



MARIN MUNICIPAL WATER DISTRICT

Water Supply Conveyance Improvements Project



TECHNICAL MEMORANDUM 3

Preferred Alternative Selection

DRAFT / November 2024





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Abbreviations

AACE	Association for the Advancement of Cost Engineering
BPS	booster pump station
CAISO	California Independent System Operator
Carollo	Carollo Engineers, Inc.
COTATI	Cotati Tanks to Reservoirs
DIDF	Distribution Investment Deferral Framework
District	Marin Municipal Water District
ESA	Environmental Science Associates
fps	feet per second
HDD	horizontal directional drilling
HGL	hydraulic grade line
ICA	Integration Capacity Analysis
ID	internal diameter
kV	kilovolt
LF	linear feet
MG	million gallons
mgd	million gallons per day
MPEG	Miller Pacific Engineering Group
NMWD	North Marin Water District
PETA	North Marin Aqueduct to Reservoirs
Petaluma	City of Petaluma
PG&E	Pacific Gas and Electric
POI	Point of Interconnection
Project	Water Supply Conveyance Improvements Project
PS A	Pump Station A
PS B	Pump Station B
Psi	pounds per square inch
STAF	Stafford Lake to Reservoirs
STS	South Transmission System
SWSA	Strategic Water Supply Assessment
TM	technical memorandum
UEI	Ulteig Engineers, Inc.
US-101	United States Highway 101
UWMP	Urban Water Management Plan

TM 3 PREFERRED ALTERNATIVE SELECTION

3.1 Purpose

Carollo Engineers, Inc. (Carollo) is assisting the Marin Municipal Water District (District) with the Water Supply Conveyance Improvements Project (Project). The Project builds on the initial alternatives evaluation performed during the Strategic Water Supply Assessment (SWSA) (Jacobs, 2023) to identify and evaluate improvements to the District's water conveyance infrastructure. The goal of the Project is to enhance the reliability, flexibility, and resiliency of the District's water system to improve service to District customers.

Carollo has prepared the following technical memoranda (TMs) as part of this project:

- TM 1 – Project Goals, Criteria, and Alternatives in January 2024 to summarize the project goals, evaluation criteria, and alternatives.
- TM 2 – Screening Criteria and Evaluation Framework in May 2024 to summarize the screening criteria and evaluation framework and screen the number of project alternatives down to three. TM 2 identified three preferred alternatives to be evaluated in additional detail during Phase 2 of the Project.

As a next step, Carollo hosted Workshop No. 3 – Phase 2 Evaluation on July 25, 2024, to summarize the evaluation of the three selected alternatives and the methodology of selecting a preferred alternative. This TM builds upon Workshop No. 3 and summarizes the evaluation of the three selected alternatives to identify a preferred alternative. The preferred alternative will move into preliminary design during Phase 3 of the Project.

Figure 3.1 provides an overview of the Project workflow.

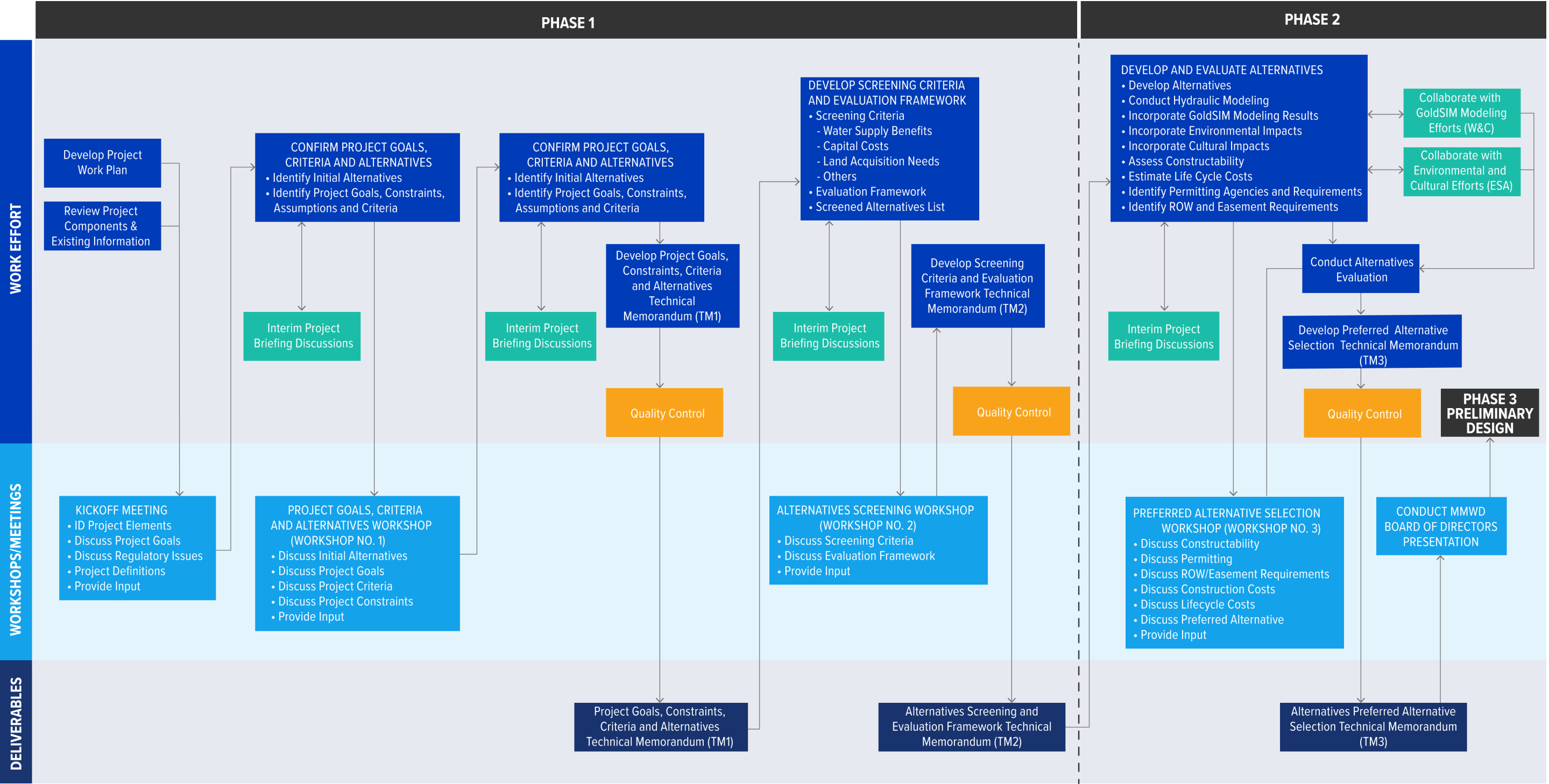


Figure 3.1 Project Workflow

3.2 Background

3.2.1 Background

The District developed the SWSA in 2023 to assess current and future hydrological conditions, performance of the District's system under these conditions, and to develop potential strategies to achieve a more resilient future water supply. The SWSA identified several potential strategies to improve the District's water supply. One of the key strategies identified is to transfer supplemental water from Sonoma Water during the wintertime. The District hired Carollo in 2023 to further evaluate the water supply alternatives developed during the SWSA as part of this Project, which evaluates alternatives that convey water from Sonoma Water during the wintertime.

3.2.2 Technical Memorandum 1 – Project Goals, Criteria and Alternatives

In January 2024, Carollo prepared TM 1 – Project Goals, Criteria, and Alternatives. TM 1 identified the Project goals and criteria to be used to evaluate project alternatives. Additionally, TM 1 identified 13 project alternatives that increase the system's capacity to transfer supplemental water from Sonoma Water during the wintertime, including:

- **South Transmission System (STS)-1:** Construct a new STS pipeline along the United States Highway 101 (US-101) corridor to convey additional water from Cotati Tanks to Reservoirs (COTATI) Tanks to the Kastania Pump Station.
- **STS-2:** Construct a new STS pipeline through the downtown area of the City of Petaluma (Petaluma) to convey additional water from Cotati Tanks to the Kastania Pump Station.
- **STS-3:** Construct a new STS pipeline through rural roads in Petaluma to convey additional water from Cotati Tanks to the Kastania Pump Station.
- **Stafford Lake to Reservoirs (STAF)-1:** Convey water from the North Marin Aqueduct to Nicasio Reservoir using existing North Marin Water District (NMWD) infrastructure, convey water from the North Marin Aqueduct to Stafford Lake, and construct new facilities to convey water from Stafford Lake to Nicasio Reservoir.
- **STAF-2:** Convey water from the North Marin Aqueduct to Soulajule Reservoir using existing NMWD infrastructure, convey water from the North Marin Aqueduct to Stafford Lake, and construct new facilities to convey water from Stafford Lake to Soulajule Reservoir.
- **STAF-3:** Convey water from the North Marin Aqueduct to Nicasio and Soulajule Reservoirs using existing NMWD infrastructure, convey water from the North Marin Aqueduct to Stafford Lake, and construct new facilities to convey water from Stafford Lake to Nicasio and Soulajule Reservoirs.
- **North Marin Aqueduct to Reservoirs (PETA)-1:** Construct new facilities to convey water from North Marin Aqueduct to Nicasio Reservoir.
- **PETA-2:** Construct new facilities to convey water from North Marin Aqueduct to Soulajule Reservoir.
- **PETA-3:** Construct new facilities to convey water from North Marin Aqueduct to Nicasio and Soulajule Reservoirs.
- **PETA-4:** Construct new facilities along San Antonio Road to convey water from North Marin Aqueduct to Nicasio and Soulajule Reservoirs.

- **Cotati Tanks to Reservoirs (COTATI)-1:** Construct new facilities to convey water from Cotati Tanks to Nicasio Reservoir.
- **COTATI-2:** Construct new facilities to convey water from Cotati Tanks to Soulajule Reservoir.
- **COTATI-3:** Construct new facilities to convey water from Cotati Tanks to Nicasio and Soulajule Reservoirs.

3.2.3 Technical Memorandum 2 – Screening Criteria and Evaluation Framework

In May 2024, Carollo prepared TM 2 – Screening Criteria and Evaluation Framework. TM 2 further refined the evaluation criteria to be used to compare the thirteen (13) alternatives in the TM. The final evaluation criteria included:

Water Reliability and Sustainability

- Contribute to a sustainable dry year water supply, consistent with the framework in the SWSA:
 - » In order to constitute contributing to a sustainable dry year water supply, an alternative should be able to increase the system's capacity to take supplemental water from Sonoma Water during the wintertime (October to May) by an amount to be determined by the District.
- Provide reliable water supply, treatment, and transmission infrastructure that meets long-term operational needs under average-day and maximum-day demand conditions.
- Improve the ability of the water system to withstand natural hazards.

Flexibility and Resiliency

- Retain and/or improve flexibility of District operations.
- Improve the resilience of District operations from changing conditions, including climate change effects.
- Integrate well with regional water systems.
- Minimize complexities of system operations and hydraulics.

Schedule and Implementation

- Implement the project in a timely manner, consistent with District goals.
- Reduce regulatory complexity (e.g., minimize number of permits required, number of agencies with involvement, etc.).
- Preserve options for future conveyance systems between Cotati Tanks and the District's facilities.
- Minimize construction complexities (e.g., alignment minimizes trenchless installation of pipe, avoids crowded utility corridors, etc.).

Water Quality

- Design improvements to meet current and foreseeable future drinking water regulations with a margin of safety.

Environmental and Social Stewardship

- Minimize long-term environmental impacts relative to other conveyance alternatives.
- Minimize disruption to existing land uses to the extent practical.

Economic and Financial

- Minimize capital costs for the District's customers relative to yield.
- Minimize life cycle costs for the District's customers relative to other conveyance alternatives.
- Integrates well with existing infrastructure.
- Leverage federal and state investments in infrastructure.
- Minimize financial risk.

TM 2 also further refined each of the alternatives. Refinements included updates to anticipated water system supply benefits and anticipated hydraulic impacts to the existing conveyance system between the Cotati Tanks and Ignacio Pump Station. Finally, TM 2 outlined the evaluation methodology developed by the District and Carollo to compare the alternatives identified in TM 1. Figure 3.2 illustrates the selected evaluation methodology.

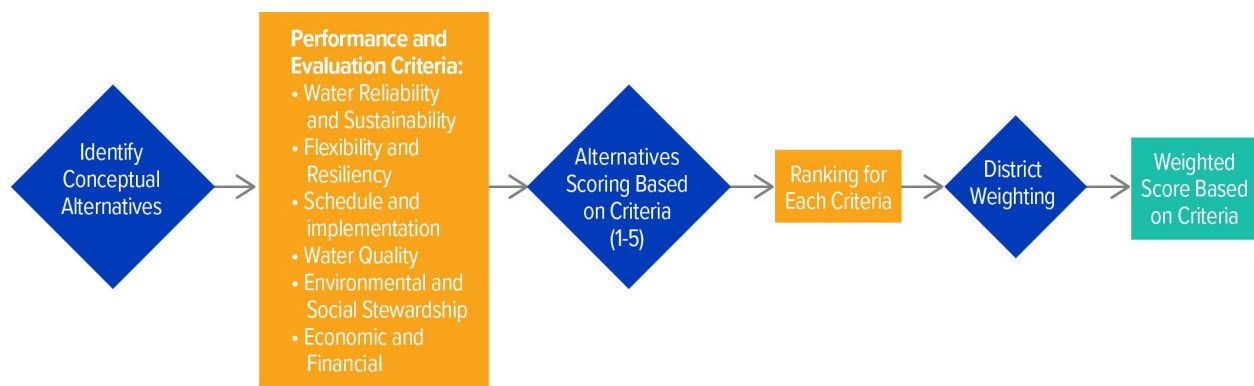


Figure 3.2 Alternatives Screening Methodology

As shown in Figure 3.2, after identifying the conceptual alternatives and evaluation criteria, each alternative was given a score of 1, 3, or 5 for each of the criteria. The higher score indicates the alternative ranks better than lower scores (a score of 5 is better than a 1). After scoring each criterion, the criteria were weighted by the project team based on relative importance on a scale from 1 to 5. If the District considered some of the criteria to be of equal weight, the criteria were weighted equally in the matrix. The final step of the evaluation was to summarize the results in an Alternative Evaluation Matrix. The three alternatives with the highest total scores were advanced into Phase 2 of the Project for further evaluation. Based on the total scores included in Table 3.1, the PETA-3, PETA-4, and COTATI-3 alternatives were selected to be advanced into Phase 2 of the Project.

Table 3.1 Alternatives Evaluation Matrix

Criteria	Water Reliability and Sustainability			Flexibility and Resiliency				Schedule and Implementation				Water Quality	Environmental and Social Stewardship		Economic and Financial					Total Score
	Contribute to a sustainable dry year water supply	Provide reliable water supply, treatment and transmission infrastructure that meets long-term operational needs under average- and maximum-day demand conditions	Improve the ability of water system to withstand natural hazards	Retain and/or improve flexibility of District operations	Improve the resilience of District operations from changing conditions, including climate change effects	Integrate well with regional water systems	Minimize complexities of system operations and hydraulics	Implement the project in a timely manner, consistent with framework identified in the SWSA	Reduce regulatory complexity	Preserve options for future conveyance systems between Cotati Tanks and the District's facilities	Minimize construction complexities	Design improvements to meet current and foreseeable future drinking water regulations with a margin of safety	Minimize long-term environmental impacts relative to other conveyance alternatives	Minimize disruption to existing land uses to the extent practical	Minimize capital costs for the District's customers relative to yield	Minimize life cycle costs for the District's customers relative to other conveyance alternatives	Integrates well with existing infrastructure	Leverage federal and state investments in infrastructure	Minimize financial risk	
Criteria Weighting	5	3	3	3	3	4	4	3	3	3	1	2	2	2	4	4	4	5	2	
STS-1	1	3	1	1	3	3	1	1	3	1	5	5	5	1	1	5	1	3	3	136
STS-2	1	3	1	1	3	3	1	1	3	1	5	5	5	1	1	5	1	3	3	136
STS-3	1	3	1	1	3	3	3	3	3	3	5	5	5	3	1	5	1	3	3	160
STAF-1	1	1	1	3	1	1	3	3	1	1	5	5	3	3	3	5	5	3	3	154
STAF-2	1	1	1	3	1	1	3	3	1	1	5	5	3	3	3	5	5	3	3	154
STAF-3	1	1	3	5	1	1	3	3	3	1	5	5	3	3	3	5	5	3	3	172
PETA-1	3	3	3	3	3	1	3	3	1	3	5	5	3	3	3	3	3	3	3	172
PETA-2	3	3	3	3	3	1	3	3	1	3	5	5	3	3	3	3	3	3	3	172
PETA-3	3	3	5	5	3	5	3	5	3	3	5	5	3	3	3	3	5	3	3	220
PETA-4	5	5	5	5	5	5	1	5	3	3	5	5	3	3	3	1	5	3	3	226
COTATI-1	5	5	5	3	3	3	1	3	3	3	5	3	1	3	1	1	1	3	3	168
COTATI-2	5	5	5	3	3	3	1	3	3	3	5	3	1	3	1	1	1	3	3	168
COTATI-3	5	5	5	5	3	3	1	5	5	3	5	3	1	3	1	1	1	3	3	186

3.3 Hydraulic Analysis of Existing System

One of the key evaluation criteria identified in TM 1 and TM 2 is an alternative's ability to contribute to a sustainable dry year water supply. Of the alternatives evaluated in TM 1 and TM 2, several interact with the existing conveyance system between the Cotati Tanks and Ignacio Pump Station, including the STAF and PETA alternatives. For these alternatives, a hydraulic analysis of the existing system was performed to determine the alternative's ability to contribute to a sustainable dry year water supply. The hydraulic evaluation considered impacts to the existing system due to transferring water from Sonoma Water, through the Petaluma and North Marin Aqueduct, to a new turnout location. A summary of the analysis performed is included below.

3.3.1 Petaluma/North Marin Aqueduct System Hydraulics

The Petaluma/North Marin Aqueduct System conveys treated surface water from the Cotati Tanks to Petaluma, NMWD, and the District. The Petaluma Aqueduct section is upstream of Kastania Tank and the North Marin Aqueduct section is downstream of Kastania Tank and Pump Station. Figure 3.3 provides an overview of the facilities used to convey water from Cotati Tanks to Ignacio Pump Station. Figure 3.4 shows the hydraulic profile of the aqueduct system.

The Ely Booster Pump Station (BPS) is operated to maintain a level in Kastania Tank between 34 and 38 feet. Under low demand conditions, Ely BPS does not need to turn on since Kastania Tank is at a lower hydraulic grade line (HGL) than the Cotati Tanks. However, when demands increase, the Ely BPS must turn on to overcome headloss through the Petaluma Aqueduct and maintain Kastania Tank levels.

Kastania Tank acts as a pressure break between the Petaluma and North Marin sections of the aqueduct. Hydraulic conditions downstream of Kastania Pump Station are controlled by the level in Kastania Tank and Kastania Pump Station settings. The control system turns pumps on and off at the Kastania Pump Station as needed to maintain minimum pressures at the downstream turnouts. Additional information on the existing Petaluma/North Marin Aqueduct system hydraulics is included in the Kastania Pump Station Rehabilitation Project Basis of Design Report (Carollo, 2021).

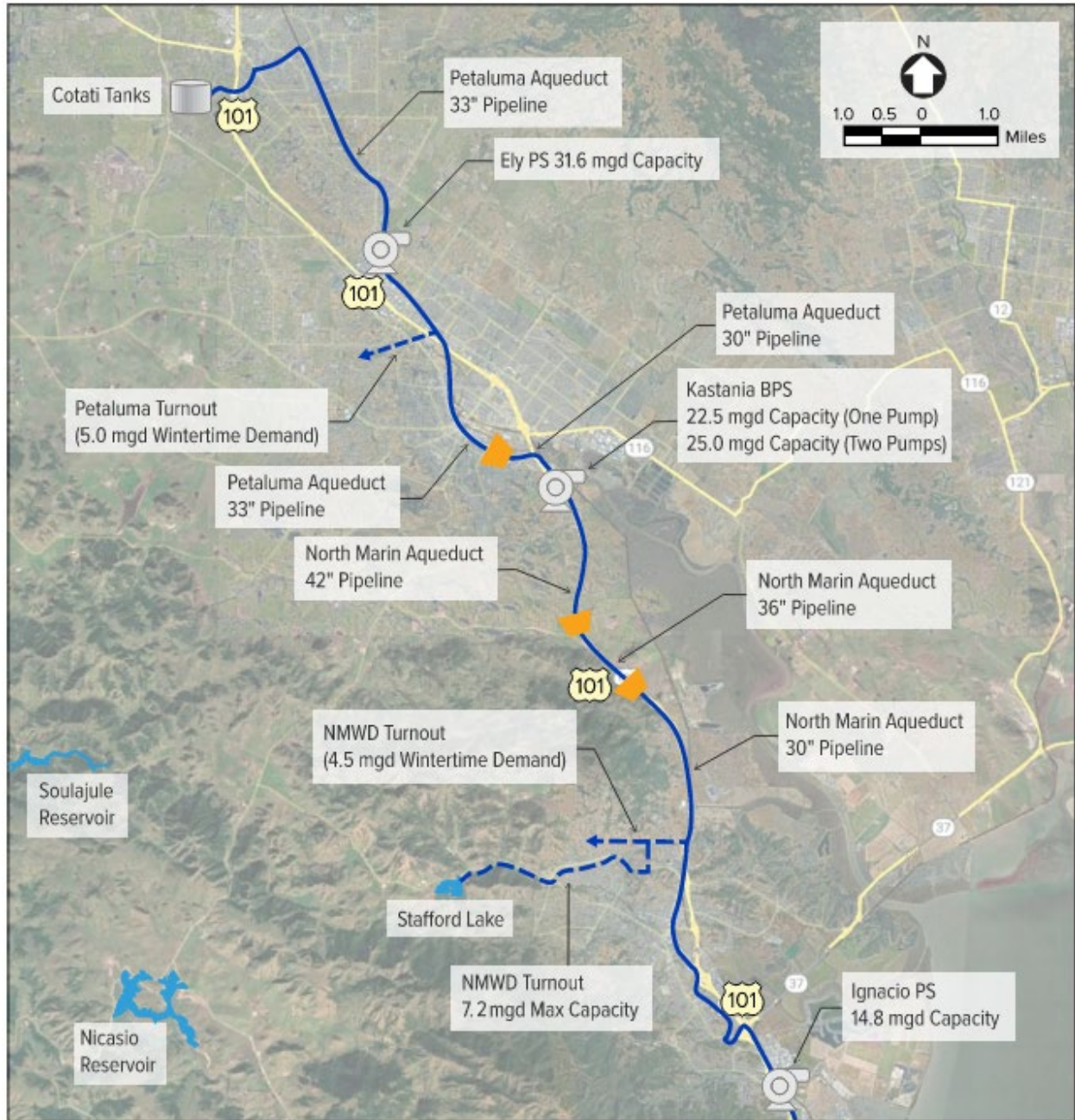


Figure 3.3 Existing Facilities Between Cotati Tanks and Ignacio Pump Station

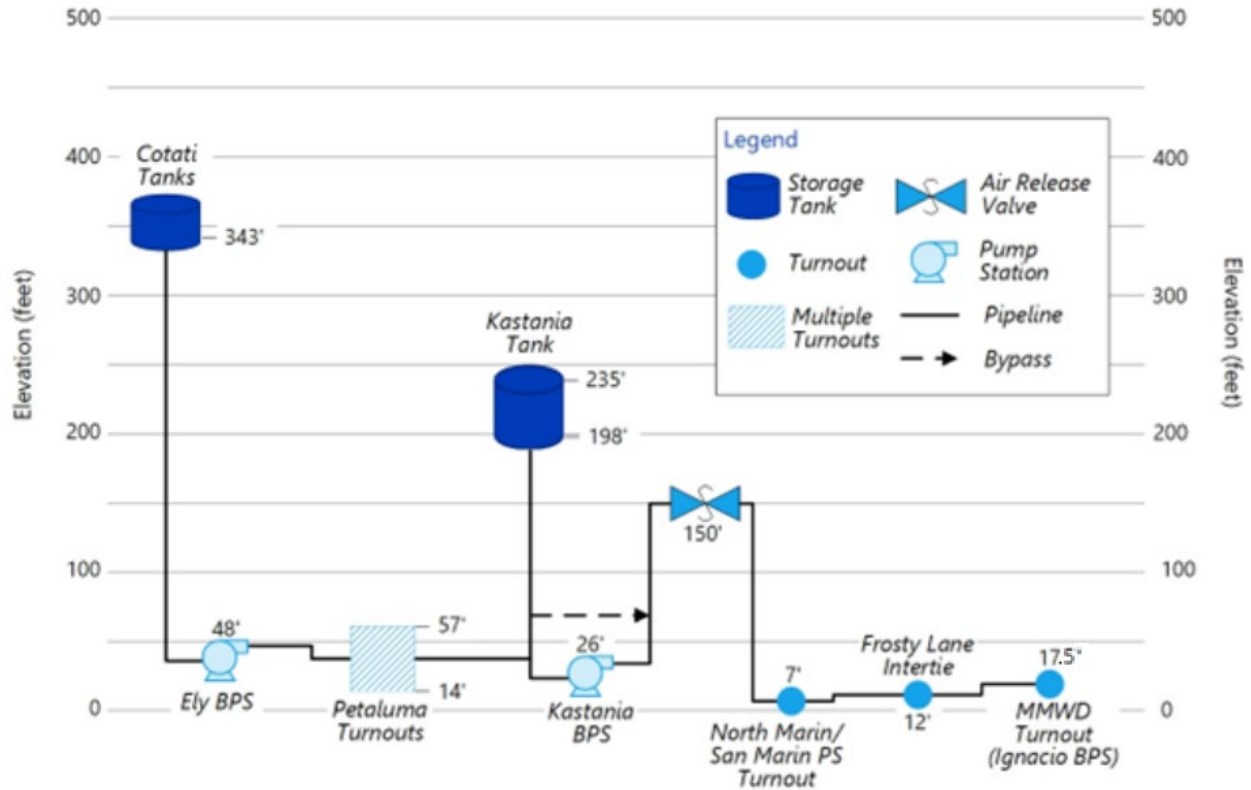


Figure 3.4 Petaluma/North Marin Aqueduct System Hydraulic Profile

3.3.2 Hydraulic Model Assumptions and Scenarios

Carollo utilized the Petaluma/North Marin Aqueduct System hydraulic model developed during the Kastania Pump Station Rehabilitation Project (Carollo, 2021) to evaluate the hydraulics of the water supply conveyance alternatives. The hydraulic model was developed in Innovyze Inc.'s InfoWater Pro software and was validated during the Kastania Pump Station Rehabilitation Project using supervisory control and data acquisition data from 2020.

The InfoWater Pro model consists of the following components:

- **Cotati Tanks:** The Cotati Tanks are represented in the model using a fixed-head reservoir at an elevation of 343 feet, which is the elevation corresponding with the lowest allowable operating level in the Cotati Tanks. The model assumes unlimited supply from Cotati Tanks to the downstream system.
- **Kastania Tank:** The Kastania Tank is modeled as a cylindrical tank with a diameter of 226 feet, a base elevation of 195 feet, and an operating range of 34 to 38 feet.

- **Ely BPS:** The Ely BPS is modeled with two pumps and a gravity bypass line. Figure 3.5 shows the modeled pump curve for both pumps. The BPS is operated according to levels in Kastania Tank (i.e., the pump station turns on when the tank level falls to 34 feet and turns off once the tank level reaches 38 feet).
- **Kastania Pump Station:** The Kastania Pump Station is modeled with two pumps and a bypass gravity line. Figure 3.5 shows the modeled pump curves for both pumps. The pump station is operated according to downstream demand conditions such that the downstream hydraulic grade does not fall below the minimum required head at each turnout. For this study, hydraulic results were generated with the pump station off, one pump on, and two pumps on under each demand condition to evaluate downstream hydraulics.
- **Turnouts:** Turnouts along the aqueduct are modeled as junctions with fixed demands. The following turnouts were considered in this analysis:
 - » **Petaluma Turnouts:** Petaluma pulls from multiple locations along the aqueduct between the Ely BPS and Kastania Pump Station with elevations ranging from 14 to 57 feet. For this study, all Petaluma demands were applied at the intersection of Petaluma Boulevard and US-101, which coincides with the southern edge of the Petaluma boundary.
 - » **North Marin/San Marin Pump Station Turnouts:** NMWD pulls from multiple locations along the aqueduct between the Kastania Pump Station and Ignacio BPS, including at the intersection of Redwood Boulevard and Atherton Avenue, which is at an elevation of 7 feet. The turnout at Redwood Boulevard and Atherton Avenue services two zones—Zone 1 and Zone 2. Zone 2, which runs through San Marin Pump Station, is used to convey water to Stafford Lake.
 - » **Ignacio BPS:** The Ignacio BPS, which is at the most downstream point of the aqueduct system, sits at an elevation of 17.5 feet.

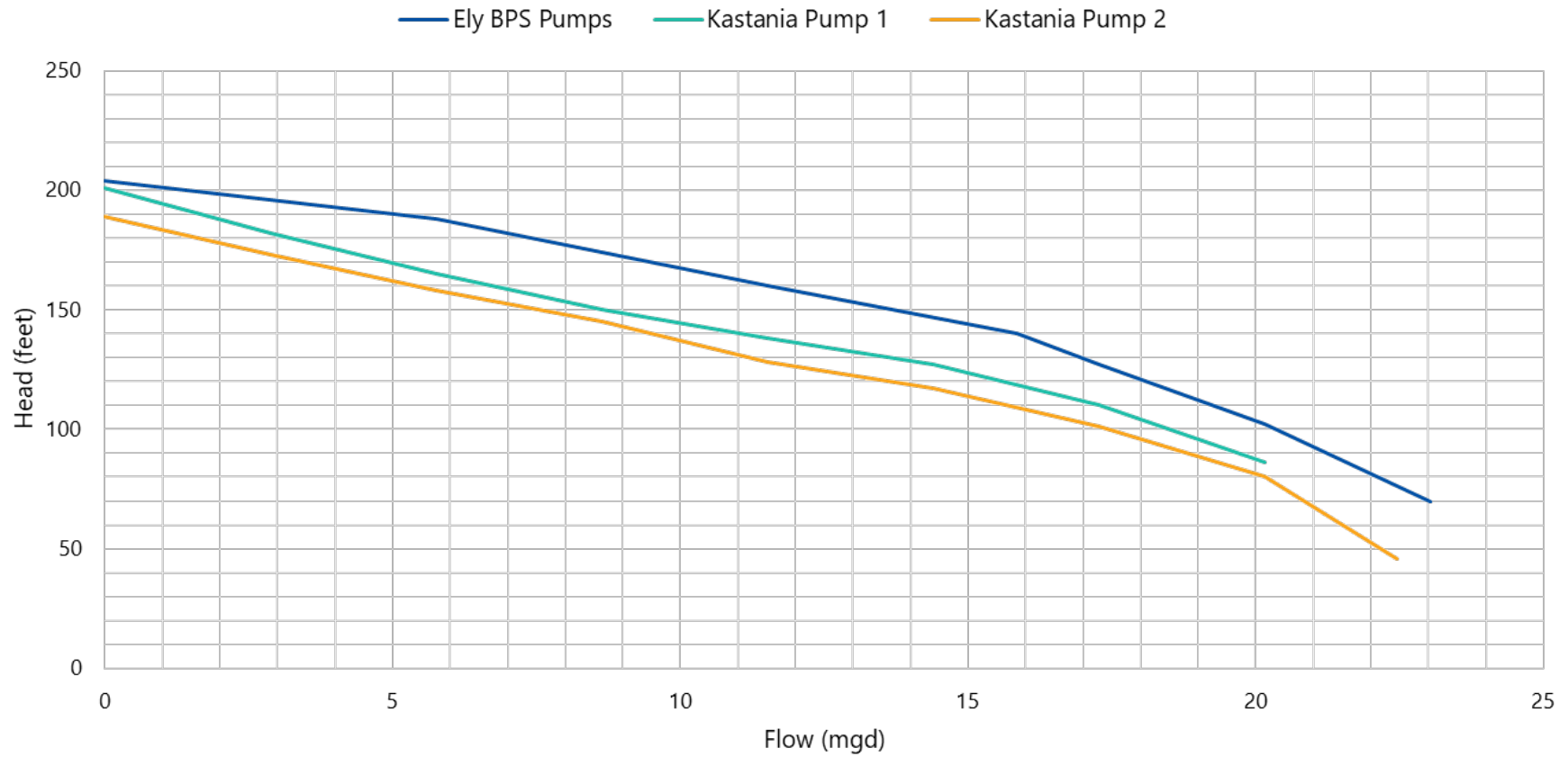


Figure 3.5 Modeled Pump Curves for Ely and Kastania Pump Stations

The hydraulic model was used to evaluate the Petaluma/North Marin Aqueduct System hydraulics with the STAF-1, STAF-2, and STAF-3 alternatives and PETA-1, PETA-2, PETA-3, and PETA-4 alternatives. Two turnout locations were considered that are downstream of Kastania Pump Station. Table 3.2 describes the primary demand and elevation assumptions for each turnout. Figure 3.6 shows the locations of the two turnouts.

Table 3.2 New Turnout Assumptions

Turnout	Location	Associated Project Alternatives	Elevation (feet)	Low Demand (mgd)	High Demand (mgd)
1	At the intersection of Redwood Boulevard and Atherton Avenue.	STAF-1, 2, and 3	7	0.0	7.2
		PETA-1, 2, and 3	7	10.4	30.0
2	On Petaluma Boulevard approximately 950 feet south of Skinner Road, near San Antonio Road.	PETA-4	33	12.5	30.0

Notes:

mgd - million gallons per day



Figure 3.6 Petaluma/North Marin Aqueduct Turnout Alternatives

Additionally, key hydraulic assumptions were made for the existing turnouts along the Petaluma/North Marin Aqueduct System based on discussions with District staff. Table 3.3 lists the key assumptions.

Table 3.3 Assumptions for Existing Turnouts

Turnout	Location	Elevation (feet)	Existing Winter Demand (mgd) ^(1, 3)	Future Winter Demand (mgd) ^(2, 3)
Petaluma	Multiple locations between Ely BPS and Kastania BPS. For this analysis, all demands were applied at the intersection of Petaluma Boulevard and US-101, which coincides with the southern edge of the Petaluma boundary.	14 to 57	5.0	6.5
North Marin/ San Marin Pump Station	The intersection of Redwood Boulevard and Atherton Avenue.	7	4.5	6.1
Ignacio BPS	North of Bel Marin Keys Boulevard approximately 1,000 feet east of US-101.	17.5	8.0	8.0

Notes:

- (1) Existing winter demands were determined in consultation with District staff after reviewing recent water delivery data from Sonoma County.
- (2) Future winter demands are consistent with 2045 demands included in Sonoma County's Urban Water Management Plan (UWMP).
- (3) Assumed demands at turnouts are approximate only.

Several scenarios were developed to evaluate the Petaluma/North Marin Aqueduct System hydraulics under existing and future demand conditions with the supply conveyance alternatives. The following lists the key assumptions related to the hydraulic analysis:

- All scenarios were run under steady state conditions (i.e., diurnal variations in demands were not considered).
- Pipes were modeled using nominal diameters. All existing pipes were assumed to have a Hazen Williams roughness coefficient of 135. All new pipes were assumed to have a Hazen Williams roughness coefficient of 120. A lower C factor was assumed to account for any minor losses.
- It was assumed that all demands would occur at Nicasio Reservoir. This is the most conservative scenario.
- The Kastania Tank initial level was set to the tank's low operating level of 34 feet. It was assumed that the upstream aqueduct system would be upgraded to enable the tank to cycle between 34 and 38 feet for selected alternatives.
- The minimum HGL at the NMWD turnout was assumed to be 170 feet; this requirement was determined during the Kastania Pump Station Rehabilitation Project and was confirmed as part of this study.
- The minimum HGLs at turnouts other than the NMWD turnout were assumed to be the HGL required to maintain 50 pounds per square inch (psi) pressure (i.e., turnout elevation plus 50 psi).

Further discussion of the system hydraulics for the PETA-3 and PETA-4 alternatives is included in TM 2 and Sections 3.5.3 and 3.6.3.

3.4 Phase 2 Evaluation Introduction

The following sections evaluate PETA-3, PETA-4, and COTATI-3 in more detail. Additionally, this TM outlines the evaluation framework used to compare the three selected alternatives and ultimately selects a preferred alternative to move into preliminary design. This evaluation is the final step towards identifying the preferred alternative to transfer supplemental water from Sonoma Water during the wintertime to the District's facilities.

3.5 Evaluation of PETA-3: North Marin Aqueduct to Nicasio and Soulajule Reservoirs

3.5.1 Introduction

The PETA-3: North Marin Aqueduct to Nicasio and Soulajule Reservoirs alternative includes a new pump station(s) and approximately 16.2 miles of pipeline to convey water from the North Marin Aqueduct to Nicasio and Soulajule Reservoirs.

This alternative requires approximately 9.2 miles of new pipeline routed along San Marin Drive and Novato Boulevard. At the intersection of Novato Boulevard and Pt. Reyes – Petaluma Road, the pipe will bifurcate with one branch heading south approximately 4.0 miles along Pt. Reyes – Petaluma Road to Nicasio Reservoir and the other branch heading northwest approximately 3.0 miles along Pt. Reyes – Petaluma Road, Hicks Valley Road, and a fire road to Soulajule Reservoir. Figure 3.7 shows the proposed pipeline alignment and profile. Refer to Appendix F for more detailed plan view drawings of the proposed pipeline alignment.

For this alternative, the additional water supply will be conveyed to Nicasio and Soulajule Reservoirs for storage. The District will have the ability to select which reservoir to fill based on reservoir levels, demands within the system, and operational considerations. By utilizing both reservoirs, the District can increase the amount of water transferred from Sonoma Water.

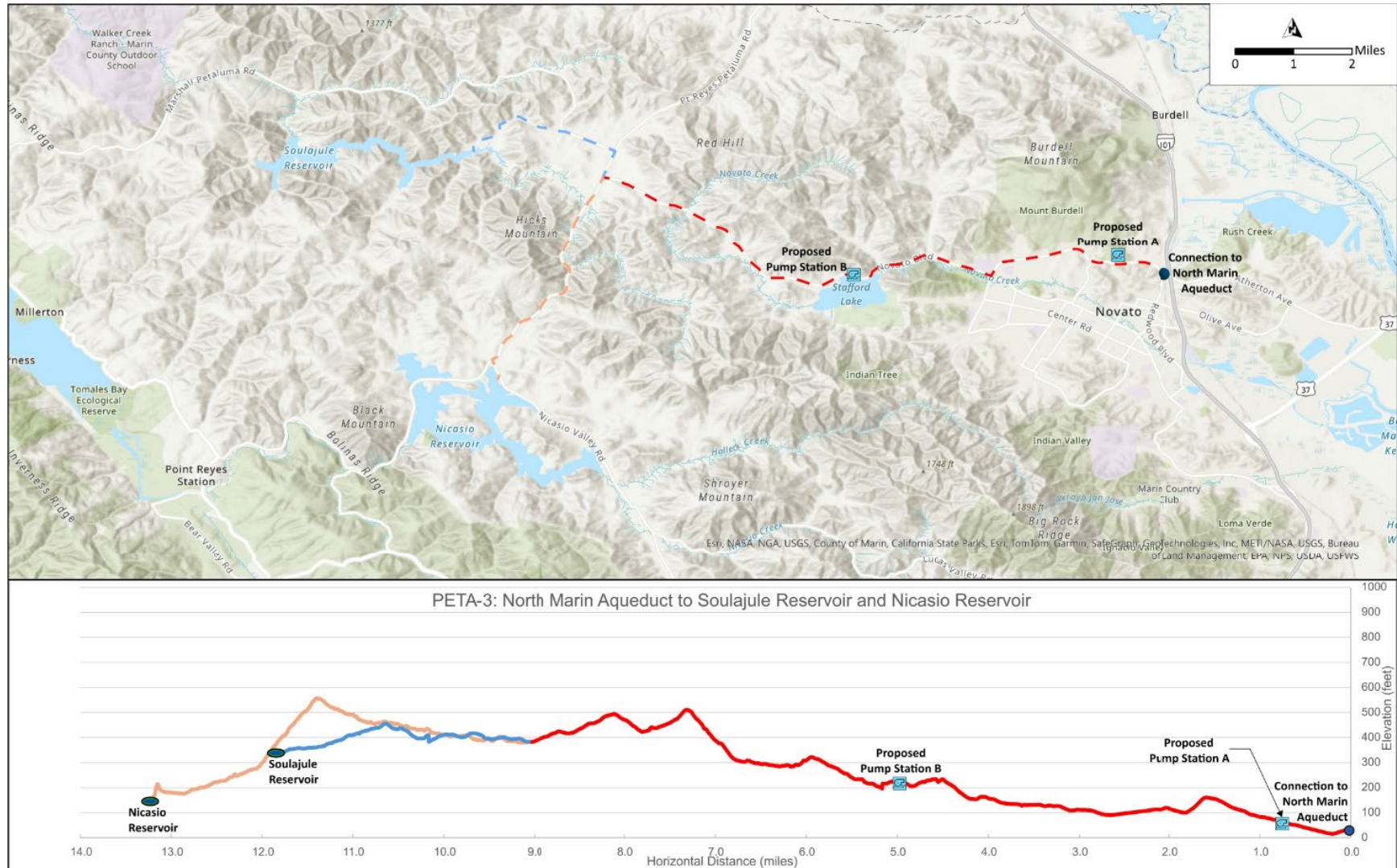


Figure 3.7 PETA-3 Alignment Overview

3.5.2 Water System Supply Benefits

3.5.2.1 PETA-3 Without Upstream Improvements

The sizing of the facilities for PETA-3 is highly dependent upon the capacity of existing infrastructure upstream of the proposed turnout location and upstream and downstream demands/diversions along the North Marin Aqueduct. Based on current operations, the existing infrastructure can convey up to 30.4 mgd of water from Cotati Tanks to Kastania Pump Station, with Petaluma diverting approximately 5.0 mgd of that during wintertime months. Therefore, up to 25.4 mgd of water can be delivered to Kastania Pump Station during wintertime months. Kastania Pump Station has a maximum capacity of 25.0 mgd.

The proposed turnout location for PETA-3 is in Novato downstream of Kastania Pump Station adjacent to the existing NMWD pipeline that conveys water from the North Marin Aqueduct to Stafford Lake. At the proposed turnout location, the existing North Marin Aqueduct has a nominal diameter of 30 inches and an internal diameter (ID) of 28.5 inches. Given the maximum allowable velocity in the aqueduct is 8.0 feet per second (fps), the existing North Marin Aqueduct has a maximum capacity of 22.9 mgd at this location. Additionally, downstream of Kastania Pump Station, the system demand during wintertime months is approximately 12.5 mgd. As previously discussed, NMWD would divert approximately 4.5 mgd during wintertime months and the District would receive and distribute approximately 8.0 mgd through Ignacio Pump Station.

The largest bottleneck in the system upstream of the proposed turnout location is the approximately 2.6-mile-long segment of 28.5-inch ID pipe in the North Marin Aqueduct. Assuming a maximum capacity of 22.9 mgd of water can be conveyed through this segment of the North Marin Aqueduct, and assuming there is a demand of 12.5 mgd during wintertime operations downstream of this segment of pipe, approximately 10.4 mgd of water is anticipated to be available to be conveyed from the North Marin Aqueduct to the reservoirs during the wintertime months.

Figure 3.8 shows a schematic of the existing system from Cotati Tanks to Ignacio Pump Station. The schematic is diagrammatic and does not show all existing or proposed facilities. The proposed turnout location is shown in green and the largest bottleneck in the existing system is shown in red. No other improvements or upgrades to the existing system are anticipated to be needed to convey 10.4 mgd from the North Marin Aqueduct to the reservoirs.

For this alternative, it is understood that actual demands for other customers may vary from the assumed demands listed above. As such, we recommend that facilities be oversized from the North Marin Aqueduct to the reservoirs to convey additional water that might be available if customers along the North Marin Aqueduct don't take their full current allotment. Final sizing of facilities should be determined with District input during preliminary design. However, for planning purposes, it was assumed that facilities should be expected to convey up to 15.0 mgd during high demand scenarios.

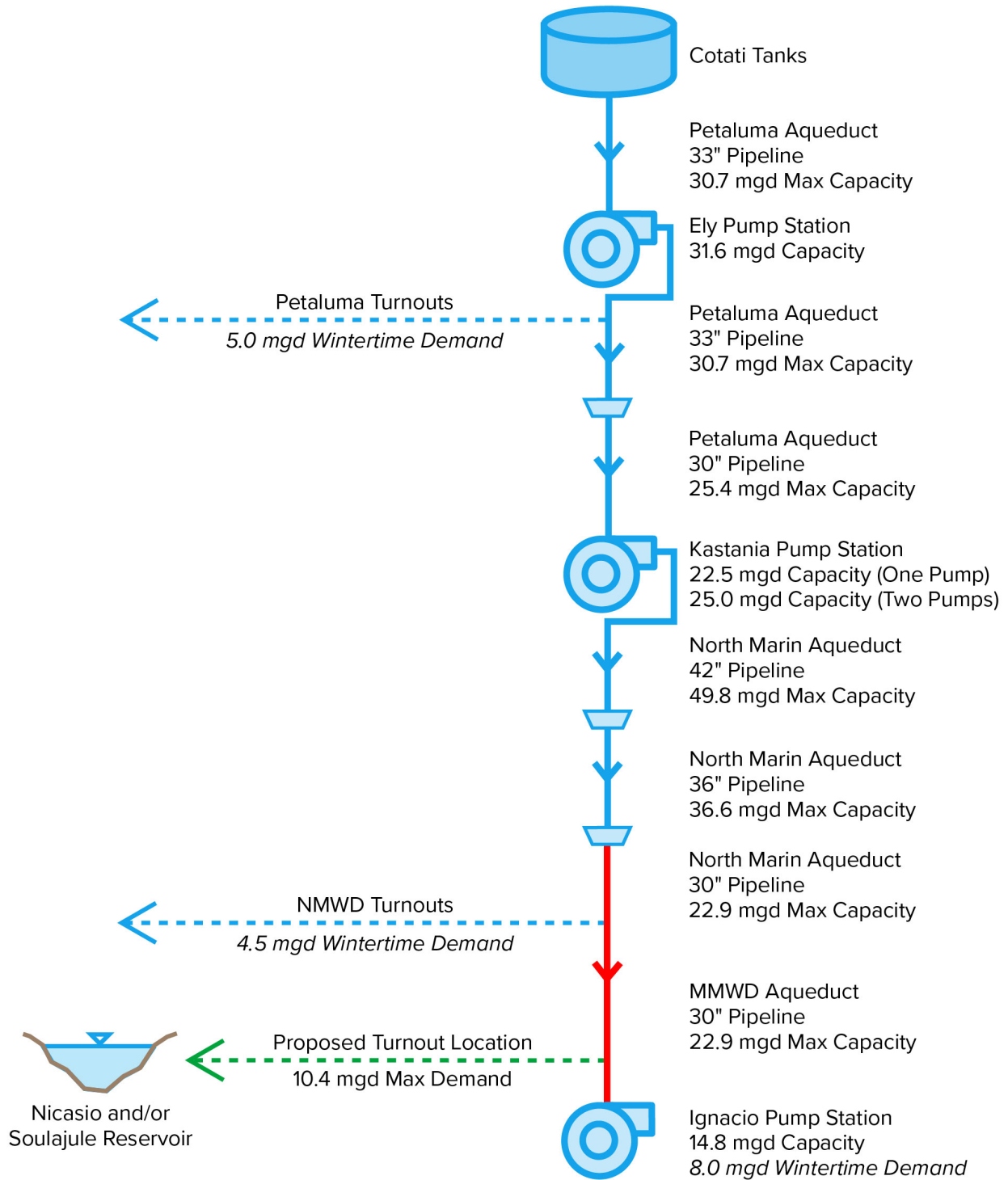


Figure 3.8 PETA-3 Without Upstream Improvements Schematic

3.5.2.2 PETA-3 with Upstream Improvements

It is possible to increase the water supply benefit of the PETA-3 alternative by implementing improvements to address bottlenecks upstream of the proposed turnout location. The improvements were selected that maximize the use of existing infrastructure and provide the most water supply benefit. The required improvements to the existing system that allow up to 30.0 mgd of water to be conveyed from the North Marin Aqueduct to the reservoirs include:

- Increasing the size of sections of the existing 30-inch and 36-inch pipeline upstream of the proposed turnout location or constructing a separate transmission main parallel to the existing 30-inch and 36-inch pipeline:
 - » As stated above, there are existing segments of 30-inch and 36-inch pipeline between Kastania Pump Station and the proposed turnout location. This includes approximately 2.6 miles of 28.5-inch ID (inside diameter) pipe, 0.4 miles of 30.0-inch ID pipe, and 0.4 miles of 36.0-inch ID pipe.
 - » The existing 28.5-inch ID pipe can convey up to 22.9 mgd of water.
 - » With the addition of a parallel 30-inch pipeline along the segments of 30-inch pipeline and 36-inch pipeline, up to 45.0 mgd of water could be delivered to the proposed turnout location from Kastania Pump Station during wintertime operations.
- Increasing the capacity of Kastania Pump Station from 25.0 mgd to 42.5 mgd.
- Constructing an STS alternative that can convey an additional 17.1 mgd from Cotati Tanks to Kastania Pump Station:
 - » The existing infrastructure can convey up to 30.4 mgd of water from Cotati Tanks to Kastania Pump Station:
 - Petaluma pulls approximately 5.0 mgd of that during wintertime operations. As such, approximately 25.4 mgd of water can currently be delivered to Kastania Pump Station via the existing Petaluma Aqueduct and Ely Pump Station during wintertime months.
 - With the addition of facilities conveying 17.1 mgd from Cotati Tanks to Kastania Pump Station, up to 42.5 mgd of water could be delivered to Kastania Pump Station during wintertime operations:
 - Improvements to the existing Petaluma Aqueduct or Ely BPS are not required.
 - » The STS alternative required to convey an additional 17.1 mgd from Cotati Tanks to Kastania Pump Station will likely need to include approx. 10 miles of new 30-inch pipeline and a booster pump station. Refer to TM1 and TM2 for further detail on potential STS alternatives.

Figure 3.9 shows a schematic of the existing system from Cotati Tanks to Ignacio Pump Station. The schematic is diagrammatic and does not show all existing or proposed facilities. The proposed turnout location is shown in green and the largest bottlenecks in the existing system are shown in red. If the improvements detailed above are implemented with the Project, the maximum capacity of the system upstream of the turnout location will be increased to 42.5 mgd, which would allow the District to convey up to 30.0 mgd from the North Marin Aqueduct to the reservoirs during wintertime operations when downstream demands are 12.5 mgd.

For this alternative, it is understood that actual demands for other customers may vary from the assumed demands listed above. As such, we recommend that facilities be oversized from the North Marin Aqueduct to the reservoirs to convey additional water that might be available if customers along the North Marin Aqueduct don't take their full current allotment. Final sizing of facilities should be determined with District input during preliminary design.

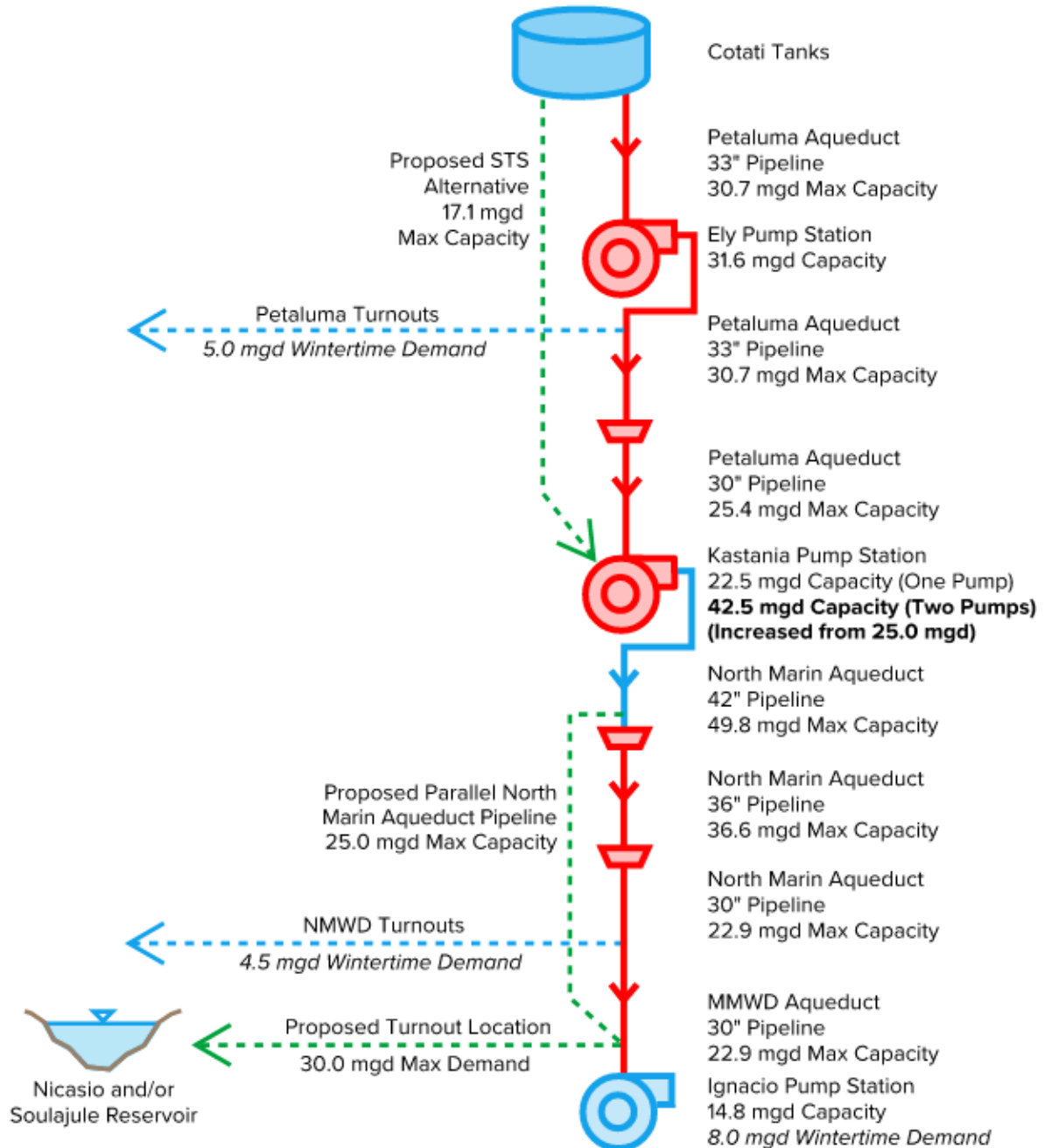


Figure 3.9 PETA-3 With Upstream Improvements Schematic

3.5.3 PETA-3 Preliminary Hydraulic Analysis

3.5.3.1 PETA-3 Hydraulic Modeling Assumptions

The system hydraulics of the PETA-3 alternative were evaluated to consider two potential scenarios. The first scenario assumes a single pump station is located near the North Marin Aqueduct that pumps to Nicasio and Soulajule Reservoirs. The second scenario considers two pump stations with one pump station located near the North Marin Aqueduct and the other located near Stafford Lake. The two pump stations would operate in series to pump water from the North Marin Aqueduct to the reservoirs.

For both scenarios, the first pump station would need to be located near the North Marin Aqueduct. The location of the first pump station is somewhat flexible from a hydraulics perspective, and we recommend evaluating potential sites in more detail. Based on discussions with NMWD and the District, there is a minimum of 170 feet of head in the existing North Marin Aqueduct at the proposed turnout location which would allow the first pump station to be located slightly away from the North Marin Aqueduct. Further discussion of pump station siting is included in Section 3.5.5.

Additionally, for both scenarios, it was assumed that a storage tank would be located directly upstream of the first pump station. The storage tank will act as a hydraulic buffer between the North Marin Aqueduct system and the new conveyance system. This hydraulic buffer will help to separate the two systems, reducing potential issues from variations in flow, pressure, and hydraulic pressure surge in the North Marin Aqueduct. We assume the tank will have a minimum operating level of 9 feet for the purposes of hydraulic modeling. Further discussion of hydraulic modeling assumptions is included in Appendix 3D.

3.5.3.2 Single Pump Station System

The single pump station system would pump from the North Marin Aqueduct (Pump Station A [PS A]), over the high points along the proposed alignment, to the reservoirs. The highest point along the proposed alignment is located near Nicasio Reservoir at elevation 551 feet. The system hydraulics for several different pipe sizes and capacities were evaluated based on this requirement, including:

- 30-inch diameter pipe with a demand of 10.4 mgd.
- 30-inch diameter pipe with a demand of 15.0 mgd.
- 36-inch diameter pipe with a demand of 10.4 mgd.
- 36-inch diameter pipe with a demand of 15.0 mgd.
- 42-inch diameter pipe with a demand of 30.0 mgd.
- 48-inch diameter pipe with a demand of 30.0 mgd.

Figure 3.10 and Figure 3.11 show the elevation profile and HGL for the pipe diameters and demands. Figure 3.10 and Figure 3.11 also include a yellow band that shows the target HGL range for the scenarios. The target HGL range shown is the sum of the elevation along the pipeline alignment plus a pipeline pressure ranging from 20 psi to 250 psi. In general, pipelines are designed to operate within this band of pressures ranging from 20 psi to 250 psi. While feasible, pipelines and pump stations with pressures exceeding 250 psi require specialty piping and appurtenances, leading to increased design complexity and construction costs.

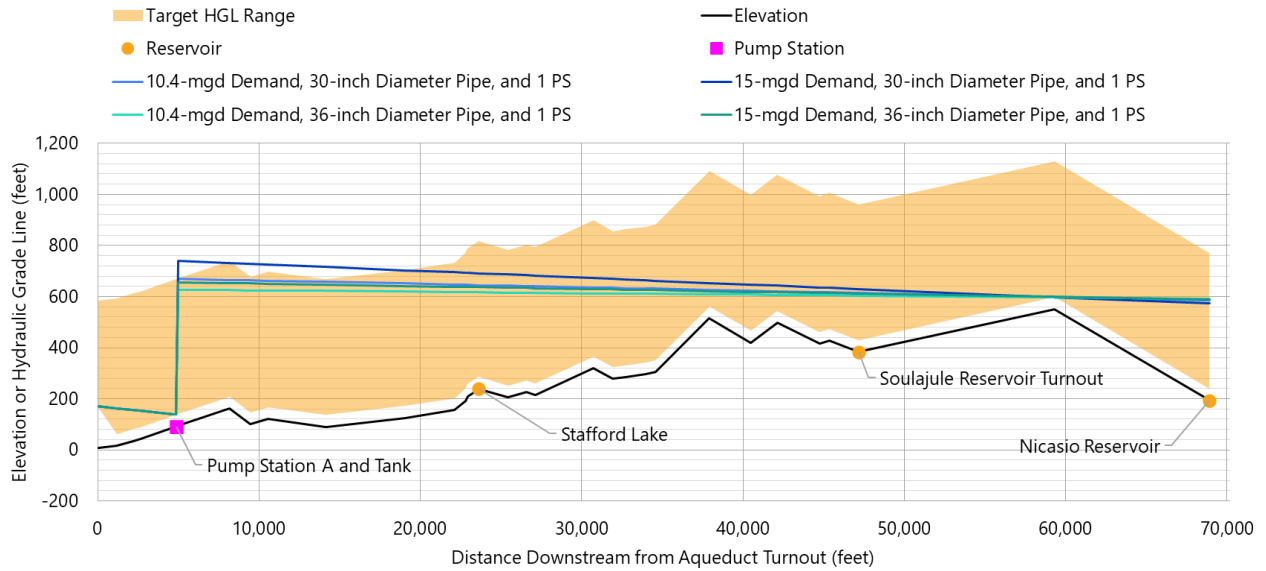


Figure 3.10 PETA-3 Single Pump Station with 10.4 mgd and 15.0 mgd Demand Hydraulic Profile

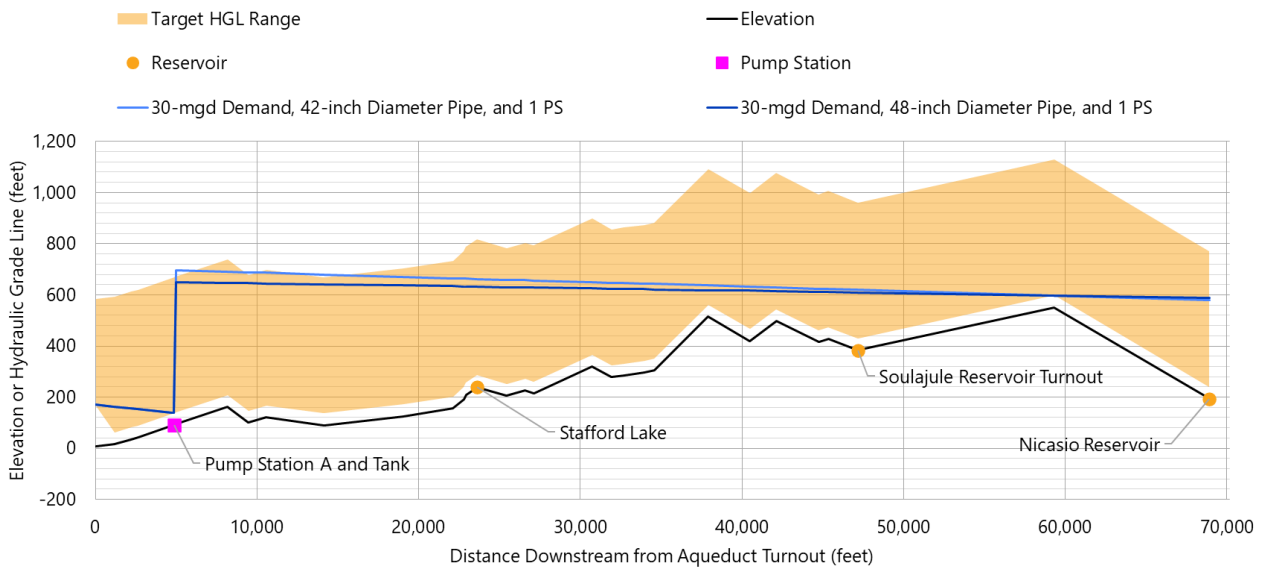


Figure 3.11 PETA-3 Single Pump Station with 30.0 mgd Demand Hydraulic Profile

As shown in Figure 3.10, for the scenarios that assume a demand of 10.4 and 15.0 mgd, the HGL remains within the yellow band of the target HGL range for both 36-inch pipe scenarios. For the scenarios that include a 30-inch pipe, the HGL downstream of the proposed PS A is higher than the yellow band of the target HGL range. We recommend considering the 36-inch pipe alternative to allow pipeline pressures to remain within the target HGL range for both the 10.4 and 15.0 mgd demands, assuming a single pump station.

As shown in Figure 3.11, for the scenario that assumes a demand of 30.0 mgd, the HGL remains within the yellow band of the target HGL range with a 48-inch pipe. For the scenario that includes a 42-inch pipe, the

HGL downstream of the proposed PS A is higher than the yellow band of the target HGL range. We recommend considering the 48-inch pipe alternative to allow pipeline pressures to remain within the target HGL range for the 30.0 mgd demand, assuming a single pump station.

3.5.3.3 Dual Pump Station System

Due to the significant head requirements anticipated for the PETA-3 alternative, a dual pump station system was also considered. The two pump stations would operate in series to pump water from the North Marin Aqueduct to the reservoirs. More specifically, the dual pump station system would pump from the pump station located near the North Marin Aqueduct (PS A) to the pump station at Stafford Lake (Pump Station B [PS B]). PS B would then pump water over the high points to the reservoirs. The highest point along the proposed alignment is located near Nicasio Reservoir at elevation 551 feet. The system hydraulics for several different scenarios were evaluated based on this requirement, including:

- 24-inch diameter pipe with a demand of 10.4 mgd.
- 24-inch diameter pipe with a demand of 15.0 mgd.
- 27-inch diameter pipe with a demand of 10.4 mgd.
- 27-inch diameter pipe with a demand of 15.0 mgd.
- 30-inch diameter pipe with a demand of 30.0 mgd.
- 36-inch diameter pipe with a demand of 30.0 mgd.

Figure 3.12 and Figure 3.13 show the elevation profile and HGL for these scenarios. Figure 3.12 and Figure 3.13 also include a yellow band that shows the target HGL range for the scenarios. The target HGL range shown is the sum of the elevation along the pipeline alignment plus a pipeline pressure ranging from 20 psi to 250 psi. In general, pipelines are designed to operate within this band of pressures ranging from 20 psi to 250 psi.

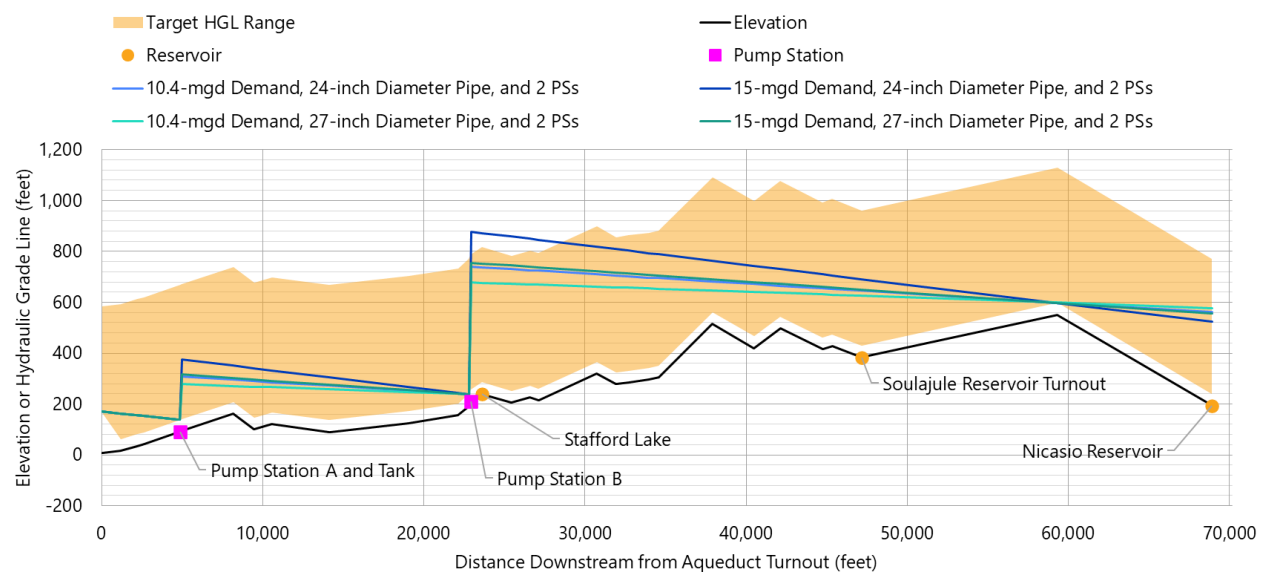


Figure 3.12 PETA-3 Dual Pump Station with 10.4 mgd and 15.0 mgd Demand Hydraulic Profile

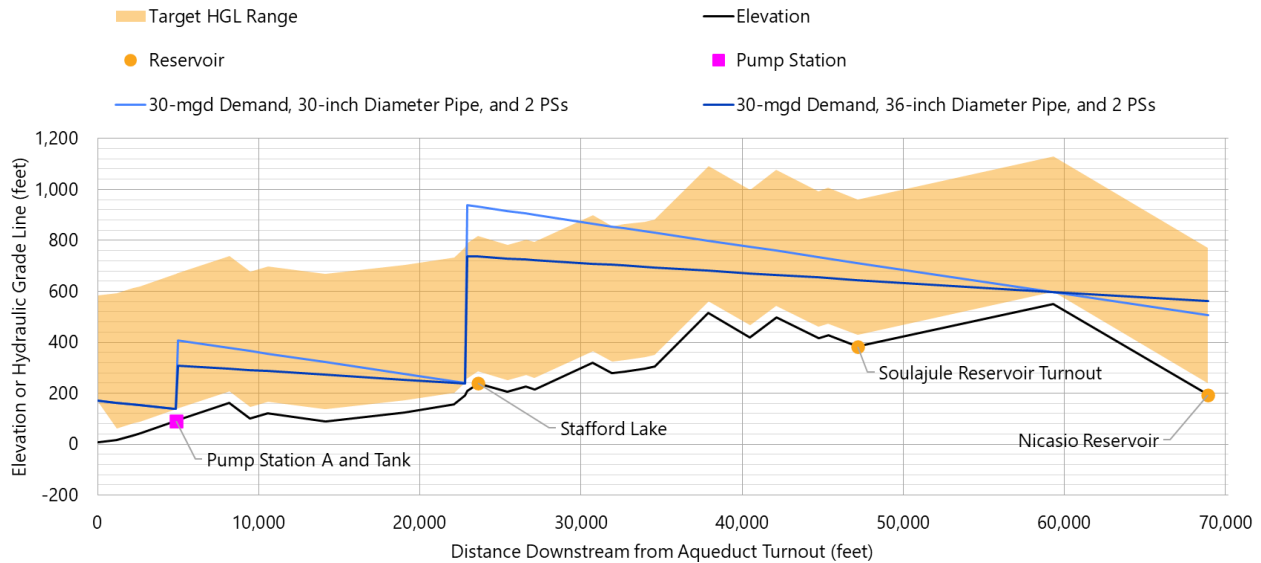


Figure 3.13 PETA-3 Dual Pump Station with 30.0 mgd Demand Hydraulic Profile

As shown in Figure 3.12, for the scenarios that assume a demand of 10.4 and 15.0 mgd, the HGL remains within the yellow band of the target HGL range for both 27-inch pipe scenarios. For the scenarios that include a 24-inch pipe, the HGL downstream of the proposed PS A is higher than the yellow band of the target HGL range. We recommend considering the 27-inch pipe alternative to allow pipeline pressures to remain within the target HGL range for both the 10.4 and 15.0 mgd demands, assuming two pump stations.

As shown in Figure 3.13, for the scenario that assumes a demand of 30.0 mgd, the HGL remains within the yellow band of the target HGL range with a 36-inch pipe. For the scenario that includes a 30-inch pipe, the HGL downstream of the proposed PS B is higher than the yellow band of the target HGL range. We recommend considering the 36-inch pipe alternative to allow pipeline pressures to remain within the target HGL range for the 30.0 mgd demand, assuming two pump stations.

3.5.3.4 Recommended Facility Sizing

Based on preliminary hydraulics, it is recommended that a 36-inch pipeline be installed for the PETA-3 alternative. With a 36-inch diameter pipeline, the system will have sufficient capacity to convey up to 15.0 mgd to Nicasio or Soulejoule Reservoir with a single pump station and up to 30.0 mgd with dual pump stations. This allows the District to construct a 15.0 mgd capacity system initially and increase the capacity to 30.0 mgd in the future.

3.5.4 Pump Station Site Layouts

3.5.4.1 Single Pump Station System

For the single pump station system described in Section 3.5.3.2, the pump station site will likely need to include a pump station building, storage tank, on-site detention basin, and other ancillary equipment. Figure 3.14. shows a general layout of the site.

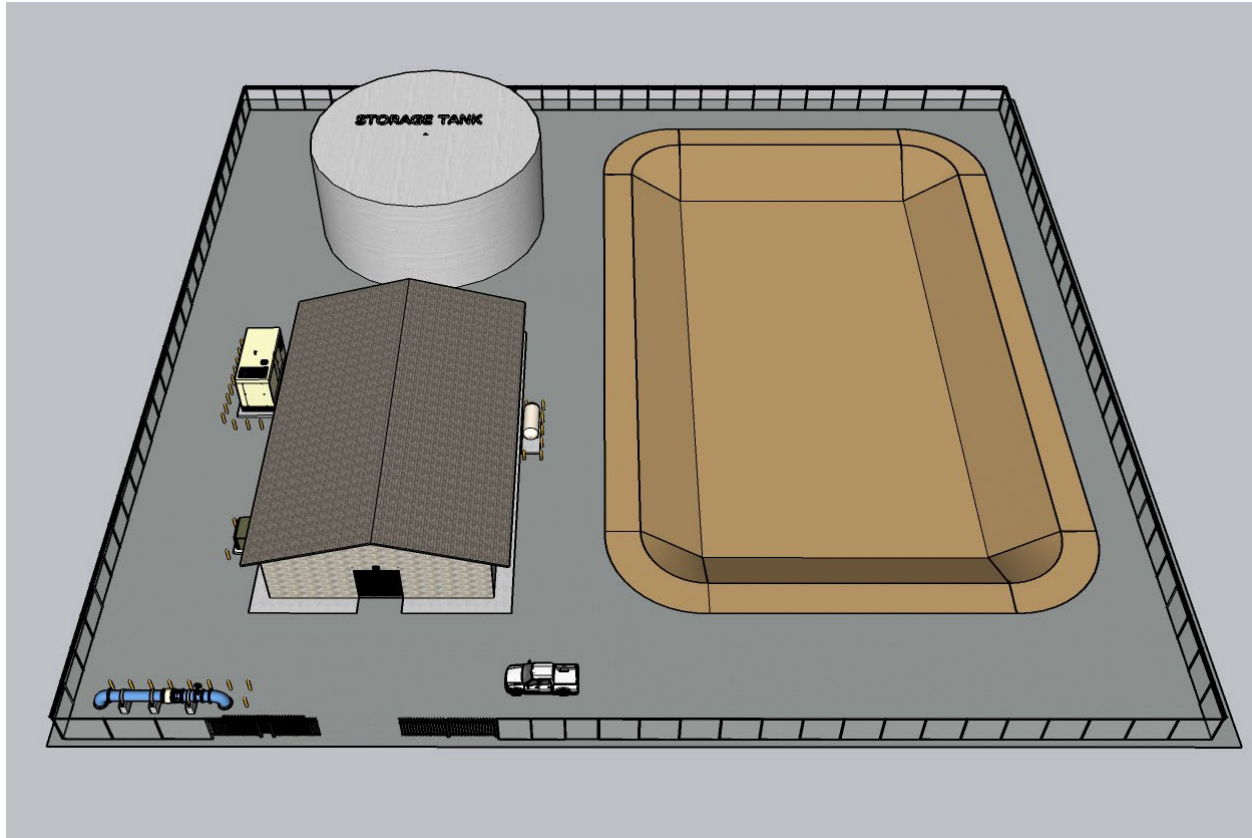


Figure 3.14 PETA-3 Pump Station A Site Layout

As discussed in Section 3.5.3.4, the pump station would likely be sized to accommodate a future capacity of 30.0 mgd. Figure 3.14 shows a typical pump station building that includes a pump mechanical room, separate electrical room, control room, and storage room. The approximate footprint of the 30.0 mgd pump station building is 120 feet by 65 feet.

The site will also need to include a storage tank. The storage tank will act as a hydraulic buffer between the North Marin Aqueduct system and the new conveyance system. This hydraulic buffer will help to separate the two systems, reducing potential issues from variations in flow, pressure, and hydraulic surge in the North Marin Aqueduct. For planning purposes, a 1-million-gallon (MG) tank is shown in Figure 3.14. However, the required capacity of the tank can vary depending on District preference and would be determined during preliminary design based on hydraulic modeling.

The site may also require an on-site detention basin. The on-site detention basin (assumed to be 1 MG to match the capacity of the storage tank) would be designed to store any overflow and/or water from draining the tank during O&M activities. The detention basin could be designed to infiltrate water into the ground or be pumped to the storm drain system. It may also be possible to send spent water from the tank directly to the storm drain system and eliminate the on-site detention basin. The viability of this alternative would depend on the storm drain capacity and ability to accept large flows in a short duration. Discharging to a storm drain system would require dechlorination before discharge.

Assuming a 120-foot-by-65-foot pump station building, 1-MG storage tank, and 1-MG on-site detention basin, the approximate required footprint of the site is 2.1 acres (300 feet by 305 feet). However, during preliminary design, the pump station site can be optimized and reduced, particularly if the on-site detention basin is eliminated.

3.5.4.2 Two Pump Station System

For the two pump station system described in Section 3.5.3.3, the first pump station site located near the North Marin Aqueduct will match the site layout described in Section 3.5.4.1.

The second pump station site located near Stafford Lake will only need to include a pump station building and other ancillary equipment. Since the second pump station will be operating in series with the first pump station and receiving water from the new conveyance system, a hydraulic buffer provided by a storage tank is not needed. Accordingly, a detention basin is not needed either. Figure 3.15 shows a general layout of the site. The approximate required footprint of the site is 0.75 acres (160 feet by 200 feet).



Figure 3.15 [PETA-3 Pump Station B Site Layout](#)

3.5.5 Pump Station Siting

As discussed previously, pump station siting will largely be driven by the new conveyance system hydraulics. Potential pump station locations are discussed in more detail below. During preliminary design, the potential pump station sites will be evaluated further and a preferred pump station site will be recommended.

3.5.5.1 Pump Station A Siting

Regardless of whether a one pump system or two pump station system is chosen, PS A will need to be located near the North Marin Aqueduct. Figure 3.16 shows the approximate limits where Pump Station A can be located.

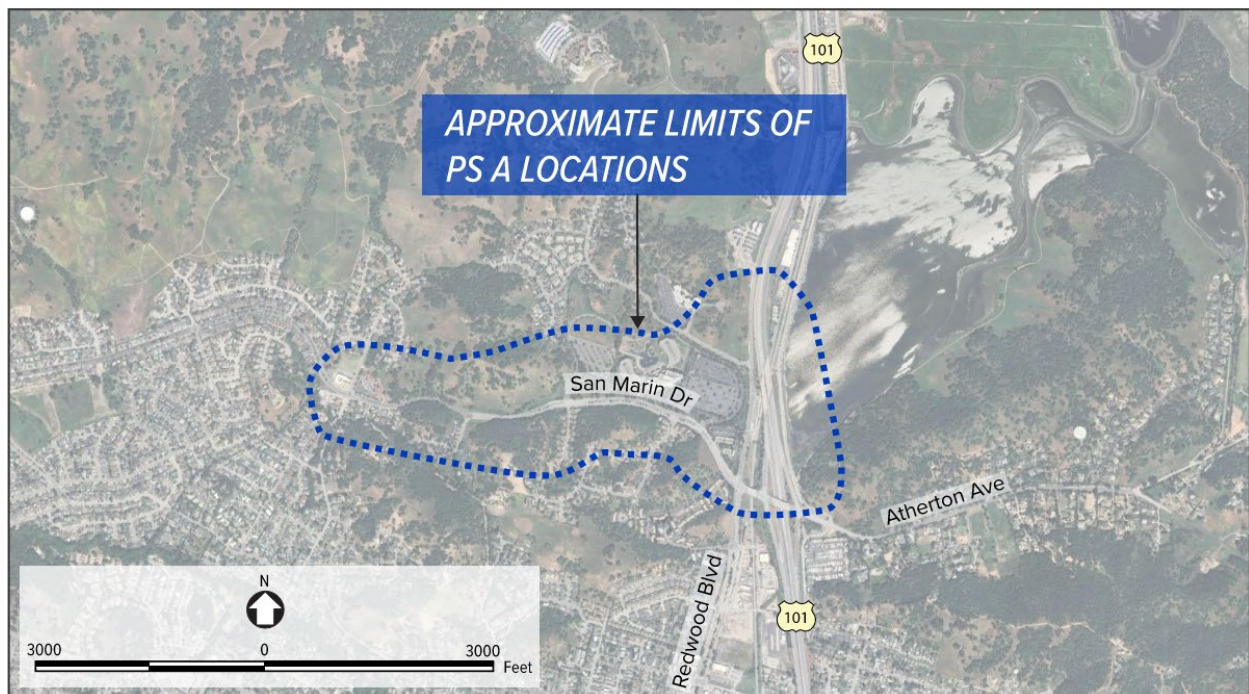


Figure 3.16 PETA-3 Pump Station A Siting

3.5.5.2 Pump Station B Siting

PS B is required in the future when the hydraulic capacity is increased beyond 15 mgd (30 mgd maximum). The pump station will need to be located near Stafford Lake. Figure 3.17 shows the approximate limits where Pump Station B can be located.

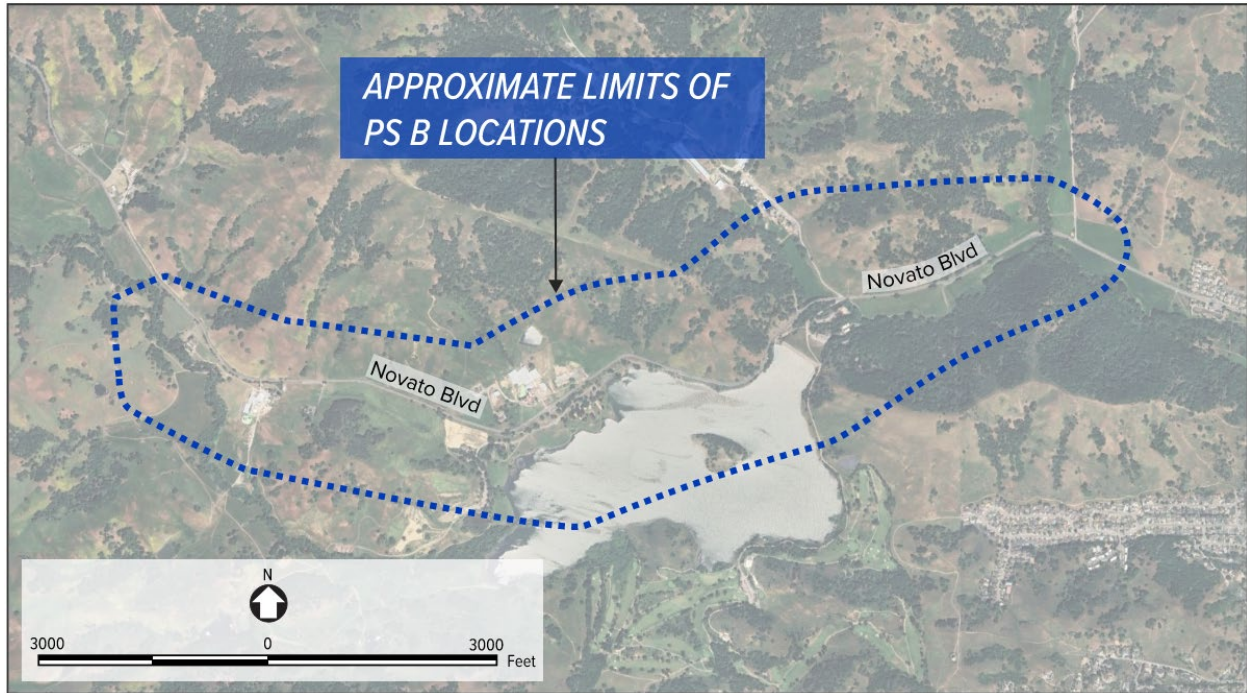


Figure 3.17 PETA-3 Pump Station B Siting

3.5.6 Environmental and Cultural Considerations

Environmental Science Associates (ESA) conducted an environmental evaluation of the initial alternatives identified in Phase 1 of the Project, including the PETA-3 alternative. No major environmental and cultural considerations were identified in the evaluation. Refer to Appendix 3A for the complete evaluation prepared by ESA.

3.5.7 Geotechnical Considerations

Miller Pacific Engineering Group (MPEG) performed a Geologic and Geotechnical Feasibility Evaluation of the initial alternatives identified in Phase 1 of the project, including the PETA-1 alternative and PETA-2 alternative which are the two components of the PETA-3 alternative. A brief summary of geologic and geotechnical issues anticipated for the PETA-3 alternative is below:

- Local landslide risks near Stafford Lake.
- Potential for naturally occurring asbestos along San Marin Drive.
- Potential for significant hard rock construction.

No major geologic or geotechnical considerations were identified in the evaluation. Refer to Appendix 3B for the complete evaluation prepared by MPEG.

3.5.8 Conservation Easements

Marin County has a significant portion of open land protected by conservation easements. A conservation easement is a legal agreement between a landowner and a conservation organization or government agency that permanently limits a property's uses to protect its conservation values. Each easement's restrictions are tailored to the particular property and to the interests of the owner and easement holder.

Figure 3.18 shows the conservation easements located along the proposed PETA-3 alignment. In general, the proposed alignment is located within the public right-of-way, with conservation easements adjacent to the right-of-way.

The one area where the proposed alignment is routed directly through conservation easement(s) is the section adjacent to Soulajule Reservoir. Through this area, there is no available public right-of-way that would allow routing of pipeline directly to the eastern finger of Soulajule Reservoir. Additionally, all land in this area is protected by conservation easements. Early coordination with the landowners and conservation organizations is needed to confirm feasibility of routing the pipeline through the conservation easement(s).

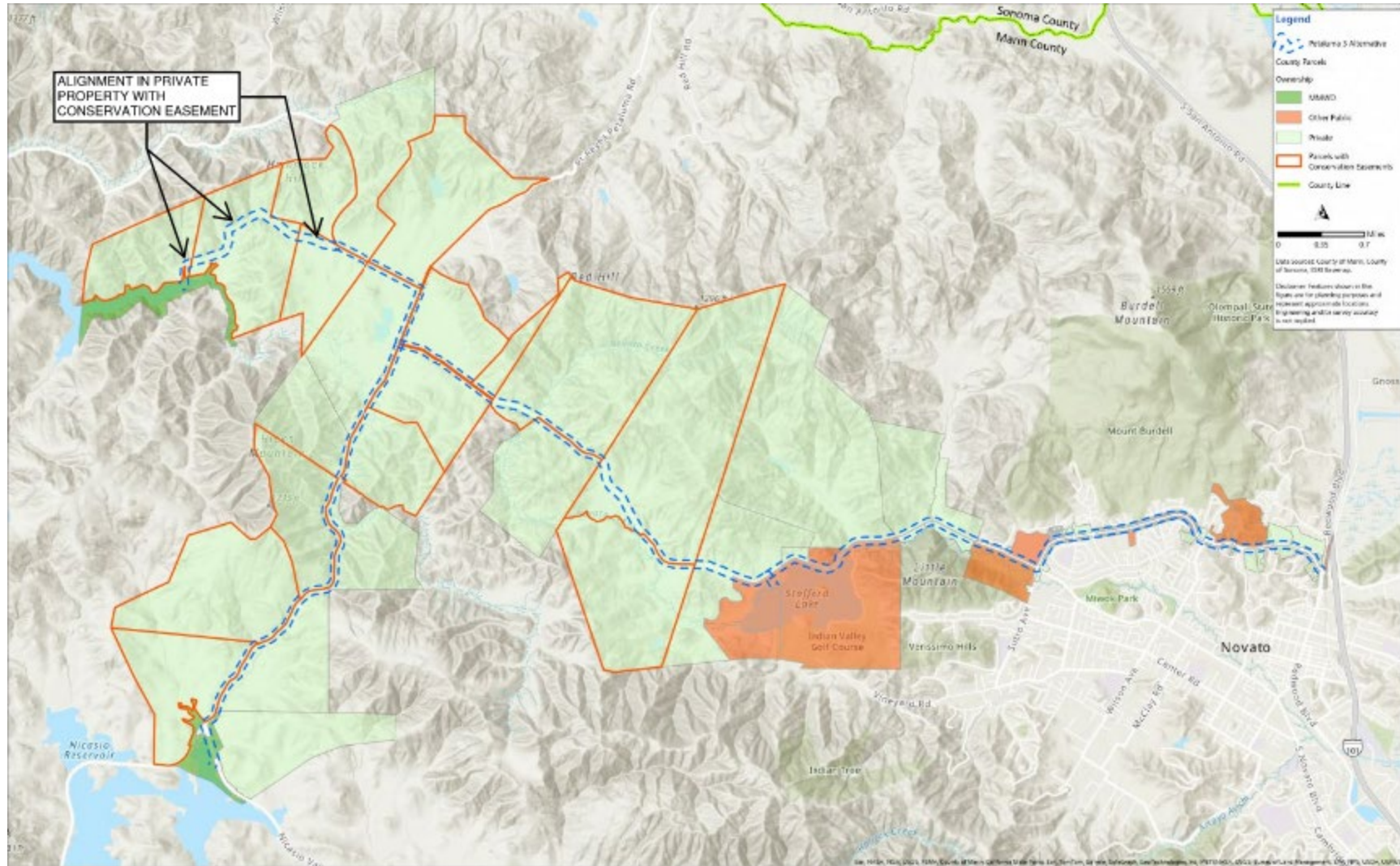


Figure 3.18 PETA-3 Conservation Easements

3.5.9 Constructability

The primary constructability concern for the PETA-3 alternative is construction of the pipeline since the alternative includes 86,000 linear feet (LF) of pipe. The segment of pipe along San Marin Drive from US-101 to Novato Boulevard (approximately 14,000 LF) is especially challenging considering there are many existing utilities.

Early investigation of existing utilities will help to determine potential pipeline alignments through congested utility corridors. Per Chapter 16 – California Waterworks Standards of Title 22 of the California Code of Regulations, the basic requirements for horizontal separation of new potable water mains are:

- 10-foot minimum horizontal clear separation from existing parallel pipeline conveying:
 - » Untreated sewage.
 - » Primary or secondary treated sewage.
 - » Secondary recycled water.
 - » Hazardous fluids such as fuels, industrial wastes, and wastewater sludge.
- 4-foot minimum horizontal clear separation from existing parallel pipeline conveying:
 - » Tertiary recycled water.
 - » Storm drainage.

For the remainder of the pipeline, the alignment is located through rural roadways, many of which are just a single lane in each direction and have a minimal shoulder. To facilitate installation of the pipeline through these areas, at least one lane of the road will need to be taken during construction. As such, traffic impacts are expected.

3.5.9.1 Pipeline Installation Methods

Open-Cut Pipeline Installation

Traditional open-cut construction allows for the installation of all pipe material and joint types. Open-cut construction is the process of installing underground pipelines by excavating. A trench is excavated to the required depth for installing a pipeline with the spoil from the excavation laid alongside the trench or off-hauled in trucks. After the pipeline is installed on bedding, the trench is backfilled with the designed backfill material and compacted to grade.

The majority of the proposed pipeline is recommended to be installed using open-cut methods due to the speed of installation and lower cost compared to other construction alternatives. Overhead power and telecom lines are located along a majority of the proposed water supply pipeline alignment. Depending on the proximity of the open trench to nearby utility poles, coordination with utility owners for support and protection of utility poles may be required. In addition, it is standard practice to lower 20-foot pipe segments into a trench using a backhoe or other large construction equipment. Potential conflicts between large construction equipment and overhead utilities are anticipated to complicate construction at some locations along the proposed alignment.

Stream Crossings

Preliminary environmental reviews have identified approximately 86 stream crossings along the proposed PETA-3 alignment. A majority of the stream crossings are anticipated to be smaller diameter culverts and can be crossed via open-cut pipeline installation. However, several culvert crossings may require trenchless pipe installation due to the larger diameter of the culverts and challenging existing site conditions. There are many different types of trenchless pipe installations that could be utilized for this project, including pipe ramming, auger boring or horizontal directional drilling (HDD). The recommended method will depend on existing site conditions, including geotechnical conditions.

3.5.10 Power Availability

Ulteig Engineers, Inc. (UEI) performed a transmission and distribution power analysis study to evaluate the impacts of the proposed pump stations on Pacific Gas and Electric's (PG&E) existing power grid. The goal of the study was to perform a high-level study of the PG&E power supply availability at each pump station site to determine if there are any fatal flaws. The analysis was based on the anticipated pump station loads and proposed pump station locations.

3.5.10.1 Transmission System Analysis

The analysis of the existing transmission system utilized California Independent System Operator (CAISO) data to evaluate the impacts of the proposed pump station loads on the existing PG&E transmission system. For each pump station, the nearest 60-kilovolt (kV) line was identified. Each transmission line was then tapped as the equivalent Point of Interconnection (POI). A thermal analysis was then conducted for both summer and winter scenarios.

For the PETA-3 alternative, the proposed pump stations are located near existing 60-kV facilities. The addition of the pump stations added minimal load contributions to the existing PG&E transmission system. However, under all scenarios, several facilities were determined to have preexisting thermal overloads. Further coordination with PG&E is needed to determine if the added load contributions are acceptable. Refer to Appendix 3C for the complete evaluation prepared by UEI.

3.5.10.2 Distribution System Analysis

The analysis of the existing distribution system utilized PG&E's Distribution Investment Deferral Framework (DIDF) and Integration Capacity Analysis (ICA) maps to evaluate the impacts of the proposed pump station loads on the existing PG&E distribution system. For each pump station, the nearest 12-kV line was identified. For PETA-3 PS A, the nearest line was approximately 3,600 LF away. Accordingly, 3,600 LF of new 3-phase line would need to be constructed to connect the existing distribution system to the new pump station. For PETA-3 PS B, the nearest line was approximately 50 LF away. Each respective distribution line was then tapped and the proposed loads were added to the system. For the PETA-3 alternative, the addition of the proposed pump station loads did not lead to excessive loads in the distribution systems. Further coordination with PG&E is needed to determine if the added load contributions are acceptable. Refer to Appendix 3C for the complete evaluation prepared by UEI.

3.5.11 Project Phasing

PETA-3 provides a unique advantage of allowing construction of the project to be phased, with future phases providing additional water supply benefit. Based on preliminary hydraulics, it is recommended that a 36-inch pipeline be installed for the PETA-3 alternative. With a 36-inch diameter pipeline, the system will have sufficient capacity to convey up to 15.0 mgd to Nicasio or Soulejoule Reservoir with a single pump station and up to 30.0 mgd with dual pump stations. This allows the District to construct a 15.0 mgd capacity system initially and increase the capacity to 30.0 mgd in the future, if upstream improvements are constructed.

Although there are several potential options for project phasing, it is recommended that the initial phase include construction of pipelines and pump station(s) to convey water from the North Marin Aqueduct to Nicasio Reservoir. Directly downstream of Nicasio Reservoir, there is a water treatment plant that treats and distributes water into the District's distribution system. For all water sent to Soulejoule Reservoir, the water must be pumped from Soulejoule Reservoir to Nicasio Reservoir prior to being able to be utilized by the District. This is the recommended initial phase, regardless of whether the PETA-3 without upstream improvements alternative or PETA-3 with upstream improvements alternative is selected.

3.5.11.1 PETA-3 Without Upstream Improvements

If the PETA-3 without upstream improvements alternative is selected, the next and final phase would then include the required pipeline to extend the project to Soulejoule Reservoir.

3.5.11.2 PETA-3 With Upstream Improvements

If the PETA-3 with upstream improvements alternative is selected, the next phase would include construction of the upstream improvements detailed in Section 3.5.2.2. These upstream improvements include:

- Upsizing sections of the existing 30-inch and 36-inch pipeline upstream of the proposed turnout location along the North Marin Aqueduct or constructing a separate transmission main parallel to the existing 30-inch and 36-inch pipeline.
- Increasing the capacity of Kastania Pump Station from 25.0 mgd to 42.5 mgd.
- Constructing an STS alternative that can convey an additional 17.1 mgd from Cotati Tanks to Kastania Pump Station.

The next and final phase would then include the required pipeline to extend the project to Soulejoule Reservoir.

3.5.12 Coordination with North Marin Water District

NMWD owns and operates water distribution facilities within the City of Novato. NMWD facilities include pipelines and pump stations that are capable of conveying water from the North Marin Aqueduct to Stafford Lake. Early in this Project, alternatives that utilized this existing infrastructure were considered. However, based on more recent discussions with NMWD, it's been determined that very limited capacity is available in the existing infrastructure to convey water from the North Marin Aqueduct to Stafford Lake for District use. As such, alternatives that considered utilizing NMWD infrastructure were not considered further.

During those discussions with NMWD it was also determined that NMWD may be interested in having a turnout at Stafford Lake that would allow NMWD to pull water from the new PETA-3 facilities. Additional coordination with NMWD is needed to confirm details, including proposed turnout location and an agreement on how much water NMWD would be able to take.

3.5.13 Project Cost Estimates

This section summarizes the budgetary project cost estimates for the PETA-3 alternative.

3.5.13.1 Basis for Estimate

The Project alternative is at the preliminary engineering phase with limited design information. The construction cost estimates are consistent with the Association for the Advancement of Cost Engineering (AACE) International Class 4 estimate with an accuracy range of +50 percent to -30 percent of the actual project cost.

Unit Costs

The project cost estimates presented are based on unit costs for the major pipeline and pump station components of the Project. The unit costs for the pipelines are based on information received from pipeline manufacturers for dollars per foot cost of the pipeline material. The pipeline installation costs are based on similar cost estimates on other recent projects. The costs for the pump stations are also based on similar cost estimates on other recent projects.

Contingencies

Contingencies are applied to a construction estimate at the design development phase to account for construction items not yet identified and design unknowns. We recommend carrying a 30 to 50 percent contingency during the planning phase of a project considering the design details are not known at this time. The contingency will be reduced as the design is developed. Agencies typically retain contingency within their project budgets, even when construction contract award values are known, to cover the cost or deal with unforeseen conditions.

The cost estimates presented in this TM include a 30 percent contingency, calculated based on the raw construction cost. This is in alignment with the recommendations for a project at an AACE International Class 4 level of development.

3.5.13.2 Cost Estimates

Table 3.4 presents a summary of the cost estimate for the PETA-3 alternative without upstream improvements. The cost estimate assumes a single pump station system with 36-inch diameter pipe is utilized.

Table 3.4 PETA-3 Without Upstream Improvements Cost Estimate

Cost Item	Cost (rounded)
General Conditions (i.e., mobilize/demobilize, temporary facilities, testing, start-up, etc.)	\$5,200,000
Pipeline	\$71,560,000
Pump Stations	\$18,000,000

Cost Item	Cost (rounded)
Power Supply	\$850,000
Ancillary Items	\$13,560,000
Direct Cost Subtotal	\$109,170,000
Contingency (30%)(1)	\$32,750,000
General Contractor Overhead, Profit, and Risk (12%)(1)	\$17,030,000
Escalation to Mid-Point (Assumes Mid-Point in April 2028) (3% per Year)(1)	\$17,890,000
Sales Tax (8.25% on Half the Direct Cost)(1)	\$7,290,000
Total Estimated Construction Cost	\$184,130,000
<i>Estimated Construction Cost Range (-30%)</i>	<i>\$128,890,000</i>
<i>Estimated Construction Cost Range (+50%)</i>	<i>\$276,190,000</i>

Notes:

(1) Indirect cost factors are applied to the running subtotal of direct costs plus preceding indirect cost; see Appendix 3E for additional information.

Table 3.5 presents a summary of the cost estimate for the PETA-3 alternative with upstream improvements. The cost estimate assumes a dual pump station system with 36-inch diameter pipe is utilized from the North Marin Aqueduct to the reservoirs, construction of a new pipeline parallel the existing North Marin Aqueduct, upsizing of the Kastania PS and construction of an STS alternative. A detailed breakdown of the construction cost estimate is included in Appendix 3E.

Table 3.5 PETA-3 With Upstream Improvements Cost Estimate

Cost Item	Cost (rounded)
General Conditions (i.e., mobilize/demobilize, temporary facilities, testing, start-up, etc.)	\$10,510,000
Pipeline	\$71,560,000
Pump Stations	\$33,500,000
Kastania Pump Station Improvements	\$10,000,000
STS Alternative	\$53,630,000
North Marin Aqueduct Parallel Pipeline	\$12,990,000
Power Supply	\$1,050,000
Ancillary Items	\$27,410,000
Direct Cost Subtotal	\$220,650,000
Contingency (30%)(1)	\$66,190,000
General Contractor Overhead, Profit, and Risk (12%)(1)	\$34,420,000
Escalation to Mid-Point (Assumes Mid-Point in April 2028) (3% per Year)(1)	\$36,140,000
Sales Tax (8.25% on Half the Direct Cost)(1)	\$14,740,000
Total Estimated Construction Cost	\$372,140,000
<i>Estimated Construction Cost Range (-30%)</i>	<i>\$260,500,000</i>
<i>Estimated Construction Cost Range (+50%)</i>	<i>\$558,220,000</i>

Notes:

(1) Indirect cost factors are applied to the running subtotal of direct costs plus preceding indirect cost; see Appendix 3E for additional information.

3.6 Evaluation of PETA-4: North Marin Aqueduct to Nicasio and Soulajule Reservoirs via San Antonio Road

3.6.1 Introduction

The PETA-4: North Marin Aqueduct to Nicasio and Soulajule Reservoirs via San Antonio Road alternative includes two new pump stations and approximately 14.8 miles of pipeline to convey water from the North Marin Aqueduct to Nicasio and Soulajule Reservoirs.

This alternative includes a turnout from the North Marin Aqueduct farther upstream than the PETA-3 alternative. This alternative requires approximately 7.8 miles of new pipeline routed along San Antonio Road and Pt. Reyes – Petaluma Road. At the intersection of Pt. Reyes – Petaluma Road and Hicks Valley Road, the pipe will bifurcate with one branch heading south approximately 4.5 miles along Pt. Reyes – Petaluma Road to Nicasio Reservoir and the other branch heading west approximately 2.5 miles along Hicks Valley Road and a fire road to Soulajule Reservoir. Figure 3.19 shows the proposed pipeline alignment and profile. Refer to Appendix F for more detailed plan view drawings of the proposed pipeline alignment.

For this alternative, the additional water supply will be conveyed to Nicasio and Soulajule Reservoirs for storage. The District will have the ability to select which reservoir to fill based on reservoir levels, demands within the system, and operational considerations. By utilizing both reservoirs, the District can increase the amount of water transferred from Sonoma Water.

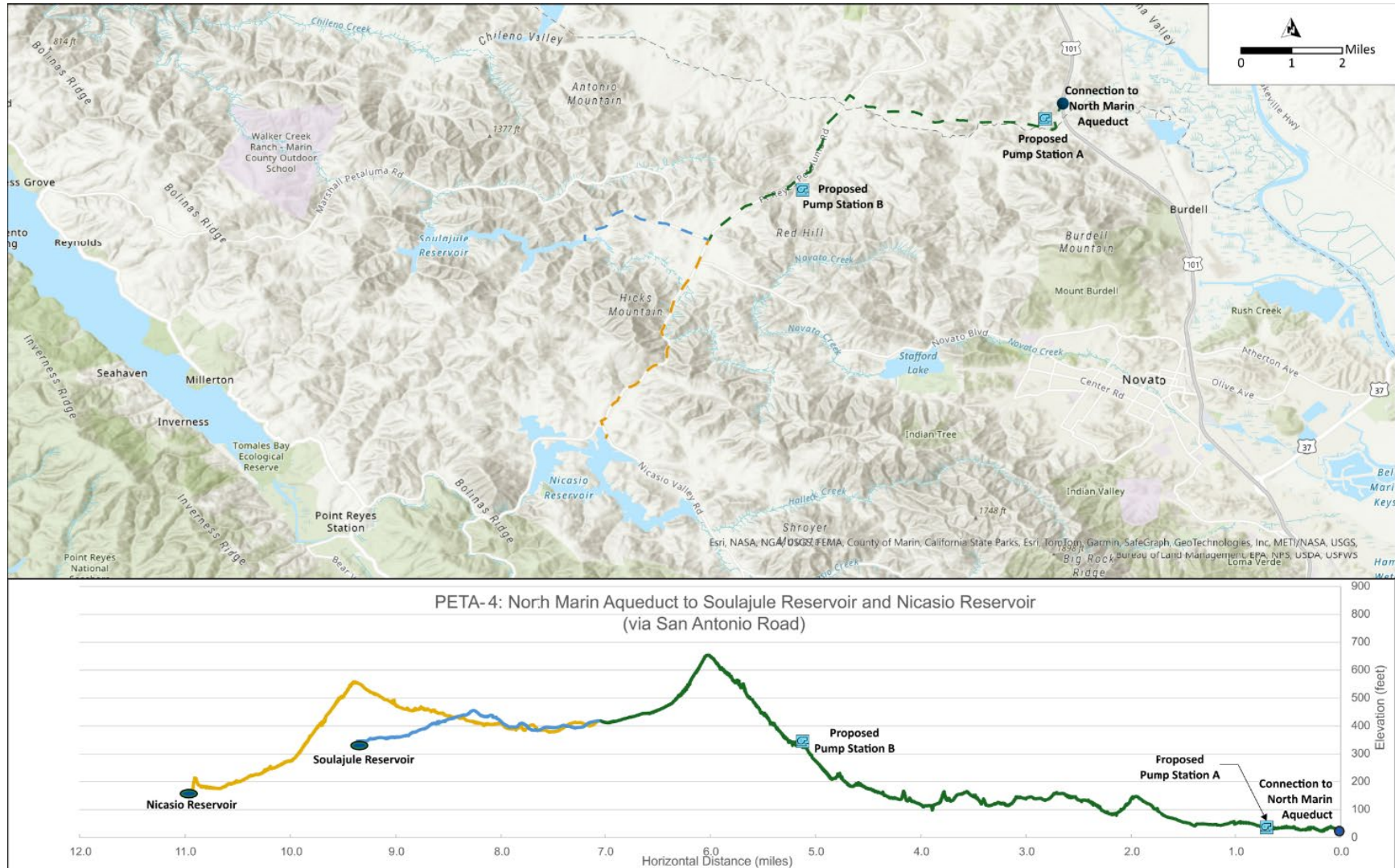


Figure 3.19 PETA-4 Alignment Overview

3.6.2 Water System Supply Benefits

3.6.2.1 PETA-4 Without Upstream Improvements

The sizing of the facilities for PETA-4 is highly dependent upon the capacity of existing infrastructure upstream of the proposed turnout location and upstream and downstream demands/diversions along the North Marin Aqueduct. Based on current operations, the existing infrastructure can convey up to 30.4 mgd of water from Cotati Tanks to Kastania Pump Station, with Petaluma diverting approximately 5.0 mgd of that during wintertime months. Therefore, up to 25.4 mgd of water can be delivered to Kastania Pump Station during wintertime months. Kastania Pump Station has a maximum capacity of 25.0 mgd.

The proposed turnout location for PETA-4 is north of Novato at San Antonio Road, downstream of Kastania Pump Station. At the proposed turnout location, the existing North Marin Aqueduct has a nominal diameter of 42 inches and can convey approximately 49.8 mgd, assuming a maximum allowable velocity in the pipeline of 8.0 fps. Additionally, downstream of Kastania Pump Station, the system demand during wintertime months is approximately 12.5 mgd. As previously discussed, NMWD would divert approximately 4.5 mgd during wintertime months and the District would receive and distribute approximately 8.0 mgd through Ignacio Pump Station.

The largest bottleneck in the system upstream of the proposed turnout location is Kastania Pump Station. Assuming that Kastania Pump Station has a maximum capacity of 25.0 mgd, and assuming there is a demand of 12.5 mgd during wintertime operations downstream of the proposed turnout location, approximately 12.5 mgd of water is anticipated to be available to be conveyed from the North Marin Aqueduct to the reservoirs during the wintertime months.

Figure 3.20 shows a schematic of the existing system from Cotati Tanks to Ignacio Pump Station. The schematic is diagrammatic and does not show all existing or proposed facilities. The proposed turnout location is shown in green and the largest bottleneck in the existing system is shown in red. No other improvements or upgrades to the existing system are anticipated to be needed to convey 12.5 mgd from the North Marin Aqueduct to the reservoirs.

For this alternative, it is understood that actual demands for other customers may vary from the assumed demands listed above. As such, we recommend that facilities be oversized from the North Marin Aqueduct to the reservoirs to convey additional water that might be available if customers along the North Marin Aqueduct don't take their full current allotment. Final sizing of facilities should be determined with District input during preliminary design.

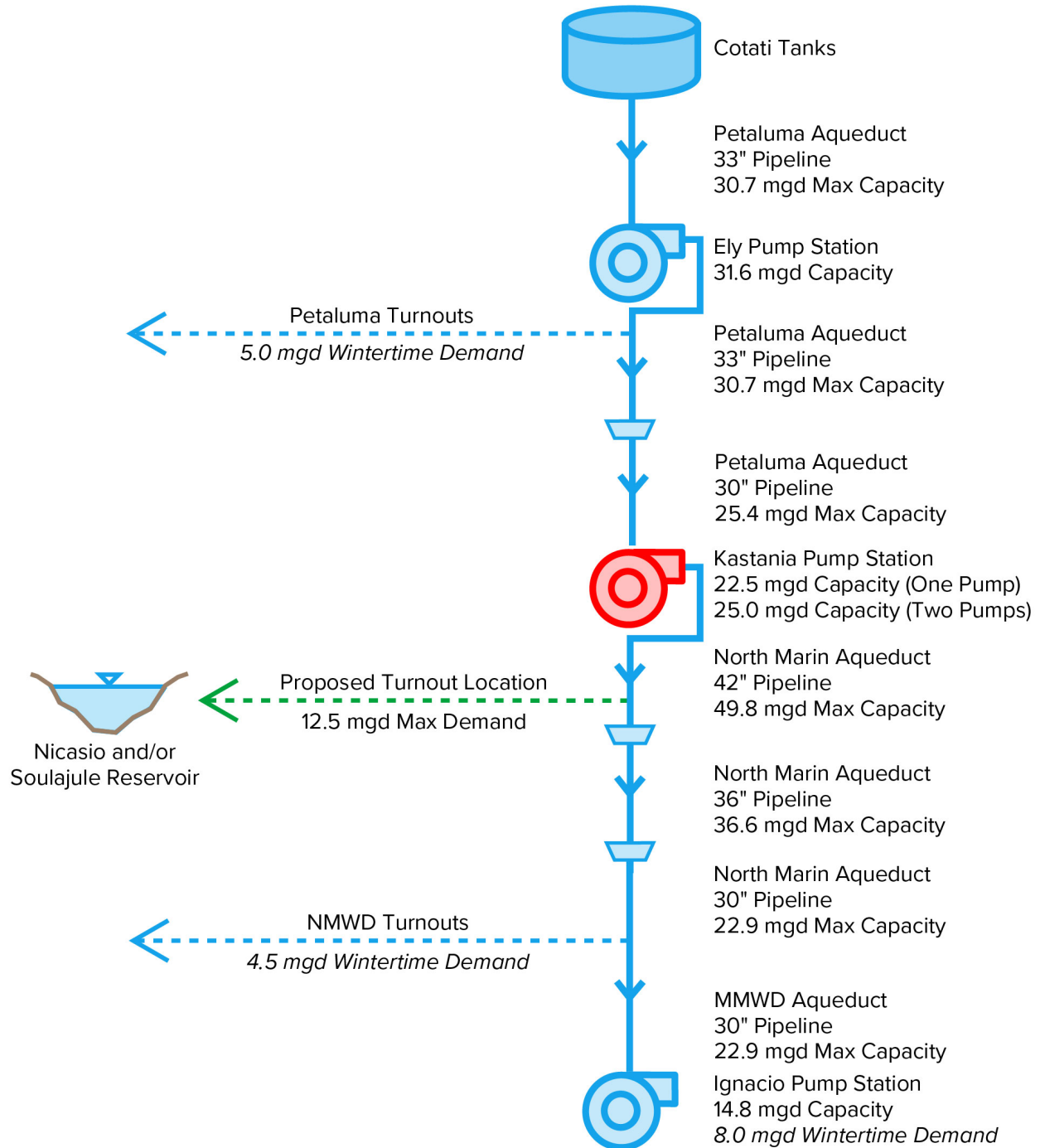


Figure 3.20 PETA-4 Without Upstream Improvements Schematic

3.6.2.2 PETA-4 with Upstream Improvements

It's possible to increase the water supply benefit of the PETA-4 alternative by implementing improvements to address bottlenecks upstream of the proposed turnout location. The improvements were selected that maximize the use of existing infrastructure and provide the most water supply benefit. The required improvements to the existing system that allow up to 30.0 mgd of water to be conveyed from the North Marin Aqueduct to the reservoirs include:

- Increasing the capacity of Kastania Pump Station from 25.0 mgd to 42.5 mgd.
- Constructing an STS alternative that can convey an additional 17.1 mgd from Cotati Tanks to Kastania Pump Station:
 - » The existing infrastructure can convey up to 30.4 mgd of water from Cotati Tanks to Kastania Pump Station:
 - Petaluma pulls approximately 5.0 mgd of that during wintertime operations. As such, approximately 25.4 mgd of water can currently be delivered to Kastania Pump Station via the existing Petaluma Aqueduct and Ely Pump Station during wintertime months.
 - With the addition of facilities conveying 17.1 mgd from Cotati Tanks to Kastania Pump Station, up to 42.5 mgd of water could be delivered to Kastania Pump Station during wintertime operations:
 - Improvements to the existing Petaluma Aqueduct or Ely BPS are not required.
 - » The STS alternative required to convey an additional 17.1 mgd from Cotati Tanks to Kastania Pump Station will likely need to include approx. 10 miles of new 30-inch pipeline and a booster pump station. Refer to TM1 and TM2 for further detail on potential STS alternatives.

Figure 3.21 shows a schematic of the existing system from Cotati Tanks to Ignacio Pump Station. The schematic is diagrammatic and does not show all existing or proposed facilities. The proposed turnout location is shown in green and the largest bottlenecks in the existing system are shown in red. If the improvements detailed above are implemented with the Project, the maximum capacity of the system upstream of the turnout location will be increased to 42.5 mgd, which would allow the District to convey up to 30.0 mgd from the North Marin Aqueduct to the reservoirs during wintertime operations when downstream demands are 12.5 mgd.

For this alternative, it is understood that actual demands for other customers may vary from the assumed demands listed above. As such, we recommend that facilities be oversized from the North Marin Aqueduct to the reservoirs to convey additional water that might be available if customers along the North Marin Aqueduct don't take their full current allotment. Final sizing of facilities should be determined with District input during preliminary design.

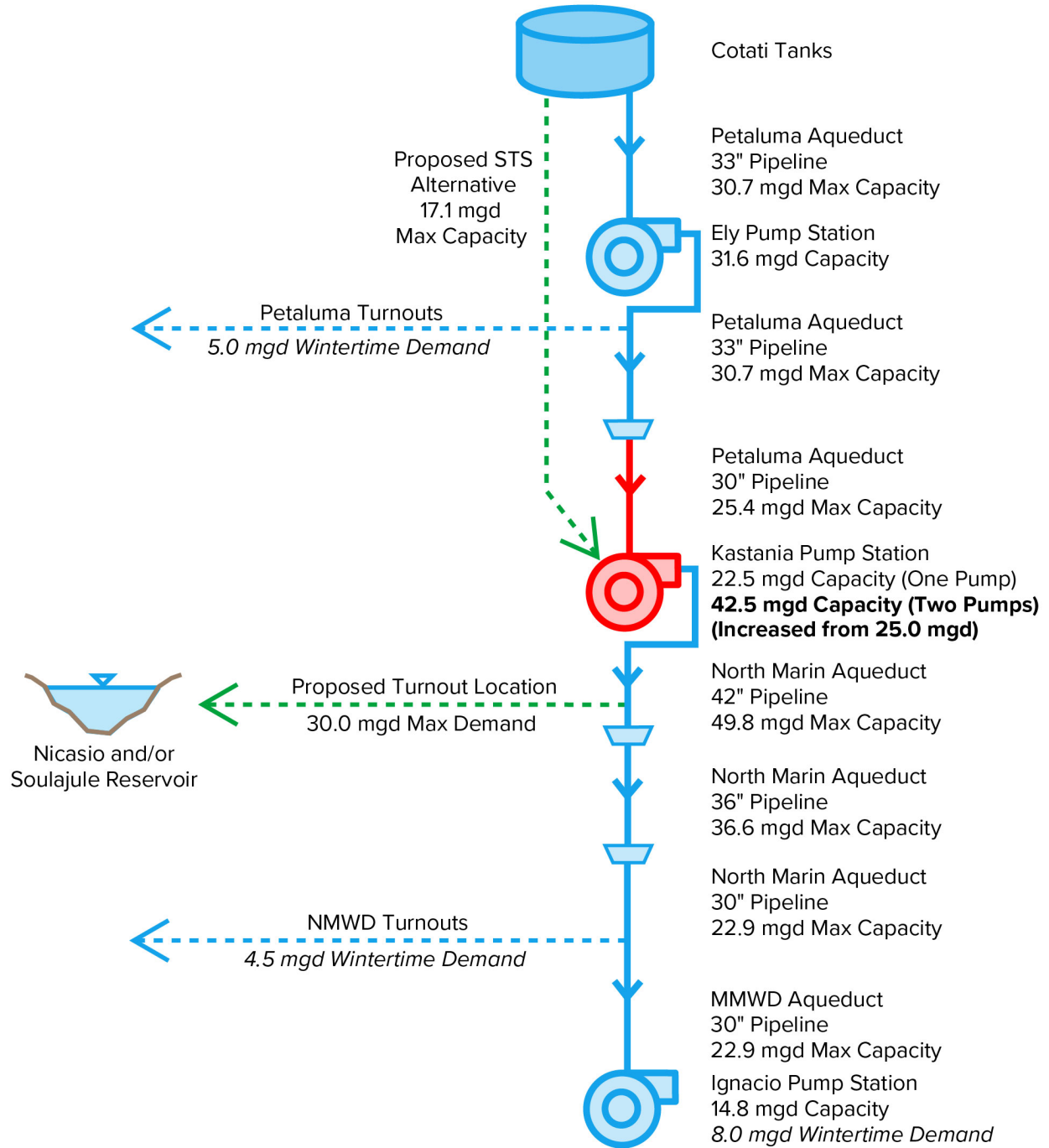


Figure 3.21 PETA-4 With Upstream Improvements Schematic

3.6.3 PETA-4 Preliminary Hydraulics

3.6.3.1 PETA-4 Hydraulic Modeling Assumptions

The system hydraulics of the PETA-4 alternative were evaluated to consider two potential scenarios. The first scenario assumes a single pump station is located near the North Marin Aqueduct that pumps to Nicasio and Soulajule Reservoirs. The second scenario considers two pump stations with one pump station located near the North Marin Aqueduct and the other located near Red Hill. The two pump stations would operate in series to pump water from the North Marin Aqueduct to the reservoirs.

For both scenarios, the first pump station would need to be located near the North Marin Aqueduct. The location of the first pump station is somewhat flexible from a hydraulics perspective, and we recommend evaluating potential sites in more detail. Based on discussions with NMWD and the District, there is approximately 170 feet of head in the existing North Marin Aqueduct at the proposed turnout location which would allow the first pump station to be located slightly away from the North Marin Aqueduct. Further discussion of pump station siting is included in Section 3.6.5.

Additionally, for both scenarios, it was assumed that a storage tank would be located directly upstream of the first pump station. The storage tank will act as a hydraulic buffer between the North Marin Aqueduct system and the new conveyance system. This hydraulic buffer will help to separate the two systems, reducing potential issues from variations in flow, pressure and hydraulic surge in the North Marin Aqueduct. We assume the tank will have a minimum operating level of 15 feet for the purposes of hydraulic modeling. Further discussion of hydraulic modeling assumptions is included in Appendix 3D.

3.6.3.2 Single Pump Station System

The single pump station system would pump from the North Marin Aqueduct (PS A), over the high points along the proposed alignment, to the reservoirs. The highest point along the proposed alignment is located near Red Hill at elevation 656 feet. The system hydraulics for several different pipe sizes and capacities were evaluated based on this requirement, including:

- 30-inch diameter pipe with a demand of 12.5 mgd.
- 30-inch diameter pipe with a demand of 30.0 mgd.
- 36-inch diameter pipe with a demand of 12.5 mgd.
- 36-inch diameter pipe with a demand of 30.0 mgd.
- 42-inch diameter pipe with a demand of 30.0 mgd.

Figure 3.22 and Figure 3.23 show the elevation profile and HGL for the pipe diameters and demands. Figure 3.22 and Figure 3.23 also include a yellow band that shows the target HGL range for the scenarios. The target HGL range shown is the sum of the elevation along the pipeline alignment plus a pipeline pressure ranging from 20 psi to 250 psi. In general, pipelines are designed to operate within this band of pressures ranging from 20 psi to 250 psi. While feasible, pipelines and pump stations with pressures exceeding 250 psi require specialty piping and appurtenances, leading to increased design complexity and construction costs.

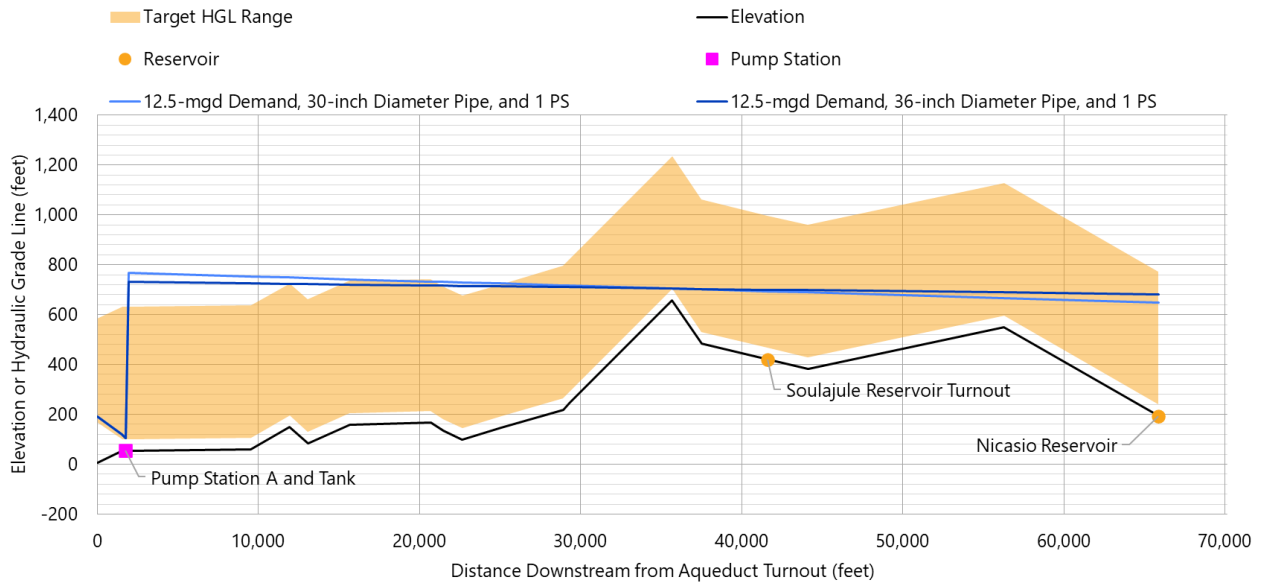


Figure 3.22 PETA-4 Single Pump Station with 12.5 mgd Demand Hydraulic Profile

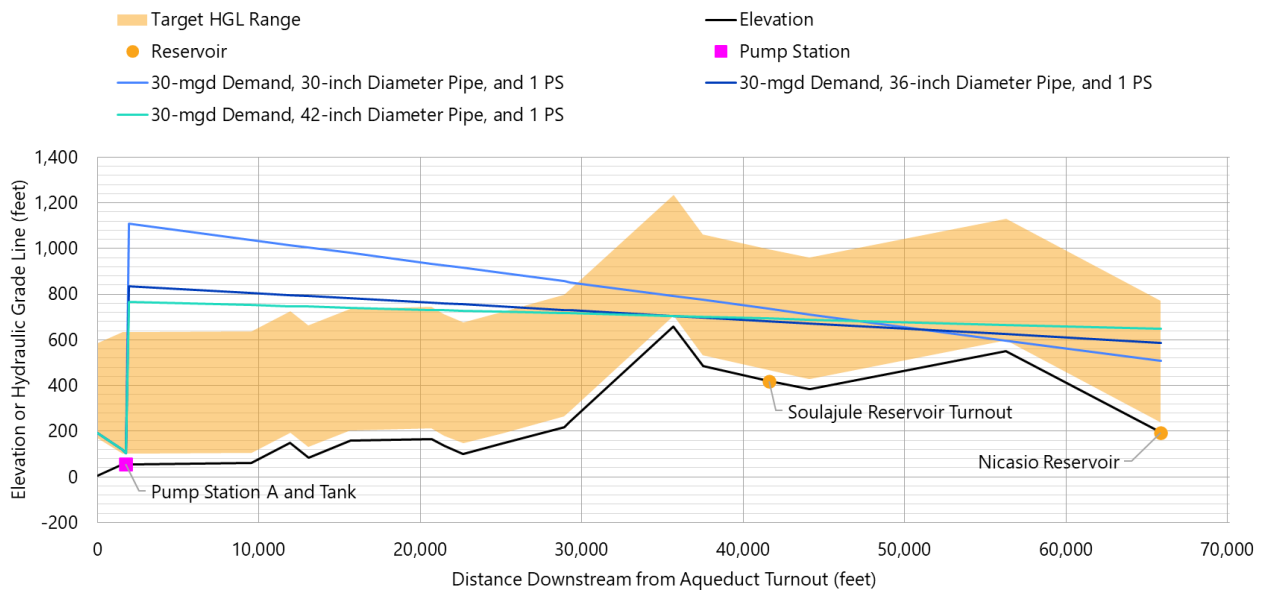


Figure 3.23 PETA-4 Single Pump Station with 30.0 mgd Demand Hydraulic Profile

As shown in Figure 3.22 and Figure 3.23, the HGL downstream of the proposed PS A for all pipeline diameters is higher than the yellow band of the target HGL range. This means that pressures in this area exceed 250 psi. Accordingly, a conveyance system with a single pump station would have pipeline pressures that exceed the target HGL range regardless of the pipeline diameter.

3.6.3.3 Dual Pump Station System

Due to the significant head requirements anticipated for the PETA-3 alternative, a dual pump station system was also considered. The two pump stations would operate in series to pump water from the North Marin Aqueduct to the reservoirs. More specifically, the dual pump station system would pump from the pump station located near the North Marin Aqueduct (PS A) to the pump station at Red Hill (PS B). PS B would then pump water over the high points to the reservoirs. The highest point along the proposed alignment is located near Red Hill at elevation 656 feet. The system hydraulics for several different scenarios were evaluated based on this requirement, including:

- 24-inch diameter pipe with a demand of 12.5 mgd.
- 27-inch diameter pipe with a demand of 12.5 mgd.
- 30-inch diameter pipe with a demand of 12.5 mgd.
- 30-inch diameter pipe with a demand of 30.0 mgd.
- 36-inch diameter pipe with a demand of 30.0 mgd.
- 42-inch diameter pipe with a demand of 30.0 mgd.

Figure 3.24 and Figure 3.25 show the elevation profile and HGL for these scenarios. Figure 3.24 and Figure 3.25 also include a yellow band that shows the target HGL range for the scenarios. The target HGL range shown is the sum of the elevation along the pipeline alignment plus a pipeline pressure ranging from 20 psi to 250 psi. In general, pipelines are designed to operate within this band of pressures ranging from 20 psi to 250 psi.

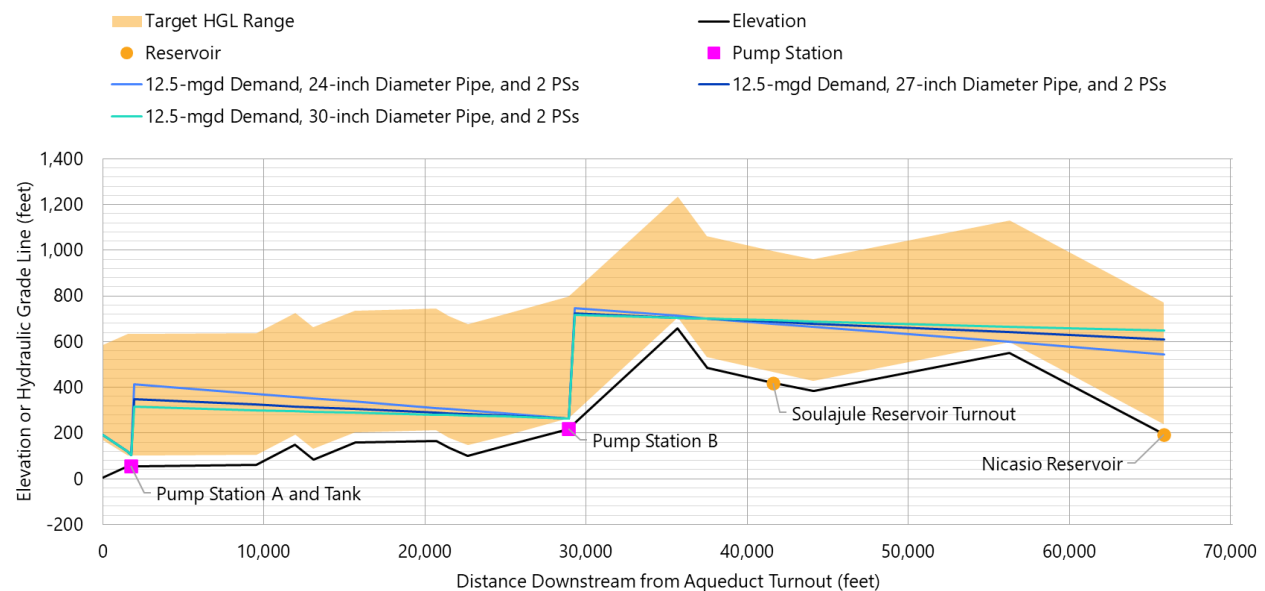


Figure 3.24 PETA-4 Dual Pump Station with 12.5 mgd Demand Hydraulic Profile

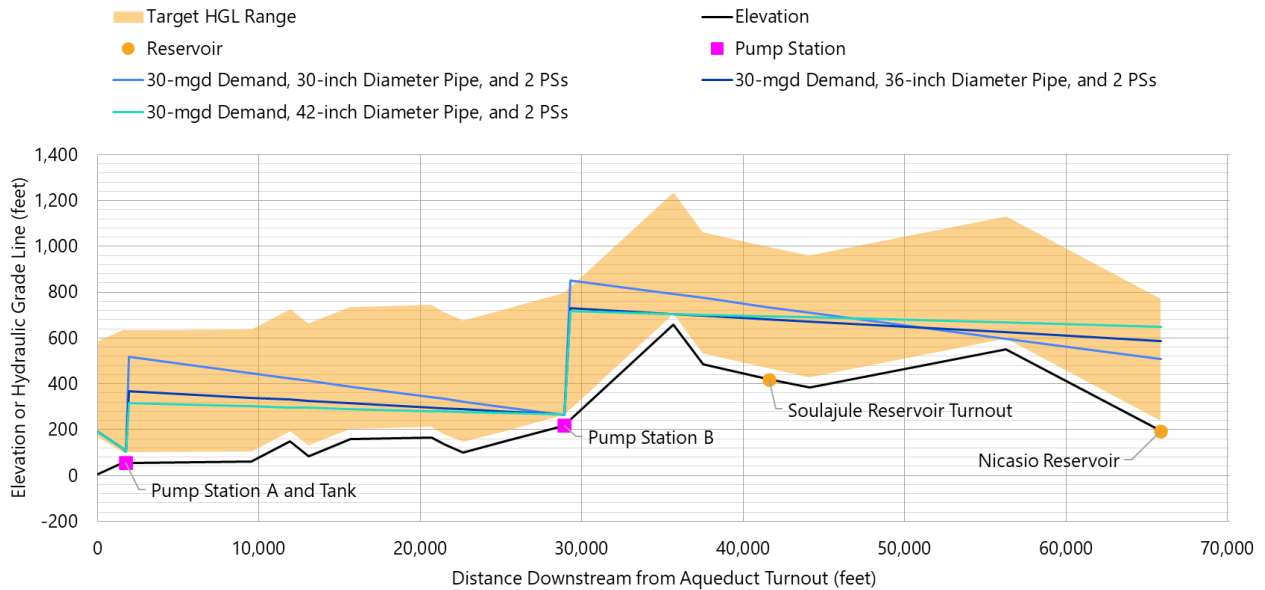


Figure 3.25 PETA-4 Dual Pump Station with 30.0 mgd Demand Hydraulic Profile

As shown in Figure 3.24, for the scenarios that assume a demand of 12.5 mgd, the HGL remains within the yellow band of the target HGL range for the proposed pipeline sizes. We recommend considering the 24-inch pipe alternative to allow pipeline pressures to remain within the target HGL range for the 12.5 mgd demand, assuming two pump stations.

As shown in Figure 3.25, for the scenarios that assume a demand of 30.0 mgd, the HGL remains within the yellow band of the target HGL range for 36-inch and 42-inch pipe. For the scenario that includes a 30-inch pipe, the HGL downstream of the proposed PS B is higher than the yellow band of the target HGL range. We recommend considering the 36-inch pipe alternative to allow pipeline pressures to remain within the target HGL range for the 30.0 mgd demand, assuming two pump stations.

3.6.3.4 Recommended Facility Sizing Based on Preliminary Hydraulics

Based on preliminary hydraulics, it is recommended that a 36-inch pipeline be installed for the PETA-4 alternative. Assuming a flow of 12.5 mgd, a 24-inch pipeline will allow the system to operate within the target HGL range with two pump stations. However, a 24-inch pipeline will not allow the conveyance system to be expanded in the future.

Assuming upstream improvements are constructed and there is a demand of 30.0 mgd, a 36-inch pipeline will allow the system to operate within the target HGL range with two pump stations. Additionally, the 36-inch pipeline will provide operational flexibility for the District in the future. Given the high pressures expected for a single pump station system, a single pump station system was not considered further.

3.6.4 Pump Station Site Layouts

3.6.4.1 Dual Pump Station System

For the dual pump station system described in Section 3.6.3.3, the first pump station site located near the North Marin Aqueduct will likely need to include a pump station building, storage tank, on-site detention basin, and other ancillary equipment. Figure 3.26 shows a general layout of the site.

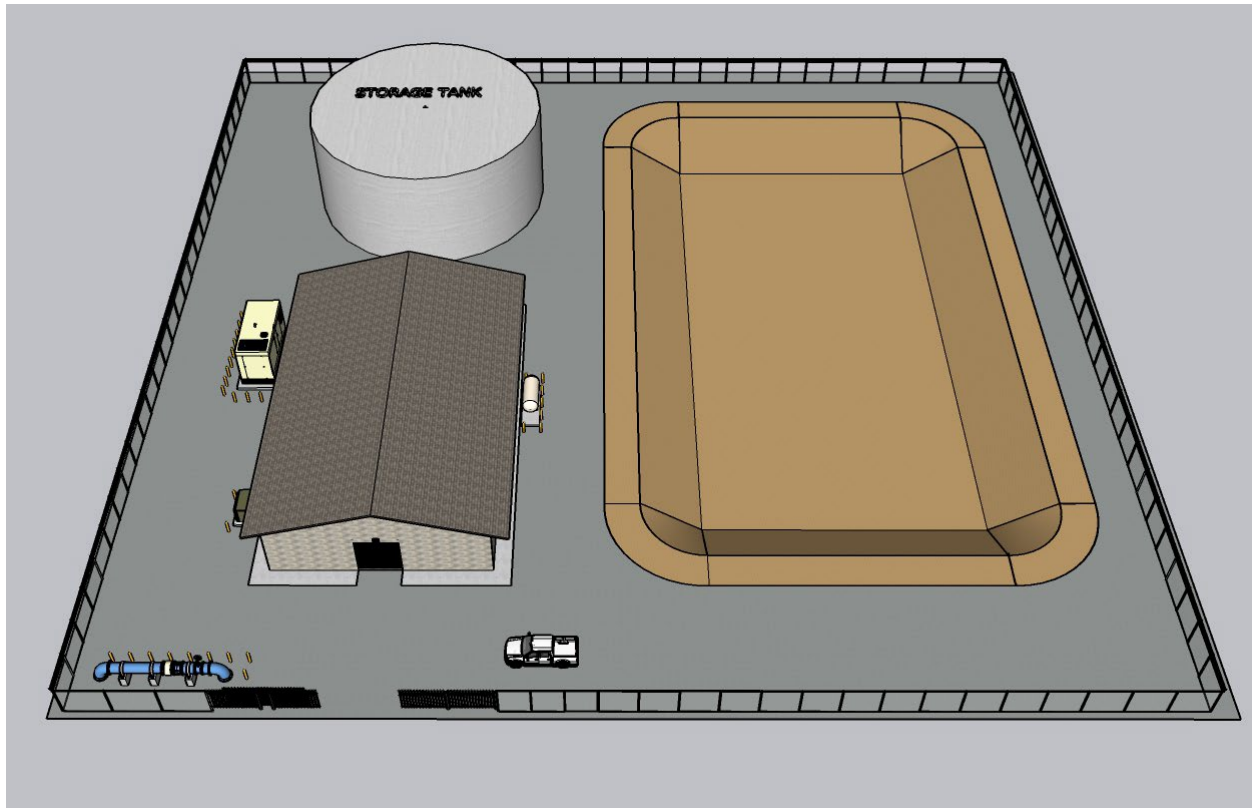


Figure 3.26 PETA-4 Pump Station A Site Layout

As discussed in Section 3.6.3.4, the pump station would likely be sized to accommodate a future capacity of 30.0 mgd. Figure 3.26 shows a typical pump station building that includes a pump mechanical room, separate electrical room, control room, and storage room. The approximate footprint of the 30.0 mgd pump station building is 120 feet by 65 feet.

The site will also need to include a storage tank. The storage tank will act as a hydraulic buffer between the North Marin Aqueduct system and the new conveyance system. This hydraulic buffer will help to separate the two systems, reducing potential issues from variations in flow, pressure and hydraulic surge in the North Marin Aqueduct. For planning purposes, a 1-MG tank is shown in Figure 3.26. However, the required capacity of the tank can vary depending on District preference and would be determined during preliminary design based on hydraulic modeling.

The site may also require an on-site detention basin. The on-site detention basin (assumed to be 1 MG to match the capacity of the storage tank) would be designed to store any overflow and/or water from draining the tank during O&M activities. The detention basin could be designed to infiltrate water into the

ground or be pumped to the storm drain system. It may also be possible to send spent water from the tank directly to the storm drain system and eliminate the on-site detention basin. The viability of this alternative would depend on the storm drain capacity and ability to accept large flows in a short duration. Discharging to a storm drain system would require dechlorination before discharge.

Assuming a 120-foot-by-65-foot pump station building, 1-MG storage tank, and 1-MG on-site detention basin, the approximate required footprint of the site is 2.1 acres (300 feet by 305 feet). However, during preliminary design, the pump station site can be optimized and reduced, particularly if the on-site detention basin is eliminated.

The second pump station site located near Red Hill will only need to include a pump station building and other ancillary equipment. Since the second pump station will be operating in series with the first pump station and receiving water from the new conveyance system, a hydraulic buffer provided by a storage tank is not needed. Accordingly, a detention basin is not needed either. Figure 3.27 shows a general layout of the site. The approximate required footprint of the site is 0.75 acres (160 feet by 200 feet).



Figure 3.27 PETA-4 Pump Station B Site Layout

3.6.5 Pump Station Siting

As discussed previously, pump station siting will largely be driven by the new conveyance system hydraulics. Potential pump station locations are discussed in more detail below. During preliminary design, the potential pump station sites will be evaluated further and a preferred pump station site will be recommended.

3.6.5.1 Pump Station A Siting

Regardless of whether a one pump system or two pump station system is chosen, PS A will need to be located near the North Marin Aqueduct. Figure 3.28 shows the approximate limits where Pump Station A can be located.

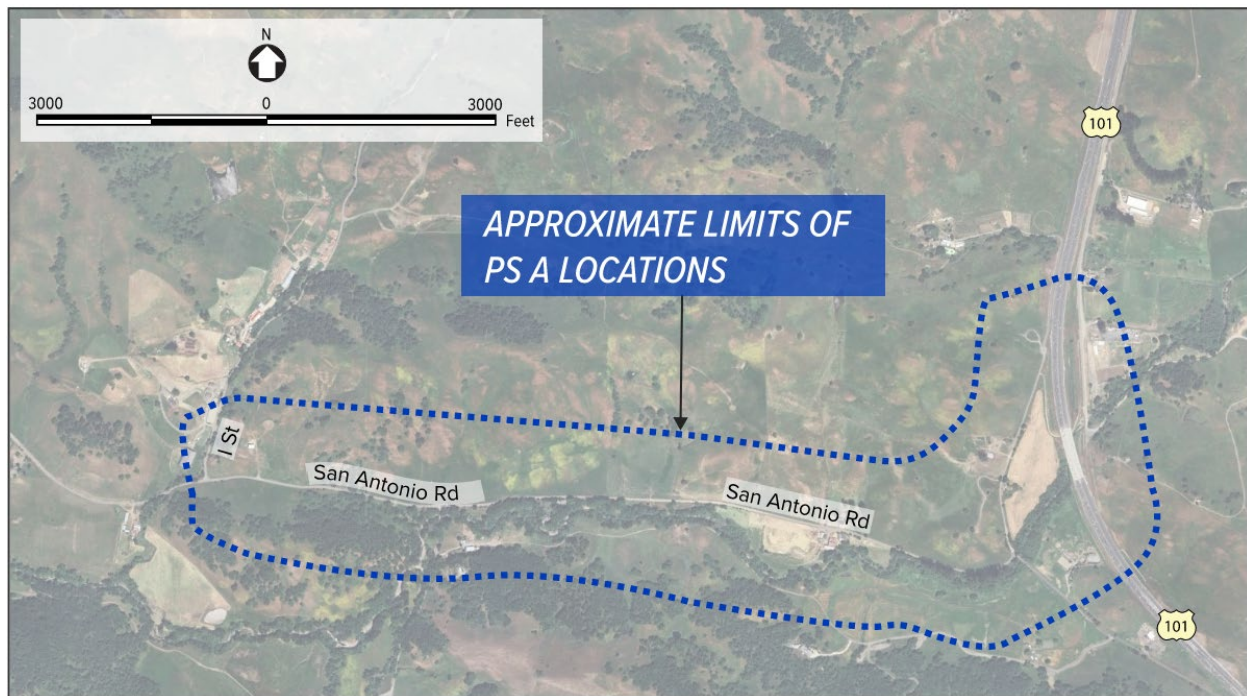


Figure 3.28 PETA-4 Pump Station A Siting

3.6.5.2 Pump Station B Siting

PS B is only needed for the dual pump station system. As previously discussed, the pump station will need to be located near Red Hill. Figure 3.29 shows the approximate limits where Pump Station B can be located.

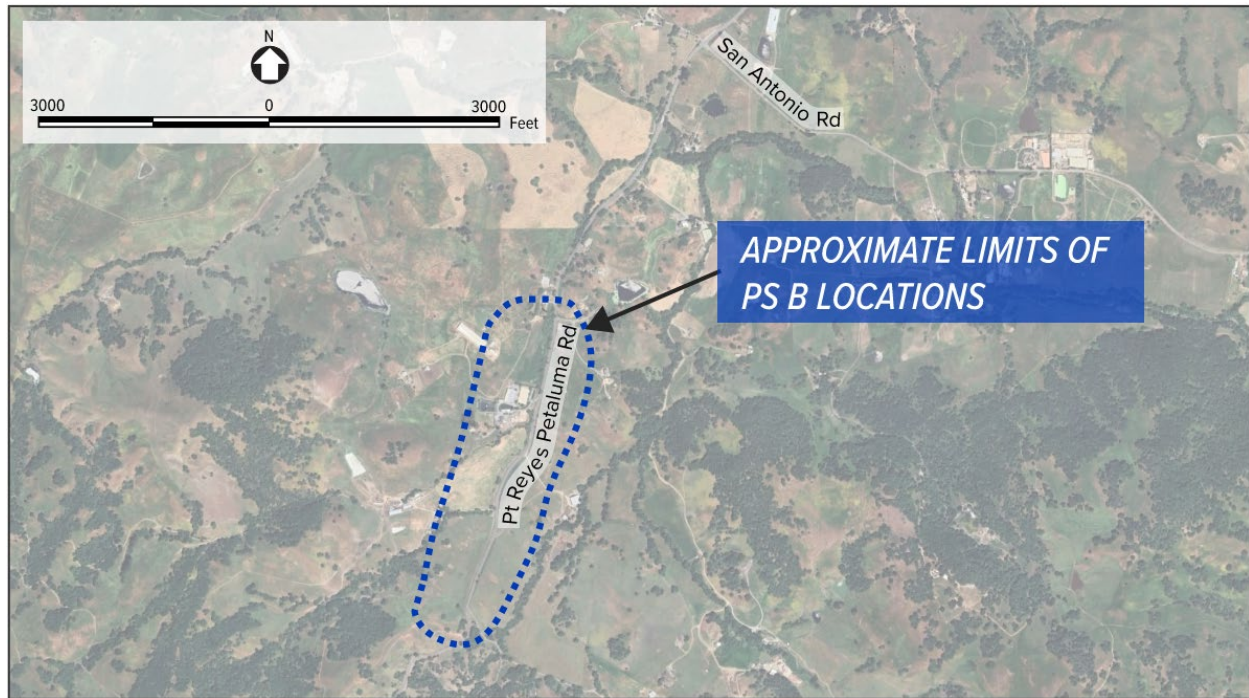


Figure 3.29 PETA-4 Pump Station B Siting

3.6.6 Environmental and Cultural Considerations

ESA conducted an environmental evaluation of the initial alternatives identified in Phase 1 of the Project, including the PETA-4 alternative. No major environmental and cultural considerations were identified in the evaluation. Refer to Appendix 3A for the complete evaluation prepared by ESA.

3.6.7 Geotechnical Considerations

MPEG performed a Geologic and Geotechnical Feasibility Evaluation of the initial alternatives identified in Phase 1 of the project, including the PETA-4 alternative. A brief summary of geologic and geotechnical issues anticipated for the PETA-4 alternative is below:

- Local landslide risks near Red Hill.
- Potential for liquefaction along major drainages, including San Antonio, Red Hill, and Hicks Valley.
- Potential for naturally occurring asbestos along Red Hill.

No major geologic or geotechnical considerations were identified in the evaluation. Refer to Appendix 3B for the complete evaluation prepared by MPEG.

3.6.8 Conservation Easements

Marin County has a significant portion of open land protected by conservation easements. A conservation easement is a legal agreement between a landowner and a conservation organization or government agency that permanently limits a property's uses to protect its conservation values. Each easement's restrictions are tailored to the particular property and to the interests of the owner and easement holder.

Figure 3.30 shows the conservation easements located along the proposed PETA-4 alignment. In general, the proposed alignment is located within the public right-of-way, with conservation easements adjacent to the right-of-way.

The one area where the proposed alignment is routed directly through conservation easement(s) is the section adjacent to Soulajule Reservoir. Through this area, there is no available public right-of-way that would allow routing of pipeline directly to the eastern finger of Soulajule Reservoir. Additionally, all land in this area is protected by conservation easements. Early coordination with the landowners and conservation organizations is needed to confirm feasibility of routing the pipeline through the conservation easement(s).

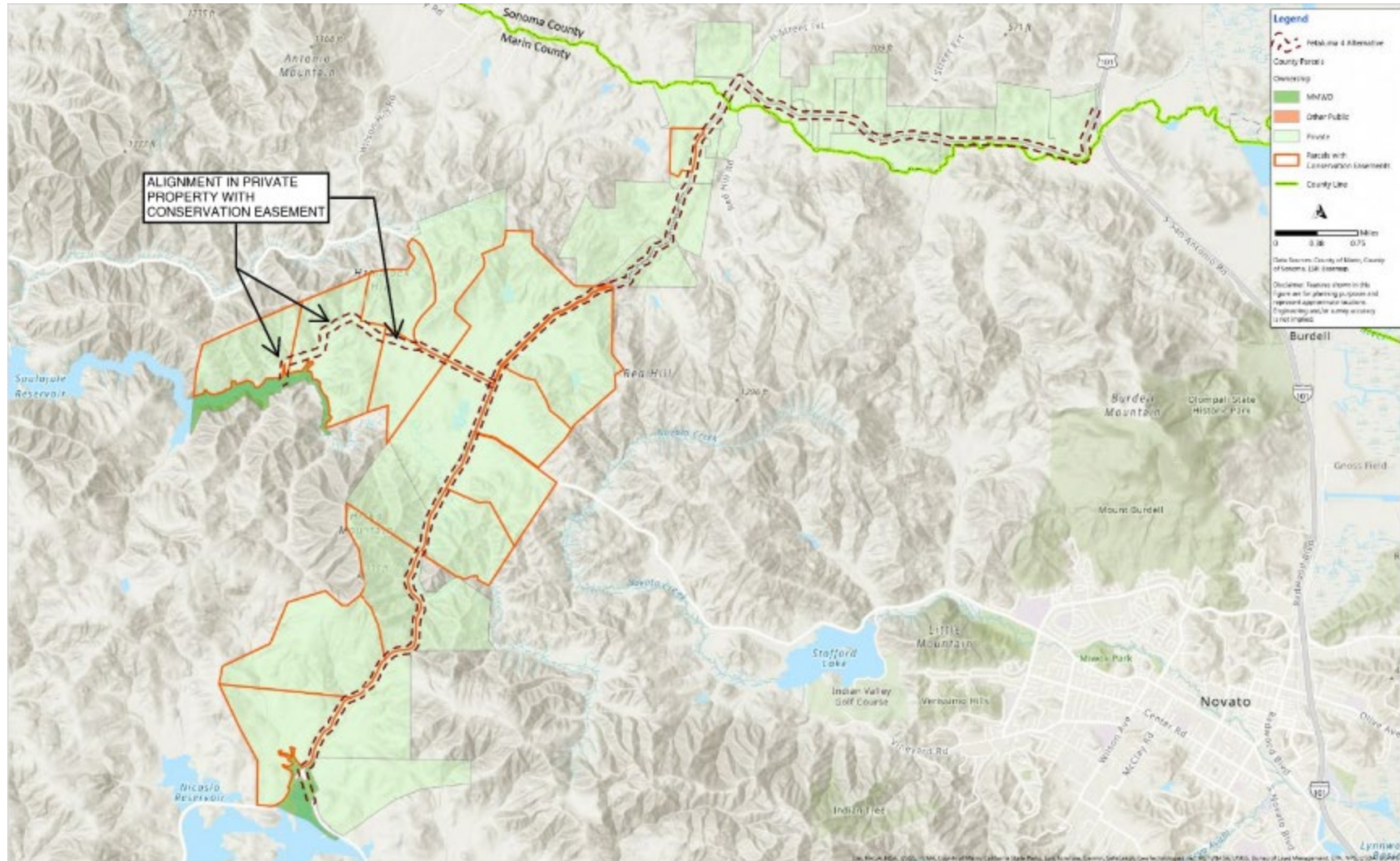


Figure 3.30 PETA-4 Conservation Easements

3.6.9 Constructability

The primary constructability concern for the PETA-4 alternative is construction of the pipeline since the alternative includes 78,000 LF of pipe. The entire pipeline alignment is located through rural roadways, many of which are just a single lane in each direction and have a minimal shoulder. To facilitate installation of the pipeline through these areas, at least one lane of the road will need to be shutdown for construction. As such, traffic impacts are expected.

Additionally, a portion of the alignment is located along Red Hill. Red Hill is a steep, curvy, narrow road that will make pipeline installation difficult. While constructable, the rate of pipeline installation along Red Hill will likely be slow.

3.6.9.1 Pipeline Installation Methods

Open-Cut Pipeline Installation

Traditional open-cut construction allows for the installation of all pipe material and joint types. Open-cut construction is the process of installing underground pipelines by excavating. A trench is excavated to the required depth for installing a pipeline with the spoil from the excavation laid alongside the trench or carted off in trucks. After the pipeline is installed on bedding, the trench is backfilled with the designed backfill material and compacted to grade.

The majority of the proposed pipeline is recommended to be installed using open-cut methods due to the speed of installation and lower cost compared to other construction alternatives. Overhead power and telecom lines are located along a majority of the proposed water supply pipeline alignment. Depending on the proximity of the open trench to nearby utility poles, coordination with utility owners for support and protection of utility poles may be required. In addition, it is standard practice to lower 20-foot pipe segments into a trench using a backhoe or other large construction equipment. Potential conflicts between large construction equipment and overhead utilities are anticipated to complicate construction at some locations along the proposed alignment.

Stream Crossings

Preliminary environmental reviews have identified approximately 77 stream crossings along the proposed PETA-4 alignment. A majority of the stream crossings are anticipated to be smaller diameter culverts and can be crossed via open-cut pipeline installation. However, several culvert crossings may require trenchless pipe installation due to the larger diameter of the culverts and challenging existing site conditions. Additionally, the pipeline alignment crosses San Antonio Creek. This crossing will likely require a significant trenchless pipe installation. There are many different types of trenchless pipe installations that could be utilized for this project, including pipe ramming, auger boring or horizontal directional drilling (HDD). The recommended method will depend on existing site conditions, including geotechnical conditions.

3.6.10 Power Availability

UEI performed a transmission and distribution power analysis study to evaluate the impacts of the proposed pump stations on PG&E's existing power grid. The goal of the study was to perform a high-level

study of the PG&E power supply availability at each pump station site to determine if there are any fatal flaws. The analysis was based on the anticipated pump station loads and proposed pump station locations.

3.6.10.1 Transmission System Analysis

The analysis of the existing transmission system utilized CAISO data to evaluate the impacts of the proposed pump station loads on the existing PG&E transmission system. For each pump station, the nearest 60-kV line was identified. Each transmission line was then tapped as the equivalent POI. A thermal analysis was then conducted for both summer and winter scenarios. For the PETA-4 alternative, PS A is located near existing 60-kV facilities. However, there are no existing 60-kV facilities located near PS B. The addition of the pump stations added minimal load contributions to the PG&E transmission system. However, under all scenarios, several facilities were determined to have preexisting thermal overloads. Further coordination with PG&E is needed to determine if the added load contributions are acceptable. Refer to Appendix 3C for the complete evaluation prepared by UEI.

3.6.10.2 Distribution System Analysis

The analysis of the existing distribution system utilized PG&E's DIDF and ICA maps to evaluate the impacts of the proposed pump station loads on the existing PG&E distribution system. For each pump station, the nearest 12-kV line was identified. For PETA-4 PS A, the nearest line is approximately 24,000 LF away. Accordingly, 24,900 LF of new 3-phase line would need to be constructed to connect the existing distribution system to the new pump station. For PETA-4 PS B, the nearest line is approximately 33,200 LF away. Accordingly, 33,200 LF of new 3-phase line would need to be constructed to connect the existing distribution system to the new pump station. Each respective distribution line was then tapped, and the proposed loads were added to the system. For the PETA-4 alternative, the addition of the proposed pump station loads did not lead to excessive loads in the distribution systems. Further coordination with PG&E is needed to determine if the added load contributions are acceptable. Refer to Appendix 3C for the complete evaluation prepared by UEI.

3.6.11 Project Phasing

There is a potential to phase the implementation of the PETA-4 alternative. We recommended that the initial phase include construction of pipelines and pump station(s) to convey water from the North Marin Aqueduct to Nicasio Reservoir. It is recommended to construct the pipeline to Nicasio Reservoir initially so that water can be utilized immediately. Directly downstream of Nicasio Reservoir, there is a water treatment plant that treats and distributes water into the District's distribution system. For all water sent to Soulajule Reservoir, the water must be pumped from Soulajule Reservoir to Nicasio Reservoir prior to being able to be utilized by the District.

3.6.11.1 PETA-4 Without Upstream Improvements

If the PETA-4 without upstream improvements alternative is selected, the next and final phase would then include the required pipeline to extend the project to Soulajule Reservoir.

3.6.11.2 PETA-4 With Upstream Improvements

If the PETA-4 with upstream improvements alternative is selected, the next phase would include construction of the upstream improvements detailed in Section 3.6.2.2. These upstream improvements include:

- Increasing the capacity of Kastania Pump Station from 25.0 mgd to 42.5 mgd.
- Constructing an STS alternative that can convey an additional 17.1 mgd from Cotati Tanks to Kastania Pump Station.

The next and final phase would then include the required pipeline to extend the project to SoulaJule Reservoir.

3.6.12 Project Cost Estimates

This section summarizes the budgetary project cost estimates for the PETA-4 Alternative

3.6.12.1 Basis for Estimate

The Project alternative is at the preliminary engineering phase with limited design information. The construction cost estimates are consistent with the AACE International Class 4 estimate with an accuracy range of +50 percent to -30 percent of the actual project cost.

Unit Costs

The project cost estimates presented are based on unit costs for the major pipeline and pump station components of the Project. The unit costs for the pipelines are based on information received from pipeline manufacturers for dollars per foot cost of the pipeline material. The pipeline installation costs are based on similar cost estimates on other recent projects. The costs for the pump stations are also based on similar cost estimates on other recent projects.

Contingencies

Contingencies are applied to a construction estimate at the design development phase to account for construction items not yet identified and design unknowns. We recommend carrying a 30 to 50 percent contingency during the planning phase of a project considering the design details are not known at this time. The contingency will be reduced as the design is developed. Agencies typically retain contingency within their project budgets, even when construction contract award values are known, to cover the cost or deal with unforeseen conditions.

The cost estimates presented in this TM include a 30 percent contingency, calculated based on the raw construction cost. This is in alignment with the recommendations for a project at an AACE International Class 4 level of development.

3.6.12.2 Cost Estimates

Table 3.6 presents a summary of the cost estimate for the PETA-4 alternative without upstream improvements. The cost estimate assumes a dual pump station system with 36-inch diameter pipe is utilized.

Table 3.6 PETA-4 Without Upstream Improvements Cost Estimate

Cost Item	Cost (rounded)
General Conditions (i.e., mobilize/demobilize, temporary facilities, testing, start-up, etc.)	\$6,100,000
Pipeline	\$64,680,000
Pump Stations	\$31,500,000
Power Supply	\$9,840,000
Ancillary Items	\$15,900,000
Direct Cost Subtotal	\$128,020,000
Contingency (30%)(1)	\$38,410,000
General Contractor Overhead, Profit, and Risk (12%)(1)	\$19,970,000
Escalation to Mid-Point (Assumes Mid-Point in April 2028) (3% per Year)(1)	\$20,970,000
Sales Tax (8.25% on Half the Direct Cost)(1)	\$8,550,000
Total Estimated Construction Cost	\$215,920,000
Estimated Construction Cost Range (-30%)	\$151,140,000
Estimated Construction Cost Range (+50%)	\$323,880,000

Notes:

(1) Indirect cost factors are applied to the running subtotal of direct costs plus preceding indirect cost; see Appendix 3E for additional information.

Table 3.7 presents a summary of the cost estimate for the PETA-4 alternative with upstream improvements. The cost estimate assumes a dual pump station system with 36-inch diameter pipe is utilized from the North Marin Aqueduct to the reservoirs, upsizing of the Kastania PS and construction of an STS alternative. A detailed breakdown of the construction cost estimates is included in Appendix 3E.

Table 3.7 PETA-4 With Upstream Improvements Cost Estimate

Cost Item	Cost (rounded)
General Conditions (i.e., mobilize/demobilize, temporary facilities, testing, start-up, etc.)	\$10,270,000
Pipeline	\$64,680,000
Pump Stations	\$40,500,000
Kastania Pump Station Improvements	\$10,000,000
STS Alternative	\$53,630,000
Power Supply	\$9,840,000
Ancillary Items	\$26,800,000
Direct Cost Subtotal	\$215,720,000
Contingency (30%)(1)	\$64,720,000
General Contractor Overhead, Profit, and Risk (12%)(1)	\$33,650,000
Escalation to Mid-Point (Assumes Mid-Point in April 2028) (3% per Year)(1)	\$35,340,000
Sales Tax (8.25% on Half the Direct Cost)(1)	\$14,410,000
Total Estimated Construction Cost	\$363,840,000
Estimated Construction Cost Range (-30%)	\$254,690,000
Estimated Construction Cost Range (+50%)	\$545,760,000

Notes:

(1) Indirect cost factors are applied to the running subtotal of direct costs plus preceding indirect cost; see Appendix 3E for additional information.

3.7 Evaluation of COTATI-3: Cotati Tanks to Nicasio and Soulajule Reservoirs

3.7.1 Introduction

The COTATI-3: Cotati Tanks to Nicasio and Soulajule Reservoirs alternative includes two new pump stations and approximately 25.9 miles of pipeline to convey water from the Cotati Tanks to Nicasio and Soulajule Reservoirs.

This alternative will connect to Sonoma Water's system at the Cotati Tanks. The alternative requires approximately 10.1 miles of new pipeline routed along mostly rural two-lane roadways, including Stony Point Road, Mecham Road, Pepper Road, Bodega Avenue, and Spring Hill Road. Along Spring Hill Road, the proposed pipeline alignment leaves the right-of-way and heads due south through private property adjacent Laguna Lake. The proposed pipeline alignment parallels Laguna Lake before returning to the right-of-way at Chileno Valley Road. The proposed pipeline alignment then follows Chileno Valley Road, Wilson Hill Road, and Hicks Valley Road for approximately 5.2 miles. At the intersection of Hicks Valley Road and 3 Peaks Fire Road, the pipeline will bifurcate with one branch routed west approximately 2.0 miles along the fire road to Soulajule Reservoir. The other branch will continue along Hicks Valley Road and Pt. Reyes - Petaluma Road for approximately 5.3 miles to Nicasio Reservoir. Figure 3.31 shows the proposed pipeline alignment and profile. Refer to Appendix F for more detailed plan view drawings of the proposed pipeline alignment.

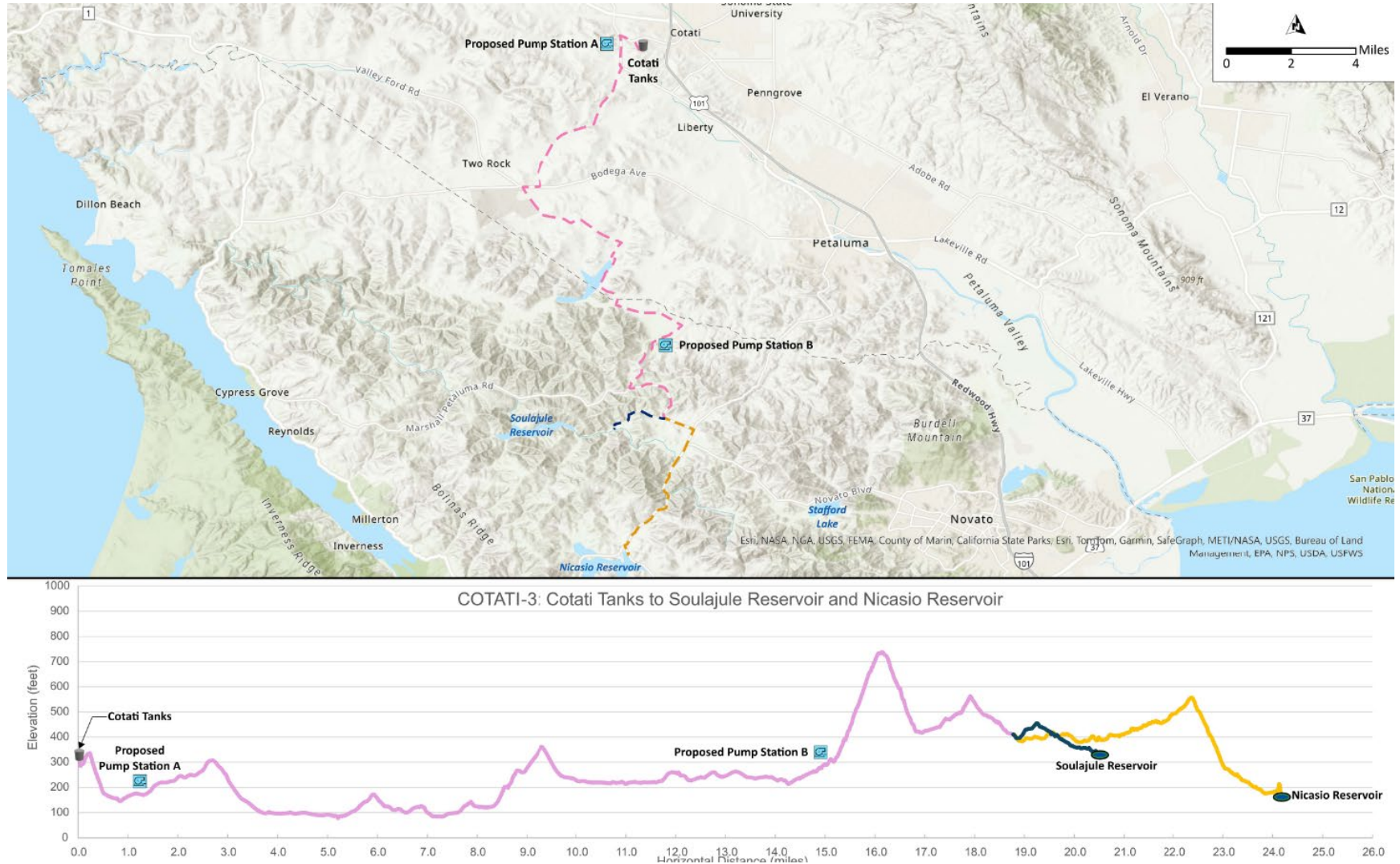


Figure 3.31 COTATI-3 Alignment Overview

For this alternative, the additional water supply will be conveyed to Nicasio and SoulaJule Reservoirs for storage. The District will have the ability to select which reservoir to fill based on reservoir levels, demands within the system, and operational considerations. By utilizing both reservoirs, the District can increase the amount of water transferred from Sonoma Water.

3.7.2 Water System Supply Benefits

As the COTATI-3 alternative includes new infrastructure that is largely independent of the existing system, the sizing of the facilities and associated water supply benefits can be defined by the District. Based on discussions with the District, this alternative would likely be sized to convey a maximum flow of approximately 30.0 mgd.

Figure 3.32 shows a schematic of the existing system from Cotati Tanks to Ignacio Pump Station. The schematic is diagrammatic and does not show all existing or proposed facilities. The proposed turnout location is shown in green.

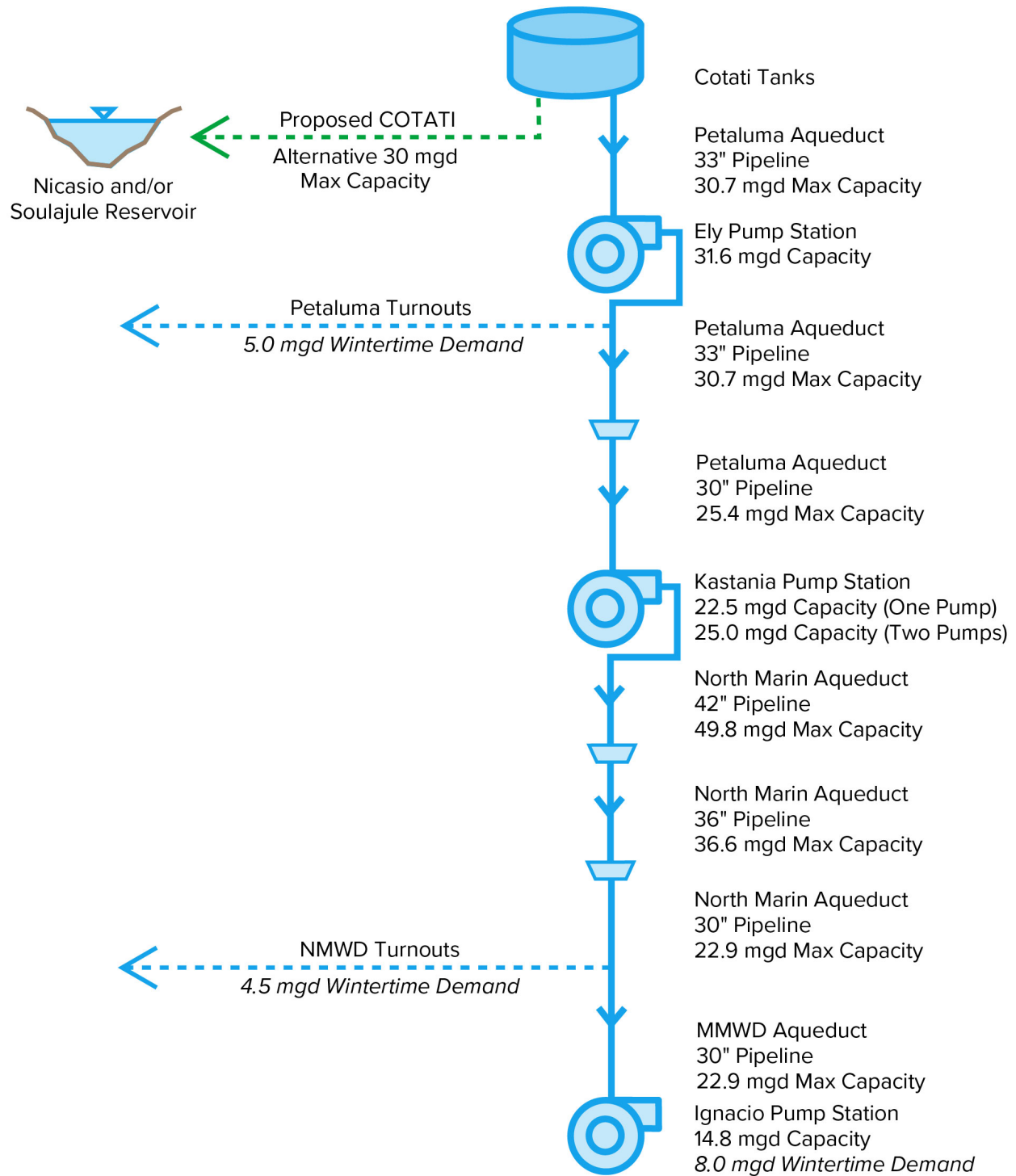


Figure 3.32 COTATI-3 Schematic

3.7.3 COTATI-3 Preliminary Hydraulics

3.7.3.1 COTATI-3 Hydraulic Modeling Assumptions

The system hydraulics of the COTATI-3 alternative were evaluated to consider two potential scenarios. The first scenario assumes a single pump station is located near the Cotati Tanks that pumps to the Nicasio and Soulajule Reservoirs. The second scenario considers two pump stations with one pump station located near the Cotati Tanks and the other located near the intersection of Chileno Valley Road and Wilson Hill Road. The two pump stations would operate in series to pump water from the North Marin Aqueduct to the reservoirs.

For both scenarios, the first pump station would need to be located near the Cotati Tanks. However, the precise location of the first pump station is flexible. The Cotati Tanks are located on a hill and have a minimum water surface elevation of 343 feet. The proposed pipeline alignment directly downstream of the tanks drops in elevation. The static head from the water level in the tanks can be utilized to allow the pump station to be located slightly downstream of the Cotati Tanks. Further discussion of pump station siting is included in Section 3.7.5. Further discussion of hydraulic modeling assumptions is included in Appendix 3D.

3.7.3.2 Single Pump Station System

The single pump station system would pump from near the Cotati Tanks (PS A), over the high points along the proposed alignment, to the reservoirs. The highest point along the proposed alignment is located near Wilson Hill at elevation 735 feet. The system hydraulics for several different pipe sizes and capacities were evaluated based on this requirement, including:

- 30-inch diameter pipe with a demand of 30.0 mgd.
- 36-inch diameter pipe with a demand of 30.0 mgd.
- 42-inch diameter pipe with a demand of 30.0 mgd.

Figure 3.33 shows the elevation profile and HGL for the pipe diameters and demands. Figure 3.33 also includes a yellow band that shows the target HGL range for the scenarios. The target HGL range shown is the sum of the elevation along the pipeline alignment plus a pipeline pressure ranging from 20 psi to 250 psi. In general, pipelines are designed to operate within this band of pressures ranging from 20 psi to 250 psi. While feasible, pipelines and pump stations with pressures exceeding 250 psi require specialty piping and appurtenances, leading to increased design complexity and construction costs.

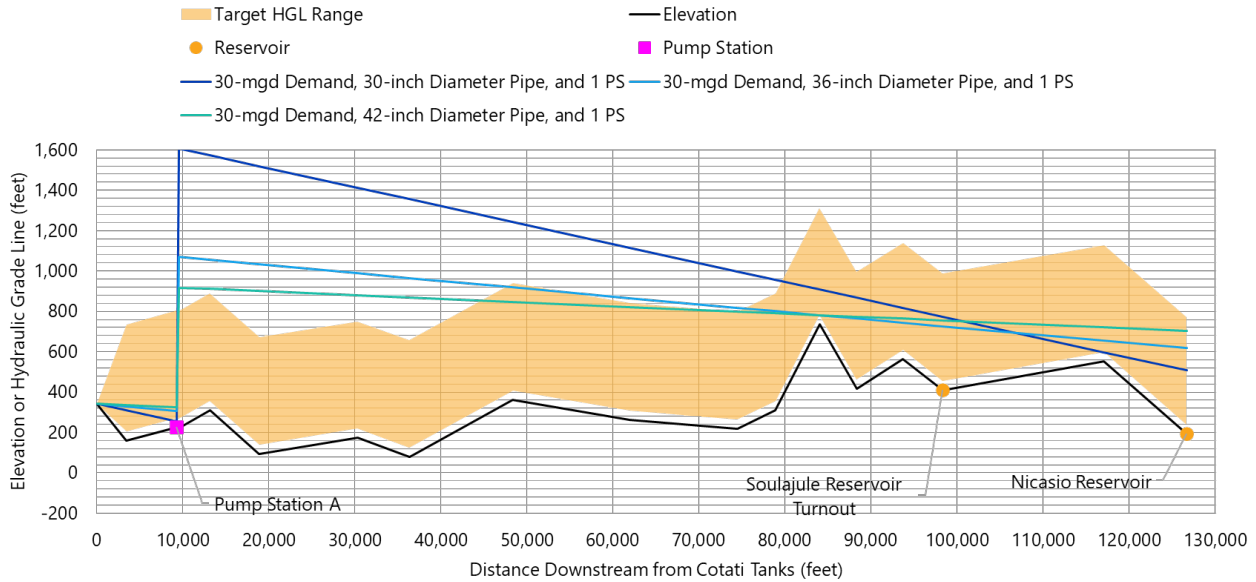


Figure 3.33 COTATI-3 Single Pump Station Hydraulic Profile

As shown in Figure 3.33, for each scenario, the HGL downstream of the proposed PS A is higher than the yellow band of the target HGL range. This means that pressures in this area exceed 250 psi. Accordingly, a conveyance system with a single pump station would have pipeline pressures that exceed the target HGL range regardless of the pipeline diameter.

3.7.3.3 Dual Pump Station System

Due to the significant head requirements anticipated for the PETA-3 alternative, a dual pump station system was also considered. The two pump stations would operate in series to pump water from the North Marin Aqueduct to the reservoirs. More specifically, the dual pump station system would pump from the pump station located near the Cotati Tanks (PS A) to the pump station near the intersection of Chileno Valley Road and Wilson Hill Road (PS B). PS B would then pump water over the high points to the reservoirs. The highest point along the proposed alignment is located near Wilson Hill at elevation 735 feet. The system hydraulics for several different scenarios were evaluated based on this requirement, including:

- 30-inch diameter pipe with a demand of 30.0 mgd.
- 36-inch diameter pipe with a demand of 30.0 mgd.
- 42-inch diameter pipe with a demand of 30.0 mgd.

Figure 3.34 shows the elevation profile and HGL for these scenarios. Figure 3.34 also includes a yellow band that shows the target HGL range for the scenarios. The target HGL range shown is the sum of the elevation along the pipeline alignment plus a pipeline pressure ranging from 20 psi to 250 psi. In general, pipelines are designed to operate within this band of pressures ranging from 20 psi to 250 psi.

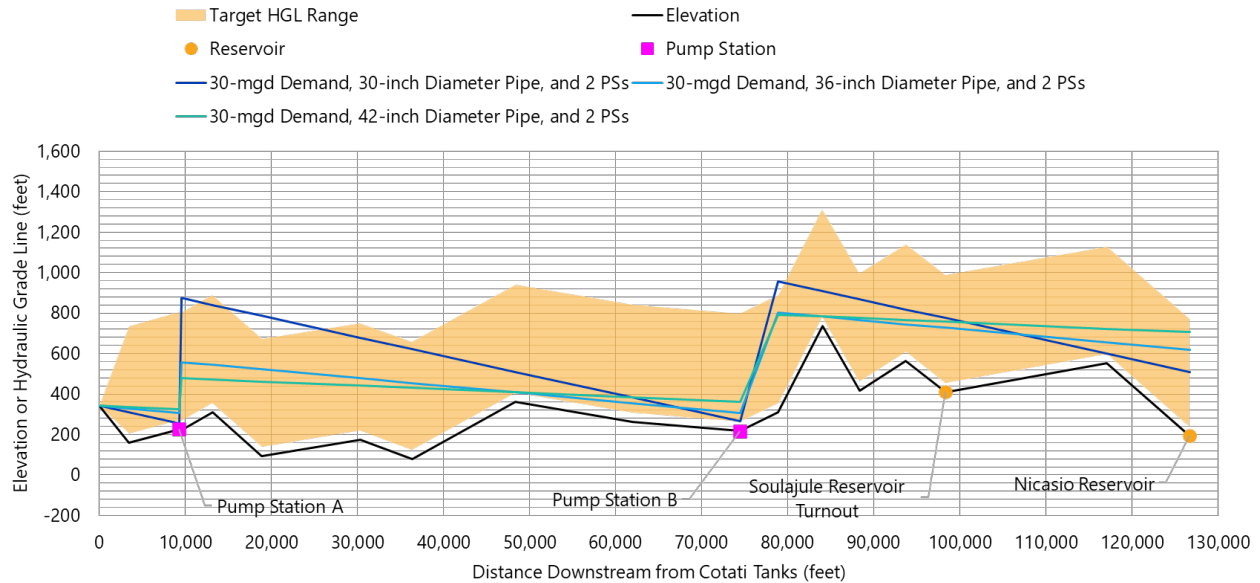


Figure 3.34 COTATI-3 Dual Pump Station Hydraulic Profile

As shown in Figure 3.34, for the scenario that includes a 30-inch pipe, the HGL downstream of the proposed PS B is higher than the yellow band of the target HGL range. This means that pressures in this area exceed 250 psi. However, for each of the other two scenarios, the HGL downstream of the proposed pump stations stays within the yellow band of the target HGL range. We recommend considering the 36-inch pipe alternative to allow pipeline pressures to remain within the target HGL range for the assumed demand (30.0 mgd), assuming two pump stations.

3.7.3.4 Recommended Facility Sizing Based on Preliminary Hydraulics

Based on preliminary hydraulics, it is recommended that a 36-inch pipeline be installed for the COTATI-3 alternative. Assuming a demand of 30.0 mgd, a 36-inch pipeline will allow the system to operate within the target HGL range with two pump stations. Given the high pressures expected for a single pump station system, a single pump station system was not considered further.

3.7.4 Pump Station Site Layouts

3.7.4.1 Dual Pump Station System

For the two pump station system described in Section 3.7.3.3, the first pump station site located near the Cotati Tanks will just need to include a pump station building and other ancillary equipment. Figure 3.35 shows a general layout of the site.



Figure 3.35 COTATI-3 Pump Stations A and B Site Layout

Figure 3.35 shows a typical pump station building that includes a pump mechanical room, separate electrical room, control room, and storage room. The approximate footprint of the 30.0 mgd pump station building is 120 feet by 65 feet.

The site will not need to include a storage tank or on-site detention basin. That is because water from the Cotati Tanks will flow by gravity to PS A. The Cotati Tanks will provide the fixed head required for the pump station to always maintain a flooded suction line. Accordingly, a detention basin is not needed either.

As shown in Figure 3.35, assuming a 120-foot-by-65-foot pump station building and other ancillary equipment, the approximate required footprint of the site is 0.75 acres (160 feet by 200 feet). However, during preliminary design, the pump station site can be optimized and reduced.

The second pump station site located near the intersection of Chileno Valley Road and Wilson Hill Road will also only need to include a pump station building and other ancillary equipment. Since the second pump station will be operating in series with the first pump station and receiving water from the new

conveyance system, a hydraulic buffer provided by a storage tank is not needed. Accordingly, a detention basin is not needed either. The general layout of the site will match the general layout of PS A, shown in Figure 3.35.

3.7.5 Pump Station Siting

As discussed previously, pump station siting will largely be driven by the new conveyance system hydraulics. Potential pump station locations are discussed in more detail below. During preliminary design, the potential pump station sites will be evaluated further, and a preferred pump station site will be recommended.

3.7.5.1 Pump Station A Siting

Regardless of whether a one pump system or two pump station system is chosen, PS A will need to be located near the Cotati Tanks. Figure 3.36 shows the approximate limits where Pump Station A can be located.

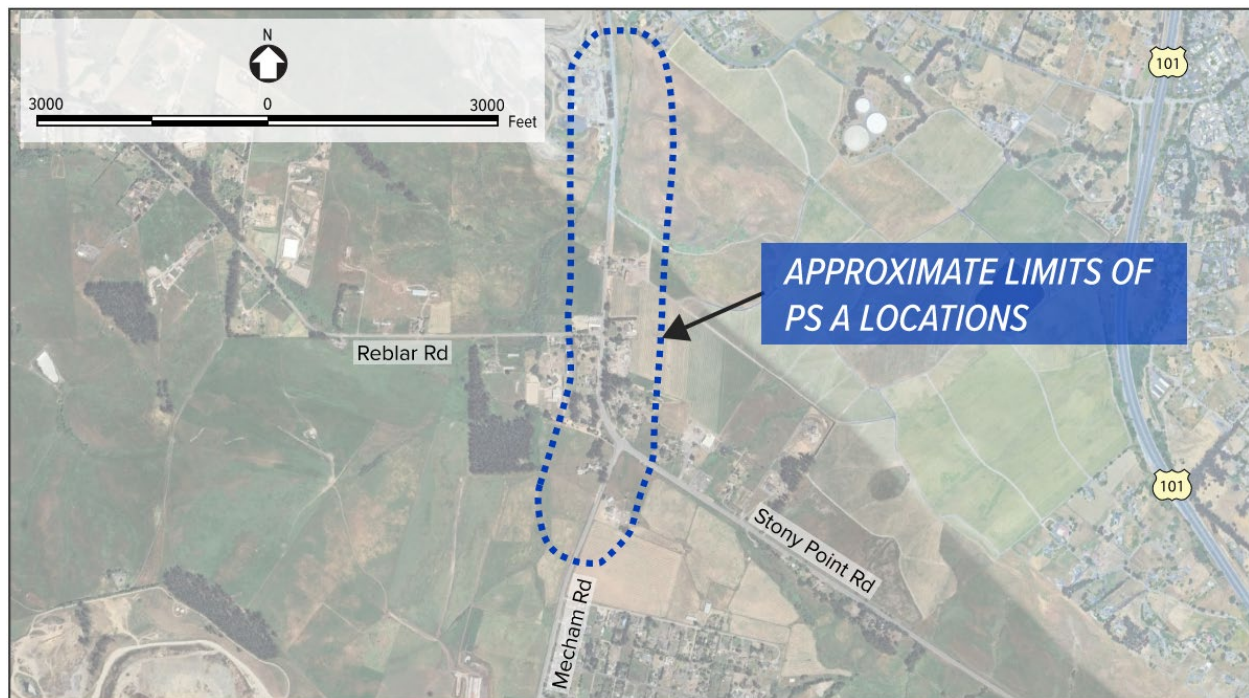


Figure 3.36 COTATI-3 Pump Station A Siting

3.7.5.2 Pump Station B Siting

PS B is only needed for the dual pump station system. As previously discussed, the pump station will need to be located near the intersection of Chileno Valley Road and Wilson Hill Road. Figure 3.37 shows the approximate limits where Pump Station B can be located.

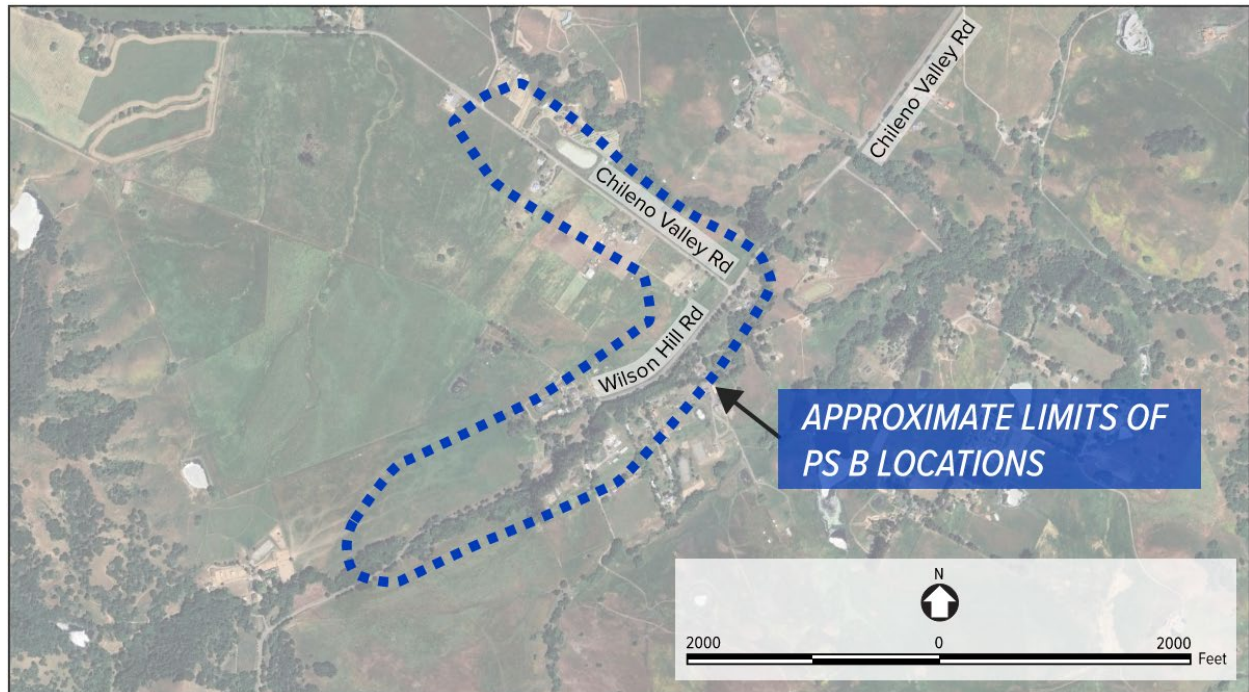


Figure 3.37 COTATI-3 Pump Station B Siting

3.7.6 Environmental and Cultural Considerations

ESA conducted an environmental evaluation of the initial alternatives identified in Phase 1 of the Project, including the COTATI-3 alternative. One consideration identified by ESA was that the proposed pipeline alignment is routed through wetlands near Laguna Lake. ESA recommended that alternative alignments through this area be evaluated if the COTATI-3 alternative is selected. No other major environmental and cultural considerations were identified in the evaluation. Refer to Appendix 3A for the complete evaluation prepared by ESA.

3.7.7 Geotechnical Considerations

MPEG performed a Geologic and Geotechnical Feasibility Evaluation of the initial alternatives identified in Phase 1 of the project, including the COTATI-1 alternative and COTATI-2 alternative which are the two components of the COTATI-3 alternative. A brief summary of geologic and geotechnical issues anticipated for the COTATI-3 alternative is below:

- Local landslide risks near Wilson Hill.
- Potential for liquefaction along major drainages, including Meachem, Pepper, and Hicks Valley.
- Potential for naturally occurring asbestos along Wilson Hill.

No major geologic or geotechnical considerations were identified in the evaluation. Refer to Appendix 3B for the complete evaluation prepared by MPEG.

3.7.8 Conservation Easements

Marin County has a significant portion of open land protected by conservation easements. A conservation easement is a legal agreement between a landowner and a conservation organization or government agency that permanently limits a property's uses to protect its conservation values. Each easement's restrictions are tailored to the particular property and to the interests of the owner and easement holder.

Figure 3.38 shows the conservation easements located along the proposed COTATI-3 alignment. In general, the proposed alignment is located within the public right-of-way, with conservation easements adjacent to the right-of-way.

The one area where the proposed alignment is routed directly through conservation easement(s) is the section adjacent to Soulajule Reservoir. Through this area, there is no available public right-of-way that would allow routing of pipeline directly to the eastern finger of Soulajule Reservoir. Additionally, all land in this area is protected by conservation easements. Early coordination with the landowners and conservation organizations is needed to confirm feasibility of routing the pipeline through the conservation easement(s).

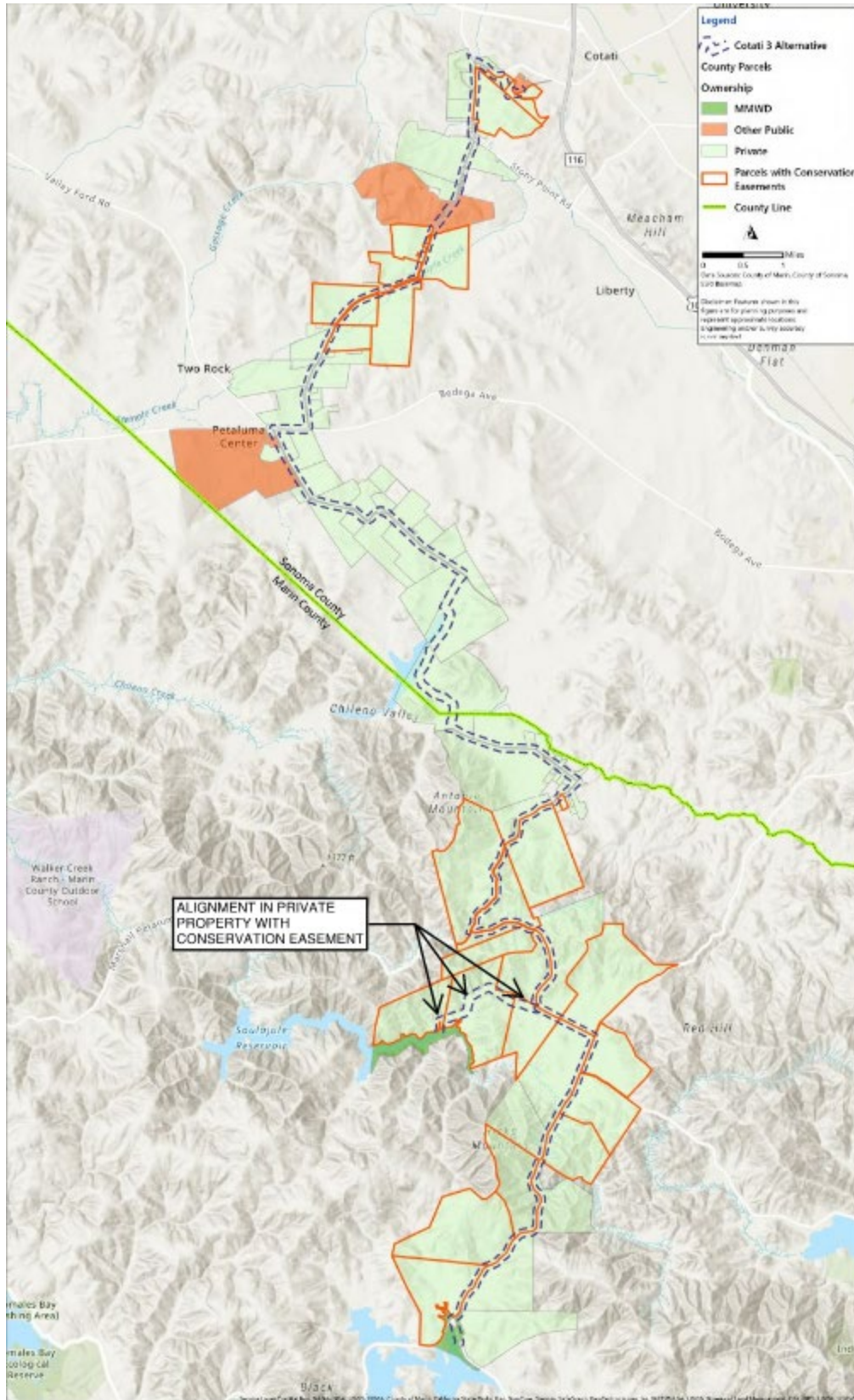


Figure 3.38 COTATI-3 Conservation Easements

3.7.9 Constructability

The primary constructability concern for the COTATI-3 alternative is construction of the pipeline since the alternative includes 137,000 LF of pipe. The entire pipeline alignment is located through rural roadways, many of which are just a single lane in each direction and have a minimal shoulder. To facilitate installation of the pipeline through these areas, at least one lane of the road will need to be taken. As such, traffic impacts are expected.

Additionally, a portion of the alignment is located along Wilson Hill. Wilson Hill is a steep, curvy, narrow road that will make pipeline installation difficult. While constructable, the rate of pipeline installation along Wilson Hill will likely be very slow.

3.7.9.1 Pipeline Installation Methods

Open-Cut Pipeline Installation

Traditional open-cut construction allows for the installation of all pipe material and joint types. Open-cut construction is the process of installing underground pipelines by excavating. A trench is excavated to the required depth for installing a pipeline with the spoil from the excavation laid alongside the trench or carted off in trucks. After the pipeline is installed on bedding, the trench is backfilled with the designed backfill material and compacted to grade.

The majority of the proposed pipeline is recommended to be installed using open-cut methods due to the speed of installation and lower cost compared to other construction alternatives. Overhead power and telecom lines are located along a majority of the proposed water supply pipeline alignment. Depending on the proximity of the open trench to nearby utility poles, coordination with utility owners for support and protection of utility poles may be required. In addition, it is standard practice to lower 20-foot pipe segments into a trench using a backhoe or other large construction equipment. Potential conflicts between large construction equipment and overhead utilities are anticipated to complicate construction at some locations along the proposed alignment.

Stream Crossings

Preliminary environmental reviews have identified approximately 85 stream crossings along the proposed COTATI-3 alignment. A majority of the stream crossings are anticipated to be smaller diameter culverts and can be crossed via open-cut pipeline installation. However, several culvert crossings may require trenchless pipe installation due to the larger diameter of the culverts and challenging existing site conditions. There are many different types of trenchless pipe installations that could be utilized for this project, including pipe ramming, auger boring or horizontal directional drilling (HDD). The recommended method will depend on existing site conditions, including geotechnical conditions.

3.7.9.2 Pavement Moratoriums

As discussed in Section 3.7.1, the majority of the proposed COTATI-3 pipeline alignment is routed along rural County roads. Several of these roads in Sonoma County have recently been paved, including Stony Point Road and Spring Hill Road. More specifically, Stony Point Road was last paved in 2023, and Spring Hill Road was last paved in 2022.

Sonoma County prohibits pavement cuts on County roads that have been paved within the previous five calendar years. As such, should this alternative be selected and should construction be scheduled to begin prior to 2028, early coordination with Sonoma County is needed to determine acceptability of pipeline installation within the recently paved roads.

3.7.10 Power Availability

UEI performed a transmission and distribution power analysis study to evaluate the impacts of the proposed pump stations on PG&E's existing power grid. The goal of the study was to perform a high-level study of the PG&E power supply availability at each pump station site to determine if there are any fatal flaws. The analysis was based on the anticipated pump station loads and proposed pump station locations.

3.7.10.1 Transmission System Analysis

The analysis of the existing transmission system utilized CAISO data to evaluate the impacts of the proposed pump station loads on the existing PG&E transmission system. For each pump station, the nearest 60-kV line was identified. Each respective transmission line was then tapped as the equivalent POI. A thermal analysis was then conducted for both summer and winter scenarios. For the COTATI-3 alternative, PS A is located near existing 60-kV facilities. However, there are no existing 60-kV facilities located near PS B. The addition of the pump stations added minimal load contributions to the PG&E transmission system. However, under all scenarios, several facilities were determined to have preexisting thermal overloads. Further coordination with PG&E is needed to determine if the added load contributions are acceptable. Refer to Appendix 3C for the complete evaluation prepared by UEI.

3.7.10.2 Distribution System Analysis

The analysis of the existing distribution system utilized PG&E's DIDF and ICA maps to evaluate the impacts of the proposed pump station loads on the existing PG&E distribution system. For each pump station, the nearest 12-kV line was identified. For COTATI-3 PS A, the nearest line is approximately 1,000 LF away. Accordingly, 1,000 LF of new 3-phase line would need to be constructed to connect the existing distribution system to the new pump station. For COTATI-3 PS B, the nearest line is approximately 36,000 LF away. Accordingly, 36,000 LF of new 3-phase line would need to be constructed to connect the existing distribution system to the new pump station. Each respective distribution line was then tapped, and the proposed loads were added to the system. For the COTATI-3 alternative, the addition of the proposed pump station loads did not lead to excessive loads in the distribution systems. Further coordination with PG&E is needed to determine if the added load contributions are acceptable. Refer to Appendix 3C for the complete evaluation prepared by UEI.

3.7.11 Project Phasing

The potential to phase the implementation of the project for the COTATI-3 alternative is limited. Although there are a couple potential options for project phasing, it is recommended that the initial phase include construction of pipelines and pump station(s) to convey water from the Cotati Tanks to Nicasio Reservoir. The next and final phase would then include the required pipeline to extend the project to Soulajule Reservoir.

It is recommended to construct the pipeline to Nicasio Reservoir initially so that water can be utilized immediately. Directly downstream of Nicasio Reservoir, there is a water treatment plant that treats and distributes water into the District's distribution system. For all water sent to Soulajule Reservoir, the water must be pumped from Soulajule Reservoir to Nicasio Reservoir prior to being able to be utilized by the District.

3.7.12 Project Cost Estimates

This section summarizes the budgetary project cost estimates for the COTATI-3 alternative.

3.7.12.1 Basis for Estimate

The Project alternative is at the preliminary engineering phase with limited design information. The construction cost estimate is consistent with the AACE International Class 4 estimate with an accuracy range of +50 percent to -30 percent of the actual project cost.

Unit Costs

The project cost estimate presented is based on unit costs for the major pipeline and pump station components of the Project. The unit costs for the pipelines are based on information received from pipeline manufacturers for dollars per foot cost of the pipeline material. The pipeline installation costs are based on similar cost estimates on other recent projects. The costs for the pump stations are also based on similar cost estimates on other recent projects.

Contingencies

Contingencies are applied to a construction estimate at the design development phase to account for construction items not yet identified and design unknowns. We recommend carrying a 30 to 50 percent contingency during the planning phase of a project considering the design details are not known at this time. The contingency will be reduced as the design is developed. Agencies typically retain contingency within their project budgets, even when construction contract award values are known, to cover the cost or deal with unforeseen conditions.

The cost estimate presented in this TM includes 30 percent contingency, calculated based on the raw construction cost. This is in alignment with the recommendations for a project at an AACE International Class 4 level of development.

3.7.12.2 Cost Estimates

Table 3.8 presents a summary of the cost estimate for the COTATI-3 alternative. The cost estimate assumes a dual pump station system with 36-inch diameter pipe is utilized. A detailed breakdown of the construction cost estimate is included in Appendix 3E.

Table 3.8 Alternatives Cost Estimate

Cost Item	Cost (rounded)
General Conditions (i.e., mobilize/demobilize, temporary facilities, testing, start-up, etc.)	\$9,450,000
Pipeline	\$108,420,000
Pump Stations	\$49,500,000
Power Supply	\$6,440,000
Ancillary Items	\$24,660,000
Direct Cost Subtotal	\$198,470,000
Contingency (30%)(1)	\$59,540,000
General Contractor Overhead, Profit, and Risk (12%)(1)	\$30,960,000
Escalation to Mid-Point (Assumes Mid-Point in April 2028) (3% per Year)(1)	\$32,510,000
Sales Tax (8.25% on Half the Direct Cost)(1)	\$13,260,000
Total Estimated Construction Cost	\$334,740,000
Estimated Construction Cost Range (-30%)	\$234,310,000
Estimated Construction Cost Range (+50%)	\$502,100,000

Notes:

(1) Indirect cost factors are applied to the running subtotal of direct costs plus preceding indirect cost; see Appendix 3E for additional information.

3.8 Other Design Considerations

For the three alternatives discussed in detail above, there are several project components that will be similar for each. Project components that will be similar for each alternative include the dechlorination facilities, among others.

3.8.1 Dechlorination

The treated water that is received from Sonoma Water will be chlorinated. Prior to discharge of this treated water to Nicasio and Soulajule Reservoir, the water must be dechlorinated. Water must be dechlorinated because chlorinated water can damage the ecosystem surrounding the reservoirs, including causing harm to plants and animals. In order to dechlorinate the water, a chemical such as calcium thiosulfate will likely be injected into the water supply, removing the chlorine.

The dechlorination facility will likely include a storage tank to hold the required chemical, chemical metering pumps, analyzers, and other ancillary items. Figure 3.39 shows the proposed layout of a dechlorination facility.

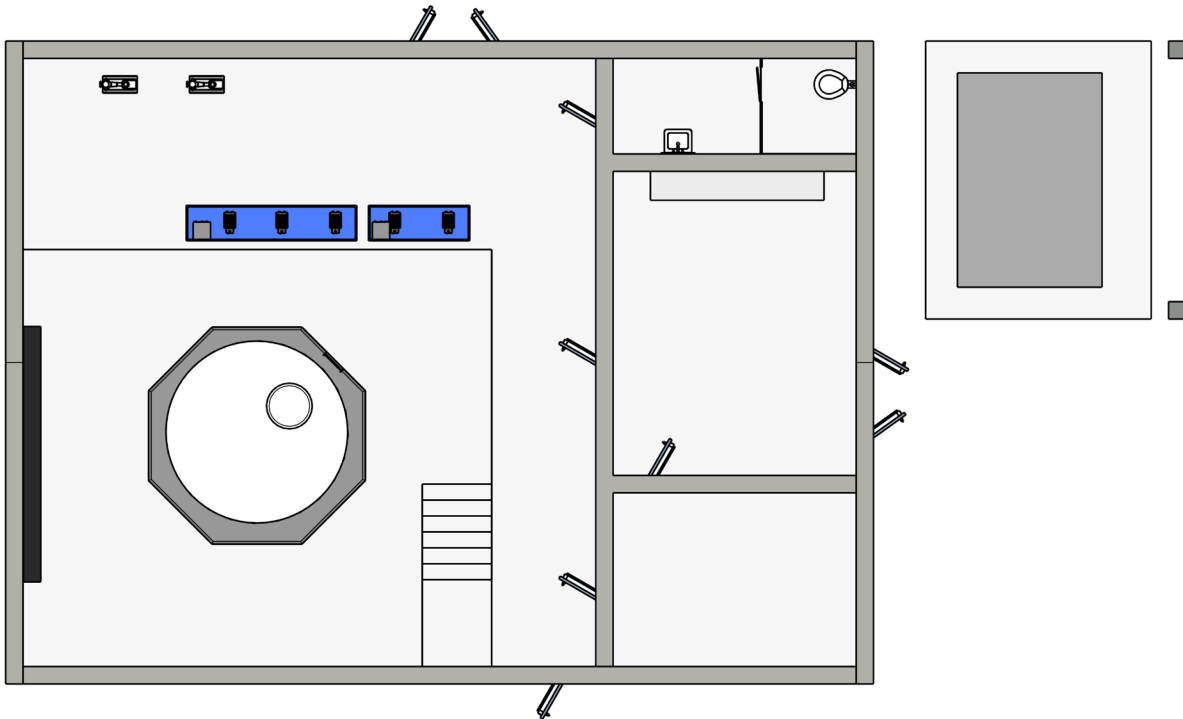


Figure 3.39 Dechlorination Facility

In general, the dechlorination facility should be located near the discharge locations at the reservoirs. This would allow the water to maintain a chlorine residual for as long as possible. Maintaining a chlorine residual in the pipelines will greatly reduce the potential for bacteriological growth within the pipeline. Instead of locating dechlorination facilities at both reservoirs, it may be more economical to have a single dechlorination facility located at the junction point where the pipeline bifurcates, with one branch going to Nicasio Reservoir and the other branch going to SoulaJule Reservoir. With a dechlorination facility located at the junction point, the remaining run of the pipe to each reservoir is short enough that bacteriological growth should be minimal. Further evaluation of the dechlorination facility and proposed siting should be performed during preliminary design.

3.9 Alternatives Evaluation

3.9.1 Alternatives Evaluation Methodology

The Phase 2 alternatives evaluation method and criteria use a similar evaluation method and criteria to that used during the Phase 1 analysis. This provides consistency between the evaluations. The Phase 1 evaluation is described in detail in "TM 2 – Screening Criteria and Evaluation".

The scoring definitions for the evaluation criteria were updated slightly from the Phase 1 evaluation. The updates include further definition of several of the evaluation criteria. Updated scoring definitions for the evaluation criteria are included in Table 3.9.

Table 3.9 Evaluation Criteria Scoring Definitions

Score	Water Reliability and Sustainability	Flexibility and Resiliency	Schedule and Implementation	Water Quality	Environmental and Social Stewardship	Economic and Financial
5	- Contributes to a sustainable dry year water supply by significantly increasing system's capacity to transfer water from Sonoma Water. - Improves reliability of transmission infrastructure and meets long-term operational needs under maximum- and average-day demand conditions. - Greatly improves the ability of system to withstand natural disasters.	- Greatly improves flexibility of District operations. - Greatly improves the resilience of District operations from changing conditions, including climate change effect. - Integrates well with regional water systems and maximizes regional benefits. - Minimizes complexities of system operations and hydraulics.	- Ability to implement project in the fastest timeline. - Reduce unnecessary regulatory complexity. - Preserves many future conveyance project options. - Minimizes construction complexities. - Minimizes new PG&E facilities required to obtain power for pump stations. - Adequate land available at proposed pump station locations.	- Enhances water quality. - Meets current and foreseeable future drinking water regulations with a margin of safety.	- Minimizes long-term environmental effects. - Minimizes disruption to existing land uses to the extent practical. - Minimizes land disturbance required at proposed pump station locations.	- Minimizes capital and life cycle costs relative to yield. - Integrates well with existing infrastructure. - Leverage federal and state grant and loan opportunities. - Minimizes financial risk.
3	- Contributes to a sustainable dry year water supply by moderately increasing system's capacity to transfer water from Sonoma Water. - Improves reliability of transmission infrastructure and meets long-term operational needs under average-day demand conditions. - Improves the ability of system to withstand natural disasters.	- Improves flexibility of District operations. - Improves the resilience of District operations from changing conditions, including climate change effect. - Integrates with regional water systems, providing some regional benefits. - Some complexities with system operations and hydraulics.	- Ability to implement project in a reasonable timeline. - Some regulatory complexity. - Preserves some future conveyance project options. - Some construction complexities. - Some new PG&E facilities required to obtain power for pump stations. - Some land available at proposed pump station locations.	Meets current drinking water regulations.	- Some long-term environmental effects. - Some disruption to existing land uses. - Some land disturbance required at proposed pump station locations.	- Moderate capital and life cycle cost relative to yield. - Integrates with some existing infrastructure. - Leverages some federal and state grant and loan opportunities. - Moderate financial risk.
1	- Contributes to a sustainable dry year water supply by minimally increasing system's capacity to transfer water from Sonoma Water. - Doesn't improve reliability of transmission infrastructure that meets long-term operational needs under average-day demand conditions. - Doesn't improve the ability of system to withstand natural disasters.	- Doesn't improve flexibility of District operations. - Doesn't improve the resilience of District operations from changing conditions, including climate change effect. - Doesn't integrate with regional water system, providing minimal regional benefits. - Significant complexities with system operations and hydraulics.	- Project complexities cause project to be implemented in the slowest timeline. - Significant regulatory complexities. - Reduces future conveyance project options. - Significant construction complexities. - Significant new PG&E facilities required to obtain power for pump stations. - Minimal land available at proposed pump station locations.	Alternative may expose water supply to lower quality surface water during storage.	- Substantial long-term environmental effects. - Substantial disruption to existing land uses. - Substantial land disturbance required at proposed pump station locations.	- High capital and life cycle cost relative to yield. - Does not integrate with existing infrastructure. - Doesn't leverage federal and state grant and loan opportunities. - High financial risk.

The next step is to score each alternative based on the scoring definitions defined in Table 3.9. Table 3.9 defines a score of 1, 3, or 5 for each criteria. It should be noted that the higher scores are better than lower scores (a score of 5 is better than a 1). Although TM 2 also scored alternatives, including the three selected alternatives, on a scale from 1 to 5, scoring of the alternatives for this TM has been updated. Scoring updates are based on further evaluation and understanding of the three selected alternatives.

After scoring each criteria, the criteria were weighted by the project team based on relative importance on a scale from 1 to 5. If the District considers some of the criteria to be of equal weight, the criteria were weighted equally in the matrix. Although TM 2 also weighted the evaluation criteria, the weighting of some criteria for this TM has been updated based on updated understanding of relative importance of project components.

The final step of the evaluation was to summarize the results in an Alternative Evaluation Matrix. The alternative with the highest total score will be advanced into Phase 3 of the Project for preliminary design.

3.9.2 Alternatives Screening Results

This section summarizes the alternative screening results. The following alternatives were reviewed and scored:

- PETA-3 Without Upstream Improvements
- PETA-3 With Upstream Improvements
- PETA-4 Without Upstream Improvements
- PETA-4 With Upstream Improvements
- COTATI-3

Table 3.10 shows the Alternatives Evaluation Matrix to summarize the scoring and weighting of each alternative. Table 3.10 includes a summation of the total scores for each alternative. Based on the total scores included in Table 3.10, the PETA-3 alternative ranks the highest and we recommend advancing PETA-3 to preliminary design.

Table 3.10 Alternatives Evaluation Matrix

Criteria	Water Reliability and Sustainability			Flexibility and Resiliency				Schedule and Implementation						Water Quality	Environmental and Stewardship			Economic and Financial					Total Score
	Contribute to a sustainable dry year water supply	Provide reliable water supply, treatment and transmission infrastructure that meets long-term operational needs under average- and maximum-day demand conditions	Improve the ability of water system to withstand natural hazards	Retain and/or improve flexibility of District operations	Improve the resilience of District operations from changing conditions, including climate change effects	Integrate well with regional water systems	Minimize complexities of system operations and hydraulics	Implement the project in a timely manner, consistent with framework identified in the SWSA	Reduce regulatory complexity	Preserve options for future conveyance systems between Cotati Tanks and the District's facilities	Minimize construction complexities	Minimize new PG&E facilities required to obtain power for pump stations	Adequate land available at proposed pump station locations	Design improvements to meet current and foreseeable future drinking water regulations with a margin of safety	Minimize long-term environmental impacts relative to other conveyance alternatives	Minimize disruption to existing land uses to the extent practical	Minimize land disturbance required at proposed pump station locations	Minimize capital costs for the District's customers relative to yield	Minimize life cycle costs for the District's customers relative to other conveyance alternatives	Integrates well with existing infrastructure	Leverage federal and state investments in infrastructure	Minimize financial risk	
Criteria Weighting	5	3	3	3	3	4	4	3	3	3	1	3	3	2	2	2	2	4	4	4	5	2	
PETA-3 Without Upstream Improvements	3	3	3	3	3	3	3	5	3	5	5	5	1	5	3	5	5	3	3	5	3	3	238
PETA-3 With Upstream Improvements	5	5	5	5	5	5	1	1	3	5	3	5	1	5	1	3	5	1	1	5	3	3	234
PETA-4 Without Upstream Improvements	3	3	3	3	3	3	3	5	3	5	5	1	3	5	3	5	3	3	3	5	3	3	228
PETA-4 With Upstream Improvements	5	5	5	5	5	5	1	1	3	5	3	1	3	5	1	3	3	1	1	5	3	3	224
COTATI-3	5	5	5	5	3	3	1	3	3	3	3	3	5	5	1	3	3	1	1	1	3	3	206

3.10 Summary

3.10.1 Recommendations

This TM summarizes the Phase 2 evaluation of the PETA-3, PETA-4, and COTATI-3 alternatives. Summaries of the PETA-3, PETA-4, and COTATI-3 alternatives are included in Figure 3.40, Figure 3.41, and Figure 3.42, respectively.

This TM also describes the criteria and methodology used to evaluate the three project alternatives and identify a preferred alternative. Based on the evaluation methodology, the PETA-3 alternative ranked the highest and we recommend advancing PETA-3 to preliminary design.

3.10.2 Next Steps

We recommend the following next steps for this Project:

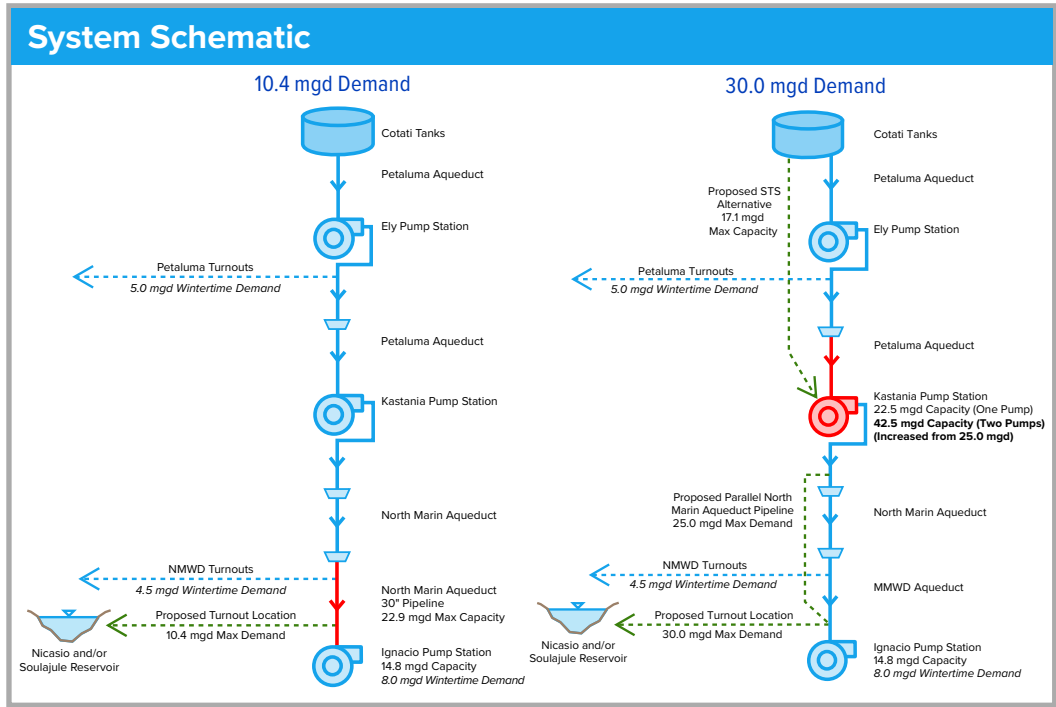
- Finalize the evaluation of water supply needs.
- Develop the contract with Sonoma Water Agency to confirm the available water supply.
- Present the recommended Project to the District's Board.
- Investigate the potential pump station sites in more detail to identify the preferred site and discuss property acquisition with the property owners.
 - » If the property is available, and the District is proceeding with the Project, we recommend purchasing the pump station site(s).
- Begin discussions with property owners that have conservation easements located along the PETA-3 alternative to understand easement restrictions and other potential impacts to the Project.
- Begin preliminary design of the PETA-3 alternative. The preliminary design should include:
 - » California Environmental Quality Act (CEQA) review of the project alternative.
 - » Topographic survey of the project area.
 - » Geotechnical investigation.
 - » 30 percent design drawings and specifications.
 - » Budgetary construction cost estimate.
 - » Detailed project design and implementation schedule.

PETA-3:

North Marin Aqueduct to Nicasio and Soulaजूle Reservoirs

Project Description

This alternative includes new pump station(s) and approximately 16.2 miles of pipeline as shown to convey water from the North Marin Aqueduct to Nicasio and Soulaजूle Reservoirs. This alternative utilizes the existing Petaluma/North Marin Aqueduct to convey water from Cotati Tanks to the proposed turnout location.



Water Supply Benefits

PETA-3 Without Upstream Improvements: 10.4 mgd

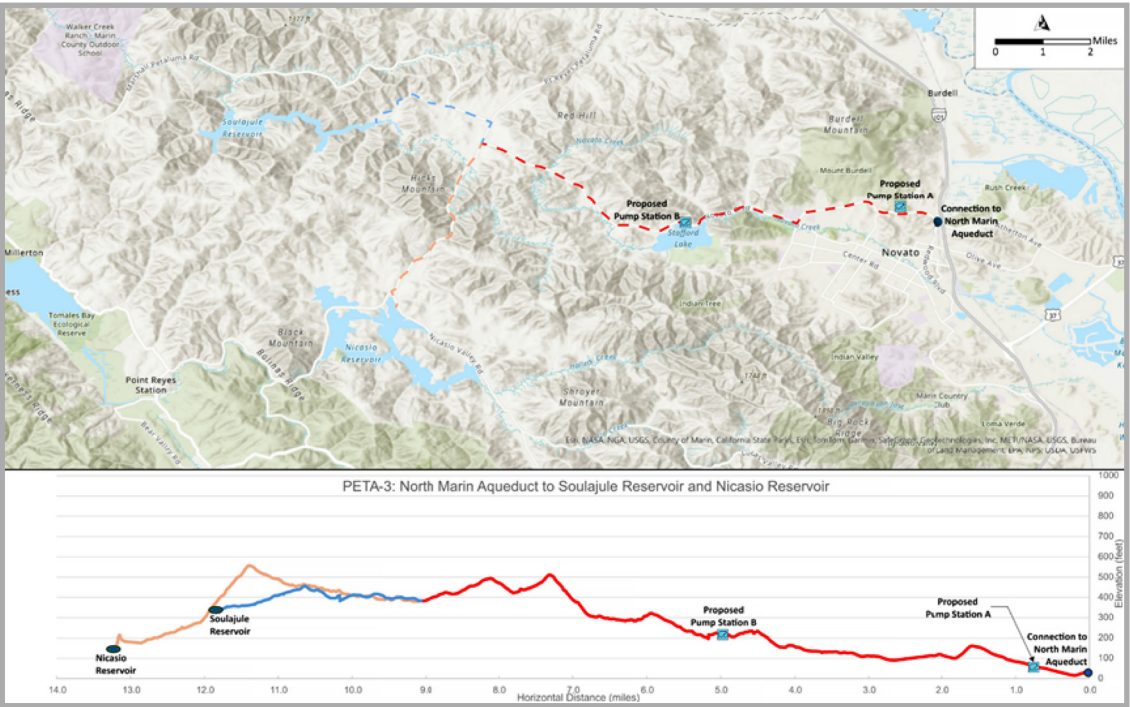
- Additional yield available if MMWD takes less through Ignacio PS

PETA-3 With Upstream Improvements: 30.0 mgd

- Required upstream improvements include:
 - Increasing the capacity of Kastania PS from 25.0 mgd to 42.5 mgd
 - Constructing a STS alternative that can convey an additional 17.1 mgd from Cotati Tanks to Kastania PS
 - Constructing a 30-inch pipeline parallel the reduced diameter sections of the North Marin Aqueduct

Constructability Challenges

- Crowded utility corridor along San Marin Dr.
- Significant number of stream crossings that may require trenchless pipe installations
- Local landslide risks near Stafford Lake
- Limited land availability to locate PS's
- Existing loads on PG&E facilities in the area are significant



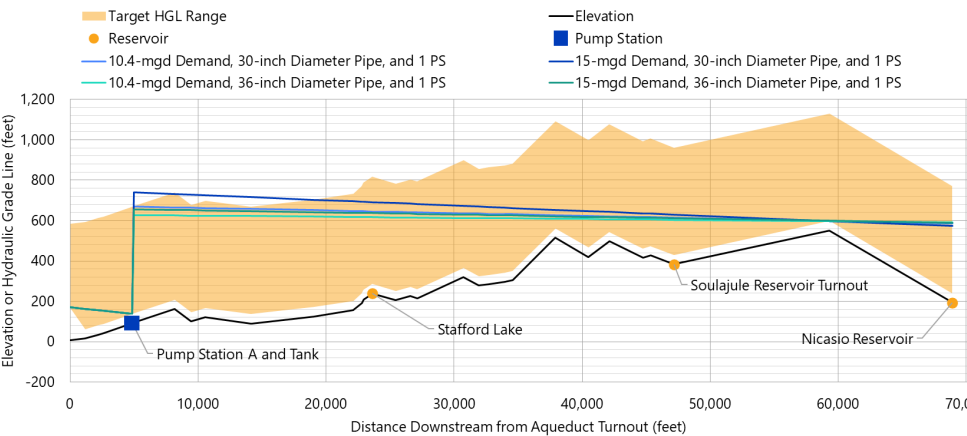
2024 TOTAL ESTIMATED COST WITHOUT UPSTREAM IMPROVEMENTS

Cost Item	Cost (rounded)
General Conditions (I.e., Mob/Demob, Temporary Facilities, Testing, Start-Up, Etc)	\$5,200,000
Pipeline	\$71,560,000
Pump Stations	\$18,000,000
Power Supply	\$850,000
Ancillary Items	\$13,560,000
Direct Cost Subtotal	\$109,170,000
Total Estimated Construction Cost Including Indirect Costs	\$184,130,000
<i>Estimated Construction Cost Range (-30%)</i>	<i>\$128,890,000</i>
<i>Estimated Construction Cost Range (+50%)</i>	<i>\$276,190,000</i>

2024 TOTAL ESTIMATED COST WITH UPSTREAM IMPROVEMENTS

Cost Item	Cost (rounded)
General Conditions (I.e., Mob/Demob, Temporary Facilities, Testing, Start-Up, Etc)	\$10,507,000
Pipeline	\$71,560,000
Pump Stations	\$33,500,000
Kastania Pump Station Imprvoements	\$10,000,000
STS Alternative	\$53,630,000
North Marin Aqueduct Parallel Pipeline	\$12,990,000
Power Supply	\$1,050,000
Ancillary Items	\$27,409,500
Direct Cost Subtotal	\$220,646,500
Total Estimated Construction Cost Including Indirect Costs	\$372,146,000
<i>Estimated Construction Cost Range (-30%)</i>	<i>\$260,502,000</i>
<i>Estimated Construction Cost Range (+50%)</i>	<i>\$558,219,000</i>

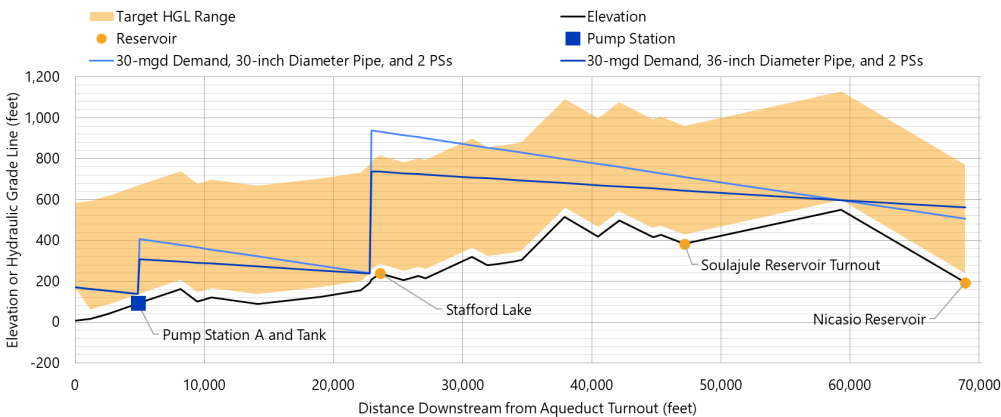
System Hydraulics



Single Pump Station Hydraulics

The system hydraulics were evaluated for two potential pumping configurations. The first configuration assumes a single pump station located near the North Marin Aqueduct that would pump water to the reservoirs. The second configuration includes two pump stations with one located near the North Marin Aqueduct and the other located near Stafford Lake. These two pump stations would operate in series to pump water from the North Marin Aqueduct to the reservoirs.

The system hydraulics graphics shown include a yellow band that shows the target HGL range for each scenario. The target HGL range shown is the sum of the elevation along the pipeline alignment plus a pipeline pressure ranging from 20 psi to 250 psi. In general, pipelines are designed to operate within this band of pressures. While feasible, pipelines and pump stations with pressures exceeding 250 psi require specialty piping and appurtenances, leading to increased design complexity and construction costs. Based on the preliminary hydraulics, it is recommended that a 36-inch pipeline be installed for the PETA-3 alternative. As shown, assuming a demand of 10.4 and 15.0 mgd, a 36-inch pipeline will allow the system to operate with a single pump station. Additionally, as shown, assuming upstream improvements are constructed and there is a demand of 30.0 mgd, a 36-inch pipeline will allow the system to operate with two pump stations.



Dual Pump Station Hydraulics

PETA-4:

North Marin Aqueduct to Nicasio and SoulaJule Reservoirs via San Antonio Road

Water Supply Benefits

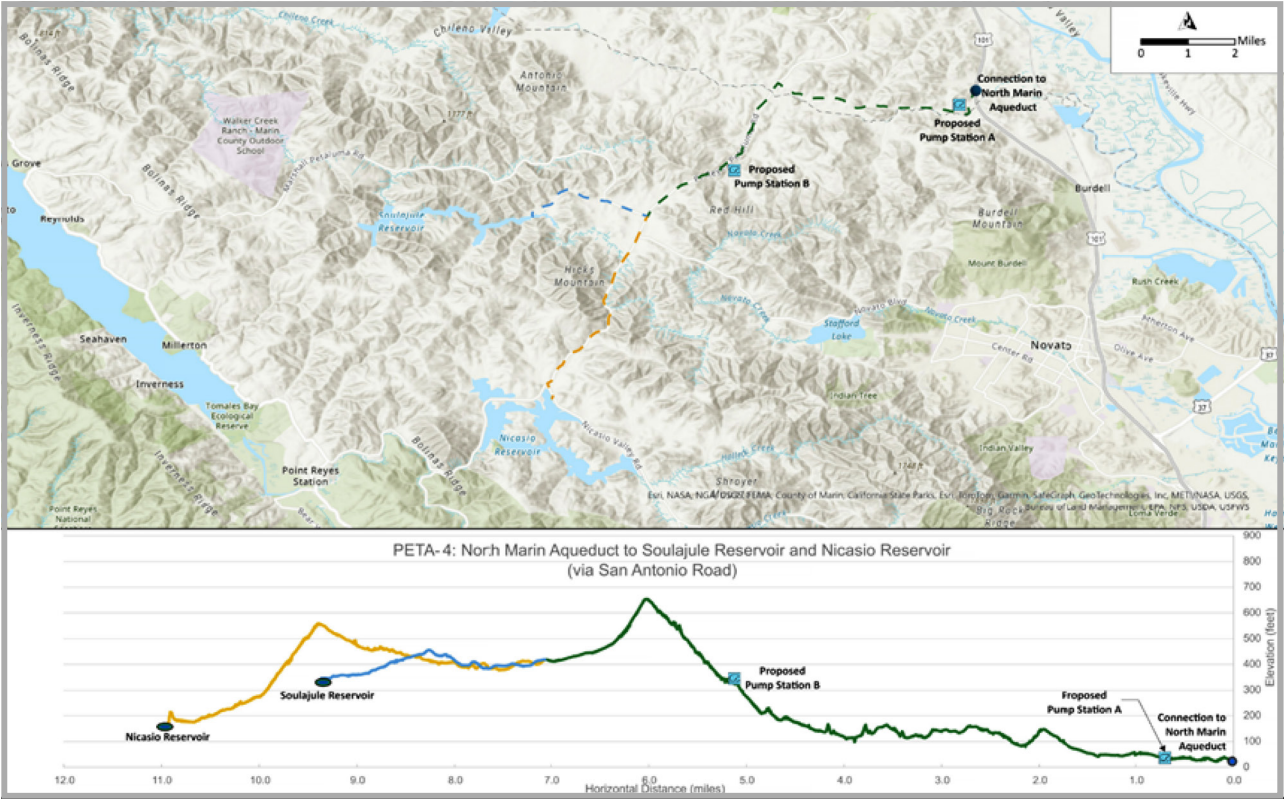
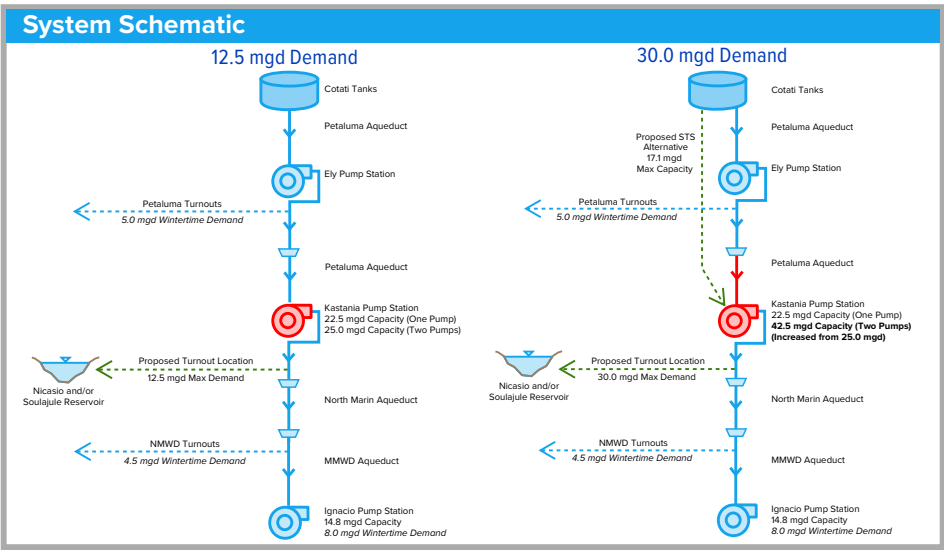
- PETA-4 Without Upstream Improvements: 12.5 mgd**
- Additional yield available if MMWD takes less through Ignacio PS
- PETA-4 With Upstream Improvements: 30.0 mgd**
- Required upstream improvements include:
 - Increasing the capacity of Kastania PS fro25.0 mgd to 42.5 mgd
 - Constructing a STS alternative that can convey an additional 17.1 mgd from Cotati Tanks to Kastania PS

Constructability Challenges

- Portion of pipeline alignment is routed along steep, curvy, narrow County roads
- Significant number of stream crossings that may require trenchless pipe installations, including San Antonio Creek
- Local landslide risks near Red Hill
- Limited existing power facilities located near the proposed PS locations

Project Description

This alternative includes two new pump stations and approximately 14.8 miles of pipeline as shown to convey water from the North Marin Aqueduct to Nicasio and SoulaJule Reservoirs via San Antonio Road. This alternative utilizes the existing Petaluma/North Marin Aqueduct to convey water from Cotati Tanks to the proposed turnout location.



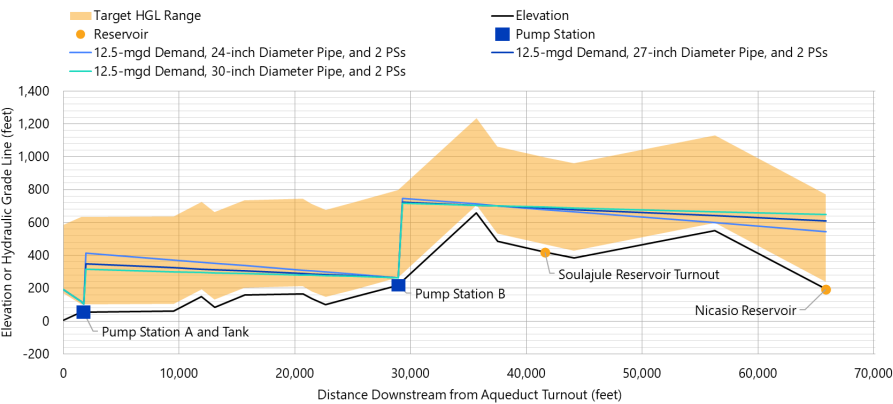
2024 TOTAL ESTIMATED COST WITHOUT UPSTREAM IMPROVEMENTS

Cost Item	Cost (rounded)
General Conditions (I.e., Mob/Demob, Temporary Facilities, Testing, Start-Up, Etc)	\$6,100,000
Pipeline	\$64,680,000
Pump Stations	\$31,500,000
Power Supply	\$9,840,000
Ancillary Items	\$15,900,000
Direct Cost Subtotal	\$128,020,000
Total Estimated Construction Cost Including Indirect Costs	\$215,920,000
<i>Estimated Construction Cost Range (-30%)</i>	<i>\$151,140,000</i>
<i>Estimated Construction Cost Range (+50%)</i>	<i>\$323,880,000</i>

2024 TOTAL ESTIMATED COST WITH UPSTREAM IMPROVEMENTS

Cost Item	Cost (rounded)
General Conditions (I.e., Mob/Demob, Temporary Facilities, Testing, Start-Up, Etc)	\$10,270,000
Pipeline	\$64,680,000
Pump Stations	\$40,500,000
Kastania Pump Station Improvments	\$10,000,000
STS Alternative	\$53,630,000
Power Supply	\$9,840,000
Ancillary Items	\$26,800,000
Direct Cost Subtotal	\$215,720,000
Total Estimated Construction Cost Including Indirect Costs	\$363,840,000
<i>Estimated Construction Cost Range (-30%)</i>	<i>\$254,690,000</i>
<i>Estimated Construction Cost Range (+50%)</i>	<i>\$545,760,000</i>

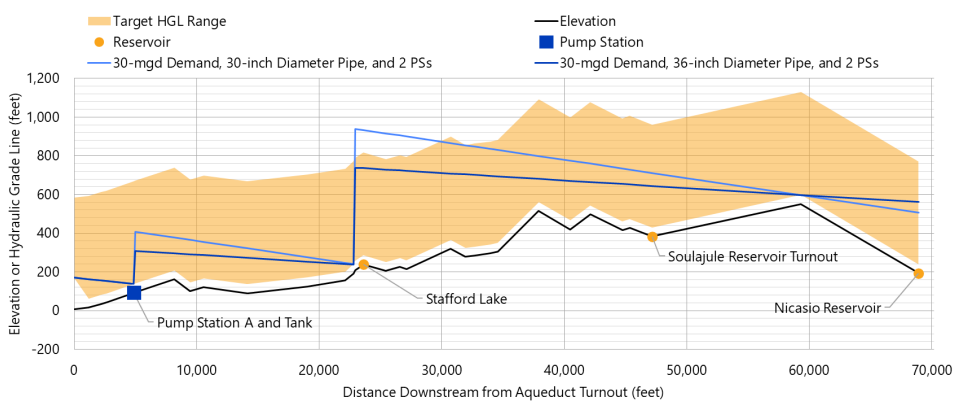
System Hydraulics



Two Pump Station Hydraulics with 12.5 mgd Demand

The system hydraulics were evaluated for two potential pumping configurations. The first configuration assumes a single pump station located near the North Marin Aqueduct that would pump water to the reservoirs. The second configuration includes two pump stations with one located near the North Marin Aqueduct and the other located near Red Hill. These two pump stations would operate in series to pump water from the North Marin Aqueduct to the reservoirs. Based on the system hydraulics, a two pump station system is anticipated to be required.

The system hydraulics graphics shown include a yellow band that shows the target HGL range for each scenario. The target HGL range shown is the sum of the elevation along the pipeline alignment plus a pipeline pressure ranging from 20 psi to 250 psi. In general, pipelines are designed to operate within this band of pressures. While feasible, pipelines and pump stations with pressures exceeding 250 psi require specialty piping and appurtenances, leading to increased design complexity and construction costs. Based on the preliminary hydraulics, it is recommended that a 36-inch pipeline be installed for the PETA-4 alternative. As shown, assuming a demand of 12.5 mgd, a 24-inch pipeline will allow the system to operate with two pump stations. However, a 24-inch pipeline will not allow the conveyance system to be expanded in the future. Assuming upstream improvements are constructed and there is a demand of 30.0 mgd, a 36-inch pipeline will allow the system to operate with two pump stations. Additionally, the 36-inch pipeline will provide operational flexibility to the District in the future.



Two Pump Station Hydraulics with 30.0 mgd Demand

COTATI-3: Cotati Tanks to Nicasio and Soulajule Reservoirs

Water Supply Benefits

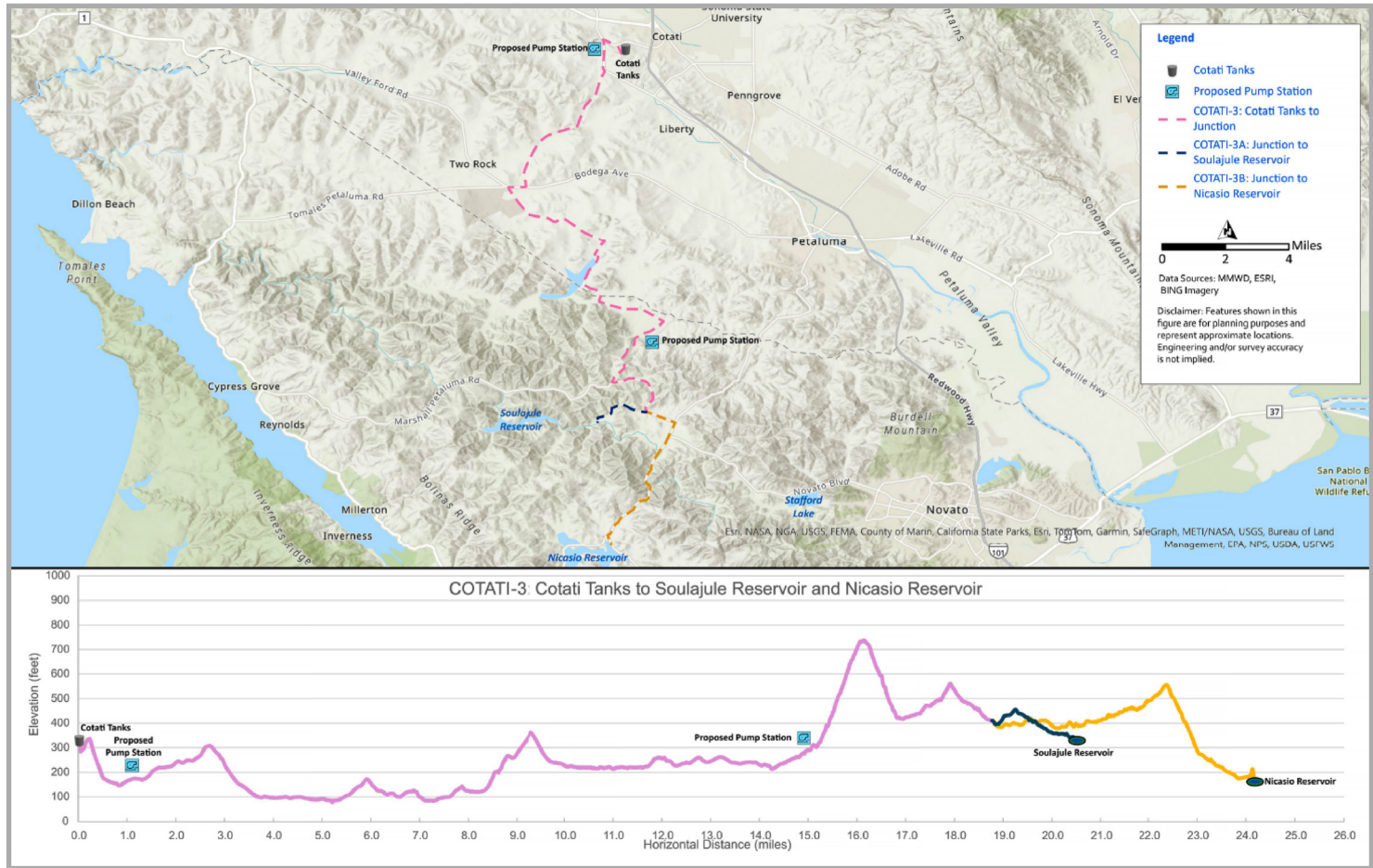
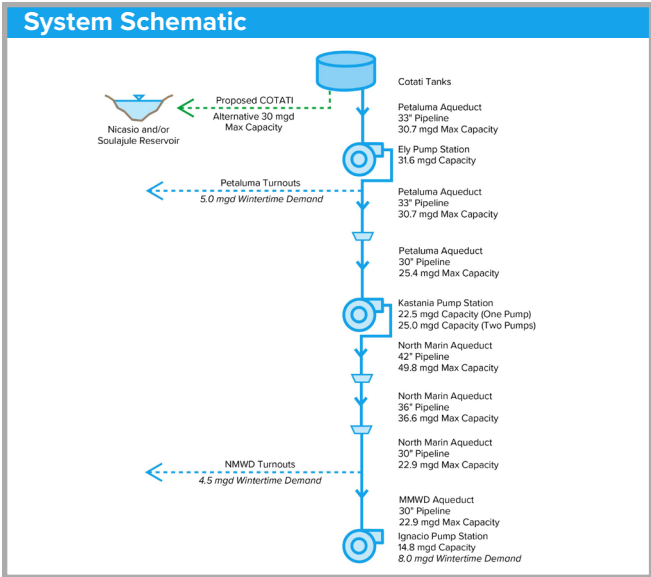
- COTATI-3: 30.0 MGD**
- Includes new infrastructure that is largely independent of the existing system
 - Sizing of the facilities and associated water supply benefits can be defined by the District

Constructability Challenges

- Portion of pipeline alignment is routed along steep, curvy, narrow County roads
- Portion of pipeline alignment is routed through wetlands
- Significant number of stream crossings that may require trenchless pipe installations
- Pavement moratorium for County roads near Cotati Tanks
- Local landslide risks near Wilson Hill
- Limited existing power facilities located near the proposed PS B

Project Description

This alternative includes two new pump stations and approximately 25.9 miles of pipeline as shown to convey water from the Cotati Tanks to Nicasio and Soulajule Reservoirs. This alternative utilizes new infrastructure to convey water from Cotati Tanks to the proposed turnout location that is independent of the existing Petaluma/North Marin Aqueduct system.



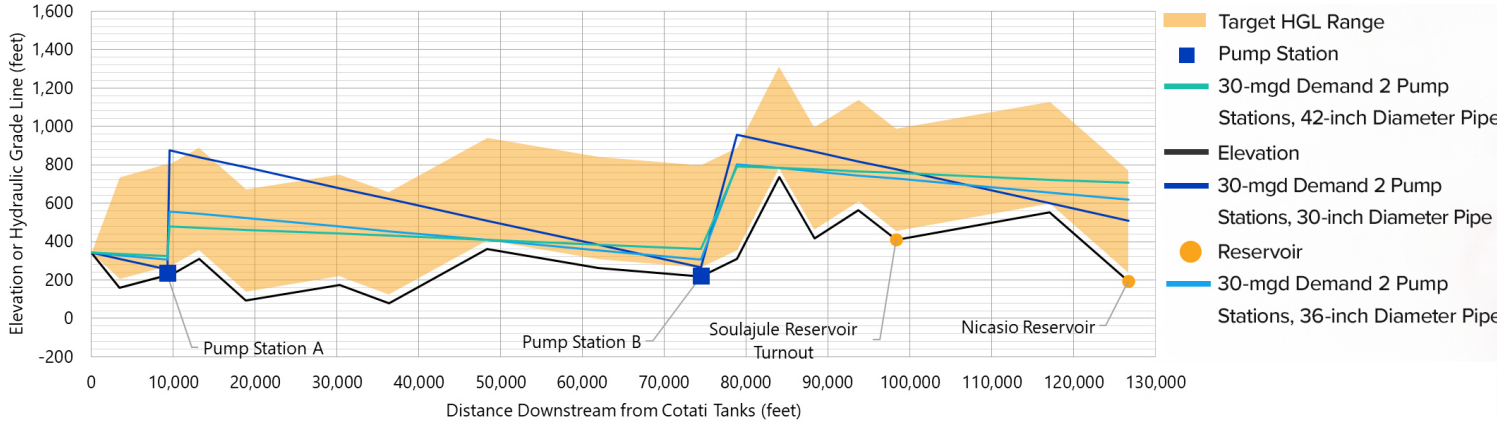
2024 TOTAL ESTIMATED COST

COST ITEM	COST (ROUNDED)
General Conditions (I.e., Mob/Demob, Temporary Facilities, Testing, Start-Up, Etc)	\$9,450,000
Pipeline	\$108,420,000
Pump Stations	\$49,500,000
Power Supply	\$6,440,000
Ancillary Items	\$24,660,000
Direct Cost Subtotal	\$198,470,000
Total Estimated Construction Cost Including Indirect Costs	\$334,740,000
<i>Estimated Construction Cost Range (-30%)</i>	<i>\$234,310,000</i>
<i>Estimated Construction Cost Range (+50%)</i>	<i>\$502,100,000</i>

System Hydraulics

The system hydraulics were evaluated for two potential pumping configurations. The first configuration assumes a single pump station located near the Cotati Tanks that would pump water to the reservoirs. The second configuration includes two pump stations with one located near the North Marin Aqueduct and the other located near Wilson Hill. These two pump stations would operate in series to pump water from the North Marin Aqueduct to the reservoirs. Based on the system hydraulics, a two pump station system is anticipated to be required.

The system hydraulics graphic shown includes a yellow band that shows the target HGL range for each scenario. The target HGL range shown is the sum of the elevation along the pipeline alignment plus a pipeline pressure ranging from 20 psi to 250 psi. In general, pipelines are designed to operate within this band of pressures. While feasible, pipelines and pump stations with pressures exceeding 250 psi require specialty piping and appurtenances, leading to increased design complexity and construction costs. Based on the preliminary hydraulics, it is recommended that a 36-inch pipeline be installed for the COTATI-3 alternative. As shown, assuming a demand of 30.0 mgd, a 36-inch pipeline will allow the system to operate with two pump stations.



APPENDIX 3A

ESA ENVIRONMENTAL EVALUATION OF INITIAL ALTERNATIVES MEMORANDUM

To be provided in future submittal.

APPENDIX 3B

MPEG GEOLOGIC AND GEOTECHNICAL FEASIBILITY EVALUATION

**GEOLOGIC AND GEOTECHNICAL FEASIBILITY EVALUATION
MARIN MUNICIPAL WATER DISTRICT
WATER SUPPLY CONVEYANCE IMPROVEMENTS PROJECT
MARIN AND SONOMA COUNTIES, CALIFORNIA**

July 24, 2024

Job No. 187.523

Prepared For:
Marin Municipal Water District
220 Nellen Avenue
Corte Madera, California 94925

Attn: Mr. Paul Sellier, Director of Water Resources

CERTIFICATION

This document is an instrument of service, prepared by or under the direction of the undersigned professionals, in accordance with the current ordinary standard of care. The service specifically excludes the investigation of polychlorinated biphenyl, radon, asbestos, or any other hazardous materials. The document is for the sole use of the client and consultants on this project. No other use is authorized. If the project changes, or more than two years have passed since the issuance of this report, the findings and recommendations must be updated.

MILLER PACIFIC ENGINEERING GROUP
(a California corporation)

REVIEWED BY



Mike Jewett
Engineering Geologist No. 2610
(Expires 1/31/25)



Scott Stephens
Geotechnical Engineer No. 2398
(Expires 6/30/25)

**GEOLOGIC AND GEOTECHNICAL FEASIBILITY EVALUATION
MARIN MUNICIPAL WATER DISTRICT
WATER SUPPLY CONVEYANCE IMPROVEMENTS PROJECT
MARIN AND SONOMA COUNTIES, CALIFORNIA**

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PLATE 3 – FAULT AND SEISMICITY MAP

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TABLE 3 PROBABILISTIC PEAK GROUND ACCELERATION

TABLE 4 PRELIMINARY ASPHALT PAVEMENT SECTIONS

DIGITAL DATA

This report is accompanied by a digital file database prepared using ESRI ArcGIS Desktop Pro, Version 3.2.2, which should be considered integral to this report. The database includes our detailed geologic mapping of the proposed alignments as well as select supporting data used in this study, including previous subsurface exploration points. A complete list of references is provided in Section 12 of this report.

**GEOLOGIC AND GEOTECHNICAL FEASIBILITY EVALUATION
MARIN MUNICIPAL WATER DISTRICT
WATER SUPPLY CONVEYANCE IMPROVEMENTS PROJECT
MARIN AND SONOMA COUNTIES, CALIFORNIA**

EXECUTIVE SUMMARY

Our Phase 1 Geologic and Geotechnical Feasibility Evaluation considered a total of nine individual pipeline alignments which would convey water from Sonoma County Water Agency (Sonoma Water) to the Marin Municipal Water District (MMWD) service area. The pipeline alignments vary in length between about 7 and 24 miles long. Each alignment traverses areas of variable topography and geologic conditions.

Our evaluation included review of readily available, published geologic mapping, regional geologic hazard maps, historic aerial photographs, and selected relevant geotechnical background data from our in-house library, field reconnaissance and geologic mapping along each alignment, evaluation of geologic hazards, and development of preliminary geotechnical recommendations and design criteria for the work.

Based on the results of our background review, field mapping, and hazards evaluation, we judge that construction of any of the proposed alignments is feasible from a geotechnical perspective. Significant geotechnical design considerations applicable to all alignments will include providing adequate seismic design and uniform foundation support for significant structures, limiting pipeline exposure to slope instability and seismic lurching risks in steeper terrain, and providing adequate flexibility in pipe materials and connections to resist damage due to seismic ground shaking, liquefaction, and/or settlement. More localized design considerations may include providing adequate mitigation if and where new pipelines are constructed in Bay Mud areas along alignment STS-2 and providing adequate evaluation of naturally occurring asbestos potential where the STAF, COTATI, and PETA alignments are underlain by Franciscan metamorphic rocks and mélange.

Significant geotechnical construction considerations applicable to all alignments will include evaluating shoring and dewatering needs in areas of weak, caving soils and/or shallow groundwater. All alignments are likely to locally encounter such conditions at stream crossings and in valley bottoms, while the STS-2 alignment in particular may require extensive shoring and dewatering where it crosses low-lying areas adjacent to the Petaluma River.

Other related considerations which may impact selection of the “preferred” alignment(s) are anticipated to include the potential for project cost and schedule impacts where hard rock may require special excavation techniques and equipment, where there are significant conflicts with existing utilities, and where naturally occurring asbestos or other hazardous materials/environmental contaminants may require special handling and disposal procedures.

1.0 INTRODUCTION

This report summarizes the results of our Phase 1 Geologic and Geotechnical Feasibility Evaluation for Marin Municipal Water District's (MMWD) Water Supply Improvements project in southern Sonoma and northern Marin Counties, California. The project generally includes evaluation of nine (9) potential alignments for potential development of significant new water conveyance pipelines. A Site Location Map showing the approximate extents and relative locations of the planned work areas is presented on Plate 1.

Our services are being provided in accordance with our Agreement authorized December 5, 2023. The purpose of our Phase 1 services is to compile existing surface and subsurface geologic data, refine existing geologic mapping, evaluate relevant geologic hazards and special geotechnical considerations that may affect the project, develop preliminary geotechnical recommendations and construction guidelines for the work. The scope of our Phase 1 services is described in our proposal letter dated June 12, 2020, and includes the following:

- Review of available, published geologic mapping and geotechnical background information from our in-house library.
- Review of available historic and current aerial photography and digital topographic mapping.
- Detailed site reconnaissance for observation of existing conditions and refinement of previous geologic mapping based on surface observations and subsurface data compilation.
- Evaluation of geologic hazards that may impact the project and conceptual mitigation measures.
- Development of preliminary geotechnical recommendations and construction guidelines for the work, including discussion of anticipated excavation conditions, special shoring and/or dewatering considerations, and preliminary recommendations for utility trench backfill materials and compaction.
- Development of general recommendations for design-level (Phase 2) subsurface exploration and laboratory testing; and
- Preparation of this report.

Future phases of work are expected to include a design-level Geotechnical Investigation with supplemental subsurface exploration and laboratory testing (Phase 2), geotechnical consultation and plan review (Phase 3), and geotechnical observation and testing during construction (Phase 4).

2.0 PROJECT DESCRIPTION

The project generally includes evaluating a total of nine (9) independent water supply conveyance pipeline alignments, typically ranging from about 7- to 24-miles long, each of which would transfer water between various existing storage tanks and reservoirs in southern Sonoma County and northern Marin County. A Site Vicinity Map showing each alignment is shown on Figure 1, and the proposed alignments are summarized below in Table A. Detailed descriptions and discussion of specific alignments are provided in Section 5 of this report.

TABLE 1 - SUMMARY OF PROPOSED PIPELINE ALIGNMENTS			
ALIGNMENT ID	ORIGIN	DESTINATION	APPROX. TOTAL LENGTH (FEET)
STS-1	Cotati Tanks	Kastania Tanks (via US Highway 101)	56,297
STS-2	Cotati Tanks	Kastania Tanks (via Petaluma streets)	64,971
STAF-1	Stafford Lake	Nicasio Reservoir	42,385
STAF-2	Stafford Lake	Soulajule Reservoir	36,223
COTATI-1	Cotati Tanks	Nicasio Reservoir	127,627
COTATI-2	Cotati Tanks	Soulajule Reservoir	108,271
PETA-1	North Marin Aqueduct (San Marin Drive)	Nicasio Reservoir	69,663
PETA-2	North Marin Aqueduct (San Marin Drive)	Soulajule Reservoir	63,501
PETA-4	North Marin Aqueduct (San Antonio Road)	Nicasio & Soulajule Reservoirs	78,355

At this time, specific project details such as pipeline size/materials, alignment depth, and ancillary improvements are unknown. However, based on our experience, we anticipate the pipelines will be of relatively large diameter (between 18 and 42 inches) and that alignment depth will generally be as shallow as practical, typically between 5 and 10 feet. Construction methods could include a combination of open-cut trenching and directional drilling or other trenchless construction techniques. Local excavation up to 20 feet or more may be required if and where deeper alignments are needed, such as at channel crossings or in areas with significant existing utility conflicts. We anticipate that at minimum, ancillary work will include restoration of asphalt pavements, concrete flatwork, and/or other improvements damaged or disturbed during the course of construction.

3.0 PURPOSE AND SCOPE

The purpose of our services is to document site geologic conditions along each alignment, evaluate relevant geologic hazards and unique geotechnical conditions that may affect the project, and develop preliminary geotechnical recommendations and guidelines for use in selection of the “preferred” alignment(s) and preliminary project design.

The scope of our services is described in our proposal letter dated October 31, 2023 and includes review of available historic and current aerial imagery and topographic data, review of published geologic mapping and geotechnical background information, field reconnaissance and geologic mapping of each proposed alignment, evaluation of relevant geologic hazards that may affect the project, development of preliminary geotechnical recommendations, and production of this report. Supplemental services may include geotechnical consultation to assist in selection of “preferred” alignments, geotechnical investigation with subsurface exploration and laboratory testing to develop design-level recommendations and criteria for the project, and geotechnical observation and testing during construction.

4.0 INVESTIGATIVE METHODS

This section outlines the means, methods, and reference information utilized in performance of this evaluation. In general, methods included review of current digital topographic mapping and historic aerial photography, review of readily available published geologic maps, review of relevant geotechnical background data from our in-house library, and performance of field reconnaissance and geologic mapping refinement along each alignment.

We used ESRI ArcGIS Pro Desktop software for the purposes of reviewing and compiling existing digital and select “paper” data, compiling field mapping and reconnaissance findings, and providing our aggregated data in digital (ESRI ArcGIS Geodatabase) format to the project team. Our findings in regard to interpreted site conditions are discussed in Section 6, and a complete list of references for background materials we reviewed is provided in Section 12.

4.1 Digital Topographic Mapping

We reviewed and incorporated into our database several digital topographic mapping products made available to the public by the Counties of Sonoma and Marin. For Sonoma County, digital topographic contours, and bare-earth hillshade data¹ were obtained through Sonoma County’s VegMap website (<https://sonomavegmap.org/data-downloads/>, accessed November 2023). Within Marin County, digital topographic contours and bare-earth hillshades derived from Marin County’s 2019 QL1 LiDAR survey (Quantum Spatial, 2019) were obtained through Marin County’s GIS Open Data website (<https://gisopendata.marincounty.gov/>, accessed May 2023).

We also reviewed the National Hydrography Dataset (NHD), maintained by the National Oceanic and Atmospheric Administration (NOAA; <https://coast.noaa.gov/digitalcoast/data/nhd.html>) to evaluate the distribution and estimated number of channel and culvert crossings along each alignment.

4.2 Regional Geologic Mapping

We reviewed a variety of regional geologic maps prepared by the United States Geological Survey (USGS) and California Geologic Survey (CGS, formerly the California Division of Mines and Geology) prepared between 1980 and the early 2000s at scales between 1:24,000 and 1:100,000. Note that these maps were prepared at a variety of scales by different researchers, and some features are not contiguous across the boundaries between different map areas.

We also reviewed a digital compilation of the more recent 2000s-era CGS mapping, available through Marin County’s GIS Open Data website. This map covers the entirety of the project area, combining portions of USGS maps by Blake, Graymer, and Jones (2000) and Blake, Graymer, and Stamski (2002). The most recent mapping by CGS (Wagner and Gutierrez, 2017) covers the entirety of the project area, but is presented at a slightly less detailed scale than the earlier 2000/2002 maps.

¹ LiDAR data and orthophotography were provided by the University of Maryland under NASA grant NNX13AP69G from NASA’s Carbon Monitoring System (Dr. Ralph Dubayah and Dr. George Hurtt, Principal Investigators).

4.3 Regional Geologic Hazard Mapping

We reviewed various fault maps produced by USGS and CGS, in both paper and digital format, which provide information on the location and nature of various active and inactive faults in the region. We also reviewed the USGS Historic Earthquake Database to review the locations of historic earthquakes in the area and reviewed Alquist-Priolo Earthquake Fault Zone maps produced by CGS which outline special regulatory zones surrounding known active faults.

Regional, reconnaissance-level landslide mapping of the project area was performed in the 1970's by the USGS and the California Division of Mines and Geology (CDMG; now CGS), which exclusively involved interpretation of aerial photographs, and did not include field mapping or "ground truthing". We also reviewed the digital CGS Landslide Inventory (<https://maps.conservation.ca.gov/cgs/lsi/>) in order to locate other mapped landslides along each of the proposed alignments.

We also reviewed paper and digital maps by USGS and the Bay Area Association of Governments (ABAG) showing relative liquefaction susceptibility and digitized FEMA flood zone maps to aid in preliminary evaluation of relevant geologic hazards.

4.4 Historic Aerial Photography

We reviewed historic aerial photographs available through the Sonoma County Vegetation and LiDAR Mapping Program (<https://sonomavegmap.org/1942/>) and the University of California, Santa Barbara Library (https://mil.library.ucsb.edu/ap_indexes/FrameFinder/) to assess historic development and land use, and to corroborate conditions shown on photo-reconnaissance landslide maps, where possible. The specific photographs we reviewed spanned the time period between 1942 and 1988 and are listed for reference in Section 12.

4.5 Geotechnical Background Data

We reviewed our in-house library to compile subsurface and laboratory test data from previous investigations undertaken throughout the project area, with a focus on areas relevant to and proximal to the planned improvements. Reports we reviewed are shown in the List of References at the end of this report, and the locations of previous exploratory borings are shown in Appendices A through H.

4.6 Site Reconnaissance

We performed a detailed site reconnaissance of eight of the alignments between January and March of 2024. Follow-up reconnaissance of alignment PETA-4 was performed in June of 2024. Our reconnaissance consisted primarily of a "windshield tour" with local stops as needed for the purpose of confirming or refining regional geologic mapping by others, identifying and locating landslides and other significant geologic features, and identifying locations where unique conditions (such as stream channel and significant culvert crossings, areas of probable hard rock, conflicts with existing improvements, etc.) may require specific supplemental evaluation and/or special geotechnical design consideration.

5.0 SITE SETTING

This section describes general site conditions in the southern Sonoma and northern Marin County areas as well as discussion regarding geologic materials, structures, and potential seismic sources in the vicinity.

5.1 Topographic Setting

The project site lies within the Coast Ranges geomorphic province of California. Regional topography within the Coast Ranges province is characterized by northwest-southeast trending mountain ridges and intervening valleys that parallel the major geologic structures, including the San Andreas Fault System. The province is also generally characterized by abundant landsliding and erosion, owing in part to its typically high levels of precipitation and seismic activity.

In general, the Cotati Tanks at the northern end of the project area lie at the crest of a low, northwest trending range of hills that separate the Laguna de Santa Rosa watershed to the north from the broad alluvial valley of the Petaluma River watershed to the south. South and west of Petaluma, topography is dominated by Mount Burdell and more closely spaced, northwest-trending ridgelines and intervening valleys, including Two Rock, Hicks, and Nicasio Valleys. Elevations along the various alignments range from a minimum of about 8-feet above sea level along the STS-1 alignment near the Petaluma River to more than 650-feet where the PETA-4 alignment would crest Red Hill at the east side of Hicks Valley. Topographic conditions along each alignment are shown in Appendices A through H, and more detailed discussion of specific conditions along each alignment is provided in Section 6.

5.2 Geologic Setting

The oldest rocks in the region are the sedimentary, igneous, and metamorphic rocks of the Jurassic- to Cretaceous-age (190- to 65-million years old) Franciscan Complex. Within Marin and Sonoma Counties, a variety of sedimentary and volcanic rocks of Tertiary (1.8- to 65-million years old) and Quaternary (less than 1.8-million years old) age locally overlie the basement rocks of the Franciscan Complex. Tectonic deformation and erosion during late Tertiary and Quaternary time (the last several million years) formed the prominent coastal ridges and intervening valleys typical of the Coast Ranges province. The youngest geologic units in the region are Quaternary age (last 1.8 million years) include a variety of sedimentary deposits, primarily alluvial deposits which partially fill most of the valleys and colluvial soils which typically blanket the lower portions of surrounding slopes.

The various proposed pipeline alignments traverse a variety of geologic materials, ranging from weak, unconsolidated soils to relatively hard and strong bedrock. More detailed descriptions of geologic materials mapped regionally and observed during our reconnaissance as underlying various portions of the proposed alignments are provided in the following sections. A Preliminary Geologic Map of the project area is shown on Plate 2, and more detailed maps of each proposed alignment are provided in Appendices A through H.

5.2.1 Surface Deposits

Regionally mapped surface deposits in the project area include man-made fills and a variety of soils deposited during Quaternary time (between 1.8 million years ago and present), including:

Artificial Fill and Bay Mud

Includes areas of man-made fill placed for development of natural marshlands (afbm). Fill (af) may include variable quantities of soil, rock, trash, and/or rubble. Bay Mud (bm) is composed of soft, saturated silty clay that is extremely prone to compression under new loads imposed by fills and structures. These materials are mapped underlying a portion of the STS-1/2 alignments near the Petaluma River in southern Petaluma. Related map units include:

Artificial Fill Over Bay Mud (afbm)

Artificial Fill (af)

Bay Mud (bm)

Alluvial Soils

Typically includes variable proportions of clay, silt, sand, and gravel deposited in stream channel, basin, and floodplain environments, and as fan deposits where stream channels emanate from upland areas onto the valley floor. Some deposits are differentiated on the basis of age or depositional environment, while others are not. In general, younger alluvium (Qhay) and fan deposits (Qf) will be poorly consolidated, while older deposits will be better-consolidated and likely of higher density and strength. Alluvial deposits mapped within various portions of the project area include:

Younger Alluvium - (Qhay)

Older Alluvium –(Qoa)

Undifferentiated Alluvium (Qal)

Fine-Grained Alluvial Fan Deposits (Qhff)

Undifferentiated Alluvial Fan Deposits (Qf)

Colluvial Soils

Although not shown on regional maps due to the relatively large scale, most areas mapped as bedrock are overlain by a layer of residual or colluvial soils, a mixture of clay, silt, sand, and rock fragments that are weathered and derived from the underlying bedrock materials. Residual soils typically refer to such materials which have weathered “in situ”, while colluvium refers to such soils which have been transported downslope via gravity and soil creep to accumulate in swales and around the base of steeper slopes. Where bedrock is more resistant to weathering residual and colluvial soils may be only a few inches or feet thick, while zones of more deeply weathered or sheared rock (such as along faults) may be overlain by significantly thicker soils.

5.2.2 Bedrock Geology

Bedrock mapped within the project area, especially within the Marin County part of the project area, consists predominantly of meta-sedimentary and meta-volcanic rocks of the Franciscan Complex. Within Sonoma and northeastern Marin County, these rocks are locally overlain by younger Tertiary-age volcanic and sedimentary rocks.

Tertiary Rocks

Burdell Mountain Volcanics (Tbm)

The Burdell Mountain volcanics consist of relatively felsic (sodium-calcium rich) andesite and dacite erupted during late Miocene time, between 11- and 12 million years ago.

Donnall Ranch Volcanics (Tdr)

The Donnall Ranch Volcanics are slightly younger, about 8.5- and 10.5 million years old, and are of more mafic andesitic and basaltic composition (richer in iron and magnesium).

Sonoma Volcanics – (Tsv and Tsa)

The remaining, undifferentiated Sonoma Volcanics includes a variety of ash tuff and lava flow deposits ranging in composition from rhyolite to basalt. These rocks are associated with volcanism in the Mayacamas Mountains and the Clear Lake volcanic field between about 2.5- and 9 million years ago. Constituent rocks range from relatively hard and erosion-resistant lava flows to weak, poorly consolidated ash tuff and lacustrine (lake) deposits. This unit is typically shown as undifferentiated (Tsv), but locally includes mapped zones of andesite and basalt flows (Tsa).

Petaluma Formation (Tp), Wilson Grove Formation (Twg) and Gravel of Cotati (Tc)

The Petaluma Wilson Grove Formations underlie much of the gently rolling terrain in southwestern Sonoma County and consist predominantly of fine siltstone, sandstone and pebble conglomerate deposited in fluvial environments and a shallow marine embayment, respectively. The gravel of Cotati consists of slightly coarser sediment deposited easterly of the Wilson Grove Formation in an estuarine environment. These formations are of similar age to the Sonoma Volcanics and tend to be weakly- to moderately cemented.

Great Valley Sequence

The Great Valley Sequence is composed of interbedded marine siltstone, sandstone, and pebble conglomerate deposited during Jurassic-Cretaceous time. These rocks are typically well-lithified and of moderate hardness and strength.

Franciscan Complex

The Franciscan Complex is a group of rocks deposited during Jurassic-Cretaceous time and represent portions of an ancient seafloor which have been subducted beneath the North American tectonic plate and then transported northward along the San Andreas fault system. These are the most common rocks in the project area, underlying some of the rolling hills southeast of Petaluma and most upland areas in northern Marin.

The Franciscan Complex is typified by resistant blocks and masses of relatively hard and strong rocks (including metagraywacke, chert, greenstone, meta-greenstone, and serpentinite) interspersed within zones of sheared shale “mélange” In some areas, resistant masses are continuous and extend for several miles or more and are mapped as individual units. In other areas, masses are discrete or too small to accurately show at

map scale, and these areas are typically shown as “Mélange”. Although sheared shale comprises the majority of the mélange terrain, embedded rocks may locally include any of the other constituent rock types. Franciscan rocks which underlie different project areas include:

Sandstone and Shale (KJfs)

Melange (KJfm)

Chert (KJfch)

Greenstone (KJfg)

Metamorphic rocks (KJsch)

Serpentine (sp)

5.3 Seismicity

The project site is located within the seismically active San Francisco Bay Area and will therefore experience the effects of future earthquakes. Earthquakes are the product of the build-up and sudden release of strain along a “fault” or zone of weakness in the earth's crust. Stored energy may be released as soon as it is generated, or it may be accumulated and stored for long periods of time. Individual releases may be so small that they are detected only by sensitive instruments, or they may be violent enough to cause destruction over vast areas. The amplitude and frequency of earthquake waves and resultant ground motions partially depends on the material through which it is moving. The earthquake force is transmitted through hard rock in short, rapid vibrations, while this energy becomes a long, high-amplitude motion when moving through soft ground materials, such as Bay Mud.

5.3.1 *Faults*

Faults are seldom single cracks in the earth's crust but are typically comprised of localized shear zones which link together to form larger fault zones. Within the Bay Area, faults are concentrated along the San Andreas Fault zone. The movement between rock formations along either side of a fault may be horizontal, vertical, or a combination and is radiated outward in the form of energy waves.

Mapped Active Faults

The California Geological Survey defines a “Holocene-active fault” as one that had surface displacement within Holocene time (the last 11,700 years). CGS mapped various faults in the region as part of their Fault Activity Map of California (CGS, 2010). Many of these faults are shown in relation to the project site on Plate 3. The nearest known Holocene-active faults to the project area are the Rodgers Creek, San Andreas, and San Gregorio Faults, each of which is located more than 8 kilometers from the nearest portion of the proposed work areas. No Holocene-active faults are mapped within the project area.

Mapped Inactive Faults

In addition to the aforementioned active faults mapped by CGS, several other faults are mapped crossing some of the proposed alignments, as summarized below. Although these faults are designed by CGS as “inactive”, some of them appear to be associated with historic earthquakes of relatively small magnitude as discussed in Section 5.3.2.

Burdell Mountain Fault

The Burdell Mountain Fault extends northwestward from the Black Point Area of eastern Novato and along the northeastern flank of Mount Burdell. East of Mount Burdell, the fault is shown offsetting the Cretaceous-age Novato Conglomerate against Tertiary-age rocks of the Sonoma Volcanics. The fault is then shown continuing northwest, concealed beneath the marsh, before crossing Highway 101 and then bending slightly west and following San Antonio Road and the PETA-4 alignment before terminating near the intersection of San Antonio Road and Red Hill Road southwest of Petaluma. Although the Burdell Mountain Fault is postulated to have been a significant regional fault during Tertiary and early Quaternary time (Blake, Graymer, and Jones, 2000), and despite the epicenter of the 1855 Black Point Earthquake lying conspicuously close to the south end of the fault zone (see section 4.2.4), it is now mapped as being of “undifferentiated Quaternary” age, and is not considered active (Bryant, 2017, #226).

Tolay Fault

The Tolay Fault is mapped by CGS as a 22-mile-long zone of sub-parallel, right-lateral faults which extend southeast from western Cotati to Sears Point. Although some of the faults are shown to be confidently located (primarily where exposed in landslide scarps along Highway 101 on the Cotati Grade and in the low hills southeast of Petaluma), most of the fault zone is shown to be concealed by younger sediment, including mapped traces within eastern Petaluma and farther south in the Petaluma River Valley where the fault zone eventually merges with the Hayward/Rodgers Creek Fault near Sears Point. The fault apparently offsets Plio-Pleistocene age rocks (about 5- to 12-million years old) and is postulated as a potential ancient northward extension of the Hayward Fault (Hart and Bryant, 1982). Although USGS postulates a slip rate of 0.2- to 1.0 mm/year, no estimated magnitude is provided. The fault is classified as “undifferentiated Quaternary time” (within the last 1.7 million years, but more than 11,000 years ago) and is therefore not considered to be active.

The northwesternmost trace within the zone is mapped as “inferred”. The trace is shown as extending from about a mile west of the Cotati tanks, then crossing Meachem Road and the COTATI-1/2 alignments about 500 feet west of Stony Point Road. From there, the fault continues southeast along the west side of Highway 101 crossing Stony Point Road and the STS-2 alignment, then crossing the highway and the STS-1 alignment just south of Petaluma Boulevard North. The fault extends another couple miles southeast and terminates just north of Washington Boulevard.

Bloomfield and Americano Creek Faults

The Bloomfield Fault is mapped as a roughly 10-mile-long zone consisting of two subparallel fault traces which cross the COTATI-1/2 alignments at Meachem Road and at Pepper Road, about a mile apart (Bryant, 2017, #224). Both faults are mapped as “certainly located” and shown to offset pre-Quaternary rocks of the Franciscan Complex and Wilson Grove Formation.

The Americano Creek Fault is located about 1 ¼ miles west of the Bloomfield Fault Zone, crossing Pepper Road and the COTATI-1/2 alignments just north of Bodega Avenue (Bryant, 2017, #225). This fault is shown as a single, 7-mile-long northwest-striking trace and is also shown offsetting Franciscan and Wilson Grove rocks.

Each of these faults exhibit evidence of rupture during “undifferentiated Quaternary time” (within the last 1.7 million years, but more than 11,000 years ago), and neither is considered active.

Unnamed Faults

Several generally northwest-striking fault zones are mapped southwest of the Americano Creek Fault, each of which offsets Tertiary-age Sonoma Volcanics and Wilson Grove rocks against various Franciscan meta-sedimentary and meta-volcanic rocks. One or more of these faults intersects each of the COATATI-1/2, PETA-1/2/4, and STAF-1/2 alignments. These are considered “pre-Quaternary” faults and are not considered to be active.

5.3.2 Historic Seismicity

Numerous earthquakes have occurred in the region within historic times. Although the most recent earthquakes are well-documented and accurately located through modern seismic monitoring and reporting networks, documentation of many older events is limited either to first- and secondhand observations and news reports. We note that some significant details of events pre-dating modern technology (such as magnitudes, shaking severity, and epicentral locations) have been subject to revision as more detailed and widespread study of active seismicity and historic events generates additional data.

The approximate epicenters and reported effects of significant historic earthquakes within the project area as reported by the USGS Earthquake Hazards Program (USGS, 2023) the CGS Historic Earthquake Online Database (CGS, 2023), and Topozada et al (1992) are summarized below:

1855 Black Point Earthquake – On August 27, 1855, a magnitude 5.5 event occurred with an epicenter in the Black Point area of Novato, near the southern end of the Burdell Mountain Fault. The earthquake reportedly resulted in severe shaking in the Petaluma River Valley, knocking plaster and paintings from walls in San Francisco and cracking the adobe walls of General Vallejo’s home east of Petaluma.

1888 Cotati Earthquake – A 4.2 magnitude earthquake struck just under 2 miles southeast of the Cotati Tanks on February 29, 1888. The epicenter is mapped within the north end of the Tolay Fault zone, just east of Highway 101 near the crest of the Cotati Grade.

1893 Santa Rosa Earthquake - On August 9, 1893, a magnitude 5.6 earthquake occurred with an epicenter along the southwest side of Taylor Mountain, on an apparent concealed splay of the Rodgers Creek Fault beneath the east edge of the Santa Rosa floodplain. This earthquake generated a Modified Mercalli intensity of VII, toppling chimneys and plaster throughout Santa Rosa and causing moderate structural damage.

1898 Mare Island Earthquake – The March 31, 1898 “Mare Island” earthquake was centered beneath San Pablo Bay near the city of Vallejo. This earthquake has been estimated between magnitude 6.2 and 6.7, generating moderate to severe shaking and an estimated Modified Mercalli intensity of VII or greater throughout portions of the North Bay area. The most significant damage was concentrated between Vallejo/Mare Island and the Sonoma and Petaluma Valleys to the northwest, and aftershock reports were also more abundant in those areas, suggesting a source near the southern end of the Rodgers Creek Fault (Topozada, et al, 1992). Damage in Santa Rosa was more moderate, limited mainly to cracked windows, fallen plaster, and a few collapsed chimneys.

Notably, the epicentral location of this event has previously been reported as being in the vicinity of Mare Island in the eastern part of Vallejo, placing it in closer proximity to the West Napa Fault. More recently, the event was assigned a magnitude of 6.4, and the epicentral location has been revised to an area between the Rodgers Creek and Tolay Fault zones, about 5 miles southwest of Petaluma. The reasoning for the updated locations is unclear. Based on current information and given the relatively large magnitude, we judge this event was most likely associated with the Rodgers Creek Fault.

1906 San Francisco Earthquake – The April 18, 1906, magnitude 8.3 earthquake occurred on the northern segment of the San Andreas Fault. The earthquake resulted in catastrophic damage throughout the greater Bay Area. Significant damage, including complete structural collapses, and 498 deaths were reported in San Francisco (CGS, 2023). Although severe shaking was reported throughout Marin and Sonoma Counties, significant damage was mostly confined to coastal areas near the fault trace, where impacts included extensive ground surface rupture and seismically induced landsliding.

1969 Santa Rosa Earthquakes – Two earthquakes of magnitudes 5.6 and 5.7 occurred within about 2 hours of each other on October 2, 1969. The resulting damage was concentrated in Santa Rosa with partial and near structural collapses. No loss of life was reported.

Similar to the 1898 Mare Island earthquake, we note that epicentral locations of these events have also been revised. Previously they had been mapped as occurring in the north-central part of Santa Rosa within the Rodgers Creek Fault zone. Currently the CGS database shows them located generally on the eastern and western margins of Santa Rosa. However, the USGS database currently plots them within the Bloomfield Fault Zone near Roblar Road, about 4.5 miles west of the Cotati Tanks. Both events are also reported as magnitude 5.3 and generating moderate shaking a Modified Mercalli intensity of VI.

1989 Loma Prieta Earthquake – The magnitude 6.9 earthquake of October 18, 1989, was centered in the Santa Cruz Mountains about 70 miles south of Novato. Significant damage was reported throughout the Bay Area, with a majority occurring in San Francisco, Oakland, and Santa Cruz in the form of structural collapses and loss of life. Significant damage was not reported in the North Bay area.

1999 Bolinas Earthquake – A 4.6 magnitude earthquake occurred at Bolinas on August 18, 1999, near the intersection of the San Andreas and San Gregorio Faults. This event resulted in light to moderate shaking with a Modified Mercalli index of VI, knocking items from shelves in southern and western Marin County. No significant damage was reported.

2014 South Napa Earthquake – The magnitude 6.0 earthquake occurred along the West Napa Fault in the Carneros area of southern Napa on August 24, 2014. The earthquake resulted in moderate to significant damage in the City of Napa and neighboring cities. Damage included surface fault rupture, partial building collapse, and structural fires. Over 300 people were injured, and one fatality was reported. Only light shaking was reported in Sonoma and Marin County.

Other Earthquakes - Several smaller earthquakes of smaller (<4.0) magnitude have occurred near the site in historic times, the nearest of which are shown for reference on Plate 3. A 1992 event of magnitude 2.7 is shown about 1.5 miles west of Nicasio Reservoir. Between 1996 and 2006, three events between magnitude 2.0 and 2.2 occurred in Cotati and Petaluma, generally along the west side of Highway 101 and the Tolay Fault zone, and in relatively close proximity

to the STS-1/2 alignments. A pair of magnitude 2.4 events occurred in 2001 and 2003 and are shown along the Americano Creek Fault zone. Several other events between magnitudes 2.0 and 2.5 are shown scattered throughout western Marin County, typically along “intraformational” faults along Big Rock Ridge, near Soulajule Reservoir, and within the San Andreas Fault zone.

5.3.3 Probability of Future Earthquakes

The site will likely experience moderate to strong ground shaking from future earthquakes originating on any of several active faults in the San Francisco Bay region. Historical records do not directly indicate either the maximum credible earthquake or the probability of such a future event. To evaluate earthquake probabilities in California, the USGS has assembled a group of researchers into the “Working Group on California Earthquake Probabilities” (USGS 2003, 2008; Field et al, 2015) to estimate the probabilities of earthquakes on active faults. These studies have been published cooperatively by the USGS, CGS, and Southern California Earthquake Center (SCEC) as the Uniform California Earthquake Rupture Forecast, Versions 1, 2, and 3. In these studies, potential seismic sources were analyzed considering fault geometry, geologic slip rates, geodetic strain rates, historic activity, micro-seismicity, and other factors to arrive at estimates of earthquakes of various magnitudes on a variety of faults in California.

Conclusions from the most recent UCERF3 and USGS (Aagard, 2016) indicate the highest probability of an earthquake with a magnitude greater than 6.7 originating on any of the active faults in the San Francisco Bay region by 2043 is assigned to the Hayward/Rodgers Creek Fault system at 33%. The Hayward Fault is located approximately 4.3 miles northeast of the site and is assigned a probability of 33 percent. The San Andreas Fault, located approximately 4.5-miles southwest of the site, is assigned a 22 percent probability of an earthquake with a magnitude greater than 6.7 by 2043. As noted previously, the Burdell Mountain Fault, Tolay Fault, and other mapped fault traces in the project area are generally considered to be inactive.

Additional studies by the USGS regarding the probability of large earthquakes in the Bay Area are ongoing. These current evaluations include data from additional active faults and updated geological data.

6.0 SURFACE CONDITIONS

This section summarizes the results of our research, field reconnaissance, and geologic mapping. In general, our reconnaissance observations corroborate conditions interpreted from our review of background information and regional geologic mapping. The following subsections provide additional detail on topography, historic development and land use, current surface conditions, and surface geology along each alignment.

6.1 Alignment STS-1

Alignment STS-1 would deliver water from the Cotati Tanks to the Kastania Tanks located just south of Petaluma, via an approximately 10.7-mile alignment along the west side of US Highway 101. The alignment would extend east from the tanks along Pine Lane and West Sierra Avenue before turning south to follow the western shoulder of the highway for about 10 miles, crossing the SMART railroad tracks and the Petaluma River, then turning west to connect to the tanks about a half mile south of the Kastania Road overcrossing.

6.1.1 *Topography*

The alignment would originate at an elevation of +315 feet at the Cotati Tanks and terminate at an elevation of +198 feet at the Kastania Tanks, for a net elevation change of -117 feet along the alignment. Site elevations range from a minimum of about +10 feet near Highway 116 (Lakeville Highway) to a maximum of +315 feet at the Cotati Tanks. Topography throughout the alignment is typically relatively gentle excepting the moderate slopes between the Cotati Tanks and the crest of the Cotati Grade along Highway 101. Locally steeper slopes exist at the south bank of the Petaluma River and between Highway 101 and the Kastania Tanks, where slopes locally exceed 25%.

Significant channel crossings along the alignment include Lichau Creek, which crosses 101 just north of Old Redwood Highway, Lynch Creek near downtown Petaluma, and the Petaluma River in the southern part of the alignment, where the channel is about 200 to 250 feet wide. Several other west-flowing drainages also cross the highway in the central part of the alignment between East Washington Boulevard and Old Redwood Highway.

6.1.2 *Historic Conditions*

As of 1953, most of the alignment was undeveloped and utilized as ranches or farmland. Pine Lane, West Sierra Avenue, Stony Point Road, East Washington Boulevard, Lakeville Highway, and the Northwestern Pacific (now SMART) railroad tracks are all visible at their approximate current locations. South Petaluma Boulevard (then Highway 101) is also visible south of the Petaluma River near the current alignment of Highway 101. At Orchard Lane (between Pepper Road and Old Redwood Highway, several residential or agricultural structures appear to have later been removed to accommodate the freeway. Similarly, residences along Lindberg Lane (north of Lakeville Highway) appear to have been removed when that road was truncated by the freeway. No other structures or significant improvements were observed in immediate proximity to the proposed alignment. No significant landslide scarps or other geologic features were apparent, save for a rock quarry south of the Petaluma River (since redeveloped as the Quarry Heights subdivision).

By 1965, Highway 101 has been constructed in its current alignment up the Cotati Grade, with new overpasses visible at West Sierra Avenue and Railroad Avenue. Prominent cuts within the Tolay Fault Zone along the east side of the highway are visible where quarried materials were used to construct embankments supporting the new southbound lanes.

Continuing south through Petaluma, other new interchanges are visible at Old Redwood Highway, East Washington Boulevard, Lakeville Highway, and South Petaluma Boulevard (at the current location of the Kastania Road overcrossing), and new overcrossings are in place at Corona Road and the SMART railroad tracks. Twin bridge spans are also visible where the north- and southbound freeway lanes cross the Petaluma River. Most of the land along the sides of the highway appears to remain agricultural, although some new subdivision development is visible to the east and expanded quarrying and barge-loading operations have appeared along the Petaluma River.

By 1987, both the Cotati and Kastania Tanks have been constructed and, aside from the freeway widening project started in the late 2000's, most of the project area appears in its current condition. Infill development along 101 in southern Petaluma includes a mix of residential subdivisions and light commercial/industrial facilities surrounding occasional patches of agricultural land. Similar development would later fill in the norther part of the city on either side of the highway.

Most of the former cut slopes along the east side of the Cotati Grade appear rounded and eroded, suggestive of previous landsliding, although the photo scale makes it difficult to discern individual landslide features. Although not visible in photographs, it is understood that the more recent widening project included some stabilization work in the area.

6.1.3 Current Conditions

Currently, Highway 101 is a 6-lane asphalt-paved roadway with a total width on the order of about 200-feet along most of the alignment, where recent construction has generally obscured evidence of historic distress but appears to be performing well. Significant highway embankments ranging to about 20-feet high provide approaches to highway and surface street overpasses at Old Redwood Highway, Rainer Avenue, the SMART railroad tracks, East Washington Boulevard, and Kastania Road and likewise do not exhibit any obvious evidence of instability. The nature of the drainage channels and creek crossings under the highway is mostly unclear.

6.1.4 Geology and Soils

Our field observations generally corroborate regional geologic mapping. However, it should be noted that the work area is extensively developed with existing structures, pavements, and landscaping, and little to no native soil/rock exposure exists in the urban parts of Petaluma.

Aside from the Sonoma Volcanics andesite rock underlying the knoll at the Cotati Tanks, the northern 3 miles of the alignment, generally including areas north of Pepper Road, are underlain primarily by Petaluma Formation. Smaller zones of volcanic rocks (Donnell Ranch Volcanics) and Wilson Grove Formation sedimentary rock underlie portions of the Cotati Grade along traces of the Tolay Fault Zone.

The central part of the alignment between Pepper Road and the Petaluma River is underlain predominantly by younger alluvial fan deposits, deposited at the western reaches of Lichau,

Lynch, and other unnamed creeks emanating from Sonoma Mountain to the east. Low-lying areas south of Lakeville Highway and the Petaluma River channel itself are underlain by artificial fill over Bay Mud/marsh deposits and the southernmost end of the alignment between the river and the Kastania Tanks is underlain by a combination of volcanic and metamorphic rocks.

We observed several large-scale landslides along the Cotati Grade, which are primarily clustered along the mapped traces of the Tolay Fault Zone. We did not observe evidence of the fault traces themselves, nor of any evidence of active faulting elsewhere along the alignment. The landslide features are generally more rounded and eroded, suggesting older sliding activity; however, several smaller landslides are also evident and appear to be more recent, smaller-scale reactivations within the larger slides. The older landslide deposits cross the alignment at several points along Cotati Grade, although the effects of previous highway stabilization work and degree to which ongoing landslide movement may be mitigated is currently unknown.

Additionally, several smaller landslides were also noted on the northeast facing slopes just west of the Kastania Tanks during our review of topographic hillshade data. Just south of the Quarry Heights development, near the southern end of the alignment, we observed outcrops of hard, strong, and moderately weathered garnet-blueschist bedrock.

6.2 Alignment STS-2

Similar to STS-1, this 12.3-mile alignment would deliver water from the Cotati Tanks to the Kastania Tanks located just south of Petaluma, except this alignment would follow surface streets through the densely developed City of Petaluma instead of following Highway 101. The alignment would extend west from the tanks along West Sierra Avenue then go south along Stony Point Road and Petaluma Boulevard before turning south where Petaluma Boulevard crosses beneath Highway 101. From there the alignment goes south along the Highway for about a mile before turning west to connect to the Kastania Tanks.

6.2.1 *Topography*

The alignment would originate at an elevation of +315 feet at the Cotati Tanks and terminate at an elevation of +198 feet at the Kastania Tanks, for a net -117 feet of elevation along its length. Topography throughout the alignment is typically relatively gentle. Locally steeper slopes exist just of the Cotati tanks, at the south bank of the Petaluma River and between Highway 101 and the Kastania Tanks, where slopes locally exceed 25%. Site elevations range from a minimum of about +12 feet along South Petaluma Boulevard, near Mountain View Avenue, to a maximum of about +336 feet along the Cotati Tank access road off West Sierra Avenue.

This alignment would cross the uppermost reach of the Petaluma River along Stony Point Road near Railroad Avenue and Lichau Creek near Denman Road. The alignment would then cross the Petaluma River again near the intersection of Old Redwood Highway and Petaluma Boulevard. before running south through downtown Petaluma. Five other east-flowing, apparently channelized drainages are shown to cross the alignment at the approximate locations of C, F, and H Streets, Mountain View Avenue, and Crystal Lane.

6.2.2 *Historic Conditions*

As of 1953, neither the Cotati Tanks nor the western driveway access have been constructed, but West Sierra Avenue and most of Stony Point Road and Petaluma Boulevard are visible in their approximate current alignments. North of Magnolia Avenue, development is primarily agricultural in nature and relatively sparse, while dense residential development along the southwest side of Petaluma Boulevard is in place to the south, extending to Mountain View Avenue. Development on the northeast side of Petaluma Boulevard consists primarily of industrial warehouses along the Petaluma River and railroad tracks through downtown. A few smaller industrial buildings are sited along the riverbank on the north side of Petaluma Boulevard east of Mountain View. Lands to the south, including the portion of the alignment between Petaluma Boulevard and the Kastania Tanks, are undeveloped except for the rock quarry (presently the Quarry Heights subdivision) just south of the river near the current Highway 101 crossing. No evidence of the natural alignment of the stream channels crossing the alignment between C Street and Crystal Lane is visible as those areas have already been developed.

In the intervening time, the Cotati and Kastania Tanks were constructed, and residential development has expended to the south of Mountain View Avenue to occupy the former rock quarry. Immediately east of the quarry, Highway 101 and the overcrossings at the Petaluma River were constructed between 1953 and 1965. Elsewhere, an assortment of infill development has occurred, although little has changed in terms of apparent land use in most of the project area, with industrial development dominating the northeast side Petaluma Boulevard and residential neighborhoods to the southwest. Lands north of Magnolia Avenue remain sparsely developed and, aside from reconstruction of the Stony Point Road/Petaluma Boulevard

intersection sometime between 1965 and 1987, no significant changes in surface conditions were evident in the northern and central part of the alignment.

6.2.3 Current Conditions

Most of the project site is surrounded by development which generally reflects local evidence of distress due to expansive soil and/or settlement, which is typically most evident in older improvements. North of downtown Petaluma, along Stony Point Road and Petaluma Boulevard, older roadway fills locally exhibit evidence of settlement, and older asphalt surfaces show evidence of damage due to expansive soils in lower-lying areas throughout the site. Newer construction in the southern part of the alignment appears to have generally performed well.

Where accessible, the large, channelized drainage outfalls along the river in the southern part of downtown Petaluma were observed to be large concrete box culverts on the order of 10 to 20 feet wide and of unknown depth, although the nature and exact location of these channels were shown to cross the alignment is unknown.

6.2.4 Geology and Soils

Excepting small zones of Sonoma Volcanics bedrock near the Cotati Tanks, the northern 3 miles of the alignment, generally including areas north of Pepper Road, are underlain primarily by older sedimentary materials (Sand and Gravel of Cotati) which are not well-exposed along the roadway but appear to be generally well-consolidated based on topography. The alignment is underlain mainly by a variety of younger alluvial deposits south of Pepper Road and north of Petaluma Boulevard. The low hills west of Petaluma Boulevard in the northern part of the City, generally between Stony Point Road and Western Avenue, are underlain mostly by Wilson Grove Formation; a small zone of dacite belonging to the Burdell Mountain Volcanics crops out at a low knoll north of Magnolia Avenue where Petaluma Boulevard bends slightly to the south. The alignment effectively straddles the contact between Wilson Grove and the volcanic rocks to the west and alluvial valley deposits to the east.

Low-lying areas south of East Washington Boulevard are underlain by a variety of alluvial deposits which interfinger with Bay Mud and marsh deposits, while upland areas are underlain by a combination of volcanic and metamorphic rocks.

As with STS-1, this alignment would cross several regionally mapped traces of the Tolay Fault Zone along Stony Point Road near the Cotati Grade and in northernmost Petaluma. We did not see significant evidence of historic or recent fault activity during our reconnaissance.

Several smaller landslides were observed on the northeast facing slopes just west of the Kastania tank during our review of topographic hillshade data, but no significant landslides were noted elsewhere along the alignment. We observed outcrops just south of the Quarry Heights development, near the southernmost end of the alignment, which exposed hard, strong, and moderately weathered garnet-bearing blueschist bedrock.

6.3 Alignment STAF-1

Alignment STAF-1 would deliver water from Stafford Lake west of Novato to Nicasio Reservoir via an 8-mile alignment along rural Novato Boulevard and Point Reyes-Petaluma Road. The alignment would extend northwest from Stafford Lake and along Novato Boulevard for about 3.9 miles, then turn southwest along Point Reyes-Petaluma Road for another 4.2 miles. The pipeline then would turn southeast along Nicasio Valley Road for about a half mile before discharging into Nicasio Reservoir.

6.3.1 *Topography*

The alignment would originate at an elevation of +200 feet at Stafford Lake and terminate at an elevation of +169 feet at Nicasio Reservoir, for a net elevation change of -41 feet. Topography throughout the alignment is typically relatively gentle, although steeper grades approaching 10% exist west of Stafford Lake and at the crest of the pass along Point Reyes-Petaluma Road between Hicks Valley and Nicasio Valley. Digital hydrographic data indicates the alignment would cross a total of approximately 50 culverts or tributary channels to Novato Creek, Arroyo Sausal, and Nicasio Creek.

6.3.2 *Historic Conditions*

In the earliest available photos from 1952, the area is sparsely developed with farms and dairy ranches. Stafford Lake, Novato Boulevard, Point Reyes-Petaluma Road, and Nicasio Reservoir are all visible, with Novato Boulevard appearing to be recently re-graded or paved. By 1965, the fire station in Hicks Valley had been constructed, but no other significant changes in land use or surface conditions are evident in the photographs between 1965 and the present day.

6.3.3 *Current Conditions*

The project area consists almost entirely of 2-lane, asphalt-paved rural roads which are typically in relatively good condition. We observed evidence of asphalt distress where a variety of landslides have locally impacted the roadway west of Stafford Lake and east of Nicasio Reservoir. Most of the culverts are of concrete or metal construction, between 1-and 3 feet in diameter, and at relatively shallow burial depths. The most significant channels are the main channel of Novato Creek about 1.25 miles southeast of Point Reyes-Petaluma Road and the main channel of Nicasio Creek near the intersection between Point Reyes-Petaluma and Nicasio Valley Roads, where channels are typically 10-to 15 feet deep and of similar width. Several other concrete box culverts and ranch/cattle under crossings, typically 7- to 10-feet wide and deep, were noted in the southern half of the alignment.

6.3.4 *Geology and Soils*

Most of the alignment along Novato Boulevard and within Hicks and Nicasio Valleys is underlain by Franciscan mélange bedrock, consisting of resistant masses of chert and greenstone surrounded by a matrix of sheared sandstone and shale. The steep slopes separating Hicks and Nicasio Valleys are underlain by harder sandstone and shale, and a few prominent zones and outcrops of hard chert bedrock within the mélange are visible as it descends westward into Nicasio Valley. Relatively small zones of alluvial channel deposits exist along Novato Creek and the shores of Stafford Lake, as well as at the Nicasio Creek channel adjacent to the reservoir. More significant portions of the alignment are underlain by alluvium along Point Reyes-Petaluma Road where it parallels Arroyo Sausal between Novato Boulevard and the pass into Nicasio Valley.

Aside from ancient, inactive faults forming contacts between Franciscan Mélange and other Franciscan rocks, no faults are mapped regionally along the alignment and no significant evidence of historic or recent fault activity was noted during our reconnaissance.

Cut slopes along the alignment typically exhibit widespread erosion and local instability, and several landslides were mapped along the uphill side of the road in steeper areas west of Stafford Lake and east of Nicasio Reservoir. Slides along the downslope side of the road were mapped in the same areas and fill slopes locally underlying the roads throughout the alignment commonly exhibit evidence of settlement and attendant asphalt distress.

6.4 Alignment STAF-2

Alignment STAF-2 would deliver water from Stafford Lake west of Novato to Soulajule Reservoir via a 6.9-mile alignment through rural northern Marin. The alignment would extend northwest from Stafford Lake and along Novato Boulevard for about 3.9 miles. The alignment would then briefly jog east on Point Reyes-Petaluma Road before continuing northwest along Hicks Valley Road and Three Peaks Fire Road for about 2.5 miles to discharge at the east end of Soulajule Reservoir. The pipeline then would turn southeast along Nicasio Valley Road for about a half mile before discharging into Nicasio Reservoir.

6.4.1 *Topography*

The alignment would originate at an elevation of +200 feet at Stafford Lake and terminate at an elevation of +330 feet at Soulajule Reservoir in rural northern Marin, gaining a net 130 feet of elevation along its length. Topography throughout the alignment is typically relatively gentle, although steeper grades approaching 10% exist west of Stafford Lake where Novato Boulevard climbs out of the Novato Creek watershed and into Hicks Valley.

Digital hydrographic data indicates the alignment would cross roughly 40 culverts or tributary channels to Novato creek and Arroyo Sausal that could pose conflicts with buried pipelines. Of these, the only significant channel crossing is at Novato Creek, about 1.25-miles south of Point Reyes-Petaluma Road.

6.4.2 *Historic Conditions*

In the earliest available photos from 1952, the area is sparsely developed with farms and dairy ranches. Stafford Lake, Novato Boulevard, Point Reyes-Petaluma Road, Hicks Valley Road, Three Peaks Fire Road, and Soulajule Reservoir are all visible, with Novato Boulevard appearing to be recently re-graded or paved. By 1965, the fire station in Hicks Valley had been constructed, but no other significant changes in land use or surface conditions are evident in the photographs between 1965 and the present day.

6.4.3 *Current Conditions*

The project area consists almost entirely of 2-lane, asphalt-paved rural roads which are typically in relatively good condition. We observed evidence of asphalt distress where a variety of landslides have locally impacted the roadway west of Stafford Lake and east of Nicasio Reservoir. Most of the culverts are of concrete or metal construction, between 1-and 3 feet in diameter, and at relatively shallow burial depths. The most significant channels are the main channel of Novato Creek about 1.25 miles southeast of Point Reyes-Petaluma Road and the main channel of Nicasio Creek near the intersection between Point Reyes-Petaluma and Nicasio Valley Roads, where channels are typically 10-to 15 feet deep and of similar width. Several other concrete box culverts and ranch/cattle under crossings, typically 7- to 10-feet wide and deep, were noted in the southern half of the alignment. We were not afforded access to the private ranchlands along Three Peaks Fire Road and Soulajule Reservoir at the north end of the alignment. Review of satellite imagery and LiDAR data in that area suggests that creekbanks along the south edge of the gravel road are very steep and commonly affected by erosion, scour, and instability.

6.4.4 *Geology and Soils*

As with STAF-1, this alignment is mostly underlain by Franciscan mélange bedrock, consisting of resistant masses of chert and greenstone surrounded by a matrix of sheared sandstone and shale. No significant outcrops of chert were mapped along the alignment itself, though several

are shown near the alignment in the southern part of the Hicks Valley area. A small zone of alluvial deposits underlies a few hundred feet of the alignment near the south end of Hicks Valley Road, and Three Peaks Fire Road is probably intermittently underlain by alluvial soils where it follows the creek into Soulajule Reservoir.

Aside from ancient, inactive faults forming contacts between Franciscan Mélange and other Franciscan rocks, no faults are mapped regionally along the alignment and no significant evidence of historic or recent fault activity was noted during our reconnaissance.

Cut slopes along the alignment typically exhibit widespread erosion and local instability, and several landslides were mapped along the uphill side of the road in steeper areas west of Stafford Lake and east of Nicasio Reservoir. Slides along the downslope side of the road were mapped in the same areas and fill slopes locally underlying the roads throughout the alignment commonly exhibit evidence of settlement and attendant asphalt distress.

6.5 Alignment COTATI-1

Alignment COTATI-1 would deliver water from the Cotati Tanks to Nicasio via an approximately 24.2-mile alignment that traverses rural 2-lane, asphalt paved roadways through areas typified by relatively sparse residential and agricultural development. The alignment would extend west from the tanks along West Sierra Avenue then go south along Stony Point, Meachem and Pepper Roads to Two Rock Valley. The alignment would briefly turn west on Bodega Avenue before continuing south along Spring Hill Road. The alignment would turn south from Spring Hill Road and traverse ranchlands near Laguna Lake to connect with Chileno Valley Road. The route would continue south along Chileno Valley, then along Wilson Hill Road and Hicks Valley Road to the intersection with Point Reyes-Petaluma Road. Then the pipe would go west along Point Reyes Petaluma Road for about 4 miles before finally turning south along Nicasio Valley Road and terminating at the western shore of Nicasio Reservoir.

6.5.1 *Topography*

The alignment would originate at an elevation of +315 feet at the Cotati Tanks and terminate at an elevation of +194 feet at Nicasio Reservoir, for a net -126-foot elevation change from start to end. Topography throughout the alignment is variable, ranging from relatively flat valley bottoms to moderately steep grades, with the steepest sections typically within Marin County in the southern half of the alignment. Site elevations range from a minimum of about +80 feet near the intersection of Bodega Avenue and Spring Hill Road to a maximum of +735 feet at the crest of Wilson Hill Road.

Digital hydrographic data indicates the alignment would cross a total of approximately 75 culverts or channels that could pose conflicts with buried pipelines. Although the majority of these are relatively shallow, small-diameter culverts draining roadside ditches, several more significant crossings exist at Stemple Creek, San Antonio Creek, Salmon Creek, Arroyo Sausal, and Nicasio Creek.

6.5.2 *Historic Conditions*

Between 1942 and the early 1950s, all of the existing roads along the alignment are in place, although they appear to be largely unpaved/unimproved. Development along the route is sparse and almost exclusively agricultural in nature, with smaller farms and orchards scattered amidst expansive ranchlands. Laguna Lake and Nicasio Reservoir are visible and appear as they do today. A few minor slump and slide scarps are apparent on the north/uphill side of Wilson Hill Road near the intersection with Hicks Valley Road.

By 1965, the rock quarry along Stony Point Road north of Meachem Road had opened, and a new landslide is visible on the slope west of Meachem Road, just south of the present landfill entrance. The landfill has not yet been constructed; however, the Two Rock Coast Guard Station has. By the mid-1980's all of the major roads had been paved, the landfill is clearly visible, and a small section of Meachem Road at the landfill entrance had been re-aligned to the west side of the creek channel. Since the mid-1980's no other significant changes are readily apparent in aerial photos.

6.5.3 Current Conditions

The project area consists almost entirely of 2-lane, asphalt-paved rural roads which are typically in relatively good condition. We observed evidence of local asphalt distress where a variety of landslides have impacted the roadway at Wilson Hill and east of Nicasio Reservoir. Most of the culverts along the alignment are of concrete or metal construction, between 1-and 3 feet in diameter, and at relatively shallow burial depths. The most significant channels are at Stemple Creek, San Antonio Creek, Salmon Creek, Arroyo Sausal and Nicasio Creek, where channels are typically 10- to 15 feet deep. Most channels are between 10- and 20 feet wide, while the Stemple Creek channel at the downstream Pepper Road crossing is about 70 feet wide. Several other concrete box culverts and ranch/cattle under crossings, typically 7- to 10-feet wide and deep, were noted at various points along the alignment.

We were not afforded access to the private ranchlands surrounding Laguna Lake between Spring Hill Road and Chileno Valley Road. Review of satellite imagery and LiDAR data suggests that ranch roads are relatively unimproved.

6.5.4 Geology and Soils

Excepting small zones of Sonoma Volcanics bedrock near the Cotati Tanks, the alignment is underlain mainly by Wilson Grove and Franciscan bedrock. Various alluvial deposits are mapped in lower-lying valley areas while upland areas are underlain primarily by the Wilson Grove Formation in Sonoma County and by Franciscan sandstone, graywacke, chert, greenstone, and mélange in the southern part of the project area.

The most significant alluvial deposits underlie portions of the alignment along Stemple Creek, San Antonio Creek, Salmon Creek, and Arroyo Sausal, while more widespread alluvium underlies areas around Laguna Lake and the bottom of Chileno Valley. Most areas mapped as mélange expose weathered sandstone and shale in between sparse outcrops of chert and greenstone. Zones of metamorphic rocks underlie the eastern lower flank of Wilson Hill and parts of the alignment in the Laguna Lake area. At the southern end of the alignment, roadcuts along Nicasio Valley road expose hard and strong weathered sandstone within the mélange. Smaller, isolated zones and outcrops of hard and strong chert are also exposed along Point Reyes-Petaluma Road, north of Nicasio Valley Road.

We observed numerous landslides on the slopes along Wilson Hill Road and Hicks Valley Road. The landslides generally originate at the tops of the intervening drainage channels and vary in size and apparent age. The alignment crosses several mapped landslides along Wilson Hill Road and along Point Reyes-Petaluma Road north of Nicasio Valley Road. The pervasive hummocky topography exhibited by the larger slides may also be a consequence of differential erosion of the underlying Franciscan Mélange bedrock.

The alignment crosses several mapped traces of the Tolay, Bloomfield, and Americano Creek Fault Zones; however, no significant evidence of historic or recent fault activity was noted during our reconnaissance.

6.6 Alignment COTATI-2

Alignment COTATI-2 would deliver water from the Cotati Tanks to Soulajule Reservoir via a 20.5-mile alignment traversing rural 2-lane, asphalt paved roadways through southern Sonoma and northern Marin Counties. The alignment would extend west from the tanks along West Sierra Avenue then go south along Stony Point, Meachem and Pepper Roads to Two Rock Valley. The alignment would briefly turn west on Bodega Avenue before continuing south along Spring Hill Road. The alignment would turn south from Spring Hill Road and traverse ranchlands near Laguna Lake to connect with Chileno Valley Road. The route would continue south along Chileno Valley, then along Wilson Hill Road and Hicks Valley Road to the intersection with Three Peaks Fire Road before turning west for just under 2 miles and discharging into Arroyo Sausal at the east end of Soulajule Reservoir.

6.6.1 *Topography*

The pipeline would originate at an elevation of +315 feet at the Cotati Tanks and terminate at an elevation of +330 feet at Soulajule Reservoir, for a net elevation gain of 15 feet along the alignment. Topography throughout the alignment is variable, ranging from relatively flat valley bottoms to moderately steep grades, with the steepest sections typically within Marin County in the southern half of the alignment. Site elevations range from a minimum of about +80 feet near the intersection of Bodega Avenue and Spring Hill Road to a maximum of +735 feet at the crest of Wilson Hill Road.

Digital hydrographic data indicates the alignment would cross a total of approximately 60 culverts or channels that could pose conflicts with buried pipelines. Although the majority of these are relatively shallow, small-diameter culverts draining roadside ditches, we noted several more significant crossings that may require special consideration. These locations include Stemple Creek, San Antonio Creek, Salmon Creek, and Arroyo Sausal.

6.6.2 *Historic Conditions*

By 1965, the rock quarry along Stony Point Road north of Meacham Road had opened, and a new landslide is visible on the slope west of Meacham Road, just south of the present landfill entrance. The landfill has not yet been constructed; however, the Two Rock Coast Guard Station has. By the mid-1980's all of the major roads had been paved, the landfill is clearly visible, and a small section of Meacham Road at the landfill entrance had been re-aligned to the west side of the creek channel. Since the mid-1980's no other significant changes are readily apparent in aerial photos.

6.6.3 *Current Conditions*

The project area consists almost entirely of 2-lane, asphalt-paved rural roads which are typically in relatively good condition. We observed evidence of local asphalt distress where a variety of landslides have impacted the roadway at Wilson Hill and east of Nicasio Reservoir. Most of the culverts along the alignment are of concrete or metal construction, between 1-and 3 feet in diameter, and at relatively shallow burial depths. The most significant channels are at Stemple Creek, San Antonio Creek, Salmon Creek, Arroyo Sausal and Nicasio Creek, where channels are typically 10- to 15 feet deep. Most channels are between 10- and 20 feet wide, while the Stemple Creek channel at the downstream Pepper Road crossing is about 70 feet wide. Several other concrete box culverts and ranch/cattle under crossings, typically 7- to 10-feet wide and deep, were noted at various points along the alignment.

We were not afforded access to the private ranchlands surrounding Laguna Lake between Spring Hill Road and Chileno Valley Road. Review of satellite imagery and LiDAR data suggests that ranch roads are relatively unimproved. Similarly, we also could not access private lands along Three Peaks Fire Road and Soulajule Reservoir at the north end of the alignment. Review of LiDAR data in that area suggests that creekbanks along the south edge of the road are very steep and commonly affected by erosion, scour, and instability.

6.6.4 Geology and Soils

Excepting small zones of Sonoma Volcanics bedrock near the Cotati Tanks, the alignment is underlain mainly by Wilson Grove and Franciscan bedrock. Various alluvial deposits are mapped in lower-lying valley areas while upland areas are underlain primarily by the Wilson Grove Formation in Sonoma County and by Franciscan sandstone, graywacke, chert, greenstone, and mélange in the southern part of the project area.

The most significant alluvial deposits underlie portions of the alignment along Stemple Creek, San Antonio Creek, Salmon Creek, and Arroyo Sausal, while more widespread alluvium underlies areas around Laguna Lake and the bottom of Chileno Valley. Most areas mapped as mélange expose weathered sandstone and shale in between sparse outcrops of chert and greenstone. Zones of metamorphic rocks underlie the eastern lower flank of Wilson Hill and parts of the alignment in the Laguna Lake area. At the southern end of the alignment, roadcuts along Nicasio Valley road expose hard and strong weathered sandstone within the mélange. Smaller, isolated zones and outcrops of hard and strong chert are also exposed along Point Reyes-Petaluma Road, north of Nicasio Valley Road.

We observed numerous landslides on the slopes along Wilson Hill Road and Hicks Valley Road. The landslides generally originate at the tops of the intervening drainage channels and vary in size and apparent age. The alignment crosses several mapped landslides along Wilson Hill Road

6.7 Alignment PETA-1

Alignment PETA-1 would deliver water from the Petaluma Aqueduct in northern Novato to Nicasio Reservoir via a 13.2-mile alignment extending through developed and rural portions of Novato and unincorporated Marin County. The alignment would originate at the aqueduct at the intersection of Redwood Boulevard and San Marin Drive in northern Novato and run west along San Marin Drive for about 2.5 miles through densely developed residential neighborhoods. The alignment would turn northwest on Novato Boulevard, traversing more rural areas and passing Stafford Lake to the intersection at Point Reyes-Petaluma Road. There, the pipeline would turn southwest, running along Arroyo Sausal, crossing into Nicasio Valley, then briefly turning east on Nicasio Valley Road before discharging into Nicasio Reservoir.

6.7.1 *Topography*

The pipeline would originate at an elevation of +36 feet at the Petaluma Aqueduct and terminate at an elevation of +200 feet at Nicasio Reservoir, for a net elevation gain of 164 feet along the alignment. Topography throughout the alignment is typically relatively gentle, excepting moderate grades west of Stafford Lake, where the alignment climbs out of the Novato Creek watershed, and on the pass between Hicks and Nicasio Valleys.

Digital hydrographic data indicates the alignment would cross a total of approximately 70 culverts or channels that could pose conflicts with buried pipelines. Most of these are relatively small-diameter culverts, and the only more significant channel crossings are the channels at Novato Creek west of Stafford Lake and Nicasio Creek at the southwest end of the alignment.

6.7.2 *Historic Conditions*

In the earliest 1952 photos, northern Novato remains largely undeveloped. The current alignment of Highway 101 has not been constructed yet, although Redwood Boulevard appears in its current configuration. Atherton Avenue is visible to the east, but the area of San Marin Drive remains occupied by undeveloped open space and ranchland. Farther west, Novato Boulevard, Stafford Lake, Point Reyes-Petaluma Road, and Nicasio Reservoir are all visible in more or less their existing conditions.

By 1965, San Marin Drive had been constructed, although Highway 101 has not yet been realigned and reconstructed. Subdivisions north and south of San Marin Drive are in various stages of construction. As of 1988, residential development has filled in along both sides of the road, and both San Marin High School and the nearby shopping center are now in place. The Fireman's Fund campus at the east end of the alignment appears partly complete, and the interchange at Highway 101/San Marin Drive/Redwood Boulevard appears in its present configuration. Other than minor residential development immediately north of San Marin High School and the appearance of County Park facilities at Stafford Lake and the Hicks Valley fire station, no other significant changes were observed elsewhere along the alignment.

6.7.3 *Current Conditions*

The southern part of the alignment along San Marin Drive traverses areas of medium-dense suburban residential development, while remaining areas between San Marin Drive and Nicasio Reservoir generally consist of rural 2-lane roads. Conditions in developed parts of Novato are typified by moderate asphalt distress but a general absence of significant slope instability along

the alignment. We observed evidence of asphalt distress where a variety of landslides have locally impacted the roadway west of Stafford Lake and east of Nicasio Reservoir.

Most of the culverts along Novato Boulevard and Point Reyes-Petaluma Road are of concrete or metal construction, between 1-and 3 feet in diameter, and at relatively shallow burial depths. The most significant channel crossings are the main channel of Novato Creek about 1.25 miles southeast of Point Reyes-Petaluma Road and the main channel of Nicasio Creek near the intersection between Point Reyes-Petaluma and Nicasio Valley Roads, where channels are typically 10-to 15 feet deep and of similar width. Several other concrete box culverts and ranch/cattle under crossings, typically 7- to 10-feet wide and deep, were noted along Novato Boulevard west of Stafford Lake.

6.7.4 Geology and Soils

The lower lying portions of the alignment along San Marin Drive, the lower reaches of Novato Creek near and south of Stafford Lake, and within Hicks Valley and the Arroyo Sausal drainage, are underlain by alluvial soils. Upland areas along San Marin Drive and south of Stafford Lake are underlain by Franciscan mélange, metamorphic rocks, and serpentinite. Particularly hard meta-greenstone and meta-chert bedrock is exposed at the Stafford Lake Dam, while scattered outcrops of hard chert and serpentinite are visible behind San Marin High School and the neighboring subdivisions. Upland areas along Novato Boulevard north of Stafford Lake are underlain by mélange, and the ridge between Hicks and Nicasio Valleys is underlain by more competent sandstone and shale rock. At the southern end of the alignment, roadcuts along Nicasio Valley road expose relatively hard and strong weathered sandstone. Smaller, isolated outcrops of hard and strong chert are also exposed along Point Reyes-Petaluma Road, north of Nicasio Valley Road.

Several landslides were observed on the north shore of Stafford Lake, just west of the dam, both upslope and downslope of the road. A variety of retaining walls are present along the south edge of the road where it was previously undermined. Several landslides were also observed upslope of the road between the dam and San Marin Drive. We also observed several landslides along Point Reyes-Petaluma Road, about 1.5 miles north of Nicasio Valley Road.

Aside from ancient, inactive faults that form contacts between individual Franciscan rock units, no significant faults are mapped along the alignment, and no evidence of historic or recent faulting was observed in the area during our reconnaissance.

6.8 Alignment PETA-2

Alignment PETA-2 would deliver water from the Petaluma Aqueduct in northern Novato to Soulajule Reservoir via a 12.0-mile alignment extending through developed and rural portions of Novato and unincorporated Marin County. The alignment would originate at the aqueduct at the intersection of Redwood Boulevard and San Marin Drive in northern Novato and run west along San Marin Drive for about 2.5 miles through densely developed residential neighborhoods. The alignment would turn northwest on Novato Boulevard, traversing more rural areas and passing Stafford Lake to the intersection at Point Reyes-Petaluma Road. There, the pipeline would turn northeast, running along Pt. Reyes-Petaluma Road before turning northwest and continuing along Hicks Valley Road. The pipeline would continue straight at the intersection with Three Peaks Fire Road, following along Three Peaks Fire Road for just under 2 miles before discharging into Arroyo Sausal at the east end of Soulajule Reservoir.

6.8.1 Topography

The pipeline would originate at an elevation of +36 feet at the Petaluma Aqueduct and terminate at an elevation of +330 feet at Soulajule Reservoir, for a net elevation gain of 294 feet along the alignment. Topography throughout the alignment is typically relatively gentle, although steeper grades approaching 10% exist west of Stafford Lake where Novato Boulevard climbs out of the Novato Creek watershed and into Hicks Valley.

Digital hydrographic data indicates the alignment would cross a total of approximately 47 culverts or channels that could pose conflicts with buried pipelines. Most of these are relatively small-diameter culverts, and the only more significant channel crossing is the channel at Novato Creek, about 1.25-miles south of Point Reyes-Petaluma Road.

6.8.2 Historic Conditions

In the earliest 1952 photos, northern Novato remains largely undeveloped. The current alignment of Highway 101 has not been constructed yet, although Redwood Boulevard appears in its current configuration. Atherton Avenue is visible to the east, but the area of San Marin Drive remains occupied by undeveloped open space and ranchland. Farther west, Novato Boulevard, Stafford Lake, Point Reyes-Petaluma Road, and Soulajule Reservoir are all visible in more or less their existing conditions.

By 1965, San Marin Drive had been constructed, although Highway 101 has not yet been realigned and reconstructed. Subdivisions north and south of San Marin Drive are in various stages of construction. As of 1988, residential development has completely filled in along both sides of the road, and both San Marin High School and the nearby shopping center are now in place. The Fireman's Fund campus at the east end of the alignment appears partly complete, and the interchange at Highway 101/San Marin Drive/Redwood Boulevard appears in its present configuration. Other than minor residential development immediately north of San Marin High School and the appearance of County Park facilities at Stafford Lake and the Hicks Valley fire station, no other significant changes were observed elsewhere along the alignment.

6.8.3 Current Conditions

The southern part of the alignment along San Marin Drive traverses areas of medium-dense suburban residential development, while remaining areas between San Marin Drive and Nicasio Reservoir generally consist of rural 2-lane roads. Conditions in developed parts of Novato are typified by moderate asphalt distress but a general absence of significant slope instability along

the alignment. We observed evidence of asphalt distress where a variety of landslides have locally impacted the roadway west of Stafford Lake and east of Nicasio Reservoir.

Most of the culverts along Novato Boulevard are of concrete or metal construction, between 1- and 3 feet in diameter, and at relatively shallow burial depths. The most significant channel crossing is the main channel of Novato Creek about 1.25 miles southeast of Point Reyes-Petaluma Road, where the channel is 10-to 15 feet deep and of similar width. Several other concrete box culverts and ranch/cattle under crossings, typically 7- to 10-feet wide and deep, were noted along Novato Boulevard west of Stafford Lake. We could not access private lands along Three Peaks Fire Road and Soulajule Reservoir at the north end of the alignment. Review of LiDAR data in that area suggests that creekbanks along the south edge of the road are very steep and commonly affected by erosion, scour, and instability.

6.8.4 Geology and Soils

The lower lying portions of the alignment along San Marin Drive, the lower reaches of Novato Creek near and south of Stafford Lake, and within Hicks Valley and the Arroyo Sausal drainage, are underlain by alluvial soils. Upland areas along San Marin Drive and south of Stafford Lake are underlain by Franciscan mélange, metamorphic rocks, and serpentinite. Particularly hard meta-greenstone and meta-chert bedrock is exposed at the Stafford Lake Dam, while scattered outcrops of hard chert and serpentinite are visible behind San Marin High School and the neighboring subdivisions. Upland areas along Novato Boulevard north of Stafford Lake are underlain by mélange, and the ridge between Hicks and Nicasio Valleys is underlain by more competent sandstone and shale rock. At the southern end of the alignment, roadcuts along Nicasio Valley road expose relatively hard and strong weathered sandstone. Smaller, isolated outcrops of hard and strong chert are also exposed along Point Reyes-Petaluma Road, north of Nicasio Valley Road.

Several landslides were observed on the north shore of Stafford Lake, just west of the dam, both upslope and downslope of the road. A variety of retaining walls are present along the south edge of the road where it was previously undermined. Several landslides were also observed upslope of the road between the dam and San Marin Drive.

6.9 Alignment PETA-4

Alignment PETA-4 would deliver water from the Petaluma Aqueduct along Highway 101 near the Marin-Sonoma County Line to both Nicasio and Soulajule Reservoirs via a 14.8-mile alignment extending through rural portions of unincorporated Marin and Sonoma Counties. The alignment would originate at the aqueduct at the intersection of Highway 101 and San Antonio Road and run west along San Antonio Road for just under 4 miles. The alignment would turn southwest on Red Hill Road/Point Reyes-Petaluma Road for nearly 4 miles, crossing Red Hill Hicks Valley. At the intersection with Hicks Valley Road, the alignment would split, with one pipeline extending north along Hicks Valley Road and Three Peaks Fire Road for about 2.5 miles to Soulajule Reservoir, and the other continuing west along Point Reyes-Petaluma Road for about 4.5-miles to Nicasio Reservoir.

6.9.1 Topography

The pipeline would originate at an elevation of +30 feet at the Petaluma Aqueduct and terminate at elevations of +330 feet at Soulajule Reservoir and +200 feet at Nicasio Reservoir. for a maximum net elevation gain of 300 feet along the alignment to Soulajule Reservoir. Site elevations range from a minimum of about +24 feet near the east end of San Antonio Road to a maximum of +656 feet at the crest of Red Hill. Topography throughout the alignment is typically relatively gentle, although steeper grades approaching 10% exist along Red Hill Road and at the pass between Hicks and Nicasio Valleys.

Digital hydrographic data indicates the alignment would cross a total of approximately 74 culverts or channels that could pose conflicts with buried pipelines. Most of these are relatively small-diameter culverts, and the only more significant channel crossing is at San Antonio Creek. along Point Reyes-Petaluma Road, about a quarter mile west of San Antonio Road., where the channel is about 100 feet wide and 10 feet deep. A few other concrete box culverts/cattle under crossings, typically about 7- to 10 feet wide and deep, exist along San Antonio and Red Hill Roads.

6.9.2 Historic Conditions

In the earliest available photos from 1952, the area is sparsely developed with farms and dairy ranches. Highway 101 is a still a 4-lane road but passes San Antonio Road in its present alignment. San Antonio, Red Hill/Point Reyes-Petaluma, and Hicks Valley Road all appear to be unpaved. Soulajule and Nicasio Reservoirs are all visible and appear largely as they do at present. By 1965, the fire station in Hicks Valley had been constructed and all of the roads except for San Antonio Road appear to have been paved. New fill embankments and cut slopes are visible along the pass over Red Hill. A portion of San Antonio Road at the intersection with Red Hill/Point Reyes-Petaluma Road is paved by 1971, but it is unclear when the rest of the road was paved.

Photos from 1982 show extensive raveling of cut slopes as well as at least one apparent landslide just east of the crest of Red Hill that appears to have partly undermined the downhill side of the roadway. No other significant changes in land use or surface conditions aside from the ongoing Highway 101 widening project are evident in the photographs between 1965 and the present day.

6.9.3 Current Conditions

The entirety of the alignment traverses rural 2-lane roads through areas typified by agricultural development. Conditions are typified by moderate asphalt distress with locally more severe

distress where moderately plastic soils exist locally along San Antonio Creek. We also observed evidence of more pronounced asphalt distress where a variety of landslides have locally impacted the roadway along the passes east and west of Hicks Valley, and where portions of Point-Reyes Petaluma, Red Hill, and San Antonio Roads are aligned parallel to incised channels.

Most of the culverts along the alignments are of concrete or metal construction, between 1-and 3 feet in diameter, and at relatively shallow burial depths. The most significant crossing is the main channel of San Antonio Creek about 0.25 miles west of San Antonio Road, where the channel is about 10 feet deep and 100 feet wide. A few other concrete box culverts/cattle under crossings, typically about 7- to 10 feet wide and deep, exist along San Antonio and Red Hill Roads. We could not access private lands along Three Peaks Fire Road and SoulaJule Reservoir at the north end of the alignment. Review of LiDAR data in that area suggests that creekbanks along the south edge of the road are very steep and commonly affected by erosion, scour, and instability.

6.9.4 *Geology and Soils*

The lower lying portions of the alignment along San Antonio Creek, and within Hicks Valley and the Arroyo Sausal drainage are underlain by alluvial soils. Upland parts of the alignment are typically underlain by Franciscan mélange and metamorphic rocks. Particularly hard meta-sandstone bedrock is locally exposed in cut slopes near the crest of Red Hill, while scattered outcrops of hard meta-chert are visible in areas underlain by mélange, most prominently along Point Reyes-Petaluma Road near Nicasio Reservoir. A small area near the west end of San Antonio Road is underlain by siltstone and sandstone of the Great Valley Sequence.

Several older landslides were observed along San Antonio Road and Red Hill Road, mostly along the upslope side of the road. More widespread and more recent landsliding was observed at Red Hill, impacting both the upslope and downslope sides of the road. A variety of retaining walls are present along the south/downslope edge of the road where it was previously undermined.

7.0 GEOLOGIC HAZARDS

This section summarizes our review of commonly considered geologic hazards, discusses their potential impacts on the planned improvements, and provides recommendations for mitigation where warranted. The primary geologic hazards which could affect the proposed project are strong seismic ground shaking, liquefaction, lurching, settlement, slope instability, erosion, and naturally occurring asbestos. Other geologic hazards are judged relatively insignificant with regard to the proposed project. Each geologic hazard considered is discussed in further detail in the following paragraphs.

7.1 Fault Surface Rupture

Under the Alquist-Priolo Earthquake Fault Zoning Act, the California Division of Mines and Geology (now known as the California Geological Survey) produced 1:24,000 scale maps showing known active and potentially active faults and defining zones within which special fault studies are required. The nearest known active faults to the site are the Hayward-Rodgers Creek and San Andreas Faults, located approximately 4.3 miles (7.0 km) northeast and 4.5 miles (7.2 km) southwest of the nearest proposed alignments, respectively. No portions of any of the proposed alignments currently lie within an Alquist-Priolo Earthquake Fault Zone, and no known active faults cross any of the proposed alignments.

Several surface traces of the Tolay Fault Zone cross alignments STS-1 and STS-2 along that Cotati Grade and in the northern part of Petaluma. The Americano Creek Fault and two mapped traces of the Bloomfield Fault Zone cross the northern parts of the COTATI-1 and COTATI-2 alignments along Meachem and Pepper Roads. The Burdell Mountain Fault follows the western end of alignment PETA-4 along San Antonio Road.

As discussed previously, these other mapped faults in the project area are not considered significant seismogenic sources, and no evidence indicative of active or significant historic faulting was observed during our reconnaissance for this study, or during previous work in the area. Despite the apparent capability of these smaller fault zones to generate minor seismic shaking, the relatively small magnitudes of the historic earthquakes and generally limited extent of the mapped fault zones suggest that the likelihood of fault surface rupture along the faults in the project area is generally remote. We therefore judge that the risk of damage due to fault surface rupture along any of the proposed alignments is low.

Evaluation: Less than significant.

Recommendations: No special engineering measures are anticipated.

7.2 Seismic Shaking

The site will likely experience seismic ground shaking similar to other areas in the seismically active Bay Area. The intensity of ground shaking will depend on the characteristics of the causative fault, distance from the fault, the earthquake magnitude and duration, and site-specific geologic conditions. Estimates of peak ground accelerations are based on either deterministic or probabilistic methods. Deterministic methods are typically used for residential developments, while “critical” facilities and larger infrastructure projects commonly use probabilistic analyses.

7.2.1 Deterministic Seismic Hazard Analysis

Deterministic Seismic Hazard Analysis (DSHA) predicts the intensity of earthquake ground motions by analyzing the characteristics of nearby faults, distance to the faults and rupture zones, earthquake magnitudes, earthquake durations, and site-specific geologic conditions. Empirical relations (Abrahamson, Silva & Kamai, Boore, Stewart, Seyhan & Atkinson, Campbell & Borzognia, and Chiou & Youngs, (2014)) for generalized “stiff soil” subsurface conditions were utilized to provide approximate estimates of median peak site accelerations. A summary of the principal active faults affecting the site, their closest distance, moment magnitude of characteristic earthquake, probable median accelerations and plus one standard deviation (+1 σ), peak ground accelerations (PGA) for earthquakes on faults near the site are shown in Table 2.

Table 2 – Deterministic Peak Ground Accelerations for Active Faults

Fault	Moment Magnitude for Characteristic Earthquake ¹	Closest Estimated Distance (km) ²	Median Peak Ground Acceleration (g) ^{3,4}	Median PGA +1 Std Dev (g) ^{3,4}
Hayward-Rodgers Creek	7.6	7.0	0.40	0.68
San Andreas	8.0	7.2	0.42	0.71
Maacama	7.6	20.5	0.25	0.42
San Gregorio	7.4	21.5	0.22	0.38
West Napa	7.0	24.3	0.18	0.30

- 1) Values determined using USGS Earthquake Scenario Map (BSSC 2014), 2024.
- 2) Values derived from USGS Quaternary Fault and Fold Database, <https://www.usgs.gov/programs/earthquake-hazards/faults>, accessed 2024, and measured from the fault to the nearest point of any of the proposed alignments.
- 3) Values determined using $V_{S30} = 260$ m/s for “Site Class D”. See Section 9.1 of this report for additional discussion regarding site classification.
- 4) Abrahamson, Silva and Kamai (2014); Boore, Stewart, Seyhan and Atkinson (2014); Campbell and Borzognia (2014); and Chiou and Youngs (2014).

7.2.2 Probabilistic Seismic Hazard Analysis

Probabilistic Seismic Hazard Analysis (PSHA) analyzes all possible earthquake scenarios while incorporating the probability of each individual event to occur. The probability is determined in the form of the recurrence interval, which is the average time for a specific earthquake acceleration to be exceeded. The design earthquake is not solely dependent on the fault with the closest distance to the site and/or the largest magnitude, but rather the probability of given seismic events occurring on both known and unknown faults.

We calculated the PGA for two separate probabilistic conditions, the 2% chance of exceedance in 50 years (2,475-year statistical return period), and the 10% chance of exceedance in 50 years (475-year statistical return period), utilizing the USGS Uniform Hazard Tool (v4.0X, accessed 2024). The results of the probabilistic analyses are presented below in Table 3.

Table 3 – Probabilistic Peak Ground Accelerations for Active Faults¹

	<u>Statistical Return Period</u>	<u>Magnitude</u>	<u>PGA</u>
2% in 50 years	2,475 years	7.7	0.79 g
10% in 50 years	475 years	7.1	0.46 g

1) Values determined using USGS Unified Hazard Tool, <https://earthquake.usgs.gov/hazards/interactive/>, accessed 2024.

The potential for strong seismic shaking in the project area is high. Due to their proximity and historic rates of activity, the Hayward/Rodgers Creek and San Andreas Faults generally present the highest potential for severe ground shaking. The significant adverse impact associated with strong seismic shaking is potential damage to structures and improvements.

Evaluation: Less than significant with special engineering measures.

Recommendations: Any new structures should be designed in accordance with the provisions of the latest edition of the appropriate design code (2022 CBC) or subsequent codes in effect when the final design occurs. In general, flexible pipes and connectors that can withstand variable ground motions during an earthquake should be considered wherever practical.

Seismic design criteria should be developed on the basis of subsurface exploration, laboratory testing, and analysis performed as part of a future design-level Investigation, once the “preferred” alignment has been identified and once the specific locations of significant structures or other critical elements are known.

7.3 Liquefaction and Related Effects

Liquefaction refers to the sudden, temporary loss of soil strength during strong ground shaking where similar soils are saturated. The strength loss occurs as a result of the build-up of excess pore water pressures and subsequent reduction of effective stress. While liquefaction most commonly occurs in loose, granular deposits, recent studies indicate that it can also occur in materials with relatively high fines content provided the fines exhibit lower plasticity. Ancillary effects of soil liquefaction can include lateral spreading (wherein liquefied materials move towards a “free face” such as a slope or bank), differential ground surface settlement, ground cracking, and other impacts. Seismic densification may occur and result in differential settlement where loose granular soils are unsaturated.

As shown on Plate 4, regional mapping indicates that valley-bottom areas of each proposed alignment typically lie in zones of “moderate” to “high” liquefaction susceptibility, and that these zones typically reflect the extents of alluvial soils shown on our Preliminary Geologic Map (Plate 2). Zones of “very high” and “high” susceptibility are typically limited to active stream channels which cross each of the alignments but are typically relatively narrow. The most significant zone of “high” susceptibility with respect to the proposed alignments lies in the central and southern parts of urban Petaluma, along Alignment STS-2. Otherwise, most valley-bottom areas are shown in zones of “moderate” susceptibility, while upland areas underlain by bedrock comprise zones of “low” susceptibility.

We judge there is a generally moderate risk of liquefaction or seismic densification (depending on groundwater elevation and local soil composition) where proposed pipeline alignments are underlain by thicker alluvial soils and/or fill and Bay Mud. In these areas, likely impacts would include differential settlements, ground cracking, and other phenomena which could cause significant damage to surface and subsurface improvements. There may be a low to moderate risk of lateral spreading where alignments are underlain by liquefiable soils and are in close proximity to steep descending slopes or channel banks, such as at some of the major channel crossings and in the area of the Petaluma River.

Evaluation: Less than significant with special engineering measures.

Recommendations: In general, flexible pipe materials and connectors capable of withstanding significant differential settlements should be considered wherever practical. "Critical" structures should be provided with foundation systems capable of resisting the effects of liquefaction. "Hollow" below-grade structures such as buried tanks, vaults, or manholes may require mitigation to resist potential buoyancy as a result of elevated pore pressures. Repair of offsets between surface features such as manholes and valves should be anticipated as a result of liquefaction-induced settlements.

The potential for liquefaction should be evaluated on the basis of subsurface exploration, laboratory testing, and engineering analyses performed as part of a future design-level Investigation, once the "preferred" alignment has been identified and once the specific locations of significant structures or other critical elements are known.

7.4 Lurching and Ground Cracking

Lurching and associated ground cracking can occur during strong ground shaking. The ground cracking generally occurs along the tops of slopes where stiff soils are underlain by soft deposits, or along steep slopes or channel banks.

In general, these conditions exist in many areas throughout each proposed alignment. Susceptibility to lurching and cracking is generally greatest where the alignments run parallel and in close proximity to incised stream banks, and where the alignment is in close proximity to steep descending slopes or existing landslide scarps.

Evaluation: Less than significant with special engineering measures.

Recommendations: Ideally new pipelines would be routed away from steep banks and other areas prone to lurching. Where avoidance is impractical, alignments could be deepened in order to reduce the risk of lateral displacement due to the lurching of overlying materials. Flexible pipe materials capable of withstanding lateral forces and displacement associated with lurching should be utilized wherever practical.

The potential for lurching should be evaluated on the basis of more detailed site mapping, subsurface exploration, laboratory testing, and engineering analyses performed as part of a future design-level Investigation, once the "preferred" alignment has been identified.

7.5 Settlement

Significant settlement can occur when new loads are applied to soft, compressible soils such as the Bay Mud that underlies the Petaluma River and adjacent areas. The rate and magnitude of potential settlements are dependent on the new (net) loads applied, the thickness of compressible material and the inherent compressibility properties of the Bay Mud.

For deep Bay Mud sites, settlement due to consolidation of the Bay Mud would be expected to take many decades to complete. The rate and magnitude of future settlements associated with existing fills will be dependent on the thickness of fill, thickness of mud, and timing of fill placement. In many areas where fills are very thin or have been in place for a long time, settlement may be complete. In areas of thicker or more recently placed fill, some long-term future settlement may occur as a result of existing loads.

New above-grade fills and structures will result in the application of new loads and inducement of additional settlement. New trench backfills in zones of Bay Mud may also induce new settlement. Conversely, construction of new ("hollow") below-grade structures will effectively reduce loading and result in no new settlement. Where settlement due to existing loads is ongoing, some vertical offset between buried structures and surrounding grades is expected.

Based on review of previous subsurface exploration performed in the area, Bay Mud deposits ranging from about 3- to 23-feet thick may be expected in the areas of Alignments STS-1 and STS-2 in southern Petaluma. Overlying fills are typically on the order of 3-to 8-feet thick. Most of these fills were placed many decades ago; however, the extent to which more recent highway construction involved new loads or additional settlement potential is unclear.

We judge the risk of damage to settlement is moderate in areas underlain by Bay Mud and will be higher where existing fills are thicker and/or were placed in the past 30 years. In general, areas at risk of settlement are limited to portions of Alignments STS-1 and STS-2 in the southern part of Petaluma where extensive development overlies Bay Mud deposits along Petaluma Boulevard and significant fill embankments underlie Highway 101.

Evaluation: Less than significant with special engineering measures.

Recommendations: In general, we recommend that excavations for below-grade improvements be minimized by use of temporary shoring instead of sloping excavations. Any abandoned below-grade structures should be backfilled with lightweight fill (cell-crete or lava rock) or left as empty space. Flexible utility connections to new structures should be considered to accommodate minor differential movements that may occur. Periodic asphalt grinding or overlays may be needed to repair future small offsets following differential settlements around below-grade structures. Detailed settlement time-rate analyses should be performed as part of a future design-level investigation if significant new structures or pipelines are planned in Bay Mud areas. The flow direction and slope of new pipelines should take into account long term settlement in Bay Mud areas.

7.6 Slope Instability/Landslides

As shown on Plate 2, landslides were mapped along portions of all proposed alignments. In general, landsliding is most prevalent in steeply-sloping areas underlain by Franciscan bedrock and along steep cut slopes and incised channel banks. In general, the majority of the landslides observed occur on the uphill side of the road, where their impacts on buried pipelines would be minimal. Where landslides occur on the downslope side of the alignments, loss of lateral support and undermining of the pipeline could occur. The likelihood of landslide occurrence or renewed movement will be higher during seismic events and following periods of intense rainfall.

Alignment STS-1 would require crossing several significant landslides along the Cotati Grade, while no significant landslides were observed underlying Alignment STS-2. Although we judge there is a moderate to high risk of slope instability in the steeper parts of those alignments along the Cotati Grade and in the Petaluma River and Kastania Tank areas, these alignments are probably the least exposed to slope instability risks of all alignments proposed.

Alignments STAF-1/2 and PETA-1/2 would each traverse areas near Stafford Lake where previous roadway undermining has required significant stabilization work, as well as other areas east and west of the lake where older slides appear to have required periodic repairs. Because of the additional landslide areas near Nicasio Reservoir, Alignments STAF-1 and PETA-1 are slightly more exposed to instability than Alignments STAF-2 and PETA-2.

Alignment PETA-4 would traverse Red Hill enroute to Hicks Valley, where several steep scarps and fill embankments exist along uphill and downhill sides of the road, as well as on the descent into Nicasio Valley at the western end of the alignment. Along San Antonio Road, the road is aligned parallel and adjacent to near-vertical creekbanks which locally exhibit evidence of instability.

Alignments COTATI-1 and COTATI-2 traverse very steep terrain along Wilson Hill Road, where widespread landsliding appears to have required periodic road maintenance. Much of each alignment follows adjacent creeks in fairly close proximity, where localized instability on steep banks is common. Of the alignments considered, the COTATI-1 alignment is probably the most exposed to slope instability risks given its overall length, the concentration of mapped slides in the Wilson Hill and Nicasio Reservoir Areas, and the extent of areas underlain by slide-prone Franciscan mélange rock.

Evaluation: Less than significant with special engineering measures.

Recommendations: The potential for damage due to slope instability should be evaluated in more detail once the "preferred" alignment has been selected. In general, mitigation measures could include avoidance (generally via aligning the pipe in the upslope/"cut" side of the roadway or opposite side from stream channels) or, where practical, deepening the alignment to place the pipe beneath any slide-prone materials. Development of design-level mitigation options and recommendations for slope instability should be predicated on subsurface exploration and laboratory testing performed as part of a future design-level Investigation.

7.7 Expansive Soil

Expansive soils will shrink and swell with fluctuations in moisture content and are capable of exerting significant expansion pressures on building foundations, interior floor slabs and exterior

flatwork. Flatwork, pavements, concrete slabs-on-grade, and other lightly loaded surface improvements are particularly vulnerable to distress due to their low bearing pressures.

Our previous experience with similar projects throughout the project area indicates that highly expansive Adobe clay, which blankets much of the Petaluma Valley, likely underlies the majority of Alignments STS-1 and STS-2. Bay Mud is also typically highly expansive and may occur in-situ or intermixed with fill materials in the southern parts of those alignments, as well as at the Redwood Boulevard/Highway 101 interchange at the east end of Alignments PETA-1/2.

Alignments COTATI-1/2 may also be underlain by Adobe clay in lower-lying parts of southern Sonoma County. Although local zones may generally exist throughout the project area, Alignments STAF-1 and STAF-2 are not anticipated to encounter significant areas of expansive soil. Alignment PETA-4 may be underlain by moderately plastic soils along San Antonio Creek and on the east side of Red Hill.

Evaluation: Less than significant with special engineering measures.

Recommendations: No special engineering measures are anticipated. If and where expansive soils are encountered, trenches should extend to minimum depths of 4 feet below grade, and non-expansive materials should be used for backfilling. If and where new structures or significant grading are planned in areas of expansive soils, mitigation recommendations should be developed on the basis of subsurface exploration and laboratory testing performed as part of a future design-level Investigation.

7.8 Erosion

Sandy soils on most slopes or clayey soils on steep slopes are susceptible to erosion when exposed to concentrated surface water flow. The potential for erosion is increased when established vegetation is disturbed or removed during normal construction activity. In areas where pipelines are installed beneath existing asphalt roadways or within mostly flat terrain, we judge the risk of damage due to erosion is typically low. However, if and where pipelines extend beyond the edge of pavement and traverse moderate to steep terrain, the risk of erosion-related damage will likely be increased. An increased risk of undermining and damage due to erosion will also exist where pipelines are planned parallel to and with little setback from active stream channels, or where buried pipelines “daylight” at crossings on steep channel banks.

Evaluation: Less than significant with mitigation.

Recommendations: In general, pipelines should be provided with reasonable setback from active channel banks. For planning purposes, setbacks would ideally be equal to 10-feet or the height of the adjacent bank. Specific recommendations for erosion mitigation should be developed on the basis of more detailed site reconnaissance, subsurface exploration, and laboratory testing performed as part of a future design-level Investigation once the “preferred” alignment(s) are identified. An erosion control and Stormwater Pollution Prevention Plan (SWPPP) should be developed prior to construction per the current guidelines of the Sonoma and/or Marin County Stormwater Pollution Prevention Program (Marin County, 2015, Sonoma County, 2022).

7.9 Flooding

Many of the low-lying portions of the project area lie within FEMA 100- or 500-year flood zones, as shown on Plate 5. Although flooding likely does not pose a significant risk to underground pipelines, designers of new improvements should consider potential flood inundation and provide appropriate corrosion protection and/or other mitigation as needed.

Evaluation: Less than significant with special engineering measures.

Recommendations: The project Civil Engineer should consider the potential for inundation by future flood events and provide adequate corrosion protection for metallic components. Any electrical components should be placed above predicted flood elevations.

7.10 Seiche and Tsunami

Seiche and tsunami are short duration earthquake-generated water waves in large, enclosed bodies of water and the open ocean, respectively. The extent and severity of a seiche or tsunami would be dependent upon ground motions and fault offset from nearby active faults. The section of the Petaluma River located south of the confluence with Lynch Creek is designated as being within a tsunami inundation zone. Although seiche likely does not pose a significant risk to underground pipelines, designers of new improvements should consider potential seiche inundation and provide appropriate corrosion protection and/or other mitigation as needed.

Evaluation: Less than significant with special engineering measures.

Recommendations: The project Civil Engineer should consider the potential for inundation by future seiche events and provide adequate corrosion protection for metallic components.

7.11 Soil Corrosion

Corrosive soil can damage buried metallic structures, cause concrete spalling, and deteriorate rebar reinforcement. Bay Mud deposits and fill soils in areas proximal to the Petaluma River may be high in soluble salts and constitute moderately to highly corrosive environments. Elsewhere, alluvial/colluvial soils and Franciscan bedrock are most often non- to slightly corrosive, and a low risk of damage due to corrosion likely exists in those areas.

Evaluation: Less than significant with special engineering measures.

Recommendations: We recommend that the potential for corrosion be evaluated by a suitably qualified professional on the basis of geochemical testing performed as part of a future design-level Investigation. Where applicable, the project Civil and Structural Engineer should specify materials that are resistant to corrosive soil or provide cathodic corrosion protection. At a minimum, we recommend a maximum water to cement ratio of 0.45 using either Type II or Type V cement for concrete structures. As is normally practiced, we recommend a minimum of 3 inches of concrete coverage over steel reinforcing where concrete is exposed to the ground. New pipes should be non-metallic or provided with a corrosion protection system.

7.12 Naturally Occurring Asbestos

Asbestos is a generic term for a group of naturally occurring fibrous minerals which, when airborne, can be hazardous to the respiratory system. Crystals of asbestiform minerals may be liberated from the host rock and become airborne during excavation, crushing or grading operations. The most common asbestos mineral is chrysotile, though other minerals such as tremolite and actinolite take the same fibrous crystal form. While relatively rare in occurrence, these minerals are commonly associated with ultramafic and related metamorphic rocks, chiefly serpentinite but also including low- to high-grade schists such as chlorite schist and blueschist. These rocks are a major constituent of the Franciscan Complex, which underlies vast portions of Marin and Sonoma Counties.

Asbestiform minerals are regulated at the state level by the California Environmental Protection Agency, Air Resources Board (CARB). In the nine-county San Francisco Bay Area, CARB regulations are enforced by the Bay Area Air Quality Management District pursuant to the California Code of Regulations, Title 17, Section 93105, "Asbestos Airborne Toxic Control Measure for Construction, Grading, Quarrying, and Surface Mining Operations" (BAAQMD, 2012). These regulations require an application package be submitted for such operations specifying mitigation measures to be implemented for control of airborne asbestos dust during construction. However, exemptions may be granted for projects on an individual residential property and/or for operations in remote locations unlikely to affect neighboring properties or the general public.

During our reconnaissance, we noted abundant blueschist exposed north of the Kastania Tanks at the south end of the STS-1/2 alignments, which frequently occurs in association with serpentinite and asbestos. We also observed abundant serpentinite and related rocks in the San Marin area of Novato along the PETA-1/2 alignments, as well as at the eastern lower foot of Wilson Hill along the COTATI-1/2 alignments. We did not observe outcrops of serpentinite at the eastern foot of Red Hill but note that exposures are mapped immediately north and south of the road. Although none was observed during our reconnaissance, smaller zones of serpentinite and other potentially asbestos-bearing rocks are likely to exist in areas underlain by Franciscan mélange.

Evaluation: Less than significant with mitigation.

Recommendations: The concentration and distribution of naturally occurring asbestos should be evaluated in more detail on the basis of subsurface exploration and laboratory examination performed during a future design-level Investigation. The Contractor should be responsible for implementing dust-control measures and other best management practices in accordance with the requirements of the Bay Area Air Quality Management District, as applicable. Likely mitigation measures include frequent watering and other dust-control measures in areas to be graded. Additionally, enhanced personal protective equipment (PPE) should be available for construction personnel during excavation into serpentinite to reduce the risk of exposure to hazardous asbestiform minerals.

7.13 Hard Rock Excavation

Many of the rock types exposed throughout the project area may be of sufficient strength and hardness as to require specialized techniques and equipment to excavate, which can result in project delays and increased excavation cost.

Based on our experience, we anticipate that the majority of the younger rocks in the project area (Tertiary and Quaternary deposits, as shown on Figure 2), as well as the sedimentary rocks of the Great Valley Sequence and Santa Margarita Sandstone, can be excavated with conventional equipment, such as large backhoes and medium-size excavators. Most Franciscan sandstone and shale, including areas of sandstone and shale within areas mapped as melange can likely be excavated with similar equipment.

There is a higher likelihood of encountering particularly hard rock in areas underlain by Sonoma Volcanics and by Franciscan chert, greenstone, metamorphic rocks, and serpentinite. Discontinuous zones of hard rock may also exist in areas mapped as melange. Where hard rock is encountered, specialized equipment such as jackhammers, hydraulic breakers, or specialized hard rock trenching machines. We judge that the likelihood that hard rock excavation conditions are encountered along any given alignment is high.

Evaluation: Less than significant with special engineering measures.

Recommendations: The distribution of hard rocks and potential for difficult excavation should be assessed in more detail on the basis of subsurface exploration performed as part of a design-level Geotechnical Investigation. We recommend that the contract documents for the project, regardless of the chosen alignment(s), include a line item and clear definition (i.e., production rate of <50 feet per day using a Cat 328 and bucket equipped with carbide “rock” teeth) for hard rock excavation.

7.14 Hazardous Materials

Although evaluation of environmental contamination or hazardous materials is generally beyond the scope of our evaluation, it should be noted that artificial fill, especially older/undocumented fills in the southern part of Petaluma, may include debris/garbage materials of unknown origin. Historic industrial development along Highway 101 and Petaluma Boulevard is also a likely source of potential contamination. In rural areas, the most likely source of potential contamination is agricultural runoff.

Evaluation: Less than significant with special engineering measures.

Recommendations: At minimum, we recommend that “screening-level” environmental testing be performed during a future design-level Investigation to assess the potential for hazardous materials within the work areas to necessitate special handling/disposal requirements. Environmental assessment should be performed by a suitably qualified professional.

8.0 CONCLUSIONS

Based on our geologic reconnaissance and experience with similar projects in the area, we judge that construction of the proposed transmission pipeline(s) along any of the nine proposed alignments is feasible from a geotechnical standpoint.

Significant geotechnical design considerations will include providing adequate seismic design and uniform foundation support for significant structures, providing adequate mitigation if and where new pipelines are constructed in Bay Mud areas, limiting pipeline exposure to slope instability and seismic lurching risks in steeper terrain, and providing adequate flexibility in pipe materials and connections to resist damage due to seismic ground shaking, liquefaction, and/or settlement.

Other related considerations which may impact selection of the “preferred” alignment(s) are anticipated to include the potential for project cost and schedule impacts where hard rock may require special excavation techniques and equipment, where there are significant conflicts with existing utilities, and where naturally occurring asbestos or other hazardous materials/environmental contaminants may require special handling and disposal procedures.

General conclusions and more detailed discussion regarding primary geotechnical considerations along individual alignments is provided below.

8.1 Alignment STS-1

The most significant geotechnical challenges associated with Alignment STS-1 will include providing adequate evaluation of slope instability risks along the Cotati Grade and crossing the Petaluma River near the south end of the alignment. It is anticipated that research regarding historic landslide activity and previous Caltrans stabilization work will be required to assess potential conflicts between existing improvements and the planned pipeline. If monitoring programs are not currently in place, installation and maintenance of slope monitoring equipment may be needed for a period of time to confirm the rate, depth, and magnitude of ongoing movement and facilitate adequate mitigation design.

At the Petaluma River, spanning the 250-foot-wide channel via directional drilling or other trenchless means would likely require subsurface exploration be performed using over-water equipment, which will add significant cost. Suspending the pipe from the existing highway overcrossing would likely be much simpler than drilling beneath the river. If the pipe will be supported by new structures, their design will be complicated by the potential for settlement due to both liquefaction and new loads in areas of Bay Mud. The potential for issues with environmental contamination may be higher given the former agricultural and current industrial development along the highway. Deeper excavations in Bay Mud areas will likely be complicated by shallow groundwater and caving/unstable soil conditions.

It is anticipated that most geologic materials along the alignment could be excavated by “typical” equipment such as medium-size excavators and backhoes. It is possible that isolated zones of harder rock may exist in the area of the Tolay Fault Zone/Cotati Grade landslides, as well as south of the Quarry Heights subdivision in areas of Franciscan metamorphic rocks. Variable excavation conditions are expected in areas of fill along the highway and near the river.

8.2 Alignment STS-2

The most significant geotechnical challenges associated with Alignment STS-2 will include providing adequate shoring and dewatering in the lower-lying areas of southern Petaluma, where undocumented fills and younger alluvial soils may present variable excavation conditions and local caving/unstable areas. Shallow groundwater in this area may also complicate construction.

It is anticipated that most geologic materials along the alignment could be excavated by “typical” equipment such as medium-size excavators and backhoes. Variable excavation conditions may exist in older undocumented fills along Stony Point Road and the north end of Petaluma Boulevard. There is also some potential for hard rock excavation conditions at the south end of the alignment where volcanic and metamorphic rocks are exposed, and where volcanic rocks locally crop out along the northern part of Petaluma Boulevard, generally between downtown and Skillman Lane.

Perhaps most significantly, construction of this alignment will be complicated by conflicts with extensive existing subsurface utilities in the urbanized portions of Petaluma. Older utility trenches may be backfilled with cohesionless materials that can present unstable excavation conditions and impact project cost and schedules. Deeper pipeline excavations in close proximity to existing structures, particularly older buildings, could undermine shallow foundations. More careful consideration of pipeline location/depth will be needed in these areas, and there could be a need for special shoring design, selective structure underpinning, and/or structure monitoring during construction. The potential for issues with environmental contamination may be higher given the historic industrial land use in the urbanized portions of Petaluma and along the river.

8.3 Alignment STAF-1

Significant geotechnical design considerations along Alignment STAF-1 are generally limited to evaluation of local landslide risks west of Stafford Lake and east of Nicasio Reservoir, where landslides on the downhill side of the road may pose some risk of undermining. Otherwise, the primary consideration is expected to be the potential for hard rock excavation. We observed very hard chert bedrock along Point Reyes-Petaluma Road near Nicasio Reservoir and noted an abundance of discontinuous zones and isolated blocks of hard rock within the mélange along Novato Boulevard.

8.4 Alignment STAF-2

As with STAF-1, significant geotechnical design considerations along Alignment STAF-2 are generally limited to evaluation of local landslide risks west of Stafford Lake, where slides on the downhill side of the road may pose some risk of undermining. Otherwise, the primary consideration is expected to be the potential for hard rock excavation. We observed very hard chert bedrock along Point Reyes-Petaluma Road near Nicasio Reservoir and noted an abundance of discontinuous zones and isolated blocks of hard rock within the mélange along Novato Boulevard. From a geotechnical perspective, this alignment is probably the simplest to design and construct of all the proposed alignments.

8.5 Alignment COTATI-1

Alignment COTATI-1 is the longest alignment and traverses the most diverse array of ground conditions amongst all alignments considered. Significant geotechnical design considerations will include evaluation of local landslide and lurching risks in steeper terrain, particularly in the area of Wilson Hill and east of Nicasio Reservoir. Liquefaction and associated settlement

potential will need to be evaluated in areas of widespread alluvium along major drainages, mostly in the Sonoma County portion of the alignment, and risks of erosion, instability, and lurching will exist where the alignment parallels incised drainages along portions of Meachem, Pepper, Hicks Valley, and Point Reyes-Petaluma Roads. Construction in these areas could be complicated by shallow groundwater and/or caving soils. Spanning Stemple Creek's 70-foot-wide channel could require moderately deep directional drilling or a carefully designed support structure if the pipe cannot be suspended below the existing bridge on Pepper Road.

It is anticipated that most of the soil materials and Tertiary-age sedimentary rocks along the alignment could be excavated by "typical" equipment such as medium-size excavators and backhoes. Hard rock excavation conditions are likely to be encountered in areas underlain by Franciscan metamorphic rocks and serpentinite and may be encountered sporadically in areas underlain by Franciscan mélange. Highly variable excavation and groundwater conditions may also exist where mapped faults cross the northern part of the alignment along Meachem and Pepper Roads. Naturally occurring asbestos may be encountered in areas of serpentinite at the eastern foot of Wilson Hill and may also locally occur in areas underlain by mélange.

8.6 Alignment COTATI-2

Similar to COTATI-1, significant geotechnical design considerations will include evaluation of local landslide and lurching risks in steeper terrain, particularly in the area of Wilson Hill and east of Nicasio Reservoir. Liquefaction and associated settlement potential will need to be evaluated in areas of widespread alluvium along major drainages, mostly in the Sonoma County portion of the alignment, and risks of erosion, instability, and lurching will exist where the alignment parallels incised drainages along portions of Meachem, Pepper, Hicks Valley, and Three Peaks Fire Roads. Construction in these areas could be complicated by shallow groundwater and/or caving soils. Spanning Stemple Creek's 70-foot-wide channel could require moderately deep directional drilling or a carefully designed support structure if the pipe cannot be suspended below the existing bridge on Pepper Road.

It is anticipated that most of the soil materials and Tertiary-age sedimentary rocks along the alignment could be excavated by "typical" equipment such as medium-size excavators and backhoes. Hard rock excavation conditions are likely to be encountered in areas underlain by Franciscan metamorphic rocks and serpentinite and may be encountered sporadically in areas underlain by Franciscan mélange. Highly variable excavation and groundwater conditions may also exist where mapped faults cross the northern part of the alignment along Meachem and Pepper Roads. Naturally occurring asbestos may be encountered in areas of serpentinite at the eastern foot of Wilson Hill and may also locally occur in areas underlain by mélange.

8.7 Alignment PETA-1

Significant geotechnical design considerations along Alignment PETA-1 will include evaluation of local landslide risks along Novato Boulevard, both east and west of Stafford Lake, and east of Nicasio Reservoir where slides on the downhill side of the road may pose some risk of undermining. Consideration should also be given to the potential for naturally-occurring asbestos in areas along San Marin Drive underlain by serpentinite – asbestos-bearing rocks may also locally exist in areas mapped as mélange or Franciscan metamorphic rocks.

Other primary considerations may include potential conflicts between existing stabilization works and the proposed pipeline may exist in landslide areas immediately west of the Stafford Lake dam and the potential for extensive hard rock excavation. Much of the bedrock underlying the San Marin area of Novato and the area around Stafford Lake dam consists of hard meta-volcanic and meta-sedimentary rocks, including very hard chert, greenstone, and serpentinite, and it is generally expected that specialized excavation equipment and techniques will be required in some of these areas.

8.8 Alignment PETA-2

As with PETA-1, significant geotechnical design considerations along Alignment PETA-2 will include evaluation of local landslide risks along Novato Boulevard, both east and west of Stafford Lake, where slides on the downhill side of the road may pose some risk of undermining. Consideration should also be given to the potential for naturally-occurring asbestos in areas along San Marin Drive underlain by serpentinite – asbestos-bearing rocks may also locally exist in areas mapped as mélange or Franciscan metamorphic rocks.

Other primary considerations may include potential conflicts between existing stabilization works (i.e., tiebacks) and the proposed pipeline that may exist in landslide areas immediately west of the Stafford Lake dam and the potential for extensive hard rock excavation. Much of the bedrock underlying the San Marin area of Novato and the area around Stafford Lake dam consists of hard meta-volcanic and meta-sedimentary rocks, including very hard chert, greenstone, and serpentinite, and it is generally expected that specialized excavation equipment and techniques will be required in some of these areas.

8.9 Alignment PETA-4

Significant geotechnical design considerations will include evaluation of local landslide and lurching risks in steeper terrain, particularly in the areas of Red Hill and east of Nicasio Reservoir, and along San Antonio Creek. Liquefaction and associated settlement potential will need to be evaluated in areas of widespread alluvium along major drainages, and risks of erosion, instability, and lurching will exist where the alignment parallels incised drainages along portions of San Antonio, Red Hill, Hicks Valley, and Three Peaks Fire Roads. Construction in these areas could be complicated by shallow groundwater and/or caving soils. Spanning San Antonio Creek's 100-foot-wide channel could require moderately deep directional drilling or a carefully designed support structure if the pipe cannot be suspended below the existing bridge on Red Hill Road.

It is anticipated that most of the soil materials and Great Valley Sequence rocks along the alignment could be excavated by "typical" equipment such as medium-size excavators and backhoes. Hard rock excavation conditions are likely to be encountered in areas underlain by Franciscan metamorphic rocks, primarily at Red Hill, and may be encountered sporadically in areas underlain by Franciscan mélange. Highly variable excavation and groundwater conditions may also exist where mapped faults are in close proximity to the eastern end of the alignment along San Antonio and Red Hill Roads. Naturally occurring asbestos may be encountered in areas of serpentinite at the eastern foot of Red Hill and may also locally occur in areas underlain by mélange.

9.0 PRELIMINARY GEOTECHNICAL RECOMMENDATIONS

The primary geotechnical considerations for project design and construction include providing appropriate seismic and structural design for any new structures, providing appropriate temporary support for deep excavations in weak, caving material, and prevention of buoyant conditions and minimization of excavation extents and associated backfills for new below-grade structures such as pump stations, wet wells, and manholes where Bay Mud soils exist. Preliminary geotechnical design criteria and recommendations for these and other project facets are presented in the following sections.

9.1 Preliminary Seismic Design

Minimum mitigation of ground shaking includes seismic design of new structures in conformance with the provisions of the most recent edition (2022) of the California Building Code. The magnitude and character of these ground motions will depend on the particular earthquake and the site response characteristics. Based on the interpreted subsurface conditions and close proximity of several nearby faults, we recommend the preliminary CBC coefficients and site values shown in Table 2 be used to calculate the design base shear of the new construction.

Table 2 – 2022 California Building Code Seismic Design Criteria

Parameter	Design Value ¹
Site Class	D (Default) ²
Site Latitude	38.21°N
Site Longitude	-122.68°W
Spectral Response (short), S_s	1.5 g
Spectral Response (1-sec), S_1	0.6 g
Site Coefficient, F_a	1.2
Spectral Response (Short), S_{MS}	1.8 g
Design Spectral Response (short), S_{DS}	1.2 g
MCE_G PGA Adjusted, PGA_M	0.6 g

- (1) SEAOC/OSHPD Seismic Design Maps Web Tool, <https://seismicmaps.org/>, accessed April 1, 2024
- (2) Site Class will vary across the project area and final design seismic design criteria should be confirmed for specific structures/work locations on the basis of subsurface exploration, laboratory testing, and additional analysis as part of a future design-level Investigation, if new structures are planned.

9.2 Site Grading

We anticipate that site grading will typically be limited to excavations on the order of 5- to 10-feet for construction of the new underground pipelines. It is expected that locally deeper excavations may be required for alignments that extend beneath the Petaluma River or other deeper channels and existing buried structures. More significant grading could happen if new structures are required to support the pipeline across and above some of these channels.

9.2.1 *Site Preparation*

Site preparation will generally include saw cutting and removing existing asphalt and/or concrete in work areas. Existing asphalt and base rock could be processed to meet the requirements for select fill (outlined below) for possible reuse as backfill for new utilities. Excavated areas (i.e., excavations for removal of old pipes) should be restored with properly moisture-conditioned and compacted fill as described in the following sections. Utilities that are abandoned in-place should be completely filled with light weight cell-crete in Bay Mud areas or controlled density fill (CDF) in other areas.

9.2.2 *Excavations*

Excavations for new improvements will encounter a wide variety of geologic materials ranging from soft Bay Mud to potentially hard bedrock, as previously described and shown on Plate 2.

In general, we expect that most excavations in soil materials and most Tertiary age sedimentary deposits throughout the project area can be accomplished with “traditional” excavation equipment, such as medium to large excavators and backhoes. Areas underlain by Franciscan sandstone and shale, and most areas underlain by *mélange*, can probably also be excavated with similar equipment. A moderate to high likelihood of “hard rock” excavation conditions is judged to exist in areas underlain by volcanic rocks, areas underlain by Franciscan chert, greenstone, and metamorphic rocks, and areas where blocks or masses of harder rocks are entrained within Franciscan *mélange*. Specialized techniques and equipment, such as jackhammers and hoe-rams, may be required to excavate these rocks. Hard rock excavation and shallow groundwater could also be encountered within mapped fault zones.

In general, most excavation spoils may be suitable for re-use as fill provided, they can be processed to meet the criteria provided in Section 9.2.5. Spoils derived from Bay Mud or of deleterious or otherwise unsuitable old fill materials (i.e., expansive clays, concrete rubble, organic material, etc.) will not be suitable for re-use as fill and should be removed from the site.

More detailed discussion regarding expected excavation conditions and specific geotechnical considerations associated with each different geologic material mapped in the project area, as shown on Plate 2, is provided in the following sections. It should be noted that the geologic contacts shown on Plate 2 are based on a combination of large-scale regional mapping, targeted site reconnaissance, limited surface soil and rock exposures, and widely scattered subsurface exploration points. Therefore, we recommend inclusion of a “contingency” or margin of error in any calculations of specific material quantities based solely on this map. If determination of specific material volumes is required, we should be consulted regarding additional subsurface exploration. In addition, it is possible that hard bedrock may be encountered beneath mapped soil deposits where deeper excavations are performed. A more specific evaluation of hard rock excavation potential, asbestos occurrence, and other significant geotechnical considerations should be performed via subsurface exploration performed as part of a future design-level investigation.

Artificial Fill (Map Unit Qaf)

Artificial fill will be encountered at various locations throughout each alignment. The most significant fills exist along Alignments STS-1/2, where thick fills underlie highway overpass approaches and reclaimed marshlands. Roadway fills are common in sloping terrain throughout the other alignments.

Artificial fill may consist of any combination of soil, rock, rubble, and garbage, and may range from poorly consolidated to dense and well-compacted. Caving conditions and/or locally difficult excavation may be encountered. All fill materials should be considered Type “C” soils.

Fill over Bay Mud (Map Unit Qaf/Qm)

Fill over Bay Mud would most likely be encountered only along Alignments STS-1/2, where thick fills underlie extensive development in reclaimed marshland areas along the Petaluma River.

Excavations in this unit will typically extend through 3- to 8-feet of loose/soft to stiff/dense clayey, gravelly, and rocky/rubbly fill material and into soft Bay Mud. Groundwater is likely to be encountered within 5-feet of the ground surface. Deeper excavations could penetrate the base of the Bay Mud layer and encounter loose sandy to medium-stiff clayey alluvial soils. In accordance with Cal/OSHA soil type designations, Bay Mud, underlying alluvium, and loose near-surface fill soils should be considered “Type C” materials for shoring purposes.

It is anticipated that excavations in these areas can be reasonably accomplished with “traditional” excavation equipment. Note that Bay Mud soil is not suitable for re-use as fill and should be removed from the site.

Landslides (Map Units Qls and Qols)

Landslide debris may be locally encountered along each of the STAF, COTATI, and PETA alignments where roads have been developed over older slide debris, or where active or dormant slides have impacted the downslope side of the road.

Landslide debris will include a chaotic mixture of soil and rock fragments; younger and more recent slides (Qls), will likely be poorly- to moderately consolidated while older slide deposits (Qols) are likely better-consolidated. Larger slides of both types could include entrained boulders or masses of hard rock where source areas are underlain by volcanic or Franciscan rock.

It is anticipated that shallow excavations into landslide areas can typically be accomplished using “traditional” excavation equipment. Groundwater conditions will likely be variable, and significant groundwater may exist along failure planes and near the toe of the slide. Excavation spoils could likely be re-used as trench backfill but may require substantial processing to meet the gradation criteria in Section 9 where large rock fragments or cobbles are encountered.

Alluvium (Map Units Qa and Qf)

Alluvial deposits will be encountered along each of the proposed alignments. The STS and COTATI alignments are generally underlain by widespread alluvial deposits, while the STAF and PETA alignments are more sparsely underlain by alluvium.

Alluvium generally consists of interbedded clay, silt, sand, and gravel in various proportions. Most deposits will likely be poorly- to moderately consolidated. Excavations in these areas will most often encounter medium stiff to stiff clays and loose to medium dense clayey sands and gravelly sands. In general, these materials will typically classify as Cal/OSHA “Type C” soils and will be prone to caving and raveling in open excavations, particularly where saturated and where well-sorted deposits are encountered (i.e., “clean” sands and gravels).

It is expected that most of these excavations may be accomplished with “traditional” equipment as described above. Groundwater is likely to be encountered within about 5- to 10-feet of the ground surface throughout the year and may be shallower during the winter months or following periods of heavy rain. Excavation spoils in these areas will likely be suitable for re-use as intermediate trench backfill.

Tertiary Sedimentary Rocks (Map units Tp, Tco, and Twg)

These rocks exist within portions of the STS alignments and within the northern (Sonoma County) parts of the COTATI alignments, but do not occur in the areas of the STAF and PETA alignments.

Excavations in these areas will generally extend through a layer of surficial residual or colluvial soils and into weakly- to moderately consolidated siltstone, sandstone, and pebble to cobble conglomerate. Residual deposits and weakly-consolidated bedrock will most likely classify as Cal/OSHA “Type B” soils.

It is expected that most of these excavations may be accomplished with “traditional” equipment as described above. Areas of shallow or “perched” groundwater may be encountered at contacts between map units, or within each of these units along individual bedding planes. Caving conditions could locally exist in areas of shallow groundwater or seepage, and in areas where rock is more deeply weathered and/or pulverized along fault zones. Excavations in these areas will typically yield sandy to gravelly mixtures that will probably be suitable for re-use as trench backfill.

Sonoma Volcanics (Map units Tsv and Tsa)

These rocks underlie relatively small portions of the STS alignments at the Cotati Tanks, Cotati Grade, and Petaluma Boulevard. The COTATI alignments would also encounter a small area of volcanic rocks at the Cotati Tanks, but these rocks were not observed along either the STAF or PETA alignments.

Excavations in these areas will likely encounter relatively hard andesite and basalt bedrock beneath a thin layer of clayey to sandy residual soil. Local zones of softer or weaker rock such as tuff (ash) may also be encountered. Residual soils, soft

or poorly consolidated tuff, and deeply weathered rock in these areas will probably classify as “Type B” materials; harder rock will most likely be “Type A”.

It is anticipated that shallow excavations into residual soil and weathered rock can be accomplished using “traditional” excavation equipment, while deeper excavations into harder and fresher rock may require hard rock excavation equipment and techniques. Although small amounts of groundwater may exist locally, significant amounts of shallow groundwater are generally not anticipated in these areas. Excavation spoils could likely be re-used as fill but may require substantial processing to meet the gradation criteria in Section 9.

Great Valley Sequence (Map unit Kqv)

These rocks crop out only along San Antonio Road and the PETA-4 alignment. Excavations in these areas will likely encounter a few feet of silty to sandy colluvium and residual soils over moderately hard, weak to moderately strong siltstone, sandstone, and pebble conglomerate. Colluvial and residual soils and bedded or fractured rock in these areas will probably classify as “Type B” materials, while harder rock will most likely be “Type A”.

It is anticipated that most excavations into these materials can be accomplished using “traditional” excavation equipment. Although generally not expected deeper excavations into metamorphosed rock along fault zones or into thick bedded to massive sandstone may require hard rock excavation equipment and techniques such as jackhammers, hydraulic breakers, or specialized hard rock trenching machines. Although small amounts of groundwater may exist locally, significant amounts of shallow groundwater are generally not anticipated in these areas. Excavation spoils could likely be re-used as fill but may require substantial processing to meet the gradation criteria in Section 9 if and where zones of thick-bedded or massive rock are encountered.

Franciscan Sandstone and Shale (Map unit KJfs)

These rocks crop out only around Nicasio Reservoir and along the ridge separating Hicks Valley from Nicasio Valley at the west end of the STAF-1, COTATI-1, and PETA-1 alignments. Excavations in these areas will likely encounter a few feet of silty to gravelly colluvium and residual soils over moderately hard to hard sandstone and shale. Colluvial and residual soils and deeply weathered rock in these areas will probably classify as “Type B” materials, while harder rock will most likely be “Type A”.

It is anticipated that shallow excavations into residual soil and weathered rock can be accomplished using “traditional” excavation equipment, while deeper excavations into harder and fresher rock or into thick bedded to massive sandstone may require hard rock excavation equipment and techniques such as jackhammers, hydraulic breakers, or specialized hard rock trenching machines. Although small amounts of groundwater may exist locally, significant amounts of shallow groundwater are generally not anticipated in these areas. Excavation spoils could likely be re-used as fill but may require substantial processing to meet the gradation criteria in Section 9 if and where zones of massive rock are encountered.

Franciscan Chert (KJfch)

Chert is mapped in small areas along the PETA, STAF, and COTATI alignments, mostly occurring as small zones and prominent outcrops along the roadside. Chert likely also underlies parts of these alignments in the form of discrete masses and inclusions within the mélange. Excavations in these areas will likely encounter a few feet of silty to gravelly colluvium and residual soils over moderately hard to very hard rock. Colluvial and residual soils and deeply weathered rock in these areas will probably classify as “Type B” materials, while harder rock will most likely be “Type A”.

It is anticipated that shallow excavations into residual soil and weathered rock can be accomplished using “traditional” excavation equipment, while deeper excavations into harder and fresher rock may require hard rock excavation equipment and techniques such as jackhammers, hydraulic breakers, or specialized hard rock trenching machines. Although small amounts of groundwater may exist locally, significant amounts of shallow groundwater are generally not anticipated in these areas. Excavation spoils could likely be re-used as fill but may require substantial processing to meet the gradation criteria in Section 9.

Franciscan Greenstone (KJfg)

Greenstone is mapped along a small portion of the STAF and PETA alignments just east of Stafford Lake. Greenstone likely underlies parts of these alignments and the COTATI alignments where it occurs as a constituent of mélange. Excavations in these areas will likely encounter a few feet to ten feet or more of clayey to gravelly colluvium and residual soils over moderately hard to very hard rock. Colluvial and residual soils and deeply weathered rock in these areas will probably classify as “Type B” materials, while harder rock will most likely be “Type A”.

It is anticipated that shallow excavations into residual soil and weathered rock can be accomplished using “traditional” excavation equipment, while deeper excavations into harder and fresher rock may require hard rock excavation equipment and techniques such as jackhammers, hydraulic breakers, or specialized hard rock trenching machines. Although small amounts of groundwater may exist locally, significant amounts of shallow groundwater are generally not anticipated in these areas. Excavation spoils could likely be re-used as fill but may require substantial processing to meet the gradation criteria in Section 9.

Franciscan Metamorphic Rocks (Map Unit KJsSch)

Franciscan metamorphic rocks underlie portions of the STAF, COTATI, and PETA alignments and also occur at the south end of the STS alignments at the Kastania Tanks. These rocks primarily consist of medium- to high-grade schists, including blueschist and eclogite derived of meta-shale and meta-basalt, respectively. These rocks are commonly associated with and may contain zones of serpentinite and variably metamorphosed chert and greenstone. Excavations in these areas will likely encounter a few inches to a few feet of clayey to gravelly colluvium and residual soils over moderately hard to very hard rock. Colluvial and residual soils

and deeply weathered rock in these areas will probably classify as “Type B” materials, while harder rock will most likely be “Type A”.

It is anticipated that shallow excavations into residual soil and weathered rock can be accomplished using “traditional” excavation equipment, while deeper excavations into harder and fresher rock may require hard rock excavation equipment and techniques such as jackhammers, hydraulic breakers, or specialized hard rock trenching machines. Excavation conditions within this unit may be highly variable over short distances. Although small amounts of groundwater may exist locally, significant amounts of shallow groundwater are generally not anticipated in these areas. Excavation spoils could likely be re-used as fill but may require substantial processing to meet the gradation criteria in Section 9.

These rocks also may contain asbestiform minerals, and construction operations in these areas may be subject to special air monitoring, material handling, and disposal requirements in the event significant deposits of asbestos are encountered.

Franciscan Melange (Map Unit KJfm)

Franciscan mélange underlies vast portions of the STAF, COTATI, and PETA alignments. These areas consist primarily of variably weathered sandstone and shale with discontinuous blocks and masses of each of the other Franciscan rock types (chert, greenstone, metamorphic rocks, and serpentinite). Excavations in these areas will likely encounter a few inches to ten feet or more of clayey to gravelly colluvium and residual soils. Overlying soils crushed or deeply weathered rock, bedded sandstone and shale in these areas, will probably classify as “Type C” materials, while more competent weathered rock will typically be “Type B”. Areas of harder rock could be “Type A”.

It is anticipated that shallow excavations into residual soil and weathered rock can be accomplished using “traditional” excavation equipment, while deeper excavations into harder and fresher rock may require hard rock excavation equipment and techniques such as jackhammers, hydraulic breakers, or specialized hard rock trenching machines. Excavation conditions within this unit may be highly variable over short distances. Although small amounts of groundwater may exist locally, significant amounts of shallow groundwater are generally not anticipated in these areas. Excavation spoils could likely be re-used as fill but may require substantial processing to meet the gradation criteria in Section 9.

These rocks also may locally contain asbestiform minerals, and construction operations in these areas may be subject to special air monitoring, material handling, and disposal requirements in the event significant deposits of asbestos are encountered.

9.2.3 Shoring Considerations

Per regulations promulgated by the California Occupational Safety and Health Administration (Cal-OSHA), trench excavations having a depth of five feet or more which will be entered by workers must be sloped, braced, or shored to protect workers from potential collapse. Cal-OSHA dictates allowable slope configurations and minimum shoring requirements based on categorized soil types as discussed above. The Contractor should be responsible for site safety and should select an appropriate shoring system for the anticipated site conditions. The chosen system should be capable of providing immediate support to the sides of the excavation in order to minimize the amount of time the excavation is unsupported.

Vertical unsupported cuts within the Bay Mud should generally be expected to “squeeze” or fail during excavation for manholes and similar structures. Additionally, the extent of the excavation (and backfill around the structure) should be minimized to lessen the risk of future settlement. If possible, we suggest excavations for manholes and other improvements be no more than 2 or 3 feet wider than the structural “footprint.” Temporary shoring support should be a required component of the construction process to facilitate near-vertical cuts and eliminate the need for sloped excavation sides. For design of shoring loads, refer to the Cal/OSHA soil-type classifications discussed in Section 9.2.2.

If and where excavations are performed in close proximity to existing structures, the shoring designer should consider surcharge loads imposed by the structure, and care should be taken to avoid undermining existing foundation elements. If excavations extend into the Zone of Influence (ZOI) of any adjacent foundation and cannot be adequately shored or braced, that foundation should be underpinned to avoid the potential for loss of lateral support and possible damage. On a preliminary/planning-level basis, the ZOI should be defined as the region below a 1.5:1 (H:V) line projected down from the base of the foundation. Where it is anticipated that underpinning may be required, we should be consulted to address the specific situation as part of a future phase of work.

9.2.4 Dewatering

Groundwater will likely be encountered in low-lying portions of each alignment where alluvial deposits or Bay Mud are mapped, and dewatering will likely be required in some areas to maintain dry working conditions where excavations extend deeper than about 5-feet. We generally recommend de-watering be accomplished by the use of submersible pumps. Dewatering wells could also be considered as an option, although they may be of limited effectiveness in areas proximal to the Petaluma River.

9.2.5 Fill Materials, Placement and Compaction

Bedding materials for utility pipes should consist of well-graded sand with 90 to 100 percent of particles passing the No. 4 sieve and no more than 5 percent finer than the No. 200 sieve. Provide the minimum bedding beneath the pipe per the manufacturer’s recommendation, typically 3 to 6 inches.

Fill materials used to backfill excavations within Bay Mud (i.e., – at elevations from which Bay Mud was excavated, such as where new alignments will be deeper than existing) should consist of lightweight lava rock having a compacted unit weight no greater than 60 pounds per cubic foot (pcf).

Fill materials used to backfill the intermediate section of trench (between top of pipe and base of pavement section) should consist of non-expansive materials that are free of organic matter, have a Liquid Limit of less than 40, a Plasticity Index of less than 20, and have a minimum R-value of 20. The intermediate fill material should contain no more than 50 percent fine grained soils (percentage of particles passing a number 200 sieve) and contain no particles larger than 4-inches in diameter. Some of the onsite fill soils may be suitable for re-use as trench backfill; Bay Mud soils are not suitable for use as trench backfill and should be removed from the site. Aggregate base rock or controlled-density fill (CDF) may also be used as trench backfill.

All trench backfill in non-Bay-Mud zones should be uniformly conditioned to near the optimum moisture content. Properly moisture-conditioned and cured materials should be placed in loose horizontal lifts no more than 8-inches thick and uniformly compacted to at least 90% relative compaction to produce a firm, non-yielding surface. The uppermost 12-inches of trench backfill in structural areas (such as beneath pavements or exterior flatwork) should be compacted to at least 95% relative compaction and must be firm and un-yielding when proof-rolled with heavy construction equipment. Relative compaction, maximum dry density, and optimum moisture content of fill materials should be determined in accordance with ASTM Test Method D 1557, "Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using a 10-lb. Hammer and 18-in. Drop".

Due to the expected depth and limited access for backfill compaction for new below grade structures such as manholes, any backfill between the manhole and surrounding soil should consist of $\frac{3}{4}$ -inch crushed rock (drain rock) that is compacted in place with vibratory equipment (i.e., concrete stinger or "jumping jack" type equipment). Shoring should be incrementally raised as fill material is placed to avoid pulling shoring after fill compaction, which would loosen the fill, resulting in voids and a higher risk of surface damage due to future backfill settlement. Placement of fill material to abandon any existing below-grade structures in Bay Mud areas should be avoided if possible. If required, lightweight fill material with a compacted unit weight less than 60 pcf should be specified.

9.3 Pipelines and Structures in Bay Mud Areas

The weight of new below-grade structures such as manholes will likely be less than the weight of the excavated soil and relatively small volume of crushed rock backfill, so settlement of the structures would not be considered a significant issue. Where the bottom of new structures extends beneath the top-of-Bay-Mud elevation, they should be designed in consideration of low bearing capacities and design skin friction values.

Where structures and utility trenches are "bottomed" in Bay Mud, we recommend over-excavating a minimum of 12 inches below the bottom of the planned foundation or pipe flowline, placing stabilization fabric (Mirafi 500X or equivalent) on the soft Bay Mud, and placing $\frac{3}{4}$ -inch lightweight lava rock to raise the grade to the specified subgrade elevation and reduce the risk of future settlement. Lava rock backfill in Bay Mud zones should be placed to at least the top-of-Bay-Mud elevation and be lightly compacted with vibratory equipment such that material is consolidated but the gradation is unaffected (i.e., use lightweight equipment that allows gentle compaction without breaking down individual lava rock pieces). Stabilization fabric should then be wrapped over the top of the lava rock.

A buoyant uplift force will develop when the water level within below-grade facilities is lower than the exterior groundwater level. Under "wintertime" (rainy season) conditions or during a flood event, the groundwater elevation should be assumed to be at the ground surface for design

purposes in some areas. The design engineer will need to determine the maximum differential between the exterior and interior water levels and should design any concrete slab floors to resist external hydrostatic pressures.

Resistance to uplift includes the weight of the structure plus the skin friction on the exterior of the structure. If necessary, the uplift resistance can be increased by structurally extending the foundation beyond the limits of the walls and including the buoyant weight of soil above the footing extensions in the total weight of the structure. Alternatively, helical anchors or other deep foundations could be utilized to provide uplift resistance. Geotechnical design criteria for below-grade structures should be developed on the basis of subsurface exploration and laboratory testing performed as part of a future design-level Investigation.

9.4 Asphalt-Concrete Pavements

New pavements will be required for backfilled areas which extend into traffic areas. We have calculated preliminary pavement sections in accordance with Caltrans procedures for flexible pavement design. We have provided a range of Traffic Indices (TI) from 4 to 9 depending on the expected traffic loads for a twenty-year design life. For preliminary design, we have assumed a subgrade R-value of 20 and 50 for undocumented fill and Class 2 aggregate subbase, respectively. Note that thinner pavement sections will be possible in areas with more supportive subgrade, which likely includes most alluvial/colluvial and bedrock materials. Preliminary recommended pavement sections are shown in Table 3. We should confirm recommended pavement sections on the basis of R-Value testing performed during a future design-level Investigation.

Table 3 – Preliminary Asphalt Pavement Sections

Select Fill Soil Backfill R-Value = 20			Aggregate Subbase Rock Backfill R-Value = 50		
<u>T.I.</u>	<u>Asphalt Concrete⁽¹⁾</u>	<u>Aggregate Base Rock</u>	<u>T.I.</u>	<u>Asphalt Concrete⁽¹⁾</u>	<u>Aggregate Base Rock</u>
4.0	3.0 inches	5.0 inches	4.0	2.5 inches	3.0 inches
5.0	3.5 inches	7.0 inches	5.0	3.0 inches	3.0 inches
6.0	4.0 inches	9.0 inches	6.0	3.5 inches	6.0 inches
7.0	5.0 inches	10.0 inches	7.0	4.0 inches	7.0 inches
8.0	6.0 inches	12.0 inches	8.0	5.0 inches	8.0 inches
9.0	7.0 inches	14.0 inches	9.0	5.5 inches	8.0 inches

- (1) Asphalt concrete should conform to the criteria for asphalt presented in Section 39 of the Caltrans Standard Specifications. The asphalt concrete shall be placed in layers not exceeding 2.5 inches in thickness and compacted to at least 95 percent relative compaction.

10.0 SUPPLEMENTAL GEOTECHNICAL SERVICES

Issuance of this letter completes the scope of our Phase 1 services. We can be available to respond to any geotechnical questions that may arise, and to assist in selection of the “preferred” alignment(s) as needed. Once the “preferred” alignment(s) have been identified and as project details become available, we can perform additional pre-design and design-level studies, including subsurface exploration, laboratory testing, and engineering analyses, to develop preliminary and design-level geotechnical recommendations and criteria for the project.

As project plans are nearing completion, we should review them to confirm that the intent of our geotechnical recommendations has been incorporated. During construction, we should be present intermittently to observe and test compacted trench backfill, and to consult regarding unanticipated soil or groundwater conditions, if encountered. The purpose of our observation and testing is to confirm that site conditions are as anticipated, to adjust our recommendations and design criteria if needed, and to confirm that the Contractor’s work is performed in accordance with the project plans and specifications.

11.0 LIMITATIONS

We believe this report has been prepared in accordance with generally accepted geotechnical engineering practices in the San Francisco Bay Area at the time the report was prepared. This report has been prepared for the exclusive use of Marin Municipal Water District and/or their assignees specifically for this project. No other warranty, expressed or implied, is made. Our evaluations and recommendations are based on the data obtained during our subsurface exploration program and our experience with soils in this geographic area. Our approved scope of work did not include a detailed environmental assessment of the site. We recommend that an environmental consultant be retained to evaluate environmental-related issues.

The evaluations and recommendations do not reflect variations in subsurface conditions that may exist between boring locations or in unexplored portions of the site. Should such variations become apparent during construction, the general recommendations contained within this report will not be considered valid unless Miller Pacific is given the opportunity to review such variations and revise or modify our recommendations accordingly. No changes may be made to the general recommendations contained herein without the written consent of Miller Pacific.

We recommend that this report, in its entirety, be made available to project team members, contractors, and subcontractors for informational purposes and discussion. We intend that the information presented within this report be interpreted only within the context of the report as a whole. No portion of this report should be separated from the rest of the information presented herein. No single portion of this report shall be considered valid unless it is presented with and as an integral part of the entire report.

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 - Frame 1K-30
 - Frame 2K-117
 - Frame 2K-119
 - Frame 2K-155
 - Frame 2K-156
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 - Frame 7K-140
 - Frame 7K-141
 - Frame 8K-92
 - Frame 8K-107
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 - Frame 41-7
 - Frame 46-170
 - Frame 46-171
 - Frame 47-46
 - Frame 47-63
 - Frame 66-154
 - Frame 66-254
 - Frame 66-255
 - Frame 66-260
 - Frame 67-2
 - Frame 67-9
 - Frame 67-113
 - Frame 67-121
 - Frame 74-4
 - Frame 74-5
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 - Frame 530-33



Legend and Key to Map Symbols

Alignment

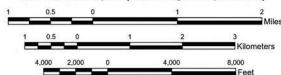
- COTATI-1
- COTATI-2
- PETA-1
- PETA-2
- PETA-4
- STAF-1
- STAF-2
- STS-1
- STS-2

Magnetic North (Degrees): 13.14°E



SITE VICINITY MAP

MAP SCALE 1:48,000 (1 INCH EQUALS 4,000 FEET)

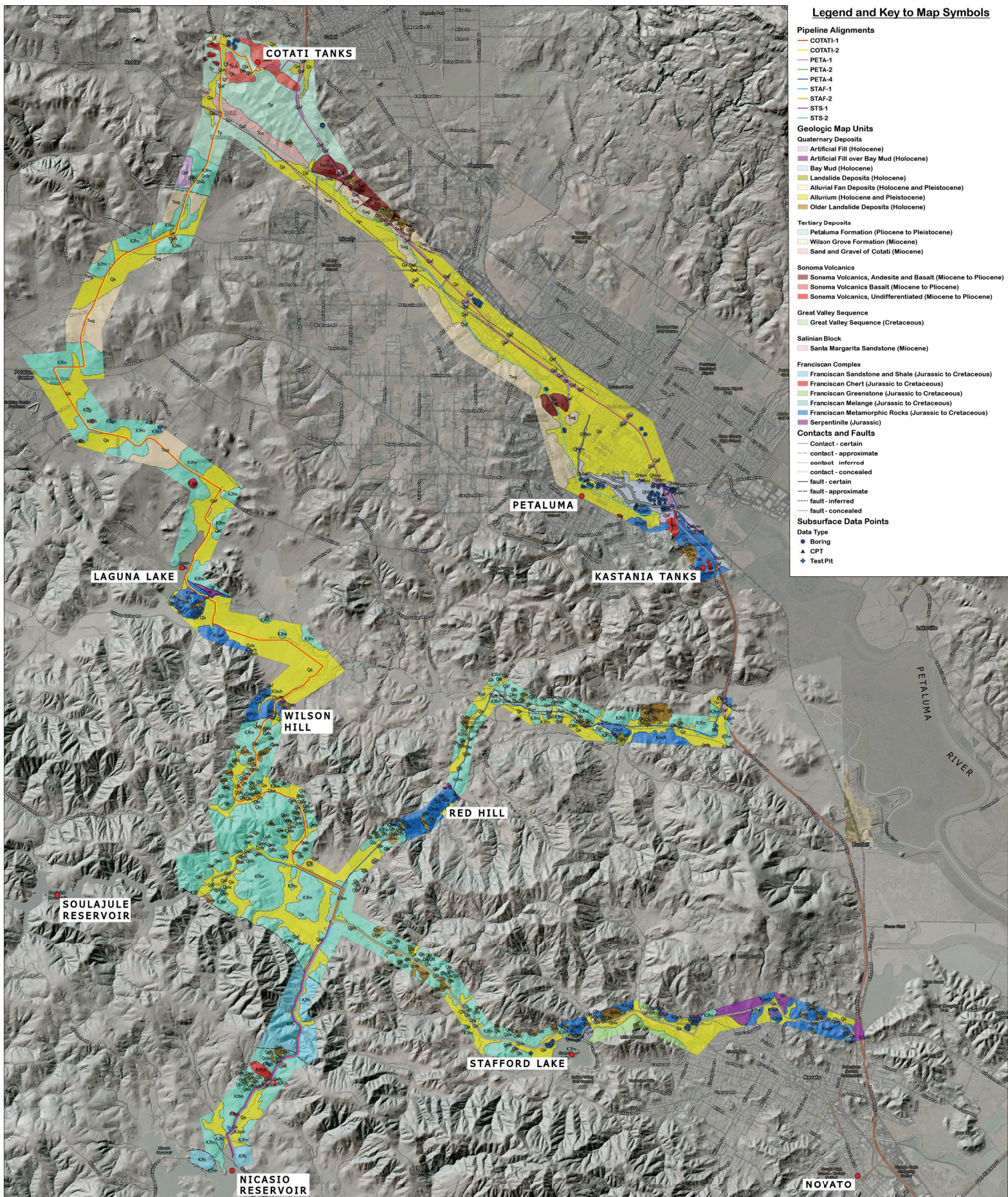


SITE VICINITY MAP

Marin Municipal Water District
Conveyance Pipeline Project
Marin and Sonoma Counties, California

Date: 7/3/2024 6:39 PM

Project No. 187.523



Legend and Key to Map Symbols

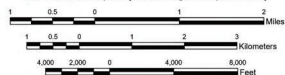
- Pipeline Alignments**
- COTATI-1
 - COTATI-2
 - PETA-1
 - PETA-2
 - PETA-4
 - STAF-1
 - STAF-2
 - STS-1
 - STS-2
- Geologic Map Units**
- Quaternary Deposits**
- Artificial Fill (Holocene)
 - Artificial Fill over Bay Mud (Holocene)
 - Bay Mud (Holocene)
 - Landslide Deposits (Holocene)
 - Alluvial Fan Deposits (Holocene and Pleistocene)
 - Alluvium (Holocene and Pleistocene)
 - Older Landslide Deposits (Holocene)
- Tertiary Deposits**
- Petaluma Formation (Pliocene to Pleistocene)
 - Wilson Grove Formation (Miocene)
 - Sand and Gravel of Cotati (Miocene)
- Sonoma Volcanics**
- Sonoma Volcanics, Andesite and Basalt (Miocene to Pliocene)
 - Sonoma Volcanics Basalt (Miocene to Pliocene)
 - Sonoma Volcanics, Undifferentiated (Miocene to Pliocene)
- Great Valley Sequence**
- Great Valley Sequence (Cretaceous)
- Salinian Block**
- Santa Margarita Sandstone (Miocene)
- Franciscan Complex**
- Franciscan Sandstone and Shale (Jurassic to Cretaceous)
 - Franciscan Chert (Jurassic to Cretaceous)
 - Franciscan Greenstone (Jurassic to Cretaceous)
 - Franciscan Melange (Jurassic to Cretaceous)
 - Franciscan Metamorphic Rocks (Jurassic to Cretaceous)
 - Serpentine (Jurassic)
- Contacts and Faults**
- contact - certain
 - contact - approximate
 - contact - inferred
 - contact - concealed
 - fault - certain
 - fault - approximate
 - fault - inferred
 - fault - concealed
- Subsurface Data Points**
- Data Type**
- Boring
 - CPT
 - Test Pit

Magnetic North (Degrees): 13.14°E



PRELIMINARY GEOLOGIC MAP

MAP SCALE 1:48,000 (1 INCH EQUALS 4,000 FEET)

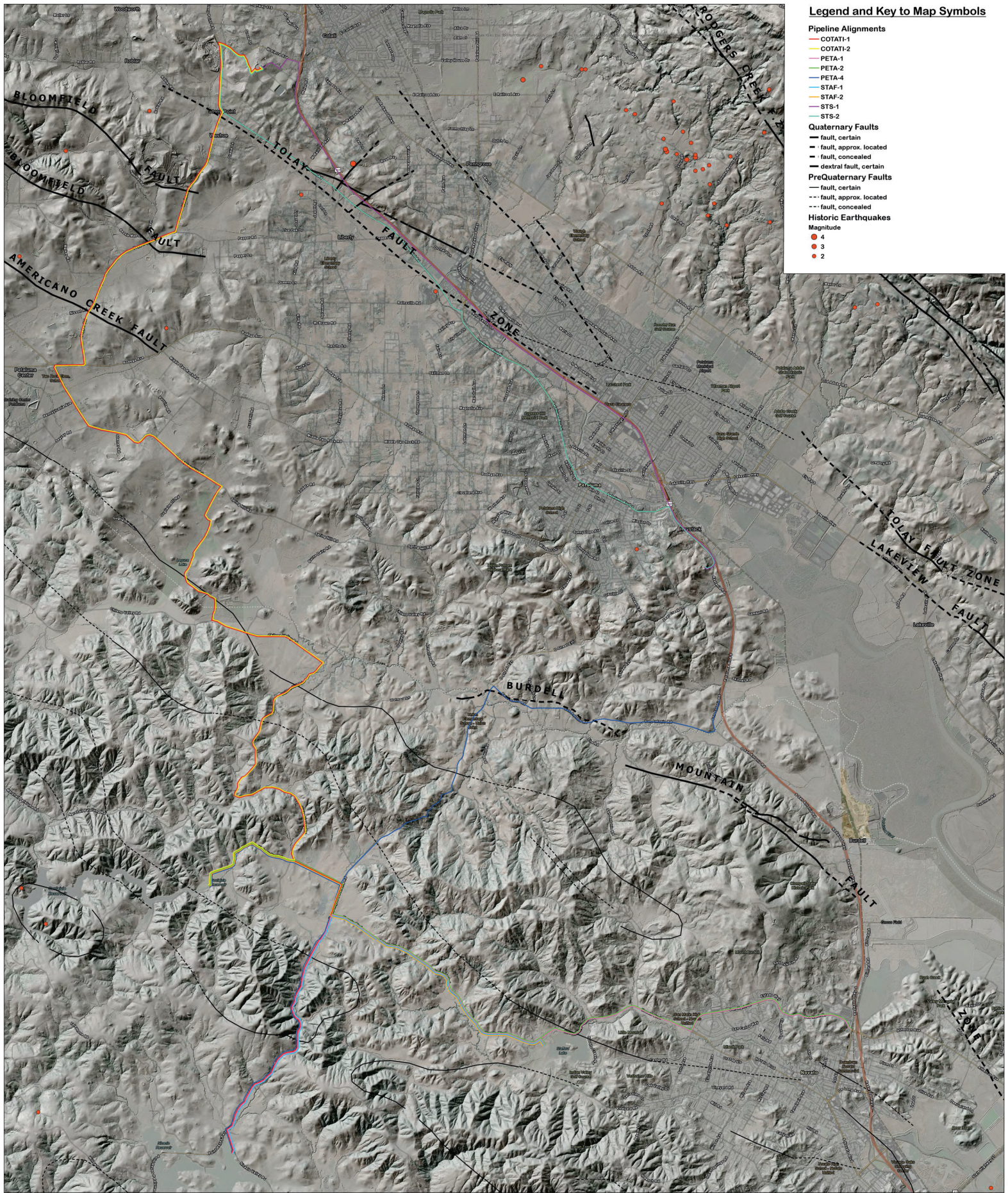


PRELIMINARY GEOLOGIC MAP

Marin Municipal Water District
Conveyance Pipeline Project
Marin and Sonoma Counties, California

Date: 7/3/2024 6:39 PM

Project No. 187.523



Legend and Key to Map Symbols

Pipeline Alignments

- COTAT-1
- COTAT-2
- PETA-1
- PETA-2
- PETA-4
- STAF-1
- STAF-2
- STS-1
- STS-2

Quaternary Faults

- fault, certain
- fault, approx. located
- fault, concealed
- destral fault, certain

PreQuaternary Faults

- fault, certain
- fault, approx. located
- fault, concealed

Historic Earthquakes

- Magnitude
- 4
 - 3
 - 2

Magnetic North (Degrees): 13.14°E



FAULT AND SEISMICITY MAP

MAP SCALE 1:48,000 (1 INCH EQUALS 4,000 FEET)



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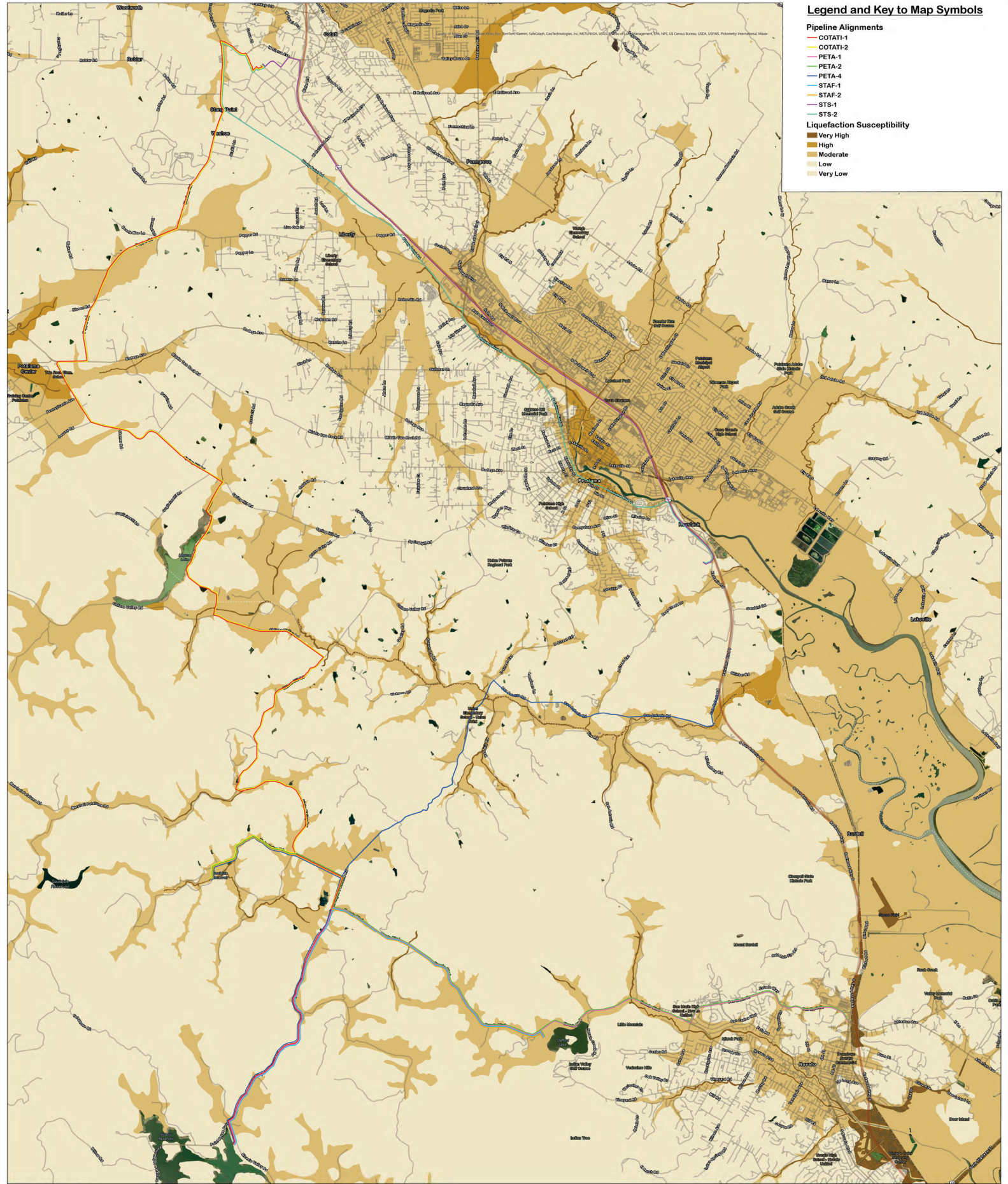
FAULT AND SEISMICITY MAP

Marin Municipal Water District
Conveyance Pipeline Project
Marin and Sonoma Counties, California

Date: 7/3/2024 6:39 PM

Project No. 187.523

Plate
3 of 5



Legend and Key to Map Symbols

Pipeline Alignments

- COTATI-1
- COTATI-2
- PETA-1
- PETA-2
- PETA-4
- STAF-1
- STAF-2
- STS-1
- STS-2

Liquefaction Susceptibility

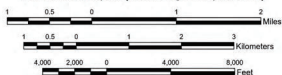
- Very High
- High
- Moderate
- Low
- Very Low

Magnetic North (Degrees): 13.14°E



LIQUEFACTION SUSCEPTIBILITY MAP

MAP SCALE 1:48,000 (1 INCH EQUALS 4,000 FEET)



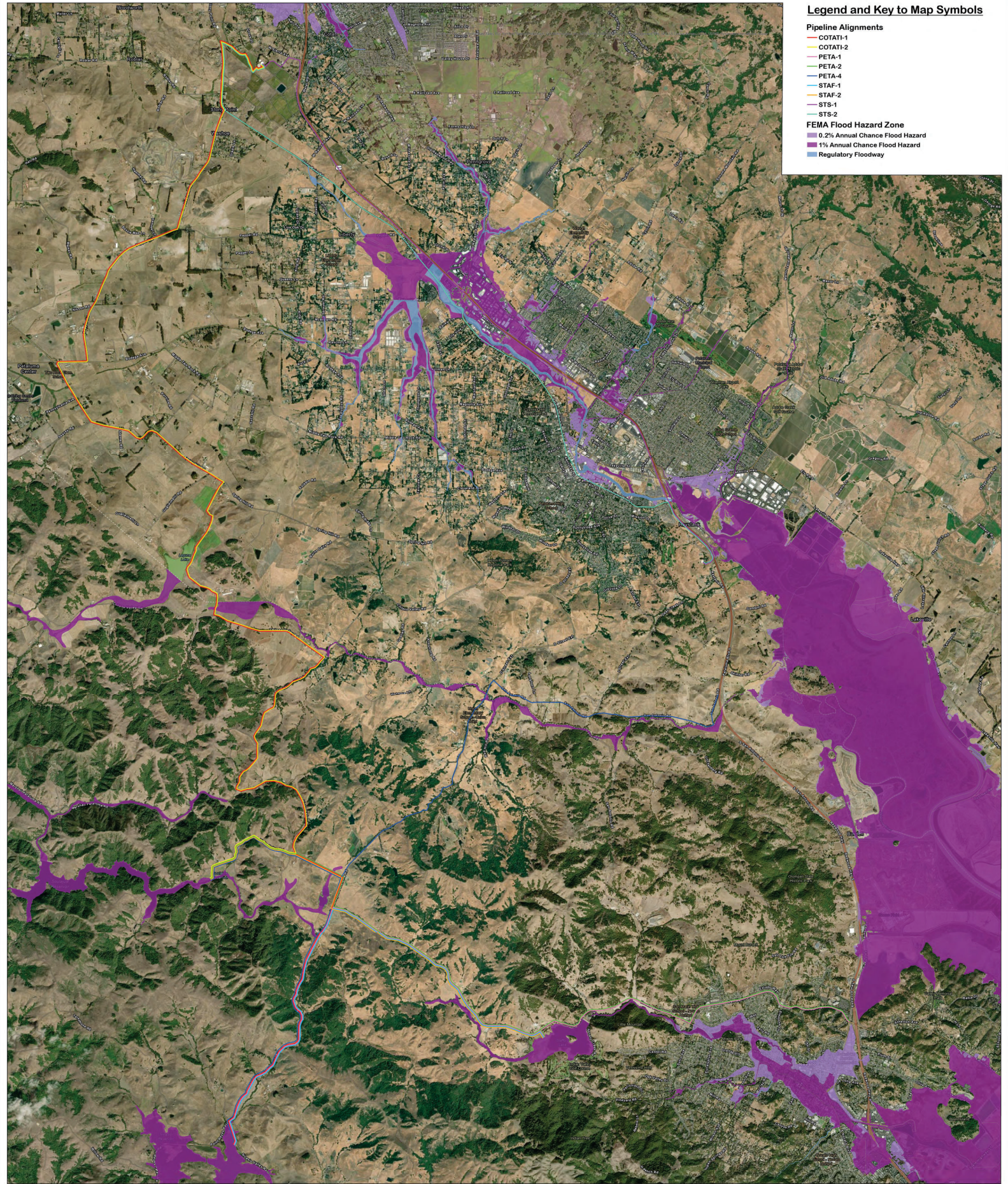
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LIQUEFACTION SUSCEPTIBILITY MAP

Marin Municipal Water District
Conveyance Pipeline Project
Marin and Sonoma Counties, California

Date: 7/3/2024 6:39 PM

Project No. 187.523



Legend and Key to Map Symbols

Pipeline Alignments

- COTATI-1
- COTATI-2
- PETA-1
- PETA-2
- PETA-4
- STAF-1
- STAF-2
- STS-1
- STS-2

FEMA Flood Hazard Zone

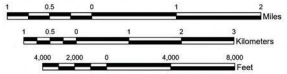
- 0.2% Annual Chance Flood Hazard
- 1% Annual Chance Flood Hazard
- Regulatory Floodway

Magnetic North (Degrees): 13.14°E



FEMA FLOOD HAZARD MAP

MAP SCALE 1:48,000 (1 INCH EQUALS 4,000 FEET)



FEMA FLOOD HAZARD MAP	
Marin Municipal Water District Conveyance Pipeline Project Marin and Sonoma Counties, California	
Date: 7/3/2024 6:39 PM	Plate 5 of 5
Project No. 187.523	

APPENDIX 3C

ULTEIG PUMP STATION POWER SUPPLY EVALUATION

Marin Municipal Water District (MMWD) Water Supply Conveyance Improvements Project – Pump Station Power Supply Evaluation



Carollo Engineers

Department

Energy Solutions

Prepared For

Carollo Engineers / Marin Municipal Water District (MMWD)

Prepared By

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Ulteig Project Number

24.02048

Revision History

Revision	Date	Description
A	7/26/2024	Draft Submittal
A	8/9/2024	Submittal
B	9/6/2024	Revision Submittal
B	9/13/2024	Revision Correction

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Legal Notice

We agree that the enclosed study has been performed by qualified personnel in accordance with generally accepted standards. The conclusions and recommendations are based upon the experience and judgment of our personnel. Because such conclusions and recommendations are based on experience and judgment, we DO NOT WARRANT that our conclusions and recommendations are the only reasonable conclusions and recommendations which might be made by a qualified consultant. We make no warranty with respect to any equipment which you may procure or cause to be built based in whole or in part on any conclusions which we provide hereunder. You shall obtain any and all warranties pertaining to such equipment from the supplier thereof.

Definition and Acronyms

CAISO	California Independent System Operator
DIDF	Distribution Investment Deferral Framework
ICA	Integration Capacity Analysis
MMWD	Marin Municipal Water District
PG&E	Pacific Gas & Electric Company
POI	Point of Interconnection
UEI	Ulteig Engineers Inc.

1 Executive Summary

Ulteig Engineers Inc (UEI) performed a transmission and distribution analysis study for Carollo Engineers, Inc (Carollo) and the Marin Municipal Water District (MMWD) to evaluate the impact of 6 proposed pump stations on the Pacific Gas & Electric (PG&E) power transmission and distribution systems. These pump stations are grouped into 3 different scenarios, outlined in **Table 1**. The primary goal of the analysis is to determine if any of the pump station sites have fatal flaws related to PG&E's ability to provide power for the new facilities. The analysis considers the impacts of the new pump station sites to PG&E's transmission and distribution grid.

Table 1: Pump Station Scenarios

Scenario	Pump Station	Horsepower (HP)	Approximate MW Equivalent
1	PETA-3 PS A	2500	1.8643
	PETA-3 PS B	1000	0.7457
2	PETA-4 PS A	3000	2.2371
	PETA-4 PS B	3500	2.6100
3	COTATI-3 PS A	3500	2.6100
	COTATI-3 PS B	4500	3.3557

Due to this study requiring both transmission and distribution analysis, the study was performed in two parts. The first part evaluated the impacts of the new pump stations loads on the PG&E transmission area using the California Independent System Operator (CAISO) summer and winter peak cases. The study concentrated on evaluating the impact of each pump station scenario interconnected on the nearby 60 kV PG&E transmission system. Each pump station was geographically analyzed through ArcGIS to identify the nearest 60 kV line. Each respective transmission line was then tapped as the equivalent Point of Interconnection (POI). Due to similar distances to 2 separate 60 kV lines, PETA-3 PS A had 2 different POIs identified. This resulted in creating scenarios 1a and 1b which are outlined in Table 2.

Thermal analysis was conducted by having a benchmark and study case for both summer and winter cases. The benchmark cases do not have the pump stations in the model and the study cases do include the pump stations modeled as a load in each model. Both the benchmark and study case were analyzed under system intact and (N-1) single contingency as a part of the thermal analysis. System intact is the simulation of the transmission system with no outages while (N-1) single contingency is the simulation of the transmission system following the loss of a single element which includes transformers, transmission lines, and generators. The thermal analysis results can be found in the tables under **Section 3**.

The second part of the study evaluated the impacts of the new pump station loads on the PG&E distribution system. To determine the interconnection at the distribution level, PG&E's Distribution Investment Deferral Framework (DIDF) & Integration Capacity Analysis (ICA) maps provided on their website were used. The ICA maps were also used in order to identify the capacity on nearby distribution circuits to provide a reasonable option for interconnection. Estimated interconnection costs for each pump station were also outlined using public per unit rates provided by PG&E which are outlined in **Section 2**.

2 Distribution Study

2.1 Assumptions

- The following analysis assumes that PG&E allows 100% loading on their 12kV circuits.
 - Historically utilities have standards in place that limit the amount of loading on their circuits. These limits are put in place for a number of reasons. The primary reason is if headroom is left on a circuit, during an outage the circuit can pick up additional load from nearby circuits to get customers power on quicker. In addition, leaving excess capacity allowed for temporary load increases in case construction crews were behind. Today, however, circuits are becoming more and more loaded, and construction costs are becoming more and more expensive and planned outages are more common. For all of these reasons companies are increasing the amount of load they allow on their circuits. There is evidence through public portals that PG&E allows loading up to 100% so the assumption is made that this loading would be allowed on these circuits.
- All motor loads are assumed to be at .95 power factor (PF) and analysis is based on kW loads not KVA.
 - A PF of 0.95 is a common assumption for motor loads. Historically older motors did not have power factor correction capacitors so motors would have much worse PF. Today most utilities have PF charges in their tariffs so motor manufacturers and sites with large motor loads have started incorporating PF correction. PF correction is rarely ever perfect as the closer to unity a designer gets the more capacitive correction is required. In addition, most utilities PF charges are for PFs that are worse than 0.95. KW calculations are used instead of KVA calculations because these are the values that are provided from PG&E. PG&E likely uses the same assumption of a 0.95 PF that we are using when completing KVA calculations so this shouldn't be a concern.

2.2 Scenario 1 - PETA-3 PS A and PETA-3 PS B

Scenario 1 represents two pump stations, PETA-3 PS A and PETA-3 PS B. PETA-3 PS A is a 2,500hp pump station and represents a 1.8387MW load on the PG&E system. PETA-3 PS B is a 1000hp pump station and represents a 0.7355MW load on the PG&E system.

2.2.1 PETA-3 PS A

The two nearest PG&E 12kV circuits are Stafford 1101, which has only 1MW of loading, and Novato 1102 which only has customer confidential load information. Neither of these sites were evaluated because they would have presented with fatal flaws to the project. Either there wasn't enough capacity to serve the pump station or there was not enough information to complete the evaluation. Should PETA-3 be identified as the selected alternative, these two circuits can be explored as alternatives to Novato 1101, which is explored below, with the understanding of the above limitations.

PETA-3 PS A is located off San Marin Dr in Novato, California. One distribution circuit serving these areas is Novato 1101. Novato 1101 has a circuit ID of 4211101 and is a 12kV feeder with a circuit rating of 8.13 MW. The 2027 projected load of Novato 1101 is 2.29MW which leaves the circuit approximately 29.52% loaded. The available capacity of Novato 1101 based on the 2027 projected load is 5.73 MW which is enough capacity to serve the 1.8387MW load of PETA-3 PS A.

For output pictures of the area around PETA-3 PS A please see Appendix B. The nearest line is a single phase tap off the main line. The distance from the pump site to the single-phase line is approximately 300 feet. The length of the single phase tap is approximately 3,600 feet. To power PETA-3 PS A it is assumed 3 phase power would be required so the installation of the pump would require building 300 feet of new 3 phase 15kv line and rebuilding approximately 3,600 feet of single-phase line to three phase line to interconnect PETA-3 PS A into the three-phase system of Novato 1101. Based on Google Earth Street view review it appears this line is overhead so building or rebuilding sections are assumed to be overhead as well. However, since this line runs through a neighborhood it might be requested to underground this section which would increase cost. In addition to building the lines, it is expected that PG&E will require switching infrastructure for the newly constructed three phase tap and additional line protection. Finally due to the size of the pump station this study assumes a primary service metering installation. The total estimated cost to interconnect this system is \$838,725.00. The cost estimate for this work was developed based on per unit rates provided by PG&E. These rates and a detailed breakdown of the estimate can be found in Appendix B.

2.2.2 PETA-3 PS B

PETA-3 PS B is located on the northeast shore of Stafford Lake near Stafford Lake Park California.

The nearest circuit is Stafford 1102. Stafford 1102 has a circuit ID of 43201102 and is a 12kV feeder with a circuit rating of 12.23 MW. The 2027 projected load of Stafford 1102 is 9.73MW which leaves the circuit approximately 80.70% loaded. The available capacity of Stafford 1102 is 2.36 MW which is enough capacity to serve the 0.7355MW load of pump PETA-3 PS B.

For output pictures of the area around PETA-3 PS B please see Appendix B. The nearest line is a three-phase tap which appears to run directly adjacent to the pump location, we are assuming only a short 50-foot connection to the PG&E system for this pump station site. To interconnect PETA-3 PS B into the PG&E system would require no line improvements or upgrades based on PG&E's D IDF maps. It is expected that PG&E will require switching infrastructure for the newly constructed interconnection and additional line protection. Finally due to the size of the pump station this study assumes a primary service metering installation. The total estimated cost to interconnect this system is \$199,680.00. The cost estimate for this work was developed based on per unit rates provided by PG&E. These rates and a detailed breakdown of the estimate can be found in Appendix B.

2.3 Scenario 2 - PETA-4 PS A and PETA-4 PS B

Scenario 2 represents two pump stations, PETA-4 PS A and PETA-4 PS B. PETA-4 PS A is a 3,000hp pump station and represents a 2.2064MW load on the PG&E system. PETA-4 PS B is a 3,500hp pump station and represents a 2.5742MW load on the PG&E system.

2.3.1 PETA-4 PS A

PETA-4 PS A is located off highway 101 west of San Antonio, California.

The nearest circuit is Petaluma C 1109. The section of Petaluma C 1109 has a capacity of 11.33MW and 2027 load of 9.51MW. This leaves a capacity of 1.82MW on the circuit which is not adequate for a 2.2064 MW pump. Since the feeder is

overloaded, it is important to understand if the limitation is also existing at the substation, known as a “Bank” at PG&E. If the feeder is overloaded and the Bank is not, it is possible to only make distribution upgrades to serve the load. If the feeder and the Bank are overloaded, then a new substation and distribution circuit would be required, which is a significant expense. Petaluma C Bank 4 has a substation ID of 4263 and is a 12kV substation with a bank rating of 15.8 MW. The 2027 projected load of Petaluma C Bank 4 is 10.52MW which leaves the bank approximately 66.58% loaded. The available capacity of Petaluma C Bank 4 is 5.28 MW which is enough capacity to serve the 2.2064 MW load of PETA-4 PS A.

For output pictures of the area around PETA-4 PS A please see Appendix B. The nearest line is the three phase Petaluma C 1109. The distance from the pump station site to Petaluma C Bank 4 is 24,905 feet. To power PETA-4 PS A it is assumed 3 phase power would be required so the installation of the pump station would require 24,905 feet of reconductor or rebuild. Based on Google Earth Street view review it appears this line is overhead so building or rebuilding sections are assumed to be overhead. In addition to building the lines, it is expected that PG&E will require switching infrastructure for the newly constructed three phase tap and additional line protection. Finally due to the size of the pump station this study assumes a primary service metering installation. The total estimated cost to interconnect this system is \$4,223,414.00. The cost estimate for this work was developed based on per unit rates provided by PG&E. These rates and a detailed breakdown of the estimate can be found in Appendix B.

2.3.2 PETA-4 PS B

PETA-4 PS B is located on the west side of Pt. Reyes – Petaluma Rd southwest of Petaluma, California.

The nearest circuit is Petaluma C 1109. The section of Petaluma C 1109 has a capacity of 11.33MW and 2027 load of 9.51 MW. This leaves a capacity of 1.82 MW on the circuit which is not adequate for a 2.5742 MW pump. Since the feeder is overloaded, it is important to understand if the limitation is also existing at the substation, known as a “Bank” at PG&E. If the feeder is overloaded and the Bank is not, it is possible to only make distribution upgrades to serve the load. If the feeder and the Bank are overloaded, then a new substation and distribution circuit would be required, which is a significant expense. Petaluma C Bank 4 has a substation ID of 4263 and is a 12kV substation with a bank rating of 15.8 MW. The 2027 projected load of Petaluma C Bank 4 is 10.52MW which leaves the bank approximately 66.58% loaded. The available capacity of Petaluma C Bank 4 is 5.28 MW which is enough capacity to serve the 2.5742 MW load of PETA-4 PS B.

For output pictures of the area around PETA-4 PS B please see Appendix B. The nearest line is the three phase Petaluma C 1109. The distance from the pump station site to Petaluma C Bank 4 is 33,236 feet. To power PETA-4 PS B it is assumed 3 phase power would be required so the installation of the pump station would require 33,236 feet of reconductor or rebuild. Based on Google Earth Street view review it appears this line is overhead so building or rebuilding sections are assumed to be overhead. In addition to building the lines, it is expected that PG&E will require switching infrastructure for the newly constructed three phase tap and additional line protection. Finally due to the size of the pump station this study assumes a primary service metering installation. The total estimated cost to interconnect this system is \$5,608,029.00. The cost estimate for this work was developed based on per unit rates provided by PG&E. These rates and a detailed breakdown of the estimate can be found in Appendix B.

2.4 Scenario 3 - COTATI-3 PS A and COTATI-3 PS B

Scenario 3 represents two pump stations COTATI-3 PS A and COTATI-3 PS B. COTATI-3 PS A is a 3,500 hp pump station and represents a 2.5742 MW load on the PG&E system. COTATI-3 PS B is a 4,500 hp pump station and represents a 3.3097 MW load on the PG&E system.

2.4.1 COTATI-3 PS A

COTATI-3 PS A is located near the intersection of Stony Point Rd and Mecham Rd.

The nearest circuit is COTATI 1105 and has a circuit ID of 42271105 and is a 12kV feeder with a circuit rating of 10.46 MW. The 2027 projected load of COTATI 1105 is 5.3 MW which leaves the circuit approximately 50.67% loaded. The available capacity of COTATI 1105 is 5.16 MW which is enough capacity to serve the 2.5742 MW load of pump COTATI-3 PS A.

For output pictures of the area around COTATI-3 PS A please see Appendix B. COTATI 1105, which is immediately adjacent to the project is nearly immediately across the street from COTATI substation. Due to this short distance Ulteig assumed a tap immediately outside the substation to minimize outages experienced at the pump station site. It is assumed that the line would be 895 feet. It is assumed this line would be built overhead but because it is such a short distance underground could be considered. In addition to building the lines, it is expected that PG&E will require switching infrastructure for the newly constructed three phase tap and additional line protection. Finally due to the size of the pump station this study assumes a primary service metering installation. The total estimated cost to interconnect this system is \$339,105.00. The cost estimate for this work was developed based on per unit rates provided by PG&E. These rates and a detailed breakdown of the estimate can be found in Appendix B.

2.4.2 COTATI-3 PS B

COTATI-3 PS B is located on the west side of Wilson Hill Rd southwest of Petaluma, California.

The nearest circuit is Petaluma C 1108. The section of Petaluma C 1108 has a capacity of 12.1MW and 2027 load of 10.71 MW. This leaves a capacity of 1.39 MW on the circuit which is not adequate for a 3.3097 MW pump. In addition, the nearest section is only a single-phase line. Since the feeder is overloaded, it is important to understand if the limitation is also existing at the substation, known as a "Bank" at PG&E. If the feeder is overloaded and the Bank is not it is possible to only make distribution upgrades to serve the load. If the feeder and the Bank are overloaded, then a new substation and distribution circuit would be required, which is a significant expense. Petaluma C Bank 4 has a substation ID of 4263 and is a 12kV substation with a bank rating of 15.8 MW. The 2027 projected load of Petaluma C Bank 4 is 10.52MW which leaves the bank approximately 66.58% loaded. The available capacity of Petaluma C Bank 4 is 5.28 MW which is enough capacity to serve the 3.3097 MW load of COTATI-3 PS B.

For output pictures of the area around COTATI-3 PS B please see Appendix B. The nearest line is the three phase Petaluma C 1108. The distance from the pump station site to Petaluma C Bank 4 is 36,218 feet. To power the COTATI-3 PS B it is assumed 3 phase power would be required so the installation of the pump station would require 36,218 feet of reconductor or rebuild. Based on Google Earth Street view review it appears this line is overhead so building or rebuilding sections are assumed to be overhead. In addition to building the lines, it is expected that PG&E will require switching

infrastructure for the newly constructed three phase tap and additional line protection. Finally due to the size of the pump station this study assumes a primary service metering installation. The total estimated cost to interconnect this system is \$6,100,059.00. The cost estimate for this work was developed based on per unit rates provided by PG&E. These rates and a detailed breakdown of the estimate can be found in Appendix B.

3 Transmission Study

The Transmission Study evaluated the impacts of the 6 potential pump stations on the surrounding PG&E transmission system. These pump stations were modeled assuming a power factor of 0.8. The analysis included performing system intact and single contingency (N-1) analysis in order to identify transmission lines that thermally overload due to the addition of the new pump stations loads. System intact is the simulation of the transmission system with no outages while (N-1) single contingency is the simulation of the transmission system following the loss of a single element which includes transformers, transmission lines, and generators. The service area being evaluated for single contingency analysis is 5 electrical buses away from the POI all within the identified PG&E area for all scenarios.

The 6 pump stations were studied in 3 scenarios, with the identified nearest line into the PG&E transmission system found using ArcGIS. The identified line was then tapped at 50% of the transmission line length which split the impedance equally (50%) of the identified POI. For example, in Scenario 1, PETA-3 PS A will be tapped 50% of the transmission line LKVLE JT to NVTO JCT to create the POI bus. This now splits the transmission line into 2 segments: POI to LKVLE JT & POI to NVTO JCT in the power flow model. Each pump station would be modeled as a load at each respective POI. The topology found in ArcGIS was also confirmed through both the CAISO Summer and Winter models used for the study. The pump station data is outlined below:

Table 2: Transmission Pump Storage Scenarios

Scenario	Pump Station	Horsepower (HP)	MW	Q (MVAR)	PF	Bus kV	POI
1	PETA-3 PS A	2500	1.8643	1.3982	0.8000	60	LKVLE JT to NVTO JCT
	PETA-3 PS B	1000	0.7457	0.5593	0.8000	60	STAF JCT to TOCA JCT
2	PETA-4 PS A	3000	2.2371	1.6778	0.8000	60	LKVLE JT to NVTO JCT
	PETA-4 PS B	3500	2.6100	1.9575	0.8000	60	LKVLE JT to NVTO JCT
3	COTATI-3 PS A	3500	2.6100	1.9575	0.8000	60	COTATI to SNMA TAP
	COTATI-3 PS B	4500	3.3557	2.5167	0.8000	60	COTATI to SNMA TAP

3.1 Scenario 1 – PETA-3 PS A and PETA-3 PS B

Scenario 1 includes two pump stations, PETA-3 PS A and PETA-3 PS B outlined in **Table 2**. PETA-3 PS A was interconnected at the Lakeville to Novato 60 kV line east of the pump station along Redwood Hwy S for Scenario 1a. Additionally, a secondary 60 kV line from Novato to Stafford was also identified for the pump station which is outlined in Scenario 1b. PETA-3 PS B was interconnected at the Stafford to Toca 60 kV line adjacent to the project. For output pictures of the area around both pumps in Scenario 1 please see **Appendix A**.

For the summer peak case, Scenario 1 saw a total of four monitored facilities with thermal overloads. Of these four overloads, all of them were pre-existing overloads under the benchmark case and were not new overloads due to the addition of the pump stations. With the addition of the pump stations, there was minimal contribution from the loads with a 0.03% thermal overload increase being the maximum change seen across the pre-existing overloads.

The winter peak case saw similar results with a total of thirteen monitored facilities with thermal overloads. No new overloads were seen due to the addition of the pump stations. There was also a minimal thermal loading increase seen due to the addition of the pump stations with the highest thermal increase being 0.08% across the pre-existing overloads. The results can be viewed in the tables below.

Table 3: Summer Peak Scenario 1 Results

Monitored Facility				Worst Study Overload AC (%)	Bench Worst Overload AC (%)	%Change
BRDGVLE	60.0	FRUTLDJT	60.0 ckt 1	105.32	105.29	0.03%
PLFLDJCT	60.0	PLAINFLDE	60.0 ckt 1	101.95	101.95	0.00%
PLFLDJCT	60.0	WINTERS	60.0 ckt 1	101.89	101.88	0.01%
LIVNGSTN	70.0	LVNGSTNT	70.0 ckt 1	101.29	101.29	0.00%
TABLE MTN	500	TM_VD_11	500 ckt 1	99.84	99.78	0.06%

Table 4: Winter Peak Scenario 1 Results

Monitored Facility				Worst Study Overload AC (%)	Bench Worst Overload AC (%)	%Change
HUMBOLDT	115	TRINITY	115 ckt 1	115.31	115.27	0.03%
IGNACIO	115	SAN RAFL	115 ckt 1	114.92	114.84	0.07%
WILDWOOD	115	COTWD_2E	115 ckt 1	112.67	112.64	0.03%
MLNO JCT	60.0	FULTON	60.0 ckt 1	112.4	112.37	0.03%
GT_MW_11	500	MIDWAY	500 ckt 1	110.07	110.06	0.01%
MARTNZ D	115	ALHAMTP1	115 ckt 1	109.62	109.57	0.05%
GATES	500	GT_MW_11	500 ckt 1	108.89	108.88	0.01%
MARTNZ D	115	ALHAMTP1	115 ckt 1	107.61	107.55	0.06%
MARTNZ D	115	ALHAMTP1	115 ckt 1	106.89	106.82	0.07%
MARTNZ D	115	ALHAMTP1	115 ckt 1	105.98	105.95	0.03%
MARTNZ D	115	ALHAMTP1	115 ckt 1	105.74	105.71	0.03%
IGNACIO	115	LS GLLNS	115 ckt 1	104.69	104.61	0.08%
CHRISTIE	60.0	FRANKLIN	60.0 ckt 2	103.58	103.57	0.01%
MN_MW_23	500	MIDWAY	500 ckt 2	99.1	99.1	0.00%

3.2 Scenario 2 – PETA-4 PS A and PETA-4 PS B

Scenario 2 represents the two pump stations, PETA-4 PS A and PETA-4 PS B outlined in **Table 2**. PETA-4 PS A was interconnected to the 60 kV line Lakeville to Novato that runs along Redwood Hwy S. PETA-4 PS B was located farther toward the coast compared to PETA-4 PS A but was also interconnected to the same 60 kV line Lakeville to Novato. For output pictures of the area around both pumps in Scenario 2 please see **Appendix A**.

For the summer peak case, Scenario 2 saw a total of nine monitored facilities with thermal overloads. All nine overloads were pre-existing overloads under the benchmark case and were not new overloads due to the addition of the pump stations. The addition of the pump stations saw a minimal contribution from the loads with a 0.05% thermal overload increase being the maximum change seen across the pre-existing overloads.

Conversely, the winter peak case saw similar results with a total of twenty-one monitored facilities with thermal overloads. No new overloads were seen due to the addition of the pump stations. There was also a minimal thermal loading increase seen due to the addition of the pump stations with the highest thermal increase being 0.11% across the pre-existing overloads. The results can be viewed in the tables below.

Table 5: Summer Peak Scenario 2 Results

Monitored Facility				Worst Study Overload AC (%)	Bench Worst Overload AC (%)	%Change
RICHMOND	115	SOBRANTE	115 ckt 2	135.32	135.32	0.00%
RICHMOND	115	ELCRTJ2	115 ckt 1	128.09	128.15	-0.05%
ELCRTJ2	115	SOBRANTE	115 ckt 1	126.51	126.54	-0.02%
NAPA	60.0	TULCY JT	60.0 ckt 1	116.12	116.1	0.02%
BRDGVLL	60.0	FRUTLDJT	60.0 ckt 1	105.34	105.29	0.05%
PLFLDJCT	60.0	PLAINFLDE	60.0 ckt 1	101.95	101.95	0.00%
PLFLDJCT	60.0	WINTERS	60.0 ckt 1	101.89	101.88	0.01%
BRDGVLL	60.0	FRUTLDJT	60.0 ckt 1	101.4	101.35	0.05%
LIVNGSTN	70.0	LVNGSTNT	70.0 ckt 1	101.29	101.29	0.00%
BRDGVLL	60.0	FRUTLDJT	60.0 ckt 1	99.95	99.9	0.05%
BRDGVLL	60.0	FRUTLDJT	60.0 ckt 1	99.92	99.87	0.05%
TABLE MTN	500	TM_VD_11	500 ckt 1	99.88	99.78	0.10%

Table 6: Winter Peak Scenario 2 Results

Monitored Facility				Worst Study Overload AC (%)	Bench Worst Overload AC (%)	%Change
RICHMOND	115	SOBRANTE	115 ckt 2	118.25	118.23	0.02%
RICHMOND	115	ELCRTJ2	115 ckt 1	115.99	115.97	0.02%
HUMBOLDT	115	TRINITY	115 ckt 1	115.31	115.27	0.03%
IGNACIO	115	SAN RAFL	115 ckt 1	114.95	114.84	0.10%
ELCRTJ2	115	SOBRANTE	115 ckt 1	114.37	114.35	0.02%
NAPA	60.0	TULCY JT	60.0 ckt 1	113.69	113.64	0.04%
WILDWOOD	115	COTWD_2E	115 ckt 1	112.68	112.64	0.04%
MLNO JCT	60.0	FULTON	60.0 ckt 1	112.42	112.37	0.04%
MARTNZ D	115	ALHAMTP1	115 ckt 1	110.13	110.01	0.11%
GT_MW_11	500	MIDWAY	500 ckt 1	110.08	110.06	0.02%
MARTNZ D	115	ALHAMTP1	115 ckt 1	109.66	109.57	0.08%
GATES	500	GT_MW_11	500 ckt 1	108.89	108.88	0.01%
MARTNZ D	115	ALHAMTP1	115 ckt 1	107.69	107.57	0.11%
MARTNZ D	115	ALHAMTP1	115 ckt 1	107.66	107.55	0.10%
MARTNZ D	115	ALHAMTP1	115 ckt 1	106.94	106.82	0.11%
MARTNZ D	115	ALHAMTP1	115 ckt 1	106.09	106.03	0.06%
MARTNZ D	115	ALHAMTP1	115 ckt 1	106.01	105.95	0.06%
MARTNZ D	115	ALHAMTP1	115 ckt 1	105.77	105.71	0.06%
IGNACIO	115	LS GLLNS	115 ckt 1	104.71	104.61	0.10%
CHRISTIE	60.0	FRANKLIN	60.0 ckt 2	103.59	103.57	0.02%
MLNO JCT	60.0	FULTON	60.0 ckt 1	100.4	100.36	0.04%
MN_MW_23	500	MIDWAY	500 ckt 2	99.11	99.1	0.01%

3.3 Scenario 3 – COTATI-3 PS A and COTATI-3 PS B

Scenario 3 represents the two pump stations, COTATI-3 PS A and COTATI-3 PS B outlined in **Table 2**. COTATI-3 PS A was interconnected at the adjacent 60 kV line Cotati to SNMA line. COTATI-3 PS B was located farther toward the coast compared to COTATI-3 PS A but was interconnected similarly to the same 60 kV line from Cotati to SNMA. For output pictures of the area around both pumps in Scenario 3 please see **Appendix A**.

For the summer peak case, Scenario 3 saw a total of four monitored facilities with thermal overloads. All four overloads were pre-existing overloads under the benchmark case and were not new overloads due to the addition of the pump stations. The addition of the pump stations saw a minimal contribution from the loads with a 0.13% thermal overload increase being the maximum change seen across the pre-existing overloads.

Conversely, the winter peak case saw results with a total of twelve monitored facilities with thermal overloads. Four of these overloads were new due to the addition of the pump stations. There was also some thermal loading increase seen due to the addition of the pump stations with the highest thermal increase being 20.62%. The results can be viewed in the tables below.

Table 7: Summer Peak Scenario 3 Results

Monitored Facility				Worst Study Overload AC (%)	Bench Worst Overload AC (%)	%Change
BRDGVLL	60.0	FRUTLDJT	60.0 ckt 1	105.43	105.29	0.13%
PLFLDJCT	60.0	PLAINFLDE	60.0 ckt 1	101.95	101.95	0.00%
PLFLDJCT	60.0	WINTERS	60.0 ckt 1	101.89	101.88	0.01%
LIVNGSTN	70.0	LVNGSTNT	70.0 ckt 1	101.29	101.29	0.00%
TABLE MTN	500	TM_VD_11	ckt 1	99.91	99.78	0.13%

Table 8: Winter Peak Scenario 3 Results

Monitored Facility				Worst Study Overload AC (%)	Bench Worst Overload AC (%)	%Change
MLNO JCT	60.0	FULTON	60.0 ckt 1	122.78	112.37	8.48%
MLNO JCT	60.0	LAGUNATP	60.0 ckt 1	119.68	97.54	18.50%
HUMBOLDT	115	TRINITY	115 ckt 1	115.36	115.27	0.08%
MLNO JCT	60.0	LAGUNATP	60.0 ckt 1	113.33	94.01	17.05%
COTATI-3	60.0	SNMA TAP	60.0 ckt 1	112.83	91.48	18.92%
WILDWOOD	115	COTWD_2E	115 ckt 1	112.73	112.64	0.08%
GT_MW_11	500	MIDWAY	500 ckt 1	110.08	110.06	0.02%
MARTNZ D	115	ALHAMTP1	115 ckt 1	109.69	109.57	0.11%
MLNO JCT	60.0	FULTON	60.0 ckt 1	109.41	100.36	8.27%
GATES	500	GT_MW_11	500 ckt 1	108.9	108.88	0.02%
CHRISTIE	60.0	FRANKLIN	60.0 ckt 2	103.59	103.57	0.02%
SNMA TAP	60.0	LAGUNATP	60.0 ckt 1	101.5	80.57	20.62%
MN_MW_23	500	MIDWAY	500 ckt 2	99.11	99.1	0.01%

4 Results

4.1 Transmission Study

For the transmission study, 3 study scenarios were reviewed for thermal overloads caused by the addition of the pump stations in the PG&E transmission system. The table below compares the total overloaded monitored facilities seen across both the summer and winter peak models. The bench results show the total overloads seen without the pump stations, while the study case includes the addition of the pump stations.

Table 9: Transmission Study Summary

Scenario	Pump Station	Horsepower (HP)	Summer Peak Case		Winter Peak Case	
			Bench Overloaded Monitored Facilities	Study Overloaded Monitored Facilities	Bench Overloaded Monitored Facilities	Study Overloaded Monitored Facilities
1	PETA-3 PS A	2500	4	4	13	13
	PETA-3 PS B	1000				
2	PETA-4 PS A	3000	9	9	21	21
	PETA-4 PS B	3500				
3	COTATI-3 PS A	3500	4	4	8	12
	COTATI-3 PS B	4500				

While the results show that all scenarios produce overloads on the PG&E transmission system, PG&E would need to mitigate pre-existing thermal overloads that are seen in the bench case. With this in mind, we first want to review new overloads that are caused with the addition of the pump stations. Once this criterion is reviewed, we then want to also consider pre-existing thermal overloads in the base case that are worsened with the addition of the pump stations.

The results show that Scenario 3 saw four new monitored facilities overloaded with the addition of the pump stations COTATI-3 PS A & COTATI-3 PS B. Because of this, Scenario 3 is the least favorable scenario. Comparing Scenario 2 and 1, we see that Scenario 2 has more total overloads identified across both summer and winter peak cases. This makes Scenario 1 the most favorable amongst the three with Scenario 2 being the second most favorable.

4.2 Distribution Study

For the distribution study, 3 scenarios were reviewed for overload conditions caused by the addition of the pump stations on the PG&E distribution system. The table below compares the total cost of each scenario with the least cost of the paired pumps at the top and the costliest at the bottom. The costs are based on expected upgrades to mitigate evaluated overloads on the PG&E system.

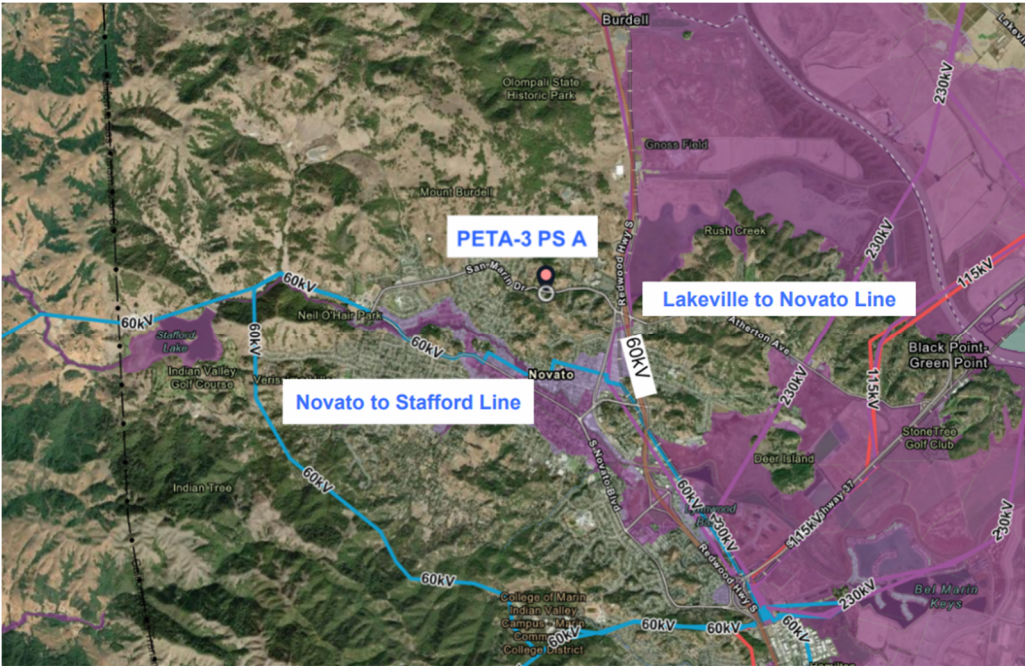
Table 10: Distribution Study Summary

Scenario	Pump Station	Horsepower (HP)	PG&E Upgrade Costs for Each Scenario
1	PETA-3 PS A	2500	\$1,038,405
	PETA-3 PB B	1000	
2	PETA-4 PS A	3000	\$9,831,443
	PETA-4 PS B	3500	
3	COTATI-3 PS A	3500	\$6,439,164
	COTATI-3 PS B	4500	

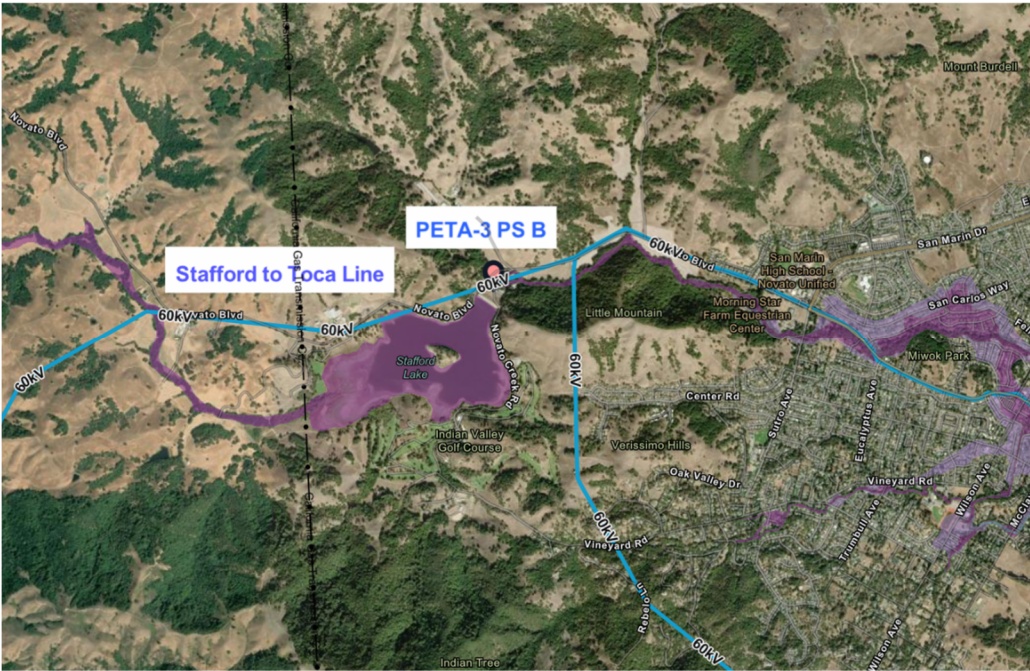
The only scenario that did not have line overloads was PETA-3. Both COTATI-3 and PETA-4 required significant line upgrades resulting in costs that exceed \$5M. The cost for PETA-3 might further be reduced if connecting into nearby circuits is explored. Utilizing these circuits for the purpose of this exploration was not possible due to the limited information provided by PG&E. COTATI-3 has a single pump site, COTATI-3 PS A that is located immediately adjacent to a substation resulting in very little cost for interconnection. COTATI-3 PS B requires significant investment however making this the 2nd most costly scenario. PETA-4 PS A and B both require significant line upgrades which made this the most expensive scenario.

Appendix A

PETA 3 PS A



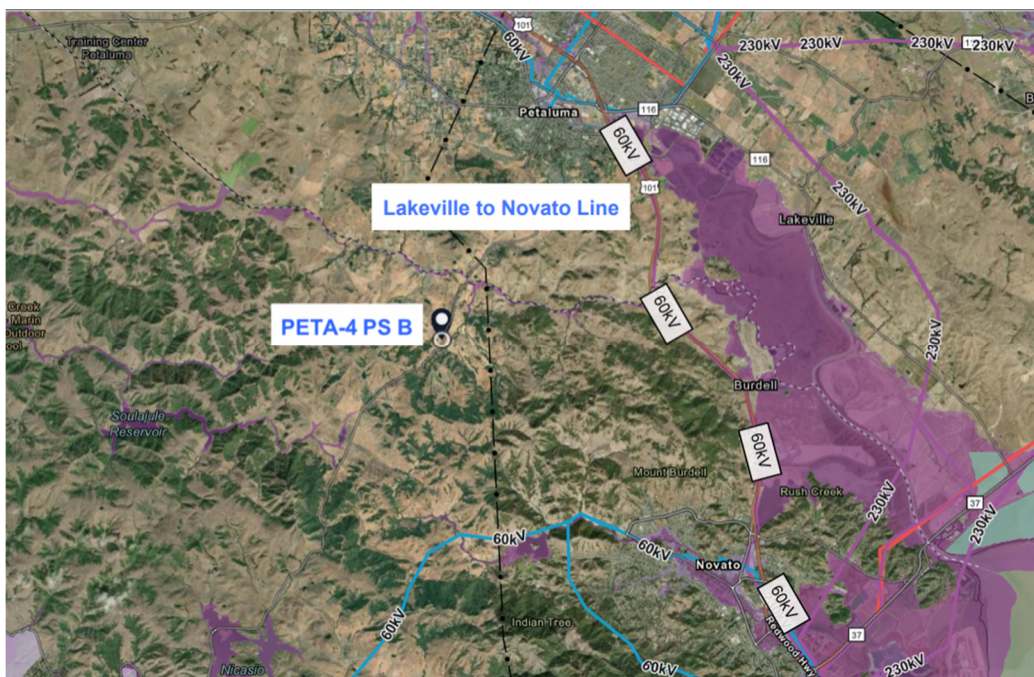
PETA 3 PS B



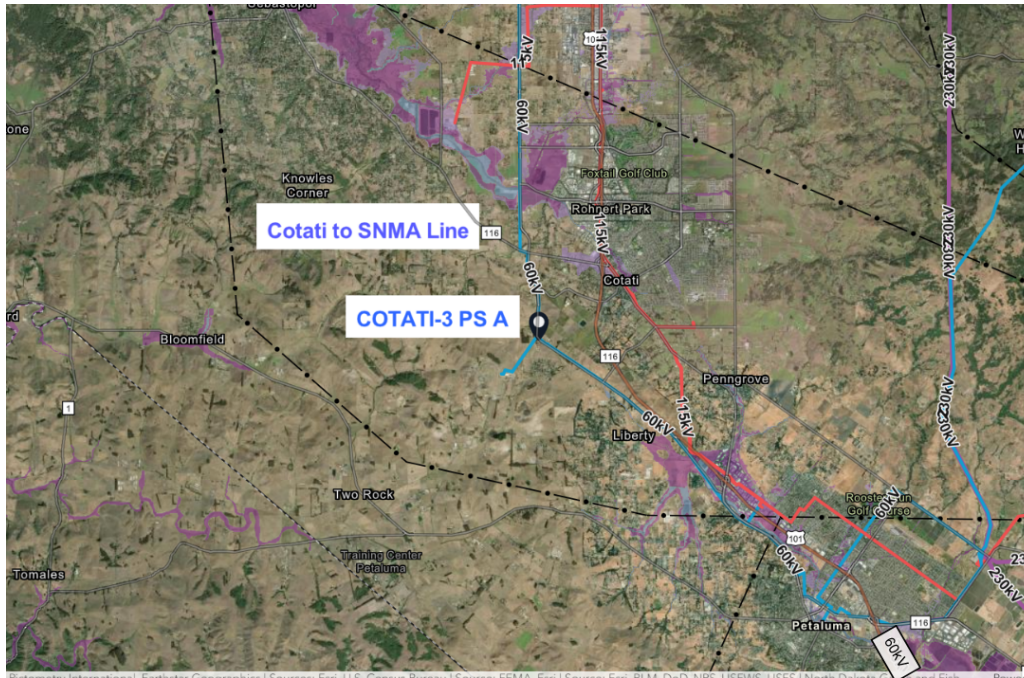
PETA-4 PS A



PETA-4 PS B



COTATI-3 PS A



COTATI-3 PS B

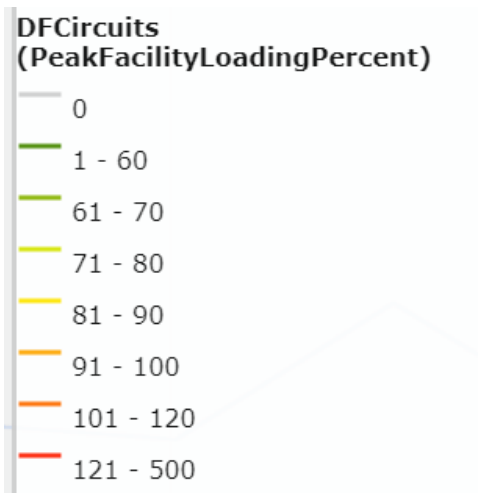


Appendix B

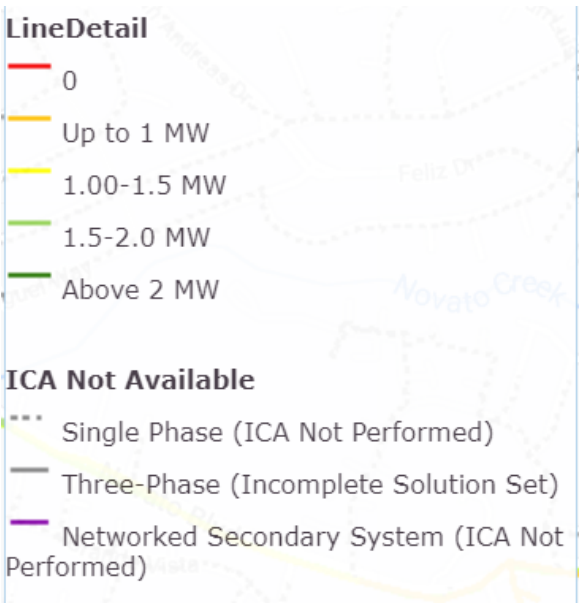
The following outputs show the pump sites located at the yellow pin in the first image with proposed lines shown by the pink lines in the satellite view. The following two images for each site represent map outputs from the ICA and DIDF maps provided by PG&E. Each map shows local distribution facilities with the lines color coded indicating available load and available interconnection capacity as green highlights. Yellow and red indicate higher loads and higher DER on the circuits or sections of the circuit.

The following legends apply to each output.

DIDF Maps



ICA Maps



PETA-3 PS A Map Output and Upgrades

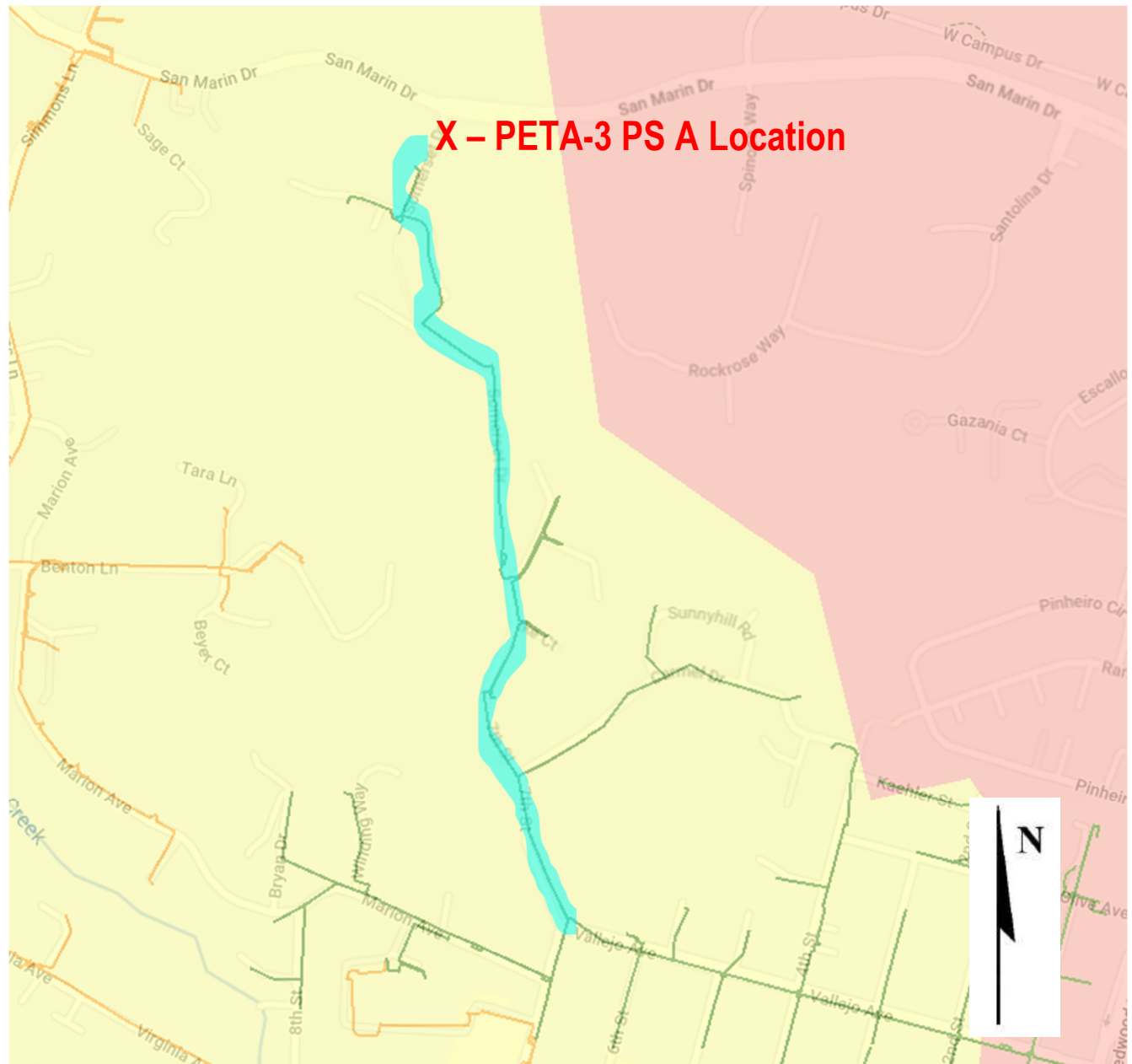
Pump Station is denoted by yellow pin.

Circuit Upgrades are denoted by pink line if applicable.



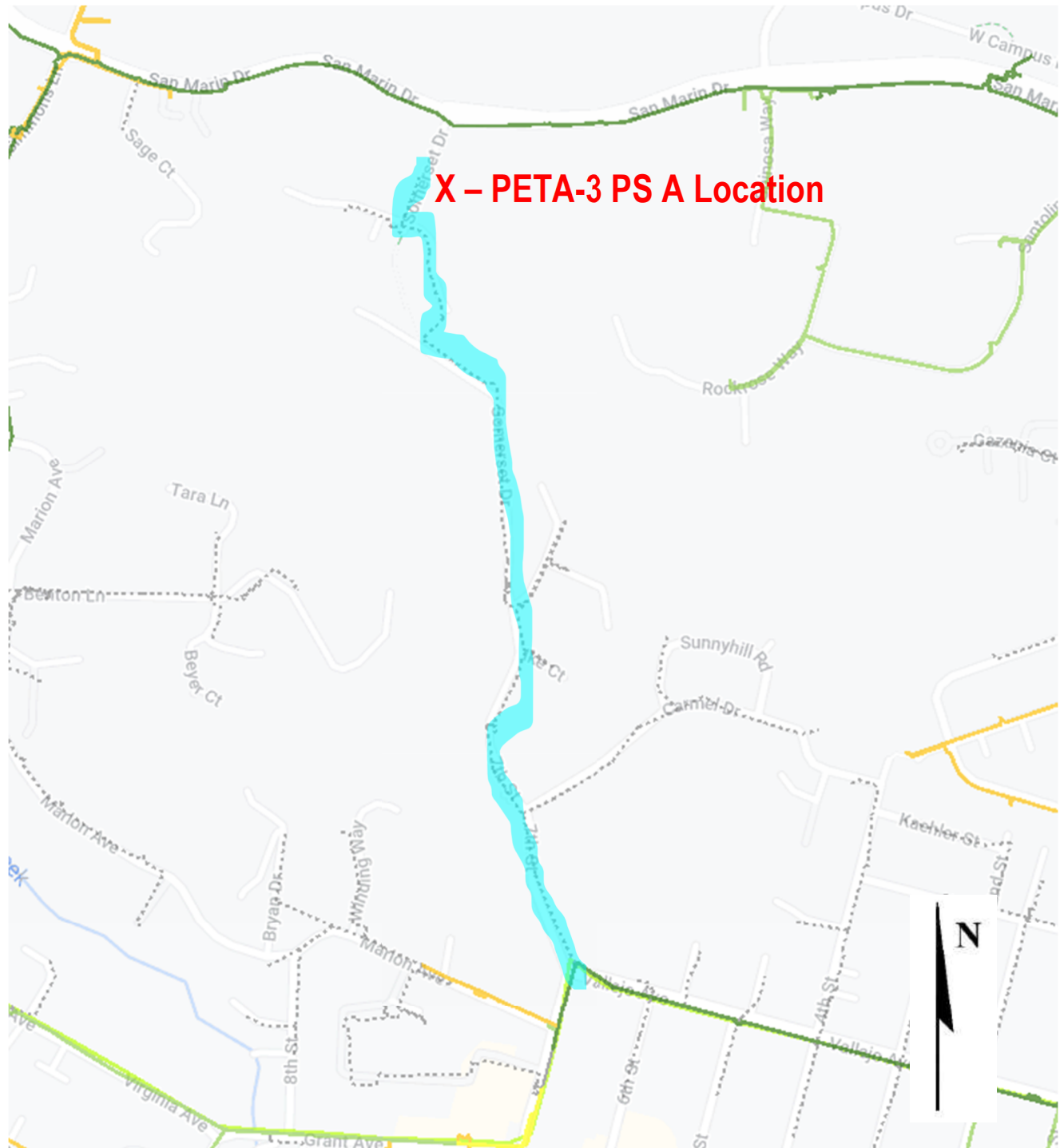
PETA-3 PS A DIDF Output

Blue highlight indicates infrastructure requiring upgrades



PETA-3 PS A ICA Output

Blue highlight indicates infrastructure requiring upgrades



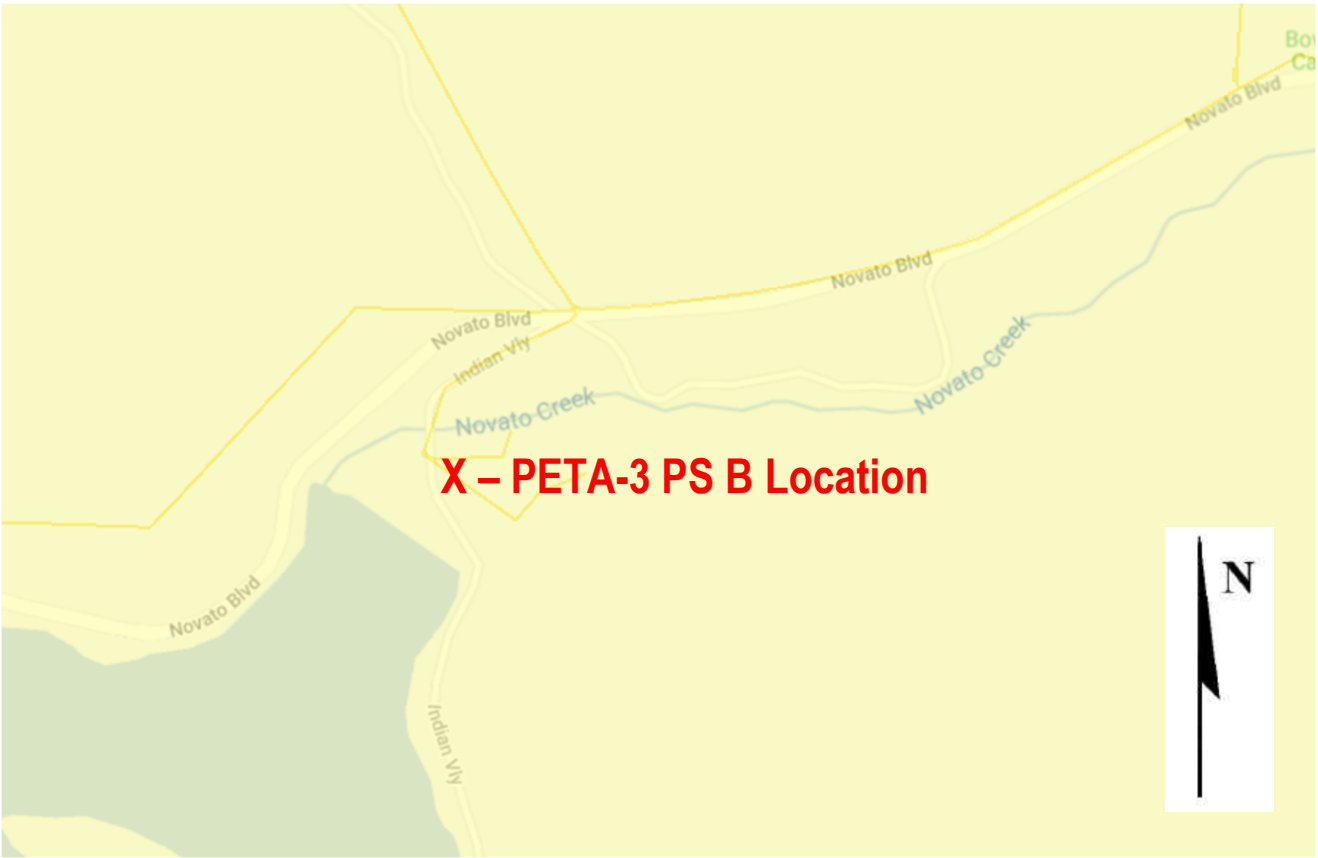
PETA-3 PS B Map Output and Upgrades

Pump Station is denoted by yellow pin.

Circuit Upgrades are denoted by pink line if applicable.



PETA-3 PS B DIDF Output



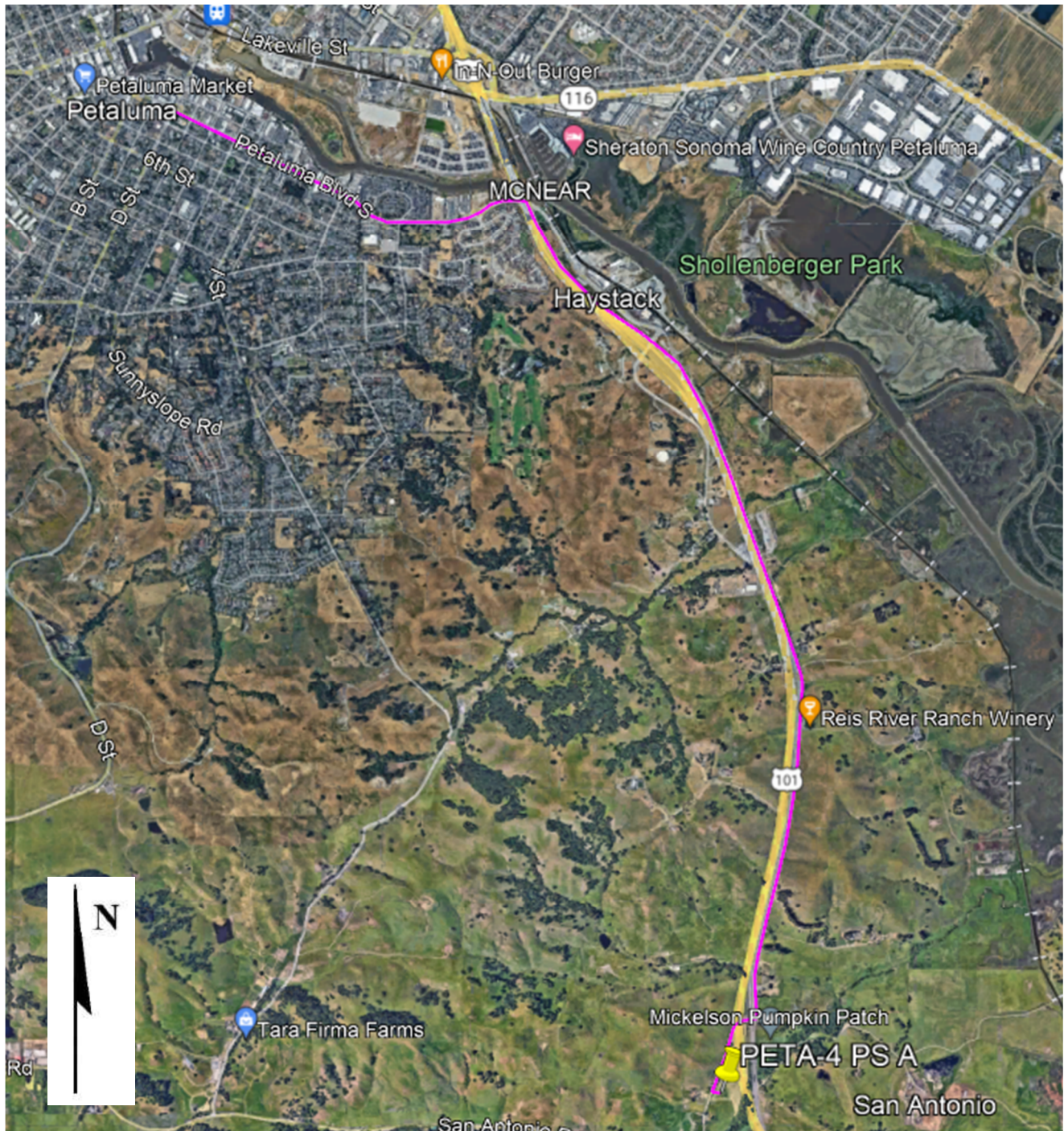
PETA-3 PS B ICA Output



PETA-4 PS A Map Output and Upgrades

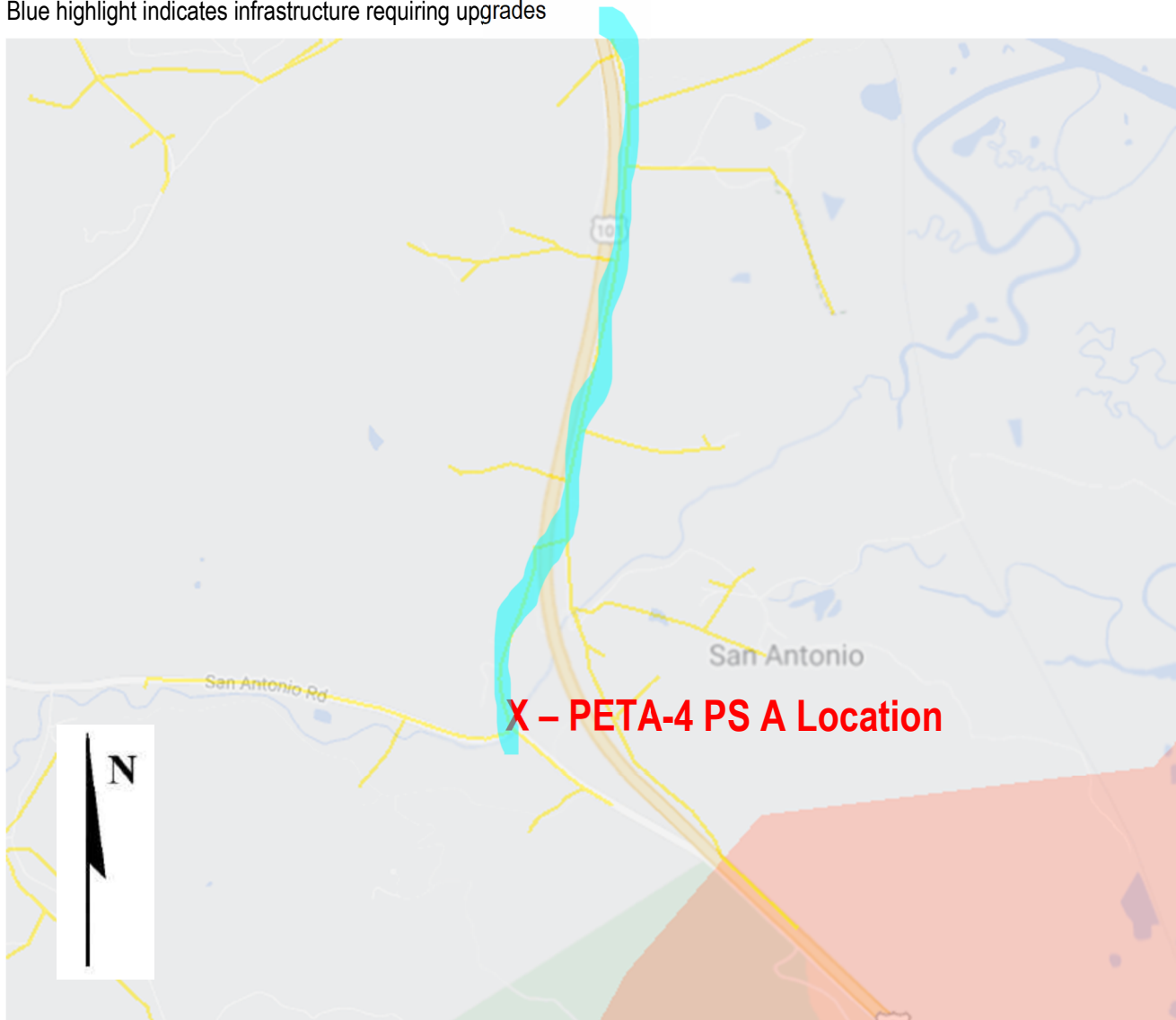
Pump Station is denoted by yellow pin.

Circuit Upgrades are denoted by pink line if applicable.



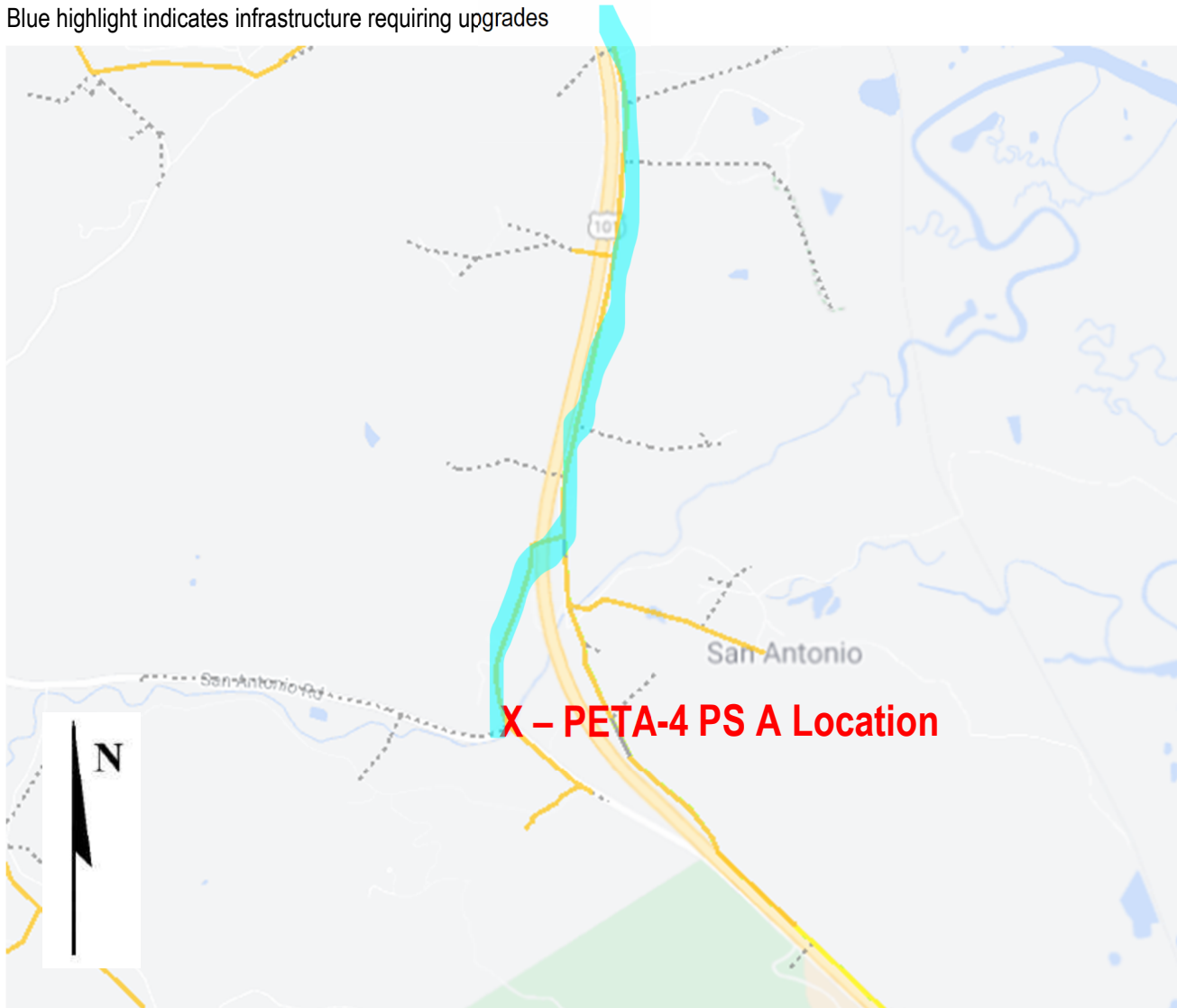
PETA-4 PS A DIDF Output

Blue highlight indicates infrastructure requiring upgrades



PETA-4 PS A ICA Output

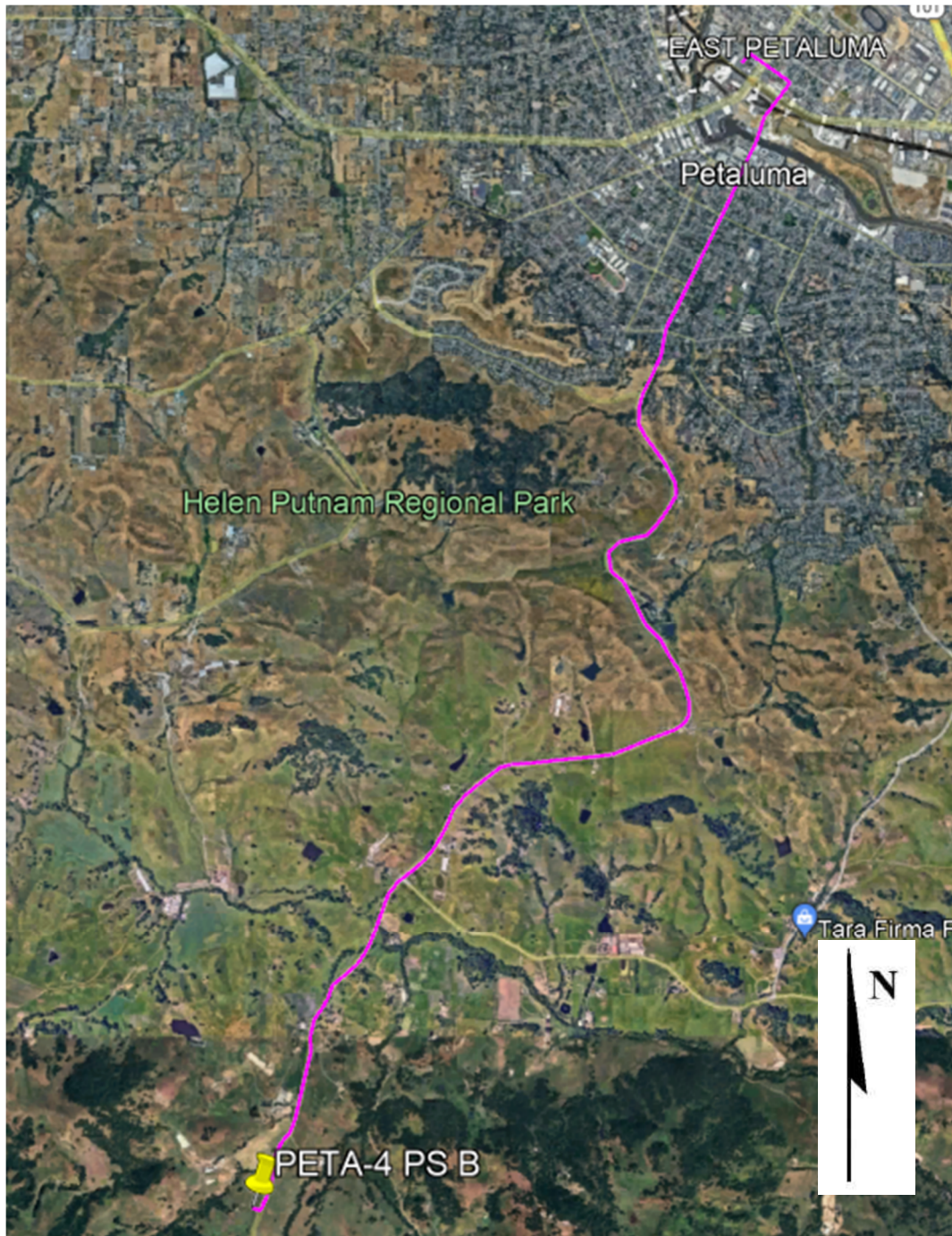
Blue highlight indicates infrastructure requiring upgrades



PETA-4 PS B Map Output and Upgrades

Pump Station is denoted by yellow pin.

Circuit Upgrades are denoted by pink line if applicable.



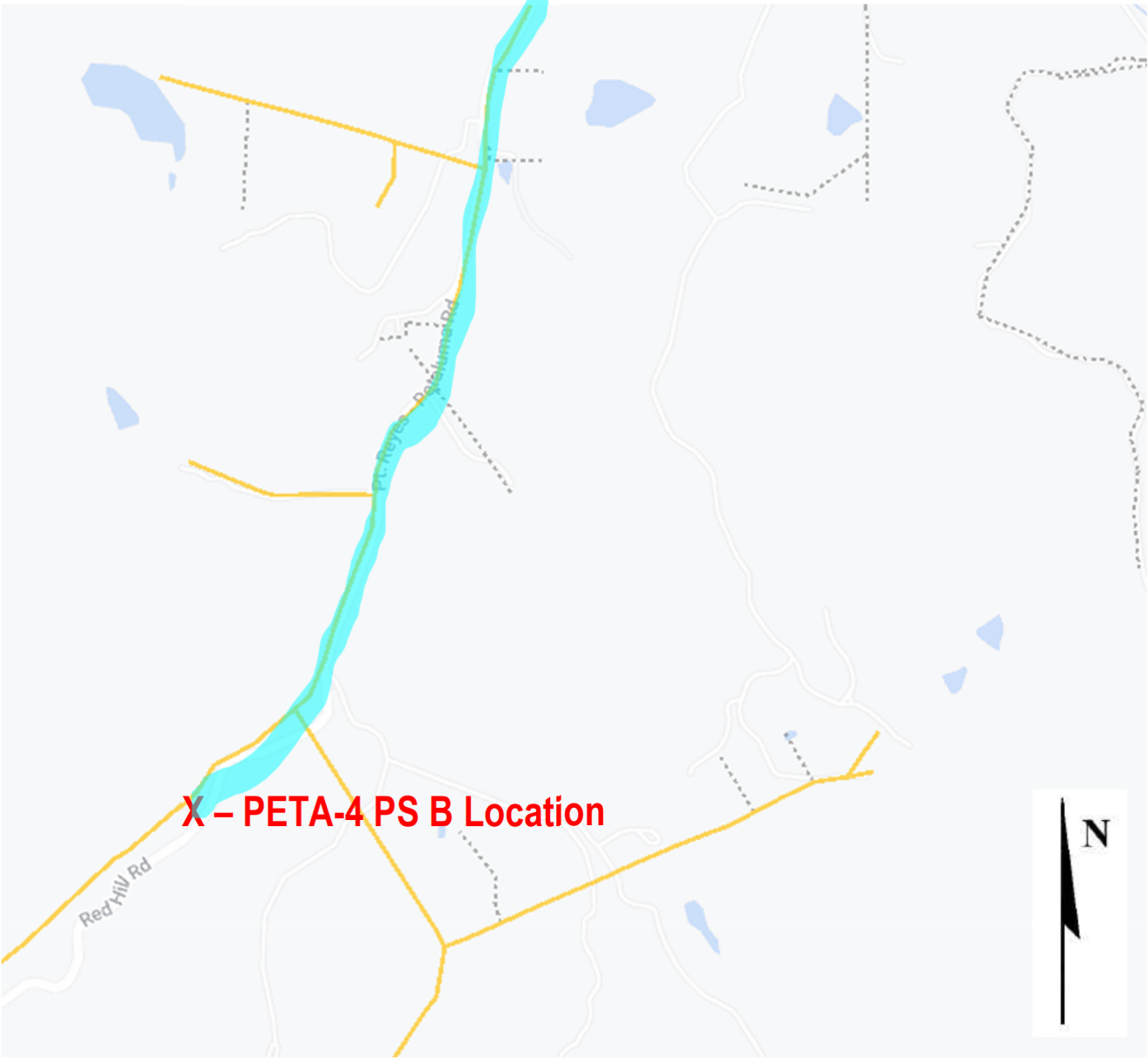
PETA-4 PS B DIDF Output

Blue highlight indicates infrastructure requiring upgrades



PETA-4 PS B ICA Output

Blue highlight indicates infrastructure requiring upgrades



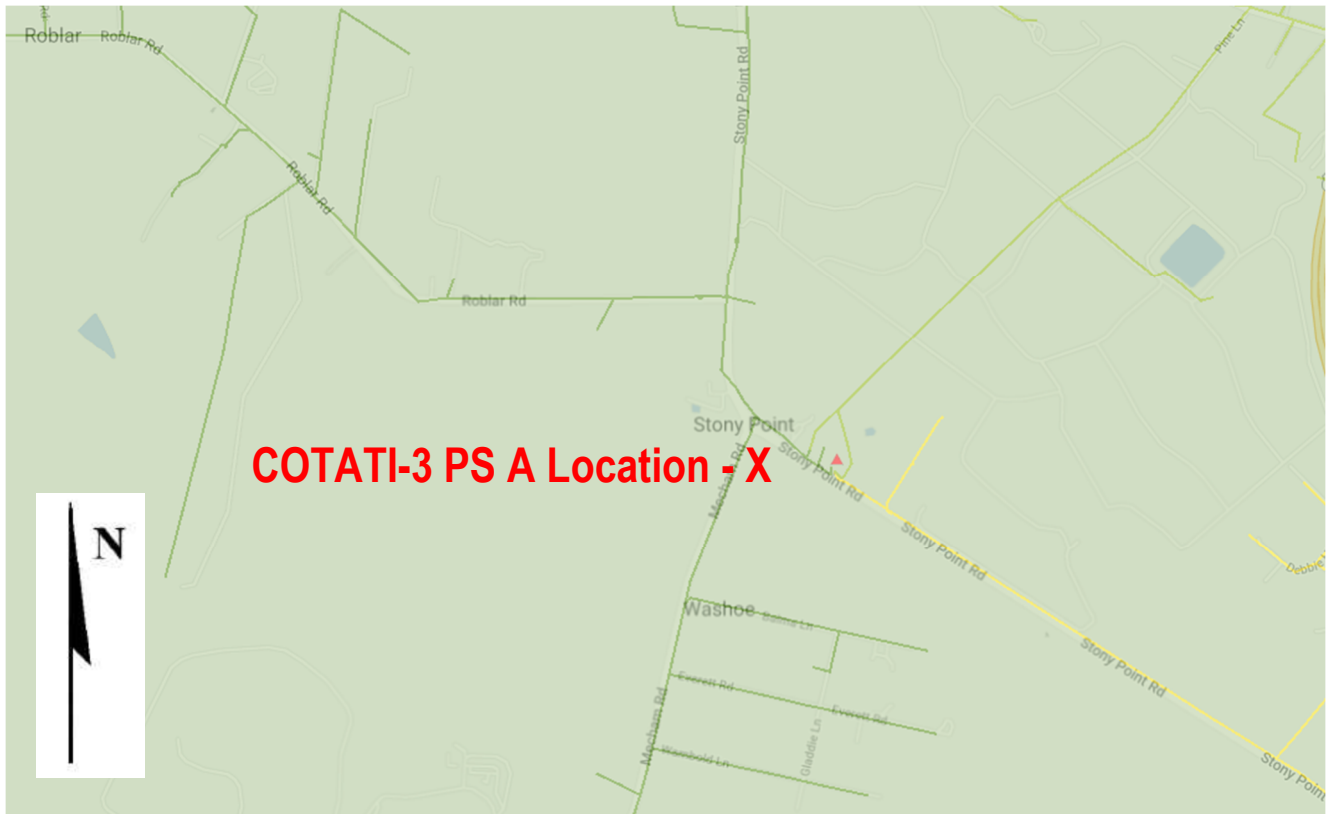
COTATI-3 PS A Map Output and Upgrades

Pump Station is denoted by yellow pin.

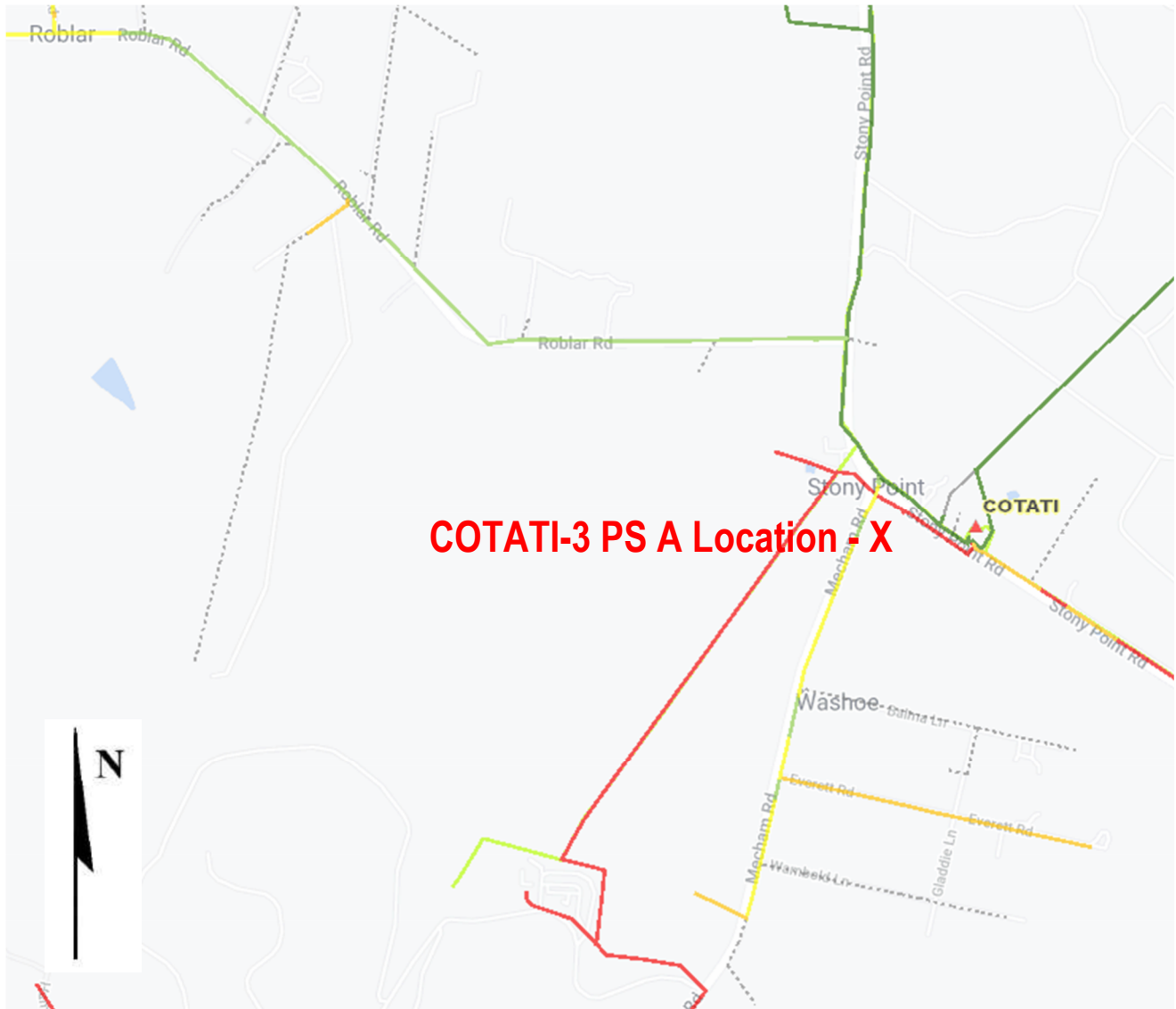
Circuit Upgrades are denoted by pink line if applicable.



COTATI-3 PS A DIDF Output



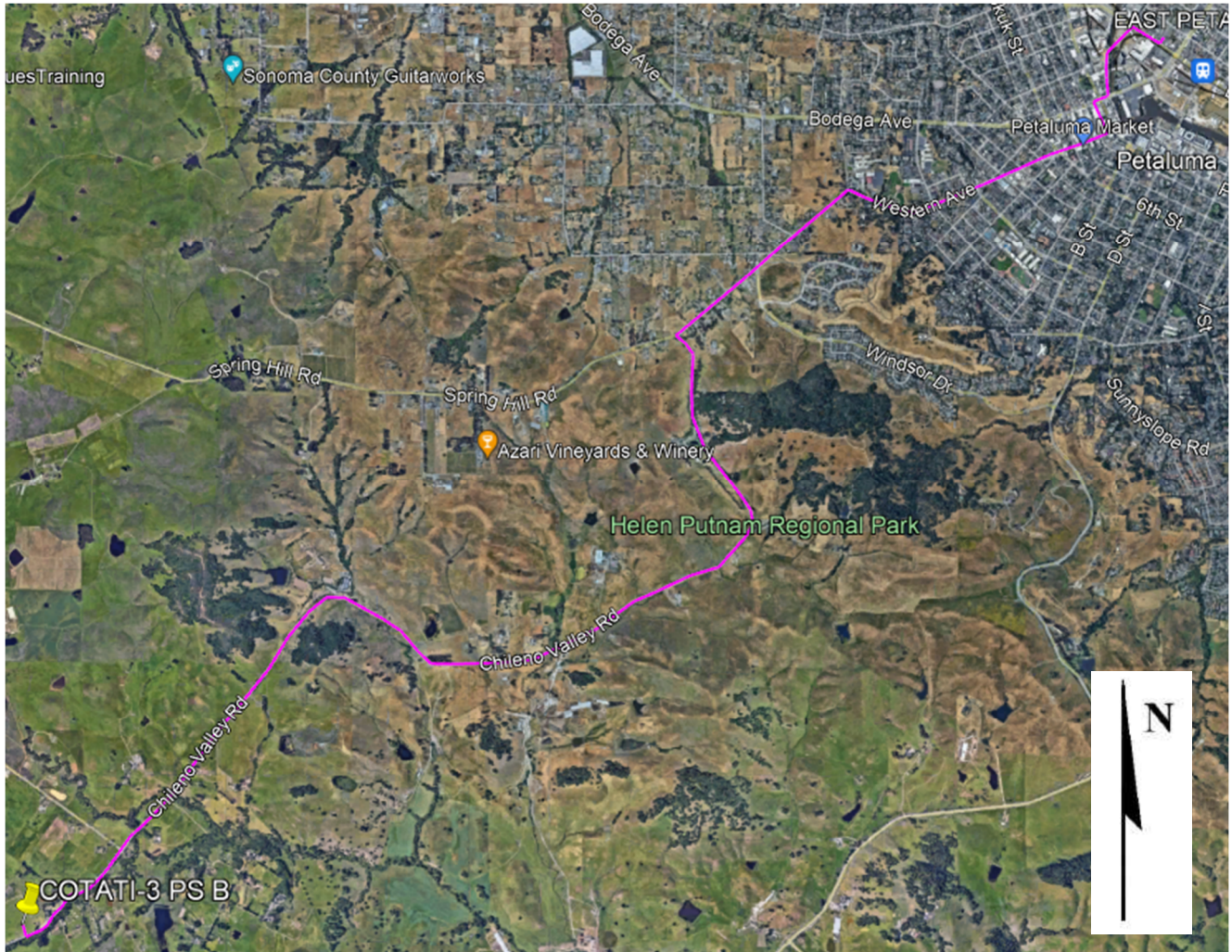
COTATI-3 PS A ICA Output



COTATI-3 PS B Map Output and Upgrades

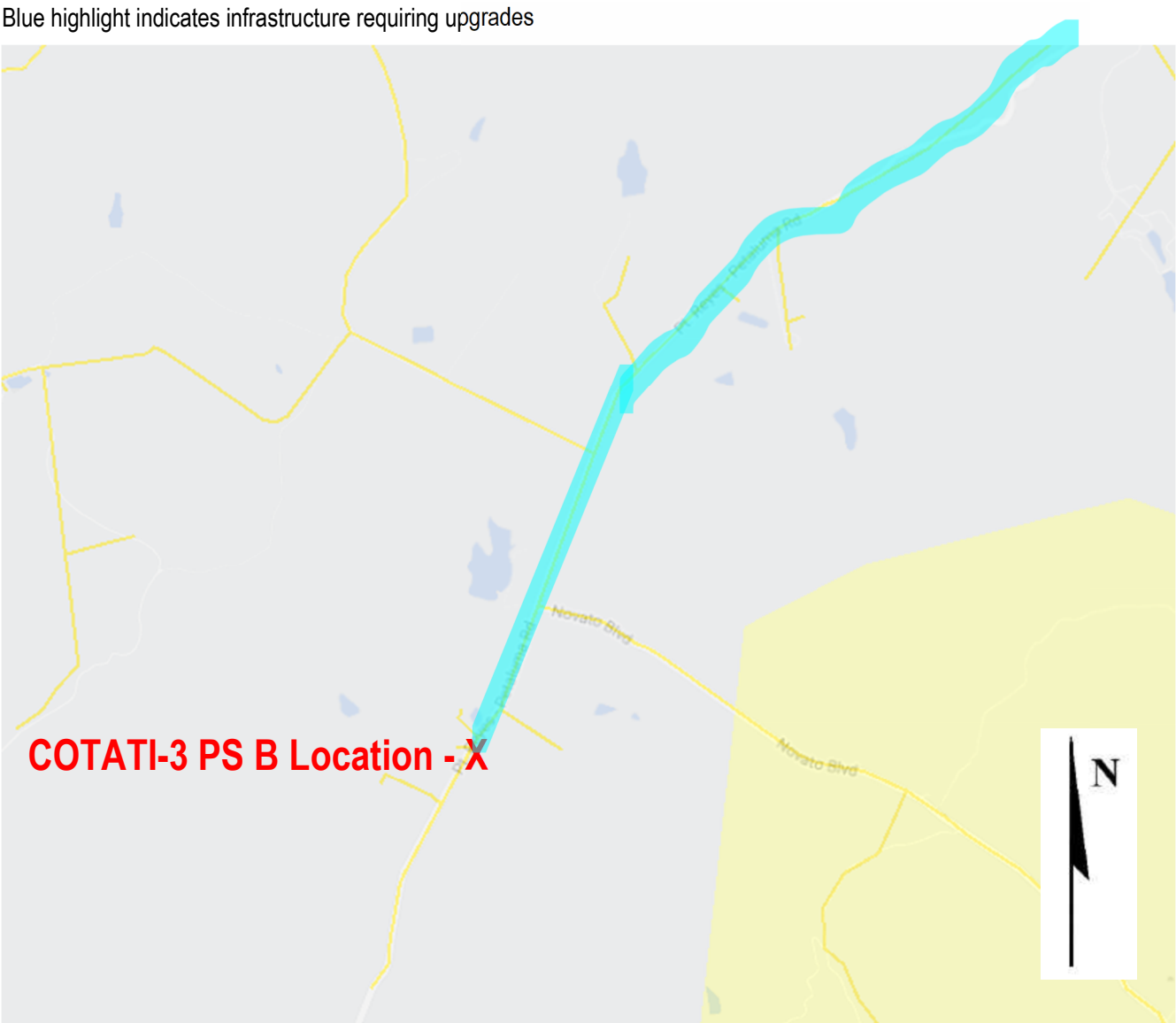
Pump Station is denoted by yellow pin.

Circuit Upgrades are denoted by pink line if applicable.



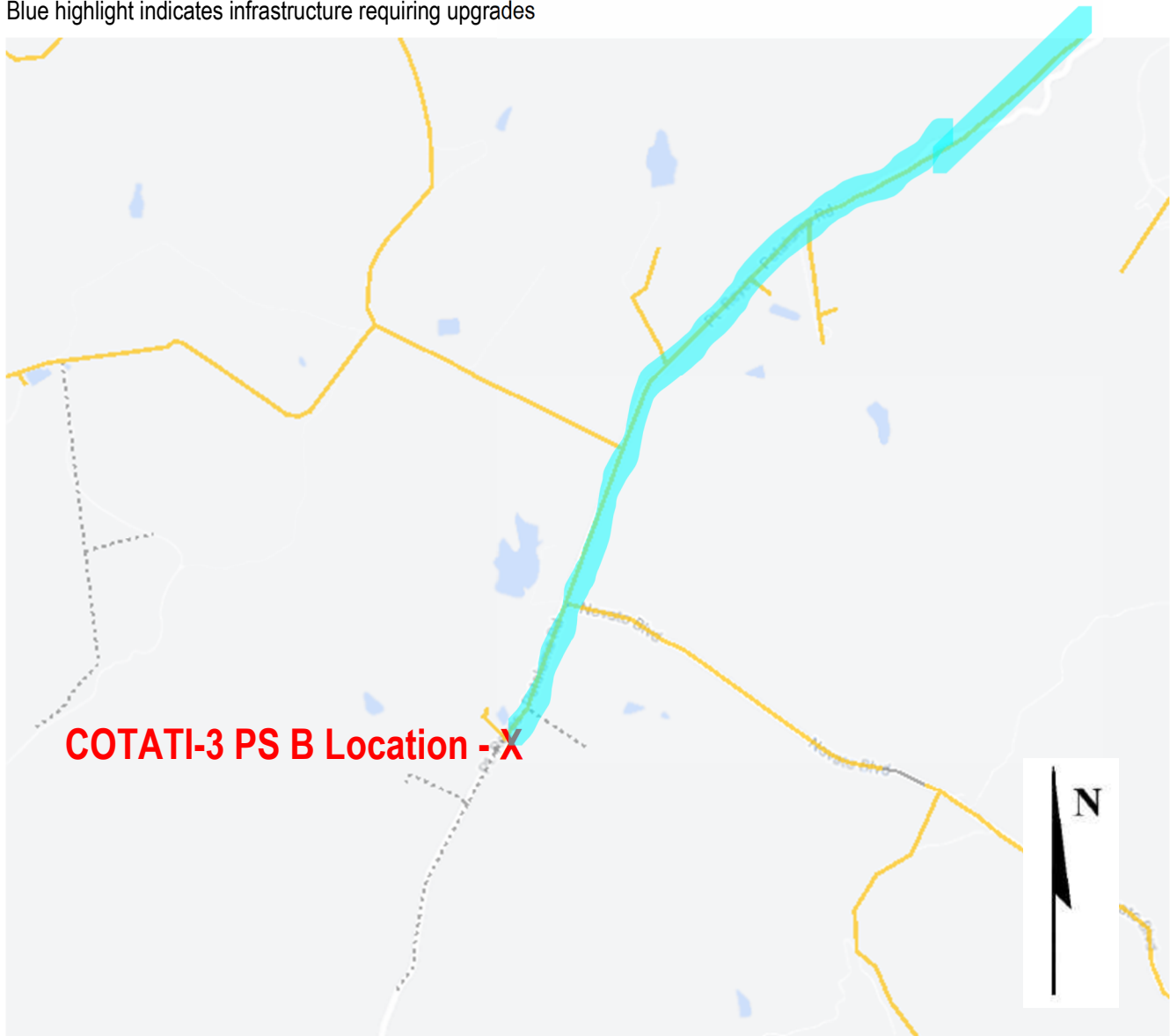
COTATI-3 PS B DIDF Output

Blue highlight indicates infrastructure requiring upgrades



COTATI-3 PS B ICA Output

Blue highlight indicates infrastructure requiring upgrades



Per Unit Pricing from PGE

Category 2 - Overhead to Underground (UG)- Set Pole and make up Cable		
Equipment	Unit Cost	Equipment
Primary UG Service up to 200ft cable	\$68,700	
Pri 350 Cable - PG&E does not separate costs for different cable size	N/A	
Pri 1000 Cable - PG&E does not separate costs for different cable size	N/A	
Category 3 - Overhead (OH) Service		
Equipment	Unit Cost	Equipment
Primary Service-OH include 1 span ovh line	\$26,700	
New Conductor extension from POI to PCC	\$165/ft (Non-Bay); \$227/ft (Bay)	
Category 5 - Metering		
Equipment	Unit Cost	Equipment
Secondary Service Metering	\$5,150	
Primary Service Metering	\$24,689	
Category 6 - Telemetry		
Equipment	Unit Cost	Equipment
Overhead SCADA Recloser	\$99,400	
Underground SCADA Switch	\$188,000	
N/A	N/A	
Mini Remote Terminal Unit	\$60,000	
Bi-directional watt transducer - Not generally used at PG&E	N/A	
Data Point addition and existing HMI - Not generally used at PG&E	N/A	
Overhead Remote Control Switch - Not generally used at PG&E	N/A	
Category 7 - System Equipment		
Equipment	Unit Cost	Equipment
New overhead Air Switch	\$31,000	
New Capacitor OH	\$47,000	
PME 5 Padmount Switch	\$67,341	
New Capacitor Padmounted	\$81,098	
New Regulator- Close Delta	\$222,000	
33kV Regulator 3-690/722 - Not generally used at PG&E	N/A	
New Voltage Regulator 600A Padmount with two switches - Not generally used at	N/A	
Grounding/Stabilizing Transformer- Pole Mounted	\$24,200	

Per Unit Pricing Continued

Grounding/Stabilizing Transformer- Padmounted	\$52,000	
Conductor (Per feet) - Overhead-Urban	\$227/ft (Bay cost)	
Reconductor (Per feet) - Overhead-Rural	\$165/ft (Non-Bay cost)	
Reconductor (Per feet) - UG	\$278/ft (Non-Bay); \$345/ft (Bay)	
New Steel Pole (not priced separately, would be part of facility supporting need for	N/A	
New Wooden Pole (not priced separately, would be part of facility supporting	N/A	
Overhead Fuses	\$15,500	
Fuse Cabinet UG 3 phase - Not generally used at PG&E	.	
Relocate Capacitor Bank	\$18,450	
Regulator Control settings modifications	\$2,575	
Relocate Regulator	\$51,500	
Add a third Regulator to close the Delta	\$56,500	
New Regulator - Closed Delta	\$222,000	
Reclose blocking	\$149,350	
Hardwire Tripping from Transformer Hi-side	\$100,000	
Substation LTC Control change out	\$61,800	
New IPAC relay cabinet for bi-direction power flow	\$128,750	
Direct Transfer Trip	\$618,000	
New Substation Circuit Breaker - Not generally used for distribution	N/A	
New 28MVA 69/12kV Transformer - Not generally used for distribution	N/A	
New 28MVA 138/12kV Transformer - Not generally used for distribution	N/A	

APPENDIX 3D

HYDRAULIC ANALYSIS RESULTS

Hydraulic Modeling

Water Supply Conveyance Improvements

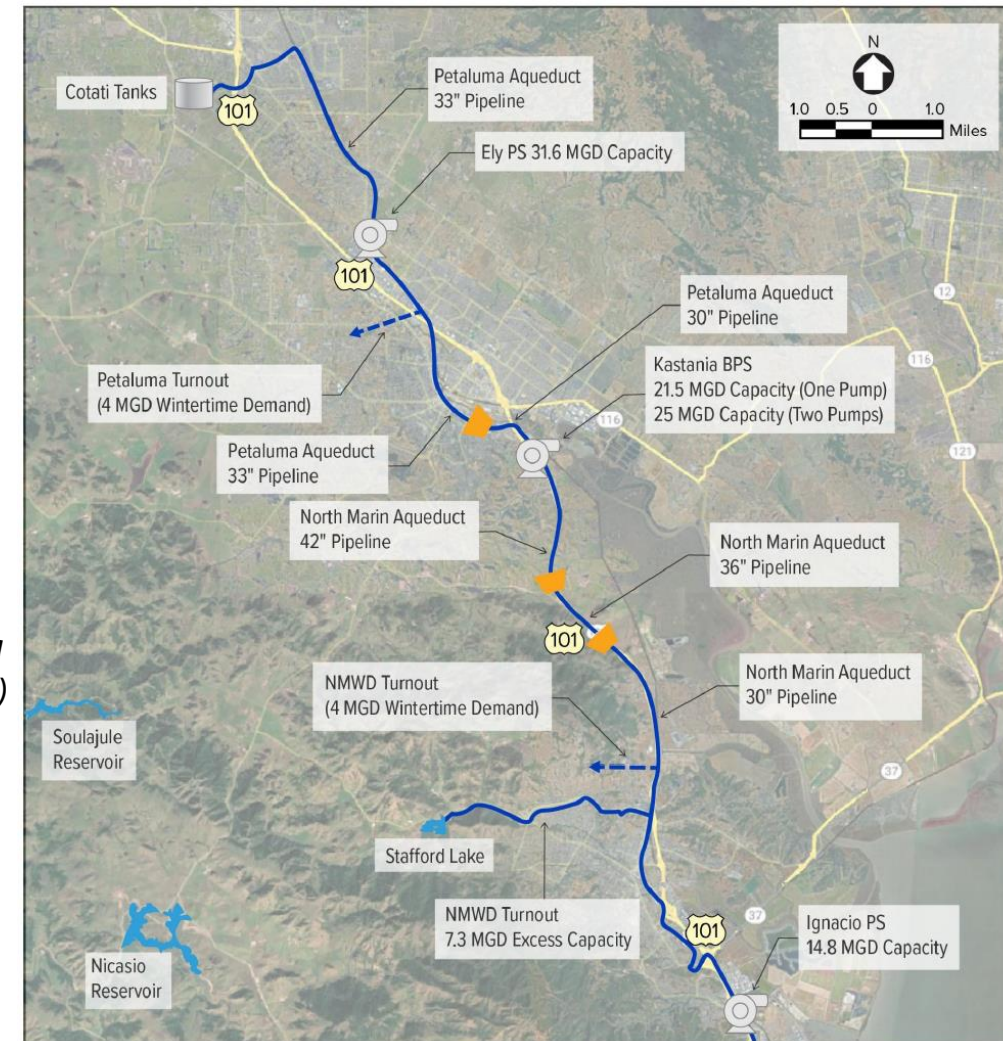
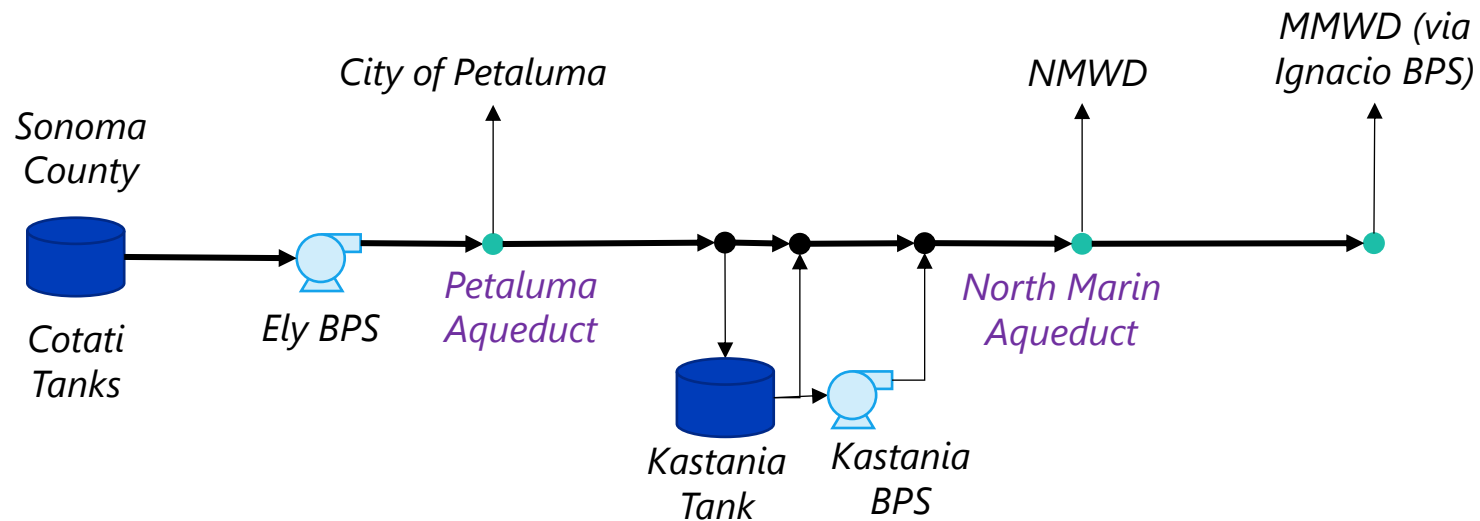
Outline

1. Aqueduct system overview
2. Model assumptions and previous analysis
 - a. Historical water delivery data
 - b. Previous model scenarios and results
3. Evaluation of proposed turnout locations
4. Hydraulic analysis of short-listed alternatives

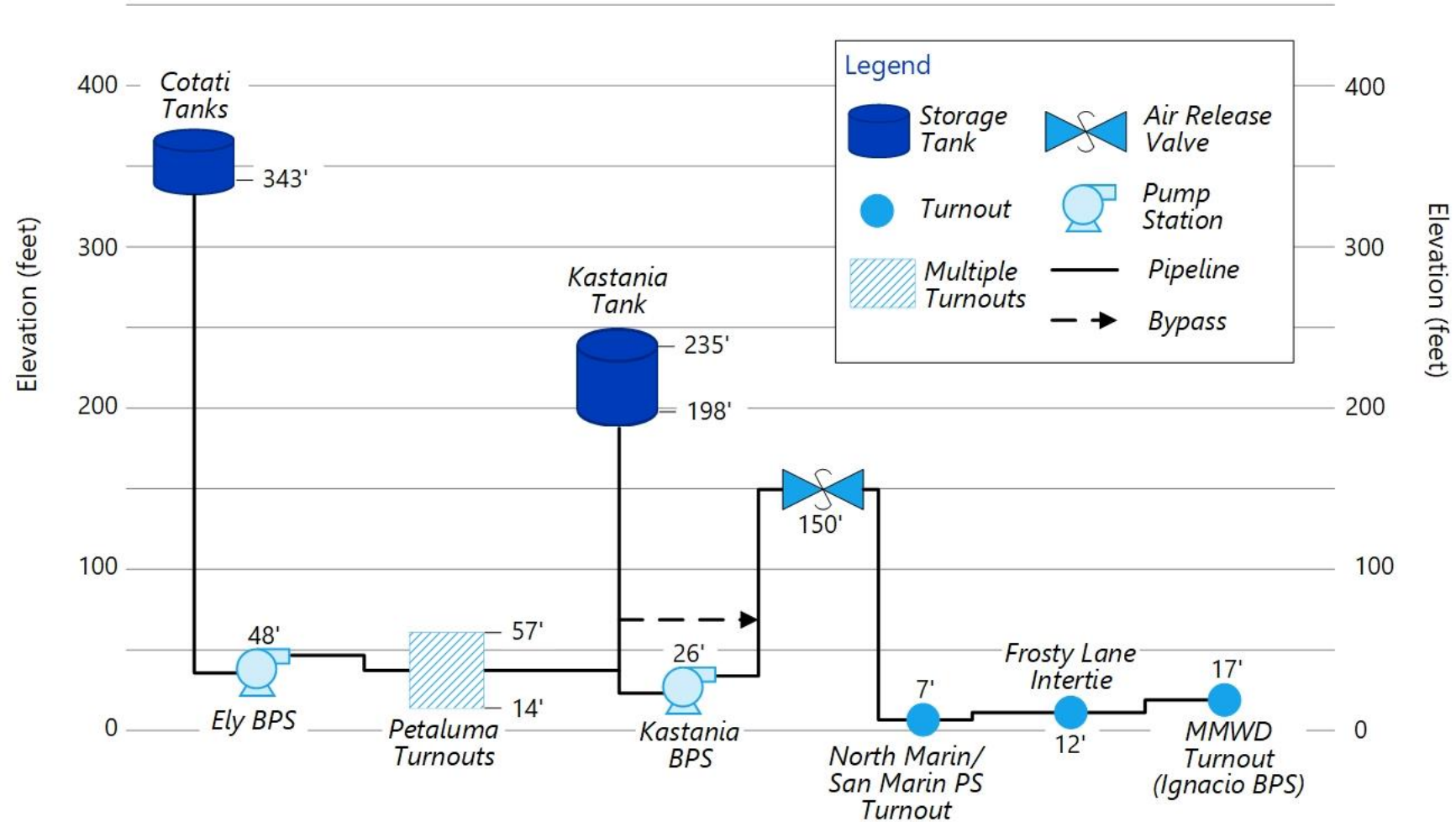
01

Aqueduct System Overview

The Petaluma/North Marin Aqueduct System conveys treated surface water from the Cotati Tanks in Sonoma County to North Marin



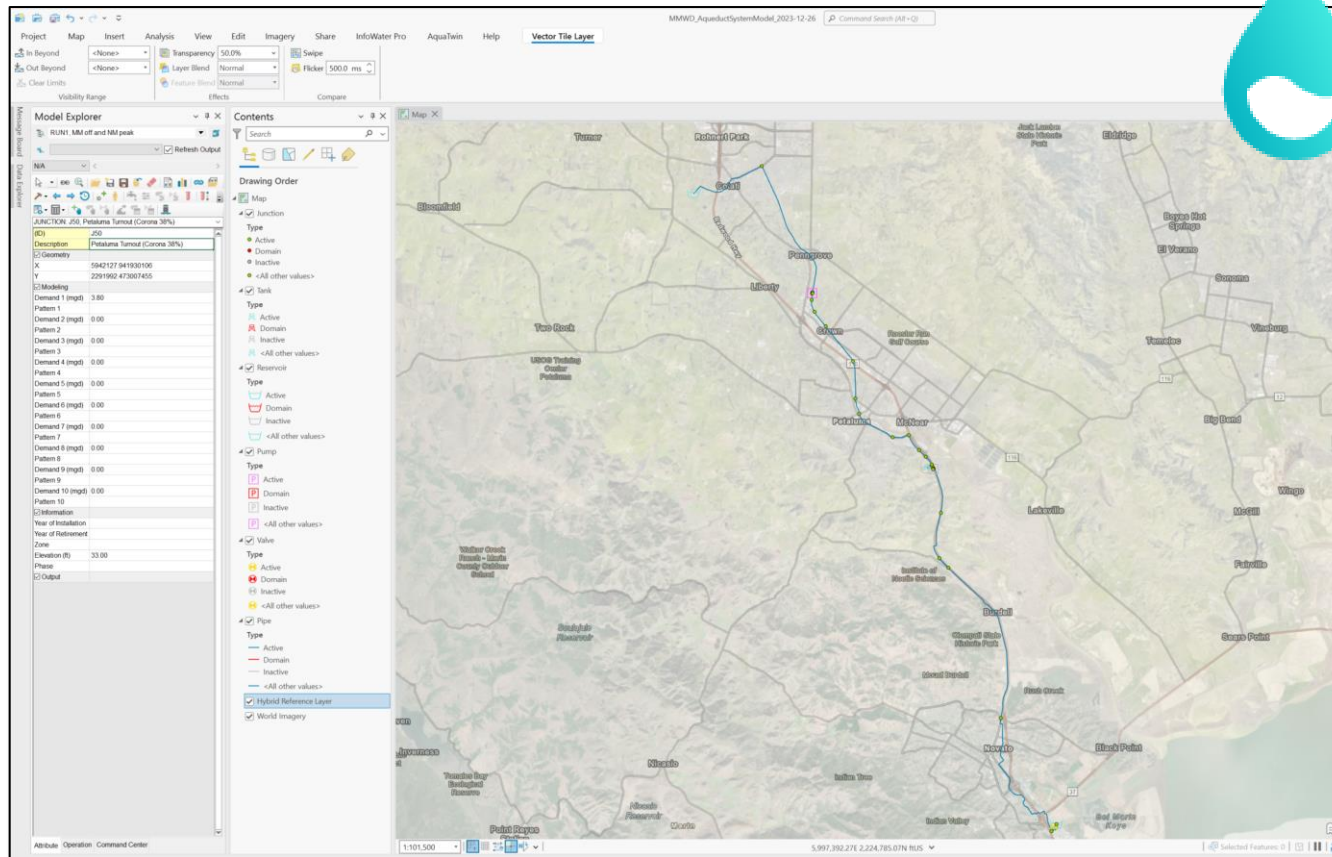
The Aqueduct System ranges from about 7 to 350 feet in elevation



02

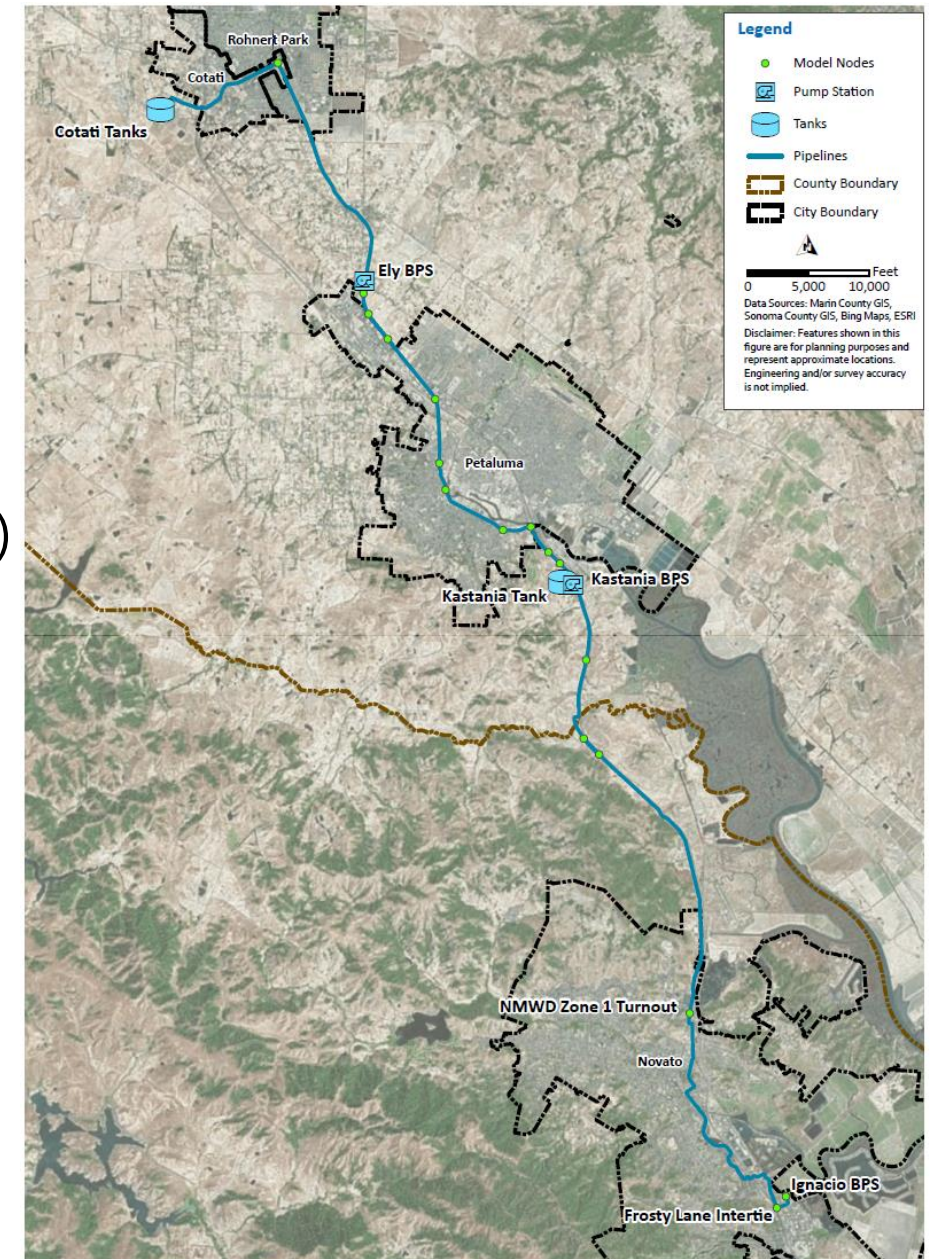
Model Assumptions and Previous Analysis

The Petaluma/North Marin Aqueduct System hydraulic model was developed in 2021 using Innovyze Inc.'s InfoWater Pro software

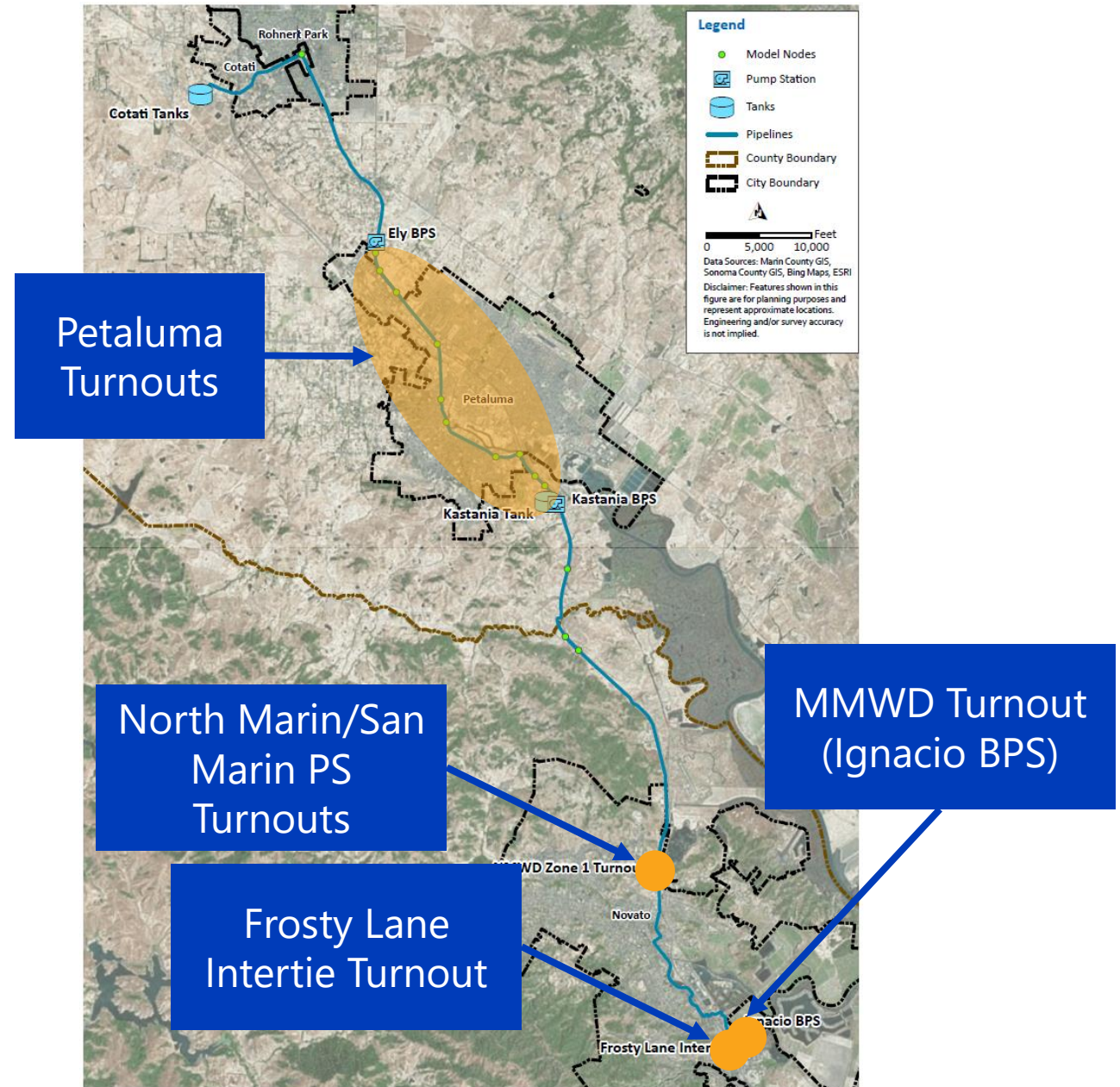


The hydraulic model consists of the aqueduct's main hydraulic components

- Storage tanks
 - » Cotati tanks (simulated as fixed-head reservoir)
 - » Kastania tank
- Pump stations
 - » Ely BPS
 - » Kastania BPS
- Turnouts
 - » Petaluma (Turnouts 1 and 2, Corona, Dynamic, Payran, McNear, South)
 - » North Marin/San Marin PS
 - » Frosty Lane Intertie
 - » Ignacio BPS (MMWD)



Turnouts are modeled as junctions with fixed demands

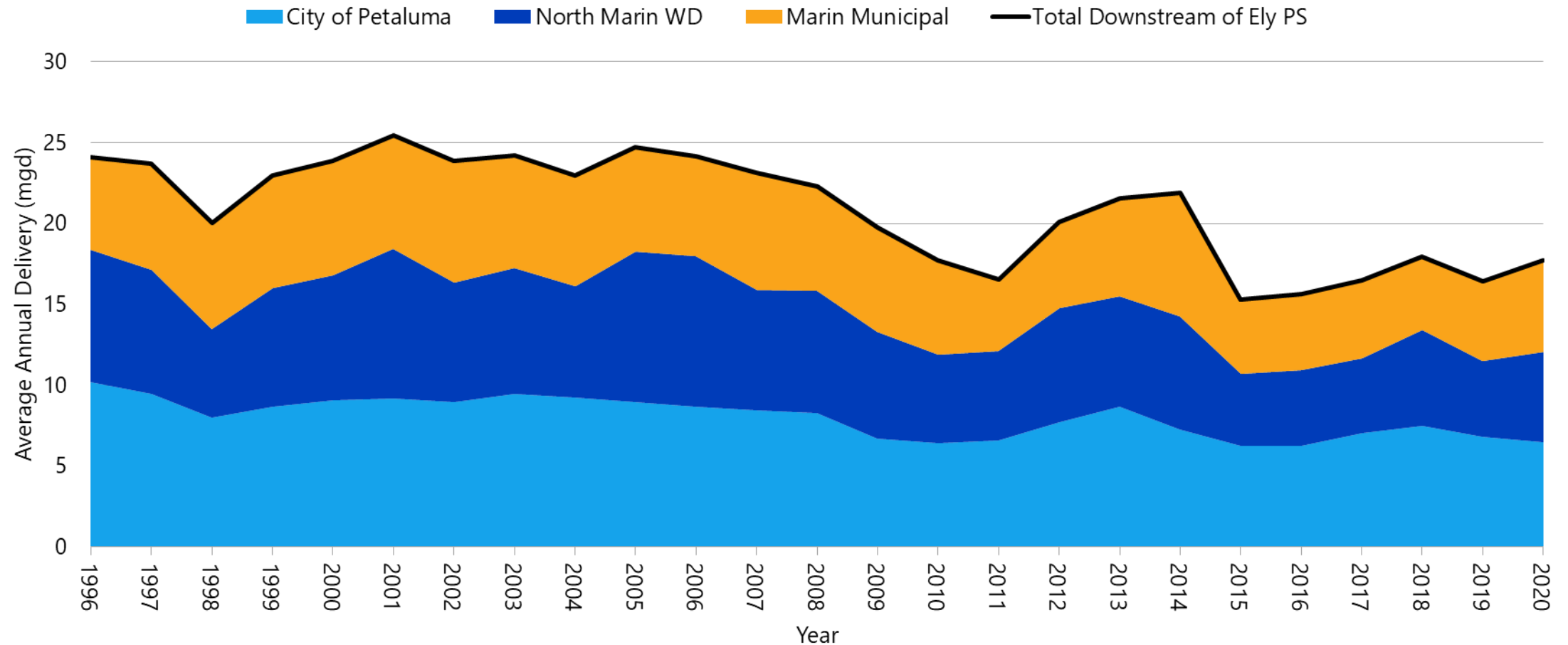


Historical Water Delivery Data

Historical water delivery data – annual averages in mgd

Year	Petaluma	NMWD	MMWD	Total
1996	7.97	8.16	5.66	24.09
1997	9.46	7.68	6.54	23.68
1998	7.97	5.48	6.60	20.06
1999	8.65	7.37	6.95	22.97
2000	9.05	7.71	7.08	23.85
2001	9.19	9.22	7.05	25.46
2002	8.94	7.39	7.52	23.85
2003	9.44	7.78	6.96	24.18
2004	9.20	6.88	6.90	22.98
2005	8.95	9.29	6.50	24.74
2006	8.64	9.33	6.16	24.14
2007	8.42	7.43	7.27	23.13
2008	8.29	7.54	6.48	22.30
2009	6.68	6.62	6.42	19.73
2010	6.37	5.52	5.84	17.73
2011	6.57	5.52	4.46	16.55
2012	7.68	7.09	5.31	20.08
2013	8.63	6.86	6.07	21.56
2014	7.26	6.98	7.64	21.89
2015	6.26	4.46	4.57	15.29
2016	6.23	4.71	4.66	15.60
2017	7.01	4.63	4.85	16.49
2018	7.49	5.92	4.57	17.97
2019	6.82	4.68	4.93	16.42
2020	6.46	5.57	5.68	17.70
5-year Average	6.80	5.10	4.94	16.84
10-year Average	7.04	5.64	5.27	17.96

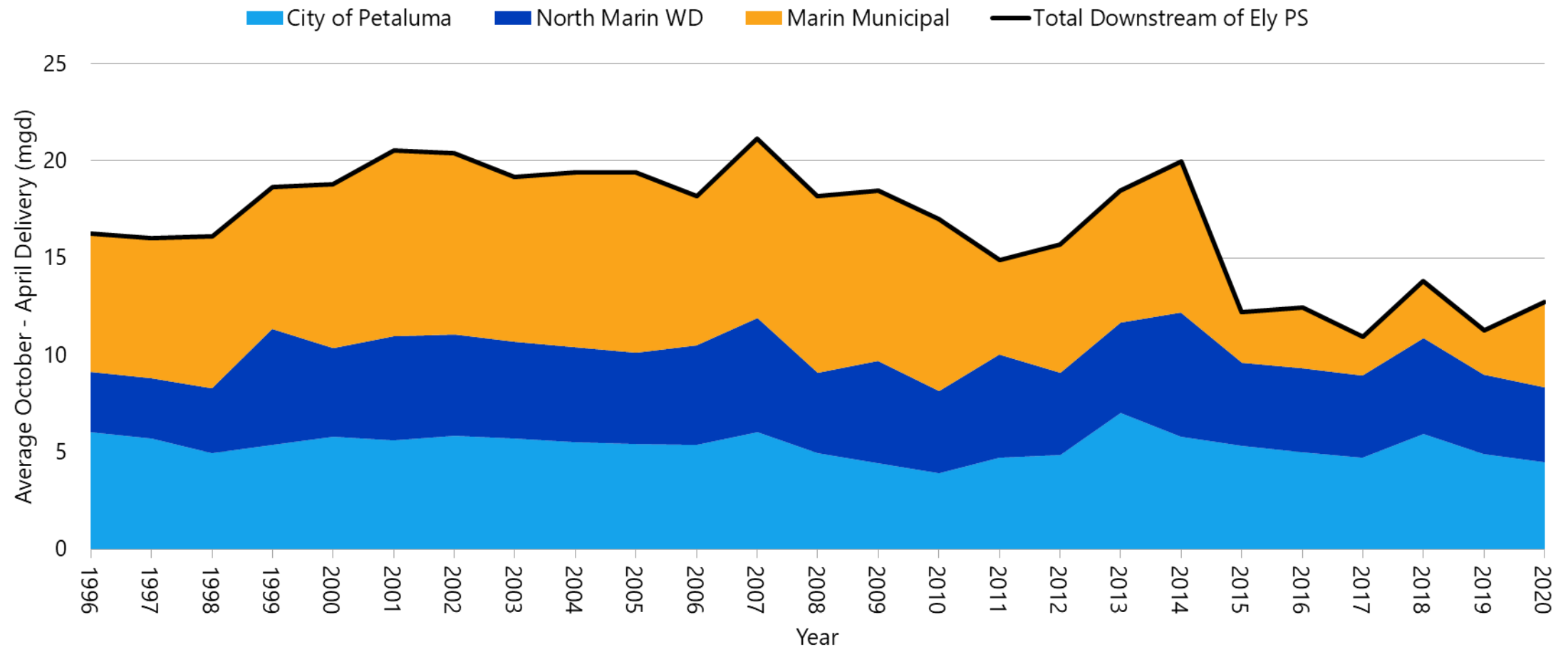
Historical water delivery data – annual averages in mgd



Historical water delivery data – Dec - Feb averages in mgd

Year	Petaluma	NMWD	MMWD	Total
1996	6.00	3.14	7.11	16.26
1997	5.68	3.11	7.23	16.03
1998	4.96	3.33	7.83	16.12
1999	5.36	5.99	7.30	18.64
2000	5.79	4.56	8.45	18.79
2001	5.61	5.35	9.58	20.54
2002	5.81	5.22	9.35	20.38
2003	5.69	4.97	8.51	19.17
2004	5.51	4.89	8.99	19.40
2005	5.39	4.73	9.26	19.38
2006	5.35	5.14	7.68	18.17
2007	6.00	5.90	9.24	21.14
2008	4.92	4.14	9.10	18.16
2009	4.44	5.23	8.77	18.44
2010	3.88	4.27	8.86	17.02
2011	4.72	5.32	4.83	14.87
2012	4.86	4.21	6.61	15.68
2013	7.03	4.64	6.78	18.45
2014	5.79	6.41	7.78	19.98
2015	5.32	4.29	2.59	12.20
2016	4.98	4.33	3.13	12.44
2017	4.72	4.20	2.01	10.93
2018	5.92	4.93	2.96	13.82
2019	4.91	4.07	2.30	11.28
2020	4.45	3.88	4.38	12.71
5-year Average	5.00	4.28	2.96	12.23
10-year Average	5.27	4.63	4.34	14.24

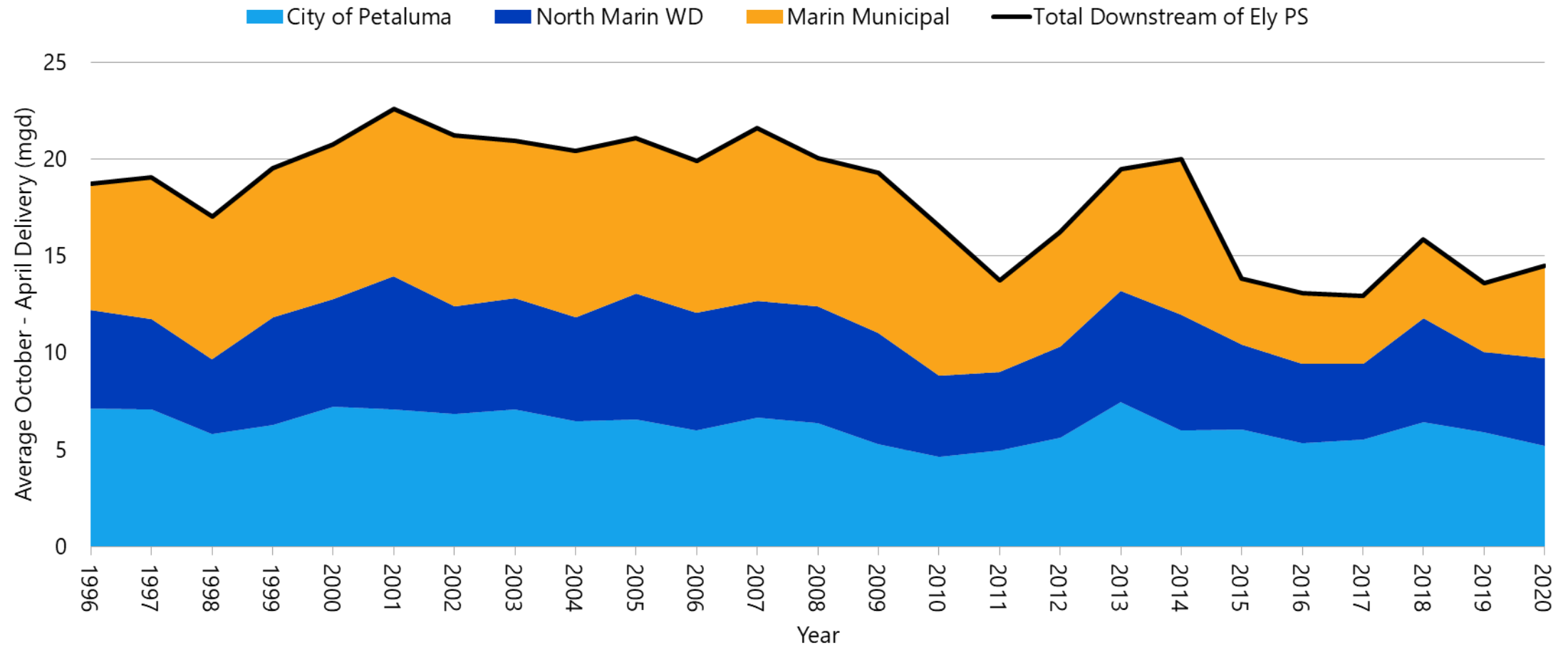
Historical water delivery data – Dec - Feb averages in mgd



Historical water delivery data – October - April averages

Year	Petaluma	NMWD	MMWD	Total
1996	7.12	5.12	6.38	18.74
1997	7.10	4.66	7.29	19.05
1998	5.83	3.86	7.36	17.05
1999	6.29	5.57	7.67	19.53
2000	7.25	5.55	7.95	20.74
2001	7.07	6.87	8.67	22.61
2002	6.86	5.53	8.83	21.22
2003	7.08	5.75	8.14	20.96
2004	6.47	5.36	8.60	20.43
2005	6.56	6.50	8.01	21.07
2006	6.02	6.04	7.83	19.90
2007	6.66	6.03	8.88	21.58
2008	6.38	6.03	7.63	20.04
2009	5.28	5.76	8.28	19.31
2010	4.66	4.17	7.73	16.55
2011	4.98	4.06	4.70	13.74
2012	5.61	4.72	5.90	16.23
2013	7.48	5.73	6.28	19.49
2014	6.00	5.98	8.03	20.00
2015	6.05	4.35	3.42	13.83
2016	5.36	4.08	3.65	13.09
2017	5.52	3.93	3.49	12.94
2018	6.43	5.35	4.10	15.87
2019	5.91	4.12	3.59	13.62
2020	5.18	4.53	4.78	14.50
5-year Average	5.68	4.40	3.92	14.00
10-year Average	5.85	4.68	4.79	15.33

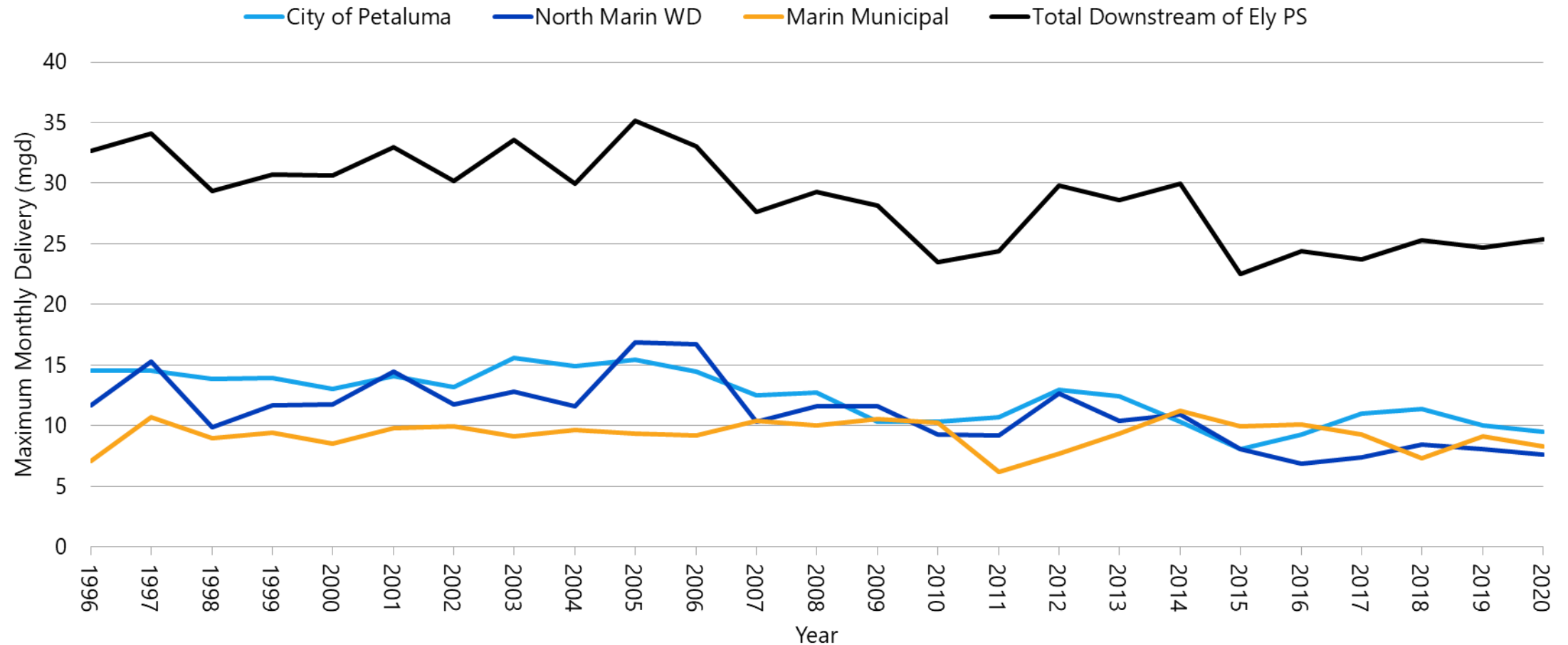
Historical water delivery data – October - April averages in mgd



Historical water delivery data – maximum monthly in mgd

Year	Petaluma	NMWD	MMWD	Total
1996	14.52	11.68	7.11	32.68
1997	14.52	15.28	10.66	34.07
1998	13.85	9.88	8.97	29.38
1999	13.96	11.67	9.41	30.73
2000	13.01	11.73	8.48	30.61
2001	14.12	14.44	9.76	32.95
2002	13.14	11.78	9.93	30.17
2003	15.56	12.81	9.12	33.59
2004	14.90	11.58	9.61	29.93
2005	15.46	16.89	9.35	35.15
2006	14.44	16.74	9.17	33.05
2007	12.53	10.31	10.39	27.62
2008	12.73	11.57	9.98	29.26
2009	10.29	11.56	10.57	28.13
2010	10.30	9.25	10.22	23.48
2011	10.72	9.19	6.22	24.39
2012	12.96	12.61	7.66	29.82
2013	12.44	10.42	9.32	28.60
2014	10.34	10.95	11.25	29.97
2015	8.08	8.04	9.95	22.51
2016	9.26	6.88	10.07	24.36
2017	10.97	7.40	9.29	23.71
2018	11.39	8.47	7.31	25.32
2019	10.01	8.05	9.14	24.70
2020	9.50	7.61	8.25	25.36
5-year Average	10.23	7.68	8.81	24.69
10-year Average	10.57	8.96	8.85	25.87

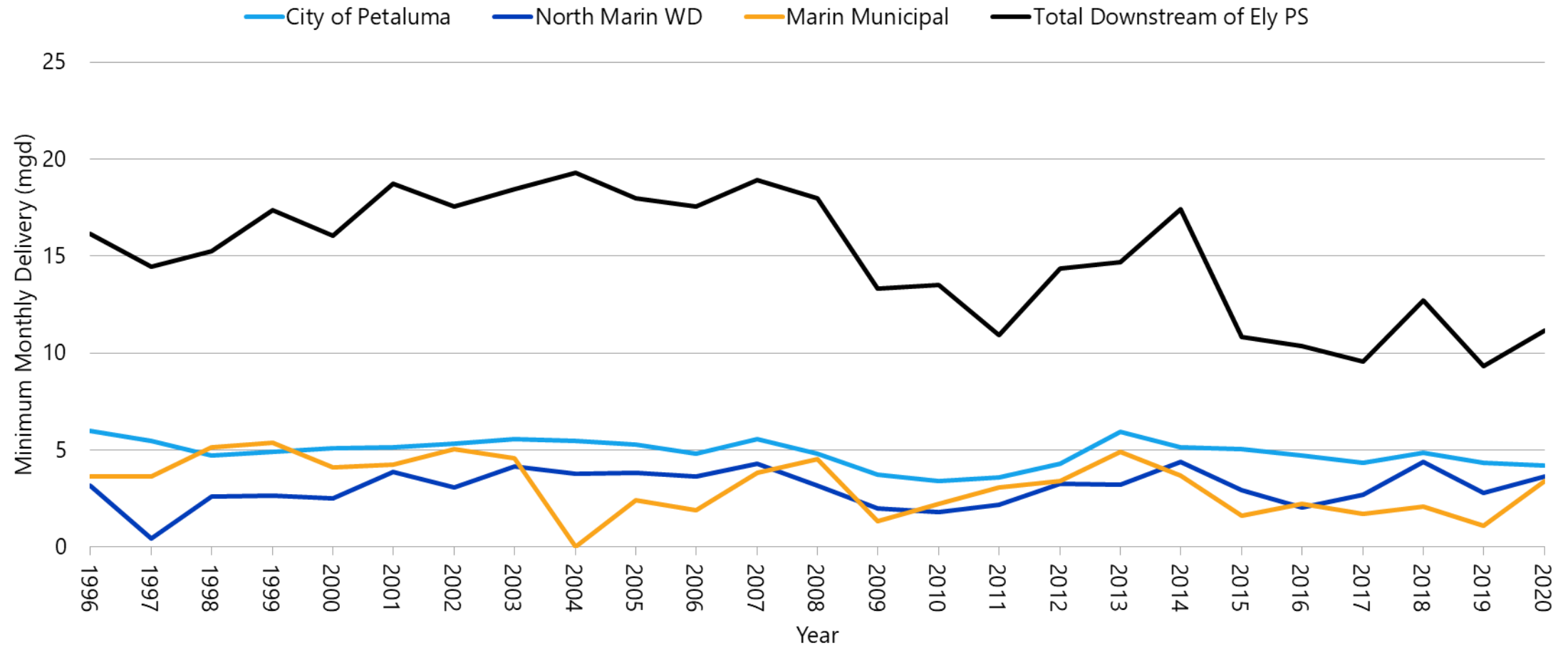
Historical water delivery data – maximum monthly in mgd



Historical water delivery data – minimum monthly in mgd

Year	Petaluma	NMWD	MMWD	Total
1996	6.00	3.14	3.63	16.14
1997	5.45	0.43	3.63	14.44
1998	4.72	2.60	5.13	15.23
1999	4.91	2.62	5.37	17.38
2000	5.10	2.48	4.11	16.04
2001	5.13	3.86	4.24	18.75
2002	5.32	3.07	5.03	17.57
2003	5.55	4.14	4.55	18.45
2004	5.45	3.79	0.00	19.31
2005	5.28	3.83	2.41	17.99
2006	4.80	3.63	1.88	17.57
2007	5.57	4.27	3.81	18.93
2008	4.82	3.17	4.54	17.97
2009	3.74	1.97	1.32	13.34
2010	3.41	1.81	2.23	13.52
2011	3.59	2.19	3.09	10.93
2012	4.30	3.24	3.38	14.37
2013	5.94	3.19	4.88	14.69
2014	5.13	4.38	3.68	17.41
2015	5.05	2.90	1.59	10.83
2016	4.69	2.03	2.19	10.33
2017	4.35	2.67	1.69	9.54
2018	4.85	4.38	2.06	12.69
2019	4.35	2.78	1.10	9.32
2020	4.18	3.61	3.38	11.17
5-year Average	4.49	3.09	2.08	10.61
10-year Average	4.64	3.14	2.70	12.13

Historical water delivery data – minimum monthly in mgd



Previous Model Scenarios and Results

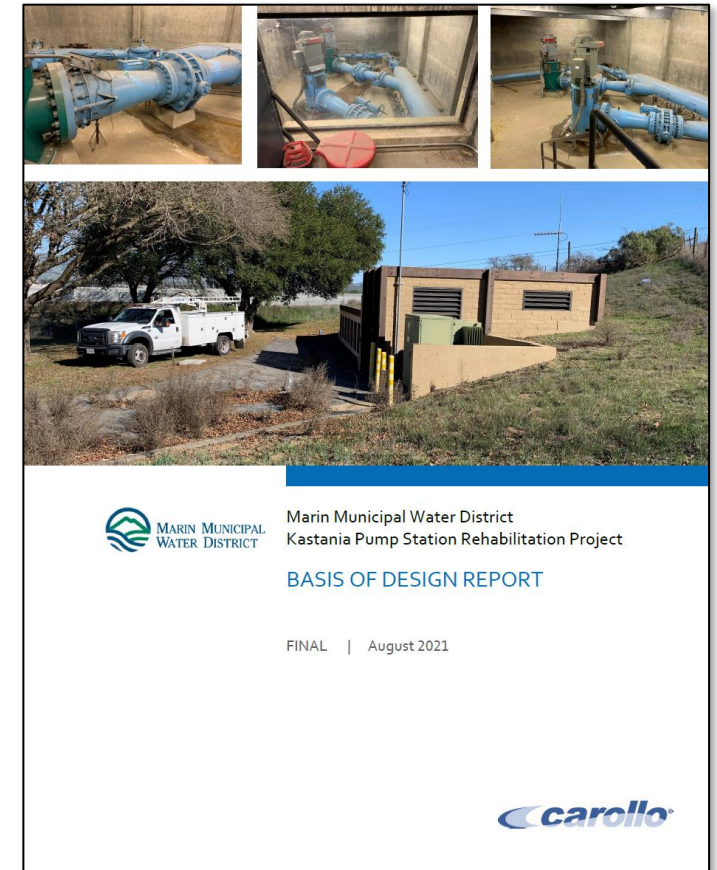
The previous model included various scenarios for the Kastania PS analysis

Table 2 Hydraulic Analysis Summary

Condition	North Marin Demands (mgd)	Available Water to MMWD (mgd)		Limiting Factor	MMWD Capacity (mgd)	Difference, mgd (%)
		Without Kastania ⁽¹⁾	With Kastania ⁽²⁾			
Current ⁽³⁾	~11	4	10.5	None	10-12 ⁽⁴⁾	6.5 (160)
Summer	8	7	13.5	Contract with SCWA	12.8 ⁽⁵⁾	5.8 (83)
April/May	4	11	17.5	MMWD Distribution System	10-14 ⁽⁴⁾	0-3 (0-27)

Notes:

- (1) Based on maximum flow through NMA of 15 mgd.
- (2) Assumes one pump (Pump No. 2) active. Based on maximum KPS flow of 21.5 mgd.
- (3) Current conditions as of April 2021. NMWD demands atypically high due to filling Stafford Lake.
- (4) Estimated capacity of MMWD distribution system.
- (5) Maximum delivery under agreement with SCWA.



03

Evaluation of Proposed Turnout Locations

Model scenarios were developed to evaluate hydraulic effects to the existing system for two proposed turnout locations under existing winter conditions

Turnout Location	Low Demand (mgd)	High Demand (mgd)
Turnout 1	12.5	15.0
Turnout 2	12.5	30.0



—

Winter demands for other turnouts were assigned according to District input

Turnout	Assumed Demand (mgd)
Petaluma (assigned to most downstream turnout)	5.0
North Marin/San Marin PS	4.5
Ignacio BPS	8.0

Minimum HGL requirements were determined or assumed for each turnout

Turnout	Required Minimum HGL	Basis
Petaluma	Varies by elevation	Assumed minimum 50 psi
Turnout 2	149 feet	Assumed minimum 50 psi
Turnout 1/North Marin/San Marin PS	170 feet	Required HGL according to Kastania PS BODR to fill NMWD tanks
Ignacio BPS	133 feet	Assumed minimum 50 psi

* It is assumed that a booster pump station will be required for each of the alternative turnout locations to pump water to the District's reservoirs. Modeled HGLs were documented for each alternative to compare potential pumping requirements downstream of the turnouts.

Maximum demand to maintain velocities less than 8 fps downstream of Kastania PS (i.e., North Marin Aqueduct)

Turnout	Minimum Diameter between Kastania PS and Turnout (inches)	Maximum Total Flow through Smallest Diameter Section (mgd)	Maximum Turnout Demand to Not Exceed 8 fps in North Marin Aqueduct (mgd) ⁽¹⁾
Turnout 1	30	25	12.5
Turnout 2	42	49 ⁽²⁾	36.5

Notes:

1. Assumes demands of 4.5 mgd and 8.0 mgd at NMWD and Ignacio PS turnouts.
2. Actual maximum flow is limited to 25 mgd due to Kastania PS capacity.

Maximum demand to maintain velocities less than 8 fps upstream of Kastania PS (i.e., Petaluma Aqueduct)

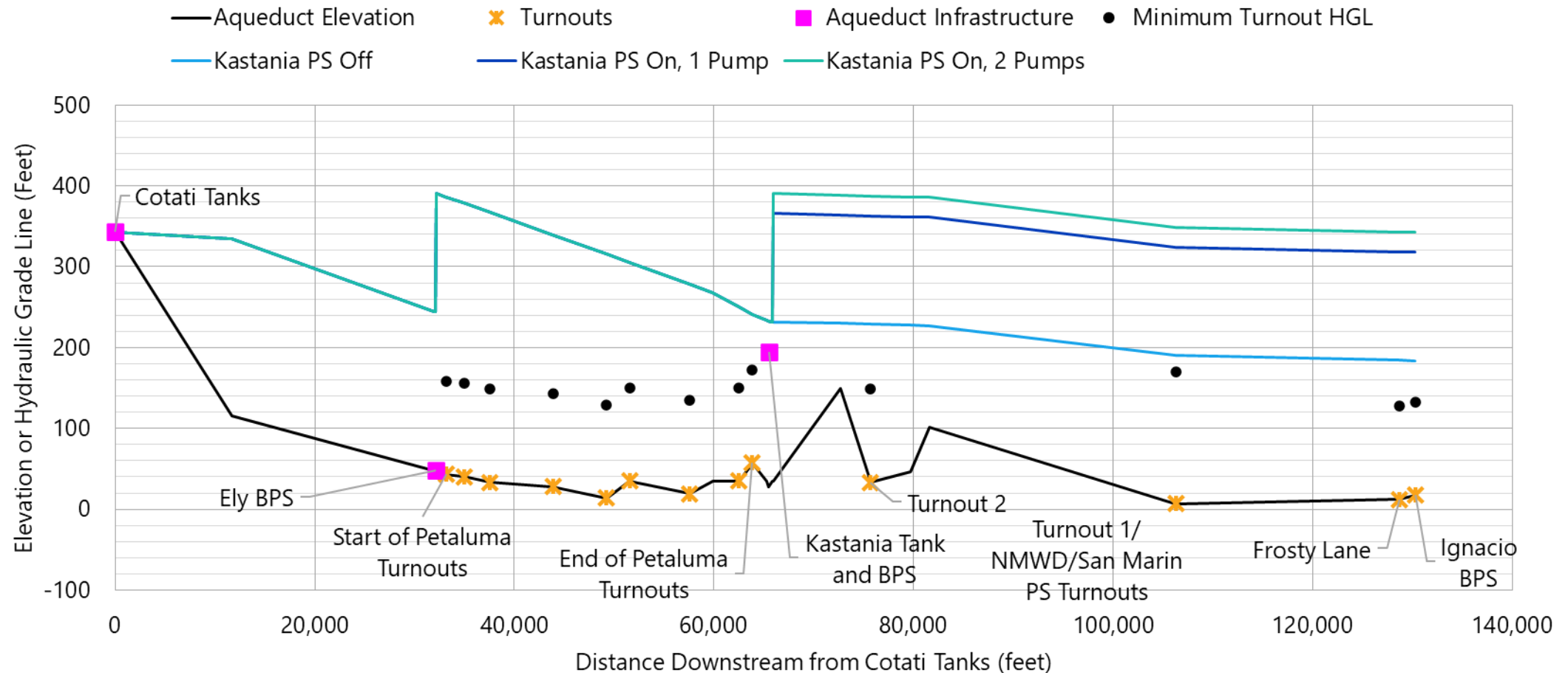
Turnout	Minimum Diameter along Petaluma Aqueduct (inches)	Maximum Total Flow through Smallest Diameter Section (mgd)	Maximum Turnout Demand to Not Exceed 8 fps in Petaluma Aqueduct (mgd) ⁽¹⁾⁽²⁾
Turnout 1	30	25	5.1
Turnout 2	30	25	5.1

Notes:

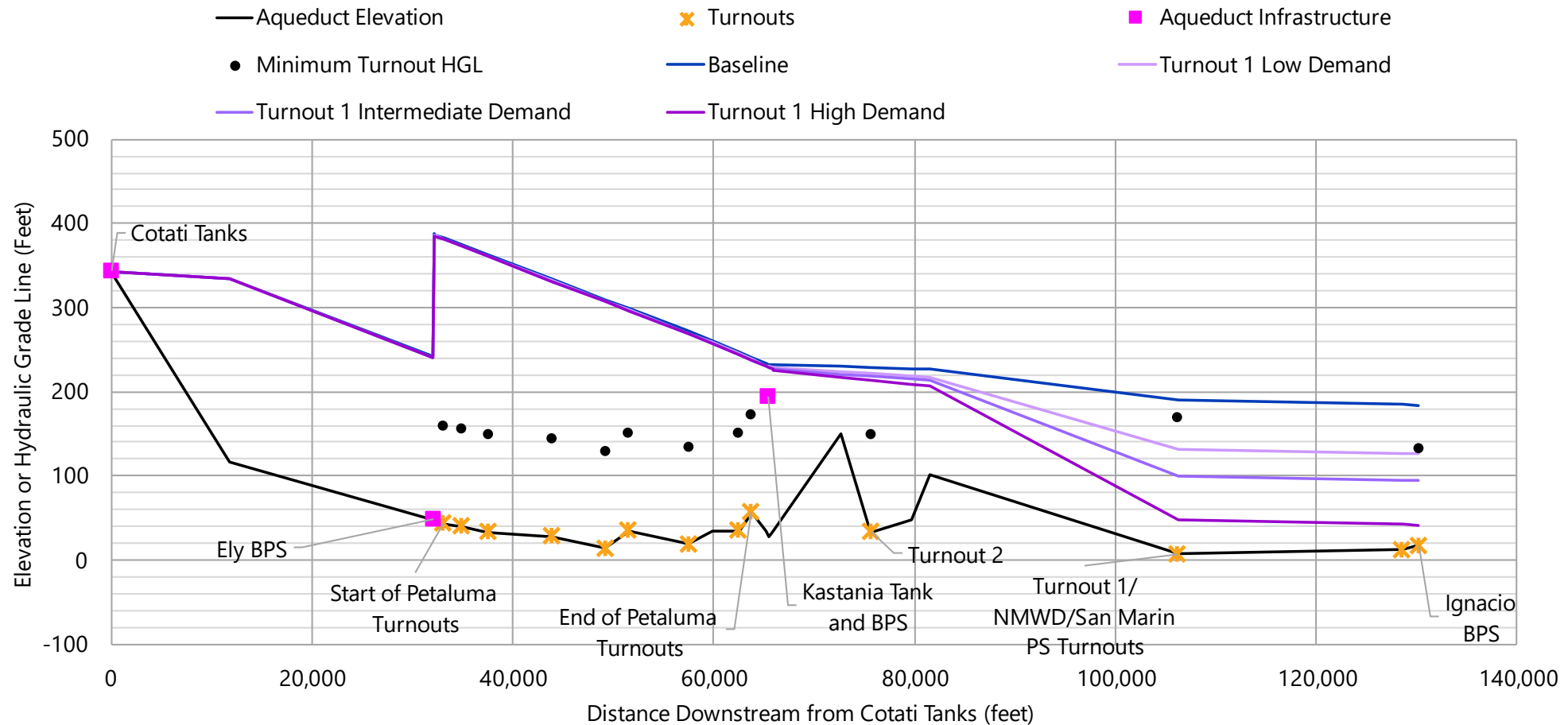
1. Assumes demands of 5.0, 4.5 mgd, and 8.0 mgd at Petaluma, NMWD, and Ignacio PS turnouts.
2. Assumes Kastania Tank fills at a rate of 2.4 mgd (i.e., rate required to increase the tank level by 4 feet from 34 to 38 feet over 12 hours) while demands are active.

Model Results

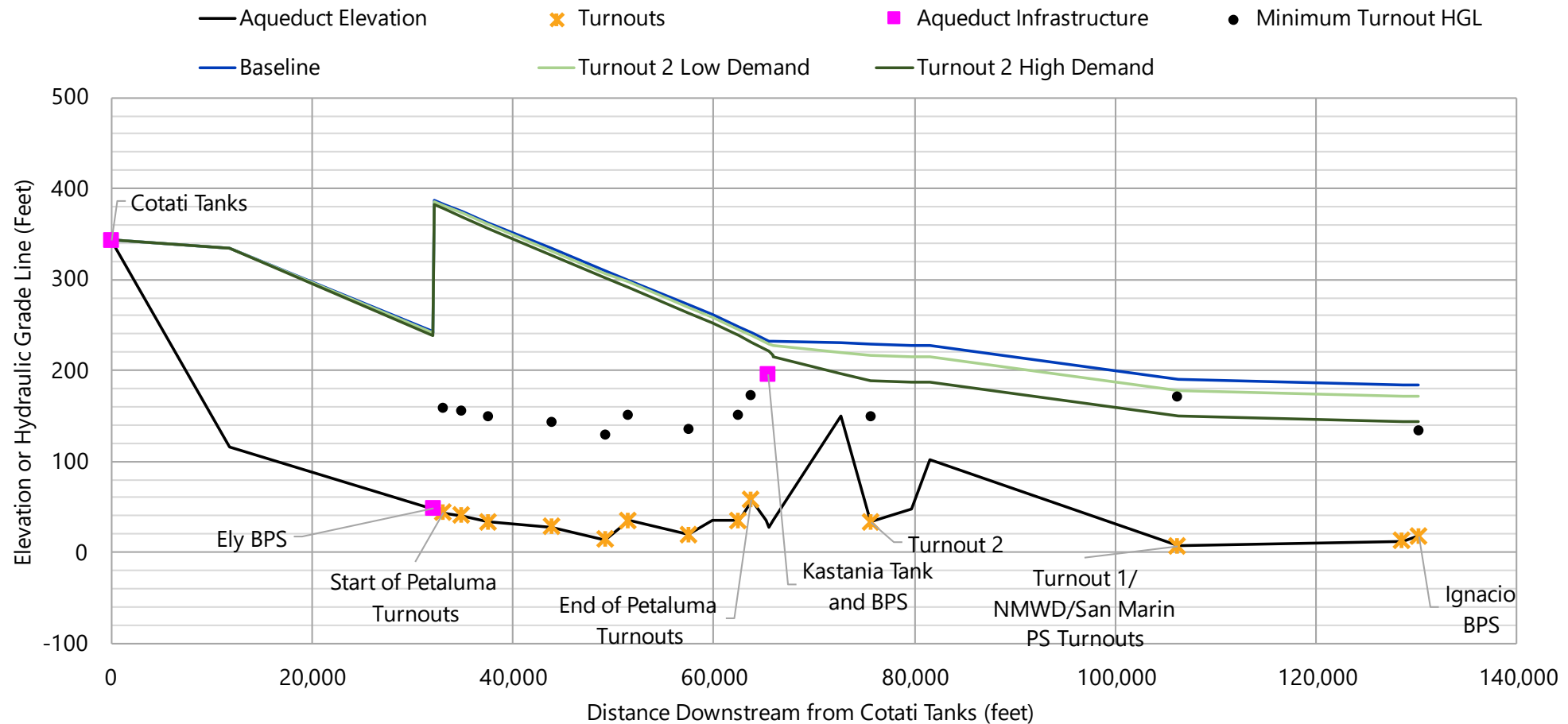
Baseline scenarios were run with and without the Kastania PS on



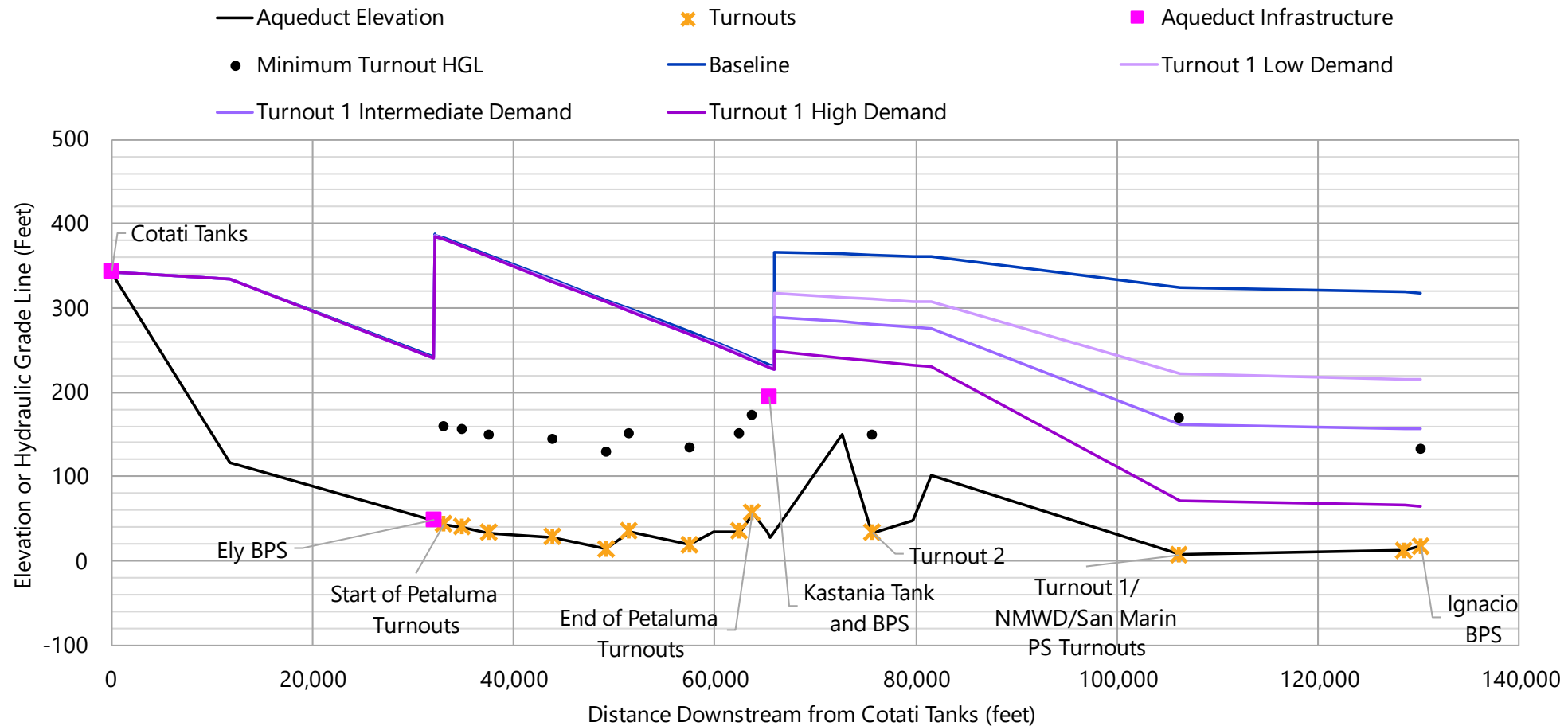
Model results with Kastania PS off - Turnout 1



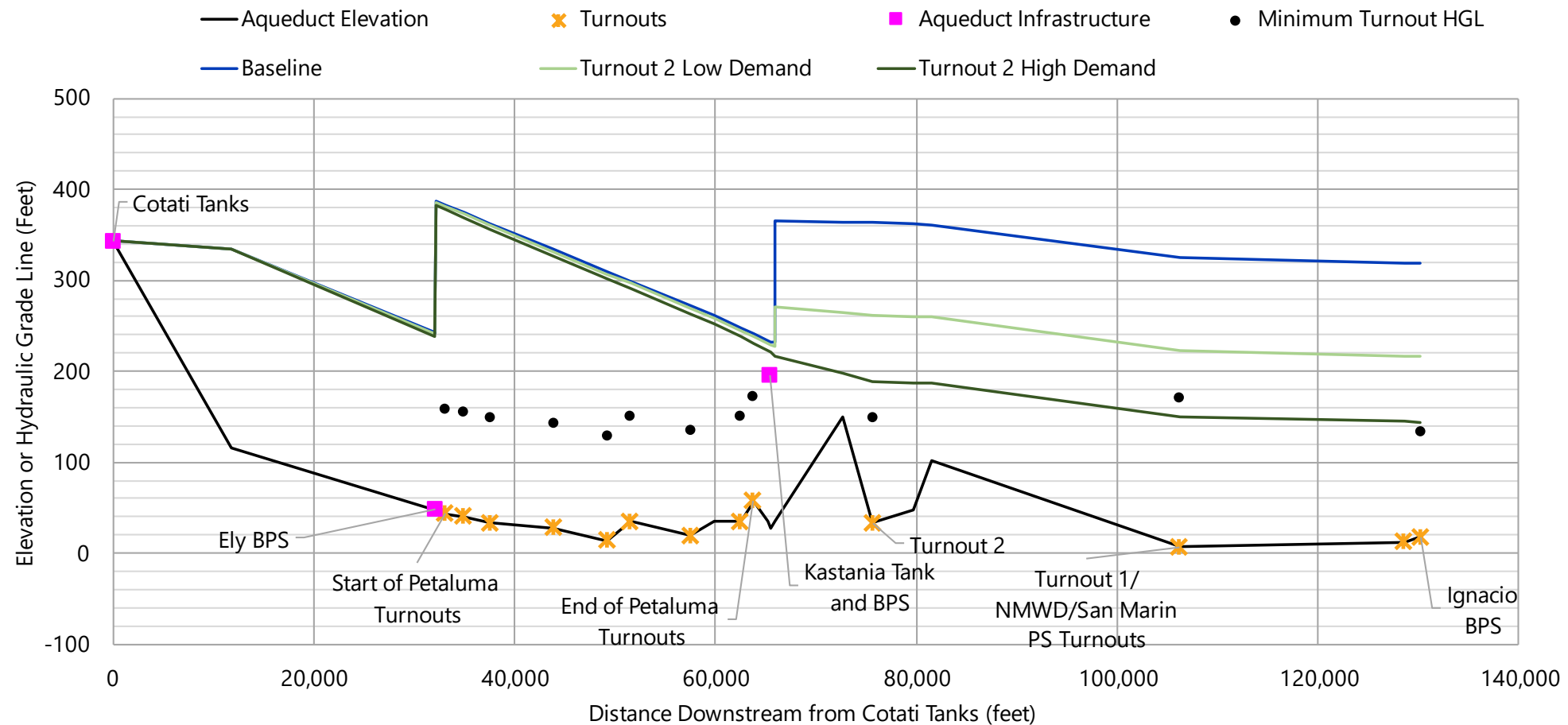
Model results with Kastania PS off - Turnout 2



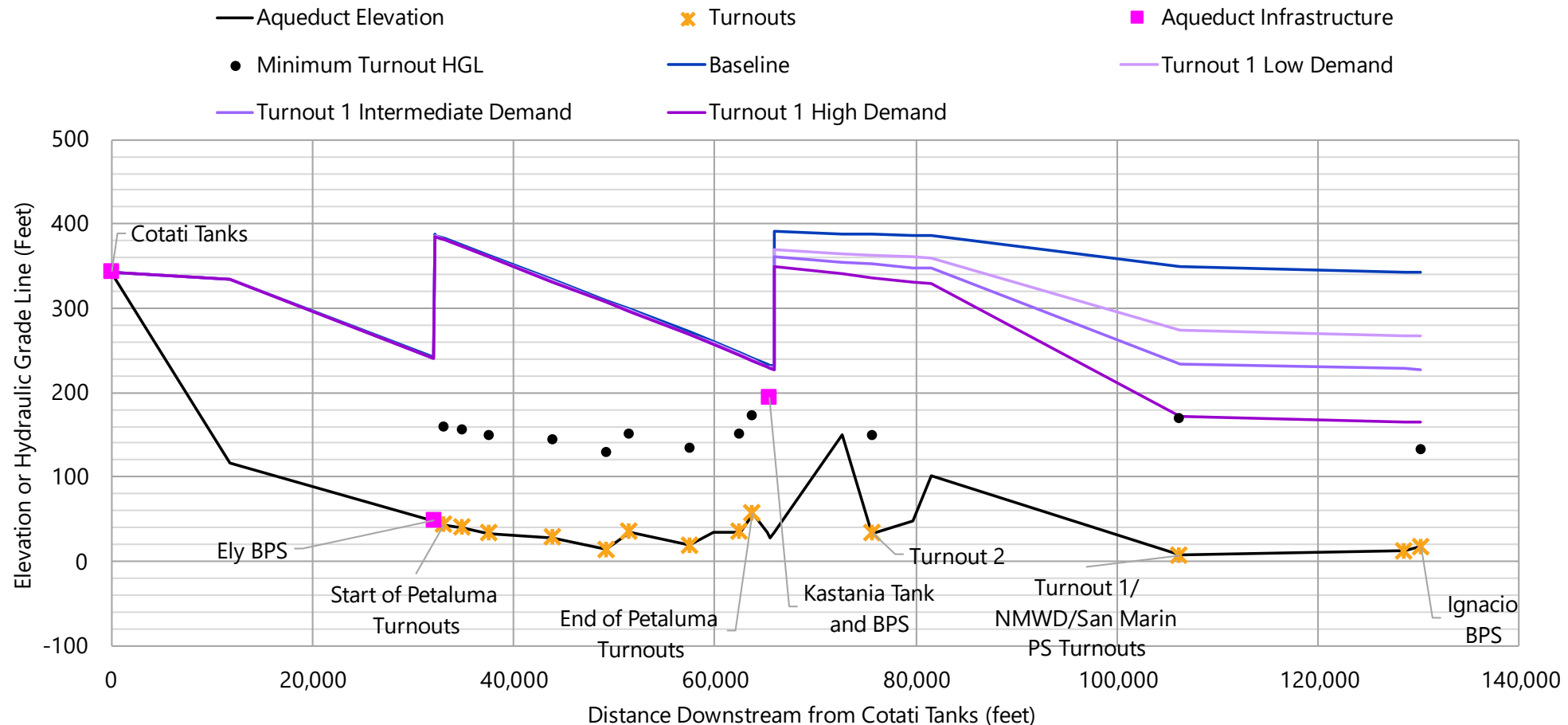
Model results with Kastania PS on (1 pump) - Turnout 1



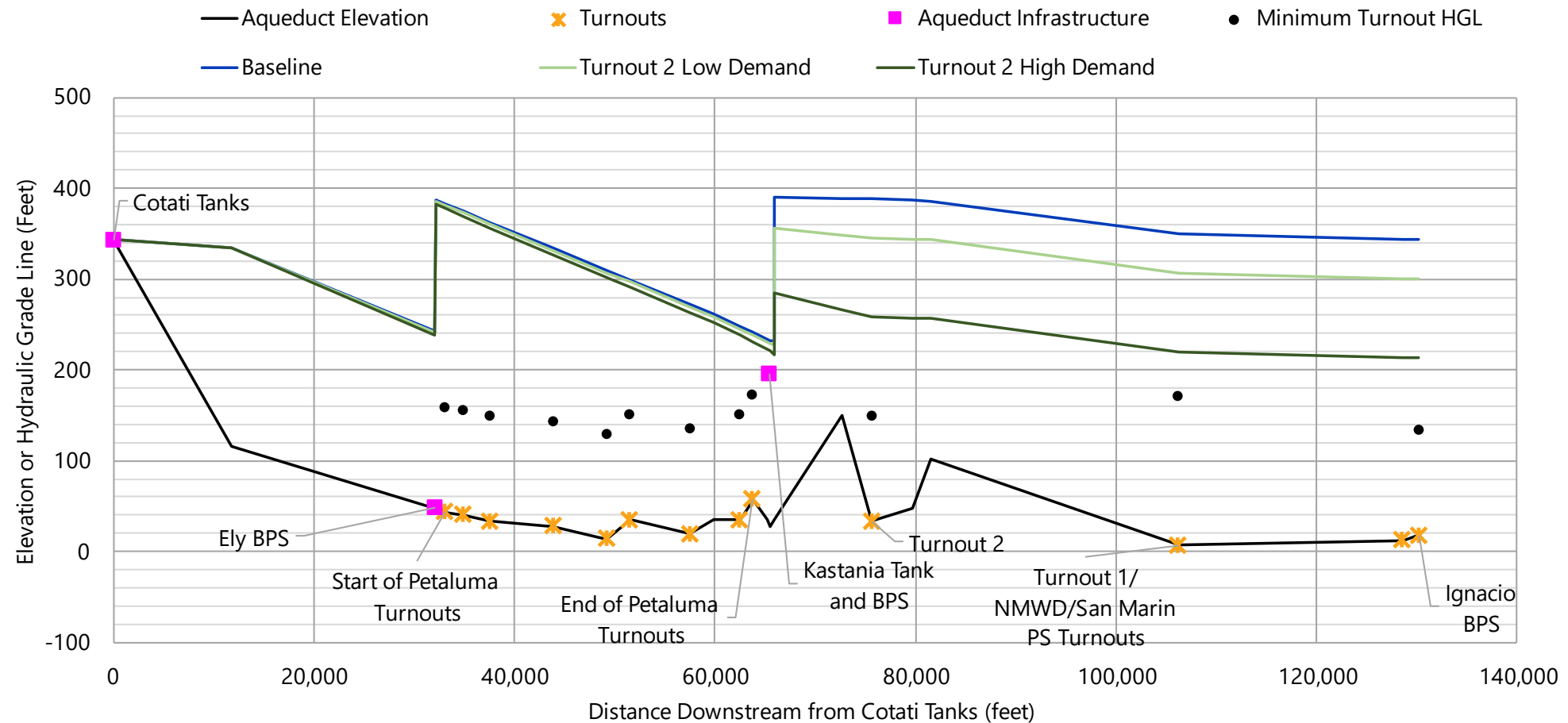
Model results with Kastania PS on (1 pump) - Turnout 2



Model results with Kastania PS on (2 pumps) - Turnout 1



Model results with Kastania PS on (2 pumps) - Turnout 2



Summary of model results at alternative turnouts

Turnout	Modeled Turnout Elevation (feet)	Modeled Turnout Demand (mgd)	Kastania PS Status	Required Minimum Head (feet) ⁽¹⁾	Modeled Head at Turnout (feet) ⁽²⁾	Modeled Head at NMWD Turnout (feet)	Maximum Pipeline Velocity (fps) ⁽³⁾
Turnout 1	7	7.2	Off	170	132	132	6.2
Turnout 1	7	10.4	Off	170	100	100	7.2
Turnout 1	7	15.0	Off	170	48	48	8.7
Turnout 1	7	7.2	1 pump on	170	221	221	6.2
Turnout 1	7	10.4	1 pump on	170	163	163	7.2
Turnout 1	7	15.0	1 pump on	170	71	71	8.7
Turnout 1	7	7.2	2 pumps on	170	274	274	6.2
Turnout 1	7	10.4	2 pumps on	170	234	234	7.2
Turnout 1	7	15.0	2 pumps on	170	171	171	8.7
Turnout 2	33	12.5	Off	149	217	178	4.0
Turnout 2	33	30.0	Off	149	188	150	6.8
Turnout 2	33	12.5	1 pump on	149	261	222	4.0
Turnout 2	33	30.0	1 pump on	149	189	150	6.8
Turnout 2	33	12.5	2 pumps on	149	345	307	4.0
Turnout 2	33	30.0	2 pumps on	149	258	220	6.8

Notes:

1. Minimum head for Turnout 1 was determined in the Kastania PS analysis as the head required to fill the NMWD tanks. Minimum head for Turnout 2 is assumed to be 50 psi.
2. Highlighted head values are at or below required minimum head. Highlighted velocities are greater than 8 fps.
3. Downstream of Kastania PS.

04

Hydraulic Analysis of Short-Listed Alternatives

General assumptions

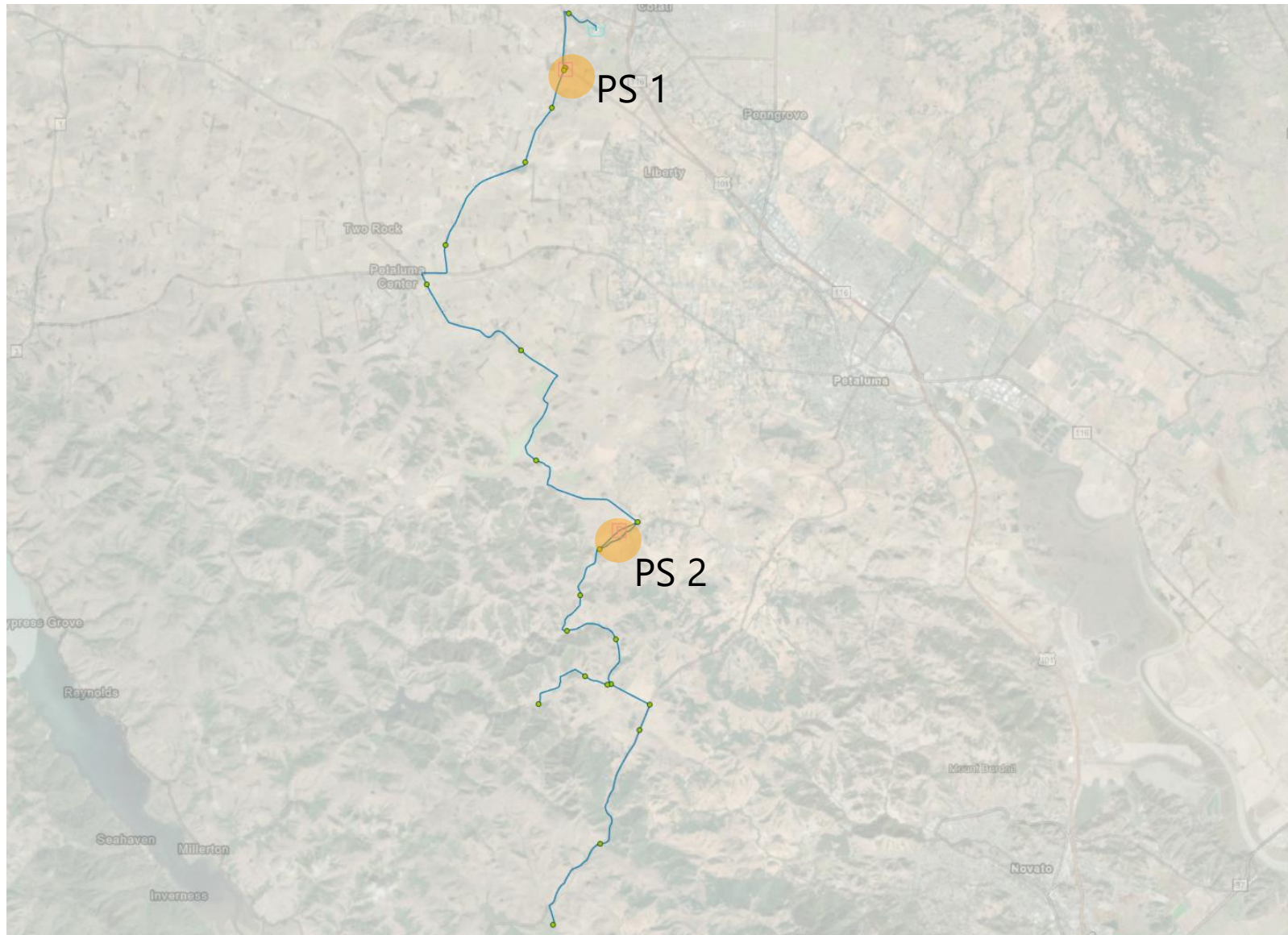
- Assumed all demands would occur at Nicasio reservoir, which is the most conservative scenario.
- Assumed C factors of 120. Lower C factor was assumed to account for any minor losses.
- Allowable HGL range along new alignment:
 - » Minimum HGL = elevation + 20 psi.
 - » Maximum HGL = elevation + 250 psi.

COTATI-3 Hydraulic Analysis

COTATI-3 Scenarios

Scenario	Pipe Diameter	Pump Station A Status	Pump Station B Status	Nicasio Reservoir Demand (mgd)	Cotati Tanks HGL (feet)
1	30	On	Off	30	343
2	30	On	On	30	343
3	36	On	Off	30	343
4	36	On	On	30	343
5	42	On	Off	30	343
6	42	On	On	30	343

Assumed Pump Station Locations

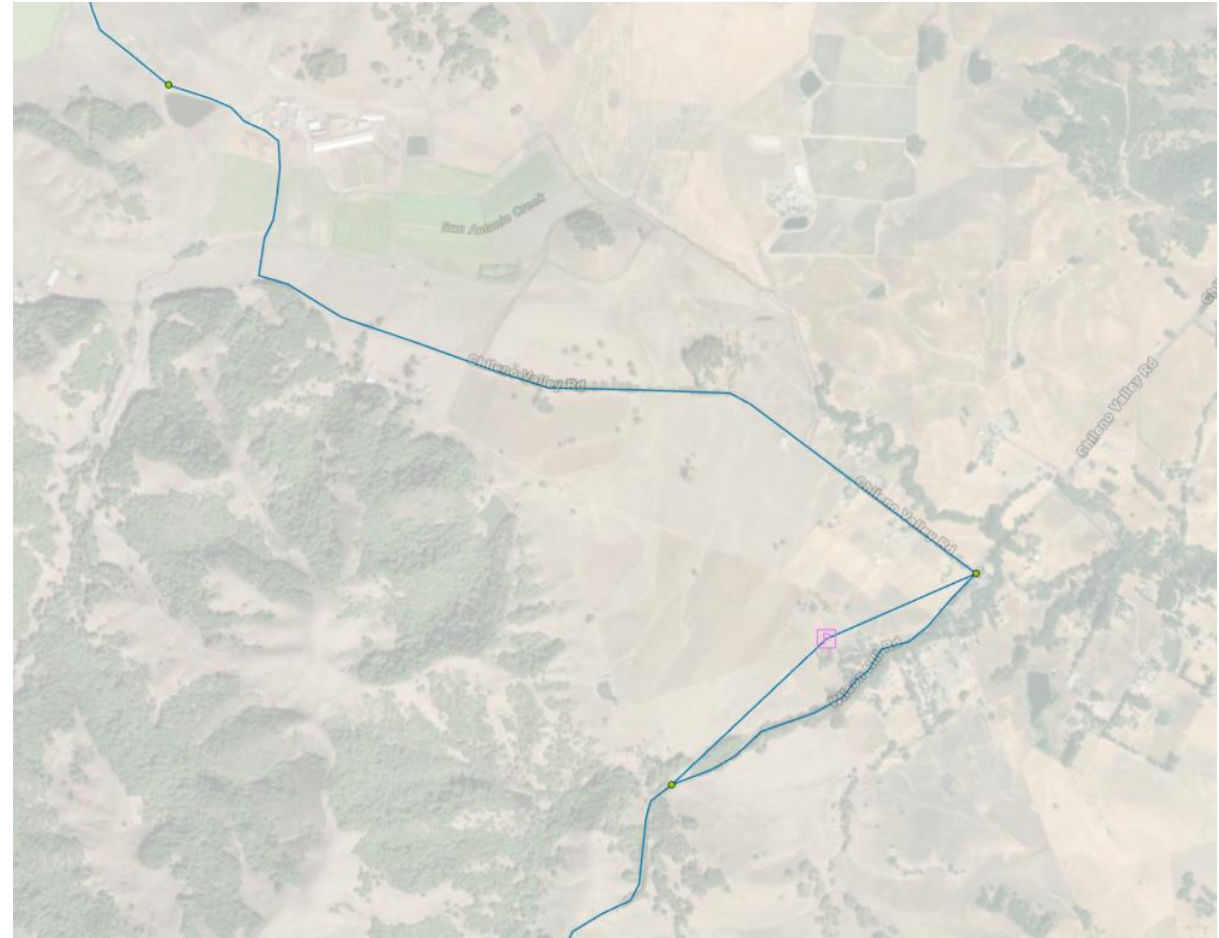


Assumed Pump Station Locations

Pump Station 1

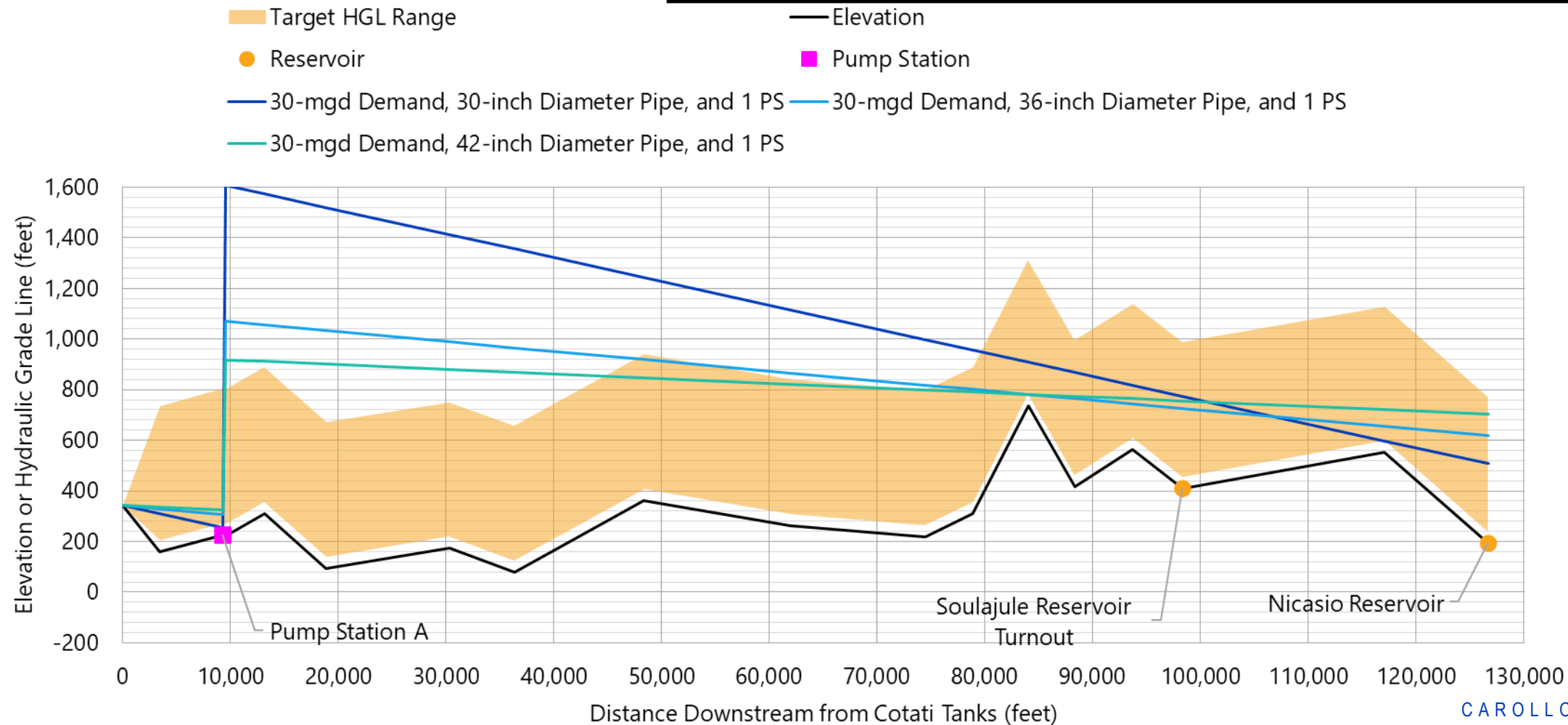


Pump Station 2



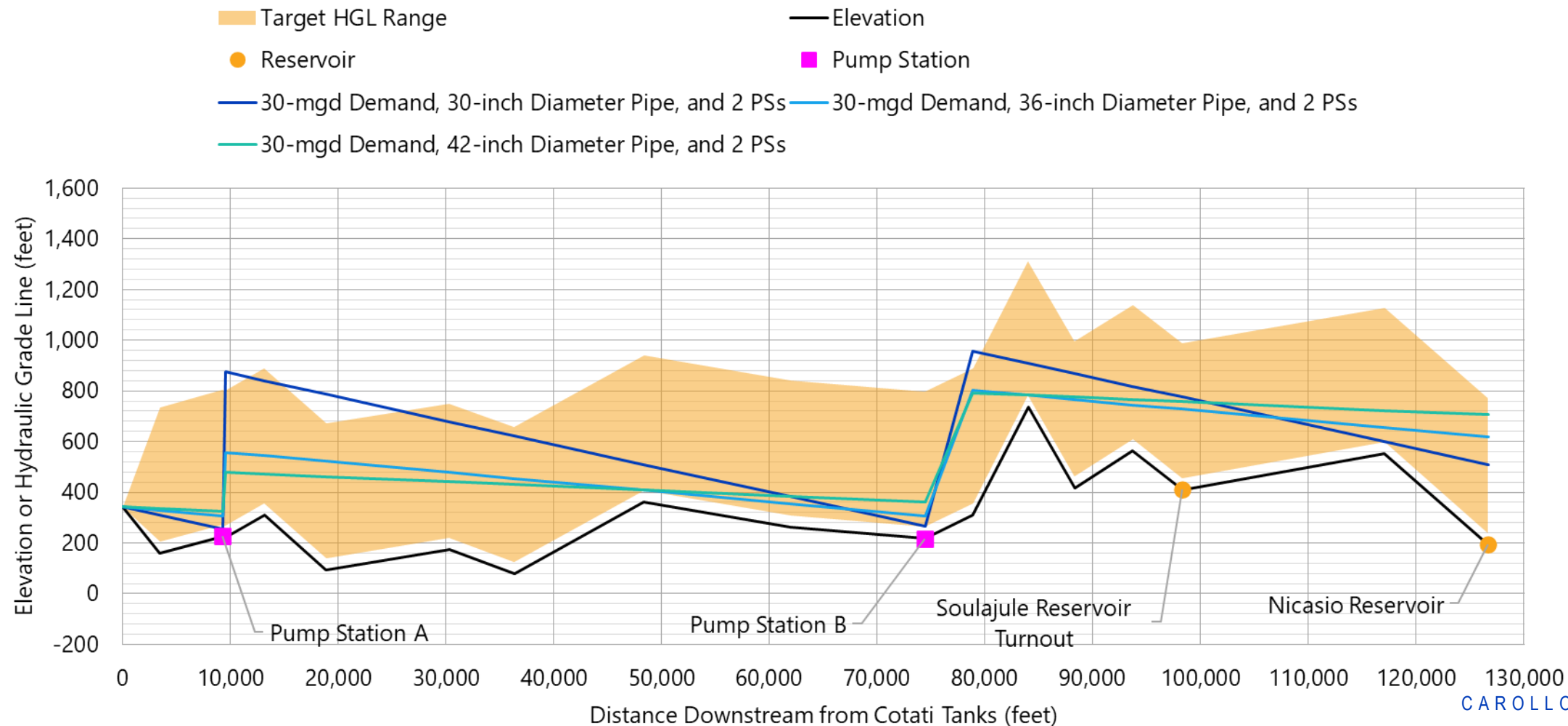
1 Pump Station

Scenario	Pipe Diameter (inches)	Pump Station A Required TDH (feet)	Maximum Pressure (psi)
1	30	1,354	618
3	36	763	407
5	42	592	350



2 Pump Stations

Scenario	Pipe Diameter (inches)	Pump Station A Required TDH (feet)	Pump Station B Required TDH (feet)	Maximum Pressure (psi)
2	30	621	734	300
4	36	252	512	213
6	42	153	440	221



Potentially feasible COTATI-3 configurations

Pipe Diameter	Pump Station A Status	Pump Station B Status	Maximum Pressure (psi)
30	On	Off	618
30	On	On	300
36	On	Off	407
36	On	On	213
42	On	Off	350
42	On	On	221

Summary of potentially feasible COTATI-3 alternatives

Pipe Diameter	Pump Station A TDH (feet)	Pump Station B TDH (feet)	Pump Station A Required Power (hp)	Pump Station B Required Power (hp)	Total Annual Energy Usage (kWh)	Total Annual Energy Cost
36	252	512	1,659	3,370	13.5M	\$5.0M
42	153	440	1,007	2,896	10.5M	\$3.9M

Pumping Assumptions:

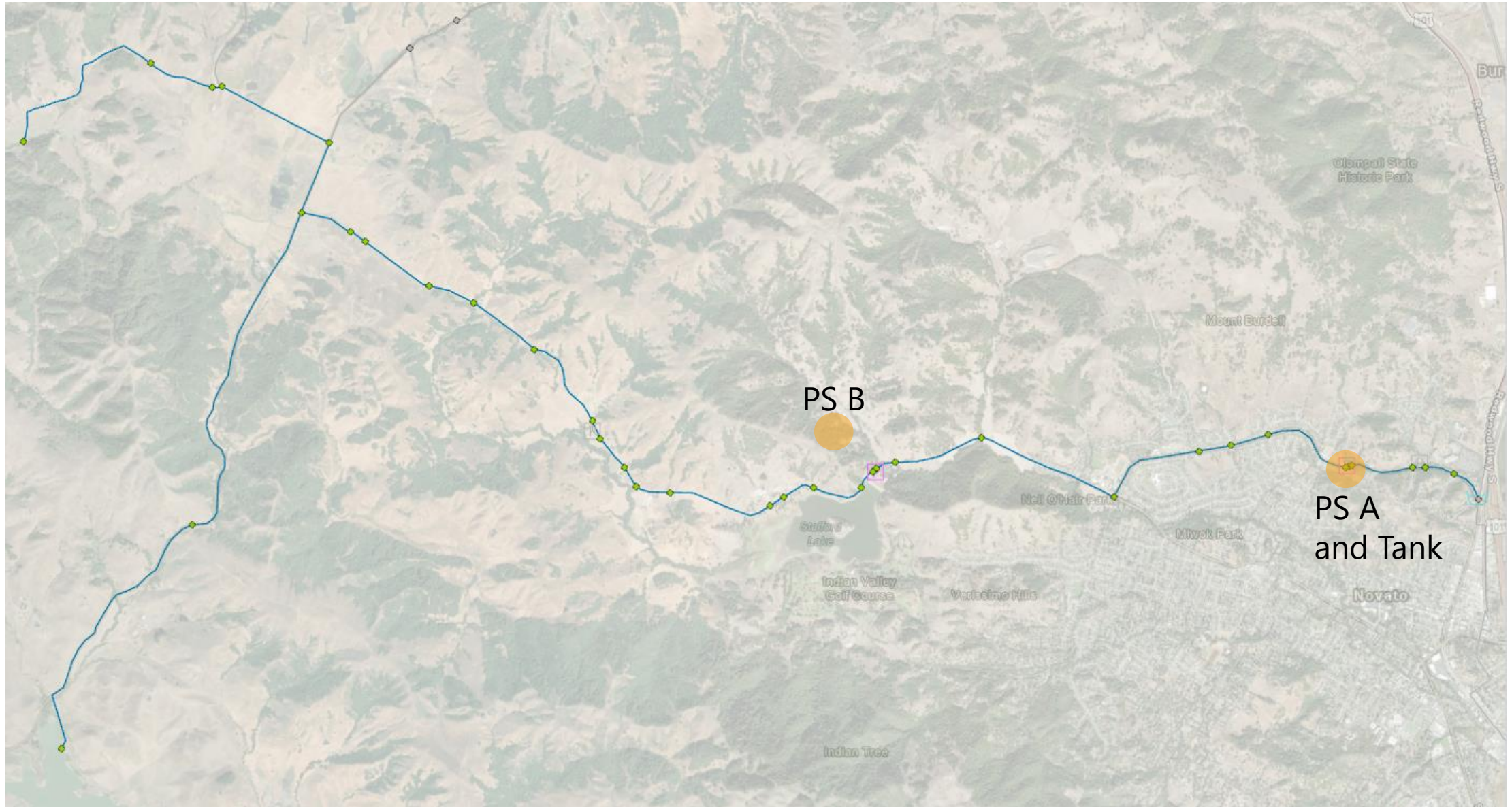
- Efficiency = 80%
- Annual operational days = 150 days/year
- Electricity cost = \$0.37/kWh

PETA-3 Hydraulic Analysis

PETA-3 scenarios

Scenario	Pipe Diameter (inches)	Pump Station A Status	Pump Station B Status	Nicasio Reservoir Demand (mgd)
1	24	On	On	10.4
2	24	On	On	15.0
3	27	On	On	10.4
4	27	On	On	15.0
5	30	On	Off	10.4
6	30	On	Off	15.0
7	30	On	On	10.4
8	30	On	On	15.0
9	36	On	Off	10.4
10	36	On	Off	15.0
11	42	On	Off	10.4
12	42	On	Off	15.0
13	30	On	On	30.0
14	36	On	On	30.0
15	42	On	Off	30.0
16	48	On	Off	30.0

Assumed Pump Station Locations



Assumed Pump Station Locations

Pump Station A and Tank



Pump Station B

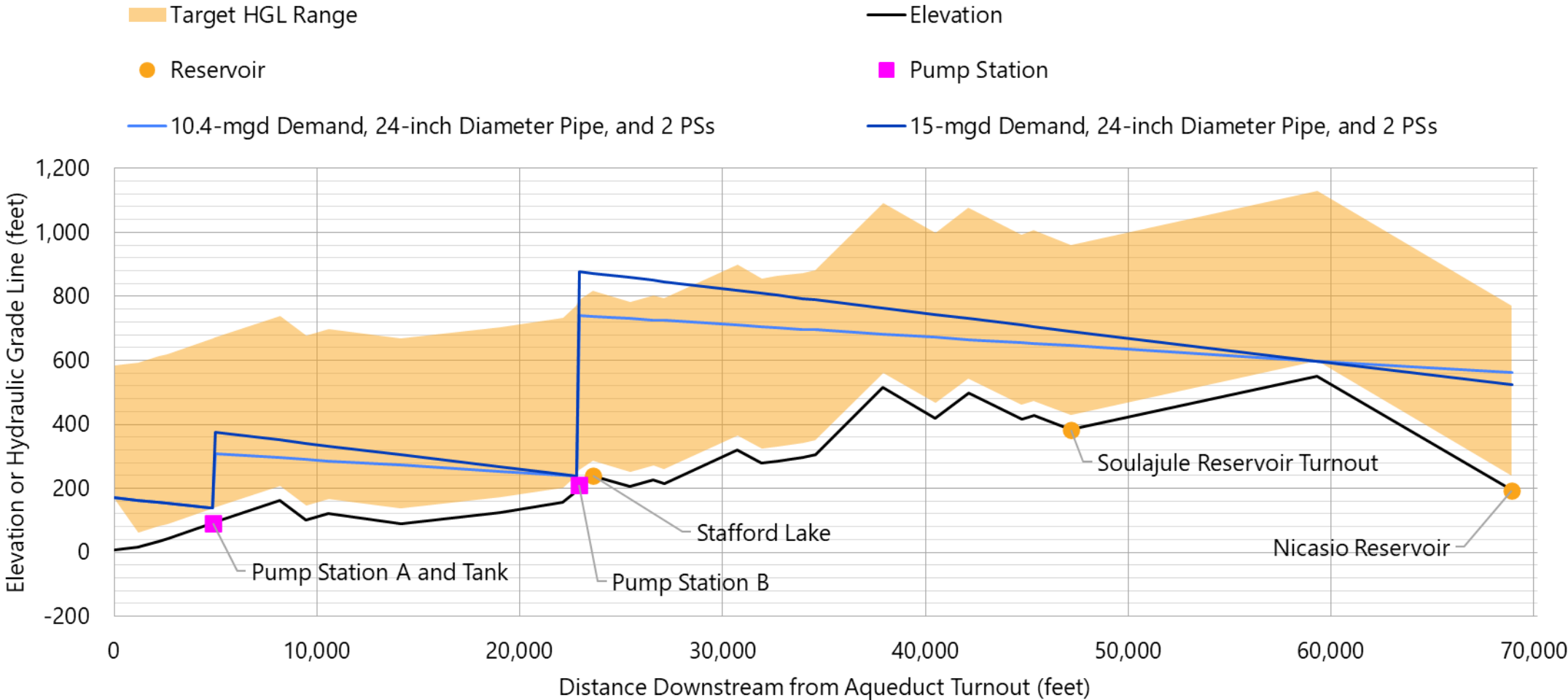


Assumed Tank Parameters

Parameter	Assumption
Location	Directly upstream of PS A, 4,850 lf downstream from Petaluma/North Marin Aqueduct turnout
Base elevation	130 feet
Minimum operating level	9 feet (to maintain minimum 20 psi pressure)
Assumed level for model analysis	9 feet

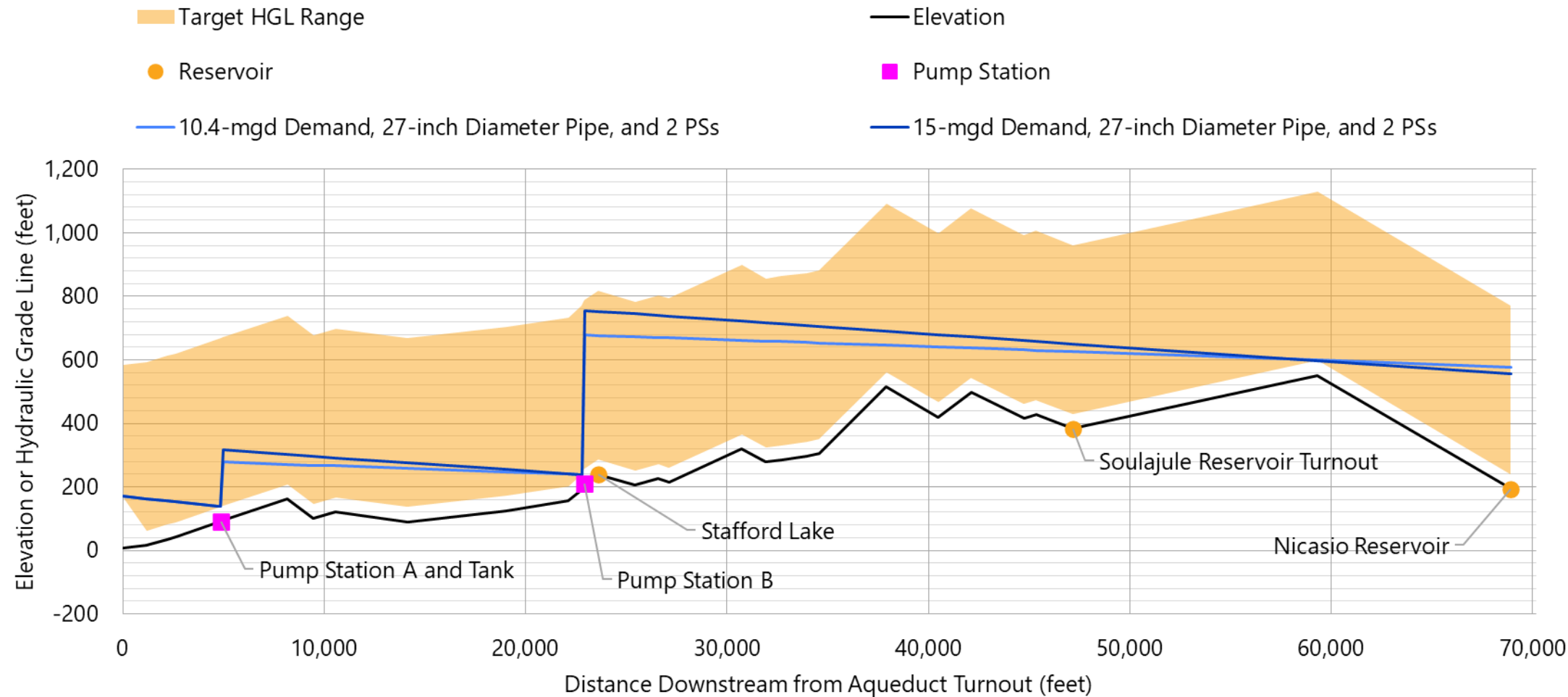
24-inch
Pipe

Scenario	Pipe Diameter (inches)	Pump Station A TDH (feet)	Pump Station B TDH (feet)	Nicasio Reservoir Demand (mgd)	Maximum Pressure (psi)
1	24	170	503	10.4	230
2	24	239	641	15.0	289



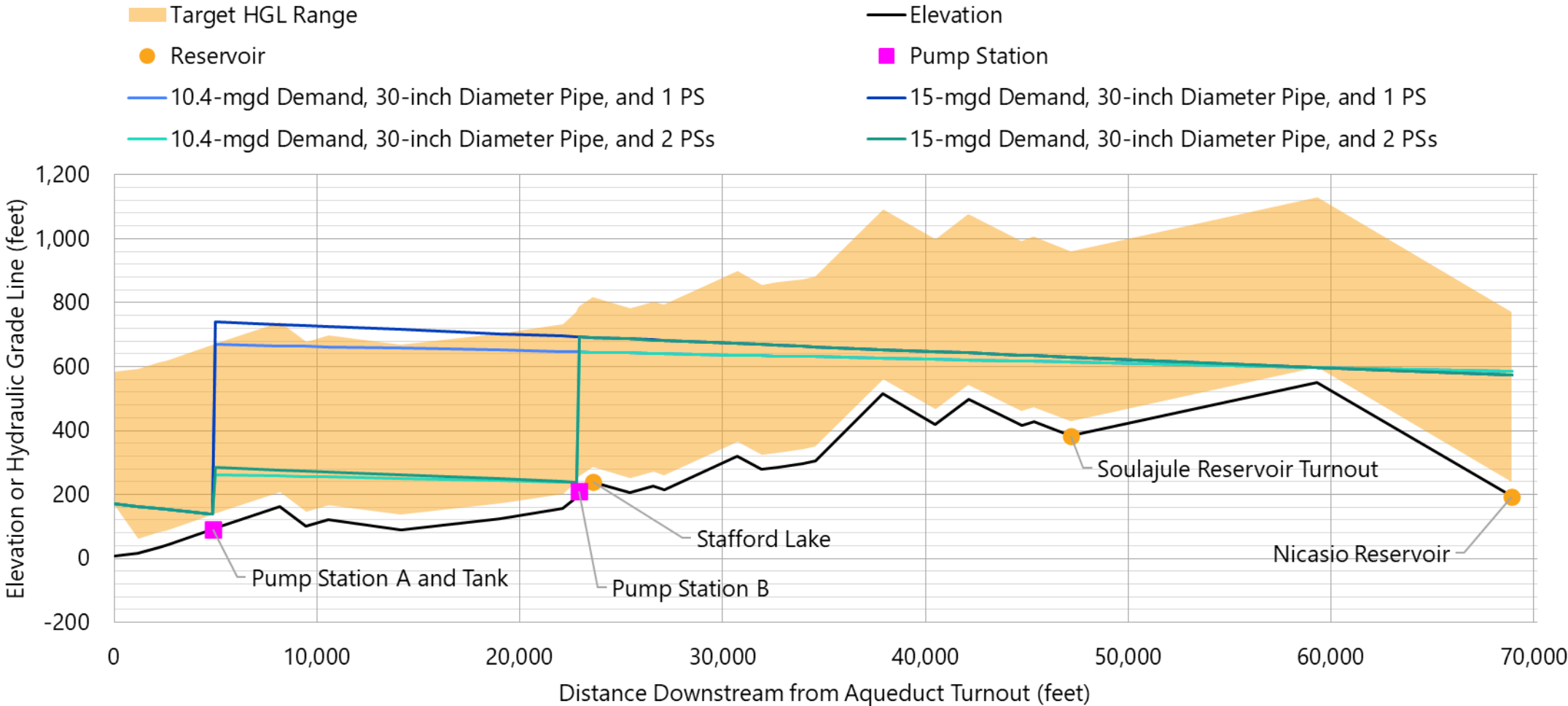
27-inch Pipe

Scenario	Pipe Diameter (inches)	Pump Station A TDH (feet)	Pump Station B TDH (feet)	Nicasio Reservoir Demand (mgd)	Maximum Pressure (psi)
3	27	140	440	10.4	203
4	27	178	518	15.0	236



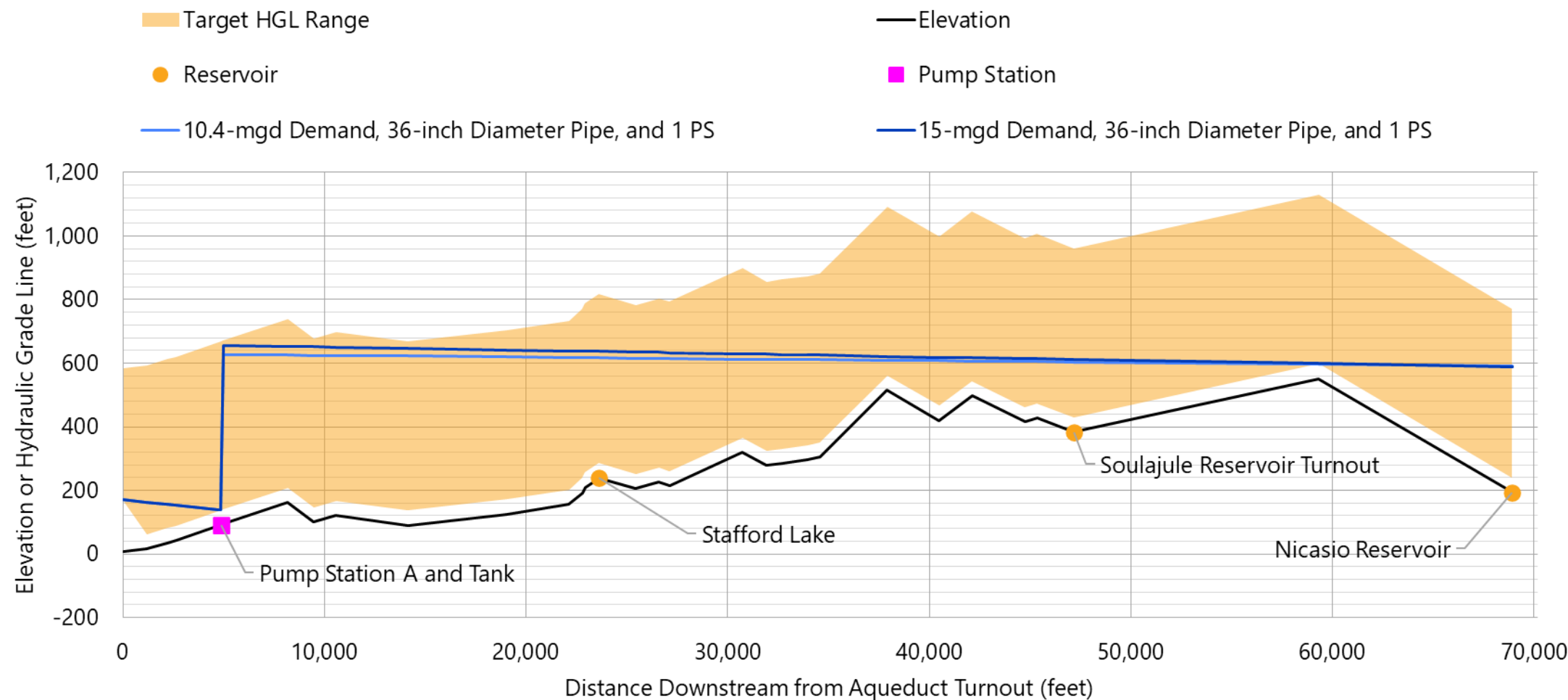
30-inch Pipe

Scenario	Pipe Diameter (inches)	Pump Station A TDH (feet)	Pump Station B TDH (feet)	Nicasio Reservoir Demand (mgd)	Maximum Pressure (psi)
5	30	531	N/A	10.4	249
6	30	601	N/A	15.0	279
7	30	123	408	10.4	189
8	30	147	454	15.0	209



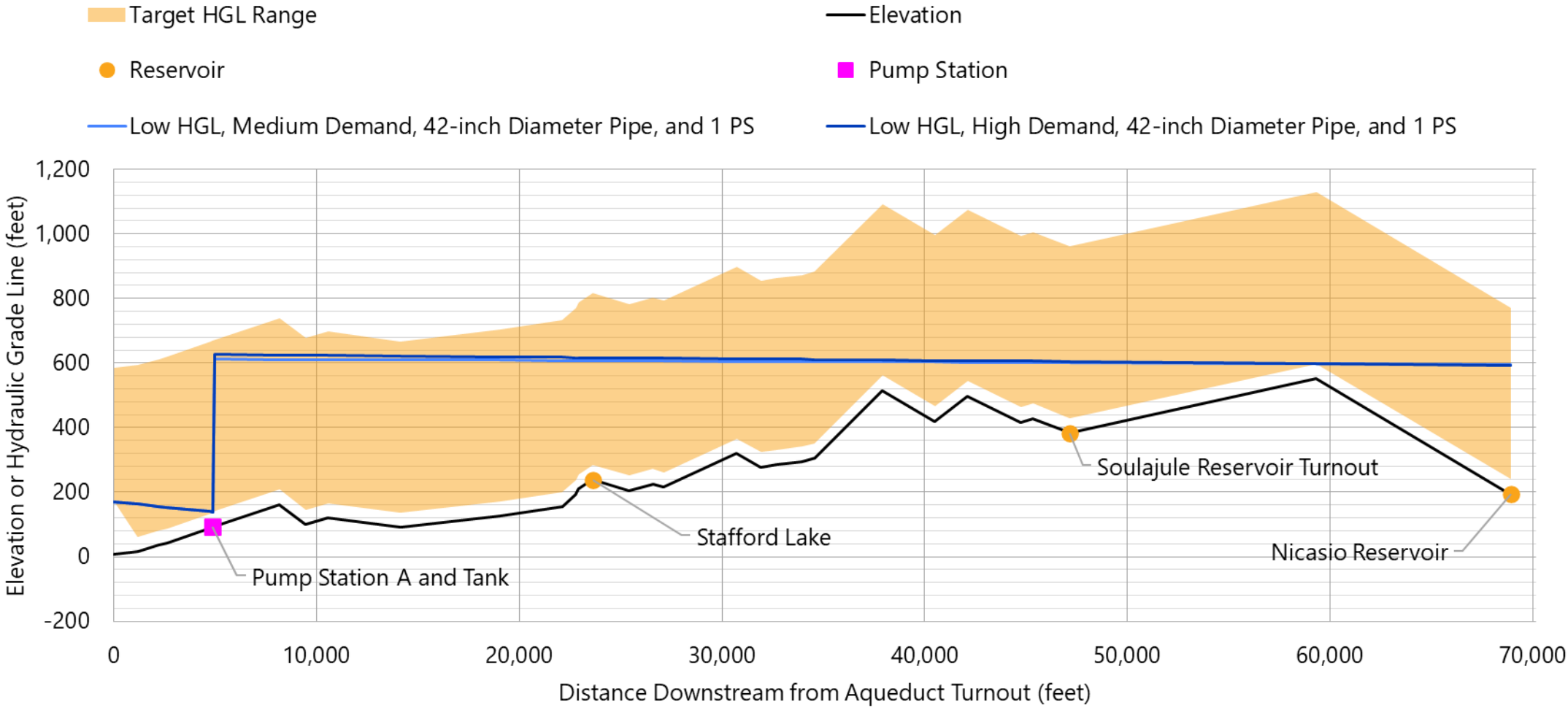
36-inch Pipe

Scenario	Pipe Diameter (inches)	Pump Station A TDH (feet)	Pump Station B TDH (feet)	Nicasio Reservoir Demand (mgd)	Maximum Pressure (psi)
9	36	488	N/A	10.4	230
10	36	601	N/A	15.0	243



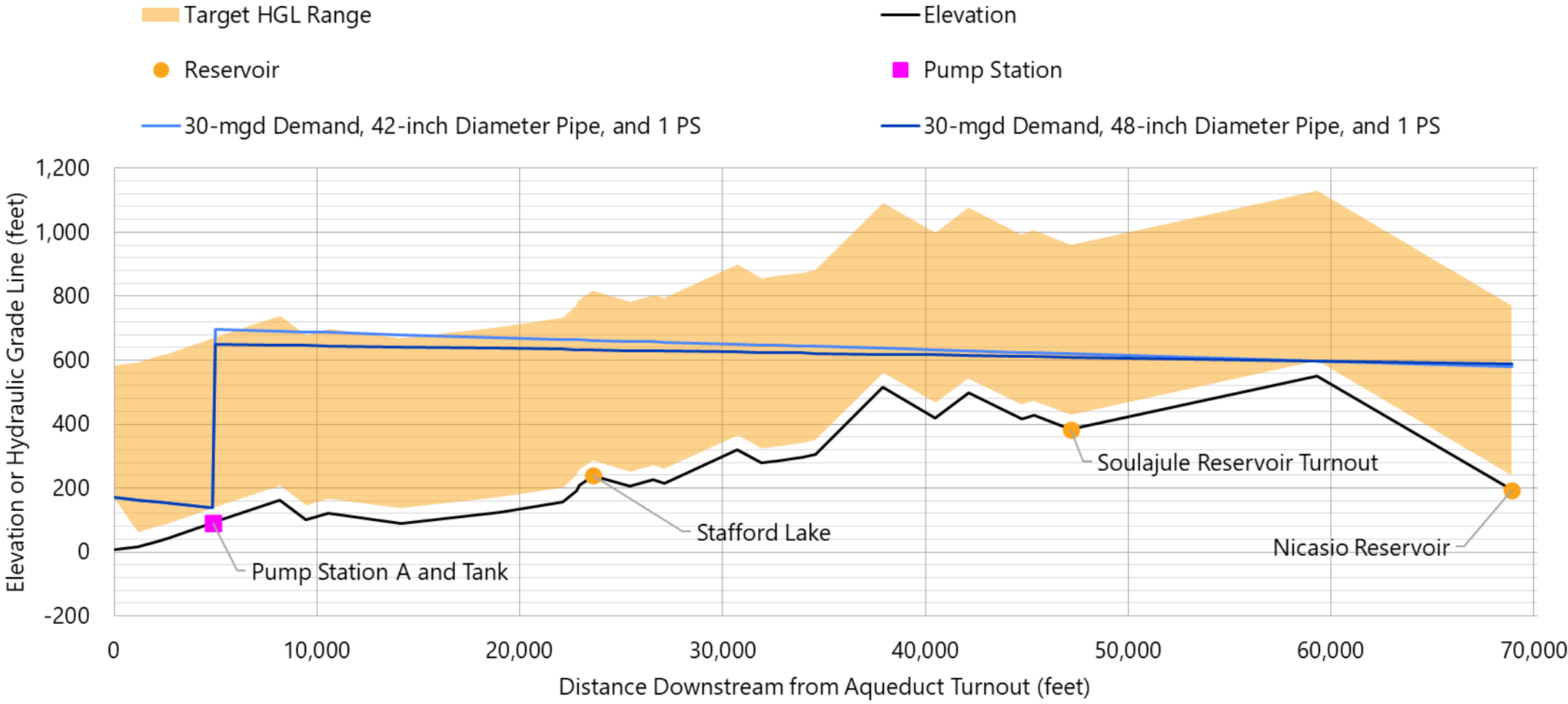
42-inch
Pipe

Scenario	Pipe Diameter (inches)	Pump Station A TDH (feet)	Pump Station B TDH (feet)	Nicasio Reservoir Demand (mgd)	Maximum Pressure (psi)
9	42	473	N/A	10.4	225
10	42	487	N/A	15.0	230



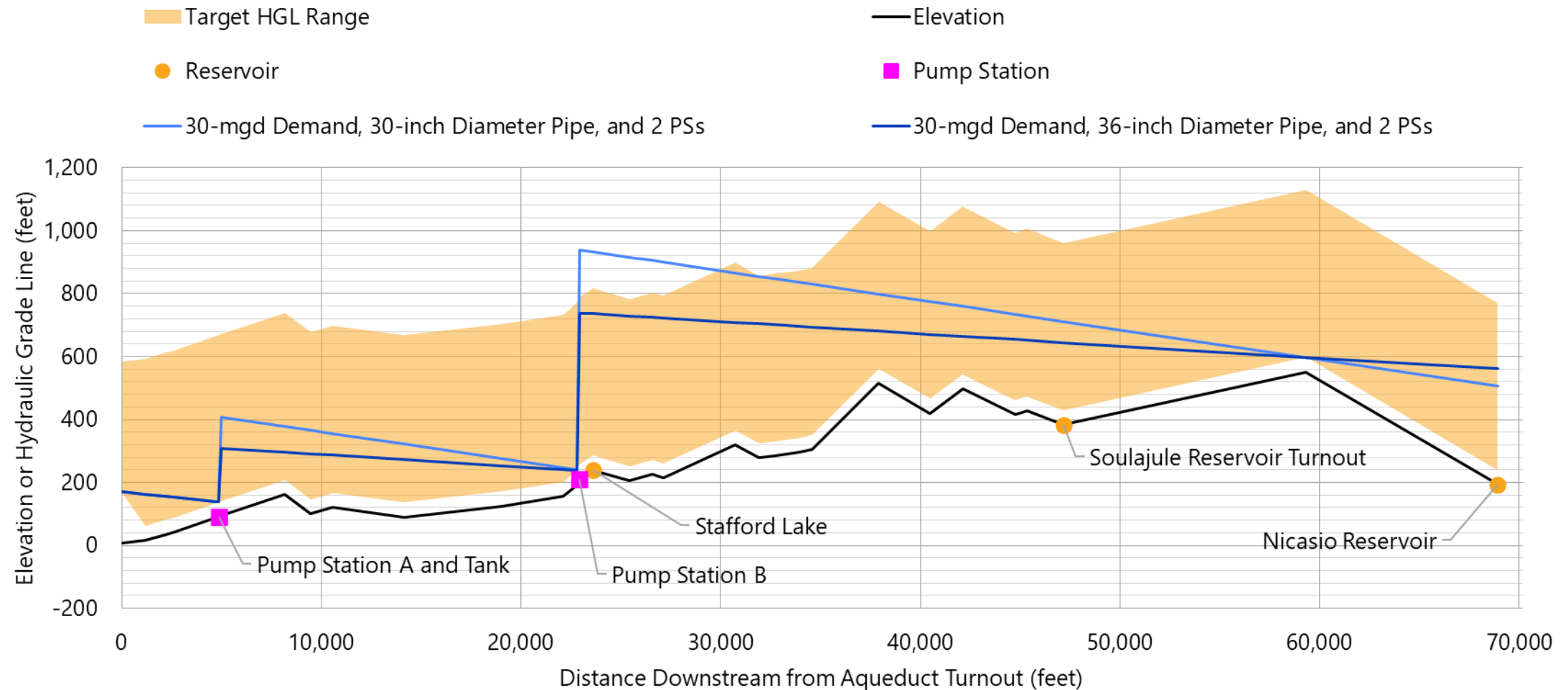
30-mgd demand, 1 PS

Scenario	Pipe Diameter (inches)	Pump Station A TDH (feet)	Pump Station B TDH (feet)	Nicasio Reservoir Demand (mgd)	Maximum Pressure (psi)
15	42	558	N/A	30.0	260
16	48	551	N/A	30.0	240



30-mgd demand, 2 PSs

Scenario	Pipe Diameter (inches)	Pump Station A TDH (feet)	Pump Station B TDH (feet)	Nicasio Reservoir Demand (mgd)	Maximum Pressure (psi)
13	30	271	701	30.0	316
14	36	170	500	30.0	229



Potentially feasible PETA-3 configurations

Scenario	Pipe Diameter (inches)	Pump Station A TDH (feet)	Pump Station B TDH (feet)	Nicasio Reservoir Demand (mgd)	Maximum Pressure (psi)
1	24	170	503	10.4	230
2	24	239	641	15.0	289
3	27	140	440	10.4	203
4	27	178	518	15.0	236
5*	30	531	Off	10.4	249
6	30	601	Off	15.0	279
7	30	123	408	10.4	189
8	30	147	454	15.0	209
9	36	488	Off	10.4	230
10	36	601	Off	15.0	243
11	42	473	Off	10.4	225
12	42	487	Off	15.0	230
13	30	558	Off	30.0	260
14	36	511	Off	30.0	240
15	42	271	701	30.0	316
16	48	170	500	30.0	229

* Scenario 5 is potentially feasible if the portion upstream of PS A is upsized to at least 36 inches in diameter.

Summary of potentially feasible PETA-3 alternatives

Pipe Diameter	Pump Station Capacity (mgd)	Pump Station A TDH (feet)	Pump Station B TDH (feet)	Pump Station A Required Power (hp)	Pump Station B Required Power (hp)	Total Annual Energy Usage (kWh)	Total Annual Energy Cost
24	10.4	158	502	388	1,148	4.1M	\$1.5M
27	10.4	119	440	319	1,004	3.6M	\$1.3M
27	15.0	168	518	586	1,705	6.1M	\$2.3M
30	10.4	531	N/A	1,212	N/A	3.3M	\$1.2M
30	10.4	99	407	281	931	3.3M	\$1.2M
30	15.0	128	454	484	1,494	5.3M	\$2.0M
36	10.4	488	N/A	1,114	N/A	3.0M	\$1.1M
36	15.0	518	N/A	1,705	N/A	4.6M	\$1.7M
42	10.4	473	N/A	1,079	N/A	2.9M	\$1.1M
42	15.0	487	N/A	1,603	N/A	4.3M	\$1.6M
36	30.0	170	500	1,119	3,291	11.8M	\$4.4M
48	30.0	511	N/A	3,363	N/A	9.0M	\$3.3M

Pumping Assumptions:

- Efficiency = 80%
- Annual operational days = 150 days/year
- Electricity cost = \$0.37/kWh

PETA-4 Hydraulic Analysis

PETA-4 scenarios

Scenario	Pipe Diameter (inches)	Pump Station A Status	Pump Station B Status	Nicasio Reservoir Demand (mgd)	HGL at North Marin Aqueduct Turnout (feet)
1	30	On	On	30.0	190
2	36	On	On	30.0	190
3	42	On	Off	30.0	190
4	42	On	On	30.0	190

Assumed Pump Station Locations



Assumed Pump Station Locations

Pump Station A and Tank



Pump Station B

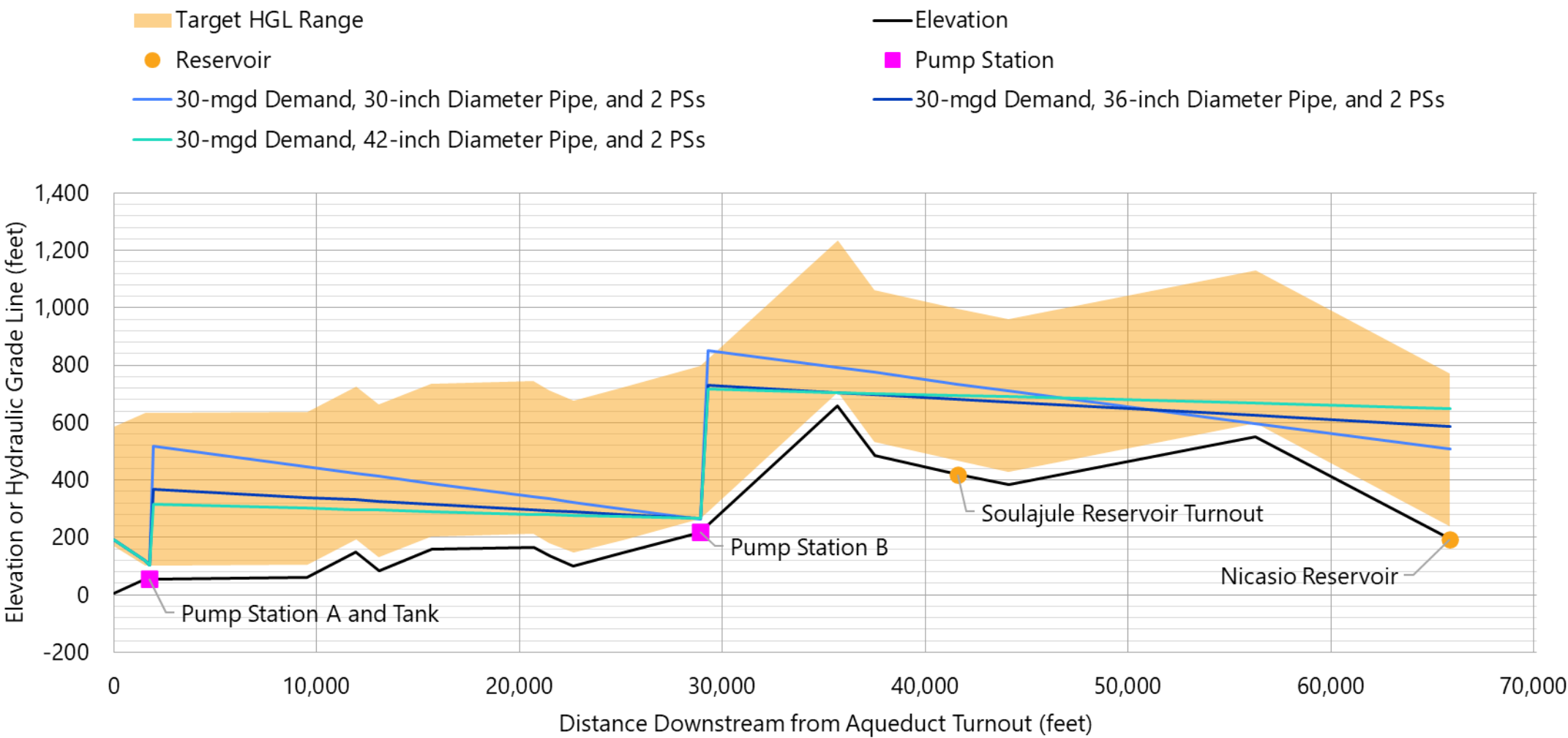


Assumed Tank Parameters

Parameter	Assumption
Location	Directly upstream of PS A, 1,600 lf downstream from Petaluma/North Marin Aqueduct turnout
Base elevation	90 feet
Minimum operating level	15 feet (to maintain minimum 20 psi pressure)
Assumed level for model analysis	15 feet

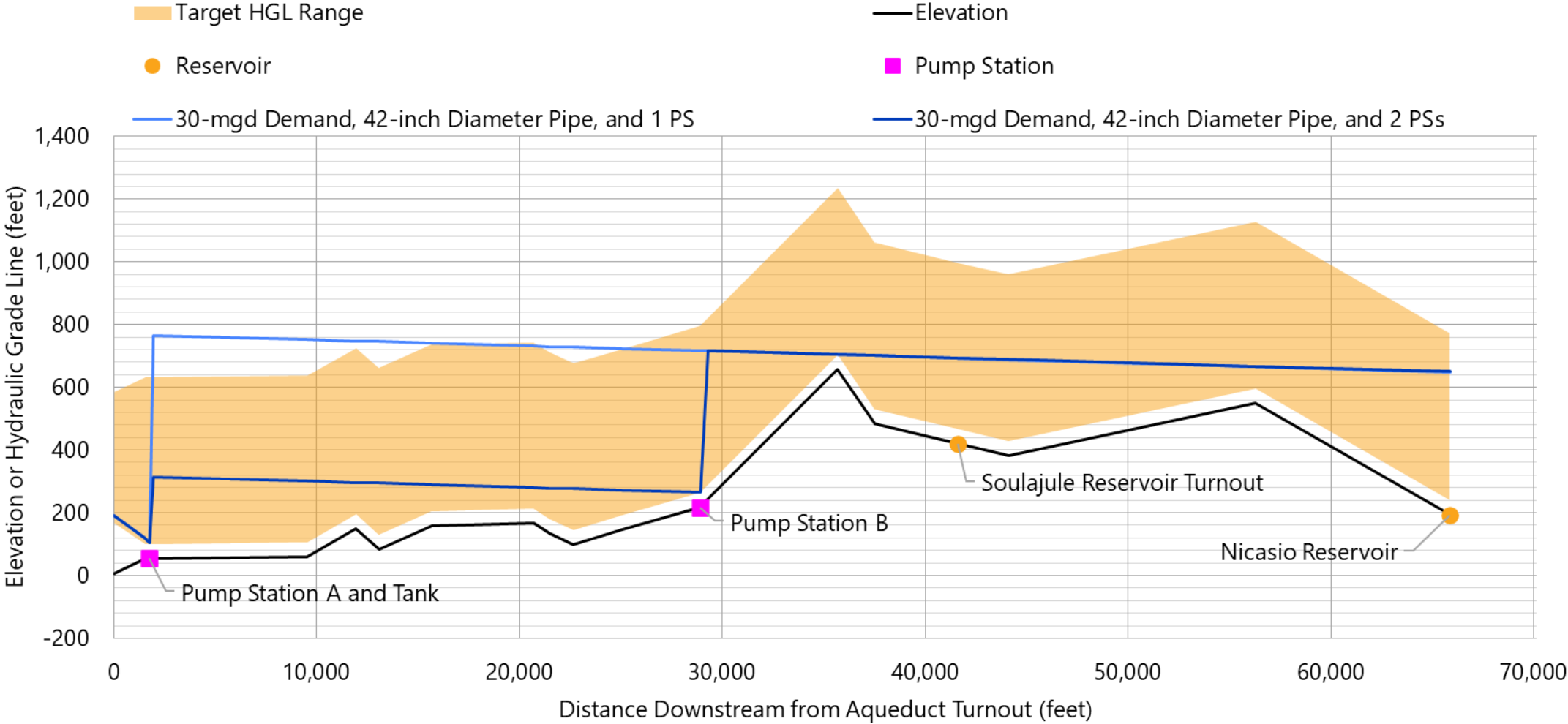
2 Pump Stations, 30- to 42-inch diameter

Scenario	Pipe Diameter (inches)	Pump Station A TDH (feet)	Pump Station B TDH (feet)	Maximum Pressure (psi)
1	30	419	592	263
2	36	267	465	210
4	42	211	452	204



42-inch diameter, 1 to 2 pump stations

Scenario	Pipe Diameter (inches)	Pump Station A TDH (feet)	Pump Station B TDH (feet)	Maximum Pressure (psi)
3	42	662	N/A	308
4	42	211	452	204



Potentially feasible PETA-4 configurations

Scenario	Pipe Diameter (inches)	Pump Station A Status	Pump Station B Status	Nicasio Reservoir Demand (mgd)	Maximum Pressure (psi)
1	30	On	On	30.0	263
2	36	On	On	30.0	210
3	42	On	Off	30.0	308
4	42	On	On	30.0	204

Summary of potentially feasible PETA-4 alternatives

Pipe Diameter	Pump Station Capacity (mgd)	Pump Station A TDH (feet)	Pump Station B TDH (feet)	Pump Station A Required Power (hp)	Pump Station B Required Power (hp)	Total Annual Energy Usage (kWh)	Total Annual Energy Cost
36	30.0	267	463	1,757	3,048	12.9M	\$4.8M
42	30.0	211	452	1,389	2,975	11.7M	\$4.3M

Pumping Assumptions:

- Efficiency = 80%
- Annual operational days = 150 days/year
- Electricity cost = \$0.37/kWh

| Summary of Potentially Feasible Alternatives

Summary of potentially feasible alternatives

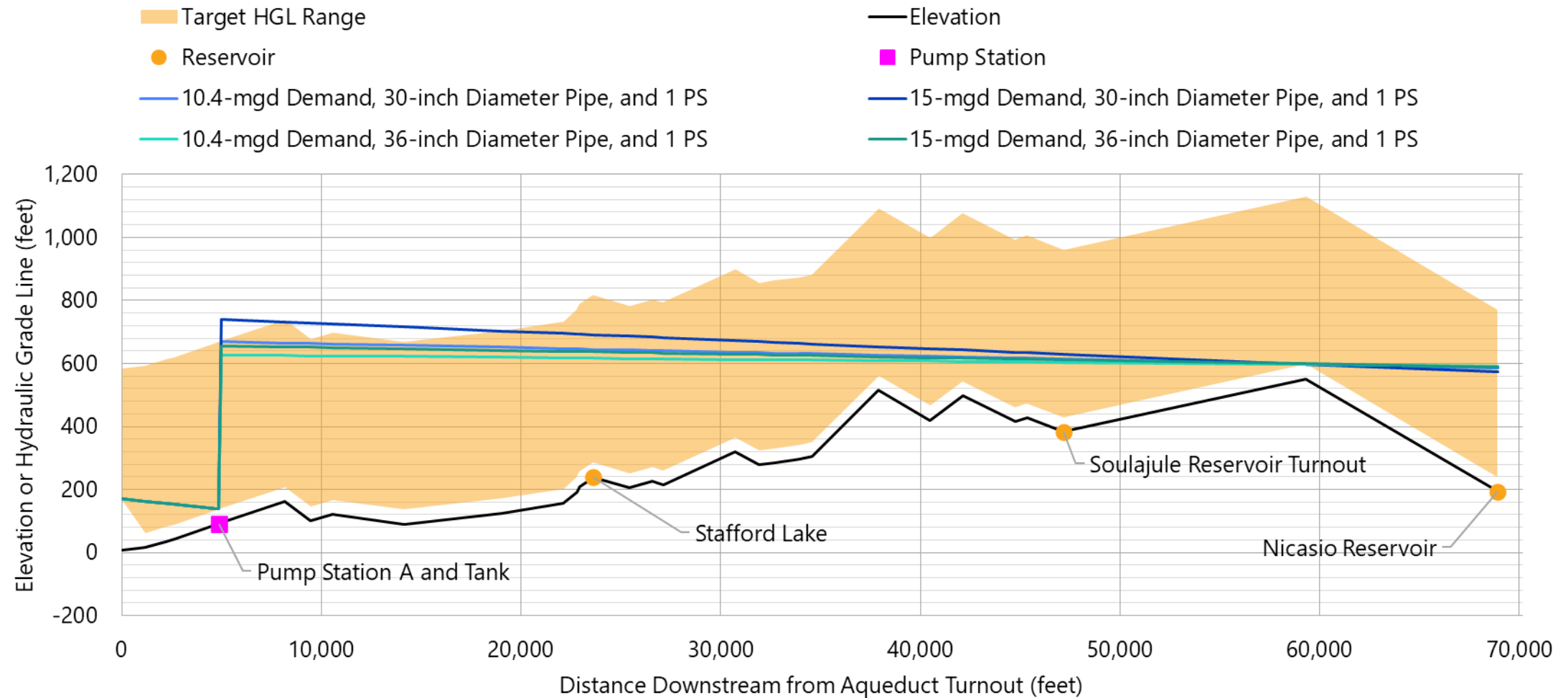
Alignment	Pipe Diameter	Pump Station Capacity (mgd)	Pump Station A TDH (feet)	Pump Station B TDH (feet)	Pump Station A Required Power (hp)	Pump Station B Required Power (hp)	Total Annual Energy Usage (kWh)	Total Annual Energy Cost
COTATI-3	36	30.0	252	512	1,659	3,370	13.5M	\$5.0M
COTATI-3	42	30.0	153	440	1,007	2,896	10.5M	\$3.9M
PETA-3	24	10.4	158	502	388	1,148	4.1M	\$1.5M
PETA-3	27	10.4	119	440	319	1,004	3.6M	\$1.3M
PETA-3	27	15.0	168	518	586	1,705	6.1M	\$2.3M
PETA-3	30	10.4	531	N/A	1,212	N/A	3.3M	\$1.2M
PETA-3	30	10.4	99	407	281	931	3.3M	\$1.2M
PETA-3	30	15.0	128	454	484	1,494	5.3M	\$2.0M
PETA-3	36	10.4	488	N/A	1,114	N/A	3.0M	\$1.1M
PETA-3	36	15.0	518	N/A	1,705	N/A	4.6M	\$1.7M
PETA-3	42	10.4	473	N/A	1,079	N/A	2.9M	\$1.1M
PETA-3	42	15.0	487	N/A	1,603	N/A	4.3M	\$1.6M
PETA-3	36	30.0	170	500	1,119	3,291	11.8M	\$4.4M
PETA-3	42	30.0	511	N/A	3,363	N/A	9.0M	\$3.3M
PETA-4	36	30.0	267	463	1,757	3,048	12.9M	\$4.8M
PETA-4	42	30.0	211	452	1,389	2,975	11.7M	\$4.3M

Pumping Assumptions:

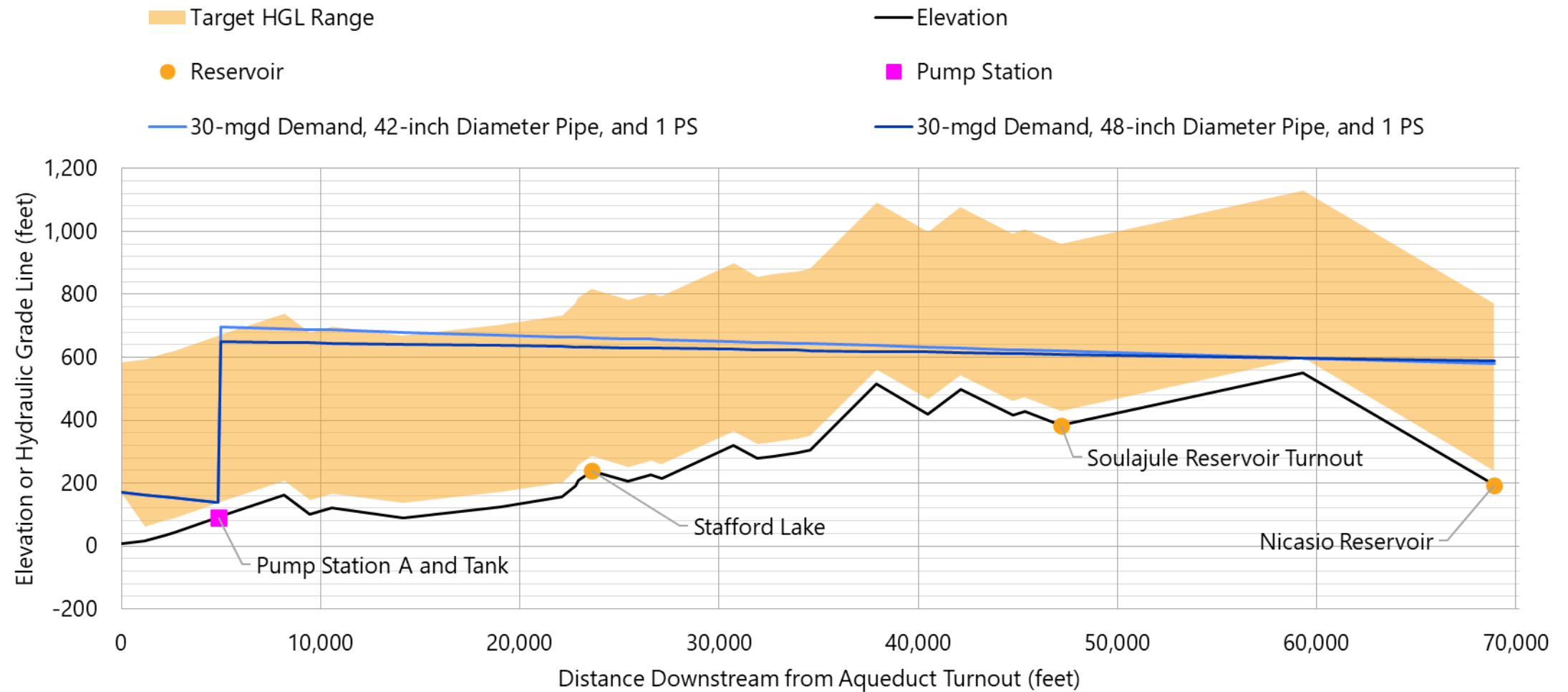
- Efficiency = 80%
- Annual operational days = 150 days/year
- Electricity cost = \$0.37/kWh

| Additional Figures

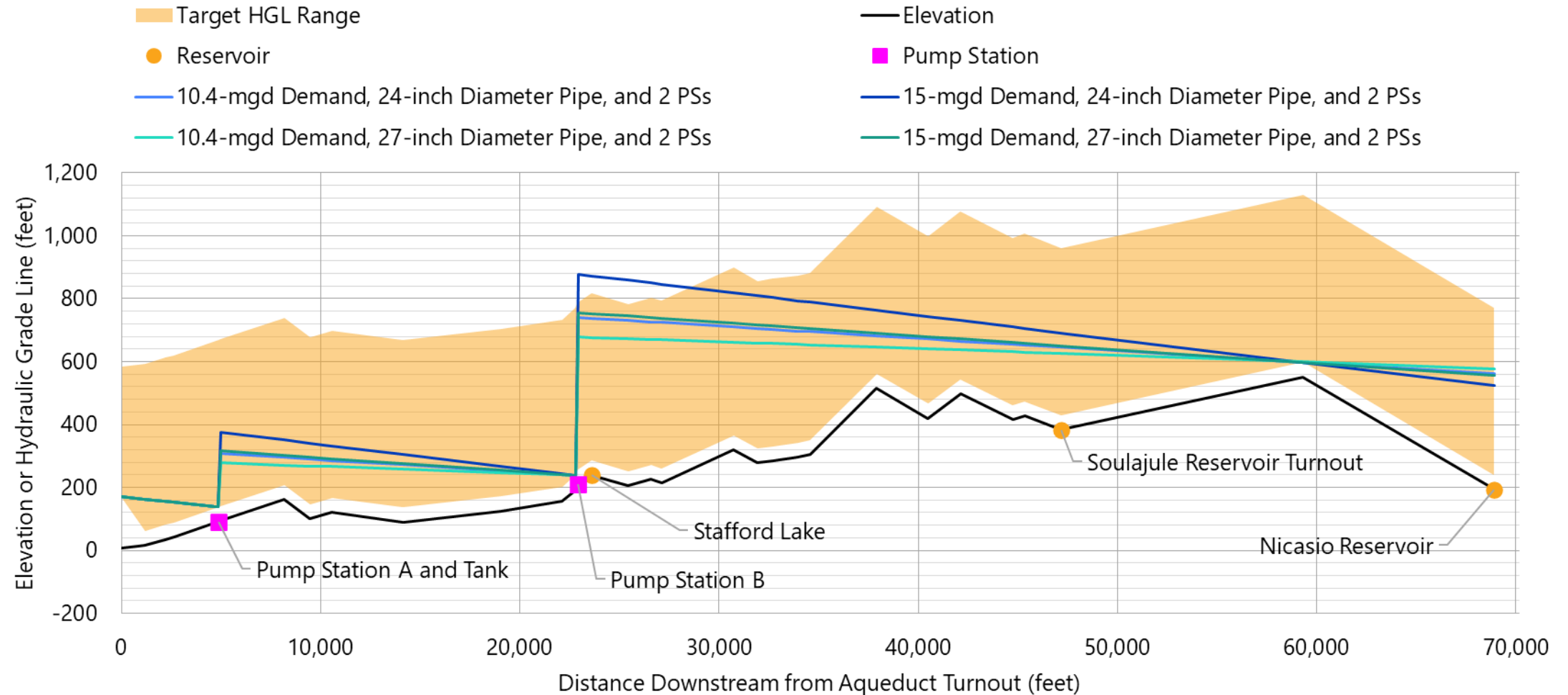
PETA-3, Single Pump Station



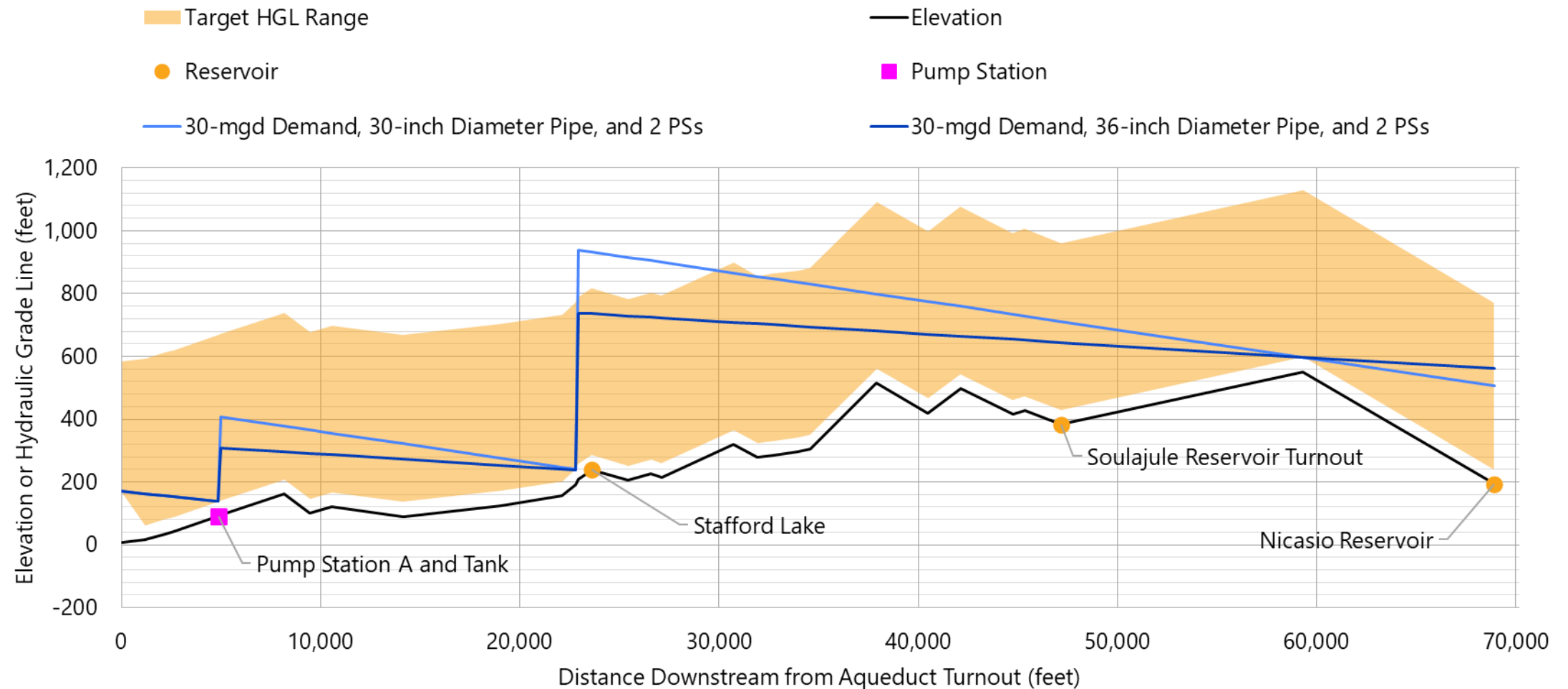
PETA-3, Single Pump Station, 30-mgd Demand



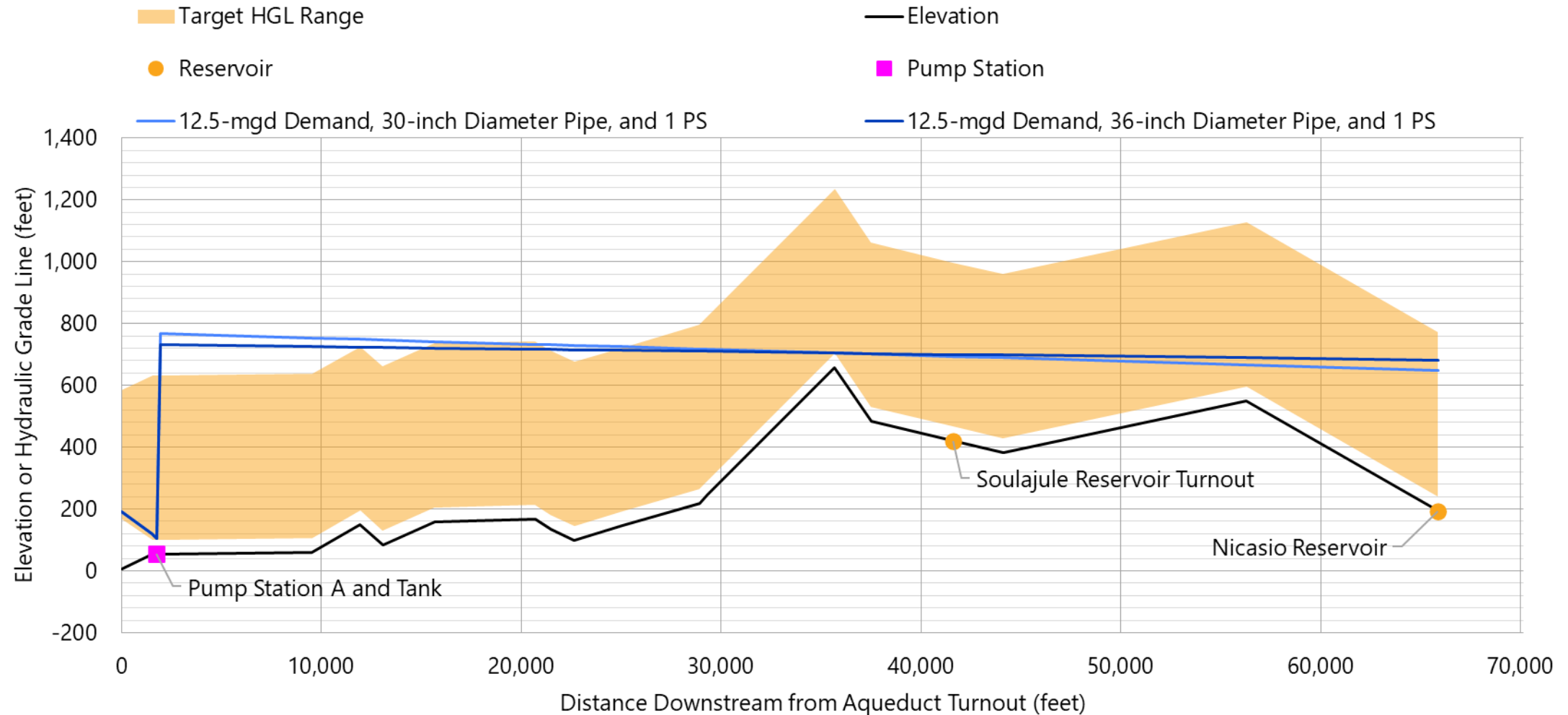
PETA-3, Dual Pump Station



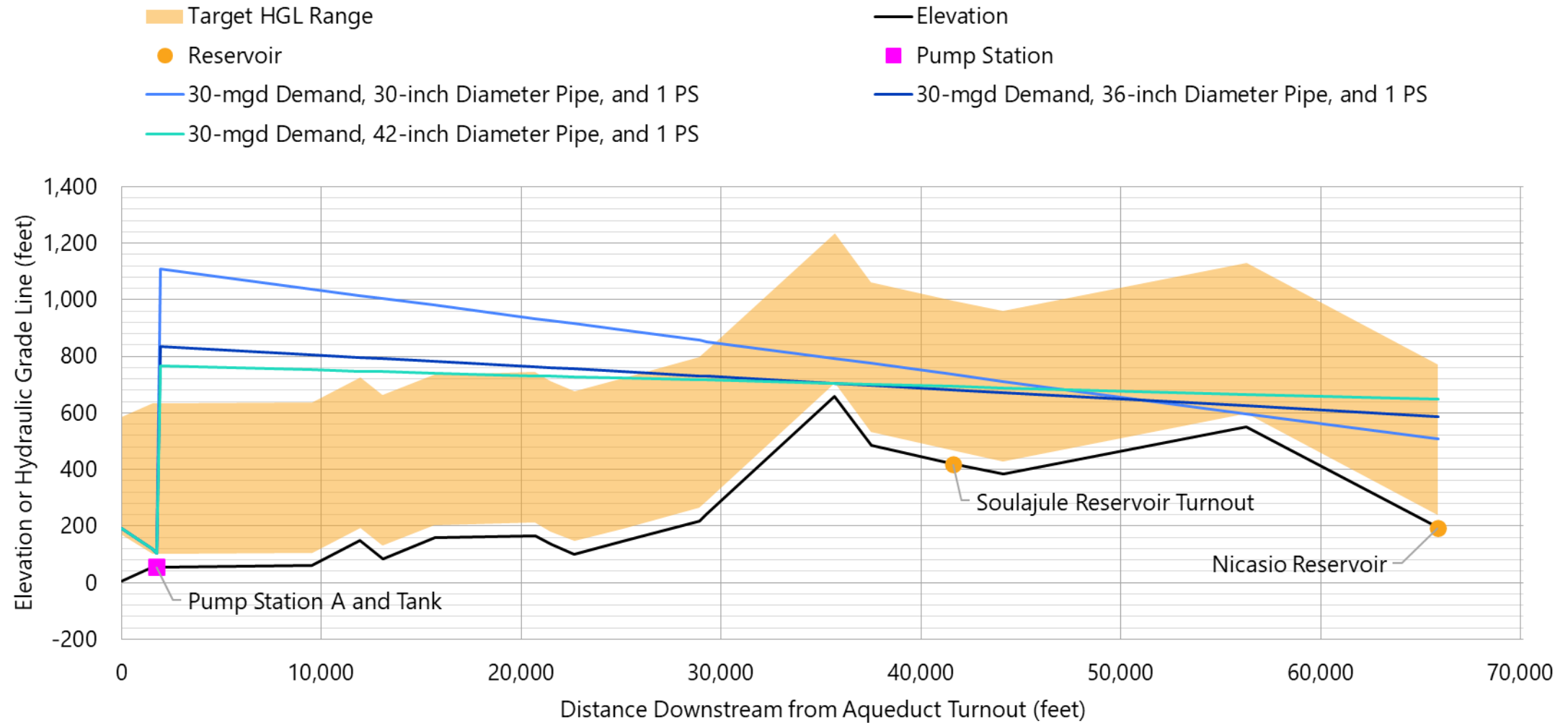
PETA-3, Dual Pump Station, 30-mgd Demand



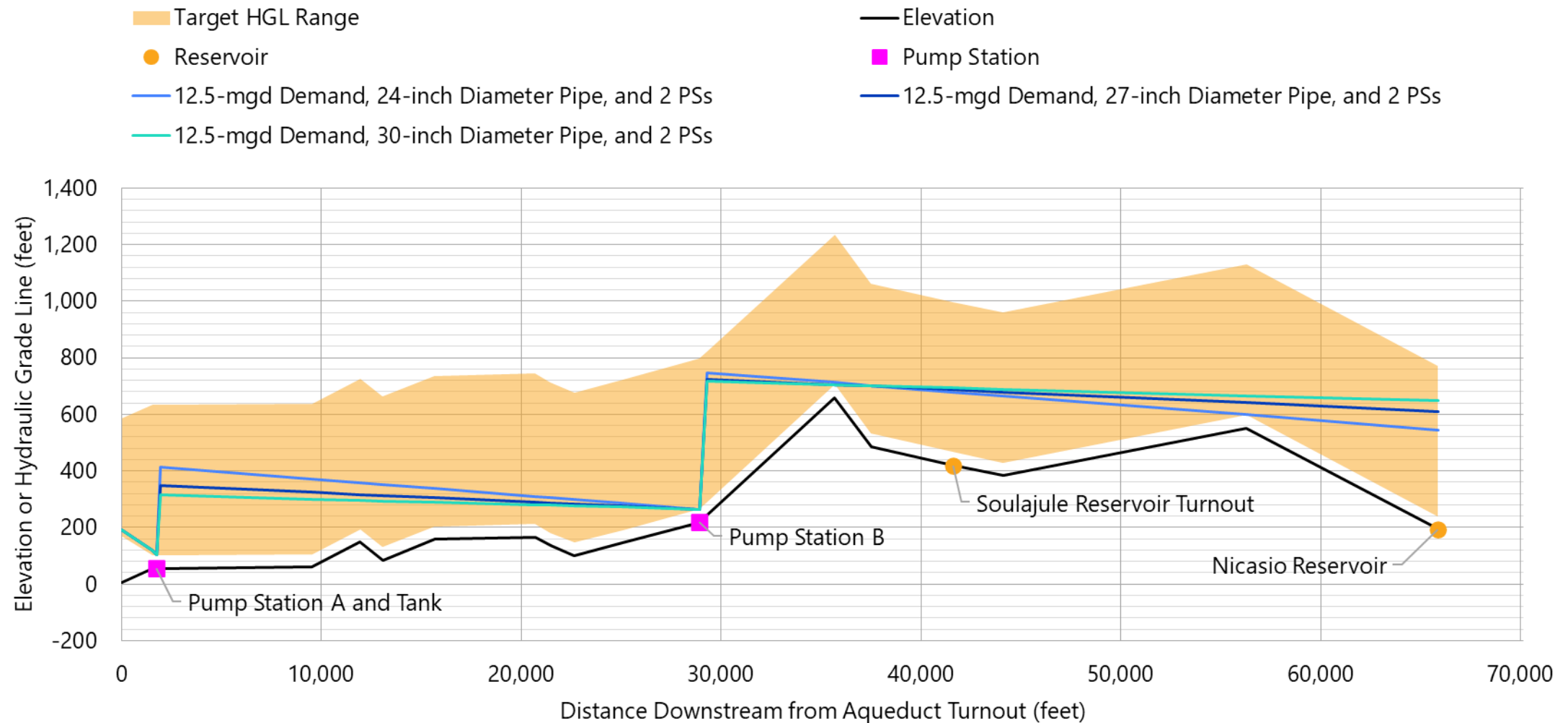
PETA-4, Single Pump Station, Demand = 12.5 mgd



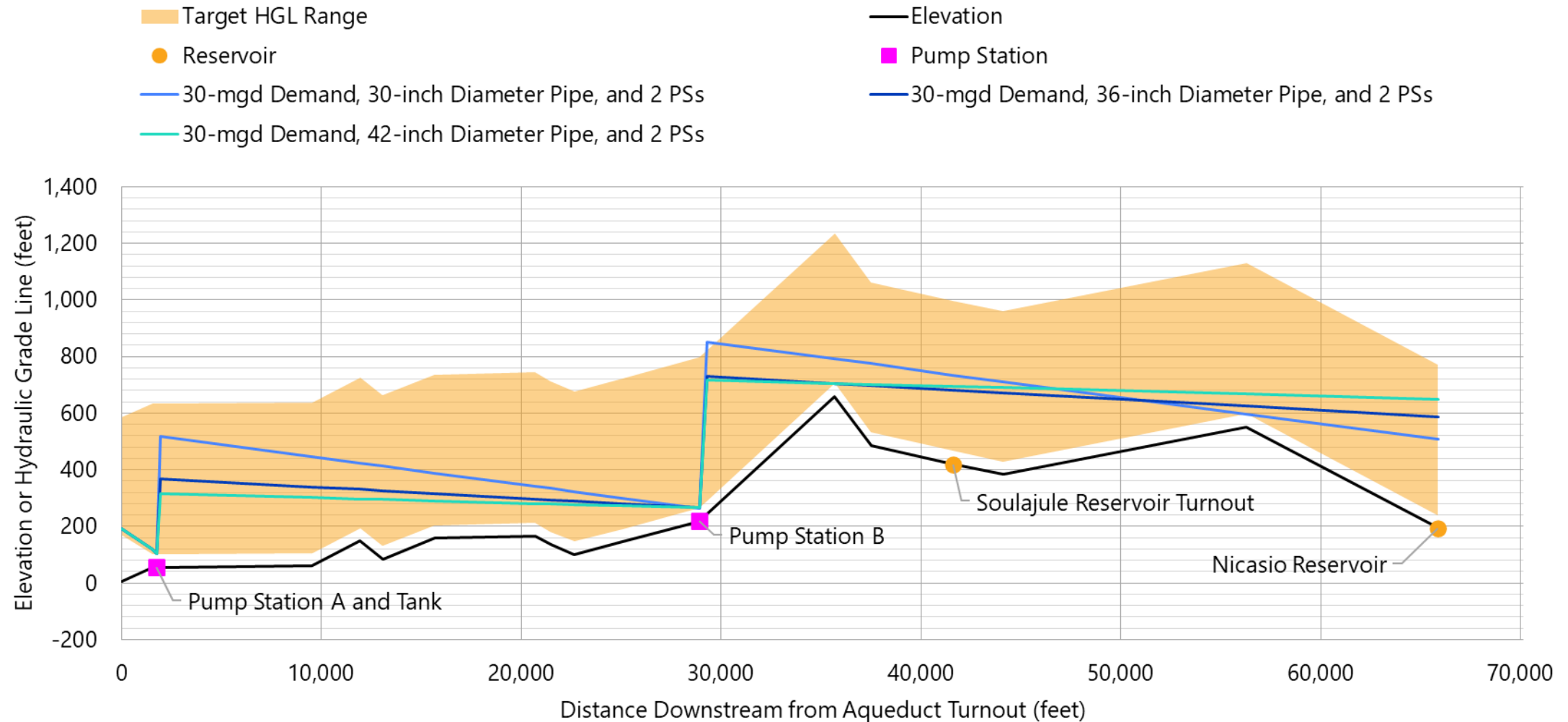
PETA-4, Single Pump Station, Demand = 30.0 mgd



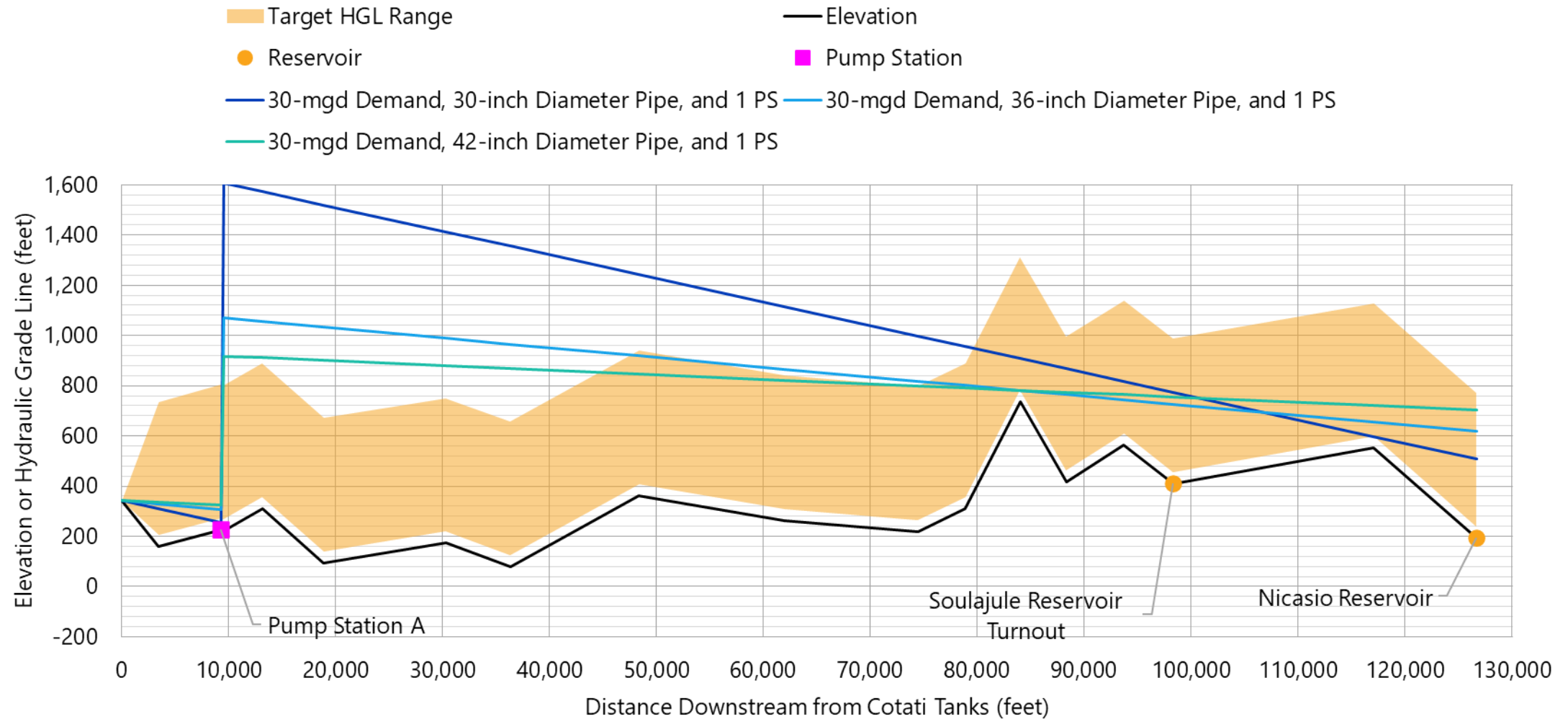
PETA-4, Dual Pump Station, Demand = 12.5 mgd



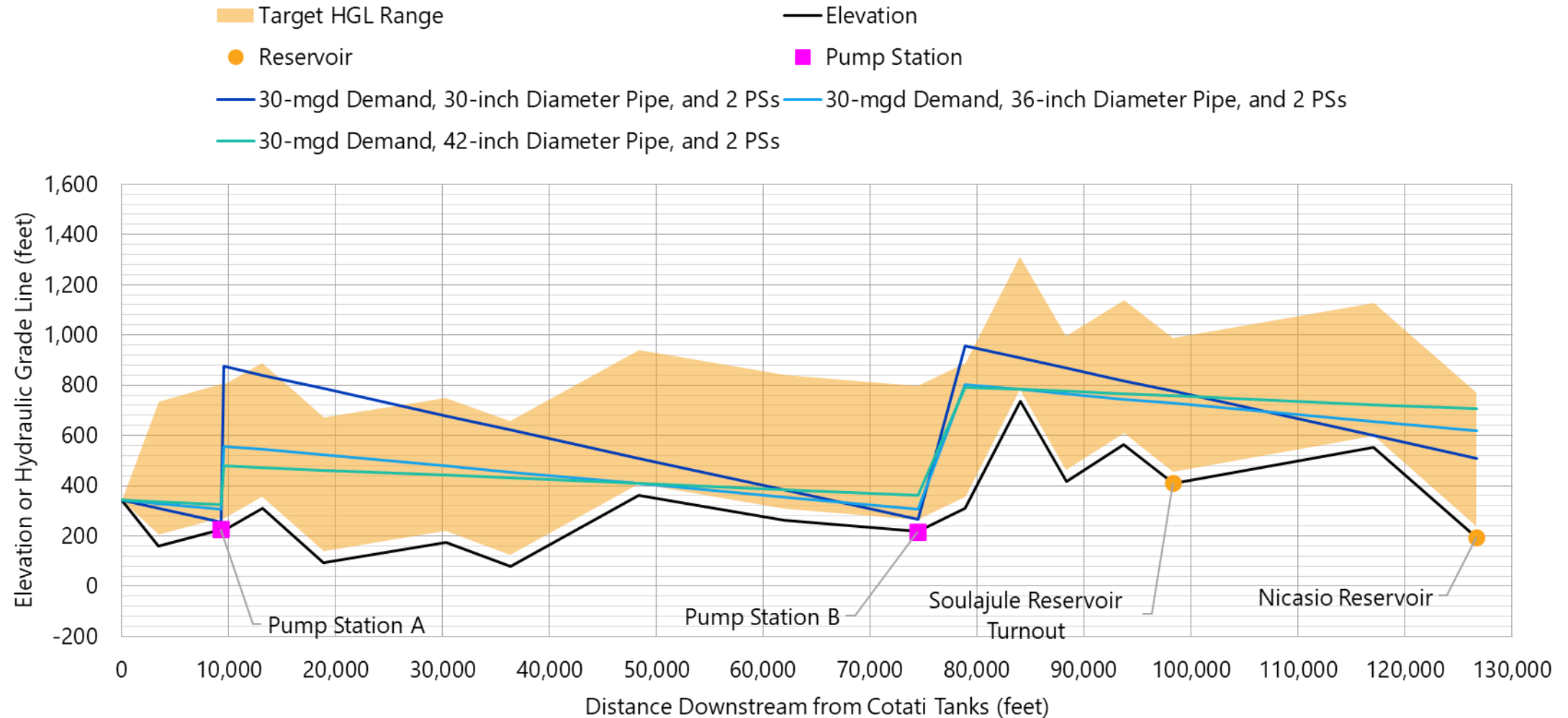
PETA-4, Dual Pump Station, Demand = 30.0 mgd



COTATI-3, Single Pump Station, Demand = 30.0 mgd



COTATI-3, Dual Pump Station, Demand = 30.0 mgd



APPENDIX 3E

BUDGETARY COST ESTIMATES

PROJECT SUMMARY

Project: Water Supply Conveyance Improvements Project
Client: Marin Municipal Water District
Location: Marin/Sonoma County
Zip Code: 94952
Carollo Job # 202554
Project Alt: PETA-3: North Marin Aqueduct to Nicasio and Soulajule Reservoir without Upstream Improvements

Estimate Class: 4
PIC: TT
PM: DGB
Date: October 25, 2024
By: AC
Reviewed: DGB

NO.	DESCRIPTION	TOTAL
01	General Conditions	\$5,198,575
02	Pipeline	\$71,560,000
03	Pump Station	\$18,000,000
04	Power Supply	\$850,000
05	Ancillary Items	\$13,561,500
TOTAL DIRECT COST		\$109,170,075
	Contingency 30.0%	\$32,751,023
	Subtotal	\$141,921,098
	General Contractor Overhead, Profit & Risk 12.0%	\$17,030,532
	Subtotal	\$158,951,629
	Escalation to Mid-Point (Assumes Mid-Point in Apr. 2028) 3.0%	\$17,882,058
	Subtotal	\$176,833,687
	Sales Tax (8.25% on half the direct cost) 8.3%	\$7,294,390
	Subtotal	\$184,128,077
	Bid Market Allowance 0.0%	\$0
TOTAL ESTIMATED CONSTRUCTION COST		\$184,128,077
	Engineering, Legal & Administration Fees 15.0%	\$27,619,212
	Owner's Reserve for Change Orders 5.0%	\$9,206,404
TOTAL ESTIMATED PROJECT COST		\$220,953,693

The cost estimate herein is based on our perception of current conditions at the project location. This estimate reflects our professional opinion of accurate costs at this time and is subject to change as the project design matures. Carollo Engineers have no control over variances in the cost of labor, materials, equipment; nor services provided by others, contractor's means and methods of executing the work or of determining prices, competitive bidding or market conditions, practices or bidding strategies. Carollo Engineers cannot and does not warrant or guarantee that proposals, bids or actual construction costs will not vary from the costs presented as shown.

PROJECT SUMMARY

Project: Water Supply Conveyance Improvements Project
Client: Marin Municipal Water District
Location: Marin/Sonoma County
Zip Code: 94952
Carollo Job # 202554
Project Alt: PETA-3: North Marin Aqueduct to Nicasio and SoulaJule Reservoir with Upstream Improvements

Estimate Class: 4
PIC: TT
PM: DGB
Date: October 25, 2024
By: AC
Reviewed: DGB

NO.	DESCRIPTION	TOTAL
01	General Conditions	\$10,506,975
02	Pipeline	\$71,560,000
03	Pump Station	\$33,500,000
04	Kastania Pump Station Improvements	\$10,000,000
05	STS Alternative	\$53,630,000
06	North Marin Aqueduct Parallel Pipeline	\$12,990,000
07	Power Supply	\$1,050,000
08	Ancillary Items	\$27,409,500
TOTAL DIRECT COST		\$220,646,475
	Contingency 30.0%	\$66,193,943
	Subtotal	\$286,840,418
	General Contractor Overhead, Profit & Risk 12.0%	\$34,420,850
	Subtotal	\$321,261,268
	Escalation to Mid-Point (Assumes Mid-Point in Apr. 2028) 3.0%	\$36,141,893
	Subtotal	\$357,403,160
	Sales Tax (8.25% on half the direct cost) 8.3%	\$14,742,880
	Subtotal	\$372,146,041
	Bid Market Allowance 0.0%	\$0
TOTAL ESTIMATED CONSTRUCTION COST		\$372,146,041
	Engineering, Legal & Administration Fees 15.0%	\$55,821,906
	Owner's Reserve for Change Orders 5.0%	\$18,607,302
TOTAL ESTIMATED PROJECT COST		\$446,575,249

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PROJECT SUMMARY

Project: Water Supply Conveyance Improvements Project
Client: Marin Municipal Water District
Location: Marin/Sonoma County
Zip Code: 94952

Estimate Class: 4
PIC: TT
PM: DGB
Date: October 25, 2024
By: AC

Carollo Job # 202554 **Reviewed:** DGB
Project Alt: PETA-4: North Marin Aqueduct to Nicasio and SoulaJule Reservoir via San Antonio Rd without Upstream

NO.	DESCRIPTION	TOTAL
01	General Conditions	\$6,096,150
02	Pipeline	\$64,680,000
03	Pump Stations	\$31,500,000
04	Power Supply	\$9,840,000
05	Ancillary Items	\$15,903,000
TOTAL DIRECT COST		\$128,019,150
	Contingency 30.0%	\$38,405,745
	Subtotal	\$166,424,895
	General Contractor Overhead, Profit & Risk 12.0%	\$19,970,987
	Subtotal	\$186,395,882
	Escalation to Mid-Point (Assumes Mid-Point in Apr. 2028) 3.0%	\$20,969,537
	Subtotal	\$207,365,419
	Sales Tax (8.25% on half the direct cost) 8.3%	\$8,553,824
	Subtotal	\$215,919,243
	Bid Market Allowance 0.0%	\$0
TOTAL ESTIMATED CONSTRUCTION COST		\$215,919,243
	Engineering, Legal & Administration Fees 15.0%	\$32,387,886
	Owner's Reserve for Change Orders 5.0%	\$10,795,962
TOTAL ESTIMATED PROJECT COST		\$259,103,091

The cost estimate herein is based on our perception of current conditions at the project location. This estimate reflects our professional opinion of accurate costs at this time and is subject to change as the project design matures. Carollo Engineers have no control over variances in the cost of labor, materials, equipment; nor services provided by others, contractor's means and methods of executing the work or of determining prices, competitive bidding or market conditions, practices or bidding strategies. Carollo Engineers cannot and does not warrant or guarantee that proposals, bids or actual construction costs will not vary from the costs presented as shown.



PROJECT SUMMARY

Project: Water Supply Conveyance Improvements Project
Client: Marin Municipal Water District
Location: Marin/Sonoma County
Zip Code: 94952

Estimate Class: 4
PIC: TT
PM: DGB
Date: October 25, 2024
By: AC

Carollo Job # 202554 **Reviewed:** DGB
Project Alt: PETA-4: North Marin Aqueduct to Nicasio and Soulajule Reservoir via San Antonio Rd with Upstream Improvements

NO.	DESCRIPTION	TOTAL
01	General Conditions	\$10,272,375
02	Pipeline	\$64,680,000
03	Pump Stations	\$40,500,000
04	Kastania Pump Station Improvements	\$10,000,000
05	STS Alternative	\$53,630,000
06	Power Supply	\$9,840,000
07	Ancillary Items	\$26,797,500
TOTAL DIRECT COST		\$215,719,875
	Contingency 30.0%	\$64,715,963
	Subtotal	\$280,435,838
	General Contractor Overhead, Profit & Risk 12.0%	\$33,652,301
	Subtotal	\$314,088,138
	Escalation to Mid-Point (Assumes Mid-Point in Apr. 2028) 3.0%	\$35,334,916
	Subtotal	\$349,423,054
	Sales Tax (8.25% on half the direct cost) 8.3%	\$14,413,701
	Subtotal	\$363,836,754
	Bid Market Allowance 0.0%	\$0
TOTAL ESTIMATED CONSTRUCTION COST		\$363,836,754
	Engineering, Legal & Administration Fees 15.0%	\$54,575,513
	Owner's Reserve for Change Orders 5.0%	\$18,191,838
TOTAL ESTIMATED PROJECT COST		\$436,604,105

The cost estimate herein is based on our perception of current conditions at the project location. This estimate reflects our professional opinion of accurate costs at this time and is subject to change as the project design matures. Carollo Engineers have no control over variances in the cost of labor, materials, equipment; nor services provided by others, contractor's means and methods of executing the work or of determining prices, competitive bidding or market conditions, practices or bidding strategies. Carollo Engineers cannot and does not warrant or guarantee that proposals, bids or actual construction costs will not vary from the costs presented as shown.



PROJECT SUMMARY

Project: Water Supply Conveyance Improvements Project
Client: Marin Municipal Water District
Location: Marin/Sonoma County
Zip Code: 94952

Estimate Class: 4
PIC: TT
PM: DGB
Date: October 25, 2024
By: AC

Carollo Job # 202554
Project Alt: COTATI-3: Cotati Tanks to Nicasio and Soulajule Reservoir

Reviewed: DGB

NO.	DESCRIPTION	TOTAL
01	General Conditions	\$9,450,700
02	Pipeline	\$108,420,000
03	Pump Station	\$49,500,000
04	Power Supply	\$6,440,000
05	Ancillary Items	\$24,654,000
TOTAL DIRECT COST		\$198,464,700
	Contingency 30.0%	\$59,539,410
	Subtotal	\$258,004,110
	General Contractor Overhead, Profit & Risk 12.0%	\$30,960,493
	Subtotal	\$288,964,603
	Escalation to Mid-Point (Assumes Mid-Point in Apr. 2028) 3.0%	\$32,508,518
	Subtotal	\$321,473,121
	Sales Tax (8.25% on half the direct cost) 8.3%	\$13,260,766
	Subtotal	\$334,733,887
	Bid Market Allowance 0.0%	\$0
TOTAL ESTIMATED CONSTRUCTION COST		\$334,733,887
	Engineering, Legal & Administration Fees 15.0%	\$50,210,083
	Owner's Reserve for Change Orders 5.0%	\$16,736,694
TOTAL ESTIMATED PROJECT COST		\$401,680,665

The cost estimate herein is based on our perception of current conditions at the project location. This estimate reflects our professional opinion of accurate costs at this time and is subject to change as the project design matures. Carollo Engineers have no control over variances in the cost of labor, materials, equipment; nor services provided by others, contractor's means and methods of executing the work or of determining prices, competitive bidding or market conditions, practices or bidding strategies. Carollo Engineers cannot and does not warrant or guarantee that proposals, bids or actual construction costs will not vary from the costs presented as shown.

MMWD Water Supply Conveyance Improvements Project - PETA-3 Pipeline Sizing																		
Project Alternative	Segment	From	To	Length (miles)	Length (feet)	Flow (mgd)	Flow (gpm)	Flow (cfs)	Max Design Velocity (ft/sec)	Minimum Pipe Diameter (in)	Design Pipe Diameter (in)	Actual Pipe Velocity (ft/sec)	Hazen Williams C factor ⁽¹⁾	Friction Headloss (ft) ⁽²⁾	Minor Losses (ft) ⁽³⁾	Starting Elevation	Max Elevation	Finish Elevation
PETA-3	1	North Marin Aqueduct	Pump Station A	0.8	3,960	30	20833.33	46.42	7	34.87	36	6.57	135	12.38	1.24	36	60	60
	2	Pump Station A	Pump Station B	3.6	19,008	30	20833.33	46.42	7	34.87	36	6.57	135	59.43	5.94	60	200	200
	3A	Pump Station B	Segment High Point	7.0	37,171	30	20833.33	46.42	7	34.87	36	6.57	135	116.23	11.62	200	551	551
	3B	Segment High Point	Nicasio Reservoir	1.8	9,504	30	20833.33	46.42	7	34.87	36	6.57	135	29.72	2.97	551	551	169

(1) - Assumes a C Factor for Cement Mortar Lined Welded Steel Pipe

(2) - Headloss Calculated Using Hazen-Williams Equation

10.44 * L * Q^{1.85}

h_f =

1.85

C

* d^{4.8655}

(3) - Minor losses assumed to be 10% of friction headloss

PETA-3 Alternative Pump Stations	
PETA-3: Pump Station A	
Friction head loss	65.4 ft
Pressure at High Point	46.2 ft
Elevation head	140 ft
Suction head	55 ft
Total Dynamic Head	196.8 ft
TDH to HP (equation 1) whp = gpm x TDH (ft) 3,960	
Flow	20840.00 gpm
TDH	196.8 ft
whp	1035.67 hp
bhp	1380.89 hp
Rounded bhp	600 hp

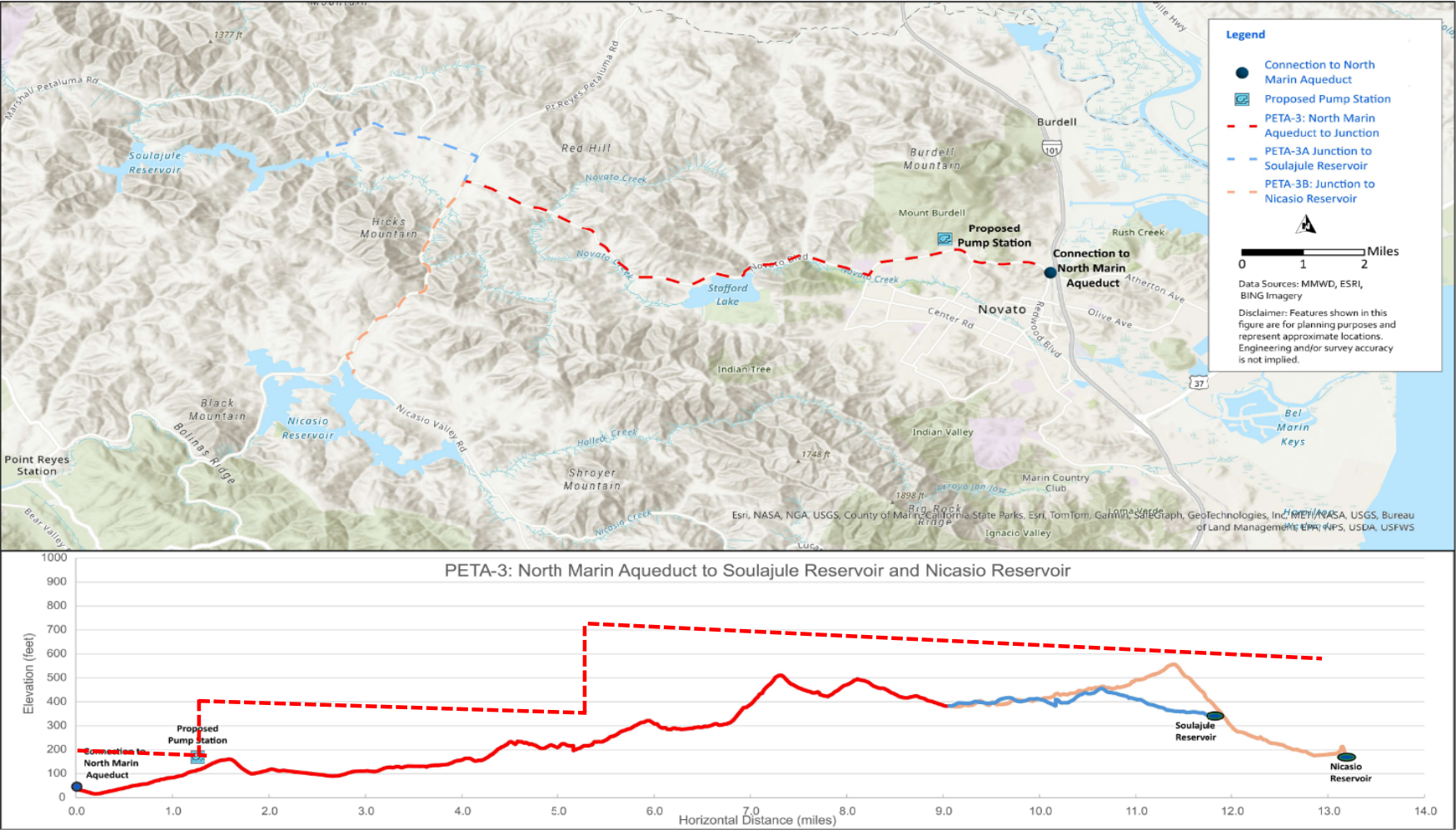
----- Includes minor losses

----- Minimum pressure at high point

----- Assuming 75% pump efficiency

PETA-3: Pump Station B	
Friction head loss	127.8 ft
Pressure at High Point	46.2 ft
Elevation head	351 ft
Suction Head	30 ft
Total Dynamic Head	495.0 ft
TDH to HP (equation 1) whp = gpm x TDH (ft) 3,960	
Flow	20840.00 gpm
TDH	495.0 ft
whp	2605.26 hp
bhp	3473.67 hp
Rounded bhp	2000 hp

----- Assuming 75% pump efficiency



MMWD Water Supply Conveyance Improvements Project - PETA-4 Pipeline Sizing																		
Project Alternative	Segment	From	To	Length (miles)	Length (feet)	Flow (mgd)	Flow (gpm)	Flow (cfs)	Max Design Velocity (ft/sec)	Minimum Pipe Diameter (in)	Design Pipe Diameter (in)	Actual Pipe Velocity (ft/sec)	Hazen Williams C factor ⁽¹⁾	Friction Headloss (ft) ⁽²⁾	Minor Losses (ft) ⁽³⁾	Starting Elevation	Max Elevation	Finish Elevation
PETA-4	1	North Marin Aqueduct	Pump Station A	1.5	7,920	30	20833.33	46.42	7	34.87	36	6.57	135	24.76	2.48	30	50	50
	2	Pump Station A	Pump Station B	4.0	21,120	30	20833.33	46.42	7	34.87	36	6.57	135	66.04	6.60	50	300	300
	3A	Pump Station B	Segment High Point	1.1	5,808	30	20833.33	46.42	7	34.87	36	6.57	135	18.16	1.82	300	735	735
	3B	Segment High Point	Nicasio Reservoir	5.6	29,568	30	20833.33	46.42	7	34.87	36	6.57	135	92.45	9.25	735	735	330
	Added STS Alt for PETA-4 w/Imps	1	Cotati Tanks	Pump Station	4.5	23,760	17.1	11875.00	26.46	7	26.33	30	5.39	135	63.76	6.38	343	343
	2	Pump Station	Kastania Pump Station	6.0	31,680	17.1	11875.00	26.46	7	26.33	30	5.39	135	85.02	8.50	40	250	250

(1) - Assumes a C Factor for Cement Mortar Lined Welded Steel Pipe

(2) - Headloss Calculated Using Hazen-Williams Equation

10.44 * L * Q^{1.85}

h_f =

C^{1.85} * d^{4.8655}

191.7

(3) - Minor losses assumed to be 10% of friction headloss

PETA-4 Alternative Pump Stations	
PETA-4: Pump Station A	
Friction head loss	72.6 ft
Pressure at High Point	46.2 ft
Elevation head	250 ft
Suction head	114 ft
Total Dynamic Head	254.4 ft
TDH to HP (equation 1) whp = $\frac{\text{gpm} \times \text{TDH (ft)}}{3,960}$	
Flow	20840.00 gpm
TDH	254.4 ft
whp	1338.72 hp
bhp	1784.95 hp
Rounded bhp	2000 hp

----- Includes minor losses

----- Minimum pressure at high point

----- Assuming 75% pump efficiency

PETA-4: Pump Station B	
Friction head loss	20.0 ft
Pressure at High Point	46.2 ft
Elevation head	435 ft
Suction Head	45 ft
Total Dynamic Head	456.2 ft
TDH to HP (equation 1) whp = $\frac{\text{gpm} \times \text{TDH (ft)}}{3,960}$	
Flow	20840.00 gpm
TDH	456.2 ft
whp	2400.69 hp
bhp	3200.91 hp
Rounded bhp	2500 hp

----- Includes minor losses

----- Minimum pressure at high point

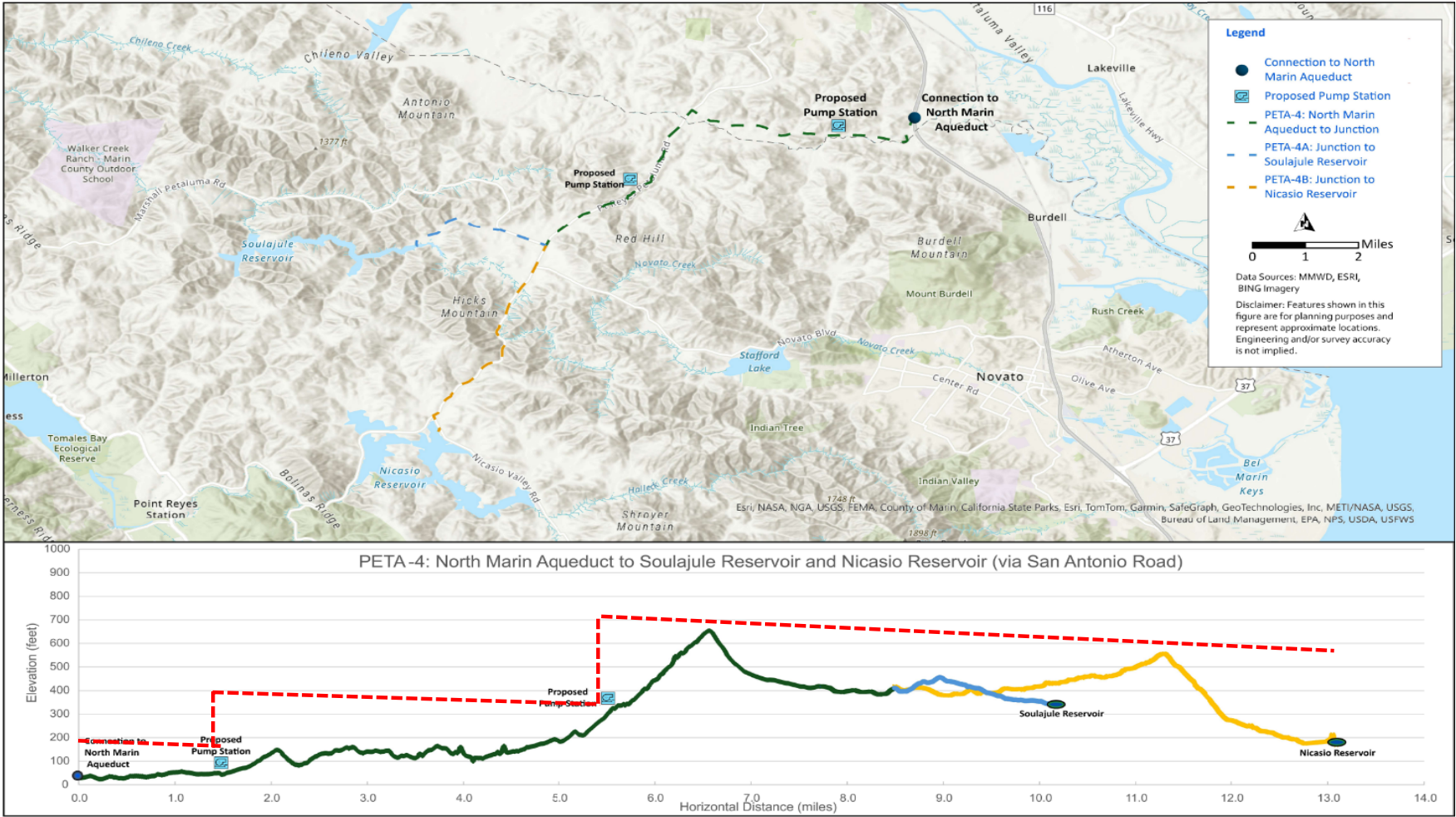
----- Assuming 75% pump efficiency

Added STS PS for PETA-4 w/Imps	
Friction head loss	93.5 ft
Pressure head	46.2 ft
Elevation head	210 ft
Total Dynamic Head	349.7 ft
TDH to HP (equation 1) whp = $\frac{\text{gpm} \times \text{TDH (ft)}}{3,960}$	
Flow	11880.00 gpm
TDH	349.7 ft
whp	1049.16 hp
bhp	1398.88 hp
Rounded bhp	1500 hp

----- Includes minor losses

----- 20 psi at Kastania Tank

----- Assuming 75% pump efficiency



MMWD Water Supply Conveyance Improvements Project - COTATI-3 Pipeline Sizing																		
Project Alternative	Segment	From	To	Length (miles)	Length (feet)	Flow (mgd)	Flow (gpm)	Flow (cfs)	Max Design Velocity (ft/sec)	Minimum Pipe Diameter (in)	Design Pipe Diameter (in)	Actual Pipe Velocity (ft/sec)	Hazen Williams C factor ⁽¹⁾	Friction Headloss (ft) ⁽²⁾	Minor Losses (ft) ⁽³⁾	Starting Elevation	Max Elevation	Finish Elevation
COTATI-3	1	Cotati Tanks	Pump Station A	1.9	10,032	30	20833.33	46.42	7	34.87	36	6.57	135	31.37	3.14	343	343	227
	2A	Pump Station A	Segment High Point	7.4	39,072	30	20833.33	46.42	7	34.87	36	6.57	135	122.17	12.22	165	361	361
	2B	Segment High Point	Pump Station B	5.2	27,456	30	20833.33	46.42	7	34.87	36	6.57	135	85.85	8.58	361	361	246
	3A	Pump Station B	Segment High Point	1.5	7,920	30	20833.33	46.42	7	34.87	36	6.57	135	24.76	2.48	246	735	735
	3B	Segment High Point	Nicasio Reservoir	8.1	42,768	30	20833.33	46.42	7	34.87	36	6.57	135	133.73	13.37	735	735	330

(1) - Assumes a C Factor for Cement Mortar Lined Welded Steel Pipe

(2) - Headloss Calculated Using Hazen-Williams Equation

10.44 * L * Q^{1.85}

h_f =

C^{1.85} * d^{4.8655}

(3) - Minor losses assumed to be 10% of friction headloss

COTATI-3 Alternative Pump Stations	
COTATI-3: Pump Station A	
Friction head loss	134.4 ft
Pressure at High Point	46.2 ft
Elevation head	196 ft
Suction head	81 ft
Total Dynamic Head	295.1 ft
TDH to HP (equation 1) whp = gpm x TDH (ft) 3,960	
Flow	20840.00 gpm
TDH	295.1 ft
whp	1552.95 hp
bhp	2070.60 hp
Rounded bhp	2000 hp

----- Includes minor losses

----- Minimum pressure at high point

----- Assuming 75% pump efficiency

COTATI-3: Pump Station B	
Friction head loss	27.2 ft
Pressure at High Point	46.2 ft
Elevation head	489 ft
Suction Head	112 ft
Total Dynamic Head	450.4 ft
TDH to HP (equation 1) whp = gpm x TDH (ft) 3,960	
Flow	20840.00 gpm
TDH	450.4 ft
whp	2370.50 hp
bhp	3160.67 hp
Rounded bhp	3500 hp

----- Includes minor losses

----- Minimum pressure at high point

----- Assuming 75% pump efficiency

The figure consists of two parts: a map and an elevation profile.

Map: A topographic map of the Cotati area in Marin County, California. It shows the proposed pipeline route (COTATI-3) from Cotati Tanks to SoulaJule Reservoir and Nicasio Reservoir. The route is divided into segments: COTATI-3: Cotati Tanks to Junction (pink dashed line), COTATI-3A: Junction to SoulaJule Reservoir (dark blue dashed line), and COTATI-3B: Junction to Nicasio Reservoir (orange dashed line). Key locations include Cotati, Penngrove, Liberty, Two Rock, Bodega Ave, Petaluma, Lakeville Rd, Adobe Rd, Sonoma Mountain, Lakeville Hwy, Novato, Stafford Lake, Nicasio Reservoir, and SoulaJule Reservoir. The map also shows major roads like Valley Ford Rd, Tomales Petaluma Rd, Marshall Petaluma Rd, Novato Blvd, and Highway 101. A legend in the top right corner identifies the symbols for Cotati Tanks, Proposed Pump Station, and the different pipeline segments. A scale bar indicates 0 to 4 miles. Data sources listed are MMWD, ESRI, and BING Imagery. A disclaimer states that features are for planning purposes and represent approximate locations.

Elevation Profile: A line graph titled "COTATI-3 Cotati Tanks to SoulaJule Reservoir and Nicasio Reservoir". The y-axis represents Elevation (feet) from 0 to 1000. The x-axis represents Horizontal Distance (miles) from 0.0 to 26.0. The profile shows the elevation of the pipeline route. It starts at Cotati Tanks (approx. 300 ft) and ends at Nicasio Reservoir (approx. 150 ft). The profile includes a proposed pump station at approximately 14.5 miles. The elevation profile is color-coded to match the pipeline segments: pink for the first segment, dark blue for the second, and orange for the third. The profile shows significant elevation changes, including a peak at approximately 16.5 miles (approx. 750 ft) and a dip at approximately 19.5 miles (approx. 400 ft).

PETA-3 without Upstream Improvements Pipeline Cost Estimate												
Project Alternative	Segment	From	To	Type of area where pipeline will be placed	Length (miles)	Length (feet)	Flow (mgd)	Design Pipe Diameter (in)	Pipe Cost (\$/LF) ⁽¹⁾	Installation Cost (\$/LF) ⁽²⁾	Pipe & Installation Unit Cost (\$/LF)	Total Pipeline Segment Cost
PETA-3	1	North Marin Aqueduct	Pump Station A	Residential	0.75	3,960	30	36	\$ 407.00	\$ 481.50	\$ 888.50	\$ 3,520,000
	2	Pump Station A	Junction	Residential	8.40	44,352	30	36	\$ 407.00	\$ 481.50	\$ 888.50	\$ 38,380,000
	4A	Junction	Soula j ule Reservoir	Rural	1.25	6,600	30	36	\$ 407.00	\$ 440.80	\$ 847.80	\$ 5,600,000
	4B			Rural - Unpaved	1.70	8,976	30	36	\$ 407.00	\$ 202.80	\$ 609.80	\$ 5,480,000
	5	Junction	Nicasio Reservoir	Rural	4.15	21,912	30	36	\$ 407.00	\$ 440.80	\$ 847.80	\$ 18,580,000

(1) - Estimate includes cost for WSP. 10% added to pipe costs to account for valves/fittings

(2) - Installation cost assumes 40% for rural areas, 50% for residential/Caltrans corridor and 60% for downtown areas, \$8/LF for traffic control and \$270/LF for pipeline installation in roadway, (Paving includes 15' width of full roadway reconstruction and 15' width of 2" grind and overlay), \$120/LF for pipeline installation in road shoulder or \$40/LF for pipeline installation in dirt road

PETA-3 Pump Station Cost Estimate		
Project Alternative	Estimated Horsepower	Total Pump Station Cost
PETA-3: Pump Station A	2000	\$ 18,000,000.00

(1) - PS sizing taken from hydraulic model

Booster Pump Station Unit Cost Assumptions		
Station Size	Unit Construction Cost (\$/hp)	Capital Cost (\$/hp)
30 hp and smaller	\$15,000	\$25,000
30-125 hp	\$12,950	\$21,500
125-500 hp	\$7,775	\$13,000
600-1,000 hp	\$6,475	\$11,000
1,000 hp and larger	\$5,175	\$9,000
Notes: Unit costs were taken from Mtn View Recycled Water Feasiblity Study		

PETA-3 Pipeline Cost Estimate												
Project Alternative	Segment	From	To	Type of area where pipeline will be placed	Length (miles)	Length (feet)	Flow (mgd)	Design Pipe Diameter (in)	Pipe Cost (\$/LF) ⁽¹⁾	Installation Cost (\$/LF) ⁽²⁾	Pipe & Installation Unit Cost (\$/LF)	Total Pipeline Segment Cost
PETA-3 w/Upstream Improvements	1	North Marin Aqueduct	Pump Station A	Residential	0.75	3,960	30	36	\$ 407.00	\$ 481.50	\$ 888.50	\$ 3,520,000
	2	Pump Station A	Pump Station B	Residential	3.60	19,008	30	36	\$ 407.00	\$ 481.50	\$ 888.50	\$ 16,890,000
	3	Pump Station B	Junction	Rural	4.80	25,344	30	36	\$ 407.00	\$ 440.80	\$ 847.80	\$ 21,490,000
	4A	Junction	Soulajule Reservoir	Rural	1.25	6,600	30	36	\$ 407.00	\$ 440.80	\$ 847.80	\$ 5,600,000
	4B			Rural - Unpaved	1.70	8,976	30	36	\$ 407.00	\$ 202.80	\$ 609.80	\$ 5,480,000
	5	Junction	Nicasio Reservoir	Rural	4.15	21,912	30	36	\$ 407.00	\$ 440.80	\$ 847.80	\$ 18,580,000
Added STS Alt for PETA-3 w/Imps	1	Cotati Tanks	Pump Station	US-101 Corridor	4.50	23,760	17.1	30	\$ 297.00	\$ 426.50	\$ 723.50	\$ 17,200,000
	2	Pump Station	Kastania Pump Station	US-101 Corridor	6.00	31,680	17.1	30	\$ 297.00	\$ 426.50	\$ 723.50	\$ 22,930,000
North Marin Aqueduct Parallel Pipeline	1	Kastania Pump Station	Turnout Location	US-101 Corridor	3.40	17,952	23.0	30	\$ 297.00	\$ 426.50	\$ 723.50	\$ 12,990,000

(1) - Estimate includes cost for WSP. 10% added to pipe costs to account for valves/fittings

(2) - Installation cost assumes 40% for rural areas, 50% for residential/Caltrans corridor and 60% for downtown areas, \$8/LF for traffic control and \$270/LF for pipeline installation in roadway, (Paving includes 15' width of full roadway reconstruction and 15' width of 2" grind and overlay), \$120/LF for pipeline installation in road shoulder or \$40/LF for pipeline installation in dirt road

PETA-3 Pump Station Cost Estimate		
Project Alternative	Estimated Horsepower	Total Pump Station Cost
PETA-3: Pump Station A	1000	\$ 11,000,000.00
PETA-3: Pump Station B	2500	\$ 22,500,000.00
Added STS PS for PETA-4 w/Imps	1500	\$ 13,500,000.00

(1) - PS sizing taken from hydraulic model

Booster Pump Station Unit Cost Assumptions		
Station Size	Unit Construction Cost (\$/hp)	Capital Cost (\$/hp)
30 hp and smaller	\$15,000	\$25,000
30-125 hp	\$12,950	\$21,500
125-500 hp	\$7,775	\$13,000
600-1,000 hp	\$6,475	\$11,000
1,000 hp and larger	\$5,175	\$9,000
Notes: Unit costs were taken from Mtn View Recycled Water Feasiblity Study		

PETA-4 without Upstream Improvements Pipeline Cost Estimate												
Project Alternative	Segment	From	To	Type of area where pipeline will be placed	Length (miles)	Length (feet)	Flow (mgd)	Design Pipe Diameter (in)	Pipe Cost (\$/LF) ⁽¹⁾	Installation Cost (\$/LF) ⁽²⁾	Pipe & Installation Unit Cost (\$/LF)	Total Pipeline Segment Cost
PETA-4 w/o Upstream Improvements	1	North Marin Aqueduct	Pump Station A	Rural	1.50	7,920	12.5	36	\$ 407.00	\$ 440.80	\$ 847.80	\$ 6,720,000
	2	Pump Station A	Pump Station B	Rural	4.00	21,120	12.5	36	\$ 407.00	\$ 440.80	\$ 847.80	\$ 17,910,000
	3	Pump Station B	Junction	Rural	2.35	12,408	12.5	36	\$ 407.00	\$ 440.80	\$ 847.80	\$ 10,520,000
	4A	Junction	Soulajule Reservoir	Rural	0.77	4,066	12.5	36	\$ 407.00	\$ 440.80	\$ 847.80	\$ 3,450,000
	4B			Rural - Unpaved	1.70	8,976	12.5	36	\$ 407.00	\$ 202.80	\$ 609.80	\$ 5,480,000
	5	Junction	Nicasio Reservoir	Rural	4.60	24,288	12.5	36	\$ 407.00	\$ 440.80	\$ 847.80	\$ 20,600,000

(1) - Estimate includes cost for WSP. 10% added to pipe costs to account for valves/fittings
(2) - Installation cost assumes 40% for rural areas, 50% for residential/Caltrans corridor and 60% for downtown areas, \$8/LF for traffic control and \$270/LF for pipeline installation in roadway, (Paving includes 15' width of full roadway reconstruction and 15' width of 2" grind and overlay), \$120/LF for pipeline installation in road shoulder or \$40/LF for pipeline installation in dirt road

PETA-4 Pump Station Cost Estimate		
Project Alternative	Estimated Horsepower	Total Pump Station Cost
PETA-4: Pump Station A	1500	\$ 13,500,000.00
PETA-4: Pump Station B	2000	\$ 18,000,000.00

(1) - PS sizing taken from hydraulic model

Booster Pump Station Unit Cost Assumptions		
Station Size	Unit Construction Cost (\$/hp)	Capital Cost (\$/hp)
30 hp and smaller	\$15,000	\$25,000
30-125 hp	\$12,950	\$21,500
125-500 hp	\$7,775	\$13,000
600-1,000 hp	\$6,475	\$11,000
1,000 hp and larger	\$5,175	\$9,000
Notes: Unit costs were taken from Mtn View Recycled Water Feasiblity Study		

PETA-4 with Upstream Improvements Pipeline Cost Estimate												
Project Alternative	Segment	From	To	Type of area where pipeline will be placed	Length (miles)	Length (feet)	Flow (mgd)	Design Pipe Diameter (in)	Pipe Cost (\$/LF) ⁽¹⁾	Installation Cost (\$/LF) ⁽²⁾	Pipe & Installation Unit Cost (\$/LF)	Total Pipeline Segment Cost
PETA-4 w/ Upstream Improvements	1	North Marin Aqueduct	Pump Station A	Rural	1.50	7,920	30	36	\$ 407.00	\$ 440.80	\$ 847.80	\$ 6,720,000
	2	Pump Station A	Pump Station B	Rural	4.00	21,120	30	36	\$ 407.00	\$ 440.80	\$ 847.80	\$ 17,910,000
	3	Pump Station B	Junction	Rural	2.35	12,408	30	36	\$ 407.00	\$ 440.80	\$ 847.80	\$ 10,520,000
	4A	Junction	Soulajule Reservoir	Rural	0.77	4,066	30	36	\$ 407.00	\$ 440.80	\$ 847.80	\$ 3,450,000
	4B			Rural - Unpaved	1.70	8,976	30	36	\$ 407.00	\$ 202.80	\$ 609.80	\$ 5,480,000
	5	Junction	Nicasio Reservoir	Rural	4.60	24,288	30	36	\$ 407.00	\$ 440.80	\$ 847.80	\$ 20,600,000
Added STS Alt for PETA-4 w/Imps	1	Cotati Tanks	Pump Station	US-101 Corridor	4.50	23,760	17.1	30	\$ 297.00	\$ 426.50	\$ 723.50	\$ 17,200,000
	2	Pump Station	Kastania Pump Station	US-101 Corridor	6.00	31,680	17.1	30	\$ 297.00	\$ 426.50	\$ 723.50	\$ 22,930,000

(1) - Estimate includes cost for WSP. 10% added to pipe costs to account for valves/fittings
(2) - Installation cost assumes 40% for rural areas, 50% for residential/Caltrans corridor and 60% for downtown areas, \$8/LF for traffic control and \$270/LF for pipeline installation in roadway, (Paving includes 15' width of full roadway reconstruction and 15' width of 2" grind and overlay), \$120/LF for pipeline installation in road shoulder or \$40/LF for pipeline installation in dirt road

PETA-4 Pump Station Cost Estimate		
Project Alternative	Estimated Horsepower	Total Pump Station Cost
PETA-4: Pump Station A	2000	\$ 18,000,000.00
PETA-4: Pump Station B	2500	\$ 22,500,000.00
Added STS PS for PETA-4 w/Imps	1500	\$ 13,500,000.00

(1) - PS sizing taken from hydraulic model

Booster Pump Station Unit Cost Assumptions		
Station Size	Unit Construction Cost (\$/hp)	Capital Cost (\$/hp)
30 hp and smaller	\$15,000	\$25,000
30-125 hp	\$12,950	\$21,500
125-500 hp	\$7,775	\$13,000
600-1,000 hp	\$6,475	\$11,000
1,000 hp and larger	\$5,175	\$9,000
Notes: Unit costs were taken from Mtn View Recycled Water Feasiblity Study		

COTATI-3 Pipeline Cost Estimate												
Project Alternative	Segment	From	To	Type of area where pipeline will be placed	Length (miles)	Length (feet)	Flow (mgd)	Design Pipe Diameter (in)	Pipe Cost (\$/LF) ⁽¹⁾	Installation Cost (\$/LF) ⁽²⁾	Pipe & Installation Unit Cost (\$/LF)	Total Pipeline Segment Cost
COTATI-3	1	Cotati Tanks	Pump Station A	Rural ⁽³⁾	1.90	10,032	30	36	\$ 407.00	\$ 418.30	\$ 825.30	\$ 8,280,000
	2A	Pump Station A	Pump Station B	Rural ⁽³⁾	10.50	55,440	30	36	\$ 407.00	\$ 418.30	\$ 825.30	\$ 45,760,000
	2B			Rural - Unpaved	2.40	12,672	30	36	\$ 407.00	\$ 202.80	\$ 609.80	\$ 7,730,000
	3	Pump Station B	Junction	Rural ⁽³⁾	4.00	21,120	30	36	\$ 407.00	\$ 418.30	\$ 825.30	\$ 17,440,000
	4	Junction	Soulajule Reservoir	Rural - Unpaved	1.70	8,976	30	36	\$ 407.00	\$ 202.80	\$ 609.80	\$ 5,480,000
	5	Junction	Nicasio Reservoir	Rural	5.30	27,984	30	36	\$ 407.00	\$ 440.80	\$ 847.80	\$ 23,730,000

(1) - Estimate includes cost for WSP. 10% added to pipe costs to account for valves/fittings
(2) - Installation cost assumes 40% for rural areas, 50% for residential/Caltrans corridor and 60% for downtown areas, \$8/LF for traffic control and \$270/LF for pipeline installation in roadway, (Paving includes 15' width of full roadway reconstruction and 15' width of 2" grind and overlay), \$120/LF for pipeline installation in road shoulder or \$40/LF for pipeline installation in dirt road
(3) - Installation cost assumes 85% of pipline installation in roadway and 15% of pipeline installation in road shoulder

COTATI-3 Pump Station Cost Estimate		
Project Alternative	Estimated Horsepower	Total Pump Station Cost
COTATI-3: Pump Station A	2000	\$ 18,000,000.00
COTATI-3: Pump Station B	3500	\$ 31,500,000.00

(1) - PS sizing taken from hydraulic model

Booster Pump Station Unit Cost Assumptions		
Station Size	Unit Construction Cost (\$/hp)	Capital Cost (\$/hp)
30 hp and smaller	\$15,000	\$25,000
30-125 hp	\$12,950	\$21,500
125-500 hp	\$7,775	\$13,000
600-1,000 hp	\$6,475	\$11,000
1,000 hp and larger	\$5,175	\$9,000
Notes: Unit costs were taken from Mtn View Recycled Water Feasiblity Study		

APPENDIX 3F

PRELIMINARY PLAN VIEW DRAWINGS

To be provided in future submittal.