Shape Our Water

Technical Memo

Seismic Risk Assessment

February 17, 2022

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Seismic Risk Assessment

Technical Memorandum

Date:	February 17, 2022
Deliverable title:	Drainage and Wastewater Seismic Risk Assessment
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Abbreviations

ALA	American Lifelines Alliance
ASCE	American Society of Civil Engineers
BC	Brown and Caldwell
BES	City of Portland Bureau of Environmental Services
City	City of Seattle
CSO	combined sewer overflow
CSZ	Cascadia Subduction Zone
DEM	digital elevation model
DNR	State of Washington Department of Natural Resources
DSA	Drainage System Analysis
DWW	Drainage and Wastewater
GIS	geographic information system
GMPE	ground motion prediction equation
GPM	gallons per minute
KCWTD	King County Wastewater Treatment Division
LiDAR	Light Detection and Ranging
M6.7	Moment Magnitude 6.7
M7.0	Moment Magnitude 7.0
M7.2	Moment Magnitude 7.2
M7.3	Moment Magnitude 7.3
M8.0	Moment Magnitude 8.0
M9.0	Moment Magnitude 9.0
MGD	million gallons per day
NAVD88	North American Vertical Datum of 1988
NSHMP	National Seismic Hazard Mapping Project
OPCD	Office of Planning and Community Development
PGA	peak ground acceleration
PGD	permanent ground deformation
PGV	peak ground velocity
QA/QC	quality assurance and quality control
SDOT	Seattle Department of Transportation
SFZ	Seattle Fault Zone
SODO	South of Downtown
SOPA	SPU System Operation, Planning, Analysis group
SPU	Seattle Public Utilities
TCLEE	(ASCE) Technical Council on Lifeline Earthquake Engineering
TM	technical memorandum
USGS	United States Geological Survey
WGS	Washington Geological Survey
WSSS	Water System Seismic Study
WWPS	wastewater pump station
WWSA	Wastewater System Analysis

Executive Summary

The Seattle area is prone to earthquakes on any of multiple faults in the region, including the Seattle Fault Zone (SFZ) and the Cascadia Subduction Zone (CSZ). A large earthquake originating on the SFZ or a great CSZ interface event will cause strong ground shaking, permanent ground deformation, liquefaction, landslides, tsunamis, and/or seiches—which could potentially impact drainage and wastewater infrastructure and disrupt Seattle Public Utilities' ability to provide essential services. Earthquakes originating in the CSZ deep intraplate zone occur more frequently, but are typically not as damaging as large SFZ or CSZ interface earthquakes.

Wastewater infrastructure is especially vulnerable to earthquakes because of the extensive networks of below-ground mainlines, pump stations, storage tanks, and combined sewer facilities. Breaks or loss of grade in the collection system, or damage to pump stations could lead to sewage backups in homes and potential releases of untreated sewage into the environment. Drainage mainlines are also susceptible to earthquake-induced damage. In the event of strong earthquake ground shaking, Seattle Public Utilities (SPU) could face significant challenges in responding to assess and repair their damaged assets due to damaged roads, bridges, power lines, and other lifeline infrastructure systems.

The Seismic Risk Assessment Team (Team), consisting of the SPU contributors and a team of consultants led by Brown and Caldwell, performed a desktop assessment of SPU's drainage and wastewater mainlines, wastewater pump stations, and combined sewer overflow (CSO) facilities to identify those that are at higher risk to damage and failure during a seismic event. The desktop assessment was based on two earthquake scenarios: (1) magnitude 7.0 earthquake occurring on the Seattle Fault Zone (M7.0 SFZ) and (2) magnitude 9.0 earthquake occurring on the Cascadia Subduction Zone (M9.0 CSZ). Scenario descriptions, ground shaking, permanent ground deformation, and tsunami/seiche inundation data are based on data previously developed by the United States Geological Survey (USGS) and Washington Department of Natural Resources (DNR) and technical work completed for King County Wastewater Treatment Division's (KCWTD's) Resiliency and Recovery Study (HDR 2018a and 2018b) and SPU's Water System Seismic Study (SPU 2018b).

The results of the desktop assessments were used to develop likelihood of failure scores. SPU then combined the likelihood of failure scores with scores representing potential consequences of failure and scores representing equity considerations. The combined risk scores were then used to categorize high-risk facilities and mainlines for subsequent planning. Seismic risk scoring data from this assessment is not intended to inform specific facility upgrades, retrofits, or improvement projects; however, it is intended to characterize the general seismic risk of the drainage and wastewater system and to inform the development of the Shape Our Water Plan.

1. Introduction

Seattle Public Utilities (SPU) is preparing *Shape Our Water*, *A 50-year Plan for Seattle's Water Resilience* to support their Drainage and Wastewater (DWW) Line of Business. The Shape Our Water Plan will provide citywide recommendations for projects, programs, and policies that will better equip SPU to be a community-centered utility and be more resilient to earthquakes, future changes in the climate, regulations, and the economy.

The Shape Our Water Plan includes a comprehensive, multi-stakeholder, engagement effort to provide a community-shaped vision and develop the plan's vision and goals. The Shape Our Water Plan will direct near and long-term investment in the partnerships, programs and projects that will improve the performance and resilience of Seattle's drainage and wastewater systems while optimizing social and environmental benefits for the community.

1.1 Seismic Risk

The Seattle area is prone to earthquakes on any of multiple faults in the region, including the Seattle Fault Zone (SFZ) and the Cascadia Subduction Zone (CSZ). A large earthquake originating on the SFZ or a great CSZ interface event will cause strong ground shaking, permanent ground deformation, liquefaction, landslides, tsunamis, and/or seiches—which could potentially impact drainage and wastewater infrastructure and disrupt SPU's ability to provide essential services. Earthquakes originating in the CSZ deep intraplate zone occur more frequently, but are typically not as damaging as large SFZ or CSZ interface earthquakes.

Wastewater infrastructure is especially vulnerable to earthquakes because of the extensive networks of below-ground mainlines, pump stations, storage tanks, and combined sewer facilities. Breaks or loss of grade in the collection system, or damage to pump stations could lead to sewage backups in homes and potential releases of untreated sewage into the environment. Drainage mainlines are also susceptible to earthquake-induced damage. In the event of strong earthquake ground shaking, SPU could face significant challenges in responding to assess and repair their damaged assets due to damaged roads, bridges, power lines, and other lifeline infrastructure systems.

1.2 Purpose and Objectives

The purpose of this task is to perform a desktop assessment of SPU's wastewater pump stations, combined sewer overflow (CSO) facilities, and drainage and wastewater mainlines to identify those that are at higher risk in a seismic event and prepare initial preliminary risk scores. These outcomes will be used to categorize high-risk facilities and mainlines for subsequent planning. Seismic risk scoring data from this assessment is not intended to inform specific facility upgrades, retrofits, or improvement projects; however, it is intended to characterize the general seismic risk of the drainage and wastewater system and to inform the development of the Shape Our Water Plan. Figure 1-1 provides a summary flowchart for the seismic risk assessment process.

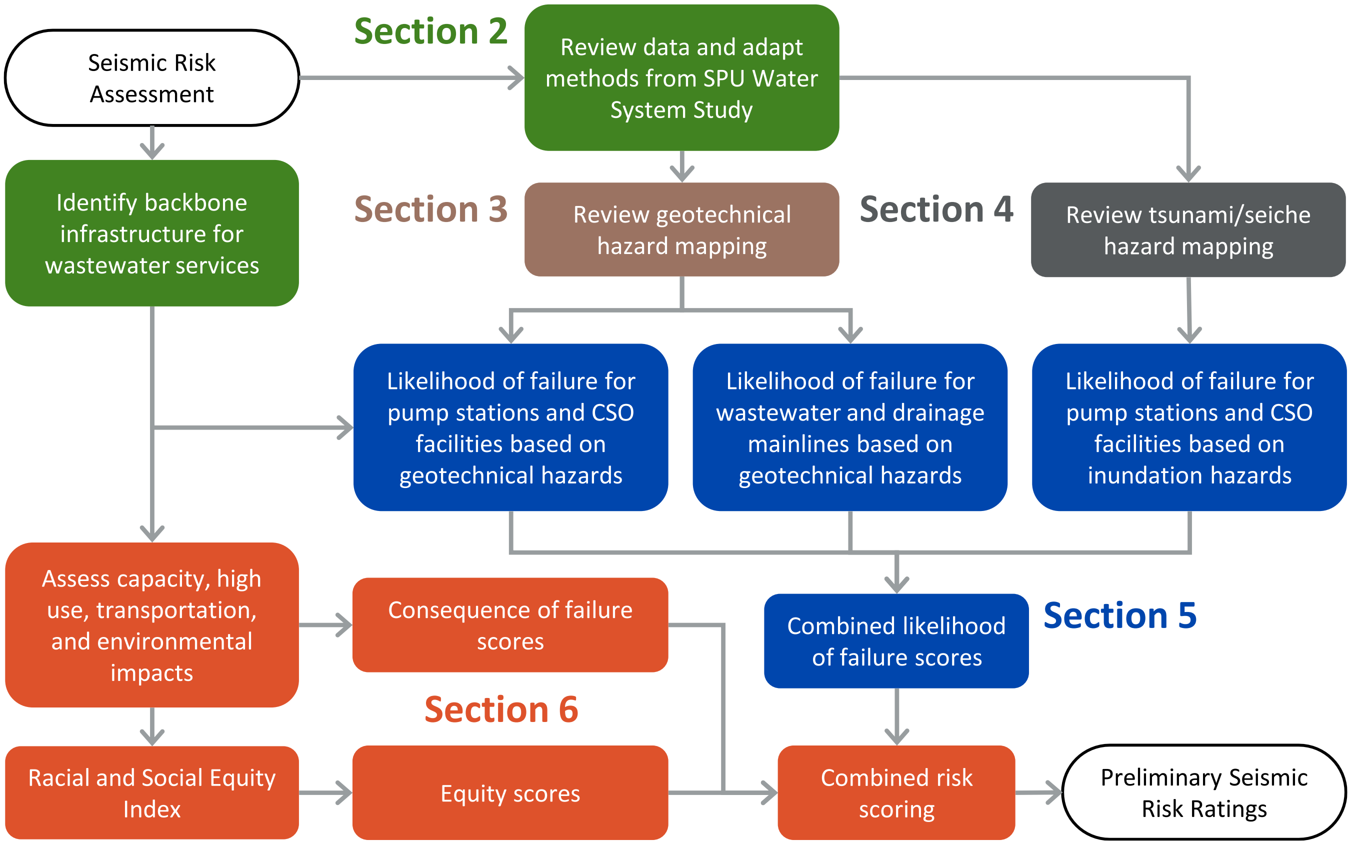


Figure 1-1. Summary flowchart for the seismic risk assessment and related report sections

The desktop assessment is based on two earthquake scenarios: (1) magnitude 7.0 earthquake occurring on the Seattle Fault Zone (M7.0 SFZ) and (2) magnitude 9.0 earthquake occurring on the Cascadia Subduction Zone (M9.0 CSZ). Scenario descriptions, ground shaking, permanent ground deformation, and tsunami/seiche inundation data are based on data previously developed by the United States Geological Survey (USGS) and Washington Department of Natural Resources (DNR) and technical work completed for King County Wastewater Treatment Division's (KCWTD's) Resiliency and Recovery Study (HDR 2018a and 2018b) and SPU's Water System Seismic Study (SPU 2018b).

2. Background

SPU's DWW Line of Business provides drainage and wastewater services to a population of approximately 747,300 and covers an area of roughly 84 square miles.

SPU's collection systems now include sanitary sewers, fully combined wastewater and stormwater sewers, and partially separated wastewater systems. Approximately 27 percent of the City's wastewater collection system is sanitary (mostly in the northern parts of the city), 33 percent is fully combined (mostly in the central core), and 40 percent is partially separated (throughout the southern parts of the city but also in several northern basins).

2.1 System Overview

SPU operates a complex wastewater collection system network consisting of 1,423 miles of separated and combined sewer mainlines and maintenance holes, 67 pump stations, and 82 permitted CSO outfalls in Puget Sound, Lake Washington, and the Duwamish Waterway (Aqualyze 2019). Map A-1 (in Appendix A) provides an overview of SPU's wastewater system, including King County interceptors.

Split ownership of the wastewater system contributes to its complex nature. Service areas for SPU's collection system typically do not exceed 1,000 acres, discharging into trunk lines owned and operated by King County's Wastewater Treatment Division (KCWTD). SPU's wastewater mainline diameters range from 4 inches to 12 feet; however, 8-inch and smaller diameter mainlines comprise over 60 percent of the network and mainlines greater than 12 inches comprise less than 18 percent of SPU's total gravity mainline inventory.

The average age of SPU's wastewater mainlines is more than 80 years, and the median year for wastewater mainline installations is between 1930 and 1940. According to SPU's Strategic Asset Management Plan (SPU 2015a), the first wastewater mainline network in Seattle was constructed in 1883 and these mainlines were made with a mixture of clay and iron slag known as "iron stone." Vitrified clay pipes were first installed in 1885, and by the turn of the century more than 30 miles of wastewater mainline had been constructed. Vitrified clay pipe continued to be the dominant material installed until the end of World War II. In the mid-1940s, concrete pipe became the primary material for constructing wastewater mainlines, and it continues to be the most common material used today. Roughly 34 percent of the mainlines are made of vitrified clay and 57 percent of the mainlines are made of concrete or reinforced concrete pipe. The remaining 7 percent are made of other materials, including asbestos cement, ductile iron, cast iron, brick, high-density polyethylene, and polyvinyl chloride.

SPU currently owns, operates, and maintains 67 wastewater pump stations that receive wastewater from enclosed gravity sewer basins and then convey the wastewater by force main to a point where it can be discharged into KCWTD's regional network of trunk line interceptors. While the first pump station was constructed in 1929, most of SPU's wastewater pump stations were constructed between 1950 and the mid-1970s. A majority of Seattle's wastewater is conveyed to the West Point Wastewater Treatment Plant owned and operated by KCWTD.

SPU currently owns, operates, and maintains 42 CSO facilities to detain and regulate combined sewer flows that exceed the conveyance capacity of the collection system during wet weather. CSO facilities consist of storage detention pipes or tanks, flow control structures, and associated electrical and mechanical equipment. CSO facilities vary in storage volume from 3,000 gallons to 2.6 million gallons (SPU 2018a). Older CSO facilities tend to be passively controlled without operable features, while newer CSO facilities are more often actively controlled to regulate flow.

Drainage Mainlines. SPU also operates a complex drainage collection system consisting of drainage mainlines, inlets, maintenance holes, catch basins, surface and subsurface stormwater control facilities (e.g., ponds, vaults, filters, and swales), stream culverts, green stormwater infrastructure (GSI), ditches, and non-stream culverts. While this study focuses on the wastewater system, drainage mainlines have also been included in the analysis because the approach to mainlines can be applied to both drainage and wastewater mainlines. Map A-2 (Appendix A) provides an overview of SPU's drainage mainlines.

2.2 Wastewater System Backbone

Critical components of the wastewater collection, conveyance, and treatment system usually include:

- treatment plant structures that are required to provide some minimal level of treatment
- trunk lines, large diameter conveyance mainlines, and associated pump stations
- small diameter collection mainlines and associated pump stations needed to connect to critical community facilities (hospitals, emergency shelters, etc.)
- certain support facilities (laboratories, maintenance shops, etc.)

Together, these critical components make up the wastewater system backbone. Following a major earthquake, the backbone system is intended to experience minimal damage so that the wastewater system will be capable of providing service to critical community facilities in support of short- and intermediate-term community recovery goals.

Since KCWTD provides treatment and trunk line interceptors, SPU's wastewater backbone system consists primarily of infrastructure components necessary to collect and convey wastewater from critical community facilities to the interceptors owned by KCWTD. SPU has identified a list of approximately 740 critical community facilities, including: hospitals, police and fire stations, shelters, schools, libraries, childcare centers, et cetera. This list of critical community facilities was used to define facilities that should be supported by the wastewater system backbone. SPU then identified a wastewater system backbone based on the following criteria:

- Mainlines that service a critical community facility
- All mainlines downstream of mainlines that serve a critical community facility up to a KCWTD interceptor or other agency sewer main
- 16 wastewater pump stations (WWPS) that are required to satisfy short- and intermediate-term community needs following a major earthquake.
- 18 CSO facilities, CSO mainline detention systems, consisting of circular or rectangular mainlines, the majority of which comprise CSO facility storage assets.

Appendix B discusses the mapping of wastewater system backbone mainlines, and Map B-1 (in Appendix B) shows the wastewater system backbone, critical facilities, and components. Table 2-1 provides a summary of SPU wastewater system assets included in the backbone. A backbone for the drainage system was not developed as part of this project.

Table 2-1. Summary of Wastewater System Backbone Infrastructure

Component	Backbone Description
Mainlines	• 0.1 miles of combined force mains • 113 miles of combined mainlines • 2 miles of sanitary force mains • 177 miles of sanitary mainlines • 0.8 miles of CSO detention mainlines
Facilities	• 16 pump stations • 18 CSO facilities

2.3 Review of Previous Studies

The Seismic Risk Assessment Team (Team), consisting of the SPU contributors and a team of consultants led by Brown and Caldwell (identified at the beginning of this Technical Memorandum), reviewed previous reports and planning documents to obtain background information for the seismic risk assessment. SPU has taken a proactive approach to managing their wastewater system assets, developing asset management plans, capital improvement plans, and condition assessments. The Team identified the following as key documents used to inform the seismic risk assessment:

- Seattle Public Utilities Sewer Pump Station Prioritized Capital Improvement Plan Report (Davido Consulting Group, Inc. 2015)
- Seattle Public Utilities Strategic Asset Management Plan Update Wastewater Collection Pipes (SPU 2015a)
- Seattle Public Utilities Wastewater Collection Pipe Criticality Criteria and Rating Scale (SPU 2015b)
- Seattle Public Utilities Critical Pipes & SSO Map (SPU 2016a)
- Seattle Public Utilities Pipe Criticality–Scoring, Process, & Current State of Data (SPU 2016b)
- Seattle Public Utilities Strategic Asset Management Plan (SAMP) Update Wastewater Pump Stations and Force Mains (SPU 2016c)
- Seattle Public Utilities Asset Management Plan (AMP) Combined Sewer Overflow Facilities (SPU 2018a)
- Wastewater System Analysis (Aqualyze 2019)

The Team also reviewed previous seismic risk studies and available seismic and tsunami hazard data as the basis for this preliminary seismic risk assessment. Key documents include the following:

- Seattle Public Utilities Water System Seismic Study Summary Report, including geospatial data for seismic hazards (SPU 2018b)
- Recommendations to Enhance the Resiliency and Recovery of King County's Regional Wastewater Treatment Facilities–Task 500 Preparedness and Recovery Recommendations (HDR 2018a)
- Recommendations to Enhance the Resiliency and Recovery of King County's Regional Wastewater Treatment Facilities–Task 600 Resiliency Recommendations (HDR 2018b)
- Tsunami Hazard Map of the Elliott Bay Area, Seattle, Washington: Modeled Tsunami Inundation from a Seattle Fault Earthquake (Walsh et al. 2003)

The following paragraphs describe the relevance of the previous studies and the associated available data.

Geotechnical and Tsunami Hazard Mapping. For the Water System Seismic Study (WSSS) (SPU 2018b), SPU evaluated the risks and vulnerabilities of their potable water system when subjected to two different earthquake scenarios: M7.0 SFZ and M9.0 CSZ (SPU 2018b). The seismic risk assessment for the wastewater system (described herein) is based on the same earthquake scenarios and corresponding geotechnical hazard data sets as the WSSS. These data include:

- Peak ground acceleration (PGA)
- Peak ground velocity (PGV)
- Spectral response acceleration parameter at short periods (~0.2 second period)
- Spectral response acceleration parameter at a period of 1 second
- Liquefaction susceptibility and probability
- Liquefaction-induced permanent ground deformation (PGD)
- Landslide susceptibility
- Landslide-induced PGD
- Fault rupture PGD (M7.0 SFZ only)

The WSSS also considered the potential impact from a tsunami generated by a M7.3 SFZ scenario earthquake. The extent of tsunami inundation in the area around the Elliott Bay coast of Puget Sound, associated with this scenario event, was based on a previous State of Washington Department of Natural Resources (DNR) study (Walsh et al. 2003). Note that a M7.3 SFZ scenario earthquake was used for the tsunami risk assessment instead of a M7.0 event, based on the available tsunami hazard data. Since the SPU Water System Seismic Study was completed, previous tsunami modeling studies have been updated and additional studies have been conducted by DNR that consider a larger portion of the South King County Puget Sound coastline than was considered in the 2003 study (WGS 2019 and DNR In Preparation). These more recent studies were used as the basis for the tsunami inundation hazard considered in this seismic risk assessment. A detailed inventory of the geotechnical seismic hazard and tsunami hazard GIS data files provided by SPU and/or obtained from DNR is provided in Appendix C.

Seismic Vulnerability of the Regional Wastewater System. In 2018, KCWTD completed a seismic resilience and recovery study for their wastewater system (HDR 2018a and 2018b). The KCWTD study evaluated the expected performance of their wastewater conveyance and treatment systems when subjected to two scenario earthquakes: M7.2 SFZ and M9.0 CSZ. The former scenario is similar to the M7.0 SFZ scenario used in the Water System Seismic Study (SPU 2018b) and the latter is equivalent. The KCWTD study also included development of mitigation strategies to address the identified seismic and tsunami vulnerabilities. As described above, the SPU collection system delivers wastewater to the KCWTD's conveyance and treatment systems. Since the SPU and KCWTD's systems must ultimately function as one integrated system, the seismic risk assessment approach and methodology used for this SPU wastewater seismic risk assessment has been generally consistent with that used for the KCWTD study.

Preliminary Risk Scoring. In 2019, SPU completed the Wastewater System Analysis (Aqualyze 2019), which evaluated the conveyance capacity of the wastewater collection system and identified potential risk areas. As part of this study, SPU developed prioritization criteria and a risk-based scoring system. The scoring system used the following equation:

$$Risk\ Score = Consequence\ Score\ \times Likelihood\ Score + Equity\ Score \qquad \textit{Eq. 2-1}$$

The *Consequence Score*, *Likelihood Score*, and *Equity Scores* each had values ranging from 1 to 5, which results in *Risk Scores* ranging from 2 to 30. This preliminary seismic risk assessment has used a similar approach to calculating seismic risk scores. As described in Section 6, the risk scores developed as part of this project are considered preliminary and “conceptual” because they are based on a desktop screening analysis without the benefit of a detailed structural analysis, on-site assessments or verifications.

3. Preliminary Geotechnical Hazard Review

As specified in the objectives for this assessment, geotechnical hazard data were reviewed for the following two earthquake scenarios: a magnitude 7.0 event on the Seattle Fault Zone (SFZ) and a magnitude 9.0 interface event on the Cascadia Subduction Zone (CSZ). Each of these earthquake scenarios are briefly described below.

M7.0 Seattle Fault Zone. The SFZ is an east-west trending, south dipping, largely concealed thrust fault that crosses the central Puget Sound near the latitude of Seattle (Brocher et al. 2001 and 2004). It produces a broad zone of active deformation, about 4 to 7 km wide, that separates bedrock to the south from thick sequences of sediments that fill the Seattle Basin to the north (Blakely et al. 2002). At the ground surface, the central SFZ deformation zone is defined by fault scarps and warped shorelines near Seattle (e.g., Nelson et al. 2003; Haugerud 2003; Kelsey et al. 2008). Paleoseismic studies suggest that these shorelines had been uplifted as much as 8 meters (m) during a single large, regional earthquake (i.e., ~ M7) above the south-dipping SFZ thrust about 1,000 years ago (AD 900 to 930) (Bucknam et al. 1992; Atwater 1999; Kelsey et al. 2008). Based on paleoseismic studies, the recurrence interval for a large rupture is about 5,000 to 6,000 years. The magnitude of the selected M7.0 scenario event is representative of one of these large SFZ events. Trench excavations and shoreline studies across the north-dipping surface fault scarps also indicate that: (a) they ruptured several times during the late Holocene Epoch producing moderate-sized earthquakes (i.e., ~M6 to 6.5) and also possibly during the A.D. 900 to 930 event (Nelson et al. 2003; Kelsey et al. 2008; Nelson et al. 2014) and (b) their rupture areas were small compared to the master fault rupture area during the AD 900 to 930 earthquake (Kelsey et al. 2008). The recurrence interval for these smaller, moderate-sized SFZ earthquakes is on the order of about 1,000 years and, unless centered on a portion of the fault beneath the city, would result in lower ground shaking and impact to the wastewater system than the selected M7.0 scenario event.

M9.0 Cascadia Subduction Zone. The CSZ is created by subduction of the Juan de Fuca Tectonic Plate beneath the North American Plate off the coast of western North America from southern Canada to northern California. Paleoseismic studies provide conclusive evidence that the CSZ generates great earthquakes (i.e., approximately M8 to M9) that actively deform this 1,000 km of coastline (e.g., Atwater and Hemphill-Haley 1997; Clague 1997; Goldfinger et al. 2003 and 2012). Geological evidence from the coastal Pacific Northwest and written records from Japan strongly suggest that the last great event that ruptured along the entire length of the subduction zone (M9) occurred about 320 years ago on January 26, 1700 (Satake et al. 1996; Atwater and Hemphill-Haley 1997; Clague 1997; Yamaguchi et al. 1997). Based on paleoseismic studies, the recurrence interval for a great M9 rupture is about 500 years. The magnitude of the selected M9.0 scenario event corresponds to a great earthquake that ruptures the entire length of the CSZ. Extensive coastal and offshore studies have refined the rupture model and some include rupture of the CSZ along segments that have shorter recurrence intervals (as short as ~200 years) and correspondingly smaller (i.e., ~M8+) earthquake magnitudes (Goldfinger et al. 2012).

As previously indicated, geotechnical hazards considered in the seismic risk assessment include:

- Ground Motions for each scenario event:
  - Peak ground acceleration (PGA)
  - Short Period (~0.2 second) Spectral Acceleration
  - 1.0-Second Spectral Acceleration
  - Peak ground velocity (PGV)
- Liquefaction-induced permanent ground deformation (PGD) and probability
- Landslide-induced PGD and probability
- Fault rupture-induced PGD (M7.0 SFZ only)

The data sets for each of these hazards are the same as those developed by SPU and used in the WSSS. Descriptions of these geospatial data sets are provided in Appendix C. Citywide maps of hazard data are provided in Appendix D. These data sets were largely based on “best available science” publicly available at the time they were developed for the SPU WSSS (i.e., circa 2017 to 2018) and were peer reviewed. As such, these data sets provide a technically sound and convenient basis for the current wastewater seismic hazard assessment.

Since development of the WSSS geotechnical hazard sets, there have been updates to the database and procedures to develop the geotechnical hazard sets. These updates may be considered qualitatively in the current hazard assessment and should be considered quantitatively in potential/future site-specific hazard assessments and/or mitigation design. The updates relative to this review of the WSSS geotechnical data sets are summarized as follows:

Ground Motion Data Sets:

- **Number of Ground Motion Prediction Equations (GMPEs) used for the M7.0 Scenario event.** The ground motion data sets are the average of five NGA-West2 GMPEs. Currently the U.S. Geological Survey (USGS) only uses four of the five NGA-West2 GMPEs in the latest seismic hazard maps from the National Seismic Hazard Mapping Project (NSHMP); they do not use the Idriss GMPE because of its lack of site factor adjustments for relatively soft-soil sites (Peterson et al. 2020). The impact of excluding this GMPE on the M7.0 scenario ground motions may be relatively small (it was reported to impact the NSHMP ground motion estimates by three percent [Peterson et al. 2020]), depending on how this issue was handled in the WSSS data set.
- **Seattle Basin Amplification.** As shown in Figure 3-1, much of Seattle lies within a sedimentary basin whose southern edge is formed by bedrock uplift on the SFZ. Constructive interference of seismic waves within a sedimentary basin, such as the Seattle Basin, amplifies ground motion relative to sites outside the basin. Amplification is especially pronounced for long-period motions (i.e., about 1 second and longer). Basin amplification was not considered in developing the WSSS datasets for estimated 1.0-second spectral acceleration and PGV for neither the M7.0 nor the M9.0 scenarios. Therefore, these values in those datasets are not conservative, and may be low in areas of the basin north of about South Spokane Street. To address basin amplification in the 2018 USGS National Seismic Hazard Maps, the USGS has developed a set of basin amplification factor values specifically for the Seattle Basin to use with the NGA-West2 GMPEs (Peterson et al. 2020). This set of values result in a basin amplification factor of approximately 1.5 for shallow crustal earthquake sources, such as the SFZ. Peterson et al. (2020) indicates that for great CSZ interface events,

basin amplification factors on the order of 2 to 3 may be expected but use a factor of 1.5 in the 2018 National Seismic Hazard Maps. Consequently, the following amplification factors are applied to locations within the Seattle Basin:

– M7.0 SFZ Scenario 1-Second Spectral Acceleration and PGV basin amplification factor = 1.5
– M9.0 CSZ Scenario 1-Second Spectral Acceleration and PGV basin amplification factor = 2

Figure 3-1. Geologic definition of the Seattle Basin

Source: Worth et al., 2018

Liquefaction susceptibility and derived data sets:

- The liquefaction susceptibility map and the derived PGD and probability of PGD data sets are based on liquefaction susceptibility mapping from the Washington State Department of Natural Resources, which are based on geologic mapping in Seattle done primarily before the year 2000. In 2005, Booth et al. published a new geologic map for the City of Seattle. The significant increase in geotechnical data available was the main driver for an update of the liquefaction susceptibility and potential mapping, sponsored by the Seattle Department of Construction and Inspection (in conjunction with the University of Washington). The results of the updated mapping were not yet available at the time this geotechnical hazard review was conducted. While the revised mapping will likely not result in large changes in areas identified as being susceptible to liquefaction, there will be some modest revisions to the locations and the relative susceptibility of some of the geologic units. This updated mapping should be considered in potential/future site-specific hazard assessments and/or mitigation design.
- The WSSS liquefaction susceptibility data set does not include the completion of major infrastructure projects designed to limit the impacts of liquefaction. Specifically, the SR 99/Alaskan Way improvements in South of Downtown (SODO) and the downtown Seattle Elliott Bay Seawall Project were designed specifically to reduce the impacts of liquefaction. Liquefaction-induced PGD for SPU facilities near and landward of these projects are likely conservatively over-estimated. As a first-order approximation to include the effects of these infrastructure projects, the estimated PGD was reduced by approximately 90 percent in the following areas (Perkins and Malinak 2019; Shannon & Wilson 2013) (see Figure 3-2):
  - Waterfront between pier 62 (north) and South Washington Street (south)
  - East side of SR99 between South Main Street (north) and South Massachusetts Street (south)
- The WSSS liquefaction-induced PGD was based on an assumption of a free-face depth of no more than 10 feet below the water level. This assumption is unconservative for some locations and results in an underprediction of PGD along the Duwamish Waterway and Elliott Bay. As a first-order approximation, the following free face depth below the water level were used (see Figure 3-2):
  - 30-feet: Duwamish waterway between 1st Avenue South Bridge (south) and South Spokane Street bridges (north)
  - 50-feet: East and West Duwamish waterways north of the South Spokane Street bridges, and Elliott Bay east of the intersection of Fairmount Avenue Southwest and Harbor Avenue Southwest, to Pier 91

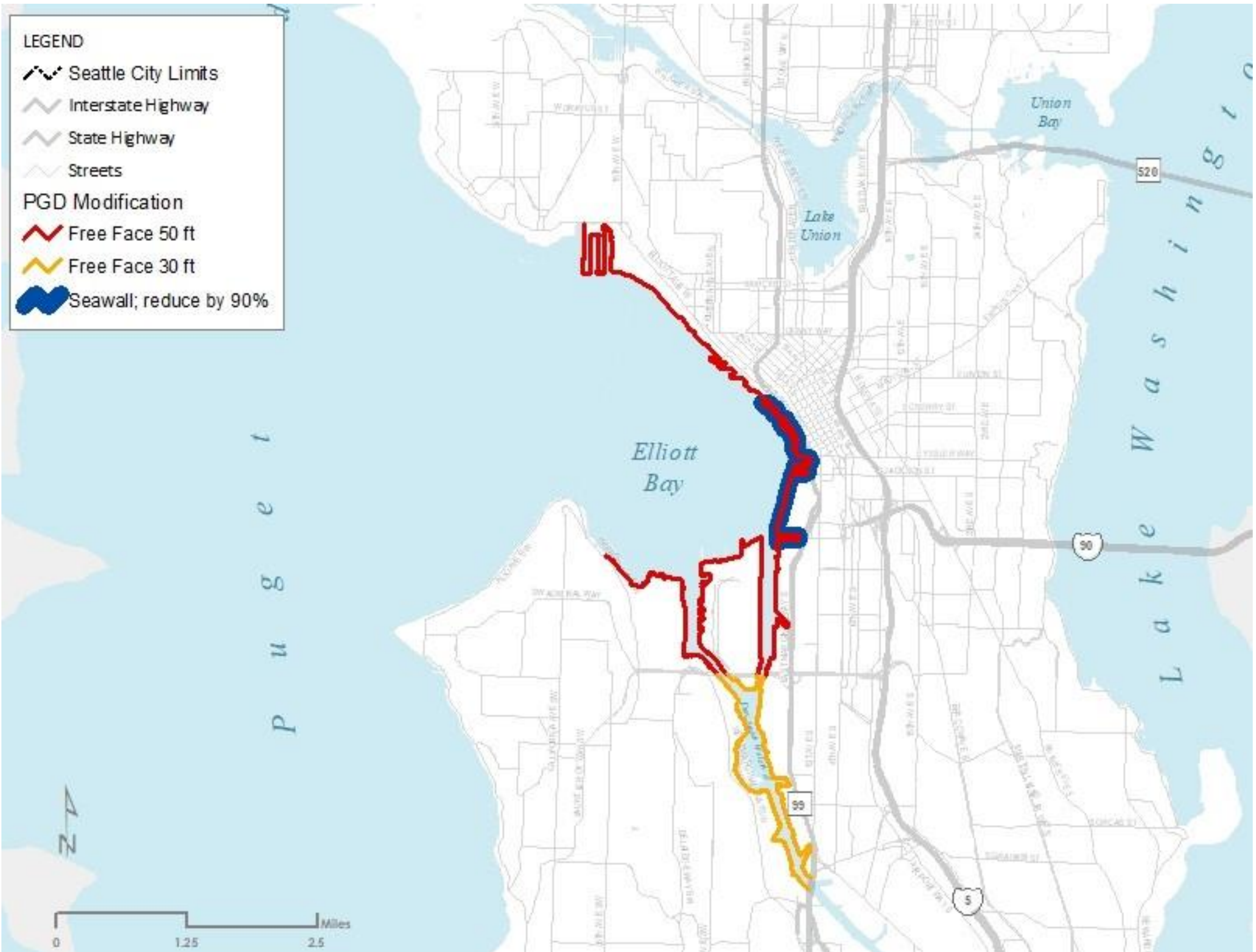


Figure 3-2. Locations for PGD modifications

4. Preliminary Tsunami and Seiche Hazard Review

Several SPU wastewater system pump stations and backbone mainlines are located in areas that may be susceptible to inundation from an earthquake-induced tsunami or seiche. This section provides an overview of the tsunami and seiche hazards potentially impacting SPU wastewater system backbone assets and describes the approach used to conduct a preliminary tsunami and seiche vulnerability assessment.

4.1 Tsunami Hazard

A tsunami is a type of water wave that can be generated by earthquake-induced ground movement or a landslide that rapidly displaces a large volume of water. In the open ocean, a tsunami's wave height is generally relatively small, but as the tsunami wave reaches land, the wave characteristics change. The wave runup may inundate low-lying areas near the shoreline and further inland, depending on topography. Tsunami runup flow velocity can approach 20 miles per hour (ASCE 2017).

A Cascadia Subduction Zone (CSZ) earthquake will generate a tsunami that will impact Washington's Pacific coastline, but tsunami modeling results indicate that the impact of the tsunami generated by a M9.0 CSZ earthquake will be minor for the Puget Sound shoreline in Seattle (City of Seattle 2019). Models simulating the tsunami generated by a M9.0 CSZ earthquake predict that Kellogg Island (a low-lying wildlife preserve in the Duwamish River) will experience the most significant impact on the City of Seattle Puget Sound shoreline, which would be subjected to approximately 15 inches of inundation depth (City of Seattle 2019). Note that there are no SPU wastewater backbone system assets located on Kellogg Island. Future sea level rise could potentially result in additional areas of the City of Seattle Puget Sound shoreline being impacted by the tsunami generated by a M9.0 CSZ earthquake, but this has not been considered as part of this seismic risk assessment.

However, the tsunami that is likely to be generated by a Seattle Fault Zone (SFZ) earthquake (master fault rupture scenario with a 5,000 to 6,000-year return period) will significantly impact the Puget Sound shoreline around Seattle, including SPU wastewater system assets. Cycles of significant tsunami wave inundation are likely to continue for several hours after the earthquake. Historical evidence suggests that a 16-foot tsunami was generated by a M7.3 SFZ earthquake that occurred around 900 A.D. (Walsh et al. 2003, City of Seattle 2019).

A tsunami could also be generated by an earthquake-induced or non-earthquake-induced landslide (e.g., 1965 Tacoma Narrows, ancient Lake Washington landslides, etc.). The inundation extents for this type of tsunami are expected to be more localized and the hazard associated with potential landslide-induced tsunamis has not been considered as part of this seismic risk assessment.

Based on post-tsunami observations from the 2010 Tohoku tsunami in Japan, it is assumed that above-grade building-like facilities in the tsunami inundation zone will likely lose their functionality for months to years, or even be a total loss. Figure 4-1(a) shows an example of a building that collapsed due to tsunami wave-generated forces, and Figure 4-1(b) shows an example of a building that overturned due to tsunami wave and buoyancy-generated forces.

Another major tsunami hazard is associated with the debris that is transported by tsunami waters. Figure 4-2 shows examples of timber log, vehicular, and boat/ship debris that can be carried by tsunami waters and result in impact damage to buildings and can create a significant logistical challenge for the transportation system and for debris removal after the event. Additionally, when tsunami waters recede, they can cause scour that damages building and bridge foundations, buried pipelines, and roadways (see Figure 4-3).

As described in Section 2.3, the tsunami inundation extents used for this preliminary tsunami vulnerability assessment were determined by a recent DNR tsunami study conducted for the South King County Puget Sound coastline based on a repeat of the M7.3 SFZ earthquake that occurred around 900 A.D. (WGS 2019 and DNR In Preparation). Map D-11 (in Appendix D) shows mapping of the tsunami inundation zone. The northern boundary of the DNR tsunami study area was located just to the north of the Lake Washington Ship Canal. There are a few SPU wastewater pump stations located to the north of this northern boundary of the DNR tsunami study area. The tsunami risk for these pump stations was based on engineering judgement.

(a) Collapsed building

(b) Overturned building

Figure 4-1. Examples of building damage due to tsunami inundation

Source: Degenkolb Engineers

(a) Timber log (source: Degenkolb Engineers)

(b) Vehicles (source: Degenkolb Engineers)

(c) Boats/ships (source: Degenkolb Engineers)

Figure 4-2. Examples of tsunami debris

(a) Foundation and pipelines exposed adjacent to buildings

(b) Pipelines exposed adjacent to road

Figure 4-3. Examples of pipelines exposed by tsunami-induced scour

Source: Degenkolb Engineers

4.2 Seiche Hazard

A seiche is a standing wave that occurs on inland water bodies and can be generated by strong winds or earthquakes. This standing wave is characterized by predominantly vertical movement of water near the shoreline and little to no vertical movement near the middle of the water body, similar to sloshing-type motion in a bathtub or swimming pool. An earthquake-induced seiche is excited by the long-period content of the ground motion, so can result from both nearby and distant earthquakes (up to several thousand kilometers away).

Historical evidence points to multiple past seiche events in Lake Union and Lake Washington, but they have not caused extensive damage. The 2002 Denali, Alaska earthquake triggered a seiche in Lake Union that caused minor damage to at least 20 houseboats (Barberopoulou et al. 2004). An 8-foot seiche was reported on Lake Washington in 1891, resulting from an earthquake near Port Angeles (City of Seattle 2019). Despite the historical occurrence of seiche events in the Seattle area, there has been very limited scientific study to characterize the expected seiche associated with a M9.0 CSZ or M7.0 SFZ earthquake. One study of the seiche hazard in Lake Union indicates that a wave height of at least 3.28 ft may result from a M8.0 CSZ earthquake and suggests that a M6.7 SFZ earthquake may cause a seiche with a wave height that does not exceed 8 inches (Barberopoulou 2006). Geotechnical basin amplification effects and the shape of the lake have been reported to contribute to the seiche hazard in Lake Union (Barberopoulou 2006). The seiche literature does not discuss historical evidence of seiche events in smaller bodies of water within the City of Seattle (e.g., Green Lake, Bitter Lake, etc.).

Due to a lack of comprehensive seiche data, the approximate extent of the seiche inundation zone for Lake Washington, Lake Union, and the Lake Washington Ship Canal has been assumed to correlate with the area inundated by a water level 8 feet above a high operating level of 18.5 feet (North American Vertical Datum of 1988, NAVD88). This assumed increase in water level was selected based on the historic report of an 8-foot seiche on Lake Washington in 1891. The same seiche inundation extents have been assumed for both the M9.0

CSZ and M7.0 SFZ scenario earthquakes. This high operating level is consistent with the elevation used for other recent SPU studies (Brown and Caldwell 2020).

The topographic data used to determine the approximate extent of the seiche inundation zone was based on a digital elevation model (DEM) dataset developed for the City of Seattle by Quantum Spatial using topographic surveys collected and compiled by King County and the Puget Sound LiDAR[1] Consortium. The DEM is based on a 2-foot grid resolution and projected into the State Plane coordinate system, North American Datum of 1983.

Since there is only very limited SPU wastewater system backbone infrastructure located adjacent to the Duwamish Waterway, the potential seiche hazard for the Duwamish Waterway has been neglected. It is recommended that the seiche vulnerability assessment be updated in the future, as additional earthquake-induced seiche modeling data becomes available.

[1] Light Detection and Ranging (LiDAR) is a remote sensing method that uses an airborne scanning laser rangefinder to measure variable distances to the ground surface. Raw LiDAR survey data are processed to develop “bare-earth” high-resolution digital surface models.

5. Likelihood of Failure Assessment

The Team performed desktop analyses and assessments to develop likelihood of failure scores for drainage and wastewater mainlines, wastewater pump stations, and selected major CSO facilities. Section 5.1 describes the assessment of potential mainline failures due to geotechnical hazards based on location and attribute data from SPU's geospatial data. Section 5.2 describes the assessment of potential failures at pump stations and major CSO facilities due to geotechnical hazards and potential structural deficiencies based on an engineer's review of available drawings, as well as tsunami and seiche hazards based on inundation mapping.

5.1 Wastewater and Drainage Mainlines

The Team performed a desktop assessment of drainage and wastewater mainlines using fragility analysis methods and tools developed for the *WSSS* (SPU 2018b). Repair rates were calculated and used to assign relative likelihood of failure scores. Tsunami and seiche hazards can cause localized scour through sustained flow around structures. Sustained flow scour depth and area extent as recommended by ASCE 7 (2017) is mainly applicable around the perimeters of a building or an above-grade structure. It would be overly conservative to estimate damaging potential of sustained flow scour on mainlines on a community scale. For this preliminary assessment, tsunami and seiche hazards were not considered for mainlines, assuming all are sufficiently buried (i.e., function is not likely to be impacted by surface flooding). However, in a future detailed study of an individual above-grade facility, scour damage of mainlines near the facility should be considered.

5.1.1 Mainline Fragility

The American Lifeline Alliance (ALA) pipeline fragility equations were used to calculate repair rates for drainage and wastewater mainlines (although the ALA document is 20 years old, the equations are reasonable and appropriate for this desktop analysis). The ALA equations were developed in the ALA document "Seismic Fragility Formulations for Water Systems", Parts 1 and 2 (ALA 2001) and more recent work by Bonneau and O'Rourke (2009). This document developed estimates of repair rates based on a regression analysis of pipeline break data from past earthquakes for buried pipes constructed with various materials and joint types. Damage is calculated as the number of breaks, or repairs required, per 1,000 feet of pipeline (i.e., mainline for SPU system). The repair rate associated with seismic wave propagation (RR_{PGV}) is calculated as follows:

$$RR_{PGV} = K_1 \times 0.00187 \times PGV \qquad \textit{Eq. 5-1}$$

where, RR_{PGV} is the number of repairs per 1,000 feet caused by seismic wave propagation based on the peak ground velocity (PGV) associated with the earthquake, and K_1 is a constant dependent on the mainline material, diameter and joint type.

The repair rate associated with permanent ground deformation (RR_{PGD}) is determined from:

$$RR_{PGD} = K_2 \times 1.06 \times PGD^{0.319} \qquad \textit{Eq. 5-2}$$

where, RR_{PGD} is the number of repairs per 1,000 feet caused by the permanent ground deformation (PGD) associated with the earthquake, and K_2 is a constant that depends on mainline material and joint type.

Table 5-1 lists the K_1 and K_2 fragility coefficients used for this study. The ALA values for K_1 and K_2 were used where appropriate. Since the ALA equations used to perform this evaluation were published in 2001, many of the K_1 and K_2 values were updated. The values for K_1 and K_2 used in this report represent the most current post-earthquake observations, results of laboratory testing, subsequent analysis and engineering judgement.

Table 5-1. Fragility Coefficients for ALA Equations with Sources

Material	K_1	K_2	Sources
Acrylonitrile Butadiene Styrene	1.0	1.0	Engineering judgment[a]
Asbestos Cement	1.5	1.5	O'Rourke, 2019 and 2020
Brick	0.7	1.3	City of Portland, 2016 and HDR, 2018a
Cast Iron	1.0	1.0	SPU, 2018b
Concrete (reinforced, DIA greater than or equal to 48 in.)	1.0	1.0	HDR, 2018a
Concrete (unreinforced)	1.3	1.3	City of Portland,2016
Corrugated Flexible Plastic	0.3	0.3	Engineering judgment[a]
Corrugated Metal Pipe	0.3	0.3	City of Portland, 2016 and HDR, 2018a
Corrugated Rigid Plastic	0.5	0.7	Engineering judgment[a]
Ductile Iron	0.5	0.5	SPU, 2018b
High Density Polyethylene	0.05	0.05	O'Rourke, 2019 and 2020
Other	1.0	1.0	SPU, 2018b
Polyvinyl Chloride	0.5	0.5	SPU, 2018b
Reinforced Concrete Box (CSO)[b]	0.5	0.5	O'Rourke, 2019
Reinforced Concrete Box (CSO on piles)[b]	0.25	0.25	O'Rourke, 2019
Reinforced Concrete Box (non-CSO)	0.8	0.8	HDR, 2018a
Reinforced Concrete Pipe (DIA greater than or equal to 48 in.)	1.0	1.0	ALA, 2001
Reinforced Concrete Pipe (DIA less than 48 in.)	1.0	1.0	SPU, 2018b
Steel	0.5	0.5	SPU, 2018b
Vitrified Clay	1.3	1.3	Cit of Portland, 2016 for VCP with brittle/cemented joints

a. Engineering judgement based on K values for mainlines with similar properties.

b. K values based on properties other than material and size.

Figure 5-1 shows the relative proportions of mainline materials comprising the drainage and wastewater mainlines. More than 80 percent of drainage mainlines are made of reinforced concrete pipe, but only 13 percent of wastewater mainlines are made of reinforced concrete pipe. Approximately 44 percent of wastewater mainlines are made of unreinforced concrete and 34 percent are made of vitrified clay. These mainlines are typically more susceptible to earthquake damage than reinforced concrete pipe. That is why vitrified clay pipe and (unreinforced) concrete pipe have higher K_1 and K_2 values than reinforced concrete pipe.

Figure 5-2 shows the relative proportions of mainline sizes by diameter for the drainage and wastewater systems. Roughly 50 percent of drainage mainlines are 12 inches or less in diameter, while approximately 83 percent of wastewater mainlines are 12 inches or less in diameter. Approximately 53 percent of the backbone is less than 12 inches in diameter. Most of the backbone mainline is (unreinforced) concrete, reinforced concrete or vitrified clay pipe making it vulnerable to earthquake damage. The mainline is assumed to have unrestrained joints.

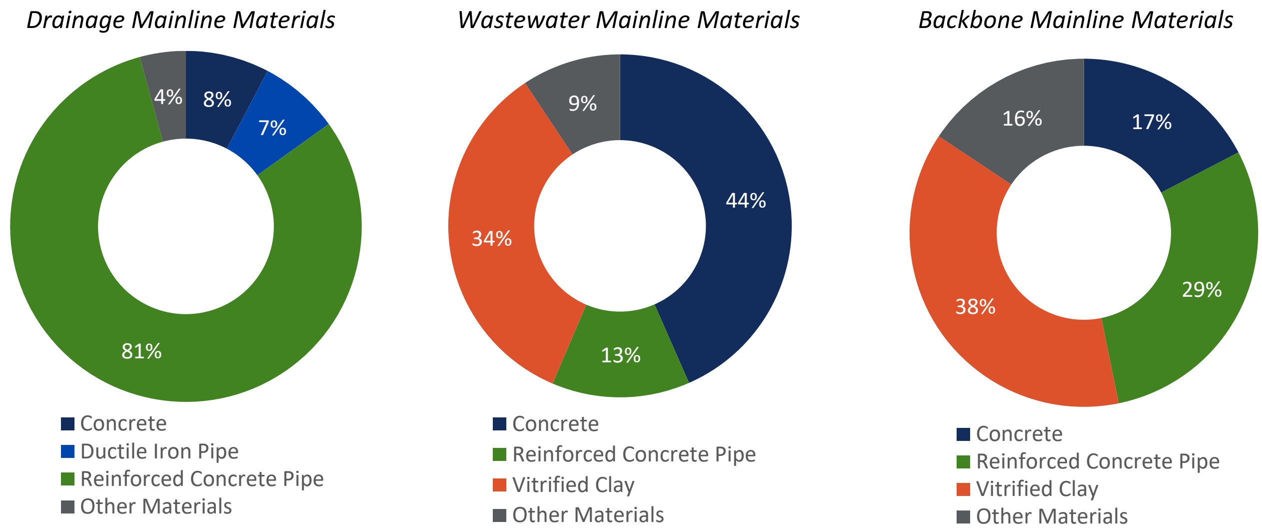


Figure 5-1. Mainline materials comprising the drainage and wastewater systems

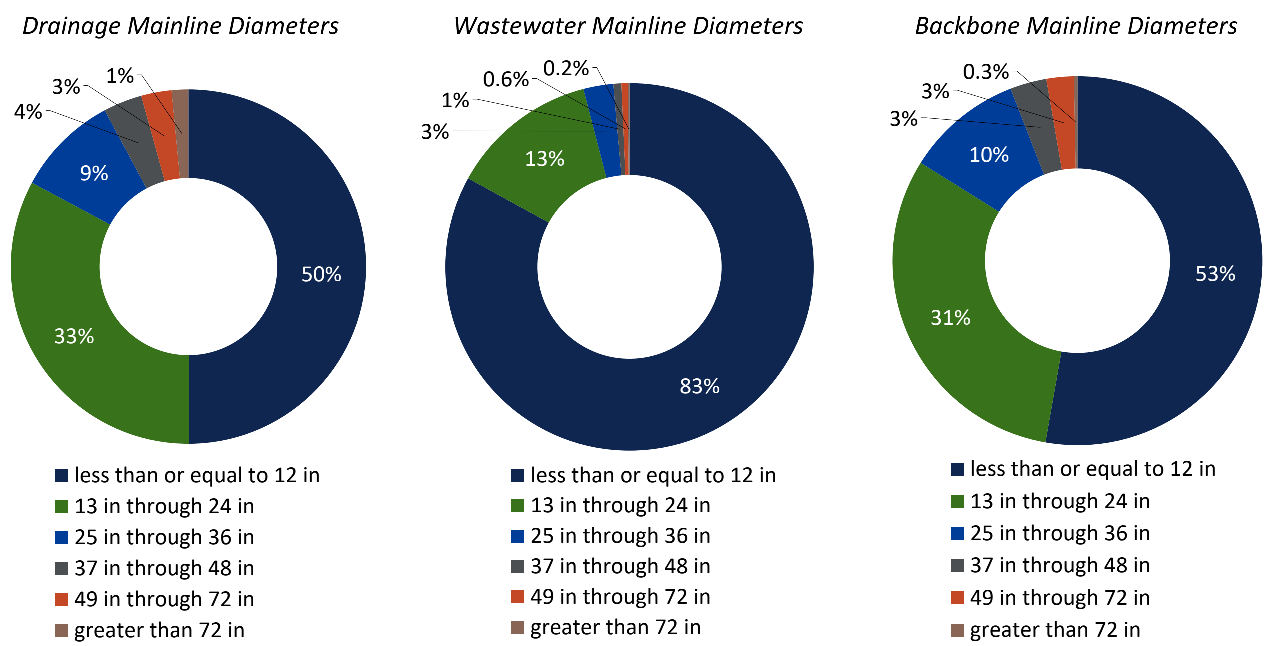


Figure 5-2. Mainline diameters for the drainage and wastewater systems

Note: non-circular shapes such as oval and rectangular are included and grouped based on the width dimension.

5.1.2 Repair Rates and Likelihood of Failure

The Team combined repair rates associated with wave propagation (RR_{PGV}) and repair rates associated with permanent ground deformation ($RR_{PGD.}$). While these values could simply be added together, this would likely overestimate the total number of mainline failures. Evaluations by ALA (2001), and more recent work by Bonneau and O'Rourke (2009), indicate that 80 percent of repairs due to PGV may be assumed to be caused by leaks and not mainline breaks. Conversely, most repairs required due to permanent ground deformation involve total mainline failure and not just leaks. Therefore, 20 percent of the PGV-based repair rates and 80 percent of the PGD-based repair rates repairs were added, as recommended by FEMA (2020), to obtain:

$$RR_{weighted} = 0.2 \times RR_{PGV} + 0.8 \times RR_{PGD} \qquad \textit{Eq. 5-3}$$

The Team then calculated combined repair rates ($RR_{combined}$) based on the results for both earthquake scenarios using the estimated return periods for each earthquake:

$$RR_{combined} = \frac{\frac{1}{5{,}000}\left(RR_{weighted,SFZ\ M7.0}\right) + \frac{1}{500}\left(RR_{weighted,CSZ\ M9.0}\right)}{\frac{1}{5{,}000} + \frac{1}{500}}$$

Eq. 5-4

After the combined repair rates were calculated using Equation 5-4, the Team assigned likelihood of failure scores to repair rate ranges by examining the relationship between repair rates and permanent ground deformation.

Figure 5-3 shows a plot of Equation 5-2 using a K_2 value of 1.3. PGD of more than 6 inches may be considered to cause mainline failures in gravity piping systems made of concrete, cast iron, or vitrified clay pipes—often because these types of pipes have joints that either pull apart or break. Accordingly, the Team assigned a likelihood of failure score of 5 to any gravity mainline with a combined repair rate estimate greater than 2.5 repairs per 1,000 linear feet. As shown in Figure 5-3, a repair rate of 2.5 repairs per 1,000 linear feet corresponds to a PGD of 6 inches. As the equation for RR_{PGD} in ALA (2001) tends to overpredict RR_{PGD} at low PGD (O'Rourke 2019 and 2020), in future studies, it should be amended so that a sensitivity analysis is performed for PGD values less are than 6 inches. This analysis will refine the RR_{PGD} and help refine the weighted repair rate ($RR_{weighted}$). The Team then assigned three additional scoring thresholds:

- A combined repair rate of 2.0 repairs per 1,000 linear feet results in a score of 4.
- A combined repair rate of 1.5 repairs per 1,000 linear feet results in a score of 3.
- A combined repair rate of 0.5 repairs per 1,000 linear feet results in a score of 2.

Mainlines with combined repair rates below 0.5 repairs per 1,000 linear feet receive a minimum score of 1.

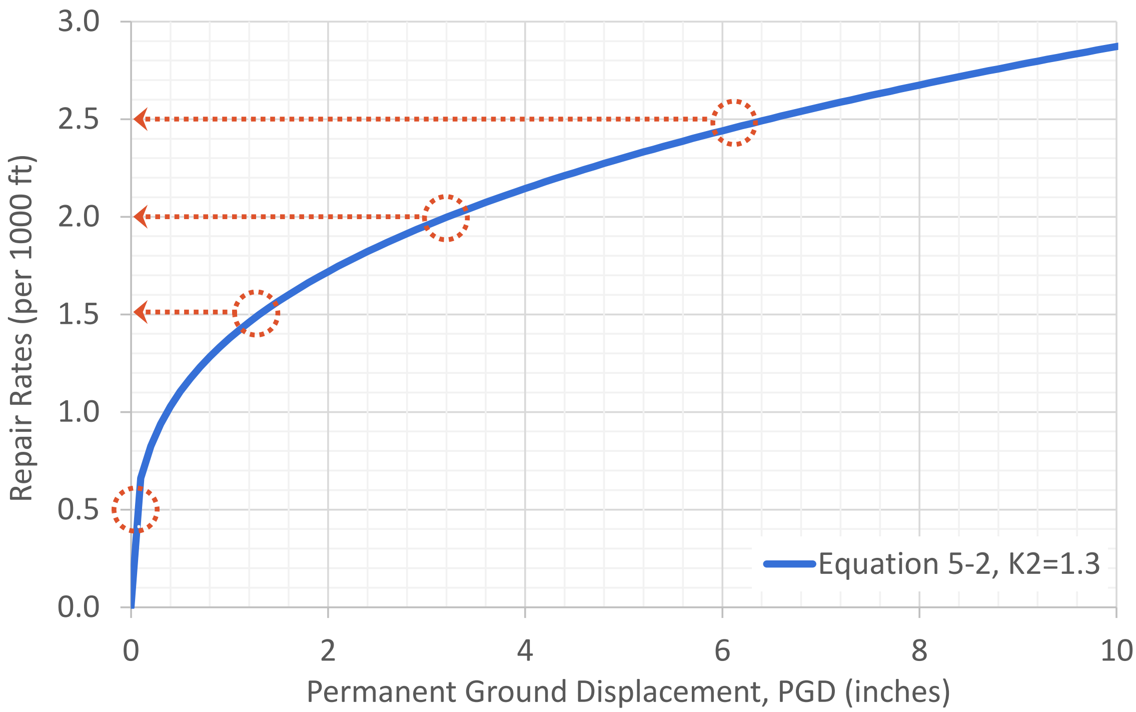


Figure 5-3. Graph showing the RR values for every PGD value from zero to 120 inches

A PGD of 6 inches is considered to be adequate movement to cause mainline failure

The gravity mainline repair rate thresholds for likelihood of failure scoring are generally higher than those for pressurized mainlines given in the *Water System Seismic Study* (SPU 2018b) because gravity systems can leak and still function, whereas pressurized water systems with leaks will be impaired in water delivery due to loss of pressure. Given that some drainage and wastewater mainlines are pressurized (i.e., force mains), the Team selected a different set of repair rate thresholds for pressurized systems that are similar to those described in the *Water System Seismic Study* (SPU 2018b).

Table 5-2 summarizes the combined repair rate ranges used for likelihood of failure scoring.

Table 5-2. Likelihood Scores Based on Repair Rates

Likelihood Score	Gravity Mainline	Force Main
1	$0.00 < RR_{combined} < 0.50$	$0.00 < RR_{combined} < 0.025$
2	$0.50 < RR_{combined} < 1.50$	$0.025 < RR_{combined} < 0.05$
3	$1.50 < RR_{combined} < 2.00$	$0.05 < RR_{combined} < 1.50$
4	$2.00 < RR_{combined} < 2.50$	$1.50 < RR_{combined} < 2.50$
5	$2.50 < RR_{combined}$	$2.50 < RR_{combined}$

5.2 Wastewater Pump Stations and CSO Structures

The Team performed a desktop assessment of the vulnerability of wastewater pump stations and selected CSO structures based on review of available drawings and the geotechnical hazards data described in Section 2.

Sixty-seven pump stations were evaluated using the desktop analysis approach described in Section 5.2.1. Eight major CSO facilities, consisting of multiple large cast-in-place concrete structures, were also evaluated using the desktop approach described in Section 5.2.1. Other CSO facilities, consisting primarily of large diameter mainline and a flow control device, were evaluated using the mainline assessment approach described in Section 5.1.

In preparation for this assessment, the Team reviewed relevant sections of the *Resiliency and Recovery Study* (KCWTD 2018) and *Water System Seismic Study* (SPU 2018b) to adopt a generally consistent technical approach. Thus, the likelihood of failure is interpreted based on impacts to system operations, similar to descriptions used in the *Water System Seismic Study* (SPU 2018b). Damage that might require repair or replacement over the long term was considered to be of relatively low impact if the facility could continue to operate after the earthquake. With this in mind, likelihood of failure on a 1 to 5 scale can be described as follows:

- 1 is considered low likelihood of failure; the facility is expected to remain operational.
- 5 is considered high likelihood of failure, the facility is not expected to be operational.
- Scores ranging between 1 and 5 have an intermediate likelihood of failure and it is less certain as to whether the facility will remain operational.

The SPU's wastewater pump stations and major CSO facilities, which were evaluated, are located at discrete locations. SPU provided tabulated geotechnical hazard data for each pump station and CSO facility site. This data was extracted from the mapped data described in Section 3, and Appendices C and D.

5.2.1 Geotechnical Hazards and Potential Structural Deficiencies

To perform these assessments, the Team reviewed original design drawings when they were available. Most of the structures are below ground structures. Data collection forms from Federal Emergency Management Agency (FEMA) *P-154 Rapid Visual Screening of Buildings for Potential Seismic Hazards* (FEMA 2015b) and the "Tier 1" checklists provided in ASCE 41-17 *Seismic Evaluation and Retrofit of Existing Buildings* (ASCE 2017) were not specifically developed for these types of buried structures. ASCE TCLEE Monograph 22 *Seismic Screening Checklists for Water and Wastewater Facilities* (TCLEE 2002) provides relevant and useful guidance for these assessments. The rapid assessment used for these structures was based upon fundamental earthquake engineering principles and key considerations contained in the checklists of ASCE 41-17 and ASCE TCLEE Monograph 22. They were focused on the following issues:

- **Landslides** induced by earthquakes can cause severe damage to pump stations or CSO facilities if they are located within a landslide area. All facilities located within a landslide susceptibility zone—based on mapping by SPU and Harp et al. (2006)—were given a likelihood score of 5 due to landslide risk. All other facilities were assumed to have a likelihood score of 1 due to landslide risk. It is important to recognize that regional landslide susceptibility maps were used as the basis for assigning landslide likelihood scores for this desktop analysis. However, this does not mean that a particular site located in an area identified as being susceptible to a landslide can or will slide during an earthquake. Future site-specific evaluations could result in changes to the landslide likelihood scores assigned as part of this desktop analysis.

- **The risk of floatation** was evaluated for pump stations or major CSO facilities within liquefiable zones. The analysis assumed: 1) that the soil provided no frictional resistance in the vertical direction against floatation; 2) if a structure had a wet well, it was assumed to be empty; 3) groundwater elevation was considered to be equal to the highest possible water level of the closest water body. It was also assumed that excess pore pressure at the bottom of a pump station or CSO facility did not contribute to uplift force acting on the structure. Depending on the excess pore pressure assumed, a different factor of safety regarding flotation would be obtained. Since an estimate of excess pore pressure was not used explicitly, conservative assumptions were used (e.g., no soil frictional resistance and weight of the structure alone) to provide a reasonable estimate of vulnerability to floatation. Structures with a safety factor greater than 1 were determined to be safe from floatation. Table 5-3 provides the likelihood of a failure score based upon the structures factor of safety against floatation. Any structure with a high likelihood of flotation is considered to have a high risk of loss of functionality.

Table 5-3. Vulnerability to Floatation

Floatation Safety Factor Range	Likelihood Score
SF ≥ 1.0	1
1.0 > SF ≥ 0.9	4
SF < 0.9	5

- **Structural failure due to damage from seismic forces.** The Team evaluated the structures to determine if the structures had a clear load path among structural components (roof slab, perimeter and interior walls, and foundation), and if they had any obvious structural deficiencies or lacked ductility due to insufficient design considerations and/or poor detailing. Each structure was evaluated to determine if the structural deficiencies could lead to the facility losing its ability to fulfill its intended function. The impact of liquification on the structural integrity was also considered in this evaluation. These evaluations and engineering judgement were then used to assign a relative score ranging from 1 to 5. Structures with identified risks are discussed in more detail in Appendix E.
- **Mainline connection failure.** The Team evaluated the risk of damage to the influent and effluent piping due to PGD. The influent and effluent mainlines were assumed to be segmented with bell and spigot joints unless shown to be continuous on the drawings. The amount of PGD was the basis used to determine if the mainline was likely to fail by being pulled apart at a segmented joint or rupture the mainline wall of a rigid pipe. The flexibility of the piping system was considered during the evaluation.

  Table 5-4 and Table 5-5 list the criteria used to determine the loss of facility functionality.

Table 5-4. Vulnerability of Connection Piping with Little or no Flexibility

Rigid Mainline Connections or Mainline Connections with Little or no Flexibility	
PGD (inches)	**Likelihood Score**
<1"	1
1" to 1.5"	2
1.5" to 2.5"	3
2.5" to 4"	4

>4"	5

Table 5-5. Vulnerability of Piping Connection with Flexibility or Multiple Joints Connecting Mainline to Structure	
Mainline connections with flexibility and multiple joints connecting mainline to structure	
PGD (inches)	**Likelihood Score**
<1"	1
1" to 2.5"	2
2.5" to 4"	3
4" to 6"	4
>6"	5

The Team also evaluated the risk of damage due to a mainline connecting two structures. A mainline interconnecting two structures can result in damage to both the mainline as well as the structures. In areas with no liquefaction, the differential movement between two structures during the earthquake can damage the mainline or the structure. If the area is subject to liquefaction-induced PGD, a mainline both rigidly and flexibly attached to two structures can be damaged or broken by differential ground movement.

Table 5-6 lists the criteria used to determine the loss of facility functionality.

Table 5-6. Vulnerability of Mainline Connecting Two Adjacent Structures	
Likelihood Score	**Construction Detail and Geotechnical Hazards**
1	If mainline has at least one joint or flexible coupling in non-liquefaction zone
2	If mainline has joints or flexible couplings in non-liquefaction zone
3	If mainline between structures has no joints or flexible couplings in non-liquefaction zone
4	If mainline between structures has no joints or flexible couplings in liquefaction zones with PGD < 2.5 inches
5	If mainline between structures has no joints or flexible couplings in liquefaction zones with PGD ≥2.5 inches

A seismic assessment of nonstructural components within a facility is typically conducted with a site visit. For this preliminary desktop assessment study, site visits to the pump stations and major CSO facilities were explicitly excluded from the scope. Although nonstructural assessment was not performed, the Team anticipates that electrical conduit and communication cable entering and exiting from facilities in liquefaction area could experience damage due to differential movement between the facilities and surrounding soil.

5.2.2 Tsunami and Seiche Hazards

The general approach implemented to consider the risk associated with earthquake-induced tsunami and seiche inundation is shown in Figure 5-4. The extent of the assumed tsunami and seiche inundation zones are described in Sections 4.1 and 4.2, respectively. Likelihood scores were assigned to wastewater pump stations and major

CSO facilities based on their proximity to the tsunami and seiche inundation zones. Pump stations and major CSO facilities located in the mapped tsunami inundation zone or in the mapped approximate seiche inundation zone were assigned a Likelihood Score equal to 5. Pump stations and major CSO facilities located outside the inundation zone were assigned Likelihood Scores that decreased with increasing distance between the asset and inundation zone boundary (see Table 5-7).

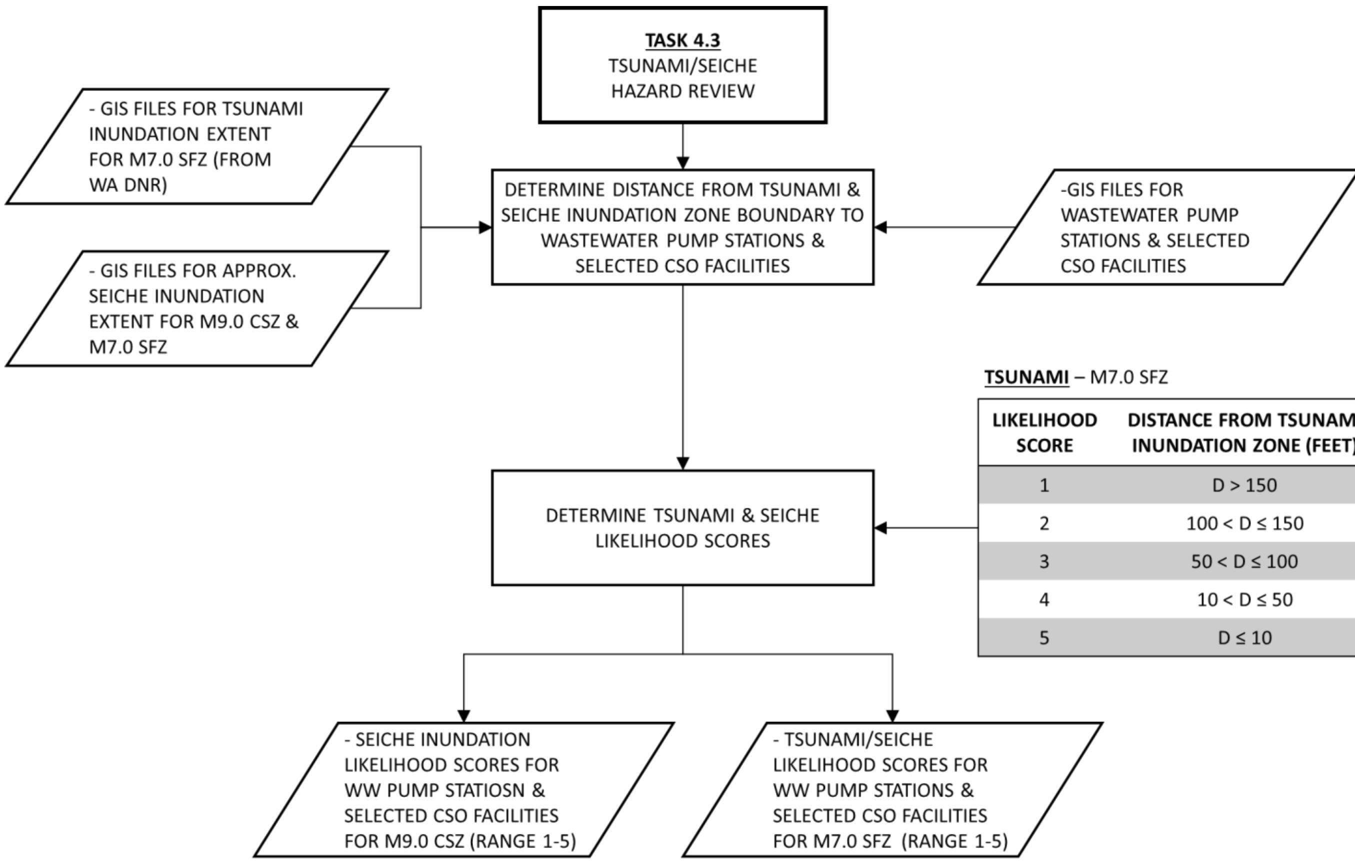
The potential damage that may be experienced by assets in the tsunami or seiche inundation zones includes:

- Pump station wet and dry wells may be filled with water, resulting in damage to electrical components, pumps, and other moisture sensitive components
- Above grade pump station components (electrical enclosures, generators, etc.) may experience damage from wave and debris impact, as well as damage from the components being submersed in water
- Pump stations and major CSO facilities may experience tsunami- or seiche-induced scour, where soil is removed from around the component by sustained flowing water, that may result in loss of support and damage to buried components.

Wastewater pump stations and major CSO facilities that are located in an area that may be subjected to inundation from a tsunami or seiche may also be in an area that is expected to experience significant earthquake-induced PGD. These assets may be damaged by PGD prior to inundation by the tsunami or seiche.

Table 5-7. Summary of Tsunami and Seiche Likelihood Scoring Criteria

Likelihood Score	Approx. Distance (*D*) from Inundation Zone Boundary (feet)	
	Tsunami	Seiche
1	$150 < D$	$150 < D$
2	$100 < D \leq 150$	$100 < D \leq 150$
3	$50 < D \leq 100$	$50 < D \leq 100$
4	$10 < D \leq 50$	$10 < D \leq 50$
5	$D \leq 10$	$D \leq 10$



TSUNAMI – M7.0 SFZ

LIKELIHOOD SCORE	DISTANCE FROM TSUNAMI INUNDATION ZONE (FEET)
1	D > 150
2	100 < D ≤ 150
3	50 < D ≤ 100
4	10 < D ≤ 50
5	D ≤ 10

SEICHE – M9.0 CSZ & M7.0 SFZ

LIKELIHOOD SCORE	DISTANCE FROM SEICHE INUNDATION ZONE (FEET)
1	D > 150
2	100 < D ≤ 150
3	50 < D ≤ 100
4	10 < D ≤ 50
5	D ≤ 10

Figure 5-4. Summary flowchart for the tsunami/seiche preliminary hazards review process

5.3 Likelihood of Failure Results Summary

A single, overall likelihood of failure score is needed for each mainline, pump station, and major CSO facility. The ALA fragility approach used for mainlines resulted in a single weighted score as described in Section 5.1. The assessments for the pump stations and major CSO facilities (described in Section 5.2) resulted in several component scores based on different potential modes of failure. While all earthquakes produce widespread ground movement or shaking, not all earthquakes will trigger landslides, tsunamis, or seiches—which suggests that the component scores for those types of failures could be weighted less than potential structural failures. However, the Team opted to not weight or further adjust for the component likelihood scores due to the uncertainty associated various modes of failure and that such adjustment factors are not readily available. Furthermore, the risk scores developed for this study are intended to flag potential vulnerabilities for follow up and further study. Therefore, the maximum component score was used as the overall combined score for pump station and major CSO facility assessments. Component scores are listed in Table 5-8.

Table 5-8. Combining Scores for Likelihood of Failure for Pump Stations and Major CSO facilities

Component Score	Scoring Range	Notes
Landslide	1-5	Based on susceptibility mapping
Floatation	1-5	Based on susceptibility mapping
Structural Failure M7.0 SFZ	1-5	Based on engineering evaluation of vulnerability to shaking
Structural Failure M9.0 CSZ	1-5	Based on engineering evaluation of vulnerability to shaking
Failure at Mainline Connection	1-5	Based on engineering evaluation of vulnerability to shaking
Inundation and Scour Tsunami	1-5	Based on available inundation mapping
Inundation and Scour Seiche	1-5	Based on approximate inundation mapping
Overall combined score	**1-5**	**Maximum of all component scores above**

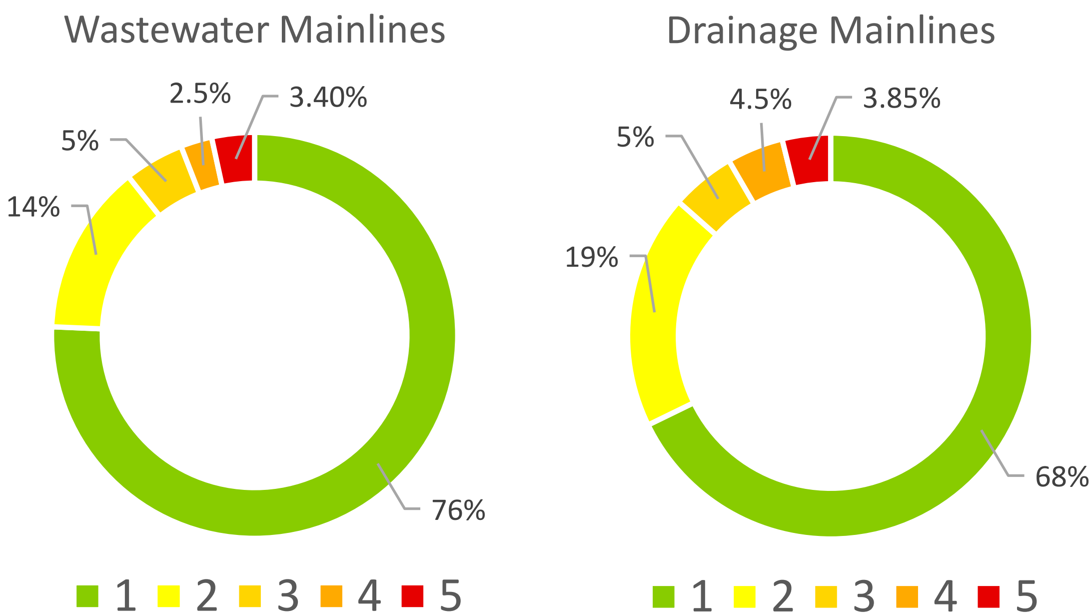


Figure 5-5 summarizes the likelihood of failure scores for drainage and wastewater mainlines. Figure 5-6 summarizes the likelihood of failure scores for pump stations and major CSO facilities.

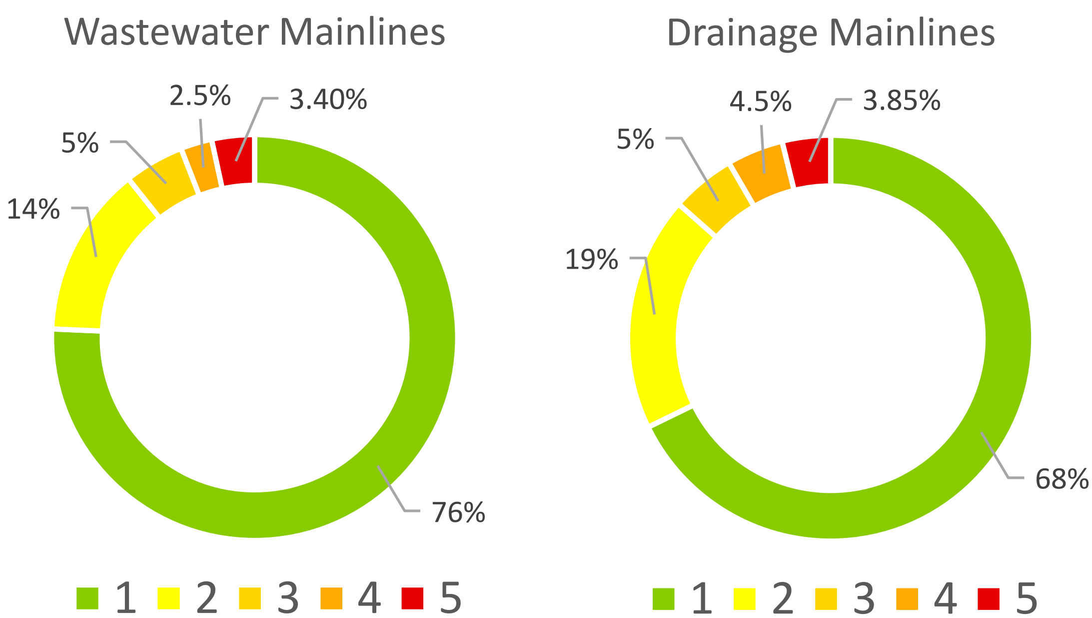
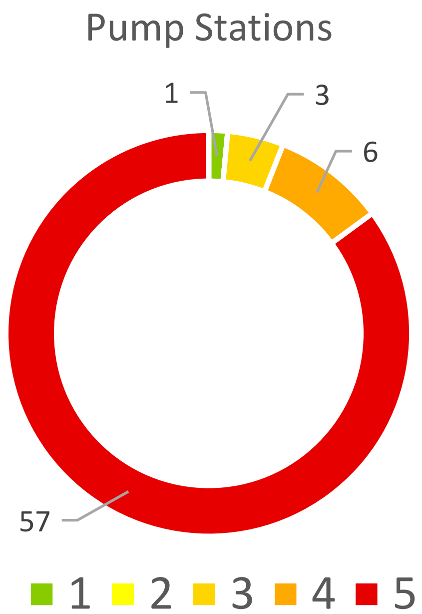


Figure 5-5. Summary of likelihood of failure scoring for mainlines



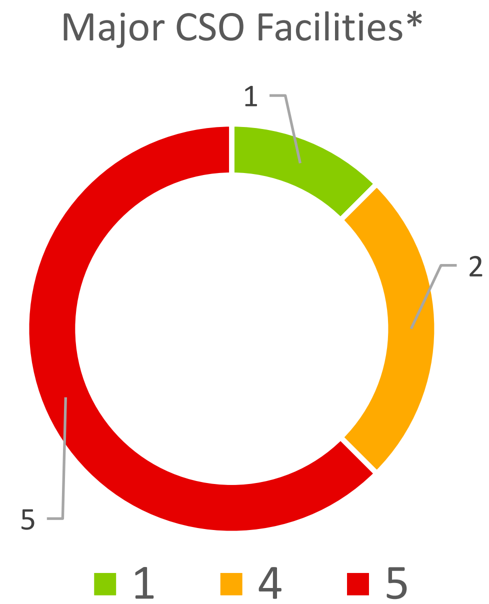


Figure 5-6. Summary of likelihood of failure scoring for pump stations and major CSO facilities

**Eight major CSO facilities, consisting of multiple large cast-in-place concrete structures, were evaluated using the desktop approach described in Section 5.2.1. Other CSO facilities, consisting primarily of large diameter mainline and a flow control device, were evaluated using the mainline assessment approach described in Section 5.1.*

The following appendices provide additional details on the likelihood of failure scoring:

- Appendix F: Example Calculation for Mainline Likelihood of Failure Score
- Appendix G: Likelihood of Failure Maps for Mainlines
- Appendix H: Example Calculation for Pump Station Likelihood of Failure Score
- Appendix I: Likelihood of Failure Data for Wastewater Pump Stations
- Appendix J: Example Calculation for CSO Facility Likelihood of Failure Score
- Appendix K: Likelihood of Failure Data for major CSO Facilities

6. Preliminary Seismic Risk Scoring

SPU developed an approach to calculating risk scores based on factors of consequence, likelihood, and equity. Scoring methods and criteria were developed based on methods outlined in SPU's *Risk Assessment Framework* (SPU 2007), staff subject matter expertise, and a review of past prioritization criteria developed and applied by SPU (SPU 2020). The basic equation for calculating risk scores is:

$$Risk\ Score\ = (Consequence\ Score\ \times\ Likelihood\ Score) + Equity\ Score$$

where the sum of all consequence scores does not exceed 5; the likelihood score ranges between 1 and 5, and the equity score ranges between 1 and 5. The resultant maximum risk score is 30. When the component scores were developed, scoring methodologies from existing and previous studies were considered, as well as the method developed for the Wastewater System Analysis (WWSA) (SPU 2019) and Drainage System Analysis (DSA) (SPU 2020). The following sections describe the scoring process based on component scores for the consequence, likelihood, and equity. Appendix L provides examples of risk score calculations.

6.1 Consequence Score

Consequence refers to the potential damages or impacts that could result from failure of the asset during a seismic event. Consequence scores were calculated by summing component scores for various criteria as shown in the equation below. Component scores were allocated to make sure the total consequence score does not exceed the maximum of 5.

$$\begin{aligned} &Consequence\ Score \\ &\qquad = Capacity\ Score + High\text{-}use\ Area\ Score + Critical\ Facility\ Score \\ &\qquad + \ Major\ Transportation\ Route\ Score\ +\ Environmental\ Impact\ Score \end{aligned}$$

Each asset category (wastewater mainlines, drainage mainlines, wastewater pump stations, and major CSO facilities) has unique characteristics that were accounted for in the consequence criteria.

- **Capacity.** The capacity criterion is based on how much wastewater or drainage flow may be impacted by an asset/facility failure. For wastewater mainlines and drainage mainlines, it is defined by the diameter of the mainline. For wastewater pump stations, it is defined by the average daily operating inflow. For major CSO facilities, it is defined by the storage volume.
- **High-use areas.** The high-use area criterion is based on whether a sewer overflow or flooding from an asset/facility failure is likely to impact an area where many pedestrians are present relative to other areas of the city. For all asset categories, it is defined by if the asset/facility is within the high use area boundary.
- **Critical facilities.** The critical facilities criterion is based on whether a critical community facility may be impacted by an asset/facility failure. For wastewater mainlines, wastewater pump stations, and major CSO facilities, it is defined by if the asset/facility is in the system backbone that provides wastewater service to the critical community facility (see Section 2.2 and Appendix B for additional information). For drainage mainlines, it is defined by if the asset is within 100 feet of the critical community facility parcel boundary.

- **Major transportation route.** The transportation impacts criterion is based on snow and ice routes for Seattle Department of Transportation, which are indicative of the major arterials within the city. In addition, lines associated with freeways (e.g., Interstate 5, Interstate 90, and State Route 520) were selected from the City's streets geodatabase. This criterion was applied only to wastewater and drainage mainlines because they are located within roadways. Wastewater pump stations and major CSO facilities are all in areas where there is limited impact to transportation.
- **Environmental impact.** The environmental impacts criterion is based on whether a sewer overflow would impact a water body or wetland. For wastewater mainlines, it is defined by if the asset is within 50 feet of a water body or within 20 feet of a wetland. For wastewater pump stations and major CSO facilities, it is defined by if the facility has an outfall to a water body. For drainage mainlines, this criterion is not included because it conveys only stormwater flow.

The consequence score is the sum of the criteria scores shown in Tables 6-1 through 6-4.

Table 6-1. Consequence Scoring for Drainage Mainlines

Scoring Criteria	Measurement	Score
Capacity	Mainline diameter is ≥ 48 inches	2.0
	Mainline diameter is ≥ 36 inches and < 48 inches	1.5
	Mainline diameter is ≥ 14 inches and < 36 inches	1.0
	Mainline diameter < 14 inches	0.5
High-use area	Impact occurs in an identified high use area	1.0
Critical facilities	Mainline is within 100 feet of critical community facility parcel boundary and impacts service to critical community facility	1.0
Major transportation route	Mainline is impacts major transportation route	1.0
Environmental impact	Not used	0.0

Table 6-2. Consequence Scoring for Wastewater Mainlines

Scoring Criteria	Measurement	Score
Capacity	Mainline diameter >14 inches	1.0
	Mainline diameter ≤ 14 inches	0.5
High-use area	Impact occurs in an identified high use area	1.0
Critical facilities	Mainline is in system backbone and impacts service to critical community facility	1.0
Major transportation route	Mainline is impacts major transportation route	1.0
Environmental impact	Within 50 feet of water bodies (lakes, creeks, rivers, Puget Sound) **OR** within 20 feet of wetlands	1.0

Table 6-3. Consequence Scoring for Wastewater Pump Stations		
Scoring Criteria	**Measurement**	**Score**
Capacity	Average annual inflow ≥ 100 GPM	2.0
	Average annual inflow ≥ 50 GPM and <100 GPM	1.5
	Average annual inflow ≥ 25 GPM and <50 GPM	1.0
	Average annual inflow <25 GPM	0.5
High-use area	Impact occurs in an identified high use area	1.0
Critical facilities	Pump Station serves critical community facility	1.0
Major transportation route	Not used	0.0
Environmental impact	Pump Station overflows to waterbody	1.0

Table 6-4. Consequence Scoring for Major CSO Facilities		
Scoring Criteria	**Measurement**	**Score**
Capacity	Storage Volume ≥ 2,000,000 gallons	2.0
	Storage volume ≥ 1,000,000 gallons and < 2,000,000 gallons	1.0
	Storage volume < 1,000,000 gallons	0.5
High-use area	Impact occurs in an identified high-use area	1.0
Critical facilities	CSO Facility serves critical community facility	1.0
Major transportation route	Not used	0.0
Environmental impact	CSO Facility overflows to waterbody	1.0

6.2 Likelihood Score

The risk increases with the probability, or likelihood, that a failure will occur. Section 5 describes the desktop analyses and assessments used to develop likelihood of failure scores for wastewater pump stations, major CSO facilities, drainage mainlines, and wastewater mainlines.

6.3 Equity Score

The equity score is used to acknowledge that areas of racial and socioeconomic disparity are at a relative disadvantage to recover from a sewer overflow or flooding. SPU provided the City's Racial and Social Equity Composite Index geospatial mapping which has polygons representing 136 census tracts throughout the city. In these data, tracts were assigned an index based on racial diversity, demographics, health outcomes, and socioeconomic factors. Data were derived from studies by the U.S. Census Bureau, Centers for Disease Control and Prevention, Washington State Department of Health, and Public Health–Seattle & King County. The range of indices was divided into five equity categories which reflect levels of disadvantage. The tracts categorized as having the highest level of disadvantage were assigned a score of 5. The areas categorized as having the lowest level of disadvantage were assigned a score of 1 (Table 6-5).

The equity score is based on the Racial and Social Equity Index developed by OPCD. The composite index includes measures of race, English speaking ability, national origin, socioeconomic disadvantage, and health disadvantage. The index is mapped by census tract and ranges from 1 (low) to 5 (high) racial and social equity disadvantage and priority as shown in Table 6-5.

Table 6-5. Equity Scores

Scoring Criteria	Level of Disadvantage	Score
Equity	High disadvantage and priority	5
	Medium-high	4
	Medium	3
	Medium -low	2
	Low disadvantage and priority	1

This element could have been incorporated into the consequence criteria. However, SPU decided to provide the equity score separately and let it be a significant factor in calculating the final score.

7. Results Summary

The Team calculated risk scores for mainlines, pump stations, and major CSO facilities as described in Section 6. Risk scores for mainlines were calculated for each mainline segment. Risk scores for pump stations and major CSO facilities were assigned to each facility as a site (not individual components).

Risk score ranges were assigned for each asset category: wastewater mainlines, drainage mainlines, wastewater pump stations, and major CSO facilities. Each asset category had a different sample size and risk score methodology which was considered when assigning the risk score range. Relative risk categories (low, medium, high, etc.) were assigned based on the assigned risk score ranges as shown in Table 7-1.

Table 7-1. Risk Categories and Scores

Relative Risk Category	Risk Score Range			
	Wastewater Mainlines	Drainage Mainlines	Wastewater Pump Stations	Major CSO Facilities
Very Low	0 - 4	0 - 5	none	none
Low	4 - 6	5 - 7	0 - 9	none
Medium Low	6 - 8	7 - 10	9 - 12	0 - 12
Medium	8 - 10	10 - 12	12 - 15	12 - 15
High	10 - 14	12 - 15	15 - 19	15 - 19
Critical	>14	>15	>19	>19

Risk scores for drainage mainlines are shown in Figures 7-1 through 7-4. Risk scores for wastewater mainlines, pump stations, and major CSO facilities are shown in Figures 7-5 through 7-8. Similar maps are provide in Appendix M.

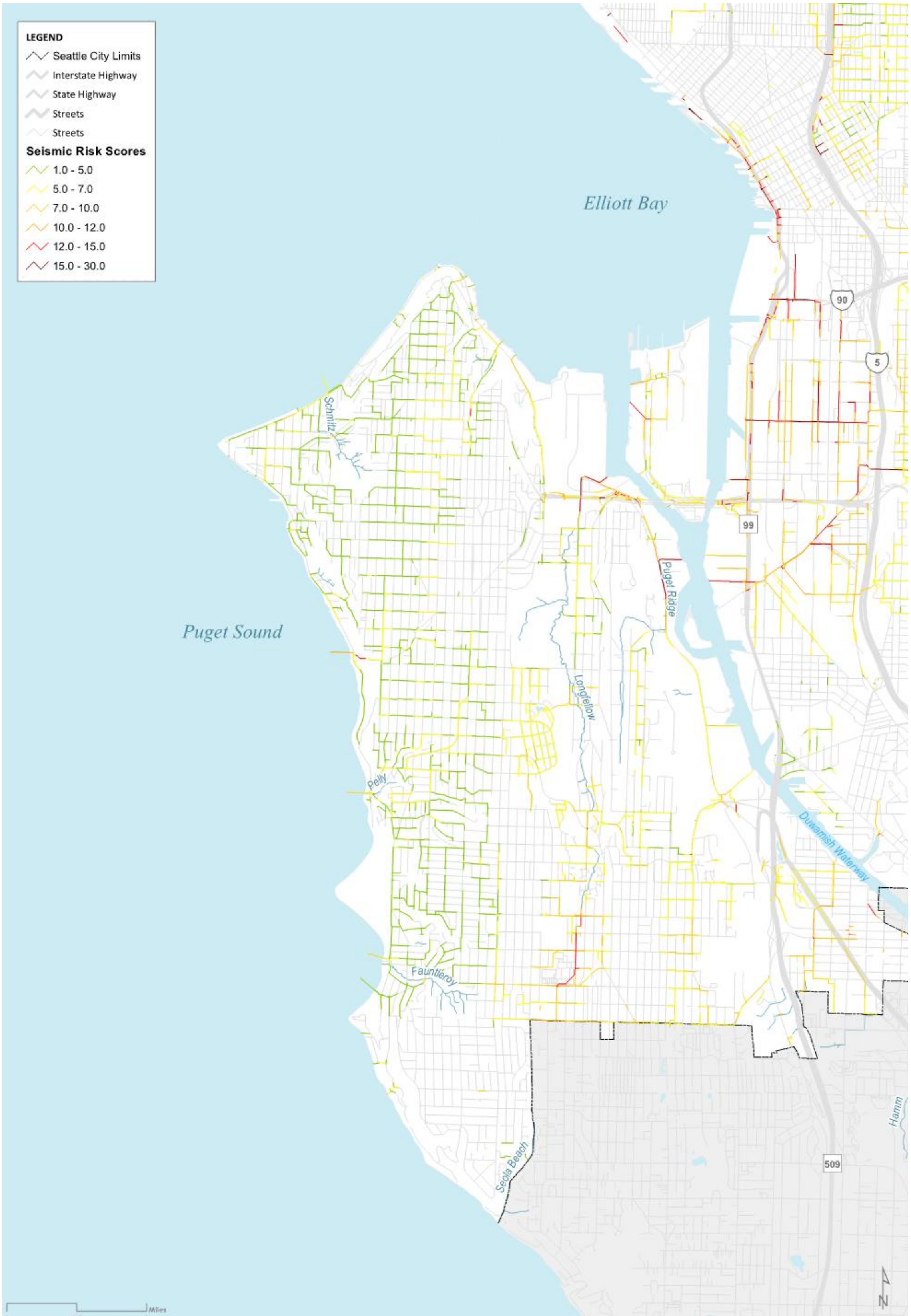


Figure 7-1. Risk scores for drainage mainlines, southwest

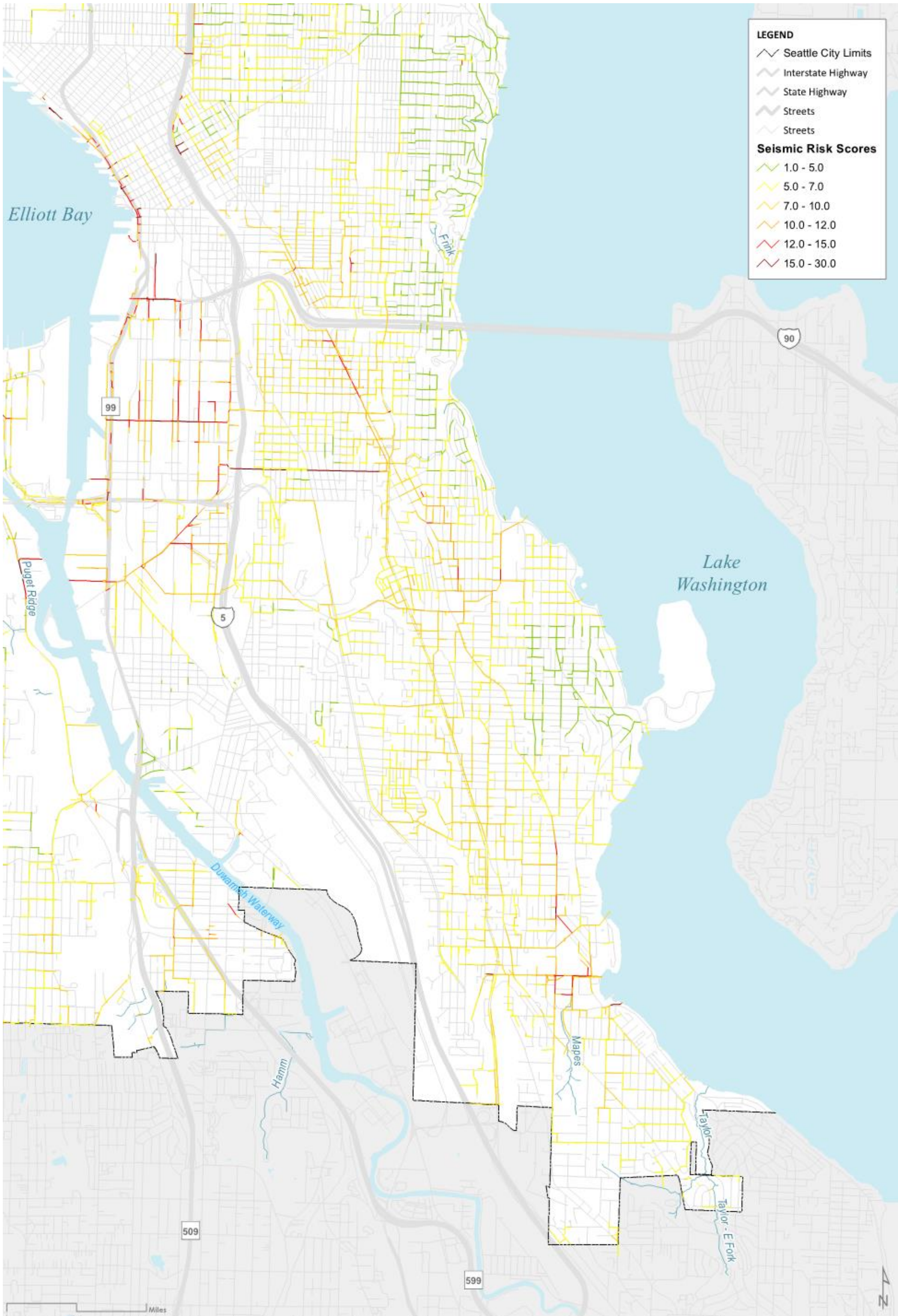


Figure 7-2. Risk scores for drainage mainlines, southeast

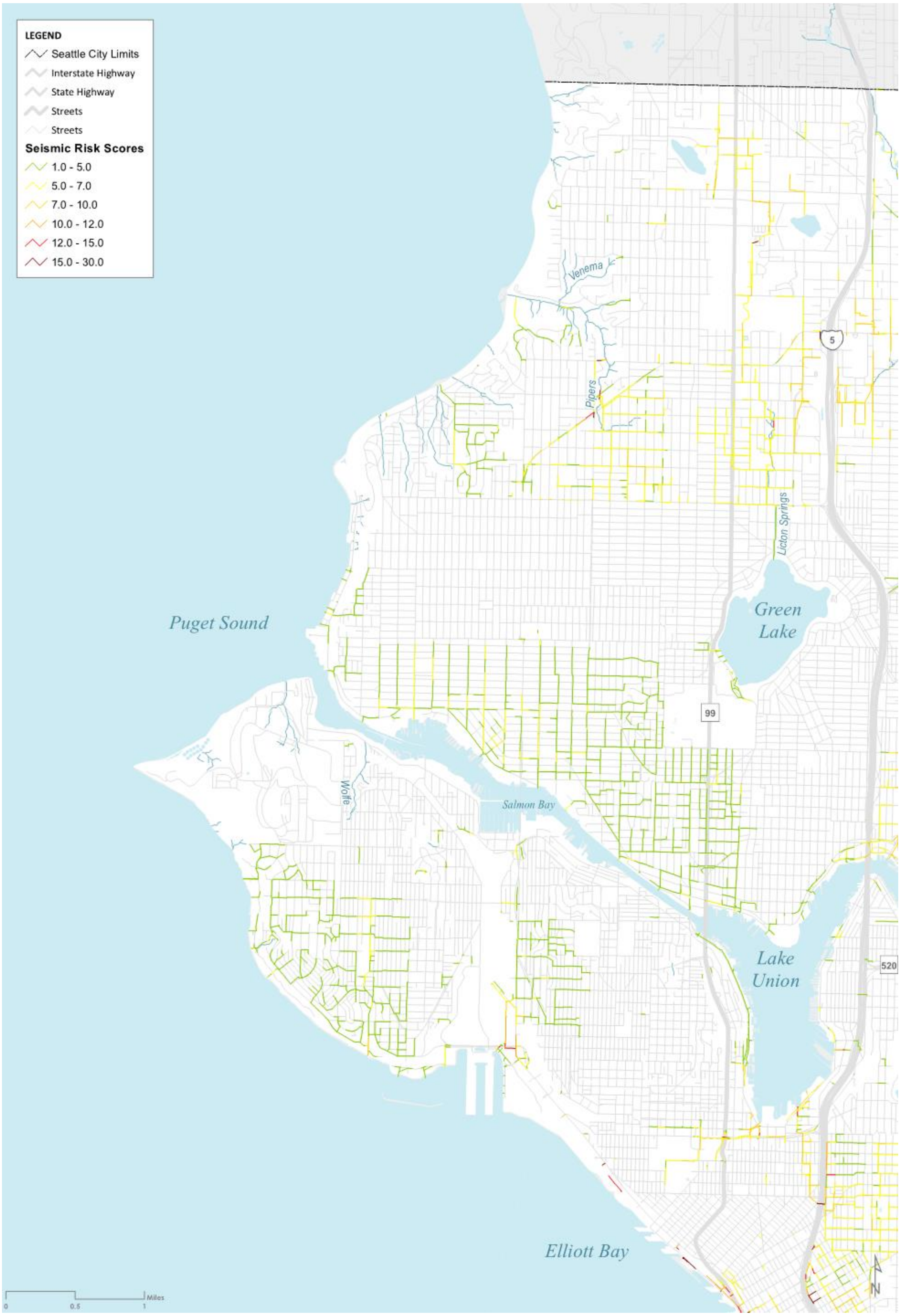


Figure 7-3. Risk scores for drainage mainlines, northwest

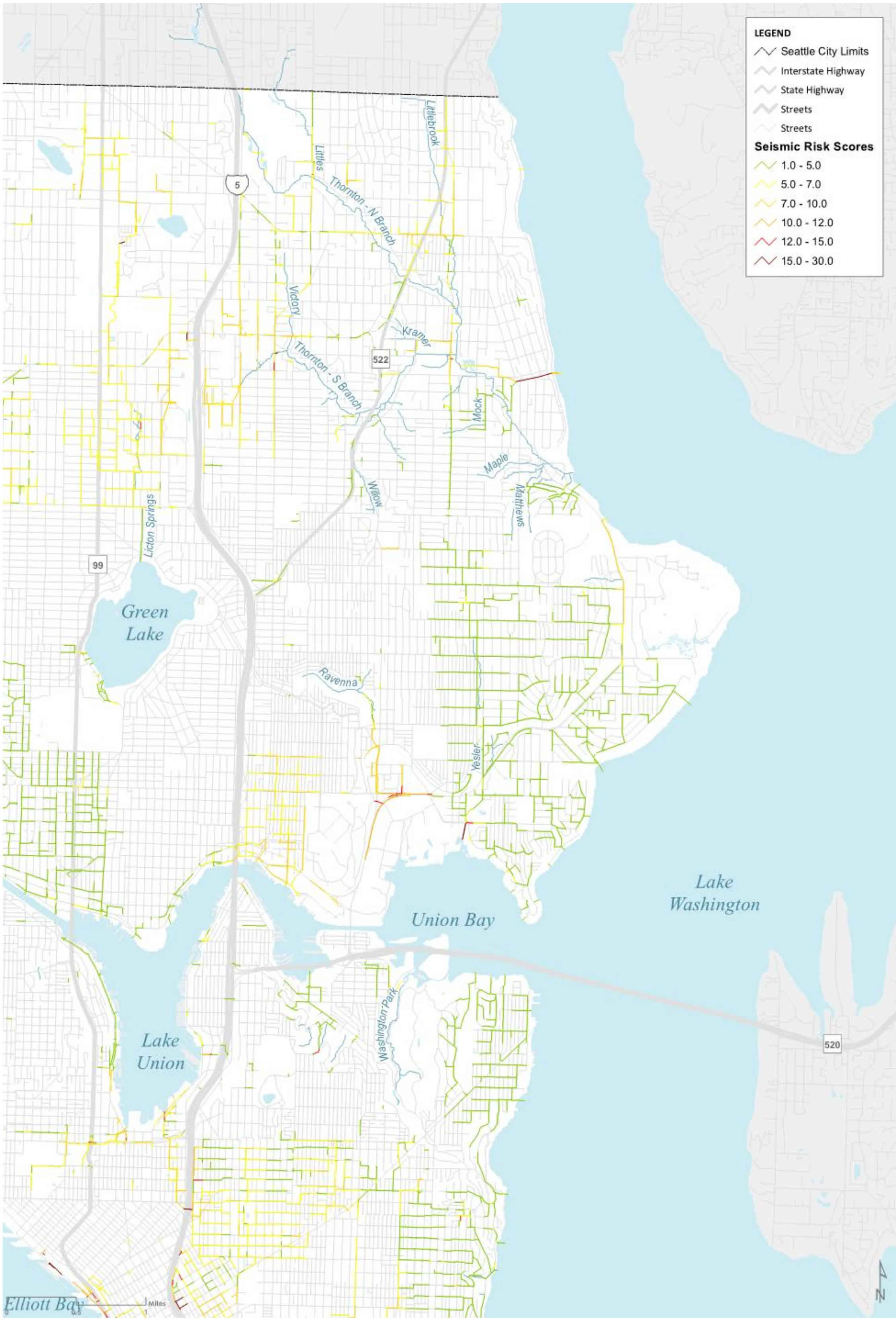


Figure 7-4. Risk scores for drainage mainlines, northeast

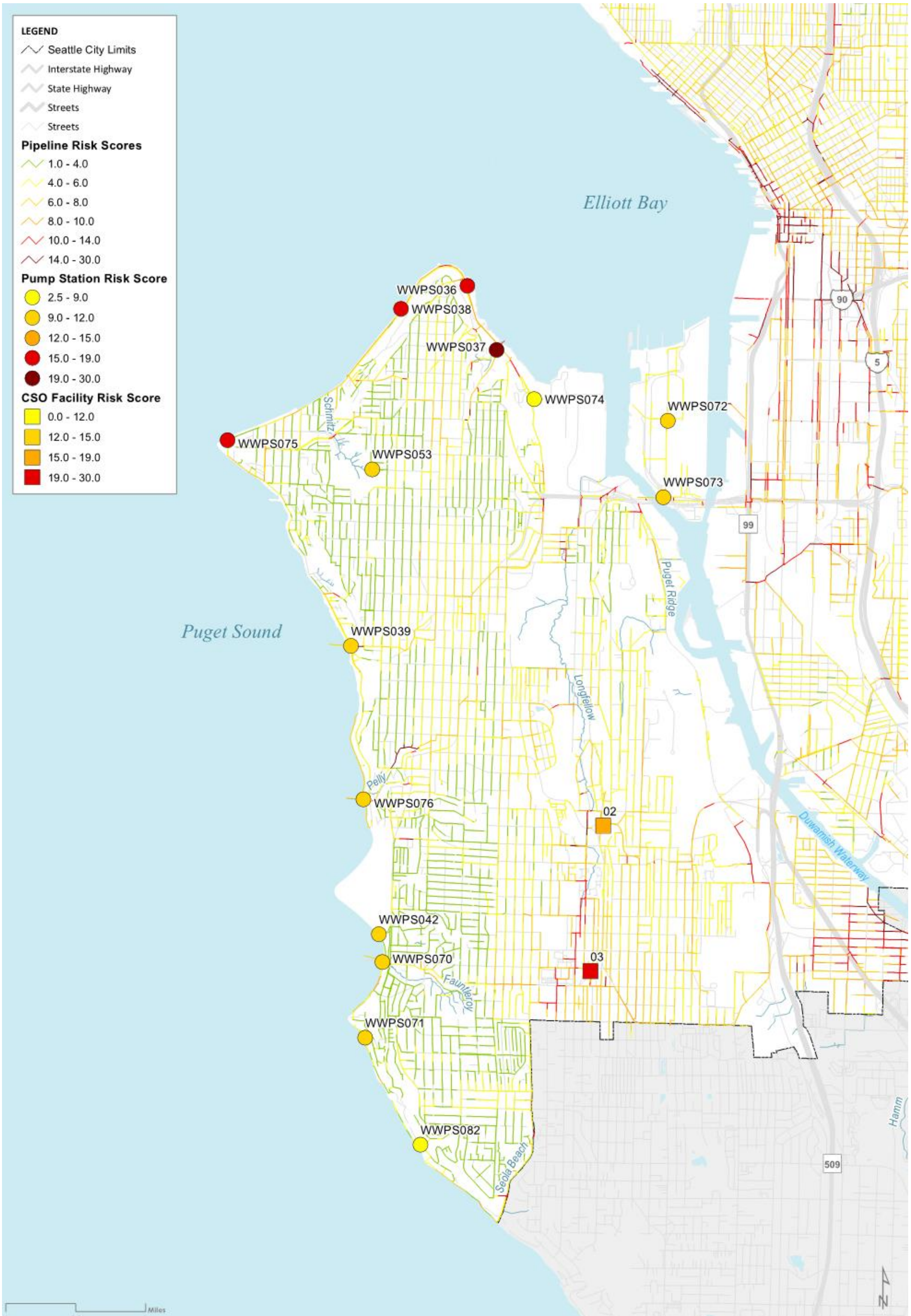


Figure 7-5. Risk scores for the wastewater system, southwest

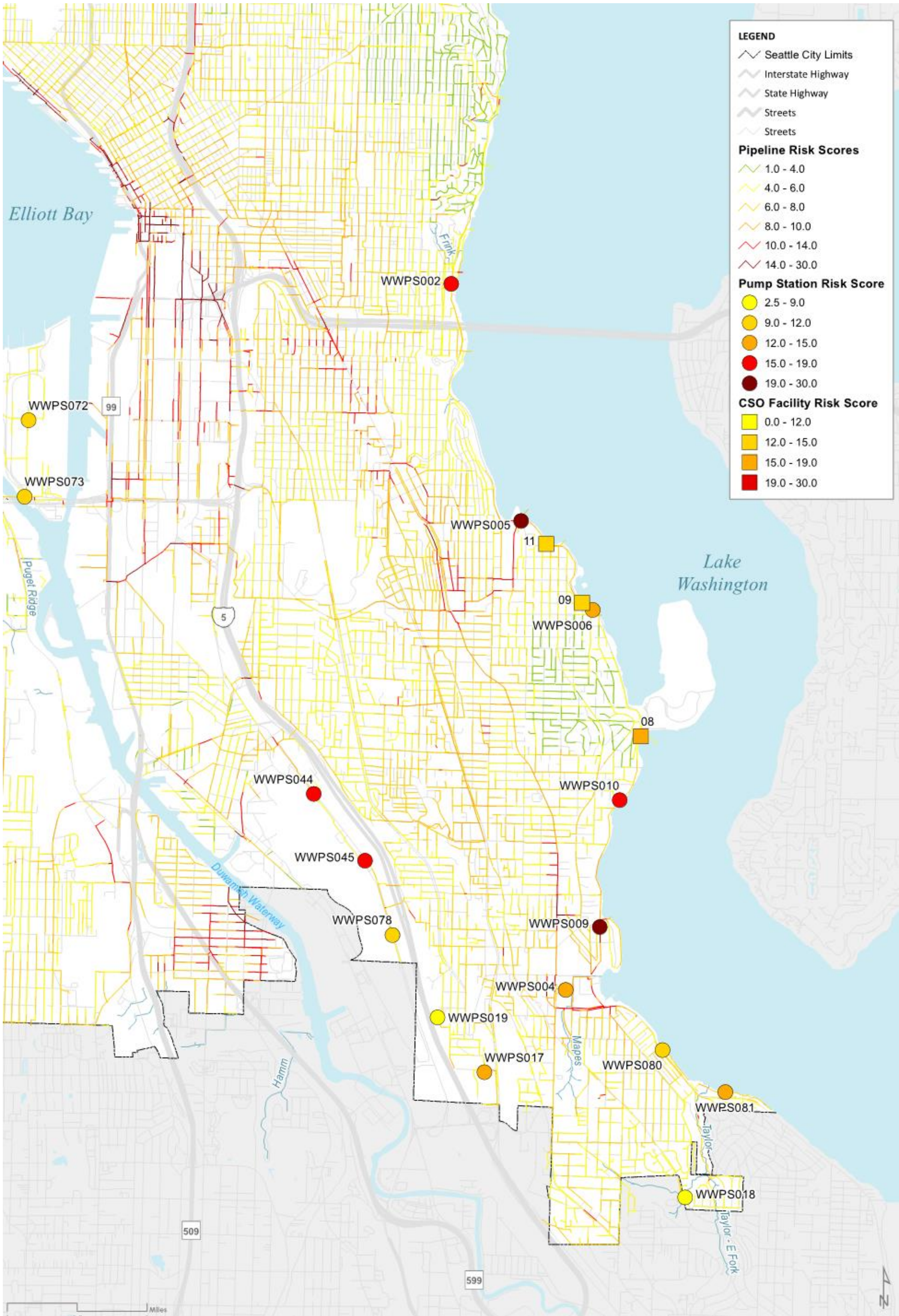


Figure 7-6. Risk scores for the wastewater system, southeast

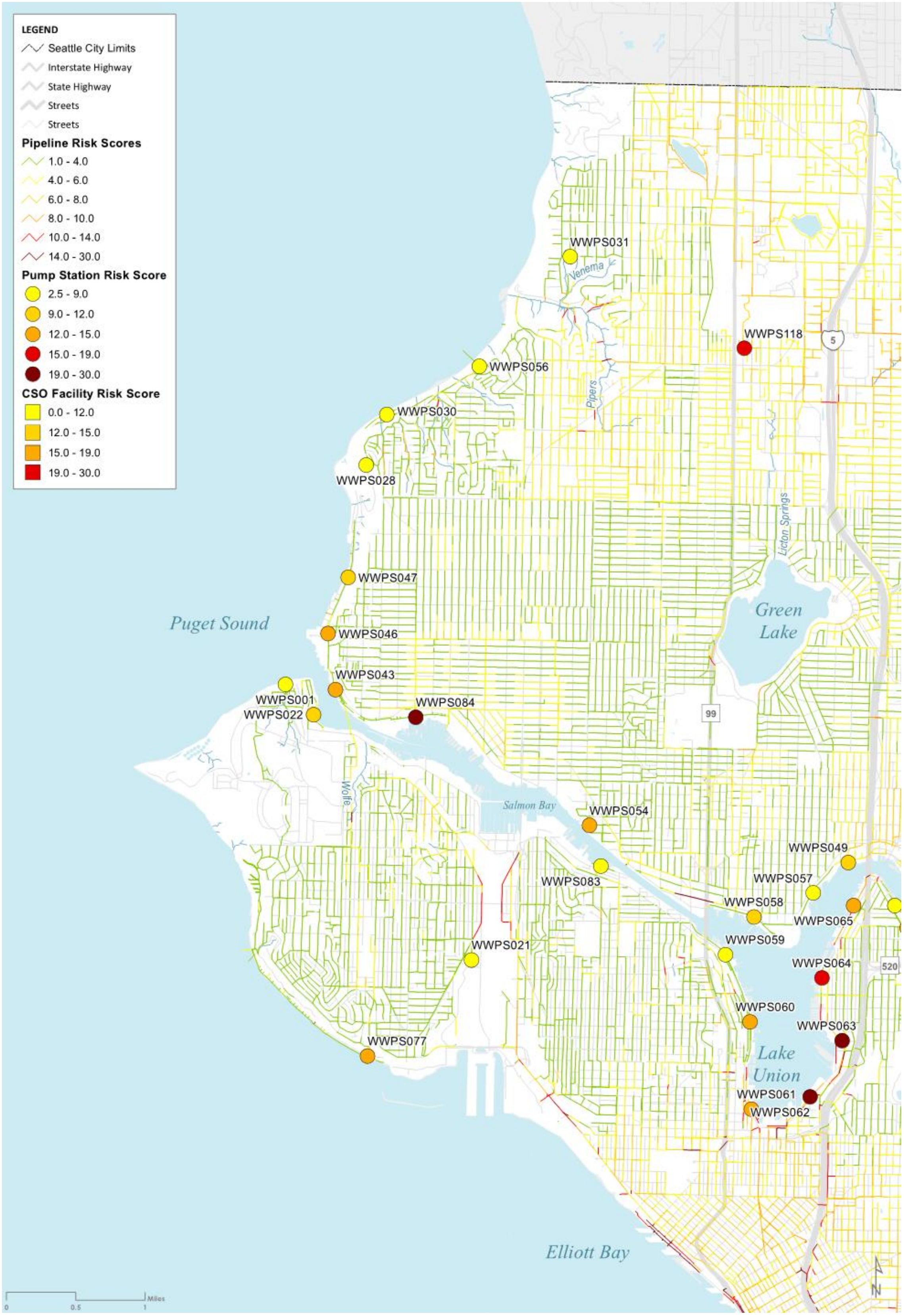


Figure 7-7. Risk scores for the wastewater system, northwest

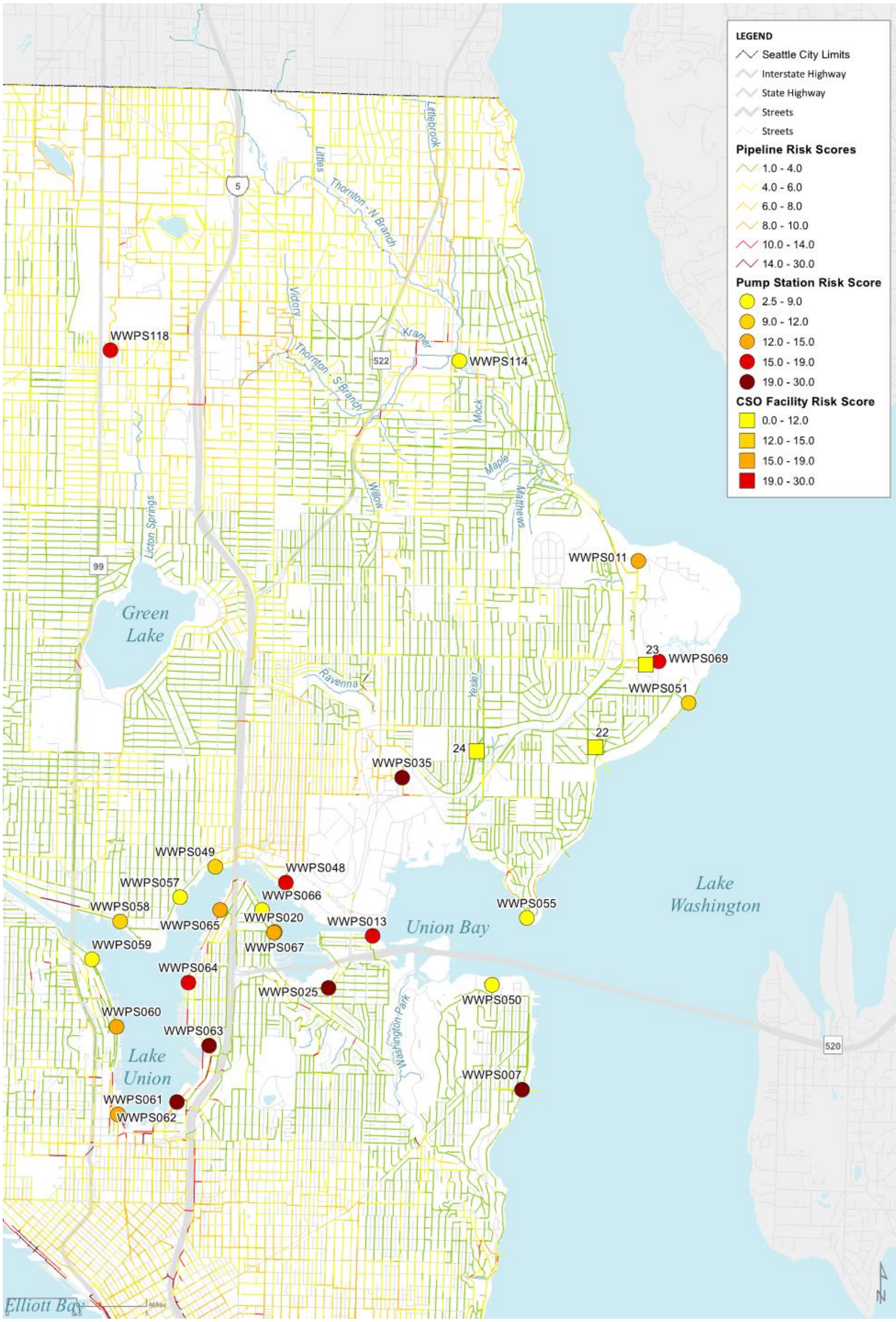


Figure 7-8. Risk scores for the wastewater system, northeast

In summarizing the results, the Team found that 3.3 percent of wastewater mainlines, 2.4 percent of drainage mainlines, 21 of 67 wastewater pump stations, and 3 of 8 major CSO facilities were categorized as either high or critical in terms of risk. Figures 7-9 through 7-12 illustrate the distribution of relative risk scoring categories for drainage mainlines, wastewater mainlines, pump stations, and major CSO facilities, respectively.

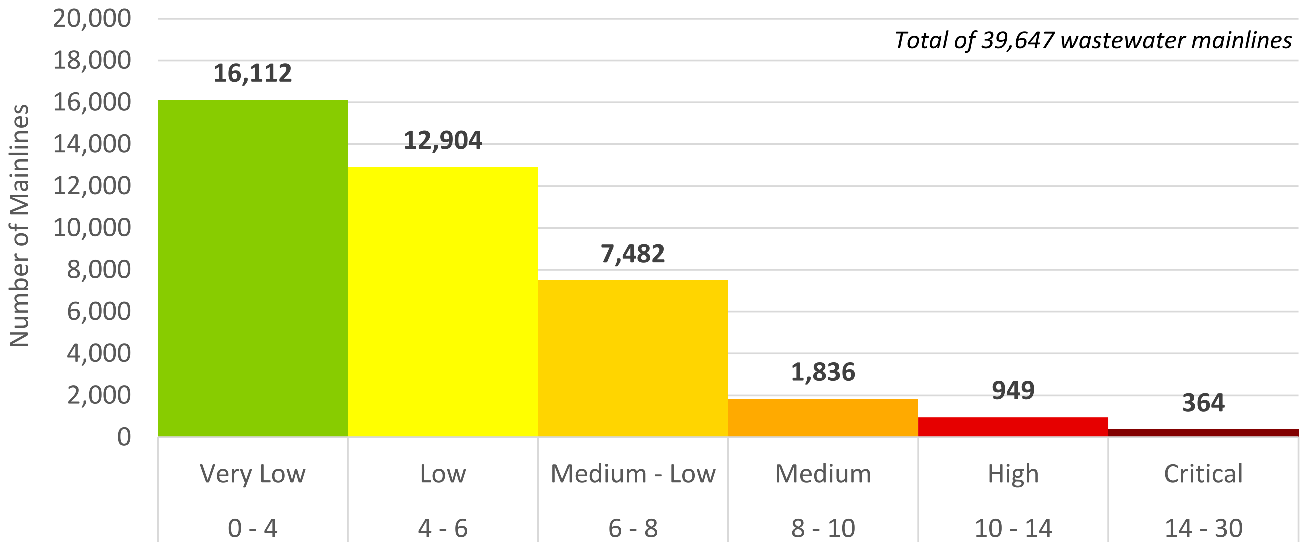


Figure 7-9. Distribution of total risk scores for wastewater mainlines

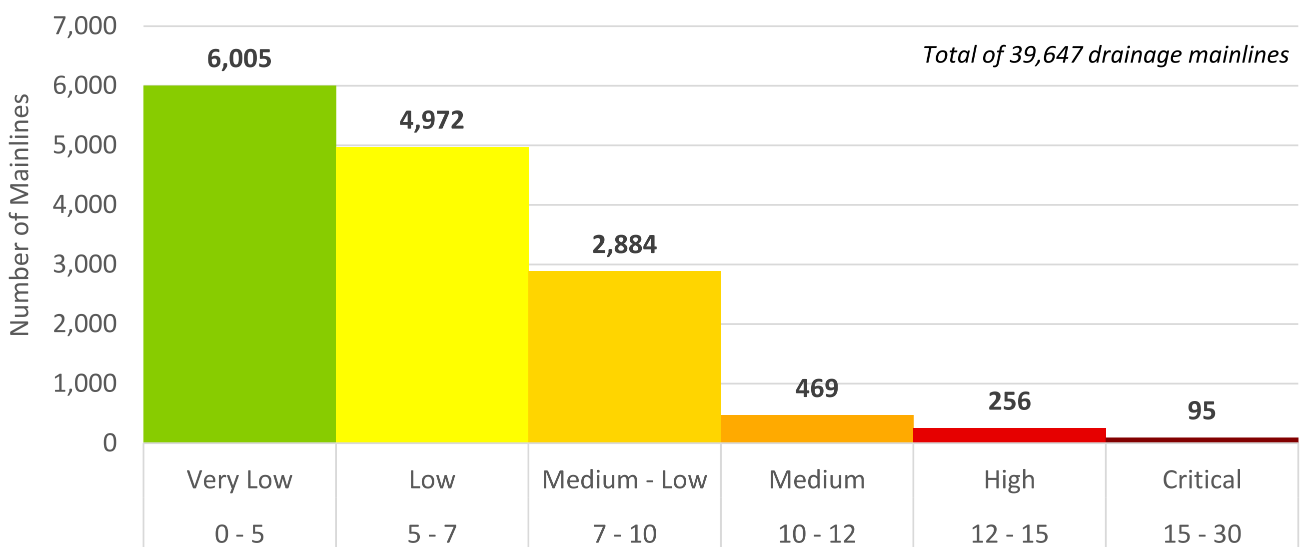


Figure 7-10. Distribution of total risk scores for drainage mainlines

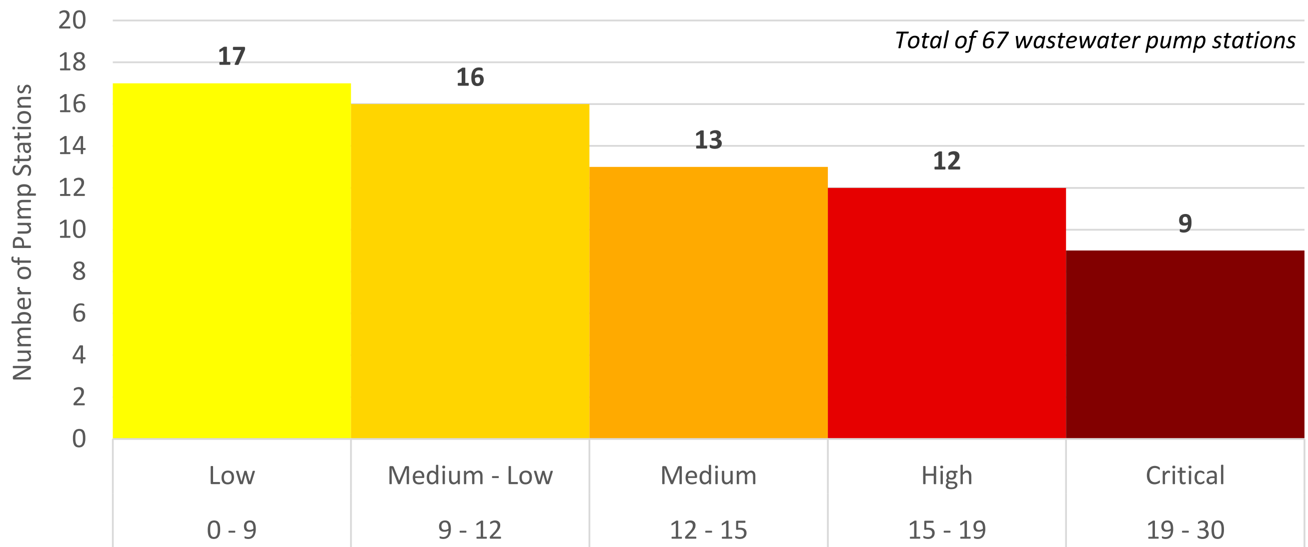


Figure 7-11. Distribution of total risk score for wastewater pump stations

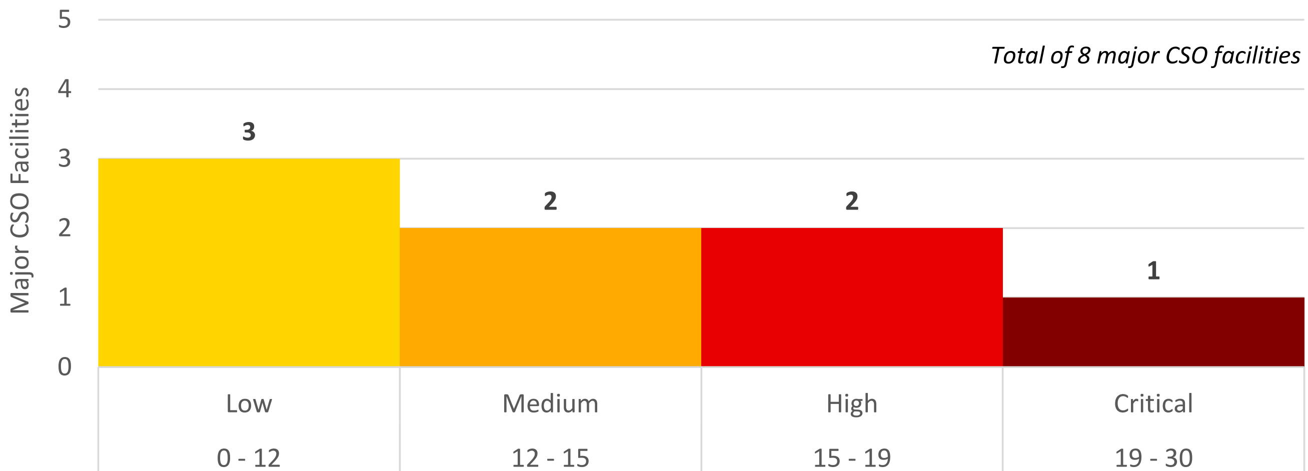


Figure 7-12. Distribution of total risk score for major CSO facilities

Eight major CSO facilities, consisting of multiple large cast-in-place concrete structures, were evaluated using the desktop approach described in Section 5.2.1. Other CSO facilities, consisting primarily of large diameter mainline and a flow control device, were evaluated using the mainline assessment approach described in Section 5.1.

As described in previous sections, likelihood of failure scores were based on assessments pertaining to two earthquakes: M 9.0 Cascadia Subduction Zone and M 7.0 Seattle Fault Zone earthquakes. The effects of transient and permanent ground deformation effects on the wastewater and drainage systems are estimated for both types of earthquakes. Simultaneously, the risk scoring assumes that flooding from tsunami and seiches, as well as landslides, will occur. However, is unlikely that all these factors will occur at the same time with the intensities assumed in the study. The risk scoring, therefore, tends to be conservative with an emphasis on low lying facilities near the waterfront. Many pump stations are near the waterfront in locations of low elevation and high water table. These locations are especially vulnerable to flooding from tsunamis and seiches, as well as liquefaction effects. The Team evaluated the basis for the high-risk assessments, as summarized below:

- Fourteen and 12 pump stations, respectively, are in the inundation zones for seiches and tsunamis and received the highest likelihood scores of 5 (see Tables I-4 and I-5 of Appendix I).
- Eleven pump stations received the highest likelihood rating of 5 because they are located in mapped landslide zones (See Table I-6 of Appendix I).
- Four pump stations received the highest likelihood rating of 5 because of potential liquefaction-induced floatation (See Table I-6).
- Twenty-seven pump stations received likelihood scores of 5 related to mainline connection failures.

The scores in the risk categories of consequence, likelihood, and equity are shown in Figures 7-13 through 7-16 and are summarized below. Risk score maps, by relative risk category, are provided in Appendix M.

- Four percent of both wastewater and drainage mainlines had a **consequence** score of 4 or greater, while 89 percent of the wastewater mainlines and 87 percent of the drainage mainlines had a **likelihood** score of 2 or less. Additionally, 3.3 percent of wastewater mainlines and 2.4 percent of drainage mainlines were categorized as either high or critical. The wastewater and drainage mainline relative risk category distribution indicates that as severity increases, there is a decreasing number of assets in each relative risk category.
- Wastewater pump stations and major CSO facilities have the highest degree of vulnerability during a seismic event (see the expanded discussion below). As shown in Figure 7-15 and Figure 7-16, a majority of both pump stations (57 of 67) and major CSO facilities (5 of 8) had a **likelihood** score of 5, signifying a likely impact during a seismic event.
- With regard to equity, 24 percent of wastewater mainlines, 34 percent of drainage mainlines, 10 of 67 pump stations, and 3 of 8 major CSO facilities fell in the highest priority/most disadvantaged category of the **equity** score.

The results show that the pump stations are particularly vulnerable to earthquake effects and associated tsunami or seiches. As indicated in Figure 7-15 **Error! Reference source not found.**, many wastewater pump stations have a high likelihood of failure, which is somewhat offset by lower consequences and relatively low equity.

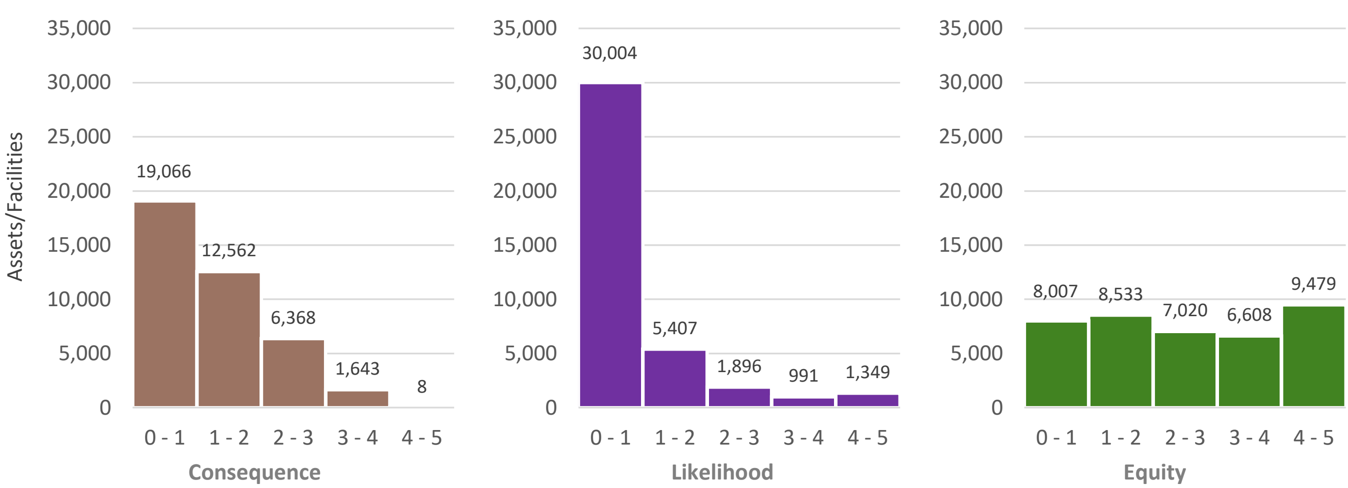


Figure 7-13. Distribution of consequence, likelihood, and equity for wastewater mainlines

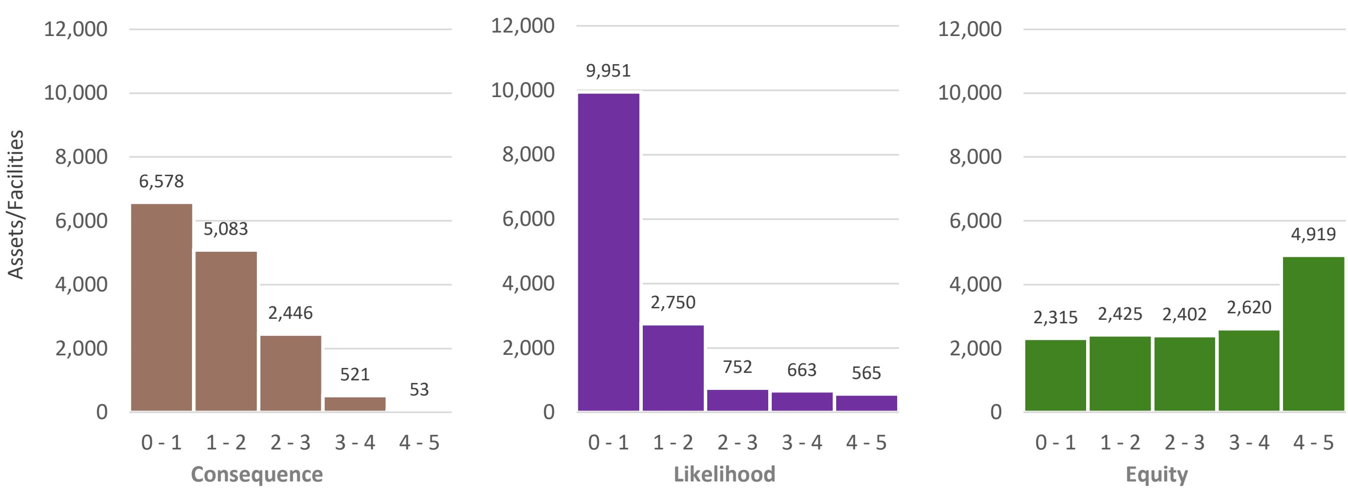


Figure 7-14. Distribution of consequence, likelihood, and equity for drainage mainlines

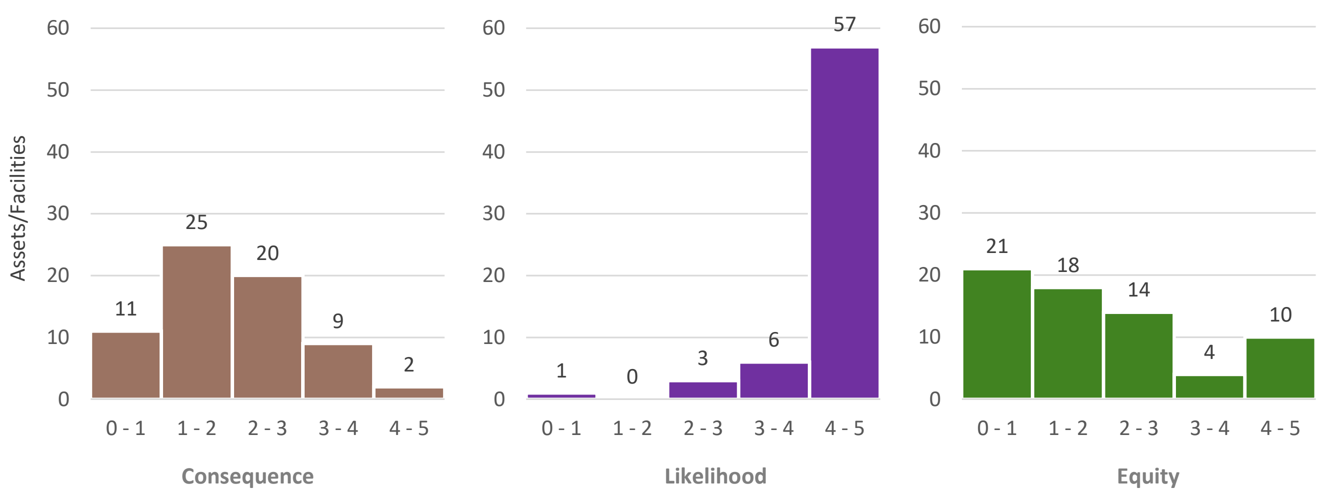


Figure 7-15. Distribution of consequence, likelihood, and equity for wastewater pump stations

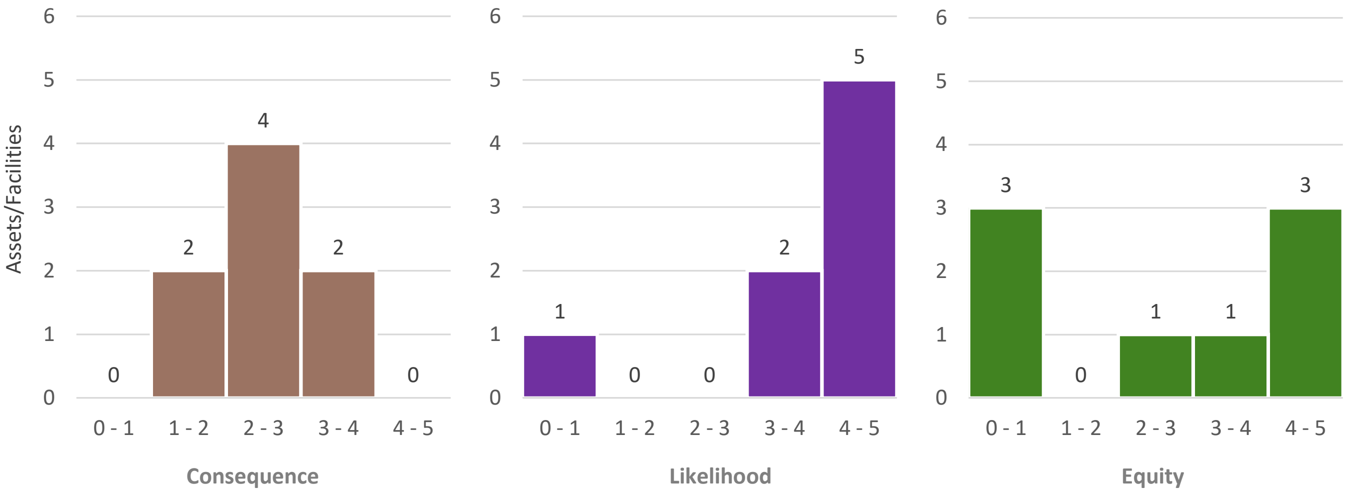


Figure 7-16. Distribution of consequence, likelihood, and equity for major CSO facilities

Eight major CSO facilities, consisting of multiple large cast-in-place concrete structures, were evaluated using the desktop approach described in Section 5.2.1. Other CSO facilities, consisting primarily of large diameter mainline and a flow control device, were evaluated using the mainline assessment approach described in Section 5.1.

Going forward, efforts to mitigate earthquake risks should include pump stations and major CSO facilities with high risk scores near the waterfront. Focusing on these areas will also address concerns associated with sea level rise and flooding related to climate change.

8. Conclusions and Recommendations

The purpose of the work summarized in this report was to perform a desktop assessment of SPU's drainage and wastewater mainlines, wastewater pump stations, and combined sewer overflow (CSO) facilities so that those at higher risk in a seismic event could be identified. In addition, risk scores are provided in the report, and will be used to categorize high-risk facilities and mainlines for subsequent planning.

The report is based on two earthquake scenarios: (1) magnitude 7.0 earthquake occurring in the Seattle Fault Zone (M7.0 SFZ) and (2) magnitude 9.0 earthquake occurring on the Cascadia Subduction Zone (M9.0 CSZ). The seismic risk assessment for the wastewater system is based on the same earthquake scenarios and corresponding geotechnical hazard data sets as the WSSS. These data include: peak ground acceleration (PGA), peak ground velocity (PGV), spectral response acceleration parameter at short periods (~0.2 second period), spectral response acceleration parameter at a period of 1 second, liquefaction susceptibility and probability, liquefaction-induced permanent ground deformation (PGD), landslide susceptibility, landslide-induced PGD, and fault rupture PGD (M7.0 SFZ only). The investigation also considered the potential impact from a tsunami generated by a M7.3 SFZ scenario earthquake as well as seiches generated in Lakes Union and Washington.

SPU's risk-based scoring system was used to evaluate the risks associated with seismic hazards. Risk scores were developed for wastewater and drainage mainlines as well as pump stations and major CSO facilities. The risk scores were developed from likelihood, consequence, and equity scores. 3.3 percent of wastewater mainlines, 2.4 percent of drainage mainlines, 21 of 67 wastewater pump stations, and 3 of 8 major CSO facilities were categorized as either high or critical in terms of risk. Assets receiving high and critical risk scores should be prioritized for more detailed evaluations and possible upgrades. SPU should consider the following recommendations for additional planning.

1) **SPU should consider developing resilience goals for the two earthquake scenarios, conducting detailed seismic assessment to understand expected performance of the drainage and wastewater systems and developing associated recovery timeframes, and developing administrative and construction strategies to address the identified seismic and tsunami vulnerabilities.** While this seismic risk assessment has identified high risk facilities and mainlines, it does not answer questions related to (a) expected seismic performance of the SPU's wastewater system associated with the two earthquake scenarios, (b) estimated timeframe for SPU to restore its wastewater collection services, and (c) potential gaps between recovery expectations based on social and economic needs within the SPU's service area and expected restoration timeframe for the system.
2) **SPU should consider performing more detailed evaluations to mitigate earthquake risks for pump stations and CSOs with high and critical risk scores, especially those near the waterfront.** Many pump stations are near the waterfront in locations of low elevation and high water table. These locations are especially vulnerable to flooding from tsunamis and seiches, as well as liquefaction effects. Focusing on these areas will also address concerns associated with sea level rise and flooding related to climate change. More detailed investigations should include site-specific investigations to verify physical conditions and refine geotechnical and structural assessments.
3) **SPU should consider collaborating with KCWTD to confirm that post-event level of services goal and recovery of the SPU system will be compatible with those of KCWTD, and pace and sequence of SPU system post-event recovery align with those of the KCWTD system. Detailed assessment of pump stations and CSOs should include (a) development of performance objectives based on their post-**

event level of services goals, (b) site-specific seismic hazard study to quantify geological hazards, (c) a site visit to assess seismic performance of nonstructural components, including assessment of mainlines and conduits at their interface with facilities, and (d) detailed structural analysis of facilities. SPU wastewater system delivers collected wastewater to the KCWTD's conveyance and treatment systems. To support social and economic needs in the SPU's service area, the SPU's and the KCWTD's systems must function as one integrated system.

4) **SPU should consider coordinating across Lines of Business (DWW and Water) and with appropriate City Departments and private service providers (e.g., communications, energy) to enhance their understanding of dependencies between the various infrastructure systems and develop coordinated recommendations for resilience improvements.** SPU wastewater infrastructure is often co-located with, or otherwise dependent on other infrastructure systems (roadways, water pipelines, gas pipelines, etc.). As SPU's Water line of business, other City Departments, and private service providers continue to develop and implement resilience improvements for their systems, it will be important for all these agencies to establish an open dialogue, and for SPU DWW line of business to make appropriate refinements to the definition of the wastewater system backbone on a routine basis. Understanding how others depend on the wastewater system, or how restoration of system services could impact other systems, will inform actions to improve the robustness of backbone system.

5) **SPU should consider refining the ALA pipeline analysis used in this study for more detailed assessments of specific infrastructure.** While reasonable for a high-level desktop assessment, the ALA guidelines for repair rates for drainage and wastewater mainlines are nearly 20 years old. As part of future detailed infrastructure assessment, more refined mainline analysis (including those CSO facilities primarily consisting of large-diameter mainlines) should be considered to reflect effects of soil-structure interaction on mainline performance as appropriate. Since the majority of mainline breaks are caused by permanent ground deformation, a sensitivity analysis should be conducted for permanent ground deformation values that are less than 6 inches. This analysis will help refine the estimates for repair rates and improve assessed likelihood of failure of SPU mainlines.

9. Limitations

This document was prepared solely for Seattle Public Utilities in accordance with professional standards at the time the services were performed and in accordance with the contract between Seattle Public Utilities and Brown and Caldwell dated August 14, 2019. This document is governed by the specific scope of work authorized by Seattle Public Utilities; it is not intended to be relied upon by any other party except for regulatory authorities contemplated by the scope of work. We have relied on information or instructions provided by Seattle Public Utilities and other parties and, unless otherwise expressly indicated, have made no independent investigation as to the validity, completeness, or accuracy of such information.

Further, Brown and Caldwell makes no warranties, express or implied, with respect to this document, except for those, if any, contained in the agreement pursuant to which the document was prepared. All data, drawings, documents, or information contained this report have been prepared exclusively for the person or entity to whom it was addressed and may not be relied upon by any other person or entity without the prior written consent of Brown and Caldwell unless otherwise provided by the Agreement pursuant to which these services were provided.

References

Aqualyze, Inc., *SPU Wastewater System Analysis*, Seattle Public Utilities, 2019.

ASCE, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures, ASCE/SEI 7-16*, American Society of Civil Engineers, 2017.

ASCE, *Seismic Evaluation and Retrofit of Existing Buildings*, *ASCE 41-17*, American Society of Civil Engineers, Reston, VA, 2017.

American Lifelines Alliance (ALA), *Seismic Fragility Formulations for Water Systems, Part 1 and 2*, American Lifelines Alliance, 2001

Atwater, B., *Radiocarbon dating of a Seattle earthquake A.D. 900-930*, Seismological Research Letters, 70(2), p. 232, 1999.

Atwater, B., and Hemphill-Haley, E., *Recurrence intervals for great earthquakes of the past 3,500 years at northeastern Willapa Bay, Washington, USGS Professional Paper 1,576*, United States Geological Survey, 1997.

Barberopoulou, A., Qamar, A., Pratt, T., Creager, K., and Steele, W., *Local Amplification of Seismic Waves from the Denali Earthquake and Damaging Seiches in Lake Union, Seattle, Washington*, Geophysical Research Letters, Vol. 31, 2004.

Barberopoulou, A., *A Seiche Hazard Study for Lake Union, Seattle, Washington*, Bulletin of the Seismological Society of America, 98(4), 2008.

Blakely, R., Wells, R., Weaver, C., and Johnson, S., *Location, structure, and seismicity of the Seattle Fault, Washington: Evidence from aeromagnetic anomalies, geologic mapping, and seismic-reflection data*, Geological Society of America Bulletin, Vol. 114, p. 169-177, 2002.

Bonneau, A. and O'Rourke, T.D., Water Supply Performance During Earthquakes and Extreme Events, Report No. MCEER-03 0009, MCEER, University at Buffalo, Buffalo, NY, 2009.

Booth, D., Troost, K., and Shimel, S., *Geologic map of northwestern Seattle (part of the Seattle north 7.5'x15' quadrangle), King County, Scientific Investigations Map SIM 2903, scale 1:12,000*, United States Geological Survey, 2005.

Brocher, T., Blakely, R., and Wells, R., *Interpretation of the Seattle uplift, Washington, as a passive-roof duplex*, Bulletin of the Seismological Society of America, 94(4), p. 1379-1401, 2004.

Brocher, T., Parsons, T., Blakely, R., Christensen, N., Fisher M., Wells, R., and the SHIPS Working Group, *Upper crustal structure in Puget Lowland, Washington, results from the 1998 Seismic Hazards Investigation in Puget Sound*, Journal of Geophysical Research, 106(B7), p. 13,541-13,564, 2001.

Brown and Caldwell, *Drainage System Analysis, Flooding Topic Area, Drainage System Capacity Evaluation*, Seattle Public Utilities, 2020.

Bucknam, R., Hemphill-Haley, E., and Leopold, E., *Abrupt uplift within the past 1700 years of the southern Puget Sound, Washington*, Science, Vol. 258, p. 1,611-1,613, 1992.

City of Portland, *Bureau of Environmental Services Resiliency Master Plan*, 2016.

City of Seattle, *Seattle Hazard Identification and Vulnerability Analysis*, City of Seattle Office of Emergency Management, 2019.

Clague, J., *Evidence for large earthquakes at the Cascadia subduction zone*, Review of Geophysics, Vol. 35, p. 439-460, 1997.

Davido Consulting Group, Inc., *Sewer Pump Station Prioritized Capital Improvement Plan Report*, Seattle Public Utilities, 2015.

DNR, *Tsunami Hazard–GIS Data (In Preparation)*, State of Washington Department of Natural Resources, In Preparation.

Federal Emergency Management Agency (FEMA), *Hazus Earthquake Model Technical Manual*, Hazus 4.2 SP 3, 2020.

Federal Emergency Management Agency (FEMA), *Hazus-MH 2.1 Earthquake Technical Manual*. Mitigation Division, 2015a.

Federal Emergency Management Agency (FEMA), *FEMA P-154 Rapid Visual Screening of Buildings for Potential Seismic Hazards: A Handbook, Third Edition*. Applied Technology Council, National Earthquake Hazards Reduction Program. Washington, D.C., 2015b.

Goldfinger, C., Nelson, C., and Johnson, J., *Holocene earthquake records from the Cascadia subduction zone and northern San Andreas Fault based on precise dating of offshore turbidites*, Annual Review of Earth and Planetary Sciences, Vol. 31, p. 555-577, 2003.

Goldfinger, C., Nelson, C., Morey, A., Johnson, J., Patton, J., Karabanov, E., Gutierrez-Pastor, J., Eriksson, A., Gracia, E., Dunhill, G., Enkin, R., Dallimore, A., and Vallier, T., *Turbidite event history–Methods and implications for Holocene paleoseismicity of the Cascadia subduction zone, USGS Professional Paper 1661–F*, United States Geological Survey, 2012.

Harp, E., Michael, J., and Laprade, W., *Shallow-Landslide Hazard Map of Seattle, Washington, USGS Open-File Report 2006-1139*, United States Geological Survey, 2006.

Haugerud, R., Harding, D., Johnson, S., Harless, J., Weaver, C., and Sherrod, B., *High-resolution lidar topography of the Puget Lowland, Washington–A bonanza for earth science*, Geological Society of America GSA Today, 13(6), p. 4-10, 2003.

HDR, *Recommendations to Enhance the Resiliency and Recovery of King County's Regional Wastewater Treatment Facilities–Task 600 Resiliency Recommendations*, King County Wastewater Treatment Division, 2018a.

HDR, *Recommendations to Enhance the Resiliency and Recovery of King County's Regional Wastewater Treatment Facilities–Task 500 Preparedness and Recovery Recommendations*, King County Wastewater Treatment Division, 2018b.

Kelsey H., Sherrod B., Nelson A., and Brocher T., *Earthquakes generated from bedding plane-parallel reverse faults above an active wedge thrust, Seattle fault zone*, Geological Society of America Bulletin, Vol. 120, p. 1581-1597, 2008.

LCI, *Final Desktop Review and Summary of the Seattle and South Whidbey Island Fault Zones for Seattle Public Utilities,* Lettis Consultants International, 2016.

Makdisi, F., and Seed, H., *Simplified Procedure for Estimating Dam and Embankment Earthquake-Induced Deformations*, Journal of Geotechnical Engineering Division, 104(7), 1978.

NAG, *Task 2a–Refinement of Liquefaction-Induced Permanent Ground Displacements*, *Report prepared for Seattle Public Utilities*, New Albion Geotechnical, 2017.

National Institute of Building Sciences, *Earthquake Loss Estimation Methodology HAZUS 97: Technical Manual*. Prepared for Federal Emergency Management Agency, Washington, D.C., 1997 .

Nelson, A., Johnson, S., Kelsey, H., Wells, R., Sherrod, B., Pezzopane, S., Bradley, L., Koehler, R., and Bucknam, R., *Late Holocene earthquakes on the Toe Jam Hill Fault, Seattle Fault Zone, Bainbridge island, Washington*, Geological Society of America Bulletin, 115(11), p. 1388-1403, 2003.

Nelson, A., Personius, S., Sherrod, B., Kelsey, H., Johnson, S., Bradley, L., and Wells, R., *Diverse rupture modes for surface-deforming upper plate earthquakes in the southern Puget Lowland of Washington State*, Geosphere, 10(4), p. 769-796, 2014.

Nelson, A., Shennan, I., and Long, A., *Identifying coseismic subsidence in tidal-wetland stratigraphic sequences at the Cascadia Subduction Zone of western North America*, Journal of Geophysical Research, Vol. 101, p. 6115-6135, 1996.

O'Rourke, T.D., Memo to Emily Meza, Kelli Reddick, Craig Davis and Steve Reel, Assessment of Pipeline Fr*agilities: Memo 2 reissued,* December 30, 2019.

O'Rourke, T.D., Memo to Jon Bray, *Pipeline Fragilities for Natural Gas Pipelines*, May 2, 2020.

Oldham, K., "Metro: Municipality of Metropolitan Seattle," *HistoryLink.org*, June 8, 2016, https://historylink.org/File/7813 (April 16, 2020).

Perkins, W., and Malinak, A., *Ground Improvement Design and Construction for Seattle's Elliott Bay Seawall Replacement and Retrofit*, Deep Foundation Institute Journal, 13(2) p. 1-11, 2019.

Petersen, M., Shumway, A., Powers, P, Mueller, C., Moschetti, M., Frankel, A., Rezaeian, S., McNamara, D., Luco, N., Boyd, O., Rukstales, K., Jaiswai, K., Thompson, E., Hoover, S., Clayton, B., Field, E., and Zeng, Y., *The 2018 update of the US National Seismic Hazard Model: Overview of model and implications*, Earthquake Spectra 36(1), p. 5-41, 2020

Quantum Spatial, *Digital Elevation Model Developed from Light Detection and Ranging (LiDAR) Data Provided by King County and Puget Sound LiDAR Consortium*, 2016.

Satake, K., Shimazaki, K., Tsuji, Y., and Ueda, K., *Time and size of a giant earthquake in Cascadia inferred from Japanese tsunami records of January 1700*, Nature, 378(18), p. 246-249, 1996.

Shannon & Wilson, Inc., *Elliott Bay Seawall Geotechnical Engineering Report for City of Seattle Department of Transportation,* Project No. 21-1-21374-480, Document No. 13.01.04, Seattle, Washington, July, 2013.

SPU, *Seattle Public Utilities Strategic Asset Management Plan Update Wastewater Collection Pipes*, Seattle Public Utilities, 2015a.

SPU, *Seattle Public Utilities Wastewater Collection Pipe Criticality Criteria and Rating Scale*, Seattle Public Utilities, 2015b.

SPU, *Seattle Public Utilities Critical Pipes & SSO Map*, Seattle Public Utilities, 2016a.

SPU, *Seattle Public Utilities Pipe Criticality–Scoring, Process, & Current State of Data*, Seattle Public Utilities, 2016b.

SPU, *Seattle Public Utilities Strategic Asset Management Plan (SAMP) Update Wastewater Pump Stations and Force Mains*, Seattle Public Utilities, 2016.

SPU, *Seattle Public Utilities Asset Management Plan (AMP) Combined Sewer Overflow Facilities*, Seattle Public Utilities, 2018a.

SPU, *Seattle Public Utilities Water System Seismic Study Summary Report*, Seattle Public Utilities, 2018b.

SPU, *Seattle Public Utilities Wastewater System Analysis,* Seattle Public Utilities, 2019.

SPU, *Seattle Public Utilities Drainage System Analysis,* Seattle Public Utilities, 2020.

TCLEE, *Seismic Screening Checklists for Water and Wastewater Facilities, TCLEE Monograph No. 22*, American Society of Civil Engineers, Technical Council on Lifeline Earthquake Engineering, Reston, VA, 2002.

Walsh, T., Titov, V., Venturato, A., Mofield, H., and Gonzalez, F., *Tsunami Hazard Map of the Elliott Bay Area, Seattle, Washington: Modeled Tsunami Inundation from a Seattle Fault Earthquake*, Washington Department of Natural Resources, 2003.

WGS, *Tsunami Hazard–GIS Data: Washington Geological Survey Digital Data Series 22, Version 1.0*, Washington Geological Survey, 2019.

Wirth, E., Chang, S., and Frankel, A., *2018 report on incorporating sedimentary basin response into the design of tall buildings in Seattle, Washington, USGS Open-File Report 2018–1149*, United States Geological Survey, 2018.

Yamaguchi, D., Atwater, B., Bunker, D., Benson, B., and Reid, M., *Tree-ring dating the 1700 Cascadia earthquake*, Nature, Vol. 389, p. 922-923, 1997.

Appendix A: System Maps

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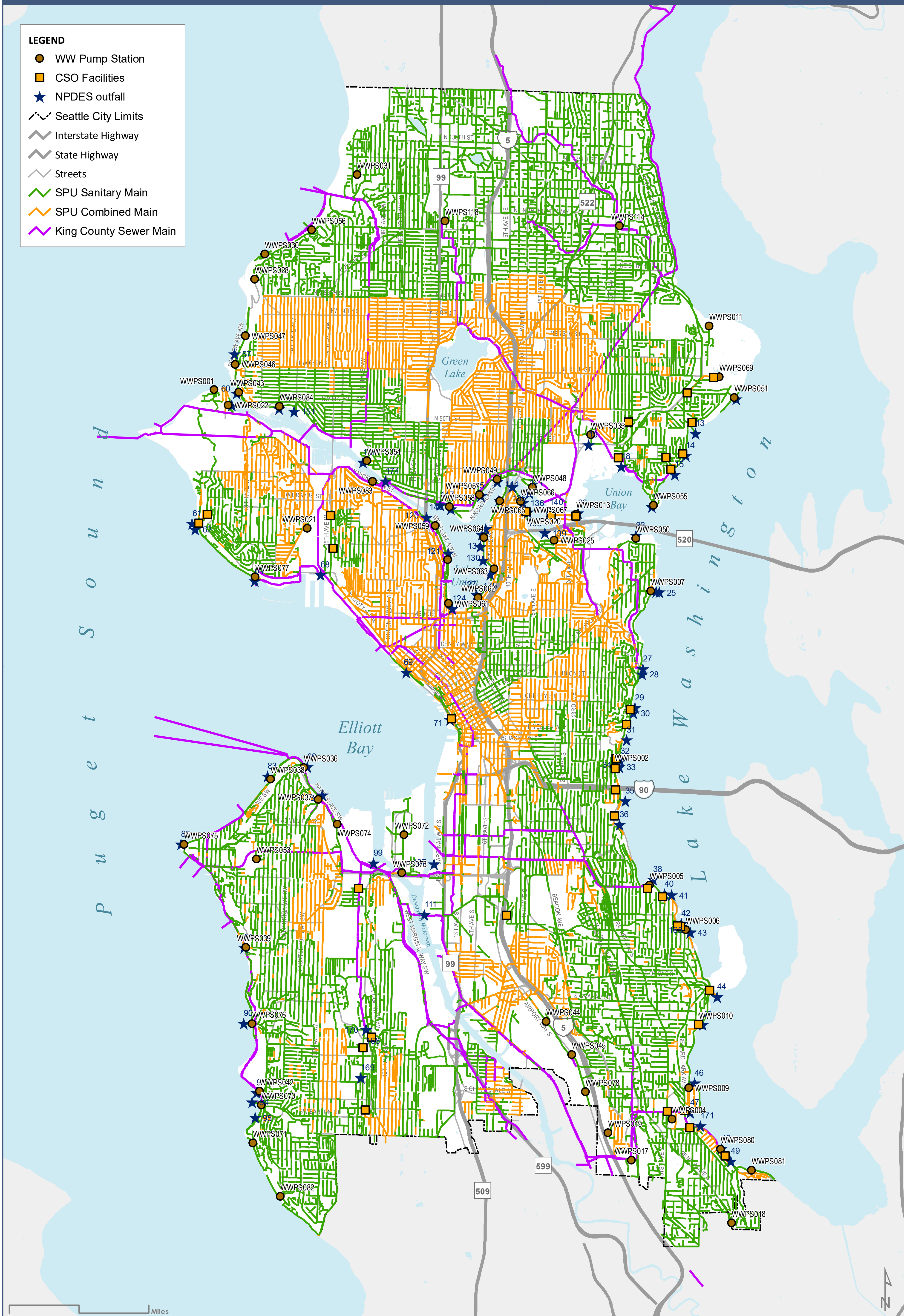
Map A-1: Overview of SPU's Wastewater System



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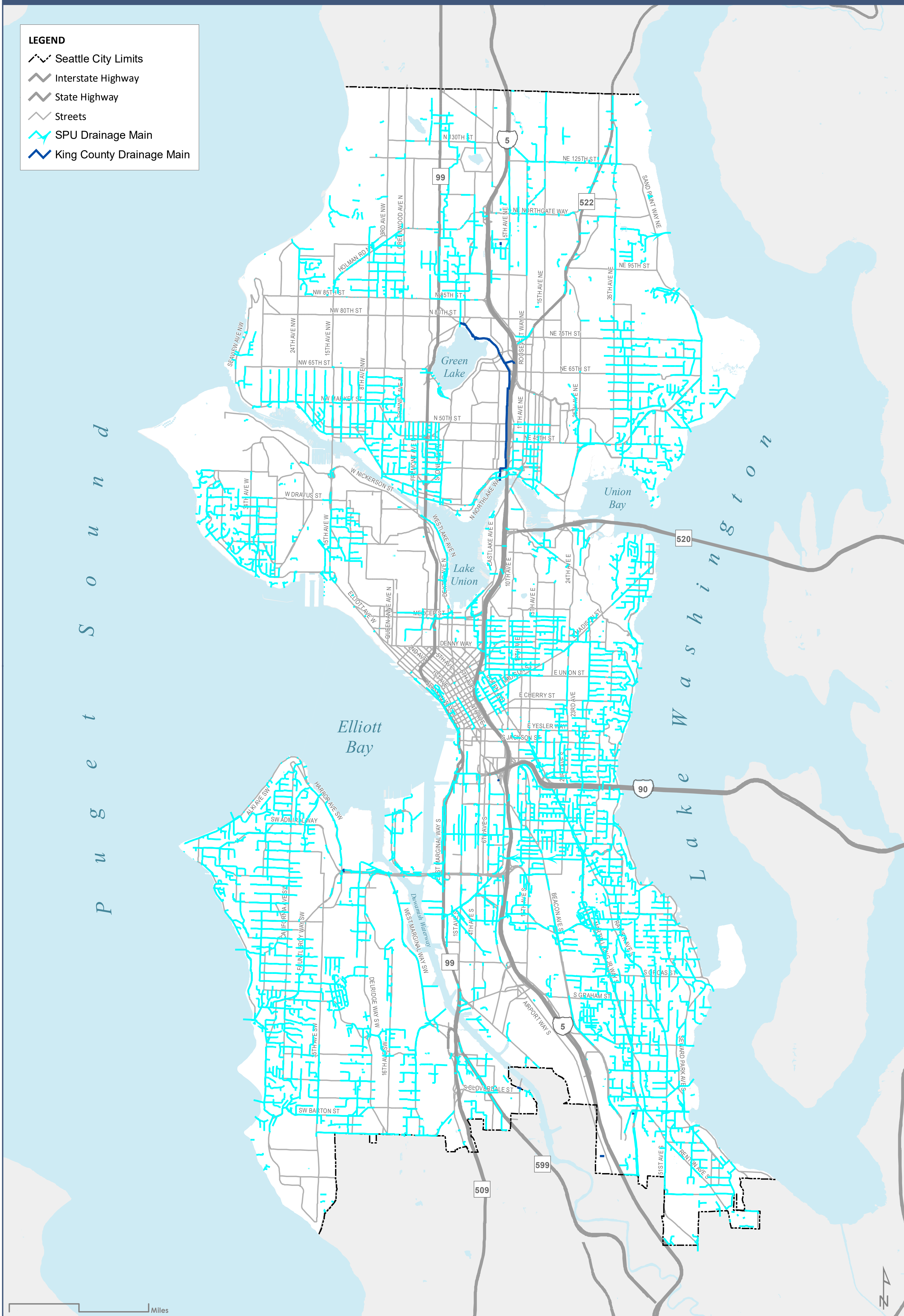
Shape Our Water | Seismic Risk Assessment

Map A-2: Overview of SPU's Drainage Mainlines



Appendix B: Wastewater System Backbone

This appendix was developed by SPU (Eric Habermeyer) to document the development of GIS data for the Wastewater System Backbone. Backbone features are described in the Section 2.2 of the report. The names of GIS feature classes are shown in *italics*. Map B-1 at the end of this appendix shows the backbone facilities.

System Backbone Features

WW System Backbone & WW Detention Mainline Backbone

GIS feature class modified for System Backbone Map. Source is *DWW Mainline (Permitted Use)* feature class and *DWW Mainline Detention Lines* feature class. Source feature classes were filtered for SPU combined and sanitary wastewater mainlines. It excludes drainage mainlines and mainlines owned by private parties or other agencies.

The mainline features included in the WW system backbone were identified as the mainlines connecting each critical customer to a point of connection with a King County Interceptor or other agency sewer main. It was developed by conducting the following GIS spatial analysis.

1. Select the parcel containing a critical customer. Each critical customer is identified by a point based on their address.
2. Select all the side sewers that connect to these parcels. The side sewers are now the start of the downstream traces (flags).
3. Identify the termination point for the traces where the SPU system goes into a KC interceptor. A point was created where an SPU mainline intersects a KC mainline (end barriers).
4. Trace is run using the existing mainline system geometric network. It selects all mainlines starting at the "flag" locations and connecting downstream until they hit a "barrier."
5. Results were reviewed manually. Any erroneous mainlines identified during the traces were removed by manually editing the feature class.

GIS Reference Features

CriticalFac_rev: GIS feature class from Wastewater System Analysis. Identifies the location of 741 critical facilities within SPU's service area. One exception is the Police Firing Range K-9 Building outside of the service boundary. Categories of critical facilities include emergency services, high population, human services, medical, protective, support, transportation, and vulnerable population. Primary use of critical facilities is identified by the attribute "PrimaryUse".

WW Pump Station: GIS feature class identifying the location of 67 SPU wastewater pump stations (WWPS). Feature class does not identify the location of sub-components (e.g., structures, I&C, etc.).

CSO Facility: GIS feature class identifies the location of 42 CSO facility sites in SPU's service. area. Feature class does not identify the location of sub-components (e.g., structures, I&C, etc.).

DWW Mainline End Points (not displayed due to scale): SPU GIS feature class of maintenance holes, catch basins, plugs, tees, outfalls, reducers, pump stations, water quality structures, et al. Feature class is included as a reference but not shown in the figure due to the scale.

WW Mainline (Permitted Use): Source is *DWW Mainline (Permitted Use)* feature class and filtered for combined and sanitary wastewater mainlines. Source identifies King County sewer main and sewer main owned by private parties or other agencies to show interconnections with other sewer systems. It excludes drainage mainlines.

DWW Mainline Detention Lines: GIS feature class of mainline detention systems. These are circular or rectangular mainlines, majority of which comprise CSO facility storage assets. Feature class attributes include dimensions (width, height, length), shape (circular, rectangular), upstream and downstream invert elevation and depth (rim to invert elevation), material, etc. It excludes drainage detention mainlines.

Other GIS Feature Classes:

- *Arterial Names*
- *Arterial Streets*
- *King County Streets* (Freeways)
- *Freeway/Highway Symbols*
- *City Limits_Purple*
- *Wateranno*
- *WaterBody_Poly*

System Backbone GIS Attribute Description

The following is a description of attribute data for select GIS system backbone feature classes. A subset of attributes is described as relevant to seismic assessment.

WW System Backbone Attribute Data Fields

- MNL_USE_1−Mainline permitted use (sanitary or combined)
- MNL_PIPE_1−Mainline shape (i.e., circular, etc.)
- MNL_MATE_1–Material description (i.e., asbestos cement, cast iron pipe, concrete)
- MNL_LENGTH–Length of Mainline (feet)
- MNL_WIDTH–Mainline width
- MNL_HEIGHT–Mainline Height
- MNL_SLOPE–Mainline slope
- MNL_UPS_DE–Mainline upstream depth (measured as Rim of maintenance hole to Invert of mainline)
- MNL_DNS_DE–Mainline downstream depth (measured as Rim of maintenance hole to Invert of mainline)
- MNL_UPS_EL–Mainline upstream elevation (NAVD88)
- MNL_DNS_EL–Mainline downstream elevation (NAVD88)
- MNL_INSTAL–Mainline installation date
- MNL_PUMP_F–Mainline is flagged as a pump station force main

WW Pump Station Attribute Data Fields

- MNLEP_DE_2–Depth of structure (rim to floor)
- MNLEP_CV_1–Rim elevation (Also referred to as curve elevation–NAVD88)

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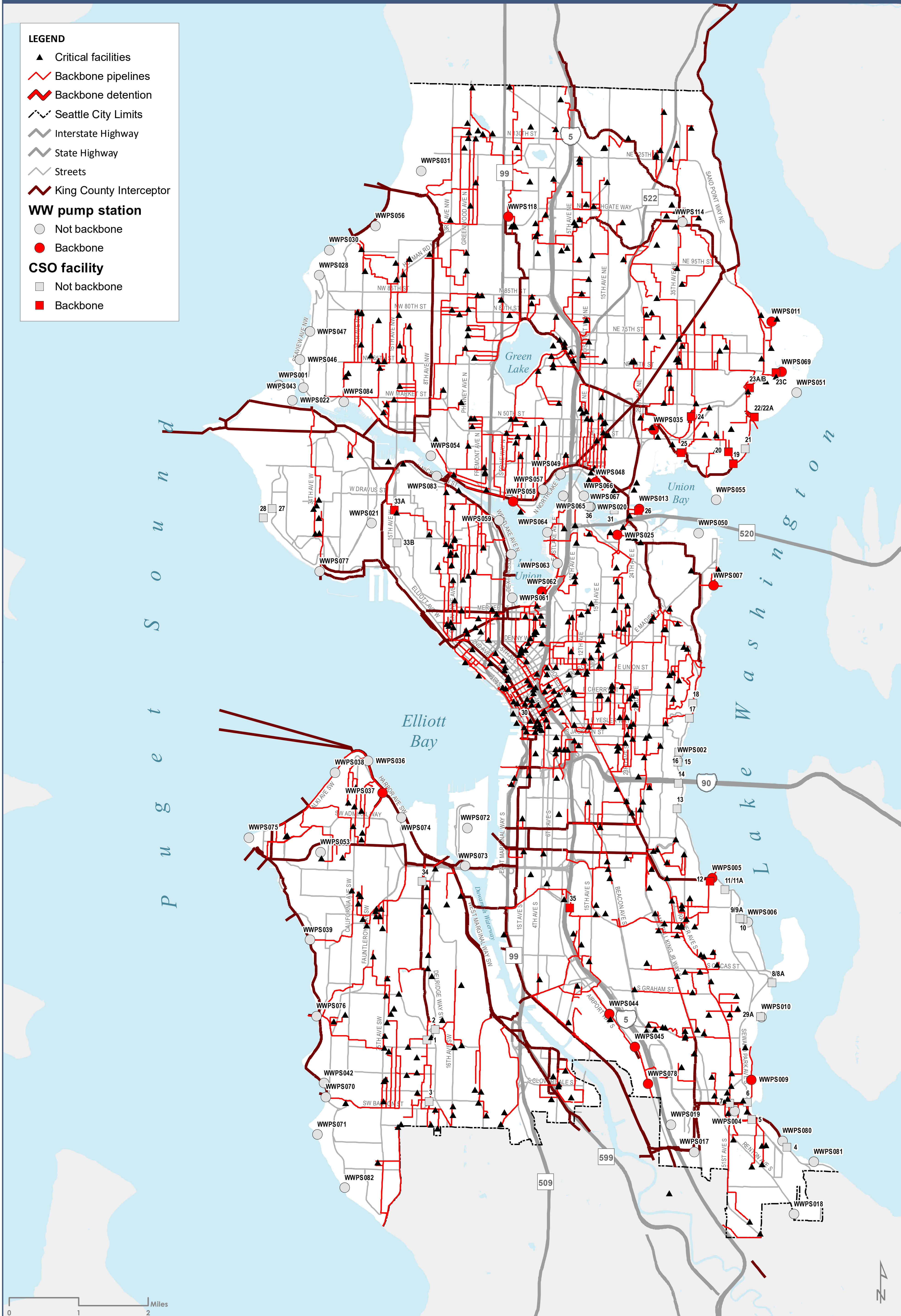
Map B-1: Wastewater System Backbone



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Appendix C: Inventory of Geospatial Hazard Data

Water System Seismic Study Geotechnical Seismic Hazard and
Tsunami/Seiche Hazard GIS Data

This appendix provides a summary of the geotechnical seismic hazard data and the associated calculation method to develop these data that were used in the SPU Water System Seismic Study (SPU 2018b). These data and/or calculations methods have also been used as the basis for the geotechnical seismic hazard data used in the SPU Shape Our Water drainage and wastewater system seismic risk assessment.

Ground-Shaking Intensity

As part of the SPU Water System Seismic Study (WSSS), ground motion prediction equations (GMPEs) were used to estimate ground-shaking intensity measures within the SPU service area and other areas where SPU water system pipelines and facilities are located (SPU, 2018b). The ground-shaking intensity measures calculated were those that are commonly used to evaluate the performance of buried pipelines and above grade facilities [peak ground acceleration (PGA), peak ground velocity (PGV), spectral response acceleration at short periods (S_s), and spectral response acceleration at a period of 1-second (S_1)]. The ground-shaking intensity values within the service area were estimated using the SERA computer program and site-specific values of the average seismic shear-wave velocity from the surface to a depth of 30 meters (V_{S30}) for the location of each grid point (spaced at 2,000 ft in both orthogonal directions). The GMPEs used for each scenario event are briefly described below. This SPU DWW seismic risk assessment used the same ground-shaking intensity measures as were used for the SPU WSSS.

M7.0 SFZ Scenario

The ground-shaking intensity measures for the M7.0 SFZ scenario earthquake were calculated using the average of the five Next Generation Attenuation Models for the Western United States (NGA-West2) GMPEs (SPU, 2018b). The following GIS files have been provided by SPU associated with ground-shaking intensity for the M7.0 SFZ scenario earthquake:

- **M7.0 SFZ Peak Ground Acceleration** (File Name: `M7_SFZ_PGA_KRIGGED.shp`) **-** GIS shape file containing geospatial distribution of PGA associated with a M7.0 SFZ scenario earthquake. *[Note: This file was created for the water system project by SPU using existing M7.0 SFZ scenario gridded data and a kriging procedure to estimate values for locations based on PGA data at adjacent grid points.]*
- **M7.0 SFZ Peak Ground Velocity** (File Name: `MapSeries_SFZ7.mpk`) **-** GIS map package file containing geospatial distribution of PGV associated with a M7.0 SFZ scenario earthquake. *[Note: This file was created for this project by SPU using existing M7.0 SFZ scenario gridded data and a kriging procedure similar to the one previously used for PGA.]*
- **M7.0 SFZ Spectral Response Acceleration at Short Periods** (File Name: `MapSeries_SFZ7.mpk`) **-** GIS map package file containing geospatial distribution of spectral response acceleration at short periods associated with a M7.0 SFZ scenario earthquake. *[Note: This file was created for this project by SPU using existing M7.0 SFZ scenario gridded data and a kriging procedure similar to the one previously used for PGA.]*
- **M7.0 SFZ Spectral Response Acceleration at a Period of 1 Second** (File Name: `MapSeries_SFZ7.mpk`) **-** GIS map package file containing geospatial distribution of spectral response acceleration at a period of 1 second associated with a M7.0 SFZ scenario earthquake. *[Note: This file was created for this project by SPU using existing M7.0 SFZ scenario gridded data and a kriging procedure similar to the one previously used for PGA.]*

M9.0 CSZ Scenario

The ground-shaking intensity measures for the M9.0 CSZ scenario earthquake were calculated using the 2012 BC Hydro GMPEs (SPU, 2018b). The following GIS files have been provided by SPU associated with ground-shaking intensity for the M9.0 CSZ scenario earthquake:

- **M9.0 CSZ Peak Ground Acceleration** (File Name: `M9_CSZ_PGA_KRIGGED.shp`) **-** GIS shape file containing geospatial distribution of PGA associated with a M9.0 CSZ scenario earthquake. *[Note: This file was created for the water system project by SPU using existing M9.0 CSZ scenario gridded data and a kriging procedure to estimate values for locations based on PGA data at adjacent grid points.]*
- **M9.0 CSZ Peak Ground Velocity** (File Name: `MapSeries_CSZ9.mpk`) **-** GIS map package file containing geospatial distribution of PGV associated with a M9.0 CSZ scenario earthquake. *[Note: This file was created for this project by SPU using existing M9.0 CSZ scenario gridded data and a kriging procedure similar to the one previously used for PGA.]*
- **M9.0 CSZ Spectral Response Acceleration at Short Periods** (File Name: `MapSeries_CSZ9.mpk`) **-** GIS map package file containing geospatial distribution of spectral response acceleration at short periods associated with a M9.0 CSZ scenario earthquake. *[Note: This file was created for this project by SPU using existing M9.0 CSZ scenario gridded data and a kriging procedure similar to the one previously used for PGA.]*
- **M9.0 CSZ Spectral Response Acceleration at a Period of 1 Second** (File Name: `MapSeries_CSZ9.mpk`) **-** GIS map package file containing geospatial distribution of spectral response acceleration at a period of 1 second associated with a M9.0 CSZ scenario earthquake. *[Note: This file was created for this project by SPU using existing M9.0 CSZ scenario gridded data and a kriging procedure similar to the one previously used for PGA.]*

2018 USGS Probabilistic Ground Motions

The 2018 United States Geological Survey (USGS) Probabilistic Ground Motions were used as the ground-shaking intensity measures (2 percent probability of exceedance in 50 years) to approximate the seismic hazard that will be specified in ASCE 7-22 and a future edition of the Seattle Building Code. In the SPU WSSS, these ground motions were only used to evaluate building-like structures, so PGV was not required.

The following GIS files have been provided by SPU associated with ground-shaking intensity for the 2018 USGS Probabilistic Ground Motions:

- **2018 USGS Probabilistic Peak Ground Acceleration** (File Name: *2018_USGS_2475_PGA.shp*) **-** GIS shape file containing geospatial distribution of PGA associated with the 2018 USGS probabilistic seismic hazard.
- **2018 USGS Probabilistic Spectral Response Acceleration at Short Periods** (File Name: *us5hz250pts.shp*) **-** GIS shape file containing geospatial distribution of spectral response acceleration at short periods associated with the 2018 USGS Probabilistic Ground Motions.
- **2018 USGS Probabilistic Spectral Response Acceleration at a Period of 1 Second** (File Name: *us1hz250pts.shp*) **-** GIS shape file containing geospatial distribution of spectral response acceleration at a period of 1 second associated with the 2018 USGS Probabilistic Ground Motions.

Liquefaction-Induced Permanent Ground Deformation

The calculation procedure implemented to estimate liquefaction-induced permanent ground deformation (PGD) for each earthquake scenario in the SPU WSSS is described below. This calculation procedure was provided by SPU (Bill Heubach, SPU WSSS Project Manager) and was developed for the WSSS in consultation with New Albion Geotechnical (NAG 2017). For each buried mainline segment, a GIS-based approach was used to calculate the liquefaction-induced PGD based on the worst-case liquefaction susceptibility and largest value of PGA that the mainline segment crosses. The associated probability of liquefaction was estimated based on this worst-case liquefaction susceptibility and largest value of PGA (using the table provided in Step 6, below). This SPU DWW seismic risk assessment used the same calculation procedure for estimating the magnitude of liquefaction-induced PGD for backbone mainline segments and facilities as was used for the SPU WSSS.

For each location (mainline segment), the liquefaction-induced PGD was calculated as follows:

1. Determine liquefaction susceptibility from Washington Department of Natural Resources (DNR) liquefaction susceptibility map. Note that the area classified by DNR as "High" from Interbay southward through downtown Seattle, South of Downtown (SODO) and Harbor Island should be classified as "Very High."
2. For each scenario (M7.0 SFZ and M9.0 CSZ), determine the PGA from the "krigged" GIS shape files.
3. Calculate the vertical PGD (*Index* PGD_v) and horizontal PGD (*Index* PGD_h) as follows (note units are in feet):

$$Index\ PGD_v = \frac{PGD_{v-max}}{1 + \left[\frac{(PGA/MSF)}{b}\right]^c}$$

$$Index\ PGD_h = \frac{PGD_{h-max}}{1 + \left[\frac{(PGA/MSF)}{b}\right]^c}$$

where, MSF = magnitude scaling factor (accounts for size of earthquake)

= 1.19 for M7.0 SFZ scenario earthquake

= 0.627 for M9.0 CSZ scenario earthquake

and *b*, *c*, *Index* $PGD_{v\text{-}max}$, and $PGD_{h\text{-}max}$ are determined from Tables C-1 and C-2.

Table C-1. Values for Index PGDv						
Parameter	Liquefaction Susceptibility					
	Very High	High	Moderate to High	Moderate	Low to Moderate	Low
$PGD_{v\text{-}max}$	1.20	0.85	0.60	0.28	0.13	0.04
b	0.25	0.25	0.25	0.32	0.32	0.32
c	-2.4	-2.6	-2.8	-2.4	-2.4	-2.4

Table C-2. Values for Index PGDh

Parameter	Liquefaction Susceptibility					
	Very High	High	Moderate to High	Moderate	Low to Moderate	Low
$PGD_{h\text{-}max}$	11.00	8.00	5.50	2.00	1.25	0.50
b	0.230	0.275	0.300	0.320	0.340	0.360
c	-3.10	-3.30	-3.50	-3.70	-3.75	-4.00

4. The horizontal PGD is affected by the distance (L) to a free face (e.g., riverbank or coastline) and the height (H) of the free face. Determine the closest distance to either the Puget Sound, Lake Washington, Duwamish River, Lake Union, Ship Canal, or Green Lake. If the distance is greater than 500 feet, then set the horizontal PGD [$PGD_h(L)$] equal to zero and also set the vertical PGD caused by soils that move horizontally [$PGD_V(L)$] equal to zero. Otherwise:
   - Estimate the height of the free face (H) to be equal to the ground elevation minus the mean water elevation of the nearest body of water plus 10 feet to account for the height of the free face height below the water line.
   - If $L/H \leq 1.5$, then

     $$PGD_h(L) = Index\ PGD_h$$

     else if $L/H > 1.5$, then

     $$PGD_h(L) = Index\ PGD_h * exp\{-1.5 * [log_{10} (L/H) - 0.176]\}$$

   - If $L/H < 1$, then

     $$PGD_v(L) = PGD_h\text{-}max * 0.6$$

     else if $L/H \geq 1$, then

     $$PGD_v(L) = PGD_h\text{-}max * 0.6 * exp\{-0.92 * [(L/H) - 1]\}$$

5. The total PGD (PGD_{total}) is:

$$PGD_{total} = \sqrt{[Index\ PGD_v + PGD_v(L)]^2 + [PGD_h(L)]^2}$$

6. The probability of liquefaction is calculated from the following table (using linear interpolation):

Table C-3. Liquefaction Probability

PGA/MSF	Liquefaction Susceptibility					
	Very High	High	Moderate to High	Moderate	Low to Moderate	Low
0.0	0.00	0.00	0.00	0.00	0.00	0.00
0.1	0.02	0.00	0.00	0.00	0.00	0.00
0.2	0.36	0.26	0.02	0.00	0.00	0.00
0.3	0.76	0.60	0.10	0.04	0.00	0.00
0.4	0.89	0.72	0.30	0.08	0.002	0.00
0.5	0.93	0.76	0.45	0.10	0.003	0.00
0.6	0.95	0.80	0.50	0.14	0.006	0.00
0.7	0.95	0.80	0.50	0.15	0.01	0.00
0.8	0.95	0.80	0.50	0.15	0.01	0.00
0.9	0.95	0.80	0.50	0.15	0.01	0.00
1.0	0.95	0.80	0.50	0.15	0.01	0.00

The following GIS files have been provided by SPU associated with susceptibility to liquefaction-induced PGD:

- **Liquefaction Susceptibility** (File Name: *liquefaction.shp*) **-** GIS shape file containing geospatial distribution of liquefaction susceptibility.

M7.0 SFZ Scenario

The following GIS files have been provided by SPU associated with liquefaction-induced PGD for the M7.0 SFZ scenario earthquake:

- **M7.0 SFZ Liquefaction PGD** (File Name: DWW_Mainlines_SF7_FOS.shp) **-** GIS shape file containing geospatial distribution of liquefaction-induced PGD for wastewater backbone system mainlines and facilities associated with a M7.0 SFZ scenario earthquake. *[Note: This file was created for this project by SPU (Nathan H.) using the procedure described above.]*

M9.0 CSZ Scenario

The following GIS files have been provided by SPU associated with liquefaction-induced PGD for the M9.0 CSZ scenario earthquake:

- **M9.0 CSZ Liquefaction PGD** (File Name: DWW_Mainlines_CSZ9_FOS.shp) **-** GIS shape file containing geospatial distribution of liquefaction-induced PGD for wastewater backbone system mainlines and facilities associated with a M9.0 CSZ scenario earthquake. *[Note: This file was created for this project by SPU (Nathan H.) using the procedure described above.]*

Landslide-Induced Permanent Ground Deformation

The calculation procedure implemented to estimate landslide-induced PGD in the SPU WSSS is described below. This calculation procedure was provided by SPU (Bill Heubach, SPU WSSS Project Manager). For each buried mainline segment, a GIS-based approach was used to calculate the landslide-induced PGD based on the lowest factor of safety of all of the landslide zones and largest value of PGA that the mainline segment crosses. This SPU DWW seismic risk assessment used the same calculation procedure for estimating the magnitude of landslide-induced PGD for backbone mainline segments and facilities as was used for the SPU WSSS.

For each location (mainline segment), the landslide-induced PGD was calculated as follows:

1. **Landslide Factor of Safety.** The landslide factor of safety is taken from the Harp, et. al. map as follows:

Table C-4. Landslide Factor of Safety

Factor of Safety	Harp et al. (2006) Map Grid Category
0.00 to 0.75	1
0.75 to 1.00	2
1.00 to 1.25	3
1.25 to 1.50	4
1.50 to 1.75	5
1.75 to 2.00	6
2.00 to 2.50	7
2.50 to 3.00	8
3 to 4	9
4 to 6	10
Greater than 6	11

If the mainline is in grid categories 7 through 11, a landslide is not considered possible unless the mainline lies in an area defined by the City of Seattle, King County, or Washington State Department of Natural Resources (DNR) maps as susceptible to landslide, in which case the factor of safety is assumed to be between 1.75 and 2.0.

A factor of safety less than 1.0 implies that the slope will slide if the slope is saturated, even under static conditions. For these areas (grid categories 1 and 2), it was assumed that the probability of sliding during an earthquake is equal to 1.0 and that the landslide-induced PGD is equal to 200 inches.

2. **Landslide Probability.** The simplified Newmark sliding block model is considered:

$k_y = (FS - 1)\, g \sin \alpha$

where:

k_y = the ground acceleration that triggers landsliding

FS = the factor of safety

g = the acceleration due to gravity

α = the slope angle

The probability density function of the factor of safety is assumed to be uniformly distributed between the lower bound and upper bound for each range. Similarly, the slope is assumed to be uniformly distributed between 30 degrees and 60 degrees. Monte Carlo simulations were run to determine the landslide probability as a function of PGA for each factor of safety as shown in the table below (for example, there is a 0.1 probability of landslide at 0.0179g for the 1.0 to 1.25 factor of safety range):

Table C-5. Landslide Probability as a Function of Peak Ground Acceleration and Factor of Safety

Landslide Probability	Factor of Safety				
	1.0 to 1.25	1.25 to 1.5	1.5 to 1.75	1.75 to 2.0	2.0 to 2.5
0	0.000	0.000	0.000	0.000	0.000
0.1	0.0179	0.181	0.326	0.468	0.654
0.2	0.0347	0.206	0.360	0.510	0.721
0.3	0.0522	0.223	0.386	0.547	0.771
0.4	0.0691	0.240	0.410	0.582	0.821
0.5	0.0858	0.257	0.434	0.613	0.865
0.6	0.102	0.275	0.458	0.643	0.912
0.7	0.119	0.297	0.484	0.674	0.965
0.8	0.137	0.323	0.515	0.710	1.03
0.9	0.161	0.355	0.554	0.757	1.11
1.0	0.215	0.432	0.648	0.864	1.29

3. **Landslide-Induced PGD.** The Makdisi and Seed (1978) methodology was used to estimate the landslide-induced PGD. Given that a landslide would occur, the PGD would not necessarily be a function of the factor of safety. However, it was assumed that lower PGD would generally occur for slopes with a higher factor of safety. The assumed k_y/k_{max} values for each factor of safety range are indicated in the table below. These assumed k_y/k_{max} values are somewhat arbitrary but were chosen so that the peak displacements for the 1.0 to 1.25, 1.25 to 1.5 and 1.5 to 1.75 factor of safety ranges produced PGD rates of repairs that were towards the maximum value. The k_y/k_{max} value for the 1.75 to 2.0 factor of safety category was chosen to produce a large but not maximum rate of repair.

   Best fit equations below were fit to the curves shown (see marked up Makdisi and Seed plots for assumed M7.0 SFZ and M9.0 CSZ relationships) below to estimate PGD.

Table C-6. Values for k_y/k_{max}

Factor of Safety	k_y/k_{max}
1.0 to 1.25	0.177
1.25 to 1.50	0.354
1.50 to 1.75	0.530
1.75 to 2.0	0.707

M7.0 SFZ Scenario:

PGD = 398.1 * exp {-[6*(k_y / k_{max})] }–0.9868 * exp [- (1–k_y / k_{max})]
M9.0 CSZ Scenario:

PGD = 3162 * exp {-[7.75*(k_y / k_{max})] }–1.362 * exp [- (1–k_y / k_{max})]

Notes:

- k_y equals the minimum ground acceleration needed to trigger a landslide
- PGD values in above equations are expressed in centimeters

For each earthquake scenario, the landslide-induced PGD values are fixed for a given factor of safety category.

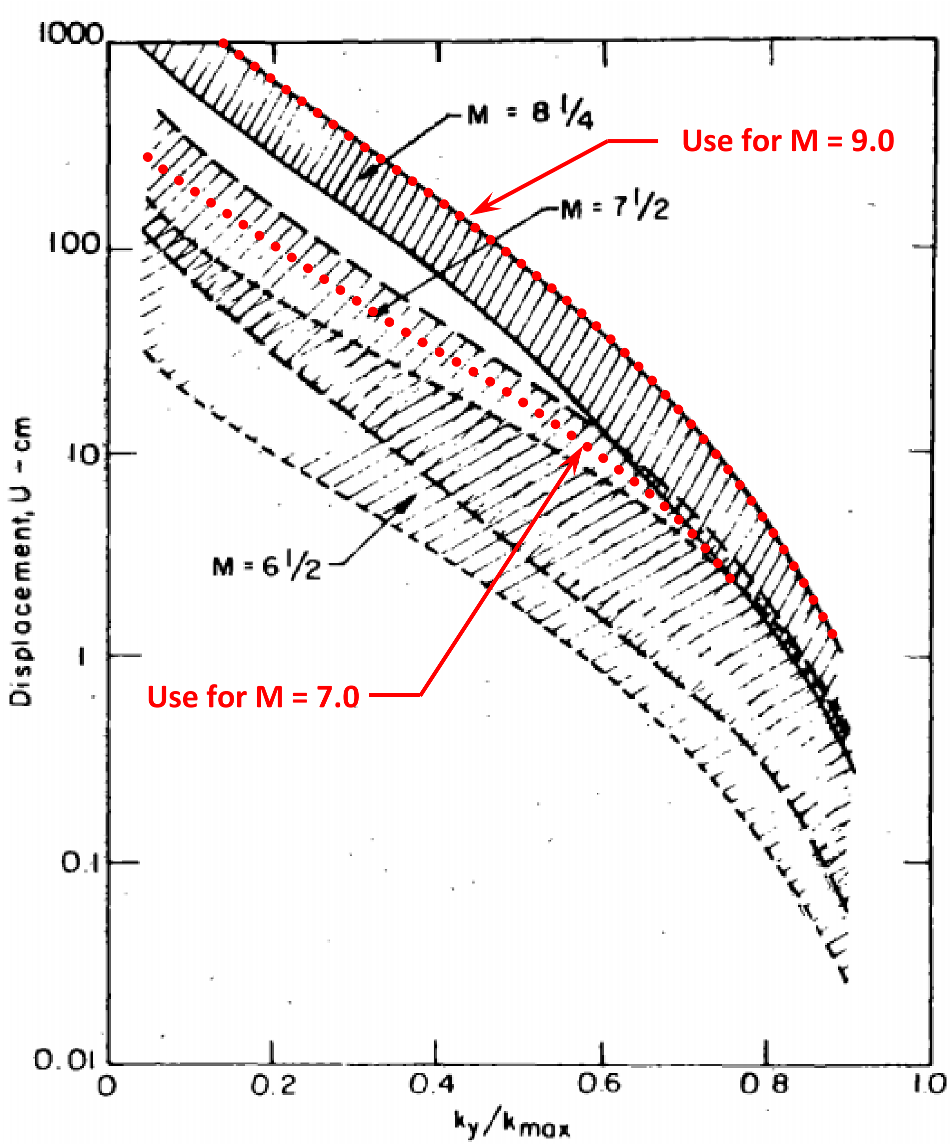

Figure C-1. Estimation of landslide-induced PGD

(from Makdisi and Seed, 1978)

The following GIS files have been provided by SPU associated with general susceptibility to landslide-induced PGD:

- **Landslide Susceptibility** (File Name: *harp.gdb*) **-** GIS geodatabase file containing geospatial distribution of areas of known or potential landslides [from USGS Open-File Report 2006-1139 (Harp, et. al. 2006)].
- **Landslide Probability** (File Name: DWW_Mainlines_CSZ9_FOS.shp and DWW_Mainlines_SF7_FOS.shp) **-** GIS shape file containing geospatial distribution of landslide probability. *[Note: This file was created for this project by SPU (Nathan H.) using the procedure described above.]*

M7.0 SFZ Scenario

The following GIS files have been provided by SPU associated with landslide-induced PGD for the M7.0 SFZ scenario earthquake:

- **M7.0 SFZ Landslide PGD** (File Name: DWW_Mainlines_CSZ9_FOS.shp) **-** GIS shape file containing geospatial distribution of landslide-induced PGD associated with a M7.0 SFZ scenario earthquake. *[Note: This file was created for this project by SPU (Nathan H.) using the procedure described above.]*

M9.0 CSZ Scenario

The following GIS files have been provided by SPU associated with landslide-induced PGD for the M9.0 CSZ scenario earthquake:

- **M9.0 CSZ Landslide PGD** (File Name: DWW_Mainlines_CSZ9_FOS.shp) **-** GIS shape file containing geospatial distribution of landslide-induced PGD associated with a M9.0 CSZ scenario earthquake. *[Note: This file was created for this project by SPU (Nathan H.) using the procedure described above.]*

Fault Rupture-Induced Permanent Ground Deformation

M7.0 SFZ Scenario

No GIS shape file is available for the geospatial distribution of fault rupture-induced PGD associated with a M7.0 SFZ scenario earthquake. In the SPU Water System Seismic Study, it was assumed that approximately 3-10 feet of discrete surface displacement could occur anywhere in either Zones A or B of the Seattle Fault Zone (SPU 2018b and LCI 2016). The number of repairs resulting from the fault rupture-induced PGD was relatively small, comparing to the overall number of repairs for the scenario. Therefore, the effects associated with the fault rupture-induced PGD were discussed but not reflected in the seismic risk assessment. A similar assumption will be made for this SPU DWW seismic risk assessment

The following GIS files have been provided by SPU associated with fault rupture-induced PGD deformation for the M7.0 SFZ scenario earthquake:

- **Seattle Fault Zone A** (File Name: *Seattle_fault_hazard_zones.shp*): GIS shape file indicating geospatial area of primary Seattle Fault Zone (Zone A).
- **Seattle Fault Zone B** (File Name: *Seattle_fault_hazard_zones.shp*): GIS shape file indicating geospatial area of back thrusting (Zone B).

M9.0 CSZ Scenario

Fault rupture-induced PGD resulting from a M9.0 CSZ scenario earthquake is not applicable to this SPU DWW seismic risk assessment.

Tsunami Hazard Data

- **M7.3 SFZ Tsunami Inundation Extents for Elliott Bay** (File Name: *Tsunami_HazardV10_3.mpk*): GIS map package file indicating geospatial area of tsunami inundation for Elliott Bay, associated with a M7.3 SFZ scenario earthquake (WGS 2019).
- **M7.3 SFZ Tsunami Inundation Extents for South King County Coast of Puget Sound** (File Name: *Seattle_L1_SFL_data.gdb*): GIS geodatabase file indicating geospatial area of tsunami inundation for the South King County coast of Puget Sound, associated with a M7.3 SFZ scenario earthquake (DNR In Preparation).

Appendix D: Seismic Hazard Maps

Seattle Public Utilities

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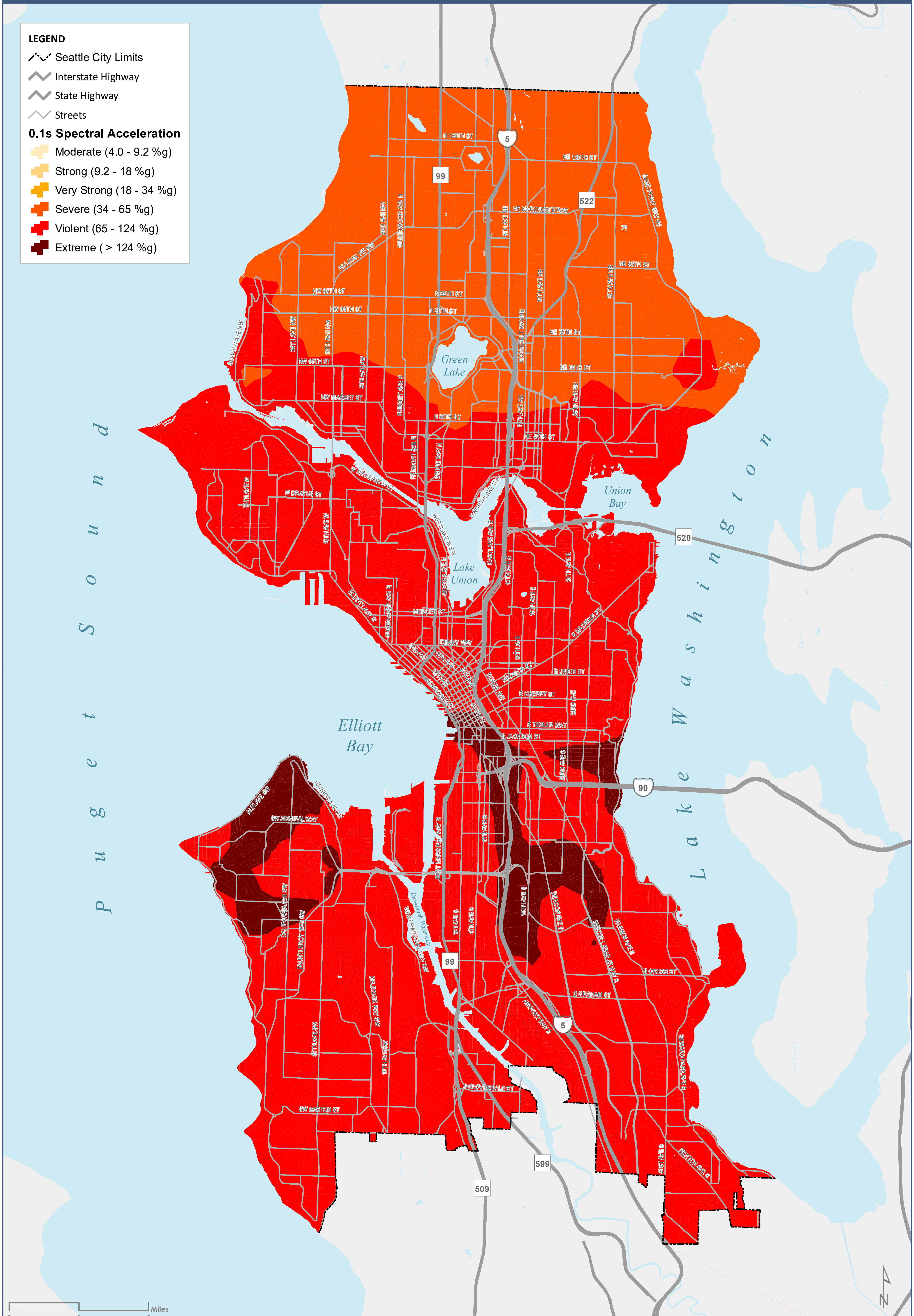
Map D-1: M7.0 SFZ 0.1s Spectral Acceleration



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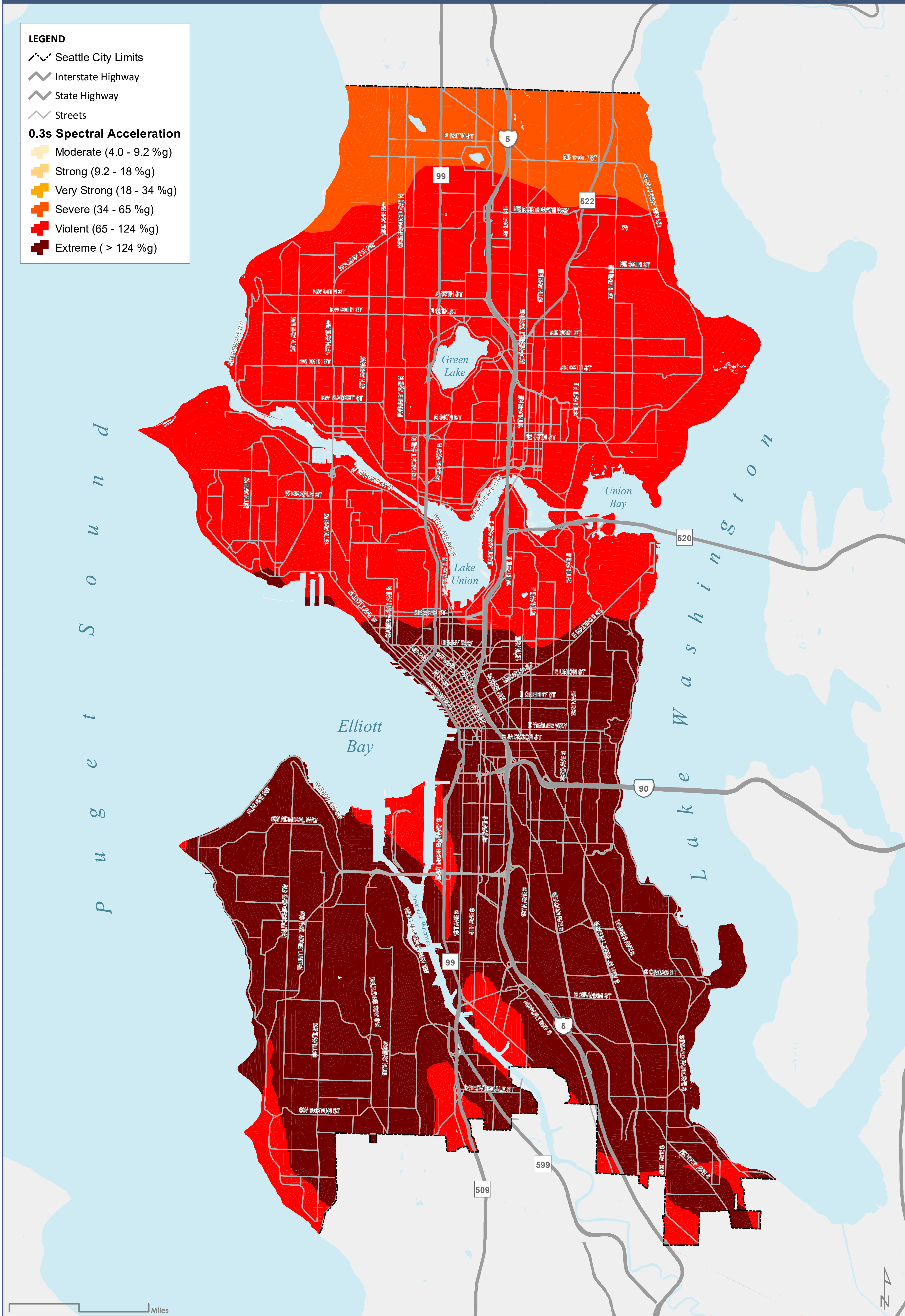
Map D-2: M7.0 SFZ 0.3s Spectral Acceleration



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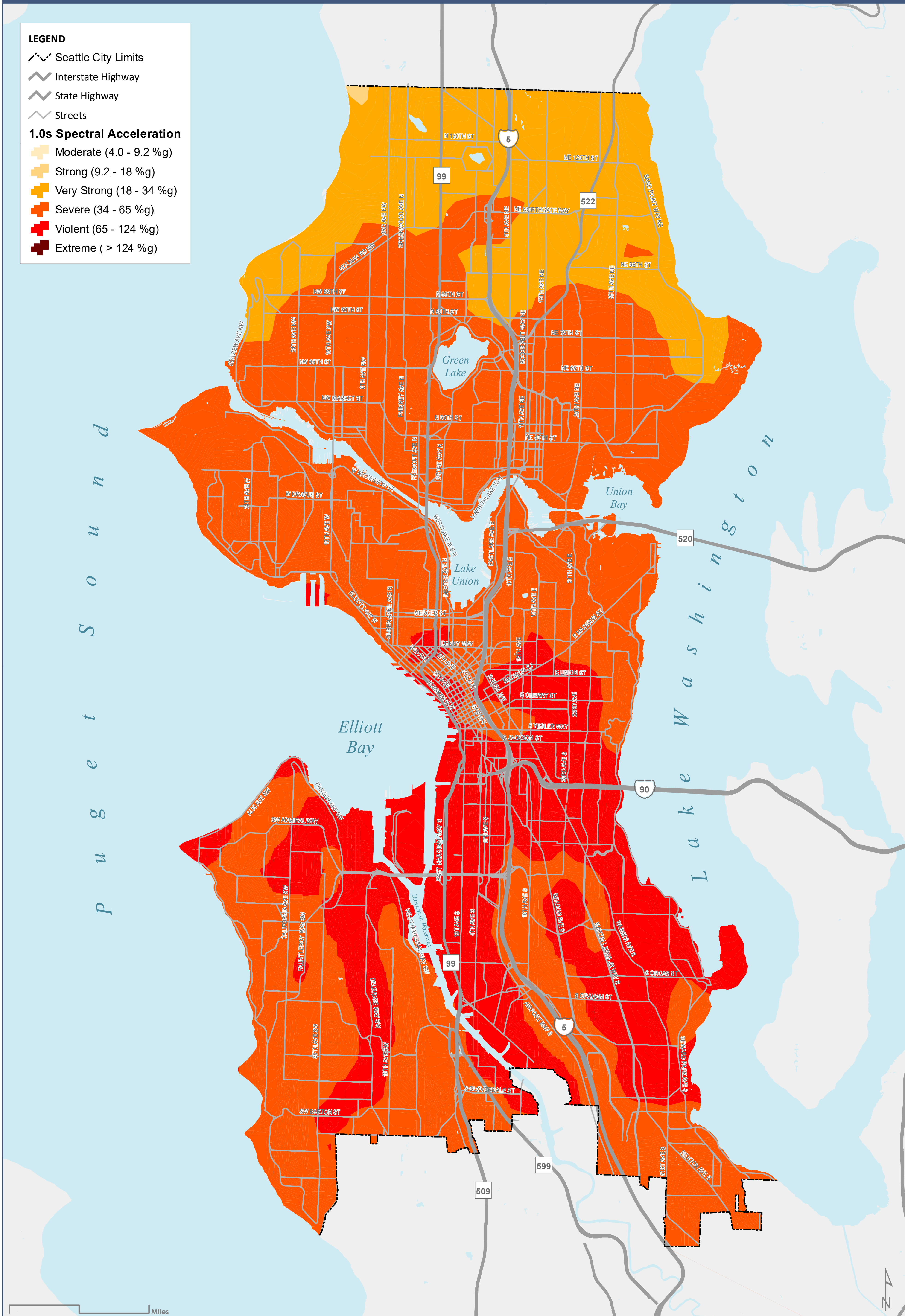
Map D-3: M7.0 SFZ 1.0s Spectral Acceleration



Author: Fill out Map Document Properties Date: 11/14/2021

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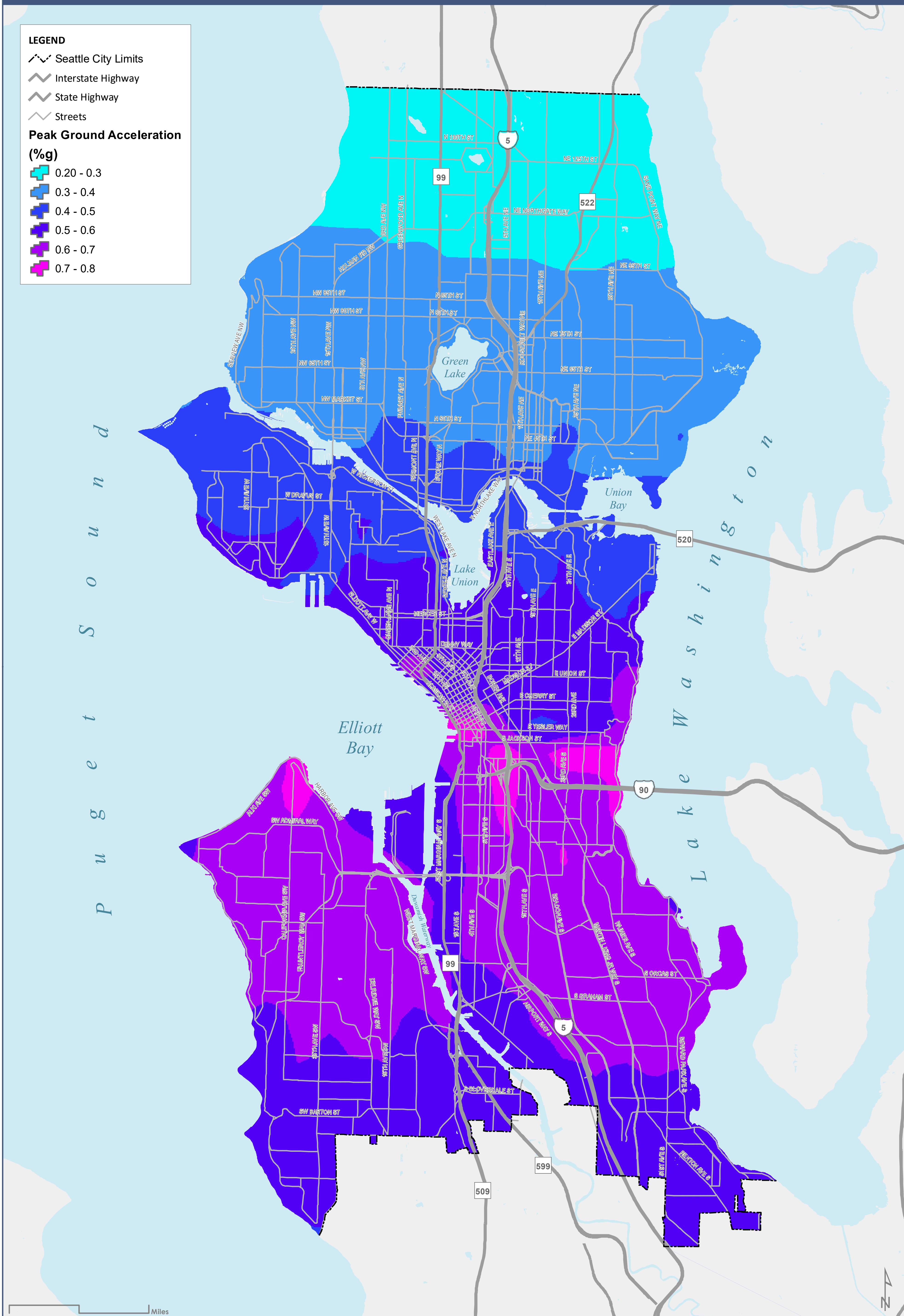
Map D-4: M7.0 SFZ Peak Ground Acceleration



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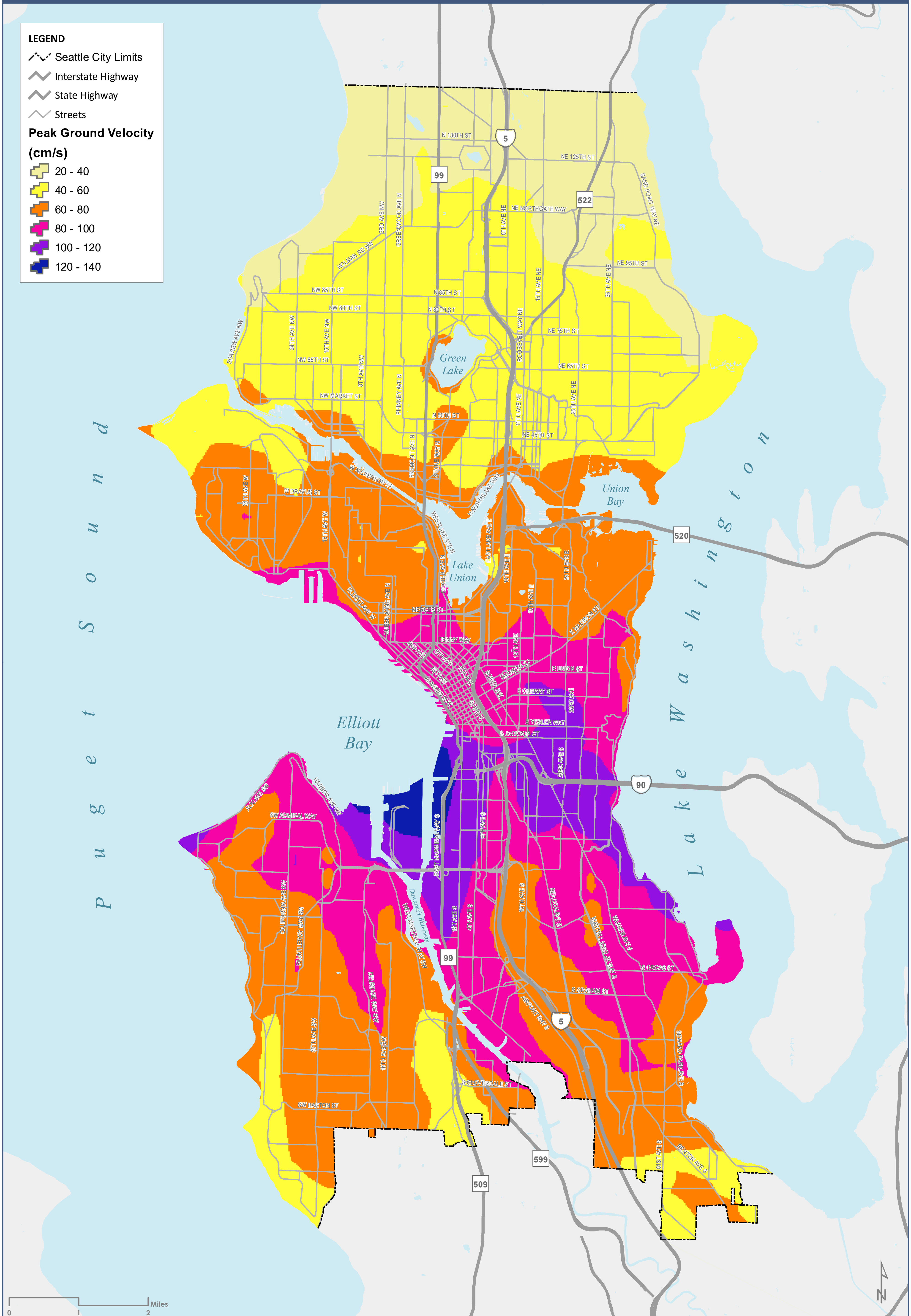
Map D-5: M7.0 SFZ Peak Ground Velocity



Author: Date: 11/29/2021 File Path: C:\Users\NFoged\AppData\Local\Temp\arc4C54\SOW_T4_MapD1.mxd

Seattle Public Utilities

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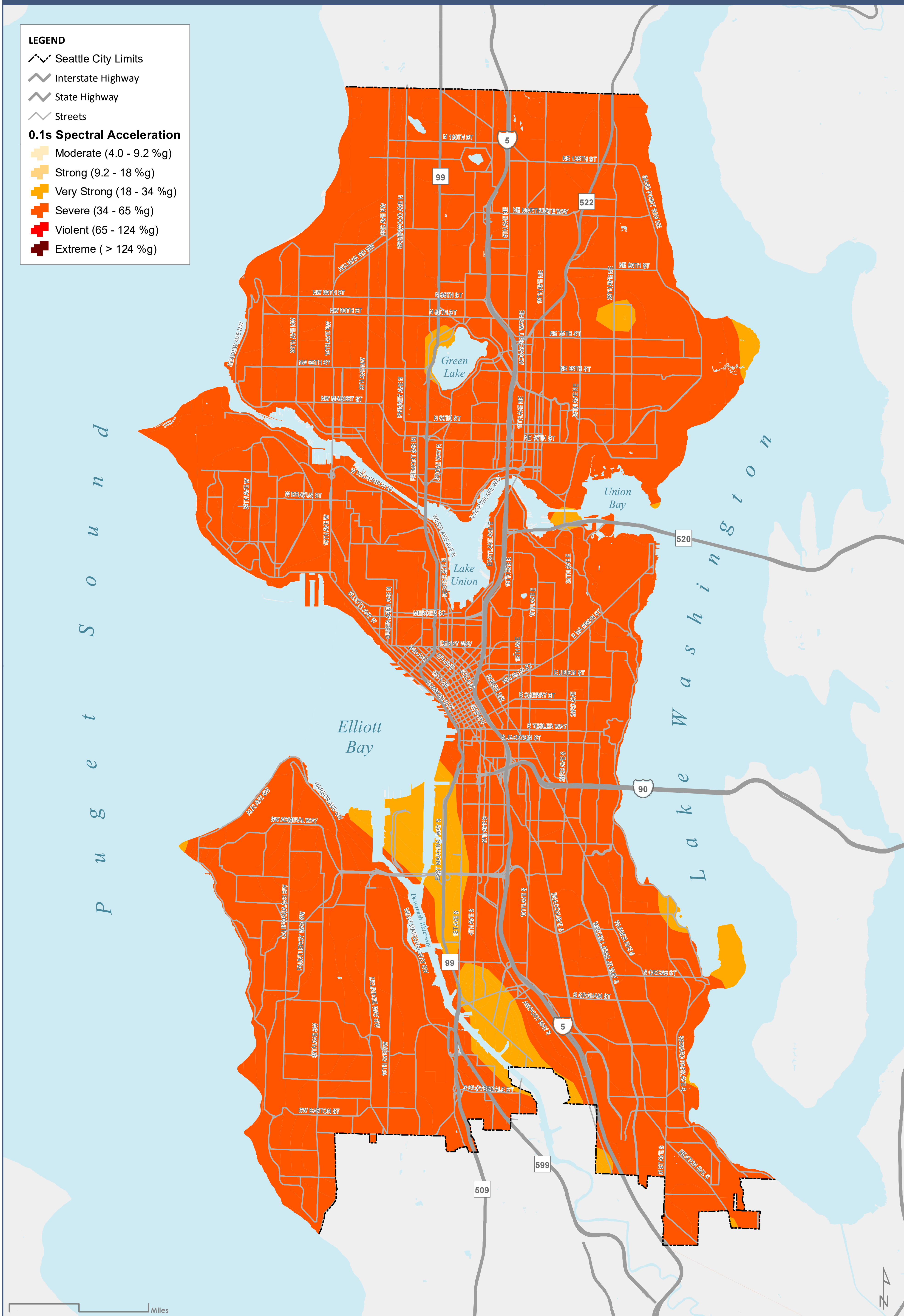
Map D-6: M9.0 CSZ 0.1s Spectral Acceleration



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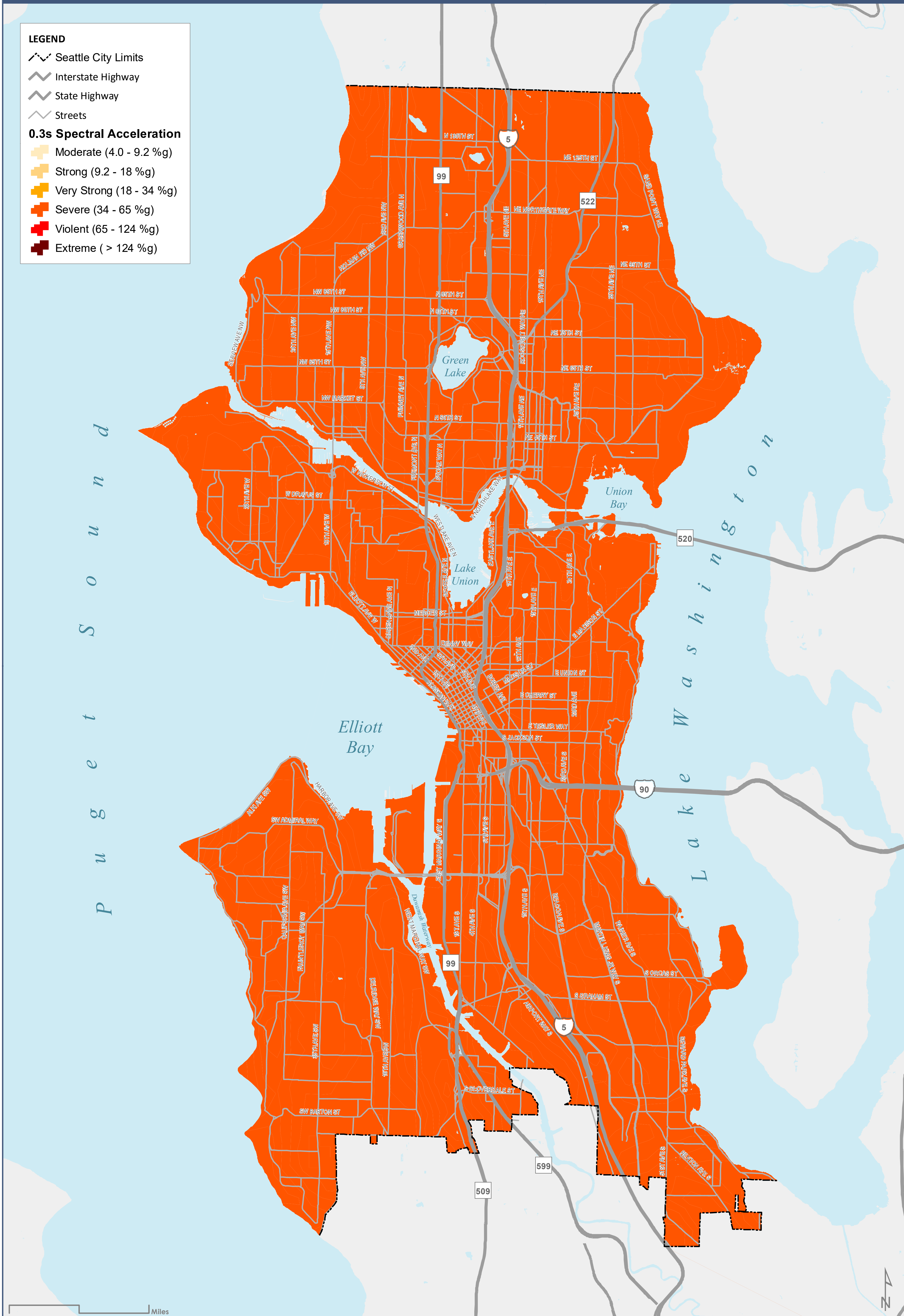
MapD-7: M9.0 CSZ 0.3s Spectral Acceleration



Author: Fill out Map Document Properties Date: 11/29/2021

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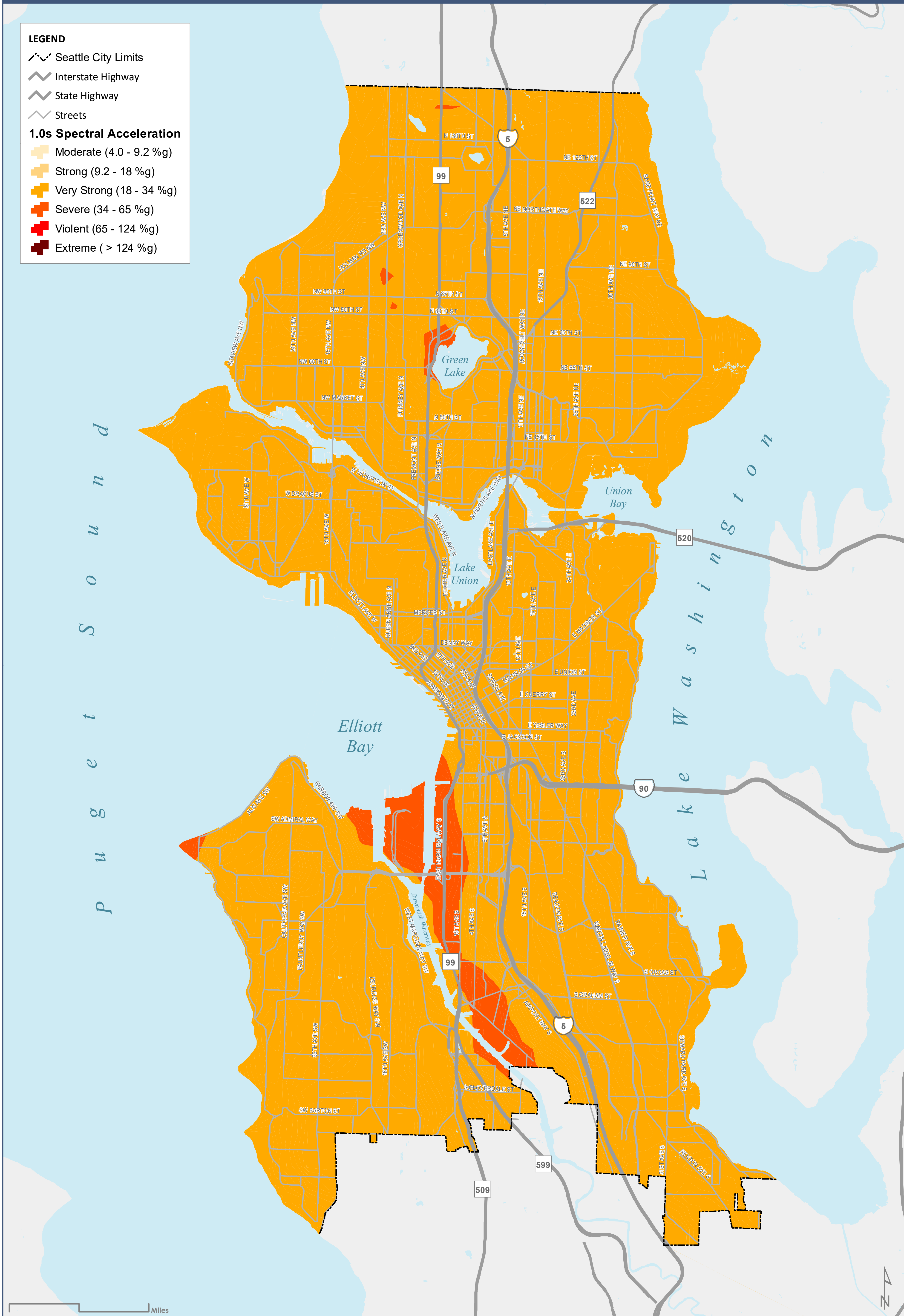
Map D-8: M9.0 CSZ 1.0s Spectral Acceleration



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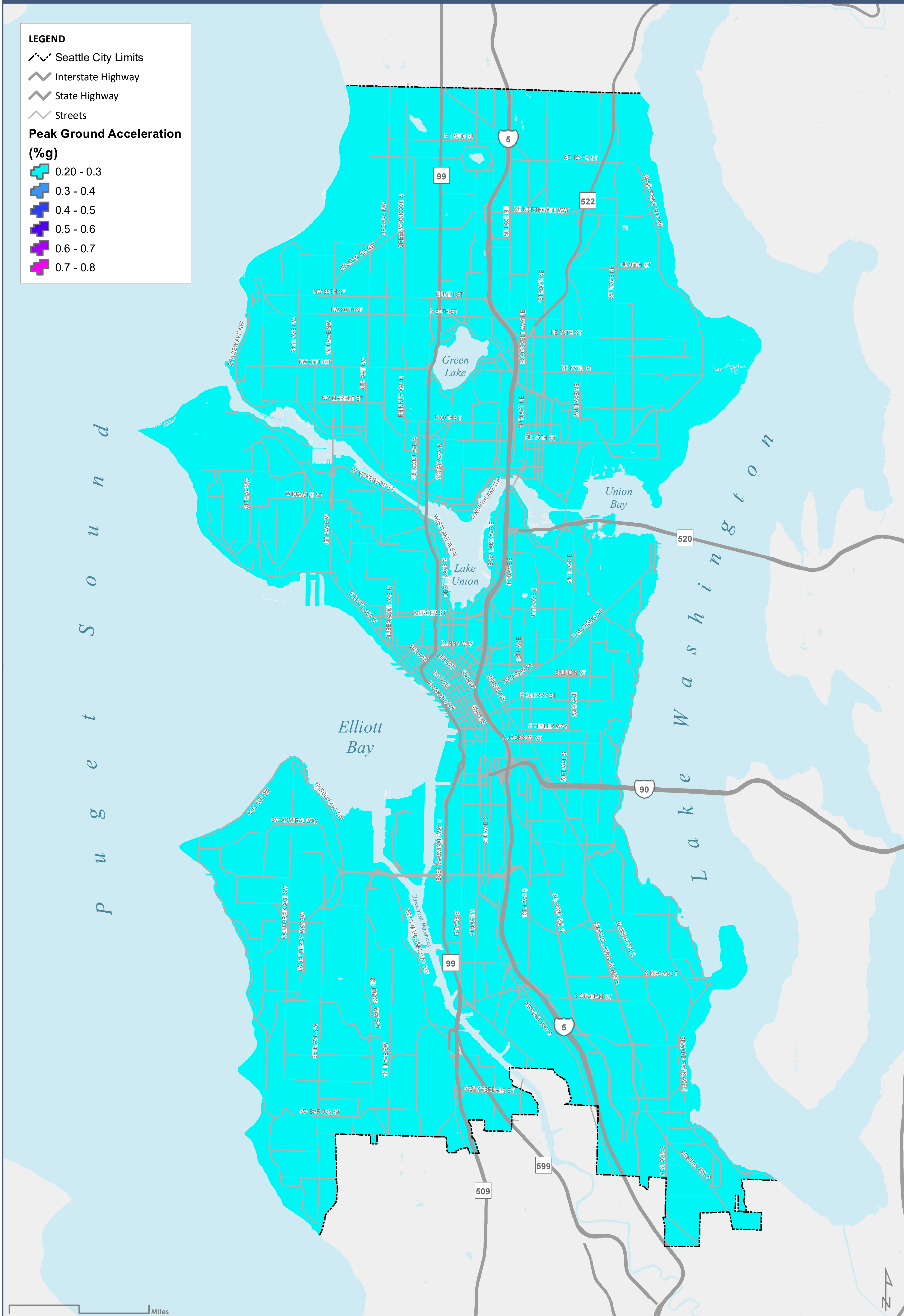
Map D-9: M9.0 CSZ Peak Ground Acceleration



Date: 11/29/2021

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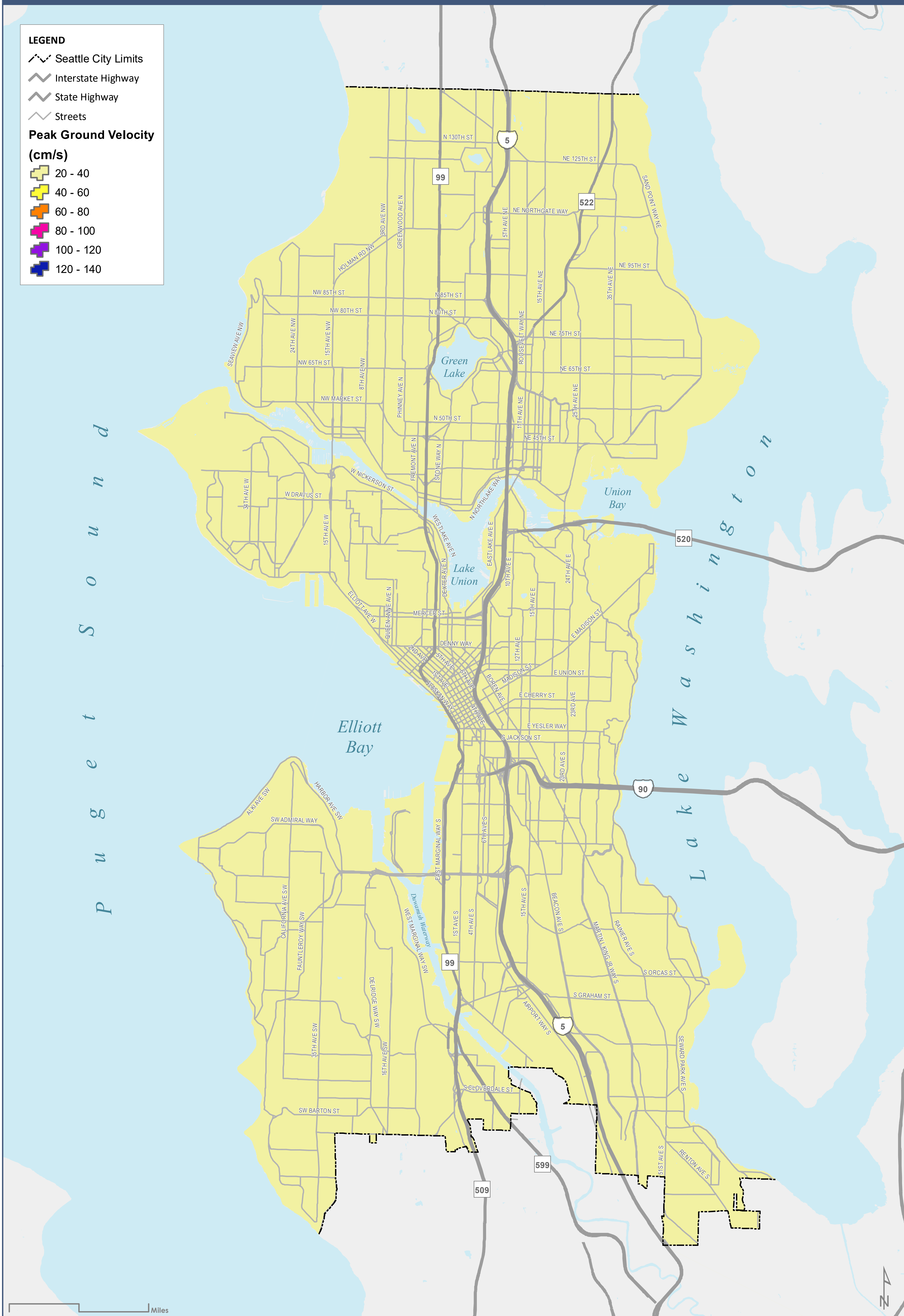
Map D-10: M9.0 CSZ Peak Ground Velocity



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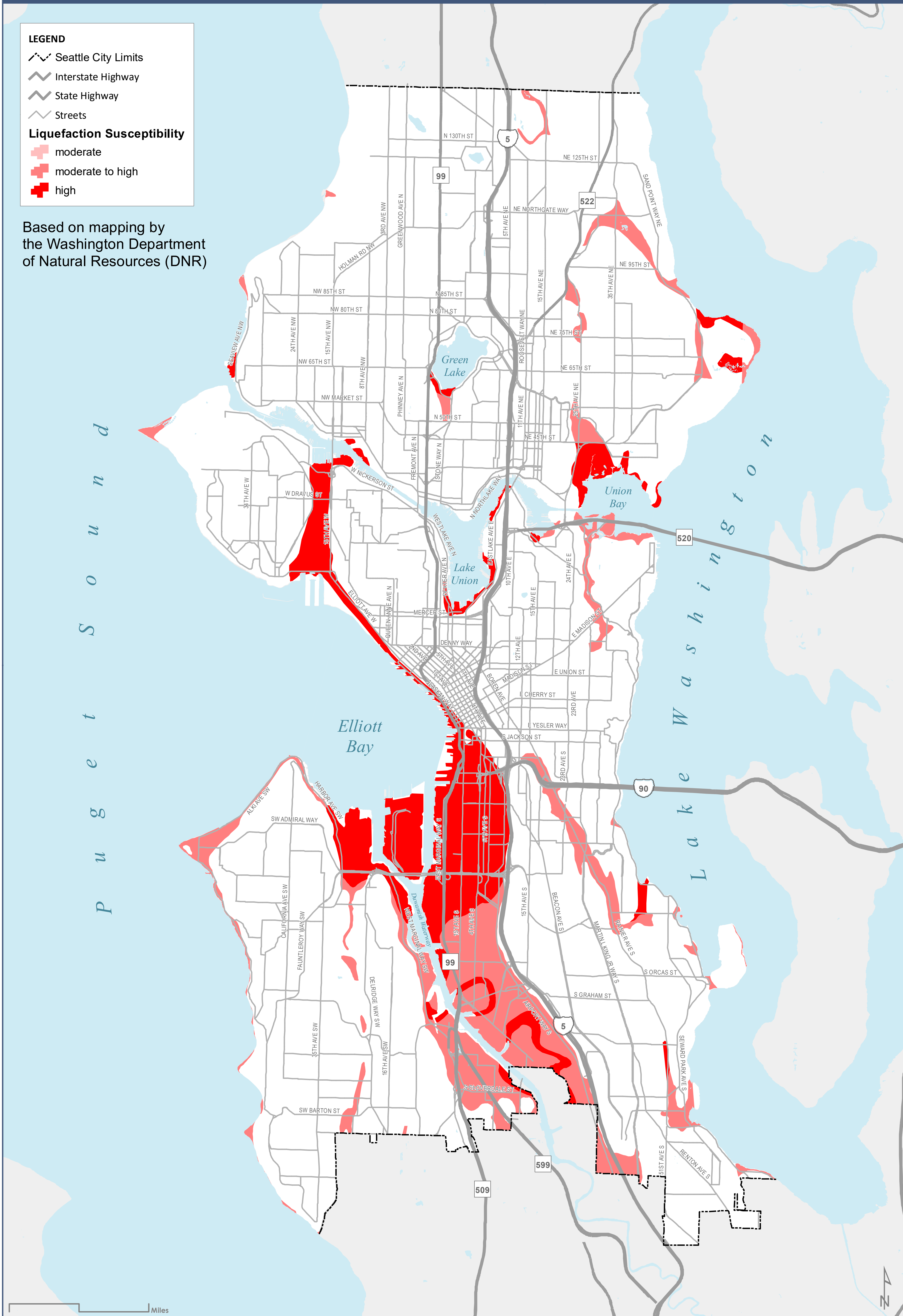
Map D-11: Liquefaction Susceptibility



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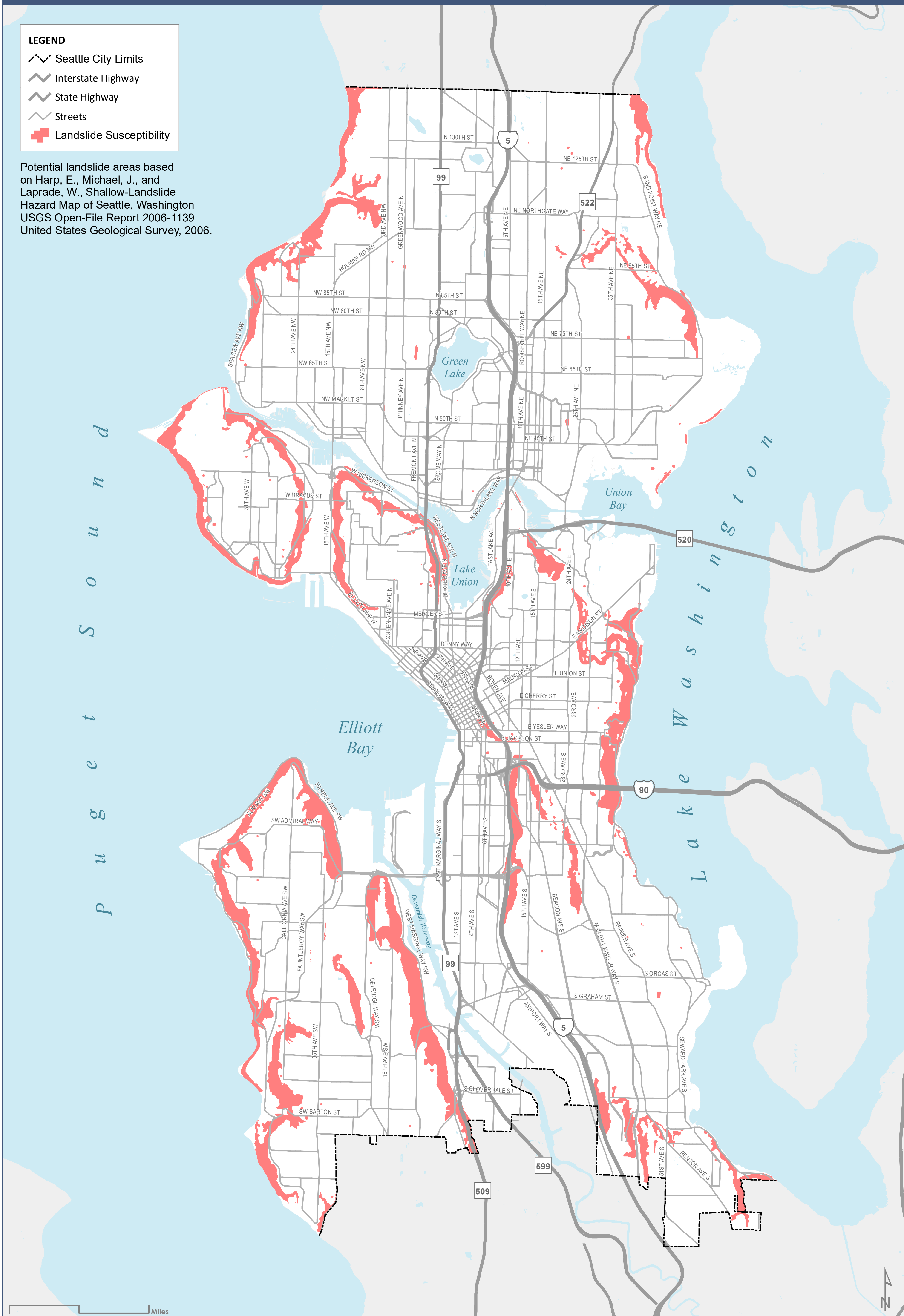
Map D-12: Landslide Susceptibility



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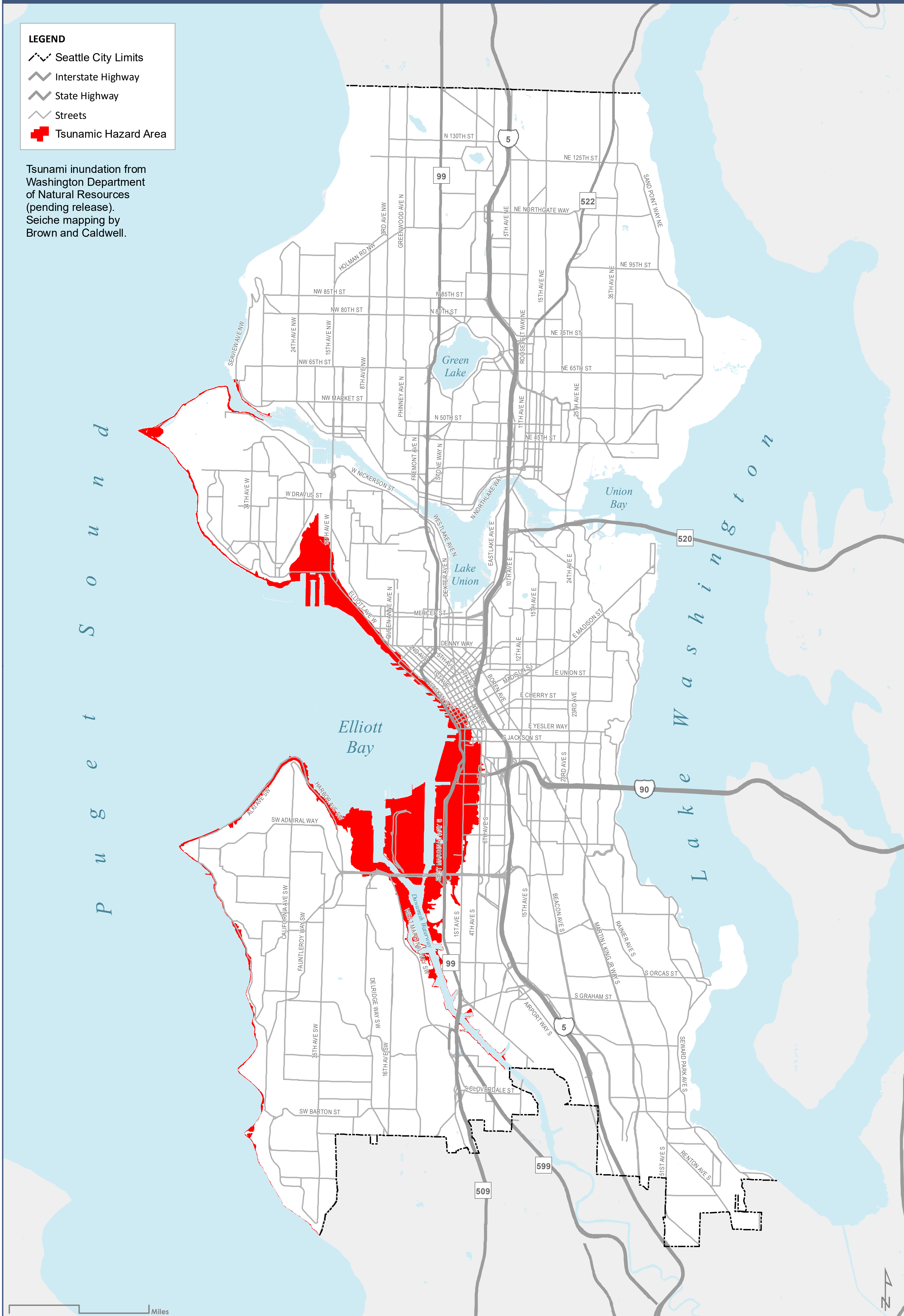
Map D-13: Tsunami Hazard Areas



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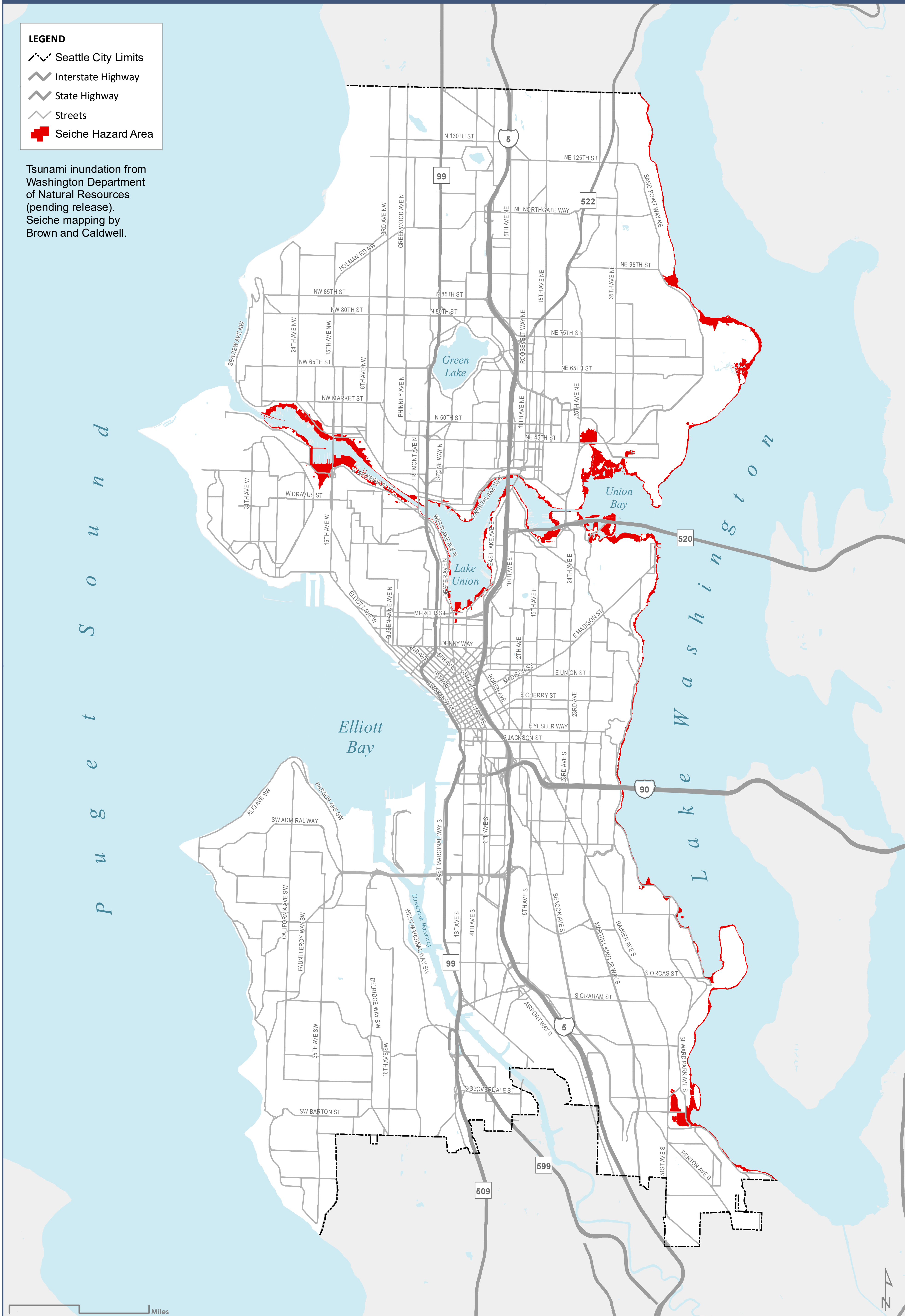
Map D-14: SeicheHazard Areas



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Appendix E: Details from Facility Assessments

Desktop assessments based on available drawings.

Pump Stations 02, 05, 07, 09, 10, 13, and 20. Built in the 1930s or ‘40s, these pump stations are circular in plan and each have a wet well. PS 02, 05, and 13 have only one layer of rebar in the perimeter walls designed to resist external pressure. There is just one layer of rebar on the tension face of the interior wall separating the wet well from the equipment room of the pump stations. Additionally, the length of lap splices for rebar of this era is typically insufficient. All of these pump stations lack ductility and potentially lack adequate strength (when unreinforced faces experience tension) and could result in pump station damage due to shaking. See Figures E-1 and E-2.

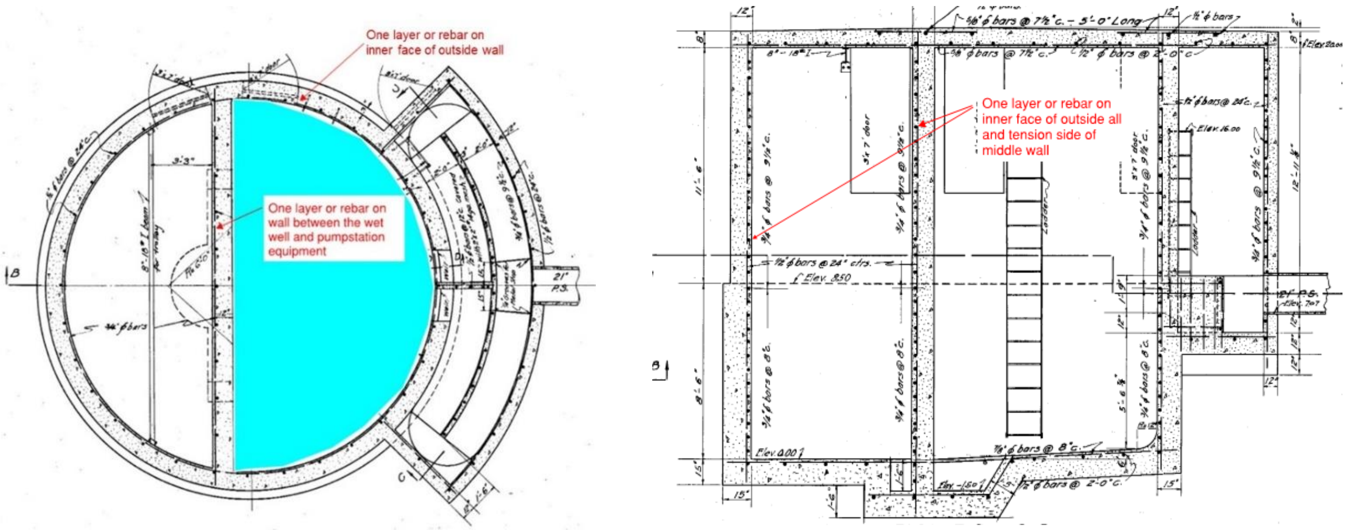


Figure E-1. Plan and section views of PS 05 showing the rebar placement within the outer and center wall

Pump Stations 09 and 13 are especially vulnerable because they also are in liquefaction zones. The amount of movement they may experience is listed in Appendix G. Moreover, Pump Station 09 is also at risk of failure due to floatation. Thus, the piping and structures are at risk of being severely damaged by the earthquake scenarios described in Section 3. There is no splice between the upper wall and lower wall in PS 20. This significantly reduces the pump station’s wall ductility and makes it vulnerable to cracking and separation between the wall segments. This pump station is at risk of being severely damaged by the earthquakes considered in Section 3. See Figure E-2.

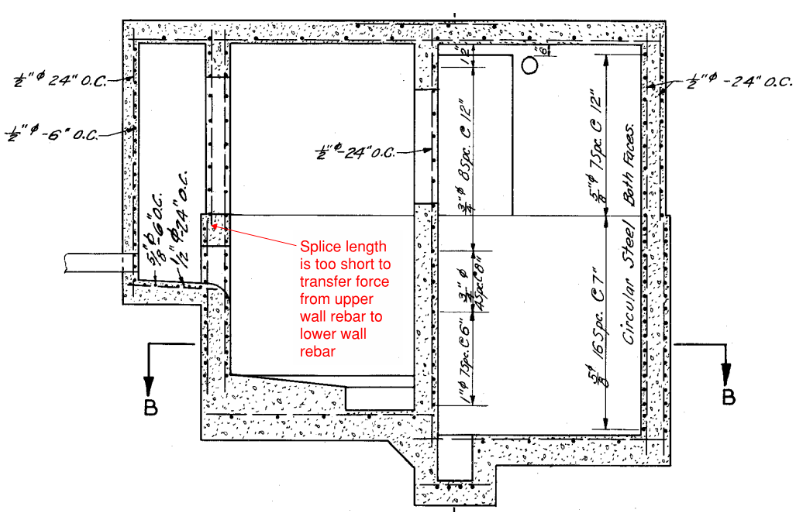


Figure E-2. Section view of PS 20 showing little or no splice where wall thickness changes between lower and upper wall

Pump Station 06. Built in 1930, this pump station is rectangular in plan, and has only one layer of rebar on the inner face of the perimeter walls. The horizontal rebar does not terminate with hooks at the wall corners. One wall is adjacent to a stairwell. The stairwell will prevent the adjacent tank wall from transferring its load to the soil making the load unbalanced and significantly increasing the risk this structure will be severely damaged by the earthquake scenarios described in Section 2. The structure potentially lacks adequate strength (when unreinforced faces experience tension) and a clear load path to transfer the load to the adjacent soil. See Figure E-3.

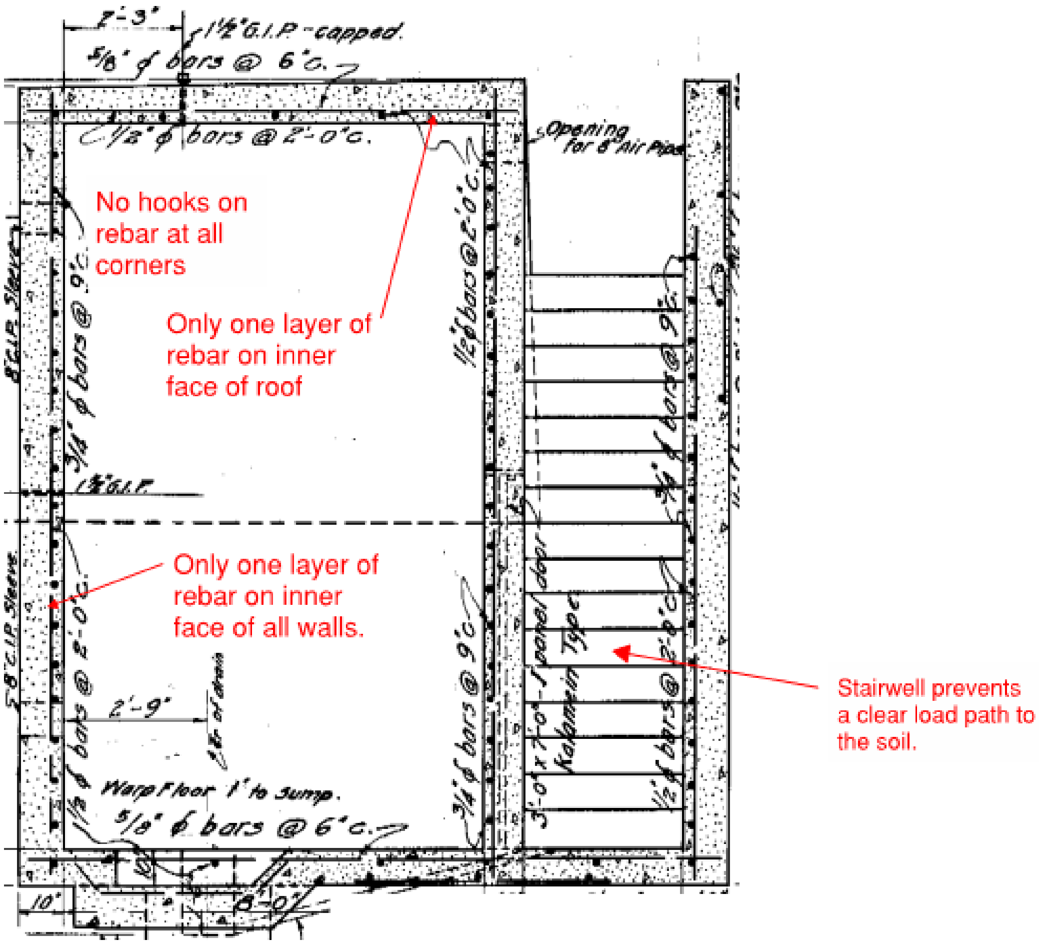


Figure E-3. Section view of PS 06 showing the rebar placement within perimeter walls of the structure

Pump Stations 22. Built in 1952, Pump Station 22 is square in plan. Although there are two layers or reinforcement in walls, reinforcement was designed or improperly detailed such that some walls can only effectively resist out-of-plane forces in one direction. The inner layer horizontal rebar hook at wall corners is improperly detailed, potentially resulting in severe spalling of corner concrete and will be ineffective in resisting out-of-plane bending. The center wall only has one layer of effective reinforcement (the other rebar is ineffective due to its length and improperly detailed rebar hook). The roof appears to be adequately attached to the walls with wall rebar.

Overall, the structure does not have adequate strength and ductility and is at risk of being severely damaged by the earthquakes considered in Section 3. See Figure E-4.

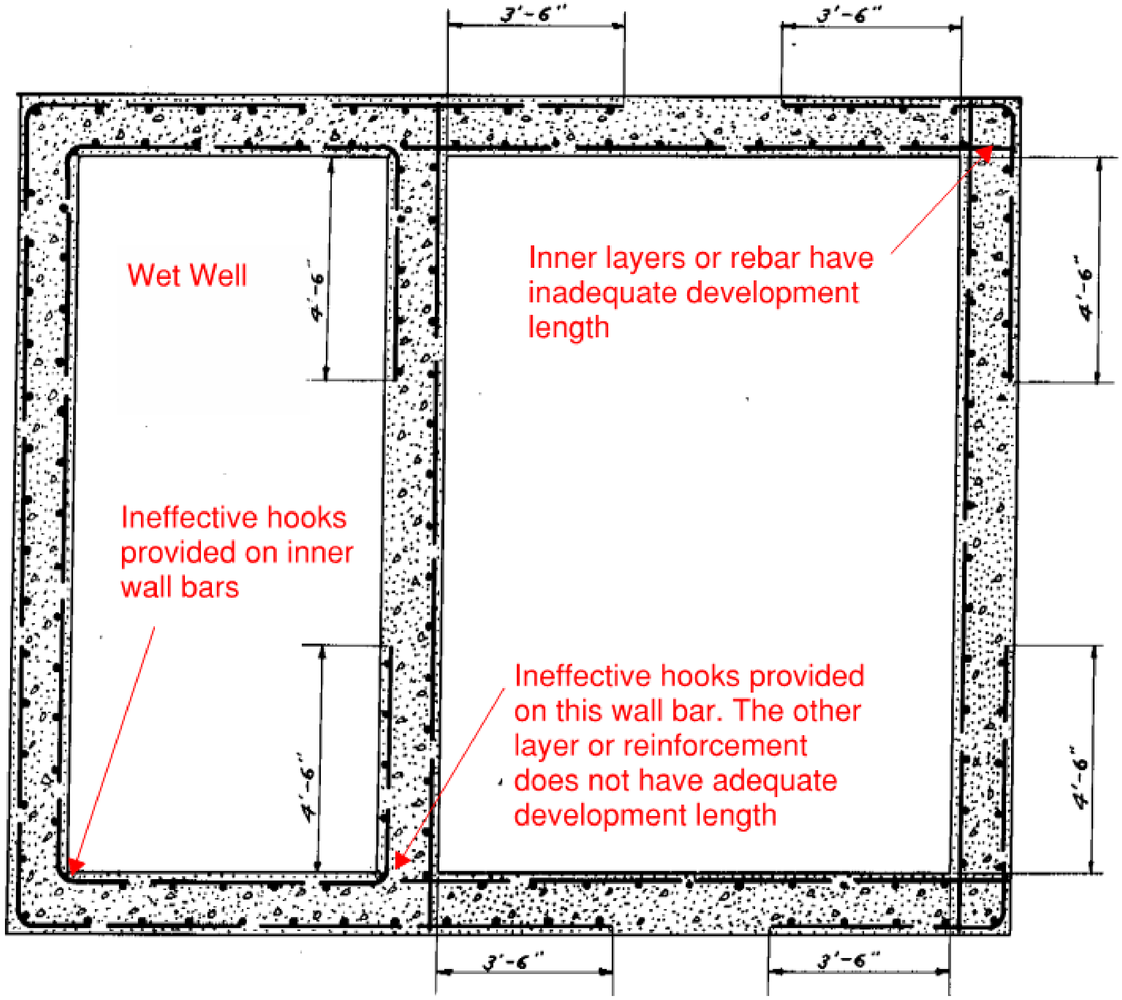


Figure E-4. Plan view of PS 22 showing ineffective hook at inner wall corners of the structure

Pump Station 25. Constructed in 1952, Pump Station 25 is a cast-in-place concrete storage tank with an inside diameter of 16 feet. An interior concrete wall divides the tank between its wet well and equipment room. The structure has two layers of rebar on the perimeter walls. The rebar in the interior wall is detailed to resist the wet well loads using a moment connection at the tank wall, however, the rebar does not have adequate embedment length to be a moment connection. The wall is improperly designed and is at severe risk of damage under the earthquake scenarios considered in Section 3. This structure will likely perform poorly due to its lack of adequate strength and ductility in the interior wall. See Figure E-5.

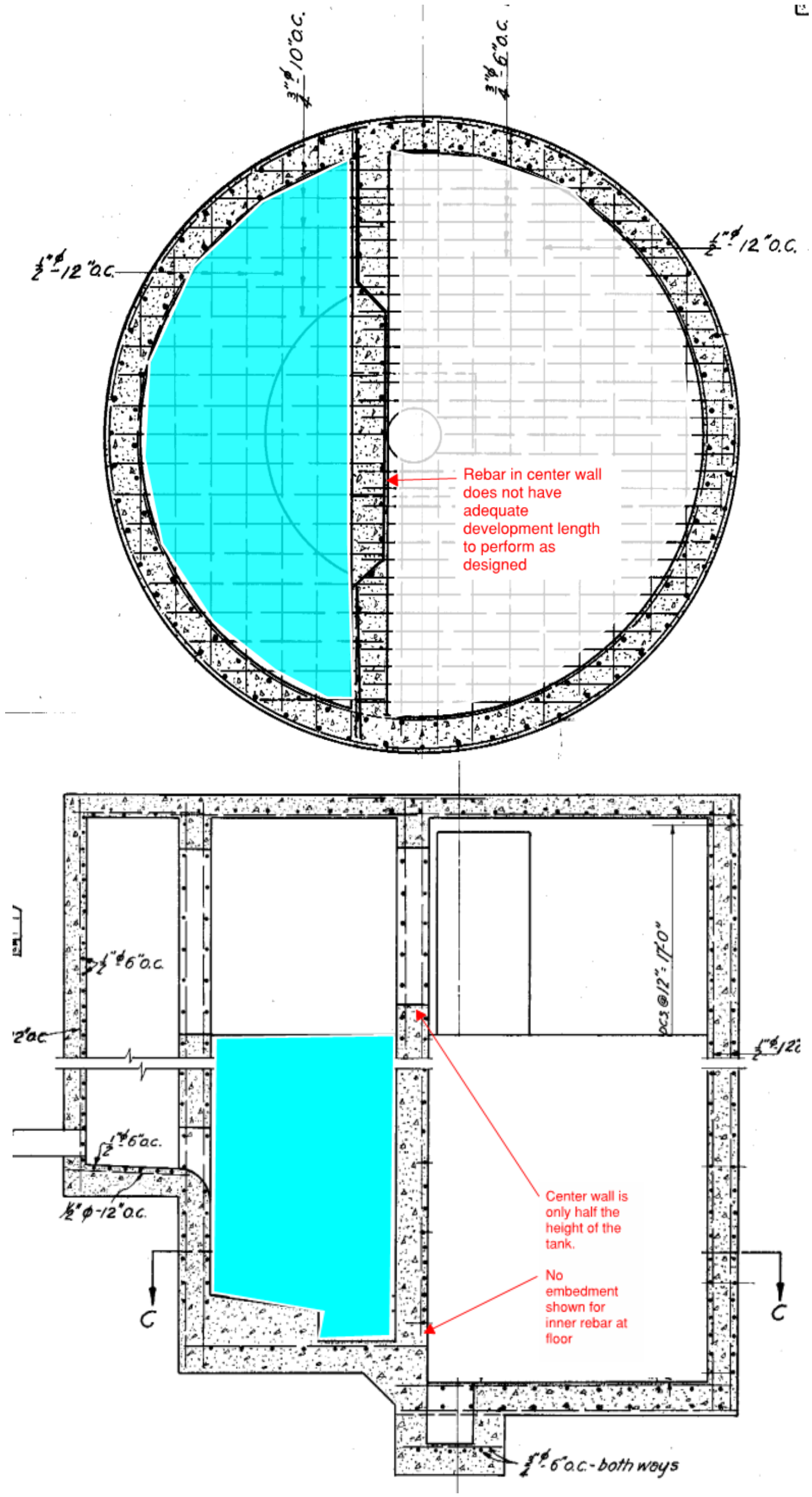


Figure E-5. Plan and section views of PS 25 showing the rebar placement of the interior wall

Pump Stations 49 and 54. Built in 1961, Pump Station 54 is rectangular in plan with approximate dimension of 17.5 ft by 13.5 ft with an overall height of 32 feet. There is an interior wall that separates the wet well from the equipment room. The interior wall is not attached to the tank walls, roof, or floor with rebar. Only a concrete key connects the interior wall to the perimeter walls and floor. This pump station is located in a liquefaction zone with an anticipated PGD of 12 inches. The roof is not connected to walls and there is no rebar in the 3-foot-thick base slab. The tank perimeter walls have no connection to the base slab except for a concrete key. The interior wall could perform poorly under the earthquake scenarios considered in this report. The roof slab is not restrained by perimeter walls, resulting in potential horizontal displacements in both directions. This could cause the pump station to become inoperable due to soil or debris entering the pump station. Overall, this pump station is at risk of being severely damaged by the earthquakes considered in Section 3.

Built in 1960, Pump Station 49 is also rectangular in plan with approximate dimensions of 10 ft by 12 ft and an overall height of approximately 18 feet. Similar to Pump Station 54, the interior wall separates the wet well from the equipment room. The interior wall is not attached to the tank perimeter walls, roof, or floor with rebar. Only a key connects the interior wall to the walls and floor. Although this pump station is not located in a liquefaction zone. The middle wall could still perform poorly under the earthquakes considered in this report. See Figures E-6 and E-7.

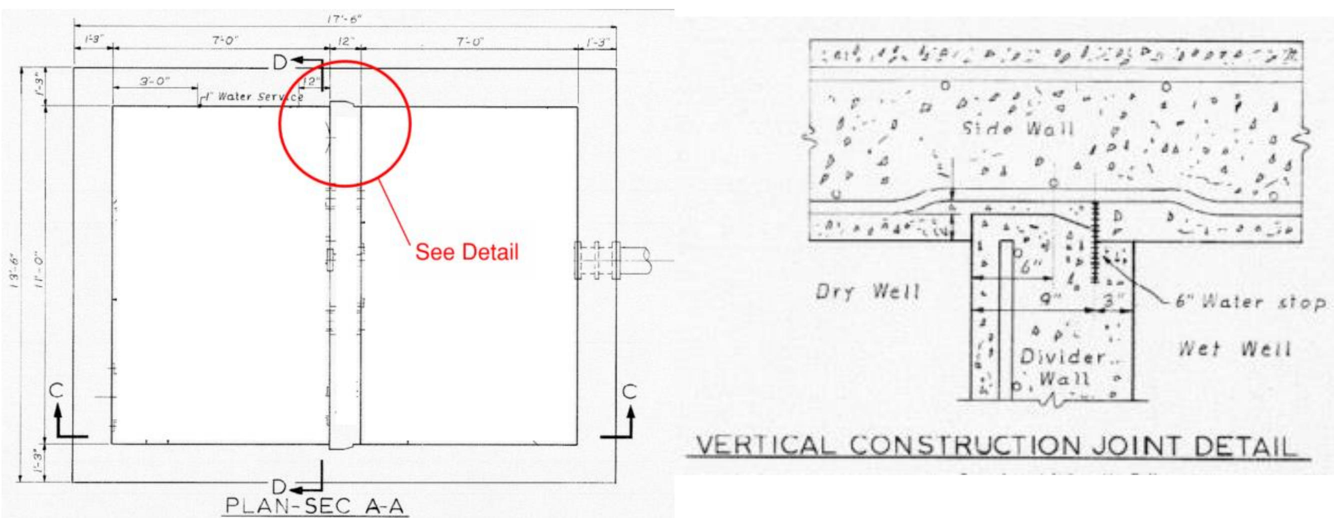


Figure E-6. Plan view of PS 54

The center wall is connected to the outer walls with only a key.

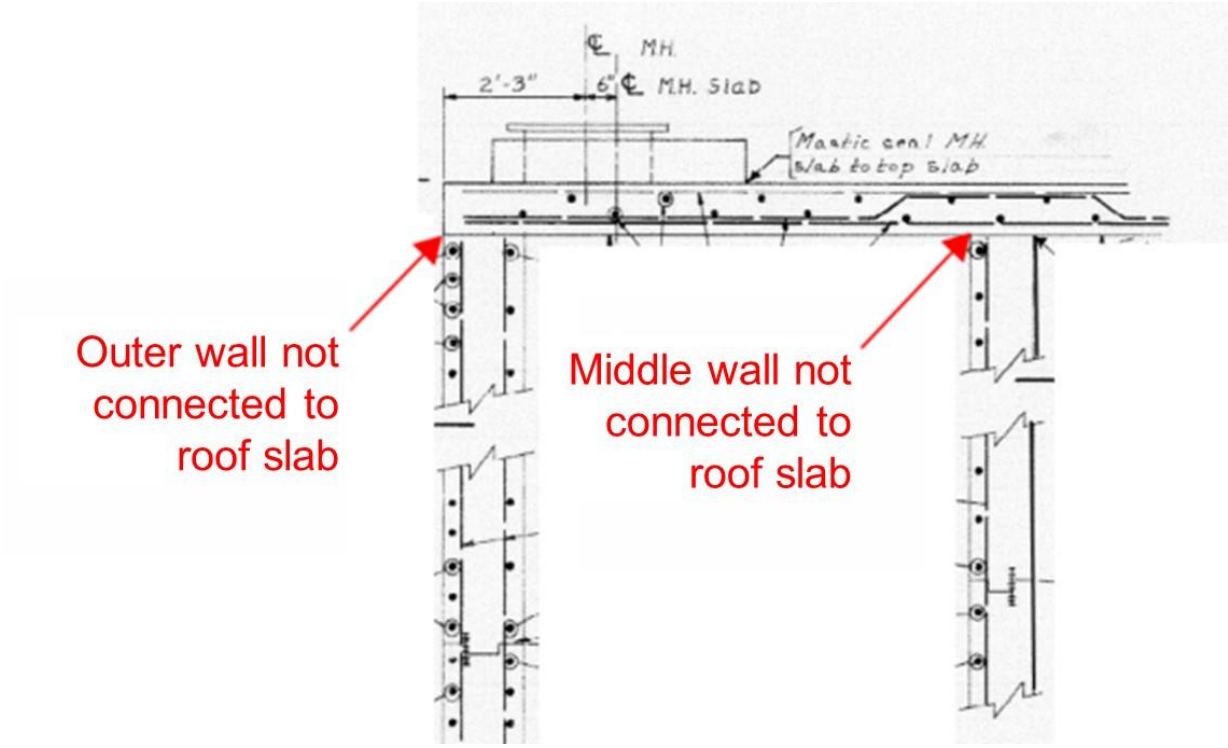


Figure E-7. Section view of PS 49

The center wall is not connected to the walls making the middle wall more vulnerable to displacement.

Pump Station 71 Built in 1963, this pump station is about 20 feet deep with an inside diameter of 13 feet. An interior wall separates the wet well from the equipment room. The interior wall is not attached to tank perimeter walls, roof, or floor with rebar. Only a concrete key connects the interior wall to the perimeter walls and floor. This pump station is also located in a liquefaction zone with an anticipated PGD of 26 inches. The roof is not connected to the walls with rebar. There is no rebar in the 3’-6” thick base slab. Tank perimeter walls have no connection to base slab except for a concrete key. The span is short but the interior wall could experience damage in the earthquake scenarios considered in Section 3 of this report. The anticipated ground movement could easily move the roof making it possible for soil or debris to enter the structure. See Figure E-8.

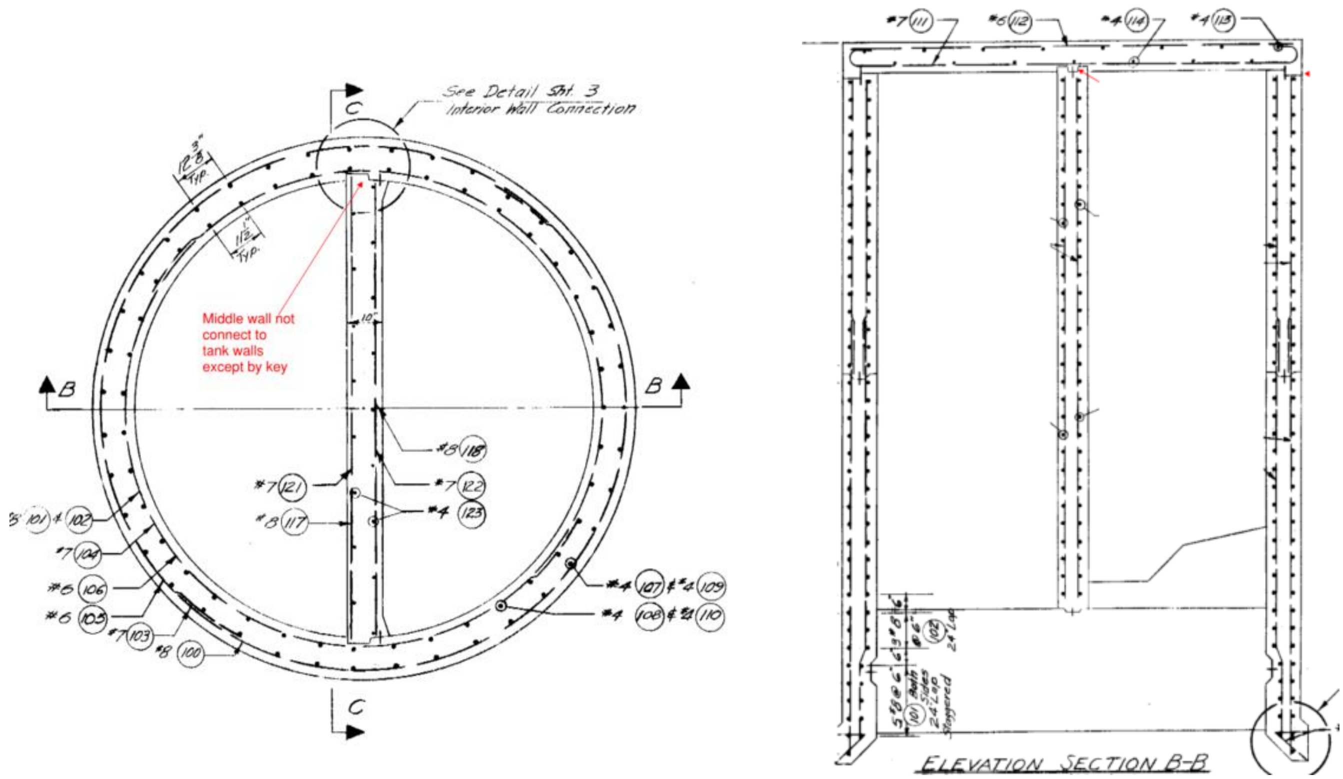


Figure E-8. Plan view of PS 71

The center wall is connected to the outer walls with only a key.

Pump Stations 11, 21, 55, and 78. All the pump stations appear to be constructed of large hollow precast concrete segment(s). The roof is not attached to perimeter walls with rebar and could be displaced from the walls of the structure. The precast concrete riser joints do not provide resistance to uplift force resulting from overturning due to lateral earth pressures on the pump stations. Because these pump stations are in a liquefaction zone, the joints could separate. The roof slab is not restrained by perimeter walls, resulting in potentially large horizontal displacement. Either could cause the pump station to become inoperable due to soil or debris entering the pump station. These pump stations will likely perform poorly in the earthquake scenarios considered in Section 3 of this report.

Pump Stations 38, 71 (weir structure), 72, and 73. All these pump stations have walls that are cast-in-place. The roof is not attached with rebar and could be displaced from the perimeter walls of the structure. Because this structure is in a liquefaction zone, the roof movement could cause the pump station to become inoperable due to soil or debris entering the pump station. These pump stations will likely perform poorly in the earthquake scenarios considered in Section 3 of this report.

Pump Stations 01, 06, and 20. These three pump stations have a mainline that is rigidly attached to two adjacent structures. These mainlines have a moderate risk of failure when not located in liquefaction zones, which applies to Pump Stations 01, 06, and 20. Since these mainlines are short and stiff, the mainlines are vulnerable to differential movement between the structures. The piping shown in Figure E-9 illustrates this issue. These pump stations will likely perform poorly in the earthquake scenarios considered in Section 3 of this report.

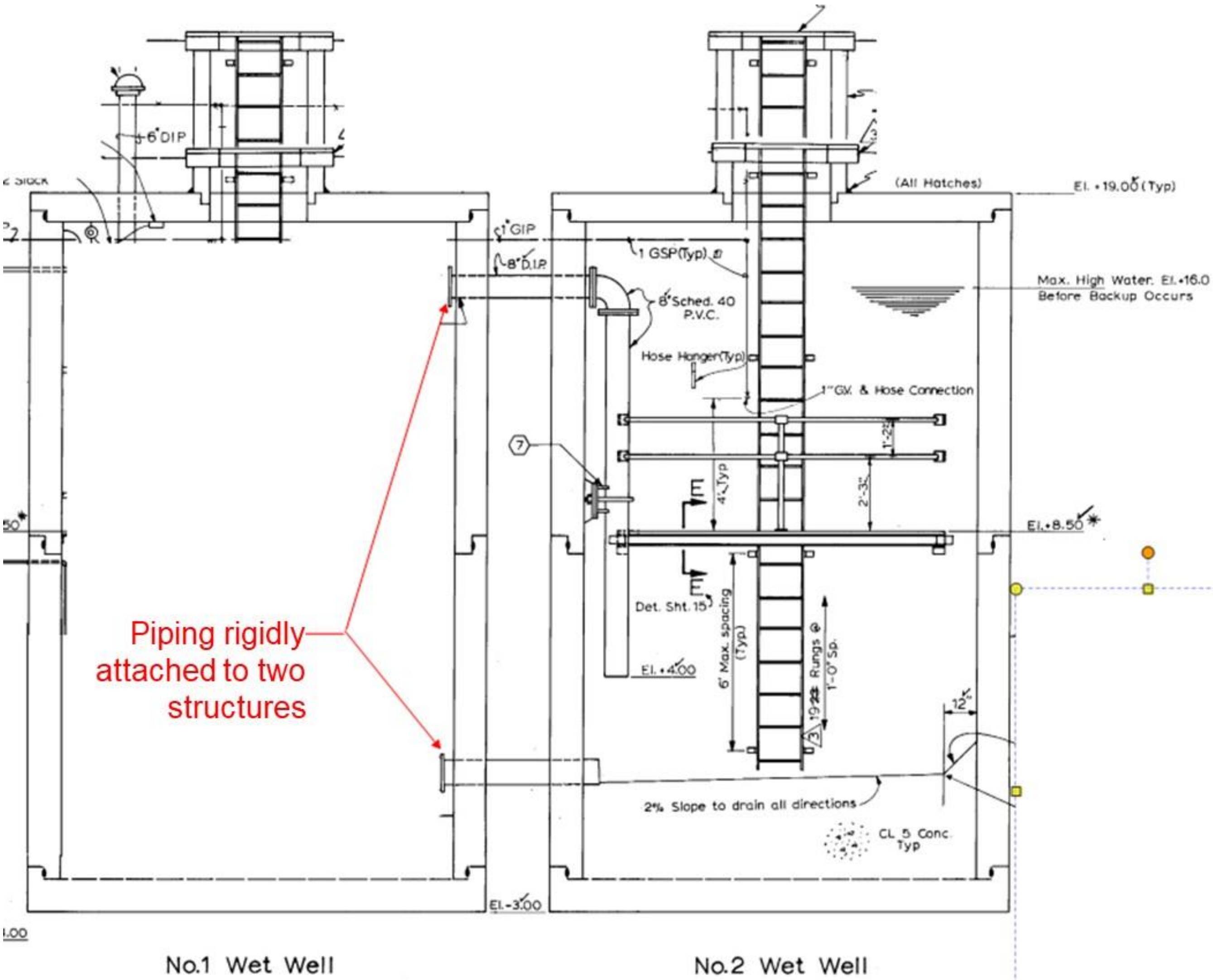


Figure E-9. Section view of PS 1

Several mainlines interconnect the two wet wells making the mainline vulnerable to damage.

Pump Stations 04, 17, 21, 36, 37, 38, 42, 44, 45, 50, 54, 55, 61, 62, 63, 65, 70, 71, 72, 73, 74, 75, 76, 78, 114, and 118. These pump stations are located in liquefaction zones. The piping leaving and entering the structures is not designed to accommodate the anticipated PGD. The vulnerability of the piping systems suggest these structures are at severe risk of being inoperable after the earthquake scenarios considered in Section 3 of this report.

Many of the structures may sustain damage, but should remain functional. However, they may not be able to operate due to the anticipated ground movement affecting electrical conduit duct banks and/or other nonstructural issues, which were not assessed for this study.

Pump Stations 09, 36, 37, 39, 50, 70, 78, and 83. These pump stations are at risk of failure due to floatation. Pump Stations 9, 37, 39, and 70 have wet wells. The wet wells were assumed to be empty in this evaluation. No equipment load was added.

CSO Facilities 2 and 3, Tanks 168 and 169. Tanks 168 and 169 are 100-foot-diameter post-tensioned tanks. The tanks were constructed in 1987. The tanks were designed per the 1979 Uniform Building Code. Based on review of the design drawings, several deficiencies have been identified. There is inadequate load path from perimeter wall to the foundation. Roof slab is not adequately attached to perimeter wall. In addition, the shear reinforcement details of interior concrete columns are not clear, requiring a future field investigation to confirm construction details. Based on the findings of actual construction details, a structural analysis will then be performed to determine if interior columns are able to deform with the rest of the structure without experience brittle shear failure. Considering the era of construction, the design earthquake and the importance of the structure, it is recommended these tanks be evaluated using a Tier 3 evaluation per ASCE 41. See Figure E-10.

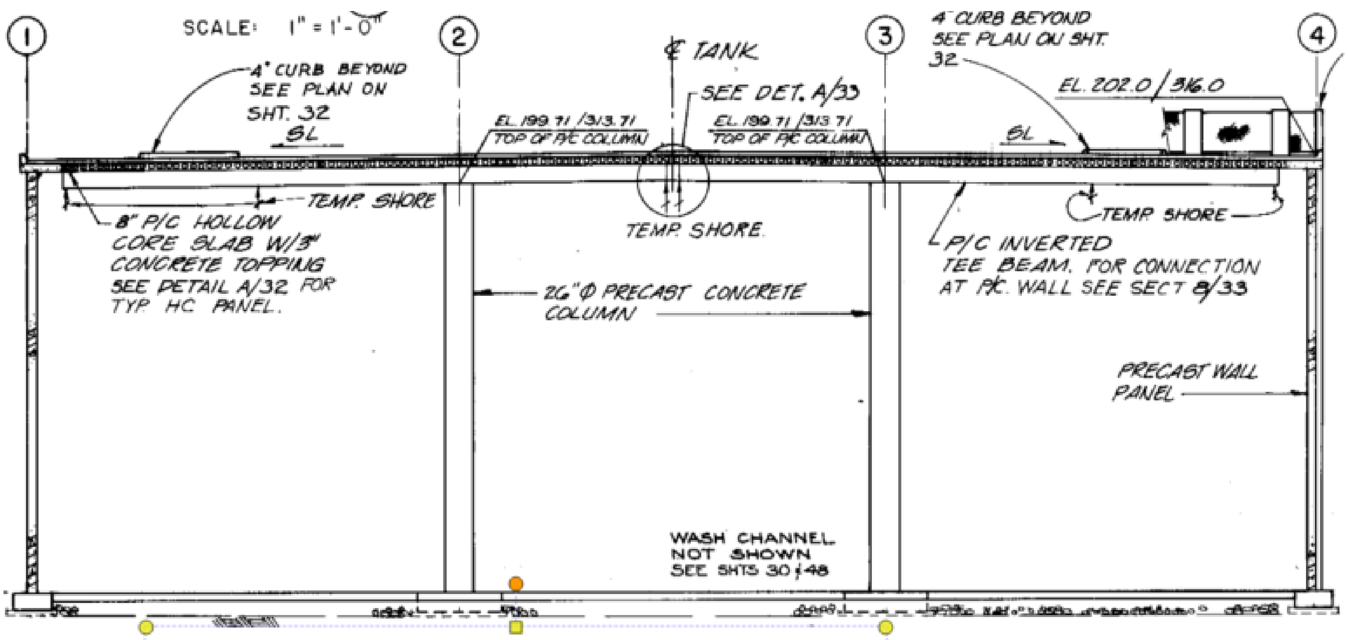


Figure E-10. Section view of Tanks 168 and 169

CSO 2 and 3 Control Building. The Control Building is 15'-4" x 29'-4" Cement Masonry Block Building (CMU) and was constructed in 1987. The building consists of a series of CMU panels that are joined together with welded connections. The building foundation has a stem wall foundation.

Based on review of the available drawings, a number of deficiencies have been identified. The CMU panels were joined together using welded connections. These connections lack ductility and may not have adequate strength. The roof diaphragm is constructed of precast hollow planks without any topping slab, and will have inadequate strength to resist seismic forces associated with the earthquake scenarios. Such an untopped precast concrete diaphragm cannot function as cross ties in the direction perpendicular to hollow planks. Considering the importance of the structure, the era of construction and design earthquakes being evaluated, it is recommended this structure be evaluated using a Tier 3 evaluation per ASCE 41. The connection between the roof and masonry walls should also be evaluated. See Figures E-11 and E-12.

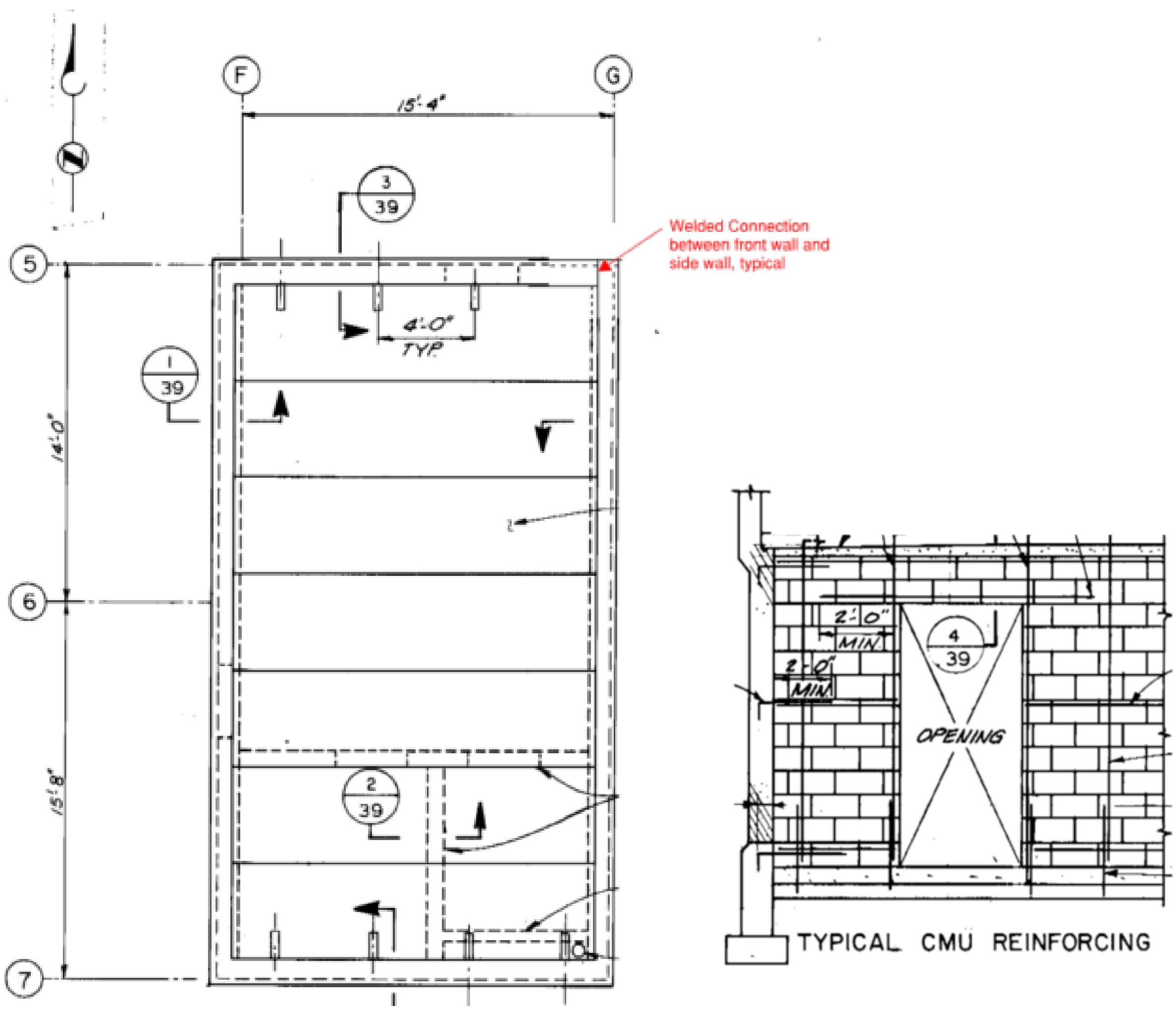
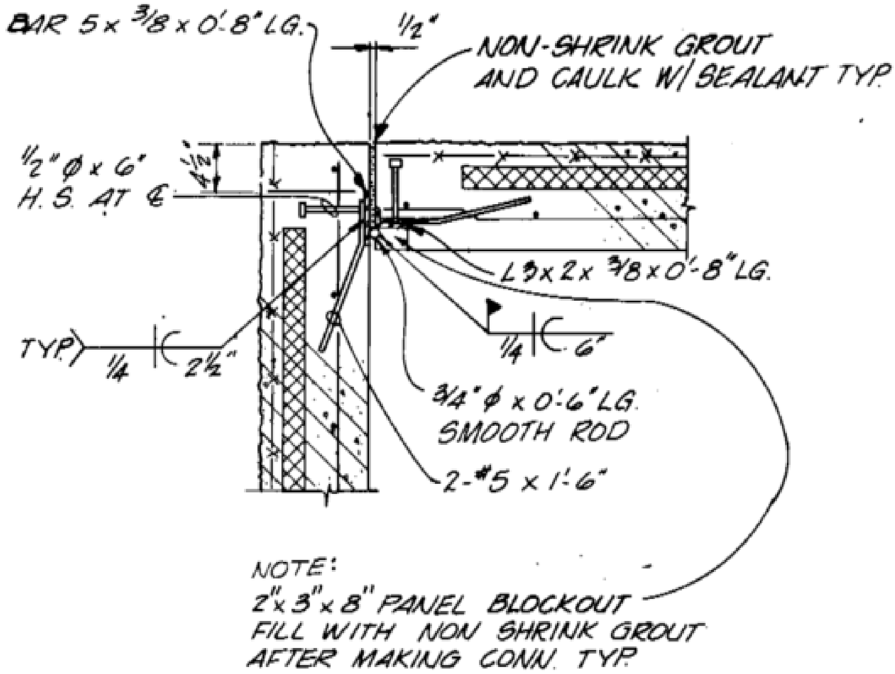


Figure E-11. Plan and section view of the CSO 2 and 3 control buildings



CORNER PANEL CONNECTION

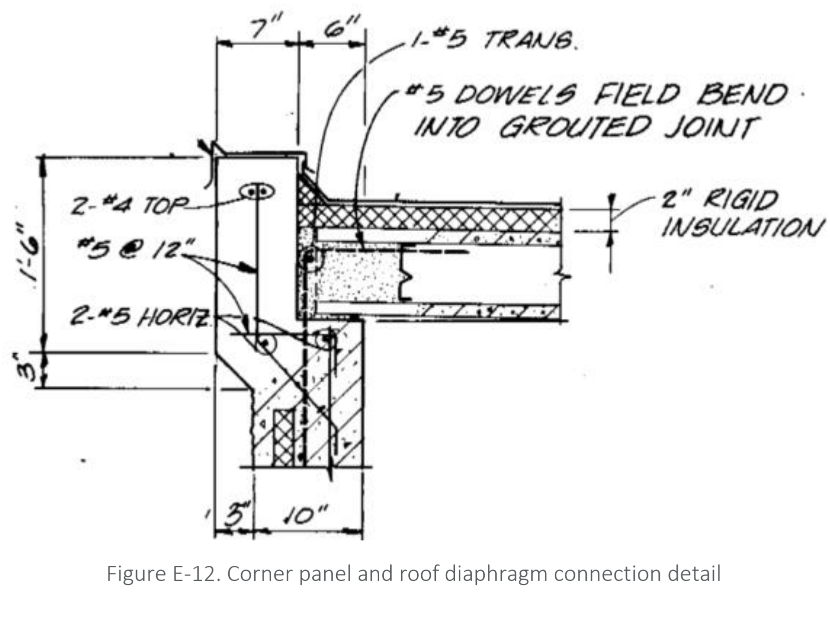


Figure E-12. Corner panel and roof diaphragm connection detail

CSO 24-Box Conduit. There are several box conduits shown in the Lake Washington North CSO 24 Project. The box conduit with the deficiency is shown on Drawing No. BC-10-C, BC-11-C, BC-12-C and BC-13-C. The conduits were constructed in 1988. These box conduits have two chambers that are 11’-8” wide by 12’ high. The conduits were built with some corner rebar improperly detailed. The inner bar hooks at the corners do not have proper embedment, resulting in reduced conduit ductility. This lack of ductility makes these structure vulnerable to the earthquakes described in section 3.

The incorrect placement of the bars is shown on BC-13-C. See Figure E-13.

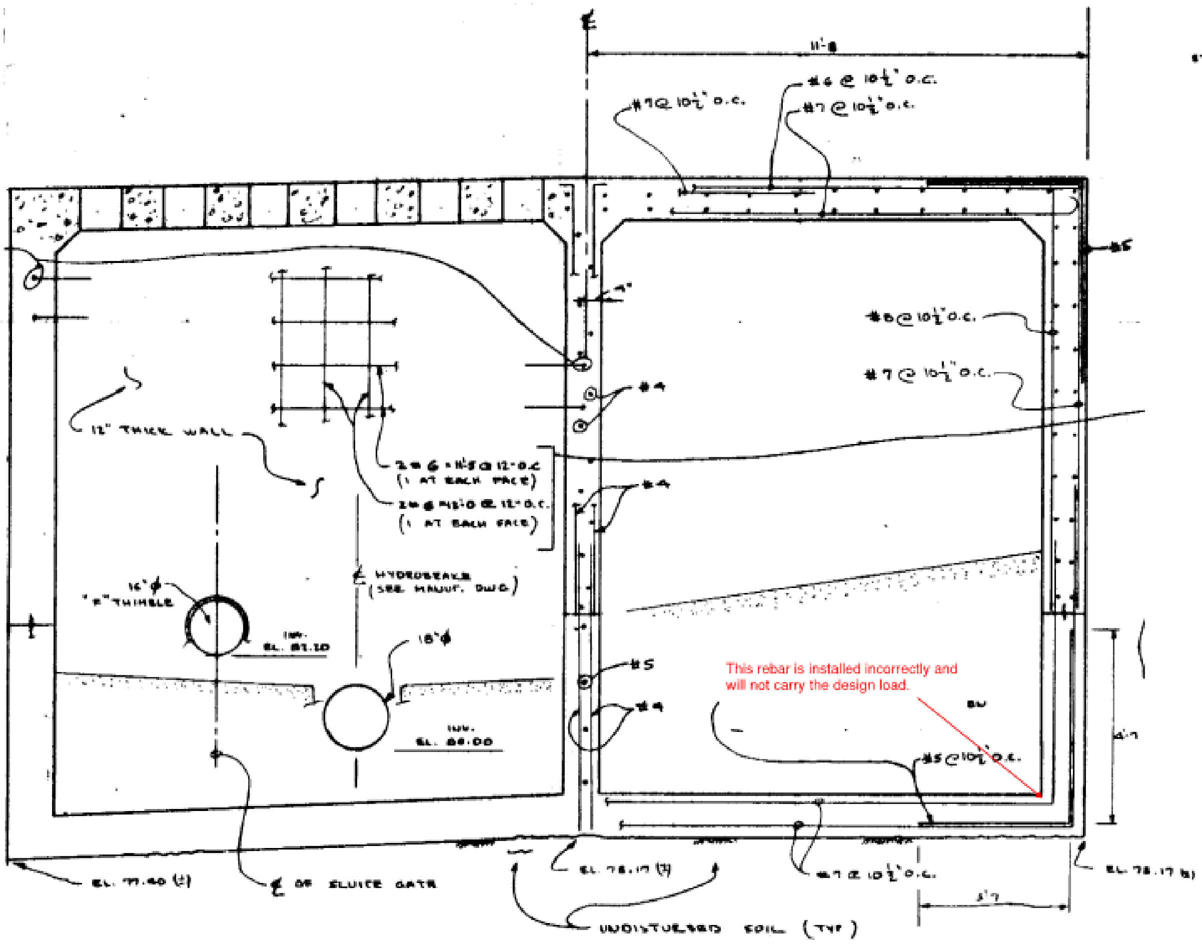


Figure E-13. Box conduit with wall to floor corner rebar shown to have hooks detailed improperly

CSO 24-Manholes 12, 13, and 20. These three manholes structures are installed on a single slab. The manholes are connected to 144-inch mainlines, each with a concrete bulkhead. A 72-inch mainline interconnects the manhole structures as well as the 144-inch-diameter mainline. There are two joints between the mainline and manhole structures. The close proximity of the structures and significant stiffness associated with these large diameter mainlines make this combination of structures and mainline vulnerable to differential movement. This connection was rated a moderate risk under the earthquake scenarios described in Section 3 of this report. See Figure E-14.

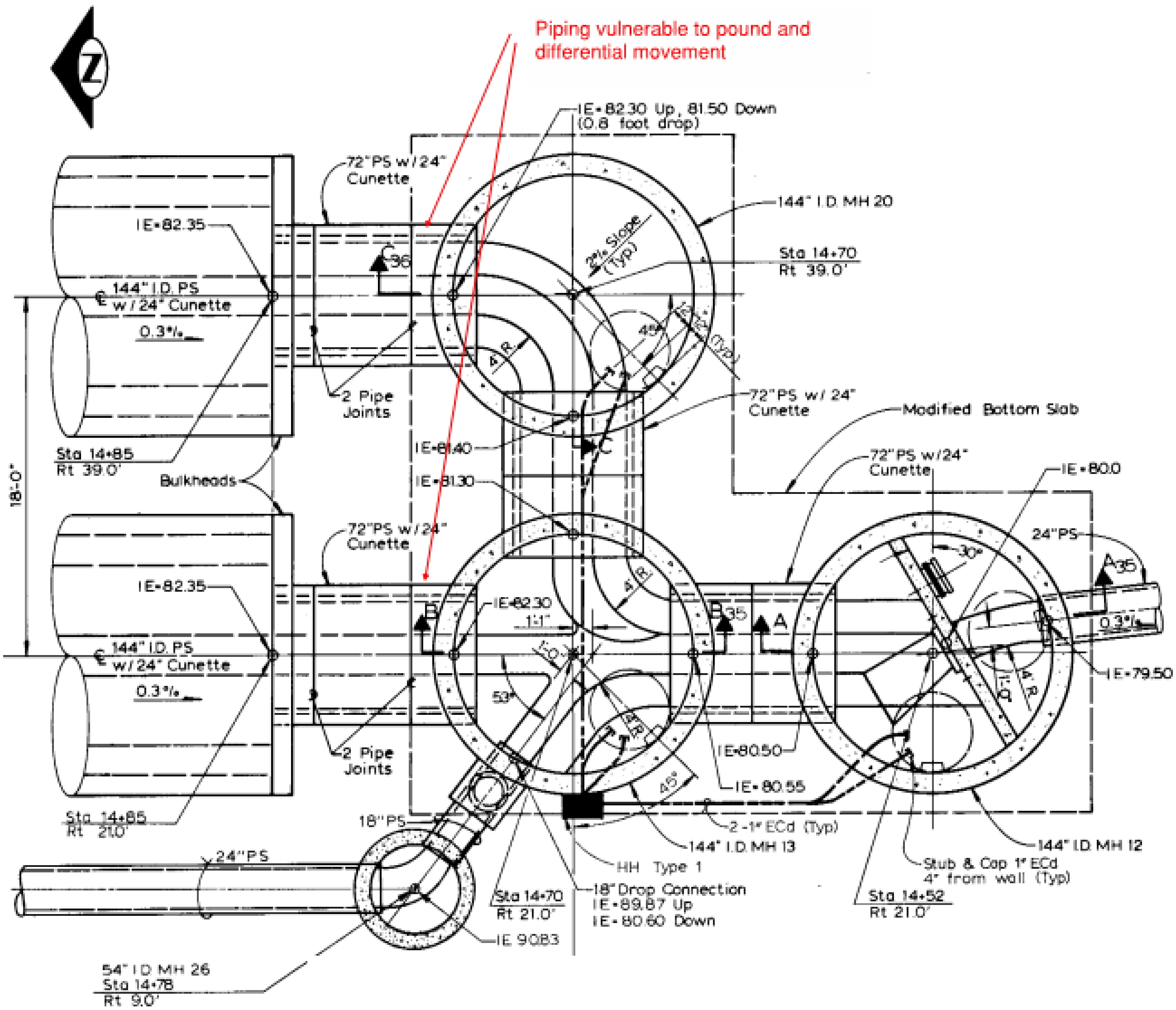
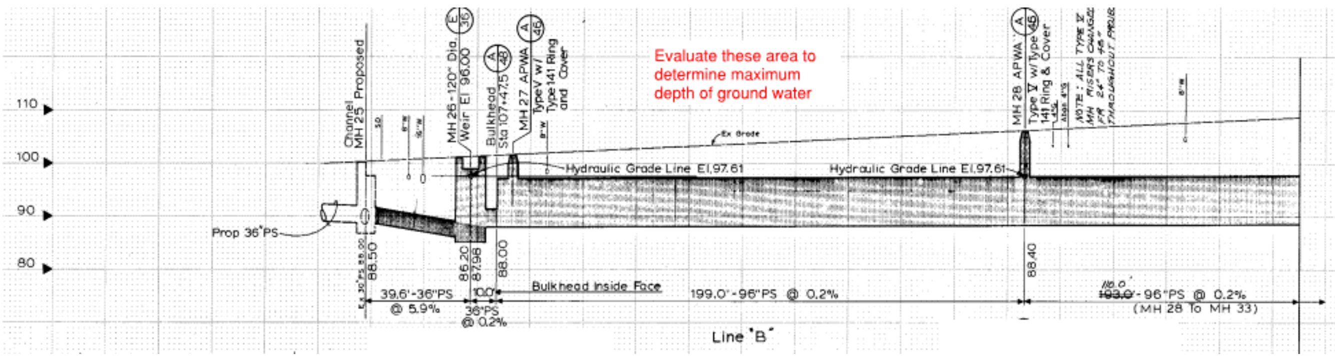


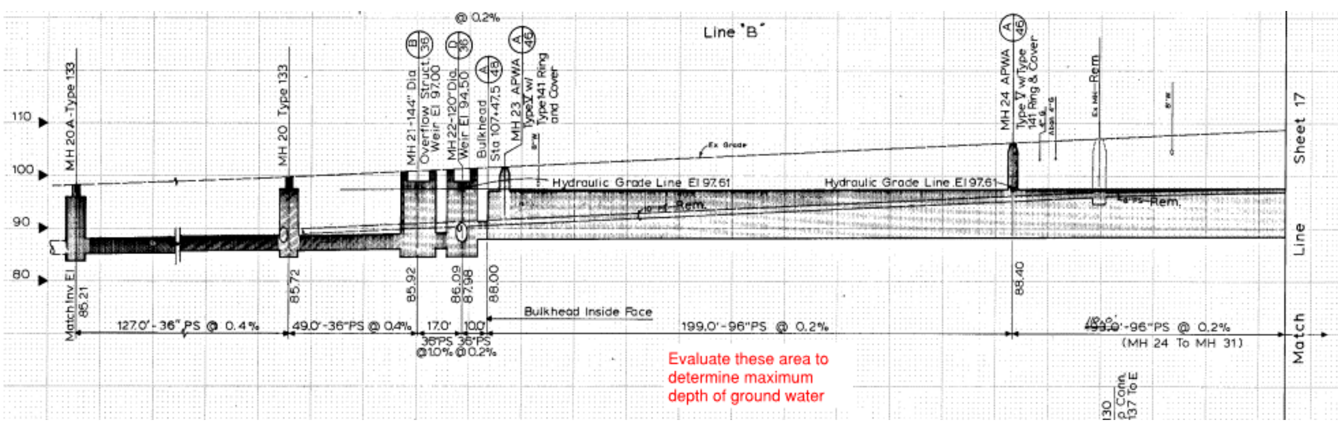
Figure E-14. Plan view of the connection between manholes 12, 13, and 20, and the 144-inch-diameter CSO piping

CSO Facility 22-Lake Washington North 72- and 96-inch Diameter Mainlines. There are several areas along the mainline alignment where the cover over the top of the mainline is less than the diameter of the mainline. This lack of cover makes it vulnerable to floatation. If the mainline floats, it will render the CSO facility inoperative.

Figure E-15 shows two profiles where the cover is less than the diameter of the mainline.



Profile 1 – 96" diameter mainline



Profile 2 – 72" diameter mainline

Figure E-15. CSO 22-Lake Washington North 72- and 96-inch-diameter mainlines

Profile 1 and 2 show locations where the cover over a 96-inch- and 72-inch-diameter mainline, respectively, is less than one mainline diameter, making them vulnerable to floatation

Appendix F: Example Calculation for Mainline Likelihood of Failure Score

Example Calculation: Drainage Mainline 2213885

Figure F-1. Location of drainage mainline 2213885 for example likelihood calculation

Identify

Identify from: <Top-most layer>

DWW_Mainlines_SF7_FOS
Seattle Public Utilities

Location: 1,262,326.441 212,759.895 Feet

Field	Value
MXM_LAST_WO_JPNUM	4567
MXM_LAST_WO_CLSFCTN	STRUCTURE INSPECT
MXM_LAST_WO_STATUS	CLOSE
MNL_MXM_LCTN_ID	<null>
NEAR_FID	388
NEAR_DIST	922.827094
NEAR_ELEV	0
PIPE_ELEV	40
LAT	47.572583
LONG	-122.365142
LIQ_SUSCEPT	very high
PGA_G	0.658
MSF_SCENARIO	1.19
PGA_MSF	0.552941
PGD_VMAX	1.2
PGD_V_INDEX_B	0.25
PGD_V_INDEX_C	-2.4
INDEX_PGD_V	1.044561
PGD_HMAX	11
PGD_H_INDEX_B	0.23
PGD_H_INDEX_C	-3.1
INDEX_PGD_H	10.319673
PGD_H_L	2.011391
PGD_V_L	0.000001
PGD_TOTAL_FT	2.266452
PGD_TOTAL_IN	27.19742
LIQ_PROBABILITY	0.940588
SLIDE_FOS	11
PGV_INCHSEC	57.011811
SLIDE_CERT	<null>
LIQ_PGD_IN	27.19742
SLIDE_FOS_1	11
SLIDE_PROBABILITY	<null>
SLIDE_KYMED	<null>
SLIDE_PGD_IN	0
K1	1
K2	1
PGV_RepairRate	0.106612
PGD_RepairRate	2.859695
Total_RepairRate	2.966307
Total_NumRepairs	2.015339
WWBackbone	<null>
RR_Weighted	2.309079

Identified 1 feature

Identify

Identify from: <Top-most layer>

DWW_Mainlines_CSZ9_FOS
Seattle Public Utilities

Location: 1,262,326.441 212,690.942 Feet

Field	Value
MXM_LAST_WO_JPNUM	4567
MXM_LAST_WO_CLSFCTN	STRUCTURE INSPECT
MXM_LAST_WO_STATUS	CLOSE
MNL_MXM_LCTN_ID	<null>
NEAR_FID	388
NEAR_DIST	922.827094
NEAR_ELEV	0
PIPE_ELEV	40
LAT	47.572583
LONG	-122.365142
LIQ_SUSCEPT	very high
PGA_G	0.249
MSF_SCENARIO	0.627
PGA_MSF	0.397129
PGD_VMAX	1.2
PGD_V_INDEX_B	0.25
PGD_V_INDEX_C	-2.4
INDEX_PGD_V	0.902716
PGD_HMAX	11
PGD_H_INDEX_B	0.23
PGD_H_INDEX_C	-3.1
INDEX_PGD_H	9.29104
PGD_H_L	1.810902
PGD_V_L	0.000001
PGD_TOTAL_FT	2.023429
PGD_TOTAL_IN	24.281144
LIQ_PROBABILITY	0.886268
SLIDE_FOS	11
PGV_INCHSEC	22.362205
SLIDE_CERT	<null>
LIQ_PGD_IN	24.281144
SLIDE_FOS_1	11
SLIDE_PROBABILITY	<null>
SLIDE_KYMED	<null>
SLIDE_PGD_IN	0
K1	1
K2	1
PGV_RepairRate	0.041817
PGD_RepairRate	2.598793
Total_RepairRate	2.640611
Total_NumRepairs	1.794057
WWBackbone	<null>
RR_Weighted	2.087398

Identified 1 feature

Identify

Identify from: <Top-most layer>

Seismic Risk Scores
Drainage

Location: 1,262,328.411 212,675.182 Feet

Field	Value
OBJECTID	48806
SHAPE	Polyline
FEA_KEY	2213885
RR_1000ft_SFZ7	2.309079
RR_1000ft_CSZ9	2.087398
RR_1000ft_2events	2.107551
Likelihood_score	4.215102
MNL_FEA_KEY	2213885
serve_crit_fac	0
pipe_diam	2
greater_14in_diam	0
high_use_area	0
mjr_transpo_rte	1
near_h20	0
Consequence	3
Likelihood	4.215102
Equity	3
Risk_Score	15.645306
SHAPE_Length	679.414232
MNL_USE_PERMIT_TEXT	Drainage

Identified 1 feature

Figure F-2. Snapshots of GIS attribute data for drainage mainline 2213885

The following steps show the likelihood of failure calculations for drainage mainline 2213885.

1. See Table 5-1 for reinforced concrete mainline greater than 48 inches diameter: K_1 = 1.00 and K_2 = 1.00
2. Obtain geotechnical hazard information for ground motion (provided with SPU geospatial data):
   - Probability of Liquefaction, SFZ M7.0 = 0.940588
   - Probability of Liquefaction, CSZ M9.0 = 0.886268
   - $PGV_{SFZ\ M7.0}$ = 57.011811 inches per second
   - $PGV_{CSZ\ M9.0}$ = 22.362205 inches per second
   - $PGD_{SFZ\ M7.0}$ = 27.19742 inches
   - $PGD_{CSZ\ M9.0}$ = 24.28114 inches
3. Calculate the repair rate associated with seismic wave propagation (RR_{PGV}):

$$RR_{PGV} = K_1 \times 0.00187 \times PGV \qquad \textbf{\textit{Eq. 5-1}}$$

$$RR_{PGV,SFZ\ M7.0} = 1.00 \times 0.00187 \times 57.011811 = 0.106612$$

$$RR_{PGV,CSZ\ M9.0} = 1.00 \times 0.00187 \times 22.362205 = 0.041817$$

4. Calculate the repair rate associated with permanent ground deformation (RR_{PGD}):

$$RR_{PGD} = K_2 \times 1.06 \times (probability\ of\ liquefaction) \times PGD^{0.319}$$

$$RR_{PGD,SFZ\ M7.0} = 1.00 \times 1.06 \times 0.940588 \times 27.19742^{0.319} = 2.859695 \text{ repairs/1000 ft}$$

$$RR_{PGD,CSZ\ M9.0} = 1.00 \times 1.06 \times 0.886268 \times 24.28114^{0.319} = 2.598793 \text{ repairs/1000 ft}$$

5. Calculate the combined PGV-based and PGD-based repair rates:

$$RR_{weighted} = 0.2 \times RR_{PGV} + 0.8 \times RR_{PGD} \qquad \textbf{\textit{Eq. 5-3}}$$

$$RR_{weighted,SFZ\ M7.0} = 0.2 \times 0.106612 + 0.8 \times 2.859695 = 2.309079$$

$$RR_{weighted,CSZ\ M9.0} = 0.2 \times 0.041817 + 0.8 \times 2.598793 = 2.087398$$

6. Calculate combined repair rates based on the results for both earthquake scenarios using the estimated return periods for each earthquake:

$$RR_{combined} = \frac{\frac{1}{5{,}000}\left(RR_{weighted,SFZ\ M7.0}\right) + \frac{1}{500}\left(RR_{weighted,CSZ\ M9.0}\right)}{\frac{1}{5{,}000} + \frac{1}{500}}$$

Eq. 5-4

$$RR_{combined} = \frac{\frac{1}{5{,}000}(2.309079) + \frac{1}{500}(2.087398)}{\frac{1}{5{,}000} + \frac{1}{500}} = 2.107551 \qquad \text{repairs/1000 ft}$$

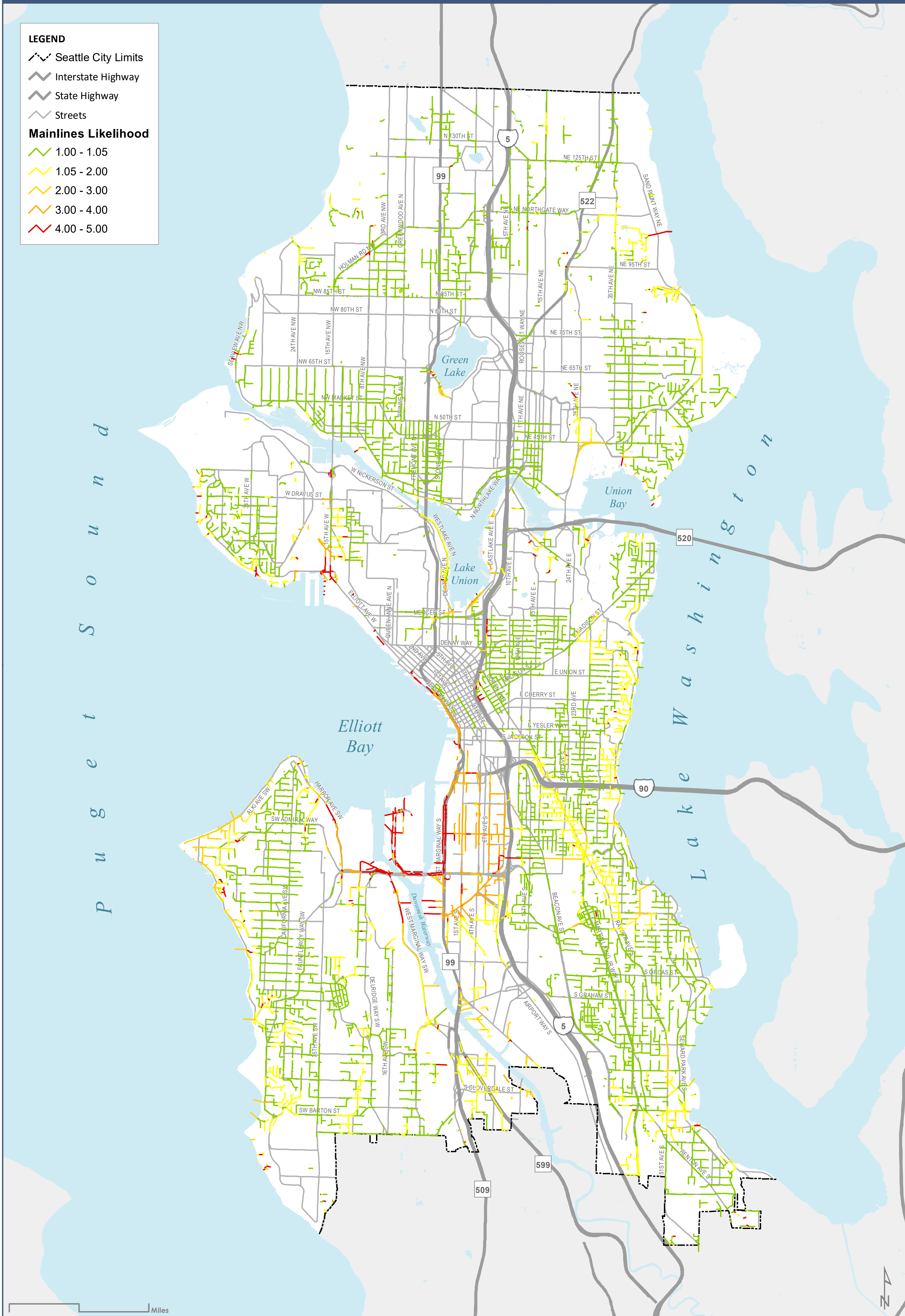
7. Use the combined repair rate to determine likelihood score from Table 5-2 (linear interpolation):

$$Likelihood\ Score = \frac{(5-4)}{(2.50-2.00)}(2.107551 - 2.00) + 4 = 4.215102$$

Appendix G: Likelihood of Failure Maps for Mainlines

Shape Our Water | Seismic Risk Assessment

Map G-1: Likelihood of Failure for Drainage Mainlines

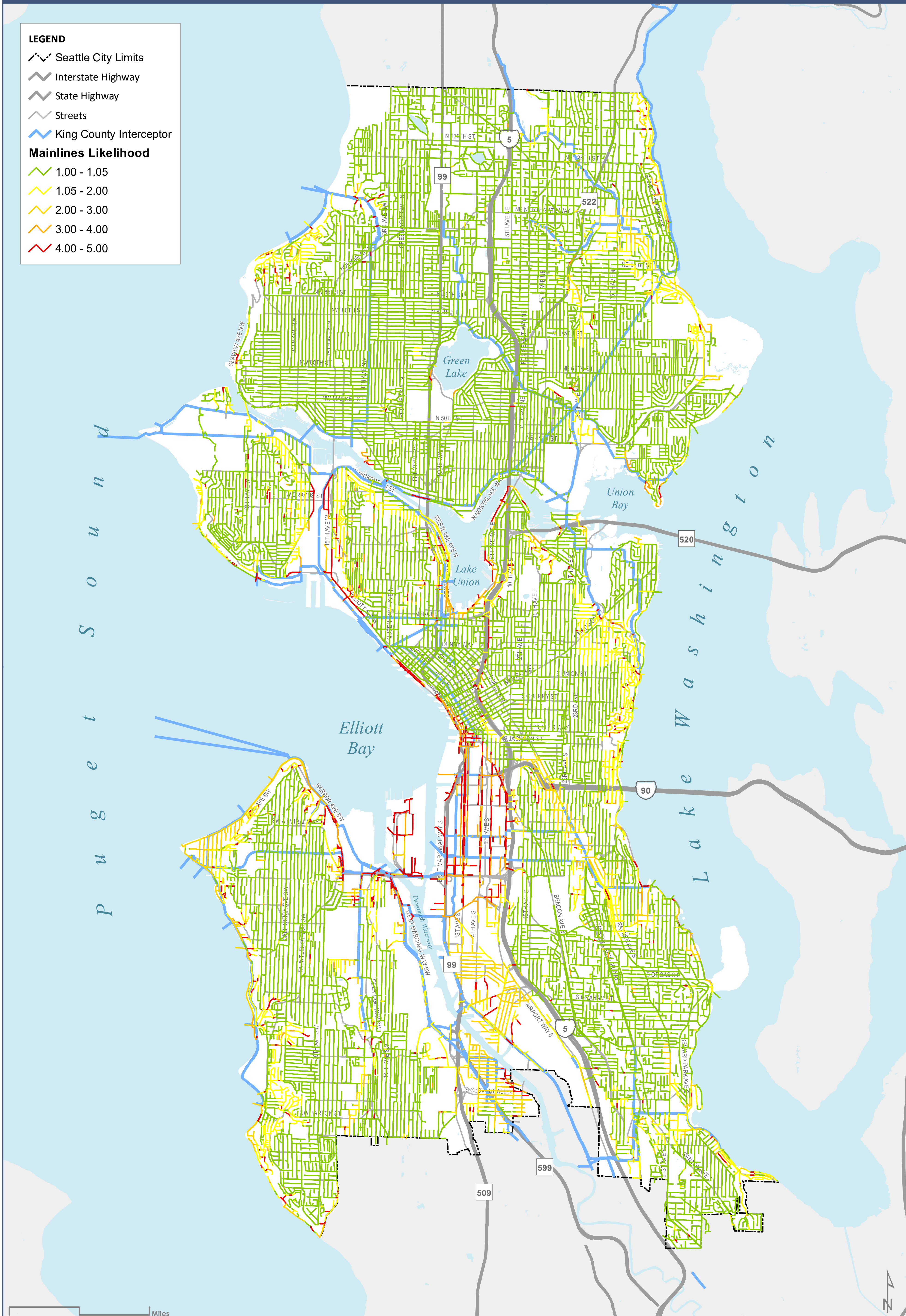


Author: Fill out Map Document Properties Date: 1/7/2022 File Path: \\bc\corp\PW_Exports\154002 - SPU Shape Our Water\Task04\GIS\MapDocs\SOW_T4_MapG1(rev).mxd

Seattle Public Utilities

Shape Our Water | Seismic Risk Assessment

Map G-2: Likelihood of Failure for Wastewater Mainlines



Author: Fill out Map Document Properties Date: 1/7/2022 File Path: \\bc\corp\PW_Exports\154002 - SPU Shape Our Water\Task04\GIS\MapDocs\SOW_T4_MapG2(rev).mxd

Appendix H: Example Calculation for Pump Station Likelihood of Failure Score

Example Calculation: Wastewater Pump Station 73

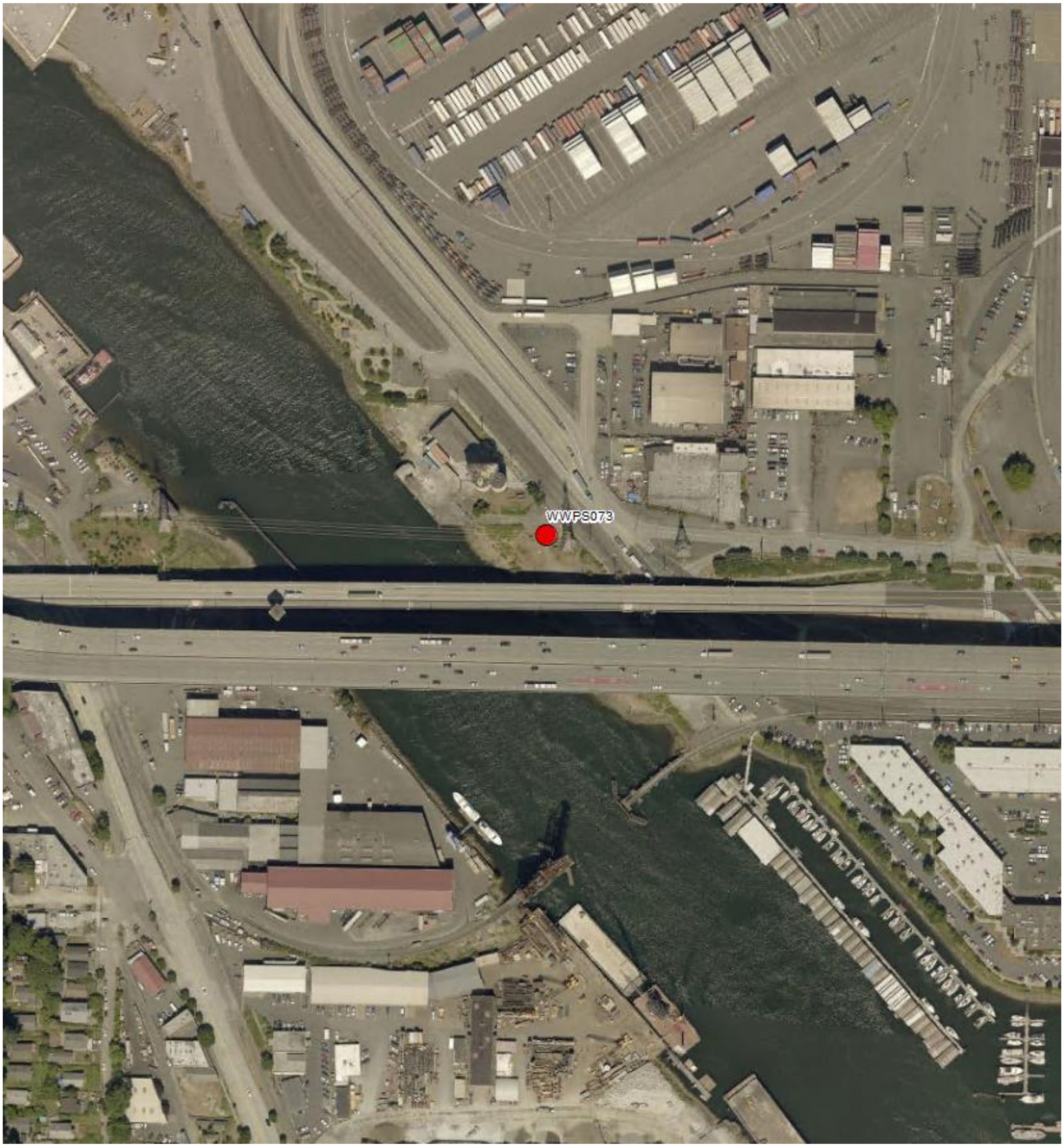


Figure H-1. Location of Pump Station 73 for example likelihood calculation

Geotechnical hazard data for Pump Station 73 are provided in Table H-1.

Table H-1. Geotechnical Hazard Data for Pump Station 73 Example

Seismic Event	Permanent Ground Deformation (PGD), inches	0.1s Spectral Acceleration ($SA_{0.1}$), g	0.3s Spectral Acceleration ($SA_{0.3}$), g	1s Spectral Acceleration (SA_{1}), g
M7.0 Seattle Fault Zone	105.28	1.01	1.44	1.26
M9.0 Cascadia Subduction Zone	96.66	0.34	0.51	0.64

- **General Description.** This is a square 23 feet by 22.33 feet x 22 feet high with center wall for wet well, floor is 6-feet thick with no rebar. Middle wall is attached to tank walls and deeply embedded in floor. It is not attached at roof except for a key.
- **Structural Assessment.** The floor does not have rebar. The walls have 20 feet spans. All walls have hooks on outer face but not on inner face. Roof is not attached to wall. Center wall only has one hook between wet well wall and tanks wall. Walls have stirrups near the wall intersections. Roof may move relative to structure below. Structure may crack at corners. The roof could displace from the structure. Since this pumpstation is in a liquefaction zone it could cause the pumpstation to become inoperable.
- **Mainline Connection Assessment.** The discharge mainline is a 12-inch CIP. It is rigidly attached to the wall. The intake mainline is a 24-inch CIP mainline. It is rigidly attached to the wall but has a bell just outside the wall to allow flexibility.

The project team examined structural and mechanical as-built drawings for Pump Station 73 (see Figure H-2).

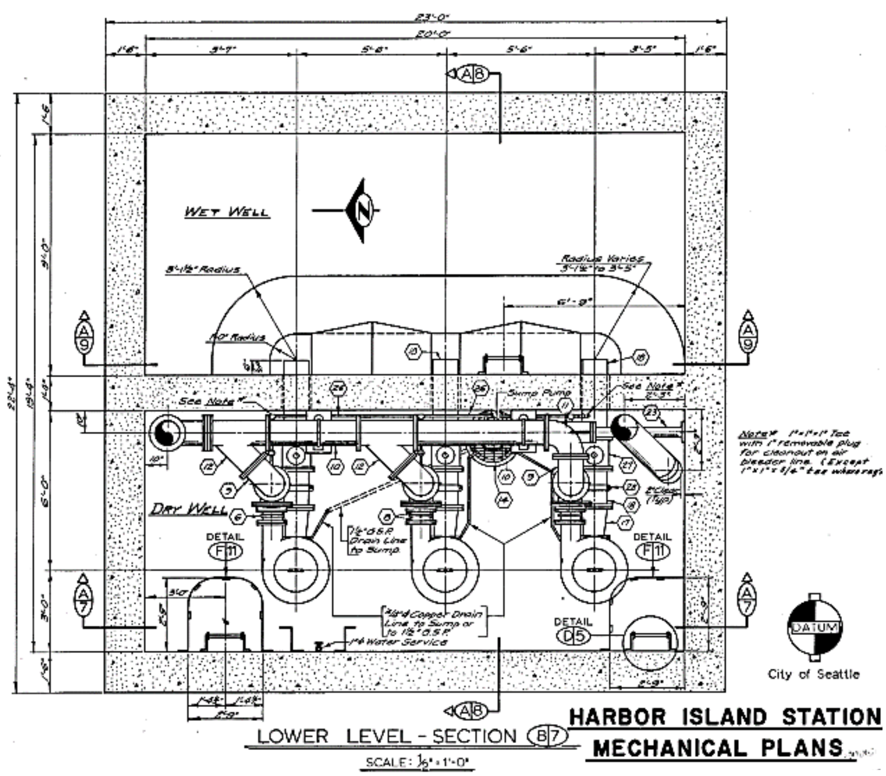


Figure H-2. Example detail from as-built drawings for Pump Station 73

The following are steps in the engineering assessment:

1. Pump Station 73 is outside of mapped landslide zones; assign a score of 1.
2. Determine that pump station is located in a “high” zone for liquefaction (see PGD values in Table H-1). Then calculate floatation safety factor (SF) based on ratio of resisting forces to buoyancy forces (liquefaction zones). For Pump Station 73, SF = 1.13, which is greater than one; assign a score of 1.
3. Perform a desktop assessment for structural failure during a M7.0 SFZ earthquake, and based on engineer’s assessment assign score of 5.
4. Perform a desktop assessment for structural failure during a M9.0 CSZ earthquake, and based on engineer’s assessment assign score of 5.
5. Perform a desktop assessment for failure at mainline connections, and based on engineer’s assessment assign a score of 5.
6. Pump Station 73 is located within a tsunami inundation zone, assign score of 5.
7. Pump Station 73 is not located in a seiche inundation zone; assign score of 1.
8. Take the highest component score from all of the above assessments and assign an overall likelihood of failure of 5.

Appendix I: Likelihood of Failure Data for Wastewater Pump Stations

Map I-1. Likelihood of Failure for Wastewater Pump Stations

Table I-1. Pump Station Geotechnical Hazard Data

Pump Station Number	M7.0 SFZ				M9.0 CSZ			
	Permanent Ground Deformation *PGD* (in)	0.1s Spectral Acceleration $SA_{0.1}$ (g)	0.3s Spectral Acceleration $SA_{0.3}$ (g)	1s Spectral Acceleration $SA_{1.0}$ (g)	Permanent Ground Deformation *PGD* (in)	0.1s Spectral Acceleration $SA_{0.1}$ (g)	0.3s Spectral Acceleration $SA_{0.3}$ (g)	1s Spectral Acceleration $SA_{1.0}$ (g)
WWPS001	0.00	0.76	0.94	0.60	0.00	0.39	0.51	0.56
WWPS002	0.00	1.35	1.64	1.05	0.00	0.35	0.45	0.48
WWPS004	6.30	1.10	1.44	0.62	5.37	0.35	0.46	0.26
WWPS005	0.00	1.11	1.58	0.82	0.00	0.34	0.47	0.28
WWPS006	0.00	1.07	1.50	0.82	0.00	0.33	0.47	0.29
WWPS007	0.00	0.89	1.17	0.84	0.00	0.35	0.46	0.52
WWPS009	9.50	1.03	1.40	0.70	7.75	0.34	0.47	0.28
WWPS010	0.00	1.17	1.48	0.62	0.00	0.35	0.45	0.24
WWPS011	7.42	0.64	0.71	0.41	9.47	0.36	0.43	0.42
WWPS013	10.87	0.69	0.97	0.85	12.19	0.33	0.49	0.62
WWPS017	6.18	1.06	1.29	0.53	5.37	0.35	0.46	0.25
WWPS018	0.00	0.97	1.28	0.59	0.00	0.33	0.46	0.27
WWPS019	0.00	1.09	1.40	0.60	0.00	0.35	0.46	0.26
WWPS020	0.00	0.77	1.05	0.76	0.00	0.35	0.48	0.54
WWPS021	11.18	0.90	1.11	0.76	10.65	0.38	0.49	0.54
WWPS022	0.00	0.73	0.94	0.66	0.00	0.38	0.52	0.60
WWPS025	0.00	0.85	1.04	0.72	0.00	0.35	0.46	0.52
WWPS028	0.00	0.65	0.74	0.42	0.00	0.39	0.49	0.50
WWPS030	0.00	0.62	0.69	0.41	0.00	0.39	0.49	0.50
WWPS031	0.00	0.54	0.57	0.31	0.00	0.38	0.46	0.44
WWPS035	4.85	0.71	0.92	0.62	5.38	0.35	0.47	0.52
WWPS036	14.09	1.31	1.61	1.03	10.97	0.38	0.49	0.52
WWPS037	26.31	1.28	1.56	1.00	20.48	0.38	0.48	0.52
WWPS038	52.73	1.27	1.53	1.00	42.76	0.39	0.49	0.52
WWPS039	0.00	1.22	1.41	0.54	0.00	0.39	0.49	0.25
WWPS042	5.45	1.06	1.26	0.52	4.46	0.38	0.49	0.27
WWPS043	0.00	0.68	0.92	0.70	0.00	0.37	0.52	0.62
WWPS044	6.36	1.19	1.36	0.51	5.32	0.36	0.45	0.22
WWPS045	6.34	1.13	1.35	0.54	5.38	0.36	0.46	0.24

Table I-1. Pump Station Geotechnical Hazard Data (continued)								
Pump Station Number	M7.0 SFZ				M9.0 CSZ			
	Permanent Ground Deformation *PGD* (in)	0.1s Spectral Acceleration $SA_{0.1}$ (g)	0.3s Spectral Acceleration $SA_{0.3}$ (g)	1s Spectral Acceleration $SA_{1.0}$ (g)	Permanent Ground Deformation *PGD* (in)	0.1s Spectral Acceleration $SA_{0.1}$ (g)	0.3s Spectral Acceleration $SA_{0.3}$ (g)	1s Spectral Acceleration $SA_{1.0}$ (g)
WWPS046	0.00	0.68	0.88	0.62	0.00	0.38	0.51	0.60
WWPS047	0.00	0.72	0.82	0.48	0.00	0.39	0.49	0.50
WWPS048	0.00	0.75	1.01	0.72	0.00	0.35	0.48	0.54
WWPS049	0.00	0.76	0.99	0.72	0.00	0.35	0.48	0.56
WWPS050	8.70	0.76	1.06	0.84	8.76	0.34	0.48	0.56
WWPS051	0.00	0.63	0.84	0.57	0.00	0.34	0.45	0.52
WWPS053	0.00	1.27	1.49	0.59	0.00	0.39	0.49	0.24
WWPS054	12.17	0.75	1.00	0.72	13.49	0.37	0.50	0.58
WWPS055	15.62	0.70	0.99	0.81	16.65	0.33	0.48	0.58
WWPS056	0.00	0.59	0.65	0.37	0.00	0.38	0.48	0.46
WWPS057	0.00	0.81	1.02	0.70	0.00	0.36	0.48	0.52
WWPS058	0.00	0.80	1.06	0.75	0.00	0.36	0.48	0.54
WWPS059	0.00	0.83	1.09	0.75	0.00	0.37	0.48	0.54
WWPS060	0.00	0.92	1.13	0.75	0.00	0.37	0.47	0.50
WWPS061	18.71	0.93	1.22	0.88	17.33	0.37	0.48	0.54
WWPS062	16.14	1.03	1.13	0.64	14.04	0.38	0.44	0.42
WWPS063	13.70	0.98	1.08	0.62	12.04	0.37	0.45	0.42
WWPS064	0.00	0.88	1.08	0.72	0.00	0.37	0.47	0.50
WWPS065	27.08	0.77	1.03	0.76	28.10	0.35	0.48	0.56
WWPS066	0.00	0.76	1.03	0.76	0.00	0.35	0.48	0.56
WWPS067	0.00	0.77	1.05	0.76	0.00	0.35	0.48	0.54
WWPS069	0.00	0.65	0.79	0.51	0.00	0.35	0.44	0.48
WWPS070	25.60	1.04	1.22	0.51	22.58	0.38	0.49	0.27
WWPS071	28.91	1.05	1.11	0.38	25.93	0.40	0.47	0.22
WWPS072	11.73	0.78	1.15	1.37	10.86	0.31	0.51	0.76
WWPS073	105.28	1.01	1.44	1.26	96.66	0.34	0.51	0.64
WWPS074	16.35	1.11	1.47	1.16	14.44	0.36	0.50	0.60
WWPS075	13.94	0.83	1.21	0.88	12.72	0.33	0.53	0.39

Table I-1. Pump Station Geotechnical Hazard Data (continued)

Pump Station Number	M7.0 SFZ				M9.0 CSZ			
	Permanent Ground Deformation *PGD* (in)	0.1s Spectral Acceleration $SA_{0.1}$ (g)	0.3s Spectral Acceleration $SA_{0.3}$ (g)	1s Spectral Acceleration $SA_{1.0}$ (g)	Permanent Ground Deformation *PGD* (in)	0.1s Spectral Acceleration $SA_{0.1}$ (g)	0.3s Spectral Acceleration $SA_{0.3}$ (g)	1s Spectral Acceleration $SA_{1.0}$ (g)
WWPS076	35.13	1.10	1.35	0.58	30.59	0.38	0.50	0.27
WWPS077	0.00	0.94	1.24	0.88	0.00	0.38	0.51	0.56
WWPS078	6.26	1.04	1.32	0.61	5.40	0.34	0.47	0.28
WWPS080	0.00	1.06	1.32	0.55	0.00	0.34	0.45	0.24
WWPS081	0.00	1.08	1.26	0.48	0.00	0.35	0.43	0.22
WWPS082	0.00	1.01	1.09	0.38	0.00	0.39	0.47	0.22
WWPS083	3.68	0.77	1.03	0.75	4.20	0.37	0.50	0.58
WWPS084	0.00	0.70	0.94	0.67	0.00	0.38	0.51	0.60
WWPS114	3.30	0.51	0.68	0.47	5.48	0.35	0.47	0.54
WWPS118	0.53	0.52	0.70	0.51	0.95	0.36	0.50	0.60

Table I-2. Summary of Pump Station PGD Information						
Pump Station Number	**Date of Construction**	**General Description**	**Landslide Zone**	**Liquefaction Zone**	**PGD, inches**	
					M7.0 SFZ	**M9.0 CSZ**
WWPS001	1978	There are three precast round tanks made of 10' ID manhole rings. One tank has equipment (dry well) and the others are wet wells.	No	No	0.00	0.00
WWPS002	1929	This is a cast-in-place round storage tank, 20' ID with a concrete dividing the tank in half.	Yes	No	0.00	0.00
WWPS004	1979	There are two round cast-in-place tanks. One is 18' high and the second is 8' diameter by 18' high.	No	Yes	6.30	5.37
WWPS005	1930	This is a cast-in-place round storage tank, 20' ID with a concrete dividing the tank in half.	No	No	0.00	0.00
WWPS006	1930	This is a 13'-6" x 16'- 8" x 15'-3" High cast-in-place concrete vault.	No	No	0.00	0.00
WWPS007	1932	This is a cast-in-place round storage tank, 26' ID with a concrete dividing the tank in half.	No	No	0.00	0.00
WWPS009	1933	This is a cast-in-place round storage tank, 22' ID with a concrete dividing the tank in half.	No	Yes	9.50	7.75
WWPS010	1933	This is a cast-in-place round storage tank, 34' ID with a concrete dividing the tank in half.	No	No	0.00	0.00
WWPS011	1998	Pump station is a concrete mainline.	No	Yes	7.42	9.47
WWPS013	1935	This is a cast-in-place round storage tank, 18' ID with a concrete dividing the tank in half.	No	Yes	10.87	12.19
WWPS017	1975	This is a cast-in-place pump station 24.5' x 23' x 28' high with a dividing wall for the wet well.	No	Yes	6.18	5.37
WWPS018	1987	There are two precast structures. The square structure is 8.33' x 4.33' x 7' high. The round structure is 72" ID x 17' high.	No	No	0.00	0.00
WWPS019	1987	There are two precast round tanks 6' diameter, 15' high.	No	No	0.00	0.00
WWPS020	1940	This is a cast-in-place round storage tank, 18' ID with a concrete dividing the tank in half.	No	No	0.00	0.00
WWPS021	1988	There are two precast round tanks 6' diameter, 22' high.	Yes	Yes	11.18	10.65
WWPS022	1952	This is a square cast-in-place 14 x 20 x 24' deep buried concrete structure.	No	No	0.00	0.00
WWPS025	1952	This is a cast-in-place round storage tank, 16' ID with a concrete dividing the tank in half.	No	No	0.00	0.00
WWPS028	1975	This pump station consists of one 6'x6' square cast-in-place structure and two 4' diameter precast manholes.	Yes	No	0.00	0.00

Table I-2. Summary of Pump Station PGD Information (continued)						
Pump Station Number	Date of Construction	General Description	Landslide Zone	Liquefaction Zone	PGD, inches	
					M7.0 SFZ	M9.0 CSZ
WWPS030	1990	This pump station has a square ~6'x4' square control chamber and a round ~8' diameter structure.	Yes	No	0.00	0.00
WWPS031	1987	This is a 6' diameter x 22' high precast manhole.	No	No	0.00	0.00
WWPS035	1955	This is a cast-in-place round storage tank, 17.33' ID with a concrete dividing the tank in half. The middle wall only extends 7.8' above floor	No	Yes	4.85	5.38
WWPS036	1957	This is a square cast-in-place 13 x 18 x 17 ft deep buried concrete structure. It has a square sump for the wet well.	No	Yes	14.09	10.97
WWPS037	1957	This is a square cast-in-place 46 x 18.67 x 25 ft deep concrete structure. It has a square wet well and an overflow chamber.	Yes	Yes	26.31	20.48
WWPS038	1959	This is a square cast-in-place 14 x 19.75 x 19 ft deep concrete structure. It has a square overflow chamber.	No	Yes	52.73	42.76
WWPS039	1957	This is a square cast-in-place 17.16 x 23.16 x 21 ft deep concrete structure. It has a square overflow chamber.	No	No	0.00	0.00
WWPS042	1957	This is a square cast-in-place 10 ft by 18.5 ft by 15.66 ft deep concrete structure. It has a square overflow chamber.	Yes	Yes	5.45	4.46
WWPS043	1957	This is a square cast-in-place 18.66 ft by 33.33 ft by 34.5 ft deep concrete structure. It has a square wet well and overflow chamber.	No	No	0.00	0.00
WWPS044	1953	This is a cast-in-place round storage tank, 12-ft ID with a concrete wall dividing the tank in half for a wet well. The middle wall extends the full height of the tank.	No	Yes	6.36	5.32
WWPS045	1967	This is a square cast-in-place 12 ft x 12 ft x 18.5 ft deep concrete structure. It has a square wet well.	No	Yes	6.34	5.38
WWPS046	1959	This is a cast-in-place round storage tank, 14-ft ID with a concrete tank.	No	No	0.00	0.00
WWPS047	1590	Pump station is a 12-ft diameter cast-in-place structure.	No	No	0.00	0.00
WWPS048	1973	This is a square cast-in-place 18' by 18' by 22' deep concrete structure. It has a square wet well.	No	No	0.00	0.00

Table I-2. Summary of Pump Station PGD Information (continued)

Pump Station Number	Date of Construction	General Description	Landslide Zone	Liquefaction Zone	PGD, inches	
					M7.0 SFZ	M9.0 CSZ
WWPS049	1960	This is a square 12 ft by 10 ft structure with wet well by 14.25 ft high with 2-feet-8-inches-thick floor	No	No	0.00	0.00
WWPS050	1961	This is a square 11 ft by 11 ft structure by 16.66' HIGH with 1'-1" Thick floor, no wet well	No	Yes	8.70	8.76
WWPS051	1961	Round tank with wet well. 14' OD and 19' high. Floor is 4 ft thick	No	No	0.00	0.00
WWPS053	1961	This is a 6 ft diameter manhole rings	No	No	0.00	0.00
WWPS054	1963	Pumpstation is square 17.5 by 13.5 ft by 32 ft high with center wall for wet well, floor is 4.75 ft thick with no rebar.	No	Yes	12.17	13.49
WWPS055	1963	Pumpstation is a Round tank. 12.33 ft OD and 22.916 ft high. Floor is 1 ft thick reinforced with 1.5 ft of tremie concrete below floor.	No	Yes	15.62	16.65
WWPS056	1984	Structure is precast mainline wall. No wet well. Structural drawings not provided. Diameter of structure was not provided. Roof not connected to structure.	Yes	No	0.00	0.00
WWPS057	1964	Pumpstation is a square 14.33 x 14.33' x 21.83' H with center wall for wet well, floor is 5 ft thick with no rebar.	No	No	0.00	0.00
WWPS058	1964	Pumpstation is a square 14.33 x 14.33' x 19.83' H with center wall for wet well, floor is 5 ft thick with no rebar.	No	No	0.00	0.00
WWPS059	1964	Pumpstation is a square 14.33 x 14.33' x 20.83' H with center wall for wet well, floor is 5 ft thick with no rebar.	Yes	No	0.00	0.00
WWPS060	1964	Pumpstation is a square 14.33 x 14.33' x 20.33' H with center wall for wet well, floor is 5 ft thick with no rebar.	Yes	No	0.00	0.00
WWPS061	1964	Pumpstation is a square 14.33 x 14.33' x 17.83' H with center wall for wet well, floor is 5 ft thick with no rebar.	No	Yes	18.71	17.33
WWPS062	1964	Pumpstation is square 14.33 x 14.33' x 20' H with center wall for wet well, floor is 5 ft thick with no rebar.	No	Yes	16.14	14.04
WWPS063	1964	Pumpstation is 14.33 x 14.33' x 21.83' H with center wall for wet well, floor is 5 ft thick with no rebar.	No	Yes	13.70	12.04

Table I-2. Summary of Pump Station PGD Information (continued)

Pump Station Number	Date of Construction	General Description	Landslide Zone	Liquefaction Zone	PGD, inches	
					M7.0 SFZ	M9.0 CSZ
WWPS064	1964	Pumpstation is a square 14.33 x 14.33' x 13.5' H with center wall for wet well, floor is 4 ft thick with no rebar.	No	No	0.00	0.00
WWPS065	1964	Pumpstation is a square 14.33 x 14.33' x 19.3' H with center wall for wet well, floor is 4 ft thick with no rebar.	No	Yes	27.08	28.10
WWPS066	1964	Pumpstation is a square 14.33 x 14.33' x 19.3' H with center wall for wet well, floor is 4 ft thick with no rebar.	No	No	0.00	0.00
WWPS067	1964	Pumpstations is a square 14.33 x 14.33' x 13.3' H with center wall for wet well, floor is 4 ft thick with no rebar.	No	No	0.00	0.00
WWPS069	1964	Pumpstation is square 14.33 x 14.33' x 14.24' H with center wall for wet well, floor is 4 ft thick with no rebar.	No	No	0.00	0.00
WWPS070	1963	Pumpstation is square 14.5 x 14.5' x 37' H with center wall for wet well, floor is 4 ft thick with no rebar.	No	Yes	25.60	22.58
WWPS071	1963	Tank: round 13' dia x 20' high cast-in-place Concrete structure. Weir: two structures 12 x 10 x 7' high square and 13 dia x 20' high round made of mainline.	No	Yes	28.91	25.93
WWPS072	1965	This is a square 23 x 22.33' x 25.66' H with center wall for wet well, floor is 6 ft thick with no rebar. Middle wall is attached to tank walls and deeply embedded in floor. It is not attached at roof except for a key.	No	Yes	11.73	10.86
WWPS073	1965	This is a square 23 x 22.33' x 22' H with center wall for wet well, floor is 6 ft thick with no rebar. Middle wall is attached to tank walls and deeply embedded in floor. It is not attached at roof except for a key.	No	Yes	105.28	96.66
WWPS074	1966	This is a square 23 x 22.33' x 22' H with center wall for wet well, floor is 9 ft thick with no rebar. Middle wall is attached to tank walls and deeply embedded in floor. It is not attached at roof except for a key.	No	Yes	16.35	14.44
WWPS075	1966	This is a round tank . 8.16' OD and 24' high. Roof not connected to tank. Floor is 4.83 ft thick with no reinforcement Floor not connected to tank walls	No	Yes	13.94	12.72

Table I-2. Summary of Pump Station PGD Information (continued)						
Pump Station Number	**Date of Construction**	**General Description**	**Landslide Zone**	**Liquefaction Zone**	**PGD, inches**	
					M7.0 SFZ	**M9.0 CSZ**
WWPS076	1965	This is a square 13.5 x 14.5' x 21.25' H with center wall for wet well, floor thickness could not be determined. Floor has no rebar. Middle wall is attached to tank walls. Attachment to floor could not be determined. Attached to roof with key only.	Yes	Yes	35.13	30.59
WWPS077	1970	This is a square 18 x 18' x 39' H with center wall for wet well, floor is 5.5 ft thick with no rebar. Middle wall attached to tank floor is not shown. Wet wall walls are attached to tank wall by rebar with two hooks. It is not attached at roof except for a key.	No	No	0.00	0.00
WWPS078	1970	Structure is precast circular mainline structure with no wet well.	No	Yes	6.26	5.40
WWPS080	1967	This pumpstation is 7.16' OD x 22' H made from manhole rings	No	No	0.00	0.00
WWPS081	1968	This is a square 14.33 x 14.33' x 16.416' H with center wall for wet well, floor is 3.5 ft thick with no rebar.	No	No	0.00	0.00
WWPS082	1972	This pumpstation is 7.16' OD x 20' H made with manhole rings	Yes	No	0.00	0.00
WWPS083	1972	This is a 7.16' OD manhole rings x 20' H made with manhole rings	No	Yes	3.68	4.20
WWPS084	1973	This is a square 14.33 x 14.33' x 18.5' H with center wall for wet well, floor is 6.25 ft thick with no rebar.	No	No	0.00	0.00
WWPS114	1953	This is a 16.75x 9 x 12H cast-in-place structure	No	Yes	3.30	5.48
WWPS118	1956	This is a 8.66 x 8.66' x 16.66' H cast-in-place square pumpstation with a round wet well that is immediately adjacent	No	Yes	0.53	0.95

Table I-3. Pump Station Vulnerability Assessment		
Pump Station Number	**Structural**	**Mainline Connection**
WWPS001	The structures are 10' diameter manhole rings with T&G "O" ring joints. The figure shows the manhole sections to be about 8 foot long. Considering the type of construction and diameter it is likely the manhole rings will crack in the transverse direction due to tension, but the structure will remain functional.	There are rigid 4" and 8" DIP that are connected to both the dry well & wet well and another set of mainlines that are connected between the two wet wells. These mainlines are embedded into the wall of the structures. These mainlines could break due to shaking.
WWPS002	The structure has only one layer of rebar on the inner face of the perimeter walls and one layer of rebar on the tension face of the middle wall. This structure will perform poorly due to its lack of ductility in the outer and inner walls. The tank walls are susceptible to cracking and leaking due to the lack of ductility.	The flow into the PS comes from a 15" mainline. It is rigidly attached to the wall. In most cases there appears to be a bell and spigot joint just outside the wall of the structure. The discharge mainline is a 10" CIP that has a bell and spigot joint. There is an immediate 90-degree bend and joint just outside the structure.
WWPS004	The structures are made with two layers of reinforcement. The amount of longitudinal reinforcing is not indicated. Considering the type of construction and diameter it is likely the manhole rings will crack in the transverse direction due to tension.	There is a 6" DI mainline that is rigidly connected between the two tanks. This mainline could be cracked or broken during the design earthquake.
WWPS005	The structure has only one layer of rebar on the inner face of the perimeter walls and one layer of rebar on the tension face of the middle wall. This structure will perform poorly due to its lack of ductility in the outer and inner walls. The tank walls are susceptible to cracking and leaking due to the lack of ductility.	The flow into the PS comes from a 21" mainline. It is rigidly attached to the wall and could be cracked or broken during the design earthquake. The discharge mainline is a series of cast iron fittings connecting two 10" diameter discharge mainlines at a wye. The discharge mainlines are rigidly attached to the wall. The discharge mainline would have some flexibility.
WWPS006	The structure has only one layer of rebar on the inner face of the perimeter walls. The rebar does not have hooks at the corner. One wall is adjacent to a stairwell. The stairwell will prevent one tank wall from transferring it load to the soil. The structure does not have a clear load path to transfer load to the soil. The structure lacks ductility and a clear load path.	The intake mainline is a cast iron mainline that is rigidly attached to both the pumpstation and the adjacent sump structure. The cast iron discharge mainline exits the pump station at a 45-degree angle to the wall. It is rigidly attached to the pumpstation wall.
WWPS007	The structure has only two layers of rebar on the perimeter walls and one layer of rebar on the tension face of the middle wall. Roof only has one layer of reinforcing. This structure will perform poorly due to the lack of ductility of the inner wall and roof. The tank walls are susceptible to cracking and leaking due to the lack of ductility.	The flow into the PS comes from a 18" mainline. It is rigidly attached to the wall and could be cracked or broken during the design earthquake. There is a 12" and 16" CI discharge mainline. They are cast into the wall. The 16" CI has a bell connection just outside the structure.

Table I-3. Pump Station Vulnerability Assessment (continued)		
Pump Station Number	**Structural**	**Mainline Connection**
WWPS009	The structure has only two layers of rebar on the perimeter walls and one layer of rebar on the tension face of the middle wall. This structure could perform poorly due to its lack of ductility in the middle wall. The tank walls are susceptible to damage due to liquefaction and lack of ductility due to rebar splice lengths.	The intake mainline is a 21" pipe. It is cast into the wall and has a bell and spigot fitting adjacent to the structure. The 21" mainline has a bell connection just outside the structure. The discharge mainline is a 10" pipe that is rigidly attached to the wall and an adjacent structure. It could be cracked or broken during the design earthquake due to differential movement between the structures.
WWPS010	The structure has only two layers of rebar on the perimeter walls and one layer of rebar on the tension face of the middle wall. Roof only has one layer of reinforcing. This structure will perform poorly due to the lack of ductility of the inner wall and roof. The tank walls are susceptible to cracking and leaking due to the lack of ductility.	The intake mainline is a 15" pipe. It is cast into the wall and has a bell and spigot fitting adjacent to the structure. The 15" mainline has a bell connection just outside the structure. The discharge mainline is an 8" CI pipe that is rigidly attached to the wall and an adjacent structure. It could be cracked or broken during the design earthquake due to differential movement between the structures.
WWPS011	The structure is a concrete mainline. The roof is not attached. Since it is in a liquefaction zone the joints could pull part and roof be displaced. Floatation could not be determined due to inability to find dimensions of pumpstation.	There is a 12-inch dimeter intake mainline and two 6-inch diameter discharge mainlines. The discharge mainlines have a coupling just outside the wall of the pumpstation. The mainlines then enter a pump control structure. There is only a couple of feet between these structures. The liquefaction movement is more than the piping is designed to resist.
WWPS013	The structure has only one layer of rebar on the inner face of the perimeter walls and one layer of rebar on the tension face of the middle wall. This structure will perform poorly due to its lack of ductility in the outer and inner walls. The tank walls are susceptible to cracking and leaking due to the lack of ductility.	The intake mainline is a 10" pipe. It is cast into the wall. The discharge mainline is a 10" CIP with flanged joints. The discharge mainline makes a 90-degree bend just outside the structure.
WWPS017	The walls bars have hooks on outer face but not on inner face. Roof is attached to wall only with a concrete key. Center wall between wet well wall has two layers of reinforcement with two hooks between wet well wall and tanks wall. Structure has adequate ductility, but the roof may move relative to the structure.	Intake piping is a 30" mainline. There is a coupling outside the wall which may provide some flexibility. The 10" discharge mainline has a coupling outside the wall which will provide some flexibility.
WWPS018	Both structures are precast manhole type structures. Considering the type of construction and diameter it is likely the manhole rings will crack in the transverse direction due to tension.	The intake piping is an 8" with a coupling outside the wall. The discharge piping is two 2" diameter mainlines which connects the two structures. These mainlines have two flexible couplings on each mainline between the structures. The final discharge mainline is a 3" pipe with a flexible coupling.

Table I-3. Pump Station Vulnerability Assessment (continued)

Pump Station Number	Structural	Mainline Connection
WWPS019	Both structures are precast manhole type structures. Considering the type of construction and diameter it is likely the manhole rings will crack in the transverse direction due to tension.	The intake piping is an 6" DIP. There may be a coupling outside the wall however it is not clear. The discharge piping is two 2" diameter mainlines which connects the two structures. These mainlines have two flexible couplings on each mainline between the structures. The final discharge mainline is a 3" pipe with a PVC adaptor. The PVC piping and this connection may not perform well.
WWPS020	The structure has only two layers of rebar on the perimeter walls and one layer of rebar on the tension face of the middle wall. This structure will perform poorly due to its lack of ductility in the inner wall. The tank walls do not have adequate splice lengths. The lap joint shown where the tanks outer wall thickness changes make this area susceptible to cracking and reduced ductility.	The intake mainline is a 10" CI pipe that is rigidly attached to two structures. This mainline is susceptible to damage due to differential movement between the structures. The discharge mainline is an 8" CI pipe that is rigidly attached to the wall and has an immediate 90-degree bend. It could be cracked or broken during the design earthquake.
WWPS021	Both structures are precast manhole type structures. Considering the type of construction and diameter it is likely the manhole rings will crack in the transverse direction due to tension.	The intake piping is an 8" DIP with a coupling outside the wall. The discharge piping is two 2" diameter mainlines. These mainlines have two flexible couplings on each mainline.
WWPS022	All walls have two layers of reinforcement. All walls have hooks, but the inner layer rebar hook is embedded improperly and will be ineffective. This rebar placement may damage the wall of the structure. Roof is attached to wall with unhooked rebar. Center wall only has one ineffective hook and one straight bar with inadequate development length. Structure does not have good ductility.	The intake piping is an 16" CI mainline that is embedded in wall of the structure. The discharge mainline is an 8" diameter pipe that makes a 90-degree bend outside the structure and uses a thrust block to resist the thrust. The mainline can pull apart or be damaged by differential movement between the thrust block and vault.
WWPS025	The structure has only two layers of rebar on the perimeter walls. The rebar in the middle wall is designed for the beam to be a moment connection at the tank wall but the rebar only has a short embedment at the tank wall. This wall is improperly designed. This structure will perform poorly due to its lack of ductility in the inner wall.	The intake mainline is a 14" CI pipe that is rigidly attached to the structure. The discharge mainline is a 8" CI pipe that is rigidly attached to the wall and has an immediate 90 degree bend. It could be cracked or broken during the design earthquake.
WWPS028	The roof of the square structure is attached with rebar and should perform well. The round manholes should perform well but may leak at the joints	No flexible couplings are shown for intake or discharge piping.
WWPS030	There are no reinforcing drawings. The existing structures are small enough that standard precast structures should perform adequately.	The intake piping consists of several DIP mainlines that project inside the structure. The discharge piping is flanged 4" line.
WWPS031	The structure is a precast manhole. Considering the type of construction and diameter it is likely the	Intake piping is an 8" mainline. There is a coupling outside the wall which may provide some flexibility.

Table I-3. Pump Station Vulnerability Assessment (continued)		
Pump Station Number	Structural	Mainline Connection
	manhole rings will perform adequately but may crack in tension.	The 2" discharge mainline should have adequate flexibility.
WWPS035	The structure has two layers of rebar on the bottom perimeter tank wall (1.33 ft thick). The structure stands about 4 feet out of the ground. The top 10 feet of the tank has only one layer or rebar on the outer face of the perimeter tank wall (8 inches thick). There is only one layer or rebar on inner wall separating water from rest of structure. The roof is attached with rebar. The middle wall could perform poorly due to its lack of ductility. Since the middle wall is only 8 feet high the risk of failure is considered low.	The intake mainline is a 12" CI pipe that is rigidly attached to an adjacent manhole with a 12" valve. This mainline is susceptible to damage due to differential movement between the structures. The valve in adjacent structure maybe able to isolate the structure. The discharge mainline is a 8" CI pipe. It exits the structure thru a 10" pipe spool and is chalked to prevent leakage. This connection has some ductility.
WWPS036	All walls have two layers of reinforcement. All walls have hooks on the outer wall and a straight bar on the inner layer of rebar at every corner. Roof is attached to wall with unhooked rebar.	The intake mainline is a 18" CI pipe that is rigidly attached to the wall of the structure and enters at 45 degrees to the structure. This mainline is susceptible to damage due to it orientation to the wall of the structure. The discharge mainline is a 8" pipe. It exits the structure through a pipe spool and takes an immediate 90-degree turn. The mainline has flanges so it should not separate from wall
WWPS037	All walls have two layers of reinforcement. All walls have hooks on the outer wall and a straight bar on the inner layer of rebar at every corner. Roof is attached to wall with hooked rebar. Center wall only has two hooks.	The discharge mainline is a 20" pipe. It is rigidly attached to the wall of the structure. It exits the structure through a pipe spool. The intake mainlines are 36" and 21' and appear to be cast into the wall structure and stop at a sluice gate.
WWPS038	All walls have two layers of reinforcement. All walls have hooks on the outer wall and a straight bar on the inner layer of rebar at every corner. The corner detail does lack ductility. Roof is not attached to the walls. The roof could displace from the structure. Since this pumpstation is in a liquefaction zone it could cause the pumpstation to become inoperable.	The intake mainline is a 18" CI pipe that is rigidly attached to the wall. There is no information on the type of mainline used. The discharge mainline is an 8" pipe. It exits the structure through a pipe spool and takes an immediate 90-degree turn. The mainline has flanges so it should not separate from wall. The material is not called out, but it is detailed similar to a steel pipe.
WWPS039	All walls have two layers of reinforcement. All walls have hooks on the outer wall and a straight bar on the inner layer of rebar at every corner. The corner detail does lack ductility. The roof is not attached to the walls and could move relative to the structure.	There are three intake mainlines. There are two 18" intake mainlines and one 24" mainline. They intersect the wall at obtuse angles making them vulnerable to damage due to shaking. They are rigidly attached to the wall. There is no information on the type of mainline used. The discharge mainline is a 12" pipe. It exits the structure through a pipe spool and takes an immediate 45-degree turn. The mainline has flanges so it should not separate from wall. The material is not called out, but it is detailed similar to a cast iron.

Table I-3. Pump Station Vulnerability Assessment (continued)		
Pump Station Number	**Structural**	**Mainline Connection**
WWPS042	All walls have two layers of reinforcement. All walls have hooks on the outer wall and a straight bar on the inner layer of rebar at every corner. The corner detail does lack ductility. The roof is attached to the walls.	The intake mainline is a 10" pipe that is rigidly attached to the wall. There is no information on the type of mainline used. The discharge mainline is a 6" CI pipe and exits the structure through a pipe sleeve. The discharge mainline is vulnerable to being pulled apart at joints.
WWPS043	All walls have two layers of reinforcement. All walls have hooks on the outer wall and a straight bar on the inner layer of rebar at every corner. Roof is attached to the walls. The corner detail does lack ductility. The roof is attached to the walls.	The intake mainline is a 36" concrete pipe and is rigidly attached to the wall. The mainline can pull apart at the joints or be damaged due to shaking. The discharge pipe is a 10" CI mainline and is rigidly attached to the wall. The mainline has flanged connections so should not separate from the walls.
WWPS044	The structure has two layers of rebar on the perimeter walls and middle wall. This middle wall is connected to the tank with a hooked bar and a straight bar. This connection lacks ductility. The tank walls will not have adequate splice lengths. The longitudinal bars in the wall do not have adequate lap lengths. The roof is connected to the tank walls with rebar. The tank should perform adequately.	The intake mainline is a 12" pipe that is rigidly attached to the wall. There is no information on the type of mainline used. The discharge mainline is a 8" CI pipe and exits the structure through a pipe sleeve. The discharge mainline is vulnerable to being pulled apart at joints.
WWPS045	All walls have two layers of reinforcement. All walls have hooks on the outer wall and a straight bar on the inner layer of rebar at every corner. The middle wall has one hook and one straight bar. The corner detail does lack ductility. The roof is attached to the walls.	The intake mainline is an 8" pipe that is rigidly attached to the wall. There is no information on the type of mainline used. The discharge mainline is a 4" pipe and exits the structure through a pipe sleeve. The discharge mainline is vulnerable to being pulled apart at joints.
WWPS046	The structure has two layers of rebar on the perimeter walls. The longitudinal bars in the wall have adequate lap lengths. The roof is connected to the tank walls with rebar.	The discharge mainline is a 8" CI pipe and exits the structure through a pipe sleeve. The discharge mainline is vulnerable to being pulled apart at joints.
WWPS047	The structure has two layers of rebar. There are two layers of rebar with hooks that attach the roof to the pumpstation. The construction joints have two layers of rebar passing through the joints. The foundation is 2'-6” thick. It has one layer of reinforcing on bottom of the slab. This structure should perform well.	The two discharge mainlines are 8" CIP. They pass through a sleeve and are not rigidly attached to the wall. The intake mainline is an 8" CIP. It is passes through a sleeve when passing through the wall and are not rigidly attached to the wall.
WWPS048	All walls have two layers of reinforcement. All walls have hooks on the outer wall and a straight bar on the inner layer of rebar at every corner. The middle wall has two hooks. The corner detail does lack ductility. The roof is not attached to the walls.	The discharge mainline is a 8" CIP pipe and exits the structure through a pipe sleeve. There is a coupling just outside the tank wall. The intake mainline is an 18" PSS pipe with a coupling outside the wall.

Table I-3. Pump Station Vulnerability Assessment (continued)		
Pump Station Number	**Structural**	**Mainline Connection**
WWPS049	No rebar connection between the concrete wet well wall and tank walls and slab except for key. Roof is not connected to walls. No rebar in slab. Middle tank wall has no connection to base slab except for key. The span is short but feel the middle wall could perform poorly. Roof could move relative to the rest of the structure.	The discharge mainline is an 8" pipe and exits the structure through a pipe sleeve. The intake mainline is an 8" PS pipe it is rigidly attached to the wall. The mainline types are unknown
WWPS050	Structure has connection between walls, floor and roof slab. Splice lengths will be shorter than current code. Structure should perform well during seismic event	The discharge mainlines are two 4" CIP and exits the structure through a pipe sleeve. The intake mainline is an 6" CIP which is rigidly attached to the wall.
WWPS051	No rebar connection between the wet well tank walls, roof or floor except a concrete key. No rebar in slab. The span is short but feel the middle wall could perform poorly.	The discharge mainline is a 6" CIP and exits the structure through a pipe sleeve. The intake mainline is an 8" PS pipe which enters through a pipe sleeve.
WWPS053	Precast manhole should perform adequately during design earthquake	The discharge mainlines are two 4" CIP pipes which are attached rigidly to the structure. The intake mainline is an 8" CIP which is rigidly attached to the wall.
WWPS054	No rebar connection between the concrete wet well wall and tank walls and slab except for key. Roof is not connected to walls. No rebar in 3' thick slab. Tank walls have no connection to base slab except for key. The span is short, but the middle wall could perform poorly. The issue is compounded by the pumpstation being located in a liquefaction zone and the roof not being connected to any of the walls.	The discharge mainline is a 6" CIP and exits the structure through a pipe sleeve. The intake mainline is an 8" CIP pipe which is rigidly attached to the tank wall.
WWPS055	The Structure walls are 108" diameter precast pipe sections. Mainline could leak but should perform adequately.	The discharge mainline is a 4" CIP and exits the structure through a pipe sleeve. The intake mainline is an 6" CIP pipe which enters through a sleeve.
WWPS056	The Structure walls are 108" diameter precast pipe sections. Mainline walls could leak but should perform adequately.	The discharge mainline is two 1.5" pipes. The intake mainline is an 6" CIP pipe which enters through a sleeve.
WWPS057	The floor does not have rebar. The walls have relatively short spans. All walls have hooks on outer face but not on inner face. Roof is attached to wall with hooked rebar. Center wall only has one hook between wet well wall and tanks wall. Structure should perform adequately during seismic event.	The discharge mainline is a 6" CIP. The discharge mainline makes a 90-degree bend just outside the structure The intake mainline is an 8" CIP pipe with several short pipes just outside the structure wall.

Table I-3. Pump Station Vulnerability Assessment (continued)		
Pump Station Number	**Structural**	**Mainline Connection**
WWPS058	The floor does not have rebar. The walls have relatively short spans. All walls have hooks on outer face but not on inner face. Roof is attached to wall with hooked rebar. Center wall only has one hook between wet well wall and tanks wall. Structure should perform adequately during seismic event.	The discharge mainline is a 6" CIP. The discharge mainline makes a 90-degree bend just outside the structure The intake mainline is an 8" CIP pipe with several short pipes just outside the structure wall.
WWPS059	The floor does not have rebar. The walls have relatively short spans. All walls have hooks on outer face but not on inner face. Roof is attached to wall with hooked rebar. Center wall only has one hook between wet well wall and tanks wall. Structure should perform adequately during seismic event.	The discharge mainline is a 6" CIP. The discharge mainline makes a 90-degree bend just outside the structure The intake mainline is an 8" CIP pipe with several short pipes just outside the structure wall.
WWPS060	The floor does not have rebar. The walls have relatively short spans. All walls have hooks on outer face but not on inner face. Roof is attached to wall with hooked rebar. Center wall only has one hook between wet well wall and tanks wall. Structure should perform adequately during seismic event.	The discharge mainline is a 6" CIP. The discharge mainline makes a 90-degree bend just outside the structure The intake mainline is an 8" CIP pipe with several short pipes just outside the structure wall.
WWPS061	The floor does not have rebar. The walls have relatively short spans. All walls have hooks on outer face but not on inner face. Roof is attached to wall with hooked rebar. Center wall only has one hook between wet well wall and tanks wall. Structure should perform adequately during seismic event.	The discharge mainline is a 6" CIP. The discharge mainline makes a 90-degree bend just outside the structure The intake mainline is an 8" CIP pipe with several short pipes just outside the structure wall.
WWPS062	The floor does not have rebar. The walls have relatively short spans. All walls have hooks on outer face but not on inner face. Roof is attached to wall with hooked rebar. Center wall only has one hook between wet well wall and tanks wall. Structure should perform adequately during seismic event.	The discharge mainline is a 6" CIP. The discharge mainline makes a 90-degree bend just outside the structure The intake mainline is an 8" CIP pipe with several short pipes just outside the structure wall.
WWPS063	The floor does not have rebar. The walls have relatively short spans. All walls have hooks on outer face but not on inner face. Roof is attached to wall with hooked rebar. Center wall only has one hook between wet well wall and tanks wall. Structure should perform adequately during seismic event.	The discharge mainline is a 6" CIP. The discharge mainline makes a 90-degree bend just outside the structure The intake mainline is an 8" CIP pipe with several short pipes just outside the structure wall.
WWPS064	The floor does not have rebar. The walls have relatively short spans. All walls have hooks on outer face but not on inner face. Roof is attached to wall with hooked rebar. Center wall only has one hook between wet well wall and tanks wall. Structure should perform adequately during seismic event.	The discharge mainline is a 4" CIP. The intake mainline is an 8" CIP pipe which is rigidly attached to the wall.

Table I-3. Pump Station Vulnerability Assessment (continued)		
Pump Station Number	**Structural**	**Mainline Connection**
WWPS065	The floor does not have rebar. The walls have relatively short spans. All walls have hooks on outer face but not on inner face. Roof is attached to wall with hooked rebar. Center wall only has one hook between wet well wall and tanks wall. Structure should perform adequately during seismic event.	The discharge mainline is a 6" CIP. The discharge mainline makes a 90-degree bend just outside the structure The intake mainline is an 8" CIP pipe with several short pipes just outside the structure wall.
WWPS066	The floor does not have rebar. The walls have relatively short spans. All walls have hooks on outer face but not on inner face. Roof is attached to wall with hooked rebar. Center wall only has one hook between wet well wall and tanks wall. Structure should perform adequately during seismic event.	The discharge mainline is a 4" CIP. The intake mainline is an 8" CIP pipe which is rigidly attached to the wall.
WWPS067	The floor does not have rebar. The walls have relatively short spans. All walls have hooks on outer face but not on inner face. Roof is attached to wall with hooked rebar. Center wall only has one hook between wet well wall and tanks wall. Structure should perform adequately during seismic event.	The discharge mainline is a 4" CIP. The intake mainline is an 8" CIP pipe which is rigidly attached to the wall.
WWPS069	The floor does not have rebar. The walls have relatively short spans. All walls have hooks on outer face but not on inner face. Roof is attached to wall with hooked rebar. Center wall only has one hook between wet well wall and tanks wall. Structure should perform adequately during seismic event.	The discharge mainline is a 4" CIP. The intake mainline is an 8" CIP pipe which is rigidly attached to the wall. There is a bell just outside the structure
WWPS070	The floor does not have rebar. The walls have relatively short spans. All walls have hooks on outer face but not on inner face. Roof is attached to wall with hooked rebar. Center wall only has one hook between wet well wall and tanks wall. Structure should perform adequately during seismic event.	The discharge mainline is a 4" CIP. The discharge mainline makes a 90-degree bend just outside the structure The intake mainline is an 8" CIP pipe. It is rigidly attached to the wall and enters the structure at a 45-degree angle. It is rigidly attached to the structure. There is no record of joints outside structure for flexibility.
WWPS071	13' diameter tank: no rebar connection between the wet well wall tank walls, roof or floor except a concrete key. No rebar in slab. Tank walls have no connection to base slab, interior wall or roof except for a key. The span is short but feel the middle wall could perform poorly. Weir Structure: The floor is 3'-8" thick and has two layers of rebar. The walls have relatively short spans with an intermediate wall. The connections at walls have hooks on both faces. Roof is not attached to wall. Structure does not have resilience at wall corners because there are no hooks at corners. Walls at corner	The weir chamber has three mainlines. Two mainlines are 12" diameter and one 10" diameter. All mainlines have a joint just outside the structure. The discharge mainline is 4" CIP. It takes a 90-degree bend outside the pump station. The mainline has flanges. The pump station has a 8" CIP intake mainline. It is rigidly attached to the tank wall.

Table I-3. Pump Station Vulnerability Assessment (continued)		
Pump Station Number	**Structural**	**Mainline Connection**
	may crack and leak. Roof may move relative to the structure below.	
WWPS072	The floor does not have rebar. The walls have 20 ' spans. All walls have hooks on outer face but not on inner face. Roof is not attached to wall. Center wall only has one hook between wet well wall and tanks wall. Walls have stirrups near the wall intersections. Roof may move relative to structure below. Structure may crack at corners. The roof could displace from the structure. Since this pumpstation is in a liquefaction zone it could cause the pumpstation to become inoperable.	The discharge mainline is a 10" CIP. The discharge mainline makes a 90-degree bend just outside the structure The intake mainline is an 18" CIP pipe. It is rigidly attached to the wall. There is no record of joints outside structure for flexibility.
WWPS073	The floor does not have rebar. The walls have 20 ' spans. All walls have hooks on outer face but not on inner face. Roof is not attached to wall. Center wall only has one hook between wet well wall and tanks wall. Walls have stirrups near the wall intersections. Roof may move relative to structure below. Structure may crack at corners. The roof could displace from the structure. Since this pumpstation is in a liquefaction zone it could cause the pumpstation to become inoperable.	The discharge mainline is a 12" CIP. It is rigidly attached to the wall. The intake mainline is an 24" CIP pipe. It is rigidly attached to the wall but has a bell just outside the wall to allow flexibility.
WWPS074	The floor does not have rebar. The walls have 20 ' spans. All walls have hooks on outer face but not on inner face. Roof is not attached to wall. Center wall only has two hooks between wet well wall and tanks wall. Walls have stirrups near the wall intersections. Roof may move relative to structure below. Structure may crack at corners. The roof could displace from the structure. Since this pumpstation is in a liquefaction zone it could cause the pumpstation to become inoperable.	The discharge mainline is a 10" CIP. It is rigidly attached to the wall. It is connected with flanges. The intake mainline is a 20" pipe. It is rigidly attached to the wall. No bells shown outside the structure.
WWPS075	No connection to roof. No rebar in slab. Tank walls have no connection to base slab. The diameter of the tank is short and has good reinforcing. Structure roof may be displaced but walls should perform adequately	The discharge mainline is a 4" CIP. The intake mainline is an 8" CIP pipe. The mainlines enter through a sleeve.
WWPS076	The floor does not have rebar. The walls have 11 ' spans. All walls have hooks on outer face but not on inner face. Roof is not attached to wall. Center wall only has one bar connecting wall to tank walls. Tank walls have stirrups near the wall intersections. Roof may move relative to structure below. Structure may crack at corners. Structure should perform adequately.	The discharge mainline is a 4" CIP. It is rigidly attached to the roof. It is connected with flanges. The intake mainlines are thee 8" pipes. Each mainline has joints and a dresser coupling. The mainline is in danger of pulling apart. They are rigidly attached to the wall.

Table I-3. Pump Station Vulnerability Assessment (continued)		
Pump Station Number	**Structural**	**Mainline Connection**
WWPS077	The floor does not have rebar. The walls have 15 ' spans. All walls have hooks on outer face but not on inner face. Roof is not attached to wall/. Center wall has two hooks between wet well wall and tanks wall. Walls have stirrups near the wall intersections. Roof may move relative to structure below. Structure may crack at corners. Structure should perform adequately.	The discharge mainline is an 8" CIP. It is rigidly attached to the wall with a dresser coupling outside the wall of the structure. The intake mainline is an existing 18" mainlines with a dresser coupling just outside the structure. The mainline is in danger of pulling apart. They are rigidly attached to the wall.
WWPS078	The structure appears to be precast manhole or pipe. Considering the type of construction and diameter it is likely the manhole rings will perform adequately but may crack in tension. The roof is not attached with rebar and could be displaced from the walls of the structure. Since this structure is in a liquefaction zone it could pull apart at the joints resulting in the structure becoming inoperable.	The discharge mainline is a 4" CIP. The intake mainline is an 6" CIP pipe. The mainlines have flanges and connect to another manhole. The mainline could be damaged by differential movement between the structures
WWPS080	Roof is not connected to PS walls. Floor is 5.5' thick. Floor has a sump and suction pump cavity. Reinforcement in floor at pump cavity. There is no connection between the various precast rings. Roof may detach, walls could leak. Should perform adequately	The discharge mainline is a 4" CIP. It is rigidly attached to the wall. It is connected with flanges. The intake mainline is an 6" DIP. It is rigidly attached to the wall. Intake mainline has flanges.
WWPS081	The floor does not have rebar. The walls have 12 ' spans. All walls have hooks on outer face but not on inner face. Roof is not attached to wall. Center wall only has two hooks between wet well wall and tanks wall. Walls have stirrups near the wall intersections. Roof may move relative to structure below. Structure may crack at corners. Structure should perform adequately.	The discharge mainline is a 4" CIP. It is rigidly attached to the wall. There is a dresser coupling just outside the wall on the discharge mainline. The intake mainline is an 8" CIP. It is rigidly attached to the wall and has a dresser coupling just outside the wall of the structure.
WWPS082	Roof is not connected to PS walls. Floor is 5.5' thick. Floor has a sump and suction pump cavity. Reinforcement in floor at pump cavity. There is no connection between the various precast rings. Roof may detach, Walls could leak. Should perform adequately	The discharge mainline is a 4" DIP. It is rigidly attached to the wall. The intake mainline is an 6" DIP. It is rigidly attached to the wall.
WWPS083	Roof is not connected to PS walls. Floor is 4.75' thick. Floor has a sump and suction pump cavity. Reinforcement in floor at pump cavity. There is no connection between the various precast rings. Roof may detach, Walls could leak. Should perform adequately	The discharge mainline is a 4" DIP. It is rigidly attached to the wall and uses flanges. The intake mainline is an 6" CIP. It is rigidly attached to the wall.

Table I-3. Pump Station Vulnerability Assessment (continued)		
Pump Station Number	**Structural**	**Mainline Connection**
WWPS084	The floor does not have rebar. The walls have 12' spans. All walls have hooks on outer face but not on inner face. Roof is not attached to wall/. Center wall only has one hook between wet well wall and tanks wall. Roof may move relative to structure below. Structure may crack at corners. Structure should perform adequately.	The discharge mainline is a 6" CIP. It is rigidly attached to the wall. It is connected with flanges. The intake mainline is an 10" CI pipe. It is rigidly attached to the wall. There is a coupling just outside the wall of the structure.
WWPS114	The drawings were inadequate to make a structural assessment.	The discharge mainline is an 8" pipe. It appears to have been cast into the pumpstation wall. It is connected to the adjacent piping with flanges. The intake mainline is an 12" pipe. No details of inlet mainline are available.
WWPS118	The floor is 8" thick and has one layer of rebar. The walls have 7' spans. All walls have hooks on outer face but not on inner face. Roof is not attached to wall. No key is shown between roof and walls. Roof may move relative to structure below. Structure may crack at corners. Structure should perform adequately.	The discharge mainline is a 4" CIP. It is connected to the adjacent piping with flanges. The intake mainline is an 8" CI pipe. This mainline is connected to the wall of the wet well and the pump station. It has flanges. The connection through the wall is not shown but it appears to be rigidly attached to the wall of both structures. The mainline could be damaged due to shaking if the structures move differentially.

Table I-4. Summary of Likelihood Scores for Pump Station Tsunami Hazard (M7.0 SFZ)				
Category	**Number of Stations**	**Station Numbers**	**Approx. Distance from Inundation Zone Boundary (ft.)**	**Likelihood Score**
In mapped tsunami inundation zone	12	021, 022, 036, 037, 038, 043, 072, 073, 074, 075, 076, 077	0	5
Outside mapped tsunami inundation zone, but within 200 feet of tsunami inundation zone boundary	6	001	50	4
		039	70	3
		042	10	5
		070	5	5
		071	10	5
		082	125	2
Adjacent to Puget Sound shoreline, but outside DNR tsunami study boundary	5	030, 046, 047	Unknown	5
		028, 056		1[a]

a. Approximate 100 ft. or greater elevation difference between Puget Sound and pump station, therefore, not considered at risk of tsunami inundation.

Table I-5. Summary of Likelihood Scores for Pump Station Seiche Hazard (M9.0 CSZ or M7.0 SFZ)				
Category	**Number of Stations**	**Station Numbers**	**Approx. Distance from Inundation Zone Boundary (ft.)**	**Likelihood Score**
In approximate seiche inundation zone	14	005, 007, 011, 020, 035, 050, 051, 057, 064, 066, 067, 081, 083, 084	0	5
Outside approximate seiche inundation zone, but within 200 feet of approximate seiche inundation zone boundary	18	002	150	2
		004	5	5
		006	20	4
		009	65	3
		010	15	4
		013	30	4
		025	200	1
		048	80	3
		049	5	5
		054	20	4
		055	145	2
		058	55	3
		059	45	4
		060	75	3
		061	110	2
		062	25	4
		065	5	5
		080	25	4

Table I-6. Pump Station Likelihood of Failure Component Scores								
Pump Station Number	Landslide	Floatation	Structural Failure		Failure at Mainline Connection	Inundation and Scour		Overall
			M7.0 SFZ	M9.0 CSZ		Tsunami	Seiche	
WWPS001	1	1	1	1	3	4	1	**4**
WWPS002	5	1	5	4	1	1	2	**5**
WWPS004	1	4	2	2	5	1	5	**5**
WWPS005	1	1	5	4	2	1	5	**5**
WWPS006	1	1	5	4	3	1	4	**5**
WWPS007	1	1	3	3	2	1	5	**5**
WWPS009	1	5	5	5	5	1	3	**5**
WWPS010	1	1	3	3	2	1	4	**4**
WWPS011	1	5	3	3	5	1	5	**5**
WWPS013	1	1	5	5	5	1	4	**5**
WWPS017	1	1	1	1	4	1	4	**4**
WWPS018	1	1	1	1	1	1	4	**4**
WWPS019	1	1	1	1	2	1	4	**4**
WWPS020	1	1	3	3	3	1	5	**5**
WWPS021	5	1	3	3	5	5	1	**5**
WWPS022	1	1	4	4	2	5	1	**5**
WWPS025	1	1	5	5	2	1	1	**5**
WWPS028	5	1	1	1	2	1	1	**5**
WWPS030	5	1	1	1	2	5	1	**5**
WWPS031	1	1	1	1	1	1	1	**1**
WWPS035	1	1	2	2	5	1	5	**5**
WWPS036	1	4	1	1	5	5	1	**5**
WWPS037	5	5	2	2	5	5	1	**5**
WWPS038	1	1	4	4	5	5	1	**5**
WWPS039	1	1	1	1	3	3	1	**3**
WWPS042	5	1	2	2	5	5	1	**5**
WWPS043	1	1	2	2	2	5	1	**5**
WWPS044	1	1	2	2	5	1	1	**5**
WWPS045	1	1	2	2	5	1	1	**5**
WWPS046	1	1	1	1	2	5	1	**5**
WWPS047	1	1	1	1	1	5	1	**5**
WWPS048	1	1	2	2	1	1	3	**3**
WWPS049	1	1	4	4	2	1	5	**5**
WWPS050	1	1	2	2	5	1	5	**5**

Table I-6. Pump Station Likelihood of Failure Component Scores (continued)								
Pump Station Number	Landslide	Floatation	Structural Failure		Failure at Mainline Connection	Inundation and Scour		Overall
			M7.0 SFZ	M9.0 CSZ		Tsunami	Seiche	
WWPS051	1	1	3	3	2	1	5	**5**
WWPS053	1	1	1	1	2	1	5	**5**
WWPS054	1	1	5	5	5	1	4	**5**
WWPS055	1	1	3	3	5	1	2	**5**
WWPS056	5	1	1	1	2	1	2	**5**
WWPS057	1	1	2	2	1	1	5	**5**
WWPS058	1	1	2	2	1	1	3	**3**
WWPS059	5	1	2	2	1	1	4	**5**
WWPS060	5	1	2	2	1	1	3	**5**
WWPS061	1	1	2	2	5	1	2	**5**
WWPS062	1	1	2	2	5	1	4	**5**
WWPS063	1	1	2	2	5	1	4	**5**
WWPS064	1	1	2	2	2	1	5	**5**
WWPS065	1	1	2	2	5	1	5	**5**
WWPS066	1	1	2	2	2	1	5	**5**
WWPS067	1	1	2	2	2	1	5	**5**
WWPS069	1	1	2	2	2	1	5	**5**
WWPS070	1	4	2	2	5	5	1	**5**
WWPS071	1	1	4	4	5	5	1	**5**
WWPS072	1	1	3	3	5	5	1	**5**
WWPS073	1	1	5	5	5	5	1	**5**
WWPS074	1	1	2	2	5	5	1	**5**
WWPS075	1	1	2	2	5	5	1	**5**
WWPS076	5	1	2	2	5	5	1	**5**
WWPS077	1	1	2	2	1	5	1	**5**
WWPS078	1	4	4	4	5	1	1	**5**
WWPS080	1	1	2	2	1	1	4	**4**
WWPS081	1	1	2	2	1	1	5	**5**
WWPS082	5	1	2	2	1	2	1	**5**
WWPS083	1	4	2	2	4	1	5	**5**
WWPS084	1	1	2	2	1	1	5	**5**
WWPS114	1	1	NA	NA	5	1	5	**5**
WWPS118	1	5	2	2	2	1	5	**5**

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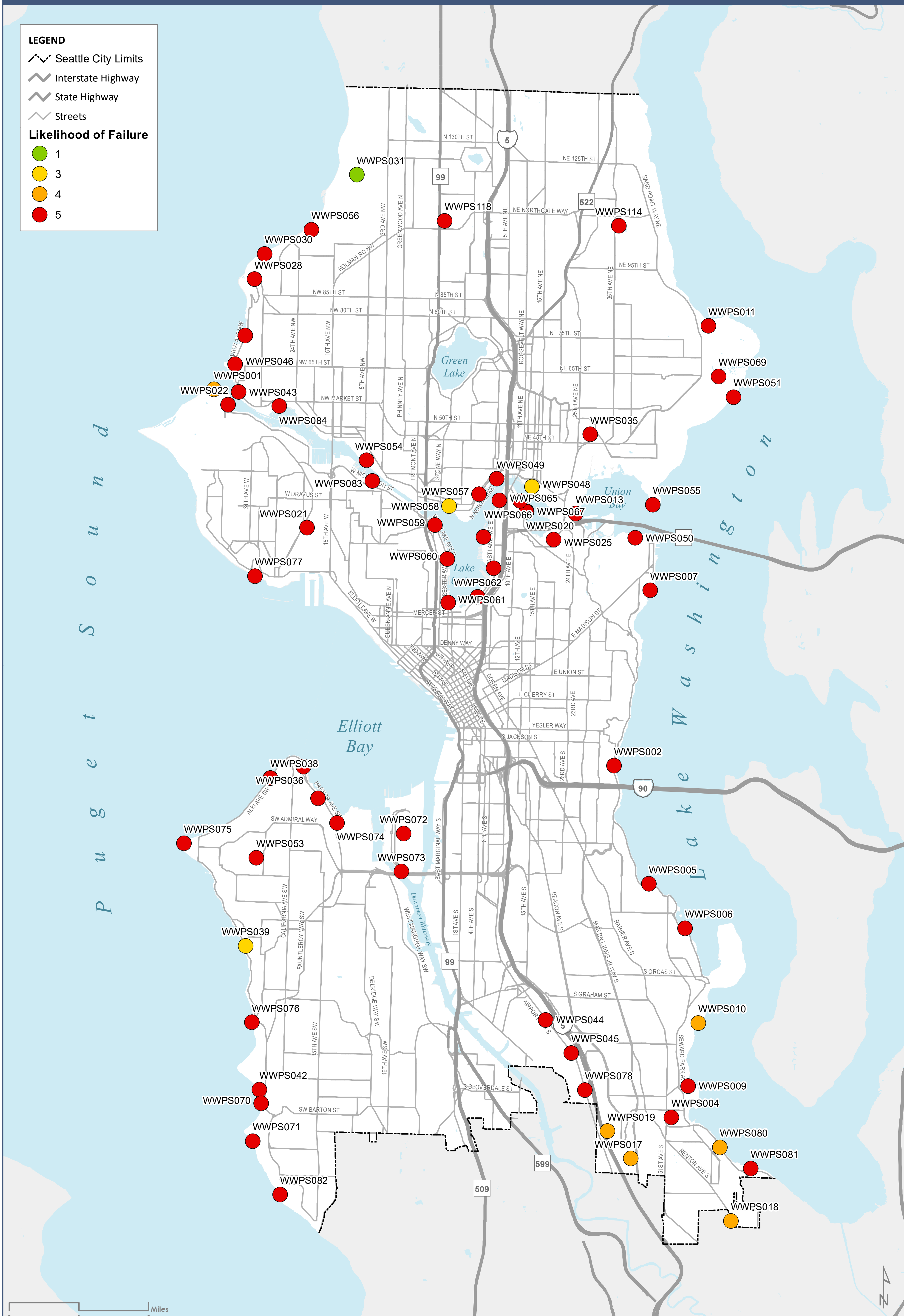
Map I-1: Likelihood of Failure for Wastewater Pump Stations



Author: Fill out Map Document Properties Date: 1/7/2022

Appendix J: Example Calculation for CSO Facility Likelihood of Failure Score

Example Calculation: CSO 24

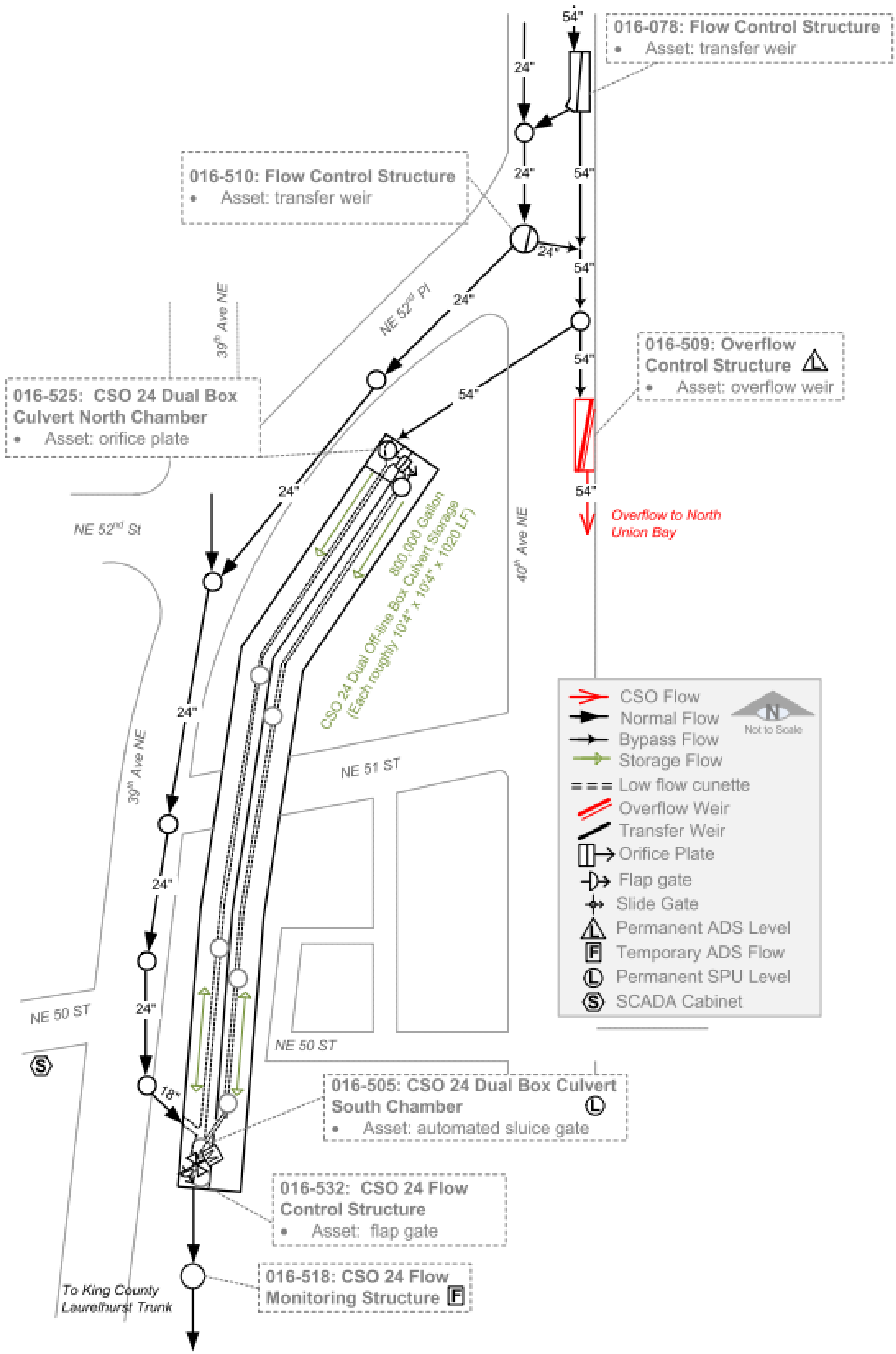


Figure J-1. Schematic for CSO 24 located between 5250 and 5130 40th Ave NE

Geotechnical hazard data for Pump Station 73 are provided in Table J-1.

Table J-1. Geotechnical Hazard Data for Pump Station 73 Example				
Seismic Event	**Permanent Ground Deformation (PGD), inches**	**0.1s Spectral Acceleration ($SA_{0.1}$), g**	**0.3s Spectral Acceleration ($SA_{0.3}$), g**	**1s Spectral Acceleration (SA_{1}), g**
M7.0 Seattle Fault Zone	0.00	1.70	2.26	1.52
M9.0 Cascadia Subduction Zone	0.00	0.89	1.17	1.32

The project team examined as-built drawings for CSO 24 facilities (see Figure J-1). The following structures were assessed:

- 144-inch inner diameter concrete mainline:
  - Structural notes: foundation and roof are cast-in-place.
  - Mainline connection notes: mainlines extend through the wall with a flex joint just outside the structure.
- Box culvert 23.33 feet by 14 feet high:
  - Structural notes: rebar in some of the walls is placed improperly, wall with improperly placed rebar lacks ductility; portions of box conduit with proper reinforcement appear to be adequate.
  - Mainline connection notes: details are typical for cast-in-place structure; no issues found.
- Manholes 12, 13 and 20:
  - Structural notes: three manholes are held down by reinforced concrete slab; structure will perform adequately; there is no connection shown between the manhole structure on top of the structure.
  - Mainline connection notes: details show one mainline connecting the manholes and adjacent structure; pounding could be an issue.
- Overflow Structure 19, which is 9 feet by 30 feet x 10 feet high:
  - Structural notes: the structure has two layers of reinforcement; corner details do not have hooks in plan view; there is no connection shown between the manhole structure on top of the structure.
  - Mainline connection notes: no special details are provided; considering the type and era of construction there does not appear to be an issue.

The following are steps in the engineering assessment:

1. CSO 24 is outside of mapped landslide zones; assign a score of 1.
2. Facilities are located outside of liquefaction susceptibility zones; assign a score of 1.
3. Perform a desktop assessment for structural failure during a M7.0 SFZ earthquake on all components; the worst case based on engineer's assessment was the condition of the box culvert; assign a score of 4.
4. Perform a desktop assessment for structural failure during a M9.0 CSZ earthquake on all components; the worst case based on engineer's assessment was the condition of the box culvert—assign a score of 4.
5. Perform a desktop assessment for failure at mainline connections; the worst case based on the engineer's assessment was the connections at the manholes—assign a score of 3.
6. CSO 24 structures are not located within a tsunami inundation zone, assign score of 1.
7. CSO 24 structures are not located in a seiche inundation zone; assign score of 1.
8. Take the highest component score from all of the above assessments and assign an overall likelihood of failure of 4.

Appendix K: Likelihood of Failure Data for Major CSO Facilities

Map K-1. Likelihood of Failure for Major CSO Facilities

Table K-1. Summary of CSO Facility Geotechnical Hazard Data

CSO Facility	M7.0 SFZ				M9.0 CSZ			
	Permanent Ground Deformation *PGD* (in)	0.1s Spectral Acceleration $SA_{0.1}$ (g)	0.3s Spectral Acceleration $SA_{0.3}$ (g)	1s Spectral Acceleration $SA_{1.0}$ (g)	Permanent Ground Deformation *PGD* (in)	0.1s Spectral Acceleration $SA_{0.1}$ (g)	0.3s Spectral Acceleration $SA_{0.3}$ (g)	1s Spectral Acceleration $SA_{1.0}$ (g)
01	1.18	2.74	3.68	1.73	0.98	0.91	1.30	0.74
02 (01)	0.00	2.72	3.68	1.78	0.00	0.91	1.30	0.76
02 (02)	0.00	2.69	3.66	1.80	0.00	0.91	1.30	0.76
03 (01)	0.00	2.64	3.51	1.65	0.00	0.91	1.30	0.76
03 (02)	0.00	2.64	3.51	1.63	0.00	0.94	1.30	0.76
04	0.00	2.67	3.23	1.35	0.00	0.89	1.12	0.61
05	11.53	2.79	3.63	1.52	9.27	0.89	1.17	0.66
06	9.23	2.79	3.68	1.60	7.51	0.86	1.17	0.66
07	0.00	2.77	3.66	1.63	0.00	0.86	1.17	0.66
08/8A	0.00	3.00	3.91	1.68	0.00	0.86	1.14	0.63
09	0.00	2.74	3.84	2.08	0.00	0.84	1.19	0.74
09 (A)	0.00	2.64	3.71	2.08	0.00	0.84	1.19	0.74
10	0.00	2.64	3.73	2.08	0.00	0.84	1.19	0.74
11	0.00	2.46	3.53	2.13	0.00	0.81	1.19	0.79
11 (A)	0.00	2.54	3.68	2.13	0.00	0.84	1.19	0.76
12	16.21	2.87	4.01	2.08	13.03	0.86	1.19	0.71
13	0.00	3.17	4.14	2.86	0.00	0.89	1.17	1.32
14	0.00	3.25	4.19	2.86	0.00	0.89	1.17	1.27
15	0.00	3.43	4.17	2.67	0.00	0.89	1.14	1.22
16	0.00	3.51	4.17	2.55	0.00	0.89	1.14	1.17
17	0.00	3.07	3.40	2.10	0.00	0.91	1.09	1.07
18	0.00	3.05	3.45	2.06	0.00	0.91	1.09	1.07
19	0.00	1.80	2.44	1.71	0.00	0.89	1.17	1.32
20	0.00	1.78	2.39	1.68	0.00	0.89	1.17	1.37
21	0.00	1.80	2.36	1.64	0.00	0.89	1.17	1.32
22/22A	0.00	1.78	2.24	1.49	0.00	0.89	1.14	1.27
22A (01)	0.00	1.75	2.24	1.49	0.00	0.89	1.14	1.27
22A (02)	0.00	1.75	2.24	1.49	0.00	0.89	1.17	1.32

Table K-1. Summary of CSO Facility Geotechnical Hazard Data (continued)

CSO Facility	M7.0 SFZ				M9.0 CSZ			
	Permanent Ground Deformation *PGD* (in)	0.1s Spectral Acceleration $SA_{0.1}$ (g)	0.3s Spectral Acceleration $SA_{0.3}$ (g)	1s Spectral Acceleration $SA_{1.0}$ (g)	Permanent Ground Deformation *PGD* (in)	0.1s Spectral Acceleration $SA_{0.1}$ (g)	0.3s Spectral Acceleration $SA_{0.3}$ (g)	1s Spectral Acceleration $SA_{1.0}$ (g)
23A/B (01)	0.00	1.65	2.11	1.41	0.00	0.89	1.17	1.32
23A/B (02)	0.72	1.68	2.06	1.37	0.89	0.89	1.14	1.27
23A/B (03)	0.72	1.65	2.08	1.41	0.89	0.89	1.14	1.27
23C	0.00	1.70	1.98	1.22	0.00	0.89	1.09	1.17
24 (01)	0.00	1.70	2.26	1.52	0.00	0.89	1.17	1.32
24 (02)	0.00	1.73	2.26	1.56	0.00	0.89	1.17	1.32
24 (03)	0.00	1.70	2.24	1.52	0.00	0.89	1.17	1.32
25	9.10	1.78	2.39	1.71	10.23	0.89	1.19	1.37
26	9.98	1.73	2.46	2.21	11.18	0.84	1.24	1.63
27	0.00	2.34	2.72	1.71	0.00	0.99	1.27	1.27
28	0.00	2.46	2.74	1.60	0.00	1.02	1.24	1.17
29A	0.00	2.97	3.73	1.57	0.00	0.89	1.12	0.61
30	15.65	3.23	4.04	2.55	12.98	0.94	1.19	1.27
31	0.00	1.80	2.51	2.13	0.00	0.86	1.24	1.52
33A	0.00	1.93	2.64	2.21	0.00	0.91	1.30	1.57
33B	0.00	2.26	2.92	2.02	0.00	0.94	1.24	1.37
34	6.48	2.97	3.99	1.90	5.64	0.94	1.27	0.74
35	0.00	3.23	3.73	1.45	0.00	0.91	1.14	0.56
36	0.00	1.96	2.67	1.94	0.00	0.89	1.22	1.37

Shaded rows show data for CSO facilities that were not included in the desktop assessment. Eight major CSO facilities, consisting of multiple large cast-in-place concrete structures, were evaluated using the desktop approach described in Section 5.2.1. Other CSO facilities, consisting primarily of large diameter mainline and a flow control device, were evaluated using the mainline assessment approach described in Section 5.1.

Table K-2. Summary of CSO Facility PGD Information

CSO Facility Number	Date of Construction	General Description	Landslide Zone	Liquefaction Zone	PGD, inches	
					M7.0 SFZ	M9.0 CSZ
02	1987	Tank 168: circular 100' diameter post tensioned tank.	No	No	0.00	0.00
02	1987	Control Building: CMU construction, The Control building is 15'-4" x 29'-4". The building consists of a series of CMU panels that are joined together with welded connections. The building is placed upon stem wall foundation.	No	No	0.00	0.00
02	1987	Diversion Structure: cast-in-place reinforced concrete structure, 14' x 14' x 35' deep.	No	No	0.00	0.00
03	1987	Tank 169: circular 100' diameter post tensioned tank.	No	No	0.00	0.00
03	1987	Diversion Structure: cast-in-place reinforced concrete structure, 14' x 14' x 35' deep.	No	No	0.00	0.00
08/8A	2017	Storage Tank: a cast-in-place concrete structure on piles that is 60' wide and ~300' long x 26' deep. There is a series of columns in the middle CSO structure.	No	No	0.00	0.00
08/8A	2017	Effluent Control Structure: cast-in-place concrete structure that is 8' x 14' x 10' deep.	No	No	0.00	0.00
08/8A	2017	Influent Control Structure: cast-in-place concrete structure that is 11' x14’ x 12' deep.	No	No	0.00	0.00
09A	2013	Diversion structure: is a cast-in-place concrete structure that is 8' x 10' x 10.16' deep.	No	No	0.00	0.00
09A	2013	Shutoff Valve Vault: cast-in-place structure that is 7.16' x 12.25' x 10.16' deep.	No	No	0.00	0.00
09A	2013	Drain Pump Valve Vault: a cast-in-place structure that is 10.66 x 9.66 x 9.5 deep.	No	No	0.00	0.00
09A	2013	Storage Tank: a pile supported cast-in-place concrete structure that is 130' x 21' x 17' deep.	No	No	0.00	0.00
09A	2013	Facility Valve Vault: cast-in-place structure that is 45.5' x 34.5' x 13' deep.	No	No	0.00	0.00

Table K-2. Summary of CSO Facility PGD Information (continued)						
CSO Facility Number	Date of Construction	General Description	Landslide Zone	Liquefaction Zone	PGD, inches	
					M7.0 SFZ	M9.0 CSZ
11A	2013	Genesee Facility Storage Tank: cast-in-place structure supported on piles at exterior of facility is 44'x 95' x 20' deep	No	No	0.00	0.00
11A	2013	Genesee Facility Drain Pump and Valve Vault: cast-in-place structure that is 10.66' diameter x 11.66 'x 9' deep	No	No	0.00	0.00
11A	2013	Genesee Facility Vault: concrete structure that is 42.5 x 31.5 x 12 deep	No	No	0.00	0.00
11A	2013	Genesee Facility Diversion and Overflow Structure: cast-in-place structure that is 13' x 14.83' W x 8' deep	No	No	0.00	0.00
22/22A	1985	Lake Washington North: 72" and 96" mainline	No	No	0.00	0.00
22/22A	1985	CSO 22A Lake Washington North: cast-in-place structure overflow structures 38 and 39 that are 20' x 10' x 10' H	No	No	0.00	0.00
23C	2015	CSO 23C Facilities Vault	No	No	0.00	0.00
23C	2015	Windermere Facility Structural Storage Tank: cast-in-place structure that is 217' x 121' x 23' H	No	No	0.00	0.00
24	1987	144" ID Concrete Mainline	No	No	0.00	0.00
24	1987	Box culvert 23.33' x 14'H	No	No	0.00	0.00
24	1987	Manholes 12, 13 and 20	No	No	0.00	0.00
24	1987	Overflow Structure 19 is 9' x 30' x 10' H	No	No	0.00	0.00

Table K-3. CSO Facility Vulnerability Assessment			
CSO Facility Number	**General Description**	**Structural**	**Mainline Connection**
02	Tank 168: circular 100' diameter post tensioned tank	The tank does not appear to have any obvious structural deficiencies. The column shear tie detailing is unclear. However, this type of structure should be given a Tier 3 assessment considering the type of construction and era of construction.	Piping connection has flexibility. No issues identified.
02	Control Building: CMU construction. The Control building is 15'-4" x 29'-4". The building consists of a series of CMU panels that are joined together with welded connections. The building is placed upon stem wall foundation.	The building consists of a series of CMU panels that are joined together at three places on the side of each panel. The panels are anchored to the foundation with a #5 dowel at each corner of the panel. At the corners the joints are welded together. The diaphragm is connected with a vertical hook which can take vertical forces but does not appear to be able to resist lateral forces. No topping slab over precast hollow planks; lack of crossties in the direction perpendicular to hollow planks; the diaphragm can be pulled apart.	Not applicable
02	Diversion Structure: cast-in-place reinforced concrete structure, 14' x 14' x 35' deep	The concrete walls are reinforced with two layers or rebar. Each layer has a hook at all corners. The roof is attached to the chamber below with rebar. The bars connecting the roof are not hooked.	All connections are embedded into the wall of the structure. The only risk is from the mainline itself coming apart outside the structure.
03	Tank 169: circular 100' diameter post tensioned tank	The tank does not appear to have any obvious structural deficiencies. The column shear tie detailing is unclear. However, this type of structure should be given a Tier 3 assessment considering the type of construction and era of construction.	Piping connection has flexibility. No issues identified.
03	Diversion Structure: cast-in-place reinforced concrete structure, 14' x 14' x 35' deep	The concrete walls are reinforced with two layers or rebar–each layer has a hook at all corners. Roof is attached to the chamber below with rebar–the bars connecting the roof are not hooked.	All connections are embedded into the wall of the structure. The only risk is from the mainline itself coming apart outside the structure.
08/8A	Storage Tank: a cast-in-place concrete structure on piles that is 60' wide and ~300' long x 26' deep. There is a series of columns in the middle CSO structure	Design Criteria Ss = 1.8g S1 = 0.49g SF = 1.25 No issues found	Piping connections consists of a mainline embedded in the wall of the structure with a joint just outside the structure. This provides good flexibility at the wall. No issues identified.

Table K-3. CSO Facility Vulnerability Assessment (continued)			
CSO Facility Number	**General Description**	**Structural**	**Mainline Connection**
08/8A	Effluent Control Structure: cast-in-place concrete structure that is 8' x 14 x 10' deep	Design Criteria Ss = 1.48g S1 = 0.49g SF = 1.25 Only standard details provided, no issues identified	Piping connections consists of a mainline embedded in the wall of the structure with a joint just outside the structure. This provides good flexibility at the wall. No issues identified.
08/8A	Influent Control Structure: cast-in-place concrete structure that is 11' x14' x 12' deep	Design Criteria Ss = 1.48g S1 = 0.49g SF = 1.25 Only standard details provided, no issues identified	Piping connections consists of a mainline embedded in the wall of the structure with a joint just outside the structure. This provides good flexibility at the wall. No issues identified.
09A	Diversion structure: is a cast-in-place concrete structure that is 8' x 10' x 10.16' deep	Design Criteria Ss = 1.51g S1 = 0.52g SF = 1.25 Only standard details provided, no issues identified	Piping connection has flexible coupling and bell and spigot joint immediately outside the structure. No issues identified.
09A	Shutoff Valve Vault: cast-in-place structure that is 7.16' x 12.25' x 10.16' deep	Design Criteria Ss = 1.51g S1 = 0.52g SF = 1.25 Only standard details provided, no issues identified except top manhole structure could move, relative to rest of structure	Piping connection has flexible coupling immediately outside the structure. No issues identified.
09A	Drain Pump Valve Vault: a cast-in-place structure that is 10.66 x 9.66 x 9.5 deep	Design Criteria Ss = 1.51g S1 = 0.52g SF = 1.25 Only standard details provided, no issues identified except top manhole structure could move, relative to rest of structure	Piping Connection has flexible coupling immediately outside the structure. No issues identified.
09A	Storage Tank: a pile supported cast-in-place concrete structure that is 130' x 21' x 17' deep	Design Criteria Ss = 1.51g S1 = 0.52g SF = 1.25 No issues identified	Discharge piping connections are with flanged mainline. No issues identified

Table K-3. CSO Facility Vulnerability Assessment (continued)			
CSO Facility Number	**General Description**	**Structural**	**Mainline Connection**
09A	Facility Valve Vault: cast-in-place structure that is 45.5' x 34.5' x 13' deep	Design Criteria Ss = 1.51g S1 = 0.52g SF = 1.25 Only standard details provided, no issues identified except top manhole structure could move, relative to rest of structure	No connection details were provided
11A	Genesee Facility Storage Tank: cast-in-place structure supported on piles at exterior of facility is 44'x 95' x 20' deep	Design Criteria Ss = 1.48g S1 = 0.57g SF = 1.25 No issues identified	Mainline passes through structure wall using sleeves. All piping is rigidly connected to adjacent mainline. No issues identified.
11A	Genesee Facility Drain Pump and Valve Vault: cast-in-place structure that is 10.66' diameter x 11.66 'x 9' deep	Design Criteria Ss = 1.50g S1 = 0.51g SF = 1.25 No issues identified except top manhole structure could move, relative to rest of structure	Mainline connection to structure is made using a spool with flanges embedded in the wall. The mainline is rigidly attached to the CSO structure.
11A	Genesee Facility Vault: concrete structure that is 42.5 x 31.5 x 12 deep	Design Criteria Ss = 1.50g S1 = 0.51g SF = 1.25 No issues identified	Connection of 24" mainline is embedded into the wall of the structure.
11A	Genesee Facility Diversion and Overflow Structure: cast-in-place structure that is 13' x 14.83'W x 8' deep	Design Criteria Ss = 1.50g S1 = 0.51g SF = 1.25 Standard details used for construction, no issues identified	RCP mainline rigidly connected to structure. No special details were found.
22/22A	Lake Washington North: 72" and 96" mainline	No issues found	Typical bell and spigot for PCCP is 3" to 4" minimum. Connection looks adequate
22/22A	CSO 22A Lake Washington North: cast-in-place structure overflow structures 38 and 39 that are 20' x 10' x 10'H	Reinforcing will provide good ductility–manhole on top of structure could separate from structure	Mainlines extend through the wall with a flex joint just outside the structure

Table K-3. CSO Facility Vulnerability Assessment (continued)

CSO Facility Number	General Description	Structural	Mainline Connection
23C	CSO 23 Facilities Vault	Design Criteria Ss = 1.26g S1 = 0.43g SF = 1.25 No issues identified	No issues identified
23C	Windermere Facility Structural Storage Tank: cast-in-place structure that is 217' x 121' x 23'H	Design Criteria Ss = 1.26g S1 = 0.43g SF = 1.25 No issues identified	No issues identified
24	144" ID Concrete Mainline	Foundation and roof are cast-in-place.	Mainlines extend through the wall with a flex joint just outside the structure
24	Box culvert 23.33' x 14'H	Rebar in some of the walls is placed improperly, wall with improperly placed rebar lacks ductility. Portions of box conduit with proper reinforcement appear to be adequate.	Details are typical for cast-in-place structures. No issues found
24	Manholes 12, 13, and 20	Three manholes are held down by reinforced concrete slab. Structure will perform adequately. There is no connection shown between the manhole structure on top of the structure.	Details show one mainline connecting the manholes and adjacent structure. Pounding could be an issue.
24	Overflow Structure 19 is 9' x 30' x 10' H	Structure has two layers of reinforcement. Corner details do not have hooks in plan view. There is no connection shown between the manhole structure on top of the structure.	No special details are provided. Considering the type and era of construction there does not appear to be an issue.

Table K-4. CSO Facility Vulnerability Assessment								
CSO Facility Number	**Landslide**	**Floatation**	**Structural Failure**		**Failure at Mainline Connection**	**Inundation and Scour**		**Overall**
			M7.0 SFZ	**M9.0 CSZ**		**Tsunami**	**Seiche**	
02	1	1	5	5	1	1	1	**5**
03	1	1	5	5	1	1	1	**5**
08/8A	1	1	1	1	1	1	5	**5**
09A	1	1	2	2	1	1	5	**5**
11A	1	1	2	2	2	1	5	**5**
22/22A	1	4	1	1	1	1	1	**4**
23C	1	1	1	1	1	1	1	**1**
24	1	1	4	4	3	1	1	**4**

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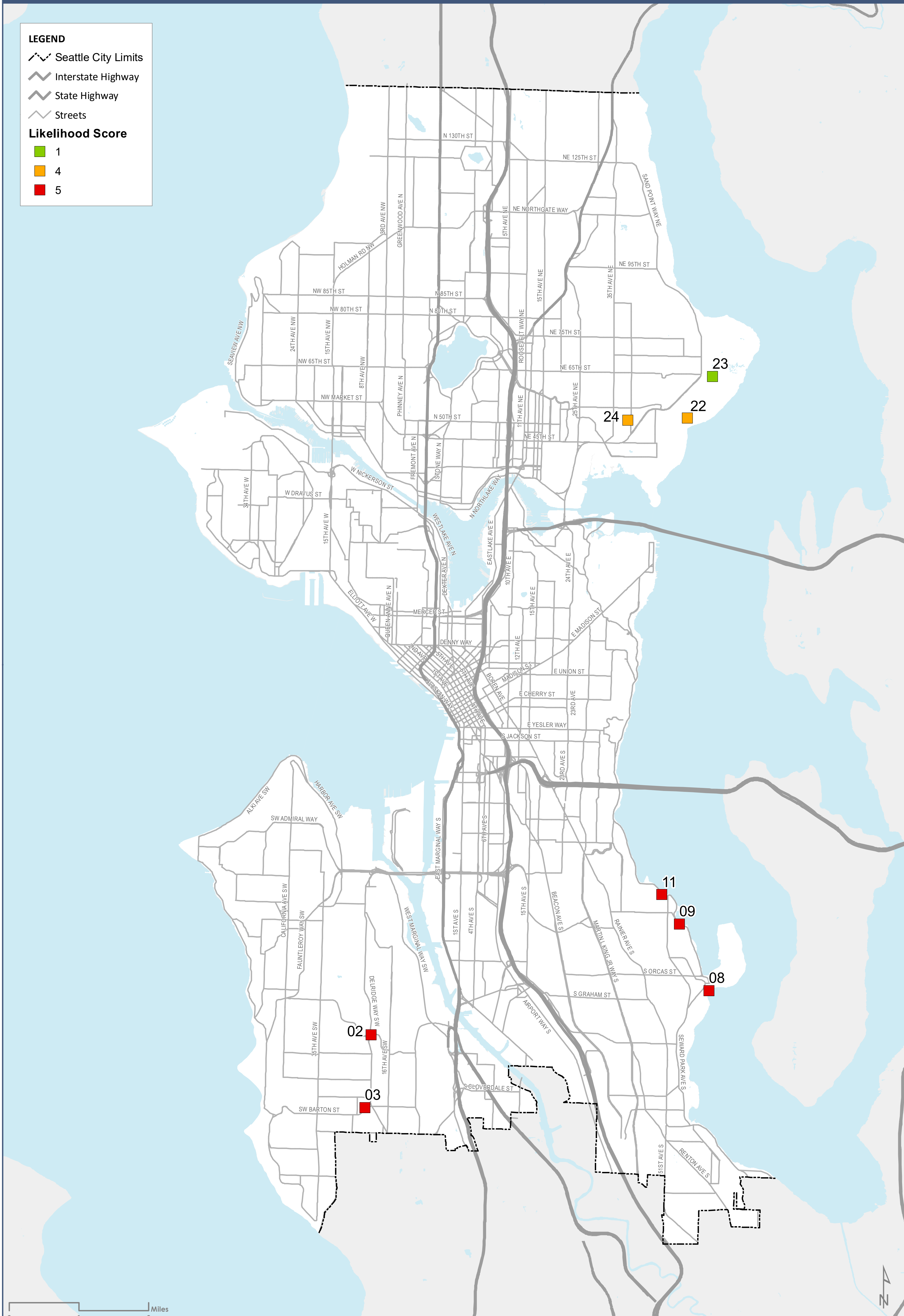
Map K-1: Likelihood of Failure for Major CSO Facilities



Appendix L: Example Calculations for Risk Scores

Example Calculation: Drainage Mainline 2213885

Calculate the Seismic Risk Score for drainage mainline 2213885 using the following steps:

1. Refer to Table 6-1 for consequence scoring criteria and attributed component scores.
2. Determine the component score for capacity based on mainline diameter. The mainline diameter is 96 inches, which is greater than 48 inches; assign a score of 2.0.
3. Check whether the mainline falls within a high-use area based on SPU's mapping data. The mainline does not fall within a high-use area; assign a component score of 0.0.
4. Determine whether the mainline falls within 100 feet of critical facility parcel boundary based on SPU's mapping data. The mainline is not near a critical facility; assign a component score of 0.0.
5. Check whether the mainline is near a major transportation route based on SPU's mapping data. The mainline does fall near a major transportation route; assign a component score of 1.0.
6. No component scores for environmental impact are assigned to drainage mainlines; assign a component score of 0.0.
7. Calculate the Consequence Score by summing the above component scores:

$$\begin{aligned} Consequence\ Score &= Capacity\ Score + High\text{-}use\ Area\ Score + Critical\ Facility\ Score \\ &\quad + Major\ Transportation\ Route\ Score + Environmental\ Impact\ Score \end{aligned}$$

$$Consequence\ Score = 2.0 + 0.0 + 0.0 + 1.0 + 0.0 = 3.0$$

8. Determine the Likelihood Score based on the likelihood of failure assessments described in Section 5 (see example calculations in Appendix F). Likelihood Score for this mainline is 4.2151.
9. Determine the Equity Score based on SPU's mapping data. The mainline falls within a "medium" level of disadvantage; assign an Equity Score of 3.0.
10. Calculate the Risk Score:

$$Risk\ Score = (Consequence\ Score \times Likelihood\ Score) + Equity\ Score$$

$$Risk\ Score = (3.0 \times 4.2151) + 3.0 = 15.6453$$

Example Calculation: Wastewater Pump Station 73

Calculate the Seismic Risk Score for Wastewater Pump Station 73 using the following steps:

1. Refer to Table 6-3 for consequence scoring criteria and attributed component scores.
2. Determine the component score for capacity based on average annual inflow. Average annual inflow is between 50 and 100 gallons per minute; assign a score of 1.5.
3. Check whether the pump station falls within a high-use area based on SPU's mapping data. The pump station does not fall within a high-use area; assign a component score of 0.0.
4. Determine whether the pump station serves a critical facility. The pump station does not serve a critical facility; assign a component score of 0.0.
5. No component scores for major transportation route are used for pump stations; assign a component score of 0.0.
6. Determine whether pump station overflows to a nearby waterbody. Assign a component score of 0.0 for environmental impact.
7. Calculate the Consequence Score by summing the above component scores:

$$\begin{aligned} Consequence\ Score &= Capacity\ Score + High\text{-}use\ Area\ Score + Critical\ Facility\ Score \\ &\quad + Major\ Transportation\ Route\ Score + Environmental\ Impact\ Score \end{aligned}$$

$$Consequence\ Score = 1.5 + 0.0 + 0.0 + 0.0 + 0.0 = 1.5$$

8. Determine the Likelihood Score based on the likelihood of failure assessments described in Section 5 (see example calculations in Appendix G). Likelihood Score for this pump station is 5.0.
9. Determine the Equity Score based on SPU's mapping data. The pump station falls within a "medium" level of disadvantage; assign an Equity Score of 3.0.
10. Calculate the Risk Score:

$$Risk\ Score = (Consequence\ Score \times Likelihood\ Score) + Equity\ Score$$

$$Risk\ Score = (1.5 \times 5.0) + 3.0 = 10.5$$

Example Calculation: CSO 24

Calculate the Seismic Risk Score for CSO 24 using the following steps:

1. Refer to Table 6-4 for consequence scoring criteria and attributed component scores.
2. Determine the component score for capacity based on storage volume. Storage volume is less than 1,000,000 gallons; assign a score of 0.5.
3. Check whether the CSO facility falls within a high-use area based on SPU's mapping data. The CSO facility does not fall within a high-use area; assign a component score of 0.0.
4. Determine whether the CSO facility serves a critical facility. The CSO facility does not serve a critical facility; assign a component score of 0.0.
5. No component scores for major transportation route are used for major CSO facilities; assign a component score of 0.0.
6. Determine whether CSO facility overflows to a nearby waterbody. Assign a component score of 1.0 for environmental impact.
7. Calculate the Consequence Score by summing the above component scores:

$$\begin{aligned} Consequence\ Score \\ &= Capacity\ Score + High\text{-}use\ Area\ Score + Critical\ Facility\ Score \\ &+ Major\ Transportation\ Route\ Score + Environmental\ Impact\ Score \end{aligned}$$

$$Consequence\ Score = 0.5 + 0.0 + 1.0 + 0.0 + 1.0 = 2.5$$

8. Determine the Likelihood Score based on the likelihood of failure assessments described in Section 5 (see example calculations in Appendix H). Likelihood Score for this CSO facility is 3.
9. Determine the Equity Score based on SPU's mapping data. The CSO facility falls within a "low" level of disadvantage; assign an Equity Score of 1.0.
10. Calculate the Risk Score:

$$Risk\ Score = (Consequence\ Score \times Likelihood\ Score) + Equity\ Score$$

$$Risk\ Score = (2.5 \times 4.0) + 1.0 = 11.0$$

Appendix M: Seismic Risk Scoring Results

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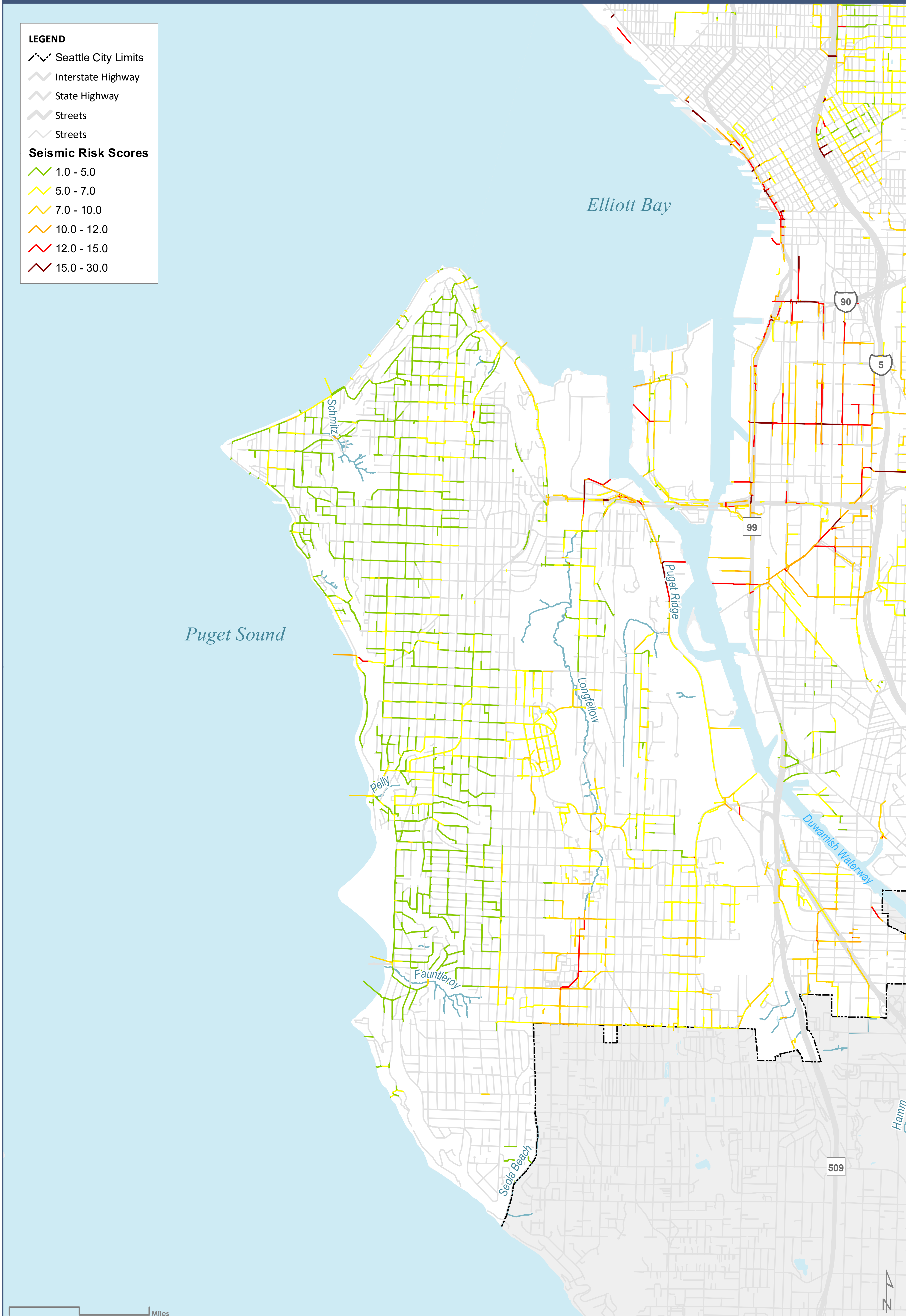
Map M-1: Risk Scores for Drainage Mainlines, Southwest



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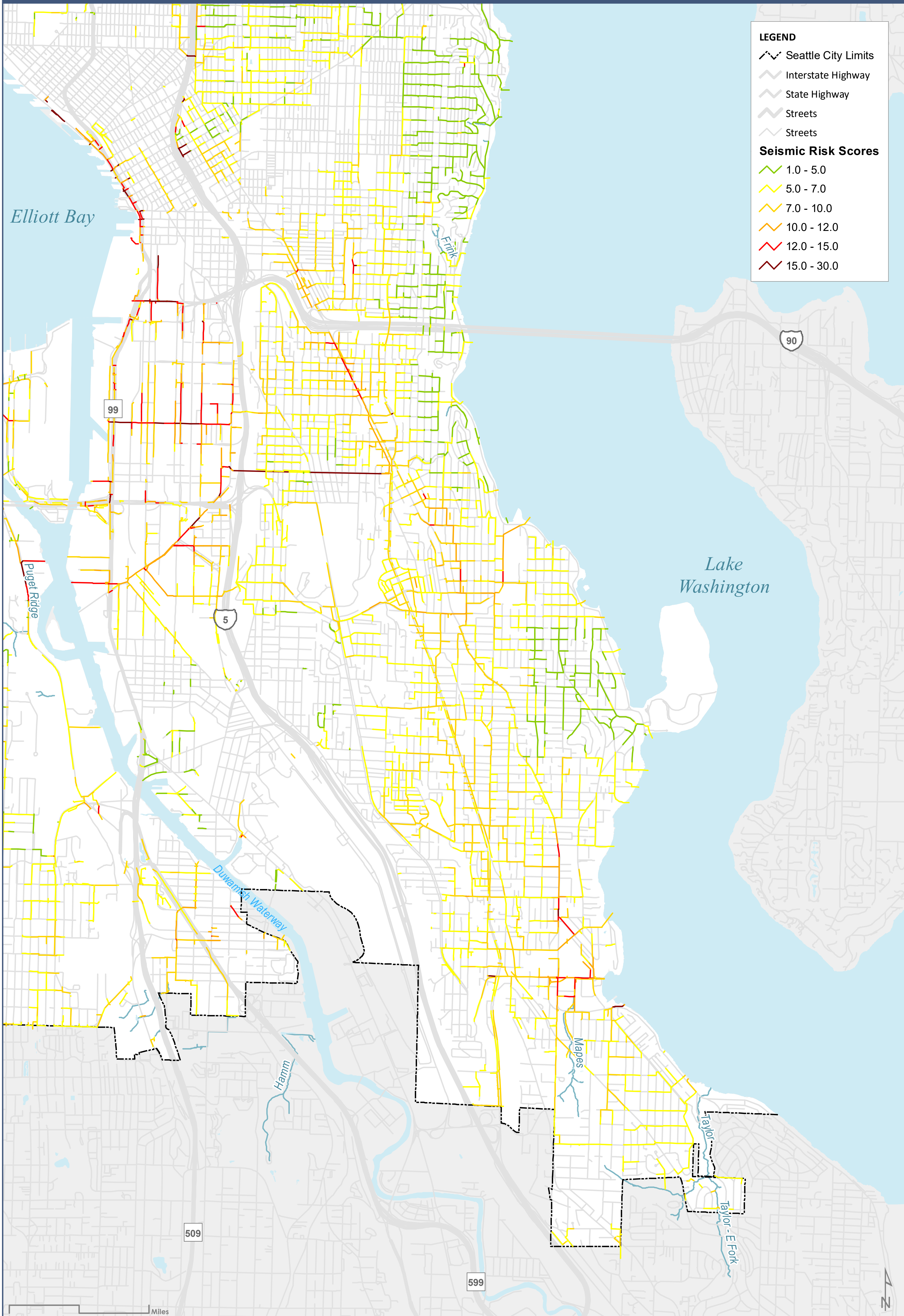
Map M-2: Risk Scores for Drainage Mainlines, Southeast



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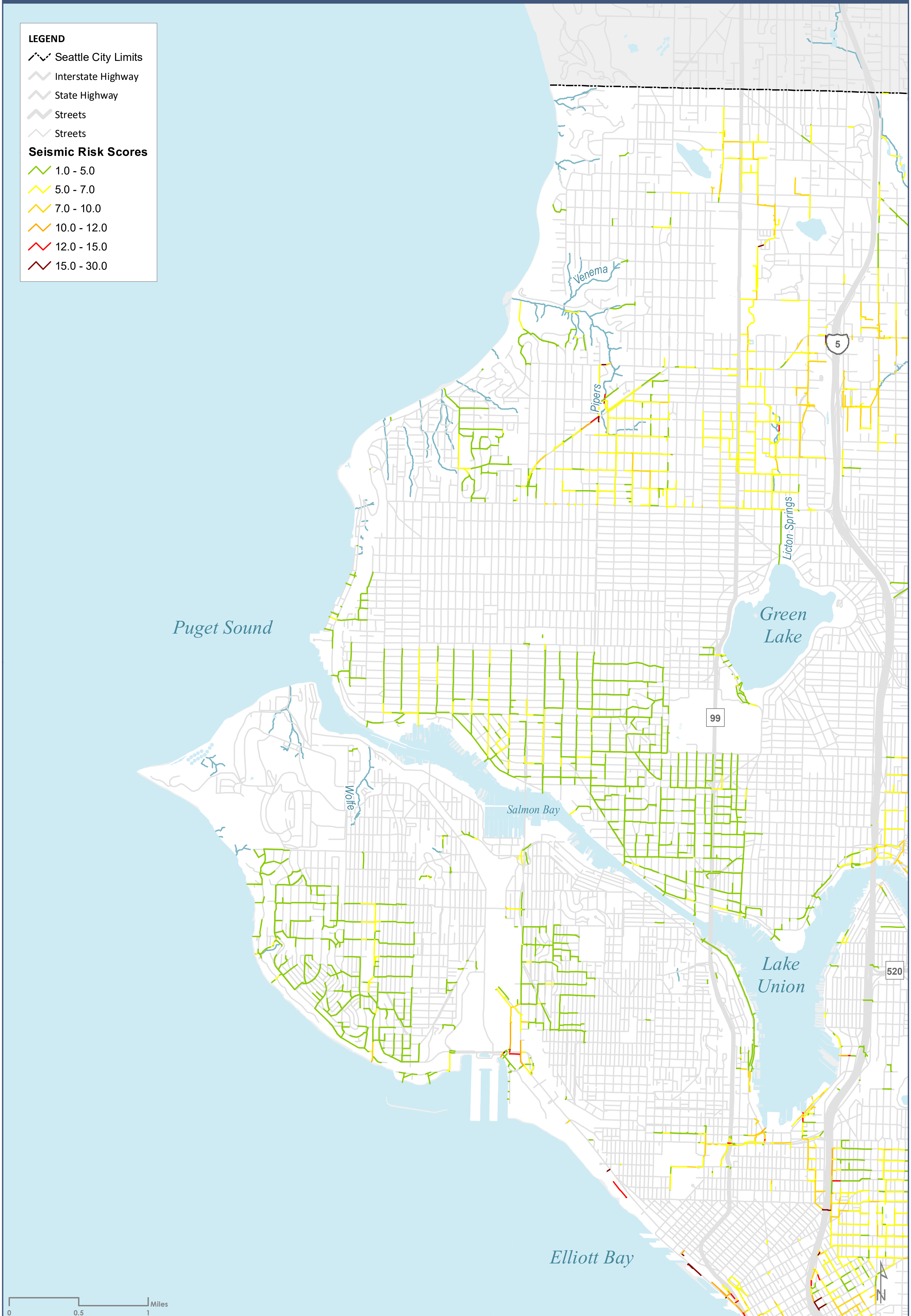
Map M-3: Risk Scores for Drainage Mainlines, Northwest



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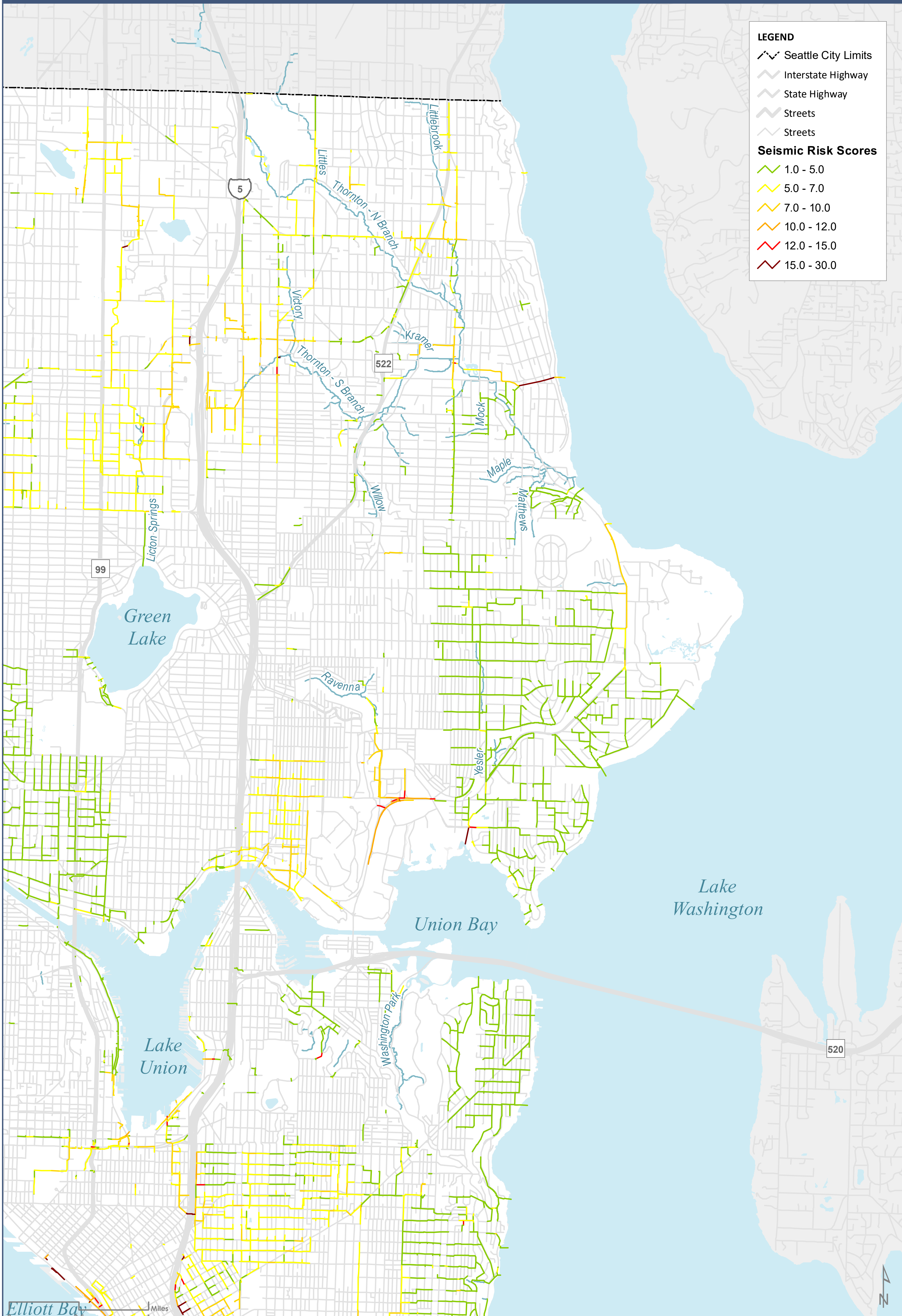
Map M-4: Risk Scores for Drainage Mainlines, Northeast



Author: Fill out Map Document Properties Date: 1/6/2022 File Path: \\bc\corp\PW_Exports\154002 - SPU Shape Our Water\Task04\GIS\MapDocs\SOW_T4_MapM4-NE(rev).mxd

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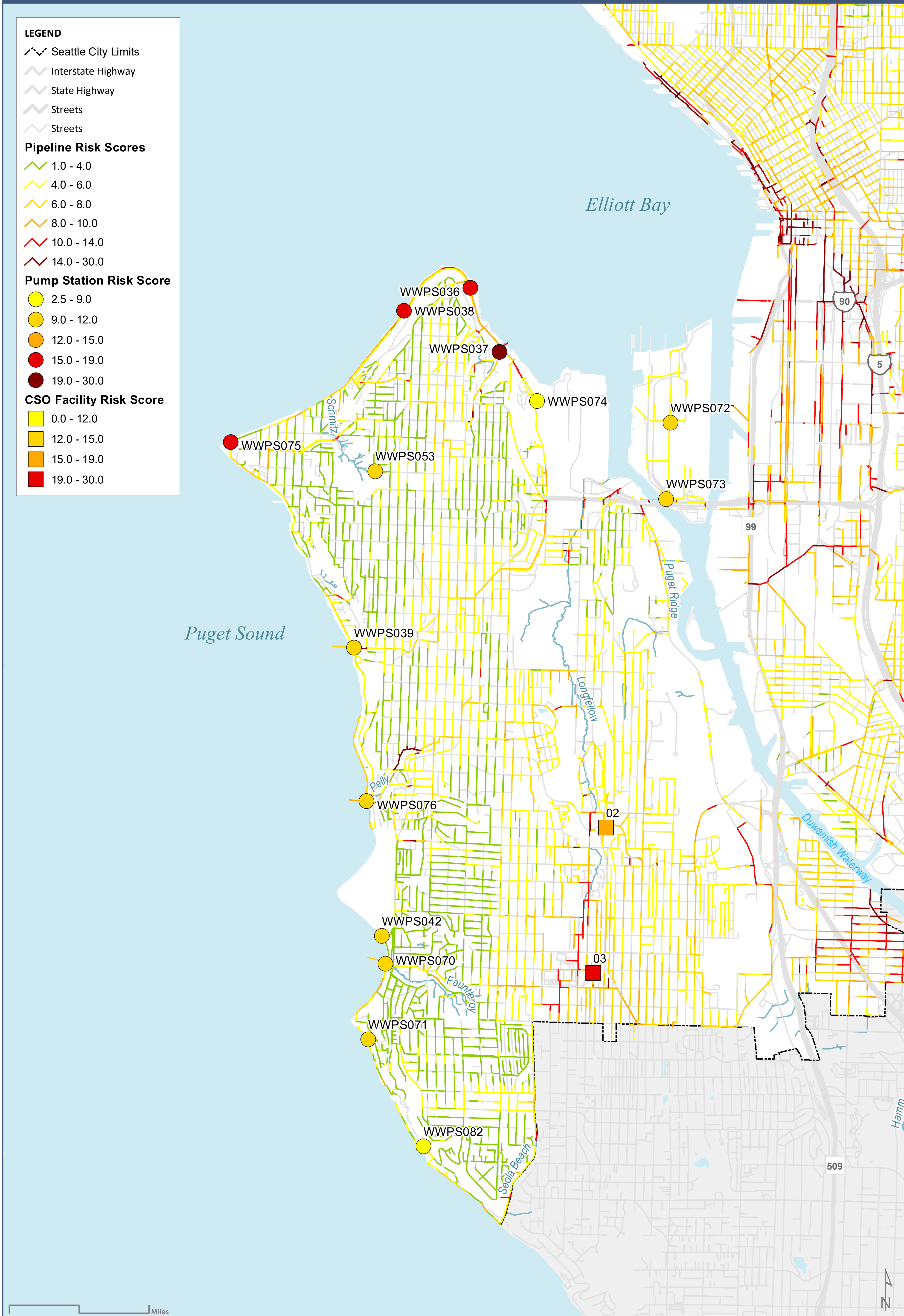
Map M-5: Risk Scores for Wastewater System, Southwest



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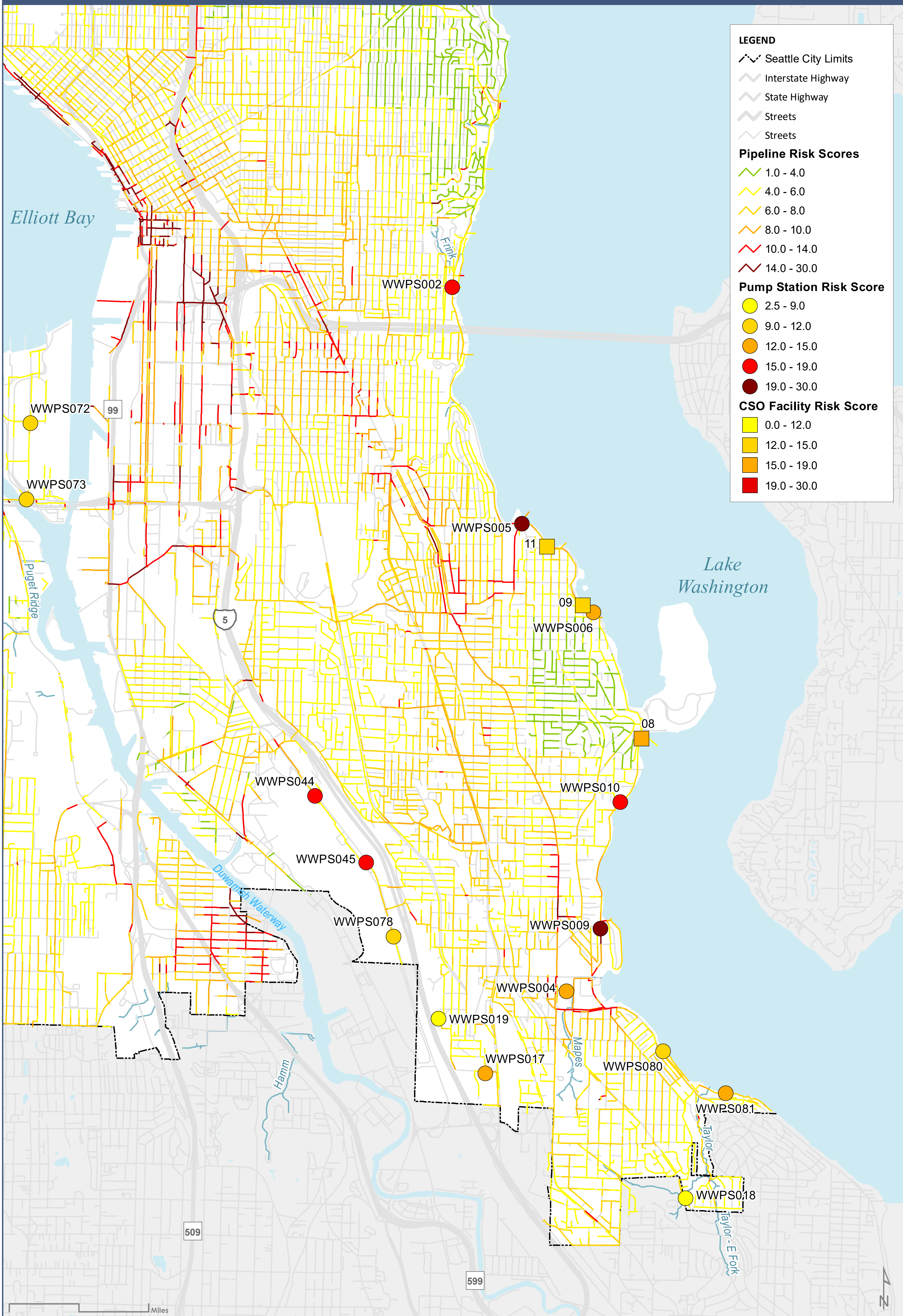
Map M-6: Risk Scores for Wastewater System, Southeast



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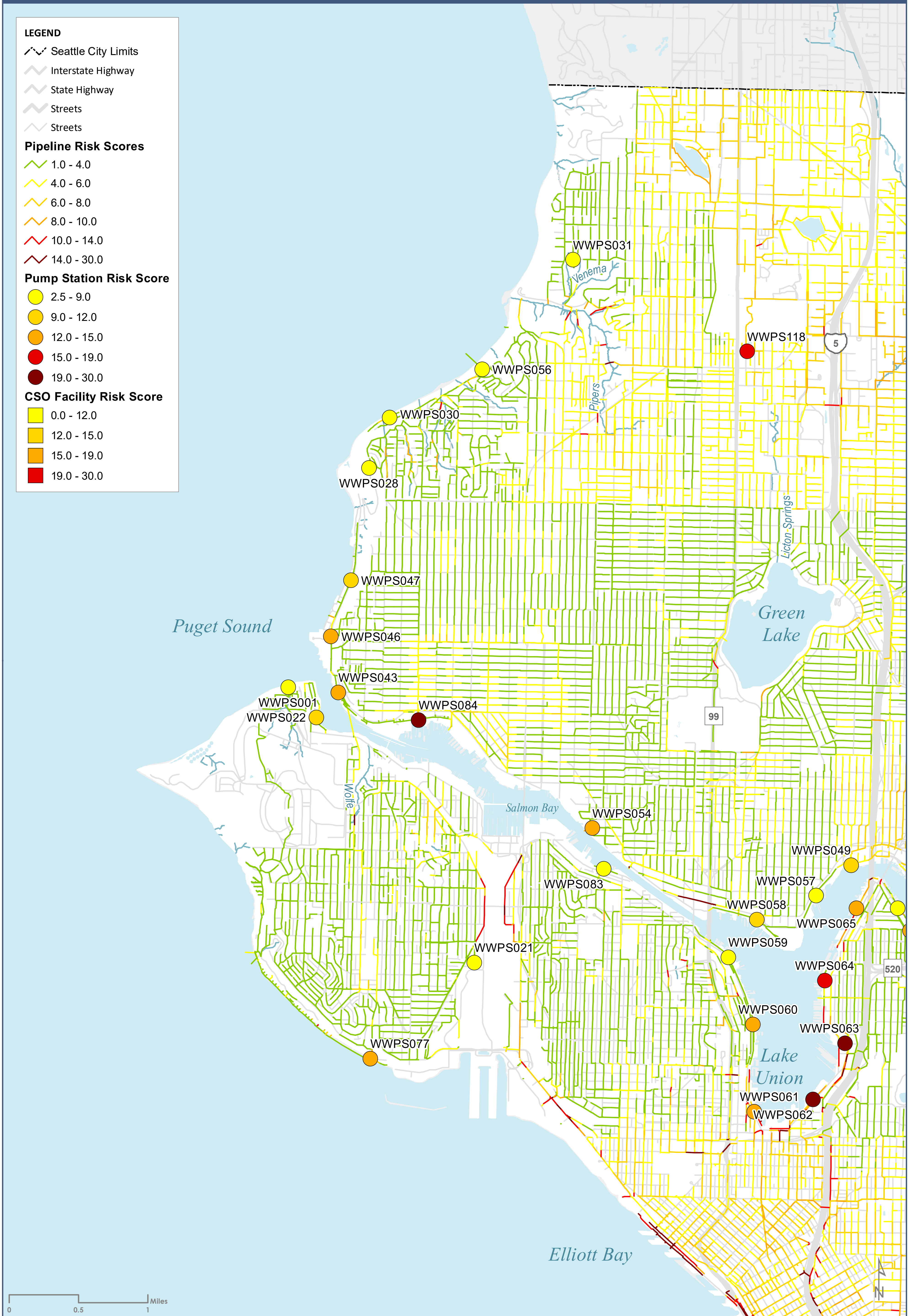
Map M-7: Risk Scores for Wastewater System, Northwest



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Map M-8: Risk Scores for Wastewater System, Northeast