Albany Water System

Prepared for:

City of Albany

by:

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1.0 Introduction and Summary

This report examines the seismic performance of the City of Albany's water system. The report is divided into the following sections.

Section 2 describes the major components of the Albany Water System.

Section 3 describes the seismic hazards for the Albany Water System.

Section 4 describes the expected performance of the Albany Water System in its as-is condition, given earthquakes that are likely to occur in the planning horizon. Damage is considered for the facilities at the Vine Street Water Treatment Plant, as well as to pipes in the distribution system.

Section 5 describes the specific weaknesses of the specific buildings, tanks and equipment at the Vine Street Water Treatment Plant. Costs to upgrade the various facilities are provided.

Section 6 provides the fragility information associated with the recommended seismic upgrade program.

Section 7 provides references.

A separate report and evaluation by Goettel and Associates Inc. uses the information in this report to perform a comprehensive Benefit Cost Analysis of the recommended seismic upgrade program.

1.1 Abbreviations

The following summarizes the abbreviations and terminology used in this report.

Abbreviations

BPA	Bonneville Power Administration
CMU	Concrete Masonry Unit
g	acceleration of gravity (=32.2 feet / second / second)
G&E	G&E Engineering Systems Inc.
GAI	Goettel and Associates Inc.
gpm	gallons per minute
HP	Horsepower
kips	kilo-pounds (1,000 pounds)
MG	Million Gallons
MGD	Million Gallons per Day
PGA	Peak Ground Acceleration (measured in g)
PGD	Permanent Ground Deformations (measured in inches)
PP&L	Pacific Power & Light
psi	Pounds per square inch
URM	Unreinforced Masonry
	(clay brick walls with straight timber roof sheathing)
WTP	Water Treatment Plant

Terminology

The following terminology is used in this report.

1.2 Acknowledgment

This report was written by John Eidinger of G&E Engineering Systems Inc. Support for the walkdown was provided by Dr. Donald Duggan of G&E and Dr. Ken Goettel of GAI. Mr. Ted Mikowski of the Public Works Department of the City of Albany provided information, access and graciously supported the effort in every instance.

1.3 Limitations

All costs presented in this report represent engineering planning estimates, and are in year 2007 dollars. The costs do not include escalation beyond 2007 or the costs of borrowing.

2.0 City of Albany Water System

2.1 History of Water System

The City of Albany, Oregon was founded in 1848. The City is located adjacent to the Willamette River immediately east of its confluence with the Calapooia River, see Figure 2-1.

The Vine Street Water Treatment Plant (WTP) historically provided essentially all the potable water for the entire water system. It is located near the middle of the water system service area, at longitude 123.115°, latitude 43.6357°. In November 2005, the new Albany-Millersburg WTP was put on line, so that now there are two WTPs serving Albany. The Vine Street WTP remains the larger of the two, with a summer time capacity over 20 MGD. The Albany-Millersburg WTP is smaller, with a dedicated capacity of 10 MGD for Albany. The impact of the new WTP has been incorporated into the overall performance of the water system in its as-is and recommended upgraded conditions, as well as in the benefit cost analyses performed by GAI.

The Vine Street WTP gets its raw water via the Santiam-Albany Canal. The Canal is about 18 miles long, and gets its water at the South Santiam River just east of the City of Lebanon. The Canal was built in 1872 for the transportation of goods. In 1892, a hydroelectric facility was brought on line, and in 1912 the current WTP was constructed adjacent to the hydroelectric turbines. Pacific Power & Light owned the hydroelectric power plant, the Canal and the water distribution system through 1984.

The City of Albany bought all the facilities of the water system from Pacific Power and Light (PP&L) in 1984. Today, the City provides water to the City of Albany, the adjacent City of Millersburg and the areas previously served by the North Albany County Service District.

The City today provides water to residential, commercial, industrial and public customers. The service area is highlighted in Figure 2-1. The population in the area is 40,852 (City of Albany only, year 2000 census), and is increasing annually in the range of about 2.5%. The service area population is expected to be about 53,200 people (year 2020), reaching ultimate building of 109,000 people (year 2074).

Average daily demand ranged from 7.5 MGD to 8.2 MGD between 1994 and 2000, with minimum winter monthly average of about 6.7 MGD. At buildout, average daily demand is expected to increase to 22.4 MGD (year 2074). By the year 2020, average daily demand is expected to be 10.1 MGD.

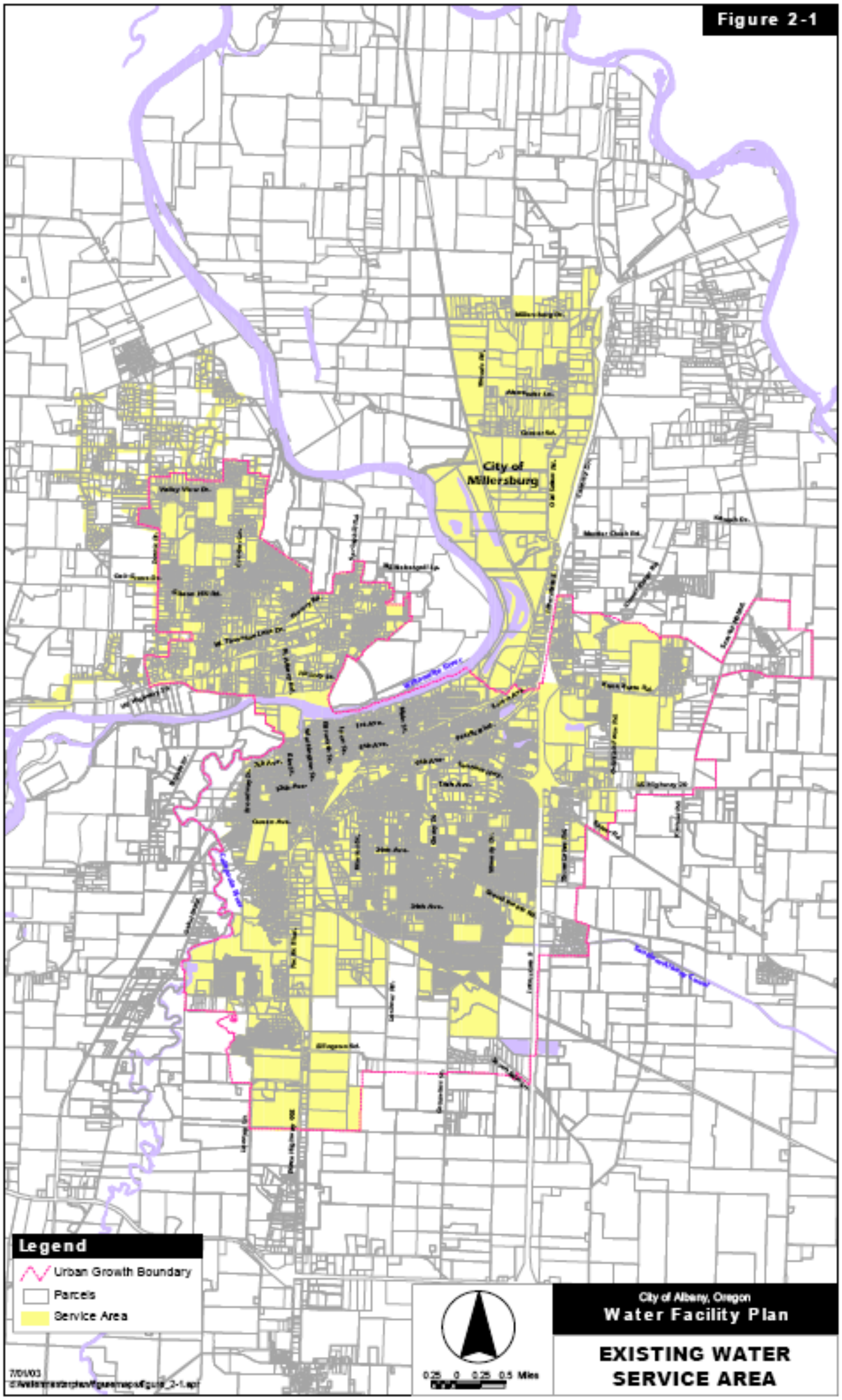
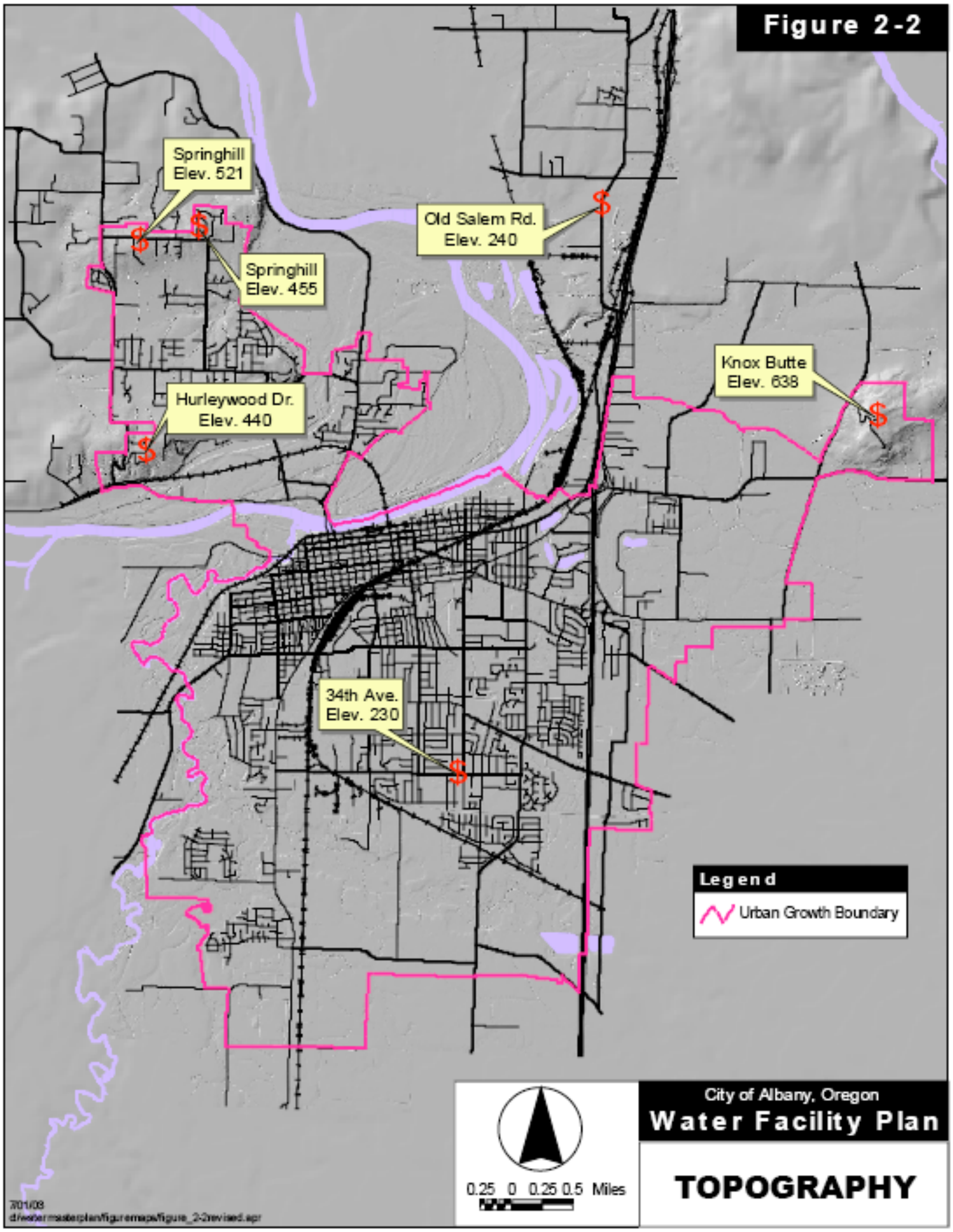


Figure 2-1
City of Millersburg
Legend
Urban Growth Boundary
Parcels
Service Area
7/01/03
0.25 0 0.25 0.5 Miles
City of Albany, Oregon
Water Facility Plan
EXISTING WATER
SERVICE AREA



2.2 Operation of the Water System

A diversion dam located about 1 miles east of Lebanon on the South Santiam River diverts water into the Santiam-Albany Canal. The Canal is 18 miles long, flowing by gravity northwestward to its terminus adjacent to the Wine Street WTP in Albany. Currently, this Canal is the only water supply for Albany.

Today, the water system is operated in three main pressure zones. Raw water enters the Vine Street WTP from the Canal and is then treated. Treated water is stored in the clearwell tank at the WTP, from which it is pumped into Zone 1. Zone 1 covers about 12,500 acres of land, serving all of downtown Albany, as well as residential and commercial and industrial areas south and east of downtown area. Pump stations are used to pump water from Zone 1 up to Zones 2 and 3.

Zone 1 is comprised of industrial, commercial and residential customers. It is served by the High Service pump station (15,950 gpm, built 1959) located at the WTP. Local in-zone storage is provided by the Queen (1 MG), 34th Avenue (2 MG) and Broadway (8 MG) reservoirs. The Queen and 34th Avenue reservoirs are located at grade, at the elevation of Zone 1, so water enters these tanks via system pressure provided by the High Service pump station; overflows are prevented by altitude valves and related controls. The water in the Queen and 34th Avenue tanks can then re-enter Zone 1 by pump stations located adjacent to the tanks (Queen 500 gpm, built 1955, 34th Avenue 2,800 gpm, built 1971). The Broadway tank (overflow 385 feet, base elevation 345 feet) is located at a hill in North Albany, and provides water into Zone 1 by gravity flow.

Zone 2 is almost entirely comprised of residential customers and is served by the Wildwood Reservoir (1.15 MG, overflow 450 feet, base elevation 430 feet) and the North Albany Pump Station (1,400 gpm, built 1980). It ranges in elevation from 230 to 350 feet. The North Albany pump station takes suction from Zone 1.

Zone 3 is almost entirely comprised of residential customers and is served by the Valley View reservoirs (3 tanks, 1.35 MG total, overflow 568 feet, base elevation 520 feet) and the Gibson Hill Road Pump Station (900 gpm, built 1998). Zone 3 has elevations from 350 to 510 feet.

A topographic relief map showing the general areas is shown in Figure 2-2. The new Albany-Millersburg WTP is located in the far east corner of Figure 2-2, drawing water from the Santiam River.

2.3 Vine Street Water Treatment Plant

The Vine Street WTP was built in 1912. An aerial view of the plant is shown in Figure 2-3. It has gone through a number of construction phases:

- In 1912, the WTP was originally built, consisting of two settling basins and six filter beds. No seismic design features are evident in the 1912 construction, with most above ground facilities constructed of unreinforced masonry (brick). Water retention structures (tanks) and some building foundations were built with reinforced concrete.
- A new raw water pump station, flocculator and clarifier were added in 1948.
- One of the sedimentation basins was converted to two filters in the mid-1960s.

- A solids contact basin and backwash ponds were added in the 1970s.
- The WTP was expanded in 1991 to increase plant capacity from 15 MGD to 20 MGD. Various systems were modified at this time to comply with water quality requirements. Two raw water pumps were added, tube settlers were installed in Clarifier #1[1], one of the settling tanks was converted into a Clarifier 2 (also with tube settlers); addition of Filters 9 and 10, and addition of a Hypalon baffle into the Maple Street Reservoir (clearwell).
- In 1995, outlet modifications were made to the backwash / sludge holding lagoons and a second drying bed was added.

A review of the drawings for the 1948, 1960s, 1970s and 1991 upgrades reveals that none were designed with significant seismic capacity in mind. The 1991 upgrade shows seismic design per UBC Zone 2; but details suggest that detailing was governed by water leak tightness, and not seismic loads.

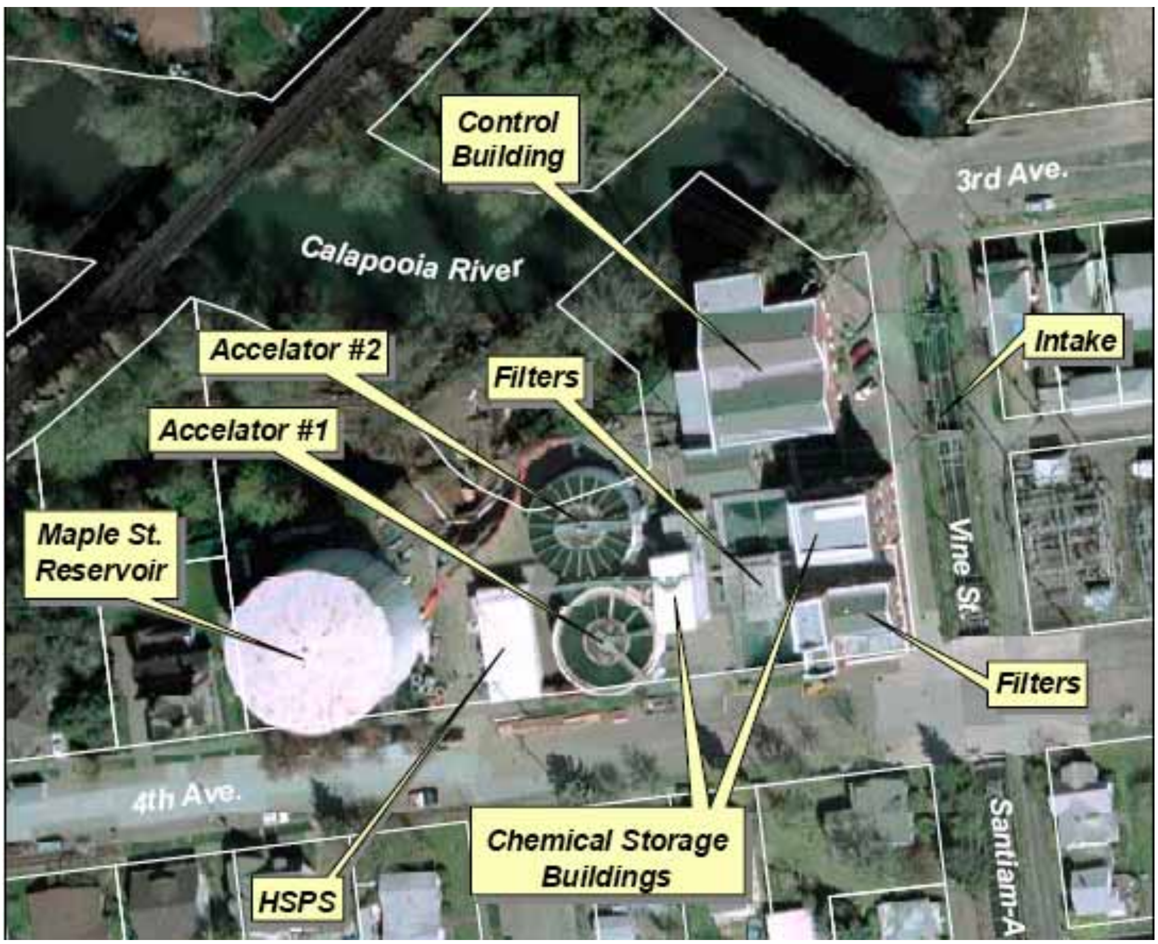


Figure 2-3. Aerial View of Vine Street WTP

[1] Clarifier 1 is sometimes also called Accelator #1

Historically, the maximum day flow produced by the Vine Street WTP has been 16 MGD.

There are nine raw water pumps, ranging in size from 50 HP to 100 HP. The firm capacity of the combined raw water pumps, assuming the largest pump out of service, is 21,000 gpm (30.2 MGD).

Clarifier 1 is 58 feet in diameter, with storage volume 295,000 gallons. It is divided in two portions. The central portion (82,300 gallons) serves as the flocculation zone. The outer portion (212,700 gallons) serves as the clarification zone.

Clarifier 2 is 60 feet in diameter, with storage volume 332,000 gallons. It is divided in two portions. The central portion (110,670 gallons) serves as the flocculation zone. The outer portion (221,330 gallons) serves as the clarification zone.

There are 6 small filters and 4 large filters, built as reinforced concrete boxes. The small filters are 10.5 x 20.3 feet in plan. The large filters are 15.3 x 32 feet in plan. The filters are layered with gravel, sand, silica sand and anthracite. Surface wash is by rotating spray. There are two backwash pumps.

There is a buried concrete tank that serves as a clearwell. It has capacity 292,000 gallons. It is 50.5 feet x 62 feet in plan, 14 feet deep.

There are two settling ponds and two sludge drying beds.

There are four low lift pumps at the plant, with rated capacity of 22.2 MGD (with largest pump out of service). These pumps move the water through the plant.

There are three chlorine ton cylinders.

There is a 5,000 gallon alum tank.

Dry chemical storage includes soda ash, polymer, fluoride and carbon.

The plant has a lime hopper, but it is no longer in service.

There are two polymer mix tanks.

The High Service pump station has five pumps, with rated capacity of 23 MGD.

The Maple Street reservoir is a 2 MG steel tank, 93 feet in diameter, 40 feet high, built in 1960. It is built on a concrete ring beam. It is unanchored. It has an internal hypalon baffle (function: to promote mixing, but recent experience suggests that the baffle is non-functional). There is one 24-inch diameter inlet pipe and two 24-inch outlet pipes. This tank is used as a chlorine contact tank.

There are four chlorine analyzers at the plant.

2.4 Canal

The Canal delivers raw water from east of Lebanon to the Vine Street WTP. Twelve miles of the canal were originally built in 1872. In 1891-1892, an additional six miles was constructed. The canal was reconfigured in 1921, and it remains much the same today as it was in 1921.

The Canal originates on the South Santiam River 325 feet upstream of the Lebanon Dam located two miles southeast of Lebanon. Over its 18 mile length, it drops about 170 feet in elevation. The Canal varies from about 20 to 40 feet in width with steep side slopes for its entire length. It has 12 flow control structures along its length.

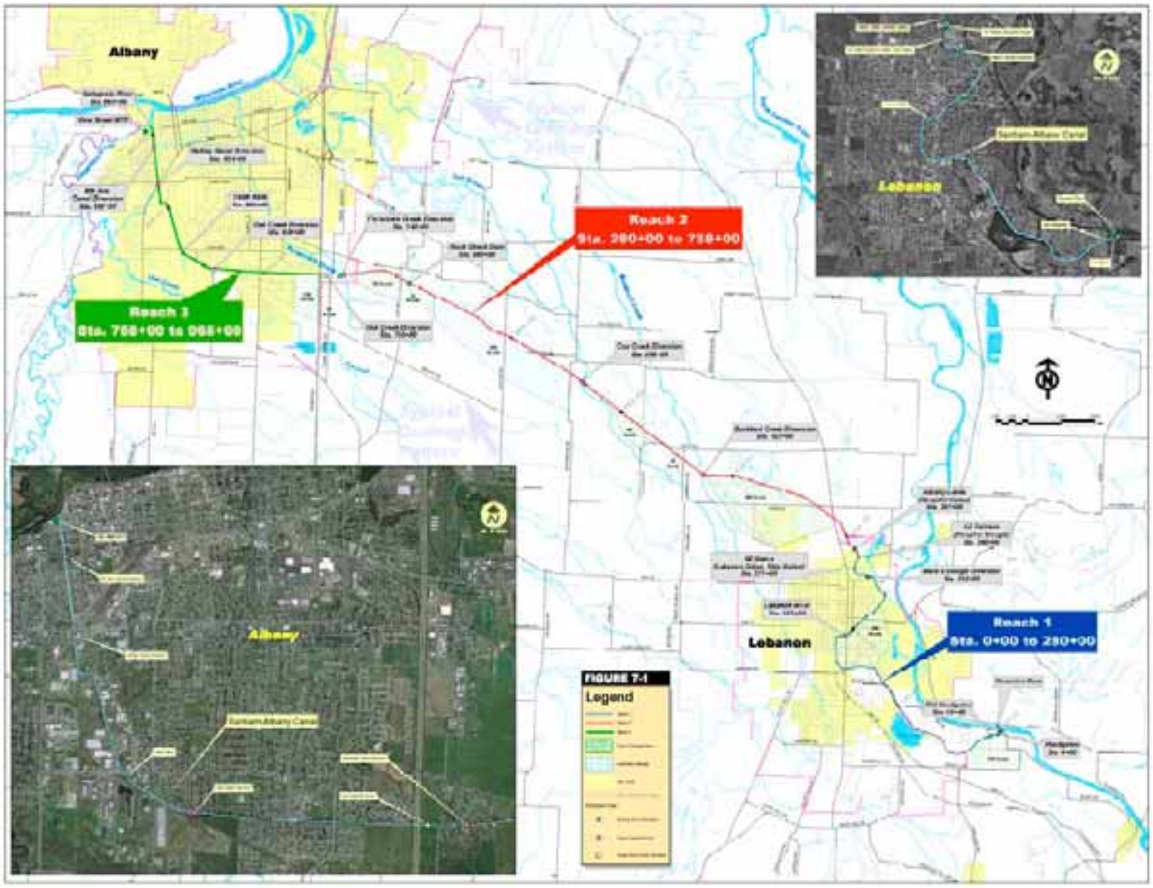


Figure 2-4. Map of the Canal Alignment

The Canal historically (though November 2005) served as Albany's sole source of drinking water and will continue to be relied upon as a source of drinking water in the future. Thus, protecting the Canal in earthquakes is a consideration in developing a suitable seismic mitigation program for the City of Albany Water System, although with the new Albany-Millersburg WTP, no Canal-specific seismic upgrades are recommended in this report.

Hazards that might impact the Canal after earthquakes include:

- Failure of bridge and culvert crossings. Any such failure could result in partial or full blockage of water supply; introduction of contaminants / turbidity spikes into the water that would impact treatment processes possibly making it impossible to (at least for some time) provide potable water. Partial or complete blockage of the Canal could also lead to flooding.

- Unstable banks could slide into the Canal, resulting in blockage and/or turbidity events.

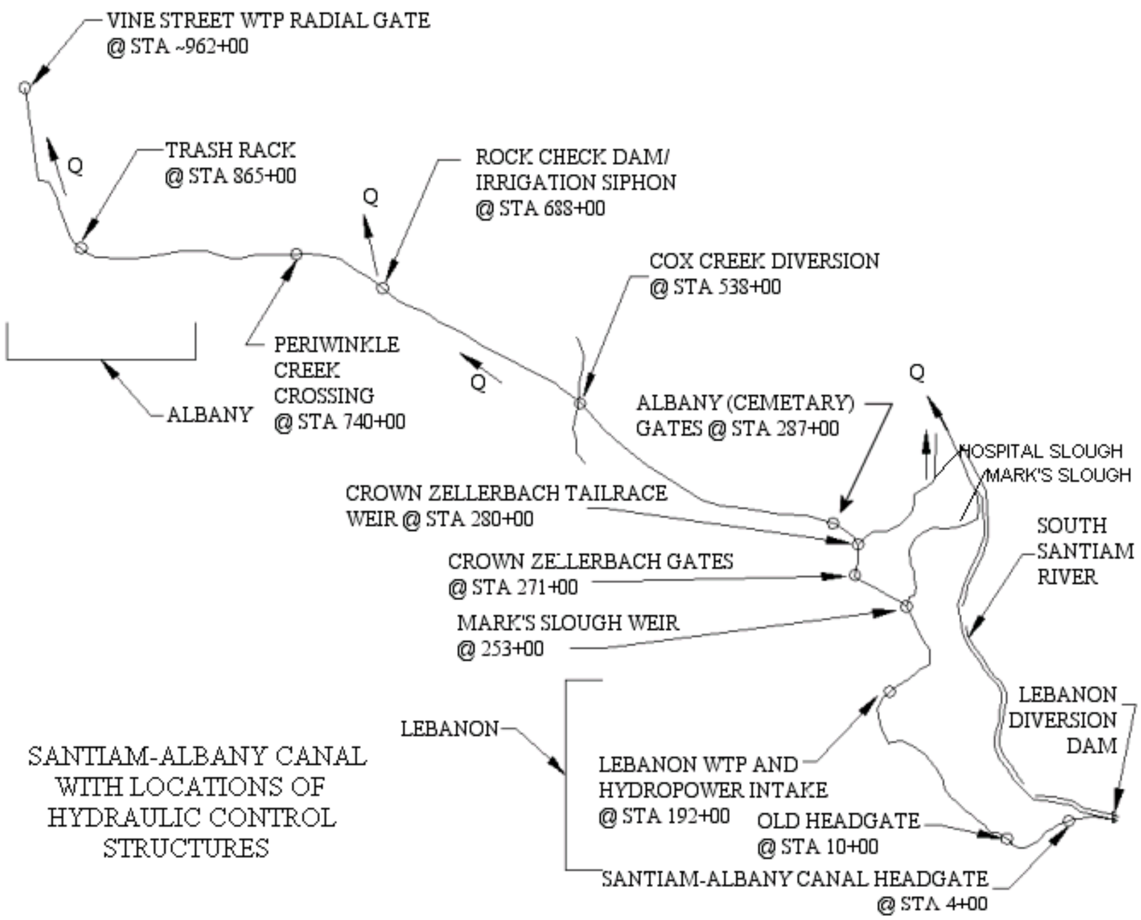


Figure 2-5. Major Features Along the Canal Alignment

The Lebanon Diversion Dam was constructed in 1925. The dam spans 435 feet across the South Santiam River. The Dam serves to locally raise the elevation of the river by 5 to 7 feet to a height sufficient for water to enter the Canal, some 325 feet upstream of the dam. The Canal entrance is unscreened, protected by a log boom to limit surface debris from entering the Canal.

Under moderate levels of shaking, the log boom system may be damaged. This should not impact the Canal. The cantilever-style concrete dam will likely be sufficient except under strong ground shaking; should it be breached, there will be loss of water supply entering the Canal.

The Canal headgate at station 4+00 was built in 1924. It is a concrete structure with four manually-operated sluice gates used to control the flow into the Canal. It has large wing walls, and the main gates are 20 feet wide and about 20 feet tall. The structure visually appears adequate to withstand moderate levels of shaking. Geologic stability above PGA = 0.25g or so is uncertain.

The Old Headgate (station 10+00) was built in the 1890s. It is a concrete structure, about 20 feet high, spanning 26 feet across the Canal, with wing walls making the entire width about 60 feet. The original sluice gates have been removed, and it is no longer used as a control structure; but the concrete structure remains.

2.5 Distribution Pipelines

There are about 228 miles of pipeline in the Albany distribution system. Pipe materials include asbestos cement (44%), cast iron (9%), wrought iron (0.9%), ductile iron (37%), galvanized (0.2%), PVC (1.2%), steel (lined and coated, 3.4%), steel (bare, 4.6%). The steel pipes are mostly smaller than 4 inches in diameter, are about 100 years old, and have had the majority of the water leaks in the system.

The oldest pipes date to the 1890s. Installation by decade is:

Decade	Percent of Total
1890s	0.1%
1900s	0.3%
1910s	0.8%
1920s	0.3%
1930s	0.5%
1940s	4.6%
1950s	8.1%
1960s	22.0%
1970s	20.8%
1980s	12.5%
1990s	26.3%
2000 - 2004	3.4%

Table 2-1. Installation Decade for Distribution Pipelines

There are 28 miles of pipe 16-inches in diameter or larger. There are 7.6 miles of pipe 24 inches in diameter or larger. There are 11 miles of pipe smaller than 4 inches in diameter (mostly steel).

The small diameter steel pipes (under 4-inches, ~8 miles) are strongly suspected as being the source of high leak rate in the existing distribution system. The City would like to replace these 8 miles of pipe, to address both the leak rate and increase pipe diameter size to address fire flows, but the cost and benefits of such replacement are not addressed in this report.

2.6 Pump Stations

The High service Pump Station is located at the Vine Street WTP. The pumps are located in a reinforced masonry building built in 1959. The building is rectangular in plan, with several window and garage door openings. It does not have emergency backup power or quick connect coupling.

The 34th Avenue pump station is located in a small rectangular reinforced masonry (CMU) building, built in 1971. The building has doors on one side, and solid walls elsewhere. It does not have emergency backup power or quick connect coupling.

Figure 2-6. 34th Avenue Pump Station

The Queen Avenue pump station is located in a small rectangular reinforced masonry (CMU) building, built in 1955. The building has doors on one side, and windows in other walls. It does not have emergency backup power or quick connect coupling.

Figure 2-7. Queen Avenue Pump Station

The North Albany pump station is located in a small rectangular reinforced masonry (brick) building, built in 1998. The building has a large door on one side, and solid walls on the other walls. It does not have emergency backup power, but does have a quick connect coupling.

Figure 2-8. North Albany Pump Station

The Gibson Hill pump station is located in a small rectangular wood frame building with wood roof, built in 1998. The building has a large door on one side, and sold walls on other walls. It does not have emergency backup power, but does have a quick connect coupling.

Figure 2-9. Gibson Hill Pump Station

2.7 Reservoirs

The City has 8 reservoirs, 7 used in the distribution system, and one at the WTP (Maple Street) that used as part of the treatment process. The 7 distribution reservoirs have total storage volume of 13.5 MG.

The Maple Street tank is a 2 MG, at grade steel tank. It rests on a concrete ring girder, unanchored. It has 1 inlet and 2 outlet pipes, an overflow pipe and a drain pipe (floor entry near wall). It has diameter 93 feet, bottom elevation 220 feet, overflow elevation 259.8 feet.

Figure 2-10. Maple Street Tank

The Queen Avenue tank is a 1 MG at grade steel tank, unanchored, built in 1955. It rests on a concrete ring girder. It has diameter 74 feet, bottom elevation 229 feet, overflow elevation 260.5 feet. It has one inlet/outlet pipe, side entry. It has an overflow pipe connected to the drain line.

Figure 2-12. Queen Avenue Tank

The 34th Avenue tank is a 2 MG at grade steel tank, unanchored, built in 1971. It rests on a concrete ring girder. It has diameter 104 feet, bottom elevation 224 feet, overflow elevation 255.5 feet. It has one inlet/outlet pipe, bottom entry. It has an overflow pipe and a drain line.

Figure 2-13. 34th Avenue Tank

The Broadway tank is a 8 MG at grade circular concrete tank, built in 1992 to AWWA seismic zone 3 requirements. It has diameter 210 feet, bottom elevation 345 feet, overflow elevation 385 feet. It has a common inlet/outlet pipe entering through the tank base, and then splitting into separate inlet and outlet nozzles, each with check valves.

Figure 2-14. Broadway Tank

The Wildwood tank is a 1.15 MG at grade concrete tank, built in 1999 to AWWA zone 3 requirements. It has diameter 140 feet, bottom elevation 430 feet, overflow elevation 450 feet. It has a common inlet/outlet pipe entering through the tank base, and then splitting into separate inlet and outlet nozzles, each with check valves.

Figure 2-15. Wildwood Tank

The Valley View tank 1 is a 0.25 MG at grade steel tank, unanchored, built in 1963. It has diameter 25 feet, bottom elevation 520 feet, overflow elevation 567.5 feet. There is a catwalk connecting between Tanks 1, 2 and 3 at this site. The inlet-outlet pipes are side entry.

The Valley View tank 2 is a 0.25 MG at grade steel tank, unanchored, built in 1967. It has diameter 25 feet, bottom elevation 520 feet, overflow elevation 567.5 feet. The inlet-outlet pipes are side entry.

The Valley View tank 3 is a 0.85 MG at grade steel tank, unanchored, built in 1982. It has diameter 55 feet, bottom elevation 520 feet, overflow elevation 569.5 feet. The inlet-outlet pipes are side entry.

Figure 2-16. Valley View Tanks

3.0 Seismic Hazards

The City of Albany is located within the Willamette River Basin. The City lies in an alluvial plain between the foothills of the Coast Range and the Cascade Range. The largest portion (12,500 acres) of the Albany distribution system is located south and east of the Willamette River, in an area with very little topographic relief, ranging in elevation from 175 feet to 275 feet.

There are two smaller zones in the water system with topgraphic relief. These areas are characterized as underlain by deposits of erosion-resistant sedimentary and volcanic intrusive rocks that form outcroppings such as Knox Butte (to elevation over 500 feet), Spring Hill and an elevated ridge in North Albany (to elevation over 600 feet).

3.1 Seismic Setting

The understanding of the seismic hazard for Albany has changed substantially over the past 20 years. Prior to 1980 (or so), The central regions of Oregon were considered to be largely aseismic. Studies suggested that the area of Albany could experience ground motions, as measured by Peak Ground acceleration, of about 0.05g once every 500 years or so. At this very low level of shaking, damage is uncommon or limited even to poorly constructed buildings, and in all practicality, local building codes excluded seismic forces as a design requirement; or if required, set at such a low level that wind-level design forces would exceed those from seismic.

Studies of the seismicity of the Pacific Northwest suggest that there are three sources of earthquakes for the Albany area. Figure 3-1 shows a map of the Pacific Northwest, highlighted the Cascadia Subduction Zone. Figures 3-2 and 3-3 show a three dimension view of the earth's crust and cross section, highlighting the three different sources of earthquakes that affect the area. Figures 3-4 and 3-5 provide a more close up map of Oregon, showing the historic and instrumentally recorded earthquakes in northern Oregon. From these figures, the seismic sources are grouped into three types:

- Cascadia subduction zone (interplate) earthquakes. A great subduction magnitude (M) 8 to 9 earthquake under the Pacific Ocean, near the Oregon coastline about 80 km west of Albany, can cause strong shaking of perhaps PGA = 0.20 to 0.35g for 60 to 90 seconds in Albany.

- Deep intraplate earthquakes. A very large M 7 to 7.5 earthquake within the subducting plate (so-called intraplate) could occur directly beneath Albany, but at substantial depth of possibly 40 to 50 km. While smaller in magnitude than a great interplate subduction earthquake, this event will still cause ground shaking in the range of PGA = 0.20 to 0.35g in Albany. This event is considered much less likely to occur near Albany as would be the case near Seattle.

- Crustal earthquakes . A moderate M 5.5 to 6.5 earthquake can occur in the top 10 to 20 km of the earth's crust, rather near Albany. Such faults include the Corvallis fault. While smaller in magnitude, this event can cause ground shaking in the range of PGA = 0.10 to 0.60g (or higher) in Albany, depending upon exactly where the earthquake occurs.

Subduction (Interplate) Zone

The area just offshore, where the Cascadia fault is near the surface of the Earth, is called the Cascadia subduction zone. Now, as at most times, there is little slip on the Cascadia fault in the subduction zone. Eastward motion of the Juan de Fuca plate is absorbed by compression of the North American plate. Records provided by buried soil layers, dead trees, and deep-sea deposits indicate to geologists that the Cascadia fault ruptures and releases this compression in large-magnitude 8 to 9-earthquakes about every 500-600 years. It is the upper portion of the shallowly dipping Cascadia fault that ruptures during these events; most of the rupture area is offshore. The last such earthquake occurred on January 26, 1700.

In Figure 3-3, the interplate earthquakes would occur at the area called "locked zone of interface"; the intraplate earthquakes would occur anywhere along the sloping lines, and the crustal earthquakes would occur at relatively shallow depths.

When the Cascadia fault ruptures, it will likely cause: 1) Severe ground motions along the coast, with shaking in excess of 1 g in many locations (1 g is equal to the acceleration of gravity, 0.5 g is half the acceleration of gravity). At "average" sites, the greater Portland area will most likely see 0.25 to 0.30 g accelerations from a subduction-zone earthquake; but there can be great variations in motions at nearby sites; but locally, ground motions could often (about 16% of the time) be considerably higher than the average. 2) Because of the very large fault area involved, slip will produce strong motions that may last for two to four minutes as the earthquake propagates along the fault, and include seismic waves of very long period (20 seconds or more). These long-period waves may particularly affect very tall structures, long structures such as bridges, and flexible conductors between transmission towers. 3) Tsunamis generated by sudden uplift of the sea floor above the fault. Effects of past tsunamis are among the evidence observed by geologists to infer the history of earthquakes in the subduction zone. 4) Effects in all of Cascadia's major population centers, from Vancouver Canada to as far south as Eureka California.

To capture the near likely upper bound of the Interplate earthquake, we selected a CSZ M 9 earthquake. This earthquake would rupture from Vancouver Canada in the north to near Eureka California in the south (Figure 3-12).

Benioff (Deep) Zone

As the Juan de Fuca plate subducts beneath North America, it becomes denser than the surrounding mantle rocks and breaks apart under its own weight, causing Benioff zone earthquakes. Beneath Western Oregon the Juan de Fuca plate reaches a depth of 40-60 km and begins to bend even more steeply downward, forming a "knee" (Figures 3-2, 3-3). It is at this knee where the largest Benioff zone earthquakes occur: both the 1949 event near Olympia (southwest of Tacoma) and the 1965 event near the Seattle-Tacoma International Airport occurred at the knee. Benioff zone earthquakes as large as magnitude 7.5 can be expected under western Oregon, but possibly not as likely as an offshore interplate earthquake, nor as likely as observed in the Puget Sound area.

Like subduction earthquakes, Benioff zone earthquakes have several distinctive characteristics. First, because they occur at depths of 40 kilometers or more, some high frequency energy becomes attenuated. At "average" hard rock sites, peak ground accelerations are about 0.20 to 0.25g, but locally, ground motions could often (about 16% of the time) be substantially higher than the average. Second, they tend to be felt over much broader areas than a shallow earthquakes of comparable magnitude.

To capture the near likely upper bound of a Benioff Intraslab earthquake, we selected a CSZ M 7.5 earthquake. This earthquake would rupture within the slab from about Allston in the north to St. Johns substation in the south (Figure 3-14).

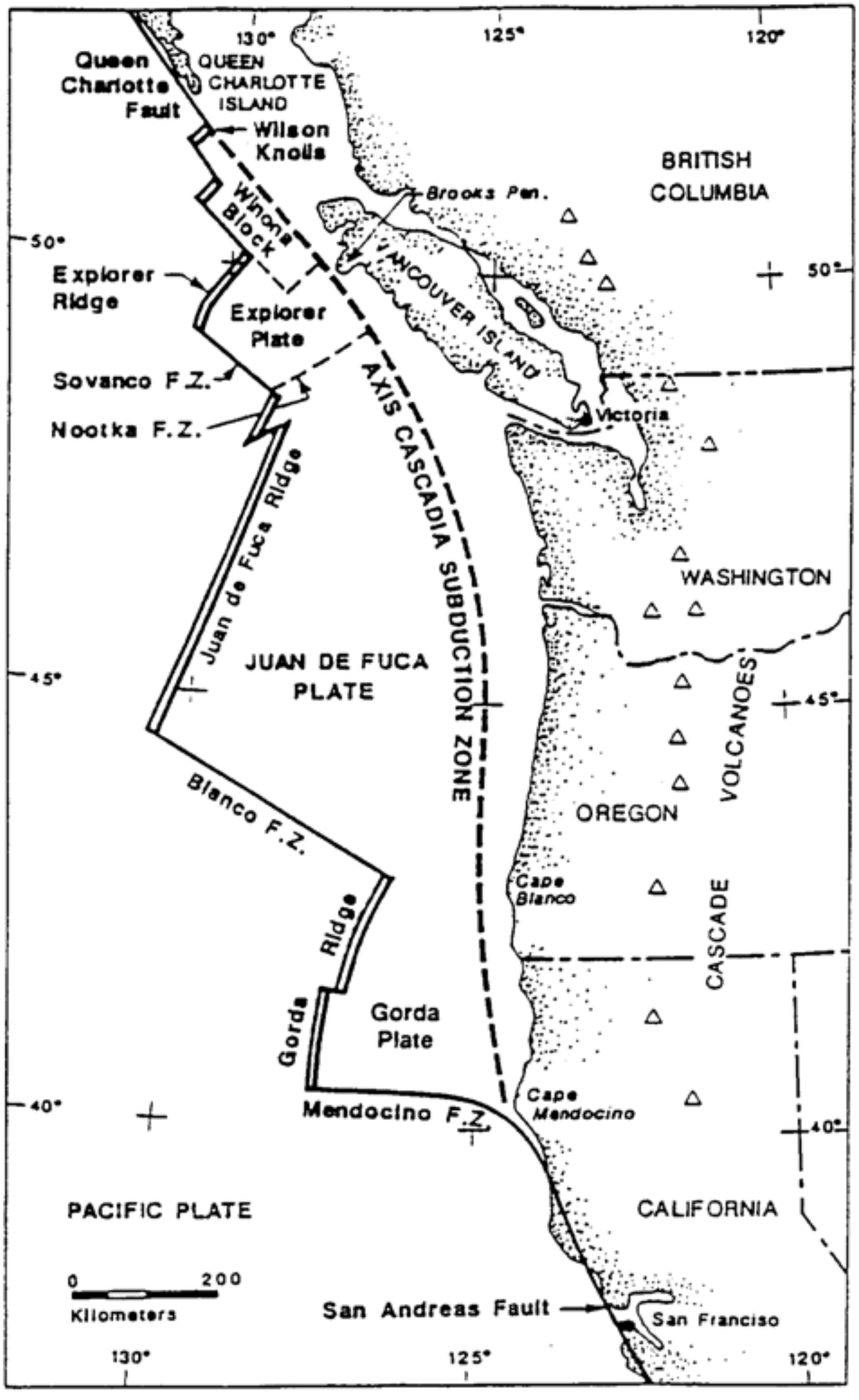


Figure 3-1. Cascadian Subduction Zone

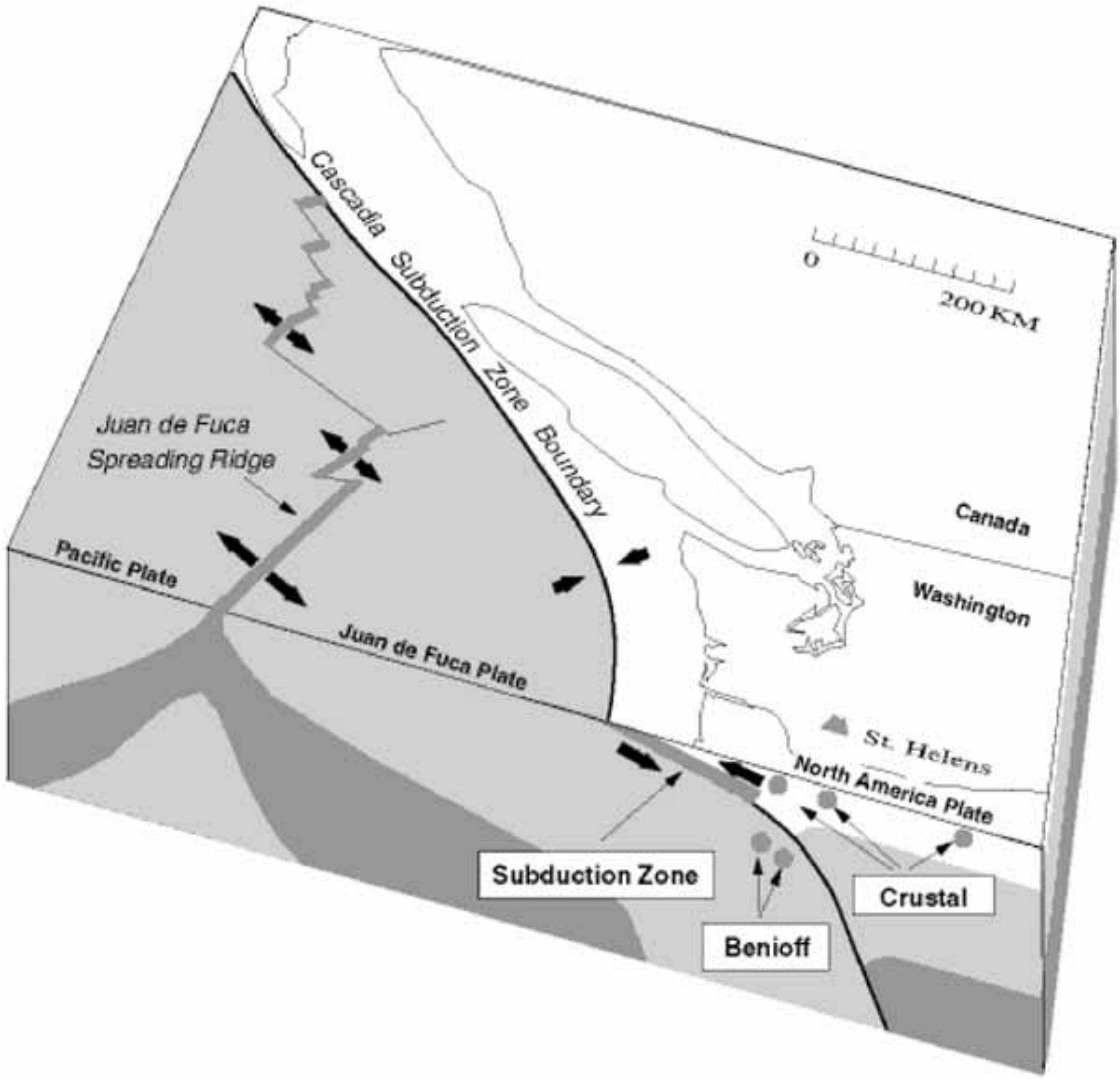


Figure 3-2. Cascadian Subduction Zone 3D View

Crustal Zone

The third source zone is the crust of the North American plate. Of the three source zones, this is the least understood.

Figure 3-3 shows the mechanisms for the 1993 Scotts Mill earthquake, located towards the northern edge of the Albany area. The 1993 Scotts Mill earthquake was an earthquake on a crustal fault.

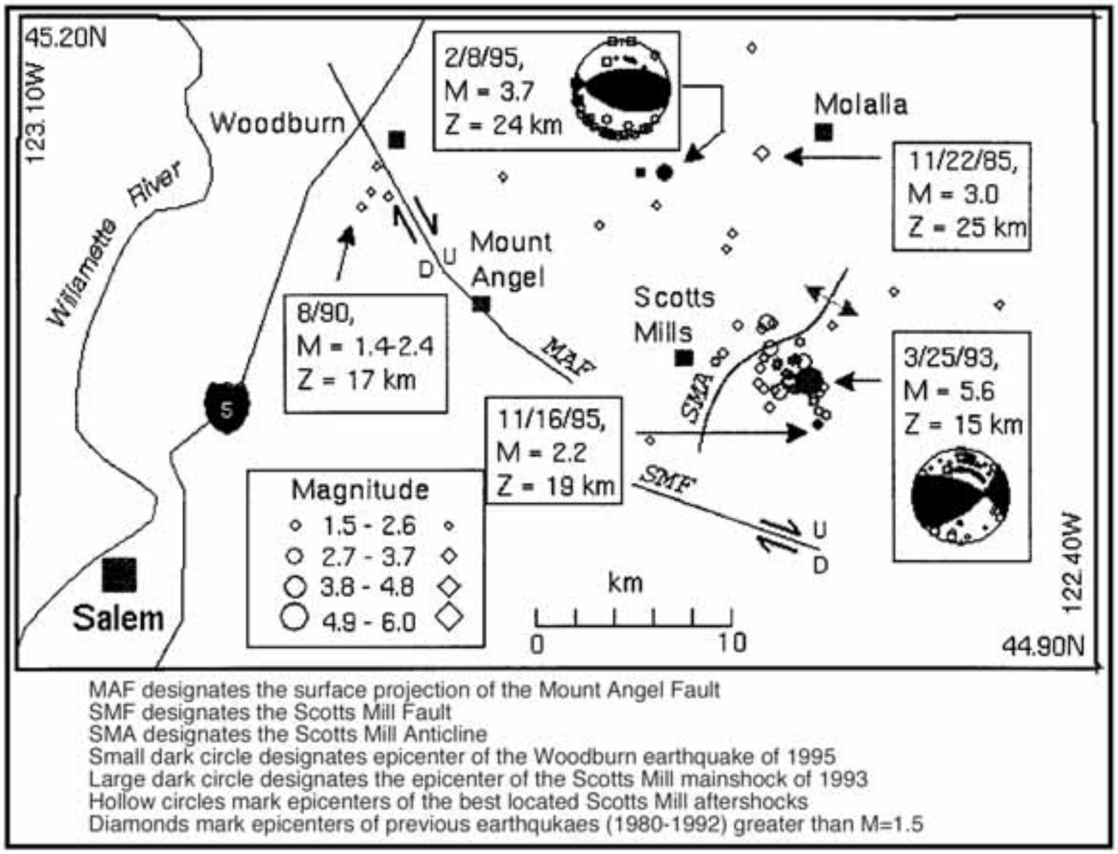


Figure 3-3. Location of Scotts Mill Earthquake, 1993

Recent work by Hull et al (2003) examines the area crustal seismicity, Figure 3-4. AS can be seen, there are a number of crustal features in the northern Oregon area. Most of the active crustal faults have low slip rates (0.2 mm per year or less), suggesting long return times (several thousand years or long for more crustal faults).

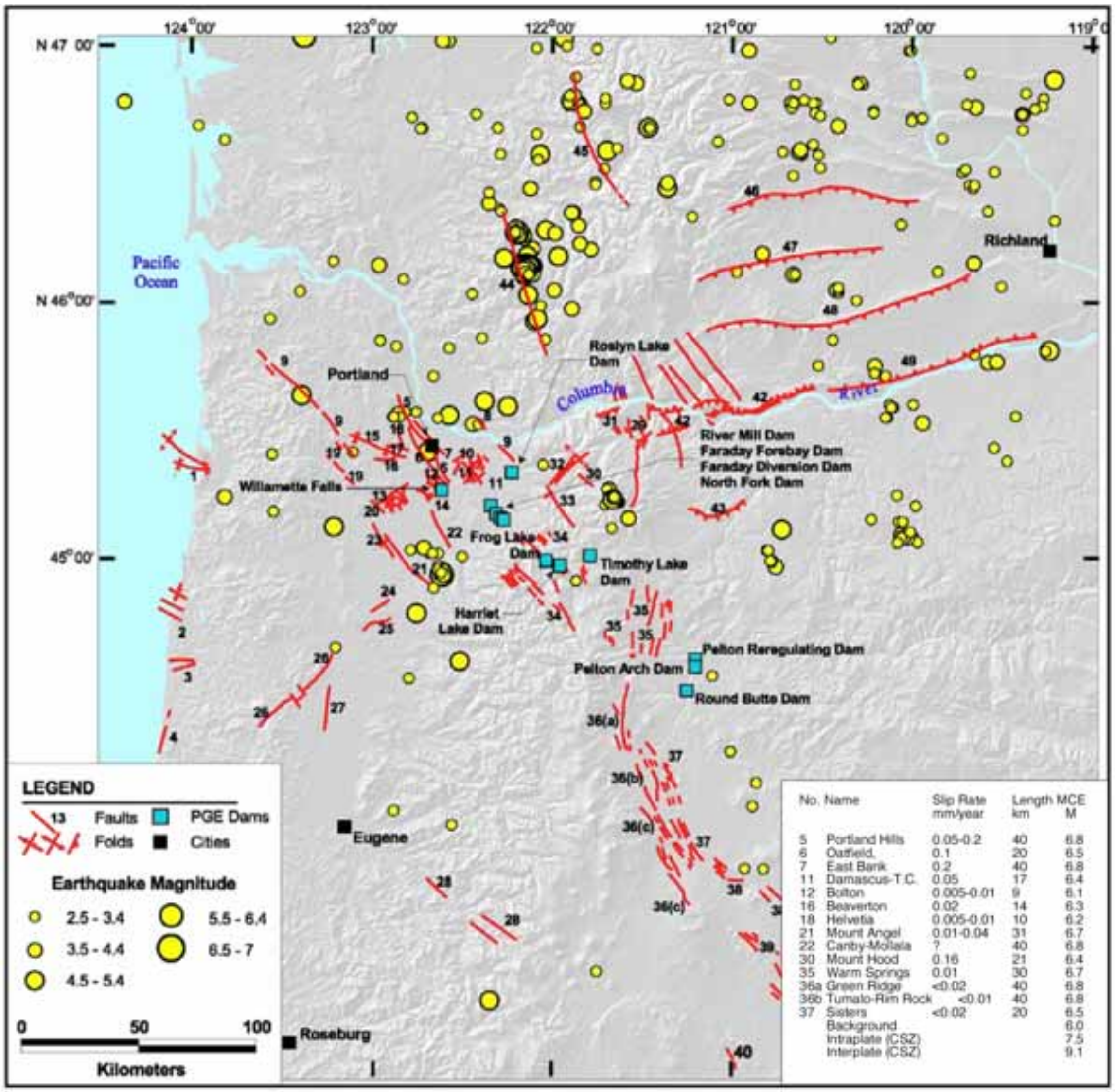


Figure 3-4. Crustal Faults in Northern Oregon (after Hull, 2003)

For crustal earthquakes, the general wave propagation characteristics of the North American crust in northern Oregon are believed to be similar to those of California; therefore, it is assumed that strong ground motion attenuation models developed from empirical data recorded in California are applicable to the Albany and surrounding area. For the interplate and intraplate earthquakes, use of California data is not appropriate because the tectonic environment of crustal earthquakes in California is different from these types of earthquakes (Sadigh 1997). For interplate and intraplate earthquake sources, an attenuation relationship specific for subduction zone earthquakes is used (Crouse 1991).

3.2 Seismic Hazards

There are five primary hazards induced by earthquakes:

- Ground shaking

- Liquefaction
- Landslide
- Surface faulting
- Tsunami / Seiche

To varying extents, the City of Albany is exposed to the first four of these hazards. For purposes of the current work, we approach the quantification of these hazards as follows.

Ground Shaking

Ground motion shaking levels are estimated at the WTP. The amount of ground shaking that will occur at the site is a function of the distance of the site to the causative earthquake fault; the magnitude of the earthquake, and the type of geology at the site. A probabilistic site hazard analysis was performed following the USGS site characterization for Oregon, as of 1996. The results are as follows:

- Once every 475 years: 0.143g or more
- Once every 975 years: 0.202 g or more
- Once every 2475 years: 0.305g or more

Figure 3-7 shows the complete seismic hazard curve for Albany. For new designs to the 1997 Edition of the UBC, the design PGA is 0.3g. It should be noted that while the design PGA of 0.3g reflects a return period longer than 475 years, a dominant source of the hazard is the potential for a moderately distant M 8.5 to 9 earthquake; this large magnitude earthquake would cause a longer duration of ground shaking (more than 60 seconds) than more common crustal events in California, and probably more damage to many types of ductile structures. Table 3-1 provides the digitized values in Figure 3-7.

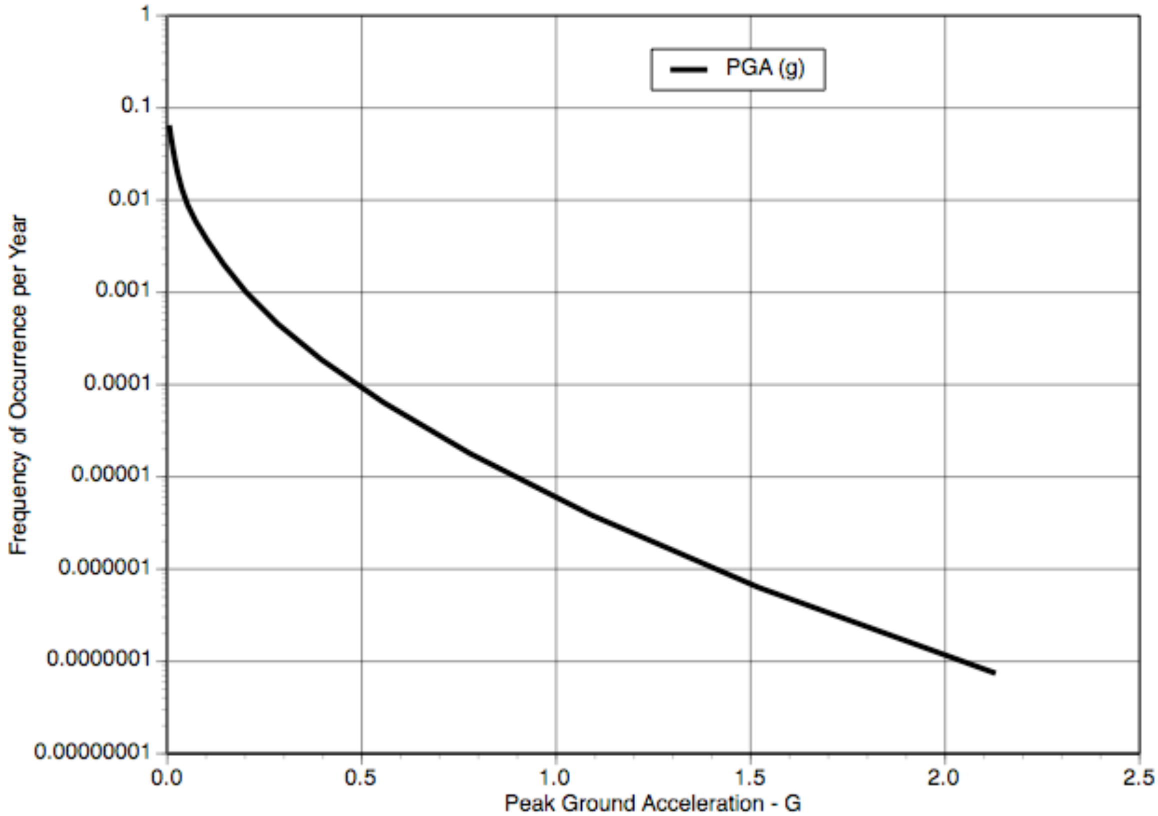


Figure 3-7. Seismic Hazard Curve for Albany

Frequency of Occurrence per Year	PGA (g)
0.06496	0.005
0.05637	0.007
0.04728	0.0098
0.03763	0.0137
0.02794	0.0192
0.0195	0.0269
0.01316	0.0376
0.008778	0.0527
0.005813	0.0738
0.003653	0.103
0.002018	0.145
0.001003	0.203
0.0004558	0.284
0.0001856	0.397
0.00006406	0.556
0.00001796	0.778
0.000003864	1.09
0.0000006339	1.52
0.00000007419	2.13

Table 3-1. Tabulated data for Figure 3-7

The Vine Street WTP is located in the west part of central Albany. This location is most likely best characterized as a deep alluvial soil site, and not a rock site. For ground motions up to about 0.3g, the soil site will tend to modestly increase PGA, as well as broaden the site response spectra to longer periods. For design purposes, it is recommended that any upgrades be designed using PGA = 0.25g coupled with a response spectra suitable for firm soil site.

Liquefaction

Firm and soft soil sites may be prone to liquefaction induced settlements and/or lateral spreads. Most of the time, the sites for the water system tanks and pump stations were chosen to avoid these types of hazards, but this may not always be the case; distribution pipelines go irregularly through liquefiable soils. Generally, the potential for liquefaction with corresponding permanent ground deformations (PGDs) at a site should be developed with understanding of the subsurface soil conditions, local slopes, and ground water conditions. These PGDs are then used to evaluate damage to items at a site, if suitable PGD-based fragility functions are provided.

As described in Section 4, we allow for no PGDs in the service area for 100 year earthquakes; 1%, 2% or 3% or distribution pipelines are assumed to traverse zones susceptible to liquefaction (or landslide) PGDs in the 475-, 975- or 2,475-year earthquakes, respectively.

Landslide

Most of the Albany water system service area is in a broad alluvial plain, not susceptible to landslide. In the two elevated pressure zones, higher slope angles may allow for some landslide susceptibility.

Earthquakes can trigger landslides. Landslides of most concern to the City of Albany distribution system are deep seated slides beneath roads that cause rotation slumps of the top 5 to 30 feet of soils. These movements, also sometimes called lateral spreads, can result in inches to several feet of downslope movements.

The prevalence of earthquake-triggered landslides will be highest when the underlying hillside soils are saturated. In the Albany area, soils become saturated on an annual basis, once there has been sufficient winter rains. In a typical winter season, soils become saturated near the end of December, and remain so until April.

As described in Section 4, we allow for no PGDs in the service area for 100 year earthquakes; 1%, 2% or 3% or distribution pipelines are assumed to traverse zones susceptible to landslide (or liquefaction) PGDs in the 475-, 975- or 2,475-year earthquakes, respectively.

Surface Faulting

Most if not all of the Water System facility sites and distribution pipelines are not situated at sites prone to surface faulting. Generally, the potential for surface faulting at a site should be developed with understanding of the location of active crustal faults. For this evaluation, fault offset hazards are not considered at any Albany facility site or distribution pipeline. On a system wide basis, it is unlikely that this hazard will have material affect to the Albany Water System as a whole.

Tsunami / Seiche

These are waves produced by earthquakes or other underwater disturbances. Within lakes, such waves are called Seiche. The height of these waves is not likely to impact any Albany Water System facility or pipeline.

4.0 Performance of Water System in Earthquakes

4.1 Existing System with Minor Upgrades

We performed a risk based analysis of the performance of the existing water system (with minimum upgrades) under four "scenario" earthquakes. Specifically, we assumed that an earthquake that produces an average PGA throughout the water system (firm soil site) of 0.04g (representing 100 year events or less); 0.14g (representing a 475 year event); 0.20g (representing a 975 year event) or 0.31g (representing a 2,475 year event).

These ground motions correspond to the hazard curve in Figure 3-7 for the 475, 975 and 2,475 year events. Since the dominant contributor of the hazard are subduction zone earthquakes, we would not expect much variation over the service area in the median-calculated PGA values (variations due to uncertainties in the attenuation model are incorporated into the evaluations). While it is understood that a crustal event would probably have quite a bit of spatial variation over the service area, crustal events are only a small contributor to the overall risk, so the approximation seems reasonable for purposes of system-wide benefit cost analysis purposes.

By "minimum upgrades", we mean that the Vine Street Water Treatment Plant upgrades listing as having either High or Moderate priority are upgraded to substantially meet the intent of current code for PGA = 0.3g motions. In Section 4.1, we discuss the impacts to overall system performance if these upgrades are not done. In Section 4.2, we discuss the impacts to overall system performance if these upgrades are done.

Experience has shown that one of the major contributors to overall water system performance is the damage to buried pipes. Buried pipes are damaged due to a combination of ground shaking and permanent ground deformations. For this evaluation, we made the following assumptions based on a reasonable estimation of the susceptibility of the ground to soil failure in the greater Albany service area.

Earthquake Return Period (Years)	PGV (inches / second)	Area exposed to PGDs	PGD in areas exposed (inches)
100	2.1	0.0 %	0.0
475	7.5	1.0 %	0.5 to 1.5
975	10.7	2.0 %	0.5 to 1.5
2475	16.1	3.0 %	0.5 to 1.5

Table 4-1. Seismic Hazards for Evaluation of Buried Pipe

Given the makeup of pipe in the system ranging from relatively good materials (ductile iron) to relatively poor materials (small diameter steel pipe), Table 4-2 provides the statistical level damage is forecasted for buried pipe. The damage is calculated using the standard fragility functions and procedures in ALA (2001).

Earthquake Return Period (Years)	Repairs due to Ground Shaking	Repairs due to Permanent Ground Deformations	Total Repairs
100	3.6	0	3.6
475	12.9	11.7	24.5
975	18.2	23.3	41.5
2475	27.5	35.0	62.4

Table 4-2. Number of Repairs for Buried Pipe

Given the damage to the buried pipe network, the City of Albany WTP staff will make repairs. It is assumed that the City can muster 8 people able to make pipe repairs after an earthquake, ramping up on the first day to full staffing by the second day. Until all major pipelines are repaired, it is assumed that staff work 12 hours days, 7 days per week; thereafter, a regular 40 hour work week. In the post-earthquake environment, it is assumed to take 120 manhours to repair a pipe 16 inches or larger; 60 hours for a pipe 12 inches in diameter; or 24 hours for pipes under 12 inches in diameter. Repairs for pipes with diameter 36-inches and larger are assumed to be done by outside contractors, taking seven days to complete. These repair times are somewhat longer than needed under a non-earthquake condition, but reflect the slow down in the post-earthquake environment due to poorer communications, transportation, etc.

The number of fire ignitions[2] that are calculated to occur within the service area within the first 24 hours after the earthquake are: 0.9 (100 year); 1.5 (475 year); 2.4 (975 year); or 3.8 (2,475 year). The estimated ignitions are calculated using the fire ignition model in Eidinger (2004), given the available inventory of buildings and local PGA.

In the first hours after the earthquake, water will be used in the three pressure zones as a combination of water used for fire flows, leaks through broken pipes, and regular

[2] The numbers of fire ignitions are listed as "statistical" fire ignitions requiring fire department response. For example, 0.9 can be interpreted that there is a 90% chance of one fire occurring and 10% chance that no fires occur. Note that once a fire ignition occurs, lack of adequate fire department response will often result in spread of the initial fire, possibly leading to conflagration, especially if it is windy at the time of the earthquake.

customer demand. For all analyses in this report, we assume average day demands for the year 2006.

At the time of the earthquake it is assumed that all storage tanks are full and the tanks are not damaged; this assumption is reasonable for earthquakes up to about return periods of 975 years, but is somewhat optimistic for ground motions with PGA = 0.3g or higher. Water in the Maple Street, Queen Avenue and 34th Avenue tanks are assumed to be unavailable immediately after the earthquake, due to a concurrent power outage. Water in the Queen and 34th Avenue tanks are assumed to be available within 24 hours after the earthquake, after mobilizing portable generators.

Regional power outages are assumed to last as follows: 6 hours (100 year earthquake); 12 hours (475 year earthquake); 24 hours (975 year earthquake); 72 hours (2,475 year earthquake). While we have not performed a detailed study of the PP&L local grid in the Albany area, these power outage times are reasonable given the level of shaking and PP&L's and BPA's ability to restore power to critical facilities given the magnitude of earthquakes described.

We assume that in the current condition, that the Vine Street WTP will have the following performance:

- 100 year EQ. No material damage, an outage of 6 hours occurs until minor repairs are made the power restored.
- 475 Year EQ. The WTP is shutdown for 12 hours. The WTP sustains minor damage to the minimum cut set (the minimum cut set includes the minimum amount of equipment / clarifiers / settlers / chemical systems needed to provide potable water, although not necessarily to full drinking water standards) needed to supply at least 8.2 MGD; more damage occurs to structures, but the plant is restored to operational service 12 hours after the earthquake.
- 975 Year EQ. The WTP is shutdown for 48 hours. The WTP sustains moderate damage to the minimum cut set needed to supply at least 8.2 MGD. The Canal suffers minor slumping and turbidity spikes. Within 48 hours, damaged structures are removed as needed to restore the minimum cut-set, the Canal is repaired (backhoe to remove slumps, etc.) and temporary repairs are made to critical pipes, motor controls and chemical systems needed to restore minimum flow through the plant. 48 hours after the earthquake, power is restored as needed to process 8.2 MGD through the plant. For 96 hours after the earthquake, turbidity spikes in the raw water supply, plus liquefaction in the filters, plus a combination of minor damage in the WTP causes incomplete treatment, and more common backwash than normal. While all water introduced into the distribution system is at least minimally treated (disinfection only), water quality will be lower than normal, suggesting a boil water alert for 4 days (in practice, the water will likely be potable, but as a safety measure, a boil water alert is put into place until sufficient water quality testing is performed to confirm that water in the distribution system remain potable).

- 2,475 Year EQ. The WTP is shutdown for 168 hours. The WTP sustains moderate to major damage to the minimum cut set needed to supply at least 8.2 MGD. The Canal suffers minor slumping and turbidity spikes at several locations, possibly with a major water quality impact. Within 168 hours, damaged structures are removed as needed to restore the minimum cut-set, repairs are made to the Canal, and temporary repairs are made to critical pipes, motor controls and chemical systems needed to restore minimum flow through the plant. 168 hours after the earthquake, there is sufficient power available to process 8.2 MGD through the plant. For 336 hours (2 weeks) after the earthquake, turbidity spikes in the raw water supply, plus liquefaction in the filters, plus a combination of minor to major damage in the WTP (damaged mechanical equipment in the clarifiers, broken pipes, loss of filters due to collapsed brick buildings, possible use of portable pumps and flex hose to bypass damaged pump building, possibly damaged / partially collapsed chemical building, loss of water quality testing laboratory due to collapse of brick control building, etc.) causes incomplete treatment, and more common backwash than normal. While all water introduced into the distribution system is at least minimally treated (disinfection only), water quality will be lower than normal, suggesting a boil water alert for 2 weeks (in practice, the water will likely be potable, but as a safety measure, a boil water alert is put into place until sufficient water quality testing is performed to confirm that water in the distribution system remain potable).

The time needed to complete repairs in the system to be able to restore at least average day demands to every customer in the system are calculated to be:

- 100 year EQ. 51 hours (2.1 days).
- 475 Year EQ. 303 hours (12.6 days).
- 975 Year EQ. 492 hours (20.5 days).
- 2,475 Year EQ. 740 hours (30.8 days).

Given these conditions, the overall system performance is shown in Figure 4-1. In Figure 4-1, the vertical axis represents the percentage of all customers that receive water at average day demand (or better) rate, at a minimum of 20 psi pressure; and the horizontal axis is the time after the earthquake (in hours, log scale). The results from the four different earthquakes are shown in thin (100 year earthquake) to thickest (2,475 year earthquake) lines. The major observations are as follows:

- The 100 year earthquake causes essentially no disruption to water service.
- The 475 year earthquake causes about 92% of customer to have water service immediately; this decreases to 20% once storage in the Broadway tank is exhausted, but then increases to 80% once power is restored to operate the WTP and pump stations. Then, water service is slowly restored to remaining customers as repairs are made to transmission and distribution pipes.
- The 975 year earthquake causes about 75% of customer to have water service immediately; this decreases to 16% once storage in the Broadway tank is

exhausted, and then 0% as the storage in Zone 2 and 3 tanks is exhausted. Once portable generators are hooked up to the Zone 1 pump stations using quick connects, water will be available to about 71% of customers, until the Queen and 34th Street tanks are exhausted. Water is restored to about 95% of customers once the WTP is put back into minimal service and regional power outages are ended. Then, water service is slowly restored to remaining customers as repairs are made to transmission and distribution pipes, with the last customer restored to service in 20.5 days.

- The 2,475 year earthquake causes about 58% of customer to have water service immediately; this decreases to 13% once storage in the Broadway tank is exhausted, and then 0% as the storage in Zone 2 and 3 tanks is exhausted. Once portable generators are hooked up to the Zone 1 pump stations using quick connects, water will be available to about 70% of customers, until the Queen and 34th Street tanks are exhausted. Water is restored to about 94% of customers once the WTP is put back into minimal service and regional power outages are ended, 1 week after the earthquake. Then, water service is slowly restored to remaining customers as repairs are made to transmission and distribution pipes, with the last customer restored to service in 30.8 days.

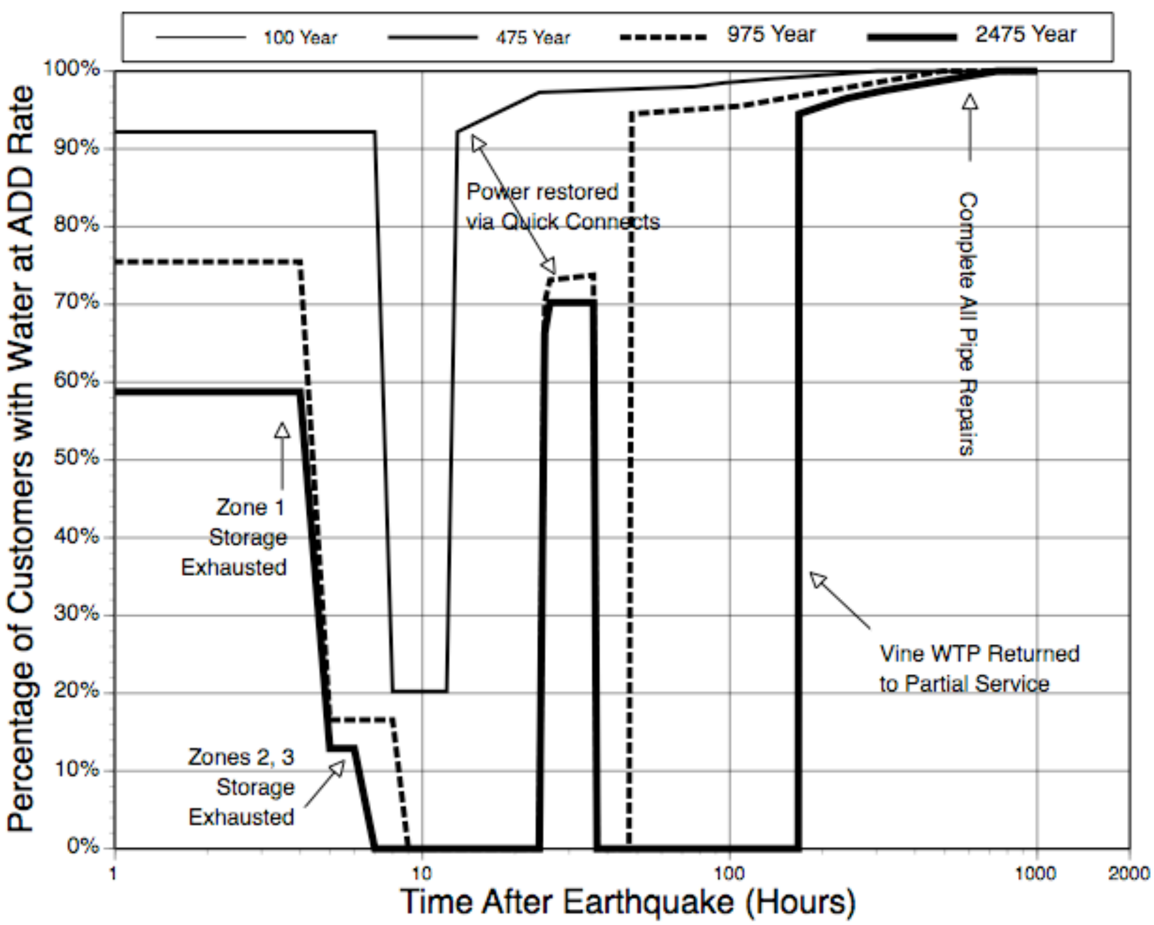


Figure 4-1. Water System Performance with Minimal Upgrades

In Figure 4-1, the area "above" the lines can be integrated to get the total "systems days lost". By "system days lost", it is meant that there is a equivalent number of days where 100% of all customers have lost all water supply. This is a measure of how severe the water outages are to the system as a whole. The results are:

- 100 year EQ. 0 system hours lost (no boil water alert)
- 475 Year EQ. 8.1 system hours lost (no boil water alert)
- 975 Year EQ. 44.7 system hours lost (96 hour boil water alert)
- 2,475 Year EQ. 166.6 system hours lost (336 hour boil water alert)

In other words, a 475-year earthquake would result in system-wide outages that could be represented as having no water anywhere in the water system for 8.1 hours.

In November, 2005, the City of Albany put into service a new, smaller capacity water treatment plant that draws water from the Santiam River. The new plant is presumed to be built for earthquakes (UBC Zone 3), suffering no damage (except for power outages) for earthquakes in the 100, 475 and 975-year earthquakes, and minor to moderate damage in the 2,475 year earthquake. According to plant staff, the new plant's intake pipeline (36") and effluent pipeline to Albany (42" to 36") are built primarily of ductile iron pipe with push-on joints, without any special provision for earthquake loads. A portion of this new pipeline may traverse zones with moderate to high liquefaction potential, so that a settlement or 1 to 3 inches (or lateral spread) might result in the failure of one or more pipe joints. Optimistically assuming that the intake and effluent lines are not damaged in large earthquakes, it is assumed that the new WTP in conjunction with the Vine Street WTP will be able to provide up to 8.2 MGD flows for Albany within 0 hours (100 year earthquake); 12 hours (475 year earthquake), 24 hours (975 year earthquake) or 48 hours (2,475 year earthquake). Factoring in this new WTP (with no intake / effluent line pipeline damage), the system hours lost would be:

- 100 year EQ. 0 system hours lost (no boil water alert)
- 475 Year EQ. 8.1 system hours lost (no boil water alert)
- 975 Year EQ. 31.8 system hours lost (no boil water alert)
- 2,475 Year EQ. 50.9 system hours lost (96 boil water alert)

The likelihood of pipe damage from the new WTP to Albany is estimated at 0% chance for the 100 year and 475 year earthquakes, and 10% chance in the 975 and 2,475 year earthquakes. Given that the 36" pipe breaks, it will take at one week to repair this pipe using outside contractor services. For economic loss estimation, then we assign a 90% chance that the lower system hour outages will occur (new water treatment plant quickly available) and 10% chance that the longer system hour outages will occur (new water treatment plant takes at least one week to return to service). System outage times after one week are primary a function of damage to distribution pipes in either case.

4.2 Existing System with Vine Street WTP Upgrades

As described in Section 2 of this report, the Vine Street WTP has a number of buildings and systems are that have clear seismic weaknesses. The bulk of these weaknesses are due to the widespread use of unreinforced brick buildings; with some other weaknesses due to the occasional use of unanchored equipment and similar issues.

For purposes of benefit cost analyses, we make the assumption that all buildings and equipment that are listed as having moderate to high priority are ultimately upgraded to have high reliability for PGA = 0.3g. This is essentially the same as the current design requirement for new facilities for western Oregon (UBC zone 3). This will involve the upgrades outlined in Section 2 and described in more detail in Section 5 of this report.

We assume that no new emergency generators are installed in the system, so that if there is a regional power outage, there will still be loss of pumping in the system until emergency generators can be installed. There will be quick connects available to operate the WTP at 8.2 MGD.

Given these upgrades, we examine how much better the water system will perform. Repeating the analyses as in Section 4.1, but with the assumption that the Vine Street WTP can be restored to service in no more than 12 hours, then the following system hours would be lost (including the impact of the new WTP) in the various earthquakes:

- 100 year EQ. 0 system hours lost (no boil water alert)
- 475 Year EQ. 4.5 system hours lost (no boil water alert)
- 975 Year EQ. 18.2 system hours lost (no boil water alert)
- 2,475 Year EQ. 41.1 system hours lost (96 hour boil water alert)

As can be seen, there is a substantial improvement (about 75%) in system performance for the 2,475-year earthquakes, and somewhat smaller improvements in the shorter return period earthquakes.

5.0 Seismic Upgrades

In 2005, CH2M-Hill performed a seismic evaluation of the Vine Street WTP. The approach used followed ASCE 31, which is a method for rapid structural seismic evaluations using so-called Tier 1 site walkdowns and checklist procedures. The approach was supplemented with calculations using AWWA D100 and ACI 350 for water retention structures. While these procedures are not as detailed as complete structural evaluations, they do form a first basis approximation to sort through which buildings / structures have more seismic deficiencies from those that have fewer.

In October 2005, the CH2M-Hill effort was supplemented by G&E. Table 5-1 summarizes the main facilities at the site, along with the target goal suggested by CH2M-Hill, and the seismic evaluation considerations by G&E. Note that the G&E evaluation incorporates a combination of seismic hazard, structural performance, life safety and benefit cost concepts, with the intent of developing a prioritized upgrade approach for the Water Treatment Plant, whereas the CH2M-Hill approach is basically "does it meet code".

Facility	Construction Style	CH2M-Hill Target Goal	G&E Evaluation
Control Building	URM	Life Safety	Significant Collapse and Life Safety Risk
Raw Water Pump Station	URM	Life Safety	Significant Collapse Risk
Soda Ash Building	URM	Life Safety	Significant Collapse Risk
Old Filter Building	URM	Life Safety	Significant Collapse Risk
Large Filter Building	Reinforced Masonry, Reinforced Concrete Basins	Life Safety	Some Collapse Risk
Chemical Building	Reinforced Concrete and Tilt Up	Immediate Occupancy	Some Collapse Risk
Accelator 1	Reinforced Concrete	Life Safety	Tank Likely OK
Accelator 2	Steel Tank Anchored	Life Safety	Tank OK
High Pressure Pump Building	Reinforced Masonry	Immediate Occupancy	Some Collapse Risk
Maple Reservoir	Steel Tank Unanchored	Immediate Occupancy	Tank OK, Add Flexible Pipes

Table 5-1. Summary of Facilities at Vine Street WTP

Generally, the CH2M-Hill goal of meeting Life Safety would be met of the existing structure could meet PGA = 0.3g, with I=1.0, versus typical modern code allowables.

Generally, the CH2M-Hill goal of meeting Immediate Occupancy would be met of the existing structure could meet PGA = 0.3g, with I=1.25, versus typical modern code allowables.

Generally, the G&E evaluation is based on developing a prioritized seismic retrofit program that is cost effective and that will produce a high benefit cost ratio.

Given the findings in the CH2M-Hill report, as supplemented by the G&E evaluation in this report, the target costs and prioritizes to upgrade the Vine Street Water Treatment Plant facilities are as follows:

Facility	Recommended Upgrade Cost, Including Non-Structural Items	G&E Recommended Upgrade Priority
Control Building	$288,000	High
Raw Water Pump Station	$90,000	High
Soda Ash Building	$325,000	Low
Old Filter Building	$250,000	Moderate
Large Filter Building	$50,000	Moderate
Chemical Building	$60,000	Moderate
Accelator 1	$120,000	Low
Accelator 2	$0	Low
High Pressure Pump Building	$25,000	Moderate
Maple Reservoir	$120,000	Moderate
Total High	$378,000	
Total High + Moderate	$883,000	

Table 5-2. Summary of Facilities at Vine Street WTP

The costs in Table 5-2 reflect mid-2007 construction costs. In addition to these costs, there will be geotechnical (2%), materials testing (2%), engineering (14%), inspection (5%), permits (2%), project management (5%) costs, estimated at 30% of total construction cost. If only the "High" priority upgrades are performed, the total upgrade cost would be $491,000. If the "High and Moderate" priority items are upgraded, the total cost would be $1,148,000. Seismic upgrades are made to PGA = 0.30g, with I=1.0 and with R values suitable for the style of construction and intended performance level (generally conforming to current code). The need to use I=1.25 is not essential for Albany, given the nature of the site specific hazards, so the extra cost needed to meet I=1.25 is not considered important.

The upgrade construction costs are based on estimated costs per square foot for URM buildings of the type at the plant, with all upgrades performed on the interior, and that the exterior shell of the building is retained with essentially the same look as in its current condition (accepting that minor holes / plates less than 6 inches square may be changed). The upgrades allow for primarily structural work, with minor provision for relocation of mechanical and electrical systems, and little changes to wall finishes; in other words, the interior of the buildings will continue to have a utilitarian finish.

The upgrades are described in the following paragraphs:

Control building. This is a pre-1900 URM Building, with additions built in 1912 and with modifications made in the 1920s/1930s. The building houses the Control Room and Water Quality Lab. The roof diaphragm is straight timber sheathing supported on steel trusses. Relatively recently installed interior CMU walls are used to provide a modern work environment for staff; but these walls provide no lateral force resisting path for the exterior brick URM building. The upgrade plan is to improve the in-plane and out-of-plane capacity of existing brick walls; modify diaphragms; and improve diaphragm connections. As this building is a historic brick structure, the upgrades would be done to preserve the look-and-feel of the exterior facades. Interior walls are generally accessible. The upgrade would include a new interior steel braced frame structure with suitable connections to the brick structure; an enhanced roof diaphragm; shotcrete walls might be used at selected interior locations. Common occupancy is 5 people (day time) or 2 people (off hours).

The water quality lab has several pieces of counter top equipment (unanchored) prone to sliding / falling under strong ground shaking. There are unanchored equipment racks with control equipment / computers (Figure 5-2).

Figure 5-1. Control Building, East Elevation

Figure 5-2. Unanchored Equipment Rack

There are several unanchored / unrestrained or poorly restrained electrical / measuring cabinets, as for example Figure 5-2. All these should be suitable anchored by adding anchor bolts into the concrete floor below (do not anchor into URM walls).

Figure 5-3. Floor Standing Cabinets

The interior CMU enclosure structure has poorly installed HVAC roof-level pieces of equipment; these should be suitably restrained to prevent damage to attached pipes.

Raw Water Pump Station building. This is a single story URM building (Figure 5-4), with concrete wet well. The recommended seismic upgrades include improving the in-plane and out-of-plane capacity of existing brick walls; modify diaphragms; and improve diaphragm connections. As this building is a historic brick structure, the upgrades would be done to preserve the roof-and-feel of the exterior facades. Interior walls are generally accessible. Possibly, the upgrade would include a new interior steel braced frame structure with suitable connections to the brick structure; with an enhanced diaphragm; shotcrete walls might be used at interior locations.

Figure 5-4. Raw Water Pump Station, East Elevation

The pump controls are in floor standing racks are unanchored[3] (Figure 5-5).

[3] During the site walkdown, the anchorage / restraint of all floor standing cabinets was reviewed. In some cases, ongoing operations prevented removal of bottom panels to confirm anchorage. Given the date of installation and site review, this report lists cabinets as unanchored if proper anchorage could not be confirmed in the field.

Figure 5-5. Control Cabinet for Pumps (Unanchored)

Soda Ash building. The building is a URM structure, two stories tall (Figure 5-6). The exterior walls are perforated by numerous windows. There are no interior walls. This building is normally unoccupied and currently has little function in terms of maintaining critical plan operations, but does provide storage space for spare parts (Figure 5-7) and the like. Interior storage racks are unanchored. CH2M-Hill recommended a full upgrade, but the lack of occupancy / key function suggests a lower priority and lower cost-effectiveness. Average occupancy is less than 0.05 people on a 24 hour basis. Structural upgrades would include improving the in-plane and out-of-plane capacity of existing brick walls; modify diaphragms; and improving the diaphragm connections. As this building is a historic brick structure, the upgrades would be done to preserve the roof-and-feel of the exterior facades. Interior walls are easily accessible. Possibly, the upgrade would include a new interior steel braced frame structure with suitable connections to the brick structure; with an enhanced diaphragm; shotcrete walls might be used at interior locations.

Figure 5-6. Old Filter Building (Left Side) and Soda Ash Building (Right Side)

Figure 5-7. Storage Rack in Old Filter Building

Old Filter building. This building is normally unoccupied (Figure 5-6). At its lower levels, there are small reinforced concrete basins that can serve as part of system operation to provide flows after earthquakes. CH2M-Hill recommended a full upgrade, but the lack of occupancy suggests a moderate priority and cost-effectiveness. Seismic upgrades would include improving the in-plane and out-of-plane capacity of the existing brick walls; modify straight sheathing wood roof diaphragm; and improving the diaphragm connections. As this building is a historic brick structure, the upgrades would be done to preserve the roof-and-feel of the exterior facades. Interior walls are easily accessible. Possibly, the upgrade would include a new interior steel braced frame structure with suitable connections to the brick structure; with an enhanced diaphragm; shotcrete walls might be used at interior locations.

There are three floor standing motor control centers, one of which is restrained and the other two should be anchored / restrained (one of which shown in Figure 5-8).

Figure 5-8. Restrained Control Cabinet

Large Filter building. The Large Filters include the primary reinforced concrete filters, pipe gallery, pump room and operations building (Figure 5-9). It is essential to post-earthquake operations. It is normally not occupied except for occasional maintenance workers (say 0.25 people average occupancy). Recommended upgrades include modification of the diaphragm connections for the CMU building (including intrusive testing to develop constructible connection details); addition of in-plane bracing, and installation of wall-foundation anchor upgrades.

Figure 5-9. CMU Building – Large Filters

Figure 5-10 shows two poorly restrained plastic chemical tanks. The steel rack legs are supported on wood shims and left unrestrained on the floor below. A floor-standing motor control cabinet is unrestrained.

Figure 5-10. Poorly Restrained Chemical Tanks

Chemical Building. This building (Figure 5-11) is two stories tall and houses the chlorine and alum tanks. It was built in 1963 using reinforced concrete tilt-up wall style construction surrounded by reinforced concrete columns. The walls are not positively tied to the structure on all sides.

The function of this building is critical to plant operations after an earthquake. Planned upgrades would include improvement of connections; and improved anchorage of the alum tank and other chemical tanks within the building.

Figure 5-11. Chemical Building

Figure 5-12. Unrestrained Tanks within Chemical Building

High Pressure Pump Building. This building is a single story reinforced masonry structure, built in the early 1960s. It is rectangular in plan, and has several window openings. The roof diaphragm uses straight 2x6 sheathing lumber supported in steel joists. This building houses the high pressure pumps needed to move water from the WTP into Zone 1. The building currently (as of late 2005) exhibits through-wall cracking (reportedly from original construction settlement), but this cracking is not considered to have seriously weakened the building to be able to survive a 475-year earthquake motion. A limited upgrade to bring the building up to Zone 3 would include adding shotcrete to the walls or adding improved wall-roof connectors.

Accelator #1 is a reinforced concrete tank (Figure 5-13). The existing tank performance is considered suitable for a 475-year earthquake. The exterior reinforced concrete columns, while using non-ductile shear reinforcement, are probably adequate as long as the in-plane shear diaphragm action of the concrete tank shell limits top level deflections so low as to preclude much lateral load being taken by the columns. Further, Accelator #1 is somewhat made redundant by Accelator #2 with regards to providing minimum post-earthquake plant capacity. Upgrades could include adding exterior reinforced concrete walls to provide additional seismic capacity, but these are considered relatively lower priority.

Figure 5-13. Accelator 1

Figure 5-14. Accelator 1 (Background)

Accelator #2 is a welded steel tank, Figure 5-15, built 1978. It is founded on a concrete ring beam, with the lowest course anchored using a steel channel embedded into the ring beam. Tank wall thickness is assumed to be 0.25 inches over the entire height. The existing tank shell performance should be adequate at PGA = 0.3g (see Table 5-3 for results from analysis). There are interior tube settlers and launders; these appear to have satisfactory capacity to take seismic sloshing loads, and should not be significantly damaged barring tank wall uplift. Rocking of the interior baffles is possible.

Figure 5-15. Accelator 2 (Left Side) and Maple Reservoir (Right Side)

Item	AWWA Code Rw=4.5	Elastic R=1	Units
Impulsive frequency	10	10	cps
Convective frequency	0.18	0.18	cps
Sai (2% damped)	1.09	1.09	g
Sac (0.5% damped)	0.13	0.13	g
Unrestricted slosh height	2.39	2.39	feet
Base overturning moment	1,043	4,694	kip-ft
Base shear	189	849	kip
Coefficient of Friction	0.35	0.70	

F.S. Sliding	4.60	2.04	
Hoop stress, top course C/D	8.1	6.3	
Hoop stress, three quarter height C/D	4.3	3.5	
Hoop stress, mid height C/D	2.9	2.5	
Hoop stress, quarter height C/D	2.3	2.0	
Hoop stress, bottom C/D	1.9	1.7	
Tank uplift ratio	0.23	1.05	
Tank uplift?	No	No	
Bottom course compressive stress if no uplift	320	590	psi
Compressive stress allowable without hoop benefit	1,335	1,335	psi
Compressive stress allowable with hoop stress benefit	4,156	4,156	psi
Buckling ratio, bottom course, C/D	4.2	2.3	No hoop
Buckling ratio, bottom course C/D	13.0	7.1	Hoop

Table 5-3. Accelator #2, PGA = 0.30g

Figure 5-16. Accelator 2 Tube Settlers and Launders

Maple Reservoir is a welded steel tank resting unanchored to a concrete ring wall foundation (Figure 5-17). Its three entry/exist pipes are all susceptible to damage due to tank uplift at PGA = 0.3g or so.

Figure 5-17. Maple Reservoir (Background), Accelator 2 (Foreground)

The tank was presumably built to the provisions of AWWA D100-59 or later, likely with no provision for seismic design. Fabrication drawings are not available for review. Therefore, the tank was evaluated for PGA = 0.30g on a firm soil site, assuming that the plate steel is relatively low yield stress (Fy = 30,000 psi), and then sizing the shell per code minimum for hydrostatic loads. The roof system is assumed to use light channel rafters, highly susceptible to lateral torsional buckling should there be imposed deflections (such as due to wall uplift). Table 5-1 shows the analysis results, assuming either Rw = 3.5 (code arbitrary knockdown factor for unanchored tanks) or R = 1.0 (elastic design).

Item	Code Rw=3.5	Elastic R=1	Units
Impulsive frequency	4.5	4.5	cps
Convective frequency	0.17	0.17	cps
Sai (2% damped)	1.09	1.09	g
Sac (0.5% damped)	0.11	0.11	g
Unrestricted slosh height	3.65	3.65	feet

Base overturning moment	40,361	141,264	kip-ft
Base shear	2,640	9,240	kip
Coefficient of Friction	0.35	0.70	
F.S. Sliding	2.20	1.26	
Hoop stress, top course C/D	1.77	1.56	
Hoop stress, three quarter height C/D	1.20	1.08	
Hoop stress, mid height C/D	1.11	1.04	
Hoop stress, quarter height C/D	1.10	1.05	
Hoop stress, bottom C/D	1.11	1.11	
Tank uplift ratio	1.52	5.32	
Tank uplift?	Yes	Yes	
Bottom course compressive stress if no uplift	3,067	2,612	psi
Compressive stress allowable without hoop benefit	2,363	2,363	psi
Compressive stress allowable with hoop stress benefit	7,436	7,436	psi
Buckling ratio, bottom course, C/D	0.77	0.90	No hoop
Buckling ratio, bottom course C/D	2.42	2.85	Hoop

Table 5-4. Maple Reservoir, PGA = 0.30g

The results in Table 5-4 suggests that the tank shell should remain integral in earthquakes up to PGA = 0.30g, with buckling prevented by allowing for some resistance offered by the hoop stress offered by internal water pressure. At PGA = 0.3g, even with the recommended pipe upgrades, damage to the roof system is expected, as the roof rafters will likely buckle given wall uplift; roof sloshing might slightly aggravate the performance if the tank is more than 90% full at the time of the earthquake. However, we do not recommend upgrades to the roof, as the tank should retain water even at PGA = 0.30g motion.

Based on these findings, we recommend that flexible connectors be provided for all side entry inlet-outlet pipes or under-tank drain pipes that be impacted by tank wall uplift on the order of 4 to 12 inches. As part of the upgrade design effort, wall thicknesses and materials should be verified either by obtaining the original fabrication drawings, or using a steel wall thickness gage tester; if the walls are much thinner than assumed (i.e., as might be possible if the tank was built using a higher grade of steel), then the conclusions about tank wall buckling might change.

Figure 5-18. Two of three Inlet-Outlet Pipes for Maple Reservoir

6.0 Seismic Fragilities

Tables 6-1 through 6-7 provide fragilities for the various structures, tanks and equipment components discussed in this report.

The prediction of damage for the buildings is done using fragility curves. For each building or tank, a fragility curve is presented for each of four damage states:

- Collapse
- Extensive
- Moderate
- Slight

These damage states are descriptive. From the descriptions of the damage states provided in this section, the user can understand the nature and extent of the physical damage to a building type from the damage prediction output. From these descriptions, life-safety, societal and monetary losses which result from the damage can be estimated. Building damage can best be described in terms of the nature and extent of damage exhibited by its components (beams, columns, walls, ceilings, etc.). For example, such component damage descriptions as "shear walls are cracked", "ceiling tiles fell", "wall panels fell out", etc., used together with such terms as "some" and "most" would be sufficient to describe the nature and extent of overall building damage.

Damage to nonstructural components of buildings (i.e., architectural components, such as partition walls and ceilings, and building mechanical/electrical systems) primarily affect monetary and societal losses while damage to structural components (i.e., the gravity and lateral load resisting systems) of buildings affect the expected casualty estimates, as well as other losses. For this project, we have provided fragility curves for damage to the structural components, and separately for the nonstructural components.

Another characteristic of building damage is that it varies from "none" to "complete" as a continuous function of building deformations (building response). Wall cracks may vary from invisible or hairline cracks to cracks of several inches width. Furthermore, damage of different nature or form may occur at different building deformations. As it is impractical to linguistically describe building damage as a continuous function, it is necessary to develop general descriptions for ranges of damage.

This methodology describes extent and severity of damage to structural components of a building separately by one of four ranges of damage or damage states: slight, moderate, extensive, and complete. General descriptions of these damage states are provided for the two central offices with reference to observable damage incurred. Damage predictions resulting from this physical damage estimation method are then expressed in terms of the probability of a building being in any of these four damage states.

Control Building, Raw Water Pump Station, Soda Ash Building, Old Filter Building

Slight Structural Damage: Diagonal, stair-step hairline cracks on masonry wall surfaces; larger cracks around door and window openings in walls with large proportion of openings; movements of lintels.

Moderate Structural Damage: Most wall surfaces exhibit diagonal cracks; some of the walls exhibit larger diagonal cracks; masonry walls may have visible separation from diaphragms; few individual masonry units may fall off the walls.

Extensive Structural Damage: At walls near relatively large area of wall openings most walls have suffered extensive cracking; some parapets cracked. Beams (joists) may have moved relative to their supports.

Complete Structural Damage: Structure has collapsed or is in imminent danger of collapse due to in-plane or out-of-plane failure of the walls.
For the tanks, the damage states refer to specific levels of damage, as listed below.

Large Filter Building

Slight Structural Damage: Diagonal hairline cracks on masonry wall surfaces; larger cracks around door and window openings in walls with large proportion of openings; minor separation of walls from the floor and roof diaphragms.

Moderate Structural Damage: Most wall surfaces exhibit diagonal cracks; some of the shear walls have exceeded their yield capacities indicated by larger diagonal cracks; some walls may have visibly pulled away from the roof.

Extensive Structural Damage: Most shear walls have exceeded their yield capacities and some of the walls have exceeded their ultimate capacities indicated by large, through-the wall diagonal cracks and visibly buckled wall reinforcement; the diaphragms may exhibit cracking and separation along joints; partial collapse of the roof may result from failure of the wall-to-diaphragm anchorages or the connections of beams to walls.

Complete Structural Damage: Structure has collapsed or is in imminent danger of collapse due to failure of the wall anchorages or due to failure of the wall panels.

Chemical Building

Slight Structural Damage: Diagonal hairline cracks on most shear wall surfaces; minor concrete spalling at a few connections of precast members.

Moderate Structural Damage: Most shear wall surfaces exhibit diagonal cracks; some shear walls have exceeded their yield capacities indicated by larger cracks and concrete spalling at wall ends; observable distress or movement at connections of precast panel connections, some failures at metal inserts and welded connections.

Extensive Structural Damage: Most concrete shear walls have exceeded their yield capacities and some have exceeded their ultimate capacities indicated by large, through-the wall diagonal cracks, extensive spalling around the cracks and visibly buckled wall reinforcement; partial collapse of the roof may result from the failure of the wall-to-diaphragm anchorages and falling of wall panels.

Complete Structural Damage: Structure has collapsed or is in imminent danger of collapse due to failure of the wall anchorages or due to failure of the wall panels.

Steel Tanks

Slight Structural Damage: Minor damage to ladder connections, attached steel.

Moderate Structural Damage: Damage to roof from sloshing or minor wall uplifts. Tank holds water. Minor buckling of upper courses, without loss of water contents.

Extensive Structural Damage: Failure of attached side entry pipes due to wall uplift. Buckling of bottom course with tears and loss of water contents. Major roof damage. Generally repair cost a small to moderate percentage of total tank replacement cost.

Complete Structural Damage: Complete loss of structure; major bucking, rapid water loss; generally repair costs exceed cost to replace.

The basis for development of the fragility data for these selected structures is past earthquake experience data, calculations, and judgment.

For each item for which fragility data was developed, two building states were considered:

- Existing structure in its pre-earthquake condition
- Existing structure in its retrofitted condition

The fragility curves represent the damage attributable to the structural load bearing part of the building.

Structure	Slight		Moderate		Major		Collapse	
Case	A	Beta	A	Beta	A	Beta	A	Beta
1. As Is	0.13	0.60	0.17	0.60	0.26	0.60	0.37	0.60
2. Upgraded	0.18	0.50	0.30	0.50	0.49	0.50	0.87	0.50

Table 6-1. Fragilities – Control Building, Old Filter Building

Case	A	Beta	A	Beta	A	Beta	A	Beta
1. As Is	0.15	0.60	0.20	0.60	0.30	0.60	0.40	0.60
2. Upgraded	0.18	0.50	0.30	0.50	0.49	0.50	0.87	0.50

Table 6-2. Fragilities – Raw Water Pump Station Building

Case	A	Beta	A	Beta	A	Beta	A	Beta
1. As Is	0.12	0.60	0.16	0.60	0.24	0.60	0.34	0.60
2. Upgraded	0.18	0.50	0.30	0.50	0.49	0.50	0.87	0.50

Table 6-3. Fragilities – Soda Ash Building

Case	A	Beta	A	Beta	A	Beta	A	Beta
1. As Is	0.16	0.60	0.20	0.60	0.29	0.60	0.54	0.60
2. Upgraded	0.18	0.50	0.30	0.50	0.49	0.50	0.87	0.50

Table 6-4. Fragilities – Large Filter Building, High Pressure Pump Station Building

Case	A	Beta	A	Beta	A	Beta	A	Beta
1. As Is	0.13	0.60	0.17	0.60	0.25	0.60	0.45	0.60
2. Upgraded	0.18	0.50	0.30	0.50	0.49	0.50	0.87	0.50

Table 6-5. Fragilities – Chemical Building

Case	A	Beta	A	Beta	A	Beta	A	Beta
1. As Is	0.15	0.50	0.40	0.50	0.75	0.50	2.00	0.50
2. Upgraded	0.15	0.50	0.40	0.50	0.90	0.50	2.00	0.50

Table 6-6. Fragilities – Maple Reservoir

For equipment, the damage states are as follows:

- Storage Racks. Racks topple. Contents fall out.

- Floor Standing Cabinets. Racks topple leading to general equipment failure. Localized equipment failures within the racks (dislodging of cards, etc.) are not considered here, but likely under 1% of components.

- HVAC. Equipment displaces and is damaged.

Equipment	Floor Standing Cabinets		Storage Racks		HVAC on Roof	
Case	A	Beta	A	Beta	A	Beta
1. As Is	0.30	0.70	0.60	0.50	0.40	0.50
2. Upgraded	2.00	0.50	1.00	0.50	0.80	0.40

Table 6-7. Fragilities- Equipment

7.0 References

ALA (2001), Seismic Fragility Formulations for Water Systems, Eidinger, J. (chairman), American Lifelines Alliance.

Eidinger, J. Editor, Fire Following Earthquake, 2004, available at http://homepage.mac.com/eidinger/ and with Scawthorn and Schiff from ASCE, 2005.

CH2M-Hill, Vine Street Water Treatment Plant, Seismic Evaluation, June, 2005.

Hull A.G., Augello, A., Yeats, R.S., Deterministic seismic hazard analysis in northwest Oregon, Pacific Conference on Earthquake Engineering, 2003.

MWH, 2003, Water Facility Plan, City of Albany, prepared by Montgomery Watson Harza.