

City of Palo Alto Water System Pipe Replacement

*Prepared for:
City of Palo Alto*

Prepared by:

*G&E Engineering Systems Inc.
6315 Swainland Rd
Oakland, CA 94611
(510) 595-9453 eidinger@geEngineeringSystems.com*

*Principal Investigators:
John Eidinger, P.E., S.E., Darlene Holston P.E.*

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TABLE OF CONTENTS

EXECUTIVE SUMMARY	1
1.0 INTRODUCTION	3
1.1 KEY FINDINGS	4
1.2 ACKNOWLEDGEMENTS	7
1.3 ABBREVIATIONS	8
2.0 SYSTEM DESCRIPTION	9
2.1 SFPUC SYSTEM SERVING PALO ALTO	9
2.2 PALO ALTO WATER SYSTEM	11
2.2.1 Overall Description, Water Demand	11
2.2.2 Historical Development of the Palo Alto Water System, 1894 to 1957	14
2.2.3 Water Demands Under Earthquake Emergency Conditions	21
2.2.4 Water Facilities – Modern System (2015)	22
2.2.5 Water Distribution System Pipelines	26
2.2.6 Service Laterals, Hydrants and Meters	50
2.2.7 Water Facilities in the Foothill Pressure Zones	52
2.3 FOOTHILLS TRANSMISSION CCP PIPELINE	55
2.3.1 Phase 1. Quarry to Boronda Reservoir Section	55
2.3.2 Phase 2. Boronda Reservoir to Monte Bello Reservoir Section	61
2.3.3 Pipe Relocation	61
2.4 SOIL RESISTIVITY	62
2.5 ABANDONED PIPES	67
2.6 PALO ALTO INCOME	70
3.0 SEISMIC HAZARDS	72
3.1 SEISMOLOGY AND SCENARIO EARTHQUAKES	72
3.2 STREAMS	82
3.3 GEOTECHNICAL HAZARDS	87
3.4 GROUND SHAKING HAZARD	88
3.5 LIQUEFACTION HAZARD	91
3.6 LANDSLIDE HAZARD	117
3.7 FAULT OFFSET HAZARDS	120
3.8 PAST EARTHQUAKES	122
3.8.1 Great San Francisco Earthquake of April 1906	122
3.8.2 Loma Prieta Earthquake of 1989	124
3.8.3 Palo Alto Park Mutual Water Company	135
3.9 BORING DATABASE	138
4.0 SEISMIC ASSESSMENT OF ALL PIPELINES	142
4.1 SERA ANALYSIS OF SITES	144
4.2 SERA ANALYSIS OF PIPES	144
4.3 SERA ANALYSIS OF PIPES – LOMA PRIETA EARTHQUAKE	150
4.4 SERA RESULTS FOR 24 SCENARIO EARTHQUAKE	162
4.5 SERA RESULTS FOR THE SAN ANDREAS M 8.0 EARTHQUAKE	165
4.6 SERA RESULTS FOR THE MONTA VISTA M 6.8 EARTHQUAKE	167
5.0 SEISMIC ASSESSMENT OF FOOTHILLS PIPELINE	169
5.1 FIELD RECONNAISSANCE – LANDSLIDE AND LIQUEFACTION	169
5.1.1 Monte Bello Reservoir	171
5.1.2 Alignment Between Monte Bello Reservoir and Dahl Reservoir	176
5.1.3 Dahl Reservoir and Booster Pump Station Site	177

5.1.4 Alignment Along Arastradero Creek, Near Corte Madera Booster Station	177
5.1.4 Alignment at Corte Madera Booster Station.....	179
5.1.5 Alignment at Corte Madera Booster Station.....	180
5.1.6 Alignment along Old Page Mill Road	182
5.1.7 Quarry Booster Pump Station	186
5.1.8 Approaches to Refined the CCP Hazard Assessment	191
5.2 FOOTHILLS WATER SYSTEM PERFORMANCE AFTER SAN ANDREAS M 8	193
5.2.1 San Andreas M 8 – Pipeline Performance in Foothills (Zones 5, 6, 7, 8, 9)	194
5.2.2 San Andreas M 8 – Reservoir Drawdown and Fire Following Earthquake	196
5.3 PIPE REPAIR STRATEGIES.....	198
5.4 PIPE MITIGATION STRATEGIES.....	199
5.5 RECOMMENDATION FOR FOOTHILLS PIPELINES (SEISMIC)	206
6.0 PIPE AGING, REPAIR ANALYSIS AND PIPE REPLACEMENT	208
6.1 HISTORICAL PIPE REPAIRS	208
6.2 PIPE REPAIR RATE AS A FUNCTION OF MATERIAL AND DIAMETER	211
6.3 PIPE REPAIR RATE AS A FUNCTION OF AGE	216
6.3.1 Pipe Repair Rate as a Function of Age – AC Pipe	218
6.3.2 Pipe Repair Rate as a Function of Age – CI and PVC Pipe	223
6.4 PIPE REPAIR RATE AS A FUNCTION OF MONTH, YEAR.....	228
6.5 PIPE REPAIR RATE AS A FUNCTION OF SOIL CORROSIVITY	231
7.0 PIPE REPLACEMENT STRATEGIES.....	236
7.1 WATER MAIN REPLACEMENT PROGRAM	236
7.2 BENEFIT COST RATIO MODEL	242
7.3 PIPE REPLACEMENT FOR AGING ISSUES.....	243
7.3.1 Benefit Cost Analysis	243
7.3.2 Refined Pipe Replacement Strategy for Aging	247
7.3.3 Maps Showing Pipe Replacement due to Aging Issues	250
7.4 PIPE REPLACEMENT FOR SEISMIC ISSUES	280
7.4.1 Benefit Cost Model - Seismic	280
7.4.2 Seismic SIP-1, SIP-2, SIP-3, SIP-4	288
7.4.3 Seismic Projects	307
7.4.4 Water Main Risk Framework	338
8.0 FUNDING.....	343
9.0 REFERENCES	348
9.1 TECHNICAL	348
9.2 ELECTRONIC	350
9.3 DRAWINGS	351
APPENDIX A. SOIL RESISTIVITY TESTS	1
A.1 OVERVIEW	1
A.2 TEST RESULTS	2
A.3 REFERENCE	6
APPENDIX B. SEISMIC PERFORMANCE OF WATER PIPELINES IN THE 2014 NAPA EARTHQUAKE	1
APPENDIX C. ASBESTOS CEMENT CORROSION	1
C.1 PALO ALTO USAGE OF AC PIPE	1
C.2 HOW DOES AC PIPE FAIL?	2
C.3 EBMUD EXPERIENCE.....	3
C.4 EXTERNAL CORROSION	7

C.5 HARDNESS..... 7

C.6 ADDITIONAL EBMUD TESTS..... 8

APPENDIX D. PALO ALTO STANDARD PIPE INSTALLATION 1

D.1 EXAMPLE PALO ALTO AC PIPELINE INSTALLATION 1

D.2 AC PIPE SPECIFICATIONS – PALO ALTO 5

D.3 AC PIPE SPECIFICATIONS - CAPCO 6

Executive Summary

Beginning in the 1990s, the City of Palo Alto water department began a long term effort to replace water pipes, called the Water Main Replacement Program (WMRP). The work was originally planned out in 34 phases, with each phase generally being accomplished in about 1 year. Phases 1 through 24 are now essentially complete. Phases 25 through 34 have yet to be implemented, which would include about 145,000 feet (27.5) miles of pipe over the next decade. Phases 25 and 26 are currently (mid-2015) in various stages of design, but construction has not yet started.

This report presents a series of analyses that suggest that, going into the future, a different pipe replacement strategy might be adopted. Considering the pipes already replaced since the 1990s, it is evident that by implementing Phases 1 through 24, the City has already replaced many of the most leak-prone and deteriorated pipes; while most of the remaining pipes planned to be replaced in future Phases 25 through 34 are currently functioning as expected and have never leaked in the past two decades.

Given these findings, this report outlines a new pipe replacement program that can be implemented over the next decade. This new pipe replacement program includes about 13.5 of pipe, of which 2 miles are in deteriorated condition, 10 miles of pipe are seismically weak, and about 1.5 miles of pipe will deteriorate over the next decade to the point where they warrant replacement. The estimated cost for this pipe replacement program is \$2.92¹ million (aging) plus \$19.01 million (seismic) plus \$2.3 million (future deteriorated pipe) or about \$24.23 million in total.

In the longer term, once the next ten years of pipe replacement is completed, there will continue to be a need to replace pipe, both for aging / deterioration purposes as well as lower priority pipes for seismic purposes. The benefit cost ratios computed for each pipe in the water system can be used to rank which pipes should be replaced. Assuming the rate of deterioration stays much the same as it is today, a long term pipe replacement rate of about 1 mile per year appears to be a cost effective strategy.

The recommended pipe replacement program (13.5 miles) is about half the currently planned effort (27.5 miles) over the next decade. This recommended pipe replacement program would significantly reduce the capital requirements for pipeline replacement over the next decade, while still cost effectively addressing ongoing aging of pipes and seismic risks in Palo Alto.

¹ All costs presented in this report are in constant \$2015, and exclude the effects of inflation. The costs of inflation and financing should be factored into actual capital budgets.

This new pipe replacement program is based on the following principles:

- All of Palo Alto's existing water pipes are aging. Many of these pipes continue to perform their function with little to no maintenance, and have adequate seismic capability if they are located in soils that are not prone to significant seismically-induced ground deformations.
- Most water pipes should not be replaced until such time that they sustain sufficient deterioration as to make them unreliable. Other pipes, located in soils prone to liquefaction, landslide or surface faulting, could be cost-effectively replaced with suitably seismic-designed pipes, if the existing pipe's failure in earthquakes will result in high adverse economic impacts to Palo Alto's customers.
- The expenditure of current capital dollars for pipe replacement should be balanced against the benefit of fewer day-to-day leaks as well as fewer damaged pipelines in future earthquakes.
- It is current US national policy that the cost to repair damage caused by future major earthquakes to Palo Alto's water system will be reimbursed at about a 75% rate by FEMA. To the extent that the current cost for Palo Alto to mitigate seismic impacts exceeds the future benefits, this report suggests that it is more cost effective for Palo Alto to accept the impacts of some level of seismic damage, and make repairs after the earthquake, the cost of which will be substantially reimbursed by FEMA.
- This report provides quantified valuation of the cost effectiveness for replacement for each pipe. This is done by computing a benefit cost ratio (BCR) for each pipe, for pipe replacement due to aging issues, for pipe replacement for seismic issues, and for both issues combined. This report uses the combined BCR of 1 to set the dividing line as to which pipes should be currently replaced (1 or higher) or left in service (under 1). Should Palo Alto wish to be more risk adverse, then more length of pipe can be replaced, by selected a lower BCR dividing line value. The databases provide the numeric BCR ranking for every pipe.

1.0 Introduction

This report describes the seismic and aging performance and mitigation strategies for the City of Palo Alto water system.

The report is in several sections:

- Section 2. Describes the inventory of pipes and other facilities in the Palo Alto water system.
- Section 3. Describes the earthquake hazards in Palo Alto.
- Section 4 describes the system-wide performance of pipelines throughout Palo Alto in earthquakes
- Section 5 describes the seismic performance of the Concrete Cylinder Pipes in the Foothills.
- Section 6 describes leak analysis and pipe replacement strategies to address aging pipes.
- Section 7 provides a pipeline replacement program. This includes a pipe replacement program that addresses aging pipes (called the Aging Improvement Plan, AIP), and a combination of emergency response and pipe replacement strategy that addresses earthquakes (called the Seismic Improvement Plan, SIP). An electronic database is provided that lists the replacement priority for each pipe.
- Section 8. Funding.
- Section 9. References.
- Appendix A. Results for Soil Resistivity Tests.
- Appendix B. Calibration of earthquake pipeline fragility models, considering the recent Napa 2014 earthquake.
- Appendix C. Asbestos Cement pipe corrosion.
- Appendix D. Palo Alto Standard AC Pipe Installation.

1.1 Key Findings

The City of Palo Alto water system includes about 236² miles of transmission and distribution mains. Over the past two decades, the City has been selectively replacing older pipes.

² About 232.3 miles of pipes as of 2010.

Pipe Aging

The report examines the historic leak and repair rates for Palo Alto's water mains. This report provides tables that show the rate of repair by pipe diameter, by pipe material, by pipe age, and by soil resistivity. Using these historic repair rates, the report describes a Benefit Cost Ratio (BCR) model that is used to rank the cost effectiveness of a pipe replacement program that addresses aging issues only. Pipes with a BCR greater than 1 are cost effective for replacement for aging issues alone. The analysis shows that there are currently about 0.7 miles of pipe that have historically had such a high repair rate that current pipe replacement is cost effective. Considering that it is often practical to replace segments of pipe from valve to valve, or street intersection to street intersection, and in consideration of street moratoria and other practicalities, there are 1.95 miles of pipes recommended to be replaced due to aging issues, costing about \$2.92 million. The specific pipes recommended for replacement are listed in Tables 7-5 and 7-6.

Earthquakes

The report examines the performance of the Palo Alto water system for 24 different scenario earthquakes. By "scenario earthquake", it is meant that a particular earthquake is assumed to occur on a specific fault, with a specific epicenter and specific magnitude. The 24 scenario earthquakes include a complete range of possible earthquakes on the nearby San Andreas fault, from Magnitude 6.0 up to 8.0, as well as large earthquakes on other nearby active faults including the Hayward, Calaveras, San Gregorio, Greenville, Mount Diablo Thrust, and Rodgers Creek faults. Two scenario earthquakes were run that represent repeats of the historic 1989 Loma Prieta and 2014 West Napa earthquakes. Additional scenarios were run on less active nearby faults, for the Monte Vista - Shannon and Zayante – Vergeles fault zones.

Depending on the earthquake, pipe damage in the Palo Alto water system can range from none or a few pipe repairs (for smaller or more distant earthquakes), up to 200 to 300 pipe repairs for a large earthquake on the nearby San Andreas fault. The bulk of the repairs will occur to older "non-seismic-designed" pipes located in areas prone to liquefaction. By "non-seismic-designed" pipes, it is meant older cast iron and asbestos cement pipes, as well as newer ductile iron and PVC pipes with push-on-type joints. Of these, cast iron and asbestos cement pipes are the worst performers when located in soils that suffer permanent ground deformations due to liquefaction or similar phenomena.

Given the pipe damage, and assuming that other facilities are suitably seismically sound (water tanks, booster pump stations, wells, service connections to the SFPUC), water outages were estimated due to the pipe repairs. System hydraulics for damaged pipeline systems, were considered as well as the time needed to repair the broken pipes. The system hydraulics depends primarily on the amount of pipe damage, while the repair times depend mostly on the number of pipe repair crews available. The gross regional product of Palo Alto was established, and the impact on the economy should there be a loss of water supply was estimated.

For the existing water system, it was assumed that the pipeline network reflects how it is currently configured, plus using only Palo Alto's in-house pipe repair capability (2 crews). In this case, the economic impact of a San Andreas M 8.0 earthquake is substantial: about \$477 million in direct economic impacts attributable to damage to the pipeline network. This includes loss of economic activity due to loss of water (about 75 to 85% of the total); the amount of damage in Palo Alto due to fires that burned structures (about 15 to 20% of the total), aggravated by the lack of water in many places, and the cost to make the actual pipe repairs (about 1% of the total).

Four possible strategies were considered to reduce the economic impacts due to pipe damage in earthquakes. These are called Seismic Improvement Plans SIP-1, SIP-2, SIP-3 and SIP-4. SIP-1 is meant to be the lowest cost with the highest benefit; SIP-2 includes SIP-1, and adds in upgrade of the most critical water pipes; SIP-3 includes SIP-2, and adds in upgrade of additional pipes; SIP-4 includes SIP-3 and adds in replacement of additional pipes. Table 1-1 lists the costs.

Item	SIP-1	SIP-2	SIP-3	SIP-4
Emergency Response	\$1,000,000	\$1,000,000	\$1,000,000	\$1,000,000
Foothills Pipeline		\$738,000	\$738,000	\$738,000
Seismic Projects				
1. 17,932 ft, Zone 3		\$6,049,820	\$6,049,820	\$6,049,820
2. 127 ft, El Camino R		\$35,560	\$35,560	\$35,560
3. 746 feet, Wilkie		\$156,660	\$156,660	\$156,660
4. 332 ft, Laguna		\$123,400	\$123,400	\$123,400
5. 576 ft, El Camino R		\$165,720	\$165,720	\$165,720
6. 2,857 ft. Los Robles		\$943,720	\$943,720	\$943,720
7. 1,203 ft. Matadero		\$278,350	\$278,350	\$278,350
8. 8,735 ft. Embarcadero		\$3,399,710	\$3,399,710	\$3,399,710
9. 911 ft. Middlefield		\$298,400	\$298,400	\$298,400
10. 1,103 ft. Park		\$364,360	\$364,360	\$364,360
11. 18,546 ft. Zone 1			\$5,452,090	\$5,452,090
12. 66,380 feet, Zone 1				\$16,616,160
12. 24,645 feet, Zone 1 Ext				\$6,370,060
13. 3,576 feet, Zone 1 Other				\$2,627,560
Total	\$1,000,000	\$13,553,700	\$19,005,790	\$44,619,570
Recommended	Very High	High	Over 10 Years	Marginal

Table 1-1. Seismic Upgrades – Pipes – Priority SIP-1, SIP-2, SIP-3 and SIP-4.

Tables 7-13 through 7-33 list the specific pipes included in Table 1-1. Total pipe length recommended for replacement for seismic reasons is 53,068 feet (10.05 miles), with an estimated cost of \$19,005,790.

The attached databases gives the benefit cost ratios for every pipe in the water system, both for aging and seismic reasons.

Overall.

Beginning in about 1990, the Palo Alto water department began a long term effort to replace water pipes, called the Water Main Replacement Program (WMRP). The work was originally planned out in 34 phases, with each phase generally being accomplished in about 1 year. Phases 1 through 24 are now essentially complete. Phases 25 through 34 have yet to be implemented, which would include about 145,000 feet (27.5) miles of pipe over the next decade or so.

The current analysis suggests that there are currently about (3,442 feet plus 6,874 feet, total 1.95 miles of pipe that should be replaced based on aging issues (total of Tables 7-5 and 7-6). This will cost about \$2.92 million. This is a lot less than the 27.5 miles planned in the current WMRP. Over time, as more pipes begin to regularly leak, it will be cost effective to replace more than this initial 1.95 mile estimate, and the BCR model described in this report can be updated as new pipe repair data becomes available. Even so, the clear observation is that replacement of pipes for aging issues along can be reduced, while still being cost effective.

The current analysis also shows that a pipe replacement program to address seismic issues is warranted. The current WMRP is not geared to lower materially improve the water system for earthquakes, for two reasons: first, most of the pipes selected for replacement in the current WMRP are already located in geologically stable soils, and thus do not present very large seismic risks; second, the replacement pipes being adopted, being PVC-C900, are not seismically sufficient to be reliable in liquefaction zones. We recommend that new pipes installed in zones prone to liquefaction be seismically-sound, and these might include: HDPE (fusion welded); PVCO (a new PVC product that is reported to be able to sustain over 1% ground strain); ductile iron seismic-chained pipe from Kubota; similar ductile iron pipe from US Pipe (or others) with chained joints; heavy wall steel pipe. All metal pipes need suitable corrosion protection systems if placed in soils with low resistivity (Rho much under 3,000 ohm-cm, which is most of the flat-land areas of Palo Alto). In areas with Rho much under 1,500 ohm-cm, an initial preference for use of plastic pipes might be suitable.

Given these findings, we suggest that Palo Alto updated its WMRP, and develop a new ten-year program to upgrade about 12 miles of pipe. These pipes should be selected from Tables 7-5, 7-6 (Aging pipes) and Table 7-34 (SIP-2 and SIP-3 pipes).

Pipes will continue to deteriorate over time in Palo Alto. As described in this report, this deterioration will manifest itself within increasing rates of pipe leaks on select pipes. There is no simple formula that will predict when each individual pipe will reach its cost effective replacement age; but monitoring ongoing leak rates is a very sound indicator. Based on the current trends, it appears that about 1,000 to 2,000 feet of pipe will be annually become so deteriorated as to warrant replacement. The BCR model for pipe aging could be updated with additional leak data once a year, and this model will identify those pipes that are good candidates for near-term replacement. The cost effective

replacement rate for pipes that are not yet leaking, and which are not identified as candidates for seismic pipe replacement, will likely be about 0.2 to 0.4 miles per year over the next decade.

The total recommended pipeline replacement program for the next decade is thus composed of three elements: replacement of pipe that are currently deteriorated (

1.2 Acknowledgements

This report was prepared by John Eidinger and Darlene Holston of G&E Engineering Systems Inc. Dr. Donald Duggan assisted with soil resistivity testing. Dr. Stephen Dickenson developed information about the Foothills pipelines. Mr. Bruce Maison (retired, EBMUD) and Prof. Mike O'Rourke (Rensselaer Polytechnic Institute) provided review of pipeline seismic and aging models. Ms. Jennifer Cioffi and Mr. Romel Antonio of the Palo Alto Utilities Department directed the work.

1.3 Abbreviations

ACP	Asbestos Cement Pipe
BDPL	Bay Division Pipeline
CCP	Concrete Cylinder Pipe
CGS	California Geological Survey
CIP	Cast Iron Pipe
cm	Centimeter
DIP	Ductile Iron Pipe
FEMA	Federal Emergency Management Agency
Fy	Yield stress for steel
g	acceleration of gravity (=32.2 feet / second / second)
G&E	G&E Engineering Systems Inc.
GIS	Geographic Information System
km	kilometer
ksi	kips per square inch (1 kip = 1,000 pounds)
M	Magnitude (Moment)
MG	Million Gallon
NEHRP	National Earthquake Hazards Reduction Program soil classifications A, B, C, D, E, F
NS	North South
PCCP	Prestressed Concrete Cylinder Pipe
PE	Polyethylene Pipe
PGA	Peak Ground Acceleration (measured in g)
PGD	Permanent Ground Deformation (measured in inches, cm)
PGV	Peak Ground Velocity (measured in inches/sec, cm/sec)
psf	pounds per square foot
psi	pounds per square inch
PVC	Polyvinyl Chloride Pipe
SFPUC	San Francisco Public Utilities Commission
ULDH	Ultra Large Diameter Hose
VCP	Vitrified Clay Pipe
Vs30	Average shear wave speed over top 30 meters of soil, meters / second
WMRP	Water Main Replacement Program

2.0 System Description

2.1 SFPUC System Serving Palo Alto

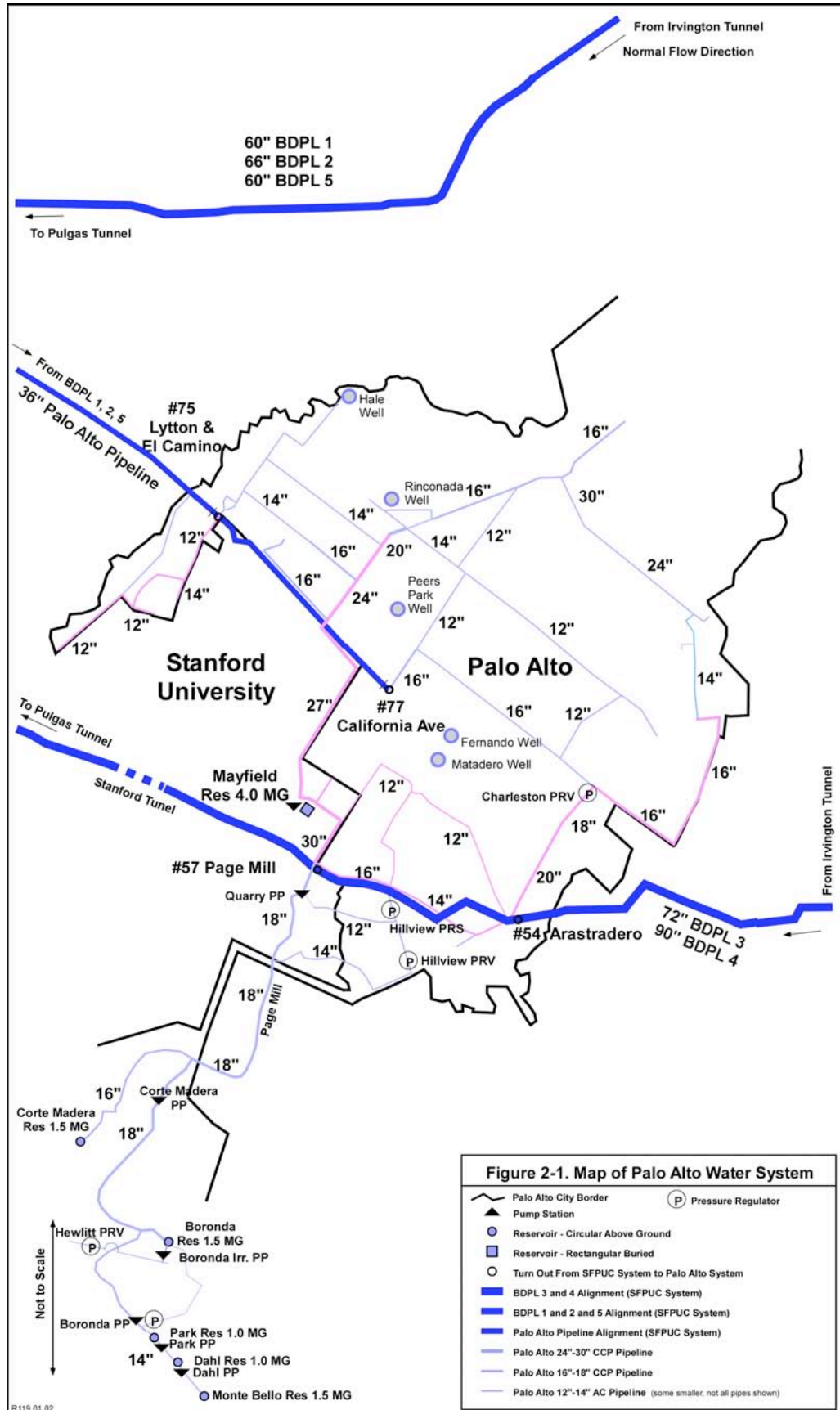
The City of Palo Alto Utilities Department owns and operates the potable water distribution system for the City of Palo Alto.

Figure 2-1 shows an overall view of the Palo Alto water system. This map shows the SFPUC pipelines that deliver potable water to Palo Alto, the major pipes within the Palo Alto water system itself, and the major facilities (booster pump stations, tanks and reservoirs, and wells) in the City water system.

There are five turnouts which connect the SFPUC system with the Palo Alto system, two of which are on the Palo Alto pipeline, and 2 of which are on BDPL 3 and 4 pipelines:

- Near the intersection of El Camino and Lytton is turnout #75 from the 36-inch SFPUC Palo Alto Pipeline to the City of Palo Alto distribution system. Near the intersection of El Camino and California Ave is turnout #77 from the 36-inch SFPUC Palo Alto Pipeline to the City of Palo Alto distribution system. A third turnout is located near Sand Hill Road, from the Palo Alto pipeline to the Palo Alto distribution system.
- Near Arastradero is turnout #54 from the 72-inch and 90-inch SFPUC BDPL 3 and 4 pipelines to the City of Palo Alto distribution system. Near the intersection of Page Mill Road and Junipero Serra Blvd is turnout #57 from the 72-inch and 90-inch SFPUC BDPL 3 and 4 Pipelines to the City of Palo Alto distribution system.

The water in the SFPUC system is normally potable. Under normal conditions, this water comes from the Hetch Hetchy reservoir in Yosemite National Park, and / or from Calaveras reservoir via the Sunol Water Treatment Plant.



2.2 Palo Alto Water System

2.2.1 Overall Description, Water Demand

The Palo Alto water system serves the City of Palo Alto and a small portion of nearby unincorporated areas. The Palo Alto water system does not serve Stanford University.

The water system is divided into nine main and three smaller pressure zones. Figure 2-2 shows a hydraulic profile of the modern (2015) water system. Figure 2-3 shows the geographic areas of each main zone.

The lower zones (zones 1, 2, 3) are gravity zones, normally served by pressure in the SFPUC's BDPL 1, 2, 3, 4, 5 and Palo Alto pipelines. About 90% of all customers are in the gravity zones. The Quarry Zone (zone 4) gets water from a pressure regulator from Zone 5, or by opening the valves between Zones 2 and 4. The Lytton pump station can boost pressure and flows into Zone 3.

There are five pumped zones (zones 5, 6, 7, 8, 9), extending almost to the highest elevation of the Peninsula mountains, with the highest served area at about elevation 2,400 feet. The function of the upper-most zones (Monte Bello, Dahl, Park) is mostly to provide water for fire fighting purposes in the upper elevations; there are a few customers in these areas. About 10% of all customers are in the pumped zones.

There are several wells in the system. Zones 1 and 2 are situated over a ground water basin. Prior to the construction of the SFPUC system (pre-1935 or so), all of the water in Palo Alto was taken via wells from the ground water basin, with 10 wells in service in the 1950s. By 1999, there were five wells remaining, of which Hale, Rinconada and Peer Park were then in then in-service with Fernando and Matadero then out-of-service. Since 2000, three additional wells have been constructed.

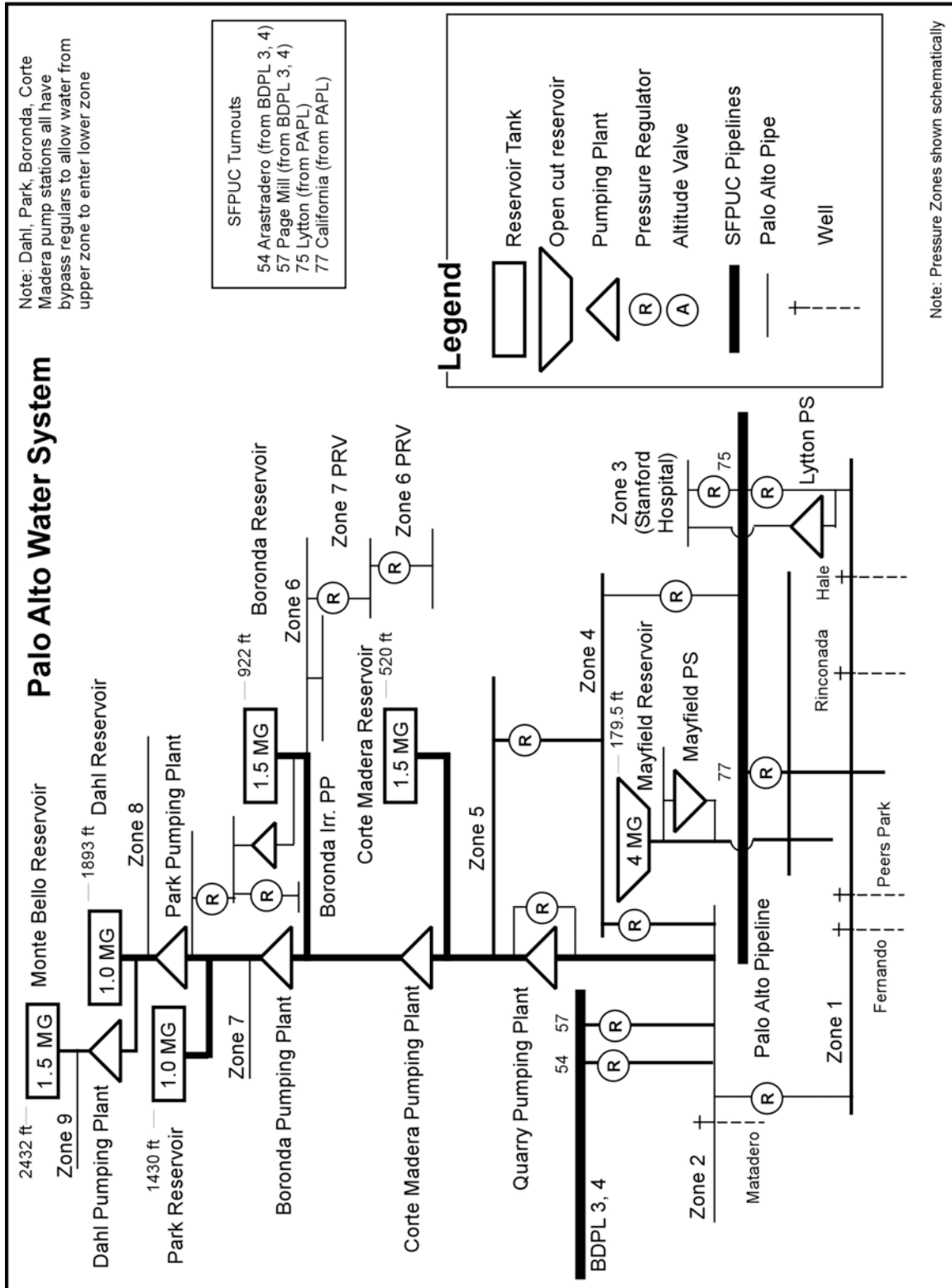


Figure 2-2. Schematic Profile

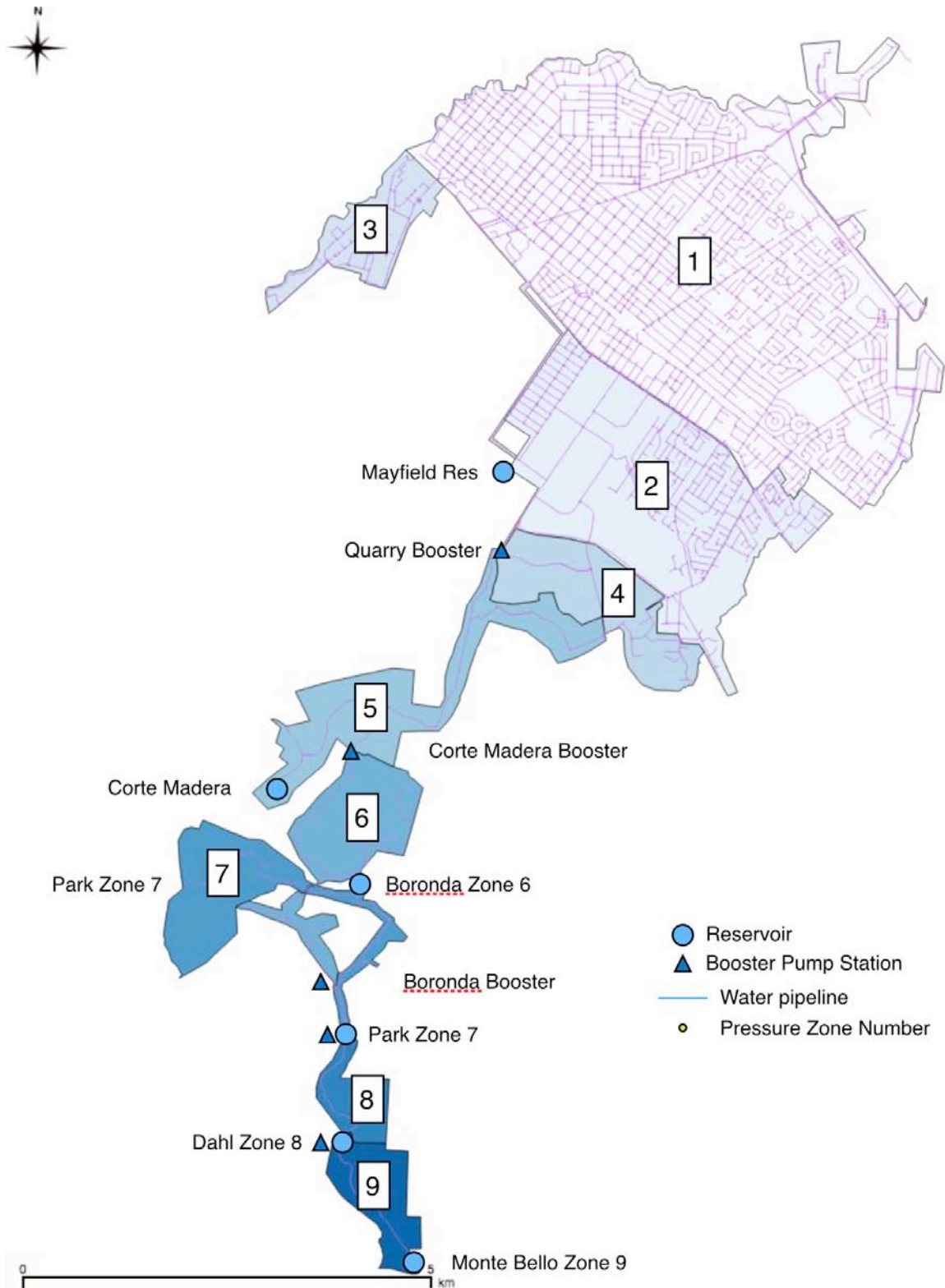


Figure 2-3. Pressure Zones

Based on a planning report from 1999 (Carollo, 1999), average day water demand (ADD) in the Palo Alto system was then about 13.8 MGD (FY 1999-2000). Annual Maximum Day water demand varied from 15 MGD to about 32 MGD during the time period from 1982 to 1998, and is expected to be about 32 MGD at build-out of the city.

By 1999, Palo Alto regularly used only one normal source of water, being the SFPUC system at turnouts 54, 57, 75, 77. As of 2015, Palo Alto continues to use nearly all of its normal water supply from the SFPUC, with local wells providing backup / emergency supply.

In the year 2000, there were five wells in Palo Alto (see Figure 2-1). These wells were taken out of normal use in 1962, when the City of Palo Alto switched over to the SFPUC system for 100% of its normal water supply. This switchover to the SFPUC system was in part a response to regional subsidence and water quality concerns. Today (2015), there are 7 wells in the Palo Alto water system, with two new wells being added over the past decade. The wells are not now (as of 2015) in normal use. The nominal combined capacity of the older five wells is 4,300 gpm, of which 3,575 gpm is currently deemed operational at three of the five wells (6.2 MGD total / 5.1 MGD operational). Under an emergency such as a large earthquake, it is reasonable to use the wells to supply water to Palo Alto, recognizing that normal water quality will be compromised.

2.2.2 Historical Development of the Palo Alto Water System, 1894 to 1957

To appreciate the configuration of the water system as of 2015, a review of the historical development of the water system is in order.

The earliest recorded history of Palo Alto dates from 1769. It is assumed that the water supply of the small community of the time was taken via surface water from the various creeks that traverse Palo Alto.

By 1855, the township of Mayfield was formed. By 1906, the northern part of Palo Alto was developed. Figure 2-4 shows a historical map of the San Francisco Peninsula and the major Spring Valley Water Company (SVWC) water works at the time of the 1906 earthquake. Figure 2-5 (modified from USGS Topographic Map of Palo Alto, California, 1/62,500, 1899) indicates that two areas of Palo Alto were then developed:

- Between Alma and Middlefield, Embarcadero and San Francisquito Creek (northwest part of modern Zone 1).
- Near the modern location of Mayfield Reservoir (modern Zone 2).



Figure 2-4. Water Works, circa 1906

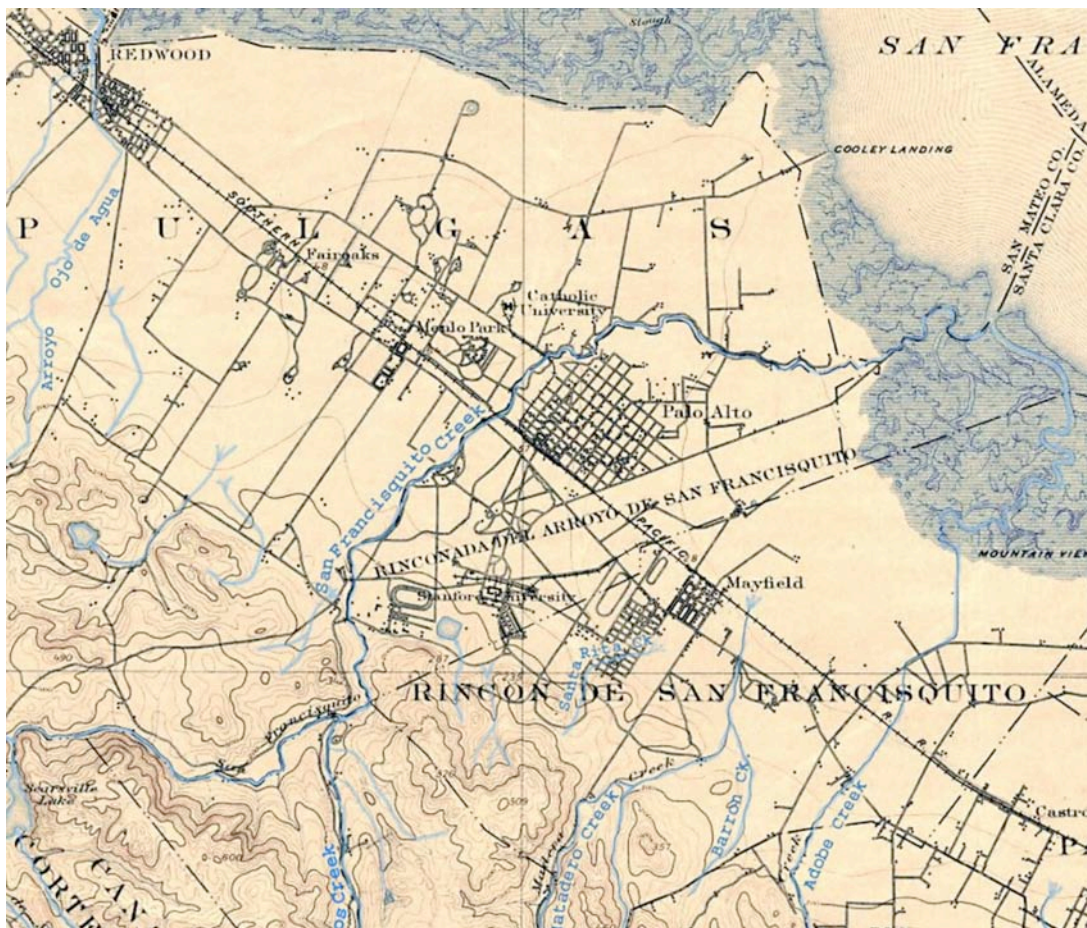


Figure 2-5. Topographic Map, Palo Alto Area, 1899

1896. The City of Palo Alto was incorporated in 1894. Prior to 1894, the small population centers (highlighted in Figure 2-5) were served by a number of small, privately-owned water companies, each of which operated one or more relatively shallow wells. In 1896, two years after incorporation, the City of Palo Alto Utilities (CPAU) began, with a bond issue for purchase by the city of the companies then serving the majority of the population. Since that time, the system was expanded both by construction of new facilities and by additional purchases of private companies. The Barron Park Water Company was purchased in 1953, having about 400 water services. The Spinks Water Company was purchased in January 1951.

1923. BDPL 1 is constructed. This is the first pipeline bringing Hetch Hetchy water to the San Francisco Bay area, terminating at Pulgas Tunnel and then emptying into Crystal Springs reservoir. For much of its length, it is a 60-inch riveted steel pipeline. BDPL 1 remains in use in 2015 (the Bay Crossing reach is expected to be abandoned in the next few years).

1932. BDPL 2 is constructed. This is the second pipeline bringing Hetch Hetchy water to communities around the San Francisco Bay, terminating at Pulgas Tunnel and then emptying into Crystal Springs reservoir. For much of its length, it is a 66-inch welded steel pipeline. It remains in use in 2015 (the Bay Crossing reach is expected to be abandoned in the next few years).

1938. The Palo Alto Pipeline (PAPL) is a 36-inch pipeline, and is a part of the SFPUC water transmission system. This pipeline is connected to BDPL 1 and 2 in Redwood City, and then continues southeastward to Palo Alto where it terminates. Water supply from the SFPUC Palo Alto 36-inch pipeline is believed to have started in 1938.

There are two turnouts from the PAPL to Palo Alto. Using the SFPUC's numbering system (see Figure 2-2), turnout 75 is a 12-inch diameter turnout, located at El Camino and Lytton Avenue; turnout 77 is a 12-inch diameter turnout, located at El Camino and California Street. Both connections are equipped with meters and pressure reducing valves. The maximum grade line in the PAPL at times of low flow in the SFPUC system might be on the order of 320 feet, with minimum about 280 feet or so, which would imply a upper pressure at sea level in Zone 1 of about 139 psi or so; at this pressure, water would overflow in the Mayfield reservoir (overflow 179.5 feet); therefore, the turnouts have pressure reducing valves.

In 1948, a consulting firm, Forbes and Jenks, examined the problem of turbidity and taste and odor in the Palo Alto water system. They recommended that a water treatment plant be constructed in Palo Alto. This was not built. It was then suggested that purchased water from the PAPL (from SFPUC) was resulting in general tuberculation, presumed for the unlined cast iron pipes, then in common use. It was thought that SFPUC water was under saturated in dissolved salts, principally calcium carbonate.

In 1950, a consulting firm, Burns and McDonnell, examined the problem of "red water" in the Palo Alto water system. They attributed the red water condition to the presence of iron bacteria, with the SFPUC upstream aqueduct tunnels as the most likely source of the infestation. They also suggested construction of a water treatment plant in Palo Alto to treat both SFPUC water as well as local well water.

In 1952, Burns and McDonnell indicated the possibility of softening only the local well water, and then mixing it with untreated SFPUC water. They also envisioned future water sources for Palo Alto from the impoundment of San Francisquito Creek (never happened) or from the Feather River (ultimately called the State Water Project, with water now being delivered to Santa Clara County for SCVWD's transmission system). While Palo Alto is in Santa Clara County, and is eligible to receive surface water from the SCVWD, there has never been a SCVWD pipeline extended to Palo Alto to make such surface water deliveries; and this report will not make any such recommendation for purposes of seismic reliability.

In 1948, the SFPUC completed construction of BDPL 3, along an alignment to the south of the PAPL. Water supply via BDPL 3 is from the same SFPUC source as PAPL. The maximum grade line in BDPL 3 at times of low flow in the SFPUC system, pressure within BDPL 3 might be as high as 320 feet or as low as 280 feet or so, which would imply an upper pressure at sea level in Zone 1 of about 139 psi or so. In 2015, water from BDPL 3 (and BDPL 4) is delivered into Zone 2 via pressure reducing valves; from Zone 2, water flows by gravity into Zone 1 (via pressure reducing valve) or into / from Zone 4 (similar elevation as Zone 2. As Zone 4 is slightly higher in elevation than Zone 2, normal flow into Zone 4 is via the Quarry booster pump station and Zone 5 via a pressure reducing valve. Should the Quarry pump station not be available, a gate valve can be opened between Zones 2 and 4.

In 1954, Brown and Caldwell reported that the SFPUC began, as of January 1951, treatment of SFPUC aqueduct water with lime, in order to maintain a condition which would inhibit corrosive action in metallic distribution pipe. It was recommended that dosing of water at each source with a polyphosphate, such as sodium hexametaphosphate; this type of treatment would inhibit corrosion and minimize the precipitation of calcium, magnesium, manganese and iron. Following that recommendation, chemical feeders were installed and polyphosphate treatment was started at all stations. Results thus obtained were reported to be generally excellent.

Figure 2-6 shows the water supply for Palo Alto, as of c. 1956. In 1956, there were 11 stations with 15 wells. The well capacity, as of 1956, assuming peak capacity of each well, was about 11 MGD. It was believed that the ground water basin in northern Santa Clara Valley, was being over drafted, and the potential of drilling new wells and increasing water supply was thought limited. Older wells were showing signs of deterioration, and were expected to decline both in yield and quality. If pumping was kept high, the yield was expected to drop to perhaps 8 MGD. However, if the wells were kept as emergency supply, and the ground water basin allowed to recover, then possibly yield on an emergency basis would increase.

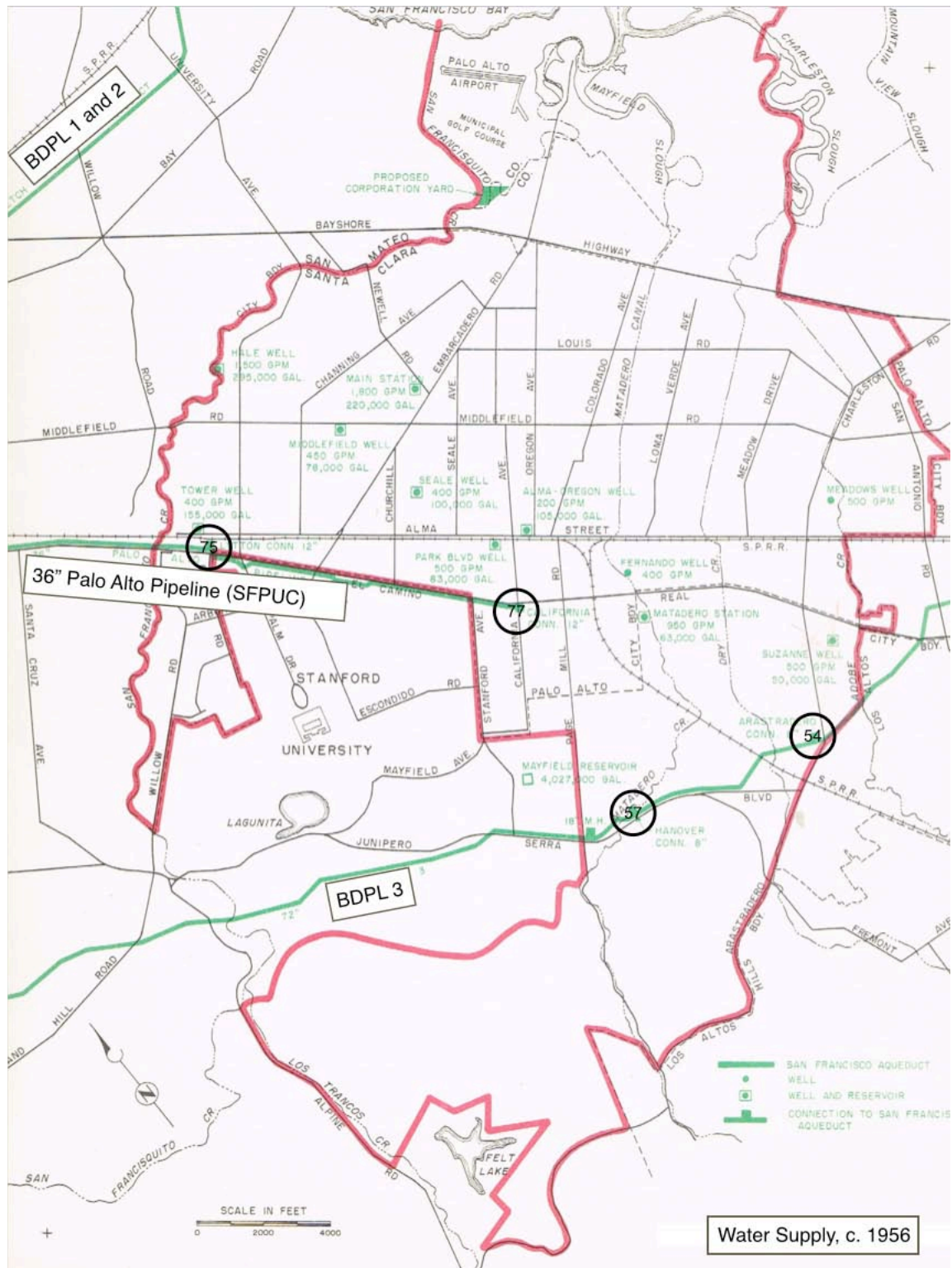


Figure 2-6. Palo Alto Water Supply, Circa 1956

In 1957, the cost of well water was about \$15 per acre-foot, and the cost of SFPUC water was about \$60 per acre-foot (in 1957 dollars). There was a desire to keep overall cost of water as low as practically consistent with the other needs of the water system, so

therefore, the concept was to ensure that the maximum reliable supply from wells could be used consistently throughout the year, supplementing local well water with SFPUC water during peak demand months.

The lengths and sizes of water pipes (6-inches and larger) in the distribution system in 1956 was 118.2 miles, with a breakdown in Table 2-1.

Diameter (inches)	Cast Iron (feet)	Asbestos Cement (feet)	Other (feet)	Total (feet)
6	161,200	240,600	3,100	404,900
8	45,400	100,600	200	146,200
10	18,000	5,900	100	24,000
12	7,400	32,600	900	40,900
14	600			6,000
16	5,800			5,800
18	1,500			1,500
Total (feet)	239,900	379,700	4,300	623,900
Total (miles)	45.4	71.9	0.8	118.2

Table 2-1. Length of Pipes, as of July 1 1956

In addition, as of July 1 1956, there were 339,100 feet (64.2 miles) of service lines, either ¾-inches (common), 1-inch (most common) or greater than 1 inch (limited).

The Cast Iron pipe are believed to be unlined bell and spigot pipe. The AC pipe are believed to have begun being installed around 1940, using rubber ring compression joints.

Brown and Caldwell (1957) report that older sections of the system had various combination of one, two three and four valves at main intersections. In some cases, the one and two valves intersections result in runs as long as 1,600 feet without a shut-off. Post 1960, it can be assumed that new four-way intersections all have three valves.

In 1957, Brown and Caldwell developed a planning document as to how to improve the Palo Alto water system. The following outlines their findings.

By 1957, water supplies were adequate, with "practically unlimited quantities" available by purchase from the SFPUC. Because, however, due to the high cost of SFPUC water, most water use in Palo Alto was from Palo Alto's own wells. The goals of the design of the water system post-1957 included:

- Ensure full use of the local ground water consistent with the safety of the ground water basin.
- Anticipate short term outages of the SFPUC water system.

- Prior to 1957, there were deficiencies in the water system, including complaints of low pressure (in some areas as low as 10 psi); taste and odor and turbidity complaints; control and instrumentation systems that were outdated.

The plan outlined in 1957 was geared to resolve these issues. Much of the infrastructure now in place (2015) reflects the improvements contemplated in 1957.

In 1962, the City of Palo Alto began purchasing nearly its entire water supply from SFPUC, largely discontinuing the use of wells. According to Palo Alto staff, this decision was based on input from the community, with the desire to have higher quality Hetch Hetchy water rather than the local ground water, even if that SFPUC water was more expensive.

In 1970, the SFPUC completed construction of BDPL 4, along an alignment parallel to BDPL 3.

In 1989, the Loma Prieta earthquake occurred. In the 1989 Loma Prieta earthquake, there was surface evidence of liquefaction in very locations in Palo Alto, near the Bay shoreline. The level of ground shaking in Palo Alto in that earthquake was likely in the $PGA = 0.10g$ to $0.15g$ range. The duration of strong ground shaking (PGA over $0.05g$) was likely in the 6 to 8 second range. Within the Palo Alto water system service area, there were few liquefaction zones and few landslides in the hills. Section 3.8.2 describes the known damage to have occurred to Palo Alto's water tanks, pump stations, pipeline and wells in the Loma Prieta earthquake. A joint pulled apart (but did not leak) in BDPL 2 where it transitions from the Dumbarton Strait reach (soon to be taken out of service) to the on-land portion in East Palo Alto.

In 2015 (or soon thereafter), the SFPUC will complete BDPL 5, including a new Bay Tunnel, along an alignment parallel to BDPL 1 and 2. As of March, 2015, water is flowing in the new Bay Tunnel. Water from BDPL 1, 2, and 5 will be able to be delivered to Palo Alto via the 36-inch PAPL. Soon thereafter, the SFPUC currently intends to retire portions of BDPL 1 and 2 for their 5-mile reach across the Bay; retirement of those two older pipelines that cross the Bay will reduce the reliability of water supply to Palo Alto, especially should the new Bay Tunnel be taken out of service (for maintenance) or due to unforeseen occurrence (damage for any reason); the cost savings to the SFPUC from avoiding future maintenance on BDPL 1 and 2 may be modest compared to the material impacts to SFPUC customers should the Bay Tunnel be damaged, especially with concurrent damage to BDPL 3 and 4. Although the risk of damage to all the BDPL pipelines at the same time, even in an earthquake, is thought to be small by design, the reduction in seismic reliability due to construction defects (such as those in BDPL 5 that were not mitigated) or unexpected seismic hazards cannot be entirely ruled out. Given these issues, keeping some or all of Palo Alto's wells in service would serve to provide a source of water after earthquakes, should the SFPUC transmission system be damaged.

2.2.3 Water Demands Under Earthquake Emergency Conditions

Under earthquake emergency conditions, loss of water supply from SFPUC turnouts 54, 57, 75, and 77 would begin to impact customers when local Palo Alto water storage is emptied, assuming no re-supply from the Palo Alto wells, and assuming there was no damage to the Palo Alto distribution system.

For planning purposes, it is assumed that it is desirable to provide at least Average Day Demand (ADD) indefinitely after major earthquakes. If the earthquake were to take place during summer time conditions, immediate water rationing would be put in place, whereby outside irrigation would be curtailed until such time that the water system could reliably deliver full summer time flows.

It is recognized that within the first hours to days after a major earthquake, there may be variations in water demand due to the following issues:

- Damage to distribution system pipelines can cause severe leakage of water in the system, resulting in drop in pressure until such time that the leaking pipelines can be valved out, and then eventually repaired. Leaks can occur in Palo Alto-owned distribution pipe as well as customer owned service connections.
- Fires may ignite after a major earthquake. If these fires spread substantially, there may be a material increase in water demand in the system for purposes of fire fighting. In the Oakland Hills firestorm of 1991, peak water demands used for fire fighting reached about 30,000 gpm (43 MGD) for about 48 hours after the initial ignition.
- Concurrent earthquake damage to residential and commercial and industrial customers will alter normal water demands by those users. Ideally, it is desired to be able to restore at least ADD to all customers within 3 days after a major earthquake; in this way, the water system will not be the limiting factor in restoring the local economy to pre-earthquake levels.

2.2.4 Water Facilities – Modern System (2015)

As of 2015, there are six primary water storage tanks in the Palo Alto system. These are listed in Table 2-2.

Reservoir	Capacity (Million Gallons)	Overflow Elev, Feet	Bottom Elev, Feet	Diam, Feet	Height, Feet	Description
Mayfield	4.027	179.5	163.0	208 x 208	16.5	Rectangular, in-ground, concrete lined. Lightweight roof, known to suffer from corrosion; recently the roof was replaced. Includes altitude valve. Built 1928.
Corte Madera	1.5	520	478	80	42	Steel tank at grade, built 1969.
Boronda	1.5	922	894	96	28	Prestressed concrete tank, built 1960. 4 feet below grade, 24 feet above grade. Walls are 8-inches thick, including 1" to 1.5" spray on gunite over prestressing wires. Roof is 8.5" flat concrete slab.
Park	1.0	1430	1402	80	28	Steel tank at grade, built 1965. Unanchored on ring beam. Flat steel roof. Scheduled to be seismically upgraded.
Dahl	1.0	1893	1871	90	22	Steel tank at grade, built 1965. Unanchored on ring beam. Flat steel roof. Scheduled to be seismically upgraded.
Monte Bello	1.5	2432	2405	100	27	Steel tank at grade, built 1965. Unanchored on ring beam. Flat steel roof. Scheduled to be seismically upgraded.
Total	10.5					

Table 2-2. Water Storage Tanks

Note: we have not surveyed the elevations of the reservoirs. Older documents likely provide elevations in 1928 vertical datums (or other), while most modern elevations usually use the 1988 NAVD. For example, the Carollo (1999) report lists the overflow of Mayfield at 179.5 feet, while the 1957 Brown and Caldwell report reports it at 167.8 feet. The overflow and bottom elevations listed in Table 2-2 are based on various source documents, some of which may use conflicting vertical datums; it is recommended that these be verified with modern survey before being used for design-related purposes.

In 2010, V&A conducted field measurements of the water tanks. Pertinent information from that report is tabulated in Table 2-3. Steel plate thicknesses are based on ultrasonic testing.

Reservoir	
Mayfield	Repair cracks in sloped concrete liner. Replace corrugated metal roof sheets. Lead found in vault pipe coating. Concrete liner is 6 inches thick (bottom) or 4 inches thick (sloped sides). 1928-vintage roof is corrugated metal supported by timber beams and concrete columns. Many cracks observed in 2010 in the sloped concrete liner, some previously repaired.
Corte Madera	Lead found in exterior tank and vault pipe coating. Recommended recoat of interior and exterior surfaces of tank. Center column. Five courses (0.67, 0.551, 0.438, 0.290, 0.271 inches, bottom to top, 0.198 inches roof, 0.474 inches columns)
Boronda	Repair roof cracks. Patch exterior crack exposing pre-stressing wire. Lead found in vault pipe coating. Concrete roof supported by center column and 6 columns.
Park	Lead found in exterior tank and vault pipe coating. Recommended recoat of interior and exterior surfaces of tank. Center column and 5 columns; all columns are pipe-type. Bottom level mixing pipe. Four courses (0.454, 0.353, 0.282, 0.236 inches, bottom to top, 0.206 inches roof, 0.269 inches columns)
Dahl	Lead found in exterior tank and vault pipe coating. Recommended recoat of interior and exterior surfaces of tank. 3 courses. Center column and 6 columns; all columns are pipe-type. Bottom level mixing pipe. Three courses (0.386, 0.265, 0.256 inches, bottom to top, 0.199 inches roof, 0.279 inches columns)
Monte Bello	Lead found in exterior tank and vault pipe coating. Recommended recoat of interior and exterior surfaces of tank. Center column and 9 columns; all columns are pipe-type. Bottom level mixing pipe. Four courses (0.505, 0.395, 0.265, 0.253 inches, bottom to top, 0.192 inches roof, 0.287 inches columns)

Table 2-3. Water Storage Tank Field Observations

Table 2-4 lists the capacities for the booster pump stations. The listed capacities assume all pumps in operation, and are the sum of individual pump capacity. Actual capacity if all pumps are in service will be somewhat smaller due to hydraulic head losses. Rated capacity should exclude the largest pump, assuming it is out of service for maintenance.

Pump Station	Capacity (gpm)	Number of pumps	Pump nameplate horsepower	Description
Mayfield	8,300	4		This pump station is used to increase flow into Zone 1 at times of maximum demand. Built 2012.
Lytton	4,650	3		Emergency lift, Zone 1 into Zone 3, if PAPL is out of service
Quarry	2,500	3	3 x 200	
Corte Madera	1,360	3	3 x 200	
Boronda	1,730			
Park	1,230			
Dahl	510	2	2 x 150	Check: 3 x 170?

Table 2-4. Water Pump Stations

Table 2-5 lists the well capacities. The capacities are based on 1974 pump tests, updated by 2010 Palo Alto report on then current capacities.

Well Station	Nominal Capacity (gpm)	Local Tank (gallons)	Tank Type (as of 1957)	Tank Description
Hale	1,425	295,000	At grade concrete	Tank exists but not in service in 2015
Rinconada	1,250	220,000	At grade concrete	Tank no longer exists
Peers Park	900	83,000	At grade concrete	Tank no longer exists
Matadero	800	63,000	At grade wood stave	Out of service in 1999. The tank no longer exists.
Fernando	800	none		Out of service in 1999. The tank no longer exists
Eleanor Pardee	1000	none		Built post 2000. No tank.
Main Library	600	none		Built post 2000. No tank.

Table 2-5. Wells

The City of Palo Alto also has pipeline interconnects with Mountain View, Stanford and East Palo Alto. The following describes the reliability of the interconnections under normal (planned outage) and post-earthquake (major event on the nearby San Andreas fault) conditions.

- Stanford interconnection. Should normally be able deliver limited flow from Stanford's water system (higher pressure) to Palo Alto zone 3 (lower pressure). Stanford interconnects can deliver limited water flows from Stanford (higher pressure) to Palo Alto zone 3 (lower pressure). May be able to provide supply from Stanford to Palo Alto's zone 3 after an earthquake.
- Mountain View. There is a 6-inch diameter pipe connection between the Mountain View and Palo Alto water systems on Silva Avenue, in Palo Alto's Zone 2, adjacent to Mountain View's Zone 2. Mountain View obtains its water supply either from the SFPUC, from the SCVWD or via local wells. Between Mountain View's Zone 2 and Palo Alto's Zone 2, the soils are generally stable, so an interconnection between Zone 2 could be somewhat reliable post-earthquake.
- In Mountain View's lowest elevation pressure Zone 1, that is southeast and adjacent to Palo Alto's Zone 1, the geologic conditions and pipelines are similar to those in Palo Alto: some moderate to very high liquefaction susceptibility, and a lot of asbestos cement pipe. There are several locations where the two systems could be temporarily interconnected between nearby fire hydrants. However, the poor soils and seismically-weak pipes means that should there be a large earthquake, Mountain View will be busy repairing pipe damage in their Zone 1, and the transfer of water between Palo Alto and Mountain View would not be reliable.
- East Palo Alto. East Palo Alto's water system does not have local storage tanks, drawing its water directly from SFPUC's BDPL 1, 2 and 5 pipelines. East Palo Alto's water system is located immediately north of Palo Alto's Zone 1, across San Francisquito Creek. From a post-earthquake point of view, it is more likely that East Palo Alto will have relatively more pipeline damage than Palo Alto, and possibly with a more limited capability to make pipe repairs. Laying hose across the creek would be a challenge. A connection located north of Highway 101 may exist, but in that area, liquefaction can be widespread, so the ability to transfer water between the systems after an earthquake, would not be immediately reliable.
- PHWD's top elevation is at the Page Mill Tank, elevation 1085 feet. It would take about a mile of flex hose (12-inch diameter) to temporarily connect this PHWD tank / pressure zone to Palo Alto's system along Page Mill Road, which would allow water from the Dahl reservoir (1,893 feet) to flow to PHWD. Due to the great change in pressure (350 psi difference) and long length of pipe required, it is doubtful that this is a practical interconnection point. In the downhill direction, water from PHWD's Page Mill tank could flow into Palo Alto's zone 6 (grade line 922 feet at Boronda reservoir), still requiring about a mile or 12-inch above ground hose / pipe to make the connection. Potential flow rates from PHWD to Palo Alto might be on the order of 500 gpm to 1,000 gpm. A test to measure flow rates and pressures would be suggested to quantify exact flow capability.

For each of these interconnections, assuming they are well maintained, they should flow potable water at modest rates (perhaps on the order of 200 to 1,000 gpm). As such, these interconnections should be considered reliable for purposes of temporary planned water outages / shutdowns for local area customers. However, these interconnections should not be envisioned as being able to supply high flow rates (2,000 gpm or higher) or be a reliable source of water supply following a major earthquake.

From a post-earthquake point of view, all four adjacent water system take most of their day-to-day supply from the SFPUC pipelines. Thus, if the SFPUC pipelines have damage, these adjacent water utilities will have a loss of water supply. Mountain View and Stanford water systems each include wells which could be used to barely supply average day demands within their systems, once power is restored to operate the wells. East Palo Alto has no wells. PHWD has no wells, but built an interconnection with Los Altos (Cal Water) to gain access to some well water from Los Altos. Overall, from a post-earthquake point-of-view, Palo Alto should not rely on water supply from any of the four adjacent water systems via these interconnections.

2.2.5 Water Distribution System Pipelines

The following information is based on GIS-based data provided by Palo Alto.

Pipeline lengths are based on the GIS, which is believed reflective of the water system as of 2010. This GIS database includes 7,315 digitized pipe segments. Summary statistics by pipeline material and diameter are provided in Tables 2-6 and 2-7.

Diameter (inches)	Cast Iron (feet)	Asbestos Cement (feet)	CCP (feet)	PVC (feet)	DIP (feet)
0.5					21
0.75					20
1					
1.25		64			09
1.5				300	
2	964	710		571	
3					2
4	3,447	37,794		955	199
6	55,193	313,788		153,265	3,002
8	27,505	214,376	4	41,280	8,319
10	10,530	29,116	60	12,775	2,526
12	9,567	72,984	626	15,092	7,499
14	557	22,572	10,870		68
16	735	18,137	27,671	4	164
18	1,304		2,459		9
20			5,969		189
24		25	2,130		5,827
27	3,717	2,770	4,000		
30			1,895		5,956
Unk	49	56		5	11
Total (feet)	113,568	712,392	55,684	224,247	33,821
Total (miles 2010)	21.51	134.92	10.69	42.47	6.41
Total (miles 1956)	45.4	71.9			

Table 2-6. Length of Pipes, as of 2010

Diameter (inches)	HDPE (feet)	CU (feet)	Steel (feet)	Unknown (feet)	Total, All Pipe (feet)
0.5					21
0.75					20
1		284			284
1.25		323		167	563
1.5		25		13	570
2		535	15	407	3,202
3					2
4				685	43,080
6	1,819			128	527,195
8	40,442		90	450	332,466
10	2,149			13	578,169
12	2,138		788	590	109,284
14	2,725				36,792
16	4,546		4	16	51,277
18				14	26,186
20					6,158
24				21	8,003
27					10,487
30				1,348	9,199
Unk				4,600	4,721
Total (feet)	53,819	1,399	897	8,452	1,226,679
Total (miles 2010)	10.19	0.26	0.17	1.60	232.33
Total (miles 1956)				0.8	118.2

Table 2-7. Length of Pipes, as of 2010

A series of maps follow that show the materials and diameters of the pipelines (based on the 2010 GIS) in Palo Alto. In all maps, the lengths are for the system as a whole (pressure zones 1 through 9). The following series of maps are provided:

- Lower Elevation Zones. By Material. Figures 2-7 to 2-13.
- Lower Elevation Zones. By Diameter. Figures 2-14 to 2-26.
- Higher Elevation Zones. Figures 2-27 to 2-30.

In all these maps, the scale is 2 km.

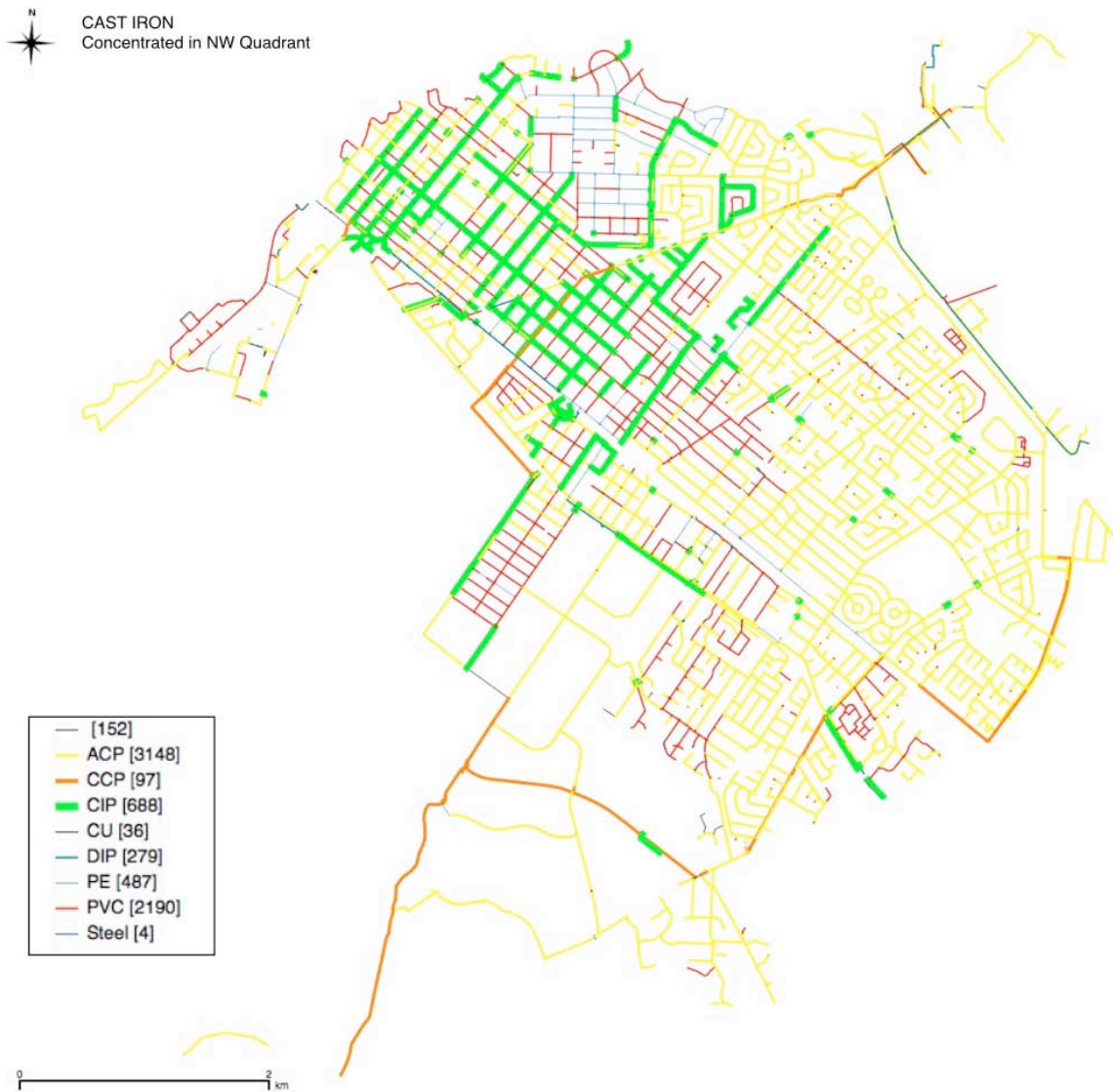


Figure 2-7. Cast Iron Pipe (Thick Green Lines, 21.51 miles)

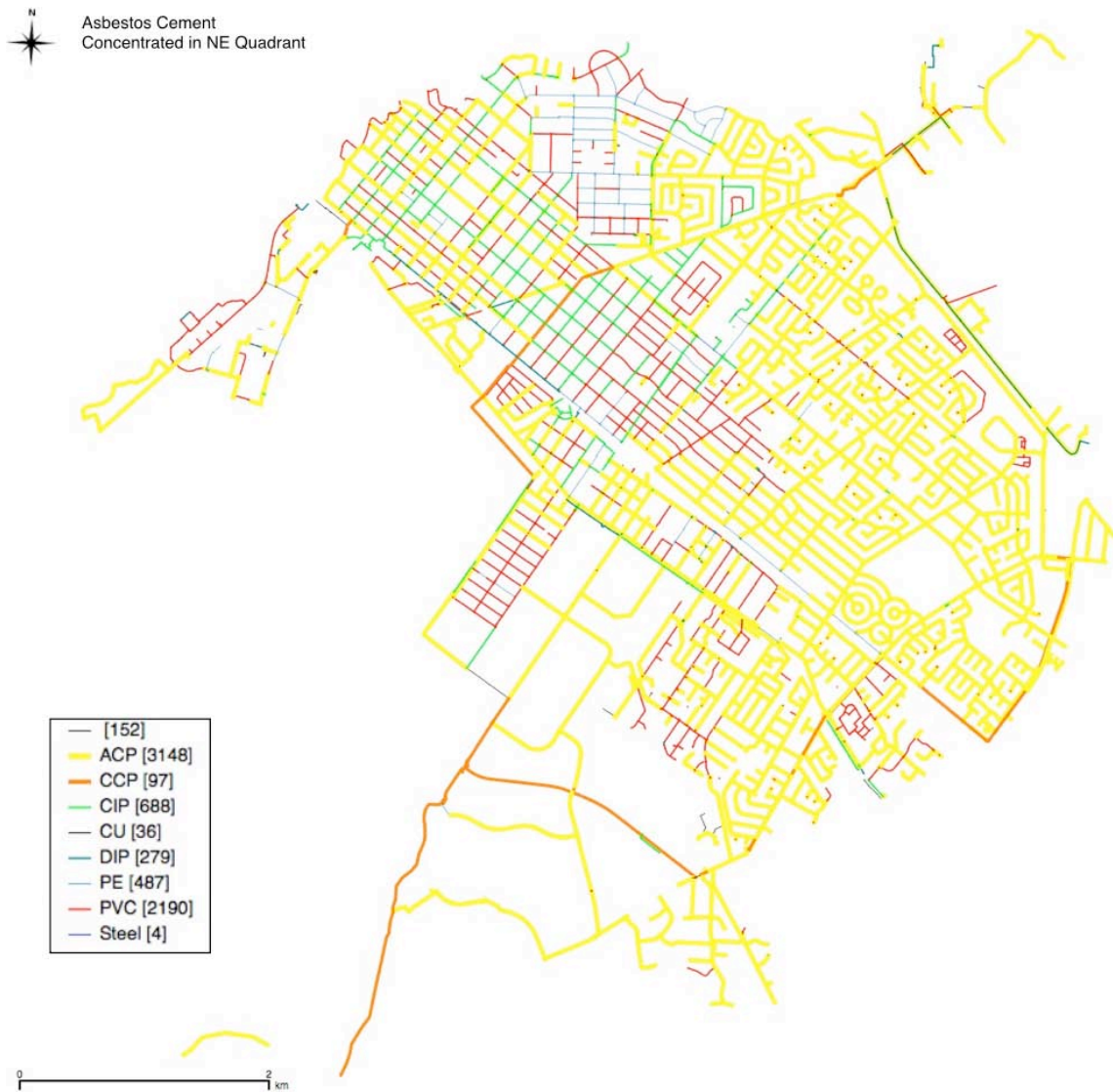


Figure 2-8. Asbestos Cement Pipe (Thick Yellow Lines, 134.92 miles)

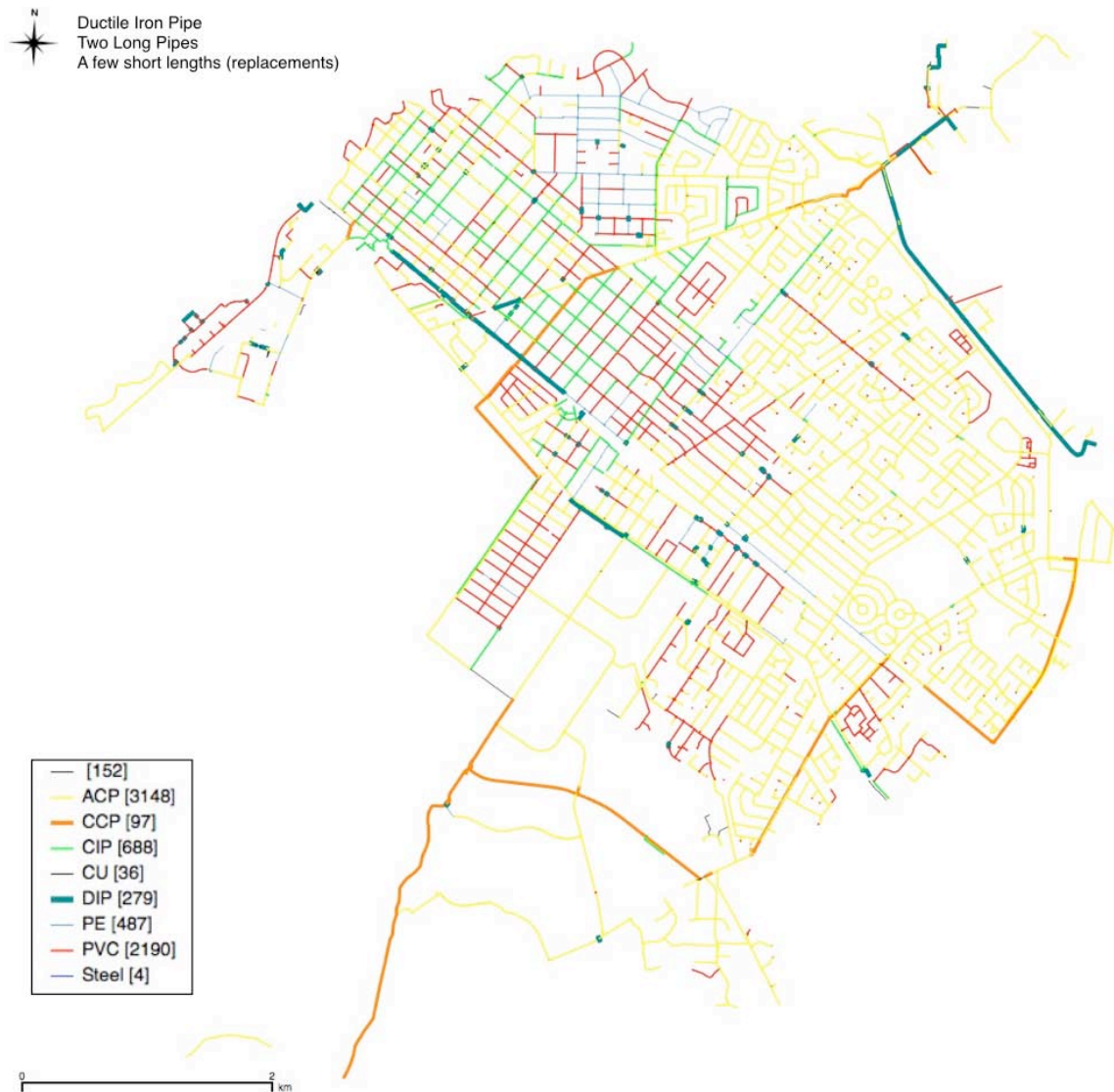


Figure 2-9. Ductile Iron Pipe (Thick Cyan Lines, 6.41 miles) and CCP Pipe (Thick Orange Lines, 14.79 miles)

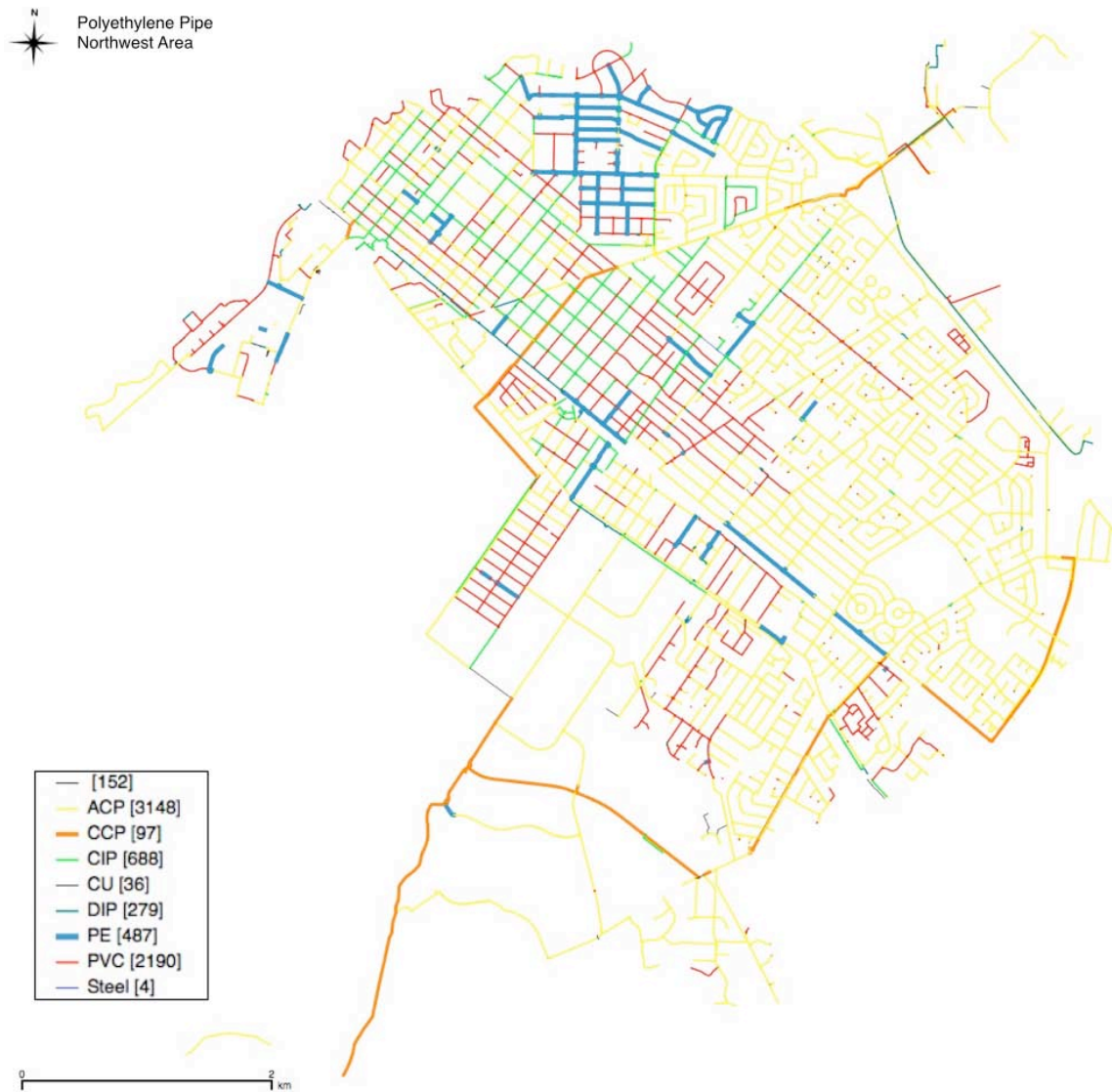


Figure 2-10. HDPE Pipe (Thick Cyan Lines, 10.19 miles)

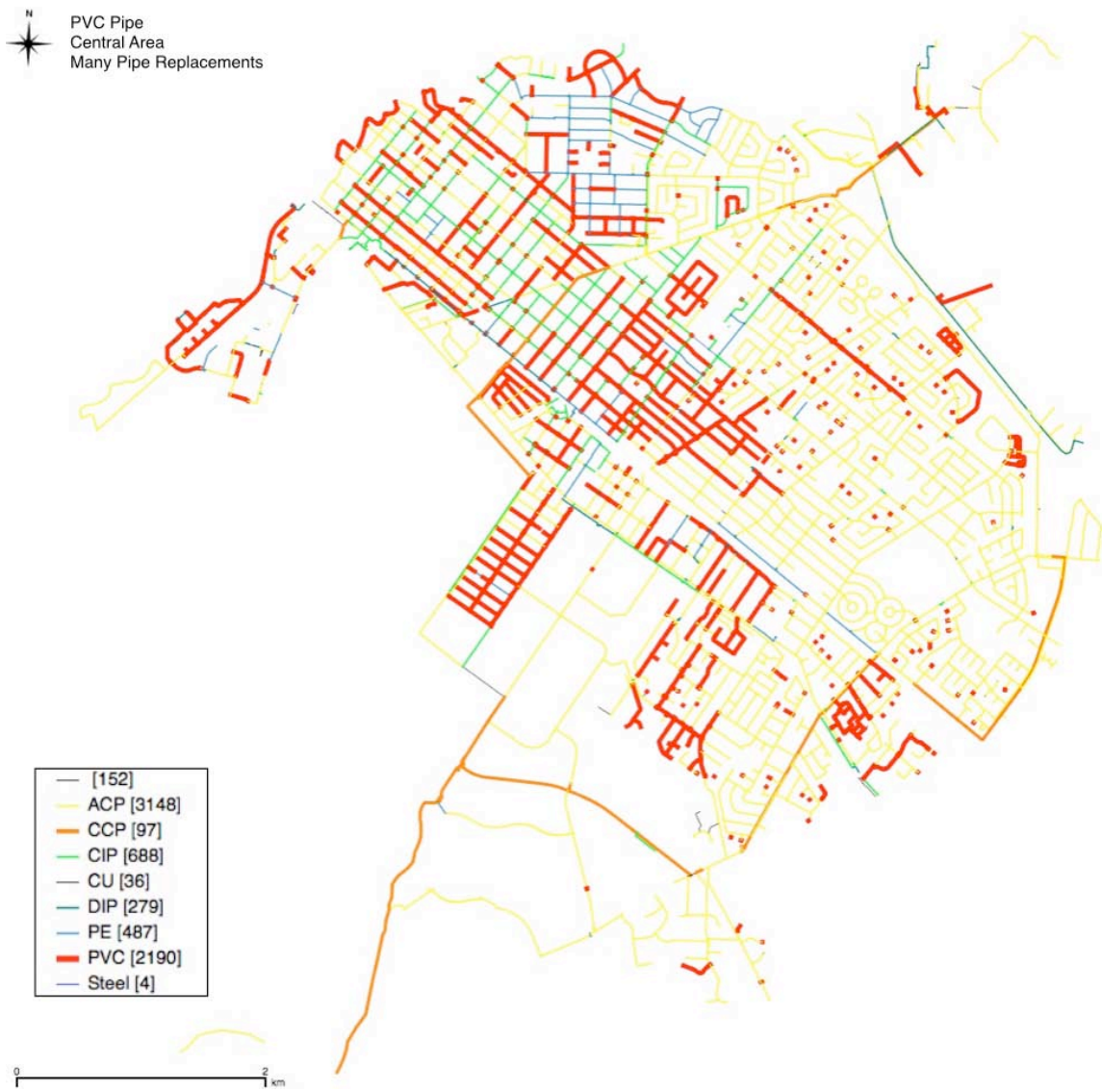


Figure 2-11. PVC Pipe (Thick Red Lines, 42.47 miles)

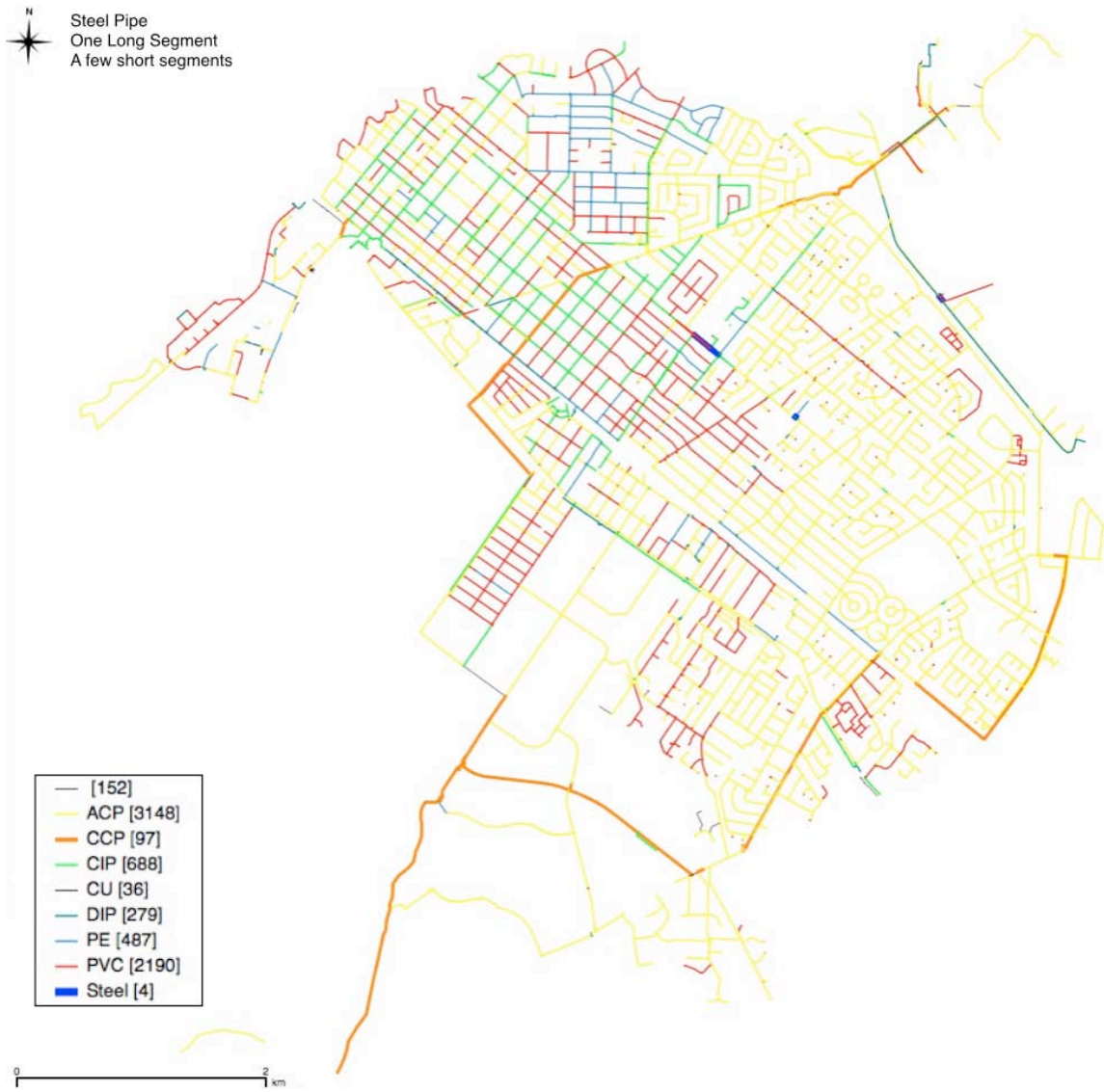


Figure 2-12. Steel Pipe (Thick Blue Lines, 0.17 miles)

Figure 2-13 shows the locations of copper pipes. These are short segments, essentially service laterals.

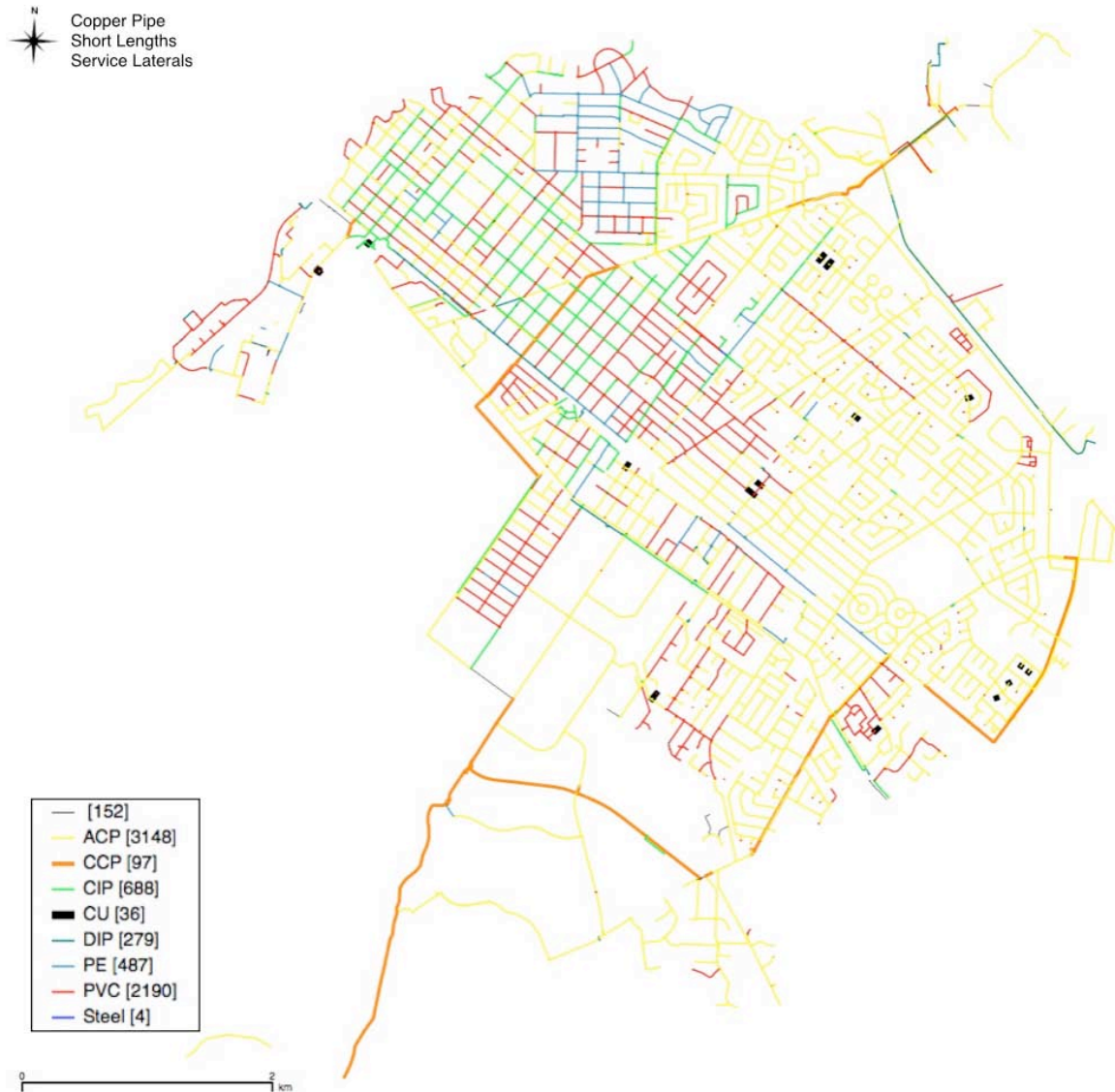


Figure 2-13. Copper Pipe (Thick Black Lines, 0.26 miles)

Figures 2-14 to 2-26 provide a series of maps to show the pipe diameters for the lower elevation parts of the Palo Alto water system. These maps are ordered to first show the largest diameter pipe, and the next map highlights the next largest diameter pipe, etc. By so ordering the maps, one can quickly visualize the hydraulic backbone of the system (18" to 30" pipes), followed by the distribution system (4" to 12" pipes). Essentially, the backbone transmission pipes take water from the SFPUC pipes (BDPL 3 and 4 in the south, Palo Alto pipeline in the center of Palo Alto). In Figures 2-14 to 2-26, full scale is 2 km.

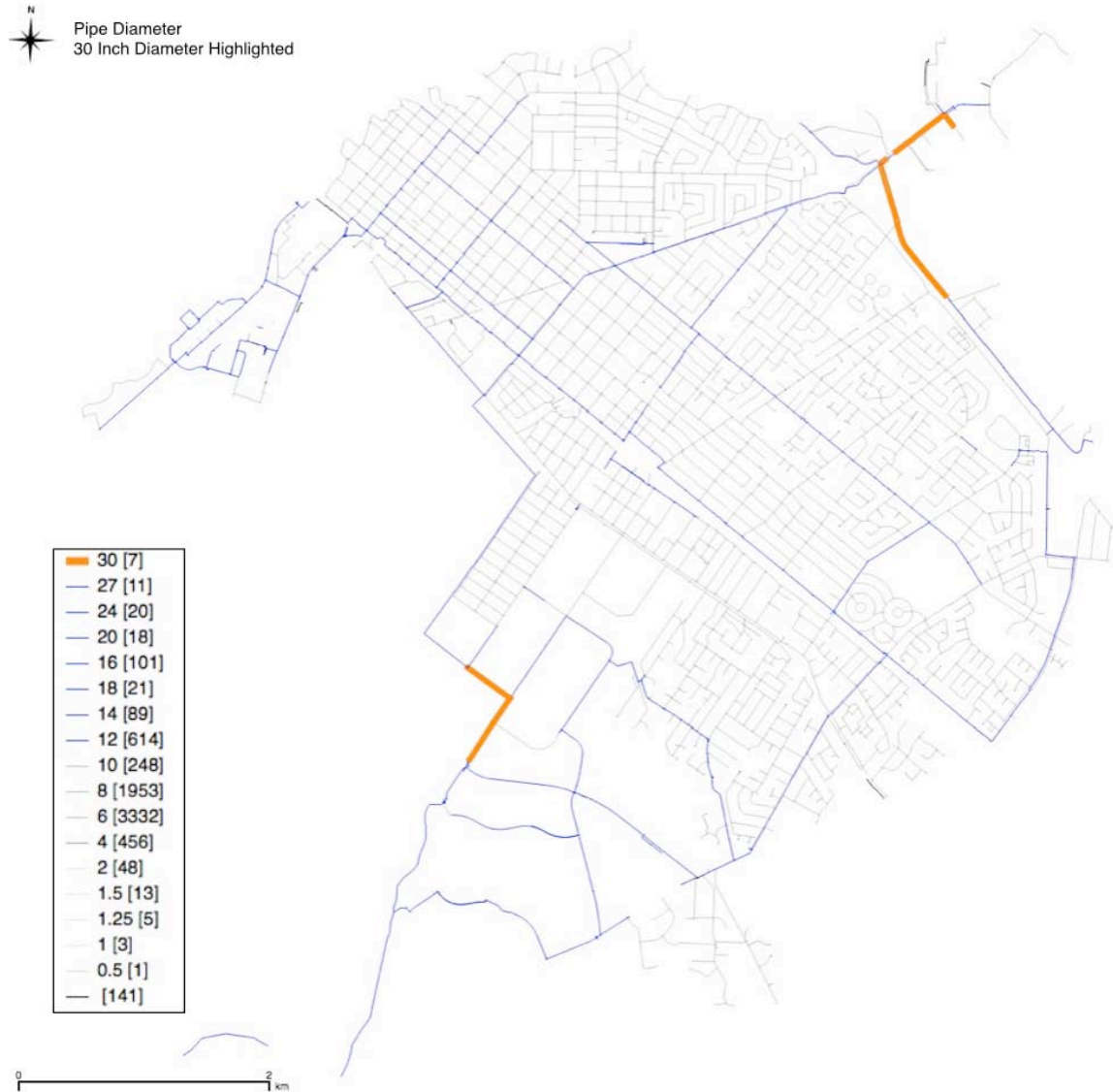


Figure 2-14. 30-inch Diameter (Thick Orange Lines, 1.74 miles)

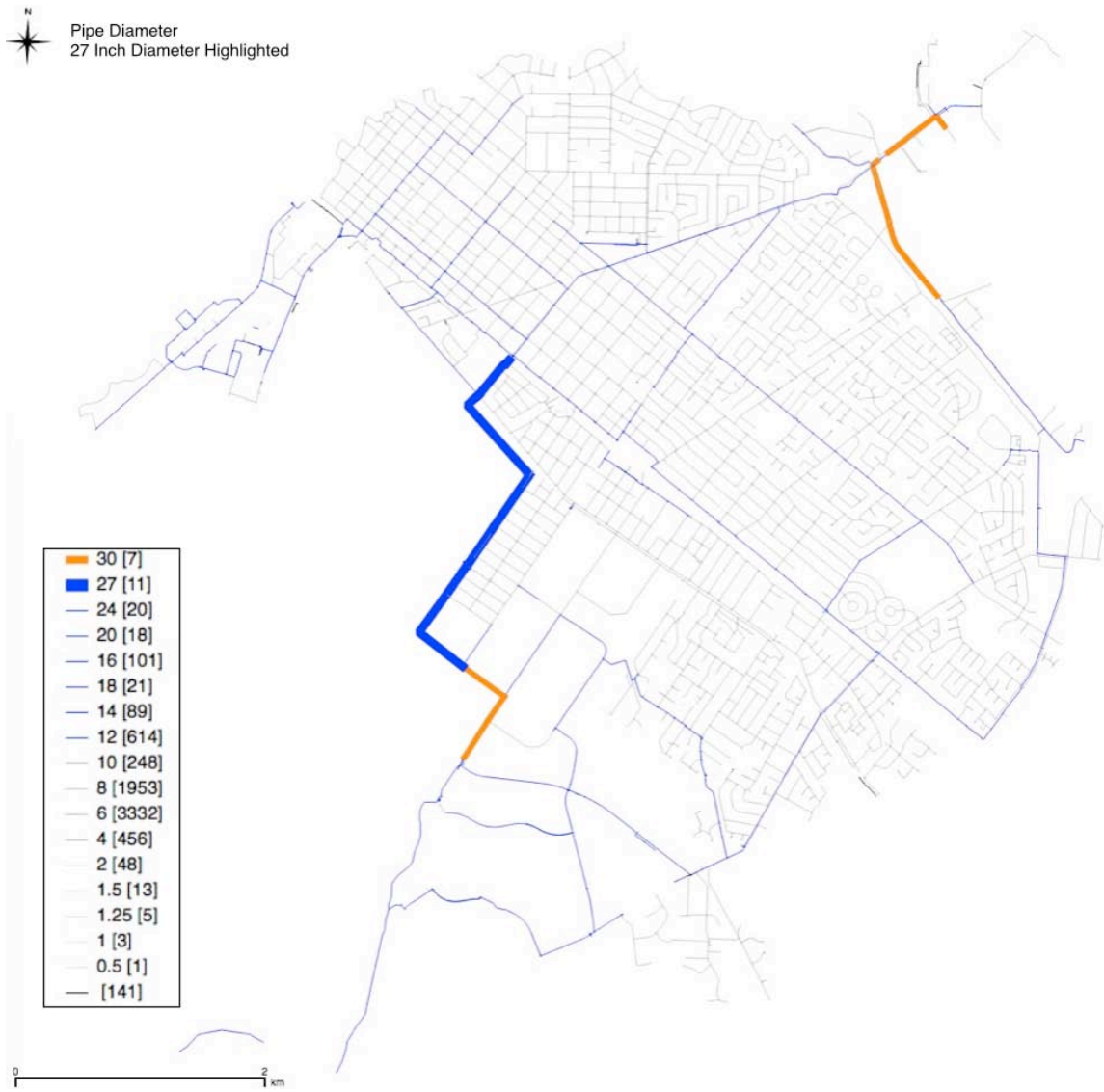


Figure 2-15. 27-inch Diameter (Thick Blue Lines, 1.99 miles)

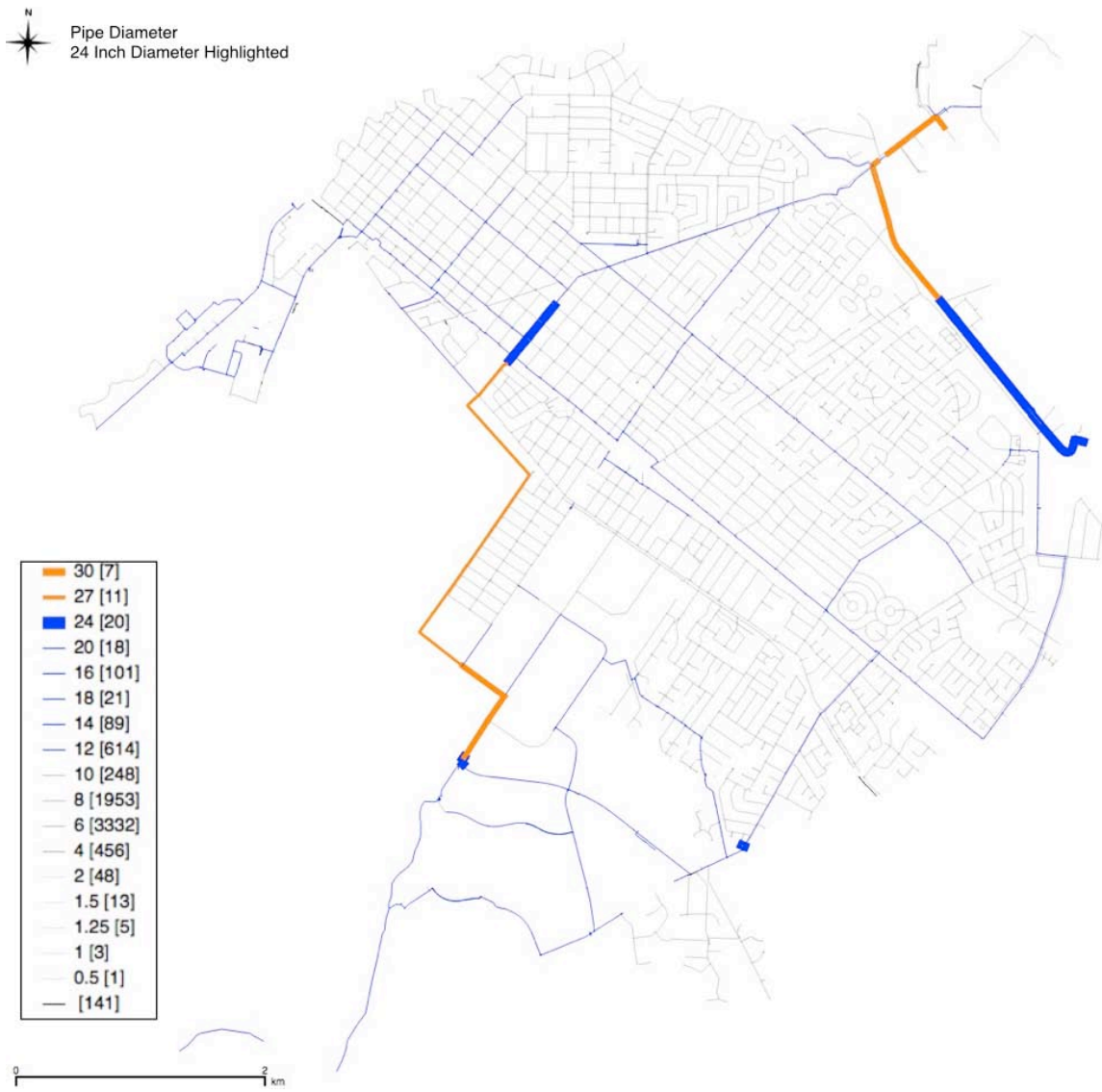


Figure 2-16. 24-inch Diameter (Thick Blue Lines, 1.52 miles)

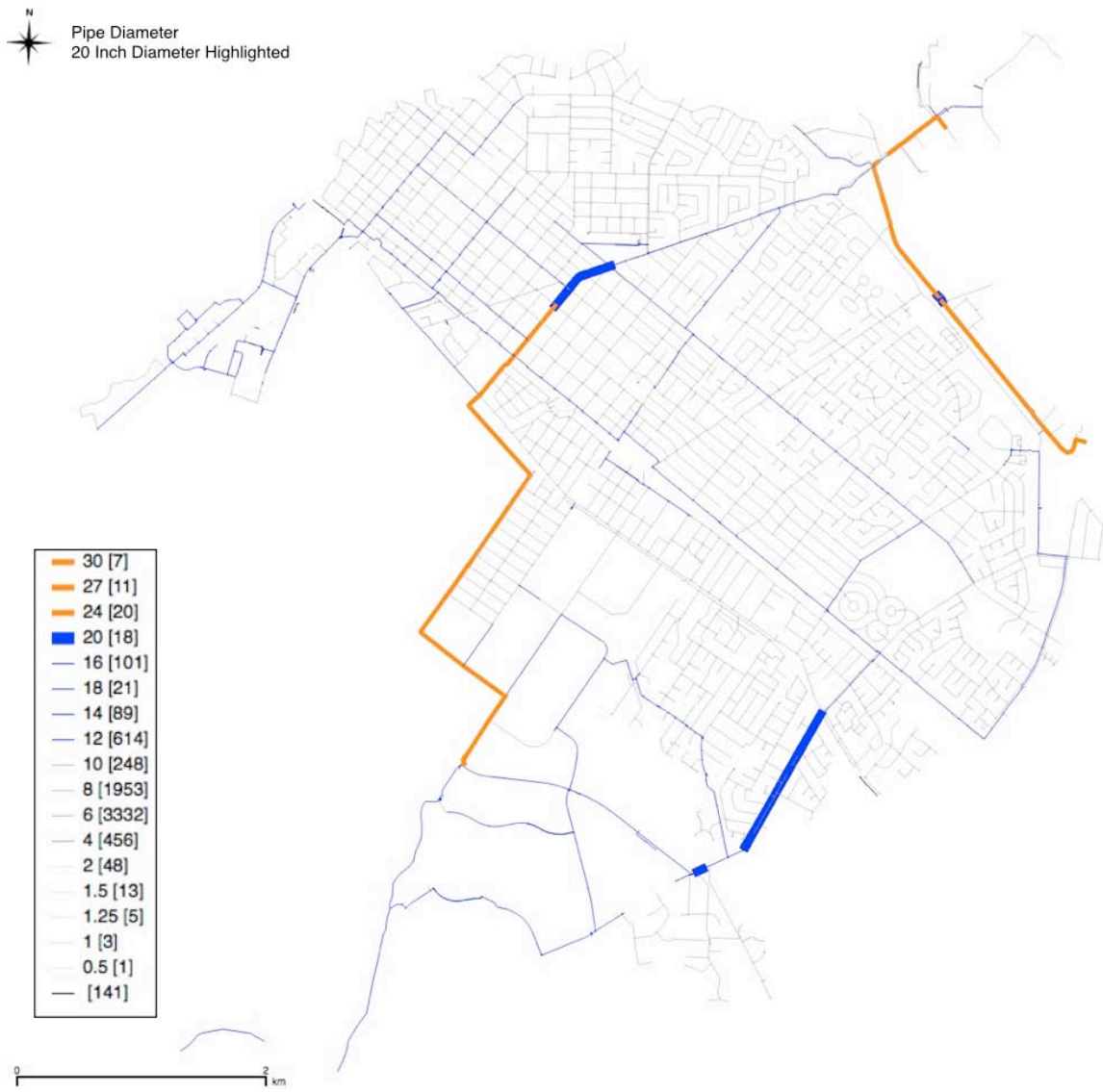


Figure 2-17. 20-inch Diameter (Thick Blue Lines, 1.17 miles)

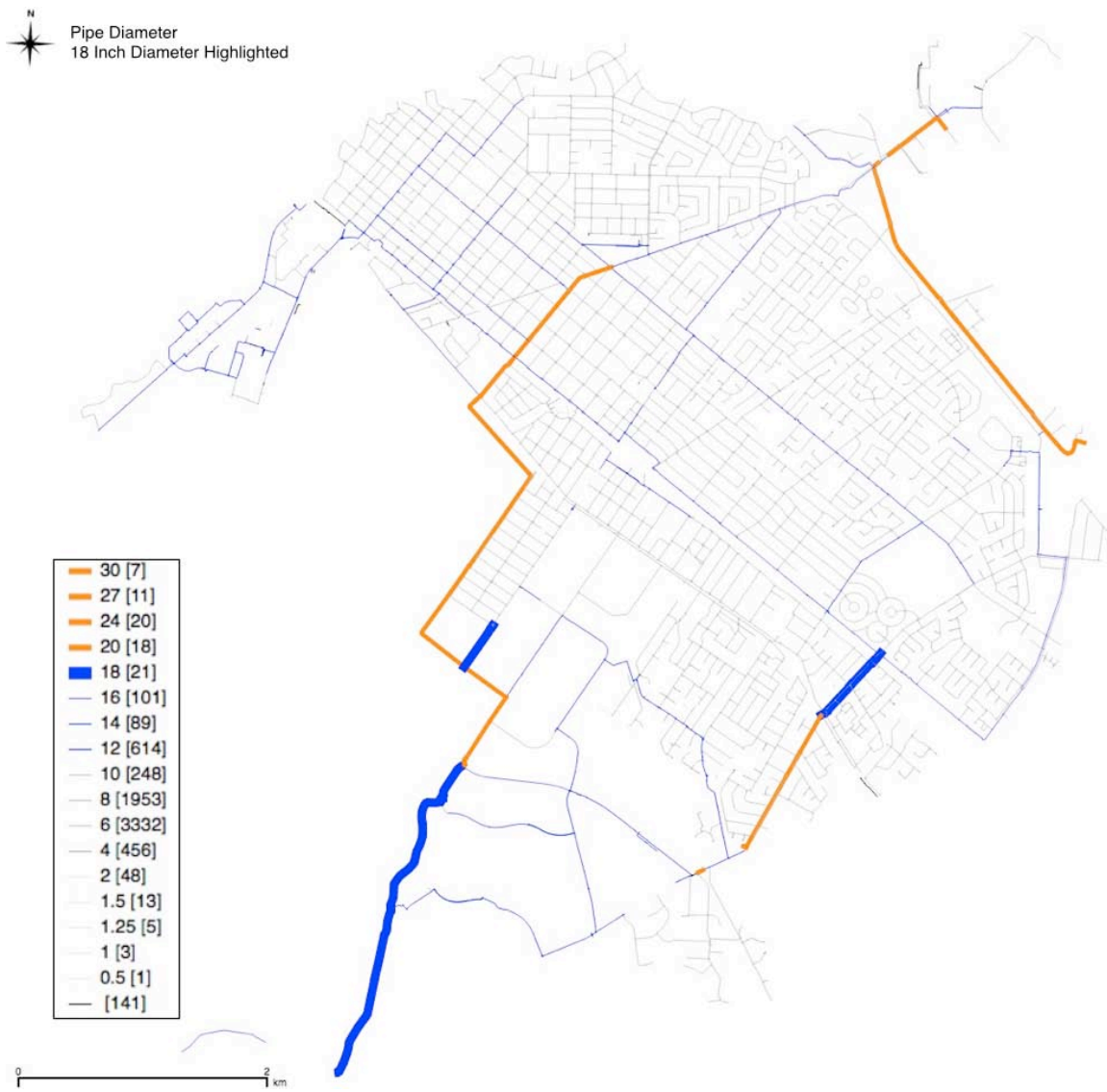


Figure 2-18. 18-inch Diameter (Thick Blue Lines, 4.96 miles)

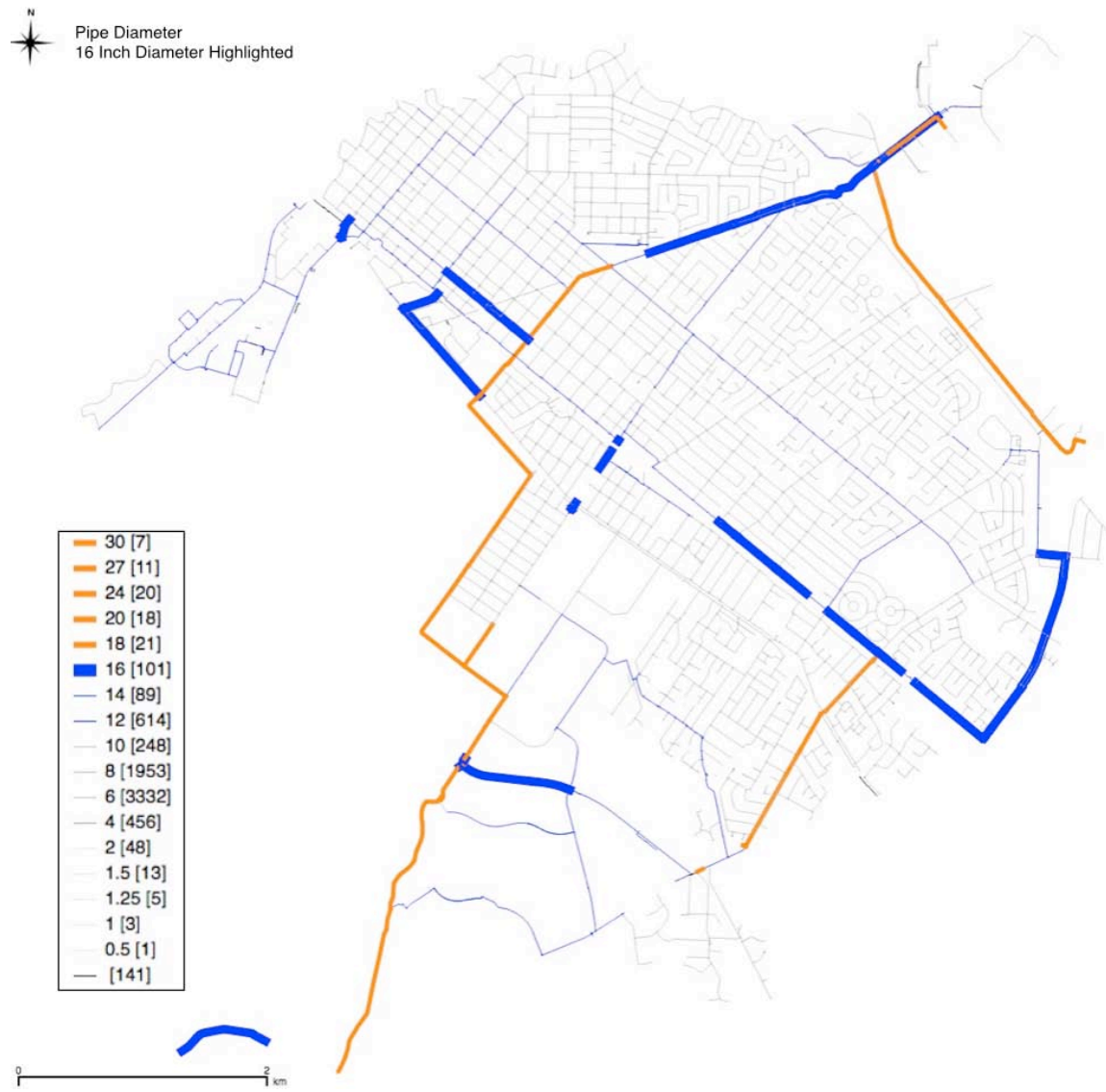


Figure 2-19. 16-inch Diameter (Thick Blue Lines, 9.71 miles)

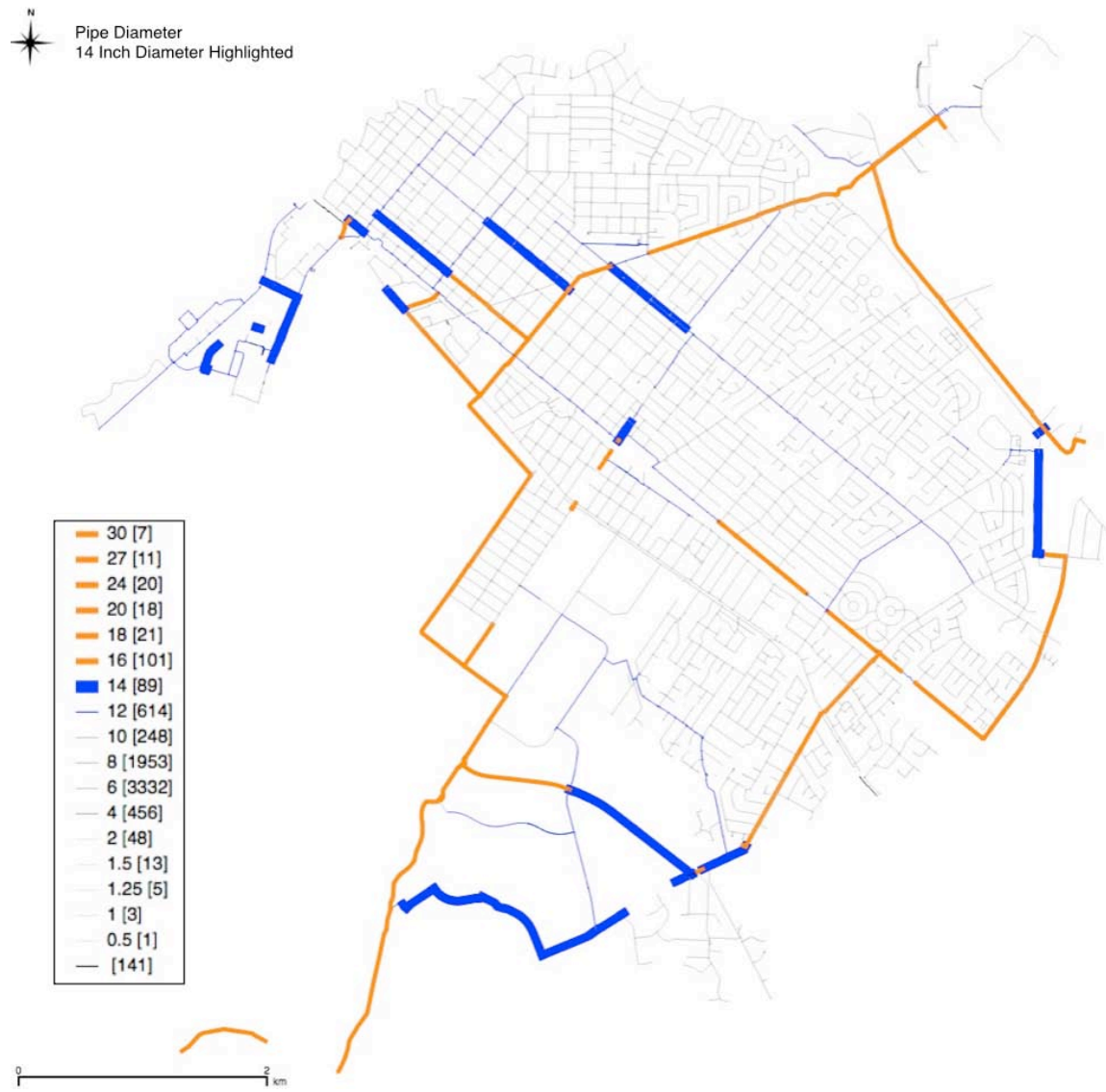


Figure 2-20. 14-inch Diameter (Thick Blue Lines, 6.97 miles)



Figure 2-21. 12-inch Diameter (Thick Blue Lines, 20.70 miles)



Figure 2-22. 10-inch Diameter (Thick Cyan Lines, 10.83 miles)



Figure 2-23. 8-inch Diameter (Thick Cyan Lines, 62.97 miles)

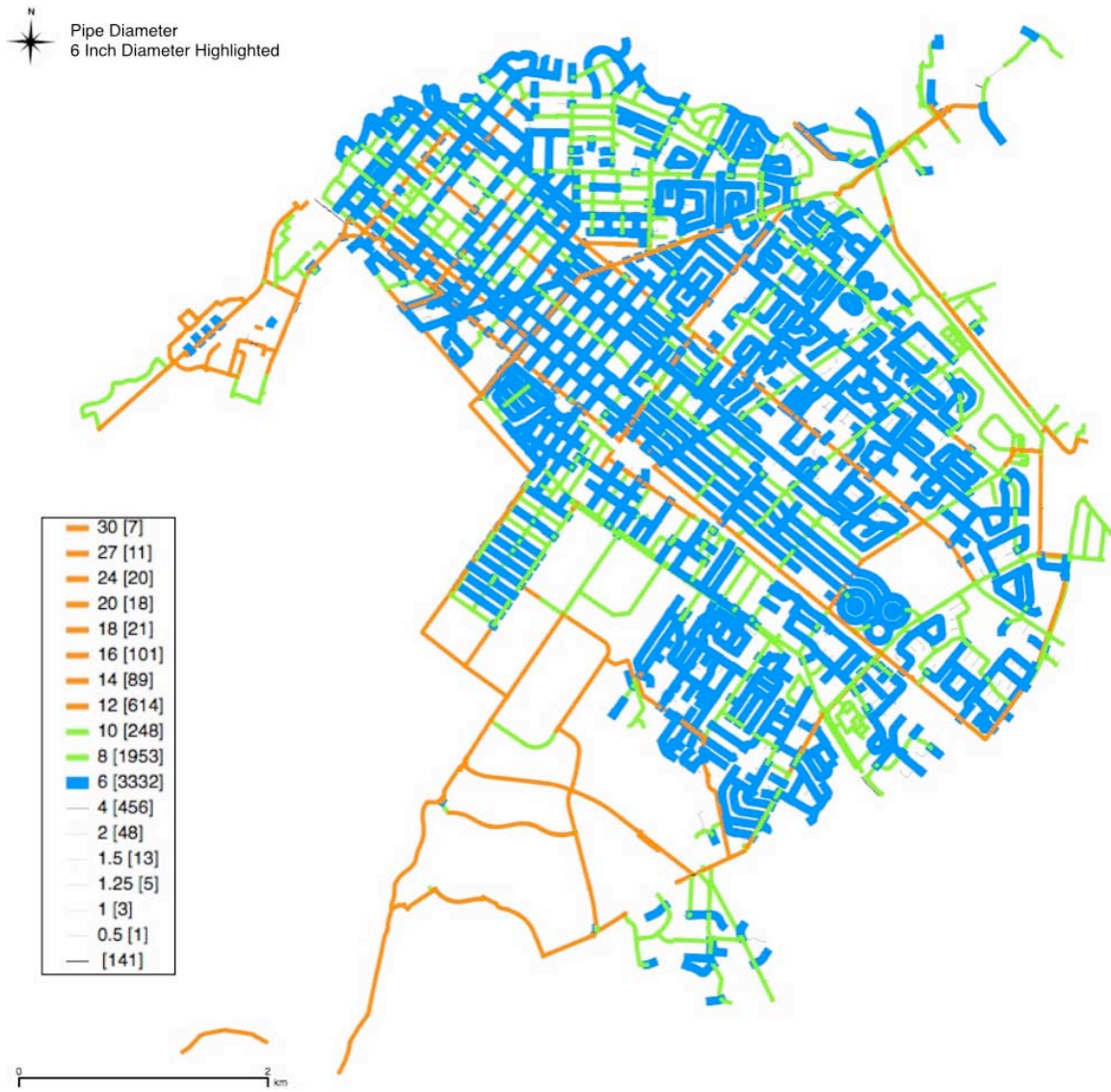


Figure 2-24. 6 inch Diameter (Thick Cyan Lines, 99.85 miles)

Figure 2-25 highlights the 4-inch pipes. It is well understood that 4-inch pipes have relatively limited capability to provide fire flows of 100 gpm or higher, especially for length of pipe much more than a couple of hundred feet. As was shown in the 1991 Oakland Hills fire, 60 to 90 year old 4" CIP pipe further limited fire flows to under 300 gpm, owing to severe tuberculation accumulation in the CIP that calibrated to a Hazen Williams "C" value of about 40. Reviewing Figure 2-25 shows that nearly all 4 inch pipes are short pipes, suggesting that the restricted fire flow rates at the end of long 4-inch pipes issue is likely minimal.

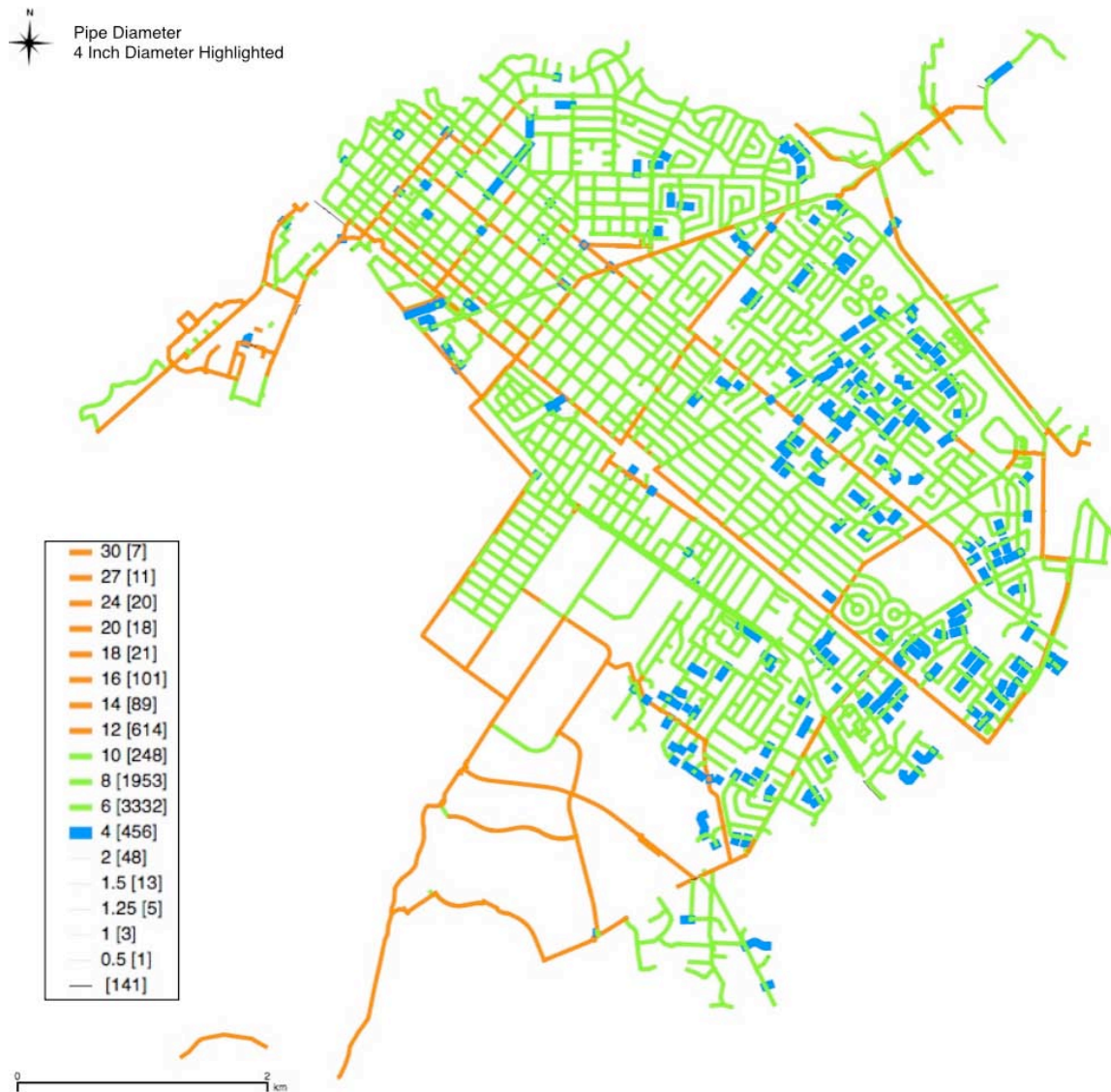


Figure 2-25. 4-inch Diameter (Thick Cyan Lines, 8.16 miles)

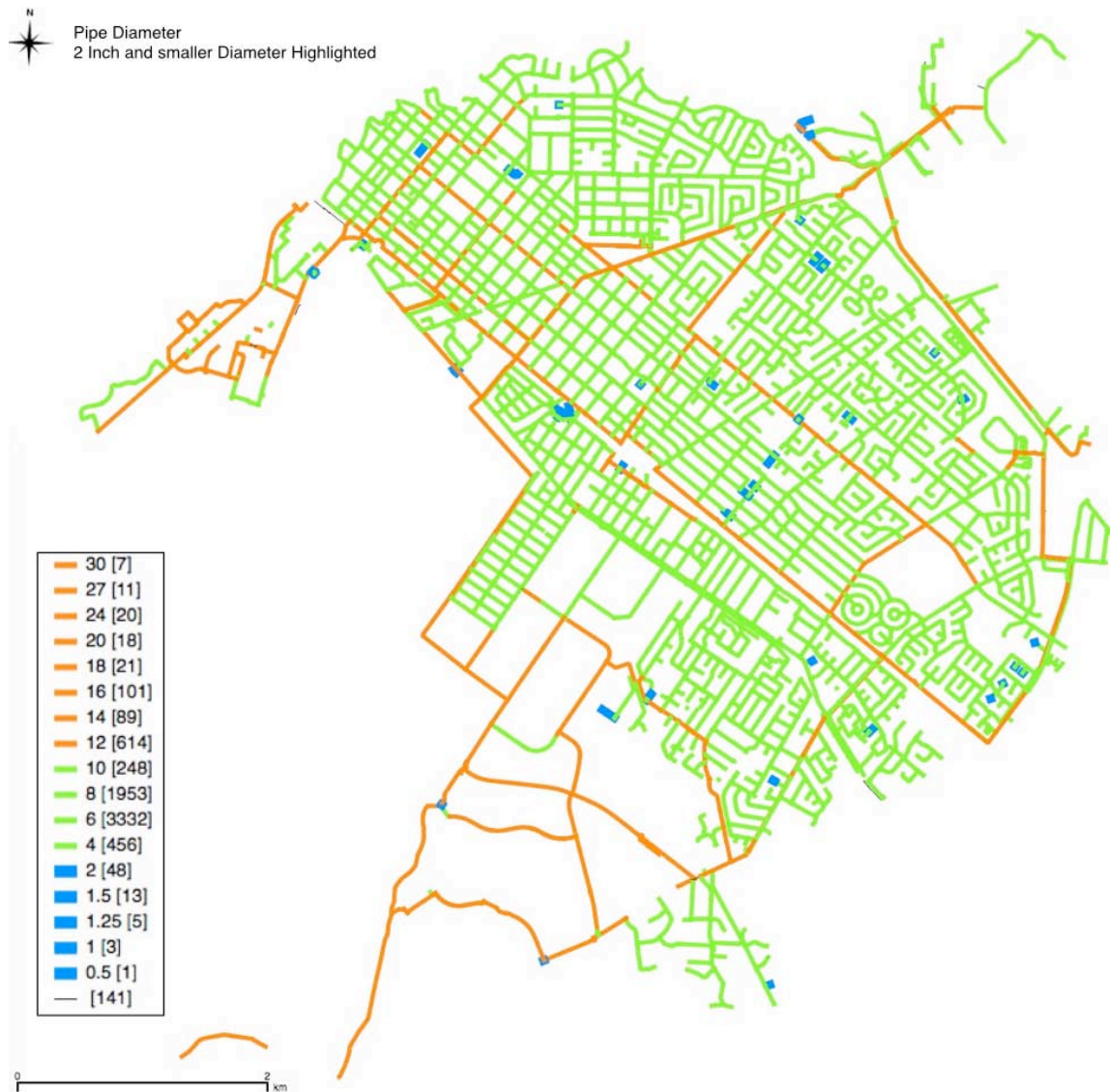


Figure 2-26. 3-inch and Smaller Diameter (Thick Cyan Lines, 0.83 miles)

The primary pipe materials are as follows:

- Asbestos Cement pipe. Most common material for pipelines from 12" to 14" in diameter. Often used in smaller (4" to 10") distribution pipe. Earliest installations around 1940. Between 1940 and 1956, the primary choice of pipeline materials for new installations, with about 72 miles installed in that time frame. Common pipe length is believe to be about 13 feet, with joinery provided by couplings with one rubber gasket on each adjacent pipe.
- Cast Iron pipe. Commonly used for small (4" – 8") distribution pipe. By 1956, 45.4 miles were installed and in service. By 2010, 21.5 miles remained in service. Cast Iron was the most common pipe type installed from 1896 through about 1940.

- PVC pipe. There is about 42.5 miles of PVC pipe in the 2010 system. Newer distribution pipe installations use PVC pipe. Used for some smaller (6" – 12") distribution pipe.
- Concrete Cylinder pipe. Used mostly for larger diameter (16" and larger) pipe.
- Ductile Iron pipe. About 6.4 miles in the system.
- HDPE pipe. About 10.2 miles in the system. All are thought to be class SDR 11 with fusion butt welds.

The pressure zones are described as follows:

- Zone 1. Gravity zone, elevation 5 to 70 feet above sea level. Normally served by BDPL 1 and 2 via the Palo Alto pipeline. SFPUC water pressure (grade line typically ~280 to ~320 feet) is reduced at the turnouts.
- Zone 2. Gravity zone, elevation 30 to 160 feet. Normally served by BDPL 3 and 4. SFPUC water pressure (grade line typically ~280 to ~320 feet) is reduced at the turnouts. Zone 2 serves many commercial businesses. Water from Mayfield reservoir can be pumped into Zone 2.
- Zone 3. Gravity zone, elevation 60 to 110 feet. Normally served by BDPL 1 / 2 via the Palo Alto pipeline. SFPUC water pressure (grade line typically ~280 to ~320 feet) is reduced at the turnouts. Should there be an outage of the PAPL, then Zone 3 can be supplied via the Lytton pump station via zone 1. Zone 3 serves the Stanford Hospital and Stanford Shopping Center.
- Zone 4. Pumped zone, elevation 90 to 233 feet. Normally served by flow from Zone 5 via the Quarry booster pumping plan, or via Zone 2.
- Zone 5. Pumped zone, elevation 140 to 515 feet. Normally served by the Quarry booster pumping plant. In-zone storage provided by Core Madera reservoir
- Zone 6. Pumped zone, elevation 410 to 900 feet. Normally served by the Corte Madera booster pumping plant via zone 5. In-zone storage provided by Boronda reservoir.
- Zone 7. Pumped zone, elevation 700 to 1,430 feet. Normally served by the Boronda booster pumping plant via zone 6. In-zone storage provided by Park reservoir.
- Zone 8. Pumped zone, elevation 1,430 to 1,870 feet. Normally served by the Park pumping booster plant via zone 7. In-zone storage provided by Dahl reservoir.

- Zone 9. Pumped zone, elevation 1,870 to 2,050 feet. Normally served by the Dahl booster pumping plant via zone 8. In-zone storage provided by Montebello reservoir.

2.2.6 Service Laterals, Hydrants and Meters

The previous Section 2.2.5 summarize the inventory of pipe mains in the water system. In this Section 2.2.6, the inventory of service laterals and hydrants is presented. This inventory is based on a GIS database of the water system, as of 2010.

There are 31,045 pipe segments in the GIS database for service laterals, hydrants and meters. Most of the pipes are small diameter, from 0.5 inches to 2 inches in diameter. There are also a few large diameter pipes, which are branches off of mains, which serve either single users or create small distribution segments. Table 2-8 provides the lengths of all the small diameter pipe (108.8 miles).

Diam (in)	ACP (ft)	CCP (ft)	CIP (ft)	CU (ft)	DIP (ft)	GI (ft)	PE (ft)	PVC (ft)	Steel (ft)	Unk (ft)	Total (ft)
0.5				360			18			12	390
0.625				70				4		27	101
0.75				10,419		646		41		1,582	12,688
1		48	41	342,653		547	121	341	86	4,578	348,415
1.25				3,761	15	207	4,959			295	9,237
1.5	72			13,380		35				488	13,975
2	113		5	18,172	22	135	6,502	275	238	651	26,113
3				199						13	212
4	3,340		682	494	359	22	296	4,345	32	1,047	10,617
6	5,438		957	226	1,541	2	2,767	6,352	469	9,613	27,365
8	2,912		467	283	302		646	1,978		4,148	10,736
10	423		47	41	60		96	396		400	1,463
12							132				132
Unk	8			1,029		54		58		111,981	113,130
Total (ft)	12,306	48	2,199	391,087	2,299	1,648	15,537	13,790	825	134,835	574,574
Total (mi)	2.33	0.01	0.42	74.07	0.44	0.31	2.94	2.61	0.16	25.54	108.82

Table 2-8. Length of Service Laterals, Hydrant and Meter Pipes, as of 2010, Including Abandoned

Some of the segments in the GIS database (1,567 entries) suggest that they may no longer be in service, because there are dates entered in the “date abandoned” field. Table 2-9 presents the same information as Table 2-8, but excluding the segments which are believed to be abandoned. This removes approximately 4.4 miles of pipe from the inventory.

If one excludes the small diameter pipes with Unknown attributes, about 85% of these pipes are copper, or which 1-inch diameter is the most common. There are also about 10

miles of 4-inch, 6-inch and 8-inch mains (mostly CI, DI, HDPE or PVC) that serve hydrants or to single large users.

Diam (in)	ACP (ft)	CCP (ft)	CIP (ft)	CU (ft)	DIP (ft)	GI (ft)	PE (ft)	PVC (ft)	Steel (ft)	Unk (ft)	Total (ft)
0.5				360			18			12	390
0.625				70				4		27	101
0.75				9,461		309				1,497	11,267
1		43	41	332,072		136	121	341	38	4,461	337,253
1.25				3475	15	38	4,928			289	8745
1.5	46			12,797		35				456	13,334
2	113		5	17,026	22	108	6,447	275	238	606	24,840
3				199						13	212
4	3,099		466	401	277	15	296	4,079	15	1,047	9,695
6	5,218		837	226	1,502	2	2,767	6003	445	9,269	26,269
8	2,820		398	283	215		646	1,810		4,138	10,310
10	423		47	41	60		96	396		400	1,463
12							132				132
Unk	8			994		9		58		106,215	107,284
Total (ft)	11,727	43	1794	377,405	2,091	652	15,451	12,966	736	128,430	551,295
Total (mi)	2.22	0.01	0.34	71.48	0.40	0.12	2.93	2.46	0.14	24.32	104.41

Table 2-9. Length of Current GIS Service Laterals, Hydrant and Meter Pipes, as of 2010, Excluding Known Abandoned

When presenting statistics about earthquake repair rates in Section 4 of this report, or maintenance / aging repairs in Section 6 of this report, the pipes in (Table 2-9) are not directly used.

- For seismic-related damage, the models presume that some of the damage will be to service laterals, and the overall total forecast seismic-caused damage includes the service lateral pipes in Table 2-9. For example, if a "lead" is forecast to be on a 8-inch diameter AC pipe, then the leak could actually be on the copper service lateral from the AC pipe to the meter.
- For in-service aging-related pipe repairs (damage without an earthquake), the leak rates described in this report are only for the leaks on the pipe mains. This report does not address long term pipe replacement of service laterals.

2.2.7 Water Facilities in the Foothill Pressure Zones

Figure 2-27 shows a map of the pipelines in the Foothills area of Palo Alto.

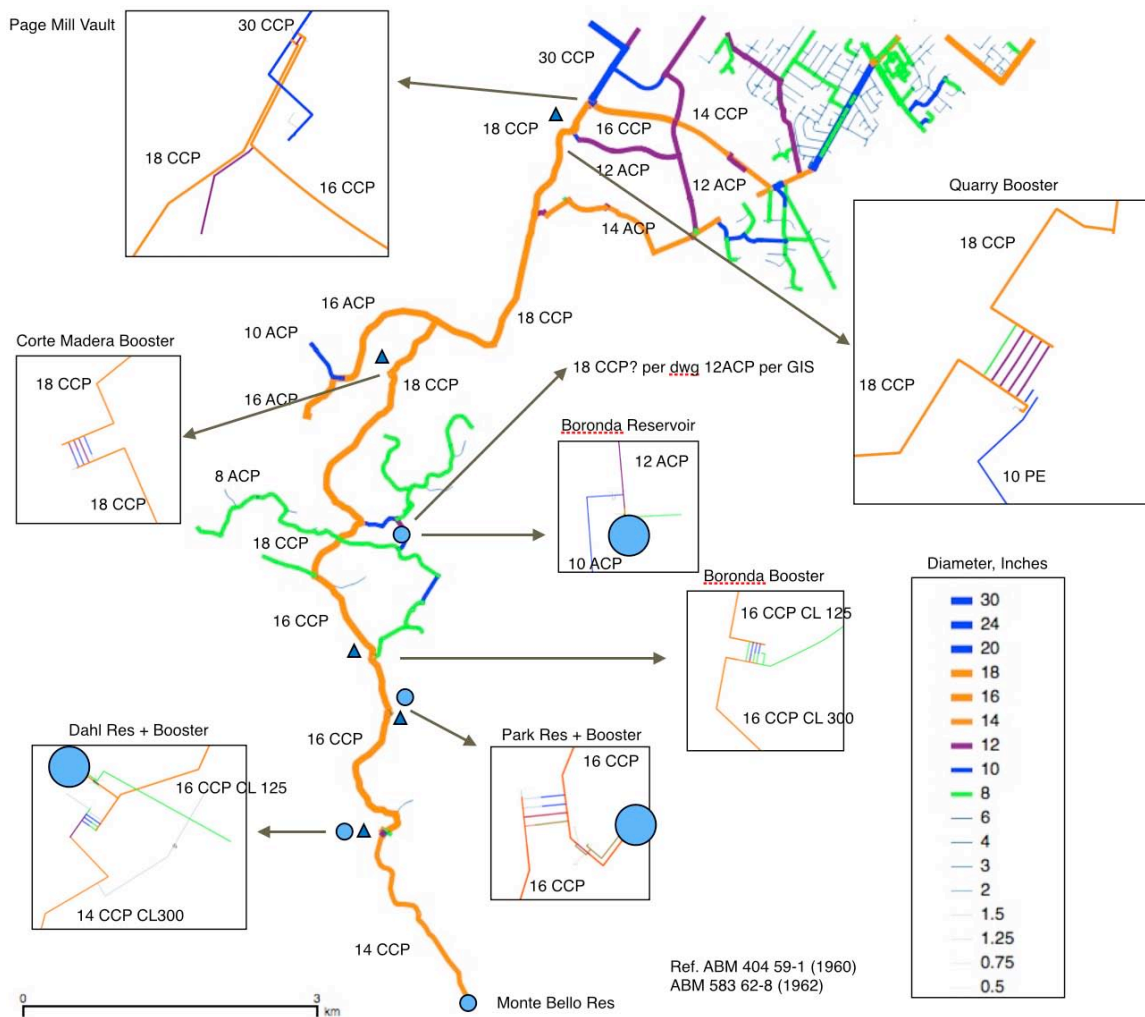


Figure 2-27. Foothills Pipeline Network

SFPUC's BDPL 3 (72" CCP) and 4 (90" PCCP) pipelines are located between the Page Mill Vault and the Quarry Booster Pump Station. The Palo Alto CCP goes under BDPL 3, and is also assumed to go under BDPL 4.

Figure 2-28 shows the same pipeline network at a larger scale, without the details at the reservoir and booster pump station locations.

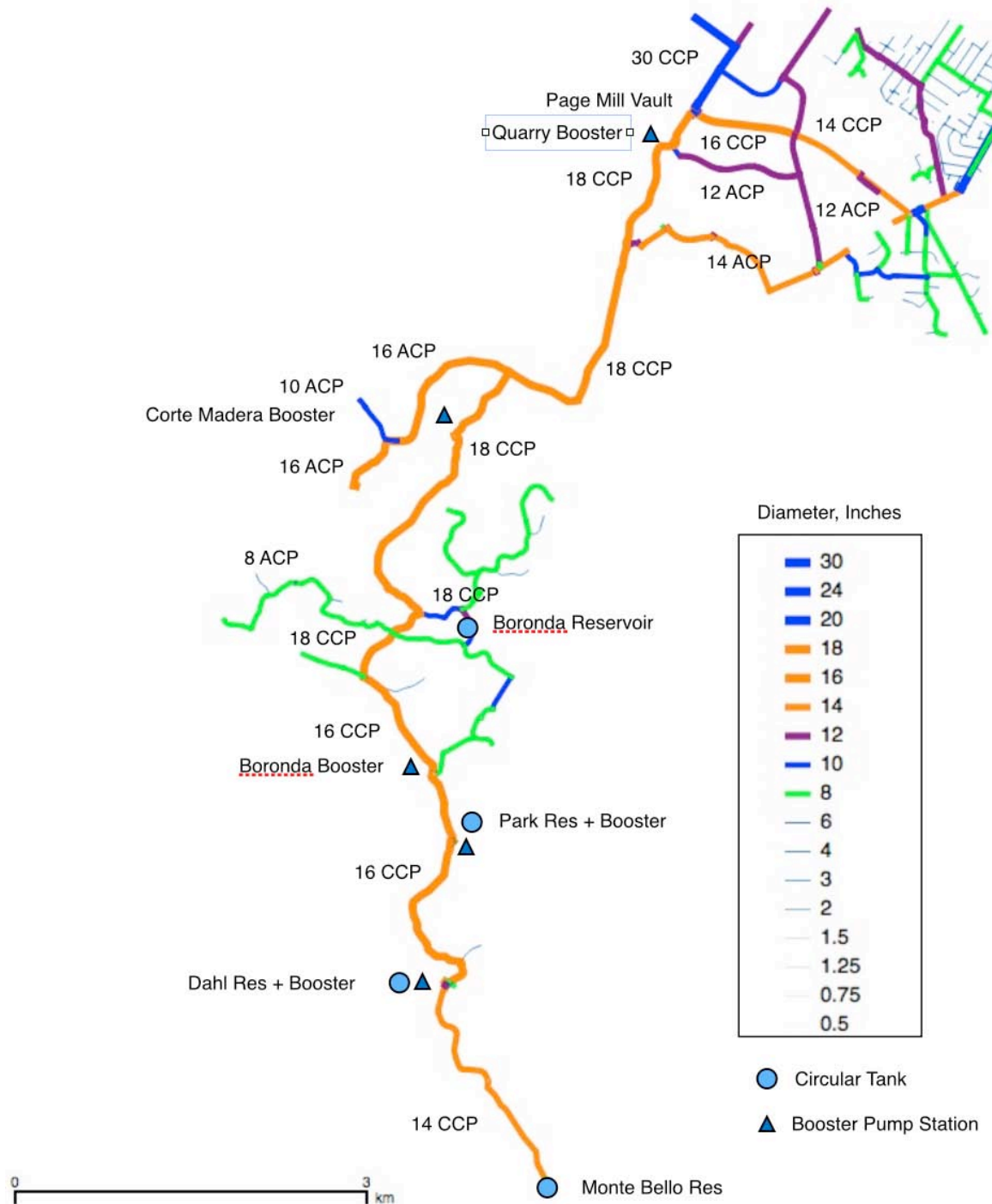


Figure 2-28. Hillside Pipeline Network – Detail

As shown in Figure 2-28, CCP pipe (orange lines) are the backbone pipes in the upper elevation zones. The backbone pipe begins at the Quarry Booster Pump Station as an 18-inch CCP. It then heads uphill to the Corté Madera Booster Pump Station, where water pressure is boosted to the Boronda Reservoir. (Note, in Figures 2-27 and 2-28, the GIS-attribute for the branch pipe to Boronda Reservoir is incorrectly shown as 12-inch ACP, and we have confirmed from the original drawings and verified that the actual pipe is 18-

inch CCP.) Water from Boronda reservoir flows downhill to Boronda Booster Pump Station, where it is lifted to the Park Reservoir. The adjacent Park Booster Pump Station lifts the water to the Dahl Reservoir. The adjacent Dahl Booster Pump Station lifts the water to the Monte Bello Reservoir.

The key elevations along this CCP pipeline are as follows (note: there are some inconsistencies with the elevations listed in Table 2-2).

Facility	Floor Elevation Feet	Comments (R = inside radius. H = height to overflow)
Boronda Reservoir	898.5	Prestressed Concrete Tank. R = 48 feet. H = 22.75 feet
Park Reservoir	1330	Steel Tank. R = 40 feet
Dahl Reservoir	1863.33	Steel Tank. R = 45 feet
Monte Bello Reservoir	2406	Steel Tank. R = 50 feet
Quarry Booster PS	137.6	
Corte Madera Booster PS	361.65	Lift = 224 feet. Static Pressure ~100 psi
Boronda Booster PS	767	Lift = ~243 feet. Static Pressure ~245 psi
Park Booster PS	1330	Lift = ~565 feet. Static Pressure ~245 psi
Dahl Booster PS	1865.7	Lift = ~570 feet. Static Pressure ~250 psi

Table 2-10. Elevations – Feet (elevations refer to mean sea level datum, 1960)

Examining the elevations in Table 2-10, it is apparent that the original design concept for this portion of the water system was to get the water to the top elevation (Monte Bello reservoir, overflow about 2432 feet) in three lifts, with each of these three lifts using "double lift" concepts. By "single lift", it is meant that nearly all pumped pressure zones used in other water systems in the San Francisco Bay Area have about a 200 to 250 foot elevation lift between pump station and tank. This strategy results in relatively "standard" strength pipes, with maximum in-zone water pressures of about 125 psi at the bottom of the zone. However, the topography and low density of construction in the Foothills in Palo Alto apparently led the original designers, circa 1960, to adopt a different strategy, as a "single lift" concept would have involved double the reservoirs and double the pump stations. Therefore, a "double lift" strategy was employed, whereby only three pump stations (Corte Madera, Dahl, Park) were used to pump water uphill nearly 1,650 feet, with the typical lift about 560 to 570 feet. This requires the pipeline between these pump stations and reservoirs to be able to sustain internal static pressures, at the lower end of each zone, of about 250 psi.

Normal water pressure within a one-to-three story structure is commonly about 60 to 80 psi at the meter. "Low" pressure would be about 40 psi at the meter. By "meter", it is meant the meter between the Palo Alto-owned water system and the customer-owned water pipes, and the meter is usually at or within 2 feet of surface elevation. Thus, for normal pressures, customers that have structures at topographic elevations more than

about 200 feet below the overflow of the tank will employ a pressure regulator (customer side of meter) to reduce the higher pressures in the Palo Alto-owned pipeline.

2.3 Foothills Transmission CCP Pipeline

The Foothills transmission pipeline is CCP pipe, ranging from 18-inch diameter at the lowest elevation at Quarry Booster pump station, to 14-inch diameter at the highest elevation to Monte Bello reservoir. In the following sub-sections, we describe this pipeline. Section 5 of this report provides a detailed seismic evaluation of the pipeline.

2.3.1 Phase 1. Quarry to Boronda Reservoir Section

Phase 1. Quarry to Boronda Reservoir (solid line in Figure 2-29). The design drawings and fabrication drawings are included in electronic file ABM #404 Foothills 59-1.pdf (52 sheets). This included the Quarry Booster Pump Station, the Corte Madera Booster Pump Station and Boronda Reservoir. The fabricator was Smith Scott Co Inc of Riverside California. The designer was Brown and Caldwell.

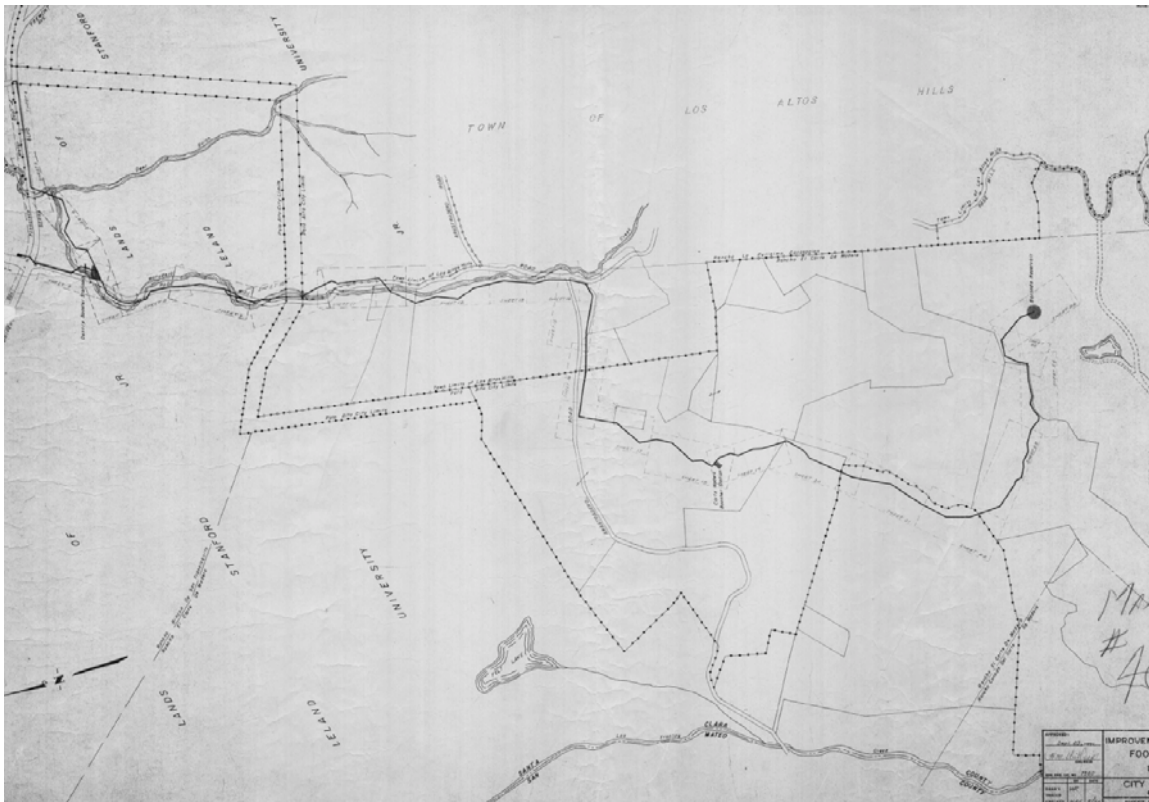


Figure 2-29. Quarry – Boronda Pipeline, 1960

This section of pipe begins at Page Mill vault, heads south to the Quarry Booster Pump Station, and then heads south, criss-crossing Matadero Creek along Page Mill Road. At Arastradero Road, the pipe heads west, then south again cross country (portions parallel to Ward Creek), terminating at Boronda Reservoir. The pipe was built using cut-and-

cover methods, with the top of pipe being about 4 feet under the finished ground surface. At creek crossings, the pipe goes under the creek, with the pipe encased in 6-inches of unreinforced concrete, generally between the creek edges, primarily for scour control (see Figure 2-30; and see Section 3.5 that discusses the weaknesses in scour protection observed at one of the creek crossing, as of 2015). The sides of the creek were reinforced with riprap for a few feet on either side of the pipe. The pipe is predominately 18" (nominal diameter) CCP, with 16" CCP used for the first ~200 feet from the Page Mill vault. Below elevation ~200 feet, the pipe is class 150. Above elevation ~250 feet, the pipe is Class 125 to the Corte Madera Booster Pump Station. South of the Corte Madera Pump Station, the pipe is Class 250 and runs parallel to a creek for a few hundred feet, crossing it twice. At elevation ~410 feet, the pipe changes to Class 225. At elevation ~460 feet, the pipe changes to Class 200. At elevation ~530 feet, the pipe changes to Class 175. At elevation ~590 feet, the pipe changes to Class 150. At elevation ~645 feet, the pipe changes to Class 125.

Along this section of pipe there are four blow offs, either 8" or 12" steel schedule 20 pipe, teed to the 18" CCP, with a corresponding gate valve. The coating for the branch pipe is asphalt dipped.

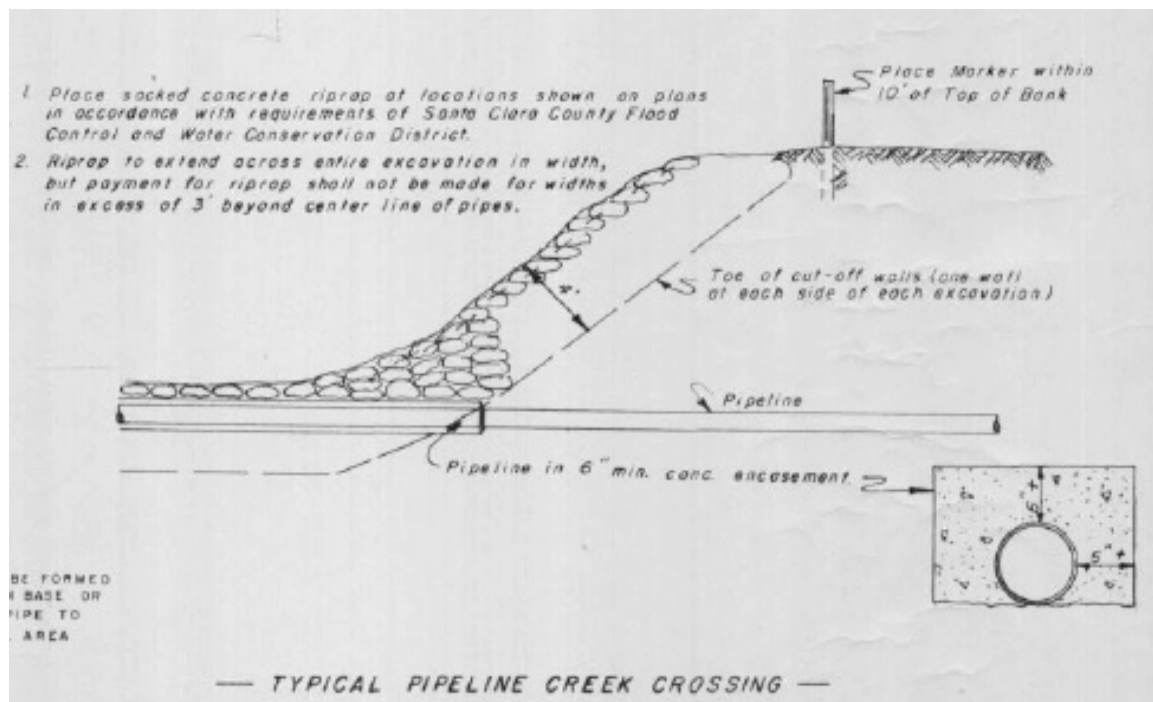


Figure 2-30. Creek Crossing – Detail

Figure 2-31 shows the standard as-designed pipe-to-pipe connection. This shows a Carnegie-type joint, highlighting the case where a welded joint is called for on the drawings (about 1% of all joints are welded). The intended joint at all other locations relies on the rubber gasket to form the leak-tight connection.

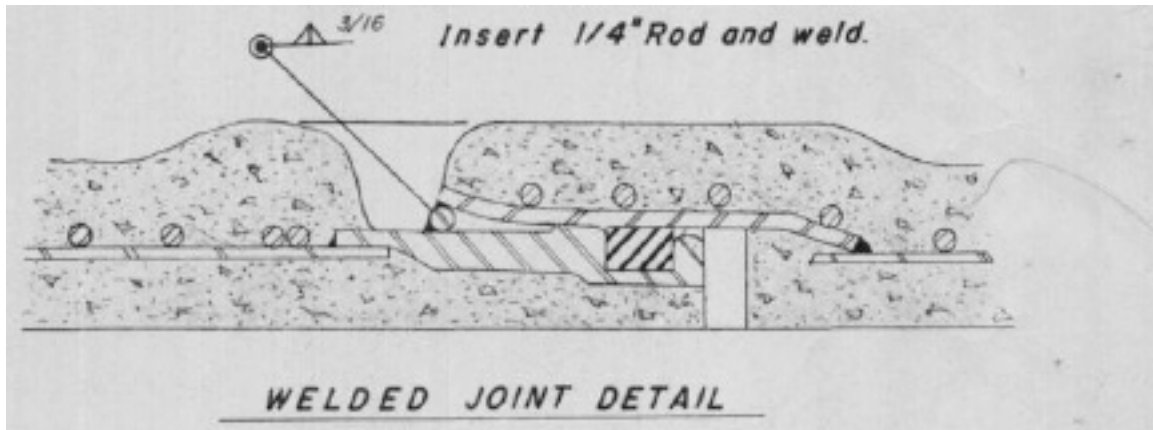


Figure 2-31. As Designed Pipe Connection – Detail

Figure 2-32 shows the pipe connection as shown on the fabrication drawings. Rather than using the welded-on Carnegie joint, the fabricator apparently elected to roll the steel cylinder into the shape needed to accept the rubber gasket, and where needed, the welded joint. Also note the difference in reinforcement bar placement between the as-designed pipe (Figure 2-31) and the fabricator's pipe, whereby the reinforcing bars are not placed directly onto the steel cylinder, but appear to be placed over a layer of concrete that has been formed onto the outside of the cylinder. In both cases, the apparent thickness of the inside lining is 0.5 inches of cement. The total thickness of the pipe wall is not specified in the available drawings.

The typical straight pipe segment was 36 feet long. The common stab length per joint was 4.25 inches, meaning that the distance from bell to bell, on two adjacent pipes, would be 36 feet, less 4.25 inches. Along the pipe, a few segments are joined using welded butt straps. Angles over 5 degrees appear to all have been manufactured as mitered butt welded steel pipe. There are about 431 pipe segments between Page Mill vault and Corte Madera Booster Pump Station.

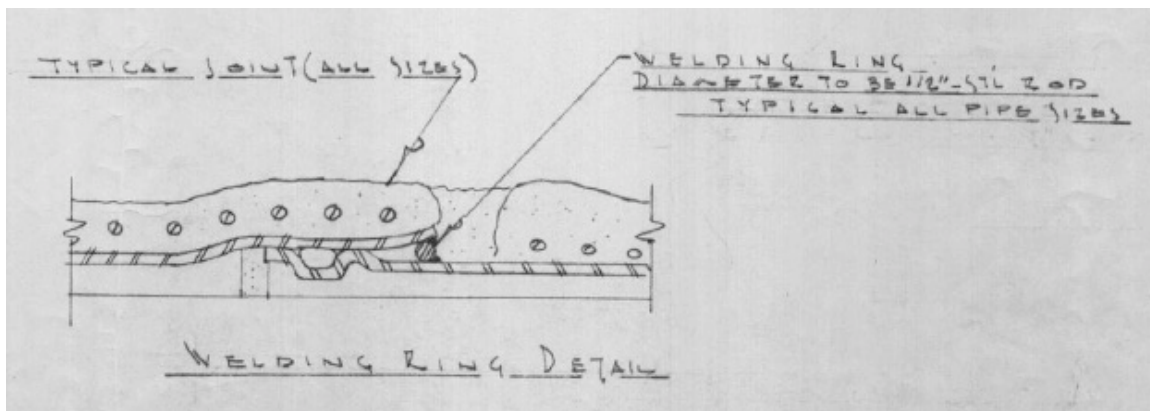


Figure 2-32. Fabricator Pipe Connection – Detail

The modern version of Concrete Cylinder Pipe is covered by AWWA C-303. This type of pipe is also sometimes called Bar-Wrapped Pipe. The 1960 design drawings are silent

as to specifications, so that the diameter, grade and post-tension, if any, of the exterior reinforcing bars is uncertain. Bar Wrapped Pipe was developed in the 1940s, and was commonly used in the 1950s and 1960s. The AWWA specification C-303 was initially developed in 1970. Today (2015), we do not know the true thickness of the exterior concrete coating, the grade, pitch and diameter of the exterior reinforcing, and whether the exterior bars are post-tensioned. However, the fabricators drawings do list the thickness of the steel cylinder of each class (125, 150, 175, 200, 225, 250) of pipe.

Most likely, the grades of steel are mild steel for both the steel cylinder and the reinforcing bars. Mild steel used for the cylinder would most likely have a minimum Fy of 30 ksi (might have been as high as 40 ksi). Mild steel for the reinforcing bars would most likely have Fy of 40 ksi.

A cement mortar coating was placed over the outside of the steel cylinder. The cement mortar places the steel cylinder in an alkaline environment that prohibits corrosion. The drawings are silent as to whether the coating uses only cement, or cement and aggregate (concrete), or the strength of the coating.

We performed a series of stress checks for hydrostatic pressures on the Foothills CCP pipelines. Table 2-11 summarizes the computed hoop-direction stresses in the Foothills CCP pipes. The information in this table is based on dimensions provided in the fabricator's drawings for 18" CCP (inside diameter 19"), as well as 16" CCP and 12" CCP.

Pressure class, pounds	Pipe Inside Diameter, inches	Nominal pressure, psi	Inside R to inside face of steel, inches	Gage	Cylinder Thickness, inches	Cylinder hoop stress, psi
250	19	250	9.5	7	0.1793	13,246
225	19	225	9.5	8	0.1644	13,002
200	19	200	9.5	9	0.1495	12,709
175	19	175	9.5	10	0.1345	12,361
150	19	150	9.5	11	0.1196	11,915
125	19	125	9.5	12	0.1046	11,353
125	17	125	8.5	12	0.1046	10,158
125	13	125	6.5	12	0.1046	7,768

Table 2-11. Pipe Hoop Stress

From Table 2-11, the key points are as follows:

- The inside steel cylinder was originally sized to keep hoop stresses due to hydrostatic pressure under about 13,500 psi. If the steel material was $F_y = 30,000$ psi, this provide somewhat more than a Factor of Safety of 2 for the 18" CCP, and about 3 to 4 for the smaller diameter CCP.
- We do not have original design calculations, and there is no data that describes how the original designer treated hydrodynamic loads (such as those due to valve closures or sudden starts / stops on pumps). The lift from Quarry to Boronda reservoir uses a booster pump station (Corte Madera) along the pipe. While both the Quarry and Corte Madera pump stations are operating, the water pressures will be a little lower than the static pressures (some loss of pressure due to friction of the moving water). We have not done new water hammer calculations to assess the impact of a sudden shut down of pumping at Quarry pump station. However, a simplified procedure to account for such water hammer effects might be to keep the stress of the steel cylinder pipe under about $0.45 F_y$, which is often thought to provide sufficient margin for water hammer effects. From Table 2-9, it is apparent that this margin was provided.
- We have no design or construction data that shows the exact type of reinforcing bars (diameter, pitch, F_y , post-tensioning) used outside the steel cylinder. As an approximation, the extra hoop capacity from this exterior reinforcement might be about 50% of that of the steel cylinder.
- The inside diameter to the steel shell is 1 inch larger than the nominal pipe size. Thus, an 18" CCP has inside diameter to the steel shell of 19 inches. Assuming 0.5 inch thick cement mortar lining was applied, then the wetted surface inside diameter of a 18" CCP is 18 inches; and the 16" CCP has inside wetted diameter of 16 inches, and the 14" CCP has inside wetted diameter of 14 inches.

We performed a field walk down of portions of the Foothills CCP pipes. We observed many service laterals, as well as fire hydrants, which are presumed connected to the CCP. The original (circa 1965) design drawings show none of these service laterals or fire hydrants. We assume that these were installed using "hot tap" methods. The common "top tap" methods for small diameter service laterals might have involved the following:

- Expose the CCP pipe
- Chip away the mortar coating to expose the reinforcing wires and cylinder.
- Insert a metal saddle where the hole is to be cut. Strap the saddle to the pipe using exterior straps and rods; or anchor to the reinforcing wires with split studs.
- Cut away the wires carefully in a manner that does not damage the cylinder.

- Install remaining hardware, corporation stop, perform a pressure test, attach a tapping machine and drill through the cylinder and concrete core.
- Coat all metal parts with cement grout.

For fire hydrants, the common lateral size might be 6 or 8 inch diameter. Conceptually, the hot tap is done in a similar manner as for a service lateral, except that a larger saddle is placed over the CCP (perhaps with 8 exterior straps), with grout injected under the saddle and the outside of the CCP to ensure a tight bond.

In either case, the integrity of the CCP is potentially compromised during the process of installing service laterals and hydrants. While the leak-tightness of the finished joint is assured by a pressure test at the time of construction, the margin of safety to accept temporary higher stress (such as from earthquake) can be compromised.

A series of soil borings were part of the 1960 design effort. The borings were done at regular intervals along the length of the pipe. Based on the drawings available for review, the following soil borings were performed:

- Boring 1. Near Junipero Serra, Quarry Booster. 5 feet of dark brown sandy clay over 4 feet of clayey sand over 2 feet of bluish-grey sand with some clay fines (wet) over volcanic gravels. (Note: there is no mention of water table; or whether the sandy clay was loose or dense).
- Borings 2, 3, 19, 18, 20, 6, 7, 8, 10, 11, 12, 13, 14, 15, 16, 17, 13A, 12A, 11A, 10A, 9A, 8A, 7A, 6A, 5A. No data. Order of borings is generally from lower elevation to higher elevation along the 18" CCP.
- Borings 1A, 2A, 3A. These are located at Boronda Reservoir.
 - Boring 1A (near center of tank). Top 11 feet. Brown, fine sandy clay, stiff greenish – brown sandy shale, hard weathered, fractured. Very hard, 1 to 2 feet. Grading harder below 11 feet.
 - Boring 2A. (near east side of tank). Top 15 feet. Dark brown fine sandy clay (CL), stiff greenish-brown sandy shale, hard, weathered, fractured. Grading very hard.
 - Boring 3A. (near west side of tank). Top 10 feet. Dark brown fine sandy clay (CL), stiff greenish-brown sandy shale, hard, weathered, fractured. Grading very hard. Very hard, 1 to 2 feet.

2.3.2 Phase 2. Boronda Reservoir to Monte Bello Reservoir Section

Figure 2-33 shows the alignment of the second phase of construction of the Quarry to Monte Bello pipeline. This section was designed in 1964. The common cover in this section is about 4 feet.

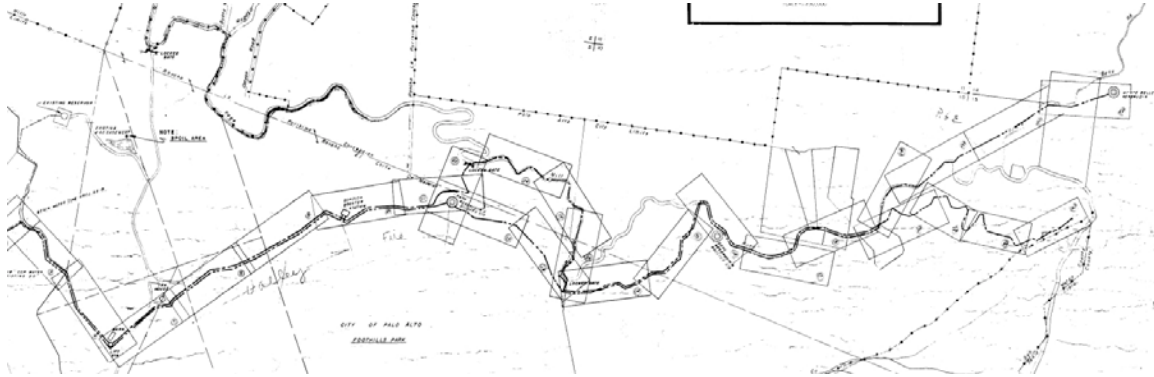


Figure 2-33. Boronda to Monte Bello Pipeline Alignment

South of the Boronda Booster, the pipe changes to 16" CCP and goes up a steep hill, rising some 670 feet over a horizontal length of about 1,600 feet (slopes about 40% to 41% common) (see sheet 10 of 49, ABM #583_Foothills 62-8.pdf). At the top of this hill is the Park Reservoir. Immediately parallel to the water pipe is a VCP sewer pipe that was constructed at the same time.

No soil borings are available over this steep slope. This slope is mapped by the CGS as being "landslide susceptible". However, there are no mapped active landslides along this alignment, and we suspect that CGS's criteria for this area was based primarily on slope.

Between the Park Booster station and Dahl reservoir, the pipe is 16" CCP, with two significant sloped sections to Dahl Reservoir (one section with uphill slope of 35%, one section with downhill slope of 15%). These sections are also mapped as landslide susceptible by CGS, and again, we believe this is due to slope, not due to observed slope movements.

From Dahl to Monte Bello, the pipe changes to 14" CCP. The uphill slopes are typically under 10%. A short section between stations ~160 to 162 (200 feet in length) is mapped as landslide susceptible, between elevations 2,032 and 2,055 feet.

2.3.3 Pipe Relocation

In October 1965, a portion of the original 1960 18" CCP pipe (about 530 meters) was relocated to the east, as part of the construction of the Junipero Serra Freeway. The relocated pipe is referred to in AB 521 (drawings not available for review).

2.4 Soil Resistivity

To help understand the reasons for the observed pipe leak repair rates in Palo Alto, as well as to provide data for the selection of new pipes, a soil-resistivity test program was conducted, so that we could quantify the effect of soil resistivity (Rho, ohm-cm) versus the observed long-term pipe leak performance in Palo Alto.

The soil resistivity test program is described in Appendix E. A total of 71 Wenner 4-point tests were performed on March 4, 9 and 16, April 13, 2015 at locations roughly equally spaced throughout Palo Alto; and on April 10, 2015 and May 6 2015 in Napa (for calibration of pipe repair rates in the August 2014 earthquake). For each site, the Rho values were tested for soil layer depths of 2.5, 5, 7.5, 10 and 15 feet. Then, the Rho value for each layer was computed using the method by Hummer.

Figure 2-34 shows the locations of the tests (colored dots), with the computed Rho values for the soil layer at f feet beneath grade indicated by the colors.

The usually interpretation of Rho for metal pipe is as follows:

- $R = 500$ to $1,500$ ohm-cm. Extremely corrosive.
- $R = 1,500$ to $3,000$ ohm-cm. Highly corrosive.
- $R = 3,000$ to $5,000$ ohm-cm. Corrosive.
- $R = 5,000$ to $10,000$ ohm-cm. Moderately corrosive.
- $R = 10,000$ to $20,000$ ohm-cm. Mildly corrosive.
- $R > 20,000$ ohm-cm. Essentially non-corrosive.

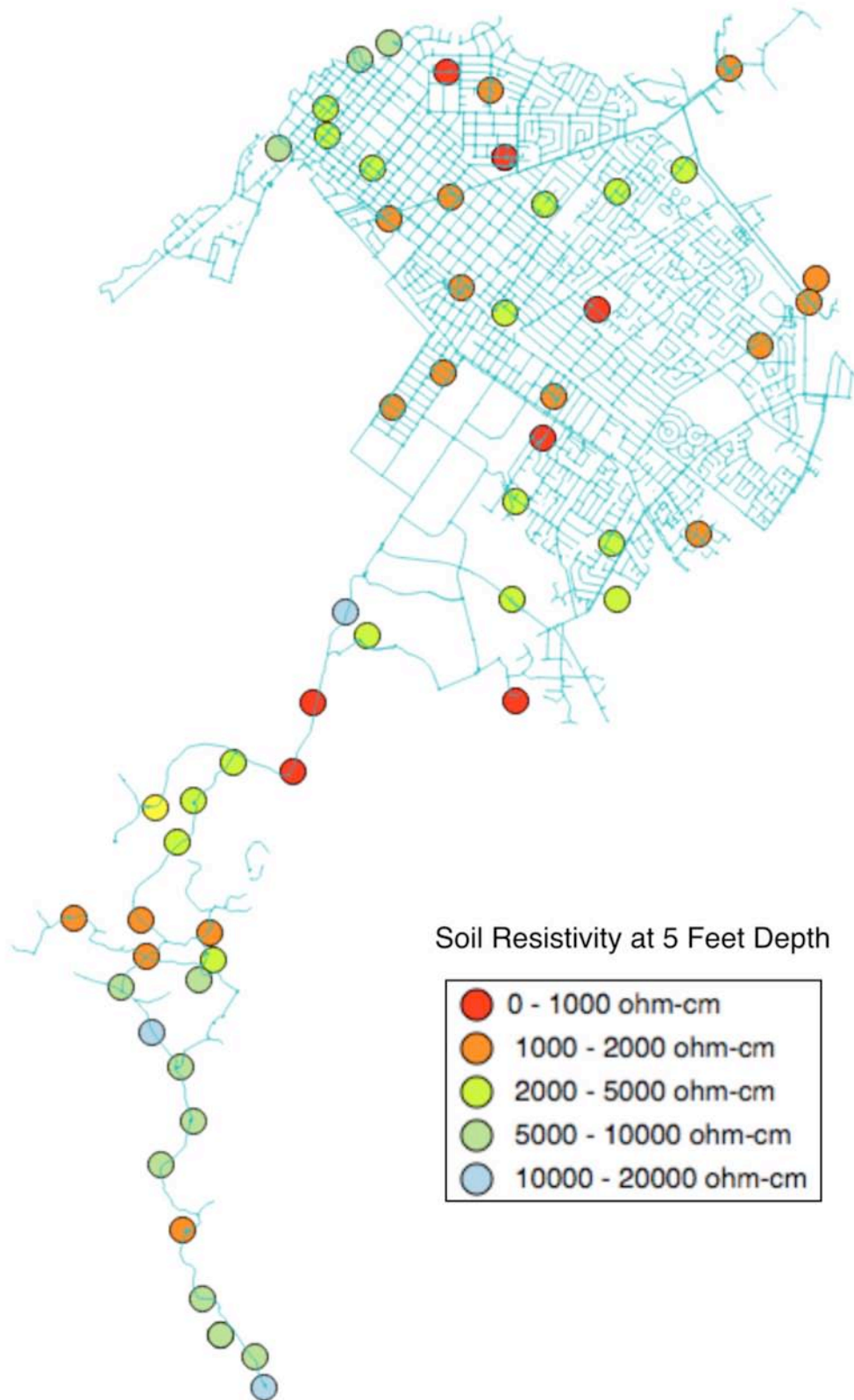


Figure 2-34. Soil Rho Test Values (5 feet depth)

We assigned the tested Rho values (ohm-cm) to each individual Palo Alto pipe main as follows:

- We computed the actual distance from the tests to the midpoint of each Palo Alto pipe as drawn in the GIS, (7,315 pipes).
- We calculated the weighted average of the closest five test points. Each pipe in the database was assigned a Rho value based on the weighted average of the five geographically-closest tests to the pipe. The weighted average reflects that if a test was within 100 feet of the pipe, and the four other tests were within 2,000 feet of the pipe, the Rho value assigned to that pipe is almost entirely based on the Rho value from the close-in test. The weighted average reflects that if all five tests were exactly the same distance from the pipe, then the Rho value assigned to that pipe is the arithmetic average of the five tests. The 68 Rho tests are included in the database, as well as the actual five tests (and distances from the test to the pipe) used to set the Rho value at each pipe.

$$Rho_{pipe} = \sum_{i=1}^5 w_i Rho_i$$

$$w_i = \frac{1/d_i}{5 \sum_{i=1}^5 1/d_i}$$

where $i = 1, 5$, representing the five closest tests to the pipe, d_i is the distance (in feet) from the test to the pipe, w_i is the weight given to each test, and Rho_i is the Rho value for each test at the depth of the pipe, (ohm-cm). For example, say there are five tests with $Rho = 480, 420, 390, 1860, 720$ ohm cm; and distances = 89, 2836, 3071, 3582, 3827 feet, respectively. The average Rho from the five tests is 774 ohm-cm. However, the data from the first test (480 ohm-cm) is likely to be more relevant, as the first test is 89 feet from the pipe, while the other 4 tests average over 3,000 feet away. Given the above weighting function, the first test is weighed 90%, and the 2nd through 5th tests are weighted about 2.5% each, and Rho_{pipe} is 512 ohm-cm.

Figure 2-35 shows the length of pipe, versus Rho values for each kind of pipe main. The vertical scale is shown as a "log" scale, so that it is easier to see the actual lengths of pipe for small values. Figure 2-36 shows the same data, but only for metal pipe.

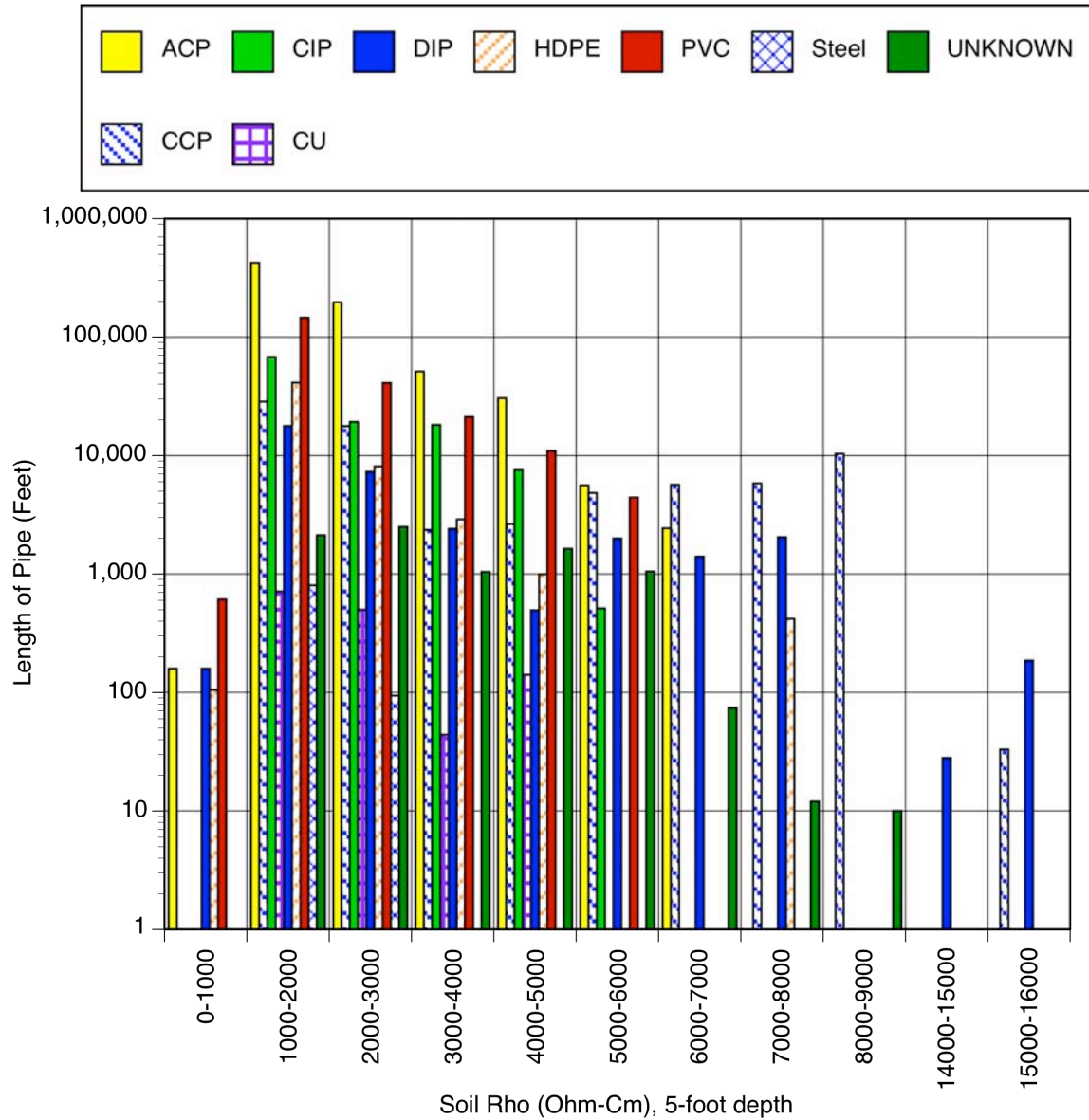


Figure 2-35. Soil Rho at Pipe (5 foot depth)

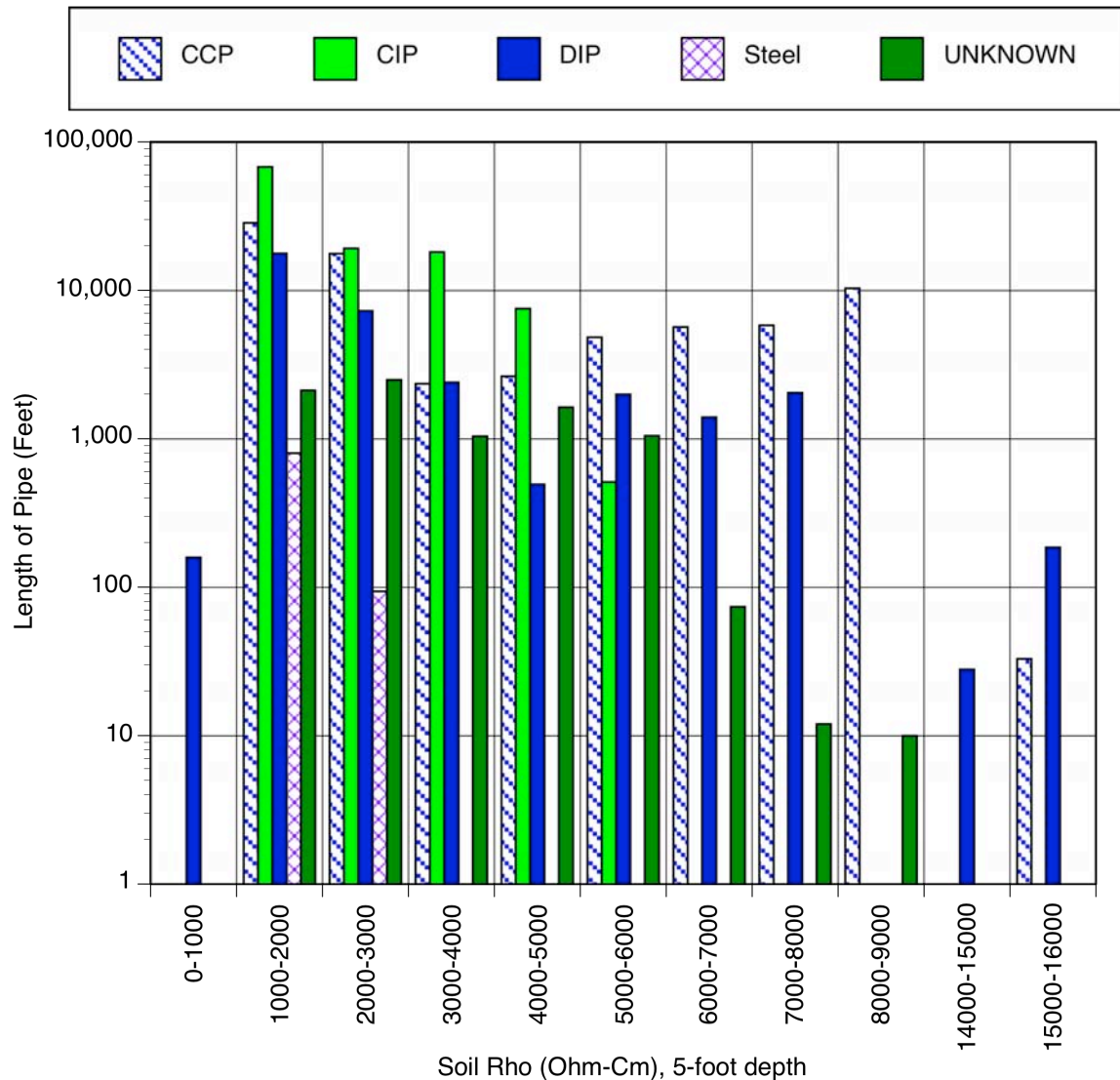


Figure 2-36. Soil Rho at Pipe (5 foot depth) (Metal Pipe only)

The pipes that will be most affected by Rho are metal pipe, with bare exterior, or with failed coating systems. The cast iron pipe (light green bars) have the longest lengths in the most corrosive soils.

The CCP pipes (blue left diagonal bars) are used both in the lowland areas near the Bay (Rho under 3000 ohm-cm) as well as in the foothills (rho typically over 5,000 ohm-cm). To the extent that the CCP coating system (a thick layer of gunite) is intact, the soil Rho will not have much affect; but if the gunite concrete is cracked, the soil will first attack the exterior rebars, and eventually the steel cylinder.

Palo Alto reports that its DIP (dark blue bars) was possibly placed with polyethylene "baggies" around the pipe. This is supposed to inhibit direct attack by the soil on the pipe. However,, we do not have factual evidence that these baggies were in fact used.

The pipes labeled "Unknown" (dark green bars) are presumed to be mostly cast iron, as the quality of records with older pipes is poorer; but this is speculative.

2.5 Abandoned Pipes

Since about 1990, Palo Alto has been replacing water pipes in the water system. Figure 2-37 shows a map of the water system, highlighting the pipes that have been already been abandoned (heavy orange lines) and pipes currently in service (as of 2010).



Figure 2-37. Abandoned Water Pipes

Comparing Figure 2-37 with the Figure 2-5 (Map of Palo Alto, 1899), and Figure 2-7 (location of currently active cast iron pipes), it is apparent that the bulk of the abandoned pipes are located in the oldest parts of Palo Alto, and that much of the abandoned pipe was older cast iron pipe. The GIS layer with the abandoned pipe information has 4856 pipe segments, with all segments having length attributes, and for a relatively small percentage, original pipe diameter and material and date of original installation data. Tables 2-12 and 2-13 show the length of abandoned pipe by pipe material and diameter.

Pipe Material	Length Miles
ACP	12.03
CCP	0.29
CIP	32.11
CU	0.05
DIP	0.13
HDPE	0.00
PVC	0.85
Unknown	28.13
Total	73.73

Table 2-12. Length of Abandoned Pipe, by Pipe Material (Miles)

Diam (Inch)	ACP	CCP	CIP	CU	DIP	HDPE	PVC	Steel	Unknown	Total
0.5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.21	0.21
1	0.00	0.00	0.12	0.00	0.00	0.00	0.02	0.00	0.02	0.16
1.25	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.37	0.37
2	0.29	0.00	0.89	0.05	0.00	0.00	0.00	0.00	1.49	2.72
3	0.00	0.00	0.08	0.00	0.00	0.00	0.00	0.00	0.33	0.41
4	1.06	0.00	16.39	0.00	0.01	0.00	0.00	0.00	1.25	18.71
6	5.91	0.01	10.36	0.00	0.01	0.00	0.65	0.00	0.78	17.72
8	2.58	0.00	2.48	0.00	0.10	0.00	0.18	0.00	0.13	5.47
10	0.30	0.00	1.05	0.00	0.00	0.00	0.00	0.00	0.01	1.36
12	1.59	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.59
14	0.29	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.29
16	0.00	0.00	0.61	0.00	0.00	0.00	0.00	0.14	0.06	0.82
18	0.00	0.28	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.28
Unknown	0.01	0.00	0.12	0.00	0.00	0.00	0.00	0.00	23.48	23.61
Total	12.03	0.29	32.11	0.05	0.12	0.00	0.85	0.14	28.13	73.73

Table 2-13. Length of Abandoned Pipe, by Pipe Material and Diameter (Miles)

The data shows that the most common abandoned pipe was cast iron (70% of all replacements with known pipe material), most commonly either 4-inch or 6-inch diameter. The most likely reason(s) for cast iron replacement were:

- 4-inch pipe cannot provide sufficient fire flows to meet modern fire-fighting criteria
- Cast iron pipe had historically high pipe repair rates, and thus replacement with new pipes reduced the economic impacts of frequent forced outages due to ongoing leaks.
- Cast iron pipe was typically unlined, meaning that over time, the equivalent internal diameter becomes reduced due to the effects of tuberculation.

The data shows that the second most common abandoned pipe was asbestos cement (26% of all replacements with known pipe material), either 6-inch or 8-inch diameter. The most likely reason(s) for cast iron replacement were:

- Some asbestos cement pipe might have had historically high pipe repair rates, and thus replacement with new pipes reduced the economic impacts of frequent forced outages due to ongoing leaks.
- Palo Alto has had a plan to replace AC pipe for some time. This plan factored in historical leak rates, aging and other factors. AS AC pipe is the second oldest style of pipe in the Palo Alto system (cast iron being oldest), it is natural to begin AC pipe replacement once the bulk of most problematic the CI pipe has been replaced.

A small percentage (about 4%) of abandoned pipe are more modern pipe material such as HDPE or PVC; these pipes are presumed to have most likely to have been abandoned because of required pipe relocations or upsizing for hydraulic purposes.

2.6 Palo Alto Income

For purposes of benefit cost analyses, the economic impacts of disruptions to the water supply are needed. In this section, we describe the general economic activity in Palo Alto, zip codes 94301 and 94305.

The median household incomes for Palo Alto are amongst the highest in California.

According to census data, the average household income in Palo Alto was \$117,251 (2012 data), with a resident population of 60,171 people (2009 data, with an aggregate household income of \$2,122,175,553 (\$2.122 billion). (This data from city-data.com, accessed March 15 2015). Median household incomes in Palo Alto about 2.01 times the median for the State of California as a whole. Median residence valuations (2009 data, single family or condominium) in Palo Alto was over \$1,000,000 (\$983,880 detached houses; townhomes \$651,117; units in 2-unit structures \$562,122; units in 3-4 unit structures \$486,340; units in 5+ unit structures \$502,094; mobile homes (\$184,523); versus an average residential valuation of \$349,000 for the State of California as a whole.

The Gross Domestic Product (GDP) of the USA for 2013 was \$15.526 trillion. Of this amount, California was \$2.051 trillion.

The Palo Alto population for 2013 was estimated to be 66,642 people. The State of California population for 2013 was 38 million.

The top employers in Palo Alto, per the City's 2011 Comprehensive Annual Financial Report, include:

- Stanford University, 10,979 employees
- Stanford University Medical Center / Hospital. 5,545 employees
- Lucile Packard Children's Hospital, 4,750 employees
- Veterans Affairs Palo Alto Health Care System. 3,850 employees
- VMware Inc. 3,509 employees
- Hewlett-Packard. 2,500 employees
- Palo Alto Medical Foundation. 2,200 employees
- SAP. 2,200 employees
- Space Systems / Loral. 3,020 employees

- Wilson Sonsini Goodrich & Rosati. 1,650 employees
- Palo Alto Unified School District. 1,318 employees
- City of Palo Alto. 1,019 employees.

Given this data, the GRP of Palo Alto might be about 2 to 3 times the per capita average for the State of California as a whole. This yields GRP of $(66,642 / 38,000,000) * \$2,051 \text{ billion} * (2 \text{ to } 3) = \$7.193 \text{ to } \$10.79 \text{ billion per year}$. Taking an average of these two values, and increasing to year 2015 dollars by 5%, yields current GRP of \$9.441 billion, or a daily GRP of $\$9.441 \text{ billion} / 365 \text{ days} = \$25,865,753 \text{ per day}$.

3.0 Seismic Hazards

Section 3 is subdivided into the following sections:

- Section 3.1. Seismology and Scenario Earthquakes
- Section 3.2. Streams
- Section 3.3. Geotechnical hazards
- Section 3.4. Ground shaking hazards
- Section 3.5. Liquefaction hazards
- Section 3.6. Landslide hazards
- Section 3.7. Surface faulting hazards
- Section 3.8. Water System Performance in Past Earthquakes (see Appendix B for the performance of the Napa water system in the Napa 2014 earthquake).
- Section 3.9. Soil borings

3.1 Seismology and Scenario Earthquakes

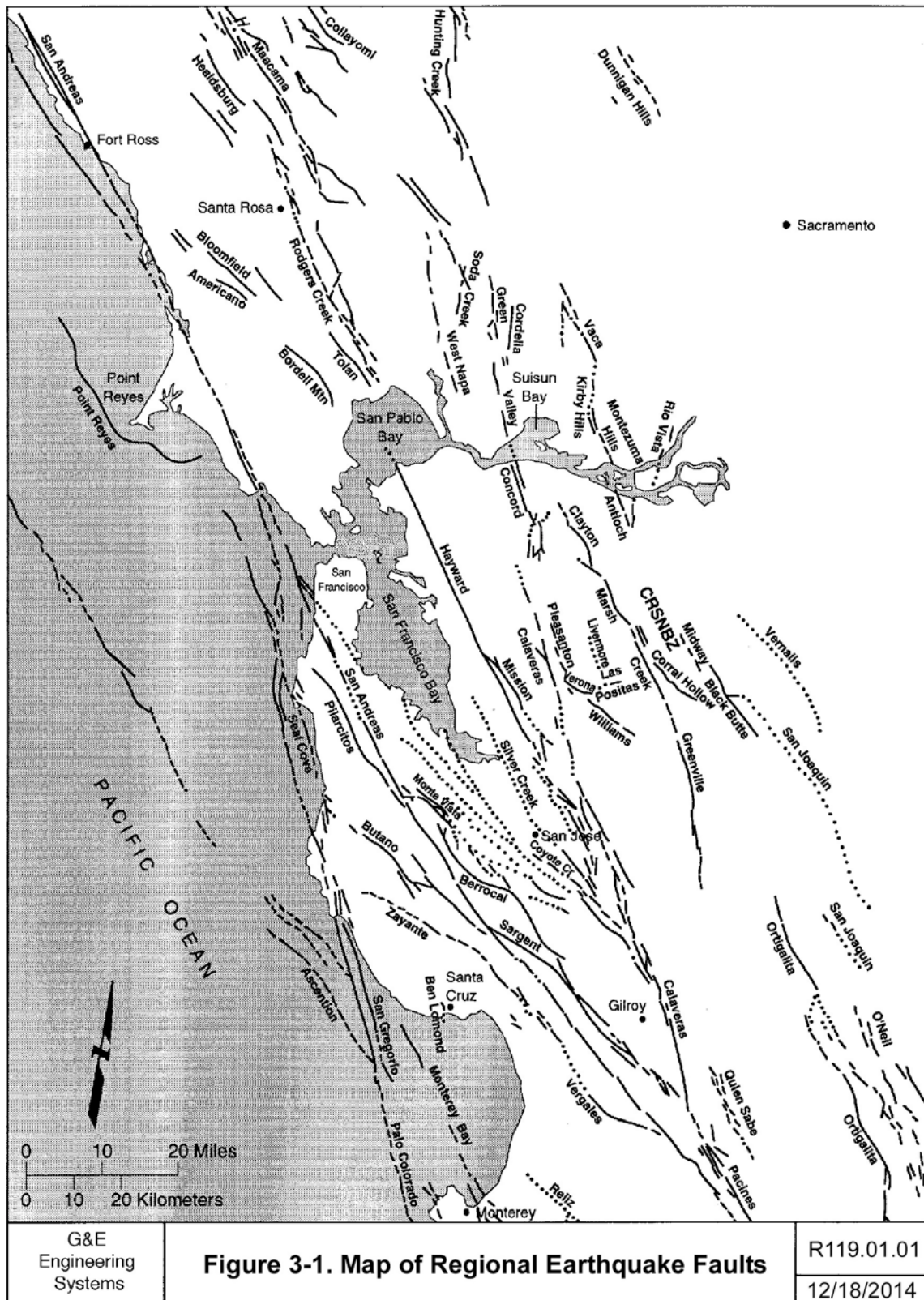
Based on its record of historic earthquakes and its position astride the North American - Pacific plate boundary, the San Francisco Bay region, within which the City of Palo Alto is located, is considered to be one of the more seismically active regions of the world. During the historical period (approximately 170 years), faults within the region have produced 14 moderate to large magnitude ($M > 6$) earthquakes affecting the Bay Area, as well as many significant smaller magnitude ($5 < M < 6$) earthquakes (ref. Toppazada et al 1979, Toppazada et al 1981 and Real et al 1978). Faults within the 80 km (50 mile) wide North American - Pacific plate boundary zone that may influence potential earthquake ground shaking and other earthquake-related hazards within the City of Palo Alto area are illustrated in Figure 3-1.

Among the historically active regional faults, those anticipated to have potential significance to the performance of the City of Palo Alto water facilities include the following:

- San Andreas
- Hayward

- Calaveras
- San Gregorio

Other faults in the region that might have some affect in Palo Alto include the Greenville, Mount Diablo, and the Rodgers Creek faults



There are other faults that bisect through the Foothills area of the City of Palo Alto. These faults include:

- Monta Vista-Shannon. Possibly active.
- Berrocal. Potentially active.

Both the Monta Vista-Shannon and the Berrocal faults might undergo secondary / sympathetic movements given a large earthquake on the San Andreas (or Hayward) faults.

Brief discussions of each of these sources are presented in the following paragraphs. Unless otherwise noted, magnitude (M) refers to moment magnitude.

San Andreas Fault. The San Andreas Fault is situated immediately to the west southwest of the Monte Bello reservoir, and within 10 km west southwest of the lower elevation areas of Palo Alto. The San Andreas Fault, which extends over 1,200 km (750 miles) from the Gulf of California to Cape Mendocino, is the major fault within the region and has generated four moderate to large earthquakes during the historical period (approximately 160 years), a M 7 event in June 1838, a M 6.3 event in October 1965, the great M 8 earthquake in April 1906, and the recent M 6.9 Loma Prieta earthquake on October 17, 1989. The Southern Santa Cruz Mountains segment of the San Andreas Fault, on which the Loma Prieta earthquake is thought to have occurred, is situated about 20 km south of the lower elevation areas of Palo Alto. The Working Group on California Earthquake Probabilities (Working Group 2008) has estimated that, during the 30-year time period between 2007 and 2036, there is a 21 percent probability of a M 6.7 or larger earthquake occurring on the San Francisco Peninsula segment of the San Andreas Fault, which extends northward from the Loma Prieta rupture segment, and a less than 5 percent probability of a M 8 earthquake along the North Coast segments of the fault. The maximum earthquake for the San Andreas Fault is judged to be in the range of M 7.75 to M 8 (moment magnitude); recent work (Niemi and Hall, 1992) indicates that, on the average, an event of such magnitude can be expected to occur approximately every 200 to 300 years.

Hayward Fault. The Hayward fault is situated about 23 km (14 miles) to the east-northeast of the lower elevations of Palo Alto. The Hayward Fault is a major component of the San Andreas Fault system in the Bay Area and extends approximately 114 km (71 miles) from its intersection with the Calaveras fault southeast of San Jose northward through and along the East Bay hills to San Pablo Bay. It has been suggested on the basis of micro-seismicity data that the Hayward Fault may connect with the Healdsburg-Rodgers Creek fault beneath San Pablo Bay (Ellsworth et al, 1982), although such a connection requires an en echelon jump between the faults. Two potential rupture segments for the Hayward Fault are commonly postulated: a southern segment extending from Warm Springs (Fremont) to the San Leandro-Mills College area (or perhaps as far north as northern Oakland), and a northern segment extending from the this transition

point to San Pablo Bay. The southern segment has been the source of a large (M 6.8) earthquake during the historical period (October 1868). The Working Group (2008) has estimated that, during the 30-year time period from 2007 to 2036, there is a 31 percent probability of a M 6.7 (or larger) earthquake occurring on the Hayward Fault. The maximum earthquake for the Hayward Fault is judged to be in the range of M 7 to M 7.25; the average recurrence of such events is estimated to be approximately 150 to 250 years.

Calaveras Fault. The Calaveras Fault is situated about 30 km (20 miles) to the east of the lower elevations of Palo Alto. The approximately 120 km (75 mi) long Calaveras Fault extends from south of Hollister to near Danville in Contra Costa County. The fault has been associated with the historical earthquakes of M 5.6 (July 1861), M 5.6 (March 1866), M 6.2 (June 1897), M 5.8 (July 1899), M 6.6 (July 1911), M 5.8 (August 1979), M 6.2 (April 1984) and M 5.1 (February 1988). The maximum earthquake for the Calaveras Fault is judged to be in the range of M 6.75 to 7; the average recurrence of such events is estimated to be approximately 150 to 300 years.

Monta Vista - Shannon Fault Zone. The Monta Vista - Shannon Fault Zone is one of the secondary faults in the hills above Palo Alto (see Figure 3-2). The USGS (2010 fault database) characterizes the fault zone as follows:

- Most recent surface-deforming earthquake occurred within the last 15,000 years. No surface deforming earthquakes since 1850.
- Slip rate is variously ascribed as being under 0.2 mm per year, or being between 0.2 mm to 1 mm per year.
- Sense of slip is thrust.
- Fault location near the CCP is poorly constrained.

The Monta Vista - Shannon Fault Zone is described as follows (after Bryant 2000). The Monta Vista - Shannon Fault Zone is late Quaternary active and possibly Holocene active, reverse to reverse-dextral oblique slip that forms what McLaughlin and others (1996) refer to as the Southwestern Santa Clara Valley thrust belt, generally located along the foothills of the northeastern Santa Cruz Mountains. The Monta Vista - Shannon Fault Zone is commonly associated with the Berrocal Fault Zone. The Monta Vista - Shannon Fault Zone offsets sediment of the Pliocene-Pleistocene Santa Clara Formation; locally colluvial deposits are thrust over fluvial gravel of Permanente Creek, indicating late Pleistocene and possibly Holocene displacement. Significant parts of the fault zone are concealed by Holocene alluvium. The speculated sense of dip is about 45 degrees (on average) to the southwest. Minor distributed coseismic contractual deformations in the urbanized areas along the northeastern flank of the Santa Cruz Mountains associated with the 1989 Loma Prieta earthquake locally was coincident with the general trend and locations of the Monta Vista - Shannon and Berrocal Fault Zones.

The Shannon Fault was first named and mapped in detail in 1964. The Monta Vista Fault was first mapped in detailed in 1975, and is thought to be the northwest extension of the Shannon fault.

If the movements of the fault are coincident with Loma Prieta-type earthquakes, then the return period of movement might be on the order of 400 years or so. With larger magnitude earthquakes of the San Andreas Fault, the range of movement along the Monta Vista – Shannon Fault Zone may be between 0% and 15% (or so) of the movement of the San Andreas. On the other hand, if the movement is independent of the San Andreas Fault, then the return interval might be much longer (5,000 to 100,000+ years).

The **Berrocal Fault Zone** is one of the secondary faults in the hills above Palo Alto. The USGS (2010 fault database) places the surface expression of the fault crossing the CCP between the Boronda Reservoir and the Boronda pump station. The Berrocal Fault Zone has the following characteristics:

- Most recent surface-deforming earthquake between 750,000 to 1,600,000 years ago
- Location of the fault is inferred or poorly constrained where it crosses the 18" CCP.
- Sense of slip is reverse

The Berrocal Fault zone is described as follows (after Bryant 2000). Herein, we use the name "Berrocal" Fault to refer to this fault zone. The Berrocal Fault is a late Quaternary, southwest dipping, reverse-dextral oblique slip fault zone that forms what McLaughlin and others (1996) refer to as the Southwestern Santa Clara Valley thrust belt, generally located along the foothills of the northeastern Santa Cruz Mountains.

The Berrocal Fault is commonly associated with the Monte Vista - Shannon Fault Zone.

The Berrocal Fault offsets sediment of the Pliocene-Pleistocene Santa Clara Formation and probably deforms late Pleistocene fluvial and alluvial fan deposits (Hitchcock et al, 1994, 1999). Bedrossian (1980) concluded that the Berrocal Fault Zone lacks evidence of Holocene displacement. Late Quaternary slip rate is poorly constrained, and the recurrence interval is unknown.

Minor distributed coseismic contractual deformations in the urbanized areas along the northeastern flank of the Santa Cruz Mountains associated with the 1989 Loma Prieta earthquake locally was coincident with the general trend and locations of the Monta Vista and Berrocal Faults.

The dip of the fault is between 20 degrees to the southwest to 90 degrees to steeply to the northeast.

If the movements of the fault are coincident with Loma Prieta-type earthquakes, then the return period of movement might be on the order of 400 years or so, with the range of movement estimated to be between 0 and 15% (or so) of the movement of the San Andreas. On the other hand, if the movement is independent of the San Andreas Fault, then the return interval might be much longer (likely under 130,000 years).

The **Hermit Fault** is one of the secondary faults in the Hills above Palo Alto. The USGS (2010 fault database) places the surface expression of the fault north of the CCP (no crossings), with the following characteristics:

- Most recent surface-deforming earthquake between 750,000 to 1,600,000 years ago
- Fault location is well located and expressed in the surface landscape
- Slip rate is under 0.2 mm per year

The **Pilarcitos Fault** is one of the secondary faults in the hills above Palo Alto. The USGS (2010 fault database) places the surface expression of the fault west of the San Andreas Fault zone, with the following characteristics:

- Most recent surface-deforming earthquake between 750,000 to 1,600,000 years ago
- Fault location is well located and expressed in the surface landscape
- Slip rate is under 0.2 mm per year

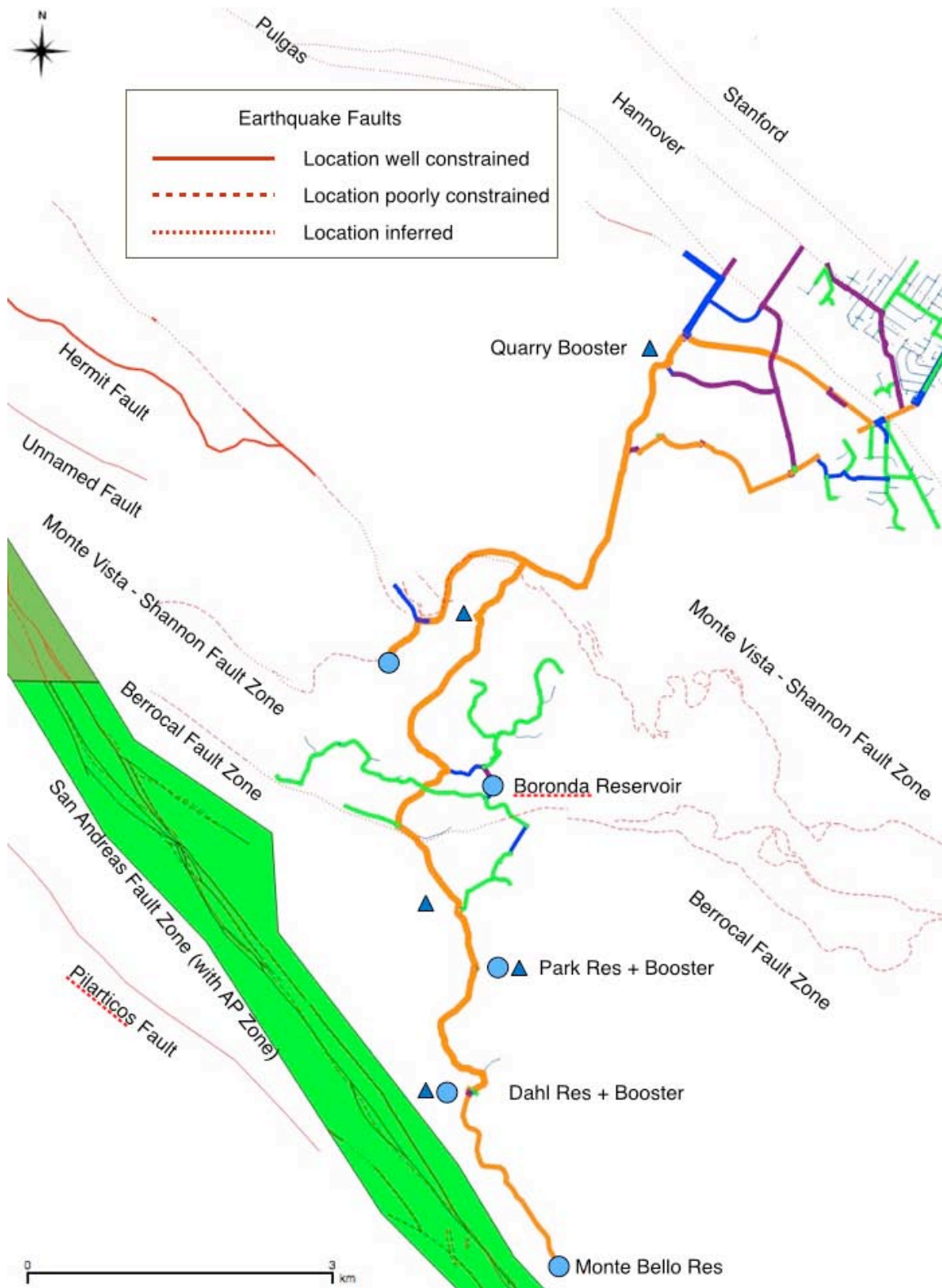


Figure 3-2. Earthquake Faults and Foothills Water System

Other Faults: As can be seen in Figure 3-1, there are a number of other faults in the San Francisco Bay Area that could impact the City of Palo Alto. Of those not already mentioned above, the Rodgers Creek Fault (north of San Pablo Bay) and the San Gregorio Fault (at the Pacific Ocean coast line) are two of the more active and capable of large magnitude earthquakes. Due to their locations, the impacts on the City of Palo Alto water system from earthquakes on these faults will be less severe than those from a great earthquake on the San Andreas Fault.

There are several other mapped faults mapped outside of the Foothills area. These include the Hermit, Pilarcitos, Stanford, Hannover, and Pulgas Faults, as shown in Figure 3-2. The activity rates on these faults is low, and they do not cross the pipelines in the Foothills.

Figure 3-3 shows a map of the major faults in the San Francisco Bay area with associated probabilities of occurrence by the year 2036 (Working Group, 2008).

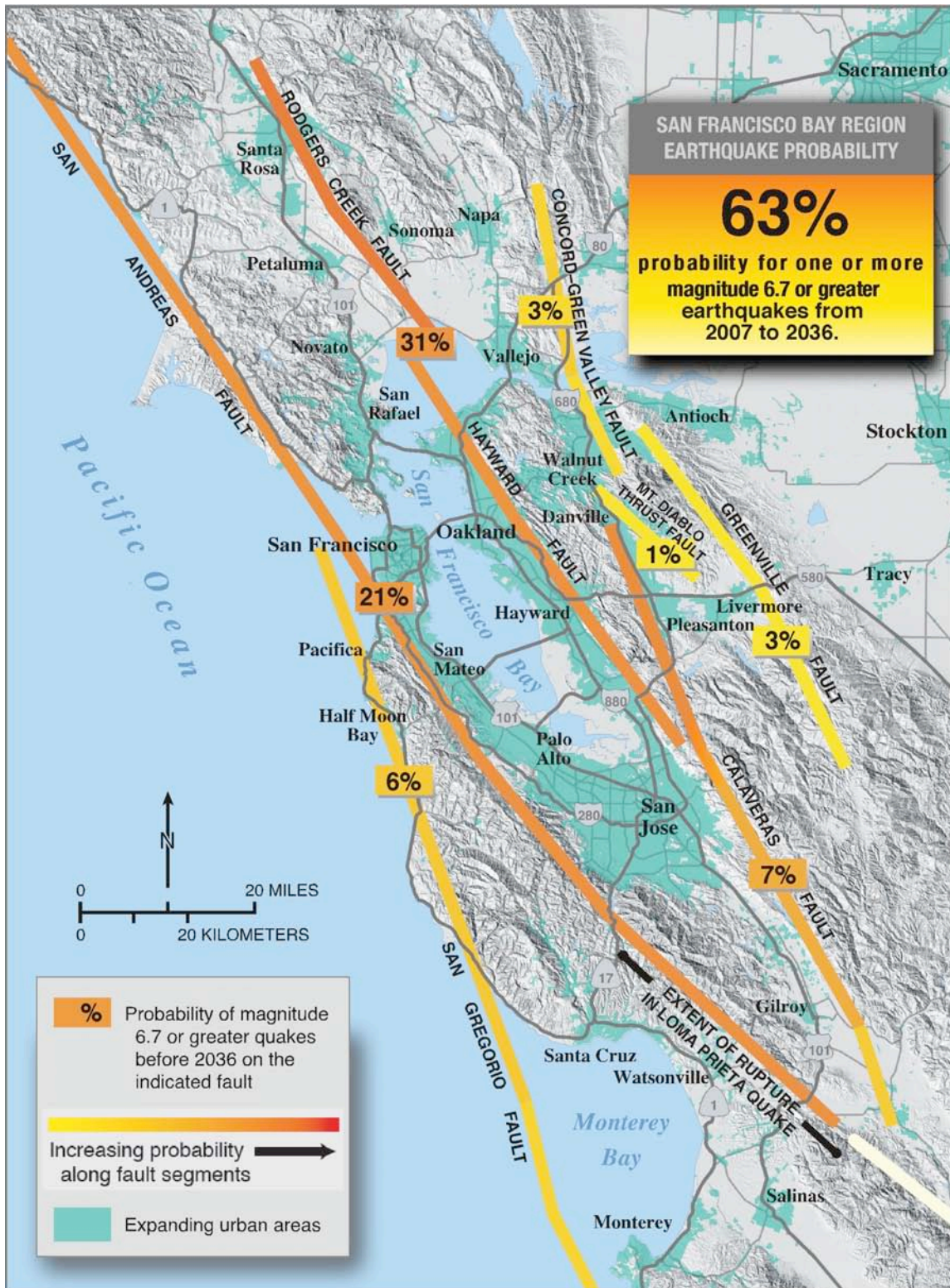


Figure 3-3. Earthquake Probabilities (after USGS, 2006)

To examine the range of possible performance of the Palo Alto pipelines in future earthquakes, we selected a range of scenario earthquakes. By "scenario", we mean that a

specific fault is assumed to rupture with assumed magnitude and epicenter location and then the performance of the water pipeline system is evaluated. Table 3-1 lists the 24 scenario earthquakes evaluated for the water system.

EQ	Fault	Magnitude	Epicenter	Comment
1	San Andreas Santa Cruz	6.9	50	Repeat Loma Prieta 1989
2	San Andreas Peninsula	6.0	50	
3	San Andreas Peninsula	6.2	50	
4	San Andreas Peninsula	6.4	50	
5	San Andreas Peninsula	6.6	50	
6	San Andreas Peninsula	6.8	50	
7	San Andreas Peninsula	7.0	50	
8	Hayward N+S	7.25	50	MCE
9	Hayward South	6.8	50	Repeat 1868
10	Hayward North	6.8	50	
11	West Napa	6.0	0	Repeat 2014
12	Rodgers Creek	7.0	50	
13	Calaveras North + Central + South	7.2	50	MCE
14	San Gregorio	7.7	100	MCE
15	Mount Diablo Thrust	6.5	50	
16	San Andreas SAN+SAP+SAS	8.0	60	MCE
17	San Andreas SAN+SAP+SAS	7.9	60	Repeat 1906
18	San Andreas SAN+SAP+SAS	7.7	60	Closest to Palo Alto
19	San Andreas SAN+SAP+SAS	7.5	60	Closest to Palo Alto
20	San Andreas SAN+SAP+SAS	7.4	60	
21	San Andreas SAN+SAP+SAS	7.2	60	
22	Monta Vista	6.8	50	MCE
23	Greenville	7.0	50	MCE
24	Zayante – Vergeles	6.9	50	MCE

Table 3-1. Scenario Earthquakes

3.2 Streams

A review of the streams in Palo Alto is presented below. The importance of the streams for purposes of this report has mainly to do with the modern locations of soils prone to liquefaction, as streams and their flood plains often are underlain by young Holocene materials, sometimes with loose sands that can liquefy under moderate to strong ground shaking in earthquakes.

Palo Alto is crossed by several creeks that flow north to San Francisco Bay:

- San Francisquito Creek (Figure 3-4) forms the western boundary of Palo Alto

- Adobe Creek is on the eastern boundary of Palo Alto
- Matadero Creek is in between these two creeks.

Arastradero Creek is a tributary to Matadero Creek, and Barron Creek is now diverted to Adobe Creek just south of Highway 101 by a diversion channel.

San Francisquito Creek is formed by the confluence of Corte Madera Creek and Bear Creek, below Searsville Lake. Los Trancos Creek is a tributary to San Francisquito Creek below Highway 280. Searsville Lake is formed by Searsville Dam, which was built in 1892. Searsville Dam does not provide potable water, flood control or hydropower, and the lake behind it has lost over 90% of its original water storage capacity as sediment has filled the lake.

In 1857, the U.S. Coast Survey indentified 1,142 acres of tidal marsh at the mouth of San Francisquito Creek. There were also two large (63-acre and 118-acre) willow groves adjacent to the tidal marsh associated with high groundwater tables and seasonal flooding. In the late 1920s, levees were constructed to re-route the creek through a new engineered channel from this former mouth, to a sharp turn north for about half a mile, then to the northeast, before exiting to the Bay. By 2004, filled areas of the former marsh lands include the Palo Alto golf course and the Palo Alto Airport. In normal winters, the creek runs sluggishly; in summer it is usually dry. The creek is capable of flooding: in 1988 storms, the creek flooded out of its banks, resulting in \$28 million flood damage in Palo Alto, Menlo Park and East Palo Alto.

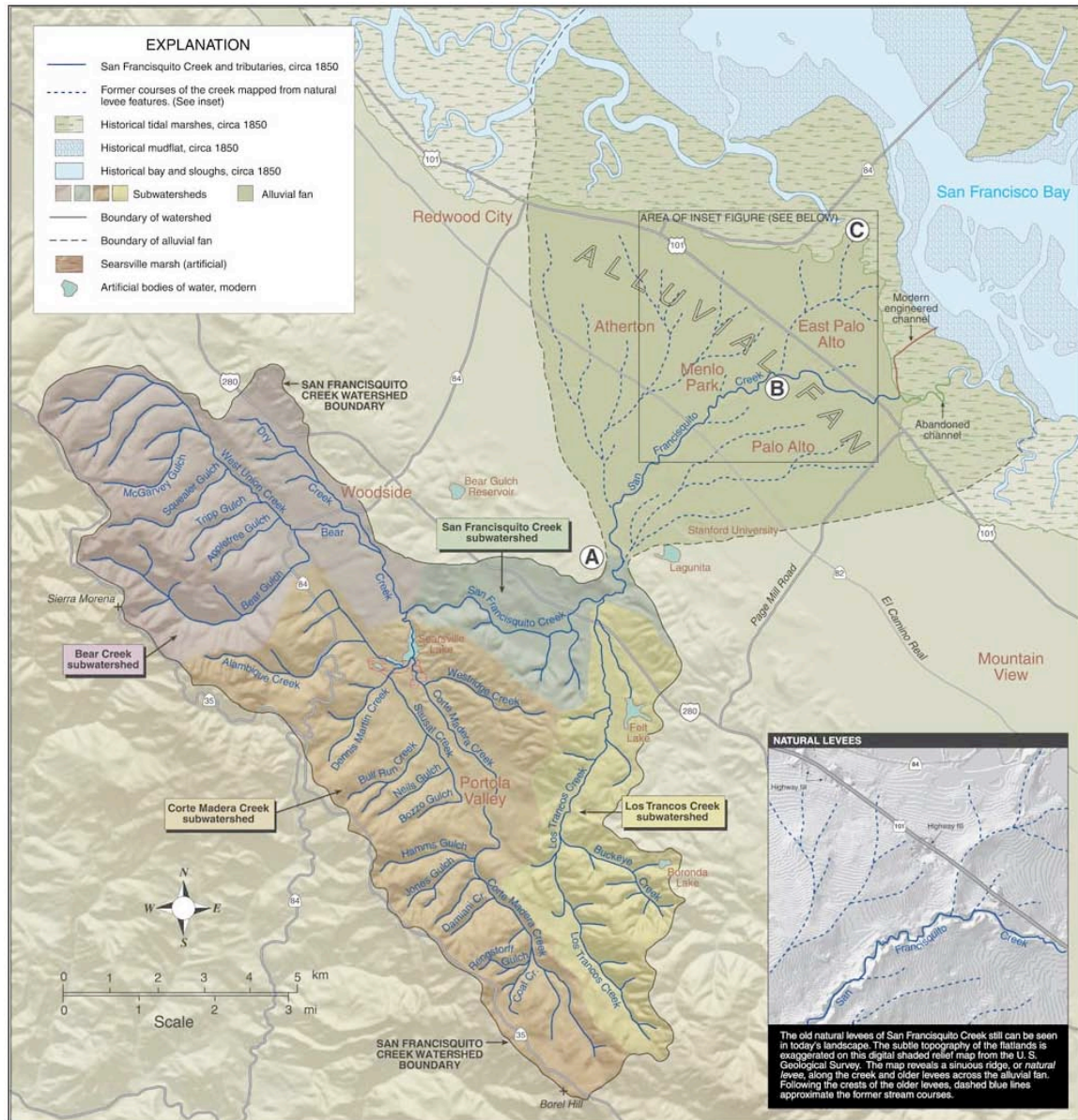


Figure 3-4. San Francisquito Watershed and Alluvial Fan (after Sowers, 2005)

Figure 3-5 shows the alignment of Matadero Creek and other creeks in central and southern Palo Alto.



Figure 3-5. Matadero, Deer, Barron and Adobe Creeks (after Sowers, 2005)

A portion of the Quarry-to-Boronda pipeline runs adjacent to Matadero Creek. The Quarry booster pump station is located immediately adjacent to Matadero Creek, just upstream of the confluence with Deer Creek.

In Figure 3-5, the northern reaches of Matadero, Barron and Adobe Creek are drawn in nearly straight lines. These reflect modern engineered channels that carry the stream flows to the Bay, see Figure 3-6. Prior to construction of these channels, the streams discharged at about the locations of the upstream ends of the modern channels. The large dotted line in Figure 3-6 indicates the former boundary of the tidal marsh near the Bay.

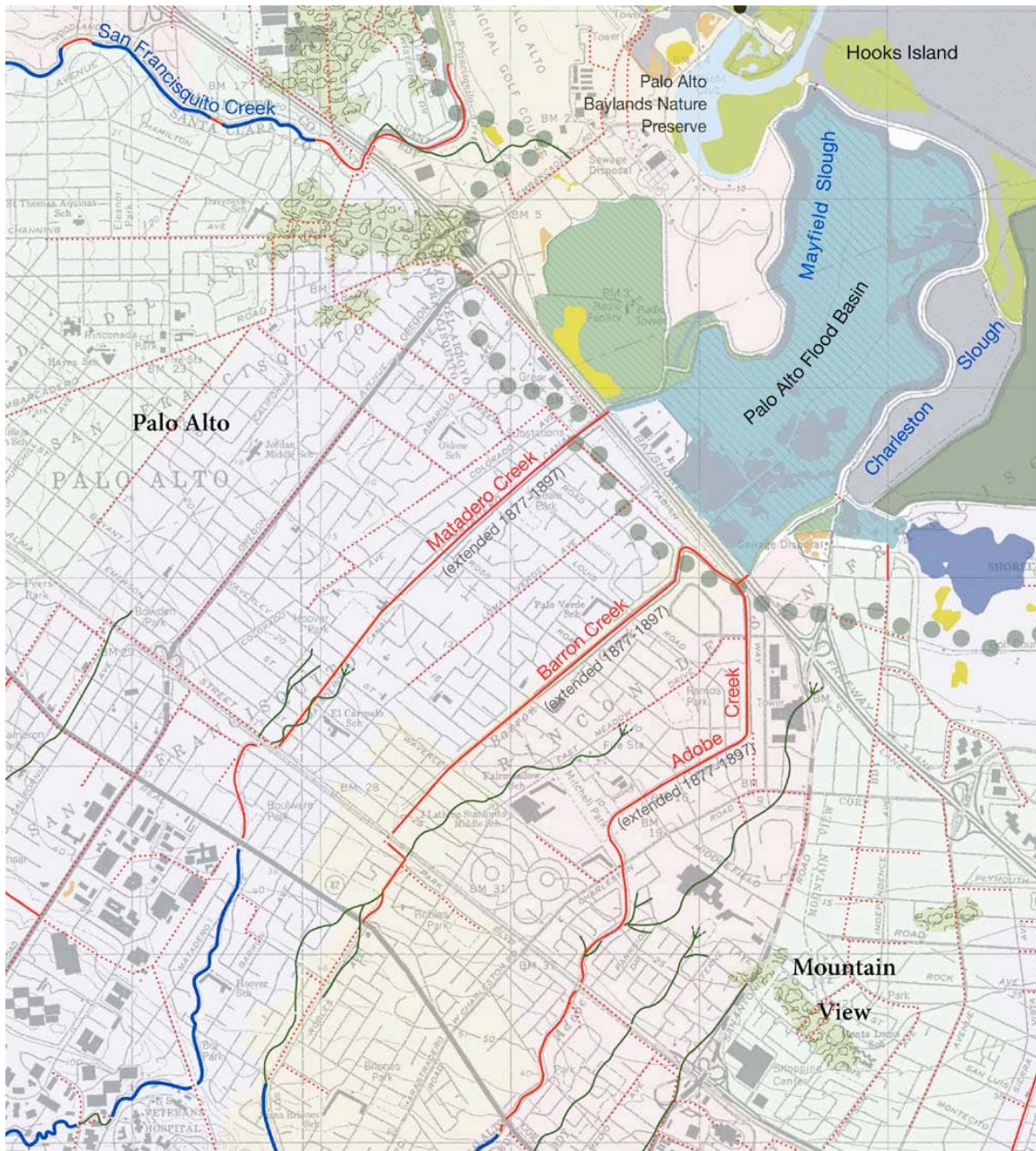


Figure 3-6. Modern Channels of Matadero, Deer, Barron and Adobe Creeks (after Sowers, 2005)

Figure 3-7 shows the Sloughs of the Bay in northern Palo Alto, prior to modern development. These areas are mapped as having very high liquefaction susceptibility.

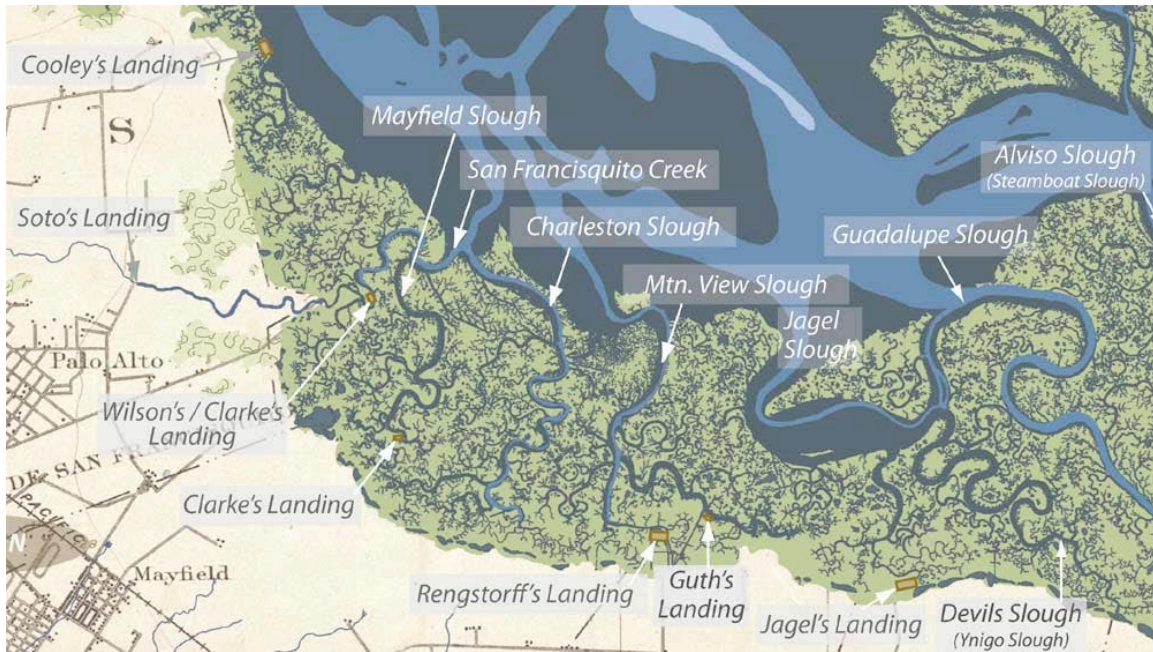


Figure 3-7. San Francisco Bay Shoreline, Circa 1899 (after Sowers, 2005)

3.3 Geotechnical Hazards

There are four primary hazards induced by earthquakes:

- Ground Shaking
- Liquefaction
- Landslide
- Surface faulting

To varying extents, the City of Palo Alto water system is exposed to all of these four hazards. For purposes of the current work, we approach the quantification of these hazards as follows:

- Ground shaking. Ground motion shaking will affect every reservoir, booster pump station, and all of the pipe network. For facilities (reservoirs, pump stations), the ground shaking hazard is best quantified in terms of Peak Ground Acceleration (PGA) with accompanying response spectral shape. For the buried pipe network, the ground shaking hazard is best quantified in terms of Peak Ground Velocity (PGV). Section 3.4 of this report quantifies these.

- Liquefaction. Section 3.5 of this report describes the liquefaction hazard and quantifies how it is treated within context of this report.
- Landslide. Section 3.6 of this report describes the landslide hazard and quantifies how it is treated within context of this report.
- Surface Faulting. Surface faulting is not expected to be a credible hazard to the Palo Alto water system from the San Andreas Fault, but independent or sympathetic movements of the Monta Vista - Shannon and Berrocal Fault Zones pose a hazards to the CCP pipes in the Foothills. Section 3.7 of this report describes the surface faulting hazard and quantifies how it is treated within context of this report.

3.4 Ground Shaking Hazard

For each of the 23 scenario earthquakes listed in Table 3-1, we computed the level of ground shaking at each of Palo Alto's facilities (pump stations, reservoirs, wells), and at each of the pipelines. The approach to calculate the ground motions is as follows:

- Assign a location (latitude / longitude) of each facility or pipeline. For facilities, this location is taken at or near the center of the site. For pipelines, this location is taken at the mid-length location along the digitized pipeline. While actual ground motions will vary spatially, the level of digitization (7,315 locations) is considered adequate for purposes of the current effort.
- For each pipe (or facility) location, estimate the local soil conditions. For modern ground motion attenuation relationships, this is done using an estimated Vs30 value at each location, representing the average shear wave velocity in the top 30 meters of soil. Vs30 ranges from about 760 meters / second (for rock-outcrop sites in the Foothills) to about 400 meters per second (stiffer soils south of Alma) to about 300 meters per second (common soils north of Alma) to as low as about 200 meters per second (in former tidal slough areas).
- Each earthquake fault is located using digitized locations, per the USGS 2008 Quaternary fault database. This includes surface locations of the fault, as well as the dip (slope) of the fault into the earth. Dip of 90° is a vertical fault, like the San Andreas or Hayward faults.
- For each scenario earthquake, the rupture (length, depth) of the fault plane is computed, based on the location of the epicenter, the magnitude, and the geometry of the fault.
- The closest distance from each facility to the ruptured fault plane is computed.

- We compute the horizontal ground motion at each site / pipe. This is done five times, for five different GMPE models: Abrahamson et al (2013), Boore et al (2013), Campbell et al (2013), Chiou et al (2013) and Idriss (2013). Depending on the GMPE model, the horizontal spectra is computed at between 50 and 125 different periods, including PGA and PGV.
- We take the average of the five sets of motion at each site / pipe. For the evaluation of buried pipe, we use PGV to estimate the damage due to ground shaking, and PGA and magnitude and local soil conditions to estimate the potential for triggering liquefaction, and if liquefaction occurs, the amount of settlements or lateral spreads.
- For each site / pipe, we compute the median (best estimate) and 84th percentile ground motions (near upper bound), for each scenario earthquake.

Appendix D provides maps that show the level of shaking in Palo Alto for each of the 23 scenario earthquakes. Two of these maps are included in the main body of the report, to help illustrate the levels of motion. Figures 3-8 and 3-9 show the likely upper bound levels of motion in Palo Alto due to a repeat of the 1989 Loma Prieta earthquake, and a future M 7.0 earthquake on the nearby San Andreas fault. The key points are as follows:

- These maps show the average of two horizontal directions for PGA.
- In the 1989 earthquake (Figure 3-8), the PGA values are estimated to be in the range of 0.15g to 0.20g through most of Palo Alto. The average motions would have been about 50% to 67% of these values. Section 3.7.2 reports the actual recorded ground motions in and near Palo Alto in the 1989 earthquake: about PGA = 0.08g (hillside areas); 0.20g in the basement of a 2 story office building in the flatland area; with several other recordings in this range.
- In a future San Andreas M 7.0 earthquake that ruptures the nearby Peninsula segment, (Figure 3-9), the PGA values will be over 0.70g in the Foothills areas (some locations as high as PGA = 1.0g), reducing to about 0.40 to 0.55g in most of the areas northeast of Alma (flatlands). The average motions would have been about 50% to 67% of these values.

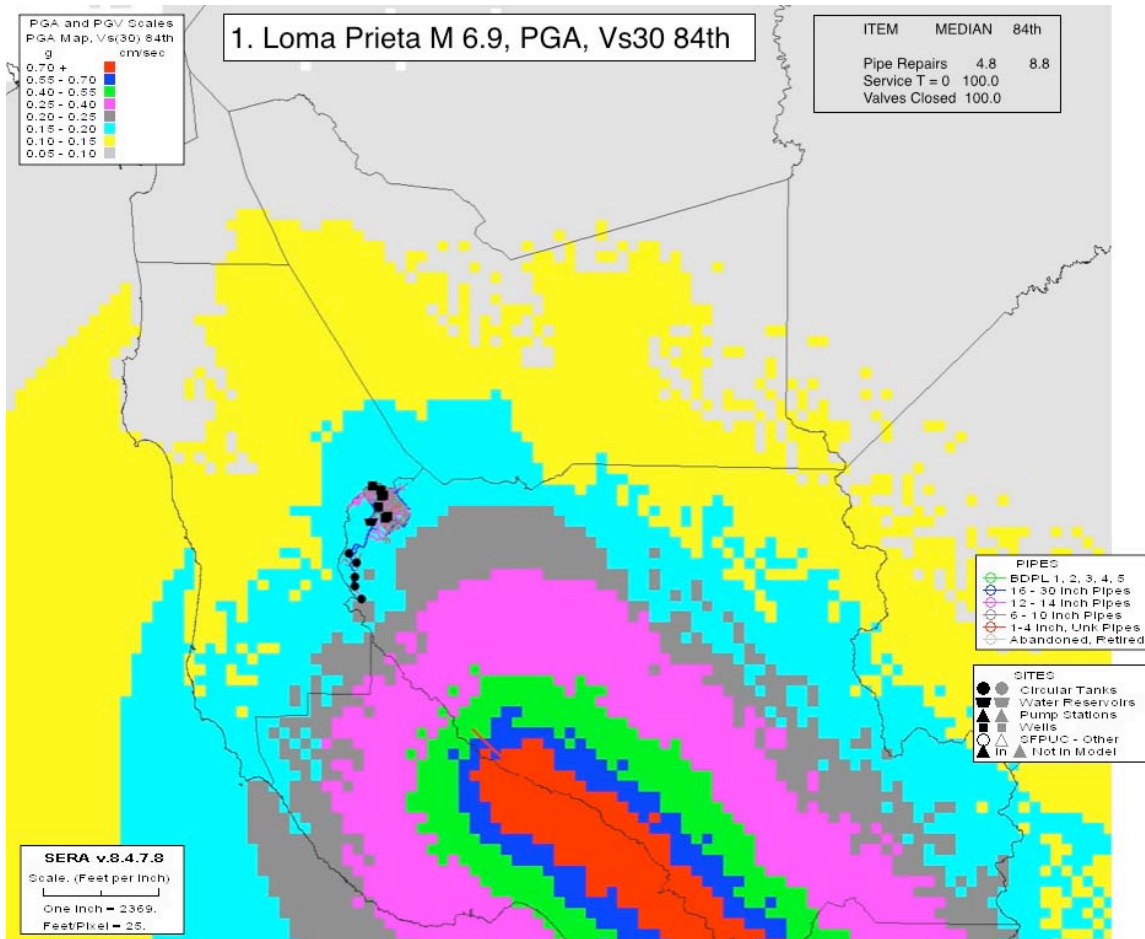


Figure 3-8. Ground Motions in Palo Alto, 1989 Loma Prieta Earthquake (84th)

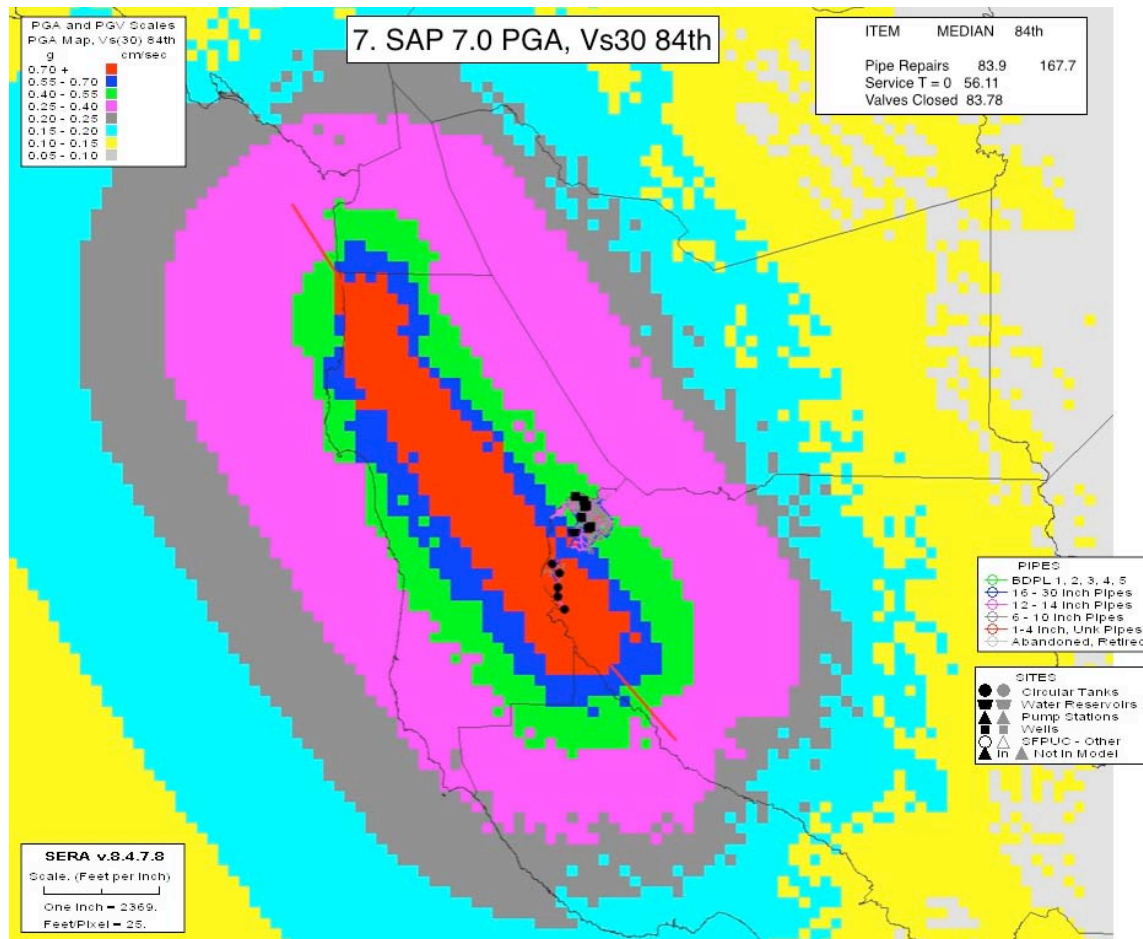


Figure 3-9. Ground Motions in Palo Alto, Future San Andreas M 7.0 Earthquake (84th)

3.5 Liquefaction Hazard

The performance of the City of Palo Alto water system after large earthquakes will be greatly influenced by what happens to the local soils during the earthquake.

Figure 3-10 shows a map of the liquefaction susceptibility in Palo Alto. Figure 3-11 shows the same map at a larger scale for the flatland portion of Palo Alto. Figure 3-12 shows the same map at a larger scale for the Foothills portion of Palo Alto. Figures 3-10 to 3-12 are based on liquefaction mapping by the UGS (Witter et al, 2006).

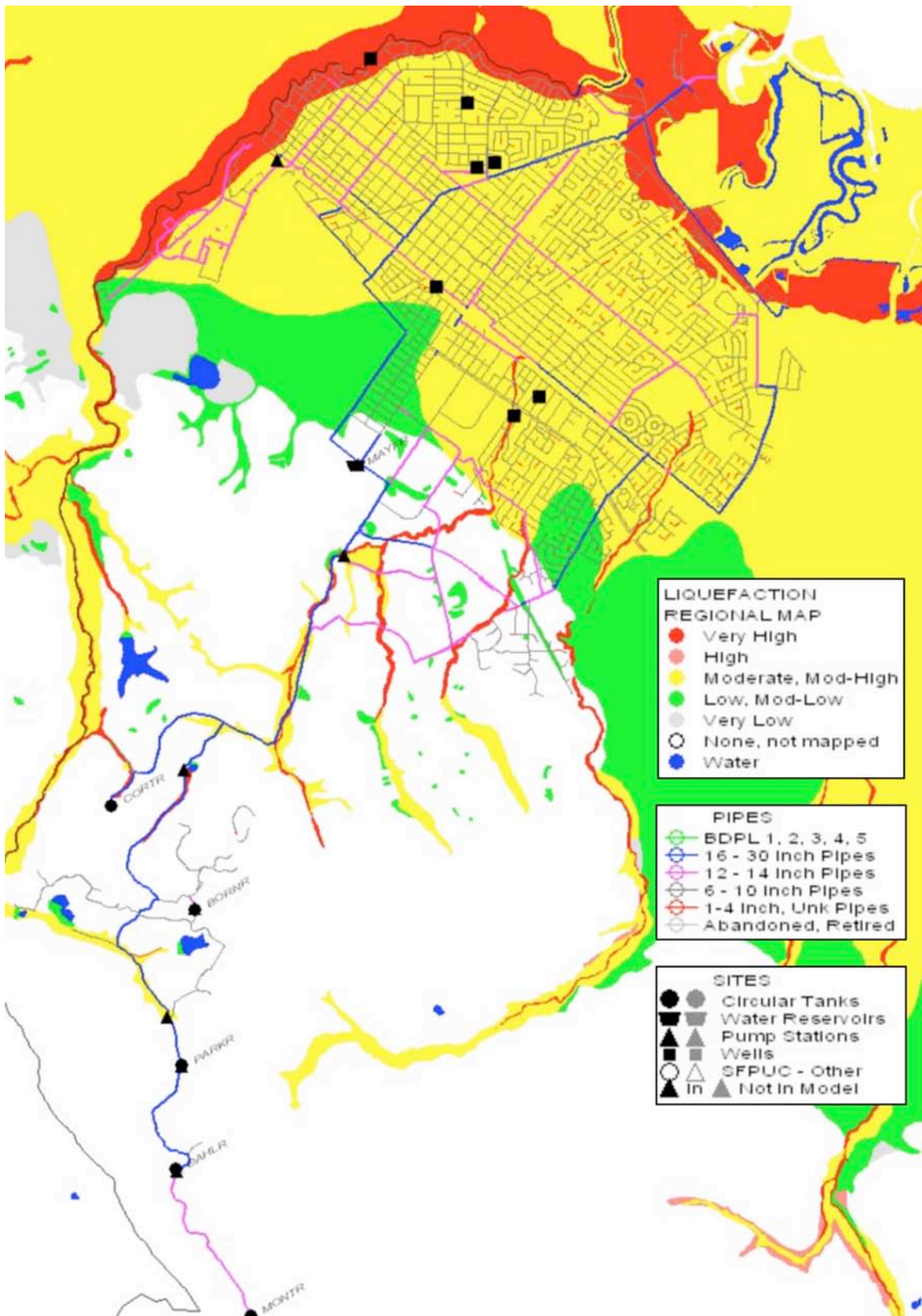


Figure 3-10. Liquefaction Susceptibility in Palo Alto

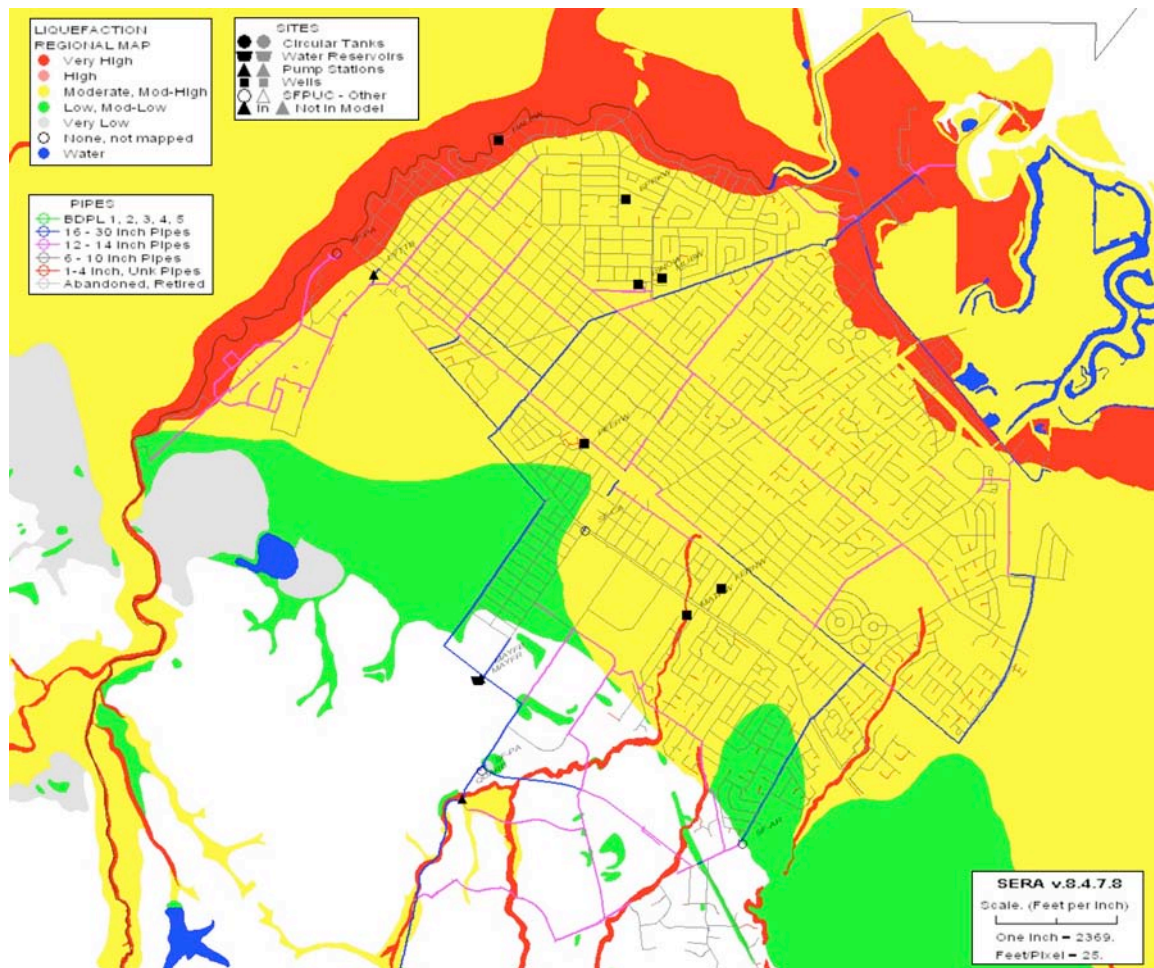


Figure 3-11. Liquefaction Susceptibility in the Lower Elevation Areas of Palo Alto

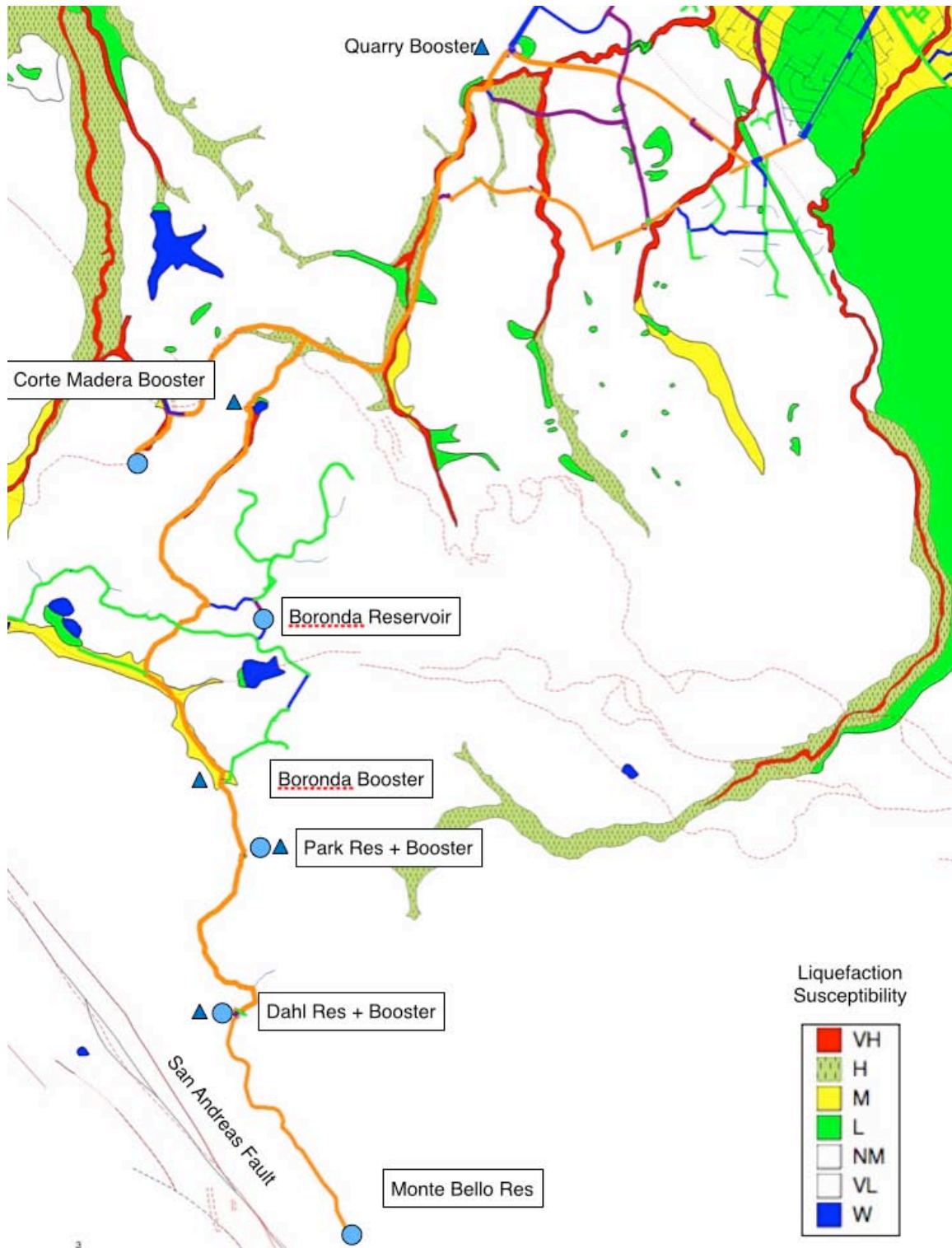


Figure 3-12. Liquefaction Zones, Foothills Area

Figure 3-13 shows a map of the Foothills area that highlights zones the Palo Alto pipelines, as well as areas prone to possible liquefaction or landslide effects. The underlying data for the liquefaction and landslide zones in Figure 3-13 is from CGS Quad sheets for the Palo Alto and Mindego Hill Quad sheets (CGS, 2005, 2006).

It is well understood that the maps provided by CGS (and used in Figure 3-13) are primarily meant to be exclusion zones, and not zones with confirmed liquefaction and landslide hazards. More important to the City of Palo Alto is the underlying data used in developing these maps. In this report, we will describe this information as it pertains to the CCP Foothills pipeline.

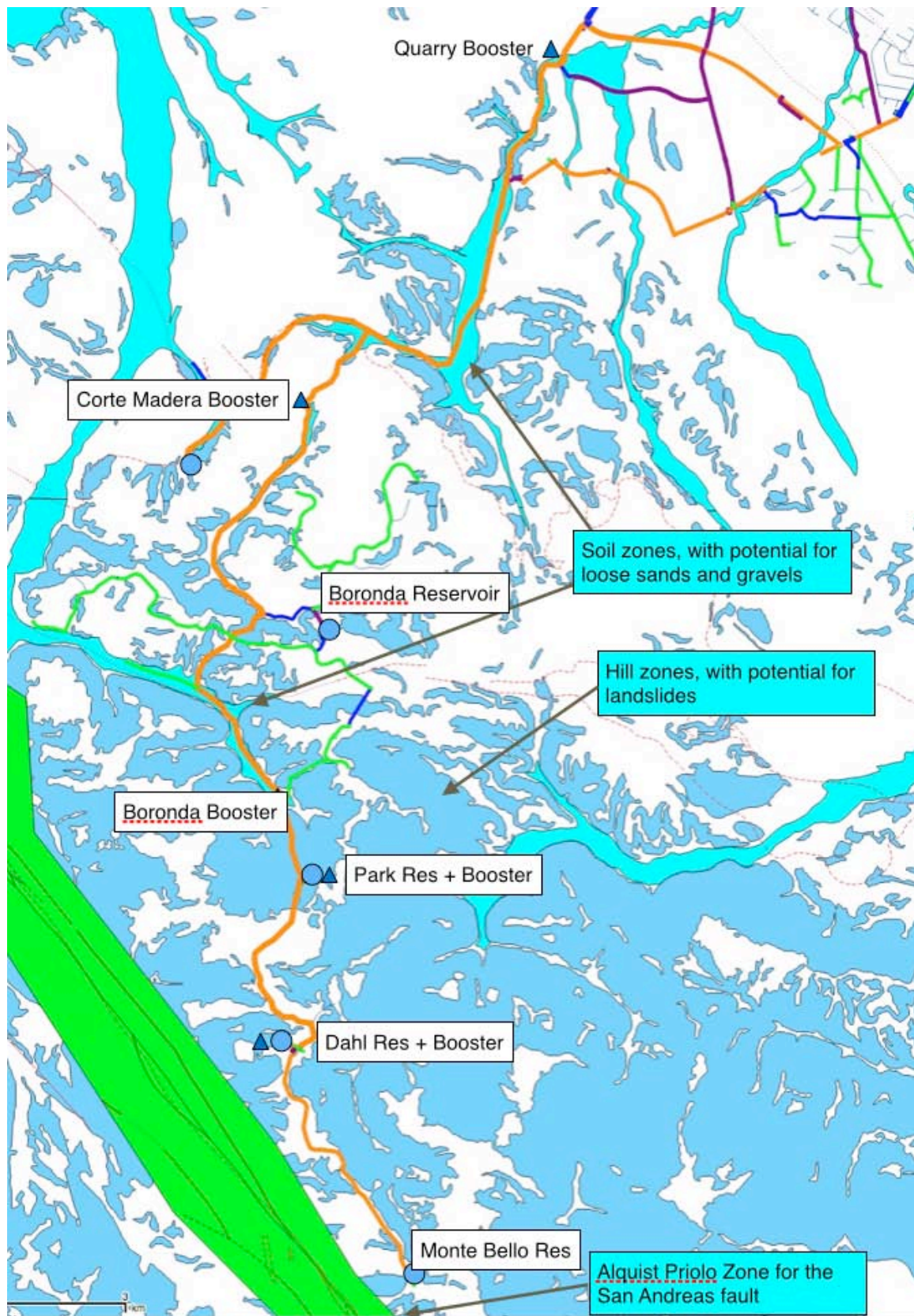


Figure 3-13. Liquefaction and Landslide Zones, Foothills Area

The scale of the map in Figures 3-12 and 3-13 is too large to enable clear understanding of the hazard zones relative to the Palo Alto pipelines in the Foothills. Figures 3-14 to 3-31 provide higher resolution maps, using the same color-coding as Figures 3-12 and 3-13. In Figures 3-18, 3-20 and 3-22, the slopes of the pipe (slope = $\Delta Y / \Delta X$, with 100% slope being the same as 45 degrees, + meaning going uphill, - meaning going downhill) are provided. Section 3.5 provides further discussion of slope.

Figures 3-14, 3-15 and 3-16 show the pipeline alignment south of Quarry Booster pump station. The aerial photo clearly shows the trees and brush that grows near the creek alignments, owing to the presence of water. The pipe follows Old Page Mill Road and Matadero Creek. At several locations, the pipe crosses under the creek, using the detail in Figure 2-30. The concrete encasement for the pipe under the creek was used for scour protection; this scour protection is failing at some locations. We were concerned about liquefaction in the soils near or under the creek, and this can cause settlements under the creek, as well as lateral spreads along the creek embankments. The highest risk to the CCP pipe is a lateral spread along the embankment that drags the pipe towards the creek centerline. These issues are further discussed in Section 5.



Figure 3-14. Quarry Booster Pump Station Area – Pipes and Liquefaction / Slide Hazard Zones

The unavailability of the soil boring data along the pipeline makes it speculative as to whether or not the pipe is prone to a lateral spread in this zone. Witter's mapping is largely based on geologic issues in this area, so the "red" zone along Matadero Creek must be considered speculative. For example, the boring data for the Quarry Booster

pump station is not considered to have "high" or "very high" susceptibility. As the pipe is buried about 4 feet, the slope instability into the creek centerline would have to reach as deep as the pipe in order to impact the pipe; damage to air valves and blow offs and branch connections would be higher risk than to the main barrel of the pipe.

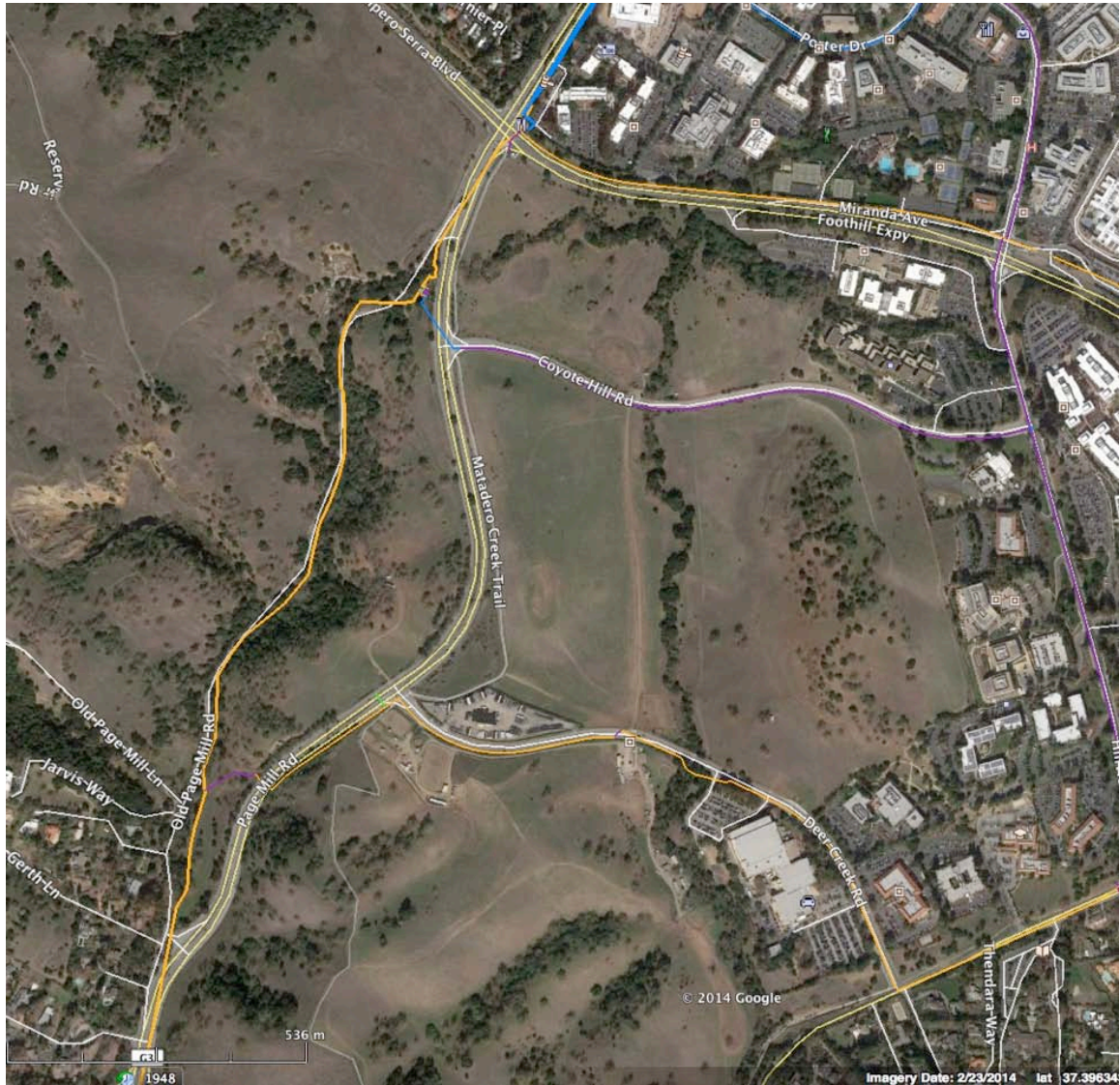


Figure 3-15. Quarry Booster Pump Station Area – Aerial

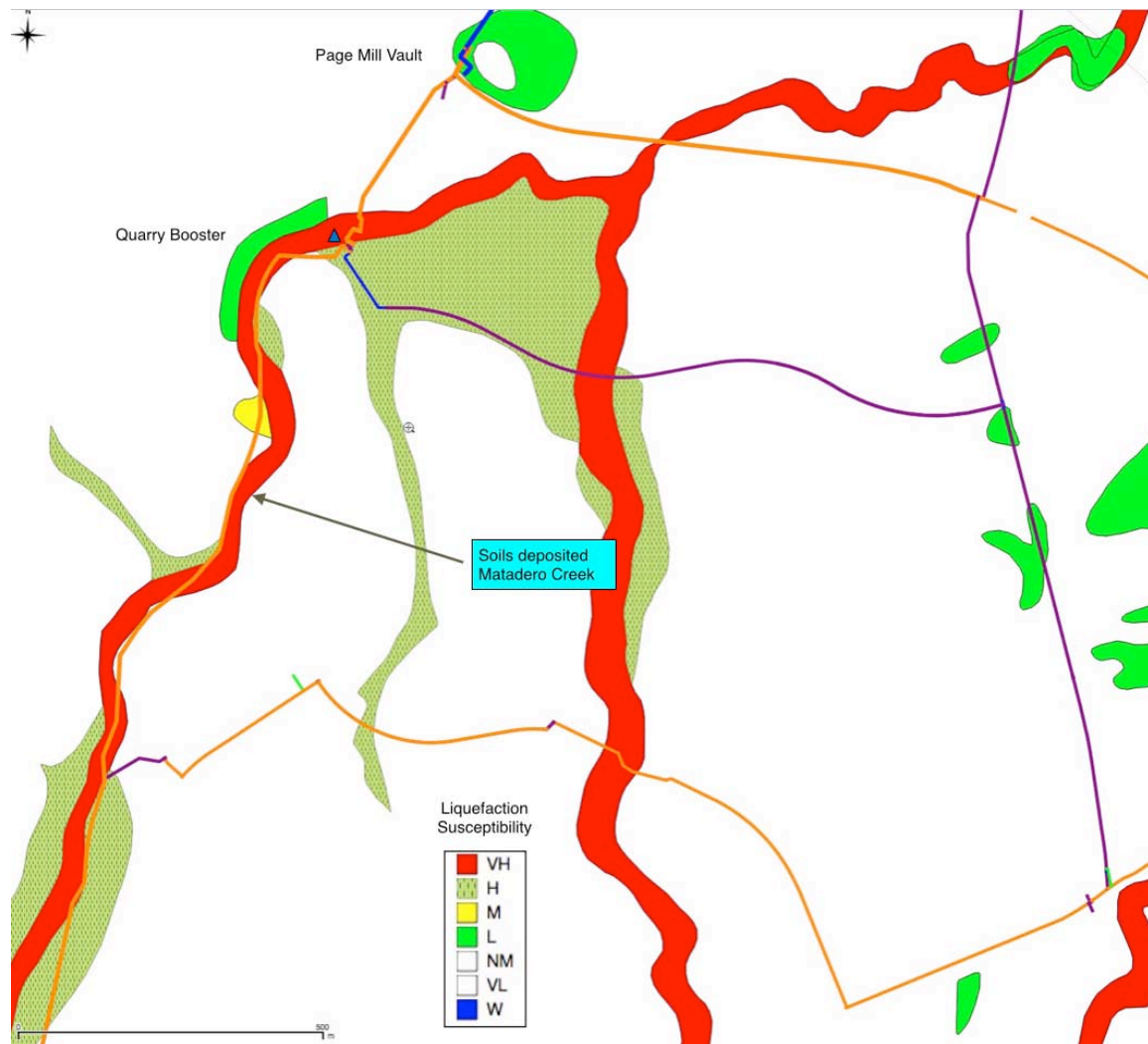


Figure 3-16. Quarry Booster Pump Station Area – Pipes and Liquefaction Hazard Zones. Full scale is 500 meters

Figures 3-17, 3-18 and 3-19 show the alignment near Corte Madera Booster pump station. The 18" CCP passes under Highway 280, where the pipe was relocated in 1965. We do not have drawings for the relocated pipe, but we speculate that Caltrans would have required the pipe to be placed either in a larger diameter pipe, or heavily encased in concrete, both to support the heavy highway loads as well as to protect the pipe. We infer that Caltrans did a good job, and that there are no settlement issues in this zone. Drawing review of the Caltrans 1965-vintage relocation would be required to verify this assumption.



Figure 3-17. Corte Madera Booster Pump Station Area – Pipes and Hazard Zones

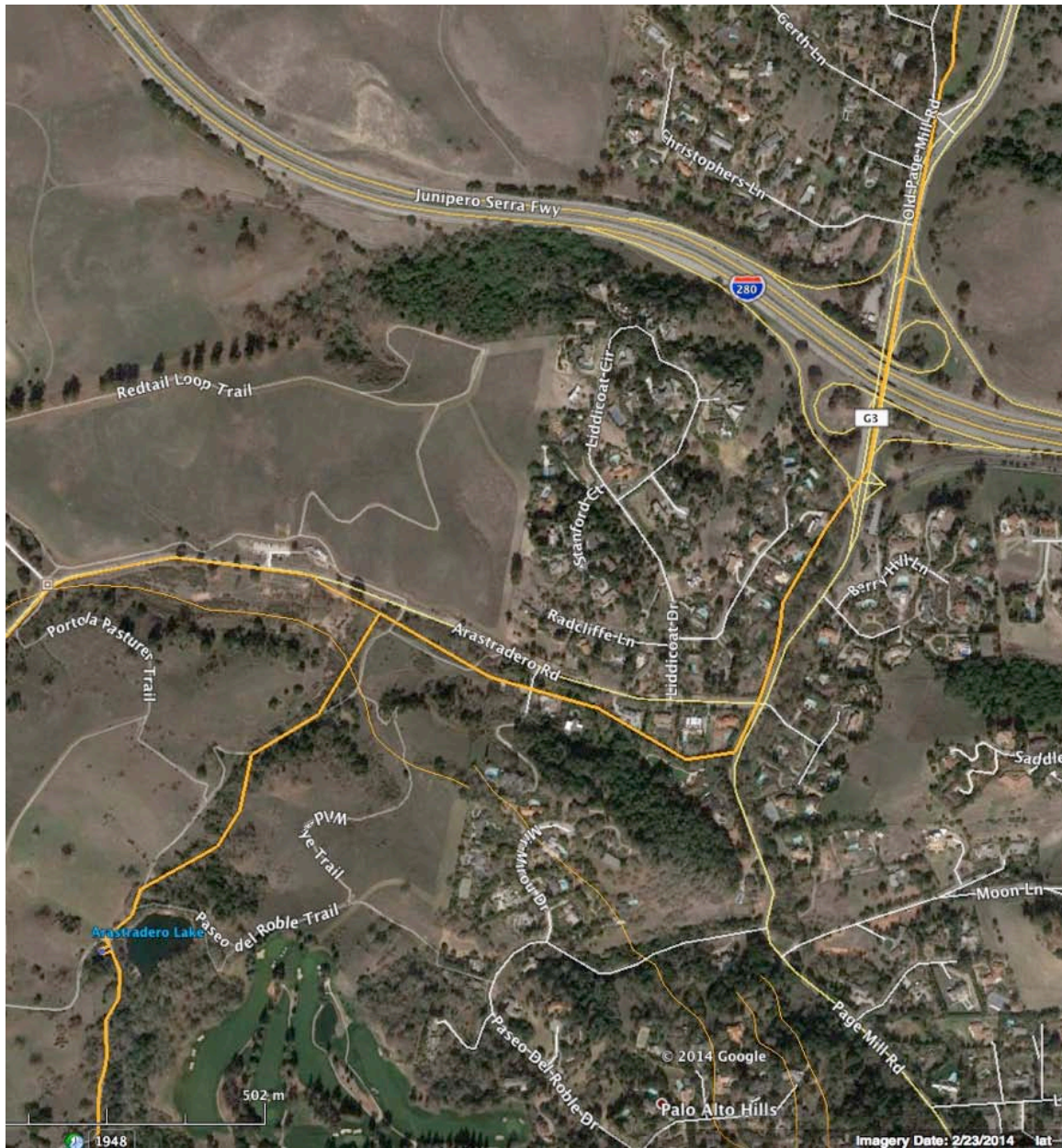


Figure 3-18. Corte Madera Booster Pump Station Area – Aerial

Figure 3-18 shows the liquefaction and fault crossing zones in this area. The interpretation of the "High" and "Very High" zones near the CCP pipe is discussed in Section 5. The potential for sympathetic faulting across the Monta Vista - Shannon Fault Zone cannot be discounted, given a San Andreas M 8.0 event, and this is addressed in Section 3.7.

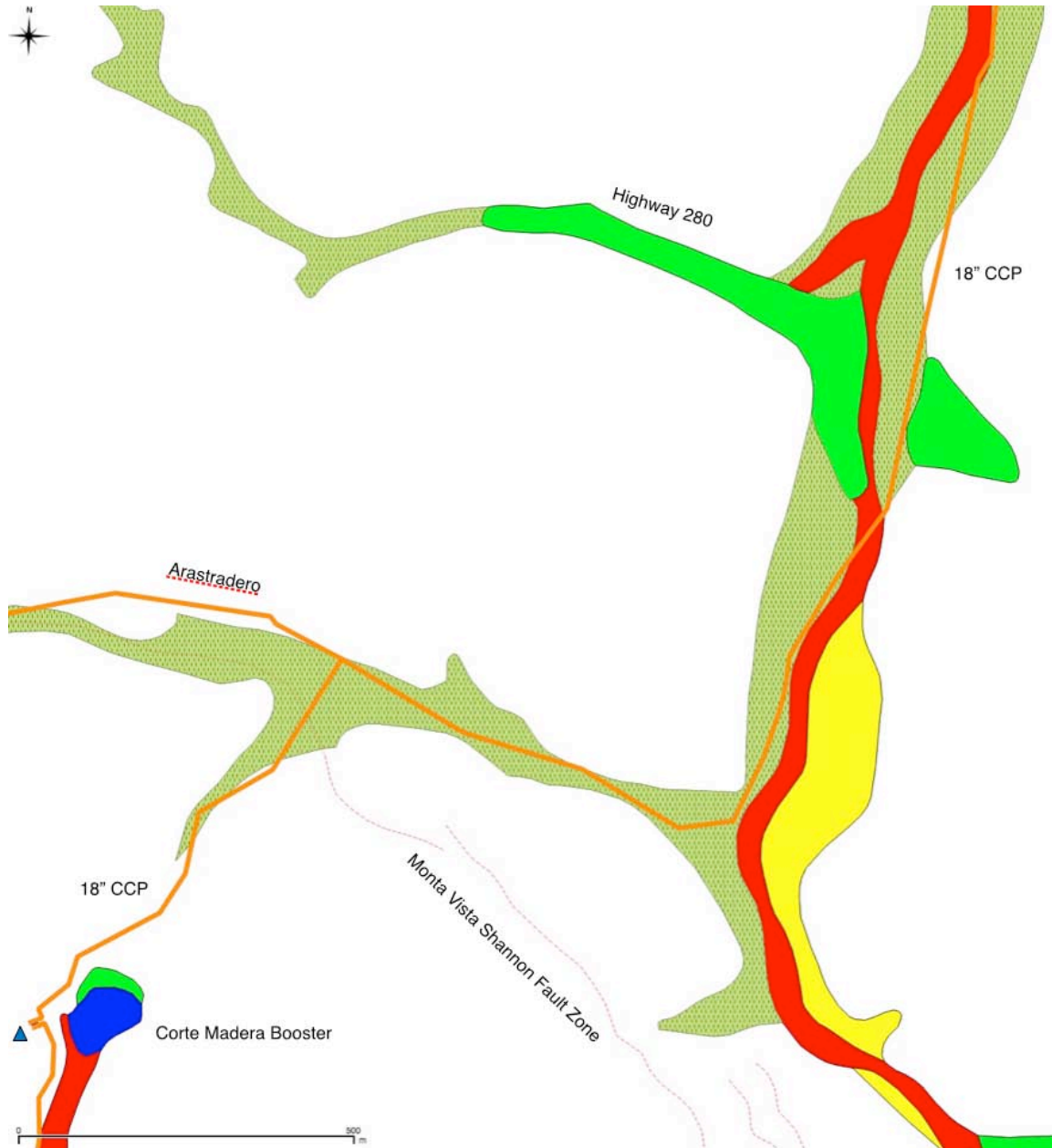


Figure 3-19. Corte Madera Booster Pump Station Area – Pipes, Liquefaction and Fault Zones

Figures 3-20, 3-21 and 3-22 show the pipe alignment from the Corte Madera Booster Pump Station to the Boronda Reservoir. This pipeline is the original 1962 18" CCP. The color-coding (blue) from the GIS for the pipe near the Boronda Reservoir is incorrect.



Figure 3-20. Corte Madera to Boronda Reservoir Area – Pipes and Hazard Zones



Figure 3-21. Corte Madera to Boronda Reservoir Area – Aerial

Figure 3-22 shows the liquefaction zones in this area. In February 2015, we did a field walkdown of the portion of the 18" CCP pipe in and next to the red zone. Section 5 describes the field observations.

The interpretation of the "Very High" zones adjacent to the 18" CCP is addressed in Section 5. From the pipe drawings, it appears that this creek is only a few feet deep. For most of the length near the red zone, the pipe is on the west side of the creek. The primary hazard is that the pipe, buried in a slope above the creek, could be dragged towards the creek due to a lateral spread / landslide. This is further discussed in Section 5.

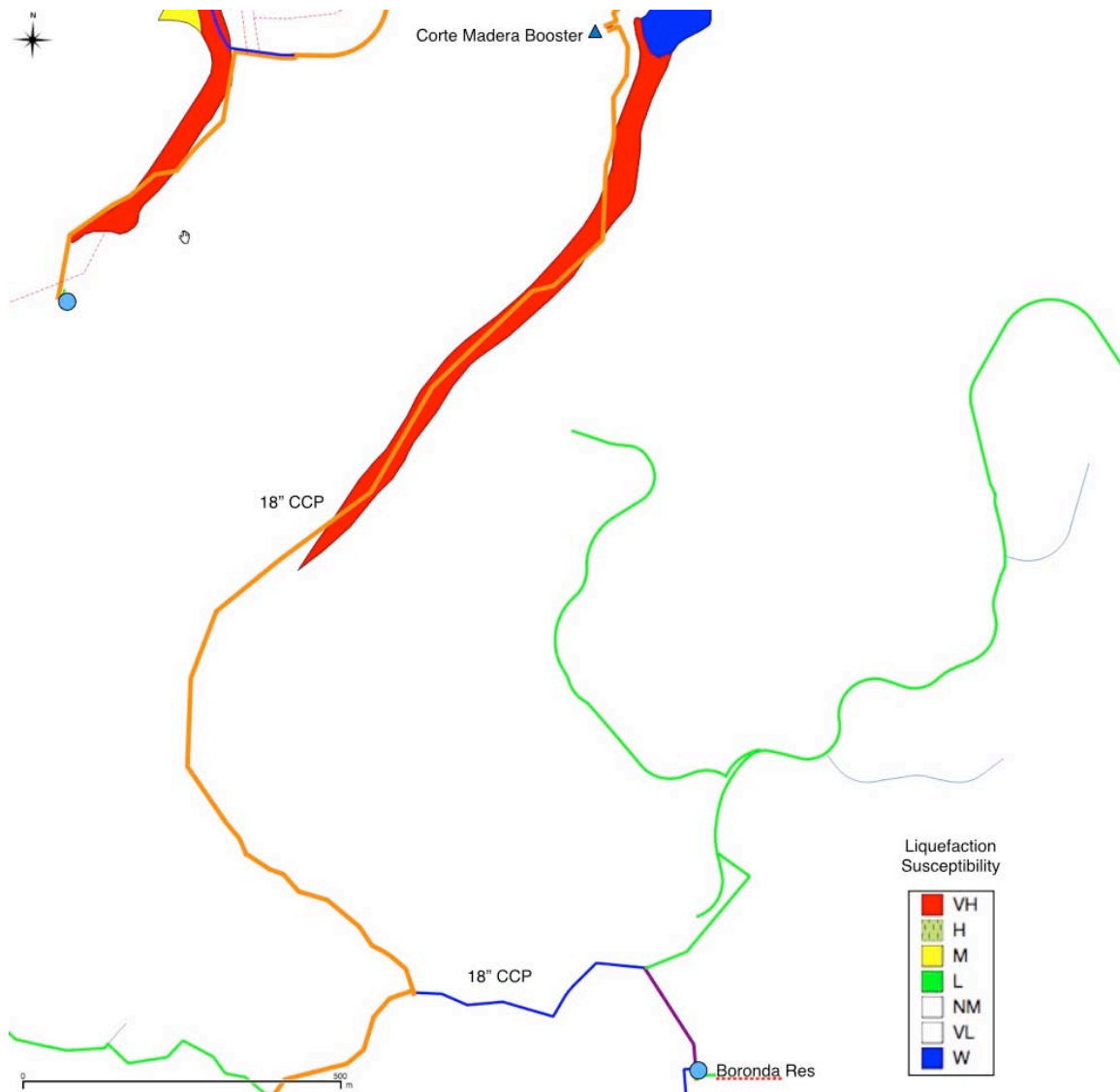


Figure 3-22. Corte Madera to Boronda Reservoir Area – Pipes and Liquefaction Hazard Zones

Figures 3-23, 3-24 and 3-25 show the pipe alignment from Boronda Reservoir to the Boronda Booster pump station. This pipeline is the original 1962 18" CCP (the blue line portion is incorrectly coded in the GIS).

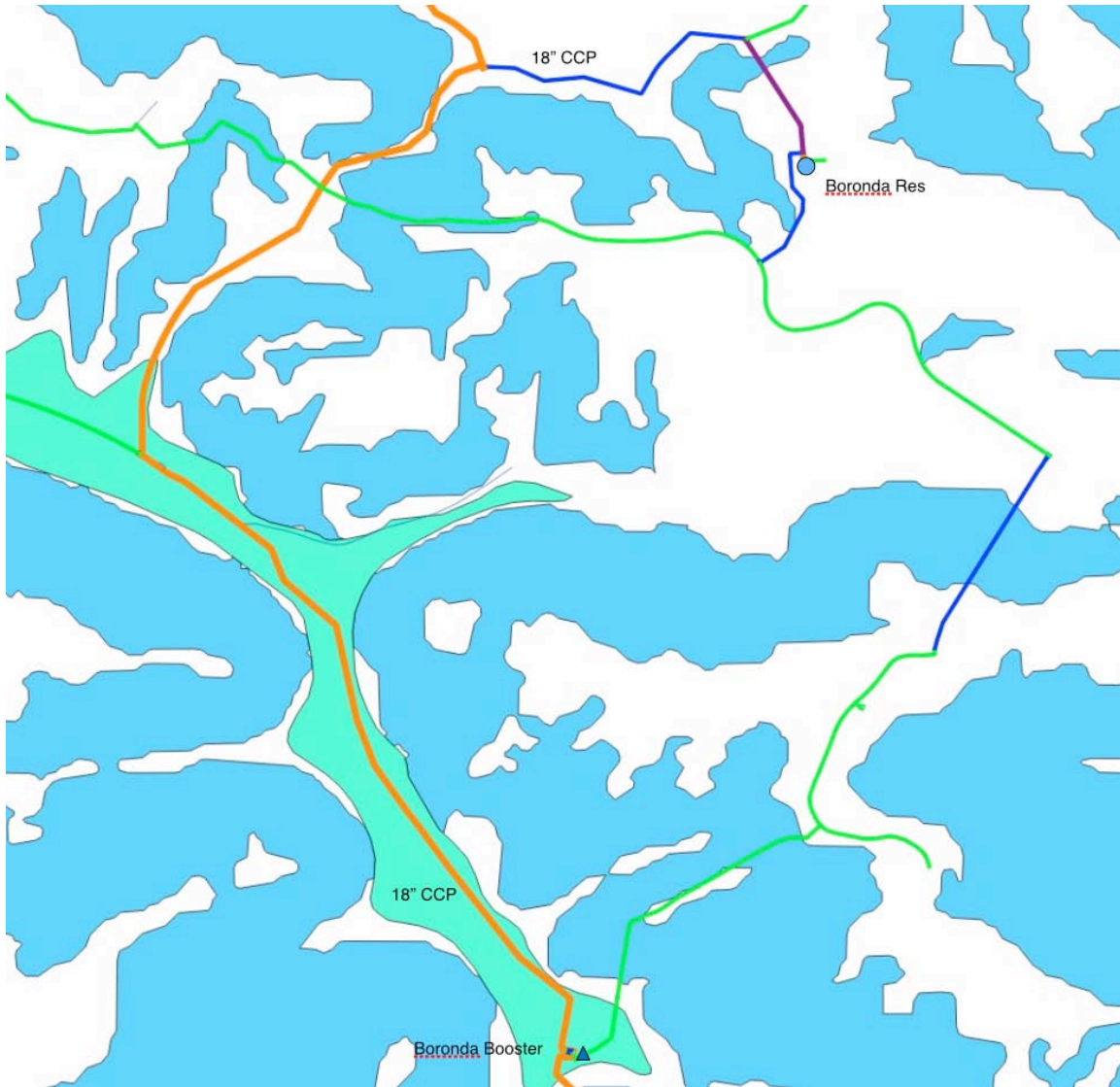


Figure 3-23. Boronda Reservoir to Boronda Booster Area – Pipes and Hazard Zones

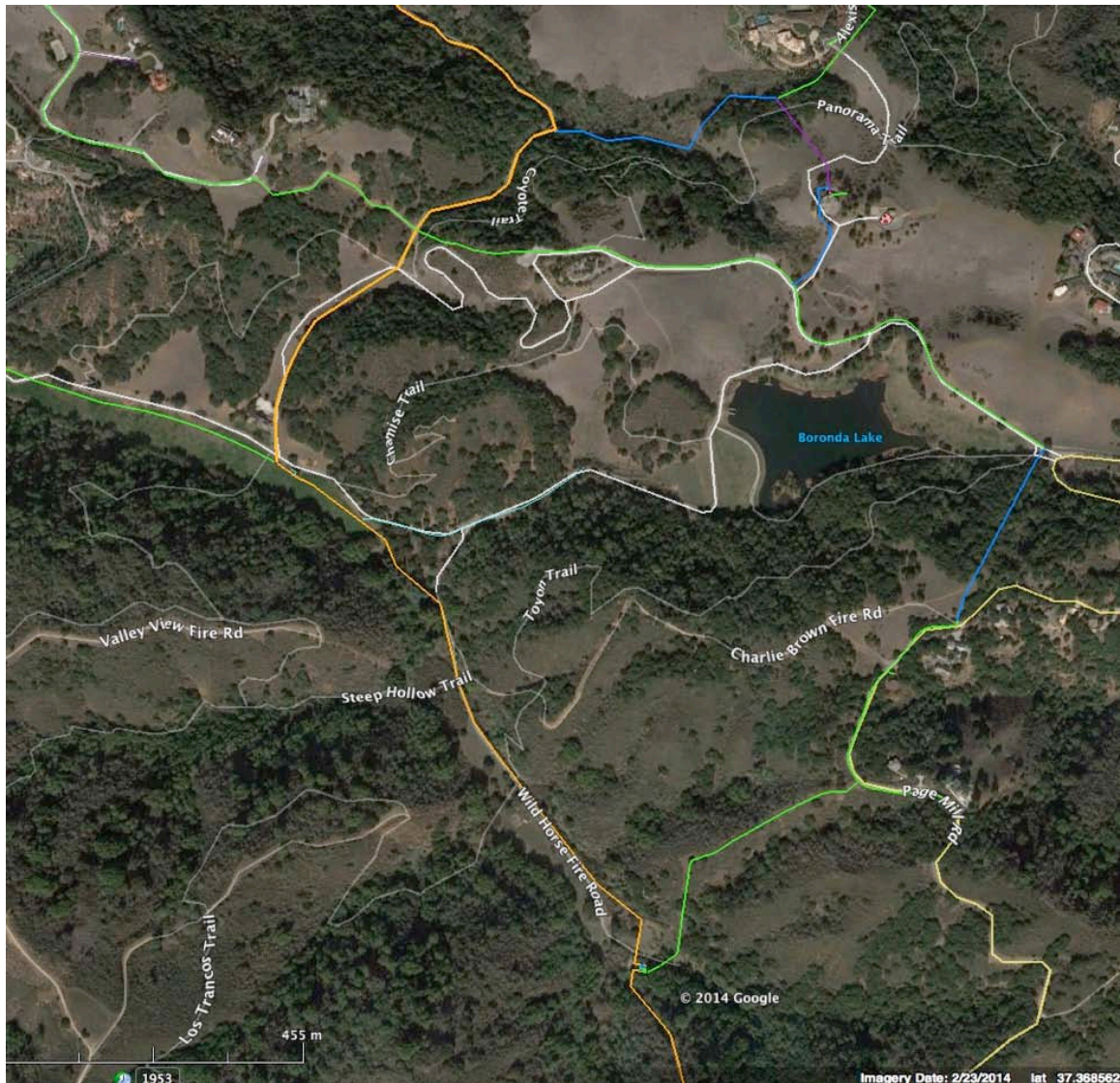


Figure 3-24. Boronda Reservoir to Boronda Booster Area – Aerial

The 18" CCP traverses the Berrocal Fault Zone, as indicated in Figure 3-25.

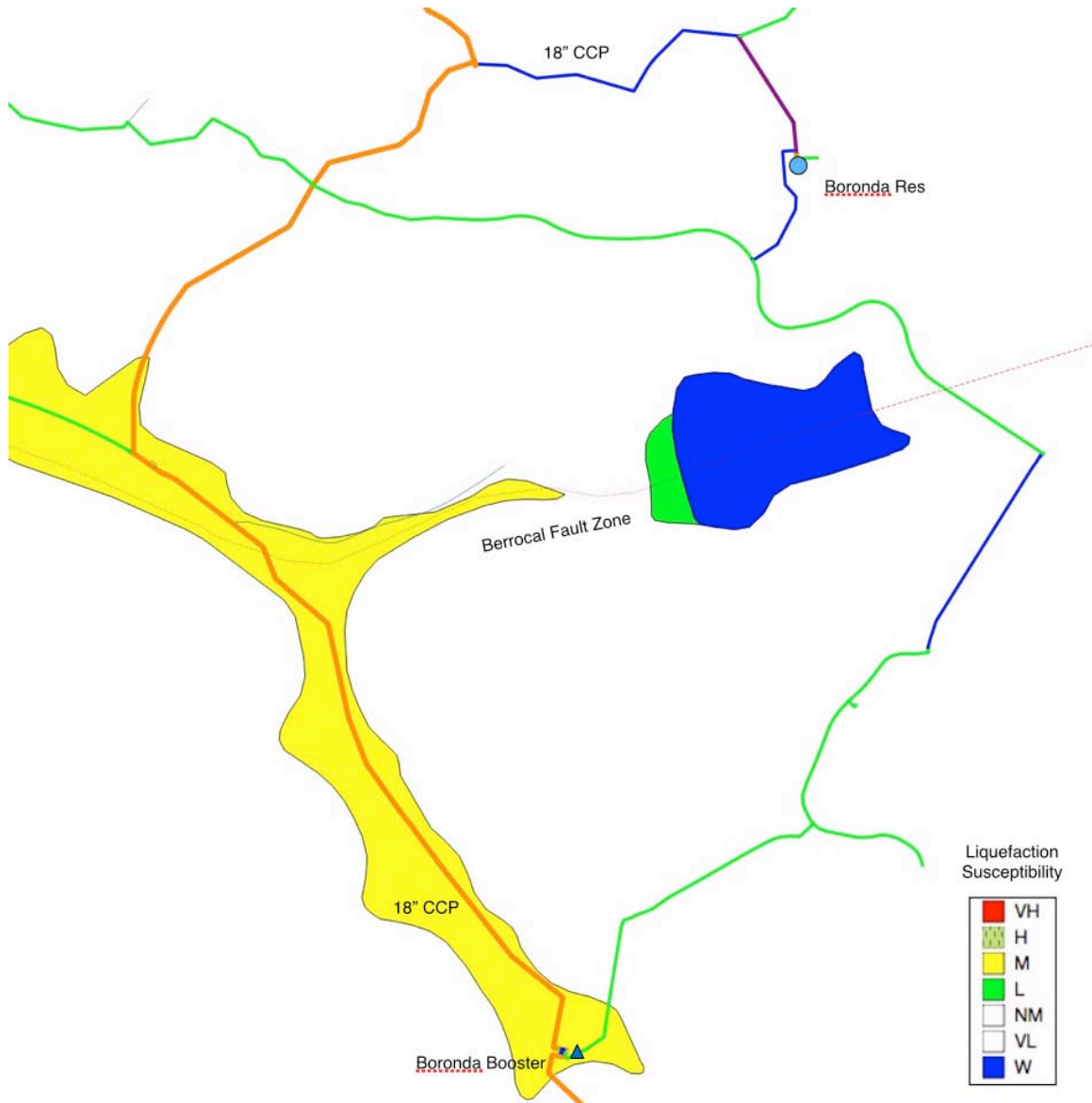


Figure 3-25. Boronda Reservoir to Boronda Booster Area – Pipes, Liquefaction and Fault Hazard Zones

Figures 3-26 and 3-27 show the pipeline from the Boronda Booster to the Park Reservoir. Just south of the Boronda Booster, the pipe goes up a steep slope (41%). The Park Reservoir and Booster Pump Station are located on a small plateau atop this hill.

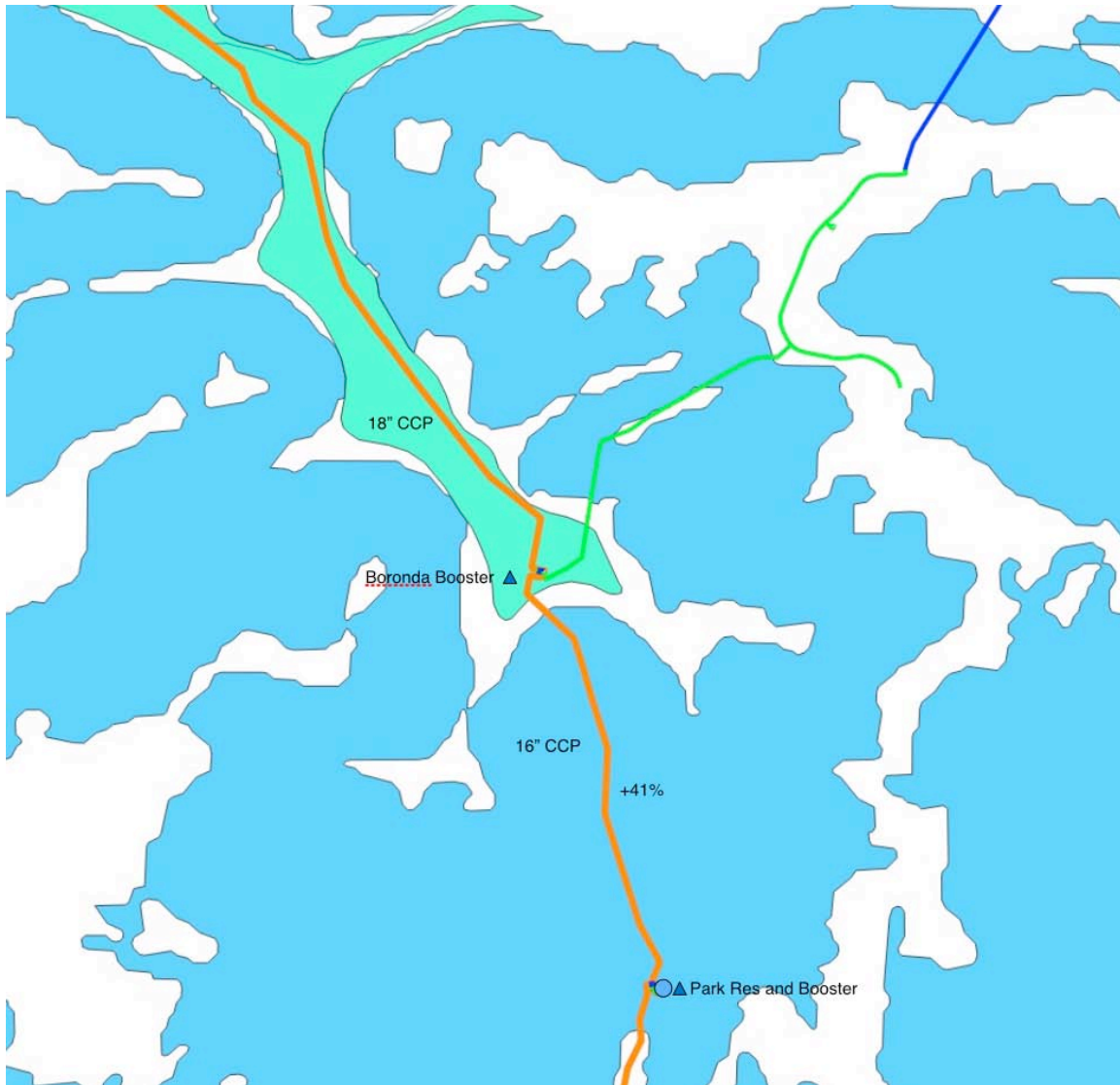


Figure 3-26. Boronda Booster to Park Reservoir Area – Pipes and Hazard Zones

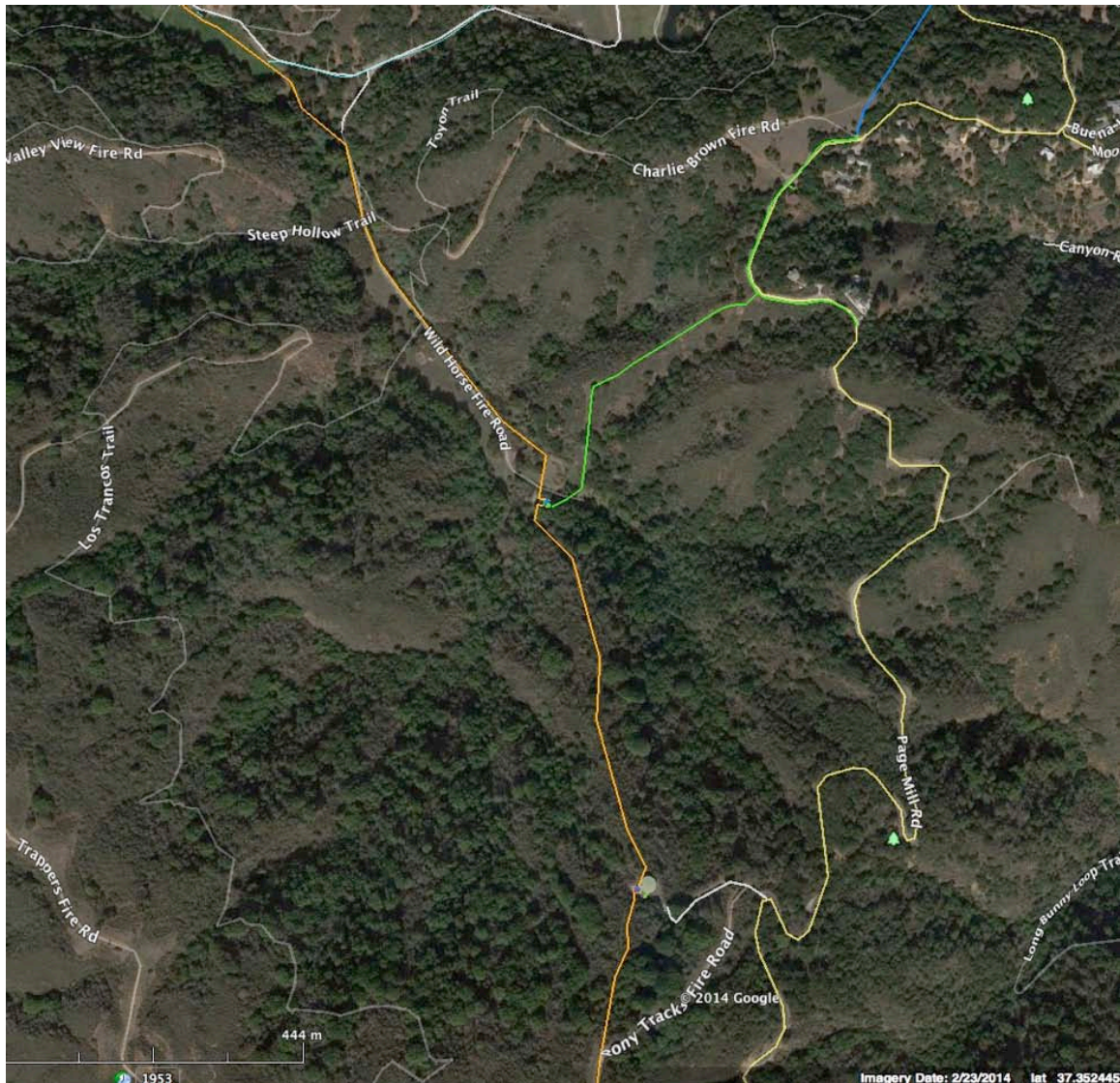


Figure 3-27. Boronda Booster to Park Reservoir Area – Aerial

Figures 3-28 and 3-29 show the pipeline from the Park Reservoir to the Dahl Reservoir. Just south of the Park Reservoir, the pipe goes up a steep slope (45%).



Figure 3-28. Park Reservoir to Dahl Reservoir Area – Pipes and Hazard Zones

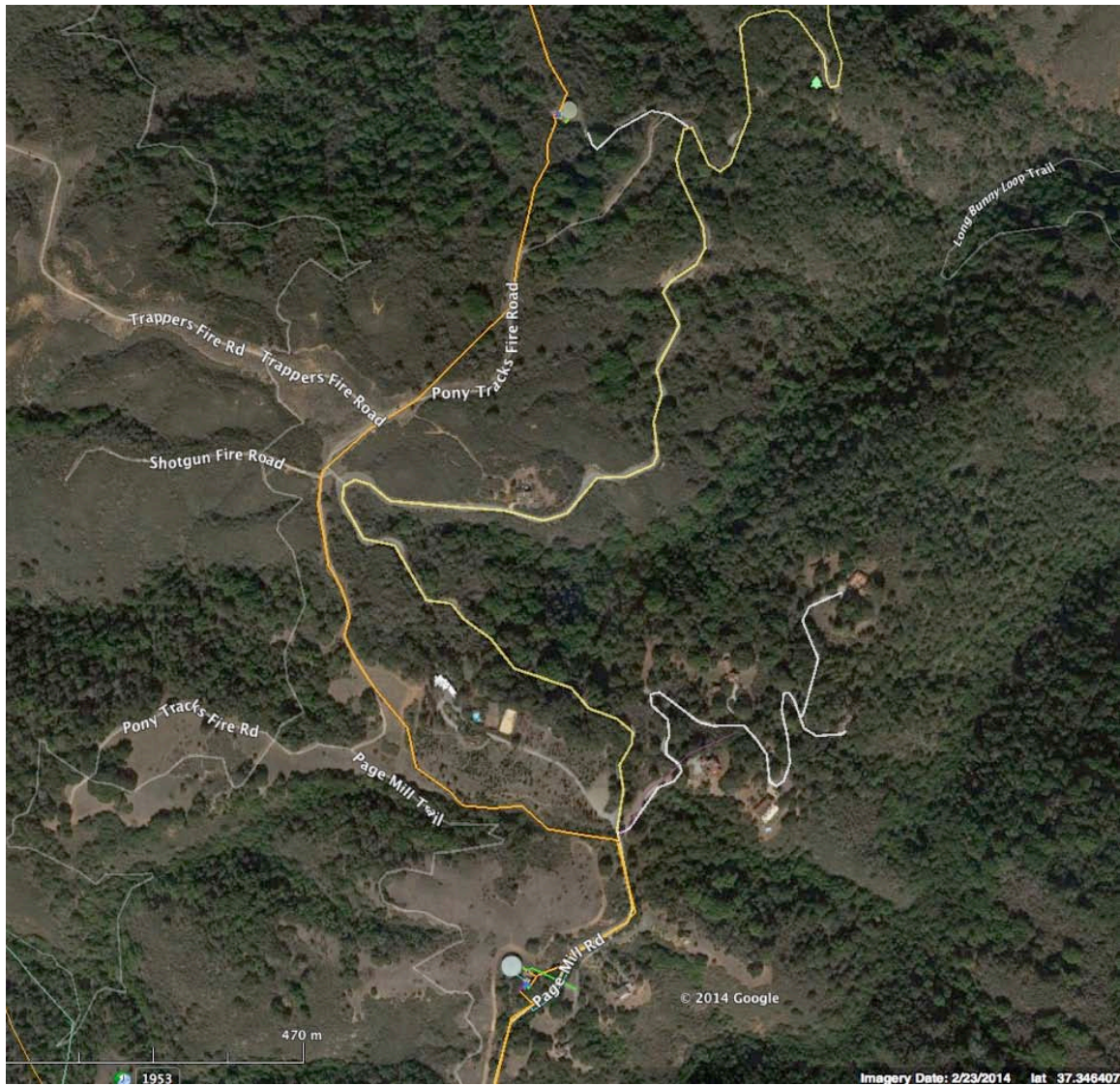


Figure 3-29. Park Reservoir to Dahl Reservoir Area – Aerial

Figures 3-30 and 3-31 show the pipeline from the Dahl Reservoir to the Monte Bello Reservoir. In February 2015, we did a field walkdown of this section of pipe between Dahl reservoir and Monte Bello reservoir. Section 5 describes the field observations.



Figure 3-30. Dahl Reservoir to Monte Bello Reservoir Area – Pipes and Hazard Zones



Figure 3-31. Dahl Reservoir to Monte Bello Reservoir Area – Aerial

Given these issues, the available information suggests that liquefaction risk for the pipeline between the Quarry Booster pump station and the Monte Bello reservoir is variable. If we assume that the entire alignment is buried in stiff soils, then the chance of a lateral spread into Matadero Creek is low.

Based on the few borings currently available, those soils appear to not be susceptible to lateral spread. Therefore, we make the following assessment for this report:

- Between Quarry and Corte Madera Booster pump stations: We assign the chance of pipe break at 10% and the chance at pipe leak at 10%. The weaknesses are primarily concentrated where the pipe crosses under Matadero Creek along Old Page Mill Road.
- Between Corte Madera Booster pump station and Boronda Reservoir: We assign the chance of pipe break at 5% and the chance at pipe leak at 5%. The primary risk is a slope failure that drags the pipe downhill and into the creek; with the more likely damage points being at blow off connections.

While the liquefaction hazard in the Foothills area might be modest, the selection of a push-on gasketed transmission pipe would not correspond to modern seismic design approaches outlined in ALA 2005. For future pipe installations, we recommend that all new pipes that are installed through stream / drainage locations be designed to accommodate the effects of liquefaction, per ALA (2005). For the Foothills area, this could be accomplished using a seismically-designed, continuously-welded steel pipeline, or a chained pipeline, as outlined in ALA 2005.

3.6 Landslide Hazard

There are several types of ground movements commonly called "landslides". Figure 3-32 show schematics of the common names given to different types of landslides.

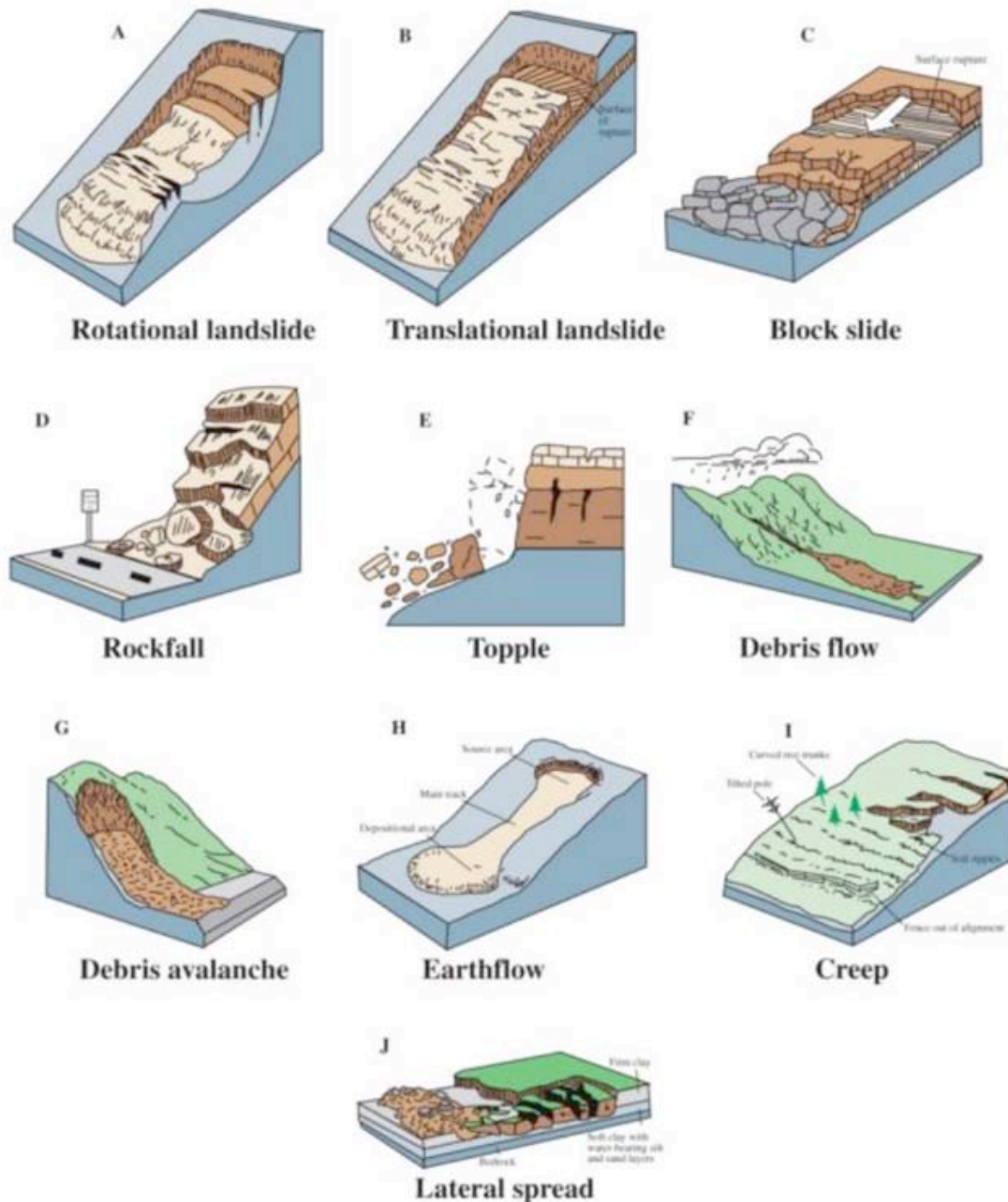


Figure 3-32. Types of Landslides with Common Names

One of the more damaging types of landslides to buried pipelines is a deep-seated rotational slump, Figure 3-33.

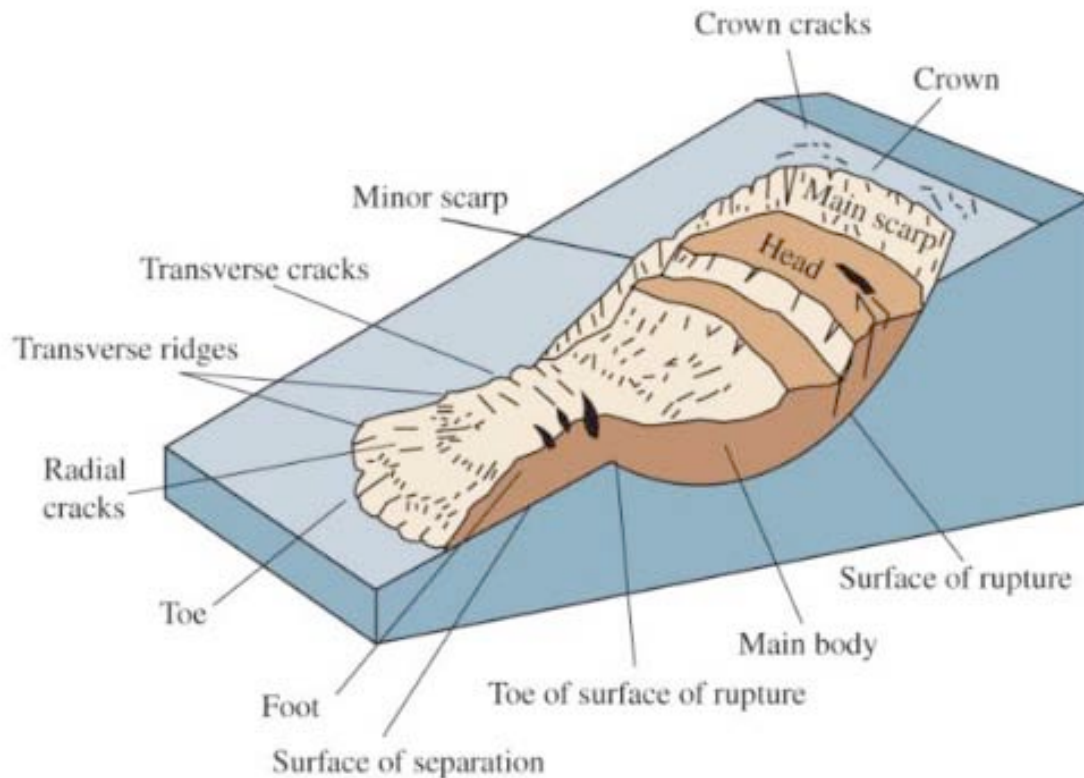


Figure 3-33. Rotational Landslide

Any CCP that traverses such a slide cannot take much more than a couple of inches of movement, before breaking.

The blue-mapped areas in Figures 3-13, 3-14, 3-17, 3-20, 3-23, 3-26, 3-28, and 3-30 have features that make them prone to one or more types of landslides. These features are:

- Steep slopes
- Surficial geologic units that are considered to be relatively weak and thus prone to landslides.

However, it is important to understand that there have been few observed landslides in the Foothills above Palo Alto since 1906. A mapping effort of previously observed landslides in the region between the Boronda Booster Pump Station and Monte Bello Reservoir is shown in Figure 3-34.

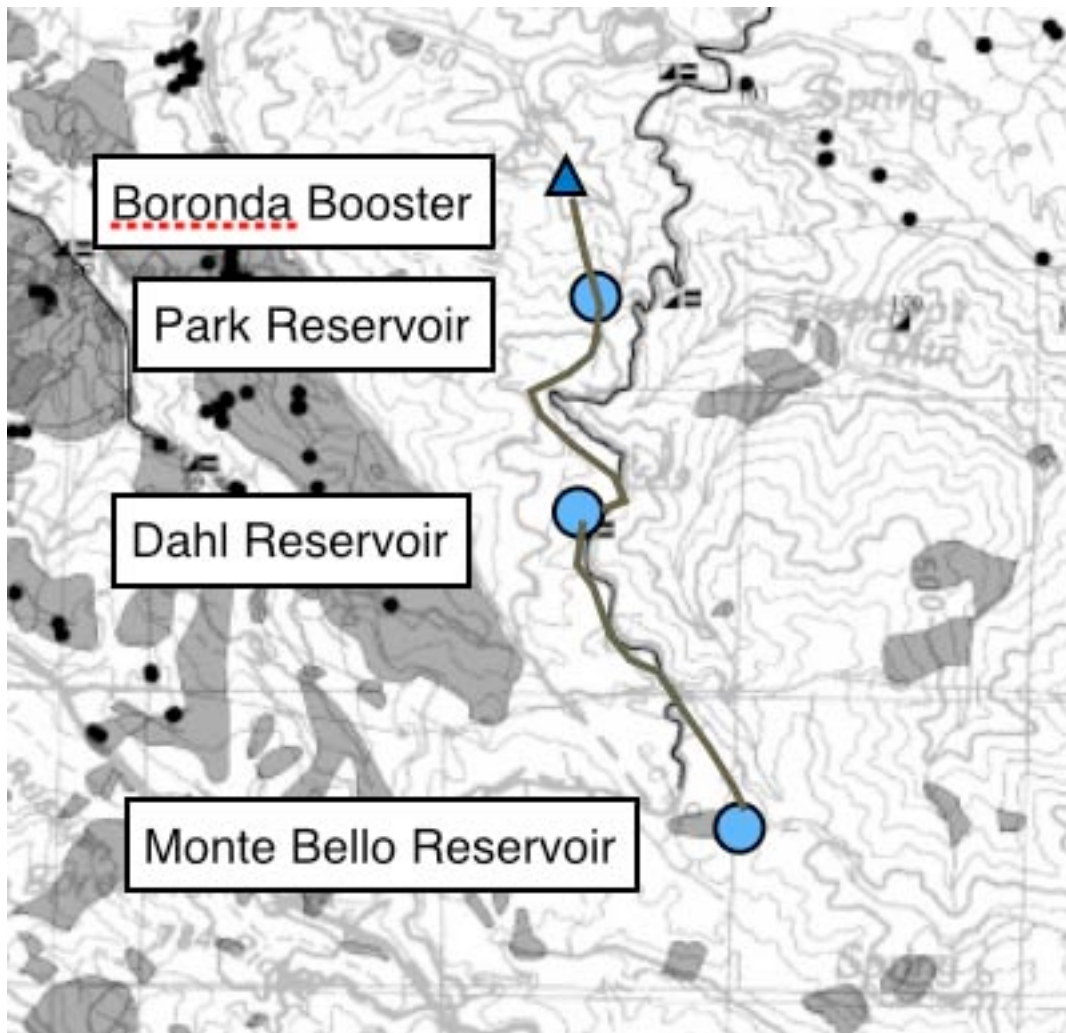


Figure 3-34. Mapped Landslides (Shaded Areas) in Foothills Area

The mapped landslides and soil movement features in Figure 3-34 is adapted from CGS (2005). There are no mapped landslides along the CCP pipe alignment or at the water tank / booster pump station sites.

While the available mapping suggests there have been no observed landslide zones along the Foothills CCP pipe alignment, this does not mean that no landslides can occur in the future. Examining Figures 3-26, 3-28, and 3-30, one sees there are several very steep sections of pipeline: with slopes of 41%, 45%, 35%, 30%, and several more with slopes under 15%. In many places, the pipeline alignment following the local ridge crests, which are less prone to slides.

However, we must consider that in a large earthquake on the nearby Peninsula segment of the San Andreas fault (M 6.0 to M 8.0), the local hillside accelerations will be on the order of 0.6g to 0.8g, and slopes that normally have an adequate Factor of Safety against landslide can momentarily have their slope stability Factor of Safety reduced beneath 1.0. Whenever the F.S. is less than 1, the slope will start to move, for the duration of time that

the driving force (gravity plus the incremental accelerations due to earthquake) exceed the slope's K_y (slope yield acceleration). Lacking borings along the pipeline alignment in the portion between the Boronda Booster and Monte Bello reservoir, a physics-based assessment of slope stability cannot be done. However, the lack of observed slides in this area over the past ~100 years (and lack of pipe leaks over the past 25 years) strongly suggests that the local geologic conditions and pipe bedding have a high Factor of Safety against slide under normal (dry and wet weather) conditions. Field reconnaissance in the hills after the 1906 and 1989 earthquakes did not mention any slide movements along the pipeline alignment.

Given these issues, the available information suggests that landslide risk for the pipeline between the Boronda Booster and the Monte Bello reservoir is low. We assign the chance of pipe break at 5% and the chance at pipe leak at 5%. Should original design and construction records for this pipeline be made available, there may be additional information that might be used to refine this assessment.

3.7 Fault Offset Hazards

Figures 3-12, 3-19, 3-25, 3-30 show fault maps in the vicinity of the Foothill areas of Palo Alto. Surface faulting hazard include primary offset, secondary offset, and sympathetic offset.

- Primary fault offset refers to fissuring and offset of the ground surface along a rupturing fault during an earthquake.
- Primary offsets on the San Andreas Fault along the Peninsula might be as little as 3 feet (magnitude 6.8 event) to about 20 feet (magnitude 8.0 event). Offset of this amount will break most below-grade water pipelines unless they have been specifically designed to take such offset. None of Palo Alto's pipes cross any known strand of the San Andreas Fault, so this hazard is not material.
- A large magnitude earthquake on the San Andreas Fault might also trigger secondary offsets mostly within 100 to 500 feet of the San Andreas Fault. Secondary offsets include ground warping and small ruptures, commonly between 5% to 25% of primary fault offset. None of Palo Alto's pipes are within 500 feet of the San Andreas Fault (although the CCP near Monte Bello Reservoir is close), so this hazard is low.
- A large magnitude earthquake on the San Andreas Fault might also trigger sympathetic offsets on nearby faults. Sympathetic offsets include ground rupture on the order of 10% to 20% of primary offset, along the surface trace of nearby faults. Sympathetic offset on nearby faults will likely not produce co-incident ground shaking, but that cannot be ruled out. This hazard is credible for the pipes in the Foothills in Palo Alto. Specifically, the Berrocal Fault (Figure 3-25) might

move 2 to 24 inches and the Monta Vista Fault (Figures 3-12, 3-19) might move 1 to 15 inches given a San Andreas M 8.0 event.

- In the 1989 Loma Prieta earthquake, there were some ground offsets noted near Highway 280 in Palo Alto. It is suspected that these offsets were due to slumps of artificial fills, and not fault offset per se. In a future M 7+ earthquake that includes rupture of the Peninsula segment of the San Andreas fault, more of these highly localized soil failures are expected, with the bulk of them likely to occur within 500 feet either side of the fault (as described above), but some could occur near Highway 280 as well as south of Highway 280.

Of these two sympathetic fault zones, movement on the Monta Vista fault is relatively more likely, based on the field observations of Hitchcock (1994, 1999).

For the push-on joint design, how much offset can the pipe sustain? The typical CCP pipe segment is 36 feet long, and if offset occurs as a knife-edge within one segment, and if a joint can take a 1-degree rotation, then the pipe can sustain about 7.5 inches of offset. However, there are serious limitations in this analogy. While the pipe could take a 1-degree rotation when initially set, by not stabbing in the full amount, once the pipe is buried in stiff soils, it cannot freely rotate around a joint. Thus, the pipe will try to accommodate offset by bending across its full barrel. The 18" CCP pipe has D/t of about 19 inches / 0.1046 inches (class 125) to 19 inches / 0.1793 inches (class 250). The most common pipe class is 125, with D/t of 181. For class 125 pipe, with a D/t of 181, the cylinder will try to wrinkle at modest strains. For class 250 pipe, with D/t of 106, the pipe could sustain considerable strain without rupture, as long as the joints do not open.

There is a lack of precise knowledge of where the Monta Vista surface rupture can occur in Figures 3-12 and 3-19, but most likely the rupture will manifest itself as thrust motions (southwest side up) along the long length of the pipe, dipping about 45 degrees, with some right lateral offset. Given this style of motion, the pipe will try to bend and shorten, and even if the barrel could sustain some strain yielding, the joints at either end will try to rotate. At the Berrocal crossing, Figure 3-25, there is less knowledge of the style of faulting, but an oblique combination of thrust (southwest side up) and right lateral offset is considered most likely.

Realistically, the CCP could reliably sustain about an inch or two of fault offset, but will be nearly certain to break at offsets of a foot or more. For intermediate offsets between 2 and 12 inches, the chance of pipe failure will increase more or less linearly from a few percent to nearly 100 percent.

Given these issues, the following describes the potential for CCP pipe failure due to sympathetic offsets:

- **Monta Vista Fault Zone.** Chance of sympathetic offset given San Andreas M 8.0 earthquake: 25% to 50%. Given offset occurs, chance of failure (pipe break that releases water) is 75%.
- **Berrocal Fault Zone.** Chance of sympathetic offset given San Andreas M 8.0 earthquake: 10% to 30%. Given offset occurs, chance of failure (pipe break that releases water) is 90%.

Combining the two fault crossing zones, the chance of 1 or more pipe repairs due to sympathetic offsets, given a San Andreas M 8.0 earthquake, is about 50%. Of this, there is a small chance that the pipe might just leak and remain serviceable until repairs can be made; but more likely, the pipe will be so damaged as to not provide hydraulic capacity.

3.8 Past Earthquakes

3.8.1 Great San Francisco Earthquake of April 1906

The M 7.8 earthquake of April 18, 1906 occurred at 5:12 am Pacific Standard Time. This earthquake has also been called the great San Francisco earthquake. The magnitude of this earthquake has previously been reported as M_s 8.0 or M_s 8.3 but has been more recently re-evaluated using the modern moment magnitude scale as M_w 7.7. The duration of strong ground shaking was about 45 to 60 seconds.

The causative fault was the San Andreas fault. Fault rupture occurred from just north of San Juan Batista (west of Hollister) to Cape Mendocino in the north. Figure 3-35 shows a map of the length of observed fault rupture and associated MMI intensities for this earthquake.

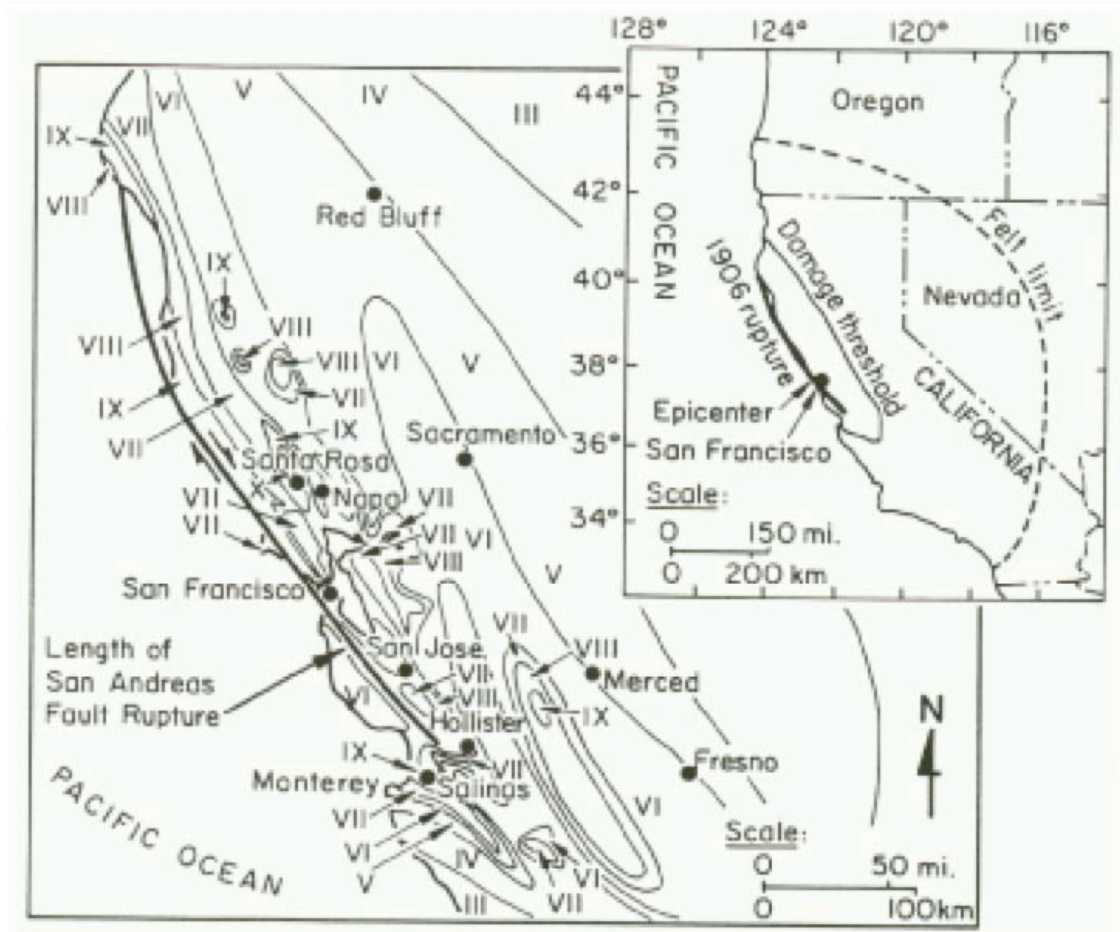


Figure 3-35. Map of 1906 San Francisco Earthquake (from O'Rourke and Hamada, 1992)

This earthquake exposed the then small City of Palo Alto water system to strong to very strong ground shaking. Figure 2-5 shows a map of the then-existing urbanized areas of Palo Alto:

- Areas northwest of Embarcadero and northeast of El Camino Real. Likely, this area was served by wells, with a cast iron distribution system. We do not have records of damage to any of this infrastructure from this earthquake. Today, some of this area remains served by cast iron pipe, although a lot of cast iron pipe in this area has already been replaced with more modern pipe materials, notably HDPE and PVC.
- Area near Mayfield reservoir. Likely, this area was served by wells, with a cast iron or possibly galvanized steel distribution system. We do not have records of damage to any of this infrastructure from this earthquake. Today, essentially all of the original 1906-vintage pipe has been replaced with modern pipe materials, notably PVC.

We do not have records of damage (or lack of damage) in the water system for Palo Alto from the 1906 earthquake. This is not to say that damage in Palo Alto did not occur; just to say that there are no modern records available that document the damage. The following describes damage to other nearby water systems in that earthquake (see Eidinger et al, 2006a, 2006b, for further details). A few miles to the southeast, four water tanks of the City of Santa Clara water system collapsed due to inertial shaking. The 36-inch Alameda pipeline, that traversed under the San Francisco Bay at Dumbarton Strait immediately adjacent to the modern BDPL 1 and 2 alignments, suffered no material damage, and remained in service. There was major damage to SVWC's 30-inch Pilarcitos pipeline where it cross-crossed the San Andreas fault, where the fault was offset 6 to 7 feet commonly; as well as at locations where it was supported on timber trestle across canyons; the damage to that pipeline was so extensive, that it had to be abandoned entirely and it has never been re-built. There was extensive damage to SVWC's 54-inch Crystal Springs pipeline, where it was supported above ground on wood trestles, through three marshes; the damage was repaired in weeks. There was localized damage to SVWC's San Andreas pipeline where it crossed a creek near Baden (believed to be Colma Creek); this was repaired in a day or so. In the City of San Francisco, the cast iron pipe distribution system suffered heavy damage (at least 300 breaks), mostly in liquefaction zones; the pipe inventory in San Francisco at that time was about 400 miles of cast iron pipe mains; with the bulk of them, located outside the liquefaction zones, suffering no or sporadic damage. Water service laterals from pipe mains were often damaged (many thousands), but it was speculated that the cause of the damage was collapsing buildings and ensuing fire, rather than direct damage from the earthquake itself; there is no way now to ascertain the exact causes.

3.8.2 Loma Prieta Earthquake of 1989

The M 6.93 (reportedly variously with surface magnitude as high as 7.1) Loma Prieta earthquake occurred on October 17, 1989. The location of this earthquake placed the nearest location of the ruptured fault to about 34 km from the central part of the City of Palo Alto (University Avenue), or about 30 km from the Monte Bello tank.

Figure 3-36 shows a map of the region. The hatched area is an estimate of the rupture area of the fault, projected to the earth's surface. The triangle symbols represent the location of strong motion instruments (free field or in basements of low rise buildings). The numbers next to the triangles represent the highest recorded horizontal ground motion, in terms of PGA (g).

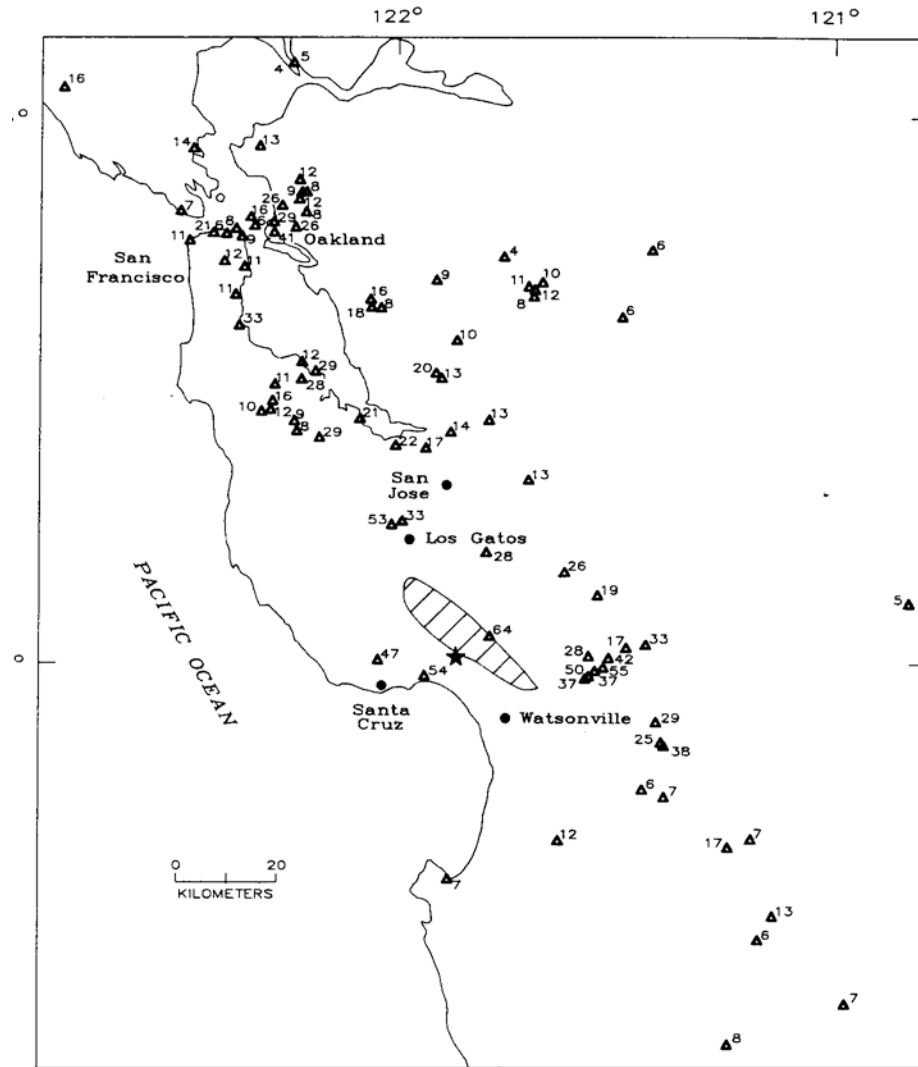


Figure 3-36. 1989 Loma Prieta Earthquake – Recorded Ground Shaking (percent g) (after Benuska, 1990)

Of relevance to the City of Palo Alto is the level of ground motions actually felt in the 1989 earthquake. The closest stations recorded the following PGA levels (after EERI, 1990):

- Palo Alto (2 story office building). PGA 37° = 0.20g, PGA 127° = 0.21g, PGA vertical = 0.09g.
- Woodside (fire station). PGA NS = 0.08g, PGA EW = 0.08g, PGA vertical = 0.05g.
- Redwood City (3 story school office building). PGA NW = 0.05g, PGA SW = 0.09g, PGA vertical = 0.04g

- Upper Crystal Springs Reservoir @ Skyline Blvd. PGA NS = 0.09g, PGA EW = 0.10g, PGA vertical = 0.04g
- Upper Crystal Springs Reservoir @ Pulgas Water Temple. PGA NS = 0.09g, PGA EW = 0.16g, PGA vertical = 0.06g
- Lower Crystal Springs Reservoir @ Dam. PGA NW = 0.06g, PGA SW = 0.09g, PGA vertical = 0.03g
- Belmont (2-story office building). PGA NNW = 0.10g, PGA WSW = 0.11g, PGA vertical = 0.04g
- Foster City – Redwood Shores. PGA NS = 0.29g, PGA EW = 0.26g, PGA vertical = 0.11g

From these records, it appears that the range of peak horizontal ground shaking in the flatlands of Palo Alto from the 1989 Loma Prieta earthquake was about PGA = 0.10g to 0.21g for locations with rock or firm alluvial soils; with amplification by soft soils to about PGA = 0.26g to 0.29g at shoreline areas underlain by thick layers of young bay mud.

Wells. Palo Alto reports that the Hale well was damaged by the Loma Prieta earthquake (damage to the casing pipe). This well had to be rehabilitated before it was restored to service. None of the other wells were known to have been damaged by the earthquake. The Hale well is located immediately adjacent to San Francisquito creek. Assuming PGA about 0.15g to 0.20g, this would have been high enough to trigger liquefaction in any loose sand layers, and a small lateral spread towards the creek might have occurred. Some soil settlement at the surface would have induced some compressive forces into the casing pipe. If the modern Hale well has not been re-built with liquefaction movements in mind, it must be considered to have a similar weakness in future moderate to large earthquakes.

In the hill areas of Palo Alto, there were no recording instruments. The nearest similar instrument was at the Woodside Fire station, located a few km to the northwest of Palo Alto. Figure 3-36 shows this instrument to have recorded a peak of 0.29g, but a review of the instrument record shows a peak of only 0.08g in either the NS or EW directions; Figure 3-37 shows the response spectra for that instrument, for the mean of the two horizontal directions (5% damping). This suggests that the thin-soil over rock-level motions at the hillside Palo Alto tanks (Monte Bello, Dahl, Park, Corte Madera, Boronda) might have also felt similar motions, perhaps PGA = 0.10g (horizontal) and 0.07g (vertical); although actual motions would have been about $\pm 50\%$ of these values (16th to 84th percentiles).

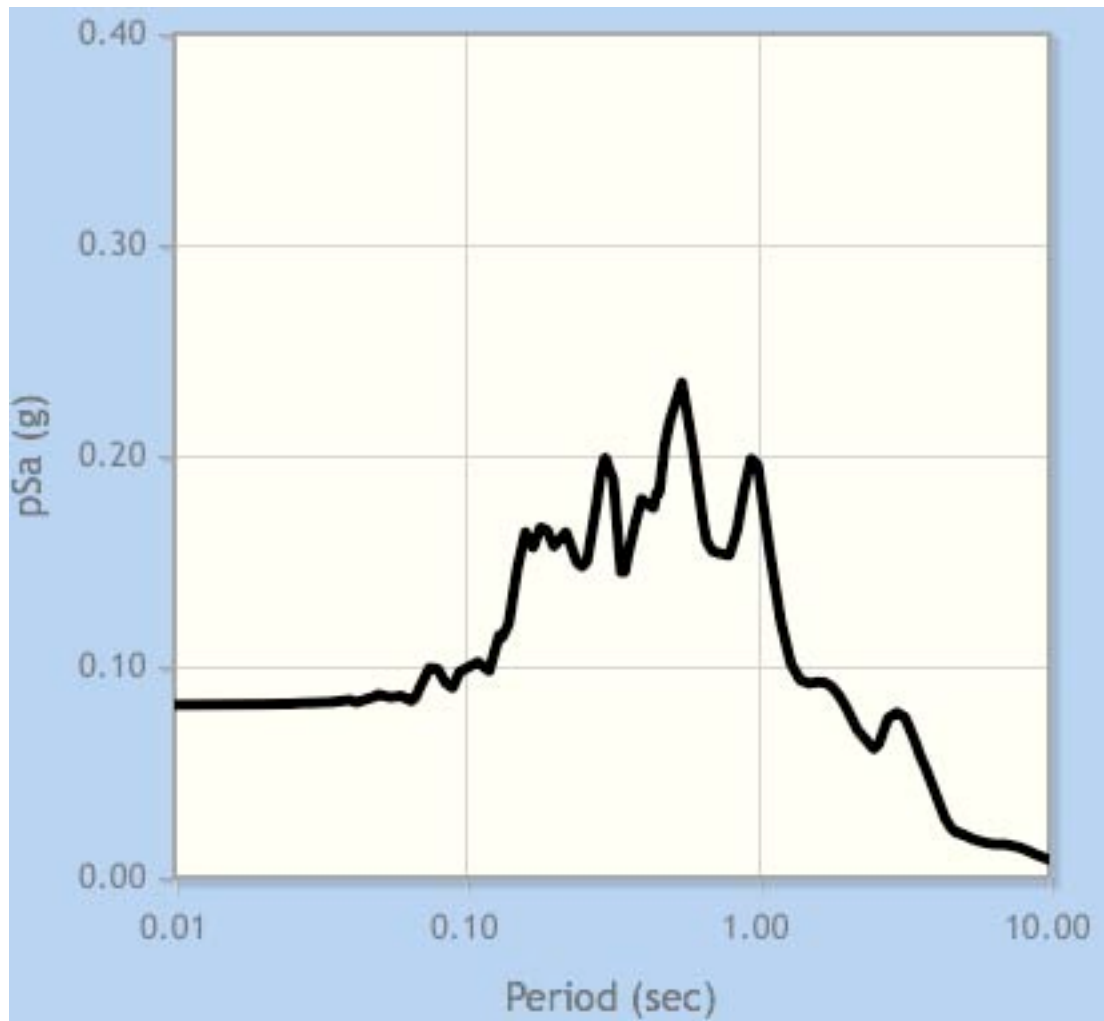


Figure 3-37. Horizontal Response Spectra, Instrument 58127 (Woodside Fire Station)

The impacts of the Loma Prieta earthquake on the Palo Alto water system were documented in a report, after the earthquake.

Water tanks. The water tanks listed in Table 2-2 were all in service at the time of the earthquake. The earthquake occurred at about 5:04 pm in the afternoon, October 17 1989. We can speculate that the water tanks in the hillside pressure zones (Corte Madera, Boronda, Park, Dahl, Monte Bello) were all nearly full (say, 80% or more) at the time of the earthquake, as water demands on these tanks, even in hot days, are relatively modest. Cwiak (1989) reported that the Dahl reservoir was full at the time of the earthquake. The water height in Mayfield reservoir is unknown.

Cwiak (1989) reported the following effects at specific reservoirs:

Monte Bello reservoir. The reservoir was observed to be in excellent shape after the earthquake. Some of the asphalt around the base of the reservoir showed signs of

movement and some asphalt chunks popped out onto the driveway. No horizontal movement of the tank was observed.

Dahl reservoir. The tank was full (or nearly so) at the time of the earthquake. The asphalt poured against the bottom steel plate (southwest side of tank, direction to the fault, as well as northeast side of the tank) was damaged, suggesting that the tank either slide sideways about half an inch, or the tank wall uplifted / rocked and pounded downwards. Water sloshed out of the roof vents; the force of the water pushing through the vents pushed out the metal screens of some of the vents; grass that was growing near the based of the vents was bent over sideways, due to the falling water coming out of the tank.

Park reservoir. No apparent damage to the tank. But, the steel roof, when walked upon, seemed "noticeably springy", suggesting that possibly there was some damage to the steel roof beams beneath.

Corte Madera reservoir. The bitumen around the base of the steel base plate was cracked. A dresser coupling in the reservoir vault was leaking, and was repaired by tightening the bolts on the coupling. A rigid conduit at the base of the roof ladder cracked, probably when the tank rocked.

Boronda reservoir. There were no apparent damage to the exterior walls. There were a few new small cracks in the roof. The vault was not damaged. The pavement around the tank was in good condition.

Mayfield reservoir. Mayfield is a 1928-vintage open cut 4 million gallon reservoir with concrete liner. There is no underdrain system for this reservoir. DiveCorr (Diver Dan Gross) did a clean water dive in the Mayfield reservoir on June 12, 1990 (about 9 months after the earthquake). He noted the following:

- Cracks in the southwest concrete wall. None were leaking at any significant rate (leaks under 1 gpm would not be observed in an underwater dive, but leaks much over 1 gpm would be observed).
- There were small leaks (0.5 to 1 gpm) at the corners of the several concrete columns that support the roof. The leaks were at the corners where the column goes under the liner.
- There was evidence that there had been prior repairs to the concrete liner; and it is assumed that Diver Dan made repairs at the observed then-current leak locations. Whether the leakage was caused or aggravated by the 1989 earthquake is speculative.

In 2010, V&A (2010) inspected the interior of the tanks. This was done by draining the tanks, and entering the tanks, and visually looking at the underside of the roof system

from the tank floor, using supplemental lighting; and then locally at the roof level using temporary support (ladders, scaffolding, etc.). The following summarizes their findings. We do not know if there were any repairs to the tank in the interim years (1989 to 2010), but suspect there were none, and Palo Alto utility staff knew of none. Over the next few years, Palo Alto intends to do substantial remedial work for these tanks, and at that time, careful close-up inspections may reveal damage that could not be observed in 2010.

- **Monte Bello.** No structural damage to roof system attributed to wave loading. Flat steel roof atop radial rafters. Radial rafters appear to be channel members, all in place, none are obviously buckled. Radial steel channel rafters supported by a center column, then atop a beam supported by 9 columns, and welded to the exterior course between air vents. Inner rafters (center column to beam) has one set of circumferential stiffeners. Outer rafters appear to have rod-type in-plane bracing. Locally, some sagging of roof steel plates.
- **Dahl.** No structural damage to roof system attributed to wave loading. Flat steel roof atop radial rafters. Radial rafters appear to be channel members, all in place, none are obviously buckled. Radial steel channel rafters supported by a center column, then atop a beam supported by 6 columns, and welded to the exterior course between air vents. No additional circumferential stiffeners.
- **Park.** No structural damage to roof system attributed to wave loading. Flat steel roof atop radial rafters. Radial rafters appear to be channel members, all in place, none are obviously buckled. Radial steel channel rafters supported by a center column, then atop a beam supported by 5 columns, and welded to the exterior course between air vents. No additional circumferential stiffeners.
- **Corte Madera.** No structural damage to roof system attributed to wave loading. Flat steel roof atop radial rafters. Radial rafters appear to be channel members, all in place, none are obviously buckled. Radial steel channel rafters supported by a center column and welded to the exterior course between air vents. There is one set of circumferential stiffeners. Locally, some sagging of roof steel plates.
- **Boronda.** The flat slab concrete roof shows radial cracks. These are likely due to incompatible movements of the exterior post-tensioned vertical walls, and in of themselves, are not indicative of material damage due to earthquakes.

Booster Pump Stations – Inspection Summary, Post 1989 Earthquake

- **Quarry Booster.** Station had no major problems or damage. An air/vacuum release valve was not setting properly and was dumping water. Action: valve to be repaired and braced to prevent breaking off in future earthquakes.
- **Meter body seals** were leaking and the seals will be tightened or replaced when time permits. The leaks do not effect the station operation.

- There were small cracks noticed in the slabs around the booster pump bases and where the underground piping surfaced through the slab. The slabs were serviceable.
- **Corte Madera Booster.** No damage.
- **Boronda Booster.** Slight damage to a thrust block on the 6-inch line. The Dresser coupling on the 6-inch pipe moved about 0.5 inches. The air-vacuum release valve leaked, similar as at Quarry.
- **Park Booster.** The support base for the plug valve on the discharge pipe to Dahl reservoir was cracked.
- **Dahl Booster.** The chlorination pipe between the booster pump station and the Dahl reservoir was out of plumb. This pipe is a small bore pipe, running up the side of the Dahl reservoir. It was likely damaged due to slight uplift of the tank.

Pipelines. Damage due to 1989 Loma Prieta Earthquake

Figure 3-38 shows the buried pipe leaks (water, gas, sewer) known to have occurred near Palo Alto in the 1989 Loma Prieta earthquake.

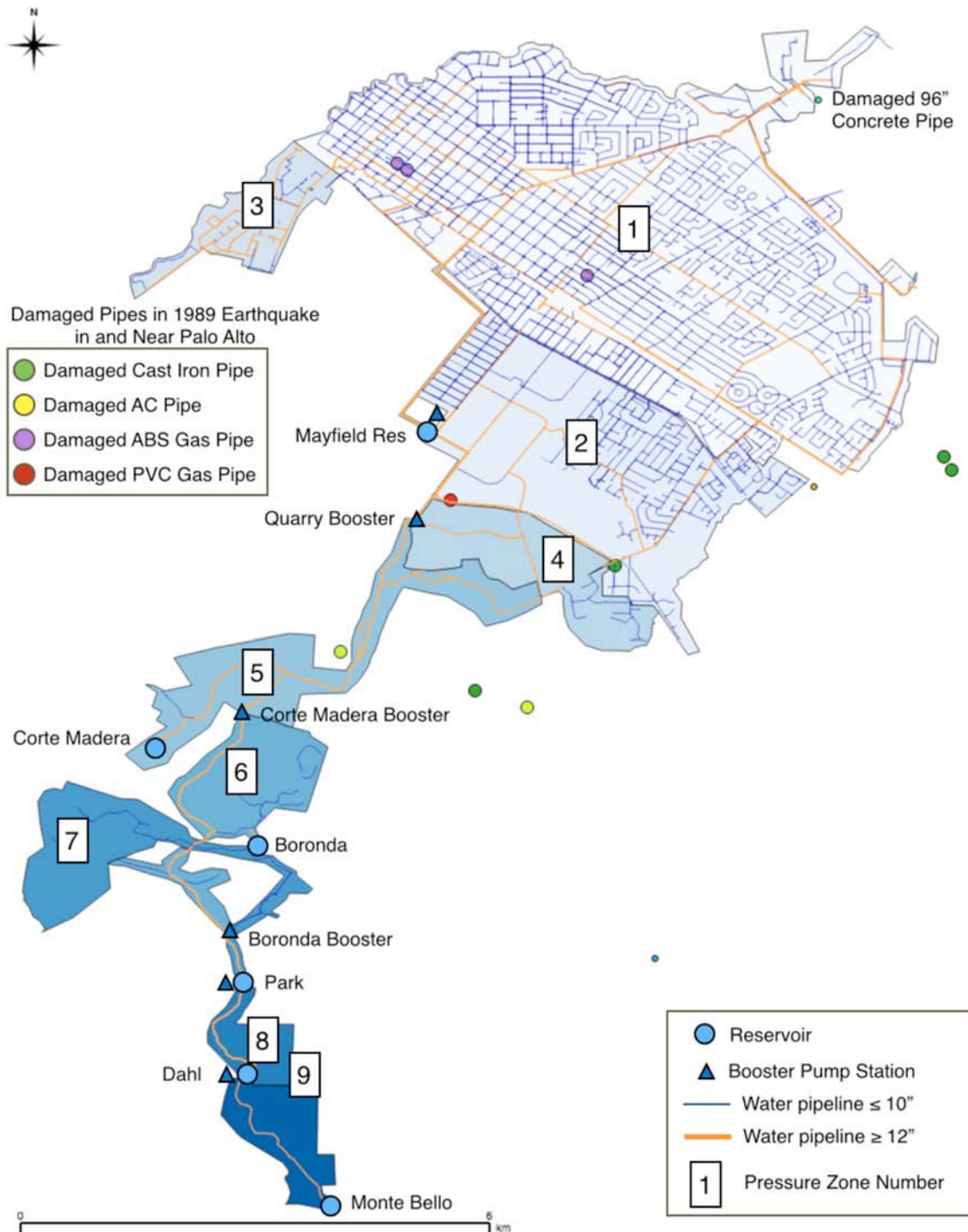


Figure 3-38. Repairs to Buried Pipe Near Palo Alto, 1989 Loma Prieta Earthquake

The following explains the various leak repairs (small or large dots) in Figure 3-38:

- The blue lines represent the modern Palo Alto water system pipes. Palo Alto reported no damage to any water pipe in the 1989 earthquake.

- At the Palo Alto water pollution control facility, a 96-inch diameter reinforced concrete pipe was damaged where it entered a junction box (small dot in northeast part of water system). This location had high ground water, and might have been subject to PGDs. It was sealed with pressure grout. This area is mapped in a red zone (very high liquefaction) in Figure 3-11.
- The Palo Alto gas department reported 4 repairs: 3" ABS, 3" ABS, 4" ABS, 6" PVC. These are indicated by the 3 large purple dots (ABS) and one large red dot (PVC). All of these are in the areas mapped as yellow zones (moderate liquefaction) in Figure 3-11.
- Mountain View reported some pipe repairs. Two cast iron pipes are shown by the large green dots on the east side of the map. A copper service lateral is indicated by a small dot on the east side of the map.
- The Purissima Hills Water District (immediately adjacent to Palo Alto in the higher elevations) had 5 water pipes that needed repair (4" steel, 6" AC, 8" CI, 10" CI, originally installed 1957 to 1962). The AC repairs are indicated by large yellow dots. The CI repairs are indicated by large green dots. The steel repair is indicated by a small dot in the southeast part of the map.
- The small dot in central Palo Alto is bad data.

Region wide, there were 862 reported repairs (as reported in a survey of 39 agencies) made to underground water and wastewater pipes (including the above mentioned Palo Alto gas pipes) (Lund and Schiff 1990). Figure 3-39 shows the location of the pipe repairs for the greater San Francisco Bay Area area, as well as the same information shown for Palo Alto in Figure 3-38. There were additional repairs made by other water agencies that did not participate in that survey (and not included in these figures), including about 66 in the City of Santa Clara water system, 1 in Redwood City, etc. When reporting, there are two stages of pipe repairs: Stage 1: those uncovered and repaired in the first few days to weeks after the earthquake; and Stage 2: those that occur in the first several months after the earthquake. Stage 1 repairs can result in widespread water outages, loss of pressure, and overtax the ability to repair all the damage quickly; these are the primary concern. Stage 2 repairs occur, more-or-less, on a "business as usual" basis, casing localized water outages and repaired in a more-or-less normal fashion. In the recent Napa August 2014 earthquake, the Napa water department has about 160 "Stage 1" water pipe repairs, with an additional 80 "Stage 2" repairs over the following 5 months. For purposes of this report, the initial "160" repairs are the primary concern but from an emergency planning point of view, once the Stage 1 repairs are made, it can be expected that a 50% to 100% higher than normal pipe repair rate will continue for several months, with the repair rate returning to more-or-less "normal" long term rate about half a year following the earthquake.

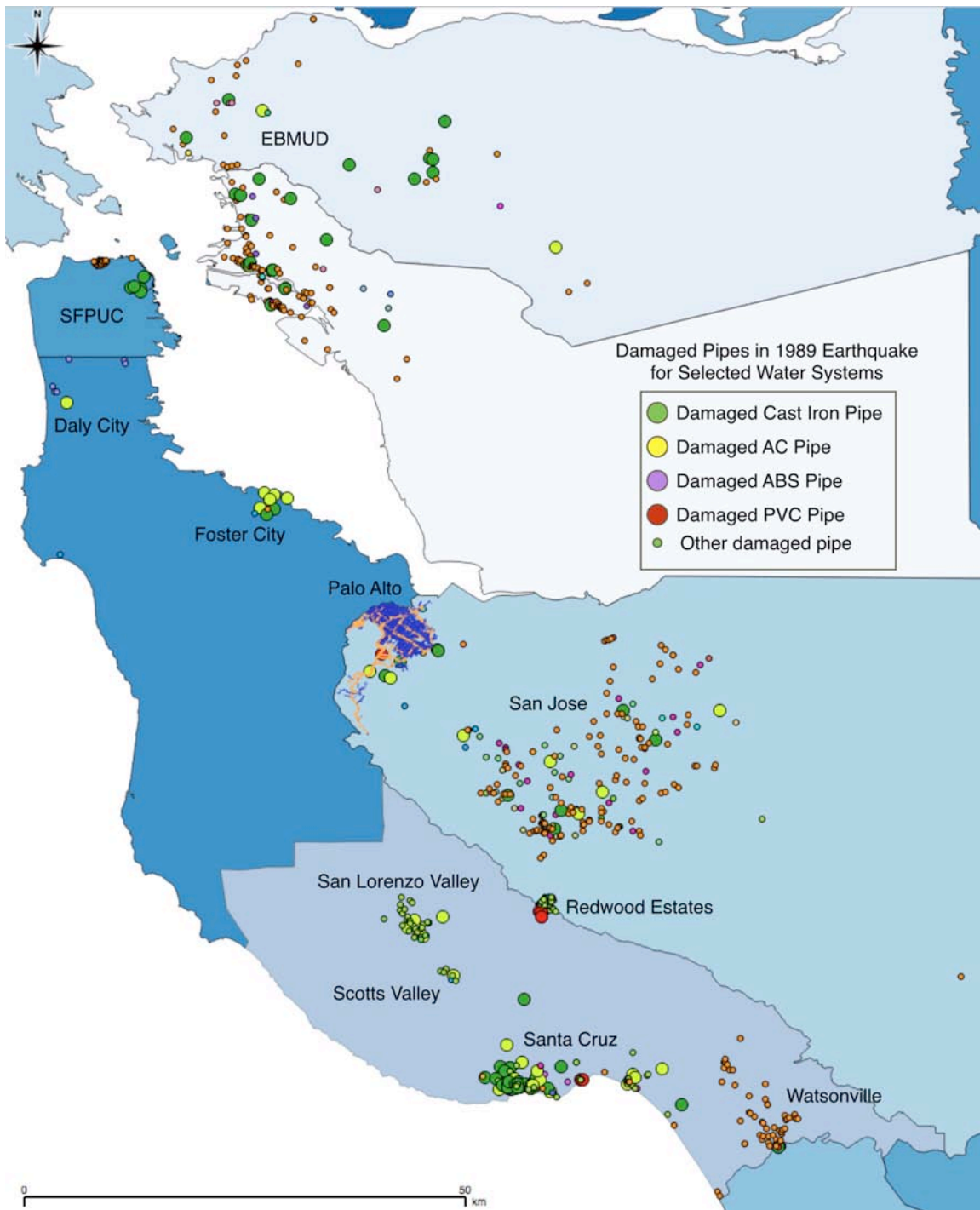


Figure 3-39. Repairs to Buried Pipe, San Francisco Bay Area (Full scale 50 km)

Figure 3-40 shows the locations of damaged water pipes (66 total) in the City of Santa Clara due to the 1989 earthquake (these dots are not included in Figure 3-39). 64 of the 66 repairs were to cast iron pipe. 1 (possibly 2) of the repairs were to AC pipe. There was no observed liquefaction at these pipe repair locations in the City of Santa Clara in the 1989 earthquake.

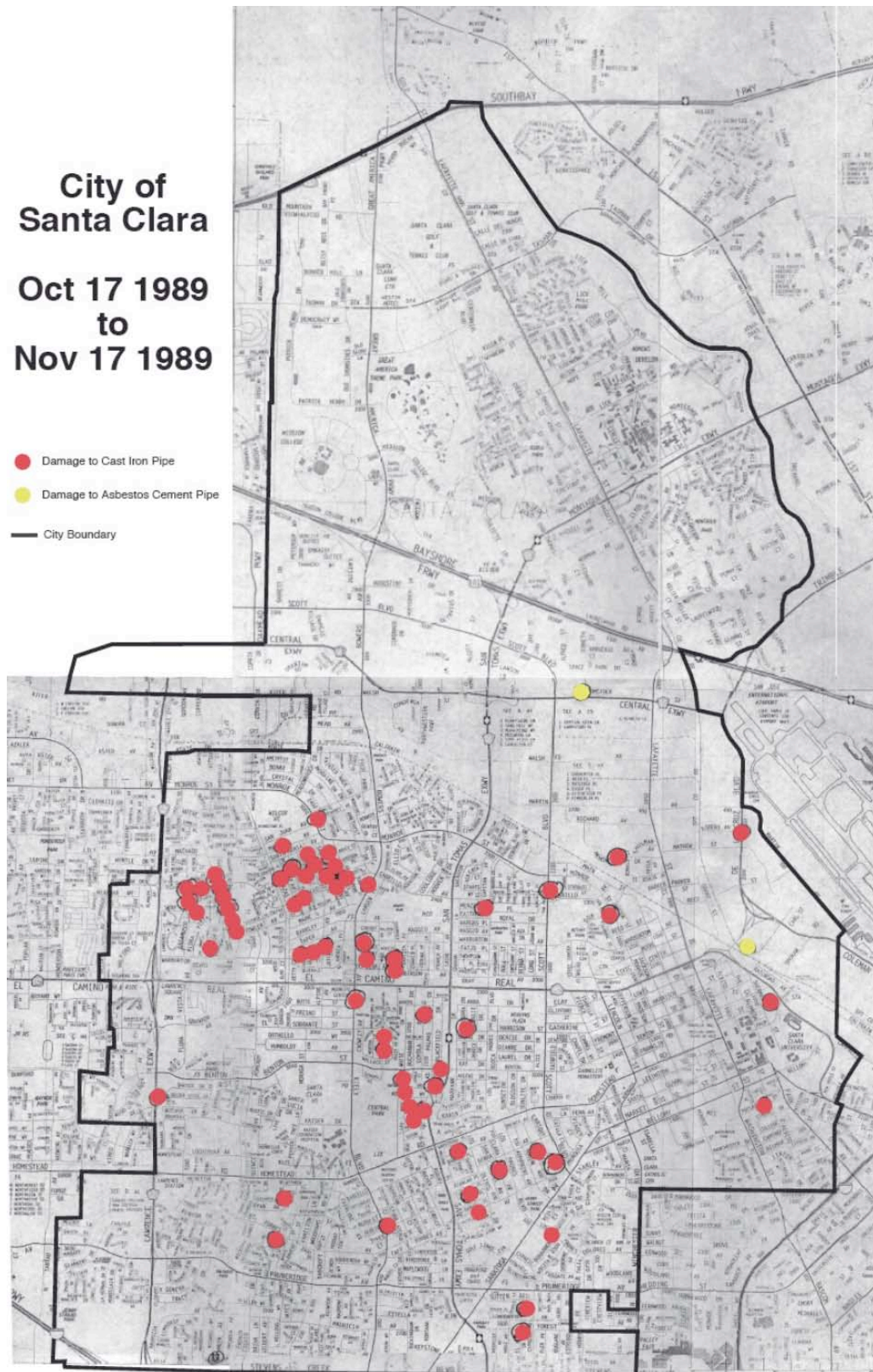


Figure 3-40. Repairs to Buried Water Pipe, City of Santa Clara

Summary. Based on the recorded motion at the Woodside Fire Station, ground motions in the Palo Alto Hills, at the time of the 1989 Loma Prieta earthquake, may have been on the order of $PGA = 0.10g$ or so. At this level of shaking, common induced wave heights (assuming no restrictions by the roof) might have been about a foot or so; and unanchored steel tanks would not have uplifted. This explains the apparent good performance. In future earthquakes, PGA levels from a M 7.0+ event on the nearby San Andreas fault can cause PGA levels (median) of about $0.7g$ (Monte Bello), lowering to about $0.5g$ (Corte Madera / Boronda), with actual motions varying by $\pm 60\%$. In other words, some of the tanks might experience 1.6 times the median motions, and some of the tanks might experience 63% of the median motions. Unrestricted wave heights can reach 5 or more feet. Unanchored steel tanks will definitely uplift their walls, possibly by several inches, and such wall uplift will damage the roof system unless the roof system is designed for wall uplift. All attached pipes are currently through the bottom plates, and if the pipe attachments are within about 18 inches of the wall, uplift will damage the attached pipe, resulting in unrestricted loss of all water contents. To limit the damage in future large earthquakes, anchoring of all steel tanks is recommended (including Dahl). As part of regular maintenance, care of interior and exterior coating systems should be done, and regularly inspected (visually inspect every 5 years, and plan on some type of improvement possibly once every 10 to 20 years, depending on the condition of the paint and cathodic protection systems).

Wells. Palo Alto reports that there was no known damage to the then-operating wells in the water system, with the exception of Hale well. Interior inspection of Hale well showed two locations where the steel casing pipe was buckled; this damage was subsequently repaired. The cause(s) of the damage to the casing pipe are not entirely certain: possibly due to some ground shaking or event liquefaction; but this seems unlikely, as the damage was at depths over 200 feet, and any near-surface manifestations of liquefaction (say, in the top 30 feet or so), would not likely induce significant forces onto the casing pipe at depths more than 170 feet lower, as skin friction between the casing pipe and the surrounding soil would absorb almost all of such force.

3.8.3 Palo Alto Park Mutual Water Company

The Palo Alto Park Mutual Company is not part of the City of Palo Alto water system. It is a non-profit mutual benefit corporation, owned by about 650 property owners in the Palo Alto Park area, a subdivision of East Palo Alto and Menlo Park. The mutual company is of interest with regards to its performance in the 1989 Loma Prieta earthquake.

Figure 3-41 shows the approximate boundaries of the Palo Alto Park water system, highlighting the location of its wells and tank.

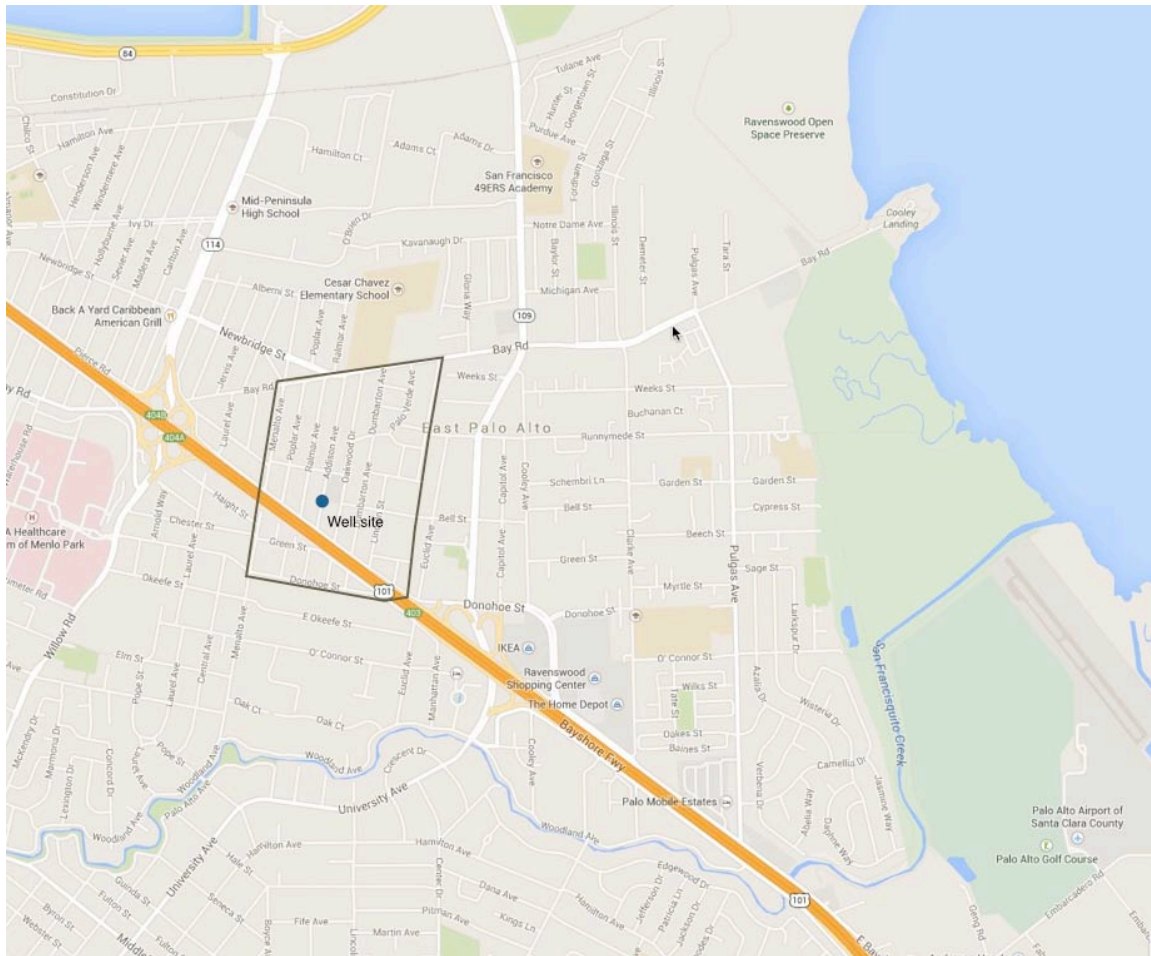


Figure 3-41. Palo Alto Park Mutual Water Company Service Area (Approximate)

Palo Alto Park is a subdivision within East Palo Alto, bordered by Palo Alto to the south, and Menlo Park to the west. Palo Alto Park is entirely in San Mateo County. In 1917, two wells were drilled to provide water for a US Army camp then located there. After 1918, the water from the wells was used mainly for irrigation. By 1924, housing began to be developed and by the 1940s more wells were drilled and deepened. In 1950, a sixth well was drilled 280 feet, and a 100,000 gallon redwood tank (at grade) was built to hold the well water.

Prior to the 1989 Loma Prieta earthquake, the water system was reported to be "in bad shape". Some streets still had 2-inch water mains. If there were leaks, the entire system had to be shut down to make repairs.

After the 1989 earthquake, the water system was "in a shambles", experiencing 2 or 3 leaks a day. The daily routine after the earthquake was to drive around spotting leaks, and then determine which could be repaired with in-house staff or with contractors. Well numbers 5 and 6 and the booster pump were damaged. The 100,000 gallon redwood tank (Figure 3-42) leaked and an attempt to repair the tank was unsuccessful, as when the tank was filled on December 20 1989, it collapsed. Without the tank, water from the wells was

being pumped into a closed system, and careful watch was needed to not over pressurize the pipes.



Figure 3-42. Palo Alto Park Mutual Water Company (pre-1989).

After the earthquake, a 7th well was drilled to 500 feet, wells 1 and 4 were abandoned, two tanks were added (11,500 gallons and 350,000 gallons) and the water system improved to use 8-inch and 6-inch mains throughout. The 350,000 gallon tank is an at-grade steel tank. FEMA paid for a portion of these improvements.

As of 2013, there were 5 wells (#2, 3, 5, 6, 7) which pump water into the storage tank, where it is blended and then pumped by two booster pumps to maintain system pressure at about 68 psi. There is an emergency generator to provide power should PG&E power be lost. With all wells working, maximum flow rate is about 1,200 gpm.

As part of the current effort, we have not done a detailed assessment of the Palo Alto Park water system. The cause(s) of the damage to the pipelines in the 1989 earthquake might have been:

- 2-inch pipes suggest that these were original pipes, dating to the 1920 to 1930 time frame. The common pipe material at that time was either galvanized steel, or cast iron. Both of these materials are susceptible to corrosion in soils like that pervasive in East Palo Alto area. Further, the subsurface of this area is prone to liquefaction, and might have experienced small settlements in the 1989 earthquake. The damage to the (likely) unanchored redwood tank possibly reflected some sliding of the tank on its base, damage to its attached pipes, loosening of the hoop bands coupled with long term wood deterioration. Inserting fiberglass (or similar) liners for pressure retention was a practice used by EBMUD for their dozen redwood tanks; EBMUD has ultimately

abandoned (since 1995) some of their redwood tanks owing to the difficulty of restraining them adequately for strong earthquake motions; replacing them with modern designed steel tanks.

3.9 Boring Database

During the course of this effort, Palo Alto provided us with a GIS-based database with soil boring data. Figure 3-43 shows the locations of reports with soil-boring data. There are 1,300 reports in the database. The GIS-based database has the following relevant attributes:

- SiteID
- Street number and Street Name
- Name of Engineer / Developer that developed the report
- Date and Year of issue of the report
- Comment field (typically to further describe the location)
- Was ground water encountered (yes/no); and depth (in feet)
- Number of borings in the report
- Depth of boring

Figure 3-44 shows the depth to water, based on the borings in Figure 3-43. Figure 3-45 shows the borings where no ground water was recorded (white dots). Heavy black lines are included to show the approximate borders for varying depths to water. This shows:

- The northeast part of Palo Alto has very shallow depth of groundwater (commonly under 10 feet)
- Away from creeks, the depth to ground water gets deeper in the flatlands, moving towards uphill to the southwest.
- By Zone 3, the depth of ground water is over 30 feet, except locally along San Francisco Creek, where the depth to ground water is at the surface of the creek.

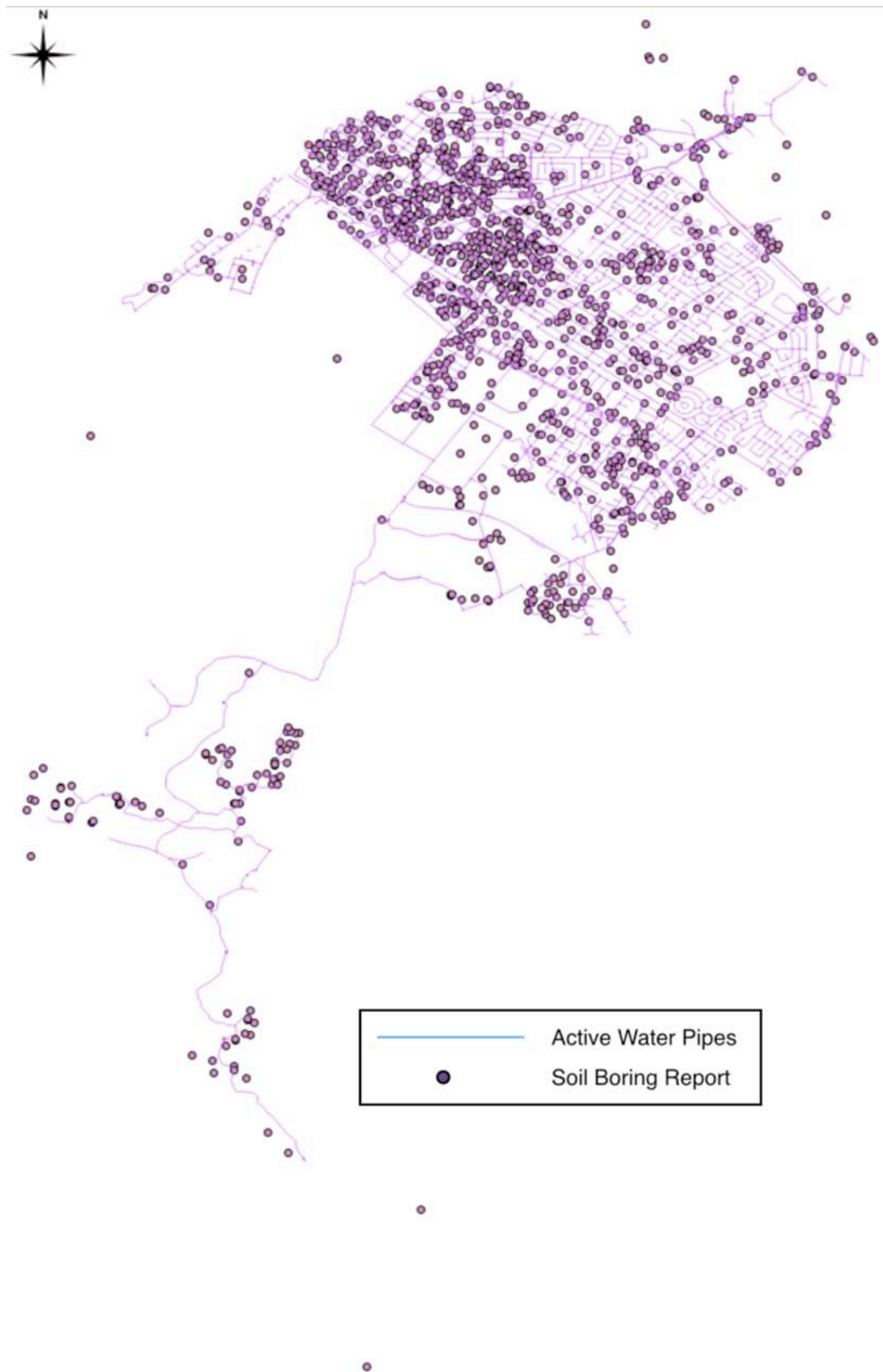


Figure 3-43. Palo Alto Soil Boring Map

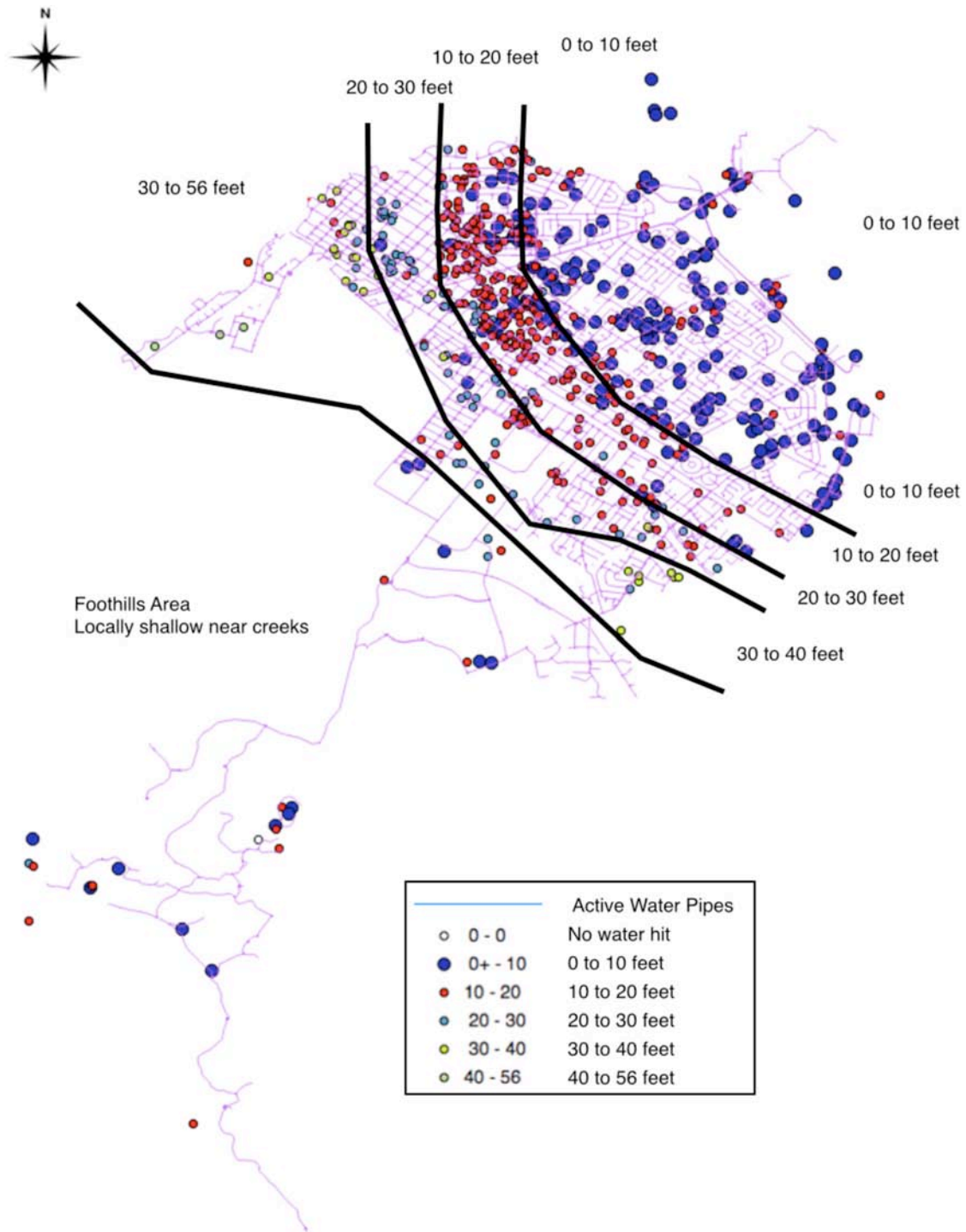


Figure 3-44. Palo Alto Depth to Water (Feet) From Borings

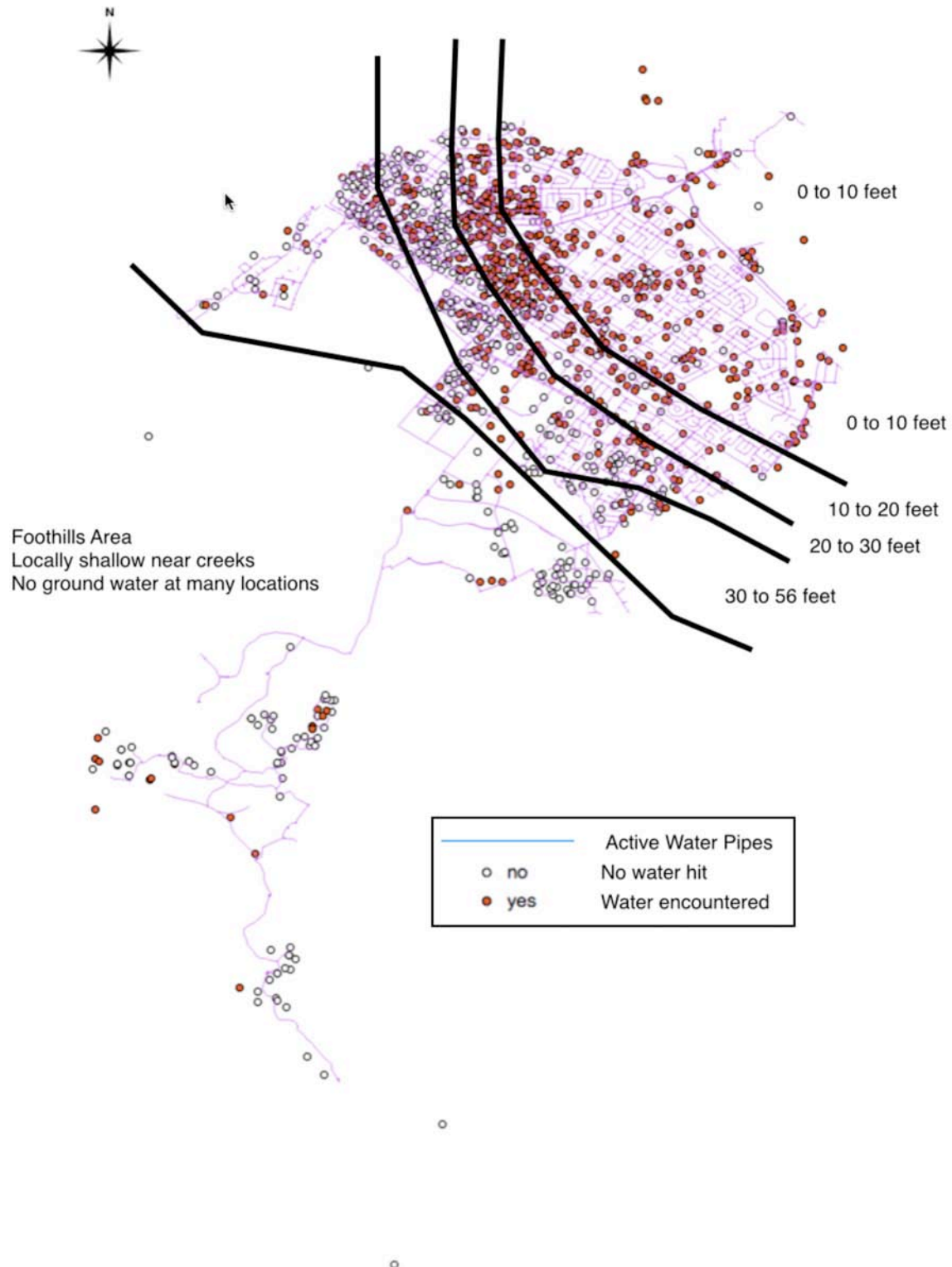


Figure 3-45. Palo Alto Depth to Water From Borings (Yes / No)

4.0 Seismic Assessment of All Pipelines

Section 4 of this report examines the performance of the pipelines in the entire water system due to 24 scenario earthquakes. In Section 5, additional more detailed evaluations are provided for the CCP pipeline between the Quarry Booster station and Monte Bello reservoir.

The seismic assessments in Section 4 were performed using the SERA program. The following outlines the approach.

- A scenario earthquake is selected (fault, magnitude, epicenter)
- SERA computes the ground motions at each of 25 sites and each of 7,315 pipes, including PGA, PGV and spectra, considering the Vs30 value for each site / pipe.
- Each pipe is assigned a pipe-specific liquefaction and landslide susceptibility. This reflects the mapping work described in Section 3 of this report.
- SERA computes the chance of triggering liquefaction at each of the pipeline locations. If liquefaction is triggered, SERA computes the ground settlement and lateral spread at the pipeline. The vector sum of settlement and lateral spread is the PGD computed for the pipe.
- SERA computes the chance of triggering a landslide at each of the pipeline locations. If a landslide is triggered, SERA computes the landslide movements. The amount of landslide movement is the PGD computed for the pipe.
- The higher of the liquefaction and landslide PGDs is used to assess the potential for damage to the pipe.
- The chance of repair is computed for each pipe, either due to PGV (all pipes are exposed to some levels of PGV) or PGD (only some pipes are exposed to PGDs). The pipeline fragility models are based on those described in ALA (2001), with adjustment for soil resistivity (corrosion) and the performance of pipelines in the recent August 23 2014 Napa earthquake (see Appendix B).
- Based on the style of pipe and hazard, the pipe repair is classified as either a leak (like a leaking service lateral) or a break (the pipe main is so badly damaged that no water can pass).

This computation is done for every pipe, and the damage is tabulated by pipe material (CIP, ACP, PVC, etc.) and diameter, as well as by pressure zone. SERA provides tables with all these statistics.

Within each pressure zone, SERA then does a rapid hydraulic analysis, examining how the pipe network will perform hydraulically, given the damage in the network. SERA does this computation at two stages:

- At Time $T = 0$ hours (immediately after the earthquake). At this time, the leaking and broken pipes are spilling water. The water pressure at the leaks is essentially zero, and the high rate of water loss also causes a lot of hydraulic head loss. The system becomes depressurized, and customers on non-damaged pipes might still lose water pressure.
- At the time when gate (or butterfly) valves are closed to isolate the leaking / broken pipes. At this time, assuming there is still water supply available, the number of customers with water will be higher than at $T = 0$, as water pressures will increase.

4.1 SERA Analysis of Sites

The SERA program allows rapid analysis and assessment of above ground water facilities and below ground water pipes. For this project, the scope of work is geared towards assessing the performance of the underground pipe network. In any case, we have included in the SERA model 25 sites in the Palo Alto water system, listed in Table 4-1, along with their locations. The above ground sites include all the reservoirs, booster pump stations and turnouts from the SFPUC pipelines. At each site, we compute ground motions for each scenario earthquake.

SITE NAME	SERA MODEL	LONGITUDE DEGREES	LATITUDE DEGREES	EASTINGS NAD1983 (FEET)	NORTHINGS ZONE 0403
QUAR Quarry Booster	Yes	-122.1575	37.4038	6080250	1973744
DAHB Dahl Booster	Yes	-122.1778	37.3396	6073944	1950502
DAHL Dahl Reservoir	Yes	-122.1779	37.3398	6073889	1950569
CORT Core Madera Booster	Yes	-122.1778	37.3813	6074214	1965679
BORB Boronda Booster	Yes	-122.1793	37.3556	6073609	1956318
BORN Boronda	Yes	-122.1760	37.3667	6074645	1960369
B.IR Boronda Irrigation	Yes	-122.1760	37.3669	6074646	1960424
LYTT Lytton Booster	Yes	-122.1670	37.4447	6077740	1988684
HALE Hale	Yes	-122.1551	37.4554	6081284	1992521
EPRK Eleanor Pardee Park	Yes	-122.1424	37.4509	6084931	1990831
RINC Rinconada Well	Yes	-122.1411	37.4443	6085266	1988422
MLIB Main Library Well	Yes	-122.1387	37.4448	6085966	1988592
PEER Peers Park Well	Yes	-122.1461	37.4318	6083735	1983886
SF-C California	Yes	-122.1459	37.4249	6083749	1981384
SF-P Palo Alto Pipeline	Yes	-122.1707	37.4463	6076688	1989302
FERN Fernando	Yes	-122.1324	37.4206	6087641	1979750
MATA Matadero Well	Yes	-122.1357	37.4185	6086670	1979002
MAYF Mayfield	Yes	-122.1561	37.4130	6080711	1977104
MAYB Mayfield Booster	Yes	-122.1558	37.4132	6080799	1977175
SF-P Page Mill	Yes	-122.1556	37.4060	6080811	1974553
SF-A Arastradero	Yes	-122.1299	37.4006	6088240	1972456
PARK Park Reservoir	Yes	-122.1773	37.3506	6074148	1954497
PARB Park Booster	Yes	-122.1774	37.3505	6074118	1954461
CORT Corte Madera Res	Yes	-122.1871	37.3774	6071475	1964305
MNTB Monte Bello Reservoir	Yes	-122.1678	37.3248	6076741	1945056

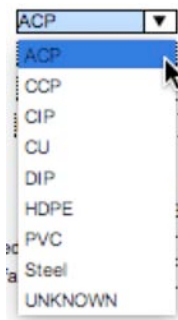
Table 4-1. Sites Included in the SERA Model

4.2 SERA Analysis of Pipes

Table 4-2 shows a page from the SERA database for pipe number 44. This page includes various attributes assigned to one of the 7,315 pipes in the database. Each pipe is assigned the following attributes:

- MainID. This is a unique number used by SERA. In Table 4-2, the number is 44, and has no meaning other than being a unique identifier for each records.
- PIPE GID. This is the corresponding ID from the Palo Alto-supplied GIS.
- Diameter. This is the diameter pipe. Supplied from the Palo Alto GIS. If blank, SERA assumes 6 inches.

- **Installation Date.** This is the date of installation of the pipe. Supplied from the Palo Alto GIS. Many such dates are missing in the database.
- **YearBuilt.** This is extracted from the installation date. "0" means no data.
- **Pipe Age.** This is computed as the age, from 2015, to the year built. "0" means no data, in which case SERA assumes the pipe installation date is 1950.
- **Material.** This is the material of the pipe. Supplied from the Palo Alto GIS. A pull-down menu provides the possible entries. If blank, SERA assumes "UNKNOWN".



- **Length.** This is the length of the pipe. Supplied from the Palo Alto GIS.
- **Pressure Zone.** This field shows the assigned pressure zone name / number. The original data was supplied from the Palo Alto GIS, in terms of a number form 1 to 9. For the SERA database, each zone is given both a number and a text name, form 1 to 9 (One to Nine). The Unknown field is used when there is no data.
- **Pressure Range.** This field shows the common pressures in the pipe mains in this zone. This data was supplied by Palo Alto.
- **Street Location.** This is a text field supplied by Palo Alto.
- **Vs30 used by SERA.** This data was developed by G&E. It is based on the regional Vs30 values for Palo Alto. No site-specific down hole testing was done.
- **VS30_USGS.** This field can hold Vs30 data developed from the USGS or other sources.
- **NEHRP_Inferred Soil.** This is the NEHRP soil designation. It can be A (hard rock), B (rock), C (stiff thin soils), D (deeper firm soils), E (deep soft soils), F (deep very soft soils). For a full description of these 6 classifications, see the IBC.

- Liquefaction. Each pipe is assigned a liquefaction susceptibility. These can be one of: VH (very high), H (high), M (moderate), L (low), Very Low (very low), W (water), NW (Not mapped).
- Landslide. Each pipe is assigned a landslide susceptibility. These can be one of: VH (very high), H (high), M (moderate), L (low), Very Low (very low), W (water), NW (Not mapped).
- Surface Faulting. Each pipe is assigned a surface faulting susceptibility. These can be one of: VH (very high), H (high), M (moderate), L (low), Very Low (very low), W (water), NW (Not mapped).
- White and yellow mini-spreadsheet. This shows the digitized points of the pipe, shown in projected space (CA 0403, NAD 1983, in feet).
- Mid Point. This is the computed mid point along the pipe. For pipes with multiple points used for digitization, this point is along the pipe, at mid-length.
- Number of Points in Pipe Element. The number of points used to digitized the pipe. In this case, there are 2 points.
- Distance to shoreline. This is the distance from the pipe to the nearest creek or shoreline or open face. This was computed using GIS techniques, comparing each pipe to all GIS coded creeks, San Francisco Bay sloughs / shoreline, including all the creeks and water boundaries shown in Figure 3-5. A default value of 10,000 feet is used for pipes where lateral spreads are assumed to not occur.
- Water Table (index, range (feet) and water table). A water table contour map was obtained from SCVWD. Each pipe segment was located to its closest contour line. Each contour line is supplied with an index, where index "1" means that the water table is between 0 and 10 feet, "2" means 10 to 20 feet, etc. The specific water depth assumed by SERA is taken at the mid-level of the assumed water table. This value can be used to refine the liquefaction model to reflect seasonal variations in the ground water table. For the results presented in this report, we assume the ground water table is at the historic high level, which is a conservative assumption, in that it will trigger the most liquefaction.
- Liquefaction Map Feature. Each pipe was compared to two liquefaction maps (Witter 2006 and Knudson 2000); the Witter (2006) map is shown in Figure 3-10. Underlying this map are a series of polygons, with each polygon representing a similar surface geologic condition. Each polygon is assigned a liquefaction susceptibility, based on historic observations from past earthquakes, borings and CPTs, and judgment. The fields "unique" and "user assigned" are polygon numbers in the underlying maps. The field "LIQ" is the liquefaction

susceptibility for that polygon, assuming highest historical ground water level. The field "LIQ_source" is either 0 or 1, with 0 meaning the value was based on original data, and "1" meaning that the user has modified the value. The field "Ptype" describes the surface level geology. The field "Ptype2" describes the top soil cap layer, if present, and is only used in the 2006 map. The field "Description" provides a text description of the surface geology.

- **Soil Resistance.** A series of soil resistivity tests were performed in 2015. Based on those tests, each pipe is assigned a soil Rho value, as a weighted average of the five nearest tests, for the soil layer at depth 5 feet. The specific test value from the nearest test is also listed. Table 4-3 shows the five closest tests for pipe number 227, where all five test locations are listed. If blank, SERA assumes 2,000 ohm-cm.
- **Leak history.** This table shows the known leaks that have been repaired on this pipe, between the period of about March 1990 to December 2014 (about 24.5 years).
- **Leaks since March 1990.** The count of the number of leaks repaired on this pipe, since March 1990.
- **Number of Leaks since Jan 1 2007.** The number of "recent" leaks repaired on this pipe, since January 1 2007. The user can change the year from which to count the number of recent leaks.
- **Years since 2015 for the most recent leak.** The number of years since the most recent leaks. This is computed using only the year field for each leak, so if the most recent leak occurred in 2013, and the year entered here is 2015, the result would be 2 (= 2015 – 2013).
- **BCR Leak Repair.** This is the Benefit Cost Ratio for replacing the pipe with a new pipe, based on the projected future leak repairs for this pipe, due to aging reasons. This value is computed in SERA and stored in this field.
- **BCR Earthquake.** This is the Benefit Cost Ratio for replacing the pipe with a new pipe, based on the projected future leak repairs for this pipe, due to earthquake reasons. This value is computed in SERA and stored in this field.
- **BCR Total.** This is the numerical addition of BCR (Leak Repair) and BCR (Earthquake).
- **Replace Cost.** This is the user-supplied value of the cost to replace this pipe for pipe aging purposes.

- Seismic Upgrade (check boxes for SIP-1, SIP-2, SIP-3, SIP-4). A checkbox is highlighted if this pipe is included in the one of the four seismic SIP upgrade strategies, SIP-1 through SIP-4.
- WMRP Phase_No. This is the WMRP phase number for this pipe, developed by Palo Alto, based on the current pipe replacement strategy.
- LiqSP. This is another field to enter the liquefaction susceptibility for this pipe. If entered, SERA uses this field entry to over-ride the value in the Liquefaction field above.

The SERA program allows rapid analysis and assessment of above ground water facilities.

MainID
 PIPE GID

DIAMETER inches
 Installation Date YearBuilt (-9 for unknown) PipeAge Years
 MATERIAL Status
 Length feet Onwership
 Pressure Zone PressureRange psi
 Street Location

Vs30 used by SERA meters/sec
 Vs30_USGS meters/sec

NEHRP_Inferred Soil Landslide
 Liquefaction Surface Faulting

Distance to shoreline feet (10,000 indicates no lateral spread hazard)
 index range (feet) water table
 WaterTable No Data feet

Number of Points in Pipe element Mid Point

Liquefaction Map Feature
 Unique User Assigned LIQ LIQ_Source PType PType2 (Top Soil Cap) Description, PType2 (2006) Ptype (2000)
 Mid Point, 2006 Map
 Mid Point, 2000 Map
 =0 Map
 =1 User Entry

Weighted Average of 5 Nearest Tests Nearest Test Rho Distance to Nearest Test
 Soil Resistance ohm-cm ohm-cm feet

Leak ID	Date	Leak Location	Address	Leak Category	Leak Description
602125	9/11/1991	Main at Service	275 Hawthorne /		

Number of Leaks since March 1990
 Number of Leaks since Jan 1
 Years since for most recent Leak Computed. Blank if there have been no leaks
 BCR Leak Repair
 BCR Earthquake (simplified)
 BCR Total

ReplaceCost This cost is for replacement for pipe aging. Fault tolerant design costs will be different

Seismic Upgrade? SIP1 SIP2 SIP3 SIP4
 WMRP Phase_No

LiqSP Special. Leave blank to use the Liquefaction index set above.
 Enter a value from the available choices if you want to change the liquefaction index for this specific pipe. LiqSP value will be used by SERA if it is entered.

Table 4-2. Pipes - Attributes Included in the SERA Model

					MainID <input type="text" value="227"/>
averageR	<input type="text" value="917.58"/>				
	1 Closest	2	3	4	5
test ID	<input type="text" value="23"/>	<input type="text" value="22"/>	<input type="text" value="24"/>	<input type="text" value="21"/>	<input type="text" value="25"/>
Distance, feet	<input type="text" value="88.50"/>	<input type="text" value="1,464.90"/>	<input type="text" value="2,512.00"/>	<input type="text" value="4,222.60"/>	<input type="text" value="4,434.50"/>
Resistance, ohmCm	<input type="text" value="807.00"/>	<input type="text" value="1,128.00"/>	<input type="text" value="2,296.00"/>	<input type="text" value="1,393.00"/>	<input type="text" value="2,888.00"/>

Table 4-3. Five Soil Resistivity Tests Closest to Pipe 227

4.3 SERA Analysis of Pipes – Loma Prieta Earthquake

The seismic analysis of the Palo Alto water pipes is done in several steps. In Section 4.3, each major step of the analysis is examined, and several of the tables developed by SERA are listed. In Section 4.4, the key findings are listed for all 24 scenario earthquakes. The SERA computer run (see Reference section) provides all the tables for all the scenario earthquakes.

SERA performs the seismic pipe analysis by pressure zone. Palo Alto has 9 pressure zones, and a tenth "zone" is assigned for pipes with "unknown" zone attribute. Table 4-4 lists the length of pipe in each zone. The Pressure Zone Name, like "1 – One" corresponds to the usual Palo Alto naming system, while the Pressure Zone Number is assigned by SERA as an internal tracking number.

PRESSURE ZONES FOR ACTIVE PIPES		
NUMBER	NAME	LENGTH (Feet) LENGTH (Miles)
1	1 – One	880661.00 166.79
2	2 – Two	194380.00 36.81
3	4 – Four	19769.00 3.74
4	3 – Three	36568.00 6.93
5	5 – Five	29151.00 5.52
6	Unknown	5254.00 1.00
7	6 – Six	27527.00 5.21
8	7 – Seven	19380.00 3.67
9	8 – Eight	6808.00 1.29
10	9 – Nine	7181.00 1.36

Table 4-4. Length of Pipe in Each Pressure Zone

The SERA program computes the ground motions at each site and each pipe, for each selected scenario earthquake. Table 4-5 lists the attributes for the San Andreas fault, Santa Cruz segment, including various parameters that define the fault.

```

=====
BEGIN EARTHQUAKE ANALYSIS
=====
***** SELECTED FAULT FOR THIS SCENARIO ANALYSIS IS:      4
        SELECTED MAGNITUDE FOR THIS ANALYSIS IS:         6.90
        SELECTED EPICENTER FOR THIS ANALYSIS IS:         50.00
=====

FAULT DESCRIPTION DATA
=====
FAULT NAME IS: N. San Andreas;SAS
NUMBER OF FAULT POINTS IS: 7
FAULT INPUT COORDINATE SYSTEM: 1. (=1 for DECIMAL DEGREES)
ARL = - SRL + 0.5*IRL*DELRL.      RL = 10**{(AL + BL*M + ARL*SIGL)}
SRL = 2.00
DELRL = 4.00
aL = -2.36 (from GUI selection)
bL = 0.58 (from GUI selection)
sigL = 0.0100 (from GUI selection)
BETA = 0.80 (=ln(10) * Richter b)
ECIN = 10.00 (=Distance between successive locations of rupture zone)
iFault = 1 (=1 for strike slip)
          (=2 for normal)
          (=3 for reverse, thrust)
          (=4 for oblique)
          (=5 for unspecified, generic)
          (=6 for subduction zone interplate)
          (=7 for subduction zone intraplate)
Hseis = 0.80 (depth of top of seismogenic part of the fault, km C9700)
hTop = 0.80 (depth to top of top of the fault C9700 ASK13)
        (focal depth H to subduction event, km Crouse 1990)
        (depth to close boundary of interplate event, km CSZ 2012)
hBot = 20.00 (depth to bottom of seismogenic part of the fault C9700)
        (depth to top of intraplate event, km CSZ 2012)
fAlpha = 90.00 (dip of the fault plane, degrees AS97 ASK13)
        (positive for dip to north or east of surface fault)
WIDTH = 14.0 (width of dipping plane, km)
*** NOTE: Variables AMMIN, AMSTEP, RATE, BETA, COEF set by SERA for SCENARIO ANALYSES

User Selected Attenuation Model(s) Selected for this Scenario
GUI Choice = 7 (= 1 Crustal Sadigh 1997 + Youngs 1997)
              (= 2 Crustal AS97, BJJF97, C9700, S9397 I919495 Equal weighted + Youngs 1997)
              (= 3 Crustal NGA 2013 + Subduction BC Hydro 2012)
              (= 4 Crustal Sadigh 1997 + Subduction Youngs 1997)
              (= 5 Crustal Sadigh 1997 + Moho Bounce at distance)
              (= 6 Central Eastern United States 2002)
              (= 7 Crustal NGA 2013, Equal weighted + CSZ 2012)
              (= 8 Crustal NGA 2013, User weighted + CSZ 2012)

```

Table 4-5. San Andreas Santa Cruz Fault Segment Fault Parameters

Table 4-6 shows various information about the location of the fault, the epicenter, the rupture length, and the distance from Mayfield reservoir to the epicenter. For example, the fault is 62.18 km long, while for a M 6.9 event, the rupture length is 43.85 km long, the epicenter is 36.3 km east and 47 km south of the Mayfield reservoir. The local center is the arithmetic average of the locations of the 25 sites, which is about 1 km from Mayfield reservoir. The "far end" of the rupture (in this case, the northwest end) is 20.07 km east and 31.3 km south of the local center (or 37.18 km from the local center).

```
***** FAULT SEGMENT LOCATIONS FROM LOCAL CENTER (+East +North)
```

LONGITUDE DEGREES	LATITUDE DEGREES	EASTINGS (KM)	NORTHINGS +east +north	EASTINGS FEET	NORTHINGS FEET
-121.4816	36.8059	59.38	-66.65	6274267	1753308
-121.5645	36.8701	52.07	-59.45	6250262	1776947
-121.6520	36.9260	44.34	-53.15	6224917	1797603
-121.7330	36.9890	37.22	-46.07	6201543	1820841
-121.8120	37.0530	30.28	-38.87	6178798	1844455
-121.9070	37.1040	21.92	-33.09	6151356	1863425
-122.0036	37.1762	13.46	-24.94	6123610	1890148

```
***** LOCAL CENTER INFORMATION
```

The geographic center of all Sites considered in this analysis

```
LONGITUDE = -122.1602 Degrees.    LATITUDE = 37.3989 Degrees (Calculated)
EASTINGS  = 6079438 Feet.          NORTHINGS = 1971983 Feet (Calculated)
EASTINGS  = 0.00 KM.               NORTHINGS = 0.00 KM (SERA Local Center)
```

The location of the Mayfield Reservoir

```
LONGITUDE = -122.1561 Degrees.    LATITUDE = 37.4130 Degrees (Calculated from Program Control)
EASTINGS  = 6080711 Feet.          NORTHINGS = 1977104 Feet (Program Control)
EASTINGS  = 0.39 KM.               NORTHINGS = 1.56 KM (from SERA Local Center)
                                   +KM = East; +KM = North; of SERA Local Center
```

- NOTE. THE CUMULATIVE FAULT LENGTH OF THE SELECTED FAULT IS: 62.18 KM

- NOTE, RUPTURE LENGTH (43.85 KM) IS COMPUTED FOR ASSIGNED MAGNITUDE: 6.90

```
EPICENTER INFORMATION
```

For Single Event, Epicenter is 50.0% along the fault's length

Selected energy center of rupture location

```
36.7 KMs +East and -45.5 KMs +North of the Local Center
36.3 KMs +East and -47.0 KMs +North of the Mayfield Reservoir
```

Single Event

Epicenter Longitude	-121.739 Degrees	Latitude	36.994 Degrees
Eastings	36.65 KM East	Northings	-45.48 KM North of Local Center

Near End Longitude	-121.556 Degrees	Latitude	36.863 Degrees
Eastings	52.85 KM East	Northings	-60.22 KM North of Local Center

Table 4-6. San Andreas Santa Cruz Segment Epicenter Information

Table 4-7 lists the hazards included in this particular analysis. In this case (and in all analyses presented in this report), liquefaction (including lateral spreads and settlements), landslide and surface faulting hazards are included, and the earthquake is assumed to occur in the winter time (ground saturated conditions).

Collateral Hazards for this Facility Analysis

Liquefaction / Lateral spreads / Settlements	= 2 (1 = Not Included, 2 = Included)
Liquefaction Bayside Modifications	= 1 (1 = Not Included, 2 = Included)
Landslides	= 2 (1 = Not Included, 2 = Included)
Surface faulting	= 2 (1 = Not Included, 2 = Included)
Wind Loads	= 1 (1 = Not Included, 2 = Included)
Glaze Ice + Concurrent Wind Loads	= 1 (1 = Not Included, 2 = Included)
Rime Ice + Concurrent Wind Loads	= 1 (1 = Not Included, 2 = Included)
Season factor	= 1 (1 = Winter, 2 = Summer)

Table 4-7. Collateral Hazards

Table 4-8 lists the computed motions at each of the 25 sites included in the SERA model. The complete SERA listings also provide the spectral acceleration values at T = 0.1, 0.3, 1.0 and 3.0 seconds, as well as the probability of triggering liquefaction, and if liquefaction occurs, the amount of settlement and lateral spread. All values are listed in terms of median (best estimate, MED) and 84th percentile (median plus one standard deviation +1S). In Table 4-8, the Liquefaction, landslide and surface faulting indices are numbers (1 to 8), with 8 corresponding to "Very High", and 1 corresponding to "None", etc.

SITE ID	SITE Abbrv	SITE Name	DISTANCE FROM FAULT KM	NEHRP Vs30 SOIL CLASS	LIQFC INDEX	LAND-SLIDE INDEX	SURFC FAULT INDEX	PGA (G) Vs(30) MED +1S	PGV (CM/SEC) Vs(30) MED +1S
1	QUARB	Quarry Booster	37.50 X	D 400.	6	3	3	0.10 0.18	11.70 21.08
2	DAHB	Dahl Booster	32.95 X	B 600.	2	3	3	0.10 0.18	10.14 19.17
3	DAHLR	Dahl Reservoir	32.97 X	B 600.	2	3	4	0.10 0.18	10.14 19.16
4	CORTB	Core Madera Booster	36.50 X	D 450.	5	4	3	0.10 0.19	11.66 22.03
5	BORB	Boronda Booster	34.36 X	B 760.	2	4	3	0.09 0.16	7.99 15.09
6	BORNR	Boronda	35.13 X	B 550.	2	4	3	0.10 0.18	10.24 19.36
7	B.IRR	Boronda Irrigation	35.14 X	B 550.	2	4	3	0.10 0.18	10.24 19.35
8	LYTTB	Lytton Booster	41.81 X	D 250.	4	1	2	0.10 0.17	14.69 25.83
9	HALEW	Hale	42.32 X	E 200.	8	2	1	0.11 0.17	16.86 29.21
10	EPRKW	Eleanor Pardee Park	41.36 X	D 270.	4	1	1	0.10 0.17	14.08 24.92
11	RINCW	Rinconada Well	40.66 X	D 280.	5	2	1	0.10 0.18	13.95 24.78
12	MLIBW	Main Library Well	40.61 X	D 280.	5	2	2	0.10 0.18	13.97 24.81
13	PEERW	Peers Park Well	39.65 X	D 300.	4	2	3	0.11 0.18	13.63 24.34
14	SF-CA	California	38.97 X	D 310.	3	2	3	0.11 0.18	13.54 24.21
15	SF-PA	Palo Alto Pipeline	42.13 X	D 260.	5	1	3	0.10 0.17	14.19 25.05
16	FERNW	Fernando	37.98 X	D 320.	3	2	3	0.11 0.18	13.58 24.30
17	MATAW	Matadero Well	37.91 X	D 330.	3	2	3	0.11 0.18	13.31 23.84
18	MAYFR	Mayfield	38.30 X	D 400.	3	3	3	0.10 0.17	11.46 20.65
19	MAYB	Mayfield Booster	38.31 X	D 400.	3	3	3	0.10 0.17	11.46 20.65
20	SF-PA	Page Mill	37.62 X	D 420.	3	3	3	0.10 0.17	11.24 20.27
21	SF-AR	Arastradero	35.93 X	D 350.	3	3	3	0.11 0.19	13.45 24.14
22	PARKR	Park Reservoir	33.83 X	B 760.	1	4	3	0.09 0.16	8.11 15.32
23	PARB	Park Booster	33.83 X	B 760.	1	4	3	0.09 0.16	8.11 15.32
24	CORTR	Corte Madera Res	36.67 X	B 550.	1	4	3	0.10 0.17	9.82 18.56
25	MNTB	Monte Bello Reservoir	31.14 X	B 650.	1	4	3	0.10 0.18	10.03 18.94

Table 4-8. Loma Prieta M 6.9. Ground Motions at Sites (PGA, PGV)

Table 4-8 shows that the common ground motion in Palo Alto would have been about $PGA = 0.10$ to $0.11g$, with near upper bound about $PGA = 0.19g$, and PGVs commonly about 8 to 14 cm/sec, with upper bound about 15 to 29 cm/sec.

Table 4-9 lists the performance of the tanks, pump stations and wells at each of the 25 facility sites in the earthquake. For the current effort, we have not included detailed models in SERA for the actual equipment, tanks, buildings, SCADA equipment, batteries, etc. Should Palo Alto wish to include this information, that can be done in a straightforward manner; and by so doing, a complete picture of what happens to the water system (facilities and buried pipe) can be easily quantified.

SUMMARY BY SITE			Median		Short Cost \$	Long Cost \$	# Items Function Lost	Highest Life Safety	# Items Damaged Number
RESULTS TOTAL FOR ALL ITEMS, ALL POSITIONS			SERA MODEL	PGA g	PGD inch				
SITE ID	ABBRV	NAME							
1	QUARB	Quarry Booster	Yes	0.10	0.00	0.	0.	0.00	0.00
2	DAHB	Dahl Booster	Yes	0.10	0.00	0.	0.	0.00	0.00
3	DAHLR	Dahl Reservoir	Yes	0.10	0.00	1.	16.	0.00	0.04
4	CORTB	Core Madera Booster	Yes	0.10	0.00	0.	0.	0.00	0.00
5	BORB	Boronda Booster	Yes	0.09	0.00	0.	0.	0.00	0.00
6	BORNRR	Boronda	Yes	0.10	0.00	0.	0.	0.00	0.00
7	B.IRR	Boronda Irrigation	Yes	0.10	0.00	0.	0.	0.00	0.00
8	LYTTB	Lytton Booster	Yes	0.10	0.00	0.	0.	0.00	0.00
9	HALEW	Hale	Yes	0.11	0.00	0.	0.	0.00	0.00
10	EPRKW	Eleanor Pardee Park	Yes	0.10	0.00	0.	0.	0.00	0.00
11	RINCW	Rinconada Well	Yes	0.10	0.00	0.	0.	0.00	0.00
12	MLIBW	Main Library Well	Yes	0.10	0.00	0.	0.	0.00	0.00
13	PEERW	Peers Park Well	Yes	0.11	0.00	0.	0.	0.00	0.00
14	SF-CA	California	Yes	0.11	0.00	0.	0.	0.00	0.00
15	SF-PA	Palo Alto Pipeline	Yes	0.10	0.00	0.	0.	0.00	0.00
16	FERNW	Fernando	Yes	0.11	0.00	0.	0.	0.00	0.00
17	MATAW	Matadero Well	Yes	0.11	0.00	0.	0.	0.00	0.00
18	MAYFR	Mayfield	Yes	0.10	0.00	0.	0.	0.00	0.00
19	MAYB	Mayfield Booster	Yes	0.10	0.00	0.	0.	0.00	0.00
20	SF-PA	Page Mill	Yes	0.10	0.00	0.	0.	0.00	0.00
21	SF-AR	Arastradero	Yes	0.11	0.00	0.	0.	0.00	0.00
22	PARKR	Park Reservoir	Yes	0.09	0.00	0.	0.	0.00	0.00
23	PARB	Park Booster	Yes	0.09	0.00	0.	0.	0.00	0.00
24	CORTR	Corte Madera Res	Yes	0.10	0.00	0.	0.	0.00	0.00
25	MNTB	Monte Bello Reservoir	Yes	0.10	0.00	1.	18.	0.00	0.04

Table 4-9. Loma Prieta M 6.9. Performance of Facilities

Table 4-10 lists the ground motions and potential for a triggered landslide; the amount of movement of each triggered landslide, and given that movement, the probability of pipeline failure. Table 4-10 lists the results for the first 25 pipes (out of 7,315 pipes) in the water system; the SERA output lists the results for all 7,315 pipes. For this earthquake, SERA predicts no triggered landslides (in the actual earthquake, no landslides are known to have occurred along the pipeline alignments).

PIPE Object ID	PIPE DIAM TYPE	YEAR	DISTANCE TO FAULT (KM)	NEHRP CLASS	C10 SLIDE INDX	PGA (G)					PGV (CM/SEC)					PROB LAND- SLIDE			PGD (INCHES)			P(FAIL)- LANDSLIDE		
						ROCK MED	A,B- +1S	SOIL MED	C- +1S	SOIL MED	D- +1S	ROCK MED	A,B- +1S	SOIL MED	C- +1S	SOIL MED	D- +1S	LAND- SLIDE	MED	+1S	LANDSLIDE MED	+1S	LANDSLIDE MED	+1S
1	6 DIP	1996	40.70	D	3	0.07	0.13	0.09	0.16	0.10	0.17	6.1	11.2	8.6	15.7	11.5	20.8	0.000	0.0	0.0	0.000	0.000	0.000	0.000
2	6 ACP	1971	39.82	D	3	0.07	0.13	0.09	0.16	0.10	0.18	6.3	11.5	8.8	16.0	11.7	21.2	0.000	0.0	0.0	0.000	0.000	0.000	0.000
3	8 ACP	1947	39.40	D	3	0.08	0.13	0.09	0.16	0.11	0.18	6.3	11.6	8.9	16.2	11.9	21.4	0.000	0.0	0.0	0.000	0.000	0.000	0.000
4	8 CIP	0	42.08	D	3	0.07	0.12	0.09	0.15	0.10	0.17	5.9	10.9	8.3	15.2	11.2	20.2	0.000	0.0	0.0	0.000	0.000	0.000	0.000
5	8 ACP	0	38.76	D	3	0.08	0.13	0.09	0.16	0.11	0.18	6.4	11.8	9.0	16.4	12.1	21.8	0.000	0.0	0.0	0.000	0.000	0.000	0.000
6	6 PVC	1994	39.15	D	3	0.08	0.13	0.09	0.16	0.11	0.18	6.4	11.7	8.9	16.3	11.9	21.6	0.000	0.0	0.0	0.000	0.000	0.000	0.000
7	6 PVC	2005	37.18	D	3	0.08	0.14	0.10	0.17	0.11	0.19	6.7	12.3	9.4	17.1	12.5	22.7	0.000	0.0	0.0	0.000	0.000	0.000	0.000
8	6 PVC	1995	38.78	D	3	0.08	0.13	0.09	0.16	0.11	0.18	6.4	11.8	9.0	16.4	12.1	21.8	0.000	0.0	0.0	0.000	0.000	0.000	0.000
9	6 ACP	1949	40.89	D	3	0.07	0.13	0.09	0.16	0.10	0.17	6.1	11.2	8.5	15.6	11.5	20.7	0.000	0.0	0.0	0.000	0.000	0.000	0.000
10	6 PVC	2002	38.96	D	3	0.08	0.13	0.09	0.16	0.11	0.18	6.4	11.7	9.0	16.4	12.0	21.7	0.000	0.0	0.0	0.000	0.000	0.000	0.000
11	6 ACP	0	35.38	C	3	0.08	0.15	0.10	0.18	0.12	0.20	7.0	12.8	9.8	17.9	13.1	23.6	0.000	0.0	0.0	0.000	0.000	0.000	0.000
12	8 HDPE	2010	41.45	D	3	0.07	0.13	0.09	0.15	0.10	0.17	6.0	11.0	8.4	15.4	11.3	20.4	0.000	0.0	0.0	0.000	0.000	0.000	0.000
13	8 HDPE	0	41.42	D	3	0.07	0.13	0.09	0.15	0.10	0.17	6.0	11.0	8.4	15.4	11.3	20.4	0.000	0.0	0.0	0.000	0.000	0.000	0.000
14	8 HDPE	2010	41.57	D	3	0.07	0.12	0.09	0.15	0.10	0.17	6.0	11.0	8.4	15.3	11.3	20.4	0.000	0.0	0.0	0.000	0.000	0.000	0.000
15	8 HDPE	2010	41.33	D	3	0.07	0.13	0.09	0.15	0.10	0.17	6.0	11.0	8.5	15.4	11.3	20.5	0.000	0.0	0.0	0.000	0.000	0.000	0.000
16	6 CIP	0	41.28	D	3	0.07	0.13	0.09	0.15	0.10	0.17	6.0	11.1	8.5	15.4	11.3	20.5	0.000	0.0	0.0	0.000	0.000	0.000	0.000
17	8 HDPE	2010	41.41	D	3	0.07	0.13	0.09	0.15	0.10	0.17	6.0	11.0	8.4	15.4	11.3	20.4	0.000	0.0	0.0	0.000	0.000	0.000	0.000
18	8 HDPE	2010	41.41	D	3	0.07	0.13	0.09	0.15	0.10	0.17	6.0	11.0	8.4	15.4	11.3	20.4	0.000	0.0	0.0	0.000	0.000	0.000	0.000
19	8 HDPE	2010	41.63	D	3	0.07	0.12	0.09	0.15	0.10	0.17	6.0	11.0	8.4	15.3	11.3	20.3	0.000	0.0	0.0	0.000	0.000	0.000	0.000
20	8 HDPE	2010	41.63	D	3	0.07	0.12	0.09	0.15	0.10	0.17	6.0	11.0	8.4	15.3	11.3	20.3	0.000	0.0	0.0	0.000	0.000	0.000	0.000
21	8 HDPE	2010	41.63	D	3	0.07	0.12	0.09	0.15	0.10	0.17	6.0	11.0	8.4	15.3	11.3	20.3	0.000	0.0	0.0	0.000	0.000	0.000	0.000
22	6 PVC	1998	41.60	D	3	0.07	0.12	0.09	0.15	0.10	0.17	6.0	11.0	8.4	15.3	11.3	20.4	0.000	0.0	0.0	0.000	0.000	0.000	0.000
23	8 HDPE	0	41.63	D	3	0.07	0.12	0.09	0.15	0.10	0.17	6.0	11.0	8.4	15.3	11.3	20.3	0.000	0.0	0.0	0.000	0.000	0.000	0.000
24	8 HDPE	2010	41.63	D	3	0.07	0.12	0.09	0.15	0.10	0.17	6.0	11.0	8.4	15.3	11.3	20.3	0.000	0.0	0.0	0.000	0.000	0.000	0.000
25	8 HDPE	2010	41.64	D	3	0.07	0.12	0.09	0.15	0.10	0.17	6.0	11.0	8.4	15.3	11.2	20.3	0.000	0.0	0.0	0.000	0.000	0.000	0.000

Table 4-10. Loma Prieta M 6.9. Performance of Pipelines due to Landslide

Table 4-11a,b is divided into two sections, so that the font size remains legible. The table lists the rate of damage due to ground shaking, liquefaction, landslide and surface faulting for the first 25 pipes; and the rightmost two columns list the number of repairs for that pipe.

PIPELINE SHAKING AND PGD ANALYSES											SHAKING ONLY	
PIPE ID	FACILITY ID	DIAMETER inches	MATERIAL	YEAR	STATUS	LENGTH feet	Median PGV In/Sec	LIQFN PGD Inch	SLIDE PGD Inch	FAULT PGD Inch	REPAIR RATE Median	84th
1	249551	6	DIP	1996	ACTIVE	1.	4.5	0.0	0.0	0.0	0.002413	0.004363
2	249559	6	ACP	1971	ACTIVE	32.	4.6	0.0	0.0	0.0	0.002465	0.004455
3	249560	8	ACP	1947	ACTIVE	184.	4.7	0.0	0.0	0.0	0.002490	0.004501
4	249561	0	CIP	0	ACTIVE	2.	4.4	0.0	0.0	0.0	0.007803	0.014112
5	249562	8	ACP	0	ACTIVE	659.	4.7	0.0	0.0	0.0	0.002529	0.004571
6	249573	6	PVC	1994	ACTIVE	46.	4.7	0.0	0.0	0.0	0.004176	0.007547
7	249574	6	PVC	2005	ACTIVE	32.	4.9	0.0	0.0	0.0	0.004389	0.007929
8	249576	6	PVC	1995	ACTIVE	636.	4.7	0.0	0.0	0.0	0.004214	0.007615
9	249578	6	ACP	1949	ACTIVE	41.	4.5	0.0	0.0	0.0	0.002403	0.004343
10	249579	6	PVC	2002	ACTIVE	550.	4.7	0.0	0.0	0.0	0.004196	0.007582
11	249584	6	ACP	0	ACTIVE	192.	3.9	0.0	0.0	0.0	0.002060	0.003755
12	260609	8	HDPE	2010	ACTIVE	234.	4.4	0.0	0.0	0.0	0.000395	0.000714
13	260610	8	HDPE	0	ACTIVE	29.	4.5	0.0	0.0	0.0	0.000395	0.000715
14	260618	8	HDPE	2010	ACTIVE	16.	4.4	0.0	0.0	0.0	0.000394	0.000713
15	260623	8	HDPE	2010	ACTIVE	9.	4.5	0.0	0.0	0.0	0.000396	0.000716
16	260625	6	CIP	0	ACTIVE	675.	4.5	0.0	0.0	0.0	0.007935	0.014347
17	260627	8	HDPE	2010	ACTIVE	8.	4.5	0.0	0.0	0.0	0.000396	0.000715
18	260628	8	HDPE	2010	ACTIVE	16.	4.5	0.0	0.0	0.0	0.000396	0.000715
19	260668	8	HDPE	2010	ACTIVE	7.	4.4	0.0	0.0	0.0	0.000393	0.000711
20	260670	8	HDPE	2010	ACTIVE	5.	4.4	0.0	0.0	0.0	0.000393	0.000711
21	260671	8	HDPE	2010	ACTIVE	20.	4.4	0.0	0.0	0.0	0.000394	0.000712
22	260673	6	PVC	1998	ACTIVE	293.	4.4	0.0	0.0	0.0	0.003938	0.007120
23	260674	8	HDPE	0	ACTIVE	17.	4.4	0.0	0.0	0.0	0.000393	0.000711
24	260675	8	HDPE	2010	ACTIVE	7.	4.4	0.0	0.0	0.0	0.000393	0.000711
25	260679	8	HDPE	2010	ACTIVE	41.	4.4	0.0	0.0	0.0	0.000393	0.000711

Table 4-11a. Loma Prieta M 6.9. Repair Rate for Pipes due to Shaking (per 1,000 feet)

SHAKING ONLY REPAIR RATE ---		LIQUEFACTION REPAIR RATE ---		LANDSLIDE REPAIR RATE ---		FAULTING NUMBER OF RPRS---		COMBINED ALL HAZARD NUMBER OF REPAIRS	
Median	84th	Median	84th	Median	84th	Median	84th	Median	84th
0.002413	0.004363	0.000000	0.000000	0.000000	0.000000	0.00	0.00	0.000	0.000
0.002465	0.004455	0.000000	0.000000	0.000000	0.000000	0.00	0.00	0.000	0.000
0.002490	0.004501	0.000000	0.000000	0.000000	0.000000	0.00	0.00	0.000	0.001
0.007803	0.014112	0.000000	0.000000	0.000000	0.000000	0.00	0.00	0.000	0.000
0.002529	0.004571	0.000000	0.000000	0.000000	0.000000	0.00	0.00	0.002	0.003
0.004176	0.007547	0.000000	0.000000	0.000000	0.000000	0.00	0.00	0.000	0.000
0.004389	0.007929	0.000000	0.000000	0.000000	0.000000	0.00	0.00	0.000	0.000
0.004214	0.007615	0.000000	0.000000	0.000000	0.000000	0.00	0.00	0.003	0.005
0.002403	0.004343	0.000000	0.000000	0.000000	0.000000	0.00	0.00	0.000	0.000
0.004196	0.007582	0.000000	0.000000	0.000000	0.000000	0.00	0.00	0.002	0.004
0.002060	0.003755	0.000000	0.000000	0.000000	0.000000	0.00	0.00	0.000	0.001
0.000395	0.000714	0.000000	0.000000	0.000000	0.000000	0.00	0.00	0.000	0.000
0.000395	0.000715	0.000000	0.000000	0.000000	0.000000	0.00	0.00	0.000	0.000
0.000394	0.000713	0.000000	0.000000	0.000000	0.000000	0.00	0.00	0.000	0.000
0.000396	0.000716	0.000000	0.000000	0.000000	0.000000	0.00	0.00	0.000	0.000
0.007935	0.014347	0.000000	0.000000	0.000000	0.000000	0.00	0.00	0.005	0.010
0.000396	0.000715	0.000000	0.000000	0.000000	0.000000	0.00	0.00	0.000	0.000
0.000396	0.000715	0.000000	0.000000	0.000000	0.000000	0.00	0.00	0.000	0.000
0.000393	0.000711	0.000000	0.000000	0.000000	0.000000	0.00	0.00	0.000	0.000
0.000393	0.000711	0.000000	0.000000	0.000000	0.000000	0.00	0.00	0.000	0.000
0.000394	0.000712	0.000000	0.000000	0.000000	0.000000	0.00	0.00	0.000	0.000
0.003938	0.007120	0.000000	0.000000	0.000000	0.000000	0.00	0.00	0.001	0.002
0.000393	0.000711	0.000000	0.000000	0.000000	0.000000	0.00	0.00	0.000	0.000
0.000393	0.000711	0.000000	0.000000	0.000000	0.000000	0.00	0.00	0.000	0.000
0.000393	0.000711	0.000000	0.000000	0.000000	0.000000	0.00	0.00	0.000	0.000
0.000393	0.000711	0.000000	0.000000	0.000000	0.000000	0.00	0.00	0.000	0.000

Table 4-11b. Loma Prieta M 6.9. Number of Pipe Repairs – First 24 Pipes

For example, pipe 3 (GID 249560) is a 8" ACP, built in 1947, 184 feet long. It is exposed to 4.7 cm/sec PGV (median), leading to a repair rate of 0.00249 per 1,000 feet, or 0.000458 repairs over 184 feet. It is not exposed to any PGDs from liquefaction, landslide or surface faulting. In other words, there is about a 1 chance in 2,000 (rounded)

that this pipe would have suffered damage in a repeat of the 1989 Loma Prieta earthquake.

Table 4-12 lists the total number of repairs expected to all of Palo Alto's water pipelines. This shows a total of 3.8 repairs (best estimate) to 6.9 repairs (84th percentile), all due to ground shaking, and none due to liquefaction, landslide or surface faulting. While the listed number values are to 6 decimal places, that is only for formatting purposes, and the values to be interpreted is "about 3.8 to 6.9" pipe repairs between the 50th and 84th percentile results. If one wanted to estimate the 16th percentile, a simple way to do that is to take the median value (3.8) times the ratio of (median to 84th), or $(3.8) * (3.8/6.9) = 2.1$. The Palo Alto leak database begins only in 1990, and from available information, there were a handful of buried pipe breaks in Palo Alto in the actual 1989 Loma Prieta earthquake (see the map in Section 3.8.2 for details). In any case, this projected handful of leaks would present not much of a problem to the water system as a whole, and appears to be reasonable.

The number of ACTIVE pipe repairs, system-wide:				
SHAKING	ONLY: Median:	3.811944	Repairs	(leaks or breaks)
SHAKING	ONLY: 84th :	6.904297	Repairs	(leaks or breaks)
LIQUEFACTION	ONLY: Median:	0.000000	Repairs	(leaks or breaks)
LIQUEFACTION	ONLY: 84th :	0.000000	Repairs	(leaks or breaks)
LANDSLIDE	ONLY: Median:	0.000000	Repairs	(leaks or breaks)
LANDSLIDE	ONLY: 84th :	0.000000	Repairs	(leaks or breaks)
FAULTING	ONLY: Median:	0.000000	Repairs	(leaks or breaks)
FAULTING	ONLY: 84th :	0.000000	Repairs	(leaks or breaks)
ALL HAZARDS	: Median:	3.811944	Repairs	(leaks or breaks)
ALL HAZARDS	: 84th :	6.904297	Repairs	(leaks or breaks)

Table 4-12. Loma Prieta M 6.9. Number of Pipe Repairs – All Pipes (needs update)

Table 4-13 lists the length of pipe subject to liquefaction and landslide. For the 1989 Loma Prieta earthquake, it is nil.

Length of Pipe (ACTIVE) exposed to Liquefaction PGDs		
Zero PGDs	: 1226679. Feet	232.33 Miles
0 to 1 Inches:	0. Feet	0.00 Miles
1 to 4 Inches:	0. Feet	0.00 Miles
4 to 8 Inches:	0. Feet	0.00 Miles
8 to 12 Inches:	0. Feet	0.00 Miles
12 to 24 Inches:	0. Feet	0.00 Miles
24+ Inches:	0. Feet	0.00 Miles

Length of Pipe (ACTIVE) exposed to Landslide PGDs		
Zero PGDs	: 1226679. Feet	232.33 Miles
0 to 1 Inches:	0. Feet	0.00 Miles
1 to 4 Inches:	0. Feet	0.00 Miles
4 to 8 Inches:	0. Feet	0.00 Miles
8 to 12 Inches:	0. Feet	0.00 Miles
12 to 24 Inches:	0. Feet	0.00 Miles
24+ Inches:	0. Feet	0.00 Miles

Table 4-13. Loma Prieta M 6.9. Length of Pipe Exposed to Liquefaction or Landslide

Table 4-14 provides a breakdown of the pipe repairs for ACP by pressure zone (median values). A pipe repair can be either a "leak" or a "break". The term "active" refers to pipes that are currently "active" in the city, and excludes pipes that are abandoned.

PIPE REPAIRS BY PRESSURE ZONE, ACTIVE PIPES, TYPE = ACP						
ZONE NUMBER	ZONE NAME	LENGTH (Miles)	ACP ONLY (Miles)	TOTAL REPAIRS	PIPE LEAKS	PIPE BREAKS
1	1 - One	166.79	97.42	1.30	1.05	0.24
2	2 - Two	36.81	22.25	0.24	0.20	0.05
3	4 - Four	3.74	3.47	0.02	0.01	0.01
4	3 - Three	6.93	3.32	0.03	0.02	0.01
5	5 - Five	5.52	3.05	0.01	0.01	0.01
6	Unknown	1.00	0.41	0.01	0.01	0.00
7	6 - Six	5.21	2.13	0.02	0.01	0.00
8	7 - Seven	3.67	2.35	0.02	0.01	0.00
9	8 - Eight	1.29	0.15	0.00	0.00	0.00
10	9 - Nine	1.36	0.00	0.00	0.00	0.00
TOTALS				1.65	1.33	0.32

Table 4-14. Loma Prieta M 6.9. Number of Pipe Repairs – AC Pipe, By Pressure Zone

Table 4-15 further breaks this down by diameter, for leaks (first table) and full breaks (second table).

PIPE ZONE	LEAKS BY PRESSURE ZONE,	ACTIVE PIPES,	DIAMETER	IN INCHES,	***	TYPE =	ACP						
ZONE	ZONE	1-4	6	8	10	12	14	16	18	20	21-30	SFPUC	UNKNOWN
1	1 - One	0.06	0.54	0.35	0.05	0.03	0.01	0.01	0.00	0.00	0.00	0.00	0.00
2	2 - Two	0.01	0.11	0.07	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3	4 - Four	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4	3 - Three	0.00	0.00	0.01	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
5	5 - Five	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
6	Unknown	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
7	6 - Six	0.00	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
8	7 - Seven	0.00	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
9	8 - Eight	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
10	9 - Nine	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
TOTALS		0.08	0.66	0.45	0.07	0.04	0.01	0.01	0.00	0.00	0.00	0.00	0.00

PIPE ZONE	BREAKS BY PRESSURE ZONE,	ACTIVE PIPES,	DIAMETER	IN INCHES,	***	TYPE =	ACP						
ZONE	ZONE	1-4	6	8	10	12	14	16	18	20	21-30	SFPUC	UNKNOWN
1	1 - One	0.01	0.11	0.07	0.01	0.03	0.01	0.01	0.00	0.00	0.00	0.00	0.00
2	2 - Two	0.00	0.02	0.01	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3	4 - Four	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4	3 - Three	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
5	5 - Five	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
6	Unknown	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
7	6 - Six	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
8	7 - Seven	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
9	8 - Eight	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
10	9 - Nine	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
TOTALS		0.02	0.13	0.09	0.01	0.04	0.01	0.01	0.00	0.00	0.00	0.00	0.00

Table 4-15. Loma Prieta M 6.9. Number of Pipe Repairs – AC Pipe, By Pressure Zone, By Diameter

The full SERA output provides similar tables all pipe types in the system, including CCP, CIP, Cu, DIP, HDPE, PVC, Steel, Unknown.

SERA then provides a set of summary tables, over all pipe types, Table 4-16 and 4-17. Table 4-16 shows that of the 3.81 forecast pipe repairs, most will be leaks (like a service lateral, air valve, pin hole, etc.), and possibly 1 will be a full break. Most of the damage will be in Zone 1, possibly 1 leak in Zone 2, and small chance elsewhere in the system.

PIPE REPAIRS BY PRESSURE ZONE, ACTIVE PIPES, ALL PIPE MATERIALS					
PIPE ZONE	ZONE NAME	LENGTH (Miles)	TOTAL REPAIRS	PIPE LEAKS	PIPE BREAKS
1	1 - One	166.79	3.04	2.46	0.58
2	2 - Two	36.81	0.52	0.42	0.10
3	4 - Four	3.74	0.02	0.02	0.01
4	3 - Three	6.93	0.08	0.06	0.03
5	5 - Five	5.52	0.03	0.02	0.01
6	Unknown	1.00	0.04	0.03	0.01
7	6 - Six	5.21	0.04	0.03	0.01
8	7 - Seven	3.67	0.03	0.02	0.00
9	8 - Eight	1.29	0.01	0.00	0.00
10	9 - Nine	1.36	0.01	0.00	0.00
TOTALS			3.81	3.05	0.76

Table 4-16. Loma Prieta M 6.9. Number of Pipe Repairs – All Pipe Materials, By Pressure Zone

Table 4-17 shows that there would be about 2 to 3 repairs to 6" pipe, 1 to 2 for 8" pipe, etc. There is a very small chance of damage to the larger transmission pipes.

PIPE REPAIRS BY PRESSURE ZONE, ACTIVE PIPES, DIAMETER IN INCHES, ALL PIPE MATERIALS													
PIPE ZONE	ZONE	1-4	6	8	10	12	14	16	18	20	21-30	SFPUC	UNKNOWN
1	1 - One	0.12	1.38	0.64	0.16	0.06	0.01	0.02	0.00	0.00	0.02	0.00	0.05
2	2 - Two	0.02	0.19	0.16	0.02	0.01	0.00	0.00	0.00	0.00	0.01	0.00	0.00
3	4 - Four	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4	3 - Three	0.00	0.00	0.01	0.01	0.02	0.00	0.00	0.00	0.00	0.00	0.00	0.01
5	5 - Five	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.00	0.00	0.00	0.00
6	Unknown	0.00	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01
7	6 - Six	0.00	0.00	0.02	0.00	0.00	0.00	0.00	0.01	0.00	0.00	0.00	0.00
8	7 - Seven	0.00	0.00	0.02	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
9	8 - Eight	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
10	9 - Nine	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
TOTALS		0.14	1.58	0.87	0.20	0.09	0.02	0.03	0.02	0.00	0.03	0.00	0.07

PIPE REPAIRS BY PRESSURE ZONE, ACTIVE PIPES, DIAMETER IN INCHES, ALL PIPE MATERIALS													
PIPE ZONE	ZONE	1-4	6	8	10	12	14	16	18	20	21-30	SFPUC	UNKNOWN
1	1 - One	0.02	0.28	0.13	0.03	0.06	0.01	0.02	0.00	0.00	0.02	0.00	0.05
2	2 - Two	0.00	0.04	0.03	0.00	0.01	0.00	0.00	0.00	0.00	0.01	0.00	0.00
3	4 - Four	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4	3 - Three	0.00	0.00	0.00	0.00	0.02	0.00	0.00	0.00	0.00	0.00	0.00	0.01
5	5 - Five	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.00	0.00	0.00	0.00
6	Unknown	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01
7	6 - Six	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.00	0.00	0.00	0.00
8	7 - Seven	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
9	8 - Eight	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
10	9 - Nine	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
TOTALS		0.03	0.32	0.17	0.04	0.09	0.02	0.03	0.02	0.00	0.03	0.00	0.07

Table 4-17. Loma Prieta M 6.9. Number of Pipe Repairs – All Pipe Materials, By Pressure Zone, By Diameter

Table 4-18 shows the forecast hydraulic response due to the leaking and broken pipes.

As part of this computation, the average day demand for Palo Alto is assumed to be 13.8 MGD. This demand is then allocated by pressure zone, as a ratio of the mileage of pipe in that pressure zone, to the total mileage (232.8 miles) in the city as a whole.

The number of "equivalent" 6"-diameter breaks is then computed for each pressure zone, using the pipe break and leak data from Table 4-17. The diameter of the damage pipe is considered by assuming that a 4" pipe break would release about $(4/6)^2$ as much water as a 6" pipe, etc., and a leak would leak about 2% as much water as a 6" break. Then the water leakage rate by pressure zone is assumed to be 1000 gpm for each equivalent 6 inch break \wedge expT. The expT value (see Table 4-18) is 1.0 for large networks with few breaks, but less than 1 if there are multiple breaks in a zone.

Over all zones, the leak rate is about 2,241 gpm (3.2 MGD). Given this leak rate, and the network connectivity, it is likely that 100% of Palo Alto customers would remain in service, as long as there was sufficient supply. Soon after the earthquake, Palo Alto repair crews would turn valves to isolate these leaks, and then make repairs. For this earthquake, the SERA model forecasts that essentially 100% of Palo Alto customers would have water before the valves are shut, and 100% after the vales are shut (in this computation, it is assumed there is sufficient supply to meet both the leak rate as well as the regular demand rate).

PRESSURE ZONE HYDRAULIC RESPONSE									
ZONE NUMBER	ZONE NAME	LENGTH (Miles)	BREAK EQUIV	BREAK RATE/KFT	expT	FLOW (gpm) NO VALVE	VALVE OPEN	VALVE CLOSED	Sdv
1	1 - One	166.79	1.54	0.0017	1.000	1539.6	1.00	1.00	
2	2 - Two	36.81	0.34	0.0018	1.000	342.3	1.00	1.00	
3	4 - Four	3.74	0.02	0.0011	1.000	21.0	1.00	1.00	
4	3 - Three	6.93	0.10	0.0027	1.000	98.2	1.00	1.00	
5	5 - Five	5.52	0.09	0.0032	1.000	94.7	1.00	1.00	
6	Unknown	1.00	0.02	0.0038	1.000	20.1	1.00	1.00	
7	6 - Six	5.21	0.08	0.0029	1.000	79.5	1.00	1.00	
8	7 - Seven	3.67	0.01	0.0007	1.000	14.1	1.00	1.00	
9	8 - Eight	1.29	0.02	0.0023	1.000	15.8	1.00	1.00	
10	9 - Nine	1.36	0.02	0.0022	1.000	15.5	1.00	1.00	
Percentage of all customers with service, immediately post-earthquake =						100.00 %			
Percentage of all customers with service, once valves are closed =						100.00 %			

Table 4-18. Loma Prieta M 6.9. Hydraulic Response, By Pressure Zone

4.4 SERA Results for 24 Scenario Earthquake

We computed the response of the Palo Alto water system for 24 scenario earthquakes. Table 4-19 lists the key findings. Results are for the PAL.RevC model, including calibration from the Napa earthquake. Complete listing of the pipe damage, by diameter, by pressure zone, by material, are included in the full computer listing in SERA.14.46.21.out.

EQ	Fault	M	Pipe Repairs Median	Pipe Repairs 84th	Sd (T = 0) Median	Sdv Valve Closed Median
1	San Andreas Santa Cruz	6.9	3.8	6.9	100.0	100.0
2	San Andreas Peninsula	6.0	5.1	19.9	98.4	99.2
3	San Andreas Peninsula	6.2	14.4	54.8	96.5	98.3
4	San Andreas Peninsula	6.4	29.3	79.0	87.6	94.5
5	San Andreas Peninsula	6.6	45.0	105.2	71.6	90.6
6	San Andreas Peninsula	6.8	60.9	131.6	57.0	84.6
7	San Andreas Peninsula	7.0	78.7	157.9	56.2	83.8
8	San Andreas SAN+SAP+SAS	7.2	77.2	158.4	57.0	84.6
9	San Andreas SAN+SAP+SAS	7.4	125.0	217.4	34.4	79.2
10	San Andreas SAN+SAP+SAS	7.5	139.5	239.1	34.3	78.6
11	San Andreas SAN+SAP+SAS	7.7	168.6	283.7	26.9	74.7
12	San Andreas SAN+SAP+SAS	7.9	198.3	328.5	23.6	73.0
13	San Andreas SAN+SAP+SAS	8.0	215.2	350.5	19.8	63.7
14	Hayward N+S	7.25	53.7	127.6	59.6	87.0
15	Hayward South	6.8	24.2	72.8	91.6	96.9
16	Hayward North	6.8	3.0	5.5	100.0	100.0
17	West Napa	6.0	0.0	0.6	100.0	100.0
18	Rodgers Creek	7.0	2.1	3.8	100.0	100.0
19	Calaveras North + Central + South	7.2	31.0	86.7	75.9	93.3
20	San Gregorio	7.7	77.8	168.5	57.0	84.6
21	Mount Diablo Thrust	6.5	2.1	3.9	100.0	100.0
22	Monta Vista	6.8	92.0	171.8	55.5	83.6
23	Greenville	7.0	3.5	6.3	100.0	100.0
24	Zayante – Vergeles	6.9	3.2	5.8	100.0	100.0

Table 4-19. Pipeline Performance, 24 Scenario Earthquakes

EQ	Fault	M	Pipe Repairs Shake Median	Pipe Repairs Shake 84th	Pipe Repairs Liq Median	Pipe Repairs Liq 84th
1	San Andreas Santa Cruz	6.9	3.8	6.9	0.0	0.0
2	San Andreas Peninsula	6.0	2.8	5.0	0.3	14.9
3	San Andreas Peninsula	6.2	4.1	7.3	10.3	47.5
4	San Andreas Peninsula	6.4	6.0	10.6	23.3	68.4
5	San Andreas Peninsula	6.6	8.1	14.5	36.9	90.8
6	San Andreas Peninsula	6.8	10.4	18.3	50.5	113.3
7	San Andreas Peninsula	7.0	12.8	22.5	65.9	135.4
8	San Andreas SAN+SAP+SAS	7.2	11.7	20.6	65.6	137.8
9	San Andreas SAN+SAP+SAS	7.4	18.3	29.5	106.7	187.9
10	San Andreas SAN+SAP+SAS	7.5	20.0	27.0	119.4	211.4
11	San Andreas SAN+SAP+SAS	7.7	23.1	17.3	145.6	265.3
12	San Andreas SAN+SAP+SAS	7.9	22.4	18.9	176.0	307.4
13	San Andreas SAN+SAP+SAS	8.0	18.9	19.9	196.3	327.5
14	Hayward N+S	7.25	8.6	15.4	45.1	112.2
15	Hayward South	6.8	5.2	9.5	19.0	63.4
16	Hayward North	6.8	3.0	5.5	0.0	0.0
17	West Napa	6.0	0.0	0.6	0.0	0.0
18	Rodgers Creek	7.0	2.1	3.8	0.0	0.0
19	Calaveras North + Central + South	7.2	6.4	11.6	24.6	75.1
20	San Gregorio	7.7	11.6	20.7	66.2	147.8
21	Mount Diablo Thrust	6.5	2.1	3.9	0.0	0.0
22	Monta Vista	6.8	13.9	23.9	73.8	138.7
23	Greenville	7.0	3.5	6.3	0.0	0.0
24	Zayante – Vergeles	6.9	3.2	5.8	0.0	0.0

Table 4-20. Pipe Damage due to Ground Shaking or Liquefaction, 24 Scenario Earthquakes

For the San Andreas M 8.0, 1 pipe is expected to fail due to landslide (3 pipes at 84th percentile motions), assuming all pipes have a median fragility of 10 inches of movement due to landslide. A few slides of 2 to 3 feet of movement are expected. The lower pipe repairs due to shaking for the San Andreas 8.0, 7.9 and 7.7 scenarios is a result of more damage that could be due to shaking or liquefaction, being assigned to liquefaction.

For the Monta Vista Shannon M 6.8 earthquake:

- Surface faulting in the Foothills is expected. The surface faulting will likely break the Foothills 18" CCP pipe, most likely in about 4 locations. Lacking mitigation,
- Several Palo Alto tanks and pump stations will be on the hanging wall side of this thrust-type earthquake, including Boronda Reservoir, Boronda Booster and Park Reservoir and Park Booster, and Corte Madera Booster and Corte Madera reservoir. These sites could experience very high levels of ground shaking (median PGA in the range of 0.9g to 1.3g, 84th percentile in the range of 1.7g to 2.3g). While this is thought to be a rare event (return period in 1000s of years), the recent design upgrades for these reservoirs presumed motions in the range of 0.6g, and there would be insufficient margin in the design to accommodate this type of earthquake; some failures might be expected.

4.5 SERA Results for the San Andreas M 8.0 Earthquake

Tables 4-19 and 4-20 show that the San Andreas M 8.0 earthquake is the worst possible earthquake that could impact the Palo Alto water system. Recognizing this, some additional details are provided for this event; similar tables are available in the full SERA output listing, for all 24 scenario earthquakes. Table 4-21 lists the ground motions at sites due to the San Andreas M 8.0 earthquake (MCE).

SITE ID	SITE Abbrv	SITE Name	DISTANCE FROM FAULT KM	NEHRP SOIL CLASS	Vs30 m/sec	LIQFC INDEX	LAND-SLIDE INDEX	SURF FAULT INDEX	PGA (G) Vs(30) MED +1S	PGV (CM/SEC) Vs(30) MED +1S
1	QUARB	Quarry Booster	6.18	X	D 400.	6	3	3	0.44 0.72	85.17 152.41
2	DAHB	Dahl Booster	0.34	X	B 600.	2	3	3	0.68 1.20	93.24 173.03
3	DAHLR	Dahl Reservoir	0.33	X	B 600.	2	3	4	0.68 1.20	93.24 173.03
4	CORTB	Core Madera Booster	3.22	X	D 450.	5	4	3	0.64 1.11	97.39 180.33
5	BORB	Boronda Booster	1.34	X	B 760.	2	4	3	0.58 1.02	73.06 135.69
6	BORNR	Boronda	2.34	X	B 550.	2	4	3	0.63 1.10	88.89 164.89
7	B.IRR	Boronda Irrigation	2.35	X	B 550.	2	4	3	0.63 1.09	88.83 164.77
8	LYTTB	Lytton Booster	8.40	X	D 250.	4	1	2	0.37 0.57	90.18 154.46
9	HALEW	Hale	9.98	X	E 200.	8	2	1	0.32 0.49	88.33 147.58
10	EPRKW	Eleanor Pardee Park	10.53	X	D 270.	4	1	1	0.35 0.55	78.43 135.89
11	RINCW	Rinconada Well	10.15	X	D 280.	5	2	1	0.36 0.57	78.86 137.26
12	MLIBW	Main Library Well	10.35	X	D 280.	5	2	2	0.35 0.56	78.04 135.85
13	PEERW	Peers Park Well	8.93	X	D 300.	4	2	3	0.38 0.61	82.02 143.98
14	SF-CA	California	8.46	X	D 310.	3	2	3	0.39 0.62	83.15 146.33
15	SF-PA	Palo Alto Pipeline	8.27	X	D 260.	5	1	3	0.37 0.59	89.75 154.54
16	FERNW	Fernando	9.08	X	D 320.	3	2	3	0.38 0.61	79.11 139.58
17	MATAW	Matadero Well	8.71	X	D 330.	3	2	3	0.39 0.63	79.74 141.02
18	MAYFR	Mayfield	6.93	X	D 400.	3	3	3	0.42 0.70	80.96 144.87
19	MAYB	Mayfield Booster	6.96	X	D 400.	3	3	3	0.42 0.69	80.77 144.54
20	SF-PA	Page Mill	6.47	X	D 420.	3	3	3	0.43 0.71	80.89 144.97
21	SF-AR	Arastradero	7.87	X	D 350.	3	3	3	0.40 0.65	81.58 144.82
22	PARKR	Park Reservoir	1.13	X	B 760.	1	4	3	0.59 1.03	73.94 137.32
23	PARB	Park Booster	1.11	X	B 760.	1	4	3	0.59 1.03	74.00 137.43
24	CORTR	Corte Madera Res	2.30	X	B 550.	1	4	3	0.63 1.10	89.10 165.27
25	MNTB	Monte Bello Reservoir	0.00	X	B 650.	1	4	3	0.67 1.18	88.58 164.44

Table 4-21. San Andreas M 8.0. Ground Motions at Sites (PGA, PGV)

Figure 4-1 shows the pattern of pipe damage in Palo Alto. The color coding is in terms of "repair per 1,000 feet of pipe", with red being the highest (widespread along San Francisquito Creek and the SF Bay margins for cast iron, ductile iron, asbestos cement or PVC pipes), and locally in a few other locations underlain by younger Holocene alluvial materials along creeks.

- In the red zones, the rate of pipe damage will be on the order of 1 to 2 repairs per city block, meaning that there will be a very high rate of leakage until these pipes are valved out, followed by extended service outages until repair are made.
- Most of the flat-land part of northern Palo Alto will have leak rates on the order from 0.06 to 0.6 repairs per 1,000 feet (yellow color coding) or 0.03 to 0.06 repairs per 1,000 feet (blue color coding). Until the leaking pipes are valved out, the high rate of water lost through the leaks will reduce system-wide pressures and many customers will be out of service. Once the leaking pipes are valved out, system pressures will increase and most customers in these areas will have water service, as long as there remains water supply either from SFPUC or from local wells.



Figure 4-1. San Andreas M 8.0. Pipe Repair Rates per 1,000 Feet

PIPE REPAIRS / K-Ft
Shaking + PGDs
Median Response

- 1.50 + Rate
- 0.60 - 1.50 Rate
- 0.06 - 0.60 Rate
- 0.03 - 0.06 Rate
- 0.00 - 0.03 Rate

In the foothills, the rate of pipeline damage will often be none to very low to moderate (grey, blue or yellow-coded pipes), except for one stretch of orange-coded pipe that runs southwards from the Corte Madera pump station. This section of 18" CCP runs parallel to an active creek, and there is significant chance the pipe will be damaged due to a lateral spread towards the creek (see Section 5 for further discussion of this pipe). Repair of this pipeline could take weeks, likely requiring road-re-building, plus complete replacement of possibly a hundred or more feet of pipe. The water from the Boronda reservoir will likely leak out entirely within 2 to 3 hours, assuming it cannot be valved out manually before that time. All customers uphill of the Core Madera booster pump station will have an extended outage. Possible mitigation options for the Foothills CCP pipe are outlined in Section 5 of this report.

4.6 SERA Results for the Monta Vista M 6.8 Earthquake

Table 4-21 lists the ground motions at sites due to the Monta Vista M 6.8 earthquake (MCE). This earthquake is assumed to have a reverse thrust mechanism. Sites listed as "S" after the distance to epicenter are located on the "hanging wall" side of the rupture, and would be exposed to very high ground motions.

SITE ID	SITE Abbrv	SITE Name	DISTANCE FROM FAULT KM	NEHRP SOIL m/sec CLASS	Vs30	LIQFC INDEX	LAND-SLIDE INDEX	SURF FAULT INDEX	PGA (G)– Vs(30)–		PGV (CM/SEC) Vs(30)–	
									MED	+1S	MED	+1S
1	QUARB	Quarry Booster	2.68	N	D 400.	6	3	3	0.48	0.79	77.63	138.30
2	DAHB	Dahl Booster	3.82	X	B 600.	2	3	3	0.46	0.81	49.81	92.88
3	DAHLR	Dahl Reservoir	3.82	X	B 600.	2	3	4	0.46	0.82	49.83	92.92
4	CORTB	Core Madera Booster	0.37	S	D 450.	5	4	3	0.82	1.42	96.95	179.31
5	BORB	Boronda Booster	2.59	S	B 760.	2	4	3	1.27	2.25	69.28	127.57
6	BORNR	Boronda	1.47	S	B 550.	2	4	3	1.00	1.73	87.95	162.34
7	B.IRR	Boronda Irrigation	1.46	S	B 550.	2	4	3	0.99	1.73	87.91	162.28
8	LYTTB	Lytton Booster	5.49	X	D 250.	4	1	2	0.37	0.57	72.13	122.95
9	HALEW	Hale	7.08	X	E 200.	8	2	1	0.31	0.48	67.81	112.75
10	EPRKW	Eleanor Pardee Park	7.46	X	D 270.	4	1	1	0.33	0.53	58.75	101.27
11	RINCW	Rinconada Well	7.00	X	D 280.	5	2	1	0.35	0.55	60.19	104.20
12	MLIBW	Main Library Well	7.18	X	D 280.	5	2	2	0.34	0.55	59.23	102.56
13	PEERW	Peers Park Well	5.67	X	D 300.	4	2	3	0.38	0.61	65.91	115.03
14	SF-CA	California	5.11	X	D 310.	3	2	3	0.40	0.64	68.55	119.94
15	SF-PA	Palo Alto Pipeline	5.41	X	D 260.	5	1	3	0.37	0.59	71.63	122.72
16	FERNW	Fernando	5.55	X	D 320.	3	2	3	0.39	0.63	64.67	113.46
17	MATAW	Matadero Well	5.18	X	D 330.	3	2	3	0.40	0.65	66.07	116.21
18	MAYFR	Mayfield	3.52	N	D 400.	3	3	3	0.45	0.75	70.89	126.30
19	MAYB	Mayfield Booster	3.56	X	D 400.	3	3	3	0.45	0.75	70.63	125.84
20	SF-PA	Page Mill	2.98	N	D 420.	3	3	3	0.47	0.78	72.77	129.86
21	SF-AR	Arastradero	4.05	X	D 350.	3	3	3	0.43	0.70	72.05	127.24
22	PARKR	Park Reservoir	2.88	S	B 760.	1	4	3	1.31	2.32	68.83	126.67
23	PARB	Park Booster	2.90	S	B 760.	1	4	3	1.31	2.32	68.80	126.62
24	CORTR	Corte Madera Res	1.25	S	B 550.	1	4	3	0.95	1.66	87.22	161.12
25	MNTB	Monte Bello Reservoir	4.45	X	B 650.	1	4	3	0.42	0.75	43.68	81.51

Table 4-22. Monta Vista M 6.8. Ground Motions at Sites (PGA, PGV)

The Monta Vista M 6.8 earthquake can result in very high levels of ground shaking (PGA >0.8g) at selected facilities in the Foothills, including the Corte Madera, Boronda, Boronda Irrigation, Park booster pump stations, Boronda and Park and Core Madera reservoirs. Recent seismic upgrades for the reservoirs have been to meet 2/3 of the 2,475 year motions defined in ASCE 7, which amount to about PGA = 0.6g for design; but the median motions could be as high as double this design value; and 84th percentile motions more than 3 times the design PGA value. For this rare event, the mitigation strategy

would be to rely on dead-end variable speed pumping, which is possible with the current pump station equipment; but with the loss of in-zone storage for weeks to months until permanent repairs can be made.

The Monta Vista M 6.8 event will also produce surface faulting, probably manifesting itself just north of the Core Madera booster pump station. Surface faulting will likely damage the 18" CCP pipe at multiple locations. Section 5 of this report discusses this issue in more detail, along with possible mitigation strategies.

5.0 Seismic Assessment of Foothills Pipeline

The Foothills transmission pipeline is CCP pipe, ranging from 18-inch diameter at the lowest elevation at Quarry Booster pump station, to 14-inch diameter at the highest elevation to Monte Bello reservoir.

Sections 2.3.1 to 2.3.3 describe the CCP pipe design and installation between the Quarry Booster pump station and Monte Bello reservoir.

Section 3 describes the seismic hazards along the Foothills pipelines. Specifically, Figures 3-12 to 3-31 show several liquefaction and landslide susceptibility maps for the Foothills pipelines, as well as aerial photos of each reach of the Foothills CCP pipes.

In (G&E, 2014), the information in Figures 3-12 to 3-31 was used to establish the potential for damage to the CCP in a San Andreas M 8 earthquake. Table 5-1 summarizes the findings from (G&E, 2014).

Earthquake Hazard	Number of Breaks	Number of Leaks	Number of Repairs
Ground Shaking	0.20	1.30	1.50
Liquefaction	0.15	0.15	0.30
Landslide	0.05	0.05	0.10
Surface Faulting	0.45	0.05	0.50
Total	0.85	1.55	2.40

Table 5-1. Performance of Foothill CCP Backbone Pipeline due to San Andreas M 8 Earthquake

Subsequent to the development of (G&E, 2014), two additional alignment-specific investigations were performed for the CCP in the Foothills:

- February 2015. Stephen Dickenson and John Eidinger walked a portion of the CCP alignment between Quarry Booster pump station and Monte Bello reservoir. Section 5.1 discusses the findings.
- March and April 2015. Darlene Holston and Donald Duggan performed soil resistivity tests at several locations along the Foothills CCP pipes. Section 5.2 discusses the findings.

The seismic results presented in Section 4 of this report factor in the findings from Section 5.

5.1 Field Reconnaissance – Landslide and Liquefaction

It is recognized that regional landslide and liquefaction maps, including those presented in Section 3 of this report, may not have enough detail to provide accurate assessments at site-specific locations. For example, the regional liquefaction for Napa includes an area

mapped as having "moderate" liquefaction susceptibility, but in the actual earthquake, there was widespread permanent ground deformations in that area that led to a concentration of damage to buried water pipelines (mostly cast iron pipes in that area). As illustrated in the Napa earthquake, site-specific investigation can shed new information that can be useful in developing more accurate hazard estimates.

On February 13, 2015, we did field reconnaissance for the zones highlighted in the heavy black boxes in Figure 5-1.

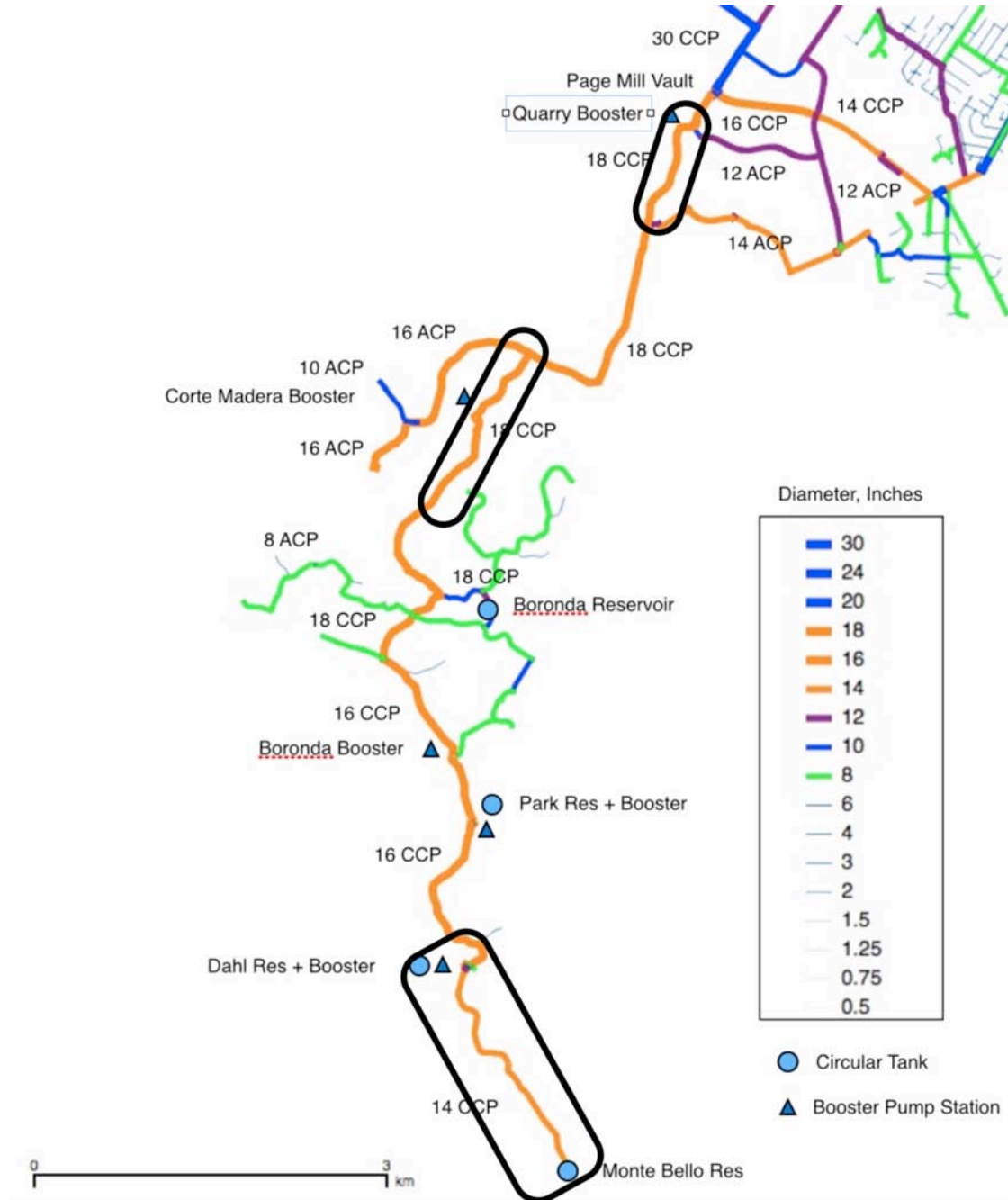


Figure 5-1. Portions of CCP Pipeline Field Inspected

The field reconnaissance focused on visual assessment of evidence for slope movement, locations where liquefaction hazards may be anticipated, and evidence of surface faulting. We looked for surface evidence for the potential for earthquake-triggered ground deformation as follows:

- Geomorphology of slope movement (tension cracks, graben development, pressure ridges and toe bulging, and hummocky topography)
- Soil creep
- Slumping or compression of fill
- Distress to pavement and/or sections of chronic repaving
- Distress to concrete slabs-on-grade, foundations, and structures
- Misalignment of tree trunks, telephone poles, and fence posts associated with surface creep or deep-seated rotational ground movement

The following sections provide a summary of the key observations made during the field reconnaissance. The objective of the field observation was to identify sites that may warrant additional, more site-specific investigation and to assist in the prioritization of assets and locations for subsequent seismic performance evaluation.

5.1.1 Monte Bello Reservoir

The plan and elevation for the Monte Bello Reservoir is provided on Sheet 22 (CPA 1964). Figures 5-2 to 5-5 provide an overview of the site conditions and features of geologic interest. The reservoir appears to be located on a cut pad. The depth to weathered rock is estimated to be shallow on the basis of the exposed cut along the E-SE side of the site and the rock (limestone) outcrops observed in proximity to the reservoir. The surficial geology in the area has been mapped (Dibble 1966, Brabb 1993) as consisting of various sub-units of the Franciscan formation (sandstone, greywacke, limestone, and chert).



Figure 5-2. Monte Bello Reservoir. View Looking East



Figure 5-3. Monte Bello Reservoir. Aerial Image (North Up)

A pronounced gully is located along the southwest portion of the reservoir; thus gully is seen in Figure 5-3 in the lower left quadrant. The gully has formed beneath a surface storm drain outlet for the tank site. The erosion and gullying provided an exposure of the soils and highly weathered bedrock adjacent to the tank site (Figures 5-4 and 5-5). The gully has incised roughly 12 feet into the soil and weathered rock. The exposed soil appears to be colluvium and residual material weathered in-place from parent rock. The composition is predominantly fine-grained with gravel-sized inclusions of limestone, siltstone, and sandstone. Relic bedrock structure was evident in portions of the gully. The gully side slopes are approximately 40° to 50°.



Figure 5-4. Active erosion and gully formed by storm drain, southeast side of reservoir



Figure 5-5. Gully, looking north and away from the reservoir, with Dickenson

This may have led to the characterization of this feature as a landslide in regional geo-hazard map (grey polygon to the west of the tank in Figure 3-34), but our field observations did not confirm evidence of active sliding to the west of the tank. It is our understanding that a primary cause of the increased rate of erosion that formed this gully is the surface run-off from the storm drain outlet, that appears to have dramatically increased the gullying since 1953 prior to construction of the reservoir (i.e., the earliest Google Earth image for the site).

The proximity of the Monte Bello tank to the San Andreas Fault Zone (about 500 meters to the southwest) is noted. Characterization of near-fault ground motions should be an important aspect of the seismic risk assessment.

5.1.2 Alignment Between Monte Bello Reservoir and Dahl Reservoir

The portion of the pipeline alignment that was observed extends from Station 192+54 (at Monte Bello tank) to 122+00 (Sheets 16 to 22, file ABM #583_Foothills 62-8.pdf). From the Monte Bello reservoir the pipeline follows a N-NW orientation along a broad ridgeline. The soils are estimated to be shallow based on observations of adjacent bedrock outcrops. No pronounced evidence of slope instability was observed between the reservoir and a swale located at approximately Station 170+00 and 167+00 (Sheet 20). The swale, or broad gully, exhibited evidence of surface seepage and surface ground instability. Shallow soil movement is expected in this area; however, the seat of movement appears to be shallow and may be above the pipeline embedded roughly 6 feet below grade (Sheet 20).

The pipeline follows Page Mill Road northerly to the Dahl Reservoir and Booster Station. This portion of the alignment was walked. Figure 5-6 shows the roadway, in which the CCP is located, along with a typical side slope. General observations are summarized as follows:

- The roadway appears to be in very good shape. No evidence of large-scale or deep-seated soil movement was evident. With the exception of general aging and localized alligator cracking, the pavement appeared overall to be in good condition and did not provide evidence for substantial soil movement.
- Shallow road cuts (generally less than 10 to 15 feet tall) were in good condition.
- Road fill along steeper slopes appears to be very isolated.
- Portions of Page Mill Road follow a broad ridgeline with steeper slopes (approximately 1V:3H to 1V:4H) on one or both sides of the roadway. Very little evidence of soil movement was observed along the roadway in these areas.



Figure 5-6. Page Mill Road Between Monte Bello and Dahl Reservoirs

5.1.3 Dahl Reservoir and Booster Pump Station Site

The plan and elevation for the Dahl Reservoir and Booster Station is provided on Sheet 16 (file ABM #583_Foothills 62-8.pdf). The reservoir appears to be located on a cut pad. The depth to weathered rock is estimated to be shallow on the basis of local geomorphology. The surficial geology in the area has been mapped (Dibble 1966, Brabb 1993) as Franciscan greenstone.

Observations made around the reservoir and booster pump station site did not reveal evidence for pronounced soil creep, slope instability, or settlement adjacent the reservoir foundation. Overall, the site appears to be in very good condition.

As with most of the Foothills pipeline network, the proximity of the Dahl site to the San Andreas Fault will require enhanced characterization of near-fault ground motions.

5.1.4 Alignment Along Arastradero Creek, Near Corte Madera Booster Station

The Arastradero Creek portion of the pipeline alignment from the vicinity of the Corte Madera Booster Station (approximately Station 128+00), upstream along Arastradero Creek to Station 207+00 (ABM #404 Foothills 59-1.pdf 1960), then to the SW between Stations 164+50 to 211+00 (ABM #583_Foothills 62-8.pdf 1964). Access to this portion of the pipeline alignment was made off of Arastradero Road along the single-lane dirt road marked as the Juan Bautista de Anza Trail to the Corte Madera Booster, then along

the dirt roadway (Arastradero Creek Trail) from the Corte Madera Booster to the gate where the Arastradero Creek Trail intersects with the paved roadway of the Open Space park.

The lower half of this portion of the pipeline alignment is mapped as a zone of Very High liquefaction hazard (Figure 3-22) due to young alluvial soil and high groundwater table adjacent the Arastradero Creek. The morphology of the valley bottom and creek channel along parts of this alignment can be characterized as a broad and shallow (Figure 5-7). The low gradient and apparently low flows along the portion of the creek observed has resulted in the deposition of predominantly fine-grained soils. A soil boring located above the Very High hazard zone (Station 174+75) revealed the following (ABM #583_Foothills 62-8.pdf 1964):

- 5 ft of sandy clay (CL) underlain by,
- Roughly 3 ft of slightly clayey fine to medium sand (SP – SC) then,
- Weathered and decomposed sandy shale.



Figure 5-7. General geomorphology along Arastradero Creek. View looking northerly. Creek is to right side of the photo. Pipeline generally follows roadway.

The broad nature of the valley floor, very shallow creek channel, and predominantly fine-grained nature of the soils observed collectively reduce the liquefaction hazard along this portion of the pipeline alignment. Along a portion of the alignment, the pipeline is embedded at an elevation that is lower than the base of the stream channel further reducing the likelihood of lateral spreading-induced damage to the pipeline.

Further upstream, the roadway has sections of steepened road fill sloping to the creek and relatively steep slopes above the roadway. The pipe alternates location on either side of the roadway. Timing precluded extensive examination of these slopes for evidence of slope instability. While the roadway was in very good condition indicating adequate static capacity, the seismic performance associated with design-level ground motions from an earthquake along the peninsula section of the San Andreas Fault may lead to permanent ground deformations along isolated sections of the roadway impacting the buried pipeline. In addition, the dense vegetation and ground cover may obscure evidence of minor sliding or slumping of the slopes above the roadway. A landslide, or lateral spread in these areas cannot be ruled out based on the available evidence.

A relatively short portion (approximately 1,000 ft) of Arastradero Creek located downstream of the Corte Madera Booster Station was observed (for a distance of about 1,000 feet northerly from the Corte Madera pump station, as mapped in Figure 3-19). This section of the alignment is adjacent to Arastradero Lake, the northern spillway from the lake, and the creek. It is our understanding that the pipeline is located in the road fill above the lake and creek. There is much more pronounced channelization of the creek downstream of the lake. This increases the possibility of slope movement toward the creek in the event of both high groundwater levels and strong ground motions. Portions of Arastradero Creek are mapped as a High hazard zone for liquefaction (Figure 3-19).

5.1.4 Alignment at Corte Madera Booster Station

The plan and elevation for the Corte Madera Booster Station is provided on Sheet 18 (C1960). The booster pump station site is located just west of and adjacent to an area characterized as a Very High liquefaction hazard (Figure 3-19). Figure 5-8 provides a general overview of the site. The booster station is located at the base of a broad gully (swale) and appears to be founded on a pad of fill. The thickness of the fill is not currently known; however, Sheet 18 (1960) indicates that a Boring was made at the site (Boring No. 11-A). We were unable to obtain a copy of this boring.

Surface water was observed on the W-SW portion (uphill) of the booster station site. The seepage and surface water demonstrates high groundwater levels and it may be assumed that most of the foundation fill is saturated from most of the year. Shallow slumping of the slopes above the booster station could be expected if strong ground motions occur during a period of high groundwater levels and active seepage.

Observations made around the booster site did not reveal evidence for pronounced soil creep, slope instability, or settlement of foundations or slab-on-grade. Overall, the booster site appears to be in very good condition.

The potential for seismically-induced settlement of the booster station fill and underlying alluvial soils should be evaluated in light of the high groundwater levels observed. A modest amount of differential movements of the booster pump station site could cause leaks or a full break of the below-grade pipes entering / leaving the pump station (more likely). Inertial shaking is not likely to cause breaks to any of the above ground pipes within the booster pump station site, but minor leaks at couplings or air release valves could still occur.



Figure 5-8. Corte Madera Booster Pump station Site. View is looking easterly, downslope. Arastradero Lake is beyond the car in the background.

5.1.5 Alignment at Corte Madera Booster Station

The portion of the pipeline alignment that was observed extends between Station 105+00 and 112+00 (1960). This portion of the alignment is located across Arastradero Road from the Enid W. Pearson Arastradero Preserve parking area (Palo Alto Open Space) and it is distinguished in the plan sheets as the location of an approximately 90 degree bend in the pipeline alignment. This section of the Foothills pipeline network is characterized as a zone of High liquefaction hazard (Figure 3-19). Figure 5-9 shows a portion of the alignment parallel to the roadway.



Figure 5-9. Pipeline alignment along Arastradero. Road is to left. Creek is in the lowlands to the right. Pipe runs parallel and between the road and creek. Eidinger stands near a vault hatch believed to be above the pipe.

In this area Arastradero Creek flows through a broad, flat valley floor. The creek is very low gradient and exhibits a shallow channel (generally less than 2 to 4 ft in the area observed). Figure 5-10 highlights a portion of the creek located approximately 250 feet to the east (downstream) of the entrance to the Pearson Arastradero Preserve. At this location the slope from Arastradero Road (i.e., the pipeline location) to the creek channel is low (approximately 1V:15H to 1V:18H). The gradient from the base of the buried pipe is much lower, indicating that at this location the potential for damaging lateral spreading due to liquefaction is minor. The soils exposed in and along the stream channel were predominantly fine-grained, or contained sandy material in a matrix of fine-grained soil. This exposed soil is judged to have a low potential for liquefaction. Taken collectively, the exposed soil type, shallow slope, distance from the pipeline alignment to the stream channel, and the depth of embedment of the CCP suggests that the liquefaction hazard at this site is very minor despite the characterization as "High" hazard on regional maps (Figure 3-19). The potential exists for lenses and layers of loose sandy soils that are vulnerable to liquefaction; however, the potential impact of these layers (of unknown vertical and lateral extent) must be evaluated in light of the slope angles and distance from the pipeline alignment to the stream channel.

It should be noted that a soil boring (No. 16) is indicated on Sheet 16 at Station 107+00. The soil log could not be found.



Figure 5-10. Arastradero Creek.

Given these issues, the pipe in the High and Very High liquefaction zones in Figure 3-19 are possibly subject to local settlements of an inch or so, and we think a very modest chance of being subject to a lateral spread towards the creek. However, given the lack of detailed subsurface information, the implications of a relatively low liquefaction risk at a few locations cannot be generalized with confidence to the entire alignment. Had this pipeline been design today, this section of the alignment (Figures 3-16 and 3-19) would probably have been specified to have fully restrained joints capable of reliably sustaining at least a couple of inches of differential settlement.

5.1.6 Alignment along Old Page Mill Road

The portion of the pipeline alignment that was observed extends between Station 500+00 (Frenchman's Tower) and the Quarry Booster Station (Station 112+00). This alignment is covered on Sheets 7, 8 and 9 (1960). This section of the Foothills pipeline network is characterized as a zone of Very High liquefaction hazard due to proximity to Matadero Creek (Figure 3-16). The pipeline alignment is alternatively located in the roadbed and along Matadero Creek is this area, including several crossings under the creek. The road has been developed in cut and fill with slope height adjacent to the creek channel of 10

feet to 15 feet. The roadway generally appeared to be in very good condition where co-located with the pipeline.

The CPA plan sheets indicate that several geotechnical borings were carried out along this portion of the alignment (boring numbers; 1, 18, 20, 6, 7, 8, 9). This information was not available. The borings may provide useful information for the depth to firm soils and or weathered rock.

Two of the four stream crossings were observed along the alignment. These are approximately located at stations 38+09.90 (Sheet 10, 1960) and 33+43 (Sheet 9, 1960). The 18-inch diameter CCP is encased in concrete with bank protection provided rip-rap at these crossings (Figure 2-30 shows the "typical" stream crossing details). The crossing at STA 33+43 is shown in Figures 5-11 to 5-13. The following items of note are evident in these figures:

- The bed load (channel fill) is predominantly loose, granular soil. This soil is highly vulnerable to liquefaction during seismic loading.
- The soil exposed in the banks of the stream is predominantly fine-grained and it appears to be either a weathering product of the parent rock (Page Mill basalt of Dibblee 1966) or older alluvium. In either case the stiff fine-grained soil along the banks and stream terrace is not considered vulnerable to liquefaction, although strong ground shaking could result in permanent deformation along over-steepened portions of the channel and areas adjacent to sloping road fill.
- The thickness of the potentially liquefiable soils appears to be small at this location based on the surface exposures and stream bank morphology.
- The concrete encasement for the CCP is exposed in the stream channel (Figure 5-11). If this location was constructed as shown in the original drawings (Figure 2-30), then over the past 60± years, the top layer of sacked concrete rip rap has been lost, followed by exposing of several inches of the unreinforced concrete encasement around the pipe. With time, more scour can be expected, and the hydraulic drop over the pipe encasement will eventually reach about 2 feet. This will result in an ongoing hydrostatic "push" of about 125 pounds per foot of pipe. As the pipe is segmented, and the encasement is unreinforced, with enough width the lateral force can be large enough to break open the pipe.

At this location, we recommend that a short term mitigation be done to restore at least the original intent of the original design. This could be achieved by placing a reinforced concrete encasement around the existing encasement (capable of withstanding the full hydraulic forces), and placing some upstream and downstream scour protection. We also recommend that all other stream crossings be inspected annually, and similar repairs made should the concrete encasement be observed to be exposed.



Figure 5-11. Matadero Creek crossing at Sta 33+43. View looking downstream (looking north)



Figure 5-12. Matadero Creek crossing at Sta 33+43. View looking upstream (looking south). The exposed concrete encasement for the CCP is evident.



Figure 5-13. Matadero Creek crossing at Sta 33+43. View looking northerly. Slope rip rap is in place. Top of concrete-encased pipe is in foreground.

5.1.7 Quarry Booster Pump Station

The plans and elevations for the Quarry Booster Station are provided on Sheets 3 and 7 (1960). This site is located in an area characterized as a Very High or High liquefaction hazard (Figure 3-16). Figures 5-14 and 5-15 provide a general overview of the site conditions and channelization of Matadero Creek adjacent to the booster station.

One geotechnical boring has been discovered for the Quarry Booster site. The soil profile was reported to consist of the following, from the existing ground surface to depth:

- 5 feet of dark brown sandy clay, over
- 4 feet of clayey sand,
- 2 feet of bluish-grey sand with some clay fines,
- Volcanic gravels

The nature of the site development made after the boring was completed is not known. Subsequent earthwork and site preparation could have altered the site conditions and soil profile. This could have been particularly significant immediately adjacent to Matadero Creek. The extent, composition, and density of possible fill layers at the site are unknown at this time. The groundwater level is assumed to be at the elevation of Matadero Creek and to seasonally fluctuate.

The Quarry Booster pump station site is immediately adjacent to a PG&E gas compressor station.

Observations made around the Quarry Booster Station site did not reveal evidence for pronounced soil creep, slope instability, settlement of concrete slabs or foundations of the block wall structure, or differential movement adjacent to the creek. Overall, the booster site appears to be in very good condition. However, the post-1989 Loma Prieta site observations (see Section 3.8.2) showed that there were small cracks in the concrete at-grade slabs around the booster pump bases and where the underground piping surface through the slab; the slab was serviceable. When we visited the site in February 2015, the concrete slabs where the pipes enter/exist the ground have apparently been cut-away, with the void filled with asphalt (Figure 5-16); we observed similar asphalt details at Corte Madera and Dahl booster pump station sites. Possibly, the concrete slabs at these pump stations were modified sometime between 1989 and 2015; these modifications might help allow the pipes to sustain minor differential movements between the soil and the above-grade portions.

Little is known about the seismic design capability of the improved Matadero creek embankments seen in Figures 5-14 and 5-15.

However, given the location of the creek, the available boring, and the local topography, we suspect that this site may actually undergo liquefaction in future large earthquakes. The primary weaknesses will be:

- Small lateral spreads at the creek banks will impose high compression and bending on the pipe over-crossing of the creek, possibly leading to pipe failure. This could be mitigated (preliminary design strategy) by installing a new section of pipe over the creek, with the new section of pipe being designed to withstand any soil movement without failure, possibly by anchoring the pipe on drilled piers either side of the creek; and with adjacent buried pipe also able to sustain the soil movements.
- Other design solutions are also possible, including burying the water pipe under the creek; and/or soil improvements; and or adding an isolation valve / bypass capability.
- All these solutions suggest that design effort include the following: collection of design and construction records of the creek embankments (possibly Palo Alto

Public Works or (maybe?) Caltrans); new subsurface explorations at the pipe crossing and possible within the pump station (CPTs and/or geotechnical borings to a depth of at least 20 feet); collection of drawings for the combined compressor station (PG&E) and water pump station (Palo Alto) underground pipes / facilities in and near the site.

Summary. The potential for differential ground movement adjacent to the stream channel is considered high during design level ground motions. A preliminary cost, considering the preliminary design strategy above, is included in this report.



Figure 5-14. Matadero Creek adjacent to Quarry Booster Pump Station. View looking upstream (looking south). Pipe in foreground is inlet water pipe to Quarry booster pump station. Eidinger stands behind box presumed to house an air release valve.



Figure 5-15. Matadero Creek adjacent to Quarry Booster Pump Station. View looking downstream (looking north). Quarry Booster pump station is to right of the photo.



Figure 5-16. Pipes at Quarry Booster Pump Station. Asphalt-filled cut-outs in concrete pads.

5.1.8 Approaches to Refined the CCP Hazard Assessment

In Sections 5.1.1 to 5.1.7, we have presented the field observations along a substantial portion of the backbone portion of the CCP between the Quarry booster pump station and the Monte Bello reservoir. We considered the field observations, plus regional maps, plus available subsurface information. In a number of ways, our evaluations could be refined, and these are listed below:

- The forecast ground motions along the CCP are included in the SERA model for 24 different scenario earthquakes. These ground motions are based on the latest (2013) GMPEs, presented for both the median and 84th percentile motions. Locally along the pipeline, the effects of directivity can be important for earthquakes on the San Andreas fault, especially if the earthquake epicenter is located more than about 50 km to the northwest of Palo Alto, with rupture southeasterly towards Palo Alto. In these cases, the directivity ("pulse") and directionality ("fling") effects can be important. Local topographic effects in the hills might also influence the ground motions. The 84th percentile results presented in this report capture much of these "pulse" and "fling" and "topographic" effects. If the epicenter is very near Palo Alto (say within a few km of the Monte Bello reservoir), the "pulse" and "fling" effects will not be important. The local shear wave velocities along the pipeline have been estimated, and these could be refined using site-specific testing. Overall, while improvements in ground motion estimates can be made, they are unlikely to change the results in this report by more than 10% to 20%.
- System-Wide Characterization of Landslide Hazard. Much of the Foothills pipeline alignment is obscured by thick vegetation, impeding efficient field reconnaissance and obscuring observation by aerial photography. The acquisition and examination of low-altitude, high-resolution LIDAR would help refine the landslide susceptibility along the entire CCP alignment. We do have LiDAR data along a 230 kV PG&E transmission line in Zone 9; which shows no landslides along the CCP pipe in Page Mill road. Should LiDAR data be made available for the entire pipeline alignment in the Foothills, a better estimate of landslide-induced damage to the pipelines could be made.
- Refined Characterization of Liquefaction Zones of Moderate to Very High or High Hazard. The rapid visual assessment of portions of the Foothills pipeline alignments indicates on the basis of surface exposures that regional hazard maps can lead to over-prediction of the seismic vulnerability of pipelines in some areas. To do a more refined assessment, we would need new subsurface information near Quarry Booster pump station, plus below, at and above the Corte Madera booster pump station. The primary weakness is a lateral spread (or landslide) of the soils surrounding the CCP pipeline, at locations near and parallel creeks, especially if the earthquake occurs in the winter with ground-saturated conditions.

For the purposes of this report, the refinements outlined above are not available. Clearly, if a new backbone pipeline is considered for the Foothills, then the additional efforts outlined above would be suitable, and be part of the design process. As will be discussed in Section 5.5 (Recommendations), we do not think that a new pipeline is currently a cost-effective strategy; but instead recommend a few upgrades with the intent to restore water supply capability along the backbone Foothills pipeline within a day or so following a large earthquake.

5.2 Foothills Water System Performance after San Andreas M 8

In (G&E, December 2014), a preliminary evaluation of the Foothills CCP pipes was made, assumed a San Andreas M 8 earthquake. In this report, we update those findings, reflecting the additional information obtained from the field inspections and local soil corrosivity tests conducted between February and April 2015.

The Foothills area of the Palo Alto water system is divided into five pressure zones (see Figure 2-3):

- Zone 5. From Quarry Booster to Corte Madera Booster pump station. Local in-zone storage in Corte Madera tank. Backbone CCP pipe is 18" diameter, running along Matadero Creek and Arastradero Creek. Primary seismic weaknesses include suction pipe section above Matadero Creek below Quarry Booster pump station (part of Zone 4); liquefaction at Quarry Booster Pump station; exposed encasement at Matadero Creek (due to ongoing scour); minor settlement issues parallel to Arastradero Creek; slop failure potential north of Arastradero lake, liquefaction under Corte Madera pump station; surface faulting potential north of Corte Madera pump station.
- Zone 6. From Corte Madera Booster pump station to Boronda pump station. Local storage in Boronda tank. Primary seismic weaknesses are minor landslide / lateral spread above Arastradero Creek along the 18" CCP alignment.
- Zone 7. From Boronda pump station to Park booster pump station. Local storage in Park tank. Primary seismic weaknesses are an unknown potential for landslide along the 16" CCP alignment (none known to exist).
- Zone 8. From Park pump station to Dahl booster pump station. Local storage in Dahl tank. Primary seismic weaknesses are an unknown potential for landslide along the 16" CCP alignment (none known to exist).
- Zone 9. From Dahl pump station to Monte Bello reservoir. Primary seismic weaknesses are an unknown potential for landslide along the 14" CCP alignment (none known to exist).

Along the CCP alignment, we performed several soil resistivity tests, with the following Rho values at a depth of 5 feet.

- Zone 5. Two tests along Matadero Creek. Both tests show Rho in the range of 800-900 ohm-cm (extremely corrosive). One test (2,200 ohm-cm) near Pearson Arastradero Preserve parking area. One test at Corte Madera pump station (2,300 ohm-cm). The low Rho values, especially along Matadero Creek, likely reflect the high ground water table.

- Zone 6. From Corte Madera Booster pump station to Boronda Booster pump station. Rho values of 2,600 ohm-cm, 1,400 ohm-cm, 1,700 ohm-cm (pipe leading to Boronda reservoir), 5,700 ohm-cm (pipe leading to Boronda Booster pump station).
- Zone 7. From Boronda Booster pump station to Park Booster pump station. Rho values of 10,200 ohm-cm, 7,200 ohm-cm.
- Zone 8. From Park Booster pump station to Dahl Booster pump station. Rho values of 6,800 ohm-cm, 5,700 ohm-cm. At Dahl Booster pump station, 1,600 ohm-cm: this low reading might have been influenced by many underground pipes (more likely), or locally high ground water table (possible, but thought less likely).
- Zone 9. From Dahl Booster pump station to Monte Bello reservoir. Rho values of 9,700, 8,000, 5,600, 16,600 ohm-cm.

The Rho tests show that for the upper Zones 7, 8, 9, the soil is not particularly aggressive, and corrosion attack is likely very limited. In the lower Zones 5 and 6, the soils are more aggressive. However, there has as yet been no leaks in the entire length of this CCP, over a ~50 year time frame, suggesting that the CCP (assumed heavy reinforced concrete coating with heavy interior cylinder) has performed well. From a corrosion / leak history point of view, there is no particular justification for pipeline replacement based on poor historical performance; the lack of historical leaks also indicates that any landslides along the alignment have not moved; or moved very little over the past 50+ years, including the moderate levels of shaking from the 1989 Loma Prieta earthquake (PGA about 0.10g to 0.15g commonly along the Foothills CCP).

5.2.1 San Andreas M 8 – Pipeline Performance in Foothills (Zones 5, 6, 7, 8, 9)

In Section 4.0 of this report, the performance of the entire water pipeline system was examined for 24 different scenario earthquakes. Of particular interest is earthquake scenario 13, namely the San Andreas M 8.0 earthquake that includes ruptures of the nearby San Andreas fault. The total damage in that earthquake is estimated to be about 215 pipe repairs (median) to 351 repairs (84th). In the Foothills, Figure 4-1 shows that the CCP pipe between the Corte Madera pump station and the branch to the Boronda reservoir had the highest potential for failure. Herein, we present additional breakdown of the forecast damage to the Foothills pipelines in this earthquake.

In the higher elevations above the Boronda pump station, the CCP pipe is underlain mostly by rock-like conditions. PGVs are estimated to be about 74 to 90 cm/sec (median) up to 165 cm/sec (84th). At lower elevations, the pipe is underlain by layers of softer soils, and the PGVs are 85 to 97 cm/sec (median), up to 180 cm/sec (84th).

The most common type of distribution pipe in the Foothills areas is ACP. Table 5-1 shows that about 4 repairs could be expected (3 on Zone 5, and 1 elsewhere).

PIPE REPAIRS BY PRESSURE ZONE, ACTIVE PIPES, TYPE = ACP						
ZONE NUMBER	ZONE NAME	LENGTH (Miles)	ACP ONLY (Miles)	TOTAL REPAIRS	PIPE LEAKS	PIPE BREAKS
1	1 - One	166.79	97.42	101.61	50.89	50.72
2	2 - Two	36.81	22.25	5.72	3.15	2.57
3	4 - Four	3.74	3.47	0.31	0.19	0.11
4	3 - Three	6.93	3.32	8.44	4.23	4.21
5	5 - Five	5.52	3.05	3.06	1.53	1.53
6	Unknown	1.00	0.41	2.99	1.49	1.49
7	6 - Six	5.21	2.13	0.26	0.19	0.07
8	7 - Seven	3.67	2.35	0.25	0.21	0.04
9	8 - Eight	1.29	0.15	0.02	0.01	0.00
10	9 - Nine	1.36	0.00	0.00	0.00	0.00
TOTALS				122.64	61.90	60.74

Table 5-1. San Andreas M 8.0. Pipe Repair by Pressure Zone (AC Pipe Only)

The backbone pipe in the Foothills in all CCP. Table 5-2 lists the forecast damage to the CCP. This shows that the CCP in Zone 6 (between Corte Madera pump station and Boronda reservoir) is forecast to have 6 repairs, of which 3 are breaks and 3 are leaks. This reflects that the pipe could be exposed to liquefaction / slide damage where it parallels Arastradero Creek (see Sections 5.1.3, 5.1.4., 5.1.5). While Table 5-2 shows a smaller potential for similar damage in Zone 5, below the Corte Madera pump station, this might be somewhat understated, and the chance of CCP damage due to liquefaction effects next to Matadero Creek near Quarry Booster pump station (see Section 5.1.7) are not captured in the model.

PIPE REPAIRS BY PRESSURE ZONE, ACTIVE PIPES, TYPE = CCP						
ZONE NUMBER	ZONE NAME	LENGTH (Miles)	CCP ONLY (Miles)	TOTAL REPAIRS	PIPE LEAKS	PIPE BREAKS
1	1 - One	166.79	4.23	2.98	1.49	1.49
2	2 - Two	36.81	3.04	0.34	0.17	0.17
3	4 - Four	3.74	0.02	0.00	0.00	0.00
4	3 - Three	6.93	0.00	0.00	0.00	0.00
5	5 - Five	5.52	2.37	0.20	0.10	0.10
6	Unknown	1.00	0.01	0.02	0.01	0.01
7	6 - Six	5.21	2.73	6.04	3.02	3.02
8	7 - Seven	3.67	0.39	0.02	0.01	0.01
9	8 - Eight	1.29	1.05	0.06	0.03	0.03
10	9 - Nine	1.36	1.31	0.08	0.04	0.04
TOTALS				9.74	4.87	4.87

Table 5-2. San Andreas M 8.0. Pipe Repair by Pressure Zone (CCP Pipe Only)

In the preliminary report for the Foothills CCP (G&E, 2014), we had forecast 2.4 repairs to the CCP backbone pipe between the Quarry Booster and Monte Bello reservoir. Table 5-2 has increased this from 2.4 to 6.4. The following highlights some of the issues:

- More length of CCP is included in this report. All CCP pipe is included in Table 5-2), including the CCP going to the Corte Madera reservoir; whereas in the preliminary report, the CCP pipe to the Corte Madera reservoir was not included. This report includes about 57% more CCP than the preliminary report.
- In this report, we allow for magnitude dependency on PGD movements. For a M 8 earthquake, this increases PGDs substantially. For a M 7.5 event, the corresponding number of repairs is 4.77. For a M 7.0 event, the corresponding number of repairs is 3.39.
- In this report, we assume there is a material chance of a landslide or lateral spread for the CCP where it parallels Arastradero Creek. In the preliminary report, the liquefaction hazard was more uniformly assumed along both Matadero Creek as well as Arastradero Creek.
- Both reports assume half the repairs due to liquefaction are full breaks and half are leaks.

Whether the number of CCP breaks is just over 1 (M 7 event) or over 3 (M 8 event), the lack of any redundant flow paths means that uphill customers will lose their only source of water re-supply, once local water tanks are drawn down.

As outlined in Section 5.1.8, there are approaches that can be done to further refine the hazard assessment along the backbone CCP; these refinements would also refine the potential for CCP pipe damage. For the Monta Vista Shannon / Berrocal fault zones, we have not recommended further investigations, recognizing the lower annual chance of such an event. At the core of the issue, though, is that the existing CCP is a segmented pipe, and any material amount of soil movement (PGDs much over a couple of inches) will have the potential to open up a pipe joint. To more accurately assess that there is nearly no potential for lateral spreads or landslides, and thus be confident that we have not missed a potential for soil movement on the order of a couple of inches, is possibly impractical. So, in context of the recommendations made in Section 5.5 (not to build a new pipeline, but rather build the ability to bypass the pipe (along Arastradero Creek), and local upgrades along Matadero Creek, through the highest known risk zones, we think the current hazard descriptions are reasonable.

5.2.2 San Andreas M 8 – Reservoir Drawdown and Fire Following Earthquake

From the point of view of a future large magnitude earthquake on the nearby San Andreas fault, or on the Monta Vista Shannon fault zone, the current evaluations show that there will be pipeline damage in the Foothills, and this will likely lead to water

outages. There are no redundant flow paths from Zone 5 to Zone 9, meaning that a break in the backbone Foothills CCP will necessarily isolate all uphill customers once in-zone storage is exhausted. Water in local tanks will be rapidly depleted in any zone that has a pipe break; depletion will typically occur within 3 to 6 hours if the break is on the backbone CCP pipe; or within 24 hours if the break is on a smaller diameter distribution branch pipeline.

With respect to the water system, there are two important time intervals after the earthquake:

- First 24 hours. During this time frame, the chance of a fire ignition is highest. The most likely type of ignition will be electrical in nature, especially once electric power is restored to the area. It is suspected that most residential structures in the Foothills area have relatively modern construction. Overall, assuming about 2,000,000 square feet of residential construction in the Foothills area, and a PGA of about 0.6g on average, the chance of a fire ignition is on the order of 10% or so, see the red line labeled "2012" in Figure 5-17. There is considerable scatter in the empirical evidence, so while the most likely chance of fire ignition is about 10%, it might be between 5% to 25% or so. Note the black line, titled "EBMUD 1195" was used to evaluate the risk of fire spread for EBMUD, circa 1995; since that time, additional earthquakes in the world have shown the fire ignition rate, using modern construction, as being substantially lower than the older pre-1989 data set (black squares), some of which reflect the 1906 earthquake, where quality of construction was often poor.
- Should an ignition occur, and the initial ignition is not controlled by local residents within a few minutes, the fire can spread within a structure, and if left uncontrolled, can spread into the wildland and to adjacent structures.

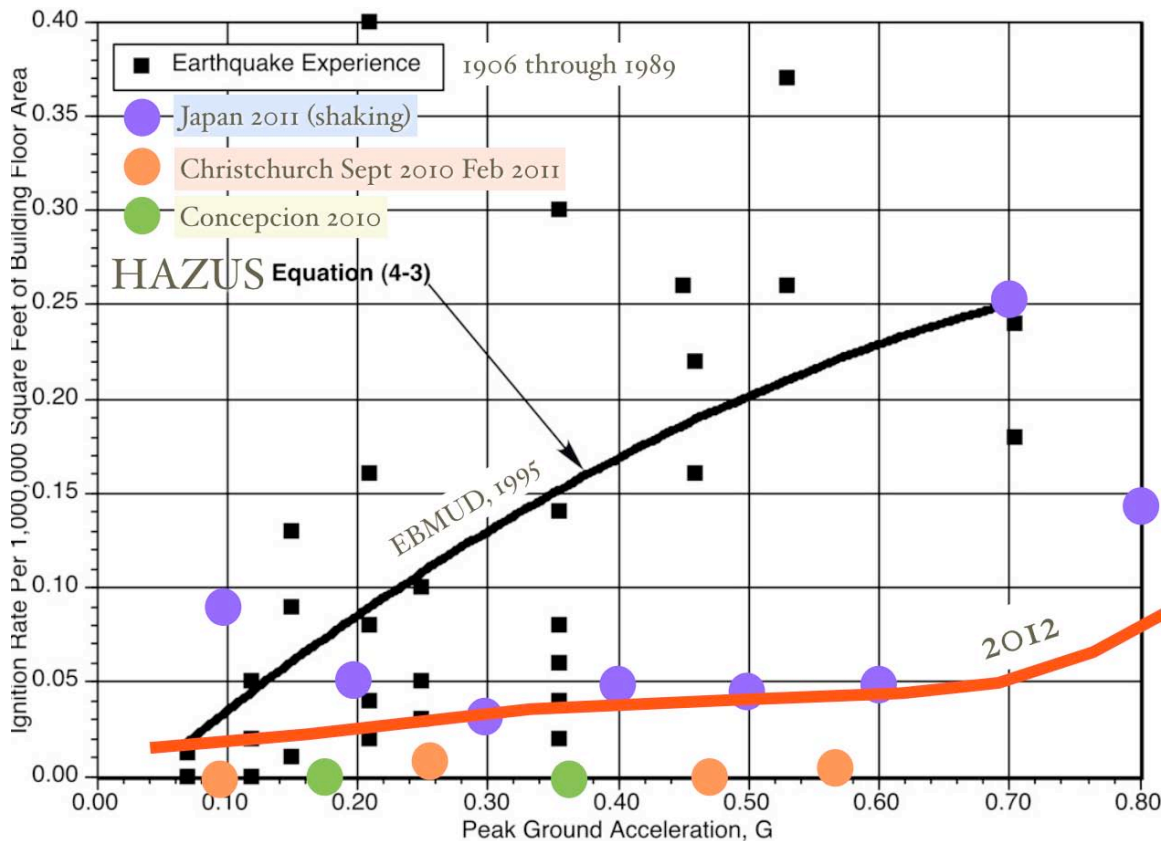


Figure 5-17. Fire Following Earthquake Ignition Rate – Empirical Evidence

5.3 Pipe Repair Strategies

Repair of a 14" to 18" CCP pipe will generally entail the following steps, if the repair site is easily accessible (on flat ground or slopes under 10% or so).

- Mobilize pipe crew with excavator, crane, dump truck and related equipment.
- Close upstream and downstream valves. Excavation of about 72 feet of pipe, to expose two pipe segments.
- Dewater the site.
- Removal of the two pipe segments, or if steel fabrication capability is possible, cut out the damaged / distorted steel cylinders at two or three locations.
- Either install new steel pipe (with suitable wall thickness to take the pressure at the location), or re-install the old CCP with repaired cylinders.
- Join the newly-installed pipe with steel butt straps using outside fillet welds.

- If new steel pipes are used, they should be cement lined and ideally concrete coated. If a dielectric lining or coating system is used, then the resulting completed pipeline will have inconsistent coating systems, and the potential for future corrosion is increased. Thus, if inconsistent coating systems are used, and if time allows, installed flanges with insulating joints; and install sacrificial anodes for the new steel pipe.
- Pressure-test the completed pipe.
- Close the trench.
- Disinfect the pipe.
- Restore the pipe to service.
- The above repair strategy is geared toward restoring service; it will not improve the capability of the pipeline to survive future earthquakes.

The time needed to make such a repair with a 6-man crew (crane operator; dump truck operator; welder; two plumbers; foreman) will be about 2 days, once mobilized. The needed pipe, butt straps, welders, and heavy crane to lift the old CCP might all be scarce after the earthquake, so unless there is a suitable emergency response plan, mobilization might take a day to a week or longer.

Should the pipe break occur along one of the steep slopes with difficult access, the time needed to make a repair could extend to weeks or longer. Further, the repair approach will have to also consider installing of additional pipe anchors to ensure that the uphill pipe is properly supported while making the repair.

Another complicating factor is that, for much of its length, the CCP pipe runs parallel to a sanitary sewer pipe of similar diameter (typically about 15", VCP). In locations where damage is due to PGDs (either due to liquefaction, landslide or surface faulting), the adjacent sewer pipe will likely also fail. It will be practical to repair both the sewer and water pipeline at the same time. Care must be taken to avoid contamination of the potable water pipeline with spilled or leaking sewage.

Practically speaking, it is possible that Palo Alto will not be able to repair a damaged CCP fast enough to avoid depletion of water from uphill storage reservoirs in pressure zones that did not sustain any pipe damage.

5.4 Pipe Mitigation Strategies

Option 1. Do nothing.

Option 2. Rebuild the Foothills backbone pipeline with a new seismically-designed pipe. This option includes new pipe through all major hazard zones.

Quarry Booster to Corte Madera Booster. Replace existing CCP with new Kubota chained seismic-resistant pipe, capable of sustaining liquefaction movements. Improve the suction pipe into Quarry Booster pump station. Cost: Allow for 16,500 feet of pipe, 18-inch diameter, \$20 per inch foot = \$5,940,000, plus one over-creek improvement, \$106,000, say \$6,046,000 total. This pipe will be designed to accommodate up to 12 inches of fault offset through the Monta Vista - Shannon Fault Zone, in part using the chained joints, and in part using a pea gravel trench through the zone. (Optional: use butt-welded steel pipe).

Corte Madera Booster part way to Boronda Reservoir, along liquefaction zone. Replace existing CCP with new Kubota chained seismic-resistant pipe, capable of sustaining liquefaction movements. Cost: Allow for 3,300 feet of pipe, 18-inch diameter, \$20 per inch foot = \$1,188,000.

Boronda Reservoir to Boronda Booster. Replace existing CCP with new Kubota chained seismic-resistant pipe, capable of sustaining liquefaction movements. Cost: Allow for 3,300 feet of pipe, 18-inch diameter, \$20 per inch foot = \$1,188,000. This pipe will be designed to accommodate up to 12 inches of fault offset through the Berrocal Fault Zone, in part using the chained joints, and in part using a pea gravel trench through the zone. (Optional: use butt-welded steel pipe).

Boronda Booster to Park Reservoir. There is no practical way to construct a landslide-resistant pipeline in this area that could sustain 10+ feet of movement. Possible alternatives would be to drill a new pipe under any deep-seated landslide depth, or to construct a parallel pipeline that would bypass the same slide zone. For cost purposes, assume that a parallel welded-steel pipe is built, with butt-welded joints. Cost is \$24 per inch-foot. Pipe is 16-inch diameter. Length of the bypass pipeline is assumed to be 6,600 feet, which would allow for spatial separation between the two pipes. Cost is \$2,534,400.

Park Reservoir to Dahl Reservoir. There is no practical way to construct a landslide-resistant pipeline in this area that could sustain 10+ feet of movement. The better alternatives would be to drill a new pipe under any deep-seated landslide depth, or to construct a parallel pipeline that would bypass the same slide zones. For cost purposes, assume that a parallel welded-steel pipe is built, with butt-welded joints, through the steep slopes. Cost is \$24 per inch-foot. Pipe is 16-inch diameter. Length of the bypass pipeline is assumed to be 4,950 feet, which would allow for spatial separation between the two pipes. Cost is \$1,900,800.

Dahl Reservoir to Monte Bello Reservoir. There is no practical way to construct a landslide-resistant pipeline in this area that could sustain 10+ feet of movement. The better alternatives would be to drill a new pipe under any deep-seated landslide depth, or to construct a parallel pipeline that would bypass the same slide zones. For cost purposes,

assume that a parallel welded-steel pipe is built, with butt-welded joints, parallel to the existing pipe where it traverses through the steep slopes. Cost is \$24 per inch-foot. Pipe is 14-inch diameter. Length of the bypass pipeline is assumed to be 2,640 feet, which would allow for spatial separation between the two pipes. Cost is \$887,040.

Reach	Option 2 New Pipe	Option 3 ULDH + Valves	Option 4 Suction Pipe + Creek	Recommended
Quarry PS to Corte Madera PS	\$6,046,000	\$540,000	\$106,000 + \$52,000	\$540,000 + \$106,000 + \$52,000 = \$698,000 total
Corte Madera PS to Boronda PS	\$1,188,000	(incl. above)		(incl. above)
Boronda PS to Boronda Res	\$1,188,000			
Boronda PS to Park PS	\$2,534,400			
Park PS to Dahl PS	\$1,900,800			
Dahl PS to Monte Bello Res	\$887,040			
Total (construction)	\$13,744,240	\$540,000	\$158,000	\$698,000
Total, including soft costs	\$16,493,000	\$580,000	\$158,000	\$738,000

Table 5-1. Costs for Foothills Pipeline Alternatives

Total construction cost, all reaches, including contingency: \$13,744,240. Add 20% for soft costs (engineering, design, project management, construction inspection), for a total project cost of \$16,493,000. This cost is in \$2015 and excludes inflation.

Benefits: For \$16,493,000, the probability of backbone pipe failure that leads to loss of water service in the Foothills area will be substantially reduced. The existing pipeline has about an 85% chance of failure; with this \$16,493,000 pipeline, the chance can be reduced to under 5%.

Option 3. Install isolation valves and bypasses and use above ground ultra large diameter hose to restore service post-earthquake, if needed.

The existing in-line valves along the CCP are shown in Figure 5-18. The schematic diagram was developed based on the GIS information provided. The large dots represent

in-line valves along the pipeline. As we have not verified the accuracy of the GIS information, this information should be considered preliminary and subject to change.

Also highlighted in Figure 5-18 are dashed lines that represent the secondary fault crossing zones.

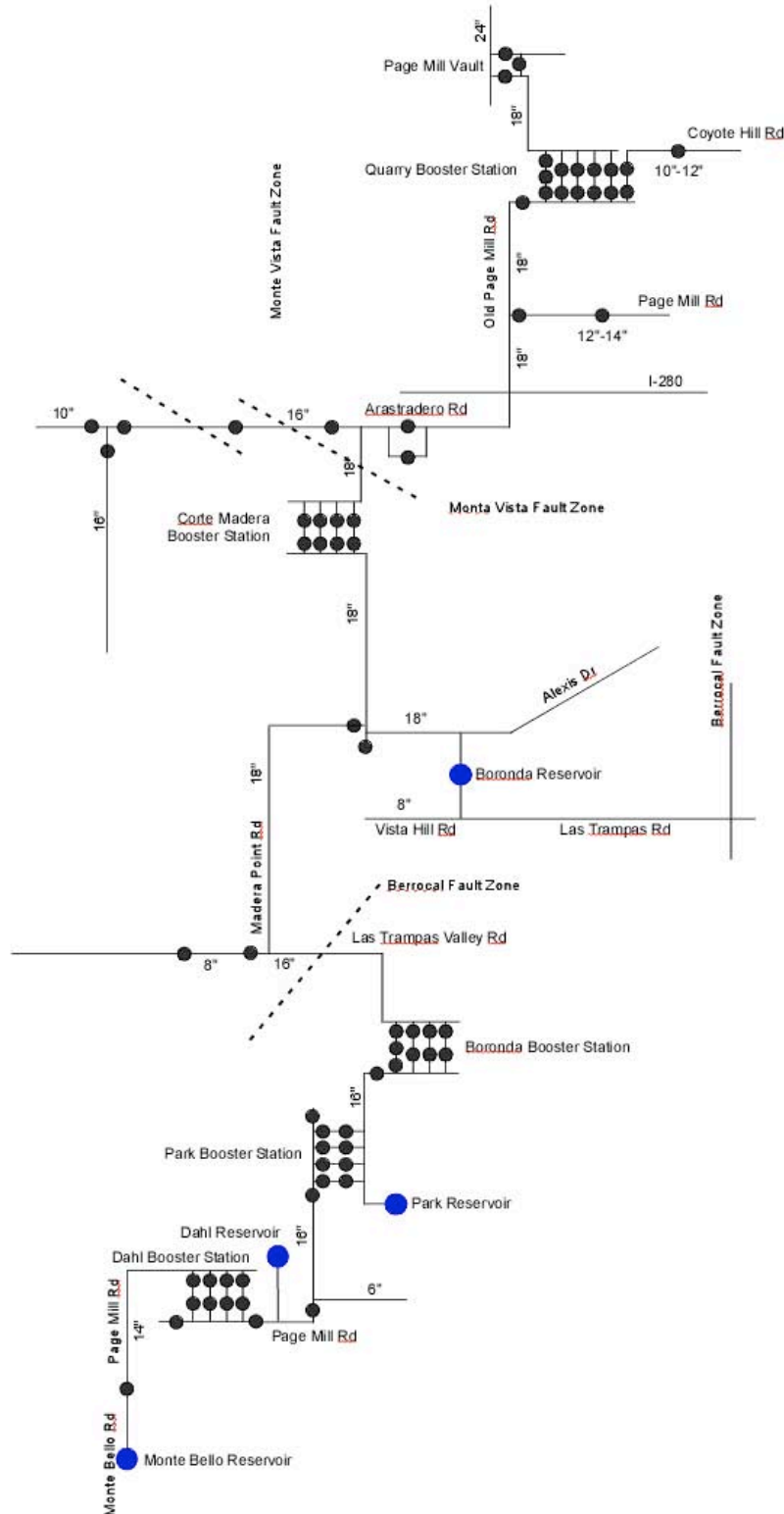


Figure 5-18. Foothill Pipe Network

The seismic mitigation concept for Option 3 is as follows:

- There are several areas where the existing CCP pipelines are most likely to leak or break in future earthquakes. Based on the available evidence, the highest risk zones are below and above the Corte Madera booster pump station, due to liquefaction or landslide movements into Arastradero creek (included in Option 3); and possibly adjacent to Quarry Booster pump station (included in Option 4).
- Adding more in-line valves will increase system reliability after earthquakes as well as for day-to-day operations. By having more in-line valves, a shorter segment of pipe can be shut down to make repairs, thus impacting fewer customers.
- As there are essentially no "loops" in the Foothills area, the shutdown of any valve (for planned maintenance or unplanned leak) requires careful planning, whereby the shutdown time must be shorter than the duration that water in in-zone tanks will last, with a suitable margin for fire flows. However, a substantial benefit occurs if a temporary above ground bypass pipe can be placed in service, to bypass the segment of pipeline shut down for maintenance or repair.
- The bypass "pipe" that is considered is a "Super Aqueduct" 12" diameter hose, by Angus or similar. Often, this size hose is called "Ultra Large Diameter Hose, ULDH". Other water agencies in the San Francisco Bay Area have already procured about 20,000 feet of such hose, and possibly, Palo Alto could use this hose by mutual aid agreement. The 12" hose is pressure rated to 400 psi, which should be sufficient for even the double lift zones, with at least a factor of safety of 1.6 (most areas, the factor of safety will exceed 3).
- The limiting factors in using the ULDH are as follows: assuming a flow rate of about 2,000 gpm, a 12" ULDH is practical over 2,000 feet or so. That means that isolation valves and suitable outlet manifolds would need to be placed ideally no more than about every 2,000 feet along the pipe. The existing pipe is about 24,000 feet long from Quarry Booster to Monte Bello Reservoir. Figure 5-18 shows that there are already about 10 such valves (with each pump station / reservoir set as being 1 effective valve). The preliminary locations for extra valves and outlets are shown schematically in Figure 5-19:
 - One valve downstream of Corte Madera PS, in the liquefaction zone.
 - Two valves at Corte Madera Pump Station. In final design, if the existing valves can be re-used, one or both of these valves might be eliminated. The existing fire hydrant outlet at Corte Madera could be modified on the downstream site to accept a 12" ULDH connection, and a new one installed on the upstream side.
 - One valve upstream of Corte Madera PS, about 2,000 feet uphill of the PS.

- Six outlet manifolds as shown.
- In final design, if funds are available a fifth valve upstream (with extra manifold outlets) of the Corte Madera PS would be useful, another ~2,000 feet upstream.
- For initial cost estimating purposes, assume that 2 new valves are cut into the backbone pipe CCP and 2 at the Corte Madera pump station. Assuming installation of each new valve is \$100,000 (in buried pipe) or \$50,000 (at the pump station). Include six outlet manifolds to allow attachment to ULDH. Assume that Palo Alto buys about 2,000 feet of ULDH, at about \$120 per foot, inclusive of the hose, fittings and flaking box, and has an emergency plan to obtain an additional 2,000 to 4,000 feet, if necessary, from EBMUD. Total cost is \$540,000. Assume soft costs (design and inspection) are \$40,000. Total cost is \$580,000.
- In final design, a review of the existing isolation valves will need to be made from original construction drawings; site inspections to determine the ideal locations for valve installation; hydraulic checks on flows via the bypass hose; discussion with EBMUD and others (City of Berkeley, etc.) as to availability of ULDH via mutual aid; etc.

Benefits: adding isolation valves and ability to use above ground bypass hose provides both earthquake and day-to-day maintenance flexibility. Deployment times can be as short as 4 hours, or possibly as long as 24 hours, requiring a crew of 2 or 3 people to lay out the hose, make the connections, and open the bypass valves and close the in-line valves. This will shorten post-earthquake outage time from several days to weeks to possible as short as a few hours. However, this option provides only limited benefit for fire fighting, as the time required for installation of the hose (a few hours or more) will not help control a fire within a single pressure zone (but might help re-supply an uphill zone that does need water for fire flows).

Option 4.

Upgrade suction pipe over Matadero Creek, inlet to Quarry Booster pump station. Assume \$40,000 (geotechnical investigation and design), two drilled piers (\$30,000), 100 feet of new pipe (\$36,000), total \$106,000.

Repair under-water creek crossing of Matadero Creek. This could be achieved by placing a reinforced concrete encasement around the existing encasement (capable of withstanding the full hydraulic forces), and placing some upstream and downstream scour protection. Cost: assume 40 feet of encasement (20 cubic yards of reinforced concrete @\$1,100 per yard placed, \$22,000, plus permitting and design (allow \$30,000), total \$52,000.

We also recommend that all other stream crossings be inspected annually, and similar repairs made should the concrete encasement be observed to be exposed.

5.5 Recommendation for Foothills Pipelines (Seismic)




We recommend that Palo Alto adopt Option 3 (valves and ULDH hose) plus Option 4 (Matadero Creek crossings) for the Foothills backbone pipe mitigation. The cost (\$738,000) is much lower than the pipe replacement strategy (\$16,493,000).

Since the existing CCP appears to have no age-related weaknesses (leak rate under 0.01 repairs per mile per year), there is no ongoing need for CCP pipe replacement in the Foothills. Should conditions change such that the leak rate of the CCP pipe increases to 0.3 or 0.4 repairs per mile per year, over a 5-year time frame, then pipe replacement of the segments with multiple leaks within a year's time frame, would be recommended.

New pipelines. As Palo Alto continues to upgrade and modify its water system, new pipelines will be installed. It is recommended that all new pipelines be designed for seismic provisions in ALA (2005). Essentially, for backbone pipelines in areas prone to liquefaction, surface faulting or landslide, this requires that the new pipelines have special seismic features, which could be either:

- Use chained pipe in liquefaction zones and zones with modest (under 12 inches or so) fault offset. Ductile Iron Pipe from Kubota with seismic joints fulfills these requirements. Butt welded HDPE pipe also meets these requirements (but should not be used where operating pressures exceed about 125 psi).
- Use ductile welded steel pipes in liquefaction zones, modest landslide zones, large fault offset zones. Use butt welds in these zones for pipes under 30-inch diameter.
- All new metal pipelines should be designed with corrosion protection, possibly using sacrificial anodes, especially where Rho is likely to be under 3,000 ohm-cm. New metal pipelines in areas with Rho likely to be under 1,500 ohm-cm will require corrosion design, possibly with impressed current.

Add ~ 3 to 4 in-line valves
 Add ~ 6 outlet manifolds
 Buy ~ 1,000 feet of ULDH

-  New Isolation Valve
-  New Outlet Manifold
-  Pipe Upgrade at Creek Crossing (see text)

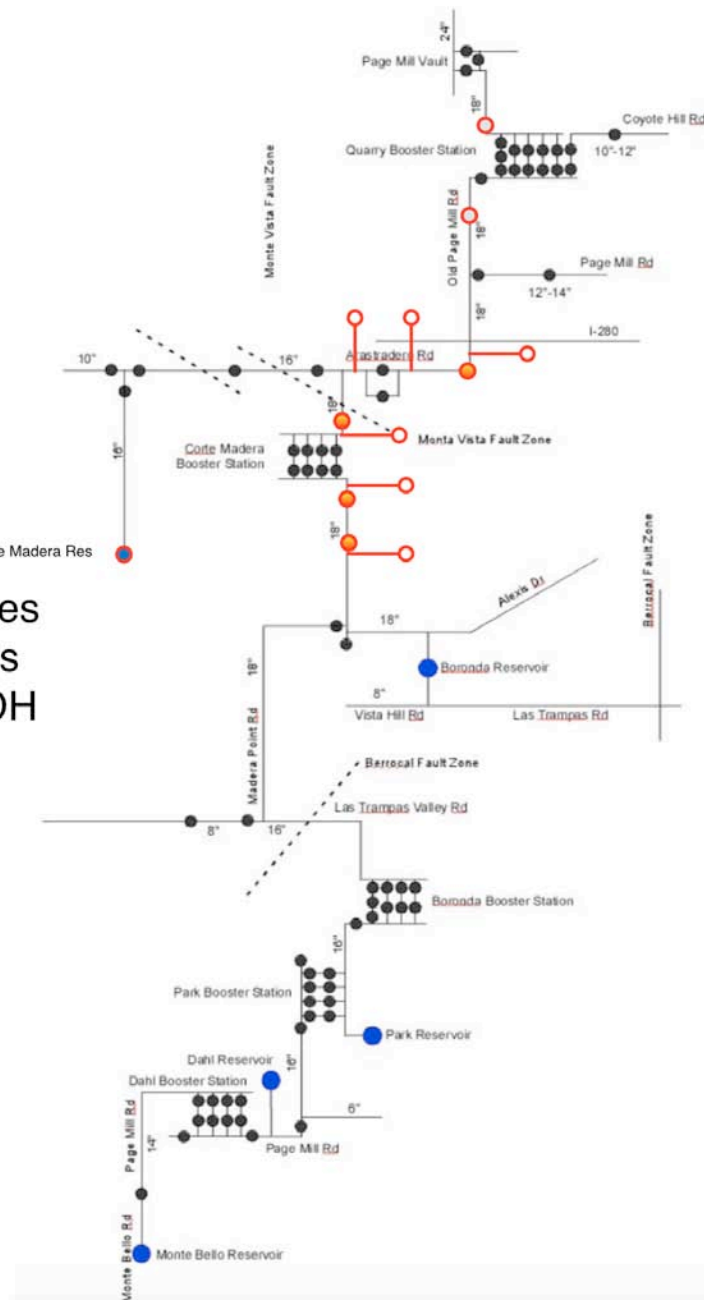


Figure 5-19. Recommended Upgrades for Foothill Pipe Network

6.0 Pipe Aging, Repair Analysis and Pipe Replacement

One of the methods to help decide whether or not a pipe has reached the end of its economic lifetime is to examine its leak rate. Leaks cause loss of water, shutdown of a pipe to make a repair, and sometimes the leaking water inundates nearby properties. There is a cost to each of these items. Further, while the pipe is shutdown to make repairs, there is economic impact to the customers who have loss of water supply.

There are other factors that lead to the decision to replace an existing water pipe. These include:

- Increased water demands in the area mean that the existing pipe has too small a diameter
- The existing pipe may have internal tuberculation, whereby its original diameter has shrunk to the point that the quantity of water available for flow (and pressure) is significantly reduced, leading to day-to-day customer complaints (less often) or much reduced fire flows (rarely needed, but when needed, a loss of fire flow can have very serious consequences).
- The pipe must be relocated, due to other needs (like construction of a new road; construction of other utilities, etc.)

6.1 Historical Pipe Repairs

Palo Alto provided us with an Excel file with a history of pipe repairs in the water system. The oldest repair was dated March 12, 1990, and the most recent repair was September 24, 2014. This suggests the database reflects 24.58 years of data.

We examined the repair history of water pipes in Palo Alto. The repair rate measure that we adopted is "repairs per mile per year". For example, if a 1,000-foot long piece of pipe has had 1 repair in 4 years, and 2 repairs since it was installed 60 years ago, then its leak rate: is:

- Last 4 years = $1 / (4 * 1000/5280) = 1.32$ repairs per mile per year
- Last 60 years = $2 / (60 * 1000/5280) = 0.176$ repairs per mile per year

To do this, we took the following steps.

Palo Alto provided us with a GIS Shapefile identifying 1,101 repairs of pipes, dating from 1990. We first removed mechanically-driven repairs, such as repairs caused by an

accident (vehicle hitting a hydrant) or construction excavation (digging and damaging pipe), since these are not indicative of pipe aging. These repairs numbered 170, leaving 931 repairs to be considered.

Next, we sorted between repairs made to transmission and distribution mains (Table 2-7, 232 miles of pipe) from those to service laterals or hydrant laterals (Table 2-9, 104 miles of pipe), as noted in the Shapefile database.

We then computed the closest current pipe segment to the remaining 931 reported repairs.

We then manually examine the description of the pipe repair (like "repair leak on 6-inch cast iron pipe") with the GIS description of the closest pipe segment; ideally these would match. However, in reviewing the GIS-matched repairs and pipes, we noted that there were many discrepancies. Most discrepancies were mismatches in pipe material in the GIS-based map inventory and the pipe material reported in the repair database. We suspect that the main reason for these mis-matches is that many of the repairs in the repair-database were made to pipes that have since been abandoned by Palo Alto (see Figure 2-37). Since about 1990, Palo Alto has been actively replacing pipe, and in part, this replacement program was based on the number of historical repairs a pipe had experienced.

We then obtained the abandoned pipe database (Figure 2-37) and manually reviewed each of the transmission and distribution repairs, assigning the repair to either a current or abandoned GIS-coded pipe. We also reviewed the population of repairs flagged in the GIS as being on service laterals, moving any repairs which could be construed as involving the transmission / distribution main (such as at a main/lateral connection) into either the current or abandoned main groups. Two pipe databases are maintained in this process: those pipes that are currently "Active" in the water system (thin blue lines in Figure 2-37), and older pipes that have already been abandoned (thick orange lines in Figure 2-37). For the abandoned pipes database, the pipe material and size were often not available. Tables 6-1 and 6-2 list the repairs in the Active and Abandoned pipe mains.

For the pipes that are still part of the Palo Alto water system, there are 282 repairs recorded over 24.58 years. In preparing Tables 6-1 and 6-2, we removed all repairs that were caused by contractor activities (like damaging the pipe due to hitting it with a backhoe, etc.).

Diam (inches)	ACP	CCP	CIP	DIP	PVC	Total
4	24		1			23
6	109		34		8	151
8	48		12			60
10	8		7			15
12	18		4	1	1	24
14	3		1			4
16		3				3
Total	210	3	59	1	9	282

Table 6-1. Leak Repairs in Past 24.58 Years on Active Pipe Mains

Diam (inches)	ACP	CCP	CIP	DIP	PVC	Total
2		1		1		2
4	6	23				29
6	39	115	1			155
8	8	19				27
10	18	6				24
12	7					7
Unknown		1			1	2
Total	78	165	1	1	1	246

Table 6-2. Repairs in Past 24.58 Years on Abandoned Pipe Mains

As seen in the repair data presented in Tables 6-1 and 6-2, most of the repairs have occurred in ACP and CIP. The Palo Alto water main replacement program appears to have targeted CIP first, as evidenced in the geographic locations of the pipe replacements (see Figure 2-37). For the current Active inventory, there are more repairs in ACP than CIP, likely because much of the worst-performing CIP has already been replaced.

The remaining 403 repairs not otherwise attributed to construction / accidental impacts, were judged to be on service laterals, hydrants or meters. The repair information for these lines is tabulated in Table 6-3. It should be noted that some (possibly in the range of 5%) of these repairs have been on abandoned service lines, but the information is not reliable, so no attempt to separate them from the current inventory was made.

Diam (inches)	ACP	CIP	CU	DIP	GI	PVC	Steel	Unknown	Total
0.75			17		61			1	79
1		3	194		15	2		1	215
1.25			3		3				6
1.5			4		6		1		11
2			15	1	9	2			27
2.5		2							2
4	7				1	1			9
6	3	3						1	7
8		1							1
10		1							1
Unknown			2					43	45
Total	10	10	235	1	95	5	1	46	403

Table 6-3. Repairs in Past 24.58 Years on Service Laterals, Hydrants and Meters

6.2 Pipe Repair Rate as a Function of Material and Diameter

For each material and diameter of pipe, we then tabulated the total length of pipe, and then calculated the repair rate per mile per year (over 24 years, 7 months, the period of time where we have repair data. The results are in Table 6-4.

Diam (inches)	ACP	CCP	CIP	DIP	PVC
4	0.125		0.062		
6	0.076		0.132		0.007
8	0.048		0.094		
10	0.059		0.143		
12	0.053		0.090	0.029	0.014
14	0.029		0.386		
16		0.023			
Total	0.064	0.012	0.112	0.006	0.006

Table 6-4. Historical Repair Rates by Pipe Material and by Pipe Diameter (for Active pipes)

In the bottom row of Table 6-4, the "total" repair rate is for all leaks on that type of pipe, divided by the total length of all diameters of that pipe; this value is not the average of the above rows, as there are some pipe diameters with no pipe repairs.

Not all entries in Table 6-4 are of "equal importance". For example, the repair rate of 0.023 for CCP (16") and zero for CCP (14") should not be interpreted that somehow 16" CCP is weaker than 14" CCP. For example, see in Table 2-6, that the length of CCP in the entire system is 55,684 feet. Thus, over a 24.58 year period, there have been 3 leaks in CCP, for a 24.58-year average repair rate of: $3 / (24.58 * (55,684/5,280)) = 0.012$. While the 3 leaks in the 16" CCP reflects about half (27,671 feet) of the entire CCP

inventory, some judgment needs to be applied before extending the repair rate over all different diameters of CCP.

One unusual result in Table 6-4 is that 4" CIP has a much lower repair rate than 6" CIP. Historically, the rate of repair for smaller diameter CIP (especially 4" and below) has been higher than for larger CIP (6" and above). For Palo Alto, two important distinctions are made:

- First. The 4" CIP has a very small inventory currently in use (3,447 feet), while the 6" CIP has a much larger inventory (55,193 feet), and CIP as a whole has an inventory of 113,568 feet. Therefore, the raw calculation of a repair rate of 0.064 for 4" CIP, based on only a single repair, may just be a statistical anomaly.
- Second. There was a much higher historical repair rate for CIP, but over the years, Palo Alto has removed (abandoned) a lot of CIP, targeting for removal those CIP with high leak rates. Section 2.5 examines the abandoned pipe inventory in more detail. This means that the worst performing CIP has already been largely removed from the system (165 repairs to 32.11 miles of CIP over 24.58 years, or a repair rate of 0.209 per year, with some adjustment up or down for uncertain duration of year the abandoned pipe was in the ground, (increases the repair rate) and how much of the "unknown" pipe material was cast iron (lowers the repair rate). In any case, the repair rate of the currently Active CIP (0.064) is a lot lower than the repair rate for the abandoned CIP (~0.209), showing that the historical pipe repair program did target the worst-leaking pipes. Of more importance to the current discussion, is the remaining CIP "good enough" with a repair rate of 0.064, or should the remaining active CIP also be replaced in the medium term (next decade or so).

Given these issues, we developed Table 6-5, which relates the repair rate by pipe type by diameter. Table 6-6 extends the data for other types of pipe in the Palo Alto system. Tables 6-5 and 6-6 can be used if one does not have any other information about the specific pipe, such as age, environmental effects (soil resistivity) or water chemistry. In Tables 6-5 and 6-6, the first row can be used for pipes for which diameter information is missing.

Diam (inches)	ACP	CCP	CIP	DIP	PVC
Unknown	0.064	0.012	0.112	0.006	0.006
<4	2.0	1.0	1.4	1.2	1.1
4	2.0	1.0	1.4	1.2	1.1
6	1.1	1.0	1.2	1.2	1.1
8	0.9	1.0	1.0	1.1	1.0
10	0.8	1.0	0.9	1.0	0.9
12	0.7	1.0	0.8	1.0	0.8
14	0.6	1.0	0.7	0.9	0.8
16	0.5	1.0	0.6	0.9	0.8
18	0.5	1.0	0.5	0.8	0.8
20	0.5	1.0	0.5	0.7	0.8
24	0.5	1.0	0.5	0.6	0.8
27	0.5	1.0	0.5	0.6	0.8
30	0.5	1.0	0.5	0.6	0.8

Table 6-5. Leak Rates by Pipe Material and by Pipe Diameter

To confirm the validity of Table 6-5, we combined these repair rates, with the actual pipe lengths from Table 2-6, and arrived at a total of 286 forecast repairs, which is essentially the same as the actual. Notes for Table 6-5:

- There is very small inventory of pipe under 4-inch diameter (0.38% of total). Very small diameter pipe generally has historically had higher repair rates.
- Only a small diameter dependency factor is given PVC. This reflects that the effects of corrosion on PVC wall thickness is thought to be much less than for other types of pipe, and thus the effects on smaller diameter PVC pipe (with thinner walls) should be less pronounced.
- There is currently (as of 2010) about 10 miles of HDPE pipe in the system. To date, there have been no repairs.
- There is currently (as of 2010) about 0.2 miles of steel pipe in the system. To date, there have been no repairs.

- There is currently (as of 2010) about 0.3 miles of copper pipe in the system (mains that perhaps could be classified as laterals). To date, there have been no repairs. However, there have been a number of repairs to copper pipe classified as service laterals (Table 6-3), and that data would suggest a repair rate of about 0.13 repairs per mile per year. We conservatively use 0.20 for the present analyses.
- In Table 6-6, for pipes that currently have "unknown" as the pipe material, we assume it is most likely to be Cast Iron pipe. This reflects that cast iron pipe is the oldest pipe in the system, and thus most likely to have "poor records". Of course, if one could do additional research for a particular pipe and find its actual pipe material, that is preferred.

Diam (inches)	HDPE	CU	Steel	Unknown
Unknown	0.01	0.20	0.150	0.112
<4	1.0	1.0	4.0	1.4
4	1.0	1.0	2.0	1.4
6	1.0	1.0	1.0	1.2
8	1.0	1.0	0.8	1.0
10	1.0	1.0	0.7	0.9
12	1.0	1.0	0.6	0.8
14	1.0	1.0	0.6	0.7
16	1.0	1.0	0.5	0.6
18	1.0	1.0	0.5	0.5
20	1.0	1.0	0.5	0.5
24	1.0	1.0	0.5	0.5
27	1.0	1.0	0.5	0.5
30	1.0	1.0	0.5	0.5

Table 6-6. Repair Rates by Pipe Material and by Pipe Diameter

We did the same repair rate calculations for service laterals, with the results shown in Table 6-7. Since we do not have good information regarding abandoned laterals and precise location of leaks, we used all 403 reported repairs against the entire lateral database (assuming that both the repair and service lateral databases include current and abandoned pipes). We believe that this gives a credible first-order estimate of the historical repair rate for the 24.58-year history.

Diam (in)	ACP	CIP	CU	DIP	GI	PVC	Steel	Unk	Total
Unk			0.417						0.086
0.75			0.350		20.3			0.136	1.340
1		15.7	0.122		5.89	1.26		0.047	0.133
1.25			0.171		3.11				0.140
1.5			0.064		36.8				0.169
2			0.177	9.76	14.3	1.56			0.222
2.5									
4	0.450				9.76	0.049			0.182
6	0.118	0.673						0.022	0.055
8		0.460							0.020
10		4.57							0.147
Total	0.175	0.977	0.129	0.093	12.4	0.078	0.260	0.073	0.151

Table 6-7. Repair Rates by Pipe Material and by Pipe Diameter, Service Lines

The repair rates in Table 6-7 vary widely, mostly because of limited lengths in some pipe sizes. For example, the repair rate for 1.5-inch galvanized iron pipe is 36.8 repairs/mile/year. There are 6 leaks for only 35 feet of pipe, which yields such a large repair rate. The following observations can be made:

- There are only 652 feet of galvanized iron pipe left in the current system; 996 feet have already been abandoned/replaced. As apparent by the large leak rate for galvanized iron, it is prudent to plan to replace all galvanized iron pipe.
- The ACP and CIP populations (4-inches and larger) in Table 6-7 could be added to that of the mains to get a slightly more accurate repair rate for those two pipe materials. There is essentially no difference between mains and service laterals. Adjusted repair rates are suggested in Table 6-8.
- Similarly, the copper pipe in the mains can be combined with that in the laterals. The length of copper pipe in the mains database is 1,399 feet, all 2-inches or smaller in diameter. There are 377,405 feet of copper pipe in current service lines. Table 6-8 shows the combined repair rates for all copper pipe.
- The repair rate for PVC service lines is somewhat higher (0.078) than for larger diameter pipe used for mains (0.006). This may reflect a higher level of quality control for larger mains than for smaller service lines; but the diameter adjustment parameter of 1.1 (Table 6-5) could be increase to about 1.3 for PVC pipe < 2 inches. There are 223,536 feet of PVC mains and 12,966 feet of PVC service lines.
- There are 15,451 feet of PE service lines, and no repairs are reported.

Diam (inches)	ACP	CIP	CU
0.75			52.2
1			3.89
1.25			0.002
1.5			0.214
2			0.232
3			
4	0.151	0.052	
6	0.077	0.142	
8	0.047	0.100	
10	0.058	0.162	
12	0.053	0.090	
14	0.029	0.386	
16			
Total	0.065	0.128	0.129

Table 6-8. Adjusted Repair Rates by Pipe Material and by Pipe Diameter, Combined Active Mains and Hydrant / Meter Laterals

6.3 Pipe Repair Rate as a Function of Age

Figures 6-1 and 6-2 show the length of Active pipe in the current Palo Alto system, as a function of its age. Notes for Figures 6-1 and 6-2:

- The horizontal axis shows the age of the pipe, in years. The age is computed from the year of installation (as recorded in the GIS) to the present time (2015). Therefore, a pipe installed in 1950 would have an age of 65 years.
- To increase legibility, the age on the horizontal axis is only shown for every second bar.
- About 32% of all pipes in the database do not have a "year installed". The data in Figure 6-1 excludes pipe that have "no age" attribute.
 - About 75% of all CIP have "no age" attribute: the majority of all cast iron pipe was installed pre-1940, some possibly as old as the late 1800s.
 - About 31% of ACP have "no age" attribute. AC pipe was installed mostly from 1940 to 1970.
 - About 85% of CCP have "no age" attribute. A lot of CCP was installed between 1960 and 1965.

- About 49% of DIP have "no age" attribute. DIP was installed between 1994 up to the current time.
- Nearly all HDPE pipe have the age attribute. Most HDPE pipe has been installed between 2005 and the current time.
- About 7% of PVC have "no age" attribute. Most PVC has been installed between 1987 up to the current time.

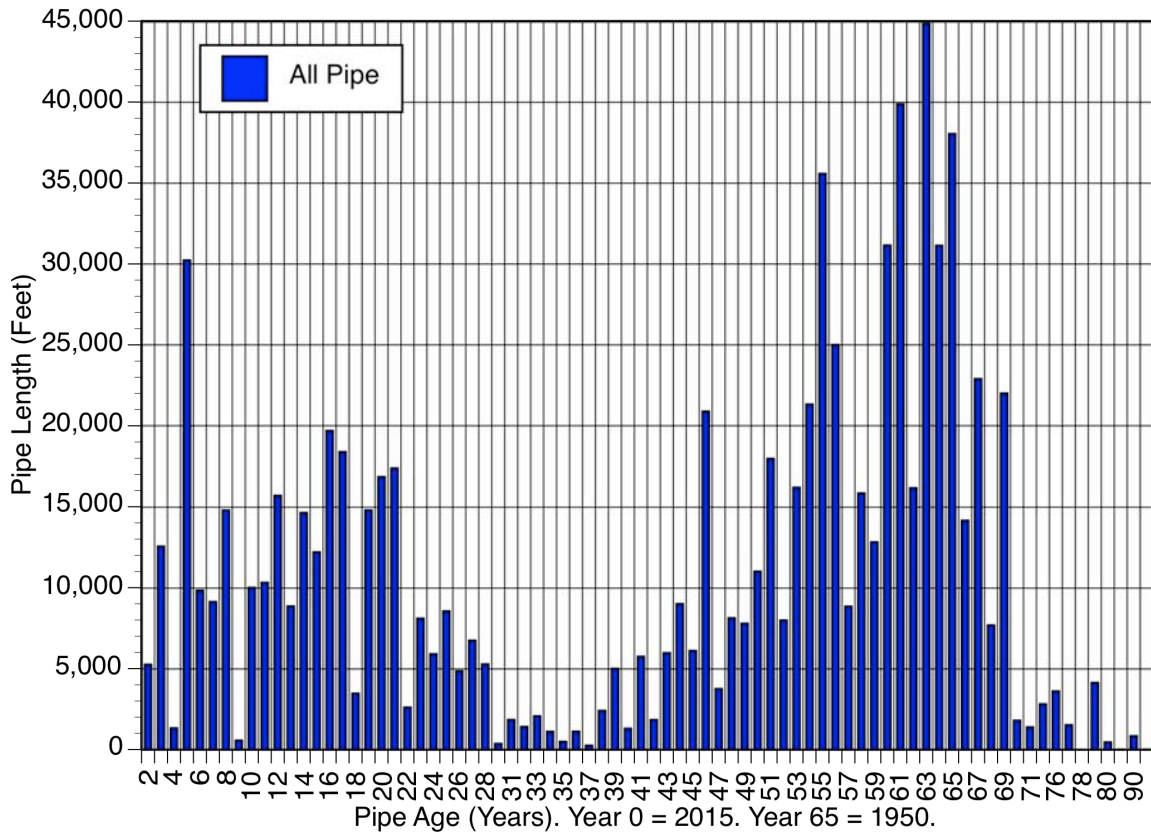


Figure 6-1. Length of Pipe versus Age

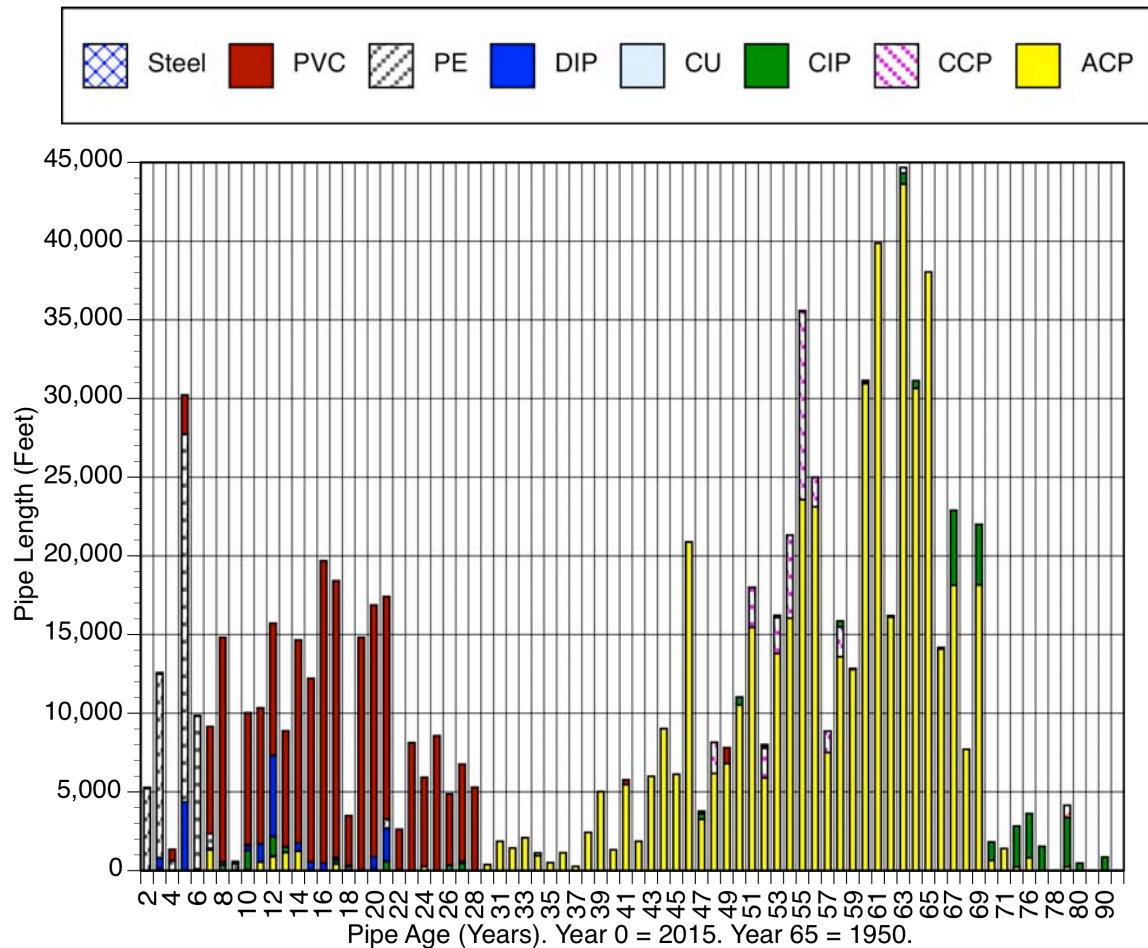


Figure 6-2. Length of Pipe, by Material, versus Age

To determine the repair rate by age, we assigned each historical repair to a specific pipe.

6.3.1 Pipe Repair Rate as a Function of Age – AC Pipe

The current largest number of historical leaks on active pipes have been to AC pipes. AC pipe is also the most widely used pipe in the water system. Therefore, we concentrate the analysis of age-based leak rates for AC pipe.

Figure 6-3 shows the repair rate (repairs per mile per year) for ACP. In Figure 6-3, a data point is included for each year with ACP pipe installed. A regression model (blue line) shows a best fit through all the data.

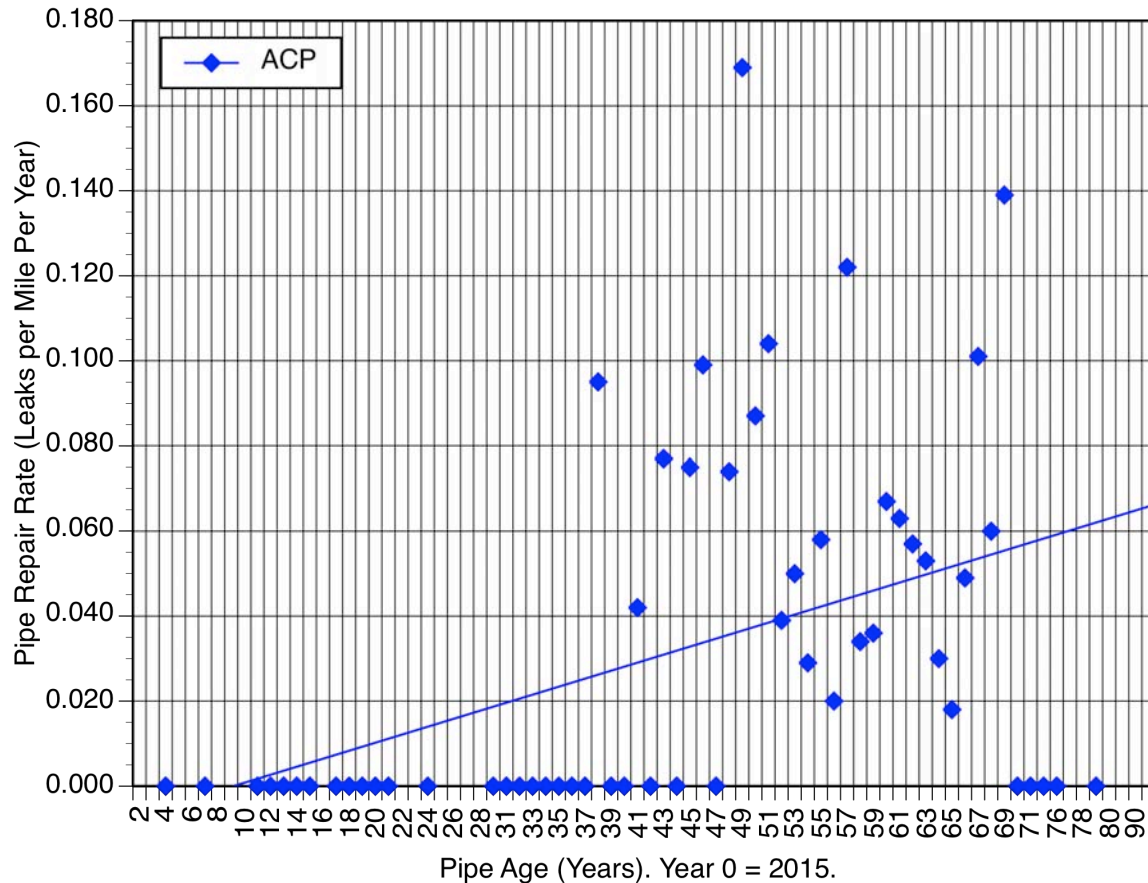


Figure 6-3. Repair Rate, ACP, versus Age (all data)

$$RR = -.007712 + 0.0009426 (\text{age}), RR \geq 0 \text{ (Figure 6-3)}$$

- RR = repairs per mile per year
- Age = age of pipe in years

The raw data in Figure 6-3 shows a zero repair rate for all years for pipe ages younger than 37 years, and zero repair rate for all pipe ages 70 years or older. There is very little ACP inventory for pipes of those ages (under 37 years or older than 70 years), so it would be incorrect to base a regression model using that data. Accordingly, Figure 6-4 shows only the repair rate data from ACP pipe installed from 1945 and 1977. This data shows a modestly increasing leak rate with age.

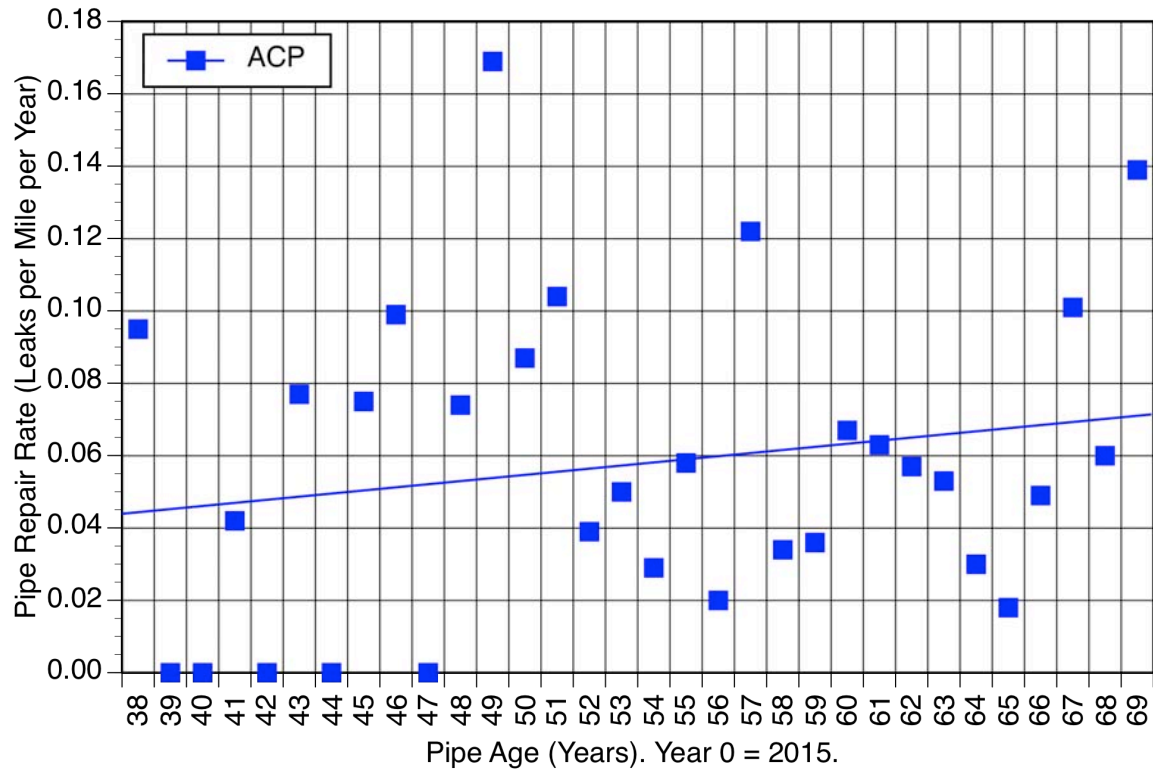


Figure 6-4. Repair Rate, ACP, versus Age (1945-1977)

$RR = 0.004353 + 0.0008602 (\text{age})$, $RR \geq 0$ (Figure 6-4)

- RR = repairs per mile per year
- Age = age of pipe in years

Figure 6-5 shows a combined plot for ACP, showing both the repair rate (blue boxes) and pipe length (red dots), by year. Figure 6-5 shows that for ACP with Age between 38 to 45 years, there was not much length of pipe installed per year (some years less than 2,000 feet), whereas it was common to install 20,000 to 40,000 feet of pipe in the older pipe (Ages 46 to 69 years).

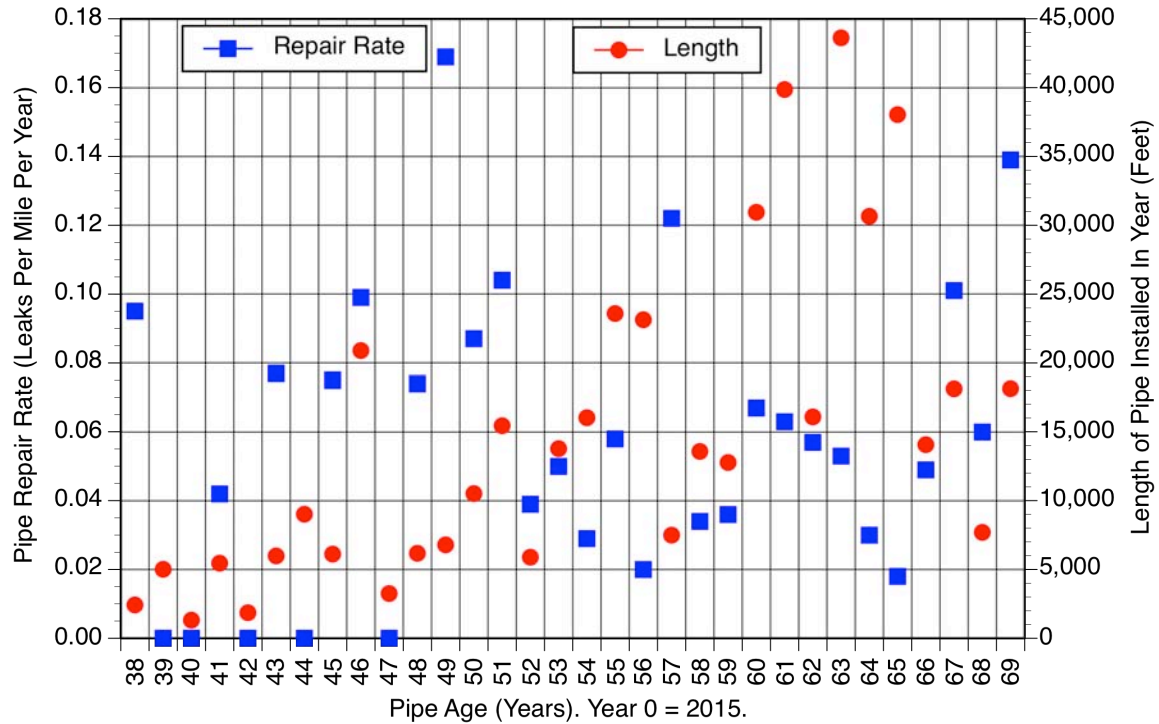


Figure 6-5. Repair Rate and Pipe Length, ACP, versus Age (1945-1977)

Figure 6-6 shows this dataset for ACP with Ages between 46 and 69 years, along with a regression model through the data.

$$RR = 0.006972 - 0.0003843 (\text{age}), RR \geq 0 \text{ (Figure 6-6)}$$

- RR = repairs per mile per year
- Age = age of pipe in years

In Figure 6-6, the regression line shows a somewhat *decreasing* leak rate with age. This result seems counter-intuitive, as it would appear to say:

- Older ACP pipe performs somewhat better than younger ACP

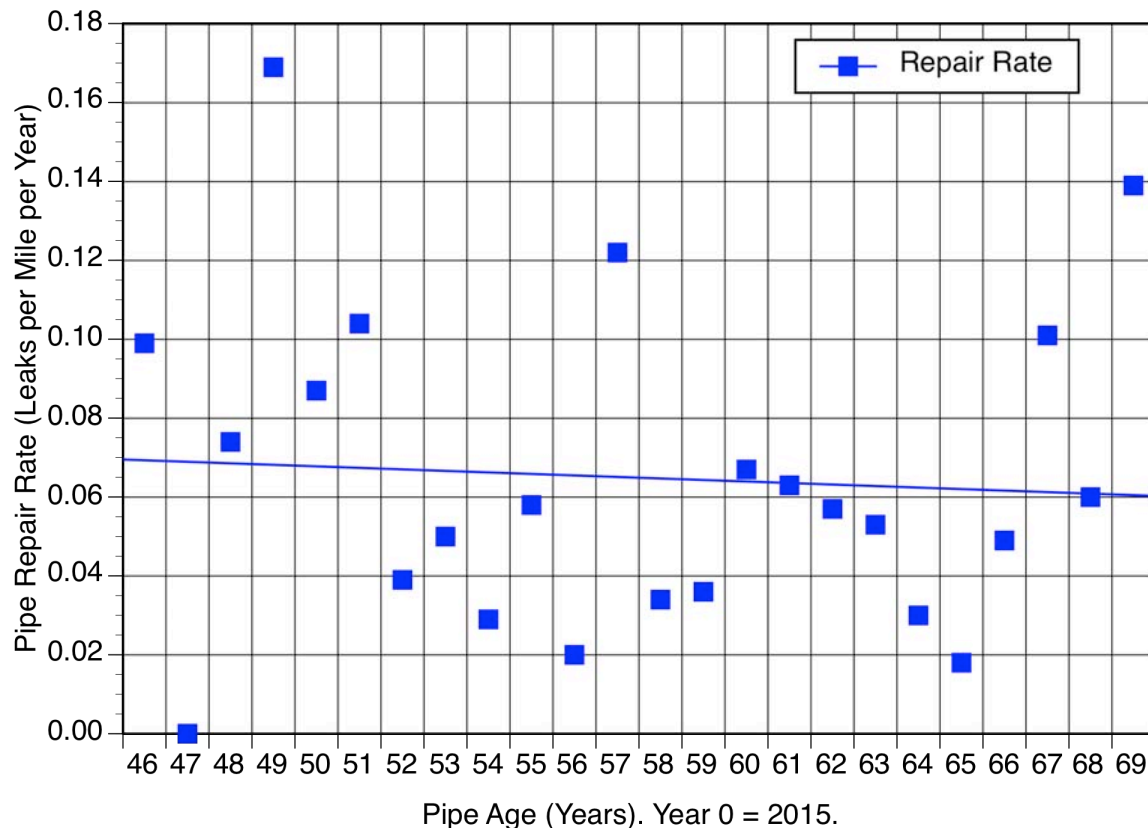


Figure 6-6. Repair Rate and Pipe Length, ACP, versus Age (1945-1969)

Even more "accurate" analyses of the repair rate dataset could be made, by equal weighting each data point by length of pipe. However, we think this is extra "refinement" will not lead to any better results.

The key findings are as follows:

- ACP in Palo Alto has had, so far, a long-term average repair rate of about 0.065 repairs per mile per year.
- There is strong evidence that smaller diameter ACP (4") has had nearly twice the repair rate than larger diameter ACP.
- There is weak evidence that older ACP has a higher repair rate than younger ACP.

This last point is of crucial importance. Many pipe replacement strategies are based on using pipe age as an important (along with pipe material and diameter) factor in deciding whether or not to replace a pipe. This concept assumes that as pipes get older the cumulative effects of internal and external corrosion should lead to pipe wall thinning; and with sufficient pipe wall thinning, the internal water pressure (including effects of cycling) will eventually burst the pipe. While we agree that this may be the common case for metallic pipe (CIP, DIP, Steel, CCP, etc.), the factual evidence in Palo Alto, for ACP, does not support this finding, at least not yet.

We believe that the actual failures in ACP are mostly due to the following:

- Soil movements. ACP pipe is rather brittle, and with excessive bending on the pipe, the barrel breaks. Soil movements occur yearly in clay-like soils, due to shrink-swell cycles.
- Stress risers. When inserting a service lateral, a clamping mechanism is often used. This clamp will locally compress the pipe, but at the edges, introduce local bending. The local bending causes high tensile stresses, and this can lead to initiation of pipe failure.
- Earthquake ground shaking. We think that the rubber gaskets at most ACP pipe joints will serve adequately to avoid build up of stresses in the pipe barrel, due to moderate levels of ground shaking: only a few ACP pipes will leak due to rubber gasket pullouts.
- Earthquake ground deformation. In areas underlain by infirm soils (liquefaction, landslide, etc.), ACP pipe will be deformed to match the movements of the permanent ground displacements (PGDs) or the ground. At PGDs of an inch or more, high bending stresses are introduced into the ACP pipe barrel. Break rates in ACP due to PGDs can be very high.

6.3.2 Pipe Repair Rate as a Function of Age – CI and PVC Pipe

Similar evaluations were performed for cast iron and PVC pipes, which had some repair data. Figures 6-7a, b show the repair rate and pipe length versus age for CIP. In Figure 6-

7b, the high repair rate for CIP pipe aged 60 years is included in the regression model, but the vertical scale is limited to 0.60, so that the trend line is easier to read.

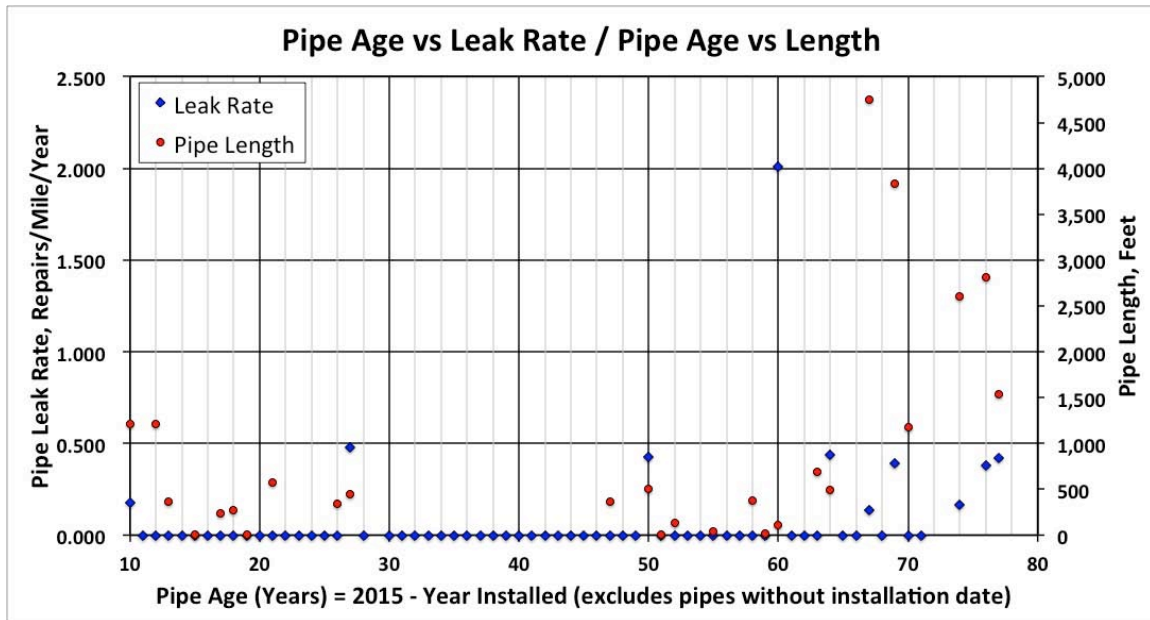


Figure 6-7a. Repair Rate and Pipe Length, CIP, versus Age (1935-2005)

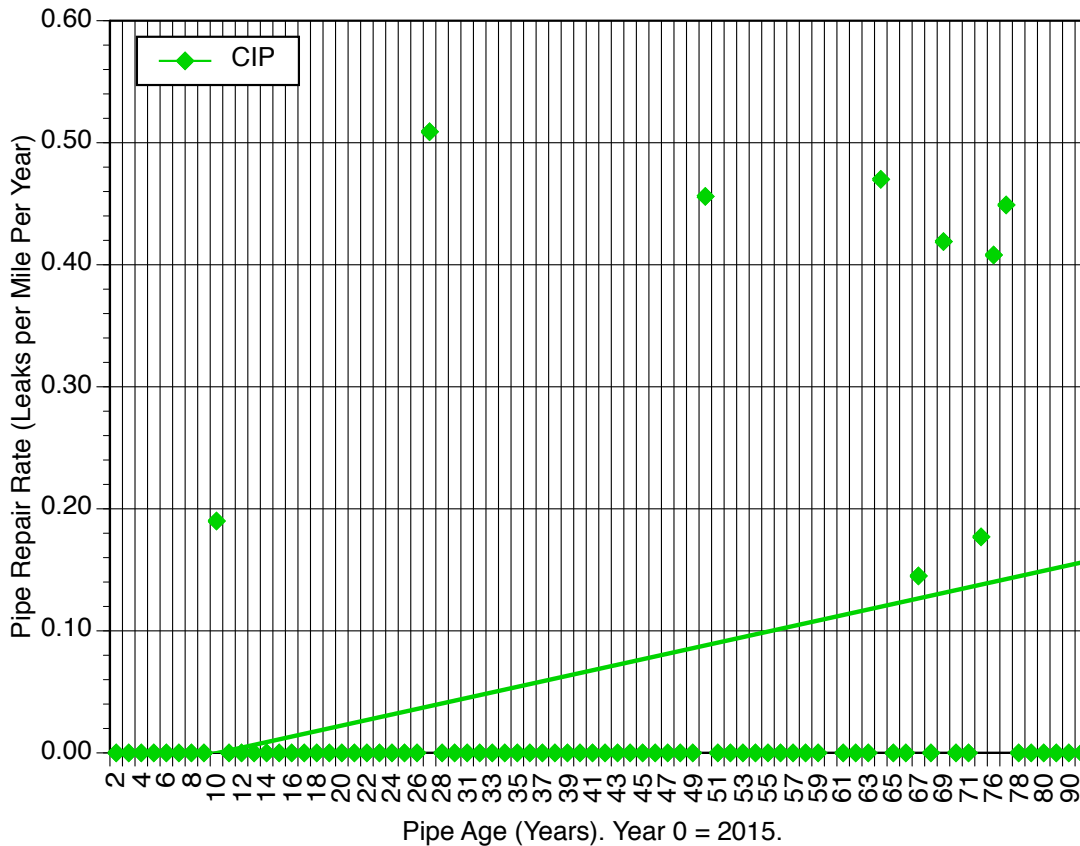


Figure 6-7b. Regression Model for Repair Rate, CIP, versus Age

Figures 6-8a, b show the repair rate and pipe length versus age for PVC.

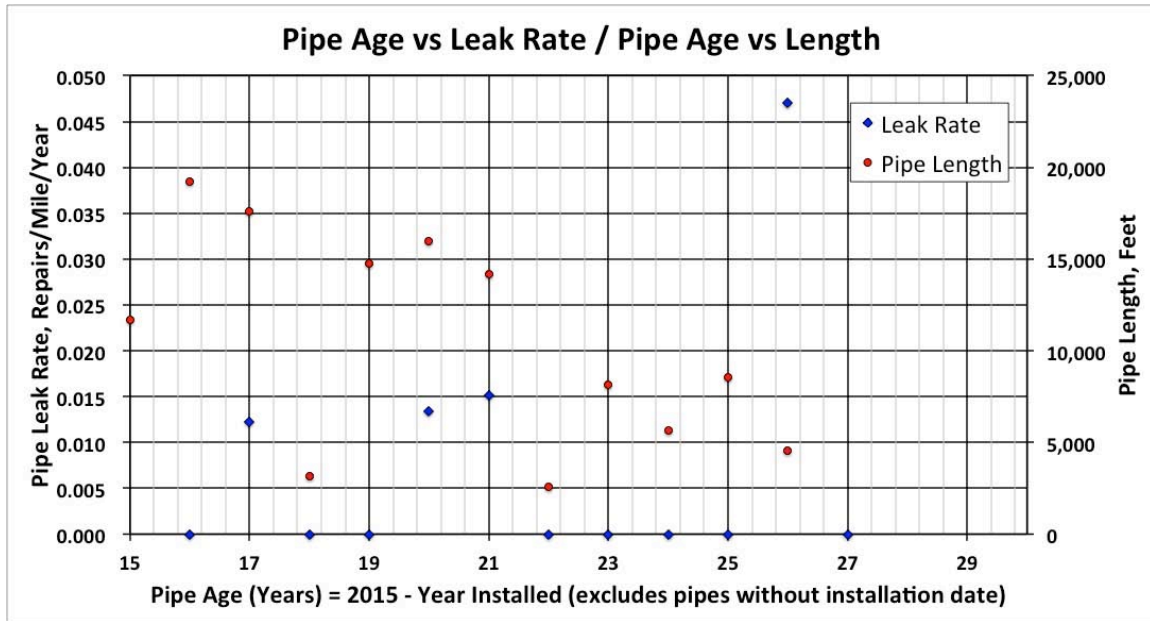


Figure 6-8a. Repair Rate and Pipe Length, PVC, versus Age (1985-2005)

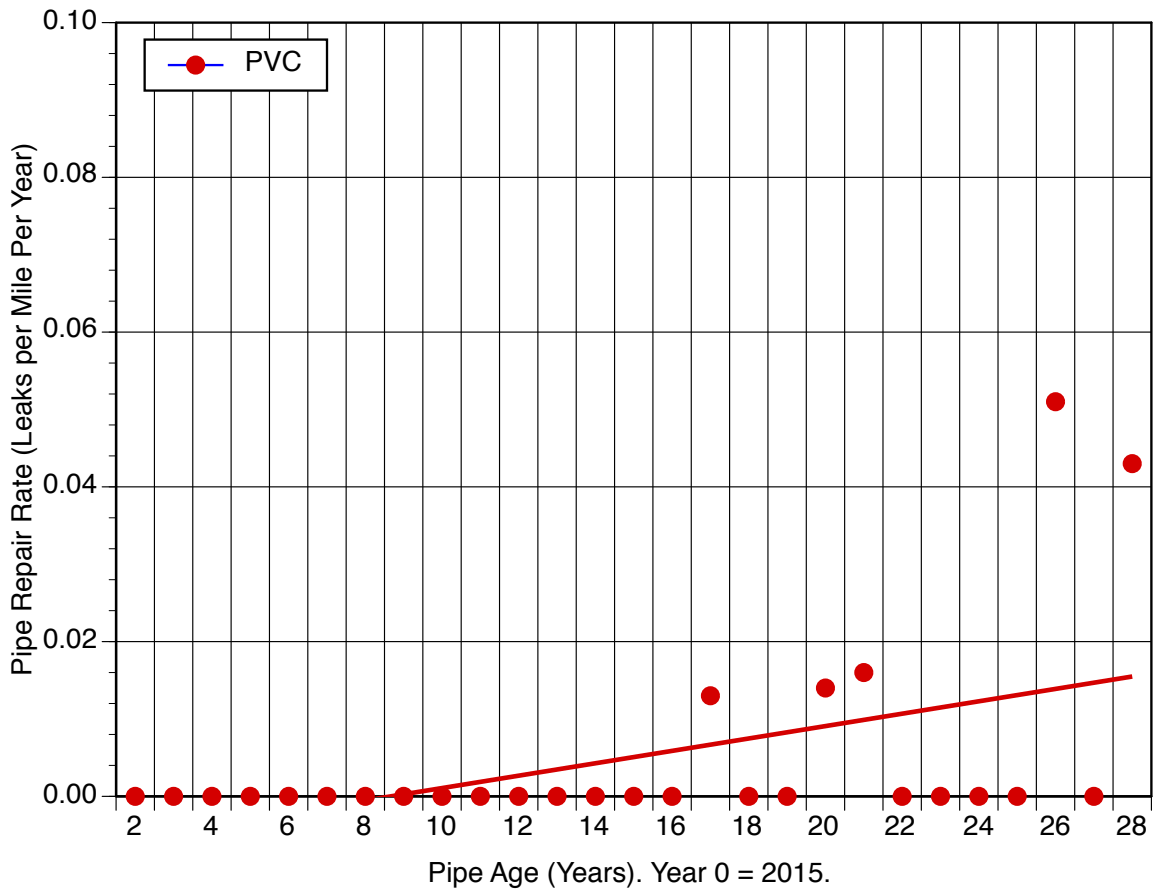


Figure 6-8b. Regression Model for Repair Rate, PVC, versus Age

$RR = -0.002062 + 0.002264(\text{age}), RR \geq 0$ (Figure 6-7b Cast Iron Pipe)

- RR = repairs per mile per year
- Age = age of pipe in years

$RR = -0.006140 + 0.0008010(\text{age}), RR \geq 0$ (Figure 6-8b PVC Pipe)

- RR = repairs per mile per year
- Age = age of pipe in years, $\text{Age} \leq 28$ years

In Tables 6-5 and 6-9, we adopt the following diameter and age models for AC, Cast Iron and PVC Pipe.

- $RR = 0.064$ if the age and diameter of the ACP are unknown. Small diameter ACP (4 inches and below) are 4 times more likely to leak than large diameter (≥ 16 inches) ACP. "Middle age" ACP (over 40 years old) are much more likely to leak than young ACP (under 40 years old; with a moderate increase in leak rate for very old ACP (81 year and older). As there are no ACP pipe that are older than about 81 years, the increase in leak rate for 100+ year old ACP is speculative, recognizing that the empirical data (up to about 70 year old) shows no increase in repair rate for pipes from 46 to 70 years old.
- $RR = 0.112$ if the age and diameter of the CIP are unknown. Small diameter CIP (4 inches and below) are about 3 times more likely to leak than large diameter (≥ 18 inches) CIP. Older CIP pipe, if Rho is unknown, are assumed to have a steeply increasing rate of repair; this is evident in the empirical data (Figure 6-7b). If Rho is known, this is further adjusted in Table 6-11, recognizing that CIP (without suitable corrosion protection) in non-aggressive soils should have a longer life / fewer repairs than in CIP in aggressive soils.
- $RR = 0.006$ if the age and diameter of the PVC are unknown. Small diameter PVC (6 inches and below) are about 38% times more likely to leak than large diameter (≥ 12 inches) PVC. Older PVC pipe, are assumed to have a slightly rate of repair; this is evident in the empirical data (Figure 6-8b).

From the results of the Palo Alto pipe age survey, and judgment, the “age factors” reported in Tables 6-9 and 6-10 are used in the reliability evaluation of pipe segments.

Pipe Age	ACP	CCP	CIP	DIP	PVC
Unknown	1	1	1	1	1
1-10	0.4	0.8	0.5	0.8	0.9
11-20	0.6	0.8	0.6	0.9	0.9
21-30	0.8	0.8	0.7	1	1
31-40	1	0.8	0.8	1	1
41-50	1.2	0.8	0.9	1.1	1
51-60	1.2	0.9	1	1.1	1.1
61-70	1.2	1	1.2	1.15	1.1
71-80	1.2	1.1	1.4	1.2	1.1
81-90	1.4	1.2	1.6	1.3	1.2
91-100	1.6	1.2	1.8	1.4	1.2
101-110	1.8	1.3	2	1.5	1.2
111-120	2	1.4	2	1.7	1.2
>120	2	1.5	2	2	1.2

Table 6-9. Repair Rates by Pipe Age, Active Mains

Pipe Age	HDPE	CU	Steel	Unknown
Unknown	1	1	1	1
1-10	0.95	0.75	0.8	0.5
11-20	0.95	0.75	0.8	0.6
21-30	1	1	0.9	0.7
31-40	1	1	0.9	0.8
41-50	1.05	1.25	0.95	0.9
51-60	1.05	1.25	0.95	1
61-70	1.1	1.5	1	1.2
71-80	1.1	1.5	1.2	1.4
81-90	1.15	2	1.4	1.5
91-100	1.15	2	1.6	1.6
101-110	1.2	2.5	1.8	1.7
111-120	1.2	2.5	2	1.8
>120	1.2	2.5	2.5	2

Table 6-10. Repair Rates by Pipe Age, Active Mains

6.4 Pipe Repair Rate as a Function of Month, Year

We counted the total number of leaks for active pipes, since 1990, and tabulated them by month and by year.

The tabulation by month is intended to see if there is any seasonality in the incidence of pipe repairs. Figure 6-9 shows the raw data, for all pipe types, by month (if the date of the repair was unknown, it is shown in the rightmost column, "UNK").

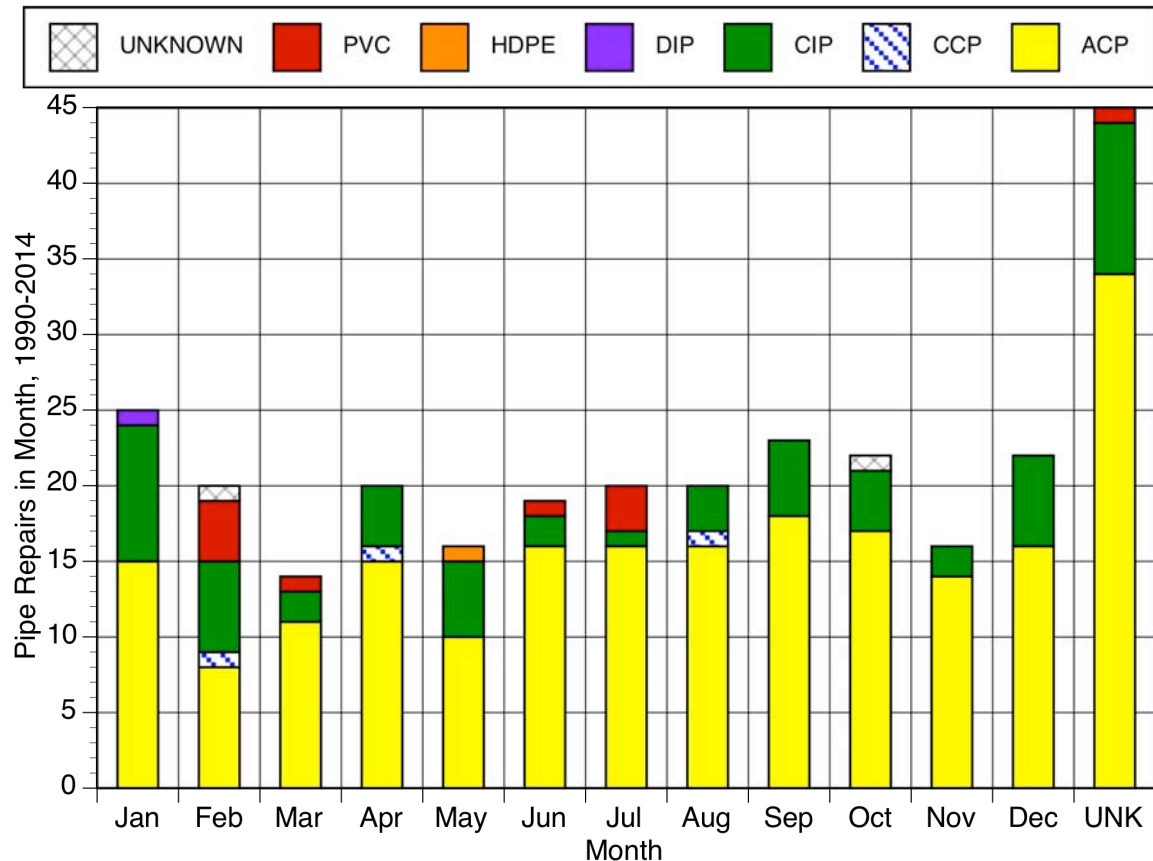


Figure 6-9. Repair History, by Month (1990-2014)

EBMUD has recently done a review of the repairs of their AC pipe, and found that in their system, AC pipe has a significantly higher repair rate during the summer months (especially August) than during the winter months. EBMUD attributes this higher rate due to the shrink-swell cycle of clay-type soils, especially in the hotter parts of their service area (San Ramon, Danville, Walnut Creek), where they think that during the summer, the soils "dry out" and thus shrink. EBMUD contends that their AC pipe failures are largely due to soil movements, and not due to internal or external corrosion (wall thinning).

To see if this trend is also seen in the Palo Alto leak history, Figure 6-10 shows just the monthly count of repairs on AC pipe, for the time period 1990-2014. Like EBMUD, there appears to be some increased rate of AC pipe failures in the hot months (83 leaks in June

through October) versus the rest of the year (89 leaks in seven months), suggesting that the "hot months" have about a 30% increase in the repair rate.

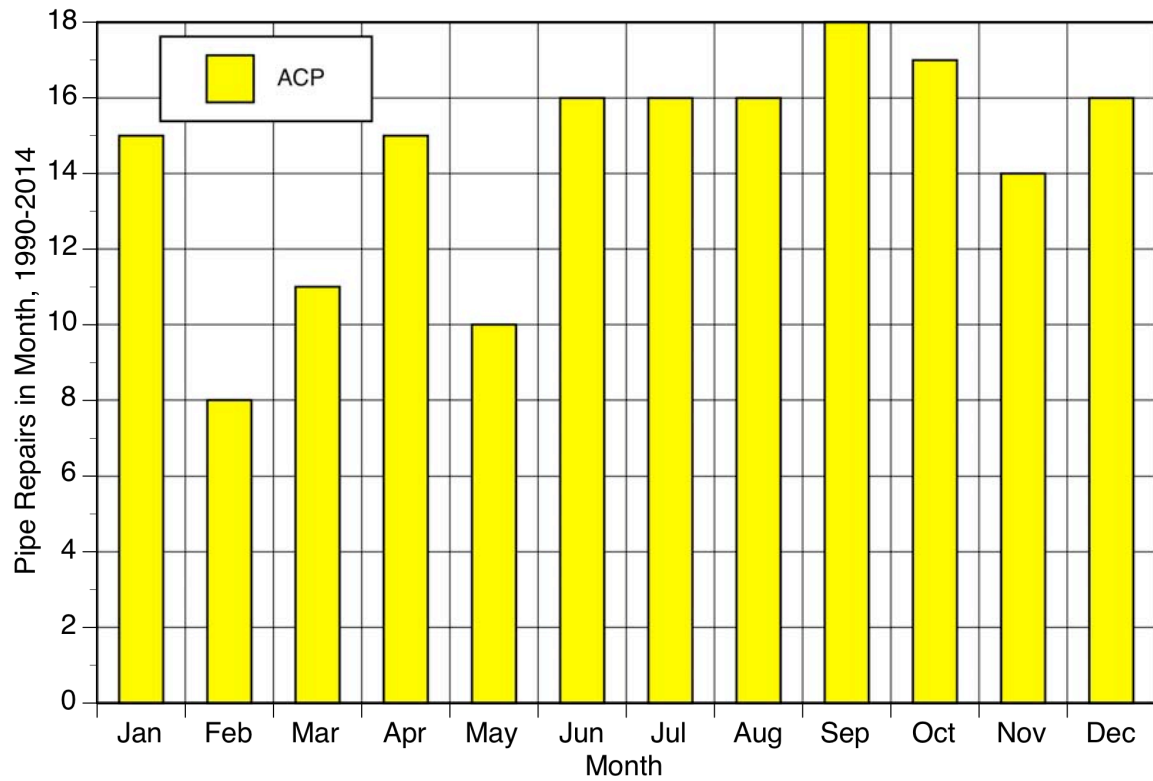


Figure 6-10. ACP Leak History, by Month (1990-2014)

Figure 6-11 shows the historical number of leak repairs for active pipe mains, by year. For the year "2014", the total repairs reflects only a partial year data. The year labeled "Unknown" reflects repairs for which a date of the repair was not available.

While Figure 6-11 shows an ongoing reduction of the number of main repairs per year, it must be recalled that this plot reflects only the "active" pipes in the water system. Palo Alto has been conducting a pipe replacement effort, and leaks on now-abandoned pipe are not included in Figure 6-11. As the pipe replacement program from 1990 to 2014 has concentrated on replacement of pipes with a known a historically high leak rate, the data in Figure 6-11 is somewhat "skewed" towards pipes that have historically had a lower repair rate.

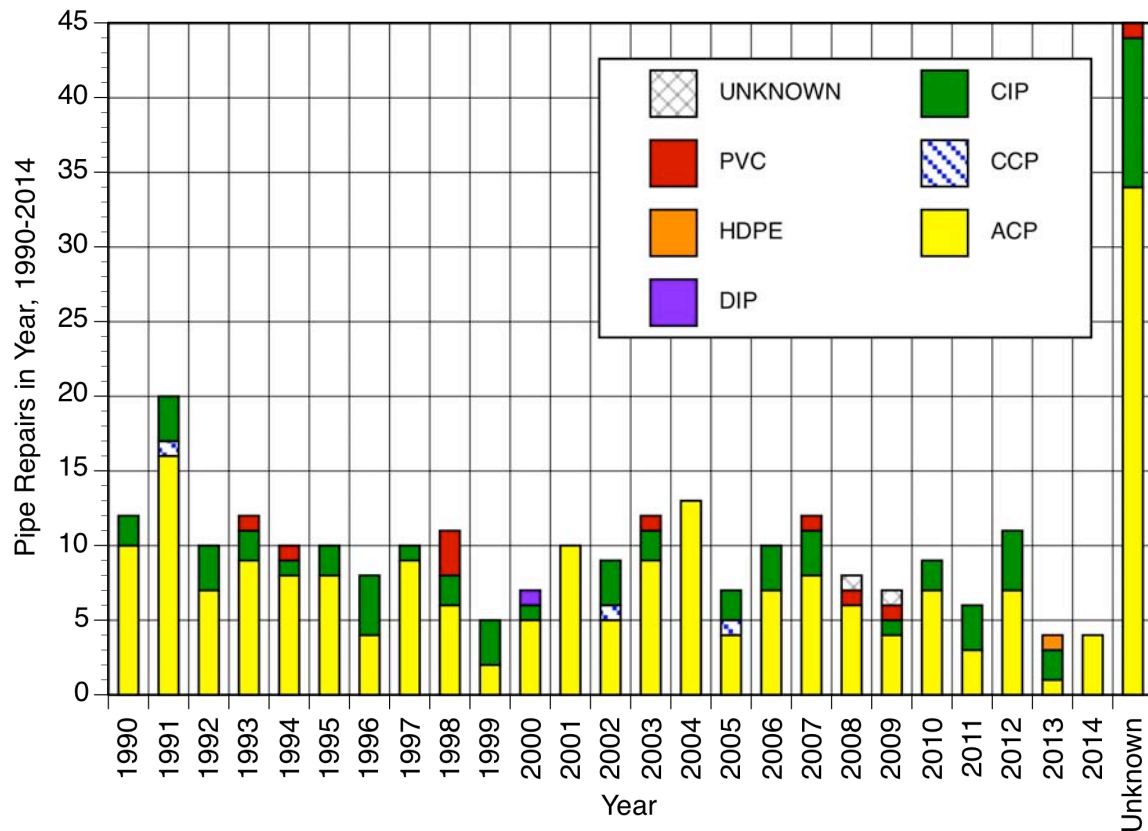


Figure 6-11. Active Pipe Repair History, by Year (1990-2014)

We were curious to see if there is a correlation of pipe repairs versus monthly rainfall. The concept we are considering is that in months with higher rainfall, there might be some trend in pipe repair rate.

Figure 6-12 shows the average monthly rainfall in Palo Alto, (data averaged from 1981 to 2010). Comparing Figure 6-12 (monthly rainfall) with Figures 6-9 and 6-10 (monthly pipe repairs), we see that there is no direct correlation between rainfall and pipe repairs.

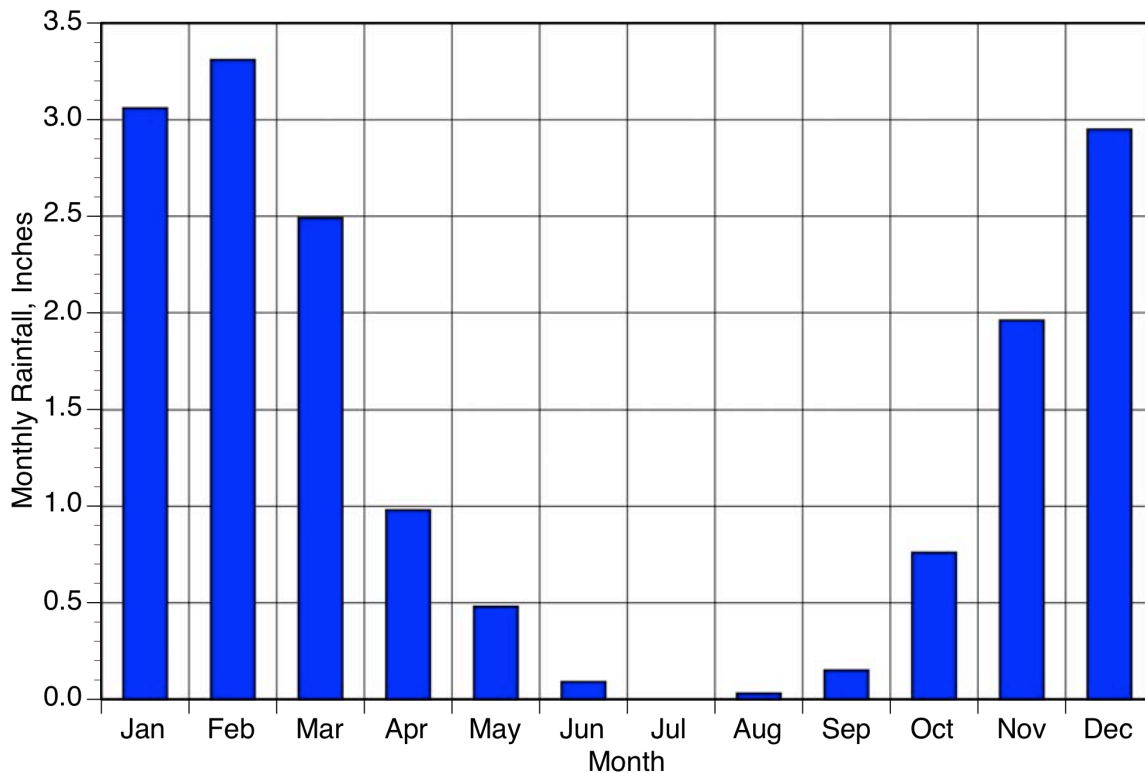


Figure 6-12. Average Monthly Rainfall, Inches (1981 to 2010)

6.5 Pipe Repair Rate as a Function of Soil Corrosivity

Metal pipes (cast iron, ductile iron, steel and copper) and pipes containing metal (concrete cylinder pipe) may be susceptible to corrosion in certain soils. Soil resistivity is often indicative of the corrosivity of the soil. We did soil testing, described in Section 2.4 (test results in Appendix A) and determined the soil resistivity, ρ , for each pipe segment. We then correlated the soil resistivity (ρ) to the historical repair rate for metal-containing pipes. The only pipes with appreciable numbers of repairs were CIP, ACP and PVC. All other material types had very limited repairs. We calculated repair rates for pipes in the 24.58-year time frame with repair records, with soil resistivity rounded to the nearest 1000 ohm-cms. For example, if the average soil resistivity for a pipe segment is 858 ohm-cm, the pipe would be assigned to the 1000 ohm-cm group.

Figures 6-13 through 6-15 show the results for CIP, ACP and PVC pipe types. Reviewing the three plots, only the CIP shows a correlation with soil resistivity. This makes sense, as we would not expect Rho to impact non-metallic pipe like AC and PVC.

Therefore, we recommend using soil resistivity factors only for CIP and Unknown (probably CIP) pipe types; those factors are reported in Table 6-11. For non-metallic pipes, this factor is taken as 1.0; that is, soil resistivity does not affect the overall repair rate.

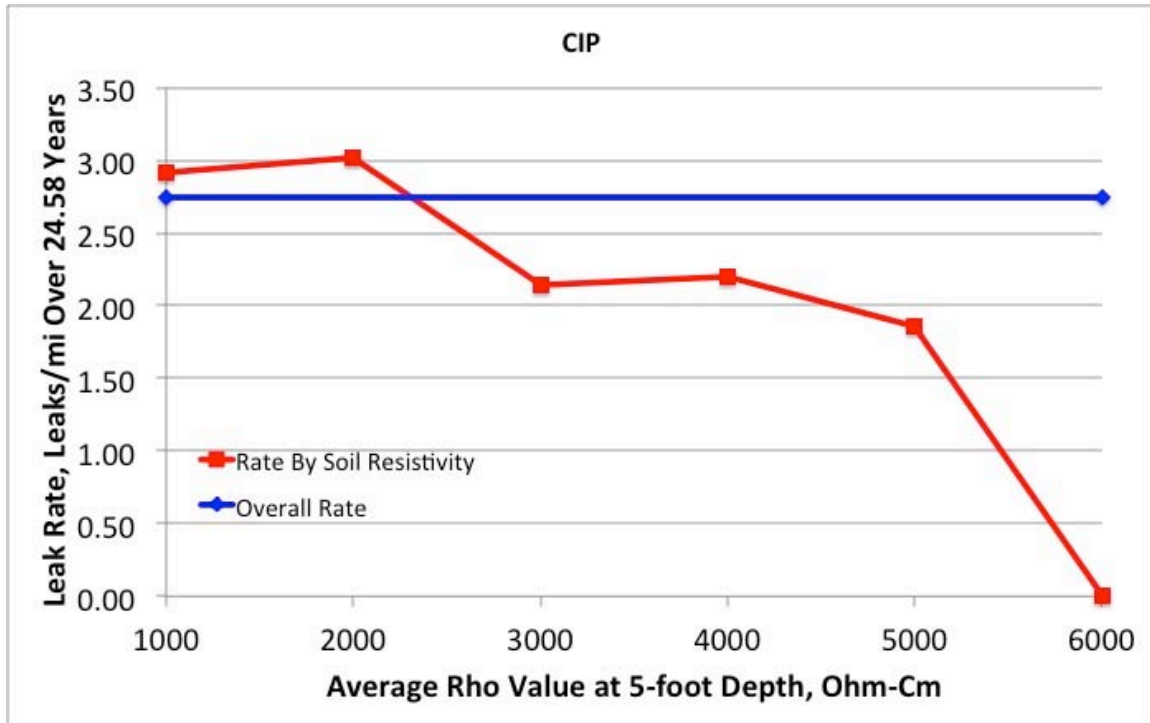


Figure 6-13. Leak Rate vs. Soil Resistivity, Cast Iron Pipe

In Figure 6-14, there would appear to be little impact on AC repair rate for $\text{Rho} \leq 3000$ ohm-cm (most of the flatlands of Palo Alto). There is some AC pipe in the Foothills of Palo Alto, where Rho is usually higher. But, the age of ACP in the Foothills is usually younger than those in the flatlands, and the Age factor for ACP pipe would mask the Rho impacts. (see Table 6-9). Therefore, we do not recommend a Rho modification factor for ACP.

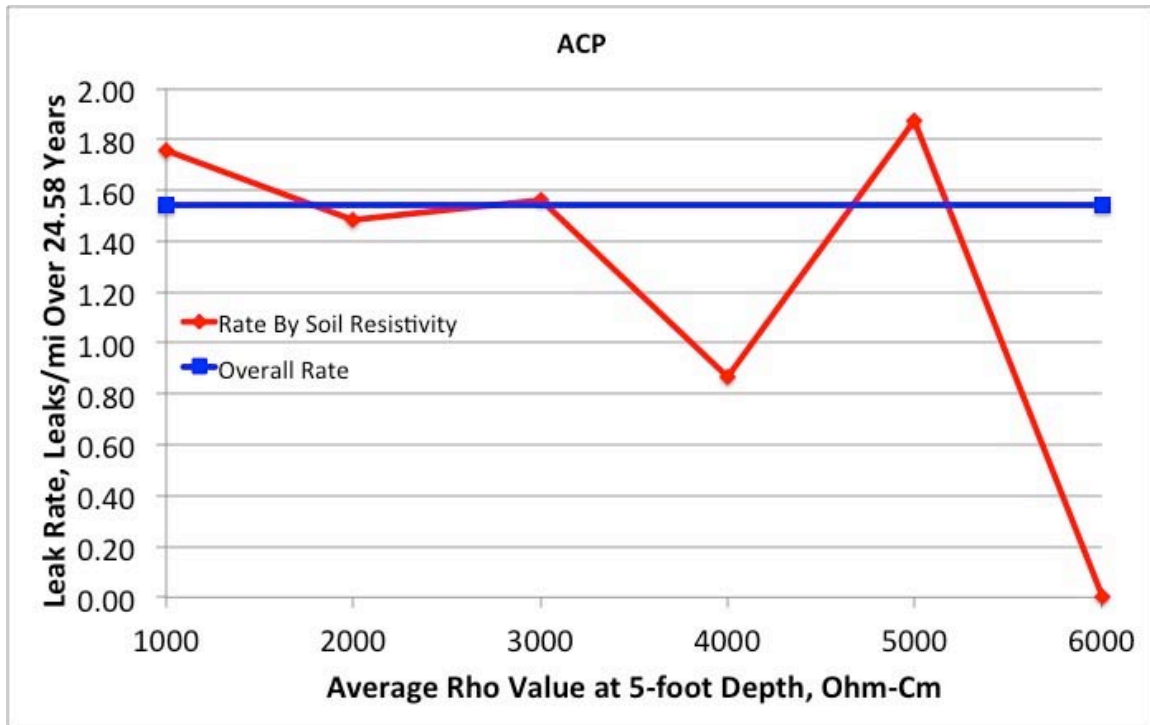


Figure 6-14. Leak Rate vs. Soil Resistivity, Asbestos Cement Pipe

In Figure 6-15, there would appear to be little impact on PVC repair rate for $Rho \leq 4000$ ohm-cm (most of the flatlands of Palo Alto). With the small quantity of PVC repairs overall (9 in 24.58 years), and recognizing that PVC should be inert to corrosion, we do not recommend a Rho factor for PVC pipe.

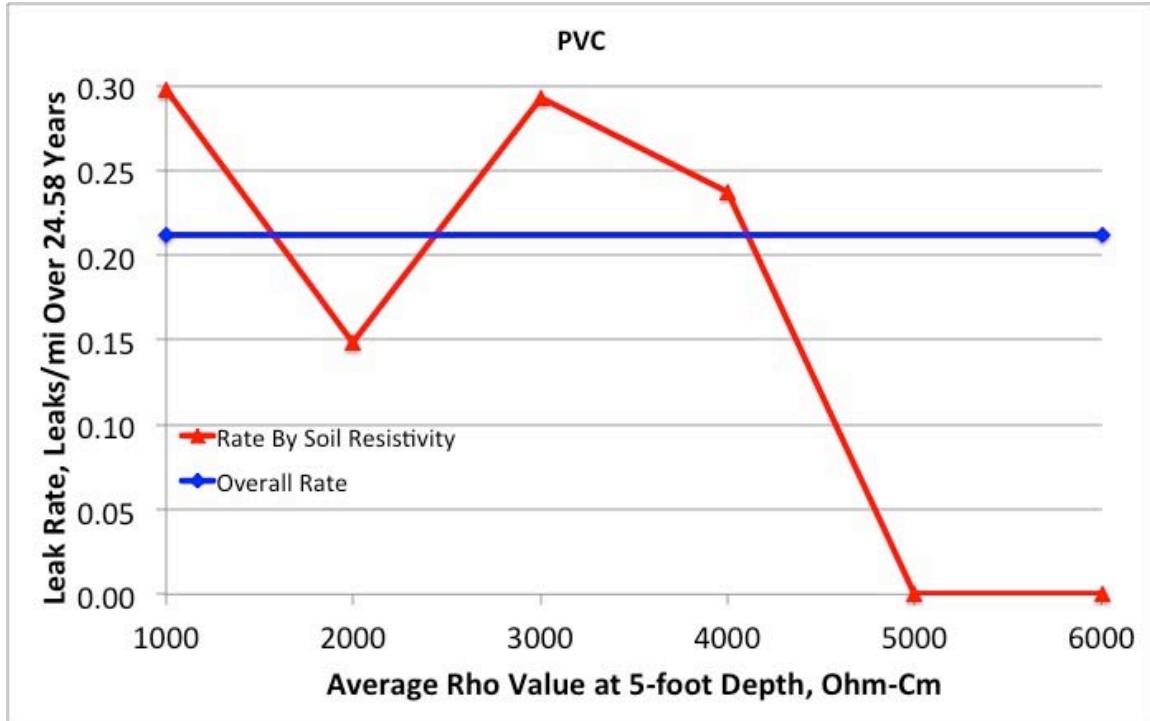


Figure 6-15. Leak Rate vs. Soil Resistivity, PVC Pipe

Soil Resistivity ohm-cm	Rho Factor, CIP, CU, Unknown	Rho Factor, DIP	Rho Factor, CCP	Rho Factor, STEEL	Rho Factor, All Other Pipes
1,000	1.2	1.1	1.1	1.15	1.0
2,000	1.1	1.0	1.0	1.1	1.0
3,000	0.8	1.0	1.0	1.0	1.0
4,000	0.7	1.0	1.0	1.0	1.0
5,000	0.6	0.9	0.9	0.9	1.0
6,000 +	0.5	0.9	0.9	0.9	1.0

Table 6-11. Soil Resistivity Factors for Pipe Repairs

7.0 Pipe Replacement Strategies

Sections 2 through 6 of this report present basic information about seismic vulnerabilities of the water pipes, as well as the repair of pipes on an ongoing basis.

In Section 7 of this report, we examine Palo Alto's water main replacement program, and make recommendations as to how to proceed into the future.

7.1 Water Main Replacement Program

Beginning in about 1990, the Palo Alto water department began a long term effort to replace water pipes, called the Water Main Replacement Program (WMRP). The work was originally planned out in 34 phases, with each phase generally being accomplished in about 1 year.

Table 7-1 shows the length of pipe involved in each of the 34 phases. Above the line, the work has already been completed (Phases 1 to 24), and the listed material reflects the replacement pipe. As can be seen the bulk of the replacement pipe has been PVC (Phases 1 through 2), HDPE (Phases 21 to 24) with some DIP. The length of pipe included in each phase has varied from about 1 mile to about 3 miles.

Sum of LENGTH	Column Labels									
Row Labels	ACP	CCP	CIP	CU	DIP	PE	PVC	Steel	(blank)	Grand Total
0	7684	628248	77844	52423	1224	22497	4127	41706	109	835862
1							5256			5256
2						138	4489			4627
3							5595			5595
4							8565			8565
5							5135			5135
6							7144			7144
7						1915	6720			8635
8		2				141	548	16513		17204
9						826	16516			17342
10		102		1		62	10038			10203
11		417				179	13542			14138
12						96	12537			12633
13		25		7		420	256	10158		10866
14		129				126	12805			13060
15		3733		2		12	7017			10764
16		247	60			5372	9884			15563
17						1174	70	14481		15725
20							181	13826		14007
21				3		68	17305	11		17387
22						125	16167			16292
23						99	13706			13805
24							1242			1242
25		1767		10143			50	37		11997
26	38	7613		21010		236	9	140		29046
27		2968		11720				66		14754
28		8924		3490				109		12523
29		12098		3569		335		1081		17083
30		11455						21		11476
31		9140		4907	175		158	76		14456
32	366	7308		727				711	788	9900
33	361	8731		1641				53		10786
34	3	9485	180	3925				15		13608
(blank)										
Grand Total	8452	712392	78084	113568	1399	33821	53819	224247	897	1226679

Table 7-1. Length of Pipe (in feet) for each Phase of the WMRP – by Pipe Material

Row "0" in Table 7-1 are lengths of pipe not scheduled to be replaced in any of the 34 phases.

Table 7-2 shows the similar data, except sorted by pipe diameter.

Row	0.5	0.75	1	1.25	1.5	2	3	4	6	8	10	12	14	16	18	20	24	27	30	Grand Total	
0	21	20	284	388	570	2833	2	36417	290336	234403	36632	86435	36234	46542	26177	6158	8003	10487	9199	4721	835862
1									4312			944									5256
2									4489	138											4627
3									5590			5									5595
4									7018	1547											8565
5									1433	59	3639			4							5135
6									6633	511											7144
7								20	4206	1758	2649	2									8635
8									15146	2058											17204
9									13425	3917											17342
10								1	9183	21	7	991									10203
11									13709	421		8									14138
12									12629	4											12633
13									5731	5135											10866
14									11620	1301	15	124									13060
15								14	3534	5410	1793	13									10764
16								300	8503	1399	90	5243		19	9						15563
17								11	9703	2251	2554	1206									15725
20								13	9723	4264		7									14007
21									188	14478	621	2085	1	14							17387
22									120	16101		71									16292
23									12	8024	1128	73		4568							13805
24										1242											1242
25								1960	9383	62		592									11997
26								321	25925	2670				130							29046
27						3		204	9684	1804	2437	622									14754
28								772	10657	56		1038									12523
29								666	4450	11242	725										17083
30								886	6372	4218											11476
31			175						5768	2590	3253	2670									14456
32						366			3892	2548		3094									9900
33									6651		1407	2171	557								10786
34								1495	7170	2834	219	1890									13608
(blank)																					
Grand Total	21	20	284	563	570	3202	2	43080	527195	332466	57169	109284	36792	51277	26186	6158	8003	10487	9199	4721	1226679

Table 7-2. Length of Pipe (in feet) for each Phase of the WMRP – By Diameter

Through 2014, 24 phases of this work have been completed. Figure 7-1 shows the locations of the water mains that have already been replaced, indicated by the heavy colored lines. The thin lines reflect all other pipe.

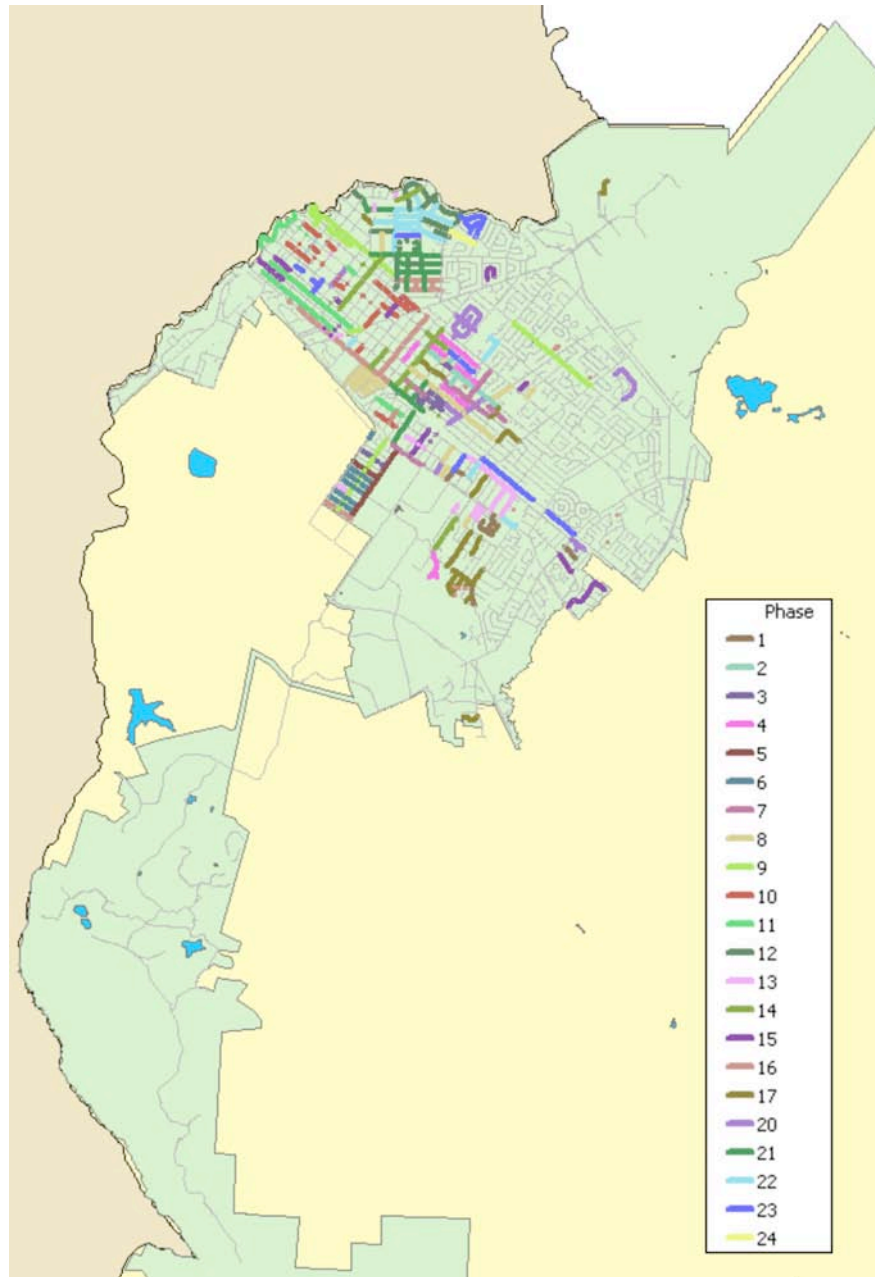


Figure 7-1. Phases 1 to 24 of the Water Main Replacement Program (WMRP)

Figure 7-1 shows that much of the replaced pipe is located in the original (pre-1900) urbanized areas (see Figure 2-5). Figure 7-2 shows the planned future phases 25 through 34.

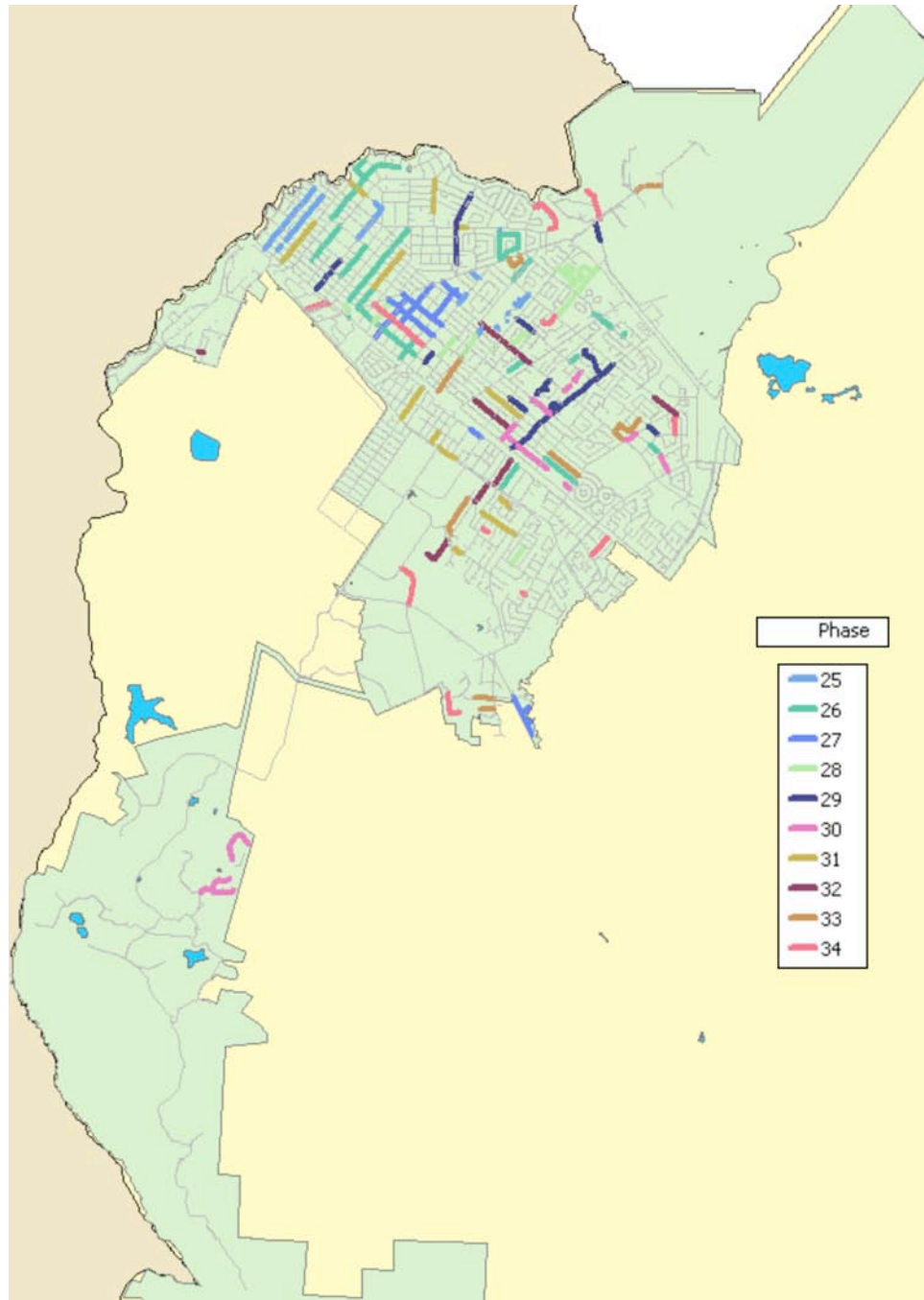


Figure 7-2. Phases 25 to 34 (Planned) of the Water Main Replacement Program (WMRP)

Figure 7-3 shows the pipe not affected by any of the 34 phases.

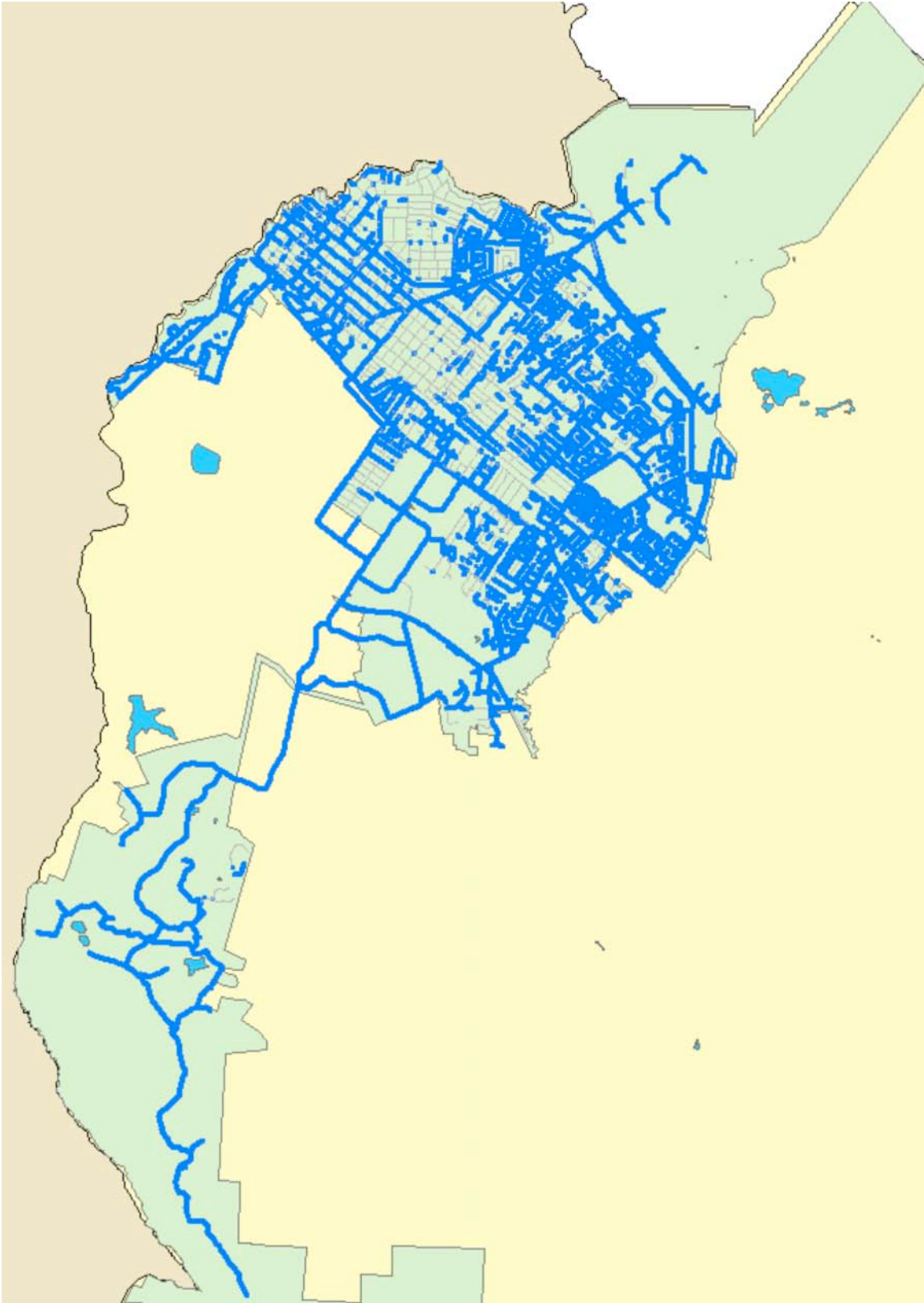


Figure 7-3. Pipe not in the Water Main Replacement Program – Heavy Blue Lines

7.2 Benefit Cost Ratio Model

In this report, we use a Benefit Cost Ratio model to sort out which pipes are most cost effective to be replaced. In Section 7, we present many tables with cost estimates, many of which were developed in computer programs. While the costs are shown to all digits per the calculations, it is recognized that the calculations are at best accurate to about 2 significant digits. We retain all the digits in Section 7 so that the reader can track the computations; for summaries in Section 1 of this report, the numbers are rounded.

We consider two reasons for pipe replacement:

- The pipe has had a high historic rate of leak, with each leak requiring a repair. The cost of the day-to-day repairs, as well as the economic impacts to customers while the repair is being made, influences whether it is cost effective to replace the pipe.
- The pipe has a high chance of being damaged in future earthquakes. The cost of the post-earthquake repairs, as well as the economic impacts to customers while the repair is being made, coupled with an increased chance of fire spread, influences whether it is cost effective to replace the pipe.

The basic computation for a Benefit Cost Ratio (BCR) is to sum up the expected future benefits (= reduction in future repair and economic costs should the pipe be replaced) divided by the current replacement costs.

$$BCR = \frac{\sum_{i=1}^{n \text{ years}} \text{ReducedRepairCostPerYear} / (1+r)^i}{\text{ReplacementCost}} \quad [\text{Eqn 1}]$$

where r = discount rate and n = number of years. A good Asset Management program should use this type of model to include both aging and seismic issues by summing up the BCRs for each pipe:

$$BCR_{\text{Total}} = BCR_{\text{Aging}} + BCR_{\text{Seismic}}$$

A Benefit Cost Ratio greater than 1 indicates that the pipe replacement is cost effective.

There are two other main reasons to replace an existing pipe:

- The pipe diameter, coupled with its inside surface roughness, results in excessive pressure losses and lack of ability to deliver the required fire flows plus customer demand at acceptable pressures. The inside surface roughness is often quantified by a Hazen Williams C coefficient, with a C value of 130 reflecting new cement-lined or smooth inside pipe wall. Unlined pipes, particularly Cast Iron pipe,

develop tuberculation on the inside walls over time, and that tuberculation results in a loss of inside diameter, coupled with a great increase in surface roughness. The only type of pipe in the Palo Alto system subject to this is unlined Cast Iron pipe. Experience with other water systems with calibrated hydraulic models shows that common C values for a 100-year old CIP are about 40 (4-inch pipe) or 60 (6 inch pipe). While these low C values might still allow adequate flow and pressure at low flow rates (commonly 50 gpm to 100 gpm, for day-to-day customer demands), under sudden high demand for water (say for fire flows of targeted at 1,000 gpm or more), the great drop in pressure at higher flow rates will not allow the pipe to deliver the desired fire flows. Examining Figure 2-7 (Cast Iron pipe in the system in 2010), and Table 2-6 (113,568 feet of cast iron pipe as of 2010), there remain more than 20 miles of CIP in the system. Very little of this pipe is 4-inch diameter. The project work scope did not include hydraulic models to quantify flows and pressures to confirm the need to replace some of the remaining CIP for hydraulic flow reasons, but it is reasonable to assume that about 20% of the remaining CIP pipe have some fire flow limitations, and therefore would be good candidates for replacement.

- There are street re-routes / construction of new highways, construction of higher density facilities requiring more water, or construction of new developments that require re-routed pipes or larger pipes new pipes for new developments to meet the change of use. The cost of these replacement pipes is often born by the developing / agency requesting the re-route or needing the water.

7.3 Pipe Replacement for Aging Issues

7.3.1 Benefit Cost Analysis

We used the pipe repair rate data discussed in Section 6 to determine repair rates and benefit cost ratios for each of the 7,315 pipe segments in the current transmission pipe inventory. For each pipe, we used the following algorithm to determine the repair rate for each pipe:

- If there were recent pipe repairs within the last 7 years, X_7 , on the segment, the repair rate is X_7 repairs / 7 years
- If there were X_{total} repairs on the segment within the 24.58 year history available in the Palo Alto system, the repair rate is X_{total} repairs / 24.58 years
- If there were no repairs on the segment, the repair rate is based on the overall rate for the specific type of pipe, calculated as follows:

$$\text{Repairs/year} = k1 * k2 * k3 * \text{segment length}$$

Where

k_1 (repairs/mi/year) = background repair rate * diameter factor (Tables 6-5, 6-6, 6-8)

k_2 = Age factor (Tables 6-9, 6-10)

k_3 = Soil resistivity factor (Table 6-11)

The number of pipe repairs per year for the pipe segment is the largest of the three above calculations. The highest repair rates are for those segments with recent repairs or those with recurring repairs.

We also computed the repair rate for the assumed replacement pipe. For all pipes located in soils with either H (high) or VH (very high) mapped liquefaction susceptibility, we assumed that the replacement pipe was HDPE with fusion butt welded joints; other types of pipe, like Kubota "chained" seismic ductile iron pipe, or competing products from US Pipe or others, can also be used, as long as the pipe can be shown to have very high reliability assuming a ground strain of 1%. For all other pipes, we assume the replacement pipe was Ductile Iron pipe with push-on type joints (no special seismic design); other types of pipe can also be used, such as PVC (C900) with push on joints. This report is not meant to be exhaustive with regards to pipeline design, and ALA (2005) gives more detailed recommendations for new pipes that are designed for earthquakes.

The parameters of the BCR model are listed below:

Outage Length. This is the length (in feet) of the pipe that will be subject to an outage while the repair is being made. This will generally be the distance between the two nearest isolation valves on the pipe to be replaced. We assume the following average lengths: 500 feet, 750, 1,000, 2,000 or 3,000 feet for the following pipe diameter ranges: $\leq 4"$, 6-10", 12-14", 16-20", $>20"$, respectively.

Outage Duration per Repair. This is the number of hours that customers will lose water service, given a leak. We assume on average 8 hours per repair for pipes $<16"$ in diameter and 16 hours per repair for pipes $16"$ in diameter or larger.

Repair Cost per Leak. This is the direct cost to make the repair for the leak. The assumed average costs are \$5,000, \$10,000, \$12,000, or \$18,000 to repair $<10"$, $<16"$, $<20"$ or $20"+$ diameter pipes.

Claim Cost per Leak. This is the direct cost to the City due to claims due to leaking water. Experience shows that about 10% to 15% of all leaks result in claims. The common claim cost might be about \$1,250 for a 6-inch pipe break, but could be over \$10,000 or more for larger diameter pipe breaks or for unusually circumstances. The model assumes the average claim cost is 10% of the repair cost.

Retail Value of Water. This is the average value of water sold to Palo Alto's retail customers. The assumed rate is \$0.004 per gallon, and assuming an average water velocity, described below.

Water Velocity. This is the 24-hour average velocity (in feet / second) of water in the pipe to be replaced, under average day demand conditions (no fire flows, no peak summer flow). The default value is 0.56 feet per second . This value is used to estimate the volume of water not-sold should there be a outage to make a repair.

Discount Rate. This is the discount rate to be used in the analyses. For FEMA analyses, current law requires 7 percent. For Palo Alto's internal decision-making purposes, the analysis uses 4% and a 60-year economic horizon (this is not to say that the pipe will only last 60 years).

Replacement Cost. We assume that the average replacement cost for new pipe is \$35 per inch-foot (pipes up to 8" diameter) and \$30 per inch-foot (pipes 10" diameter and larger). This cost reflects all design, installation and inspection costs for a common pipe in urban streets, usually constructed using cut-and-cover methods. This translates to about \$1,500,000 to build one mile of 8-inch pipe, or \$1,900,000 to build one mile of 12-inch pipe. Actual costs will vary and will be dependent on many street-specific factors, as well as the total length of pipe included in the project. However, we feel these costs are indicative of common costs for the Palo Alto area.

Economic Activity. We used Department of Commerce data, the year 2010 gross domestic product for customers served by the Palo Alto water system is about \$9.44 billion (see Section 2.6 for derivation). This data is developed using California county de-aggregation, with San Mateo County being one of the highest GDP per-capita counties in California.

The total cost of a repair is the sum of the following values:

- Actual repair
- Claim cost of repair (10% of repair cost)
- Loss of GDP during the repair outage (prorate the GDP contribution of any pipe segment by its length, versus the total length of pipe in Palo Alto, then say 70% of that GDP portion is lost during that period. This assumes that some commerce can continue, even with disruption of water service.)
- Sales loss of water during the repair outage

The above items are added up, then the yearly cost is calculated by multiplying the repair rate (repairs per year) by the total repair cost. Then, the present value of the yearly cost is calculated, using a discount rate of 4% over 60 years. The multiplier for the yearly cost to

achieve that discount rate and term is 22.62. The above computation is done for the existing pipe, and again with the assumed replacement pipe. The "benefits" of the upgrade are the reduction in the expected leak rate for the replacement pipe.

Main ID	PIPE GID	Pipe Age (Yrs)	Length (feet)	MATERIAL	Diam (inch)	Liq	Zone	Replace Cost (\$)	BCR Repair	WMRP Phase No
237	250071	61	42	ACP	6	M	2 - Two	8,820	1.37	WMR 31
572	251073	0	6	UNKNOWN		VH	1 - One	5,000	6.87	
865	251919	0	32	ACP	6	M	1 - One	6,720	1.45	
1025	252352	21	22	PVC	6	M	1 - One	5,000	1.95	WMR 8
1205	252885	69	62	ACP	6	M	1 - One	13,020	2.64	WMR 29
1507	253733	0	43	ACP	8	M	1 - One	12,040	1.12	
1679	254256	0	37	ACP	6	M	1 - One	7,770	4.42	
1753	254504	57	315	ACP	12	VH	2 - Two	113,400	2.96	WMR 31
1819	254692	0	27	ACP	12	VL	2 - Two	9,720	4.92	WMR 34
1987	255186	0	63	CIP	8	M	2 - Two	17,640	1.00	
2354	256268	0	62	ACP	6	M	1 - One	13,020	5.27	WMR 32
2398	256433	0	24	CIP	6	M	1 - One	5,040	1.94	WMR 29
2455	256588	76	35	CIP	6	M	1 - One	7,350	4.67	
2490	256688	0	37	ACP	6	M	1 - One	7,770	4.42	
2496	256703	0	12	ACP	6	M	1 - One	5,000	6.87	
2758	257478	0	42	ACP	12	VL	2 - Two	15,120	3.16	
3109	258057	67	33	CIP	6	M	1 - One	6,930	1.41	
3274	258275	0	17	DIP	12	M	1 - One	6,120	5.77	
3346	258366	0	110	UNKNOWN		M	1 - One	23,100	1.49	
3461	258511	0	40	CIP	6	M	1 - One	8,400	4.09	
3581	258654	0	55	CIP	10	M	1 - One	16,500	1.41	
3766	258910	0	30	ACP	6	VH	2 - Two	6,300	6.76	
3770	258914	0	48	ACP	10	M	3 - Three	14,400	2.07	
4238	259555	0	248	CIP	10	M	1 - One	74,400	1.10	WMR 33
4639	260118	0	169	CIP	10	M	1 - One	50,700	1.61	WMR 27
5234	261957	76	280	CIP	6	M	1 - One	58,800	1.17	WMR 25
5330	262183	0	32	CIP	6	M	1 - One	6,720	1.45	WMR 26
5661	262990	57	122	ACP	12	M	2 - Two	43,920	3.82	
5669	263009	0	121	CIP	6	M	1 - One	25,410	1.35	WMR 26
5698	263081	0	36	CIP	8	M	1 - One	10,080	1.33	
5837	263394	0	130	ACP	8	VH	1 - One	36,400	1.30	
6112	264027	59	5	ACP	12	M	1 - One	5,000	7.06	
6183	264261	77	49	CIP	6	M	1 - One	10,290	3.34	WMR 26
6453	264871	0	166	CIP	8	M	1 - One	46,480	1.02	WMR 29
6478	264925	0	10	CIP	6	M	1 - One	5,000	1.96	WMR 26
6819	265729	58	146	ACP	4	M	1 - One	20,440	1.11	WMR 30
6963	266026	69	56	ACP	6	M	2 - Two	11,760	1.03	
6998	266093	55	170	ACP	6	M	2 - Two	35,700	1.19	

Main ID	PIPE GID	Pipe Age (Yrs)	Length (feet)	MATERIAL	Diam (inch)	Liq	Zone	Replace Cost (\$)	BCR Repair	WMRP Phase No
7093	266293	0	394	ACP	12	M	1 - One	141,840	1.75	
7207	266552	0	606	ACP	10	VH	1 - One	181,800	1.35	WMR 29
		Total	3934					1,098,920		

Table 7-3. Pipe Segments with Benefit Cost Ratios > 1.0 (Aging only)

The replacement cost is calculated, and the ratio of the net present value of the repairs to the replacement cost gives the BCR. The complete calculations are in SERA file SERA.14.21.26.out. 40 segments of the 7,315 main pipes have BCRs in excess of 1.0, totaling 3,934 feet of pipe, with replacement cost of \$1,098,920. They are listed in Table 7-3. The actual length of pipe to be replaced will be higher, considering valve-to-valve length as a reasonable replacement length, or taking into account actual street conditions. By extending each replacement length until a BCR of 1 is reached for that segment, the total replacement length would be 9,406 feet.

Table 7-4 extends the results from Table 7-3, but using different BCR ratios as a cut off. Table 7-4 presents the results for pipe leaks only (BCR leak), seismic only (BCR seismic) and the combined effects of leaks and seismic issues (BCR total).

BCR Criteria	Num Pipes	Length (Feet)	Length (Miles)	Replacement Cost for Length
BCR leak > 1.5	22	1,565	0.30	\$499,180
BCR leak > 1	40	3,934	0.75	\$1,098,920
BCR leak > 0.6	54	6,762	1.28	\$1,743,850
BCR seismic > 1.5	349	95,930	18.17	\$24,505,553
BCR seismic > 1	432	111,368	21.09	\$30,494,724
BCR seismic > 0.6	472	120,725	22.86	\$36,191,045
BCR total > 1.5	374	97,654	18.50	\$25,039,400
BCR total > 1	475	114,291	21.65	\$31,289,500
BCR total > 0.6	533	125,502	23.77	\$37,624,230

Table 7-4. Pipe Lengths and Costs with Various Benefit Cost Ratios

The bottom line of the BCR evaluation for aging is that the City of Palo Alto's current plan to replace an average of 2 to 3 miles of pipe per year for aging for the next decade or so may be "too much", given the generally low rates of repairs seen since records have been kept (1990 onward). Additionally, as the Water Main Replacement Program has been ongoing since 1987, it is thought that much of the worst performing pipe has already been replaced.

7.3.2 Refined Pipe Replacement Strategy for Aging

At a meeting with Palo Alto on June 15, 2015, it was discussed that Phases 25 and 26 of the WMRP have already been, or are in the process of being implemented (including survey, design, etc.). Therefore, five pipe segments in Table 7-3 are already in the

process of being replaced (or are in the process of being replaced). Table 7-5 reflects these updates.

Main ID	PIPE GID	Pipe Age (Yrs)	Length (feet)	MATERIAL	Diam (inch)	Liq	Zone	Replace Cost (\$)	BCR Repair	WMRP Phase No
237	250071	61	42	ACP	6	M	2 - Two	8,820	1.37	31
572	251073	0	6	UNKNOWN		VH	1 - One	5,000	6.87	
865	251919	0	32	ACP	6	M	1 - One	6,720	1.45	
1025	252352	21	22	PVC	6	M	1 - One	5,000	1.95	8
1205	252885	69	62	ACP	6	M	1 - One	13,020	2.64	29
1507	253733	0	43	ACP	8	M	1 - One	12,040	1.12	
1679	254256	0	37	ACP	6	M	1 - One	7,770	4.42	
1753	254504	57	315	ACP	12	VH	2 - Two	113,400	2.96	31
1819	254692	0	27	ACP	12	VL	2 - Two	9,720	4.92	34
1987	255186	0	63	CIP	8	M	2 - Two	17,640	1.00	
2354	256268	0	62	ACP	6	M	1 - One	13,020	5.27	32
2398	256433	0	24	CIP	6	M	1 - One	5,040	1.94	29
2455	256588	76	35	CIP	6	M	1 - One	7,350	4.67	
2490	256688	0	37	ACP	6	M	1 - One	7,770	4.42	
2496	256703	0	12	ACP	6	M	1 - One	5,000	6.87	
2758	257478	0	42	ACP	12	VL	2 - Two	15,120	3.16	
3109	258057	67	33	CIP	6	M	1 - One	6,930	1.41	
3274	258275	0	17	DIP	12	M	1 - One	6,120	5.77	
3346	258366	0	110	UNKNOWN		M	1 - One	23,100	1.49	
3461	258511	0	40	CIP	6	M	1 - One	8,400	4.09	
3581	258654	0	55	CIP	10	M	1 - One	16,500	1.41	
3766	258910	0	30	ACP	6	VH	2 - Two	6,300	6.76	
3770	258914	0	48	ACP	10	M	3 - Three	14,400	2.07	
4238	259555	0	248	CIP	10	M	1 - One	74,400	1.10	33
4639	260118	0	169	CIP	10	M	1 - One	50,700	1.61	27
5661	262990	57	122	ACP	12	M	2 - Two	43,920	3.82	
5698	263081	0	36	CIP	8	M	1 - One	10,080	1.33	
5837	263394	0	130	ACP	8	VH	1 - One	36,400	1.30	
6112	264027	59	5	ACP	12	M	1 - One	5,000	7.06	
6453	264871	0	166	CIP	8	M	1 - One	46,480	1.02	29
6819	265729	58	146	ACP	4	M	1 - One	20,440	1.11	30
6963	266026	69	56	ACP	6	M	2 - Two	11,760	1.03	
6998	266093	55	170	ACP	6	M	2 - Two	35,700	1.19	
7093	266293	0	394	ACP	12	M	1 - One	141,840	1.75	
7207	266552	0	606	ACP	10	VH	1 - One	181,800	1.35	29
		Total	3442					992,700		

Table 7-5. List of Pipe Segments with Benefit Cost Ratios > 1.0 (Aging only)

Each of the segments in Table 7-5 was reviewed to determine practical pipe lengths to be included in the replacement project. To do this, each of the segments in Table 7-5 was studied in the GIS map. Surrounding pipe segments were reviewed, up to the next inline valve or street intersection in each direction. The adjacent segments were added to the replacement project if they exhibited strong cause for replacement, such as having relatively high BCRs (say above 0.5), have experienced historical repairs, are in a future WMRP phase, or their addition would make sense from an operability point of view. Table 7-6 lists the pipe segments for the extended population.

Main ID	PIPE GID	Pipe Age (Yrs)	Length (feet)	MATERIAL	Diam (inch)	Liq	Zone	Replace Cost (\$)	BCR Repair	WMRP Phase No
881	251961	0	36	ACP	6	M	1 - One	7,560	0.02	33
2016	255272	0	39	CIP	6	M	1 - One	8,190	0.02	28
2595	256981	52	1509	ACP	6	M	1 - One	316,890	0.03	33
2650	257140	0	859	ACP	12	VL	2 - Two	309,240	0.03	34
2959	257851	0	494	ACP	12	M	1 - One	177,840	0.02	
3347	258367	0	8	UNKNOWN		M	1 - One	5,000	0.01	
3769	258913	58	27	ACP	10	M	3 - Three	8,100	0.03	
3817	258974	0	564	ACP	6	M	1 - One	118,440	0.02	
3929	259120	56	598	ACP	12	M	1 - One	215,280	0.02	
3930	259121	13	8	PVC	12	M	1 - One	5,000	0	15
4085	259344	56	185	ACP	12	M	1 - One	66,600	0.53	
4086	259345	56	15	ACP	12	M	1 - One	5,400	0.02	
4303	259640	0	434	CIP	10	M	1 - One	130,200	0.04	27
4542	259973	0	4	CIP	8	M	1 - One	5,000	0.01	
5339	262255	69	458	ACP	6	M	2 - Two	96,180	0.02	
5688	263062	0	400	CIP	8	M	1 - One	112,000	0.02	29
5691	263066	0	8	CIP	8	M	1 - One	5,000	0.01	
5692	263067	0	3	CIP	8	M	1 - One	5,000	0	
5699	263082	8	5	PVC	8	M	1 - One	5,000	0	
5700	263083	0	222	CIP	8	M	1 - One	62,160	0.02	
5701	263092	67	322	CIP	6	M	1 - One	67,620	0.03	
5836	263393	0	284	ACP	8	VH	1 - One	79,520	0.02	
5838	263395	3	5	PVC	8	VH	1 - One	5,000	0	
6454	264872	0	211	CIP	8	M	1 - One	59,080	0.23	29
6455	264873	3	9	PVC	8	M	1 - One	5,000	0	
6820	265730	58	153	ACP	6	M	1 - One	32,130	0.02	30
6821	265731	8	9	PVC	4	M	1 - One	5,000	0	30
6822	265732	58	5	ACP	4	M	1 - One	5,000	0	30
		Total	6874					1,922,430		

Table 7-6. List of Extended Pipe Segments For Aging Replacement

The combined list of pipe segments in Tables 7-5 and 7-6 would form the basis of the actual pipe replacement effort. The total cost of the pipes in Table 7-5 is \$992,700 and

the total cost of the pipes in Table 7-6 is \$1,922,430. The total cost of the combined replacement effort (10,316 feet, 1.95 miles) is estimated at \$2,915,130.

During final design, the project engineer might further refined this list to reflect pipe- or location-specific conditions. For example, depending on the type of pipe weakness (say the historical weakness causing damage was due to a poorly installed / weakened valve), then the final design might replace a segment of pipe to either side of the valve, but not the entire adjacent segment to the next street / valve. It should be noted that the locations of historical repairs were matched to pipe segments using best judgment; during actual final design, should additional information be found that would suggest that the original pipes have not had historical damage, then that pipe would have a lower priority for early replacement.

7.3.3 Maps Showing Pipe Replacement due to Aging Issues

Given the results in Tables 7-5 and 7-6, Section 7.3.3 provides a series of maps that show the physical locations of the pipes recommended for replacement due to aging issues.

Figures 7-4 through 7-30 provide the 27 maps. In these maps:

- The blue segments are those pipes listed in Table 7-5
- The violet segments are those pipes listed in Table 7-6
- The red, orange segments are pipes recommended for replacement as part of the seismic program, described in Section 7.4.
- The background color (red, yellow, cyan) indicate areas mapped as having very high (red), moderate (yellow) or low (cyan) liquefaction susceptibility.

Table 7-7 provides a list of the segments for each project and the street location – this is said to constitute the AIP or Aging Improvement Program. It should be noted that Project 22 is included in Seismic Improvement Plan 2 (SIP-2) and should not be double-counted if both the AIP and SIP-2 are selected.

Project No.	Main IDs	Pipe GIDs	Material	Diam (in)	Total Length (Ft)	Street Location / Notes	Figure No.
1	237	250071	ACP	6	42	La Donna @ Barron	7-4
2	572	251073	UNKNOWN		6	Valve on Faber	7-5
3	865	251919	ACP	6	32	Valve on Colonial @ Moreno	7-6
4	1025	252352	ACP	6	22	Bryant @ Oregon	7-7
5	1205	252885	ACP	6	62	Valve on Ross @ Moreno	7-8
6	1507	253733	ACP	8	43	Wilton @ El Camino Real	7-9
7	1679 3817	254256 258974	ACP	6	601	Louis between Loma Verde and Ames	7-10
8	1753	254504	ACP	12	315	Valve on Laguna	7-11
9	1819	254692	CIP	12	928	Intersection of Hillview/Porter/Hanover	7-12
10	1987	255186	ACP	8	63	El Camino Real @ Acacia	7-13

Project No.	Main IDs	Pipe GIDs	Material	Diam (in)	Total Length (Ft)	Street Location / Notes	Figure No.
11	2354	256268	CIP	6	62	Ramona @ El Dorado	7-14
12	2398	256433	CIP	6	24	Homer @ Ramona	7-15
13	2455	256588	ACP	6	35	Guinda @ Embarcadero	7-16
14	2490	256688	ACP	6	37	Moreno @ Greer	7-6
15	2496 881 2595	256703 251961 256981	ACP	6	1,557	Christine	7-17
16	2758	257478	CIP	12	42	Hillview, mapped @ Porter/Hanover, but may actually be S of Foothill – needs verification	7-12
17	3109 5701	258057 263092	DIP CIP	6	355	Park btwn Leland and Stanford	7-18
18	3274	258275	UNKNOWN	12	17	Alma @ Homer	7-19
19	3346 3347	258366 258367	CIP	8	118	Valve on Embarcadero South of Alma	7-20
20	3461	258511	CIP	6	40	Embarcadero South of Alma	7-20
21	3581 4639 2016 4303	258654 260118 255272 259640	ACP CIP CIP CIP	10 10 6 10	697	Waverley between Embarcadero and Churchill	7-21
22	3766	258910	ACP	6	30	Matadero @ Tippawingo – also included in SIP-2	7-22
23	3770 3769	258914 258913	CIP ACP	10	75	Pasteur	7-23
24	4238	259555	CIP	10	248	California btwn Bryant and South	7-24
25	5661	262990	CIP	12	122	Matadero btwn Chimalus and Laguna	7-11
26	5698 6453 6455 6454 4542 5688 5700	263081 264871 264873 264872 259973 263062 263083	ACP ACP PVC CIP CIP CIP CIP	8	1,048	Newell btwn Harker and Embarcadero	7-16
27	5837 5836	263394 263393	ACP	8	414	Embarcadero Near the Bay	7-25
28	6112 3929 3930 4085 4086	264027 259120 259121 259344 259345	CIP ACP PVC ACP ACP	12	884	Park btwn Grant and Sheridan	7-26a 7-26b
29	6819 6820 6821	265729 265730 265731	ACP ACP PVC	4 6 4	308	Layne	7-27
30	6963 5339	266026 262255	ACP	6	514	Floraes btwn Campana and Verdosa and Verdosa btwn Floraes and Vista	7-28
31	6998	266093	ACP	6	170	El Centro btwn Paul and Timlott	7-29
32	7093 2959	266293 257851	ACP	12	888	Alma btwn El Dorado and El Carmelo	7-14
33	7207	266552	ACP	10	606	Bayshore South of Embarcadero – also included in SIP-3	7-30
				Total	10,405		

Table 7-7. Aging Replacement Program (AIP) List of Projects

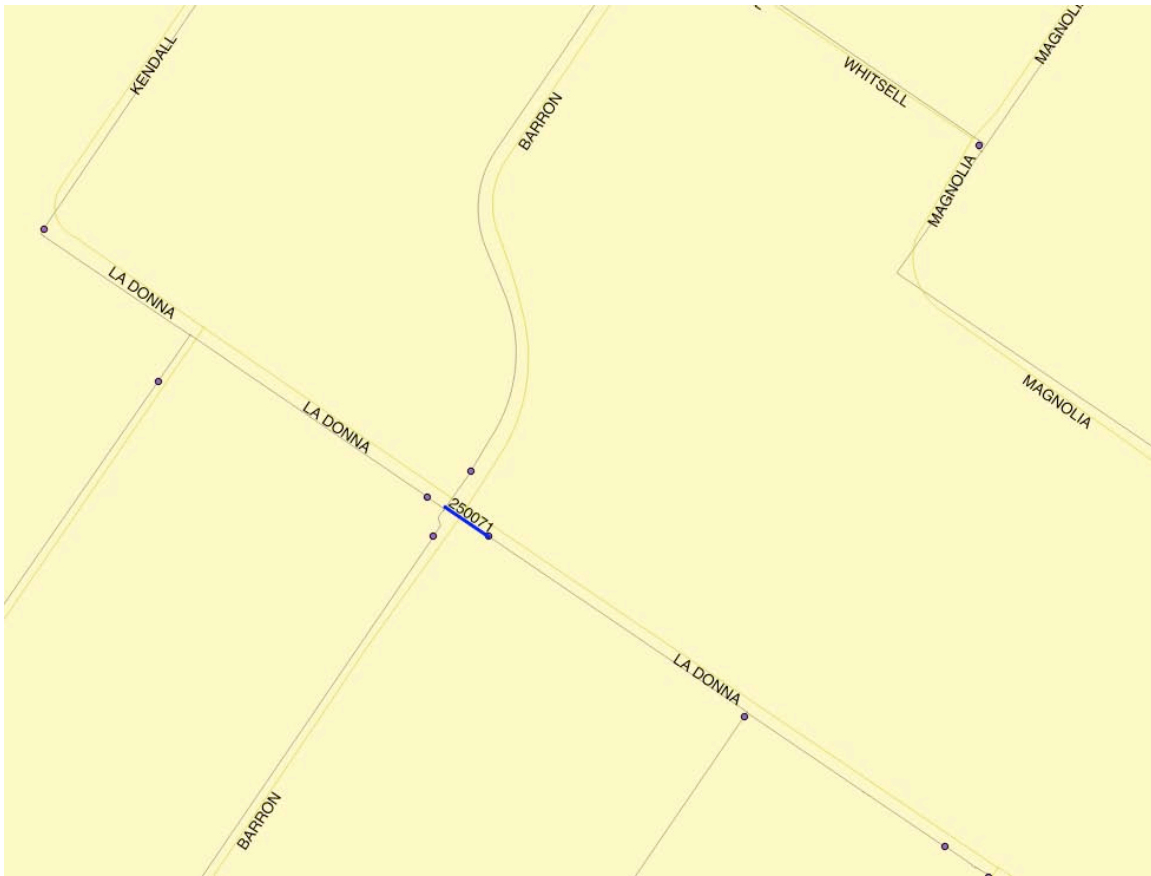


Figure 7-4. AIP Project 1 - LaDonna Replacement

Project No.	Main IDs	Pipe GIDs	Material	Diam (in)	Total Length (Ft)	Street Location / Notes	Figure No.
1	237	250071	ACP	6	42	La Donna @ Barron	7-4

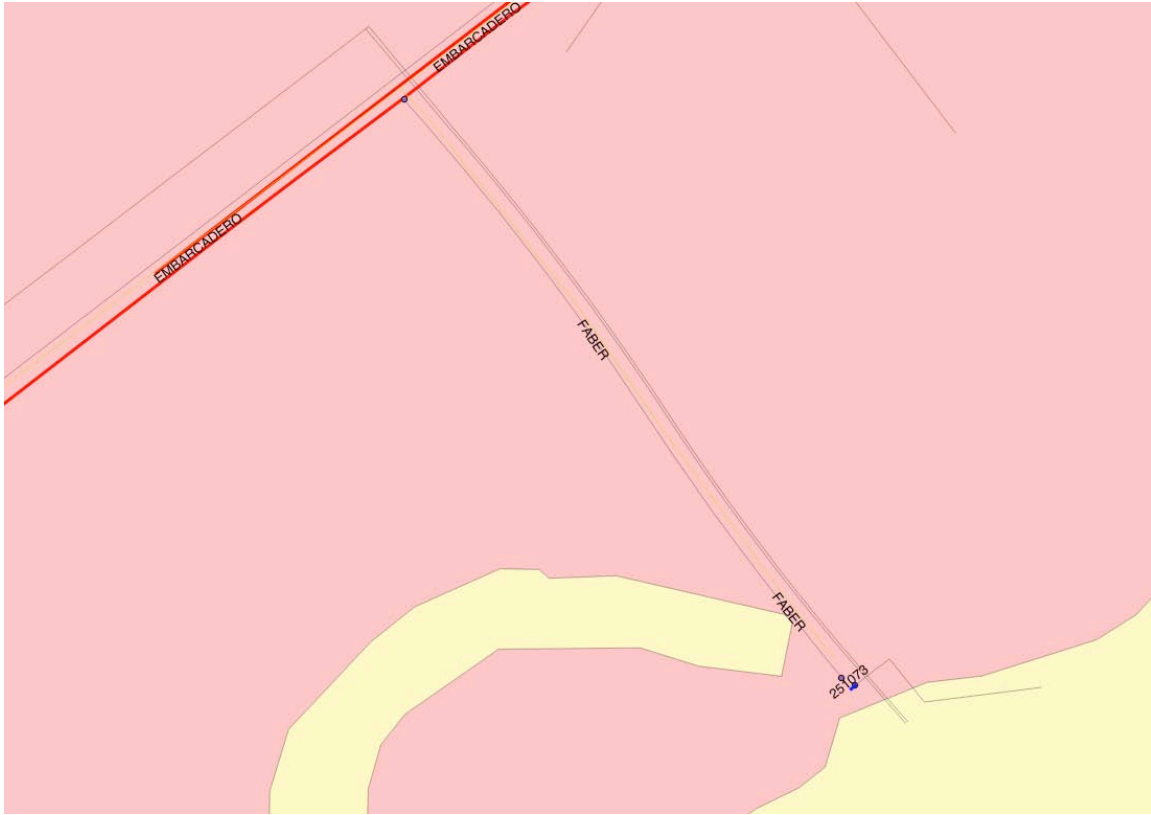


Figure 7-5. AIP Project 2 - Faber Replacement

Project No.	Main IDs	Pipe GIDs	Material	Diam (in)	Total Length (Ft)	Street Location / Notes	Figure No.
2	572	251073	UNKNOWN		6	Valve on Faber	7-5

Note: the red lines in Figure 7-5 along Embarcadero reflect pipes recommended for replacement due to seismic issues, see Section 7.4 for more details.

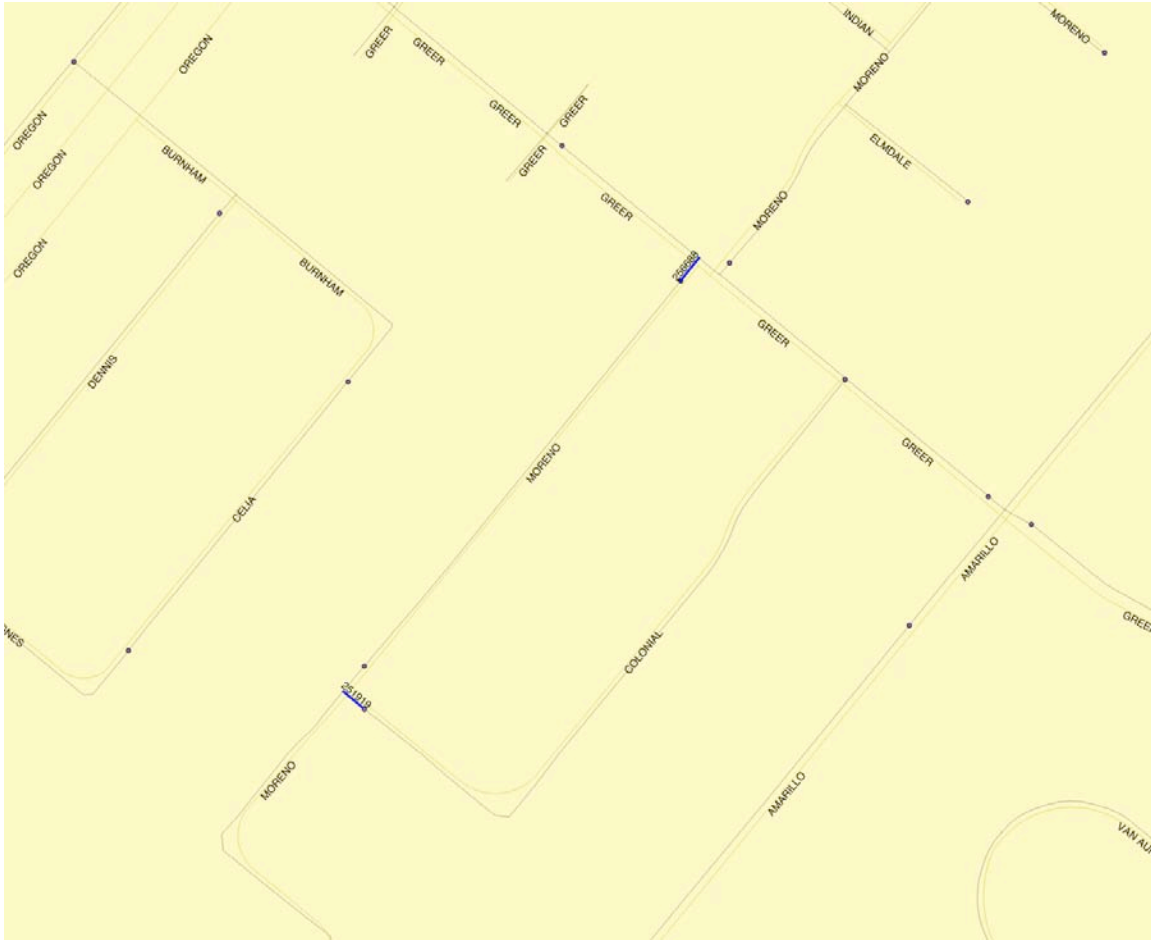


Figure 7-6. AIP Projects 3 and 14 – Colonial and Moreno Replacements

Project No.	Main IDs	Pipe GIDs	Material	Diam (in)	Total Length (Ft)	Street Location / Notes	Figure No.
3	865	251919	ACP	6	32	Valve on Colonial @ Moreno	7-6

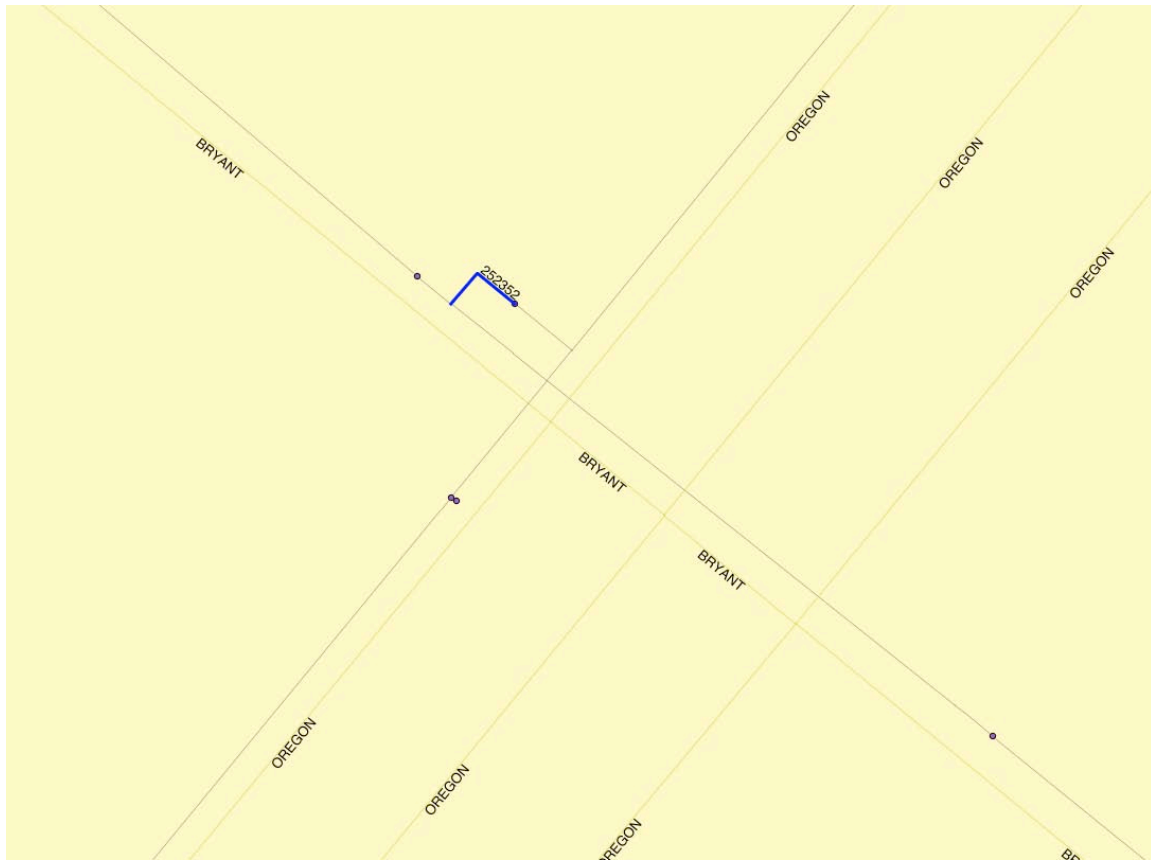


Figure 7-7. AIP Project 4 – Bryant Replacement

Project No.	Main IDs	Pipe GIDs	Material	Diam (in)	Total Length (Ft)	Street Location / Notes	Figure No.
4	1025	252352	ACP	6	22	Bryant @ Oregon	7-7

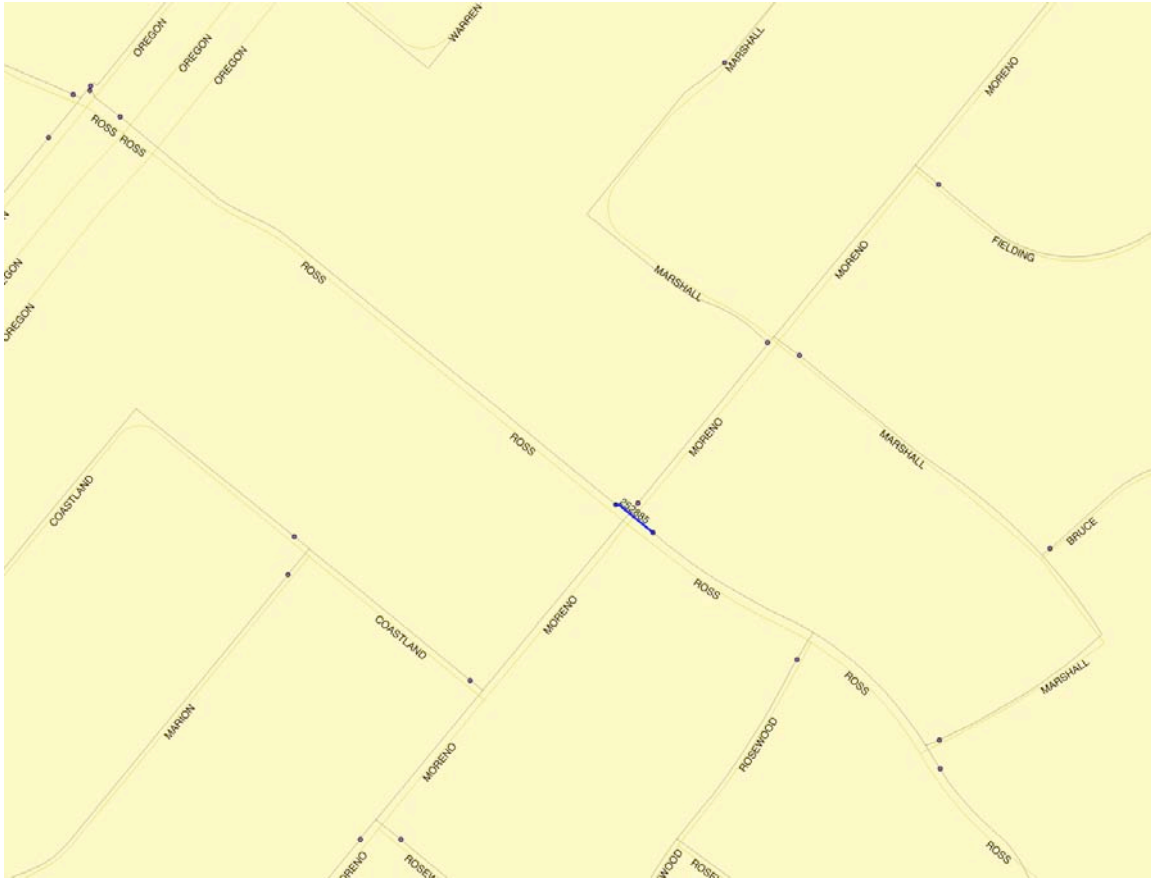


Figure 7-8. AIP Project 5 – Ross Replacement

Project No.	Main IDs	Pipe GIDs	Material	Diam (in)	Total Length (Ft)	Street Location / Notes	Figure No.
5	1205	252885	ACP	6	62	Valve on Ross @ Moreno	7-8

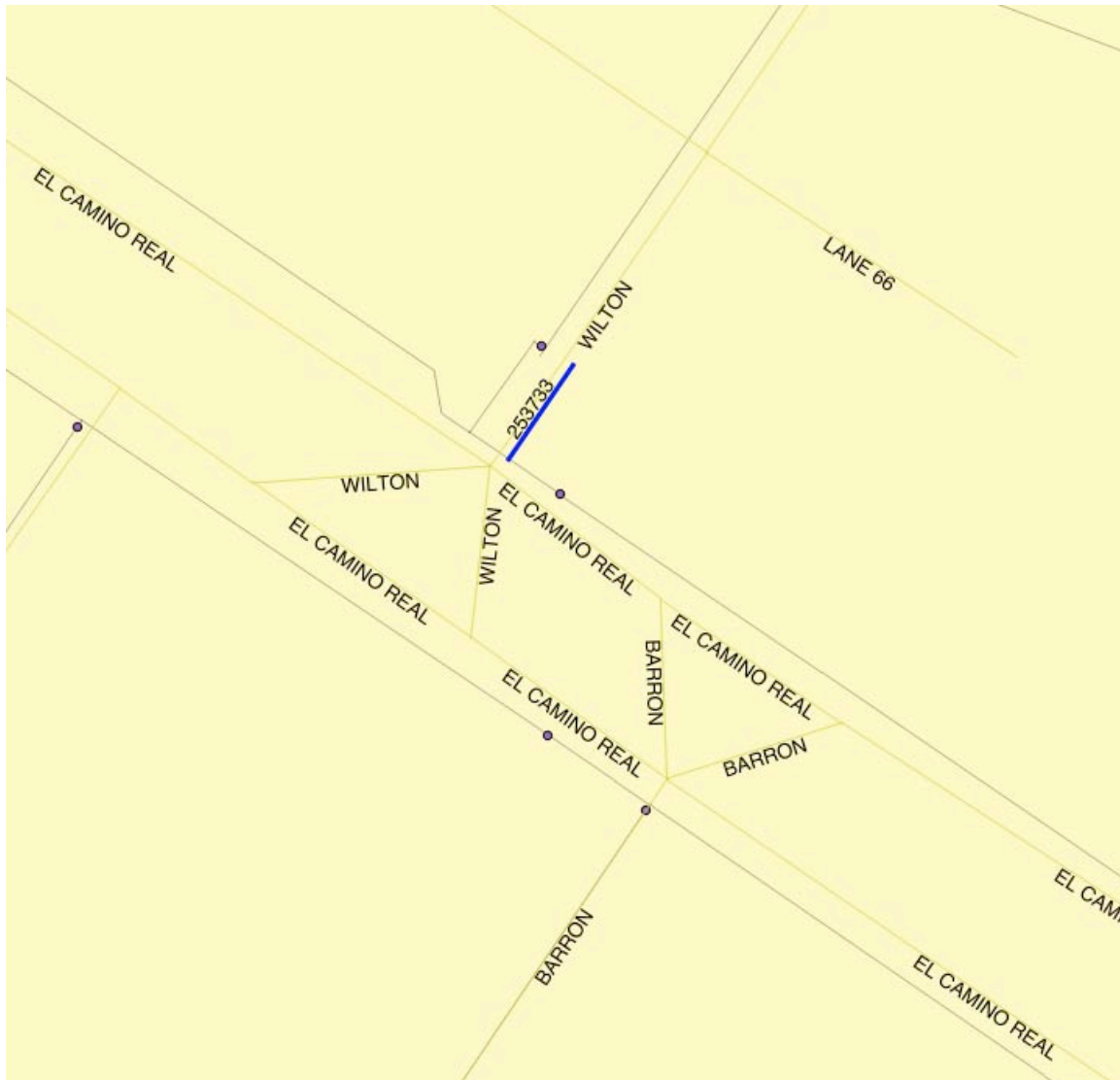


Figure 7-9. AIP Project 6 – Wilton Replacement

Project No.	Main IDs	Pipe GIDs	Material	Diam (in)	Total Length (Ft)	Street Location / Notes	Figure No.
6	1507	253733	ACP	8	43	Wilton @ El Camino Real	7-9

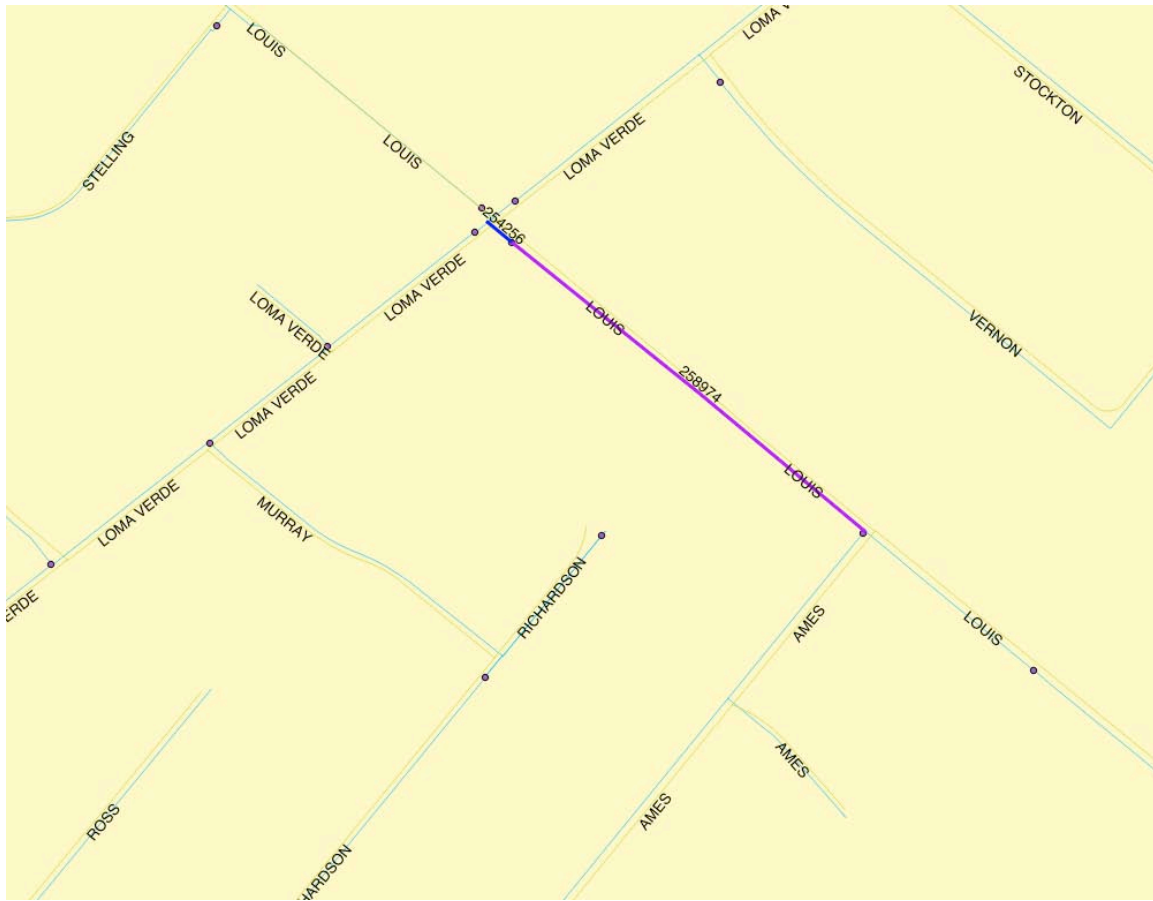


Figure 7-10. AIP Project 7 – Louis Replacement

Project No.	Main IDs	Pipe GIDs	Material	Diam (in)	Length (Ft)	Street Location / Notes	Figure No.
7	1679 3817	254256	ACP	6	37	Louis between Loma Verde and Ames	7-10
		258974	ACP	6	564		
					Total 601		



Figure 7-11. AIP Projects 8 and 25 – Laguna and Matadero Replacements

Project No.	Main IDs	Pipe GIDs	Material	Diam (in)	Length (Ft)	Street Location / Notes	Figure No.
8	1753	254504	ACP	12	315 27 Total 342	Valve on Laguna Extension to valve (orange)	7-11
25	5661	262990	CIP	12	122	Matadero btwn Chimalus and Laguna	7-11

Note; the orange line to the southeast along Laguna (Project 8) reflects a recommended extension of the pipe replacement to reflect seismic issues (to provide adequate anchor zone for liquefaction loading); see Section 7.4 for further details.



Figure 7-12. AIP Projects 9 and 16 – Hillview Replacements

Project No.	Main IDs	Pipe GIDs	Material	Diam (in)	Length (Ft)	Street Location / Notes	Figure No.
9	1819	254692	CIP	12	27	Intersection of Hillview/Porter/Hanover	7-12
	2650	257140	ACP	12	859		
	5661	257478	ACP	12	42		
					Total 928		

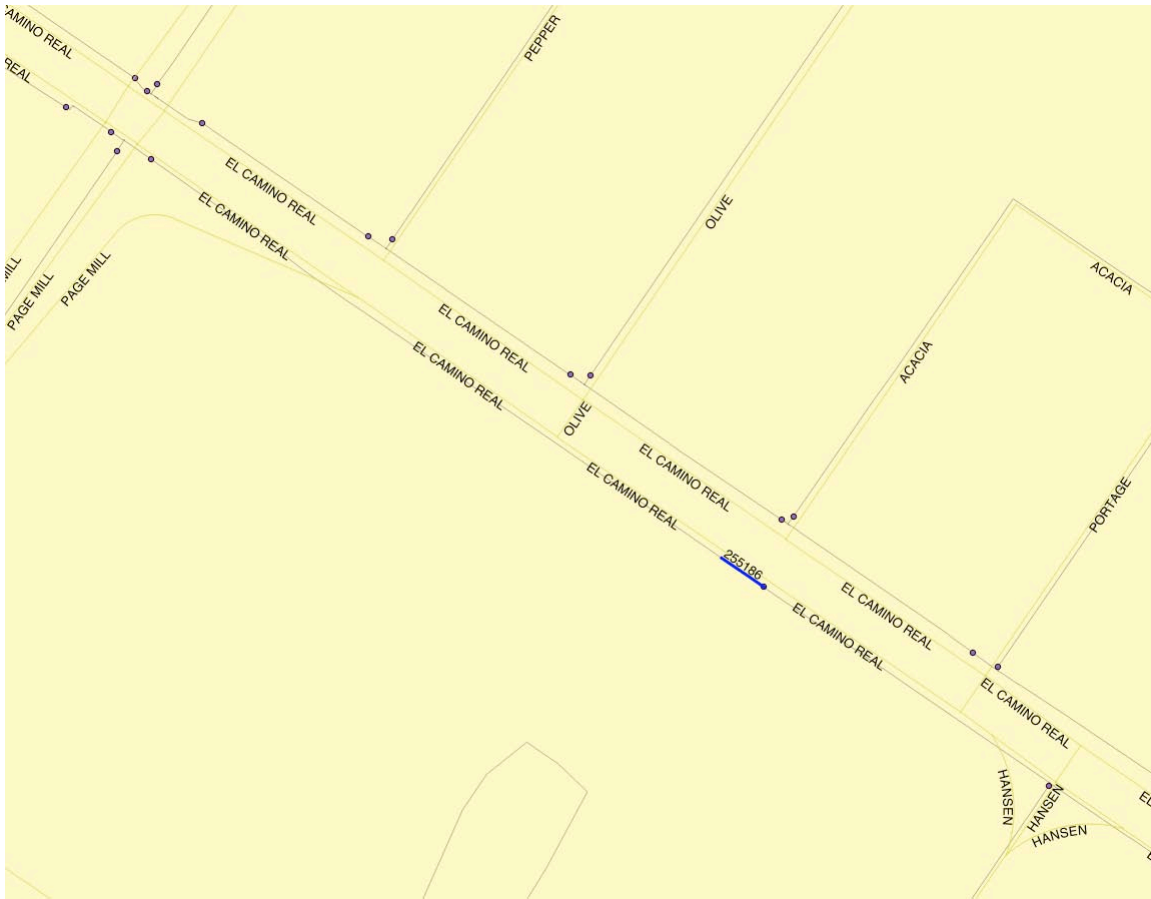


Figure 7-13. AIP Project 10 – El Camino Real Replacement

Project No.	Main IDs	Pipe GIDs	Material	Diam (in)	Total Length (Ft)	Street Location / Notes	Figure No.
10	1987	255186	ACP	8	63	El Camino Real @ Acacia	7-13

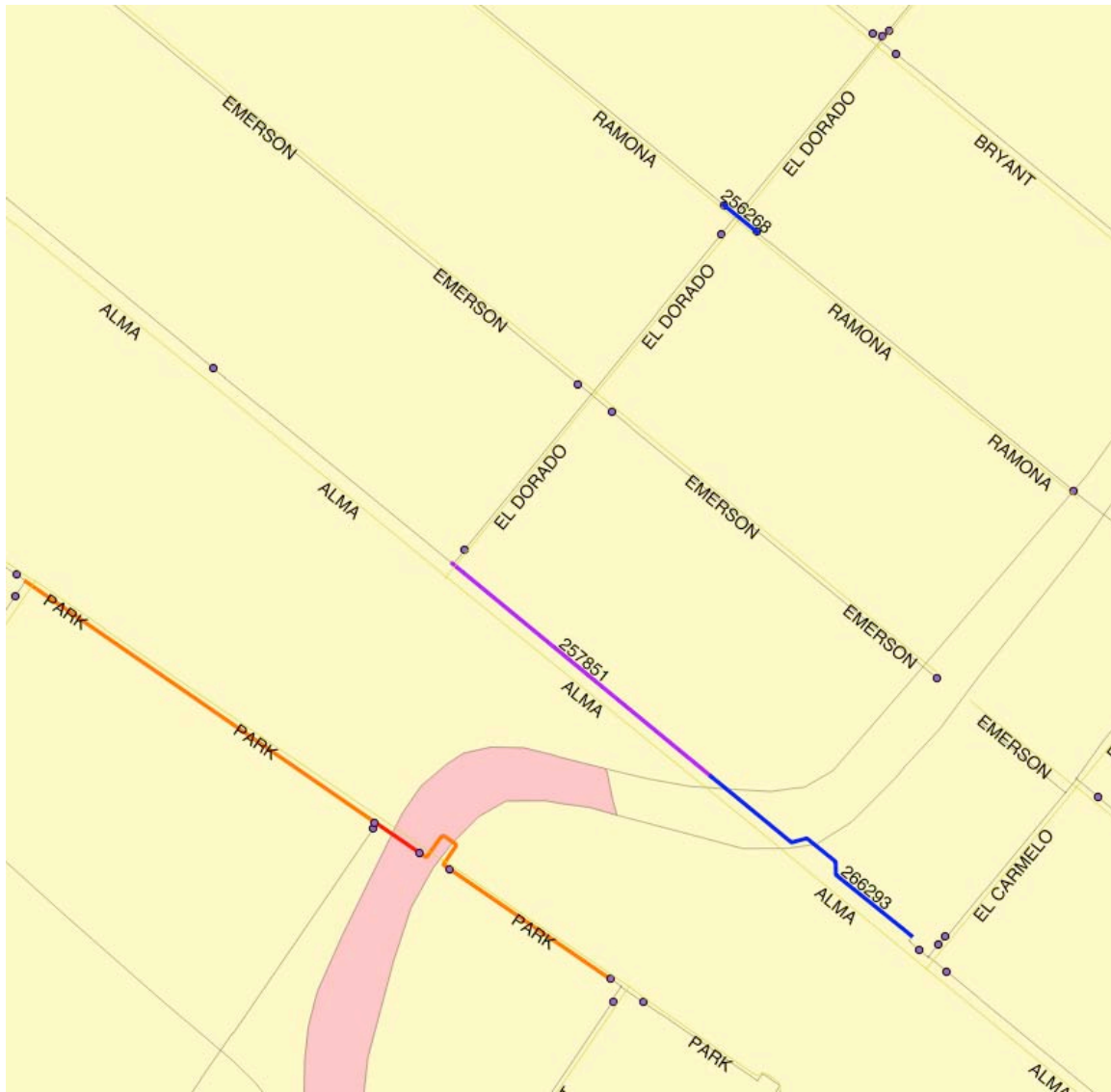


Figure 7-14. AIP Projects 11 and 32 – Ramona and Alma Replacements

Project No.	Main IDs	Pipe GIDs	Material	Diam (in)	Length (Ft)	Street Location / Notes	Figure No.
11	2354	256268	ACP	6	62	Ramona @ El Dorado	7-14
32	7093 2959	266293 257851	ACP	12	394 494 Total 888	Alma btwn El Dorado and El Carmelo	7-14

The orange pipe in Figure 7-14 is for a seismic upgrade, described in Section 7.4.

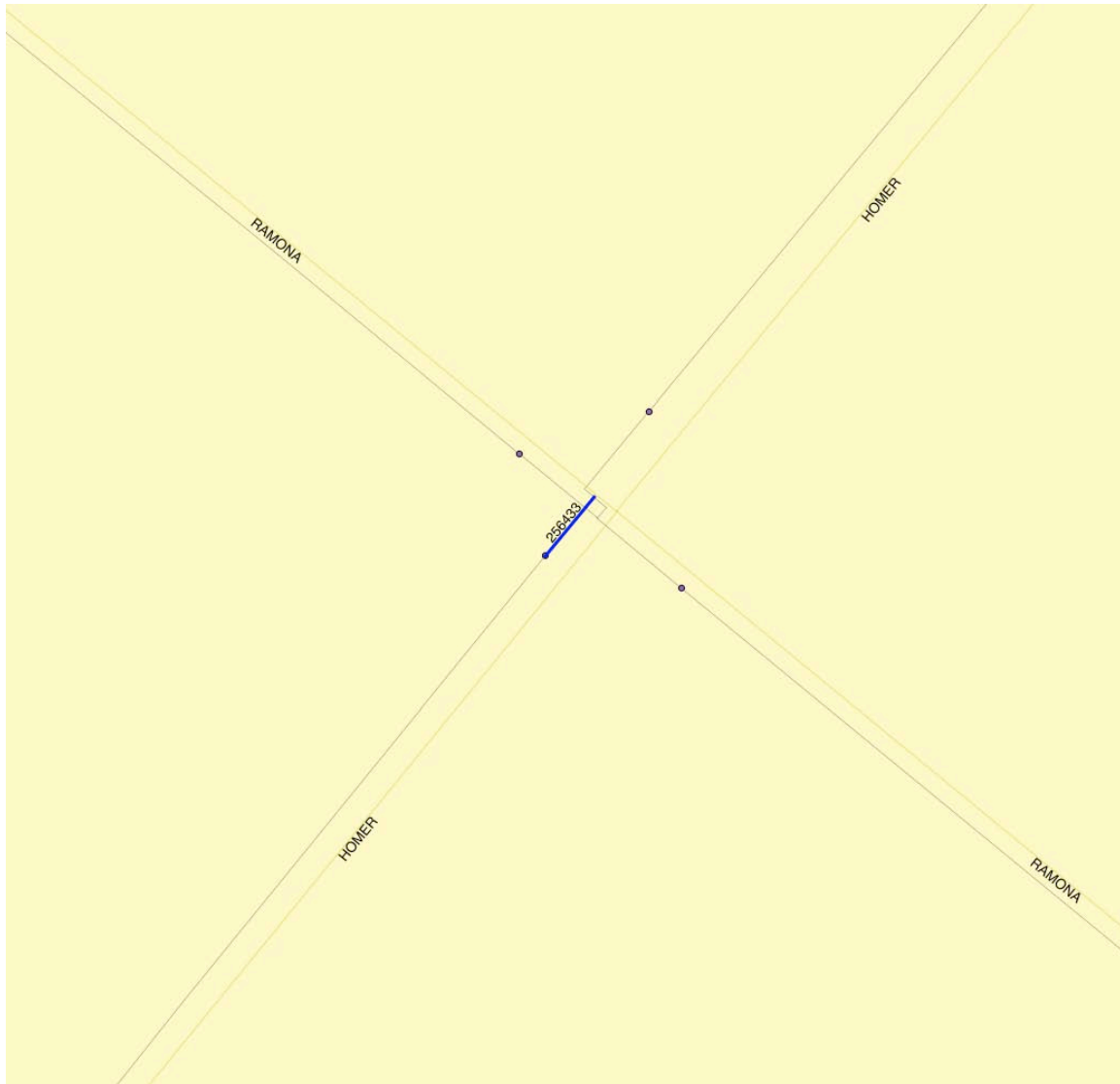


Figure 7-15. AIP Project 12 – Homer Replacement

Project No.	Main IDs	Pipe GIDs	Material	Diam (in)	Total Length (Ft)	Street Location / Notes	Figure No.
12	2398	256433	CIP	6	24	Homer @ Ramona	7-15

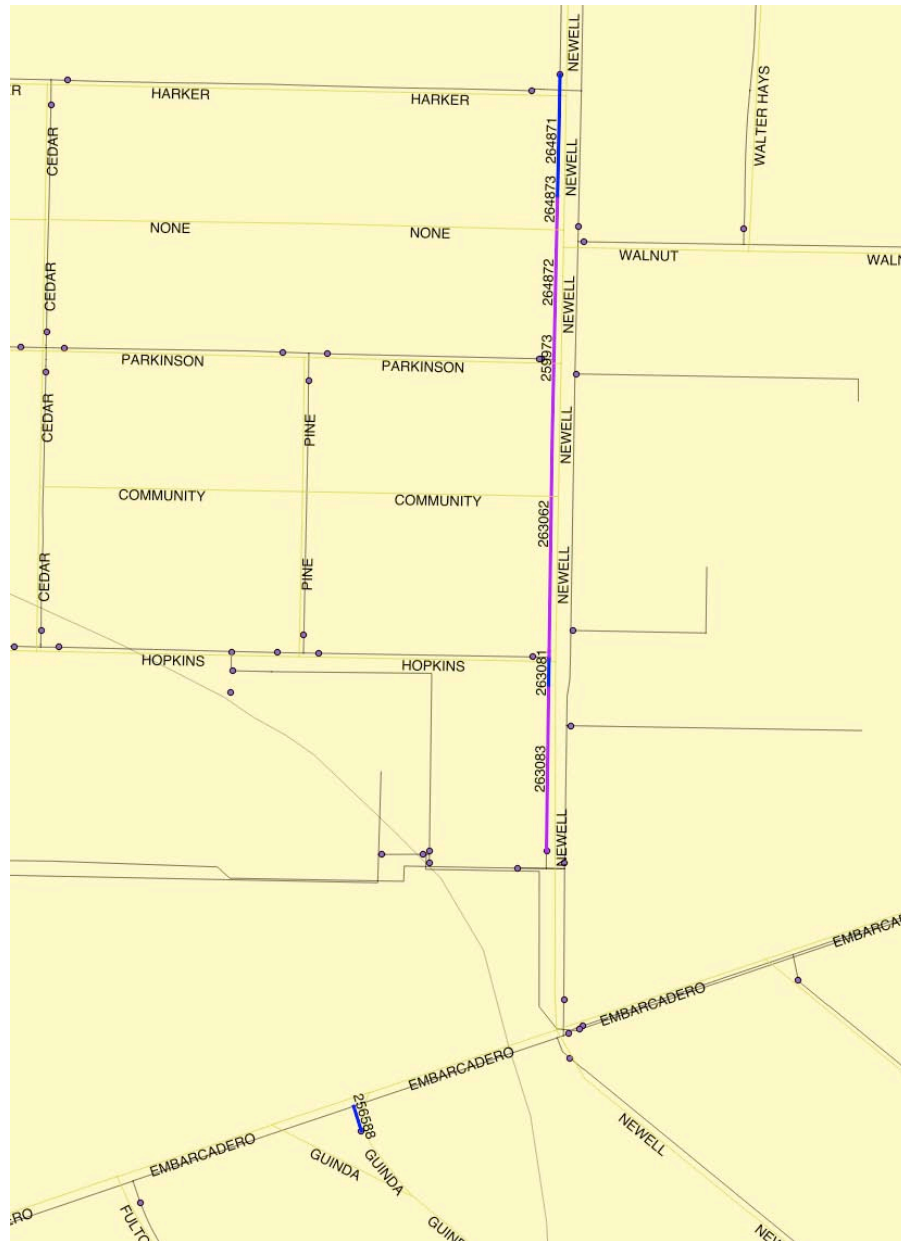


Figure 7-16. AIP Projects 13 and 26 – Guinda and Newell Replacements

Project No.	Main IDs	Pipe GIDs	Material	Diam (in)	Length (Ft)	Street Location / Notes	Figure No.
13	2455	256588	ACP	6	35	Guinda @ Embarcadero	7-16
26	5698	263081	ACP	8	36	Newell btwn Harker and Embarcadero	7-16
	6453	264871	ACP		166		
	6455	264873	PVC		9		
	6454	264872	CIP		211		
	4542	259973	CIP		4		
	5688	263062	CIP		400		
	5700	263083	CIP		222		
					Total		
					1,048		

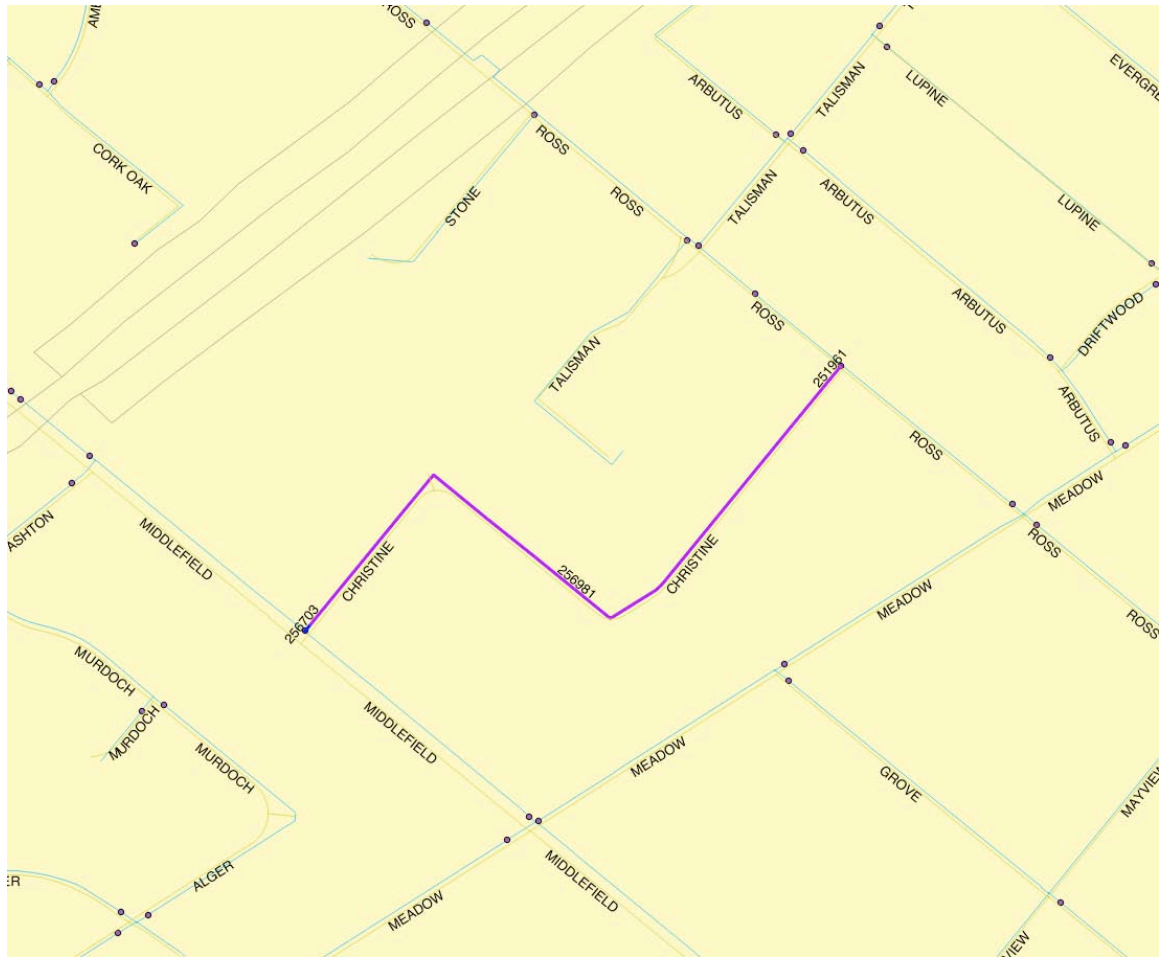


Figure 7-17. AIP Project 15 – Christine Replacement

Project No.	Main IDs	Pipe GIDs	Material	Diam (in)	Total Length (Ft)	Street Location / Notes	Figure No.
15	2496 881 2595	256703 251961 256981	ACP	6	12 36 1509 Total 1,557	Christine	7-17

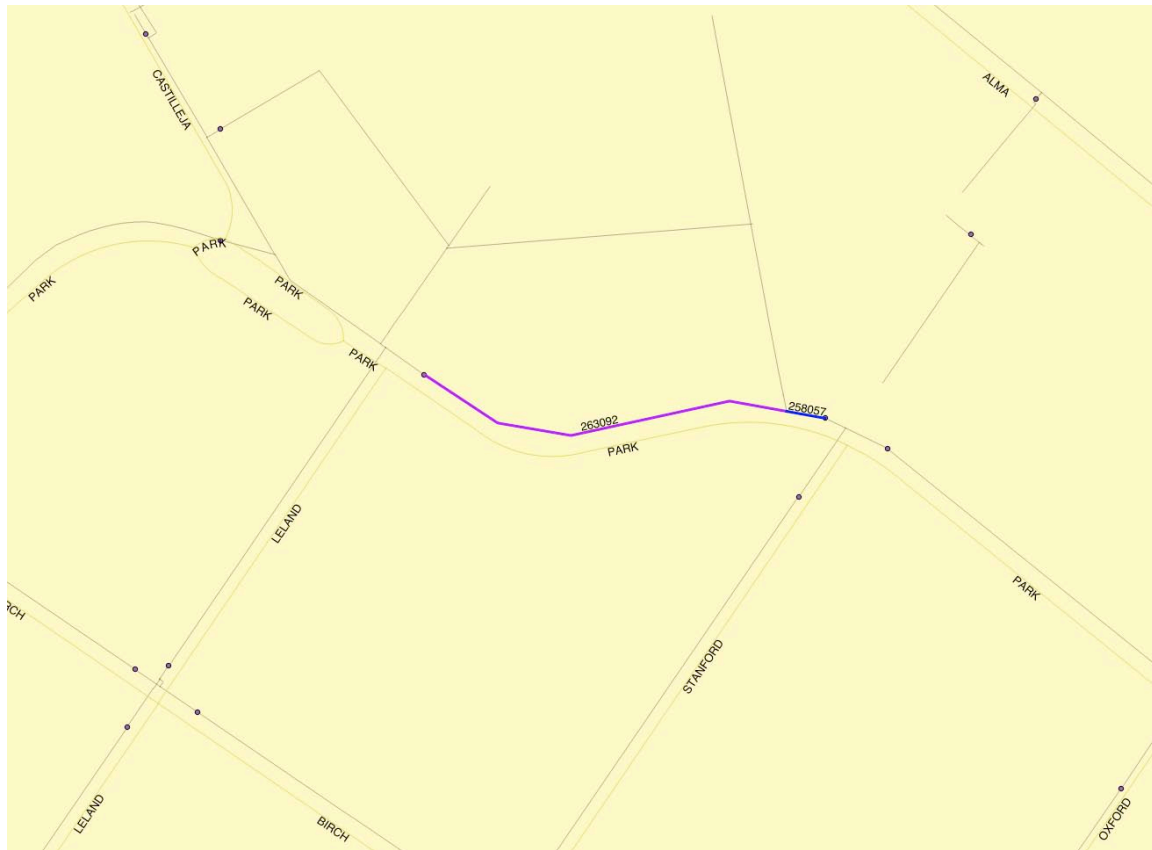


Figure 7-18. AIP Project 17 – Park Replacement

Project No.	Main IDs	Pipe GIDs	Material	Diam (in)	Length (Ft)	Street Location / Notes	Figure No.
17	3109 5701	258057 263092	DIP CIP	6	33 322 Total 355	Park btwn Leland and Stanford	7-18



Figure 7-19. AIP Project 18 – Alma Replacement

Project No.	Main IDs	Pipe GIDs	Material	Diam (in)	Length (Ft)	Street Location / Notes	Figure No.
18	3274	258275	UNKNOWN	12	17	Alma @ Homer	7-19

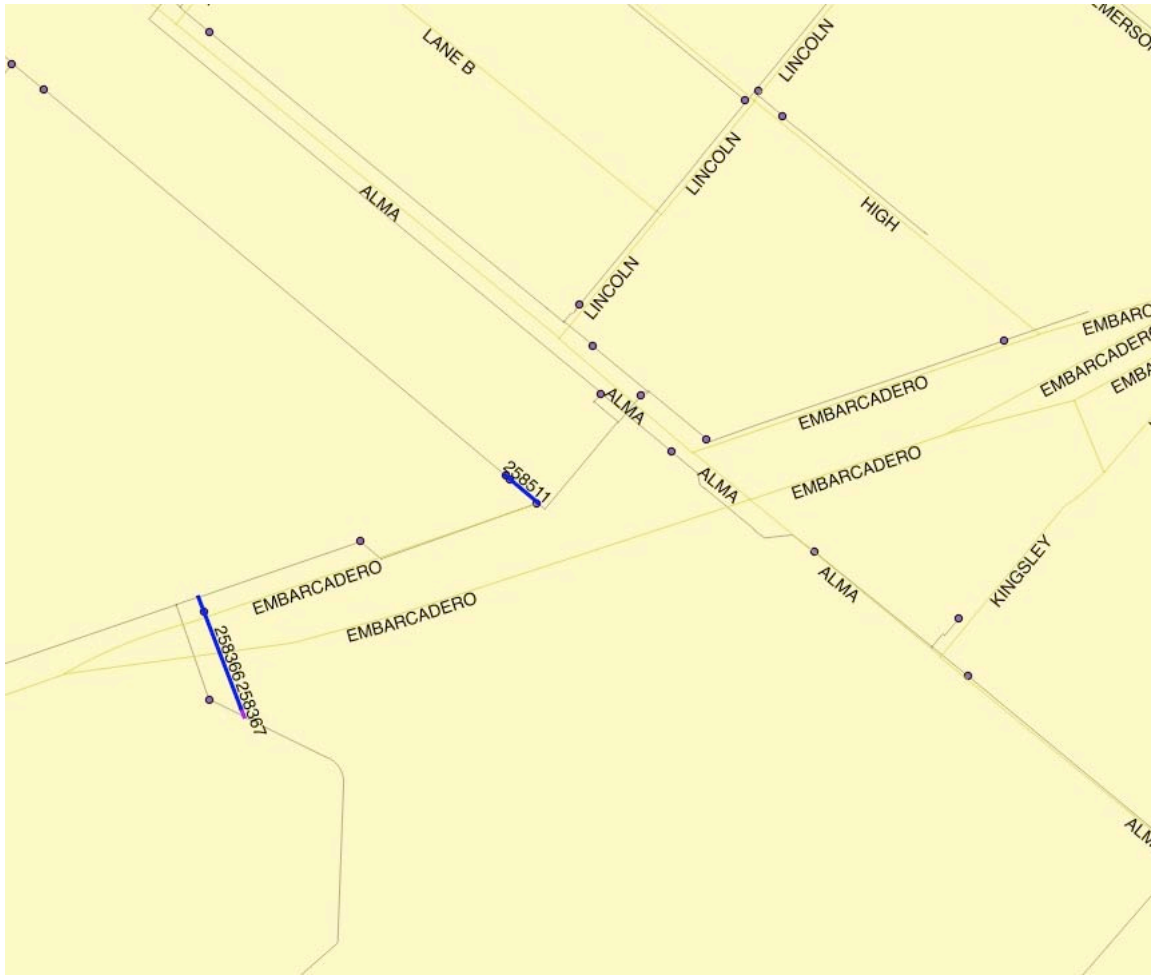


Figure 7-20. AIP Projects 19 and 20 – Embarcadero Replacements

Project No.	Main IDs	Pipe GIDs	Material	Diam (in)	Length (Ft)	Street Location / Notes	Figure No.
19	3346 3347	258366 258367	CIP	8	110 8 Total 118	Valve on Embarcadero South of Alma 8-foot stub: if needed to accommodate replaced valve	7-20
20	3461	258511	CIP	6	40	Embarcadero South of Alma	7-20

[map missing]

Figure 7-21. AIP Project 21 – Waverley Replacement

Project No.	Main IDs	Pipe GIDs	Material	Diam (in)	Total Length (Ft)	Street Location / Notes	Figure No.
21	3581	258654	ACP	10	55	Waverley between Embarcadero and Churchill	7-21
	4639	260118	CIP	10	169		
	2016	255272	CIP	6	39		
	4303	259640	CIP	10	434		
					625		

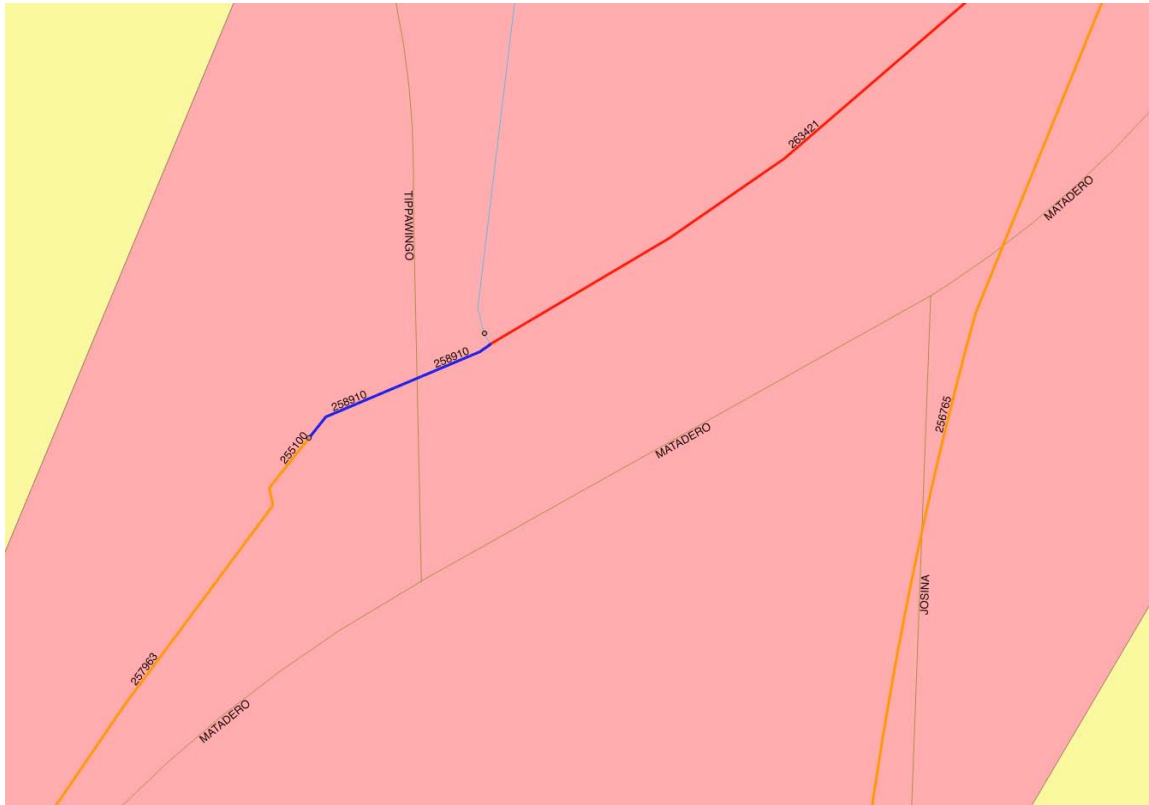


Figure 7-22. AIP Project 22 – Matadero Replacement

Project No.	Main IDs	Pipe GIDs	Material	Diam (in)	Length (Ft)	Street Location / Notes	Figure No.
22	3766	258910	ACP	6	30	Matadero @ Tippawingo – also included in SIP-2	7-22

Note: this pipe segment (blue line) is also part of the seismic upgrades that are further described in Section 7.4 (red and orange lines).

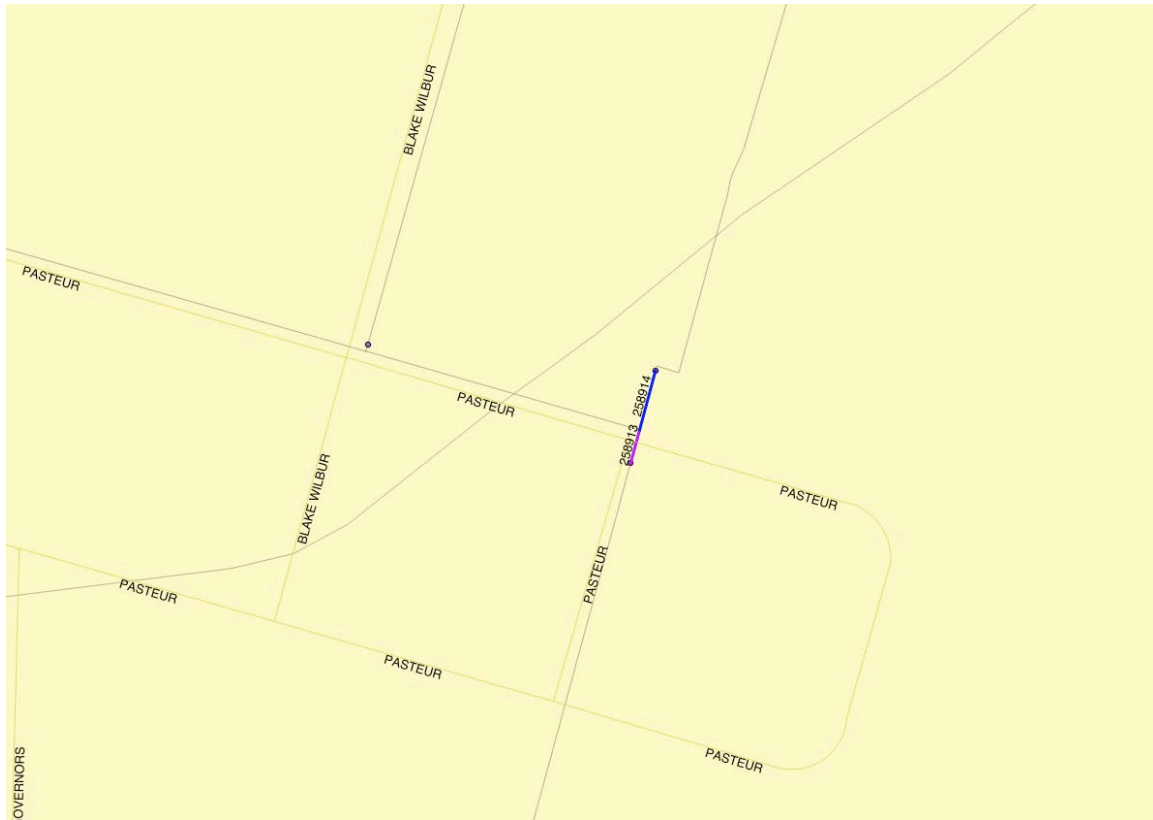


Figure 7-23. AIP Project 23 – Pasteur Replacement

Project No.	Main IDs	Pipe GIDs	Material	Diam (in)	Length (Ft)	Street Location / Notes	Figure No.
23	3770 3769	258914 258913	CIP ACP	10	48 27 Total 75	Pasteur	7-23

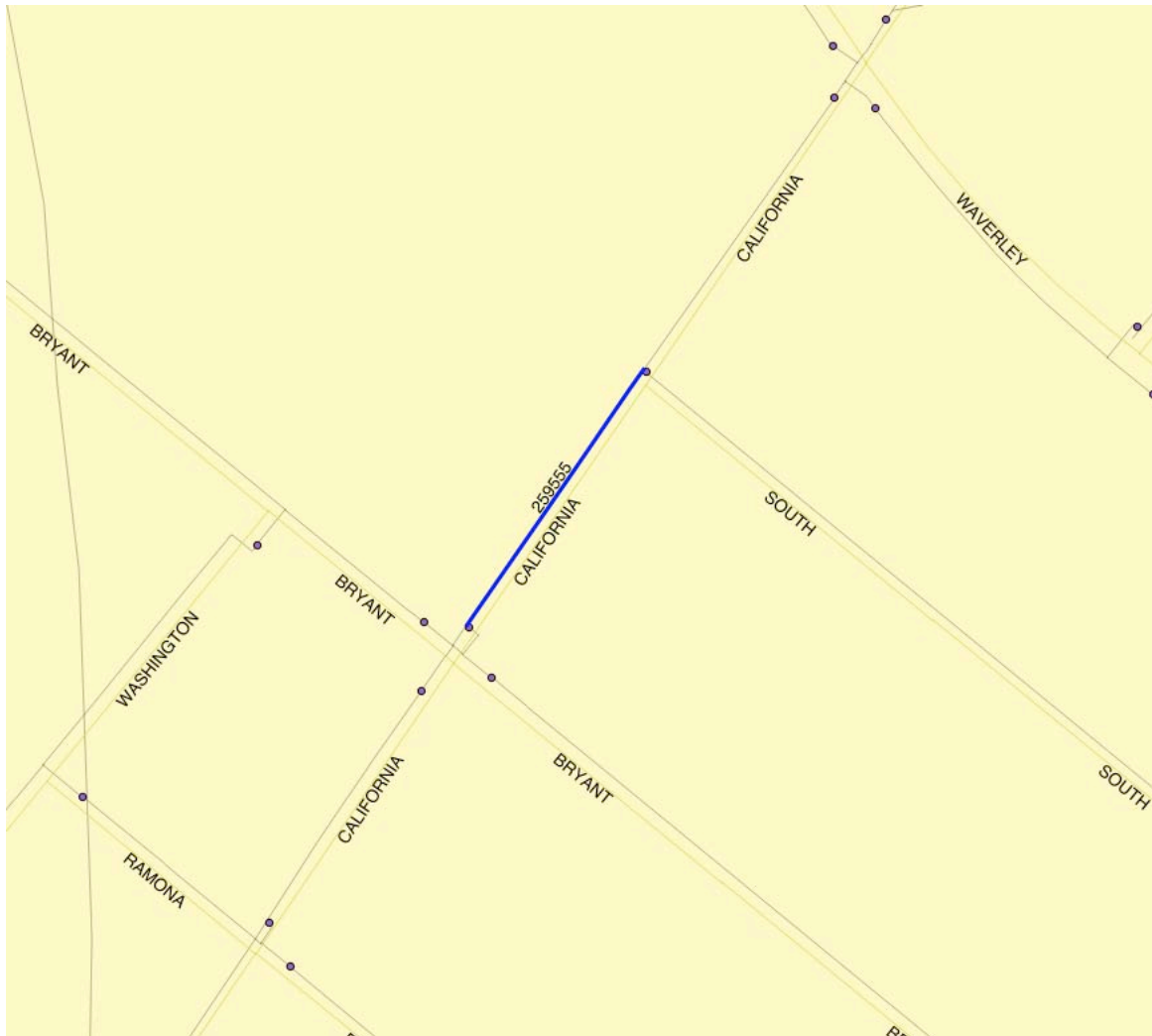


Figure 7-24. AIP Project 24 – California Replacement

Project No.	Main IDs	Pipe GIDs	Material	Diam (in)	Length (Ft)	Street Location / Notes	Figure No.
24	4238	259555	CIP	10	248	California btwn Bryant and South	7-24

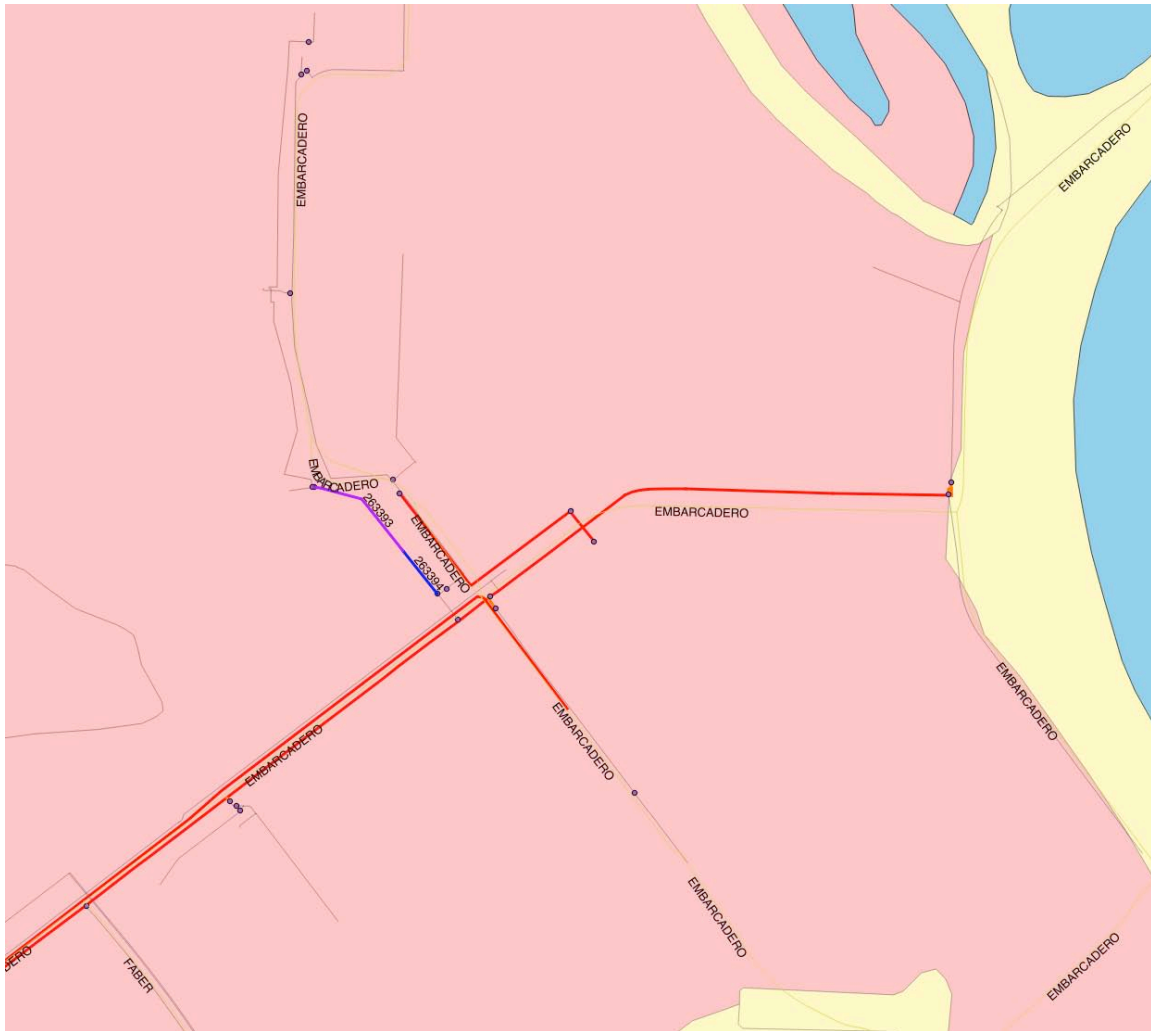


Figure 7-25. AIP Project 27 – Embarcadero Replacement

Project No.	Main IDs	Pipe GIDs	Material	Diam (in)	Length (Ft)	Street Location / Notes	Figure No.
27	5837 5836	263394 263393	ACP	8	130 284 Total 414	Embarcadero Near the Bay	7-25

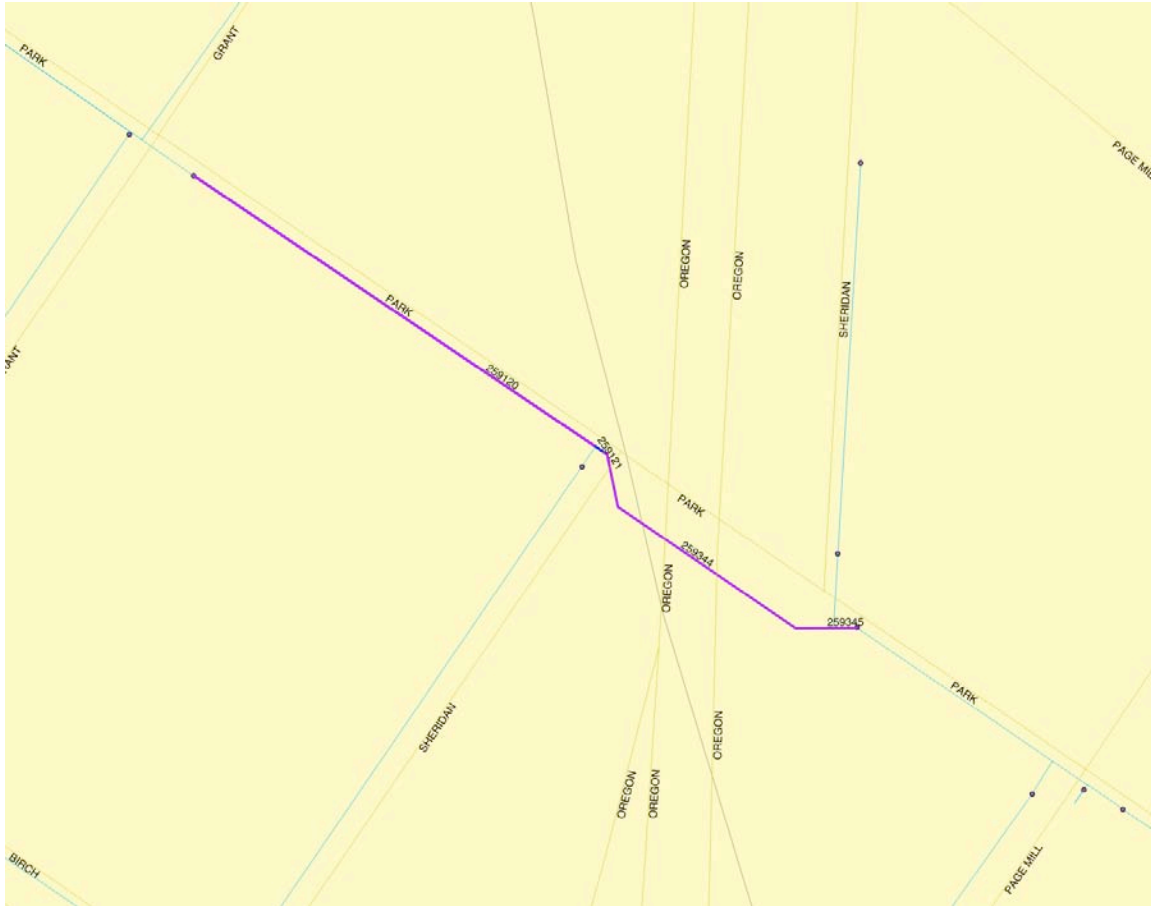


Figure 7-26a. AIP Project 28 – Park Replacement – Overall

Project No.	Main IDs	Pipe GIDs	Material	Diam (in)	Length (Ft)	Street Location / Notes	Figure No.
28	6112	264027	CIP	12	5	Park btwn Grant and Sheridan	7-26a 7-26b
	3929	259120	ACP		598		
	3930	259121	PVC		8		
	4085	259344	ACP		185		
	4086	259345	ACP		15		
					Total 884		

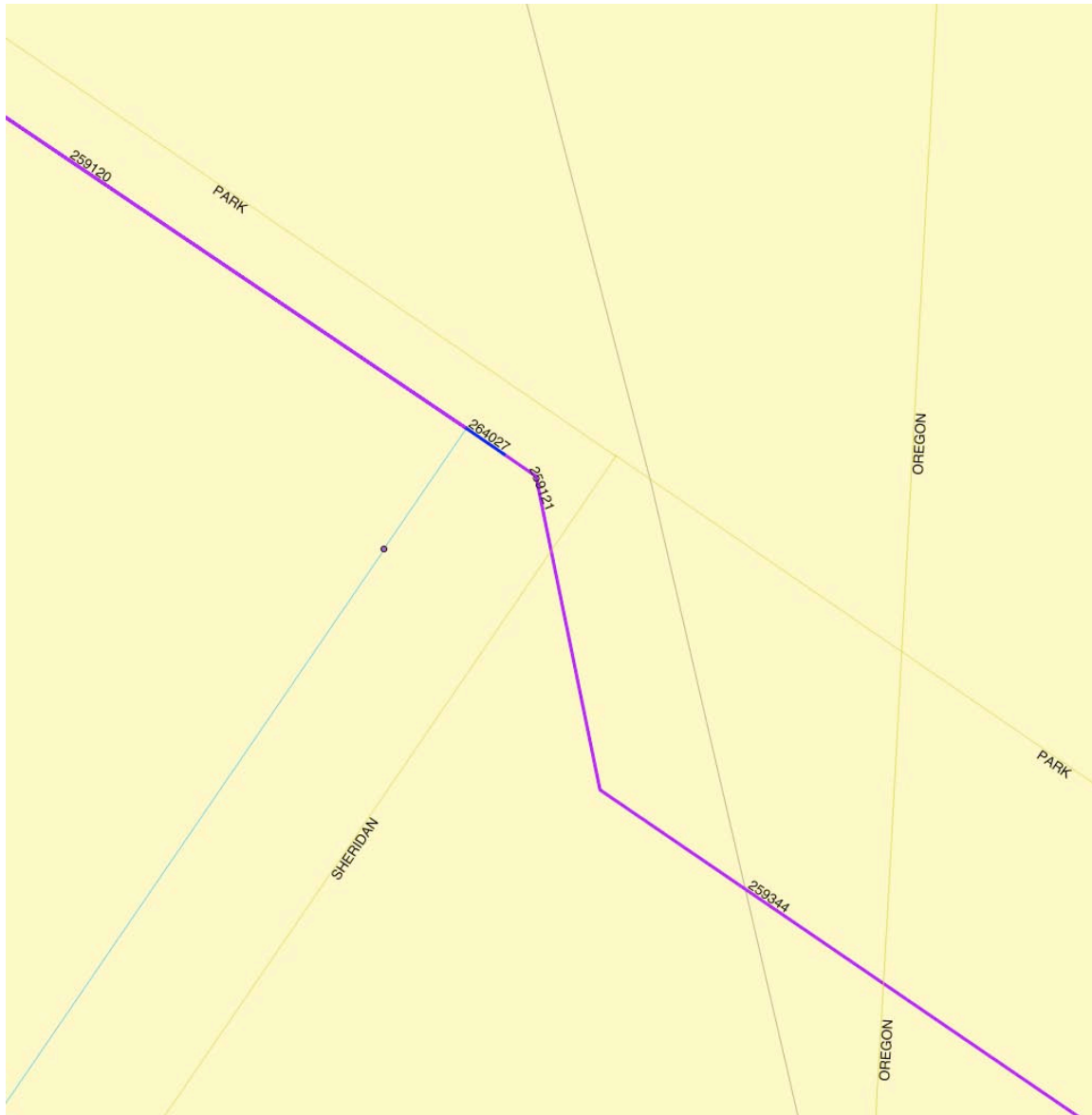


Figure 7-26b. AIP Project 28 – Park Replacement – Detail

Project No.	Main IDs	Pipe GIDs	Material	Diam (in)	Length (Ft)	Street Location / Notes	Figure No.
28	6112	264027	CIP	12	5	Park btwn Grant and Sheridan	7-26a 7-26b
	3929	259120	ACP		598		
	3930	259121	PVC		8		
	4085	259344	ACP		185		
	4086	259345	ACP		15		
					Total 884		

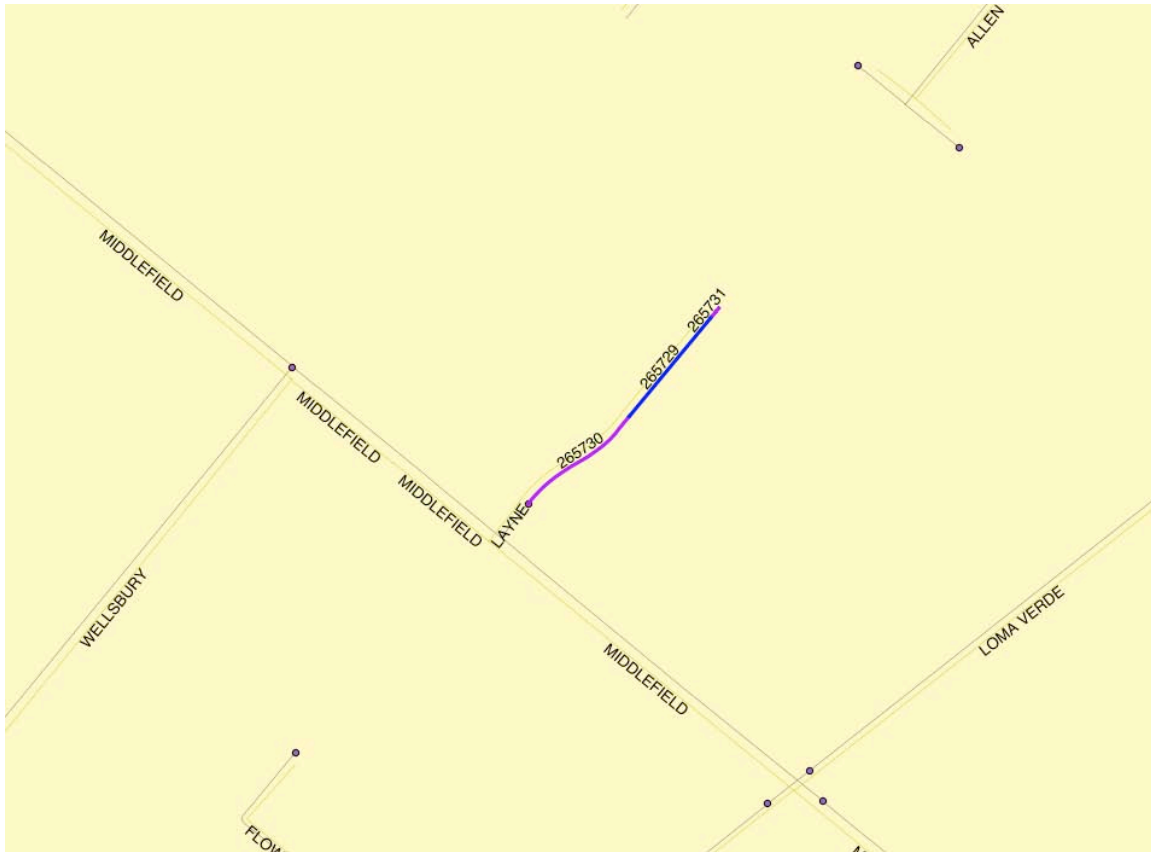


Figure 7-27. AIP Project 29 – Layne Replacement

Project No.	Main IDs	Pipe GIDs	Material	Diam (in)	Total Length (Ft)	Street Location / Notes	Figure No.
29	6819	265729	ACP	4	146	Layne (Replace with 6" or larger for entire length)	7-27
	6820	265730	ACP	6	153		
	6821	265731	PVC	4	9		
					Total 308		

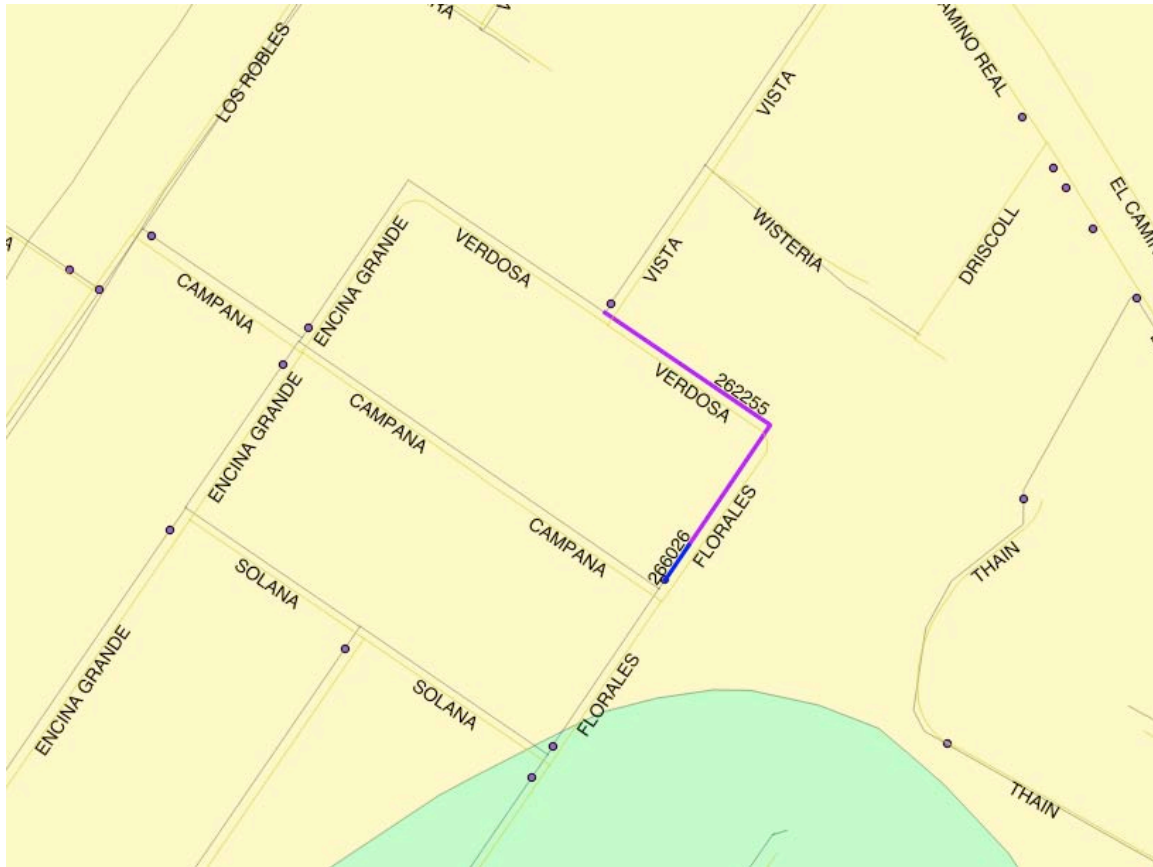


Figure 7-28. AIP Project 30 – Flores and Verdosa Replacement

Project No.	Main IDs	Pipe GIDs	Material	Diam (in)	Length (Ft)	Street Location / Notes	Figure No.
30	6963 5339	266026 262255	ACP ACP	6 6	56 458 Total 514	Flores btwn Campana and Verdosa and Verdosa btwn Flores and Vista. 458-foot segment 262255 might be eliminated in final design as it has had no history of leakage; but as it was installed in 1946, it is included herein to upgrade from valve to valve	7-28

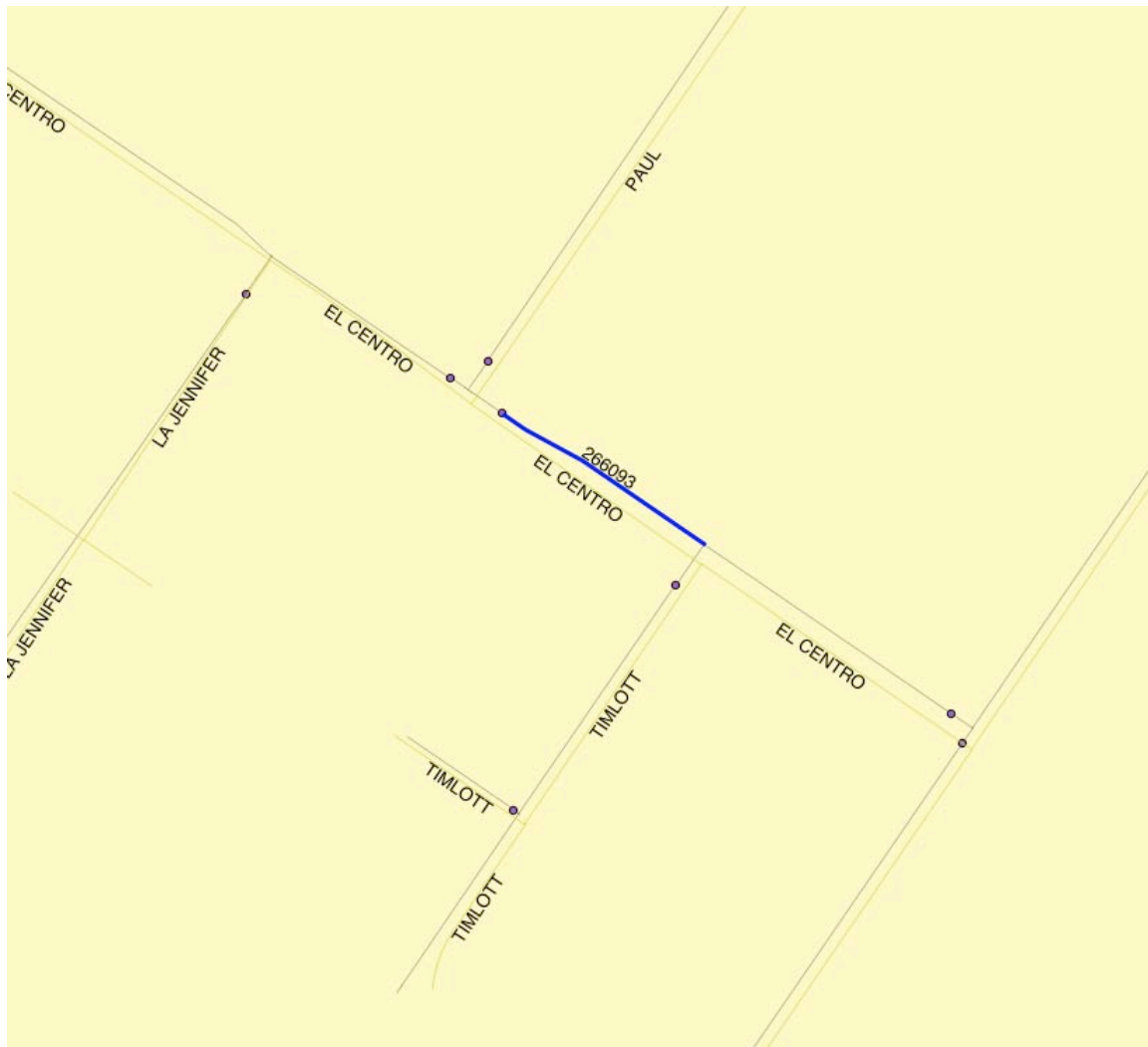


Figure 7-29. AIP Project 31 – El Centro Replacement

Project No.	Main IDs	Pipe GIDs	Material	Diam (in)	Length (Ft)	Street Location / Notes	Figure No.
31	6998	266093	ACP	6	170	El Centro btwn Paul and Timlott	7-29



Figure 7-30. AIP Project 33 – Bayshore Replacement

Project No.	Main IDs	Pipe GIDs	Material	Diam (in)	Length (Ft)	Street Location / Notes	Figure No.
33	7207	266552	ACP	10	606	Bayshore South of Embarcadero – also included in SIP-3	7-30

7.4 Pipe Replacement for Seismic Issues

7.4.1 Benefit Cost Model - Seismic

The need to replace older pipes in the Palo Alto water system for seismic issues is discussed in Section 7.4. In order to perform the benefit cost computation in equation 1, we need to establish the "do nothing / as-is" option, and then consider various strategies to improve the performance of the water system in earthquakes.

Tables 4-19 and 4-20 list the number of pipe repairs expected in each of 24 different scenario earthquakes. In Section 7, we make the simplifying assumption that Palo Alto has seismically upgraded all of its other infrastructure (like water tanks and pump stations and wells) and that the SFPUC is able to deliver water to the Palo Alto turnouts, and/or the Palo Alto wells are able to produce water, in each of the earthquakes, generally within 24 hours of the earthquake.

Table 7-8 lists the level of effort needed to make the pipe repairs in each of the 24 scenario earthquakes.

EQ	Fault	M	Manhours for repair Median	Manhours for repair 84th	Days to complete repairs, median	Days to complete repairs, 84th
1	San Andreas Santa Cruz	6.9	190	345	2.0	3.6
2	San Andreas Peninsula	6.0	254	994	2.6	10.4
3	San Andreas Peninsula	6.2	719	2,742	7.5	28.6
4	San Andreas Peninsula	6.4	1,464	3,950	15.2	41.1
5	San Andreas Peninsula	6.6	2,251	5,261	23.4	54.8
6	San Andreas Peninsula	6.8	3,047	6,581	31.7	68.5
7	San Andreas Peninsula	7.0	3,934	7,896	41.0	82.3
8	San Andreas SAN+SAP+SAS	7.2	3,861	7,920	40.2	82.5
9	San Andreas SAN+SAP+SAS	7.4	6,248	10,872	65.1	113.3
10	San Andreas SAN+SAP+SAS	7.5	6,973	11,955	72.6	124.5
11	San Andreas SAN+SAP+SAS	7.7	8,432	14,187	87.8	147.8
12	San Andreas SAN+SAP+SAS	7.9	9,917	16,427	103.3	171.1
13	San Andreas SAN+SAP+SAS	8.0	10,762	17,527	112.1	182.6
14	Hayward N+S	7.25	2,684	6,381	28.0	66.5
15	Hayward South	6.8	1,212	3,641	12.6	37.9
16	Hayward North	6.8	151	274	1.6	2.8
17	West Napa	6.0	2	28	0.0	0.3
18	Rodgers Creek	7.0	104	189	1.1	2.0
19	Calaveras North + Central + South	7.2	1,550	4,334	16.1	45.1
20	San Gregorio	7.7	3,890	8,426	40.5	87.8
21	Mount Diablo Thrust	6.5	107	194	1.1	2.0
22	Monta Vista	6.8	4,598	8,592	47.9	89.5
23	Greenville	7.0	173	313	1.8	3.3
24	Zayante – Vergeles	6.9	159	288	1.7	3.0

Table 7-8. Level of Effort Needed to Make Pipe Repairs, Days to Complete (2 crews)

The column "Manhours for repair, Median" refers to the number of manhours by pipe repair personnel, in the field, needed to complete all the pipe repairs, Median in Table 4-19. Similarly, the column "Manhours for repair, 84th" refers to the number of manhours

by pipe repair personnel, in the field, needed to complete all the pipe repairs, 84th, in Table 4-19. The pipe repairs in Table 4-19 are those that occur immediately or after the earthquake, up to the time that the initial set of pipe repairs are made and water is restored to nearly every customer service lateral (up to the meter). We use an average of 50 manhours needed to make each repair: for 8-inch and smaller pipe, the average pipe repair time might be somewhat under 40 manhours, while for larger diameter pipe, it might be over 60 manhours, so the average of 50 manhours is suitable in the post-earthquake environment. The work will generally involve identifying the location of the pipe repair; closing upstream and downstream valves; USA alerts (might be bypassed in an emergency, but assumed not), using an excavator to dig a hole to reach the pipe; dewatering the hole; examining the type of damage; choosing a pipe repair method (clamp, replace a segment or two of pipe, repair a service lateral connection, etc.); making the repair; opening a valve for a pressure test to see if the repair does not leak; disinfection of the pipe; fill the hole; compact the soil; repair the street surface. Under day-to-day type leaks, such a repair for a 8" pipe might take about 30 manhours (4-man crew taking about 6 clock hours to perform, plus some time leaving the yard, returning to the yard). In a post-earthquake environment, the repair time would be longer, on average, in that communications will be sub-par, road closures might occur, some spare parts might not be easily available, etc.

The columns "Days to Complete Repairs" (median and 84th) are calculated assuming that there are two 4-person crews available right after the earthquake, and each person works 12-hour shifts until all repairs are completed. This recognizes that under emergency conditions, the normal 40-hour work-week is extended to 72 hours per person during the emergency situation.

If we use the median values, we see that the time needed to restore water service to the last customer is between 0 to 2 days for smaller, more distant earthquakes (Like Napa 2014, Mount Diablo thrust, Greenville, Hayward North, etc.). However, the restoration times, using only two crews, extend to 1 week (San Andreas M 6.2 or nearly 3.5 months (San Andreas M 8.0).

With just 2 crews and this amount of pipe damage, there will be lengthy water outages in Palo Alto to some customers.

The lack of water will lead to economic impacts to those customers without water. For example, lacking potable water for an extended time, hospitals will likely have to evacuate all their patients; laundromats cannot operate; most commercial offices will close. Residents at homes will be greatly inconvenienced by lack of water, although sufficient water for consumption and sanitation could be purchased. From an economic point of view, we take the following approach to quantify these impacts:

- We estimate the total Gross Regional Product produce in Palo Alto, in one year, to be about \$9.441 Billion, or \$25,986,753 per day (see Section 2.6.). Of this, about 70% is lost if there is no water supply, or \$18,106,027.

- Using the outage times (Table 7-4) and initial service levels after the earthquake (before and after valves are turned closed), we estimate the equivalent number of Service Days Lost (SDLs) for each scenario earthquake. We assume that once valves are closed, each incremental pipe repair will correspond to a linear increase in the number of customers that receive water. In making this calculation, we do not know for sure "which pipes get fixed first", so we make the assumption that the repair effort will linearly increase the number of customers restored, over time.
- The economic losses are a combination of loss of economic activity due to loss of water (this is the largest portion, about 80% of the total), plus some losses of structures burned due to fire, plus the cost to make the pipe repairs themselves. For example. Assume $SDL = 20.972$ days. The total economic activity would be $\$18,106,027 * 20.972 = \$379,725,326$. Add to this the cost to make the pipe repairs, which in this example is 10,762 manhours times \$130 per manhour = \$1,399,060. The \$130 per manhour represents the labor and benefits for the pipe repair person; plus small parts / spare parts; plus equipment costs (excavators, dump trucks, etc); plus office overhead. Add to this the loss due to fires in Palo Alto, which prior studies have shown, for large earthquakes with a severely impacted water supply, can be approximated at about 25% of the direct economic losses (results will vary a lot, but the 25% value is representative for purposes of the benefit cost analysis).

Table 7-9 lists the results for each of the 24 scenario earthquakes. Table 7-5 shows that for smaller or distance earthquakes, the Economic losses are very small, and this would imply that no mitigation is worthwhile to address these smaller / distant earthquakes.

EQ	Fault	M	SDLs Total, Median	SDLs Total, 84th	Scenario Economic Loss, Median	Scenario Economic Loss, 84th
1	San Andreas Santa Cruz	6.9	0.000	0.000	\$24,700	\$44,850
2	San Andreas Peninsula	6.0	0.022	0.053	\$536,290	\$1,322,706
3	San Andreas Peninsula	6.2	0.089	0.265	\$2,106,183	\$6,356,889
4	San Andreas Peninsula	6.4	0.514	1.224	\$11,839,889	\$28,250,554
5	San Andreas Peninsula	6.6	1.339	2.812	\$30,644,995	\$64,436,385
6	San Andreas Peninsula	6.8	2.794	5.625	\$63,734,264	\$128,357,068
7	San Andreas Peninsula	7.0	3.674	7.015	\$83,798,413	\$160,056,519
8	San Andreas SAN+SAP+SAS	7.2	3.442	6.689	\$78,519,128	\$152,658,577
9	San Andreas SAN+SAP+SAS	7.4	7.333	12.353	\$167,043,013	\$281,430,182
10	San Andreas SAN+SAP+SAS	7.5	8.317	13.868	\$189,452,607	\$315,917,688
11	San Andreas SAN+SAP+SAS	7.7	11.728	19.321	\$266,960,695	\$439,819,117
12	San Andreas SAN+SAP+SAS	7.9	14.585	23.746	\$331,906,186	\$540,431,497
13	San Andreas SAN+SAP+SAS	8.0	20.972	33.767	\$476,815,103	\$767,730,961
14	Hayward N+S	7.25	2.157	4.660	\$49,238,416	\$106,455,173
15	Hayward South	6.8	0.265	0.658	\$6,157,635	\$15,392,364
16	Hayward North	6.8	0.000	0.000	\$19,565	\$35,555
17	West Napa	6.0	0.000	0.000	\$195	\$3,640
18	Rodgers Creek	7.0	0.000	0.000	\$13,520	\$24,570
19	Calaveras North + Central + South	7.2	0.747	1.713	\$17,124,026	\$39,406,022
20	San Gregorio	7.7	3.465	7.093	\$79,050,639	\$161,891,533
21	Mount Diablo Thrust	6.5	0.000	0.000	\$13,845	\$25,155
22	Monta Vista	6.8	4.301	7.723	\$98,103,975	\$176,194,347
23	Greenville	7.0	0.000	0.000	\$22,425	\$40,690
24	Zayante – Vergeles	6.9	0.000	0.000	\$20,605	\$37,375

Table 7-9. System Days Lost, Scenario Economic Losses

However, the economic losses rise rapidly for increasing magnitude earthquakes on the nearby San Andreas fault, reaching about \$477 million for a San Andreas M 8 event (median).

Next, we factor in the annual chance of each of the scenario earthquakes. This is done in Table 7-10. At the bottom of Table 7-10, we add in the annualized loss for each of the 24 scenario earthquake, and add 50% more to reflect all the other possible earthquakes that were not considered as part of the suite of 24 scenario earthquakes. For example, there is a range of smaller magnitudes possible on the San Gregorio fault (we only considered a M 7.7), and each in turn contribute to the overall losses. Examining Table 7-10, it is clear that we have captured essentially all the losses for the full range of earthquakes on the San Andreas fault, but we have not captured all the possible losses from smaller magnitude, but more common earthquakes on the Calaveras, San Gregorio, Hayward, Rodgers Creek and other faults. Further, we include some allowance for other faults not listed in Table 7-6, especially unknown faults along the Peninsula.

EQ	Fault	M	Annual interval probability of earthquake in the range of the listed M	Annual Loss, Median	Annual Loss, 84th
1	San Andreas Santa Cruz	6.9	0.02	\$494	\$897
2	San Andreas Peninsula	6.0	0.003	\$1,609	\$3,968
3	San Andreas Peninsula	6.2	0.003	\$6,319	\$19,071
4	San Andreas Peninsula	6.4	0.003	\$35,520	\$84,752
5	San Andreas Peninsula	6.6	0.0023	\$70,483	\$148,204
6	San Andreas Peninsula	6.8	0.0018	\$114,722	\$231,043
7	San Andreas Peninsula	7.0	0.0015	\$125,698	\$240,085
8	San Andreas SAN+SAP+SAS	7.2	0.0014	\$109,927	\$213,722
9	San Andreas SAN+SAP+SAS	7.4	0.0013	\$217,156	\$365,859
10	San Andreas SAN+SAP+SAS	7.5	0.0012	\$227,343	\$379,101
11	San Andreas SAN+SAP+SAS	7.7	0.001	\$266,961	\$439,819
12	San Andreas SAN+SAP+SAS	7.9	0.0008	\$265,525	\$432,345
13	San Andreas SAN+SAP+SAS	8.0	0.0006	\$286,089	\$460,639
14	Hayward N+S	7.25	0.002	\$98,477	\$212,910
15	Hayward South	6.8	0.01	\$61,576	\$153,924
16	Hayward North	6.8	0.01	\$196	\$356
17	West Napa	6.0	0.002	\$0	\$7
18	Rodgers Creek	7.0	0.006	\$81	\$147
19	Calaveras North + Central + South	7.2	0.01	\$171,240	\$394,060
20	San Gregorio	7.7	0.003	\$237,152	\$485,675
21	Mount Diablo Thrust	6.5	0.003	\$42	\$75
22	Monta Vista	6.8	0.0002	\$19,621	\$35,239
23	Greenville	7.0	0.005	\$112	\$203
24	Zayante – Vergeles	6.9	0.0002	\$4	\$7
	Total, 24 Scenario Earthquakes			\$2,316,345	\$4,302,108
	Total, Other faults and M			\$1,158,173	\$2,151,054
	Total, All Earthquakes			\$3,474,518	\$6,453,162

Table 7-10. Annualized Seismic Economic Losses (As Is)

The annual probability values for the San Andreas fault have the largest influence in determining the overall annual losses. In Table 7-10, the cumulative chance of a M 6.6 to M 8.0 is 35.7%. This compares reasonably well with recent USGS estimates of a greater

than ~25% chance of a M 6.7 or greater earthquake on the San Andreas fault in the next 30 years.

Another facet of the results in Table 7-10 is that the Monta Vista Shannon M 6.8 event, while very damaging to Palo Alto should it occur, is statistically only about 0.6% of the overall annualized loss (\$19,621 versus \$3,474,518). In other words, the need to spend a lot of money to seismically upgrade the Foothills Pipeline for fault offset across this fault zone, is only likely to be cost effective if the cost is not much over \$450,000 or so ($\$19,621 * (\text{NPV } \$1, 4\%, 60 \text{ years} = 22.62) = \$443,827$).

In the "do nothing" alternative, then the total Net Present Value (NPV) of earthquake losses is $\$3,474,518 * 22.62 = \$78,593,597$ (say \$79 million).

This \$79 million value is an important benchmark. It says that a solution / mitigation strategy is cost effective if its present value (\$2015) less than \$79 million.

One possible strategy (which we think is a poor strategy) would be to replace all "non-seismic" pipe in the Palo Alto water system with "seismically-designed" pipe. Today (2015), the only pipe in the Palo Alto system that would be considered "seismically-designed" is about 53,800 feet (10 miles) of recently-installed HDPE pipe. This leaves about 222 miles of "non-seismic" pipe. Outright replacement of all 222 miles of pipe might cost about \$1.66 million per mile (average over all diameters), or \$369 million. Even if the replaced pipe was "seismic-proof" (and no pipe is absolutely seismic-proof), the benefit cost ratio of such a strategy is low: \$79 million / \$369 million = 0.21. In other words, this is an economically-unsound strategy.

Another possibly strategy (which we think has some merit) is to replace all the non-seismic pipe in locations with VH and H liquefaction susceptibility, and replace it with HDPE, or DIP Chained (like Kubota pipe, US Pipe, etc.) or other similarly-designed pipe that can sustain at least 1% ground strain with rare chance of failure. System-wide, there are 24.07 miles of water pipe in these zones. By replacing these pipes with seismically-designed pipes, the number of pipe repairs in future earthquakes will be dramatically reduced (see Table 4-20, where the typical large earthquake, about 75% to 85% of all pipe damage is due to liquefaction).

In the SERA model, the chance of pipe failure for existing pipe (like Cast Iron) is very high in a liquefaction zone, whereas the chance of failure for a HDPE pipe in the same location is very low. As a reasonable approximation, we assume that 95% of the total seismic losses could be reduced if all the seismically-weak pipes in liquefaction zones were replaced with seismically-designed pipes. This results in 432 pipe segments, or 111,368 feet of pipe, or about 21.09 miles of pipe (see Table 7-8 for summary, see computer database for complete listing of each individual segment). The pipe replacement costs would be about \$30.5 million (about \$1.5 million pipe mile). Comparing this capital cost versus the expected benefits (reduction of future losses from \$79 million to about \$2 million), or a net benefit of about \$77 million, we see that the

overall benefit cost ratio of this strategy is about \$77 million (benefits) to \$30.5 million (capital costs), or about 2.5. While there are a number of assumptions that could reduce this BCR ratio (say, instead of 95% reduction in losses, it was only 75%), the BCR would still be about 2, or a good project.

While the "brute force" approach of pipe replacement appears attractive (BCR in the 2 to 2.5 range), the "emergency response" strategy should also be considered. By "emergency response" we mean a combination of strategies, including:

- Be able to ramp up with more than 2 repair crews, rapidly, after any earthquake. This can be done using the Cal Warn (or similar) mutual aid agreement, and pre-set contract agreements. In the recent Napa 2014 earthquake, the City of Napa was able to get 10 mutual aid crews to assist in their pipe repair effort, thus lowering the time needed to restore most customers to about 15 days (see Appendix B). For a San Andreas M 7+ event, the rapid availability of repair crews from EBMUD, ACWD and other Bay Area water companies will be severely restricted, as those utilities will need those same people to make repairs in their own systems. Thus, the availability of crews will be more difficult for Palo Alto in a large San Andreas M event. We think that practically speaking, Palo Alto, through an effective emergency response plan, could ramp up to 10 crews, each crew being 5 people (1 person from Palo Alto, the rest from the Mutual Aid agency), with each person working 10 hours per day (70 hour weeks) until the initial pipe repair effort is complete. More crews are theoretically possible, but practically speaking, we find that Palo Alto will need 1 person (Palo Alto crew person) assigned to each mutual aid crew, and this will de-facto limit the total crews feasible to about 10.
- The cost to use mutual aid is "not free", and generally, Palo Alto will have to pay for all the costs of the mutual aid crews, including labor, equipment, spare parts, logistics (housing, food, etc.). Using current rules, FEMA will reimburse Palo Alto for a percentage (currently 75%) of these mutual aid costs, as long as Palo Alto fulfills all of FEMA's obligations (a lot of book keeping for sure, as well as having a suitable hazard mitigation plan.).
- To ensure that the mutual aid is effective, it is recommended that Palo Alto conduct annual desktop / field "earthquake drills", so that the approaches to accept and manage mutual aid become clearly defined and part of the Water Department's capabilities.

Table 7-11 shows the annualized losses, assuming that a very effective emergency response plan is implemented. In Table 7-11, we see no improvement in small or distant earthquakes, as Palo Alto's own crews would be sufficient; but great reduction in losses due to nearby large San Andreas earthquakes.

EQ	Fault	M	Annual interval probability of earthquake in the range of the listed M	Annual Loss, Median	Annual Loss, 84th
1	San Andreas Santa Cruz	6.9	0.02	\$494	\$897
2	San Andreas Peninsula	6.0	0.003	\$641	\$1,721
3	San Andreas Peninsula	6.2	0.003	\$2,882	\$5,968
4	San Andreas Peninsula	6.4	0.003	\$12,567	\$22,803
5	San Andreas Peninsula	6.6	0.0023	\$24,057	\$39,706
6	San Andreas Peninsula	6.8	0.0018	\$34,251	\$57,252
7	San Andreas Peninsula	7.0	0.0015	\$34,569	\$57,156
8	San Andreas SAN+SAP+SAS	7.2	0.0014	\$30,723	\$51,249
9	San Andreas SAN+SAP+SAS	7.4	0.0013	\$55,688	\$84,871
10	San Andreas SAN+SAP+SAS	7.5	0.0012	\$56,610	\$86,375
11	San Andreas SAN+SAP+SAS	7.7	0.001	\$63,208	\$97,001
12	San Andreas SAN+SAP+SAS	7.9	0.0008	\$61,035	\$93,612
13	San Andreas SAN+SAP+SAS	8.0	0.0006	\$62,429	\$96,369
14	Hayward N+S	7.25	0.002	\$31,905	\$54,652
15	Hayward South	6.8	0.01	\$25,633	\$45,915
16	Hayward North	6.8	0.01	\$196	\$356
17	West Napa	6.0	0.002	\$0	\$7
18	Rodgers Creek	7.0	0.006	\$81	\$147
19	Calaveras North + Central + South	7.2	0.01	\$72,613	\$118,319
20	San Gregorio	7.7	0.003	\$66,151	\$115,296
21	Mount Diablo Thrust	6.5	0.003	\$42	\$75
22	Monta Vista	6.8	0.0002	\$5,191	\$8,274
23	Greenville	7.0	0.005	\$112	\$203
24	Zayante – Vergeles	6.9	0.0002	\$4	\$7
	Total, 24 Scenario Earthquakes			\$641,082	\$1,038,233
	Total, Other faults and M			\$320,541	\$519,116
	Total, All Earthquakes			\$961,623	\$1,557,349

Table 7-11. Annualized Seismic Economic Losses (P1, With Mutual Aid)

7.4.2 Seismic SIP-1, SIP-2, SIP-3, SIP-4

Table 7-12 lists the seismic improvement plan costs, ranked in four possible levels of funding, called SIP-1, SIP-2, SIP3 and SIP-4. SIP-1 is the package with highest priority improvements / upgrades. SIP-4 is the package that includes all the potential improvements / upgrades. Each SIP is cumulative, meaning that SIP-2 includes all the SIP-1 items, SIP-3 includes all the SIP-2 items and SIP-4 includes all the SIP-3 items. The following describes the reasons for placing each element into one of the four SIP packages.

As an planning level estimate, assume that the cost for this emergency response plan is \$1,000,000, which includes about \$321,000 in the first year to develop and set up mutual aid agreements with various other agencies and contractors, including initial costs for logistic support (spare parts, hoses, etc.), and about \$30,000 per year for desktop / training exercises per year (conducted every year into the future, using 4% discount rate over 60 years, this is $\$30,000 * 22.62 = \$679,000$).

Is this level of emergency planning worthwhile? Yes. Using Table 7-11 total annualized losses of \$961,623, the NPV of losses with an enhanced emergency response plan that heavily uses mutual aid is \$22 million, while the NPV losses with no mutual aid is \$79 million, or a net benefit of \$57 million, for an investment of about \$1 million. This is clearly a very cost effective approach to reduce (but not eliminate) the economic impacts of broken water mains in earthquakes.

Assuming that the Mutual Aid effort is adopted, then the next question is which pipes might still be cost effective for pipe replacement? With \$22 million of NPV losses, and assuming no more than about 95% effectiveness for installing new pipes, then a seismic pipe replacement program might cost about \$21 million. Therefore, replacing all 21.09 miles of pipe is likely to capture some pipes that are more worthwhile, and other pipes that are less worthwhile.

By examining the water maps, it is observed that Zone 3 has a substantial risk of liquefaction, as well as very high value customers (Stanford Hospital, etc.). In Zone 3, there are 11,162 feet of pipe in VH liquefaction zones with BCR ≥ 1 .

In Zone 2, there is high economic activity, with 2,513 feet of pipe in creek crossing zones (VH), and all of this pipe (except existing HDPE) should be upgraded. These are listed in Table 7-14.

Within Zone 1, there are two primary liquefaction zones: those along San Francisquito Creek (primarily residential) and those north of Highway 101 (primarily commercial / industrial). The total Zone 1 length in VH zones is 19.51 miles, of which 16,655 feet is 12 inches and larger. Of these pipes, some in the database are for the reclaimed water pipeline (30" pipe) which is not part of the potable water system and are removed from the upgrade program. Of the remaining pipe, 7,024 feet (\$2,947,080) have the highest benefit cost ratios, and these are included in SIP-2; the remainder in SIP-4 (3,675 feet, \$2,627,560) have BCR ratios commonly in the 0.7 to 0.8 range, and thus are included in the SIP-4 replacement effort, recognizing that once the decision is made to re-build all the water pipes in a locality, it may be best to do this all at once, rather than piecemeal, reflecting practical issues like street moratoria, customer impacts during construction etc.

As relatively higher priorities, it is recommended to upgrade of those 12-inch and larger pipes in VH zones in Zone 1. The Zone 1 large diameter pipes with the higher priority are included in SIP-2; the others in SIP-3.

As outlined in Section 5.4, for the Foothills pipeline it is recommended to adopt "Options 3 and 4", costing \$738,000. This should be sufficient to reasonably assure restoring water supply in the Foothills after large earthquakes on the San Andreas fault, and is included in SIP-2.

Item	SIP-1	SIP-2	SIP-3	SIP-4
Emergency Response	\$1,000,000	\$1,000,000	\$1,000,000	\$1,000,000
Foothills Pipeline		\$738,000	\$738,000	\$738,000
Zone 1 Liq (7,024 feet SIP 2) (15,795 feet SIP 3) (66,380 feet SIP 4) (3,675 feet SIP 4)		\$2,947,080	\$2,947,080 \$4,738,500	\$2,947,080 \$4,738,500 \$16,616,160 \$2,627,560
Zone 2 Liq (2,583 feet SIP 2)		\$803,370	\$803,370	\$803,370
Zone 3 Liq (11,162 feet SIP 2)		\$3,721,020	\$3,721,020	\$3,721,020
Total	\$1,000,000	\$9,209,470	\$13,947,970	\$33,191,690
Recommended	Very High	High	Possible	Marginal

Table 7-12. Seismic Upgrades – Pipes – Priority SIP-1, SIP-2, SIP-3 and SIP-4 (excludes extensions of pipes described in Section 7.4.3)

For SIP-4, we would replace all the non-seismic water pipes in H or VH liquefaction zones (about 22 miles). This is estimated to cost about \$31.5 million, plus the Foothills pipeline upgrades, plus emergency response. While there is merit in doing this, we suggest putting the replacement for the 8 inch and smaller pipes (66,380 feet) into the lowest SIP-4, recognizing that the very long term requirement of ultimately replacing all pipes, that these 8-inch and 6-inch pipes should ultimately be replaced with seismic-designed pipe.

For SIP-3, we would upgrade all water pipes in VH and H zones, the Foothills pipelines upgrades, and emergency response, to limit the total capital cost to about \$16.6 million. This is the limit to reach a BCR of about 1. Compared with SIP-2, SIP-3 adds in 10" diameter pipes in Zone 1, plus the remaining larger diameter (12"+) water pipes in Zone 1, prioritized by those directly serving commercial zones.

Main ID	PIPE GID	Pipe Age (Yrs)	Length (feet)	MATERIAL	Diam (inch)	Liq	Zone	Replace Cost (\$)	BCR Total	WMRP Phase No
354	250387	16	247	PVC	12	VH	3 - Three	88920	1.39	
390	250508	0	10	DIP	12	VH	3 - Three	5000	1.00	
468	250752	15	202	PVC	8	VH	3 - Three	56560	1.78	
546	250997	0	131	ACP	8	VH	3 - Three	36680	2.26	
704	251442	16	291	PVC	12	VH	3 - Three	104760	1.39	
779	251646	0	375	ACP	10	VH	3 - Three	112500	1.69	

Main ID	PIPE GID	Pipe Age (Yrs)	Length (feet)	MATERIAL	Diam (inch)	Liq	Zone	Replace Cost (\$)	BCR Total	WMRP Phase No
847	251848	0	15	UNKNOWN		VH	3 - Three	5000	1.51	
849	251854	16	317	PVC	12	VH	3 - Three	114120	1.39	
943	252130	0	671	ACP	10	VH	3 - Three	201300	1.69	
1098	252575	16	346	PVC	12	VH	3 - Three	124560	1.39	
1108	252601	15	244	PVC	8	VH	3 - Three	68320	1.78	
1187	252831	0	21	PVC	12	VH	3 - Three	7560	1.39	
1389	253406	0	250	PVC	8	VH	3 - Three	70000	1.79	
1501	253705	0	12	DIP	12	VH	3 - Three	5000	1.20	
1555	253897	56	375	ACP	12	VH	3 - Three	135000	1.42	
1725	254420	16	85	PVC	12	VH	3 - Three	30600	1.39	
1773	254583	0	11	DIP	12	VH	3 - Three	5000	1.10	
1845	254777	0	14	DIP	12	VH	3 - Three	5040	1.39	
1920	254987	0	102	ACP	4	VH	3 - Three	14280	3.60	
1943	255045	16	81	PVC	12	VH	3 - Three	29160	1.39	
1992	255200	0	44	PVC	12	VH	3 - Three	15840	1.39	
1998	255225	0	21	PVC	12	VH	3 - Three	7560	1.39	
2024	255296	16	144	PVC	12	VH	3 - Three	51840	1.39	
2093	255501	16	230	PVC	12	VH	3 - Three	82800	1.39	
2117	255568	0	523	ACP	10	VH	3 - Three	156900	1.69	
2218	255863	16	153	PVC	12	VH	3 - Three	55080	1.39	
2226	255883	16	290	DIP	12	VH	3 - Three	104400	1.39	
2240	255919	0	280	ACP	8	VH	3 - Three	78400	1.80	
2285	256049	16	374	PVC	12	VH	3 - Three	134640	1.39	
2339	256217	16	287	PVC	12	VH	3 - Three	103320	1.39	
2343	256226	0	10	DIP	12	VH	3 - Three	5000	1.00	
2475	256653	16	28	DIP	8	VH	3 - Three	7840	1.79	
2539	256814	0	267	ACP	8	VH	3 - Three	74760	1.80	
2541	256821	0	794	ACP	10	VH	3 - Three	238200	1.69	
2603	257011	0	221	DIP	8	VH	3 - Three	61880	1.79	
2634	257085	16	44	DIP	8	VH	3 - Three	12320	1.79	
2818	257631	0	22	DIP	12	VH	3 - Three	7920	1.39	
3077	258005	0	61	ACP	8	VH	3 - Three	17080	1.80	
3137	258091	16	32	PVC	12	VH	3 - Three	11520	1.39	
3138	258092	16	55	PVC	12	VH	3 - Three	19800	1.39	
3190	258165	16	173	PVC	12	VH	3 - Three	62280	1.39	
3193	258168	16	70	PVC	12	VH	3 - Three	25200	1.39	
3872	259046	16	68	PVC	12	VH	3 - Three	24480	1.39	
3873	259047	16	17	PVC	12	VH	3 - Three	6120	1.39	
4056	259305	16	53	PVC	12	VH	3 - Three	19080	1.39	
4057	259306	16	61	PVC	12	VH	3 - Three	21960	1.39	
4407	259783	56	241	ACP	12	VH	3 - Three	86760	1.42	
4408	259784	56	15	ACP	12	VH	3 - Three	5400	1.42	

Main ID	PIPE GID	Pipe Age (Yrs)	Length (feet)	MATERIAL	Diam (inch)	Liq	Zone	Replace Cost (\$)	BCR Total	WMRP Phase No
4593	260057	16	394	PVC	12	VH	3 - Three	141840	1.39	
4601	260066	56	357	ACP	12	VH	3 - Three	128520	1.42	
4602	260067	56	11	ACP	12	VH	3 - Three	5000	1.13	
4772	260301	16	588	PVC	12	VH	3 - Three	211680	1.39	
4807	260382	16	971	PVC	12	VH	3 - Three	349560	1.39	
5745	263198	15	53	DIP	12	VH	3 - Three	19080	1.39	
5752	263216	14	194	DIP	12	VH	3 - Three	69840	1.39	
5753	263217	14	192	DIP	12	VH	3 - Three	69120	1.39	
5754	263218	15	24	DIP	12	VH	3 - Three	8640	1.39	
		Total	11,162					\$3,721,020		

Table 7-13. Zone 3 Pipes in SIP-2 with Benefit Cost Ratios > 1.0

Main ID	PIPE GID	Pipe Age (Yrs)	Length (feet)	MATERIAL	Diam (inch)	Liq	Zone	Replace Cost (\$)	BCR Total	WMRP Phase No
244	250086	0	127	DIP	8	VH	2 - Two	\$35,560	1.79	
1306	253164	55	746	ACP	6	VH	2 - Two	\$156,660	2.40	
1408	253450	0	2	ACP	6	VH	2 - Two	\$5,000	0.20	
1753	254504	57	315	ACP	12	VH	2 - Two	\$113,400	4.35	WMR 31
1964	255100	0	5	PVC	6	VH	2 - Two	\$5,000	0.50	
2096	255504	0	87	CIP	8	VH	2 - Two	\$24,360	1.82	
2987	257883	0	3	ACP	12	VH	2 - Two	\$5,000	0.31	
3182	258153	28	403	PVC	12	VH	2 - Two	\$145,080	1.39	WMR 1
3508	258569	0	2	CCP	18	VH	2 - Two	\$5,000	0.21	
3766	258910	0	30	ACP	6	VH	2 - Two	\$6,300	9.14	
4287	259622	0	4	ACP	12	VH	2 - Two	\$5,000	0.41	
4288	259623	0	13	ACP	12	VH	2 - Two	\$5,000	1.33	
4441	259826	28	313	PVC	12	VH	2 - Two	\$112,680	1.39	WMR 1
4816	260393	0	129	CCP	18	VH	2 - Two	\$69,660	0.95	
5820	263360	0	120	ACP	12	VH	2 - Two	\$43,200	1.42	
5844	263421	0	115	ACP	6	VH	2 - Two	\$24,150	2.40	WMR 32
5845	263422	3	8	DIP	6	VH	2 - Two	\$5,000	0.80	Well Rehab Project
5846	263423	3	13	DIP	6	VH	2 - Two	\$5,000	1.30	Well Rehab Project
5847	263425	0	16	ACP	6	VH	2 - Two	\$5,000	1.61	
5848	263426	3	3	DIP	6	VH	2 - Two	\$5,000	0.30	Well Rehab Project
5850	263428	3	11	DIP	8	VH	2 - Two	\$5,000	1.10	Well Rehab Project
5852	263431	3	44	DIP	8	VH	2 - Two	\$12,320	1.79	Well Rehab Project
5991	263741	10	4	PVC	6	VH	2 - Two	\$5,000	0.40	WMR 17
	Total		2,533					\$803,370		

Table 7-14. Zone 2 Non-Seismic Pipes in SIP-2 with Benefit Cost Ratios > 1.0 or located in VH Zones

Main ID	PIPE GID	Pipe Age (Yrs)	Length (feet)	MATERIAL	Diam (inch)	Liq	Zone	Replace Cost (\$)	BCR Total	WMRP Phase No
403	250528	0	361	UNKNOWN	12	VH	1 - One	129,960	1.43	WMR 33
1056	252455	0	720	ACP	12	VH	1 - One	259,200	1.41	
1146	252715	0	294	ACP	12	VH	1 - One	105,840	1.41	WMR 33
2789	257547	0	377	ACP	12	VH	1 - One	135,720	1.41	
4751	260267	48	1971	CCP	16	VH	1 - One	946,080	1.05	
4755	260274	48	99	ACP	16	VH	1 - One	47,520	1.08	
4756	260275	0	414	ACP	12	VH	1 - One	149,040	1.41	WMR 33
4987	261102	32	455	ACP	12	VH	1 - One	163,800	1.40	
5309	262127	0	153	ACP	12	VH	1 - One	55,080	1.41	
7155	266446	0	72	ACP	12	VH	1 - One	25,920	1.41	
7186	266531	0	1348	CCP	16	VH	1 - One	647,040	1.05	
7211	266556	0	11	CCP	16	VH	1 - One	5,280	1.05	
7212	266557	0	30	CCP	16	VH	1 - One	14,400	1.05	
7213	266558	0	28	CCP	16	VH	1 - One	13,440	1.05	
7227	266588	0	95	PVC	12	VH	1 - One	34,200	1.39	GMR 19B-20-21
7228	266589	0	596	PVC	12	VH	1 - One	214,560	1.39	GMR 19B-20-21
	Total		7,024					\$2,947,080		

Table 7-15. Zone 1 Pipes in SIP-2 (12-Inch and larger with BCR ≥ 1)

Main ID	PIPE GID	Pipe Age (Yrs)	Length (feet)	MATERIAL	Diam (inch)	Liq	Zone	Replace Cost (\$)	BCR Total	WMRP Phase No
15	249844	0	1761	ACP	10	VH	1 - One	528,300	1.69	
1608	254040	11	579	DIP	10	VH	1 - One	173,700	1.67	
2065	255410	48	522	ACP	10	VH	1 - One	156,600	1.69	
2315	256139	0	681	ACP	10	VH	1 - One	204,300	1.78	
2356	256275	11	368	DIP	10	VH	1 - One	110,400	1.67	
2462	256609	48	295	ACP	10	VH	1 - One	88,500	1.69	
2672	257210	0	399	ACP	10	VH	1 - One	119,700	1.69	
2947	257839	0	245	ACP	10	VH	1 - One	73,500	1.69	
2948	257840	0	383	ACP	10	VH	1 - One	114,900	1.69	
3123	258074	48	17	ACP	10	VH	1 - One	5,100	1.69	
3588	258661	0	76	ACP	10	VH	1 - One	22,800	1.69	
3960	259166	54	150	ACP	10	VH	1 - One	45,000	1.69	
4221	259536	0	17	ACP	10	VH	1 - One	5,100	1.69	
4306	259645	0	374	ACP	10	VH	1 - One	112,200	1.69	
4558	259995	0	222	ACP	10	VH	1 - One	66,600	1.69	
4752	260271	0	1169	ACP	10	VH	1 - One	350,700	1.69	
4757	260276	0	694	ACP	10	VH	1 - One	208,200	1.69	
5961	263678	11	32	DIP	10	VH	1 - One	9,600	1.67	
7184	266529	54	2594	ACP	10	VH	1 - One	778,200	1.69	
7185	266530	55	2349	ACP	10	VH	1 - One	704,700	1.69	
7202	266547	54	516	ACP	10	VH	1 - One	154,800	1.69	
7207	266552	0	606	ACP	10	VH	1 - One	181,800	3.01	WMR 29
7208	266553	0	119	ACP	10	VH	1 - One	35,700	1.69	WMR 29
7210	266555	55	94	ACP	10	VH	1 - One	28,200	1.69	
7221	266582	0	1533	PVC	10	VH	1 - One	459,900	1.67	GMR 19B-20- 21
137	249690	0	5	ACP	12	VH	1 - One	\$5,000	0.508	
7191	266536	0	1249	DIP	24	VH	1 - One	\$899,280	0.727	
7194	266539	0	760	DIP	24	VH	1 - One	\$547,200	0.727	
7195	266540	0	872	DIP	24	VH	1 - One	\$627,840	0.727	
7196	266541	0	123	DIP	24	VH	1 - One	\$88,560	0.727	
7197	266542	0	189	DIP	20	VH	1 - One	\$113,400	0.852	
7201	266546	0	474	DIP	24	VH	1 - One	\$341,280	0.727	
7229	266590	0	3	PVC	12	VH	1 - One	\$5,000	0.3	GMR 19B-20- 21
Total	BCR ≥ 1		15,795					\$4,738,500		SIP-3
Total	BCR < 1		3,675					\$2,627,560		SIP-4

Table 7-16. Zone 1 Pipes in SIP-3 and SIP-4

Table 7-17 lists all the pipes in Zone 1 with diameter 8 inches or smaller, in VH liquefaction zones, excluding pipes already replaced with HDPE pipe. Several short segments (commonly under 10 feet) are listed with $BCR < 1$. The actual BCR for these very short segments would be >1 if the work is done in conjunction with replacement of adjacent pipes, and not as a stand-alone project; therefore, they are included in Table 7-17. During the course of final design, if there are pipes in Table 7-17 serving commercial, government or industrial customers, upgrade of these pipes would have higher economic impact and could be prioritized along with SIP-2 or SIP-3; those pipes serving residential customers are SIP-4.

Main ID	Pipe GID	Pipe Age (Yrs)	Length (Feet)	MATERIAL	Diam (inch)	Zone	BCR Total	Replace Cost (\$)
30	260707	0	132	CIP	6	1 - One	2.40	\$27,720
32	260711	0	3	CIP	6	1 - One	0.30	\$5,000
33	260712	0	3	CIP	6	1 - One	0.30	\$5,000
34	260713	0	84	ACP	8	1 - One	1.80	\$23,520
36	260715	26	4	PVC	8	1 - One	0.40	\$5,000
37	260716	0	109	CIP	6	1 - One	2.40	\$22,890
38	260717	0	14	CIP	6	1 - One	1.41	\$5,000
39	260718	17	5	PVC	6	1 - One	0.50	\$5,000
40	260721	0	240	CIP	6	1 - One	2.40	\$50,400
41	260722	0	11	CIP	6	1 - One	1.11	\$5,000
42	260723	17	4	PVC	6	1 - One	0.40	\$5,000
43	260724	17	4	PVC	6	1 - One	0.40	\$5,000
44	260725	0	801	CIP	6	1 - One	2.44	\$168,210
47	260728	0	253	CIP	6	1 - One	2.40	\$53,130
49	260731	0	253	CIP	6	1 - One	2.40	\$53,130
50	260732	0	4	CIP	8	1 - One	0.41	\$5,000
51	260733	0	40	CIP	8	1 - One	1.81	\$11,200
52	260734	0	40	CIP	6	1 - One	2.40	\$8,400
53	260735	0	21	CIP	6	1 - One	2.12	\$5,000
54	260736	50	244	CIP	6	1 - One	2.40	\$51,240
55	260737	50	256	CIP	6	1 - One	2.56	\$53,760
56	260740	39	390	ACP	6	1 - One	2.39	\$81,900
57	260743	0	248	CIP	6	1 - One	2.40	\$52,080
63	249967	46	694	ACP	8	1 - One	1.80	\$194,320
79	250548	0	2	PVC	6	1 - One	0.20	\$5,000
116	249622	39	531	ACP	8	1 - One	1.80	\$148,680
118	249628	53	768	ACP	6	1 - One	2.40	\$161,280
148	249739	0	3	ACP	8	1 - One	0.30	\$5,000
193	249892	0	633	ACP	6	1 - One	2.64	\$132,930
195	249897	41	3	ACP	8	1 - One	0.30	\$5,000
205	249923	0	673	ACP	8	1 - One	1.80	\$188,440
206	249926	0	10	ACP	4	1 - One	1.01	\$5,000
234	250064	0	10	ACP	8	1 - One	1.01	\$5,000
242	250078	0	14	CIP	8	1 - One	1.42	\$5,000
254	250120	39	350	ACP	6	1 - One	2.39	\$73,500
260	250132	16	481	PVC	6	1 - One	2.38	\$101,010
263	250147	0	2	ACP	8	1 - One	0.20	\$5,000
273	250177	8	656	PVC	8	1 - One	1.79	\$183,680
280	250190	47	152	DIP	6	1 - One	2.38	\$31,920
307	250258	0	122	DIP	8	1 - One	1.79	\$34,160
311	250269	0	7	ACP	6	1 - One	0.71	\$5,000

Main ID	Pipe GID	Pipe Age (Yrs)	Length (Feet)	MATERIAL	Diam (inch)	Zone	BCR Total	Replace Cost (\$)
377	250461	41	2	ACP	8	1 - One	0.20	\$5,000
386	250501	0	22	CIP	6	1 - One	2.22	\$5,000
393	250511	19	12	PVC	6	1 - One	1.20	\$5,000
398	250521	0	145	ACP	6	1 - One	2.70	\$30,450
456	250703	0	62	ACP	8	1 - One	1.80	\$17,360
492	250820	39	27	ACP	8	1 - One	1.80	\$7,560
494	250823	16	27	PVC	6	1 - One	2.38	\$5,670
544	250989	57	20	ACP	8	1 - One	1.80	\$5,600
545	250994	39	674	ACP	6	1 - One	2.39	\$141,540
576	251092	19	393	PVC	6	1 - One	2.38	\$82,530
590	251132	0	1	ACP	6	1 - One	0.10	\$5,000
591	251134	0	178	ACP	6	1 - One	2.40	\$37,380
648	251297	17	3	PVC	6	1 - One	0.30	\$5,000
668	251350	0	29	CIP	6	1 - One	2.40	\$6,090
671	251362	66	42	ACP	6	1 - One	2.40	\$8,820
690	251418	16	639	PVC	6	1 - One	2.38	\$134,190
706	251448	0	140	CIP	6	1 - One	2.40	\$29,400
717	251468	0	41	ACP	8	1 - One	1.80	\$11,480
749	251578	0	2	ACP	8	1 - One	0.20	\$5,000
775	251633	39	2	ACP	6	1 - One	0.20	\$5,000
802	251728	16	0	PVC	6	1 - One	0.00	\$5,000
814	251758	39	319	ACP	6	1 - One	2.39	\$66,990
823	251772	39	247	ACP	8	1 - One	1.80	\$69,160
856	251890	39	1	ACP	6	1 - One	0.10	\$5,000
867	251922	17	501	PVC	6	1 - One	2.38	\$105,210
871	251930	0	6	ACP	6	1 - One	0.60	\$5,000
913	252038	0	28	CIP	6	1 - One	2.40	\$5,880
924	252071	19	165	PVC	6	1 - One	2.38	\$34,650
930	252083	0	6	ACP	8	1 - One	0.61	\$5,000
940	252114	0	362	ACP	8	1 - One	1.80	\$101,360
959	252166	0	417	ACP	2	1 - One	7.19	\$29,190
973	252212	65	38	ACP	6	1 - One	2.40	\$7,980
988	252255	41	2	ACP	8	1 - One	0.20	\$5,000
1033	252374	56	59	ACP	8	1 - One	1.80	\$16,520
1073	252510	60	1100	ACP	6	1 - One	2.40	\$231,000
1076	252526	0	14	ACP	6	1 - One	1.41	\$5,000
1077	252529	39	32	ACP	8	1 - One	1.80	\$8,960
1085	252543	0	1	ACP	6	1 - One	0.10	\$5,000
1093	252561	17	553	PVC	6	1 - One	2.38	\$116,130
1111	252609	0	52	ACP	6	1 - One	2.40	\$10,920
1112	252612	0	1	ACP	6	1 - One	0.10	\$5,000
1119	252637	0	2	CIP	6	1 - One	0.20	\$5,000
1154	252738	0	90	Steel	8	1 - One	1.78	\$25,200

Main ID	Pipe GID	Pipe Age (Yrs)	Length (Feet)	MATERIAL	Diam (inch)	Zone	BCR Total	Replace Cost (\$)
1198	252863	19	40	PVC	6	1 - One	2.38	\$8,400
1201	252873	17	1	PVC	6	1 - One	0.10	\$5,000
1267	253068	17	37	PVC	6	1 - One	2.38	\$7,770
1293	253132	40	322	ACP	8	1 - One	1.80	\$90,160
1297	253145	19	32	PVC	6	1 - One	2.38	\$6,720
1300	253153	39	24	ACP	6	1 - One	2.39	\$5,040
1308	253172	0	5	CIP	4	1 - One	0.50	\$5,000
1360	253304	0	21	ACP	6	1 - One	2.11	\$5,000
1362	253313	0	30	ACP	6	1 - One	2.40	\$6,300
1372	253345	71	260	ACP	6	1 - One	2.40	\$54,600
1405	253445	0	43	ACP	8	1 - One	1.80	\$12,040
1454	253585	0	65	CIP	6	1 - One	2.40	\$13,650
1471	253628	19	666	PVC	6	1 - One	2.38	\$139,860
1564	253924	0	4	ACP	6	1 - One	0.40	\$5,000
1584	253977	49	463	ACP	8	1 - One	1.80	\$129,640
1624	254078	17	49	PVC	6	1 - One	2.38	\$10,290
1717	254392	0	16	ACP	8	1 - One	1.61	\$5,000
1758	254518	21	72	DIP	8	1 - One	1.79	\$20,160
1761	254531	16	235	PVC	6	1 - One	2.38	\$49,350
1768	254556	0	50	ACP	8	1 - One	1.80	\$14,000
1784	254610	45	344	ACP	6	1 - One	2.40	\$72,240
1798	254637	16	1046	PVC	6	1 - One	2.38	\$219,660
1811	254676	0	273	ACP	8	1 - One	2.40	\$76,440
1834	254743	17	2	PVC	6	1 - One	0.20	\$5,000
1844	254775	0	33	CIP	6	1 - One	2.40	\$6,930
1856	254803	0	5	ACP	4	1 - One	0.50	\$5,000
1887	254907	0	2	ACP	6	1 - One	0.20	\$5,000
1922	254993	0	26	ACP	8	1 - One	1.80	\$7,280
1932	255022	41	2	ACP	6	1 - One	0.20	\$5,000
1935	255031	0	21	ACP	6	1 - One	2.11	\$5,000
1937	255035	0	36	ACP	6	1 - One	2.40	\$7,560
1956	255072	0	2	PVC	8	1 - One	0.20	\$5,000
1961	255086	19	139	PVC	8	1 - One	1.79	\$38,920
1974	255140	39	1	ACP	6	1 - One	0.10	\$5,000
1984	255181	39	10	ACP	6	1 - One	1.01	\$5,000
2055	255386	17	32	PVC	6	1 - One	2.38	\$6,720
2111	255547	65	914	ACP	6	1 - One	2.40	\$191,940
2201	255807	0	2	ACP	8	1 - One	0.20	\$5,000
2278	256032	0	12	ACP	8	1 - One	1.21	\$5,000
2308	256126	17	2	PVC	6	1 - One	0.20	\$5,000
2312	256134	0	521	ACP	8	1 - One	1.80	\$145,880
2316	256140	0	13	CIP	4	1 - One	1.31	\$5,000
2323	256161	45	337	ACP	6	1 - One	2.52	\$70,770

Main ID	Pipe GID	Pipe Age (Yrs)	Length (Feet)	MATERIAL	Diam (inch)	Zone	BCR Total	Replace Cost (\$)
2347	256248	0	3	ACP	8	1 - One	0.30	\$5,000
2349	256254	58	143	CIP	8	1 - One	1.81	\$40,040
2410	256457	17	1	PVC	6	1 - One	0.10	\$5,000
2425	256500	0	3	ACP	6	1 - One	0.30	\$5,000
2426	256501	0	2	ACP	8	1 - One	0.20	\$5,000
2440	256543	0	568	PVC	8	1 - One	1.79	\$159,040
2445	256559	0	71	ACP	6	1 - One	2.40	\$14,910
2451	256575	0	32	ACP	8	1 - One	1.80	\$8,960
2464	256616	16	3	PVC	6	1 - One	0.30	\$5,000
2472	256635	8	733	PVC	8	1 - One	1.79	\$205,240
2486	256681	0	4	ACP	8	1 - One	0.40	\$5,000
2499	256707	38	19	ACP	8	1 - One	1.80	\$5,320
2510	256732	19	25	PVC	6	1 - One	2.38	\$5,250
2545	256830	65	37	ACP	6	1 - One	2.40	\$7,770
2549	256848	0	430	ACP	6	1 - One	2.40	\$90,300
2553	256857	17	467	PVC	6	1 - One	2.38	\$98,070
2554	256859	41	3	ACP	8	1 - One	0.30	\$5,000
2558	256869	66	40	ACP	6	1 - One	2.40	\$8,400
2593	256973	0	18	CIP	8	1 - One	1.81	\$5,040
2641	257111	0	22	ACP	8	1 - One	1.80	\$6,160
2719	257367	0	73	ACP	8	1 - One	1.80	\$20,440
2731	257406	21	325	PVC	6	1 - One	2.38	\$68,250
2737	257416	39	300	ACP	8	1 - One	1.80	\$84,000
2769	257514	0	1003	ACP	6	1 - One	2.40	\$210,630
2770	257519	17	560	PVC	6	1 - One	2.38	\$117,600
2775	257525	17	1140	PVC	6	1 - One	2.38	\$239,400
2838	257675	16	31	PVC	6	1 - One	2.38	\$6,510
2843	257687	64	824	ACP	6	1 - One	2.40	\$173,040
2898	257767	16	13	PVC	6	1 - One	1.30	\$5,000
2899	257768	16	15	PVC	6	1 - One	1.50	\$5,000
2901	257770	41	135	ACP	8	1 - One	1.80	\$37,800
2904	257778	0	228	ACP	4	1 - One	3.60	\$31,920
2905	257779	0	78	ACP	4	1 - One	3.60	\$10,920
2936	257821	0	45	ACP	6	1 - One	2.40	\$9,450
3003	257902	0	312	ACP	8	1 - One	1.80	\$87,360
3022	257927	0	4	CIP	4	1 - One	0.40	\$5,000
3023	257928	0	8	CIP	4	1 - One	0.80	\$5,000
3032	257940	0	431	ACP	8	1 - One	1.80	\$120,680
3037	257946	19	168	PVC	6	1 - One	2.38	\$35,280
3039	257948	0	65	ACP	6	1 - One	2.40	\$13,650
3040	257949	0	16	ACP	6	1 - One	1.61	\$5,000
3046	257956	49	510	ACP	8	1 - One	1.80	\$142,800
3047	257957	49	19	ACP	8	1 - One	1.80	\$5,320

Main ID	Pipe GID	Pipe Age (Yrs)	Length (Feet)	MATERIAL	Diam (inch)	Zone	BCR Total	Replace Cost (\$)
3091	258026	39	28	ACP	8	1 - One	1.80	\$7,840
3102	258041	41	19	ACP	8	1 - One	1.80	\$5,320
3160	258121	41	17	ACP	8	1 - One	1.71	\$5,000
3161	258122	41	546	ACP	8	1 - One	1.80	\$152,880
3196	258173	16	17	PVC	6	1 - One	1.70	\$5,000
3217	258204	0	30	CIP	6	1 - One	2.40	\$6,300
3218	258205	0	285	CIP	6	1 - One	2.40	\$59,850
3240	258231	17	34	PVC	6	1 - One	2.38	\$7,140
3241	258232	17	270	PVC	6	1 - One	2.38	\$56,700
3277	258282	7	563	ACP	6	1 - One	2.39	\$118,230
3278	258283	7	187	ACP	6	1 - One	2.39	\$39,270
3313	258327	0	13	ACP	8	1 - One	1.31	\$5,000
3314	258328	0	18	ACP	8	1 - One	1.80	\$5,040
3372	258405	17	44	PVC	6	1 - One	2.38	\$9,240
3385	258420	45	35	ACP	6	1 - One	2.40	\$7,350
3386	258421	0	4	ACP	8	1 - One	0.40	\$5,000
3513	258574	0	56	PVC	8	1 - One	1.79	\$15,680
3545	258612	7	152	ACP	8	1 - One	1.79	\$42,560
3546	258613	7	410	ACP	8	1 - One	1.79	\$114,800
3569	258640	41	4	ACP	8	1 - One	0.40	\$5,000
3587	258660	0	4	ACP	8	1 - One	0.40	\$5,000
3597	258673	17	22	PVC	6	1 - One	2.20	\$5,000
3598	258674	17	174	PVC	6	1 - One	2.38	\$36,540
3629	258719	0	239	CIP	6	1 - One	2.40	\$50,190
3640	258736	51	461	ACP	8	1 - One	1.80	\$129,080
3679	258790	19	40	PVC	6	1 - One	2.38	\$8,400
3684	258795	0	393	ACP	8	1 - One	1.80	\$110,040
3685	258796	0	20	ACP	8	1 - One	1.80	\$5,600
3687	258799	49	5	ACP	8	1 - One	0.50	\$5,000
3688	258800	49	26	ACP	8	1 - One	1.80	\$7,280
3696	258812	32	4	ACP	6	1 - One	0.40	\$5,000
3697	258813	32	505	ACP	6	1 - One	2.39	\$106,050
3726	258850	0	118	DIP	6	1 - One	2.38	\$24,780
3727	258851	0	9	DIP	6	1 - One	0.90	\$5,000
3729	258853	71	157	ACP	6	1 - One	2.40	\$32,970
3736	258860	0	34	CIP	6	1 - One	2.40	\$7,140
3738	258864	39	32	ACP	6	1 - One	2.39	\$6,720
3739	258866	17	15	PVC	6	1 - One	1.50	\$5,000
3740	258867	17	18	PVC	6	1 - One	1.80	\$5,000
3747	258887	0	39	ACP	6	1 - One	2.40	\$8,190
3783	258930	0	227	ACP	8	1 - One	1.80	\$63,560
3784	258931	0	307	ACP	8	1 - One	1.80	\$85,960
3906	259093	71	201	ACP	6	1 - One	2.40	\$42,210

Main ID	Pipe GID	Pipe Age (Yrs)	Length (Feet)	MATERIAL	Diam (inch)	Zone	BCR Total	Replace Cost (\$)
3907	259094	71	371	ACP	6	1 - One	2.40	\$77,910
3908	259095	39	19	ACP	6	1 - One	1.91	\$5,000
3909	259096	39	11	ACP	6	1 - One	1.11	\$5,000
3954	259160	0	528	ACP	6	1 - One	2.40	\$110,880
3955	259161	0	20	ACP	6	1 - One	2.01	\$5,000
3965	259172	16	260	PVC	6	1 - One	2.38	\$54,600
3966	259173	16	145	PVC	6	1 - One	2.38	\$30,450
4004	259220	16	8	PVC	6	1 - One	0.80	\$5,000
4005	259221	16	72	PVC	6	1 - One	2.38	\$15,120
4006	259226	51	501	ACP	8	1 - One	1.80	\$140,280
4051	259296	55	844	ACP	8	1 - One	1.80	\$236,320
4052	259297	55	396	ACP	8	1 - One	1.80	\$110,880
4059	259311	0	16	ACP	6	1 - One	1.61	\$5,000
4060	259312	67	699	ACP	6	1 - One	2.40	\$146,790
4095	259358	39	131	ACP	6	1 - One	2.39	\$27,510
4131	259406	0	502	CIP	6	1 - One	2.40	\$105,420
4132	259407	0	2	CIP	6	1 - One	0.20	\$5,000
4215	259529	0	363	ACP	8	1 - One	1.80	\$101,640
4249	259570	39	4	ACP	8	1 - One	0.40	\$5,000
4250	259571	39	29	ACP	6	1 - One	2.39	\$6,090
4361	259720	62	358	ACP	6	1 - One	2.40	\$75,180
4397	259771	19	49	PVC	6	1 - One	2.38	\$10,290
4398	259772	19	236	PVC	6	1 - One	2.38	\$49,560
4425	259804	19	12	PVC	6	1 - One	1.20	\$5,000
4426	259805	19	16	PVC	6	1 - One	1.60	\$5,000
4485	259890	41	42	ACP	8	1 - One	1.80	\$11,760
4486	259891	41	9	ACP	8	1 - One	0.91	\$5,000
4513	259933	65	294	ACP	6	1 - One	2.40	\$61,740
4514	259934	65	243	ACP	6	1 - One	2.40	\$51,030
4529	259956	50	3	CIP	6	1 - One	0.30	\$5,000
4559	259997	16	4	PVC	6	1 - One	0.40	\$5,000
4574	260021	51	230	ACP	8	1 - One	1.80	\$64,400
4608	260078	66	283	ACP	6	1 - One	2.40	\$59,430
4732	260244	39	139	ACP	6	1 - One	2.39	\$29,190
4733	260245	39	4	ACP	8	1 - One	0.40	\$5,000
4767	260289	69	531	ACP	6	1 - One	2.40	\$111,510
4771	260300	45	410	ACP	6	1 - One	2.40	\$86,100
4776	260306	66	258	ACP	6	1 - One	2.40	\$54,180
4844	260438	19	771	PVC	6	1 - One	2.38	\$161,910
4845	260439	19	4	PVC	6	1 - One	0.40	\$5,000
4846	260440	19	9	PVC	6	1 - One	0.90	\$5,000
4882	260501		1	DIP	6	1 - One	0.10	\$5,000
4884	260505	16	384	PVC	6	1 - One	2.38	\$80,640

Main ID	Pipe GID	Pipe Age (Yrs)	Length (Feet)	MATERIAL	Diam (inch)	Zone	BCR Total	Replace Cost (\$)
4885	260506	16	31	PVC	6	1 - One	2.38	\$6,510
4931	260744	39	16	ACP	6	1 - One	1.61	\$5,000
4932	260745	39	34	ACP	6	1 - One	2.39	\$7,140
4933	260746	0	8	CIP	6	1 - One	0.81	\$5,000
4934	260747	0	3	CIP	6	1 - One	0.30	\$5,000
4935	260748	0	241	CIP	6	1 - One	2.40	\$50,610
4936	260749	39	8	ACP	6	1 - One	0.80	\$5,000
4984	261098	0	37	CIP	6	1 - One	2.40	\$7,770
4985	261099	0	9	CIP	6	1 - One	0.91	\$5,000
4986	261100	32	9	ACP	6	1 - One	0.90	\$5,000
4988	261104	0	4	CIP	6	1 - One	0.40	\$5,000
4989	261105	32	7	ACP	6	1 - One	0.70	\$5,000
4990	261106	0	19	ACP	6	1 - One	1.91	\$5,000
4991	261107	0	2	CIP	6	1 - One	0.20	\$5,000
4994	261110	0	217	CIP	6	1 - One	3.13	\$45,570
5022	261263	0	399	ACP	6	1 - One	2.40	\$83,790
5023	261264	0	7	ACP	6	1 - One	0.71	\$5,000
5025	261266		1	DIP	6	1 - One	0.10	\$5,000
5221	261913	0	3	CIP	6	1 - One	0.30	\$5,000
5222	261915	0	51	CIP	6	1 - One	2.40	\$10,710
5255	262063	7	1	PVC	8	1 - One	0.10	\$5,000
5256	262064	7	99	PVC	8	1 - One	1.79	\$27,720
5257	262065	7	1	PVC	8	1 - One	0.10	\$5,000
5258	262066	7	1	PVC	8	1 - One	0.10	\$5,000
5259	262067	7	220	PVC	8	1 - One	1.79	\$61,600
5260	262068	7	1	PVC	8	1 - One	0.10	\$5,000
5261	262069	7	1	PVC	8	1 - One	0.10	\$5,000
5262	262070	7	199	PVC	8	1 - One	1.79	\$55,720
5263	262071	7	1	PVC	8	1 - One	0.10	\$5,000
5264	262072	7	114	PVC	8	1 - One	1.79	\$31,920
5265	262073	7	2	PVC	8	1 - One	0.20	\$5,000
5266	262074	7	160	PVC	8	1 - One	1.79	\$44,800
5267	262075	7	1	PVC	8	1 - One	0.10	\$5,000
5268	262076	7	109	PVC	8	1 - One	1.79	\$30,520
5269	262078	7	1	PVC	8	1 - One	0.10	\$5,000
5270	262079	7	1	PVC	8	1 - One	0.10	\$5,000
5271	262080	7	215	PVC	8	1 - One	1.79	\$60,200
5272	262081	7	17	PVC	8	1 - One	1.70	\$5,000
5273	262082	7	1	PVC	8	1 - One	0.10	\$5,000
5274	262083	7	1	PVC	8	1 - One	0.10	\$5,000
5275	262084	7	128	PVC	8	1 - One	1.79	\$35,840
5276	262085	7	56	PVC	8	1 - One	1.79	\$15,680
5277	262086	7	1	PVC	8	1 - One	0.10	\$5,000

Main ID	Pipe GID	Pipe Age (Yrs)	Length (Feet)	MATERIAL	Diam (inch)	Zone	BCR Total	Replace Cost (\$)
5278	262087	7	1	PVC	8	1 - One	0.10	\$5,000
5280	262089	7	170	PVC	8	1 - One	1.79	\$47,600
5281	262090	7	75	PVC	8	1 - One	1.79	\$21,000
5282	262091	7	184	PVC	8	1 - One	1.79	\$51,520
5283	262092	7	72	PVC	8	1 - One	1.79	\$20,160
5284	262093	7	108	PVC	8	1 - One	1.79	\$30,240
5285	262094	7	1	PVC	8	1 - One	0.10	\$5,000
5286	262095	7	2	PVC	8	1 - One	0.20	\$5,000
5287	262096	7	21	PVC	8	1 - One	1.79	\$5,880
5288	262097	7	57	PVC	8	1 - One	1.79	\$15,960
5289	262098	7	1	PVC	8	1 - One	0.10	\$5,000
5290	262099	7	1	PVC	8	1 - One	0.10	\$5,000
5291	262100	0	1	ACP	8	1 - One	0.10	\$5,000
5292	262101	0	48	ACP	8	1 - One	1.80	\$13,440
5293	262102	0	147	ACP	8	1 - One	1.80	\$41,160
5294	262103	7	1	PVC	8	1 - One	0.10	\$5,000
5295	262104	7	1	PVC	8	1 - One	0.10	\$5,000
5296	262105	7	41	PVC	8	1 - One	1.79	\$11,480
5297	262106	7	122	PVC	8	1 - One	1.79	\$34,160
5298	262107	7	68	PVC	8	1 - One	1.79	\$19,040
5299	262108	7	9	PVC	8	1 - One	0.90	\$5,000
5301	262110	56	255	ACP	8	1 - One	1.80	\$71,400
5829	263384	0	528	CIP	8	1 - One	1.80	\$147,840
5830	263385	0	48	CIP	8	1 - One	1.80	\$13,440
5831	263386	4	2	PVC	8	1 - One	0.20	\$5,000
5832	263387	4	2	PVC	8	1 - One	0.20	\$5,000
5836	263393	0	284	ACP	8	1 - One	1.80	\$79,520
5837	263394	0	130	ACP	8	1 - One	3.08	\$36,400
5838	263395	3	5	PVC	8	1 - One	0.50	\$5,000
5854	263437	0	11	ACP	8	1 - One	1.11	\$5,000
5856	263439	0	2	ACP	6	1 - One	0.20	\$5,000
5866	263465	0	14	ACP	6	1 - One	1.41	\$5,000
5929	263592	0	567	ACP	8	1 - One	1.80	\$158,760
5930	263593	0	215	ACP	8	1 - One	1.80	\$60,200
5962	263679	11	18	DIP	6	1 - One	1.80	\$5,000
5963	263680	0	2	ACP	8	1 - One	0.20	\$5,000
6079	263957	0	4	CIP	6	1 - One	0.40	\$5,000
6080	263958	0	3	CIP	6	1 - One	0.30	\$5,000
6081	263959	0	20	CIP	6	1 - One	2.02	\$5,000
6231	264364	16	37	PVC	6	1 - One	2.38	\$7,770
6232	264365	16	18	PVC	6	1 - One	1.80	\$5,000
6291	264483	41	2	ACP	8	1 - One	0.20	\$5,000
6292	264484	41	0	ACP	8	1 - One	0.00	\$5,000

Main ID	Pipe GID	Pipe Age (Yrs)	Length (Feet)	MATERIAL	Diam (inch)	Zone	BCR Total	Replace Cost (\$)
6293	264485	17	33	PVC	6	1 - One	2.38	\$6,930
6325	264568	17	25	PVC	6	1 - One	2.38	\$5,250
6326	264569	17	300	PVC	6	1 - One	2.38	\$63,000
6327	264570	17	32	PVC	6	1 - One	2.38	\$6,720
6328	264571	17	6	PVC	6	1 - One	0.60	\$5,000
6329	264572	17	6	PVC	6	1 - One	0.60	\$5,000
6330	264573	17	16	PVC	6	1 - One	1.60	\$5,000
6360	264681	38	706	ACP	6	1 - One	2.44	\$148,260
6361	264682	0	39	CIP	6	1 - One	2.40	\$8,190
6362	264683	16	10	PVC	6	1 - One	1.00	\$5,000
6363	264684	16	10	PVC	6	1 - One	1.00	\$5,000
6364	264685	16	14	PVC	6	1 - One	1.40	\$5,000
6365	264686	16	2	PVC	6	1 - One	0.20	\$5,000
6366	264687	16	5	PVC	6	1 - One	0.50	\$5,000
6367	264688	16	2	PVC	6	1 - One	0.20	\$5,000
6368	264689	16	2	PVC	6	1 - One	0.20	\$5,000
6395	264762	26	343	CIP	6	1 - One	2.40	\$72,030
6398	264765	14	340	PVC	8	1 - One	1.79	\$95,200
6399	264766	14	39	PVC	8	1 - One	1.79	\$10,920
6400	264767	14	336	PVC	8	1 - One	1.79	\$94,080
6401	264768	14	22	PVC	8	1 - One	1.79	\$6,160
6404	264771	14	3	PVC	8	1 - One	0.30	\$5,000
6408	264775	14	5	PVC	6	1 - One	0.50	\$5,000
6409	264776	14	2	PVC	6	1 - One	0.20	\$5,000
6410	264777	16	29	PVC	6	1 - One	2.38	\$6,090
6411	264778	14	4	PVC	6	1 - One	0.40	\$5,000
6412	264779	14	5	PVC	8	1 - One	0.50	\$5,000
6444	264853	19	29	PVC	6	1 - One	2.38	\$6,090
6445	264854	19	7	PVC	6	1 - One	0.70	\$5,000
6446	264855	19	10	PVC	8	1 - One	1.00	\$5,000
6447	264862	0	36	CIP	6	1 - One	2.40	\$7,560
6448	264863	17	4	PVC	6	1 - One	0.40	\$5,000
6449	264864	0	19	CIP	6	1 - One	1.91	\$5,000
6538	265072	19	42	PVC	6	1 - One	2.38	\$8,820
6539	265073	49	393	ACP	6	1 - One	2.50	\$82,530
6540	265074	19	216	PVC	6	1 - One	2.38	\$45,360
6543	265077	21	30	PVC	6	1 - One	2.38	\$6,300
6544	265078	19	33	PVC	6	1 - One	2.38	\$6,930
6545	265079	19	9	PVC	6	1 - One	0.90	\$5,000
6647	265322	65	339	ACP	6	1 - One	2.40	\$71,190
6693	265455	19	6	ACP	8	1 - One	0.60	\$5,000
6695	265457	20	38	ACP	8	1 - One	1.79	\$10,640
6696	265458	20	25	ACP	8	1 - One	1.79	\$7,000

Main ID	Pipe GID	Pipe Age (Yrs)	Length (Feet)	MATERIAL	Diam (inch)	Zone	BCR Total	Replace Cost (\$)
6770	265624	51	598	ACP	8	1 - One	1.80	\$167,440
6771	265625	51	28	ACP	8	1 - One	1.80	\$7,840
6936	265970	39	248	ACP	8	1 - One	1.80	\$69,440
6937	265971	39	33	ACP	8	1 - One	1.80	\$9,240
6938	265972	39	34	ACP	6	1 - One	2.39	\$7,140
6939	265973	39	26	ACP	6	1 - One	2.39	\$5,460
6940	265978	0	521	CIP	6	1 - One	2.47	\$109,410
7062	266239	41	354	ACP	8	1 - One	1.80	\$99,120
7063	266240	41	190	ACP	8	1 - One	1.80	\$53,200
7172	266474	55	1003	ACP	8	1 - One	1.80	\$280,840
7173	266475	55	257	ACP	8	1 - One	1.80	\$71,960
7174	266476	55	25	ACP	8	1 - One	1.80	\$7,000
7175	266477	55	760	ACP	8	1 - One	1.80	\$212,800
7183	266528	0	554	ACP	8	1 - One	1.80	\$155,120
7187	266532	0	161	ACP	8	1 - One	1.80	\$45,080
7188	266533	54	225	ACP	8	1 - One	1.80	\$63,000
7189	266534	54	217	ACP	8	1 - One	1.80	\$60,760
7193	266538	0	10	ACP	8	1 - One	1.01	\$5,000
7203	266548	0	257	ACP	8	1 - One	1.80	\$71,960
7204	266549	0	10	ACP	8	1 - One	1.01	\$5,000
7214	266559	0	2	ACP	8	1 - One	0.20	\$5,000
7215	266560	0	24	ACP	8	1 - One	1.80	\$6,720
7216	266561	0	156	ACP	8	1 - One	1.80	\$43,680
7218	266579	0	974	ACP	8	1 - One	1.83	\$272,720
7219	266580	0	1142	ACP	8	1 - One	1.80	\$319,760
7222	266583	0	990	PVC	8	1 - One	1.79	\$277,200
7224	266585	40	334	ACP	8	1 - One	1.80	\$93,520
7225	266586	40	55	ACP	8	1 - One	1.80	\$15,400
7226	266587	40	284	ACP	8	1 - One	1.80	\$79,520
7231	266592	40	39	ACP	8	1 - One	1.80	\$10,920
7232	266593	40	39	ACP	8	1 - One	1.80	\$10,920
7234	266600	0	419	ACP	8	1 - One	1.80	\$117,320
7235	266601	57	1114	ACP	8	1 - One	1.80	\$311,920
7236	266602	56	1098	ACP	8	1 - One	1.80	\$307,440
7237	266603	0	31	ACP	8	1 - One	1.80	\$8,680
Total			66,380				Total	\$16,616,160

Table 7-17. Zone 1 Pipes in SIP-4 (8-inch and smaller)

7.4.3 Seismic Projects

The pipe-by-pipe seismic upgrade priorities are described in Section 7.4.2. Using this information as a starting point, we then grouped the pipe-by-pipe upgrades into practical "Seismic Projects", numbered from 1 to 12.

Each of the segments in SIP-2, SIP-3 and SIP-4 was reviewed to determine additional pipe segments to be included in the replacement project. To do this, each of the segments in Tables 7-13 to 7-17 was studied in GIS maps that include an overlay of the underlying liquefaction hazards. The technical issue is that should a pipe be exposed to lateral spreads, which can happen where there is an open face for the soils to move towards, then those pipes will also try to "drag" attached pipes that are outside the liquefaction zone. Therefore, to provide a suitable anchor zone for the pipes in the non-liquefied zones, additional pipes are included in the Seismic Projects to provide the anchorage capability.

For each pipe segment in Tables 7-13 to 7-17, the surrounding pipe segments were reviewed to determine a suitable anchor zone extension distance. Since the aim of the seismic replacement is to protect the pipe from permanent ground movements, the adjacent segments are extended outside the mapped VH liquefaction zones for a distance of approximately $200 * D$, where D is the pipe diameter in feet. For example, a 6-inch pipe would have an extended segment of at least 100 feet outside the VH liquefaction zone. This length is intended to provide a suitable anchor zone for the pipe in stable soils. It is recommended that as part of the detailed design, manual isolation valves be placed at more locations than would be normal: one valve at all pipes at every intersection of 3 or more pipes; and one valve (with an upstream hydrant and possibly a downstream slip joint in case there is any question of providing an adequate anchor zone length) at the limits of reconstruction in stable soils. In this way, residual damaged pipes can be isolated with the fewest customer impacts; and if necessary, above ground hoses could be laid (from the hydrant) with the shortest distance to customers within the liquefaction zone.

In final design, should budgets otherwise restrict complete replacement efforts, then adding additional isolation valves (plus hydrants plus slip joints) to meet this design strategy could be appropriate. For example, if all of SIP-2 and SIP-3 can be funded, but only a portion of SIP-4 can be funded, then include just the extra isolation valves (and hydrants and slip joints) for the pipes in SIP-4, which could be done for about 15% of the budget for the incremental pipes in SIP-4. In other words, a "SIP-3.5" (intermediate between SIP-3 and SIP-4) would include all the pipe replacements in SIP-2 and SIP-3, and the remaining pipes, just the valve / hydrant / slip joint installations. All valves can be manual gate or butterfly type. Using the costs in Table 7-12, SIP-3.5 would be \$13.9 million + $0.15 * (\$33.2 - \$13.9 \text{ million}) = \$13.9 \text{ million} + \$2.9 \text{ million} = \16.8 million .

Since we do not presently have the design drawings for WMRP phases 25 and 26, we have left them in the SIP-4 population. Should the final design for these pipes adopt seismically-designed pipe (like HDPE, Kubota chained ductile iron pipe, US Pipe chained ductile iron pipe, etc.), then those pipes and their extended portions can be removed from the list of pipe candidates.

Given these issues, we then took the pipe-by-pipe upgrades data in Tables 7-13, 7-14 and 7-15, and developed so-called Seismic Projects 1, 2 and 3. This was done in a manner similar to that done for the AIP in Table 7-7.

- Seismic Project 1. All of the pipe segments in Table 7-13 are in Pressure Zone 3 area around San Francisquito Creek. As there are many pipe segments in this area, these segments, with their extended segments, are listed as Seismic Project 1 in Table 7-18 and summarized in Table 7-19. The total cost for Seismic Project 1 is \$6,018,400.
- The remaining segments in Tables 7-14 and 7-15, along with their extended segments, constitute smaller projects and are listed similarly in Tables 7-20 through 7-28 and summarized in Table 7-29. It should be noted that, in these tables, the replacement cost reported is for the segment, independent of any other segments, and as a result, the actual cost of a project could vary from the sum of costs for individual segments.

Figures 7-31 through 7-39 show the locations of the seismic projects. The red segments are in the initial replacement list in Table 7-13; the orange segments are the extended segments.

Main ID (SIP2)	PIPE GID (SIP2)	Main ID (SIP2Ext)	PIPE GID (SIP2Ext)	Pipe Age (Yrs)	Length (feet)	MATERIAL	Diam (inch)	Liq	Replace Cost (\$)	WMRP Phase No
354	250387			16	247	PVC	12	VH	88920	
		376	250459	0	2	ACP	10	VH	5000	
390	250508			0	10	DIP	12	VH	5000	
468	250752			15	202	PVC	8	VH	56560	
546	250997			0	131	ACP	8	VH	36680	
704	251442			16	291	PVC	12	VH	104760	
779	251646			0	375	ACP	10	VH	112500	
847	251848			0	15	UNKNOWN		VH	5000	
849	251854			16	317	PVC	12	VH	114120	
		890	251979	0	796	ACP	10	L	238800	
943	252130			0	671	ACP	10	VH	201300	
1098	252575			16	346	PVC	12	VH	124560	
1108	252601			15	244	PVC	8	VH	68320	
1187	252831			0	21	PVC	12	VH	7560	
1389	253406			0	250	PVC	8	VH	70000	
1501	253705			0	12	DIP	12	VH	5000	
1555	253897			56	375	ACP	12	VH	135000	
		1607	254035	56	20	ACP	12	M	7200	
		1636	254119	16	491	PVC	12	M	176760	
		1667	254226	56	1060	ACP	12	M	381600	
1725	254420			16	85	PVC	12	VH	30600	

Main ID (SIP2)	PIPE GID (SIP2)	Main ID (SIP2Ext)	PIPE GID (SIP2Ext)	Pipe Age (Yrs)	Length (feet)	MATERIAL	Diam (inch)	Liq	Replace Cost (\$)	WMRP Phase No
1773	254583			0	11	DIP	12	VH	5000	
1845	254777			0	14	DIP	12	VH	5040	
1920	254987			0	102	ACP	4	VH	14280	
		1938	255037	16	33	PVC	12	M	11880	
1943	255045			16	81	PVC	12	VH	29160	
		1982	255174	0	732	ACP	10	L	219600	
1992	255200			0	44	PVC	12	VH	15840	
1998	255225			0	21	PVC	12	VH	7560	
2024	255296			16	144	PVC	12	VH	51840	
2093	255501			16	230	PVC	12	VH	82800	
2117	255568			0	523	ACP	10	VH	156900	
		2211	255844	16	336	PVC	12	M	120960	
2218	255863			16	153	PVC	12	VH	55080	
2226	255883			16	290	DIP	12	VH	104400	
2240	255919			0	280	ACP	8	VH	78400	
2285	256049			16	374	PVC	12	VH	134640	
2339	256217			16	287	PVC	12	VH	103320	
2343	256226			0	10	DIP	12	VH	5000	
2475	256653			16	28	DIP	8	VH	7840	
2539	256814			0	267	ACP	8	VH	74760	
2541	256821			0	794	ACP	10	VH	238200	
2603	257011			0	221	DIP	8	VH	61880	
2634	257085			16	44	DIP	8	VH	12320	
2818	257631			0	22	DIP	12	VH	7920	
		2945	257833	0	529	ACP	8	M	148120	
3077	258005			0	61	ACP	8	VH	17080	
3137	258091			16	32	PVC	12	VH	11520	
3138	258092			16	55	PVC	12	VH	19800	
3190	258165			16	173	PVC	12	VH	62280	
3193	258168			16	70	PVC	12	VH	25200	
		3194	258169	16	267	PVC	12	M	96120	
		3553	258621	16	499	PVC	12	M	179640	
		3575	258648	16	779	PVC	12	M	280440	
		3690	258803	58	10	ACP	12	M	5000	
		3691	258804	58	74	ACP	12	M	26640	
3872	259046			16	68	PVC	12	VH	24480	
3873	259047			16	17	PVC	12	VH	6120	
		3892	259075	56	916	ACP	12	L	329760	
4056	259305			16	53	PVC	12	VH	19080	
4057	259306			16	61	PVC	12	VH	21960	
4407	259783			56	241	ACP	12	VH	86760	
4408	259784			56	15	ACP	12	VH	5400	

Main ID (SIP2)	PIPE GID (SIP2)	Main ID (SIP2Ext)	PIPE GID (SIP2Ext)	Pipe Age (Yrs)	Length (feet)	MATERIAL	Diam (inch)	Liq	Replace Cost (\$)	WMRP Phase No
4593	260057			16	394	PVC	12	VH	141840	
4601	260066			56	357	ACP	12	VH	128520	
4602	260067			56	11	ACP	12	VH	5000	
4772	260301			16	588	PVC	12	VH	211680	
4807	260382			16	971	PVC	12	VH	349560	
		5744	263197	15	6	DIP	12	VH	5000	
5745	263198			15	53	DIP	12	VH	19080	
		5746	263199	15	2	DIP	12	VH	5000	
		5748	263201	15	9	UNKNOWN	10	VH	5000	
5752	263216			14	194	DIP	12	VH	69840	
5753	263217			14	192	DIP	12	VH	69120	
5754	263218			15	24	DIP	12	VH	8640	
		5755	263219	15	5	DIP	12	VH	5000	
		5756	263220	15	6	DIP	12	VH	5000	
		5757	263221	15	0	DIP	12	VH	5000	
		6886	265872	2	24	PVC	12	M	8640	
		6888	265875	16	39	PVC	12	M	14040	
		6889	265876	16	135	PVC	12	M	48600	
				Total	17,392				\$6,049,820	

Table 7-18. Seismic Project 1 Detailed List

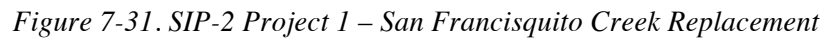


Table 7-19. Project 1 Summary – San Francisquito Creek

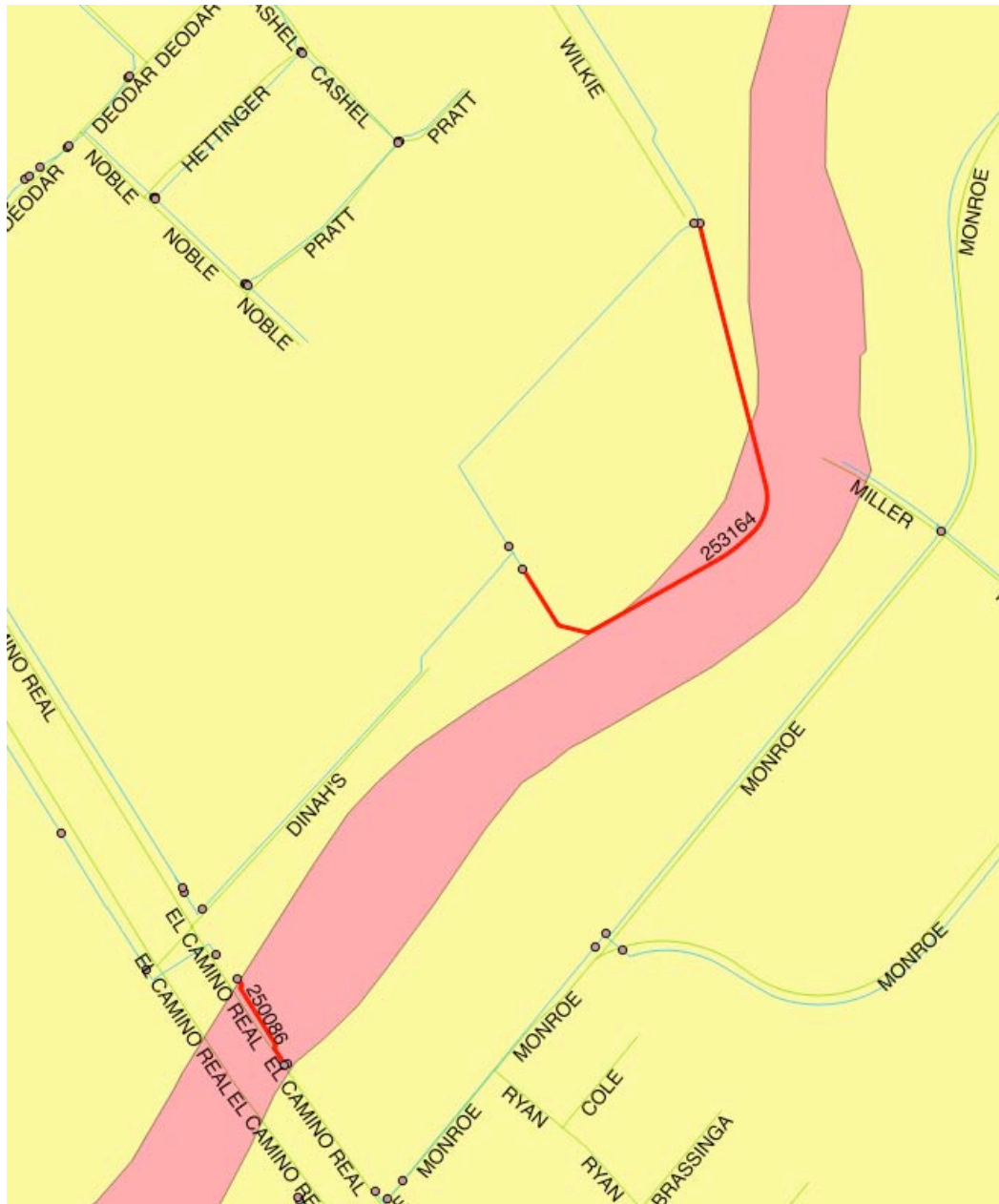


Figure 7-32. Seismic Projects 2 and 3 – El Camino Real and Wilkie Replacements

Main ID (SIP2)	PIPE GID (SIP2)	Main ID (SIP2Ext)	PIPE GID (SIP2Ext)	Pipe Age (Yrs)	Length (feet)	MATERIAL	Diam (inch)	Liq	Replace Cost (\$)	WMRP Phase No
244	250086			0	127	DIP	8	VH	35560	

Table 7-20. Seismic Project 2 – El Camino Real

Main ID (SIP2)	PIPE GID (SIP2)	Main ID (SIP2Ext)	PIPE GID (SIP2Ext)	Pipe Age (Yrs)	Length (feet)	MATERIAL	Diam (inch)	Liq	Replace Cost (\$)	WMRP Phase No
1306	253164			55	746	ACP	6	VH	156660	

Table 7-21. Seismic Project 3 – Wilkie



Figure 7-33. Seismic Project 4 – Laguna Replacement

Main ID (SIP2)	PIPE GID (SIP2)	Main ID (SIP2Ext)	PIPE GID (SIP2Ext)	Pipe Age (Yrs)	Length (feet)	MATERIAL	Diam (inch)	Liq	Replace Cost (\$)	WMRP Phase No
1753	254504			57	315	ACP	12	VH	113400	31
		6036	263855	10	10	PVC	12	M	5000	17
		6456	264875	3	7	PVC	12	M	5000	
			Total		332				\$123,400	

Table 7-22. Seismic Project 4 – Laguna

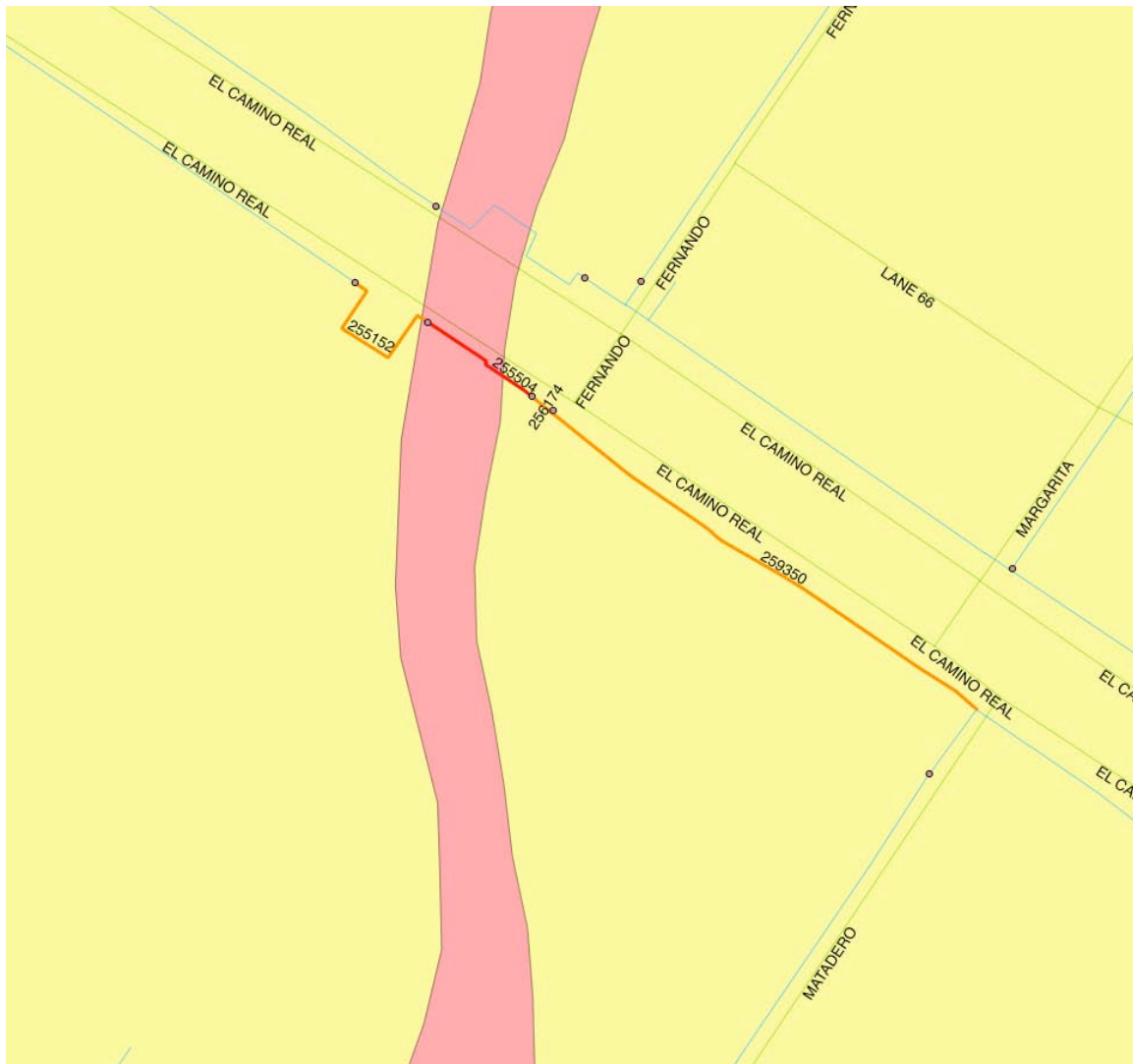


Figure 7-34. Seismic Project 5 – El Camino Real Replacement

Main ID (SIP2)	PIPE GID (SIP2)	Main ID (SIP2Ext)	PIPE GID (SIP2Ext)	Pipe Age (Yrs)	Length (feet)	MATERIAL	Diam (inch)	Liq	Replace Cost (\$)	WMRP Phase No
		1976	255152	0	120	CIP	8	M	33600	
2096	255504			0	87	CIP	8	VH	24360	
		2328	256174	0	2	ACP	8	M	5000	
		4091	259350	47	349	CIP	8	M	97720	
		4092	259351	47	18	CIP	8	M	5040	
			Total		576				165720	

Table 7-23. Seismic Project 5 – El Camino Real

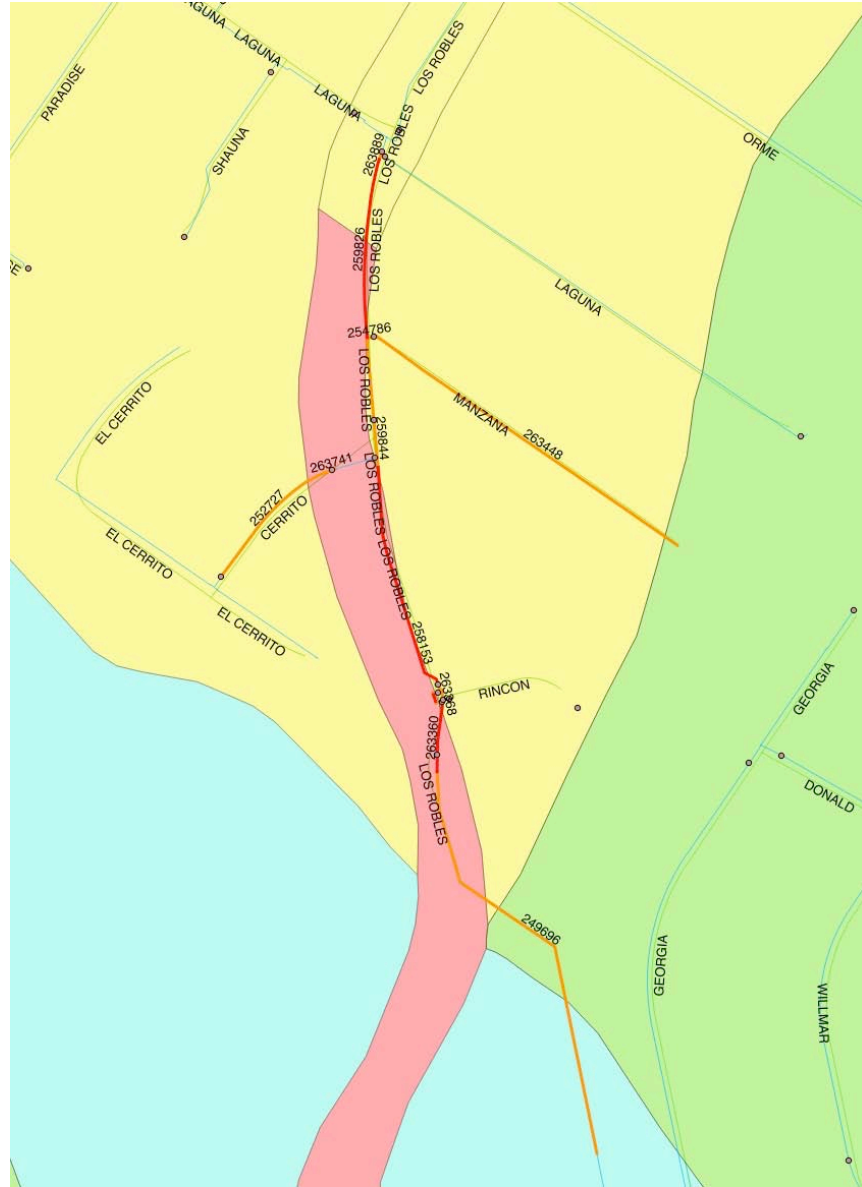


Figure 7-35a. Seismic Project 6 – Los Robles Replacement (Overall)

Main ID (SIP2)	PIPE GID (SIP2)	Main ID (SIP2Ext)	PIPE GID (SIP2Ext)	Pipe Age (Yrs)	Length (feet)	MATERIAL	Diam (inch)	Liq	Replace Cost (\$)	WMRP Phase No
		138	249696	51	766	ACP	12	L	275760	
		1034	252384	28	3	PVC	6	M	5000	1
		1150	252727	10	274	PVC	6	M	57540	17
		1210	252902	10	9	PVC	6	M	5000	17
		1850	254786	28	14	PVC	6	M	5000	1
		2787	257544	10	3	PVC	6	M	5000	17
		2987	257883	0	3	ACP	12	VH	5000	
3182	258153			28	403	PVC	12	VH	145080	1
		3361	258391	48	3	ACP	6	M	5000	

Main ID (SIP2)	PIPE GID (SIP2)	Main ID (SIP2Ext)	PIPE GID (SIP2Ext)	Pipe Age (Yrs)	Length (feet)	MATERIAL	Diam (inch)	Liq	Replace Cost (\$)	WMRP Phase No
		3656	258755	28	145	PVC	12	M	52200	1
		4287	259622	0	4	ACP	12	VH	5000	
4288	259623			0	13	ACP	12	VH	5000	
4441	259826			28	313	PVC	12	VH	112680	1
		4451	259843	28	18	PVC	12	M	6480	1
		4452	259844	28	65	PVC	12	M	23400	1
5820	263360			0	120	ACP	12	VH	43200	
		5821	263361	0	6	ACP	12	M	5000	
		5822	263362	3	3	PVC	12	M	5000	
		5823	263364	5	3	PVC	12	M	5000	
		5824	263366	0	1	ACP	4	M	5000	
		5826	263368	5	21	PVC	12	M	7560	
		5827	263369	5	3	PVC	12	M	5000	
		5859	263448	48	642	ACP	6	M	134820	
		5990	263740	10	4	PVC	6	M	5000	17
		5991	263741	10	4	PVC	6	VH	5000	17
		6045	263888	10	3	PVC	12	M	5000	17
		6046	263889	10	11	PVC	12	M	5000	17
			Total		2857				943720	

Table 7-24. Seismic Project 6 – Los Robles

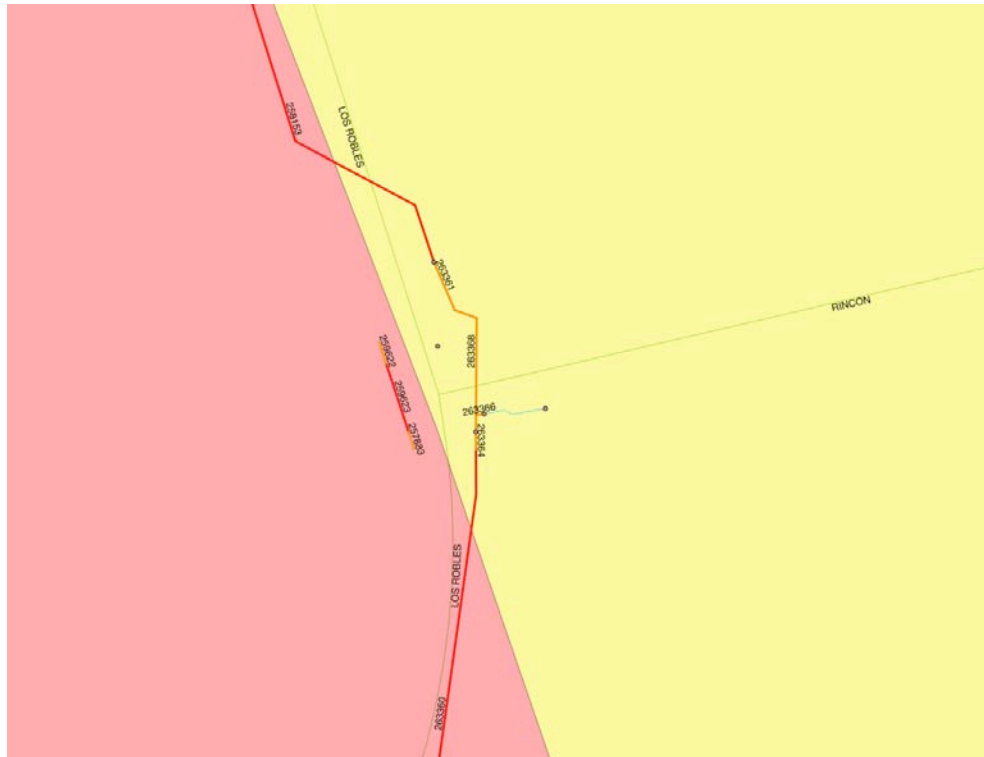


Figure 7-35b. Seismic Project 6 – Los Robles Replacement (Detail)

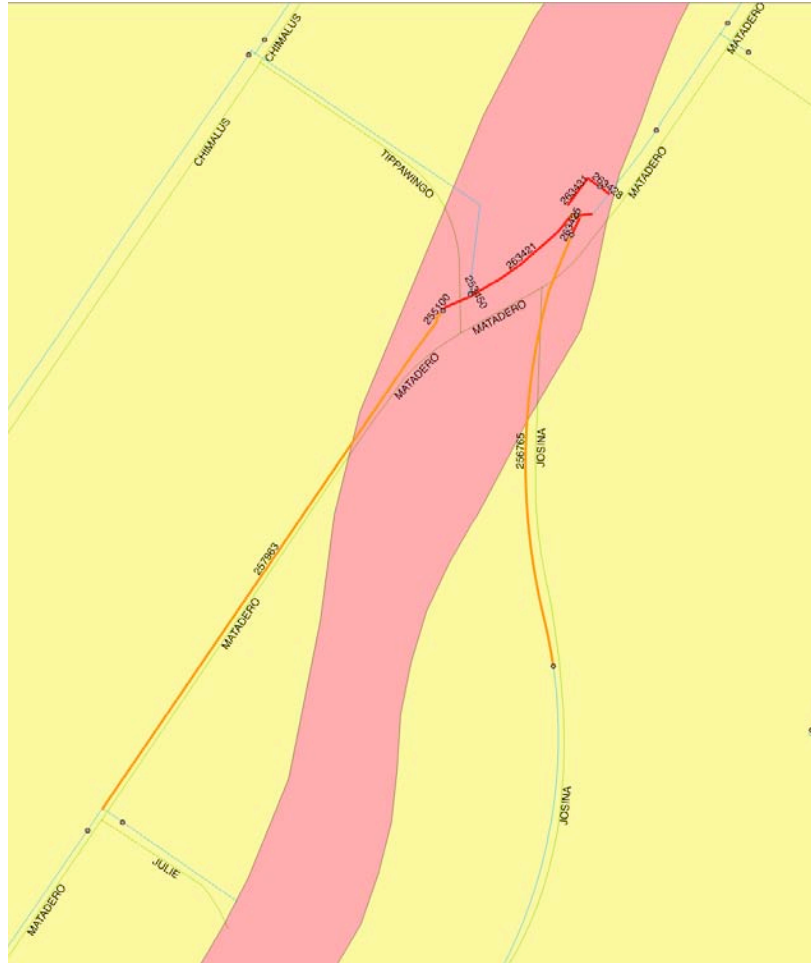


Figure 7-36. Seismic Project 7 – Matadero Replacement

Main ID (SIP2)	PIPE GID (SIP2)	Main ID (SIP2Ext)	PIPE GID (SIP2Ext)	Pipe Age (Yrs)	Length (feet)	MATERIAL	Diam (inch)	Liq	Replace Cost (\$)	WMRP Phase No
		1408	253450		2	ACP	6	VH	5000	
		1964	255100		5	PVC	6	VH	5000	
		2522	256765	0	405	ACP	6	M	85050	
		3052	257963	14	551	PVC	6	M	115710	14
3766	258910			0	30	ACP	6	VH	6300	
5844	263421			0	115	ACP	6	VH	24150	32
		5845	263422	3	8	DIP	6	VH	5000	
5846	263423			3	13	DIP	6	VH	5000	
5847	263425			0	16	ACP	6	VH	5000	
		5848	263426	3	3	DIP	6	VH	5000	
5850	263428			3	11	DIP	8	VH	5000	
5852	263431			3	44	DIP	8	VH	12320	
				Total	1203				278530	

Table 7-25. Seismic Project 7 – Matadero

Main ID (SIP2)	PIPE GID (SIP2)	Main ID (SIP2Ext)	PIPE GID (SIP2Ext)	Pipe Age (Yrs)	Length (feet)	MATERIAL	Diam (inch)	Liq	Replace Cost (\$)	WMRP Phase No
		1408	253450		2	ACP	6	VH	5000	
		1964	255100		5	PVC	6	VH	5000	
		2522	256765	0	405	ACP	6	M	85050	
		3052	257963	14	551	PVC	6	M	115710	14
3766	258910			0	30	ACP	6	VH	6300	
		137	249690	0	5	ACP	12	VH	5000	
		148	249739	0	3	ACP	8	VH	5000	
		203	249919	0	1	ACP	10	VH	5000	
		279	250188	0	4	UNKNOWN		VH	5000	
403	250528			0	361	UNKNOWN	12	VH	129960	33
		419	250601	0	6	ACP	8	M	5000	
		803	251730	0	12	UNKNOWN		VH	5000	
1056	252455			0	720	ACP	12	VH	259200	
1146	252715			0	294	ACP	12	VH	105840	33
		1935	255031	0	21	ACP	6	VH	5000	
		1937	255035	0	36	ACP	6	VH	7560	
		2347	256248	0	3	ACP	8	VH	5000	
		2404	256441	0	656	ACP	12	M	236160	
		2426	256501	0	2	ACP	8	VH	5000	
		2451	256575	0	32	ACP	8	VH	8960	
2789	257547			0	377	ACP	12	VH	135720	
		3002	257901	0	119	ACP	8	M	33320	
		3003	257902	0	312	ACP	8	VH	87360	
		3148	258104	0	33	UNKNOWN		M	6930	
4751	260267			48	1971	CCP	16	VH	946080	
4755	260274			48	99	ACP	16	VH	47520	
4756	260275			0	414	ACP	12	VH	149040	33
5309	262127			0	153	ACP	12	VH	55080	
7186	266531			0	1348	CCP	16	VH	647040	
7211	266556			0	11	CCP	16	VH	5280	
7212	266557			0	30	CCP	16	VH	14400	
7213	266558			0	28	CCP	16	VH	13440	
7227	266588			0	95	PVC	12	VH	34200	
7228	266589			0	596	PVC	12	VH	214560	
				Total	8735				3399710	

Table 7-26. Seismic Project 8 – Embarcadero

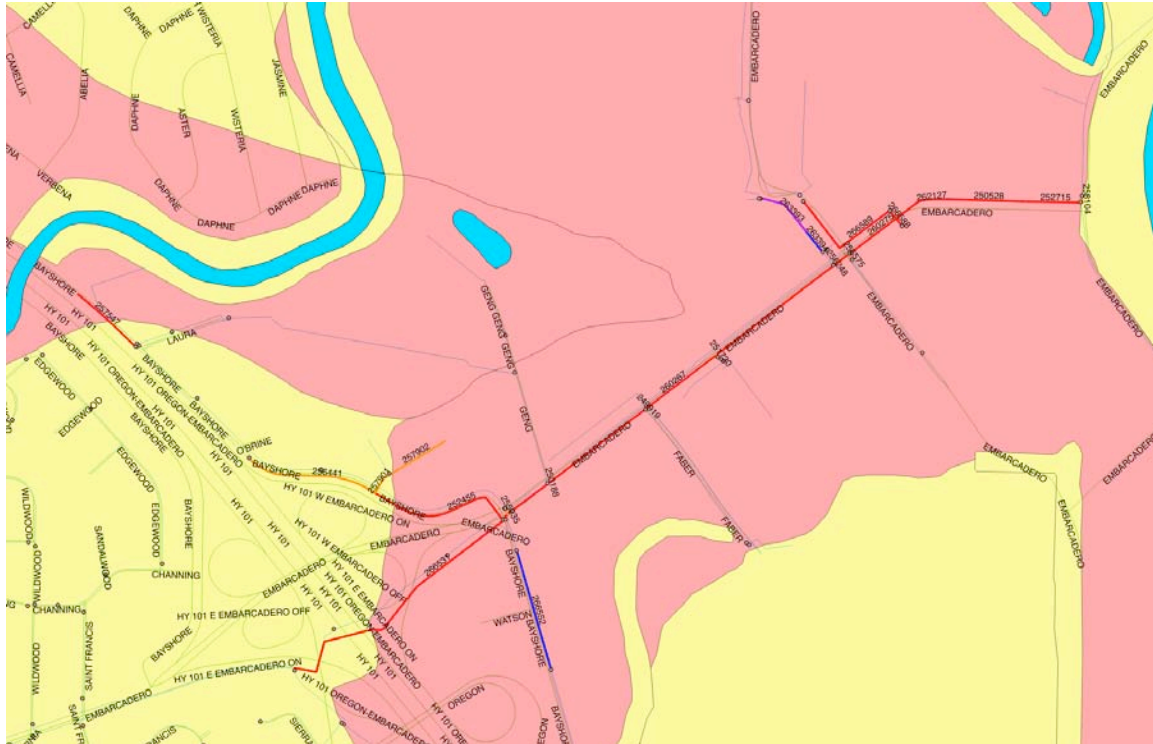


Figure 7-27. Seismic Project 8 – Embarcadero Replacement

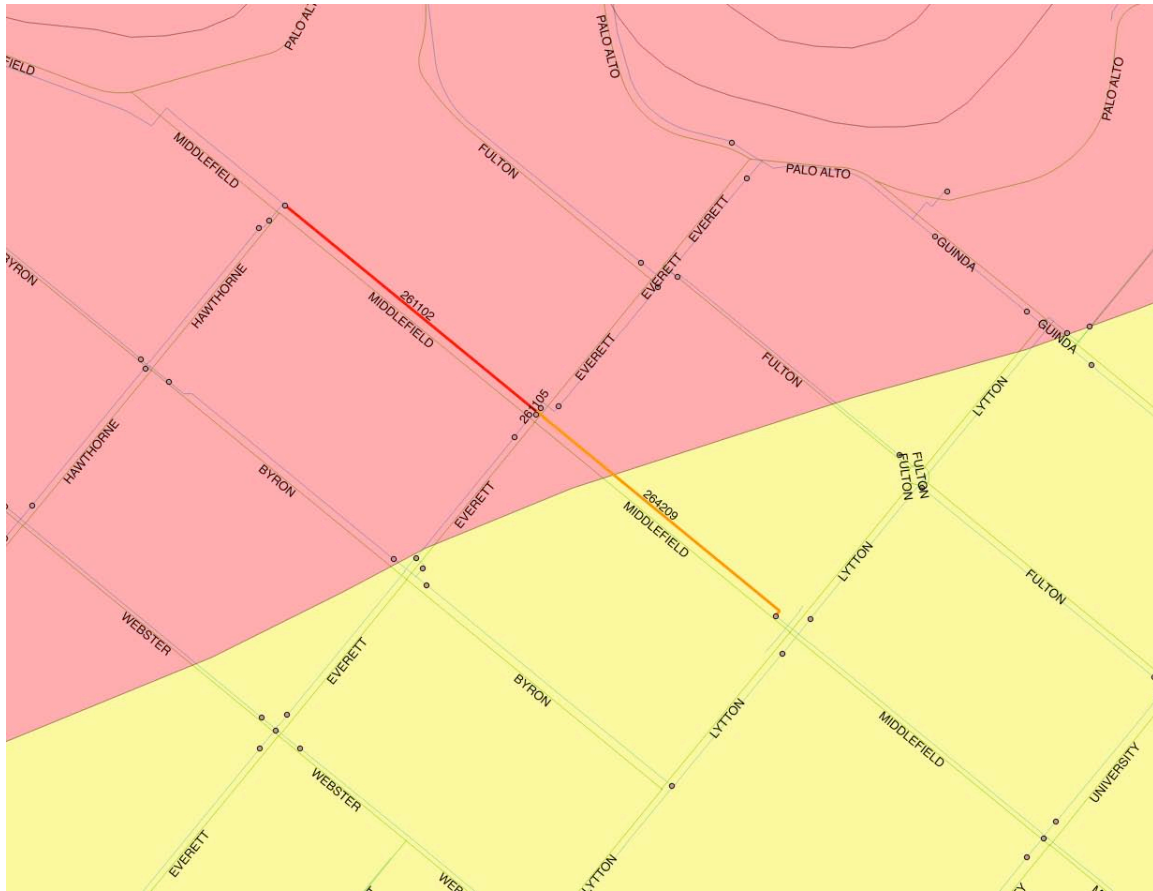


Figure 7-38. Seismic Project 9 – Middlefield Replacement

Main ID (SIP2)	PIPE GID (SIP2)	Main ID (SIP2Ext)	PIPE GID (SIP2Ext)	Pipe Age (Yrs)	Length (feet)	MATERIAL	Diam (inch)	Liq	Replace Cost (\$)	WMRP Phase No
		98	264209	32	445	ACP	8	M	124600	
4987	261102			32	455	ACP	12	VH	163800	
		4988	261104	0	4	CIP	6	VH	5000	
		4989	261105	32	7	ACP	6	VH	5000	
				Total	911				298400	

Table 7-27. Seismic Project 9 – Middlefield

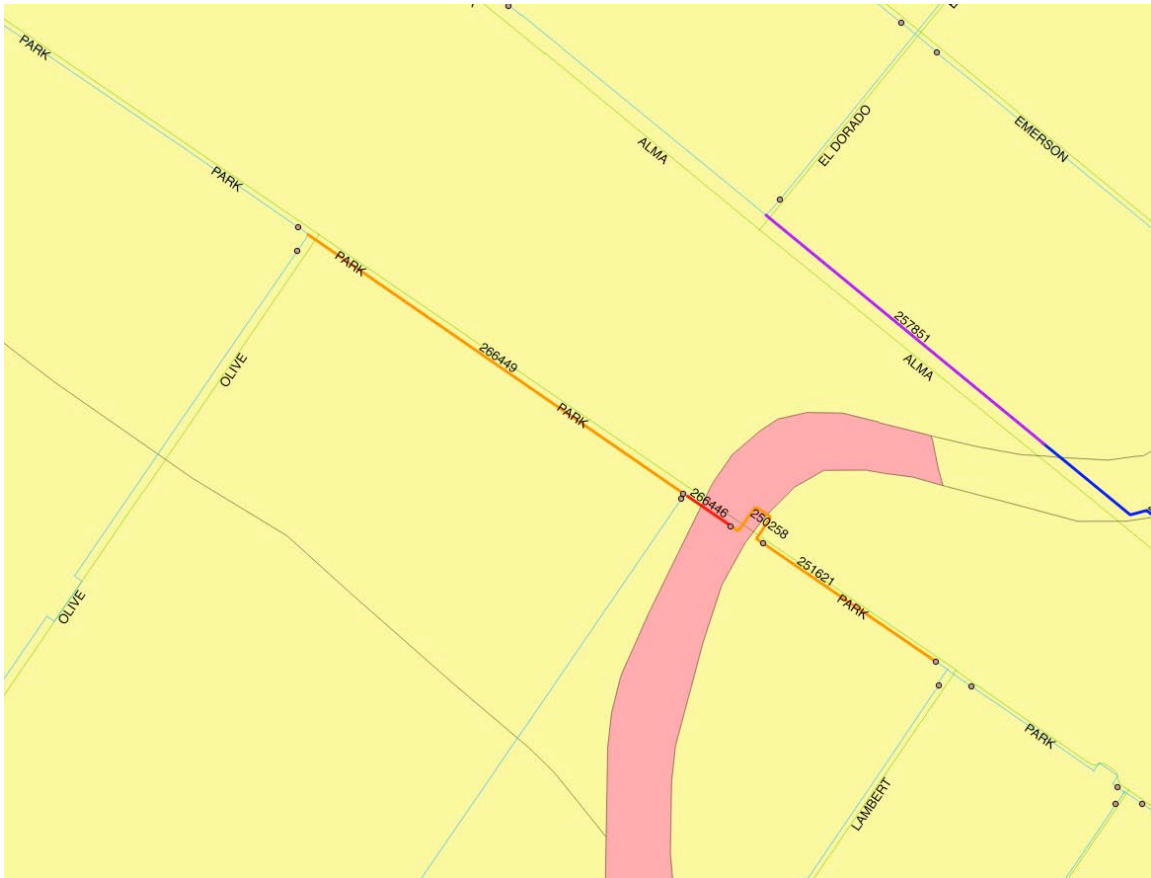


Figure 7-39. Seismic Project 10 – Park Replacement

Main ID (SIP2)	PIPE GID (SIP2)	Main ID (SIP2Ext)	PIPE GID (SIP2Ext)	Pipe Age (Yrs)	Length (feet)	MATERIAL	Diam (inch)	Liq	Replace Cost (\$)	WMRP Phase No
		307	250258	0	122	DIP	8	VH	34160	
		771	251621	15	287	PVC	8	M	80360	13
7155	266446			0	72	ACP	12	VH	25920	
		7158	266449	64	622	ACP	12	M	223920	27
				Total	1103				364360	

Table 7-28. Seismic Project 10 – Park

Seismic Project No.	Pipe Diameter (in)	Total Length (ft)	Replacement Cost (\$)	Street Location / Notes
2	8	127	35,560	El Camino Real btwn Dinah's and Monroe
3	6	746	156,660	Wilkie @ Dinah's (this section is also in AIP)
4	12	332	119,520	Laguna west of Barron
5	8	576	161,280	El Camino Real west of Matadero
6	4 6 12	1 956 1,900	140 200,760 684,000 Total 884,900	Los Robles from Laguna to south of Rincon Manzana (entire length) Cerrito (entire length)
7	6 8	1,148 55	241,080 15,400 Total 256,480	Matadero from Julie to north of Josina Josina from Matadero to south
8	6 8 10 12 16 Unknown	57 477 1 3671 3487 49	11,970 133,560 300 1,321,560 1,673,760 23,520 Total 3,164,670	Embarcadero from west of US101 to the Bay Bayshore NW of Laura Bayshore west of Embarcadero to O'Brine
9	6 8 12	11 445 455	2,310 124,600 163,800 Total 290,710	Middlefield btwn Hawthorne and Lytton
10	8 12	409 694	114,520 249,840 Total 364,360	Park btwn Olive and Lambert

Table 7-29. Seismic Projects 2 through 10 Summary

We then examined the pipes in SIP-3, to develop Seismic Project 11. For Seismic Project 11, we included the additional pipes in Zone 1 with $BCR \geq 1$, and all in the Embarcadero / Bayshore area. Figure 7-40 shows the area with the addition of the SIP-3 pipes to the SIP-2 pipes. In Figures 7-40 through 7-42, the original pipes in Table 7-16 are shown in dark green, and the extended pipe segments are shown in light green. Table 7-30 presents detailed lists of the segments in SIP-3, and Table 7-31 provides the summary for Project 11. Note that the pipe segments listed for SIP-3 are in addition to those for SIP-2; that is, if SIP-3 is pursued, all pipes in SIP-2 should be added to those listed in Table 7-30.

Main ID (SIP3)	PIPE GID (SIP3)	Main ID (SIP3Ext)	PIPE GID (SIP3Ext)	Pipe Age (Yrs)	Length (feet)	MATERIAL	Diam (inch)	Liq	Replace Cost (\$)	WMRP Phase No
180	249844			0	1761	ACP	10	VH	528300	
		263	250147	0	2	ACP	8	VH	5000	
		398	250521	0	145	ACP	6	VH	30450	
		406	250562	0	2	DIP	10	VH	5000	
		572	251073	0	6	UNKNOWN		VH	5000	

Main ID (SIP3)	PIPE GID (SIP3)	Main ID (SIP3Ext)	PIPE GID (SIP3Ext)	Pipe Age (Yrs)	Length (feet)	MATERIAL	Diam (inch)	Liq	Replace Cost (\$)	WMRP Phase No
		717	251468	0	41	ACP	8	VH	11480	
		828	251792	0	189	ACP	10	M	56700	
		947	252138	0	14	UNKNOWN		M	5000	
		1154	252738	0	90	Steel	8	VH	25200	
		1465	253611	0	230	UNKNOWN		VH	48300	
1608	254040			11	579	DIP	10	VH	173700	17
		1642	254132	0	6	ACP	10	M	5000	
		1887	254907	0	2	ACP	6	VH	5000	
		1971	255130	0	225	ACP	14	M	94500	
2065	255410			48	522	ACP	10	VH	156600	
		2200	255804	0	20	ACP	8	M	5600	
		2201	255807	0	2	ACP	8	VH	5000	
		2290	256073	54	497	ACP	10	M	149100	
2315	256139			0	681	ACP	10	VH	204300	
2356	256275			11	368	DIP	10	VH	110400	17
2462	256609			48	295	ACP	10	VH	88500	
		2486	256681	0	4	ACP	8	VH	5000	
2672	257210			0	399	ACP	10	VH	119700	
2947	257839			0	245	ACP	10	VH	73500	
2948	257840			0	383	ACP	10	VH	114900	
		3039	257948	0	65	ACP	6	VH	13650	
		3040	257949	0	16	ACP	6	VH	5000	
3123	258074			48	17	ACP	10	VH	5100	
		3124	258075	0	2	ACP	10	VH	5000	
		3317	258331	0	583	ACP	4	M	81620	
		3318	258332	0	2	ACP	10	M	5000	
		3474	258529	0	6	ACP	12	M	5000	
		3475	258530	0	771	ACP	12	M	277560	
		3587	258660	0	4	ACP	8	VH	5000	
3588	258661			0	76	ACP	10	VH	22800	
		3630	258721	0	9	ACP	10	VH	5000	
3960	259166			54	150	ACP	10	VH	45000	
		4220	259535	0	9	ACP	10	VH	5000	
4221	259536			0	17	ACP	10	VH	5100	
		4262	259584	0	4	ACP	10	M	5000	
4306	259645			0	374	ACP	10	VH	112200	
		4557	259994	0	40	ACP	10	M	12000	
4558	259995			0	222	ACP	10	VH	66600	
4752	260271			0	1169	ACP	10	VH	350700	
4757	260276			0	694	ACP	10	VH	208200	
5961	263678			11	32	DIP	10	VH	9600	17
		5962	263679	11	18	DIP	6	VH	5000	17

Main ID (SIP3)	PIPE GID (SIP3)	Main ID (SIP3Ext)	PIPE GID (SIP3Ext)	Pipe Age (Yrs)	Length (feet)	MATERIAL	Diam (inch)	Liq	Replace Cost (\$)	WMRP Phase No
		5963	263680	0	2	ACP	8	VH	5000	
7184	266529			54	2594	ACP	10	VH	778200	
7185	266530			55	2349	ACP	10	VH	704700	
7202	266547			54	516	ACP	10	VH	154800	
7207	266552			0	606	ACP	10	VH	181800	29
7208	266553			0	119	ACP	10	VH	35700	29
7210	266555			55	94	ACP	10	VH	28200	
		7214	266559	0	2	ACP	8	VH	5000	
		7220	266581	40	244	ACP	6	M	51240	
		7229	266590	0	3	PVC	12	VH	5000	
		7230	266591	0	1029	ACP	6	VH	216090	
				Total	18546				5452090	

Table 7-30. Seismic Project 11 – Embarcadero Zone 1

Pipe Diameter (in)	Total Length (ft)	Replacement Cost (\$)	Street Location / Notes
4	583	81,620	Embarcadero from west of US101 to the Bay
6	1,519	326,430	Laura from Bayshore to Geng
8	167	72,280	Faber (entire length)
10	16,555	4,531,400	Bayshore SE of Laura
12	780	287,560	Bayshore east of Embarcadero to SE of Corporation
14	225	94,500	(one segment also included in AIP, see Table 7-7,
Unknown	250	58,300	Project 33)
		Total 5,452,090	

Table 7-31. Seismic Project 11 Summary Zone 1

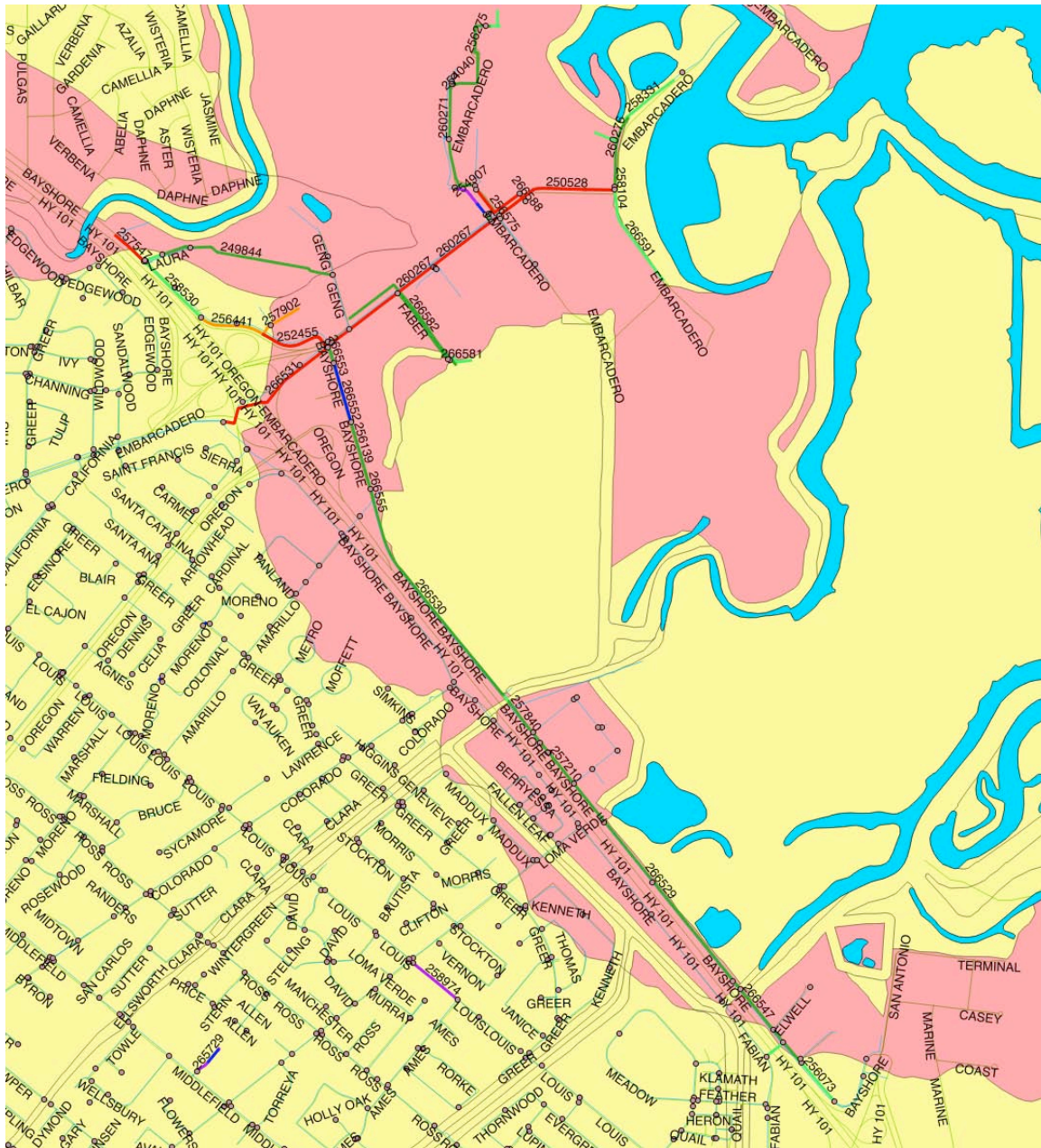


Figure 7-40. Seismic Project 11 – Embarcadero / Bayshore Replacement

Seismic Project 12 includes additional pipes with $BCR \geq 1$ in Zone 1. Figures 7-41 through 7-45 show five areas, with the addition of the SIP-4 pipes to the AIP, SIP-2 and SIP-3 pipes. In Figures 7-41 through 7-45, the original pipes in Table 7-17 are shown in fuschia, and the extended pipe segments are shown in pink. Table 7-32 presents detailed lists of the extended segments in Seismic Project 12, and Table 7-33 provides the summary. Note that the pipe segments listed for Seismic Project 12 are in addition to those for SIP-2 and SIP-3.

It should be noted that Tables 7-17 (SIP-4) and 7-32 (SIP-4 Extension) have many pipes which are noted to be in WMRP phases 25 and 26, which are currently in final design or

are in the process of completion. The pipe materials shown in the database are the original material and not the replaced material. If the replaced pipe is design to accommodate liquefaction (say ground strain of 1%), those pipe segments can be removed from the Seismic Project 12 populations; however, they are left in Tables 7-17 and 7-32 for purposes of this report.

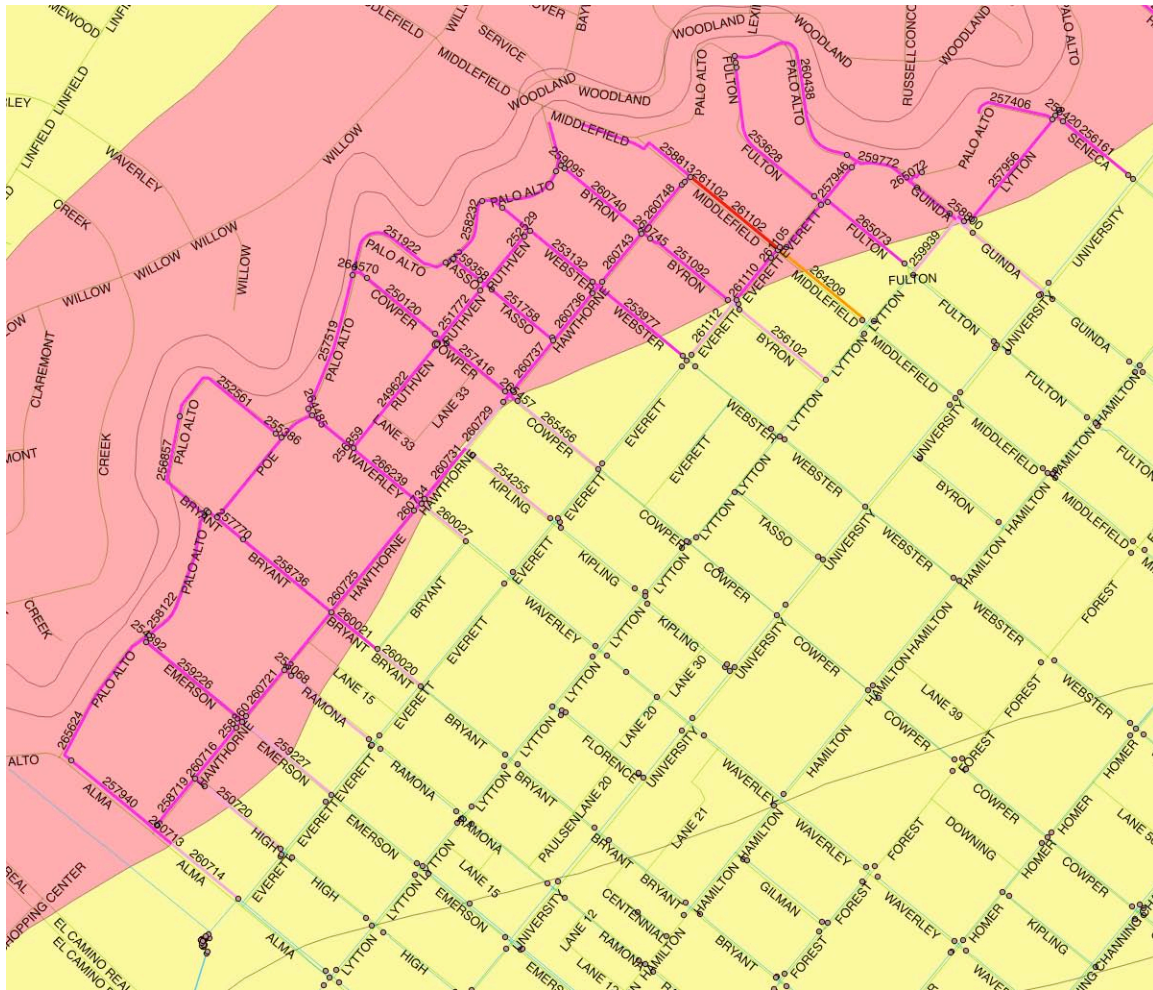


Figure 7-41. Seismic Project 12 – Area 1

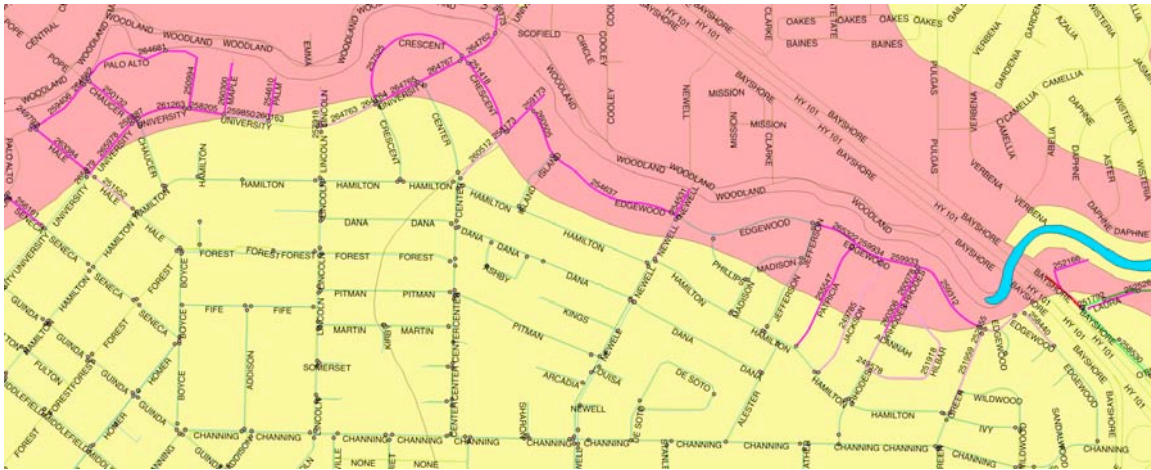


Figure 7-42. Seismic Project 12 – Area 2

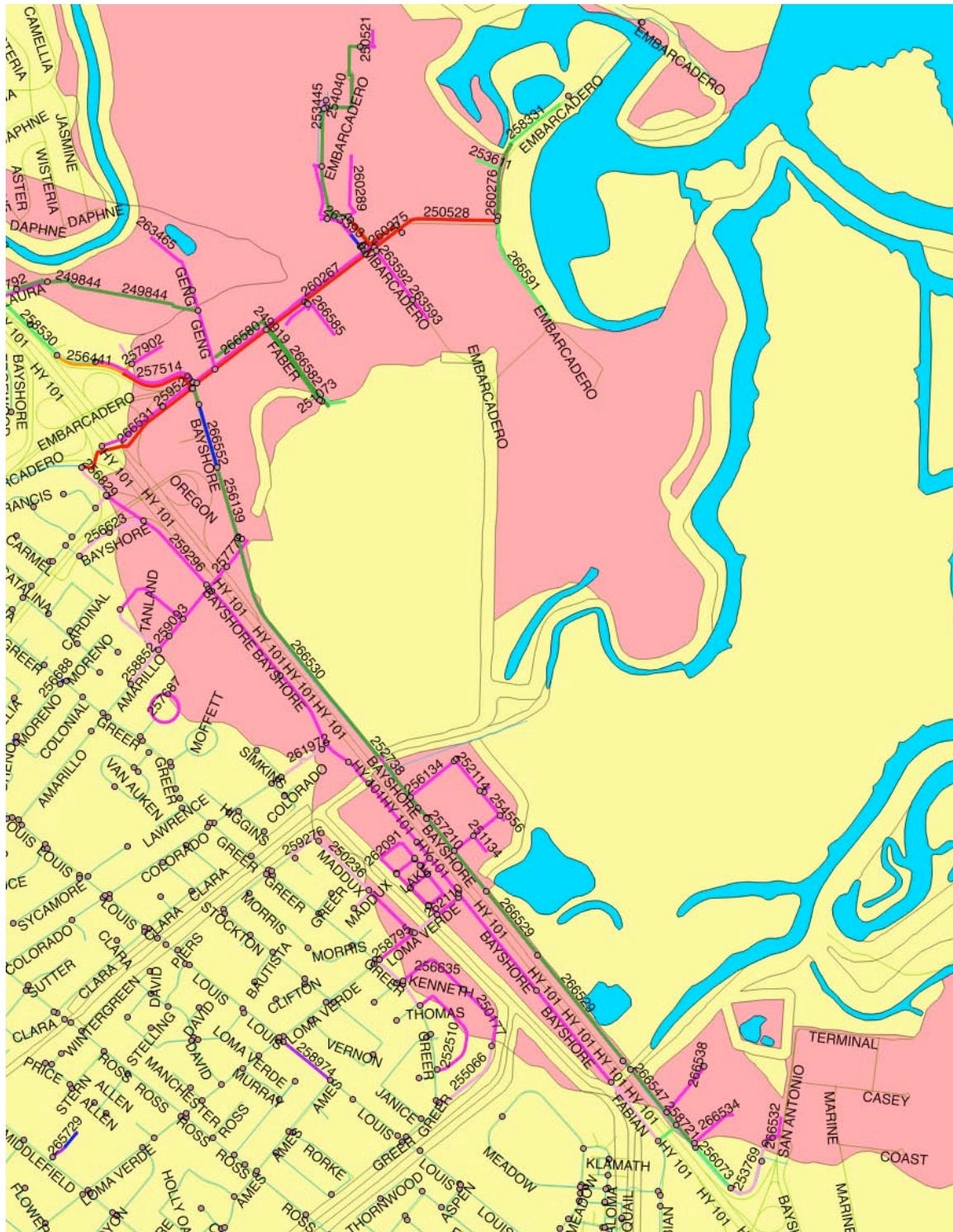


Figure 7-43. Seismic Project 12 – Area 3

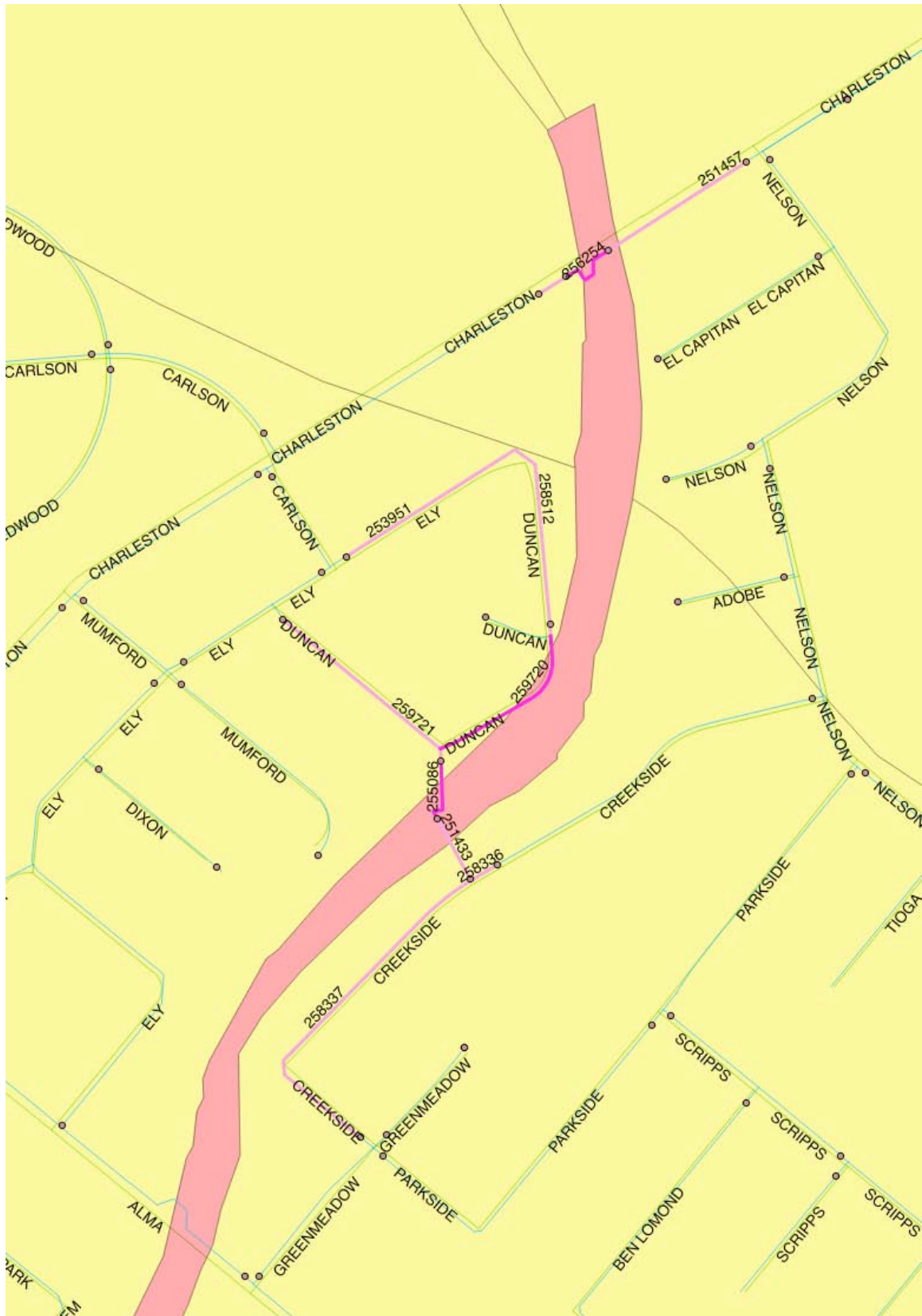


Figure 7-44. Seismic Project 12 – Area 4

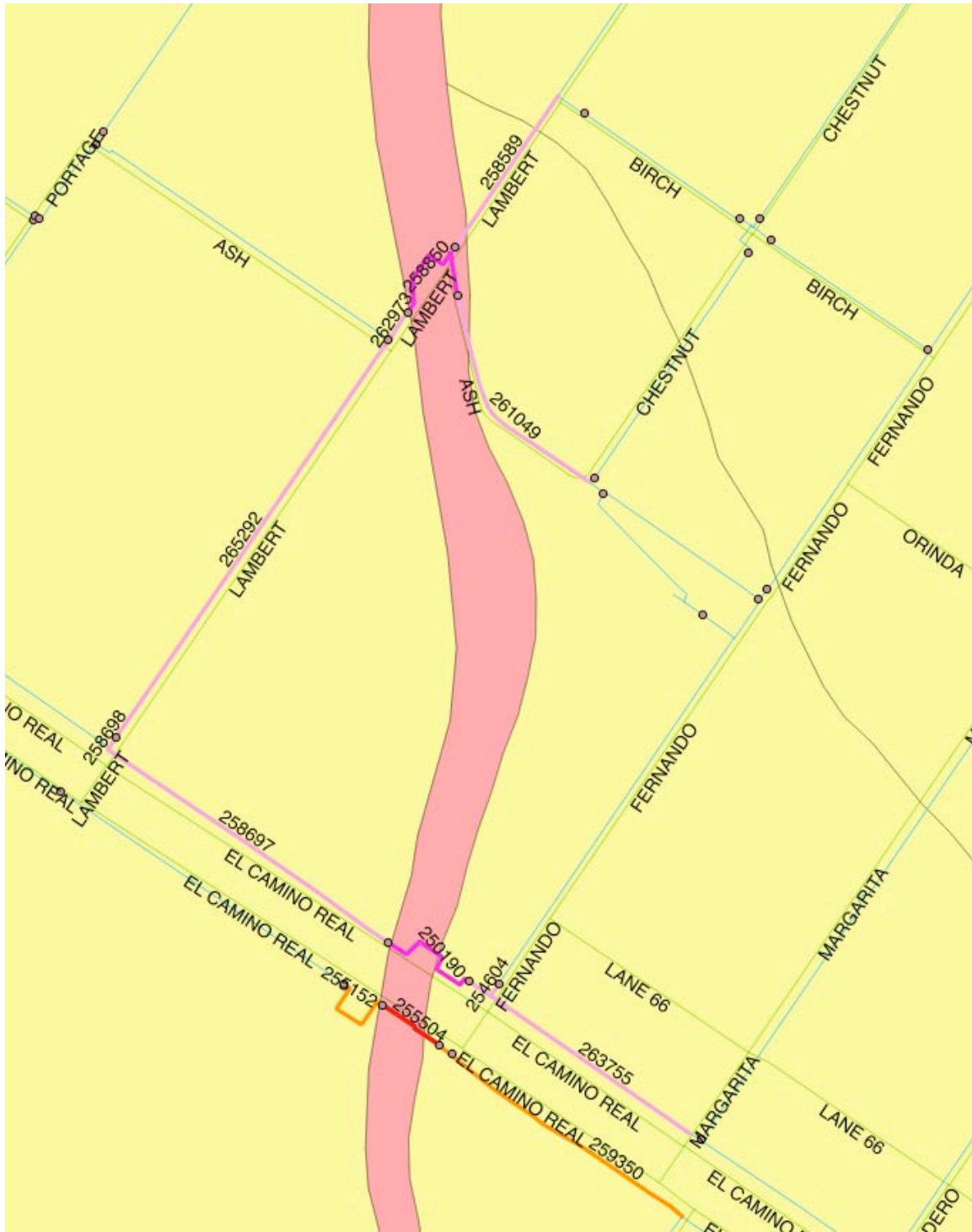


Figure 7-45. Seismic Project 12 – Area 5

Main ID (SIP4Ext)	PIPE GID (SIP4Ext)	Pipe Age (Yrs)	Length (feet)	MATERIAL	Diam (inch)	Liq	Replace Cost (\$)	WMRP Phase No
9	249578	66	41	ACP	6	M	8610	
31	260710	0	236	CIP	6	M	49560	25
32	260711	0	3	CIP	6	VH	5000	

Main ID (SIP4Ext)	PIPE GID (SIP4Ext)	Pipe Age (Yrs)	Length (feet)	MATERIAL	Diam (inch)	Liq	Replace Cost (\$)	WMRP Phase No
33	260712	0	3	CIP	6	VH	5000	
35	260714	0	348	ACP	8	M	97440	
42	260723	17	4	PVC	6	VH	5000	25
43	260724	17	4	PVC	6	VH	5000	25
45	260726	19	4	PVC	6	M	5000	10
46	260727	19	5	PVC	6	M	5000	10
48	260729	0	237	CIP	6	M	49770	25
50	260732	0	4	CIP	8	VH	5000	25
79	250548	0	2	PVC	6	VH	5000	
162	249785	65	1017	ACP	6	M	213570	
163	249792	3	139	CIP	10	VH	41700	
195	249897	41	3	ACP	8	VH	5000	
294	250236	0	652	ACP	6	M	136920	
311	250269	0	7	ACP	6	VH	5000	
360	250410	66	32	ACP	6	M	6720	
366	250429	15	6	PVC	6	M	5000	13
377	250461	41	2	ACP	8	VH	5000	
438	250657	0	1	PVC	6	M	5000	
460	250720	17	392	PVC	6	M	82320	11
518	250885	0	236	ACP	6	M	49560	
581	251105	8	21	PVC	8	M	5880	20
590	251132	0	1	ACP	6	VH	5000	
648	251297	17	3	PVC	6	VH	5000	11
697	251433	61	142	ACP	8	M	39760	
711	251457	62	339	ACP	8	M	94920	
742	251552	59	397	ACP	8	M	111160	
749	251578	0	2	ACP	8	VH	5000	
775	251633	39	2	ACP	6	VH	5000	
856	251890	39	1	ACP	6	VH	5000	
864	251918	66	1182	ACP	6	M	248220	
871	251930	0	6	ACP	6	VH	5000	
880	251959	67	682	ACP	8	M	190960	
961	252169	61	4	ACP	8	M	5000	
988	252255	41	2	ACP	8	VH	5000	25
1016	252329	0	80	ACP	6	M	16800	34
1027	252355	67	40	ACP	8	M	11200	
1112	252612	0	1	ACP	6	VH	5000	
1119	252637	0	2	CIP	6	VH	5000	
1157	252748	15	2	CIP	6	M	5000	13
1201	252873	17	1	PVC	6	VH	5000	11
1208	252898	19	22	PVC	6	M	5000	10
1270	253077	60	21	ACP	6	M	5000	

Main ID (SIP4Ext)	PIPE GID (SIP4Ext)	Pipe Age (Yrs)	Length (feet)	MATERIAL	Diam (inch)	Liq	Replace Cost (\$)	WMRP Phase No
1308	253172	0	5	CIP	4	VH	5000	
1374	253349	62	26	ACP	8	M	7280	
1451	253580	49	24	ACP	8	M	6720	
1460	253595	19	36	PVC	6	M	7560	10
1521	253769	52	630	ACP	8	M	176400	
1557	253904	0	66	ACP	8	M	18480	
1572	253951	62	231	ACP	6	M	48510	
1599	254011	0	199	ACP	8	M	55720	
1662	254214	0	58	ACP	8	M	16240	
1678	254255	18	402	PVC	6	M	84420	10
1779	254590	0	2	UNKNOWN		VH	5000	
1782	254604	60	19	ACP	8	M	5320	
1856	254803	0	5	ACP	4	VH	5000	
1901	254936	17	398	PVC	6	M	83580	11
1932	255022	41	2	ACP	6	VH	5000	
1953	255066	8	767	PVC	8	M	214760	20
1956	255072	0	2	PVC	8	VH	5000	
1974	255140	39	1	ACP	6	VH	5000	
2129	255604	0	1	CIP	6	M	5000	
2141	255626	0	12	CIP	4	M	5000	
2163	255697	19	89	PVC	6	M	18690	9
2164	255700	0	3	PVC	8	M	5000	
2176	255736	55	534	ACP	6	M	112140	28
2302	256102	18	441	PVC	6	M	92610	10
2385	256373	0	58	CIP	6	VH	12180	
2410	256457	17	1	PVC	6	VH	5000	25
2425	256500	0	3	ACP	6	VH	5000	
2453	256580	16	351	PVC	6	M	73710	12
2464	256616	16	3	PVC	6	VH	5000	12
2468	256623	51	694	ACP	6	M	145740	
2538	256811	67	50	ACP	8	M	14000	
2544	256829	56	371	ACP	8	M	103880	
2554	256859	41	3	ACP	8	VH	5000	
2564	256893	19	15	PVC	6	M	5000	10
2612	257028	60	107	CIP	8	M	29960	34
2659	257168	19	2	PVC	6	M	5000	10
2797	257577	0	2	ACP	6	M	5000	
2814	257622	0	216	ACP	4	M	30240	
2953	257845	67	125	ACP	8	M	35000	34
2954	257846	60	153	ACP	8	M	42840	34
2990	257888	11	227	ACP	6	M	47670	
3014	257918	15	25	PVC	6	M	5250	13

Main ID (SIP4Ext)	PIPE GID (SIP4Ext)	Pipe Age (Yrs)	Length (feet)	MATERIAL	Diam (inch)	Liq	Replace Cost (\$)	WMRP Phase No
3022	257927	0	4	CIP	4	VH	5000	
3023	257928	0	8	CIP	4	VH	5000	
3127	258079	60	20	ACP	8	M	5600	
3322	258336	61	64	ACP	8	M	17920	
3323	258337	61	771	ACP	8	M	215880	
3362	258392	17	30	PVC	6	M	6300	12
3363	258393	17	36	PVC	6	M	7560	12
3386	258421	0	4	ACP	8	VH	5000	
3401	258439	60	31	ACP	8	M	8680	34
3402	258440	60	322	ACP	8	M	90160	34
3462	258512	62	589	ACP	6	M	123690	
3527	258589	64	227	ACP	6	M	47670	
3569	258640	41	4	ACP	8	VH	5000	
3615	258697	0	423	ACP	6	M	88830	
3616	258698	0	15	ACP	6	M	5000	
3642	258738	20	368	PVC	6	M	77280	9
3687	258799	49	5	ACP	8	VH	5000	
3696	258812	32	4	ACP	6	VH	5000	
3727	258851	0	9	DIP	6	VH	5000	
3728	258852	71	412	ACP	6	M	86520	28
3815	258972	0	26	PVC	8	M	7280	26
3816	258973	14	11	PVC	8	M	5000	14
3852	259020	49	19	ACP	8	M	5320	
3853	259021	49	3	ACP	8	M	5000	
4004	259220	16	8	PVC	6	VH	5000	12
4007	259227	51	460	ACP	8	M	128800	15
4041	259276	0	323	ACP	6	M	67830	
4112	259380	0	40	DIP	6	M	8400	
4113	259381	0	19	DIP	6	M	5000	
4132	259407	0	2	CIP	6	VH	5000	26
4214	259528	66	286	ACP	6	M	60060	
4249	259570	39	4	ACP	8	VH	5000	
4318	259665	47	18	ACP	6	M	5000	
4319	259666	47	33	ACP	6	M	6930	
4346	259704	55	24	ACP	14	M	10080	
4362	259721	62	420	ACP	6	M	88200	
4383	259755	11	28	ACP	6	M	5880	
4456	259850	0	721	CIP	6	M	151410	26
4486	259891	41	9	ACP	8	VH	5000	
4517	259939	49	281	ACP	8	M	78680	
4529	259956	50	3	CIP	6	VH	5000	
4559	259997	16	4	PVC	6	VH	5000	12

Main ID (SIP4Ext)	PIPE GID (SIP4Ext)	Pipe Age (Yrs)	Length (feet)	MATERIAL	Diam (inch)	Liq	Replace Cost (\$)	WMRP Phase No
4567	260008	0	157	ACP	8	M	43960	
4573	260020	51	231	ACP	8	M	64680	15
4578	260027	0	231	CIP	8	M	64680	
4671	260163	0	8	CIP	6	M	5000	
4733	260245	39	4	ACP	8	VH	5000	
4778	260308	66	38	ACP	6	M	7980	
4882	260501		1	DIP	6	VH	5000	21
4889	260512	16	515	PVC	6	M	108150	12
4933	260746	0	8	CIP	6	VH	5000	
4934	260747	0	3	CIP	6	VH	5000	
4936	260749	39	8	ACP	6	VH	5000	
4973	261049	0	298	ACP	6	M	62580	
4985	261099	0	9	CIP	6	VH	5000	
4986	261100	32	9	ACP	6	VH	5000	
4991	261107	0	2	CIP	6	VH	5000	
4995	261111	0	24	CIP	6	M	5040	25
4996	261112	0	254	CIP	6	M	53340	25
4997	261113	19	4	PVC	6	M	5000	10
5023	261264	0	7	ACP	6	VH	5000	26
5221	261913	0	3	CIP	6	VH	5000	
5240	261971	56	192	ACP	8	M	53760	
5241	261972	56	219	ACP	8	M	61320	
5255	262063	7	1	PVC	8	VH	5000	
5257	262065	7	1	PVC	8	VH	5000	
5258	262066	7	1	PVC	8	VH	5000	
5260	262068	7	1	PVC	8	VH	5000	
5261	262069	7	1	PVC	8	VH	5000	
5263	262071	7	1	PVC	8	VH	5000	
5267	262075	7	1	PVC	8	VH	5000	
5269	262078	7	1	PVC	8	VH	5000	
5270	262079	7	1	PVC	8	VH	5000	
5273	262082	7	1	PVC	8	VH	5000	
5274	262083	7	1	PVC	8	VH	5000	
5277	262086	7	1	PVC	8	VH	5000	
5278	262087	7	1	PVC	8	VH	5000	
5279	262088	7	19	PVC	8	M	5320	
5285	262094	7	1	PVC	8	VH	5000	
5286	262095	7	2	PVC	8	VH	5000	
5289	262098	7	1	PVC	8	VH	5000	
5290	262099	7	1	PVC	8	VH	5000	
5291	262100	0	1	ACP	8	VH	5000	
5294	262103	7	1	PVC	8	VH	5000	

Main ID (SIP4Ext)	PIPE GID (SIP4Ext)	Pipe Age (Yrs)	Length (feet)	MATERIAL	Diam (inch)	Liq	Replace Cost (\$)	WMRP Phase No
5295	262104	7	1	PVC	8	VH	5000	
5299	262108	7	9	PVC	8	VH	5000	
5300	262109	56	265	ACP	8	M	74200	
5652	262972	4	3	PVC	6	M	5000	
5653	262973	0	36	ACP	6	M	7560	
5654	262974	4	3	PVC	6	M	5000	
5965	263686	60	15	ACP	8	M	5000	
5966	263687	60	297	ACP	8	M	83160	
5967	263688	8	6	PVC	8	M	5000	20
5995	263755	47	276	ACP	6	M	57960	
5996	263756	10	4	PVC	6	M	5000	17
6079	263957	0	4	CIP	6	VH	5000	
6080	263958	0	3	CIP	6	VH	5000	
6170	264169	15	248	PVC	6	M	52080	13
6291	264483	41	2	ACP	8	VH	5000	
6328	264571	17	6	PVC	6	VH	5000	11
6329	264572	17	6	PVC	6	VH	5000	11
6362	264683	16	10	PVC	6	VH	5000	26
6363	264684	16	10	PVC	6	VH	5000	12
6365	264686	16	2	PVC	6	VH	5000	12
6366	264687	16	5	PVC	6	VH	5000	12
6367	264688	16	2	PVC	6	VH	5000	12
6368	264689	16	2	PVC	6	VH	5000	12
6396	264763	14	439	PVC	8	M	122920	14
6397	264764	14	25	PVC	8	M	7000	14
6402	264769	15	3	PVC	6	M	5000	13
6403	264770	14	19	PVC	8	M	5320	14
6404	264771	14	3	PVC	8	VH	5000	14
6405	264772	14	2	PVC	8	M	5000	14
6406	264773	14	7	PVC	8	M	5000	14
6407	264774	14	20	PVC	8	M	5600	14
6408	264775	14	5	PVC	6	VH	5000	14
6409	264776	14	2	PVC	6	VH	5000	14
6411	264778	14	4	PVC	6	VH	5000	14
6412	264779	14	5	PVC	8	VH	5000	14
6445	264854	19	7	PVC	6	VH	5000	9
6446	264855	19	10	PVC	8	VH	5000	9
6448	264863	17	4	PVC	6	VH	5000	11
6545	265079	19	9	PVC	6	VH	5000	9
6635	265292	66	592	ACP	6	M	124320	
6636	265294	0	2	CIP	4	M	5000	
6693	265455	19	6	ACP	8	VH	5000	

Main ID (SIP4Ext)	PIPE GID (SIP4Ext)	Pipe Age (Yrs)	Length (feet)	MATERIAL	Diam (inch)	Liq	Replace Cost (\$)	WMRP Phase No
6694	265456	50	426	ACP	8	M	119280	
6941	265979	0	36	CIP	6	M	7560	26
6942	265980	59	49	ACP	8	M	13720	26
6943	265981	0	21	CIP	8	M	5880	26
7233	266599	56	692	ACP	8	M	193760	

Table 7-32. Seismic Project 12 Extension Segments

SIP4	Pipe Diameter (in)	Total Length (ft)	Replacement Cost (\$)
Original list (Table 7-17)	2	417	29,190
	4	351	72,840
	6	34,481	7,550,570
	8	31,131	8,963,560
	10	0	0
			Total 16,616,160
Extended list (Table 7-32)	4	252	60,140
	6	14,213	3,257,240
	8	10,015	2,995,800
	10	139	41,700
	14	24	10,080
	Unknown	2	5,000
			Total 6,370,060
Combined SIP-4 and Extended	2	417	29,190
	4	603	132,980
	6	48,694	10,807,810
	8	41,236	11,959,360
	10	139	41,700
	14	24	10,080
	Unknown	2	5,000
			Total 22,986,120

Table 7-33. Seismic Project 12 Summary

Table 7-34 lists the seismic upgrades, based on the Seismic Projects described in Section 7.4.3.

Item	SIP-1	SIP-2	SIP-3	SIP-4
Emergency Response	\$1,000,000	\$1,000,000	\$1,000,000	\$1,000,000
Foothills Pipeline		\$738,000	\$738,000	\$738,000
Seismic Projects				
1. 17,932 ft, Zone 3		\$6,049,820	\$6,049,820	\$6,049,820
2. 127 ft, El Camino R		\$35,560	\$35,560	\$35,560
3. 746 feet, Wilkie		\$156,660	\$156,660	\$156,660
4. 332 ft, Laguna		\$123,400	\$123,400	\$123,400
5. 576 ft, El Camino R		\$165,720	\$165,720	\$165,720
6. 2,857 ft. Los Robles		\$943,720	\$943,720	\$943,720
7. 1,203 ft. Matadero		\$278,350	\$278,350	\$278,350
8. 8,735 ft. Embarcadero		\$3,399,710	\$3,399,710	\$3,399,710
9. 911 ft. Middlefield		\$298,400	\$298,400	\$298,400
10. 1,103 ft. Park		\$364,360	\$364,360	\$364,360
11. 18,546 ft. Zone 1			\$5,452,090	\$5,452,090
12. 66,380 feet, Zone 1				\$16,616,160
12. 24,645 feet, Zone 1 Ext				\$6,370,060
13. 3,576 feet, Zone 1 Other				\$2,627,560
Total	\$1,000,000	\$13,553,700	\$19,005,790	\$44,619,570
Recommended	Very High	High	Possible	Marginal

Table 7-34. Seismic Upgrades – Pipes – Priority SIP-1, SIP-2, SIP-3 and SIP-4 (includes extensions of pipes described in Section 7.4.3)

It is recommended that SIP-3 be adopted, costing \$19,005,790 (\$19 million), and implemented over the next ten years. The incremental work included in SIP-4 could be adopted in the following decades or as pipes are placed due to aging, hydraulic requirements, relocations or other factors.

Within the recommended \$19 million SIP-3 program, the work should be ideally prioritized as follows: Emergency Response (take action immediately); SIP-2 items (implement in years 1 through 7); SIP items (implement in years 8 through 10); variations in actual year-by-year implementation can be made, recognizing practicalities of combining this work with the SIP and other needs in the city.

7.4.4 Water Main Risk Framework

The City of Palo Alto uses risk framework to select pipes for replacement, Figure 7-46.

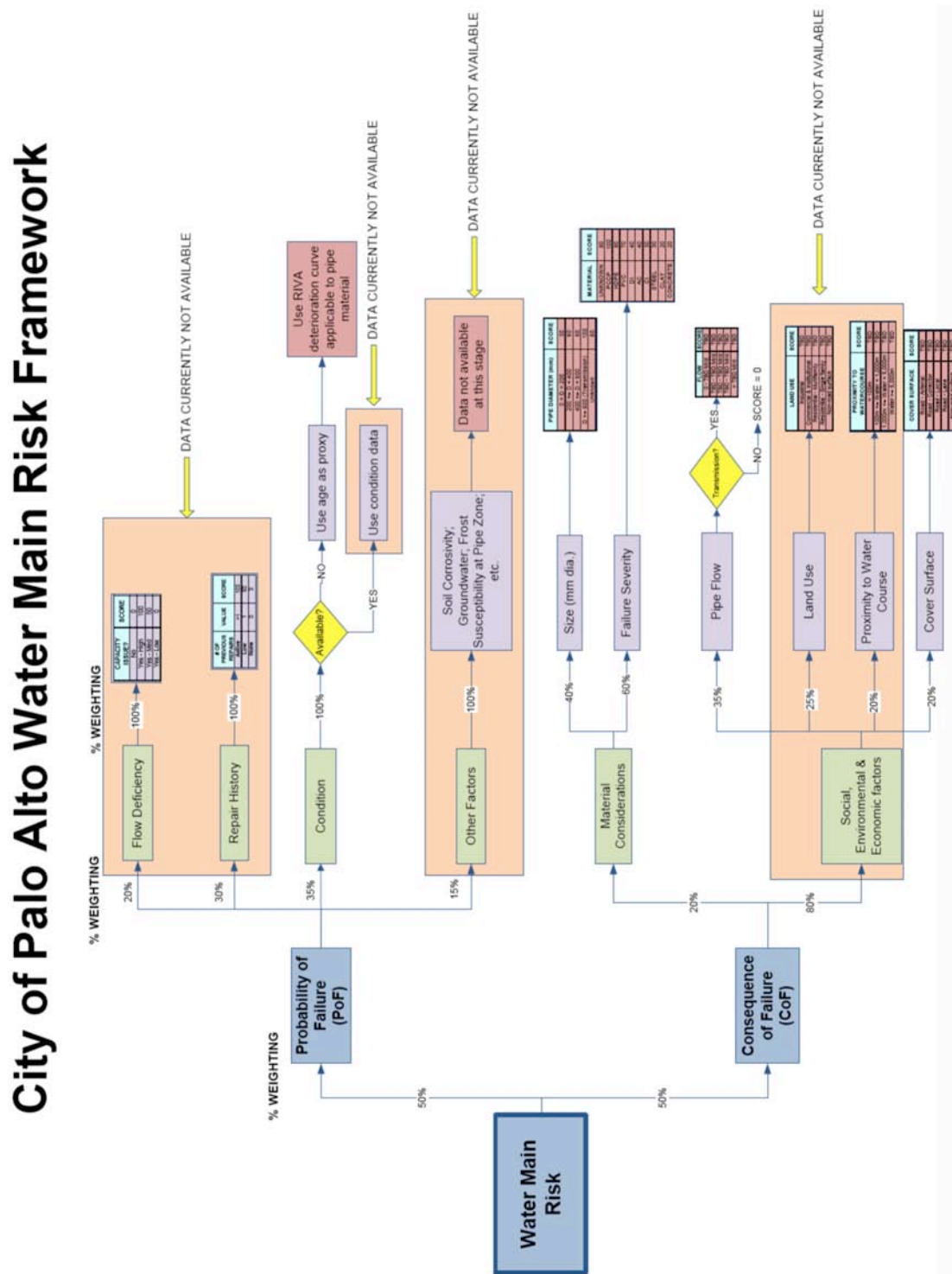


Figure 7-46. City of Palo Alto Water Main Risk Framework

This report provides various kinds of information that can be used to refine this framework. Specifically:

Probability of Failure

- **Repair History.** The databases developed for this report include the leak history for each pipe in the water system. This can be sorted by GIS ID number to see how many leaks (if any) on an individual segment of pipe, the date(s) of the repairs, the number of repairs over the past 7 years, etc. The Framework uses a "point weighting" system of 0, 60 to 100 to "score" each pipes. The Benefit Cost model applies a more detailed scoring system that factors in costs, economic impacts of outages, etc. for each pipe. If Palo Alto wishes to retain the Risk Framework, it is suggested that the "score" values of 100, 60 or 0, be replaced by the Benefit Cost ratio (from 0.001 to about 5) for each pipe, and then multiplied by a suitable value to suitably weight the repair history. Essentially, if the BCR ratio is over 1, the pipe (and its nearby neighboring segments to make a logical work effort) should be prioritized for replacement. Note: The BCR ratios exclude repairs made due to contractor action (hit pipe by excavator, etc.) as it was felt that these types of repairs do not have any relevance with regards to age-related deterioration of the pipe.
- **Flow Deficiency.** The scoring system places 0, 50 or 100 points for "deficiencies". How these "deficiencies" are determined is not described. It is suggested that these deficiencies be based on hydraulic models with the following criteria:
 - Establish baseline pressures at Average Day Demand. Use C values that factor in the age and lining of the pipe, with $C = 130$ (or so) for new pipe with smooth linings; $C = 100$ for any cast iron pipe with cement linings older than 50 years (considers that there might be some lining degradation over that time frame); $C = 40, 60, 80$ for unlined 4-inch, 6-inch, 8-inch unlined cast iron pipe older than 70 years; interpolate for intermediate ages. The baseline pressures could be calibrated using flow and pressure tests.
 - Establish pressures at peak hour demand, using the calibrated model. For pipes with more than a 20 psi drop in pressure (or drop in pressure below 20 psi), score 100 points. For pipes with less than a 10 psi drop, score 0 points. Interpolate for intermediate results.
 - Establish pressures at target fire flow demand, with concurrent ADD. For residential zones, assume 1,000 gpm fire flows. For commercial areas, assume 2,000 to 4,000 gpm fire flows (or whatever the fire marshal requires). For special fire zones (subject to urban interface fires / grassland fires), assume two 1,000 gpm fire flows within the same pressure zone at

the same time. Note: the fire flows can be met by multiple hydrants on multiple pipes, if they are located within about 500 feet of the fire location. A 4-inch cast iron pipe on a long dead-end street (more than 500 feet from a larger source of water) with no other nearby hydrants from other pipes, is a good candidate for replacement in a 10 to 20 year time frame. For each pipe that cannot sustain a minimum of 20 psi residual pressure for "regular" fires, score 50 points; for each pipe that cannot sustain a minimum of 10 psi residual pressure for "regular" fires, score 75 points. For each pipe that cannot sustain a minimum 10 psi residual pressure for "urban interface" fires, score 50 points. Total all points for all kinds of fires.

- Add up total points due to flow deficiencies. For pipes with ≤ 60 points, take no immediate action. For pipes ≥ 100 points, place in 10 year replacement program. Note: these exact point scores should be calibrated to reflect economic impacts. Not being able to meet normal flows at peak hour demand is the highest priority for replacement, as these cause regular (annual) impacts. A weighting factor for Flow Deficiencies due to fires could be adopted as follows: total up total estimated fire losses in Palo Alto over the past 50± years; map those fire losses; determine the percentage of those fire losses that were in areas with pipes that do not meet the above criteria. Get an annualized fire loss. Normalize the rate of fire loss to the length of pipe that does not meet the fire flow criteria listed above. Assume that if the pipes were replaced, the rate of fire loss would be reduced by a percentage (this needs some judgement, but possibly 10% for regular fires and 50% for interface fire events).
- Alternatively, take a conservative view, that if the existing pipes do not meet current fire flow criteria, they must be upgraded, as these are required by the Fire Chief. Palo Alto should be careful about adopting such a strategy, as "retrofit" of existing systems that were never designed to modern fire flow requirements is not a normal practice by other water agencies. To the extent that the Palo Alto customers accept such a retrofit policy, and the costs, then that is acceptable. However, the benefit cost approach outlined above might be a more cost effective strategy.
- Condition. The seismic + aging BCR models, as outlined in this report, could be used to assess Condition. The BCR value (generally between 0.01 and 5.0) could be multiplied by 100 to get a scoring system. Pipes with Condition > 100 should be included in a 10 year pipe replacement program.
- Other Factors. The effects of Soil Corrosivity and Groundwater and Leak History are already captured in the above recommended approaches. For pipes subject to frost heave (or shrink-swell soils), to some extent this is already captured by the above models. Therefore, this "branch" would be redundant.

- Weighting, PoF. Until a fire loss study is conducted, it is unclear what is the ideal "eight" applied to pipe upgrades for fire flows. Lacking such a study, possible put the following weights: 40% for Flow Deficiencies, 20% for Repair History, 40% for Condition (which includes seismic). Alternatively, the BCR ratios in this report can be used directly for Repair History and Seismic (upgrade those pipes with $BCR > 1$), and the Flow Deficiency pipes separately added into the overall pipe replacement plan.

Consequences of Failure

General observations. In the Benefit Cost models described in this report, the consequences of failure are addressed by quantifying the total economic impacts of loss of water (as-is) versus the reduced impacts if the pipe is replaced. The BCR ratios directly take into account the diameter of the pipe (larger diameter pipes have more customers between valves, and thus have more impact in outages), the type of pipe (the more fragile pipe materials are more prone to failure in seismic events) and adjustments to account for the general rate of economic activity (higher impact in Zones 2 and 3 commercial areas, lower impact in residential areas), adjusted to reflect the total economic activity in Palo Alto as a whole. The BCR models indirectly take into account "Proximity to Water Course", other than the fact that most liquefaction zones are next to creeks, so in this report, those pipes are identified as good candidates for replacement. The BCR model does not take into account cover surface, but in practice, this would be factored into design (pipes in highly congested urban streets cost more to replace than those in less congested or semi-rural areas).

Combined Ranking

The BCR models in this report provide a numeric ranking for every pipe in the Palo Alto system for pipe replacement, either due to aging / leak issues, or seismic issues. These BCR values are used to establish the pipe replacement program outlined in this report.

The BCR models in this report do not address pipe replacement due to flow deficiencies. The above descriptions suggest an approach to account for flow deficiencies that is based on hydraulic models, to address normal demands, peak demands, regular fires, and urban interface fires. There are two key points:

- For new installed pipes, for new subdivisions, the cost for the new pipe is usually born by the developer / owner of the new properties. These costs can be presented to the developer / owner, and they can make the decision to "go / no go".
- For replacement of old pipes, the cost for such replacement can be high. There are generally no code-based requirements for water departments to upgrade the old water system to the latest flow requirements or latest seismic design concepts. "Who pays" sometimes becomes an issue, in that these upgrades might benefit a

few customers, and whether these costs should be spread to all customer in the entire water system can be an issue.

- The assumption in this report is that there is no legal requirement to upgrade older water pipes. Instead, this report uses benefit cost effective principles to identify individual pipes for upgrade.

8.0 Funding

This report outlines a pipe replacement program that can be implemented over the next 10 years. This pipe replacement program includes about 13 to 14 miles of pipe, of which 2 miles are in sufficiently deteriorated condition as to warrant replacement, plus 10 miles of pipe that are seismically weak, plus an allowance of about 1 to 2 miles of pipe to allow for existing pipes that will deteriorate over the next decade to the point where they warrant replacement.

- The estimated cost for this aging-related pipe replacement program is \$2.92 million.
- The estimated cost for this seismic-related pipe replacement program is \$19.01million.
- During the next 10 years, we estimate that an additional 1 to 2 miles of pipe will continue to deteriorate, making them cost effective to replace, costing about \$2.3 million (for 1.5 miles of pipe).

In total, over a 10 year time frame, the total pipe placement costs are about \$24.23 million for about 13.5 miles of pipe.

The above recommendations are based on the following assumptions:

- Palo Alto can fund the effort in this time frame, costing about \$2 million per year.
- Palo Alto already have a capital plan to replace about 27.5 miles of pipe in this same time frame. While we do not have details of the exact capital budget, in 2015 dollars replacing 27.5 miles of pipe might cost well over \$40,000,000, or about \$4 million per year.

Hydraulic upgrade. This report does not address pipe replacement due to current or future-limited hydraulic capacity issues. Section 7.4.4 outlines hydraulic models and evaluations of the existing pipe network to consider day-to-day water demands, peak hour water demands, and various kinds of fire flows, in order to better quantify the location and length of pipe that might warrant upgrade to improve hydraulic performance. We do not know of any current location that would require such upgrade, but the following observations are made:

- Hydrants located on streets with long lengths (several hundred feet or more) of 4-inch diameter old cast iron pipe (likely with tuberculation), will be unable to provide high flow rates (much over 500 gpm) out of hydrants during fires. While small fires can often be controlled with one or two hand-lines at 200 gpm or less, should there be a major fire (especially one that spreads beyond the initial

- ignition), water demands could easily reach 1,000 gpm or much higher levels. In these cases, the lack of water flows might hamper control of the fire, leading to further fire spread, especially if it is windy. If such situations exist, these pipes are good candidates for replacement with 6-inch (possibly 8-inch) diameter pipe.
- The higher elevation pressure zones (Foothills area) are all subject to wildland grass fires. The fuel load locally can be heavy, and canopy fires in some areas cannot be ruled out during the drier months, especially on windy hot days. The water system in the Foothills provides reasonable protection for regular fires to protect spread of fire ignitions that begin at structures located within a hundred feet or so from roads where fire department trucks and personnel have good access. However, the Foothills water system does not extend into the rural areas. Therefore, the likely strategy should a wildland fire start is that the city fire department will wait until the fire approaches the structures before attempting to control the fire; or, even better, the initial ignition will be identified within minutes, before it spreads very far, and the fire controlled.
 - It is not likely cost effective for the Palo Alto water department to extend or upgrade its water system in the Foothills to fight major wildland fires. The existing water tanks in the Foothills can provide, by gravity flow to each pressure zone, about 8,000 gpm for two hours, before they are exhausted. Within 2 hours, either the fuel load near most structures will be exhausted, or the structures will have been burned. So, in either case, the need for more storage in the Foothills would not have much gain.

Who pays. The ~13.5 miles of pipe targeted for replacement includes replacement in Zones 1, 2 and 3, as well as along critical pipelines in the Foothills. This suggests that the total cost should be absorbed city-wide, as there would be benefits to all areas in the city. Trying to allocate the cost to different pressure zones, or sub-areas within individual pressure zones, does not appear to be warranted.

Long Term Pipe Replacement. Over the next decade, this report recommends about 13.5 miles of pipe replacement. This represents about $13.5 / 236 / 10 = 0.57\%$ pipe replacement per year. If this rate of replacement were extended beyond 10 years, this would suggest about a 175 year cycle to replace all pipes. While some water agencies in Japan are adopting a 40 to 65 year water pipe replacement cycle, in part to address earthquake risks, such a rapid (and expensive) replacement cycle is not likely to be cost effective in California. The rate of major earthquakes in Japan is about 3 times higher than in California. There is very little information in California (or anywhere else) as to the lifetime of water pipes for water pipes older than about 100 years... within California, nearly all pipes (galvanized steel, wood pipes) that were installed > 100 years ago have already been replaced due to their very high leak rates, or increases in water demands over time. Within Palo Alto, there is currently no type of pipe remaining with very high leak rates (0.5 repairs per mile per year), and most of the remaining pipes have rather low repair rates (event cast iron pipe has an average repair rate under 0.15 repairs per mile per year). The approach to address aging / deterioration of pipe, as outlined in this report,

factors in the pipe's actual repair history, and thus should provide a long term strategy for pipe replacement, with the objective to replace pipe only when it becomes cost effective to do so. This should keep capital expenditures on pipe replacement at levels which are economically sound.

With respect to seismic issues, there is no way to predict exactly in which year when a major earthquake will occur. Ideally, all seismic-related upgrades should be complete just prior to that earthquake. Given these issues, this report suggests a 10-year cycle to upgrade the most vulnerable pipe (about 10 miles) in the areas that will likely have the most seismic impacts. This will reduce, but not eliminate, the adverse impacts of pipe breakage due to future earthquakes. As Palo Alto eventually replaces more and more pipe, beyond the next 10 years, more of the seismic weaknesses can be eliminated. Eventually, all pipe in zones prone to liquefaction, landslide or surface faulting, could be replaced, and this would further reduce the impacts due to earthquakes; but this might take an additional 10 to 100 years to implement. In the short term, a strong emergency response capability to rapidly fix broken pipes, is the most cost effective strategy in the short term.

Source of Funding. The main source funding for pipe replacement can be from Palo Alto's own capital budgets.

FEMA (Federal Emergency Management Agency) provides a possible additional source of funding. There are several programs:

- After a Presidential-declared disaster, FEMA currently has the mandate to co-pay for repair work done by public agencies. Palo Alto is eligible. The current co-pay rate is 75% (historically, it was 90%, and sometimes even 100%, depending on political factors). For example, if Palo Alto suffers 200 pipe repairs, and the average cost to make a repair is \$10,000, then the repair cost is \$2,000,000, and Palo Alto could expect to be reimbursed by FEMA about \$1,500,000 (presuming Palo Alto follows all of FEMA's many requirements, including documentation). Generally, FEMA does not pay for any improvements (in other words, if the pipe breaks, FEMA will pay to make a pipe break repair by adding a clamp or similar, but not replacement of a 6-inch pipe along an entire street with a new 8-inch pipe). This same program would provide repair co-pays for damage to water tanks, etc. Consequential damage (like due to fires, economic impacts due to no water, etc.) is generally not covered.
- After a Presidential-declared disaster, FEMA currently has the mandate to co-pay for mitigation work done by public agencies. Palo Alto is eligible. The current co-pay rate is 75% (historically). In other words, for items that are not damaged, Palo Alto could still get a mitigation grant to prevent future damage. The amount of money available for these types of grants is usually about 7.5% (or so) of the total money spend on repair work. In order to be eligible, each mitigation project must have a benefit cost ratio of 1 or higher.

- Prior to a disaster, FEMA currently has the mandate to co-pay for mitigation work done by public agencies, as part of the HMGP-C program (Hazard Mitigation Grant Program – Competitive). Palo Alto is eligible. The current co-pay rate is 75%, or up to \$3,000,000 per project, whichever is lower. The amount of money available for these types of grants has varied, year-to-year, from as high as \$250,000,000 per year to as low as \$75,000,000, per year. All 50 states are eligible. These grants cover earthquake, fire, flood and similar natural disasters. Should FEMA get more grant applications than they have money available, FEMA will select the projects based on many criteria, one of which is that the project must have a benefit cost ratio greater than 1 (generally, the higher the better).

All FEMA grants are made through the applicant (which is the State of California), with Palo Alto being the sub-applicant. The California Office of Emergency Services (CA OES) manages the programs. As such, CA OES has their own ranking system as to which grants within California should be funded, and their weighting / ranking is a factor in FEMA's ultimate decision making process.

There are other possible sources of funding through various State of California bonds, etc. This report does not address these other sources.

FEMA Grant. Based on the findings in this report, it is evident that there are seismic vulnerabilities in Palo Alto's water system. Palo Alto is eligible for FEMA mitigation grant co-funding. All such projects would require a benefit cost analysis, that follows FEMA's rules (7% discount rate, 50 year project lifetime, etc). The types of items that may be most likely to be co-funded include the following (in the past, all these types of projects have been successfully co-funded by FEMA):

- Seismic upgrade of water tanks. For example, adding flexible pipe attachments (\$20,000 or so per tank) and anchorage of unanchored steel tanks (\$100,000 to \$300,000 per tank). Palo Alto already have plans to mitigate some tanks. Any tank that has started construction of the mitigation automatically becomes ineligible for FEMA co-funding; but tanks that have been designed for upgrade, but where upgrade has not actually started, remain eligible.
- Anchorage of non-structural items. We observed in the course of this work some unanchored SCADA equipment (batteries). These are low-cost items to seismically upgrade, and often have large benefit cost ratios. A full inventory of such items would need to be established, costs developed.
- Adding valves to pipelines and outlet manifolds: closing the valves avoids system de-pressurization due to downstream damage due to surface faulting, liquefaction or landslide; using the bypass manifolds to use above-ground hose to quickly restore emergency water service to affected downstream customers.

- Pipe replacement through especially high seismic hazard zones, such as surface faulting, liquefaction or landslide.

While there have been exceptions, FEMA usually does not co-fund procurement of portable generators or pumps.

The seismic weaknesses in the existing water systems are such that Palo Alto can develop a grant application to FEMA to potentially obtain co-funding in any of the above four areas. The decision as to what exactly should be included in the grant application will depend in part on the amount of total FEMA funding, and how to best address the competitive nature of the grant application process. For example, if FEMA announces that they have \$50,000,000 available, and that each of 50 states must get 1 mitigation project before any State gets 2 mitigation projects, then an application for \$3,000,000 in co-funding might not be the best strategy... rather, in such a case, a grant application for about \$1,000,000 and for projects with a high benefit cost ratio, might be a better strategy.

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Working Group, *The Uniform California Earthquake Rupture Forecast, Version 2 (UCERF 2)*, The 2007 Working Group on California Earthquake Probabilities, USGS Open File Report 2007-1437, CGS Special Report 203, SCEC Contribution #1138, 2008.

9.2 Electronic

The seismic results presented in this report are from run SERA.14.46.21.out. The benefit cost analysis results presented in this report are from run SERA.14.21.26.out. The database with all pipe data and results is in file: Palo Alto 062215.fmp12.

A series of electronic files were used in the course of this effort. Shapefiles provided by Palo Alto are in NAD83, California zone 3 (feet), EPSG 2227.

T_WT_PM2. Shapefile. 7315 features (lines). In service pipes in Palo Alto system.

T_WT_PM5. Pipe names. 438 features, annotation for selected pipes in hillside pressure zones.

T_WT_CP1. Shapefile. 1506 features. Fittings. Cap. Reducer. Branch Saddle.

T_WT_H2. Hydrants.

T_WT_VM1. Valves. 1911 features. Gate, Butterfly. Main, Service, Hydrant, FS.

T_WT_PR3. Shapefile. 9 features (polygons). Pressure zone boundaries.

T_WT_PR5. Shapefile. 9 features (points). Pressure zone names for annotation.

Abandoned Pipes. Shapefile.

TC_RNAT4. 5028 features (points). Street name labels, with angle for plotting. Last update November 20 2013. Plots well at Scale of 1:2000.

NF_WC2. 271 features (Creeks). Centerline of creeks (not including lower San Francisquito creek). Last update 2013.

NF_WC5. 1 feature. San Francisquito Creek Name.

NF_WE2. Creeks. Lower San Francisquito and other creeks. Channels. Sloughs. Combine with NF_WC2 to get full streams.

NF_WE3. Lakes.

SD_CH2. Storm Drains. Centerlines of drainages and sloughs.

T_WT_LE1. 1107 features. Leaks. Mains, Services, etc.

T_WT_PM1. Motors at pump stations

9.3 Drawings

ABM #404 Foothills 59-1.pdf (52 sheets), 1960.

ABM #583_Foothills 62-8.pdf (49 sheets), 1964.

Appendix A. Soil Resistivity Tests

A.1 Overview

A series of soil resistivity tests were performed throughout Palo Alto, conducted in March and April, 2015.

The tests were done using the Wenner 4-point method. The Wenner 4-point method requires the insertion of four equally spaced in-line electrodes into the test area. Then, a known current from a constant current generator (I) is passed between the outermost two electrodes, and the potential drop (V) is measured across the innermost two electrodes. As $V = IR$, then R (resistance) is directly measured, in ohms. The soil resistivity is then computed as:

$$Rho = \rho = 2\pi RL$$

where R = the soil resistance (ohm) as measured, and L = distance between the two innermost electrodes (in cm).

At each test site, five tests were performed, with L = 2.5, 5, 7.5, 10 and 15 feet.

The tests were conducted using a AEMC 4620 soil resistivity tester. Prior and after testing, the instrument calibration was verified.

According to Roberge (2006), soil resistivity is a function of soil moisture and the concentration of ionic soluble salts. Soil resistivity is considered to be a good indicator of a soil's corrosivity. The lower the resistivity, the higher will be the corrosivity, see Table A-1.

Soil resistivity (ohm-cm)	Corrosivity Rating
>20,000	Essentially non-corrosive
10,000 to 20,000	Mildly corrosive
5,000 to 10,000	Moderately corrosive
3,000 to 5,000	Corrosive
1,000 to 3,000	Highly corrosive
< 1,000	Extremely corrosive

Table A-1. Corrosivity Ratings Based on Soil Resistivity

Soil resistivity generally decreases with increasing water content and concentration of ionic species. Sandy soils are high up on the resistivity scale, and are therefore often the least corrosive. Clay soils, especially those contaminated with saline water, are on the opposite end of the spectrum.

Field soil resistivity measurements are often conducted using the Wenner 4-pin method and a soil resistance meter. The Wenner method uses four metal electrodes, driven into the ground along a straight line, equidistant from each other. Soil resistivity is a function derived from the voltage drop between the center pair of pins, with current flowing between the two outside pins. This method can estimate soil resistivity as follows:

- $Rho = \rho = 2\pi RL$, where L = distance between measuring probes (cm), R = measure resistance (ohms).
- The distance between the measuring probes is set to evaluate the soil resistivity at the depth of the water pipelines. For the most part, water pipes in Palo Alto are buried with about 4 feet of soil above them, putting the spring line (centerline) of the pipe about 4.5 to 5 feet beneath grade. There are likely to be many exceptions where the pipe is shallower or deeper. In part to capture the range of possible pipe burial depth, as well as an indirect method to examine the local variation in soil Rho (and possibly water table and soil type), 5 tests were conducted at each location, time and space permitting. At a few locations, where time and/or space did not permit, then the most important tests would be for a 5-foot depth of pipe.
- For the Palo Alto tests, the distance between measuring probes was set at 80 cm (for 2.5 foot test), 160 cm (for 5-foot test), 240 cm (for 7.5 feet test), 320 cm (for 10 feet test), and 480 cm (for 15 feet test). The resistivity values so-obtained represent the average resistivity of the soil to a depth equal to the pin spacing.
- The indicated resistivity to a depth is the weighted average of the soils from the surface to that depth. Therefore, if the average Rho for all five depth readings is similar to the individual readings, one can assume fairly uniform soil conditions. If the average Rho increases with depth, then the actual top layer Rho is likely to be higher than the average Rho measured for that layer's depth. If the average Rho decreases with depth, this might be indicative of a relatively high water table or a transition to clay-type soils at depth.

The field test data is then processed to obtain layer resistivity. To do this, we used the two-layer methods outlined by Hummel's relationships in Andres and Weibenga (1965). Hummel's model is approximately valid if the assumption is made that there is no flow of current across either of the boundaries, working best when the soil resistivity decreases with depth. For locations where the top few feet are a soil cap, and the ground water table is located several feet beneath the surface (generally the most common case in Palo Alto), this is a reasonable approximation.

A.2 Test Results

Table A-1 presents the results from the tests for a depth of 5 feet. Figure 2-34 shows a map with the results.

- TestIDs 1 through 58 (except 31) were done in Palo Alto.
- TestID 31 was done in Redwood Shores, at a site located near sea level and over young bay mud, known to be highly corrosive to metal pipes. The test results for test 30 (Rho about 1,403 ohm-cm) confirms that site is highly corrosive to metal pipes, and helps confirm the calibration of the test instrument.
- TestIDs 59 through 71 were done in Napa. These Napa tests were done to calibrate the pipeline seismic fragility models used for Palo Alto, which are further discussed in Appendix B.

At two sites, tests were conducted in orthogonal directions. This was necessary as at these sites, the initial tests indicated high current, suggesting that there were metal pipes (or similar) that were oriented parallel to the test. These two tests were excluded from further analysis, as the test data primarily reflects the resistivity of the pipe, and not the soil.

Most test sites were located at parks / open space in Palo Alto where we had ready access. Several tests sites are adjacent to Palo Alto water facilities, like wells, pump stations or reservoirs. We avoided doing any tests on private property. At each site, we set up the test at locations where it was unlikely to have nearby buried metal utilities, and in the direction that would be orthogonal to nearby metal pipes.

Overall. At only a few sites in the Foothills, or very near San Francisquito Creek, did the test data suggest moderate- to non-corrosive soils (Rho much over 5,000 ohm-cm.). At the Hale well site, the test data shows Rho about 6,000 ohm-cm, and this suggests a relatively deep water table at that site (the adjacent creek is about 20 feet below the ground surface at the well), and possibly soils with a high granular content (as might be expected due to the delta of San Francisquito creek). Much of the rest of the flatlands of Palo Alto have alluvial soils with high clay content, which is naturally aggressive. There are many areas with moderately high Rho in the hillside (Foothills) area where there is commonly a thin layer of soil over the Franciscan formation, and deep water table; but locally, Rho can be low in the Foothills.

Test ID	Test Number	Location	Latitude	Longitude	Rho, Ohm-cm
1	1	Ellwood Bike Trail	37.4333	-122.1031	1,312
2	2	Ellwell @ East Bayshore	37.4309	-122.1040	1,275
3	3	Hale Well (999 Palo Alto Avenue)	37.4554	-122.1556	6,015
4	4	Elanor Park Well	37.4511	-122.1432	1,104
5	5	Rinconada Park	37.4445	-122.1413	696
6	5A	Rinconada Park - Orthogonal Direction	37.4445	-122.1413	958
7	6	Heritage Park	37.4433	-122.1573	2,615
8	7	Cogswell Park	37.4463	-122.1629	2,706
9	8	Johnson Park	37.4489	-122.1631	2,055
10	9	782 Palo Alto Avenue (across Road by creek)	37.4538	-122.1590	6,197
11	10	Median on Forest Avenue	37.4527	-122.1486	779
12	11	El Camino Park	37.4450	-122.1688	7,567
13	12	Park @ Embarcadero and Alma	37.4384	-122.1552	1,542
14	13	Bowling Green Park	37.4406	-122.1478	1,979
15	14	Peers Park	37.4318	-122.1464	1,047
16	15	Palo Alto Airport	37.4535	-122.1141	1,489
17	16	West Greenwich Place	37.4401	-122.1363	2,175
18	17	Median, Oregon Expressway @ Agnes	37.4413	-122.1274	2,354
19	18	Bowden Park	37.4294	-122.1409	2,084
20	19	Werry Park	37.4200	-122.1543	1,404
21	20	Mayfield Park	37.4234	-122.1483	1,393
22	21	Fernando Well / Boulware Park	37.4214	-122.1348	1,128
23	22	Matadero Well	37.4173	-122.1361	807
24	23	Bol Park	37.4111	-122.1391	2,296
25	24	Briones Park	37.4073	-122.1274	2,888
26	25	Terman Park	37.4018	-122.1267	3,886
27	26	Monroe Park	37.4082	-122.1168	1,005
28	27	Ramos Park	37.4267	-122.1097	1,459
29	28	Hoover Park	37.4300	-122.1297	845
30	29	Greer Park	37.4435	-122.1195	3,358
31	30	Redwood Shores - Marine Parkway / Twin Dolphin Drive	37.5288	-122.2638	1,066
32	31	Page Mill Road at PG&E Monta Vista - Jefferson 230 kV Power Lines	37.3297	-122.1733	8,016
33	31A	Page Mill Road at PG&E Monta Vista - Jefferson 230 kV Power Lines -	37.3297	-122.1733	9,252

Test ID	Test Number	Location	Latitude	Longitude	Rho, Ohm-cm
		Orthogonal Direction			
34	32	Foothills Park - near Pony Tracks Trail - Gate 4	37.3461	-122.1810	5,652
35	33	Boronda Reservoir	37.3661	-122.1750	2,873
36	34	Boronda Lakeside	37.3641	-122.1768	5,401
37	35	Arastradero Creek Trail	37.3663	-122.1832	1,792
38	36	Top of Alexis Drive	37.3688	-122.1756	1,734
39	37	Arastradero Road, Gate B	37.3807	-122.1825	585
40	38	In Private Development off Los Trancos Road.	37.3698	-122.1920	1,131
41	39	Page Mill Road @ Deer Creek Road	37.3978	-122.1571	2,265
42	40	Esther Clark Park	37.3918	-122.1387	914
43	41	VA Hospital - Miranda Avenue	37.4016	-122.1395	2,984
44	42	Page Mill Road	37.3331	-122.1757	9,736
45	43	Dahl Reservoir	37.3398	-122.1781	1,628
46	44	Park Reservoir	37.3503	-122.1771	6,776
47	45	Monte Bello Reservoir	37.3246	-122.1680	16,639
48	46	Monte Bello Road	37.3277	-122.1690	5,569
49	47	Boronda Booster Station	37.3557	-122.1787	7,248
50	48	Wild Horse Creek Trail	37.3589	-122.1824	10,186
51	49	Foothill Park Interpretive Center	37.3632	-122.1861	5,747
52	50	Arastradero Creek Trail	37.3698	-122.1839	1,390
53	51	Arastradero Creek Trail	37.3774	-122.1797	2,570
54	52	Corte Madera Booster Station	37.3816	-122.1778	2,266
55	53	Arastradero Road / Bautista Trail	37.3853	-122.1730	2,160
56	54	Arastradero Road /Page Mill Road	37.3845	-122.1659	834
57	55	Page Mill Road @ I 280 next to Caltrans yard	37.3912	-122.1634	879
58	56	Old Page Mill Road	37.4002	-122.1597	14,780
59	Napa 1	American Canyon - close to Highway Substation	38.1668	-122.2534	687
60	Napa 2	Dead end at Broadmoor, somewhat near Hacienda Water Tank	38.3165	-122.3474	2,226
61	Napa 3	Morningside at Maidu Court	38.3067	-122.3500	1,202
62	Napa 4	end of Stonybrook and across Bridge near creek	38.3049	-122.3471	1,302

Test ID	Test Number	Location	Latitude	Longitude	Rho, Ohm-cm
63	Napa 5	Redwood Road	38.3156	-122.3389	2,921
64	Napa 6	Open Yard behind strip mall in western Napa	38.2889	-122.3095	2,369
65	Napa 7	Health Center	38.2880	-122.2982	1,415
66	Napa 8	Fuller Park	38.2930	-122.2935	942
67	Napa 9	Napa County Fair Grounds Office Building	38.2989	-122.2789	1,802
68	Napa 10	End of Valle Verde in Park	38.3310	-122.2998	3,022
69	Napa 11	Dry Creek Picnic Area	38.3212	-122.3301	1,918
70	Napa 12	Stone Bridge School (Los Carneros)	38.2569	-122.3228	923
71	Napa 13	Mannering Near Hilltop Drive	38.2925	-122.3067	745

Table A-1. Soil Layer Resistivity Test Data

A.3 Reference

Andrews, T. and Weibenga, W.A., Two-Layer Resistivity Curves for the Wenner and Schlumberger Electrode Configurations, Bureau of Mineral Resources, Geology and Geophysics, Record No. 1965/18, Commonwealth of Australia.

Hummel, J. N., A theoretical study of apparent resistivity in surface potential methods, in Trans. Amer. Inst. Min. Metal Engrs, 97, pp 392-422, 1932.

Roberge, P.R., (Ed.), Corrosion basics: An Introduction, Second Edition, NACE Press, 2006.

Appendix B. Seismic Performance of Water Pipelines in the 2014 Napa Earthquake

Seismic Performance of Water Pipelines in the 2014 Napa Earthquake

Acknowledgements. Appendix B was prepared by G&E Engineering Systems Inc. Drafts of this document were reviewed by Mr. Bruce Maison (EBMUD, Retired), and Prof. Mike O'Rourke (RPI). Mr. Maison and Prof. O'Rourke both participated in developing the ALA (2001) Water Pipeline Fragility report. They report that the rate of water pipeline damage, as forecast using the seismic hazards and pipe fragility models in this report, appear reasonable. Ms. Joy Eldredge, General Manager of the City of Napa City Water Department, provided pipe damage statistics. Several images in this document were provided by the City of Napa.

B.1 Ground Shaking

At 3:20 am, Pacific time, on August 24, 2014, an earthquake occurred near the city Napa, California. Figure B-1 shows a map of the region, with major places and populations, along with a map¹ of the level of ground shaking.

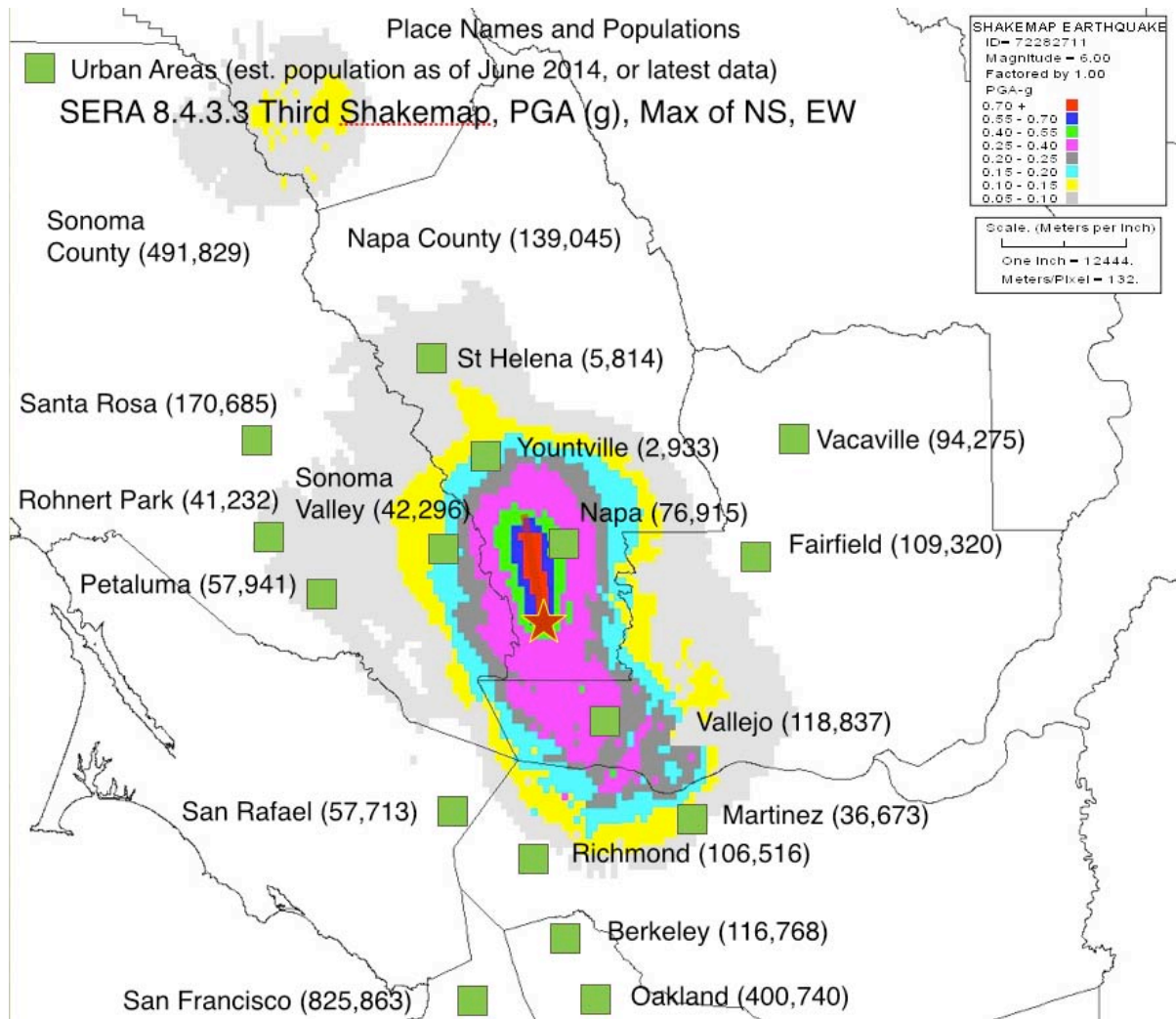


Figure B-1. Place Names and Populations, SERA ShakeMap-v3 (PGA)

In Figure B-1, the bright red north-south trending line corresponds to the surface rupture of the earthquake; the red star shows the location of the epicenter of the earthquake (latitude 38.22, longitude -122.31, depth 11.3 km); the green squares represent the major population centers (population in brackets).

The colors of the shaking in Figure B-1 correspond to:

¹ All ShakeMaps in Appendix B were prepared using SERA (System Earthquake Risk Assessment, © G&E Engineering Systems Inc., 2015. Figure B-1 shows PGA, maximum of NS and EW components, based on a ShakeMap using 665 recording instruments.

- Red. PGA $\geq 0.70g$
- Blue. PGA = 0.55g to 0.70g
- Green. PGA 0.40g to 0.55g
- Magenta. PGA 0.25g to 0.40g
- Dark Grey. PGA 0.20g to 0.25g
- Cyan. PGA 0.15g to 0.20g
- Yellow. PGA 0.10g to 0.15g
- Light Grey. PGA 0.05g to 0.10g
- White (no color). PGA $< 0.05g$

B.2 Geologic Conditions and Liquefaction

Figure B-2 shows a geologic map of the Napa area. This geologic map was prepared by Sowers et al (1998).

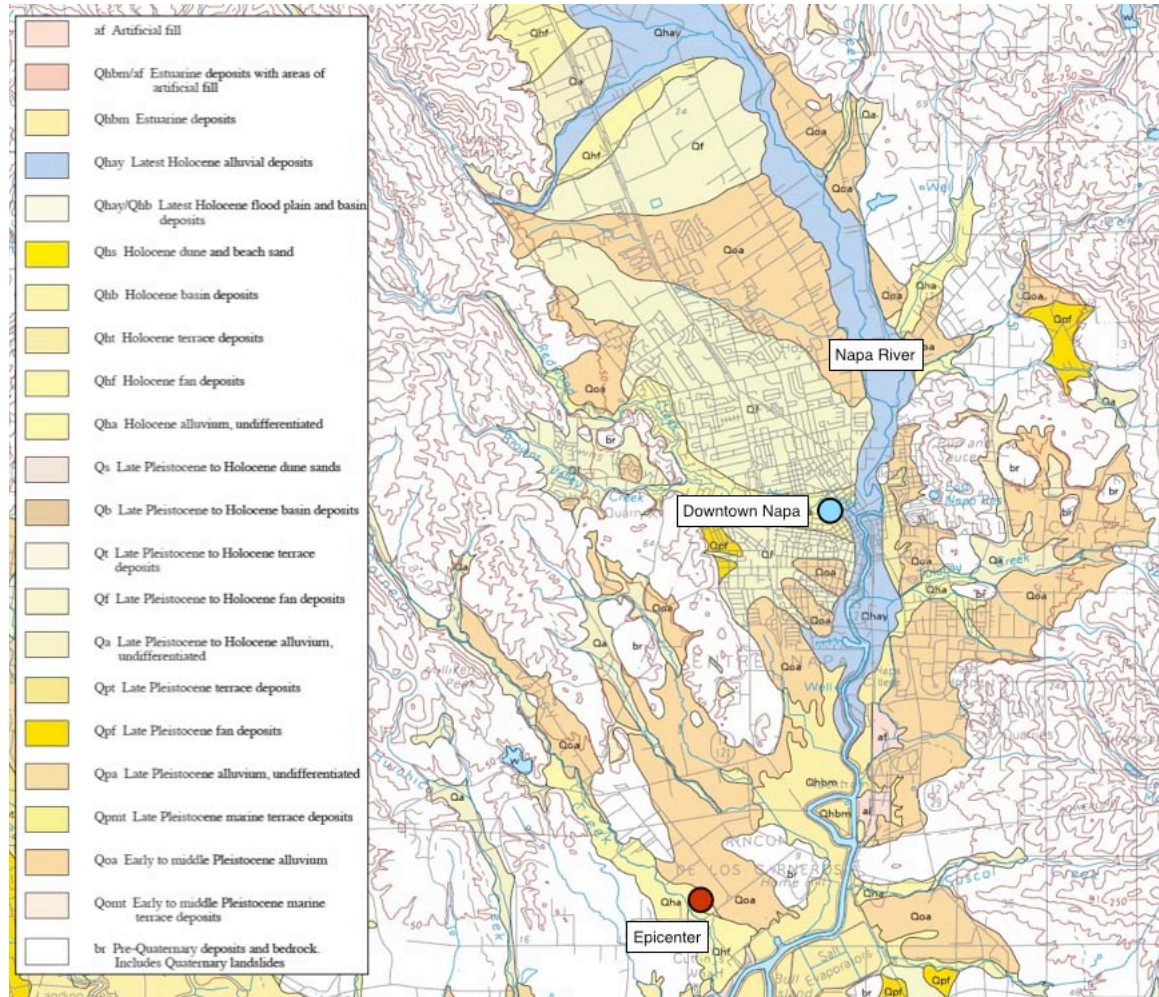


Figure B-2. Geologic Map (After Sowers, 1998)

In preparing the map, Sowers et al were particularly concerned by the geologic conditions of greatest concern that can lead to liquefaction:

- Holocene estuarine deposits
- Holocene stream deposits
- Eolian sands
- Artificial fills

In developing this geologic map, Sowers et al first interpreted aerial photographs and topographic maps to determine the depositional environment and to estimate relative age by evaluating landforms and geomorphic relationships. Second, published soil survey data (dated 1956 through 1985) was reviewed to assess the character and age of near-surface deposits.

Figures B-3 and B-4 show the liquefaction susceptibility map for the San Francisco and Napa areas (after Knudsen 2000), respectively. In these maps, red = very high; pink = high; yellow = moderate; green = low; grey = non / very low; white = none / not mapped; heavy red line = West Napa fault location.

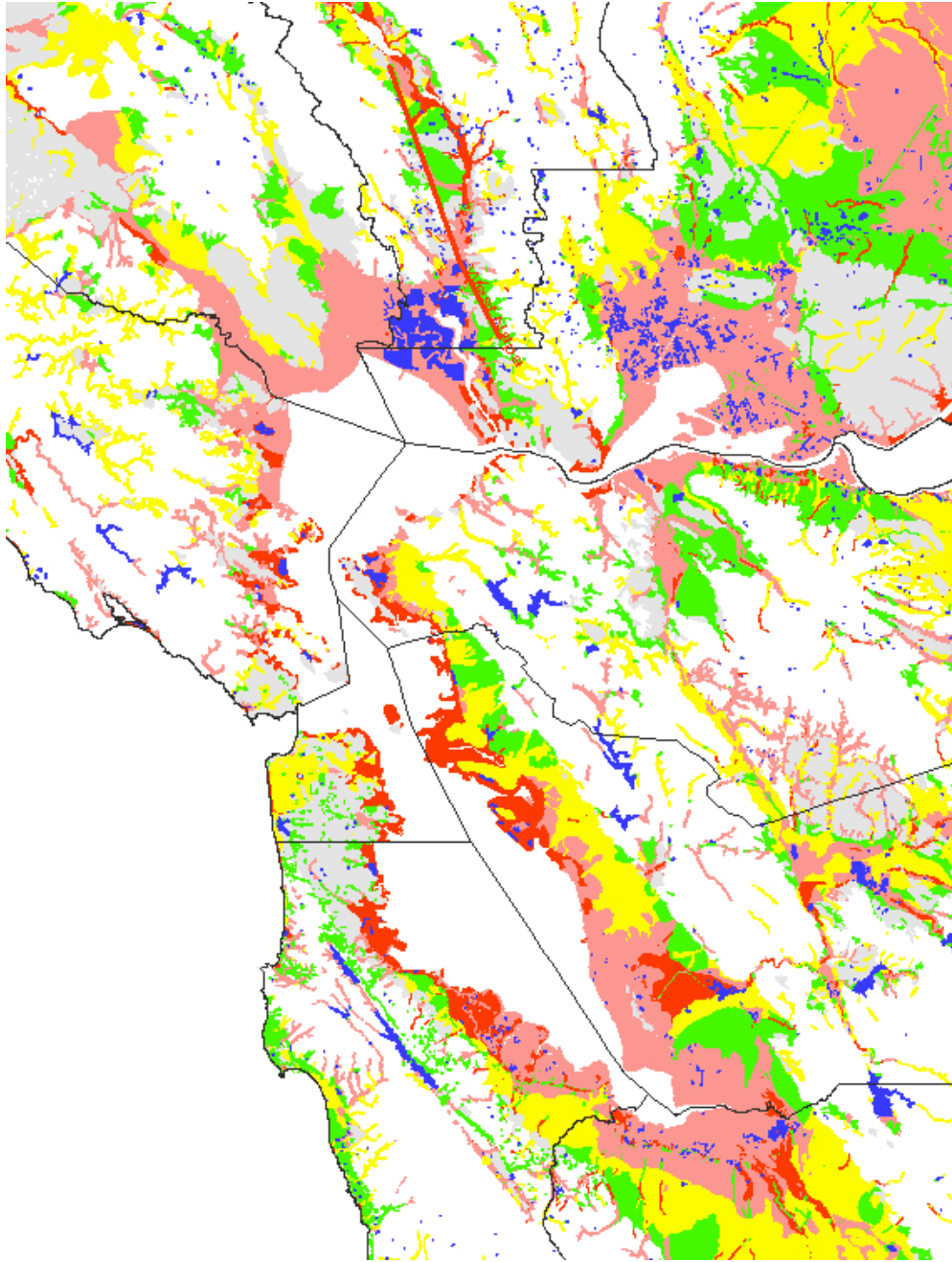


Figure B-3. Liquefaction Susceptibility Map, San Francisco Bay Area

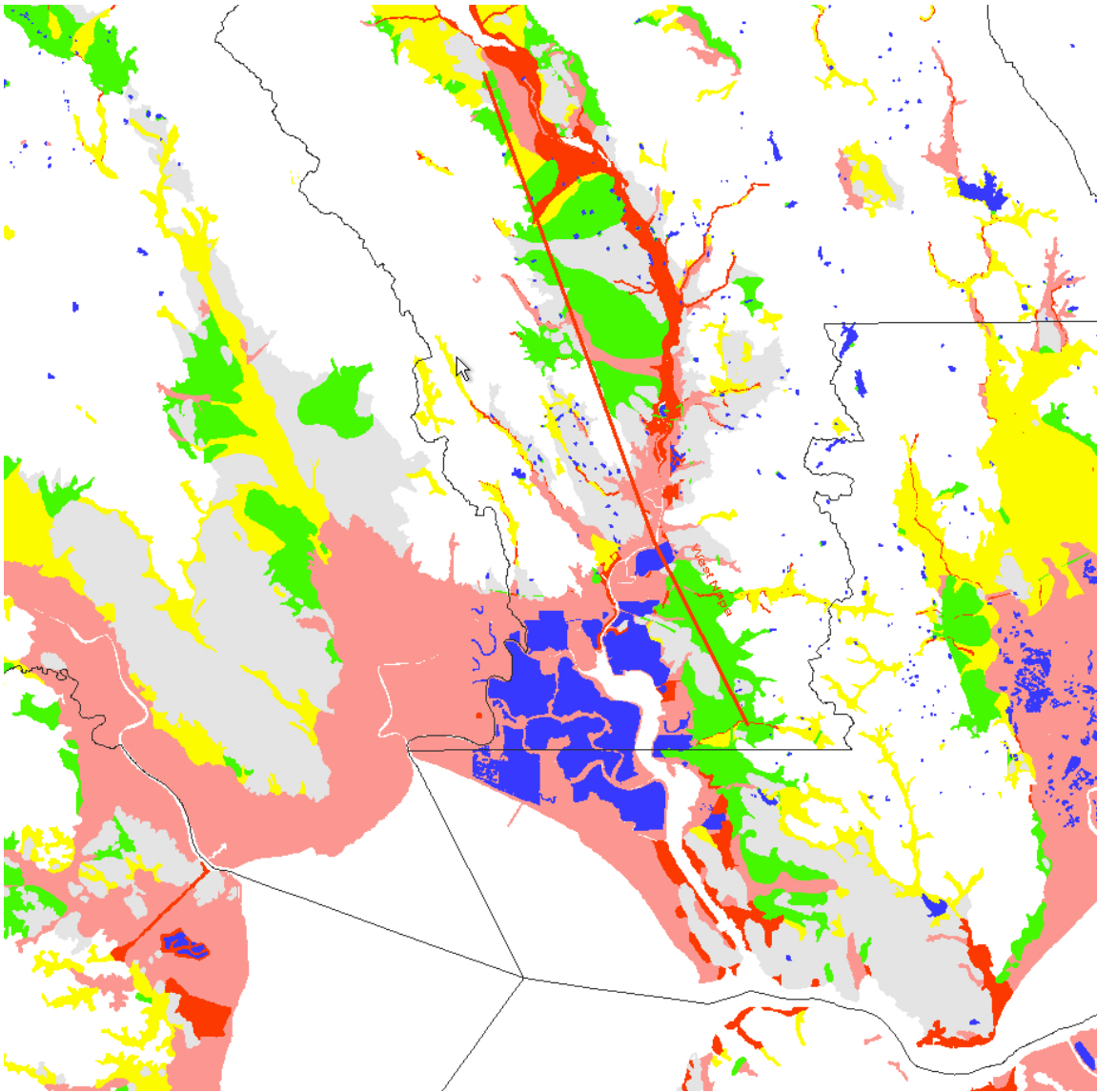


Figure B-4. Liquefaction Susceptibility Map, Napa and Sonoma Valley Area

In Figure B-4, the heavy red line shows the location of the West Napa fault used to develop the USGS national seismic hazard maps through 2012. As will be described later in this report, this heavy red line does not correlate exactly with the observations of the surface location of the fault, as seen in the actual August 2014 earthquake; at some locations, the heavy red line is 2 km distant from the surface rupture; at other locations, the heavy red line is nearly coincident with the actual surface rupture.

In 2006, Witter (Witter et al, 2006) updated portions of Knudson's (2000) regional liquefaction susceptibility map. Underlying the 2006 Witter Liquefaction Map (Figure B-5) is a version of Sowers 1998 map (Figure B-2). Figure B-5 shows a portion of the liquefaction susceptibility map (Witter 2006) along the west side of the City of Napa, showing both the liquefaction susceptibility (red / orange / yellow / green for very high, high, moderate, low, as per Figure B-4), as well as the mapped geologic unit. Full scale in Figure B-5 is 1 km.

Figure B-5 shows a close-up of the liquefaction map for the west side of Napa where there was a high concentration of water pipe repairs.

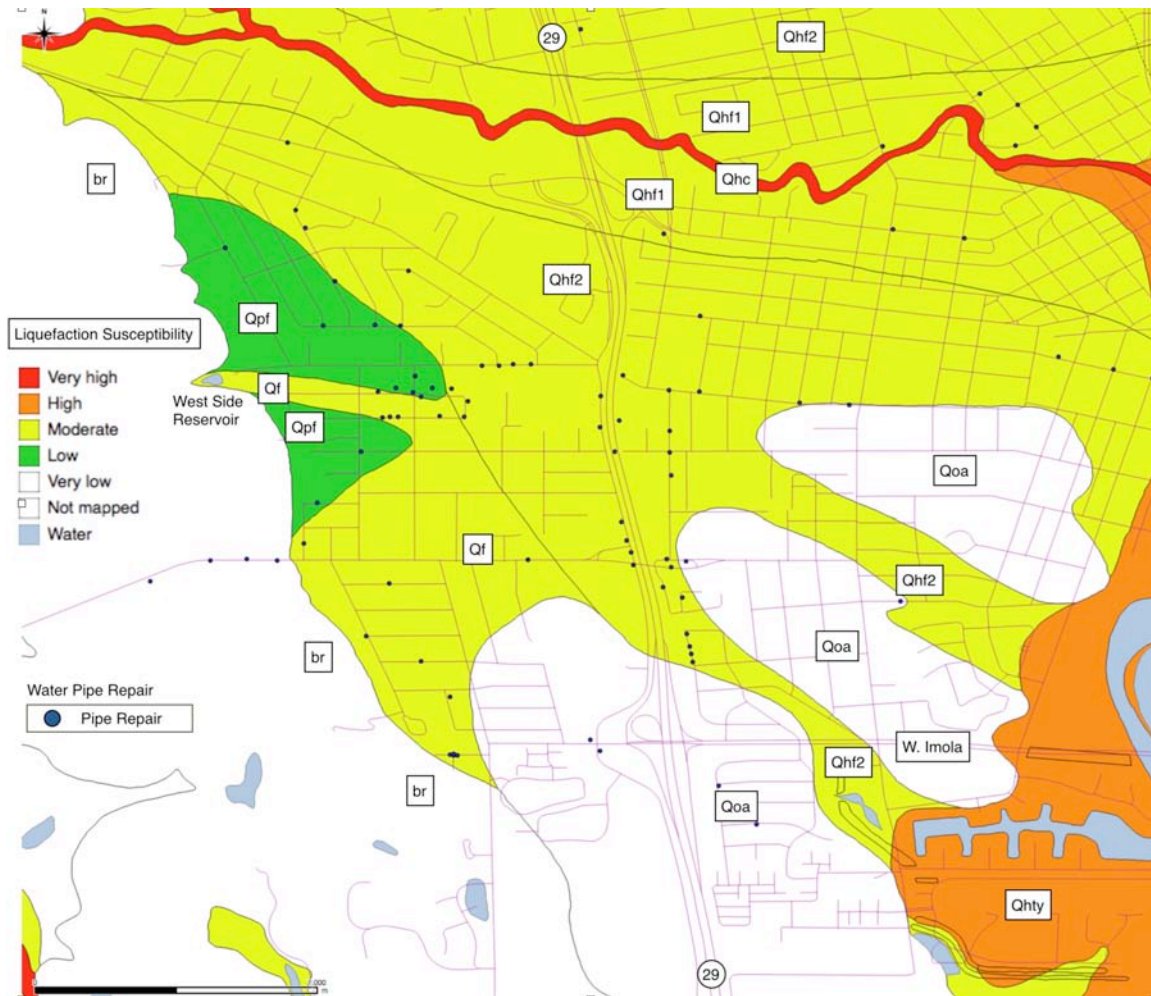


Figure B-5. Pipe Repair, Liquefaction Susceptibility and Geologic Map, West Side of Napa

The geologic units in Figure B-5 are listed in Table B-1.

Geologic Unit	Description
br	Bedrock. Generally Jurassic to Pliocene. This units may include landslides, colluvium, small stream channel deposits that are not delineated at the map scale.
Qpf	Alluvial fans, Pleistocene age. Low liquefaction susceptibility
Qf	Gently sloping, fan shapes, undissected alluvial surfaces with unknown age (could be Holocene or latest Pleistocene), or with thin patches of Holocene over latest Pleistocene. Mapped liquefaction susceptibility is Moderate, but this presumes a ground water table from 10 to 30 feet; where ground water table is under 10 feet, liquefaction susceptibility is generally High.
Qoa	Early to late Pleistocene alluvial deposits. Liquefaction susceptibility low where ground water table is < 10 feet, very low where ground water table is >10 feet
Qhf2	Older Holocene alluvial fan deposits. Liquefaction susceptibility is moderate where ground water depth is < 15 feet or where local streams are not mapped
Qhf1	Younger Holocene alluvial fan deposits. Liquefaction susceptibility is moderate where ground water depth is < 15 feet or where local streams are not mapped
Qhc	Modern stream deposits. Historical (<150 years) deposits.
Qhty	Latest Holocene (< 1,000 years). Stream terraces deposited as point bar and overbank deposits by the Napa River. Include sand, gravel, silt and minor clay. Liquefaction susceptibility judged high based on abundance of sandy, cohesionless sediment, high ground water level, presence of free-face at banks

Table B-1. Geologic Units

By examining Figures B-2, B-3, B-4 and B-5 and Figure B-1 (PGA), one might expect that perhaps 20% to 40% of the areas mapped as being Holocene (young) and exposed to $PGA > 0.3g$, with the shallowest ground water tables, should have shown evidence of liquefaction. While a team of geologists from GEER (GEER 2015) found few widespread effects of liquefaction in the region, field work by Bruce Maison and John Eidinger on May 7 2015 showed clear evidence of settlements and minor lateral spreads in the alluvial fan areas produced from the West Side Reservoir. These same areas had a very high rate of water pipe damage in the earthquake, followed up by many further breaks in the few months following the earthquake. Continued movement of liquefied soils is a not uncommon phenomena. It is evident that the areas mapped as Qf (downstream of the West Side Reservoir) and Qhf2 (the same alluvial fans created from the drainage feeding the West Side Reservoir) with highest ground water depths did liquefy, and that largely explains the cluster of non-seismically-designed (mostly Cast Iron) water pipe damage in that area.

With regards to liquefaction, the following can be surmised:

- The low magnitude of this earthquake (M 6.0) results in relatively short duration of very strong shaking ($PGA > 0.2g$). Liquefaction in moderate to high susceptible soils tends to be triggered at $PGA > 0.15g$ at a minimum (sometimes $PGA > 0.3g$), and the short duration results in small settlements (volumetric strains) or lateral spreads, even where liquefaction occurs. In other words, we expect that the areas that high concentration of non-seismic pipe damage in Figure B-5, will have even higher concentrations of pipe damage in future higher magnitude earthquakes.
- There was evidence of liquefaction along the banks of the Napa river; in this area, the liquefaction maps in Figures B-3 and B-4 are reasonable. There (has yet) been little to no evidence of liquefaction (sand boils, lateral spreads) in the pink (high) zones north of San Pablo Bay; these areas should probably be re-classified as "moderate"; or use a soft strain PGD model in these areas rather than a granular material liquefaction model in these areas. As there are no water pipes owned by the City of Napa in these areas, these implications are not relevant in context of Appendix B.

On May 7 2015, John Eidinger and Bruce Maison field inspected area in Napa that had concentrated clusters of water pipe breaks. Figure B-6 shows a map of the pipe break cluster along Hilltop Drive and Mannering Street and Cypress Streets, up to the West Side Reservoir.

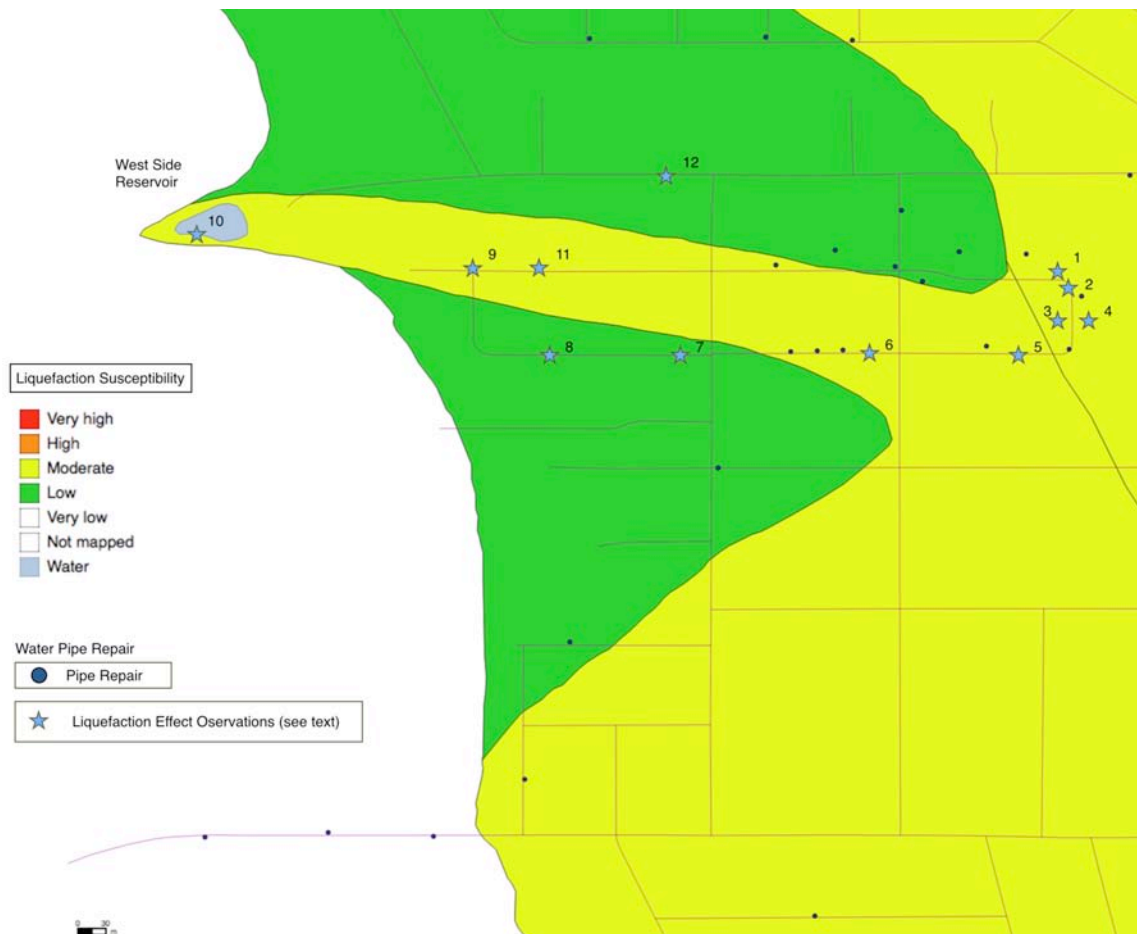


Figure B-6. Liquefaction Effects, West Side Reservoir to Mannering Streets

- Location 1. The road has been re-surfaced after the original water pipe breaks (dots in Figure B-6) were repaired. Subsequently, new water pipe repairs were made. The scatter of the dots in Figure B-6 (off-road) reflects slightly inaccurate geo-coding.
- Location 2. The road has been re-surfaced after the original water pipe breaks (dots in Figure B-6) were repaired. Subsequently, new water pipe repairs were made.
- Location 3. Severe cracking in unreinforced concrete slab-on-grade patio. Cracks are commonly one-half inch wide, spaced 3 to 5 feet apart.
- Location 4. Severe cracking in unreinforced concrete slab-on-grade driveway. Cracks are commonly one-half inch wide, spaced 3 to 5 feet apart. Local ground water depth is under 5 feet. Original water outage was "several days". Homeowner confirms that he shut off the natural gas pipe, and as soon as PG&E crews came by to re-light, he had gas again. PG&E confirmed no pipe breaks in their gas distribution system, anywhere in Napa; common PG&E distribution pipes are fused medium density polyethylene.

- Location 5. The road has been re-surfaced after the original water pipe breaks (dots in Figure B-6) were repaired. Subsequently, new water pipe repairs were made.
- Location 6. The road has been re-surfaced after the original water pipe breaks (dots in Figure B-6) were repaired. Subsequently, new water pipe repairs were made.
- Location 7. Slight uphill grade (about 1%). Cracks in unreinforced concrete driveways.
- Location 8. Slight uphill grade (about 1%). Buckled sidewalk.
- Location 9. Rotated sidewalk curb.
- Location 10. West Side Reservoir. (see Figure B-7). This "reservoir" is in fact a debris basin, design to catch intense rainfall run-off from the hills immediately to the west. As observed in May 2015, the concrete-lined basin was empty. It is assumed that prior to modern development in the c. 1950s, the drainage formed the oblong-shaped "moderate" Qf polygon shown in Figures B5 and B-6. Local home owners confirmed that within that polygon, the groundwater table is " < 5 feet, near the surface), whereas just a few hundred feet away (outside the Qf polygon), the ground water table is "much deeper".
- Location 11. Slight uphill grade (about 1%). Rotated sidewalk curb.
- Location 12. No damage to concrete curbs or other discernibly effects from permanent ground movement observed along this portion of the street (mapped Qpf, older Pleistocene alluvial).



Figure B-7. West Side Reservoir

Based on the field observations at Location 1 to 12, we feel that the common range of liquefaction effects in this area was on the order of 1 to 2 inches of differential settlement (clear evidence, along Mannering Street and the lower elevations of Hilltop Drive and Cypress Streets), or more subtle PGDs in the sloped areas. The geologic mapping of the Qf zone is possibly off by a few tens of feet along the lower elevations of Hilltop Drive. Where the depth of groundwater in the Qf zone was shallow (under a few feet), liquefaction did occur.

The count of pipe repairs in this cluster is 13, and we think nearly all of these are best attributable to PGDs on the order of 1 to 2 inches.

Due to time constraints, we were able to do "drive-by" surveys of other pipe break clusters in the former creek / alluvial fans that are downhill from this Qf zone. We did observe more sidewalk cracking of the nature described above, although somewhat less frequent. Of the ~23 pipe repairs in these zones, we think perhaps 10-15 of these are at locations with evidence of PGD settlements on the order of about an inch.

B.3 Surface Faulting

The West Napa fault is considered to be a Late Pleistocene and Holocene active dextral strike-slip fault, generally located along the western Napa Valley. Prior to the 2014 earthquake, detailed reconnaissance-level mapping existed for most of the fault.

The West Napa fault and its branches were first mapped by Weaver (1949). Wesling and Hanson (2008) presented a summary of the then-known information for the West Napa fault, along with updated maps.

It is now known that the M 5.0 Yountville earthquake of September 3 2000, originally ascribed to have occurred on the West Napa fault, in fact occurred on an unnamed fault about 5 km to the west of the West Napa fault.

In the literature from 1949 to 1999, the West Napa fault is often described in two sections. Holocene slip rate and recurrence interval data had not been determined in the literature for this fault. Several site-specific studies (Hart and Bryant 1997) documented the located and approximate age of most recent faulting, but detailed paleoseismic investigations had not been done.

- The northern section, often called the Browns Valley section, is delineated by a zone of north-northwest striking late Pleistocene faults that generally lack geomorphic evidence of Holocene displacement (Bryant 1982). The 2014 event ruptured this section, and the fault is clearly active.
- The southern section, named the Napa County Airport section, is delineated by northwest striking dextral-slip faults that exhibit geomorphic evidence of Holocene displacement (Helley and Herd, 1977, Bryant 1982). The 2014 event ruptured this section, and the fault is clearly active.

Wesling and Hanson updated the prior section names, and divided the West Napa fault into 5 reaches, Figure B-8a. From north to south, they named the reaches:

- St. Helena Dry Creek
- Yountville to North Napa
- North Napa to Napa River (this section broke in 2014)
- Napa River – American Canyon. This section was known to be Holocene Active, and was designated as a Alquist-Priolo Earthquake Fault Zone.
- American Canyon to Carquinez Strait

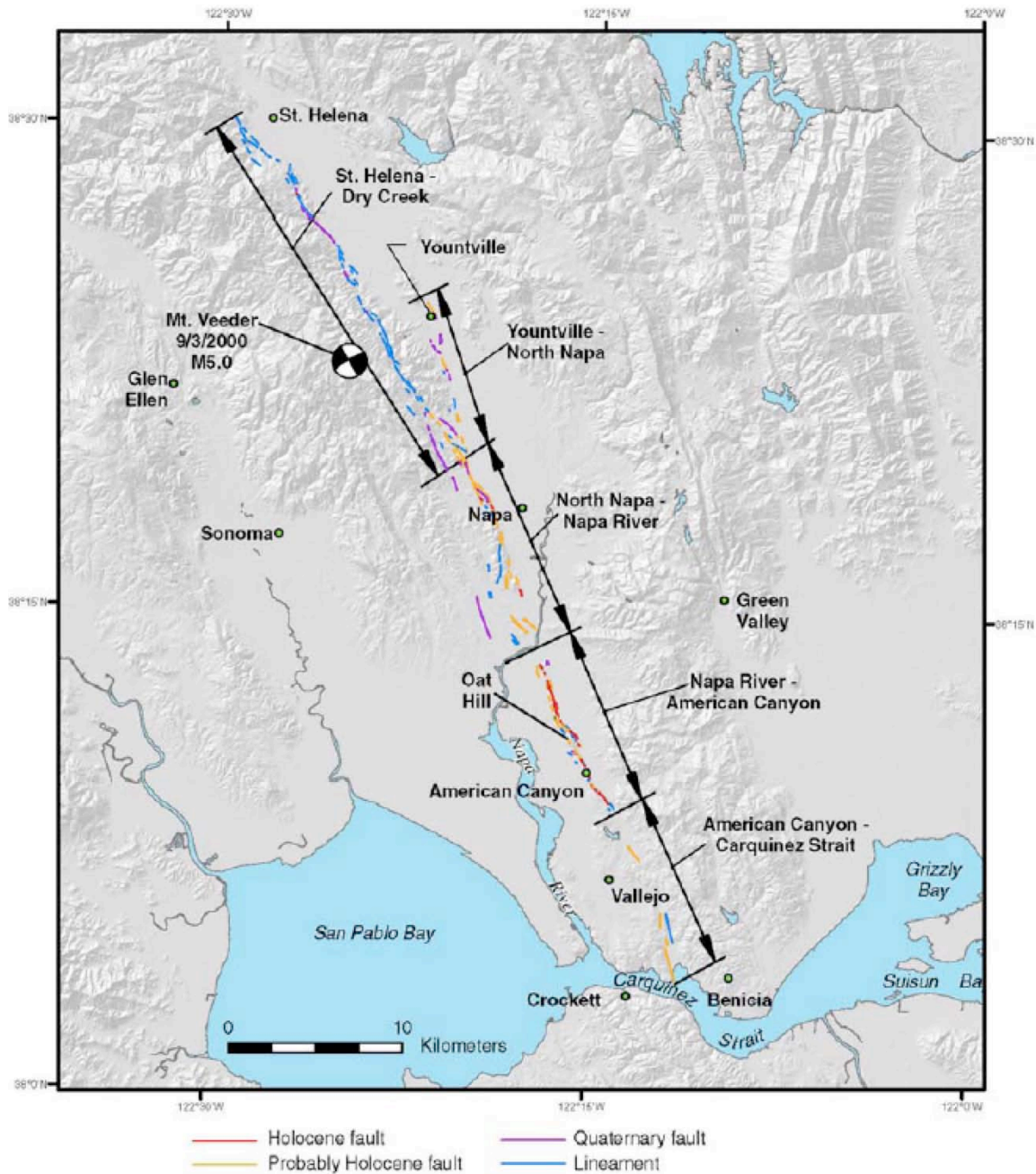


Figure B-8a. Map Showing Quaternary-active faulting and reaches along the West Napa fault zone (after Wesling and Hanson, 2008)

Prior to the 2014 earthquake, the West Napa fault had been assigned a long term slip rate of 1 mm/year, ± 1 mm/year; Wesling had suggested a higher slip rate, suggesting they there was evidence of surface rupture on the order of 600 to 700 years ago, and possibly a slip rate up to 4 mm/year.

Surface faulting in the 2014 earthquake actually occurred at many locations to the west of downtown Napa. Figure B-8 shows a map that highlights the prior mapped locations of

lineaments in the West Napa fault zone (green and blue lines) and field observations of surface faulting (red dots).

David Schwartz of the USGS reports that the locations of surface faulting in 2014 (red dots in Figure B-8b) do not correspond closely to the prior mapped locations of the West Napa fault (green and blue lines in Figure B-8b).

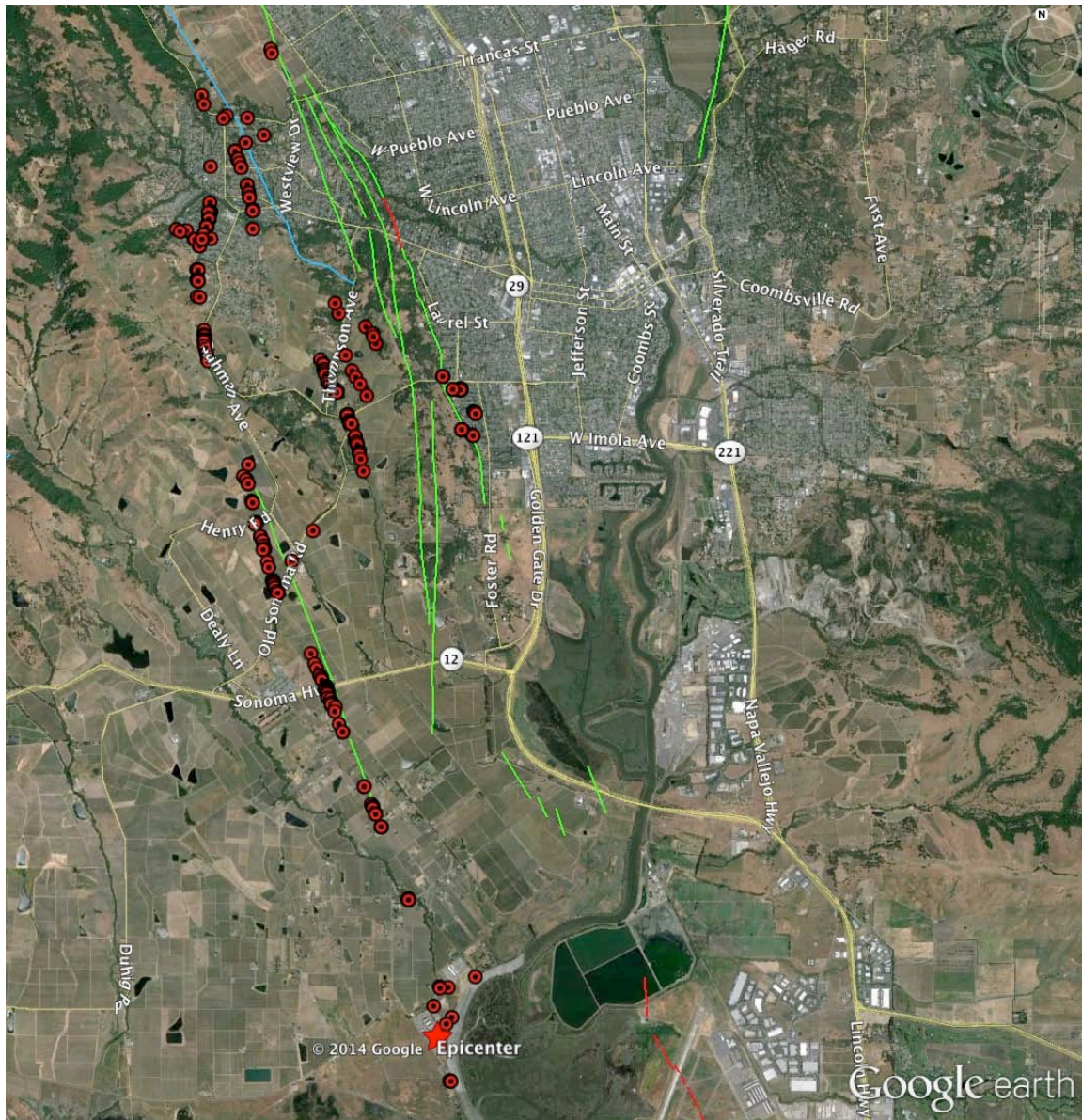


Figure B-8. Map of Surface Faulting Observations (Red Dots)

Figures B-9 and B-10 show buckled sidewalks at two of the fault crossing locations (red dots in Figure B-8b). These figures show two of many such sidewalk buckles in the west part of Napa near Meadowbrook Drive. In this area, there were about 8 streets, running nearly perpendicular to the strike of the fault. At each street, there was commonly one or two sidewalk buckles (one each side of the street), and if one maps the location of the

buckles, one gets a more-or-less straight line of red dots, as shown in the top left part of Figure B-8. Inspecting these buckles, one observes that there was some right lateral movement, as well as possibly some thrusting movement at the surface, resulting in net compression in the sidewalk slabs. This compression then results in the uplift of two adjacent concrete slabs as seen in Figures B-9 and B-10. Underneath the ground, the ground movements are also applied to buried water pipes, and this will be further described later in Appendix B.



Figure B-9. Buckled Sidewalk at Fault Offset Location



Figure B-10. Buckled Sidewalk at Fault Offset Location

B.4 City of Napa Water System

The City of Napa was incorporated in 1872. Figure B-11 shows that the City's water service area contains three boundaries:

- Designated water service area, which includes most of lower Napa Valley and the city of Napa (blue line),
- Rural Urban Limit line (RUL) (orange line).
- City limits (red line, nearly the same as the orange line).

The majority of potable water delivery is within the RUL (orange line). Outside the RUL, some potable water is delivered to the Monticello / Silverado community, the Congress Valley Water District and to agricultural customers along the Conn Transmission Main.

The City also exports water to the Cities of American Canyon (daily), St. Helena (daily), Calistoga (daily), Yountville (rare) and the California Veterans Home (rare). By "daily", it is meant that this is normally done. By "rare" , it is meant that the infrastructure is in

place to deliver water, but those communities have separate water supplies that are used normally.

Water demand peaks at about 25 MGD during a hot spell in July and drops to about 7 MGD during the winter months. Landscape irrigation represents about half of yearly water demand. All potable water is provided from three surface water sources; no ground water is used. Of the total demand, about 53% is single family residential, 16% is multi-family residential, commercial is about 15%, institutional about 7%, landscape about 5%, St. Helena about 2%, agricultural about 1%, construction about 0.3%.

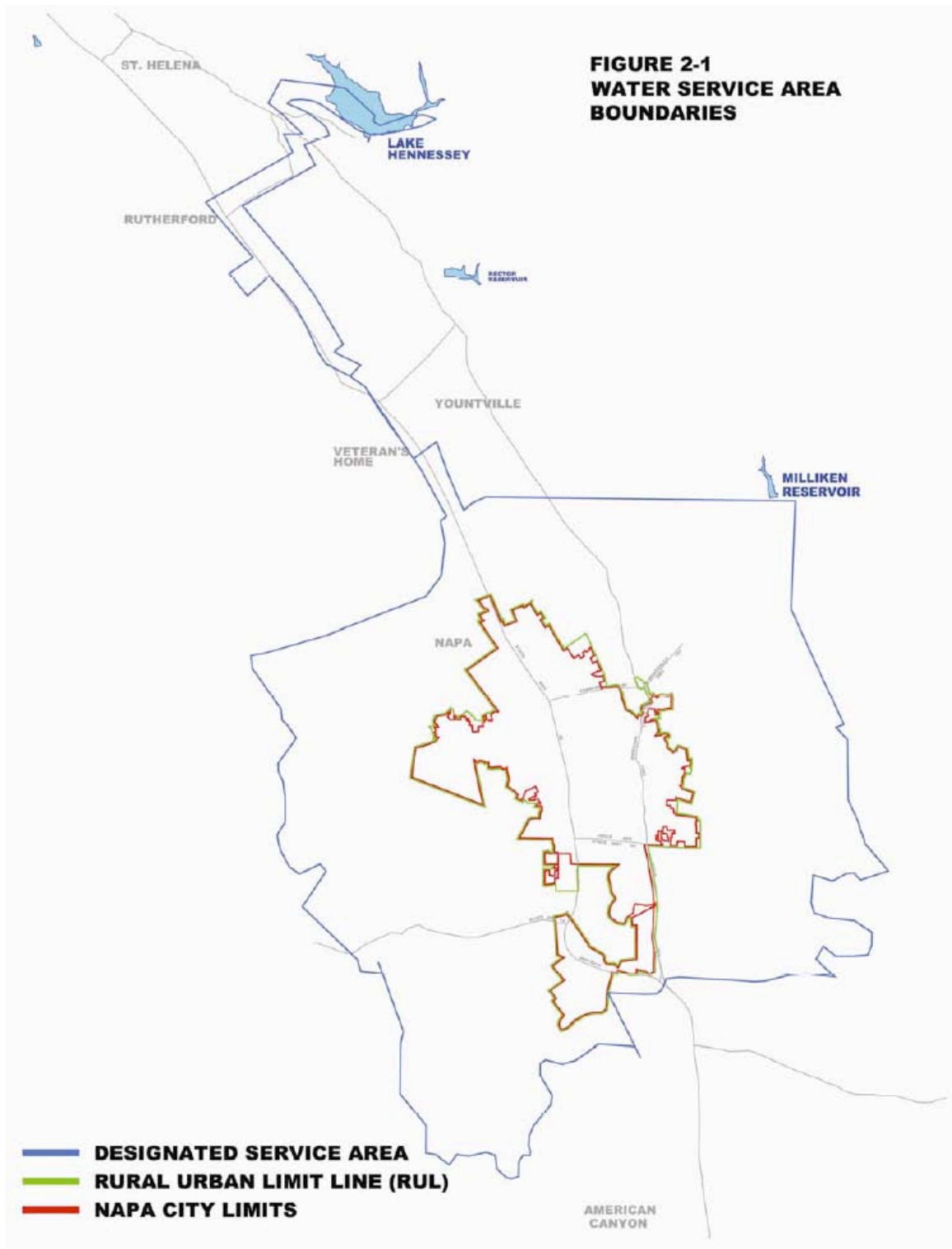


Figure B-11. Napa Water System Service Area

Figure B-12 shows the locations of the three water treatment plants serving Napa, as well as the major transmission pipelines in the water system.

- **Lake Hennessey.** Located about 14 miles north of the City of Napa water system. The lake was created by building Conn Dam in 1946. The Hennessey WTP began operations in 1981, with nominal treatment capacity of 20 MGD. PGA from this earthquake was about 0.1g. Facilities includes a circular concrete intake tower, flash mixing, coagulation, flocculation, sedimentation, filtration and disinfection. Treated water is stored in a buried 5.0 million gallon concrete clearwell tank on site. This treated water is delivered to the distribution system via a steel pipeline, 36-inch diameter Conn Transmission Main, about 20 miles long. This transmission pipeline suffered no damage.
- **Milliken WTP.** Prior to 1946, this was Napa's sole water supply. Milliken Dam was built about 1923. Normally, this WTP is used only during summer months. The storage capacity of the reservoir is limited due to seismic stability concerns of the dam. Raw water is diverted from Milliken Creek about 2 miles downstream of the dam, to the WTP, via a one-mile-long 16-inch diameter above ground raw water pipe. The current WTP was built in 1976, with a capacity of 4 MGD. It is a direct filtration plant, with a contract/reaction tank and four horizontal filters. Treated water is stored in a 2.0 million gallons clearwell tank. Water from the clearwell tank is delivered to the distribution system via the 3-mile-long Milliken Transmission Line. PGA about 0.15g. Milliken did not come on-line in response to the earthquake.
- **Jamieson Canyon.** PGA about 0.15 – 0.20g. Raw water for this WTP comes via the State Water Project's North Bay Aqueduct. The SWP diverts water from the Sacramento-San Joaquin Delta at Barker Slough pumping plant east of Fairfield (PGA < 0.05g) and the NBA moves the water about 21 miles westward to Cordelia Forebay; from there, the SWP water is pumped an additional 6 miles to two NBA terminal 5 MG reservoirs (built 2008) located at the WTP. The WTP provides potable water to the cities of Napa, Calistoga and American Canyon. Raw water from the WTP can also serve agricultural customers in American Canyon. Facilities includes a 5 MG tank. The WTP was originally constructed in 1968, expanded to 12 MGD in 1988, and expanded again in 2011 to 20 MGD. The WTP includes pre-and intermediate-ozonation, as well as rapid mixing, flocculation, sedimentation with tube settlers, gravity filtration, and disinfection. There is a 5 MG clearwell. The 42-inch diameter Jamieson Transmission Line delivers potable water to the City, splitting into 36-inch diameter and 24-inch diameter lines near the intersection of Highway 29 and 221. After this split, the west-side transmission pipeline is a rubber gasketed asbestos cement pipe; this pipe did not fail in the first week after the earthquake; but did suffer 3 leaks during the months afterward.

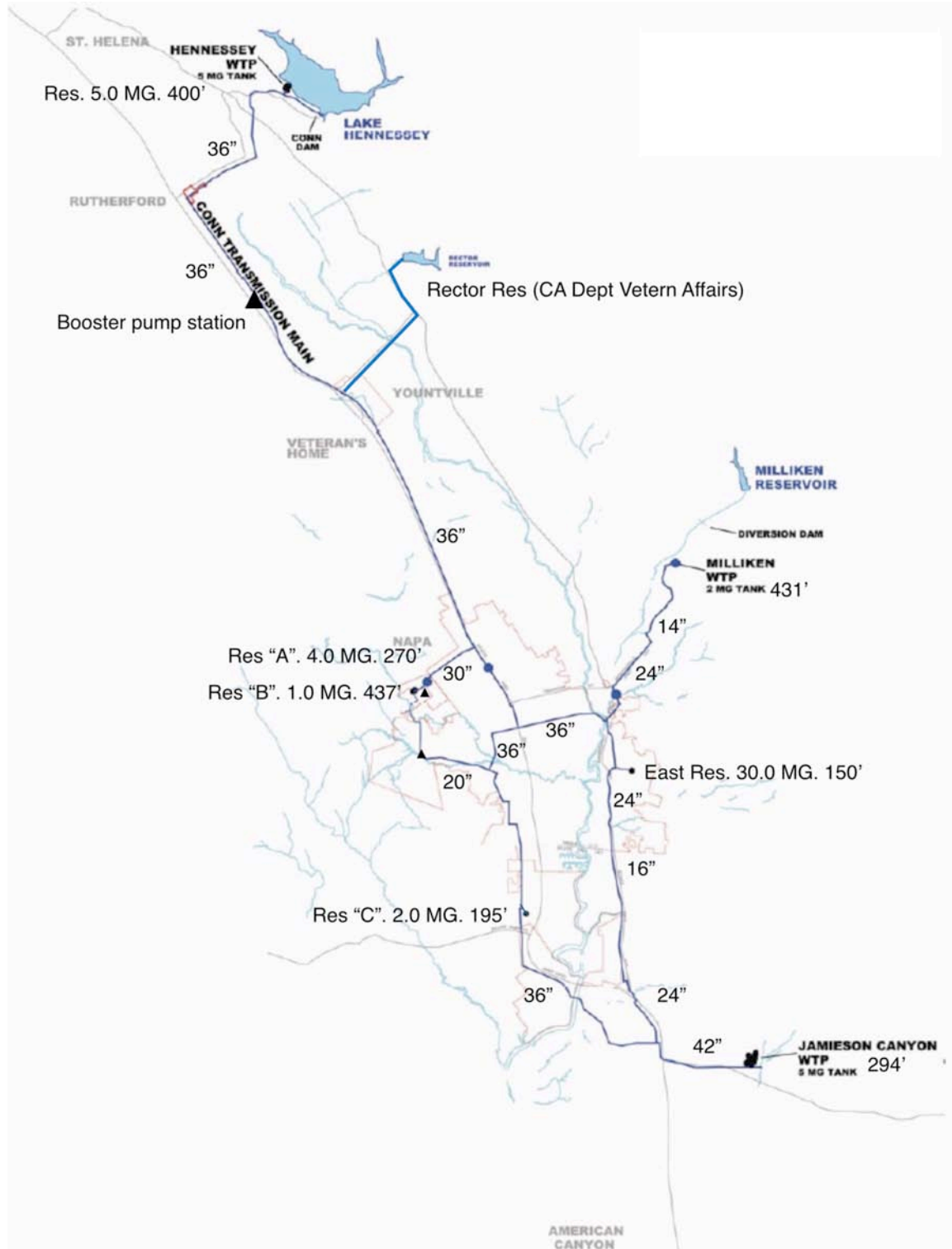


Figure B-12. Napa Water System Major Pipelines and Location of Water Treatment Plants

The Napa Water Department (NWD) serves a population of about 84,000 people, via 25,000 services. NWD has 51 employees. The water system includes 3 water treatment plants, about 380 miles of pipe, 12 storage tanks with a total of 30 million gallon storage, 9 pump stations, 14 pressure regulating stations and is operated in 5 pressure zones. The 9 pump stations serve about 10% of the population; and about 90% of the populations is served by gravity flow coming from the water treatment plants.

As of September 30, 2014, Napa reported about 172 water pipe repairs. Through November 15, 2014, Napa reported 185 pipe repairs. Through late January 2015, Napa reported having completed 243 pipe repairs. By late January 2015, Napa reported that the repair rate for water pipes had reduced to "about" the long term average repair rate.

Table B-2 lists the lengths of water pipes in the Napa water system (2012 data). Napa reports that in a typical year, there are about 80 to 100 pipe leaks in the city, system wide (about 0.21 to 0.26 repairs per mile per year). This leak rate for water pipes is consistent with industry average (about 0.24 to 0.27 leaks per mile per year). Even so, other water departments in California with primarily cast iron pipe of similar vintages as in Napa (but with non-aggressive soils) have system wide leak rates on the order of 0.06 leaks per mile per year. A tentative finding is that the rate of water pipe damage in earthquakes will tend to be relatively high, if the historical leak rate is also high.

Age (years)	PVC	DI	CI	AC	RCCP	STL	Total	Pct of Total
< 20	6,600	225,600				100	232,300	13%
20-40	24300	370,500	83,400	14,100		100	492,400	28%
40-60		12,300	466,700	167,200	9,900	59,800	715,900	40%
60-80			173,100			100,400	273,500	15%
80-100			55,100				55,100	3%
> 100			10,300				10,300	1%
Total	30,900	608,400	788,500	181,300	9,900	160,400	1,779,500	100%
	2%	34%	44%	10%	1%	9%	100%	

Table B-2. Length of Water Pipe Mains – Napa (Feet)

Table B-3 lists the number of repairs by pipe material. The total number of repairs (163) reflects the amount completed as of about September 15 2015, at which time water was restored to essentially all customers. As discussed above, for several months afterward, the rate of pipe damage was about 2 to 3 times higher than normal: about 80 more repairs in about 4.5 months; whereas the long term rate was about 90 per 12 months.

This higher leak repair rate in the months following an earthquake is not unusual, reflecting that many pipes were highly stressed / deflected by the earthquake (but not immediately broken), but over time, these highly loaded pipes fail at a higher rate than

normal. One would expect that the long term pipe repair rate will decrease over time; the City of Napa confirmed this to be the case, in that by about late January 2015, the leak rate seemed to have reduced to about the long term rate.

Material	Repairs	% Repairs	% Pipe	Repair per Mile
AC	8	5%	10%	0.23
PVC	2	1%	2%	0.34
CI	123	75%	44%	0.82
DI	18	11%	34%	0.16
Steel	3	2%	9%	0.10
Other / unk	7	4%		
Total	163	100%		

Table B-3. Repair Rates for Water Pipe

Table B-3 shows that cast iron is the most vulnerable of pipe materials, with AC, PVC, DI and Steel all performing much better than Cast Iron. Further examination of the repairs to correlate against location (in terms of PGV and PGD) and age (especially with respect to corrosion) is done in Section B.5 below.

Figure B-13 shows the locations of the 163 pipe repairs listed in Table B-3. In Figure B-13, the locations shown by dots have attributes (like diameter, material); the locations shown by triangles have missing attributes.

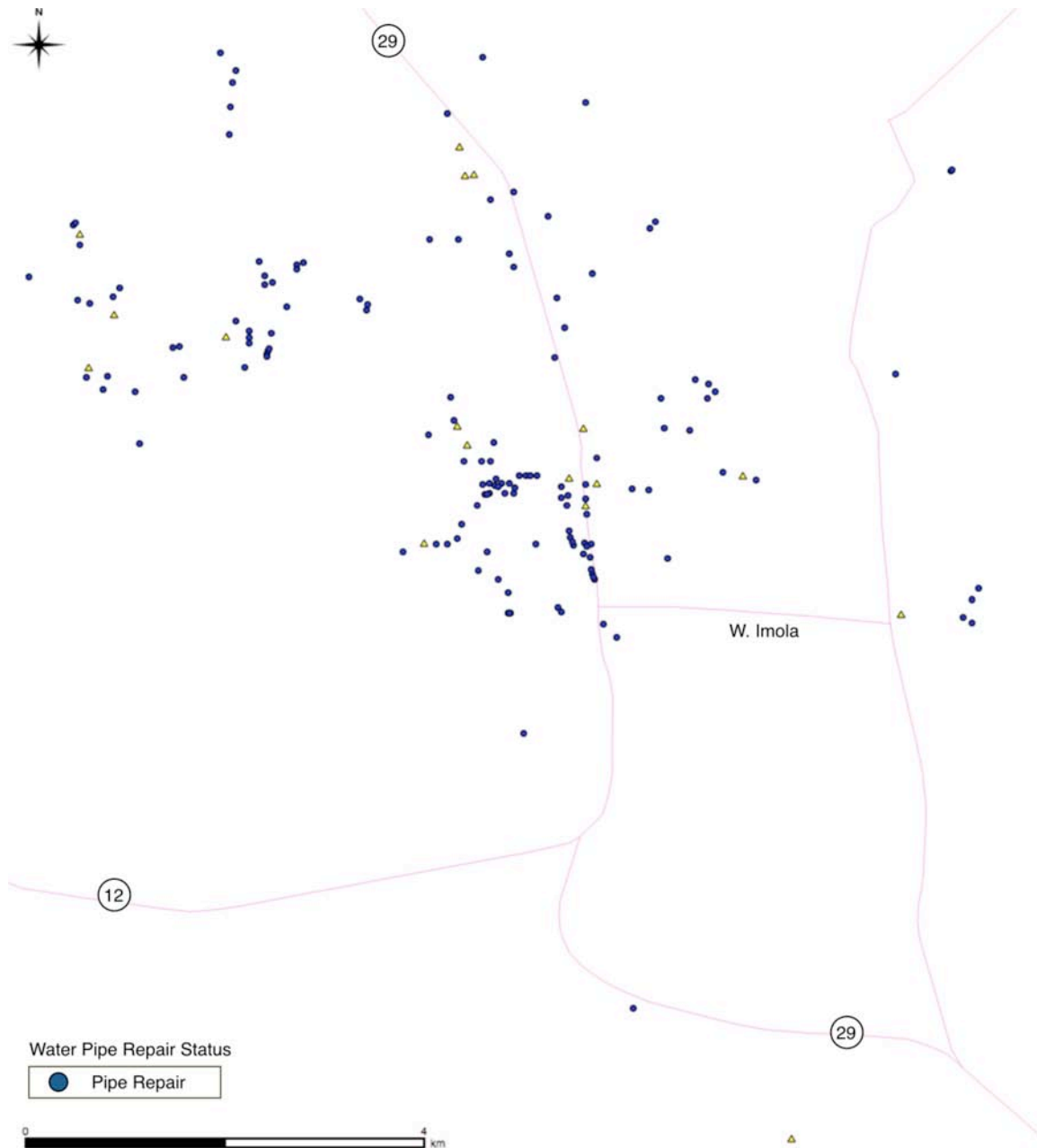


Figure B-13. Location of Pipe Repairs for Napa Water Pipe

Figure B-14 shows the same information, along with results from soil resistivity (Rho) tests of the same type as performed in the City of Palo Alto. Each Rho test is color coded, and the actual test value is shown next to the symbol.

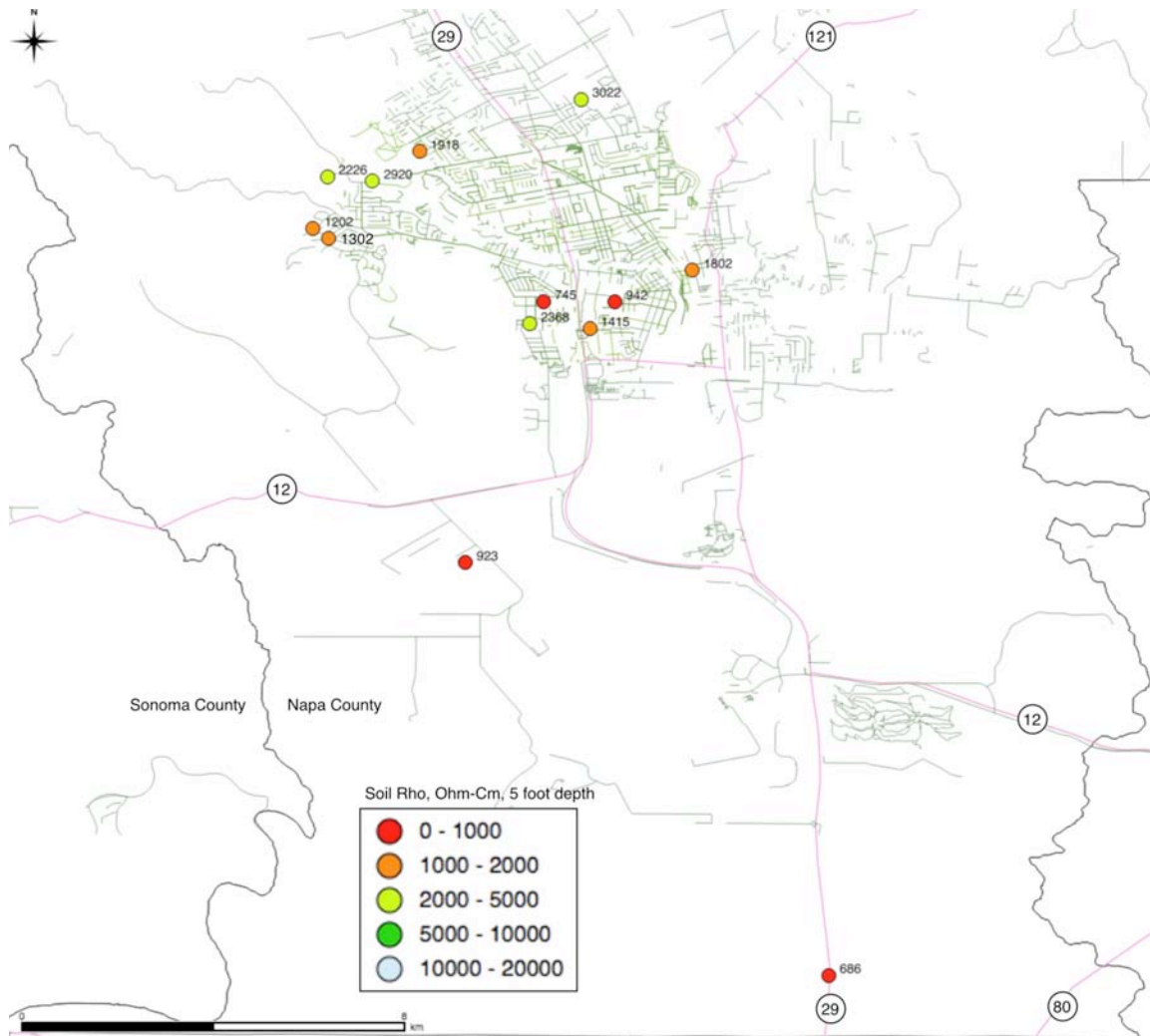


Figure B-14. Soil Resistivity and Location of Pipe Repairs for Napa Water Pipe

Figure B-14 shows that the soils in Napa are extremely aggressive. This may be in part due to the volcanic origins of the soils; many are clayey-in nature. At three test locations, Rho was under 1,000 ohm-cm (extremely corrosive); four more tests showed Rho from 1,000 to 2,000 ohm-cm (very aggressive); and four more tests showed Rho from 2,000 to 3,022 ohm-cm.

The Napa water department reports that their ductile iron pipe has had a fairly high leak rate, even without the earthquake, and this reflects that Napa apparently did not install polyethylene "baggies" over the ductile iron pipe; the very aggressive nature of the soils; coupled with the relatively thinner wall thickness of ductile iron pipe (under 0.2 inches) relative older but thicker cast iron pipe (commonly 0.5 inches or thicker).

In the northwest part of the water system, there was surface faulting. Figure B-15 shows a map of the 71 locations with observed surface faulting (yellow stars), along with the pipe repair locations. The light green lines indicate streets; light blue lines indicate major roads / highways.

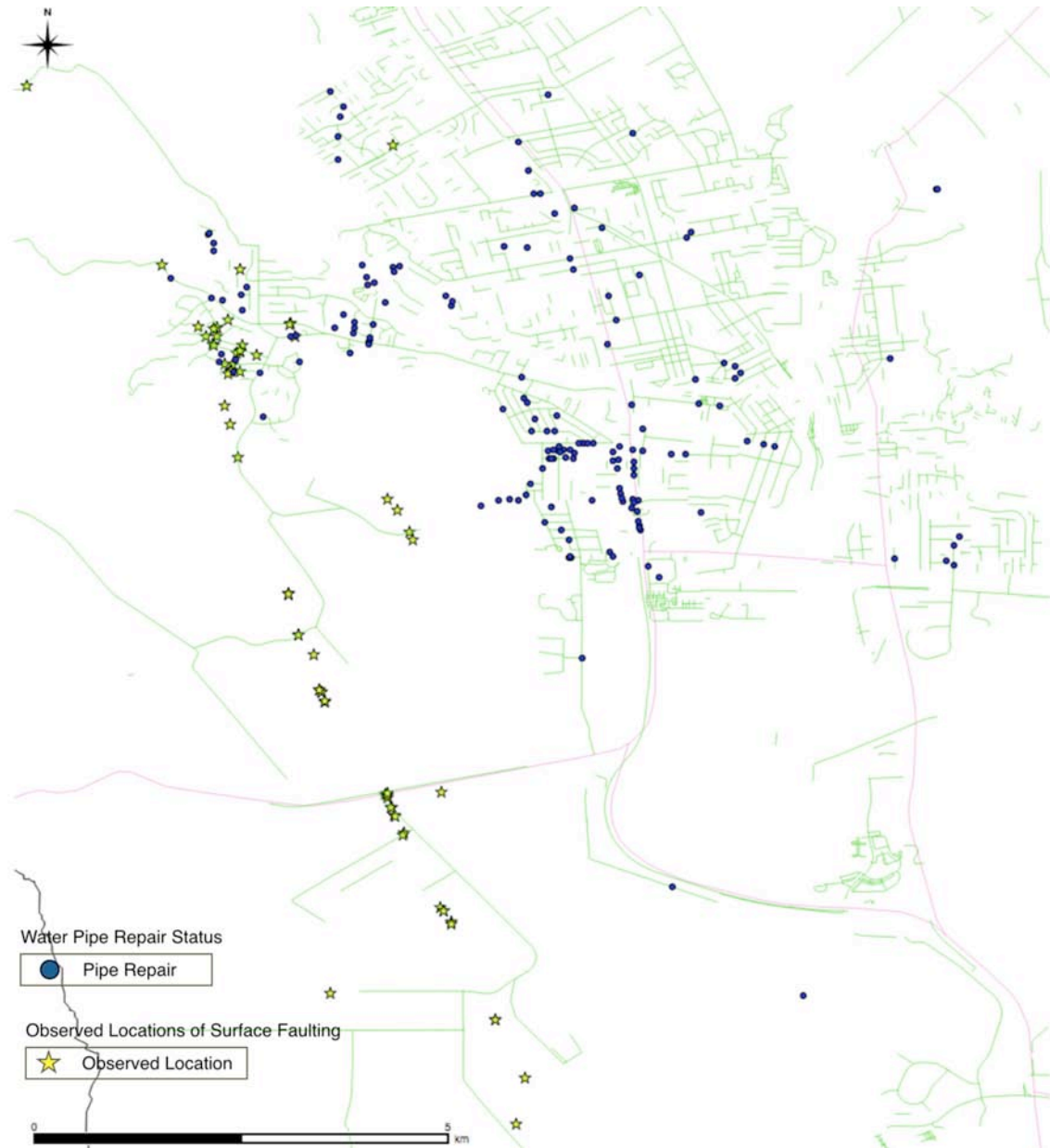


Figure B-15. Location of Surface Faulting and Water Pipe Repairs

Figure B-16 shows the location of pipe repairs, with the pipe main material indicated by color. Most repairs were to cast iron pipe (green dots) followed by ductile iron pipe (light blue dots).

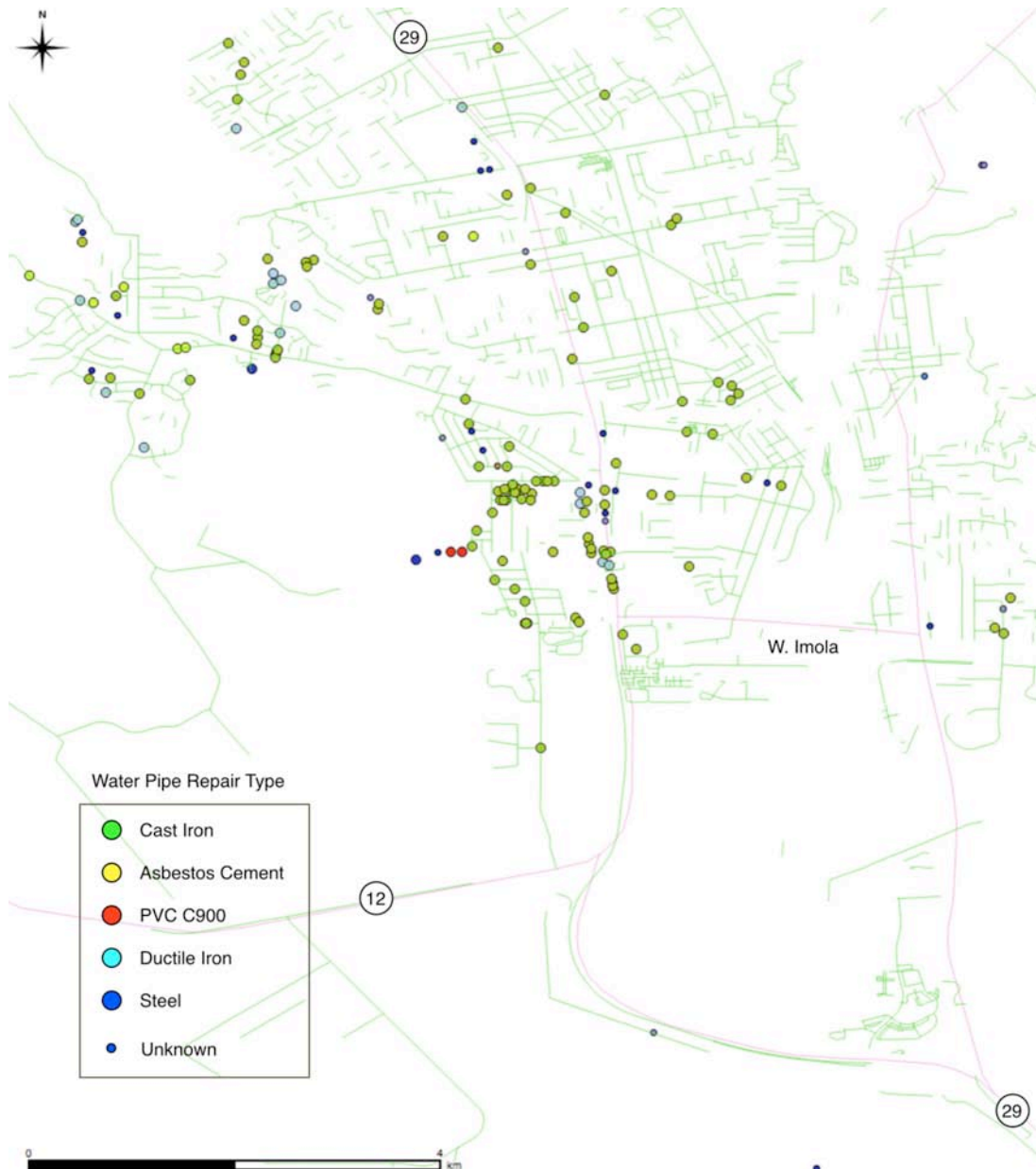


Figure B-16. Water Pipe Repairs – By Material

Figure B-17 shows the location of pipe repairs by diameter. The largest pipe repair made in the two weeks following the earthquake was 12-inches. In the 4 months afterward, a 36-inch diameter AC transmission pipe broke 3 times; that pipe ran north-south along the west side of Napa, as seen in Figure B-12.

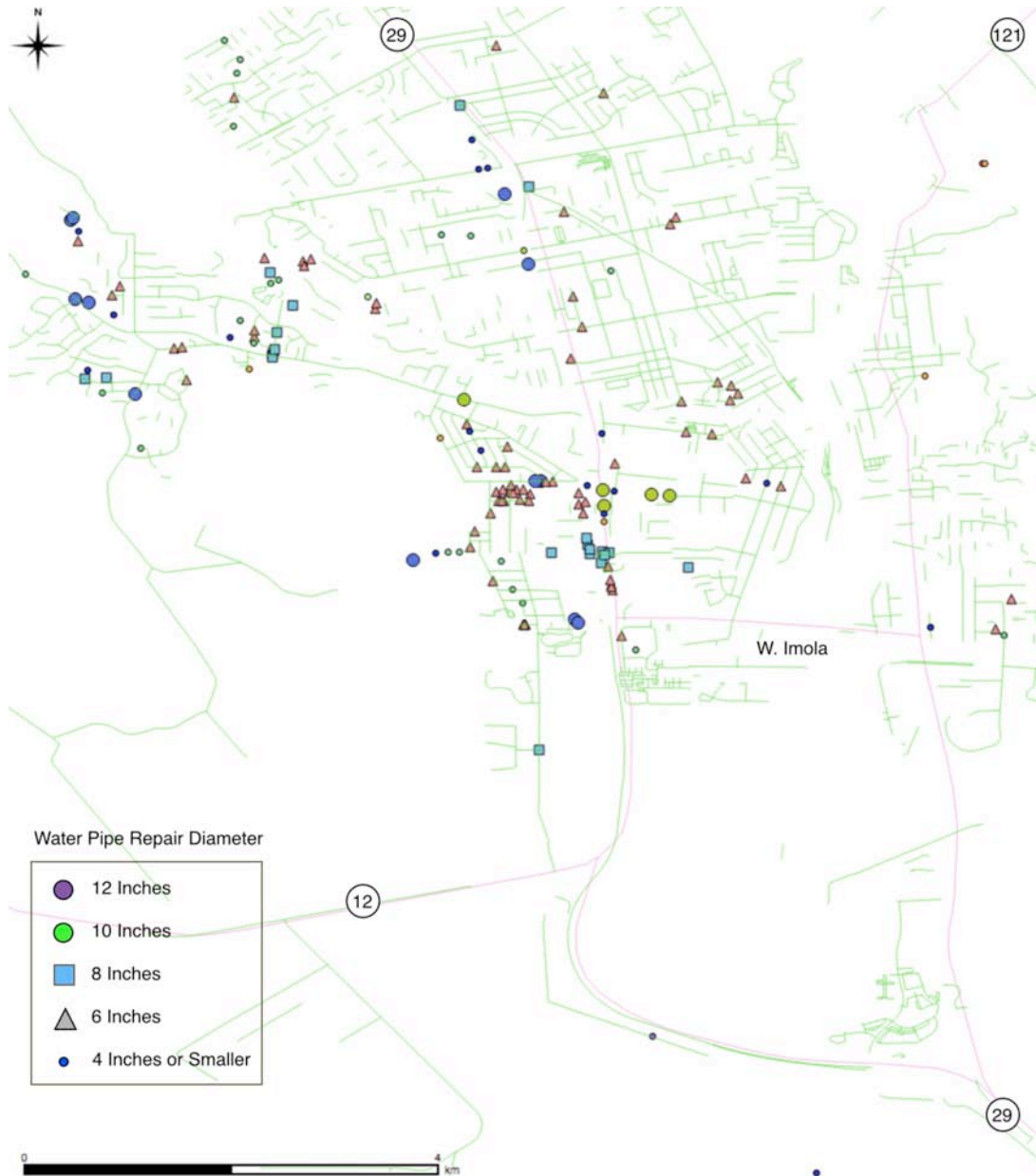


Figure B-17. Water Pipe Repairs – By Diameter

B.5 Seismic Evaluation of City of Napa Water System Pipes

Figure B-18 shows the PGV map for the Napa area. This map shows the maximum of NS or EW PGV, based on the USGS ShakeMap. Each "box" represents an area of about 0.8 km (east-west) by 1 km (north-south). The symbols are at the locations of the pipe repairs as shown in Figure B-18: solid dot: cast iron; triangle: ductile iron; open dot = unknown; polygon: asbestos cement; square: PVC (C900).



Figure B-18. ShakeMap PGVs (Maximum of NS, EW)

The colors of the PGV shaking in Figure B-18 correspond to:

- Red. PGV \geq 85 cm/sec
- Blue. PGV = 70 cm/sec to 85 cm/sec

- Green. PGV = 55 cm/sec to 70 cm/sec
- Magenta. PGV = 40 cm/sec to 55 cm/sec
- Dark Grey. PGV = 30 cm/sec to 40 cm/sec
- Cyan. PGV = 20 cm/sec to 30 cm/sec
- Yellow. PGV = 10 cm/sec to 20 cm/sec
- Light Grey. PGV = 5 cm/sec to 10 cm/sec
- White (no color). PGV < 5 cm/sec

To help appreciate the uncertainty in Figure B-18, there were only 3 instruments in the strong ground shaking area (PGV >20 cm/sec) entire area, shown in Figure B-19.

- Instrument NP.1765. Located inside Fire Station 3 at the north end of Napa. PGV recordings were 64, 87, 45 cm/sec in the NS, EW and vertical directions, respectively.
- Instrument NC.N016. Located near downtown Napa. PGV recordings were 32, 34, 17 cm/sec in the NS, EW and vertical directions, respectively.
- Instrument CE.68150.N016. Located south of Napa (and just off the south end of the map). PGV recordings were 55, 41, 15 cm/sec in the NS, EW and vertical directions, respectively.

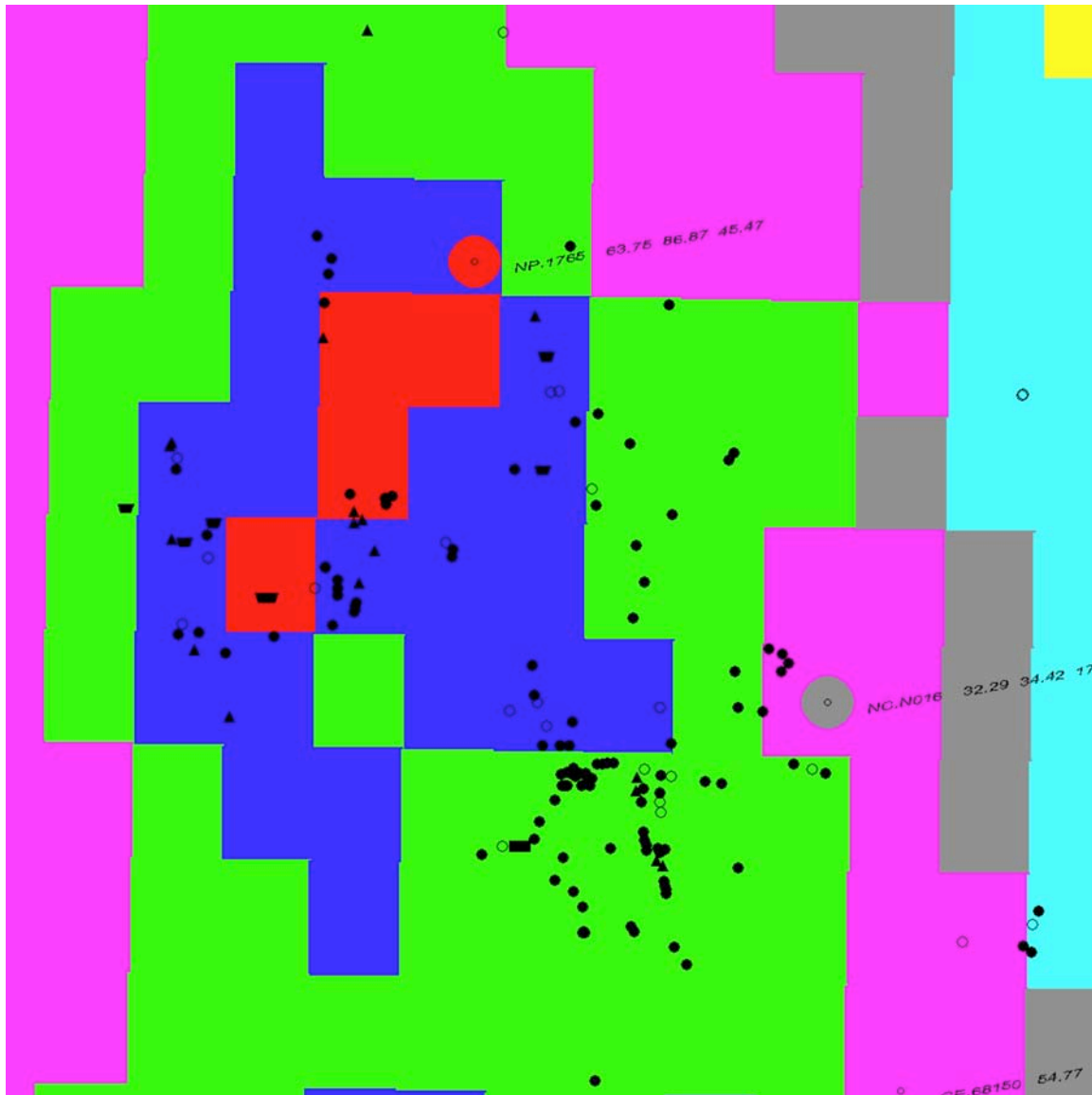


Figure B-19. ShakeMap PGVs and Instrument Locations (Dots)

Using the corresponding PGV ShakeMap levels of shaking from Figure B-18, Table B-4 shows the number of pipe repairs that were subjected to various levels of PGV (totals add to 164; 163 elsewhere in Appendix B; the 1 extra pipe has uncertain attributes).

	<i>20-30 cm/sec</i>	<i>30-40 cm/sec</i>	<i>40-50 cm/sec</i>	<i>50-60 cm/sec</i>	<i>60-70 cm/sec</i>	<i>70-80 cm/sec</i>	<i>80-90 cm/sec</i>	<i>Total</i>
<i>AC</i>					2	2	3	7
<i>CI</i>	3		2	16	38	41	10	110
<i>DI</i>				2	4	4	7	17
<i>PVC</i>					2			2
<i>STL</i>					1	1		2
<i>UNK</i>	3	1	2	1	9	7	2	25
<i>Total</i>	7	1	4	19	56	55	22	164

Table B-4. Water Pipe Repairs vs. PGV (cm/sec)

We \reviewed the number of pipes that were located within about 500 feet from locations of surface faulting; and attributed that damage to surface faulting and not ground shaking or liquefaction effects. As outlined in Section B.2, we also differentiated the number of pipes that were damaged due to liquefaction. Table B-5 shows the results. This reduces the total number of pipes damaged from shaking to 122, with 41 repairs due to PGD effects.

Pipe Type	Length, System-wide (miles)	Repairs due to Shaking (PGV)	Repairs due to Liquefaction (PGD)	Repairs due to Surface Faulting (PGD)	Total Repairs, August 24 to Sept 15 2014
AC	34.34	2	0	5	7
CI	149.34	86	19	5	110
DI	115.23	8	4	5	17
PVC	5.85	2	0	0	2
STL	30.38	2	0	0	2
RCCP	1.88	0	0	0	0
UNK		22	0	3	25
Total	337.01	122	23	18	163

Table B-5. Water Pipe Repairs – PGV and PGD (faulting)

ALA (2001) provides pipe fragility models that factor in PGV and pipe material and diameter, but not the soil resistivity / corrosion effects. In Napa, the corrosion effects should be important, especially for Cast Iron and Ductile Iron pipes. We ran a forecast using modified ALA (2001) models, including the effects of corrosion, and the variations of ground shaking levels throughout Napa. The results are in Table B-5.

Pipe Type	Length, System-wide (miles)	Actual Repairs due to Shaking	Forecast Repairs due to Shaking
AC	34.34	2	2.4
CI	149.34	86	88.5
DI	115.23	8	12.3
PVC	5.85	2	0.4
STL	30.38	2	5.0
RCCP	1.88	0	0.1
UNK		22	
Total	337.01	122	108.8

Table B-5. Water Pipe Repairs – Forecast and Actual (due to PGV)

In preparing Table B-5, the following fragility models were used.

Level of ground shaking. The PGVs, based on ShakeMaps, were subdivided into 8 categories. We then estimated the length of the water pipeline within each category:

- 25 cm/sec (10% of pipe inventory)
- 35 cm/sec (10% of pipe inventory)
- 45 cm/sec (10% of pipe inventory)
- 55 cm/sec (10% of pipe inventory)
- 65 cm/sec (20% of pipe inventory)
- 75 cm/sec (20% of pipe inventory)
- 85 cm/sec (20% of pipe inventory)

The repair rate model for repairs due to shaking was assumed to be:

$RR = k_1 * k_2 * k_3 * 0.00187 * PGV$, where

- RR = repairs per 1,000 feet of pipe
- PGV = ground shaking, inches per second
- k_1 = factor to account for corrosion
- k_2 = factor to account for pipe diameter (k_2 would be higher than 1 for very small diameter pipe (1 to 4 inches), but few of these very small diameter pipes are installed in Napa).
- k_3 = factor to account for pipe material

Pipe Type	k1 corrosion	k2 Diameter (≤ 12 inches)	k3 material	k1 ALA 2001
AC	1.0	1.0	0.3	0.5
CI	1.0 to 3.0	1.0	1.0	1.0
DI	1.5	1.0	0.3	0.5
PVC	1.0	1.0	0.3	0.5
STL	1.0	1.0	0.7	0.7
RCCP	1.0	1.0	0.2	0.2

Table B-6. Pipe Fragility Model: Napa, ALA (2001)

Considering the situation in Napa, we suggest applying corrosion (k_1) as follows for metal pipes:

- $\text{Rho} < 1500$ ohm-cm. Prior to 1920. $k1 = 3.0$. Post 1960. $k1 = 1.0$. 1920 to 1960, interpolate.
- Rho from 1500 to 2500 ohm-cm. Prior to 1920. $k1 = 2.0$. Post 1960. $k1 = 1.0$. 1920 to 1960, interpolate.
- $\text{Rho} > 2500$ ohm-cm. $k1 = 1.0$.

For ductile iron, asbestos cement and PVC, we suggest reducing the $k3$ factor from 0.5 to 0.3. The main issues with shaking for these pipes is the pullout of joints, and all rubber gasketed pipes have similar joint pull out capabilities.

There are potentially other factors which would help explain clustered pipe damage:

- A "basin edge effect". These areas may have had even higher PGVs than suggested by the ShakeMaps.
- A poor installation effort. Possibly, the pipes were installed with cement-caulked rather than lead-filled joints. Cast iron pipe with lead-caulked joints are somewhat more "forgiving" with respect to accepting differential settlements. During the second world war, from about 1941-1945, the use of lead for water pipeline installations was limited due to the war time efforts. Several of these neighborhoods were developed in this time frame, or a few years post-war, and possibly the pipe joinery in these neighborhoods used cement ground (more brittle) than leaded-joints (somewhat less brittle). We do not have enough information to confirm these theories.
- Directivity effects. Some areas may have been exposed to a pulse of high PGV in the east-west (fault normal) direction, due to the direction of the rupture (from south to north) in this area. There are an insufficient number of ground instruments to verify this, but modern ground modeling techniques suggest that this could have happened. At Napa Fire station #3, the instrument clearly shows higher longer-period (over 1-second) ground motions in the east-west (fault normal) direction than the north-south (fault parallel) direction. However, there were no instruments in or near the Qf area in Figure B-6, so the directivity effects are speculative.
- Hydrodynamic effects. The areas with pipe damage clusters in Figure B-6 are gravity-fed area, so water hammer due to sudden pump / activation shutdown should not have been at play. However, if these areas had original pipes that were relatively low pressure (say 150 psi) and the rest of Napa had pipes with a 200 psi pressure rating, the cumulative effects of corrosion, or joint insertion distance, etc.

may have had important influence to explain the cluster of pipe failures in the cluster in Figure B-6.

- The ground motions in the areas with pipe damage clusters in Figure B-6 could have included a strong component of slower moving surface Rayleigh waves, which induce high ground strain, even if ground velocity is not excessive. There are no instruments in this area to verify this theory.

The repair rate model for repairs due to liquefaction was assumed to be:

$RR = k_1 * k_2 * k_3 * 1.06 * (PGD ** 0.319)$, where

- RR = repairs per 1,000 feet of pipe
- PGD = permanent ground deformation, inches
- k_1 = factor to account for corrosion (value in brackets if $Rho > 1500$ ohm-cm)
- k_2 = factor to account for pipe diameter (k_2 would be higher than 1 for very small diameter pipe (1 to 4 inches), but few of these very small diameter pipes are installed in Napa).
- k_3 = factor to account for pipe material

Pipe Type	k1 corrosion	k2 Diameter (≤ 12 inches)	k3 material	k1 ALA 2001
AC	1.0	1.0	0.8	0.8
CI	1.0 (0.8)	1.0	1.0	1.0
DI	1.0 (0.8)	1.0	0.5	0.5
PVC	1.0	1.0	0.8	0.8
STL	1.0 (0.8)	1.0	0.7	0.7
RCCP	1.0	1.0	0.7	0.7

Table B-6. Pipe Fragility Model: Napa, ALA (2001)

Based on the field observations, we estimate the following lengths of pipe were subjected to liquefaction:

- Cast iron. 20,000 feet. Average PGD = 1 inch. Total repairs = 21.2.
- Ductile iron. 4,000 feet. Total repairs = 4.3.

Pipe Type	Length, System-wide (miles)	Actual Repairs due to Liquefaction	Forecast Repairs due to Liquefaction
AC	34.34		
CI	149.34	19	21.2
DI	115.23	4	4.3
PVC	5.85		
STL	30.38		
RCCP	1.88		
UNK			
Total	337.01	23	25.5

Table B-7. Water Pipe Repairs – Forecast and Actual (due to Liquefaction PGDs)

B.6 Napa Water System Emergency Response

Figure B-20 shows the restoration of water service after the earthquake.

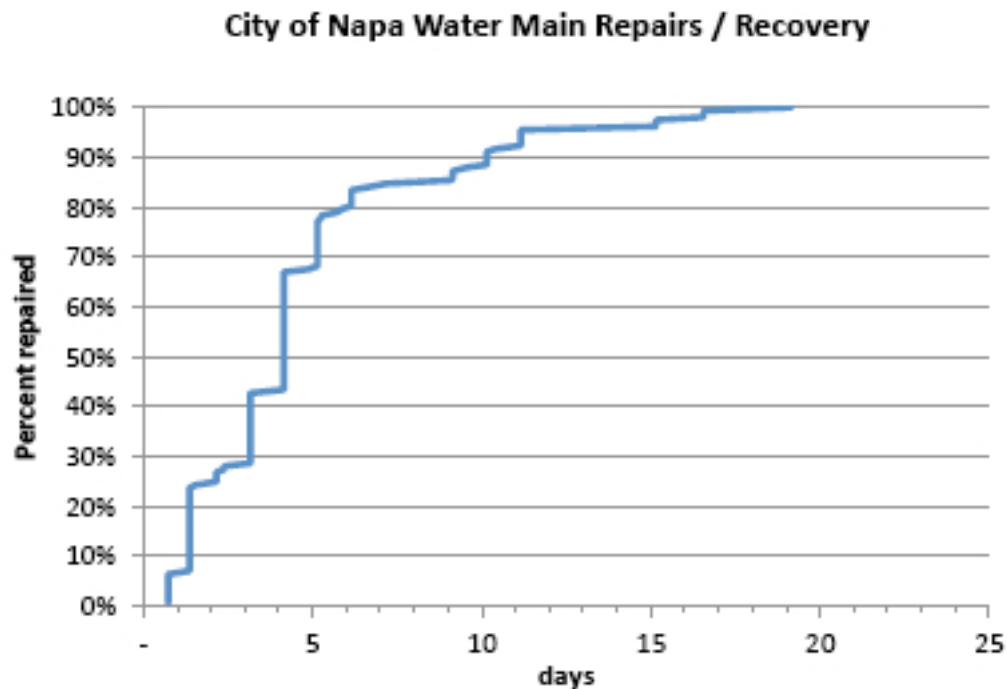


Figure B-20. Napa Water Recovery

In making repairs, one of the required steps prior to digging up the area around the leaking pipe, is to perform "USA" markings (Underground Service Alert). This entails having each utility with underground facilities (gas, water, sewer, communications, etc.) to pre-mark the location of their pipes / conduits, prior to the commencement of digging to get to the leaking pipe. In the post-earthquake environment, when all utilities are also busy, this effort can slow down the entire restoration process. Following this process possibly slowed down the restoration effort for the water pipes. Still, the potential of accidentally damaging a gas pipe, or otherwise damaging third party utilities, cannot be discounted, so this effort should be accounted for in emergency restoration plans.

Water demand increased by 200% or more immediately following the earthquake. This reflected normal overnight water demands, as well as water lost through leaking pipes, and some water being used for fire flows. As measured at the water treatment plants, the following water rates were recorded:

Time	Water Flows from WTPs	Then Known Water Leaks
Day 1. Sunday Aug 24	32 MGD	60 leaks
Day 2. Monday	28 MGD	90 leaks
Day 3. Tuesday	24 MGD	100 leaks
Day 4. Wednesday	22 MGD	105 leaks
Day 5. Thursday	20 MGD	110 leaks
Day 6. Friday	19 MGD	120 leaks
Day 20. Sept 12		167 leaks
Day 37. Oct 1		179 leaks
Day 68. Nov 1		193 leaks
Day 99. Dec 2		220 leaks
Day 153. Jan 26 2015		241 leaks

Table B-8. Napa Water Flows, Million Gallons per Day, Number of Repaired Leaks

The average water demand for late August, without earthquakes is about 18 MGD. The actual water flows in the entire distribution system is uncertain, but in the first day would be much higher than the 32 MGD listed in Table B-8, in that much of the water in the 12 storage tanks (with 30 MG capacity, mostly full at the time of the earthquake) also drained in the first few hours.

The Napa Water Department was aided by several regional utilities in making pipe repairs. Mutual aid (via Cal Warn) pipe crews were provided by:

- EBMUD: 5 crews. Three crews were initially dispatched within 24 hours of the earthquake, and two days later, two additional crews. EBMUD reports that the EBMUD crews helped make repairs for 56 pipe leaks.

- Contra Costa Water district: 1 crew
- City of Fairfield: 2 crews
- Alameda County Water District: 1 crew

All crews arrived with spare parts, trucks and equipment, typically 5 people per crew. All mutual aid crews were released by August 29 2014 (note the slow down in the rate of pipe repairs after Day 5, Figure B-20). It was initially thought that the mutual aid crews were sufficient to effect nearly all the pipe repairs by August 29; however, over time, as the last pipe repairs were made, additional pipe leaks were identified as repaired pipes were re-pressurized.

The Napa Water Department estimated they spent about \$200,000 on spare parts.

There were no regional-wide boil water alerts issued during or after the earthquake. Water quality at the water treatment plants was reported to be acceptable. The general population was encouraged to use bottled water (many did). There were boil water alerts to all customers who lost water supply, owing the concern of possible cross contamination from nearby potentially damaged sewer lines (no evidence that this occurred); or bacterial growth in empty water pipes.

The lack of a regional boil water alert was in part due to the negotiations between the Napa Water Department and other state-wide agencies. The state-wide agencies wanted a large scale boil water alert, being concerned that damaged sewer pipes might be leaking sewage into damaged water pipes. However, the City of Napa noted that they had no damage in the transmission pipes, and the water treatment plants were able to keep up with the increased water demand (for normal use lost water due to leaks, and fire fighting purposes), and positive pressure was maintained in the majority of the water system, so there was no need for such a regional boil water alert. Had the water transmission pipes been damaged, or the water treatment plants been unable to keep up with post-earthquake water demands, large portions of the water system would have become de-pressurized, and this would have had much more impacts on customers (more outages, boil water alerts, longer restoration times, etc.).

At some locations, the City of Napa installed hose bibs, Figure B-21, at working fire hydrants with potable water, and alerted residents where they could go get potable water. These were located close to locations where there were water pipe breaks and with residents without piped potable water.



Figure B-21. Hose Bib Used During Recovery

By August 28, there were about 500 Napa customers without water. By August 29, this was reduced to about 400 customers, with the intent to have all customers back in service by August 30.

B.7 Napa Water System – Other Facilities

The Napa water system includes 3 water treatment plants, 12 storage tanks (30 MG capacity), 380 miles of pipes, up to 42" diameter, 9 pump stations, 5 pressure zones and 14 pressure regulating stations.

Figure B-22 shows an aerial view of the Jamieson WTP. The site was likely exposed to PGA about 0.10g. The plant was originally constructed in 1968, with raw water being delivered by the Department of Water Resources (DWR)'s North Bay Aqueduct (72-inch steel pipeline). In 1968, the plant was rated for 16 MGD maximum flow, and then included a 7 MG steel tank for raw water (owned by the DWR) and a 5 MG steel clearwell.

The plant was expanded in 2011, adding pre-treatment basins (tube settling-type), two new filters, a new chemical storage facility, new pre- and intermediate ozone injection systems, emergency backup power, new washwater recovery clarifiers, increasing maximum capacity to 24 MGD.

There was no damage at this WTP. DWR reports no damage to the North Bay Aqueduct (ground motions typically PGA = 0.05g to 0.15g).



Figure B-22. Jamieson WTP, Aerial View, Looking South

The Hennessey WTP was constructed in 1981, Figure B-23. The 5 MG clearwell is a buried tank. Ground shaking would have been modest, on the order of $PGA = 0.05g$ to $0.10g$.



Figure B-23. Hennessey WTP, Aerial View

The Milliken WTP was built in 1975. Figure B-23 shows a rockfall atop the 16" raw water pipeline to the Milliken WTP; the pipe was thought not to be damaged.



Figure B-24. Milliken Pipeline with Rockfall

Figures B-25 and B-26 show the damage to the corrugated metal roof for Tank B.



Figure B-25. Tank B. Damaged Corrugated Metal Roof



Figure B-26. Tank B. Corrugated Metal Roof Damage (Side View)

Tank B is a welded steel 1.0 MG tank, designed in 1960. The tank rests on a concrete ring beam, without anchorage. The roof is supported by nine interior steel posts, with a series of timbers running in one direction, and then wood rafters in the other direction, all supporting a corrugated metal roof.

Prior to the earthquake, the inlet-outlet pipes had been retrofitted with EBAA-type ball-and-slip joints, Figure B-27.

During the earthquake, the tank water depth was at 31.5 feet, with overflow at about 37.5 feet. The partially-filled tank likely rocked, with the tank walls uplifting. The unrestricted water slosh height would have been perhaps 3 to 5 feet (this presumes a certain level of shaking, which in reality is unknown), but with the tank water elevation at about 6 to 7 feet beneath the wood timbers and rafters, there might have been no major high-pressure water impacts from the sloshing into the roof system; all the same, the adjacent Verizon telecom building that is located at the water tank site, just below the tank, did show some evidence of water inundation, suggesting that the water slosh height was at least enough to have water spill through the ventilation screens near the top of the tank; so this suggests that the actual water slosh height was more than 5 feet. The uplifting of the walls would have tried to lift the wood beams upwards, and as they were not designed for this movement, they were damaged, possibly falling sideways, and then shifting the corrugated metal roof sideways, as seen in Figures B-25 and B-26. There was no evidence of steel shell buckling at the base of the tank.



Figure B-27. Tank B. Inlet Outlet Pipes, Pipe with Ball and Slip Joints



Figure B-28. Tank B. Inlet Outlet Pipes, Cracked Concrete Thrust Block

Subsequent to the earthquake, Napa replaced the original corrugated roof system on this tank with an aluminum-dome type system.

B.8 References

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Appendix C. Asbestos Cement Corrosion

Appendix C examines some of the technical issues on the performance of AC pipe over time.

C.1 Palo Alto Usage of AC Pipe

As of 2010, systemwide, there was 232 miles of distribution pipe in the City of Palo Alto water system. This includes 134.9 miles of AC pipe, or 58% of the total by length. Table C-1 lists the lengths of AC pipe in the Palo Alto water system, as of 2010.

Diameter (inches)	Asbestos Cement (feet)
0.5	
0.75	
1	
1.25	64
1.5	
2	710
3	
4	37,794
6	313,788
8	214,376
10	29,116
12	72,984
14	22,572
16	18,137
18	
20	
24	25
27	2,770
30	
Unk	56
Total (feet)	712,392
Total (miles 2010)	134.92

Table C-1. Length of AC Pipes, as of 2010

Historically, Palo Alto began installing AC pipe around 1940, and stopped by about 1986. By 1956, there was 71.9 miles of AC pipe in the system. Since 1990, about 12.03 miles of AC pipe have been removed from the system. By 2010, there was 134.92 miles of AC pipe in the system. Today (2015), no new AC pipe is being installed.

The AC pipe is mostly used for 6-inch and 8-inch diameter distribution mains (about 100 miles, or 74% of all AC pipe), with lesser amounts used for 4 (7 miles), 10 (6 miles), 12 (14 miles), 14 (4 miles), 16 (3 miles) inch diameters and about 0.5 miles of 27 inch pipe, and cumulative less than 1 mile in all other pipe sizes (1.25 inch, 2 inch, 24 inch and unknown diameter).

Today (2015), we could find no current AC pipe installation specifications for Palo Alto's AC pipe. Appendix D of this report provides the AC pipe installations used for one project in Palo Alto dated 1982. It may be there are older Palo-Alto specific AC pipe installation specifications, but as yet, we have not located them.

C.2 How does AC Pipe Fail?

Pipe failures in AC pipe will generally occur due to three conditions:

- Internal water pressure exceeding the hoop strength of the pipe wall. Under normal design, there is enough strength in new AC pipe wall to resist at least 4 to 5 times the internal water pressure. Therefore, this failure mode is likely to occur when internal or external corrosion has resulting in sufficient "thinning" of the pipe wall such that the pipe cannot longer sustain the water pressure. When AC pipe bursts in this mode, it tends to burst catastrophically (a major break, not a minor leak). When a damaged AC pipe shows cracks solely in the longitudinal direction of the pipe, it could be assumed that the root-cause of the failure was due to excessive internal pressure; however, once the AC pipe begins to crack, the direction of the cracks might change rapidly from longitudinal direction to transverse direction, so it might not be entirely obvious from the observed crack pattern once the pipe is dug up for repair, as to the root cause failure mode.
- Soil movements that induce longitudinal forces in the pipe wall. High longitudinal forces can result in tensile rupture of the wall; compressive buckling of the wall (rare for AC pipe); or more commonly, pull out of the pipe from the rubber gasket coupling (a common earthquake failure mode due to very strong wave passage effects); or high bending on the pipe barrel, resulting in rupture of the wall (a common earthquake failure mode due to permanent ground deformations due to liquefaction, landslide or fault offset effects).
- Local stress risers are introduced into the AC pipe at service lateral or fire hydrant taps. Commonly, when a lateral is installed, a clamping device is placed around the pipe to resist the thrust forces introduced by the lateral, as well as to tighten the connection so it does not leak (Figure D-5). Very small diameter taps might rely only on threaded connections in the pipe wall, but such connections are usually only made in metal pipe.

One is interested in trying to forecast the events that can lead to any of these failure modes. If the chance of occurrence of a pipe failure is high enough, and the adverse

impacts of pipe failure high enough (loss of water supply until repairs are made, economic impacts, etc.), then it might be economically justifiable to replace the AC pipe prior to such failure. Section 7 of this report presents these benefit cost analyses.

C.3 EBMUD Experience

Internal corrosion due to soft water can play a role in AC pipe deterioration. Internal corrosion can occur when the water being transported has a pH less than 7. Below 7, the water is "acidic", and will tend to attract positive ions from adjacent materials, including the Calcium (++) charge) in the AC pipe cement matrix.

The East Bay Municipal Utilities District (EBMUD) serves water to 23 cities in the East Bay area, serving about 1,300,000 people. The EBMUD water system includes about 4,000 miles of pipe, of which about 1,150 miles is AC pipe. Systemwide, EBMUD experiences about 1,000 pipe repairs per year, of which about 120 repairs are to AC pipe. Between 1990 and 2010, EBMUD was then replacing between 5 to 10 miles of pipe per year; even at the 10 mile per year rate, the replacement cycle would be 400 years. This led EBMUD to do detailed assessments of its pipe inventory, to develop a rational and economic strategy to replace its aging pipe.

EBMUD's usual water source is from Pardee Reservoir, with source water from the Mokelumne River in the Sierras. This water is soft, low alkalinity water (typically 15 mg/l as CaCO₃, 20 mg/l alkalinity). This water is corrosive towards the cement part of the AC mix, as well as mortar cement lining used in steel (or other types) of pipes. The Langlier Saturation Index (LSI) is one of several chemical indices that quantifies how corrosive the water might be; a negative LSI implies corrosion of cement ought to be expected.

Until the early 1980s, EBMUD's treatment goal was an LSI of slightly greater than zero, to minimize any impact on cementitious materials (including AC pipe, and cement linings on other types of pipe). LSI goals were met by adding lime prior to water entering the raw water aqueducts at the headworks at Pardee reservoir in the Sierras.

Before 1983, EBMUD did not coagulate at its in-line filtration plants, because the Pardee water was already clear enough to satisfy turbidity requirements of the time. Starting in 1983, EBMUD lowered its target LSI to -0.5, which increased the water's aggressiveness. In the 1990s, the lime used at WTPs was replaced with caustic soda; and this made the water more corrosive for a given pH. In about 2000, the typical pH was slightly lowered as part of an attempt to optimize compliance with the Lead and Copper rule.

In 2010, EBMUD conducted tests at 18 AC pipe locations, and analyzed historical leak data. It was identified that AC pipe aged 60 years or more (installed in 1948 and 1949) had a significantly higher historical pipe leak rate than newer pipe. Leak history suggested that 48% of all repairs indicated circumferential breaks, suggesting the loads that induced the breaks were due to soil movements; and there was a seasonality: higher

repair rates in the dry summer season, with full circumferential breaks 3 times more likely in the dry summer months and large blowout holes that are greater than 1-inch 2 times more likely in the dry summer months. EBMUD attempted to correlate these repair rates with the water chemistry or soil conditions at the locations of the pipe failures.

EBMUD conducted a series of phenolphthalein³ staining and hardness tests on the AC pipe. Phenolphthalein staining is used to measure the thickness of calcium hydroxide (lime) left in the pipe matrix. The approach in these tests was to spray on phenolphthalein on a freshly exposed, smoothly ground, cross section of a AC pipe sample. Figure C-1 shows one such test sample. It is assumed that the cross section areas that still have lime turn pink (basic) and the areas where lime has leached out do not change color (acidic). At pH under 8.2, the dye remains colorless. At pH over 8.4, the dye turns pink (red).



Figure C-1. Results of Stain Test, EBMUD AC pipe

Figure C-1 shows a sample AC pipe phenolphthalein test. The image shows a cross section of pipe. The pinkish center band shows high pH, while the inner and outer layers did not stain pinkish, suggesting zones which are acidic, and indicating that leaching or corrosion has occurred.

For about half the samples, deterioration on the inside face was thicker than on the outside face.

The staining tests seemed to indicate a clear evidence of loss of pipe hoop strength. Given the known age of the pipes at the time of the tests were performed, a projection of "loss of hoop strength" over time was developed. Based on this set of tests, a forecast was made that EBMUD should begin replacing about 41 miles of AC pipe per year; the reasoning is outlined below.

³ Phenolphthalein is used in acid-based tests: it turns colorless in acidic solutions, and pink in basic solutions.

As suggested in Figure C-1, the AC pipelines showed clear signs of aging due to chemical attack from within and without, in roughly equal measure.

However, the loss of hoop strength did not account for the higher rate of failure in the summer months. EBMUD theorized that the increased break rates observed in the summer months was attributed to drier soils, leading to soil shrinkage, leading to increased external bending load on the AC pipe. Such soil shrinkage would be dominant in clay-like soils.

Given these findings, EBMUD then developed a AC pipe replacement model. For corrosion, EBMUD developed a predictive equation to forecast when the wall thinning would reach 20% of its original thickness (in other words, the initial Factor of Safety against burst, originally 4 to 5, would be reduced to about 1, and the pipe should burst). This should correlate with a blowout. Based on this model, EBMUD projected that 41 miles of AC pipe should be replaced per year by the year 2020, and by 2050 an average of 27 miles per year should be replaced.

Given that increasing the annual pipe replacement efforts from about 10 miles per year to about 40 miles per year would require a major investment, and an increase in water rates, in parallel, EBMUD considered what they could do in the interim to slow down the rate of AC pipe aging. Accordingly, EBMUD considered modifications to its water quality goals to reduce internal corrosion of AC pipe, while at the same time without creating adverse impacts such as metal corrosion, chloramine instability, nitrification, and distribution system regrowth. These goals were to change the water quality at the five normally operational EBMUD WTPs as follows:

Interim goals.

- Orinda, Walnut Creek, Lafayette WTPs. Calcium Carbonate Precipitation Potential (CCPP): 2.0 mg/L, pH < 9.5
- Sobrante and USL WTPs. CCPP: 4.0 mg/L. pH 8.5 to 9.0

Long Term goals.

- Orinda, Walnut Creek, Lafayette WTPs. Calcium Carbonate Precipitation Potential (CCPP): 4.0 mg/L, pH 8.5 to 9.0. Alkalinity (as CaCO₃): 40 mg/L.
- Sobrante and USL WTPs. 4.0 mg/L, pH 8.5-9.0. Alkalinity (as CaCO₃) 40 mg/L.

EBMUD has 1,150 miles of AC pipe, with about 120 AC pipe repairs per year. This corresponds to a repair rate of $120 / 1150 = 0.104$ repairs per mile per year. In contrast, Palo Alto AC pipe repair rate currently is about 0.064 repairs / mile per year.

EBMUD splits its AC pipes into two categories: Type I and Type II.

- Type I. 15.5% of total weight is free lime. Most EBMUD AC pipe is Type I.
- Type II. <1% of total weight was free lime, and a higher amount of silica was combined with free lime in cement products. This improves the resistance of the AC pipe to acid attack.
- In 1973, the asbestos content in AC pipe was reduced from about 12% to less than 0.2%. EBMUD has 290 miles of post-1973 AC pipe. Low asbestos fiber content pipe might be a more likely candidate for replacement by pipe bursting.

The first AWWA standard for AC pipes was approved in 1953 (C400-53T). AC pipe could be purchased in pressure classes 100, 150 and 200, (class = maximum working pressure in psi); there were no criteria for sulfate resistance or use of silica. In 1964, AWWA C400-64 required silica as a constituent and included methods to measure free lime content.

There have been no standards regarding AC pipe thickness. Thickness varies between manufacturer, and even within a manufacturer. For class 150, 6-inch pipe the wall thickness varied between 0.55 to 0.71 inches, while for 8-inch pipes it varied between 0.91 and 1.1 inches. Assuming a working pressure of 150 psi, this would translate to the following hoop stresses:

Pipe Diameter (inches)	Wall t (inches)	Hoop Stress (psi)
6	0.55	818
6	0.71	634
8	0.91	659
8	1.1	545

Table C-1. Hoop Stress in AC pipe at 150 psi internal pressure

ASTM C 296 requires as-manufactured AC pipe to sustain a hydrostatic pressure of 4 times the rated working pressure for the class of pipe (600 psig for Class 150) for not less than 5 seconds.

EBMUD tested two of the pipe samples that the testing had suggested a loss of hoop strength. The results were surprising:

- AC pipe sample X-101 (Century Way) burst at 1,190 psi.
- AC pipe sample X-102 reached 750 psig prior to leaking at one of the couplings.

In other words, the pipes, even with indications of loss of hoop strength per Figure C-1, remained as strong, or stronger, than original band-new installation requirements.

C.4 External Corrosion

AC pipes are subject to external damage caused by an aggressive soil and groundwater environment. Acidic soils and groundwater leach out the calcium hydroxide (lime) and calcium silicate from the external pipe surface. Sulfate in the surrounding soils react with calcium silicate hydrates and other minerals in the AC pipe cement. This causes swelling in the pipe external matrix and results in the destruction of the cementitious material of the AC pipe.

Soil aggressiveness can be measured by soil pH and sulfate, with soils containing organic material and clay usually having higher concentration of sulfates. In addition, sulfate reducing bacteria increase soil acidity and aggressiveness toward AC pipes.

To some extent, soil Rho (resistivity) values might have a direct correlation with soil pH and sulfate. We attempted to correlate Rho with pipe repairs for AC pipe (see Figure 6-14), but we found no clear evidence for such a correlation.

C.5 Hardness

The purpose of a hardness test is to gain an indirect indication of the amount of calcium that has been lost in a pipe. To measure hardness, a Rockwell "L" hardness instrument or a Shore "D" durometer is pressed into the AC pipe wall. By applying a standard amount of force and measuring the penetration into the pipe wall, a hardness value is obtained.

We obtained from Palo Alto several samples of AC pipe, and conducted both Rockwell "L" and Shore "D" tests. We found that the Shore "D" tests were mostly non-conclusive, as the hardness range for Shore "D" is limited to items like soft to hard rubber; and AC pipe is generally harder than that, so many tests were "off scale". We then used the Rockwell "L" tests, that can reasonably forecast concrete strengths of up to 5,000 to 8,000 psi or higher; clearly, AC pipe would be no harder than very sound concrete. We tested a few samples, usually 12 tests per sample, tossing out the low and high tests, and taking an average of the remaining 10. The tests were done on the inside face of the AC pipe sample, by placing the curved sample down on a concrete surface, and using the instrument from above, with the load path directly through the AC pipe sample into the concrete surface. We found common equivalent concrete strengths in the range of 3,000 to 4,000 psi; with no obvious indication of a weakened inside surface. We tried the same test on the outside surface, but since each sample was curved, the impact energy from the hammer caused bending in the curved pipe, so no useful data could be gathered.

Summary. We did not observe any particular trend of soft AC pipe in the specimens we tested. It could be that further testing, by preparing each sample, could provide more results; but recognizing the health concerns with cutting AC pipe into small samples, and the likely lack of clear results, we did not pursue those types of tests.

C.6 Additional EBMUD Tests

AC Pipe test samples from EBMUD's water system were obtained at 5 locations in Danville (circa 2007) and 16 samples system wide (address data lost for most), (circa 2009).

Laboratory tests show the following:

- Soil sulfates. No substantive correlation to pipe deterioration
- Soil resistivity (by lab sample). Some possible correlation to pipe deterioration
- Soil pH. No obvious correlation of soil pH (7 to 9) to pipe deterioration
- Soil chlorides. No substantive correlation to pipe deterioration
- Soil resistivity (by in-situ test). No substantive correlation to pipe deterioration
- LSI values from water samples from spigots near the failed pipes suggest moderately aggressive to AC pipe.
- No correlation was found between pipe density and percent of sound pipe as indicated by phenolphthalein testing.
- At two failure points in Danville, the remaining wall thickness and estimate rate of deterioration suggested a short remaining life (19 to 23 years), but adjacent pipe, located 10 to 20 feet from the failure location. showed no weaknesses (tested by Levelton). This suggests that the rate of wall thinning (pinkish layer in Figure C-1) may vary rapidly over short lengths of pipe, and that possibly the rate of thinning (from failed test samples) might not be a good indicator of remaining life.... but that local leak history might be a better indicator.

Crush tests showed all samples of existing pipe failures showed all but one sample met strength requirements for new pipe.

EBMUD leak history data (1977 to 2009) showed:

- Higher repair rates for 4, 6, and 8 inch AC pipe than larger diameter AC pipe
- Much higher repair rate for AC pipe 60 years or older (pre-1949 or so)
- About 48% of repairs were full circumferential, suggesting the cause is bending induced by soil movements

- Pipe repairs increase dramatically in the summer months, peaking in August. The failures in August are more than 3 times the failure rate during January.
- EBMUD's AC repair rate increased significantly since the year 2004; but this data might be affected by an anomaly in the data for 2008.

Appendix D. Palo Alto Standard Pipe Installation

D.1 Example Palo Alto AC Pipeline Installation

Appendix D addresses the common AC pipe installations in Palo Alto.

Figures D-1 through D-4 are taken from drawings for Tract No. 7025 (William Hewlitt 1501 Page Mill Road), dated 1982, and are presumed to be consistent with common installation practices in Palo Alto of that time period.

Figure D-1 shows a 1982-vintage fire hydrant installation, from a 8" ACP pipe main. The lateral is AC pipe. Most common hydrants have three outlets: two 2-1/2" hose and 1-4.5" pumper.

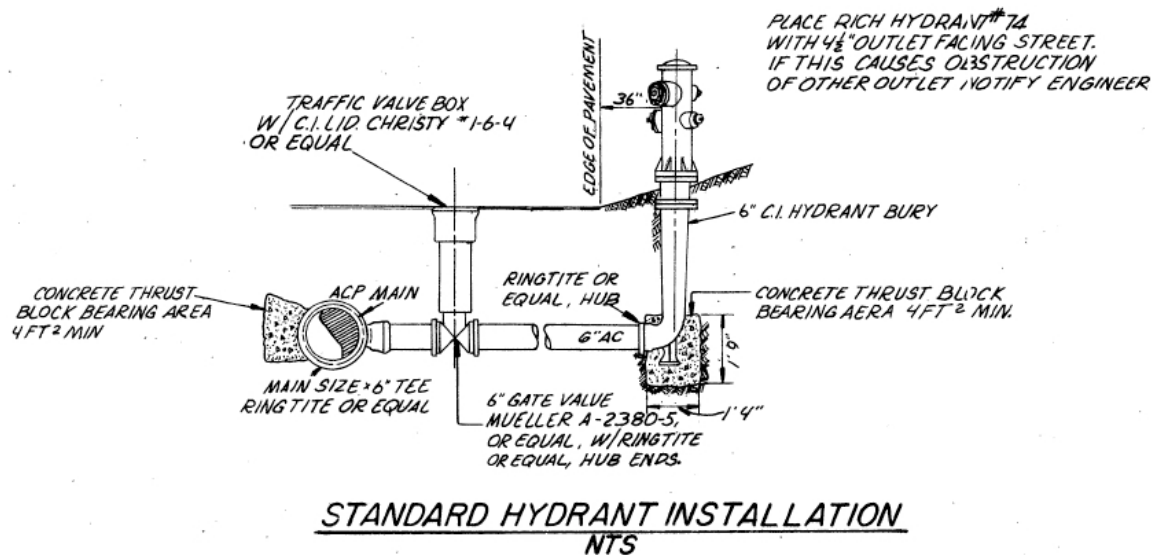


Figure D-1. Fire Hydrant, circa 1982

Figure D-2 shows a common air release valve installation. Circa 1982.

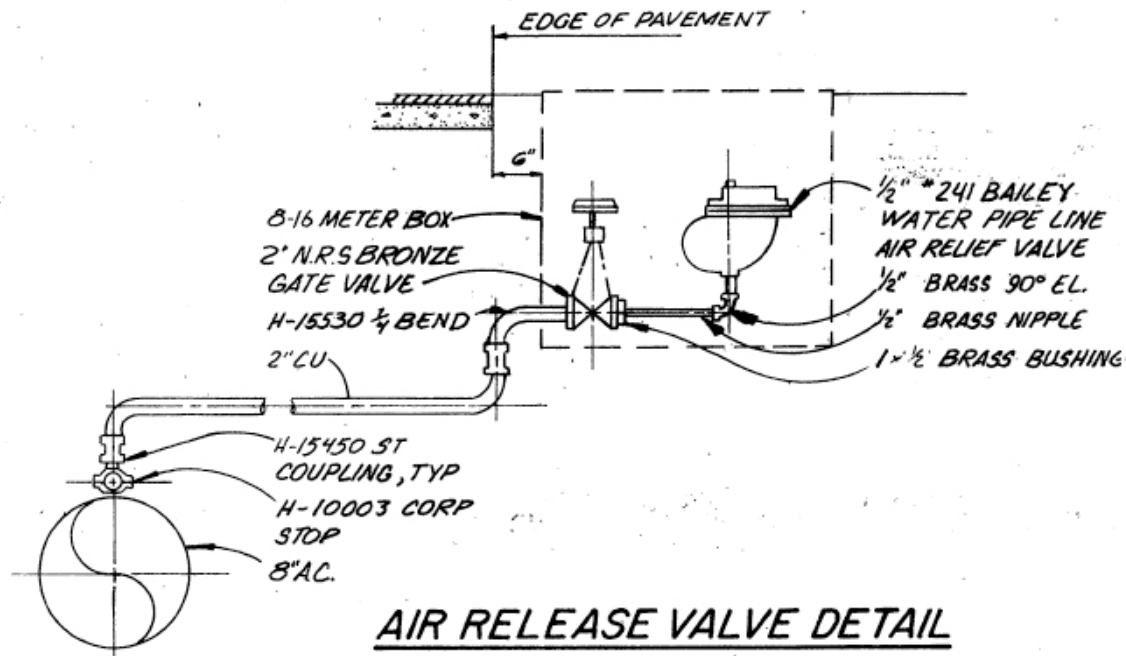


Figure D-2. Air Release Valve, circa 1982

Figure D-3 shows the common installation of a gate valve on a AC distribution pipe.

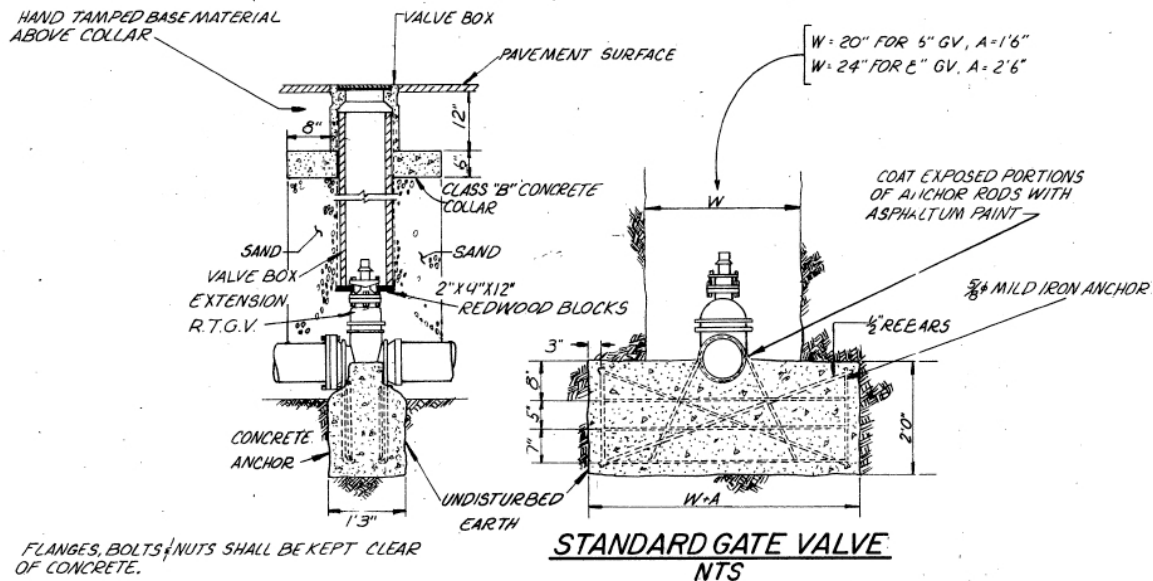


Figure D-3. Gate Valve on AC pipe, circa 1982

Figure D-4 shows the common AC pipe installation of 1982. The notes suggests maximum pipe lengths for 8" AC was 13 feet, and installed per the Johns Manville installation guide.

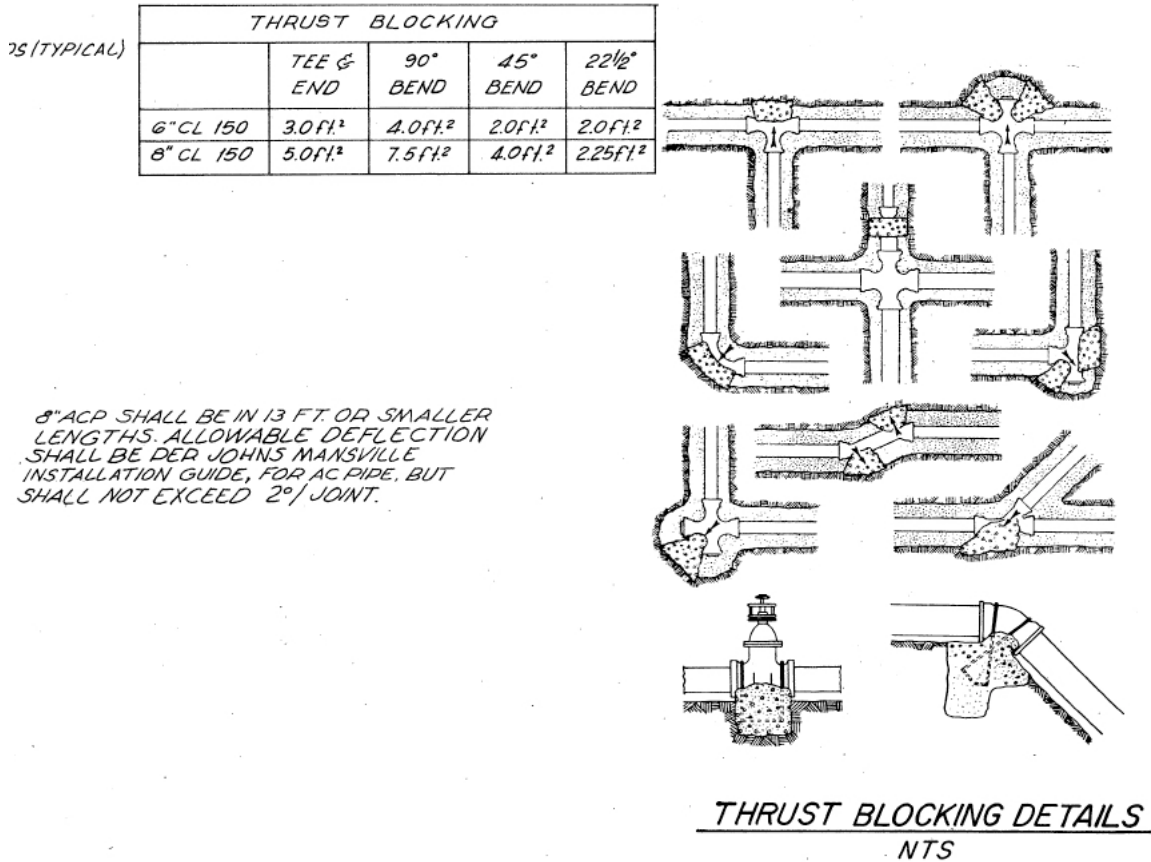


Figure D-4. AC pipe Installation, circa 1982

Figure D-5 shows a 2" service lateral from a AC pipe, circa 1982.

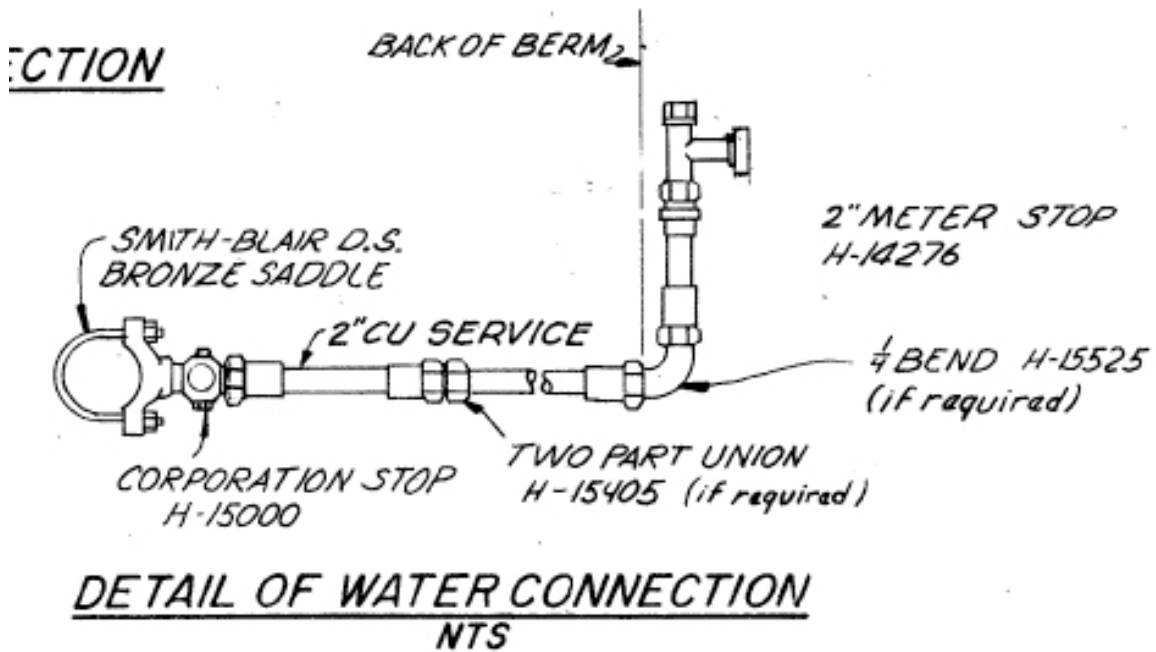


Figure D-5. Service Lateral from AC Pipe, circa 1982

Figure D-6 shows a water pipe trench, circa 1982.

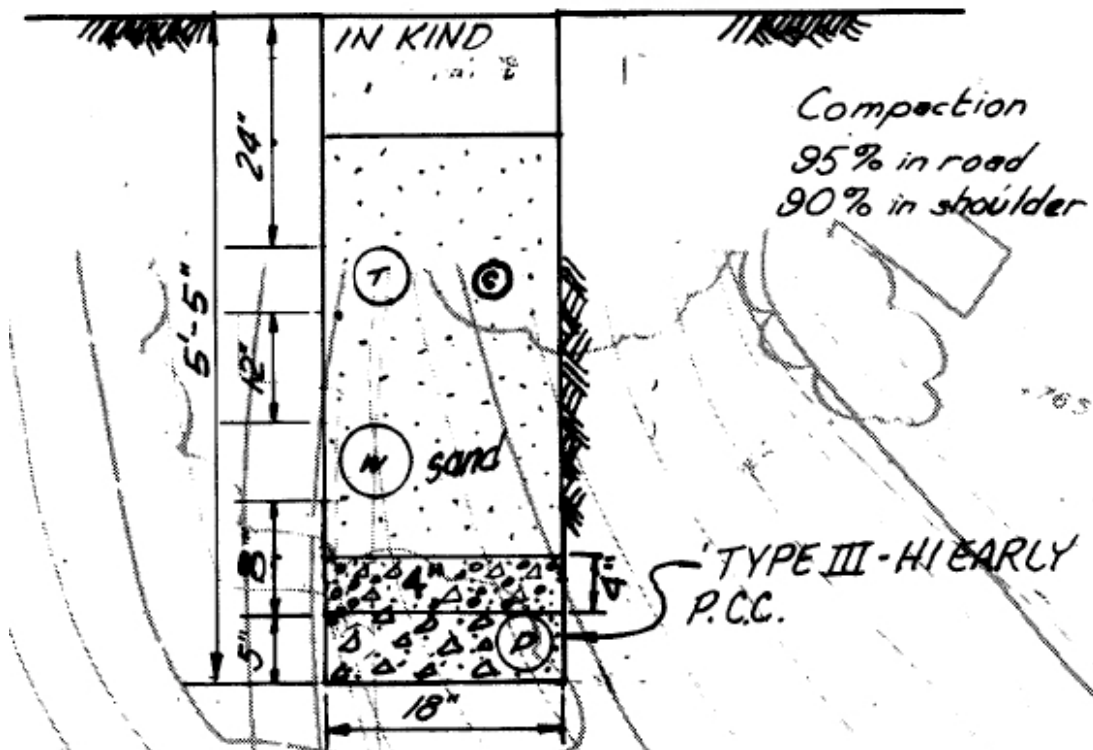


Figure D-6. AC Water Pipe Trench, Joint with Telephone, Cable, circa 1982

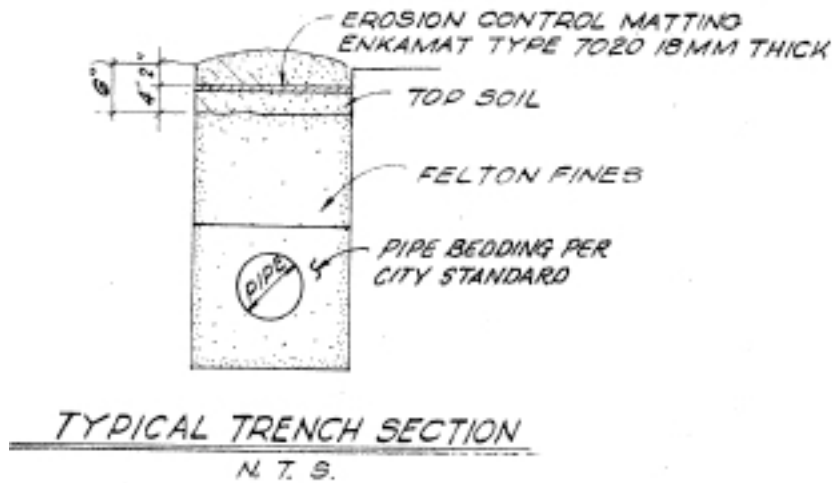


Figure D-7. AC Water Pipe Trench, Non-Road Area, circa 1982

D.2 AC Pipe Specifications – Palo Alto

Corresponding specifications for asbestos cement (abbreviated) for water pipe, from design documents for a pipeline project in Palo Alto from 1982, indicate the following:

- Water pipe shall be class 150 or class 200 AC (Transite) pipe. Pipe to be procured per AWWA C400-77.
- AC pipe ends shall be machined for standard couplings, and compatible with short body cast-iron fittings for asbestos-cement pipe.
- Couplings are Fluid-Tite, designed as manufactured by the Keasby and Mattison Company, Ring Tite as manufactured by Johns Mansfield Company, or approved equal.
- Flanges are ASA 16.1-1948, class 125. Flange gaskets.
- Bell end fittings for AC pipe, 3 to 12 inches, to conform to ASA A31.10-1952 (AWWA C110-52). Cast iron bell-end fittings for rubber gasketed AC pipe shall be furnished with hub ends to receive rubber-gasketed AC pipe ends.
- Tapping valves are flanged one end with Ring-Tite connection at the other end.
- All water service lines shall be copper.
- Trenches: contractor to save 6 inches of top soil to be placed over pipe when the trench is completed. After the pipe is laid, compact the trench well enough to prevent any settling, approximately 85% to 90% per city specifications.

- After the pipe is installed, it will be disinfected and tested per city specifications.

D.3 AC Pipe Specifications - CAPCO

The following information is based on a AC pipeline manufacturer (CAPCO) literature from 1990.

AC pipe was then available as either class 150 or class 200. The class number refers to the rated working pressure for the pipe, in psi. All pipe conform to AWWA C400. Pipe is commonly furnished in 13-foot lengths; for common projects, up to 10% might be shorter, but not shorter than 7 feet. Couplings are made of AC pipe sleeve with two solid rubber rings of uniform cross section.

Hydrostatic test. Each standard AC pipe is designed to have sufficient strength to withstand an internal hydrostatic pressure of 4 times the rated working pressure.

The uncombined calcium hydroxide present in CAPCO AC water pipe and CAPCO couplings shall not be more than 1%.

The pipe and couplings are composed of a mixture of Portland cement, silica and asbestos fiber, and free from organic substances. The rubber rings are a vulcanized rubber compound to meet ASTM-D1869.

For a 4-, 6-, 8- and 10-inch pipe, the collar length is 7 inches, the rubber gasket slot width is 0.9 inches, and the "stab length", if fully stabbed equally on both sides of the collar, is 1.68 inches. The last 0.75 inches of each pipe is narrowed. This means, that for an idealized pipe installation, the AC pipe can be pulled out of the collar 0.93 inches with no chance of leak, or 2.60 inches with 100% chance of leak, with an increasing chance of leak between 0.93 inches and 2.60 inches of pull out. For larger diameter pipes, the collar lengths are longer, and there is modestly more capability for withdraw before leak.

In practice, not all collars are perfectly centered, and in areas where the pipe is laid to form a curve, there is unequal pipe insertion lengths.

If one assumes that for earthquake ground shaking that the joint withdraw movement is correlated to PGV and c , and that the pipe segment length is 13 feet, and PGV is 30 inches per second, and c is 12,000 feet per second for vertically propagating shear waves, and the pipe moves with the ground (no slippage) and all ground strain is accumulated as movement at the rubber gasket, then: $\text{ground strain} = \text{pipe strain} = 30 / (12,000 * 12 * 2) = 0.000104$, and the movement at the gasket location is $13 \text{ feet} * 12 \text{ (inch/foot)} * 0.000104 = 0.016 \text{ inches}$. If we assume that locally the ground velocity is 50% higher, and the wave propagation speed is 50% lower, and the bulk of the ground motion is from surface waves, then the joint opening is 0.073 inches. At bottom of hills there will be a localized increase in PGV, and at transitions between stiffer to softer soils there will be some amplifications. Even with these conditions combining at one location, the joint

opening movement is still less than about 0.25 inches in tension. In compression, a fully inserted pipe at a collar will bear directly on the adjacent pipe, (note: manufacturer's recommendations is to leave a gap, but how this is achieved in the field is not certain), and with high compressive capability, there should be no failure; but the pounding between opening movements and closing movements could locally impact the pipe and crack it; if the crack is small, it will be contained by the ring and no leak occurs; but a large crack that extends beyond the rubber gasket cannot be entirely ruled out.