Alameda-North Bay Farm Island Crossings Master Plan

East Bay Municipal Utility District

November 2014

MASTER PLAN

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ACRONYMS

CEQA	California Environmental Quality Act
CFS	Underwater Pipeline Crossings Feasibility Study
EBMUD	East Bay Municipal Utility District
fps	feet per second
gpm	gallon per minute
HDD	Horizontal Directional Drilling
HDPE	High-density polyethylene
JA	Jacobs Associates
LOS	level of service
MDD	Maximum Day Demand
mgd	million gallons per day
MT	Microtunnel
MTBM	Microtunnel Boring Machine
OBM	Old Bay Mud
PID	Pipeline Infrastructure Division
RCS	Rate Control Station
R/W	Right-of-Way
SD-1	East Bay Municipal Utility District, Special District No. 1
YBM	Young Bay Mud
WDPD	Water Distribution Planning Division
WOH	West of Hills

EXECUTIVE SUMMARY

The Alameda-North Bay Farm Island Master Plan was conceived to determine the number and location of pipeline estuary crossings needed to meet existing and future demands for the City of Alameda with a high in-service reliability. The Master Plan addresses issues of long-term service to the island, vulnerabilities of the existing crossings, impacts due to major seismic events and recommendations for new crossings and their construction methods. The Master Plan also identifies the reduction in level of service related to losing any of the remaining crossings or combination of crossings.

Based on the hydraulic analyses, three primary crossing areas were identified as critical to maintaining level of service (LOS) to Alameda: the Oakland Inner Harbor crossing in the vicinity of Posey Tube, the crossing to North Bay Farm Island from Alameda and the crossing in the vicinity of Derby Avenue. A key point of the hydraulic findings is that forecast growth in demands causes a 10-20 psi drop in pressure in Alameda from current pressures. In addition, hydraulic restrictions are apparent in the transmission pipelines on the Oakland side.

The existing and active Park Street crossing can fail without consequence to the LOS on Alameda Island. A failure of the active Alameda-Bay Farm Island crossing causes pressures on Bay Farm Island to drop from approximately 56 to approximately 30 psi, so the condition of this crossing needs periodic review to plan for its replacement before it fails. The same attention should be paid to the existing Blanding crossing.

Alternative underwater crossing alignments were identified based on the need for adequate construction staging, shortest underwater crossing distances, and close proximity to the existing distribution grid and backbone pipeline at both ends of the crossings. Four of the eleven alignments identified in the initial screening were developed further through the Underwater Pipeline Crossings Feasibility Study (CFS - Volume 1 - DOX #2191923 and Volume 2 - DOX #2191925) completed by Jacobs Associates in March 2014.

The CFS examined crossings in terms of major risks and the best trenchless construction methods to reduce those risks, while maximizing survivability and minimizing repair-related water service outages following a major seismic event. Repair cost, construction cost and construction duration estimates were prepared for each of the preferred alignments.

Given that ground conditions are subject to liquefaction, lateral spread, and shoreline slope instability after a major earthquake, this study looked to microtunneling and horizontal directional drilling (HDD) technology to construct new underwater pipeline crossing in the deeper, more stable soil conditions at each crossing location.

Results of the CFS were used in the development of Info-Gap models to analyze uncertainty. Info-Gap models were developed in order to investigate robustness of predictions about the failure rate of new crossings and the time to repair those crossings in the event one does fail. The goal of the Survivability Info-Gap model analysis was to assess the robustness of survivability predictions. From an Info-Gap standpoint, the robustness of the survivability prediction for one HDD crossing alternative is good enough given the redundancy provided by the other in-service crossings. The goal of the Reparability Info-Gap model analysis was to assess the robustness of the predicted times to repair. No amount of reparability provided by the crossing alternatives was deemed good enough from a robustness perspective; therefore the recommendation is to provide redundancy in the number and location of crossings along with a resilient design approach as opposed to relying on the ability to make major repairs to failed crossings.

The first project recommended is the replacement of the existing Alice-Webster crossing at the Alternative 1D alignment (from Fallon Avenue near Estuary Park in Oakland to Marina Village Parkway in Alameda) using HDD tunneling methods and jet-grouted soil improvements at either end of the crossing. The plan and profile of the tunnel crossing alignment are shown in Figure 2-2. The new crossing will be HDPE pipeline with an additional 10,000 feet of 24-inch steel pipeline to connect to Lincoln Avenue on the Alameda side and 9th Street on the Oakland side. The tunnel and connecting pipeline alignments are shown in Figure 2-1.

Total cost of the recommended project is estimated at $14M, including environmental documentation, design, construction, construction management, and contingencies. The project is scheduled for construction in FY 2017.

In the event of a Bay Farm Island #2 pipeline crossing failure, this crossing should be valved out and replaced by a new crossing at alternative alignment 2A shown in Figure 6-2. In the event of a Blanding Street crossing failure, this crossing should be valved out and replaced by a new crossing at Derby Street (i.e., the Alternative 3A alignment shown in Figure 6-4). The existing Park Street crossing can be valved-out and removed from service without any further improvements once it fails. In any case, it is recommended that the three new crossings be budgeted for an installed prior to any failures.

CHAPTER 1 - INTRODUCTION

Background

Alameda Island (island) including the U.S. Naval Air Station at Alameda is a unique part of East Bay Municipal Utility District (EBMUD) service area. Water service for the 40,000 services (mostly residential with some commercial and industrial users) is completely dependent on several underwater pipeline crossings from Oakland to the island; there is no water storage on the island. To date, seven underwater crossings have been constructed, but only four of these crossings remain in service today (Figure 1-1).

After the most recent crossing failure (Derby Street crossing in 2009), hydraulic model investigations determined that the failure of one of the remaining crossings would lead to a reduction in available fire flow rates on the island. Further investigation was recommended in order to determine vulnerabilities of existing crossings and impacts of those failures.

The Alameda-North Bay Farm Island Master Plan was conceived to determine the number and location of pipeline crossings needed to meet existing and future demands for the City of Alameda. The Master Plan addresses issues of long-term service to the island, vulnerabilities of the existing crossings, impacts due to major seismic events, recommendations for new crossings and their construction methods, and steps in the event of failure of any of the remaining crossing or combination of crossings.

An all-pipe hydraulic model of the Central-Central Pressure Zone was developed to analyze the number, general location, and size of new crossings to Alameda. Jacobs Associates was contracted to complete a CFS to identify feasible construction methods for new crossings to the island. Results of the CFS were used in the development of Info-Gap models to investigate the uncertainty of probabilities of failure and estimated times to repair.

Goals and Objectives

1. Design at least one crossing to survive a major seismic event in order to provide minimum levels of service.

2. Design one crossing to be repairable in a short amount of time after a major seismic event.

3. Determine the vulnerability of existing crossings in order to provide minimal redundancy in the event of the failure of one or more crossings.

4. Identify cross-island transmission improvements in order to provide minimal levels of service after a major seismic event.

CHAPTER 2 - PROJECT RECOMMENDATIONS

Project recommendations were determined based on findings from the hydraulic analysis presented in Chapter 4, the March 2014 CFS (Appendix A), and the Info-Gap analyses (Appendix B). In addition, a detailed connection location and street alignment evaluation was conducted to identify crossing connection locations and alternative pipeline replacement alignments (Appendix C). Refer to Chapter 5 for detail description and discussion of alternative estuary crossings.

Alternatives 1A and 1D located in the vicinity of the existing Alice-Webster crossing were selected for further investigation in the CFS prepared by Jacobs Associates based on feasibility and cost-effectiveness. Alternative 1A is estimated to be less expensive and has good construction access on the Oakland side and fair construction access on the Alameda side. Alternative 1A has the shortest length of additional pipeline improvements required to connect to a reliable transmission main. Alternative 1D requires a long underwater tunnel, but incorporates pipeline improvements to replace more than 2,000 feet of old cast iron pipe in connecting to an existing 30-inch backbone pipeline. Coupled with good construction access at both landings, Alternative 1D is recommended as the preferred alignment to replace the existing Alice-Webster crossing.

The selected project includes the installation of a new 24-inch diameter underwater crossing at the Alternative 1D alignment along with new in-ground and street pipeline installation on the Oakland and Alameda side of the estuary; see Figure 2-1. A plan and profile drawing of the new 24-inch high density polyethylene (HDPE) pipeline crossing installation is shown in Figure 2-2. In addition, approximately 10,000 feet of new 24-inch steel pipeline will be needed to connect to Lincoln Avenue on the Alameda side and to 9th Street on the Oakland side, also shown in Figure 2-1. The new HDD drilling profile and the jet-grouted soil improvements are shown respectively in Figures 2-3 and 2-4. Jet grout supports are intended to strengthen the conductor casing through the young bay mud (YBM) into the less liquefiable older bay mud (OBM).

HDD with jet-grouted soil improvements is the selected construction method based on total cost and robustness against failure. The Info-Gap analysis also indicates that three reliable crossings provide adequate redundancy and are very robust against failure due to anticipated external events such as a major earthquake. The planned crossings at Fallon Street near Estuary Park (Alternative1D), Blanding (Alternative 3A) and Bay Farm #2 (Alternative 2) are geographically separate, significantly reducing the likelihood of multiple failures during a single earthquake.

Cost

The total estimated construction cost for Alternative 1D improvement is $4.2M based on CFS estimates for a 24-inch underwater tunnel crossing. Associated connection pipeline improvements are estimated at $4.3M. The total project cost including planning, design, construction, construction management and contingences is estimated at $14M (rounded). Costs are described further in Chapter 6 of this report. Upsizing the underwater crossing to a 30-inch HDPE pipe to accommodate potential unadjusted future demands would marginally increase the cost of the tunnel component.

Schedule

Environmental documentation is scheduled to begin in FY 2015, with design and construction to follow in FY 2016 to FY 2017, respectively.

Potential Additional Improvements and Recommended Testing

The existing Alice-Webster crossing is to be abandoned in place once the new pipeline is placed in service. Other options include the potential for slip-lining the 24-inch with a much smaller pipe for potential other uses such as recycled water if feasible.

In addition, it is recommended that the Pipeline Infrastructure Division (PID) review the mitigation methods for reducing pipeline vulnerabilities recommended in Chapter 7 of the CFS (Appendix A) for the remaining active crossings and determine the value of those improvements.

PID should consider a small-volume water pressure test of the Blanding Street and Bay Farm Island #2 crossings to test for any existing leaks. The CFS identified and expressed concern over the integrity of the Blanding pipeline installation due to short radius bends. A leak detection study conducted by an external vendor in 2011-2012 found a “potential” leak on Bay Farm #2 crossing. Pressure tests should be conducted soon, before the failure of any remaining crossings. If there is any concern about a leak or pending failure of these crossings, PID should also initiate the design and construction of a new pipeline at Alternative 2A or 3A and remove Park Street from service, as outlined in Table 2-1. This test should be repeated regularly to ensure the continued reliability of these crossings.

TABLE 2-1
Recommendations in Event of Crossing Failure

Failed Crossing	Recommended Action*
Alameda-Bay Farm #2	Valve out, remove from service, and initiate construction of new HDD – Alternative 2A Alignment
Blanding	Valve out, remove from service, and initiate construction of new HDD – Alternative 3A Alignment (Derby Street)
Park Street	Valve out and remove from service

* *Chapter 6 (Preferred Projects) provides more details on crossings alignments 2A and 3A.*

Off-Island Improvements

Figure 4-6 identifies transmission mains in Oakland that supply the island that must be maintained and eventually considered for upsizing as the opportunity becomes available.

These pipelines should be added to the Large Diameter Master Plan as backbone facilities critical to maintaining adequate LOS for Alameda and Bay Farm Island in the future. Some of these reaches may be in liquefiable area and should be relocated if so.

CHAPTER 3 - EXISTING CONDITION OF SUBMARINE CROSSINGS

Crossings Inventory

Table 3-1 is a summary inventory of the existing crossing materials and age. Figure 1-1 also shows each crossing location while the as-builts are contained in Appendix D.

TABLE 3-1
Pipeline Crossing Inventory

No.	Crossing	Size *(inches)*	Material	Joints	Install Year	Age *(years)*	Lining	Coating	Status
1	Alameda-Bay Farm 1	16	Cast Iron/ Steel	USIFLEX Class D, Flanged and Welded	1952	62	U/M	B/M	Removed from Service
2	Alameda-Bay Farm 2	24	Steel	Double Lap Welded Bell and Spigot	1983	31	M	BM	In Service
3	Alice-Webster	24	Cast Iron	USIFLEX Class D Ball and Socket	1946	68	M	U	In Service
4	Blanding	24	Steel 1/4 inches thick	Double Lap Welded Bell and Spigot	1987	27	M	BM	In Service
5	Derby	20	Cast Iron	Flanged Pipe and Expansion Joints	1935	79	U	U	Removed from Service
6	High	12	Cast Iron	Unknown	1918	96	U	U	Removed from Service
7	Park	16	Cast Iron	Flexible Joint Pipe	1918	96	U	U	In Service

Installation Year/Age *Age of pipe crossing taken from existing data inventory from 2012*
Lining *M = Mortar and U = Unlined*
Coating *U = Uncoated, BM = Insulating material with mortar overcoat, and M = Mortar*
Status *Operational status of the pipeline, either in service or killed*

<u>Alameda Bay Farm 1 Crossing</u> (out of service) is hung from the bottom of the Bay Farm Island Bridge, which connects Alameda Island to the Bay Farm Island area on the south side of Alameda. The tunnel crossing is located in the public R/W at the end of Otis Drive/ Bridge View Isle on the Oakland side and within Caltrans Bay Farm Island Bridge/Doolittle Drive R/W on the Bay Farm Island side. The total length of the 1,100-foot bridge crossing includes approximately 800 feet of pipeline suspended on the side of the bridge. In addition approximately 300 feet of pipeline runs along the bottom of the channel and was installed by open trench excavation per as-built 2050-G.

<u>Alameda Bay Farm 2 Crossing</u> is located at the south end of Alameda Island and connects to the Bay Farm Island area, about 150 feet west of the Bay Farm Island Bridge. The crossing is in the public R/W at the end of Otis Drive/Bridge View Isle on the Oakland side and in the

public R/W of the Veterans Court cul-de-sac on the Bay Farm Island side. The approximately 1,300-foot crossing was installed by open trench excavation per as-built W-5463-3.

Alice-Webster Crossing is located on the west end of the island, approximately 300 feet south of the Posey Tube automobile tunnel connecting Oakland and Alameda. The tunnel crossing is located in an 81-foot right-of-way (R/W 2731) at the southwest end of Alice Street on the Oakland side and in a 10-foot R/W 647 adjacent to the silos within the Barnhill Marina property limit on the Alameda side. The approximately 1,100-foot crossing was installed in 1946 by open trench excavation per as-built W-419.

Blanding Street Crossing at Oak Street is located on the east side of the island, about 700 feet northwest of the Park Street Bridge. The tunnel crossing is located in a 20-foot R/W 3528 and 3443 southwest of Kennedy Street through a motor boat yard on the Oakland side and in a 20-foot R/W 3419 in a paved parking lot on the Alameda side. The 823-foot tunnel crossing was installed by directional drilling per as-built W-6497.

Derby Street Crossing (out of service) is located on the east side of the island, approximately 900 feet northwest of the Fruitvale Bridge. The tunnel crossing is located in the public R/W in Derby Avenue on the Oakland side and in a 20-foot R/W 2636 through a concrete sidewalk and a paved parking lot in front of Nob Hill supermarket on the Alameda side. The 475-foot crossing was installed by open trench excavation per as-built 379-G.

High Street Crossing (out of service) is located on the east side of the island, adjacent to the High Street Bridge, in the public R/W in High Street on both the Oakland and Alameda side. The 410-foot crossing was installed using open trench excavation per as-builts W-47 and W-48.

Park Street Crossing is located on the east side of the island, adjacent to the Park Street Bridge. The tunnel crossing is located in the public R/W in 29th Avenue on the Oakland side and in Park Street on the Alameda side. The 450-foot crossing was installed via unknown installation methods. The pipeline profile is shown in drawing W-50.

Leak History

In 2011-2012, Echologics Engineering assessed the Alice-Webster, Park Street, Blanding (Oak Street) and Alameda-Bay Farm No. 2 crossings for potential leaks using acoustic correlation technology (DOX #2190335). The results of this investigation though inconclusive are summarized in Appendix E. One potential leak was detected approximately 150 feet from the north end of Alameda Bay Farm #2 crossing, but was not investigated further to verify its existence.

While there are no known leaks on any of the four in-service crossings, there is a history of leaks along the on-shore alignments. Failures on the Alameda side of the Alice-Webster tunnel occurred twice in the past 10 years where the pipeline is about eight feet deep. The leak history shown in the Aqueduct Leak Detection Study (Appendix E) includes pipe repairs compiled from the General Work Order system database. Table 3-2 includes a summary of recent pipeline repairs in the vicinity of the existing crossings.

TABLE 3-2
Summary of Recent Pipeline Repairs

Approximate Distance to Crossing *(feet)*	GWO #	Address	B-Map	Diameter *(inches)*	Pipe Material	Installation Year	Pipe Extension	Date of Leak Repair
Alice-Webster								
2,600	1519078	718 Alice Street Oakland	1488B476	24	CM	1946	E-25661	5/29/2012
400	1177233	55 Alice Street Oakland	1485B474	6	CM	1944	E-25181	12/28/2006
1300	1232415	Webster 550 South of Mariner Square Loop Alameda	1485B472	24	CM	1946	E-25661-A	11/11/2007
Park Street								
500	1444733	2900 Glascock Street Oakland	1497B466	12	C	1929	E-15784	1/9/2011
1700	668150	1611 Park Street Alameda	1497B466	12	S	1920	E5014	11/11/1999
Blanding Street								
600	1322151	421 23rd Avenue Oakland	1497B468	6	A	1978	45017	4/10/2009
1000	1451902	2832 Chapman Street Oakland	1497B468	6	C	1910	E-1138	1/31/2011
900	1108982	2814 Chapman Street Oakland	1497B468	6	C	1910	E-1138	12/6/2005
1050	932998	614 23rd Street Oakland	1497B468	24	CM	1944	E-25252-C	7/19/2003
Derby Street								
700	1444733	2900 Glascock Street Oakland	1497B466	12	C	1929	E-15784	1/9/2011
600	477217	510 Derby Avenue Oakland	1497B466	8	CM	1947	E-26559	1/9/1997
400	1122069	400 Derby Avenue Oakland	1497B466	6	C	1924	E-9994	2/28/2006
650	1179872	2934 Ford Street Oakland	1497B466	6	CM	1940	E-22665	1/16/2007
High Street								
1500	633443	2007 Cambridge Drive Alameda	1500B464	6	A	1938	E-21301	5/4/1999
500	819340	3221 Fernside Drive Alameda	1500B464	6	CM	1954	E-31525	1/22/2002
950	1112117	500 Howard Street Oakland	1500B464	6	SMM	1966	40248	12/26/2005
1200	1179690	1700 Cornell Drive Alameda	1500B464	6	C	1925	E-11156	1/13/2007
500	1372191	3045 Marina Drive Alameda	1500B464	8	A	1940	E-22406	3/9/2010
650	1408867	3230 Fernside Drive Alameda	1500B464	6	C	1914	E-2528	9/24/2010
1500	1461309	1915 Cambridge Drive Alameda	1500B464	6	A	1938	E-21301	3/3/2011
400	1511939	3105 Marina Drive Alameda	1500B464	8	A	1940	E-22406	11/21/2011
Alameda-Bay Farm Island Crossings								
200	1294001	3300 Otis Drive Alameda	1497B458	16	SMM	1952	E-30355	11/7/2008
450	1325695	75 Veterans Court Alameda	1497B458	16	SMB	1980	45247	5/4/2009

Operation and Maintenance Concerns

Alameda Bay Farm 1 Crossing is currently out of service due to a leak. Repair of the submarine cast iron section of the pipe also requires dredging and underwater construction. Repair of the steel pipe section suspended under the bridge requires use of a barge and scaffolding. Repair of the existing buried or submarine pipe sections may require an encroachment permit from Caltrans or the City of Alameda.

Alameda Bay Farm 2 Crossing currently supplies approximately one tenth of the total water supply to Alameda Island. The pipe and isolations valves located in Veteran's Court on Bay Farm Island are buried over 10 feet deep, making access and repair difficult. In addition, the valve extensions are rusting due to salt water exposure. Repairs to the submarine section of the 24-inch diameter steel pipe would require dredging and underwater construction.

The main on the Oakland side located in the public R/W at the end of Otis Drive/Bridge View Isle is accessible for repairs or new pipeline installation based on space and ease of access. Mains on the Bay Farm Island side located in the public R/W in Island Drive and Veterans Court (cul-de-sac) are also accessible for repairs or new pipeline installation based on space and ease of access.

Alice-Webster Crossing currently supplies approximately half of the total water supply to the Alameda and Bay Farm Island based on July 2006 summer day demands. Leak repairs on the submarine pipe sections would be extremely difficult to make, given that the bottom of the pipeline is about 45 feet below the water surface and covered with about 10 feet of backfill. In order to provide adequate water to the island, a temporary pipeline would have to be installed in the Posey Tube automobile tunnel or placed on the bottom of the channel.

Repair and maintenance access is limited on the Alameda side within a narrow 10-foot wide R/W inside the Barnhill Marina property. Pipeline maintenance and repair access is further limited where the pipeline alignment is closer to the southern row of silos.

The 24-inch main crosses an open field and railroad track then emerges on Webster Street in Alameda just outside the tunnel. Two pipeline failures on the Alameda side in the past 10 years flooded the Posey Tube. The pipe is 8 feet deep in Webster Street and would be difficult to find and access. As the pipe emerges from the open field off Marina Loop, it passes under the freeway on ramp.

Blanding Street Crossing at Oak Street currently supplies approximately 30 percent of the total water supply to Alameda Island based on July 2006 summer day demands. The deepest part of the submarine tunnel section is about 45 feet below the surface and thus difficult to repair.

The main on the Oakland side is located in a 20-foot R/W inside a motor boat yard and crosses through a boat repair parking lot; valves were not found during the site visit. The

24-inch main then crosses under Pacific Union rail road tracks inside a 36-inch steel culvert. The pipeline on the Alameda side located inside an asphalt-paved parking lot in front of a storage building and has limited accessibility**.**

Derby Street Crossing failed on May 5, 2009. DRS Marine Inc. was contracted by EBMUD in June 2009 to locate and repair the damaged 20-inch cast iron pipe section. However their efforts were unsuccessful given the fluidity of the channel sediment[1]. The Derby Crossing was not repaired, but was valved out and removed from service.

The main on the Oakland side is located in the public R/W on Derby Avenue and is accessible for installation or repairs based on space and ease of access. The pipeline on the Alameda side located in a 20-foot R/W located across the entire front of the Nob Hill market/parking area. Repair access is good. The alignment nearest the waterfront is under a wide concrete sidewalk walkway which would need to be restored in case of any pipeline installation or repair work, as would the asphalt parking area.

High Street Crossing is currently out of service due to a leak. Repair of the submarine pipe section is not recommended due to its age, material and condition. The 12-inch cast iron pipe was installed in 1918 and is unlined and uncoated. Severe corrosion of the submarine pipe is anticipated based on the pipe's age and material.

The pipeline on both of Oakland and Alameda sides has limited accessibility due to crowded roads and intersections.

Park Street Crossing currently supplies approximately one tenth of the total water supply to Alameda Island from the 16-inch cast iron submarine pipeline installed in 1918. If a leak occurs on the submarine pipe section, the repair would require the assistance of a diver.

The main on both the Oakland and Alameda side is located in the 29th Avenue and Park Street public R/W. Accessibility is limited on the Alameda side near the bridge due to the close proximity of the bridge on-ramp retaining wall.

Vulnerability Assessment

In September 2012, EBMUD's Materials Engineering Section (Materials Engineering) performed a preliminary geotechnical evaluation for the existing Alameda/Bay Farm Crossings (Appendix F). The purpose of this evaluation was to rank the vulnerability of each of the seven pipeline crossings to failure caused by a seismic event, based on subsurface soil types, potential for liquefaction and seismic shaking amplification, and the pipeline depth and alignment relative to these soil types. Materials Engineering relied

1. *DRS Marine Inc. report "20" Water Pipeline Estuary Crossing dated June 22, 2009.*

on previous geotechnical investigation reports to make an assessment[2]. No subsurface investigations were conducted for this evaluation.

The CFS, completed in March 2014, includes a vulnerability assessment for the existing and proposed Alameda pipeline crossings based on seismic risk due to wave propagation and peak ground displacement, as well as the corrosion/aging vulnerability for the seven existing pipelines. Included in Appendix A, the CFS is based on available information and no subsurface investigations were performed. The pipes were analyzed under four possible scenario earthquakes: Hayward M 7.0 (which also accounts for a more distant San Andreas M 8.0), Hayward M 6.0, Calaveras M 6.75, and Concord M 6.5. The information is intended to be used as part of an overall risk assessment for reliability of existing crossings to Alameda Island.

The Materials Engineering Evaluation memo and the CFS generally agree on the vulnerability ranking of each of the crossings. However, there are significant differences in the ranking of Park Street Crossing and High Street Crossing. Materials Engineering ranks both as high risks while CFS ranks both as low risks. The Materials Engineering Evaluation uses more conservative assumptions for these crossings and puts more emphasis on site specific information (e.g., soil types and layers) whereas CFS puts more emphasis on a general seismic analysis of the area (e.g., wave propagation and peak ground displacement). The Materials Engineering Evaluation is more conservative, potentially leading to a more robust result.

<u>Pipeline Seismic Vulnerability</u> - During the event of an earthquake, the most concerning factors that pose risks to pipeline failure are the presence of YBM and liquefiable soils. Pipeline vulnerability is significantly related to the type of soils the pipeline traverses through and the soil layers located beneath the pipeline, which can move and shake the pipeline during a seismic event. Soil layers located above the pipeline do not affect the pipeline vulnerability; these layers can move and shake without affecting the pipeline stability.

YBM is soft, unconsolidated silty clay, saturated with water. Due to its high water content and loose packing, YBM has low resistance to penetration, which makes the soil very weak. Although YBM has a low likelihood of liquefaction, it remains a seismic hazard due to its propensity of high seismic shaking amplification. These strong levels of shaking induce differential stresses that can potentially rupture a pipeline that traverses through YBM or above YBM. OBM is more compacted than YBM and has high resistance to penetration. Therefore, pipelines located in or below OBM layers are not as vulnerable to failure during a seismic event.

In addition to YBM, liquefiable soils are present throughout the Oakland-Alameda estuary and pose a high risk to the pipeline installations. During an earthquake, liquefiable soils are susceptible to liquefaction. If shear stresses result from sloping

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2. *Primary reference used was Final Geotechnical and Environmental Investigation, EBMUD Alameda NAS Discharge Pipeline and Siphon Project, prepared by Olivia Chen Consultants, dated September 24, 2000.*

terrain (i.e., the crossing pipeline approaches) during liquefaction, lateral spreading can occur, generating ground failures that can stretch the pipeline to the point of breakage. Seismically induced liquefaction is most likely to occur in beds of loose, water-saturated, well-sorted sand within 100 feet of the ground surface. Therefore, the sandy layers within the area of the pipeline crossings increase the vulnerability to liquefaction.

Artificial fill is liquefiable and acts similar to a sand layer. Alice Crossing, Alameda-Bay Farm No. 1 and No. 2 Crossings, and High Street Crossings traverse through artificial fill. Portions of the Alameda Island are still settling due to consolidation of the underlying soil from the weight of the fill. Pipeline sections that traverse the artificial fill are at high risk of failure during a seismic event, especially on sloping sections, which could liquefy and cause lateral spreading.

On flat ground, liquefiable soils settle during an earthquake. This type of settlement is typically not significant enough to break the pipe. However, on sloped surfaces, liquefiable soils settle and spread during an earthquake (lateral spreading), which is a much higher risk for pipeline failure. Lateral spreading can easily stretch the pipeline to the point of failure. Therefore, liquefiable soils on the sloped pipeline approaches pose a higher risk to the pipeline vulnerability than liquefiable soil layers within the submarine portion of the pipeline.

<u>Vulnerability Rankings</u> - The soil layers in the vicinity of the approaches and submarine section of each existing crossing are listed in Table 3-3. A summary of the vulnerability analysis and the vulnerability ranking of each crossing is also provided. The vulnerability is ranked on a scale of 1 to 5 (1 being the least vulnerable, and 5 being the most vulnerable).

Alice-Webster Crossing is the most vulnerable crossing. Alameda-Bay Farm No. 2, High Street, and Park Street Crossings have a high vulnerability ranking as well.

TABLE 3-3
Alameda Pipeline Crossings Vulnerability Evaluation

Crossing	Soil Layers in Vicinity of Pipeline Profile *(in order of top layer to bottom layer)*			Vulnerability Assessment Summary	Vulnerability Ranking [2]
	Approach on Alameda Side	**Submarine Portion**	**Approach Opposite Alameda Side[1]**		
Alice-Webster	Artificial Fill YBM Sand, Clayey Sand OBM	OBM / YBM Sand, Clayey Sand OBM	Artificial Fill YBM Sand, Clayey Sand OBM	Pipeline traverses through sandy layers (loose and liquefiable) in the submarine portion and the sloping Oakland approach, and through artificial fill at both approaches, putting pipe at high risk of lateral spreading and pipe failure during a seismic event. Pipe also traverses YBM at both approaches, which can undergo large amplifications of seismic shaking and break pipe during a seismic event.	5
Blanding	Silty Clay	Silty Clay	Silty Clay, Silty Sand	Pipeline is embedded deep in the soil (avoiding loose, potentially liquefiable layers above) and traverses mostly through silty clay layer, which does not pose a high risk to the pipeline.	1-2
Park	Mud Fine Yellow Sand Stiff Gray Clay	Mud Stiff Gray Clay	Mud Sand Stiff Gray Clay	Information is very limited for this crossing. A sand layer appears to be in both approaches and can pose a risk to liquefaction and lateral spreading in a seismic event. Liquefaction cannot be ruled out, and the crossing is assumed a high risk.	4-5
Derby[3]	Stiff black adobe clay Yellow sandy clay	Stiff gray/yellow clay	Stiff yellow/blue clay (with considerable amount of sand)	Information is very limited for this crossing. In the submarine portion, pipeline is embedded deep in the soil, avoiding YBM and sand layers above. The 2:1 slope at the approaches contains clay layers with a considerable amount of sand (which could potentially be loose and liquefiable), and has potential for lateral spreading and pipe failure during a seismic event. However, more soil information is needed to provide an accurate vulnerability ranking. The crossing is assumed a medium-high risk.	3-4
High[3]	Gray sandy clay Soft Mud Loose sand and gravel Stiff clay	Soft Mud Gray sandy clay	Soft Mud Gray sand Sandy clay	Pipeline could not be located at the Alameda approach, so more information is needed for accurate vulnerability assessment at this approach. Sand layers are present below this pipeline and are potentially liquefiable and pose risk to lateral spreading and pipeline damage during a seismic event. However, more soil data is required for an accurate assessment. The crossing is assumed a high risk.	4-5
Alameda-Bay Farm[3] #1	Above Soil	Mostly Above Soil Partial Embedment: Likely YBM OBM	Above Soil (Bay Farm Side)	Pipeline runs along the bridge span, except for a portion that goes under the deck of the draw bridge. Little information is known about the embedded portion of pipeline. The bridge has deep foundation piles, so the section of pipe along the bridge span is at low risk of failure.	2
Alameda-Bay Farm #2	Artificial Fill YBM Sandy Silt (low plasticity) OBM	YBM OBM	Bay Farm Approach: Artificial Fill YBM OBM	Pipeline traverses through YBM, with high risk of amplified ground shaking that could break the pipeline during a seismic event. Artificial Fill on both approaches is prone to liquefaction-induced lateral spreading during a seismic event.	4-5

1. *Oakland side except for Alameda-Bay Farm crossings, which is the Bay Farm Island Approach.*
2. *Vulnerability Ranking is from 1-5 (5 being the most severe risk)*
3. *Out of Service*

CHAPTER 4 - HYDRAULIC ANALYSIS

An all-pipe hydraulic model of the Central-Central Pressure Zone (G0A3-G0A6) was developed to analyze the effects of forecast demands on the island's LOS and future pipeline crossing configurations; the model area is shown in Figure 4-1. This chapter includes a detailed discussion of the hydraulic model results, including the potential impact on LOS if key crossings are not in place, as well as impacts to available fire flows.

Pressure Zone Area

The Central Pressure Zone is divided into seven planning subzones designated G0A1 through G0A7 owing to its large size and area-to-area connectivity. The Central-Central portion of this pressure zone is the area served by Central Reservoir and contains subzones G0A3 through G0A6. In the north, the Central-Central Pressure Zone is separated from the North Reservoir area (G0A1 and G0A2) at 59th Street in the Cities of Emeryville and Oakland. In the south, the Central-Central Pressure Zone is separated from the South Reservoir area (G0A7) by a line of mostly closed zone gates along 96th Avenue in Oakland. Some water enters the study area from the adjacent subzones due to differences in hydraulic grade lines and the interconnection of smaller distribution pipelines. For conservative sizing of the crossing pipelines, these other subzone contributions were not considered as part of the Crossings Master Plan analysis.

The Central-Central Pressure Zone is supplied entirely by gravity flow through five rate control stations (RCS) and Central Reservoir. Most of the supplied water comes from the Orinda Water Treatment Plant via the Aqueduct Pressure Zone (G1Aa). The existing overflow elevation of Central Reservoir is 202 feet. **Central Reservoir is scheduled to be replaced at a higher elevation; therefore the higher overflow elevation of 222 feet was used for this analysis.**

Hydraulic Model

A seven-day extended period simulation was performed using a combined Central and Aqueduct Pressure Zone all-pipe model. This model was verified using July 21-27, 2005 Op/Net data and these values are used for all existing maximum day conditions listed in this hydraulic modeling analysis. Once existing conditions were established, static runs of the Central-Central Pressure Zone were conducted to evaluate future level of service conditions and fire flows.

<u>Demands</u> - Current and future demands are summarized in Table 4-1, including adjusted and unadjusted projections for 2030. Unadjusted demands do not reflect savings from future water conservation efforts, whereas adjusted demands do. Demands from the heat wave that occurred July 22-29, 2006 applied to the Central-Central Pressure Zone (G0A3-G0A6) were used to identify existing flow and pressure conditions. Future scenarios were then developed based on projected 2030 maximum day demands (MDD) based on the 2040 Demand Study. The hydraulic

analysis used Central-Central 2030 MDD of approximately 83 million gallons per day (mgd), which is about 40 mgd higher than seen during the July 2006 heat wave (43 mgd).

TABLE 4-1
Modeled Maximum Day Demands
(mgd)

		2030 Projected Demands	
Area	**2006 Actual**	**Adjusted[1]**	**Unadjusted[2]**
Central-Central	42.7	-[3]	82.5
Alameda Island[4]	9.9	15.7	20.7
Bay Farm Island	3.3	4.2	5.0

1. *Reflects forecast conservation.*
2. *Excludes forecast conservation.*
3. *Adjusted 2030 MDD demands were not modeled over the Central-Central Pressure Zone region given current estimate that localized MDD may not be affected by annual conservation practices.*
4. *Includes Alameda Point development (former Naval Air Station) in projected demands, 1.5 mgd (adjusted) and 4.2 mgd (unadjusted), based on the 2040 Demand Study.*

Though the water conservation goals may be reached, there is concern that conservation activities that reduce (adjust) the average annual demands may not significantly reduce the peaking factor on a periodic maximum day. Flow and pressure conditions were checked for both unadjusted and adjusted demand scenarios to gauge the change in the LOS. Unadjusted/higher demand scenarios were used to verify crossing sizes. As time goes by, this condition can be further evaluated.

Facilities **-** Central Reservoir and the San Pablo Clearwell provide the majority of the storage in the Central-Central Pressure Zone for Alameda. While the West of Hill (WOH) model shows that Central-South Pressure Zone (G0A7) can supply water north into the Central-Central Pressure Zone, via Dunsmuir and South Reservoirs, at full 2030 build out, no contribution from Central-South Pressure Zone was applied here as ignoring this small contribution is more conservative in sizing the new crossings. The contribution from Dunsmuir was however considered in sizing South Reservoir. Central, Church, and Genoa Rate Control Stations (RCSs) provide flow into the pressure zone from the Aqueduct Pressure Zone.

Monitoring Nodes **-** The node locations shown in Figure 4-2 were used to monitor LOS changes at representative high and low elevation services across the study area.

Existing and Future Level of Service Summary

Static model runs were conducted with Alameda and Bay Farm Island only and then with the full Central-Central model incorporated. Service pressures at several monitoring nodes are summarized in Table 4-2, including those for 2030 adjusted and 2030 unadjusted demands. Corresponding to the pressures, crossing flows are summarized in Table 4-3.

TABLE 4-2
Monitoring Node LOS Results
(psi)

	2006 Demands[1]	2030 Adjusted[1]	2030 Unadjusted[2]
Alameda Point	76	57	46
SW Island off Central Avenue	76	64	54
Mariner Square Drive	76	65	56
Central Island off Lincoln Avenue	68	57	50
Park Street at Lincoln	68	58	52
Otis Drive	75	62	56
Harbor Bay Parkway on Bay Farm Island	76	61	56

1. *Four existing crossings in service*
2. *New/replacement crossings at Estuary Park, Derby and Bay Farm Island; Park Street crossing not in service*

TABLE 4-3
Crossing Flows Into Alameda
(gpm)

	2006 Demands[1]	2030 Adjusted[1]	2030 Unadjusted[2]
Existing Alice-Webster Crossing	4,400	6,600	
Existing Blanding Crossing	2,400	4,500	
Existing Park Crossing	1,000	1,800	
Existing Bay Farm #2 Crossing	(950)	(2,000)	
New 24-inch Crossing at 1D			8,000
New 24-inch Crossing at 2A			(1,700)
New 24-inch Crossing at 3A			8,000

1. *Four existing crossings in service*
2. *New/replacement crossings at Estuary Park, Derby and Bay Farm Island; Park Street crossing not in service*

The results of some 15 model runs are shown in Table 4-4 (Hydraulic Model Run Summary) including references to Figures 4-3,4-4, 4-5, 4-7, 4-8, and 4-9 in Chapter 4.

TABLE 4-4
Hydraulic Model Run Summary

Run No.	Run Description[2]	Flows Into Alameda (gpm)					Pressures at Monitoring Node Locations (psi)[1]							Remarks
		Alice-Webster	Blanding	Park	New Derby	Bay Farm Island[3]	Alameda Point Node 1	SW Island, off Central Avenue Node 2	Mariner Square Drive Node 3	Central Island off Lincoln Node 4	Park Street at Lincoln Node 5	Otis Drive Node 6	Harbor Bay Parkway on Bay Farm Island Node 7	
0	2006 MDD, Existing conditions	4400	2400	1000		-950	76	76	76	68	68	75	76	see Figure 4-3
1	2030 MDD, Existing crossings in service	6600	4500	1800		-2000	57	64	65	57	58	62	61	see Figure 4-4
2	2030 MDD, After Alice-Webster Failure, Blanding and BF#2 Remain In-service		10700			-200	36	42	42	39	50	55	58	see Figure 4-7
3	2030 MDD, New 24" Alice-Webster @ 1D, existing crossings in service	6200	4700	1900		-1900	55	62	62	56	57	62	61	
4	2030 MDD, New 24" Alice-Webster @ 1D, After Park Failure	6400	6300			-1800	55	62	62	56	57	62	61	
5	2030 MDD, New 24" Alice-Webster @ 1D, After Blanding Failure	7300		4800		-1200	53	60	60	54	54	60	60	
6	2030 MDD, New 24"Alice-Webster @ 1D, After Blanding and& Park Street Failures	10000				850	46	52	53	45	43	52	56	see Figure 4-8
7	2030 MDD, New 24" Alice-Webster @ 1D and New 24" Derby @ 3A, After Blanding and Park Failures	6500			6200	-1800	55	61	62	55	57	62	61	
8	2030 MDD, New 24"Alice-Webster @ 1D, After BF#2 Failure	5500	3900	1500			57	63	64	58	59	66	58	
9	2030 MDD, New 24" Alice-Webster @ 1D and New 27" Derby @ 3A, After Blanding & Park Failures	6300			6500	-1900	55	62	62	56	58	62	61	
10	2030 MDD, New 24" Alice-Webster @ 1D and New 36" Derby @ 3A, After Blanding & Park Failure	6100			6900	-2100	55	62	63	56	58	62	61	
11	2030 MDD, New 24" Alice-Webster @ 1D and New 24" Derby @ 3A; no other crossings in service	5800			5000		56	63	63	57	58	66	58	
A	2030 MDD- *Unadjusted, Full Central Model*, New 24" Alice-Webster @ 1D, existing Blanding and Park	7500	6100	2500		-1800	47	56	58	51	54	57	57	
B	2030 MDD- *Unadjusted, Full Central Model*, New 24" Alice-Webster @ 1D, Replace Derby @ 3A (24")	7400			7000		48	57	58	52	55	61	28	see Figure 4-9
C[4]	2030 MDD- *Unadjusted, Full Central Model*, New 24" Alice-Webster @ 1D, Replace Derby @ 3A (24"), New 24" crossing @2A/BF#2	8000			8000	-1700	46	54	56	50	52	56	56	see Figure 4-5

(1) Monitoring Node locations, listed from west to east.
(2) 2030 MDD adjusted to reflect forecast conservation, unless otherwise noted.
(3) Negative values reflect flows to Bay Farm Island from Alameda.
(4) Recommended long-term configuration.

Based on 2006 MDD in the Central-Central Pressure Zone, Alameda and Bay Farm Islands typically see pressures of 68-76 psi across Alameda Island. Typical flows through the four active crossings as well as pressures at select nodes on Alameda and Bay Farm Islands are shown in Figure 4-3 reflecting existing conditions. Positive numbers reflect flow into Alameda while negative (bracketed) values denote flows leaving Alameda Island. As shown in Figure 4-4, pressures are expected to drop to about 57-65 psi across the study area under adjusted 2030 maximum day demands with the same four existing crossings in service.

If water conservation does not significantly reduce demand on the maximum day, unadjusted 2030 maximum day demands with the recommended configuration cause pressures to drop up to 10 psi to 46-56 psi as shown in Figure 4-5. As these reduced pressures are unacceptable given customer plumbing, fire sprinklers, and current fire hydrant flows, upsizing some of the supply pipelines from Oakland will be critical to maintaining service pressures closer to the “existing conditions”. Figure 4-6 shows the general location critical pipes serving Alameda. These transmission mains must eventually be upsized to maintain the existing level of service for Alameda and Bay Farm Island.

Failure Scenarios

In case of a failure of the Alice-Webster crossing (or the new Estuary Park crossing), pressures drop to the levels shown in Figure 4-7. In this case, Blanding (or the new Derby crossing if in service) provides all the flow to Alameda and Bay Farm Island. The most significant pressure reduction is seen in the Alameda Point area (modeled as one node) where pressures fall below 40 psi to 36 psi.

The loss of a connection in the northeast corridor of the island (either the existing Blanding or the new Derby crossing) does not cause a significant reduction in LOS across the island as shown in Figure 4-8. The flow through the Bay Farm Island connection reverses and Bay Farm Island then serves Alameda.

Each of the above failure scenario results are presented with adjusted MDD scenarios. The failure of Bay Farm Island shown in Figure 4-9 is presented with unadjusted demands because this scenario represents the largest reduction in pressure and therefore most conservative results. In this case, pressures drop to 28 psi significantly reducing the level of service to Bay Farm Island; however, Alameda maintains acceptable levels of service (48-61 psi).

Fire Flow

A fire flow analysis was performed using the 2030 adjusted maximum day demand over the Central-Central Pressure Zone and with the "new" Central Reservoir at 70 percent of capacity. Pipeline velocities were limited to 10 feet per second. Available fire flows were analyzed for the future crossing configuration and in the case of failure of any of the new crossings. The results are summarized in Table 4-5.

TABLE 4-5
Residual Pressure (psi) at 1,500 gpm,
Adjusted 2030 MDD

Area	Final Crossing Configuration	Alice-Webster Failure	Bay Farm Island Failure	Blanding/Derby Failure
Alameda Point	27	900 gpm*	1,300 gpm*	22
Eastern Alameda	38	54	30	35
Bay Farm Island	53	48	350 gpm*	51

* *Available fire flow at 20 psi*

Conclusions

1. Three reliable crossings are required long-term to maintain a reliable LOS to Alameda Island are Fallon Street (Estuary Park) to Marina Village Parkway, across the San Leandro Channel from Bridge View Isle to Veterans Court, and near the existing Derby crossing from Derby Avenue to Broadway.
2. Forecast growth in demands causes a 10-20 psi drop in pressure in Alameda from current pressures.
3. A failure of the active Alameda-Bay Farm Island crossing causes pressures on Bay Farm Island to drop from about 56 to 28 psi in 2030.
4. Hydraulic restrictions are apparent in the transmission pipelines on the Oakland side with underwater crossings modeled with a 24-inch inside diameter.
5. Velocities in the 24-inch crossings modeled under the 2030 MDD scenario are approximately 5 to 6 fps with flows ranging from 7,000 gpm to 8,000 gpm.
6. Supply pipelines on the Oakland mainland side must also be upsized (and a few relocated) to be able to maintain service pressures closer to the "existing conditions" as demands increase.
7. New/replacement crossings should not be sized for less than 24-inches inside diameter.
8. The Park Street crossing can fail without consequence to the LOS on Alameda Island.

CHAPTER 5 - ESTUARY CROSSING ALTERNATIVES

Initial Alternative Screening

Based on the results of the hydraulic modeling analysis of Alameda's water supply and distribution, three pipeline connection locations around Alameda Island were identified as robust toward delivering Alameda's water supply during and after an emergency. These three locations are identified as: Alice-Webster, Alameda-Bay Farm Island, and Northeast Corridor vicinities (Figure 5-1). At least one reliable pipeline crossing is required for each of these three areas. Based on these vicinities, several pipeline crossing alignment alternatives were identified for evaluation.

Eleven alternative alignments were identified based on the need for adequate construction staging, length of the underwater crossing (shorter is preferred), and close proximity to the existing distribution grid and backbone pipelines at both ends of the crossing. These alternative alignments were further investigated and analyzed, based on construction accessibility on both sides of the crossing, distance of additional piping needed to connect to a reliable transmission main (not just the closest part of the distribution grid), geology and geotechnical considerations including soil liquefaction susceptibility (Figure 5-2) and construction costs. The summary of crossing descriptions, advantages, and disadvantages for each of the developed crossing alignment alternatives is described below and summarized in Table 5-1. The four alignments selected for further study are both underlined and highlighted in yellow in Table 5-1.

Alice-Webster Vicinity Crossing Alignments Alternatives

Presently, the existing 24-inch Alice-Webster crossing installed in 1946 is the only potable water pipeline crossing located in this area. The cast iron pipeline is currently in service and is crucial to Alameda's water supply. A crossing in this area will also supply water for the future Alameda Point residential development, which will substantially increase the water demands in the northwest vicinity of the island.

Five crossing alignments were examined as alternatives to replace the existing Alice-Webster crossing (Figure 5-3). All five alternative alignments avoid the congested Mariner Square Drive and Webster Street area in Alameda and avert the need for a Caltrans R/W permit to work near the Posey Tube entrance and Webster Street Tunnel exit. Furthermore, all five alternative alignments connect to the new 16-inch Lincoln Avenue transmission pipeline in Alameda, in order to ensure a reliable connection to the distribution system.

TABLE 5-1
Crossing Replacement Alternatives

Alt#	Alternative Description	Crossing Length[1] *(feet)*	Length to Connect to Existing System[2] *(feet)*	Length of Additional Improvements[3] *(feet)*	Construction Accessibility[4] *(North South)*	Addresses Old/Leaking Pipes in Vicinity of Crossing[5]	Replaces Pipe Within Liquefaction Zone[6]	Construction Cost Estimate[7] ($ M)
Alice-Webster								
1A	Existing Alignment Connect to Constitution Way/Lincoln Avenue on Alameda side, replace 24CM46 north to 9th Street on Oakland side	1,300	0	7,700	good to fair	good	good	**$ 4.7**
1B	Existing Alignment Reroute around Mariner Square Drive and connect to Webster at Willie Stargell Avenue on Alameda side, connect at 9th and Alice Streets on Oakland side	1,300	3,400	9,700	good to fair	good	good	**$ 7.1**
1C	New Alignment Main Street to Union Pacific, west of existing crossing alignment and also west of the Turning Basin	1,900	1,300	14,000	poor to fair	poor	fair	**$ 8.8**
1D	New Alignment Fallon Street to Marina Village Parkway, east of existing crossing alignment	1,700	400	8,400	good to fair	fair	good	**$ 5.4**
1E	New Alignment Washington Street to Mitchell Avenue, west of existing crossing alignment	1,200	2,700	11,200	fair to good	fair	good	**$ 7.3**
Alameda-Bay Farm Island								
2A	Existing Alignment Cross bay only	900	500	0	fair to good	fair	fair	**$ 1.2**
2B	Broadway to Sea View Parkway West of existing crossing alignment	2,200	0	1,100	fair to good	fair	fair	**$ 1.5**
Northeast Corridor								
3A	Existing Derby Avenue Alignment Connect to Ford Street and 29th Avenue in Oakland, connect to Lincoln Avenue and Park Street in Alameda	500	0	5,000	good to good	good	fair	**$ 2.8**
3B	Existing Park Street Alignment Connect to Ford Street and 29th Avenue in Oakland, connect to Lincoln Avenue and Park Street in Alameda	500	0	2,300	fair to poor	good	fair	**$ 2.6**
3C	Fruitvale Ave Bridge Connect to Ford Street and 29th Avenue in Oakland, connect to Lincoln Avenue and Park Street in Alameda	600	0	7,100	poor to fair	food	fair	**$ 4.5**
3D	Existing High Street Alignment Connect to Lincoln Avenue and Park Street in Alameda	500	0	5,500	poor to poor	poor	fair	**$ 4.0**
3E	Everett to Peterson Between existing Derby and Park crossings	500	200	3,700	poor to poor	poor	fair	**$ 3.3**

1 *Crossing Length = Distance from shoreline to shoreline*
2 *Length to Connect to Existing System = Distance from shoreline to closest point in the distribution grid.*
3 *Length of Additional Improvements = Additional distance to connect to a reliable transmission main in the distribution grid.*
4 *Construction Accessibility = adequate space for construction laydown and minimal disturbance to nearby residences, businesses, traffic (etc., on either end of the crossing)*
5 *Addresses Old/Leaking Pipes in Vicinity of Crossing = Replaces aging pipelines and pipes with leak history (an additional benefit)*
6 *Replaces pipe within Liquefaction Zone = Replaces approach pipeline within the Liquefaction Zone*
7 *Preliminary construction cost estimate based on HDD and open trenching unit costs; HDD portal costs based on construction accessibility rating.*

<u>Alternative 1A</u> – On the Oakland side of the crossing, the landing is located at the dead-end of Alice Street, one block south of railroad crossing. The landing area is within a public R/W in the street and located next to an empty lot located on the north side, and multi-story housing located on the south side (Figure 5-4). Although close to condominium residences, this location provides good accessibility for construction of the new crossing. However, recommended pipeline improvements continue farther into downtown Oakland, which is very congested, intersects Highway 880, and crosses a 30-inch PG&E gas line at 2nd Street and an EBMUD SD-1 interceptor in the Embarcadero.

On the Alameda side of the crossing, the landing is located in an existing 10-foot R/W in the congested Barnhill marina parking lot, which contains large silos and warehouses. However, as the alignment continues farther into Alameda, the Southern Pacific Railroad R/W provides easy access for construction and on-going maintenance for 2,100 feet of pipeline; however, this R/W would need to be obtained.

<u>Alternative 1B</u> is similar to Alternative 1A, which follows the existing Alice-Webster Crossing alignment, and has the same alignment on the Oakland side of the crossing, but the Alameda side has a different pipeline alignment after the landing.

For the pipeline section on the Oakland side of the crossing, and the pipeline section from the estuary crossing to the Alameda landing, Alternative 1B follows the same alignment as Alternative 1A. Past the Alameda landing, the alignment makes a sharp turn in the western direction after the Barnhill marina parking lot. The pipeline crosses into an area with current development plans, which may lead to a paving moratorium before the recommended alignment can be installed, creating potential R/W complications. Additionally, construction work while crossing over the transportation tunnels may require a Caltrans R/W permit.

<u>Alternative 1C</u> is located farthest west in the Alice-Webster vicinity, which starts at the pipeline in the Schnitzer Steel Yard in Oakland and crosses the estuary to connect to the pipeline in Main Street in Alameda.

On the Oakland side of the crossing, the landing is located on the Schnitzer Steel property in an existing 10-foot R/W. However, EBMUD recently abandoned the pipeline on this property due to poor accessibility, and therefore, the crossing landing in Oakland would be impractical. Additionally, the estuary crossing of this alignment is located just before the Port of Oakland turning basin, and major boat traffic would cross over this alignment, which could potentially damage the crossing when boats are anchored or when the port of Oakland is dredged.

On the Alameda side of the crossing, the landing is located in a small park and parking lot, just west of the Alameda Ferry Terminal. This landing has good construction accessibility, but the connection to the existing distribution network would require a long distance of pipeline improvements. In addition, both sides of the crossing are known to have a high risk of contaminated soils, which would create greater installation hazards and costs.

Alternative 1D is the farthest east alignment alternative in the Alice-Webster vicinity, which starts at the pipeline in Marina Village Parkway in Alameda and crosses the estuary to connect to the pipeline in Fallon Street in Oakland.

The landing on the Oakland side of the crossing is located in 40-foot R/W 676 west of Estuary Park in Fallon Street south of Embarcadero West, providing good construction accessibility for a new crossing (Figure 5-5). Additionally, the pipeline improvements have the potential to provide improved LOS to the proposed mixed-use Brooklyn Basin development, which will be located from Oak Street to 9th Street. Pipeline construction across the Embarcadero will also cross under an EBMUD SD-1 interceptor. On the Alameda side of the crossing, the landing is located in an existing 40-foot R/W in a small office parking lot, beyond a boat marina and a small public park space, connecting into Tynan Avenue. There is good accessibility for construction at this location, but there would be temporary construction impacts to the office parking lot.

This alignment is located well beyond the shipping channel and away from the existing fragile Alice-Webster Crossing, so that construction of the new crossing would not impact the existing crossing. Also, the existing crossing would eventually be removed from service.

Alternative 1E is located just west of the existing Alice-Webster crossing, which starts at the pipeline in Washington Street in Jack London Square in Oakland and crosses the estuary to connect to the pipeline in Mitchell Avenue in Alameda.

On the Oakland side of the crossing, the landing is located in a 15-foot R/W in a hardscape area between Waterfront Hotel and other commercial properties, which could potentially limit future maintenance access due to its location within Jack London Square (Figure 5-6). However, this area provides adequate construction accessibility. As the alignment continues farther into downtown Oakland, the new pipeline must immediately cross the Embarcadero, a PG&E gas transmission line, Southern Pacific Railroad, Kinder Morgan, BART, and Highway 880 to tie into the transmission main in Alice Street.

On the Alameda side of the crossing, the landing is located in a 25-foot R/W for EBMUD's wastewater interceptor, which is within open space slated for redevelopment (currently serving as a construction laydown space). This side of the crossing provides good construction accessibility, but a long distance of additional pipeline improvements is required to provide good connection to a reliable transmission main in Lincoln Avenue on the Alameda side.

Alameda-Bay Farm Vicinity Alignment Alternatives

Presently, there are two pipeline crossings located in the Alameda-Bay Farm (A-BF) vicinity (A-BF #1 and A-BF #2). Currently, A-BF #1 is out of service due to a leak near the shoreline valve pit that is too deep to repair. A-BF #2 is the newest crossing and traverses the estuary at a shallow depth, passing through highly vulnerable soil layers. The A-BF #2 is currently in service and is crucial to maintaining Bay Farm Island levels of service. Two proposed crossing alignments were examined as alternatives to replace the existing AB-F #2 crossing (Figure 5-7).

Alternative 2A is located along the existing alignment of the A-BF #2 Crossing. On the Alameda side of the crossing, the landing or connection is located in Bridge View Isle, a dead-end street and public R/W, with an adjacent gravel and grass area (Figure 5-8). Although Bridge View Isle is a residential street, the landing would be located on the east end of the street, not in front of any homes, which provides good construction access. On the Bay-Farm side of the crossing, the landing is located in Veteran's Court, a dead-end street, adjacent to tennis courts. This location is also in a public R/W providing good construction access with limited impact to the neighboring community. Alternative 2A is the best alignment for the replacement crossing to Bay Farm Island based on the study criteria, and was analyzed further in the CFS.

Alternative 2B is located west of the existing Alameda-Bay Farm No. 2 alignment, starting at the pipeline in Shoreline Drive in Alameda and crosses the estuary to connect to the pipeline in Seaview Parkway in Bay Farm Island.

On the Alameda side of the crossing, the connection or landing is located at the corner of Shoreline Drive and Broadway Street, adjacent to the border of Elsie Roemer Bird Sanctuary, an East Bay Regional Park (Figure 5-9). The construction laydown and assembly would be on Broadway, which is a public street, but close to the Regional Park. This location could also affect the high volume of traffic on Broadway. On the Bay Farm side of the crossing, the landing is located in a small waterfront park off of Seaview Parkway, with residential condominiums across the street.

Northeast Corridor Vicinity Alignment Alternatives

Presently, there are four pipeline crossings located in the Northeast Corridor vicinity — Blanding, Park, Derby, and High Street Crossings. The Derby and High Street Crossings are out of service due to unrepairable leaks. Installed in 1987, the 24-inch Blanding Street Crossing is the youngest and most reliable crossing in this area and is currently in service. The 16-inch Park Street Crossing, installed in 1918, is currently in service.

Five alternative crossing alignments were identified for the Northeast Corridor area and all five need extensions to connect to the new 16-inch Lincoln Avenue transmission pipeline in Alameda in order to ensure a reliable connection to the distribution system (Figure 5-10).

Alternative 3A is located along the existing alignment of Derby Crossing, which is out of service. The crossing starts at the pipeline in Derby Avenue in Oakland and crosses the estuary to connect to a pipeline in Blanding Avenue in Alameda.

On the Oakland side of the crossing, the landing is located at the dead end of Derby Avenue, providing good construction accessibility within the public R/W (Figure 5-11). Since this dead end experiences little traffic, this landing would minimize traffic impacts. On the Alameda side of the crossing, the landing is located in an existing 20-foot R/W, in the shopping center parking lot, in front of Nob Hill Foods. Although there is good accessibility for construction, there would likely be impacts on local traffic and the businesses during construction.

Alternative 3A has good potential for maintenance access and good connectivity to the distribution network on both sides of the crossing. There is good public R/W access on the Alameda side to Lincoln Avenue via Tilden Way for additional pipeline improvements. The landings have good accessibility on both sides of the crossing and an existing easement in the alignment.

<u>Alternative 3B</u> is located along the existing alignment of Park Street Crossing, which is currently in service. The crossing starts at the pipeline in 29th Avenue in Oakland and crosses the estuary to connect to a pipeline in Park Street in Alameda.

On the Oakland side of the crossing, the landing is located in a narrow, dead end of 29th Avenue, adjacent to a large parking lot on the south side of the Park Street Bridge (Figure 5-12). Although somewhat narrow, this area provides adequate space for construction laydown, and causes little disturbance to surrounding areas. On the Alameda side of the crossing, the landing is located in a narrow (approximately 30-foot wide) dead end of Park Street, which serves as a congested parking lot, between the bridge overpass and private residences. Construction accessibility in this location is considered poor. However, there is an existing R/W in the last 30-foot section of the street.

<u>Alternative 3C</u> is located parallel to the Fruitvale Bridge, between the existing Park Street Crossing and High Street Crossing. This alternative starts in the pipeline at the corner of Fruitvale Avenue and Alameda Avenue in Oakland and crosses the estuary to connect to the pipeline at the corner of Marina Drive and Versailles Avenue in Alameda.

On the Oakland side of the crossing, the landing would be located in a small grassy park outside Owen-Illinois glass factory on Fruitvale Avenue (Figure 5-13). This park provides fair construction accessibility, although construction could impact traffic at the nearby intersection and also interfere with truck access to the adjacent glass factory. On the Alameda side of the crossing, the landing is located in a narrow dead-end section of Versailles Avenue, between the Fruitvale Bridge overpass and a private residence. This location as described provides for poor construction and maintenance access.

<u>Alternative 3D</u> is located along the existing High Street Crossing alignment, starting in the pipeline at the corner of High Street and Tidewater Street in Oakland and crossing the estuary to connect to the pipeline at the corner of Marina Drive and High Street in Alameda.

On the Oakland side of the crossing, the landing located in High Street in a narrow roadway (Figure 5-14). On the Alameda side of the crossing, the landing is located in High Street as well, in a busy intersection with high traffic.

Alternative 3E is located between the existing Park Street Crossing and Derby Crossing, starting at the pipeline at the corner of Glascock Street and Peterson Street in Oakland and crossing the estuary to the pipeline in Everett Street in Alameda.

On the Oakland side of the crossing, the landing is located within narrow private streets, surrounded by multi-story condominium units (Figure 5-15). However, there is an existing 20-foot R/W through the multi-unit housing. On the Alameda side of the crossing, the landing is located in a private steel fabrication yard, which would require an easement. Overall, both of these landing locations are on private property, in congested areas, and do not provide much room for construction access.

Construction Methods

The CFS (Appendix A) identified multiple design/construction approaches for each of the four preferred alignments with the goal of maximizing survivability and minimizing repair-related water service outages attributable to a major seismic event.

The CFS identified microtunneling and HDD as the two most feasible trenchless construction methods for the crossings. This determination was made based on crossing constraints, groundwater levels, and ground conditions at each of the selected alignments. A microtunnel installation was recommended at Alignment 1A while an HDD installation was recommended at Alignment 1D. Alignments 2A and 3A could accommodate either a microtunnel or HDD installation. To avoid unstable ground conditions, such as hydraulic fills and soft soils overlying bedrock, deeper underwater pipeline crossings are needed. Both microtunneling and HDD concepts are developed to be in the deeper, more stable ground conditions at each crossing.

Contract construction costs in 2014 dollars for the tunnel sections are summarized in Table 5-2. Total project costs including planning, design and construction for the selected projects are detailed in Chapter 6.

In addition to the recommended methods of installation, the CFS provided estimated probabilities of failure for each of the design iterations as well as estimates to repair if the crossings were to fail. These estimates are the best professional judgment of the experienced consulting team. These values were used as inputs to the Info-Gap model developed and discussed in the next section of this chapter. A risk register, also developed as part of the study is included in Volume 2 (Appendix G) of the CFS. Risks were identified and quantified for each of the four proposed pipeline crossing alignments.

TABLE 5-2
Estimated Crossing Construction Costs

Alternative	Length *(feet)*	Construction Cost *($M)*
1A - Microtunnel	1,216	$7.8
1D - HDD	1,780	$4.2
2A – Microtunnel	903	$7.0
2A – HDD	1,250	$3.3
3A – Microtunnel (long)	1,150	$6.7
3A – Microtunnel (short)	950	$6.1
3A – HDD	1,300	$3.4

<u>Microtunneling</u> **-** The microtunneling (MT) approach consists of a jacking shaft from which the microtunneling boring machine (MTBM) and casing are advanced to a receiving shaft for retrieval of the MTBM. The jacking and receiving shafts are constructed using secant piles and have a diameter of 28- and 18-feet, respectively. The shaft depths were selected to place the underwater tunnels below the fill and YBM and into the deep stable soils not prone to liquefaction. The microtunnel crossing depths vary from 60-80 feet, which also places them deep enough to avoid any future dredging of the channel. The microtunnel crossing is a 48-inch steel casing with a 24-inch inside diameter HDPE carrier pipe (30- inch outer diameter). The carrier pipe is grouted inside the steel casing.

Three concepts were developed for bringing the water main riser pipe through the shaft to the surface connection:

1. Free standing and mounted to the shaft wall with struts or tie-downs and supported at the bottom with a concrete saddle.
2. Encased and protected by concrete, with the remaining portion of the shaft left open.
3. Encased in concrete with the remaining portion of the shaft backfilled with compacted structural backfill.

Although fully backfilling the shafts provides the most protection against failure in the event of an earthquake, leaving all or a portion of the shaft open for access allows for much easier access for maintenance of microtunnel pipeline in the future. The CFS provided estimates on the probability of failures for each of these design iterations for use in the Info-Gap model, as described in the next section of this chapter.

<u>Horizontal Directional Drilling</u> **–** HDD is a three stage construction method which originates from the surface. A U-shaped pilot hole is drilled first, and then reamed and enlarged to the required size for pipe pullback, typically approximately 30 percent larger in outside diameter than the carrier pipe to be installed. The third stage is pulling of the carrier pipe into the hole. A 24-inch inside diameter HDPE pipeline, with a 30-inch outside diameter will be installed. The pipeline is typically assembled as one long pipe string and pressure-tested before pullback into the reamed hole. Construction

layout space controls which side of the crossing is the pipeline exit point and which side is the pipeline entry point.

The depth of the crossing is dictated by the clearance requirements of the water body (the Alameda Channel in this case), including potential for future dredging and to locate the crossing within a particular stable soil horizon. For the alignments studied, the entry and exit angles will be 10-15 degrees. Oversized conductor conduits are required at each end of the HDD installation to control fluid pressure to prevent hydraulic fracturing to the surface. The conductor casing/conduit is typically installed in the shallow reaches near the entry and exist pits using pipe ramming and can be installed up to 200 feet. Using an entry angle of 15 degrees for the casing, a 200-foot long casing would reach a depth of 50 feet, which is the approximate boundary between liquefiable and non-liquefiable soils. For this reason, the entry and exit pits are located at least 200 feet onshore to allow installation of 200-foot conductor casings and any associated ground improvement work before the pipeline is under the estuary.

In order to develop an HDD crossing that is housed in non-liquefiable soils from surface to surface, concepts are developed in the CFS which incorporate ground improvements, such as jet-grouted columns or cement soil-mixed panels, to support the conductor casings in the liquefiable soils. The recommended ground improvement includes supporting a 200-foot conductor casing with 8-foot diameter jet-grouted columns. The distance between jet-grouted columns is typically 20-inch center-on-center but this spacing will be refined for site-specific soil profiles.

Info-Gap Model Summary

Info-Gap decision methodology uses models of uncertainty rather than probability owing to the lack of information or data about the problem[3].

For this project objective, Info-Gap models were developed in order to investigate robustness of predictions about the physical failure rate of the proposed new crossings to Alameda and the time to repair those crossings in the event one does fail. The two Info-Gap models address these predictions: Survivability and Reparability. What follows is a brief summary of the Info-Gap models and the conclusions drawn based on the results. A full discussion of the Info-Gap models and results is included as Appendix B.

<u>Survivability</u> - The Survivability model was developed to investigate the question: how wrong can the estimates about probability of failure be and still provide the minimum critical survivability? The CFS estimates there is essentially a near zero chance of a pipeline failure in the submarine portion of a new tunnel. The goal of the Info-Gap model analysis was to assess the robustness of these survivability predictions. Robustness is calculated as the percentage error tolerance on each estimated probability.

[3] Info-Gap Decision Theory, Decision Under Severe Uncertainty; Yokov Ben-Haim

Probabilities of failure of the different portions of the tunnels are summarized in Table 5-3. The system model is simply the sum of the probabilities of failure of each portion of the pipeline minus the joint probabilities. Since joint probabilities were not provided, they were assumed to be equal to the minimum known individual probability. A robustness function was derived based on the system model described above and shown in detail in Appendix B.

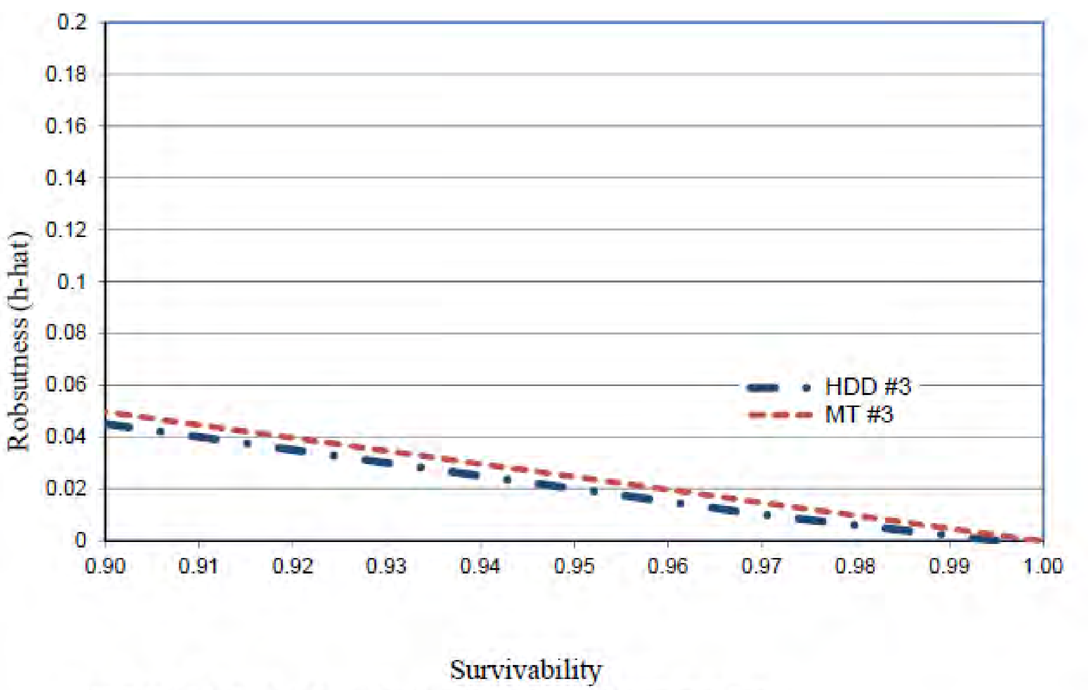
TABLE 5-3
Survivability Info-Gap Model Inputs

Horizontal Directional Drilling Alternatives	near-surface	submarine portion	end of conductor casing
Directly drilled (no conductor casing)	1-5%	0-1%	n/a
Conductor casing (100' long)	n/a	0-1%	<5%
Jet-grouting with conductor casing	n/a	0-1%	0-1%

Microtunneling Alternatives	submarine portion	riser
Free-standing riser	0–0.1%	<1%
Riser encased in concrete, open shaft, bolted cover	0–0.1%	0–0.1%
Riser encased in concrete, shaft backfilled	0–0.1%	0–0.05%

Graph 5-1 shows a comparison of robustness curves for two designs: one HDD alternative and one MT alternative. The horizontal axis represents the critical survivability, while the vertical axis is the corresponding robustness.

GRAPH 5-1
Survivability Model Results



HDD #3: 20-foot conductor casing left in place plus jet-grouting
MT #3: Riser encased in concrete, shaft backfilled

The MT alternative has higher robustness at every point along the curve. Since the probability of failures of the MT alternative is estimated as extremely low, the higher robustness leads to extremely large tolerance to uncertainty relative to those estimated probabilities of failures. The tolerance to uncertainty on the HDD alternative, on the other hand, is not only smaller but much smaller in relative comparison to the estimated probabilities.

For example, at 90 percent critical survivability, the HDD alternative has a robustness of >0.04. The probability of failure of the submarine portion and end of conductor casing for this alternative is given as 0-1 percent (Table 5-3). According to the uncertainty model, the actual probability is the estimated probability plus robustness. So the actual probability of failure that can be tolerated in this case is 4 to 5 percent, or up to five times the estimated probability. On the other hand, the microtunneling alternative has a robustness of approximately 0.05 at 90 percent survivability. The probability of failure of the submarine portion of this alternative was given as 0-0.1 percent, so the actual probability of failure that can be tolerated in this case is 5 percent or up to 50 times the estimated probability.

Despite the higher robustness of the MT alternative, the project team determined the robustness of the survivability prediction for one HDD crossing alternative is “good enough” from an Info-Gap perspective, given the redundancy provided by other in-service crossings.

<u>Reparability</u> - The Reparability model was developed to investigate the question: how wrong can the repair estimates be and still be within the maximum allowable downtime (i.e., the critical time?). Obtaining estimated times to repair proved to be both challenging and conflicting. Time to repair estimates used in the Info-Gap model for both tunneling methods are summarized in Table 5-4 with full descriptions of the estimates in Appendix B. The system model is simply the sum of all the repair times.

TABLE 5-4
Reparability Info-Gap Model Inputs

τ_n = time to perform n step

	Microtunneling (Shaft Only) Alilgnments: 1A, 2A, 3A	τ_1 = isolate approach*	τ_2 = pump out	τ_3 = ventilate	τ_4 = enter & stabilize	τ_5 = excavate & rebuild liner	τ_6 = rebuild pipe
1	Free-standing riser	1 hr	24 hr	1 hr	24 hr	n/a	48 hr
2	Riser encased in concrete, open shaft, bolted cover	1 hr	24 hr	1 hr	n/a	n/a	n/a
3	Riser encased in concrete, shaft backfilled	n/a	n/a	n/a	n/a	1 wk	n/a
4	Increased casing diameter for second carrier pipe	f(1,2,3)	f(1,2,3)	f(1,2,3)	f(1,2,3)	n/a	f(1,2,3)

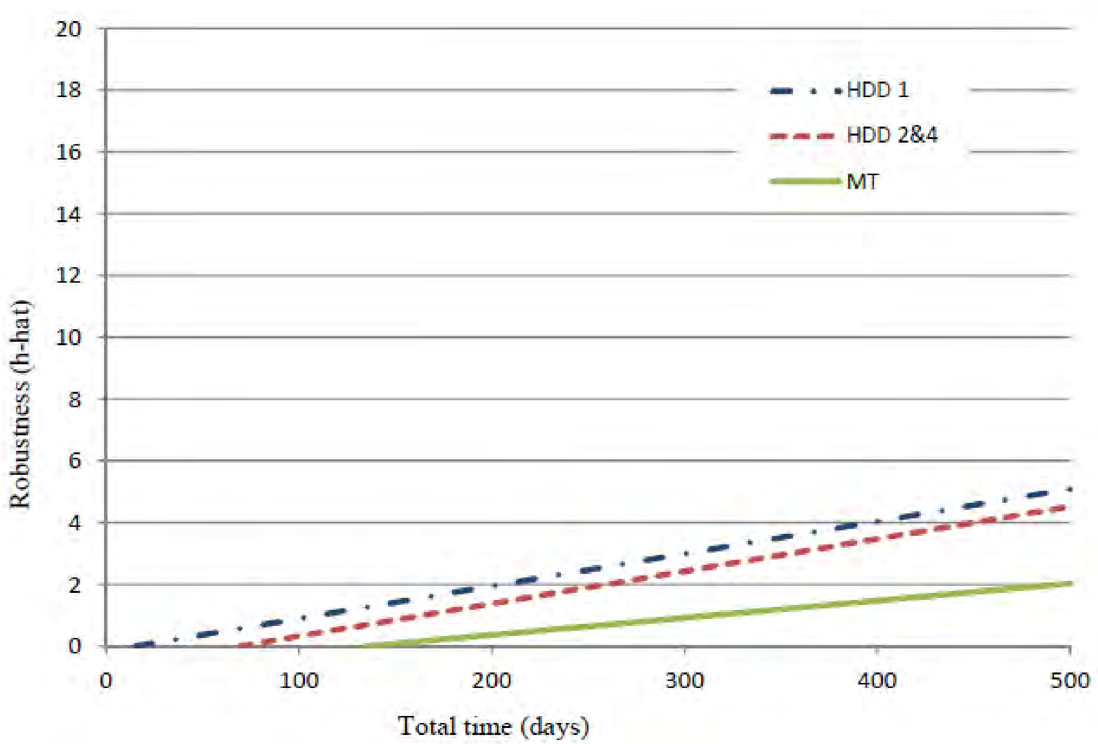
* steps identified on p 85 of Jacobs' draft report

	Directionally Drilling (HDD) Alignments: 1D, 2A	τ_1 = isolate approach*	τ_2 = pump out	τ_3 = CCTV	τ_4 = insert new pipe	τ_5 = ??	τ_6 = ??
1	Directly drilled	1 hr	n/a	24 hr	1 wk		
2	Conductor casing (50-100' long)	1 hr	n/a	24 hr	1 wk		
3	Steel casing to house carrier pipe, entire length	1 hr	n/a	24 hr	1 wk		
4	Jet-grouting with conductor casing	1 hr	n/a	24 hr	1 wk		

A robustness function was derived based on the system model described above and is shown in detail in Appendix B. What follows here is a brief discussion of the results.

Graph 5-2 shows a comparison of three robustness curves. HDD #1 represents a directly drilled crossing with no conductor casing. This design would be *expected* to fail near the surface in the liquefiable soils and therefore has a lower *expected* estimate. HDD 2 and 3 represent alternatives with a conductor casing which are expected to require a submarine repair. The microtunnel alternative is based on a general estimate for all microtunnel designs. The horizontal axis represents the critical time to repair, while the vertical axis is the corresponding robustness.

GRAPH 5-2
Reparability Model Results



As shown in Graph 5-2, the HDD alternatives have higher robustness at every point on the curve. But no amount of reparability was deemed “good enough” to be counted on for restoring service to Alameda and Bay Farm Islands after a seismic event. Therefore the project team chose to provide redundancy (another model of robustness) in the number and location of crossings, as stated in the Survivability Model section.

Environmental Factors Potentially Affected

Construction of the planned underwater crossings and related pipeline improvements could potentially result in short term construction impacts on biological resources, greenhouse gas emissions, transportation/traffic, hazards and hazardous materials, utilities, air quality, geology/soils, hydrology/water quality, noise and possibly recreation.

Chapter 4 of the CFS (Appendix A) describes feasible trenchless construction methods and materials that would be used for the preferred crossing locations discussed earlier in this chapter. While horizontal directional drilling is the preferred approach for each of the three crossing alignments, microtunneling was also evaluated. Information on the street piping alignments related to the tunnel crossings alternatives is shown in Appendix C.

Environmental documentation for tunnel crossings under the Oakland Estuary will need to address potential impacts to marine structures (such as dock pilings), marinas and house boats, and buried utilities as well as those to water quality and biological resources. Hazardous/contaminated soil conditions are also concerns in some areas.

Short-term construction noise and traffic impacts are also of concern given the mix of nearby residential, commercial and recreational land uses near the shoreline. Depending on the specific project location, traffic to and from local shopping areas and business could also be affected, potentially involving detours. Traffic will also be affected when the HDPE pipe is fused and placed along the street or median to accommodate pullback during the evening hours.

Agency involvement is anticipated to include the Bay Conservation and Development Commission (BCDC), Regional Water Quality Control Board (RWQCB), Department of Fish and Game, Caltrans, California Air Recourses Board (CARB), Department of Boating and Waterways, State Coastal Commission, Department of Parks and Recreation, City of Alameda, City of Oakland/Parks and Recreation, Port of Oakland, and Southern Pacific Railroad.

CHAPTER 6 - PREFERRED PROJECTS

All new crossings will be HDPE pipe, installed using horizontal directional drilling methods. Crossings that traverse liquefiable soils will utilize soil improvements such as the jet-grouting recommended by Jacobs Associates in drawing E-5 of the CFS. This section contains representative drawings of the projects selected for construction.

Alternatives 1A and 1D located in the vicinity of the existing Alice-Webster crossing were selected for further investigation in the CFS prepared by Jacobs Associates based on feasibility and cost-effectiveness. Alternative 1A is estimated to be less expensive with good construction access on the Oakland side and fair construction access on the Alameda side. Alternative 1A also requires the shortest length of additional pipeline improvements required to connect to a reliable transmission main. Alternative 1D requires a long underwater tunnel and several thousand feet of connecting pipeline improvements, but also replaces more than 2,000 feet of old cast iron pipe. Coupled with good construction access at both landings, Alternative 1D is recommended as the preferred alignment to replace the existing Alice-Webster crossing.

Alignment 1D

Alignment 1D is the recommended new crossing alignment in the Alice-Webster crossing area and detailed planning, design, and construction should proceed. The HDD alignment crosses the Alameda Channel from a gated parking lot at Estuary Park in Oakland to a private parking lot between 1080 and 1100 Marina Village on Alameda Island and is approximately 1,780 feet in length (Figure 2-2). The laydown area for the pipe fusion is along the east side of the median strip of Marina Village Parkway on the Alameda side (Figure 6-1).

Pipeline improvements in the streets will extend from the crossing to the new 16-inch pipeline in Lincoln Avenue at 8th Street on the Alameda side and to the 30-inch transmission line at 9th and Alice Streets on the Oakland side. The proposed Oakland-side pipeline extension will replace approximately 2,300 feet of existing 24CM46 cast iron pipe in Alice Street (which exhibits a high consequence of failure based on the leak history) and about 300 feet of 24SMM53 pipe encased under the Hwy 880 embankment**.** The recommended alignment is shown in Figure 2-1 and a detailed alignment analysis is contained in Appendix C.

Alignment 2A

Upon sign of failure of the existing Alameda Bay Farm Island #2 crossing, a new crossing at this same location is recommended. Alignment 2A crosses the Alameda Channel from Veterans Court on North Bay Farm Island to Towata Park, just beyond Bridgeview Isle, on the Alameda side and is approximately 1,250-feet in length (Figure 6-2). The laydown area for the pipe fusion would be along Veterans Court and then turning south along Island Drive on Bay Farm Island (Figure 6-3). For an assessment of recommended in-ground pipeline improvements associated with this alignment, see Appendix C.

Alignment 3A

Upon sign of failure of the existing Blanding (Oak) Street crossing, a new crossing in Alignment 3A (the old Derby Street crossing) is recommended. Alignment 3A crosses the Alameda Channel from Derby Avenue in Oakland to Broadway (just beyond the parking lot of the Nob Hill Market Shopping Center) on the Alameda side (Figure 6-4). The laydown area for the pipe fusion would be along the center stripe of Broadway on the south side of Tilden Way on the Alameda side (Figure 6-5). For an assessment of recommended in-ground pipeline improvements associated with this alignment see Appendix C.

Cost Estimates

Cost estimates for the three selected projects are included in Table 6-1. The HDD construction costs are based on estimates provided in the CFS (Appendix A) and include the cost to construct the specified length of 24-inch HDD pipeline as well as the recommended jet-grouting soil improvements. In-ground pipe installation cost estimates are based on the recommended street alignments contained in Appendix C. CEQA documentation costs are based on the level of effort estimated for a Mitigated Negative Declaration. Design, Construction Management, and contingency estimates are listed in the table.

TABLE 6-1
Total Project Cost Estimates ($M)[1]

	Alternative 1D	Alternative 2A	Alternative 3A
HDD Construction cost[2]	$ 4.2 1,780 feet	$ 3.3 (1,250 feet)	$ 3.4 (1,300 feet)
In-ground pipe installation[3]	$ 4.3 10,000 feet	$4.7 (10,100 feet)	$3.1 (7,200 feet)
Construction subtotal	$ 8.5	$ 8.0	$ 6.5
CEQA Documentation[4]	0.5	0.5	0.5
Design (15 percent)	1.3	1.2	1.0
Construction Management (15 percent)	1.3	1.2	1.0
Project subtotal	$ 11.6	$ 10.9	$ 8.9
Contingency (20 percent)	2.3	2.2	1.8
Total Project Cost	**$ 13.9**	**$ 13.1**	**$ 10.7**

1. *March 2014 dollars*
2. *HDD estimates provided in CFS, Appendix A*
3. *Street alignment estimates provided in Appendix C*
4. *Mitigated Negative Declaration*

CHAPTER 7 – ACKNOWLEDGEMENTS

This master plan is the culmination of contributions made by the project team who provided experience and insights that guided the formation and recommendations.

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In addition, this master plan included information based on the combined experience, knowledge and dedicated efforts of the following contributors.

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CHAPTER 8 – REFERENCES

DRS Marine Inc., June 22, 2009. 20” Water Pipeline Estuary Crossing Derby, Oakland – Broadway, Alameda, Informal Progress Report Data on Excavation Operations to Locate Damage Pipe Section (DOX #1679052).

East Bay Municipal Utility District, February 2009. 2040 Demand Study (DOX #1640446).

East Bay Municipal Utility District, 2009. Distribution System Hydraulics - Outage of the Derby Street Crossing Oakland/Alameda Technical Memorandums (3) to Water Treatment and Distribution from Water Distribution Planning (DOX #s1712414, 1670354, 1667251).

Jacobs Associates by G&E Engineering Systems Inc., September 2, 2013. Alameda Bay Farm Island Pipes Report, (DOX #2212444).

Jacobs Associates, March 2014. Alameda-North Bay Farm Island Crossings Master Plan, Underwater Pipeline Crossings Feasibility Study, (Volume 1 - DOX #2191923 and Volume 2 - DOX #2191925).

Yokov Ben-Haim, 2006 and 2001. Info-Gap Decision Theory, Decision Under Severe Uncertainty (ISBN-13: 978-0-12-373552-2 and ISBN-10: 0-12-373552-1).

Figures

Alameda North Bay Farm Island Crossings Master Plan

Figure 1-1

Alameda Island Overview

Figure 2-1

Selected Project Overview

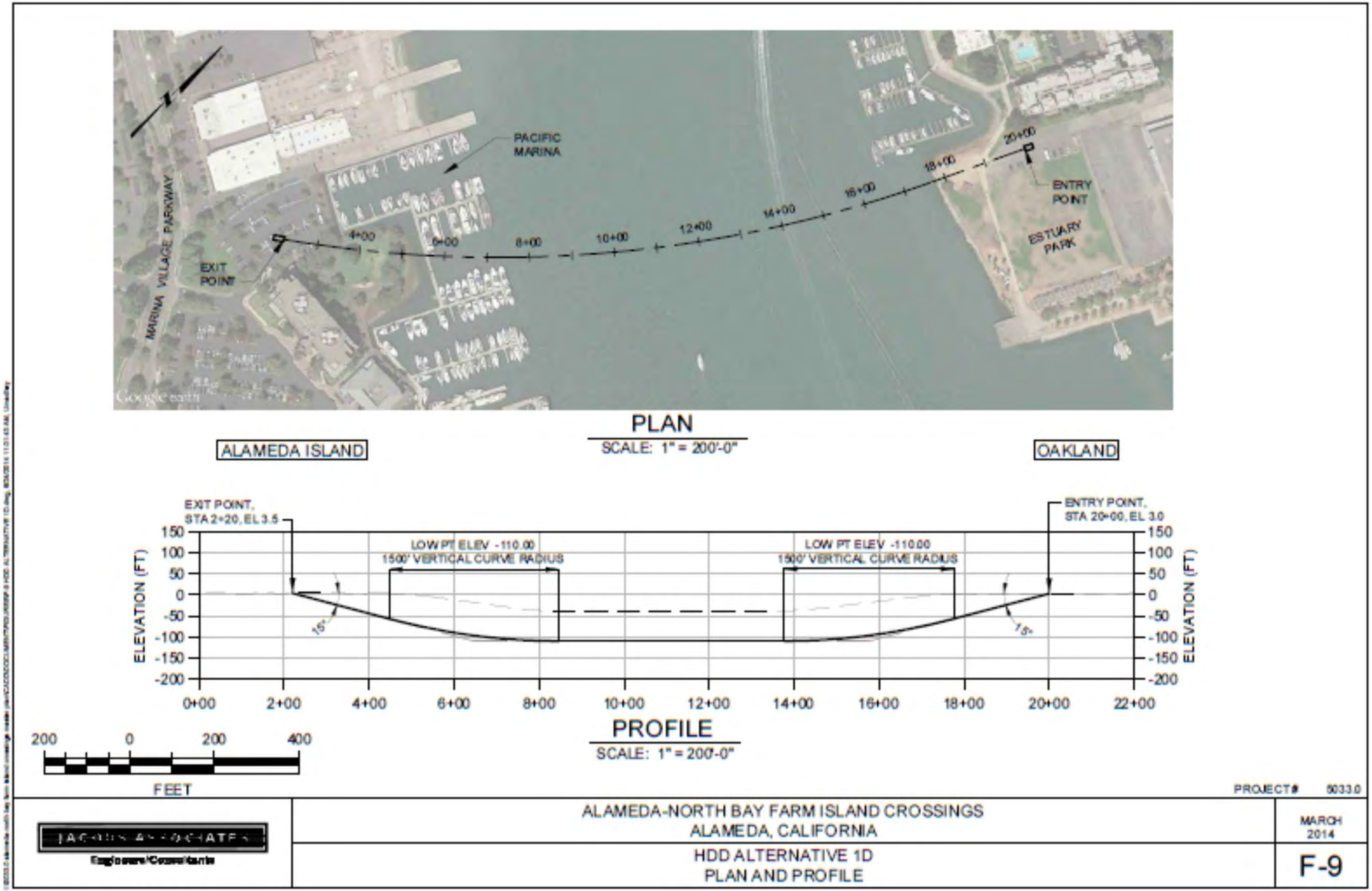
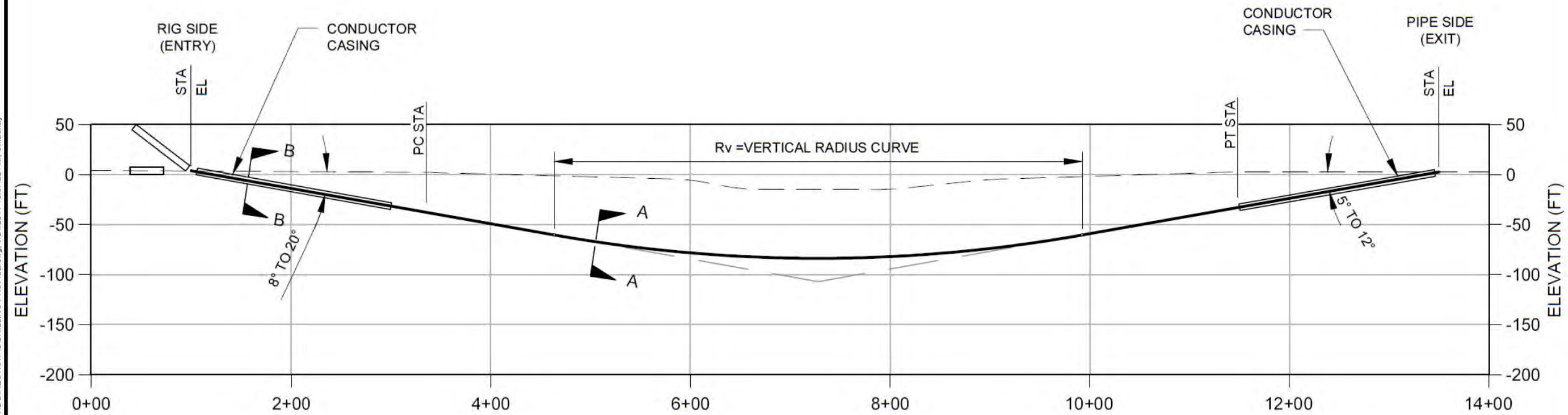


Figure 2-2

HDD Alternative 1D Plan and Profile

NOTES:

1. FOR STEEL Rv = PIPE OD (IN INCHES) x 100'
2. FOR HDPE Rv = PIPE OD (IN INCHES) x 50'



PROFILE
SCALE: 1" = 100'-0"

PROJECT # 5033.0

JACOBS ASSOCIATES
Engineers/Consultants

ALAMEDA-NORTH BAY FARM ISLAND CROSSINGS
ALAMEDA, CALIFORNIA

TYPICAL HORIZONTAL DIRECTIONAL DRILLING PROFILE

MARCH 2014

E-1

Figure 2-3
Typical Horizontal Directional Drilling Profile

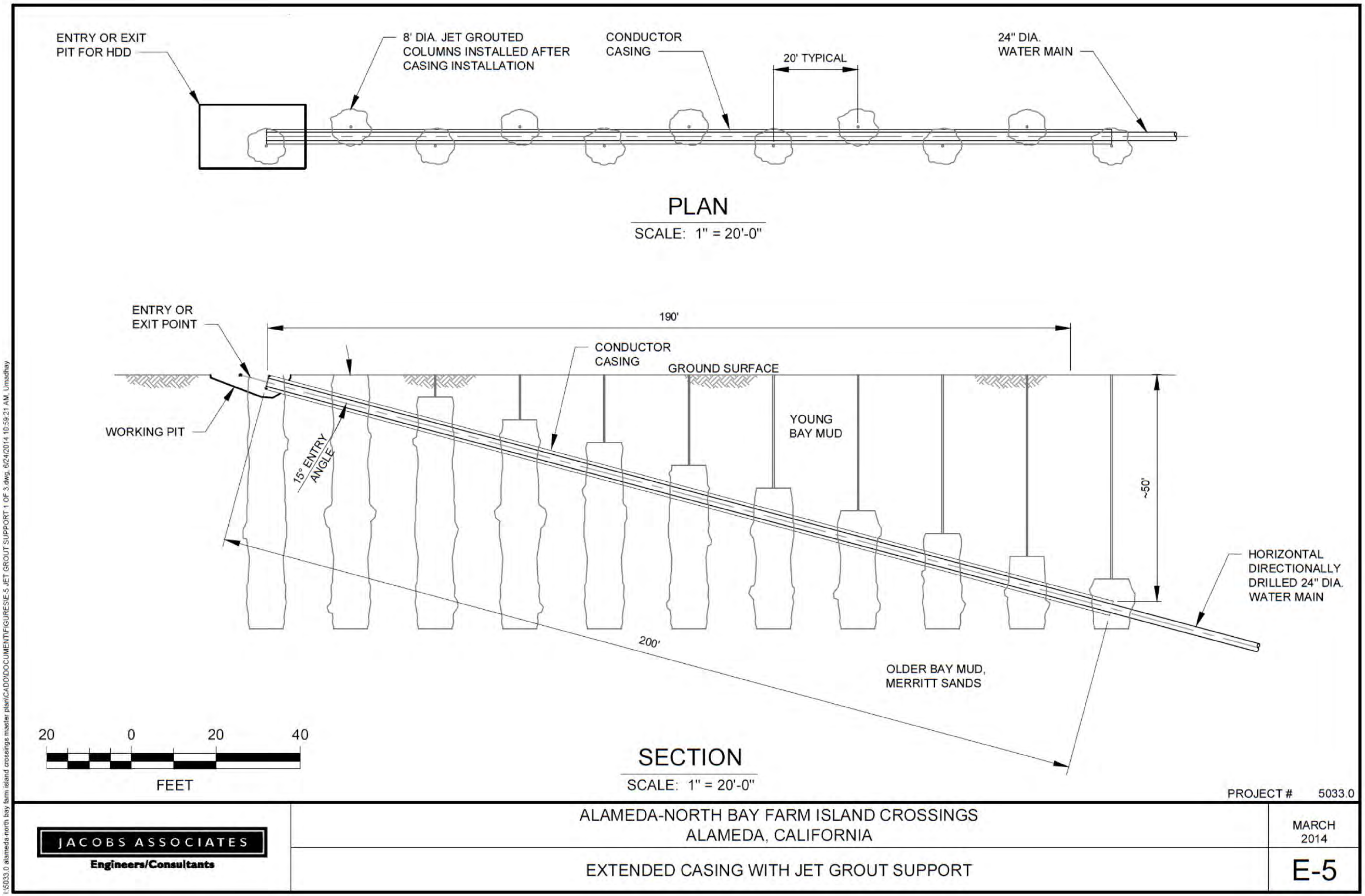


Figure 2-4

Extended Casing with Jet Grout Support

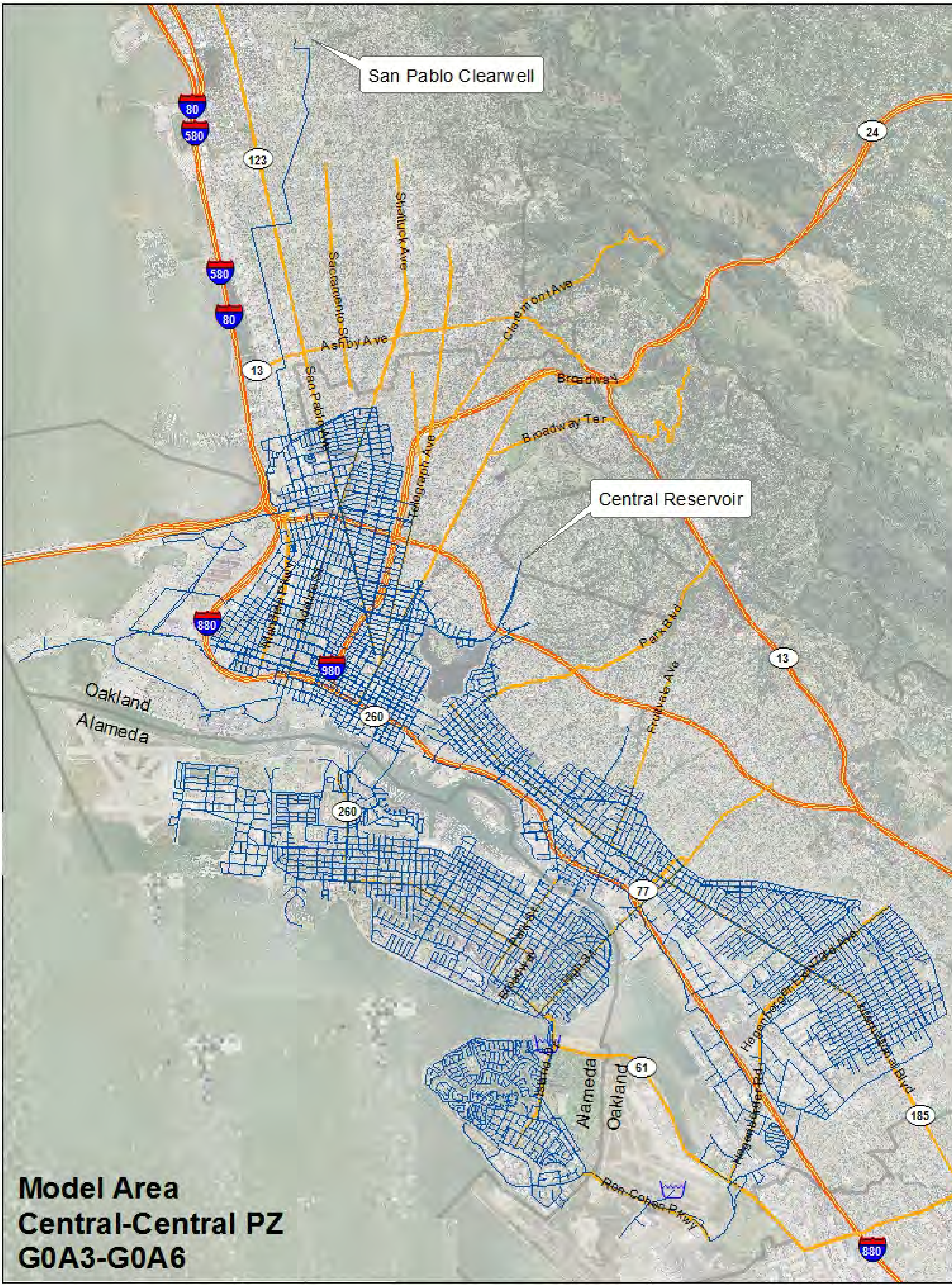


Figure 4-1

Central-Central Pressure Zone All-Pipe Mode

Figure 4-2
Monitoring Node Locations

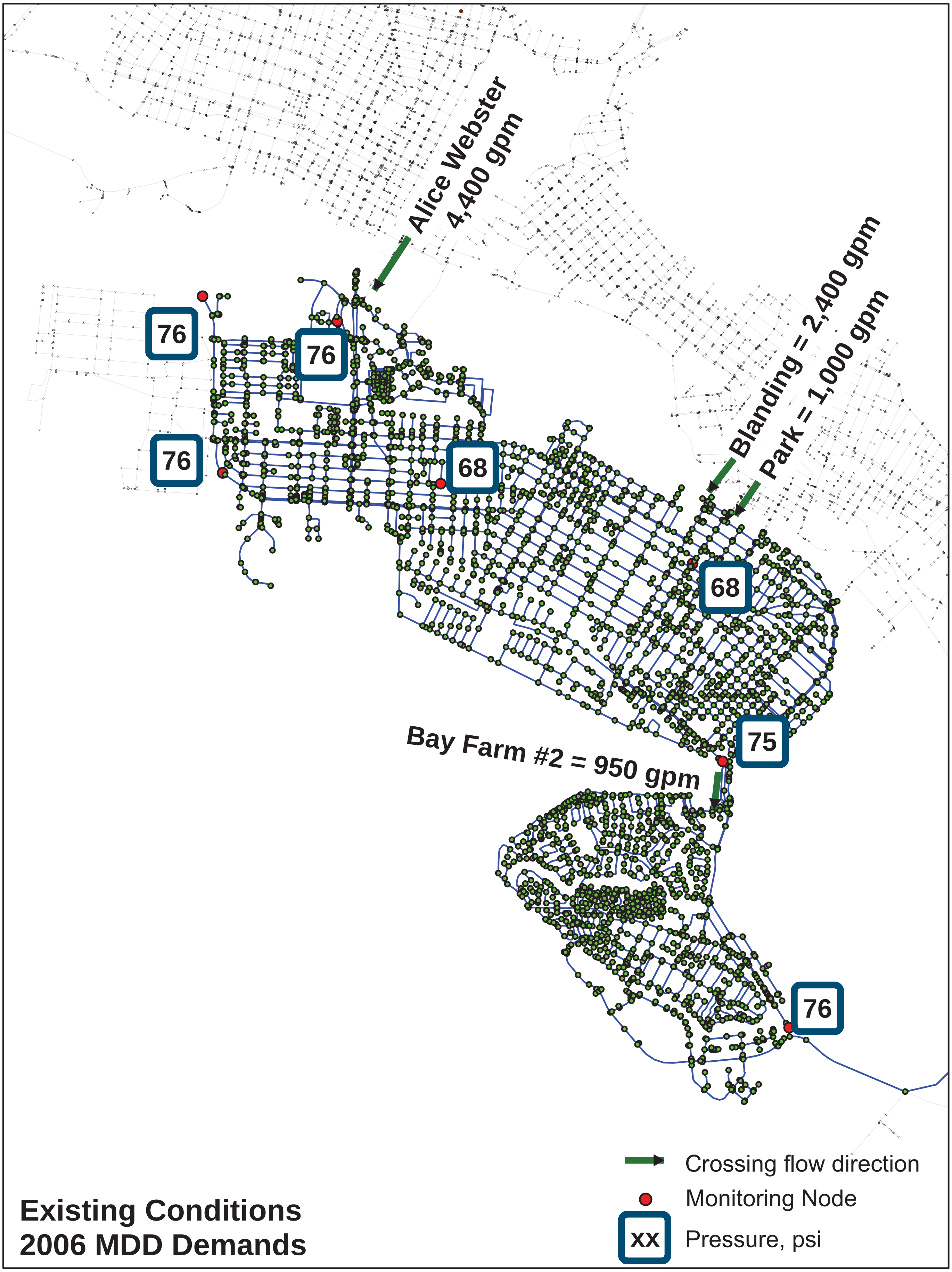


Figure 4-3
Existing Crossing Flows and Pressures

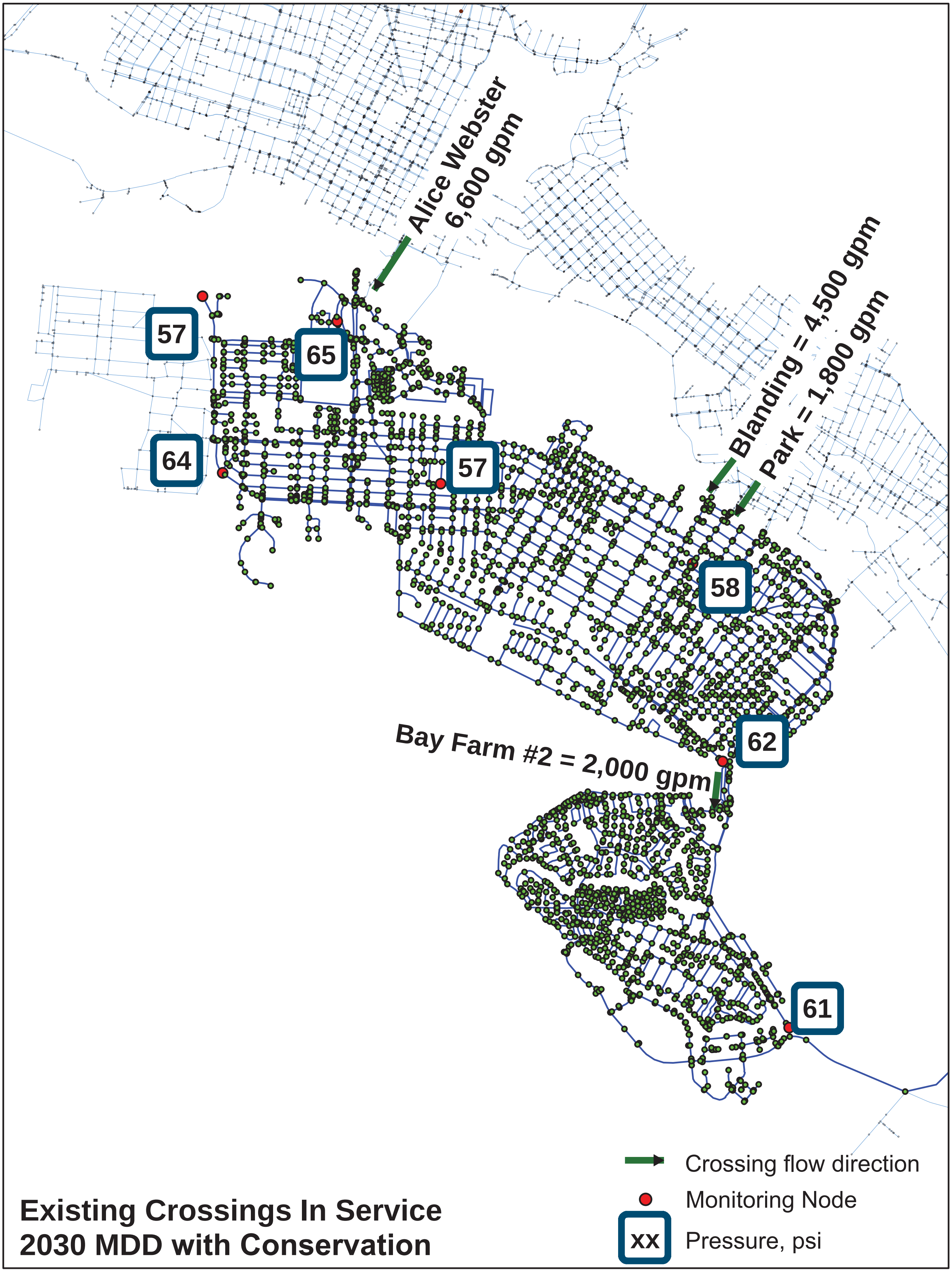


Figure 4-4

Existing Crossing in Service, 2030 Crossing Flows and Pressures (with Conservation)

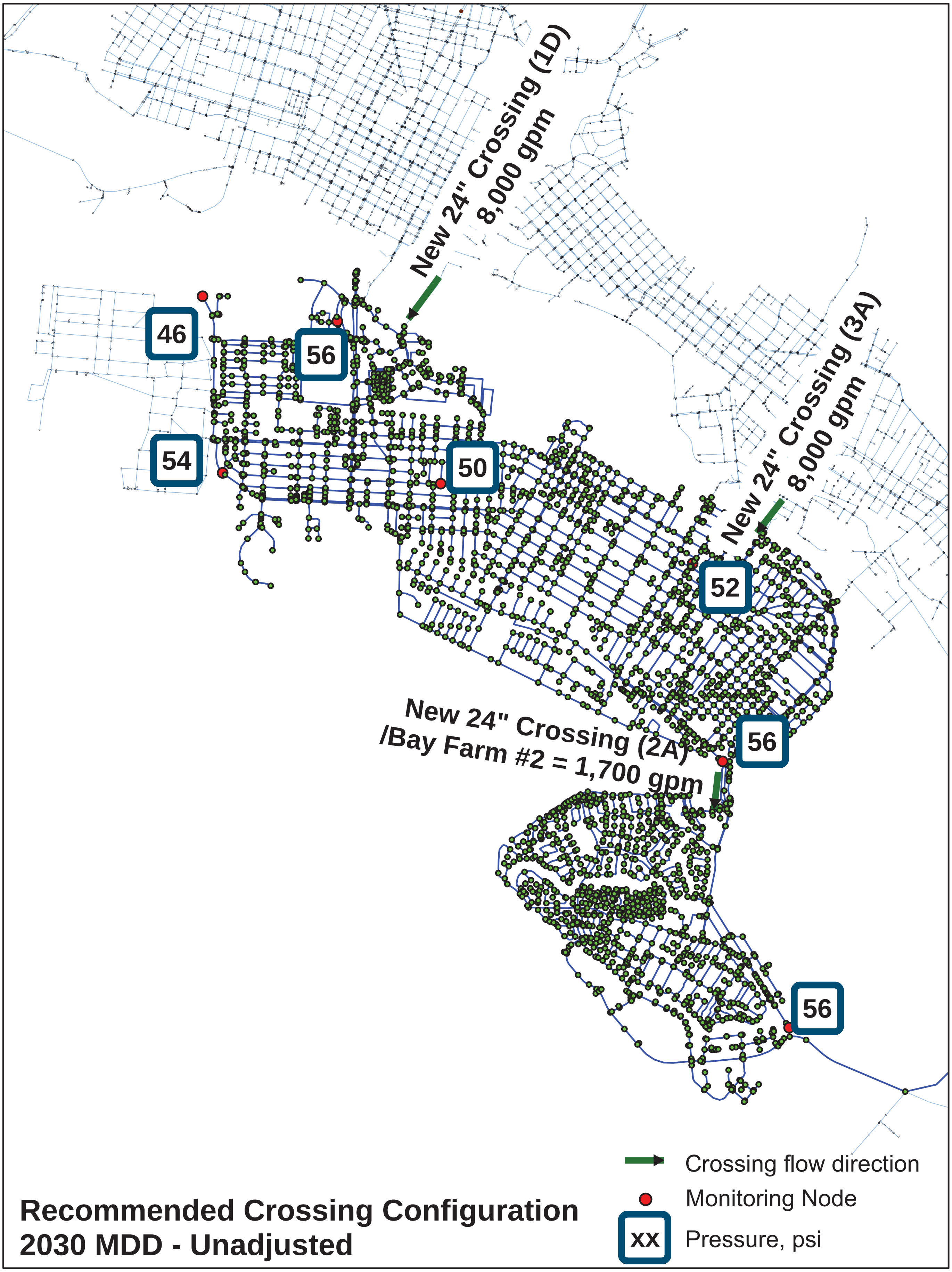


Figure 4-5

Recommended Crossing Configuration 2030 Flows and Pressures (unadjusted)

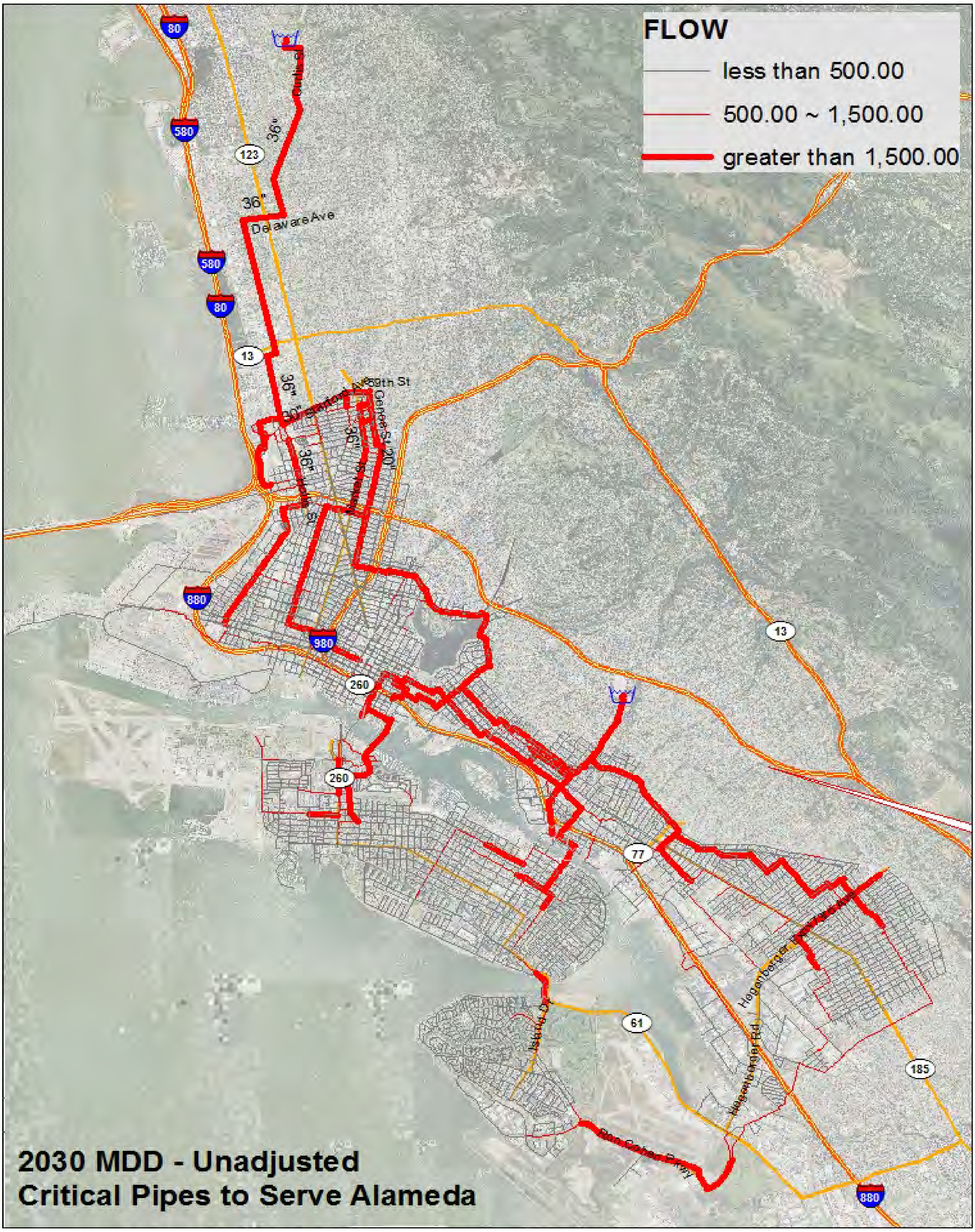


Figure 4-6
Transmission Pipelines to Almeda and Bay Farm Island

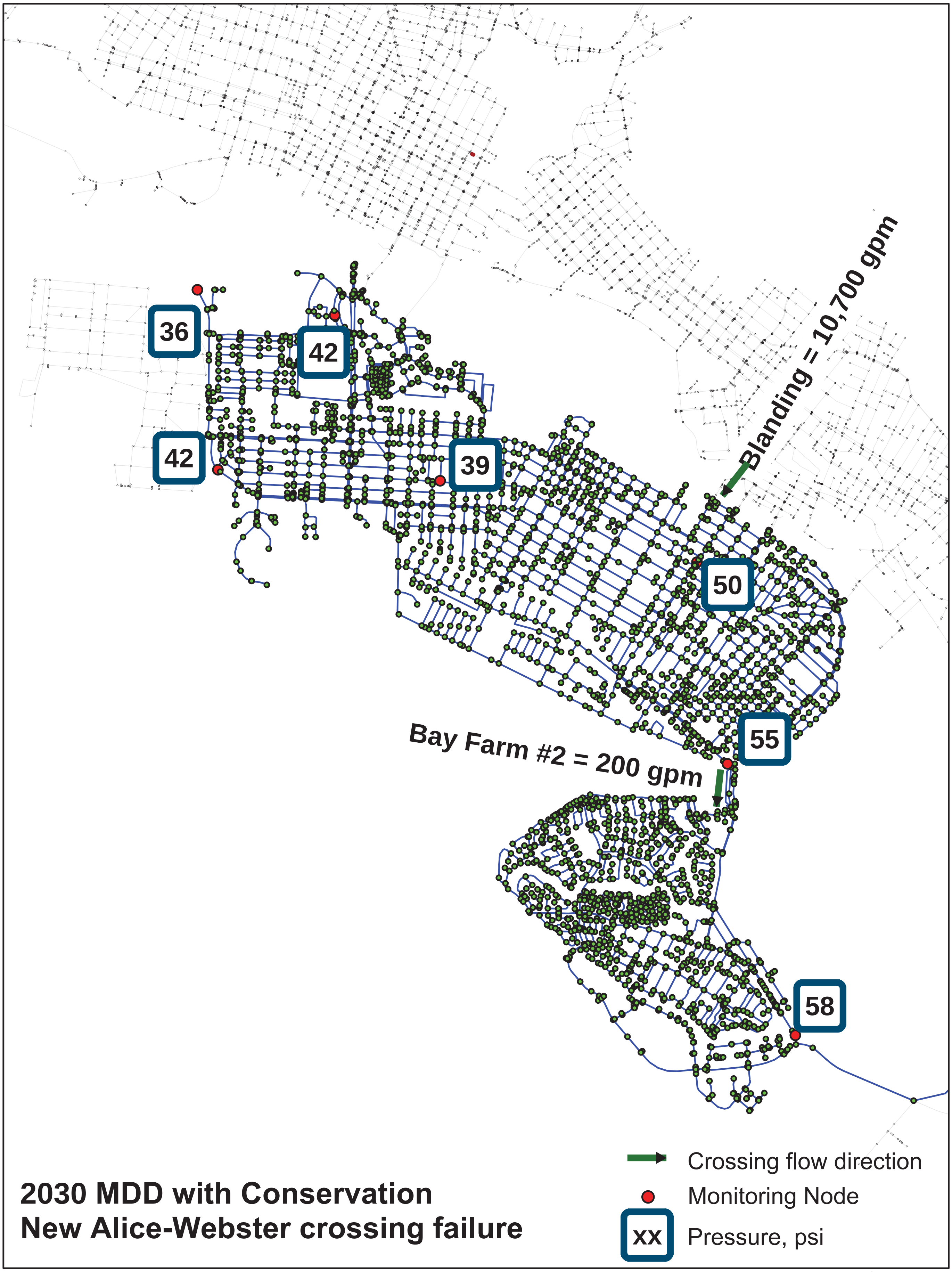


Figure 4-7

Alice-Webster Crossing Failure, 2030 Crossing Flows and Pressures

Figure 4-8

Blanding Crossing Failure, 2030 Existing Crossing Flows and Pressures

Figure 4-9

Bay Farm Island Crossing Failure, 2030 Existing Crossing Flows and Pressures

Figure 5-1

Critical Pipeline Crossing Locations

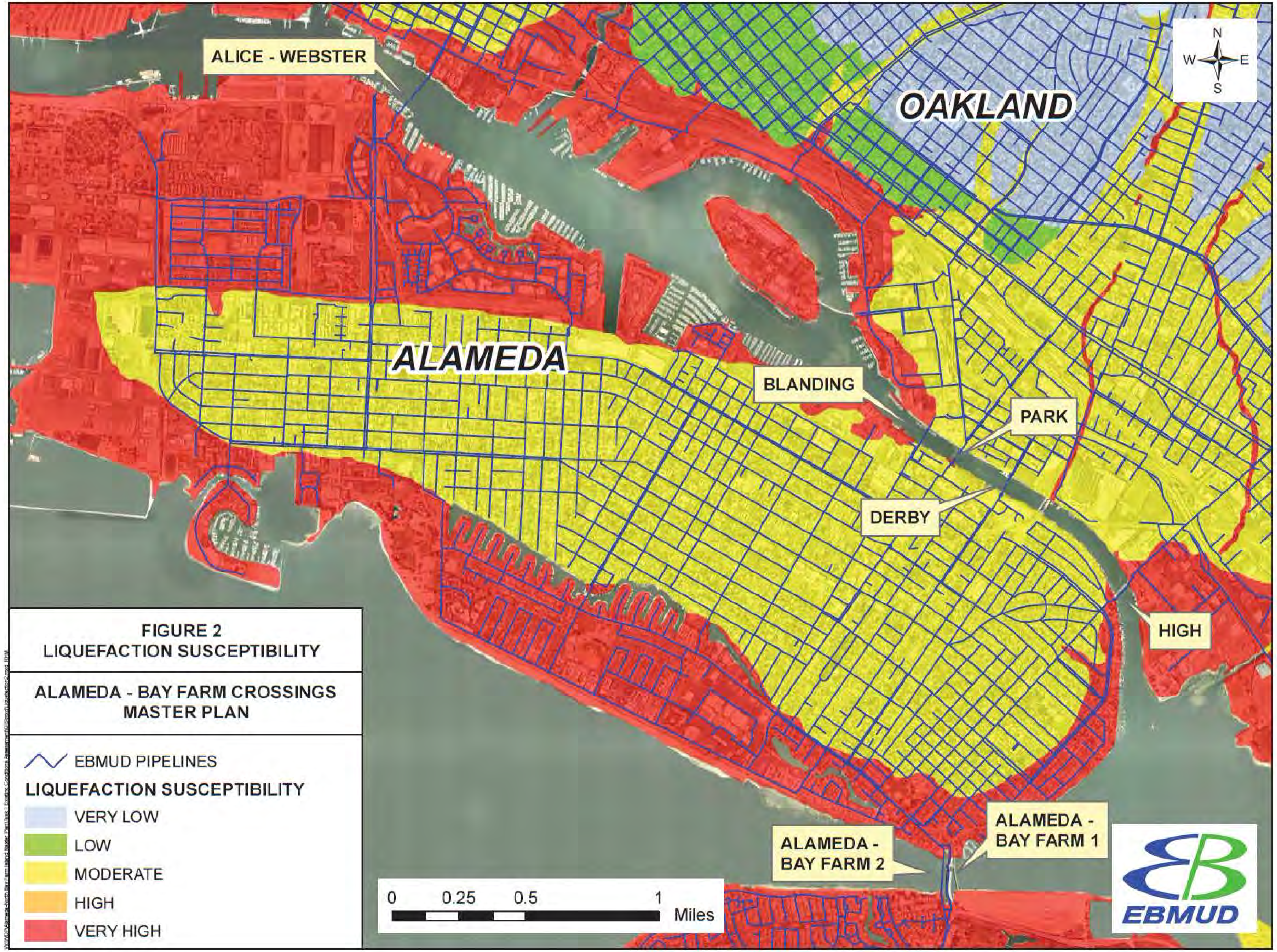


Figure 5-2

Soil Susceptibility to Liquefaction

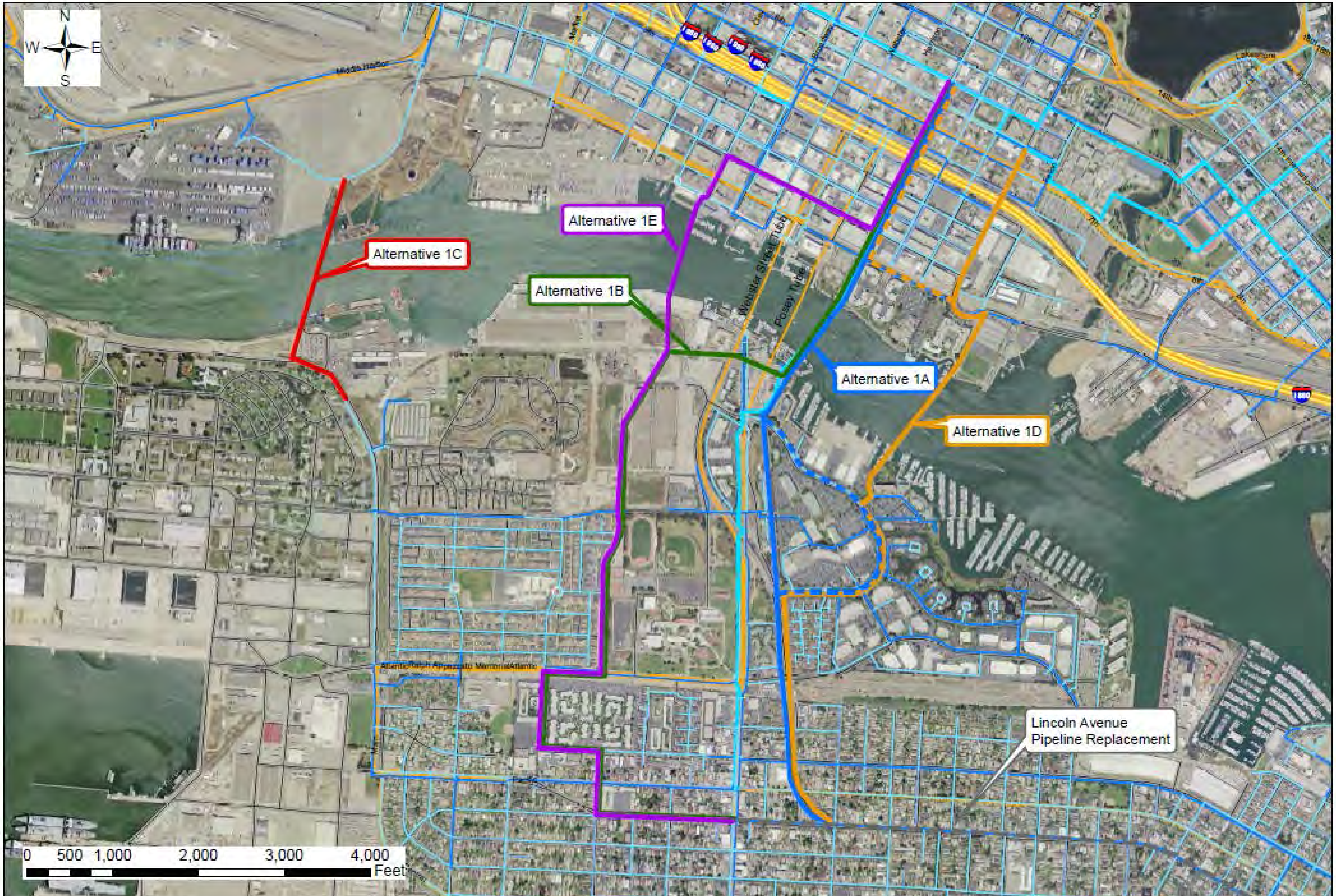


Figure 5-3

Alternative Alignments in Alice-Webster Vicinity

ALICE-WEBSTER CROSSING - ALTERNATIVES 1A & 1B

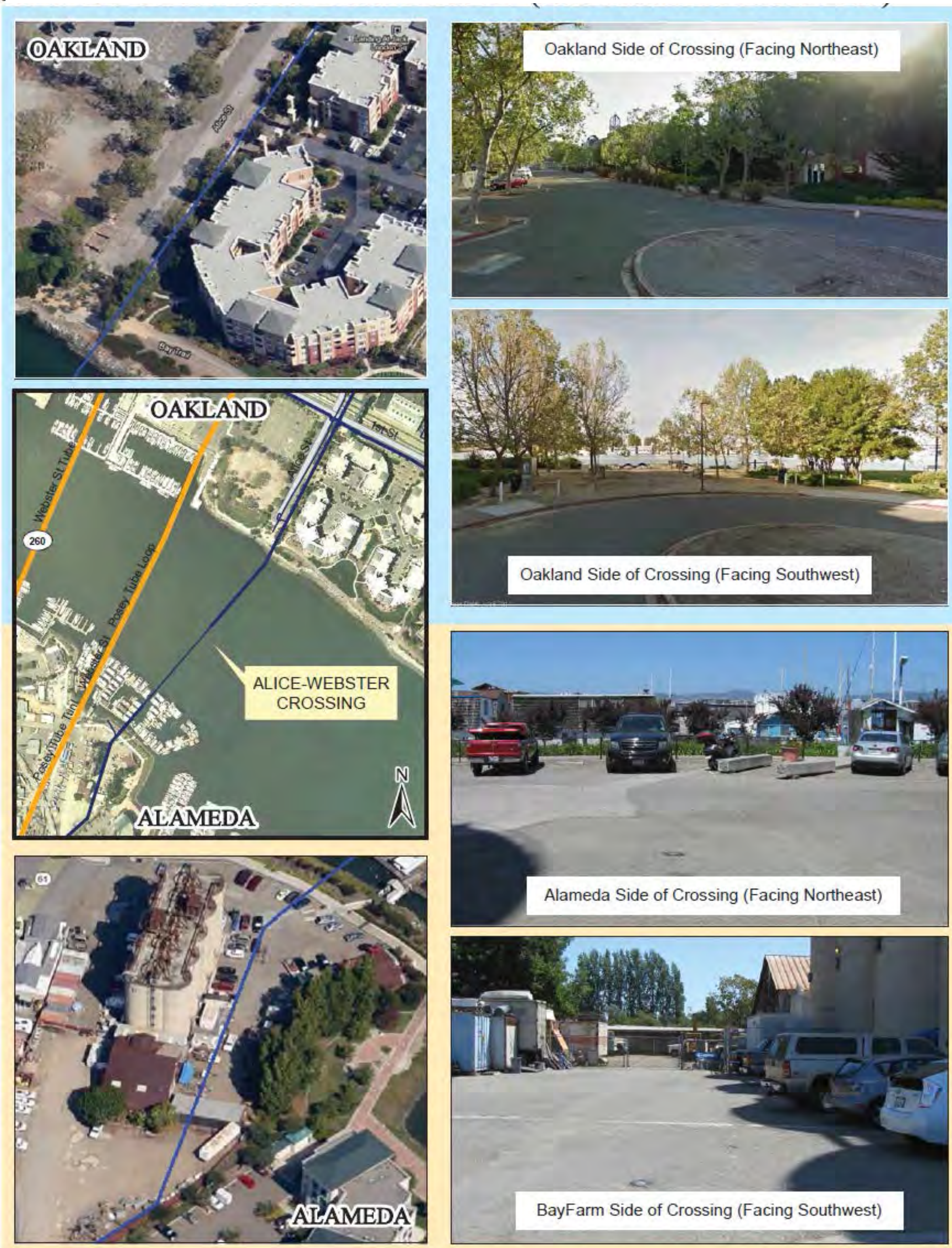


Figure 5-4

Alternative Alignments 1A and 1B Photos and Aerial

ALTERNATIVE 1D

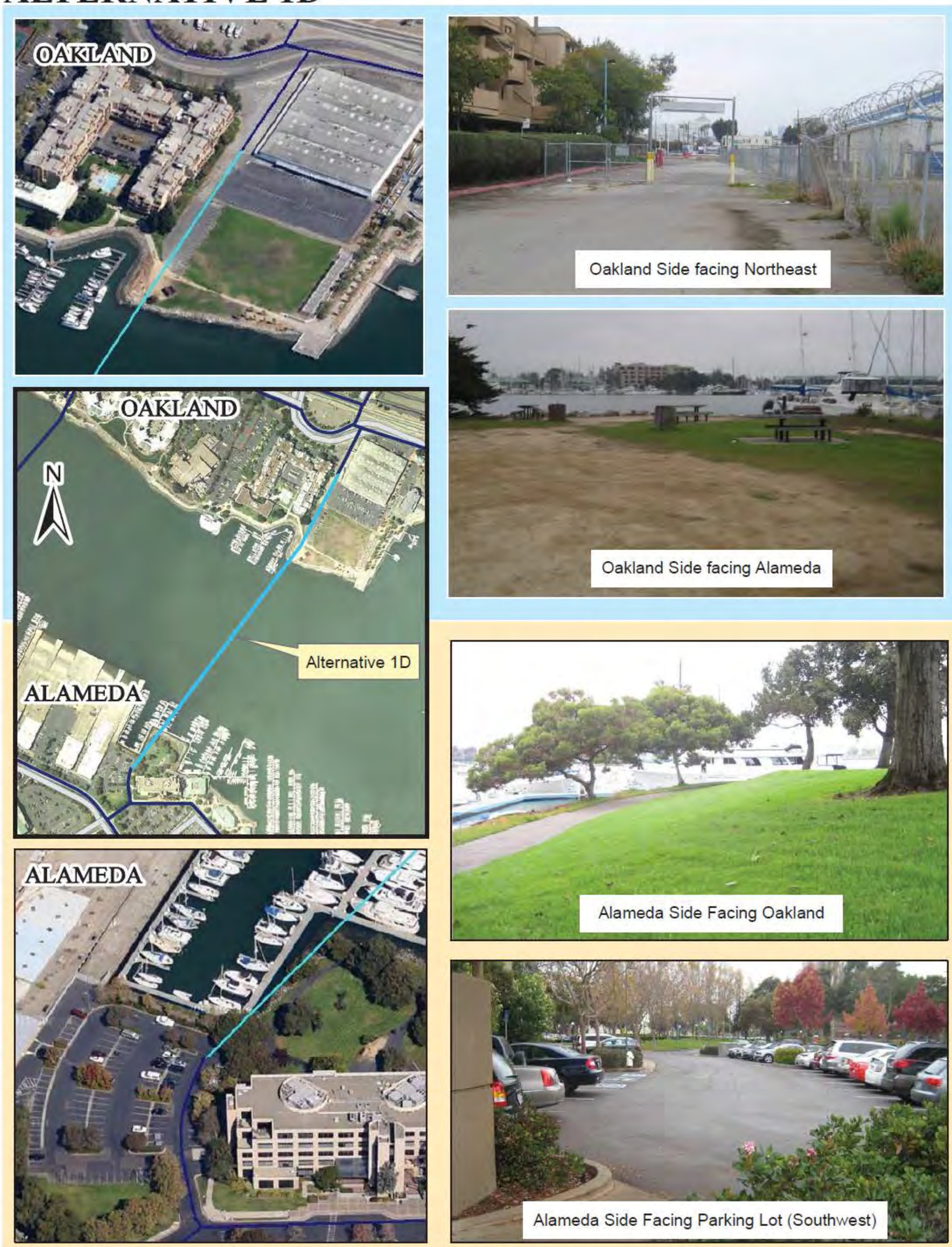


Figure 5-5

Alternative Alignment 1D Photos and Aerial

ALTERNATIVE 1E

Figure 5-6

Alternative Alignment 1E Photos and Aerial

Figure 5-7
Alternative Alignments in Alameda Bay Farm Island Bridge Vicinity

ALAMEDA BAY FARM 2 CROSSING – ALTERNATIVE 2A

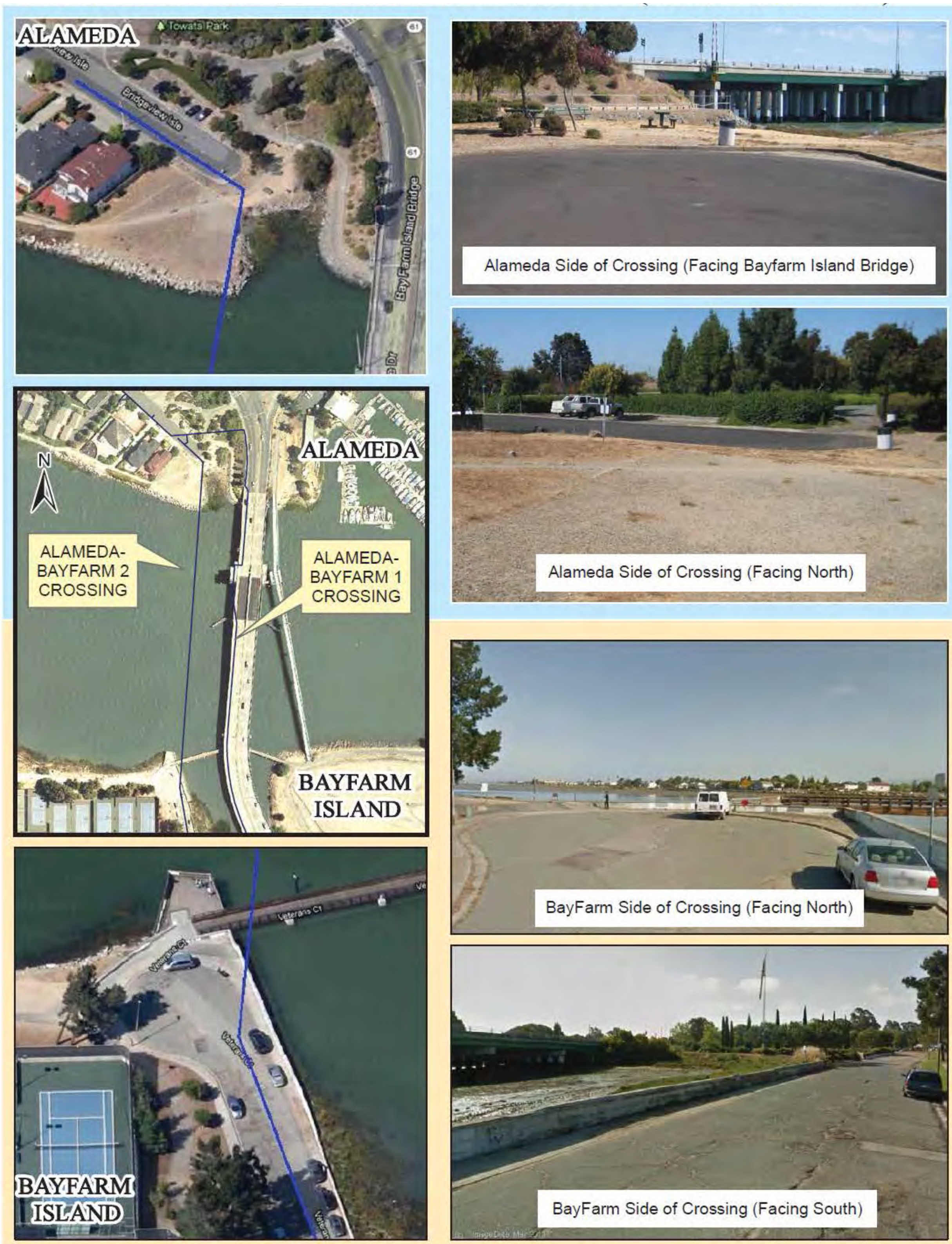


Figure 5-8

Alternative Alignment 2A Photos and Aerial

ALTERNATIVE 2B

Figure 5-9

Alternative Alignment 2B Photos and Aerial

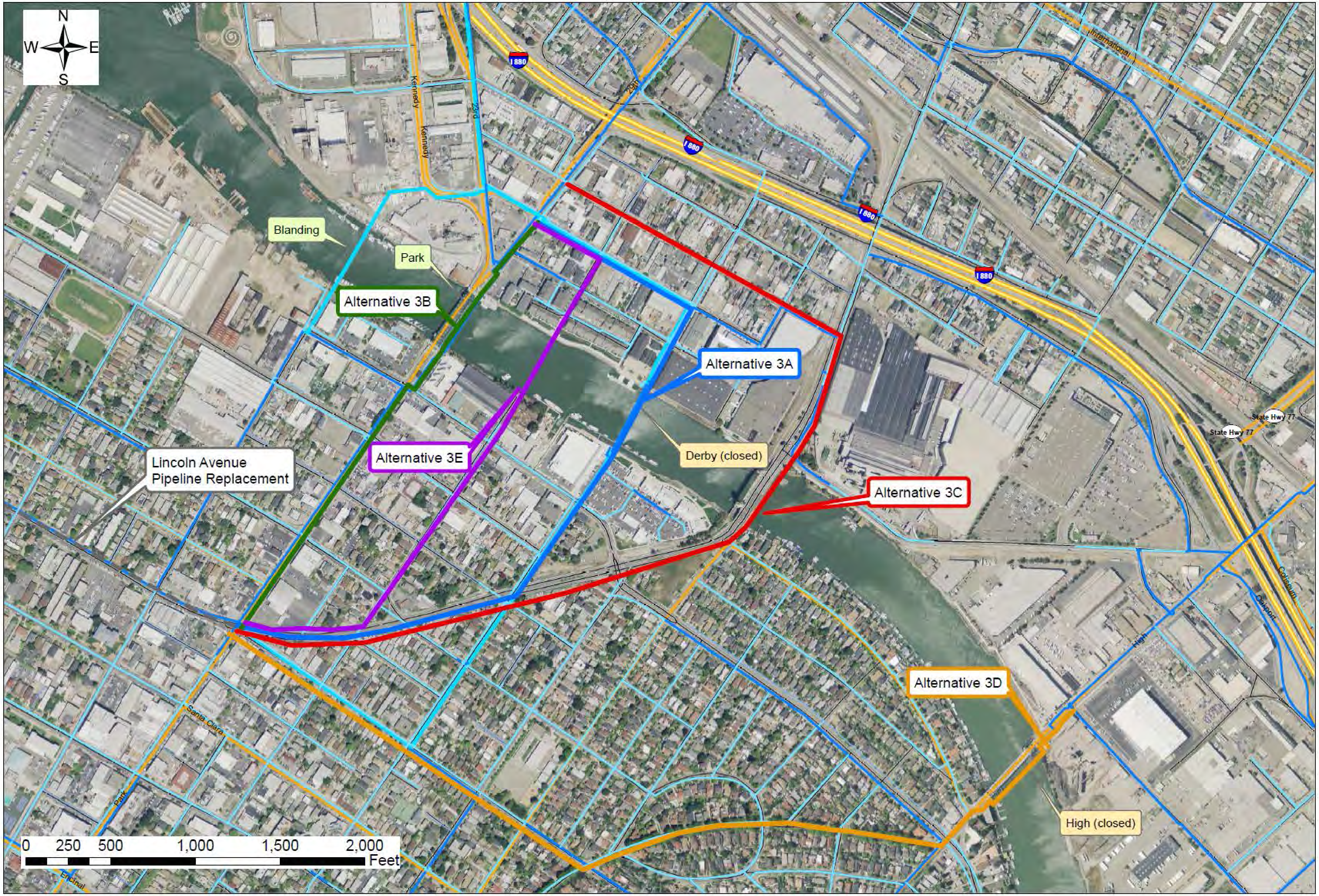


Figure 5-10

Alternative Alignments in Northeast Corridor Vicinity

DERBY CROSSING - ALTERNATIVE 3A

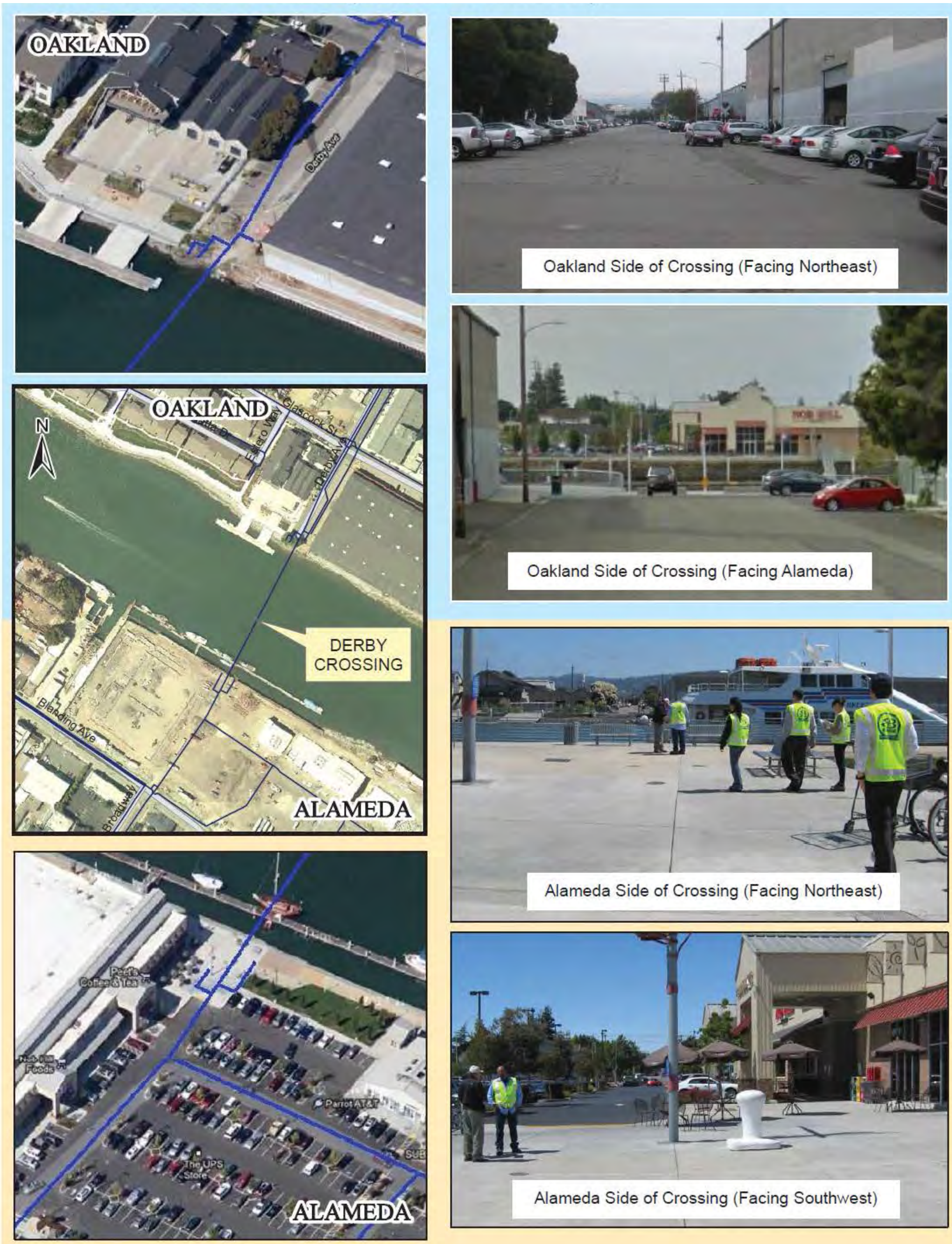


Figure 5-11

Alternative Alignment 3A Photos and Aerial

PARK CROSSING - ALTERNATIVE 3B

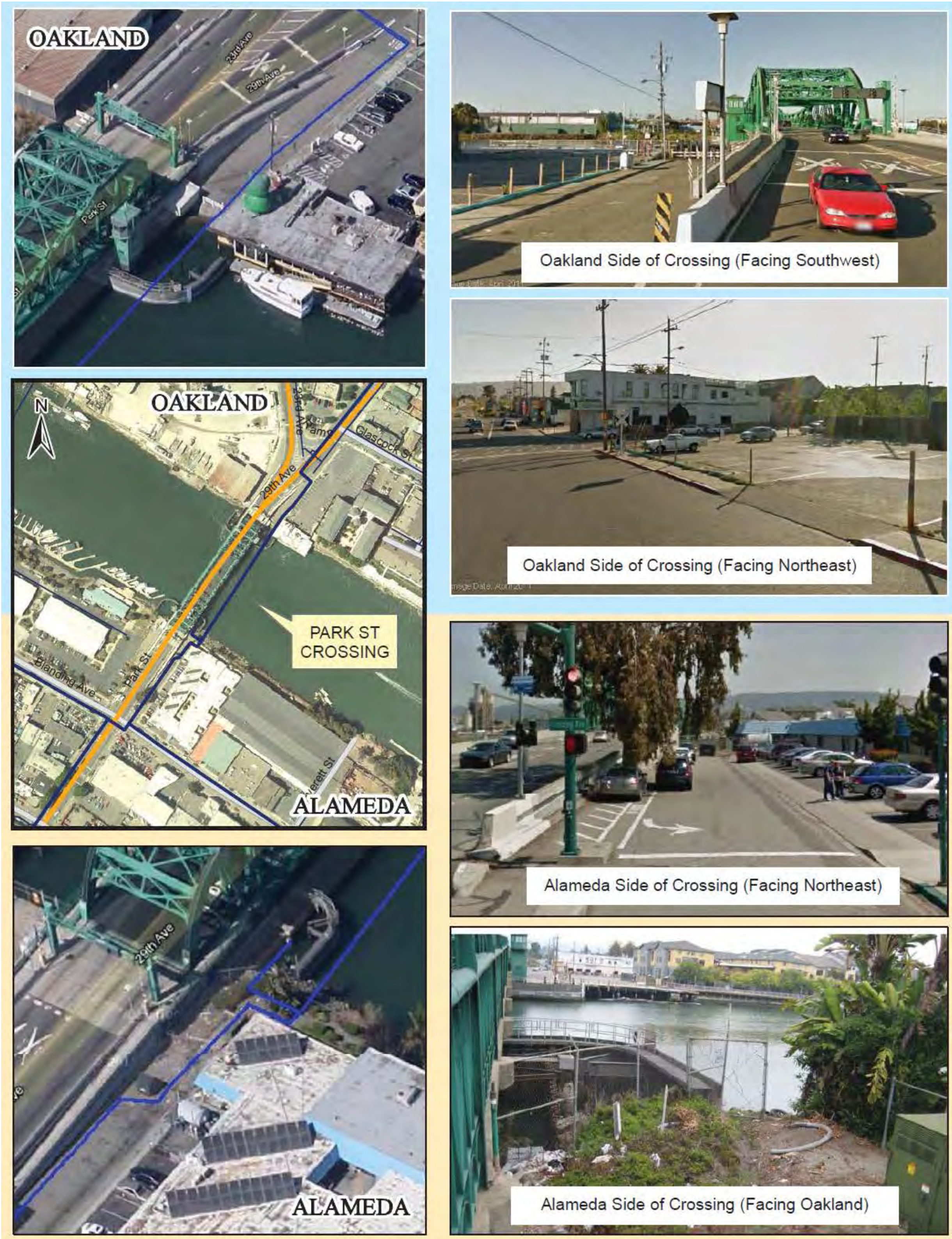


Figure 5-12

Alternative Alignment 3B Photos and Aerial

ALTERNATIVE 3C

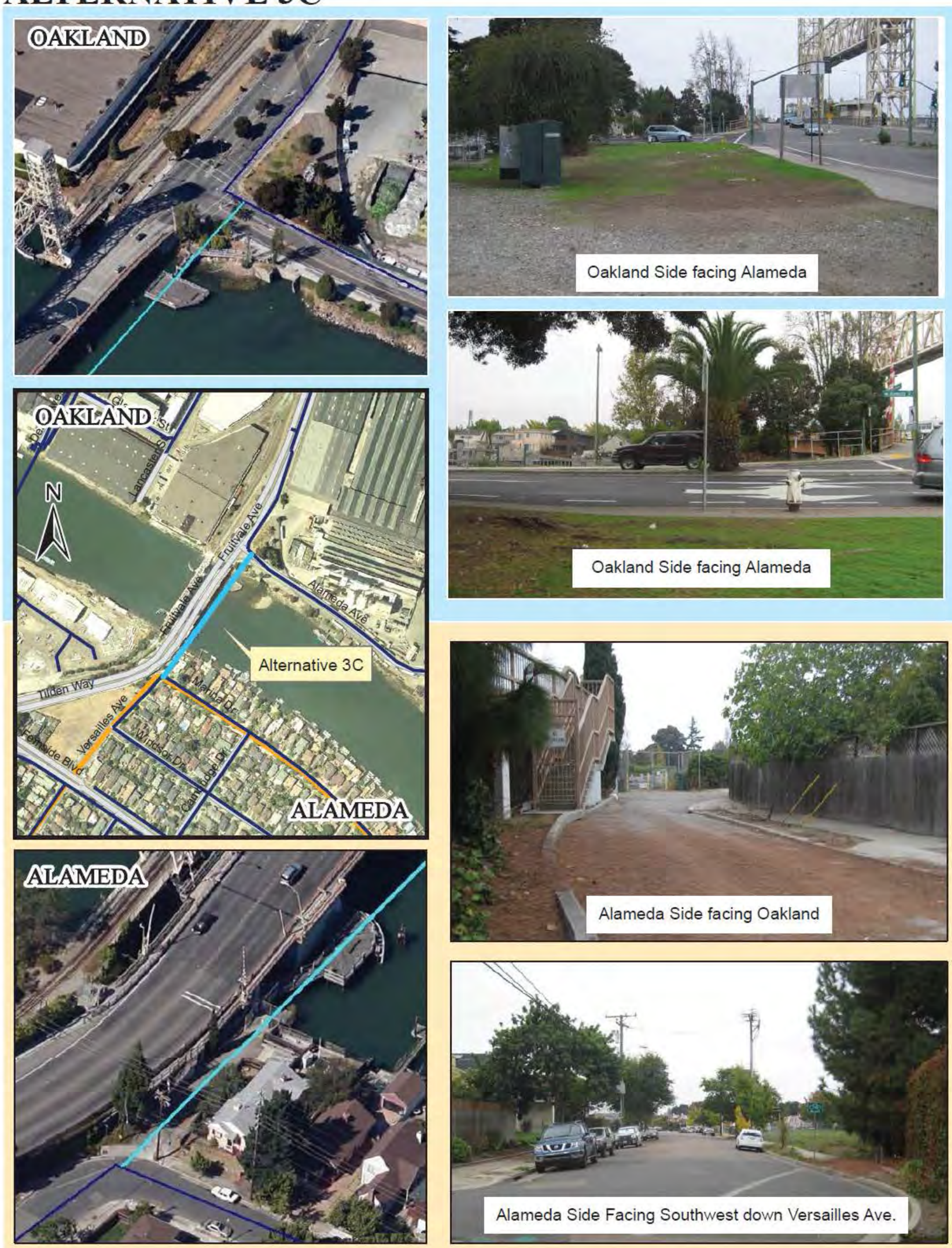


Figure 5-13

Alternative Alignment 3C Photos and Aerial

HIGH ST. CROSSING - ALTERNATIVE 3D

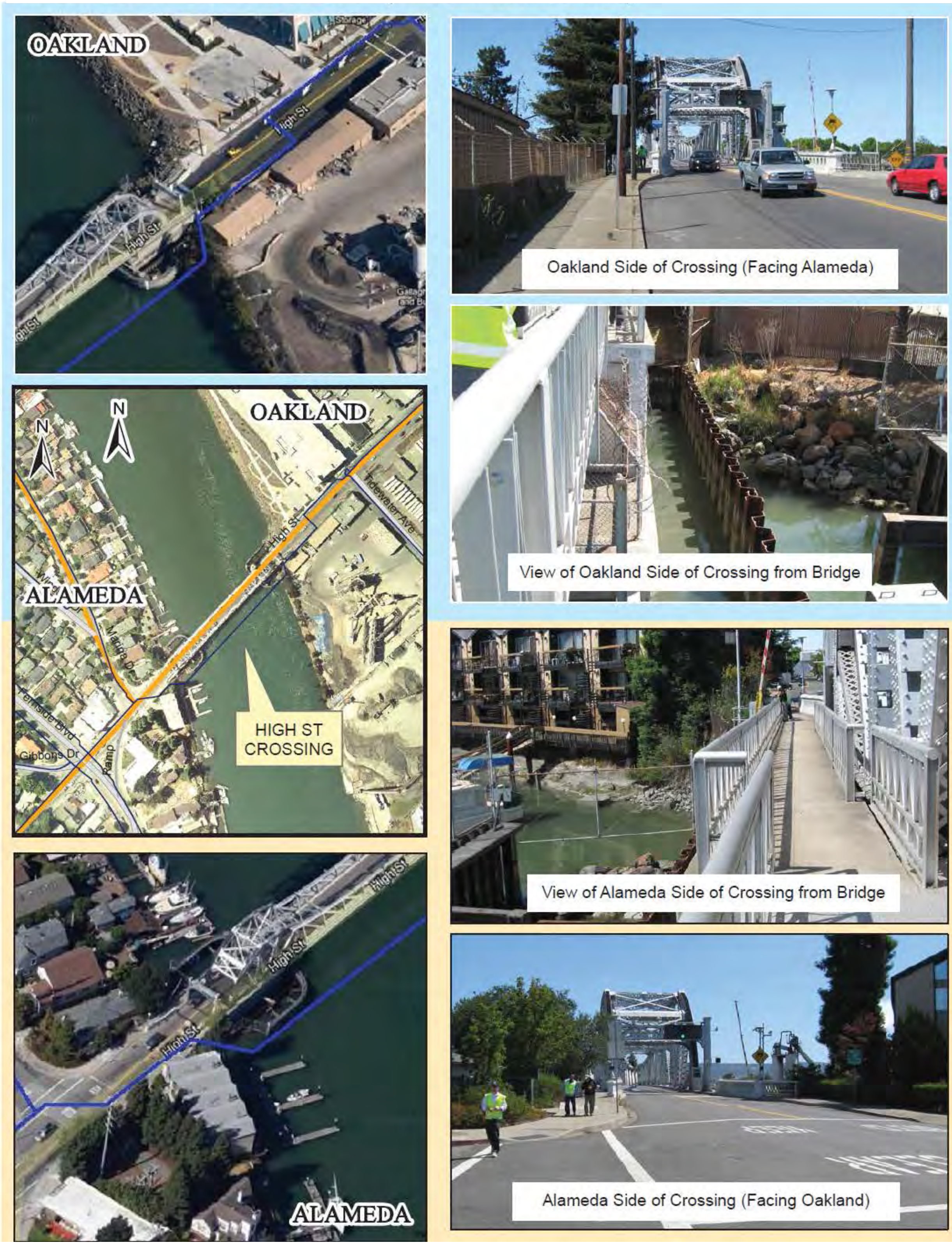


Figure 5-14

Alternative Alignment 3D Photos and Aerial

ALTERNATIVE 3E

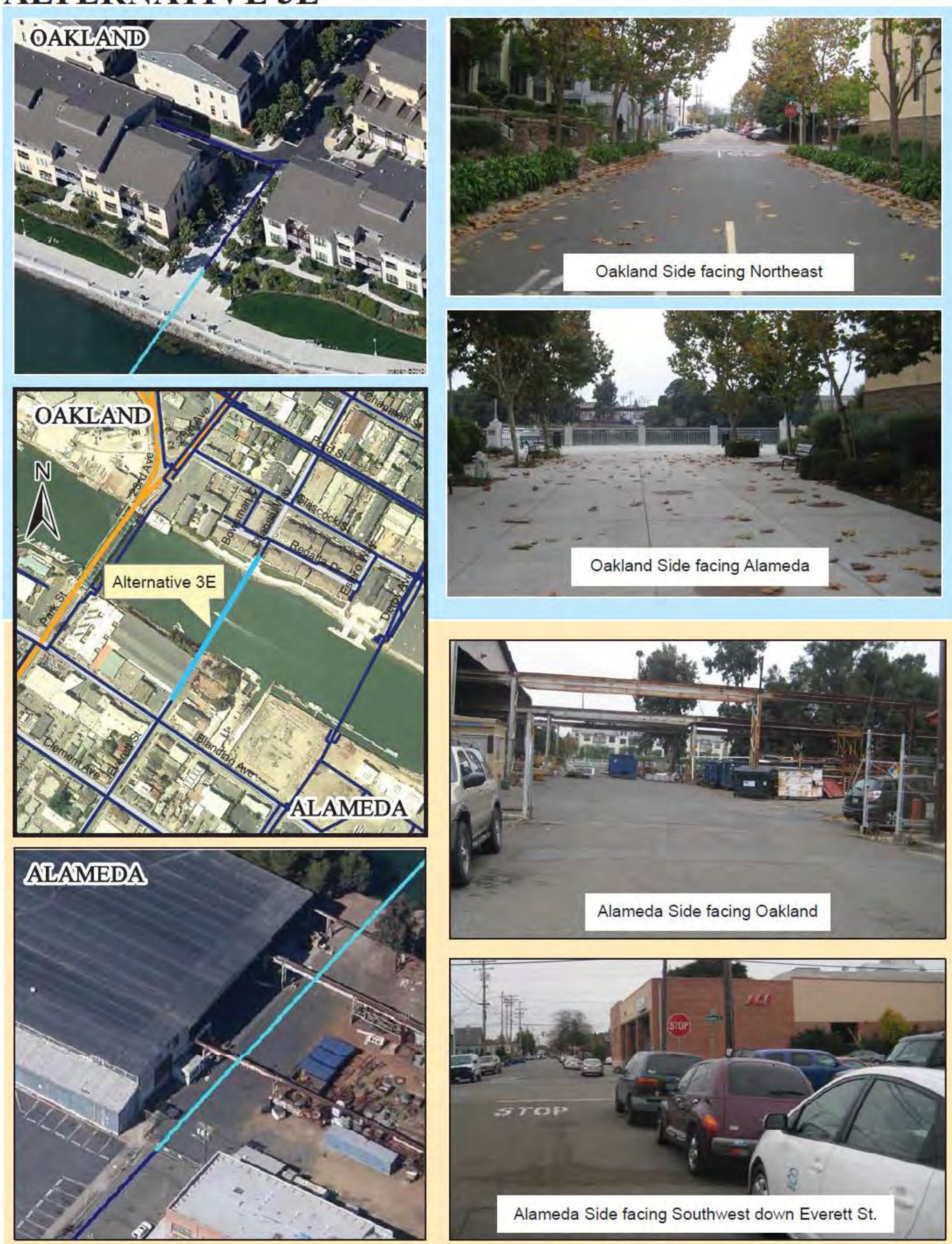


Figure 5-15

Alternative Alignment 3E Photos and Aerial

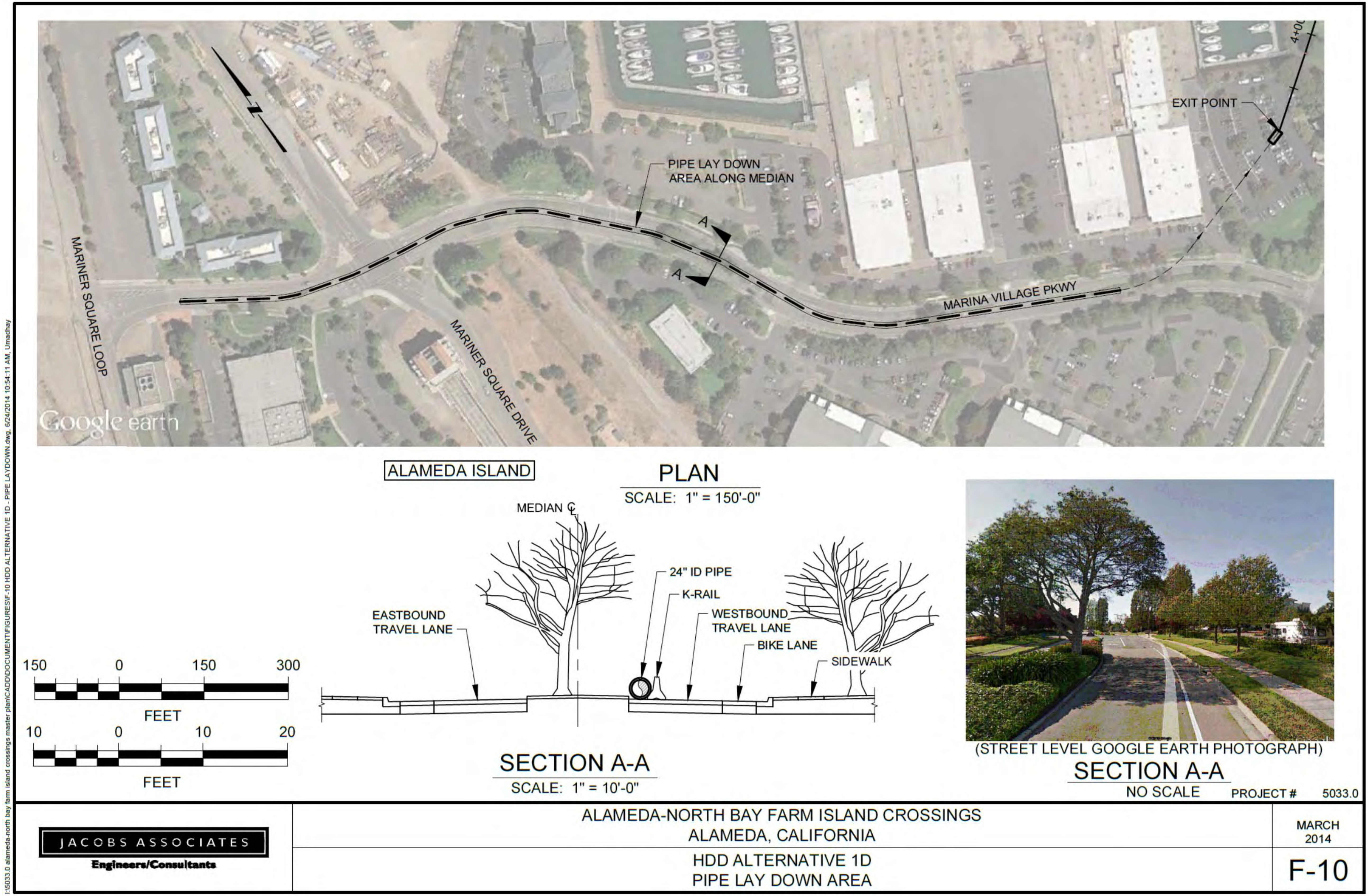


Figure 6-1
HDD Alternative 1D Pipe Laydown Area

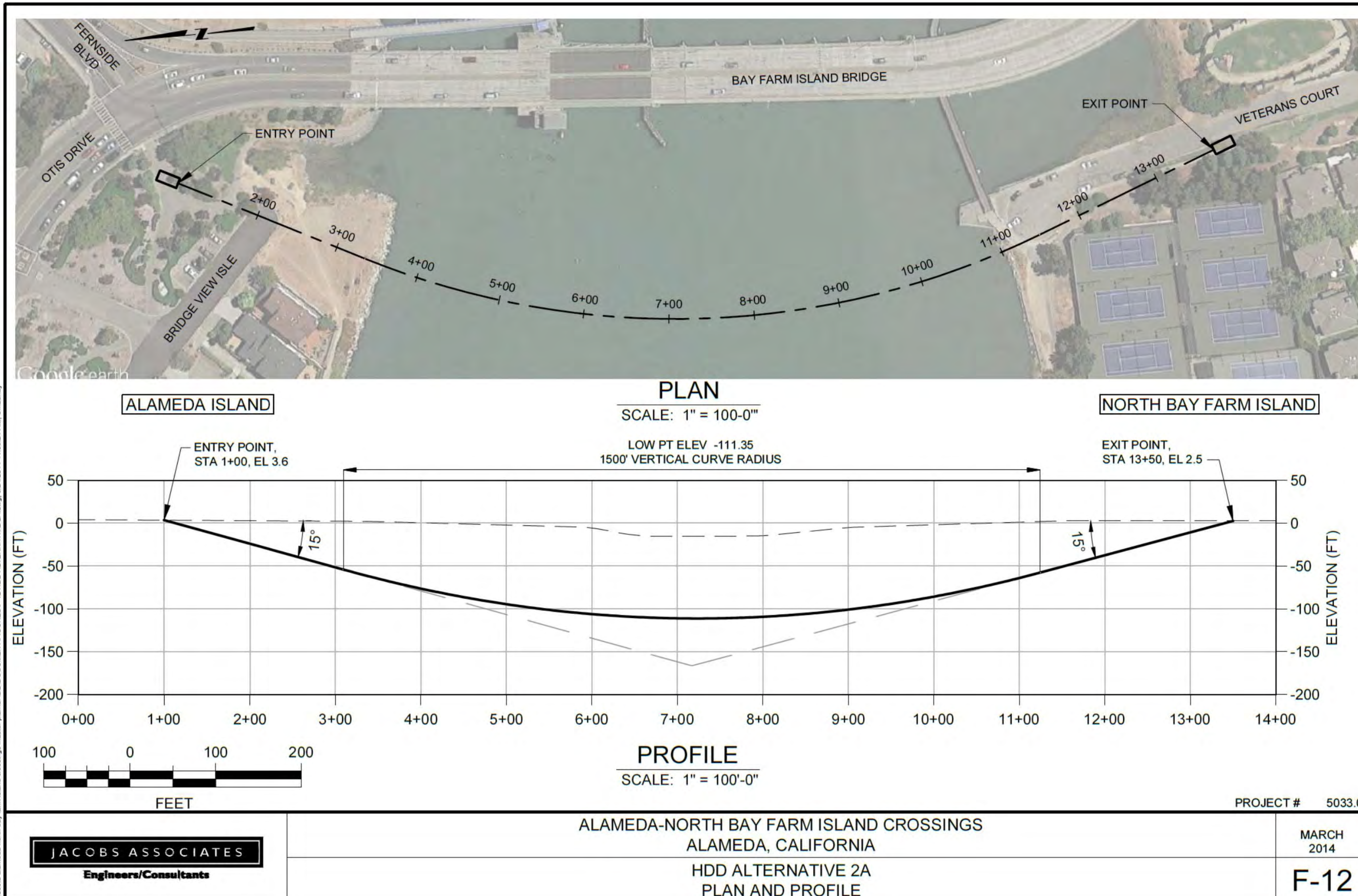


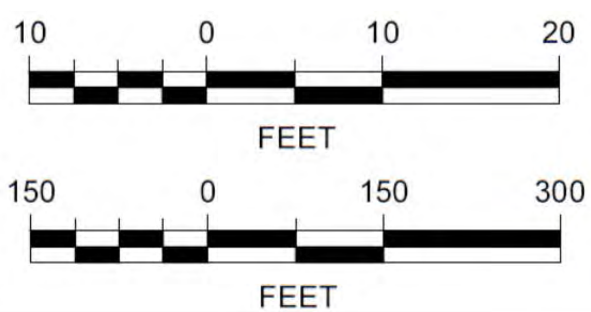
Figure 6-2

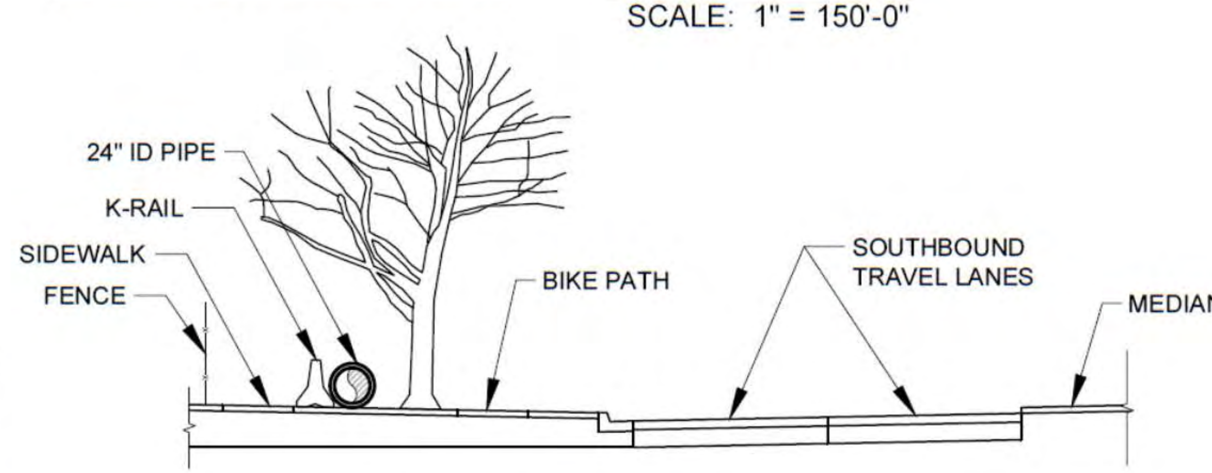
HDD Alternative 2A Plan and Profile

NORTH BAY FARM ISLAND

PLAN

SCALE: 1" = 150'-0"





SECTION A-A

SCALE: 1" = 10'-0"

(STREET LEVEL GOOGLE EARTH PHOTOGRAPH)

SECTION A-A

NO SCALE

PROJECT # 5033.0

JACOBS ASSOCIATES Engineers/Consultants	ALAMEDA-NORTH BAY FARM ISLAND CROSSINGS ALAMEDA, CALIFORNIA	MARCH 2014
	HDD ALTERNATIVE 2A PIPE LAY DOWN AREA	F-13

I:\5033.0 alameda-north bay farm island crossings master plan\CADD\DOCUMENT\FIGURES\F-13 HDD ALTERNATIVE 2A - PIPE LAYDOWN.dwg, 6/24/2014 11:08:47 AM, Umadhay

Figure 6-3

HDD Alternative 2A Pipe Laydown Area

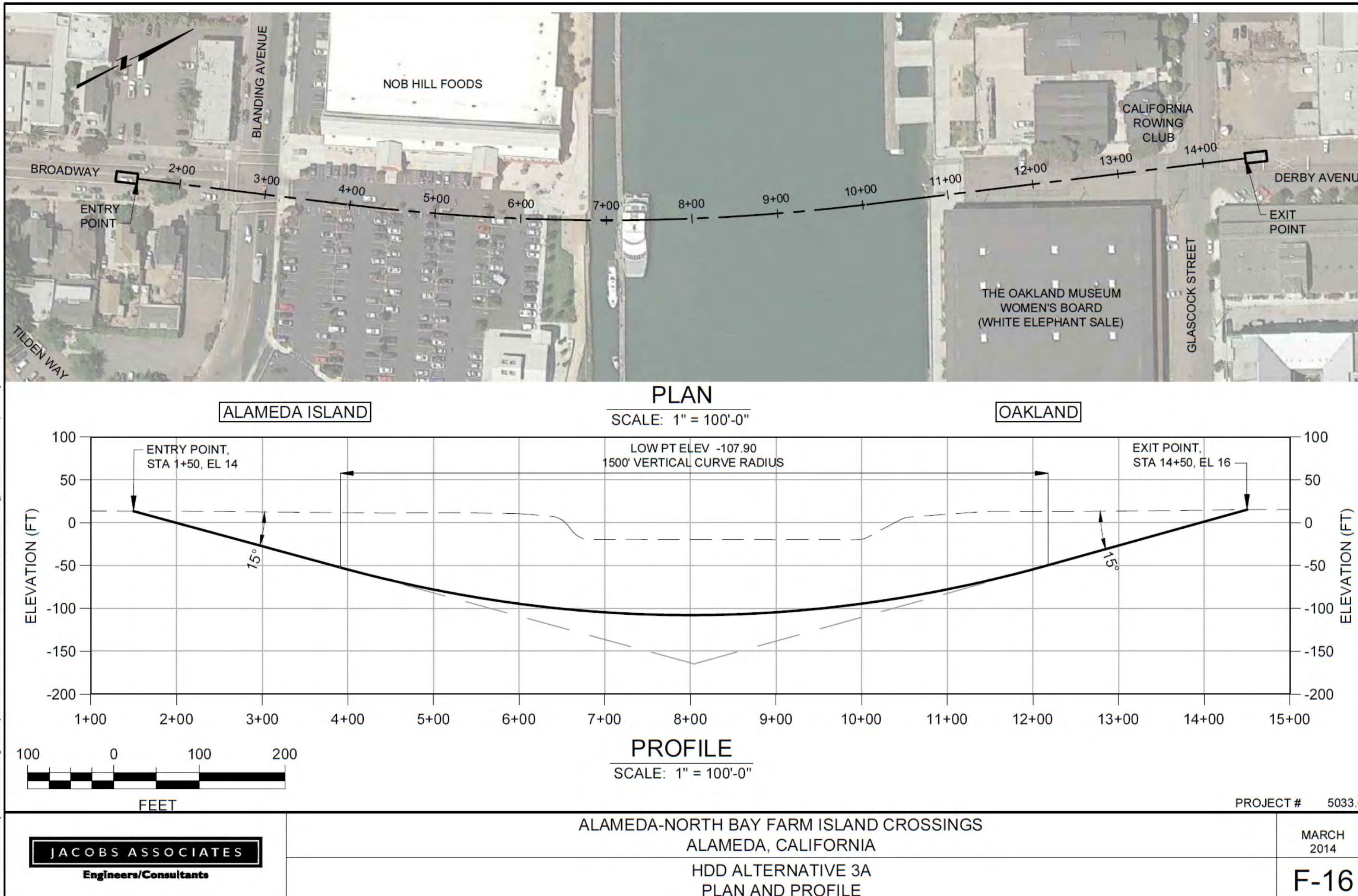


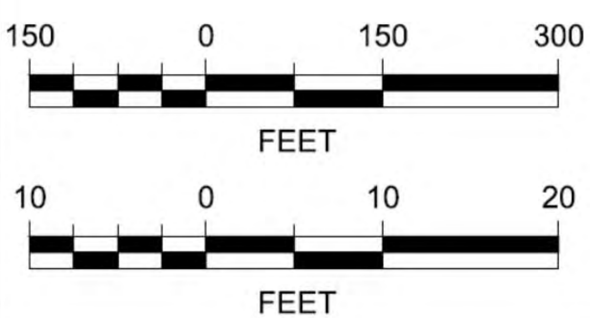
Figure 6-4

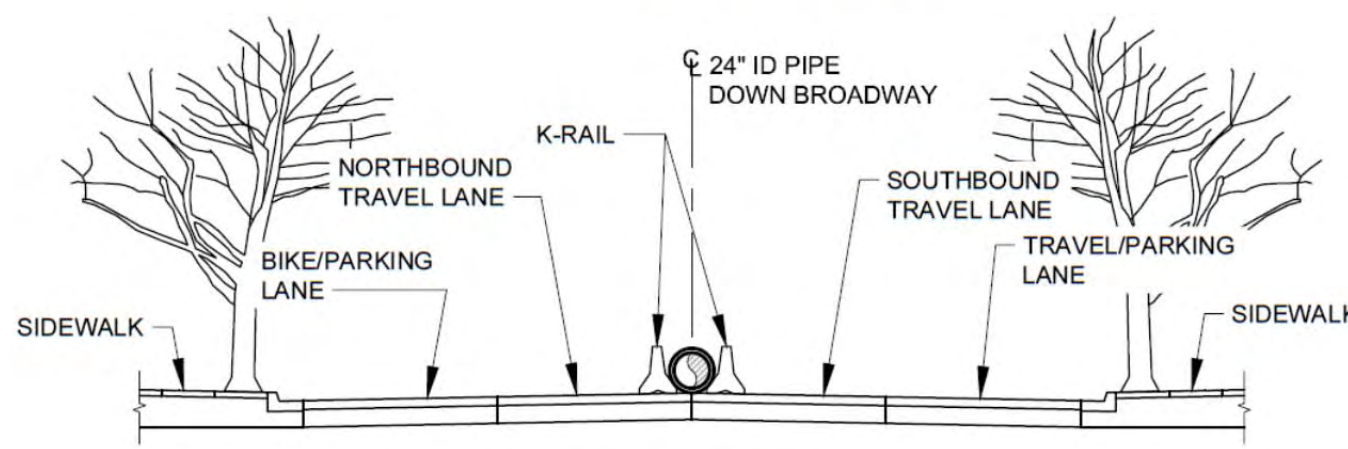
HDD Alternative 3A Plan and Profile

ALAMEDA ISLAND

PLAN

SCALE: 1" = 150'-0"





SECTION A-A

SCALE: 1" = 10'-0"

(STREET LEVEL GOOGLE EARTH PHOTOGRAPH)

SECTION A-A

NO SCALE

PROJECT # 5033.0

JACOBS ASSOCIATES Engineers/Consultants	ALAMEDA-NORTH BAY FARM ISLAND CROSSINGS ALAMEDA, CALIFORNIA	MARCH 2014
	HDD ALTERNATIVE 3A PIPE LAY DOWN AREA	F-17

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Figure 6-5

HDD Alternative 3A Pipe Laydown Area

Appendix A

Underwater Pipeline Crossings Feasibility Study, Volumes 1 and 2
(Volume 1 – DOX #2191923 and Volume 2 – DOX #2191925)

Alameda–North Bay Farm Island Crossings Master Plan

Underwater Pipeline Crossings Feasibility Study

March 2014

EAST BAY MUNICIPAL UTILITY DISTRICT

Prepared by:

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Reviewed By:	Glenn M. Boyce, PhD, PE Jacobs Associates

Revision Log

Revision No.	Date	Revision Description
0	November 10, 2013	Pre-Draft for Review
1	January 28, 2014	Draft for Review
2	March 28, 2014	Final

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Appendices (Volume 2)

Executive Summary

To achieve its goal of having a functional water pipeline crossing to Alameda Island following a major seismic event, the East Bay Municipal Utility District (District) through its Master Plan for the Alameda–North Bay Farm Island is considering three areas for new underwater pipeline crossings from Oakland to Alameda and North Bay Farm Islands. The three crossing areas are identified as follows:

- Oakland Inner Harbor crossing in the vicinity of the Posey Tube (Alice Street) – Area 1
- Crossing to North Bay Farm Island – Area 2
- Crossing in the vicinity of Derby Avenue – Area 3

The District has identified four preferred alignment alternatives at the three crossing areas: Alternatives 1A and 1D (Oakland Inner Harbor), 2A (Crossing to North Bay Farm Island), and 3A (Derby Avenue Crossing). This report presents the results of the feasibility study that examines the four proposed pipeline crossings in terms of:

- Major risks
- The best trenchless construction methods to reduce those risks while maximizing survivability and minimizing repair-related water service outages after a major seismic event
- Preliminary cost estimates and construction durations for the preferred alignments

The major risk for any water pipeline crossing from Oakland to Alameda and North Bay Farm Islands is damage from a major earthquake. The Maximum Considered Earthquake (MCE) for the project will include one of the following scenarios:

- Hayward M 7.0, on any segment of the Hayward fault (also accounts for a more distant San Andreas M 8.0 event)
- Hayward M 6.0, on the nearby northern segment of the Hayward fault
- Calaveras M 6.75, on the northern segment of the Calaveras fault
- Concord M 6.5, on the Concord fault

Since much of Alameda and North Bay Farm Islands are composed of hydraulic fills and soft soils overlying bedrock, the ground conditions are subject to liquefaction, lateral spread, and shoreline slope instability after a major earthquake. The existing underwater pipeline crossings constructed between 1912 and 1983 used open trench construction. The shallow depth of the existing pipelines places them in the ground conditions vulnerable to earthquake damage. To avoid these ground conditions, deeper underwater pipeline crossings are needed. This study looked to using either microtunneling or horizontal directional drilling (HDD) to construct new underwater pipeline crossing in the deeper, more stable ground conditions at each crossing location. The concept of using microtunneling or HDD was developed for Areas 1, 2, and 3.

A typical and recommended microtunnel crossing approach will consist of a jacking shaft from which the microtunneling boring machine (MTBM) and casing are advanced and a receiving shaft for retrieval of the MTBM. For the crossing evaluations, we assumed the following conditions for the shafts and trenchless crossings:

- Assume the use of secant piles for a circular jacking shaft with a working diameter of 28 feet for 20-foot-long pipe segments.
- Secant pile shafts are watertight and a circular configuration is selected for its efficiency in resisting ground forces in ring compression thereby eliminating internal struts, braces or walers.
- H-beams would be placed in selected secant piles to resist jacking forces.
- Assume the use of secant piles for a circular receiving shaft with a working diameter of 18 feet.
- The use of the receiving shaft is typically limited to retrieval of the MTBM. During non-use times, the receiving shaft can be plated over to maintain local traffic.
- The shaft depths would be selected deep enough to place the microtunneled pipeline crossing below the fills and Young Bay Mud and into the deep stable soils not prone to liquefaction.
- Microtunneling would install a 48-inch-diameter steel casing to obtain the distance needed for the crossings.
- The 24- or 36-inch diameter carrier pipe could be either (1) HDPE pipe or (2) steel pipe with a coating and lining.
- The carrier pipe would be grouted inside the steel casing.

A typical and recommended HDD crossing approach will consist of an entry and exit pit on each of side of the channel. For the crossing evaluations, we assumed the following conditions for the horizontal directionally drilled crossings:

- Assume an equal entry and exit angle varying from 10 to 15 degrees.
- Choice of the entry and exit angle will control the depth of the crossing under the channel.
- The entry and exit pits will be located at least 200 feet onshore to allow installation of the longer conductor casings and any associated ground improvement work.
- If the pilot hole hits a buried object, the drill steel will be removed from the bored hole, and new similar alignment will be drilled.
- The HDD crossing will install 24-inch ID HDPE carrier pipe with a 30-inch OD.
- The bored hole diameter will be 39 inches or larger to allow pullback of carrier pipe.
- The minimum vertical curve radius will be 1,500 feet (R = 50 x 30).
- Available space to layout the pipeline will control which side of the crossing is the exit point, and which side is the entry point.
- Sufficient easement space would be available for pipe fusion and pullback.
- The pipeline will be assembled as one long pipe string and tested before pullback.
- The carrier pipe will sit in the bored hole where the drilling mud will obtain gel strength to lock the pipe in-place after installation.
- The annular space between the carrier pipe and conductor casing will be filled with backfill grout.
- The ground under the conductor casing on both sides of the crossing will be improved with jet grouting to prevent liquefaction of the near-surface portions of the crossings.

The three crossing areas resulted in site specific alignments needed to position jacking/receiving shafts and entry/exit points for the trenchless crossings in stable ground, far enough back from the shoreline to avoid zones of seismic instability. These start and end points resulted in different lengths of the crossings. Historical records and maps were also collected, and the alignments modified to avoid potential buried objects along the trenchless alignment.

Preliminary cost estimates and construction durations were established by preparing a bottom-up cost estimate for the microtunnel and HDD options. The cost estimates were generated in the same manner as what a contractor does to prepare a bid. Using the cost estimates, direct unit costs for the key items were developed. To obtain a total cost for the preferred alignments at each area, the quantities were extended per item, and then mobilization, overhead, profit, and contingency percentages were applied. Table ES-1 reports the budgetary costs for each of the alignments included in this Feasibility Study. The HDD options (*) include an additional lump sum cost of $706,000 for ground improvement under the entry and exit conductor casings for improved seismic resistance to shoreline slope instability.

Table ES-1. Summary Cost Estimate for Proposed Alignments by Means of Construction

Options	Length	Construction Costs
1A- Microtunnel	1,216 feet	$7,800,000
1D - HDD	1,780 feet	$4,216,000*
2A - Microtunnel	903 feet	$6,990,000
2A -HDD	1,250 feet	$3,296,000*
3A Microtunnel (Long)	1,150 feet	$6,660,000
3A –Microtunnel (Short)	950 feet	$6,100,000
3A - HDD	1,300 feet	$3,396,000*

The total construction duration from Notice to Proceed for the microtunnel alignments (1A, 2A, and 3A) is 10 to 12 months. This includes six months for the submittal process together with procurement of pipe and microtunnel equipment for all alignments. During the procurement period, the contractor can mobilize and construct the jacking and receiving shafts. The activities to setup the MTBM, complete tunneling, install pipe, grout, and backfill shafts have an anticipated duration of three to five months. One month is expected for punch list work and demobilization. Given that the majority of the activity durations are fixed, such as microtunnel equipment procurement, the variations in tunnel length and shaft depth do not result in significant differences in construction duration estimates at this feasibility level.

The total construction duration from Notice to Proceed for the HDD alignments (1D, 2A, and 3A) is five to seven months. This includes two to three months for the submittal process together with procurement of pipe and equipment for all alignments. The HDD setup, pilot hole, and multiple reaming passes, including placement of the pipe, have an anticipated duration of one to two months. One month is expected for punch list work and demobilization. Given that the majority of the activity durations are fixed, the difference in pipe length does not result in a significant difference in construction duration estimates at this feasibility level.

The new trenchless crossings can be designed to survive the identified earthquake scenarios. We list the probability of failure as “very very low” (<0.5%) for all microtunnel alternatives and “very low” (<1%) for all directionally drilled alternatives, and for all earthquakes. This assumes that the tunnel/directional drill pipes will be founded in competent materials and that the entry/exit points are located outside of, or otherwise designed to accommodate any settlements and lateral spreads that might occur.

The results and information presented in this Feasibility Study will be used in the District’s uncertainty model, which will be run as a separate analysis to identify the preferred alternative alignment(s).

1 Introduction

To achieve its goal of having a functional water pipeline crossing to Alameda Island following a major seismic event, the East Bay Municipal Utility District (District) through its Master Plan for the Alameda–North Bay Farm Island is considering three areas for new underwater pipeline crossings from Oakland to Alameda and North Bay Farm Islands. The three crossing areas are identified as follows:

- Oakland Inner Harbor crossing in the vicinity of the Posey Tube (Alice Street) – Area 1
- Crossing to North Bay Farm Island – Area 2
- Crossing in the vicinity of Derby Avenue – Area 3

The District has identified four preferred alignment alternatives at the three crossing areas shown in Figure 1-1: Alternatives 1A and 1D (Oakland Inner Harbor), 2A (Crossing to North Bay Farm Island), and 3A (Derby Avenue Crossing). This report presents the results of the feasibility study that examines the four proposed pipelines in terms of:

- Major risks
- The best trenchless construction methods to reduce those risks while maximizing survivability and minimizing repair-related water service outages after a major seismic event
- Preliminary cost estimates and construction durations for the preferred alignments

The results and information presented in this Feasibility Study will be used in the District's uncertainty model, which will be run as a separate analysis. Also presented in this report is information regarding the seven existing crossings that may be relevant as input to the uncertainty model.

The District constructed these existing pipeline crossings over the last approximately 100 years. Five go from the Oakland to Alameda Island, and two go from the Alameda Island to North Bay Farm Island. The existing pipeline crossings are part of the Central Pressure Zone (G0A). The seven existing crossings are:

1. Alice Street Channel Crossing
2. Oak Street (Blanding) Channel Crossing
3. Bay Farm #2 Crossing
4. Park Street Crossing
5. Broadway to Derby Avenue (Derby) (Closed)
6. Bay Farm #1 Island Bridge (Closed)
7. High Street (Closed)

Figures 1-2 and 1-3 show these seven existing pipeline crossings. Figure 1-2 also shows the locations of actual pipeline damage in the 1989 Loma Prieta earthquake (small squares). Figure 1-3 highlights the style of pipeline and year of original installation for the seven existing pipeline crossings. In Figures 1-2 and 1-3, the shading of the soils depicts the local surficial soil conditions (dark grey being Young Bay Mud and stippled depicting Merritt sands).

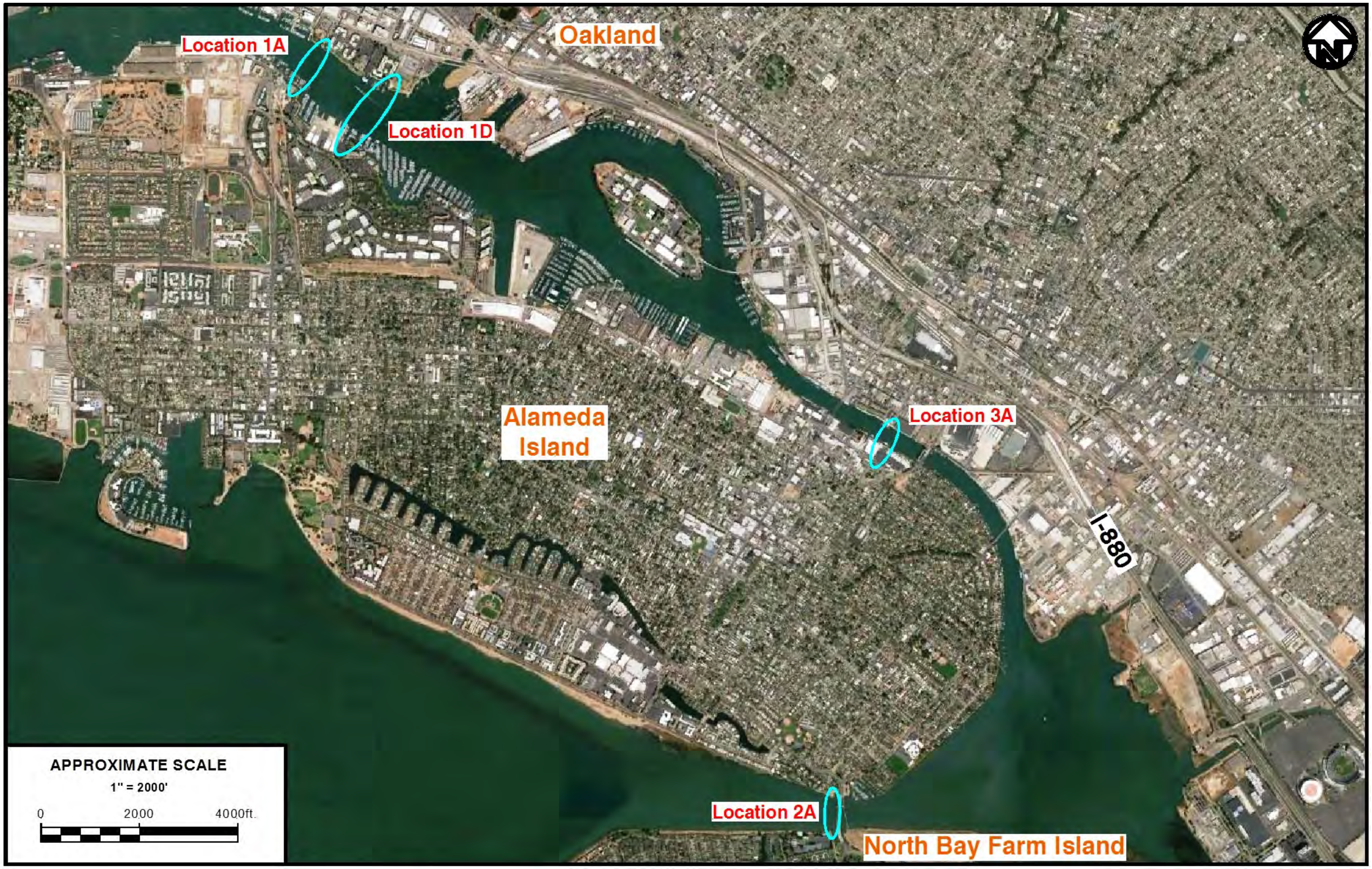


Figure 1-1. Proposed Pipeline Crossing Alternatives 1A and 1D (Oakland Inner Harbor), 2A (Bay Farm Crossing), and 3A (Derby Avenue)

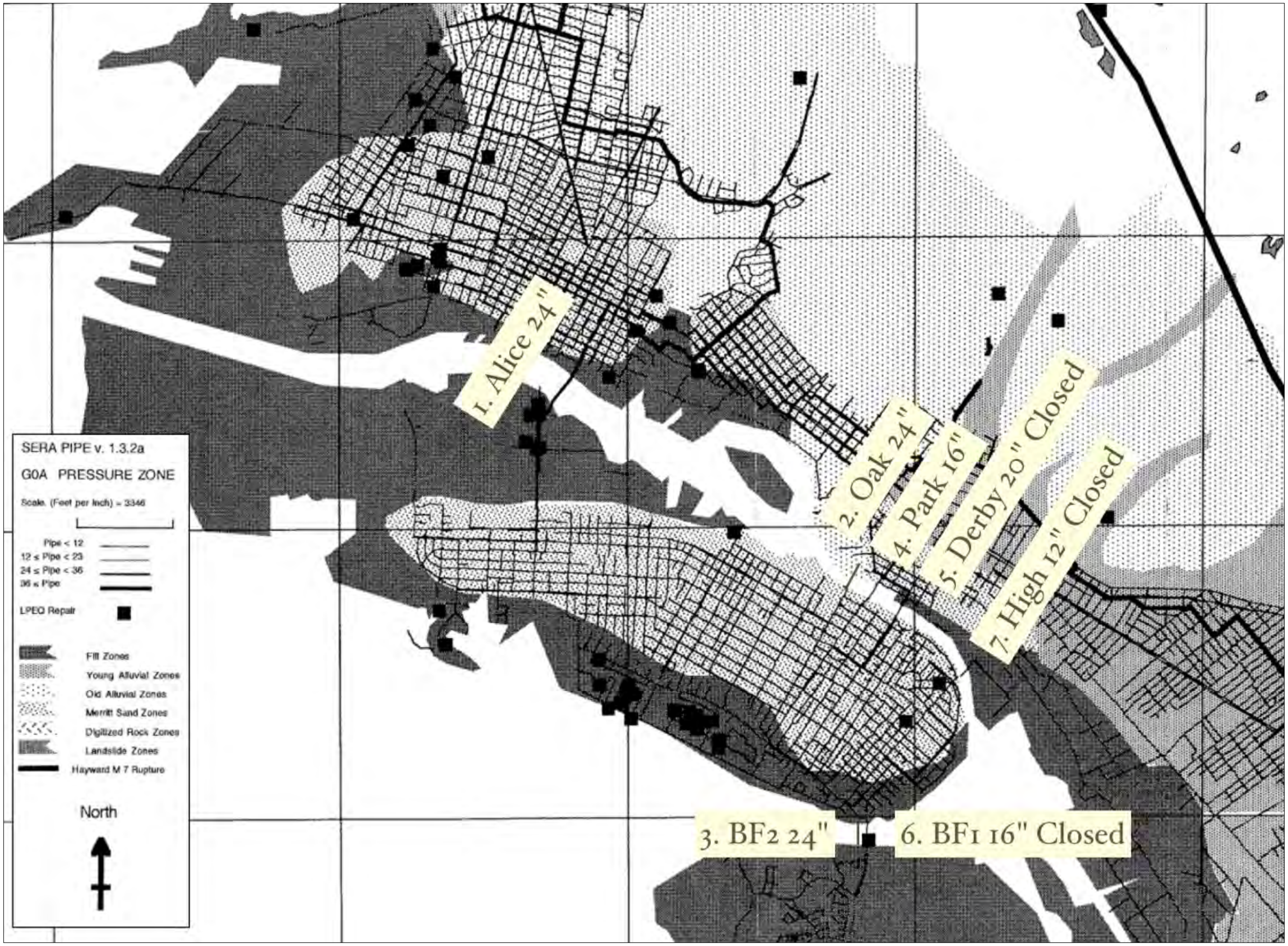


Figure 1-2. Existing Pipeline Crossings (2014) to Alameda and North Bay Farm Islands (gridlines are at 10,000-foot intervals)

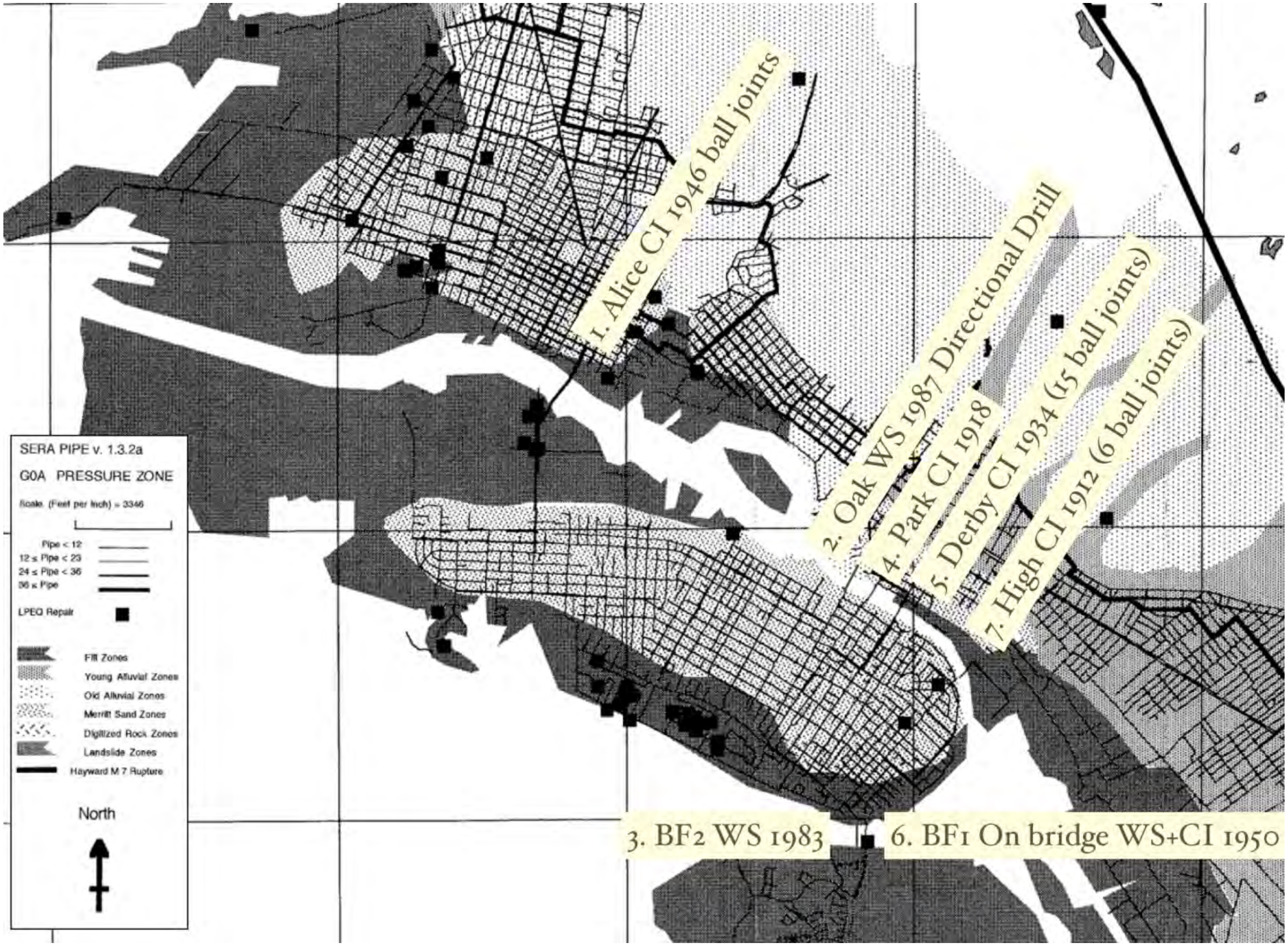


Figure 1-3. Existing Pipeline Crossings: Type of Construction and Age of Original Installation (grid lines are at 10,000-foot intervals)

By the mid-1990s, the oldest existing pipeline crossing, the 12-inch-diameter High Street pipeline built in 1912 was closed. By 2009, the Derby 20-inch-diameter pipe built in 1934 was leaking, and it too has since been closed. The District reports that one of the valves for the Bay Farm #1 pipe built in 1950 was leaking, so this pipe has also been closed. Therefore, as of 2014, four pipe crossings remain in service:

- Alice Street – 24-inch Cast Iron – built in 1946
- Oak Street (Blanding) – 24-inch Welded Steel – built in 1987
- Park Street – 16-inch Cast Iron – built in 1918
- Bay Farm #2 – 24-inch Welded Steel – built in 1983.

This Feasibility Study is intended to look at the four new pipeline crossing locations and help the District with the development of a trenchless approach and any other information to identify a preferred alignment for the crossing(s).

This report contains a large quantity of maps, figures, concepts, and tables. To simplify and eliminate duplication, we have assembled a Volume 2 for the appendices. The appendices contain most of the maps, figures, and concepts needed for the report. We envision using the Volume 2 report as a companion as you read this Volume 1 report. Through this report, all references to "Figure" with a **L**etter-**N**umber designation refer to the figures/pages within Volume 2. A general summary of the appendices in Volume 2 are:

- Appendix A: Existing Pipeline Information
- Appendix B: Geotechnical Information
- Appendix C: Historical Summary and Figures
- Appendix D: Microtunneling Concepts
- Appendix E: Horizontal Directional Drilling Concepts
- Appendix F: Proposed Crossing Plans and Profiles
- Appendix G: Risk Register
- Appendix H: Cost Estimates
- Appendix I: Photographs from the May 28, 2013 Site Visit

2 Criteria and Assumptions for Proposed Pipeline Crossings

This section summarizes the design criteria and assumptions for the construction feasibility analyses of trenchless methods at the four alternative project crossing locations of the Underwater Pipeline between Oakland and Alameda Island. Design criteria and assumptions are based on data collected to-date.

2.1 Design Criteria

- The new pipeline crossings are to survive the Maximum Considered Earthquake (MCE) and remain functional after a seismic event.
- The new pipeline crossing shall provide a minimum 24-inch ID pipe to bring flows between Oakland and Alameda Island.
- The new pipeline crossing shall provide a minimum 24-inch ID pipe to bring flows between Alameda Island and North Bay Farm Island.
- The depth of the pipeline crossings beneath the shipping channel will be placed in non-liquefiable soils and at an elevation 20 feet below any known channel dredging requirements.
- Corrosion mitigation methods will be implemented to protect the pipeline installed as part of the crossings.

2.2 Assumptions

1. The Maximum Considered Earthquake (MCE) for the project will include any one of the following scenarios (see Figure B-17 for project area and fault locations):
   - Hayward M 7.0, on any segment of the Hayward fault (also accounts for a more distant San Andreas M 8.0 event)
   - Hayward M 6.0, on the nearby northern segment of the Hayward fault
   - Calaveras M 6.75, on the northern segment of the Calaveras fault
   - Concord M 6.5, on the Concord fault
2. Enough flow can reach the crossing locations through the existing pipeline network.
3. The pipelines approaching the crossings will be upgraded to help maintain water service after the earthquake.
4. The pipelines approaching the crossing locations if damaged will be repaired after the earthquake.
5. Valves will be included at both ends of each new pipeline crossing to isolate the crossing from the pipelines in Oakland, on Alameda Island, and on North Bay Farm Island if needed after an earthquake.
6. Geotechnical investigations undertaken during future designs will make an effort to identify man-made buried objects.
7. The depth of future dredging is set at El. -53; i.e., 50 feet of draft below lowest low water level.
8. The use of steel pipe is acceptable.
9. The use of high density polyethylene (HDPE) pipe is acceptable.
10. If HDPE is exposed to contaminated ground or groundwater, the pipe will be housed in a casing.

11. Groundwater levels in the lower flatlands of Oakland, Alameda Island, and North Bay Farm Island are controlled by the San Francisco Bay and will have tidal fluctuations.
12. Ground shaking in the project area during future earthquakes in the region will be strong enough to cause liquefaction and related effects (e.g., lateral spreading) in relatively shallow depths (less than 50 feet), in loose saturated silts and sands including Young Bay Mud and some artificial fills.

3 Existing Conditions for the Existing and Proposed Pipeline Crossings

3.1 Existing Pipeline Crossings

The District has constructed seven pipeline crossings to Alameda Island over the last approximately 100 years. See Figure A-1 for the specific locations of the crossings. Five go from the Oakland to Alameda Island, and two go from the Alameda Island to North Bay Farm Island. The seven existing pipeline crossings are:

1. Alice Street Channel Crossing (Active)
2. Oak Street (Blanding) Channel Crossing (Active)
3. Bay Farm #2 Crossing (Active)
4. Park Street Crossing (Active)
5. Broadway to Derby Avenue (Derby) (Closed)
6. Bay Farm #1 Island Bridge (Closed)
7. High Street (Closed)

3.1.1 Alice Street Channel Crossing

The Alice Street estuary crossing is the primary main water supply line serving Alameda Island from Oakland. It is located on the West end of Alameda Island, and crosses between Alice Street on the Oakland side to Mariner Square Drive (Webster Street) on the Alameda Island side. It is about 300 feet south of the Posey Tube automobile tunnel connecting Oakland and Alameda Island. The District's maps and plan and profile for the Alice Street Channel Crossing are provided as Figures A-2 through A-6.

The water main is just south of the three Alameda Island sewer interceptors. One of the sewer interceptors was built in 2000 using horizontal directional drilling. A copy of the technical paper on the sewer interceptor construction written by Staheli, et.al (2001) is provided as Figures A-24 through A-28. Copies of the two drawings from the construction documents are provided as Figures A-29 and A-30.

Constructed in 1946, the nominal 24-inch-diameter mortar-lined pipeline consists of a submarine portion and onshore approaches. The submarine portion is cast iron pipe having "Usiflex" flexible ball-and-socket unit joints. The buried pipeline is located about 45 feet below mean low level water elevation. It traverses about 1,100 feet under the estuary.

On each side of the crossing, the pipeline is connected to pile-supported concrete anchor blocks. Connected to the anchor blocks are the onshore approaches consisting of bell-and-spigot cast iron pipe having lead caulked joints. The pipelines approaching the anchor blocks are buried. Figure A-4 shows an elevation view of the submarine portion with the pile-supported concrete anchor blocks.

Both approaches and the submarine pipe traverse former tidal flat areas. Figure 3-1 depicts the former tidal flat area in relation to the modern shoreline and pipeline alignment. Oakland is toward the top and

Alameda Island is toward the bottom of the image. The shaded areas indicate the former tidal flat areas. The present day shoreline is at the edge of the former tidal flats. Hence, the onshore approach pipelines traverse significant distances of these soft soils.

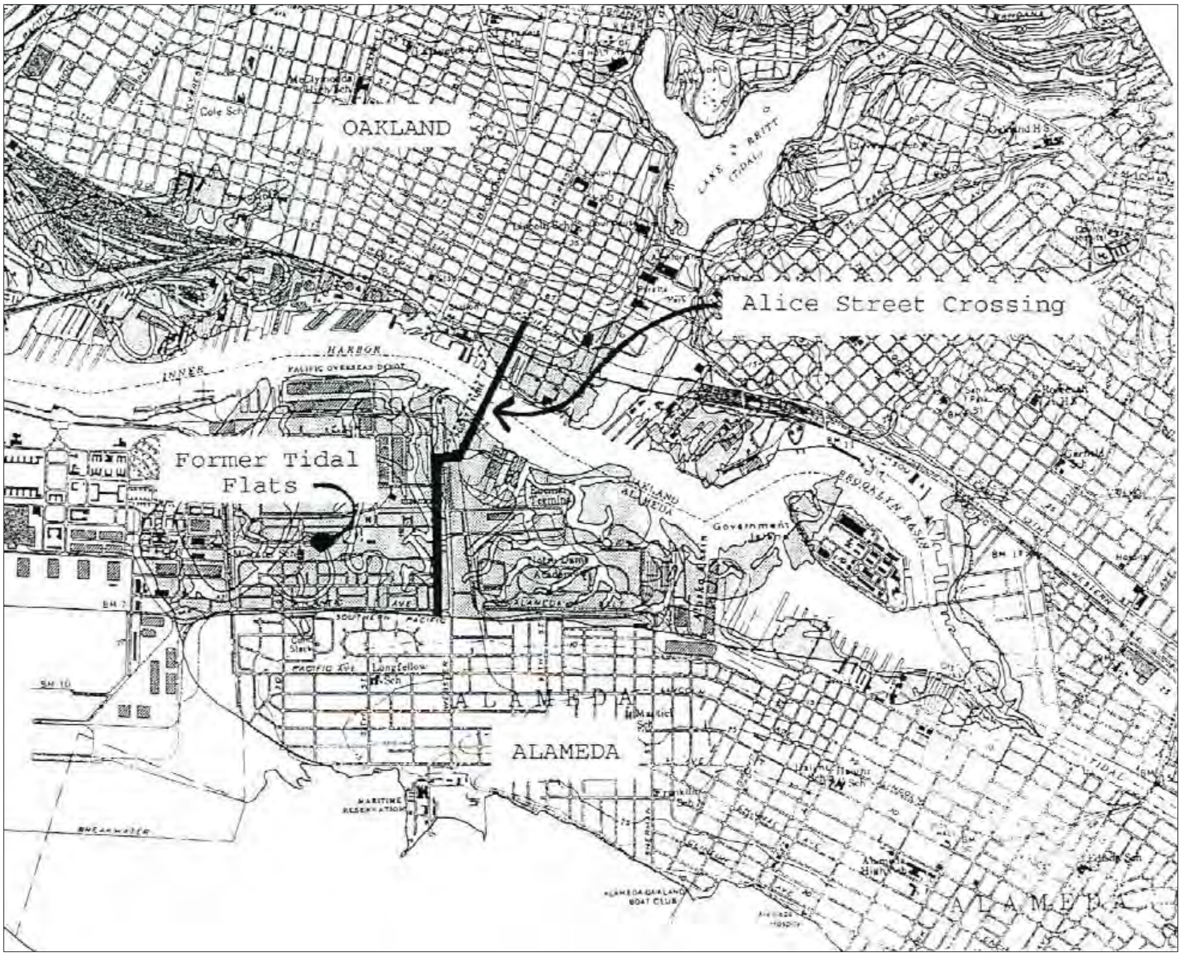


Figure 3-1. Alice Street: Plan View Showing Formal Tidal Flats Areas

3.1.2 Oak Street (Blanding) Channel Crossing

The Oak Street estuary crossing is located on the Alameda Island east end and extends from the intersection of 23rd Avenue and Ford Street on the Oakland side to the intersection of Oak Street and Blanding Avenue on the Alameda Island side. It is indicated by the pink and green lines in Figures A-7 and A-8, located about 700 feet northwest of the Park Street Bridge.

Constructed in 1987, the nominal 24-inch-diameter pipeline consists of a submarine portion and onshore approaches. The submarine portion, 823 feet long, is Koppers 300-M HB coal tar epoxy coated (EBMUD 1987 Change Order 1) and cement mortar-lined steel pipe having welded bell-and-spigot unit joints with a wall thickness of 0.25 inch. The submarine pipe run was installed under the estuary by a directionally controlled horizontal drilling procedure. At the pipeline's lowest point, it is located about 45 feet below mean low level water level. Given the style of installation, today it remains uncertain as to the condition of the exterior coating system (in other words, we do not know if the exterior coating system was damaged during installation as the pipeline was pulled into the bored hole); where damaged, the corrosive nature of surrounding soils could result in rusting of the pipe.

The approach pipelines on each side of the submarine crossing are buried cement mortar coated and mortar lined steel pipe having welded bell-and-spigot unit joints, 719 feet long total.

Figure A-9 shows an elevation view of the submarine portion. This figure (taken from a District drawing), indicates that the final bend radius was 1,500 feet; this is relatively tight for a butt welded steel pipe (bend radius usually limited to at least 100 times the pipe diameter, or 2,400 feet), but might have been accomplished using lap welded joints.

3.1.3 Bay Farm #2 Crossing

The North Bay Farm Island area is separated from Alameda Island by an estuary that connects the San Francisco and San Leandro Bays. The North Bay Farm area is supplied with water via two pipelines from Alameda Island. A third pipeline from the Oakland airport was constructed in the last decade or so, but without special seismic design features. For purposes of this report, this pipeline from the Oakland Airport is assumed to provide no drinking water to North Bay Farm Island after a large earthquake.

For purposes of this report, the pipeline on the Bay Farm Island Bridge is referred to as Bay Farm #1 Crossing (BF #1). The other pipe from Alameda Island buried under the San Leandro Channel, about 100 feet northwest of the bridge, is referred to as Bay Farm #2 Crossing (BF #2). This section presents the evaluation of the buried BF #2 channel crossing. BF #1 is currently out of service.

Located at the south end of Alameda Island, BF #2 traverses from Bridge View Isle on Alameda Island to Seal Point Court (now named Veterans Court) on North Bay Farm Island (Figures A-16 and A-17). The pipeline is indicated by the orange lines in Figures A-16 and A-17. The pipeline crossing is part of the Central Pressure Zone (G0A). The isolation valves on the 16-inch-diameter BF #1 pipeline, at either side of the Bay Farm Island Bridge, do not appear on 2009-dated B maps. We have not researched which version is correct. Presently (2014), there is a pedestrian bridge on the east side of the Bay Farm Island Bridge. This pedestrian bridge does not impact the seismic evaluations of the existing pipelines, but may factor into the installation of a new proposed pipeline crossing at Veterans Court.

The nominal 24-inch-diameter pipeline was constructed in 1983. It is a welded steel pipe having cement mortar lining and enamel coating with mortar overcoat. The pipe has a 25.75-inch OD steel cylinder, 0.281-inch wall thickness, and a 28.25-inch OD to the mortar overcoat. The pipeline is buried and located about 16 feet below mean low level water elevation. It traverses about 1,010 feet under the estuary. On each side of the pipeline crossing are pile-supported concrete pipe vault/anchors. Inside each vault the pipe has slip-type joint couplings, and on each side of the vault are pipe ball-and-socket joints.

Figures A-18 and A-19 show the profile of the submarine portion of the BF#2 pipeline crossing. The horizontal scale is 10 times the vertical scale indicating that the pipeline run is very flat with about 1:13 slopes. The evaluation considers all of the nominal 24-inch diameter pipes shown in these figures. Starting on Alameda Island at Sta. 0+25, the pipeline runs southeast for about 100 feet, and then turns to the south, goes through a pipe vault/anchor and proceeds under the channel. On the North Bay Farm Island side at Sta. 10+21, the pipeline makes a slight turn to the southeast and runs up through a pipe vault/anchor and proceeds to Sta. 13+23.

The existing pipeline crossing is located in a former tidal flat area. The present-day shorelines extend out to the edges of the former tidal flats. Note that much of the current North Bay Farm Island area is built-up over former tidal flats. The present day shorelines were formed by sinking World War I era Navy ships, and then backfilling behind them.

3.1.4 Park Street Crossing

This pipeline crossing consists of a 16-inch-diameter cast iron pipe, with the submarine portion installed in 1918 (per B-Map) (dated 1931 per District drawing W-50, but may have been redrawn from earlier records). It is about 450 feet long. Based on available information, it appears the pipeline was constructed using cut-and-cover methods, generally having about 3 feet of soil cover over the pipe, including through the submarine location. It is presumed the pipeline is unlined. See Figures A-7 and A-10 for plan and profile (District drawing E-4202, E-18616, W-49, E-18613, E-18614, E-18615). It is indicated by the green line in Figure A-7. We do not have details as to the style of cast iron pipe, and whether (or not) any ball joints or slip joints were included in the original installation.

The profile in Figure A-10 suggests that the Alameda Island approach is underlain by loose grey sands (Boring 9). The 9 soil logs in Figure A-10 are provided in larger scale in Figures B-7, B-8, and B-9. All the submarine borings suggest there is no Young Bay Mud under the pipeline; one boring (Boring 2) shows about a 5 foot-thick layer of loose clay, sand and gravel. U.S. Geological Survey (USGS) maps suggest a Merritt sand formation.

The borings in Figures B-7, B-8, and B-9 classify the soils beneath the pipeline as "sand," but the density is not specified. Merritt sands are considered to generally have a low probability of liquefaction.

3.1.5 Broadway to Derby Avenue (Closed)

This pipeline crossing is composed of a 20-inch-diameter cast iron pipe, installed in 1934. It is about 475 feet long. The pipeline is presumed to be unlined. The Oakland and Alameda Island approach pipelines are 24-inch-diameter steel, bitumen lined and coated, installed in 1935. See Figures A-7, A-11, and A-12 for plan and profile (see Figures B-10 and B-11 for additional geotechnical information). The pipeline is indicated by the orange line in Figure A-7.

The pipeline was installed with concrete anchor blocks at both approaches, concrete anchor blocks at the bottom of both slopes in the channel, with 15 ball joints at regular intervals over the length of the submarine portion, and with four expansion joints (one on each slope, two in the submarine portion) (District drawings 379-G, W-122).

Available subsurface information includes eight borings (details in Figure B-10 from 1934). These borings show the following:

- Hole 3 – Alameda Island approach. Underlain by a layer of loose yellow sand.
- Hole 2 – Alameda side slope, edge of water. Underlain by stiff yellow clay.
- Hole 1 – Alameda side, bottom of channel. Underlain by stiff clay.
- Hole 4 – Near the center of channel. Underlain by stiff clay.
- Hole 5 – Oakland side of channel. Underlain by stiff clay.
- Hole 6 – Oakland side, bottom of channel. Underlain by stiff clay and yellow sand.
- Hole 7 – Oakland side slope, edge of water. Underlain by stiff clay.
- Hole 8 – Oakland approach. Underlain by a layer of loose yellow sand; this loose layer extends laterally to the channel.

We presume the submarine soils are firm clay, dense and not prone to liquefaction, but modern borings along the upper shorelines would be needed to confirm. At the Oakland and Alameda Island approaches, there are layers of loose sand under the near-surface pipe near the concrete anchors, and a lateral spread at the Oakland side is possible.

In June 2009, DRS Marine was retained to try to determine the location of a damaged submarine pipe section. They noted fresh water seeping up from the bottom of the channel, about mid-channel, in about 21 feet of water depth. They dredged a hole 8 feet deep without finding the pipeline, and a diver suggested something hard was a further 7 feet deep. Ultimately, the dredged hole was as much as 20 feet deep, without finding the pipeline.

3.1.6 Bay Farm Island #1 Bridge (Closed)

This 16-inch-diameter pipeline was built in 1950. The pipe is hung from the bottom of the Bay Farm Island Bridge, Figures A-20 and A-21. Figure 3-2 shows details of the pipe hanger locations for the north side of the bridge. The details for the south side bridge are similar. The pipe hung from the bridge is composed of a 16-inch OD x 3/16-inch mortar lined and coated steel pipe, on rod hangers, with Dresser-type coupling expansion joints (style 38 or similar) located between every other bridge span, along the fixed portion of the bridge (Figure 3-2). At the channel crossing, the pipe material changes to cast iron, the pipe drops down from the bridge, goes to one side of the draw bridge anchor block, and runs about 160 feet along the bottom of the channel (using Usiflex joints and fiberglass coating), and then rises up on the other side of the crossing. The total length of the bridge crossing is about 1,000 feet. (Reference: District drawings 2020G, 2021G, 2050G, 2051G, 2052G, 2053G,2054G, 2055G, 932B, 934B dated 1950.)

Figure 3-2. Bay Farm Island #1 Pipe Details (May 2013)

3.1.7 High Street (Closed)

This pipeline crossing goes from Oakland to Alameda Island. The 12-inch-diameter pipe is about 410 feet long, cast iron, presumed unlined. The plan and profile are shown in Figures A-13, A-14, and A-15.

At both the Oakland and Alameda Island sides, the pipeline is anchored in a small concrete block, and then slopes down to the underwater portion. The pipeline appears to have been constructed using conventional cut-and cover methods, assumed to have a few feet of cover (as installed). To accommodate some soil settlements, six special joints were installed along the length of the pipeline, presumed to be a type of ball joint (drawing indicate special joints type E-1284, E-1285 C488, details are unknown).

Figures B-13 and B-14 show a drawing with soil borings along the alignment. The drawing was dated in 1934, and is presumed to be based on data developed for the original installation about 15 years earlier. There are ten borings shown. These borings show the following (from Alameda Island to Oakland):

- Hole 10 – Alameda Island approach. Underlain by stiff sandy clay.
- Hole 7 – Alameda Island side slope, edge of water. Underlain by clay and sand, one layer of mud (might have been removed?).
- Hole 4 –Alameda Island side, bottom of channel. Underlain by clay and sand.
- Hole 5 – Near the center of channel. Underlain by clay and sand. Note: "could not locate pipe," suggesting the 1934 drawing was created in response to some field effort.
- Hole 6 – Near the center of channel. Underlain by clay and sand.

- Hole 8 – Near the center of channel. Underlain by clay and sand.
- Hole 3 – Near the center of channel. Underlain by clay (some stiff) and sand.
- Hole 2 – Oakland side, bottom of channel. Underlain by soft mud, under which is clay and yellow sand. It does not make sense that the soft mud layer was left in place, and the boring data might have reflected the preconstruction situation, not the as-built situation.
- Hole 1 – Oakland side slope, edge of water. Underlain by soft mud, under which is clay and yellow sand. It does not make sense that the soft mud layer was left in place, and the boring data might have reflected the preconstruction situation, not the as-built situation.
- Hole 9 – Oakland approach. Underlain by soft mud, under which is clay and yellow sand. It does not make sense that the soft mud layer was left in place, and the boring data might have reflected the preconstruction situation, not the as-built situation.

3.2 Proposed Pipeline Crossings

This section summarizes conditions for the new pipeline crossings at Alternative 1A, 1D, 2A, and 3A locations (Figures 1-1 and F-1). Information provided in this section is based on the (1) District-Jacobs Associates-G&E Engineering Systems' site visit on May 28, 2013, (2) background information provided by the District, and (3) references cited herein. Copies of the photographs taken during the May 28, 2013 site visit are provided in Appendix I.

All cited elevations (El.) are approximate and in feet.

3.2.1 Alignment 1A

The location of Alternative 1A pipeline crossing is in the general vicinity of (1) the District's 24-inch-diameter Alice-Webster water pipeline constructed in 1946, and (2) the District's Third Alameda Interceptor Siphon constructed in 2000. Plans for the Alice-Webster project show the mean lower low water level as El. 0 (assumed USGS datum) and the channel bottom as El. -35. Plan and profile for the Third Alameda Interceptor Siphon show the mean lower low water level as El. -3 (assumed City of Alameda datum) and the channel bottom as El. -40 (see Figures A-29 and A-30).

3.2.1.1 Historic Conditions

Historic topographic maps and aerial photographs were assembled as Figure C-2 and illustrate the following urban development at Alternative 1A:

- In 1856 Alternative 1A consisted of a tidal bay and tidal flats.
- By 1915 the tidal flats were reclaimed and define the Oakland and Alameda Island shorelines.
- By 1950 dry docks and railroad loading wharfs were constructed along the Alameda Island shoreline, and railroad loading wharfs and channel pilings were constructed along the Oakland shoreline.

Plans for the 1946 Alice-Webster project show the following:

- A pipeline invert beneath the channel of El. -48.
- Concrete pilings with tip El. -22 for support of pipeline at both onshore crossing ends.
- A piling tip El. -30 for a former wharf located on the order of 50 feet west of the Oakland shoreline.
- A pier head/bulkhead line of unknown depth about 60 feet west of the Oakland shoreline and 150 east of the Alameda Island shoreline.
- Piles to El. -40 about 200 feet east of the Alameda Island shoreline, for the Santa Cruz Portland Cement Company Wharf.
- An Oakland shoreline seawall pile bent tips on the order of El. -22.

Plans for the 2000 Third Alameda Interceptor Siphon (Figures A-29 and A-30) show the following:

- 36-inch outside diameter HDPE sanitary sewer pipeline installed by horizontal directional drilling (HDD) with a channel crossing depth of El. -70, and within a couple hundred feet north of the Alice-Webster water pipeline.
- Telephone, 12-inch-diameter gas, 30-inch-diameter, and 48-inch-diameter siphon channel crossings, probably shallow but of unknown depth and timing located within a couple of hundred feet north of the Alice-Webster water pipeline.

Notes and observations made during the May 28, 2013 site visit were:

- Foundations of unknown size, depth, and type for concrete silos for the Santa Cruz Portland Cement Company near the west end of Alternative 1A on the Alameda Island side.
- Piles of unknown depths for a sea wall and for house-boat moorings on the Alameda Island side.
- Piles or other sheeting to develop a sea wall on the Oakland side.

3.2.1.2 Soil Conditions

Alternative 1A is underlain by several hundred feet of soil over bedrock. Soils at Alternative 1A that are particularly subject to liquefaction consist of saturated loose sands and soft silts within artificial fills and within Young Bay Mud.

- The Young Bay Mud near Alternative 1A is mapped to vary in thickness from 0 to as much as 80 feet thick (see Figure B-20).
- A geologic profile constructed near Alternative 1A from test borings drilled for design of the Third Alameda Interceptor Siphon project shows Young Bay Mud extending to El. -50 beneath the channel and isolated sands to El. -70 beneath the channel.
- The profile indicates that Young Bay Mud thickens and deepens to at least El. -80 towards the southwest.
- The lateral extent of highly liquefiable soils at Alternative 1A extends 1,000 feet northeast of the Oakland shoreline to Second Street, and 3,500 feet southwest of the Alameda Island shoreline to Eagle Avenue (see Figure B-19).

Based on the information identified above and from geotechnical information for the existing Alice Street Channel crossing, we have developed a generalized geotechnical profile for Alternative 1A as shown on Figure B-22.

3.2.1.3 Surface Conditions

Alternative 1A crosses the Alameda Channel from a cul-de-sac in Alice Street in Oakland to a private parking lot for residents of houseboats at the Barnhill Marina on Alameda Island (see Figure F-2). Visible surface conditions along Alternative 1A are illustrated in photographs provided on Figures I-1 and I-2.

Surface conditions visible near the alignment from the Oakland side toward the channel include a multi-story residential structure (The Landing at Jack London Square), a landscaped turn around median in Alice Street, asphalt pavements with subsurface utility covers, concrete curbing and walkways, San Francisco Bay Trail, trail signage, rip-rap shoreline, and storm drain outlets.

Surface conditions visible near the alignment from the Alameda Island side toward the channel include historic cement silos, asphalt parking area pavement with subsurface utility covers, pier-retained concrete shoreline paths, signage, rip-rap shoreline, and houseboats attached to pier-supported docks.

3.2.2 Alignment 1D

The location of Alternative 1D is in the general vicinity of (1) the District's 24-inch Alice-Webster water pipeline constructed in 1946, (2) the District's Third Alameda Interceptor Siphon constructed in 2000, and (3) Alameda Municipal Power's planned 2013 Alameda to Coast Guard Island HDPE Conduit Crossing. Plans for the Alice-Webster project show the mean lower low water level as El. 0 (assumed USGS datum) and the channel bottom as El. -35. Plans for the Third Alameda Interceptor Siphon show the mean lower low water level as El. -3 (assumed City of Alameda datum) and the channel bottom as El. -40 (see Figures A-29 and A-30). Plans for the Alameda to Coast Guard Island HDPE Conduit Crossing show a channel bottom of approximately El. -37 (unknown datum) (see Figures A-31, A-32, and A-33).

3.2.2.1 Historic Conditions

Historic topographic maps and aerial photographs were assembled in Figure C-2 and illustrate the following urban development at Alternative 1D:

- In 1856 Alternative 1D consisted of a tidal bay and tidal flats.
- By 1915 the tidal flats were reclaimed and define the Oakland and Alameda Island shorelines.
- By 1950 dry docks and railroad loading wharfs were constructed along the Alameda Island shoreline, and railroad loading wharfs and channel pilings were constructed along the Oakland shoreline.
- By 1993 some of the railroad loading wharfs and channel pilings were replaced by boat marinas and moorings.

Plans for the 1946 Alice-Webster project show the following:

- A pipeline invert beneath the channel of El. -48.
- A piling tip El. -30 for a former wharf located on the order of 50 feet west of the Oakland shoreline.
- A pier head/bulkhead line of unknown depth about 60 feet west of the Oakland shoreline and 150 east of the Alameda Island shoreline.
- Piles to El. -40 about 200 feet east of the Alameda Island shoreline, for the Santa Cruz Portland Cement Company Wharf.
- Oakland shoreline seawall pile bent tips on the order of El. -22.

Plans for the 2000 Third Alameda Interceptor Siphon (Figures A-29 and A-30) show the following:

- 36-inch outside diameter HDPE sanitary sewer pipeline installed by horizontal directional drilling (HDD) with a channel crossing depth of El. -70, and within a couple hundred feet north of the Alice-Webster water pipeline.
- Telephone, 12-inch-diameter gas, 30-inch-diameter, and 48-inch-diameter siphon channel crossings, probably shallow but of unknown depth and timing located within a couple of hundred feet north of the Alice-Webster water pipeline.

The 2013 plans for Alameda Municipal Power’s Alameda to Coast Guard Island HDPE Conduit Crossing (Figures A-31, A-32, and A-33) show the following:

- Bulkheads and wharf piles along the western shore of Coast Guard Island to El. -34, El. -47, and El. -75.
- A conduit crossing invert of El. -100.

Notes and observations made during the site visit were:

- Foundations of unknown size, depth, and type for the old dry docks near the west end of Alternative 1D on the Alameda Island side.
- An inscription on a concrete pile for the boat marina on the Alameda Island side dated May 9, 1985 that indicates a pile depth of 54 feet.
- Sheet piling of unknown depths for a sea wall on the Alameda Island side.
- Piles or other sheeting to develop a sea wall on the Oakland side.

3.2.2.2 Soil Conditions

Alternative 1D is underlain by several hundred feet of soil over bedrock. Soils at Alternative 1D that are particularly subject to liquefaction consist of saturated loose sands and soft silts within artificial fills and in Young Bay Mud.

- The Young Bay Mud near Alternative 1D is mapped to vary in thickness from 0 to as much as 80 feet thick.

- A geologic profile constructed at Alternative 1A (a few hundred feet northwest of Alternative 1D) from test borings drilled for design of the Third Alameda Interceptor Siphon project shows Young Bay Mud extending to El. -50 beneath the channel and isolated sands to El. -70 beneath the channel (see Figure B-23).
- The profile indicates that Young Bay Mud thickens and deepens to at least El. -80 towards the southwest.
- The lateral extent of highly liquefiable soils at Alternative 1D extends several thousand feet northeast of the Oakland shoreline and southwest of the Alameda Island shoreline (see Figure B-19).

Based on the information identified above and from geotechnical information for the existing Alice Street Channel crossing, we have developed a generalized geotechnical profile for Alternative 1D as shown on Figure B-23.

3.2.2.3 Surface Conditions

Alternative 1D crosses the Alameda Channel from a gated parking lot at Estuary Park in Oakland to a private parking lot between 1080 and 1100 Marina Village on Alameda Island (see Figure F-9). Visible surface conditions along Alternative 1D are illustrated in photographs provided on Figures I-3 and I-4.

Surface conditions visible near the alignment from the Oakland side toward the channel include adjacent multi-story residential structure (Portabello Development), asphalt pavements, a monitoring well cover, open-space with concrete pads and benches, San Francisco Bay Trail, signage, rip-rap shoreline, and storm drain outlets.

The open-space known as Estuary Park and land to the southeast is to become a part of a new waterfront mixed-use redevelopment area known as the Brooklyn Basin. Search of the Brooklyn Basin web site indicates the Oakland side of Alignment 1D will include:

- 65-acre environmentally-sustainable mixed-use urban master plan
- Waterfront location adjacent to Jack London Square
- 3,100 residential units
- 200,000 square feet of retail and commercial space
- 30 acres of parks, public trails, and open space plus new marinas and renewed wetlands

The developer of the property is the Oakland Harbor Partners and Zarsion Holdings Group, Beijing: Signature Development Group. The web site provides no information on when development of the property will begin. Figure 3-3 is an artist rendering of the proposed property development. Based on the rendering, the entry point for Alignment 1D will be located in a proposed tree-lined cul-de-sac and the pipeline will extend along a paved area to Embarcadero East, the main road adjacent to Interstate 880.

Figure 3-3. Artist rendering of the proposed Brooklyn Basin Development

Surface conditions visible near the alignment from the Alameda Island side toward the channel include an adjacent multi-story structure (Telecare Corporation), asphalt parking area walkways and pavements, trees and lawn landscaping, rip-rap shoreline, sheet-pile retained sea walls, and pier-supported boat docks.

3.2.3 Alignment 2A

The location of Alternative 2A is in the general vicinity of (1) the District's Estuary to Bay Farm Island #1 water pipeline project that was constructed and attached to the existing bridge in the early 1950s, and (2) the Bay Farm #2 water pipeline project (also referred to as the San Leandro Channel Utilities Crossing project) that was constructed in 1983. Plans for the Estuary to Bay Farm Island project show the mean lower low water at El. 0 (USGS datum) and the deepest portion of the channel bottom as El. -10. Plans for the Bay Farm #2 project show the mean lower low water level as El. -3.1 (City of Alameda datum), and the channel bottom as El. -11.59 (USGS datum = City of Alameda datum +3.41).

3.2.3.1 Historic Conditions

Historic topographic maps and aerial photographs were assembled in Figure C-3 and illustrate the following urban development at Alternative 2A:

- From 1897 through 1950 a bridge occupied the area of Alternative 2A that was formerly a tidal bay (San Leandro Bay) and tidal flats.

- Sometime after 1950 and before 1959, a new parallel bridge (Bay Farm Island Bridge) connecting Otis Drive to the north and Doolittle Drive to the south) was constructed to the east of the first bridge. The first, older bridge was removed. The working assumption is the bridge foundations were left in place and are potential buried objects for any new pipeline crossing alignment.
- Ten WWI-era Navy destroyers (ships) were purchased as scrap in the 1920s or 1930s and placed as breakwaters, end-to-end along the northwest end of Bay Farm Island. These ships can be observed on air photographs taken prior to 1966, after which hydraulic filling to reclaim tidelands around Bay Farm Island concealed them. The class of destroyer believed to be buried is 314 feet long, 30.5 feet wide, with a hull depth of 18 feet (12 feet of draft and 6 feet of freeboard). The hull was made of ¼-inch steel. One of the ships may be visible in the 1950 photo, west of the old bridge.

Plans for the Estuary to Bay Farm Island project show District piles designed to penetrate to El. -85. Plans for the 1983 Bay Farm #2 project show the following:

- A pipeline invert beneath the channel of El. -21.
- End vaults and anchors supported on piles of unknown depth.
- A possible future channel bottom at El. -17.1.
- A seawall replacement on the Bay Farm side to El. -3.1.
- PT&T conduits and a de-energized and abandoned 12KV submarine cable east of the Bay Farm #2 pipeline.
- Sheet piling of unknown depth and a fishing pier north of Maitland Drive (also referred to as Seal Point Court and Veterans Court) along the North Bay Farm Island shoreline.
- Wood and concrete headwalls along the Alameda Island side shoreline.

3.2.3.2 Soil Conditions

Alternative 2A is underlain by a several hundred foot thick sequence of soil over bedrock. Soils at Alternative 2A that are particularly subject to liquefaction consist of saturated loose sands and soft silts within artificial fills and in Young Bay Mud.

- Estuary to Bay Farm Island project plans shows the deepest depth to stiff blue clay below the channel at El. -62.
- Logs of 6 borings for the Bay Farm Island #2 project (see Figure B-16), unidentified as to their geographic location, indicate that the bottom of Young Bay Mud varies from El. -21 to El. -52.
- A schematic soil profile is provided in Figure B-24, based on interpolation of two CPT logs by the USGS. The bottom of Young Bay Mud approaches El. -10 at the CPT on the Alameda Island side, and El. -50 at the CPT on the North Bay Farm Island side.
- The lateral extent of highly liquefiable soils extends over a thousand feet north of the Alameda Island shoreline and several thousands of feet south of the Alameda Island shoreline (see Figure B-19).

Based on the information identified above and from geotechnical information for the existing Bay Farm #2 crossing, we have developed a generalized geotechnical profile for Alternative 2A as shown on Figure B-24.

3.2.3.3 Surface Conditions

Alternative 2A microtunneling crosses the Alameda Channel from Veterans Court on North Bay Farm Island to Bridge View Isle on Alameda Island (see Figure F-11). Alternative 2A HDD crosses the Alameda Channel from Veterans Court on North Bay Farm Island to Towata Park (just beyond Bridge View Isle) on Alameda Island (see Figure F-12). Visible surface conditions along Alternative 2A are illustrated in photographs provided on Figures I-5 and I-6.

Surface conditions visible near the alignment from the North Bay Farm Island side toward the channel include asphalt pavements with subsurface utility covers, concrete curb and walking paths, trees and landscaping, adjacent tennis courts, pier-supported pedestrian walkways, signage, and rip-rap shoreline.

Surface conditions visible near the alignment from the Alameda Island side toward the channel include trees and landscaping (Towata Park), adjacent homeowner residences, concrete curbs, asphalt pavement with subsurface utility covers, open space, signage, and rip-rap shoreline.

3.2.4 Alignment 3A

The location of Alternative 3A is in the general vicinity of the District's Derby Avenue water pipeline project that was constructed across the channel in 1934. Plans for that project show the mean lower low water at El. 0 (assumed USGS datum), a channel bottom at El. -21.1, official channel bottom at El. -25, and a proposed channel bottom at El. -30.

3.2.4.1 Historic Conditions

Historic topographic maps and aerial photographs were assembled in Figure C-4 and illustrate the following urban development at Alternative 3A:

- In 1897 the area of Alternative 3A was entirely in dry ground above high tide elevation.
- By 1915 a tidal channel was constructed through the area of Alternative 3A to interconnect San Leandro Bay to the southeast with San Antonio Creek to the northwest. The tidal channel separated the Oakland side to the northeast from the Alameda Island side to the southwest.
- Since 1915 boat moorings and wharfs were constructed along both the Alameda Island and Oakland sides.
- In 1950 above-ground tanks existed near Alternative 3A on the Oakland side, and lumber storage existed on the Alameda Island side.
- In 2009 a 25-foot-long by 15-foot-wide hole was dredged to 20 feet deep within the channel in an unsuccessful attempt to locate a damaged section of the Derby Avenue pipeline.

Plans for the 1934 Derby project show the following:

- A pipeline invert beneath the channel of El. -39.
- A Great Western Power Company cable of unknown depth a few tens of feet to the southeast of the water pipeline.
- A Loop Lumber Company Wharf of unknown support type and depth on the Alameda Island side.
- Eight unknown "ARK" (ship moorings?) on the Oakland side.
- Pierhead/bulkhead lines of unknown depths on both the Alameda Island and Oakland sides.
- A Shell Oil Company Building on the Oakland side.
- Wood box culverts emptying into the channel on the Alameda Island side.

Based on the information identified above and from geotechnical information for the existing Derby Avenue crossing, we have developed a generalized geotechnical profile for Alternative 3A as shown on Figure B-25.

3.2.4.2 Soil Conditions

Alternative 3A is underlain by a several hundred foot thick sequence of soil over bedrock, and no Young Bay Muds. Logs of the 1987 Blanding Project (see Figures B-5 and B-6) indicate a generalized profile of silty sand (SM) over low plastic clay (CL). The blow counts on Figure B-6 show dense to very dense (blow count 30 to 50+) relative density of the silty sands and a very stiff consistency (blow count 15 to 30) of the low plasticity clays. Logs of 9 borings for the Park Street project (see Figures B-7, B-8, and B-9) show inter-bedded layers of yellow sandy clay (mostly sand), yellow clay and stiff gray clay to a depth of 56 feet below Mean Sea Level. Two of the Park Street project borings on the Alameda Island side show a "loose" layer of sand, clay sand and gravel at a depth of El. -10 to -15 feet. Logs of 8 borings for the 1934 Derby Avenue project (see Figures B-10 and B-11) indicate a "loose" sand layer between El. 0 and -10 that is shown to be generally overlain and underlain by stiff clays. No blow counts or lab data was provided to support the Park Street or Derby Avenue projects to support the "loose" description. Soils at and surrounding Alternative 3A are reported by the USGS to have a relatively low likelihood to liquefy during a Magnitude 7.1 earthquake (see Figure B-19).

3.2.4.3 Surface Conditions

Alternative 3A microtunneling crosses the Alameda Channel from Derby Avenue in Oakland to the parking lot of the Nob Hill Foods Shopping Center on Alameda Island (see Figures F-14 and F-15). Alternative 3A HDD crosses the Alameda Channel from Derby Avenue in Oakland to Broadway (just beyond the parking lot for the Nob Hill Market Shopping Center) on Alameda Island (see Figure F-16). Visible surface conditions along Alternative 3A are illustrated in photographs provided in Figures I-7 and I-8.

Surface conditions visible near the alignment from the Oakland side toward the channel include asphalt pavements with subsurface utility covers in Derby Avenue, adjacent commercial structures (Oakland Museum Women's Board/White Elephant Sale and California Rowing Club buildings), concrete pier- and sheet pile-supported concrete promenades and sea walls and steps, and pile-supported boat docks.

Structures and tanks of a petroleum terminal for the Shell Oil Company were formerly visible west of Derby Avenue, between the Alameda Channel and Glascock Street, dating back to at least 1934. Remedial measures performed by a developer to remove petroleum-impacted soil from the property in the 2003-2004 included excavation and off hauling to depths up to 10 to 13 feet below the original ground surface. For purposes of shoreline stability, contaminated soil that was allowed to remain was covered by a protective high density polyethylene (HDPE) geomembrane liner and rip-rap cap. It does not appear that soils were removed east of the property, including from beneath Derby Avenue.

In 2009, a petroleum seep was observed along the margin of the geomembrance cap at the shoreline intertidal zone. The seep was believed to be due to either a localized tear in or flowing around the end of the geomembrane. To mitigate the seep, the existing geomembrane was left in place and a new section of geomembrane was installed to encompass the seep. The vertical and horizontal extent of contaminated ground beyond the former Shell Oil Company property is not known to us at this time. The contaminated ground could exist beneath Derby Avenue (URS, November 30, 2012, Seep Investigation Technical Report, The Estuary, 2901-2999 Glascock Street, Oakland, California). For this reason the jacking shaft for the microtunneled pipeline crossing (short and long options) and the entry pit for the HDD pipeline crossing were moved about 300 feet to the northeast side of Glascock Street on Derby Avenue.

For the HDD pipeline crossing, HDPE is the preferred carrier pipe material. A check of published sources does show that contaminated groundwater, especially BTEX [benzene, toluene, ethylbenzene, xylenes] will permeate into HDPE pipe. Once contaminated, the HDPE pipe cannot be “cleaned”.

Literature suggests that permeation of organic chemicals and hydrocarbons through polyethylene pipe is possible, while actual cases of soil contaminated hydrocarbon permeation are extremely rare. Hydrocarbons do not degrade polyethylene but can diffuse through the wall of the pipe in areas of gross contamination. The exterior contact may affect sidewall fusions and or butt fusions; thus, after polyethylene pipes have been exposed to grossly contaminated soils, mechanical connections may be preferred. There are several ways to address gross hydrocarbon contamination of soil surrounding the pipe including removal and replacement of the contaminated soil with good clean soil of Class I or Class II materials, placing the carrier pipe inside a casing, and rerouting the pipeline around the contaminated area.

Surface conditions visible near the alignment from the Alameda Island side toward the channel include adjacent residential and small business structures, asphalt pavements (Broadway, Blanding Avenue, and parking lot for Nob Hill Foods Shopping Center), trees, and landscaping, adjacent structures (e.g., Nob Hill Foods), concrete curbs and walkways and flatwork, storm drain inlets and outlets, retaining walls with tie-back features, and concrete pile-supported boat docks.

4 Feasible Construction Methods and Materials for Proposed Pipelines

A number of key factors were considered in evaluating feasible trenchless construction methods for the different crossings. These factors include:

- Length of installation
- Size of pipe
- The potential conflict of existing utilities and structures
- The elevation and special site constraints
- Anticipated subsurface soil and groundwater conditions along the alignment
- Separation clearance at existing utility/water body crossings
- Accuracy of installation
- Available construction staging areas
- Construction costs

Although different trenchless methods exist for installation of pipe, crossing constraints, groundwater levels, and ground conditions rule out most of the trenchless methods leaving microtunneling and horizontal directional drilling (HDD) as the two most viable trenchless methods for the crossings. Both methods satisfy crossing constraints and work well below the groundwater table and in the ground conditions present. A discussion of these two methods and their applicability at the project crossings are discussed below.

4.1 Microtunneling Overview

Microtunneling is a pipe jacking process that simultaneously excavates the ground with a microtunneling boring machine (MTBM), counterbalances groundwater pressure with an engineered drilling fluid, removes the excavated spoil via the slurry/drilling fluid, and advances pipe segments to support the excavation. The MTBM is remotely controlled, laser guided, and steerable. The carrier pipe (or casing) is installed behind the machine in a pipe string to transfer jacking forces to simultaneously jack pipe and advance the machine into the ground. Excavation is carried out by the MTBM in front of the lead pipe section. The MTBM and transport slurry exert continuous and controllable pressure at the face of the excavation to support the ground at the same time the slurry/drilling fluid counterbalances the groundwater pressures. A typical MTBM and pipe installation operation are shown in Figures 4-1 and 4-2.

Excavated material is removed from a chamber behind the cutter wheel of the machine at a rate that is synchronized with the advance rate of the machine. These materials are typically transported back to the jacking shaft in slurry suspension. The drilling fluid/slurry not only transports the material, but it also counterbalances the hydrostatic pressures at the heading. The excavated materials are then separated from the slurry at the separation plant. The spoils, together with some residual drilling fluid, are hauled away from the site for disposal while the bulk of the drilling fluid is recycled back into the tunneling operation and pumped to the face of the MTBM. A typical microtunnel slurry separation plant layout is shown in Figure 4-3.

Figure 4-1. Typical Microtunnel Boring Machine (MTBM)

Figure 4-2. Placing Pipe for Jacking during Microtunneling Operation

Figure 4-3. Typical Slurry Separation Plant

Microtunneling machines are equipped with a sophisticated guidance system that utilizes a laser beam to establish a fixed reference to the design line-and-grade. The laser is independently supported in the jacking shaft with the beam set to the design line and grade. The laser beam is aimed at a target located in the rear of the MTBM. The operator is located in a surface control room and provided with a digital and/or closed circuit display of the laser beam's position on the target. The operator uses this information to make steering corrections to maintain the laser beam on the target. The recommended tolerance for grade is 3 percent of the diameter of the MTBM or 1 inch, whichever is greater. For our project, we are looking at installing 48-inch diameter steel casings. At 3 percent, the microtunneled installation will be ±1.5 inches. Since the water mains are pressure systems, the need for a tight grade tolerance is not needed.

A pipeline installed by microtunneling is constructed in a series of drives from a jacking shaft to a receiving shaft. The drive length (or distance from the jacking shaft to the receiving shaft) for microtunneling methods typically range from a few hundred feet to over 1,500 feet. The ultimate drive length is a function of the pipe diameter and pipe materials, machine capabilities, and ground conditions. For diameters greater than 36 inches, intermediate jacking stations (IJSs) can be installed in the pipe string to extend drive lengths. Figure 4-4 shows a typical IJS. With the addition of IJSs to increase the overall system jacking capacity, drive lengths can be increased to well over 1,000 feet.

The size and configuration of a jacking shaft vary with the specific requirements of a project and with the type of equipment used. For the ground conditions anticipated at the Alameda – North Bay Farm Island pipeline crossing locations, we anticipate that circular water tight shafts utilizing secant piles or cutter soil mixed panels will be used to capitalize on the efficiency of circular hoop stress design. The diameter of a circular jacking shaft is generally a function of the casing or pipe length being installed. We have assumed that 20-foot long casing segments would be used thus requiring a jacking shaft approximately 28 feet in diameter. See Figure D-1 for typical jacking shaft configuration. The receiving shaft only needs to be large enough to remove the MTBM. Removal of the MTBM can generally be accomplished inside an 18-foot-diameter shaft (see Figure D-2).

Figure 4-4. Intermediate Jacking Station (IJS)

4.2 Typical Microtunnel Crossing Approach

A typical microtunnel crossing approach will consist of jacking shaft from which the MTBM and pipe are advanced and a receiving shaft for retrieval of the MTBM. For the pipeline crossing evaluations, we assumed the following conditions for the shafts and trenchless crossings:

- Assume the use of secant piles for a circular jacking shaft with a working diameter of 28 feet for 20-foot-long pipe segments (see Figure D-1 for typical jacking shaft).
- Secant pile shafts are watertight and a circular configuration is selected for its efficiency in resisting ground forces in ring compression thereby eliminating internal struts, braces or walers.
- H-beams would be placed in selected secant piles to resist jacking forces.
- Assume the use of secant piles for a circular receiving shaft with a working diameter of 18 feet (see Figure D-2 for a typical receiving shaft).

- The use of the receiving shaft is typically limited to retrieval of the MTBM. During non-use times, the receiving shaft can be plated over to maintain local traffic.
- The shaft depths would be selected deep enough to place the microtunneled pipeline crossing below the fills and Young Bay Mud and into the deep stable soils not prone to liquefaction.
- Microtunneling would install a 48-inch-diameter steel casing to obtain the distance needed for the crossings (see Figure D-3).
- The 24- or 36-inch diameter carrier pipe could be either (1) HDPE pipe or (2) steel pipe with a coating and lining.
- The carrier pipe would be grouted inside the steel casing (see Figure D-3).

In an effort to provide additional flexibility to the District, the carrier pipe could be inserted into the casing on spacers without grouting the annular space. In this scenario, if the carrier pipe is damaged during a seismic event and the casing survives, the entire damaged carrier pipe could be removed from the casing as a post-earthquake repair and a new carrier pipe installed. Another scenario for the District to consider is inserting a large diameter carrier pipe (36-inch) within the casing and grouting the carrier pipe in place. By installing a 36-inch diameter carrier pipe, if the carrier pipe is damaged during a seismic event, the 36-inch ID carrier pipe could be slip lined with a 24-inch ID pipe to provide water flow to the Alameda Island as a post-earthquake repair.

After the tunnel is completed and the new carrier pipe is installed in the casing, there are different options available to bring the riser pipe up through the shaft to the surface connection. The riser options are presented in order of increasing robustness which corresponds to decreasing risk. The riser options are not specific to any particular pipeline crossing and can be applied at any shaft. The following identifies the options available to extend the riser pipe up through the shaft:

- Water main riser is free standing, mounted to the shaft wall with struts/bracing/tiedowns (see Figure D-4), and supported at the bottom with a concrete saddle. Shaft is left open and fully accessible for inspection/repair/replacement of the water main.
- Water main riser is encased and protected by concrete (see Figure D-5). The remaining portion of the shaft is left open for access. With this option, a T-section and bolted flange can be installed on the water main for access to the water main.
- Water main riser is encased in concrete and the remaining portion of the shaft is backfilled with low strength flowable fill or compacted structural backfill (see Figure D-6). With this option, shaft access is not available.

A variation on any of the options outlined above would be to increase the casing diameter so that a second carrier pipe can be installed in the casing for added redundancy.

The initial concept for microtunneling a channel crossing is to tunnel from a jacking shaft to a receiving shaft. If the idea is to provide a water main crossing which survives a seismic event and the pipeline needs to be within non-liquefiable soils, another option is to include a series of microtunneled drives where the water main extends from stable ground in Oakland to stable ground on Alameda Island. The total trenchless crossing would be divided into multiple microtunnel drives. The jacking shafts would be design to allowing jacking in two directions (see Figure D-7). The receiving shafts would be design to receive MTBMs in two directions (see Figure D-8).

4.3 Horizontal Directional Drilling (HDD) Overview

HDD is a three-stage construction method that originates from the surface thereby eliminating the need for shafts (Figure 4-5). The first stage consists of drilling a pilot hole with a U-shaped configuration with the apex located near the mid-point of the crossing (see Figure E-1). The second stage involves reaming and enlarging the pilot hole to the required size for pipe pullback. The third stage involves pulling the final carrier pipe into the stabilized hole. Alternately, a steel casing can be pulled into the excavated hole after which the final carrier pipe is inserted through the casing.

Figure 4-5. HDD Setup with Surface Conductor Casing

The pilot hole is excavated from a shallow, relatively small surface pit using a steerable guided drilling method that follows a prescribed path. The pilot hole starts at the ground surface with an entry angle that is generally between 8 and 20 degrees. The hole traverses a tangent followed by a large radius vertical curve to the design depth. The hole can then transition to a horizontal traverse before resuming a vertical curve and tangent back to the surface or the hole can transition directly into a vertical curve and tangent to the surface.

The radius of the vertical curve is a function of the diameter of the pipeline to be installed. A rule of thumb equation used to determine the minimum pipe bend radius of steel pipe is given by:

$$R \text{ (in feet)} = 100 \times \text{pipe diameter (in inches)}$$

When HDPE or PVC pipe is used, the constant in the equation can be changed to 50. For example, to install a 24 inch inside diameter (30 inch outside diameter) HDPE or PVC pipe, the drill path must use a vertical curve with radii greater than about 1,500 feet, or about 3,000 feet if a steel casing is installed. Curves with large radii require long run-out distances to obtain the changes in elevation needed between the starting point and the deepest point in the alignment.

There have been other projects, including the District's Alameda Siphon project (see pages A-24 to A-28) that have used vertical curves with radii tighter than the recommended 50 x D bend radius for HDPE or PVC pipe. For our project, we recommend using the industry standard of 50 x D bend radius. It reduces the tensile stress imposed on the pipe during installation and in the ground. For the proposed pipeline crossing profiles, the use of the 50 x D bend radius installs the pipe into the deeper more stable soils for a longer distance, which is what we want. It also makes the profile easier to drill with less vertical curves to deal with. When dealing with underground construction, Murphy's Law is always presence. Keeping it simple will reduce risks, decrease the chance of something going wrong, and increase the pipe's ability to survive the earthquake.

The depth is usually dictated by clearance requirements for the water body crossing including the potential for future dredging of the crossing, depth of cover necessary to prevent slurry returns to the surface during the HDD operations, or the desire to locate the alignment within a particular soil horizon. The exit angle is typically between 5 and 12 degrees because this is quite often the pipe insertion side. Steeper exit angles require higher break over points to maintain the pipe within the safe bend radius as the pipe is pulled into the angled hole.

HDD's distinguishing features include a guided and steerable drill tool used to develop a pilot bore through the ground in an inverted arc profile. The path of the drill tool is monitored by setting up a magnetic surface coil (see Figure 4-6). Spoils are typically washed back to the insertion pit at the surface through the excavated opening utilizing slurry (drilling mud). Once the pilot hole has been established, it is enlarged by attaching a reamer to the drill string and pulling the reamer back and forth (Figure 4-7). With each successive ream, the reamer size is increased to enlarge the size of the hole. Depending on the final hole size, the reaming process could take several passes. During the entire process the hole remains open by keeping the hole filled with slurry to prevent collapse of the reamed hole. The excavated opening (reamed hole) is oversized approximately 50 percent by volume (~30 percent by diameter) to accommodate the pullback of the final carrier pipe (see Figure E-2).

Oversized conductor casings are required at each end to control fluid pressures and fluid collection at each end as well as prevent hydraulic fracturing to the surface (see Figure E-3). There are number of options available regarding the casing. The easiest way to install the conduit casing is to use pipe ramming. An air hammer is attached to the open ended steel casing. The casing is set on guide rails at the desired entry angle and driven into the ground like a battered pipe pile. The soils at the surface for the different HDD alignments are relatively loose, and pipe ramming the casing for a distance of 100 feet (Figure E-3) or 200 feet (Figure E-4) should be considered. Since the longer length of casing is desirable, the casing can be rammed to refusal, and then the ground inside the casing removed with an auger. Once the casing is unloaded, the ramming of the casing can continue to get additional penetration into the ground. Using an entry angle of 15 degrees, a 200-foot long casing would reach a depth of 50 feet. This 50-foot depth is about the boundary between liquefiable and non-liquefiable soils.

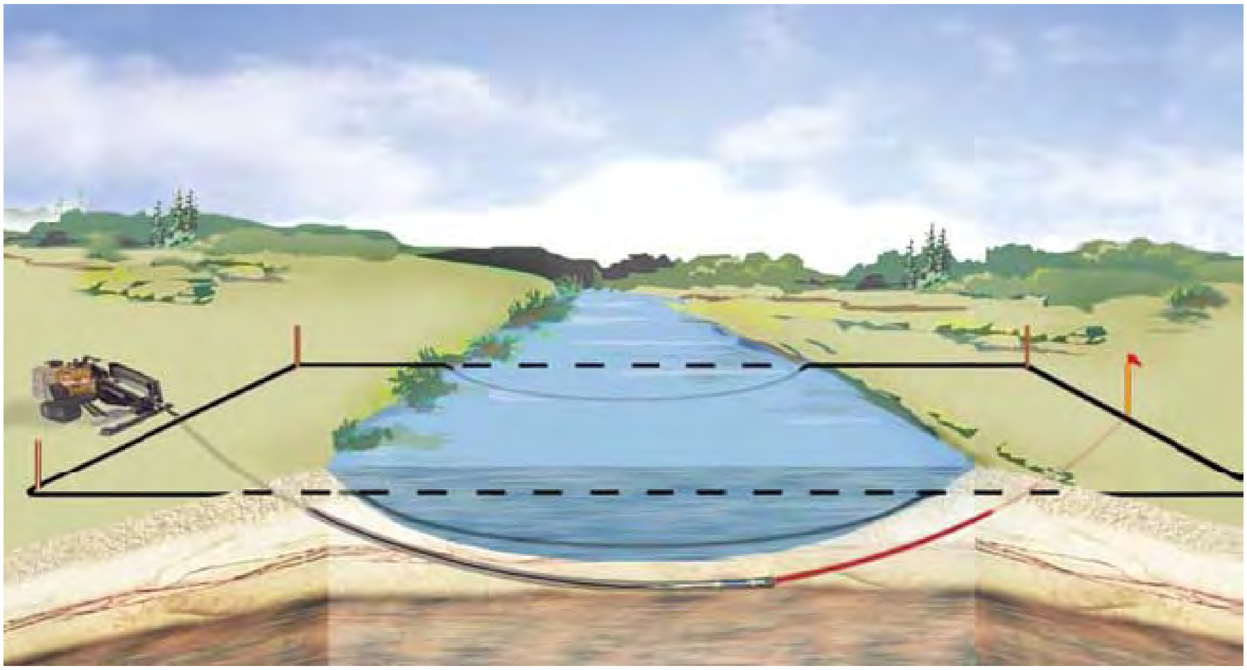


Figure 4-6. Surface Coil System to Monitor the HDD Drilled Path

Figure 4-7. HDD Reaming Tools

In an effort to develop an HDD pipeline crossing that is housed in non-liquefiable soils from surface to surface, we have developed some additional concepts where the conductor casings are supported in the liquefiable soils with jet grouted columns or cement soil mixed panels. Figure E-5 shows use of a 200-foot long casing, and then supports the casing with 8-foot diameter jet grouted columns. The jet grouting is done after the pipeline is installed in the hole. The HDD pipeline extends through the stabilized casing, into the hole drilled through the deep non-liquefiable soils, and then extends back to the surface through a stabilized casing. The use of the longer 200-foot casing with jet grouted columns as support is the recommended approach for the proposed pipeline crossing.

Figure E-6 uses a 100-foot-long casing. Jet grouted columns are spaced under the casing for support. The ground from the end of the casing to a hole depth founded in non-liquefiable soils would also be improved with jet grouted columns drilled tangent to each other. The ground improvement work would be completed before the HDD pilot hole was drilled. The concept shown on Figure E-7 is similar to the concept on Figure E-6, but the jet grouted columns are drilled after the HDD pipeline is installed. One concern with the concept shown on Figure E-7 is the potential for damaging the water main with the jet grouting process.

Figure E-8 shows one additional concept developed where cement soil mixed panels are installed from the surface to a depth of non-liquefiable soils instead of jet grouted columns. A 200-foot-long casing would then be rammed into place through the cement soil mixed panels. Since the panels would have increased compressive strengths, the ramming process would involve hammering the casing into or up to the panel, placing to auger to bore through the panel, and then continue the hammering process to the next panel.

In discussions with the District, we have also considered installing a 100-foot-long casing to control the fluid pressures at the surface during drilling and then removing the casing after the HDD pipeline is installed. With this scenario, the new water main would extend through liquefiable soils on both sides of the crossing. The use of a HDPE pipe would be needed to span through the liquefiable zone of the crossing. While potentially able to survive the seismic event, installing a pipeline founded in non-liquefiable soils and with ground improvement extending to the surface is recommended.

After the hole has been enlarged to the required size, the pipeline (final carrier pipe or steel casing) is installed in the hole in one continuous pullback operation (Figure 4-8). Pressure testing of the carrier pipe as shown in Figure 4-9 is completed before pullback is started.

The annular space (space between the excavated hole and pipe exterior) can be filled with a cement grout, but it is more common with the HDD process to leave the drilling mud as the final backfill. Post HDD installation research has demonstrated that there is an equalization of the surrounding soil with the slurry filled annular space over time.

Figure 4-8. 24-inch ID HDPE Pipe Being Pulled into the Final Hole

Figure 4-9. Pre-installation Pressure Test on Fused HDPE Pipe

4.4 Typical HDD Crossing Approach

A typical HDD crossing approach will consist of an entry and exit pit on each of side of the channel. For the crossing evaluations, we assumed the following conditions for the horizontal directionally drilled pipeline crossings:

- Assume an equal entry and exit angle varying from 10 to 15 degrees (see Figure E-1).
- Choice of the entry and exit angle will control the depth of the crossing under the channel.
- The entry and exit pits will be located at least 200 feet onshore to allow installation of the longer conductor casings and any associated ground improvement work.
- If the pilot hole hits a buried object, the drill steel will be removed from the bored hole, and new similar alignment will be drilled.
- The HDD crossing will install 24-inch ID HDPE carrier pipe with a 30-inch OD (see Figure E-2).
- The bored hole diameter will be 39 inches or larger to allow pullback of carrier pipe.
- The minimum vertical curve radius will be 1,500 feet (R = 50 x 30).
- Available space to layout the pipeline will control which side of the crossing is the exit point, and which side is the entry point.
- Sufficient easement space would be available for pipe fusion and pullback.
- The pipeline will be assembled as one long pipe string and tested before pullback (see Figures F-10, F-13, and F-17).
- The carrier pipe will sit in the bored hole where the drilling mud will obtain gel strength to lock the pipe in-place after installation (see Figure E-2).
- The annular space between the carrier pipe and conductor casing (if left in-place) will be filled with backfill grout (see Figure E-2).
- The ground under the conductor casing on both sides of the crossing will be improved with jet grouting to prevent liquefaction of the near-surface portions of the crossings.

4.5 Trenchless Approaches for the Proposed Pipeline Crossings

The following sections describe the application of these two trenchless methods to the individual pipeline crossings. Also described are different levels of robustness with corresponding levels of risk that can be incorporated in the crossing designs to achieve the goal of having a functional pipeline crossing following a major seismic event. System robustness can be achieved through the trenchless installation, e.g., installation from good ground to good ground versus installation through unstable ground, or through the system components, e.g., design accessibility to new pipe or permanently grout pipe in place.

4.5.1 Microtunneling

Our historical reviews of the alignments, as well as our geologic/geotechnical review of the trenchless crossings, indicate that microtunneling can be used to construct single-drive crossings for Alternatives 1A, 2A, and 3A. The controlling criterion is that the tunnel horizon be founded in the Old Bay Clays which are not prone to seismically induced liquefaction or lateral spreading. Depending on the level of robustness desired, shafts returning the riser pipe to the surface can be located to extend through materials known to be unstable during seismic events, or microtunnel drives can be extended so that the riser pipe can be brought to the surface through ground not prone to seismic instability.

4.5.1.1 Alternative 1A

For Alternative 1A, the depth down to the casing crown will be Elevation (El) -80 feet to address the potential of any future shipping channel dredging. The target elevation accounts for future dredging as well as bottom separation for protection against anchor drops or drag.

A single drive layout of Alternative 1A is depicted in Figure F-2. As shown, the drive extends from shoreline to shoreline resulting in a drive of about 1,216 feet. The casing depth criteria will require shafts about 85 feet deep.

As a good ground to good ground variation, a microtunneled crossing was developed from 3rd Street in Oakland to Eagle Avenue on Alameda Island (see Figure F-3). The concept is to install seven shafts (three jacking shafts and four receiving shafts) with drive lengths of 1,200 feet to install the pipeline in non-liquefiable soils from Oakland to Alameda Island. The alignment extends south along Alice Street from 3rd Street and under the Amtrak railroad tracks, across the channel to Alameda Island (see Figures F-4 to F-8). Once on Alameda Island, the alignment turns and runs south along Constitution Way to Eagle Avenue.

4.5.1.2 Alternative 2A

For Alternative 2A, depth to the casing is not controlled by dredging. Consequently, depth to the casing crown is a function of setting the tunnel horizon in the Old Bay Clays that are not prone to seismic instability. For Alternative 2A, the top of casing is set at El. -60 feet.

A single drive layout of Alternative 2A is depicted in Figure F-11. As shown, the drive extends from shoreline to shoreline resulting in a drive of about 903 feet. The casing depth criteria will require shafts about 65 feet deep founded in materials not prone to seismic instability. The upper portions of the shafts would extend through ground prone to liquefaction during a seismic event. The resulting surface settlement is not expected to impact the shaft as much as the pipeline at the transition from shaft to unstable ground.

The alignment of the microtunnel drive has been moved slightly to the west in an effort miss any buried foundations from the historical bridge alignment that crossed the channel prior to the new Bay Farm Island Bridge built in the 1950s. More investigations of the old bridge foundation locations are needed if this alternative is selected.

4.5.1.3 Alternative 3A

For Alternative 3A, depth to the casing is not controlled by dredging. Consequently, depth to the casing crown is a function of setting the tunnel horizon in the Old Bay Clays that are not prone to seismic instability. For Alternative 3A, the top of casing is set at El. -60 feet.

A single drive layout of Alternative 3A is depicted in Figures F-14 and F-15. As shown in Figure F-14, the drive extends from a jacking shaft located on the northeast side of Glascock Street on Derby Avenue in Oakland to a receiving shaft in Blanding Avenue on Alameda Island resulting in a drive of about 1,150 feet. The jacking shaft on Derby Avenue is about 350 feet onshore to be away from the identified contaminated ground on the south side of the Glascock Street, the former site of the Shell Oil Company tanks. The receiving shaft was placed in Blanding Avenue in an effort to remain within the public right-of-way.

In an effort to reduce construction costs, a shorter microtunneled crossing was identified and is shown in Figure F-15. The drive extends from the same jacking shaft location on the Oakland side to a receiving shaft in the Nob Hill Food Market's parking lot. The resulting drive length is about 950 feet.

For both options, the casing depth criteria will require shafts about 65 feet deep founded in materials not prone to seismic instability. The upper portions of the shafts would extend through loose materials, which is ground that could potentially liquefaction during a seismic event. The resulting surface settlement is not expected to impact the shaft as much as the surface pipeline at the transition from shaft is in stable ground.

4.5.2 Horizontal Directional Drilling (HDD)

Our historical reviews of the alignments, as well as our geologic/geotechnical review of the trenchless crossings, indicate that HDD can be used to construct a single-drive crossing for Alternatives 1D, 2A, and 3A.

The controlling criterion that will be incorporated into the HDD crossing designs include the alignment founded in Old Bay Clays not prone to seismic instability at the channel crossings. The depth needed to be in the stable soils is obtained by the selection of the entry and exit angles. HDD will be used to directly install a 24 inch ID (~30 inch OD) carrier pipe. The reamed hole in which the carrier pipe is pulled into must be at least 39 inches in diameter. The conductor casing at each end of the crossing would be on the order of 48 inches in diameter.

An HDD alignment would be need to be set in the Old Bay Clays where a hole of this size would have the best chance of remaining stable long enough for the pipe pullback. Conductor casings would be required at both ends to extend from the surface through the fills and Young Bay Muds preferably into the underlying Old Bay Clays. Finding windows through the existing utilities to install the inclined conductor casings could be a difficult task.

4.5.2.1 Alternative 1D

The plan and profile of a single drive layout of Alternative 1D is depicted in Figure F-9. The drive extends from a pilot bore pit in the roadway in Estuary Park on the Oakland side to the Telecare Corporation's parking lot on Alameda Island. The alignment would be threaded down under the shore protection on the Oakland side. A compound curve will be utilized to position the pipeline between the

Pacific Marina dock pilings and under the Alameda Island shore protection. The exit point is located in the parking lot, which is open for receiving the pilot hole. The HDD alignment is 1,780 feet long.

The laydown area for the pipe fusion would be along the east side of the median strip of Marina Village Parkway (see Figure F-10). The parkway provides a long usable distance for the laydown area. The initial position of the pipeline is in the median to maintain westbound and eastbound travel along Marina Village Parkway. Access to all driveways will be maintained during fusion and testing of the pipeline. At locations where the pipeline crosses an intersection, K-rail will be used to protect the pipeline. Cross traffic will be detoured around the pipeline using the open westbound and eastbound lanes of Marina Village Parkway. The duration of the traffic detours would be about three weeks. This timing assumes a week to position and weld the pipe segments together. We would allow one week to pressure test the pipe string. And we should allow a week for coordination between when the reamed pilot hole is finished, cleaned, and prepared for pullback.

Pullback will be scheduled to start in the evening after the local businesses closed for the day and vehicle traffic reduced. Initially, the pipeline will be walked across Marina Village Parkway to the parking lot. The westbound lane of Marina Village Parkway will be closed during pullback. As the pipeline goes into the hole, the K-rail used at the intersections will be removed and the intersections along Marina Village Parkway will be opened to cross traffic. Pullback will be completed over a 12 to 14-hour period. The westbound lane of Marina Village Parkway will be reopened to traffic the next morning.

4.5.2.2 Alternative 2A

A single drive layout of Alternative 2A is depicted in Figure F-12. As shown, the drive extends from a pilot bore pit on Alameda Island to a surface pipe pullback pit in Veterans Court on North Bay Farm Island. The alignment would utilize a compound curve to establish an orientation that veers away from the historical bridge alignment, thus away from any remnant bridge foundations, and then back towards the bridge alignment on the other side of the crossing. The HDD alignment is 1,250 feet long.

The laydown area for the pipe fusion would be along Veterans Court, and then turning south along Island Drive (see Figure F-12). The Veterans Court and Island Drive provide a long usable distance for the laydown area. The initial position of the pipeline is in Veterans Court along the west edge. This will allow access to Veterans Court and bike traffic to continue north to the coast bike trail. The pipeline will continue south along Island Drive positioned between the sidewalk and paved bike path. K-rail will be used to keep the pipeline off the sidewalk. The pipeline will not cross any driveways or intersections. The K-rails would be left in place for about three weeks. This timing assumes a week to position and weld the pipe segments together. We would allow one week to pressure test the pipe string. And we should allow a week for coordination between when the reamed pilot hole is finished, cleaned, and prepared for pullback.

Pullback will be scheduled to start in the evening after most bike riders are off the trails. Initially, the pipeline will be walked north along Veterans Court to the exit pit. The travel lanes along Veterans Court will be closed during pullback. As the pipeline goes into the hole, the K-rail used along the sidewalk and bike path will be removed. Pullback will be completed over a 12-hour period. The travel lanes and bike path of Veterans Court will be reopened to traffic the next morning.

4.5.2.3 Alternative 3A

A single drive layout of Alternative 3A is depicted in Figure F-16. As shown, the drive extends from a pilot bore pit northeast of Glascock Street on Derby Avenue in Oakland to a surface pipe pullback pit in Broadway just south of Blanding Avenue on Alameda Island. The alignment would utilize a compound curve to establish an orientation that veers around the storm outlets on Alameda Island. The HDD alignment is 1,300 feet long.

The laydown area for the pipe fusion would be along the center stripe of Broadway on the south side of Tilden Way (see Figure F-17). Broadway provides a long usable distance for the laydown area. With the use of K-rail, local traffic along Broadway will be maintained. Access to all driveways will be maintained during fusion and testing of the pipeline. At locations where the pipeline crosses an intersection, K-rail will be used to protect the pipeline. Cross traffic will be detoured around the pipeline using the open southbound and northbound lanes of Broadway. The duration of the traffic detours would be about three weeks. This timing assumes a week to position and weld the pipe segments together. We would allow one week to pressure test the pipe string. And we should allow a week for coordination between when the reamed pilot hole is finished, cleaned, and prepared for pullback.

Pullback will be scheduled to start in the evening after vehicle traffic along Tilden Way is reduced. Initially, the pipeline will be walked across Tilden Way to the entry pit. The eastbound and westbound lanes of Tilden Way will be closed during pullback. As the pipeline goes into the hole, the K-rail used at the intersections will be removed and the intersections along Broadway will be opened to cross traffic. Pullback will be completed over a 14-hour period. The eastbound and westbound lanes of Tilden Way will be reopened to traffic the next morning.

5 Seismic Vulnerabilities of Existing and Proposed Pipeline Crossings

5.1 Existing Pipeline Crossings

For three of the pipeline crossings, the results presented in this section are based on detailed analyses, including nonlinear structural pipeline models. These three crossings are:

- Alice Street Estuary Crossing (from Oakland to Alameda Island, Active)
- Oak Street Estuary (Blanding) Crossing (from Oakland to Alameda Island, Active)
- Bay Farm #2 San Leandro Channel Crossing (from Alameda Island to North Bay Farm Island, Active)

For the other four crossings, the results are based on extrapolations from the detailed evaluations for the pipeline crossings named above. The extrapolated results are for the following pipeline crossings:

- Park Street Estuary Crossing (from Oakland to Alameda Island, Active)
- Broadway–Derby Estuary (Derby) Crossing (from Oakland to Alameda Island, Closed)
- Bay Farm #1 Bridge Crossing (from Alameda Island to North Bay Farm Island, Closed)
- High Street Crossing (from Oakland to Alameda Island, Closed)

The pipes are examined for performance under four possible scenario earthquakes:

- Hayward M 7.0, on the nearby northern segment of the Hayward fault
- Hayward M 6.0, on the nearby northern segment of the Hayward fault
- Calaveras M 6.75, on the northern segment of the Calaveras fault
- Concord M 6.5, on the Concord fault

By “scenario,” it is meant that the earthquake is assumed to occur, without any recurrence interval associated with it. These four scenario events are thought to bound the range of earthquake loads in the District’s system, for near maximum earthquakes (Hayward M 7.0, Calaveras M 6.75, Concord M 6.5), and more probable earthquakes (Hayward M 6.0). A large event on the somewhat more distant San Andreas fault, possibly M 8.0, would produce somewhat lower shaking in the District service area than the Hayward M 7.0; liquefaction effects might be similar as for the Hayward M 7.0 event for Alameda Island area.

The probability of these four events occurring within the next 50 to 200 years is estimated below. By probability of occurrence, we mean earthquakes within 0.25 magnitudes of the median listed: for example, Hayward M 7.0 refers to a range of M 6.75 to 7.25. While “definitive” percentages can be calculated for each event, based on common assumptions such as Poisson processes, magnitude-recurrence intervals, fault memory, etc., for purposes of the new pipeline selection process, we think these approximate values are suitable for factoring in the possible choices. In the long term, we must emphasize that the Hayward M 7.0 event is nearly a certainty, and project objectives require that a minimum of reliable pipeline (to each place) remain in service and one pipeline be repairable to allow for a rapid return to service after this event.

- Hayward M 7.0. 20% to 60% chance in the next 50 years (assumes long term return period of 150± years and no event since 1868). Over 90% chance of occurrence in the next 200 years.
- Hayward M 6.0. 30% to 80% chance in the next 50 years. Over 95% chance of occurrence in the next 200 years.
- Calaveras M 6.75. Under 10% chance in the next 50 years. About 20% chance of occurrence in the next 200 years.
- Concord M 6.5. Under 10% chance in the next 50 years. About 15% chance of occurrence in the next 200 years.

5.1.1 Alice Street Channel Crossing

5.1.1.1 Earthquake Hazards

Two types of seismic hazards are considered. The first is a seismic wave propagation (WP) effect. This causes deformations to the pipe by the transient shaking of the surrounding soil due to the passage of seismic ground waves. Key parameters influencing the WP effect include the local Peak Ground Velocity and Peak Ground Acceleration during the earthquake. The median peak ground motions at the crossing under each of the four scenario earthquakes are contained in Table 5-1. Given the earthquake, the median values in Table 5-1 could vary by ±50% (16th to 84th percentiles). Lacking site-specific analyses, any new pipeline should be designed for the Hayward M 7.0 median values (PGV and PGA and corresponding spectra) plus 50%.

Table 5-1. Ground Motions (Median)

Earthquake	PGV (in./sec)	PGA (g)
Hayward M 7.0	33	0.45
Hayward M 6.0	17	0.35
Calaveras M 6.75	17	0.25
Concord M 6.5	13	0.20

The second hazard is permanent ground deformations (PGD) caused by ground failure due to seismic shaking. The crossing and approaches are located in a filled tidal flat area which is susceptible to liquefaction-induced lateral spreading. The soil surrounding the pipeline at the Oakland Inner Harbor contains clayey-sand with low blow counts. The soil near the Alameda Island side of the pipeline is underlain with very soft, high plasticity silt. Based on soil borehole information (EBMUD, 1945) as well as PGD data from other earthquakes (Bartlett, 1992), estimated PGD data are listed in Table 5-2. Larger PGDs are assumed to move laterally towards the shoreline. Smaller PGDs away from the shoreline are predominantly settlement. Location of ground fissures is likely to be closer to the shoreline, with the fissures taking up much of the lateral spread. All PGDs in Table 5-2 are median, with actual displacements generally being within a factor of 2 (i.e., multiplied or divided by 2). It is recommended that if the Alice Street area were selected for a new pipeline crossing installation, that the initial design values consider the values in Table 5-2, and that the final design be based on site specific investigations.

Table 5-2. Ground Displacements

Earthquake	Oakland Side PGD (in.)	Oakland Side Distance from shoreline (ft)	Alameda Island Side PGD (in.)	Alameda Island Side Distance from shoreline (ft)
Hayward M 7.0	> 60 36 to 60 12 to 36 Negligible	< 150 150 to 300 300 to 1100 > 1100	> 60 36 to 60 12 to 36	< 250 250 to 500 500 to > 1500
Hayward M 6.0	6 to 12 3 to 6 <3	< 50 50 to 150 > 150	6 to 12 3 to 6 <3	< 100 100 to 300 > 300
Calaveras M 6.75	6 to 12 3 to 6 <3	< 100 100 to 300 > 300	>12 6 to 12 3 to 6 <3	< 100 100 to 200 200 to 500 >500
Concord M 6.5	3 to 6 <3	< 50 > 50	3 to 6 <3	< 100 > 100

In 2000, a geotechnical evaluation was developed for the nearby EBMUD third sewer, the so-called Alameda Siphon project (KJC and OCC, 2000). The siphon portion of that project was to a be (about) a 1,750-foot long inverted siphon, about 36-inch diameter, under the Oakland Inner Harbor, parallel to the District's existing double barrel siphon (and immediately north of, and roughly parallel to the 24-inch Alice Street potable water pipeline). The style of construction was intended to be horizontal directional drilling. As part of the 2000 effort, 10 exploratory borings were made along the siphon: 3 on Alameda Island, 3 in the inner harbor channel, and 4 in the City of Oakland. Two additional borings (EB-3 and EB-4) from nearby projects were also projected onto the alignment. (See Figure B-1 for the interpreted soil layering across the sewer siphon alignment from that effort.) The actual borings were not available for this study. The following characterizes the findings along the force main siphon alignment, based on the available report (KJC and OCC, 2000):

- Most of the approaches of the Alice Street Crossing on the Alameda Island side was former marshland, and was filled in the 1920s to create the present-day land features. Portions of Alameda Island are still settling due to consolidation of the underlying soil from the weight of fill. Generally, the soil profile consists of man-made fill (approach segments) overlying varying thicknesses of Young Bay Mud, overlying sandy or silty clay, overlying Old Bay Clays; several loose to dense poorly-graded sand lenses are also present.
- The Young Bay Mud on Alameda Island side is up to 80 feet thick, and on the Oakland side up to 6 to 14 feet thick. On the Oakland side, a layer of sand, silty sand, and clayey sand, about 23 to 32 feet thick, underlies the Young Bay Mud. The clayey sand layer was also found in the Alameda Island side on one of the three borings.

- The entire submarine portion is underlain by Old Bay Clays. The top of the Old Bay Clays is about -45 feet on the Oakland side, and lowers to about -80 feet on the Alameda Island side.
- Bedrock might be between -400 to -500 feet deep at the site.
- The depth to groundwater is between 1 to 2 feet below grade near either shoreline, deepening to as much as -18 feet at some distance from the Oakland shoreline.

Generally, we find that the soil layering (Figure B-1) is consistent with the PGD estimates described in Table 5-2. Lateral spreads and accompanying settlements can occur into the channel, initiated either in the near-surface artificial fills, or possibly in the underlying sand / clayey sand layer. Lateral spreads or settlements in the Old Bay Clay layer are unlikely.

5.1.1.2 Onshore Crossing Approach Evaluation

The Alameda Island approach analysis covers the bell-and-spigot cast iron pipe extending from the anchor block near the shoreline to the intersection of Tynan and Webster Avenues. This is about 2,000 feet of pipeline. At Tynan and Webster, the pipe changes from cast iron to welded steel. Figure 5-1 shows the pipe routing (from Drawings 1485-B-472, 1485-B-474). On the Alameda Island approach, the pipe goes from cut-and cover over native soils, to a portion that traverses over the Posey Tube; there could be differential settlements where the pipe transitions from over the Caltrans highways (concrete filled steel box tunnels presumed founded on deeper stable formations) to native soils.

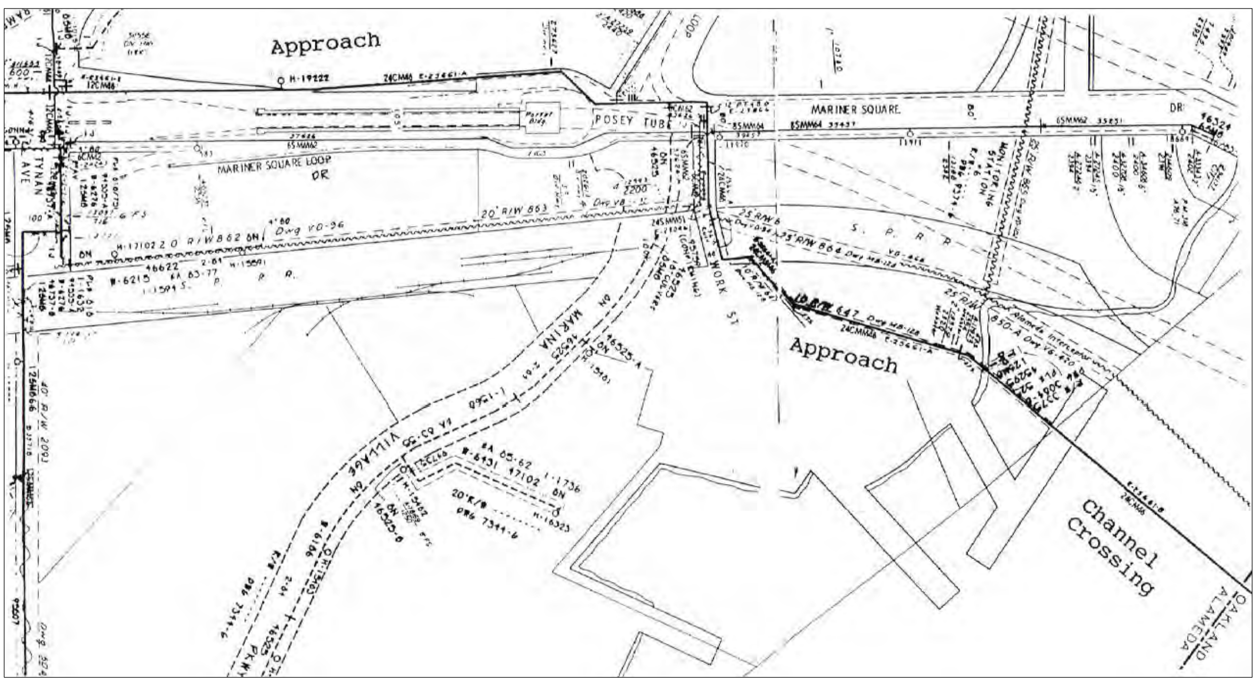


Figure 5-1. Alice Street 24-inch Pipe: Plan, Alameda Island Side Approach

The Oakland approach analysis covers all the bell-and-spigot cast iron pipe extending from the anchor block near the shoreline to about the intersection of Alice and 2nd Streets. This is about 1,000 feet of pipeline. Beyond about 1,100 feet from the shoreline, the soil conditions change from that of a tidal fill to

Merritt sands. The Merritt sands are not generally susceptible to lateral spreading PGDs. Figure 5-2 shows the pipe routing (from Drawings 1485-B-474, 1488-B-474).

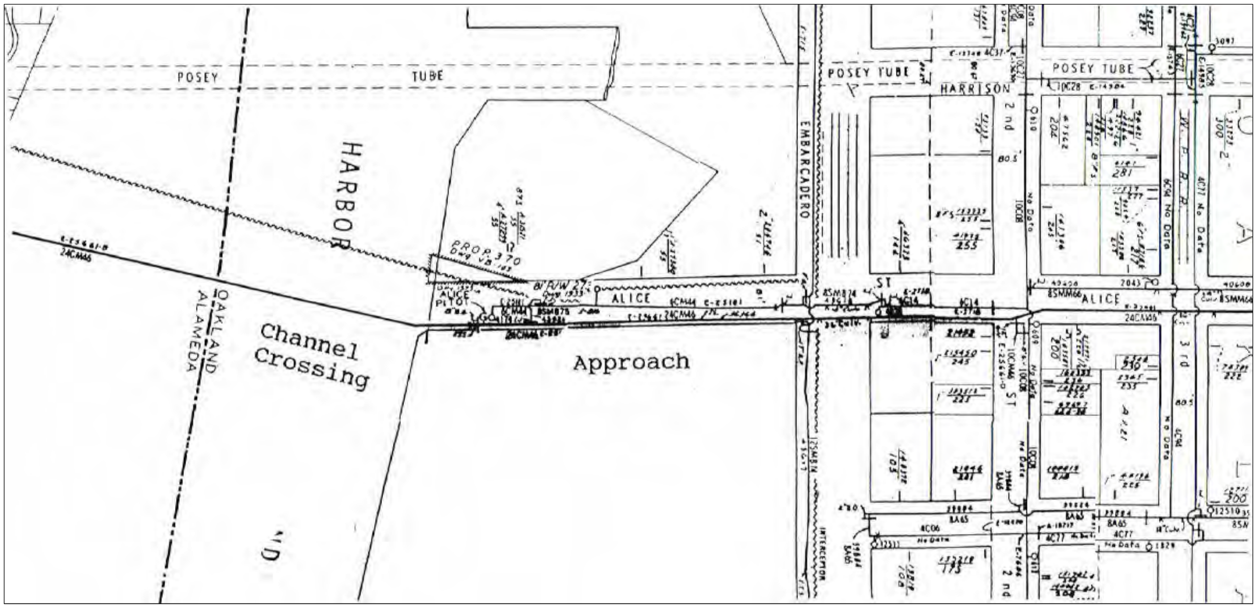


Figure 5-2. Alice Street Pipe: Plan, Oakland Side Approach

The approaches consist of Class 150 bell-and-spigot cast iron pipe having nominal diameter of 24 inches (0.73 inch wall thickness), and the joints are lead-caulked. The pipe segment lengths are 18 feet. The weak points are the joints which are most vulnerable to damage from tensile pull-out or differential joint rotations. Three limit state criteria were defined: joint pull-out, rotation, and pipe wall ultimate strain. The values are contained in Table 5-3. The joint pull-out displacement defining leakage is based on the interpretations of prior experimental tests (Prior, 1935; O'Rourke and Trautmann, 1980; Elhmadi and O'Rourke, 1989), and the displacement for a total break is taken as the pipe socket depth. The limiting joint rotation and pipe wall strain are based on prior test values (O'Rourke and Trautmann, 1980).

Table 5-3. Onshore Cast Iron Pipe Limit State (Alice Street)

Limit State Mode	Criteria
Joint pull-out displacement	2 inches (for leakage) 4.5 inches (for break)
Joint rotation	0.4 degrees (for break)
Pipe wall ultimate strain	0.52% (for break)

Wave Propagation Effects

Assuming the pipe is essentially rigid versus the joint flexibility, then upper bounds on the relative displacement (D) and rotation (R) in the joints are as follows (O'Rourke et al, 1985).

$$D = V\,L / C$$
$$R = A\,L / C^2$$

where, V = peak ground velocity, A = peak ground acceleration, L = pipe segment length, and C = apparent wave propagation velocity in the soil.

In addition, the pipe is evaluated as if it were continuous (i.e., having rigid joints), and fully bounded to the soil. In this case the peak strain (ε) in the cast iron pipe wall is

$$\varepsilon = V / C + A\,d / C^2$$

where, d = pipe outer diameter. This latter case is very conservative since the joints are relatively flexible.

The joint pull-out displacements, rotations, and pipe wall strains are computed and compared to the limit state criteria (i.e., "allowable"). The apparent seismic wave propagation velocity, C, is taken as 3,000 feet/sec, which presumes very soft soils and presumes most of the energy is in the form of surface wave. The results are in Table 5-4.

Table 5-4. Estimated Pipe Performance: Alice Street Approaches, Ground Shaking

Earthquake	Joint Pull-Out (% of Allowable)	Joint Rotation (% of Allowable)	Pipe Wall Strain (% of Allowable)
Hayward M 7.0	10%	0.3%	18%
Hayward M 6.0	5%	0.3%	9%
Calaveras M 6.75	5%	0.2%	9%
Concord M 6.5	4%	0.1%	7%

The peak responses are well within the limit state criteria (Table 5-3) thus indicating that wave propagation effects are not damaging to the approach pipeline. However, these results are computed in a deterministic manner, and hence represent average values at each joint. Empirical earthquake experience data suggests that cast iron pipe can have a repair rate of 0.2 Repairs per 1,000 feet under a peak ground velocity of 33 inches/sec. As there are 3,000 feet of such pipelines, this indicates about a 45% chance of failure due to wave propagation alone, due to the Hayward magnitude 7.0 scenario earthquake. Therefore, the PGD effects may have more potential to damage the pipe than the analysis above suggests.

Permanent Ground Deformation Effects

Experience from previous earthquakes indicates that the ground displacement field can be complicated. That is, the soil PGD is often characterized by movements of soil block masses separated by fissures or ground cracks. The ground strains are large in the vicinity of the fissures, and small within the soil blocks. Because of this complexity, the effects of PGD on the approach pipelines were evaluated in two different ways.

The first method assumes that the ground strain is constant over reduced lengths. The purpose is to simulate a pattern of large ground strains associated with fissures, and small strains within the soil blocks. This is representative of actual displacement. The reduced lengths are taken as 10% of the distances given in Table 5-2. This may be thought of as resulting from soil blocks moving as rigid bodies with numerous fissures concentrated in small zones of deformations between the blocks. The pipe joint deformations are then evaluated assuming that the pipe is essentially rigid versus the joint flexibility.

The second method is an empirical technique that relates the pipe repair rate per 1,000 feet to PGD magnitude. Note that the term "pipe repair" used here is likely to mean a detectable water leakage that triggers a repair activity. The number of pipe repairs is computed by multiplying the repair rate by the length of pipe experiencing the particular PGD magnitude (i.e., the average PGD in each length).

The pipe was analyzed two ways as described above, i.e., concentrated spread and empirical methods. The average of these analyses is shown in Table 5-5. Due to the longer approach length, and somewhat worse soil conditions on the Alameda Island side, the total damage to the Alameda Island side approach is higher than for the Oakland side approach.

Table 5-5. Alice Street Crossing: Approach Pipelines

Earthquake	Approach Side	Estimated Repairs
Hayward M 7.0	Alameda Oakland	13 6
Hayward M 6.0	Alameda Oakland	2 1
Calaveras M 6.75	Alameda Oakland	2 2
Concord M 6.5	Alameda Oakland	2 1

5.1.1.3 Submarine Portion Evaluation

The submarine pipeline evaluation covers all of the cast iron pipe extending between the anchor block on the Alameda Island side to the anchor block on the Oakland side. This is about 1,100 feet of pipeline. Figures A-2 and A-4 show the pipe arrangement in plan and profile views.

The submarine portion consists of Class D cast iron pipe having nominal diameter of 24 inches (1.16 inch wall thickness). The joints are flexible Usiflex joints. These are ball-and-socket arrangements that permit a limited free joint rotation. The joint is rigid with respect to direct tension or compression. The pipe segment lengths are 12 feet. Four limit state criteria were developed: free joint rotation limit, pipe barrel ultimate force, pipe barrel ultimate movement, and pipe wall ultimate strain. The values are contained in Table 5-6.

The free joint rotation is the maximum joint deflection per discussions with the manufacturer (Bogs, 1993). The joint becomes locked after a 15 degree deflection. Data indicating the ultimate strengths of the 1945 vintage Usiflex pipe joints are not available. Discussions with the manufacturer (1993) indicate that the ultimate moment and compressive capacity of the pipe should be governed by the pipe barrel since the Usiflex joint is of larger diameter and thicker wall. Review of the joint details (US Pipe 1943) confirms this and the ultimate force and moment in Table 5-6 are based on strength-of-materials unit calculations using the barrel section properties. The limiting pipe wall strain is based on prior test values. Once the joints become locked, increasing PGD cause forces and moments in the pipe barrel, and the pipe fails when the limit state criteria are reached.

Table 5-6. Submarine Cast Iron Pipe Limit State (Alice Street)

Limit State Mode	Criteria
Free joint rotation	15 degrees
Pipe barrel ultimate force	2,570 kips
Pipe barrel ultimate moment	1,290 kip-feet
Pipe wall ultimate strain	0.52%

Wave Propagation Analysis Evaluation

Assuming the pipe is continuous and fully bonded to the soil, then the peak strain (ε) in the cast iron pipe wall is computed as follows.

$$\varepsilon = V / C + A\ d / C^2$$

Because the joints do not have unlimited free rotation, an additional check is made for the joint rotation (R) as follows.

$$R = A\ L / C^2$$

The computed pipe strains and joint rotations are evaluated against the pipe limit state criteria in Table 5-6 to assess possible damage. The joint rotations were negligible and the pipe wall strain evaluations are as shown in Table 5-7.

Table 5-7. Submarine Pipeline: Performance due to Wave Propagation

Earthquake	Pipe Wall Strain (% of Allowable)
Hayward M 7.0	18%
Hayward M 6.0	9%
Calaveras M 6.75	9%
Concord M 6.5	7%

The peak strains are well within the limit state criteria thus indicating that wave propagation effects are not expected to cause damage to much (if any) of the submarine pipeline. However, given the uncertainties in each joint capacity, it is estimated that wave propagation effects have about a 20% chance of breaking a joint somewhere in the submarine portion.

PGD Evaluation

Starting at the Alameda Island anchor block, the submarine pipeline crossing angles downward at about 12 degrees for about 270 feet, then assumes a horizontal run for about 770 feet, then angles upward at roughly 32 degrees for about 110 feet to the Oakland anchor block. At both the Alameda Island and Oakland sides, the pipe extends down through a soil layer, likely to have PGDs, into a stable soil layer having no movement. The soil displacement field is that of the upper layer moving horizontally over the stable soil layer. Assuming that the anchor blocks move under the action of the PGDs, then the pipe experiences primarily a compressive loading, i.e., the pipe is pushed by the upper soil, and it is restrained in the lower soil layer.

The behavior of the Usiflex-jointed pipe run under compression is essentially that of a series of links connected by hinges. Should the soil surrounding the pipe be strong enough not to permit lateral pipe movements, then the response of the pipe would be that of an axially loaded rod. The failure mode would be compressive fracture of the pipe. On the other hand, should the surrounding soil permit the pipe to buckle laterally, then the failure mode would be fracture of the pipe due to bending moment once the pipe joints lock-up.

Which of the above behavior patterns governs depends on many factors including the pipe as-built geometry, the properties of the soil, and the magnitude of the PGD. In order to account for these effects, nonlinear structural analysis was performed[1] using the ANSR computer program. Separate computer models were formulated for the Alameda Island and the Oakland sides. The models start at the anchor blocks and end at points under the channel. Each pipe segment was modeled by a truss element having hinges at the ends. The element formulation includes large deformation effects in order to capture the lateral buckling behavior of the segmented pipe system.

There is uncertainty regarding the accuracy of the pipe position. For example, the noticeable angle change in the pipe run on the Oakland side (Figure 5-2) does not correspond to a location of a pipe joint (when superimposing 12 foot pipe segments lengths on the sketch). Moreover, pipe settlement over time in the soft bay fill materials may reduce abrupt kinks in the pipe alignment due to pipe catenary action.

The soil is accounted for by translational springs connected to the joints. The soil springs are oriented parallel (axial spring) and perpendicular (lateral spring) to the pipe run. The axial springs model the soil resistance to longitudinal pipe movements. This is primarily due to skin friction between the pipe and the soil. The lateral springs model the soil resistance to transverse pipe movements. The lateral springs are linear-elastic and their stiffness is based on a coefficient of subgrade reaction. The axial springs are elasto-plastic and the ultimate force is based on friction strength. The stiffness of the axial springs are taken as the same as that for the lateral springs.

The pipe is anchored in the stable soil at a point in the horizontal run under the channel. The PGD is simulated by horizontal displacements applied at the anchor block and at the ends of the soil springs for

[1] These analyses were performed under the EBMUD SEP project.

those located in the soil layer experiencing movements. The response quantities of interest are the peak axial force in the pipe and the peak relative joint rotation as a function of PGD. Several analyses were performed in order to bound the actual behavior.

Cases A and B were analyses of the Alameda Island side. Case A uses upper bound soil properties per ASCE (ALA 2005) guidelines and Japanese study of permanent ground deformations. Figure 5-3 shows the deflected shape of the pipe under a PGD of 33.7 inches. Note the "accordion" type pattern of upward then downward deflection of the pipe segments. This mechanism was typical in each of the analyses. Case B is the same as Case A except reduced soil properties are used for the soils in the layer experiencing PGDs. The coefficient of subgrade reaction and friction strength are reduced by a factor of 10. This is to account for the expected large decrease in soil stiffness and strength due to liquefaction. Cases C and D were analyses of the Oakland side. Case C uses the upper bound soil properties, and Case D uses reduced properties in the soil layer having PGDs. The analyses indicate that the joints lock before a limit state axial force is developed in the pipe.

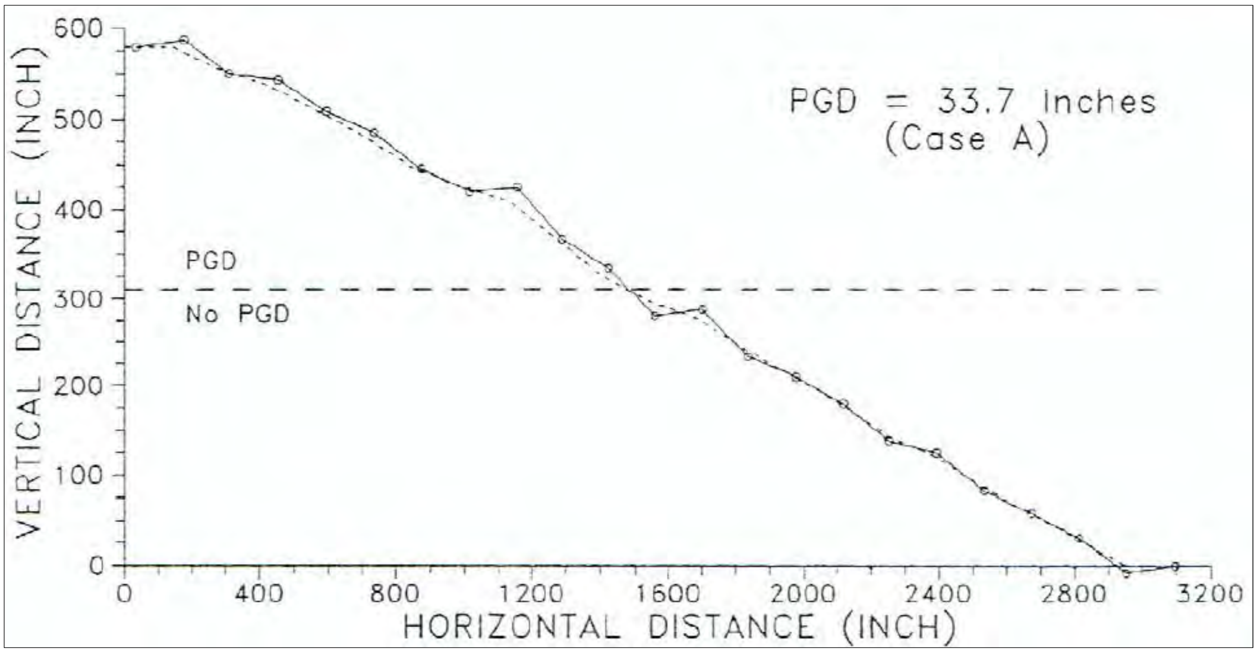


Figure 5-3. Submarine Pipe: Alameda Island Side Deflected Shape (Case A)

Because the joints lock after reaching their rotation limit, subsequent PGD loading causes axial and bending moments in the pipe. Additional beam-on-inelastic foundation analyses were performed to determine the additional PGD before the limit state condition is developed in the pipe barrel. A model was analyzed assuming a locked joint having a 15 degree “kink,” and this joint between two pipe segments is fixed.

Soil springs were included to account for lateral soil resistance. Opposing horizontal forces were applied to the ends of the two kinked pipe segments. The analyses indicate that the pipe can take only a small amount of additional PGD before failure of the pipe barrel, i.e., on the order of 20 percent of the PGD needed to lock up the joints. Assuming that at least one joint has an initial rotation of about 7 degrees, then failure PGD levels will be reached at less than those calculated assuming every joint is perfectly aligned. Based on the range of analyses performed, the most likely PGDs to cause a pipe failure are as follows: 16 inches for the Alameda side, and 10 inches for the Oakland side. These limit states are compared to the scenario earthquake PGDs in Table 5-8.

Table 5-8. Alice Street Crossing: Submarine Portion

Earthquake	Channel Side	PGD (in.)	Limit Criteria (in.)	D/C	Damage Likelihood
Hayward M 7	Alameda	84	16	5.3	High
	Oakland	84	10	8.4	High
Hayward M 6	Alameda	12	16	0.8	Moderate
	Oakland	12	10		Moderate
				1.2	
Calaveras M 6.75	Alameda	18	16	1.1	Moderate
	Oakland	12	10	1.2	Moderate
Concord M 6.5	Alameda	6	16	0.4	Low
	Oakland	6	10	0.6	Low

The D/C ratio is a measure of the PGD demand (D) versus pipe capacity (C) to withstand the movement, based on the deterministic analyses. Ratios greater than one indicate that demand is greater than capacity thus suggesting pipe damage. Due to uncertainties in both the demand and capacity, the assessment is described qualitatively in terms of pipe damage likelihood.

Should damage occur to the pipeline, the analyses indicate that it would be in the underwater inclined portions of the pipeline as it slopes upward toward the anchor blocks. The postulated pipe damage mode is compressive buckling of the pipe segments with failure of the pipe under bending after certain Usiflex joints exceed their free rotational capacity.

5.1.2 Oak Street (Blanding) Channel Crossing

5.1.2.1 Earthquake Hazard

The seismic hazard for the submarine portion is a seismic wave propagation effect. This causes deformations to the pipe by the transient shaking of the surrounding soil due to the passage of seismic ground waves. Key parameters influencing the wave propagation effect include the local Peak Ground Velocity and Peak Ground Acceleration during the earthquake. The estimated peak values at the pipeline crossing (firm soil at the surface) for each scenario earthquake are contained in Table 5-9.

The site is classified as Merritt sands terrain unit which are dense formations having low potential for liquefaction. Figures B-5 and B-6 show the plan, profile and boring logs for six borings along the alignment. The local soil conditions under the pipeline through its entire submarine crossing consist primarily of silty sand soils. While there are potentially liquefiable soils above the pipeline along the embankments, these do not intersect the pipeline and appear to pose no risk to the 24-inch diameter pipeline itself.

Table 5-9. Ground Motions (Median)

Earthquake	PGV (in./sec)	PGA (g)
Hayward M 7.0	22	0.45
Hayward M 6.0	10	0.38
Calaveras M 6.75	8	0.28
Concord M 6.5	6	0.23

Based on this data, it is inferred that there is no significant ground failure/deformation hazard along the submarine portion of the Oak Street pipeline crossing. Accordingly, the pipeline crossing is evaluated for the wave propagation effects only.

5.1.2.2 Seismic Evaluation

The pipeline consists of bell-and-spigot steel pipe having pipe barrel outside diameter of 25.75 inches and pipe wall thickness of 0.25 inch. Assuming a maximum operating pressure in the Central pressure zone of 100 psi, this results in a hoop stress of 4,800 psi, which is very low. Even if the pipe is exposed to the grade line of the Aqueduct zone, there is no issue with hoop stress. Thus, the wall thickness is controlled by installation methods as well as seismic conditions, and possibly also with an allowance for potential wall thinning due to potential corrosion.

Based on a review of the installation (discussion with M. Falarski), it is believed that the pipe joints are welded on both the inside and outside of the pipe (double fillet welds); this would allow for the relatively tight radius (indicated as 1,500 feet) along the profile. The limit state criterion is as follows.

- Incipient wall wrinkling strain = 0.15%

The criterion that the strain in the pipe should not exceed the compressive strain at which the pipe wall buckles within the barrel (i.e., wrinkles) is used because after wrinkling, the pipe loses strength, and also because local strains in the vicinity of the wrinkle can become sufficiently large as to initiate fracture. It is recognized that the onset of wrinkling does not necessarily imply pipe wall fracture and associated leakage.

This criterion is taken as one-half of the wrinkling strain as computed using the ALA (2005) guideline for wrinkling strains of steel cylinders. A factor of one-half is used to reduce the wrinkling strain to account for the pipe section offsets at the connection of pipe segments due to the bell-and-spigot joints. This is based on analogy with the ASME code joint efficiency factor of 0.55 assigned for double fillet welded joints (i.e., regarding stress, the double fillet welded connection is about one-half as efficient as a butt welded joint). In reality, this is a very conservative criterion, and we use it only because the computed pipe strains are still below this limit.

Wave Propagation Evaluation

Assuming the pipe is fully bounded to the soil, the peak strain in the pipe is found using the methodology described in Section 5.1.1.

The analysis covers all the welded steel pipe having bell-and-spigot joints, from the intersection of 23rd Avenue and Ford Street on the Oakland side to the intersection of Oak Street and Blanding Avenue on the Alameda Island side. This is about 2,000 feet of pipe.

The computed peak pipe wall strains are compared to the limit state criteria. The apparent seismic wave propagation velocity is taken as 3,000 feet/sec, which conservatively presumes the damaging seismic waves are surface waves traversing soft soils. The results are in Table 5-10.

Table 5-10. Oak Street Pipeline Crossing

Earthquake	Strain (% of allowable)
Hayward M 7	41%
Hayward M 6	19%
Calaveras M 6.75	15%
Concord M 6.5	11%

The peak responses in Table 5-10 are within the limit state criteria thus indicating that wave propagation effects are not likely to cause damage to the pipe under any of the scenario earthquakes.

For the submarine pipeline, the strains computed above are especially conservative because the pipe-to-soil friction is expected to be very low. Slippage implies that the ground strains cannot be developed in the pipe. The reason for the low friction value is that the horizontal directional drilling construction process is assumed to have required a heavy viscous drilling fluid (slurry) in order to prevent collapsing of the bored hole, and the bored hole is typically 30 percent larger than the pipe diameter. Thus, when the pipe was installed in the hole, a layer of the drilling fluid would likely have remained around the pipe. Because the pipe is below water level, the drilling fluid is likely to remain in a somewhat viscous state (Cherrington, 1993). This would permit the soil to slip relative to the pipeline.

These analyses assume that no PGDs occur at the approaches to the submarine pipeline.

5.1.3 Bay Farm #2 Crossing

5.1.3.1 Earthquake Hazards

Figure B-16 shows profile logs from 6 borings from 1978 to 1981, used as part of the design and installation process for this pipeline. The logs indicate that the upper layers consist of highly plastic organic materials (Young Bay Mud), underlain by medium to medium-stiff old bay clay.

The thickness of the Young Bay Mud, as reported in the six logs, is as follows (in feet): 20; 20; 50; 45; 12; 45. The geographic locations of the six logs are not known. The one boring with the shallow Young

Bay Mud lens (12 feet) is underlain by 21 feet of stiff to hard silty sands and clay, which in turn is underlain by about 5 feet of soft wet clays with organics, which in turn is underlain by old bay clay; while unconfirmed, this boring might go through some man-made fill.

Two types of seismic hazards are considered. The first is a seismic wave propagation effect which causes deformations to the pipe by the transient shaking of the surrounding soil due to the passage of ground waves. Key parameters influencing the wave propagation effect include the local Peak Ground Velocity and Peak Ground Acceleration during the earthquake. The expected peak values at the crossing for each scenario earthquake are contained in Table 5-11.

Table 5-11. Ground Motions (Median)

Earthquake	PGV (in./sec)	PGA (g)
Hayward M 7.0	33	0.45
Hayward M 6.0	16	0.35
Calaveras M 6.75	17	0.25
Concord M 6.5	13	0.20

The second hazard is permanent ground deformations (PGD) caused by ground failure due to seismic shaking. The crossing is located in soft soils which have the potential for slope instabilities. Table 5-12 contains estimated median PGD data for each scenario earthquake based on soil borehole information (see Figure B-13); actual PGDs are expected to be within a factor of 2 of these values. The PGDs can occur on both sides of the channel. The mechanism is characterized by horizontal movement of soil mass toward the center of the channel. The PGDs are uniform with depth including the pipe elevation. The extent of the PGDs along the pipe for all earthquake scenarios is contained in Table 5-13.

Table 5-12. Bay Farm #2 Crossing Ground Motions (Median)

Earthquake	PGD (in.)
Hayward M 7.0	24
Hayward M 6.0	3
Calaveras M 6.75	3
Concord M 6.5	< 3

Table 5-13. Bay Farm #2 Crossing PGD Lengths

Location	From Station	To Station	Total Length (ft)
North Bay Farm Island Side	8+00*	12+00	400
Alameda Island Side	0-60	3+70	430

* - Stations per Figures A-18, A-19.

5.1.3.2 Seismic Evaluation

The pipeline consists of bell-and-spigot steel pipe having pipe cylinder outside diameter of 25.75 inch and pipe wall thickness of 0.281 inch. It is welded on both the inside and outside of the pipe (double fillet welds). The pipe is cement mortar lined and coated. The coating is reinforced with continuous, spirally wound 15 gage wire at a 1 inch pitch. Individual pipe segments have maximum lengths of 40 feet.

The pipe limit state criterion is that the strain in the pipe should not exceed the compressive strain at which the pipe wall buckles (i.e., wrinkles) because after wrinkling, the pipe loses strength, and also because local strains in the vicinity of the wrinkle can become sufficiently large as to initiate fracture. Note that the onset of wrinkling does not necessarily imply pipe wall failure. The methodology to determine the limit strain is the same as that used for the Oak Street Crossing analysis above. This is based on a bare steel cylinder, and hence is conservative since the actual reinforced mortar coating will mitigate wrinkling to some degree.

Inside of each vault/anchor the pipe has a flanged coupling having a sleeve arrangement which acts as a slip joint, Figure 5-4. The joint geometry nominally allows for a 1 inch relative contraction of the pipe segments before pipe end bearing develops, and 17.5 inch relative expansion movement of the pipes before the pipe separates from the sleeve. The limit state criterion for pipe expansion joint pull-out is taken as 80% of the ultimate separation distance, i.e., 14 inches.

The pipe as it passes through the vault/anchors rests on adjustable steel pedestals that only provide vertical support to the pipe. Thus, relative horizontal movement between the pipe and vault can occur. Since the vault/anchors are in the soil mass that is forecast to experience PGDs, they will move toward the channel. Review of the clearance between the flanged coupling hardware and the steel plate forming the vault wall indicates that about 20 inches of relative pipe-to-vault/anchor movement can be accommodated before the coupling binds on the wall, see Figure 5-4. This displacement is taken as the limit state criterion.

The limit state criteria area summarized in Table 5-14.

Table 5-14. Bay Farm #2 Crossing Limit State Criteria

Limit State Mode	Criteria
Incipient pipe wall wrinkling strain	0.16 %
Flanged joint pull-out displacement	14 in.
Relative pipe-to-vault / anchor displacement	20 in.

Wave Propagation Effects

Assuming the pipe is continuous and fully bonded to the soil, the peak strain in the pipe is found using the methodology described in Section 5.1.1.

The peak pipe strains are computed and compared to the limit state criteria in Table 5-15.

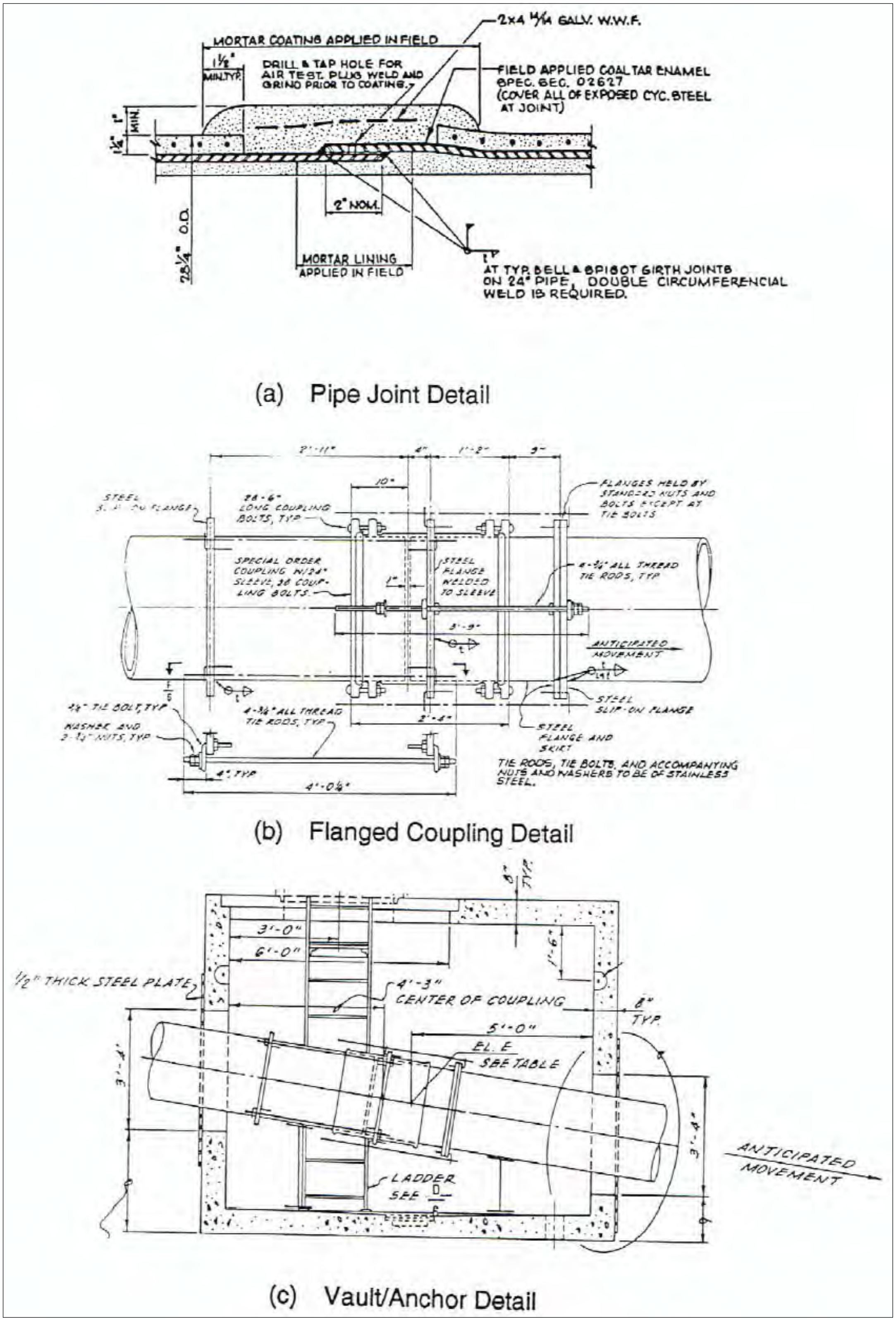


Figure 5-4. Bay Farm #2 Crossing Girth Joint / Vault / Anchor Details

Table 5-15. Bay Farm #2 Crossing: Wave Passage

Earthquake	Strain (% of allowable)
Hayward M 7	58%
Hayward M 6	28%
Calaveras M 6.75	31%
Concord M 6.5	23%

The peak responses are within the limit state criteria thus indicating that wave propagation effects are not likely to cause damage to the pipe under any of the scenario earthquakes.

Permanent Ground Deformation Effects

The evaluation considers all nominal 24-inch-diameter pipe shown in Figures A-18 and A-19. This is about 1,300 feet of pipe.

Both the Alameda Island and North Bay Farm Island sides have postulated PGDs resulting from the scenario earthquakes. On the Alameda Island side, the 24-inch diameter pipe starting on Otis Drive originates in the soil mass experiencing PGDs, and proceeds through the vault/anchor and under the channel into stable soil having no PGDs. On the North Bay Farm Island side, the 24-inch diameter pipe starting on Seal Point Court (now Veterans Court) is in (presumed) stable soil, runs through the soil mass experiencing PGDs, and proceeds through the vault/anchor and under the channel into stable soil having no PGDs. The soil displacement field is that of the soil mass moving horizontally toward the channel center. The pipe is dragged along with the moving soil mass. However, the pipe is also restrained in the adjacent stable soil. This causes primarily a compressive loading to the pipe.

The response of the crossing under PGDs is evaluated by nonlinear structural analysis using the ANSR computer program. Separate models are analyzed and the results compared to the limit state criteria presented previously. The analyses are described in the following.

North Bay Farm Island Side Pipe Evaluation

The model consists of a straight run of pipe starting at the channel center (Station 5+85) and extending onshore (Station 13+30). The pipe is discretized by a series of truss elements. The soil is accounted for by translational springs connected to the joints. The soil springs are oriented parallel to the pipe run to model the soil resistance to longitudinal pipe movements due to skin friction between the pipe and the soil. The soil springs are elasto-plastic having stiffness and ultimate force properties based on ALA (2005). The pipe is backfilled with gravel. A gap (no tension) element is located at the position of the vault/anchor to model the flanged coupling slip joint. The model is anchored at a point under the channel. The PGD is modeled by support displacements applied to the ends of the soil springs.

Figure 5-5 shows the pipe axial strain plotted along the pipe run for three levels of PGD. The strains increase with increasing PGD. However, the pipe strain becomes independent of PGD once the soil reaches its ultimate resistance, i.e., the pipe slides through the soil. Under a PGD of 48 inches, a peak pipe compressive strain of 0.12% occurs at a point near the front of the moving soil mass. The pipe strain is zero at the vault/anchor because the slip joint is opening and cannot resist tension. The key responses are compared to the limit state criteria (i.e., allowable) in Table 5-16.

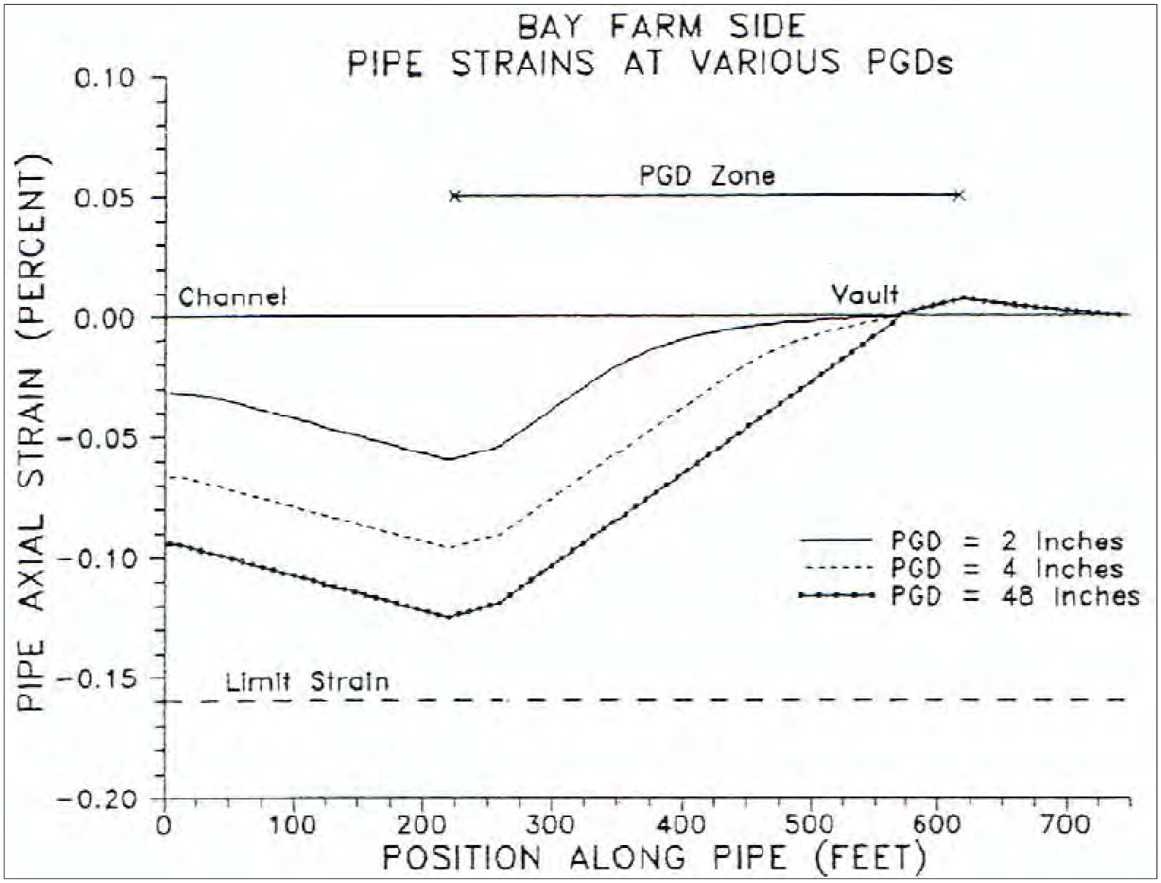


Figure 5-5. Bay Farm #2 Crossing: Pipe Axial Strain Along Pipe, North Bay Farm Island Side

Table 5-16. Bay Farm #2 Crossing: North Bay Farm Island Side, PGDs

Earthquake	Pipe Wall Strain (% of allowable)	Flange Coupling Gap Opening (% of allowable)
Hayward M 7	75%	39%
Hayward M 6	50%	20%
Calaveras M 6.75	50%	20%
Concord M 6.5	50%	20%

The peak responses are within the limit state criteria thus indicating that pipe wall wrinkling and/or flanged coupling pull-out are not likely under any of the scenario earthquakes. The possible binding of the flanged coupling with the vault/anchor wall due to relative coupling-to-vault/anchor movement is evaluated separately and is discussed below.

Alameda Island Side Pipe Evaluation

The model consists of a straight run of pipe starting at the channel center (Station 5+85) and extending back to the vault/anchor (Station 1+40). The model has the same features as that for the North Bay Farm Island side except the effects of the onshore pipe beyond the vault/anchor are accounted for by a spring that is connected to the gap element modeling the flanged coupling. This is because the onshore pipe changes direction as it goes down Otis Drive, and therefore soil resistance to pipe transverse and longitudinal movements comes into play. A separate analysis is performed to compute the equivalent spring properties.

The onshore pipe model consists of a run of pipe with a 51 degree angle change. The pipe is modeled with beam-column elements. The soil is modeled by translational springs connected to the nodes. Soil springs are oriented both parallel and perpendicular to the pipe run. The springs have bi-linear force-displacement relations having stiffness and ultimate force properties based on ALA (2005) guidelines. The pipe is anchored at the end near the anchor/vault and the PGD is modeled by applied displacements at the ends of the soil springs. The force-deformation of the system was determined and an equivalent nonlinear spring was applied at the flanged coupling.

The analysis results of the straight run of pipe (from Station 1+40 to Station 5+85) are shown in Figure 5-6. The behavior is similar to that for the North Bay Farm Island side described above. The pipe strains become independent of PGD when the soil attains its ultimate resistance. Under a PGD of 48 inches, a peak pipe compressive strain of 0.10% occurs at a point near the front of the moving soil mass. A compressive pipe strain occurs at the vault/anchor because the slip joint is in compression and therefore an axial force develops in the pipe. The key responses are compared to the limit state criteria in Table 5-17.

Regarding the pipe on the channel side of the vault/anchor, the peak strains are within the limit state criteria. Likewise, for the pipe on Otis Drive, the peak compressive force developed at the flanged coupling indicates that peak strains are within the limit state criteria as well. Thus, the results indicate that pipe wall wrinkling is not likely under any of the scenario earthquakes. The possible binding of the flanged coupling with the vault/anchor wall due to relative coupling-to-vault/anchor movement is evaluated next.

Table 5-17. Bay Farm #2 Crossing: Alameda Island Side, PGDs

Earthquake	Pipe Wall Strain (% of allowable)	Otis Drive Pipe Strain (% of allowable)
Hayward M 7	63%	88%
Hayward M 6	50%	6%
Calaveras M 6.75	50%	6%
Concord M 6.5	50%	6%

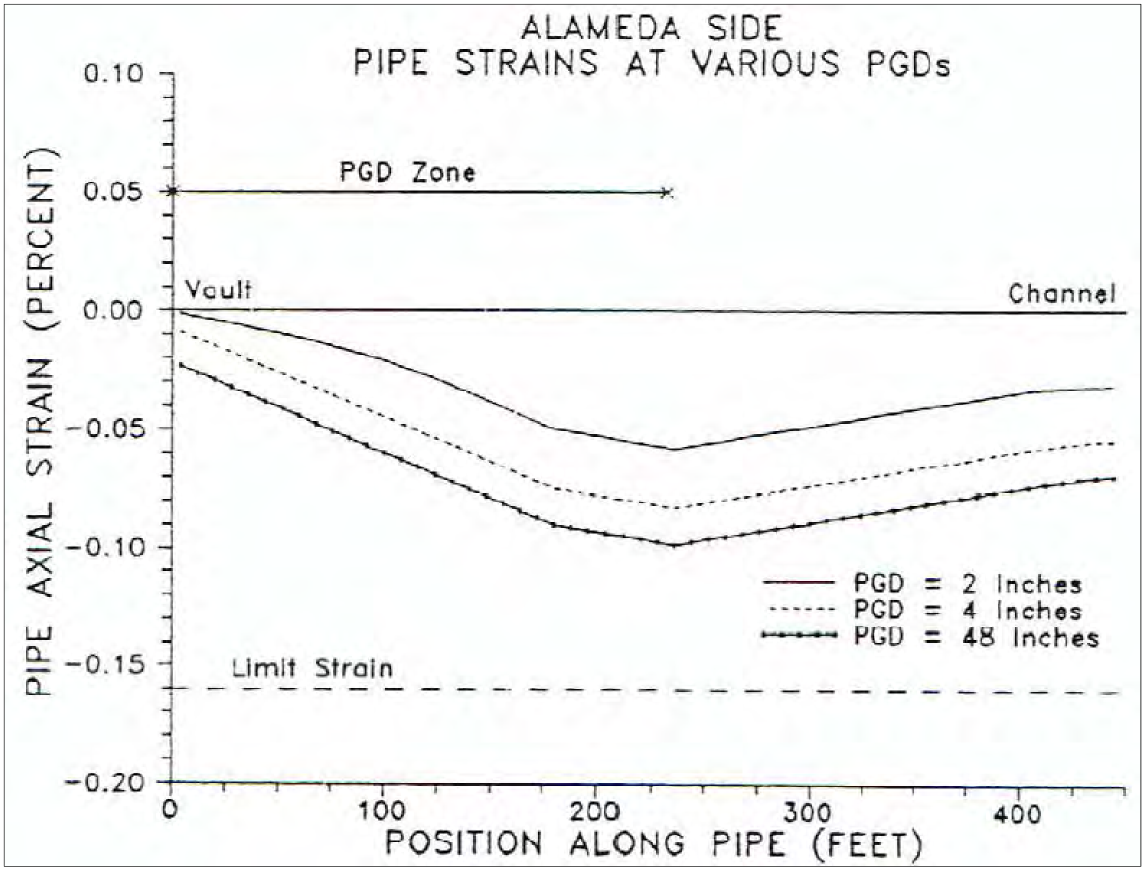


Figure 5-6. Bay Farm #2 Crossing: Pipe Axial Strain Along Pipe, Alameda Island Side

North Bay Farm Island Side Vault/Anchor Evaluation

An analysis is performed to determine the amount of vault/anchor movement under PGD. The purpose is to determine the relative pipe-to-vault/anchor movement for checking whether the flanged coupling could bind with the vault/anchor wall. That is, the vault/anchor may move toward the channel under the action of the PGD, but the flanged coupling may have little movement because the pipe extends beyond the PGD zone and is fixed in the stable soil.

Figure 5-7 depicts the vault/anchor model. The piles and vault/anchor are modeled with beam-column elements. The soil is modeled by translational springs connected to the nodes (at both the piling and vault/anchor). Soil springs are oriented both parallel and perpendicular to the pile axis. The springs have bi-linear force-displacement relations having stiffness and ultimate force properties based on ALA guidelines (2005). The vault/anchors have pilings that extend down into the stable soil having no PGD. The PGD is modeled by applied displacements at the ends of the soil springs in the soil experiencing PGDs.

The analyses indicate that after about a 2-inch PGD, the vault/anchor movement follows closely to the soil PGD, i.e., the vault/anchor is very compliant with the soil PGD.

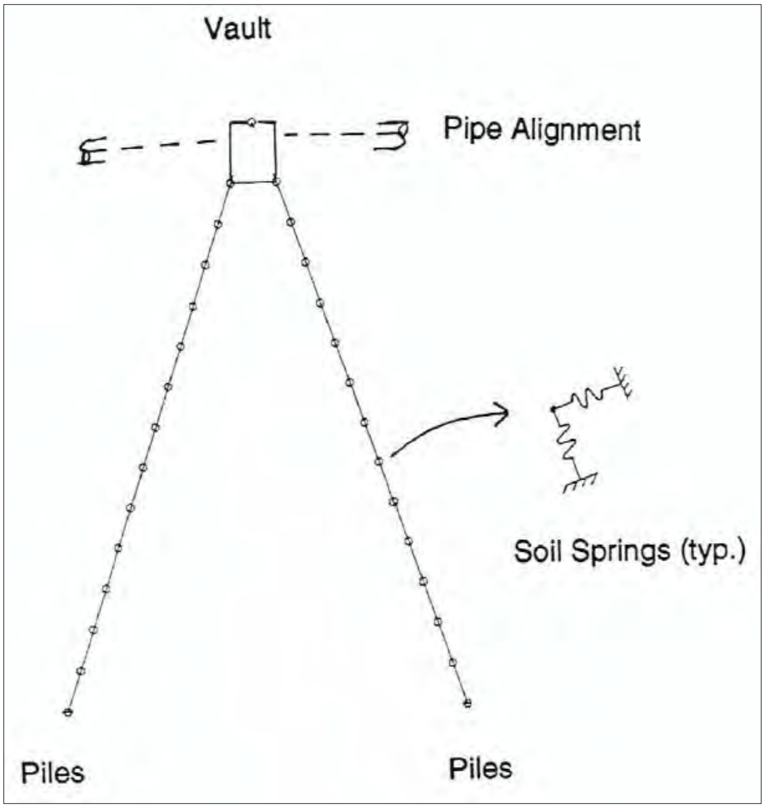


Figure 5-7. Bay Farm #2 Crossing: Vault / Anchor Model

The relative displacements for checking possible binding of the flanged coupling with the vault/anchor wall due to relative coupling-to-vault/anchor movement were computed and compared to the limit state (allowable) value of 20 inches in Table 5-18.

The relative displacement to allowable value ratios indicates that the coupling may touch the vault/anchor wall only under the Hayward M7 earthquake.

Table 5-18. Bay Farm #2 Crossing: Vaults

Earthquake	Bay Farm Island Side (% of allowable)	Alameda Side (% of allowable)
Hayward M 7	110%	90%
Hayward M 6	10%	0%
Calaveras M 6.75	10%	0%
Concord M 6.5	10%	0%

Summary

The submarine pipeline extending between the vault/anchors on each side of the channel is not likely susceptible to damage under any of the earthquake scenarios. The computed pipe strains are below the limit state strain defining incipient pipe wall wrinkling. The flange coupling slip joints inside the vaults/anchors are not likely to exceed their working expansion displacement during any of the earthquake scenarios. Interference (or binding) between the vault/anchor walls and the flanged coupling due to relative vault-to-pipe movements is possible only under the Hayward magnitude 7 earthquake. The analyses using the best estimates for the Hayward magnitude 7 PGD (24 inches) indicates that the interference is minor and may not cause sufficient damage to result with leakage. For larger PGDs (i.e., 48 inches), damage causing leakage is likely. Damage estimates are made in Table 5-19.

Table 5-19. Bay Farm #2 Crossing Damage Estimates

Earthquake	Chance of Pipe Failure
Hayward M 7	~ 10-50% chance that binding of the coupling in vault/anchor could lead to pipe failure (either side)
Hayward M 6	negligible chance of pipe failure
Calaveras M 6.75	negligible chance of pipe failure
Concord M 6.5	negligible chance of pipe failure

5.1.4 Park Street Crossing

The primary potential for earthquake failure is assumed to be seismic wave propagation. Based upon this assumption, there is about a 5% to 10% chance of failure of this pipe due to the Hayward magnitude 7 scenario earthquake. There is about a 2% to 5% chance of failure due to the Hayward magnitude 6 scenario earthquake. There is about a 1% chance of failure due to the Calaveras magnitude 6.75 scenario earthquake. There is about a 0.5% chance of failure due to the Concord magnitude 6.5 scenario earthquake.

It should be noted that these probabilities of failure can increase substantially if local soil conditions are different than assumed. Given the high levels of PGA and long shaking durations for a Hayward M 7 event, and the possibility that there are some loose sand layers, the potential for damage in the Hayward M7 event may be somewhat higher than listed above.

The borings in Figures B-7, B-8, and B-9 classify the soils beneath the pipe as "sand," but the density is not specified. Merritt sands are considered to generally have a low probability of liquefaction. Overall, we think there is low chance of liquefaction for the submarine portion (except locally at Boring 2), and some chance of settlement due to liquefaction for the Alameda Island approach pipeline.

Lacking modern soil corrosivity tests, we assume that the sandy nature of the surrounding soils makes for relatively moderate corrosion susceptibility along this alignment.

5.1.5 Broadway to Derby Avenue, Closed

For seismic evaluation of the submarine pipeline, soil conditions are inferred based upon firm soil conditions for the submarine portion. The submarine portion is considered to have low probability of liquefaction. Therefore, the potential for earthquake failure is assumed to be seismic wave propagation for the submarine portion.

Based upon these assumptions, there is about a 5% to 10% chance of failure of the submarine pipeline due to the Hayward magnitude 7 scenario earthquake. There is about a 2% to 5% chance of failure due to the Hayward magnitude 6 scenario earthquake. There is about a 1% chance of failure due to the Calaveras magnitude 6.75 scenario earthquake. There is about a 0.5% chance of failure due to the Concord magnitude 6.5 scenario earthquake.

Liquefaction damage to the approaches (about 30 feet of length on either shoreline) cannot be ruled out. It is possible that the existing slip joints could accommodate small movements to the shoreline side of the concrete anchor blocks, but the pipe behind the blocks (landward side) might then be pulled out. Should such damage occur, it could be repaired using conventional methods.

The cause of the pipe leak in 2009 is uncertain. It may be due to long term corrosion (graphitization) of the cast iron pipe. It may be due to accumulated stress in the pipe from differential settlements under the estuary, coupled with long term wall thinning, leading to a hoop pressure failure, or joint failure at a ball joint or push-on / flanged joint (type uncertain).

5.1.6 Bay Farm Island #1 Bridge, Closed

The seismic performance of the 16-inch pipeline is dependent upon several factors:

How well the bridge performs. The bridge is on piles that go through Young Bay Mud, into older stiff blue clays below. Lacking detailed analyses, it is thought that the bridge will be flexible, with large seismic deflections (several inches) at the road deck level. The decks of the bridge have been seismically upgraded, which should limit the chance that the decks will deflect laterally far enough to collapse into the water below, but which do not keep the decks from still undergoing significant (many inches) of relative deck-to-deck displacements. It would have to be checked to confirm that the existing dresser couplings can sustain these displacements; likely, they cannot.

Transition from the at-grade pipe (either side) to the bridge. These will be highly loaded points on the pipe, and a substantial flexible connection would be needed to accommodate the differential movements; local bends in the pipe (as well as attachments to the 24-inch No. 2 Bay Farm Island pipe) will act as anchors, attracting high loads should there be any lateral spreading of the shorelines.

Transition from the hanging pipe to the submarine crossing. Hydraulic-imposed water thrust forces (static and hydrodynamic) at this corner will be large, and the pipe would have to be checked to see if it able to accommodate these forces. The pipe would have to accommodate the seismic movements of the bridge at this location.

The underwater portion of the pipeline, being on stiff clays, should be satisfactory, presuming no impact of corrosion since the pipe was originally installed.

For this effort, a detailed evaluation of the Bay Farm Island Bridge was not performed. It is assumed that the shoreline lateral soil permanent ground deformations which affect the nearby buried San Leandro Bay crossing (about 2 feet in a Hayward magnitude 7 earthquake) will similarly affect the bridge. Thus, there is high likelihood that the pipe on the at-grade-to-bridge transitions will break in a Hayward magnitude 7 earthquake.

The smaller soil lateral displacements in the other three scenario earthquakes (up to about 3 inches) will lessen, but not eliminate the probability of failure of the pipe, especially at the transition from the shoreline to the pile-supported bridge. It is expected that the pipe will break or seriously leak at coupling locations due to the Hayward M 6 and possibly the Calaveras M 6.75 scenario earthquakes, although this damage should be easily repaired in a short amount of time.

5.1.7 High Street, Closed

It is believed that this pipe was damaged in the 1989 earthquake and was subsequently valved out and remains closed. Given the vintage of the pipe (nearly 100 years old) and the presumed highly corrosive nature of the local soils, it would not be surprising if the pipe has been "corroded" from the outside (graphitized) and undergone some amount of tuberculation from the inside. While the submarine portion might be vulnerable, the approaches, lacking liquefaction, should not be especially vulnerable to earthquake shaking.

5.2 EBMUD Pipe Damage: 1989 Loma Prieta

Figures 5-8 and 5-9 show maps with the locations of damaged pipelines in the District’s system as a result of the 1989 Loma Prieta earthquake. In Figure 5-8, the background shaking indicates areas with high corrosivity soils (light stipples, near the coastline of the Bay), moderate corrosivity (darker stippling, generally somewhat inland from the Bay), and low corrosivity (clear areas, generally inland).

In Figure 5-9, the dark background indicates Young Bay Muds / fills; the darker stippling indicates younger alluvial soils; the light stippling indicates older alluvial soils / Merritt sands; the clear areas indicate rock; the lines show the District’s pipe grid in the central pressure zone (thickest lines being pipes up to 36-inch diameter); the heavy black line on the right (east) shows the location of the Hayward fault.

There were a total of 113 pipe mains and 22 service lateral repairs (135 total repairs) due to the 1989 Loma Prieta earthquake. As can be seen in Figures 5-8 and 5-9, the bulk of the repairs were concentrated in areas underlain by Young Bay Mud and artificial fills, which are generally highly corrosive. In a future Hayward M 7 earthquake, the total number of pipe repairs may total 3,000 or more; the concentration of the repairs will likely show a pattern similar to that seen in Figures 5-8 and 5-9 (heavy concentrations of damage in fill and highly corrosive soils), as well as other damage due to triggered landslides or fault offsets.

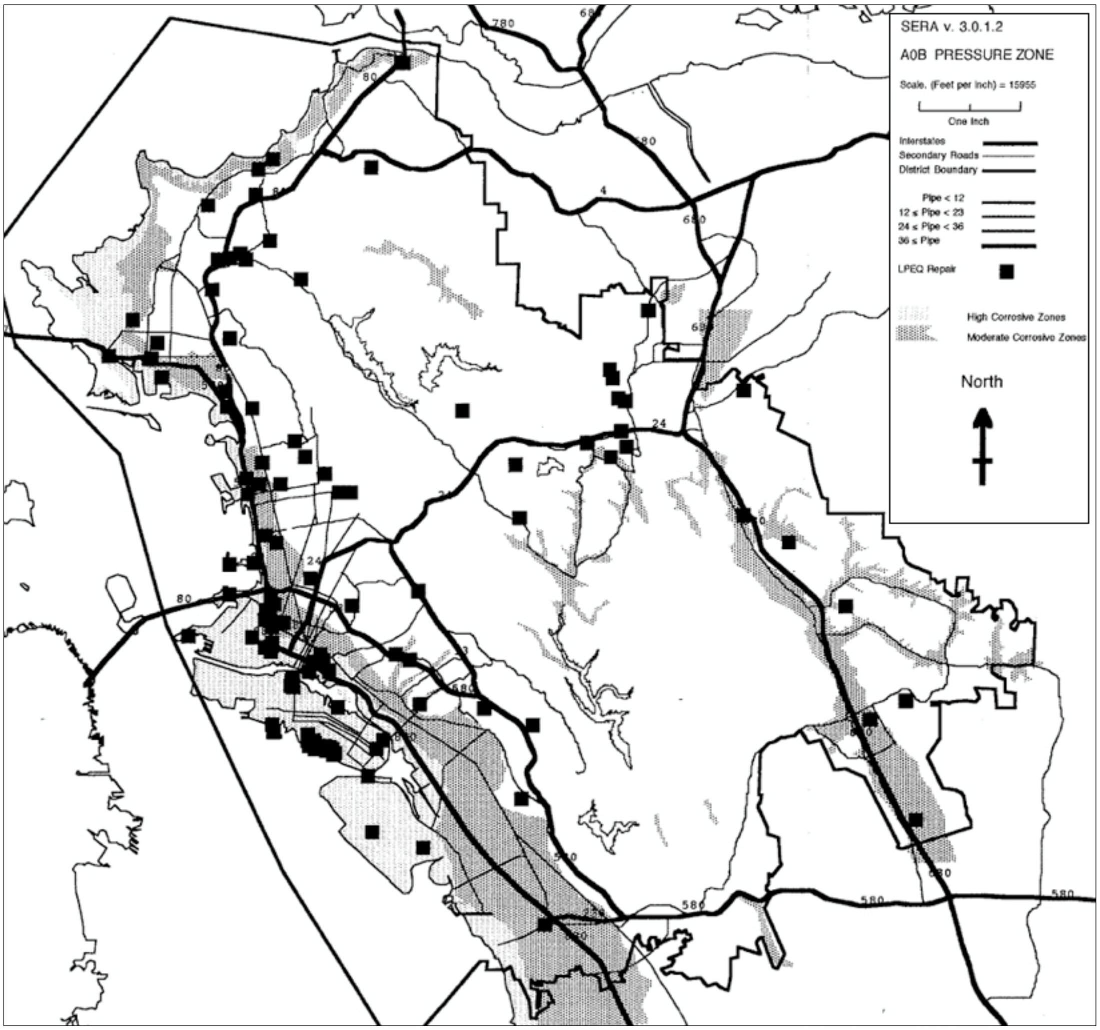


Figure 5-8. EBMUD Pipe Damage: 1989 Loma Prieta Earthquake

As indicated in Figure 5-9, there was a leak on the Bay Farm Island pipeline (believed to be BF#1 pipe on the bridge). There were also several smaller diameter pipes with repairs on Alameda Island near the Posey tube (the 24-inch Alice Street pipeline was not damaged). The heavy concentration of damage to pipes on the south shore of Alameda Island were typically 6-inch welded steel pipes, having poor welds and subjected to corrosive soils.

The lack of damage to the pipe network on North Bay Farm Island in the 1989 Loma Prieta earthquake was attributed to sufficiently small motions that did not trigger liquefaction in that area. But, with a future Hayward M7 event, the motions will be higher, and the most common style of small diameter (6 to 8 inch) pipe in that area, being Asbestos Cement with push in joints, will suffer widespread damage should the soils liquefy (more recently installed small diameter pipes are PVC, and larger diameter pipes, 12" and larger, are lap-welded steel).

Figure 5-9. EBMUD Pipe Damage: 1989 Loma Prieta Earthquake, Near Alameda Island

Figure 5-10 shows a map of the downtown Oakland–Alameda Island area, highlighting locations with observed liquefaction effects from past earthquakes (1906 San Andreas and 1989 Loma Prieta).

Figure 5-10 shows that both ground settlements and lateral spreads have been observed near the Alice Street pipeline crossing location in past earthquakes. The numbers assigned in Figure 5-10 refer to ground failure sites, adapted from Youd and Hoose (1978) and Knudson et al. (2000).

5.3 Proposed Pipeline Crossings

5.3.1 Microtunnel Vulnerabilities

Proposed Alternatives 1A, 2A, and 3A consider the microtunnel style of construction. Microtunnels have the following design features:

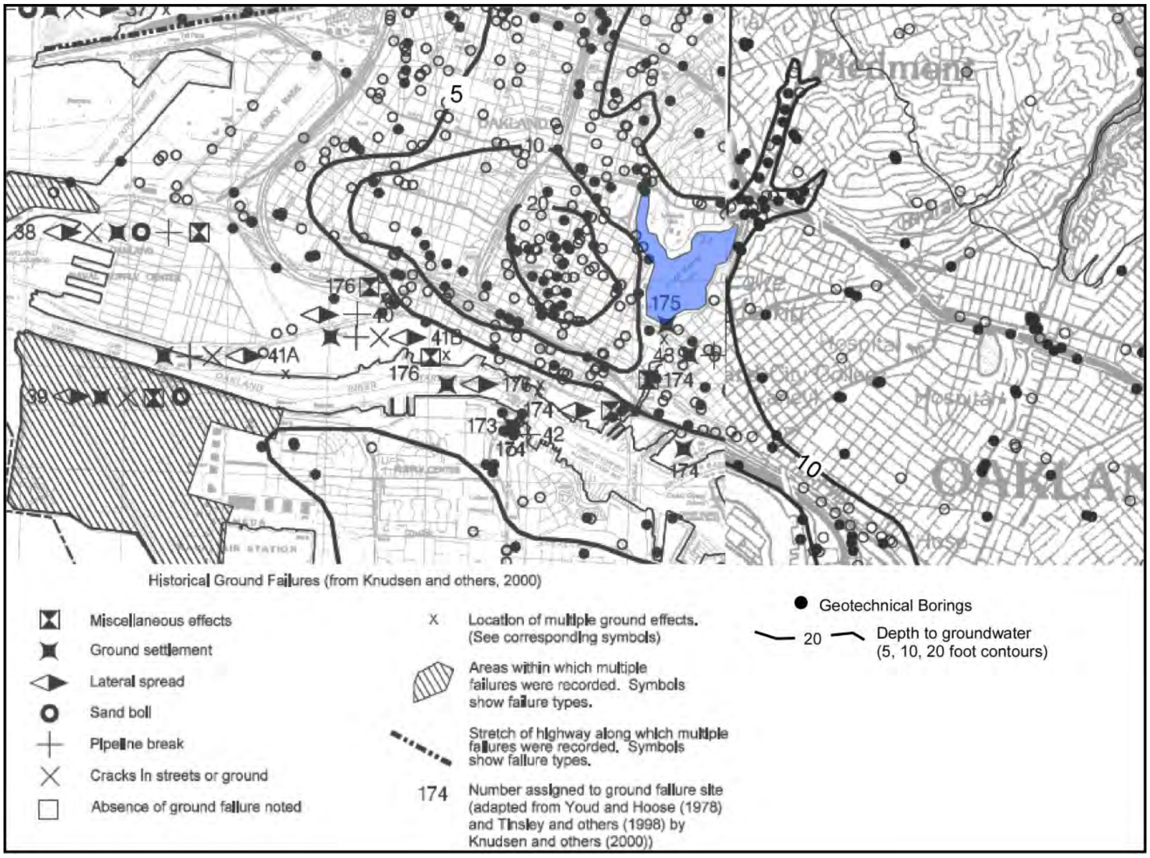


Figure 5-10. Observed Liquefaction Effects

- The construction of the tunnel should be safe in the deeper and stable soils. Free to select the best tunnel horizon.
- Use of a casing pipe will provide additional protection to the water main carrier pipe.
- Grouting between the casing and the carrier pipe can provide corrosion protection to the exterior of the pipe.
- The jacking and receiving shafts that are located at Young Bay Mud sites, (most sites), extend from the deeper stable soils to the top level soft (young bay mud) soils.
- There may be remnants of piles from an old bridge near alignment 2A. Hitting a pile during microtunnel construction can be problematic. Therefore, knowing the precise locations of existing piles (or other obstructions) will be important.

5.3.2 Horizontal Directional Drilling Vulnerabilities

Alternatives 1D, 2A, and 3A consider the horizontal directional drilling (HDD) style of construction. HDD has the following design features:

- Horizontal directionally drilled crossing should be safe being in the deeper and stable soils. This allows the designer to freely select the best tunnel horizon.
- Use of a conductor casing will provide protection to the carrier pipe at the two ends. The conductor casings can be 100 to 200 feet long. At a length of 200 feet and 15-degree entry angle, carrier pipe is protected for the top 50 feet in the Young Bay Mud, avoiding the upper liquefiable zones (see Figure 5-11).
- Robustness can be enhanced by providing jet grout supports beneath the conductor casing that would otherwise be prone to settlement to liquefaction.
- Backfill grouting between the conductor casing and carrier pipe can provide stability in the Young Bay Mud for the top 50 feet.
- Need to consider whether the conductor casings are backfilled with a cement-based grout. Consider using sand as an alternate backfill material.
- There will be a weak point (hard/soft) at the end of the casing and the start of the driller's mud filled HDD hole.
- The entry and exit points of the HDD crossing will be designed to be more than 200 feet from the shoreline, so any weak point at the end of the casing will be onshore and accessible from the surface (see Figure 5-11).
- The HDD crossing on the two sides will be in soft soils and exposed to lateral spread of the channel embankments.
- A potential second weak point exists where the pipe exits the conductor casing and joins with the at-grade pipelines. The connection of the HDD to the at-grade pipe will need to factor potential for differential movements due to PGDs. If a coupling unit is used, the coupling must be sized to accommodate differential movements (likely more than can be accommodated with a standard Dresser-type coupling, suggesting that long throw couplings and possibly ball joints should be used if the ends are located in materials prone to PGDs).

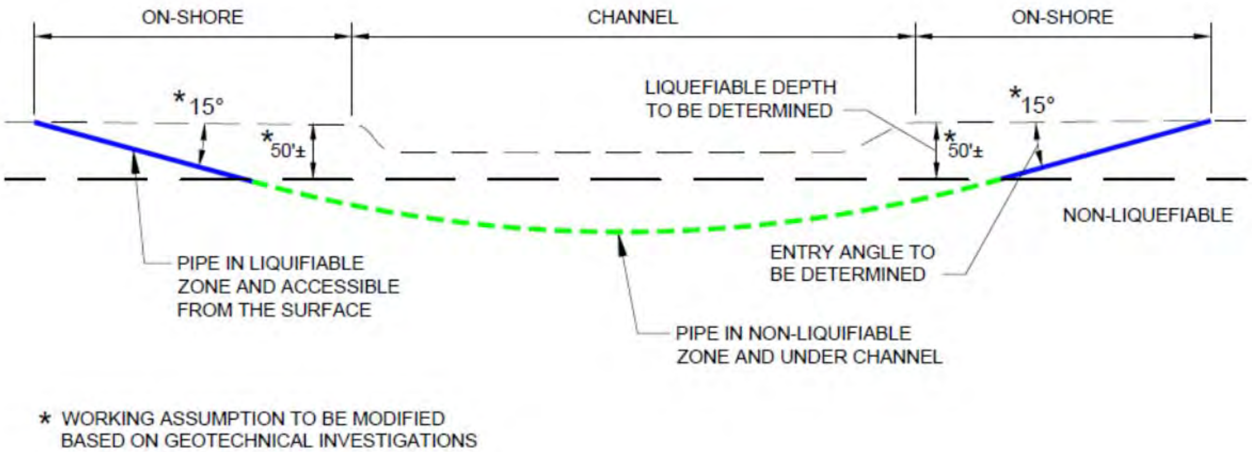


Figure 5-11. Typical Layout to Reduce the Vulnerability of the HDD Pipeline Crossings

5.3.3 Seismic Assessment of New Pipeline Crossings

The alternatives outlined in Sections 5.3.1 and 5.3.2 are assessed for their potential to survive future earthquakes. As the final design details of each alternative are at this time not yet known, these assessments are based on best estimates, and assuming that seismic designs is done for the submarine and approach sections.

Table 5-20 lists the vulnerabilities to the submarine portions due to earthquake failure.[2] We list the probability of failure as "very very low" for all microtunnel alternatives and "very low" for all directionally drilled alternatives, and for all earthquakes. This assumes that the tunnel/directional drill pipes will be founded in competent materials and that the entry/exit points are located outside of, or otherwise designed to accommodate any settlements and lateral spreads that might occur. Should further refinement be needed for purposes of selecting between alternatives, the risk of failure for the Hayward M 6, Calaveras M 6.75, and Concord M 6.5 events can be assumed to be one-half that of the Hayward M 7 event; and the San Andreas M 8 event can be assumed to be similar as for the Hayward M 7 event.

Table 5-20. Probability of Earthquake Failure: New Pipeline Crossings, Submarine Portion

Alternative	Hayward M 7	Hayward M 6	Calaveras M 6.75	Concord M 6.5
1A MT	Very Very Low	Very Very Low	Very Very Low	Very Very Low
1A Long MT	Very Very Low	Very Very Low	Very Very Low	Very Very Low
1D HDD	Very Low	Very Low	Very Low	Very Low
2A MT	Very Very Low	Very Very Low	Very Very Low	Very Very Low
2A HDD	Very Low	Very Low	Very Low	Very Low
3A MT	Very Very Low	Very Very Low	Very Very Low	Very Very Low
3A HDD	Very Very Low	Very Very Low	Very Very Low	Very Very Low

We list the seismic risk as "very very low" for all microtunnel alternatives (about 0.5% chance of failure). We based this on the following factors:

- The tunnel/pipeline alignments will be supported in competent materials.
- There are no known faults through any of the alignments.
- In a survey of 217 tunnels (constructed between 1890 and 1995) that have gone through ten actual earthquakes (1906 San Francisco through 1995 Kobe) (ALA 2001), each having been exposed to ground motions with PGA > 0.10g, only 4 have been materially damaged because of ground shaking effects.
- Bored tunnels (38) with reinforced concrete or steel liners have suffered, at most, small cracks in the concrete (no collapses). Any microtunnel option described in this report is thought to be at least as capable (if not better) than a reinforced concrete liner system.

[2] Risk during construction is not addressed in Table 5-20.

- Of the 4 tunnels with material damage, 3 reached this damage state due to landslides or poor geologic conditions along the alignment. Landslides are not applicable to the microtunnel or HDD options considered in this report. All microtunnel or HDD options place the pipe within/supported by competent materials (old bay clay or similar formations).
- The one tunnel of the survey that was heavily damaged was a 76-meter-long and 6-meter-wide road tunnel, where about 16 meters of the liner collapsed in the center of the tunnel due to a M6.8 offshore earthquake. The failure was attributed to ground deterioration around the tunnel that had progressed over a long period of time, with voids already existing behind the liner at the time of the earthquake. These voids allowed high stressed in to the concrete lining, leading to the failure.

Considering these factors, it is unlikely that even very strong ground shaking (PGAs up to 50 inches/sec, PGAs up to 0.68g) will collapse the carrier pipe within the casing (microtunnel system) or seriously distort the directional drilled carrier pipe in the submarine sections.

The probability of failure for the HDD alternatives is very low (less than 1% for the submarine portion). This assumes that the HDD would use a single HDPE carrier pipe in the submarine portion. The carrier pipe extending from the submarine portion to the surface through the near surface soils will be housed in a casing. It is assumed the carrier pipe will be grouted and protected by the casing. There is uncertainty as to whether or not the pulling of the HDPE carrier pipe through the hole will accidently scrape the exterior of the pipe. To protect against scraping, the wall thickness of the pipe will be increased, using a SDR of 9 or 11. Additional HDD options, including separate casing and carrier pipes, are discussed in Section 7.

Table 5-21 lists the probability of failure of the new approach pipes for each alternative. Alternative 1A Long MT includes completely new approach pipes. All other alternatives include short new approach pipelines (seismically designed) to attach to existing pipelines.

Table 5-21. Probability of Earthquake Failure: New Pipeline Crossings, New Approach Portion

Alternative	Hayward M 7	Hayward M 6	Calaveras M 6.75	Concord M 6.5
1A MT	Very Low	Very Low	Very Low	Very Low
1A Long MT	Very Very Low	Very Very Low	Very Very Low	Very Very Low
1D HDD	Very Low	Very Low	Very Low	Very Low
2A MT	Very Low	Very Low	Very Low	Very Low
2A HDD	Very Low	Very Low	Very Low	Very Low
3A MT	Very Low	Very Low	Very Low	Very Low
3A HDD	Very Low	Very Low	Very Low	Very Low

Table 5-22 lists the probability of failure of the existing approach pipelines for each alternative. All alternatives include a transition piece to the existing pipe network. The existing pipe considered in Table 5-22 is that portion of pipe that is located in Young Bay Mud as indicated by the dark shaded areas in Figure 1-2.

Table 5-22. Probability of Earthquake Failure: New Pipeline Crossings, Existing Approach Portion

Alternative	Hayward M 7	Hayward M 6	Calaveras M 6.75	Concord M 6.5
1A MT	Very high	High	High	High
1D HDD	Very high	High	High	High
2A MT	Moderate	Low	Low	Low
2A HDD	Moderate	Low	Low	Low
3A MT	Low	Low	Low	Low
3A HDD	Low	Low	Low	Low

The following describes the key issues associated with the approach pipelines:

- **Alternative 1A MT.** The approach pipelines are essentially unchanged (slightly shorter) from the existing approach pipelines. They have essentially the same risk as in Table 5-2.
- **Alternative 1A Long MT.** The existing approach pipelines are replaced (or paralleled) with new seismically-designed HDPE (or heavy wall steel) pipes, that are designed to accommodate PGDs. The seismic risk for the new pipelines is very low.
- **Alternative 1D HDD.** The existing approach pipelines are presumed to be mostly retained on the Alameda Island and Oakland sides, with new pipes (seismically designed) from the drill entry/exit points built to the existing pipelines.
- **Alternative 2A MT.** The connections to existing pipelines are likely still in Young Bay Mud zones. Until the existing pipelines are upgraded / replaced, there remains some risk that they will be damaged in future large earthquakes.
- **Alternative 2A HDD.** The connections to existing pipelines are likely still in Young Bay Mud zones. Until the existing pipes are upgraded / replaced, there remains some risk that they will be damaged in future large earthquakes.
- **Alternative 3A MT.** The connections to existing pipelines are likely in competent soils. As the existing pipelines are aging and may use non-seismic styles of construction, their seismic risk is low if they are exposed only to strong ground shaking (no PGDs).
- **Alternative 3A HDD.** The connections to existing pipelines are likely in competent soils. As the existing pipelines are aging and may use non-seismic styles of construction, their seismic risk is low if they are exposed only to strong ground shaking (no PGDs).

6 Risk Analysis Summary

6.1 Existing Pipeline Crossings

Table 6-1 summarizes the potential for existing pipeline failure due to earthquake in the submarine portion, based on the analysis from Section 5. Generally, such damage will mean that repair efforts can takes weeks to several months, once the District has mobilized specialized crews and equipment. The quantified meanings of the terms "Very high, high, etc." follow Table 6-1, and are used throughout Section 6.

While we have not formally quantified the results for a nearby San Andreas M 8.0 event, for purposes of the current discussion, it would be reasonable to assume similar performance as for a Hayward M 7 event.

Table 6-1. Probability of Earthquake Failure: Existing Pipeline Crossings, Submarine Portion

Crossing	Hayward M 7	Hayward M 6	Calaveras M 6.75	Concord M 6.5
1. Alice 24-inch	High	Moderate	Moderate	Low
2. Oak (Blanding) 24-inch	Low	Low	Low	Low
3. Bay Farm #2 24-inch	Moderate	Low	Low	Low
4. Park 16-inch	Low	Very Low	Very Low	Very Low
5. Broadway to Derby, 20" (currently damaged, closed)	Low	Low	Very Low	Very Low
6. Bay Farm #1 16-inch (channel portion) (currently closed)	Moderate	Low	Low	Low
7. High Street 12" (currently closed)	Low-Moderate	Low	Low	Low

Very high: 80% to 100%; High: 60% to 80%; Moderate: 40% to 60%; Low-Moderate: 20% to 40%; Low: Under 20%; Very low: Under 5%.

Note: The "very low" designation is only used where the evaluations clearly show a very low chance of failure.

Table 6-2 lists the potential for damage to the approach sections of each pipeline crossing due to earthquake. Generally, should the pipe be damaged in the approach section, but not in the submarine section, we believe that the District can repair the pipe using District forces, using conventional cut-and-cover type of efforts (including trench shields, etc.).

Table 6-3 provides risk elements due to ongoing pipeline aging processes. This includes the effects of corrosion (graphitization) and existing un-repaired pipe damage. Table 6-3 contemplates a 50-year time window (through the year 2064). Table 6-3 only addresses the submarine portions, as it is assumed that the approach sections, if broken, can be repaired using District forces using conventional methods, over a short period (commonly 1 day), without leading to widespread water outages (requires that at least one parallel pipe remain in service while repairs are made).

Table 6-2. Probability of Earthquake Failure: Existing Pipeline Crossings, Approach Portions

Crossing	Hayward M 7	Hayward M 6	Calaveras M 6.75	Concord M 6.5
1. Alice 24-inch	Very high	High	High	High
2. Oak (Blanding) 24-inch	Low	Low	Low	Low
3. Bay Farm #2 24-inch	Moderate	Low	Low	Low
4. Park 16-inch	Low - Moderate	Low	Low	Low
5. Broadway to Derby, 20" (currently closed)	Low - Moderate	Low	Low	Low
6. Bay Farm #1, 16-inch: Bridge portion (currently closed)	High	Moderate	Low to Moderate	Low to Moderate
7. High Street, 12-inch (currently closed)	Low to Moderate	Low	Low	Low

Table 6-3. Probability of Non-Earthquake Failure: Existing Pipeline Crossings, Submarine Portions

Crossing	Existing Condition (2014)	Corrosion Potential (submarine portion)	Potential for Submarine Break / Failure by the year 2064 (without major earthquake)
1. Alice, 24-inch	In Service	Very high	Moderate
2. Oak (Blanding), 24-inch	In Service	Moderate	Low
3. Bay Farm #2, 24-inch	In Service	Moderate	Low
4. Park 16-inch	In Service	Moderate	Moderate
5. Broadway to Derby, 20-inch	Closed – underwater break	Moderate to High	Already Occurred
6. Bay Farm #1, 16-inch	Closed – unknown reason	Moderate	Uncertain
7. High Street, 12-inch	Closed – break	High	Already Occurred

Under the column "Potential for Submarine Break / Failure by the year 2064," we include potential for failure due to:

- Corrosion (graphitization) of the pipe, resulting in pipe wall thinning and thus pipe pressure burst. While we do not have R values along the pipelines, we based this on generalized studies of corrosion for cast iron and steel pipes; an assumed high ground water table; the nature of the soils surrounding the pipe (clays being more corrosive, sands being less corrosive); the original pipe wall thickness (presuming a normal operating pressure of about 80 psi); the current age of the pipe; this history of pipe repairs to other District pipes in the vicinity.
- Ongoing soil settlement (non-earthquake).

For all pipelines, the potential for pipe damage due to dragging of ship anchors, dredging operations, fishing activities, etc., is assumed to be rather low, owing to several feet (at least) of pipe cover. However, we do not have video of the pipe alignments to confirm that the original depth of pipe cover has not been reduced or otherwise compromised.

Table 6-4 describes the potential impacts should a single pipeline be closed due to earthquake or non-earthquake reasons. Under "earthquake," we presume a Hayward M 7 event, with concurrent damage to other pipelines.

Table 6-4. System Impact of Pipe Damage: System Redundancies, Hydraulic Issues

Crossing	Earthquake	Non-Earthquake
1. Alice, 24-inch	High. Should the Oak pipeline survive, this would likely result in flow and pressure impacts to Alameda Island, especially the western portion.	Moderate. Increased flows via Oak and Park may result in pressure drops in Oakland (modest) and western Alameda Island. A large concurrent fire in western Alameda Island might have restricted flows.
2. Oak (Blanding), 24-inch	Very High. Any earthquake damage to Oak would also likely damage Alice. Then only Park would be available, and it too might be compromised. While this event is unlikely, should it occur, there is potential for a very long and widespread outage to Alameda Island.	Low. Adjacent Park and Alice Street pipelines should be adequate for short term operation until repairs are made.
3. Bay Farm #2, 24-inch	Very High. Currently, the only open pipeline to North Bay Farm Island. Assumes damage to the on-shore pipeline from the Oakland Airport.	Low. Currently, the only existing on-shore pipeline by Oakland Airport can serve North Bay Farm Island.
4. Park, 16-inch	Moderate, as long as adjacent Oak street pipeline remains in service.	Low. Adjacent Oak and Alice Street pipelines should be adequate for short term operation until repairs are made.
5. Broadway to Derby, 20-inch	Low. Already closed with no known major operational issues.	Low. Already closed with no known major operational issues.
6. Bay Farm #1, 16-inch	Low. Already closed with no known major operational issues.	Low. Already closed with no known major operational issues.
7. High Street,12-inch	Low. Already closed with no known major operational issues.	Low. Already closed with no known major operational issues.

6.2 Proposed Pipeline Crossings

6.2.1 Risk Due to Earthquake

Section 5.3 discusses in detail the potential for future pipe failure, given four possible earthquakes.

6.3 Summary of Risk Register

Risks were identified and quantified for each of the four proposed pipeline crossing alignments. A detailed risk register for the alignments is provided in Appendix G. These risks were scored in terms of both the likelihood and potential impact of occurrence. In addition, design measures and construction measures were provided for management/reduction of risk. The level of risk was quantified for mitigated and unmitigated conditions. Included in the risk assessment is a description of each alternative's long-term risk due to catastrophes such as earthquakes.

The four proposed pipeline crossing alignments had similar unmitigated scores and mitigated scores for a number of risk/hazard scenarios. However, it should be noted that soils at the 1A, 1D, and 2D alignments are subject to liquefaction. These soils consist of saturated loose sands and soft silts within artificial fills and within Young Bay Mud. Alignment 3A has no Young Bay Muds and is the native Merritt Sands.

6.3.1 Alignment 1A Risk Register

Alternative 1A is in the general vicinity of the District's 24-inch diameter Alice-Webster water pipeline that was constructed in 1946, and its Third Alameda Interceptor Siphon. Unlike the other alignments, construction will require many utility relocations both before and after mitigation (risk scores of 20 before and 12 after[3]; compared to 25 before and 6 after for other 3 alignments). Both 1A and 1D have an unmitigated risk score of 15 for construction impacts adjacent to railroads, and a mitigated score of 8. In comparison, Alignments 2A and 3A have unmitigated risk scores of 1, so no mitigation was recommended. For the risk of construction impacting the San Francisco Bay Trail, both 1A and 1D have an unmitigated risk score of 12 and a mitigated risk score of 8. By comparison, Alignments 2A and 3A have unmitigated risk scores of 1, so no mitigation was recommended.

6.3.2 Alignment 1D Risk Register

The location of Alternative 1D is in the general vicinity of (1) the District's 24-inch diameter Alice-Webster water pipeline, (2) the District's Third Alameda Interceptor Siphon, and (3) Alameda Municipal Power's planned 2013 Alameda to Coast Guard Island HDPE Conduit Crossing. See Section 5.3.1 above for a general comparison of 1A and 1D risk scores with other alignments. In addition, Alignment 1D and 2A have an unmitigated risk score of 16 and mitigated risk score of 4 for future development or improvements impairing access to new water lines. In comparison, 1A and 3A have scores of 8 and 4 for unmitigated vs. mitigated.

6.3.3 Alignment 2A Risk Register

The location of Alternative 2A is in the general vicinity of (1) the District's Estuary to Bay Farm Island water pipeline project and (2) the Bay Farm #2 water pipeline. See Section 5.3.2 above for a general comparison of 1D and 2A risk scores with other alignments. In addition, as discussed in Section 5.3.1 above, both 2A and 3A have an unmitigated risk score of 1 for construction impacts adjacent to railroads,

[3] Risk scores range from 25–12 (high); to 11–5 (medium), to 4–1 (low).

so no mitigation was recommended. For the risk of construction impacting the San Francisco Bay Trail, both 2A and 3A have an unmitigated risk score of 1, so no mitigation was recommended.

6.3.4 Alignment 3A Risk Register

The location of Alternative 3A is in the general vicinity of the District's Derby Avenue water pipeline. See Section 5.3.3 above for a general comparison of 2A and 3A risk scores with other alignments. A historical review of the area indicates contaminated ground conditions, specifically hydrocarbons from the old Shell Oil storage tanks on the Oakland side. To mitigate hitting the contaminated ground, the jacking shaft was moved to the east. The entry point for the HDD was also moved to east to allow the HDD distance to get deeper and below the contamination.

7 Mitigation Methods for Reducing Pipeline Vulnerabilities

7.1 Existing Pipeline Crossings

Mitigation methods were provided for all pipeline crossings except for Oak Street and High Street.

7.1.1 Alice Street Channel Pipeline Crossing

The Alice Street pipeline crossing has a significant likelihood of damage during most of the scenario earthquakes. The damage would probably cause leakage and/or breaks that would take the pipeline crossing out of operation. Key factors contributing to the crossing vulnerability are:

- Soil conditions consisting of fill materials along either shoreline of the estuary are susceptible to large PGDs, and
- The approach pipelines consist of segmented cast iron pipe which can suffer brittle failure—joint pull-out. The submarine pipeline is expected to fail due to lock-up of the Usiflex joints.

Several options for improving the reliability of the existing Alice Street pipeline crossing are as follows.

- Soil Improvement. The purpose is to reduce the probability and magnitude of earthquake-induced PGDs at the pipeline crossing. This would involve soil replacement and/or densification. However, soil improvement is not likely feasible due to the large extent of soil that would require improvement; nor does it address pipe aging issues.
- Pipe Replacement. Heavy wall and corrosion protected welded steel pipe generally has better seismic performance versus cast iron pipe. However, under the relatively large PGDs postulated for this site, the integrity of thin-walled steel pipe (as commonly used by the District) could not be assured if the pipe is simply placed in the same trench as the old cast iron pipe. Instead, a heavy wall (about 0.5 inch) butt welded steel pipe, with suitable corrosion protection, would likely be required, and the trench would require a combination of CDF (CLSM) and sand layers for anchorage purposes, to protect both the new and non-replaced original pipe. Due to the high levels of potential PGDs (over 5 feet), the use of "chained" ductile iron pipe (such as manufactured by Kubota of Japan) might be considered, as part of the final design process, but may be impractical due to cost.

The following additional recommendations are made.

- Locate and test the operation of valves that can isolate the existing Alice Street 24-inch pipeline crossing from the supply network.
- Assess the flow demands to western Alameda Island with the Alice Street pipeline crossing out of operation. The purpose is to determine if certain minimum levels of water supply can be provided if the Alice Street pipeline crossing has to be shutdown, until it could be repaired or replaced. Assuming that the Alice Street pipeline crossing and those portions of the distribution system

underlain by former tidal flats are out of service, model to determine whether the remaining water network in Alameda Island can sustain suitable flows and pressures.

- Should additional water supply be required, then plan for a temporary supply pipeline to be constructed through the Posey tube. If this is not acceptable, then improvements in the Alameda distribution pipe network (to reduce head loss to acceptable levels) might be required to allow water to flow to western Alameda Island from the undamaged water feeds at the eastern end of the island (for example, via the Oak and Park Street crossings).

7.1.2 Bay Farm #2 Crossing

The seismic evaluation in Section 5.1.3 suggests that the existing pipeline is marginally acceptable for a large magnitude earthquake on the Hayward fault. Should the pipeline be damaged, the most likely location is at the vaults at either shoreline, possibly due to pinching / damage to the expansion joints. Given this, the District has two choices for the existing pipeline crossing:

- Do nothing. This is the lowest cost choice. Before adopting this choice, we recommend that the District confirm the seismic capacity of the Bay Farm #1 pipeline on the Bay Farm Island Bridge, as otherwise, the entire North Bay Farm Island community has the potential to lose water service during any outage required for repairs. For damage in the vaults, outage time might be on the order of 1 day once the District mobilizes a repair crew.
- Upgrade the vaults / confirm operable valves. The evaluation suggests that the pipeline might bind up at the existing vaults either side of the channel crossing. This could be potentially mitigated by modifying the vault / slip joint / vault walls to accommodate up to 4 feet of movement. Also, confirm that the isolation valves for BF #1 crossing are operable, and that any damage to BF #1 pipeline would not impact the operability of these valves, nor the approach pipelines that serve both BF #1 and Bay Farm #2 pipelines. These upgrades might be implemented as part of a new parallel third crossing, so that there would be two reliable pipelines to North Bay Farm Island.

7.1.3 Park Street Crossing

The seismic evaluation in Section 5.1.4 suggests that the existing pipeline is likely acceptable for a large magnitude earthquake on the Hayward fault.

For additional reliability, the District has two choices for the existing pipeline:

- Do nothing. This is the lowest cost choice.
- Do a modern soil boring for the approach section on Alameda Island, and assess the liquefaction potential. Obtain measurements of soil resistivity to be used for ongoing corrosion evaluations.

7.1.4 Broadway to Derby Avenue, 20-inch, Closed

Repair of the pipe will be expensive. Given the pipe age, it is unclear if single-location repair is the right long term solution. The seismic evaluation in Section 5.1.5 suggests that the existing submarine pipeline is likely acceptable for a large magnitude earthquake on the Hayward fault.

However, the pipe is already damaged in the submarine portion. For additional reliability, the District has two choices for the existing pipeline:

- Do nothing. This is the lowest cost choice.
- Repair the submarine portion. First, do a modern soil boring for the approach section on the Oakland and Alameda Island sides, assess the liquefaction potential, and obtain measurements of soil resistivity to be used for corrosion evaluations. Then, insert a ~16-inch HDPE liner (OD about 20 inches) within the existing pipeline. This is doable if the submarine damage is not too extensive and the changes in direction along the alignment permit this type of effort.

7.1.5 Bay Farm Island #1 Bridge, Closed

The seismic evaluation in Section 5.1.6 suggests that the existing 16-inch submarine pipe is likely acceptable for a large magnitude earthquake on the Hayward fault. However, the pipeline is already closed, for unknown reasons. For additional reliability, the District has three choices for the existing pipeline:

- Do nothing. This is the lowest cost choice.
- Make repairs, as needed, for the existing pipeline. Assuming the damage is due to leaking pipes under the bridge, this damage can be corrected as part of the seismic upgrades described in the next bullet.
- Make seismic upgrades. This will require a coupled bridge-pipe seismic evaluation, and the development of suitable upgrade details. Likely, this will involve installation of more substantial pipe anchors (every other span) to force the pipe to move with the bridge decks (or energy absorbers to limit forces into the bridge); replacement of existing dresser couplings with long-throw couplings (10 inch expansion / contraction); installation of suitable ball and expansion joint couplings at either end of the bridge; correction of existing paint failures along the exposed pipeline (check to see if the paint has lead; if yes, use a suitable encapsulation paint system).

7.2 Proposed Pipeline Crossing Alignments

For three of the four alignments there is high risk from liquefaction-induced ground movements caused by alignments crossing or locating within soils subject to significant strength loss during an earthquake. Mitigation methods are preconstruction (in design phase), and include:

- Conduct study to identify zones susceptible to liquefaction (exploration, site characterization, analysis, etc.)
- Locate alignments in soils that are not susceptible to liquefaction

- Use ground improvement to reduce liquefaction potential
- Design pipe to minimize damage and facilitate repair
- Use flexible pipe material (e.g. steel, HDPE, fusible PVC) which can accommodate larger strains
- Use a two pass system with the carrier pipe within a casing
- Deepen the crossing to soils less prone to ground movement

In addition, below are specific mitigation methods to reduce vulnerability in the proposed alignments, for the construction option(s) recommended, as well as the failure mechanism.

7.2.1 Microtunneling

Even though the chance of failure of the submarine portion of the microtunnels is listed as “very very low” (Table 5-20), it is near zero. For the microtunnel alternatives, using a 48-inch inside diameter casing and inside include the carrier pipe. It might be possible to make repairs, as outlined below:

- Isolate the approach pipelines.
- Pump out the tunnel. If the leak rate is under 100 gpm, a sump pump should be able to readily maintain a nearly dry shaft. Redundant sump pump systems would be needed for worker safety.
- Ventilate the tunnel. Redundant ventilation systems may be needed for worker safety.
- Enter the tunnel and use a grouting system to stabilize the collapsed area of the carrier pipe.
- Excavate and rebuild the liner through the failed zone all within the casing.
- Rebuild the carrier pipe through the failed zone.

The estimated time needed to make the repairs would be 3 to 6 months (possible in 2 months if work proceeds smoothly and with good emergency planning to allow mobilization by specialty contractors very soon after the earthquake). The estimated repair cost is $2,000,000.

We think that there would be a high chance (90% or better) that tunnel failures could be repairable. Still, there is some chance (under 10%) that the nature of the tunnel collapse would mean that the tunnel was not repairable.

7.2.1.1 Alignment 1A

Probability of Failure and Expected Failure Mechanism

The upper portions of the shafts would extend through ground prone to liquefaction and lateral spreading during a seismic event. The shaft would be capable of withstanding liquefaction, but it could be damaged depending on shaft interior build out (partial shell or fully concreted) if there was significant lateral spreading of the ground.

Potential consequences are ground movements that are excessive and damage the new pipeline; ground movements cause approach pipelines to sag or deform; approach pipelines becomes nonfunctioning; or shafts or access points are damaged.

Mitigation Method(s)

For microtunneling, the robustness of this crossing can be fortified against liquefaction and lateral spreading by extending the crossing past all of the ground prone to liquefaction and lateral spread. Moving the shafts farther from the shoreline would place more of the shaft in stable ground. To ensure the entire pipeline crossing is in stable ground, the long version of Alignment 1A would require five additional shafts (seven total shafts) to microtunnel about 6,500 feet of casing to provide the most robust pipeline crossing configuration.

7.2.1.2 Alignment 2A

Probability of Failure and Expected Failure Mechanism

Although Alternative 2A conceivably could be constructed by microtunneling, there is the inherent risk that this crossing is obstructed with the buried remnant foundations of a historical bridge within the very same horizontal alignment. The exact locations of the historical bridge foundations would have to be known in order to pick and select a microtunnel alignment through this obstructed corridor.

Mitigation Method(s)

Mitigation methods to consider for this alignment include:

- Conduct extensive exploration program to explore areas of known buried objects (e.g. pot-holing, probing, geophysical methods, etc.);
- Perform detailed research of existing structures which could conflict with alignment;
- Select a deep tunnel horizon below the potential buried objects;
- Require contractor to provide contingency plans for obstruction removal;
- Require contingency in contract for obstruction removal shaft.
- Confirm contractor complies with specifications; implement contingency plan

7.2.1.3 Alignment 3A

Probability of Failure and Expected Failure Mechanism

As Alternative 3A has no Young Bay Muds in either the submarine or approach areas, there is relatively low likelihood of liquefaction and lateral spread along this alignment. There is potential for failure associated with large PGDs. The contaminated ground is not a failure mechanism, most be addressed during design.

Mitigation Method(s)

The general mitigation methods outlined at the beginning of Section 7.2 are recommended for this alignment. The contaminated ground can be avoided by moving the jacking shaft to the east of Glascock Street on Derby Avenue.

7.2.2 HDD

For the horizontal directional drilling alternatives, the chance of failure is under 1% ("very low"), even under very strong ground motions. However, should the pipe fail in the submarine zone, it is likely that it will not be repairable, as the small diameter (24-inch) pipeline would preclude any safe way to inspect and make repairs. Possibly, the repair strategy would be to abandon the pipe in place, and build a new pipe. Possibly, the contractor could try to pull part of the casing pipe, and re-drill the hole. In either case, the cost and time needed to make the repair would approach that of building a new replacement directionally-drilled pipeline.

To further reduce the chance of failure for the HDD alternatives, the HDD construction could include a separate carrier pipe (carrying water) and casing pipe (outside pipe), with the annulus between the two pipes, and the casing pipe and ground, filled with grout. While this is the most robust option, filling the annulus with grout can introduce additional construction risk, as over-pressurization can ovalize the interior carrier pipeline. Assuming the grouting operation is done effectively, then the grout provides an additional barrier for long term corrosion protection, and the combined casing / grout / carrier pipe assembly can provide additional strength to internal and external loading. However, by having the double pipe and grout system, some flexibility of the overall system is lost, and the low friction offered by the residual drilling fluid in the bored hole is lost by the stiffer grout system. Overall, we speculate that the seismic risk is halved, and the potential functional life (excluding seismic) is doubled.

Another option is to install separate casing and carrier pipe, without grouting the annulus between the two, thus allowing the carrier pipe to be removed and replaced or repaired in the event of a future failure. Additionally, the single casing/carrier pipe option could be oversized, thus allowing for future repairs or future installation of a smaller carrier pipe inside the casing.

7.2.2.1 Alignment 1D

Probability of Failure and Expected Failure Mechanism

The low point of the HDD installation would be around El. -110 feet or about 70 feet below the existing channel bottom and 50 feet below future dredging. Although a significant portion of an HDD alignment would be placed in ground not prone to seismic instability, the ends of the alignment would be in ground prone to seismic instability. To stabilize these reaches, ground improvement will be included under the conductor casing on both sides of the crossing to prevent settlement and pipe movement.

Potential consequences are ground movements that are excessive and damage the new pipeline crossing; ground movements causing approach pipelines to sag or deform; pipeline becomes nonfunctioning; or access points are damaged.

Mitigation Method(s)

Conductor casing can be used at the surface to encase the carrier pipe through the weaker materials that are prone to seismic instability. A 200-foot long casing at 15 degree entry angle will protect the carrier pipe within the upper 50 feet of Young Bay Mud. The entry and exit points can be positioned more than 200 feet from the shoreline to allow direct access to the casing from the surface (see Figure 5-11). Even if robustness is built into those sections that rise through the weak ground (i.e., jet grout below surface conductor casing to mitigate settlement), the interface connections with the approach pipelines will still be vulnerable weak points.

7.2.2.2 Alignment 2A

Probability of Failure and Expected Failure Mechanism

The low point of the HDD installation would be around El. -110 feet or about 90 feet below the existing channel bottom. Although a significant portion of an HDD alignment would be placed in ground not prone to seismic instability, the ends of the alignment would transition up in ground prone to seismic instability. To stabilize the entry and exit reaches, ground improvement will be included under the conductor casing on both sides of the pipeline crossing to prevent settlement and pipe movement.

Potential consequences are ground movements that are excessive and damage the new pipeline crossing; ground movements causing approach pipelines to sag or deform; pipeline becomes nonfunctioning; or access points are damaged.

Mitigation Method(s)

Conductor casing can be used at the surface to encase the pipe through the weaker materials that are prone to seismic instability. A 200-foot long casing at 15 degree entry angle will protect the carrier pipe within the upper 50 feet of Young Bay Mud. The entry and exit points can be positioned more than 200 feet from the shoreline to allow direct access to the casing from the surface (see Figure 5-11). Even if robustness is built into those sections that rise through the weak ground (i.e., jet grout below surface conductor casing to mitigate settlement), the interface connections with the approach pipelines will still be vulnerable weak points.

7.2.2.3 Alignment 3A

Probability of Failure and Expected Failure Mechanism

The low point of the HDD installation would be around El. -110 feet or about 90 feet below the existing channel bottom. The entire HDD alignment would be placed in ground not prone to seismic instability. As an added precaution, conductor casing will be used to protect against contaminated ground known to existing at the site. Ground improvement will be included under the conductor casing on both sides of the crossing to prevent settlement and pipe movement.

Mitigation Method(s)

Conductor casing can be used at the surface to encase the pipe through the potentially contaminated ground.

8 Costs

8.1 Introduction

Cost estimates are provided for the repair of some existing pipeline crossings as well as the four proposed alignments. All cost estimates in this report are developed solely for selecting a design approach for possible new pipelines to Alameda Island and North Bay Farm Island. The costs are based on current year (2014) pricing, and an escalation of 3 percent/year should be applied if the cost estimates are extended for budgetary planning of construction in the future. Actual costs for repairs will vary, depending on the style of existing pipeline damage, as well as the actual final design approach. A more detailed cost estimate should be developed as part of the design when the preferred crossing alignment(s) is selected.

8.2 Existing Pipeline Crossings

8.2.1 Alice Street Channel Crossing

8.2.1.1 Repair Cost and Time Estimates

Onshore Approaches

The approach pipeline is in a conventional trench buried mode. Repair to such pipelines can be usually performed by District forces. Table 8-1 describes the process assumed for repair of a leaking or broken 24-inch cast iron pipe, in the approach areas. Costs include District forces, 5-man crew with excavator, dump truck, pick-up track, compactor, other equipment, replacement pipe (pipe, couplings, etc.), and assume full time work (overnight as needed) at the site until the repair is complete. The assumed cost for a single repair event was estimated to be $30,000. The column "clock hours" gives ranges that are thought to be representative of the average, plus or minus one standard deviation. The column "Cost" is for the average repair.

Table 8-1. Alice Street Crossing: Time and Cost to Make One Above-ground Repair

Description	Clock hours	Cost ($2014)
Identification of leak	1 – 12	$0
Mobilization	6 – 10	$2,000
Excavation, isolate valves	3 – 5	$2,000
Cut and remove damaged pipe	2 – 6	$3,000
Install new pipe	8 – 16	$10,000
Disinfect	1 – 3	$1,000
Time to restore water service	21 – 52 (average 32)	$18,000
Backfill	1 – 3	$1,000
Pavement restoration	1 – 4	$1,000
Subtotal		$20,000
Contingency (25%)		$5,000
Engineering, project management (20%)		$5,000
Total		$30,000

Average repair time for each site is about 32 clock hours, working 24-hours per day, or 4 8-hour days (about 130 total repair crew man-hours in the field). Depending on the actual type of damage and availability of crews, equipment and parts, we think the common range of repair times, per site, would vary from 21 to 52 hours. Total repair time for multiple repairs would depend on number of crews and number of work shifts. Using the averages, the total cost and man-hour estimates for repairs to the approach piping are in Table 8-2.

Table 8-2. Repair Costs: Alice Street Crossing, Approach Pipelines

Earthquake	Cost	Man-hours	Notes
Hayward M 7	$570,000	2,470	19 repairs
Hayward M 6	$90,000	390	3 repairs
Calaveras M 6.75	$120,000	520	4 repairs
Concord M 6.5	$90,000	390	3 repairs

Submarine Pipeline Crossing

Repair of the submarine pipeline crossing would be a complex operation that would require outside help and considerable time. Two typical estimates are prepared for the Oakland side. The Deep Section applies to the lower portion of the slope (should the damaged pipe occur near the point where the pipe run is horizontal) and the Shallow Section applies to the upper portion of the slope (should the pipe damage occur near the shoreline). The Shallow Section estimate can be used for the Alameda Island side. Table 8-3 shows the repair strategy; all values are best estimate, and could vary -50% to +100% (plus or minus one standard deviation).

Table 8-3. Alice Street Crossing: Time and Cost to Make One Submarine Repair

Description	Days	Cost ($2013) Deep Section	Cost ($2013) Shallow Section
Identification of leak, retain contractor	28	$0	$0
Mobilization	5	$18,000	$14,000
Sheet piling	10	$120,000	$84,000
Dewater	2	$12,000	$12,000
Excavate and off-haul	5	$25,000	$25,000
Remove damaged pipe, dimension and order replacement pipe	2	$5,000	$5,000
Replacement pipe (District cost)		$120,000	$120,000
Install new pipe (new pipe to be provided by District). 6 weeks	42±	$10,000	$10,000
Backfill	1	$5,000	$5,000
Disinfect	0.25	$2,000	$2,000
Subtotal	95.25±	$317,000	$277,000
Contingency (30% of contractor cost)		$59,100	$47,100
Engineering, project management (25% of contractor cost)		$64,025	$51,025
Total		$440,000	$375,000

The estimated costs (per submarine repair) and time to repair are shown in Table 8-4.

Table 8-4. Repair Effort: Alice Street Crossing, Submarine Section

Location	Estuary	Oakland Side	Alameda Island Side
Water Depth	**Deep Section**	**Shallow Section**	**Shallow Section**
Repair Cost	$440,000	$375,000	$375,000
Repair Time	13 weeks, ±3 weeks	13 weeks, ±3 weeks	13 weeks, ±3 weeks

The basis for the estimates is as follows.

- Repair work is assumed to be done in one eight-hour shift day, five days per week, and no overtime. Materials (except pipe), labor and equipment are assumed to be readily available. Costs to resolve construction conflicts with other utilities and to relocate boat houses on Alameda Island side are not included. All repair work is assumed to be located within 80 feet from shoreline (no barge use). Sheet piling cost is based on two months rental, pull, and salvage.
- Prior to repair, District will prepare design, purchase pipe, select contractor, negotiate contract and isolate the pipeline crossing. Four weeks are included in the Repair Time Estimate for these activities. The District also will obtain all required permits from Agencies having jurisdiction, notify and coordinate with other public utilities where necessary, and obtain rights-of-way for access to the construction site. (In a post-earthquake emergency environment, it is assumed that these permits will be issued in an expedited fashion). Repairs are to be done in kind.
- Three pipe lengths are assumed to need replacement. Therefore, a 40-foot-long by 10-foot-wide trench is used for estimating the sheet piling work.
- U.S. Pipe Company is no longer manufacturing the 24-inch-diameter cast iron pipe (12-foot segments) with Usiflex joints. Usiflex ductile iron extra wall pipe (15° rotation, boltless, 18 foot segments) is used in the estimate. The pipe is manufactured in Birmingham, Alabama and it takes 8 to 12 weeks for delivery. As these pipes come in standard 18-foot (±) lengths, and the actual length needed for replacement will only be known once the damaged pipe is exposed, it may take some time to obtain the correct pipe; it is doubtful that "straightening out" the existing potentially kinked (but otherwise undamaged) pipeline will be practical; special order piece(s) (flanged end pipe, etc.) may be required. The unit price of the pipe is assumed to be $1,000 per foot. Optionally, the District could pre-order this pipe with suitable fittings (albeit with some risk that the pipe will not fit once the old pipe is exposed).
- Repairs are assumed to be designed in a manner so as to be able to retain water pressure so the pipe can be restored to service; but not to mitigate the pipe for future earthquakes.
- Based on the evaluations, the submarine portion could have three damage locations for the Hayward M 7, and one location (possible none or two) for each of the Hayward M 6 and Calaveras M 6.75 scenario earthquakes. With multiple breaks, it may require the contractor to

work under a force account, as the extent of the damage will not be entirely known until the repair work is well underway.

Table 8-5 summarizes the repairs, repair costs, and resulting time needed to restore the pipeline, from end to end, to service. All values are best estimates (average), and could vary lower or higher as indicated earlier.

Table 8-5. Repair Effort: Alice Street Crossing, Submarine Section

Earthquake	Approach Repairs Cost (number)	Submarine Repairs Cost (number)	Total Repairs Cost (number) *Outage time*
Hayward M 7.0	$570,000 (19)	$1,190,000 (3)	$1,760,000 (22) *20 weeks*
Hayward M 6.0	$90,000 (3)	$400,000 (1)	$490,000 (4) *13 weeks*
Calaveras M 6.75	$120,000 (4)	$400,000 (1)	$520,000 (5) *13 weeks*
Concord M 6.5	$90,000 (3)	$0 (0)	$90,000 (3) *1 week*

Alternative Water Source

It might be possible to install a temporary 12-inch diameter portable pipeline within the Webster or Posey tubes should the closure of the Alice Street pipeline crossing prove to result in unacceptable water flow rates / pressures to Alameda Island. Then, a temporary channel crossing could be constructed by routing a pipe (using 12" ultra-large diameter Super Aqueduct, a 12-inch flanged steel or aluminum pipe, or similar) on the pedestrian sidewalk located in the Posey Tube automobile tunnel. The following outlines some considerations for such an emergency temporary pipe in the Posey Tube:

- A 12-inch diameter pipeline is assumed to be the largest pipe that could be located on the 36-inch pedestrian sidewalk. There are existing waterlines close to the Alameda Island and Oakland Portals for tie-in. This operation could be performed by District forces.
- This style of emergency installation is not risk-free. Damage to the pipeline could occur, resulting in major leak within the Posey Tube, and possible large forces from jetting water. Coordination with Caltrans will be required.
- Installation effort will take about one day to deploy the hose/pipeline, and 2 days to make the connections

The cost to install the pipeline is: $15,000 to deploy; $20,000 to make each connection (and temporary bulkheads); or a total of $55,000, taking 4 days to complete (faster if pre-built emergency connections are put into place). It is assumed that a portion of the District's 17,000± feet of Super Aqueduct can be deployed for this purpose, although it is recognized that this Super Aqueduct might be also needed elsewhere in the system after a Hayward M 7 event.

Pressure sensing flow valves could be installed at either end of the pipeline to automatically shut off flow to the pipe within the tunnel should a break occur in the Posey Tube. These flow valves may be necessary as a safety precaution to prevent flooding in the Posey Tube should unexpected break in the pipeline occur. The extra costs for these valves are not included in the above cost estimate.

8.2.2 Bay Farm #2 Crossing

Should the pipe be damaged, the most likely location is at the vaults at either shoreline, possibly due to pinching / damage to the expansion joints. The repair process is outlined in Table 8-6. It is assumed that the repair would be done as quickly as practical, and without seismic upgrade, meaning that the replacement pipe would remain vulnerable to future earthquakes and/or ongoing soil settlements.

Table 8-6. Bay Farm #2 Crossing: Time and Cost to Make One Above-ground Repair

Description	Clock hours	Cost ($2013)
Identification of leak	1 – 12	$0
Mobilization	6 – 10	$2,000
Isolate valves, excavate vault	3 – 5	$2,000
Cut and remove damaged pipe	2 – 6	$3,000
Install new pipe and coupling units	12 – 24	$20,000
Disinfect	1 – 3	$1,000
Time to restore water service	25 – 60 (average 36)	$28,000
Backfill	1 – 3	$1,000
Pavement restoration	1 – 4	$1,000
Subtotal		$30,000
Contingency (25%)		$7,500
Engineering, project management (20%)		$6,000
Total		$43,500

Average repair time for each site is about 36 clock hours, working 24-hours per day, or 4.5 8-hour days (about 150 total repair crew man-hours in the field); depending on the actual style of damage and availability of crews, equipment and parts, we think the common range of repair times, per site, would vary from 25 to 60 hours. Total repair time for multiple repairs would depend on number of crews and number of work shifts. Using the averages, and conservatively assuming damage to both vaults in a Hayward M 7 earthquake (might also be applicable to a San Andreas M 8 earthquake), the total cost and man-hour estimates for repairs to the approach pipeline are in Table 8-7.

Table 8-7. Repair Costs: Bay Farm #2 Crossing, Approaches

Earthquake	Cost	Man-hours	Notes
Hayward M 7	$87,000	300	2 repairs
Hayward M 6	$0	0	0 repairs
Calaveras M 6.75	$0	0	0 repairs
Concord M 6.5	$0	0	0 repairs

8.2.3 Park Street Crossing

Do a modern soil boring for the approach section on Alameda Island, and assess the liquefaction potential as well as soil resistivity. Research the original installation methods (ball joints, slip joints, etc.) to determine if the pipe can safely accommodate any likely PGDs. If not, locally modify the pipeline to accommodate a few inches of PGDs at the above-water shoreline segment (possibly add in suitable slip joints and ball joints along the Alameda Island approach; inspect the replaced pipe for corrosion / graphitization / tuberculation). This might cost about $150,000 and would be designed to increase the pipeline’s seismic reliability. Confirm the longer term potential for corrosion / aging failures for this pipeline.

8.2.4 Broadway to Derby Avenue, 20-inch, Closed

Repair of the pipe will be expensive. Given the pipeline’s age, it is unclear if a single-location repair is the right long term solution.

The pipe is already damaged in the submarine portion. To repair the submarine portion, given the apparent location of the existing damage, the cost might be similar to that listed under “Deep Section” in Table 8-3 (about $440,000). Should the District proceed with this repair, then the reason for the pipe failure should be determined. If the reason is due to some unique defect, then potentially the decision can then be made to make the repair (single location). However, if the reason is due to corrosion, which might also affect the rest of the submarine alignment, the decision might be made to abandon the pipeline.

First, do a modern soil boring for the approach section on the Oakland and Alameda Island sides, and assess the liquefaction potential as well as soil resistivity. Research the original installation methods (ball joints, slip joints, etc.) to determine if the pipe can safely accommodate any likely PGDs. If not, locally modify the pipeline to accommodate a few inches of PGDs at the above-water shoreline segment (possibly add in suitable slip joints and ball joints along both the Oakland and the Alameda Island approaches; do an internal ultrasonic / camera inspection of the buried pipe for corrosion / graphitization / tuberculation). This might cost about $300,000 (over and above the submarine repair), and would be designed to increase the pipeline's seismic reliability. Confirm the longer term potential for corrosion / aging failures for this pipeline.

Insert a ~16-inch HDPE liner (OD about 20 inches) within the existing pipeline. The HDPE pipe would be designed to take the full operating pressure. This could be done if the submarine damage is not too extensive, and the changes in direction along the alignment permit this type of effort. Pulling the HDPE pipe through might uplift the existing submarine concrete anchor blocks, and this might potentially be successful if the existing ball joints and slip joints be sufficiently flexible to accommodate the straightening out of the pipe. Allowing a $1,000 per foot installation cost, this upgrade might cost $500,000. Additional study of the potential for a successful installation would be required; if uplift of the anchor blocks is not possible while maintaining stress within the HDPE under about 1,500 psi, then this alternative would not be feasible.

8.2.5 Bay Farm Island #1 Bridge, Closed

Repairs and seismic upgrades are estimated at $670,000, including painting, but not removal of any lead-based paint. This price should be considered a placeholder until design details are developed. The pricing includes the following elements: $50,000 for twin ball joint / expansion couplings at either end of the bridge; about 11 new/upgraded pipe supports at $15,000 each; 11 long-throw pipe couplings at $20,000 each; $30,000 allowance for painting; 20% contingency; 20% engineering and project management.

8.3 Costs of New Installations

Preliminary cost estimates and construction durations were established by preparing a bottom-up cost estimate for the microtunnel and HDD options (see Appendix H). The approach identifies the crew sizes needed for each construction activities and the associated materials. The cost estimates were generated in the same manner as what a contractor does to prepare a bid. Using the bottom-up cost estimates, direct unit costs for the key items were developed and are provided in Table 8-8. These unit costs are based on current year (2014) pricing. To obtain a total cost for the preferred alignments at each area, the quantities were extended per item, and then mobilization, overhead, profit, and contingency percentages were applied (see Appendix H). Table 8.9 reports the budgetary costs for each of the new trenchless crossing alignments included in this Feasibility Study. The HDD options (*) include an additional lump sum cost of $706,000 for ground improvement under the entry and exit conductor casings for improved seismic resistance against shoreline slope instability. Other options for ground improvement of the conductor casings for the HDD crossings were also estimated and are provided in Table 8-10.

Table 8-8. Unit Costs for Key Items

Unit	Cost/Unit
Jacking Shaft: 28-foot-diameter Secant Pile Shaft, with carrier pipe and backfill	$13,925
Receiving Shaft: 18-foot-diameter Secant Pile Shaft, with carrier pipe and backfill	$8,550
Microtunnel w/ Carrier Pipe	$1,256
HDD Pit	$27,775
HDD pilot, reaming, and carrier pipe	$775

Table 8-9. Summary Cost Estimate for Proposed Alignments by Means of Construction

Options	Length	Construction Costs
1A- Microtunnel	1,216 feet	$7,800,000
1D - HDD	1,780 feet	$4,216,000*
2A - Microtunnel	903 feet	$6,990,000
2A -HDD	1,250 feet	$3,296,000*
3A Microtunnel (Long)	1,150 feet	$6,660,000
3A –Microtunnel (Short)	950 feet	$6,100,000
3A - HDD	1,300 feet	$3,396,000*

Table 8-10. Summary Cost Estimate for Optional Ground Improvement for HDD

Options	Unit	Construction Costs
5 Adder – Jet Grout – Figure E-5	LS	$706,000
6 Adder – Jet Grout – Figure E-6	LS	$1,105,000
7 Adder – Jet Grout – Figure E-7	LS	$1,013,000
Soil Mixed Columns – Figure E-8	LS	$614,000
Removing HDD Casings	LS	$225,000

8.3.1 Microtunnel Construction Costs and Durations

8.3.1.1 Costs

Figures H-1 and H-2 outline the cost estimate components for microtunneling at Alignments 1A, 2A, and 3A. These estimates are based upon the assumptions outlined in Section 4.2. For all three alignments, the cost of shaft construction exceeds the cost of the microtunnel (including the carrier pipe).

8.3.1.2 Estimated Construction Duration

The total construction duration from Notice to Proceed for all three microtunnel alignments (1A, 2A, and 3A) is 10 to 12 months. This includes a duration of approximately six months for the submittal process together with procurement of pipe and microtunnel equipment for all alignments. During the procurement period, the contractor can mobilize and construct the jacking and receiving shafts. The activities to setup the MTBM, complete tunneling, install pipe, grout, and backfill shafts have an anticipated duration of three to five months. One month is expected for punch list work and demobilization. Given that the majority of the activity durations are fixed, such as microtunnel equipment procurement, the variations in tunnel length and shaft depth do not result in significant differences in construction duration estimates at this feasibility level.

8.3.2 Horizontal Directional Drilling Construction Costs and Durations

8.3.2.1 Costs

Figures H-1, H-2, and H-3 outline the cost estimate components for HDD construction methods at Alignments 1D, 2A, and 3A. These estimates are based upon the assumptions outlined in Section 4.4.

8.3.2.2 Estimated Construction Duration

The total construction duration from Notice to Proceed for the HDD alignments (1D, 2A, and 3A) is five to seven months. This includes a duration of approximately two to three months for the submittal process together with procurement of pipe and equipment for all alignments. The HDD setup, pilot hole, and multiple reaming passes, including placement of the pipe, have an anticipated duration of one to two months. One month is expected for punch list work and demobilization. Given that the majority of the activity durations are fixed, the difference in pipe length does not result in a significant difference in construction duration estimates at this feasibility level. The estimated construction duration is supported by the District's Alameda Siphon project built in 2000, which was completed in 1 month (see pages A-24 to A-28). Construction of Miami-Dade's 1,500-foot-long 24-inch ID water main under Government Cut was completed in 5 months in 2011 and included downtime for drill steel breakage and retrieval. Construction of Miami-Dade's 1,300-foot-long 16-inch ID water main under Bear Cut was also completed in 5 months in 2013 and includes time for making connections, flushing, and disinfecting.

8.4 Repair Time Estimates for Tunnels

Tunnels have been observed to sustain two types of damage due to earthquakes.

- Mode 1. Small cracks in liner, induced by ground shaking and weak liners. Generally, these can be repaired once inspection is complete, often in a few days once the tunnel is dewatered, and ventilated (perhaps one to two week time frame). If repaired quickly, before ongoing damage can accumulate, the repair times can be modest. If the damage is left unprepared, and with adverse soil conditions behind the liner, the initial minor damage can lead to major damage (see below).

  For water tunnels without a pressure resistant liner (i.e., no steel pipe or similar), the damage to a liner due to ground shaking / poor quality liner can result in water entering / leaving the tunnel, with scour / erosion of the ground behind the liner. Over time (days to a few months), if the damage is left unrepaired, the damage can propagate into a local failure of the liner and major accumulation of debris within the tunnel.

- Mode 2. Major damage to the liner, usually due to fault offset through the liner, or landslide through the liner. For highway / railway tunnels, passage is blocked. For water tunnels, flow of water in nearly completely blocked off or required to follow a new flow path.

  Access to the tunnel may take many weeks, until aftershocks end, and after the tunnel is dewatered and ventilated. Temporary supports will often be required, either custom made post-earthquake, or pre-made to expedite the time needed for repair. In California, safety steps and inspections by the owner and third parties may take weeks to months to complete. Usually, a third party specialty contractor will need to be hired (can takes days to months, depending on pre-existing contracts, availability of special crews).

If the owner has developed a sound emergency response plan, with pre-selected contractors, the time needed to make a tunnel repair might be as little as 3 months (ideal conditions) to as long as 2 years (highly adverse conditions).

For microtunnels, much of the same repair issues will occur, if no pressure-retaining liner is used (e.g., concrete pipe is jacked). In other words, either Mode 1 or Mode 2 damage could occur. If the damage is severe enough to result in break-through to the bay, then the dewatering effort will be extensive.

If a pressure retaining liner is used, either steel or HDPE, then the question of repair times become a matter of deciding what type of damage occurred in the first place. For microtunnels under Alameda Channel, the major hazard is liquefaction, generally near the shoreline and up to about 30 feet below the water level. A large lateral spread that transects the microtunnel could damage the tunnel, even with a liner. However, prudent design would likely eliminate this hazard entirely, as the underwater portion of the tunnel would be placed below the lateral spread zone to start with.

This leaves the possible damage to the vertical shaft for the tunnel that may have to transect the liquefaction zone. Prudent design would include design features for the vertical shaft to preclude any but minor damage to the shaft due to liquefaction.

This leaves the possible damage of the surface-level pipeline connection to the shaft. Again, prudent design would include a feature to accommodate differential PGDs from the approach pipeline to the shaft. But even if these features are omitted (or under-designed), the repair time for a 24-inch pipeline-to-shaft connection could likely be done by a 5-man crew in 12 hours (ideal conditions, once mobilized), or 10-man crew in 48 hours (adverse conditions, once mobilized). This assumes that suitable equipment (excavators, welding machines, dump trucks, trench shields, spare parts, etc.) are also available.

For a microtunnel with seismic design, it is thought most likely that either no damage occurs, or only Mode 1 damage occurs. After a major earthquake, an interior inspection would be advisable within 90 days post-event, even if there are no obvious signs of distress to the tunnel. It is not likely that with proper design, that a Mode 2 type of damage could occur.

To provide a numeric value, the chance of Mode 2 damage for a seismically-designed microtunnel is 0.001 (1 in 1,000), given occurrence of a Hayward M 7+ event (worst case). Given that a Mode 2 failure mode occurred, the repair times would be 3 months (ideal conditions) to 1 year (adverse conditions). Mode 2 damage requires damage to the HDPE (or steel) liner; repair may require insertion of a localized liner (steel, HDPE, polymer-type, etc., with design of the repair to factor in whether the liner must be able to resist external pressure (harder), or only to maintain a smooth flow path (easier to install).

The idea with the microtunneling is to install a 48-inch casing. Inside of the casing, we will insert and install a 24-inch or 36-inch diameter carrier pipe and then backfill the annular space with grouting. The use of the 48-inch diameter casing is to allow us to jack the casing the distances of 1,000 to 1,500 feet needed for the crossings. The jacking process will scratch the outside of the casing. The insertion of the slurry lines, and other equipment will scratch the inside of the casing. The insertion of the carrier pipe allows the carrier pipe to have all the coatings and linings you want if a steel pipe is used. If the carrier pipe is HDPE, then the selection of the material is the coating and lining and nothing additional is added. The steel pipe or HDPE pipe is grouted in place within the casing. To broke the tunnel, we need to shear the 1-inch thick steel casing, shear the 6 to 12 inches of backfill grout (with no free space to shear to) and then we need the carrier pipe to shear. If we use HDPE, we are talking about a 3-inch wall thickness.

Mode 2 requires a fault rupture. We have no fault rupture zone within the crossing alignments.

The only scenario we can see where there is damage is if a steel carrier pipe is used. The steel carrier pipe might have a cement mortar lining (CML). With a major earthquake, the carrier pipe might see some minor bending and the CML might crack. Worst case scenario, maybe some of the CML chips off. So we have some damaged CML in the pipe. The water is still flowing to Alameda Island. Any post-event inspection might show the CML is damaged. The repair could happen anytime later. One way to avoid the potential CML damage problem is to use HDPE as the carrier pipe.

If somehow the microtunneling portion is sheared off, the only scenario for repair is to build a new microtunneled crossing. The timeline for the repair would be the same for the new construction (see cost estimates in Appendix H).

9 References

ALA. April 2001. Seismic Fragility Formulations for Water Systems, Parts 1 and 2, April 2001.

ALA. November 2005. Seismic Guidelines for Water Pipelines, American Lifelines Alliance, March 2005 with Addenda.

American Society of Mechanical Engineers. 1980. ASME Boiler and Pressure Vessel Code, Section VIII, Division 1.

Bartlett, S.F., and T.L. Youd. 1992. Empirical Prediction of Lateral Spread Displacement. In *Proceedings 4th Japan-U.S. Workshop on Earthquake Resistant design of Lifeline Facilities and Countermeasures for Soil Liquefaction*, Technical Report NCEER-92-0019, August 1992.

Bogs, G.J. 1993. Personnel Communications, United States Pipe and Foundry Co., Birmingham, AL.

EBMUD. 1982. Specification 1413, San Leandro Channel Utilities Crossing.

EBMUD. 1987. Specification 1494, Estuary Crossing Oak Street, Alameda – 23rd Avenue, Oakland – a Compilation of significant information, Compiled by R. Choy, Resident Engineer.

EBMUD. Specification LS-327, "Installation of 24-inch Water Main In Alice Street, Oakland; Webster Street, Alameda; and Across Tidal Canal," 1945.

EBMUD. August 1950. General Plan & Profile of Estuary Crossing to Bay Farm Island.

Elhmadi, K., and M.J. O'Rourke. August 1989. Seismic Wave Propagation Effects on Straight Jointed Buried Pipelines. National Center for Earthquake Engineering Research Technical Report NCEER-89-0022.

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

Holzer, T.L., M.J. Bennett, T.E. Noce, A.C. Padovani, and J.C. Tinsley, III. 2002. Liquefaction Hazard and Shaking Amplification Maps of Alameda, Berkeley, Emeryville, Oakland, and Piedmont: A Digital Database, *U.S. Geol. Surv. Open-File Rep. 02-296.* http://pubs.usgs.gov/of/2002/of02-296.

Holzer, T.L., et al. 2010. USGS CPT logs from USGS. Open File Report 2010-1136.

Jacobs Associates, Project Memorandum, N. Joyal and G. Boyce to D. Cicala, EBMUD. Feasible Trenchless Construction Methods, Alameda-North Bay Farm Island Crossing Master Plan, Oakland to Alameda Island, Alameda County, California, dated August 29, 2013.

Knudson, K., J. Sowers, R. Witter, C. Wentworth, and E. Helley. 2000. Preliminary maps of quaternary deposits and liquefaction susceptibility, nine-county San Francisco Bay region, California, U.S. Geological Survey Open file Report 00-444, V. 1.0.

Martin Cherrington, Personnel Communications, Cherrington Corp. (The Project Drilling Contractor), Sacramento, CA, 1993.

OCC. September 29, 2000. Final Geotechnical and Environmental Investigation, EBMUD Alameda NAS Discharge Pipeline and Siphon Project, prepared for Kennedy Jenks Consultants.

O'Rourke, T. D., and C.H. Trautmann. July 1980. Analytical Modeling of Buried Pipeline Response to Permanent Earthquake Displacements. Geotechnical Engineering Report 8-4, Cornell University.

O'Rourke, T.D., M.D. Grigoriu, and M.M. Khater. 1985. Seismic Response of Buried Pipelines. In *Pressure Vessel and Piping Technology—A Decade of Progress, American Society of Mechanical Engineers*, 1985.

Pacific Aerial Surveys. April 14, 1950.

Prior, J.C. 1935. Investigation of Bell and Spigot Joints in Cast Iron Water Pipes. *Bull. 87, Ohio State University Engineering Experiment Station*.

Staheli, K., G. Hermanson, P. Hopkins, and C. Vandaelle. 2001. Inverted Siphon Installed from Alameda to Oakland Using HDD. In *No-Dig Proceedings, North American Society for Trenchless Technology*.

United States Pipe and Foundry Co. 24-inch Usiflex Joint Model 41, Drawing 15934, Rev. Dec. 29, 1943.

USGS and Google Earth. 1993. Aerial photo.

USGS and Google Earth. 2012. Aerial photo.

USGS. 1897. 15 Minute Quadrangle, Concord.

USGS. 1899. 15 Minute Quadrangles, Concord (1897) and Hayward.

USGS. 1915. 15 Minute Quadrangle, Concord.

USGS. 1915. 7.5 Minute Quadrangles, Oakland East and San Leandro.

USGS. 1959. 7.5 Minute Quadrangle, Oakland East

USGS. 1959. 7.5 Minute Quadrangles, Oakland East and San Leandro.

Youd, T.L., and T.N. Hoose. 1978. Historic Ground Failures in Northern California Associated with Earthquakes, U.S. Geological Survey Professional Paper 993.

Appendix B
Info-Gap Report

APPENDIX B

INFO-GAP REPORT

SURVIVABILITY INFO-GAP MODEL

Introduction

An Info-Gap model (IGM) was developed to investigate the robustness of predictions about the failure rate of new crossings to Alameda. The survivability model asks the question: How wrong can the estimates be about probably of various types of failures and still have an acceptably large overall probability that a crossing survives? In other words, that the survivability is greater than some critical survivability.

This model is a probability-based model that seeks to estimate the probability of survival, after accounting for the typical ways in which an earthquake could cause failure, such as landslides, liquefaction, and shaking. We aim to calculate robustness, which in this case is the percentage error tolerance on each estimated probability.

The challenge is that the experts analyzing this project believe there is essentially zero chance of a pipeline failure in the submarine portion of the tunnels and therefore believe the likelihood of seismically-induced failure is negligible. The goal in this analysis is to assess the robustness – to uncertainties in the experts' beliefs – of our survivability prediction.

System Model

Survivability of the pipelines is modeled, though the probabilities are given as probability of failure. The system model is simply the sum of the probabilities (P) of all single and multiple failures minus the joint probabilities (as shown below in Figure 1 for two- and three-failure scenarios). Survivability is then 1 minus the total probability of failure.

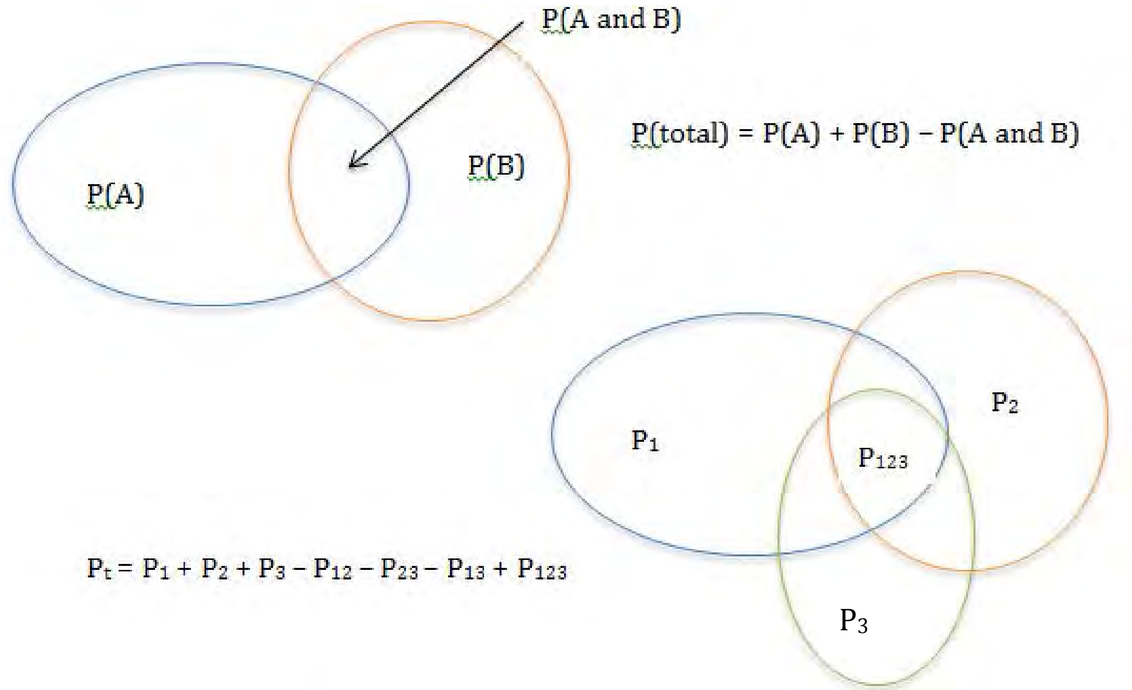


Figure 1-System Model

The Info-Gap analysis will only consider portions of the crossings that may fail under water, therefore ignoring the terms associated with the connection to existing piping and approach pipelines, since these pipes are relatively easy to access and repair compared to the submarine portions. Extremely high robustness is desired for portions submarine portions, which are difficult to access and repair. We will therefore only consider an IGM with 2-points of failure. In the HDD alternative, these points are the submarine portion and the end of the conductor casing, when present. For microtunneling, these two points are the riser and the submarine portion.

Joint probabilities have not been estimated but cannot be assumed to be zero and in fact are likely highly correlated, which is a conservative assumption. However, since joint probabilities are unknown, the joint probability will be related to the minimum expected value of the individual, or “known”, probabilities.

Performance Requirement

The survivability (S) must be no less than the critical survivability (S_c), whose value is determined by the performance requirement for the system. Survivability is defined as “no failure”, i.e.: 1 minus the probability of failure.

For the two-failure event, the probability of failure is given as P(total) = P(A) + P(B) – P(A and B). The Survivability is then S = 1 – P(total).

Uncertainty Model

The design consultants provided an estimate of the chance of failure. Chances of failure are given as “less than” terms. For example, the chance of failure of the submarine portion of the microtunnel alternative is “less than 0.1%”, or a probability of 0.001. The estimate of the chance of failure in this example as taken is the middle of the range (0.05%) and the error range is 0%-0.1%.

We choose to work with absolute error terms, as opposed to fractional or nominal, as these will produce a lower inner minimum and inversely a VERY large h-hat (robustness), which we seek. Utilizing fractional error will result in robustness-times-range. When the range is very small, the robustness term is reduced to near-zero and the nominal estimate reigns. Since there is a high uncertainty regarding the reliability of the estimates with very small ranges, we do not want these terms to essentially not be info-gapped.

Robustness Function

Next a robustness function will be derived to determine how potentially wrong these estimates can be given the intrinsic uncertainty.

System Model

$$P_f = P_1 + P_2 - P_{12}$$

$$S = 1 - P_f$$

Performance Requirement

The performance requirement is that the survivability (S) is greater than the critical survivability (S_C). For example, if 90% survival is required: $S_c = 0.9$.

$$S \geq S_C$$

$$S = 1 - P_f$$

Uncertainty Model

The equation below describes the uncertainty model which states that the difference between the actual probabilities (P_i) and their estimated values ($\tilde{P}_i$) is less than the horizon of uncertainty (h). In addition, all probabilities must adhere to rules of probability, and robustness must be greater than zero.

$$\mathcal{U}(h) = \left\{ \begin{array}{ll} P_1, P_2, P_{12}: & P_1 \geq 0, P_2 \geq 0, P_{12} \geq 0 \\ & P_{12} \leq \min(P_1, P_2) \\ & P_1 + P_2 - P_{12} \leq 1 \\ & \left|P_i - \tilde{P}_i\right| \leq h, \ i = 1,2 \\ & \left|P_{12} - \tilde{P}_{12}\right| \leq h \end{array} \right\} \quad h \geq 0$$

Evaluating the Robustness

Robustness ($\hat{h}$) is the maximum horizon of uncertainty (h) such that the minimum survivability (S) is greater than the critical survivability (S_c) over the range of actual probabilities (P_1, P_2, P_{12}) that satisfy the uncertainty model.

$$\hat{h} = max\left\{h: \left(\min_{P_1, P_2, P_{12} \in \mathcal{U}(h)} S\right) \geq S_c\right\}$$

The inner minimums are the values of the probability terms that would maximize $\hat{h}$, the horizon of uncertainty. Thus P_1 is a minimum when the estimated value of probability ($\tilde{P}_1$) is combined with the horizon of uncertainty, only when the horizon of uncertainty is less than the survivability $(1 - \tilde{P}_1)$.

$$\left|P_1 - \tilde{P}_1\right| \leq h \quad \rightarrow \quad P_1 = \tilde{P}_1 + h \qquad\qquad P_1 = \begin{cases} \tilde{P}_1 + h, if\ h \leq 1 - \tilde{P}_1 \\ 1,\ else \end{cases}$$

$$\left|P_2 - \tilde{P}_2\right| \leq h \quad \rightarrow \quad P_2 = \tilde{P}_2 + h \qquad\qquad P_2 = \begin{cases} \tilde{P}_2 + h,\ if\ h \leq 1 - \tilde{P}_2 \\ 1, else \end{cases}$$

And P_{12} is a minimum when the horizon of uncertainty is subtracted from the estimated value of probability ($\tilde{P}_{12}$), when the horizon of uncertainty is less than that probability.

$$\left|P_{12} - \tilde{P}_{12}\right| \leq h \quad \rightarrow \quad P_{12} = \tilde{P}_{12} - h \qquad\qquad P_{12} = \begin{cases} \tilde{P}_{12} - h,\ if\ h \leq \tilde{P}_{12} \\ 0,\ else \end{cases}$$

After accounting for all the constraints, the Uncertainty Model can be rewritten as:

$$r(x) = \begin{cases} 1,\ \ x > 1 \\ x,\ \ 0 \leq x \leq 1 \\ 0,\ \ x < 0 \end{cases}$$

$$\mu(h) = 1 - P_1 - P_2 + P_{12}$$

$$\mu(h) = 1 - \left[r\left(\tilde{P}_1 + h\right) + r\left(\tilde{P}_2 + h\right) + r\left(\tilde{P}_{12} - h\right)\right]$$

Inputs

The probability of failure inputs provided by the design consultants are as shown in Table 1 for HDD and microtunnel alternatives. Though estimates of failure were provided for the submarine portion and for approach pipelines, we chose to consider only the probability of failure for the submarine portion. The submarine portion of the tunnels must be highly robust.

Table 1- Failure Probabilities

Horizontal Directional Drilling Alternatives	near-surface	submarine portion	end of conductor casing
Directly drilled (no conductor casing)	1-5%	0-1%	n/a
Conductor casing (100' long)	n/a	0-1%	<5%
Jet-grouting with conductor casing	n/a	0-1%	0-1%

Microtunneling Alternatives	submarine portion	riser
Free-standing riser	0–0.1%	<1%
Riser encased in concrete, open shaft, bolted cover	0–0.1%	0–0.1%
Riser encased in concrete, shaft backfilled	0–0.1%	0–0.05%

Results

Figure 2 shows robustness curves for two designs: HDD alternative #3 and MT alternative #3. The horizontal axis is the critical survivability, (S_c), while the vertical axis is the corresponding robustness ($\hat{h}$). The dashed red line is the robustness curve for the microtunnel design and the blue dot-dashed line is the robustness curve for the horizontal directional drilling design.

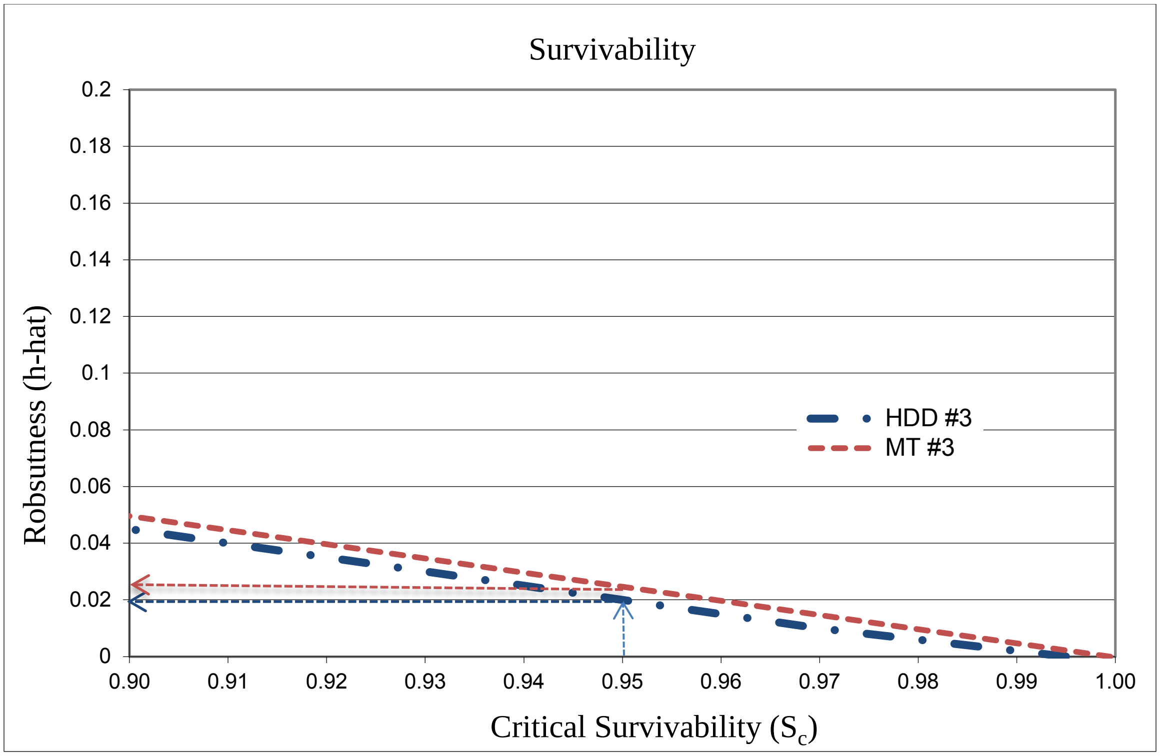


Figure 2. Robustness Results

Zeroing. The estimated survivability (evaluated directly from the estimated subunit failure probabilities) has zero robustness against uncertainty. The horizontal-intercept of the curve is equal to the estimated survivability based on the submarine failures provided by the experts. Note that the robustness is precisely zero at the horizontal intercept; hence we cannot assess the designs in terms of the estimated survivabilities because the robustness is zero and we should have no confidence in those estimates.

Trade-off. The negative slope of the robustness curve expresses a trade-off: lower (that is, worse) critical survivability gives higher (that is, better) robustness. It is necessary to use this information in comparing various designs.

Interpretation of robustness values. In order to interpret the robustness results, it is necessary to consult the uncertainty model. The uncertainty model says that the actual probability can be as large as the estimated probability plus robustness, for individual probabilities:

$$P_1 = \rho_1 + h$$

In the case of joint probabilities, the actual probability is equal to the estimated probability minus robustness:

$$P_{12} = \rho_{12} - h$$

In the example plot above, HDD has a robustness of approximately 0.01 at 95% survivability. For this particular HDD alternative, ρ_1 is given as 0.005, which means that, when requiring 95% survivability, the largest actual probability of failure that can be tolerated (P_1) is equal to 0.01 + 0.005 = 0.015, or three times the estimated probability of submarine pipeline failure. Similar conclusions can be drawn for ρ_2 and ρ_{12}.

From the red-dashed curve we see that, at 95% survivability, the MT design has a robustness of approximately 0.03. This MT alternative has an estimated probability of submarine failure (ρ_1) of 0.0005. The actual probability of submarine failure that can be tolerated (P_1) is therefore equal to 0.0305, which is many times the estimated probability of submarine failure.

The microtunnel alternative has higher robustness at every point on the curve. In addition, since the probability of failures of the microtunnel alternative are estimated as extremely low values, the higher robustness leads to *extremely large* tolerance to uncertainty relative to those estimated probability of failures, at 95% survivability. The horizon of uncertainty on the HDD alternative, on the other hand, is not only smaller, but it is *much smaller* as a relative comparison to the estimated probabilities.

REPARABILITY INFO-GAP MODEL

Introduction

An Info-Gap model (IGM) was developed to investigate the robustness of predictions about the time to repair crossings to Alameda. The reparability model asks the question: How wrong can the estimates be about time to repair various types of failures and still have an acceptably short overall estimated time to repair? In other words, that the time to repair is less than some critical time.

System Model

The system model is represented by the total time to repair. For the microtunnel alternatives, the total time to repair is the sum of the times required to isolate the approach, pump out the crossing, ventilate, enter and stabilize, excavate and rebuild the liner, and rebuild the damaged crossing section. For the HDD alternatives, it is the sum of times required to isolate the approach, pump out the crossing, and insert a new pipe.

$$T = T_{ID} + T_S + T_{INV} \ldots$$

Performance Requirement

The time to repair (T) must be no less than the critical time (T_c), whose value is determined by the performance requirement for the system.

$$T \geq T_C$$

Uncertainty Model

The equation below describes the uncertainty model which states that the difference between the actual times to repair (T_i) and their estimated values ($\tilde{T}_i$) is less than the horizon of uncertainty (h). In addition, robustness must be greater than zero.

$$\mathcal{U}(h) = \left\{ \begin{array}{c} T_{ID}\colon\ \tilde{T}_{ID} - s_{d,l}h \ \leq\ T_{ID} \ \leq\ \tilde{T}_{ID} + s_{d,u}h \qquad P_2 \geq 0, P_{12} \geq 0 \\ T_S\colon\ \tilde{T}_{ID} - s_{d,l}h \leq T_{ID} \leq \tilde{T}_{ID} + s_{d,u}h \\ T_{INV}\colon\ \tilde{T}_{ID} - s_{d,l}h \leq T_{ID} \leq \tilde{T}_{ID} + s_{d,u}h \\ T_R\colon\ \tilde{T}_{ID} - s_{d,l}h \leq T_{ID} \leq \tilde{T}_{ID} + s_{d,u}h \\ \\ \\ \end{array} \right\}$$

Evaluating the Robustness

Model Inputs

The design consultants provided a time estimate for each of the steps to repair a crossing should it fail; see Table 2 below.

Table 2- Repair Time Estimates

	Estimate	Lower	Upper
HDD with no conductor casing[1]			
Identification	12 hrs	1 hr	28 days
Set-up, mobilization, excavation	15 hrs	9 hrs	15 days
Investigate/dewater	16 hrs	0 hrs	10 days
Execute repair	10 days	4 days	50 days
HDD with conductor casing[2]			
Identification	21 days		28 days
Set-up, mobilization, excavation	11 days		15 days
Investigate/dewater	16 hrs		10 days
Execute repair	37 days		50 days
Microtunnel[3]			
All steps	4.5 months	3 months	6 months

[1] Expected failure mode = near-surface, accessible; estimated upper limits for submarine portion, based on G&E provided repair times for existing Alice-Webster crossing
[2] Expected failure mode = submarine portion of crossing; estimated repair times based on G&E provided repair times for existing Alice-Webster crossing
[3] Microtunnel alternative *may* be repaired by entering the tunnel and making repairs from the inside; estimate provided based on assumption that a repair is possible

Results

<u>Interpretation</u>

From the IGM, actual repair times can be as large as:

$T_i = \tau_i + s_{i,u}{*}h$ τ_i: estimated time for $\boldsymbol{i}$ repair step
$s_{i,u}$: upper bound of error estimate $\boldsymbol{i}$ step

Table 3- At 300 day critical repair time

HDD1	HDD2&3	Microtunnel
h-hat < 3	h-hat = 2	h-hat < 1
τ_r = 10 days	τ_r = 37 days	τ_{total} = 4.5 month

$S_{r,u}$ = 40 days	$S_{r,u}$ = 13 days	$S_{total,u}$ = 6 months
T_r = 130 days	T_r = 63 days	T_{total} = 5.5 months

Appendix C

Connection Locations and Street Alignments

APPENDIX C

CONNECTION LOCATIONS AND STREET ALIGNMENTS

Submarine pipelines will connect on-shore to the existing distribution and transmission system via streets and public right-of-way. This report identifies on-shore connection locations and pipeline/replacement alignments for each of the four preferred submarine tunnel crossing alternatives developed in Chapter 5 of the Alameda Crossings Master Plan and also listed below:

- Tunnel Alternative 1A extends from Alice Street in Oakland to Webster Street in Alameda under the Oakland Inner Harbor east of the Posey Tube.

- Tunnel Alternative 1D, located approximately 1.600 feet south of Alternative 1A, extends from Fallon Street near Estuary Park in Oakland to Marina Village Parkway in Alameda under the Oakland Inner Harbor.

- Tunnel Alternative 2 extends from Bridge View Isle in Alameda to Veterans Court in Bay Farm Island under the San Leandro Channel.

- Tunnel Alternative 3A extends from Derby St. in Oakland to Broadway St. in Alameda under the Tidal Canal.

Preliminary concerns associated with easements, utilities, permits and costs are also presented.

CONNECTION LOCATIONS FOR TUNNEL ALTERNATIVE 1A and 1D

Alternative 1A - Oakland Connections

Connection piping for Tunnel Alternative 1A begins at the southern end of Alice Street south of Embarcadero (First Street). Figure 1 presents the B-Map extent (B-Map 1485B474) of the connection location.

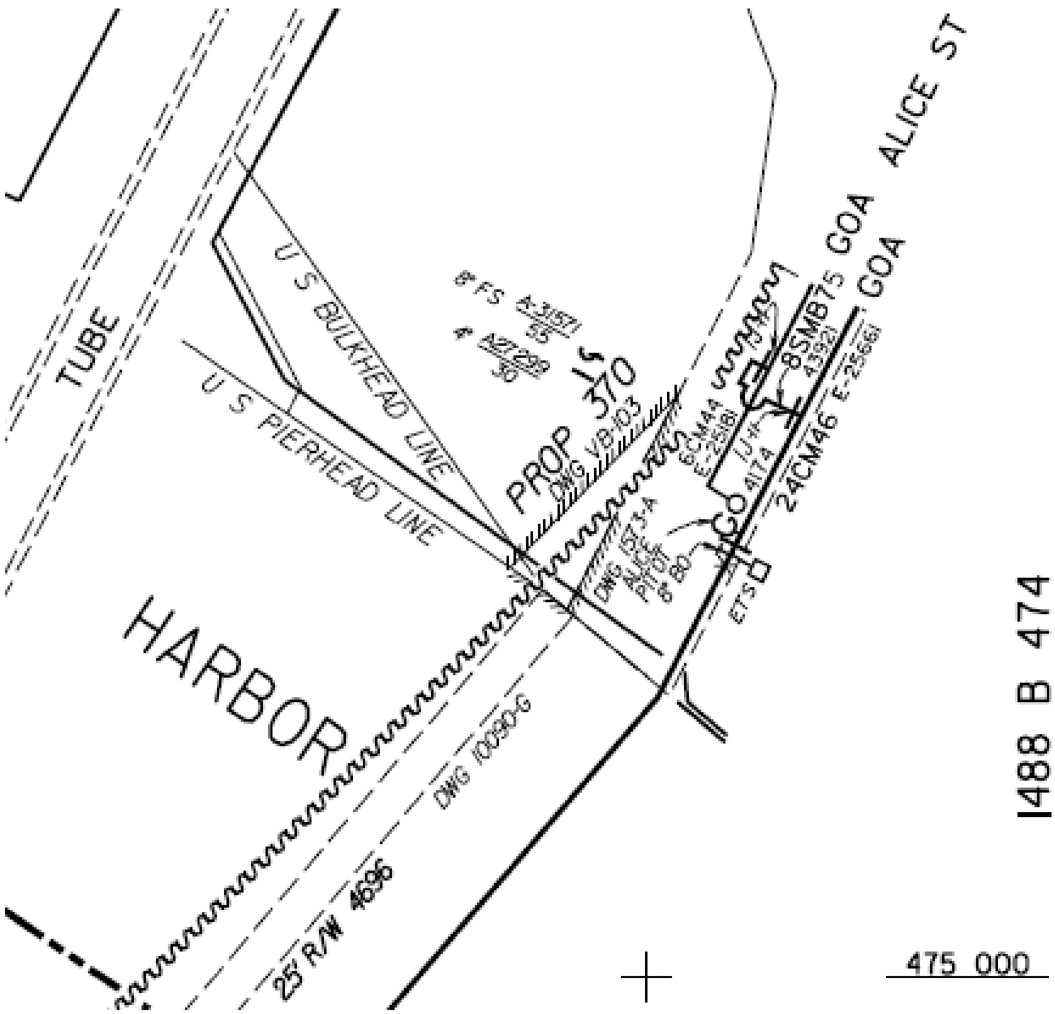


Figure 1: Alternative 1A – South Alice Street Connection Location

The Oakland side street piping will extend from the east-end of the submarine crossing on Alice Street to the intersection of 9th Street and Alice, connecting to the existing 30SMM37 (E-20537-B) pipeline at a T-connection. Figure 2 presents the 9th Street and Alice Street intersection, taken from B-Map 1488B476. By connecting the new alignment at this location, the District could eventually replace approximately 2,300 feet of existing 24CM46 (E-25661) cast iron pipe in Alice Street (which exhibits a high consequence of failure based on the leak history) and about 300 feet of 24SMM53 pipe encased under the Hwy 880 embankment**.**

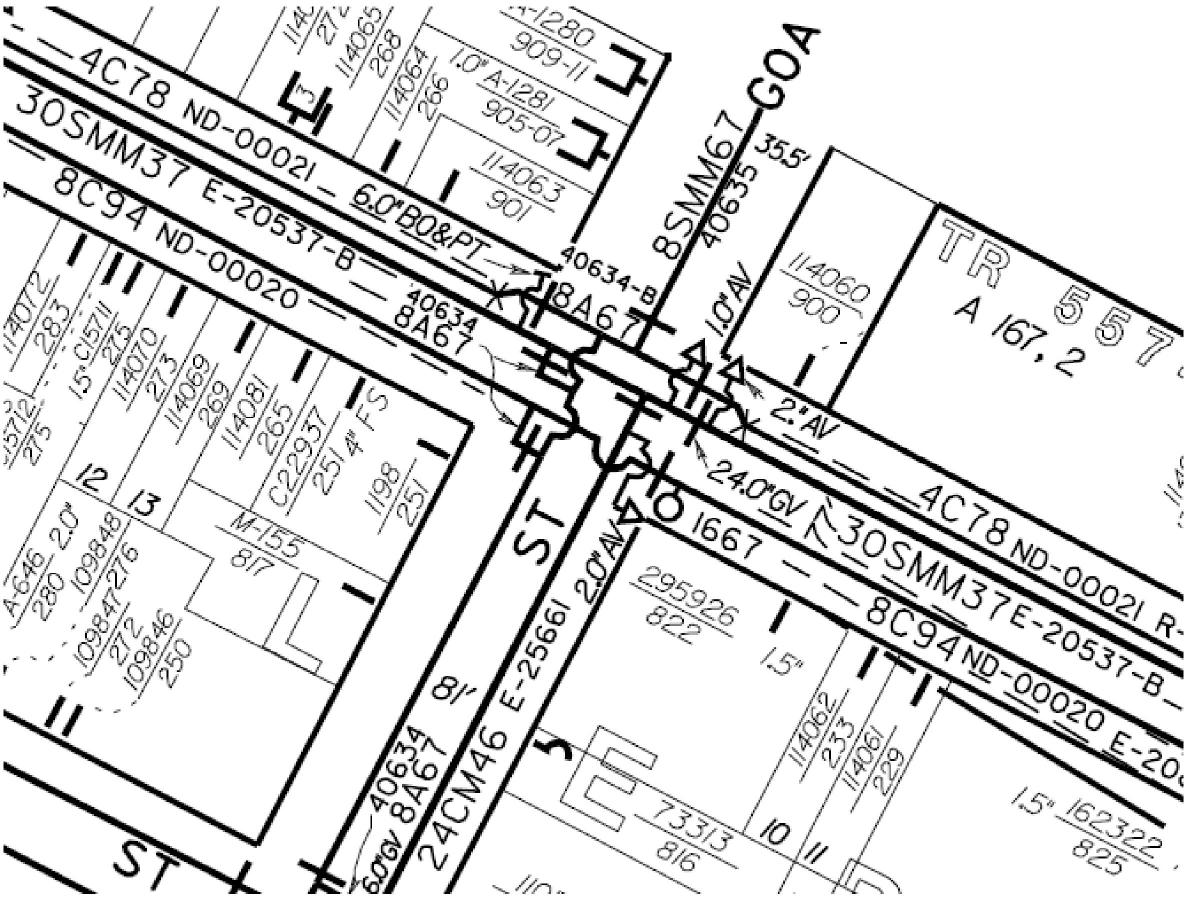


Figure 2: Alternative 1 – Alice at 9th Street Connection Location

Alternative 1A - Alameda Connections

The connection or tie-in is located in the vicinity of the Barnhill Marina and the existing 24CMM46 (E-25661-A) crossing near the 10' R/W 647 (DWG MB-128) as shown in Figure 3 and B-Map 1485B474. The area is northeast of the Mariner Square Drive and Marina Village Parkway intersection.

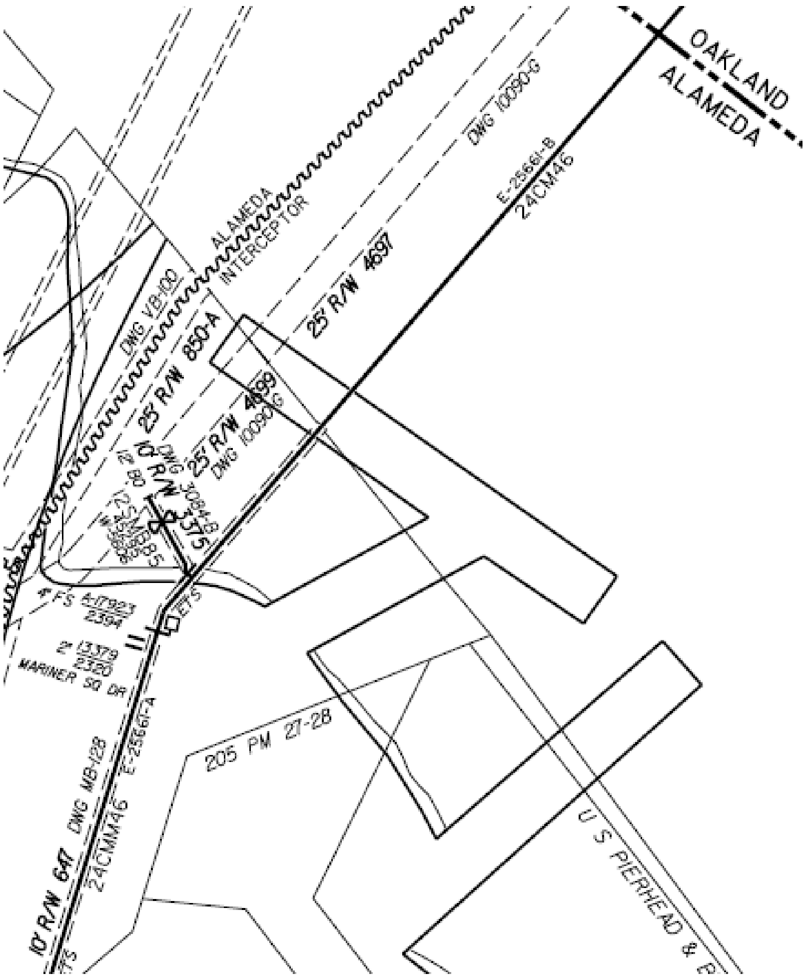


Figure 3: Alternative 1A – Alameda Connection at Barnhill Marina

The submarine crossing for Alternative 1A will surface at Barnhill Marina and ultimately extend south about 7,000 feet to the intersection of Constitution Way, Lincoln Avenue, and 8th Street, in order to connect to the new 16-inch pipeline installed in Lincoln Avenue between 8th St. and Park Avenue. Figure 4, taken from B-Map 1485B468, shows the existing piping at the connection point including the 20SMM36 (E-19979) that will be removed from service.

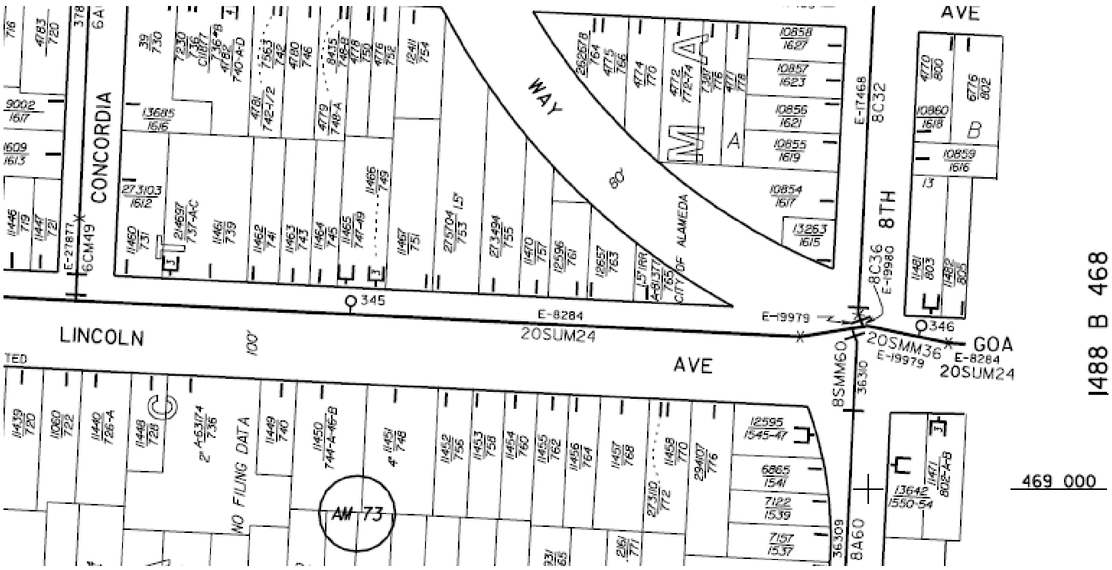


Figure 4: Alternative 1 – Alameda Connection at 8th and Lincoln

Alternative 1D - Oakland Connections

The connection or tie-in on the Oakland side is located at the southern end of Fallon Street, south of Embarcadero (First Street). Figure 5 (taken from of B-Map 1488B474) shows the street piping route leading from the Oakland Inner Harbor towards Embarcadero West. An aerial of the same vicinity is also shown below in Figure 6.

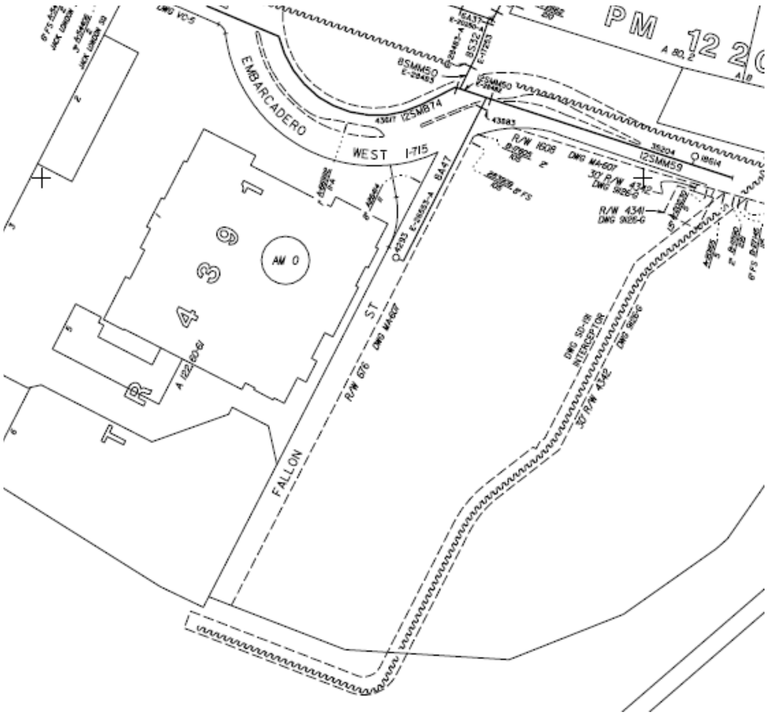


Figure 5: Alternative 1D – Oakland Connection at South Fallon Street

Figure 6: Alternative 1D – South Fallon Street Aerial

The street alignment extends north along Fallon to the same connection point described in Alternative 1A at the intersection of 9th Street and Alice Street. The existing 24CM46

(E-25661-A) in Alice Street requires replacement regardless of alternative route. Refer to Plate 3 for alternative street alignments.

Alternative 1D - Alameda Connections

The Alameda side connection will be located within the paved parking area of Telecare Corporation, a mental health company, on the Alameda estuary shore along Marina Village Parkway. This site provides room for construction staging in the parking lot of the office park and minimal utility confilcts. Figure 7 shows an inset of B-Map 1488B472, including the location of the connection to the distribution system.

Figure 7: Alternative 1D - Alameda Connection at Telecare Corp. Parking

Figure 8 shows an aerial photo of the parking lot where the submarine tunnel pipeline would connect to the street pipline buried in the streets.

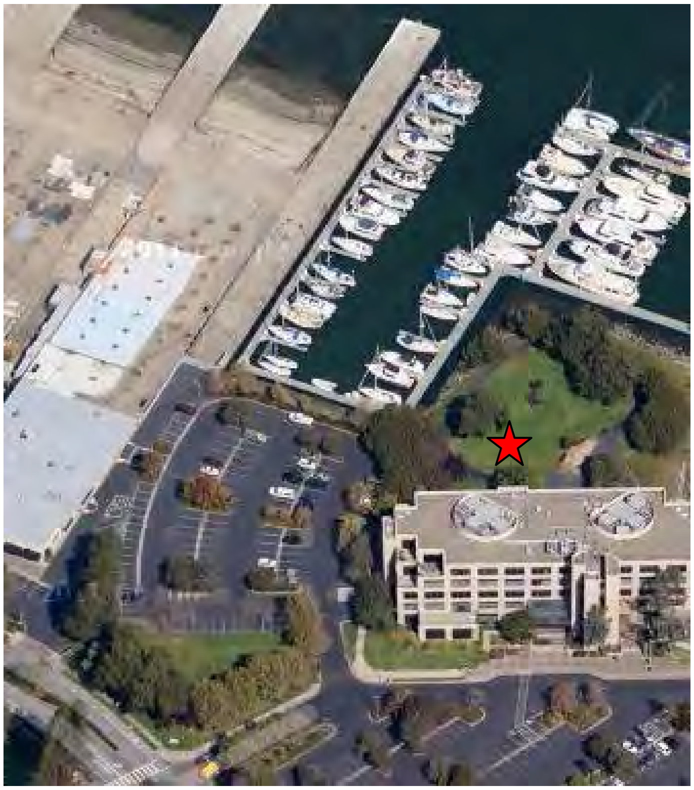


Figure 8: Alternative 1D - Alameda Connection at Telecare Corp. Parking Lot

Alternative 1D on the Alameda side would extend from the Telecare Corp. parking area on Marina Village Parkway south to the same connection point described in Alternative 1A at the intersection of Constitution Way, Lincoln Avenue, and 8th Street, as shown on Plate 4. Plate 4 also shows for alternative street alignments on the Alameda side associated with this submarine or tunnel crossing alternative.

STREET ALIGNMENTS FOR TUNNEL ALTERNATIVES 1A AND 1D

Potential street alignments were identified to connect the submarine or tunnel estuary crossing projects to the distribution system. Plates 3 and 4 present aerial photos delineating the alignments for the Oakland side and Alameda side, respectively, as described below.

<u>Tunnel Alternatives 1A and 1D – Oakland Pipeline Alignments:</u>

Four street alignments were identified on the basis of reducing potential utility conflicts and construction costs. The four alignments described below traverse from the tie-in at Alice St. and Ninth St. to the submarine crossings that surface at the south end of Alice in the case of Crossing Alternative 1A, and at the south end of Fallon St. for Alternative 1D:

- *Alignment 1*: Approximately 3,700 feet – follows 9th Street to Jackson Street to 5th Street to Alice Street to Estuary Crossing 1A. This alignment most resembles the existing alignment along Alice Street, except for the crossing under Highway 880 (Nimitz Freeway) along Jackson Street.

- *Alignment 2*: 3,700 feet – follows Alice Street to 8th Street to Jackson Street to Embarcadero to Alice Street to Estuary Crossing 1A. Alignment 2 follows Jackson Street for most of the replacement, which provides open corridors within the right-of-way to install the new pipeline.

- *Alignment 3*: 4,500 feet – follows Alice Street to 8th Street to Jackson Street to 5th Street to Oak Street to Embarcadero to Fallon Street to the Estuary Park Entrance to Estuary Crossing 1D.

- *Alignment 4*: 4,500 feet – follows 9th Street to Jackson Street to 7th Street to Oak Street to 5th Street to Fallon Street to the Estuary Park Entrance to Estuary Crossing 1D. Alignment 4 follows Oak Street under the Highway 880 (Nimitz Freeway) crossing.

<u>Tunnel Alternatives 1A and 1D – Alameda Pipeline Alignments:</u>

Four alignments or routes were identified with the goal of eliminating the flooding issues in the Posy Tube due to pipeline breaks. All four of the alignments share the Constitution Way right-of-way from Marina Village Parkway south. The four alignments are described below:

- *Alignment 1*: Approximately 5,600 feet – follows the alignment of the existing 24-inch diameter pipeline before turning down an old abandoned railroad right-of-way, then onto Constitution Way, connecting at the intersection of 8th Street and Lincoln Avenue. Alignment 1 follows the most direct path connecting to Crossing Alternative 1A.

- Alignment 2: Approximately 7,200 feet – follows the alignment of the existing 24-inch pipeline but then turns onto Marina Village Parkway south to Constitution Way to the connection at 8th Street and Lincoln Avenue. This is the longest route on the Alameda side for estuary Crossing Alternative 1A, but it avoids potential permit and easements issues associated with the abandoned railroad right-of-way.

- *Alignment 3*: Approximately 5,500 feet – begins in the Telecare Corporation parking area then follows Marina Village Parkway south to Constitution Way to the connection at 8th Street and Lincoln Avenue. Alignment 3 provides the shortest route connecting estuary crossing Alternative 1D to the tie-in at 8th Street and Lincoln in Alameda.

- *Alignment* 4: Approximately 6,600 feet – begins in Telecare Corporation parking area then turns northwest up Marina Village Parkway to the old abandoned railroad right-of-way to Constitution Way to the Alameda connection at 8th Street and Lincoln Avenue. Alignment 4 has the longest route connecting estuary crossing Alternative 1D to the tie-in at 8th Street and Lincoln in Alameda.

EASEMENTS FOR TUNNEL ALTERNATIVES 1A AND 1D

Permanent and construction easements will be required for each of the submarine crossing alignments. A description of each easement is provided below.

<u>Oakland side</u>

Construction easements will be required on the Oakland side of the tunnel crossing for Alternative 1D. Fallon Road, south of Embarcadero West, provides access to Cash & Carry Wholesale Groceries (formerly Jetro, but now vacant), the San Francisco Bay Trail, Estuary Park, and the Oakland Inner Harbor**.** Figure 9 presents the pipeline alignment from the Embarcadero to the San Francisco Bay Trail and nearby parking areas. Coordination with the parcel owners will be required to determine if construction easements are required within this reach for pipeline and tunnel construction activities.

Figure 9: Fallon Road Terminus at Crossing 1D

Alameda side:

Submarine Crossing Alternative 1A surfaces in an area called Mariner Square located on the west side of the Oakland Estuary. Owned by Barnhill Construction Company, Mariner Square is a mix of house boats, residential, commercial, and industrial activities. EBMUD currently operates and maintains both water transmission and wastewater interceptor facilities located below grade on the Mariner Square property. Figures 10 and 11 show the general location of any new pipeline alignment through Mariner Square. The property owners would need to be consulted if pipeline alignments 1 and 2 are developed further, thus adding to the utility congestion that exits today.

Figure 10: Mariner Square Parking Area – Looking South

Figure 11: Mariner Square Region for Alignments 1 and 2

Pipeline alignments for submarine Crossing Alternative 1D on the Alameda side begin in the Telecare Corporation parking area. An easement would be required to work and install the new pipeline alignment within this property. Refer to Figure 12 for the tentative alignment (red line) located in the parking area.

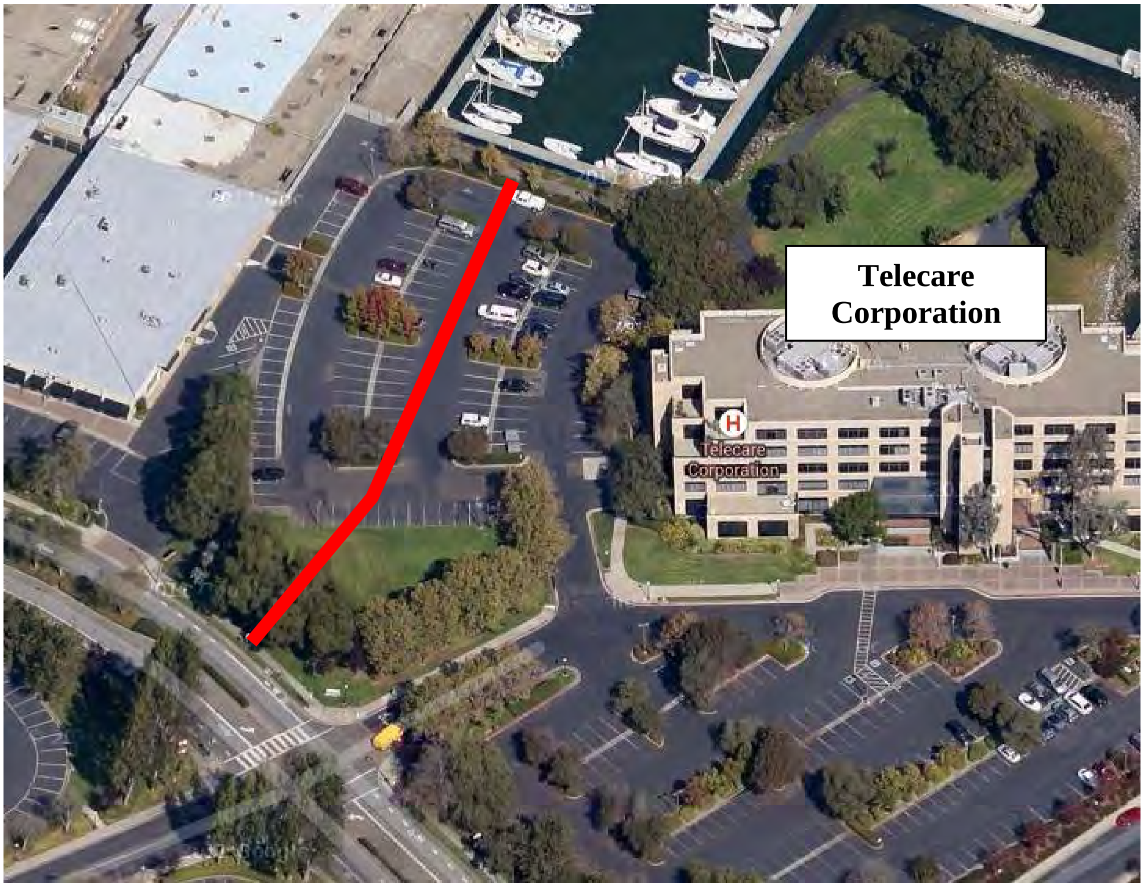


Figure 12: Telecare Corporation Park Area – Alignments 3 and 4

Shown in Figure 13, the abandoned railroad right-of-way associated with Alignments 1 and 2 between Marina Village Parkway and Constitution Way appears to be owned by Southern Pacific (S P CO 872-1-65A-2) or the City of Alameda. Further investigation by Real Estate Services will be needed if the alignment is pursued. An easement will be required for the installation of a new pipeline alignment along this corridor.

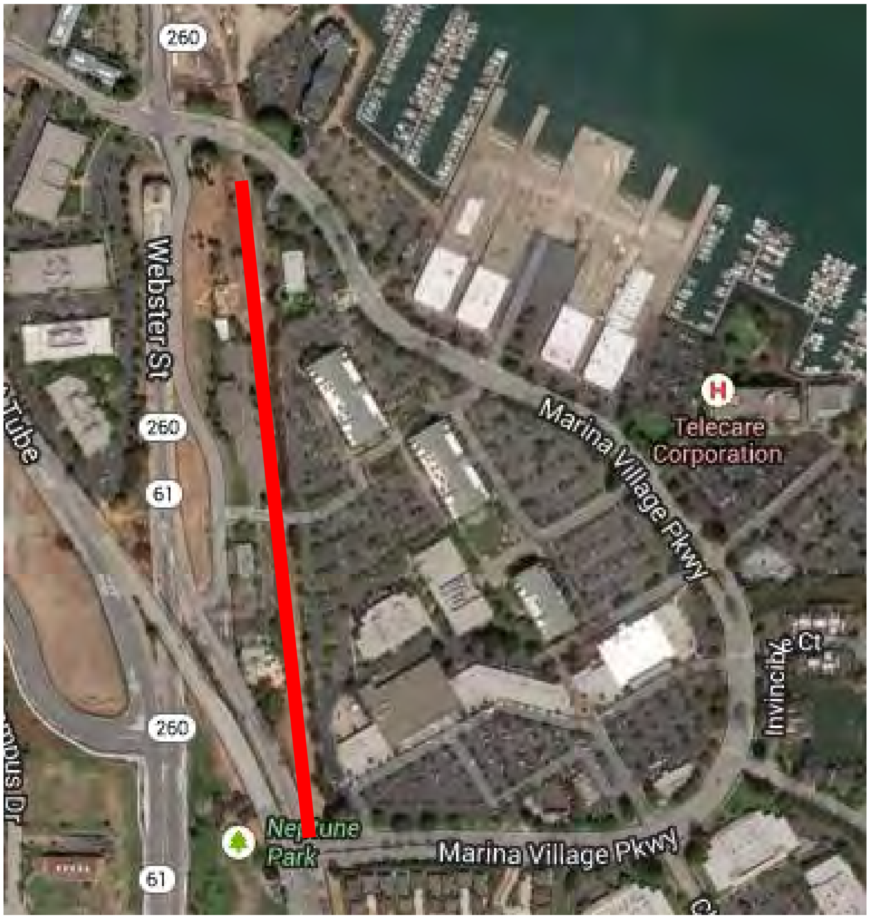


Figure 13: Abandoned Railroad Easement – Alignments 1 and 4

UTILITY ISSUES FOR TUNNEL ALTERNATIVES 1A AND 1D

A summary of the current utility information is provided below. Refer to Plates 5 and 6 for GIS maps identifying the utility locations on the Oakland side and Alameda side connections, respectively.

Oakland Utility Conflicts

Kinder Morgan owns 10-inch and 12-inch high pressure refined petroleum product lines that travel in the east-west direction along the railroad tracks parallel to Embarcadero West. All the proposed alignment will cross these Kinder Morgan lines.

PG&E owns a 24-inch gas transmission main along 2nd Street in the east-west direction. All the proposed alignment will cross this PG&E gas line. A 12-inch gas transmission main runs along Alice Street crossing the estuary. Alignments 1 and 2 will run parallel to the gas crossing.

PG&E owns two 115kV underground electrical transmission mains. The first follows 8th Street to Fallon Street and continues east through Laney College. The second travels along 6th Street to Oak Street and continues east along 10th Street. All of the proposed street alignments will cross both electrical transmission mains.

EBMUD owns the Alameda Interceptor that crosses the estuary and the South Interceptor that follows along 1st Street as shown in Plate 5. All four of the alignments will cross the 84-inch South Interceptor along 1st Street perpendicularly. Alignments 1 and 2 parallel the 96-inch Alice Street interceptor and the three-siphon submarine interceptor crossings to Alameda.

The Lake Merritt BART Station is located at 9th Street between Madison Street and Oak Street. The BART runs underground along 9th Street west of the Lark Merritt Station; refer to B-Map 1488B476 and Plate 5. All four alignments will require coordination with BART.

Alameda Utility Conflicts

EBMUD owns the Alameda Interceptor that crosses the estuary in Mariner Square and runs south along the western side of the abandoned railroad to Constitution Way. At the intersection of Constitution Way and Buena Vista Avenue, the interceptor turns east along Buena Vista Avenue to collect from the eastern region of Alameda. Pipeline alignments 1 and 4 will run parallel to the Alameda interceptor in the Mariner Square area. All four of the Alignments will run parallel and share the right-of-way along Constitution Way, and cross the Alameda Interceptor on Constitution Way between Eagle Avenue and Buena Vista Avenue. Coordination with the Wastewater Division will be required during the design phase to provide adequate clearance parallel alignments and crossings.

PG&E owns three separate high priority facility gas crossings at Marina Village Parkway (10-inch), Tynan Avenue (6-inch), and Eagle Avenue (8-inch). Refer to Plate 6 for each PG&E Gas crossings with the four considered alignments. Alignments 1 and 4 cross the Marina Village Parkway and Tynan Avenue gas transmission mains. All four of the alignments cross the Eagle Avenue 8-inch gas transmission main. Coordination with PG&E will be required during the design of the new pipeline to confirm clearance requirements.

Table 5.1 provides a summary of the utility crossings associated with Tunnel Alternatives 1A and 1D on the Oakland and Alameda side. A description of each utility is provided:

- **EBMUD Interceptor**: EBMUD interceptors 24-inch diameter and larger
- **PG&E Electrical**: 115kV high voltage electrical and higher
- **PG&E Gas**: 6-inch diameter high pressure gas lines are larger
- **Kinder Morgan**: any size high pressure refined petroleum products lines
- **BART**: alignment of the BART

Lengths listed in Table 1 correspond to the length of new pipeline alignment within the utility right-of-way. The separation clearances for high priority utilities varies for electrical, gas, and petroleum lines, and creates design issues within congested streets.

TABLE 1
Utility Crossing Summary for Street Alignments Associated with Tunnel Alternatives 1A and 1D

Street Alignment, Oakland side	EBMUD Interceptor	PG&E Electrical	PG&E Gas	Kinder Morgan	BART
1	1 Crossing 600’ R/W	2 Crossings	1 Crossing 1,000’ R/W	1 Crossing	1 Crossing 400’ R/W
2	1 Crossing 1,100’ R/W	2 Crossings 400’ R/W	1 Crossing 600’ R/W	1 Crossing 400’ R/W	1 Crossing
3	1 Crossing	2 Crossings 400’ R/W	1 Crossing 300’ R/W	1 Crossing	1 Crossing
4	1 Crossing	2 Crossings 300’ R/W	1 Crossing	1 Crossing	1 Crossing 400’ R/W
Street Alignment, Alameda side	**EBMUD Interceptor**	**PG&E Electrical**	**PG&E Gas**	**Kinder Morgan**	**BART**
1	1 Crossing 900’ R/W	None	3 Crossings 500’ R/W	None	None
2	1 Crossing 900’ R/W	None	2 Crossings 500’ R/W	None	None
3	1 Crossing 900’ R/W	None	2 Crossings 500’ R/W	None	None
4	1 Crossing 900’ R/W	None	3 Crossings 500’ R/W	None	None

PERMITS FOR ALTERNATIVES 1A AND 1D

Pipeline Alignments 1 through 4 will require construction/encroachment permits from several entities.

Alameda side

An excavation permit is required by the City of Alameda for each road or street segment excavated to install a new pipeline. The number of permits can add up depending on the number of streets encountered along the pipeline alignment. In addition, a Traffic Control Plan may be required for the permit application, which may require help from a traffic consultant to assist with the traffic control plan and contract documents (plans and specifications).

Oakland side

A Utility Excavation Permit is required by the City of Oakland for each road or street segment excavated for the new pipeline. The number of permits can again add up depending on the number of streets encountered along the pipeline alignment. In addition, a Traffic Control Plan may be required for the permit application, which may require help from a traffic consultant to assist with the traffic control plan and contract documents (plans and specifications). For more information on the requirements, refer to the City of Oakland – Utility Excavation Permit Application Checklist.

A Utility Survey Permit and Pipeline Crossing Permit are required by Union Pacific for the pipeline crossings. EBMUD must also submit a permit application before performing the survey on Union Pacific railroad property. Before construction of the new pipeline, EBMUD must also obtain a Pipeline Crossing Permit. Figure 14 shows the Union Pacific railroad crossing at the intersection of Alice Street and Embarcadero looking north on Alice Street. The Union Pacific Railroad crossing also requires a casing for the carrier pipeline. A trenchless technology (pipe ramming, slick bore, or horizontal boring) will be used to install the carrier pipe under the existing railroad tracks. Union Pacific requirements call for a non-flammable pipe material, thus potentially eliminating the use of HDPE as the carrier pipe in this application.

Figure 14: Union Pacific Railroad Crossing at Alice Street and Embarcadero

A Standard Encroachment Permit is required by the State of California (Caltrans) for the Highway 880 (Nimitz Freeway) crossing. Even though the new pipeline alignments will not interfere with traffic on the highway, an encroachment permit is required because construction is located within the highway right-of-way. All of the new street alignments in Oakland associated with Tunnel Crossing Alternatives 1A and 1D pass under highway overpasses at Jackson Street or Oak Street, while the existing 24SMM53 in Alice Street crosses under Highway 880 through a 36-inch culvert. Figure 15 provides a street view of the Jackson Street overpass looking south.

Figure 15: Highway 880 (Nimitz Freeway) Crossing at Jackson Street

The Dublin/Pleasanton and Fremont BART underground train lines follow 9th Street to Lake Merritt Station located between 9th and 8th, and Madison Street and Fallon Street. Figure 16 presents the Lake Merritt BART Station area. The four Oakland side alignments will require work within the BART right-of-way above the underground tunnels. As a result, EBMUD will have to submit a Permit Application for Construction to BART to summarize the scope of work for the new pipeline installation. BART will respond to the Permit Application with requirements for the design and construction phases.

Figure 16: Lake Merritt BART Station Location

DESIGN AND CONSTRUCTION ISSUES FOR ALTERNATIVES 1A AND 1D

The following areas will require more thorough investigation during the CEQA and design phase of the project:

<u>Traffic Control During Construction</u>

All of the street alignments fall within residential, commercial, highway on-ramp/off-ramps, and city streets. Additional assistance from a traffic control consultant will be required to complete the contract documents and meet the requirements of the City of Oakland permit application process.

Liquefaction Susceptibility

The Alameda-Oakland Estuary area falls in a highly susceptible liquefaction seismic hazard zone. Plates 7 and 8 identify Liquefaction Susceptibility zones ranging from very low, low, moderate, high, to very high using the Maps of Quaternary Deposits and Liquefaction Susceptibility in the Central San Francisco Bay Region, California (Witter et al. 2006), respectively for the Oakland and Alameda sides. The alignments fall into liquefaction susceptibility levels of moderate to very high given the close proximity to the estuary. Table 2 breaks down the length of each alignment by level of liquefaction susceptibility.

TABLE 2
Oakland and Alameda sides Liquefaction Susceptibility Summary

Street Alignments		**Liquefaction Susceptibility Level**			
Oakland side	**Total Length [Feet]**	**Moderate Length [Feet]**	**Moderate Percent [%]**	**Very High Length [Feet]**	**Very High Percent [%]**
1	3,700	2,800	76%	900	24%
2	3,700	2,400	65%	1,300	35%
3	4,500	2,400	53%	2,100	47%
4	4,500	2,400	53%	2,100	47%
Alameda side	**Total Length [Feet]**	**Moderate Length [Feet]**	**Moderate Percent [%]**	**Very High Length [Feet]**	**Very High Percent [%]**
1	5,600	1,800	32%	3,800	68%
2	7,200	1,800	25%	5,400	75%
3	5,500	1,800	33%	3,700	67%
4	6,600	1,800	27%	4,800	73%

Estuary Park

The estuary crossing for Alternative 1D on the Oakland side surfaces at the south end of Fallon St. just north of Estuary Park, a seven acre waterfront park adjoining a vacant wholesale grocery building and parking lot located on the Embarcadero at Fallon Street. The park incorporates the Jack London Aquatic Center, a community facility providing youth and adult programs in rowing. It also includes a popular grassy field, a public boat launching ramp and a group picnic area.

Measure DD was recently passed by the City of Oakland, which funds numerous park restoration projects. Estuary Park is included with Measure DD for renovation and expansion of the park. The plan includes the reshaping of the shoreline, providing improved access along the water's edge, and redesigning parking to provide for additional landscape areas and a green edge. Figure 17 presents an image of the renovation to Estuary Park. The Measure DD Estuary Park Project is on hold pending site cleanup, scheduled for 2012, and finalization of adjacent Oak to 9th Project development plans. The District is currently waiting for additional information from the City of Oakland on the park renovation. The Design phase will coordinate efforts with the City of Oakland.

Figure 17: Estuary Park Renovation per Measure DD

Highway 880 (Nimitz Freeway) Crossing

The installation of pipe under the Highway 880 overpass will create some issues for the construction contractor. Limits to the vertical access will decrease the pipeline installation efficiency and slow the construction pace of the project within this area. Coordination with Caltrans will be required during the Design phase to meet the requirements of the permit application to work within the highway right-of-way.

Geotracker Information

Figure 18 shows the nearby cleanup sites in both Oakland and Alameda reported in Geotracker for the tunnel crossings located east of the Posey Tube.

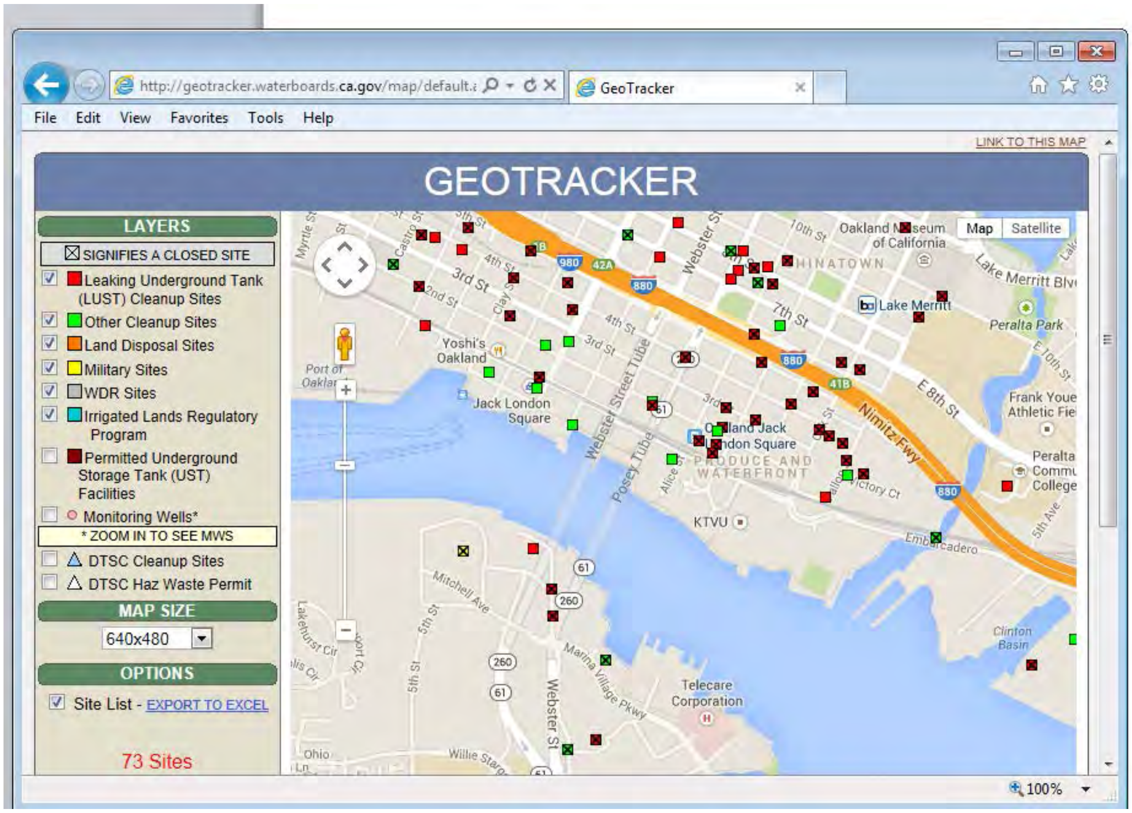


Figure 18: Geotracker Sites east of the Posey Tube

COST ESTIMATES FOR ALTERNATIVES 1A AND 1D

EBMUD collected cost estimates from two pipe manufacturers and three local Bay Area construction contractors to understand the unit cost for pipeline installation in the street alignments associated with Tunnel Crossing Alternatives 1A and 1D. Table 3 provides a tabular summary of the unit cost used throughout the alternatives analysis report, including the unit costs for a 24-inch diameter steel pipeline with 3/16-inch wall thickness at 42 ksi minimum yield strength.

TABLE 3
Pipe Installation Unit Cost Summary

Steel Pipe Manufacturer	**Unit Cost [$/ft]**
Northwest Pipe	$60
NOV Ameron	$100
Average Unit Cost for Pipeline	$80
Pipe Installation Contractor	**Unit Cost [$/ft]**
McGuire & Hester	$330
Mountain Cascade	$150
Ranger Pipeline	$330
Average Unit Cost for Installation	$270
Total Combined Costs	**Unit Cost [$/ft]**
Combined Average Unit Cost	$350
Conceptual Level Safety Factor	$80
Unit Cost for Analysis	**$430**

STREET ALIGNMENT SUMMARY AND RECOMMENDATIONS for ALTERNATIVES 1A AND 1D

Summary

Tables 4 and 5 summarize the key factors to be considered in the selection of the preferred street alignments on the Oakland and Alameda side, respectfully, for Tunnel Crossing Alternatives 1A and 1D. The matrix includes pipeline length, alignment pros and cons, and planning level construction costs for each alignment. The liquefaction susceptibility percentage (LIQ) is the percentage of the pipeline length highly susceptible to liquefaction.

TABLE 4
Street Alignment Summary for Tunnel Alternative 1A and 1D, Oakland side

Alignment #	Length [feet]	Cost [$]	Pros	Cons
1	3,700	$1.6M	LIQ 24% Very High Shorter Length – Lower Cost	Congestion at 1a Crossing 6 Utility Crossings 2,000’ Utility R/W
2	3,700	$1.6M	LIQ 35% Very High Shorter Length – Lower Cost	Congestion at 1A Crossing 6 Utility Crossings 2,500’ Utility R/W
3	4,500	$1.9M	No Utilities at 1D Crossing Alignment with Limited Dual Transmission Mains	LIQ 47% Very High Estuary Park 6 Utility Crossings 700’ Utility R/W
4	4,500	$1.9M	No Utilities at Crossing Alignment with Limited Dual Transmission Mains	LIQ 47% Very High Estuary Park 6 Utility Crossings 700’ Utility R/W Most Elbows in Alignment

TABLE 5
Street Alignment Summary for Tunnel Alternative 1A and 1D, Alameda side

Alignment #	Length [feet]	Cost [$]	Pros	Cons
1	5,600	$2.4M	Shorter Length – Lower Cost Fewest Elbows in Alignment	LIQ 68% Very High Congestion in Mariner Square Abandoned Railroad R/W 4 Utility Crossings 1,400’ Utility R/W
2	7,200	$3.1M	No Abandoned Railroad R/W Open Telecare Parking at 1D	Longest Length – Highest Cost LIQ 75% Very High Congestion in Mariner Square 3 Utility Crossings 1,400’ Utility R/W
3	5,500	$2.4M	Shorter Length – Lower Cost Open Telecare Parking at 1D	LIQ 67% Very High 3 Utility Crossings 1,400 Utility R/W
4	6,600	$2.8M	No Abandoned Railroad R/W Alignment with Limited Dual Transmission Mains	LIQ 73% Very High Abandoned Railroad R/W 4 Utility Crossings 1,400 Utility R/W Most Elbows in Alignment

Recommendations

Pipeline routing recommendations for Tunnel Alternatives 1A and 1D are presented below.

Alternatives 1A – Oakland side. Alignment 1 is recommended for further review during the design phase if tunnel crossing 1A is selected. This alignment minimizes pipeline installation in suspected liquefaction zones and high priority utility right-of-ways. Potholing will be required as part of the subsurface investigation to confirm clearances between the many utilities. In addition, the Union Pacific Railroad crossing will require a casing design to contain the new transmission pipe within the railroad right-of-way.

Alternatives 1D – Oakland side. Alignment 3 is recommended for further review if tunnel crossing 1D crossing is selected. Alignment 3 is more direct and shorter than Alignment 4. Coordination with the City of Oakland on the Estuary Park Improvements will be needed. Similar to Alternative 1A, the Union Pacific Railroad crossing will require a casing design to contain the new transmission pipe within the railroad right-of-way.

Alternatives 1A – Alameda side. Alignment 1 is recommended for further review if tunnel crossing 1A is selected. This alignment provides one of the shorter routes to the connection at Lincoln Avenue, but will require an encroachment permit to install the pipeline in the old/abandoned railroad corridor. Utility congestion in Mariners Square will need to be studied to find space for a new pipeline. Potholing will also be required to delineate the locations of the other utilities.

Alternatives 1D – Alameda side. Alignment 3 is recommended for further study if tunnel crossing 1D is selected. Alignment 3 offers one of the shorter routes, also reducing the amount of pipeline installation in liquefiable soils. The Telecare Corporation Parking area also provides space for the estuary crossing installation/receiving pit; encroachment permits will also be required.

CONNECTION LOCATIONS FOR TUNNEL ALTERNATIVE 2

Alternative 2 - Alameda Connections

The estuary crossing on the Alameda site will connect to a 16-inch mortar lined and coated steel main (16SMM51) at the south-eastern end of Bridge View Isle Drive and eventually extend back to Lincoln Avenue near Park St. via any number of street alignments as shown in Plate 18. Figure 19 shows a portion of B-map 1497B460 at the Bridge View Isle Drive connection point.

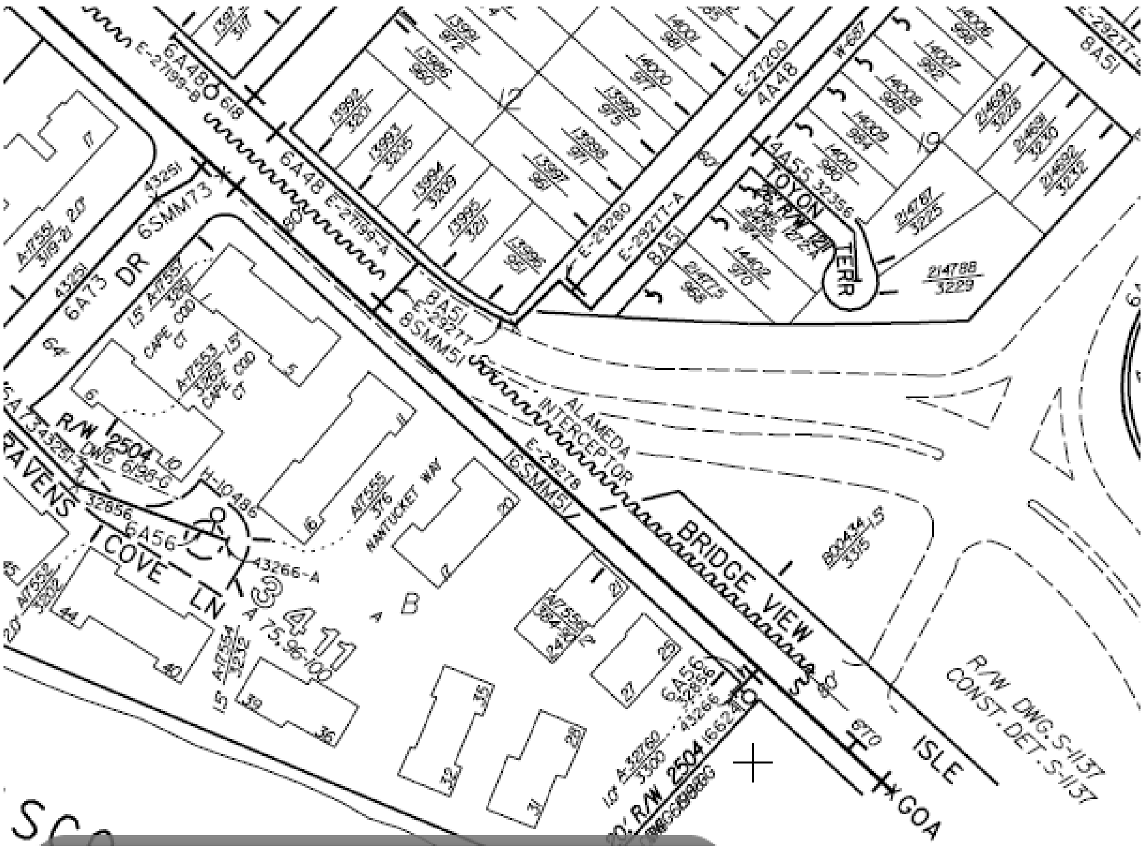


Figure 19: Alternative 2 – South Otis Drive Connection Location

The goal is to ultimately connect the new estuary crossing to the existing 20-inch unlined and mortar coated steel main (20SUM24) located at the corner of Lincoln Avenue and Park Street. Figure 20 presents the B-map extent (B-map 1497B464) of the Lincoln Avenue and Park Street connection point.

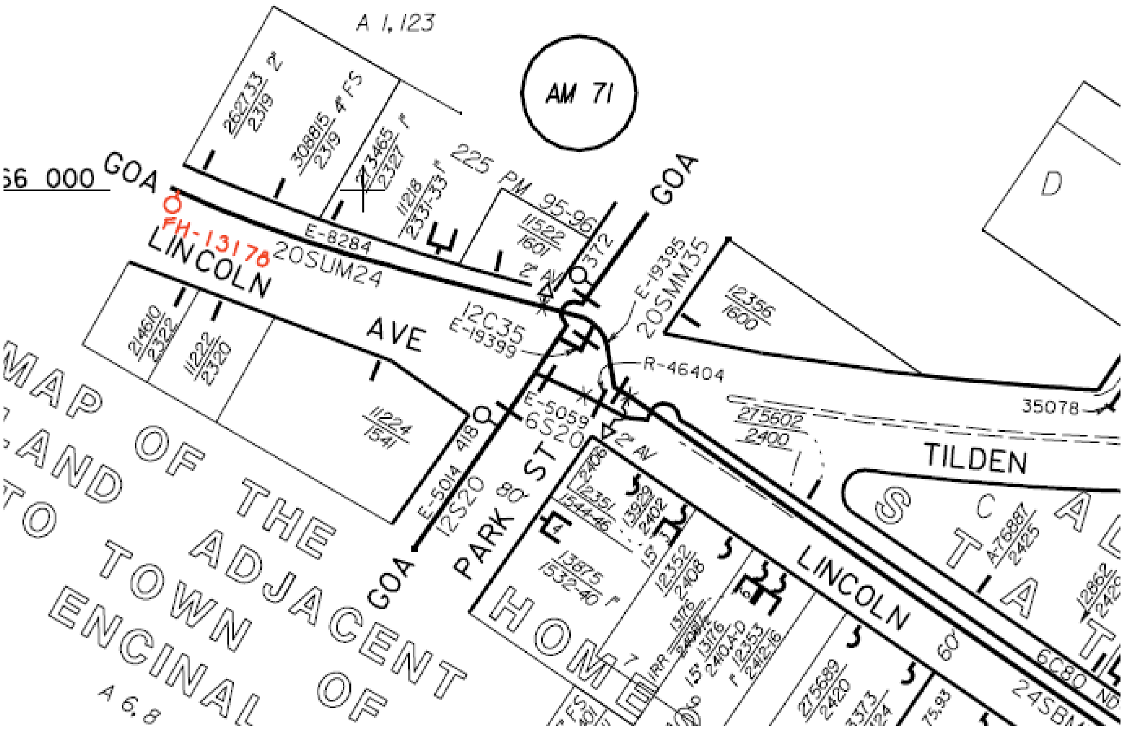


Figure 20: Alternative 2 – Lincoln Avenue Connection Location

<u>Alternative 2 - Bay Farm Island Connections</u>

The tunnel crossing on the Bay Farm Island side will connect to an existing 16-inch mortar lined and coated steel pipe on the north end of Veterans Court (which borders the estuary) and then extends southerly to connect to an existing 16-inch transmission main at the intersection of Island Drive and Robert Davey Jr. Drive.

Figure 21 presents the B-map extent (B-map 1497B458) of the connection at the end of Veterans Court.

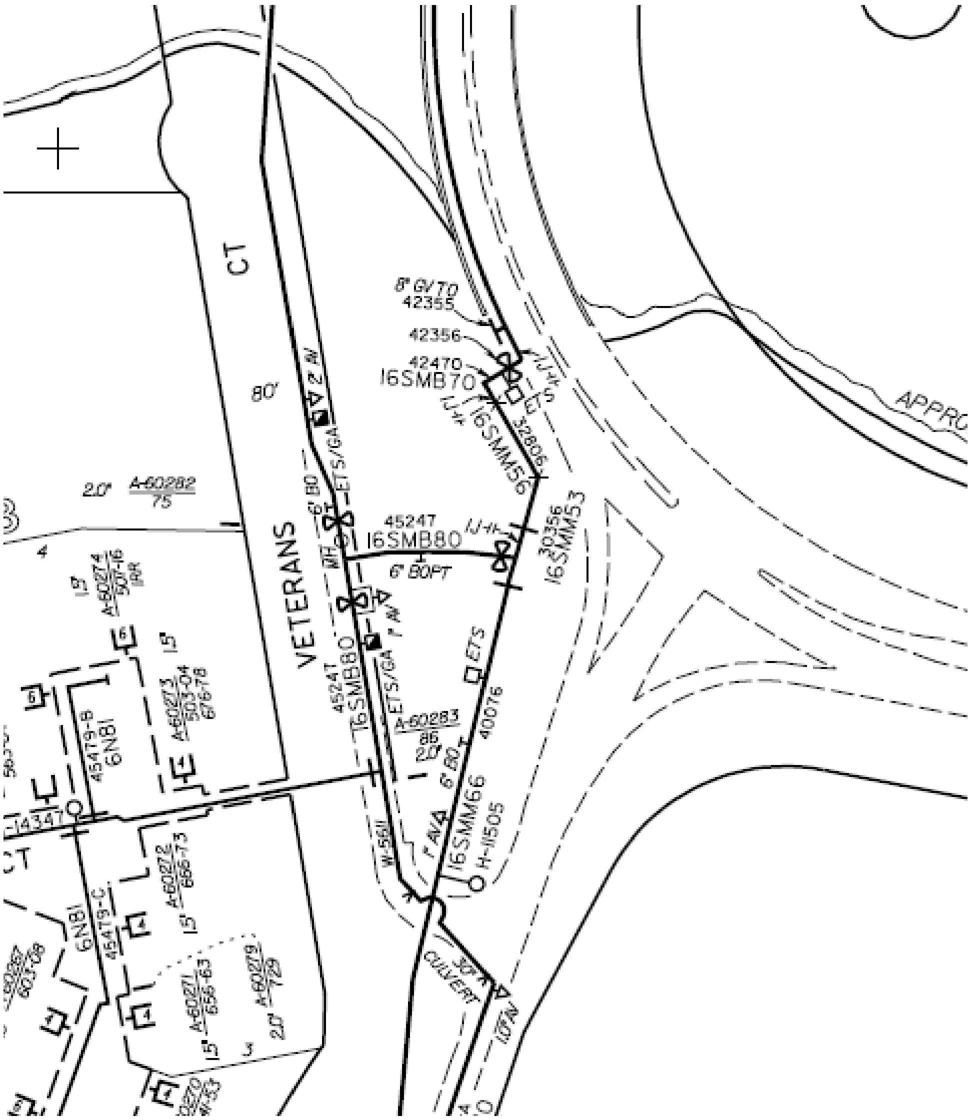


Figure 21: Alternative 2 – Veterans Court Connection Location

Figure 22 presents an inset of B-Map 1497B456 of the connection at the intersection of Robert Davey Jr. Drive and Island Drive.

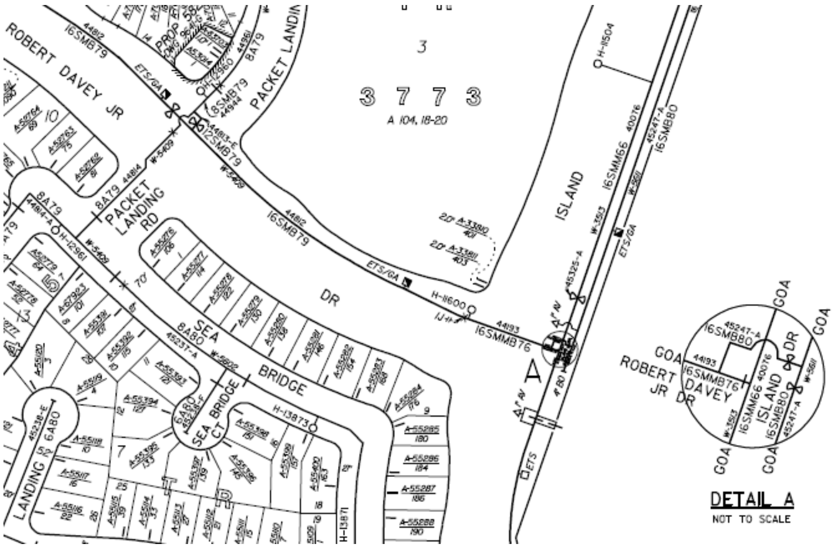


Figure 22: Alternative 2 – Robert Davey Jr and Island Dr. Instercation Connection Location

STREET ALIGNMENTS FOR TUNNEL ALTERNATIVE 2

Five alignments were identified on the Alameda site while three were identified on the Bay Farm Island side of the tunnel crossing. These alignments connect to open-cut buried pipe connection points. Plates 9 and 10 present GIS maps delineating the alignments described below.

<u>Alternative 2 – Alameda side:</u>

- Alignment 1: Approximately 8,300 feet – follows Lincoln Avenue to Pearl Street (in the south-west direction) to Otis Drive and connecting to the Bridge View Isle connection point.
- Alignment 2: Approximately 8,300 feet - follows Park Street (in the south- east direction) to Otis Drive and to connecting to the Bridge View Isle connection point connection point.
- Alignment 3: Approximately 8,300 feet - follows Park Street to High Street (traveling in the south-east direction) to Otis Drive and connecting to the Bridge View Isle connection point.
- Alignment 4: Approximately 8,300 feet - follows Park Street to Versailles Avenue (traveling in the south-east direction) to Otis Drive and connecting to the Bridge View Isle connection point.

- Alignment 5 – Approximately 8,300 feet - follows Park Street to Broadway (traveling in the south-east direction) to Otis Drive and connecting to the Bridge View Isle connection point.

<u>Alternative 2 – Bay Farm side:</u>

- Alignment 1: Approximately 1,800 feet – follows Veteran Court to the northbound lane of Island Drive to the connection point near the intersection of Robert Davey Jr. Drive and Island Drive.

- Alignment 2: Approximately 1,800 feet - follows Veteran Court to the southbound lane of Island Drive (traveling in the southern direction to the connection point near the intersection of Robert Davey Jr. Drive and Island Drive.

EASEMENTS FOR TUNNEL ALTERNATIVE 2

All the proposed alignments on the Alameda and Bay Farm side stay within the public right-of-way and will not require permanent or temporary construction easements.

UTILITY ISSUES FOR TUNNEL ALTERNATIVE 2

The Project Team coordinated with local utility departments to obtain current utility information for the alignments on the Alameda and Bay Farm side of the crossing. A summary of the high priority utilities are provided below. Plates 11 and 12 present the GIS figures for the Alameda and Bay Farm side Utility Crossings.

<u>Alameda side Utility Conflicts</u>

Alameda Municipal Power owns an underground 12kV transmission line that travels along the Otis Drive in the north-south direction. All of the proposed alignments cross this transmission main.

EBMUD owns the Alameda Interceptor that travels along Otis Drive in the north-south direction, continues to Bridgeview Isle and crosses the estuary to Bay Farm. All the proposed alignments will cross the interceptor near the intersection of Bridgeview Isle and Driftwood Lane. The Alameda Interceptor also travels along Pearl Street and Versailles Avenue in the north-south direction. Four alignments cross the Alameda Interceptor when it travels along Pearl Street and Versailles Avenue.

<u>Bay Farm side Utility Conflicts</u>

Alameda Municipal Power owns an underground 12kV transmission line that travels along Veterans Court, in the north –south direction, and then along the southbound lane of Island Drive. All of the proposed alignments cross the transmission main.

Table 6 provides a summary of the high priority utility crossings for Alternative 2 crossings at the Alameda and Bay Farm side.

The lengths listed in Table 6 followed by R/W (right-of-way) present the length of new pipeline alignment that falls within the same right-of-way as the utility. The separation clearances for high priority facility utilities ranges for electrical, gas, and petroleum lines and creates design issues within congested streets.

TABLE 6
Utility Crossing Summary for Street Alignments Associated with Tunnel Alternative 2

Alameda side	**EBMUD Interceptor**	**PG&E Electrical**	**PG&E Gas**	**Kinder Morgan**	**Alameda Municipal**
1	5 Crossing 3,400’ R/W	None	None	None	1 Crossing 300’ R/W
2	3 Crossing 6,000’ R/W	None	None	None	1 Crossing 300’ R/W
3	5 Crossing 1,500’ R/W	None	None	None	1 Crossing 300’ R/W
4	7 Crossing 6,600’ R/W	None	None	None	1 Crossing 300’ R/W
5	3 Crossing 2,000’ R/W	None	None	None	1 Crossing 300’ R/W
Bay Farm side	**EBMUD Interceptor**	**PG&E Electrical**	**PG&E Gas**	**Kinder Morgan**	**Alameda Municipal**
1	None	None	None	None	1 Crossing 600’ R/W
2	None	None	None	None	1 Crossing 1,800’ R/W

PERMITS FOR TUNNEL ALTERNATIVE 2

For the Alameda and Bay Farm pipelines in Alternative 2 the following permits are anticipated to be obtained for the construction of a new pipeline.

An excavation permit is required by the City of Alameda for each road or street segment excavated for the trench of the new pipeline. The number of permits can add up depending on the number of streets encountered along the pipeline alignment. In addition, a Traffic Control Plan may be required for the permit application, which may require help from a traffic consultant to assist with the Traffic Control Plan and contract documents: plans and specifications.

DESIGN AND CONSTRUCTION ISSUES FOR TUNNEL ALTERNATIVE 2

Traffic Control during Construction

All of the alignments considered in the alternatives analysis stay within city streets and are in residential and commercial areas. The District understands that additional assistance from a traffic control consultant may be required for the contract drawings and specifications to meet the requirements of the City of Alameda permit application process. During the design, the Project Design Team will meet and review the requirements for the proposed alignment.

Liquefaction Susceptibility

The Alameda-Oakland Estuary area falls into a liquefaction seismic hazard zone. Plates 13 and 14 present the Liquefaction Susceptibility zones from very low, low, moderate, high, to very high using the Maps of Quaternary Deposits and Liquefaction Susceptibility in the Central San Francisco Bay Region, California (Witter et al. 2006), for the Alameda and Bay Farm sides. The alignments fall into liquefaction susceptibility levels of moderate to very high. Table 7 presents the length of alignment in each level of liquefaction susceptibility and percentage for the Alameda and Bay Farm.

TABLE 7
Alameda and Bay Farm Sides Liquefaction Susceptibility Summary

Street Alignments		Liquefaction Susceptibility Level			
Alameda side	**Total Length [Feet]**	**Moderate Length [Feet]**	**Moderate Percent [%]**	**Very High Length [Feet]**	**Very High Percent [%]**
1	8,300	5,500	64%	2,800	34%
2	8,300	4,000	48%	4,300	52%
3	8,300	6,500	78%	1,800	22%
4	8,300	5,800	70%	2,500	30%
5	8,300	5,300	64%	3,000	36%
Bay Farm side	**Total Length [Feet]**	**Moderate Length [Feet]**	**Moderate Percent [%]**	**Very High Length [Feet]**	**Very High Percent [%]**
1	1,800	0	0%	1,800	100%
2	1,800	0	0%	1,800	100%

Geotracker Information

Geotracker sites in the vicinity of Tunnel Crossing Alternative 2 (i.e., in the vicinity of Otis Drive and Doolittle Drives) are shown in Figure 23.

Figure 23: Geotracker Sites near Otis and Doolittle Drives

COST ESTIMATES FOR TUNNEL ALTERNATIVE 2

The District collected cost estimates from two pipe manufacturers and three local Bay Area construction contractors to understand the unit cost for pipeline installation for the Alameda and Bay Farm side Alternative 2 alignments. Table 3 provides a tabular summary of the unit cost used throughout the alternatives analysis report. Table 3 provides the unit costs for a 24-inch diameter steel pipeline with 3/16-inch wall thickness at 42 ksi minimum yield strength. Tables 8 and 9 present the conceptual level construction costs for Alternative 2 Crossing Alignments.

STREET ALIGNMENT SUMMARY AND RECOMMENDATIONS FOR TUNNEL ALTERNATIVE 2

Summary

Tables 8 and 9 provide the summary matrix for the Alameda side and Bay Farm side Alternative 2 Alignments. The matrix includes the length, cost, pros, and cons for each alignment option and the previously discussed utility crossing and alignment concerns. The liquefaction susceptibility percentages provide the ratio of alignment length in the very high level of liquefaction susceptibility.

TABLE 8
Street Alignment Summary for Tunnel Alternative 2, Alameda side

Alignment #	Length [feet]	Cost [$]	Pros	Cons
1	8,300	$3.6M	LIQ 34% Very High In Public Right-of-Way	3,400’ Utility R/W 6 Utility Crossings
2	8,300	$3.6M	In Public Right-of-Way	3 Utility Crossings LIQ 52% Very High
3	8,300	$3.9M	LIQ 22% Very High In Public Right-of-Way	6 Utility Crossings
4	8,300	$3.9M	LIQ 30% Very High In Public Right-of-Way	8 Utility Crossings 6,600’ Utility R/W
5	8,300	$3.9M	In Public Right-of-Way LIQ 36% Very High	3 Utility Crossings

TABLE 9
Street Alignment Summary for Tunnel Alternative 2, Bay Farm side

Alignment #	Length [feet]	Cost [$]	Pros	Cons
1	1,800	$0.8M	Length Constant	LIQ 100% Very High
2	1,800	$0.8M	Length Constant	LIQ 100% Very High Has same alignment as 12kV transmission line

<u>Recommendations</u>

The intent of the Alternatives Analysis is to present some potential open trench alignment connection options for the various estuary crossings and provide initial recommendations to assist with the design phase. Street alignment recommendations are listed below for both sides of the estuary crossing.

Alternatives 2 – Alameda side. Alignments 3 or 5 appear to be the best alignment options. Alignment 3 is located in the least amount of very high liquefiable soil but has a moderate number of high priority utility crossings. Alignment 5 is located in a moderate amount of very high liquefiable soil but has the lowest amount of high priority utility crossings.

Alternatives 2 – Bay Farm side. Alignments 1 and 2 are very comparable. Both are located in very high liquefiable soil and have the same construction costs. Alignment 2 travels in the same right-of-way as an underground 12kV electrical transmission main but the alignment can be designed to minimize or eliminate any conflicts.

CONNECTION LOCATIONS FOR TUNNEL ALTERNATIVE 3

Alternative 3 - Oakland Connections

The estuary crossing on the Oakland side will connect to a 24SBM35 at the south end of Derby Avenue and eventually extend northwesterly toward the intersection of 29th Avenue and Ford Street via either Glasscock Street and/or Ford Street. Figure 24 presents the B-map extent (B-map 1497B468) of the 29th Avenue and Ford Street connection point.

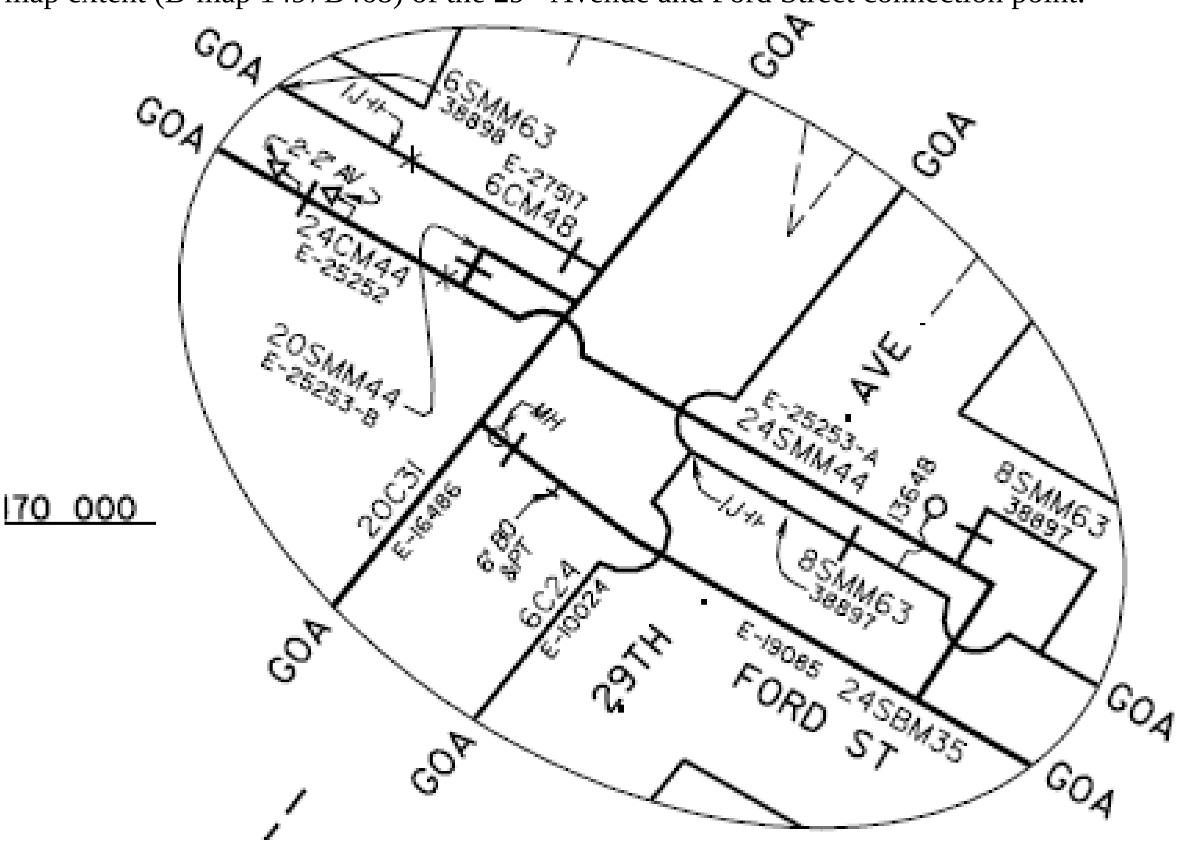


Figure 24: Alternative 3 – Ford St. and 29th Ave. Connection Location

Figure 25 shows the B-map extent (B-map 1497B466) of the connection point at the end of Derby Avenue at the estuary.

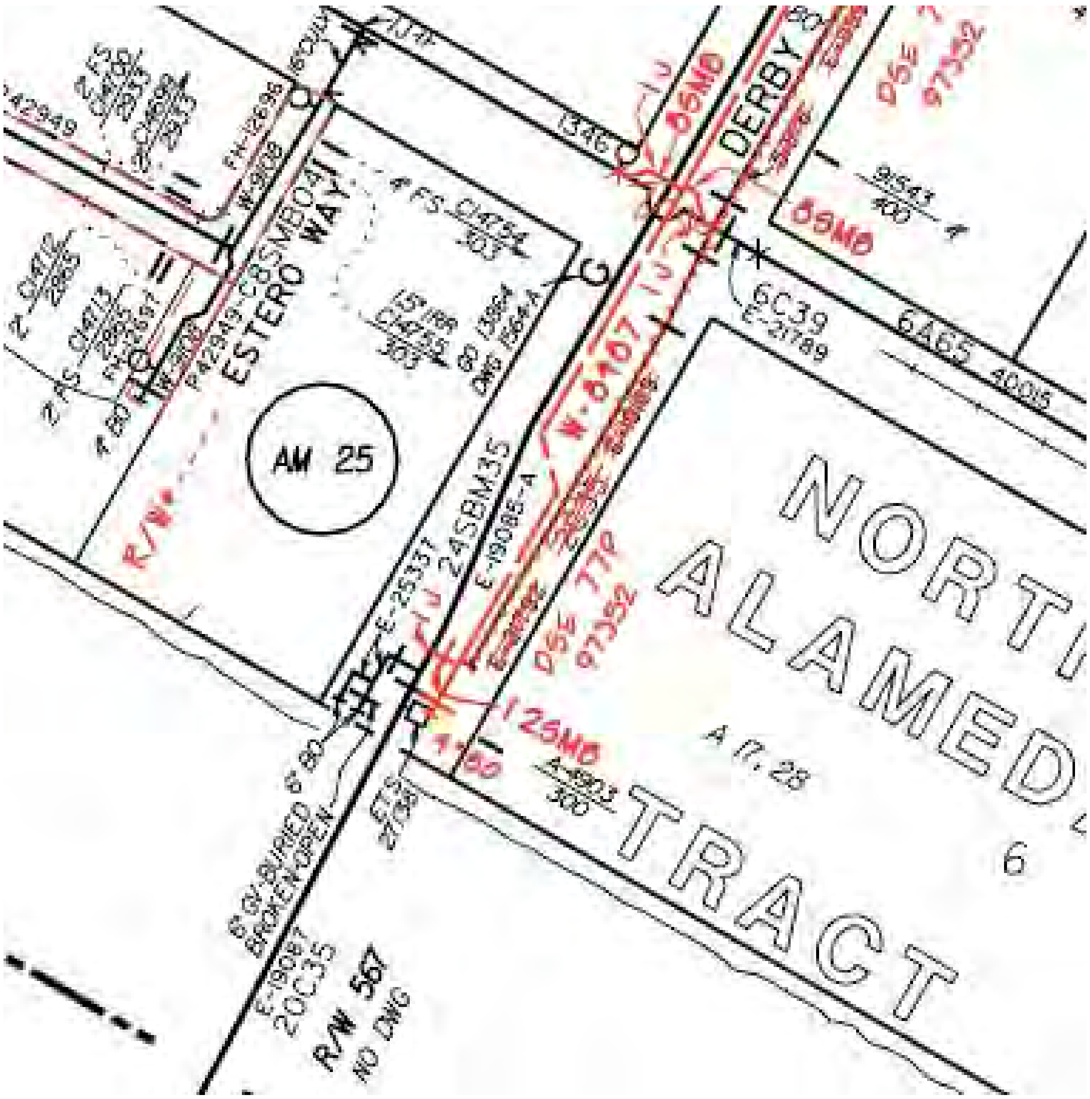


Figure 25: Alternative 3 – End of Derby Avenue Connection Location

Alternative 3 - Alameda Connection

The tunnel crossing on the Alameda side will connect to the 24SBM35 steel main in the Bridgeside Shopping Center located at the corner of Blanding and Broadway (directly across the estuary from Derby Ave) and ultimately extend southwesterly, connecting to a 24-inch steel at the intersection of Lincoln Avenue and Park Street. Figure 26 presents the B-map extent (B-map 1497B468) of the Bridgeside Shopping Center connection.

Figure 26: Alternative 3 – Bridgeside Shopping Center Connection Location

The connection point will occur in the 20-foot right-of-way (R/W 2636) which is located in the shopping center parking lot. Figure 27 shows the location in the parking lot where the submarine tunnel pipeline would connect to the on-shore pipeline.

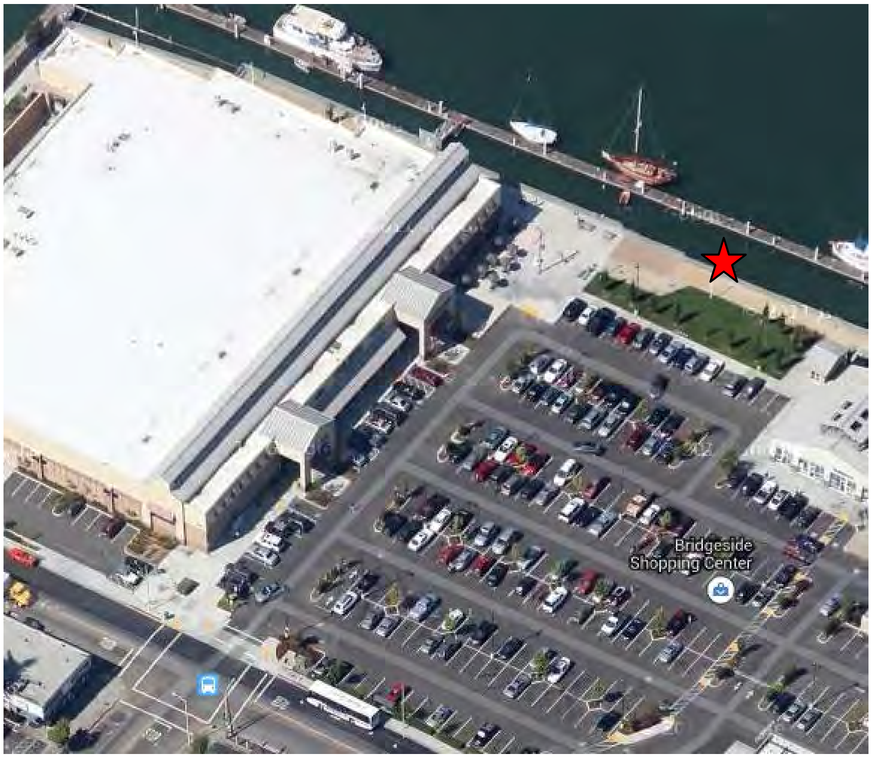


Figure 27: Alternative 3 – Bridgeside Shopping Center Parking Lot Location

Figure 28 presents the B-map extent (B-map 1497B468) of the Lincoln Avenue and Park Street connection.

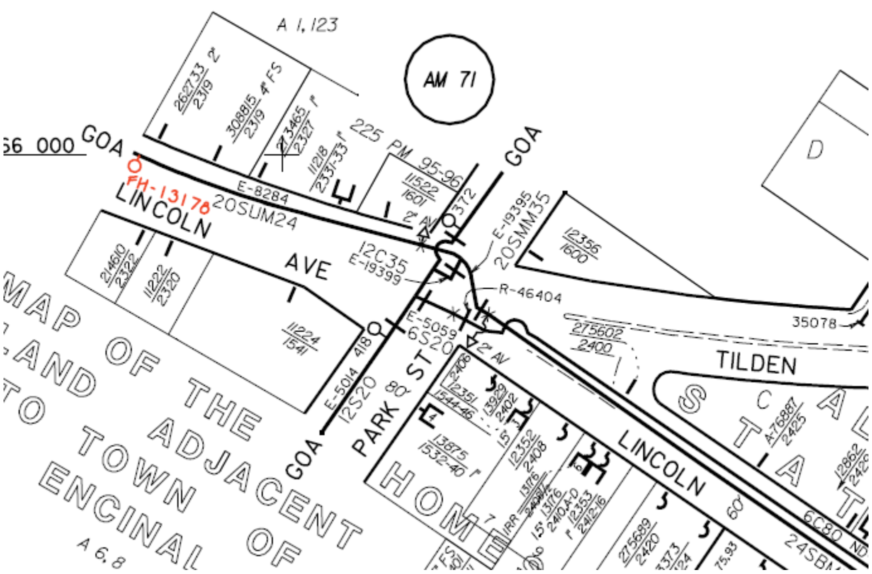


Figure 28: Alternative 3 – Lincoln Avenue and Park Street Connection Location

STREET ALIGNMENTS FOR TUNNEL ALTERNATIVES 3

Several street alignments were identified on the Oakland side of the tunnel crossing as well as the Alameda side of the crossing. These alignments connect to open-cut buried pipe connection points. Plates 15 and 16 present GIS maps delineating the alignments described below.

Alternative 3 – Oakland side:

- Alignment 1: Approximately 3,400 feet – follows Ford Street (traveling east) to Derby Avenue (traveling south) and then connects at the end of Derby Avenue.

- Alignment 2: Approximately 3,400 feet - follows 29th Avenue to Glascock Street to Derby Avenue (traveling south) and connects at the end of Derby Avenue near the estuary.

- Alignment 3 – Approximately 3,400 feet - follows Ford Street (traveling east) to Peterson Street (traveling south) to Glascock (traveling east) to Derby Avenue (traveling south) and connect at the end of Derby Avenue.

Alternative 3 – Alameda side:

- Alignment 1: Approximately 3,800 feet – travels through the Bridgeside Shopping Center parking lot (R/W 2636) and continues on Broadway to Lincoln Ave (travels west) and connects at Lincoln Avenue and Park Street.

- Alignment 2: Approximately 3,800 feet – travels through the Bridgeside Shopping Center parking lot (R/W 2636) to Blanding Avenue (travels west) to Park Street (travels south) and connect at the intersection of Lincoln Avenue and Park Street.

- Alignment 3: Approximately 3,800 feet – travels through the Bridgeside Shopping Center parking lot (R/W 2636) to Clement Avenue (travels west) to Park Street (travels south) and connect at the intersection of Lincoln Avenue and Park Street.

- Alignment 4: Approximately 3,800 feet – travels through the Bridgeside Shopping Center parking lot (R/W 2636) to Eagle Avenue (travels west) to Park Street (travels south) and connect at the intersection of Lincoln Avenue and Park Street.

EASEMENTS FOR TUNNEL ALTERNATIVE 3

All of the proposed alignments on the Oakland side are within the public right-of-way and will require encroachment permits. Temporary construction easements may also be needed**.**

The proposed alignment on the Alameda side will stay within the public right-of-way and the established right-of-way (R/W 2636) in the Bridgeside Shopping Center parking lot. No additional surface easements will be required for this project. A temporary construction easement will be required when installing pipeline in R/W 2636 to ensure adequate construction space.

UTILITY ISSUES FOR TUNNEL ALTERNATIVE 3

The Project Team coordinated with local utility departments to obtain current utility information for the alignments on the Alameda and Bay Farm side of the crossing. A summary of the high priority utilities are provided below. Plates 17 and 18 present the GIS figures for the Oakland and Alameda Utility Crossings.

<u>Oakland side Utility Conflicts</u>

PG&E owns two 24-inch gas transmission mains in the area of this crossing. One main travels along East 7th Street and the other travels along Chapman. These large gas mains do not directly conflict with the proposed alignment but are near the general project work area.

<u>Alameda side Utility Conflicts</u>

Alameda Municipal Power has an underground 12kV transmission line travels along Broadway, in the north-south direction, between Blanding Avenue and Tilden Way.

EBMUD owns the Alameda Interceptor which travels along Clement Avenue in the east-west direction and travels along Broadway in the north-south direction.

Table 10 provides a summary of the utility crossings associated with Tunnel Alternative 3. The lengths listed in Table 10 followed by R/W (right-of-way) present the length of new pipeline alignment that falls within the same right-of-way as the utility. The separation clearances for high priority facility utilities varies for electrical, gas, and petroleum lines and creates design issues within congested streets.

TABLE 10
Utility Crossing Summary for Street Alignments Associated with Tunnel Alternative 3

Oakland side	EBMUD Interceptor	PG&E Electrical	PG&E Gas	Kinder Morgan	Alameda Municipal
1	None	None	None	None	None
2	None	None	None	None	None
3	None	None	None	None	None
Alameda side	**EBMUD Interceptor**	**PG&E Electrical**	**PG&E Gas**	**Kinder Morgan**	**Alameda Municipal**
1	2 Crossings 1,400’ R/W	None	None	None	700’ R/W
2	1 Crossing	None	None	None	None
3	2 Crossings 1,200’ R/W	None	None	None	350’ R/W
4	2 Crossings 300’ R/W	None	None	None	1 Crossing 700’ R/W

PERMITS FOR TUNNEL ALTERNATIVE 3

For the Oakland and Alameda Street Alignments, the following permits are anticipated to be obtained for the construction of a new pipeline:

City of Oakland

A Utility Excavation Permit is required by the City of Oakland for each road or street segment excavated for the new pipeline. The District understands that assistance from a traffic control consultant may be needed to prepare the contract documents and permit application. Staff will also meet with the permitting agencies during the Design phase.

For more information on the requirements, refer to the City of Oakland – Utility Excavation Permit Application Checklist.

City of Alameda

An excavation permit is required by the City of Alameda for each road or street segment excavated for the new pipeline. In addition, a Traffic Control Plan will also be required as noted above for work in the City of Oakland.

The District has a right-of-way (R/W 2636) in the Bridgeside Shopping Center parking and will need to coordinate construction activities with the local businesses and community given the short-term construction impacts.

DESIGN AND CONSTRUCTION ISSUES FOR TUNNEL ALTERNATIVE 3

Traffic Control during Construction

All of the street alignments considered in this analysis need to meet the requirements of the City of Oakland and City of Alameda permit application process. Staff will meet with the respective agencies as part of the design process.

Liquefaction Susceptibility

The Alameda-Oakland Estuary area falls into a liquefaction seismic hazard zone. Plates 19 and 20 present the Liquefaction Susceptibility zones from very low, low, moderate, high, to very high using the Maps of Quaternary Deposits and Liquefaction Susceptibility in the Central San Francisco Bay Region, California (Witter et al. 2006) for the Oakland and Alameda side Alternative 3 Alignments. The Oakland and Alameda alternative alignments fall into liquefaction susceptibility levels of moderate. Table 11 presents the length of alignment in each level of liquefaction susceptibility and percentage for the Oakland and Alameda sides.

TABLE 11
Oakland and Alameda Sides Liquefaction Susceptibility Summary

Street Alignments		Liquefaction Susceptibility Level			
Oakland side	**Total Length [Feet]**	**Moderate Length [Feet]**	**Moderate Percent [%]**	**Very High Length [Feet]**	**Very High Percent [%]**
1	3,400	3,400	100%	N/A	N/A
2	3,400	3,400	100%	N/A	N/A
3	3,400	3,400	100%	N/A	N/A
Alameda side	**Total Length [Feet]**	**Moderate Length [Feet]**	**Moderate Percent [%]**	**Very High Length [Feet]**	**Very High Percent [%]**
1	3,800	3,800	100%	N/A	N/A
2	3,800	3,800	100%	N/A	N/A
3	3,800	3,800	100%	N/A	N/A
4	3,800	3,800	100%	N/A	N/A

Geotracker Information

Geotracker sites on both the Alameda and Oakland side of the estuary are shown in Figure 29. Listed sites on Broadway in Alameda and Derby Avenue in Oakland will need to be considered in the preliminary design of the tunnel crossing alternatives at this location.

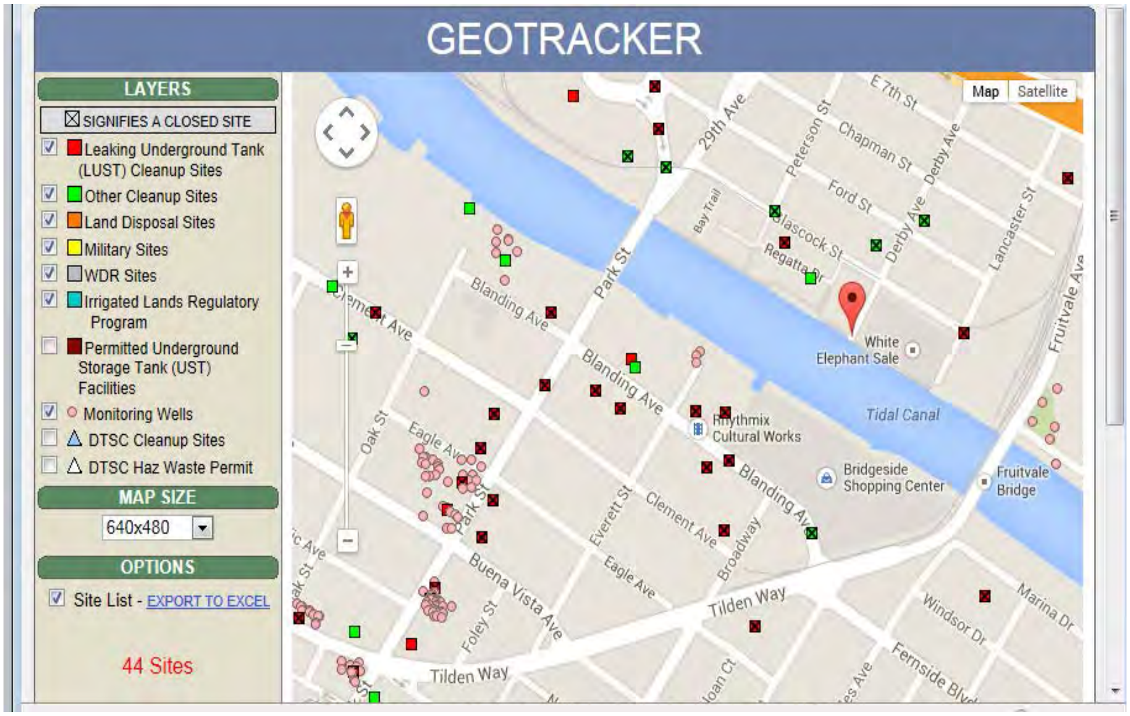


Figure 29 Geotracker Information, Park and Fruitvale Bridge Area

COST ESTIMATES FOR TUNNEL ALTERNATIVE 3

The District collected cost estimates from two pipe manufacturers and three local Bay-Area construction contractors to understand the unit cost for pipeline installation for the Oakland and Alameda side Alternative 3 Alignments. Table 3 provides a tabular summary of the unit cost used throughout the alternatives analysis report based on a 24-inch diameter steel pipeline with 3/16-inch wall thickness at 42 ksi minimum yield strength. Tables 12 and 13 present the conceptual level construction costs for each alternative 3 Crossing Alternative Alignment.

STREET ALIGNMENT SUMMARY AND RECOMMEDATIONS FOR TUNNEL ALTERNATIVE 3

<u>Summary</u>

Tables 12 and 13 provide the summary matrix for the Oakland and Alameda side Alternative 3 Alignments, respectively. The matrix includes the length, cost, pros, and cons for each alignment option and the previously discussed utility crossing and alignment concerns. The liquefaction susceptibility percentages provide the ratio of alignment length in the very high level of liquefaction susceptibility**.**

TABLE 12
Street Alignment Summary, Oakland Side

Alignment #	Length [feet]	Cost [$]	Pros	Cons
1	3,400	$1.5M	LIQ 0% Very High No High Priority Utility Crossings	
2	3,400	$1.5M	LIQ 0% Very High No High Priority Utility Crossings	
3	3,400	$1.5M	LIQ 0% Very High No High Priority Utility Crossings	

TABLE 13
Street Alignment Summary, Alameda Side Summary Matrix

Alignment #	Length [feet]	Cost [$]	Pros	Cons
1	3,800	$1.6M	LIQ 0% Very High	
2	3,800	$1.6M	LIQ 0% Very High	
3	3,800	$1.6M	LIQ 0% Very High	
4	3,800	$1.6M	LIQ 0% Very High	3 Utility Crossings 700 Utility R/W

Recommendations

The intent of the Alternatives Analysis is to present some potential open trench alignment connection options for the various estuary crossings and provide initial recommendations to assist with the design phase. The Pipeline Design Team will review the recommendations provided in this document and continue the design past the conceptual level. The following recommendations for Alternative 3 are provided below for the Oakland and Alameda sides.

Alternatives 3 – Oakland side. All of the proposed alignment options are very comparable. There is no one issue that puts one alignment above another. The Pipeline Design Team will have to complete a further review of each proposed alignment.

Alternatives 3 – Alameda side. All of the proposed alignment options are very comparable. There is no one issue that puts one alignment above another. The Pipeline Design Team will have to complete a further review of each proposed alignment.


ALICE - WEBSTER
OAKLAND
880
880
ALAMEDA
BLANDING
PARK
DERBY
77
77
HIGH
N
W
E
S
ALAMEDA - BAY FARM 2
ALAMEDA - BAY FARM 1
BAY FARM
0
0.25
0.5
1
Miles
EBMUD
Plate 1
Area Overview
Alameda-North Bay Farm Island
Crossings Master Plan
Pipeline Alternatives Analysis
Transmission Mains
Distribution Mains
Crossing Status
In Service
Out of Service

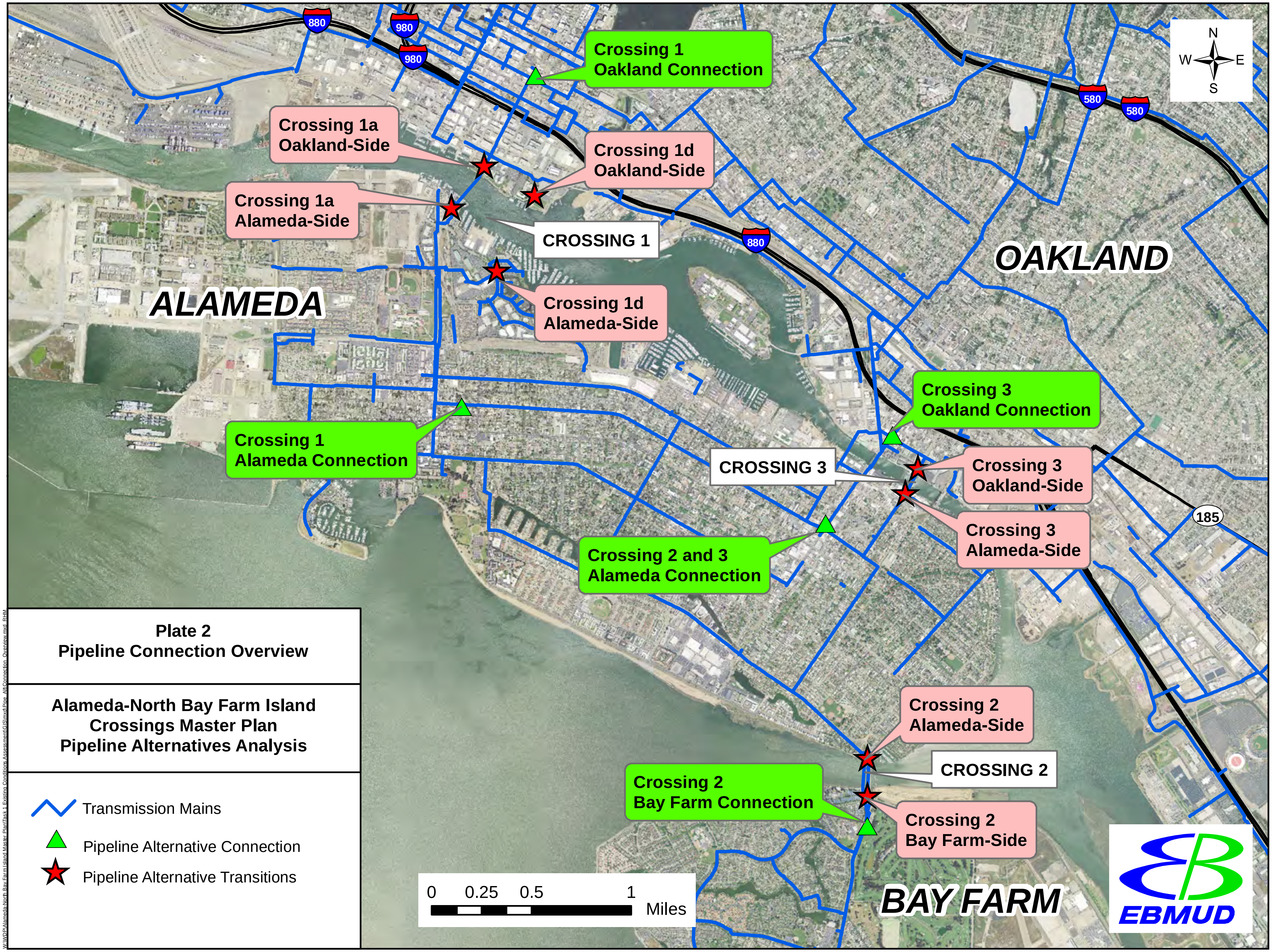


Plate 2
Pipeline Connection Overview

Alameda-North Bay Farm Island
Crossings Master Plan
Pipeline Alternatives Analysis

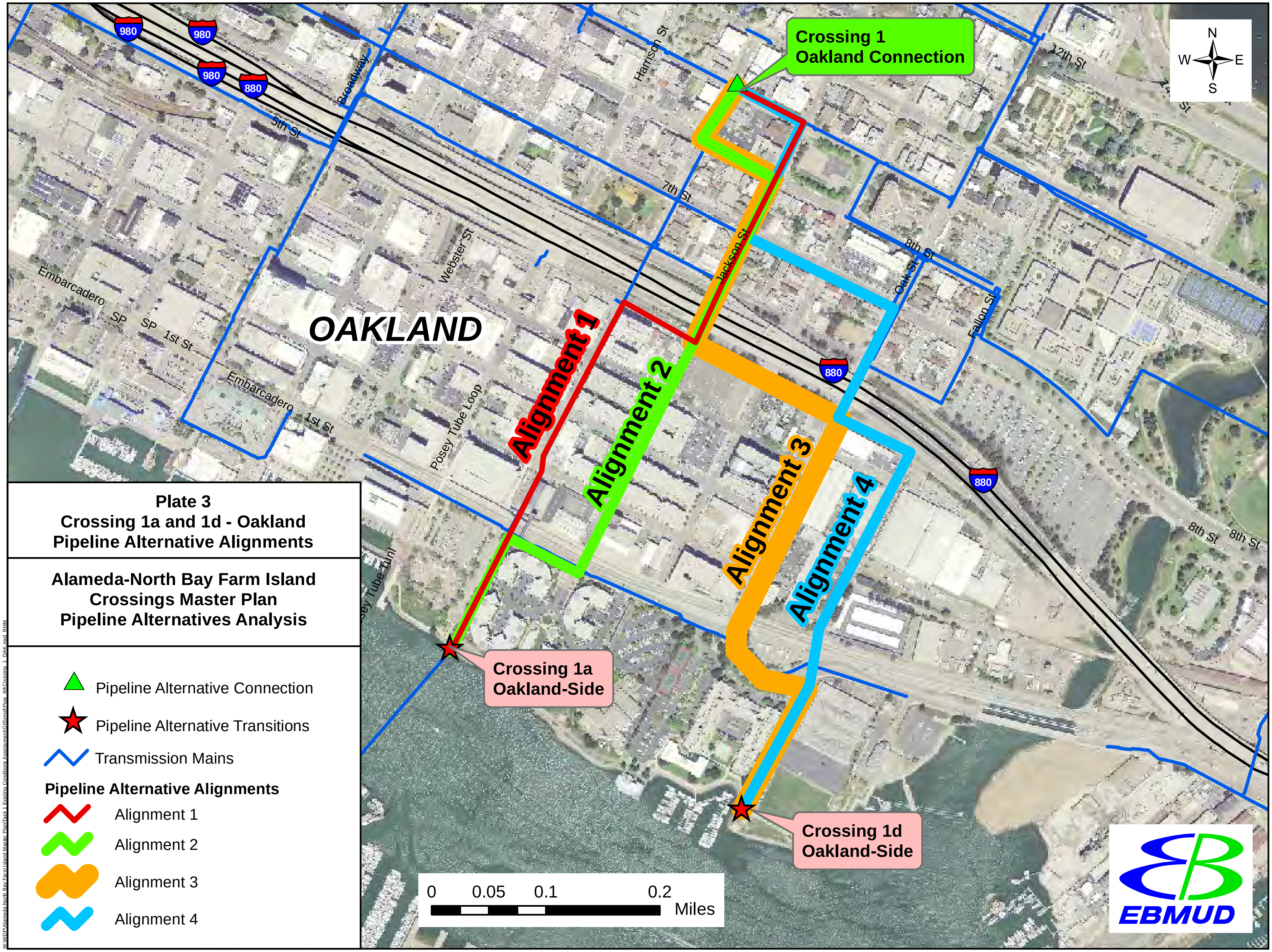


Plate 3
Crossing 1a and 1d - Oakland
Pipeline Alternative Alignments
Alameda-North Bay Farm Island
Crossings Master Plan
Pipeline Alternatives Analysis
Pipeline Alternative Connection
Pipeline Alternative Transitions
Transmission Mains
Pipeline Alternative Alignments
Alignment 1
Alignment 2
Alignment 3
Alignment 4
Crossing 1
Oakland Connection
Crossing 1a
Oakland-Side
Crossing 1d
Oakland-Side
OAKLAND
Alignment 1
Alignment 2
Alignment 3
Alignment 4
980
880
5th St
7th St
8th St
12th St
Broadway
Harrison St
Jackson St
Webster St
Oak St
Fallon St
Posey Tube Loop
Embarcadero
SP
SP 1st St
Embarcadero 1st St
0 0.05 0.1 0.2
Miles
N
W
E
S
EBMUD

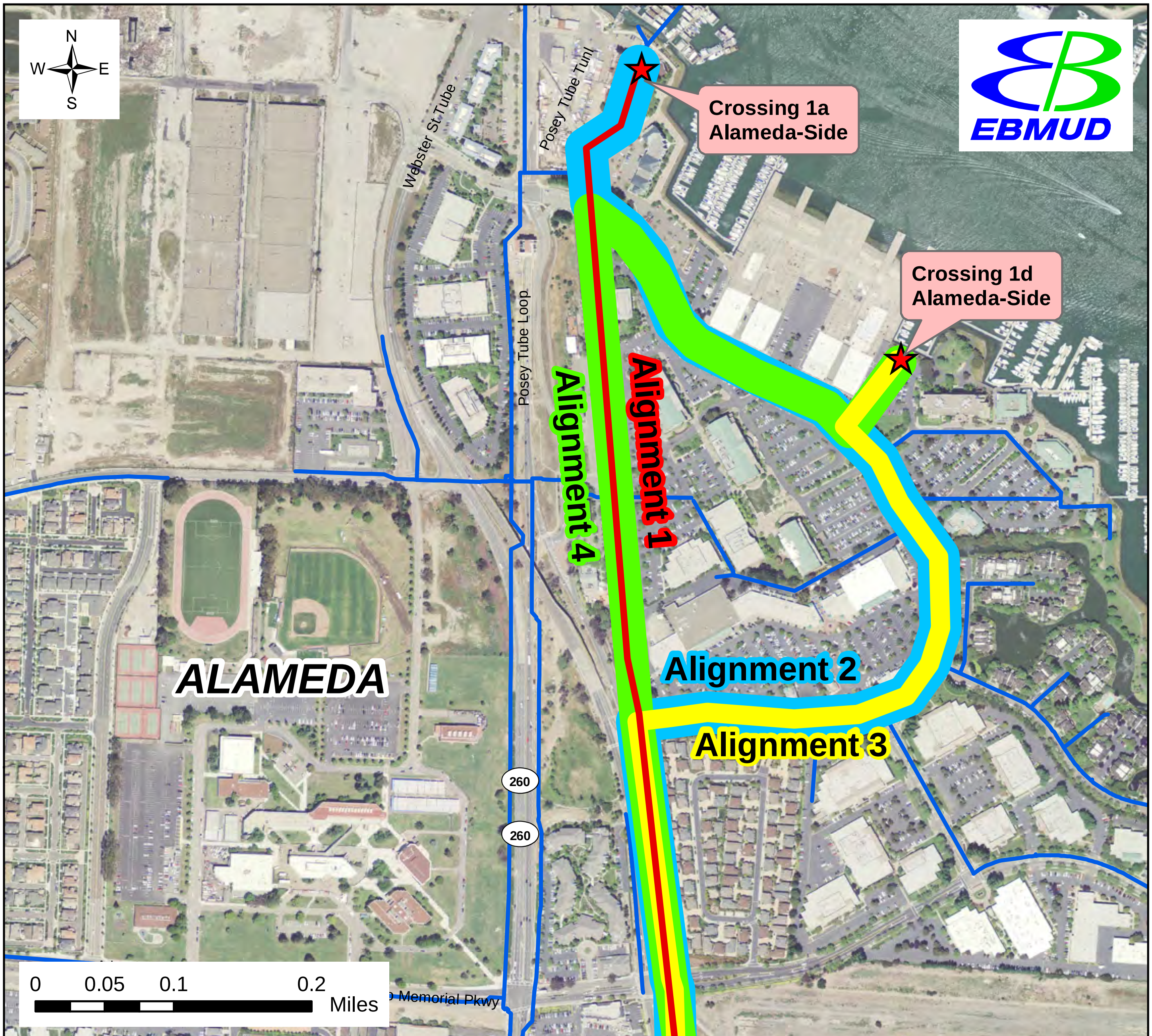


Plate 4
Crossing 1a and 1d - Alameda
Pipeline Alternative Alignments

Alameda-North Bay Farm Island
Crossings Master Plan
Pipeline Alternatives Analysis

Pipeline Alternative Connection
Pipeline Alternative Transitions

Pipeline Alternative Alignments

Alignment 1
Alignment 2
Alignment 3
Alignment 4

W:\WDP\Alameda-North Bay Farm Island Master Plan\Task 1 Existing Conditions Assessment\GIS\mxd\Pipe_Alt\Crossing_1_ALA.mxd RHM

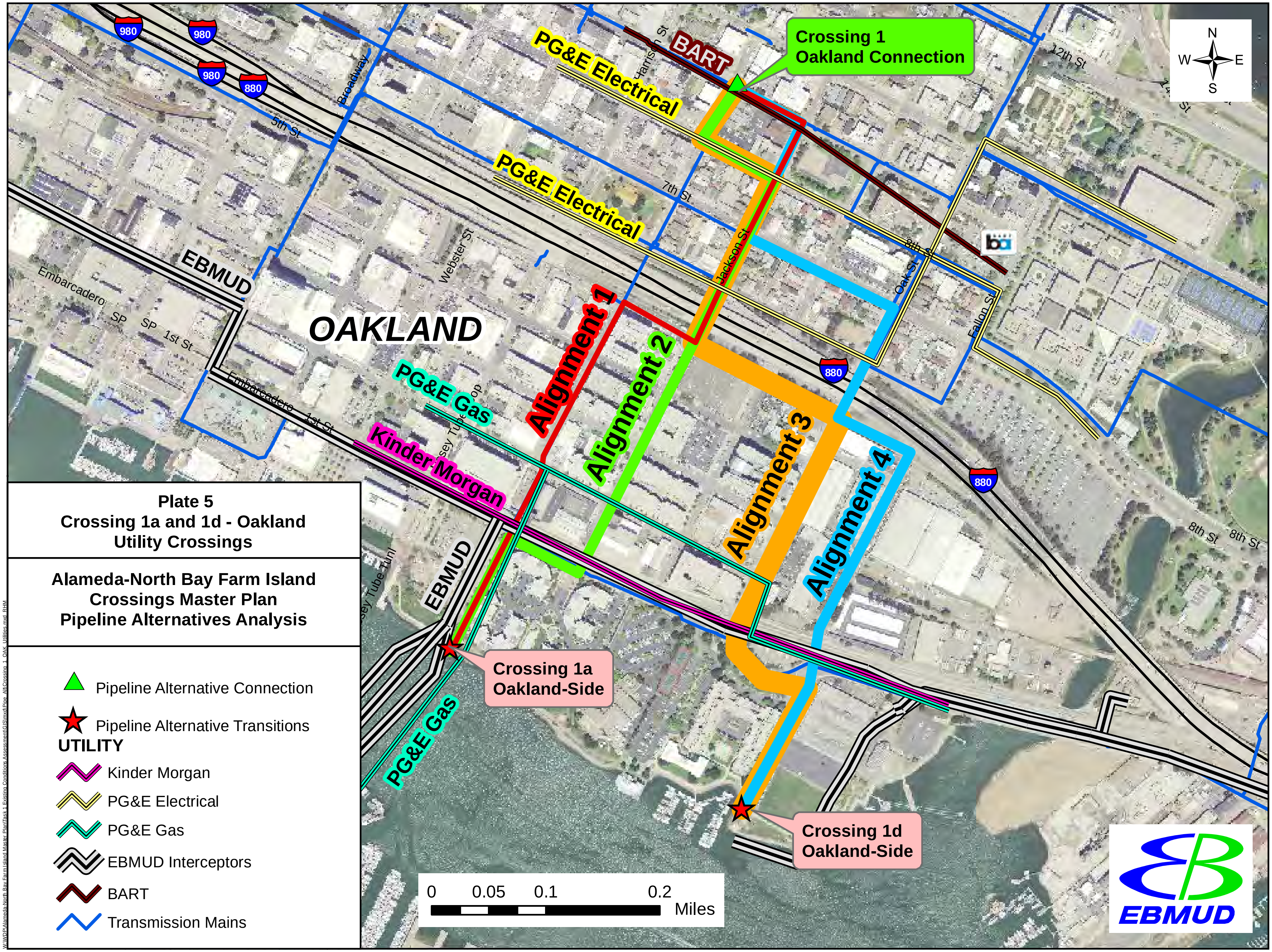


Plate 5
Crossing 1a and 1d - Oakland
Utility Crossings
Alameda-North Bay Farm Island
Crossings Master Plan
Pipeline Alternatives Analysis
Pipeline Alternative Connection
Pipeline Alternative Transitions
UTILITY
Kinder Morgan
PG&E Electrical
PG&E Gas
EBMUD Interceptors
BART
Transmission Mains
Crossing 1
Oakland Connection
Crossing 1a
Oakland-Side
Crossing 1d
Oakland-Side
Alignment 1
Alignment 2
Alignment 3
Alignment 4
OAKLAND
PG&E Electrical
PG&E Electrical
PG&E Gas
PG&E Gas
Kinder Morgan
BART
EBMUD
EBMUD
980
980
880
880
880
5th St
7th St
8th St
8th St
8th St
12th St
Broadway
Webster St
Harrison St
Jackson St
Oak St
Fallon St
Embarcadero
Embarcadero
SP
SP
1st St
1st St
N
S
E
W
0
0.05
0.1
0.2
Miles
EBMUD

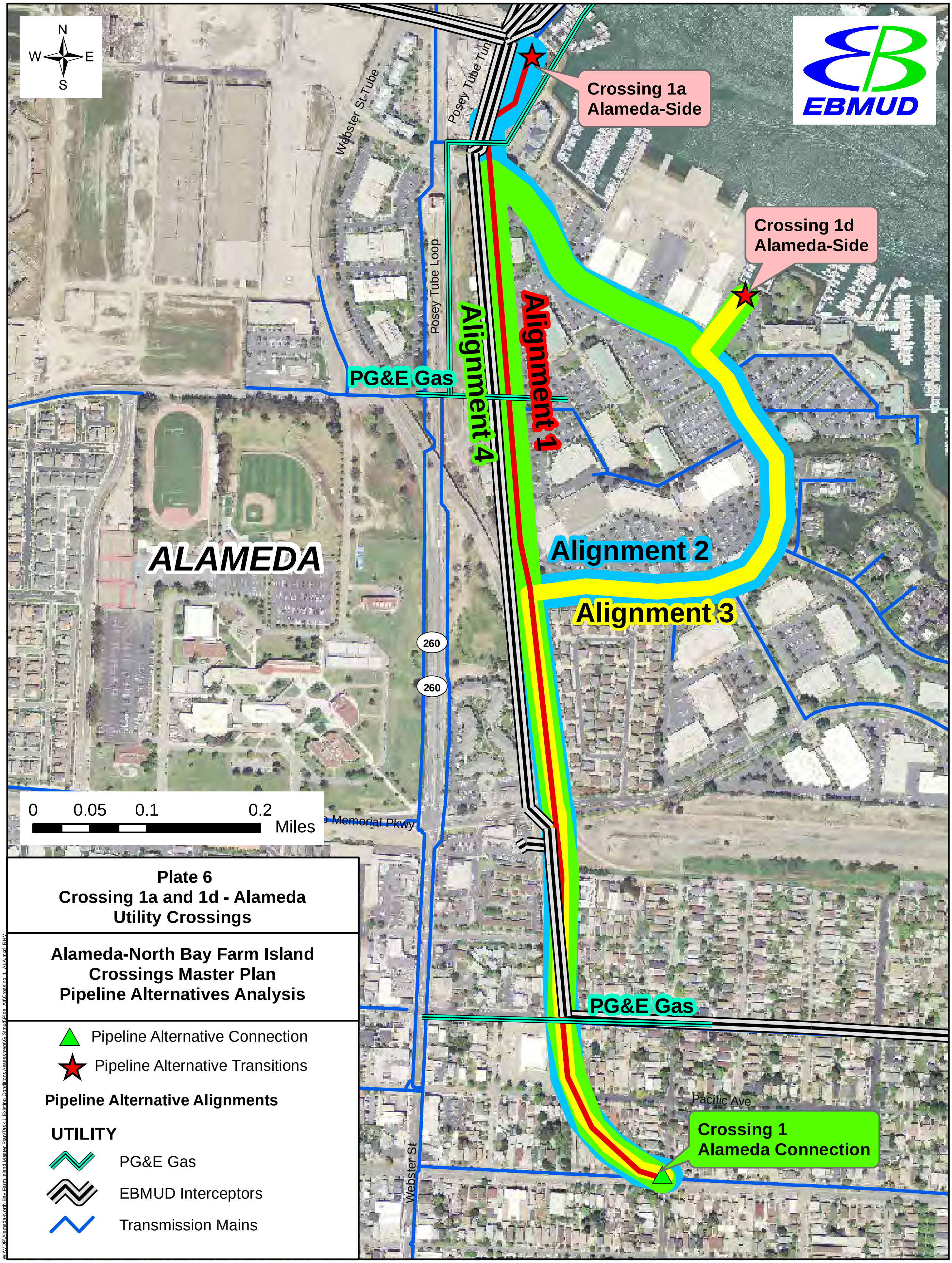


Plate 6
Crossing 1a and 1d - Alameda
Utility Crossings

Alameda-North Bay Farm Island
Crossings Master Plan
Pipeline Alternatives Analysis

Pipeline Alternative Connection

Pipeline Alternative Transitions

Pipeline Alternative Alignments

UTILITY

PG&E Gas

EBMUD Interceptors

Transmission Mains

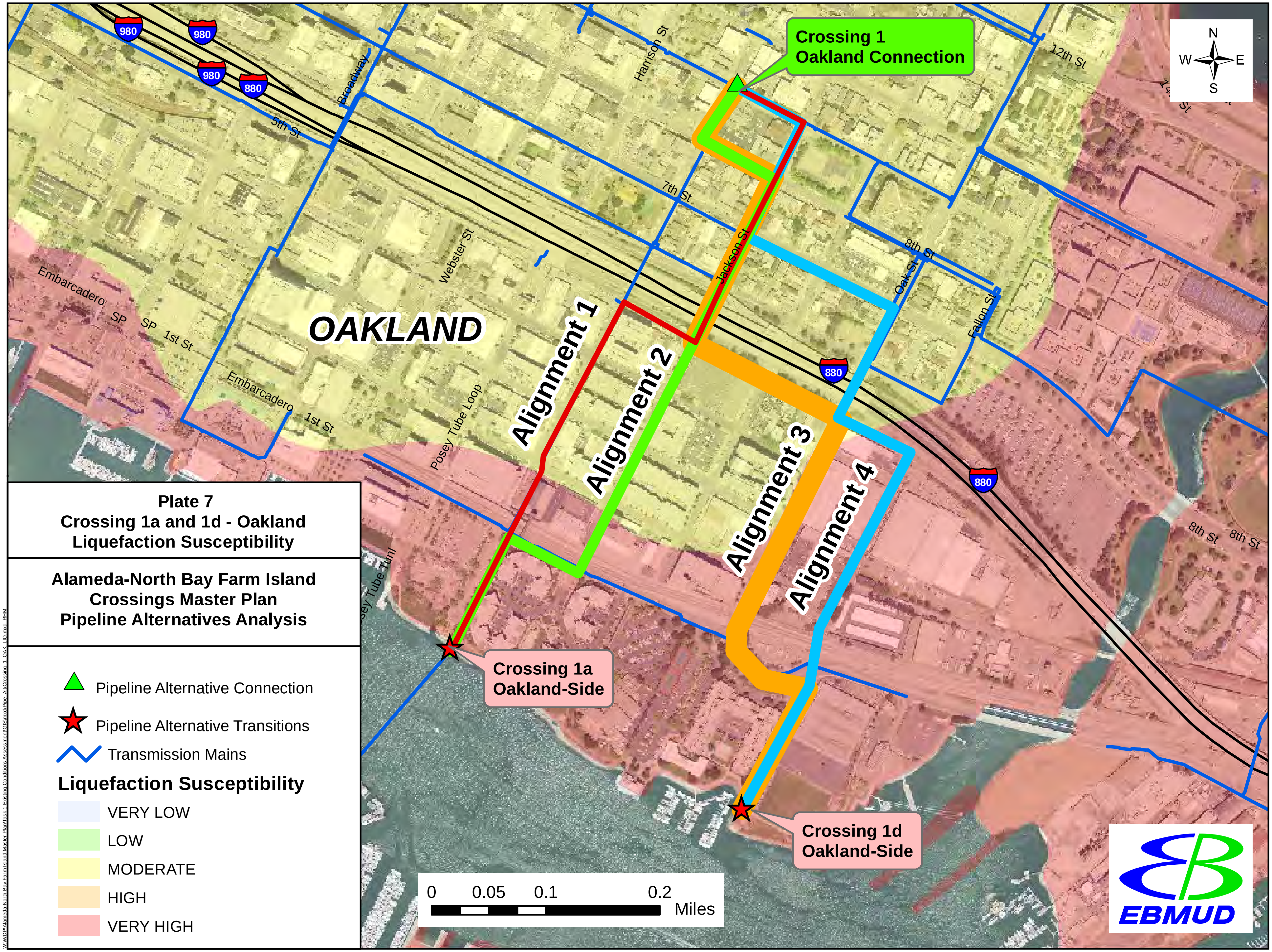


Plate 7
Crossing 1a and 1d - Oakland
Liquefaction Susceptibility

Alameda-North Bay Farm Island
Crossings Master Plan
Pipeline Alternatives Analysis

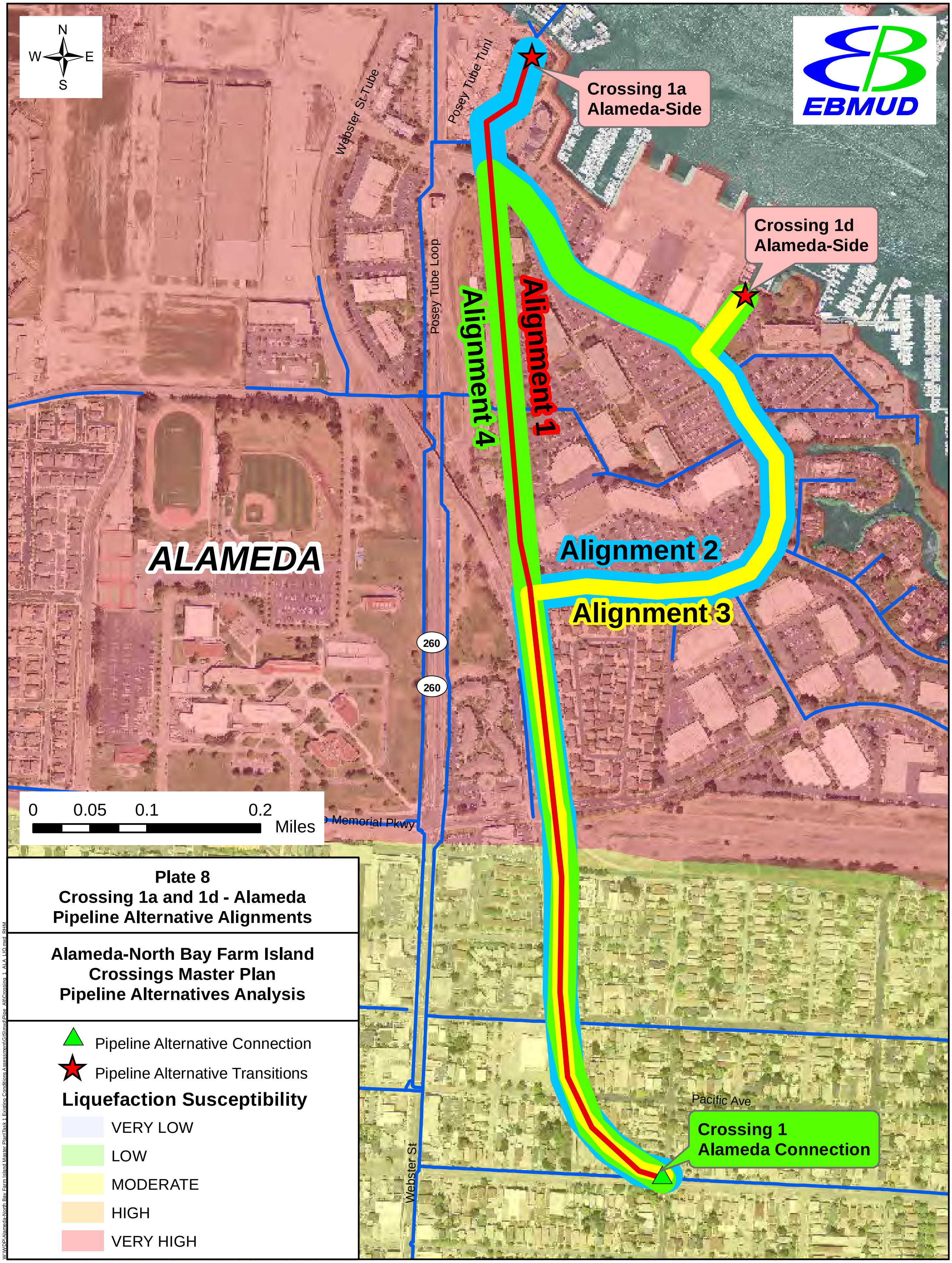


Plate 8
Crossing 1a and 1d - Alameda
Pipeline Alternative Alignments

Alameda-North Bay Farm Island
Crossings Master Plan
Pipeline Alternatives Analysis

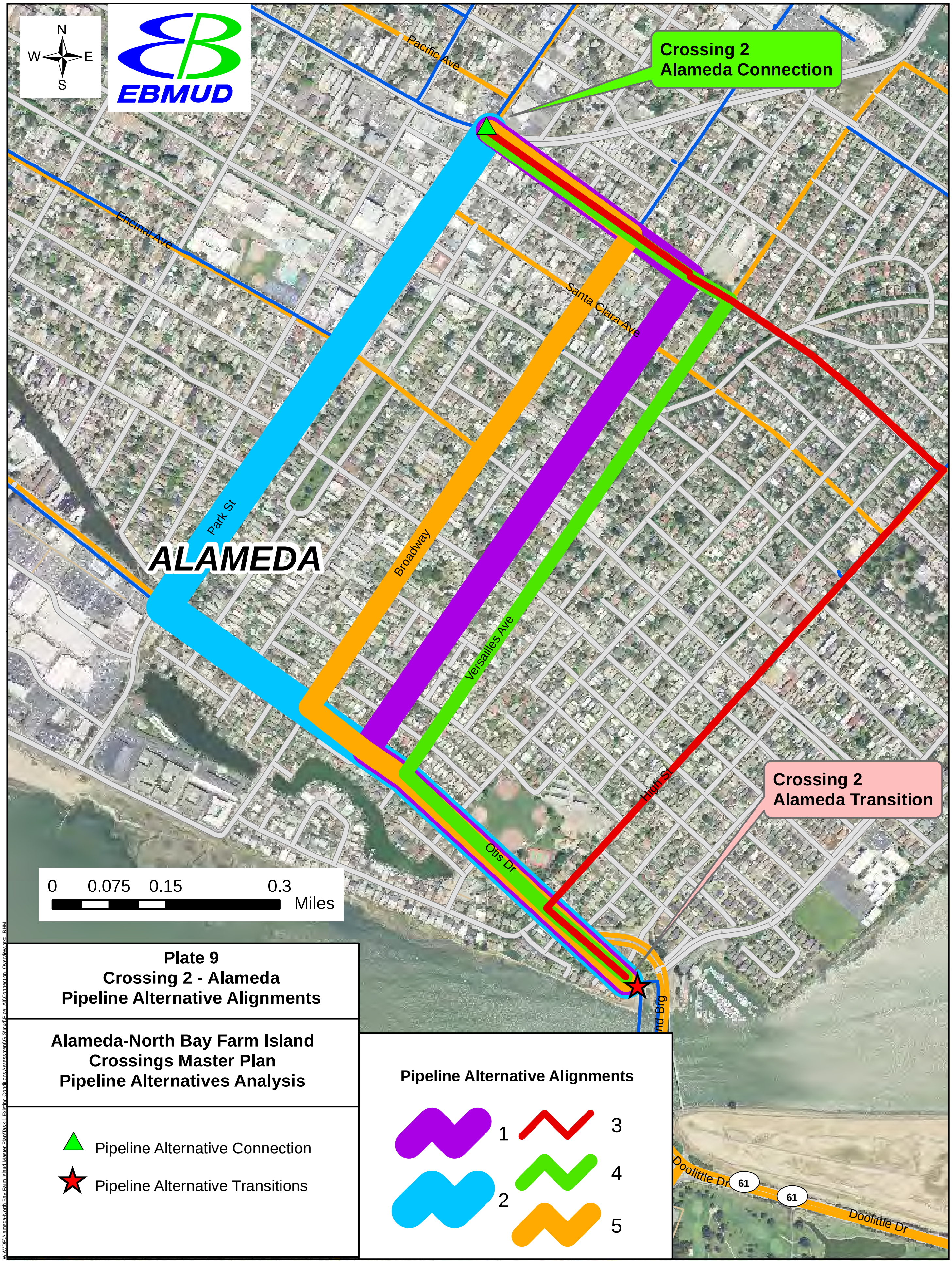
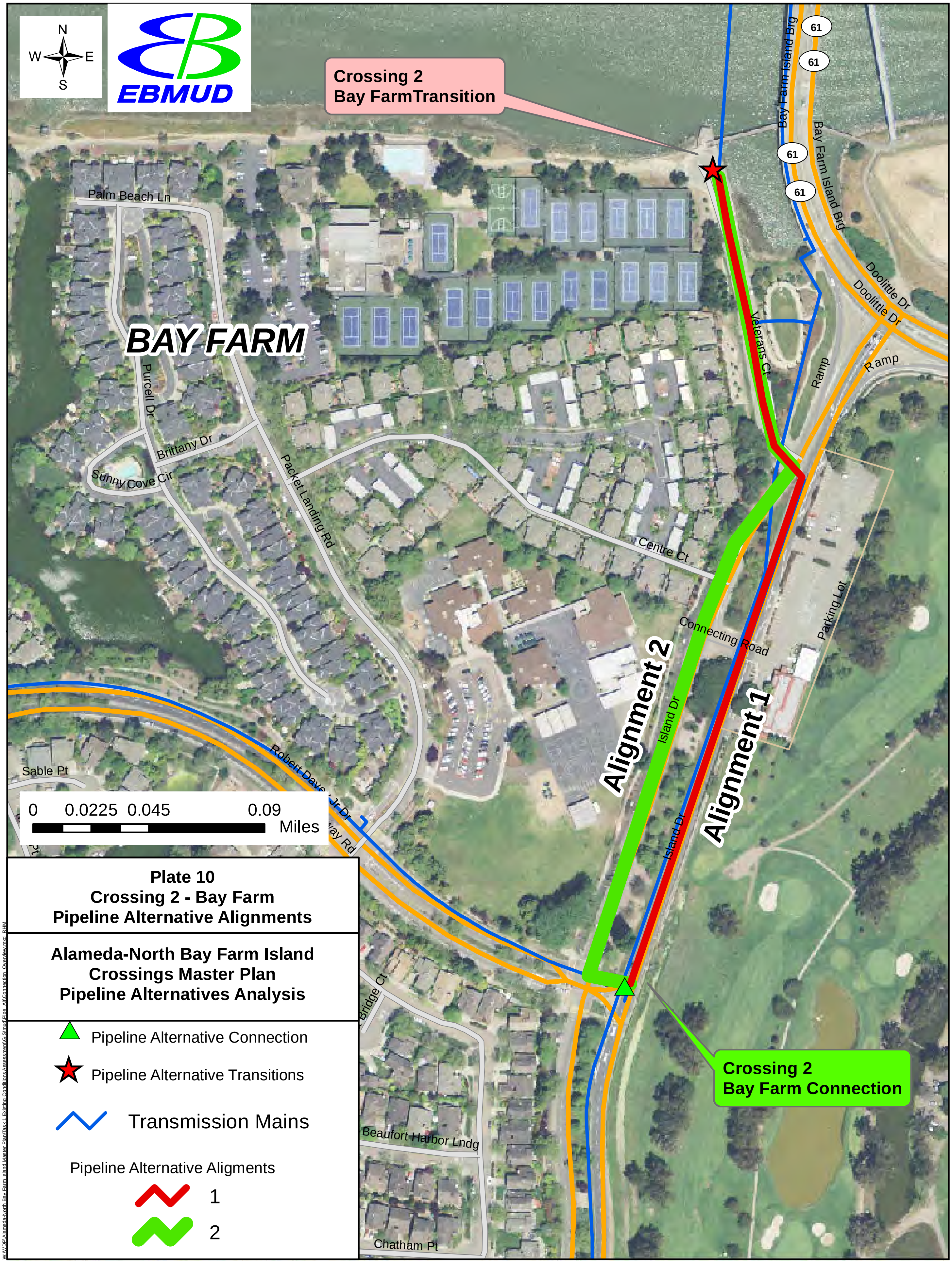


Plate 9
Crossing 2 - Alameda
Pipeline Alternative Alignments

Alameda-North Bay Farm Island
Crossings Master Plan
Pipeline Alternatives Analysis


N
W
E
S
EBMUD
Crossing 2
Bay FarmTransition
BAY FARM
Palm Beach Ln
Purcell Dr
Brittany Dr
Sunny Cove Cir
Packet Landing Rd
Centre Ct
Veterans Ct
Connecting Road
Parking Lot
Island Dr
Alignment 2
Alignment 1
Bay Farm Island Brg
61
Doolittle Dr
Ramp
Robert Davey Jr Dr
Sable Pt
Bridge Ct
Beaufort Harbor Lndg
Chatham Pt
0 0.0225 0.045 0.09
Miles
Crossing 2
Bay Farm Connection
Plate 10
Crossing 2 - Bay Farm
Pipeline Alternative Alignments
Alameda-North Bay Farm Island
Crossings Master Plan
Pipeline Alternatives Analysis
Pipeline Alternative Connection
Pipeline Alternative Transitions
Transmission Mains
Pipeline Alternative Aligments
1
2

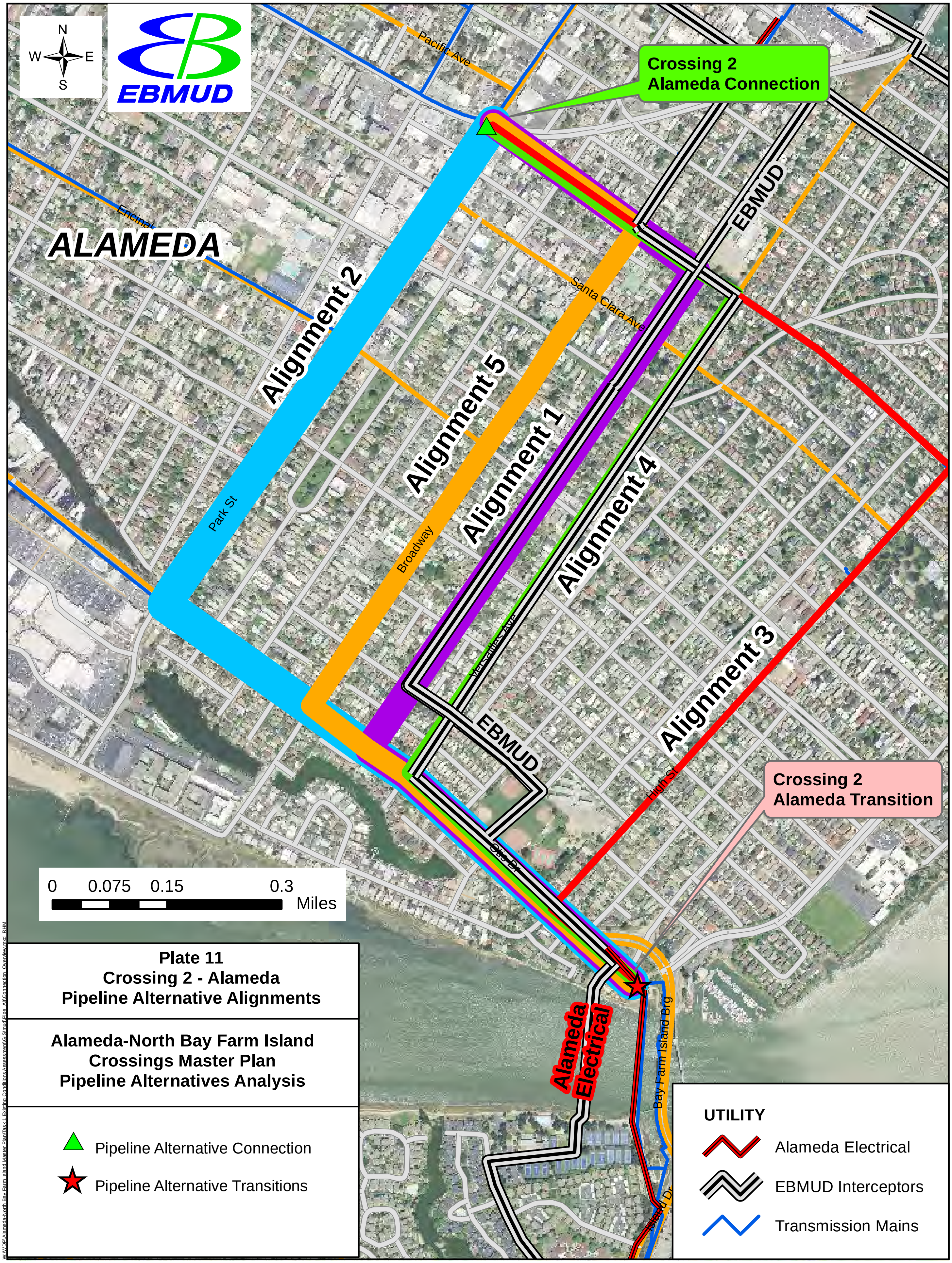


Plate 11
Crossing 2 - Alameda
Pipeline Alternative Alignments

Alameda-North Bay Farm Island
Crossings Master Plan
Pipeline Alternatives Analysis

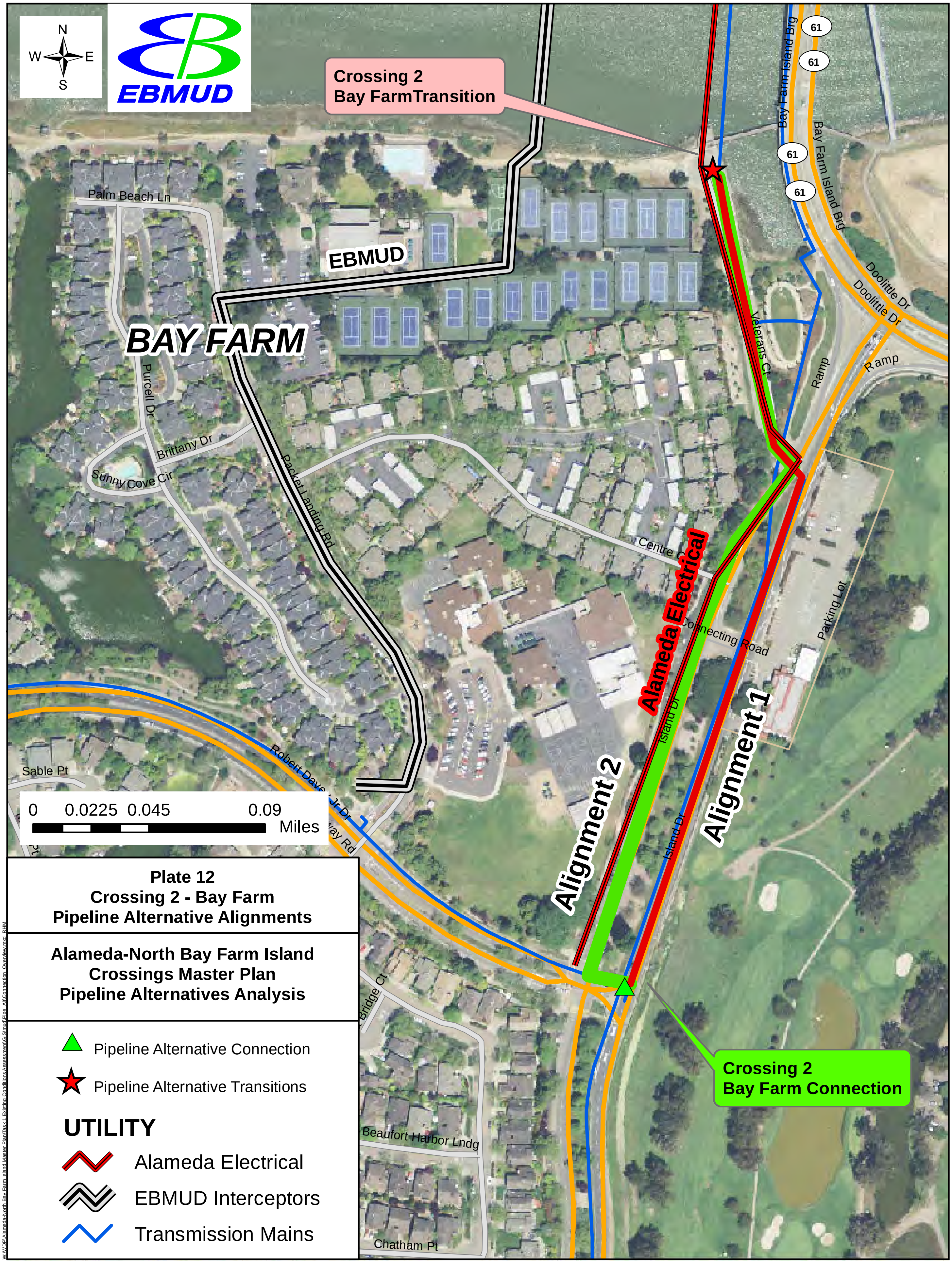


Plate 12
Crossing 2 - Bay Farm
Pipeline Alternative Alignments

Alameda-North Bay Farm Island
Crossings Master Plan
Pipeline Alternatives Analysis

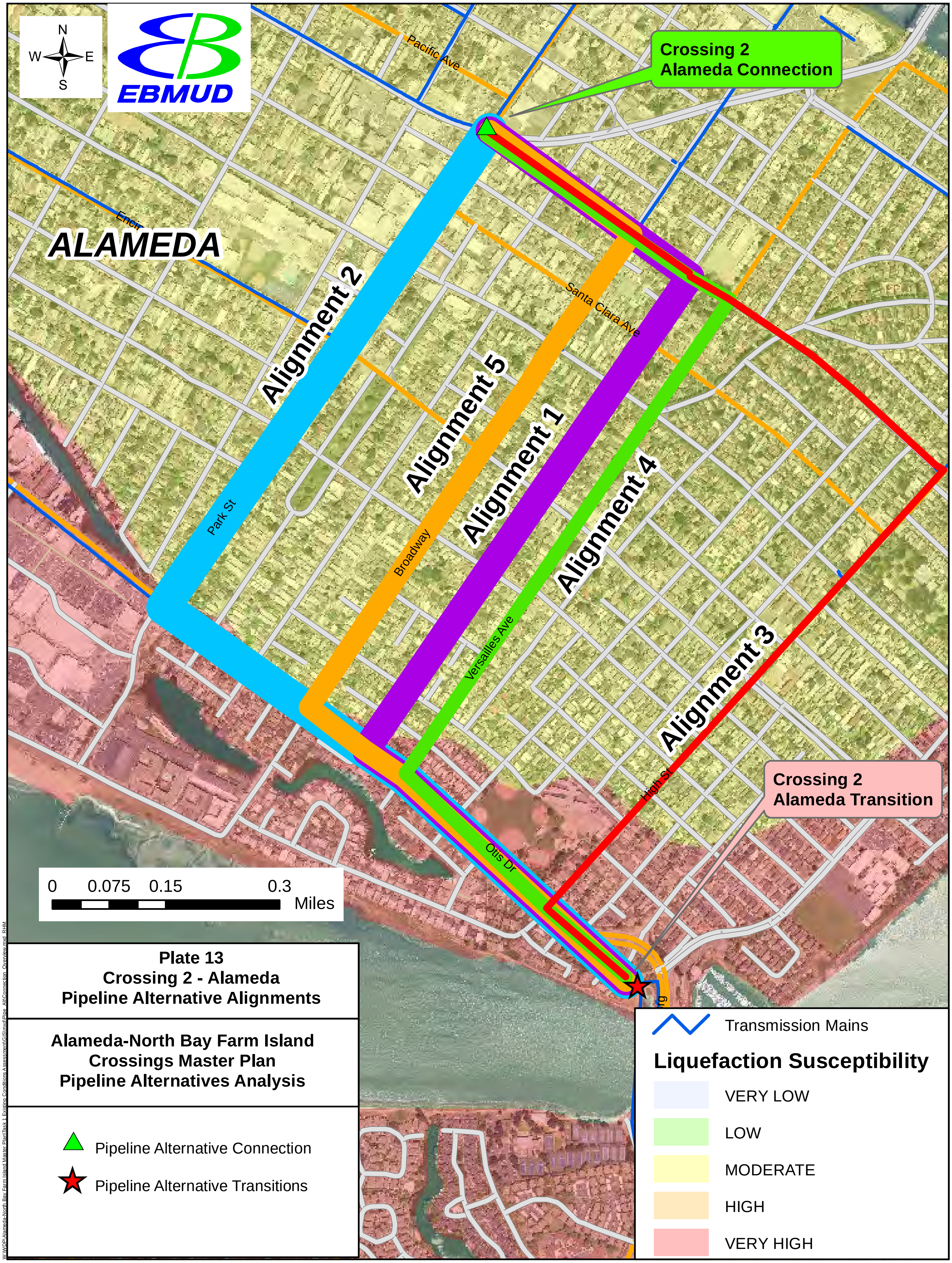


Plate 13
Crossing 2 - Alameda
Pipeline Alternative Alignments

Alameda-North Bay Farm Island
Crossings Master Plan
Pipeline Alternatives Analysis

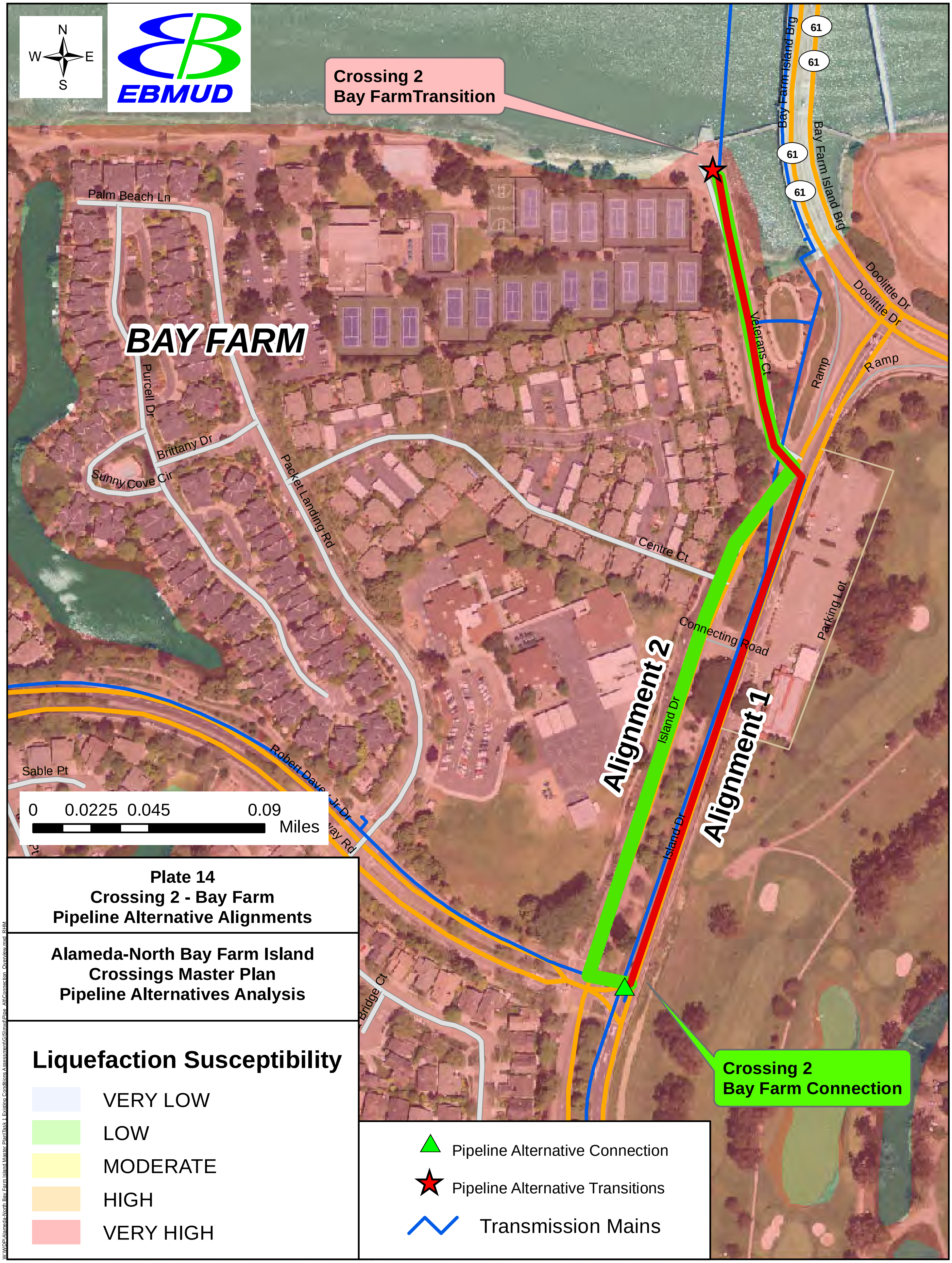
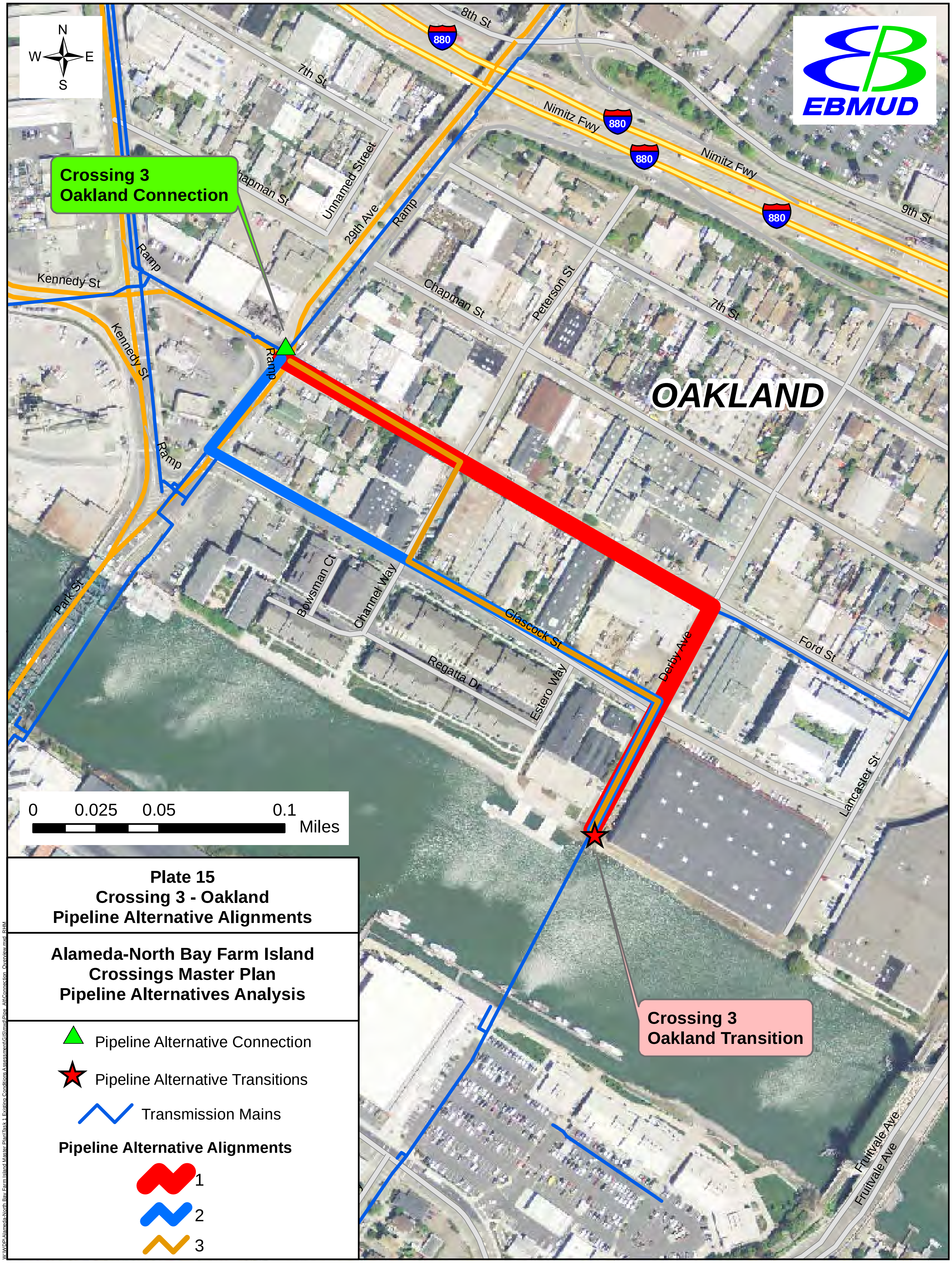


Plate 14
Crossing 2 - Bay Farm
Pipeline Alternative Alignments

Alameda-North Bay Farm Island
Crossings Master Plan
Pipeline Alternatives Analysis



N
W
E
S
EBMUD
Crossing 3
Oakland Connection
Crossing 3
Oakland Transition
OAKLAND
8th St
7th St
Nimitz Fwy
Nimitz Fwy
9th St
880
880
880
880
Chapman St
Unnamed Street
29th Ave
Ramp
Ramp
Ramp
Ramp
Kennedy St
Kennedy St
Chapman St
Peterson St
7th St
Park St
Bowsman Ct
Channel Way
Glascock St
Regatta Dr
Estero Way
Derby Ave
Ford St
Lancaster St
Fruitvale Ave
Fruitvale Ave
0
0.025
0.05
0.1
Miles
Plate 15
Crossing 3 - Oakland
Pipeline Alternative Alignments
Alameda-North Bay Farm Island
Crossings Master Plan
Pipeline Alternatives Analysis
Pipeline Alternative Connection
Pipeline Alternative Transitions
Transmission Mains
Pipeline Alternative Alignments
1
2
3

Plate 16
Crossing 3 - Alameda
Pipeline Alternative Alignments

Alameda-North Bay Farm Island
Crossings Master Plan
Pipeline Alternatives Analysis

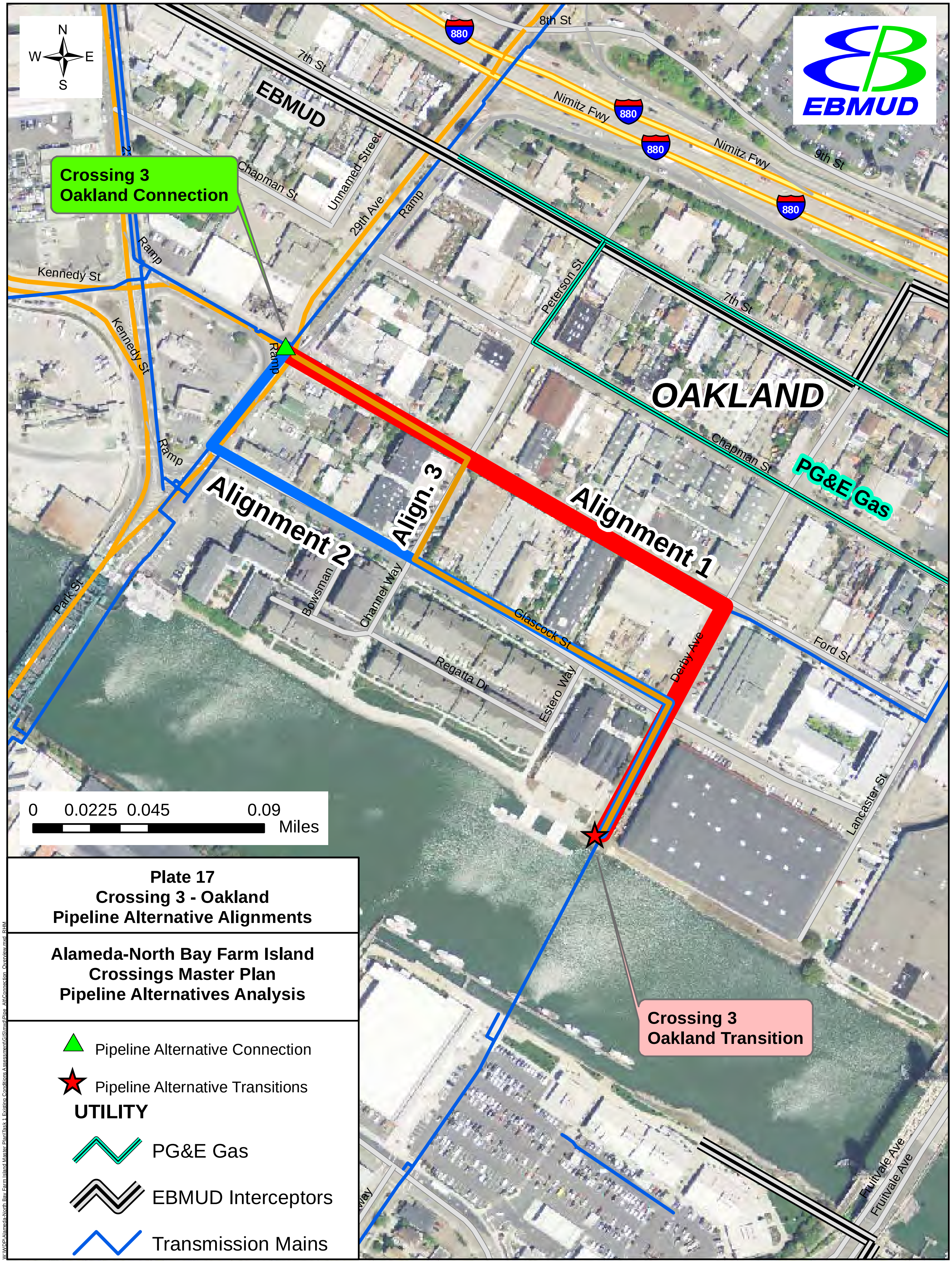


Plate 17
Crossing 3 - Oakland
Pipeline Alternative Alignments

Alameda-North Bay Farm Island
Crossings Master Plan
Pipeline Alternatives Analysis

Pipeline Alternative Connection

Pipeline Alternative Transitions

UTILITY

PG&E Gas

EBMUD Interceptors

Transmission Mains


N
W
E
S
EBMUD
Crossing 3
AlamedaTransition
Crossing 3
Alameda Connection
Alignment 1
Alignment 2
Alignment 3
Alignment 4
Alameda
Electrical
EBMUD
EBMUD
ALAMEDA
Blanding Ave
Clement Ave
Oak St
Buena Vista Ave
Park St
Pacific Ave
Eagle Ave
Everett St
Foley St
Tilden Way
Tilden Way
Ramp
Ramp
Ramp
Ramp
Fernside Blvd
Eagle Ave
Times Way
Webb Ave
Gould Ct
Noble Ave
Tregloan Ct
Edison Ct
Pearl St
St Margaret Ct
Broadway
Janis Cir
Versailles Ave
Lincoln Ave
Moreland Dr
Northwood Dr
Gibbons Dr
Johnson Ave
Glascock St
Regatta Dr
Estero Way
0
0.035
0.07
0.14
Miles
Plate 18
Crossing 3 - Alameda
Pipeline Alternative Alignments
Alameda-North Bay Farm Island
Crossings Master Plan
Pipeline Alternatives Analysis
Pipeline Alternative Connection
Pipeline Alternative Transitions
UTILITY
Alameda Electrical
EBMUD Interceptors
Transmission Mains

Plate 19
Crossing 3 - Oakland
Pipeline Alternative Alignments

Alameda-North Bay Farm Island
Crossings Master Plan
Pipeline Alternatives Analysis

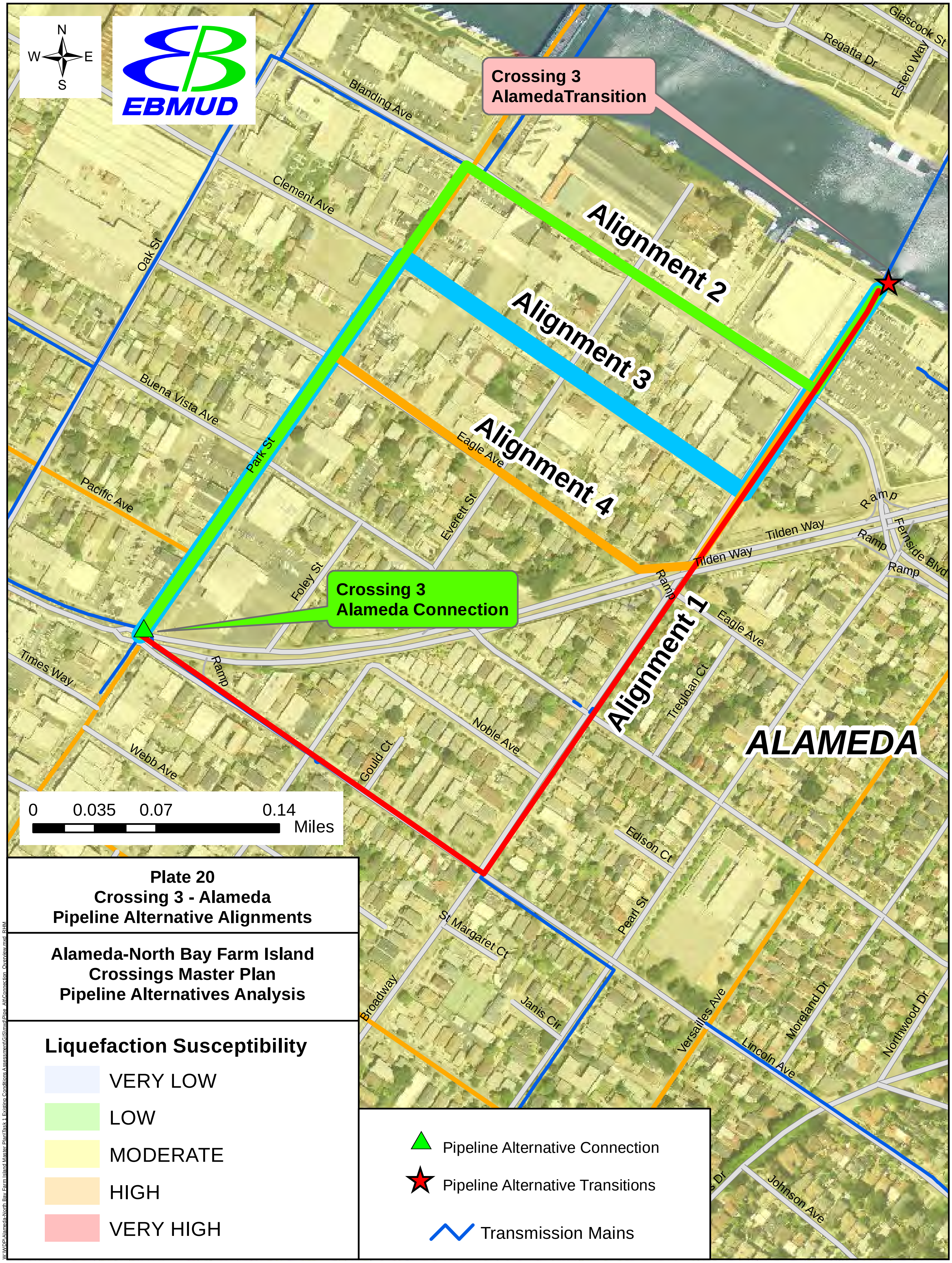


Plate 20
Crossing 3 - Alameda
Pipeline Alternative Alignments

Alameda-North Bay Farm Island
Crossings Master Plan
Pipeline Alternatives Analysis

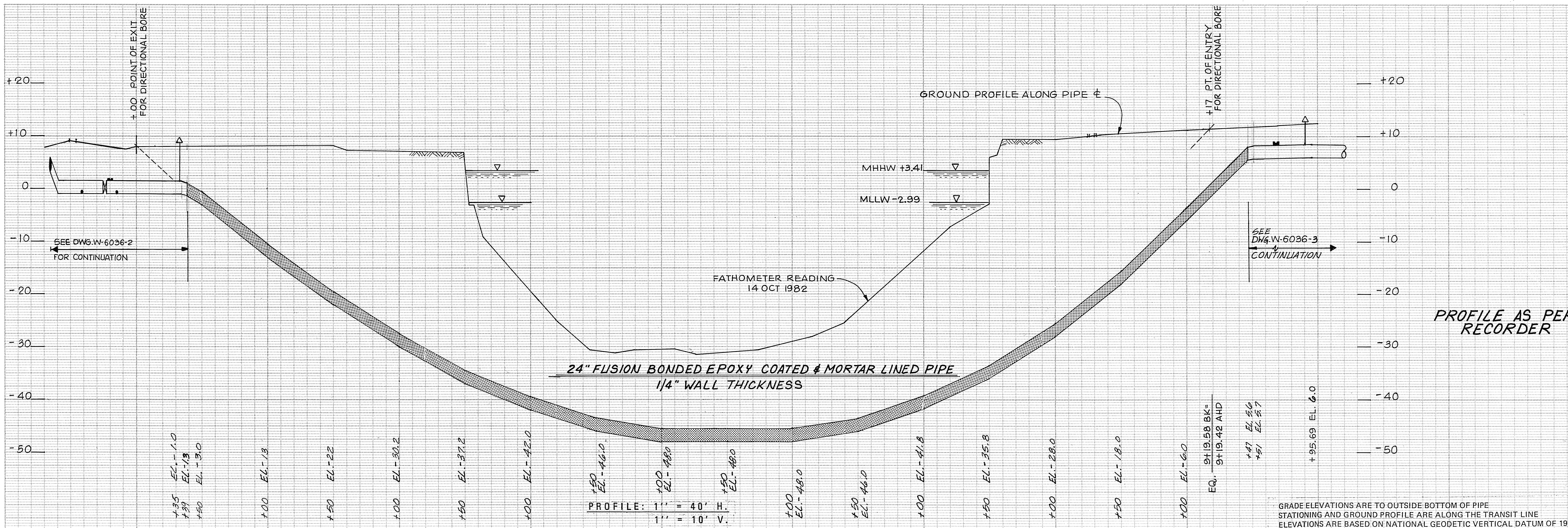
Appendix D

Existing Crossings As-Builts

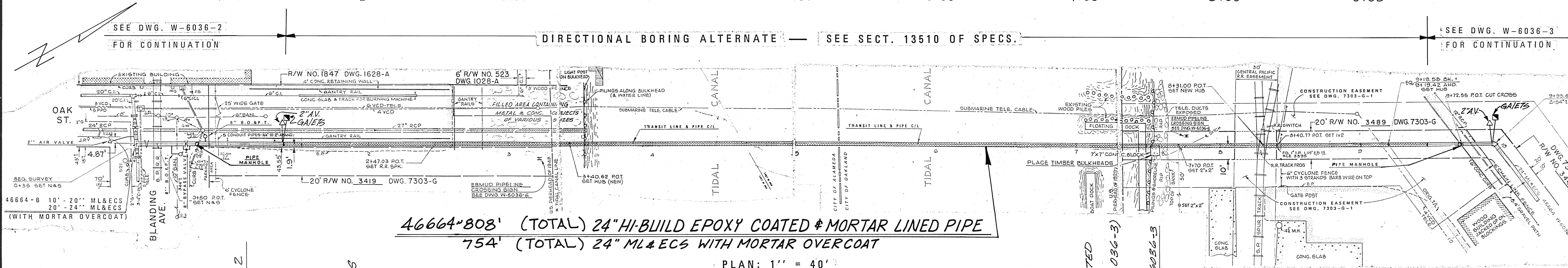
EAST BAY MUNICIPAL UTILITY DISTRICT
OAKLAND, CALIFORNIA
J. S. LONGWELL, CHIEF ENGINEER & GENERAL MANAGER

ESTUARY CROSSING
FOR 24" C.I. PIPE LINE AT
ALICE ST., OAKLAND

DRAWN BY I.W.R	SCALE: 1"= 20'
TRACED BY I.W.R.	DATE: APRIL 25, 1945
CHECKED BY	
APPROVED BY	No. W419



GRADE ELEVATIONS ARE TO OUTSIDE BOTTOM OF PIPE
STATIONING AND GROUND PROFILE ARE ALONG THE TRANSIT LINE
ELEVATIONS ARE BASED ON NATIONAL GEODETIC VERTICAL DATUM OF 1929



AS BUILT
STARTED 4 DEC 86
COMPLETED 23 MAR 87
IN SERVICE 7 MAR 87
RECORDED BY R. CHOY
REVIEWED BY S.R. NYLANDER
REVISED BY S.H. CHAVARIN
CHECKED BY H. LOUIE
DATE 17 AUG 95

46664

46664

CONTINUATION FROM W-6036-2

1+39 BUTTSTRAP END 24" ML & ECS (SEE W-6036-2)
BEG. HI-BUILD EPOXY COATED & MORTAR LINE PIPE

9+47 END HI-BUILD EPOXY COATED & MORTAR LINE PIPE
BEG. 24" ML & ECS (SEE W-6036-3)

FOR CONTINUATION SEE W-6036-3

APPROVED [signature]
CHIEF ENGINEER, R.P.E. NO. C26724

NO.	DATE	REVISION	BY	REC.	APP.

DESIGNED BY	J Choy
DESIGN CHECKED BY	[signature]
DRAWN BY	A. DIAZ
CORROSION CHECK BY	[signature]
SR. CIVIL ENGR. R.P.E. NO. C 27714	[signature]
MGR. OF DESIGN R.P.E. NO. C 10873	[signature]
ASST. CH. ENG. FOR DES. & CONST., R.P.E. NO. C 20035	[signature]

EAST BAY MUNICIPAL UTILITY DISTRICT
OAKLAND, CALIFORNIA

ESTUARY CROSSING, ALAMEDA – OAKLAND
WELDED STEEL
PIPE INSTALLATION
DIRECTIONAL BORING ALTERNATE
PLAN & PROFILE
STA. 1+00 TO STA. 9+17±

STRUCTURE OR ZONE DESIGNATION GOA
SCALE AS NOTED
DATE 4-19-86

W-6497

DISTRIBUTION SYSTEM MAP NOS. 1497 B 466, 468

CONSTRUCTION BY CONTRACT
CONTRACT NO.
JUSTIFICATION IMPROVING SYSTEM

PAVING REFERENCE PLE 641 F.B. 3691 7832-G
DESIGN NO. 49757

E-162 • 4/84

5


PARK ST.
BLANDING AVE.
HARRISON AVE.
23RD AVE.
29TH AVE.
Pierhead Line
Break in Sidewalk Floor
Break in Roadway Floor
Telephone Cable
Curb
Floor Beams
Upper lateral system
Lower lateral system not complete
East Bay St. Ry. Ltd. Cables
Timber Fender
Operators House
Fire Hydrant
Recover gate
Plug tee
Plug cross
Reducer
20"x12" Cross
16" Ward Joint C.I.C. Cast Iron Pipe
OFFICE
AUTOS
PLANING MILL
SWIFT LUMBER CO.
Storm drain
Proposed 20" C.I.
Figures refer to elevations U.S.E.D. at Oakland
OAKLAND ESTUARY
Ground Surface
Map. 1497-B-466
E-18616
see (W-49) E-18613, 14, 15
E-18613
18614
18615
Revised Nov. 28, 1933 by I.W.R.
EAST BAY MUNICIPAL UTILITY DISTRICT
OAKLAND~CALIFORNIA
F.W. Hanna~Chief Engineer and General Manager
DISTRIBUTION SYSTEM
PLAN & PROFILE
OF ESTUARY CROSSING
AT PARK STREET
OAKLAND SIDE
Drawn by Maclean
Traced by L.A. Thomas
Checked by
Approved by
Scale 1"=20'-0"
Date Aug. 13, 1931
No (W-50)
53R

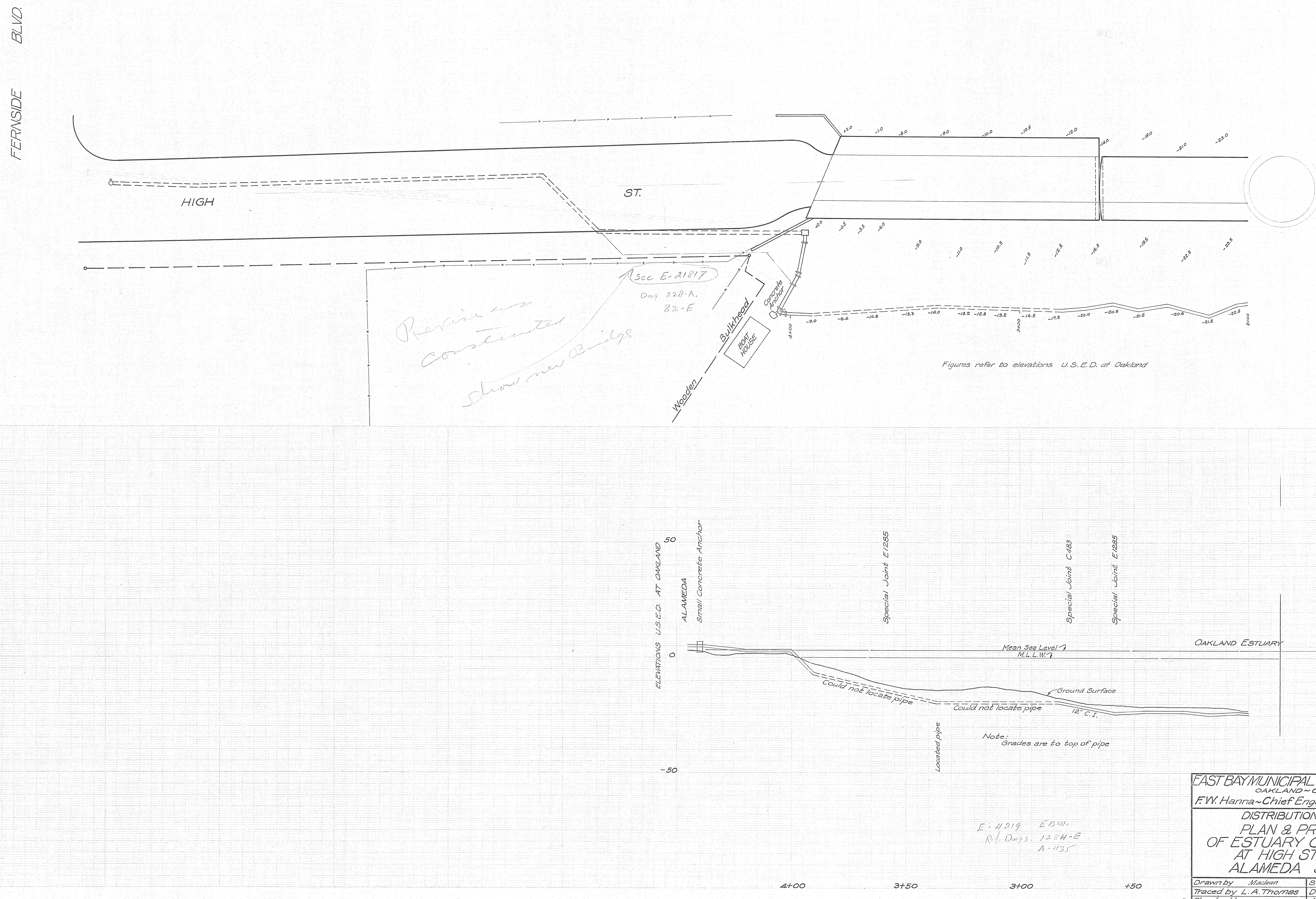
Revised Oct. 31, 1935 by L.A.T.

EAST BAY MUNICIPAL UTILITY DISTRICT
OAKLAND CALIFORNIA
J.S. Longwell Chief Engineer and General Manager

DISTRIBUTION SYSTEM
INNER HARBOR CROSSING
DETAIL OF
20" SUBMARINE PIPE LINE
DERBY AVE. - OAKLAND TO
BROADWAY - ALAMEDA

Drawn by E.C. Mills	Scale 1"=8'
Traced by L.A. Thomas	Date Oct. 29, 1934
Checked by T.L.E.H.	№ 379-G
Approved by R.C.K.	

APPROVED [signature]
Chief Engineer and General Manager



EAST BAY MUNICIPAL UTILITY DISTRICT
OAKLAND ~ CALIFORNIA
F.W. Hanna ~ Chief Engineer and General Mgr.

DISTRIBUTION SYSTEM
PLAN & PROFILE
OF ESTUARY CROSSING
AT HIGH STREET
ALAMEDA SIDE

Drawn by Maclean	Scale 1"=20'-0"
Traced by L.A.Thomas	Date Aug. 18, 1931
Checked by	№ W-47
Approved by	

Revised Feb. 27, 1934 by L.A.T.

12" C.I. Installed E-4219

Ref. E-1284 EBW
1285 EBW
1286 EBW.

EAST BAY MUNICIPAL UTILITY DISTRICT
OAKLAND~CALIFORNIA
F.W. Hanna~Chief Engineer and General Manager

DISTRIBUTION SYSTEM
PLAN & PROFILE
OF ESTUARY CROSSING
AT HIGH STREET
OAKLAND SIDE

Drawn by Maclean	Scale 1"=20'-0"
Traced by L.A. Thomas	Date Aug 17, 1931
Checked by	№ W-48
Approved by	

Notes;
1. See Dwg. 2050-G for General Notes.

2021-G 16" Steel Pipeline - Layout on Bridge.
2050-G Submarine Crossing - Plan & Profile.
2051-G Gen'l Arrangement of Pipeline at Pier 7.
2052-G Steel Specials for 16" O.D. Pipe
2053-G Precast Conc. Pipe Support & C.I. Expansion Joint.
2054-G Pipe Hanger Support & Misc. Det's.
2055-G Bill of Materials.
932-B Field Weld'g Det's for 16" Dia. Lined & Coated Steel Pipe.
934-B 16" Insulat'g Jt. for Pipeline on Bridge

NO.	DATE	REVISION	BY	APP.
4	2-19-70	INSTALLED 2 MAGNESIUM ANODES	AEW	
3	8-23-51	Raised Pipe line	N	C.Y.
2	7-13-51	Revised aux. pile bent	JNP	REL
1	11-14-50	Pipe line moved	JNP	REL

APPROVED ______ CHIEF ENGINEER

E-29453

EAST BAY MUNICIPAL UTILITY DISTRICT
OAKLAND, CALIFORNIA

ESTUARY CROSSING
TO BAY FARM ISLAND

GENERAL PLAN & PROFILE

DESIGNED BY JNP
DRAWN BY JNP
CHECKED BY REL
RECOMMENDED BY R.E. Layton
APPROVED BY JWJ
TRACED BY
SCALE As shown
DATE Aug 15, 1950
NO. 2020-G


PROFILE AS PER RECORDER.
PT&T MANHOLE SEE DWG. W-5463-16
GROUND LINE
+25± 24" BALL JOINT
+36± 24" FLG. COUP.
+36± ℄ PIPE VAULT & ANCHOR
+47± 24" BALL JOINT
EXIST. 16" ML&CS
VAULT
MEAN HIGHER HIGH WATER
CONTRACTOR SHALL PROTECT EXISTING HEADWALLS AND FACILITIES AND RESTORE EXISTING BANK CONTOURS AND SLOPES ALONG NORTH SHORELINE USING GRAVEL BACKFILL MATERIAL AS SPECIFIED IN SPECIFICATIONS. BACKFILL BELOW THE LOW TIDE LINE SHALL BE ALLOWED TO ATTAIN ITS OWN ANGLE OF REPOSE. MAINTAIN 6' MIN. COVER.
- 3.10 M.L.L.W. (USGS DATUM)
PRESIDIO (CAL-TRANS) STATE CHART 11/19/51
(- 6.51 ALAMEDA DATUM)
APPROX. TIDE LOT LINE (SHORELINE)
APPROX. UPPER LIMIT LINE OF IMPORTED BACKFILL (SEE DWG. W-5463-5)
14' BELOW M.L.L.W. (POSS. FUTURE CHANNEL BOTTOM) (EL. -20.51 ALAMEDA DATUM)
P.T.&T. CONDUITS
+28± 6" B.O.&P.T. G.A./E.T.S.
+35± PIPE MANHOLE
+50± 24" TEE
+65± 24" VALVE
+70± 2" AIR VALVE
+00± PIPE MANHOLE
+20± RING GIRDER
+52± RING GIRDER
+25 EL. -5.5
+20 EL. -5.5
+52 EL. -10.0
+75 EL. -13.2
+15 EL. -17.5
+55 EL. -20.8
+95 EL. -23.0
+35 EL. -24.4
+75 EL. -24.5
+50 EL. -24.5
24" CEMENT MORTAR LINED & ENAMEL COATED STEEL PIPE, TYPE S - ALTERNATE "C" (WITH MORTAR OVERCOAT)
25-3/4" O.D. STL. CYL. X 0.281 WALL THICK.
28-1/4" O.D. COATING - (SEE DWG. W-5463-5)
+10 0 -10 -20
0+00 1+00 2+00 3+00 4+00 5+00 6+00
GROUND LINE ALONG PIPE
EXIST. 16" ML&CS
16" TEE
16" VALVE
24"x16" TAPER
24" TEE
16" ML&CS POLYWRAPPED
EL. -2.0
EL. -5.5
+4 +2 0 -2 -4 -6
16" CROSS CONNECTION
GRADE ELEVATIONS ARE TO OUTSIDE BOTTOM OF PIPE
STATIONING AND GROUND PROFILE ARE ALONG THE TRANSIT LINE
ELEVATIONS ARE BASED ON CITY OF ALAMEDA DATUM (OTIS-HIGH B.M. AS EL 2.81)
ELEV. 0.00 ALA. DATUM = ELEV. 3.41 USGS DATUM
STA. 1+20 "C-1" Δ=51°32'27" RT.
PROPERTY NO. DWG. NO. 7313-G
DITCH
TOP
MEAN HIGHER HIGH WATER
STA. 2+44.74 P.O.T. WK. PT.
ORIGINAL "C"
BALL JOINT
PIPE VAULT & ANCHOR
BALL JOINT
STA. 1+46.45 P.O.T.
WOOD HEADWALL- PROTECT DURING EXCAVATION
TOE
DEENERGIZED & ABANDONED 12KV SUBMARINE CABLE
TRANSIT LINE AND PIPE C/L
℄ PT&T CONDUITS
BAY FARM ISLAND BRIDGE
STA. BRIDGE CHISELED "+" IN W. SIDEWALK
SAN LEANDRO EBB TIDE CHANNEL
EXCAVATION LIMITS MUST NOT ENCROACH ON BRIDGE FENDER PILING-PROVIDE PROTECTION WITH SHEET PILING IF NECESSARY.
MATCH LINE-STA. 6+50 SEE DWG. W-5463-4
16" CROSS CONNECTION POLYWRAPPED
16" ML&CS
HI-VOLT BOX
PT&T M.H. 7558 SEE DWG W-5463-16
OTIS MARK
E.P.
BEG. SURVEY 0+00
TO BE REMOVED BY OTHERS
WK. PT.
GUARD RAIL
CONC. HEADWALL
4" STORM DRAIN
60'
D.O.T. R/W 9-1137
45097
1306'-24" WELDED STEEL
32'-16" ML&PCS-POLYWRAPPED CROSS CONNECTION
CABLE CROSSING SIGN SEE DWG. W-5463-15
MEAN LOW LOWER WATER
10'
14' N.T.S.
8'±
PERALTA GRANT LINE TIDE LOT LINE
ISLE
BRIDGE VIEW (OTIS DR.)
HI-VOLT BOX
ELEC. BOX
ELEC. BOX
TEL. BOX
ELEC. BOX
PIPE MANHOLE
2" AIR VALVE
G.A./E.T.S.
PIPE MANHOLE
6" B.O.&P.T.
TEMP. 6" BYPASS AND WET TAP BY DISTRICT FORCES. SEE SPEC. SEC. 1A
30'
TEL. BOX
HI-VOLT BOXES
POWER STA.
P.P.
T.P.
292.78' W-687
80'
45097
0+25.4 24" BUMPED HEAD (3/16" THICK) BEGIN 24" WELDED STEEL
0+28.4 6" BLOWOFF & PUMPING TEE
0+35.4 PIPE MANHOLE-SEE SPEC. SEC. 1A
0+50.4 24" TEE (SEE DWG. W-5463-5) CONNECT TO 16" CROSS. CONN.
0+53.6 GALVANIC ANODE WITH ELECTROLYSIS TEST STATION DWG. 286-EA, FIG. A-Y LEDGER NO. 82-04-19-02
0+65.4 24" VALVE, BUTTERFLY, FLGD. & SKIRT. M: PRATT O: RT. T: 43
0+70.4 TAP FOR 2" AIR VALVE DWG. 189-EA, TYPE "B"
1+00.4 PIPE MANHOLE
1+17.2 BEGIN COMB. BEND @ 51.887° (VERT. Δ = 8.315° HORIZ. Δ = 51.408°)
BK 1+20.4 / AHD 1+20.0 P.I. STA. 1+20±
1+23.2 END COMB. BEND
1+25.2 BALL JOINT
1+36.0 ℄ PIPE VAULT & ANCHOR-SEE DWGS. W-5463-5 & W-5463-6
1+47.2 BALL JOINT
2+50 TIDE LOT LINE
16" CROSS CONNECTION
32'~16" ML & PCS
0+00 CONNECT TO EXIST. 16" ML&CS WITH 16" TEE
16" NIPPLE
16" VALVE, BUTTERFLY, FLGD. & SKIRT. M: MUELLER O: RT. T: 23
SEE SPEC SEC. 1A
0+21.4 16" ELL, (COMB.), 45° Δ RT. & DN.
0+33.9 16" INSULATING JOINT, DWG. 3186-B
0+24.4 16" VALVE, BUTTERFLY, FLGD. & SKIRT. M: MUELLER O: RT. T: 23
0+28.4 16" x 24" TAPER (SEE DWG. W-5463-5)
0+31.7 24" TEE (45097) = 0+50.4
AS BUILT
STARTED 01 OCT 82
COMPLETED 10 JUN 83
IN SERVICE 29 JUN 83
RECORDED BY T.C. Farata
REVIEWED BY SRD
REVISED BY N. McALGER
DATE 22 JUL 92
45097
APPROVED
CHIEF ENGINEER R.P.E. NO. C-8216
DESIGN BY RJR & OTHERS
DESIGN CHECK BY
DRAWN BY DIAZ
MECH ELECT STRUCT
PIPELINE CORR FNDS
DRAFTING
SR. CIVIL ENGR., P.L.E. R.P.E. NO. C 18774
ASST. CH. ENG., DES. & CONST. R.P.E. NO. C 26724
EAST BAY MUNICIPAL UTILITY DISTRICT
OAKLAND, CALIFORNIA
SAN LEANDRO CHANNEL UTILITIES CROSSING
ALAMEDA
WELDED STEEL PIPE INSTALLATION
SCHEDULE II
BRIDGE VIEW ISLE TO STA. 6+50
STRUCTURE OR ZONE DESIGNATION G0A
SCALE 1"=40'H(1:480) 1"=4'V(1:48)
DATE 22 APR 82
W-5463-3
18
NO. DATE REVISION BY APP.
DISTRIBUTION SYSTEM MAP NO. 1497 B 458
SPEC. NO. 1413
E-162 • 6/76

NOTE

1. UPON COMPLETION OF THE BACKFILLING OF THE UTILITY LINES, THE CONTRACTOR SHALL PLACE A 6-INCH LAYER OF 2-INCH± COARSE AGGREGATE BETWEEN THE LOW TIDE LINE AND THE SEAWALL OVER ALL AREAS NEAR SEAL POINT COURT THAT HAVE BEEN DISTURBED BY THE PROJECT CONSTRUCTION, EXCLUSIVE OF THE SEAWALL STONE FACING.

2. FOR LOCATION OF PROPOSED EAST BAY REGIONAL PARK BOARDWALK IN THE VICINITY OF PIPELINE STATION 9+50, SEE DRAWING W-5463-14.

AS BUILT

STARTED 01 OCT 82
COMPLETED 10 JUN 83
IN SERVICE 29 JUN 83
RECORDED BY T.C. Faroke
REVIEWED BY S.P. Dorovic
REVISED BY H. McALEER
DATE 22 JUL 92

APPROVED Orvin W. Harder
CHIEF ENGINEER R.P.E. NO. C-8216

DESIGN BY	RJR & OTHERS
DESIGN CHECK BY	
DRAWN BY	DIAZ
MECH / ELECT / STRUCT	
PIPELINE / CORR	AEW / ENGR
DRAFTING	
SR. CIVIL ENGR., P.L.E., R.P.E. NO. C 18774	
ASST. CH. ENG., DES. & CONST. R.P.E. NO. C 26724	

EAST BAY MUNICIPAL UTILITY DISTRICT
OAKLAND, CALIFORNIA

SAN LEANDRO CHANNEL UTILITIES CROSSING
ALAMEDA

WELDED STEEL
PIPE INSTALLATION

SCHEDULE II
STA. 6+50 TO SEAL POINT CT.

STRUCTURE OR ZONE DESIGNATION: G0A
SCALE: 1"=40'H(1:480) 1"=4'V(1:48)
DATE: 22 APR 82

W-5463-4
18

NO.	DATE	REVISION	BY	APP.

DISTRIBUTION SYSTEM MAP NO. 1497 B 458 SPEC. NO. 1413

E-162 • 6/76

Appendix E
Leak History

AQUEDUCT LEAK DETECTION STUDY
Alameda Crossings

INTRODUCTION

During the period of September 2011 to April of 2012, all four in-service Alameda pipeline crossings were inspected for leaks by Echologics Engineering (Echologics), a company specializing in leak detection on large diameter transmission mains using a new acoustic correlation technology. Echologics was one of the three leak detection companies hired by East Bay Municipal Utility District (EBMUD) for the Aqueduct Leak Detection Study to help assess the condition of several critical, large diameter pipelines throughout the District. All in-service Alameda pipeline crossings were assessed by Echologics as part of the Aqueduct Leak Detection Study, which include: Alice, Park, Blanding (Oak St.), and Alameda-Bay Farm 2.

LEAKFINDER RT CORRELATION SYSTEM

Echologics uses a technology called the Leakfinder RT Correlation System (Leakfinder RT) to identify potential leaks in large diameter pipelines. The Leakfinder RT uses a cross correlation methodology which involves the use of two sensors connected to the water column on both ends of the pipeline segment under investigation. See Figure 1 below for a schematic of the Leakfinder RT set-up.

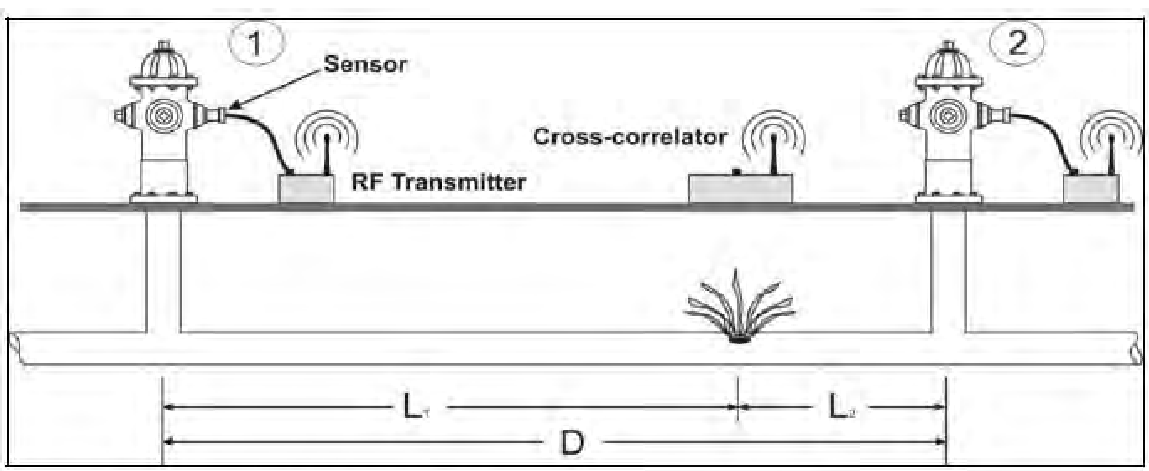


Figure 1 Schematic of the Leakfinder RT Set-Up

Figure 5.1. Schematic of the Leakfinder RT Set-up.

The two sensors act as hydrophones, detecting potential leaks by determining the location of a noise source based on the time difference that a noise is heard by the two separate stations. Each station contains a wireless radio transmitter that broadcasts the sound signals from the station to a central receiver, which then connects to a laptop that calculates the noise source location. The distance formula used for the location calculation (Distance = Velocity * Time). In this case, the sound velocity is used for the velocity term and is calculated using information on the pipe type, diameter, and thickness.

In reality, the Leakfinder RT is a noise correlator, not a leak correlator. Although the noises detected by the technology are often produced by leaks, the noise can be produced by a number of other sources, such as ground vibrations from vehicles, interference in the transmission of sound from electrical systems, noise from pumps and regulating valves, wi-fi networks, radio transmissions, etc. Echologics performs extensive post-processing of recorded sounds, eliminating background noise in order to isolate a particular noise of interest, which also allows them to locate more than one leak on a single run. In some cases, depending on the quality of the signal, Echologics is able to pinpoint a noise source. In other cases, only an approximate range is possible.

Figure 2 is an example of acoustic information that is used to determine the location of a possible leak or other point of interest. In this example, the two stations are located at air valves, approximately 2,600 feet apart. The pipe material is concrete and the pipe diameter is 108 inches so the predicted sound wave velocity is calculated to be 2,993 ft/sec; site photos are shown in Figure 3. The two top graphs in Figure 2 depict the distributions of sound frequency heard at each listening station. The bottom left graph is the Coherence Function which measures the degree of similarity in the frequency of sound signals from each sensor and which frequencies have the most commonality. The bottom right graph in Figure 2 is the Correlation Function which determines the time differential between sounds with the best coherence heard by each station. This time differential is then translated to a distance measurement using the sound velocity. In this example, the leak is initially estimated to be 2,392 feet from the Blue station and 236 feet from the White station.

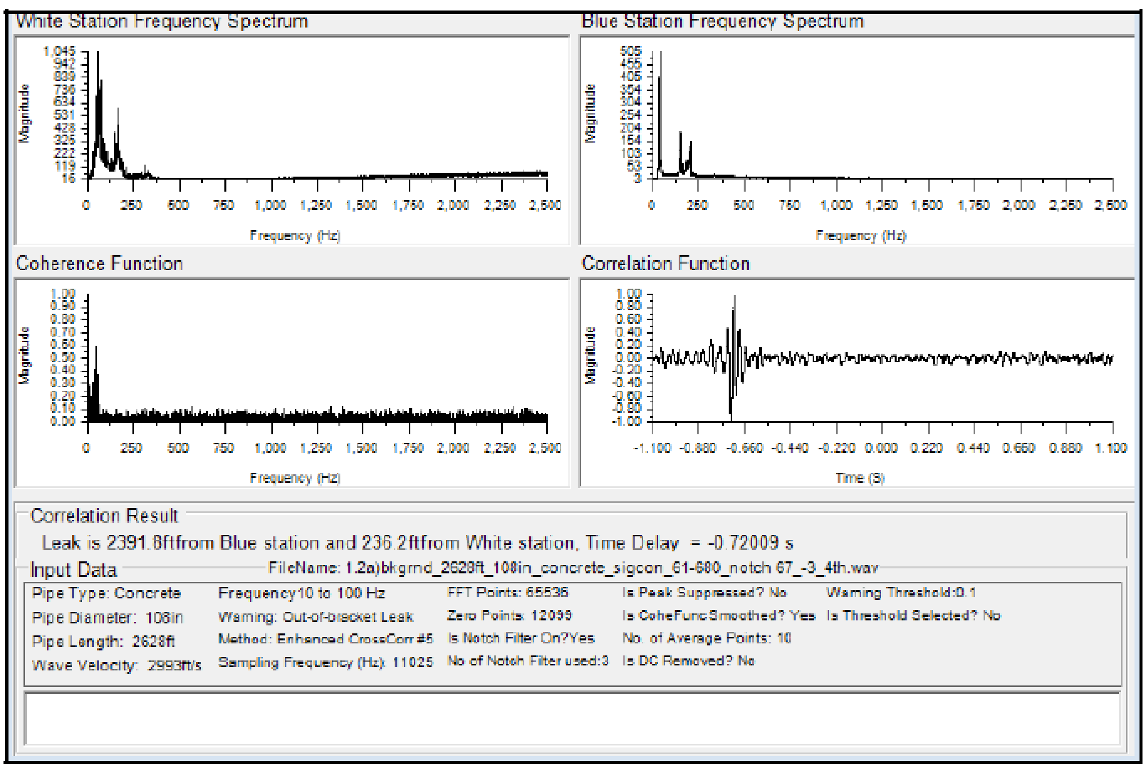


Figure 2 Example of Echologics RT noise signal processing of suspected leak location.

Figure 3 Photos taken during Echologics investigation.

FIELD RESULTS

Alice Street Crossing

The Alice Street Crossing, a 24-inch diameter steel pipe, was tested by Echologics on April 12, 2011. A total of 8,890 feet of pipe was surveyed in four consecutive pipe segments; the limits of the survey are shown in Figure 3. The estuary crossing alone is approximately 900 feet.

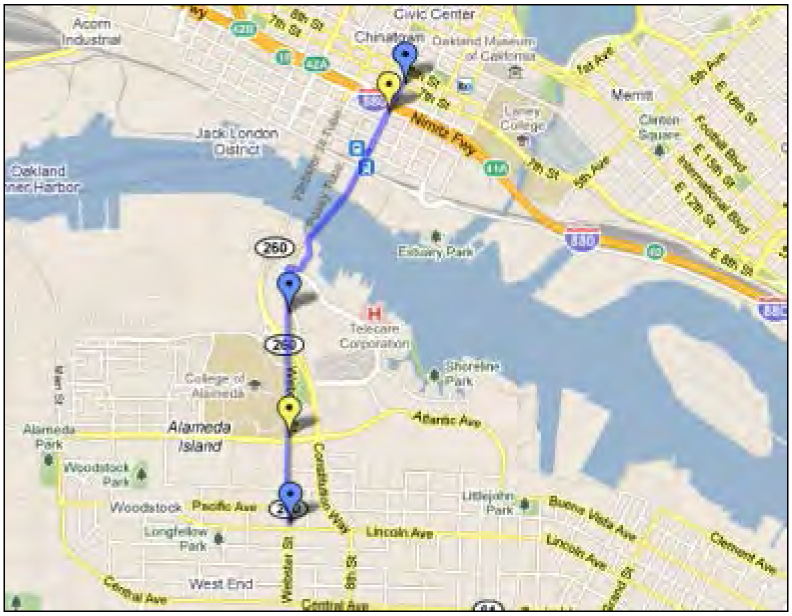


Figure 3. Alice Street estuary crossing investigation by Echologics.

The blue and yellow dots represent the location of the transmitter stations for each segment tested. The first segment started at Hydrant 5269 on the corner of Alice St. and 7th St. in Oakland to a 2-inch air valve at the intersection of Webster and Lincoln St in Alameda. The survey extended into Alameda, across the estuary, and finished at the intersection of Webster St. and Lincoln Ave. No potential leaks were detected during the survey, and Echologics concluded that the probability of leakage for Alice Street Crossing is low.

<u>Park Street Crossing</u>

The Park Street Crossing, a 20-inch diameter steel and cast iron pipe, was tested by Echologics on September 23, 2010. A total of 4,715 feet of pipe was surveyed in four consecutive pipe segments; see Figure 4.

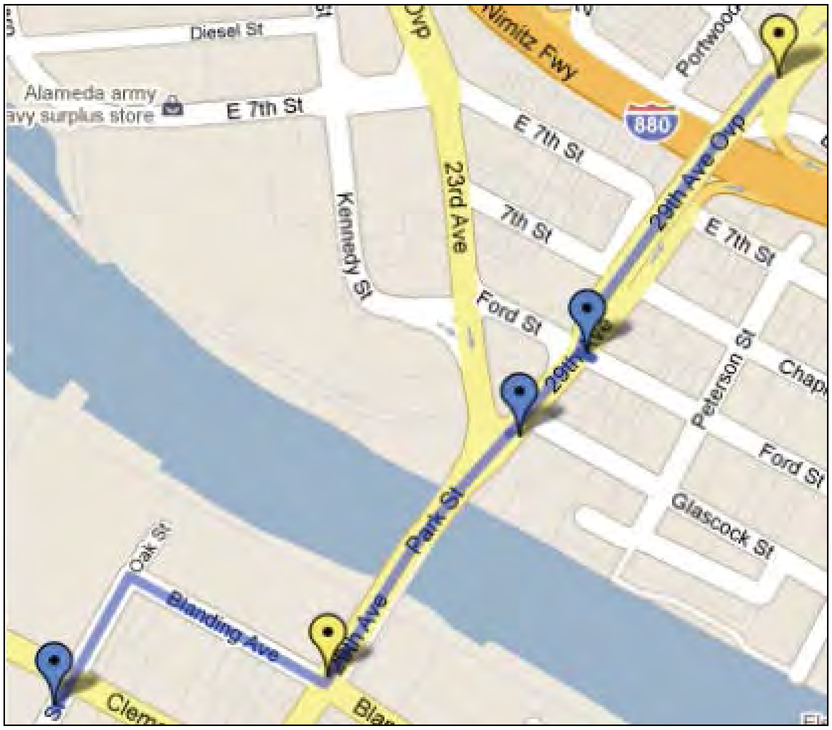


Figure 4. Park Street Crossing investigation by Echologics.

The survey investigation started at the intersection of Ford St. and 29th Ave/Park St. in Oakland, and ended on Oak St, just south of Clement Ave in Alameda. Hydrant and/or air valve information was not provided in the report for this crossing. No potential leaks were detected during the survey, and Echologics concluded that the probability of leakage for Park Street Crossing is low.

<u>Blanding (Oak Street) Crossing</u>

The Blanding Crossing Leak Detection report could not be located.

<u>Alameda-Bay Farm 2 Crossing</u>

The Alameda-Bay Farm 2 Crossing, a 16-inch and 20-inch diameter steel pipe, was tested by Echologics on April 13, 2011. A total of 1,430 feet of pipe was surveyed, with three consecutive pipe segments for the initial testing; see Figure 5.

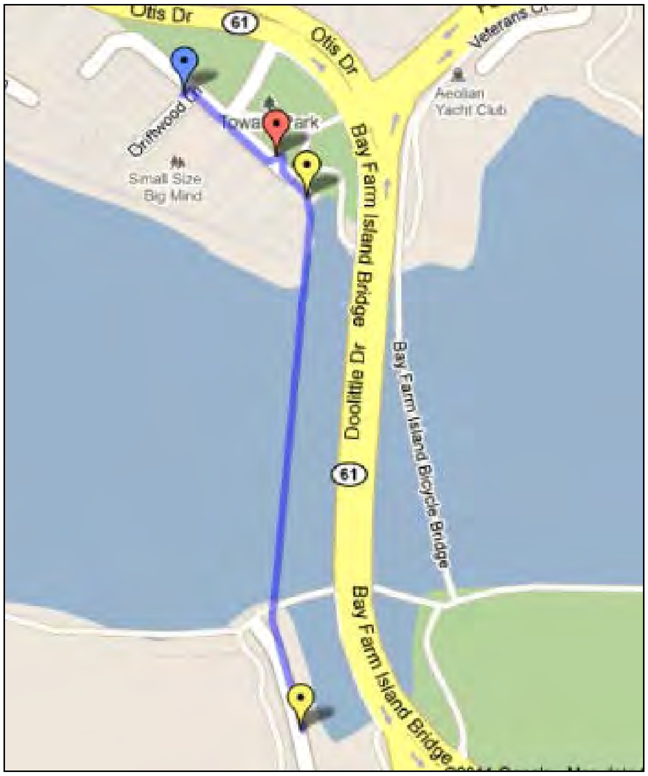


Figure 5. Alameda-Bay Farm 2 Crossing investigation by Echologics

The survey investigation started from Hydrant 16624 at the intersection of Driftwood Ln. and Bridgeview Isle, and ended at the 2-inch air valve in Veterans Ct. One potential leak was found on Bridgeview Isle, and Echologics concluded the probability of leakage is high. The detected leak's location is shown in Figure 6.

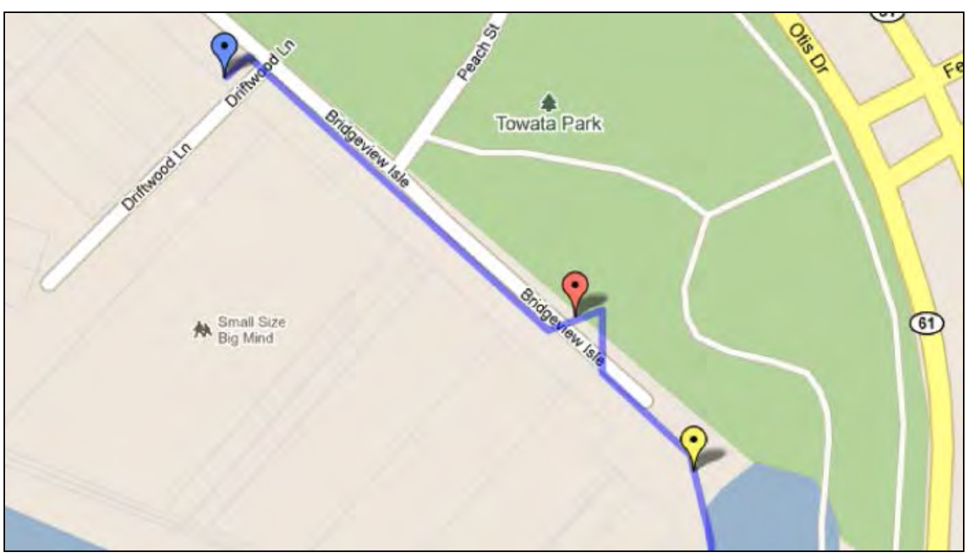


Figure 6. Potential leak located near Alameda-Bay Farm 2 Crossing.

The blue station in Figure 6 is Hydrant 16624 and the yellow station is a manhole. Two access points exposing the pipe barrel were made available for surface mounted sensors to further pinpoint the leak. The detected leak is approximately 196.3 feet from the blue station and 152.7 feet from the yellow station, as determined by the correlation result shown in Figure 7.

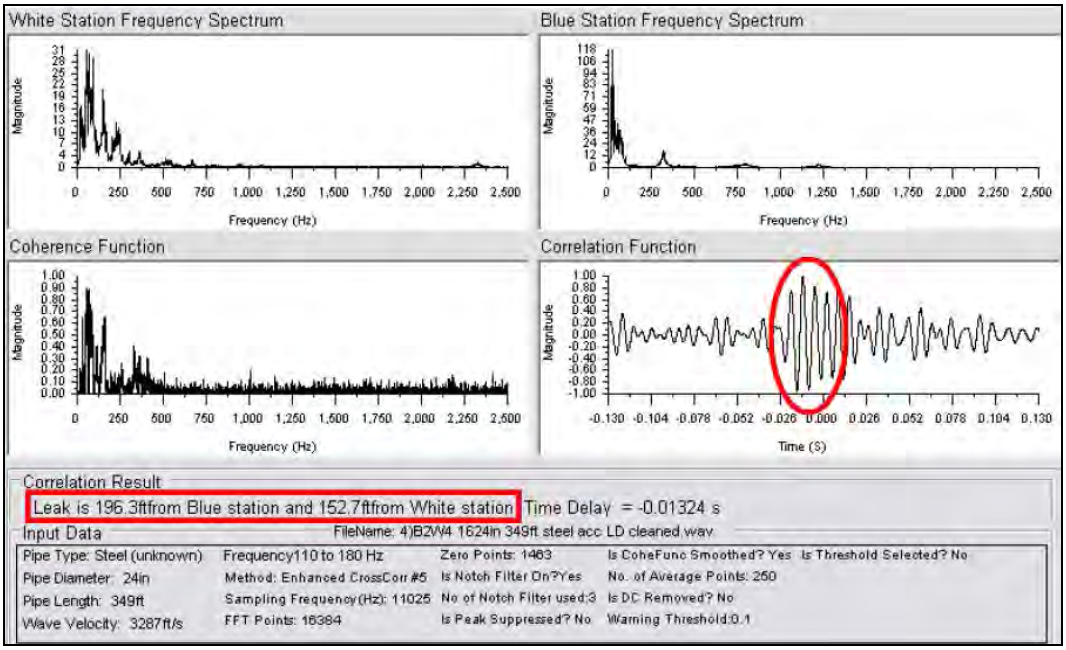


Figure 7. Correlation Report for potential leak on Bridgeview Isle.

In addition to detecting a potential leak, a vault located some distance along the crossing was observed to have significantly corroded tie rods supporting the pipeline. If the tie rods were to fail, the pipeline crossing would fail.

CONCLUSION

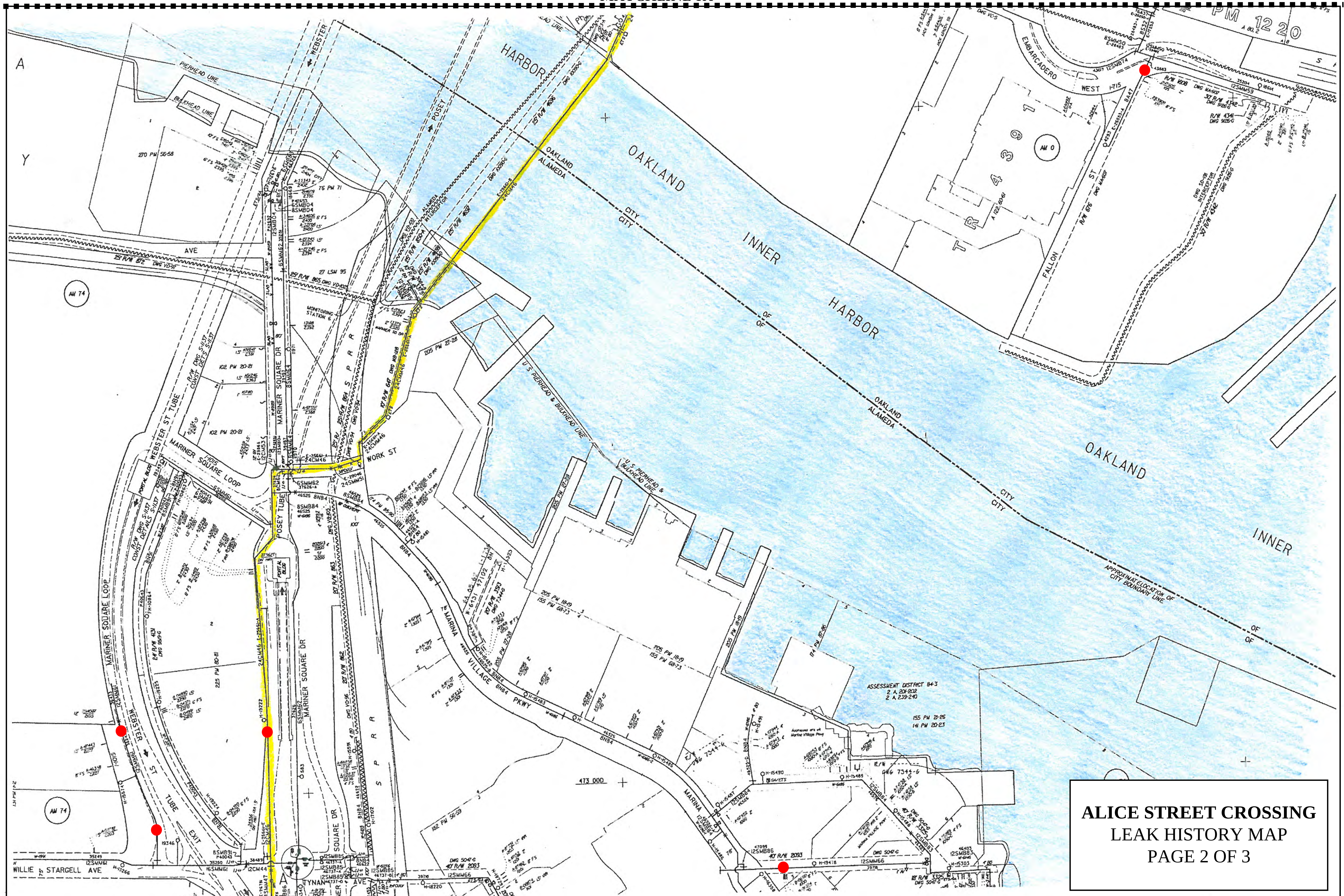
Echologic's survey detected one potential leak approximately 150 feet from the north end of the Alameda-Bay Farm 2 Crossing. No leaks were detected by Echologics during the survey of Alice, Park, Blanding (Oak St.) crossings. Echologics leak detection methods are not an exact science, and may not find all leaks.

The potential leak detected in Alameda-Bay Farm Crossing, as well as the significantly corroded tie rods in the crossing's vault may indicate that this crossing is in poor condition; however, further investigation is recommended including inspections of the pipe interior.

ALICE STREET CROSSING
LEAK HISTORY MAP
PAGE 1 OF 3

MATCHLINE 1A

MATCHLINE 1A

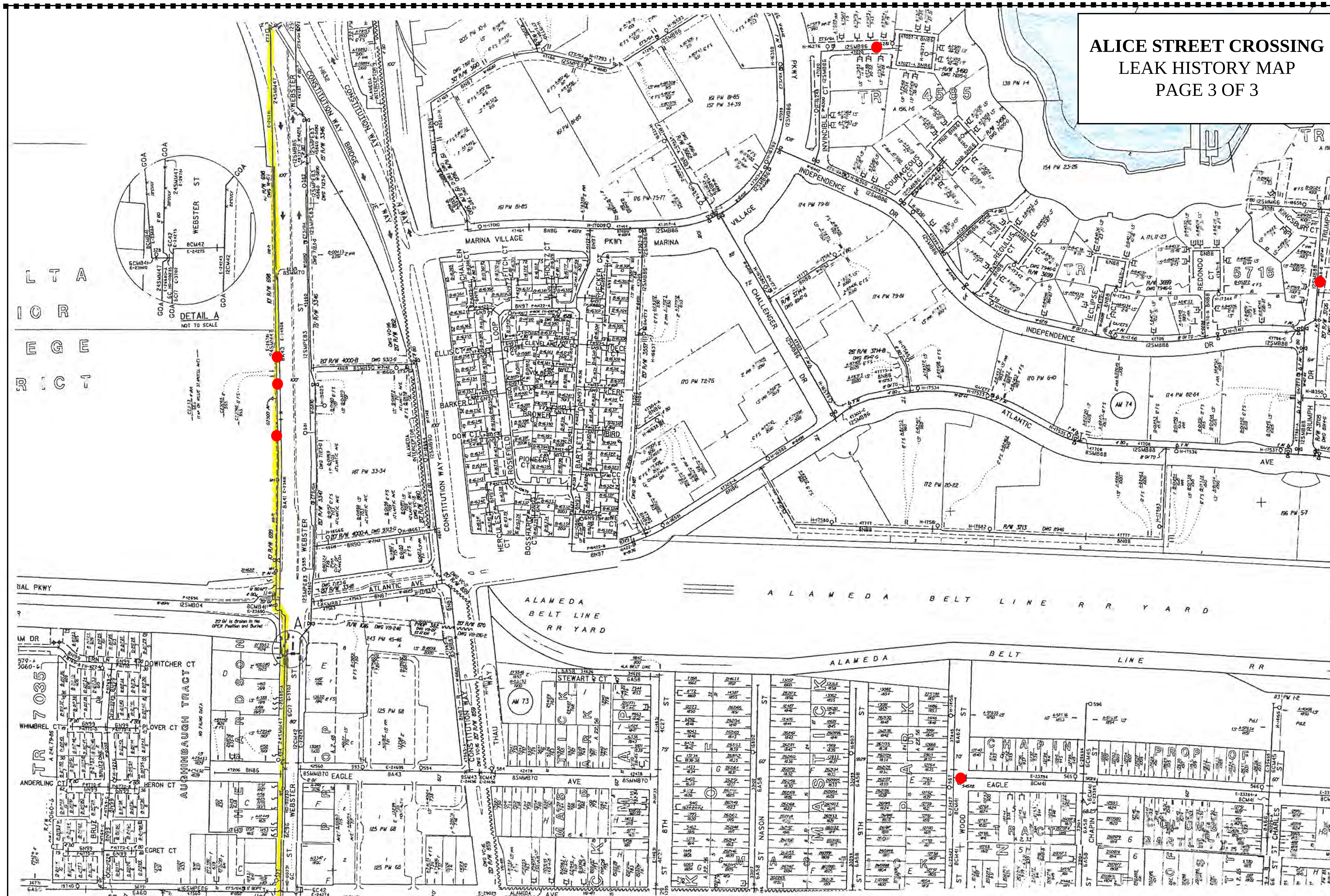


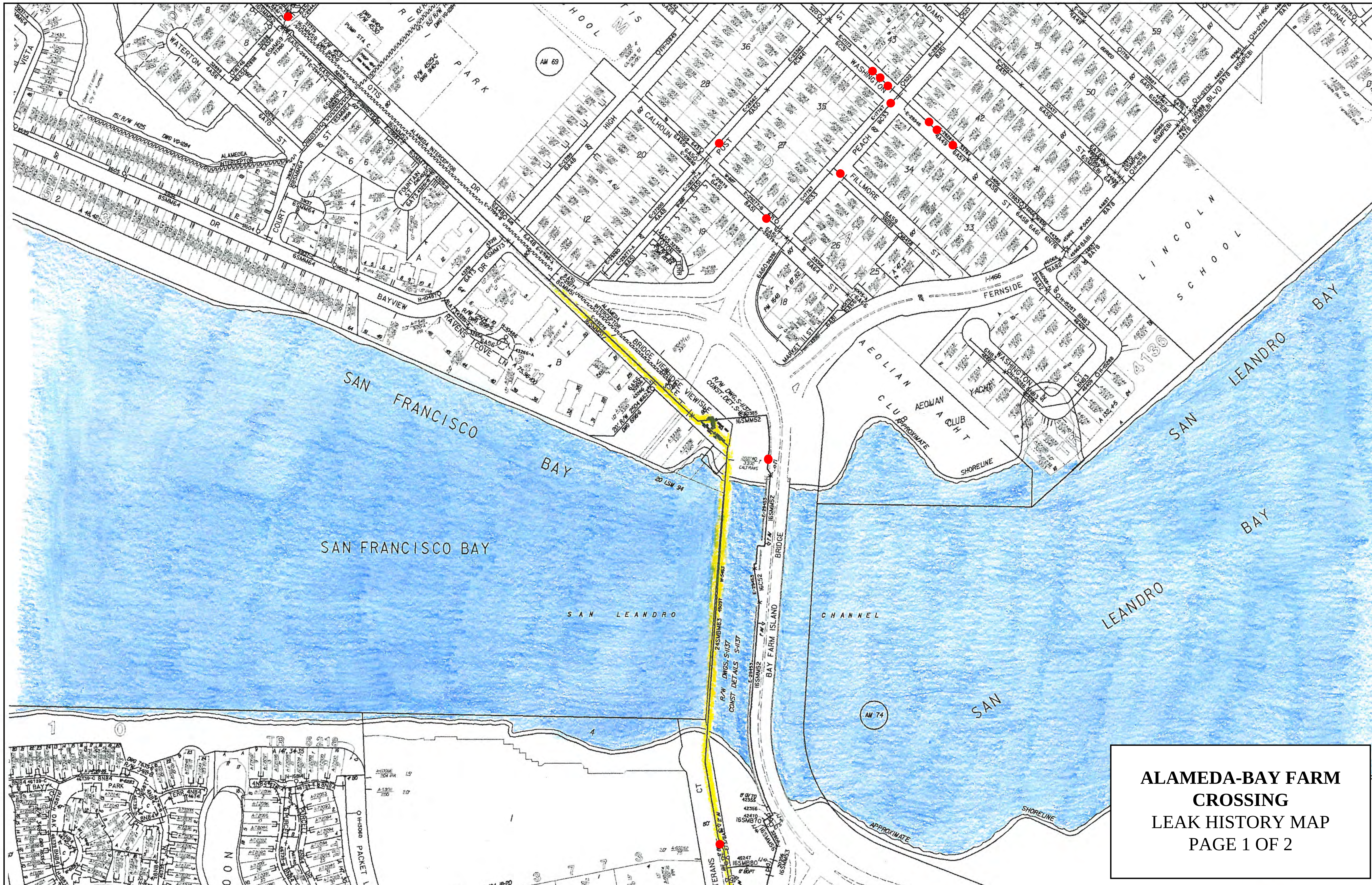
MATCHLINE 2A

ALICE STREET CROSSING
LEAK HISTORY MAP
PAGE 2 OF 3

MATCHLINE 2A

ALICE STREET CROSSING
LEAK HISTORY MAP
PAGE 3 OF 3





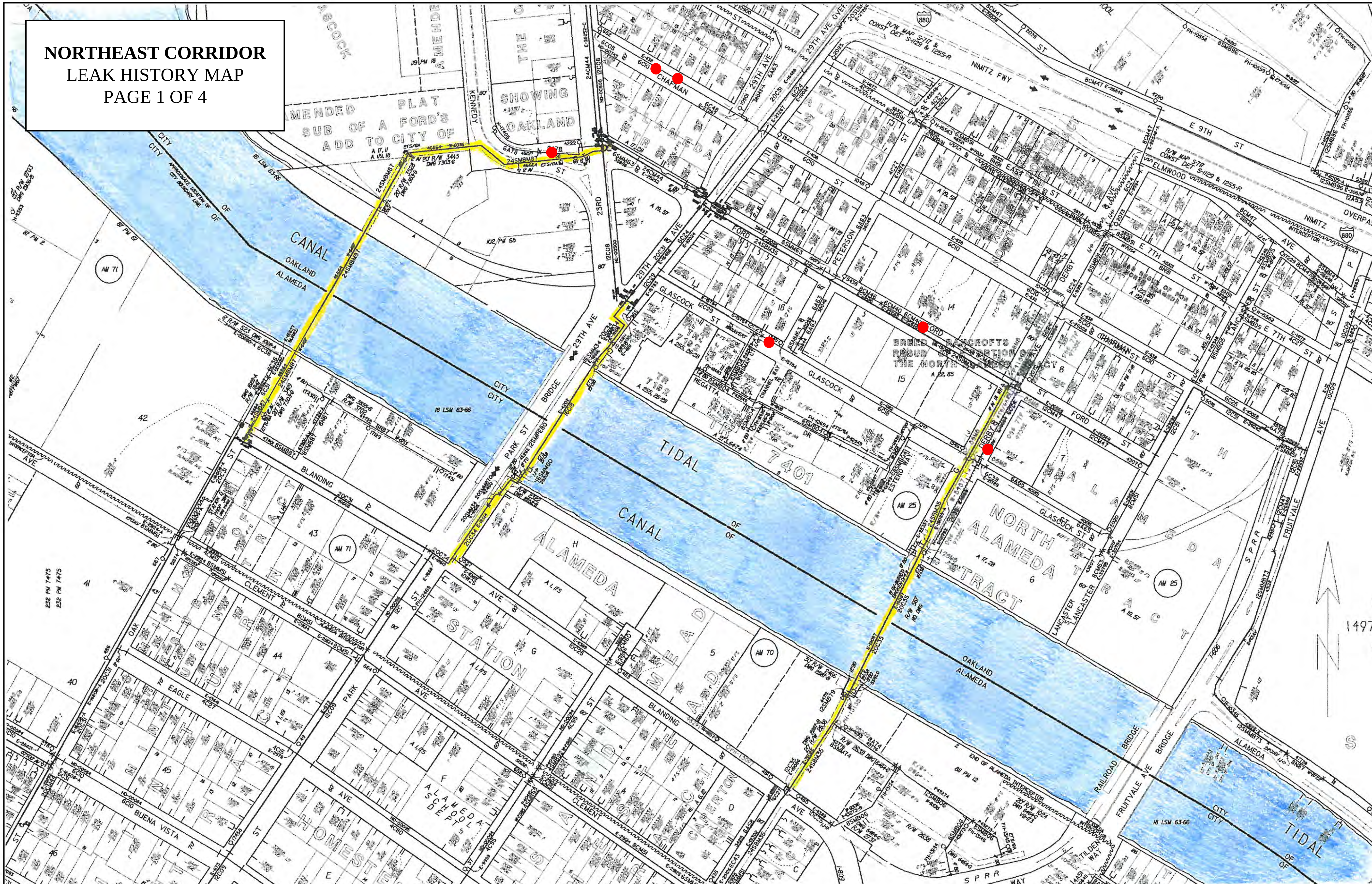
ALAMEDA-BAY FARM CROSSING
LEAK HISTORY MAP
PAGE 1 OF 2

MATCHLINE 1B

MATCHLINE 1B

ALAMEDA-BAY FARM CROSSING
LEAK HISTORY MAP
PAGE 2 OF 2

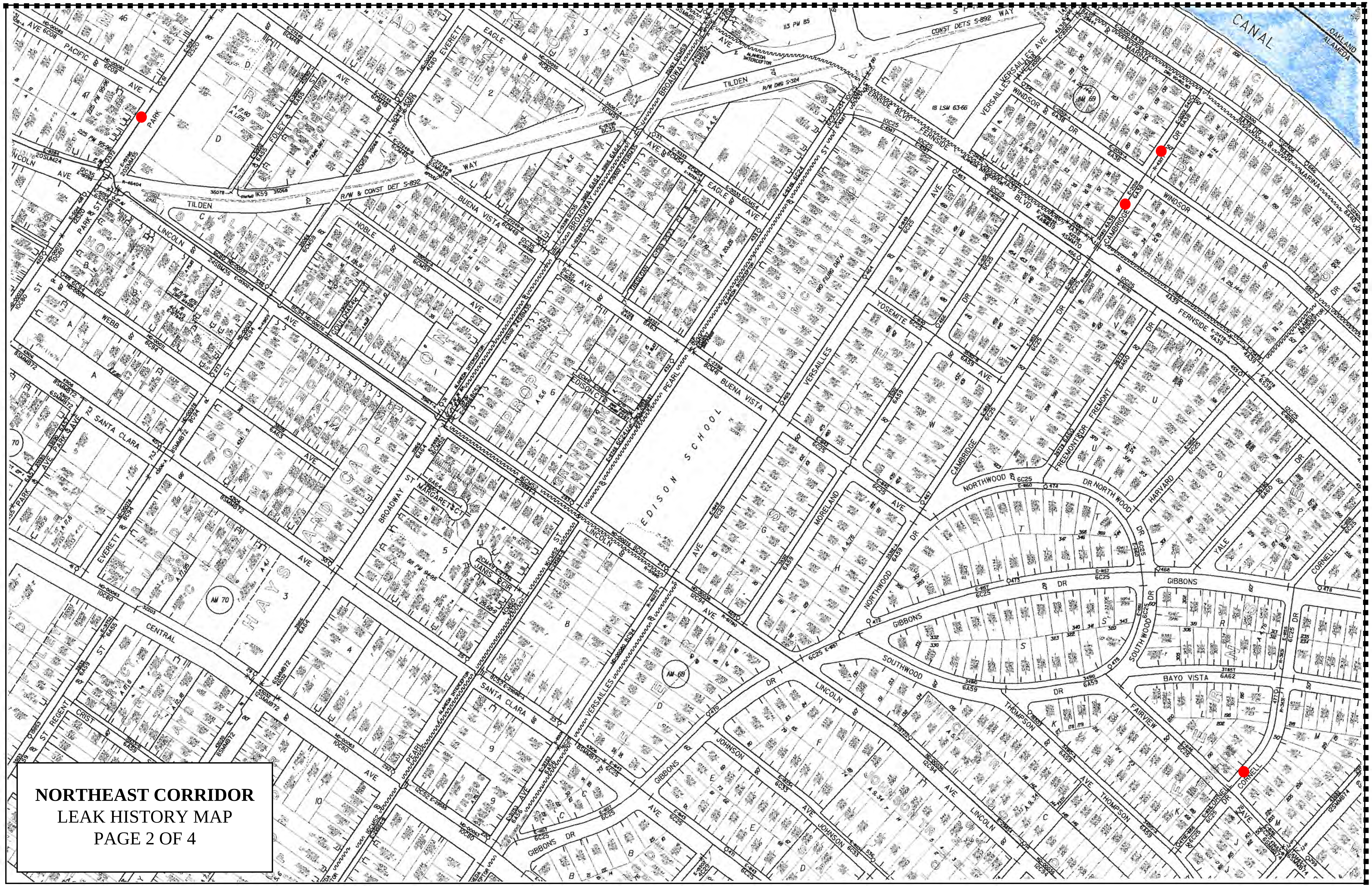
NORTHEAST CORRIDOR
LEAK HISTORY MAP
PAGE 1 OF 4



MATCHLINE 2 C

MATCHLINE 1C

MATCHLINE 1C



MATCHLINE 3C

NORTHEAST CORRIDOR
LEAK HISTORY MAP
PAGE 2 OF 4

NORTHEAST CORRIDOR
LEAK HISTORY MAP
PAGE 3 OF 4

MATCHLINE 2C

MATCHLINE 4C

MATCHLINE 4C

MATCHLINE 3C

NORTHEAST CORRIDOR
LEAK HISTORY MAP
PAGE 4 OF 4

Appendix F

Vulnerability Assessment

EAST BAY MUNICIPAL UTILITY DISTRICT

DATE: September 19, 2012

MEMO TO: Denise Cicala, Associate Civil Engineer

THROUGH: Atta B. Yiadom, Senior Civil Engineer

FROM: Yogesh Prashar, Associate Civil Engineer

SUBJECT: Geotechnical Evaluation for the Alameda/Bay Farm Crossings, Alameda, CA

INTRODUCTION

This memo presents the results and recommendations of a geotechnical evaluation, prepared by the Materials Engineering Section (MES), for seven pipeline crossings located in Oakland and Alameda, to Bay Farm channels, California.

Background and Understanding:

Project understanding is based on our meeting and e-mail correspondences with you. The purpose of this project is to qualitatively rank the relative vulnerability of each of the seven pipeline crossings. We reviewed the existing soil borings, data, reports, and pipeline alignments you provided to us for an understanding of the site conditions. The project entails a geotechnical evaluation of 7 pipeline crossings. Five of these pipelines cross from Oakland to Alameda, and two cross from Alameda to the Bay Farm Island.

Scope of Work:

The purpose of this evaluation was to collect, review, and evaluate the existing geologic and geotechnical data at each pipeline crossing, and to develop a pipeline channel crossing vulnerability assessment. In order to accomplish the work, MES reviewed all pertinent geotechnical information, drawings, and construction documents to gain understanding of the project. In addition, MES prepared this memorandum, which provides a qualitative geotechnical evaluation.

REVIEW OF EXISTING INFORMATION

For the initial phase of our geotechnical evaluation, MES reviewed the available geologic and geotechnical data from previous investigations in the general areas of the site, as provided and included in the references. We also reviewed project drawings and reports you provided for each of the seven crossings. The following subsections summarize the findings of our review:

<u>Existing Conditions</u>

The six crossings evaluated are shown below in Illustration 1. Each pipeline crossing is located below ground surface, with the exception of Alameda-Bay Farm 1, which is suspended from the bridge deck for most of the channel crossing.

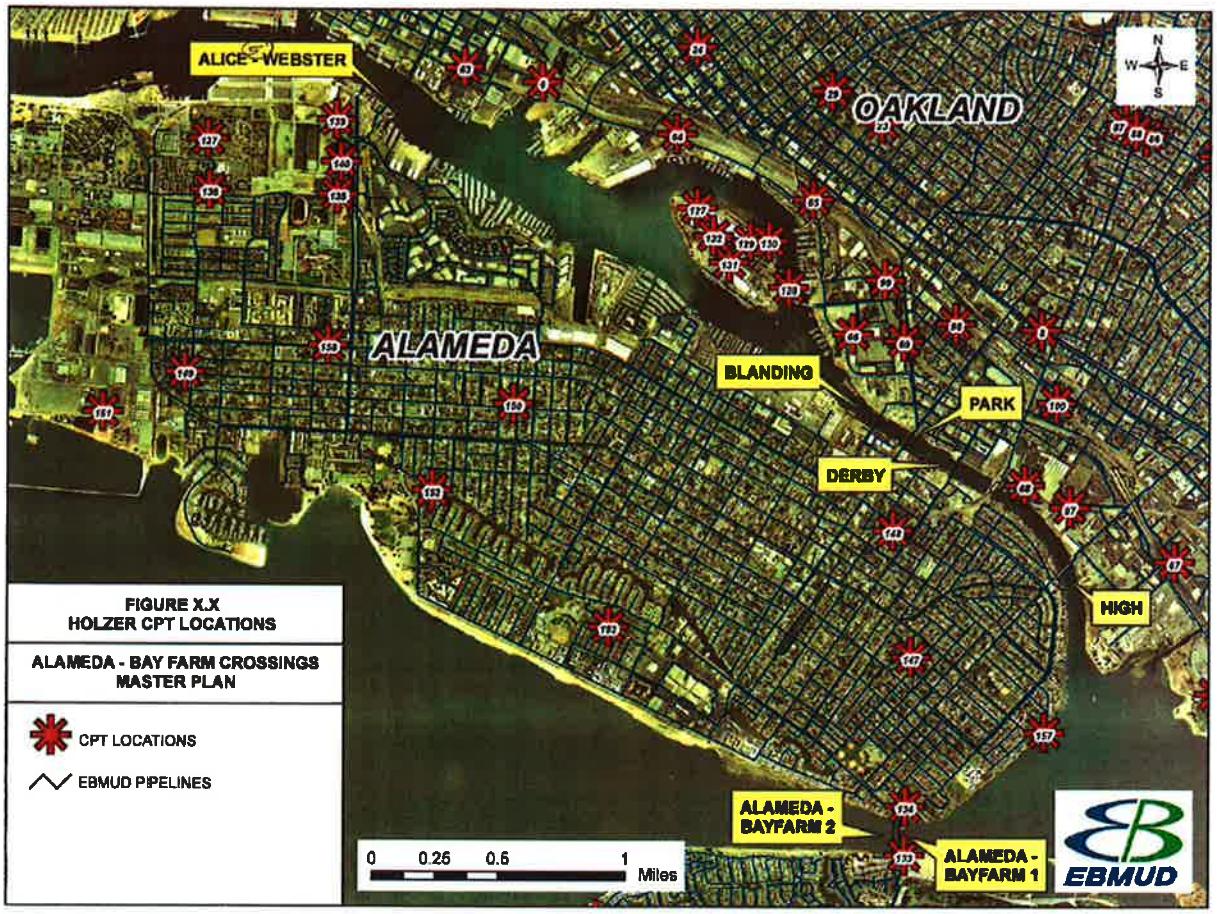


Illustration 1: Alameda Crossing Map Indicating Location of Pipeline

No subsurface investigations were conducted at this site, and we relied upon the previous geotechnical investigation reports to make an assessment. Ground water was high in most cases since soils and pipelines are below the groundwater table. The figures attached to the end of this report (see Figures 1 to 7) show the soil stratigraphy and approximate pipeline alignment for each crossing. From previous investigations, there are no buried utilities underneath the project site. The subsurface soils at each crossing can be generalized as follows:

1. Alice-Webster Crossing (Ref. Figure 1A): Multiple boring logs were done at the Alice-Webster crossing. The Alice-Webster pipeline crosses the Oakland Inner Harbor. In the vicinity of the pipeline, the soil near the Alameda side of the crossing is underlain with very soft, high plasticity silt. The soil surrounding the pipeline at the Oakland Inner Harbor contains clayey-sand with low blow counts. This clayey-sand layer is loose and can liquefy

under a strong earthquake. This soil poses a high risk to the pipeline and it is susceptible to large deformation and possible damage in the event of an earthquake. The referenced 2000 Olivia Chen Consultants report concluded that soils in the vicinity of the pipeline are liquefiable and require mitigation.

2. Blanding Crossing (Ref. Figures 2A & B): Blanding is located on the northeast side of Alameda. The soil throughout the vicinity of the pipeline can be described as a soft, silty clay. Additionally, liquefiable soils are also present in the crossing. This pipeline, however, is **not** under the risk of damage from an earthquake **because** it is embedded deeper in the soil; it is located below the sandy layers, so liquefaction-induced settlement will not affect the pipeline.
3. Park Crossing (Ref. Figures 3A, B, & C): The park crossing is located east of the Blanding crossing. The information provided here was limited; the soil profile only described the subsurface conditions qualitatively. There is a sand layer located underneath near the pipeline that could pose a liquefaction hazard. There is also a risk of lateral spreading and slope instability due to the liquefaction of this sand layer. However, a complete assessment can only be done through further tests, and this assessment is based on limited data.
4. Derby Crossing (Ref. Figures 4A, B, & C): Subsurface soils surrounding the pipeline generally are potentially liquefiable and available boring data is sparse. Subsurface soils surrounding the pipeline consist of soft Bay Mud followed by layers of sandy clay to clayey sands with gravel. Boring stick logs did not contain strength data or field blow count data.
5. High Street Crossing (Ref. Figures 5A & B): The pipeline burial depth is fairly shallow (~10 feet). Once again, since the boring stick logs do not contain details of any soil strength, index properties, and blow count data, we assume based on descriptions of Bay Mud overlying alternating layers of clayey sand to sandy clays that soils are potentially liquefiable.
6. Alameda Bay Farm 1 Crossing (Ref. Figure 6A): The pipe is suspended from underneath the bridge connecting Otis Drive in Alameda to Doolittle Drive in Bay Farm. About a 160-foot stretch (in plan view) of pipeline submerges and is embedded into bay channel soils. Since the pipeline is suspended on a bridge deck which is pile supported, we conclude that provided the bridge deck is seismically stable, the pipeline should also perform adequately. The embedded (~160 feet) portion needs to be evaluated. Boring logs were not available and additional research will be required to acquire this from the City or County of Alameda.
7. Alameda Bay Farm 2 Crossing (Ref. Figure 7A): Figure 7A shows boring stick logs along the Alameda-Bay Farm 2 pipeline described below. The pipeline is embedded into soft organic clays and potentially liquefiable sandy soils. Based on the soil descriptions and blow count data provided in the stick logs shown, pipeline vulnerability is considered to be high.

SUMMARY OF PIPELINE FRAGILITY

Based on our review of:

- Subsurface soil types
- Ground water depth
- Potential for liquefaction
- Presence of soft clays (Bay Mud)
- Pipeline alignment relative to surface and sub-surface soil types

We have developed a table (see Table 1) that summarizes the above criterion and have developed our conclusions on the pipeline fragility. In summary only two (Blanding and Alameda-Bayfarm-1) crossings are relatively resilient in the present condition to earthquake threat. All other crossings are considered to be quite fragile to earthquakes. These five crossings will also require additional evaluations to quantify the fragility. With the exception of Alice-Webster crossing all remaining six crossings will require further studies.

SUPPORTING DOCUMENTS

"Final Geotechnical and Environmental Investigation, EBMUD Alameda NAS Discharge Pipeline and Siphon Project, Oakland, California", prepared by Olivia Chen Consultants, dated September 24, 2000.

CLOSING

If you have any comments with regard to the contents of this memo, we will be pleased to discuss them with you. If you have any questions, please call Yogesh Prashar at ext. 0520.

YP:gh

Attachments:

Table 1	Alameda Pipeline Crossings Vulnerability Study
Figure 1A	Alice-Webster General Soil Profile
Figure 2A	Blanding General Soil Profile
Figure 2B	Blanding Detailed Soil Profile
Figure 3A	Park General Soil Profile
Figure 3B	Park General Soil Profile Continued
Figure 3C	Park General Soil Profile Continued
Figure 4A	Derby Detailed Soil Profile
Figure 4B	Derby General Soil Profile
Figure 4C	Derby Plan View
Figure 5A	High General Soil Profile
Figure 5B`	High General Soil Profile Continued
Figure 6A	Alameda-Bay Farm 1 General Plan View
Figure 7A	Alameda-Bay Farm 2 Detailed Soil Profile

cc: Yogesh Prashar
Records Management
Project File
MERF

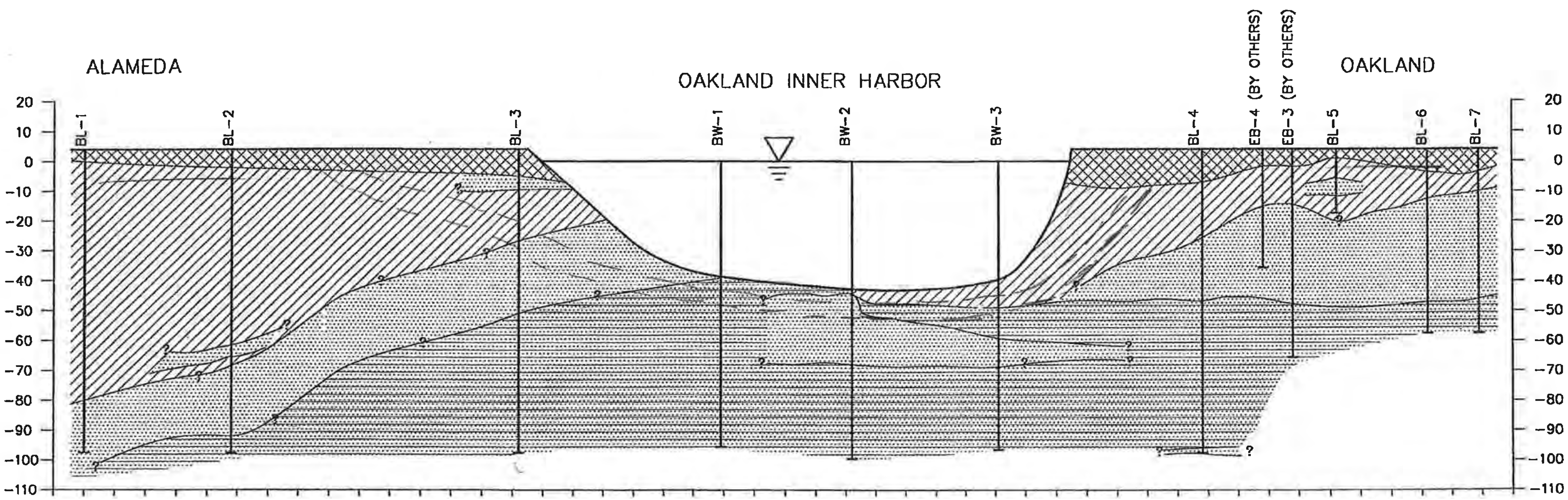
W:\WORKGRP\DATA\Yprashar\Projects\Alameda_Crossings\Geotech Memo Draft Alameda Crossings.doc

TABLE 1: Alameda Pipeline Crossings Vulnerability Study

Crossing:	Alameda Side*	Oakland-Alameda Channel*	Oakland Side*	Discussion	Ranking (1-5, 5 most severe)
Alice-Webster	High plasticity silt with very low blow counts.	Sand, Clayey-Sand. Older Bay Mud.	Pipeline goes through fill.	At the sloping area near the Oakland Inner Harbor, the pipeline passes through a clayey-sand layer. This layer is loose and liquefiable; it poses a high risk to the pipeline and can cause slope instability as well.	5.
Blanding	Silty Clay	Silty Clay	Silty Clay – Silty Sand.	This pipeline is embedded deeper in the soil and is located 15' from the ground surface. It is located below the loose, potentially liquefiable layers; it does not pose a high risk to the pipeline.	1-2.
Park	Stiff grey clay, some fine yellow sand.	Stiff grey clay, clay.	Grey sandy clay. Stiff grey clay,	Information is limited for this crossing. There is a sand layer located near the pipeline that could pose a liquefaction hazard. There is also a risk of lateral spreading and slope instability due to liquefaction.	4-5
Derby	Stiff black adobe clay, yellow sandy clay	Stiff grey/yellow clay	Stiff yellow/blue clay	Technical information is very limited for this site. There is a 2:1 slope and layers of sand that could liquefy. However, additional technical information is needed.	4
High	Stiff clay of various colors.	Grey sandy clay	Soft mud, grey sand	There are sand layers below this pipeline which could be liquefiable. However, more technical information is needed.	4-5
Alameda – Bayfarm 1	Above soil (Alameda side)	Above soil (partially goes under deck)	Above soil (Bay Farm area, not Oakland)	This pipeline is located along the span of the bridge except for a portion that goes under the deck of the draw bridge. However, there are deep foundation piles there and assuming they work, the soil doesn't pose a high risk to the pipeline. The embedded portion needs to be evaluated.	2
Alameda – Bayfarm 2	Sandy silt, low plasticity (Alameda side)	Very soft high plasticity silty clay	Very soft clayey sandy silt (Bay Farm area, not Oakland)	The pipeline goes through a soft silty clay layer, which poses a risk because it can amplify ground shaking and induce differential stresses on the pipeline.	4-5

*Note: Soil description is for the soil in the vicinity of the pipeline.

ALICE-WEBSTER



LEGEND

MAN-MADE FILL

YOUNGER BAY MUD

SAND, CLAYEY SAND

OLDER BAY MUD

GENERAL PIPELINE LOCATION

NOTE:

STRATIGRAFIC LAYERS DEPICTED IN THIS DRAWING ARE ONLY APPROXIMATE AND ARE INFERRED BASED ON INFORMATION FROM WIDELY-SPACED BORINGS. ACTUAL SUBSURFACE CONDITIONS MAY VARY FROM THOSE SHOWN HERE.

0 100 200

HORIZONTAL SCALE: 1" = 200'

0 20 40

VERTICAL SCALE: 1" = 40'

Olivia Chen Consultants, Inc. Civil · Environmental · Geotechnical Job No. 1256	EAST BAY MUNICIPAL UTILITY DISTRICT ALAMEDA NAS DISCHARGE PIPELINE AND SIPHON PROJECT	ALAMEDA INTERCEPTOR SIPHON AND JUNCTION STRUCTURES CROSS SECTION	SCALE: AS SHOWN

G:\PROJECTS\EBMUD\1256\ALAMEDA\X-SECT1.DWG

Figure 1A
Alice Webster Soil Profile

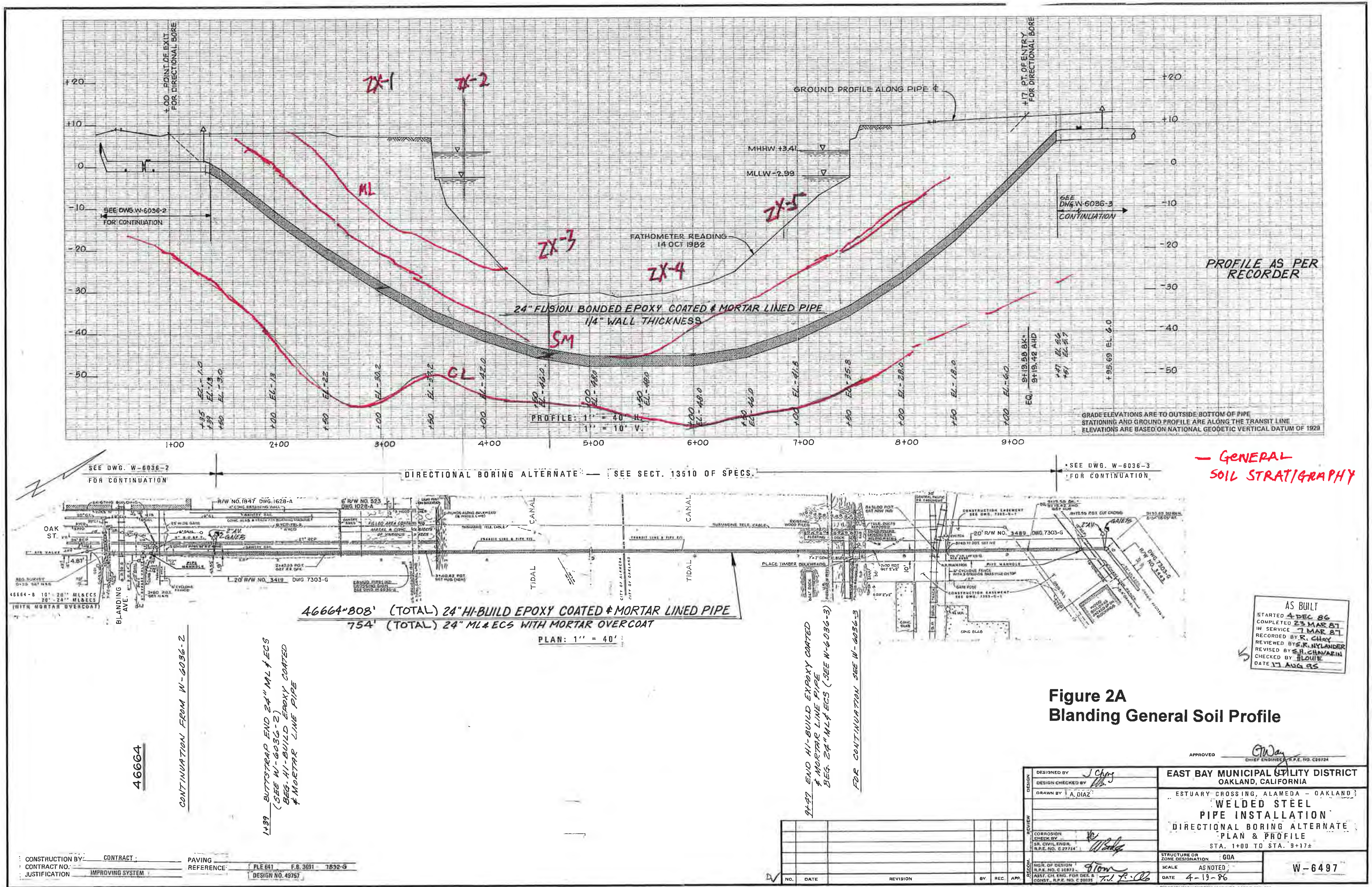
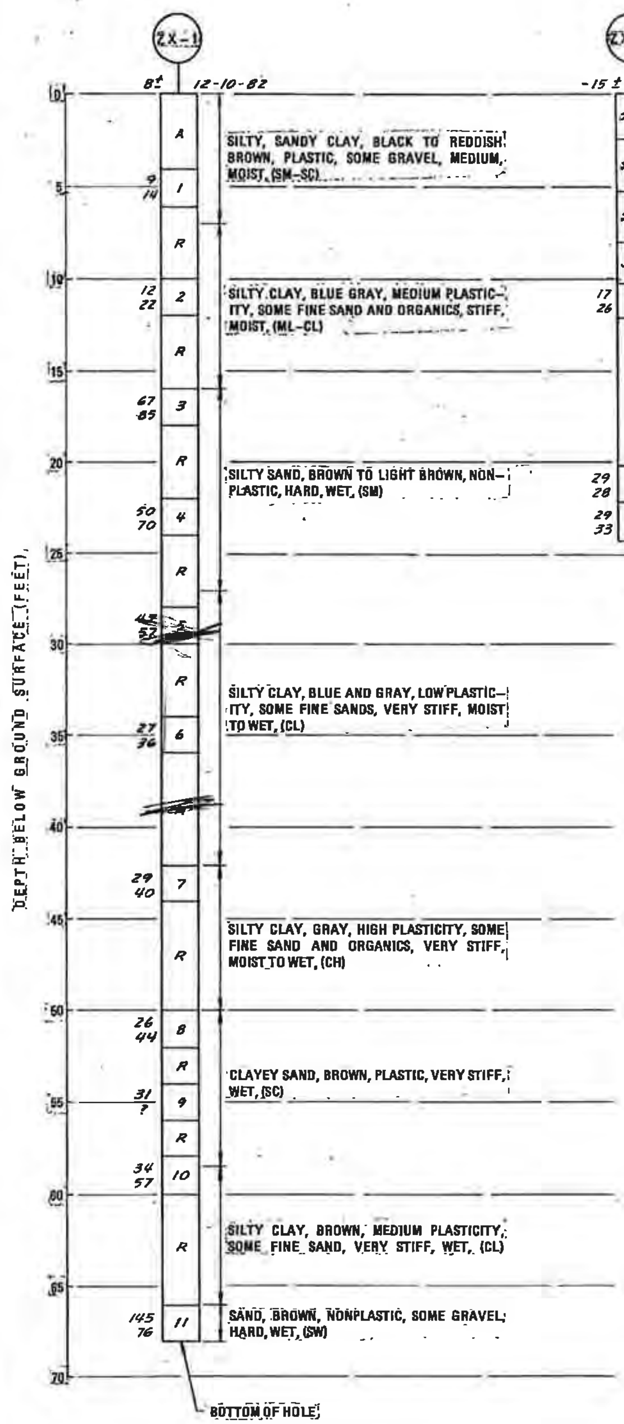
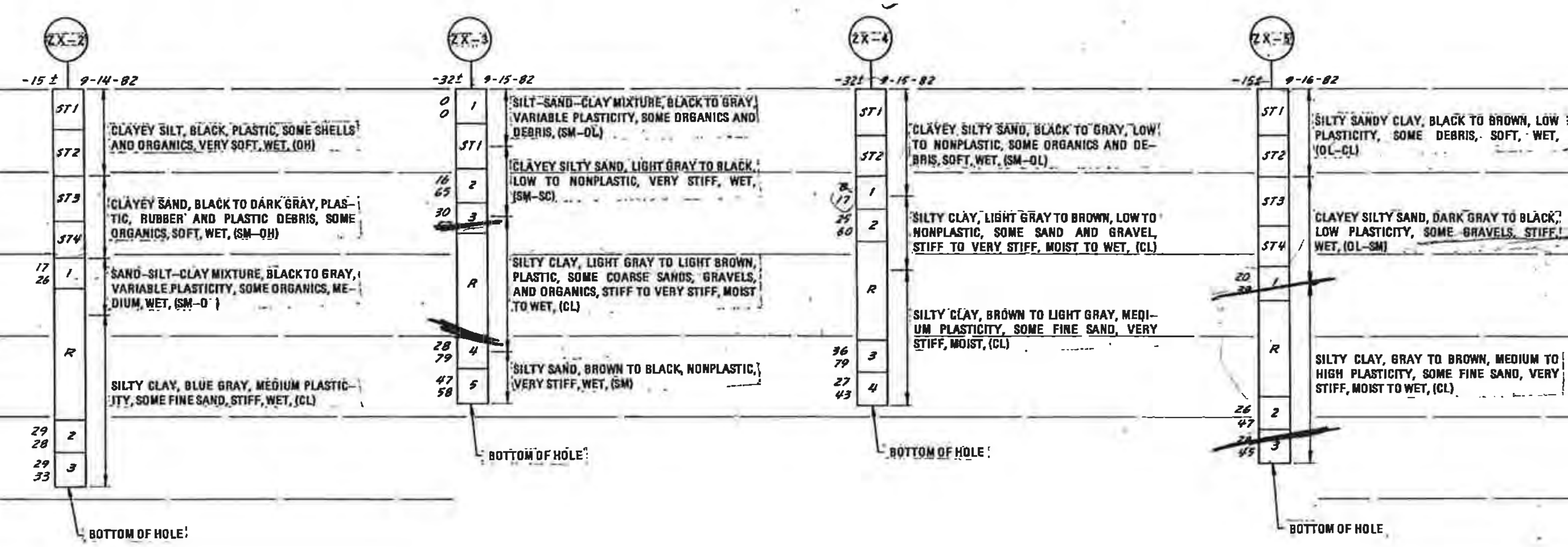
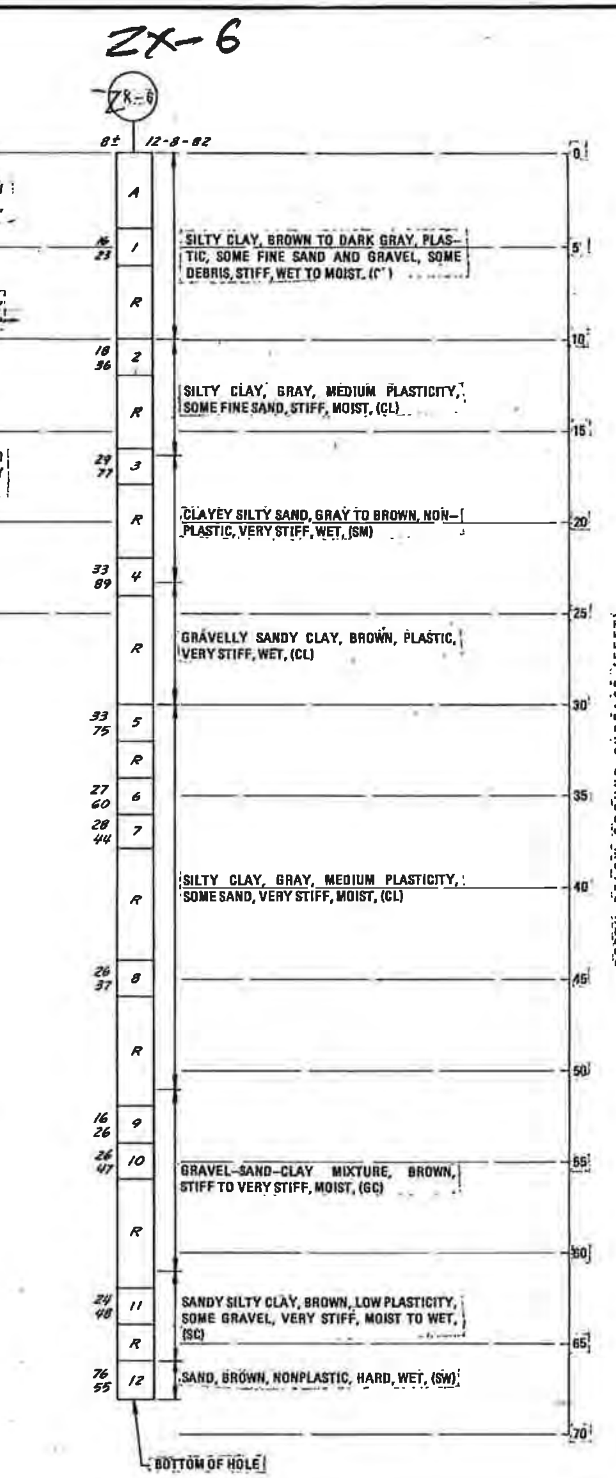


Figure 2A
Blanding General Soil Profile





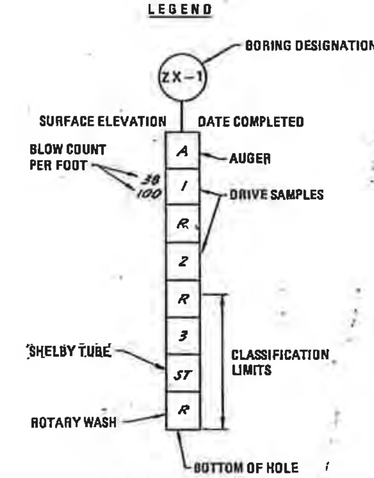


NOTES

1. FOR LOCATION OF BORINGS, SEE DRAWING 7270-G-1 & 2.
2. ELEVATIONS ARE TO USC&GS STANDARD MEAN SEA LEVEL DATUM (1929).
3. ALL DISTRICT BORINGS ON THIS PROJECT HAVE BEEN DESIGNATED BY PREFIX ZX AND THE NUMBER OF THE BORING.
4. BORINGS LOGGED ON THESE SHEETS WERE DRILLED BY THE PITCHER DRILLING COMPANY UNDER CONTRACT TO THE DISTRICT. THE EQUIPMENT CONSISTED OF A FAILING ~~MODEL 750~~ DRILL RIG USING 4-INCH DIAMETER HELICAL-FLIGHT AUGERS AND ROTARY JET.
5. SAMPLES WERE OBTAINED BY MEANS OF:
   A. A MODIFIED CALIFORNIA-TYPE SPLIT-SPOON SAMPLER DRIVEN BY A 140-LB. HAMMER DROPPED 30 INCHES.
   B. THREE-INCH DIAMETER SHELBY THIN-WALLED TUBES DRIVEN BY AN OSTERBERG SAMPLER.

   DRIVE SAMPLES WERE RETAINED IN 2-INCH DIAMETER BY 6-INCH BRASS LINER TUBES, CAPPED AND SEALED. SHELBY TUBES WERE RETAINED, CAPPED AND SEALED.
6. CLASSIFICATIONS ARE ACCORDING TO THE SYSTEM INDICATED ON DRAWING 3125-GB. ALL MATERIALS, WHETHER OVERBURDEN OR BEDROCK FORMATION, HAVE BEEN ASSIGNED A BASIC SOIL CLASSIFICATION: GEOLOGIC CLASSIFICATIONS OR NAMES ARE INDICATED IN PARENTHESES AS ADJECTIVES. CLASSIFICATIONS WERE ASSIGNED MAINLY ON THE BASIS OF VISUAL EXAMINATION ONLY AND SUBSTANTIATED WITH A MINIMUM NUMBER OF LABORATORY TESTS.
7. REPRESENTATIVE UNDISTURBED SAMPLES, AS RECOVERED, MAY BE EXAMINED AT THE DISTRICT'S SOIL LABORATORY, 2500 E. 29TH STREET, OAKLAND, CALIFORNIA. FIELD LOGS OF ALL BORINGS ARE ON FILE AND A REPORT DESCRIBING ALL PHASES OF THE DISTRICT'S SOILS AND FOUNDATION INVESTIGATION, INCLUDING LABORATORY TEST RESULTS AND PERTINENT DESIGN CONSIDERATIONS, MAY BE EXAMINED AT THE DISTRICT'S ENGINEERING OFFICE, 2130 ADELINE STREET, OAKLAND, CALIFORNIA.

LEGEND



= GENERAL PIPELINE LOCATION

Figure 2B
Blanding Detailed Soil Profile

APPROVED
CHIEF ENGINEER, R.P.E. NO. C 8216

EAST BAY MUNICIPAL UTILITY DISTRICT
OAKLAND, CALIFORNIA

ESTUARY CROSSING
OAKLAND-ALAMEDA PIPELINE

LOG OF BORINGS

DESIGN BY	
DESIGN CHECK BY	
DRAWN BY	M. MURRAY
PIPELINE	CORR FNDN
ASST. CH. ENG., DES. & CONST. R.P.E. NO. C 26734	
STRUCTURE OR ZONE DESIGNATION	
SCALE	1" = 5' VERT.
DATE	13 JAN 83
DRAWING	7296-G

NO.	DATE	REVISION	BY	APP.

MISC.

E-160 3/82

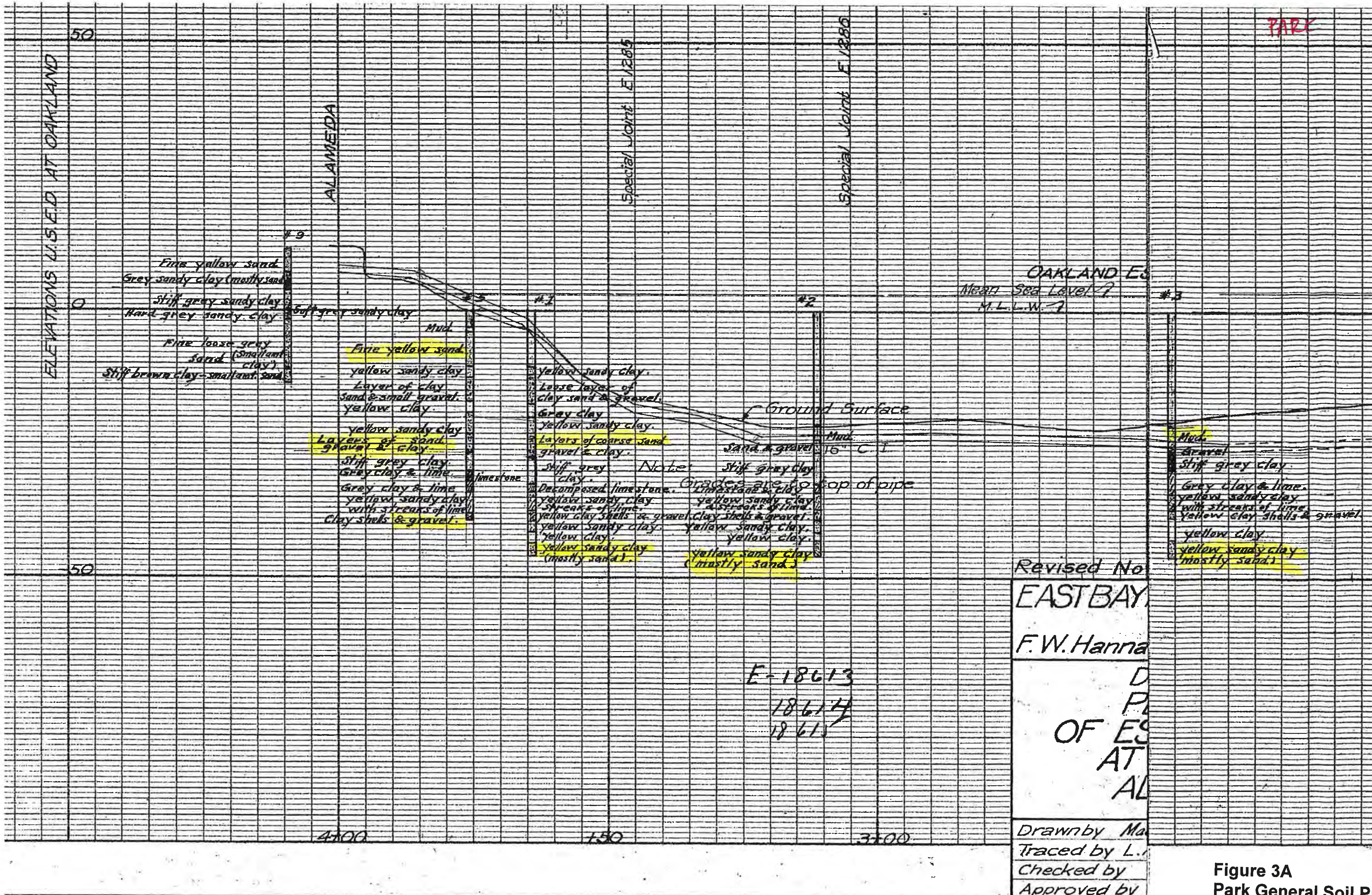


Figure 3A
Park General Soil Profile

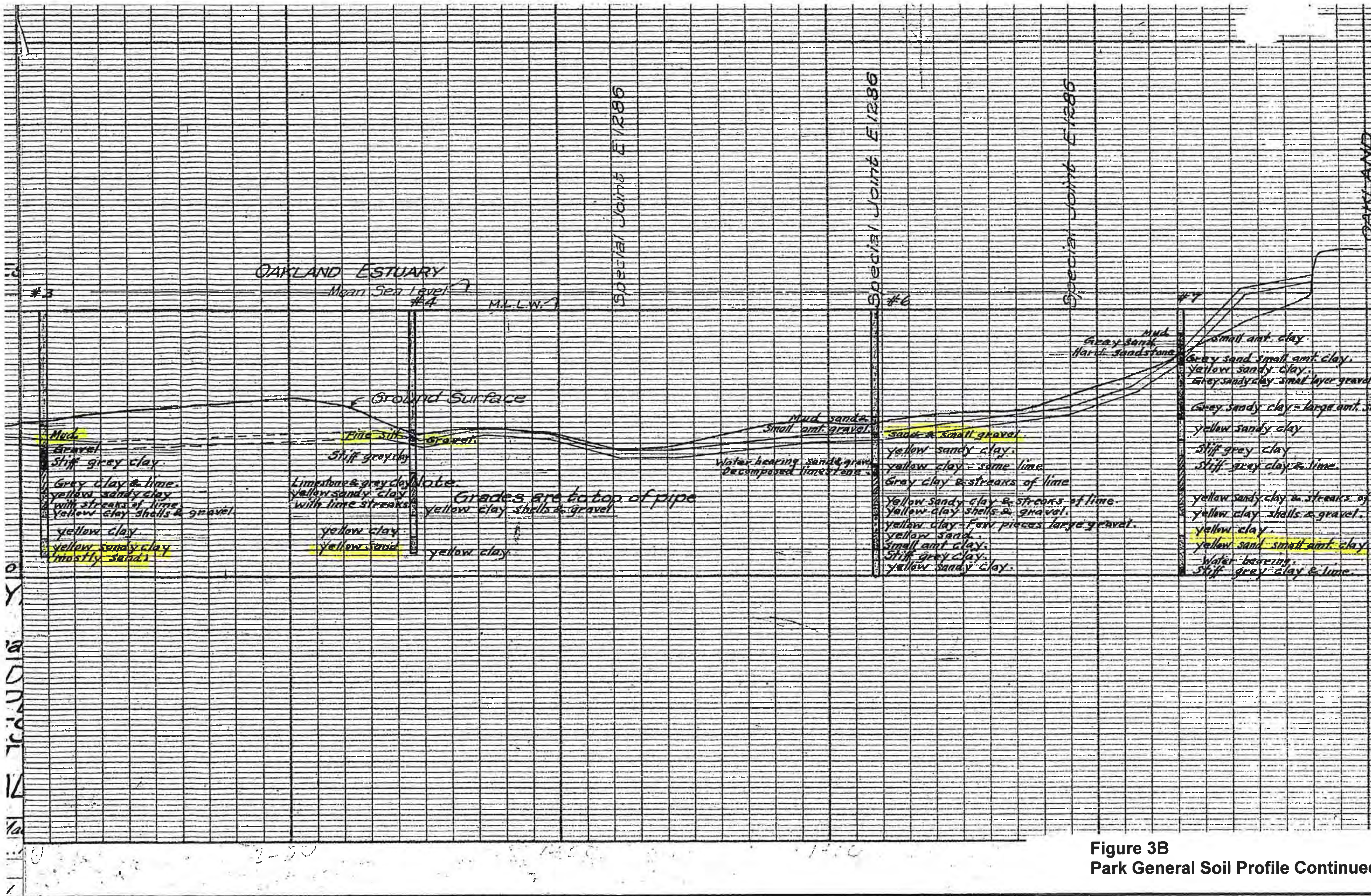


Figure 3B
Park General Soil Profile Continued

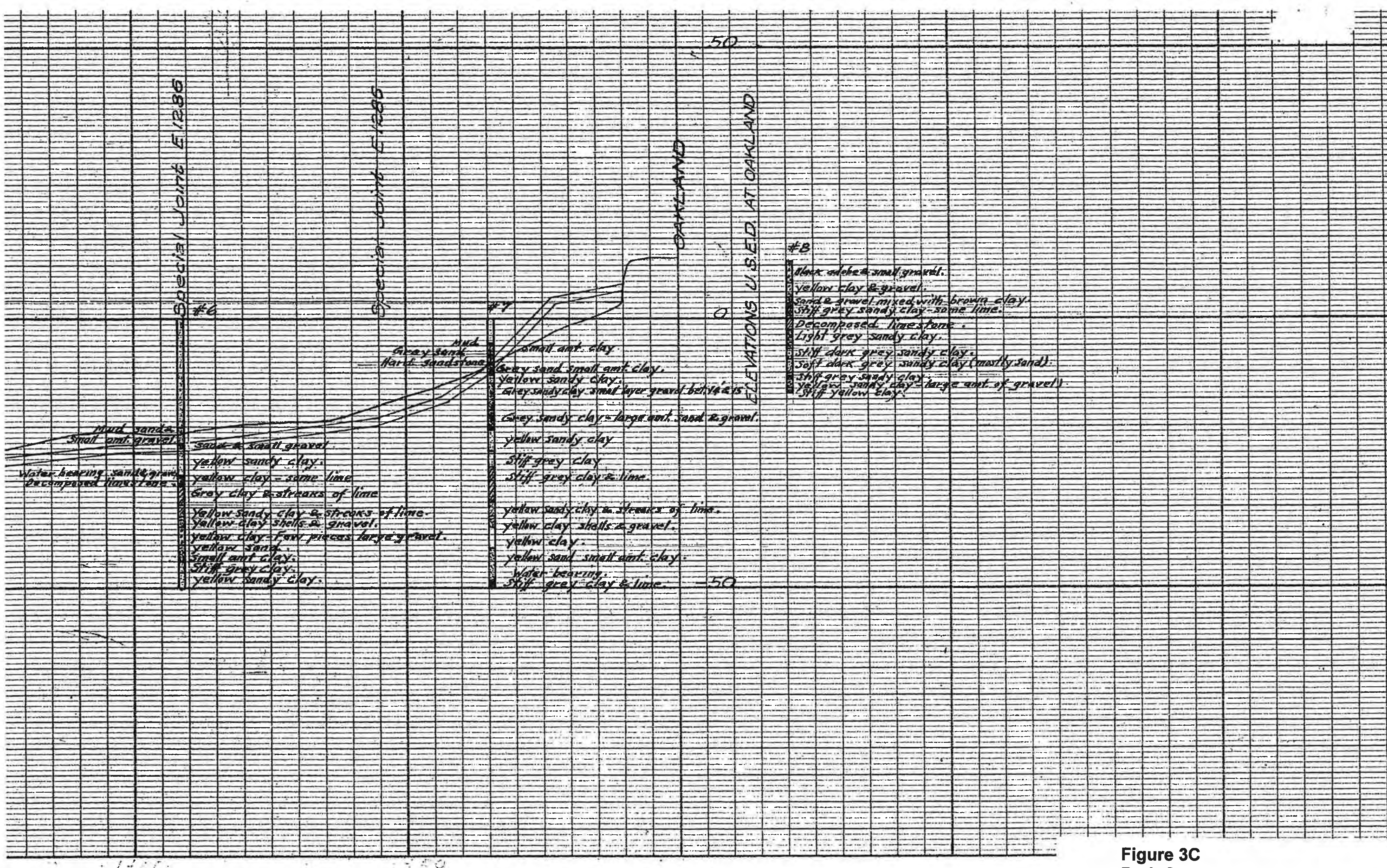


Figure 3C
Park General Soil Profile Continued

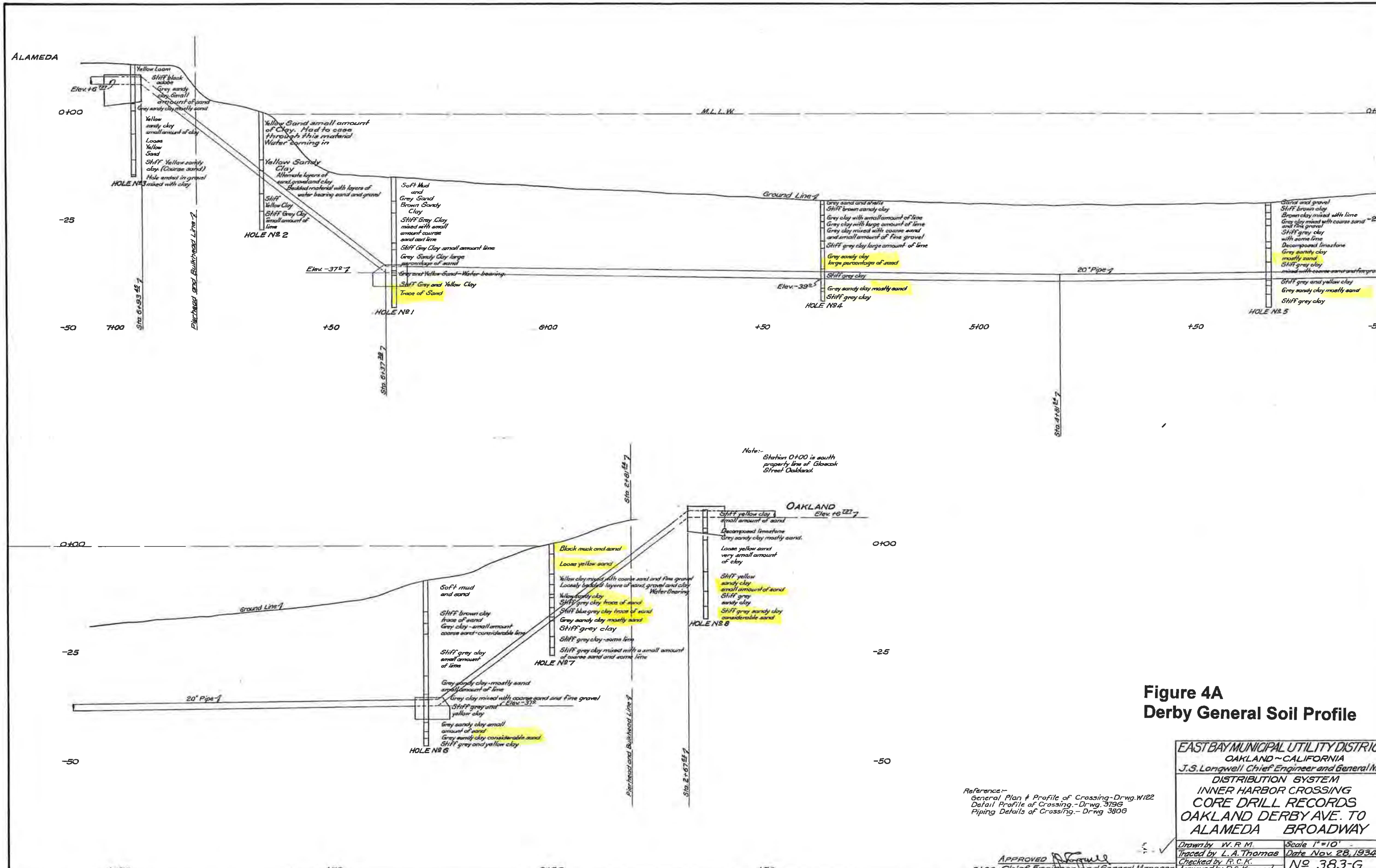
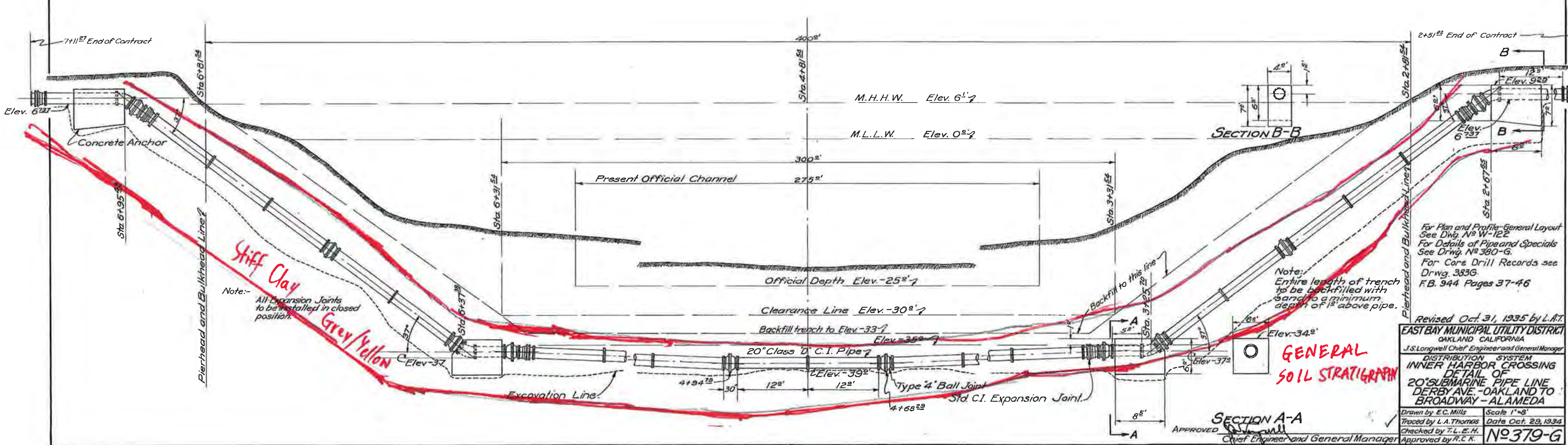


Figure 4A
Derby General Soil Profile

Figure 4B
Derby General Soil Profile



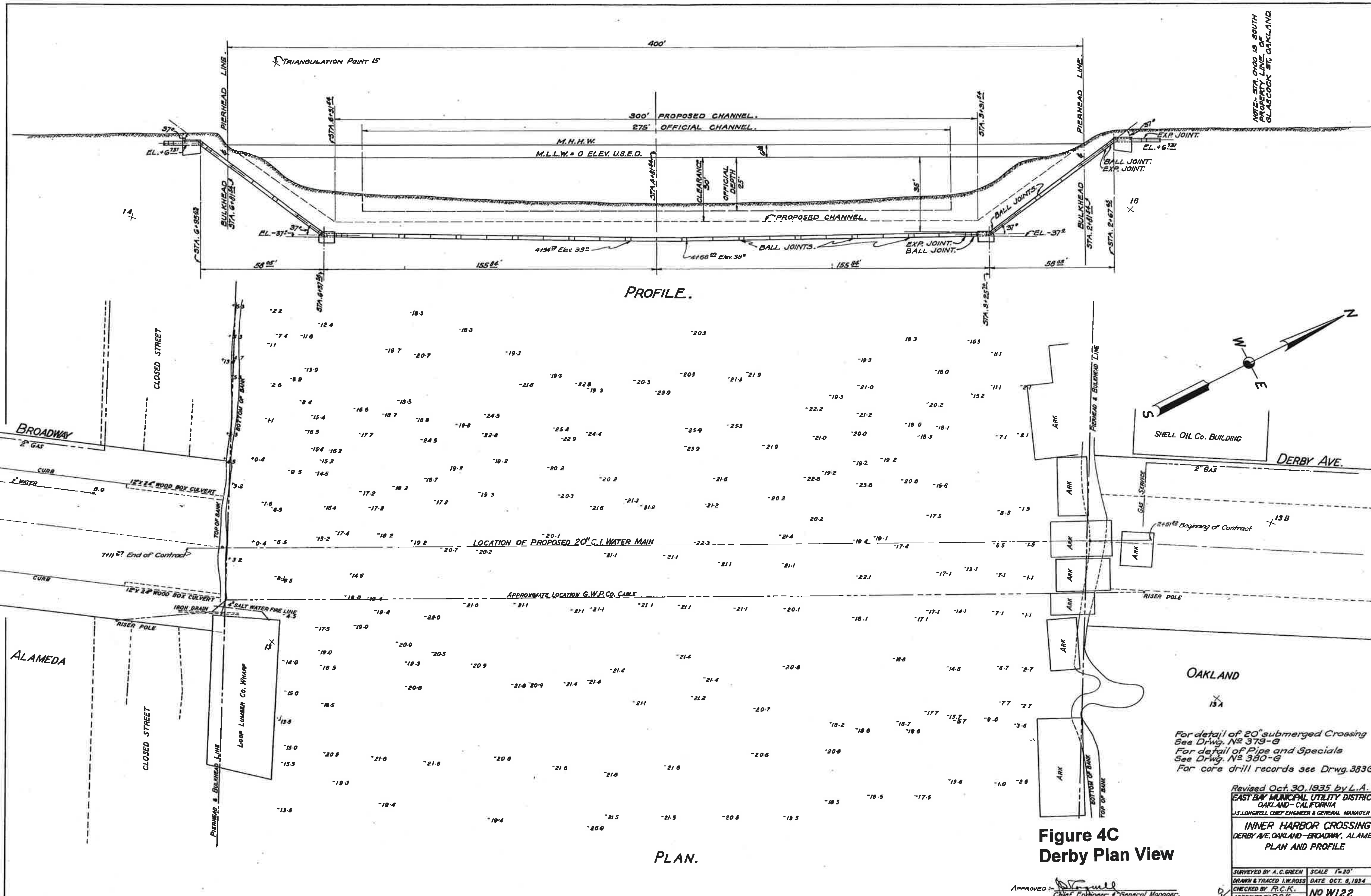


Figure 4C
Derby Plan View

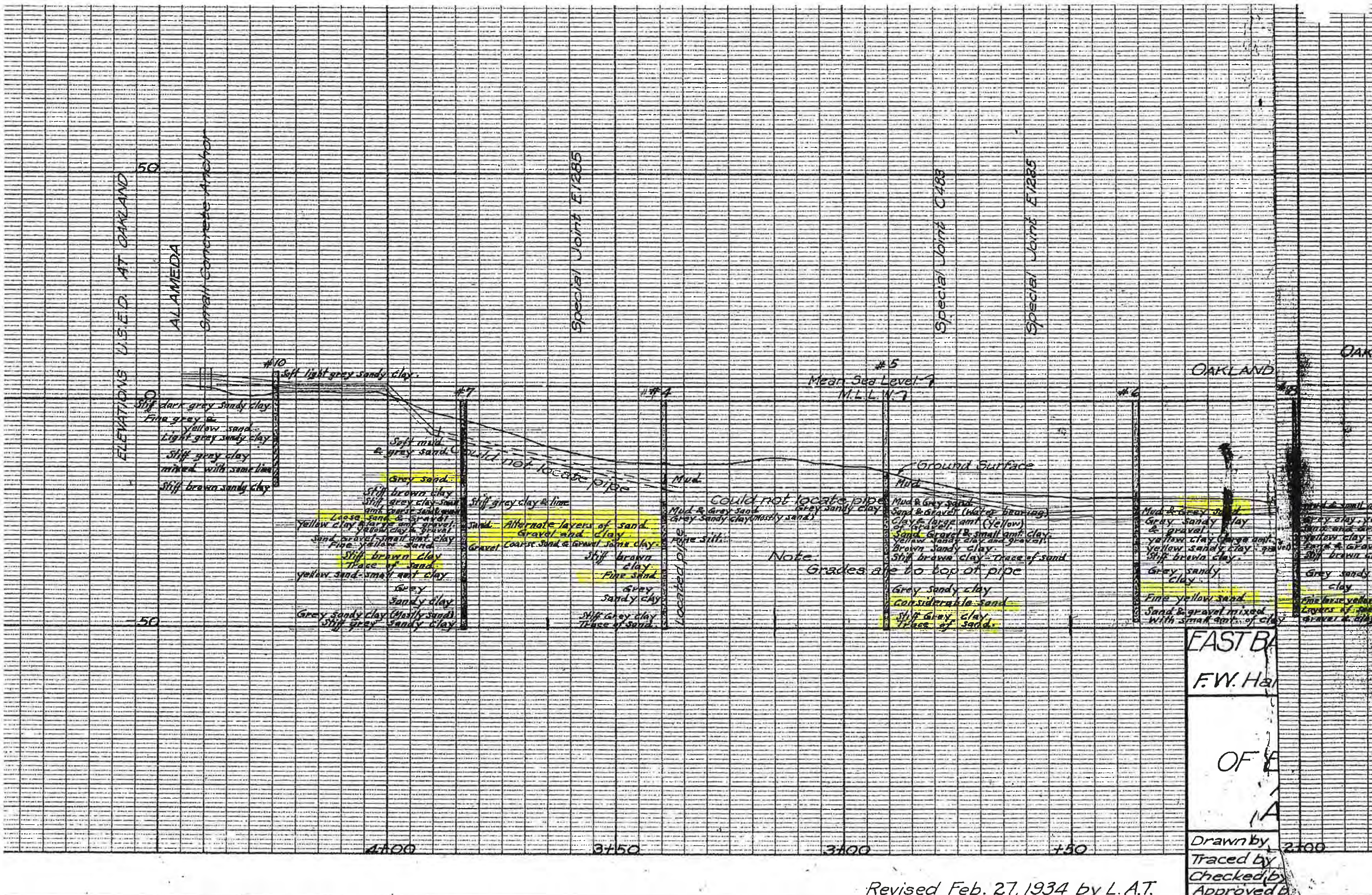


Figure 5A
High General Soil Profile

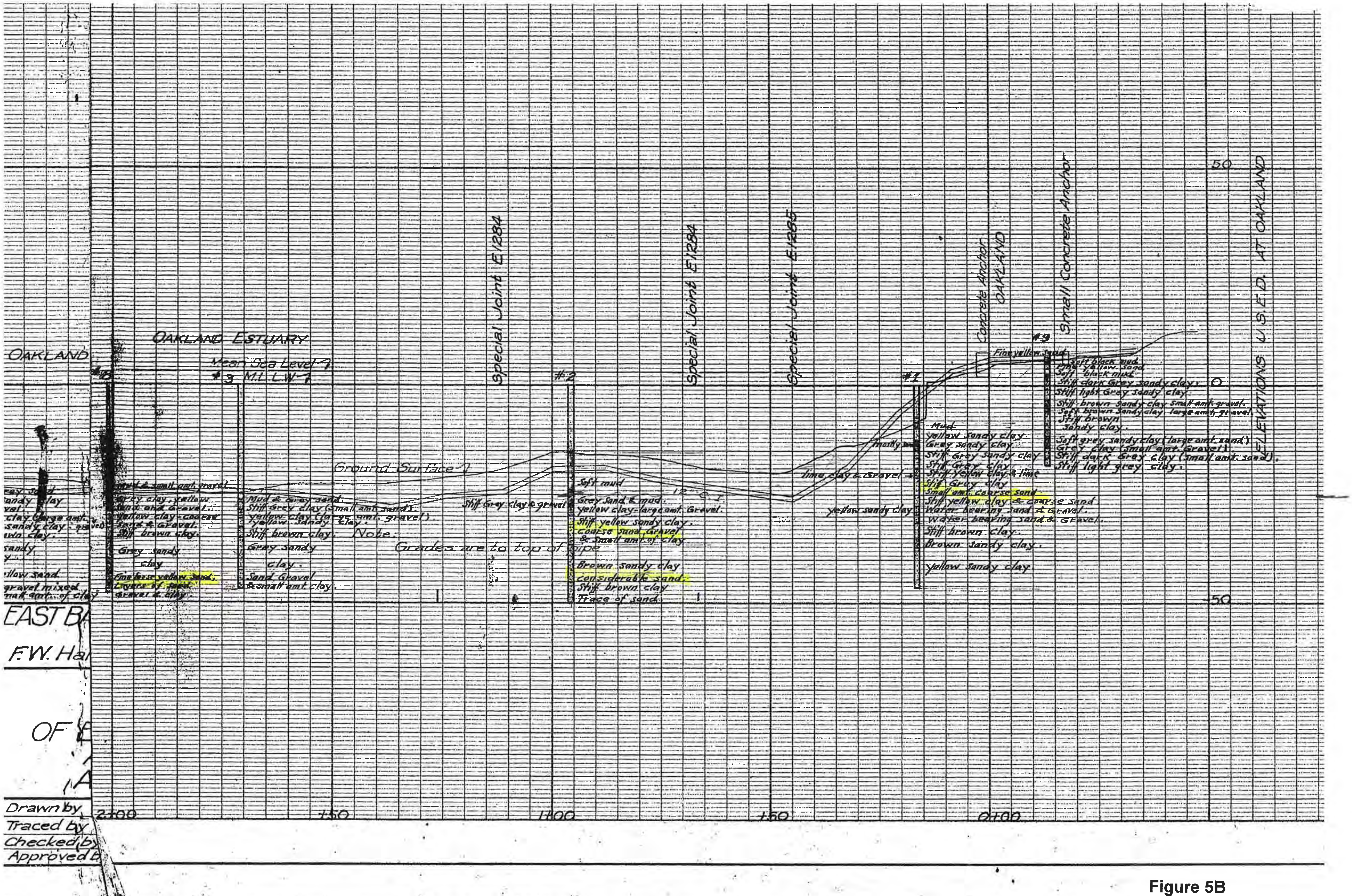


Figure 5B
High General Soil Profile

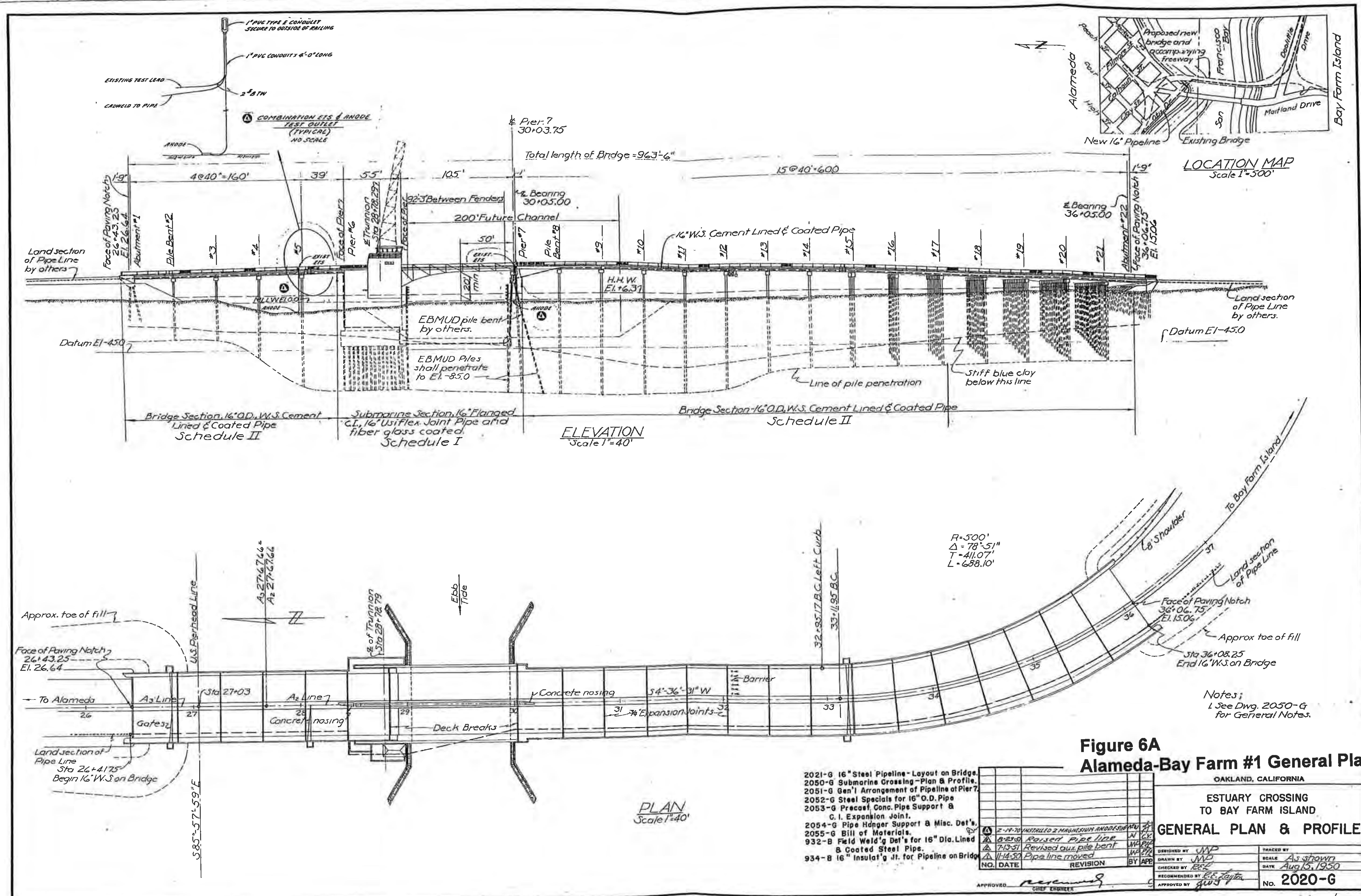
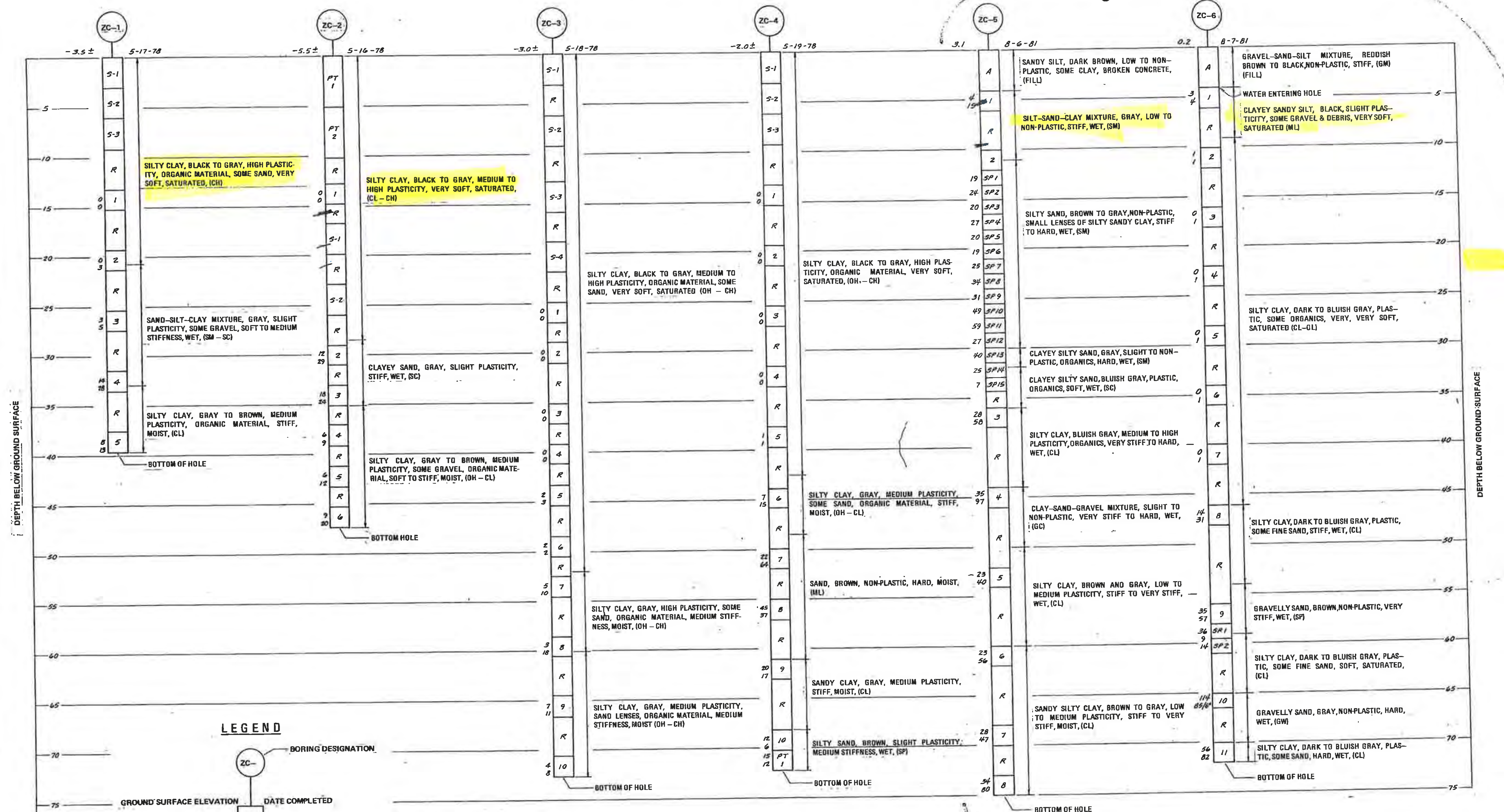
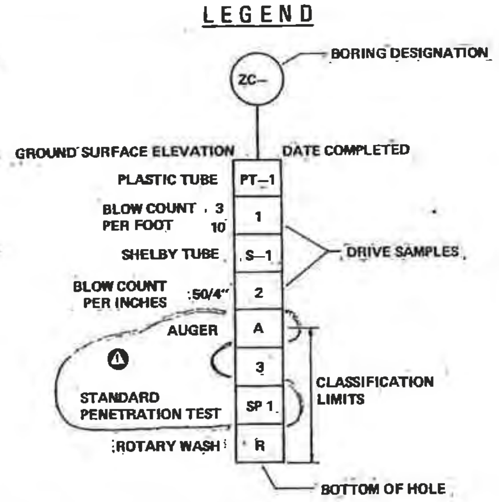


Figure 6A
Alameda-Bay Farm #1 General Plan



LEGEND



NOTES

1. ELEVATIONS ARE TO THE CITY OF ALAMEDA DATUM OF 1929.
2. BORINGS HAVE BEEN DESIGNATED BY A PREFIX ZC AND THE NUMBER OF THE BORING.
3. BORINGS WERE ADVANCED BETWEEN SAMPLES BY ROTARY WASH AND AUGER.
4. SAMPLES WERE OBTAINED BY MEANS OF A MODIFIED CALIFORNIA-TYPE SPLIT-SPOON SAMPLER AND STANDARD PENETRATION TEST SAMPLER DRIVEN BY A 140-LB. HAMMER WITH A 30" DROP, PLASTIC TUBE AND SHELBY TUBE.
5. DRIVE SAMPLES WERE RETAINED IN 2" DIAMETER BY 6" BRASS LINER TUBES, CAPPED AND SEALED, OR CAPPED GLASS JARS.
6. CLASSIFICATIONS ARE ACCORDING TO THE UNIFIED SOIL CLASSIFICATION SYSTEM AS INDICATED ON DWG. 3125-G6. ALL MATERIALS, WHETHER OVERBURDEN OR BEDROCK FORMATION, HAVE BEEN ASSIGNED A BASIC SOIL CLASSIFICATION. GEOLOGIC CLASSIFICATION OR NAMES ARE INDICATED IN PARENTHESES AS ADJECTIVES. CLASSIFICATIONS WERE ASSIGNED MAINLY ON THE BASIS OF VISUAL EXAMINATION AND SUBSTANTIATED WITH A MINIMUM NUMBER OF LABORATORY TESTS.
7. REPRESENTATIVE SAMPLES, AS RECOVERED, MAY BE EXAMINED AT THE DISTRICT'S LABORATORY AT 2500 EAST 29TH STREET, OAKLAND, CALIFORNIA. FIELD LOGS OF ALL BORINGS ARE ON FILE AND A REPORT DESCRIBING ALL PHASES OF THE DISTRICT SOILS AND FOUNDATION INVESTIGATION, INCLUDING LABORATORY TEST RESULTS AND PERTINENT DESIGN CONSIDERATIONS, MAY BE EXAMINED AT THE DISTRICT'S ENGINEERING OFFICE, 2130 ADELINE STREET, OAKLAND, CALIFORNIA.

Figure 7A
Alameda-Bay Farm #2 Detailed Soil Profile

LOG OF BORINGS

NO.	DATE	REVISION	BY	APP.
Δ	8-21-81	ADDED BORINGS ZC-5 & ZC-6		

REVIEW: MECH ELECT STRUCT PIPELINE CORR FNDN
MGR., PROJ. ENG'G. R.P.E. NO. C 8566
DIRECTOR OF ENG'G R.P.E. NO. C 13447
STRUCTURE OR ZONE DESIGNATION
SCALE 1" = 5" (VERT.)
DATE OCT 78

6835 – G – 2

E-160 • 7/77