Concept Design Report

Water Reclamation Facility (WRF) Lift Stations and Offsite Pipelines

FINAL DRAFT

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TABLE OF CONTENTS

TABLE OF CONTENTS ii
LIST OF APPENDICES iv
1 EXECUTIVE SUMMARY 5
1.1 Summary of Work to Date and Supporting Studies 6
1.2 Project Design Flows 8
1.3 Offsite Pipelines Route Study 10
1.3.1 Alignment Alternatives Development 11
1.3.2 Preliminary Assessment and Fatal Flaws 13
1.3.3 Preferred Alignment Alternative Assessment and Selection 17
1.3.4 Preferred Offsite Pipelines Alignment Alternative 19
1.4 Pump Station Alternatives Assessment 20
1.4.1 Pump Station Configuration and Location 20
1.4.2 Site Improvements 20
1.4.3 Pump Station Hydraulics and Pumping Scenarios 21
1.4.4 Preferred Pump Station Alternative 23
1.5 Preferred Alternative and Total Project Cost 25
1.6 Project Next Steps 27
2 BACKGROUND 29
2.1 Existing WWTP and Sewer System 29
2.2 Project Objectives 32
3 DESIGN FLOWS 33
3.1 Wastewater Flows 33
3.1.1 OneWater Hydraulic Model Results 33
3.1.2 Wastewater Design Flow Criteria 36
3.2 Brine and IPR Flows 38
4 OFFSITE PIPELINES ROUTE STUDY 39
4.1 Alignment Alternatives 39
4.1.1 Inlet & Discharge Locations 39
4.1.2 Working Alignment Alternatives 39
4.2 Alternatives Assessment Methodology 42
4.3 Pipeline Design Criteria & Constraints 44
4.3.1 Pressurized Pipeline Materials 44
4.3.2 Communication Conduit Material 48
4.3.3 Pipeline Casing Materials 48
4.3.4 Pipeline Size and Hydraulics Criteria 51
4.3.5 Open Cut Construction Criteria 58
4.3.6 Utility Conflict Constraints 61
4.3.7 Outfall Constraints 63
4.3.8 Highway 1 and Highway 41 Crossing Constraints 66
4.3.9 Morro Creek Crossing 74
4.3.10 Bike Path and Drainage Crossing 77
4.3.11 Quintana Roundabout Crossing 79

4.3.12 Trenchless Design Constraints and Feasibility ... 80
4.3.13 Traffic Control & Fencing Constraints ... 99
4.3.14 Right-of-way, Easement, and Encroachment Constraints ... 101
4.3.15 Geotechnical Constraints ... 102
4.3.16 Environmental Constraints ... 104
4.3.17 Cultural Resources Constraints ... 112
4.3.18 Concerns of Outside Stakeholders ... 112
4.4 Alignment Alternatives Development ... 118
4.4.1 Preliminary Assessment & Fatal Flaws ... 120
4.4.2 Final Alignments ... 123
4.4.3 Preferred Alignment Alternative Assessment and Selection ... 126
4.4.4 Unit Costs (Direct Construction Costs) ... 126
4.4.5 Indirect Project Costs ... 129
4.4.6 Project Costs ... 131
4.4.7 Non-Cost Project Impacts ... 131
4.4.8 Preferred Alternative Alignment ... 134
5 PUMP STATION ALTERNATIVE ASSESSMENT ... 135
5.1 Pump Station Configuration and Location ... 135
5.1.1 Single Pump Station ... 137
5.1.2 Dual Pump Station – West Alignment ... 138
5.1.3 Dual Pump Station – Embarcadero Alignment ... 140
5.2 Required Site Improvements for Pump Station Alternative Sites ... 142
5.2.1 Groundwater ... 142
5.2.2 Geotechnical Considerations ... 143
5.2.3 Soil Improvements ... 143
5.2.4 Flood Protection ... 144
5.2.5 Tsunami Inundation ... 146
5.2.6 Pump Station Configuration and Wet Well Design ... 147
5.2.7 Security and Access ... 150
5.2.8 Control Building ... 152
5.2.9 Emergency Generator and Storage ... 154
5.2.10 Odor Control ... 155
5.2.11 Forcemain Maintenance ... 156
5.3 Pump Station Hydraulics ... 157
5.3.1 West Alignment ... 157
5.3.2 Embarcadero Alignment ... 165
5.4 Pumping Schemes ... 170
5.4.1 Scenario 1 ... 170
5.4.2 West Scenario 2 ... 172
5.4.3 Embarcadero Scenario 2 ... 174
5.5 Pumping Scenarios Assessment ... 176
5.5.1 Pumping Scenarios ... 176

5.5.2 Project Costs 179
5.5.3 Non-Cost Project Constraints 180
5.5.4 Preferred Alternatives 183
6 Additional Pump station ALTERNATIVE 184
6.1 Scenario 3 Overall Description 184
6.1.1 Pumping from PS-A to PS-B 187
6.1.2 Pumping from LS-2 to PS-B 187
6.1.3 Pumping from PS-to the WRF 190
6.1.4 PS-A and PS-B Pump Station Preliminary Layouts and Site Plan 193
6.1.5 Additional Considerations for Scenario 3 196
6.1.6 Scenario 3 Costs 197
6.1.7 Preferred Pumping Scenario Selection 198
7 TOTAL PROJECT COST AND PREFERRED ALTERNATIVE 200
7.1 Offsite Pipelines Preferred Alternative 200
7.1.1 WRF Onsite Pipeline Alignment Coordination 200
7.1.2 IPR Site Coordination 201
7.1.3 Brine and IPR Hydraulic Coordination 201
7.1.4 Communication Conduit Coordination 201
7.1.5 Environmental Assessments 201
7.1.6 Potholing and Additional Utility Research 202
7.2 Offsite Pump Stations Preferred Alternative 202
7.3 Total Project Cost 202
7.4 Construction Bid Package 204
7.5 Construction Schedule and Phasing 204

LIST OF APPENDICES

APPENDIX A: 30% Plan & Profile of Preferred Alternative Alignment

APPENDIX B: Right-of-way Map

APPENDIX C: Preliminary Geotechnical Report

APPENDIX D: Preliminary Offsite Pipeline Alignment Figures

APPENDIX E: Preferred Alternative Alignment Offsite Pipelines Costs

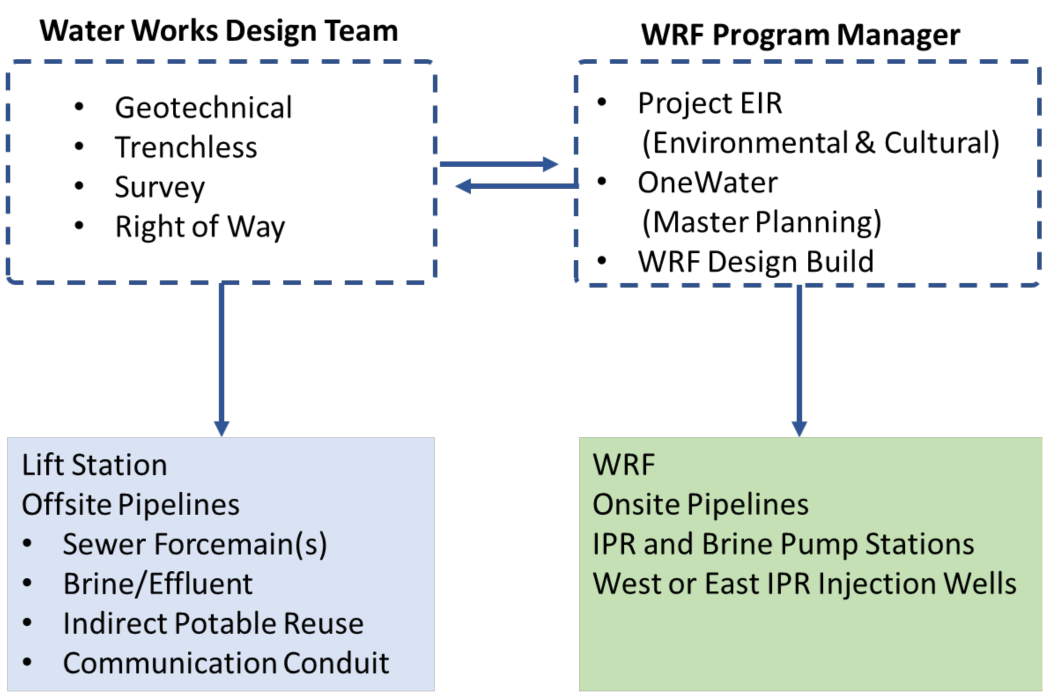
APPENDIX F: Preferred Alternative Pump Station Costs

1 EXECUTIVE SUMMARY

WaterWorks Engineers, LLC (WaterWorks) is under contract with the City of Morro Bay (City) to provide professional engineering services for the Water Reclamation Facility (WRF) Lift Stations and Offsite Pipelines projects. The overall project objectives are:

- Identify, Develop, Assess, and Recommend via workshops and a final Concept Design Report – Final Draft (CDR)
    - Site, design criteria, and project constraints for WRF Lift Station(s)
    - Alignment, design criteria, and project constraints for the offsite pipelines (sewer forcemains, brine line, and communication conduit), as well as the indirect potable reuse line (IPR).
- Complete final design PS&E (plans, specifications, & estimates) of recommended lift stations and pipelines with focus on cost effectiveness, long term quality and viability, and schedule compliance
    - Incorporate City staff input into final design
    - Coordinate closely with WRF Program Team
    - Provide engineering services during construction

A graphical representation of the role of WaterWorks in the overall WRF Project is visualized below.



1.1 Summary of Work to Date and Supporting Studies

This Concept Design Report (CDR) is the result of several workshops and field investigations which are listed below:

- 11/15/2017 Kick-Off Meeting
- 01/10/2018 Combined Workshop
    - Hydraulics, WRF Master Plan Design Criteria Review and Modifications
    - Alignment fatal flaws, Constraints and Construction Methodology
    - Environmental/Cultural Constraints, EIR Support Review
- 04/12/2018 Combined Workshop
    - Utility Research Results
    - Route Analysis – Preliminary Costs, Non-Cost Criteria, & Recommendation Status
    - Pump Station Alternatives
- 07/11/2018 Combined Workshop
    - Updated Pump Station Alternatives
    - Pump Station mechanical layout, architecture, odor control, aesthetics
    - Pump Station operations planning, electrical systems, construction sequencing
- 09/27/2018 Combined Agency Workshop
    - Route Study
    - Dynegy/Bike Path, Outfall Location, Preliminary IPR/Brine hydraulics
    - Caltrans Permitting Coordination Meeting
    - Division of Drinking Water (DDW) Permitting Coordination Meeting
- 01/22/2019 City Council Meeting
    - WRF Status and Actions
    - WRF Design/Build Schedule
    - South Bay Boulevard Property Acquisition
    - Land Use Permitting
    - Conveyance Facilities Project
        - Project Assumptions
        - Design Capacity
        - Route Analysis Summary
        - Pump Station Analysis Summary
        - EIR Revisions
    - Budget
- Field Investigations (WaterWorks Design Team)
    - Right-of-Way Acquisition Mapping (Praxis/Guida)
    - Preliminary Geotechnical Report (Yeh & Associates)
    - Utility A Research
- Project EIR Supporting Documentation
    - Archeological Survey and Cultural Resources Recommendations (Far Western and ESA)
    - Biological Resource Assessment (KMA)

- OneWater hydraulic model results and flow meter data (Carollo and V&A)

1.2 Project Design Flows

WaterWorks utilized updated design flow data from the WRF Program Team to size the forcemains, pump stations, and Brine/IPR pipelines. A flow frequency analysis based on the facility master planning and City historical meter data produced the following curve:

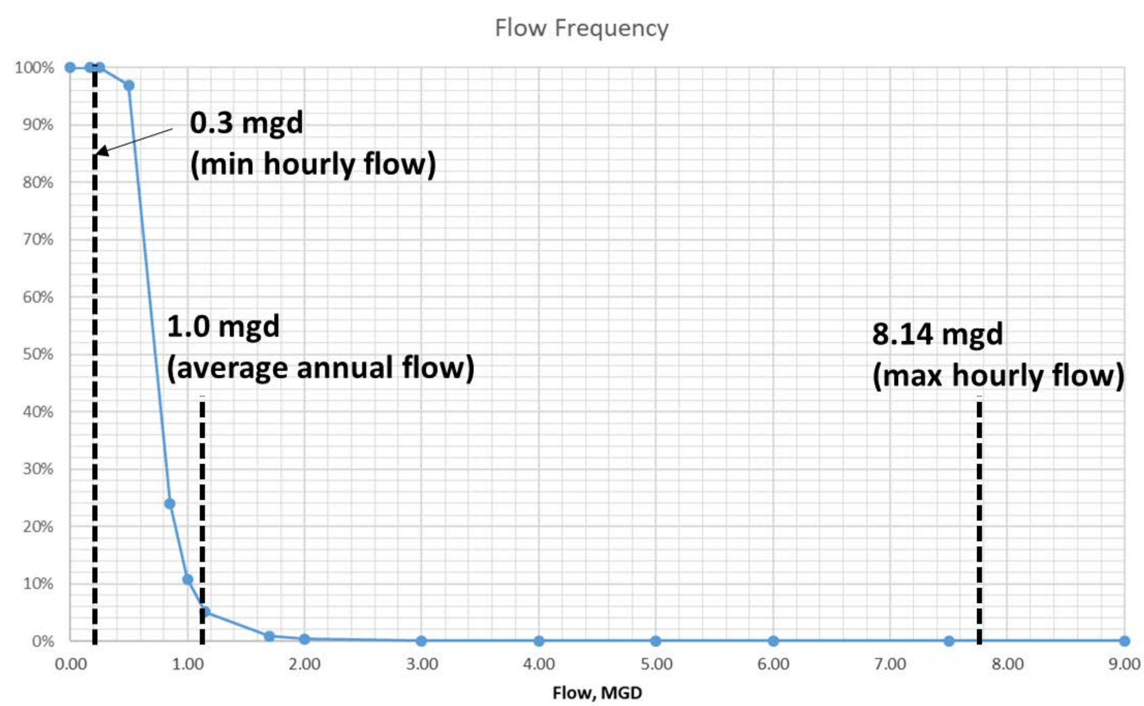


Figure 1-1: Flow Frequency

An average annual flow of 1.0 MGD was observed – with flows being less than 1.15 MGD 95% of the time. A minimum hourly flow of 0.3 MGD and a maximum hourly flow of 8.14 MGD were also observed. These observations along with the separation flows from Cayucos Sanitary District, led to the design flows for the City of Morro Bay (only) summarized in Table 1-1 and Table 1-2 below, which are presented fully in Chapter 2.

Table 1-1: Wastewater Design Flows

Existing Conditions*			2040 Future Conditions		
Flow Regime	Hourly (MGD)	Hourly (gpm)	Flow Regime	Hourly (MGD)	Hourly (gpm)
High Winter	5.85	4,060	High Winter	8.14	5,650
High Summer	2.08	1,443	High Summer	2.74	1,901
Average Annual	0.90	623	Average Annual	1.00	695
Low Summer	0.27	188	Low Summer	0.30	209

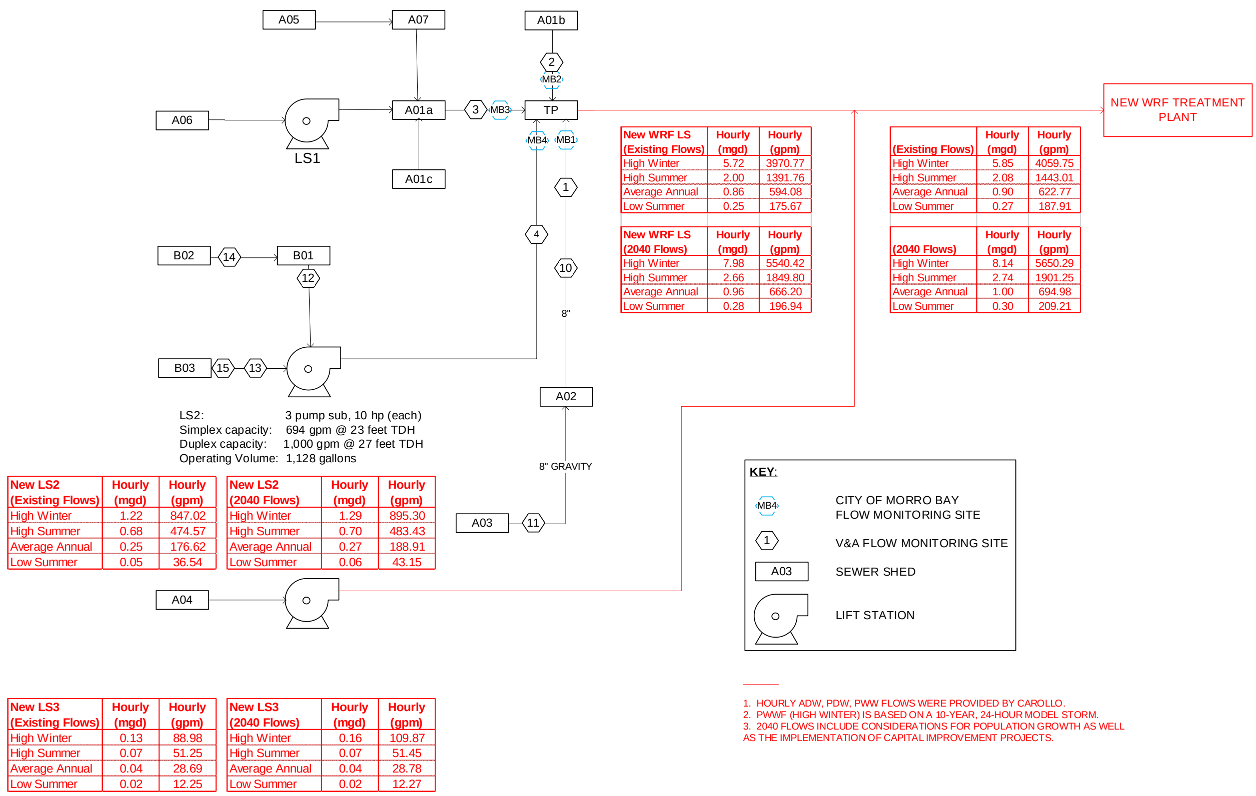
** Cayucos Sanitary District existing flows are not incorporated in the WRF project*

The maximum hourly flow for the brine pipeline is based on the maximum hourly flow seen at the existing WWTP. The indirect potable reuse (IPR) line is based on the *Lower Morro Valley Basin Screening-Level Groundwater Modeling for Injection Feasibility by GSI Water Solutions, Inc.* In that report, the amount of potential recycled water to be used for groundwater injection was determined to be 825 acre-feet/year.

Table 1-2: Brine/IPR Design Flow Data

	Brine Pipeline	IPR Pipeline
Max Flow (mgd)	8.14	0.93

Given the proximity of LS-3 to the new WRF, double pumping LS-3 flows can be eliminated by direct connecting to the new forcemain(s). This approach reduces hourly design flows for the new WRF LS to 7.98 mgd (2040 Future Conditions). A depiction of the wastewater design flows is reflected in Figure 1-2 below.



New WRF LS (Existing Flows)	Hourly (mgd)	Hourly (gpm)
High Winter	5.72	3970.77
High Summer	2.00	1391.76
Average Annual	0.86	594.08
Low Summer	0.25	175.67

New WRF LS (2040 Flows)	Hourly (mgd)	Hourly (gpm)
High Winter	7.98	5540.42
High Summer	2.66	1849.80
Average Annual	0.96	666.20
Low Summer	0.28	196.94

(Existing Flows)	Hourly (mgd)	Hourly (gpm)
High Winter	5.85	4059.75
High Summer	2.08	1443.01
Average Annual	0.90	622.77
Low Summer	0.27	187.91

(2040 Flows)	Hourly (mgd)	Hourly (gpm)
High Winter	8.14	5650.29
High Summer	2.74	1901.25
Average Annual	1.00	694.98
Low Summer	0.30	209.21

New LS2 (Existing Flows)	Hourly (mgd)	Hourly (gpm)
High Winter	1.22	847.02
High Summer	0.68	474.57
Average Annual	0.25	176.62
Low Summer	0.05	36.54

New LS2 (2040 Flows)	Hourly (mgd)	Hourly (gpm)
High Winter	1.29	895.30
High Summer	0.70	483.43
Average Annual	0.27	188.91
Low Summer	0.06	43.15

New LS3 (Existing Flows)	Hourly (mgd)	Hourly (gpm)
High Winter	0.13	88.98
High Summer	0.07	51.25
Average Annual	0.04	28.69
Low Summer	0.02	12.25

New LS3 (2040 Flows)	Hourly (mgd)	Hourly (gpm)
High Winter	0.16	109.87
High Summer	0.07	51.45
Average Annual	0.04	28.78
Low Summer	0.02	12.27

Figure 1-2: Proposed Process Flow Diagram with LS-3 Diversion

1.3 Offsite Pipelines Route Study

WaterWorks conducted a comprehensive route study to select a preferred offsite alignment and pipelines option that focused on cost effectiveness, long term quality and viability, and schedule compliance. The overall process is summarized below:

1. Identified working alternative alignments and pipeline options that vary by size and material
2. Conducted a preliminary assessment and identify fatal flaws that disqualify the alternative based on constructability issues and hydraulic design criteria including various lift station alternatives
3. Identified pipeline design criteria and project constraints that affect the final alignments
4. Developed construction unit costs that will contribute to direct construction costs (estimate of the contractor's bid costs)
5. Developed indirect costs that will affect the City prior to or during construction (not reflected in the contractor's bid costs)
6. Assessed the final alignment alternatives based on total project costs (direct + indirect) and non-cost considerations
7. Selected and recommended an alignment alternative for design and construction

1.3.1 Alignment Alternatives Development

For the first step in the assessment, WaterWorks identified five working alignments which are summarized below and presented in **Error! Reference source not found.** and in more detail in Section 4.1.

- **West Alignment** – Runs east along Atascadero, southeast along the existing bike path, and down Quintana Rd parallel to HWY-1 on the southwest side. This alignment was originally identified as the "west" alignment because it is located west of HWY-1.
- **East Alignment** – Runs east along Atascadero, and then southeast on Main St and then via new easement parallel to HWY-1 on the northeast side. This alignment was originally identified as the "east" alignment because it is primarily located east of HWY-1. Note that a "hybrid" West and East alignment was also analyzed, whereby the West alignment would be utilized up to Main St, northwards and under Hwy-1, and then would utilize an East alignment to the WRF site.
- **Embarcadero Alignment** – Runs west and then south along Embarcadero, then east along Pacific, and Quintana parallel to HWY-1 on the southwest side.
- **Hills-Creek Alignment (Little Morro Creek – open cut, or Long HDD)** – Runs east along Atascadero, northeast along HWY-41 and then cuts across the rolling hills above the City and into the County limits. This represents the shortest possible alignment to the WRF.
- **Hills-Radcliff Alignment (Main St – Long HDD)** – Runs east along Atascadero, along Main St and then cuts through rolling hills above the City and crosses into the County limits.

Through this process WaterWorks identified pipeline design criteria and constraints for each of the working alignments which was utilized to inform the preliminary assessment. A summary of these design criteria and constraints are listed below:

- Pipeline Hydraulics (pipe material, number and size of pipelines)
- Construction Methodology
  - Open Cut Construction Criteria
  - Trenchless Design & Feasibility
- Hwy-1/Hwy-41 Crossing
- Utility Conflicts
- Morro Creek / Drainage Crossing
- Quintana Roundabout Crossing
- Right-of-way, Easement, Encroachment
- Geotechnical
- Traffic Control and Fencing
- Cultural Resources
- Environmental
- Concerns of Outside Stakeholders

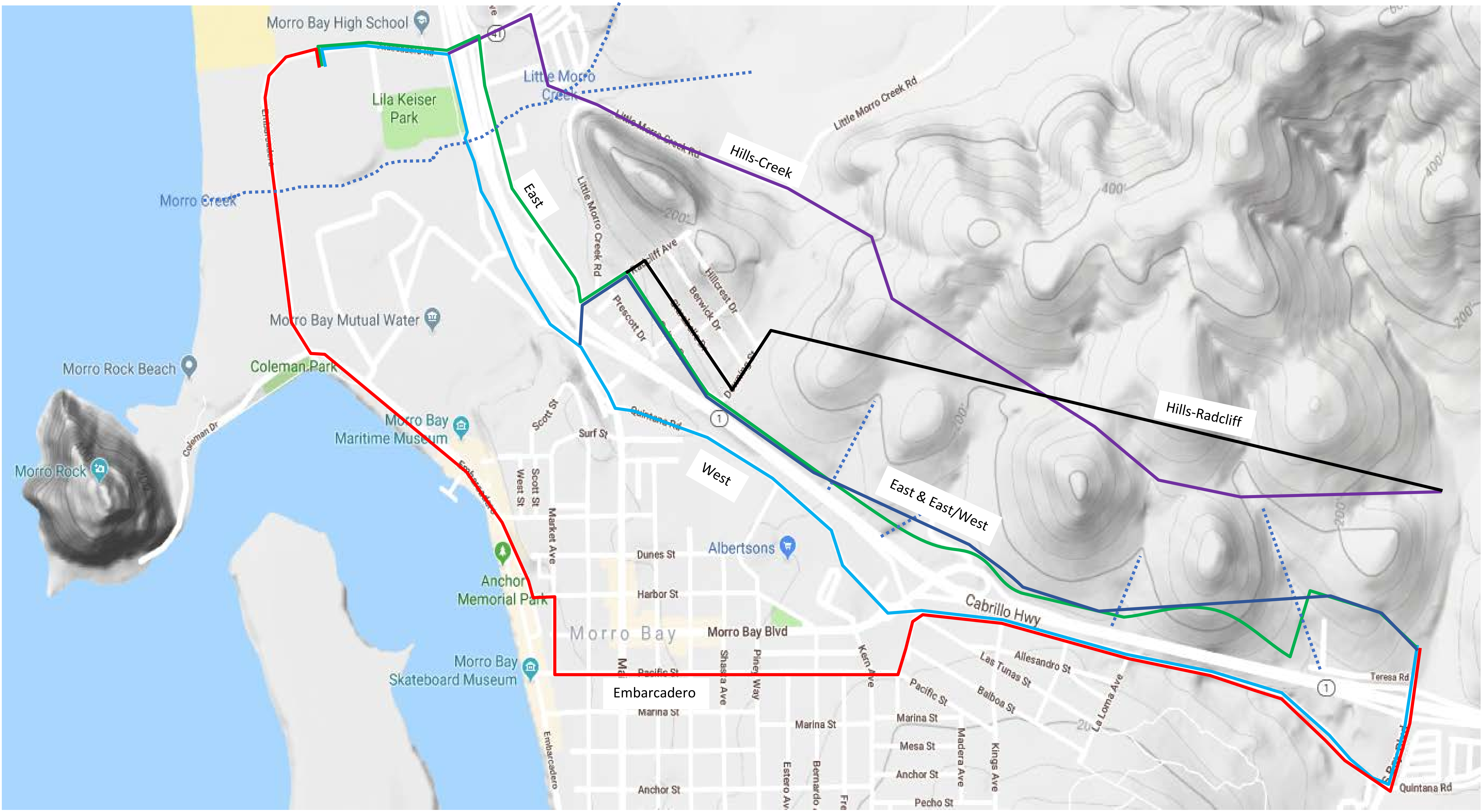


Figure 1-3: WRF Offsite Pipelines Alignment Alternatives (dashed blue lines indicate drainage features)

1.3.2 Preliminary Assessment and Fatal Flaws

WaterWorks began the preliminary assessment by identifying fatal flaws from the pipeline design criteria and constraints for several working alignments whereby the alignment or pipeline option was disqualified from further assessment primarily due to constructability concerns (feasibility). In addition, an alignment may be fatal flawed due to an unacceptable risk of not meeting the overall project schedule due to extensive permitting or right-of-way acquisition lead time. This analysis is detailed in Section 4.3. The different fatal flawed working alignments are listed in Table 1-3 below with a description of the fatal flaws.

Table 1-3: Preliminary Assessment Fatal Flaws

Working Alignment	Fatal Flaws
East Alignment	• Constructability: No practical way to cross Morro Creek: limited space for pipe bridge, trenchless crossing impacted by existing bridge piles • Environmental: The four ephemeral drainage lines and wetlands in the grassland northeast of HWY-1 would require separate environmental permits (404/401/1602) that would introduce unacceptable risks to the project schedule. Numerous trenchless crossings would be cost prohibitive. • Hydraulic: Extreme elevation changes close to HWY-1 introduce significant static head requirements (>220’) which produce maximum TDH values that are infeasible for preferred pump station alternatives.
East/West Alignment	• Constructability: Two long HDD operations would be required that would put the pipelines at extreme depths. These may be feasible based on preliminary research into site geology, but the cost of the operation and a minimum two casings to meet DDW requirements at that extreme depth and length (> 100’ deep) would be prohibitive and would add at least $4M more than the West Alignment
Hills-Creek Alignment - (Little Morro Creek) Open Cut	• Constructability: No practical way to cross Morro Creek and Little Morro Creek: no location for pipe bridge, and HDD would pass very closely to houses near the alignment which is not preferred • Environmental: Multiple ephemeral drainage lines in the may require separate environmental permits (404/401/1602) that would introduce unacceptable risks to the project schedule. Numerous trenchless crossings would be cost prohibitive. • Hydraulic: Extreme elevation changes along the hillside portion of the alignment introduce significant static head requirements (>250’) which produce maximum TDH values that are infeasible for preferred pump station alternatives
Hills-Radcliff Alignment – Long HDD	• Constructability: No practical way to cross Morro Creek and Little Morro Creek: no location for pipe bridge, and HDD would pass very closely to houses near the alignment which is not advisable. • Hydraulic: The two Hills alignments which utilize Long HDDs (>6000’) may be feasible based on preliminary research into site geology, but the cost of the operation and a minimum two casings to meet DDW requirements at that extreme depth and length (> 100’ deep) would be prohibitive and would add at least $10M more than the West Alignment

The West Alignment in plan and profile is presented in Figure 1-4: West AlignmentFigure 1-4. Subsequent figures illustratively compare other alignment alternatives to the West Alignment with indications of if, where and how alternatives were fatal flawed as described in the previous table.

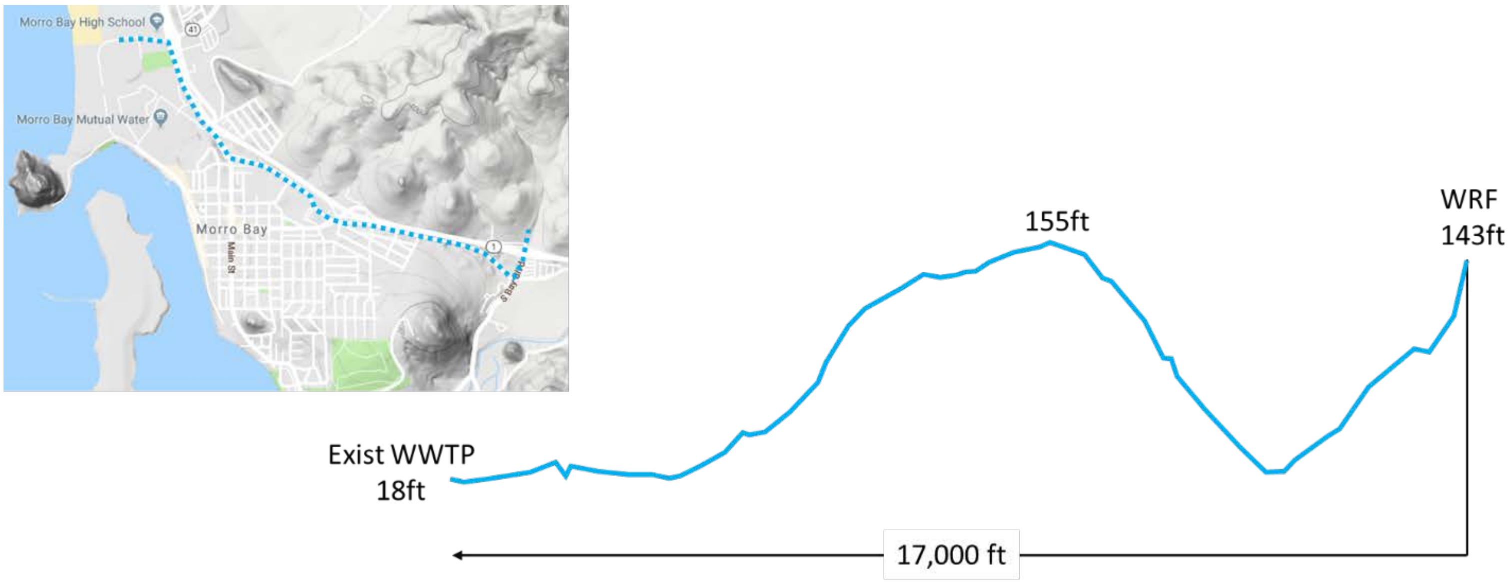


Figure 1-4: West Alignment

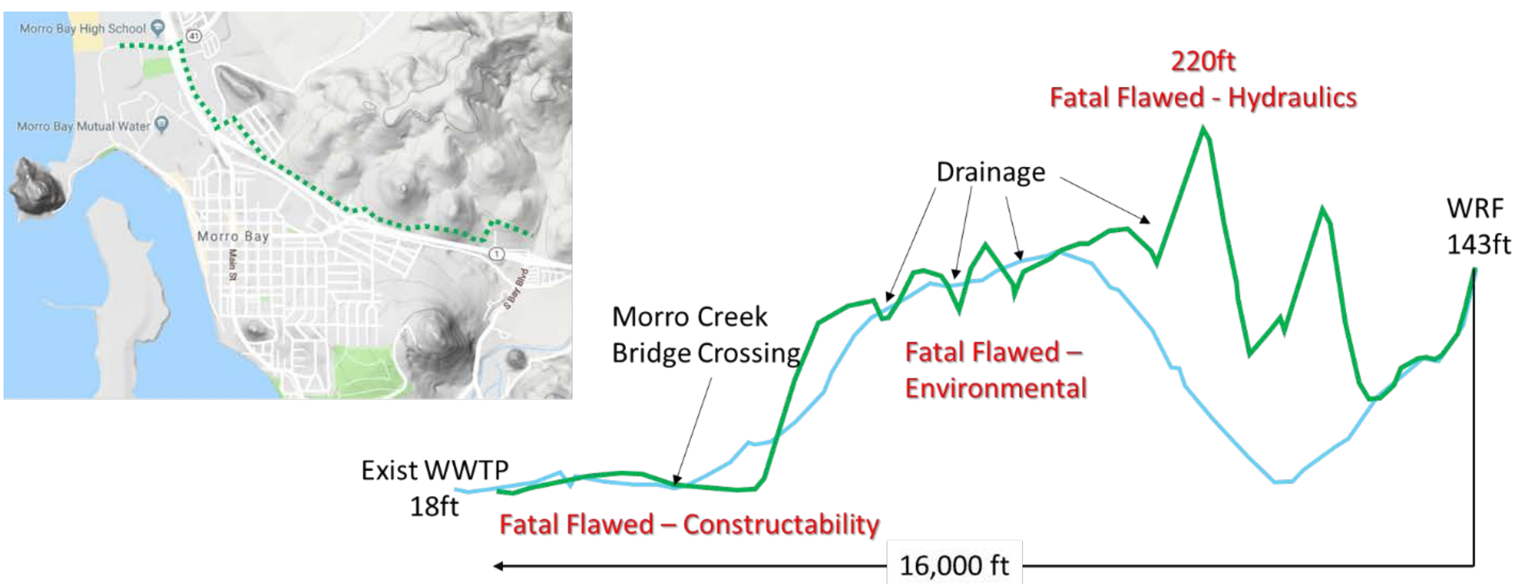


Figure 1-5: East Alignment

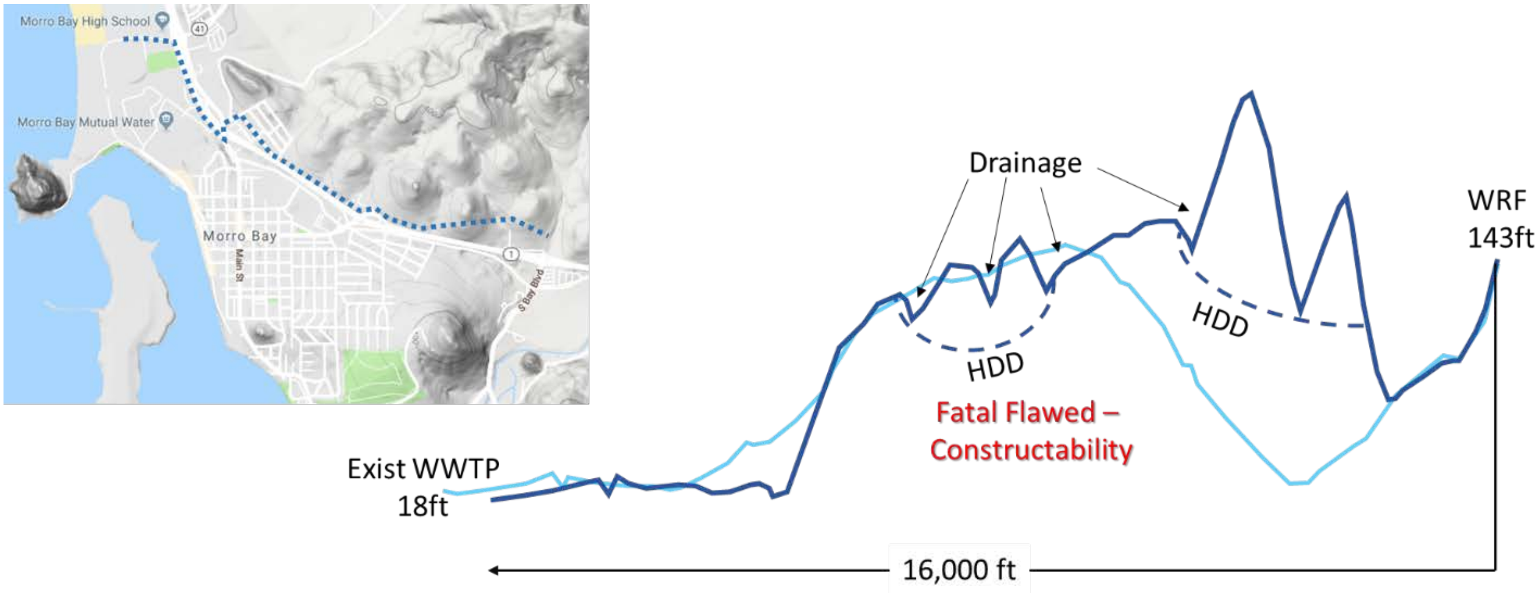


Figure 1-6: East/West Alignment

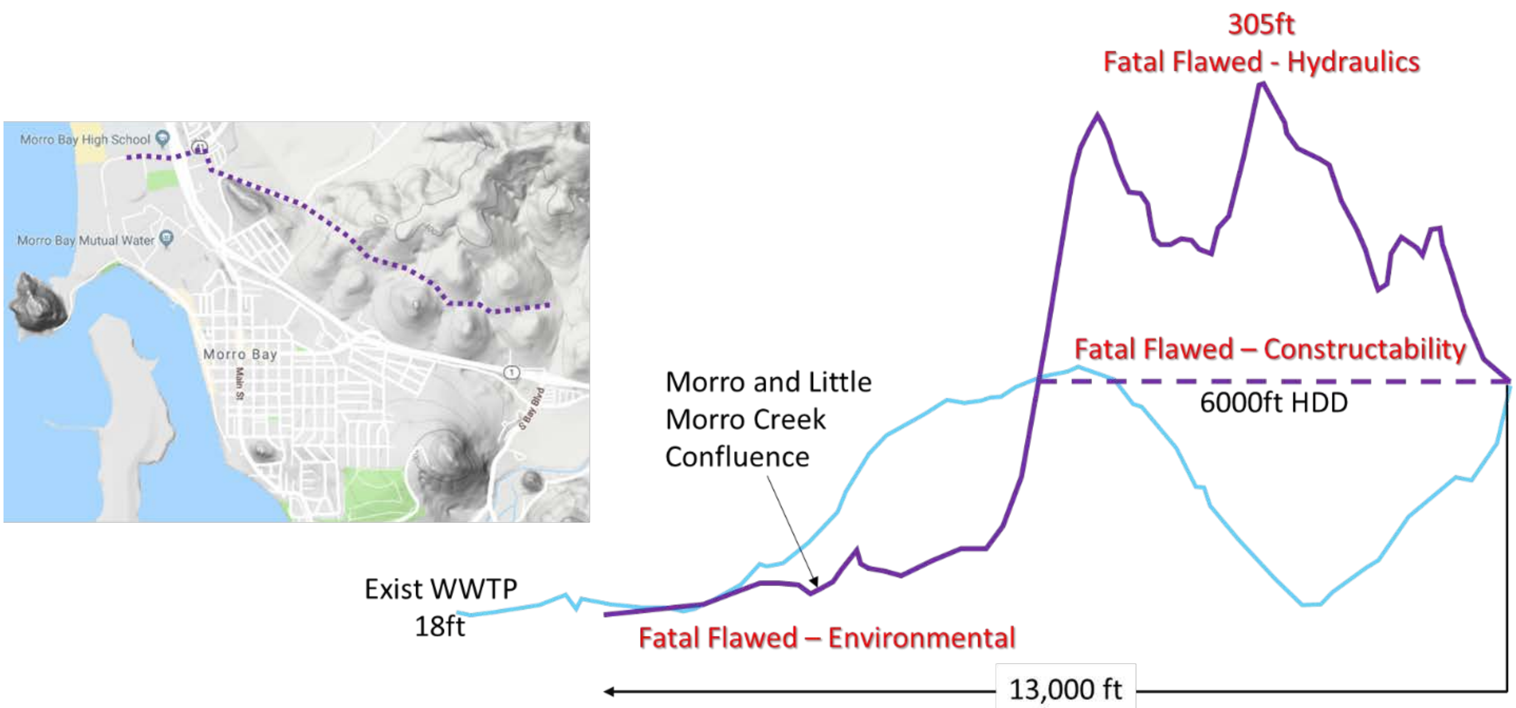


Figure 1-7: Hills-Creek Alignment

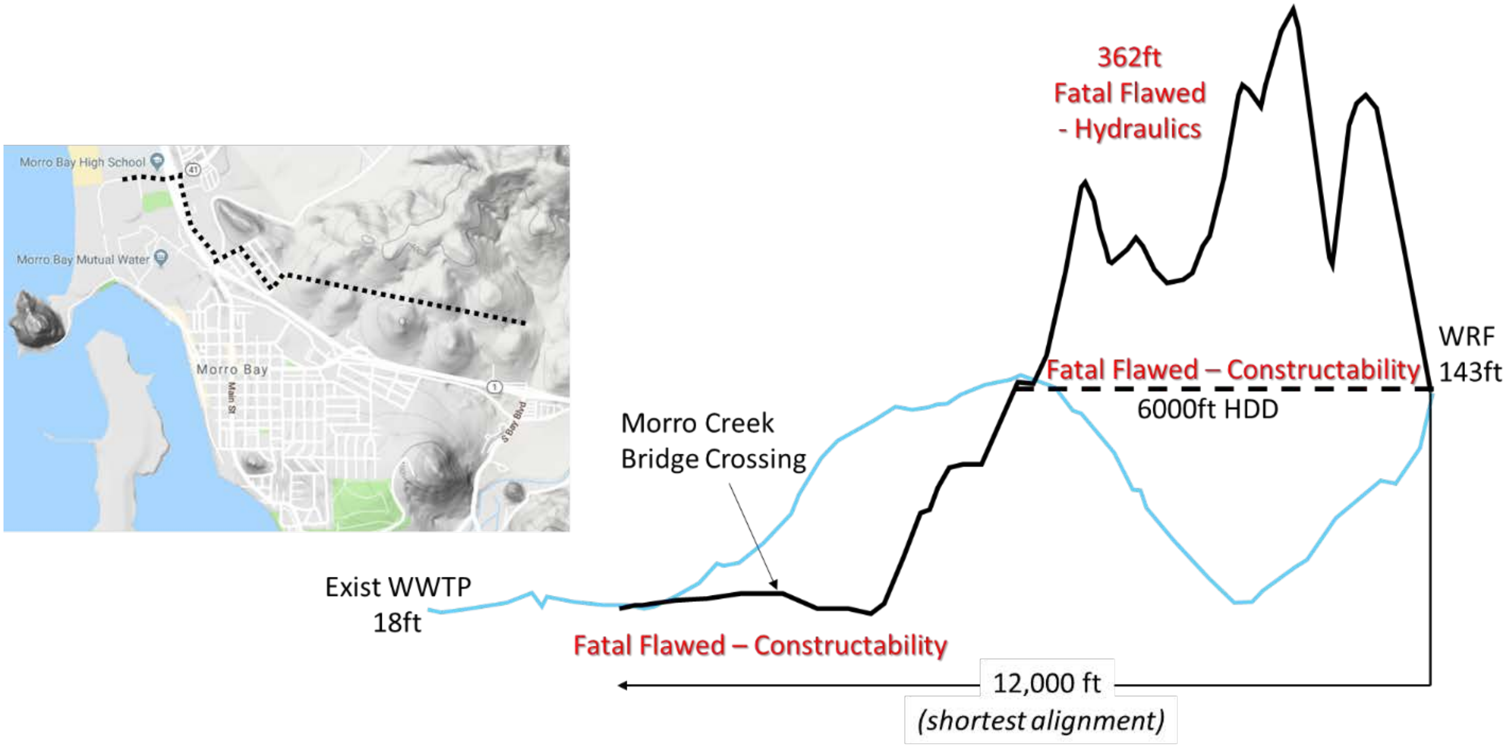


Figure 1-8: Hills-Radcliff Alignment

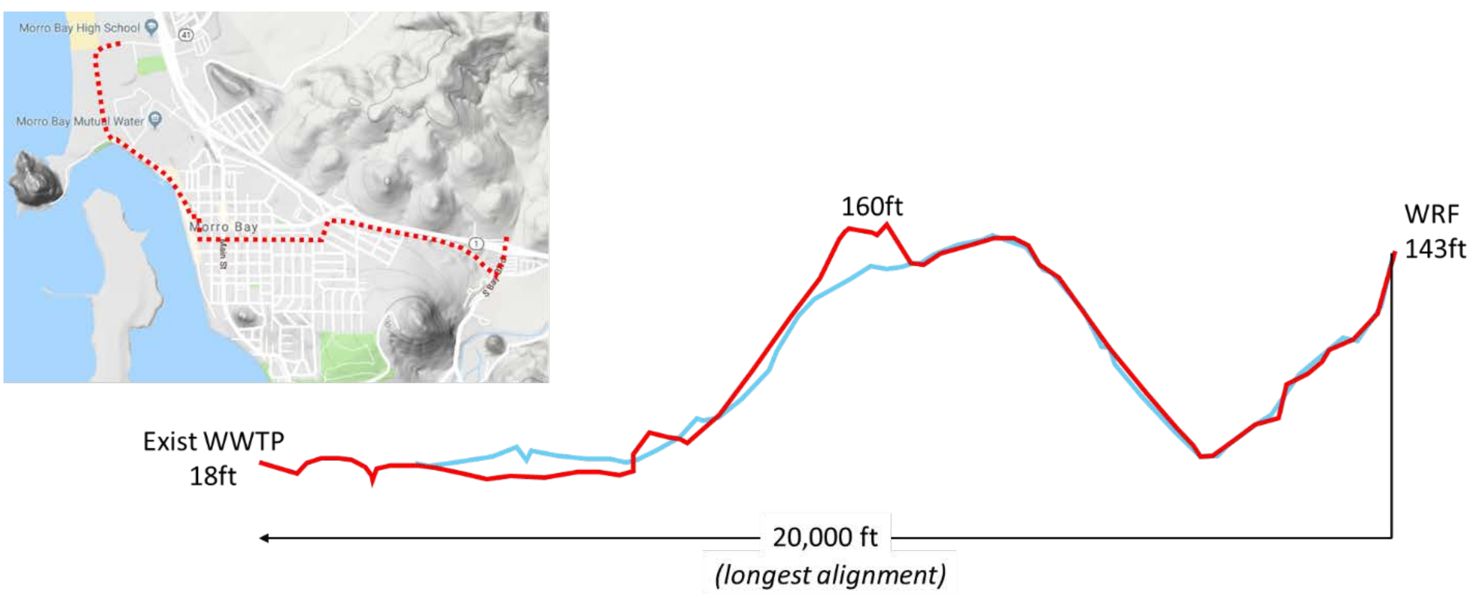


Figure 1-9: Embarcadero Alignment

This preliminary assessment does not fatal flaw the Embarcadero Alignment which is a viable alternative.

1.3.3 Preferred Alignment Alternative Assessment and Selection

The final alignment alternatives that were not fatal flawed were then assessed based on total project costs and non-cost project impacts which is detailed in Section 4.4.

Total Offsite Pipelines Project Costs

WaterWorks developed direct construction costs (estimated contractor bid costs) and indirect construction costs associated with the particular-alignment and pipeline options that would impact the City during the design phase. These indirect construction costs include right-of-way acquisition/procurement and permitting costs. Sewer forcemain, brine and IPR pipeline alignment alternatives varying sizes size and material of pipelines were analyzed and are presented as nominal pipe sizes and material (FPVC: Fusible Polyvinyl Chloride and HDPE: Fusible High Density Polyethylene) in the table below. In addition, a communication conduit for fiber optic will be installed in a common trench. An example trench section is shown below. The total construction project costs for the final alternative alignments are listed in Table 1-4.

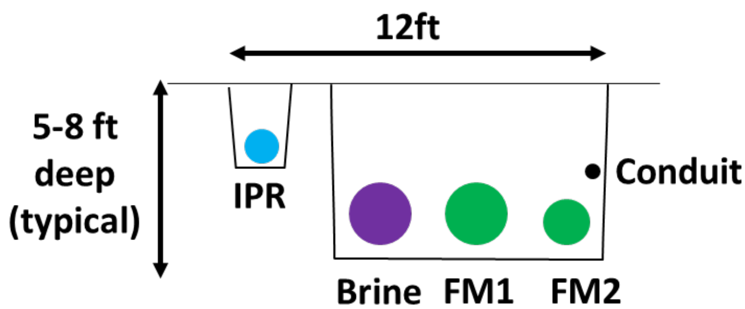


Figure 1-10: Typical Trench Section

Table 1-4: Final Alternative Alignments Total Project Costs

Total Offsite Pipelines Project Costs*					
Alignment	Pipeline Option	No IPR + Conduit	West IPR	East IPR	Communication Conduit
West	12"FM + 16"FM + 16"Brine DR 18 FPVC	$12,614,700	$14,814,700	$15,784,700	$ 414,700
	14"FM + 20"FM + 20"Brine DR 13.5 HDPE	$13,874,700	$16,184,700	$17,147,700	$ 414,700
Embarcadero	12"FM + 16"FM + 16"Brine DR 18 FPVC	$14,124,000	$17,024,000	$17,874,000	$494,000
	14"FM + 20"FM + 20"Brine DR 13.5 HDPE	$15,594,000	$18,624,000	$19,735,000	$494,000

**Reflects 20% construction & 10% design contingency applied to direct construction costs*

Non-Cost Project Impacts

The selection of the preferred alternative was not only driven by project cost but by non-cost constraints and project impacts.

The first non-cost considerations WaterWorks assessed was the use of dual or single forcemains. Utilizing a single forcemain would reduce costs compared to a dual forcemain and would be preferable if considering cost alone. When considering the significant operational risk associated with a single FM, however, the dual FM option becomes preferable. In summary, dual sewer forcemains provide the best combination of hydraulics for pumping, O&M flexibility, and redundancy. Consequently, WaterWorks selected the dual forcemain option as the preferred alternative. This is summarized in Table 1-5 below and is discussed in further detail on Table 4-30 in Section 4.4.7.

Table 1-5: Single Forcemain Non-Cost Project Constraints

Forcemain Options Non-Cost Project Constraints		
Non-Cost Constraint	**Single**	**Dual**
Redundancy	-1	+1
Maintenance & Reliability	-1	+1
Odor Production	-1	+1
Total Score	-3	+3

The West alignment is the most cost competitive of the two final alignments, but the Embarcadero alignment has fewer non-cost constraints as summarized in Table 1-6 below. Table 4-31 in Section 4.4.7 has a detailed discussion of the non-cost constraints, some of which were combined in the table below.

Table 1-6: Final Alignments Non-Cost Project Constraints

Key Criteria and Constraints	West	Embarcadero
Hydraulics	+1	-1
Environmental / Schedule Risks	0	+1
Geotechnical	+1	-1
Cultural Resources	+1	-1
Accessibility / O&M	+1	+1
Dual Pump Station Integration	+1	-1
Constructability	0	+1
Right-of-way Acquisition	0	+1
Traffic / Public / Commercial Impacts	-1	-1
Total Score	+4	-1

From the analysis presented above, the Embarcadero alignment generally has fewer constraints than the West Alignment but is approximately $2.0 million more expensive due to pipeline related costs (if including the cost of the East IPR). In addition, use of the Embarcadero alignment cannot leverage diverting local flows into a secondary booster pump station. The West alignment can leverage local flow diversions, however, and consequently is significantly less expensive if much smaller dual pump stations are used. This is discussed in more detail in the pump station assessment and project costs Chapter 5& 6.

1.3.4 Preferred Offsite Pipelines Alignment Alternative

Based on the comprehensive route study and alternative assessment presented in Section 4.4.8, WaterWorks recommends the 8”IPR-12”FM-16”FM-16”Brine FPVC or 8”IPR-14”FM-20”FM-20”Brine HDPE West Alignment as the preferred alignment/pipeline option alternatives. The preferred West Alignment alternative is summarized by its largest component construction segments in Table 1-7.

Table 1-7: Preferred Alternative Alignment Construction Segments

Start STA	End STA	Length	Construction Segment
10+00	14+50	450	Outfall Improvements & Tie In
14+50	26+75	1225	Atascadero to Caltrans
14+50			PS-A Tie-In
26+75		1476	East IPR - Hwy-1 Crossing - Hwy-41 Caltrans Encroachment
26+75	33+00	625	SB Hwy-1 Connector Caltrans Encroachment
33+00	36+00	200	Morro Creek Bridge Crossing
36+00	51+00	1500	Bike Path
39+75		500	West IPR
51+00	58+00	700	Bike Path Drainage Crossing + Caltrans Encroachment
51+00		1910	LS-2 12" SSFM
58+00			PS-B Tie-In
58+00	94+00	3600	Lower Quintana (Main St to Roundabout)
94+00	101+00	700	Quintana Roundabout Crossing
101+00			LS-3 Bypassing
101+00	151+00	5000	Upper Quintana (Roundabout to South Bay)
150+00			LS-3 Tie In (to 12” SSFM)
151+00	161+00	1000	South Bay Blvd and Hwy-1 Crossing

A preliminary plan and profile of this alternative along with the potential East and West IPR lines is displayed in APPENDIX A: 30% Plan & Profile of Preferred Alternative Alignment. References to tie-ins at PS-A and PS-B are included in Table 1-7.These reflect the preferred WRF PS alternative, the assessment and selection of which is summarized in the following Section 1.4 and detailed in Chapter 5 & 6

1.4 Pump Station Alternatives Assessment

WaterWorks conducted a pump station alternatives assessment based on the final alignment alternatives developed as part of the offsite pipelines assessment. The pump station alternatives assessment is detailed in Chapter 5.

1.4.1 Pump Station Configuration and Location

With the intent of developing a pumping solution that met the high static head (~160') and dynamic head associated with the wide range of design flow conditions (0.35 – 8.14 MGD) with cost-effective improvements that support successful long-term operations and maintenance, both single and dual pump station configurations were evaluated for the West and Embarcadero alignments. In a multi-pump station configuration, PS-A would be located near the existing WWTP and PS-B would be an intermediate pump station between PS-A and the new WRF.

There are three potential sites for a single station/PS-A at/near the existing WWTP:

- PS Site 1: Re-use Existing Influent Pump Station
- PS Site 2: South of Atascadero, City Property
- PS Site 3: North of Atascadero

On the West Alignment, the following locations were evaluated for PS-B:

- West Site 1A & B – Quintana Road
- West Site 2 – City-Owned Parcel (Main Street at Highway 1)

On the Embarcadero Alignment, the following locations were evaluated for PS-B:

- Embarcadero Site 1 – City-Owned Parcel (Pacific Street and Market Avenue)
- Embarcadero Site 2 – Bank of America Parcel (Pacific Street and Monterey Avenue)

1.4.2 Site Improvements

WaterWorks identified several design considerations for incorporation into the pump station(s) site improvements. This included:

- Impacts of shallow groundwater
- Geotechnical considerations for seismic design
- Soil improvements to mitigate liquefaction
- Identification of 100-year flood plain and requirements to protect critical equipment
- Identification of tsunami inundation design zone
- Wet well configurations for single and dual pump stations
- Security for and access to the pump stations
- Control building structural, architectural and electrical considerations

- Emergency power options and identification of emergency storage potential
- Odor control options
- Pig launching and receiving opportunities

1.4.3 Pump Station Hydraulics and Pumping Scenarios

The single and multi-pump stations were hydraulically evaluated at the alternative sites. Achievable flows, combination of required forcemains at varying flows and associated velocities were calculated and evaluated.

Scenario 1: Single Pump Station

Scenario 1 was based on a single pump station configuration. A single station near the existing WWTP was sized and evaluated for both the Embarcadero and West Alignments. The three locations that were identified as potential pump station sites (PS Site 1, 2 & 3) were in such proximity to one another that they were determined to be hydraulically equivalent. A single station at/near the existing WWTP with a design flow of 7.98 MGD required an 8-pump configuration: 1+1 (1 duty + 1 stand-by) 60-HP jockey pumps on variable frequency drives (VFDs) and 5+1 250-HP duty pumps. Flows from 0.35 up to 1.15 MGD can be handled by the jockey pumps' VFDs and flows above 1.15 MGD, up to 7.98 MGD, would be handled with a varying number of 250 HP pumps. A single station with so many pumps, in varying sizes, needed a trench-style self-cleaning wet well.

Proposed Concept

Original Concept

Maximize use of existing infrastructure

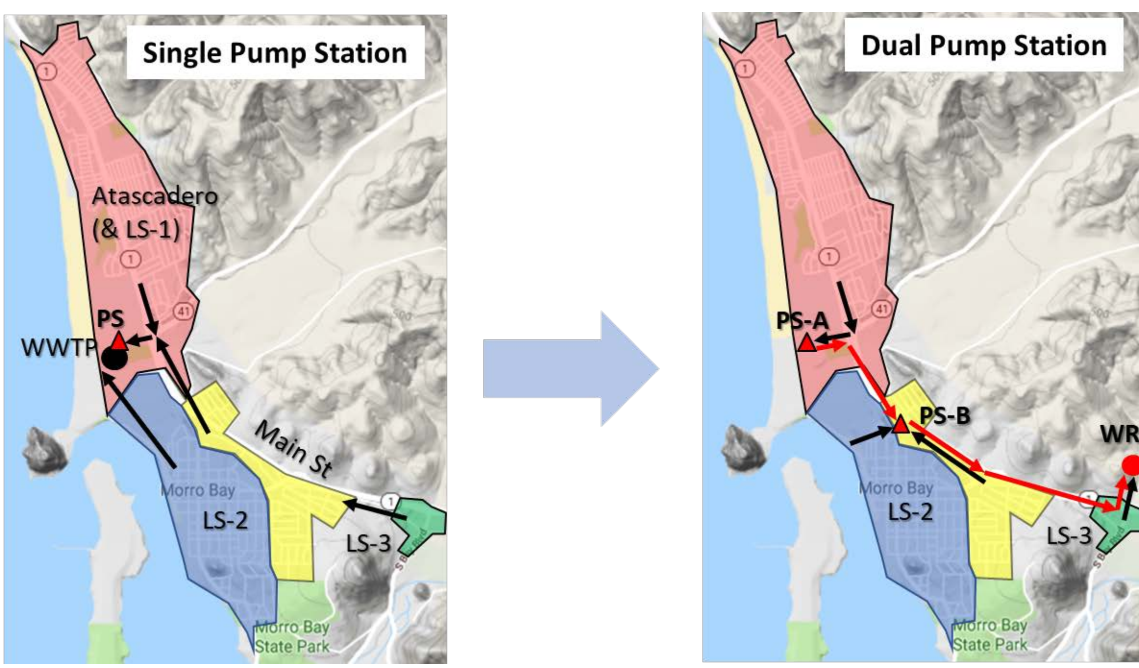


Figure 1-11: New Pump Station Concepts

Scenario 2: Secondary Stormwater PS-B

Scenario 2 was identified as a dual pump station configuration with the intent of downsizing the number of pumps in a single pump station. This allowed the pump stations to have smaller precast wet wells. PS-A was evaluated near the WWTP and a secondary pump station was evaluated at several sites along both the West (W1A, W1B & W2) and Embarcadero (E1 & E2) Alignments. In this scenario, PS-A typically pumped to the WRF, but in instances of PWWF, PS-B was activated via a series of valves. During PWWF, PS-A pumped to PS-B and then PS-B subsequently pumped to the new WRF. This resulted in 2+1 140-HP pumps at PS-A and 2+1 250-HP pumps at PS-B – a total of 6 pumps.

Scenario 3: Full-Time PS-B

Scenario 3 was identified another multiple pump station configuration that was the result of workshop discussions with City staff. The goal for this scenario was to divert enough flows to PS-B and downsize the pumps at PS-A such that both pump stations can be used full-time. The diversion of Lift Station 2 (LS-2) flows was deemed feasible by the current existence of a secondary 8" forcemain that can convey flows in the direction of the West Alignment (when needed). Due to the size, age and unknown condition of the secondary forcemain, for this scenario to be viable with the existing LS-2 pumps, it needed to be replaced with a new primary 12" forcemain. In addition, local gravity manholes near the alternative PS-B sites on the West Alignment were able to be diverted to PS-B – bringing the design flow of PS-A to 5.81 MGD. This scenario resulted in 2+1 60-HP pumps at PS-A and 2+1 250-HP pumps at PS-B – also a total of 6 pumps.

In summary, pump station utilization for each scenario is shown in Table 1-8.

Table 1-8: Pump Station Utilization

Scenario	1	2	3
Description	Single Station	Stormwater Booster (PS-B)	Full-Time Boosters
Non-Storm Flows	LS-2 → PS-A PS-A → WRF LS-3 → WRF	LS-2 → PS-A PS-A → WRF LS-3 → WRF	LS-2 → PS-B or PS-A PS-A → PS-B PS-B → WRF LS-3 →WRF
Storm Conditions	LS-2 → PS-A PS-A → WRF LS-3 → WRF	LS-2 → PS-A PS-A → PS-B PS-B → WRF LS-3 → WRF	LS-2 → PS-B PS-A → PS-B PS-B → WRF LS-3 →WRF

The criteria used for comparing the identified scenarios are summarized below and are further explained in Table 6-5, Table 6-6, and Table 6-7 in Section 6.1.7. They are color coded to aide in indicating which items under specific scenarios are the most beneficial to the City (green = best; red = worst):

Table 1-9: Pump Station Assessment Summary

Key Criteria and Constraints	Single	Dual
# of New Stations	0	-1
Single vs. PS-A Footprint	-1	+1
Standard Wet Well Configuration	-1	+1
Facility Maintenance Impacts	0	-1
Pipe Length for Pigging	-1	+1
LS-2 FM Redundancy	0	+1
Total Score	-3	+2

The anticipated costs for each scenario are summarized in Table 1-10. These costs do not include property acquisition associated with the alternative pump station sites but do reflect 20% construction and 10% design contingency applied to direct construction costs.

Table 1-10: Cost Comparison Summary

Scenario	1	2	3
Estimated Capital Cost	$11.0M	$8.4M	$8.4M
Estimated O&M Cost	$59K	$70K	$83K
Replacement Funds	$230K	$171k	$157K
Estimated 20-yr NPW (O&M + Replacement Funds)	$3.6M	$3.0M	$3.0M
Total NPW	**$14.9M**	**$11.6M**	**$11.6M**

1.4.4 Preferred Pump Station Alternative

Scenario 3 was the preferable alternative as the benefits and costs savings associated with a multi-station setup outweighed that of a single station. Since the City would not have to procure additional property, PS Site 2 – South of Atascadero is preferable for PS-A, and West Site 2 – Main Street at Highway 1 is preferable for PS-B.

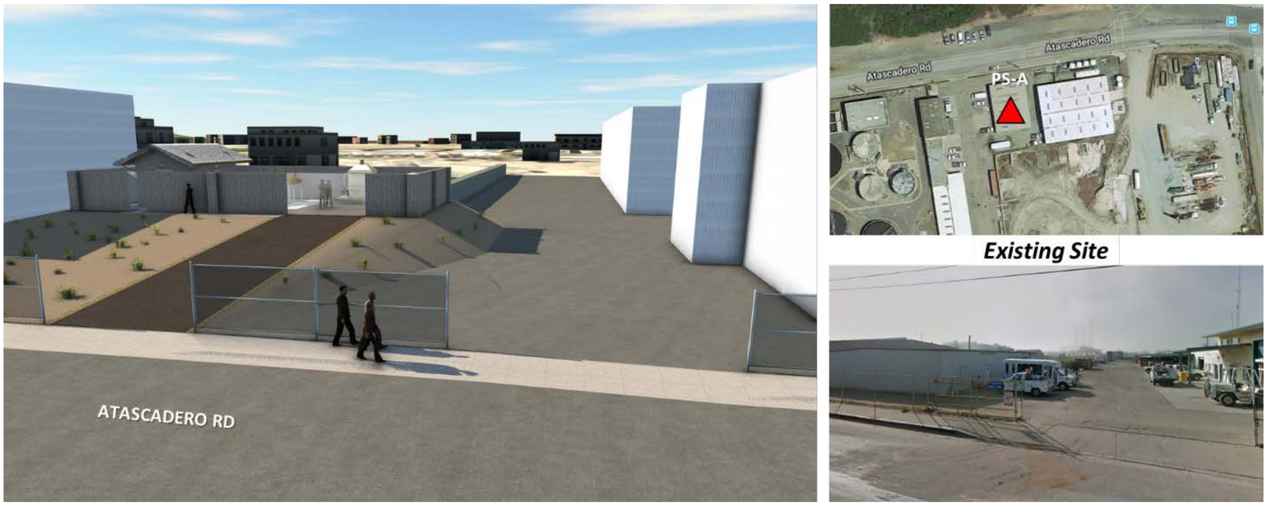


Figure 1-12: PS-A Site

Despite having to maintain two stations, the full-time PS-B booster pump station option:

- eliminates the complexity of pumping operations and valving associated with only using PS-B during PWWF
- has less pumps to maintain than a single station
- has the least amount of idle/unused infrastructure
- has higher velocity, shorter and potential for redundant forcemains

Figure 1-13: PS-B Site

1.5 Preferred Alternative and Total Project Cost

WaterWorks recommends that the City approve the West Scenario 3 with dual forcemains (FPVC or HDPE) as the preferred project alternative which is visualized in the diagram below.

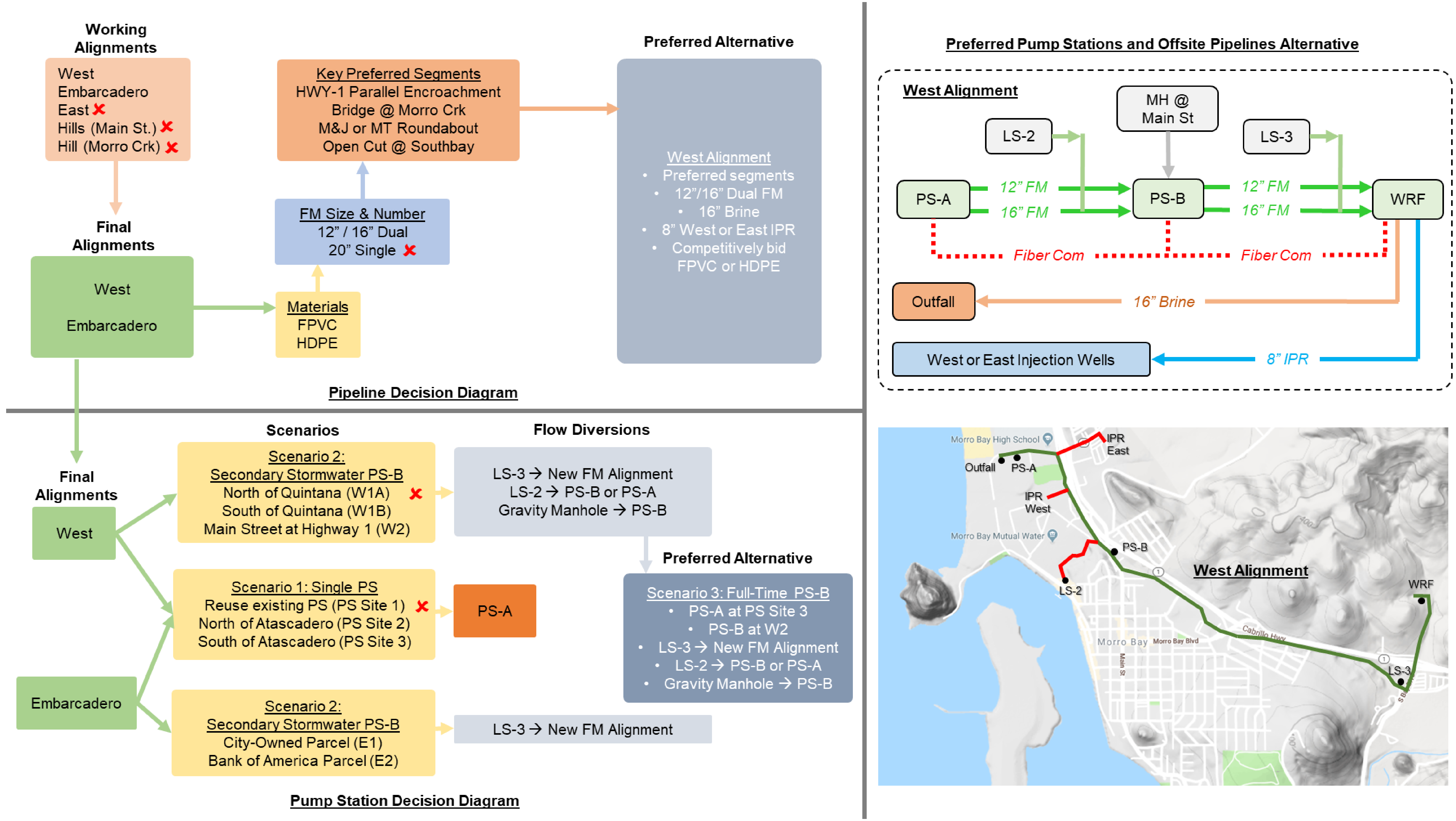


Figure 1-14: Preferred Alternative

A summary of why this alternative best fits the overall project goals is listed below:

- Least expensive alternative that incorporates redundancy via dual forcemain and maximizes long-term operability and ease of maintenance.
- Uses existing City right-of-way or easements where feasible along the West Alignment. Reduces traffic impacts by utilizing a trenchless crossing of the Morro Bay/Quintana roundabout.
- Avoids known cultural resources in the Lila Keiser Park area via the Caltrans parallel encroachment on the SB HWY-1 connector shoulder.
- Leverages local flow diversions via dual pump stations which provide the best flow range per pump station and optimizes pump sizing. In addition, potentially reduces or eliminates sanitary sewer capacity improvement projects on Main St due to the PS-B diversion. In addition, the alternative eliminates the LS-3 forcemain and provides a new LS-2 forcemain.

The total project costs for the Offsite Pipelines and Pump Stations are displayed in Table 1-11.

Table 1-11: WRF Offsite Pipelines and Pump Station Total Project Costs

WRF Offsite Pipelines and Pump Station Total Project Costs*						
Alignment	Item	Offsite Pipelines + Conduit	Pump Station	Additional West 8” IPR	Additional East 8” IPR	Comm. Conduit
West	12"FM + 16"FM + 16"Brine DR 18 FPVC	$ 12,614,700	-	$14,814,700	$15,784,700	$414,700
	14"FM + 20"FM + 20"Brine DR 13.5 HDPE	$ 13,874,700	-	$16,184,700	$17,147,700	$414,700
	Pump Station Estimated Capital Cost	-	$ 8,400,000	-	-	-
	Pump Station Estimated 20-yr NPW (O&M + Replacement Funds)	-	$ 3,004,000	-	-	-

**Reflects 20% construction & 10% design contingency applied to direct construction costs*

1.6 Project Next Steps

The next project deliverable is the 60% PS&E which will incorporate the final alignment, modeled pump stations and pipelines. A summary of next steps in support of this deliverable are as follows:

- Coordinate with WRF Program Team
  - Confirm West or East IPR injection well selection and common alignment tie-in locations
  - Confirm draft surge/air-relief mitigation criteria for the IPR and Brine offsite pipelines given the WRF Design Build team is conducting the hydraulics assessment for those two lines
  - Review any additional biological assessment, coordinate draft environmental permit applications and biological construction support documentation via KMA
  - Review any additional cultural resources studies, coordinate draft cultural resources monitoring and mitigation plan (if required) via Far Western and ESA

    - Confirm communication conduit design criteria and supervisory control and data acquisition (SCADA) standards
- Complete final studies and field investigations
    - Additional utility locating along the alignment and at the pump stations (potholing and/or ground penetrating radar)
    - Final geotechnical field investigation and updated geotechnical criteria
    - Complete design level survey
    - Work closely with the City to examine/negotiate existing easements and draft new plats and legals (P&Ls) for new proposed PE and TCE
    - Prepare a draft Caltrans encroachment permit and draft cultural resources support letter from the City

2 BACKGROUND

2.1 Existing WWTP and Sewer System

The existing Morro Bay – Cayucos Wastewater Treatment Plant (WWTP) (last upgraded in 1981) that jointly serves the City of Morro Bay (City) and Cayucos Sanitary District (CSD) will be replaced with a new Water Reclamation Facility (WRF) located further inland and at higher elevation. The catalyst for this project was when the California Coastal Commission denied a Coastal Development Permit (CDP) to the City to make substantial upgrades to the existing WWTP to comply with new SWRQB NPDES requirements in 2013. Extensive environmental and siting studies identified the South Bay Blvd / HWY-1 site as the preferred location to construct a new WRF. In addition, the City included a recycled water component to augment existing City water supplies via indirect potable reuse via injection wells. To support the new WRF location and proposed injection wells, new lift station(s) are required to divert wastewater flows from the old WWTP site to the new WRF site, along with pipelines to convey treated water to the injection wells and to the existing ocean outfall. This is summarized in Figure 2-1 below.

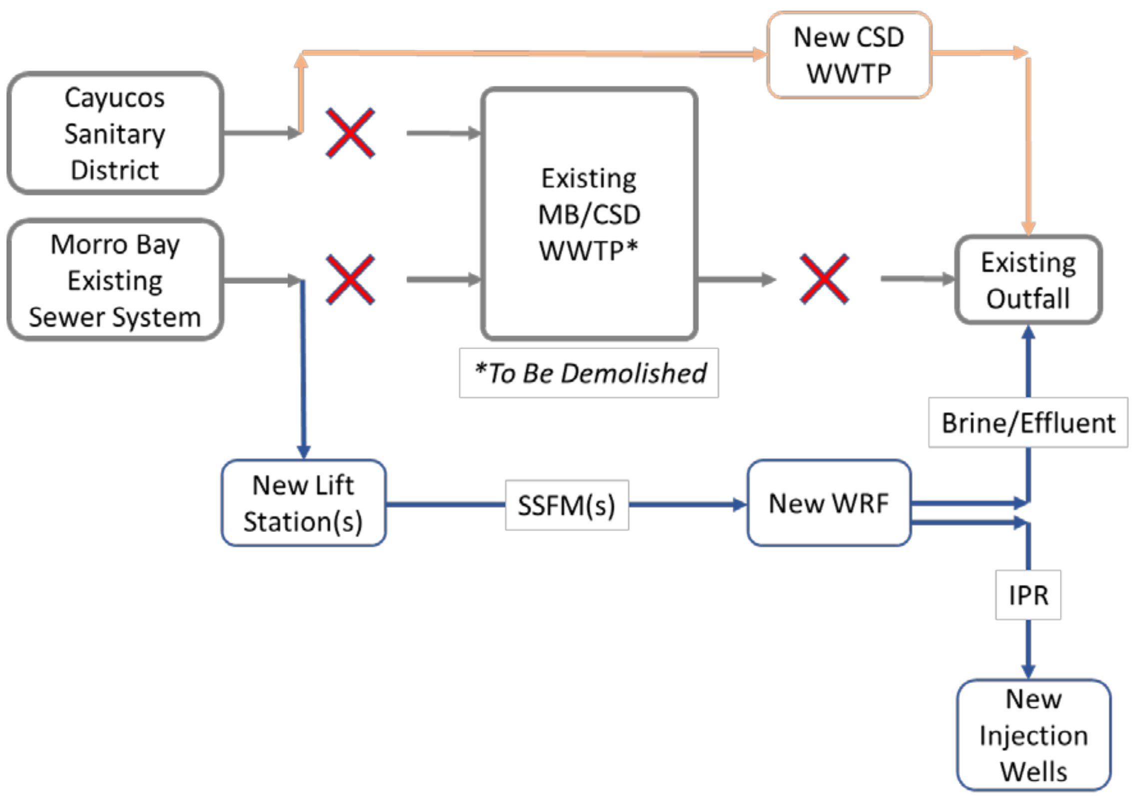


Figure 2-1: New Wastewater System Schematic

The existing Morro Bay sanitary sewer collection system incorporates 14 separately identifiable sewer basins. Note that three lift stations' forcemains operate within the system, which are highlighted in Figure 2-2 and Figure 2-3 below.

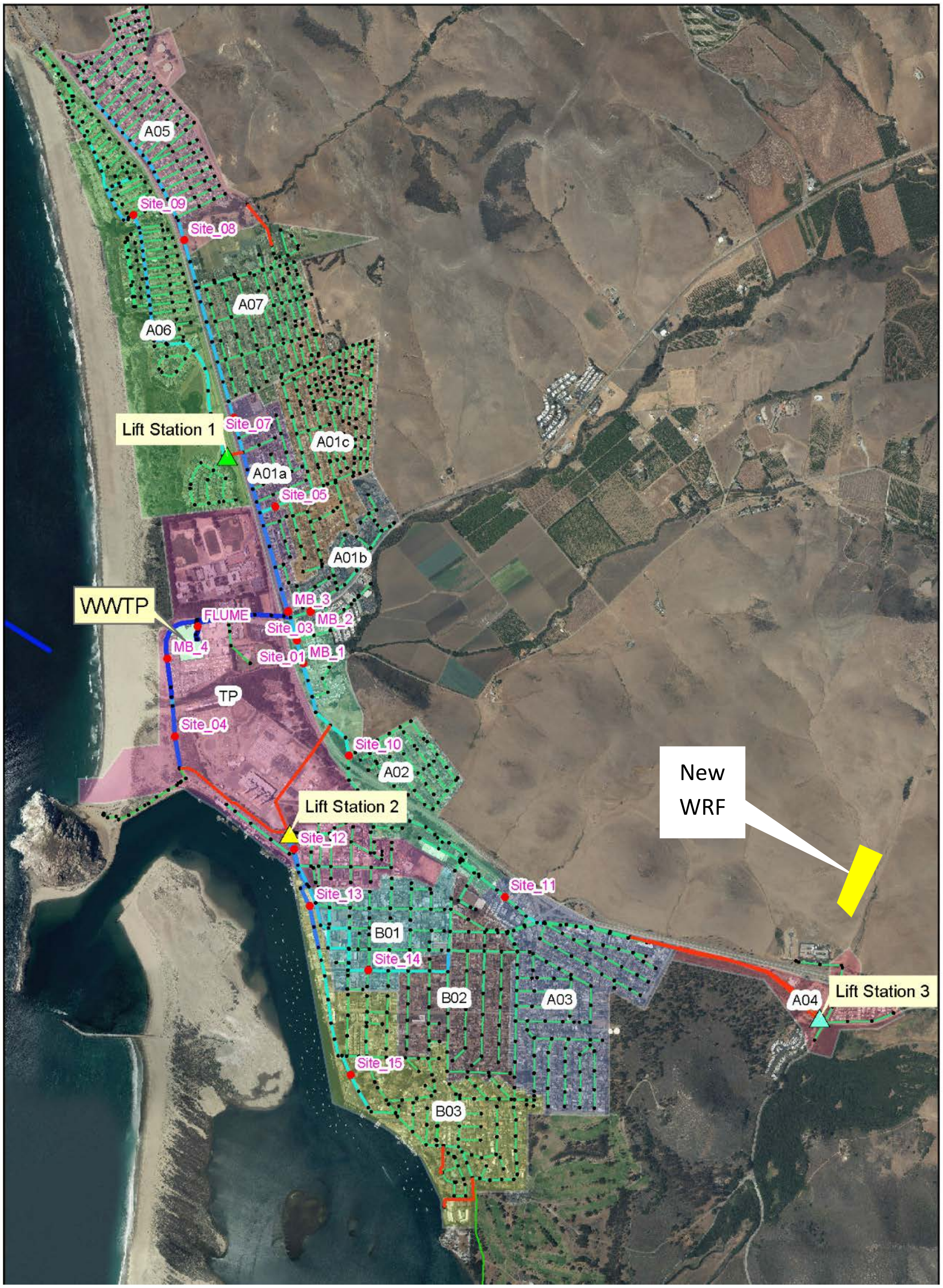


Figure 2-2: Existing Wastewater Collection System Map

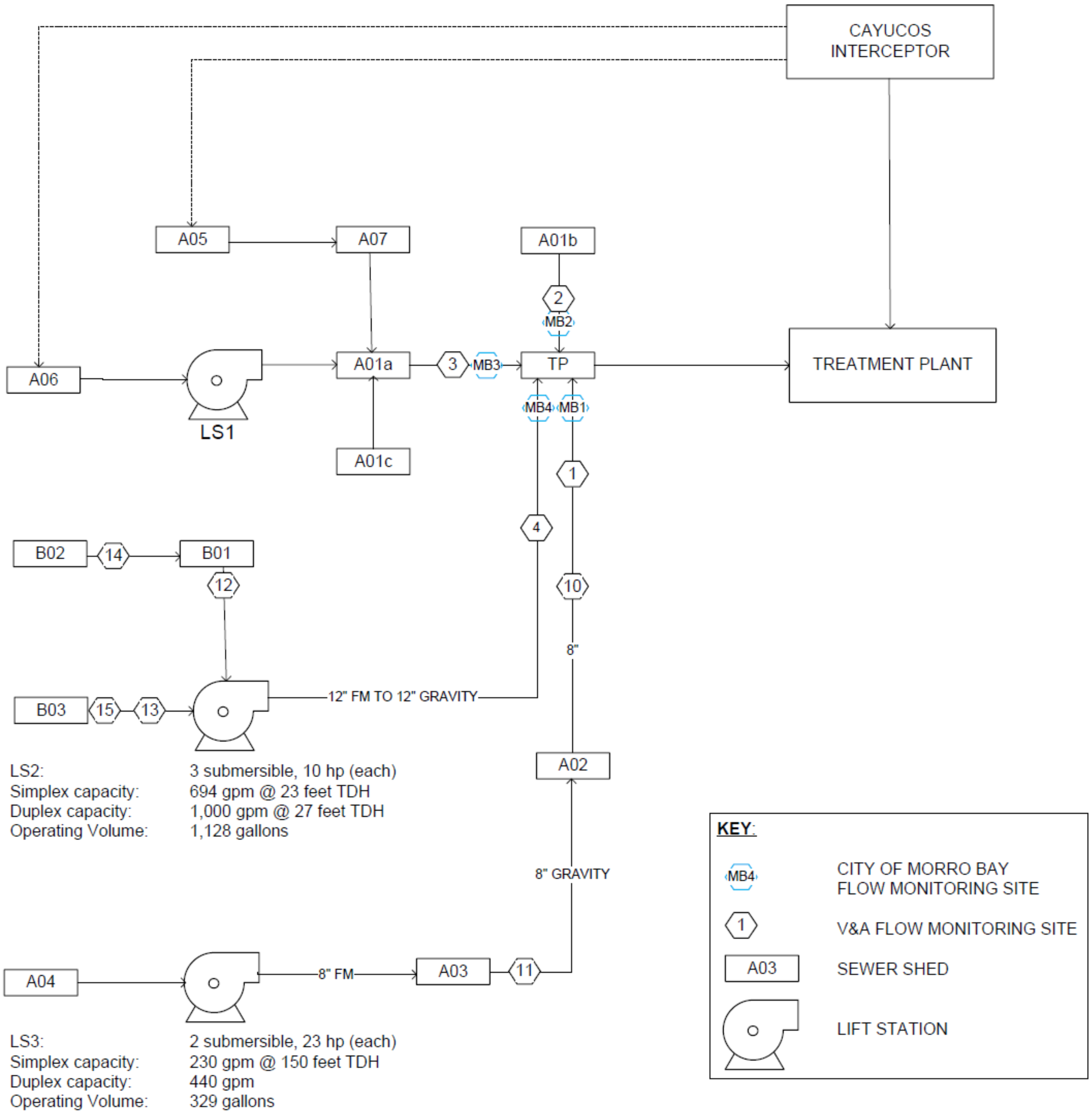


Figure 2-3: Existing Process Flow Diagram (PFD) Schematic

2.2 Project Objectives

WaterWorks Engineers, LLC (WaterWorks) is under contract with the City of Morro Bay (City) to provide professional engineering services for the Water Reclamation Facility (WRF) Lift Stations and Offsite Pipelines Project. The overall project objectives are:

- Identify, Develop, Assess, and Recommend via workshops and a final Concept Design Report (CDR)
    - Site, design criteria, and project constraints for WRF Lift Station(s)
    - Alignment, design criteria, and project constraints for the offsite pipelines (sewer forcemains, brine line, and communication conduit), as well as the indirect potable reuse line (IPR).
- Complete final design PS&E of recommended lift stations and pipelines with focus on cost effectiveness, long term quality and viability, and schedule compliance
    - Incorporate City staff input into final design
    - Coordinate closely with WRF Program Team
    - Provide engineering services during construction

A graphical representation of the role of WaterWorks in the overall WRF Project is visualized below.

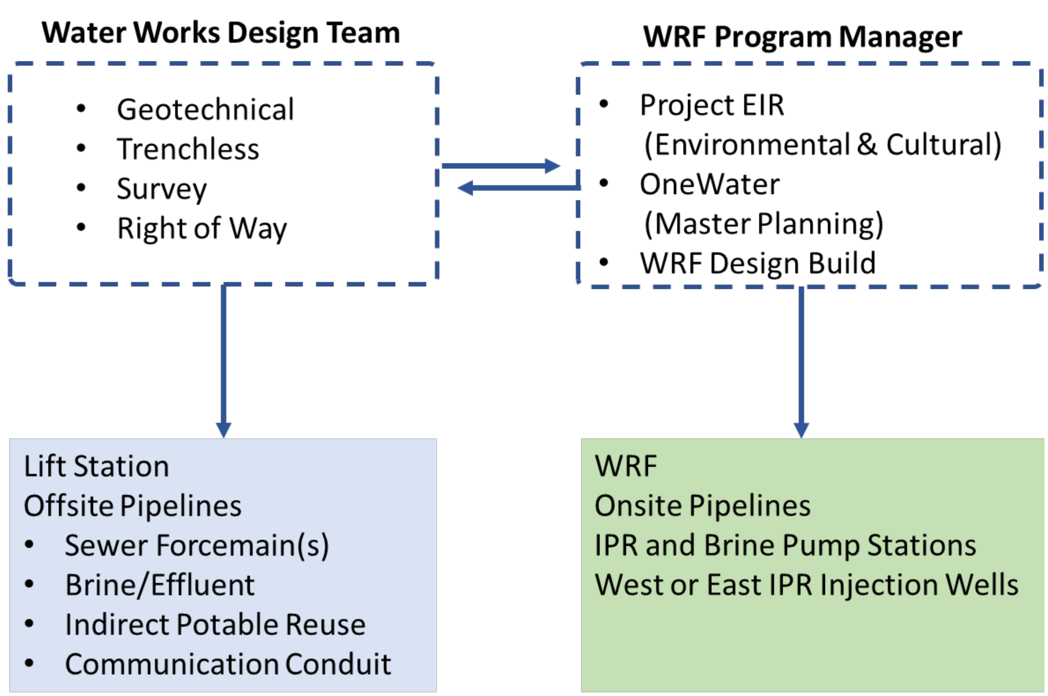


Figure 2-4: Project Team

3 DESIGN FLOWS

3.1 Wastewater Flows

WaterWorks received flow meter data and hydraulic model results from the City to facilitate the sizing of new infrastructure. This section reviews and summarizes this data which was subsequently used to identify design flows for the water reclamation facility pump station and offsite pipelines.

3.1.1 OneWater Hydraulic Model Results

Existing Conditions – WWTP Influent Flume

The City of Morro Bay has influent flume data for the existing WWTP since 2011. The data collected is graphically represented in Figure 3-1. It is to be noted that this data includes the flows of Cayucos Sanitary District (Cayucos). Currently and in the future, Cayucos is pursuing its own efforts such that the two agencies will operate independent of one another.

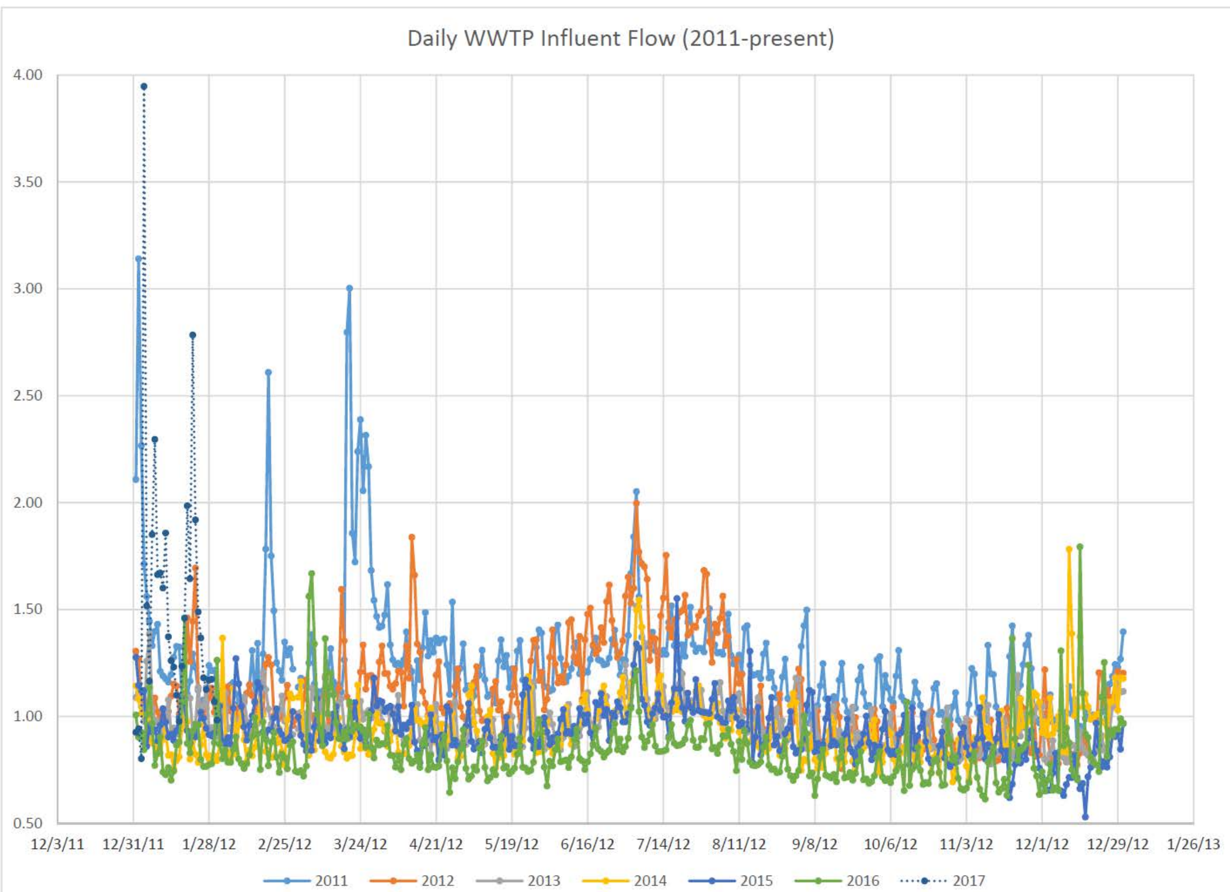


Figure 3-1: Existing WWTP Flume data

Historical data is important in evaluating flow as it relates to time of year and possible effects of water conservation. Anomalies are also more readily identifiable when patterns can be established. The data shows patterns of high flow during typically wet weather months as well as times of high tourism.

Existing Conditions – Studies

In addition to the influent flume data, the City also conducted a “2017 Sewer Flow Monitoring and Inflow/Infiltration Study” and a “2017 Wastewater Treatment Plant Flume Services” with V&A Consulting Engineers. These studies included the installation of numerous flow meters and data collection which were then modeled and analyzed by Carollo Engineering, Inc. (Carollo). This additional information helped separate Cayucos flows as well as identify some data that could have been affected by the limitations of the current WWTP influent flume design. The monitoring and influent WWTP data are summarized below:

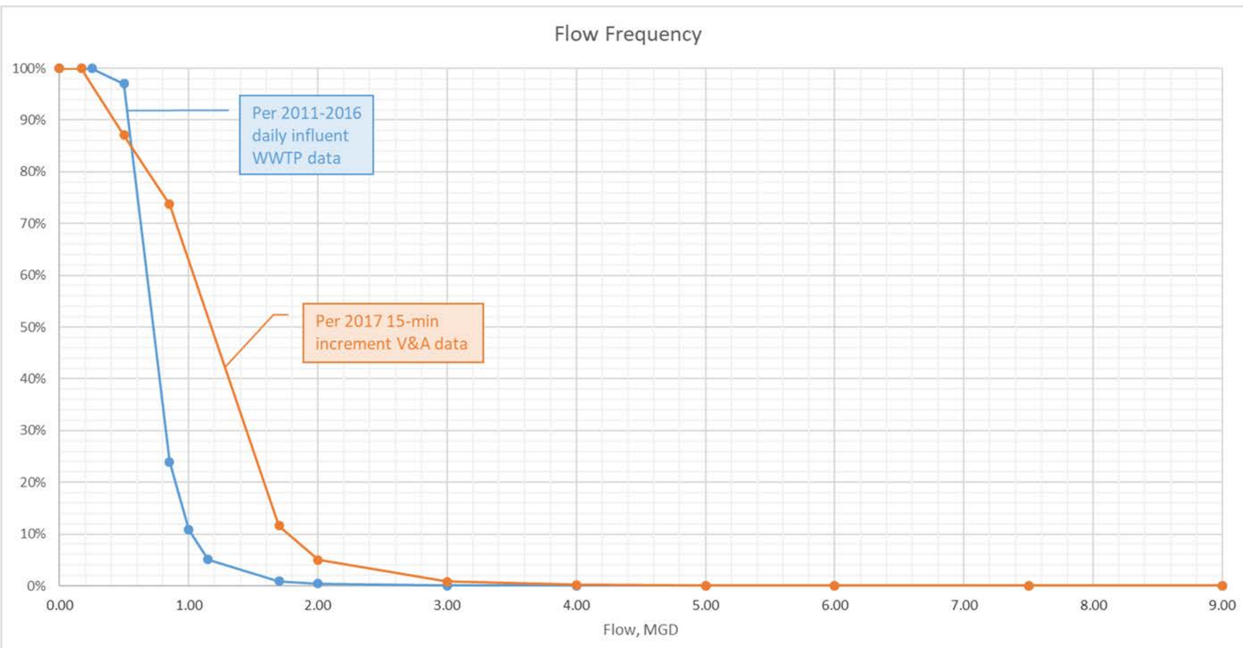


Figure 3-2 Frequency Curve

From this historical frequency curve, it can be derived that 95% of the time, flows are less than 1.15 MGD. This includes a maximum daily recorded flow of ~4 MGD as well as one count at 8.14 MGD from the 2017 V&A data.

Using the analysis provided by Carollo, a master planning effort known as “OneWater” is being conducted in parallel to this project. The following summarizes the historical flows as acknowledged by OneWater, which is the basis of design flows for the Water Reclamation Facility Lift Station and Offsite Pipeline Project.

Historical Wastewater Flows

		Morro Bay Influent(1)				Cayucos Influent(1)		Total Influent(1)		ADWF/Annual Average Ratio		
Year	City of Morro Bay Population	Annual Avg (mgd)	Per Capita Flow (gpcd)	ADWF(2) (mgd)	Per Capita Flow (gpcd)	Annual Avg (mgd)	ADWF(2) (mgd)	Annual Avg (mgd)	ADWF(2) (mgd)	Morro Bay	Cayucos	Total
1997	9,696	1.34	138	1.33	137	0.32	0.30	1.66	1.63	0.99	0.94	0.98
1998	9,845	1.58	161	1.61	164	0.38	0.34	1.96	1.96	1.02	0.91	1.00
1999	9,871	1.40	142	1.47	149	0.29	0.30	1.69	1.77	1.05	1.04	1.05
2000	10,410	1.50	144	1.55	148	0.27	0.26	1.77	1.81	1.03	0.97	1.02
2001	10,486	1.21	115	1.14	109	0.27	0.26	1.48	1.40	0.95	0.95	0.95
2002	10,510	0.89	84	0.79	76	0.25	0.28	1.14	1.08	0.90	1.11	0.94
2003	10,510	0.79	75	0.82	78	0.27	0.29	1.06	1.11	1.04	1.09	1.05
2004	10,522	0.81	77	0.79	75	0.28	0.29	1.09	1.08	0.98	1.02	0.99
2005	10,270	0.94	91	0.93	90	0.32	0.31	1.25	1.24	0.99	0.99	0.99
2006	10,491	0.86	82	0.88	84	0.33	0.34	1.19	1.22	1.02	1.04	1.02
2007	10,436	0.82	79	0.85	81	0.27	0.28	1.09	1.22	1.03	1.04	1.12
2008	10,506	0.82	78	0.85	81	0.28	0.28	1.10	1.22	1.04	1.00	1.11
2009	10,555	0.83	78	0.85	81	0.26	0.27	1.09	1.22	1.03	1.02	1.12
2010	10,234	0.88	86	0.87	85	0.31	0.28	1.19	1.16	0.99	0.93	0.98
2011	10,296	0.94	91	0.94	92	0.31	0.30	1.24	1.24	1.01	0.97	1.00
2012	10,331	0.85	82	0.94	91	0.25	0.26	1.10	1.20	1.10	1.05	1.09
2013	10,427	0.73	70	0.74	71	0.24	0.25	0.96	0.99	1.02	1.05	1.03
2014	10,547	0.72	68	0.74	70	0.22	0.22	0.93	0.96	1.03	1.02	1.03
2015	10,601	0.74	70	0.77	73	0.19	0.20	0.93	0.97	1.05	1.04	1.05
2016	10,714	0.62	57	0.60	56	0.23	0.22	0.84	0.83	0.98	0.98	0.98
2010-14 Average		0.82	79	0.85	82	0.26	0.26	1.08	1.11	1.03	1.00	1.02
2015-16 Average		0.68	64	0.69	65	0.21	0.21	0.89	0.90	1.02	1.01	1.02
2010-14 Maximum		0.94	91	0.94	92	0.31	0.30	1.24	1.24	1.01	0.97	1.00

Notes:

(1) Source: City of Morro Bay Influent Data.

(2) ADWF is defined as the weighted average flow during the months May through September.

Figure 3-3: OneWater Historical Wastewater Flows

Future Conditions

Existing flows as well as build-out flows for 2040 were provided by Carollo and are listed in Table 3-1. Flows were categorized as: High Winter, High Summer, Average Annual (ADWF) and Low Summer. High Winter flows are representative of wet weather flows during the high use times in winter months, Peak Wet Weather Flow (PWWF). High Summer flows capture high flows during peak tourist times in the summer months, Peak Dry Weather Flow (PDWF). Low Summer flows are representative of the lowest flows in the year, summer months with low tourist activity.

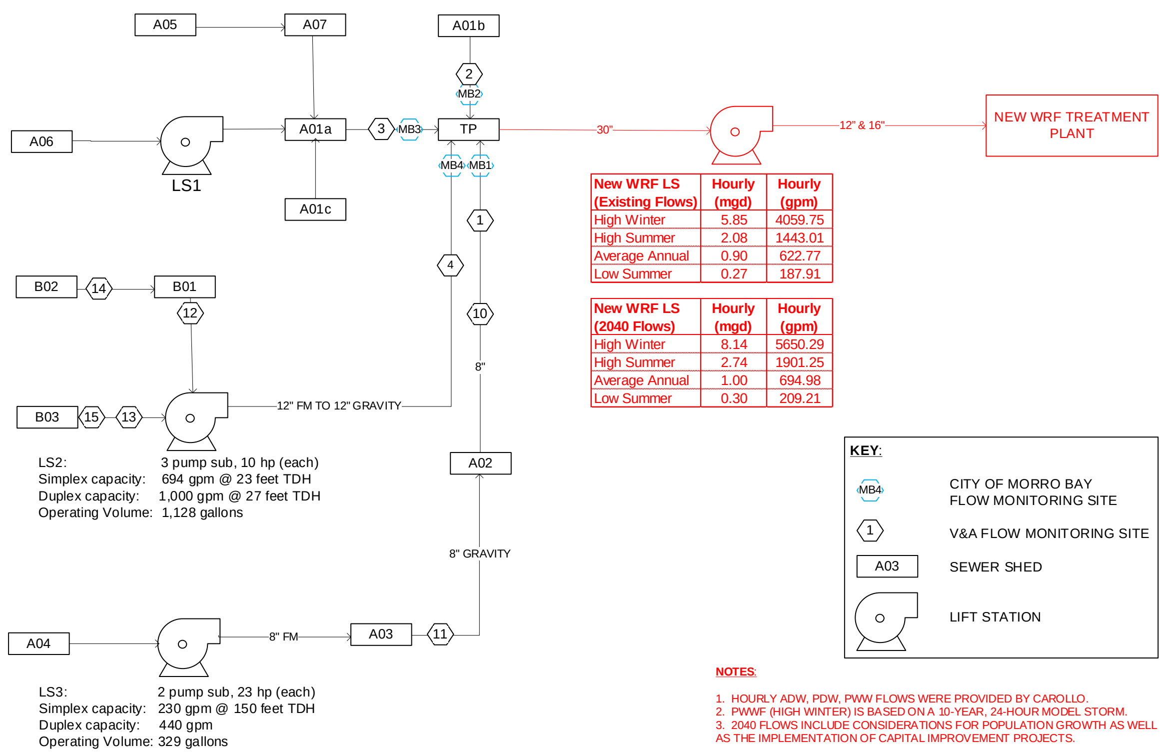
Table 3-1: Wastewater Design Flows

Existing Conditions*			2040 Future Conditions		
Flow Regime	Hourly (MGD)	Hourly (gpm)	Flow Regime	Hourly (MGD)	Hourly (gpm)
High Winter	5.85	4,060	High Winter	8.14	5,650
High Summer	2.08	1,443	High Summer	2.74	1,901
Average Annual	0.90	623	Average Annual	1.00	695
Low Summer	0.27	188	Low Summer	0.30	209

**Does not include CSD existing flows which are not incorporated in the WRF project*

3.1.2 Wastewater Design Flow Criteria

In order to size the offsite pump station(s) and pipeline, an understanding of how current flows get to the existing WWTP was necessary. Flows from numerous sewer basins and lift stations convey wastewater to the exiting WWTP. As is such, all flows entering the WWTP would need conveyance to the new WRF. Based on the design flows identified, new infrastructure needs to be sized for 2040 buildout system flows of 8.14 MGD (which accounts for the removal of Cayucos). The following is a schematic process flow diagram of how the existing system and proposed improvements would be integrated. A new lift station or combination of lift stations would take all flows which currently go to the influent flume at the existing WWTP and collectively get them to the new WRF. Items in black represents existing infrastructure while items in red are representative of improvements.



New WRF LS (Existing Flows)	Hourly (mgd)	Hourly (gpm)
High Winter	5.85	4059.75
High Summer	2.08	1443.01
Average Annual	0.90	622.77
Low Summer	0.27	187.91

New WRF LS (2040 Flows)	Hourly (mgd)	Hourly (gpm)
High Winter	8.14	5650.29
High Summer	2.74	1901.25
Average Annual	1.00	694.98
Low Summer	0.30	209.21

NOTES:

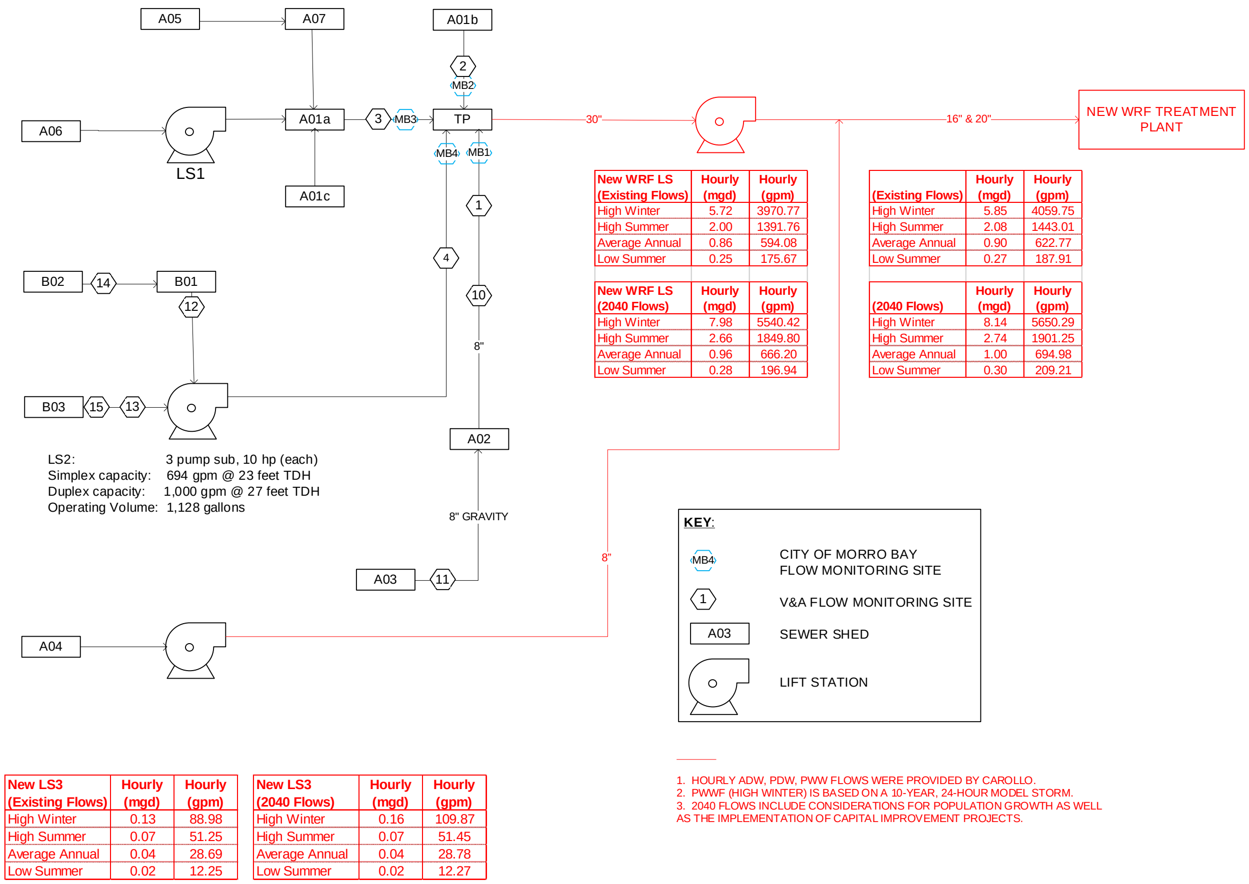
1. HOURLY ADW, PDW, PWW FLOWS WERE PROVIDED BY CAROLLO.
2. PWWF (HIGH WINTER) IS BASED ON A 10-YEAR, 24-HOUR MODEL STORM.
3. 2040 FLOWS INCLUDE CONSIDERATIONS FOR POPULATION GROWTH AS WELL AS THE IMPLEMENTATION OF CAPITAL IMPROVEMENT PROJECTS.

Figure 3-4: PFD Schematic

Lift Station 3 Diversion

Lift Station 3 (LS-3) is located in close proximity to the proposed WRF. It is WaterWorks' recommendation that flows from LS-3 be pumped directly to the new forcemains as illustrated in Figure 3-5, rather than double pumped to the existing WWTP and back to the proposed WRF.

The existing pumps at Lift Station 3 have the capacity of 230 gpm each. The existing TDH for the pumps is 150 ft. With a 2040 projected peak wet weather flow of 110 gpm and a similar anticipated TDH to the new WRF, preliminary analysis expects that the existing pumps can be reused with adequate capacity to handle the diversion to the new SSFM.



New WRF LS (Existing Flows)	Hourly (mgd)	Hourly (gpm)
High Winter	5.72	3970.77
High Summer	2.00	1391.76
Average Annual	0.86	594.08
Low Summer	0.25	175.67

(Existing Flows)	Hourly (mgd)	Hourly (gpm)
High Winter	5.85	4059.75
High Summer	2.08	1443.01
Average Annual	0.90	622.77
Low Summer	0.27	187.91

New WRF LS (2040 Flows)	Hourly (mgd)	Hourly (gpm)
High Winter	7.98	5540.42
High Summer	2.66	1849.80
Average Annual	0.96	666.20
Low Summer	0.28	196.94

(2040 Flows)	Hourly (mgd)	Hourly (gpm)
High Winter	8.14	5650.29
High Summer	2.74	1901.25
Average Annual	1.00	694.98
Low Summer	0.30	209.21

New LS3 (Existing Flows)	Hourly (mgd)	Hourly (gpm)
High Winter	0.13	88.98
High Summer	0.07	51.25
Average Annual	0.04	28.69
Low Summer	0.02	12.25

New LS3 (2040 Flows)	Hourly (mgd)	Hourly (gpm)
High Winter	0.16	109.87
High Summer	0.07	51.45
Average Annual	0.04	28.78
Low Summer	0.02	12.27

Figure 3-5: Proposed Lift Station 3 Diversion

Alternative Scenarios

For this CDR, several alternatives and scenarios were examined for conveying existing and buildout flows to the new WRF. Different combinations of forcemain sizes, material types as well as pump station configurations and locations are presented later in this report. The general flow scenario alternatives for are as follows:

1. Constructing a single pump station at existing WWTP and the associated forcemain(s)
2. Constructing dual pump stations (PS-A and PS-B) and their associated forcemain(s)

These scenarios were considered along several alignments, which are further discussed in Chapter 4.

1. West Alignment
2. East Alignment

3. Embarcadero Alignment
4. Hills Alignment

The evaluation of the configuration and location(s) of the pump station(s) is discussed in Chapter 5.

3.2 Brine and IPR Flows

The WRF design build team will be designing the onsite effluent/brine and IPR pump stations and which will govern the maximum pressures and flow rates that the brine and IPR pipelines will see. However, the brine pipeline is designed to match the peak design inflow for the sewer forcemain(s), but ideally will be operating at very low flows ("brine" flows with high TDS concentrations) when the tertiary treated recycled water is pumped via the IPR line into the groundwater injection wells at a maximum design flow 0.93 mgd. The anticipated maximum design flow for the brine line is 8.14 mgd. This is summarized in Table 3-2 below.

Table 3-2: Brine/IPR Design Flows

Pipeline	Brine Pipeline	IPR
Max Flow (mgd)	8.14	0.93

4 OFFSITE PIPELINES ROUTE STUDY

WaterWorks conducted a comprehensive route study to select a preferred offsite alignment and pipeline option that focused on cost effectiveness, long term quality and viability, and schedule compliance. The study is organized into the following elements:

1. Overview of working alignment alternatives
2. Overview of assessment methodology
3. Development of all pipeline design criteria and pertinent project constraints
4. Assessment of the working alignment alternatives and refinement into final alignment alternatives
5. Selection of a preferred alternative based on project costs and non-cost, constraint-based considerations

4.1 Alignment Alternatives

4.1.1 Inlet & Discharge Locations

Pump Station and WRF Tie ins

The connection locations for the pump station(s) and at the discharge location on the WRF site will be controlled via isolation valves and will incorporate above-grade valving to facilitate pigging operations. It is currently assumed that that the tie in elevation at the WRF site is 143.5 ft and will freely discharge into a headworks structure. The preliminary location was based on the conceptual site plan laid out in the 2017 WRFP Master Plan (Black & Veatch). It is understood that this location and elevation are "worst-case" and that it may be further optimized and improved during the WRF design-build process. The design of the onsite pipelines (Edge of Teresa Rd onwards) will be conducted by the WRF Design Build Team. It is assumed that WaterWorks will provide shallow stubbed connection points at the edge of Teresa Rd. It is also assumed that the City will require that the same pipe material be utilized to the WRF discharge.

Outfall

The brine pipeline will connect to the existing air relief structure at the existing WWTP or at the 27" outfall line. Various options for connecting to the outfall are discussed herein.

4.1.2 Working Alignment Alternatives

WaterWorks identified five working alignments for further assessment which is summarized below and presented in Figure 4-1.

- **West Alignment** – Runs east along Atascadero, southeast along the existing bike path, and down Quintana Rd parallel to HWY-1 on the southwest side. This alignment was originally identified as the "west" alignment because it is located west of HWY-1.

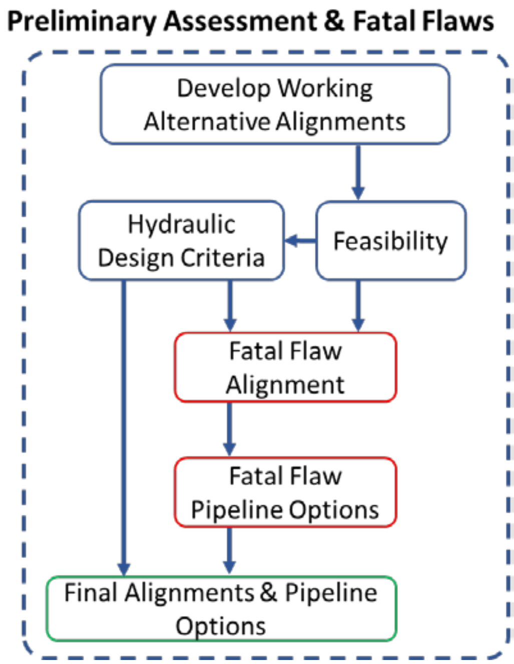
- **East Alignment** – Runs east along Atascadero, and then southeast on Main St and then via new easement parallel to HWY-1 on the northeast side. This alignment was originally identified as the “east” alignment because it is primarily located east of HWY-1. Note that there is a hybrid version of this alignment, whereby the West Alignment is utilized to Main St, and then the pipelines run north to the Radcliff area.
- **Embarcadero Alignment** – Runs west and then south along Embarcadero, then east along Pacific, and Quintana parallel to HWY-1 on the southwest side.
- **Hills-Creek Alignment (Little Morro Creek – open cut, or Long HDD)** – Runs east along Atascadero, northeast along HWY-41 and then cuts across the rolling hills above the City and into the County limits. This represents the shortest possible alignment to the WRF
- **Hills-Radcliff Alignment (Radcliff Avenue – Long HDD)** – Runs east along Atascadero, along Main St and Radcliff Avenue and then cuts through rolling hills above the City and crosses into the County limits.

Figure 4-1: Working Alignment Alternatives (note that Hills Main St is also known as the "Hills-Radcliff" alignment)

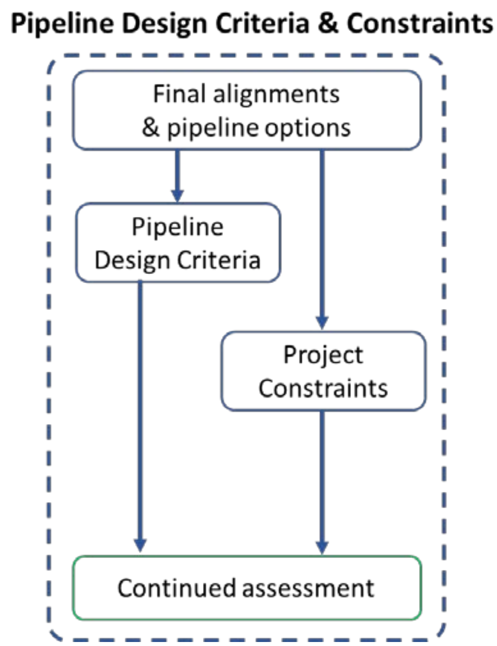
4.2 Alternatives Assessment Methodology

The working alternative alignments previously identified were assessed via comprehensive process which is outlined below:

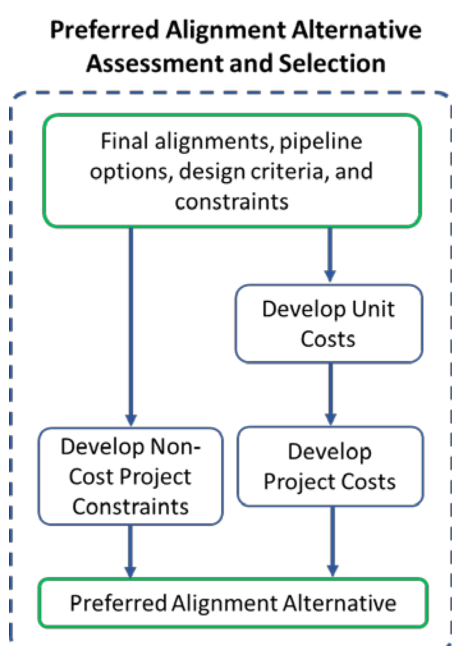
1. Identify multiple pipeline options (varying by number, size, and material) for each working alignment alternative
2. Conduct a preliminary assessment and identify fatal flaws that disqualify the alternative based on constructability issues and hydraulic design criteria.



3. Identify additional pipeline design criteria and project constraints that affect the final alignments
    a. Pipeline Hydraulics (pipe material, number and size of pipelines)
    b. Construction Methodology
        i. Open Cut Construction Criteria
        ii. Trenchless Design & Feasibility
    c. Hwy-1 / Hwy-41 Crossing
    d. Utility Conflicts
    e. Morro Creek / Drainage Crossing
    f. Quintana Roundabout Crossing
    g. Right-of-way, Easement, Encroachment
    h. Geotechnical
    i. Traffic Control and Fencing
    j. Cultural Resources
    k. Environmental
    l. Concerns of Outside Stakeholders



4. Develop unit costs that will contribute to direct construction costs (estimate of the contractor's bid costs). Develop indirect costs that will affect the City prior to or during construction (not reflected in the contractor's bid costs). Finally, assess the final alignment alternatives based on total project costs (direct + indirect) and non-cost considerations.



5. Select and recommend an alignment alternative for design and construction.

4.3 Pipeline Design Criteria & Constraints

WaterWorks identified preliminary criteria and constraints to inform the route study. Preliminary design criteria incorporates pipeline materials, sizes, construction methodology, burial depth, and other items. In comparison, project constraints determine how the design criteria can be implemented when encountering site-specific issues such as a highway or drainage crossing, traffic impacts or other items.

4.3.1 Pressurized Pipeline Materials

The offsite pipelines (except for the communication conduit) are functionally pressurized pipelines that will be made of typical materials listed in Table 4-1.

Table 4-1: Pressurized Pipeline Materials

<table>
<tr><th rowspan="2">Parameters</th><th colspan="6">Offsite Pipeline Material (FM, Brine, IPR)</th></tr>
<tr><th>High Density Polyethylene (HDPE)</th><th>Fused Polyvinyl Chloride (FPVC)</th><th>Polyvinyl Chloride (PVC)</th><th colspan="2">Ductile Iron Pipe (DIP)</th><th>Fiberglass Reinforced Polymer Mortar (FRPM)</th></tr>
<tr><td>Lining / Coating</td><td>None</td><td>None</td><td>None</td><td colspan="2">Ceramic Epoxy / Zinc or Asphaltic+PE or Ceramic Epoxy</td><td>None</td></tr>
<tr><td>Class</td><td>DR13.5 IPS PE4710 (SF 1.6)</td><td>DR18 DIPS C-900 (SF 2.0)</td><td>DR 18 DIPS C-900 (SF 2.0)</td><td>CL350 (TR Flex)</td><td>CL50 (MJ)</td><td>SN46-200</td></tr>
<tr><td>Working Pressure</td><td>160 PSI (FM) – SF 1.6
130 PSI SF – 2.0</td><td>235 PSI (FM)</td><td>235 PSI (FM)</td><td colspan="2">350 PSI</td><td>200 PSI (FM)</td></tr>
<tr><td>Fitting & Connection Style</td><td>Butt fused - Flange + MJ Adapter</td><td>Butt fused – MJ & Flange-MJ Adapter</td><td>Segmental – MJ</td><td colspan="2">Segmental – Locking, MJ</td><td>Segmental - Non-restrained coupling, Flange + MJ adapter</td></tr>
<tr><td>Typical fitting material</td><td>Mitered butt fused HDPE</td><td>DIP fitting with Ceramic Epoxy lining/coating</td><td></td><td colspan="2">DIP fitting with Ceramic Epoxy lining/coating</td><td></td></tr>
<tr><td>Hazen-Williams C Coefficient</td><td>140</td><td>130</td><td>130</td><td colspan="2">140</td><td>140</td></tr>
</table>

Segmental vs Continuous (fused)

Segmental pipe and continuously fused pipe have different advantages that can be leveraged to meet project constraints. Segmental pipe is typically easier to install in high density utility corridors when there are many utility crossings. On long clear stretches of alignment, however, continuously fused pipe has an advantage, whereby long sections of fused pipe can be efficiently installed in a trench and the contractor can leverage the lack of utility crossings. Fused pipelines also significantly reduce the number of mechanical joints, which offers better long-term reliability (i.e. joints are more likely to fail than pipe), and often allow closer construction to parallel water utilities. Fused pipe typically does not have the same longitudinal restraint requirements as segmental pipe. Segmental pipes typically require an additional mechanical coupling or thrust block for full restraint. Due to their size and immobility, thrust blocks are typically not preferred.

PVC

Fusible and segmental Polyvinyl Chloride (PVC C-900 Ductile Iron Pipe Size-DIPS) was identified as a potential material and is considered a very common sewer forcemain material (corrosion-free). PVC is petroleum-based thermoplastic made of a strong and amorphous (long & linear) polymer cell structure. A consequence of this structure is that it has a high tensile/pull strength (relative to HDPE, for instance), but in the event of a catastrophic pressure-induced failure, PVC will likely crack and split linearly along the pipe. Conversely, the HDPE failure mechanism is a radial bursting failure at one location due to the semi-crystalline polymer cell structure of HDPE.

From a pipe life cycle analysis, PVC experiences increased pipe fatigue than HDPE in scenarios with high surges and frequent pump cycling over the pipe lifecycle (HDPE flexibility effectively dampens shockwaves). Thus, an additional transient analysis will be conducted during the design phase to identify additional PVC pressure requirements beyond the system working pressure (pumped) that is comparatively utilized for HDPE pipe. At this preliminary stage, DR18 C-900 PVC (235 PSI) was identified as the likely maximum pipe class necessary, but this will be revisited and refined during the design phase.

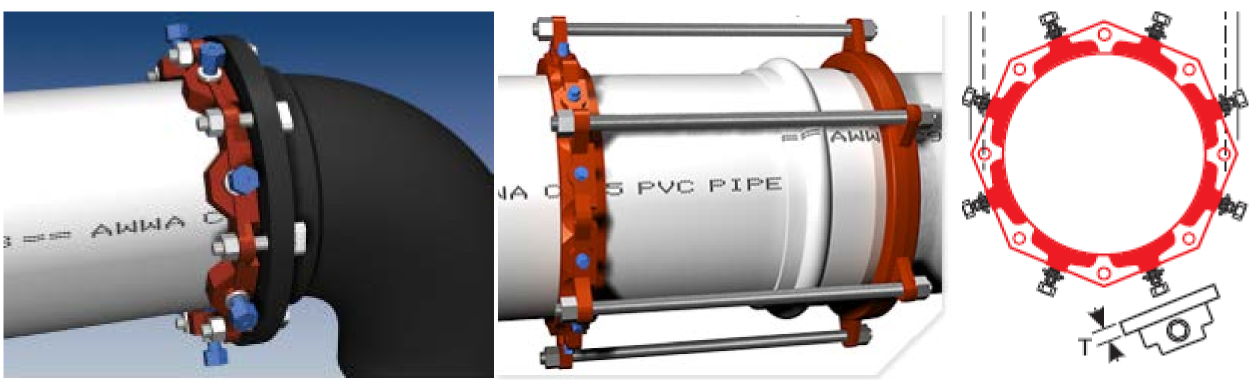


Figure 4-2: PVC Mechanical Joint (L) & PVC Restraining Harness (M) & wedge locking restraint section (R)

Fusible PVC has the advantage of minimizing the number of joints (only at connection points with appurtenances), while segmental PVC would incorporate joints every 20 lineal feet (LF). Segmental pipe theoretically increases the locations and causes for potential failure. A mechanical joint, flange adapter, and restraining harness (see Figure 4-2) would be used for PVCxPVC and PVCxDIP (fitting) connections. In addition, linear pipe restraint (natural soil reaction) and thrust blocks would be utilized, wherever prudent. A sample minimum linear pipe restraint length for 16” PVC at a 90-degree horizontal bend is displayed in Figure 4-4 below.

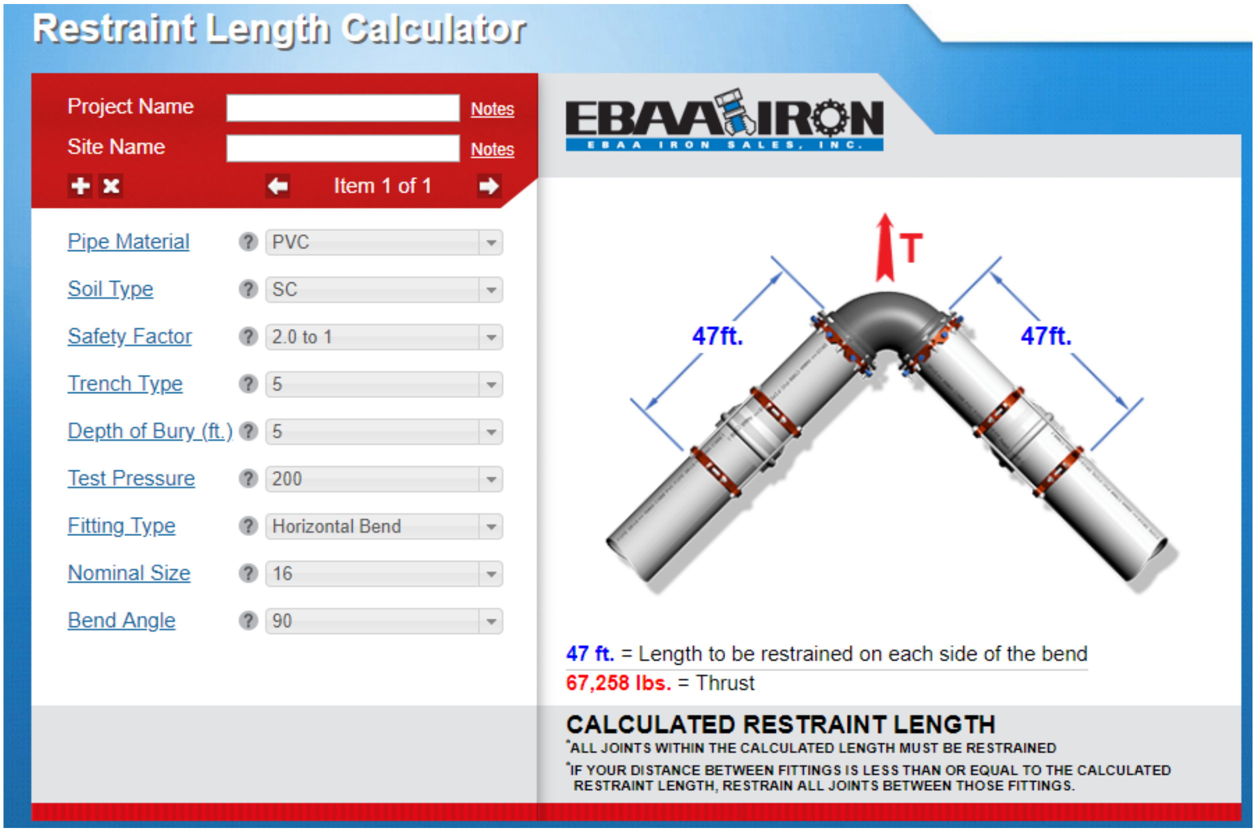


Figure 4-4: Sample 16" PVC Pipe Restraint Length

Fusible PVC is butt fused (heat from friction) via a fusion machine operated by a crew of two or more subcontractors and is typically laid out for long sections on the side of the trench as the primary contractor is excavating and preparing trench (see Figure 4-3 below). The pipe can then be dropped into place and/or longitudinally shifted under a transverse utility crossing.

Figure 4-3: Butt Fusion Machine for FPVC

HDPE

HDPE is generally 2-3x thicker than PVC to achieve a similar pressure rating, and since it is made of semi-crystalline polymeric cell structure, it can be effectively butt fused at much larger diameters than FPVC (limited to 36”). HDPE has the disadvantage of having a relatively large OD with iron pipe sizing (IPS). HDPE can be supplied in ductile iron pipe sizing (DIPS), but it is not common to the West Coast and is likely not competitive in pricing. This means that any connection between HDPE and fittings (DIPS sizing) requires a ductile iron reducer along with a fused flanged or MJ adapter. This connection can take up more linear space and requires a larger vault if the connection is to not be buried (see Figure 4-5 below). Because of spacing requirements, an ARV/blowoff/CARV connection with HDPE is best accomplished via sidewall fusion in lieu of an independent tee fitting (which is what PVC would require).

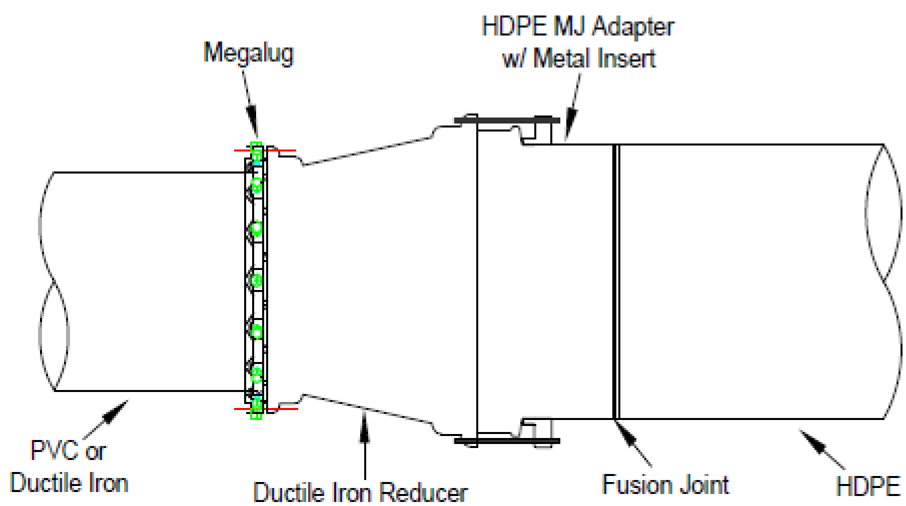


Figure 4-5: HDPE to fitting connection

HDPE is formulated with different strength resins that offer different performance characteristics. HDPE 4710 (often listed as PE100) is the newest and most common HDPE resin formulation utilized for sewer applications. The previous resin that was commonly utilized (HDPE 3608) achieves a working safety factor of 2.0, but the newest resin 4710 has effectively decreased the safety factor from 2.0 to 1.6. During the design phase, it may be necessary to effectively upsize the HDPE PE4710 thickness to achieve a 2.0 safety factor, in which case a DR13.5 pipe would be increased to DR11, for example. Alternatively, the maximum working pressure rating for the HDPE pipe could be de-rated to achieve a 2.0 safety factor (i.e., 160 psi class for DR13.5 is really 130 PSI).

DIP

Ductile iron pipe is a flexible pipe material commonly used in pressurized piping systems. The primary benefits of DIP are that it is a high strength material which can be utilized in shallow bury application (< 3 feet). It is not corrosion proof, however, and requires lining and coating. It is generally more expensive pipe material over PVC/HDPE, utilizes joints, and in this preliminary stage is not a preferred material for the forcemain(s). It may be best suited for tight mechanical configurations at pump station tie-ins.

4.3.2 Communication Conduit Material

As part of the overall project, the City is linking the SCADA system at the pump station(s) to the WRF via a communication conduit for the fiber optic communication line directly between the new lift station(s) and WRF within the trench of the sewer forcemains. Based on preliminary design criteria provided by the City, an approximately 4” diameter (DIA) pipe is the maximum size required for the communication line. At this preliminary design level, it is assumed that a high-strength corrosion proof pipe material such as 4”DIA SCH 40 segmental PVC will be utilized. Several other potential pipe materials are listed in Table 4-2 below. Conduit material for trenchless crossings will be further refined during the design phase.

Table 4-2: Fiber Optic Conduit Materials

Parameters	Offsite Fiber Optic Conduit Materials		
	Rigid		Flexible
	High Density Polyethylene Pipe (HDPE)	Polyvinyl Chloride Pipe (PVC)	PE Conduit (coilable)
Fitting & Connection Style	Butt fused (≥4”); electrofusion coupling	Butt fused (≥4”); segmental solvent welded; mechanical coupler	Mechanical coupler
Type	SDR 9-17 typical	SCH 40 or SCH 80	SDR 9-17 typical
Description	Medium – High Stiffness	High stiffness	Low stiffness; cable in conduit optional (preinstalled)
Pull Length* (approximate)	1000 LF	1000 LF	< 1000 LF or N/A
Relative ease of installation	Hard	Hard-Moderate	Easy
Relative Cost Differences	High cost if fused	Medium cost if segmental; high cost if fused;	Low cost

4.3.3 Pipeline Casing Materials

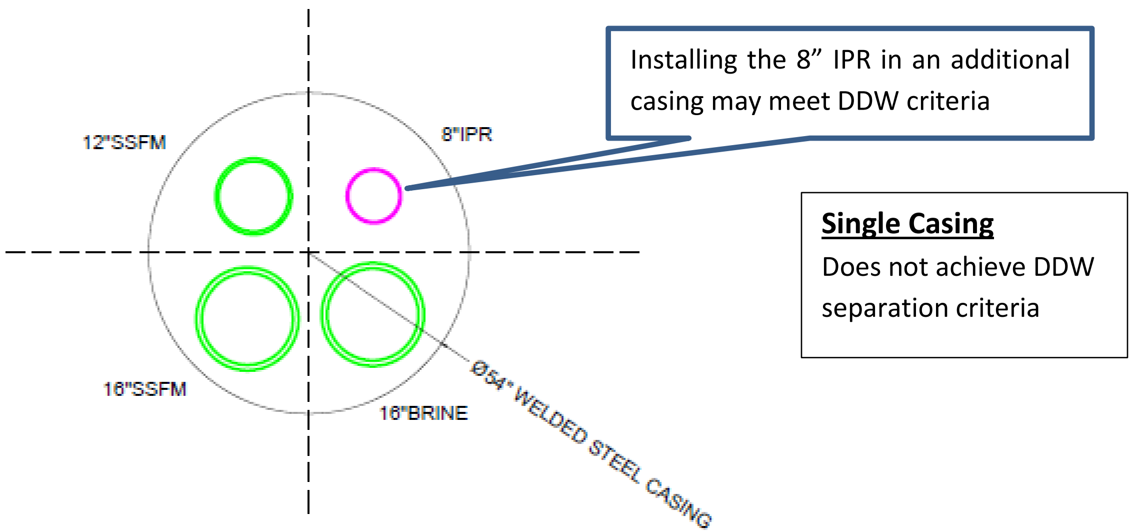
Various locations that will require casings for trenchless or open cut construction will be designed to meet the governing design criteria. In most instances, the casing design will meet Caltrans standards. Recommended casing materials are listed in Table 4-3 below.

Table 4-3: Casing Materials

Construction Methodology	Material	Range of Sizes	Caltrans standard
Open Cut	HDPE	10” and upwards	Yes
	WSP (1/4” thickness min)	10” and upwards	Yes
	FPVC	10” up to 36”	Exception required
	FRP	10” and upwards	Exception required

The thickness of the casing material in open cut applications will be designed to meet dead and live load criteria. The casing material in a trenchless application is typically designed for dead load and safe pulling force (or thrust force).

To capitalize on economy of scale (better wholesale pricing) it will be preferable to match the casing material with the carrier pipe (brine, IPR, SSFMs) if possible, but the recommendation will also depend on the casing configuration. HDPE, steel, and fiberglass reinforced plastic (FRP) for instance can utilize larger sizes that would facilitate 3 carrier pipes in a single casing. FPVC would likely only be able to house 2 carrier pipes. This is highlighted in the casing diagrams below.



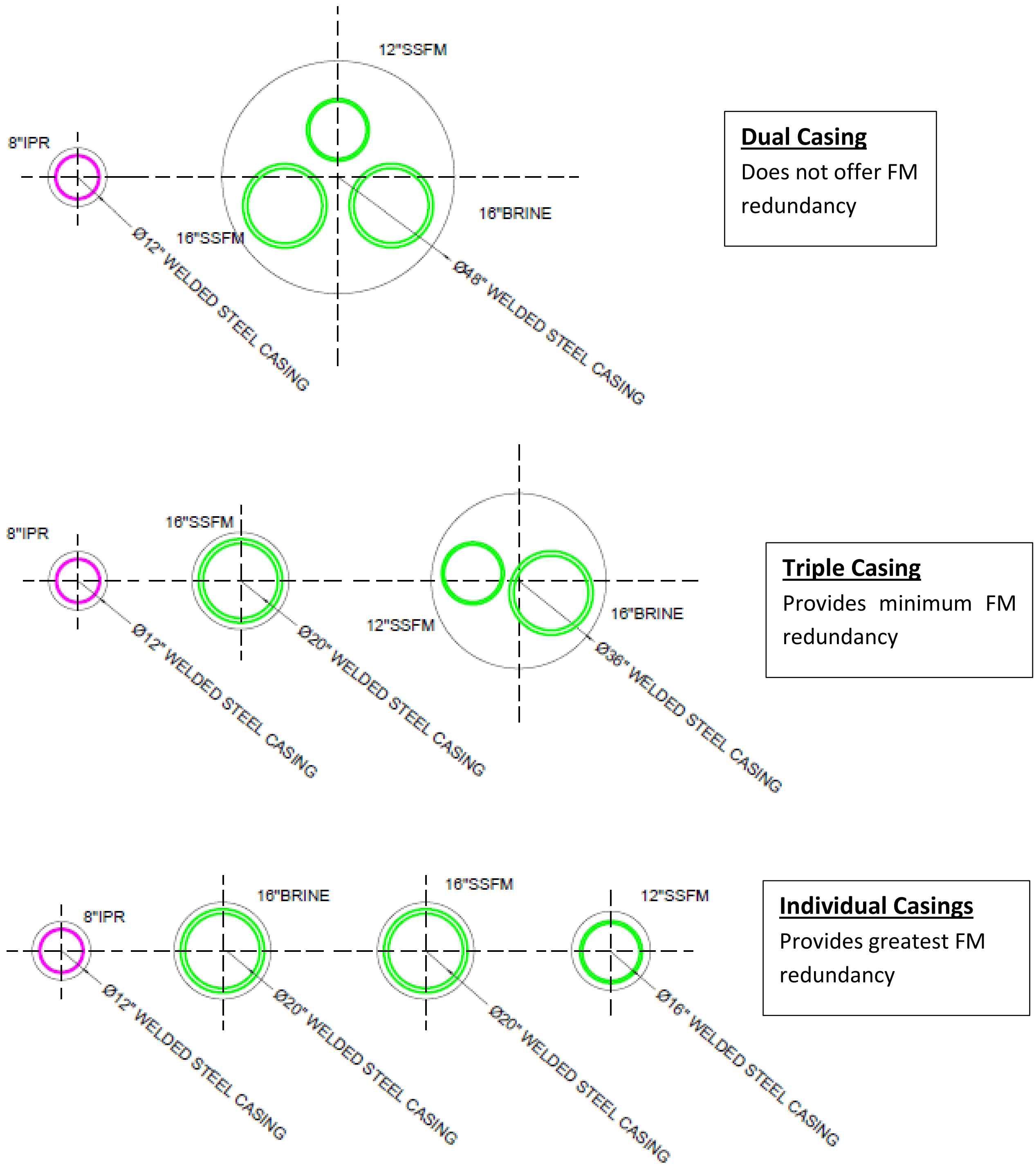


Figure 4-6: Casing Configurations

4.3.4 Pipeline Size and Hydraulics Criteria

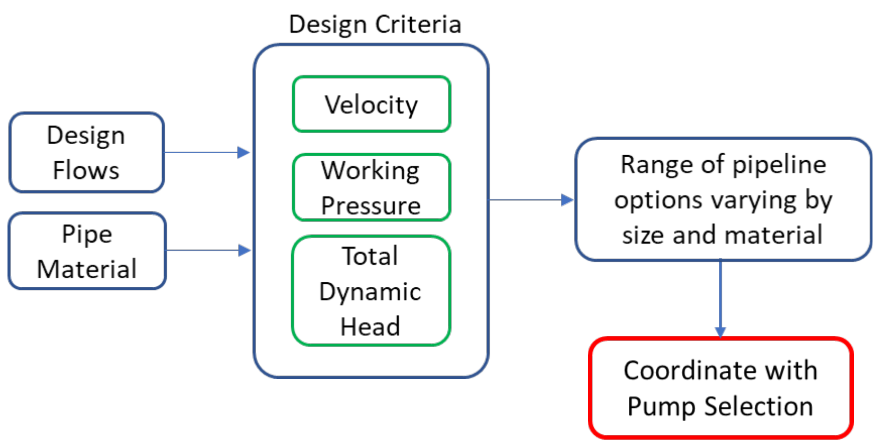
Sewer Forcemains

The design flows utilized in the sizing of the offsite pipelines are presented in detail in Chapter 3 and are summarized below in Table 4-4.

Table 4-4: Design Flows

Flow Conditions	Existing Scenario (peak hourly MGD)	2040 Scenario (peak hourly MGD)
High Winter	5.85	8.14
High Summer	2.08	2.74
Average Annual	0.90	1.00
Low Summer	0.27	0.30

Based on these design flows, several design criteria were utilized to develop various potential sizes for the forcemain(s), which is visualized in the diagram below.



Velocity

It is recommended that the velocity in a pressurized sewer forcemain operate in an optimal range to reduce long term maintenance needs. That optimal range is between 2 and 8 feet per second (ft/s). Velocities outside this range have effects to the pipeline which require mitigation as outlined below.

Table 4-5: Forcemain Velocities

	Sustained Low Velocities < 2 ft/s	Optimal Velocities 2 ft/s ≤ V ≤ 8 ft/s	High Velocities > 8 ft/s
Velocity effect	Solids deposition: - Gas pocket accumulation; odor production (solubilized or gas); H2S induced corrosion in crown - Reduction in cross sectional area (headloss) - Sliding beds (pipe bed abrasion)	Typical Conditions	Increased headloss, turbulence, surge/water hammer, cavitation issues
Mitigation	Consistent daily cleaning cycles with higher velocities; seasonal "pigging"; corrosion-proof pipe material; odor control units at ARVs	As-needed maintenance	Larger pumps at PS; more robust surge and air-relief valving/controls

Working Pressure

The maximum working pressure of the pipeline is found immediately downstream of pumps before static head (elevation rise) and dynamic head (velocity-based headloss) in the pipe decreases the pressure over the length of the pipe. The maximum working pressure will be confirmed once the pump station design is formalized, however it is anticipated that pressures will not exceed 130 psi.

TDH

The total dynamic head (TDH) of a pressurized pipeline is the calculated maximum pressure that a downstream pump would theoretically have to pump "against" to achieve a fully filled pipe at the highest elevation in the alignment:

> Maximum TDH at Peak Flow =
>
> Dynamic Headloss (inlet to discharge)
>
> + Static Head (relative elevation from inlet to highest point in the alignment)

The maximum available TDH given an alignment could be driven by the available pipe pressure class, but it is more commonly limited by the practicalities associated with centrifugal pump motor sizes. Based on the preliminary centrifugal pump sizing for the various pump station alternatives (as delineated in Chapter 5) is understood that maximum practical TDH is 300 ft.

Brine Pipeline

The WRF design build team will be designing the onsite effluent pump station which will govern the maximum pressures and flow rates that the brine pipeline will see. The brine pipeline is designed to match the peak design inflow for the sewer forcemains, but ideally, will be operating at very low flows ("brine" flows with high TDS concentrations) when the tertiary treated recycled water is pumped via the IPR line into the groundwater injection wells.

IPR Pipeline

The WRF design build team will be designing the onsite IPR pump station which will govern the maximum pressures and flow rates that the IPR pipeline will see. However, the IPR pipeline is designed to discharge approximately 70% of typical average dry weather flow (ADWF) from the WRF and up to approximately 0.93 MGD into the groundwater injection wells

Surge & Air Relief Design Criteria

Surge & air relief/vacuum valves will be incorporated at key locations along the alignment to address the hydraulic conditions reflected in Table 4-6 and Figure 4-7.

Table 4-6: Surge and Air Relief Hydraulic Conditions

Hydraulic Condition (in increasing severity)		Mitigation
Case 1	**Low volume air/gas discharge** Cause: Accumulation of air pockets to be vented Effect: Air pockets can restrict flow and increase headloss and decrease energy efficiency. Air pockets can also cause surges or water hammer if they move. Depending on the pipe material, exposure to air in the pocket becomes a point of corrosion and potential failure.	Air Release Valve (ARV)
Case 2	**Regular operation** Cause: Pump/pressure cycles, flow changes. Effect: Water momentum (i.e., hammer, or surge) cycles forward and backwards and develops pressure spikes that concentrate at location of flow stoppage or where flow substantially changes (i.e., velocity increase/decrease). Cyclical pressure spikes are eventually dampened; however high cycling can cause eventual pressure de-rating fatigue in host pipe (especially PVC), or can cause damage to non-flexible/brittle ceramic liners of DIP.	Combination Air Release Valve (CARV) + Surge Relief; Surge Anticipator Valve (at PS); Pump-Well bleed line (at PS)
Case 3	**High volume air intake – vacuum relief – pipeline draining** Cause: Pipeline draining for maintenance; catastrophic pipeline failure Effect: To drain water from the pipeline, air needs a way in otherwise a vacuum will form which could potentially collapse the pipe.	CARV; vacuum breaker; check valve

Case 4	**High volume air/gas discharge – pipeline filling** Cause: Pipeline filling for initial service or maintenance Effect: Air pockets can develop when filling a pipe. As previously mentioned, air pockets restrict flow, increase headloss, decrease energy efficiency, potentially cause surges or water hammer and can become a point of corrosion and potential failure.	ARV/CARV; vacuum breaker
Case 5	**Surge dampening – high velocity air/gas discharge, liquid column separation & liquid oscillation** Cause: Catastrophic pump failure; pump power failure; valve sudden closure; operator closing "the wrong" valve. Effect: At high pressures/velocities/developed pipeline length: Water column (plug flow) continues moving forward initially and produces a vacuum behind it (e.g., vapor cavity or air-water column separation); pipe can respond by "pancaking" and collapsing, and/or water column will eventually be pulled back into place by vacuum and cause pipe at connection point to burst.	CARV + Surge Relief; Surge Anticipator Valve; Pump-Well bleed line; check valve

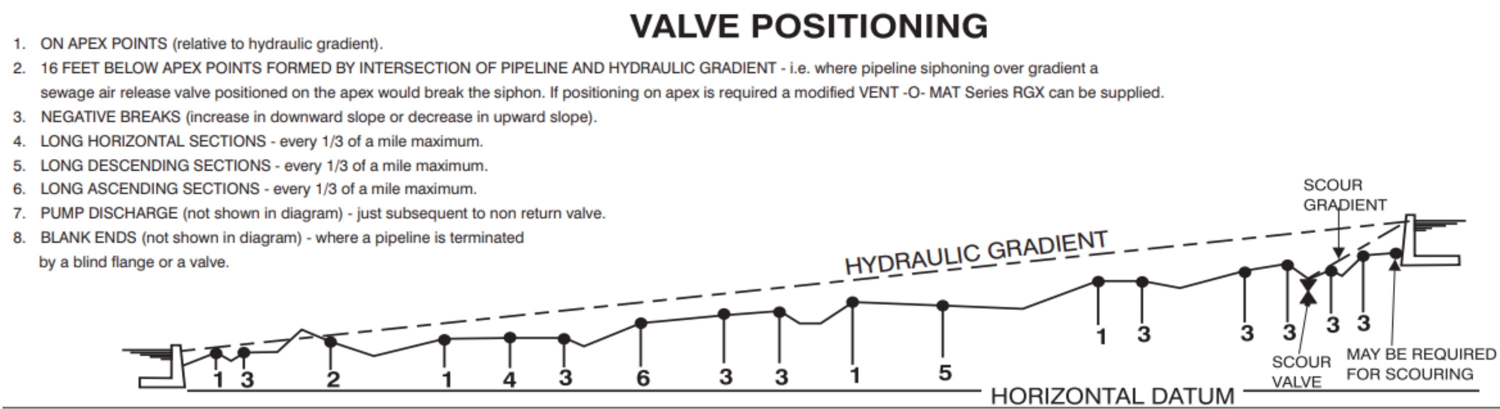


Figure 4-7: Sample Surge/Pressure/Air Valve Placement Criteria

Combination Air Relief Valves

There are a variety of different styles of sewer-capable combination air valves (+ vacuum breaker, and + surge control). Some sample configurations are displayed in Figure 4-9 below. Note that ARVs are assumed to be located in subgrade structures for this preliminary analysis but can be located above grade as well which would likely reduce costs. A sample ARV manhole configuration is displayed in Figure 4-8. Odor control units can also be installed and connected to air release valves if required (see next section).

Figure 4-9: Sample Combination Air Release Valves

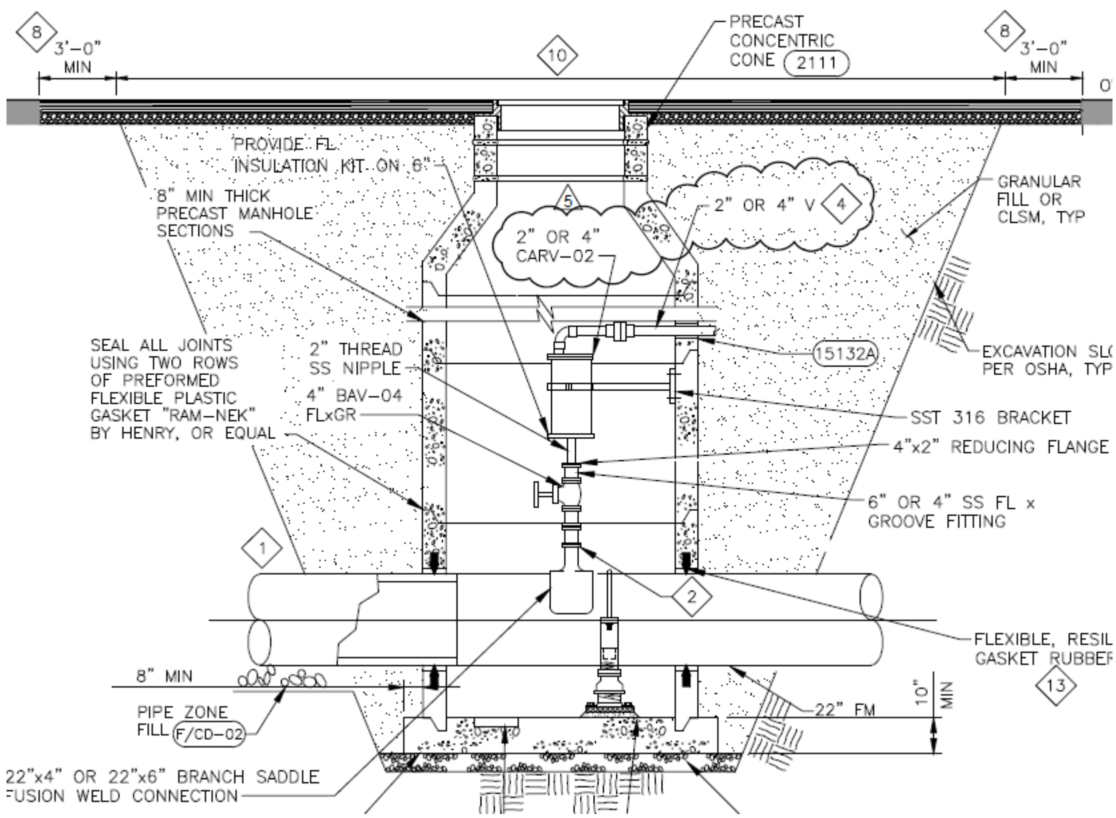


Figure 4-8: Sample ARV MH configuration

Forcemain Odor Control

Small odor control units can be added to an ARV vault, whereby for any vented air from a combination of forcemains (maximum of two) can be directed towards a single odor control unit. An above grade odor control unit is typical for this application due to the small volumes of air that are released. The need for odor control units at air release valves are a function of sensitive receptor proximity and expected odor production in the wastewater (e.g., from low velocities). Adding odor control units to air relief valves add cost by increasing vault size, easement acquisition, and are additional sources of maintenance. To reduce overall project costs, it is recommended that the City limit their use to areas that may impact residents and commercial properties, as listed in Table 4-7.

Table 4-7: Odor Control Units

Odor Control Units in ARV vaults	Cost
Residential/Commercial Areas Only	Low
+ Recreational areas	Medium
+ Remote areas	High
All Locations	Highest

A sample subgrade odor control unit layout in a valve vault is displayed in Figure 4-10 below.

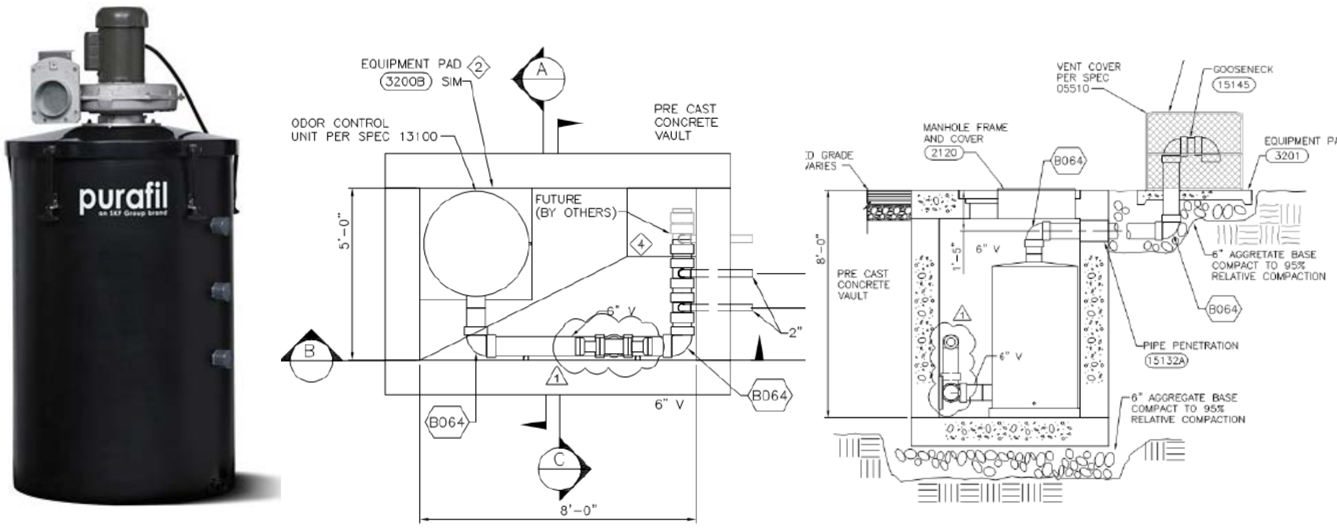


Figure 4-10: Sample ARV and Subgrade Odor Control Valve Vault

Forcemain Maintenance

Given the likely low velocity ranges that a dual forcemain would experience and the great length of the alignment alternatives, it highly recommended that the City commit to a regular forcemain maintenance program.

A common forcemain maintenance activity is to clean the forcemain via a "pigging" device as displayed in Figure 4-11 below.

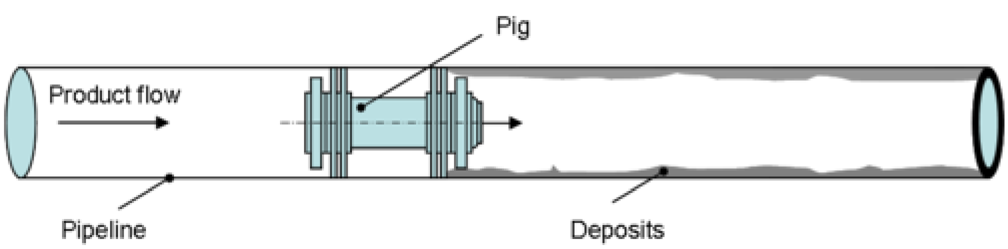


Figure 4-11: Forcemain "pigging"

The "pigging" tool is pushed down the pipeline via hydraulic pressure and typically conducted when the pipeline is offline in a controlled setting (in a dual forcemain application). The "pig" launching facility requires an extensive mechanical layout and is typically sized for each forcemain pipeline (i.e., a 12" and 16" forcemain requires two separate pig launchers, while a dual 12" forcemain requires one pig launcher).

In a single forcemain application, regular maintenance via pig launching facilities becomes critical to the long-term operation of the pipeline because of the lack of operational redundancy. Given the length of the pipeline and topography, it is highly recommended that one or two mid-way subgrade pig catching facilities be installed along the pipeline. A sample midway pig lauching facility (subgrade) is displayed in Figure 4-12 below.

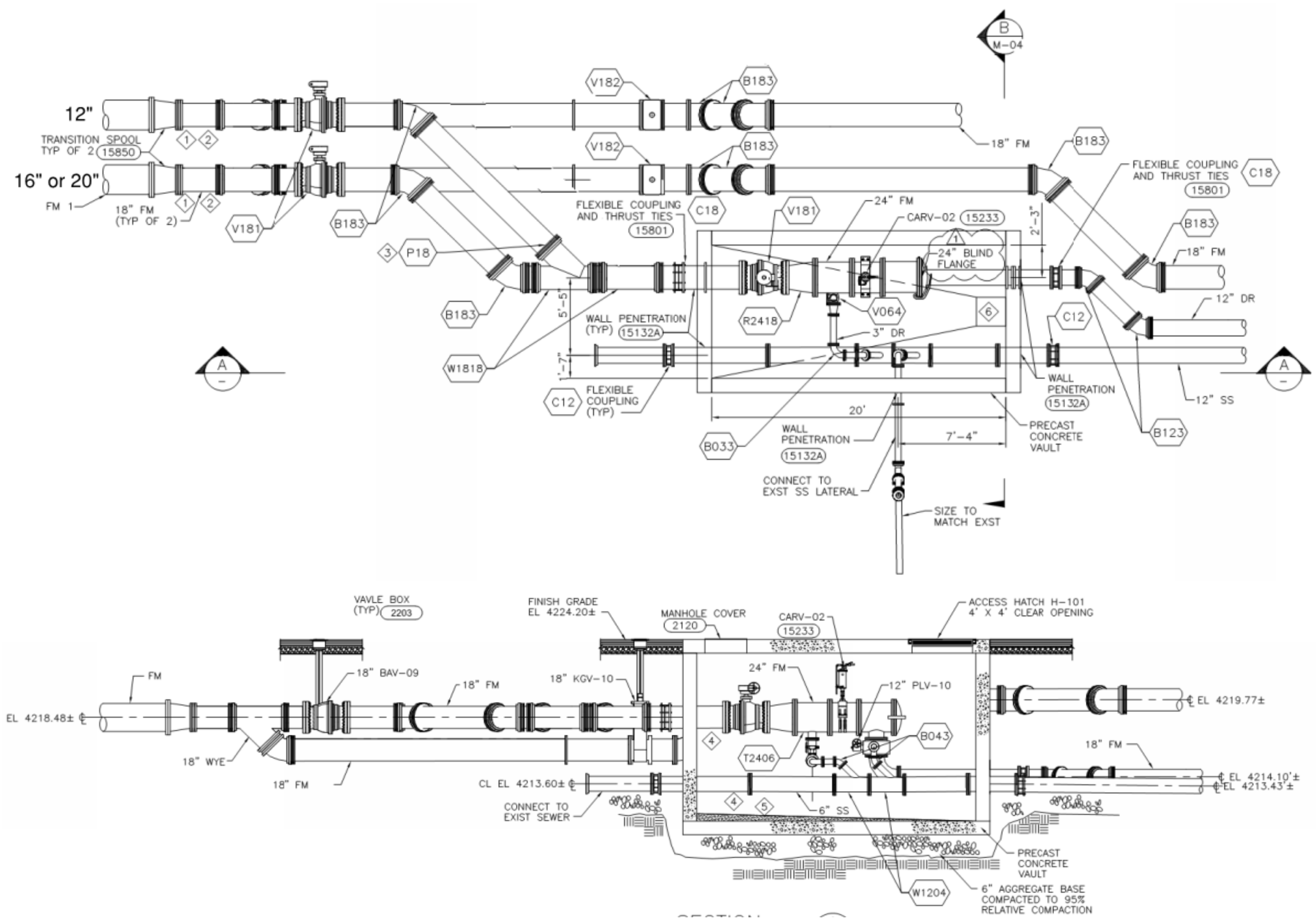


Figure 4-12: Midway Pig Launching Facility

4.3.5 Open Cut Construction Criteria

Depth

The typical pipeline minimum soil cover required to meet loading requirements for vehicular traffic is 4 ft of cover. For this study it was assumed that pipeline in areas with low parallel or transverse utilities density would be installed at minimum cover. Otherwise, it was assumed that the pipeline would be installed with 7 ft of cover (least 1 ft lower than a parallel water utility assumed to be 4 to 5 ft deep).

State and County Separation Requirements

In 2017 DDW released new guidance on separation of water mains and non-potable pipelines. The new guidance memorandum supersedes the Guidance Memo 2003-02, in which specific vertical and horizontal separation requirements (and material recommendations) were detailed for a new water main or new sewer pipeline are installed within 10 LF. The new guidance requires that any pipeline with separation under 10 LF must be approved directly by the regional DDW District 06 (D-06) via a standard application checklist process.

WaterWorks coordinated directly with DDW D-06 to discuss the required separation between the IPR line, brine line, sewer forcemains, and parallel utilities. Based on comments from DDW, several primary constraints were highlighted that will be incorporated into the project design:

- The IPR line is a tertiary treated indirect-potable reuse water source that will be pumped into a groundwater injection well; undergo a regulated minimum residence/travel time; undergo water quality testing; and then be pumped via the City's existing public water wells into the potable water system. Ideally, the IPR line should achieve 10 ft separation from sewer/storm drain utilities and potable water lines to avoid any need for a separation waiver from DDW.

- The brine line is a tertiary treated effluent that is not intended for indirect potable reuse and is effectively treated as a typical non-potable source. As such, the IPR line will need 10 ft separation from the brine line to avoid any need for a separation waiver from DDW.

- If avoidance is not feasible and 10-ft separation cannot be achieved, a specific DDW waiver will be applied for, whereby some vertical and horizontal clearance will be required along with mitigation measures to achieve "equal protection" such as:

    - Utilization of jointless pipe only (Fusible HDPE or Fusible C900 PVC)

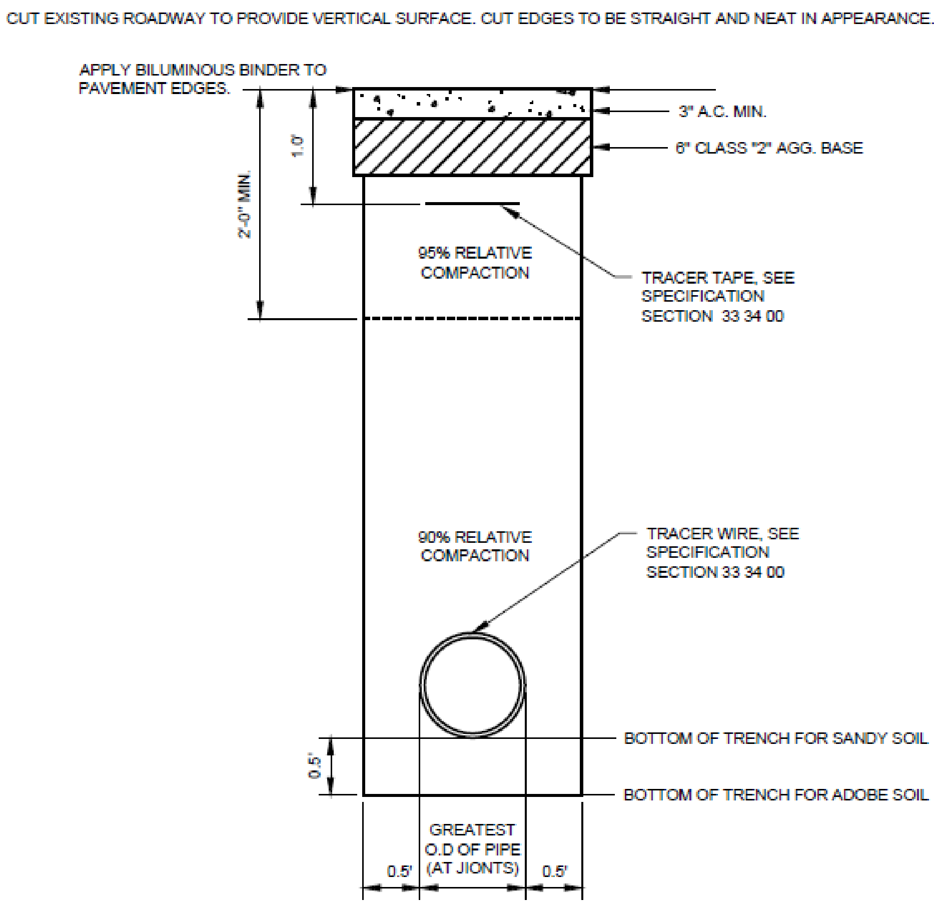
    - Utilization of CLSM bedding/backfill for the SSFM/brine pipes as opposed to using a typical aggregate media which is more porous and less durable

    - Separation of the IPR line which must be in a separate trench that is higher and offset from the brine/SSFM trench (some minimum horizontal and vertical separation)

Based on these constraints WaterWorks developed a proposed "compact" trench profile that would provide equal protection as the 10-ft minimum separation criteria and could be used to apply for a DDW waiver. This profile is presented in the section below.

Trench Dimensions and Backfill Considerations

The City standard trench detail W-6 shown in Figure 4-13 will be used as the basis for minimum trench dimensions and backfill considerations but will likely be adapted with specific geotechnical constraints and more specific pavement replacement details during the design phase.



NOTE:

1. ALL BACKFILL SHALL BE SELECT SAND OR SANDY LOAM AS APPROVED BY THE CITY ENGINEER.
2. PLACE 3" A.C. HOT-MIX PAVEMENT OR REPLACE WITH SAME DEPTH AS EXISTS, WHICH EVER IS GREATER.
3. HOT-MIX A.C. SHALL BE 1/2" MAX. AR 8000.
4. 85% RELATIVE COMPACTION PERMITTED IN NON-ROADWAY TRENCHES WHEN NO STRUCTURES ARE TO BE CONSTRUCTED OVER TRENCH.
5. PONDING, FLOODING, OR JETTING NOT PERMITTED UNLESS SPECIFICALLY ALLOWED BY CITY ENGINEER.
6. CONTRACTOR SHALL INSTALL AND MAINTAIN TEMPORARY PAVEMENT FOR 2 WEEKS PRIOR TO INSTALLATION OF PERMAMENT HOT-MIX PAVEMENT.

Figure 4-13: City Standard Trench Detail

Given the quantity of new parallel pipelines (potentially IPR, brine, 2x SSFMs), space constraints, and DDW constraints, WaterWorks developed the compact trench profile in Figure 4-14. Ideally the pipelines will have minimum 4’ of cover, but depths will likely be driven by the required vertical separation criteria, in which case 5’ to 6’ coer is more typical.

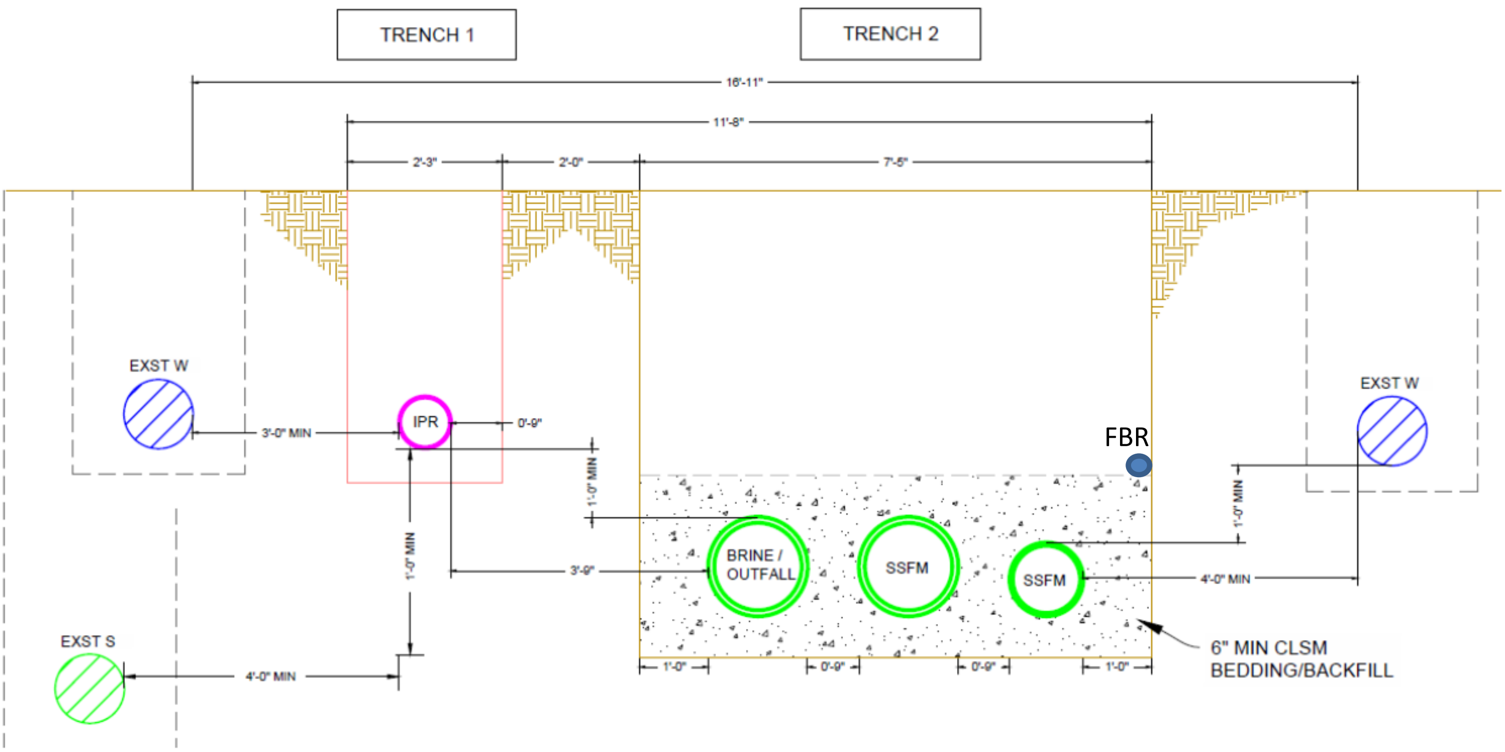


Figure 4-14: Proposed Compact Trench Section with DDW Waiver

Pavement Repair

For the purposes of preparing preliminary cost estimates, WaterWorks assumed that pavement would be repaired within anticipated trench limits and 2 feet beyond each edge. If a four-pipe trench section is utilized, there may be 16’ wide corridors of pavement repair which is equivalent to a typical lane. Conversely, a three-pipe trench section will lead to 12’ wide pavement repair. The City may choose to require certain roadways or sections be fully replaced (EP to EP), however this would add significant additional costs to the project and is not preferred. For the purposes of preliminary cost estimates it is assumed that the minimum pavement section replaced is 3”AC/8”AB, however this may vary by site (and jurisdiction) and will be confirmed with field work.

4.3.6 Utility Conflict Constraints

Utilities in Project Area

Extensive buried and overhead facilities are within the project area and are summarized below:

- Petroleum – Decommissioned petroleum oil lines that are slated for removal under the Embarcadero Multi-Use Bike Trail (next to beachfront).

- **Gas** – SoCal Gas operates and maintains local gas lines and a large 16" high pressure gas main that is present along Main Street and Quintana Road.

- Water – City of Morro Bay water mains, saltwater desalinization intake lines (and outfall) are located throughout the project area. Of particular significance is the 12" blended water line which runs down the center of the bike trail and parallel to Highway 1 (behind Dynegy/PGE property).

- Water -- SLO County operates a water line that cuts across Quintana, Highway 1, and runs east along Teresa Drive.

- Groundwater Well – City of Morro Bay operates numerous saltwater and regular groundwater wells in the project area along Embarcadero and in the Lila Keiser Park vicinity.

- Groundwater Well – Morro Bay Mutual Water Company (Dynegy) operates 2 wells near the Highway 1 bike trail.

- Raw Water – Morro Bay Mutual Water Company (Dynegy) operates a collection of 4" water mains in the vicinity of the bike trail that runs parallel to Highway 1.

- Raw Water – City of San Luis Obispo operates the Whale Rock water line which runs along the City and County border on the northeast side of the project area.

- Raw Water – Dynegy owns numerous intake and discharge lines to and from the existing and decommissioned power plant located on Embarcadero.

- Storm Drain – City of Morro Bay storm drain lines are located throughout the project area.

- Sewer – City of Morro Bay sewer lines are located throughout the project area.

- Sewer – City of Cayucos sewer lines are located throughout the project area and discharge into the common inlet location at the existing WWTP at Atascadero.

- Communications: Charter Communications operates extensive local buried and overhead communications cables (fiber, cable, phone) in the project area. These are typically located under sidewalks.

- Communications: AT&T operates extensive local buried and overhead communications cables (fiber, cable, phone) in the project area. These are typically located under sidewalks,

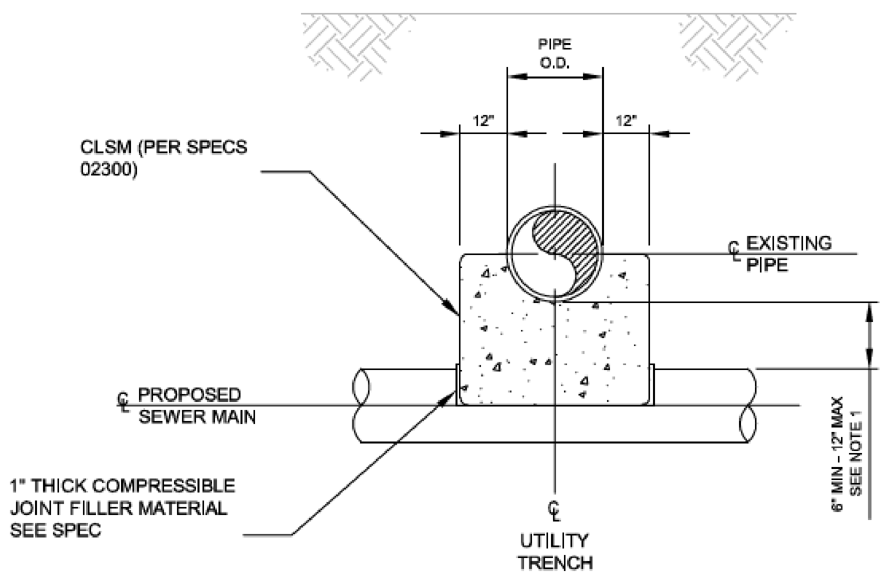
- Power: PG&E operates local buried and overhead power lines in the project area.

Potholing & Utility Marking

During the design phase extensive potholing, ground penetrating radar, and tracing will be conducted throughout the preferred alignment at key locations where it is necessary to know the horizontal placement and depth of the utility. In addition, any geotechnical boreholes will require 811 utility marking (Dig Alert or USA North), which will be utilized to further refine utility locations.

Utility Crossing Requirements

For the purposes of preliminary cost estimates, it is assumed that a close-proximity crossing (with saddle) and nominal transverse crossings of buried utilities constitute an additional cost. This is intended to appropriately define the level of effort required to construct in high-density utility areas. A sample proximity crossing is visualized in Figure 4-15.



NOTES

1. CLSM ENCASEMENT IS REQUIRED IF SEPARATION BETWEEN EXISTING UTILITY AND PROPOSED SEWER PIPELINE IS LESS THAN 12 INCHES.
2. CLSM ENCASEMENT IS ALSO REQUIRED IF SPECIFICALLY CALLED OUT OUT ON PLANS.

Figure 4-15: Close Proximity Saddle Crossing

Parallel Utilities in High Density Corridors

Utility research has highlighted several high utility density corridors whereby a very compact trench will be required to provide sufficient horizontal clearance to safely construct the proposed offsite pipelines. This compact trench detail was previously highlighted above in Figure 4-14.

4.3.7 Outfall Constraints

The existing ocean outfall and elevated air relief structure at the WWTP is not planned for demolition along with the rest of the plant. The air relief structure is approximately 8’ deep and is elevated 5’ above existing grade. The structure can be accessed via stairs and an aluminum walkway. Three options have been identified to connect with the existing ocean outfall, which are listed in Table 4-8 below and visualized in Figure 4-16.

Table 4-8: Outfall Connection Options

Option	Description
A	Make a buried connection to the existing outfall out within the Atascadero ROW and utilize a buried CARV with an elevated stand pipe above the 100yr floodplain (at least 7 ft high)
B	Construct a new elevated (at least 7 ft high) outfall/air relief structure within Atascadero ROW that utilizes energy dissipation, a flexible coupling, and flexible check valve. It is anticipated this structure would negatively impact the aesthetics of the area and might require some architectural treatment or should be fenced and screened for security/anti-vandalism purposes.
C	Connect to the existing air relief structure via an elevated structure or an at grade structure and stand pipe that utilizes energy dissipation, a flexible coupling, and flexible check valve. Screened fencing is recommended around the two structures to facilitate future demolition of the WWTP site. For the purposes of this preliminary cost estimate, it is assumed that this is the preferred option.

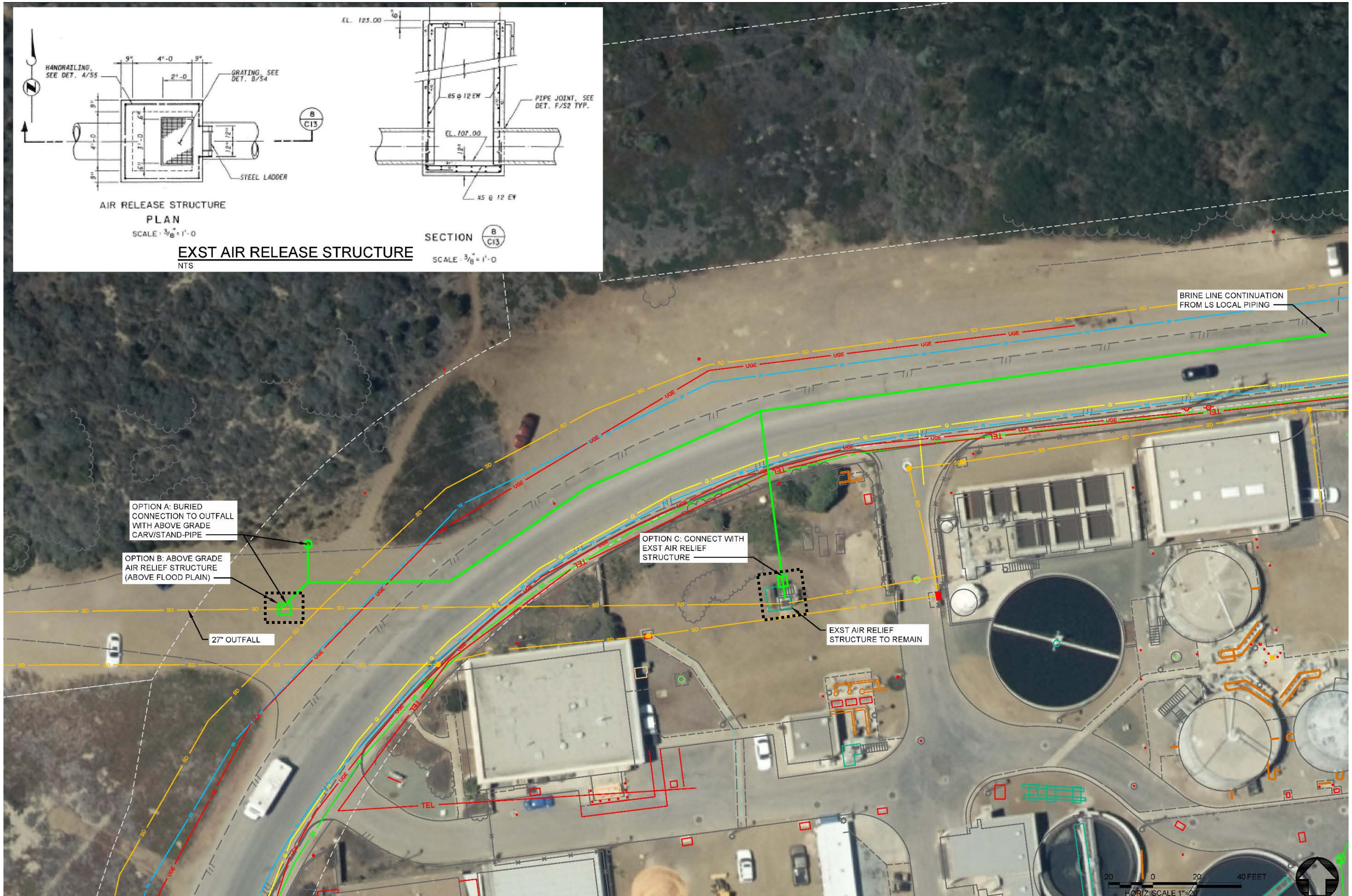


Figure 4-16: Outfall Connection Options

WATERWORKS ENGINEERS

4.3.8 Highway 1 and Highway 41 Crossing Constraints

Several alternative alignments cross Highway 1 (and Highway 41) and would be subject to Caltrans transverse and longitudinal encroachment requirements. A summary of these constraints is listed below:

- Pressurized pipeline requires a secondary casing for a transverse or longitudinal crossing
    - HDPE or welded steel is the standard preferred casing material for Caltrans. Fusible PVC would require additional coordination for approval.
- Open cut construction under the Atascadero Road and South Bay Blvd. undercrossings are permissible if construction is outside of the foundation zone of influence (assumed to be a 1:1 plane from existing grade of the feature)
- Structures (bridges) are not allowed within Caltrans ROW
- Tree removal should be avoided where at all possible due to additional coordination that would be required.
- Nightwork may be required for under the South Bay Blvd. undercrossing due to the constricted work area.
- Disturbance of sloped shoulders within Caltrans ROW requires replacement with grades slopes that are a maximum 4H:1V (25% grade).
- Any disturbance of existing Caltrans pavement will require full structural replacement (near trench) and additional overlay beyond the limits of excavation to the closest fog line or construction joint.
- Any disturbance to a sidewalk or ramp (ADA) will require replacement with an updated design that meets current standards and will require special submittal and full plan check from Caltrans. If possible, avoidance is preferred.
- Any disturbance to a railing (not anticipated at this time) will require replacement with an updated design that meets current standards and delivered via submittal and full plan check from Caltrans.

Transverse Crossing – Atascadero Undercrossing

A transverse crossing via standard open cut construction under the Atascadero HWY-1 undercrossing is feasible based on as-built research and utility records. An overview of the crossing is displayed along with imagery in Figure 4-17. It is understood that the intersection of HWY-41 and Main St are planned for modification when Caltrans and the City work together to install a roundabout.

Figure 4-17: Atascadero Road HWY-1 Undercrossing

A sample section view of the HWY-1 undercrossing at Atascadero Road is displayed in Figure 4-18.

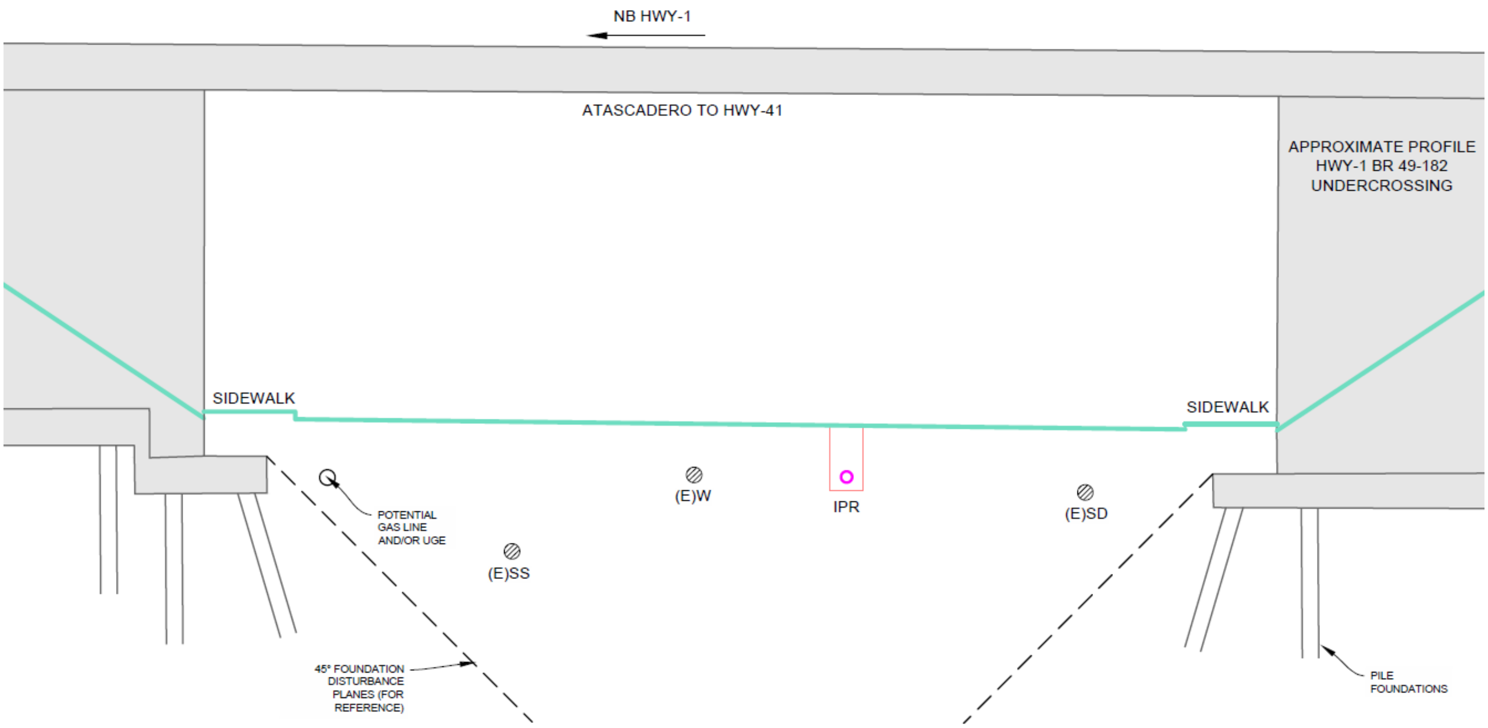


Figure 4-18: Approximate Section View of BR 49-182

Parallel Encroachment – Atascadero

Locating the forcemain/brine alignment in HWY-41 or utilizing the corridor for the East IPR alignment is feasible but will require a longitudinal encroachment exception and may be subject to shorter working hours to accommodate peak traffic in the morning. This alignment will likely only be utilized for the East IPR line if the east area injection wells are selected. A plan view of the alternative is displayed in Figure 4-19 below. Preliminary research shows that the State ROW extends north past the roadway and covers a parallel grassy/ornamental area that would be ideal for locating the East IPR alignment and would serve to minimize disruptions to the paved roadway.

Figure 4-19: HWY-41 Encroachment

Parallel Encroachment – SB HWY-1 Connector

The parallel encroachment of the southbound (SB) HWY-1 connector (west shoulder) has been identified as an ideal corridor for the West Alignment to avoid the cultural resources present in the Lila Keiser Park area and the public water wells. This is shown in Figure 4-20.

Figure 4-20: SB HWY-1 Connector Encroachment

Transverse Crossing – Quintana to Teresa

An alternative crossing of HWY-1 to get from Quintana Road to the WRF-side of the highway is via a trenchless crossing from Quintana Road to Teresa Drive, as shown in Figure 4-21. The primary constraints are the utility conflicts on the west end of Teresa Drive, whereby there is very limited space to locate a receiving pit. In addition, there is a large culvert that crosses Teresa Drive and runs southeast, which would likely drive the trenchless crossing deeper.

Figure 4-21: HWY-1 Crossing At Teresa Drive

Transverse Crossing – Southbay Blvd Undercrossing

An alternative location for a trenchless crossing of HWY-1 is at South Bay Blvd. An open-cut option is feasible within the pavement of the undercrossing. Due to the low overhead bridge superstructure, smaller equipment will need to be utilized to work within that space. In addition, there may be working hour constraints imposed by Caltrans to accommodate peak traffic in the mornings. An overview of this option is displayed in Figure 4-22.

Figure 4-22: South Bay Blvd Undercrossing

An approximate section view of the South Bay Blvd. undercrossing and the proposed location of the new pipelines is displayed in Figure 4-23 below. Note that there is only 15 feet of headspace under the bridge which require the contractor to utilize smaller equipment

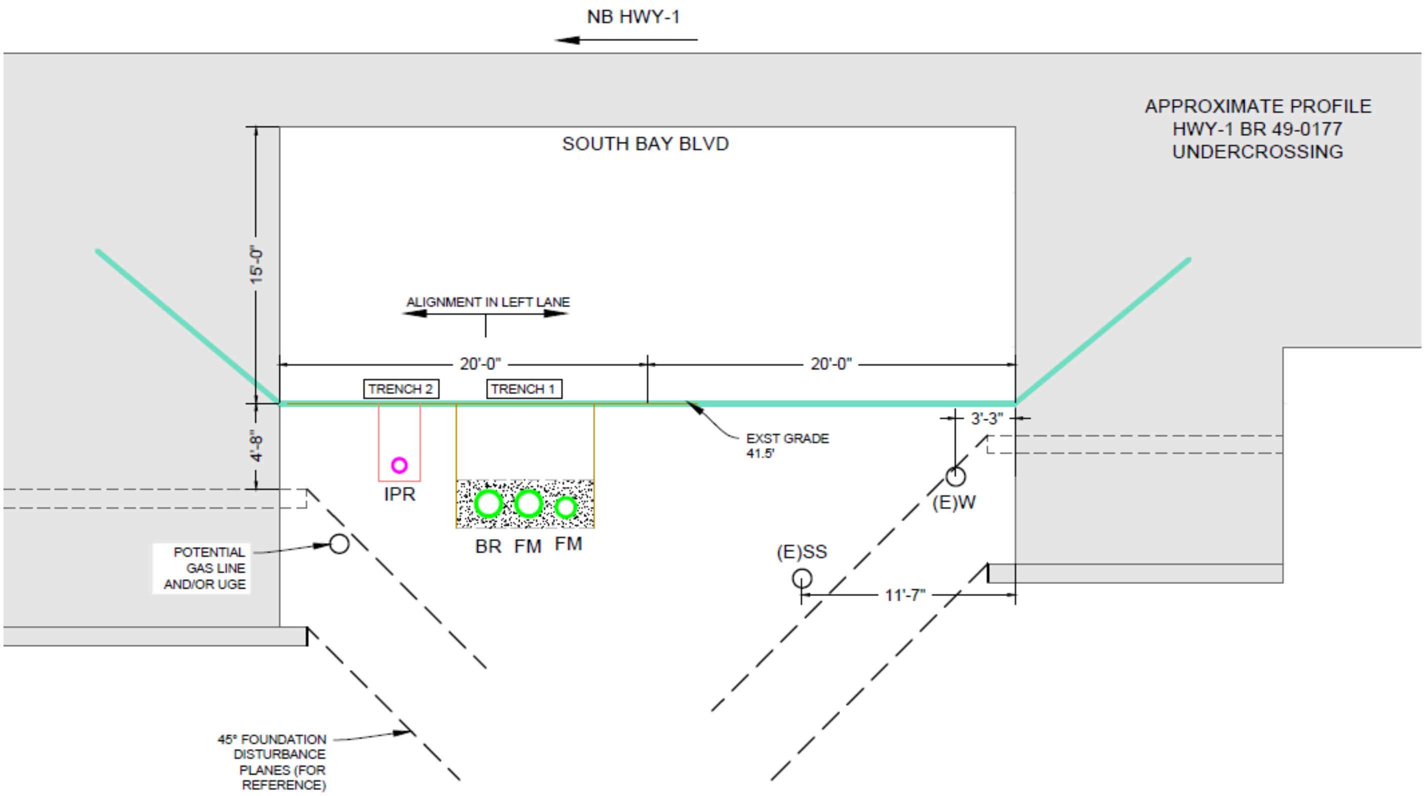


Figure 4-23: Approximate Section View of BR 49-177

4.3.9 Morro Creek Crossing

WaterWorks identified and analyzed four methods of crossing Morro Creek for the West (80-100 feet) and Embarcadero (130-150 feet) alignment alternatives. These four options are displayed in Table 4-9 below.

Table 4-9: Morro Creek Crossing Options

Option	Morro Creek Crossing	Analysis Summary
A	New Pipe Bridge	The new pipe bridge is the most feasible Morro Creek Crossing option; incorporating pedestrian and/or vehicular access would likely increase cost. A utility bridge on the Embarcadero Alignment would have to be offset towards the east due to existing utilities, in which case private easement would be required. Due to the subsurface conditions near the coast line extensive
B	Trenchless Crossing	Feasible but not preferred due to cost considerations
C	Open-Cut Construction	Infeasible given seasonal and environmental permitting constraints
D	Modify/Reuse Pedestrian Bridge	Likely infeasible given the number of pipelines (up to 4) that would need to be supported

Renovate Existing Bridge

The existing pedestrian bridge at Lila Keiser Park and the existing pedestrian bridge on Embarcadero (vehicular emergency option) are likely not able to be adapted to support 2 to 4 pipelines without removing

New Pipe Bridge

An exposed pipe crossing of Morro Creek via a pipe bridge could be constructed in several different styles are listed and discussed in Table 4-10 below. A sample section view of some typical pedestrian/vehicular bridges are shown in Figure 4-24. Note that a security feature (fencing + screening) may be necessary to add to discourage vandalism and pedestrian access.

Table 4-10: Pipe Bridge Options

<table>
<tr><th>Type of Bridge</th><th>Above 100yr level (sea rise adjusted)</th><th>Pile Foundation</th><th>Height (above existing grade)*</th><th>Width*</th><th>Cost</th></tr>
<tr><td>Base Pipe Bridge</td><td rowspan="3">Yes</td><td rowspan="3">Required @ Embarcadero, not required at Lila Keiser Park</td><td>7 ft</td><td>~5-8-ft</td><td>Lowest</td></tr>
<tr><td>+ Pedestrian Access</td><td>5 ft</td><td>~8-9-ft</td><td>High</td></tr>
<tr><td>+ Vehicular Access</td><td>6 ft</td><td>~13 ft</td><td>Highest</td></tr>
</table>

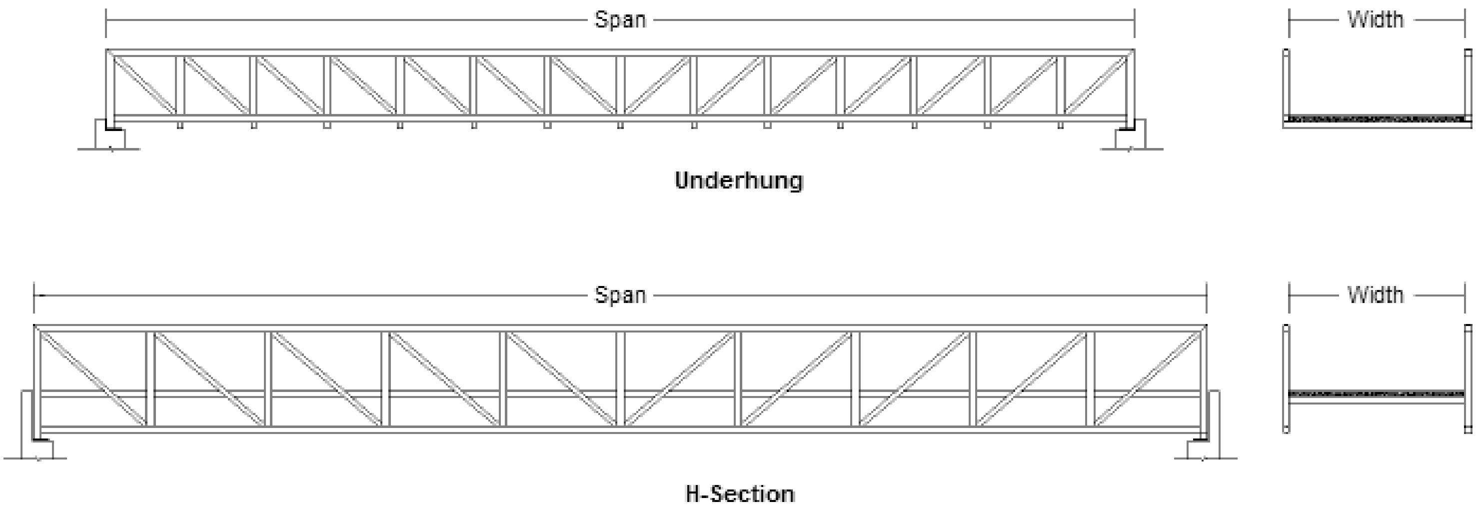


Figure 4-24: Common Pipe Bridge Options

At one or both ends of each pipeline as it breaks grade and crosses the bridge, a flexible expansion joint will be added to provide seismic redundancy and allow one or both ends of the pipeline to deflect in the axial direction, angular, and translational (longitudinal) movement. The expansion joint can be placed flat in a large precast vault, or can be placed in an angled position (exposed). One sample flexible expansion joint is displayed in Figure 4-25 below (EBAA Force Balanced FlexTend).

Figure 4-25: Sample Flexible Expansion Joint

<u>Double Containment, Spill Containment & Leak Monitoring</u>

As discussed previously, an exposed pipeline crossing of the pipe bridge will utilize a single or dual flexible expansion joint (one or both ends of the bridge) to improve the seismic performance of the pipeline. In addition, the pipeline will be vertically placed above the sea-level-rise-adjusted 100-yr floodplain. Additional spill protection or containment measures are discussed below:

- Trough Spill Containment: A lightweight U-shaped trough that envelopes each suspended pipeline represents the least costly containment measure available. However, this would only provide protection against small leaks at segmental joints and not a full pipe rupture. The trough could be made to drain into the flexible expansion joint vault, where a level sensor would alert the City to any flow into the vault. The practicality and feasibility of the trough is significantly reduced, however, if it is not covered and insulated from overhead precipitation. Due to these impracticalities, trough containment is not recommended for this application.

- Individual double containment (via casing): The carrier pipe would be inserted into a casing (with spacers) and supported on top or slung under the bridge. This represents the most comprehensive and costly containment option due to the added casing cost, dead load on the bridge, and increased pipe clearance. A moisture sensor would be placed at the end of the casing which would register any spills in the casing. It is likely that the bridge will be wide enough for a vehicle. If the pipelines are suspended under the deck, the bridge could be used for pedestrians and vehicles, and would be more aesthetically pleasing. This is displayed in Figure 4-26 through below.

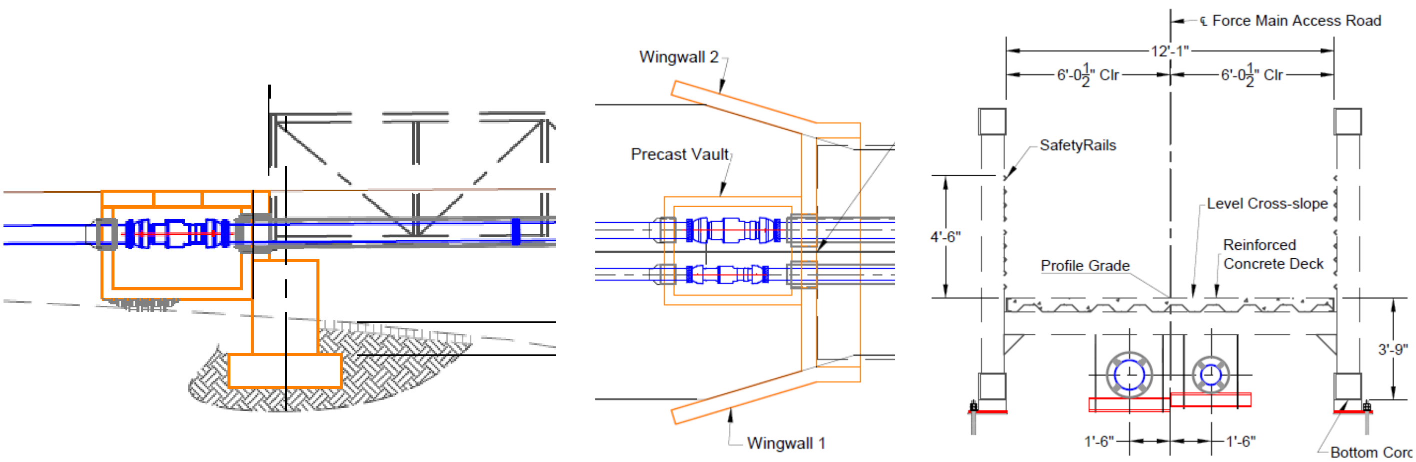


Figure 4-26: Sample Pedestrian Bridge with Individual Casings (3rd pipe not shown)

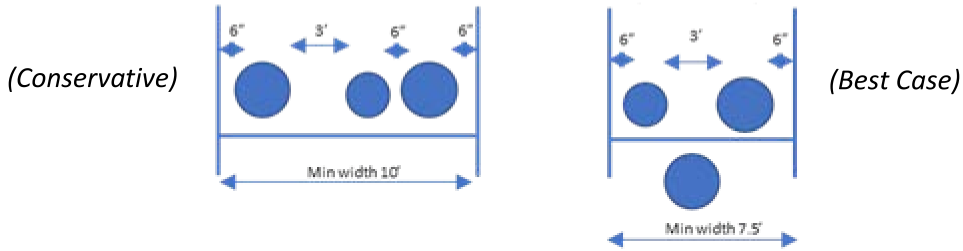


Figure 4-27: Sample Utility Bridge Configurations with Individual Casings

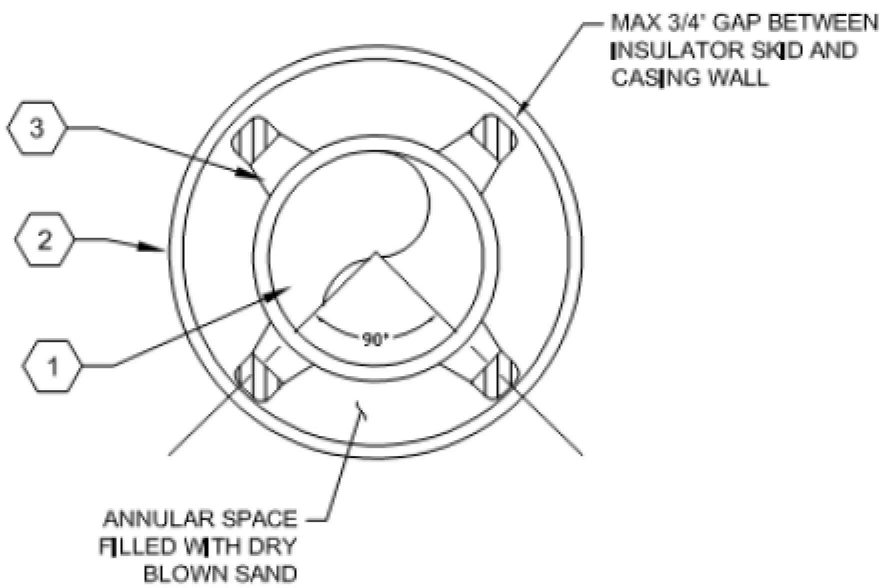


Figure 4-28: Sample carrier pipe in casing (dry blown sand in annulus is optional)

4.3.10 Bike Path and Drainage Crossing

The 20'W water and bike path easement that is parallel to HWY-1 will be utilized as a corridor for the West Alignment. Existing as-built records highlight that 8' wide 2.5" AC bike path runs along the centerline of this the easement. A typical section is visualized in Figure 4-29 below. A 12" PVC blended water line runs in this bike path corridor and is horizontally located in varying locations relative to the centerline of the bike path. Fencing runs along both sides of the bike path (at its tightest location, there is 20' inside clearance). Many ornamental trees line the fencing along with a parallel overhead wire and power poles. A portion of the bike path has a parallel 4" raw water line that is operated intermittently by MBMWC.

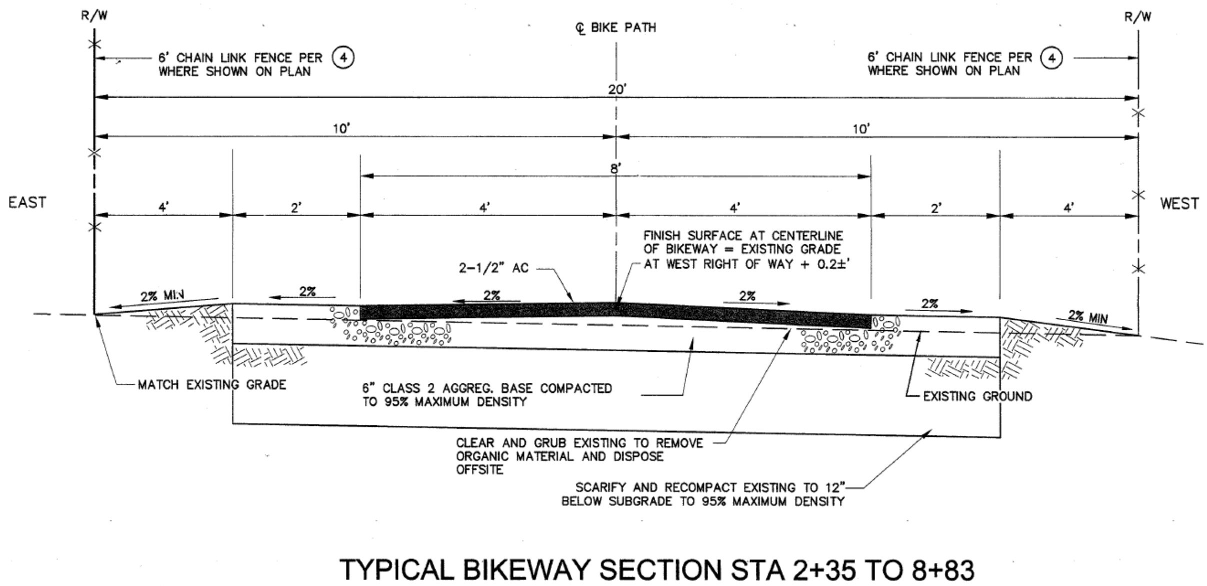


Figure 4-29: Typical Bike Path Section

Near the end of the bike path the right-of-way transitions from PGE/Dynegy to State ROW. At that transition location there is an existing drainage box culvert (see Figure 4-30 below). This drainage culvert crossing presents a significant constraint due to the number of proposed pipelines in that corridor.

Figure 4-30: Culvert/Drainage Crossing on Bike Path

There are several options to cross the culvert which are listed in Table 4-11 below.

Table 4-11: Culvert/Drainage Crossing Options

Option	Quintana-Morro Bay Roundabout Crossing	Analysis Summary
A	Open-Cut Construction – Replace-in-in place culvert	Would likely trigger 401/404/1602 Environmental Permits. Highly sensitive to construction date. Permit requirements would improve considerably if construction were completed during dry summer conditions. Because the culvert is partially located in State ROW, there is the potential for additional requirements from Caltrans (drainage study, etc.). This option presents some risk to the overall project schedule. This option is preferred over a trenchless crossing due to cost considerations. The existing 12" blended water line would likely need to be relocated.
B	Trenchless Crossing	Feasible, but not preferred due to cost considerations. Will be explored more during the design phase as an alternative to open cut.

4.3.11 Quintana Roundabout Crossing

The Quintana-Morro Bay roundabout on the West Alignment along the Quintana corridor overlays an area with a very high density of parallel and crossing utilities.

Due to the anticipated number of utility conflicts, WaterWorks considers open cut construction to be infeasible for all the pipelines because utilities would need to be rerouted at great expense. In addition, open cut construction would highly impact traffic conditions in a high-volume corridor that is an important gateway to Morro Bay from Hwy-1. The alternative crossings are listed in

Table 4-12: Quintana Roundabout Crossing Options

Option	Quintana-Morro Bay Roundabout Crossing	Analysis Summary
A	Trenchless Crossing	See Trenchless Design Constraints and Feasibility Section
B	Open-Cut Construction	Considered infeasible due to numerous utility conflicts

4.3.12 Trenchless Design Constraints and Feasibility

A range of trenchless construction methodologies was assessed for the various locations where trenchless construction was feasible and competitive to other alternatives. Please note that this trenchless analysis is limited to the final alignment alternatives (West and Embarcadero) which are the outcome of the preliminary alternative alignment assessment which is detailed in Section 4.4.2. The following locations are listed as suitable for trenchless construction:

1. Morro Creek Crossing @ Lila Keiser Park (West alignment)
2. Morro Creek Crossing @ Embarcadero (Embarcadero alignment)
3. Roundabout @ Morro Bay and Quintana (West alignment)
4. Transverse HWY-1 Crossing @ Quintana to Teresa (West or Embarcadero alignments)

Note that there is also a potential trenchless crossing required of a culverted drainage feature that crosses the West alignment bike path. The trenchless assessment for this crossing may be required during the design phase, but at this point, an open cut crossing is preferable.

Other locations that were evaluated for trenchless construction but were deemed unsuitable and best served by open cut-construction were the Caltrans longitudinal encroachments under HWY-1 undercrossings at Atascadero (8” East IPR) and South Bay Blvd (4 pipelines), Bike Path to main St (4 pipelines) and parallel to the SB HWY-1 connector at Atascadero (4 pipelines). There is sufficient horizontal clearance for open cut construction relative to parallel utilities and Caltrans structures. In addition, a trenchless operation would likely impact traffic more than open cut construction due to the extended duration (3-4 months) and top-side equipment surface footprint that operation would entail.

Possible Trenchless Construction Methodologies

Auger Bore and Jack (ABJ)

A successful ABJ operation depends on firm ground conditions where the tunnel heading is self-supporting and is not appropriate below groundwater. Under flowing ground conditions, face loss will be excessive resulting in large scale surface settlements impacting overlying utilities. Providing sufficient dewatering to control groundwater may not be practical if extraordinary pumping volumes are required, which over an extended period, could also cause subsidence issues in fine grained deposits. In coarse grained sediments with cobbles and possible bounders, maintaining line and grade is difficult. A summary of the process is listed below:

- Open ended, free face (groundwater control required @ invert)
- Typical for 4” to 96” casings
- Jacking/Receiving shafts; min inside dimensions 12’Wx38’L / 12’SQ or 12’DIA
- Straight line segments, typical installation lengths 200 to 500 feet
- Additional 3000-4000 SF working space required (top side equipment)
- Risk Factors: groundwater, unstable soils, contaminated soils, cobbles and boulders, obstructions, systemic settlement

Figure 4-31: Auger Bore and Jack

Pilot Tube Guided Boring (PTGB)

A successful PTGB operation is highly dependent on the pilot tubes being able to push through "displaceable" soils. Coarse-grained sediments may not be displaceable. A summary of the operation is listed below:

- Three step process; 1) Steerable pilot tube, 2) auger drill casings, 3) jack product pipe
- Typical 4" to 48" casings
- Open ended, free face (groundwater control required < 10' above invert)
- Jacking/Receiving shafts; min inside dimensions 12'DIA / 8'DIA
- Straight line segments, typical installation lengths < 300 ft
- Additional 3000-4000 SF working space required (top side equipment)
- Risk Factors: groundwater, unstable soils, contaminated soils, cobbles and boulders, obstructions, systemic settlement

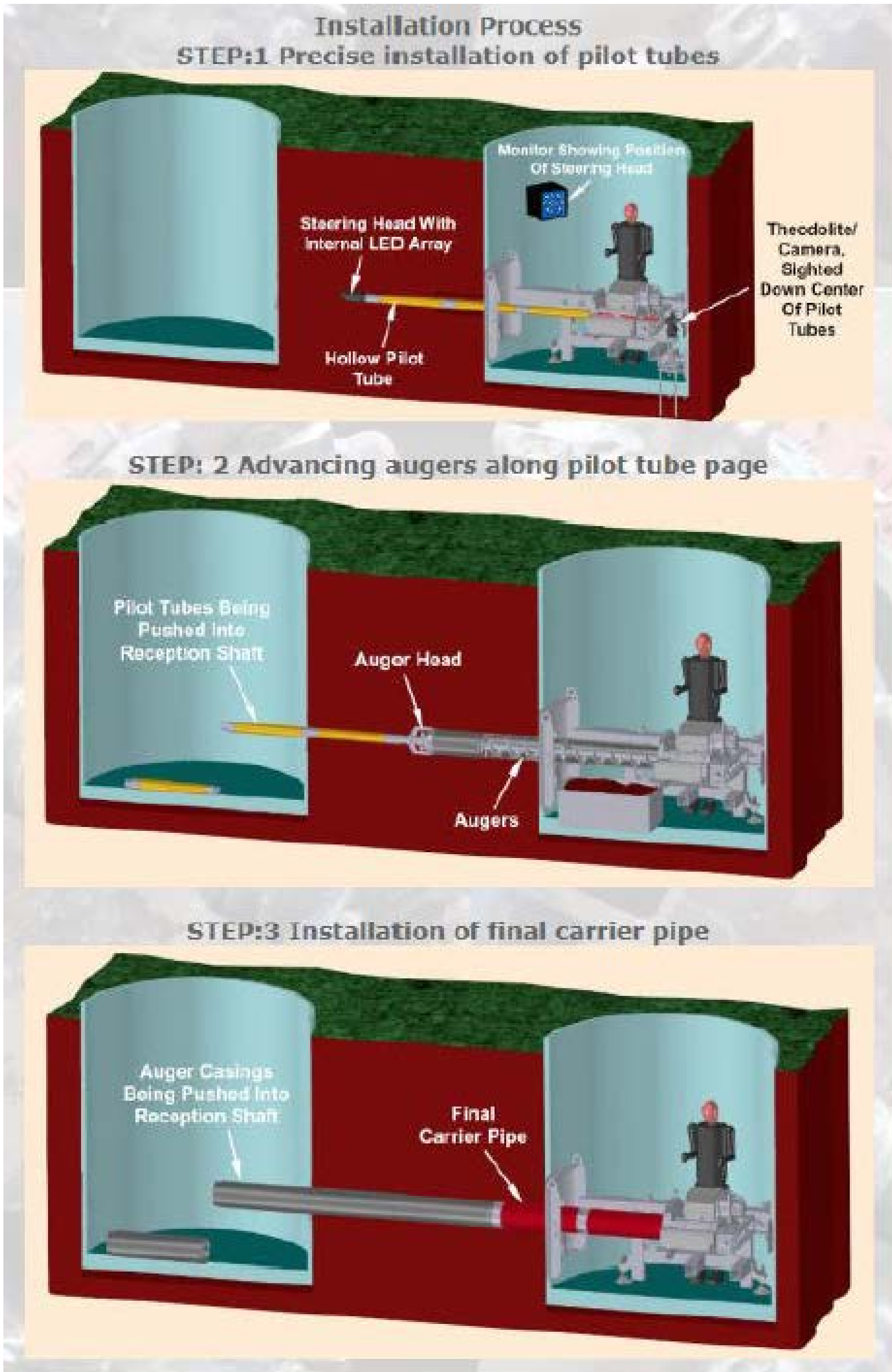


Figure 4-32: Pilot Tube Guided Bore and Jack

Front Steer Guided Boring (FSGB)

A successful FSGB operation is highly dependent on the strength of the soil to provide sufficient bearing capacity to “push” against and steer. A summary of the operation is listed below:

- Similar to PTGB but without pilot tube, uses steerable cutting head instead
- Typical 12” to 48” casings
- Three step process: 1) Steerable pilot tube, 2) auger drill casings, 3) jack product pipe
- Open ended, free face (groundwater control required < 10’ above invert)
- Jacking/Receiving shafts; min inside dimensions 12’DIA / 8’DIA
- Straight line segments, typical installation lengths < 300 ft
- Additional 3000-4000 SF working space required (top side equipment)
- Risk Factors: groundwater, unstable soils, contaminated soils, cobbles and boulders, obstructions, systemic settlement

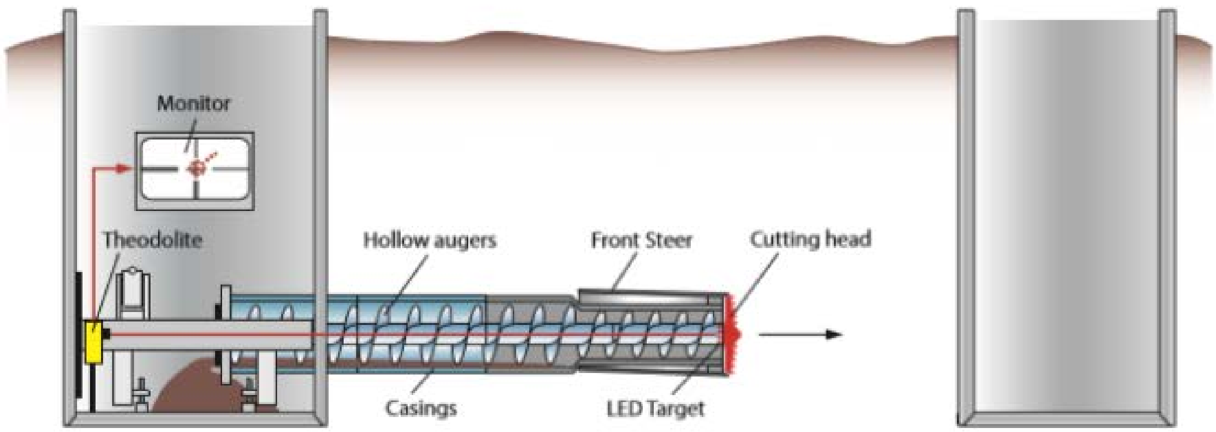


Figure 4-33: Front Steer Guided Boring

Pilot Tube Guided Auger Boring (PTGAB)

A successful PTGAB operation is highly dependent on the pilot tubes being able to push through “displaceable” soils. Coarse-grained sediments may not be displaceable. A summary of the operation is listed below:

- Similar to PTGB but without pilot tube, uses steerable cutting head instead
- Typical 12” to 48” casings
- Three step process: 1) Steerable pilot tube, 2) auger drill casings, 3) jack product pipe
- Open ended, free face (groundwater control required < 10’ above invert)
- Jacking/Receiving shafts; min inside dimensions 12’DIA / 8’DIA
- Straight line segments, typical installation lengths < 300 ft
- Additional 3000-4000 SF working space required (top side equipment)

- Risk Factors: groundwater, unstable soils, contaminated soils, cobbles and boulders, obstructions, systemic settlement

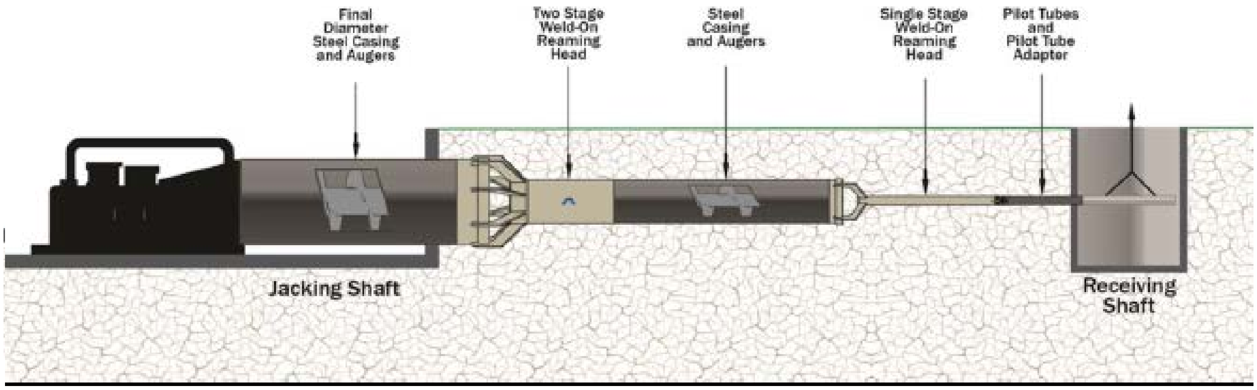


Figure 4-34: Pilot Tube Guided Auger Bore and Jack

<u>Horizontal Directional Drilling (HDD)</u>

- Typical 6" to 48" casing or carrier pipe
- 100 ft to 2000 ft (and greater depending on soil conditions)
- Surface launched (no shafts)
- Top side equipment working area 3000 SF
- Pull back side requires continuous length free to lay down product pipe (i.e., equal to length of HDD, on edge of road or sidewalk)
- Curved bore path (combination of vertical/horizontal possible)
- First a pilot tube and bit is steered by drill rod rotation. Successive enlargement of the pile bore hole is then accomplished via an incremental reaming passes (borehole diameter increased until 1.5x diameter of product pipe)
- Borehole kept stable with heavy drilling fluids ("frac" fluids)
- Not as accurate using bore and jack or Microtunneling trenchless methodologies. Some excavation may be required at the pull back end, after the HDD operation is complete, to properly line up pipelines if the alignment must be strictly adhered to in tight utility configurations or within easements constraints.
- Risk Factors:
  - Clean gravels and cobbles drain drilling fluids and borehole collapses
  - Cobbles and boulders
  - Loose ground that can't support steering corrections
  - Hydrofracture where drilling fluid exceeds soil overburden/shear strength

- Inadvertent drilling fluid returns (pressured drilling fluid breaks into parallel utilities, pile foundations, well casings, trench lines, etc.)
- Large systemic settlement due to significant overcut required; sensitive to product pipe diameter and depth

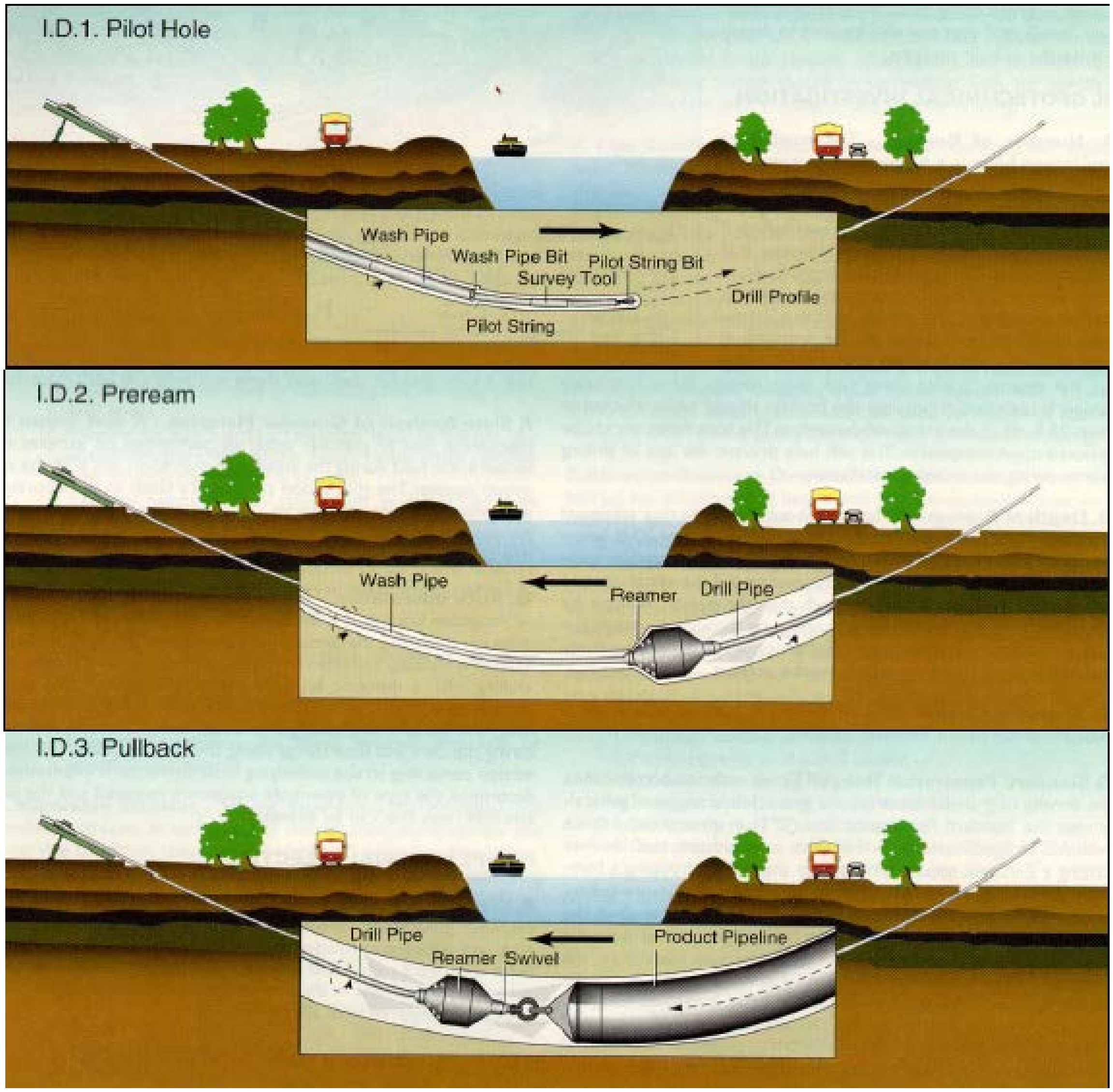


Figure 4-35: Horizontal Directional Drilling

Figure 4-36: HDD Rig

<u>Microtunneling (MT)</u>

- Uses microtunnel boring machine (MTBM)
- Typically 24” to 96”DIA casings
- Jacking/Receiving Shafts; min inside dimensions 12’Wx32’L/12’SQ or 12’DIA
- Remotely controlled; guided and steerable (laser if straight alignment and machine articulation)
- Uses pressurized continuous slurry in overcut space
- MTBM pushed forward via pipe segments and a jacking system
- Watertight continuous tunnel face support (can be operated in high groundwater conditions or under an active river)
- Large working area required for top side equipment (7000 SF)
- Typically straight segments, 400-500ft long and up to 1000ft for large diameter installations
- Very accurate
- Soil cuttings transported to jacking pit
- Risk Factors:
  - Cobbles and boulders
  - Very soft and loose ground; must support MTBM steering corrections
  - Obstructions
  - Wood (pilings, sunken logs, etc.)
  - Hydrofracture of slurry
  - Systemic settlement

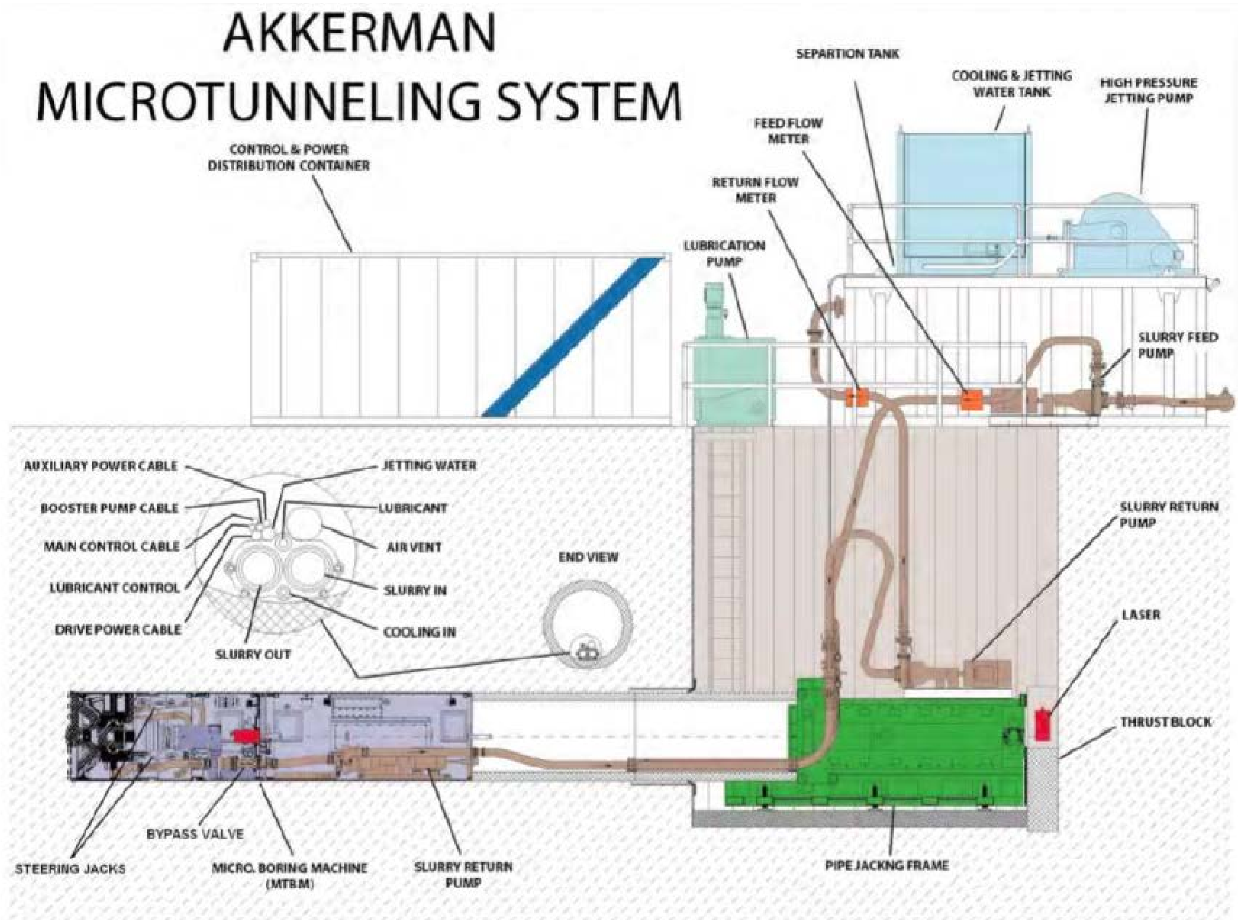


Figure 4-37: Typical Microtunneling Diagram

Figure 4-38: Microtunnel jacking pit preparation and MTBM positioning

Figure 4-39: MT equipment to process cuttings and provide slurry

Trenchless Crossing Constraints

Morro Creek @ Lila Keiser Park

- High groundwater due to proximity of creek
- Maintain sufficient vertical separation to minimize systemic settlement impacts to the parallel utility crossings (2x water lines) and pedestrian bridge (spread footing foundation)
- Maintain sufficient clearances to the parallel Caltrans HWY-1 bridges over Morro Creek due to pile foundations
- MBMWC Well #2 would need to be shut down to avoid contamination from pressurized drilling fluids

Morro Creek @ Embarcadero

- High groundwater due to proximity of creek and tidal influence

- Maintain sufficient clearances to parallel utility crossings (water and sewer lines) and pedestrian bridge (pile footings)

Roundabout Intersection Morro Bay and Quintana

- Intermediate depth groundwater levels (15’ to 30’ deep), flows northwest
- Potential groundwater contamination at gas stations. If plume is present, it would be affected by the groundwater flow direction.
- High density of utilities (24”SD, 2x36”SD, 12”W, 10”W, 16”HPG, 6”+3”G, 10”SS, fiber, cable, UGE). The deepest utility is the 12”W which is approximately 14’ deep. The typical maximum systemic settlement constraint is 1”, but this may be refined due to segmental gravity utilities (Sewer/Storm) which are more sensitive to settlement. In addition, the presence of the 16” HPG indicates that systemic settlement constraints may be more stringent.
- Surface improvements
- Minimal disruption to Quintana/traffic conditions is strongly preferred due to importance of commercial corridor

Highway 1 – Quintana to Teresa

- Caltrans systemic settlement constraints (< 0.5 inches)
- Deep storm drain from the end of Teresa across Hwy-1 would drive down the depth of the crossing.
- Limited space within Teresa for a receiving pit

Recommended Trenchless Crossings

Morro Creek @ Lila Keiser Park

Microtunnel installation of a 48” to 60” steel casing (as a function of pipe spacer design) for the likely preferred alternative which is the 16”SSFM, 16”Brine, and 12” SSFM. The 8” IPR could potentially be installed within the same pipe if utilizes a casing, but DDW may not permit this due to the sewer/water horizontal clearance constraints, even with the additional casing mitigation. For the purposes of providing a preliminary cost estimate, it is assumed that a single microtunnel would house all four pipelines, but the 8” IPR could potentially be slung from the existing pedestrian bridge. The existing MBMWC Well #2 would need to be shut down due to the potential drilling fluid impact from the MBTM

Morro Creek @ Embarcadero

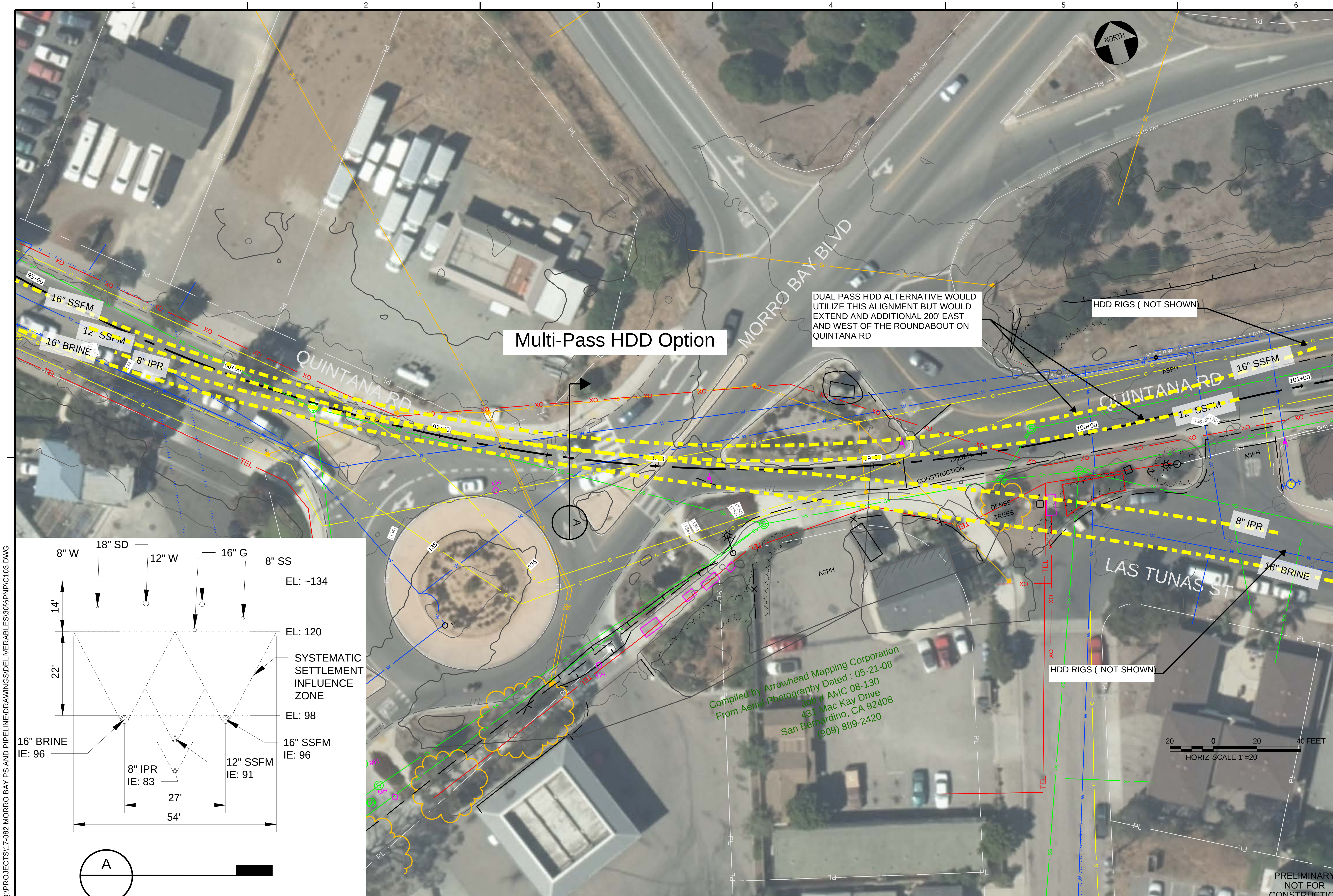
Microtunnel installation of a 48” to 60” steel casing (as a function of pipe spacer design) for the likely preferred alternative which is the 16”SSFM, 16”Brine, and 12” SSFM. The 8” IPR could potentially be installed within the same pipe if it utilizes an (additional) casing, but DDW may not permit this due to the sewer/water horizontal clearance constraints, even with the additional casing mitigation. For the purposes of providing a preliminary cost estimate, it is assumed that a single microtunnel would house all four pipelines, but the 8” IPR could potentially be slung from the existing pedestrian bridge. The microtunnel would need to be sufficiently offset from the existing pedestrian/vehicular bridge to not impact the pile foundation.

Roundabout Intersection Morro Bay and Quintana

The Morro Bay / Quintana roundabout crossing faces significant constraints due to the tight ROW corridor, directional change through the roundabout, surface improvements, mid to shallow groundwater, potential groundwater contamination, and high-density utilities which includes a critical high-pressure gas pipeline. WaterWorks identified two HDD alternatives (single pass or multi-pass) and three pipeline jacking alignments across the roundabout. These are summarized in the following Table 4-13 and the following figures.

Table 4-13: Roundabout Trenchless Alternative Alignments

Alternative Alignment	Description	Surface Impacts
Multi Pass HDD	Requires 4x entries (2x on Quintana, 2x on Las Tunas).	Extended traffic impact to Quintana Rd and Las Tunas due to multiple HDD rigs required. Pull/End side of HDD would require significant excavation to locate and cut individual carrier pipes. May affect Las Tunas residences (sensitive noise receptor)
Dual Pass HDD	Requires 2x entries (on Quintana). One for 8” IPR, one for remaining pipes.	Extensive impact to entry and exit locations on Quintana Rd due to anticipated systemic settlement
Pipe Jacking Option A	20’Wx40’L jacking pit in Las Tunas, pipe jack single casing 300’ to 20’DIA receiving pit on 899 Quintana Rd (UHAUL property).	Requires add’l easement from UHAUL property. May affect Las Tunas residences (sensitive noise receptor). Reduced traffic impacts to Quintana Rd.
Pipe Jacking Option B	ROW required. 20’Wx40’L jacking pit on 899 Quintana Rd (UHAUL property) pipe jack single casing 200’ to a 20’DIA receiving pit at 948 Morro Bay (Morro Bay Coffee)	Requires add’l easement from UHAUL property and Morro Bay Coffee property. Reduced traffic impacts to Quintana Rd.
Pipe Jacking Option C	2x pipe jacking operations; 2x 20’Wx40’L jacking pits on either end of the roundabout on Quintana Rd, pipe jack 230’ to 20’DIA receiving pit within roundabout.	No additional easement required. Extended traffic impact to Quintana Road and would require traffic control measures and lane merging within roundabout (due to receiving pit)



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DESIGN T. LEWIS
DRAWN T. LEWIS
CHECKED M. FISHER
APPROVED M. FISHER

WATERWORKS ENGINEERS

CITY OF MORRO BAY
WRF OFFSITE PIPELINES PROJECT

30%PNP

C-23

DATE

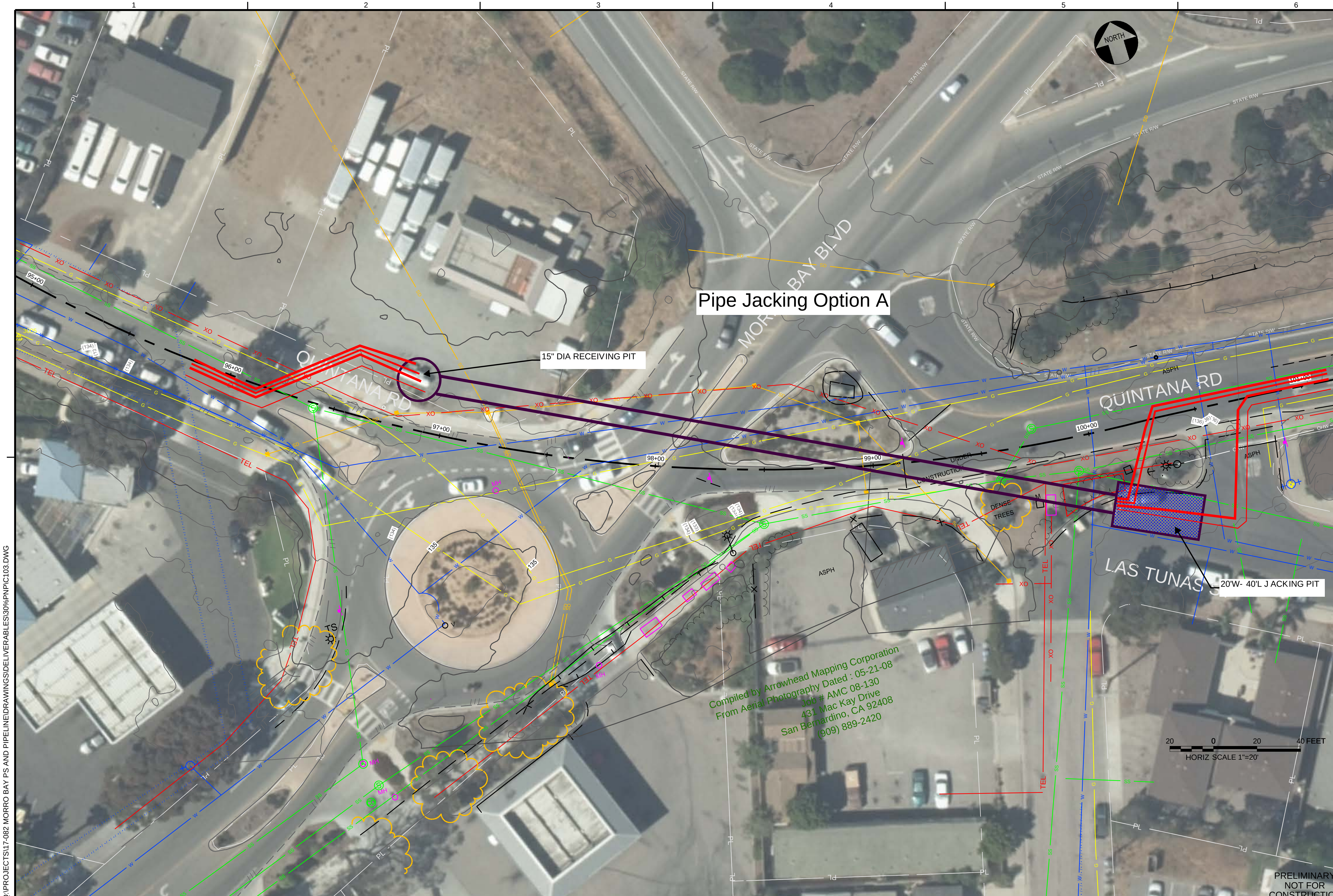
PROJECT NUMBER

DRAWING NUMBER
ROUNDABOUT

SHEET NUMBER 23

PRELIMINARY NOT FOR CONSTRUCTION

JOB NUMBER: 00-000 FILENAME: C103.DWG PLOT DATE: 11/13/18 PLOT TIME: 09:13:08



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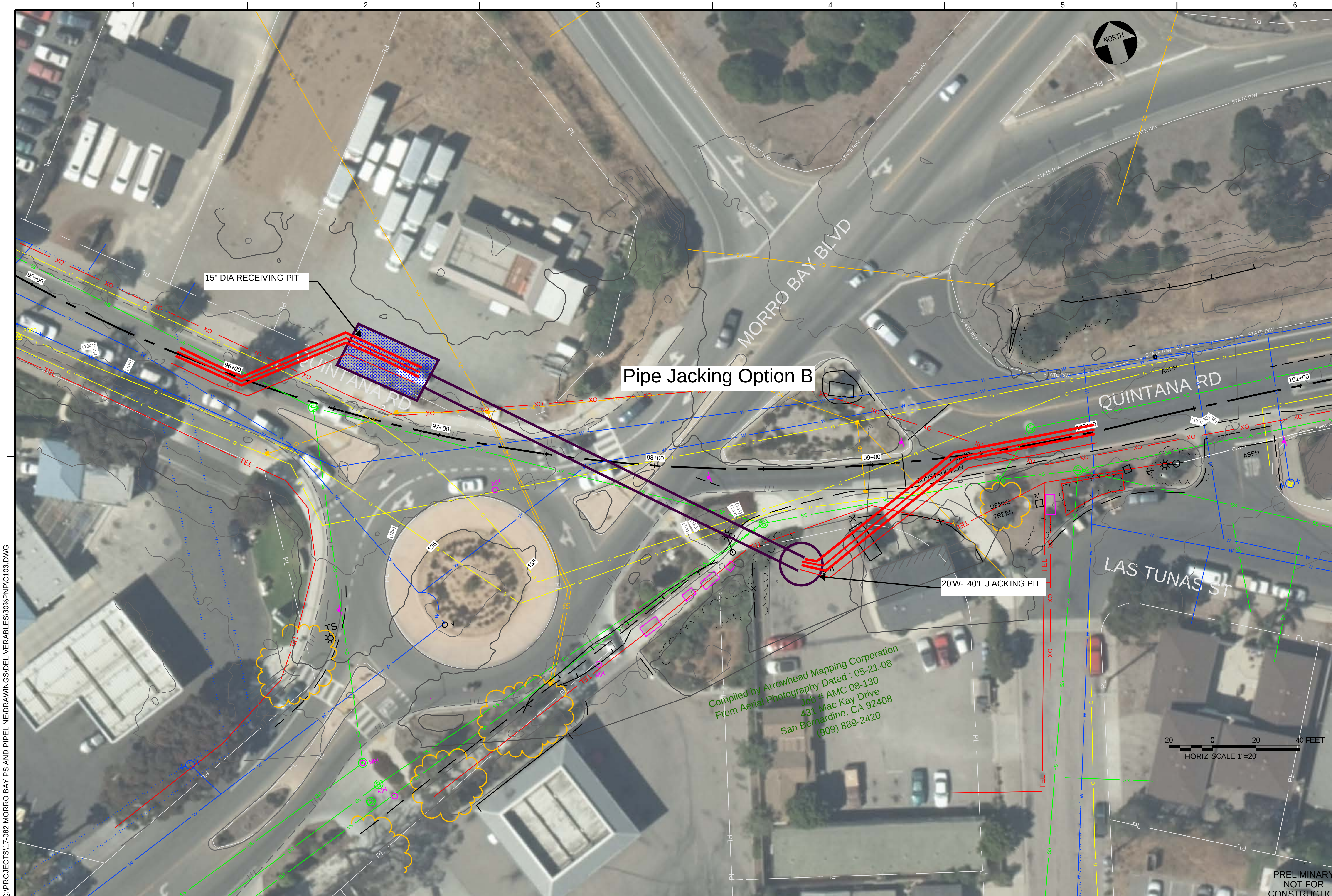

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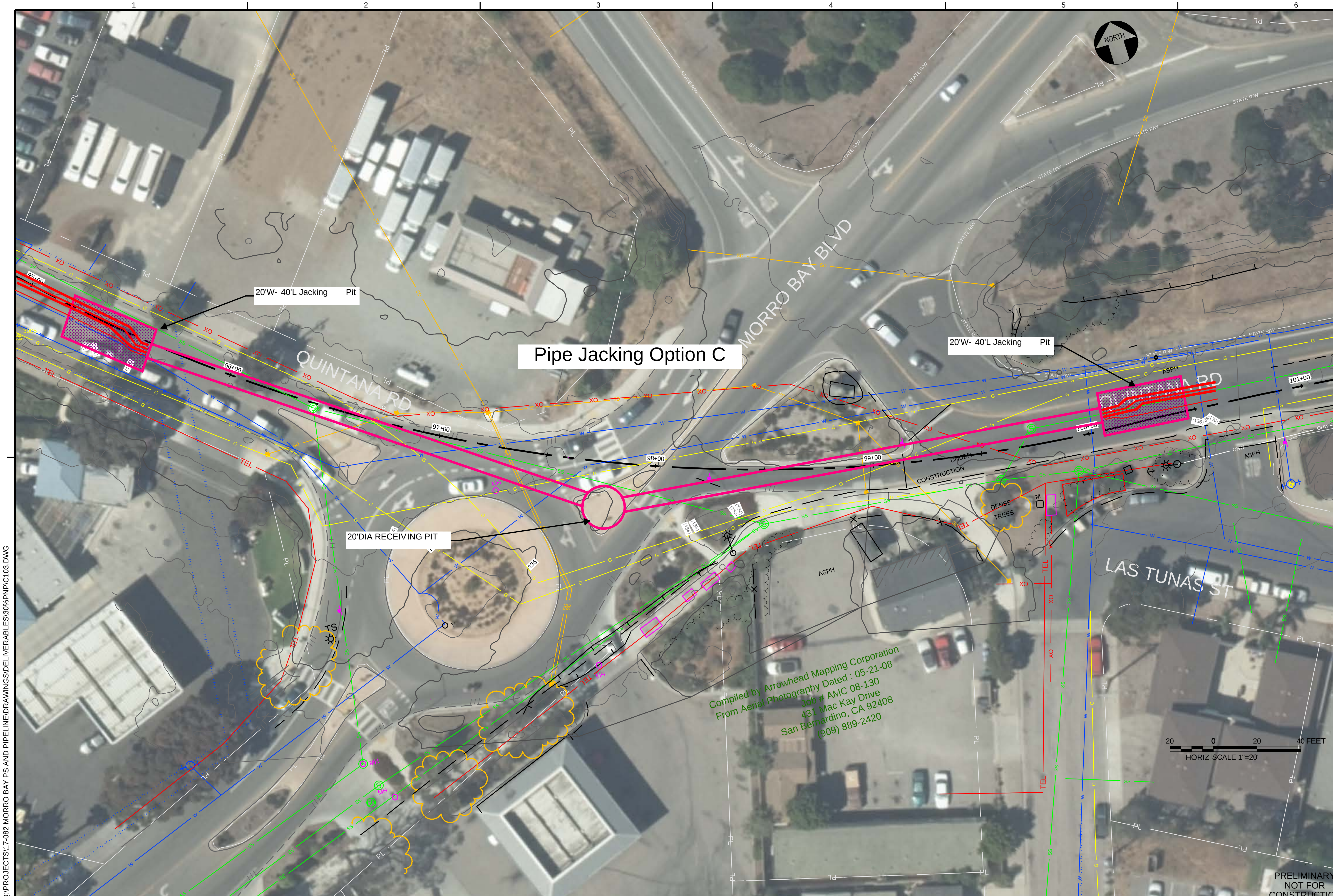

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Based on these alternative alignments, key trenchless constraints are summarized and listed in Table 4-14 below.

Table 4-14: Feasibility Comparison of Trenchless Alternatives Under Various Constraints for the Quintana/MB Roundabout

Constraint	Dual Pass HDD	Multi Pass HDD (offset)	Pipe Jacking Alternatives				
			MT	ABJ*	PTGB*	FSGB*	PTGAB*
Relative Cost	High	Medium	Highest	Lowest	Low	Low	Low
Dewatering effort	None	None	Low	High	Medium	Medium	Medium
Groundwater/Soil treatment	None	None	May be required				
Casing Size	Bundled Carriers	Individual Carrier	48” to 60” Typical		48” to 60” (not Typical)		
Systemic Settlement Risk	High	Medium	Low				
Depth (driven by systemic settlement tolerances)	130’	40’-60’	25’: 5’-10’ below deepest utility at minimum				
Length	>1000’	500’	300’-500’				
Extended traffic impact on Quintana	Medium	High	Low				
Working Area	Low	Low	High	Medium			
Season dependent	No	No	No	Summer preferred			

**Feasibility unknown pending additional geotechnical boreholes and groundwater level and WQ analysis*

A discussion of the relative comparative analysis between the HDD and Pipe Jacking Alternatives, and between the Trenchless Methodology Alternatives is listed below:

- Dual Pass HDD: The large overcut required for the bundled carrier pipelines would require a significant borehole diameter of 52” and 15” (IPR), which would need to be installed at 130’ deep to meet maximum systemic settlement criteria (<1”) at the nearest utility. Because of the extreme depth and the maximum entry angle that can be achieved (16 degrees), the Single Pass HDD alignment would have to be at least 1000’ long to achieve allowable systemic settlement *under the roundabout*. This would still lead to extensive systemic settlement near the entry and exit locations on Quintana Road, and may require repairing affected utilities, and raising the roadway in the area. These anticipated impacts disqualify this alternative from further assessment (fatal flawed).

- Multi Pass HDD: Utilizing single multi-pass HDDs would reduce borehole diameters but would still require significant offsets so that the overlapping systemic settlement troughs equal less than 1” at the lowest utility crossed. The resulting horizontal footprint would require spacing out the entry locations across Las Tunas and Quintana. The four pipelines would exit at staggered locations along Quintana Road and may cause localized systemic settlement issues that could lead to repairing some utilities and roadway. This alternative does not require additional easement, but would likely leady to widespread traffic impacts on Quintana Rd past the roundabout, and is not preferred.

- A discussion on the various pipe jacking alignment options are listed below:
  - Jacking Pipe Option A is assumed to be the preferred option, as it balances additional easement requirements, business impacts, overall length, and is the least impactful to Quintana Road. Note that this alternative would require extensive working area from the UHAUL property and noise mitigation may be required for nearby sensitive receptors (residences) on Las Tunas. Long term permanent easement would follow the footprint of the pipelines (and casing) and would encroach approximately 20’ from the property line. This is not anticipated to affect the long-term ability of the property to develop, as typical commercial zoning setbacks mean that a building would likely not be able to encroach that space in the first place.
  - Jacking Pipe Option B represents the shortest alignment and is the option with the least traffic impacts to Quintana and Las Tunas, but also requires the most easement from the two affected properties. Similarly to Option A, the long term permanent easement required is not anticipated to affect the long-term ability of the property to develop, as typical commercial zoning setbacks mean that a building would likely not be able to encroach that space in the first place.
  - Jacking Pipe Option C is the only option that does not require additional easement. But due to change in direction across the roundabout, requires two separate pipe jacking operations to a central location within the roundabout Consequently, this option introduces the most traffic impacts to Quintana and the roundabout itself, which is not preferred.

To summarize the information listed in Table 4-14, the Microtunnel trenchless construction methodology represents the least risk due to the ability to control groundwater and would be able to be constructed during the Fall/Winter season, which is outside of the May to October tourism season for Morro Bay. The bore and jack alternatives would likely be less costly but cannot be directly assessed for feasibility until additional geotechnical investigations are conducted (boreholes at the jacking/receiving pits at a minimum) and groundwater monitoring/testing.

At this preliminary stage, a microtunnel installation is preferred, whereby a 48” to 60” steel casing (as a function of pipe spacer design) for the likely preferred alternative would encase the 16”SSFM, 16”Brine, and 12” SSFM. The 8” IPR could potentially be installed within the same pipe if it utilizes an (additional) casing, but DDW may not permit this due to the sewer/water horizontal clearance constraints, even with the additional casing mitigation. For the purposes of providing a preliminary cost estimate, it is assumed that a single microtunnel would house all four pipelines, but in a worst-case scenario, 8” IPR could be offset via an HDD installation.

Highway 1 – Quintana to Teresa

For the Highway 1 transverse crossing from Quintana to Teresa, a pilot tube guided auger boring (PTGAB) above the groundwater is recommended to install a single steel casing (48” or 60” as a function of pipe spacer design). The 8” IPR could potentially be installed within the same pipe if it utilizes an (additional) casing, but DDW may not permit this due to the sewer/water horizontal clearance constraints, even with the additional casing mitigation. For the purposes of providing a preliminary cost estimate, it is assumed that a single microtunnel would house all four pipelines, but in a worst-case scenario, 8” IPR could be offset via a separate PTGAB 12” steel casing.

It is understood that parallel research into the alternative crossings have highlighted that open-cut construction via a longitudinal encroachment on South Bay Blvd is preferable to a trenchless crossing of Highway 1.

4.3.13 Traffic Control & Fencing Constraints

Traffic Control Plan Measures

For cost estimating purposes, four levels of traffic control plan measures were determined and assigned to every section of the alternative alignments. The intent of this categorization is to not match exactly with CA MUTCD traffic control plan requirements, but to characterize the level of effort and equipment utilized by the contractor to provide reasonable traffic control. This is displayed and listed in more detail in Figure 4-40 and Table 4-15 below.

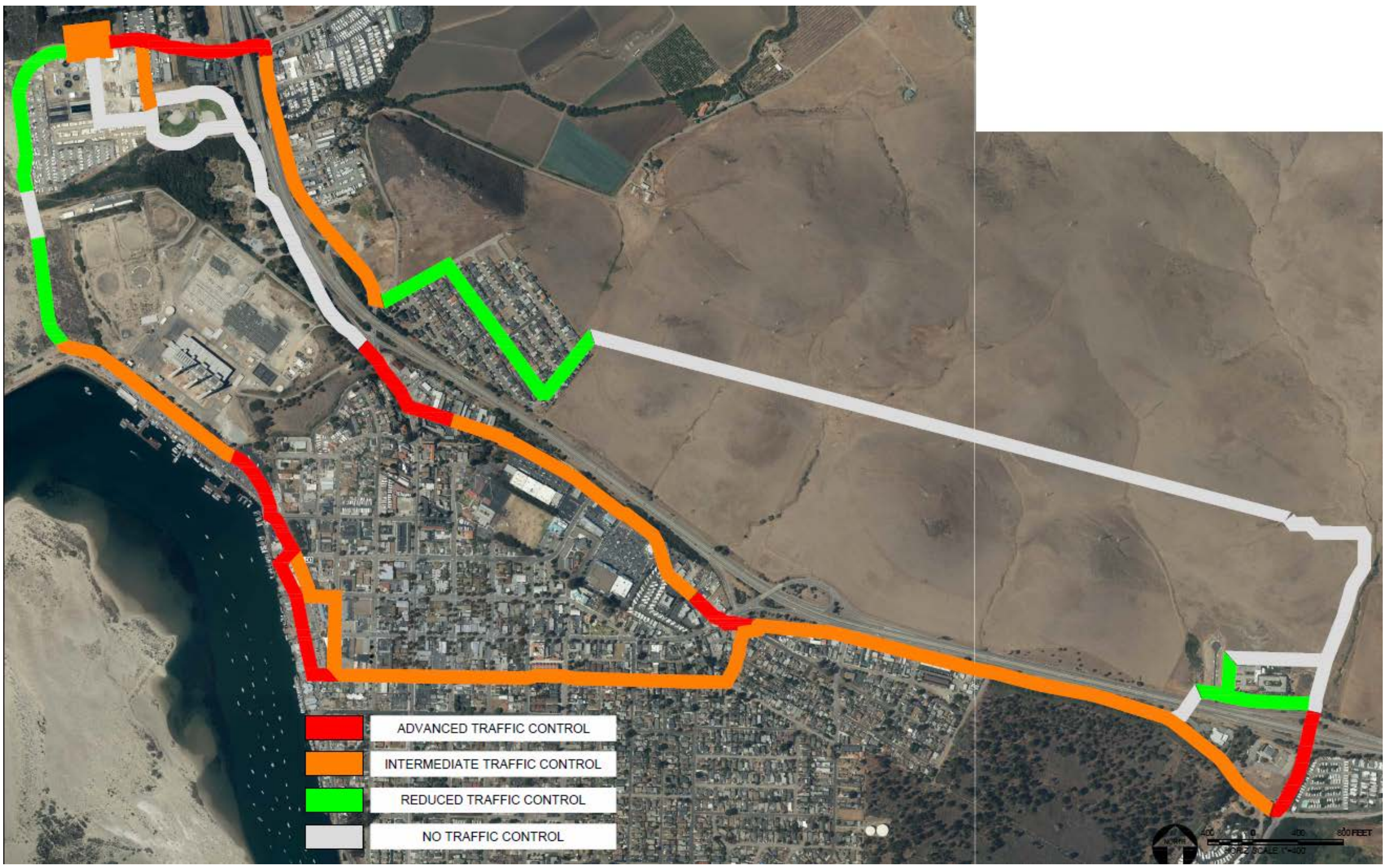


Figure 4-40: Traffic Control Areas by Alignment

Table 4-15: Traffic Control Plan Measures

TCP Level	Cost Impact	Flaggers	Active Signage	Barriers	Passive Signage
Night Work Only*	Very High				
Advanced	High				
Intermediate	Medium				
Reduced	Low				
None Required	Nominal				

*If required for permitting purposes

It is assumed that typical traffic control devices will be utilized to meet the traffic control plan measures. A typical list is below:

- Temporary Barricades (K-Rail and Crash Cushions))
- Mounted signs
- Message Boards
- Arrow Boards
- Flaggers (manual direction directed at traffic via traffic control specialist)

Extended Detours

Extended detours are anticipated for sections of Quintana Road because the working road corridor may be too narrow to provide continuous access to the public via a single lane. This will be further analyzed during the design phase.

Public Beach Access

Public access to coast/beach areas via common points of easement shall be maintained by workers during construction or a reasonable alternate route be provided.

Seasonal Schedule Restrictions

It is anticipated that the City will impose schedule constraints whereby construction will not be allowed during summer months (up to after Harbor Festival) along a portion of the Embarcadero and commercial Quintana corridor. In addition, it is anticipated that the City will impose schedule restrictions whereby construction in Atascadero will not be allowed when Morro Bay High School is in session.

Commercial/Residential Access

Commercial and Residential access along the alternative alignment will be addressed in design phase via right-of-way agreements and during pre-construction public outreach. Workers must maintain access to business during open hours and give sufficient notification and actively pursue coordination with residents and business owners if access is temporarily blocked. It is anticipated that the City and the contractor will work directly with adjacent business owners to develop a traffic control strategy that is amenable to them.

Emergency Services Access

Emergency services must be granted access to any contiguous location along the alignment for worker and public safety.

Trench Plating

Exposed trench excavation construction will be required to be plated and locked in place with cold patch (at a minimum) when workers vacate the site. If the trench is too wide to plate, then every direction that a vehicle can access must be blocked with a continuous segment of temporary k-rail and fencing. There are also Caltrans-specific trench plating requirements that will be addressed during the design phase.

Noise Sensitive Receptors

Noise-sensitive receptors may exist at various locations throughout the project, and could require mitigation, whereby construction activity is limited via more specific hours and or sound barriers are erected during the construction activity. This is anticipated to be discussed in more detail during the design phase of this project after the selection of the preferred alternative.

4.3.14 Right-of-way, Easement, and Encroachment Constraints

Right-of-way

WaterWorks researched existing assessor maps and the City GIS to evaluate potential alternative alignments and maximize use of public right-of-way. Public right-of-way is owned and administered by the City and is primarily occupied by paved road. Utility construction with public right-of-way is dependent on permission from the City via a typical encroachment permit.

Permanent Easement

All the alternatives will require extensive new private easement or a modification of existing easements. Acquiring new easement does introduce risk that the owner will introduce unfavorable preconditions (i.e., modify the appraisal-based compensation or schedule constraints) or refuse to grant access, leading the City to have to expend significant legal resources. The easement procurement process is often dictated by the availability and responsiveness of the land owner or owner's representative which can highly impact project start dates. To that end, it is desirable to maximize use of public right-of-way or existing City easements wherever possible to minimize procurement expenses and schedule constraints. A preliminary right-of-way map detailing property lines and property numbers is displayed in APPENDIX B: Right-of-way Map.

Temporary Construction Easement

All the alternatives will require extensive Temporary Construction Easement (TCE) to ensure space for the contractor to store equipment/materials and retain continuous vehicular access to the work area. Significant working area constraints on the contractor is feasible, will likely affect schedules and prolong construction. Generally, it is recommended that up to a 100-ft wide TCE corridor (including PE) be retained to provide sufficient working area for the contractor.

House Procurement

At the high point of the Embarcadero alignment alternative at the end of the Pacific Ave, the proposed pipelines will directly cross under a house on a private parcel that will have to purchased and demolished. It is anticipated that this parcel will require extensive lead time and City resources to procure and presents a potential risk to the overall project schedule.

Encroachment Permits

It is anticipated that the contractor will pursue a standard encroachment permit with the City prior to construction starting. Typical encroachment permit conditions mandate construction hours; placement of materials; adherence to traffic control plans; and provide a schedule basis.

Caltrans Permitting

An updated utility agreement and transverse/parallel encroachment permit applications will be required with Caltrans. This is discussed in more detail in Section 4.3.18.

4.3.15 Geotechnical Constraints

A draft preliminary geotechnical report was prepared by Yeh and Associates for the WRF Lift Station and Offsite Pipelines and can be found in APPENDIX C: Preliminary Geotechnical Report.

. A summary of the geotechnical constraints identified in that report for the offsite pipelines have been adapted and presented below.

Flooding and Tsunami Risks

The area around Atascadero, near Morro Creek, up a portion of the bike path, and along Embarcadero are all areas within the adjusted 100-yr floodplain or may be inundated during a tsunami. These conditions may cause the ground to become completely saturated which decreases the effective vertical weight of the soil. This in turn can cause an empty pipe to float due to buoyant forces if it is not buried deep enough. Consequently, the minimum depth of a proposed pipeline is not governed solely by overhead vehicular/soil loads, but also by the effective vertical weight of the saturated soil it resides in overcoming an empty pipe buoyant force. Once more geotechnical information is determined along the selected alignment, specific design calculations will be made to mitigate for flooding/tsunami risks.

Liquefaction Considerations

Due to underlying soft, saturated, loose sandy soils found within Atascadero and Embarcadero, these areas are considered at risk of liquefaction during a seismic event (i.e., earthquake). Liquefaction could be manifested as adding buoyant forces to pipelines, loss of thrust resistant along the pipe or thrust blocks, displacement, reduction of bearing resistance, and settlement.

Naturally Occurring Asbestos

Naturally occurring asbestos (NOA) is associated with serpentine rock which is known to exist in the hillsides north-east of HWY-1 (the WRF site), and potentially in select locations along Quintana, and Teresa Dr. Consequently, during the design phase, it may be necessary to implement NOA mitigation into the project specifications, in which case air-control monitoring and mitigation measures are utilized for worker and pedestrian safety. It is important to note that the construction in proximity to the memory care/nursing facility located on Teresa Rd may require additional mitigation.

Over Excavation in Soft Soils

If very soft and muddy soils are identified in the field at the bottom of trenchlines, it may be necessary for the contractor to overexcavate and replace the unsuitable material with engineered fill. This requirement will be further identified during the design phase.

Dewatering

Groundwater levels are anticipated to be relatively high along Embarcadero and the Atascadero (closer to the coast) and may require standard trench dewatering operations when excavating below 5-10 of depth. Groundwater is anticipated to be at deeper elevations along Quintana (approximately 15’-20’ below the surface at the Quintana/Morro Bay roundabout).

Concrete Pavement on Quintana Road

Quintana Road previously utilized concrete pavement in lieu of asphaltic concrete and is still underlying the majority of Quintana Road. The anticipated impact on construction activities is both negative and positive. It is likely that the contractor will have to sawcut through both concrete and AC layers on much of the proposed alignment, which adds complexity and time to excavation activities. On the other hand, the presence of concrete will likely support the edges of trenches, and facilitate tighter, rectangular trenches (instead of large V-shaped open cut which is more common in soft soils).

Embarcadero Revetment and Hydraulic Fill

Much of Embarcadero (fronts Morro Bay or the Pacific Ocean) is buttressed by rip-rap revetments and filled with low-strength hydraulic fill. This creates poor subsurface soil conditions for pipeline construction given the high groundwater table (tidal influenced). In addition, the soils are highly corrosive due to their high salt content. In general, the Embarcadero alignment has more adverse geotechnical constraints versus other (inland) alignments.

Existing Utilities

Close parallel construction to existing utilities is anticipated for most of the alignment. The pipeline will likely stay shallow which will facilitate close construction, but there may be instances where specific design mitigation will be required to ensure safety. It is recommended that excavation be limited to within 3’-5’ horizontal feet of the high-pressure gas pipeline (SoCal Gas) that runs on the northeast edge the majority of Quintana Rd.

Summary of Open-Cut Geotechnical Recommendations

A summary of recommendations to mitigate the geotechnical constraints (for open cut construction) are listed in Table 4-16 below and will be formally incorporated during the design phase.

Table 4-16: Geotechnical Recommendations for Open Cut Construction

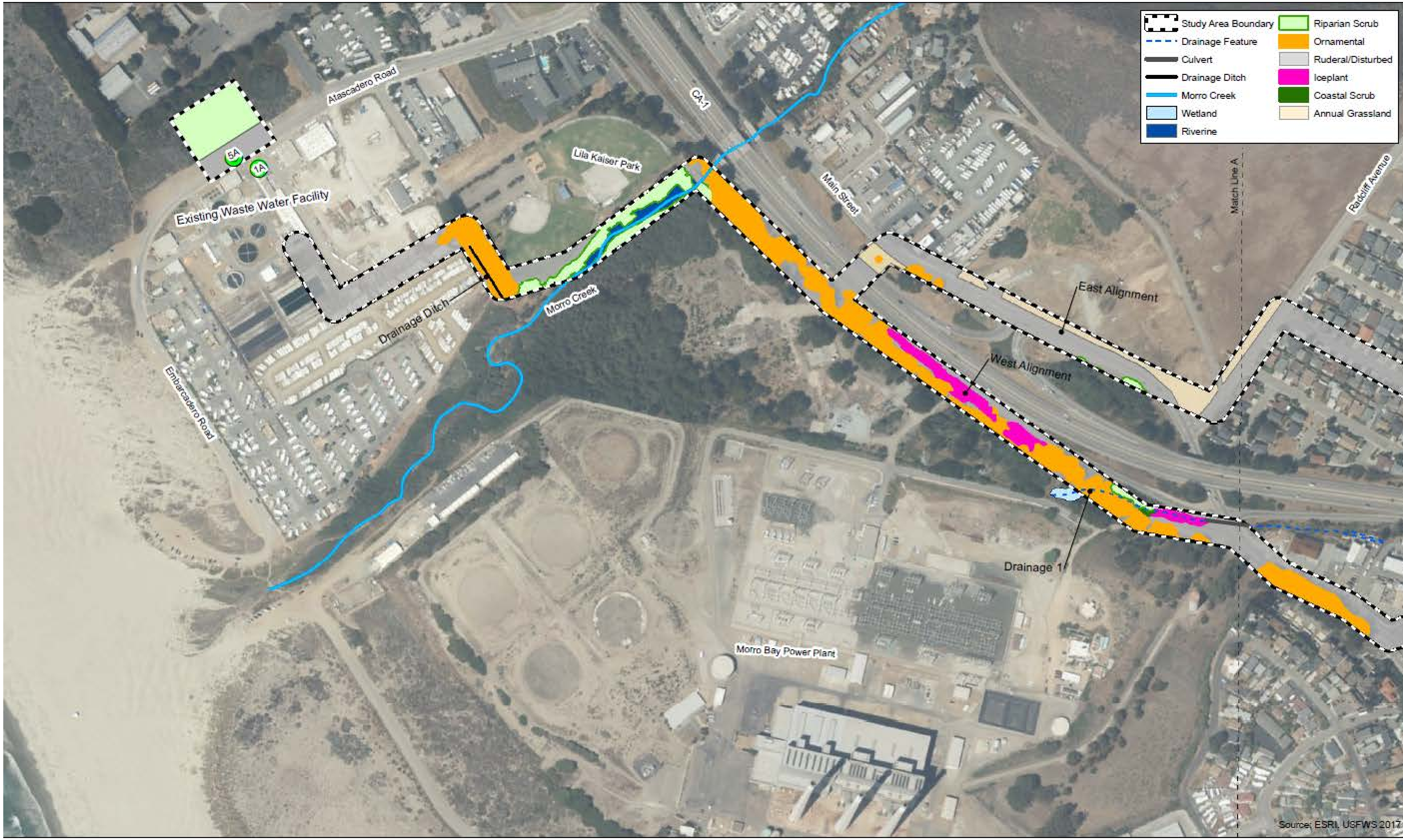
Geotechnical Constraint	Mitigation
Flooding/Tsunami/Liquefaction	Design against flotation by enforcing a minimum depth, utilizing 90%-95% relative compaction of backfill soils (decrease available pore spaces),
Lateral displacement	Utilize durable, flexible pipeline (jointless/ fused pipe is preferable)
Loss of thrust resistance/bearing capacity	Use a truly self-restrained pipeline (no thrust blocks) and do not utilize passive sleeve resistance on the pipeline in calculations
Flexibility connections	Provide sufficient lateral clearances within vaults to accommodate longitudinal movement in the pipe. At the pump station, bridge, and outfall connections, utilize flexible connections (i.e., flexible expansion joints).

4.3.16 Environmental Constraints

The WRF Biological Resources Assessment report was produced by KMA and is summarized below and in overall project EIR documentation.

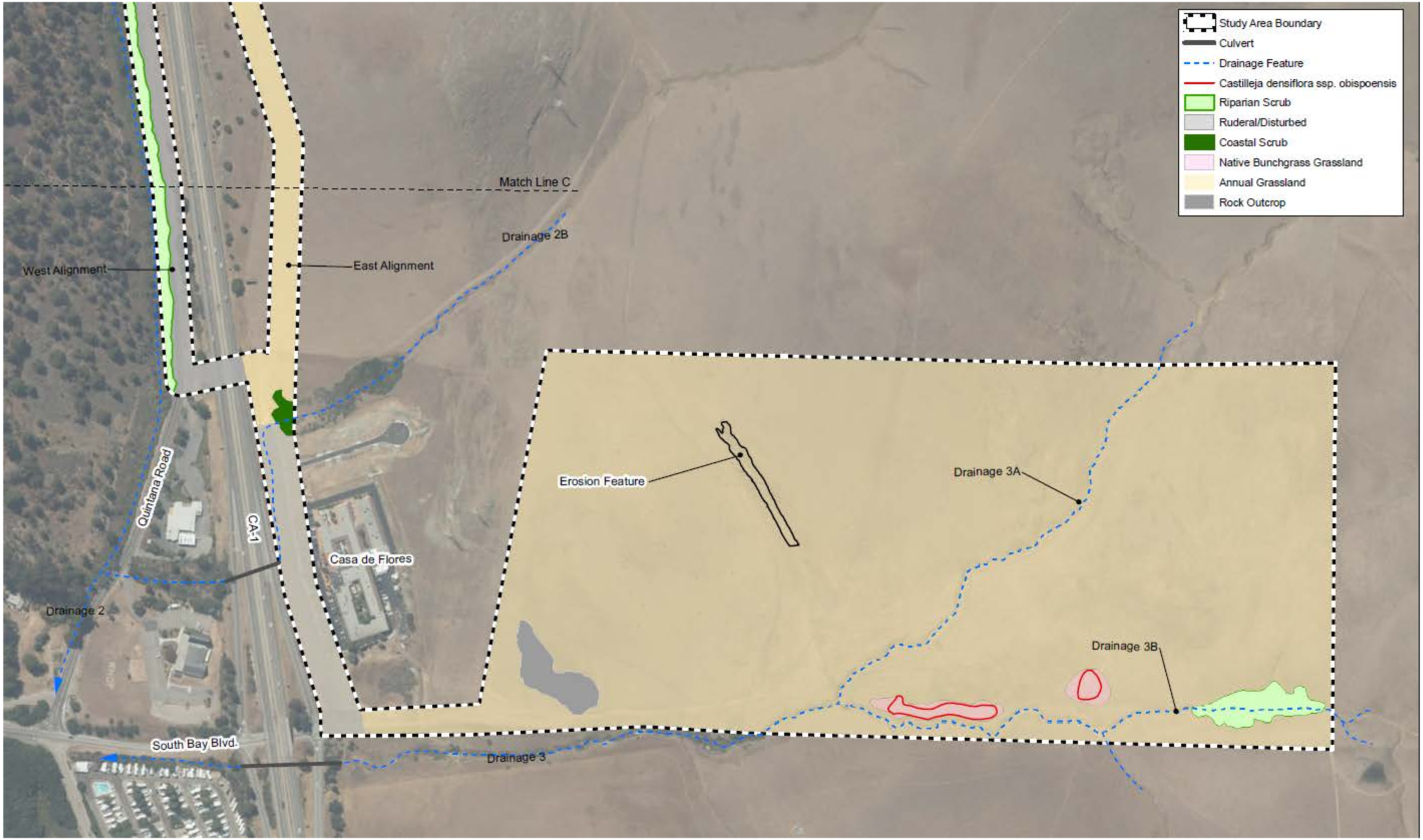
Environmentally Sensitive Areas

Based on the biological resources study conducted for the project EIR, numerous environmentally sensitive areas were delineated at several locations within the City that may impact the alignments. The biological mapping is displayed in the following figures.


Study Area Boundary
Drainage Feature
Culvert
Drainage Ditch
Morro Creek
Wetland
Riverine
Riparian Scrub
Ornamental
Ruderal/Disturbed
Iceplant
Coastal Scrub
Annual Grassland
Atascadero Road
Existing Waste Water Facility
Lila Kaiser Park
CA-1
Main Street
Radcliff Avenue
Match Line A
East Alignment
West Alignment
Drainage Ditch
Morro Creek
Embarcadero Road
Drainage 1
Morro Bay Power Plant
5A
1A
Source: ESRI, USFWS 2017


Study Area Boundary
Drainage Feature
Culvert
Riparian Scrub
Ornamental
Ruderal/Disturbed
Iceplant
Wetland
Coastal Scrub
Annual Grassland
Rock Outcrop
Match Line A
Match Line B
Radcliff Avenue
Drainage 1A
Drainage 1B
East Alignment
Rock Outcrop
Drainage 1
CA-1
West Alignment
Main Street
Morro Bay Blvd.
Source: ESRI, USFWS 2017


Study Area Boundary
Culvert
Drainage Feature
Riparian Scrub
Ornamental
Ruderal/Disturbed
Wetland
Rock Outcrop
Drainage 1A
Drainage 1B
Drainage 2A
Drainage 2B
Drainage 2
Match Line C
Match Line D
Rock Outcrop
East Alignment
West Alignment
CA-1
Morro Bay Blvd.


Study Area Boundary
Culvert
Drainage Feature
Castilleja densiflora ssp. obispoensis
Riparian Scrub
Ruderal/Disturbed
Coastal Scrub
Native Bunchgrass Grassland
Annual Grassland
Rock Outcrop
Match Line C
Drainage 2B
West Alignment
East Alignment
Quintana Road
CA-1
Erosion Feature
Drainage 3A
Casa de Flores
Drainage 2
Drainage 3B
South Bay Blvd.
Drainage 3

Ephemeral Drainage

Several ephemeral/seasonal drainage areas are located parallel to and perpendicular to the various alternative alignments. There are 4 ephemeral drainages along the east alignment and one along the west alignment (the large culvert on the bike path near Main St). It is anticipated that these drainages would be impacted via traditional open cut construction methodology. If trenchless construction is not selected as the preferred method to cross these locations, then the following recommended mitigation measures will be implemented during the design phase:

1. Consult with regulatory agencies and identify all applicable permits that will likely include the following:

    a. USACE Section 404 permit (waters of the state)

    b. RWQCB Section 401 WQ Certification

    c. CDFW Section 1602 Streambed Alteration Agreement (beneficial wildlife habitat)

    d. ESHA (environmentally sensitive habitat areas) from the Morro Bay LCP and the CCC

It is anticipated that these permits would require extensive permit coordination and introduces some risk to the overall project schedule.

2. Project footprint and impacts are calculated to federal and state jurisdictional areas. A Habitat Mitigation and Monitoring Plan (HMMP) will be prepared and implemented during construction and for estimated period of 5 years afterwards.

3. A biological monitor will be utilized to ensure compliance to all permit requirements (pre-construction onwards).

4. A Diversion and Dewatering Plan may be required in case there is flowing or ponded water in the work area.

5. Other environmental project documentation will be prepared and followed during construction which may include the following items: Erosion Control Plan, Spill Prevention Plan, fueling/staging BMPs, washouts,

SWPPP

A NPDES General Construction Storm Water Permit will be required for this linear underground project (LUP) as the disturbed area will exceed 1 acre. In accordance with the final EIR released on June 2018, the City will prepare a SWPPP plan prior to construction that includes BMPs to control erosion, sediment, and hazardous materials release. In lieu of installing SWPPP BMPs along the entire project alignment, the City may be able to apply for a dry weather amendment to reduce BMP application in certain areas during summer months. Various common BMPs that are used on linear projects are listed below:

- Temporary silt fence: tarp-like fencing with gravel toe that naturally filters runoff
- Covers: temporary application of plastic tarps to cover stockpiles and disturbed ground
- Sediment basins: temporary lined basins for dewatering operations to encourage settling of suspended solids for eventual land discharge, discharge to drainage/streams, or to sewer.
- Check dams: fiber rolls placed perpendicular to drainage lines
- Fiber rolls: placed along contour lines to facilitate filtering of runoff and discouraging scouring.
- Inlet protection: fiber rolls and fabric used to cover drainage inlets and filter runoff)
- Hydroseeding (+ mulch & soil binders): typically used for land restoration purposes during post construction activities; seed mix is dependent upon environmental recommendation.

The extra application of SWPPP BMPs are occasionally utilized to fulfill specific environmental mitigation requirements related to proximity to environmentally sensitive areas such as riparian areas, wetlands, waterways, etc. For the purposes of cost estimating it is assumed that there are additional biological constraints and BMPs while near Morro Creek and along certain portions of the Highway 1 bike path that front drainage features.

Dewatering & Groundwater Discharge

It is anticipated that the contractor will seek to discharge nuisance dewatering from excavation activities. This flow may be discharged via several methodologies which are listed in Table 4-17 below.

Table 4-17: Dewatering Discharge

Methodology	Description	Permitting Requirements
Land applied discharge	Water is applied to flat areas and allowed to percolate	Low – Set up through project SWPPP
Discharge to natural drainage features	Discharge directly into a natural drainage feature (i.e., creek)	High – Requires NPDES permit and treatment
Discharge to City storm drains	Discharge controlled volume into storm drain inlets	High – City process, may require separate NPDES permit and treatment
Discharge to City sewer	Discharge controlled volume into adjacent City sewer	Medium – City process; may require treatment

Trees and Foliage

The alignment alternatives are primarily located in public right-of-way and roadways, but there many trees near the bike path. Pending a formal tree survey by an arborist, it appears that most of the trees along the edge of the bike path are ornamental (cypress, pine, or eucalyptus). For the purposes of producing a preliminary cost estimate, WaterWorks has identified that there may be up to 20 trees that would be impacted by the West Alignment and could require pruning or removal. During the design phase, WaterWorks will coordinate closely with an arborist and property owner (PG&E and Dynegy) to clearly identify impacted trees and provide a tree protection plan and recommend mitigation procedures. Some common mitigation procedures for protected trees consist of high-visibility fencing, temporary tree “bumpers”, and temporary bollards/fencing.

Special Status Wildlife and Nesting Birds

Special status wildlife (such as the Morro shoulderband snail or MSS) and nesting birds could potentially be present within the study area and affected by construction. Preconstruction surveys will be required to confirm their presence or absence. Coordination with USFWS may be necessary to facilitate receipt of a concurrence determination if MSS is absent from the project area. Utilizing silt fencing (a SWPPP BMP) is a common recommended methodology to isolate construction areas from impacting MSS habitat areas. Biological monitoring during construction and training will likely be required.

It is recommended that construction near or in environmentally sensitive areas that may have nesting birds (removal of trees or annual grassland habitat disturbance) be limited to September 1 and February 14 if feasible. If that is not possible, then a qualified biologist should conduct pre-construction surveys for nesting birds within the project limits. Active nests that are observed would require specific mitigation, buffers, and project documentation that may be submitted to USFWS and CDFW.

Surface Restoration

Hydroseeding and landscaping may be required to restore native surfaces where ground is disturbed within construction limits. Construction within Lila Keiser Park, for instance, will require turf replacement. In locations where there is no pavement and the City wishes to retain a semi-permanent and drivable surface (and no overlying foliage) it may be necessary to specify 3”-6” gravel or aggregate base. There may be additional requirements post-construction to irrigate hydroseeded areas to promote growth for a specified frequency and duration. Surface restoration in environmentally sensitive areas would likely be required to meet permit requirements.

Fugitive Dust Control

A fugitive dust control permit will be required from SLOAPCD. Typical mitigation measures will be spraying down open and disturbed areas via water trucks or hand-operated hoses. This may be an avenue for a SWPPP Land Applied Discharge to facilitate the removal of groundwater from dewatering operations. For the purposes of cost estimating, dust control measures are included in the cost of excavation.

4.3.17 Cultural Resources Constraints

WaterWorks accessed existing EIR documentation, public comments, and coordinated with the City to ascertain known and potential cultural resources constraints that would affect the development and assessment of alignment alternatives.

Summary of Findings

Research conducted in support of the overall project EIR highlight numerous cultural resources areas that are in proximity to several alignments but appear to be concentrated near Morro Creek.

High Buried Potential

The area-specific potential for excavating buried cultural resources as estimated within the EIR support documentation provided the basis for the likelihood of cultural resource monitoring. WaterWorks estimated cultural resource monitoring for each alignment alternative and assigned a preliminary cost estimate to the activity.

Monitoring and Mitigation Measures

Cultural resource monitoring and mitigation is an official process outlined within the project EIR whereby a qualified individual would be onsite to monitor excavation activities by the contractor. If buried cultural resources are uncovered during construction, specific mitigation procedures are implemented.

4.3.18 Concerns of Outside Stakeholders

WRF Project Schedule

The current WRF Project Schedule lists the following milestones and schedule constraints in Table 4-18.

Table 4-18: WRF Projection Schedule

Milestone	Date
WRF Design Build Award	August 2018
Offsite Pipelines/Lift Station Award	November 2019
Construction Completion	October 2021
RWQCB permit compliance	February 23, 2022

Based on the schedule constraints, it is anticipated that the City has a low tolerance for extensive permitting or ROW acquisition during the design phase that would delay the project start date of November 2019.

Caltrans

Any transverse or parallel crossing of Caltrans Right-of-way (State of California) will require an encroachment permit and updated utility agreement. Various locations along the preferred alignment that will involve permitting with Caltrans are listed below:

1. Parallel to SB HWY-1 Connector – It is assumed that the continuous chain-link fencing running parallel to Hwy-1 represents the property line. Construction that crosses the fence or impacts the fence requires permitting and coordination with Caltrans. A route along the unpaved shoulder of the SB HWY-1 connector from Atascadero was identified that would provide the means to avoid impact to cultural resources in the Lila Keiser Park area and along west Atascadero/Embarcadero areas.

2. Atascadero Road (West of Hwy-1) – City owned, however construction and traffic control measures that may affect Hwy-1 on/off ramps to Atascadero Rd may require coordination with Caltrans

3. Atascadero @ Hwy-1 – The ROW transitions from City-owned to State ROW near Hwy-1, under bridge (49-182) HWY-1, and up State Route 41 ("Morro Rd").

4. End of Bike Path to Unnamed Road to Main St– City bike path easement runs through PGE property and then transitions to State ROW and onto an unnamed roadway up to Main St.

5. South Bay Blvd @ Hwy-1 – ROW transitions from City-owned to State ROW near Hwy-1, under bridge (BR49-177)

The map of these numbered areas is displayed in Figure 4-41.

Figure 4-41: Caltrans Encroachment Sections

Any crossing of Caltrans Right-of-way will require a standard transverse encroachment permit. In addition, any longitudinal encroachment (Atascadero/HWY-41 and SB HWY-1 connector) will require a longitudinal exception. WaterWorks has coordinated with the Caltrans District 05 lead permit engineer to discuss Caltrans constraints required for permit approval.

WaterWorks and the City will be requesting casing/double containment exemptions at several locations (Locations 2/3 and 4 in the image above) for the reasons listed below:

- The pipeline that is being utilized is flexible, self-restrained, jointless pipe with no service connections (i.e., points of failure). WaterWorks would be amenable to increasing the pipe thickness as an additional safety factor in lieu of providing double containment via a casing.
- The Atascadero to Highway-41 corridor is in a central location, utilized by residents as a common thoroughfare, and encompasses several other City-owned parallel utilities with

no casings. In addition, the unnamed road from Main St to the bike path is a low-volume access point for the back of the Dynegy and PGE properties and is assumed to not be a critical facility to Caltrans. Given the City's long partnership with Caltrans in maintaining their existing facilities in State ROW and given the City's capacity to immediately address and manage emergency repairs, it is requested that this be an additional source of consideration in waiving the additional casing exemption.

- Avoidance is not feasible due to the constraints listed within this report and given the size and scope of the overall WRF project, the City is under significant budgetary constraints whereby the fiscal impact of requiring a fully separate casing is prohibitive.

In addition, WaterWorks will request approval for the use of FPVC as a potential casing material (standard casings are currently HDPE and steel). Existing research has shown that Caltrans has approved FPVC casings in over a half dozen examples across California, which are listed in the Figure 4-42 below which was recently provided by AEGION Underground Solutions.

Permit No.	District	Engineer	Project	Scope
1012-NUJ-0693	Santa Nella	NV5 Reid Johnson	Santa Nella Well #1 Water Main	8" DR 18 Fusible PVC® Casing with 6" DR 18 Fusible PVC® Carrier
0412-NUJ-0860	City of Napa	City of Napa Megan Thomas	2013 Water Main Freeway Crossings (F Street)	16" DR 18 Fusible PVC® Casing with 12" DR 14 Fusible PVC® Carrier
0413-NUJ-0135	City of Napa	City of Napa Megan Thomas	2013 Water Main Freeway Crossings (Sierra Ave)	24" DR 18 Fusible PVC® Casing with 16" DR 18 Fusible PVC® Carrier
0410-NUJ-1316	City of Napa	City of Napa Megan Thomas	2013 Water Main Freeway Crossings (Salvador Ave)	24" DR 18 Fusible PVC® Casing with 16" DR 18 Fusible PVC® Carrier
0409-6DP-1946	City of Napa	City of Napa Megan Thomas	Lincoln Avenue Crossing	12" DR 18 Fusible PVC® Carrier by HDD under offramp right-of-way
California Department of Transportation Utility Agreement 1780.1	North Marin Water District	NMWD Drew McIntyre	Aqueduct Relocation AEEP Reaches A-D/MSN B3	**D1 Line** - 12" DR 18 Fusible PVC® Casing with 4" DR 14 Fusible PVC® Carrier pipe **D2 Line** – 12" DR 18 Fusible PVC® Casing with 4" DR 14 Fusible PVC® Carrier pipe and 2" Sch 40 conduit **D3 Line** - 6" DR 18 Fusible PVC® Casing with 2" Sch 80 PVC Carrier pipe **D4 Line** – 16" DR 18 Fusible PVC® Casing with 8" DR 14 Fusible PVC® Carrier pipe
0414-NUJ-0331	City of Pleasanton	Kimley-Horn Eric Biland	Meadowlark Sewer Siphon	12" DR 18 Fusible PVC® Casing pipe with 8" DR 18 Fusible PVC® Carrier pipe
0314-NUS0840	City of Sacramento	Kennedy Jenks Sean Maguire		24" DR 18 Fusible PVC® Casing pipe with 12" DR 18 Fusible PVC® Carrier pipe
TBD	City of Redding	Pace Engineers Rick Bowser	Lakeport HDD	16" DR 18 Fusible PVC® Casing pipe with 12" DR 18 Fusible PVC® Carrier pipe

Figure 4-42: FPVC Carrier and Casing Caltrans Approval Examples

DDW

DDW horizontal sewer-potable water separation requirements were previously discussed in Section 4.3.5.

Cayucos Sanitary District

It is understood that Cayucos Sanitary District is constructing a new wastewater treatment facility, forcemain improvements, and pump station that will be in the Atascadero Rd area. In addition, it is assumed that the discharge from the pump station will be directed towards the existing gravity outfall structure maintained by the City within the limits of the existing wastewater treatment plant. WaterWorks anticipates coordination with Cayucos Sanitary District will be conducted on an as-needed basis through the City.

Native American Tribes

During the draft EIR public comment process Native American tribal representatives commented on the proposed offsite pipeline alignments and objected to the siting of the pipelines through or near to the cultural resource areas near Morro Creek (Lila Keiser Park) and along Embarcadero. Subsequent discussions with this representative led to the development of an alternative alignment that would utilize Atascadero Rd and the shoulder of the SB HWY-1 connector (parallel to the bike path in State ROW) would minimize impacts to known cultural resource areas in the Lila Keiser Park area.

Dynegy (Vistra Energy) & Morro Bay Mutual Water Company (MBMWC)

Dynegy merged with Vistra Energy in 2017 and owns a 90-acre private property which encompasses the Lila Keiser Park area and the bike path that runs parallel to HWY-1. The Morro Bay Mutual Water Company operates within Dynegy and sources raw groundwater from a single well (Well #2). An additional well (Well #4) was constructed in 2010 as a backup but is not in compliance with CCC. According to correspondence with Dynegy, the operational capacity of Well #2 is significantly reduced (showing signs of failure). The raw water is the primary source for filling the elevated raw water tank, which provides an additional source of fire water for the power plant, Marine Mammal Center, Pacific Wildlife Care facilities, and PG&E switchyard. There are two additional water connections to City (one on Embarcadero, and one emergency connection near Well #2 at the bike path). Given that the plant is now decommissioned, pumping operations for MBMWC at Well #2 are very intermittent and incidental. Based on routing analysis conducted by WaterWorks, Well #2 will need to be decommissioned due to the close proximity of the proposed West Alignment. To make this amenable to MBMWC and Dynegy, the City could negotiate providing additional water supplies to the parcel or continue using the existing water connections (2x connections).

As previously discussed, the City has several easements within the Dynegy property for the bike path, the old 8" SSFM from LS-2, and several water lines. It is anticipated that the City will need to either clean-up and renegotiate the 20' wide bike path easement to include the proposed forcemains or purchase additional parallel easement.

PG&E

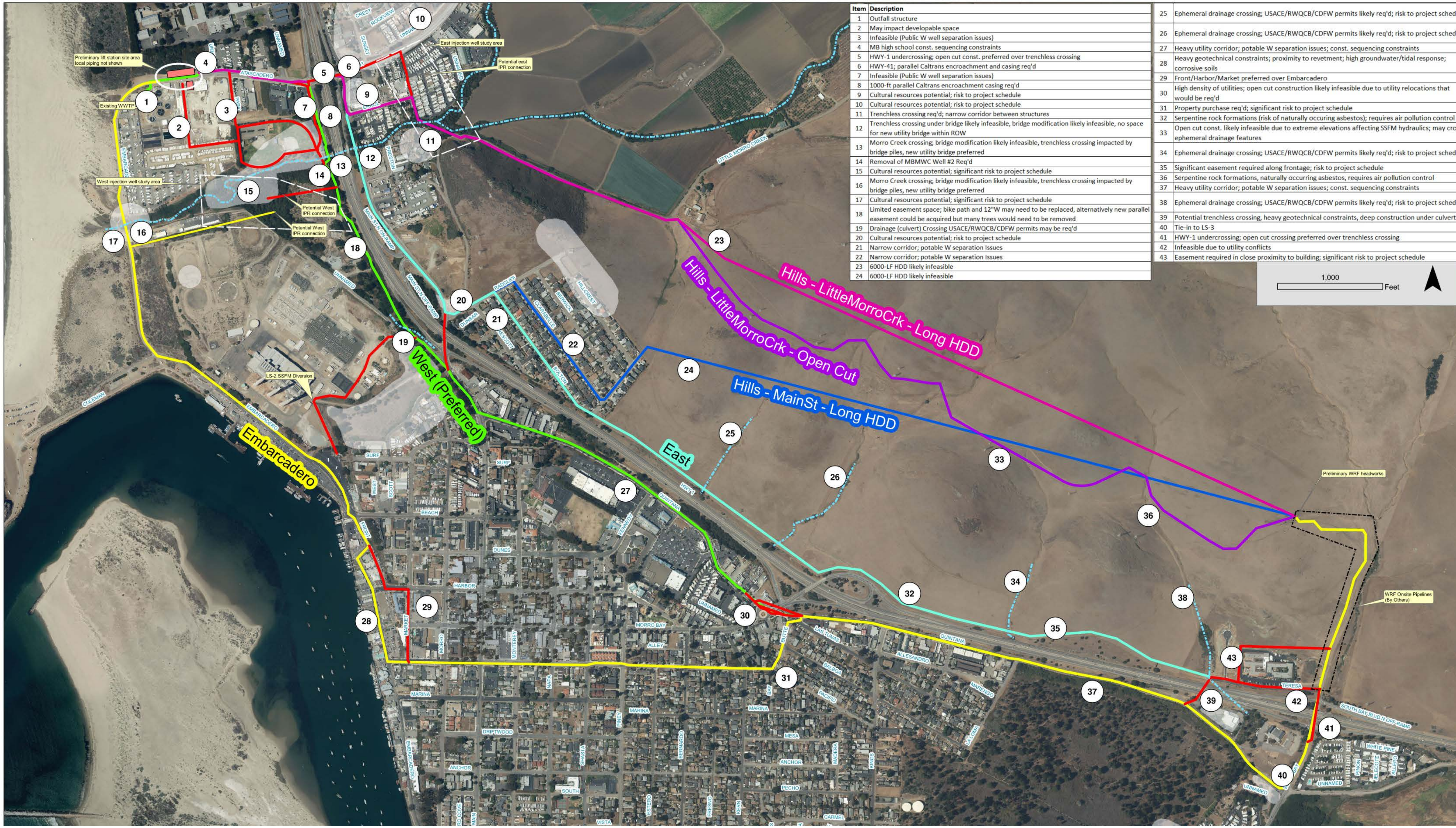
The large property that is surrounded by Dynegy and State ROW is owned by PG&E and is primarily occupied by a large switchyard and several major overhead power lines. The City has several easements within the PG&E property for the bike path, the old 8" SSFM from LS-2, and several water lines. It is anticipated that the City will need to either clean-up and renegotiate the 20' wide bike path easement to include the proposed forcemains or purchase additional parallel easement.

4.4 Alignment Alternatives Development

For the first step in the assessment, WaterWorks identified five working alignments which are summarized below.

- **West Alignment** – Runs east along Atascadero, southeast along the existing bike path, and down Quintana Rd parallel to HWY-1 on the southwest side. This alignment was originally identified as the "west" alignment because it is located west of HWY-1.
- **East Alignment** – Runs east along Atascadero, and then southeast on Main St and then via new easement parallel to HWY-1 on the northeast side. This alignment was originally identified as the "east" alignment because it is primarily located east of HWY-1.
- **Embarcadero Alignment** – Runs west and then south along Embarcadero, then east along Pacific, and Quintana parallel to HWY-1 on the southwest side.
- **Hills-Creek Alignment (Little Morro Creek – open cut, or Long HDD)** – Runs east along Atascadero, northeast along HWY-41 and then cuts across the rolling hills above the City and into the County limits. This represents the shortest possible alignment to the WRF
- **Hills-Radcliff Alignment (Main St – Long HDD)** – Runs east along Atascadero, along Main St and then cuts through rolling hills above the City and crosses into the County limits.

The pipeline design criteria and identified project constraints for all of the working alignments is highlighted in Figure 4-43 below.

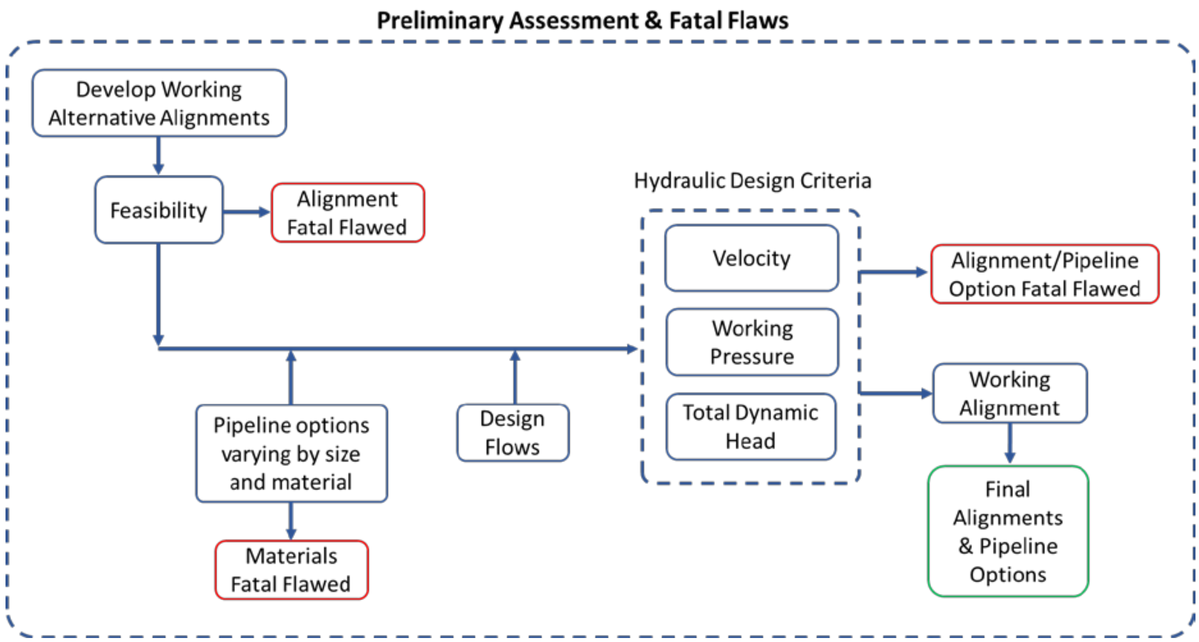


Item	Description
1	Outfall structure
2	May impact developable space
3	Infeasible (Public W well separation issues)
4	MB high school const. sequencing constraints
5	HWY-1 undercrossing; open cut const. preferred over trenchless crossing
6	HWY-41; parallel Caltrans encroachment and casing req'd
7	Infeasible (Public W well separation issues)
8	1000-ft parallel Caltrans encroachment casing req'd
9	Cultural resources potential; risk to project schedule
10	Cultural resources potential; risk to project schedule
11	Trenchless crossing req'd; narrow corridor between structures
12	Trenchless crossing under bridge likely infeasible, bridge modification likely infeasible, no space for new utility bridge within ROW
13	Morro Creek crossing; bridge modification likely infeasible, trenchless crossing impacted by bridge piles, new utility bridge preferred
14	Removal of MBMWC Well #2 Req'd
15	Cultural resources potential; significant risk to project schedule
16	Morro Creek crossing; bridge modification likely infeasible, trenchless crossing impacted by bridge piles, new utility bridge preferred
17	Cultural resources potential; significant risk to project schedule
18	Limited easement space; bike path and 12"W may need to be replaced, alternatively new parallel easement could be acquired but many trees would need to be removed
19	Drainage (culvert) Crossing USACE/RWQCB/CDFW permits may be req'd
20	Cultural resources potential; risk to project schedule
21	Narrow corridor; potable W separation Issues
22	Narrow corridor; potable W separation Issues
23	6000-LF HDD likely infeasible
24	6000-LF HDD likely infeasible
25	Ephemeral drainage crossing; USACE/RWQCB/CDFW permits likely req'd; risk to project schedule
26	Ephemeral drainage crossing; USACE/RWQCB/CDFW permits likely req'd; risk to project schedule
27	Heavy utility corridor; potable W separation issues; const. sequencing constraints
28	Heavy geotechnical constraints; proximity to revetment; high groundwater/tidal response; corrosive soils
29	Front/Harbor/Market preferred over Embarcadero
30	High density of utilities; open cut construction likely infeasible due to utility relocations that would be req'd
31	Property purchase req'd; significant risk to project schedule
32	Serpentine rock formations (risk of naturally occuring asbestos); requires air pollution control
33	Open cut const. likely infeasible due to extreme elevations affecting SSFM hydraulics; may cross ephemeral drainage features
34	Ephemeral drainage crossing; USACE/RWQCB/CDFW permits likely req'd; risk to project schedule
35	Significant easement required along frontage; risk to project schedule
36	Serpentine rock formations, naturally occurring asbestos, requires air pollution control
37	Heavy utility corridor; potable W separation issues; const. sequencing constraints
38	Ephemeral drainage crossing; USACE/RWQCB/CDFW permits likely req'd; risk to project schedule
39	Potential trenchless crossing, heavy geotechnical constraints, deep construction under culvert
40	Tie-in to LS-3
41	HWY-1 undercrossing; open cut crossing preferred over trenchless crossing
42	Infeasible due to utility conflicts
43	Easement required in close proximity to building; significant risk to project schedule

Figure 4-43: Overall Project Constraints By Working Alignment

4.4.1 Preliminary Assessment & Fatal Flaws

WaterWorks then conducted a preliminary assessment of the working alternative alignments which is visualized in the diagram below and further discussed.



Preliminary Feasibility Assessment

Based on the pipeline design criteria and identified constraints, fatal flaws were identified for several working alignments whereby the alignment or pipeline option was disqualified from further assessment primarily due to constructability concerns (feasibility). In addition, an alignment may be fatal flawed due to an unacceptable risk of not meeting the overall project schedule due to extensive permitting or right-of-way acquisition lead time. It is anticipated that the offsite pipeline (and pump station) construction will take 18 months to complete, and the City is planning on the construction project starting in November 2019 and completing by October 2021.

The different fatal flawed alignments are listed in Table 4-19 below with a description of the fatal flaws.

Table 4-19: Working Alignment Alternative Fatal Flaws

Working Alignment	Fatal Flaws
East Alignment	• Constructability: No practical way to cross Morro Creek: limited space for pipe bridge, trenchless crossing impacted by existing bridge piles • Environmental: The four ephemeral drainage lines and wetlands in the grassland northeast of HWY-1 would require separate environmental permits (404/401/1602) that would introduce unacceptable risks to the project schedule. Numerous trenchless crossings would be cost prohibitive. • Hydraulic: Extreme elevation changes close to HWY-1 introduce significant static head requirements (>220') which produce maximum TDH values that are infeasible for preferred pump station alternatives.
East/West Alignment	• Constructability: Two long HDD operations would be required that would put the pipelines at extreme depths. These may be feasible based on preliminary research into site geology, but the cost of the operation and a minimum two casings to meet DDW requirements at that extreme depth and length (> 100' deep) would be prohibitive and would add at least $4M more than the West Alignment
Hills-Creek Alignment - (Little Morro Creek) Open Cut	• Constructability: No practical way to cross Morro Creek and Little Morro Creek: no location for pipe bridge, and HDD would pass very closely to houses near the alignment which is not preferred • Environmental: Multiple ephemeral drainage lines in the may require separate environmental permits (404/401/1602) that would introduce unacceptable risks to the project schedule. Numerous trenchless crossings would be cost prohibitive. • Hydraulic: Extreme elevation changes along the hillside portion of the alignment introduce significant static head requirements (>250') which produce maximum TDH values that are infeasible for preferred pump station alternatives
Hills-Radcliff Alignment – Long HDD	• Constructability: No practical way to cross Morro Creek and Little Morro Creek: no location for pipe bridge, and HDD would pass very closely to houses near the alignment which is not advisable. • Hydraulic: The two Hills alignments which utilize Long HDDs (>6000') may be feasible based on preliminary research into site geology, but the cost of the operation and a minimum two casings to meet DDW requirements at that extreme depth and length (> 100' deep) would be prohibitive and would add at least $5-10M more than the West Alignment

Sewer Forcemain Size and Material Selection

Based on DDW horizontal-separation criteria and geotechnical constraints, WaterWorks recommends that Fusible PVC and Fused HDPE be competitively bid against each other as the preferred materials of choice. Segmental pipe (PVC, FRP, and DIP) was effectively fatal flawed as mainline pipe for these reasons but may still be utilized at connection points to intermediate valves and pump station(s).

Consequently, WaterWorks developed a range of FPVC and HDPE size and thickness (DR rating) pipeline options. This is listed in Table 4-20 below for the sewer forcemains.

Table 4-20: Preliminary Sewer Forcemain Pipeline Material and Sizes

# of FM	Material	Nominal Diameter	Equivalent Inner Dia.	DR	Max Working Pressure
Single FM	FPVC (DIPS C900)	16”	15.35”	18	235 psi
		20”	19.06”	18	235 psi
	HDPE (IPS PE4710)	20”	16.86”	13.5	160 psi
		22”	18.54”	13.5	160 psi
Dual FM	FPVC (DIPS C900)	12” + 12”	16.48” (11.65 & 11.65)	18	235 psi
		12” + 16”	19.27” (11.65 & 15.35)	18	235 psi
	HDPE (IPS PE4710)	14” + 14”	16.69” (11.8 & 11.8)	13.5	160 psi
		14” + 20”	20.58” (11.8 & 16.86)	13.5	160 psi

**The safety factors utilized for the max working pressure are not the same between 4710PE and C900 and are further discussed in in Section 4.3.1*

Sewer Forcemain Hydraulic Assessment

The hydraulic performance of the different working alternative alignments and range of pipeline options were assessed to identify what the maximum TDH and velocities were across a range of flow conditions. This is summarized in Table 4-21.

Table 4-21: Final Alignment Forcemain Hydraulic Performance

Alignment	Pipeline Scenario	Equiv. ID	Buildout Conditions + CIPs 8.14 mgd (10yr24hr PWWF) Max TDH	Buildout Conditions + CIPs 8.14 mgd (10yr24hr PWWF) Velocity in all available FM	Flow Conditions 0.3 mgd (low summer) Velocity in Single Smallest FM	Flow Conditions 2.0 mgd (>95% of tot. flow 2017) Velocity in Single Smallest FM	Flow Conditions 2.74 mgd (high summer) Velocity in all available FM
West Alignment (Preferred Segments)	14"FM + 20"FM + 20"Brine DR 13.5 HDPE	20.58 (11.8 & 16.86)	229	5.45	0.61	4.07	1.84
	14"FM + 14"FM + 20"Brine DR 13.5 HDPE	16.69 (11.8 & 11.8)	359	8.29	0.61	4.07	2.79
	12"FM + 16"FM + 16"Brine DR 18 FPVC	19.27 (11.65 & 15.35)	253	6.22	0.63	4.18	2.09
	12"FM + 12"FM + 16"Brine DR 18 FPVC	16.48 (11.65 & 11.65)	369	8.51	0.63	4.18	2.87
	20"FM + 20"Brine DR 13.5 HDPE	16.86	350	8.12	0.3	2	2.73
	16"FM + 16"Brine DR 18 FPVC	15.35	457	9.8	0.36	2.41	3.3
	22"FM + 20"Brine DR13.5 HDPE	18.54	278	6.72	0.25	1.65	2.26
	20"FM + 16"Brine DR 18 FPVC	19.06	258	6.36	0.23	1.56	2.14
East Alignment (Preferred Segments)	14"FM + 20"FM + 20"Brine DR 13.5 HDPE	20.58 (11.8 & 16.86)	237	5.45	0.61	4.07	1.84
	14"FM + 14"FM + 20"Brine DR 13.5 HDPE	16.69 (11.8 & 11.8)	382	8.29	0.61	4.07	2.79
	12"FM + 16"FM + 16"Brine DR 18 FPVC	19.27 (11.65 & 15.35)	284	6.22	0.63	4.18	2.09
	12"FM + 12"FM + 16"Brine DR 18 FPVC	16.48 (11.65 & 11.65)	412	8.51	0.63	4.18	2.87
	20"FM + 20"Brine DR 13.5 HDPE	16.86	371	8.12	0.3	2	2.73
	16"FM + 16"Brine DR 18 FPVC	15.35	512	9.8	0.36	2.41	3.3
	24"FM + 20"Brine DR13.5 HDPE	20.23	260	5.64	0.21	1.39	1.9
	20"FM + 16"Brine DR 18 FPVC	19.06	290	6.36	0.23	1.56	2.14

	Fatal Flawed
	Feasible, but not advisable
	Feasible and within typical performance standards

Based on this assessment, most of the pipeline options drop out of consideration due to these following fatal flaws:

1) Max TDH > 300 ft
2) Velocity in smallest FM < 1.5 ft/s at @ 2.0 mgd.

The remaining pipeline options (not fatal flawed) are listed below in Table 4-22.

Table 4-22: Final Sewer Forcemain Pipeline Material and Sizes

# of FM	Material	Nominal Diameter	Equivalent Inner Dia.	DR	Max Working Pressure
Single FM	FPVC (C900 DIPS)	20”	19.06”	18	235 psi
	HDPE IPS (IPS PE4710)	22”	18.54”	13.5	160 psi
Dual FM	FPVC (C900)	12” + 16”	19.27” (11.65 & 15.35)	18	235 psi
	HDPE IPS (PE4710)	14” + 20”	20.58” (11.8 & 16.86)	13.5	160 psi

Brine and IPR Size and Material Selection

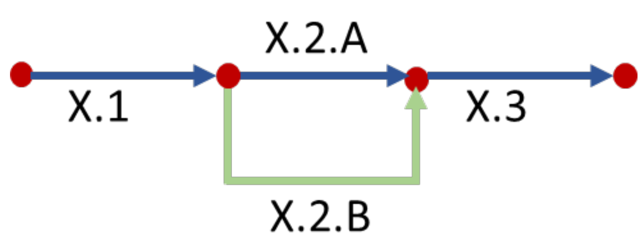
The Brine and IPR lines should match the material of the sewer forcemains to leverage economy of scale (better pricing). Based on the design criteria presented herein, the following materials are recommended in Table 4-23. Note that IPR and Brine pipelines will be further optimized during the design phase in conjunction with the WRF Program Team

Table 4-23: Brine/IPR Pipeline Material and Sizes

# of FM	Material	Nominal Diameter	Equivalent Inner Dia.	DR	Max Working Pressure
IPR	FPVC (C900)	8”	7.98”	18	235 psi
	HDPE IPS (PE4710)	8”	7.27”	13.5	160 psi
Brine	FPVC (C900)	16”	15.35”	18	235 psi
	HDPE IPS (PE4710)	20”	16.86”	13.5	160 psi

4.4.2 Final Alignments

The final alignments that were selected are the West and Embarcadero alignments (which share a common corridor down Quintana past the Morro Bay/Roundabout). Multiple options to traverse a certain area were also accounted for. The alignment segment notation is explained in the following diagram.



The final alignments are listed in Table 4-24 by segment and within Figure 4-44. An initial set of figures which follow the final alignments and display color coded utilities are shown APPENDIX D: Preliminary Offsite Pipeline Alignment Figures, which was utilized during a project workshop with the City.

Table 4-24: Final Alignment Alternative Segments

Offsite Pipeline Final Alignment Alternatives			
Segment		Quick Description	Length (ft)
Embarcadero	E1	Atascadero to Embarcadero	1300
	E2A	Morro Creek MT Crossing	600
	E2B	Morro Creek Bridge Crossing	600
	E3	Embarcadero	3800
	E4A	Embarcadero	1400
	E4B	Embarcadero-Front-Harbor-Market	1351
	E5	Pacific to Quintana	4128
West	W1A	WWTP -Lila Keiser Park	1060
	W1B	Atascadero-Park Rd-Lila Keiser Park (not feasible)	900
	W1C	Atascadero-Bike Path-Lila Keiser Park (not feasible)	1300
	W1D	Atascadero-Caltrans Encroachment-Utility Bridge	2302
	W2A.1	Lila Keiser Park-Morro Creek HDD Creek Crossing	1598
	W2A.2	Lila Keiser Park-Morro Creek Utility Bridge Crossing	1598
	W2B	Parallel Creek & Bridge Crossing	1710
	W3A	Replace Bike Path; 12" W unmodified	1390
	W3B	Offset Bike Path & New PE	1390
	W4	Main St Quintana Paved	4120
	W5A	Roundabout Open Cut (not feasible)	602
	W5B	Roundabout HDD (trenchless, not feasible)	600
	W5C	Roundabout B&J (trenchless)	598
	W5D	Roundabout MT (trenchless)	592
Quintana (common)	Q1	Upper Quintana	1370
	Q2A	Upper Quintana	2400
	Q2B	Upper Quintana Reuse SSFM	2400
	Q3A.1	South Bay Hwy-1 Open Cut	2250
	Q3A.2	South Bay Hwy-1 HDD	2250
	Q3B	Hwy-1 HDD Teresa	1420
	Q3C	Hwy-1 HDD Behind Teresa	1830

	Segment	Quick Description	Length (ft)
Embarcadero	E1	**Atascadero to Embarcadero**	1300
	E2A	Morro Creek MT Xing	600
	E2B	**Morro Creek Bridge Xing**	600
	E3	**Embarcadero**	3800
	E4A	Embarcadero	1400
	E4B	**Embarcadero-Front-Harbor-Market**	1351
	E5	**Pacific to Quintana**	4128
West	W1A	WWTP -LilaKeiserPark	1060
	W1B	Atascadero-ParkRd-LilaKeiserPark (not feasible)	900
	W1C	Atascadero-BikePath-LilaKeiserPark (not feasible)	1300
	W1D	**Atascadero-Caltrans Encroachment-Utility Bridge**	2302
	W2A.1	LilaKeiserPark-Morro Creek HDD Crk Xing	1598
	W2A.2	LilaKeiserPark-Morro Creek Utility Bridge Xing	1598
	W2B	Parallel Creek & Bridge Xing	1710
	W3A	**Replace Bike Path; 12" W unmodified**	1390
	W3B	Offset Bike Path & New PE	1390
	W4	**Main St Quintana Paved**	4120
	W5A	Roundabout Open Cut (not feasible)	602
	W5B	Roundabout HDD (trenchless, not feasible)	600
	W5C	Roundabout B&J (trenchless)	598
	W5D	**Roundabout MT (trenchless)**	592
Quintana (common)	Q1	**Upper Quintana**	1370
	Q2A	**Upper Quintana**	2400
	Q2B	Upper Quintana Reuse SSFM (not feasible)	2400
	Q3A.1	**South Bay Hwy-1 Open Cut**	2250
	Q3A.2	South Bay Hwy-1 HDD	2250
	Q3B	Hwy-1 HDD Teresa	1420
	Q3C	Hwy-1 HDD Behind Teresa	1830

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WRF Offsite Pipelines Alignment Alternatives

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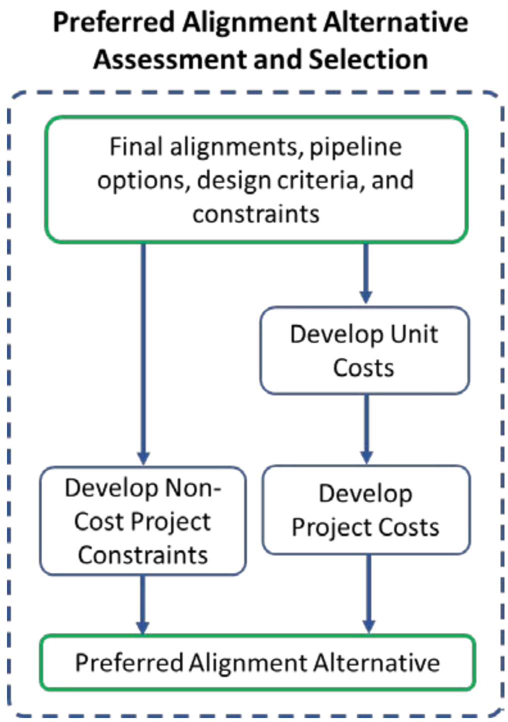
4.4.3 Preferred Alignment Alternative Assessment and Selection

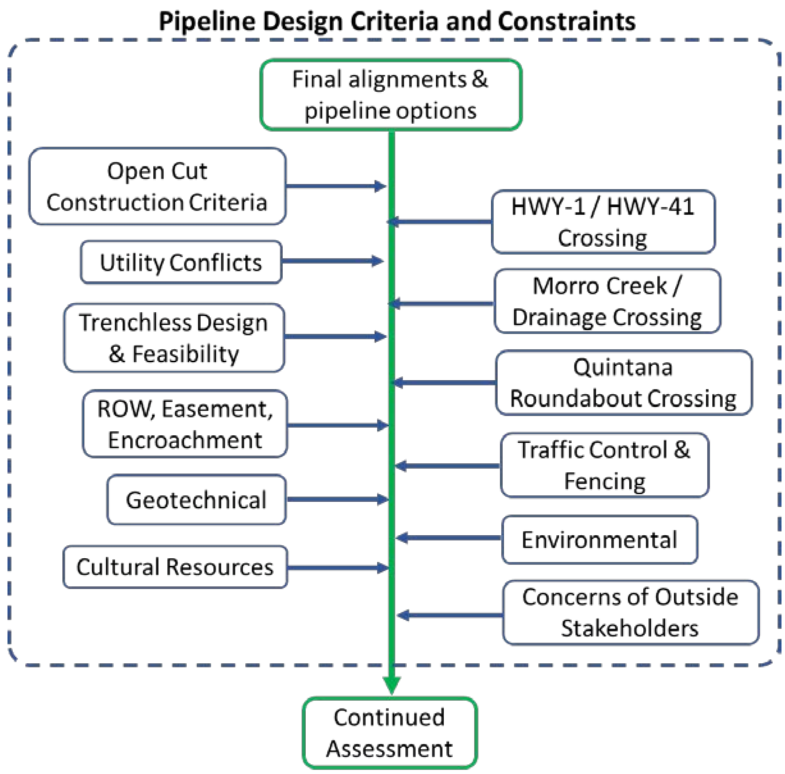
After disqualifying several alignment and pipeline options due to aforementioned fatal flaws, WaterWorks took the final alignment alternatives and pipeline options and continued to assess them by estimating and comparing project costs and non-cost constraints. This process is visualized in the diagram below.



Several other non-forcemain project components were inserted into the assessment process at this point, which include outfall connection/improvements and tie-ins to various pump stations.

4.4.4 Unit Costs (Direct Construction Costs)

WaterWorks utilized the previously identified pipeline design criteria and constraints to develop unit-based costs that inform the direct project costs (construction bid costs) which are visualized in the diagram below.



A list of the preliminary unit costs that were developed along with descriptions are listed in Table 4-25 below.

Table 4-25: Offsite Pipelines Unit Costs

Item	Unit Type	Unit Cost	Description
Mob / Demob	LS	$100,000	Mobilization and demobilization costs for offsite pipeline contractor
SWPPP	LF	$20	Application of linear SWPPP BMPs at various locations along the alignments that will likely require them. Assumed to be required on at least of a quarter of alignments on paved roads.
Tree removal	EA	$500	Typical cost of removing a tree (does not include replacement if that is required per tree removal permit)
Cultural Constraints	Days	$320	Cost to have a cultural monitor onsite during excavation activities
Biological Constraints	LF	$10	Cost of additional linear SWPPP BMPs that front creeks, wetlands, and drainage features.
12" DR18 FPVC	LF	$30	Various carrier pipe materials and sizes for the different SSFMs, Brine, or IPR lines pipeline options.
16" DR18 FPVC	LF	$51	
14" DR13.5 Fused HDPE	LF	$41	
20" DR13.5 Fused HDPE	LF	$79	

22" DR13.5 Fused HDPE	LF	$95	
20" DR1 8 FPVC	LF	$78	
8" DR13.5 Fused HDPE	LF	$19	
8" DR18 FPVC	LF	$21	
24" DR13.5 Fused HDPE	LF	$112	
24" WSP (casing)	LF	$330	
36" WSP (casing)	LF	$330	
16" DR17 Fused HDPE (Open Cut Casing)	LF	$43	
18" DR17 Fused HDPE (Open Cut Casing)	LF	$53	
22" DR17 Fused HDPE (Open Cut Casing)	LF	$77	
Trench 4 - Shallow	LF	$262	Cost of excavation, bedding and backfill, compaction, shoring, and displaced soils removal for a shallow trench that fits the numbers of pipes with an approximate depth of 6 feet. This is the default depth for pipe installation.
Trench 3 - Shallow	LF	$196	
Trench 2 - Shallow	LF	$138	
Trench 1 - Shallow	LF	$100	
Trench 4 - Deep	LF	$349	Cost of excavation, bedding and backfill, compaction, shoring, and displaced soils removal for a deep trench that fits the numbers of pipes with an approximate depth of 10 feet. This is assumed to be required in high utility density corridors with many crossings.
Trench 3 - Deep	LF	$249	
Trench 2 - Deep	LF	$175	
Trench 1 - Deep	LF	$122	
Hydroseeding/Landscape	LF	$2.00	Linear cost to hydroseed and replant any landscaping that is disturbed along the alignment (not used in paved areas).
Dewatering	LF	$50	Extra trench dewatering that may be required due to high groundwater (tidal influenced)
CARV Setup 4	EA	$18,000	Typical costs for combo air relief valve stations given various parallel pipelines.
CARV Setup 3	EA	$13,000	
CARV Setup 2	EA	$9,000	
CARV Setup 1	EA	$5,000	
New Pipeline Cleaning, Inspection and Testing	LF	$10	Linear cost of pipeline cleaning, inspection, and testing.
Pavement Restoration - Bike Path	LF	$105	Linear cost to replace the bike path
Pavement Restoration 4	LF	$131	Cost to replace pavement (T-trench, 2’ beyond edges of assumed trench excavation) given various pipeline options. It is assumed that the pavement thickness is 3”AC over 8”AB.
Pavement Restoration 3	LF	$96	
Pavement Restoration 2	LF	$72	
Pavement Restoration 1	LF	$54	
Surface Restoration 4	LF	$35	Trench restoration costs if there is no overlying pavement. It is assumed that 6” AB will be installed which will produce a drivable surface for maintenance activities.
Surface Restoration 3	LF	$25	
Surface Restoration 2	LF	$17	
Surface Restoration 1	LF	$12	

TCP 1 (advanced)	LF	$50	Assumes that flaggers, active signage, barriers, and passive signage will be required to construct within the alignment.
TCP 2 (intermediate)	LF	$30	Assumes that active signage, barriers, and passive signage will be required to construct within the alignment.
TCP 3 (reduced)	LF	$20	Assumes that barriers, and passive signage will be required to construct within the alignment.
TCP 4 (none)	LF	$5	Assumes that the area can be closed off to pedestrians and vehicular traffic
Utility Transverse Crossing	EA	$250	Additional cost to cross a utility on the alignment
Utility Transverse Crossing + Saddle	EA	$2,000	Additional cost to cross a utility in close vertical separation that would require a formal concrete saddle.
HDD Trenchless Construction	LF	$450	The cost to install each pipeline via HDD trenchless construction
B&J Trenchless Roundabout	LS	$805,000	Cost to bore and jack 400 feet under the Quintana/MB roundabout
MT Trenchless Roundabout	LS	$1,200,000	Cost to microtunnel 400 feet under the Quintana/MB roundabout
MT Trenchless Morro Creek	LS	$825,000	The cost to microtunnel 200 feet under Morro Creek
Pipe Bridge Embarcadero-MorroCreek (130 LF)	EA	$400,000	Cost of steel H-style utility bridge (similar design to existing bridges across Morro Creek) for the Embarcadero alignment.
Pipe Bridge Bike Path-Morro Creek (80 LF)	EA	$300,000	Cost of steel H-style utility bridge (similar design to existing bridges across Morro Creek) for the West alignment.
LS-3 Modifications and Tie-In	LS	$50,000	The cost of making mechanical modifications to LS-3 at the foot of Quintana to tie it into the forcemains
Single FM Midway Pigging Station	EA	$125,000	The cost of a subgrade single FM pigging receiving station at an intermediate location
Dual FM Midway Pigging Station	EA	$150,000	The cost of a subgrade dual FM pigging receiving station at an intermediate location
Outfall Modifications	LS	$100,000	Cost of improvements to connect to the existing outfall

4.4.5 Indirect Project Costs

In addition to the direct construction costs (estimated contractor's bid costs), WaterWorks estimated the indirect construction costs associated with the particular-alignment and pipeline options that would impact the City during the design phase. This is highlighted in Table 4-26.

Table 4-26: Unit Based Indirect Project Costs

Item	Indirect Unit Cost
Permanent Easement (PE)	$1.00/SF
Temporary Construction Easement (TCE)	$0.25/SF
Procurement Cost (coordination with each owner)	$5000/owner
Caltrans encroachment permit fee and plan check review fee	$100,000 (5% to 15% of cost of improvements in Caltrans ROW)
USACE/RWQCB/CDFW 404-401-1602 Permit	$100,000/crossing

PE/TCE/Procurement costs are listed by alternative segment and APN in the following Table 4-27.

Table 4-27: Easement Costs by Alternative Segment

Permanent Easement and Temporary Construction Easement Procurement															
Alternative	APN	Property Owner	Permanent Easement (PE)					Temporary Construction Easement (TCE)					Total Easement(s) Cost	Procurement Costs	Total Cost
			Length (LF)	Width (ft)	Area (SF)	$/SF	Cost	Length	Width (ft)	Area (SF)	$/SF	Cost			
W1D	066-331-040	Dynegy	200	20	4000	$ 1.00	$ 9,000	798	50	43,900	$ 0.25	$ 10,975	$ 19,975	$ 5,000	$ 24,975
W2A.1	066-331-040	Dynegy	1598	20	31960	$ 1.00	$ 31,960	798	50	43,900	$ 0.25	$ 10,975	$ 42,935	$ 5,000	$ 47,935
W2A.2	066-331-040	Dynegy	1048	20	20960	$ 1.00	$ 20,960	1520	20	30,400	$ 0.25	$ 7,600	$ 28,560	$ 5,000	$ 33,560
W2B	066-331-040	Dynegy	1160	20	23200	$ 1.00	$ 23,200	1710	20	34,200	$ 0.25	$ 8,550	$ 31,750	$ 5,000	$ 36,750
W3A	066-331-040	Dynegy	*renegotiate existing 20'W PE*				$ 5,000	250	50	12,500	$ 0.25	$ 3,125	$ 8,125	$ -	$ 8,125
W3A	066-331-036	PG&E					$ 5,000	1140	50	57,000	$ 0.25	$ 14,250	$ 19,250	$ 5,000	$ 24,250
W3B	066-331-040	Dynegy	250	20	5000	$ 1.00	$ 5,000	250	30	7,500	$ 0.25	$ 1,875	$ 6,875	$ -	$ 6,875
W3B	066-331-036	PG&E	1140	20	22800	$ 1.00	$ 22,800	1140	30	34,200	$ 0.25	$ 8,550	$ 31,350	$ 5,000	$ 36,350
E3	066-461-002	Dynegy	480	10	4800	$ 1.00	$ 4,800						$ 4,800	$ 5,000	$ 9,800
E5	066-084-037	Domenghini		0	5000	$ -	$ 700,000						$ 700,000	$ 50,000	$ 750,000
Q3B	068-411-002	Mortuary	100	20	2000	$ 1.00	$ 2,000	100	20	2,000	$ 0.25	$ 500	$ 2,500	$ 5,000	$ 7,500
Q3C	068-411-002	Mortuary	100	20	2000	$ 1.00	$ 2,000	100	20	2,000	$ 0.25	$ 500	$ 2,500	$ 5,000	$ 7,500
Q3C	068-411-017	Seashell Comm. /	823	20	16460	$ 1.00	$ 16,460	823	20	16,460	$ 0.25	$ 4,115	$ 20,575	$ 5,000	$ 25,575
Q3C	068-411-007	Shepard	360	20	7200	$ 1.00	$ 7,200	360	20	7,200	$ 0.25	$ 1,800	$ 9,000	$ 5,000	$ 14,000
IPR West	068-411-007	Dynegy	650	10	6500	$ 1.00	$ 6,500	650	20	13,000	$ 0.25	$ 3,250	$ 9,750	$ -	$ 9,750
IPR East	068-371-007	Jennings	400	10	4000	$ 1.00	$ 4,000	400	20	8,000	$ 0.25	$ 2,000	$ 6,000	$ 5,000	$ 11,000
12"SSFM	066-331-036	PG&E	782	10	7820	$ 1.00	$ 7,820	782	20	15,640	$ 0.25	$ 3,910	$ 11,730	$ -	$ 11,730
12"SSFM	066-331-040	Dynegy	497	10	4970	$ 1.00	$ 4,970	1128	20	22,560	$ 0.25	$ 5,640	$ 10,610	$ -	$ 10,610
B&J or MT	066-280-013	UHAUL	150	20	3000	$ 1.00	$ 3,000	150	60	9,000	$ 1.25	$ 11,250	$ 14,250	$ 5,000	$ 19,250

Additional environmental permitting costs are incorporated in Table 4-28.

Table 4-28: Environmental Permitting Costs

Permitting Costs			
Agency	Permit	Standard Review Time	Procurement Costs
USACE/RWQCB/CDFW	404/401/1602	6-8 months (expedited)	$ 100,000

4.4.6 Project Costs

The total project construction costs were calculated based on total project indirect costs (costs that the City will see) and total project direct costs (estimated bid costs from the contractor) and are summarized in Table 4-29 and are also listed in APPENDIX E: Preferred Alternative Alignment Offsite Pipelines Costs.

Table 4-29: Offsite Pipelines Total Project Costs

Total Offsite Pipelines Project Costs*					
Alignment	Pipeline Option	No IPR + Conduit	West IPR	East IPR	Communication Conduit
West	12"FM + 16"FM + 16"Brine DR 18 FPVC	$12,614,700	$14,814,700	$15,784,700	$ 414,700
	14"FM + 20"FM + 20"Brine DR 13.5 HDPE	$13,874,700	$16,184,700	$17,147,700	$ 414,700
	20"FM + 16"Brine DR 18 FPVC	$11,224,700	$13,064,700	$13,974,700	$ 414,700
	22"FM + 20"Brine DR13.5 HDPE	$12,024,700	$14,064,700	$14,864,700	$ 414,700
Embarcadero	12"FM + 16"FM + 16"Brine DR 18 FPVC	$14,124,000	$17,024,000	$17,874,000	$494,000
	14"FM + 20"FM + 20"Brine DR 13.5 HDPE	$15,594,000	$18,624,000	$19,735,000	$494,000
	20"FM + 16"Brine DR 18 FPVC	$12,304,000	$14,744,000	$15,704,000	$494,000

**Reflects 20% construction & 10% design contingency applied to direct construction costs*

4.4.7 Non-Cost Project Impacts

Single vs Dual Forcemains

The selection of the preferred alternative is not only driven by project cost but by non-cost constraints that were previously identified.

The first non-cost considerations to analyze is utilizing dual vs single forcemains. Utilizing a single forcemain would reduce costs compared to a dual forcemain and is preferable from considering cost alone. When considering the significant operational risk associated with a single FM, however, the dual FM option becomes preferable. These are further discussed in Table 4-30 below.

Table 4-30: Forcemain Options Non-Cost Project Constraints

Forcemain Options Non-Cost Project Constraints			
Non-Cost Constraint	**Single**	**Dual**	**Discussion**
Redundancy	-1	+1	No redundancy with single FM. Pipeline runs through seismic liquefaction prone areas resulting in elevated risk that can be partially mitigated by pipeline redundancy. Construction access to perform repairs in an emergency scenario are extremely difficult due to high parallel utility density. Single FM requires by-pass lines during prolonged (>4 hours) repair activities.
Maintenance & Reliability	-1	+1	Single FM results in consistently low velocities (lower than industry recommended values), which would contribute to significant solids buildup and necessitate an elevated frequency proactive maintenance program, including pigging. Pigging operations for a single large FM would be done as a multi-pass operation (smaller to larger diameter pigs). This requires significant time per pig (4-8 hours) due to the volume of single FM and required flow velocities to move the pig, whereas the dual forcemain allows for pigging at controlled velocities on the inactive line. For single FM, two additional midway pigging stations (pig catching facilities) are recommended to provide maximum flexibility during maintenance procedures. Single FM relies on high frequency inspection and maintenance of surge/air relief valves to ensure continuous successful operation of FM.
Odor Production	-1	+1	Single FM long residence times and solids buildup/settling promote gas production and elevated solubilized H2S, which will increase odor release at air relief valves resulting in the need for enhanced odor control measures along the alignment and increased odor production at the WRF.

In summary, dual sewer forcemains provide the best combination of hydraulics for pumping, O&M flexibility, and redundancy. Consequently, WaterWorks has selected the dual forcemain option as the preferred alternative.

West vs Embarcadero Alignment

The West alignment is the most cost competitive of the two final alignments, but the Embarcadero alignment has fewer non-cost constraints and impacts as summarized in below.

Table 4-31: Final Alignments Non-Cost Project Constraints

Final Alignments Non-Cost Project Constraints			
Non-Cost Constraint	**West**	**Embar-cadero**	**Discussion**
Accessibility/O&M	+1	+1	The Embarcadero alignment is accessible via vehicles in paved areas and maximizes use of existing City ROW

Constructability / Traffic Impacts	0	+1	Heavy pedestrian/vehicular traffic areas (Atascadero school traffic, Quintana commercial corridor, Embarcadero waterfront) are mostly avoided via the Embarcadero alignment
Traffic / Public / Commercial Impacts	-1	-1	
Environmental / Schedule Risks	0	+1	The Embarcadero alignment reduces environmental impacts by maximizing use of existing City ROW and paved roads. The West alignment would impact more trees and may require 401/404/1602 permits. The Embarcadero alignment does pass through a cultural resource area, however, and would likely require cultural resource monitoring and mitigation. There is some risk that this would impact the overall project schedule.
Utility Coordination	-1	+1	The Embarcadero alignment generally avoids high utility density areas and may reduce (but not eliminate) DDW water/sewer separation waivers.
Right-of-Way Acquisition	0	+1	The Embarcadero alignment would require the purchase of a single-family home at the top of Pacific. This is anticipated to be a sensitive issue and could affect the overall project schedule. The West alignment would require easement negotiation with Dynegy and PGE, but the City has already successfully negotiated with these utility agencies on past projects.
Geotechnical	+1	-1	The Embarcadero alignment runs through less favorable geotechnical conditions due its proximity to Morro Bay/Pacific Ocean, which increase its exposure to tsunami-inundation, corrosive soils, and seismic liquefaction. Dewatering is anticipated to be a significant constraint due to tidal-influenced groundwater.
Key stakeholder coordination	-1	+1	The Embarcadero alignment reduces or eliminates permitting with Caltrans, Dynegy, PG&E, and DDW.
Dual Pump Station Integration	+1	-1	Use of the West alignment leverages more flow diversions which in turn better balances flows in a dual pump station scenario. This results in more economical pump sizing, reduced long term O&M, and a smaller PS-A which is advantageous given the seismic, flooding, and tsunami inundation constraints in the Atascadero/Embarcadero area.
Cultural Resources	+1	-1	

From the analysis presented above, the Embarcadero alignment generally has fewer constraints than the West Alignment but is $2.0 million more expensive from pipeline related costs (if including the cost of the East IPR). In addition, use of the Embarcadero alignment cannot leverage diverting local flows into a secondary booster pump station. The West alignment can leverage local flow diversions, however, and consequently is significantly less expensive if much smaller dual pump stations are used. This is discussed in more detail in the pump station assessment and project costs Chapter 5 & 6.

4.4.8 Preferred Alternative Alignment

Based on the assessment presented in the previous sections, WaterWorks recommends the 8"IPR-12"FM-16"FM-16"Brine FPVC and 8"IPR-14"FM-20"FM-20"Brine HDPE West Alignment as the preferred alignment/pipeline option alternative. In addition, based on the analysis presented in Chapter 5 & 6 below, the preferred pump station scenario is the dual PS-A and PS-B pump station scenario (@ Main St). The alignment alternative is broken up into its largest program components in Table 4-32.

Table 4-32: Preferred Alternative Alignment Program Components

Start STA	End STA	Length	Construction Segment
10+00	14+50	450	Outfall Improvements & Tie In
14+50	26+75	1225	Atascadero to Caltrans
14+50			PS-A Tie-In
26+75		1476	East IPR - Hwy-1 Crossing - Hwy-41 Caltrans Encroachment
26+75	33+00	625	SB Hwy-1 Connector Caltrans Encroachment
33+00	36+00	200	Morro Creek Bridge Crossing
36+00	51+00	1500	Bike Path
39+75		500	West IPR
51+00	58+00	700	Bike Path Drainage Crossing + Caltrans Encroachment
51+00		1910	LS-2 12" SSFM
58+00			PS-B Tie-In
58+00	94+00	3600	Upper Quintana (Main St to Roundabout)
94+00	101+00	700	Quintana Roundabout Crossing
101+00			LS-3 Bypassing
101+00	151+00	5000	Lower Quintana (Roundabout to South Bay)
150+00			LS-3 Tie In to forcemains
151+00	161+00	1000	South Bay Blvd and Hwy-1 Crossing

A preliminary plan and profile of this alternative along with the potential East and West IPR lines is displayed in APPENDIX A: 30% Plan & Profile of Preferred Alternative Alignment. An in-depth discussion of construction phasing is presented in Section 7.5.

5 PUMP STATION ALTERNATIVE ASSESSMENT

The pump station alternatives that were assessed for this project include alternative pumping scenarios, as well as alternative sites for the different stations and the accompanying hydraulics. The site improvements and alternatives involved with the pump station site design are also discussed.

5.1 Pump Station Configuration and Location

As described in Chapter 4, the preferred alternatives for the forcemains are the West and Embarcadero alignments. To design the pumping system, pump station configurations were examined to determine the most cost-effective way to pump wastewater through these pipe alignments. These configurations included single and dual pump station scenarios along both the West and Embarcadero alignments. Pump station sites along both alignments were identified which were hydraulically advantageous and practically implementable (in terms of access, land ownership, proximity to the pipe alignment, elevation, etc.) and can be seen in **Figure 5-1**.

Figure 5-1: Pump Station Location Alternatives Along the West and Embarcadero Alignments

5.1.1 Single Pump Station

As described in Chapter 3, flows currently enter the existing WWTP at the influent flume which receives flows from several basins, Lift Stations 1-3 and Cayucos Sanitary District. Any new infrastructure would be sized to exclude Cayucos flows, as Cayucos Sanitary District will be treating its own wastewater moving forward.

The simplest and most straightforward approach would be to size a single pump station (Pump Station A – PS-A) located at or near the existing WWTP to intercept influent nearly all flows (with the exception of those flows going to PS-3) and convey them to the new WRF. Three potential location alternatives for a single pump station were identified and can be seen in **Figure 5-2**. The three site alternatives are described below:

PS-A Site 1: Re-use Existing Influent Pump Station

The existing WWTP has an influent pump station. The existing pump station has three (3) vertical non-clog 25 horsepower (hp) pumps, with a design flow of 2,300 gpm at 31.6’ TDH. It is a possibility that parts of the structure could be reused and/or retrofitted for use as a single station/PS-A but the associated costs do not present significant benefits over those associated with brand new infrastructure. There are issues with flood elevation, aging structures and electrical infrastructure, and accessing the site at such time as the existing WWTP site is redeveloped. After workshop discussion regarding the possibilities of reusing the existing influent pump station, the re-use option was not further pursued as a viable alternative.

Alternative pump station locations were originally identified and vetted in the *2017 Water Reclamation Facility Master Plan* by Black & Veatch. Of those analyzed, two were recommended.

PS-A Site 2: South of Atascadero – City Property

Previously identified PS-A Site 2 is adjacent to the existing WWTP, where the existing parks maintenance shed is currently. This location would require no land acquisition or permitting as the City already owns this property. This site would maximize the opportunity of redeveloping the existing WWTP site, compared to reusing the existing influent PS. Existing grade is approximately 17-18 feet above sea level and has easy access to Atascadero Road. Demolition of existing facilities would be needed prior to the construction of a new pump station but would not affect the operation of the WWTP.

PS-A Site 3: North of Atascadero

PS-A Site 3 was previously identified as a recommended site due to its proximity to the existing WWTP influent confluence. The site is adjacent to Atascadero Road which lends to using right-of-way for access. The site is at approximately 15-19 feet above sea level, similar to PS-A Site 2 which makes the alternatives hydraulically equivalent. Construction at this location will not impact the existing WWTP. There are no existing buildings to demolish, although there is a high likelihood that some utilities will need relocation – i.e. desalination line, water line. The locations for existing utilities would be confirmed via potholing during design. To minimize the potential for additional property acquisition, the pump station site would need to be long and narrow, which may not be conducive to operations and maintenance.

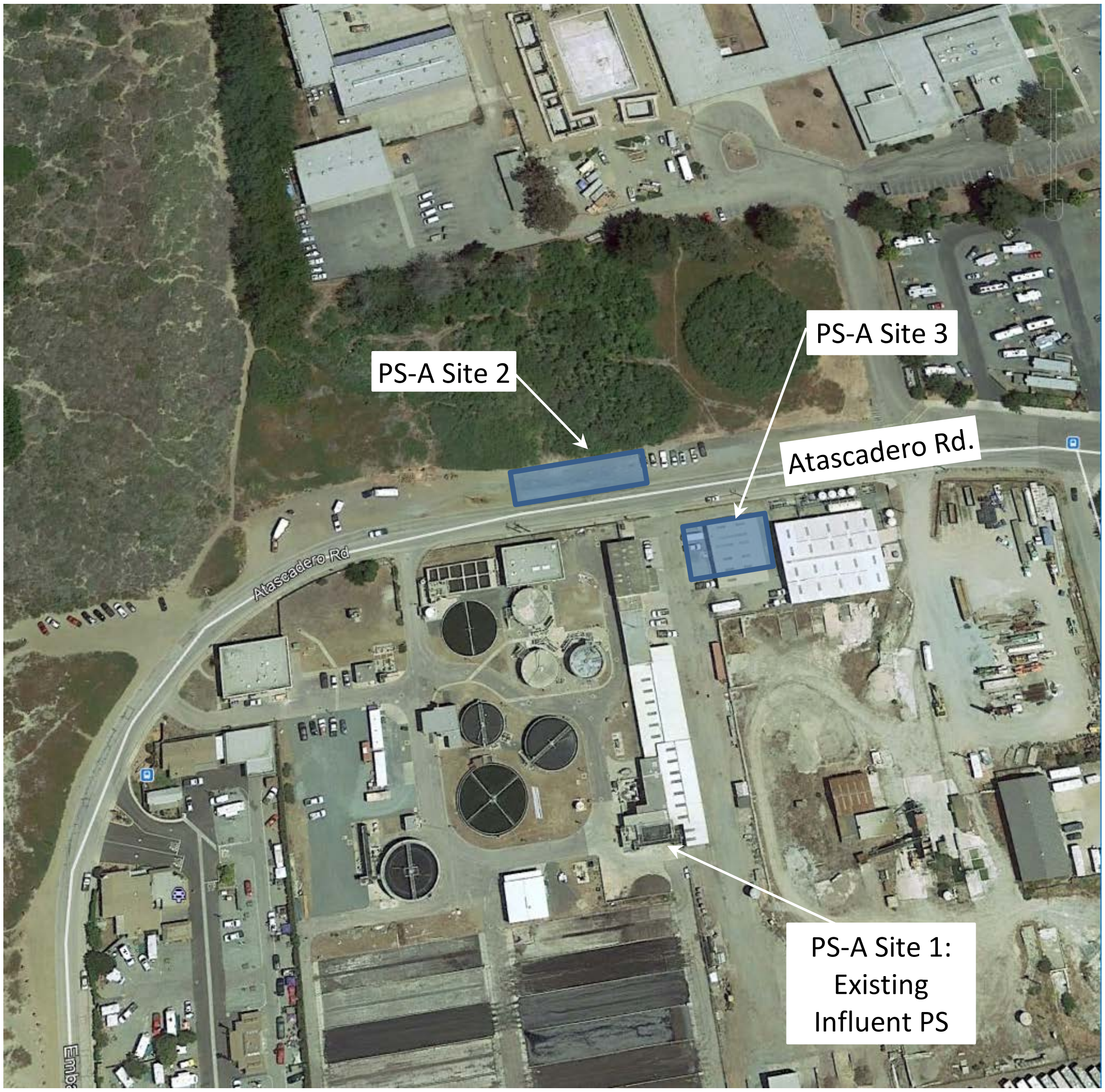


Figure 5-2: WWTP Pump Station Site Alternatives

5.1.2 Dual Pump Station – West Alignment

A dual pump station configuration is intended to help decrease the size and number of pumps required for a single pump station. In a dual station setup, one pump station would intercept flows at the existing WWTP, located at one of the single station sites identified. This would be referred to as Pump Station A (PS-A). A second pump station, Pump Station B (PS-B) would be located such that pump sizing is optimized for both stations.

Two sites were identified as potential PS-B locations along the West Alignment. These sites were chosen due to their parcel availability, as well as hydraulic placement along the alignment.

West Site 1 (W1)– Quintana Road

A pair of vacant lots along Quintana Road are to be viable locations for an intermediate station. The lots are located east of the Couch Potato and Morro Bay Veterinary Clinic. Both parcels have space for the pump station, electrical building, odor control and emergency generator. The sites have good access to utilities and for operation and maintenance vehicles.

Figure 5-3: Quintana Road Pump Station Site Alternatives (W1A & W1B)

W1A is on the north side of Quintana Road, adjacent to the shopping center. The dirt lot is at grade with the road, at approximately 56 feet elevation. Existing water, gas and sewer run along the south side of the lot, parallel to Quintana Road. Trees and shrubs line the northerly and easterly sides of the parcel.

On the south side of Quintana Road (W1B), there is a small elevated lot located adjacent to the parking lot of a commercial strip mall. Access would be from the existing parking lot, away from Quintana Road. Electrical utility is readily available at this site. The lot is 15-20 feet higher than the road. This elevation difference would artificially elevate this point along the West Alignment requiring larger pumps at PS-A and effectively negating a significant amount of the advantage of adding PS-B. For this reason, Site W1B was removed from further consideration.

West Site 2 (W2) – Main Street at Highway 1

Another vacant lot was identified on Main Street near the onramp to Highway 1, next to Lemos Feed & Pet Supply. It currently appears to be used for additional/overflow parking for the store. It has been confirmed that the parcel is currently owned by the City of Morro Bay. Water, telephone, and gas utilities are readily available at the site. This location is highly accessible from Highway 1 and Main Street and is at approximately 32 feet elevation.

Figure 5-4: Main Street Pump Station Site Alternative (W2)

5.1.3 Dual Pump Station – Embarcadero Alignment

Sites identified for PS-B along the Embarcadero alignment were close in elevations to the locations identified on the West Alignment.

Embarcadero Site 1 (E1)– City-Owned Parcel

The City of Morro Bay owns a parcel at the north east corner of the Pacific Street and Market Avenue intersection. The lot is currently used for parking for the nearby businesses. Access to the parcel could be from either Market Avenue or Pacific Street. Electrical, water, and fiber are all readily available at the site. E1 is at an elevation of approximately 32 feet.

Figure 5-5: Market Avenue Pump Station Site Alternative (E1)

Embarcadero Site 2 (E2)– Bank of America Parcel

The existing Bank of America, located at the northwest corner of the intersection of Monterey Avenue and Pacific Street, is slated to close which will make this lot available for purchase. Access could be from either Monterey Avenue or Pacific Street. This E2 location is higher than E1, at approximately 67 feet above sea level. Water, telephone, and gas are all readily available at the site.

Figure 5-6: Monterey Avenue Pump Station Site Alternative (E2)

5.2 Required Site Improvements for Pump Station Alternative Sites

The goal is to design a cost-effective pump station that provides existing and future capacity requirements and ensures reliable and continuous operation. This section addresses geotechnical, mechanical, structural, electrical, aesthetic and impact concerns that need to be considered, addressed and incorporated into design for all the pump station alternative locations and configurations discussed above so that a comparison of alternative pumping arrangements can be made.

5.2.1 Groundwater

During geotechnical investigations conducted by Yeh and Associates in January 2017 (APPENDIX C: Preliminary Geotechnical Report), groundwater was encountered at approximately 6 to 7 feet below existing grate at the existing WWTP. Groundwater is anticipated to fluctuate based on seasonal precipitation and the level of water in Morro Creek. Underground structures such as the wet well, vaults and pipelines should consider the buoyancy potential and be designed assuming groundwater at grade. The location of the pump station(s) as it relates to the 100-year floodplain also warrants this design assumption.

Furthermore, subsidence is to be anticipated during dewatering. The soils found near the WWTP are prone to subsidence and the nature of construction for the lift station would require a substantial amount of dewatering. Therefore, the design and construction sequencing for the pump station(s) should plan for and mitigate subsidence.

5.2.2 Geotechnical Considerations

The overall project site is in a generally seismically active area. At this time, the lift stations and associated structures are expected to have fundamental periods of less than 0.50 seconds. This qualifies the proposed improvements to be classified as Site Class D. Preliminary Seismic Data presented by Yeh and Associates is shown below.

Table 5-1: Preliminary Seismic Data

Seismic Parameter	Value
Latitude, degrees	37.3799
Longitude, degrees	-120.8605
Site Class	D
Earthquake Magnitude	6.7
Peak ground acceleration 2% in 50 years	0.48
S_s, Seismic Factor for Site Class B at 0.2 seconds	1.16
S_1, Seismic Factor for Site Class B at 1 second	0.427
F_a, Site Specific Site Coefficient	1.04
F_v, Site Specific Site Coefficient	1.57
S_{MS}, Site Specific Response Parameter at 0.2 seconds	1.20
S_{M1}, Site Specific Response Parameter at 1 second	0.672
$S_{DS} = 2/3\ S_{MS}$	0.801
$S_{D1} = 2/3\ S_{M1}$	0.448

5.2.3 Soil Improvements

In addition to designing in accordance to the 2016 California Building Code (CBC), additional considerations for liquefaction should be made for a Pump Station A (PS-A), located in the general area of the existing WWTP. The Yeh and Associates geotechnical report, generated in April 2018 identifies that the potential lift station sites near the existing WWTP are prone to liquefaction and seismic settlement. Mitigation measures suggested to reduce the effect of liquefaction include: vibrocompaction/vibroreplacement; permanent shoring cofferdam; deep soil mixing; or deep foundations.

Vibrocompaction/Vibroreplacement

Vibroreplacement was used at the WWTP and involves stone columns that are used to increase the soil bearing capacity of the site prior to construction. This has been used for some of the construction at the existing WWTP.

Permanent Shoring Cofferdam

A permanent cofferdam would surround the lift station and isolate it from the surrounding area during a seismic event. The shoring would help with dewatering but could also help with permanent flood protection.

Deep Soil Mixing

Deep soil mixing would entail altering the existing ground at the lift station site with Portland cement.

Deep Foundations

Constructing foundations such that they extend below the potentially liquefiable layers is an option.

Regardless of which liquefaction mitigation measure is chosen, it is recommended that the life station site be overexcavated prior to the placement of any fill. Some fill is anticipated in raising the site for flood protection reasons. Excavated materials for the pump station may be suitable for compacted fill or trench backfill but not for bedding, pipe zone, base, aggregate, or drainage type materials.

5.2.4 Flood Protection

As shown in Figure 5-7 the location of the single station/PS-A sites are within the 100-year floodplain per the current Flood Insurance Rate Map per the Federal Emergency Management Agency (FEMA). The flood potential in the project area further emphasizes the need for anti-flotation design considerations for any underground structures.

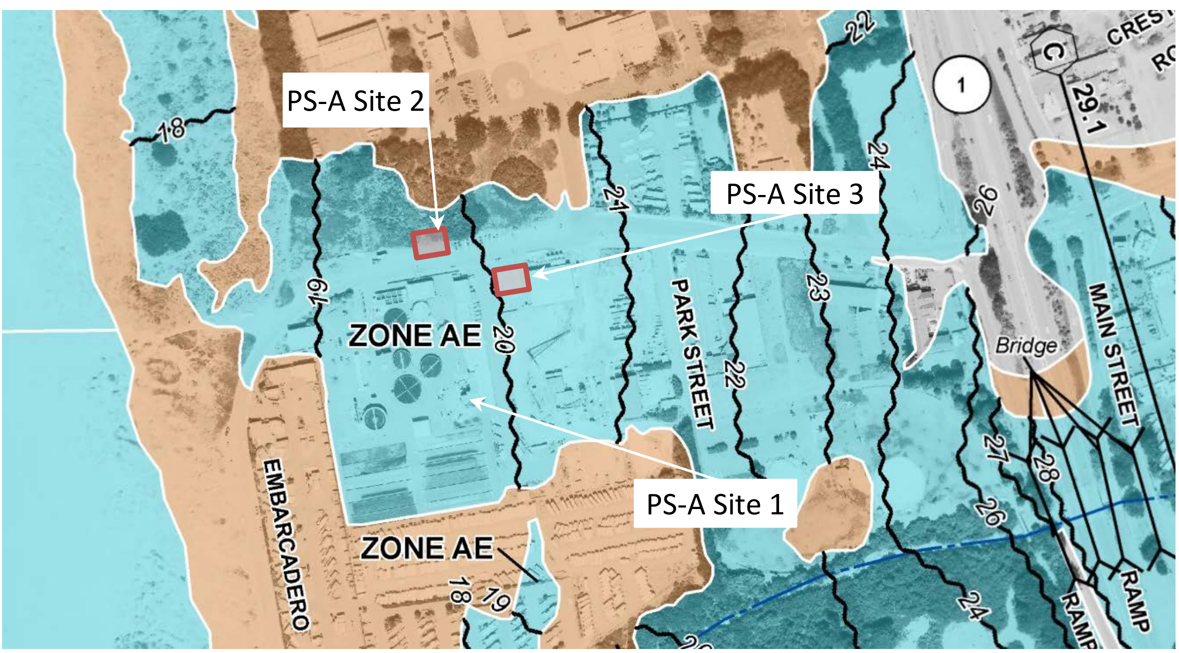


Figure 5-7: FEMA Flood Map

Construction in flood damage prevention areas (i.e. 100-year floodplain) is allowed under the City's and FEMA's regulations as long as compliance with the elevation standards are met - i.e. the facilities impacted by a 100-year flood need to be elevated or protected to two feet above the base flood elevation. This is a standard construction method in the floodplain and will also be employed in the construction of a new lift station near the existing WWTP site.

To protect critical equipment, it was decided that structures and equipment are to be set at a minimum of two feet above the 100-year flood elevation. The minimum site elevation for the two alternative sites for the singe station/PS-A are as follows:

Table 5-2: Single Station/Pump Station A Flood Elevations

Criteria	PS-A Site 2	PS-A Site 3
100-year Flood Elevation (with no tide adjustment)	20.61	19.97
Minimum Top of Concrete Elevation	22.61	21.97
Existing Grade Elevation (Ranges)	17-18	15-19

Mitigation measures to protect the fill used to raise the site from washout and erosion under flood conditions will be implemented. In addition to settlement concerns regarding groundwater pumping and consolidation of soils, settlement of fill used to raise the site will also be accounted for in construction sequencing and backfill design for the site.

5.2.5 Tsunami Inundation

There is potential for tsunami inundation or flooding of the lift station sites according to the ASCE Tsunami Hazard Tool. The single station/PS-A and PS-B locations do fall within the tsunami design zone. PS-B locations are, however located towards the outer edges of the design zone. The pump stations at this time are not planned to be designed to withstand a tsunami and would have to be repaired or replaced following such a large natural disaster. Below grade structures and equipment (wet wells, pumps, valve

Figure 5-8: ASCE Tsunami Hazard Tool

vaults) would be less susceptible to damage. Above grade structures and equipment (electrical buildings, odor control, generators, etc.) would be more exposed to damage. A wall surrounding the sites would help to dampen the effects of the tsunami and reduce damage. Constructing all buildings from concrete masonry blocks will also help to minimize damage, however following a tsunami, repairs of the affected lift stations should be expected.

PS-B West Site 2, along Main Street, does have and existing “drainage, flood control & water conservation easement” along its northwesterly side. If this site is chosen, a wall along this easement would be anticipated to help keep overflow from the drainage ditch off of the pump station site.

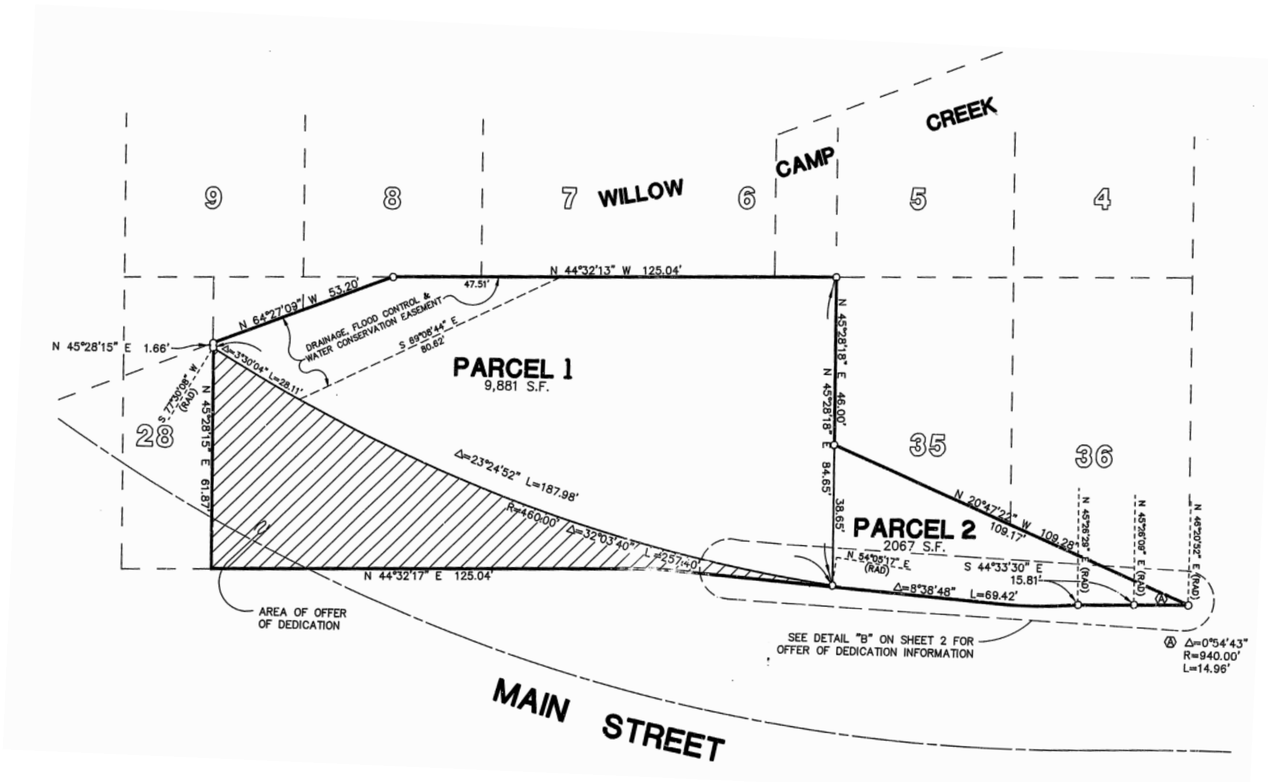


Figure 5-9: Flood Control Easement

5.2.6 Pump Station Configuration and Wet Well Design

Single Pump Station

The approximate 8 mgd of PWWF anticipated for a single pump station configuration, a self-cleaning trench style is recommended. Influent sewage would enter the wet well via ogee ramp, which facilitates ease of cleaning scum and sludge. In this configuration, pump inlets would be located lower than the upstream pipe which allows for more uniform suction. Due to the large range of flows the station would have to accommodate, variable frequency drives (VFDs) will be utilized which can also help reduce the required wet well size and sedimentation. The submersible pumps would be housed in a separate dry pit.

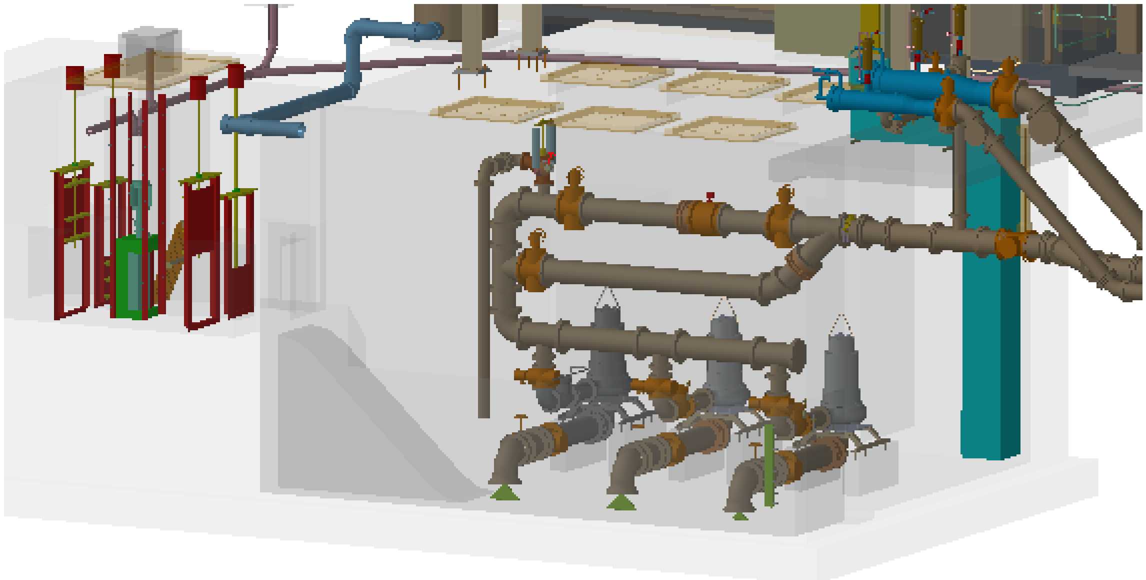


Figure 5-10: Trench-Type Sump, Single Station

Dual Pump Stations

The intent for a dual pump station configuration is to have two smaller stations rather than a single large station. The smaller stations lend themselves to more traditional precast concrete wet well layouts. Typically, these stations would consist of a precast wet well with submersible pumps in it, followed by two precast vaults - one vault for valves and another for a flowmeter and bypass. Lift Station 2 is similar to the planned layout of the two smaller stations, see Figure 5-11.

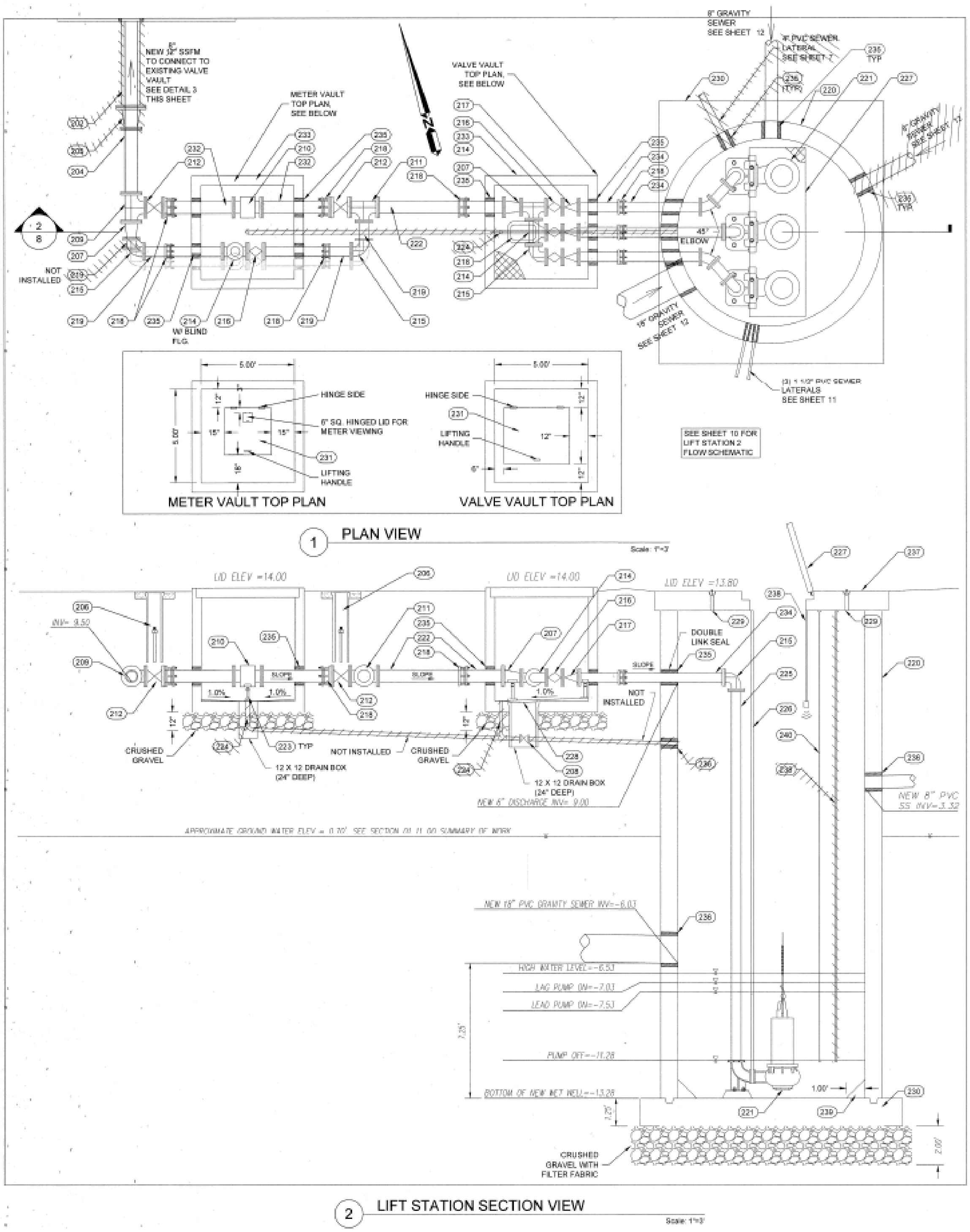


Figure 5-11: Existing Lift Station 2

5.2.7 Security and Access

It would be in the City's best interest to have security measures in place for the pump station site(s). Considerations for access, fencing and alarms would help deter unwanted visitors at these public utility facilities.

Access

The sites will have space for vehicular access and parking. There will be space for a truck to drive to the fuel storage tank for fuel deliveries. Access into the site could require key or keypad entry for vehicle and man-gates.

For flood protection reasons, the single station/PS-A site would require an inclined driveway to get to design grade.

CMU Block Fencing

Fencing for the site will be 8-ft tall CMU block fencing with columns every 100-ft, matching the main block style and color of the building (split face block, light tan color). CMU fencing will be 8x8x16 reinforced CMU with a continuous concrete footing and grout fill in the cells with reinforcing. The post and fence cap block will be matching color to the split face field block. An example of what the fencing will look like is shown in Figure 5-12.

Figure 5-12: Example of Planned CMU Block Fence

Wrought-Iron Fencing

Wrought-iron vehicle and personnel access gates will be provided with automatic openers and keypad entry systems. Vehicle gate will be a 20-ft wide rolling gate to allow for large vehicle access. Gate color will match block wall color (light tan). Gates will be designed for climb prevention and general vehicle security but will not be anti-ram gates or meet any specific anti-ram requirements of ASTM F2656 - 07 *Standard Test Method for Vehicle Crash Testing of Perimeter Barriers*. Gates will be similar to those shown in Figure 5-13.

Figure 5-13: Example of Planned Wrought Iron Access Gates

Alarms

Security systems for the control building will include door alarms and motion detectors inside of the building as well as smoke alarms, all wired to a single alarm panel which will be able to alarm to SCADA as well as dial out to a security provider of the City's choosing.

Additional security measures could include motion-activated site lighting and intrusion alarms on access hatches.

5.2.8 Control Building

At the request of the City, the control building would house the site electrical equipment, motor control center, switchgear, and pump controls as well as a restroom for operation and maintenance staff use. There is a potential to allow the public to use the restroom at the PS-A site.

Structural

The control building will have at least 9 feet high ceilings to provide enough clearance height for the various electrical equipment. Two doors will be installed to open in the direction of egress travel, with the door height being at least 8 feet.

Generally, the building will be a concrete slab foundation, CMU block (either 8x8x16 or 12x8x16 CMU, as required by the structural design) building with a concrete tile roof. CMU block will be completely grout filled. The structure will be designed to meet 2016 California Building Code standards.

Architectural

The architectural considerations for the control building are as shown:

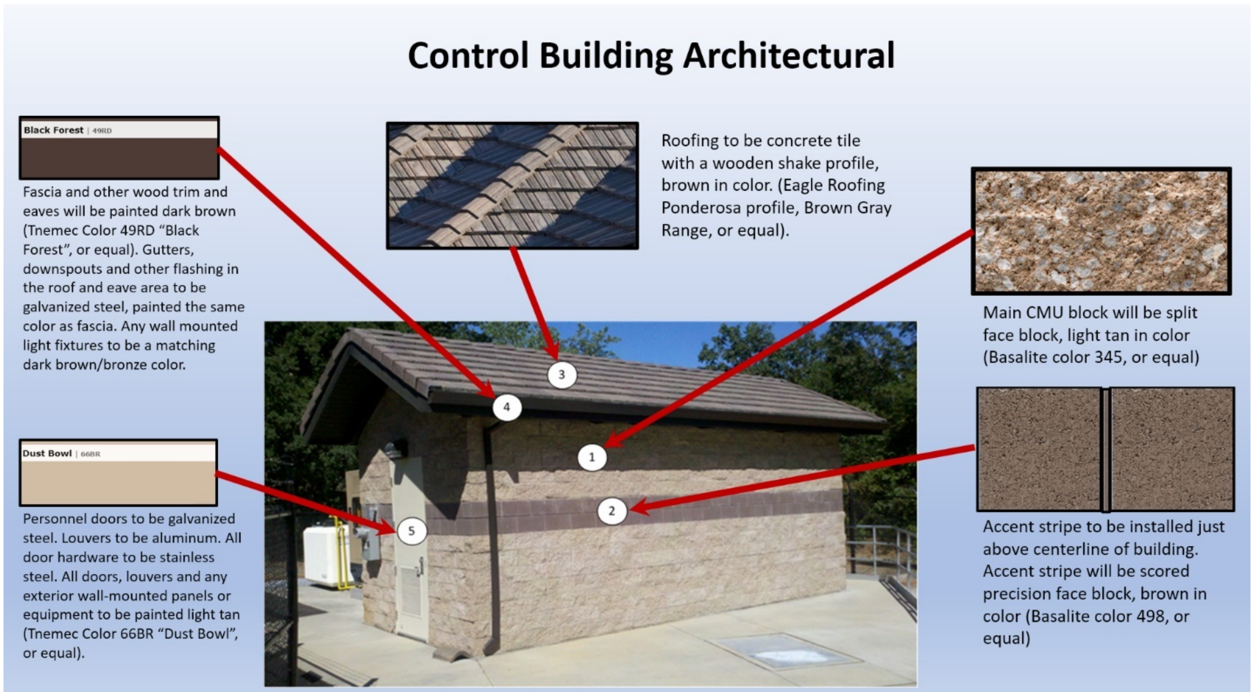


Figure 5-14: Control Building Material and Colors

Generally, split faced block and concrete tile are the primary building materials to be used in the construction of the building. Colors and textures are meant to allow the building to blend into its surroundings. Durability and the longevity of the materials were also considered. The overall color scheme is a beige/brown combination so as to be unobtrusive and easy to maintain. This architectural design is similar Lift Stations 2 and 3.

Electrical

The control building would be equipped with lighting and HVAC. In a dual pump station configuration, fiber optic conduit would be required to connect the two stations. It is understood that a fiber communication conduit is planned to be installed with the offsite pipelines and will connect the new pump stations to the WRF site. In the scenario of two pump stations in which PS-B would operate as a stormwater pump station, the PS-B pumps would have soft starters. In any other scenario, variable frequency drives (VFDs) would be utilized to accommodate the large range of flows and for power efficiency.

5.2.9 Emergency Generator and Storage

An emergency generator and automatic transfer switch will provide power for the entire pump station in the event of a power outage. A minimum of 4 feet of space around the generator should be provided. Given the location and proximity of the pump station site(s) to residences and public areas, the emergency generator(s) should be weatherproof and have sound attenuation.

Natural Gas

The City has expressed interest in potentially using natural gas as the fuel for the emergency backup generator(s). Fuel source reliability as well as the location of the single pump station/PS-A in a floodplain should be further assessed. Preliminary findings indicate that for a single large station, a natural gas emergency generator option is not viable. For the dual pump station scenarios, natural gas generators are more expensive than their diesel counterparts.

Diesel

Diesel is traditional for backup power. Although a diesel generator would be larger, it would not be impacted by flood or the reliability of the natural gas provider. Diesel would require a fuel storage tank will be on-site which will store sufficient fuel for at least 24 hours of continuous operation of the emergency generator under full load.

Table 5-3 briefly summarizes the differences in providing emergency power at the different pump stations as they differ based on the configuration.

Table 5-3: Emergency Generator

	PS-A		PS-B (1000 kW required)
	Scenario 2 (600 kW required)	Scenario 3 (250 kW required)	
Diesel Tier 3 EPA-Certified for Stationary Emergency Applications	• 270" x 74" x 127" • 1038 gallon • 485-600 kW Standby • $280k • Sound enclosure	• 197.2" x 52.7" x 113.9" • 555 gallon • 230-255 kW Standby • $135k • Sound enclosure	• 332" x 103" x 155" • 1749 gallon • 975-1000 kW Standby • $500k • Sound enclosure
Natural Gas EPA-Certified for Stationary and Non-Emergency Applications	• 174.3" x 80" x 96.7" • 675 kW Continuous • $400k • Custom sound enclosure	• 178.2" x 70.3" x 96.9" • 170-260 kW Standby • $150k • Sound enclosure	• 212.1" x 66.5" x 97.1" • 1030 kW Continuous • $750k • Custom sound enclosure

Emergency Storage

At the request of the City, emergency storage that would provide 30 minutes of PWWF will be incorporated into the pump station designs – either incorporated into the sizing of the wet well and/or other onsite storage at the project's design stage. This emergency storage time is intended to allow operations and maintenance staff additional response time in the event of a power outage or malfunction at a station, without surcharging the system.

5.2.10 Odor Control

The potential pump station sites are in areas where odors would be concerning. The existing WWTP is close to the coast and near access to a public beach. PS-B sites are in areas near businesses which have public access/exposure. To mitigate the odors from the lift stations, treatment processes would need to be implemented. In sizing these processes, the ventilation requirements for pumping stations will need to reflect the following:

1. Confined space status should be assumed for the wet well as defined by OSHA. The minimum number of air changes per hour should be six (6). If entry to the wet well is necessary (e.g. for cleaning or maintenance), a portable fan should be used to increase the number of air changes per hour to the minimum requirements established by NFPA 820. The minimum number of continuous air changes for the dry well is six changes per hour.

2. Depth of the air duct in the wet well should be set to avoid entry of wastewater into the duct during periods of high-water level. Duct shall be constructed of fiberglass reinforced plastic (FRP).

3. Ventilation fan should be a centrifugal type with adjustable belt drive.

Acceptable foul air treatment processes are as follows:

Biofilters

Biofilters provide the conditions and substrate to grow both autotrophic and heterotrophic bacteria to grow. A foul air fan conveys foul air from the head space of the wetwell to the biofilter and through the media bed, allowing the bacteria to consume or biodegrade the odorous compounds in the air stream. The primary waste stream from the biofilter is leachate which must be disposed of to the sewer lift station. A supply of water is required to keep the system moist. These systems work well with low to medium air flows and medium to high odor concentrations.

Air Scrubber

Air scrubber systems use a foul air fan to move the foul air out of the wetwell head space and into the scrubber, running counter-current to a water stream which contains treatment chemicals. The water is distributed into the air stream in droplets, using distribution troughs and plastic packing media. As the air runs counter-current to the water droplets, odor compounds are transferred from the gas phase to the liquid phase. The primary waste stream is periodic liquid blow-down. These systems require potable water makeup systems and sometimes water softeners, depending on water quality. These systems work well with larger air flows and more consistent odor loadings.

Activated Carbon

Absorbent systems have absorbent media housed in a non-corrosive vessel. Air can be moved through the media using passive or active means. Activated carbon removes odorous compounds via adsorption onto the carbon media. Once the media is exhausted, it is replaced which can be a labor intensive and expensive process depending on the frequency of media change out. Waste streams include periodic media disposal/replacement and some condensate drainage back to the wetwell. These systems do not require outside utilities, are low capital cost and work well with low to medium (up to 1500 cfm) variable air flows and variable odor loadings.

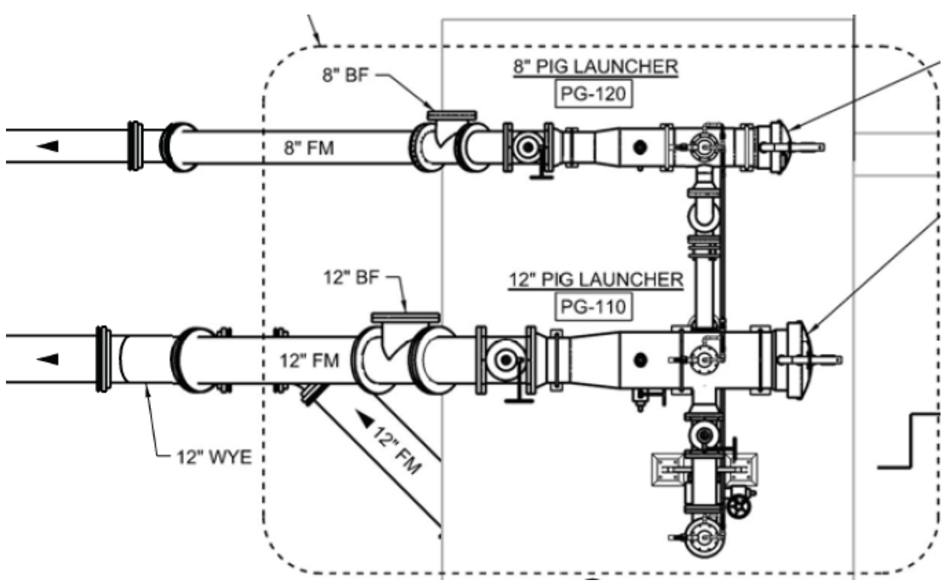
UV/Photoionization

Photoionization is relatively new to the United States but has a three-decade track record in Europe. Photoionization uses ultraviolet (UV) light and a catalyst to breakdown odorous compounds via oxidation. Compounds not oxidized pass into the catalyst (activated carbon based) where they are trapped and broken down by various reactions, including catalysis. This system requires minimal maintenance – mainly catalyst and bulb replacement. The profile is typically compact and has a low lifecycle cost.

In the single lift station scenario, odor control would need to be sized to treat the wet well and dry pit. The wet wells and vaults for the dual pump station scenarios are much smaller. In the dual pump station scenario which has PS-B as a stormwater booster pump station, odor control would not be needed at PS-B as the station would not have wastewater 95% of the time.

5.2.11 Forcemain Maintenance

As mentioned in Chapter 4, pigging is a standard maintenance activity to clean sewer forcemains. It is assumed that all the proposed pump stations will utilize dual pig launching stations (for dual forcemains). It is recommended that the launchers be constructed of stainless steel and be located downstream of pumps and pump station appurtenances, within the pump station site. Figure 5-15 shows an example configuration for dual pig launchers. Additional midway pigging catching facilities may be necessary, which is discussed in Chapter 4.



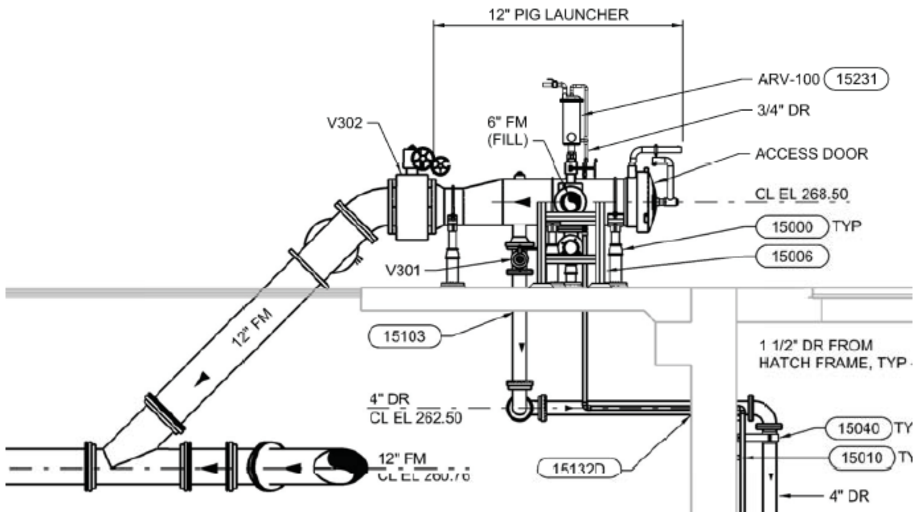


Figure 5-15: Pig Launcher

5.3 Pump Station Hydraulics

The West and Embarcadero alignments were analyzed for viable hydraulics and for the purposes of this analysis, the forcemains are assumed to be 12” and 16” fusible polyvinyl chloride (FPVC). Equivalent diameters for high-density polyethylene pipe would be 14-inch and 20-inch. This discussion of hydraulics assumes the diversion of LS-3 flows to the new 12” SSFM directly. Furthermore, the pumps were assumed to be Xylem/Flygt submersible pumps.

5.3.1 West Alignment

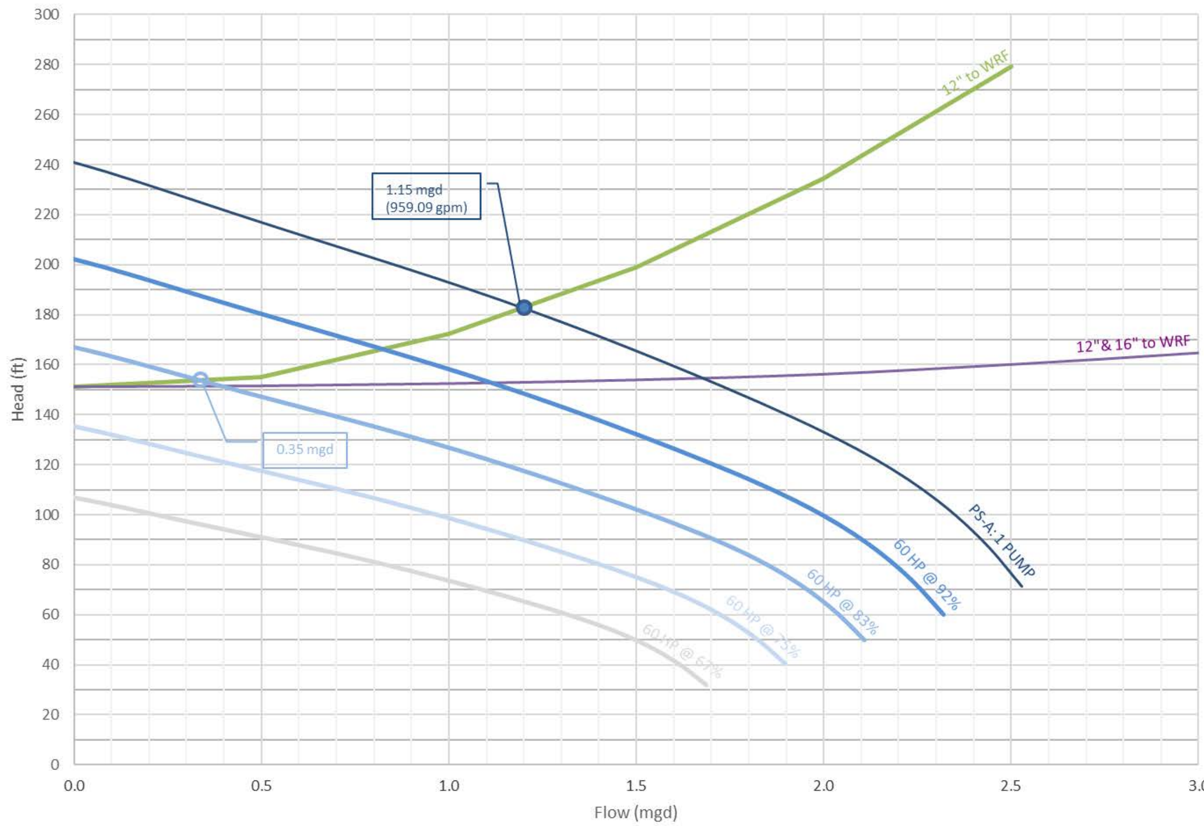
The West Alignment goes easterly on Atascadero Road from the WWTP, parallels the westerly side of Highway 1 to Main Street and along Quintana Road. The static lift from the WWTP to the new WRF is anticipated to be approximately 150 feet.

West Scenario 1 - Single Station

The single pump station location would be near the existing WWTP where existing grade ranges between 15-19 feet. In a single station scenario, pumps would have to be sized to pump 7.98 MGD. This represents the entire system’s flows (minus LS3) projected out to the year 2040. At this flow, a single pump would have to overcome 268 feet of TDH to get flow to the new WRF. Because of the similar locations and elevations, the hydraulics of the alternative single station sites are considered equivalent.

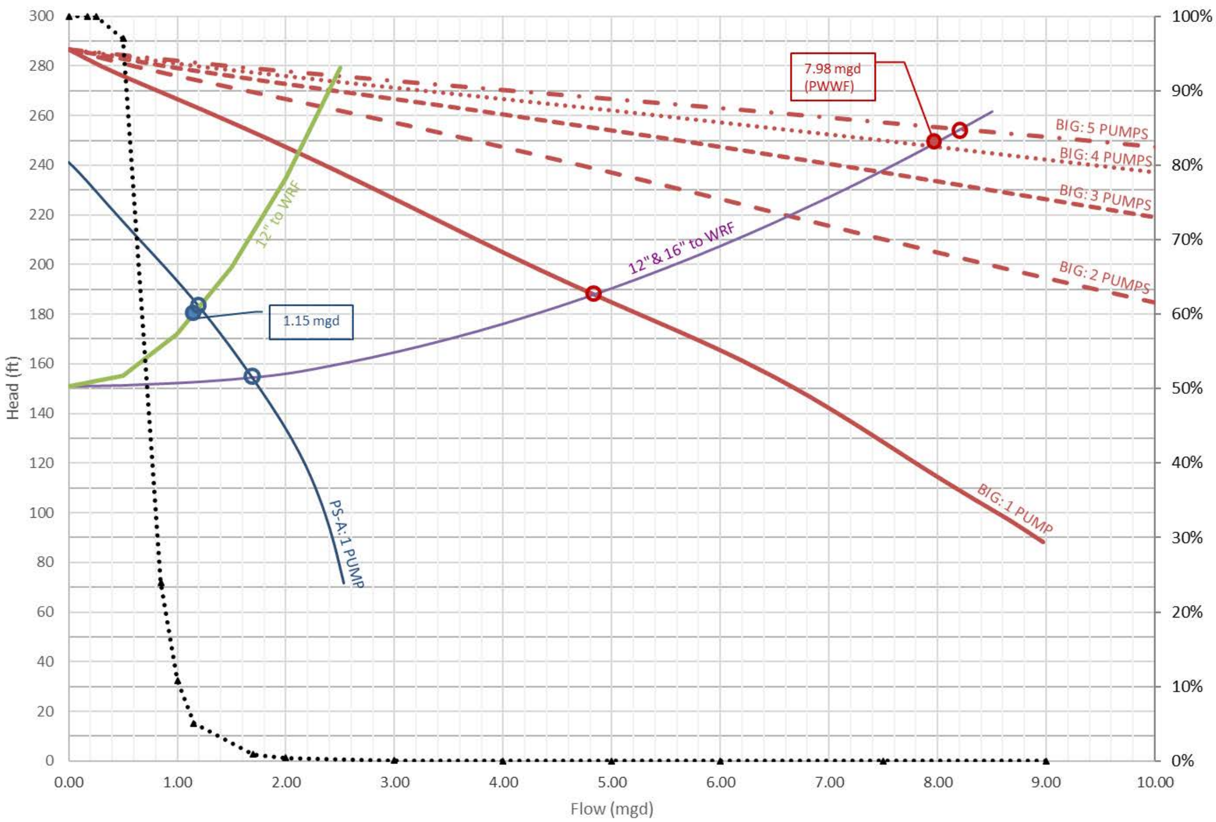
Due to the large range of flows that a single pump station would be designed for, more than one size of pump is necessary. Preliminary sizing indicates flows up to 1.15 MGD, can be pumped by a jockey pump configuration – one duty and one backup (1+1), 60-HP pumps. This assumes the utilization of the 12” forcemain to allow for higher velocities in the pipe. The jockey pump would have a variable frequency drive (VFD) to accommodate lower flows, down to 0.35 MGD as shown in Figure 5-16. From 0.3 MGD to 1.15 MGD, the velocities in the 12” forcemain would range between 06.-2.3 feet per second (ft/s). A minimum velocity of 2.0 ft/s is ideal for resuspension of solids.

Figure 5-16: Jockey Pump for West Scenario 1



For flows larger than 1.15 MGD, a combination of 250-HP pumps will be needed. To pump PWWF of up to 7.98 MGD, a "5 duty + 1 backup" pump configuration of 250-HP will be needed, with the utilization of both 12" and 16" forcemains. In total, a single station would require 8 pumps: 1 jockey, 5 duty and a stand-by pump for each size. Figure 5-17 shows the ranges of flows covered by the different pump and forcemain sizes. A more detailed breakdown of the number of pumps and forcemains required for specific flow ranges are discussed later in Section 5.4.

Figure 5-17: West Scenario 1



Preliminary sizing resulted in a trench-style wet well that is over 68-ft long and over 30-ft deep. **Error! Reference source not found.** shows dimensions based on the *Pumping Station Design (3rd ed. Jones, Sanks, et.al., 2008)* as well as recommendations from Xylem/Flygt pumps. The depth of the wet well considers the 100-year floodplain finished grade elevation and the inverts of existing WWTP influent pipelines. Figure 5-19 is a 3-dimensional representation of the single station wet well and the scaled sizing for an accompanying emergency diesel generator and biological odor control system.

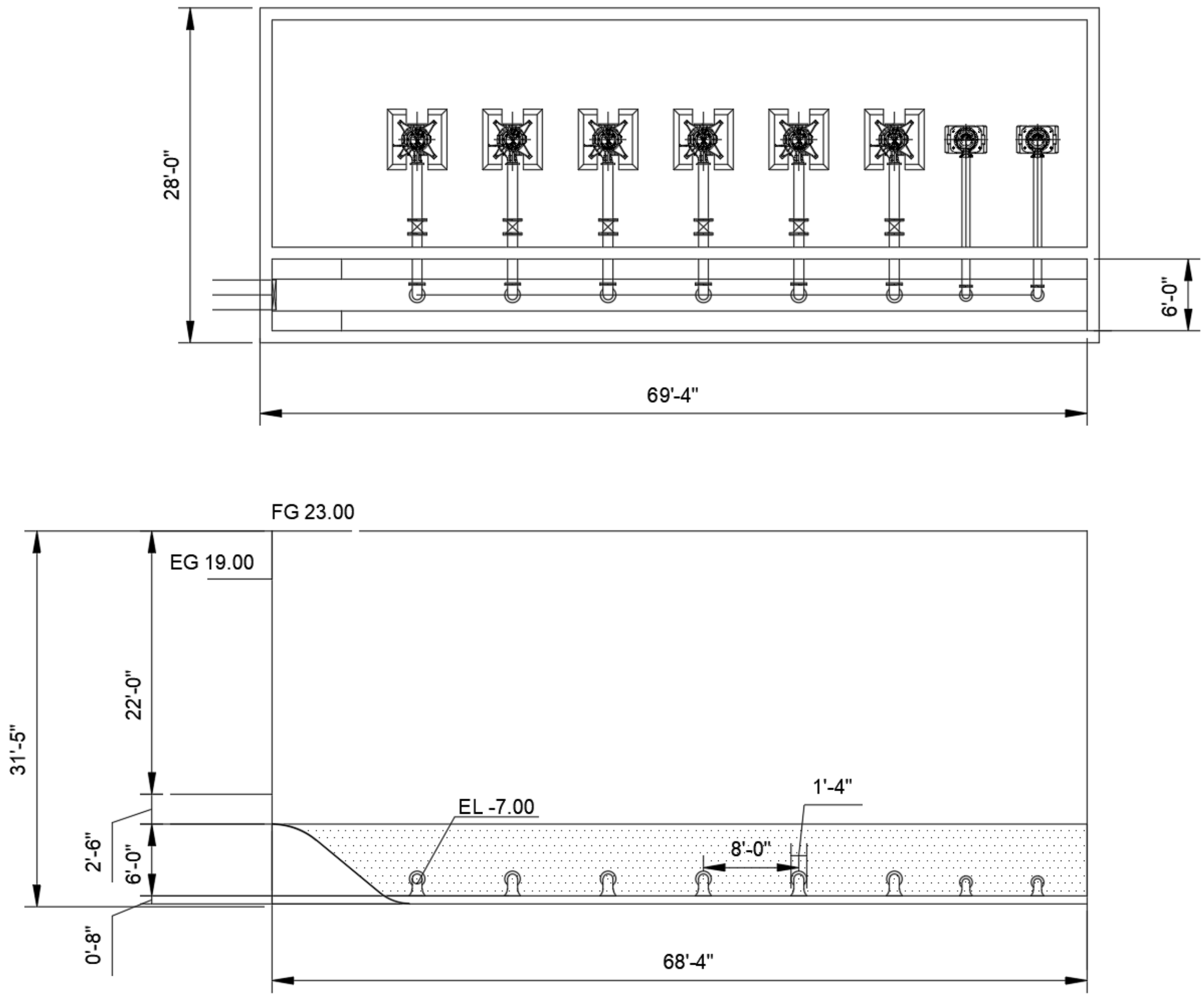


Figure 5-18: Preliminary Scenario 1 Wet Well Sizing

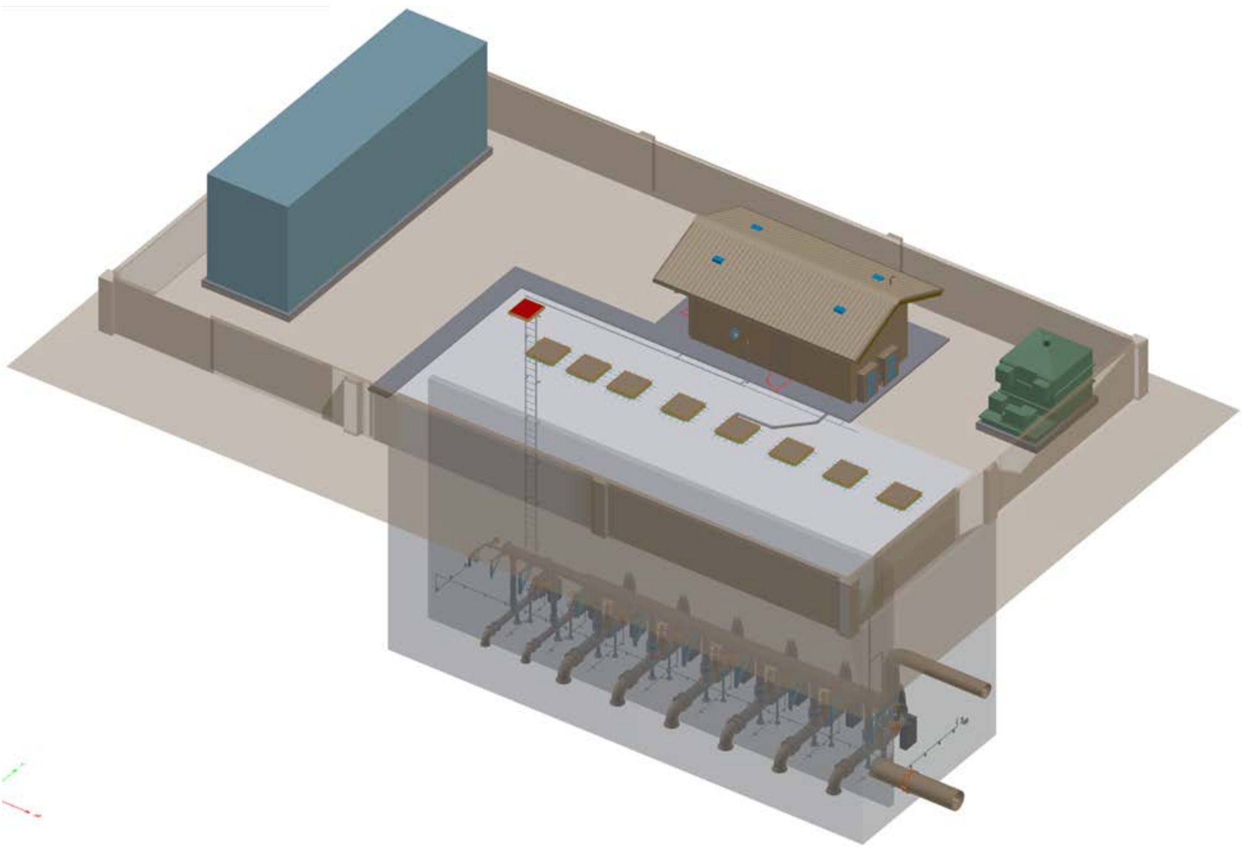
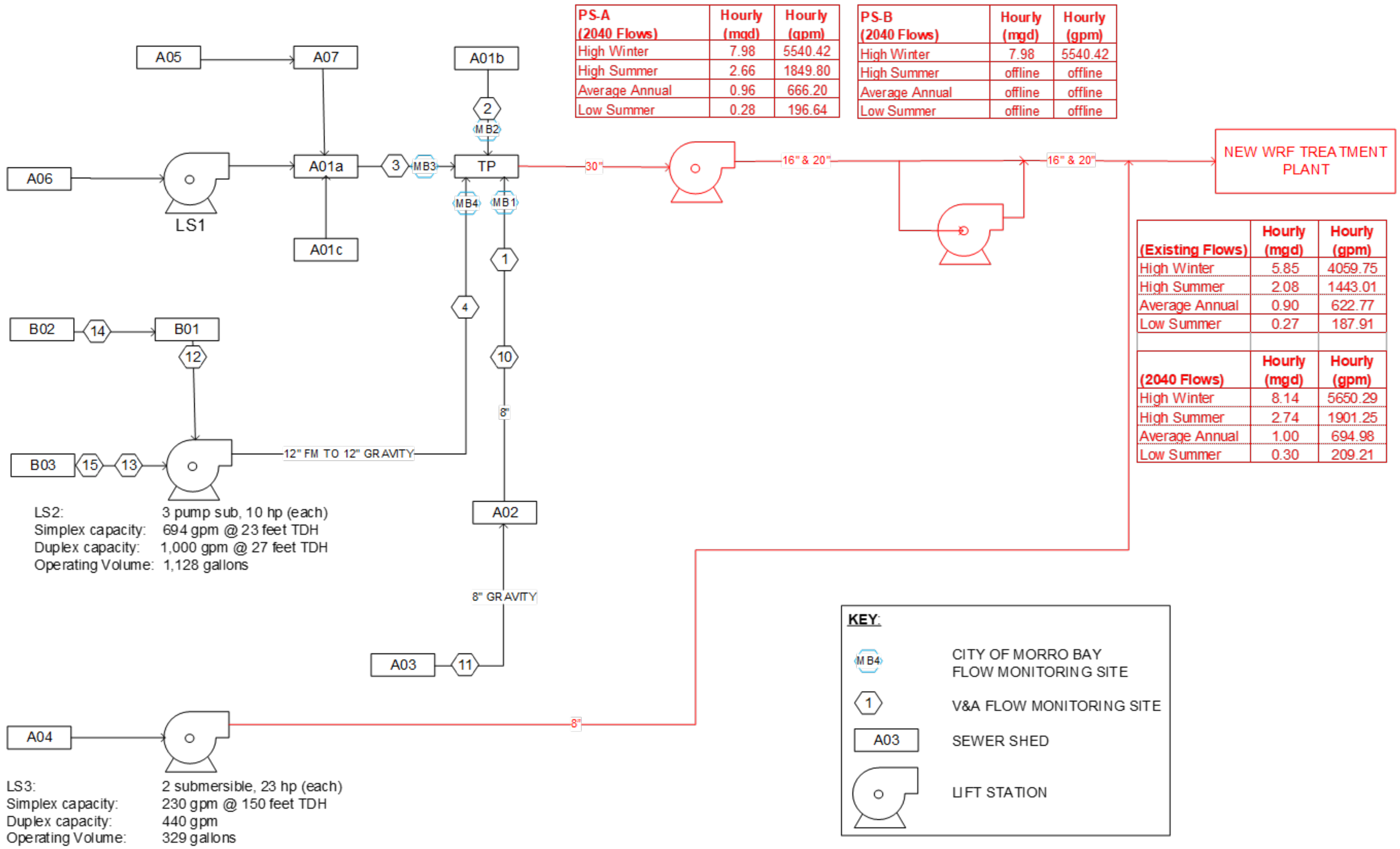


Figure 5-19: Scenario 1 3-D Model

West Scenario 2 - Secondary Stormwater PS-B

A multiple pump station alternative seeks to downsize the number and sizes of pumps required to achieve PWWF. This was addressed by optimizing the hydraulics of the individual stations and by looking at various locations for a secondary pump station (PS-B). As previously discussed, the two locations that were identified for PS-B were based on site availability and hydraulics, WS1: Quintana Road and WS2: Main Street at Highway 1.

One approach to utilizing a multi-pump station configuration involves a pump station at the existing WWTP that could potentially operate in all flow scenarios (PS-A) with the help of a second station (PS-B) located between PS-A and the new WRF, which would be used only in PWWF scenarios. A series of valves would allow for the bypass of PS-B during flows in which PS-A pumps could produce enough TDH to pump all the way to the new WRF. Valves would be activated to utilize PS-B when the head and flow conditions are too great for PS-A to pump on its own. Figure 5-20 is a schematic representation of the Scenario 2: Secondary PS-B configuration.



PS-A (2040 Flows)	Hourly (mgd)	Hourly (gpm)
High Winter	7.98	5540.42
High Summer	2.66	1849.80
Average Annual	0.96	666.20
Low Summer	0.28	196.64

PS-B (2040 Flows)	Hourly (mgd)	Hourly (gpm)
High Winter	7.98	5540.42
High Summer	offline	offline
Average Annual	offline	offline
Low Summer	offline	offline

(Existing Flows)	Hourly (mgd)	Hourly (gpm)
High Winter	5.85	4059.75
High Summer	2.08	1443.01
Average Annual	0.90	622.77
Low Summer	0.27	187.91

(2040 Flows)	Hourly (mgd)	Hourly (gpm)
High Winter	8.14	5650.29
High Summer	2.74	1901.25
Average Annual	1.00	694.98
Low Summer	0.30	209.21

LS3 (Existing Flows)	Hourly (mgd)	Hourly (gpm)
High Winter	0.13	88.98
High Summer	0.07	51.25
Average Annual	0.04	28.69
Low Summer	0.02	12.25

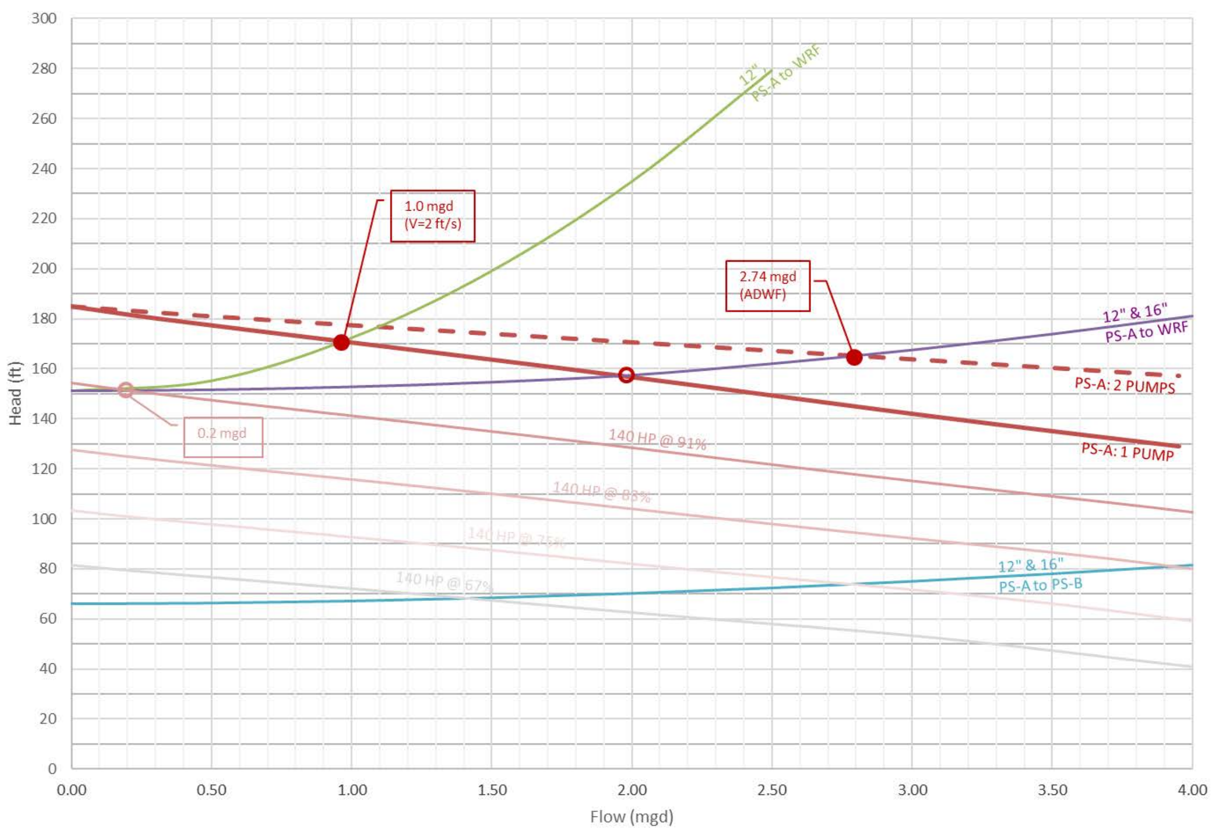
LS3 (2040 Flows)	Hourly (mgd)	Hourly (gpm)
High Winter	0.16	109.87
High Summer	0.07	51.45
Average Annual	0.04	28.78
Low Summer	0.02	12.27

Figure 5-20: Scenario 2 – Secondary Stormwater PS-B

West Scenario 2 at W1

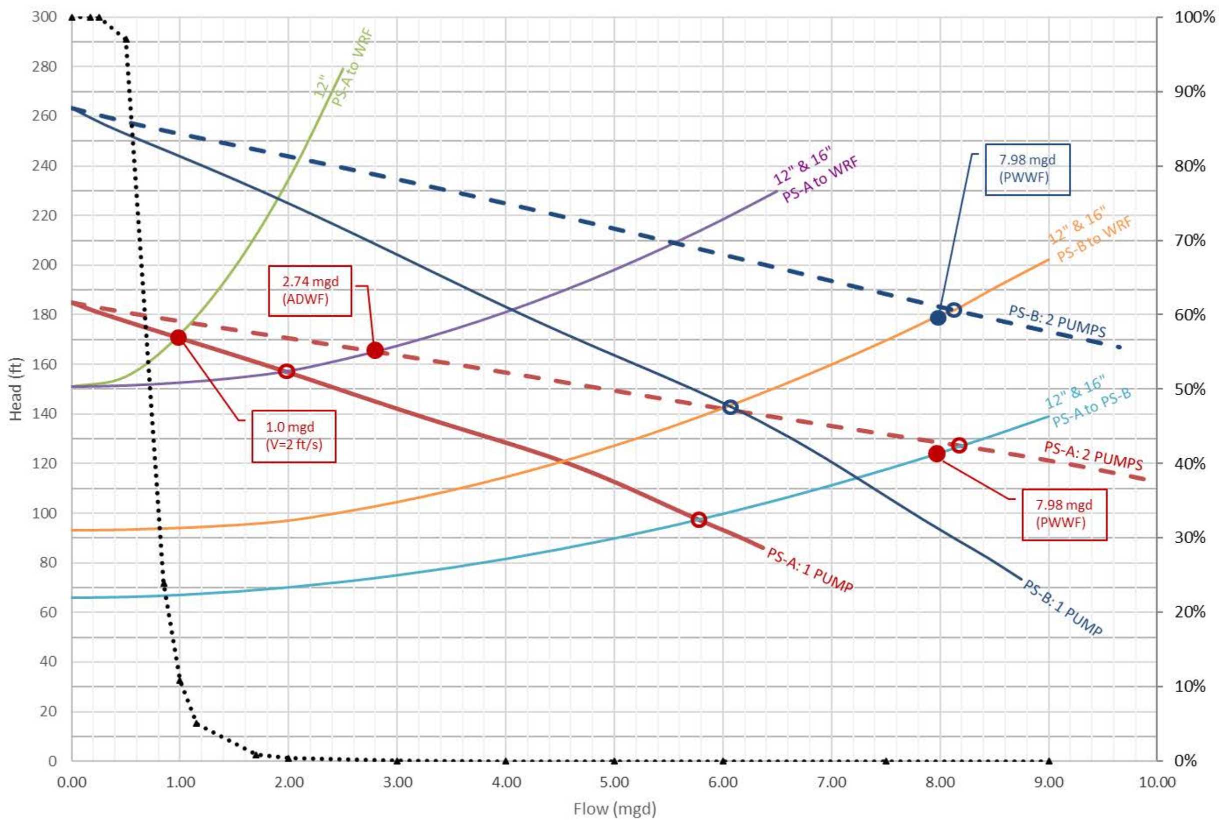
With a secondary pump station (PS-B) at W1 (Quintana Road), PS-A could be reduced from a 6+2 station to a 2+1 station, with only 140-HP pumps. In a dual pump station setup, PS-A would be able to pump flows up to approximately 2.74 MGD directly to the new WRF, which satisfies the anticipated system build-out high summer (PDWF) (Figure 5-20). Figure 5-21 shows that with the use of VFDs, the pumps at PS-A can pump as low as 0.2 MGD, which in the 12" forcemain would be a velocity of 0.45 ft/s. Velocities in the 12" forcemain get up to 2.0 ft/s at 0.95 MGD. For flows between 0.95 – 2.74 MGD, the 12" and 16" forcemains would both be used to get flows to the new WRF. A more detailed breakdown of the number of pumps and forcemains required for specific flow ranges are discussed later in Section 5.4.

Figure 5-21: PS-A Pump for West Scenario 2 at W1



When flows are greater than 2.74 MGD, PS-B would be utilized. Instead of having to overcome 150 feet of static head when pumping to the new WRF, PS-A would only need to overcome 66 feet of static head to get flow to PS-B. Because the pipeline length is reduced going only to PS-B, the dynamic head is also reduced, so the slope of the system curve is made more shallow. This enables the PS-A pumps to accommodate a higher range of flows. With the activation of a series of valves to divert flows to PS-B, PS-A would be able to pump the system buildout flow of 7.98 MGD with only the two 140-HP pumps. PS-B would receive those flows and then pump to the new WRF as a 2+1, 250 HP station. Figure 5-22 shows the pump and system curves for PS-A and PS-B, if PS-B were to be operated as a part-time stormwater pump station.

Figure 5-22: West Scenario 2 at W1

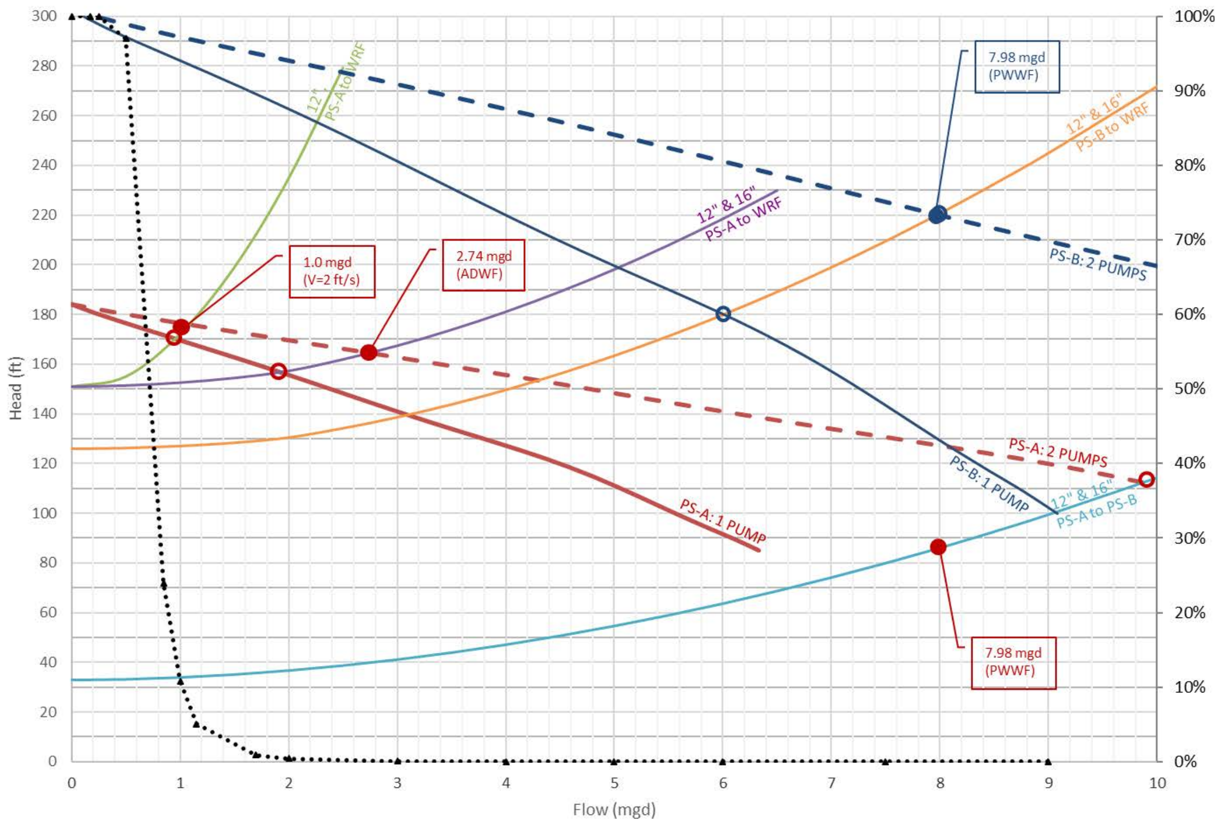


West Scenario 2 at W2

A City owned parcel along Main St., near the Highway 1 underpass, is also a hydraulically viable option for an intermediate PS-B location.

With PS-B located at W2, PS-A would be a 2+1, 140 HP pump setup and PS-B would be a 2+1, 250 HP configuration – similar to W1. This location would have similar pumping ranges as the other potential PS-B site on the West Alignment, on Quintana Road (Figure 5-23). PS-A could pump flows up to 2.74 MGD direct to the WRF. As previously discussed, VFDs can bring the flows down to 0.2 MGD. From 0.2-0.95 MGD the 12" forcemain will see velocities between 0.42-2.0 ft/s. Flows between 2.74-7.98 MGD would be pumped from PS-A to PS-B and PS-B pumps would pump up to 7.98 MGD to the new WRF. A more detailed breakdown of the number of pumps and forcemains required for specific flow ranges are discussed later in Section 5.4.

Figure 5-23: West Scenario 2 at W2



5.3.2 Embarcadero Alignment

The Embarcadero Alignment goes westerly on Atascadero Road from the WWTP, follows Embarcardero and Pacific Street then to Qunitana Road. The Embarcadero Alignment is approximately 2,500 linear feet (LF) longer than the West Alignment, with the same static head.

Embarcadero Scenario 1 - Single Station

The single station/PS-A location is the same for the Embarcadero and West Alignments. Per Figure 5-25, at 7.98 MGD, a single pump station would have to overcome 259 TDH. Similar to West Scenario 1, due to the large range of flows and head conditions, a single station for Embarcadero Scenario 1 will require 1+1, 60 HP jockey pumps to pump flows up to 1.15 MGD and 5+1, 250 HP PWWF pumps to pump up to 7.98 MGD (Figure 5-25) – an 8-pump station. VFD's allow the jockey pumps to get as low as approximately 0.27 MGD as shown on Figure 5-24. Figure 5-25 shows the ranges of flows covered by the different pump and forcemain sizes. A more detailed breakdown of the number of pumps required for specific flow ranges are discussed later in Section 5.4.

Figure 5-24: Jockey Pump for Embarcadero Scenario 1

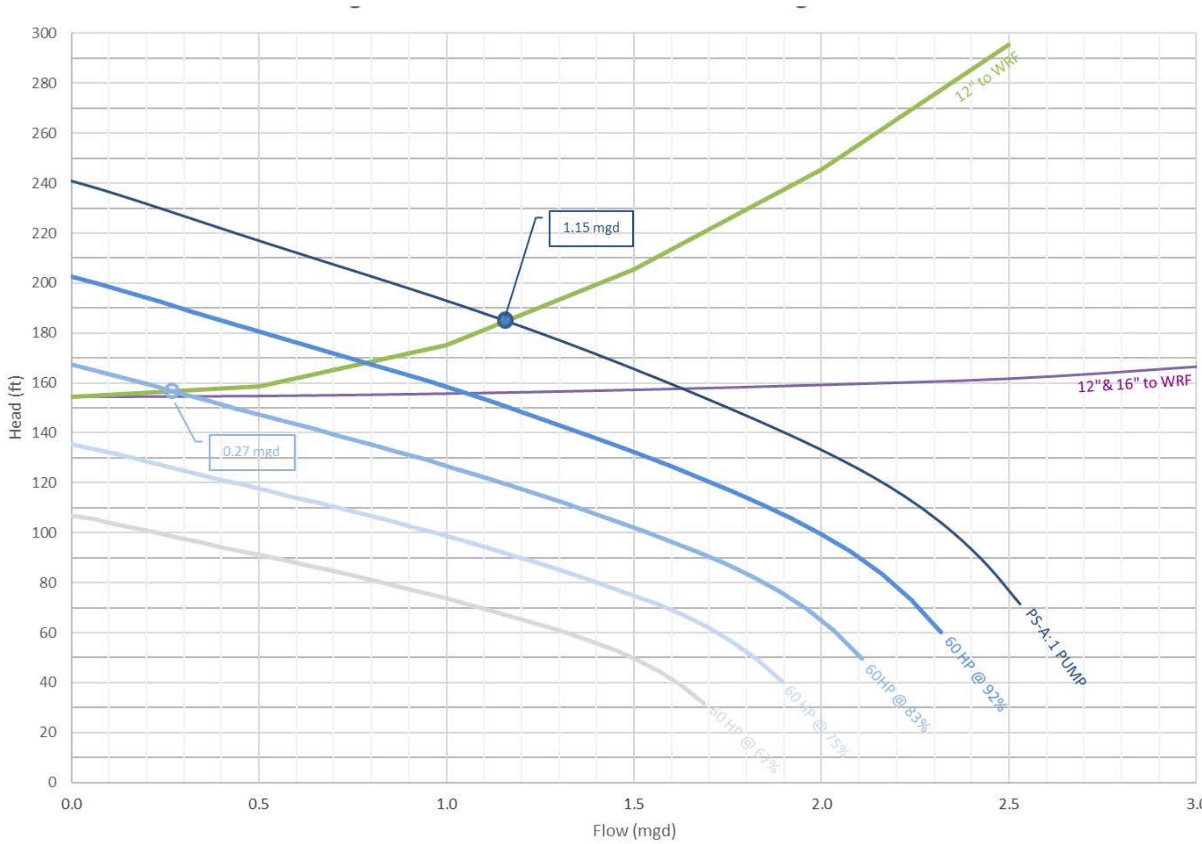
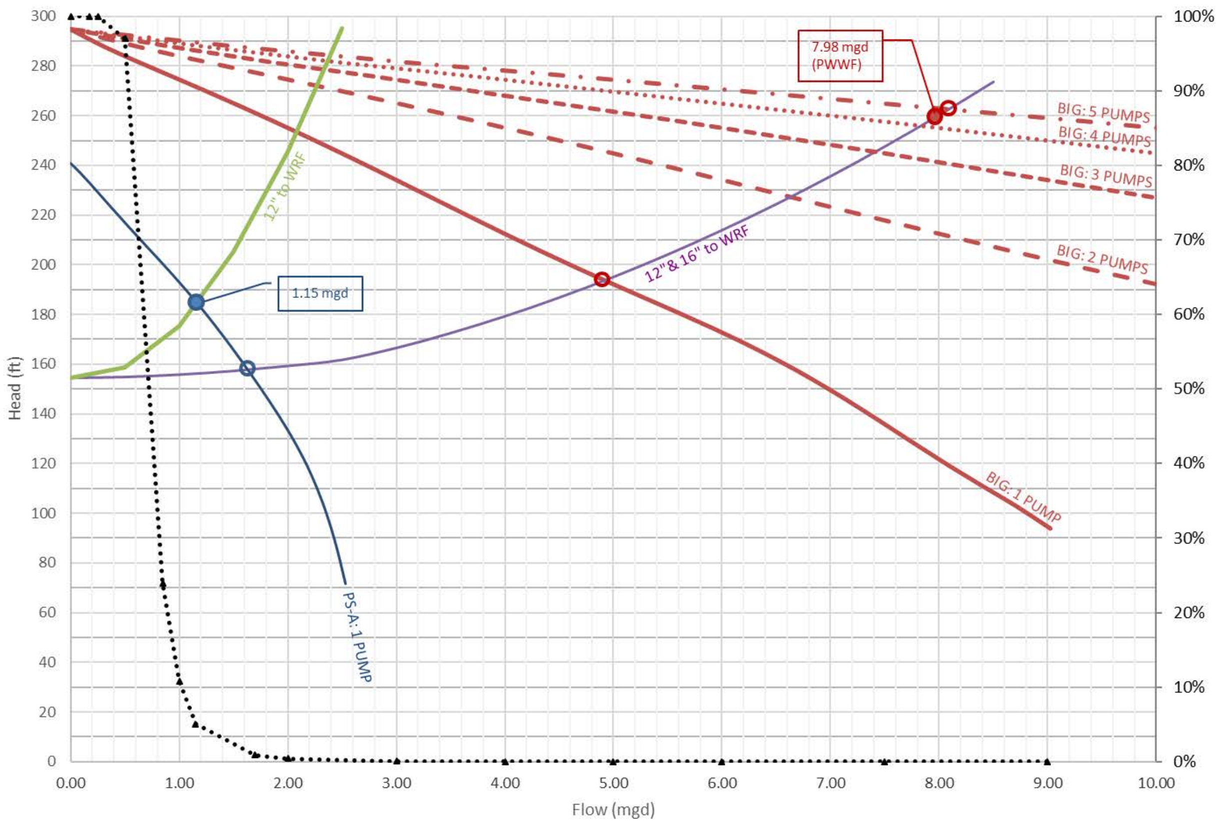


Figure 5-25: Embarcadero Scenario 1



Embarcadero Scenario 2 – Secondary Stormwater PS-B

Two locations (E1 and E2) were identified on the Embarcadero alignment that were hydraulically similar to the sites identified on the West Alignment.

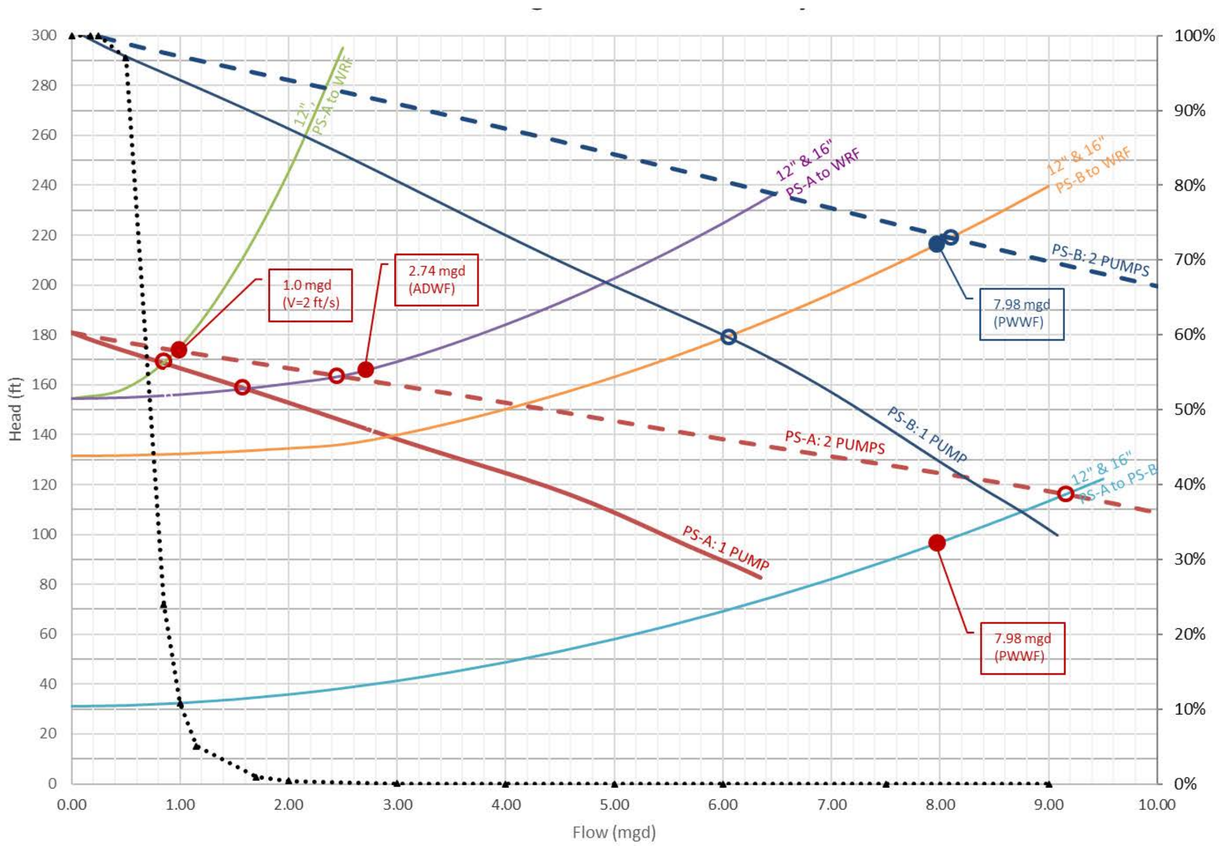
As shown on Figure 5-20, in the pump station scenario in which PS-B is used as a stormwater booster station, PS-A and PS-B are sized to pump 7.98 MGD. PS-A pumps these flows singularly up to a certain flow and then works in conjunction with PS-B to get build-out flows to the new WRF.

Embarcadero Scenario 2 at E1

With PS-B at E1 (City-Owned parcel), PS-A would have approximately 31 feet of static head to overcome when pumping to PS-B during PWWF, instead of 149 feet if it were pumping to the new WRF. PS-A's pumps could pump approximately 2.5 MGD directly to the new WRF. The Scenario 1 pump station goes from an 8+2 single station, to a 2+1, 140-HP pump station for Scenario 2.

Anything above 2.5 MGD up to 7.98 MGD would require the use of PS-B's 2+1, 250-HP pumps. Flows up to 7.98 MGD would be pumped from PS-A to PS-B, then from PS-B to the new WRF. Figure 5-26 shows the system and pump curves for the dual pump stations as well as for the different forcemains.

Figure 5-26: Embarcadero Scenario 2 at E1



Embarcadero Scenario 2 at E2

Through discussions with the City, it was suggested that the existing Bank of America parcel (E2) could potentially be purchased and used for the secondary PS-B site. In comparison to the City-owned parce (E1), this location would give PS-A a static head of 71 feet. Similar to the other PS-B location and those of the West alignment, this would also require PS-A to be a 2+1, 140 HP lift station and PS-B to a 2+1, 250 HP booster station.

PS-A would pump flows up to 2.74 MGD to the new WRF with two pumps running and the use of both 12" and 16" forcemains. The combined use of PS-A and PS-B would achieve flows up to 7.98 MGD with two duty pumps running at both stations.

Figure 5-27: Embarcadero Scenario 2 at E2

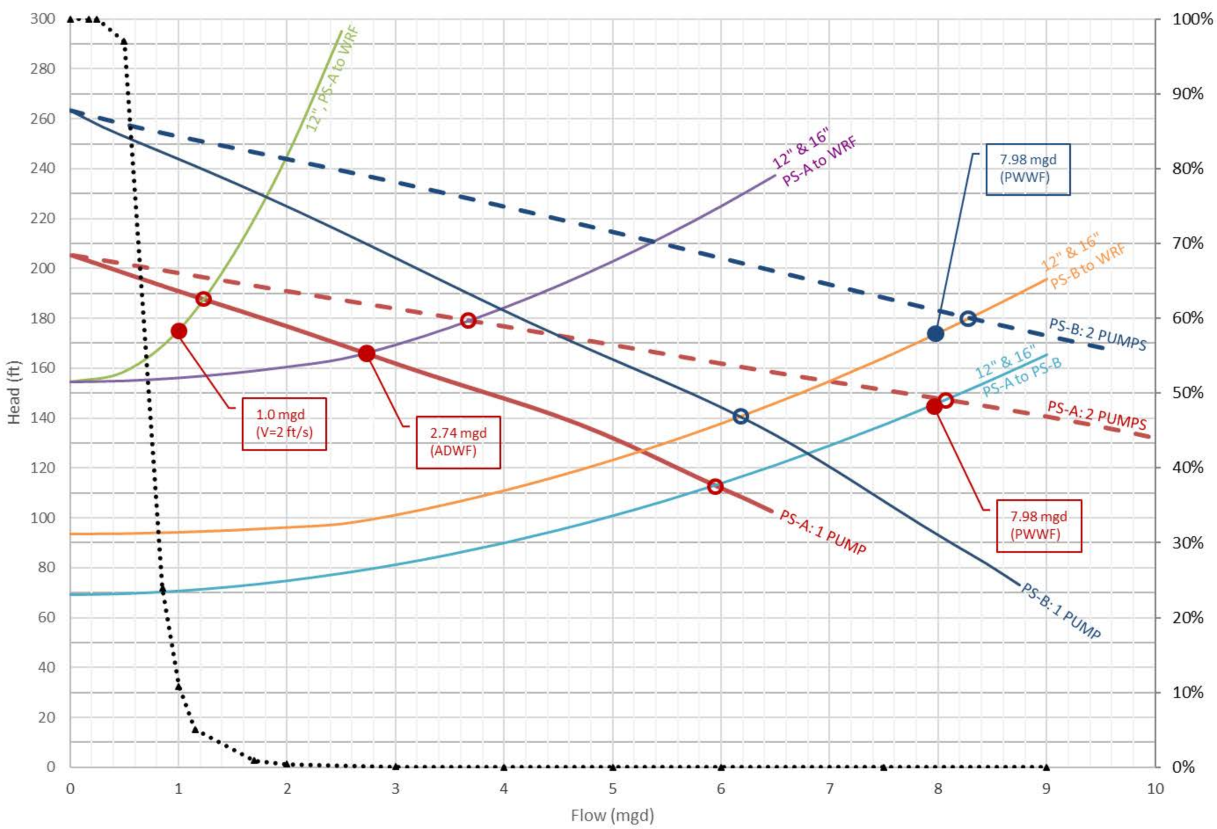
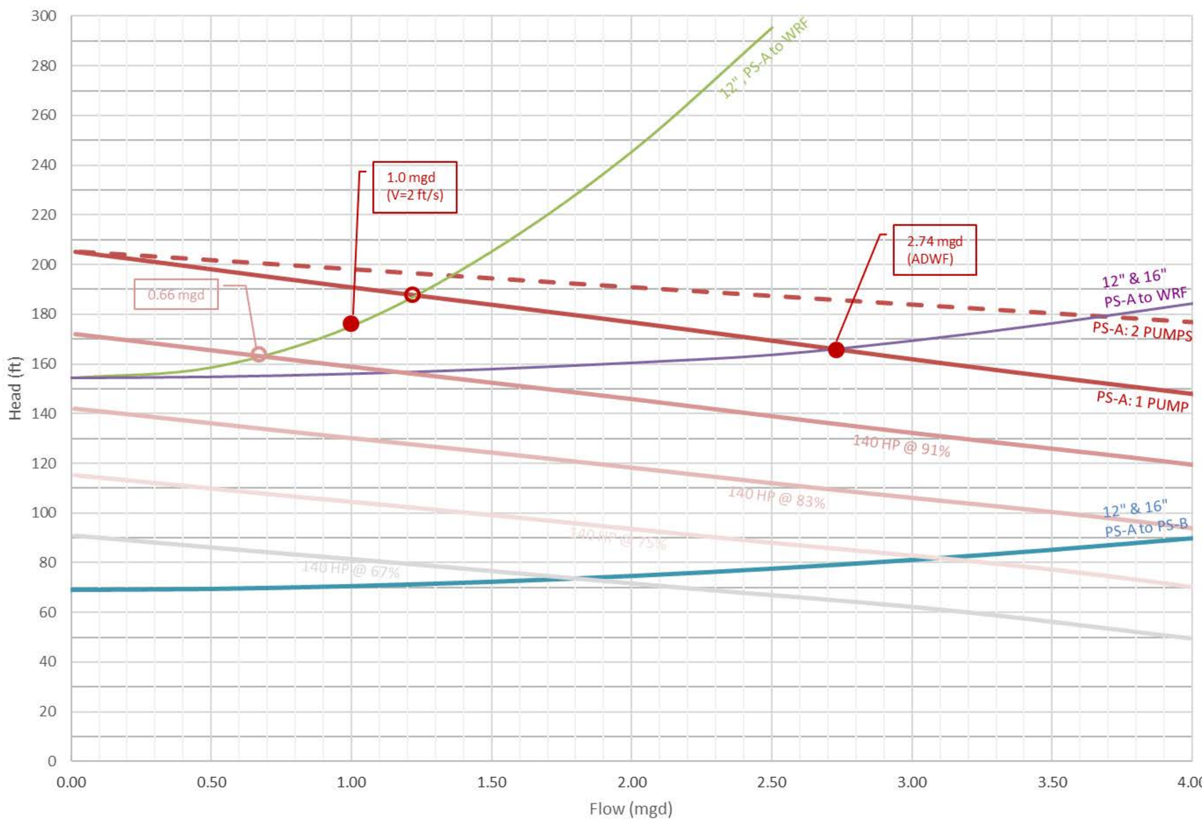


Figure 5-28 shows that with VFDs, PS-A could pump flows as low as 0.66 MGD.

Figure 5-28: PS-A Pump for Embarcadero Scenario 2 at E2



5.4 Pumping Schemes

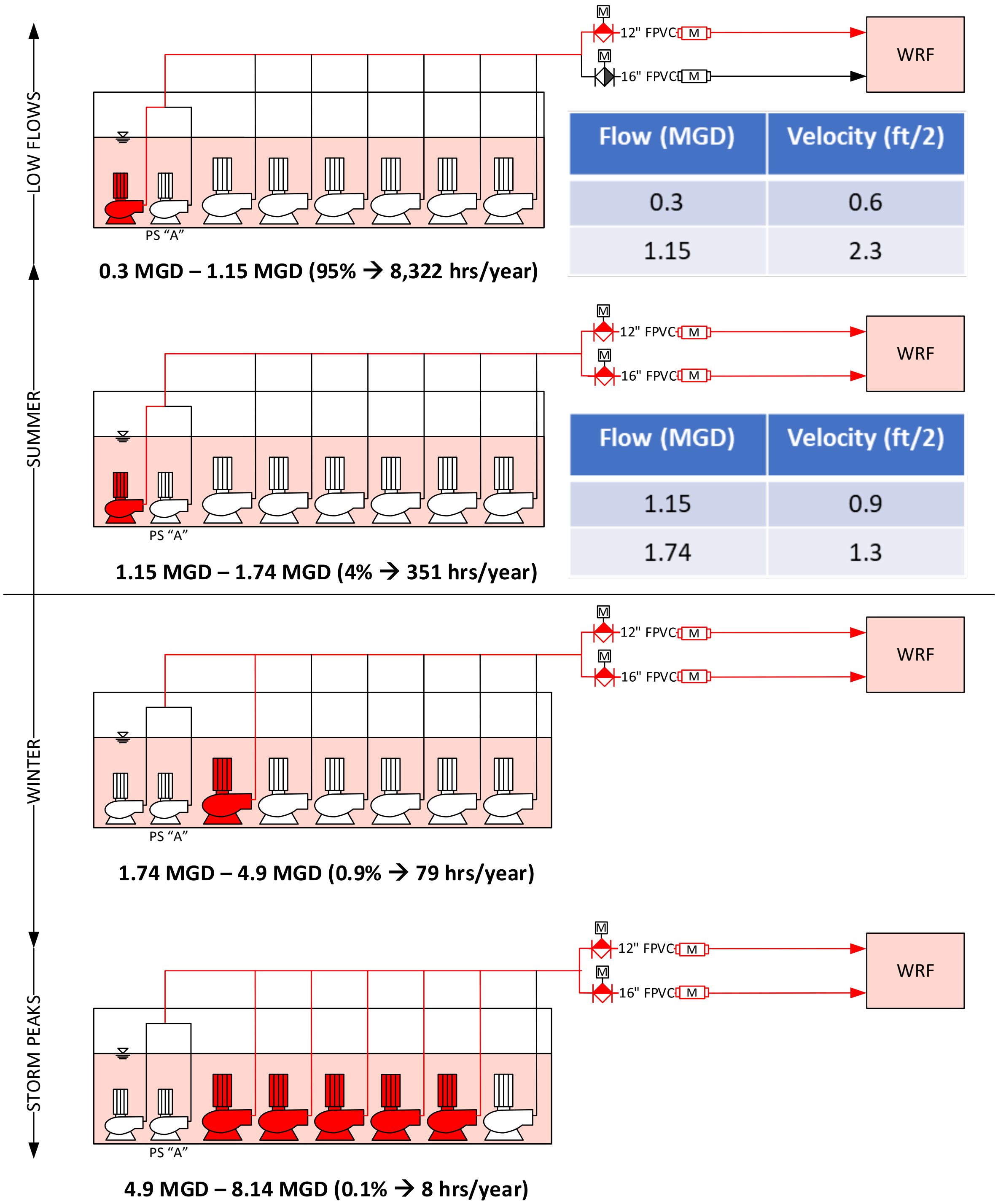
The following summarizes the pumping scheme for the Scenario 1 and Scenario 2 pump station configurations. The operation of each pump is dependent on flows and head through a single 12" or dual (12" and 16") forcemains. Velocities are noted and the frequency of use for each pump is also analyzed as this would affect operation and maintenance costs.

5.4.1 Scenario 1

Due to the hydraulically similar requirements of the pumps along both West and Embarcadero Alignments, the pumping scenario for a Scenario 1 is identical.

A 60 HP jockey pump can pump up to 1.15 MGD through the 12" forcemain, which accounts for flows 95% of the year. A VFD will allow the jockey pump to pump lower flows, as low as 0.3 MGD. From 0.3-1.15 MGD, velocities in the 12" forcemain are approximately 0.6-2.3 ft/s. Utilizing both forcemains (12" and 16"), summer flows up to 1.74 MGD can be achieved by the jockey pump with velocities up to 1.3 ft/s.

Higher summer flows above 1.74 MGD would require the use of the large 250-HP pumps. A single 250-HP pump could pump up to 4.9 MGD. Based on historical flows, any 250-HP pump would only ever run 5% of the time (maximum). Per Figure 5-29, two 250-HP pumps would pump flows up to 6.6 MGD; three pumps would pump 7.5 MGD. Peak wet weather flow up to 7.98 MGD would require the operation of all 5 duty 250-HP pumps, which is anticipated to be needed less than 1% of time.



Flow (MGD)	Velocity (ft/2)
0.3	0.6
1.15	2.3

Flow (MGD)	Velocity (ft/2)
1.15	0.9
1.74	1.3

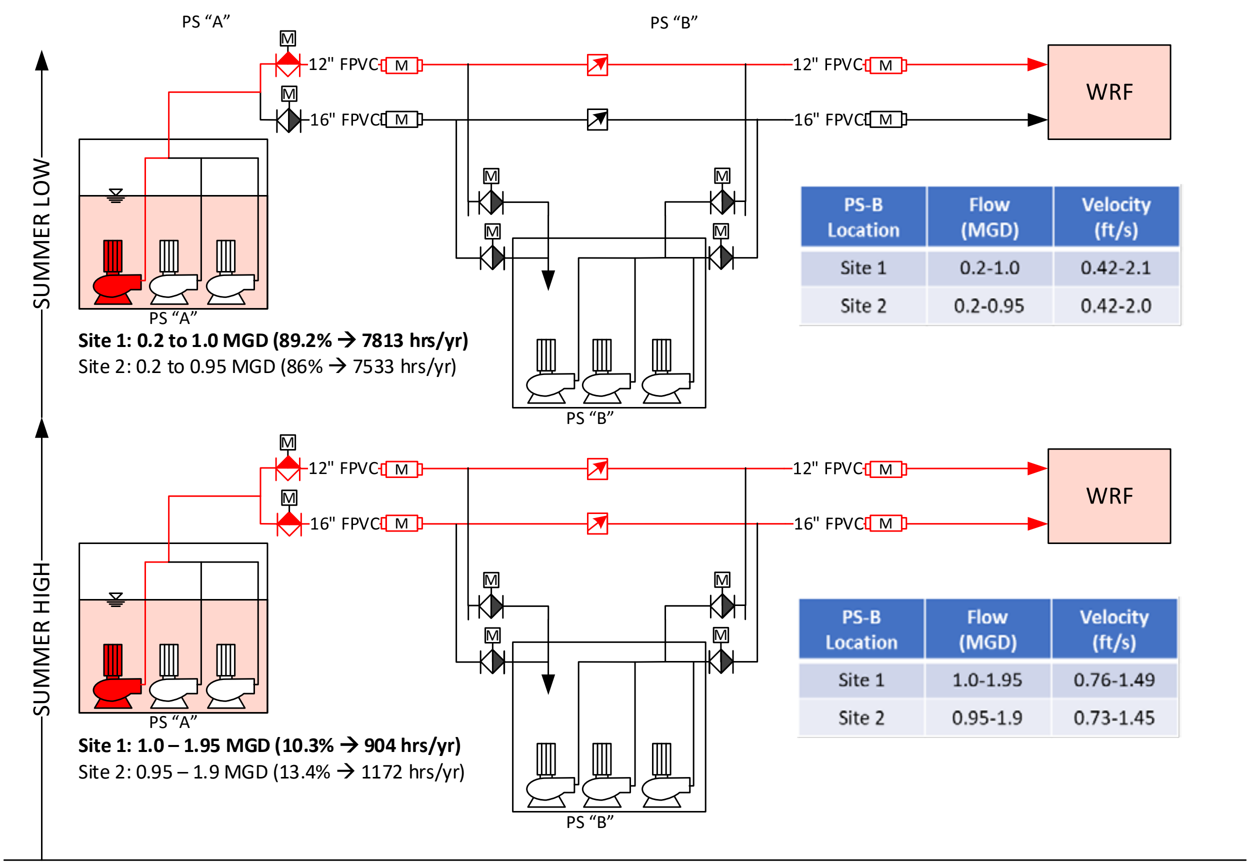
Figure 5-29: Scenario 1 Pumping Scheme

5.4.2 West Scenario 2

PS-A in West Scenario 2 can handle 99% of the station's flows regardless of which site PS-B is located.

For PS-B at W1, one pump at PS-A could pump up to 1.0 MGD through the 12" forcemain with a velocity of 2.1 foot per second (ft/s). With PS-B at W2, this flow would be reduced slightly to 0.95 MGD at 2.0 ft/s.

A single pump at PS-A could be used with the 12" forcemain to achieve up to 0.95 MGD. With the use of both forcemains and both duty pumps at PS-A achieve 2.74 MGD. For flows greater than 2.74 MGD, PS-B would be utilized. PS-A would switch to pumping to PS-B and PS-B would then pump flows up to 7.98 MGD to the new WRF. PS-B would also be a 2+1, 250-HP station. It is anticipated that PS-B would be used less than 1% of the year.



PS-B Location	Flow (MGD)	Velocity (ft/s)
Site 1	0.2-1.0	0.42-2.1
Site 2	0.2-0.95	0.42-2.0

PS-B Location	Flow (MGD)	Velocity (ft/s)
Site 1	1.0-1.95	0.76-1.49
Site 2	0.95-1.9	0.73-1.45

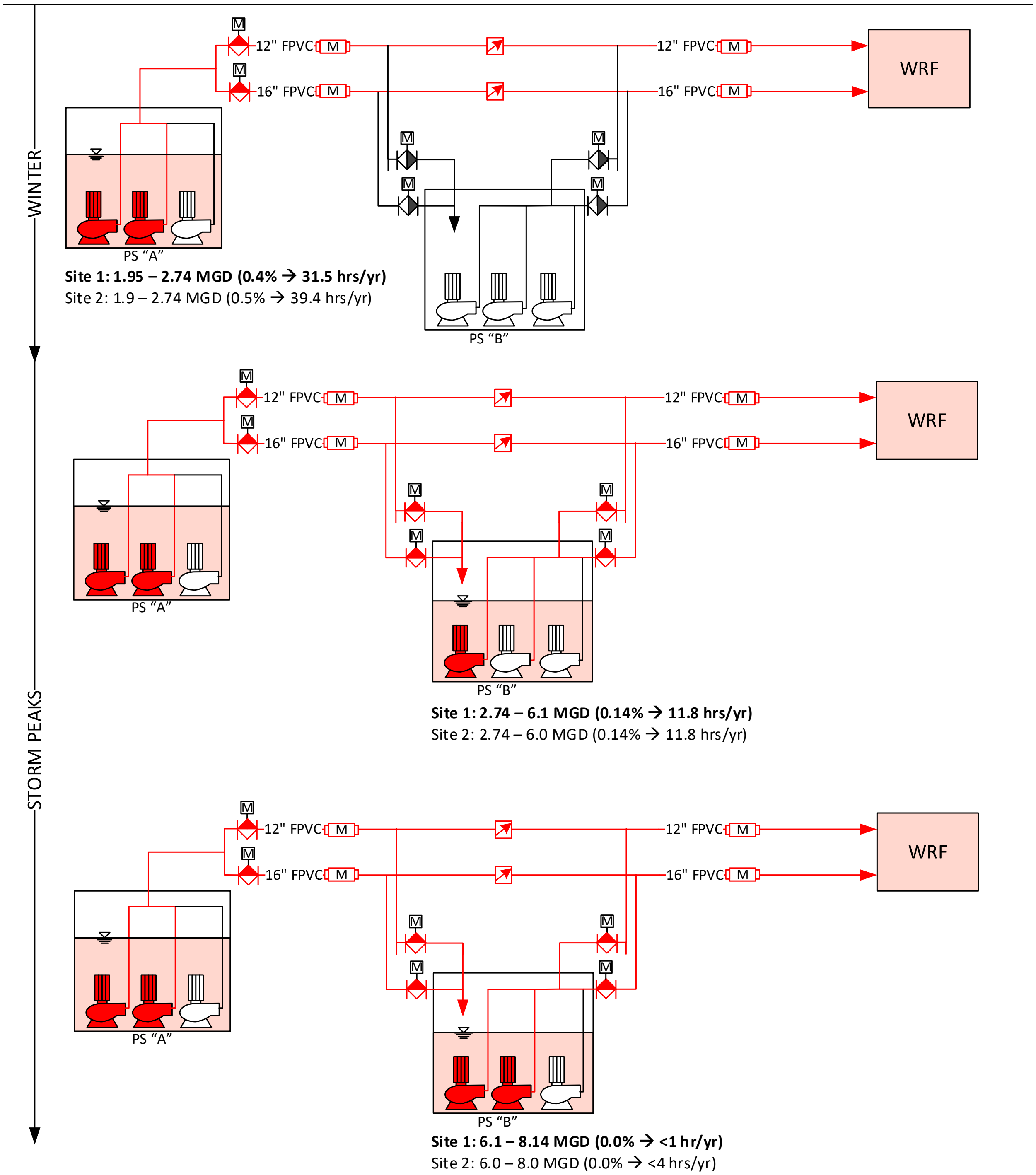
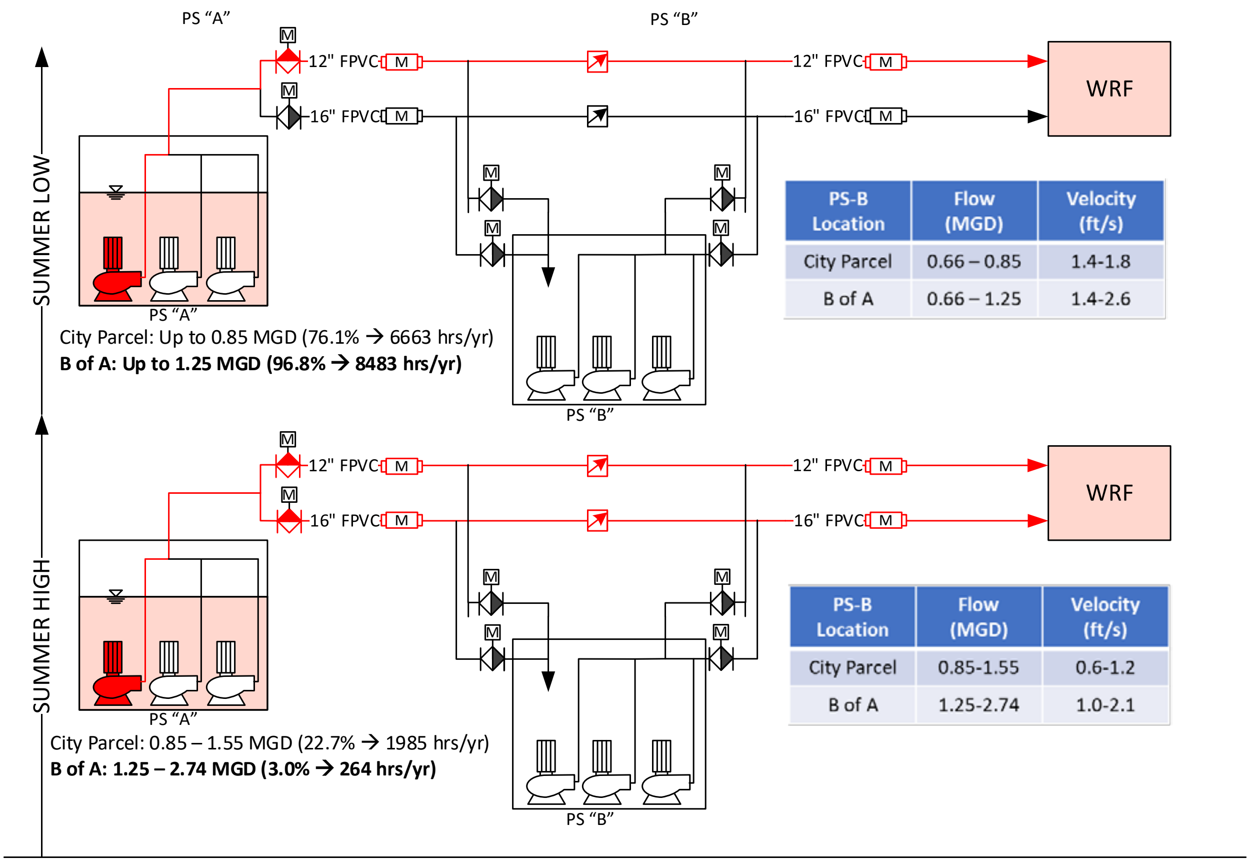


Figure 5-30: West Scenario 2 Pumping Scheme

5.4.3 Embarcadero Scenario 2

Unlike the West Scenario 2, there are differences in flow ranges and velocities between the potential PS-B locations on the Embarcadero Alignment.



PS-B Location	Flow (MGD)	Velocity (ft/s)
City Parcel	0.66 – 0.85	1.4-1.8
B of A	0.66 – 1.25	1.4-2.6

PS-B Location	Flow (MGD)	Velocity (ft/s)
City Parcel	0.85-1.55	0.6-1.2
B of A	1.25-2.74	1.0-2.1

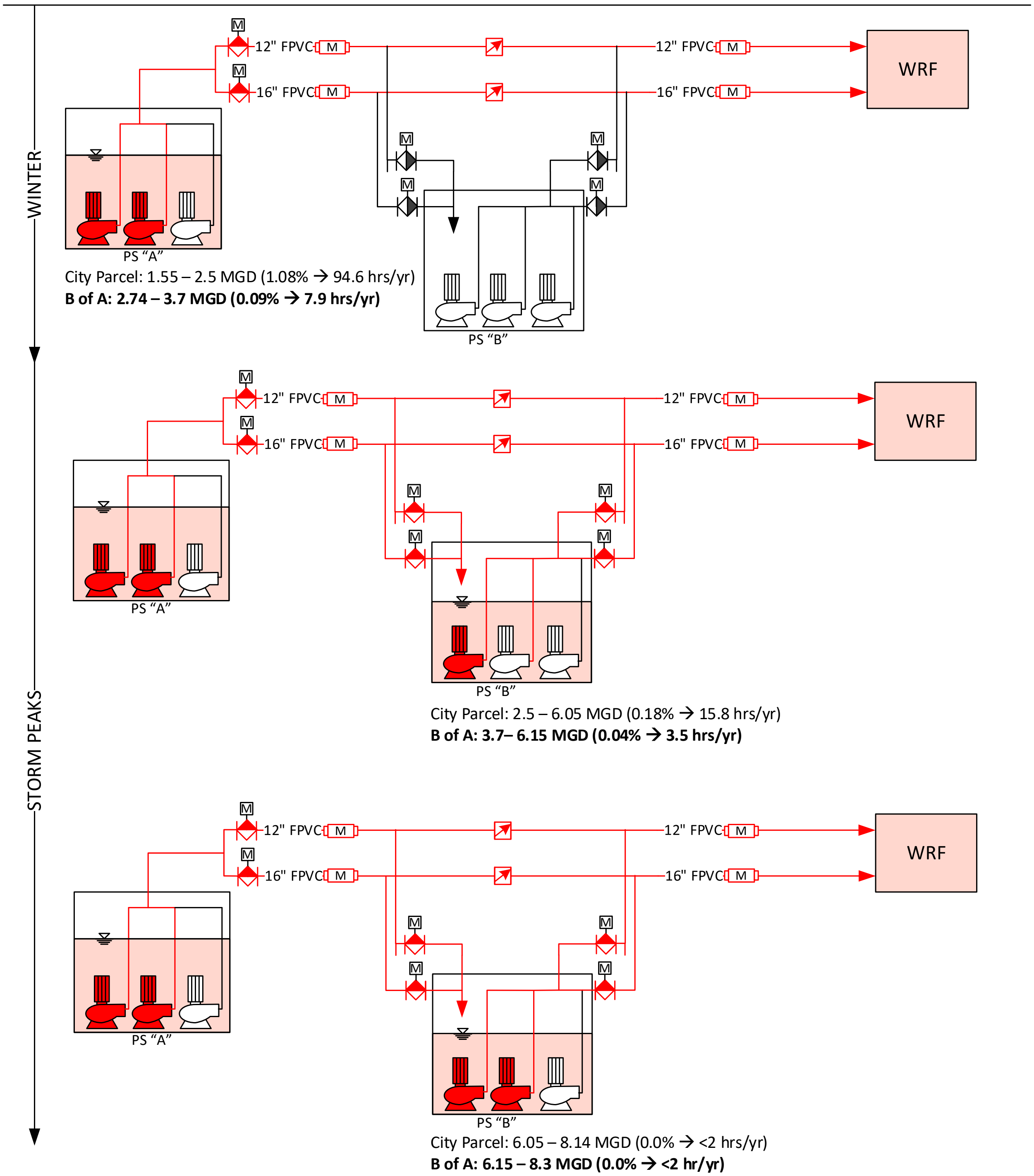


Figure 5-31: Dual Pump Station Pumping Operations, Embarcadero Alignment

E1 (City-Owned Parcel)

With PS-B at E1, one pump at PS-A could handle up to 0.85 MGD to the new WRF through the 12" forcemain with a velocity of 1.86 foot per second (ft/s). With both the 12" and 16" forcemains, one PS-A pump can reach velocities of 1.2 ft/s at flows up to 1.55 MGD. Two pumps operating at PS-A and the use of both forcemains would give PS-A the capacity of 2.5 MGD to the new WRF.

Flows greater than 2.5 MGD would activate the valves at PS-B and flows from PS-A would switch from going to the new WRF to discharging into the PS-B wet well. PS-B would operate with one 250-HP pump for flows between 2.5-6.05 MGD. Flows up to 7.98 MGD would require the operation of both duty pumps at PS-B. It is anticipated that PS-B would be used less than 1% of the year.

E2 (Bank of America Parcel)

With PS-B at E2, flows with one pump running at PS-A would be increased to 1.25 MGD, with a velocity in the 12" forcemain of 2.6 ft/s. With the use of both forcemains, a single pump can get flows up to 2.74 MGD, with velocities around 2.1 ft/s. The second duty pump at PS-A can deliver 3.7 MGD directly to the new WRF via both forcemains.

With the addition of a single pump at PS-B, combined PS-A and PS-B would be able to transport up to 6.15 MGD to the new WRF. Two 250-HP pump at PS-B take the system up to the 7.98 MGD build-out flows.

5.5 Pumping Scenarios Assessment

5.5.1 Pumping Scenarios

Pumping scenarios for both alignments have been developed as follows:

- **Scenario 1**: Single Pump Station → pumping from near the existing WWTP to the new WRF. Alternative locations for a single pump station:
  - **PS-A Site 1**: Single station located at the existing influent pump station
  - **PS-A Site 2**: Single station located at the existing City parcel, South of Atascadero
  - **PS-A Site 3**: Single station located on the north side of Atascadero
- **West Scenario 2**: Secondary Stormwater Pump Station → PS-A (at any Scenario 1 site) would pump along the Embarcaderto alignment to the new WRF the majority of the time but during PWWF, pump to PS-B instead. PS-B would then convey flows to the new WRF. Alternative PS-B locations:
  - **W1A**: PS-B at lot north of Quintana Road
  - **W1B**: PS-B at lot south of Quintana Road
  - **W2**: City-owned parcel on Main Street at Highway 1

- **Embarcadero Scenario 2**: Secondary Stormwater Pump Station → PS-A (at any Scenario 1 site) would pump along the Embarcardero alignment to the new WRF the majority of the time but during PWWF, pump to PS-B instead. PS-B would then convey flows to the new WRF. Alternative PS-B locations:
  - **E1**: City-Owned Parcel at Pacific St. and Market Ave.
  - **E2**: Bank of America Parcel at Pacific St. and Monterey Ave.

Site Assessment

A preliminary site feasibility assessment was conducted, and the summary of the findings are listed below.

Pump Station Site	Pros	Cons
PS-A Site 1	• No property procurement as it is currently City property • No permit requirement	• Requires removal of existing mechanical and electrical equipment • Requires structural retrofit for flooding • Requires structural adaptation for new pumps • Requires new mechanical and electrical equipment • Requires retrofit of influent flume for flows greater than 7 MGD • No storage • Oversized structure/footprint for PS-A in dual pump station scenario • Not appealing for selling existing WWTP property • Construction sequencing with existing WWTP operations → **FATAL FLAWED due to magnitude of cons**
PS-A Site 2	• Adjacent to public right-of-way • No construction impact to existing WWTP operations • Maximizes WWTP property for resale	• Potential for property procurement • Requires potential relocation of existing utilities • Requires permitting
PS-A Site 3	• No property procurement as it is currently City property • Allows for more contiguous parcel for selling WWTP property	• Requires demolition prior to construction

	• Access from Atascadero Road • No construction impact to existing WWTP operations • No permitting requirement	
W1A	• At grade with Quintana Road • Undeveloped parcel • Access from Quintana Road	• Requires property procurement
W1B	• Undeveloped parcel	• Requires property procurement • Significantly higher than Quintana Road which adds static head to PS-A → **FATAL FLAW**
W2	• No property procurement as it is currently City property • Undeveloped parcel • Access from Main Street and Highway 1	
E1	• No property procurement as it is currently City property	• Loss of parking spaces
E2	•	• Requires property procurement

Hydraulic Assessment

A preliminary hydraulic assessment was conducted, and the summary of the findings are listed below

Scenario 1 – Single Pump Station	
PROS	**CONS**
• One station to maintain • Majority of equalization can be done at station • Smaller pumps (jockey pumps) used most of the time – easier/cheaper to rebuild/replace	• Lower velocities in FM • Oversized wetwell during majority of flows • 5 large pumps with long idle time

Scenario 2 – Secondary Stormwater Pump Station	
PROS	**CONS**
• Smaller footprints • Higher minimum velocities in FM • Less pumps to maintain • Conducive to pigging & flushing	• Pump cycling • 3 large pumps at separate facility with idle time • Larger pumps at PS-A being used most of the time – more expensive to maintain/replace • Two locations (two: buildings, generators, etc.)

5.5.2 Project Costs

Preliminary capital, operational and replacement cost assessments were conducted, and the summary of the findings are listed below.

Direct Construction Costs

The direct construction costs included in Table 5-4 apply to those costs the contractor would bid.

Table 5-4: Direct Construction Costs

DIRECT CONSTRUCTION COSTS			
	SCENARIO 1	SCENARIO 2 (WEST OF EMBARCADERO)	
		PS-A	PS-B
Site Work	$1,134,000	$493,000	$237,400
Piping, Valves, Fitting & Appurtenances	$304,400	$165,200	$172,100
Equipment	$1,588,400	$665,600	$637,300
Structural	$1,161,200	$366,600	$272,400
Electrical	$4,109,300	$1,377,500	$1,942,500
Project Subtotal 1	**$8,297,300**	**$3,067,900**	**$3,261,700**
20% Design Contingency	$1,659,460	$528,660	$736,500
TOTAL CONSTRUCTION BID COST	$9,956,760	$3,171,960	$4,419,000
10% Construction Contingency	$995,676	$317,196	$441,900
TOTAL PROJECT CONSTRUCTION COST	$10,952,436	$3,489,156	$4,860,900
TOTAL PROJECT COMPARISON (nearest $10,000)	***$11,000,000***	***$8,400,000***	

As seen in Table 5-4, Scenario 2 represents considerable capital cost savings over Scenario 1.

Indirect Construction Costs

Indirect construction costs apply to those costs incurred by the City which would not be included in the contractor's bid. These costs would include costs associated with property procurement, permitting, environmental mitigation, etc. Property procurement costs would vary for individual sites. It is assumed that permitting and the purchase of property would be upward of $1 million for any non-City owned site for PS-A or PS-B. Even adding $1M to Scenario 2 to account for permitting and property acquisition, if necessary, it still represents a significant cost savings over Scenario 1.

Annual Operations and Maintenance Costs

Table 5-5 represent the estimated ongoing costs that would be incurred by the City for the operation of the pump stations annually.

Table 5-5: Annual Operations and Maintenance Costs

OPERATIONAL COSTS (2018 dollars, nearest $1,000)		
	SCENARIO 1	SCENARIO 2
	$/year	$/year
Total Annual Pumping Cost	$37,000	$47,000
Ancillary Power Usage	$5,000	$6,000
Odor Control Media Replacement	$4,000	$3,000
Mechanical Maintenance	$13,000	$14,000
	$59,000	**$70,000**

Based on preliminary pump selections, the Total Annual Pumping Cost for Scenario 2 is greater due to the differences in efficiencies between the Scenario 1 60-HP jockey pumps and the PS-A 140-HP pumps. It is expected that during the design phase, additional pumps and pump manufacturers will be vetted and efficiencies will be optimized.

Table 5-6 represent the estimated occasional costs that the City would incur for the replacement of equipment and maintenance of the pump stations.

Table 5-6: Replacement Funds

REPLACEMENT FUNDS		
	SCENARIO 1	SCENARIO 2
	$/year*	$/year
Recoating & Repainting	$9,000	$4,000
Pump Replacement	$53,000	$40,000
Switchgear/VFD Replacement	$168,000	$127,000
	$230,000	**$171,000**

*Assumes annualized cost of 20-year replacement program

Property procurement costs would vary for individual sites. It is assumed that permitting and the purchase of property would be upward of $1 million for any non-City-owned site for PS-A or PS-B.

5.5.3 Non-Cost Project Constraints

Single Pump Station/PS-A Sites

The potential sale of the WWTP property is a significant non-cost constraint. The location of the existing WWTP is prime beachfront real estate. The ability to sell the property as a contiguous piece of land is more appealing than that of a segmented parcel such as one which would keep the existing influent pump station location a functioning piece of the City's wastewater collection system.

Disruption to the existing physical WWTP is also a potential non-cost concern. Re-use of the existing pump station would affect construction sequencing as well as operations at the WWTP.

A plan for by-passing the influent pump station would be needed. The demolition associated with PS Site 3 is not anticipated to affect the existing WWTP and can be done at any time.

The existing treatment plant is about 40-years old. The age and condition of the existing structure would need to be assessed and evaluated for its anticipated remaining life. Consideration for installing new mechanical and electrical equipment into a retrofitted, considerably aged structure should be accounted for.

Non-cost considerations for the location of a single pump station or PS-A are summarized below. Based on these constraints, reusing the existing influent pump station would be the least desirable alternative.

Single Pump Station/PS-A Options Non-Cost Project Constraints				
Non-Cost Constraint	**PS Site 1**	**PS Site 2**	**PS Site 3**	**Discussion**
Sale of existing WWTP	-	+	-	PS Site 2 – North of Atascadero would allow for the largest sale of contiguous existing WWTP property.
WWTP Impacts / Constructability	-	+	+	Re-using the existing influent PS would be the most disruptive to WWTP operations. PS Site 3 would require the demolition of an existing building but would not affect the operations of the WWTP.
Aging Infrastructure	-	+	+	The existing influent PS (PS Site 1) is approaching 40 years old.
Environmental	+	-	+	The site north of Atascadero would require property procurement as well as permitting for construction. Protective fencing may be needed along the northern construction limit.

PS-B Sites

The evaluation of PS-B sites should be considered along with the assessment of the West Alignment vs. Embarcadero alignment discussed in Chapter 4.

The West alignment has the potential for diverting flow from Lift Station 2 and potentially decreasing the size of PS-A. Gravity flows near the PS-B sites can potentially be diverted into the wet well which could have a positive effect on potential capital improvement projects (CIP) in the area.

Lift Station 2 has an existing 8” forcemain that goes towards the general direction of the West Alignment that could be looked into as a possible additional flow diversion to downsize PS-A.

The West Alignment sites are all on vacant lots which have minimal adjacent neighbors. The pump stations would be on the edge of development rather than in the middle of a business area. Embarcadero sites would require the demolition of existing pavement and lessen the number of available public parking spaces if the lots were kept to serve that purpose. Construction would also be more impactful in areas where there are surrounding businesses and parking becomes inaccessible/unusable.

Because the Embarcadero sites are already developed parcels, there are no environmental concerns. The undeveloped West sites do have areas that may require protection.

Non-cost considerations for the location of PS-B for a dual pump station scenario are summarized below. These constraints further the argument to go with the preferred the West Alignment.

PS-B Options Non-Cost Project Constraints					
Non-Cost Constraint	**W1A & W1B**	**W2**	**E1**	**E2**	**Discussion**
Potential for Gravity Sewer Diversion	+	+	-	-	Diverting gravity flows from near the Embarcadero sites would not help decrease the size of PS-A and possibly eliminate CIPs.
Potential for Lift Station 2 Flow Diversion	+	+	-	-	LS-2 has an existing 8" FM that goes in the general direction of the West Alignment.
Construction Impacts / Public Visibility	+	+	-	-	The sites along Embarcadero are surrounded by businesses and would likely be surrounded by public parking.
Environmental	-	-	+	+	West Sites 1A and 2 are bordered by unnamed drainage ditch that parallels Highway 1 and would require protection during construction.

Scenario 1 vs. Scenario 2

Non-cost considerations for a single pump station versus the option for a secondary pump station are summarized below.

Pump Station Options Non-Cost Project Constraints			
Non-Cost Constraint	**Scenario 1**	**Scenario 2**	**Discussion**
Sale of existing WWTP	-	+	Dual pump stations reduce the size of the pump station at/near the existing WWTP. The size of the property that the City would have to retain is minimized.
Constructability / Traffic Impacts	+	-	Construction of a single pump station would only affect the area in and around Atascadero Road (i.e. Morro Bay High School). A PS-B station would have some traffic affects to frontage streets during construction.

5.5.4 Preferred Alternatives

In assessing the various pump station sites, several alternatives were fatal flawed and based on the pros and cons and non-cost constraints, the preferred sites are as follows:

- West Alignment → for the reasons discussed in Chapter 4 along with the potential to divert gravity flows along the West alignment make this alternative preferable
- PS-A Site 3 → does not require permitting or property procurement, does not impact the existing WWTP operations and allows for a brand new station
- W2 → does not require property procurement

Hydraulic assessments of Scenario 1 and Scenario 2 show Scenario 2 is preferred due to:

- Higher velocities in the forcemains
- Better sizing of structures
- Smaller quantity of pumps
- Shorter forcemains to clean

The anticipated project costs further emphasize that the preferred alternative is to have dual pump stations, with PS-A operating all times of year and PS-B operating only during storm events (Scenario 2) on the West Alignment. For the reasons stated in the comparisons above, it is preferred to locate PS-A at PS-A Site 3 (North side of Atascadero) and PS-B at site W2 (the City owned parcel on Main Street at Highway 1).

6 ADDITIONAL PUMP STATION ALTERNATIVE

After presenting Scenario 1 and 2 to the City and making a recommendation for a preferred alternative described above, Scenario 2 was agreed to by all parties. In further discussions during the workshop, however, there was a concern regarding the infrequent operation of PS-B and a desire to revisit the pumping scenarios in order to make PS-B a full-time pump station. These workshop discussions prompted the idea of diverting flows from LS-2 to PS-B and using PS-B full time. This would facilitate further downsizing PS-A and could require little or no modification to LS-2. Full-time stations are appealing to operations and maintenance staff as they would not be forced to rely on typically idle infrastructure during worst-case-scenario PWWF. This approach was called Scenario 3 and was investigated further to test its viability as a refinement of the preferred Scenario 2.

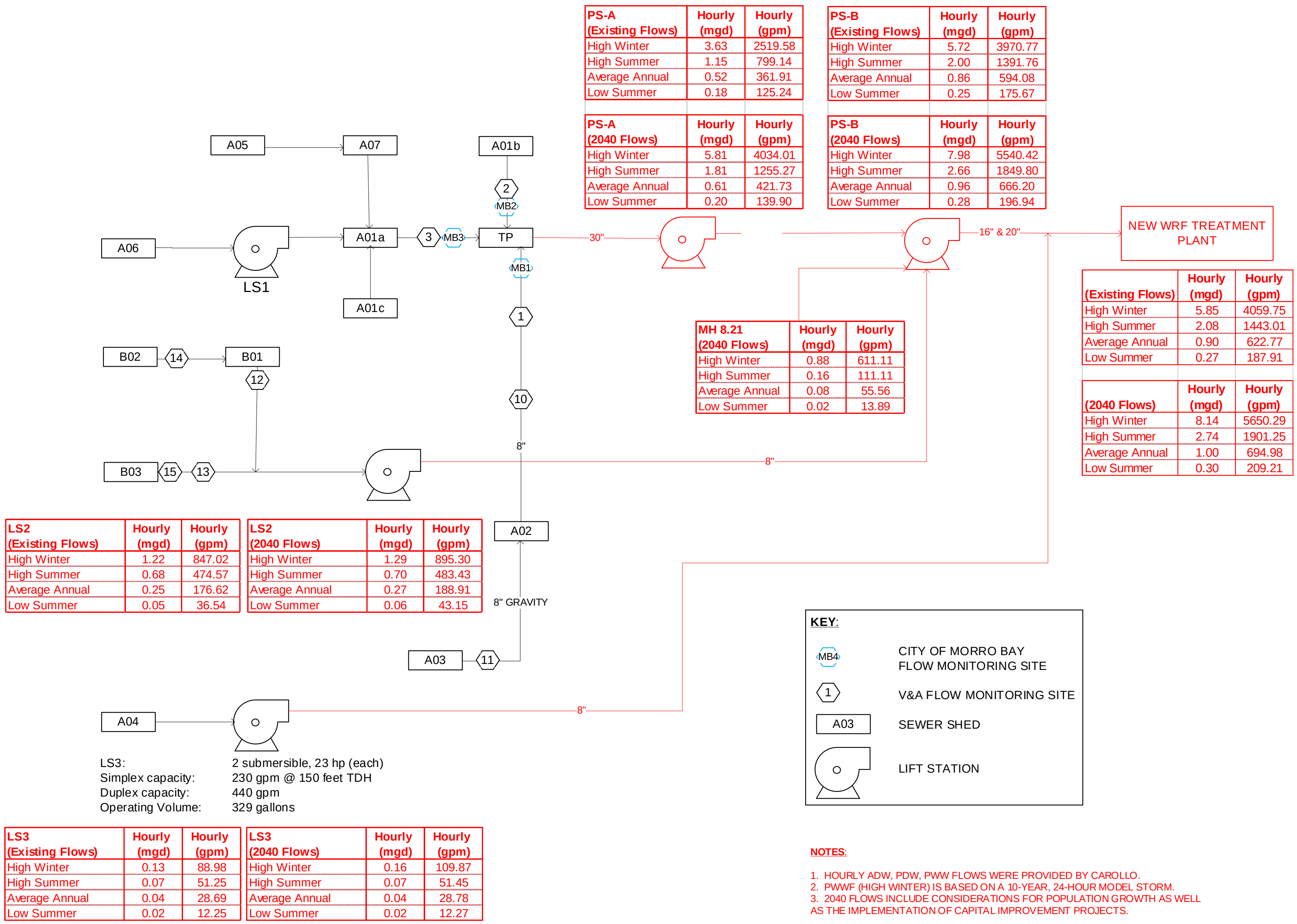
6.1 Scenario 3 Overall Description

The Scenario 3 approach to dual pump stations is as follows:

- LS-2 flows (1.29 MGD) would be pumped directly to PS-B,
- Gravity flows near PS-B (0.88 MGD, namely from MH 8.21, seen in Figure 6-1) would be diverted directly to PS-B
- PS-A buildout flows would therefore be decreased from 7.98 MGD (in Scenario 2) to 5.81 MGD.

Figure 6-2 illustrates the flow distribution for Scenario 3.

Figure 6-1: Gravity Manholes near PS-B



LS3:
Simplex capacity: 230 gpm @ 150 feet TDH
Duplex capacity: 440 gpm
Operating Volume: 329 gallons
2 submersible, 23 hp (each)

PS-A (Existing Flows)	Hourly (mgd)	Hourly (gpm)
High Winter	3.63	2519.58
High Summer	1.15	799.14
Average Annual	0.52	361.91
Low Summer	0.18	125.24

PS-A (2040 Flows)	Hourly (mgd)	Hourly (gpm)
High Winter	5.81	4034.01
High Summer	1.81	1255.27
Average Annual	0.61	421.73
Low Summer	0.20	139.90

PS-B (Existing Flows)	Hourly (mgd)	Hourly (gpm)
High Winter	5.72	3970.77
High Summer	2.00	1391.76
Average Annual	0.86	594.08
Low Summer	0.25	175.67

PS-B (2040 Flows)	Hourly (mgd)	Hourly (gpm)
High Winter	7.98	5540.42
High Summer	2.66	1849.80
Average Annual	0.96	666.20
Low Summer	0.28	196.94

(Existing Flows)	Hourly (mgd)	Hourly (gpm)
High Winter	5.85	4059.75
High Summer	2.08	1443.01
Average Annual	0.90	622.77
Low Summer	0.27	187.91

(2040 Flows)	Hourly (mgd)	Hourly (gpm)
High Winter	8.14	5650.29
High Summer	2.74	1901.25
Average Annual	1.00	694.98
Low Summer	0.30	209.21

MH 8.21 (2040 Flows)	Hourly (mgd)	Hourly (gpm)
High Winter	0.88	611.11
High Summer	0.16	111.11
Average Annual	0.08	55.56
Low Summer	0.02	13.89

LS2 (Existing Flows)	Hourly (mgd)	Hourly (gpm)
High Winter	1.22	847.02
High Summer	0.68	474.57
Average Annual	0.25	176.62
Low Summer	0.05	36.54

LS2 (2040 Flows)	Hourly (mgd)	Hourly (gpm)
High Winter	1.29	895.30
High Summer	0.70	483.43
Average Annual	0.27	188.91
Low Summer	0.06	43.15

LS3 (Existing Flows)	Hourly (mgd)	Hourly (gpm)
High Winter	0.13	88.98
High Summer	0.07	51.25
Average Annual	0.04	28.69
Low Summer	0.02	12.25

LS3 (2040 Flows)	Hourly (mgd)	Hourly (gpm)
High Winter	0.16	109.87
High Summer	0.07	51.45
Average Annual	0.04	28.78
Low Summer	0.02	12.27

KEY:
- MB4 – CITY OF MORRO BAY FLOW MONITORING SITE
- 1 – V&A FLOW MONITORING SITE
- A03 – SEWER SHED
- LIFT STATION

NOTES:

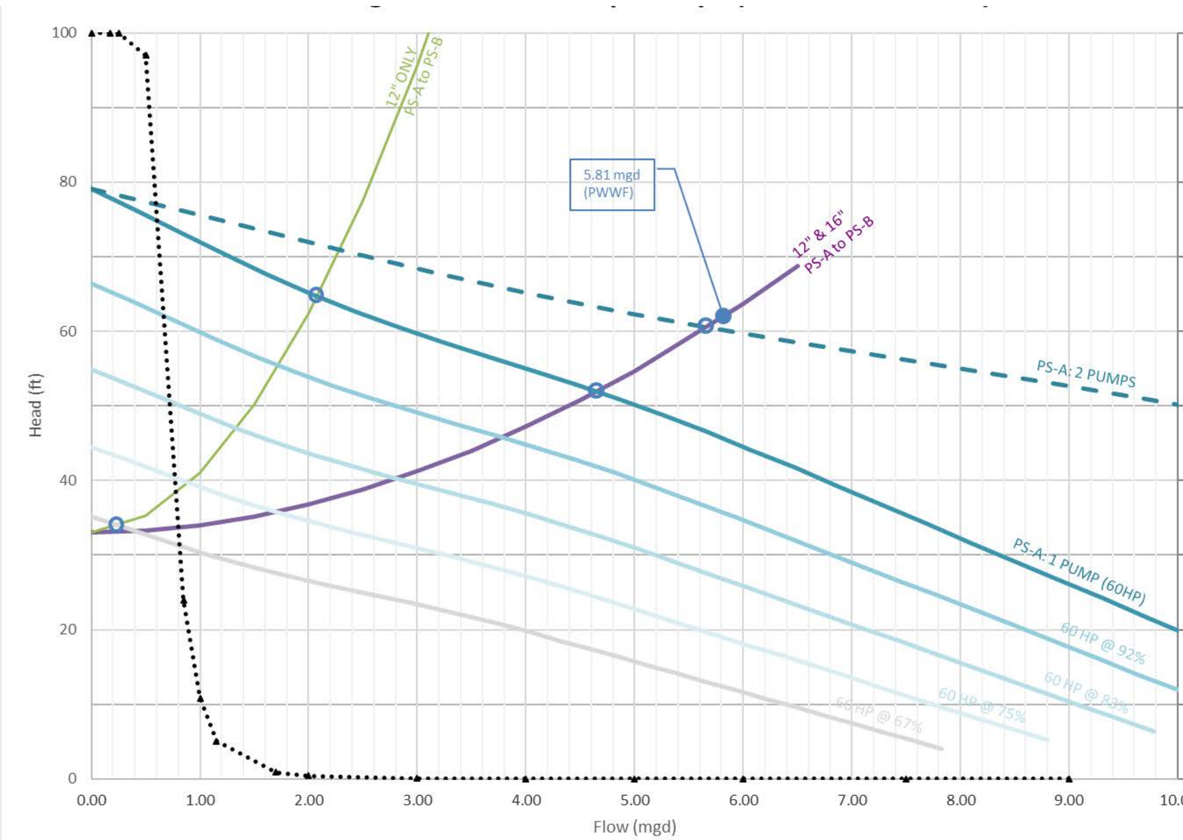
1. HOURLY ADW, PDW, PWW FLOWS WERE PROVIDED BY CAROLLO.
2. PWWF (HIGH WINTER) IS BASED ON A 10-YEAR, 24-HOUR MODEL STORM.
3. 2040 FLOWS INCLUDE CONSIDERATIONS FOR POPULATION GROWTH AS WELL AS THE IMPLEMENTATION OF CAPITAL IMPROVEMENT PROJECTS.

Figure 6-2: Scenario 3 Schematic and Flows

6.1.1 Pumping from PS-A to PS-B

For the sites analyzed on the West Alignment, the City-Owned parcel was identified as preferable as the City would not need to procure any new property. Figure 6-3 shows the range of flows that PS-A can achieve with 60-HP pumps on VFDs pumping to PS-B located at the City-owned parcel at Site W2 - Highway 1 and Main Street. Flows as low as 0.25 MGD up to 5.81 MGD can be pumped by PS-A.

Figure 6-3: Pumping from PS-A to PS-B on West Alignment



6.1.2 Pumping from LS-2 to PS-B

As shown in Figure 1-2: Proposed Process Flow Diagram with LS-3 Diversion, it is anticipated that LS-2 will have a build-out flow of 1.29 MGD. Diverting this flow to PS-B directly, rather than through PS-A will decrease PS-A flow to 5.81 MGD and PS-B will remain at 7.98 MGD. The previously described PS-A 140-HP pumps in the dual pump station scenario with a PS-B stormwater booster, are now reduced to 60-HP with the same 2+1 configuration. PS-B would remain unchanged with 2+1, 250-HP pumps.

Existing LS-2

Lift Station 2 is currently a 2+1 pump station with 10 HP pumps. The lift station capacity is currently 1,000 gpm when run as a duplex station. The original design TDH is 27 feet. The existing discharge forcemain that goes to the WWTP is a 12", though the station is set up to divert to an existing 8" ACP SSFM if needed.

New Forcemain

To divert flows from Lift Station 2 to PS-B, it was initially thought to use the existing 8” ACP SSFM and re-connect it to the proposed West Alignment. Upon further consideration, it was determined that since the condition of the existing 8” ACP SSFM is unknown and due to the desire to re-use the existing pumps at LS-2, a larger forcemain would be needed to get flows from LS-2 to PS-B. A 12” SSFM was determined to be best suited to pump buildout flows and allow for the continued use of the existing LS-2 pumps.

The existing 8" SSFM and associated 10-ft wide sewer easement through the Dynegy and PGE properties would be the preferred alignment to route the new 12" SSFM (see Chapter 5), but there are two significant constraints tied to this approach. First, the existing 8" ACP SSFM runs under an existing switchyard and electrical infrastructure within PGE’s property. WaterWorks anticipates that open-cut or trenchless construction for a new pipeline under this area would not be approved by PGE due to potential impacts and safety issues. The second constraint is that the existing 8" ACP SSFM is concrete encased under an existing drainage ditch. Open cut construction across the drainage ditch is anticipated to trigger Section 401/404/1602 permits, which would present significant risk to the overall schedule, and would likely cost upwards of $100,000 in permitting. The alternative would be to utilize trenchless construction, but this approach is not cost effective. Consequently, WaterWorks has identified a parallel but offset alignment that would run within the PGE property along the fence line (no impacts to PGE infrastructure), divert around the drainage area, and allow for connection with the West Alignment within the paved road. This preferred alternative is summarized below and in Figure 6-4 (along with Appendix A)

- Install 1,910-ft of 12" DR18 FPVC or 14" DR13.5 HDPE SSFM
- Quitclaim 742' existing 10-ft wide SSFM PE easement from PGE (under electrical substation), procure 782' of new 10-ft wide SSFM PE along PGE’s eastern fence line
- Quitclaim 421' existing 10-ft wide SSFM PE easement from Dynegy, procure 497' of new 10-ft wide SSFM PE
- Formally abandon 1,163' of existing 8" ACP SSFM in-place

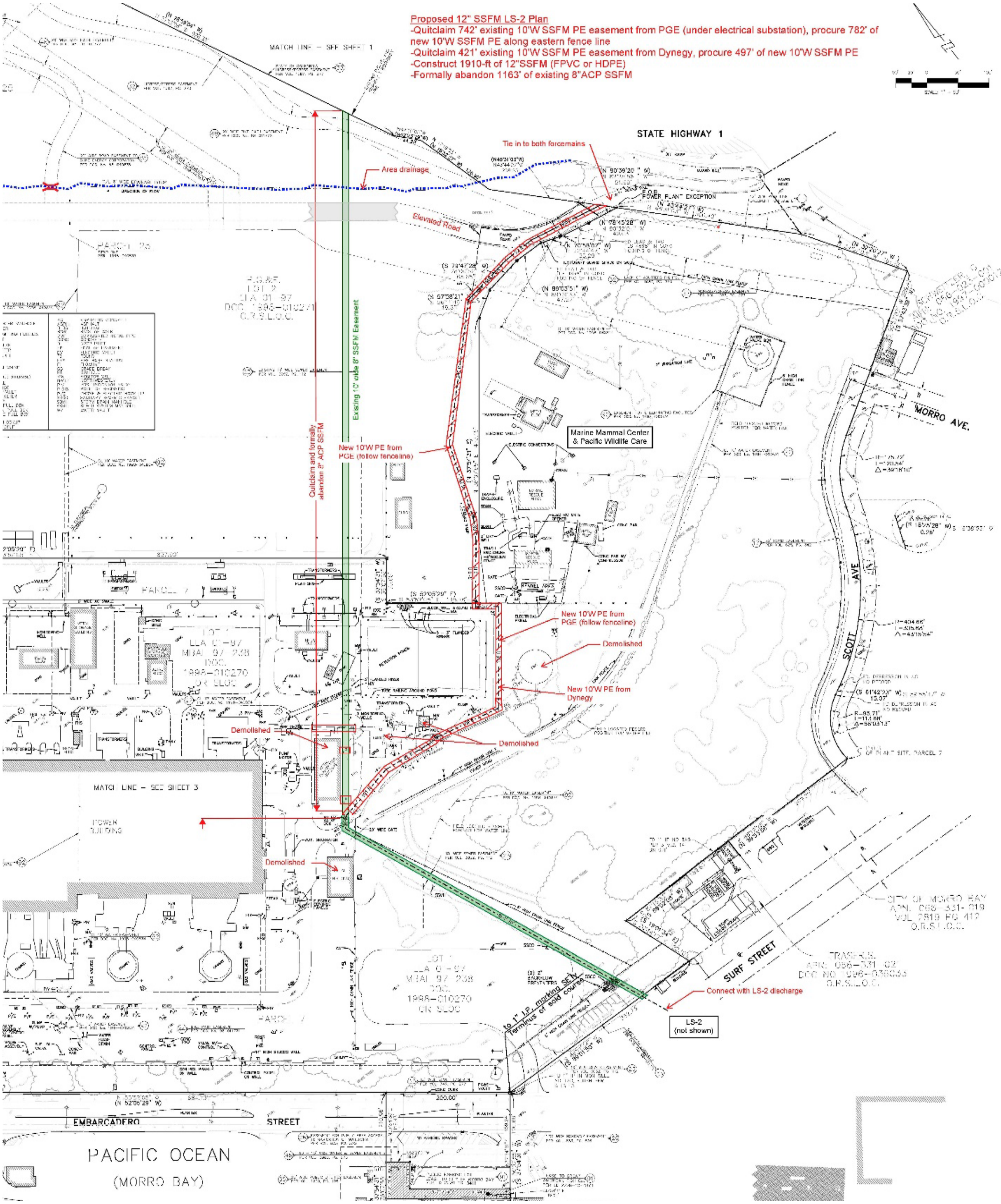


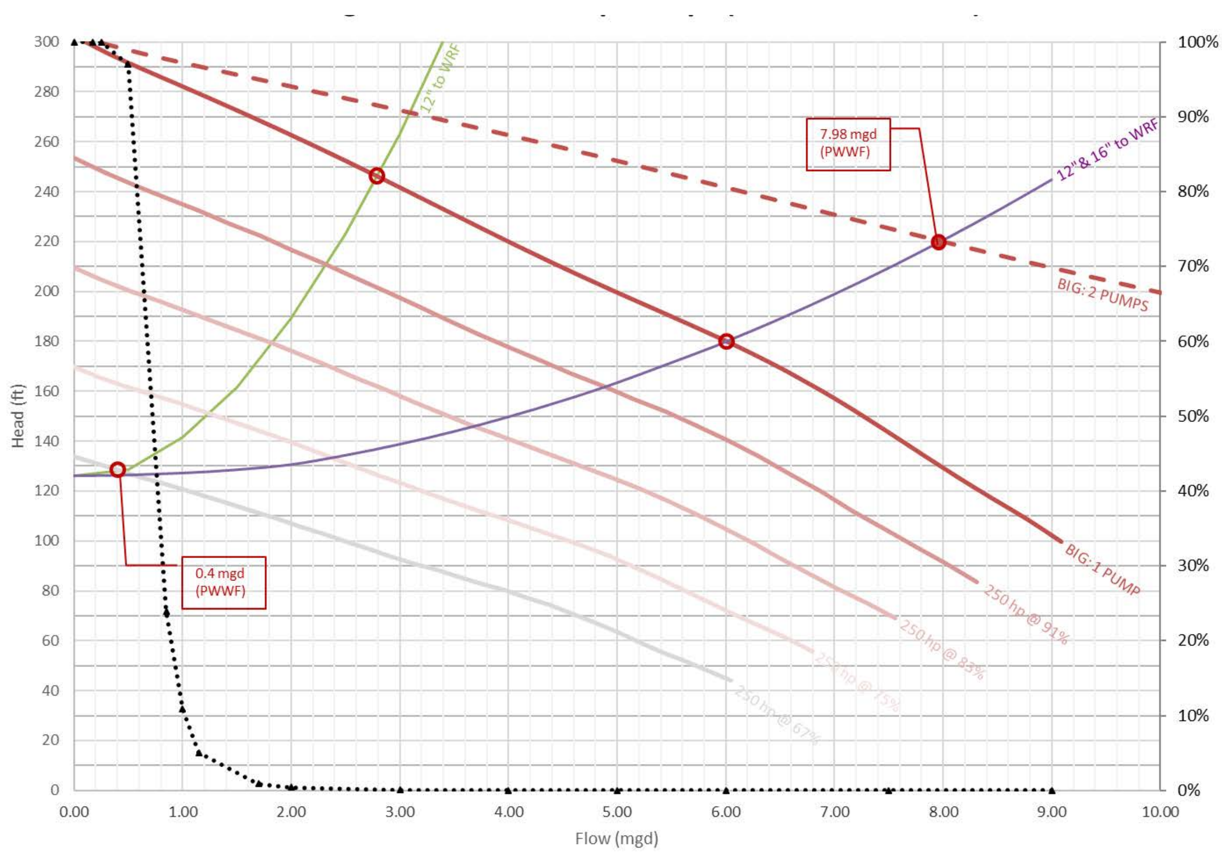
Figure 6-4: Existing 8" LS-2 Diversion and Replacement (Presented in greater detail in Appendix A)

The replacement of the existing 8" ACP with a new 12" forcemain would make the new 12" to PS-B the primary forcemain but still keep LS-2's ability to pump to PS-A as needed. With 60-HP pumps at PS-A in the full-time booster scenario, PS-A would not be able to handle PWWF from LS-2 but diverting lower flows would be an option.

6.1.3 Pumping from PS-to the WRF

Flow from both PS-A and PS-2 would be combined with local gravity flows at PS-B and all pumped together to the WRF. Figure 6-5 shows the range of flows for PS-B with VFDs as well as the flows at which the different size forcemains are used.

Figure 6-5: West Alignment PS-B (Dual Booster PS)

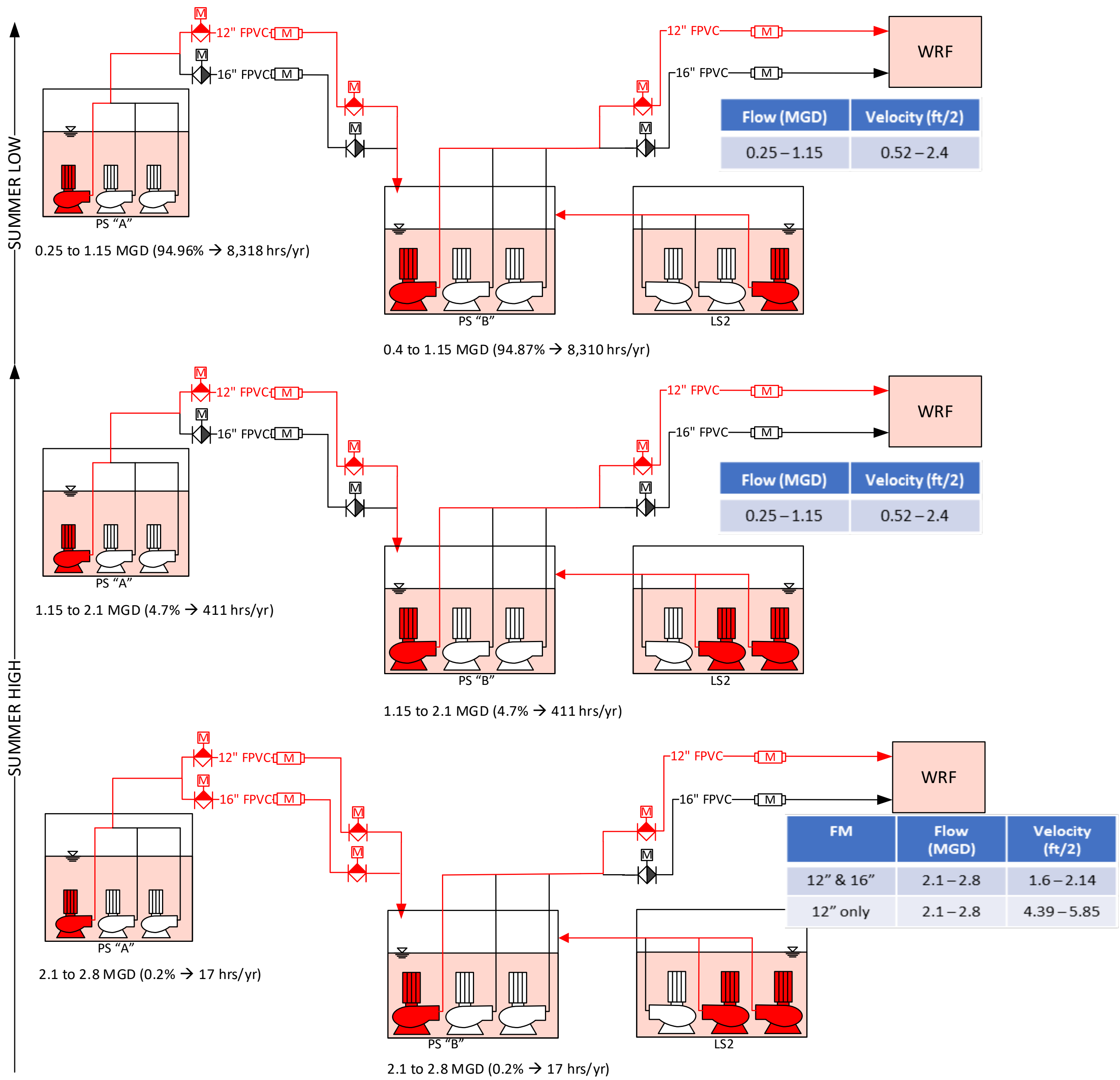


Operation of the full-time boosters would be partially contingent upon the flows and operations of Lift Station 2. Figure 6-6 shows the details of which pumps are used in conjunction with the forcemains and flows from Lift Station 2.

Flows between 0.25-1.15 MGD in the 12" forcemain see velocities between 0.52-2.4 ft/s. As flows get up to 2.1 MGD, velocities up to 4.39 ft/s can be achieved. A single 60-HP pumps at PS-A can pump flows up to 4.65 MGD. The second duty pump would be required to meet 5.81 MGD. A single 250-HP at PS-B can pump flows up to 6.0 MGD and the second duty pump would be needed to reach 7.98 MGD.

PS “A” PUMPS 60-HP

PS “B” PUMPS 250-HP



Flow (MGD)	Velocity (ft/2)
0.25 – 1.15	0.52 – 2.4

Flow (MGD)	Velocity (ft/2)
0.25 – 1.15	0.52 – 2.4

FM	Flow (MGD)	Velocity (ft/2)
12” & 16”	2.1 – 2.8	1.6 – 2.14
12” only	2.1 – 2.8	4.39 – 5.85

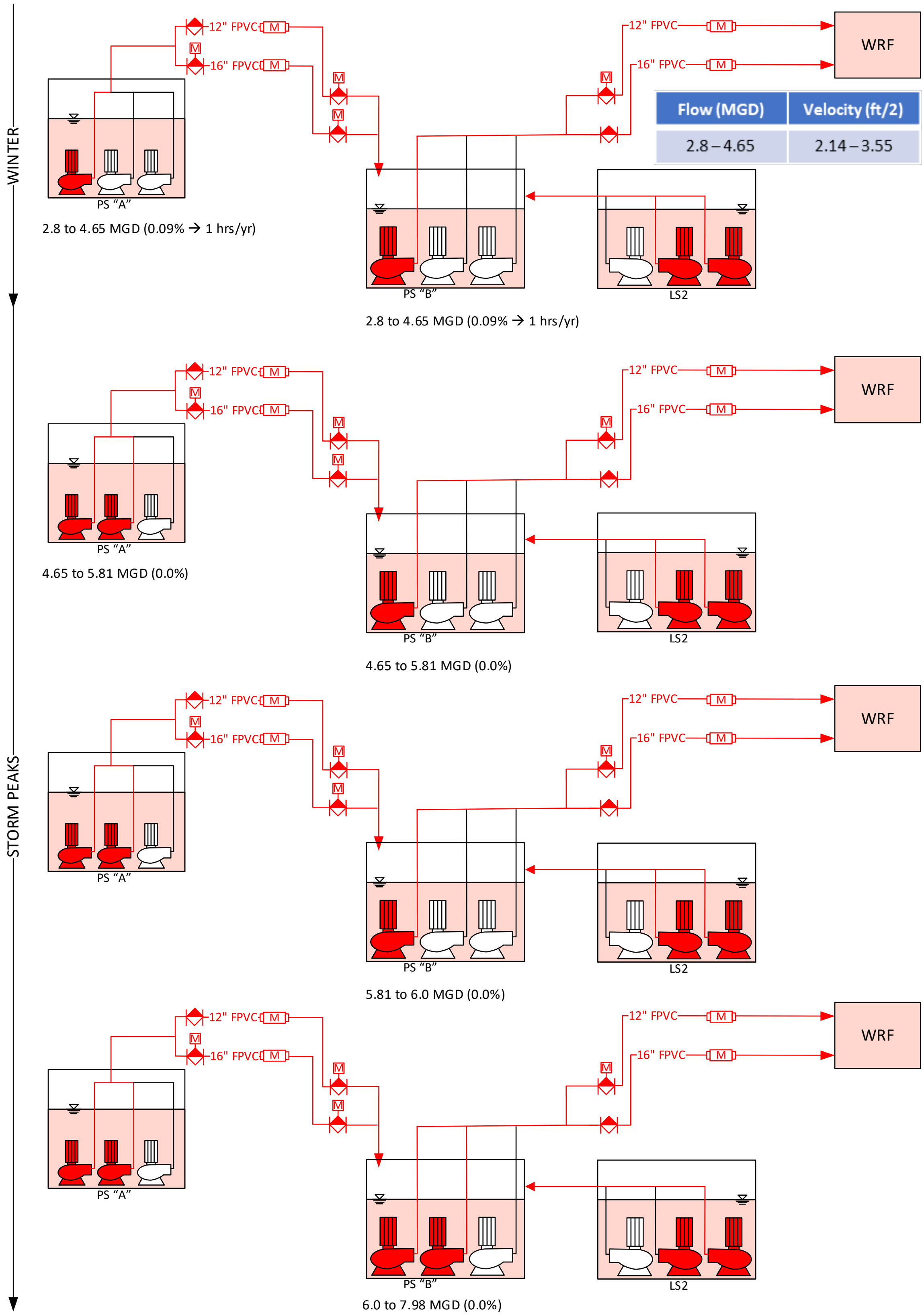

WINTER
STORM PEAKS
12" FPVC
16" FPVC
M
WRF
Flow (MGD)
Velocity (ft/2)
2.8 – 4.65
2.14 – 3.55
PS "A"
PS "B"
LS2
2.8 to 4.65 MGD (0.09% → 1 hrs/yr)
2.8 to 4.65 MGD (0.09% → 1 hrs/yr)
4.65 to 5.81 MGD (0.0%)
4.65 to 5.81 MGD (0.0%)
5.81 to 6.0 MGD (0.0%)
6.0 to 7.98 MGD (0.0%)

Figure 6-6: Scenario 3, Dual Pump Station, Full Time Boosters Operations

6.1.4 PS-A and PS-B Pump Station Preliminary Layouts and Site Plan

3-D renderings of PS-A and PS -B have been generated at their preferred locations and are included in this section. These will be further developed in the final design process.

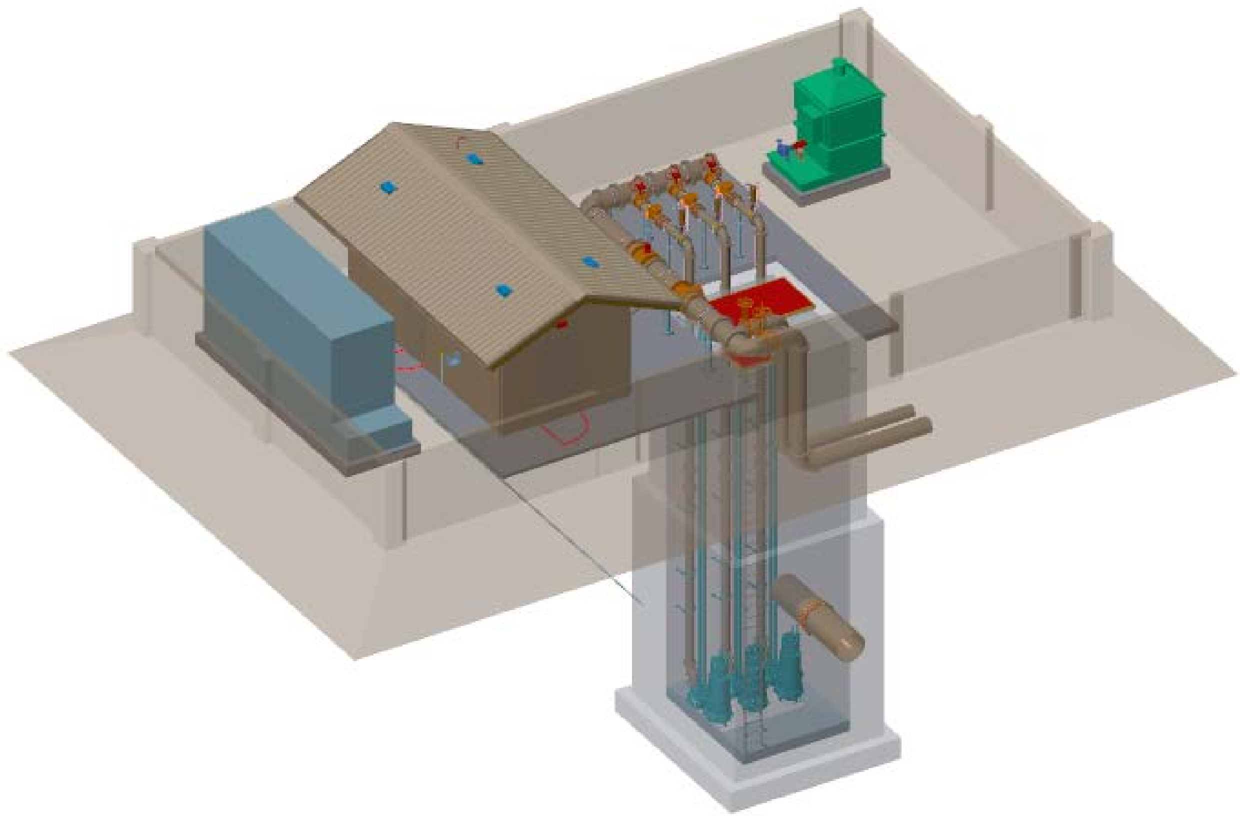


Figure 6-7: PS-A 3-D Rendering

Figure 6-8: PS-A at PS-A Site 2

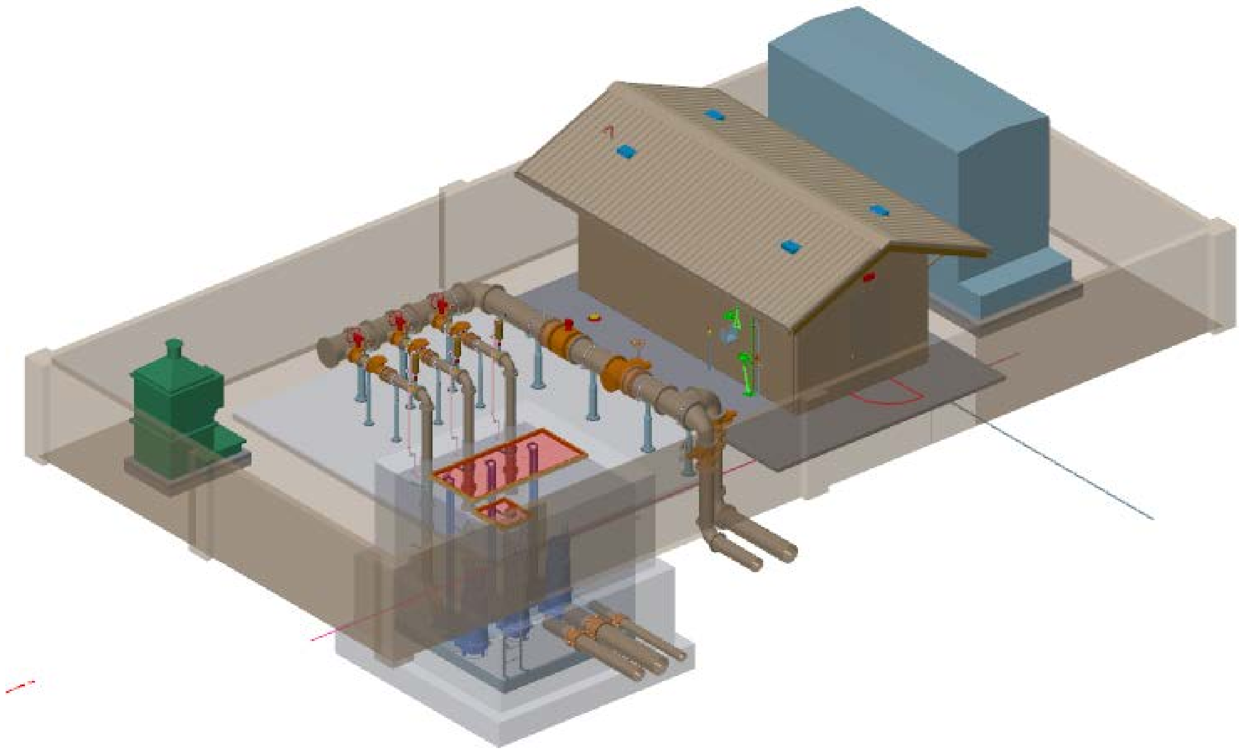


Figure 6-9: PS-B 3-D Rendering

Figure 6-10: PS-B at West SIte 2

6.1.5 Additional Considerations for Scenario 3

To switch PS-B from a stormwater booster pump station to a full-time booster pump station, several additional pump station design considerations need to be added as summarized below:

- Less valving is required as there is no longer a need for PS-A to pump direct to the new WRF
- Odor control would be needed at PS-B as there would constantly be wastewater in the PS-B wet well
- Full-time operation costs for using PS-B would be offset by the smaller pumps at PS-A
- Pigging from PS-A to PS-B and from PS-B to new WRF

6.1.6 Scenario 3 Costs

This section summarizes the Scenario 3 costs: direct and indirect (operational and maintenance). Detail cost estimates can be found in APPENDIX F: Preferred Alternative Pump Station Costs.

Table 6-1: Scenario 3 Direct Construction Costs

DIRECT CONSTRUCTION COSTS		
	SCENARIO 3	
	PS-A	PS-B
Site Work	$493,000	$239,200
Piping, Valves, Fitting & Appurtenances	$155,400	$512,100
Equipment	$516,500	$716,300
Structural	$366,600	$272,400
Electrical	$1,111,800	$1,942,500
Project Subtotal 1	**$2,643,300**	**$3,682,500**
2% Design Contingency	$528,660	$736,500
TOTAL CONSTRUCTION BID COST	$3,171,960	$4,419,000
10% Construction Contingency	$317,196	$441,900
TOTAL PROJECT CONSTRUCTION COST	$3,489,156	$4,860,900
TOTAL PROJECT COMPARISON (nearest $10,000)	***$8,400,000***	

Table 6-2: Scenario 3 Operational Costs

OPERATIONAL COSTS (2018 dollars, nearest $1,000)	
	SCENARIO 3
	$/year
Total Annual Pumping Cost	$45,000
Ancillary Power Usage	$6,000
Odor Control Media Replacement	$5,000
Mechanical Maintenance	$27,000
	$83,000

Table 6-3: Scenario 3 Replacement Funds

REPLACEMENT FUNDS	
	SCENARIO 3
	$/year*
Recoating & Repainting	$4,000
Pump Replacement	$32,000
Switchgear/VFD Replacement	$121,000
	$157,000

*Assumes annualized cost of 20-year replacement program

6.1.7 Preferred Pumping Scenario Selection

After establishing the West Alignment as the preferred forcemain route and setting a preference for pump station locations, the three pump station scenarios that have been developed and discussed are as follows:

- **Scenario 1**: Single Pump Station
- **Scenario 2**: Secondary Stormwater PS-B
- **Scenario 3**: Full-Time PS-B

The following table re-summarizes pump station utilization in each scenario during different flow conditions:

Table 6-4: Pump Station Utilization

Scenario	1	2	3
Description	Single Station	Stormwater Booster (PS-B)	Full-Time Boosters
Non-Storm Flows	LS-2 → PS-A PS-A → WRF LS-3 → WRF	LS-2 → PS-A PS-A → WRF LS-3 → WRF	LS-2 → PS-B or PS-A PS-A → PS-B PS-B → WRF LS-3 →WRF
Storm Conditions	LS-2 → PS-A PS-A → WRF LS-3 → WRF	LS-2 → PS-A PS-A → PS-B PS-B → WRF LS-3 → WRF	LS-2 → PS-B PS-A → PS-B PS-B → WRF LS-3 →WRF

The decision matrices that follow are intended to the effects of choosing one scenario over another under different criteria: facility impacts, pump maintenance and reliability, forcemain impact and cost. They are color coded to aide in indicating which items under specific scenarios are the most beneficial to the City (green = best; red = worst)

Facility maintenance and public impacts of each scenario are summarized here:

Table 6-5: Facility Maintenance Impacts Decision Matrix

Scenario	1	2	3
# of New Stations	1	2	2
PS-A Footprint	Large	Medium	Medium
PS-B Footprint	None	Medium	Medium
Odor Control	One Site	One Site	Two Sites

Pump maintenance and reliability are summarized here:

Table 6-6: Pump Maintenance and Reliability Decision Matrix

Scenario	1	2	3
# of New Pumps			
PS-A (Small)	2 (60 HP)	3 (140 HP)	3 (60 HP)
PS-A (Large)	6 (250 HP)	n/a	n/a
PS-B (Large)	n/a	3 (250 HP)	3 (250 HP)
Total	**8**	**6**	**6**
# of Idle Pumps during Non-Storm Flows	1 - PS-A Small 6 - PS-A Large	0 - PS-A 3 - PS-B	0 - PS-A 0 - PS-B
Size of Duty Pumps	Smaller	Larger	Smaller
Pump Cycling	Less	More	More
Control Complexity	Medium	Complex	Simple

A summary of the impact on forcemain operations is presented here:

Table 6-7: Forcemain Impact Decision Matrix

Scenario	1	2	3
Forcemain Velocities	Low	Higher	Higher
Pipe Length for Pigging	Longer	Shorter	Shorter
LS-2 FM Redundancy	No	No - Could be added ($400k)	Yes

The cost comparison for these scenarios is as follows:

Table 6-8: Cost Comparison Decision Matrix

Scenario	1	2	3
Estimated Capital Cost	$11.0M	$8.4M	$8.4M
Estimated O&M Cost	$59K	$70K	$83K
Replacement Funds	$230K	$171k	$157K
Estimated 20-yr NPW (O&M + Replacement Funds)	$3.6M	$3.0M	$3.0M
Total NPW	**$15.1M**	**$11.8M**	**$11.7M**

The cost of Scenario 3 is very similar to the cost of Scenario 2, both in capital and in long term Operation and Maintenance costs. The fact that Scenario 3 keeps all pump stations operating all year is a significant benefit to Scenario 3 in terms of resilience and reliability of the system as a whole to be ready for and able to pump peak storm related flows through the system when called upon to do so. For that reason, Scenario 3 is the final recommended pumping alternative.

7 TOTAL PROJECT COST AND PREFERRED ALTERNATIVE

WaterWorks recommends the West Scenario 3 with dual forcemains (FPVC or HDPE) as the preferred project alternative for City approval. A summary of why this alternative best fits the overall project goals is listed below:

- Least expensive alternative that incorporates redundancy via dual forcemain and maximizes long-term operability and ease of maintenance.
- Maximizes use of existing City right-of-way or easements via the West Alignment. Reduces traffic impacts by utilizing a trenchless crossing of the Morro Bay/Quintana roundabout.
- Avoids known cultural resources in the Lila Keiser Park area via the Caltrans parallel encroachment on the SB HWY-1 connector shoulder.
- Leverages local flow diversions via two new pump stations which provide the best flow range per pump station and optimizes pump sizing. In addition, potentially reduces or eliminates sanitary sewer capacity improvement projects on Main St. due to the LS-2 diversion. In addition, the alternative eliminates double-pumping of LS-3 flows and provides a new LS-2 forcemain.

7.1 Offsite Pipelines Preferred Alternative

The preferred alignment alternative is the 8”IPR-12”FM-16”FM-16”Brine FPVC (or 8”IPR-14”FM-20”FM-20”Brine HDPE) West Alignment. In addition, a communication conduit will be installed within the same trench as the offsite pipelines for fiber optic cable installation by the WRF Program Team.

A preliminary plan and profile of the preferred alternative along with the potential East and West IPR lines is displayed in APPENDIX A: 30% Plan & Profile of Preferred Alternative Alignment.

Due to parallel design work on the overall project between the WRF Program Team (Injection Wells, WRF, IPR PS, Brine PS) and WaterWorks (Offsite Pipelines and Pump Stations), there are many coordination items that need to be resolved during the design phase.

7.1.1 WRF Onsite Pipeline Alignment Coordination

The preliminary hydraulic analysis presented herein which informs current pump station sizing is based off several assumptions that require further confirmation with the WRF Program Team during the design phase. These are listed below:

- The approximate length of the onsite forcemains from Teresa to the discharge location is 2300-lf
- The worst-case discharge elevation at the headworks is 143.5 feet (ft)
- WaterWorks assumes pipe material will be the same onsite as offsite. If alternative material is desired / requested by the DB team, these will require review and approval by WaterWorks to confirm compatibility with offsite design hydraulics and operations.

7.1.2 IPR Site Coordination

The WRF Program Team is anticipated to select the IPR injection well locations during the start of the design phase. It is important to note that selecting the East IPR would require a Caltrans longitudinal encroachment permit which would be included in the overall project Caltrans permitting effort. As the entire permit must be completed and sent as one package, early confirmation of pertinent design criteria that affects the pipeline in Caltrans ROW is critical. In summary, the items that need confirmation are the following:

- Injection well area
- Common tie in location (i.e., how far should the IPR main encroach into the proposed injection well study areas?)

7.1.3 Brine and IPR Hydraulic Coordination

The WRF Program Team will be incorporating the Brine and IPR pump stations into the overall WRF site and control the pump selection. Due to this constraint, WaterWorks cannot conduct a design-level hydraulic analysis on the pipelines, and more importantly, size the surge and air release valves. Therefore, it is important to note that many items need to be coordinated and reviewed with the WRF Design Build Team during the Offsite Pipelines design phase. The items are summarized below:

- Final required size
- It is assumed that the pipe material will match offsite pipelines to achieve better economy of scale/pricing (FPVC or HDPE)
- Surge/CARV design criteria based on WRF Program Team hydraulic assessment

7.1.4 Communication Conduit Coordination

WaterWorks will also coordinate additional design criteria with the WRF Program Team with regards to installing the communication conduit along with the offsite pipelines. It is understood that WaterWorks will install the pipeline but the fiber optics cable will be installed by the WRF Program Team. The following items needs to be confirmed:

- Number of conduits
- Location of conduit in trenchline
- Size and material of conduit
- Frequency of junction boxes and standard details
- SCADA, communications, and instrumentation standards (pump stations)

Again, the addition of the communication conduit within the overall offsite pipelines Caltrans permit means that items listed above need to be discussed in a timely manner to not impact schedule.

7.1.5 Environmental Assessments

It is anticipated that further coordination with the WRF Program Team to review any additional biological/cultural assessments, draft environmental permit applications, and support documentation during construction will be conducted during the design phase.

7.1.6 Potholing and Additional Utility Research

Extensive utility potholing and/or ground penetrating radar (along with convention wire tracing) is anticipated during the design phase for long sections of Quintana and near the roundabout area. These items will be coordinated with the WRF Program Team.

7.2 Offsite Pump Stations Preferred Alternative

The preferred configuration is a dual pump station scenario with PS-A near the existing WWTP and PS-B located at the City-owned parcel on Main Street at Highway 1. This scenario would require the diversion of flows from LS-2 as well as gravity flows local to PS-B (MH 8.21). PS-A would house 2 duty + 1 standby 60-HP pump and PS-B would have 2+1 250-HP pumps and both would operate as full-time booster pump stations – PS-A pumping to PS-B, then PS-B pumping to the new WRF.

PS-A flows could potentially be further reduced by taking additional flow from MH 13.25. The impact of diverting these additional gravity flows should be modeled as they could affect the need for planned capital improvement projects (CIP) downstream of Highway 1 and Main Street.

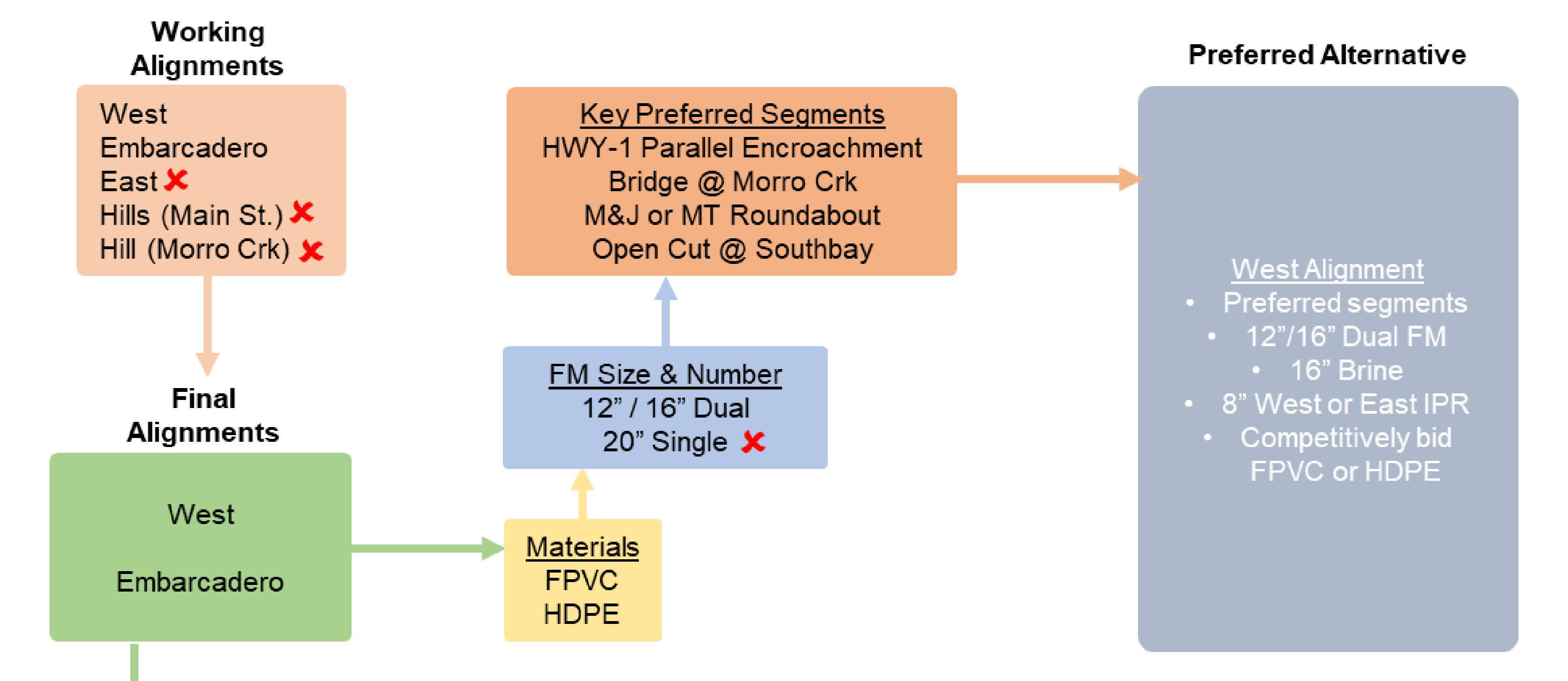
7.3 Total Project Cost

The total project costs for the Offsite Pipelines and Pump Stations (including 20-year O&M and Replacement costs) are listed in Table 7-1 below. An overview figure of the preferred alternative assessment process and results is presented in the following figure.

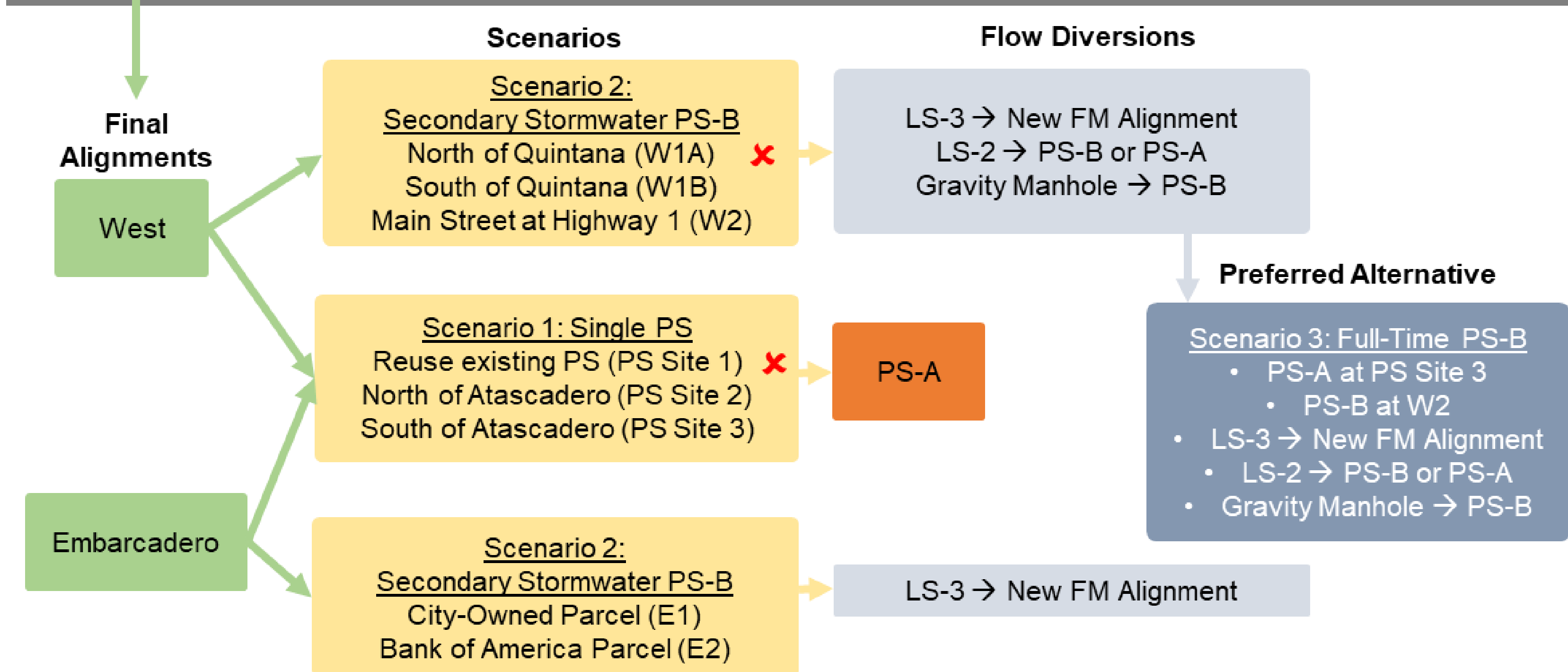
Table 7-1: WRF Offsite Pipelines and Pump Station Total Project Costs

WRF Offsite Pipelines and Pump Station Total Project Costs*						
Alignment	Item	Offsite Pipelines + Conduit	Pump Station	Additional West 8” IPR	Additional East 8” IPR	Comm. Conduit
West	12"FM + 16"FM + 16"Brine DR 18 FPVC	$ 12,614,700	-	$14,814,700	$15,784,700	$414,700
	14"FM + 20"FM + 20"Brine DR 13.5 HDPE	$ 13,874,700	-	$16,184,700	$17,147,700	$414,700
	Pump Station Estimated Capital Cost	-	$ 8,400,000	-	-	-
	Pump Station Estimated 20-yr NPW (O&M + Replacement Funds)	-	$ 3,004,000	-	-	-

Reflects 20% construction & 10% design contingency applied to direct construction costs

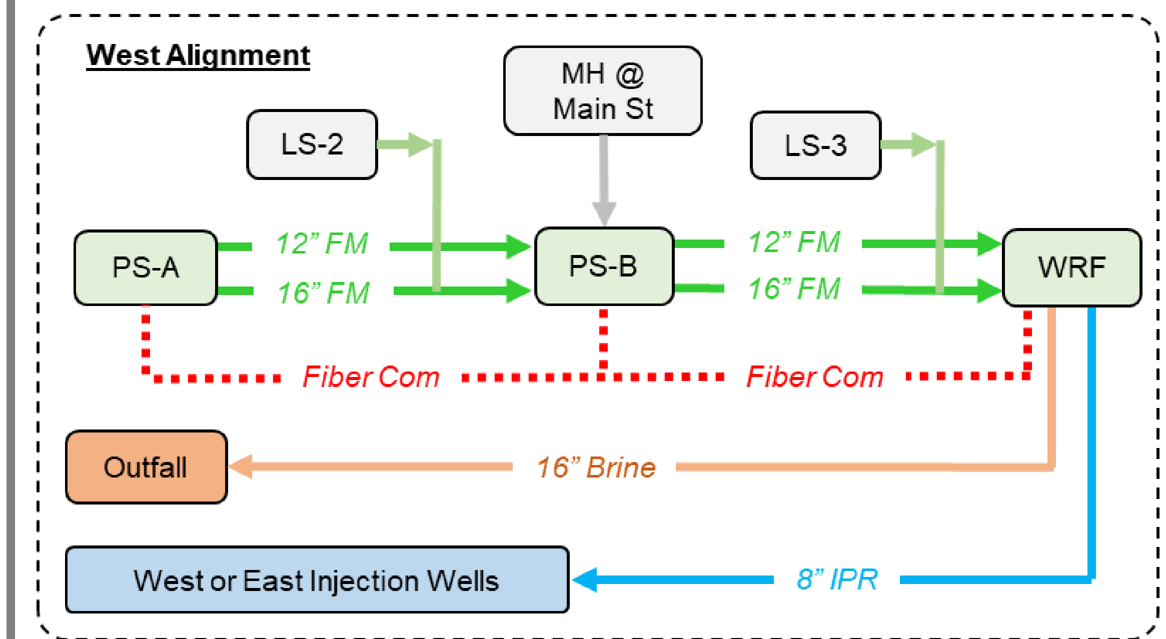


Pipeline Decision Diagram



Pump Station Decision Diagram

Preferred Pump Stations and Offsite Pipelines Alternative



7.4 Construction Bid Package

Early in the design phase WaterWorks will coordinate with the City and confirm the construction bid package strategy for the offsite pipelines and pump stations project. There are two options for the bid package: A) provide a combined bid package whereas as a single Contractor would bid on the Pump Stations and Pipelines, or B) two separate Contractors bid on the pump stations and pipelines separately. There several advantages and disadvantages to both approaches that are discussed in Table 7-2 below.

Table 7-2: Project Bid Package Options

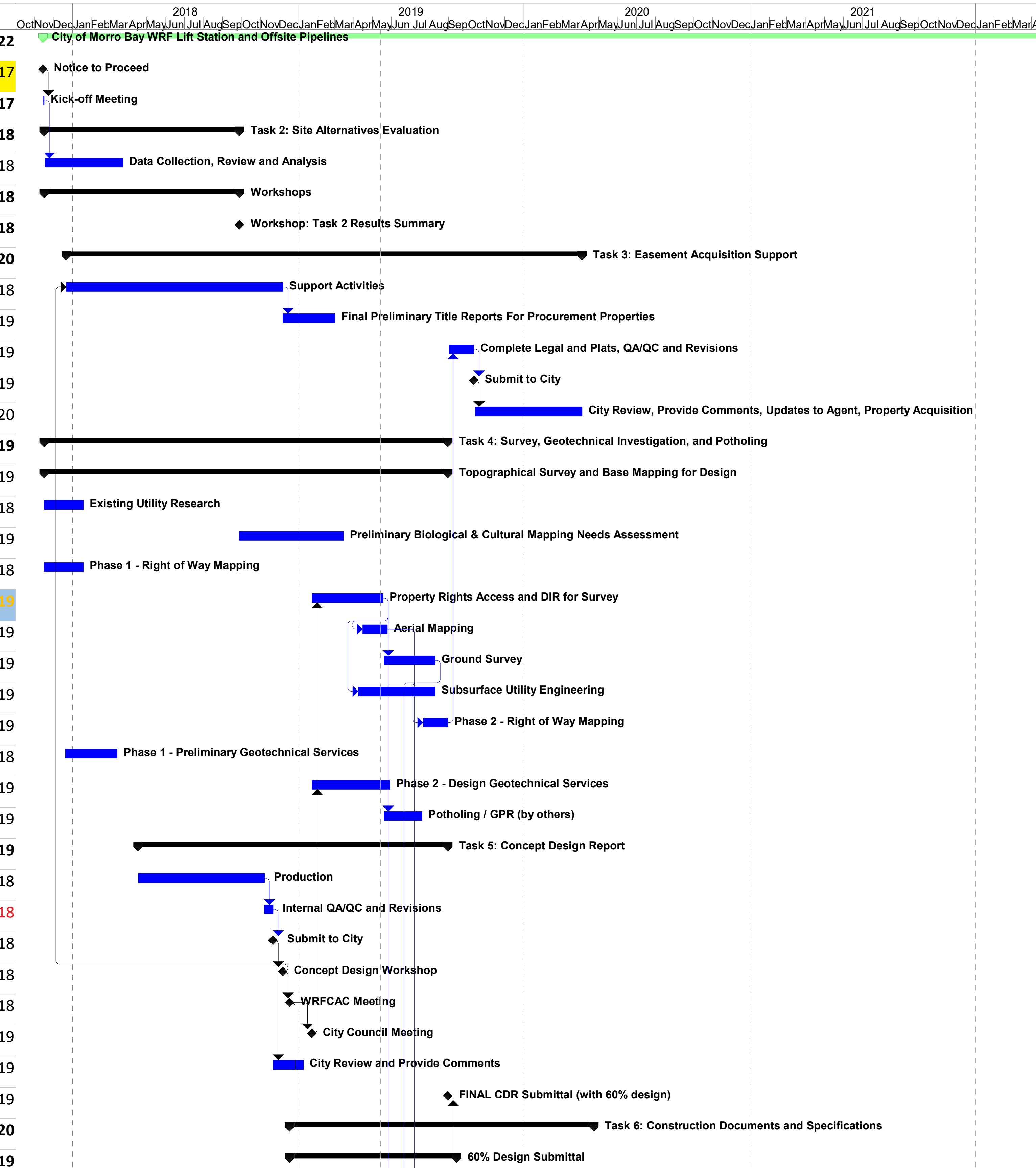
Impacts	Combined Bid Package	Separate Bid Packages
Bidding Environment	Likely decreased pool of contractors; potentially larger bid costs; incentivizes larger regional or national contractors	Improved due to larger pool; may decrease bid costs; leverages smaller contractors that specialize in civil pipeline or structural/mechanical work; may incentivize local and smaller regional contractors
City Staff	Ideal	Moderate increase in project coordination/communication (more parties involved); more permits
Construction Management (assumes single CM for entire project)	Ideal	Significant increase in project coordination/communication & documentation
Eng. Services During Construction	Ideal	Moderate increase
Environmental Permits & Compliance	Ideal	Moderate increase
Construction Schedule/Sequencing	Assumed same impact	

7.5 Construction Schedule and Phasing

Upon final City approval of the preferred pump station and offsite pipelines alternatives, WaterWorks will work to deliver the 60% PS&E. The updated project schedule is reflected in the following figures along with anticipated construction schedule and sequencing plan which provides additional details about schedule constraints.

WRF Lift Station and Offsite Pipelines Project Schedule

ID	Task Name	Duration	Start	Finish
1	**City of Morro Bay WRF Lift Station and Offsite Pipelines**	**1159 days**	**Tue 11/14/17**	**Fri 4/22/22**
2	Notice to Proceed	0 days	Tue 11/14/17	Tue 11/14/17
3	**Kick-off Meeting**	**1 day**	**Wed 11/15/17**	**Wed 11/15/17**
4	**Task 2: Site Alternatives Evaluation**	**226 days**	**Wed 11/15/17**	**Thu 9/27/18**
5	Data Collection, Review and Analysis	90 days	Fri 11/17/17	Thu 3/22/18
6	**Workshops**	**226 days**	**Wed 11/15/17**	**Thu 9/27/18**
19	**Workshop: Task 2 Results Summary**	**0 days**	**Thu 9/27/18**	**Thu 9/27/18**
22	**Task 3: Easement Acquisition Support**	**596 days**	**Fri 12/22/17**	**Fri 4/3/20**
23	Support Activities	250 days	Fri 12/22/17	Thu 12/6/18
24	Final Preliminary Title Reports For Procurement Properties	60 days	Fri 12/7/18	Thu 2/28/19
25	Complete Legal and Plats, QA/QC and Revisions	30 days	Mon 9/2/19	Fri 10/11/19
26	Submit to City	0 days	Fri 10/11/19	Fri 10/11/19
27	City Review, Provide Comments, Updates to Agent, Property Acquisition	125 days	Mon 10/14/19	Fri 4/3/20
28	**Task 4: Survey, Geotechnical Investigation, and Potholing**	**467 days**	**Thu 11/16/17**	**Fri 8/30/19**
29	Topographical Survey and Base Mapping for Design	467 days	Thu 11/16/17	Fri 8/30/19
30	Existing Utility Research	45 days	Thu 11/16/17	Wed 1/17/18
31	Preliminary Biological & Cultural Mapping Needs Assessment	120 days	Fri 9/28/18	Thu 3/14/19
32	Phase 1 - Right of Way Mapping	45 days	Thu 11/16/17	Wed 1/17/18
33	Property Rights Access and DIR for Survey	83 days	Wed 1/23/19	Fri 5/17/19
34	Aerial Mapping	30 days	Mon 4/15/19	Fri 5/24/19
35	Ground Survey	60 days	Mon 5/20/19	Fri 8/9/19
36	Subsurface Utility Engineering	90 days	Mon 4/8/19	Fri 8/9/19
37	Phase 2 - Right of Way Mapping	30 days	Mon 7/22/19	Fri 8/30/19
38	Phase 1 - Preliminary Geotechnical Services	60 days	Wed 12/20/17	Tue 3/13/18
39	Phase 2 - Design Geotechnical Services	90 days	Wed 1/23/19	Tue 5/28/19
40	Potholing / GPR (by others)	45 days	Mon 5/20/19	Fri 7/19/19
41	**Task 5: Concept Design Report**	**359 days**	**Tue 4/17/18**	**Fri 8/30/19**
42	Production	146 days	Tue 4/17/18	Tue 11/6/18
43	Internal QA/QC and Revisions	10 days	Wed 11/7/18	Tue 11/20/18
44	Submit to City	0 days	Tue 11/20/18	Tue 11/20/18
45	Concept Design Workshop	0 days	Thu 12/6/18	Thu 12/6/18
46	WRFCAC Meeting	0 days	Mon 12/17/18	Mon 12/17/18
47	City Council Meeting	0 days	Tue 1/22/19	Tue 1/22/19
48	City Review and Provide Comments	35 days	Wed 11/21/18	Tue 1/8/19
49	FINAL CDR Submittal (with 60% design)	0 days	Fri 8/30/19	Fri 8/30/19
50	**Task 6: Construction Documents and Specifications**	**352 days**	**Tue 12/18/18**	**Wed 4/22/20**
51	**60% Design Submittal**	**194 days**	**Tue 12/18/18**	**Fri 9/13/19**



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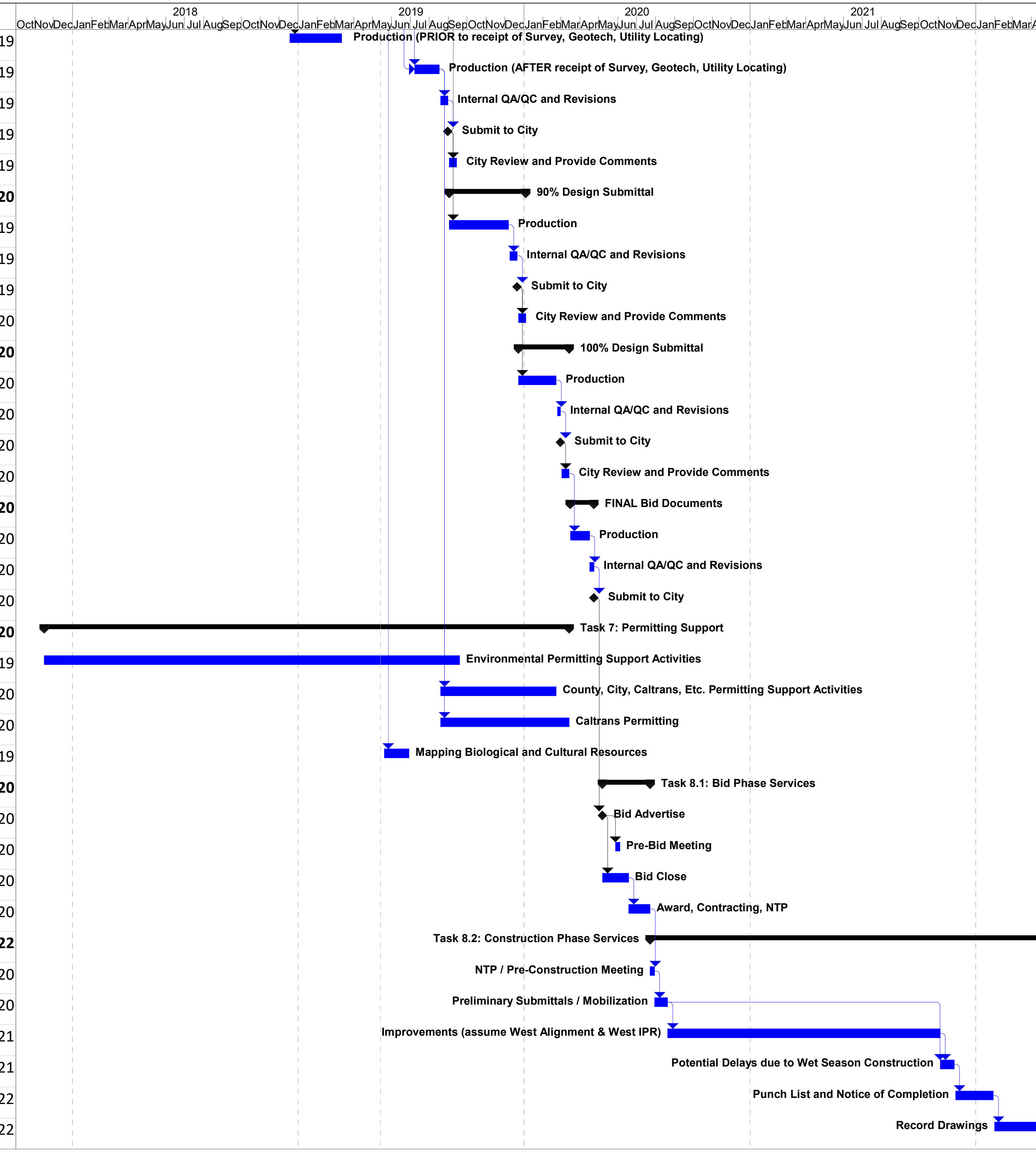
Task	Summary	External Milestone	Inactive Summary	Manual Summary Rollup	Finish-only
Split	Project Summary	Inactive Task	Manual Task	Manual Summary	Progress
Milestone	External Tasks	Inactive Milestone	Duration-only	Start-only	Deadline

WRF Lift Station and Offsite Pipelines Project Schedule

ID	Task Name	Duration	Start	Finish
52	Production (PRIOR to receipt of Survey, Geotech, Utility Locating)	60 days	Tue 12/18/18	Mon 3/11/19
53	Production (AFTER receipt of Survey, Geotech, Utility Locating)	30 days	Mon 7/8/19	Fri 8/16/19
54	Internal QA/QC and Revisions	10 days	Mon 8/19/19	Fri 8/30/19
55	Submit to City	0 days	Fri 8/30/19	Fri 8/30/19
56	City Review and Provide Comments	10 days	Mon 9/2/19	Fri 9/13/19
57	**90% Design Submittal**	**90 days**	**Mon 9/2/19**	**Fri 1/3/20**
58	Production	70 days	Mon 9/2/19	Fri 12/6/19
59	Internal QA/QC and Revisions	10 days	Mon 12/9/19	Fri 12/20/19
60	Submit to City	0 days	Fri 12/20/19	Fri 12/20/19
61	City Review and Provide Comments	10 days	Mon 12/23/19	Fri 1/3/20
62	**100% Design Submittal**	**60 days**	**Mon 12/23/19**	**Fri 3/13/20**
63	Production	45 days	Mon 12/23/19	Fri 2/21/20
64	Internal QA/QC and Revisions	5 days	Mon 2/24/20	Fri 2/28/20
65	Submit to City	0 days	Fri 2/28/20	Fri 2/28/20
66	City Review and Provide Comments	10 days	Mon 3/2/20	Fri 3/13/20
67	**FINAL Bid Documents**	**28 days**	**Mon 3/16/20**	**Wed 4/22/20**
68	Production	23 days	Mon 3/16/20	Wed 4/15/20
69	Internal QA/QC and Revisions	5 days	Thu 4/16/20	Wed 4/22/20
70	Submit to City	0 days	Wed 4/22/20	Wed 4/22/20
71	**Task 7: Permitting Support**	**607 days**	**Thu 11/16/17**	**Fri 3/13/20**
72	Environmental Permitting Support Activities	480 days	Thu 11/16/17	Wed 9/18/19
73	County, City, Caltrans, Etc. Permitting Support Activities	135 days	Mon 8/19/19	Fri 2/21/20
74	Caltrans Permitting	150 days	Mon 8/19/19	Fri 3/13/20
75	Mapping Biological and Cultural Resources	30 days	Mon 5/20/19	Fri 6/28/19
76	**Task 8.1: Bid Phase Services**	**55 days**	**Wed 5/6/20**	**Wed 7/22/20**
77	Bid Advertise	0 days	Wed 5/6/20	Wed 5/6/20
78	Pre-Bid Meeting	5 days	Thu 5/28/20	Wed 6/3/20
79	Bid Close	30 days	Thu 5/7/20	Wed 6/17/20
80	Award, Contracting, NTP	25 days	Thu 6/18/20	Wed 7/22/20
81	**Task 8.2: Construction Phase Services**	**457 days**	**Thu 7/23/20**	**Fri 4/22/22**
82	NTP / Pre-Construction Meeting	5 days	Thu 7/23/20	Wed 7/29/20
83	Preliminary Submittals / Mobilization	15 days	Thu 7/30/20	Wed 8/19/20
84	Improvements (assume West Alignment & West IPR)	315 days	Thu 8/20/20	Wed 11/3/21
85	Potential Delays due to Wet Season Construction	17 days	Thu 11/4/21	Fri 11/26/21
86	Punch List and Notice of Completion	45 days	Mon 11/29/21	Fri 1/28/22
87	Record Drawings	60 days	Mon 1/31/22	Fri 4/22/22



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Task	Summary	External Milestone	Inactive Summary	Manual Summary Rollup	Finish-only
Split	Project Summary	Inactive Task	Manual Task	Manual Summary	Progress
Milestone	External Tasks	Inactive Milestone	Duration-only	Start-only	Deadline

Const Item	Start Date	End Date	Duration (months)	Start STA	End STA	Length	Construction Segment	Schedule Constraints
1	8/1/2021	10/1/2021	1-2	10+00	14+50	450	Outfall Improvements & Tie In	None
2	6/15/2020	8/15/2020	1-2	14+50	26+75	1225	Atascadero to Caltrans	Signficant school impact - Summer Only
3	9/1/2021	10/31/2021	1-2	14+50			PS-A Tie-In	None
4	6/1/2021	8/30/2021	2-3	26+75		1476	East IPR - Hwy-1 Crossing - Hwy-41 Caltrans Encroachment	Reduced school impacts - Summer Preferred
5	5/1/2020	7/31/2021	2-3	26+75	33+00	625	SB Hwy-1 Connector Caltrans Encroachment	None
6	5/1/2020	10/31/2020	4-6	33+00	36+00	200	Morro Creek Bridge Crossing	Dry Weather Preferred To Reduce SWPPP/Dewatering
7	5/1/2020	10/31/2020	4-6	36+00	51+00	1500	Bike Path	Dry Weather Preferred To Reduce SWPPP/Dewatering; Sep-Feb Preferred for Nesting Season
8	7/1/2020	8/31/2020	1-2	39+75		500	West IPR	None
9	7/1/2020	8/31/2020	1-2	51+00	58+00	700	Bike Path Drainage Crossing + Caltrans Encroachment	Dry Weather Preferred To Reduce SWPPP/Dewatering
10	8/1/2020	9/30/2020	1-2	51+00		1910	LS-2 12" SSFM	None
11	8/1/2020	10/31/2021	1-2	58+00			PS-B Tie-In	Reduced commercial impacts - Mid October to Late May Preferred
12	9/1/2020	4/30/2021	4-6	58+00	94+00	3600	Lower Quintana (Main St to Roundabout)	Signficant commercial impacts - Mid October to Late May Only
13	10/1/2020	3/30/2021	3-5	94+00	101+00	700	Quintana Roundabout Crossing	Signficant commercial impacts - Mid October to Late May Only
14	5/1/2021	8/31/2021	3-4	101+00			LS-3 Bypassing	None
15	5/1/2021	8/31/2021	3-4	101+00	151+00	5000	Upper Quintana (Roundabout to South Bay)	None
16	9/1/2021	10/31/2021	1	150+00			LS-3 Tie In	None
17	7/1/2021	9/1/2021	1-2	151+00	161+00	1000	South Bay Blvd and Hwy-1 Crossing	None

WATERWORKS ENGINEERS

APPENDIX A: 30% Plan & Profile of Preferred Alternative Alignment

APPENDIX B: Right-of-way Map

APPENDIX C: Preliminary Geotechnical Report

APPENDIX D: Preliminary Offsite Pipeline Alignment Figures

APPENDIX E: Preferred Alternative Alignment Offsite Pipelines Costs

APPENDIX F: Preferred Alternative Pump Station Costs