FINAL

TECHNICAL MEMORANDUM

SAN FRANCISQUITO CREEK URBAN REACH 2 – TUNNEL DIVERSION FEASIBILITY STUDY

BLACK & VEATCH PROJECT NO. 423823
BLACK & VEATCH FILE NO. 423823.40

PREPARED FOR

SAN FRANCISQUITO CREEK JOINT POWERS AUTHORITY

28 JULY 2026

Reviewed by: ______________________________ ____________

Signature **Date**

____

Printed Name

Reviewed by: ______________________________ ____________

Signature **Date**

____

Printed Name

Professional Engineer: Mark Lander July 28, 2026

Signature **Date**

Mark Lander. P.E.

Printed Name

C34953, Expires 9/30/2027

License No.

Approved by: ______________________________ ____________

Signature **Date**

____

Printed Name

Table of Contents

LIST OF TABLES

LIST OF FIGURES

Limitations of Use

This report is issued for San Francisquito Creek Joint Powers Authority (SFCJPA) review and should not be distributed externally at this time.

EXECUTIVE SUMMARY

This report presents a preliminary evaluation of the feasibility of constructing an underground bypass tunnel to convey floodwaters, as part of the San Francisquito Creek (SFC) Reach 2 Project. This project aims to remove flood risk to the residents of Palo Alto, Menlo Park, and East Palo Alto from a 7,200 cfs storm, equivalent to a 70-year storm or the 1998 storm of record.

The SFCJPA released the Public Draft Alternatives Evaluation Report (WRA, 2025), which includes of the evaluation of four potential project alternatives consisting of channel widening, floodwalls, and modification to the Pope-Chaucer Bridge. After the release of this report, there has been interest from the community to understand if an underground tunnel could be constructed in place of other project elements to reach the project design goal of a 70-year storm, and if possible, the 100-year storm.

This scope of this study included:

- Review of past studies and bypass alignments considered
- Performance of a desk study/site reconnaissance to understand potential constraints
- Evaluation of tunnel alignments, feasibility of tunneling construction methods, operations and maintenance (O&M) considerations, and costs

For the purpose of this study, an existing creek capacity of 5,400 cfs was used, which was previously established from the Public Draft Alternatives Evaluation Report. Thus, the required hydraulic conveyance of the tunnel is calculated to be 1,800 cfs to meet the 70-year storm goal, and 3,100 cfs for the 100-year storm goal.

This study evaluated the viability of three tunnel construction methods: Tunnel Boring Machine (TBM) tunneling, Microtunneling, and Horizontal Directional Drilling (HDD). The following table summarizes each construction method and its viability to the Reach 2 Project.

Table ES-1 Summary of tunnel construction methods and viability for San Francisquito Creek

Considerations/ Tunneling Method	Method Description	Viability for SFC
TBM Tunneling	TBMs can be used to construct tunnels of up to 58 ft in diameter and can excavate tunnels through a variety of ground conditions. It uses a rotating cutter head at the front of the machine to dig and break up the earth, while simultaneously installing concrete segments to line the tunnel and support the surrounding ground. TBMs are ideal for long, large-diameter tunnel projects, but require large areas of workspace for TBM entry and exit (approximately 3.7 acres).	No. This is the only technology that could provide protection for both 70-year and 100-year storm but would require approximately 4 acres of land for input and output shafts, with a subset of this space permanently used for ongoing operations and maintenance. This open space is not available in or near the project area. Tunneling under private property requires significant property/easement acquisition, which may not be possible.

Considerations/ Tunneling Method	Method Description	Viability for SFC
Microtunneling	A Microtunnel Boring Machine (MTBM) is a remote-controlled pipe jacking method that installs pipelines of up to 10 ft diameter. This tunneling method is particularly well suited for urban environments where surface disruption must be minimized, and it is often used for sewer, stormwater, and water supply systems. Less workspace area is required for MTBM entry and exit: • Launch Shaft - 7,000 square feet (0.16 acres) • Receiving Shaft - 4,000 square feet (0.1 acres)	Yes, but requires further evaluation. Tunnelling would be significantly deeper than the existing infrastructure in the streets or other public right of way. With a 10' diameter pipe, microtunneling can accommodate approximately 33% – 39% of the excess flow for the 70-year event and 19% – 23% for the 100-year event, depending on alignment. Although it does not fully meet the project goal, it may be used in conjunction with other project elements to provide protection for a 70-year storm.
HDD	HDD is a surface-launched, trenchless method used primarily for installing pipelines and conduits beneath surface obstacles like rivers, roads, railways and buildings. The maximum pipe diameter is typically 4.5 ft. The laydown area required is 100 feet x 150 feet, plus a 50 feet wide corridor for staging the pipe on the surface.	No. HDD is limited to a 4.5' diameter pipe, which was found to provide less than 8% of the required capacity needed for the project goal. HDD is also generally not applicable to gravity-fed systems.

The study identified six potential alignments along public roadways that may be achievable through microtunneling. All alignments focused on diverting water from a location upstream of Pope-Chaucer Bridge (either Middlefield Road or Pope-Chaucer) and discharging it at a location downstream of Newell Road Bridge (either Newell Road or West Bayshore culvert), bypassing the section of the creek with the least capacity. Details and maps of these alignments are shown in Figure ES-1 and Appendix C – Aerial Views of Considered Tunnel Alignments.

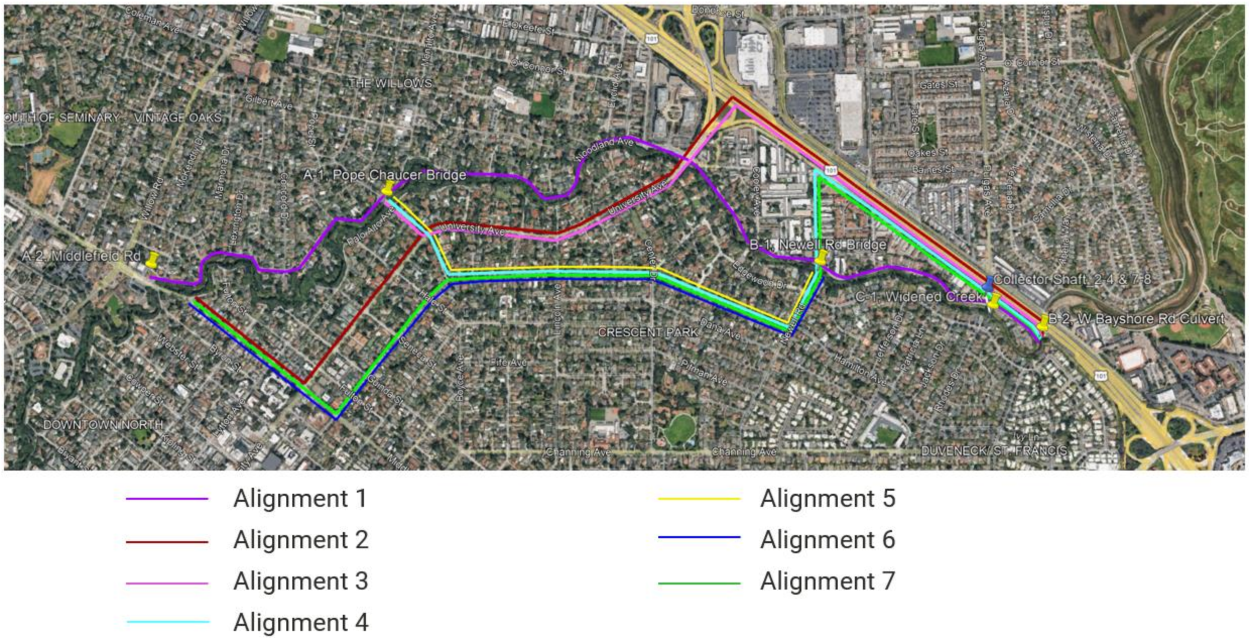


Figure ES-1 Summary of All Alignments

An alignment along Woodland Ave that follows the creek, or a route parallel to the creek on the south (City of Palo Alto) side of the creek was deemed infeasible due to the number and extent of curves, which could not be achieved using any construction method.

A summary of alignments and American Association of Cost Engineering (AACE) Level 5 cost estimates is provided in Table ES-2 below. Alignment 1 was not viable and was eliminated from further evaluation. The cost estimates were developed based on experience and benchmarks from projects of similar magnitudes and ground conditions.

Table ES-2 Summary of costs for potentially viable alignment options using microtunneling

Alignment ID	Description	# of Shafts	Tunnel capacity (cfs)	Percent of 70-year storm	Estimated Cost (Millions USD)
2 (Red)	Middlefield Rd (A-2) – University Ave – W Bayshore Rd – West Bayshore Rd Culvert (B-2)	8	703.3	39%	103
3 (Pink)	Pope Chaucer Bridge (A-1) – Chaucer St – University Ave - West Bayshore Rd – West Bayshore Rd Culvert (B-2)	7	636.5	35%	76
4 (Light blue)	Pope Chaucer Bridge (A1) – Chaucer St – Hamilton Ave – Newell Rd (B-1) - West Bayshore Rd – West Bayshore Rd Culvert (B-2)	8	598.0	33%	85

Alignment ID	Description	# of Shafts	Tunnel capacity (cfs)	Percent of 70-year storm	Estimated Cost (Millions USD)
5 (Yellow)	Pope Chaucer Bridge (A-1) – Chaucer St – Hamilton Ave – Newell Rd (B-1)	5	585.2	33%	51
6 (Blue)	Middlefield Rd (A-2) – Hamilton Ave – Newell Rd – West Bayshore Rd – Woodland Ave/Widened SF Creek (C-1)	6	705.2	39%	78
7 (Green)	Middlefield Rd (A-2) – Hamilton Ave – Newell Rd – West Bayshore Rd – West Bayshore Rd Culvert (B-2)	9	678.3	38%	112

Notes:

- Alternatives # 2, 3, 4, and 7 have the option of discharging water at Point C-1, near the intersection of West Bayshore Road and Woodland Avenue, allowing elimination or deferral of the lower 650' of West Bayshore Road tunnel. This option is shown on respective figures as a white colored alternate alignment running between the collector shaft and point C-1.
- All alternatives crossing the SF Creek will require more in-depth assessment to ensure there is sufficient cover above the tunnel.
- Further investigation is required to verify the availability and/or acquisition of space for each shaft along the respective alignments, as well as to ensure adequate workspace or laydown areas.
- Cost estimates are based on construction and materials costs and do not include design, O&M, property/easement acquisition, or utility relocation.

Conclusions and Recommendations

Based on the results of the evaluation, the study concludes that microtunneling is the only feasible technology for San Francisquito Creek due to project and site constraints. While microtunneling may be feasible, it would not be able to meet the project goal without other measures. A tunnel would also take several years to evaluate, design, and permit. In addtion, a full CEQA review would be required.

The construction of an underground diversion tunnel system would have significant impacts to the community and environment. The system would require significant active monitoring and maintenance activities. This will include inspection of the intake structures throughout the year, with more frequent inspection and cleaning of the trash/debris racks during rain events to ensure that the tunnel capacity is available when needed. In addition, a pump station would be needed to empty the tunnel after each storm storms.Sediment and debris that enters the tunnel would need to be removed, at least once a year.

Additional considerations for microtunneling include:

1) From a technical standpoint, Black & Veatch suggests that microtunneling may be worth considering as an option since it will provide an additional 700 cfs of flow capacity. By itself, it could provide up to 39%of the additional flow capacity needed for the 70-year storm protection(increasing capacity from

5,400 cfs to 6,100 cfs, reducing the current capacity gap from 1,800 cfs to 1,100 cfs). In conjunction with other previously identified improvements, which provide 1,800 cfs additional capacity, capacity would be increased to 7,900 cfs. This exceeds the 70-year storm (7,200 cfs). This also provides 80% of the additional flow capacity needed for the 100-year storm protection (increasing capacity from 5,400 cfs to 7,900 cfs, reducing the current gap from 3,100 cfs to 600 cfs). Additional capacity may be achievable by installing a second tunnel along the same alignment or using two tunnels on different alignments. This could provide double the capacity, but at double the cost.

2) Additional study would be needed to confirm that microtunneling will work and will achieve the desired level of flood protection. This includes determination of available space within the public rights-of-way for the shafts, identification of utility conflicts, detailed hydraulic studies, geotechnical investigation, and environmental review

3) Microtunneling will have significant impacts on adjoining neighborhoods during construction, with lesser ongoing post-construction impacts. Tunnelling will have significant operations and maintenance requirements and costs.

4) The benefits of tunneling should be weighed against the impacts and compared to other previously identified options for flood risk reduction.

1.0 INTRODUCTION

1.1 Project Background

The San Francisquito Creek Joint Powers Authority (SFCJPA) and its member agencies-the City of Palo Alto, the City of Menlo Park, the City of East Palo Alto, the Santa Clara Valley Water District (Valley Water), and the San Mateo County Flood Control and Sea Level Rise Resiliency District (OneShoreline) desire to reduce flooding along San Francisquito Creek Urban Reach 2.

The creek currently has capacity for approximately the 25-year storm; the goal of this evaluation is to determine if a bypass tunnel can provide a level of flood protection for the 70-year storm, with protection up to the 100-year storm desired if possible.

The Urban Reach 2 area extends from West Bayshore Road to El Camino Real, with capacity issues beginning upstream of the Middlefield Road Bridge (Figure 1-1).

Target design flows are 7,200 cubic feet per second (cfs) for the 70-year storm and 8,500 cfs for the 100-year storm. The current creek capacity is 5,400 cfs from the Newell Road Bridge upstream to Middlefield Road – this capacity is based on the replacement of the existing Newell Road Bridge, which is currently under construction. From the new Newell Road Bridge downstream to West Bayshore Road, capacity is estimated to be about 8,500 cfs; however, this capacity is dependent on existing floodwalls that are reaching the end of their useful life. Additional capacity is needed from Newell Road to Middlefield Road due to inadequate capacity in the existing channel. In addition, while the lower portion of the creek has capacity to handle the 100-year storm, additional capacity via a bypass tunnel, if cost-effective, may be beneficial for reducing or eliminating the need to replace the aging floodwalls downstream of Newell Road.

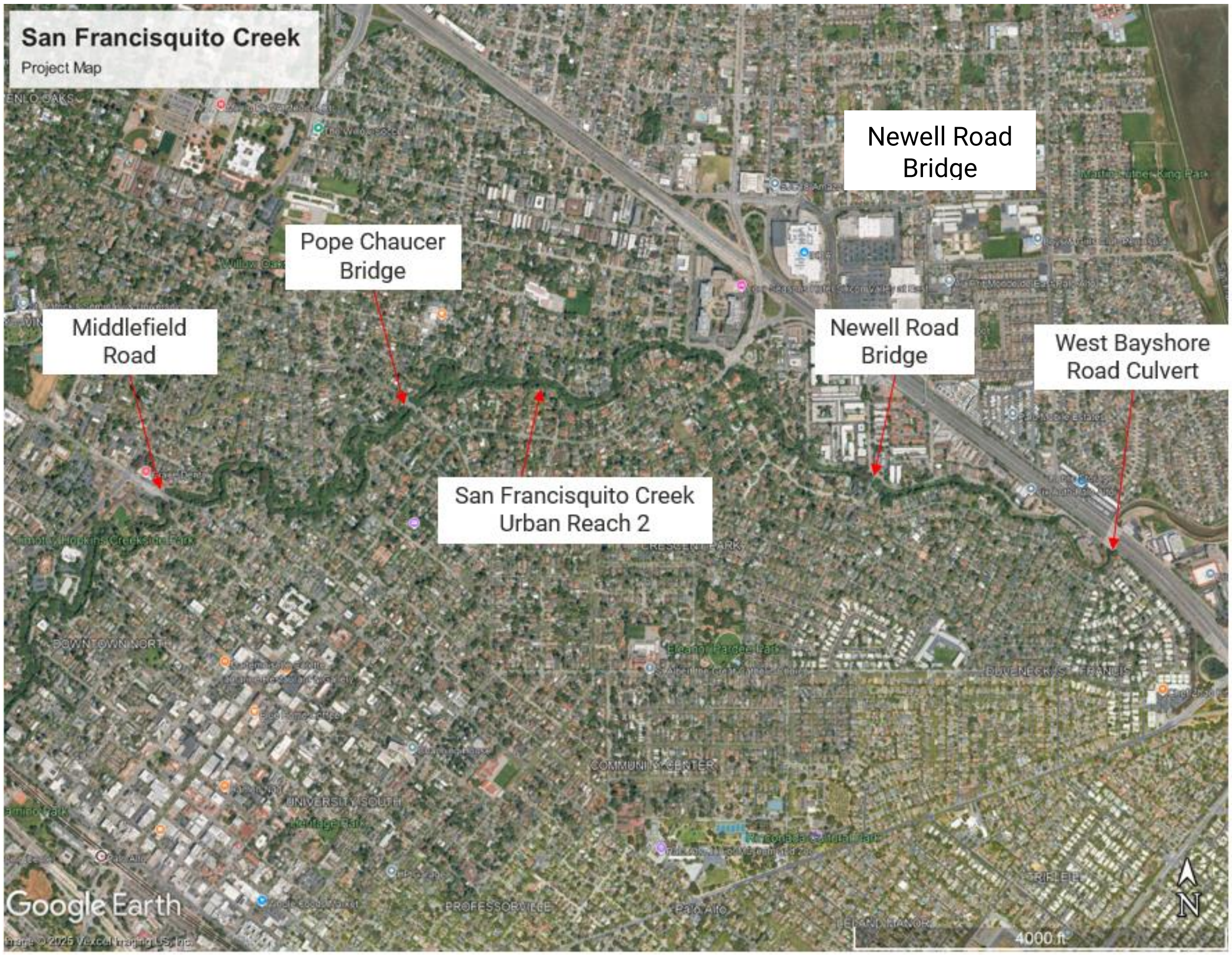


Figure 1-1 San Francisquito Creek Project Map

Reach 1, or Estuary Reach, from West Bayshore Road downstream to San Francisco Bay, has been previously improved to accommodate the 100-year storm, so there are no capacity issues downstream of West Bayshore Road.

The SFCJPA recently issued a Public Draft Alternatives Evaluation Report (Report), prepared by WRA, which included the evaluation of four potential design alternatives including channel widening, floodwalls, and modification to the Pope-Chaucer Bridge to increase creek capacity. In addition to these alternatives, the SFCJPA is interested in evaluating an underground bypass tunnel that would be capable of conveying some or all of the flows that exceed the current creek capacity, eliminating or reducing the scope of improvements in the Report.

As part of the project initiation, Black & Veatch discussed with SFCJPA staff the installation of a parallel diversion pipe along Woodland Drive using open-cut trenching. Staff expressed concerns with this method due to the narrow width of neighborhood streets, which could not accommodate trenching operations while maintaining access for residents and emergency vehicles and that the trench would have conflicts with existing utilities. Also, the creek corridor is believed to have extensive cultural resources which would be impacted by open trenching. Tunneling was viewed as an option to avoid the disruption caused by open-cut trenching.

1.2 Project Scope

The scope of this study is limited to:

- Review of past studies and bypass alignments: relevant information such as underground conditions considered during this study's alternative development and evaluation.
- Perform desk study/site reconnaissance: potential constraints (rights-of-way, existing residential and commercial properties) and opportunities such as alternative tunnel alignments are included in this study.
- Evaluate potential tunnel alignments: Alternative alignments will be assessed based on hydraulic constraints (maximum tunnel size allowed by tunneling methods) and the feasibility of construction methods. This study will determine the volume of water that can be conveyed through different pipe sizes that can be constructed given current technologies. An Opinion of Construction Costs (OPCC) and annual Operation and Maintenance Costs (O&M) will be prepared. The study will include trenchless technologies limitations, site-specific conditions and impacts to adjoining communities.

2.0 SUMMARY OF PREVIOUS STUDIES

2.1 Past Studies and Bypass Alignment

The following previous studies were reviewed:

- US Army Corps of Engineers Survey Report on San Francisquito Creek for Flood Control and Allied Purposes (1972)
- Early Implementation Project (EIP) Preliminary Analysis Document (2008)

An underground diversion option was considered in the 1972 US Army Corps of Engineers report. This option was named the “Willow Road Diversion Scheme” and consisted of a concrete-lined channel three-quarters of a mile long from El Camino Real to a diversion point just upstream from Middlefield Road, followed by an underground conduit along Willow Road to Bayshore Freeway, and a trapezoidal earth channel with levees to Ravenswood Slough. The alignment is shown in Figure 2-1. The US Army Corps of Engineers determined that this option could not be economically justified.

The idea of an underground tunnel bypass was considered but screened out in the Reach 2 Upstream Environmental Impact Report (2019), stating that although a culvert could convey 1,800 cfs around the floodplain area there would be major construction considerations, including traffic, utility, noise impacts, and temporary relocation of adjacent residents. New easements would be required. In addition, the bypass tunnel would have significant impacts on creek bank vegetation and potential impact from trapping aquatic species. Finaly, significant O&M would be required (SFJCPA, 2019).

A bypass tunnel was also screened out in WRA’s Draft Alternatives Evaluation Report for the same reasons listed above.. In addition, this option would require frequent maintenance, and any tunnel work poses additional safety concerns due to confined space. The necessity of cleaning debris and sediment from the tunnel before the start of the wet season and after storm events would require higher a maintenance burden that could outweigh advantages of installation. The project goals of enhancing habitat would not be realized with this approach. (WRA, 2024)

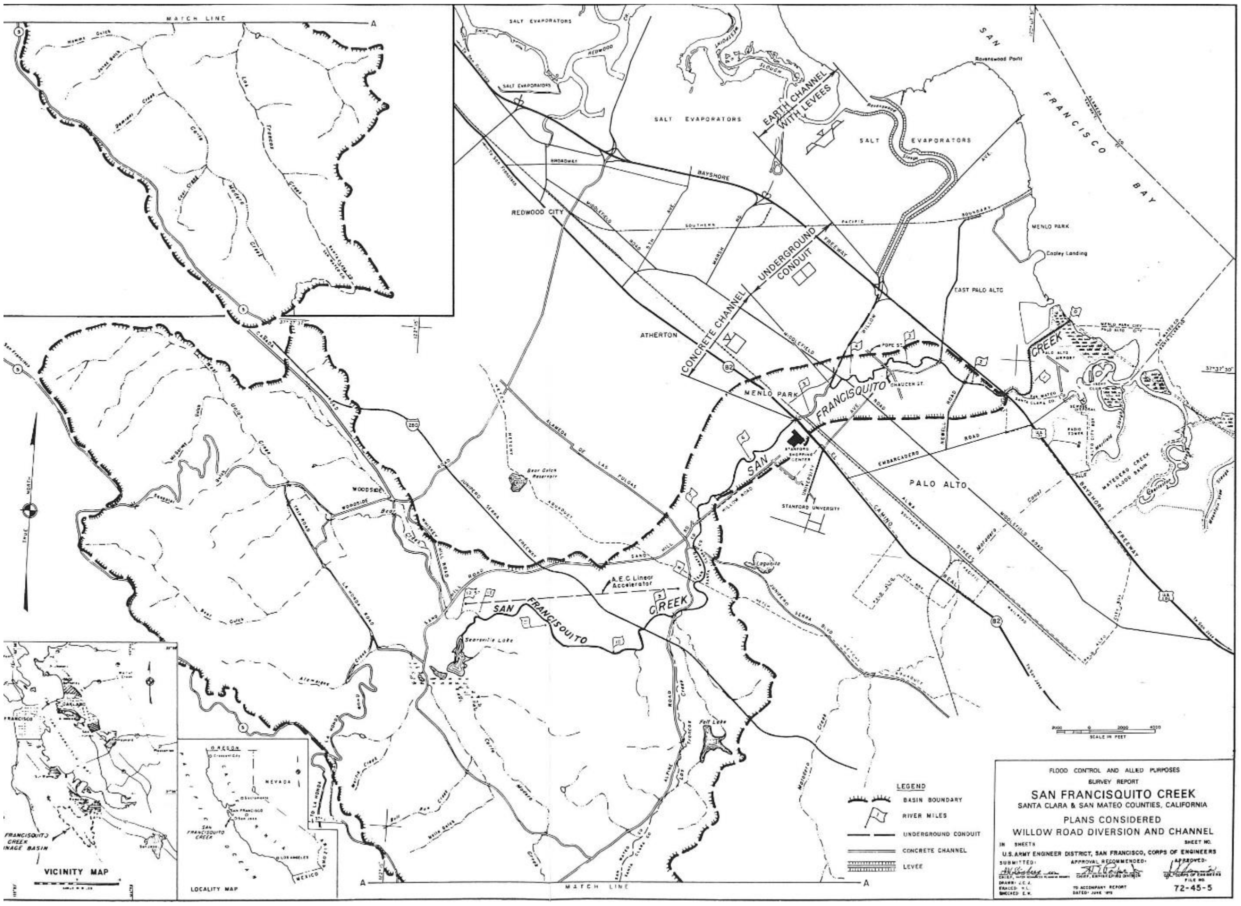


Figure 2-1 Alignment of Willow Road Diversion Channel (Survey Report on San Francisquito Creek, 1972)

A similar option, as shown in Figure 2-2, was considered in the 2008 Early Implementation Project Preliminary Analysis Document. The option presented in the 2008 report (Option 10) consisted of an underground pipeline along Willow Road from Middlefield Road to an outfall into Ravenswood Slough near the Bayfront Expressway. Constraints that were identified for this option in the 2008 report consisted of the cost of real estate, conflicts with other underground utilities, traffic disruptions on Willow Road during construction, and scour at Ravenswood Slough due to the high velocity pipeline discharge. These constraints still exist.

The Willow Road option was not considered for this study for the following reasons:

1) No suitable discharge location at the downstream end of the alignment near Willow Road and Bayfront Expressway.
2) Limited site area for a tunneling shaft at the upstream end of the alignment (Willow Road and Middlefield Road)
3) The tunnel would be within Caltrans right-of-way for about half its length.
4) The tunnel would divert SF Creek flows to another watershed.
5) The tunnel would not take advantage of existing capacity improvements in Reach 1 of SF Creek.

Figure 2-2 Alignment of Willow Road Diversion Pipeline (adapted from EIP Preliminary Analysis Document, 2008)

2.2 Desktop Study/Site Reconnaissance

This project is in a densely developed urban corridor, extending from the intersection of Middlefield Road and Woodland Avenue to West Bayshore Road near Highway 101, adjacent to the San Francisquito Creek itself. This area contains roads, utilities, and residential properties that have been built up to the edges of the creek bank through the urban Reach 2 area.

3.0 REVIEW OF EXISTING GEOTECHNICAL INFORMATION

A desktop review of previous geotechnical investigations in the vicinity of the potential project site was performed. Black & Veatch selected Appendix E, the Preliminary Foundation Report Newell Road Bridge (Replace), included in the Basis of Design for the San Francisquito Creek Reach 2 Project [1] as the most relevant and closest site information available currently. A summary of existing borehole information is provided in Table 3-1 below.

Table 3-1 Summary of Existing Borehole Information

Parameters	Borehole R-12-001	Borehole R-12-002
Location	Southern side of existing bridge	Northern side of existing bridge
Date Drilled	May 10th, 2012	May 11th, 2012
Depth Drilled	Approximately 81.5 ft	Approximately 80.5 ft
Drilling Methods	Hollow-stem auger to locate groundwater, rotary wash to bottom	
Sampling Methods	Modified California (MC) and Standard Penetration Test (SPT) samplers	
Ground Surface Elevation	Estimated 29-30 feet	
Soil Stratigraphy	~13.5 ft silty sand; predominately lean clay and sandy lean clay to 81.5 ft; sporadic clayey sand lenses between 30-60 ft; sandy soils loose to medium dense; clayey materials medium stiff to very stiff	~13 ft medium dense silty sand; mostly stiff to very stiff lean clay and sandy lean clay to 80.5 ft; medium dense gravel lens at ~42 ft
Groundwater	Encountered at ~20 ft depth during drilling	Not encountered in upper 20 ft; not measured below 20 ft due to drilling method
Site Geology	Underlain primarily by Latest Holocene alluvial fan levee deposits (Qhly); creek has stream channel deposits (Qhc)	
Corrosion Potential	Chloride content 733.2 ppm	Chloride content 47.1 ppm
Corrosion Test pH	7.41	7.45
Corrosion Test Sulfate Content	236.2 ppm	177.2 ppm
Liquefaction Potential	Sandy materials in upper ~13 ft susceptible	

Overburden soil conditions typical of alluvial fan and stream deposits with no bedrock were encountered within the investigated depth based on Boreholes #R-12-001 and #R-12-002 [1] completed on both the north and south sides of the existing Newell Road Bridge located at San Francisquito Creek/Newell Road intersection in May 2012. The upper sandy layers are loose to medium dense and susceptible to liquefaction while the presence of medium stiff to very stiff clay layers are expected to provide some natural ground support at depth. Groundwater was found at about 15-20 feet depth.

Prior to advancing further with any tunnel design and alignment planning, additional geotechnical and hydrogeological information will be required to confirm subsurface conditions and validate preliminary assumptions made in this technical memo. This information will be critical for refining construction methodologies, assessing risks, and ensuring the long-term performance and safety of the tunnel infrastructure. It is possible that a more thorough geotechnical investigation could find conditions that would rule out microtunneling

4.0 DESCRIPTION OF TUNNELING METHODS

This study will evaluate the viability of alternative alignments for the following three tunnel construction methods:

- Tunnel Boring Machine (TBM) Tunneling
- Horizontal Directional Drilling (HDD) and
- Microtunneling

Table 4-1 below provides a comparison of the tunneling methods and summarizes the limitations of each method.

Table 4-1 Comparison of Tunneling Methods

Considerations\Tunneling Method	TBM Tunneling	Microtunneling	HDD
Maximum Pipe Internal Diameter (feet)	>35	10	4.5
Typical Minimum Cover/Clearance	2 x Pipe Diameter	2 x Pipe Diameter	Determined from hydrofracture calculations and alignment geometry
Typical Minimum Horizontal Curve Radius (feet)	1,000	1,000	Dependent on pipe diameter and material
Minimum Laydown/Workspace Area (square feet)	160,000 (3.7 acres)	• Launch Shaft - 7,000 square feet (0.16 acres) • Receiving Shaft - 4,000 square feet (0.1 acres) [4]	100 feet x 150 feet, plus a 50 feet wide corridor along the pipe alignment [5]
Minimum Shaft Internal Diameter (feet)	40	• Launch Shaft – 40 feet • Receiving Shaft – 28 feet (For 10 feet pipe)	N/A
Typical Length between shafts, entry/exit (feet)	>10,000	<2,500	<5,000
Ground Conditions	All Ground Conditions	All Ground Conditions (Typically not recommended if boulders or obstructions greater than 1/3 of the excavation diameter are anticipated)	Soil or Rock (not ideal in gravelly conditions; not recommended for ground with boulders or obstructions present)

4.1 Tunnel Boring Machine (TBM) Tunneling

TBMs can be used to construct tunnels of up to 58ft diameter. They are cylindrical machines used to excavate tunnels through a variety of ground conditions, including soil and rock. They operate by rotating a cutterhead at the front of the machine, which breaks the ground while simultaneously removing excavated material using a conveyor system or slurry system that transports the excavated material back to the launch shaft. TBMs are ideal for long, large-diameter tunnel projects such as those used in subway systems, roadways, and water conveyance projects. TBM tunneling can be executed using either a one-pass or two-pass lining system, depending on project requirements, ground conditions, and structural design criteria. In a one-pass system, the TBM advances while simultaneously excavating and installing precast concrete segments to form the final tunnel lining, creating a continuous and stable structure. This approach is efficient and commonly used when the segmental lining meets all structural, hydraulic, and durability requirements. It reduces construction time and cost by eliminating the need for a secondary lining phase. For a two-pass system, a temporary support lining is installed during TBM excavation, typically using rock bolts, wire mesh and/or shotcrete. After the TBM completes its drive, a cast-in-place (CIP) final lining is constructed within the tunnel. Another type of two-pass system consists of installing precast concrete segments during the first pass, then installing a carrier pipe within the precast concrete segments during the second pass and grouting the carrier pipe in place. This method allows for enhanced structural performance, waterproofing and is often used in high-load or high-pressure environments.

The choice between one-pass and two-pass systems depends on factors such as tunnel depth, ground stability, internal and external water pressure, intended use, and long-term durability requirements. Both methods offer flexibility and precision, making TBM tunneling a preferred solution for complex underground infrastructure.

Figure 4-1 shows a TBM used for constructing the 35ft diameter Mill Creek Drainage Relief Tunnel project in Dallas, Texas. Figure 4-2 below shows a section of the Precast Concrete Tunnel Lining installed at the 16ft diameter 3RPORT Tunnel in Fort Wayne, Indiana.

Figure 4-1 35ft diameter TBM at the Mill Creek Tunnel in Dallas, TX for flood protection.

Figure 4-2 16ft diameter PCTL installed at the 3RPORT Combined Sewer Overflow (CSO) Tunnel project in Fort Wayne, IN.

The following are key phases of the TBM tunneling process:

1. Site Investigations: Detailed geotechnical and hydrogeological studies are conducted to assess ground conditions along the tunnel alignment and potential risks. Utility surveys are completed to identify existing utilities which may require relocation at shaft sites or along the alignment.

2. Shaft Construction: Launch and reception shafts are constructed to assemble and disassemble the TBM and facilitate material handling and segment installation.

3. TBM Assembly & Launch: The TBM is assembled in the launch shaft and connected to backup systems for spoil removal, segment handling, and guidance. Initial thrust is applied to begin excavation, and the cutterhead starts rotating to break ground.

4. Tunnel Excavation & Spoil Management: The TBM excavates material continuously while advancing along the tunnel alignment. Excavated material (spoil) is transported via muck carts, conveyor belts or slurry pipelines to the surface for disposal or reuse. Real-time monitoring systems track TBM position, cutterhead performance, and ground response including settlement, lateral ground movement, groundwater inflow, face pressure changes and vibrations near the tunnel.

5. Segment Installation (for a one-pass system): Precast concrete segments are installed within the TBM shield immediately after excavation to form the tunnel lining. Segments are bolted and gasketed to ensure structural integrity and watertightness. In two-pass systems, a secondary lining (typically CIP concrete) is installed after completion of TBM tunneling.

6. TBM Breakthrough & Retrieval: Upon reaching the reception shaft, the TBM is dismantled and removed. Final tunnel inspections are conducted to verify alignment, structural performance, and waterproofing.

7. Post-Installation Activities: Surface restoration and shaft decommissioning are completed as per project requirements.

Benefits and limitations of TBM tunneling are summarized below:

Benefits:

- Large diameter (greater than 10 feet) installations with minimal surface disruption
- Ability to install precast concrete tunnel segments which can serve as both initial and final tunnel support
- TBMs are capable of mining over 10,000 feet between shafts, which minimizes the number of shafts required.

Limitations:

- High initial cost
- Significant mobilization and demobilization time
- Requires large launch and retrieval shafts and staging areas – see details in Table 4-1

The Urban Reach 2 area is fully developed and does not have the available land for the TBM, with almost 4 acres of land (two city blocks) at entrance and exist shafts. The land would be needed for construction, but a large portion would be need to be retained permanently to accommodate the tunnel and shaft footprint, and to provide access for maintenacne. It was therefore ruled out for application to this area.

4.2 Horizontal Directional Drilling (HDD)

HDD is a trenchless method used primarily for installing pipelines and conduits beneath surface obstacles like rivers, roads, railways and buildings. HDD is surface-launched and highly maneuverable, allowing for curved alignments that navigate around existing infrastructure. It is most effective in softer soils and is commonly used for utility installations such as water, gas, and telecommunications lines. The process involves drilling a pilot hole along a predetermined path, enlarging it to the required diameter, and pulling the fused product pipe into place, all with minimal surface disruption.

The following are key phases of the HDD process:

1. Site Investigations: A comprehensive geotechnical investigation is conducted to assess subsurface conditions, including soil type and groundwater levels. A utility survey is conducted to determine the location and depth of existing utilities.

2. Alignment Selection: The drill path is designed to accommodate the required depth, curvature, and entry/exit angles, ensuring avoidance of underground obstructions and compliance with regulatory requirements.

3. Pilot Hole Drilling: A small-diameter pilot hole is drilled along the predetermined path using a steerable drill head. A real-time tracking system guides the drill head, allowing navigation through curved alignments.

4. Hole Enlargement (Reaming): Once the pilot hole is complete, it is enlarged using a series of reamers to achieve the required diameter for the product pipe. Drilling fluid (typically bentonite-based mud) is circulated to stabilize the borehole, remove cuttings, and reduce friction.

5. Product Pipe Preparation: The final pipeline or conduit is typically connected into a continuous string before installation. For HDPE (High-Density Polyethylene) pipes or Fusible Polyvinyl Chloride (FPVC) pipes, fusion is commonly used to ensure a leak-proof and structurally sound joint. The continuous pipe string is laid out in a staging area, typically along the pipeline alignment and aligned with the exit point.

6. Pullback Operation: The reamed borehole is cleaned and conditioned before the product pipe is pulled into place using the drill rig. A swivel is used between the reamer and the pipe to prevent torque transfer, and a reamer or barrel reamer may be used during pullback to maintain borehole integrity.

7. Post-Installation Testing & Restoration: The installed pipeline may undergo pressure testing, pigging, or inspections to verify integrity and alignment. Surface restoration is performed at entry and exit pits, and any disturbed areas are rehabilitated.

Benefits and limitations of HDD are summarized below:

Benefits:

- Minimal surface disruption
- Steerable and allows curved alignments

- Quick setup and execution for shorter distances
- Cost effective for small diameter installations
- Shafts not required (only launch and exit pits)

Limitations:

- Rarely used for gravity flow pipelines due to poor installation accuracy
- Limited to small diameter pipelines (up to 5 feet)
- Not recommended in ground containing cobbles and boulders
- Challenging in gravelly ground conditions
- Accuracy may decrease over long distances or with increasing depth depending on the tracking system used
- Risk of drilling fluid loss (frac-out)
- Long staging area required to lay out pre-fused pipe string.

These limitations coupled with the low flow capacity permitted by the maximum pipe size achievable with HDD (as discussed in section 6) render this tunneling method unsuitable for the project needs. It was therefore ruled out.

4.3 Microtunneling

This tunneling method utilizes a Microtunnel Boring Machine (MTBM), which is a remote-controlled pipe jacking method that installs pipelines with high precision and minimal surface disruption. Typical pipeline diameters installed using microtunneling range from 24 inches to 10 feet (internal diameter). The MTBM is typically launched from a shaft and pushed forward by hydraulic jacks, which simultaneously install prefabricated pipe segments behind it. Guided using a laser system, microtunneling maintains accurate alignments even in challenging ground conditions. This tunneling method is particularly well suited for urban environments where surface disruption must be minimized, and it is often used for sewer, stormwater, and water supply systems. Unlike HDD, microtunneling requires launch and reception shafts whose typical sizes are shown in Table 4-1.

The following are key phases of the microtunneling process:

1. Site Investigation & Shaft Construction: A thorough geotechnical investigation is conducted to evaluate soil conditions and groundwater levels. A utility survey is completed to identify the location and depth of existing infrastructure. A launch shaft is constructed to launch the MTBM and facilitate pipe jacking operations. A reception shaft is constructed to retrieve the MTBM at the other end of the microtunnel drive.

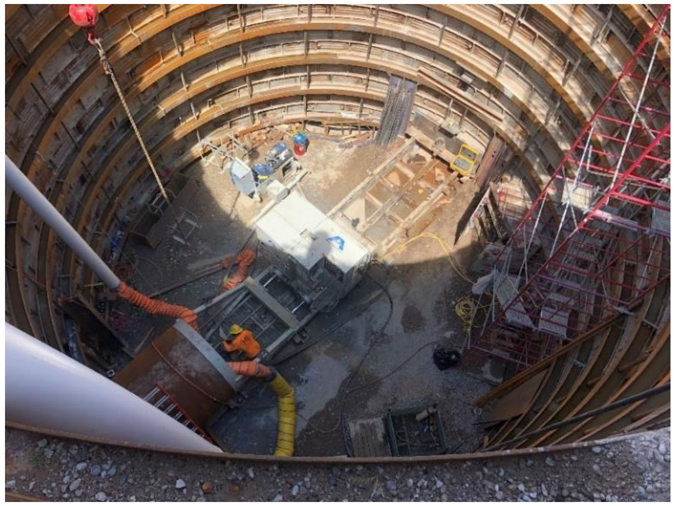


Figure 4-3 Typical Microtunnel Launch Shaft

2. MTBM Setup & Launch: The MTBM is lowered into the launch shaft and set up within the jacking frame. Hydraulic jacks are positioned to push the pipe segments forward as the MTBM excavates the tunnel.

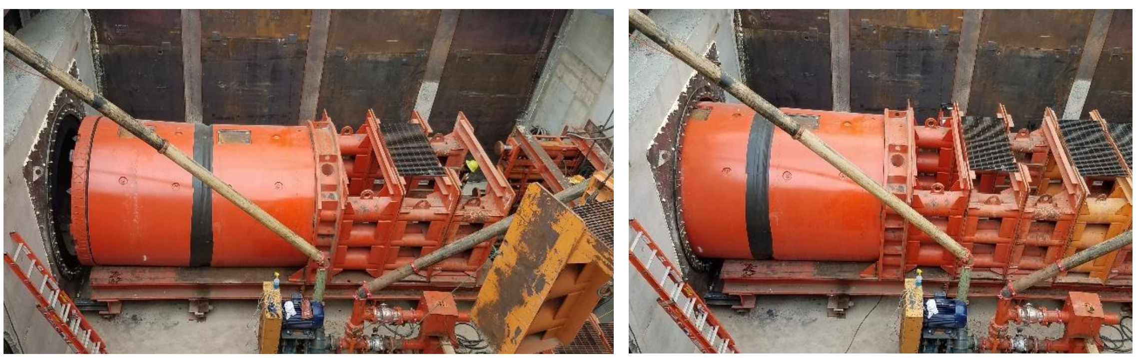


Figure 4-4 Typical Microtunnel Setup and Launch

3. Tunnel Excavation & Pipe Jacking: The MTBM excavates soil while simultaneously advancing and installing pipe segments. Spoil is typically removed through a slurry system that pumps the slurry back to the surface near the launch shaft. A slurry separation system adjacent to the launch shaft separates the soil from the slurry (typically a mixture of water and bentonite). A laser guidance system monitors the horizontal and vertical alignment throughout the drive.

Figure 4-5 Typical Microtunnel Excavation and Pipe Jacking Operations

4. Pipe Installation: Pipes are jacked into place behind the MTBM in a continuous sequence. Pipe joints are sealed to ensure structural integrity and watertightness, often using gasketed connections for concrete pipes and welded joints for steel pipes.

Figure 4-6 Typical Microtunnel Pipe Installation

5. Reception & Retrieval: Upon reaching the reception shaft, the MTBM is retrieved from the shaft. Final pipe alignment and grade are verified to ensure compliance with design specifications.

6. Carrier Pipe Installation: If a two-pass system is required, a carrier pipe is installed within the microtunnel jacking pipe and grouted in place. On some projects, a one-pass system is sufficient (microtunnel jacking pipe is used as the final product pipe).

7. Post-Installation Testing & Restoration: The installed pipeline may undergo inspection via CCTV, pressure testing, or other verification methods. Shafts are backfilled and surface areas are restored to pre-construction conditions or better.

Benefits and limitations of microtunneling are summarized below:

Benefits:

- Remote controlled and highly accurate alignment tolerances (well suited for gravity systems)
- Suitable for a wide range of ground conditions, including through cobbles and boulders depending on the MTBM diameter

- Minimal surface disruption
- Can be used below the groundwater table
- Less site area required for launch and reception shafts compared to TBM tunneling see Table 4-1 for more details.

Limitations:

- Typically limited to pipe diameters up to 10 feet (ID) though pipe diameters up to 11 feet have been accomplished
- Requires launch and reception shafts with an estimated minimum ID of 40 feet and 28 feet ID, respectively (See Table 4-1).
- Typically higher cost than HDD
- Limited to straight or slightly curved alignments
- Further study is needed to confirm if any property acquisition, or temporary or permanent easements are needed.

5.0 EVALUATION OF TUNNEL ALIGNMENTS

To convey water from upstream at the intersection between the San Francisquito (SF) Creek and either Middlefield Road or Pope Chaucer Bridge, seven tunnel alignment alternatives were considered and summarized in Table 5-1 below. Given the site constraints and the needs of the other tunneling methods, microtunneling is the proposed method for these alignments. These alignments are shown in Appendix C – Aerial Views of Considered Tunnel Alignments.

Table 5-1 Summary of Tunnel Alignments

Alignment ID	Description	# of Shafts	Proposed Tunneling Method
1 (Purple)	Middlefield Rd (A-2) – Woodland Ave – West Bayshore Rd – West Bayshore Rd Culvert (B-2)	N/A	N/A
2 (Red)	Middlefield Rd (A-2) – University Ave – W Bayshore Rd – West Bayshore Rd Culvert (B-2)	8	Microtunneling
3 (Pink)	Pope Chaucer Bridge (A-1) – Chaucer St – University Ave - West Bayshore Rd – West Bayshore Rd Culvert (B-2)	7	Microtunneling
4 (Light blue)	Pope Chaucer Bridge (A1) – Chaucer St – Hamilton Ave – Newell Rd (B-1) - West Bayshore Rd – West Bayshore Rd Culvert (B-2)	8	Microtunneling
5 (Yellow)	Pope Chaucer Bridge (A-1) – Chaucer St – Hamilton Ave – Newell Rd (B-1)	5	Microtunneling
6 (Blue)	Middlefield Rd (A-2) – Hamilton Ave – Newell Rd – West Bayshore Rd – Woodland Ave/Widened SF Creek (C-1)	6	Microtunneling
7 (Green)	Middlefield Rd (A-2) – Hamilton Ave – Newell Rd – West Bayshore Rd – West Bayshore Rd Culvert (B-2)	9	Microtunneling

Notes:

- Alternative 1 not feasible due to the curvature of Woodland Avenue.
- Alternatives # 2, 3, 4, and 7 have the option of discharging water at Point C-1, near the intersection of West Bayshore Road and Woodland Avenue, allowing elimination or deferral of the lower 650’ of West Bayshore Road tunnel. This option is shown on respective figures as a white colored alternate alignment running between the collector shaft and point C-1.
- All alternatives crossing the SF Creek will require more in-depth assessment to ensure there is sufficient cover above the tunnel.

- Further investigation is required to verify the availability and/or acquisition of space for each shaft along the respective alignments, as well as to ensure adequate workspace or laydown areas.
- Based on tunneling methods comparison discussed in section 4.0 and summarized in Table 4-1 above, TBM tunneling was not considered a feasible option. Additional information can be found in section 9.1.1.

5.1 Alignment 1

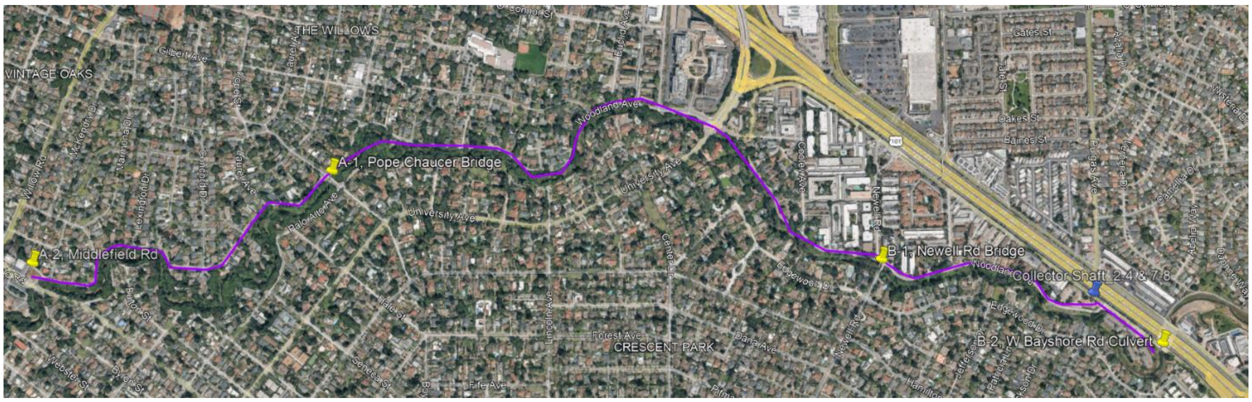


Figure 5-1 Alignment 1

This alternative starts from Point A2 (Middlefield Road and SF Creek), then follows the right of way along Woodland Avenue, passing both the Pope Chaucer Bridge (Point A-1) and Newell Road Bridge (Point B-1,) then connecting to West Bayshore Road through the Collector Shaft from which it discharges the conveyed water flow downstream at the existing West Bayshore Road/ Highway 101 Culvert (Point B-2).

Alignment 1 shown in Figure 5-1 above <u>is not feasible</u> given the number and extent of curves it would require. It would only be constructible with multiple shafts to accommodate the horizontal curves, which would cause significant cost increases, surface disruption, and impacts to traffic and residents.

5.2 Alignment 2

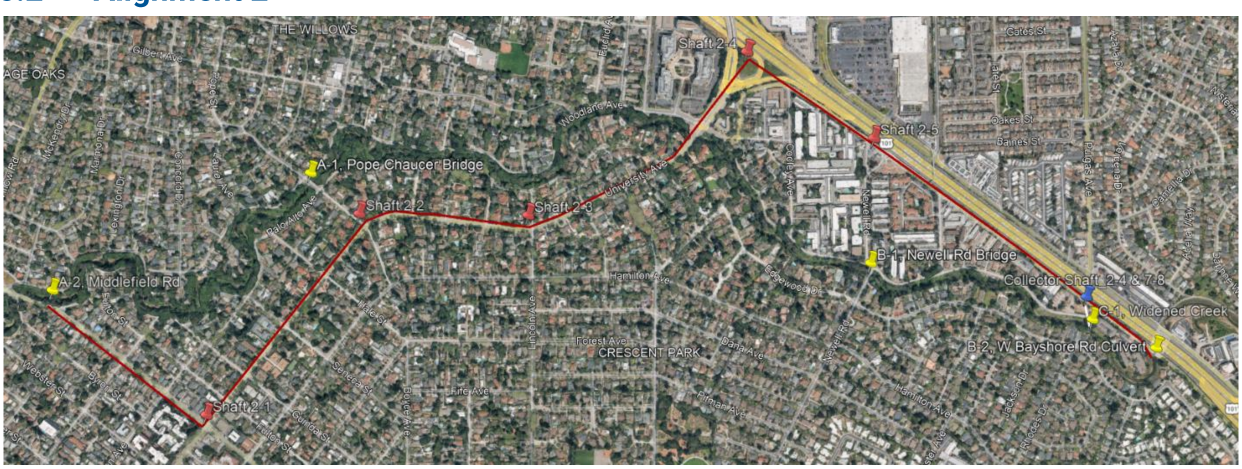


Figure 5-2 Alignment 2

Alignment 2 shown in Figure 5-2 above starts from Point A-2 and follows Middlefield Road, then changes direction at the intersection between Middlefield Road and University Avenue. It continues along University Avenue crossing the SF Creek at Woodland Avenue and turning right toward West Bayshore Road from the

Highway 101/University Avenue interchange. From there, this alignment continues along West Bayshore Road discharging water at Point B-2.

Alternative 2 would be constructed with a Microtunnel Boring Machine (MTBM) which would require 8 shafts located at the following intersections:

- Middlefield Road/Palo alto Ave
- Middlefield Road/University Avenue
- Chaucer Street/University Avenue
- University Avenue/Lincoln Avenue
- University Avenue/West Bayshore Road (Hwy 101 Right-of- Way)
- West Bayshore Road/Newell Road
- Collector Shaft
- West Bayshore Road/West Bayshore Road Culvert

Alternative 2 appears to not be a preferred option for the following reasons:

- Crossing the SF Creek at University Ave/Woodland Ave would require a significantly deeper tunnel alignment to ensure adequate minimum cover above the tunnel.
- Approval is needed from Caltrans regarding availability of space for construction of Shaft #2-4 located at the University Avenue/ Hwy 101 interchange.
- Managing traffic impacts associated with construction along this alignment would be challenging, especially at the Highway 101/University Avenue exit.

5.3 Alignment 3

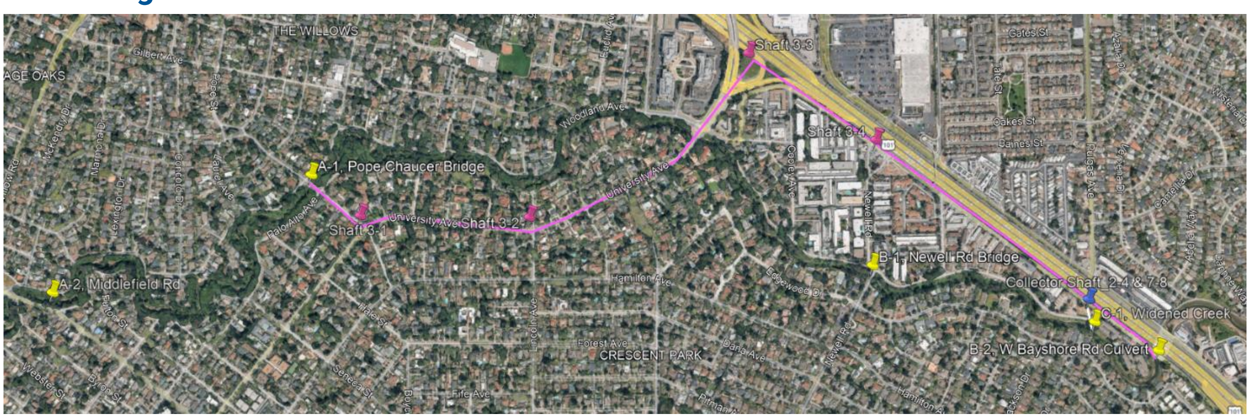


Figure 5-3 Alignment 3

Alignment 3 shown in Figure 5-3 above is similar to alignment 2, except it starts from the Pope Chaucer Bridge. For microtunneling, it would require construction of 7 shafts located at the following intersections:

- Chaucer Street/Woodland Avenue
- Chaucer Street/University Avenue
- University Avenue/Lincoln Avenue
- University Avenue/W Bayshore Road
- West Bayshore Road/Newell Road
- Collector Shaft
- West Bayshore Road/West Bayshore Road Culvert

5.4 Alignment 4

Alignment 4 shown in Figure 5-4 below draws water from the SF Creek from the intersection of Pope Chaucer Bridge and the SF Creek, then continues along Chaucer Street until it changes direction towards Hamilton Avenue, Newell Road, and West Bayshore Road, and discharges at the West Bayshore Road Culvert downstream.

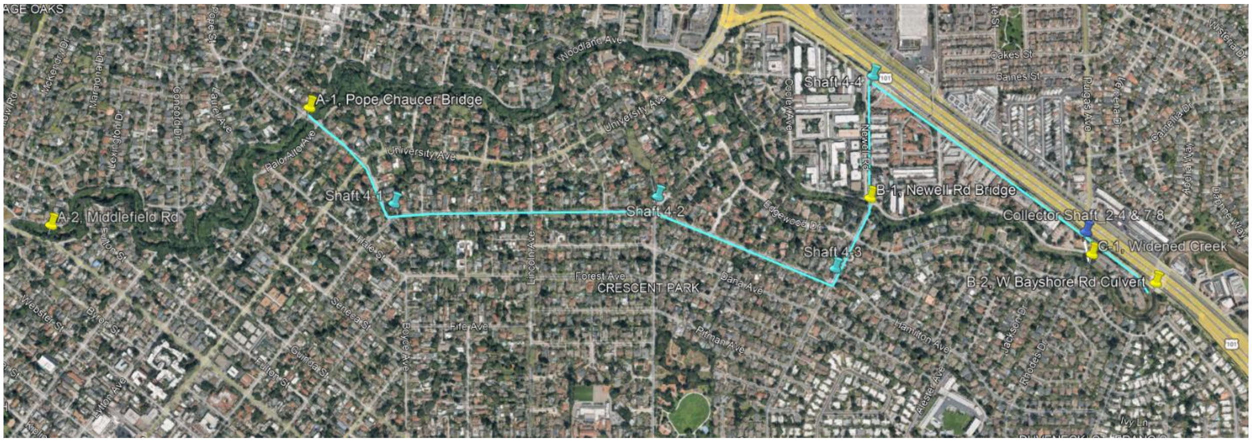


Figure 5-4 Alignment 4

This alternative would require construction of 8 shafts for microtunneling located at the following intersections:

- Chaucer Street/Woodland Avenue
- Chaucer Street/Hamilton Avenue
- Hamilton Avenue/Center Drive
- Newell Rd/Hamilton Avenue
- Newell Rd/Woodland Avenue
- West Bayshore Road/Newell Road
- Collector Shaft
- West Bayshore Road/West Bayshore Road Culvert

5.5 Alignment 5

Alignment 5 shown in Figure 5-5 below draws water from the intersection of Pope Chaucer Bridge and SF Creek, then continues along Chaucer Street until it changes direction towards Hamilton Avenue and Newell Road to finally discharge water back into the creek at its intersection with Newell Road.

This alternative would require construction of 5 shafts for microtunneling located at the following intersections:

- Chaucer Street/Woodland Avenue
- Chaucer Street/Hamilton Avenue
- Hamilton Avenue/Center Drive
- Newell Rd/Hamilton Avenue
- Newell Rd/Woodland Avenue

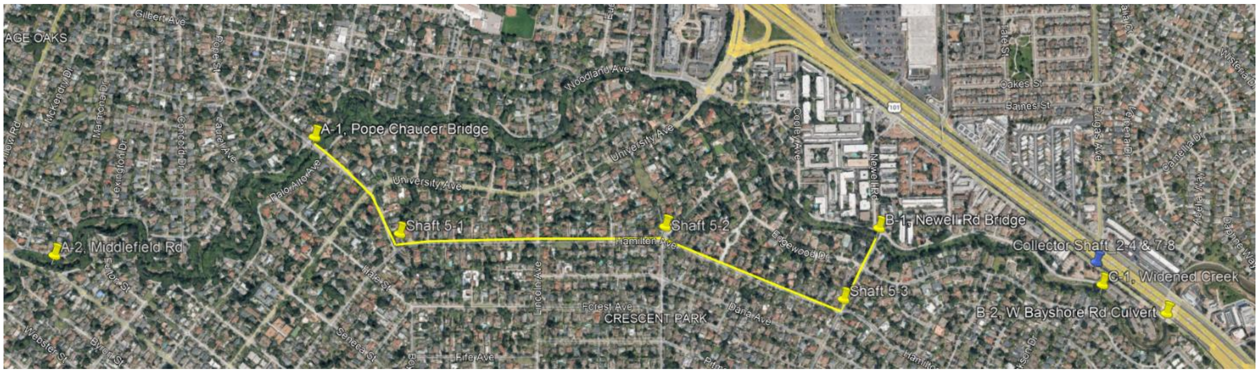


Figure 5-5 Alignment 5

This alternative presents the shortest tunnel alignment, with the least construction challenges. It would be built using microtunneling. The alternative would not address flooding issues upstream of the Pope-Chaucer Bridge, and would not address the need to replace the existing floodwalls between Newell Road and Hwy 101. For these reasons, this was not considered to be the preferred alternative.

5.6 Alignment 6

Alignment 6 shown on Figure 5-6 below is a variation of alternative 5 where water is drawing from the creek further upstream at its intersection with Middlefield Road and discharges it back at the SF Creek's intersection with Newell Road.

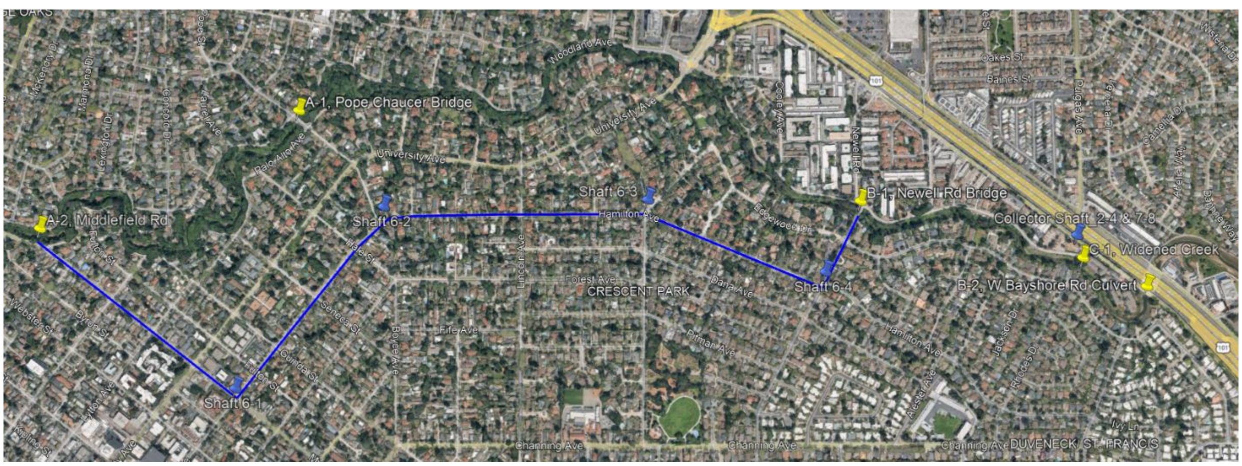


Figure 5-6 Alignment 6

Similarly, this alignment would be built by microtunneling. Given the ability of this alternative to draw water from the creek further upstream at Middlefield Road, it is considered one of the preferred alternatives. A plan/profile drawing was developed for this alignment and included in Appendix A – Conceptual Plan and Profile Drawings.

5.7 Alignment 7

Alignment 7 shown on Figure 5-7 below is a variation of alternative 4 where water is drawn from the creek at its upstream intersection with Middlefield Road. It allows reduction of the water flow from this location of the creek and discharges back at its downstream intersection with the West Bayshore Road culvert.

This alternative presents the longest tunnel alignment. It is anticipated to be feasible by microtunneling. Similar to Alignment 6, a plan/profile drawing was developed and included in Appendix A – Conceptual Plan and Profile Drawings .

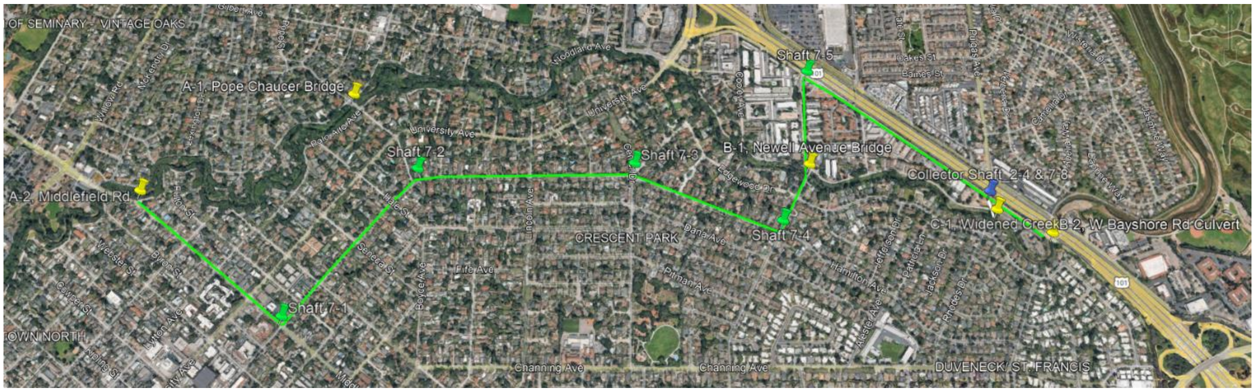


Figure 5-7 Alignment 7

Plan/profile for alternatives 6 and 7 were developed since they represent the most preferred alternatives given their construction feasibility and flow capacity.

6.0 HYDRAULIC ANALYSIS

This section describes the conveyance capacity analysis of the proposed tunnel by considering hydraulic conditions of the SF Creek and the constraints associated with the various alignments. Section 6.1 outlines the existing channel capacity of SF Creek and the excess flows during large rainfall events that need to be conveyed by other means. Section 6.2 describes the methodology used to evaluate tunnel capacity, and Section 6.3 presents the analysis results for each alignment.

It should be noted that preliminary calculations were performed for the three tunneling methods mentioned in Section 4.0. However, because of the limitations of using TBM (Section 4.1) and HDD (Section 4.2), these methods were not considered feasible and only the hydraulic capacity assuming use of microtunneling is presented in this section. TBM tunneling allows for construction of a sufficiently large tunnel to convey the excess flow from the 100-year rainfall event, but the large staging areas and property and easement acquisition make it unfeasible. In the case of HDD tunneling, while it also presents large staging area requirements and poor installation accuracy, the main issue affecting capacity is the maximum pipe diameter this method allows (up to 5 feet) compared to microtunneling (up to 10 feet). HDD tunneling provided between a fourth or fifth of the hydraulic capacity of the microtunneling; therefore, it was not considered further.

6.1 Existing Channel Capacity and Excess Flows

Based on the review of previous reports described in Section 1.1, and coordination with SFCJPA staff during the meeting held on August 14, 2025, tunnel conveyance capacities were evaluated for both the 70-year and 100-year rainfall events. Table 6-1 summarizes the total flow associated with each event, along with the existing channel capacity and excess flows. The resulting excess flows for the portion of the SF Creek between the Middlefield Road crossing and the Newell Road crossing are approximately 1,800 cfs for the 70-year event and 3,100 cfs for the 100-year event.

Table 6-1 Existing Channel Capacity and Additional Flow Capacity Required for 70-Year and 100-Year Rainfall Event

Rainfall Event	Total Flow (cfs)	Existing Channel Capacity (cfs)	Additional Flow Capacity Required (cfs)
70-year	7,200	5,400	1,800
100-year	8,500	5,400	3,100

6.2 Tunnel Conveyance Capacity Considerations

Figure 6-1 illustrates a schematic representation of the tunnel system hydraulics. The tunnel would be designed to act as a siphon, in which water enters the tunnel system through an inlet structure adjacent to the creek and is directed down a drop shaft to the elevation of the tunnel. Water would be conveyed through the tunnel and intermediate shafts and eventually reach an outlet shaft where it would flow upwards to the elevation of the creek. Intermediate shafts are incorporated to shorten the tunnel segments between shafts, considering the maximum allowable distances for both horizontal directional drilling (HDD) and microtunneling. These shafts also facilitate directional changes in tunnel alignmentDuring large storm events, when excess flow needs to be conveyed by the tunnel, the flow would be transported by gravity. After the rainfall events, the remaining water in the tunnel would be removed by pumping. Pumps should be designed to allow the tunnels to be emptied within 48-72 hours to avoid loss of dissolved oxygen in the water (an impact to aquatic life) and mosquito breeding at the intake and discharge shafts.

. Shafts will be retained for post-construction maintenance access. Shafts can be retained as constructed, or a manhole riser can be constructed within the shaft and backfill placed between the manhole and the shaft. Each shaft will include a manhole to allow post-construction access.

The upstream and downstream shafts can be retained and modified to serve as the intake and outlet structures. Additional design work will be needed to determine the size and elevation of the intake and outlet openings. Debris racks will also be needed to keep large debris out of the tunnel.

Pump sizing, pump station locations, and energy source were beyond the scopes of this study. However, pumps would not need to convey peak storm flows. Based on a length of 9,500 lineal feet for the tunnels and a diameter of 10 feet, the volume of stored water would be approximately 750,000 cubic feet. Based on a drain time of 48 hours (172,800 seconds), the require flow rate would be 4.3 cubic feet per second. It is expected that the downstream shaft could be used as the wet well for the pump, avoiding the need for another structure. Construction of the pump station could begin once the downstream shaft is no longer and could occur in parallel with construction of other pipe segments. Pump station would not likely impact the overall duration of the project.

The hydraulic capacity of the tunnel system can be described using the principle of conservation of energy. According to this principle, the water surface elevation difference (available head, ΔH) between the upstream and downstream diversion sections equals to the total energy losses (head loss, h_T) incurred as water flows through the system.

The total head loss within the tunnel results from the combined effect of friction losses (h_f) and local losses (h_l), expressed as:

$$\Delta H = h_T = h_f + h_l$$

Friction losses are caused by contact of the fluid with the conduit walls, viscosity of the fluid and turbulence. Local losses are typically caused by changes in flow velocity due to inlet and outlet structures, bends in the tunnel, variations in conduit cross-sectional area, and other structural features or obstructions. The methods used to estimate both friction and local losses are described in detail below.

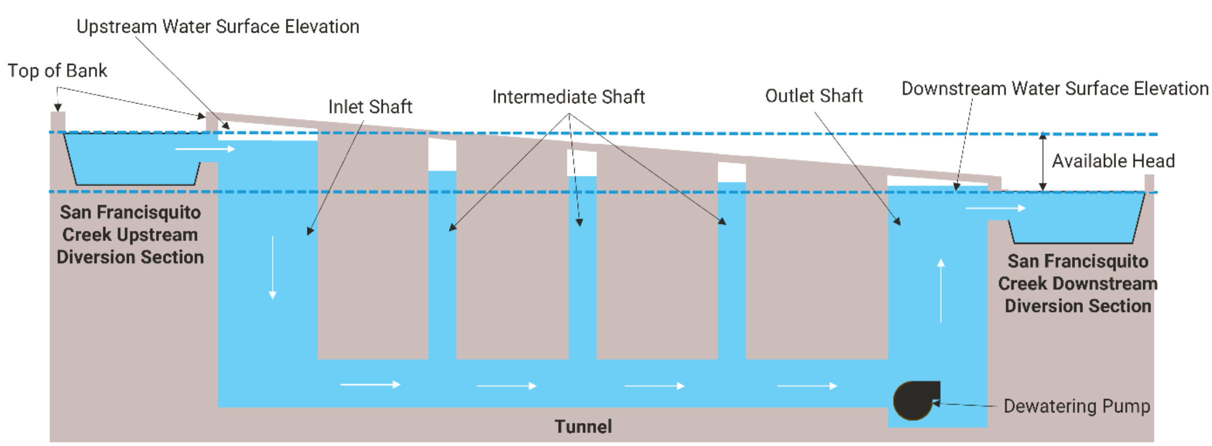


Figure 6-1 Typical Tunnel Profile

6.2.1 Friction Losses

For circular conduits, friction losses can be calculated using the Darcy-Weisbach equation, as shown below:

$$h_f = f \frac{L}{D} \frac{v^2}{2g}$$

where h_f is the friction loss (foot), f is friction factor (dimensionless), L is length of the conduit (foot), D is conduit diameter (foot), v is flow velocity (foot per second), and g is acceleration of gravity (32.2 feet per second squared).

The friction factor can be estimated using the Colebrook-White equation, as shown below:

$$\frac{1}{\sqrt{f}} = -2\log(\frac{k_s}{3.7D} + \frac{2.51}{Re\sqrt{f}})$$

$$Re = \frac{VD\rho}{\mu}$$

where k_s is pipe wall roughness expressed as an equivalent uniform sand grain size (foot), Re is Reynolds number (dimensionless), μ is dynamic viscosity of water (pound-force seconds per square foot), and ρ is density of water (slugs per cubic foot).

For tunnel construction, a segmented concrete lining is typically used for TBM tunneling, concrete pipes are typically used for single-pass microtunneling, and high-density polyethylene (HDPE) pipe or fusible polyvinyl chloride (FPVC) pipe are commonly used for HDD. For this preliminary calculation, since only microtunneling is considered, a pipe wall roughness of 0.007 foot was assumed for concrete tunnels, in accordance with the recommendations in *Internal Flow Systems* by D.S. Miller.

6.2.2 Local Losses within the Tunnel

Local head losses for this analysis can generally be categorized into four types: inlet and outlet shaft losses, tunnel curvature losses, intermediate shaft losses, and tunnel entrance/exit losses.

Inlet and outlet shaft losses occur when water is diverted from the stream into the inlet shaft and when it returns from the outlet shaft back to the stream. These losses result from changes in flow conditions as water passes through various control structures required for the safe operation of the tunnel conveyance system. For preliminary calculations, a head loss of 2 feet was assumed for the inlet shaft and 1 foot for the outlet shaft.

Other types of local losses (h_l) are typically proportional to the velocity head ($V^2/2g$), with the proportionality constant (k) determined empirically. These losses were estimated using the following equation:

$$h_l = k \frac{v^2}{2g}$$

Tunnel curvature losses were applied only to alignment 1 due to its significant curvature. The bend loss was accounted by assuming 5 bends per mile and a loss coefficient of 0.05 per bend, resulting in a total loss coefficient of 0.25 per mile. Currently, the number of curves and radius of curvature is not known.

For alignment 2 through alignment 7, the location of intermediate shafts has been determined, and the tunnel is relatively straight between intermediate shafts. **Intermediate shaft losses** were assumed to act as 90-degree bends without rounding. A loss coefficient of 1.1 per intermediate shaft was assumed, as recommended in *Internal Flow Systems* by D.S. Miller.

Lastly, **tunnel entrance and exit losses** were estimated using coefficients of 0.5 and 1.0, respectively, for preliminary design purposes.

6.3 Preliminary Tunnel Conveyance Capacity Estimates

As discussed in Section 6.1, the maximum flow capacities of microtunneling for Alignments 1 through 7 were calculated and compared against the additional flow capacities required during 70-year and 100-year rainfall events. Table 6-2 presents the maximum tunnel conveyance capacity under the 70-year event. Table 6-3 provides the corresponding values for the 100-year rainfall event.

Overall, microtunneling can accommodate approximately 33% – 44% of the excess flow for the 70-year event and 19% – 26% for the 100-year event. Tunnel velocities for microtunneling under the design storm events for Alignment 1 through 7 are below 10 feet per second (fps), which are considered to be within normal best practice range. If further design of the tunnel is performed, flow velocities in the tunnel will be evaluated to ensure they are within best practices to mitigate sedimentation. Among these alignments, Alignment 1 is technically infeasible because of the excessive number of curves. Alignments 2 and 6 provide the highest conveyance capacity — approximately 39% of the 70-year excess flow and 23% of the 100-year excess flow. This is primarily because Alignment 2 begins at Middlefield Road and ends at the West Bayshore Road culvert, offering the greatest available head among all combinations. Alignment 6, on the other hand, has relatively fewer intermediate shafts, resulting in lower local head losses.

Table 6-2 Tunnel Conveyance Capacity by Microtunneling for 70-Year Rainfall Event (Pipe Diameter = 10 feet)

Alignment	Excess Flow (cfs)	Tunnel Conveyance Capacity (cfs)	Tunnel Conveyance Percentage	Residual Flow Not Conveyed (cfs)
1[a]	1,800	796.5	**44%**	1,003.5
2		703.3	**39%**	1,096.7
3		636.5	**35%**	1,163.5
4		598.0	**33%**	1,202.0
5		585.2	**33%**	1,214.8
6		705.2	**39%**	1,094.8
7		678.3	**38%**	1,121.7

Notes:

a. The hydraulic calculation does not account for energy losses through intermediate shafts. Due to the excessive number of curves, this alignment is technically unfeasible.

Table 6-3 Tunnel Conveyance Capacity by Microtunneling for 100-Year Rainfall Event (Pipe Diameter = 10 feet)

Alignment	Excess Flow (cfs)	Tunnel Conveyance Capacity (cfs)	Tunnel Conveyance Percentage	Residual Flow Not Conveyed (cfs)
1[a]	3,100	796.5	**26%**	2,303.5
2		703.3	**23%**	2,396.7
3		636.5	**21%**	2,463.5
4		598.0	**19%**	2,502.0
5		585.2	**19%**	2,514.8

6		705.2	**23%**	2,394.8
7		678.3	**22%**	2,421.7

Notes:

a. The hydraulic calculation does not account for energy losses through intermediate shafts. Due to the excessive number of curves, this alignment is technically unfeasible.

7.0 OPINION OF PROBABLE CONSTRUCTION COST (OPCC)

American Association of Cost Engineering (AACE) Level 5 cost estimates for alternatives 2, 3, 4, 5, 6 and 7 are summarized in Table 7-1 below. These cost estimates were developed based on experience and benchmarks from projects of similar magnitudes and ground conditions.

Table 7-1 Cost Estimates

Alternatives ID	Cost Estimate[2] (Million USD)
Alternative 2	103
Alternative 3	76
Alternative 4	85
Alternative 5	51
Alternative 6	78
Alternative 7	112

Note on Costs:

Note 1: Alignments 2 and 7 construct the tunnel from Middlefield Road to the Highway 101 culvert. Alignments 3-6 do not cover the entire length between these two points and do not provide the flood protection provided by Alignments 2 and 7. Alignments 3-6 would provide a lower level of flood protection which could be implemented for lower cost and in less time. Future construction of additional improvements to provide the level of flood protection provided by Alignments 2 and 7 would raise the total costs to a level equal to that of Alignments 2 and 7.

Note 2: The cost of any utility relocation has not been included.

Note 3: The above costs do not include a pump station to drain the tunnel between storms. It is expected that the downstream shaft can be utilized as the wet well. The pump itself would only be required to handle the low flows (<5 cfs) to drain the tunnel. It is suggested that an allowance of an additional $1M be provided for the pump.

Note 4: The costs include construction of the tunneling shafts, and presumes that shafts will be retained for future maintenance access.

Note 5: Operation and Maintenance (O&M) costs are covered in Section 8. The annual estimated cost is calculated to be approximately $125,000 (rounded). A detailed O&M plan will need to be developed based on the actual project design; the plan may result in revisions to the O&M cost estimate.

A cost estimate summary sheet is in Appendix B – Summary of Cost Estimates.

8.0 OPERATION AND MAINTENANCE (O&M) COSTS

The tunnel will require ongoing Operation and Maintenance (O&M).

Based on design work performed by Black & Veatch on a large tunnel project for the Harris County Flood Control District (Houston, TX Metropolitan Area), O&M activities will include the following:

- Dewatering Pump Operation (Electricity/Staff)
- Pump Station Maintenance and Repair
- Staff Costs for Ongoing Maintenance and Security Inspections
- Trash Rack Cleaning
- Trash Rack Maintenance and Repair
- Sediment and Debris Removal
- Five-Year Tunnel Inspections

As part of the Harris County project, Black & Veatch prepared O&M costs for the eight tunnel segments included in the project. The tunnels range from 8 to 25 miles in length, and 35-40' in diameter. The average cost of maintenance was estimated at $78,654/year for each mile of tunnel. The estimates were completed in 2021; applying a 5% annual cost increase results in a cost of $95,958/year per mile for 2025.

The longest tunnel alignment is Alignment 7, at 13,269' or 2.5 miles long. Since the tunnel diameter (10') is smaller than the tunnels in the Harris County study, the cost was reduced by 50% to an annual cost of $47,979/ per mile. This presumes that some costs will be proportional to the diameter and volume of the tunnel, while others will be fixed (such as mobilization). Applying the adjusted Harris County average cost result in an annual estimated cost of $119,947 for Alignment 7; the annual estimated cost is rounded to $125,000. This represents the average annual cost; actual costs will vary due to storm severity and frequency of repairs. The five-year inspections will also skew the costs during the years that inspections are performed.

The costs do not include tunnel repairs or capital replacement costs. The SFCJPA would need to consider setting aside funds for shorter-term replacements such as the pumps and debris racks, as well as longer-term repairs and remediation of the tunnels and shafts. Reserve amounts would be set aside annually based on replacement costs and life expectancy of improvements.

9.0 SUMMARY AND CONCLUSIONS

9.1 Summary of Findings

Microtunneling is the only tunneling method considered feasible and/or beneficial. Alignments 2 through 7 present construction challenges. Tunneling by itself would not provide the desired level of flood protection. Alignment 1, paralleling the creek and using the Woodland Avenue right-of-way is not considered an option for a tunnel alignment.

9.1.1 Tunnel Boring Machine

Use of a Tunnel Boring Machine, although the only method capable of providing full capacity for the 70-year and 100-year storm, was not considered viable due to the lack of available space for the shafts and easements that would be required.

In addition, a large-diameter tunnel poses the following challenges:

- Property acquisitions would likely be required at all shaft locations to provide sufficient space for shaft construction and tunneling logistics. Acquisitions would be a combination of temporary construction easements or right-of-entry as well as permanent easements or fee acquisition over shafts, tunnels, and other improvements.
- Acquisition of the relevant easements to construct a tunnel below private property. Presuming a pipe size of 17’ and 10’ of clearance on both sides of the pipe, an easement of approximate width of 60’ would be required. The length of the tunnel would be approximately 10,000 lineal feet, resulting in an easement totaling 600,000 square feet from multiple private properties.,
- In-depth assessment on potential ways to connect the deep tunnel to the creek, at both upstream and downstream, would be required. A diversion structure would be needed at the upstream end, set to divert flows that exceed the creek capacity. At the downstream end, an outlet structure would be needed to direct flows back into the creek. At both locations, consideration would need to be given to turbulence or erosion, impacts to wildlife, and debris.

9.1.2 Microtunnel Boring Machine

Use of microtunneling may be feasible for Alignments 2-7. The need for smaller shaft construction footprints for microtunneling may allow the public rights-of-way to be used for construction, avoiding the need to route the tunnel under private property. Constructing the shafts in the public rights-of-way would require partial road or intersection closures at each of the shaft locations while the shafts are being constructed and while the microtunnel completes its drive between the shafts. Microtunnel work is typically completed in a staged process where the microtunnel equipment is moved from one completed drive to the next. After finishing a drive, the microtunnel boring machine is retrieved at the reception shaft and relocated to the next launch shaft to begin the next drive. After each drive is completed, the shafts that are no longer required for microtunnel operations are either fitted with a maintenance hole if future access is required or backfilled, and roads/intersections are reopened. This method ensures that road closures are kept to a minimum and are only required during active microtunneling operations. The duration of shaft and tunnel construction varies based on ground conditions, presence of groundwater, shaft support of excavation methods used, and contractor efficiency.

A drawback of microtunneling is the maximum pipe size of 10 feet internal diameter. As noted in Section 6, a pipe of this size would only convey 600-700 cfs. This is only 33-39% of the 1,800 cfs needed to be conveyed for 70-year flood protection and 19-23% needed for 100-year flood protection; additional improvements identified in the WRA Report would be needed to achieve the desired levels of flood protection.

Because of the need to maintain a minimum of 20 feet of cover for the microtunnel alignments (assuming a 10 feet diameter pipe), the tunnel outlet would be below the creek flowline, which is only 15-17 feet deep. The tunnel would act as a siphon during high flow events, with a pump needed to drain the standing water between storms and to remove debris.

9.1.3 Horizontal Directional Drilling

As noted earlier, use of Horizontal Directional Drilling was dropped as an alternative due to the lack of available space for laydown for the pipe string which must be pre-fused prior to installation, difficulty in maintaining a gravity profile, and pipe size limitations that provided minimal flood protection improvements.

9.1.4 Options for Location of Downstream Discharge Point

Alignments 5 and 6 would discharge runoff into the creek at Newell Road, rather than conveying it along West Bayshore Road to the existing culvert crossing under Hwy 101. In addition, Alignments 2, 3, 4 and 7 have the option of discharging runoff into the creek near the West Bayshore/Woodland Avenue intersection, about 650' upstream of the Hwy 101 culvert.

An option to consider is to address the floodwalls as a longer-term issue, treating the flooding problems upstream of Newell Road as a higher priority. Shortening the diversion tunnel by moving the discharge point to a location upstream of the Hwy 101 culvert would allow available resources to be focused on the upstream flooding. The floodwalls could be addressed as a future issue. If this option is chosen, the upstream system should be designed to accommodate the future downstream system.

A benefit of providing additional capacity with a parallel tunnel downstream of Newell Road is that the water surface elevation downstream of Newell Road could be lowered. By removing the higher water surface elevation caused by the floodwalls, the lowered water surface elevation might continue upstream of Newell Road, providing additional flooding relief in the portions of the creek which currently are under capacity. For example, improvements upstream of Newell Road might result in 70-year storm protection, with the completion of the lower segment providing protection up to the 100-year storm. This would require additional hydraulic analysis.

9.1.5 Neighborhood Impacts

Construction will result in impacts that, while temporary, will be challenging. Shaft construction, generally located at intersections, will likely require continuous (24/7) closure of the intersections for the duration of individual microtunnel drives as described in section 9.1.2, impacting neighborhood traffic circulation. Individual driveways could also be impacted. Neighborhood streets would be subjected to ongoing construction traffic. Projects of this nature could take multiple years to complete, and any work in the creeks would be subject to a dry-weather work window that could force completion into the next year.

Post-construction maintenance would result in temporary street closures at the shaft access points to the tunnel (manholes would be installed in the shafts after completion of tunneling). Vacuum trucks or other equipment would be needed to remove sediment and debris trapped in the tunnel. Closures would also be needed to access the pump stations, unless these can be located outside of the street.

9.1.6 Environmental Impacts

The tunnel would create less surface impact to the creek and surrounding neighborhoods than the surface improvements in the WRA Report. However, intake and discharge structures would be needed at the upstream and downstream ends of the tunnel at the junctions with the creek. The structures would require excavation of the creek bank and loss of any trees or vegetation in the excavation area.

Any construction within the creek area for the intake and discharge structures would be limited to the dry season, June to October. These limitations would not apply to the tunnel itself. The tunnel operation could continue throughout the wet season, although surface operations could be slowed.

The structures have not been designed as part of this study; however, the structure openings to and from the creek would need to accommodate the design flow in the tunnel and would likely need an opening at least as wide as the tunnel. Turbulence and velocities at the structures would need to be analyzed to determine the potential for scour. Bank armoring would likely be needed at the transition from the structures to the existing bank

A trash/debris rack would be needed at the intake/discharge holes to prevent both large debris from entering the tunnels, and to prevent people and/or wildlife from being swept into the tunnel. The debris racks would reduce the effective open area of the openings, and an allowance should be provided for partial blockage of the racks during storms. This would require that the overall opening size be increased to compensate for lost capacity.

SF Creek is a route for steelhead migration. Impacts to the steelhead passage would require study, permitting by regulatory agencies, and implementation of mitigation measures to offset impacts. Measures to keep fish out of the tunnel could conflict with the flow capacity needed at the intake and discharge structures. Assessment of impacts to steelhead or other biological impacts was beyond the scope of this study.

9.1.7 Operation and Maintenance

As noted above, the trash/debris racks would require ongoing monitoring and cleaning during high flow events to ensure that the tunnel capacity is maintained.

In addition to wet weather monitoring, ongoing monitoring throughout the year should be performed to ensure security (vandalism, trespassing) and to determine if clearing or minor repairs to the trash racks is needed prior to wet weather

Between storms, the tunnels would need to be pumped out to remove stagnant water and prevent mosquito breeding. The pumps will require ongoing maintenance and a connection to the electrical grid for power.

The pump system will require periodic maintenance including exercising (running) the pumps on a regular basis which may produce noise.

Maintenance and repair operations will require adequate access and potentially temporary equipment parking or material laydown areas.

Stagnant water in the tunnel may lose dissolved oxygen. Release of this water into a creek may deprive aquatic life of needed oxygen. Although this could possibly be avoided by pumping out the tunnels within a few days after a storm, further study would be needed to assess impacts and regulatory agency approval would be needed.

Sediment and smaller debris will accumulate in the shafts. Use of vacuum trucks or other equipment will be needed to remove this material. Testing of sediment may be required prior to disposal to determine if hazardous materials are present. Sediment will need to be transported to a disposal site; the presence of pollutants may require a lengthier and more costly haul.

Clearing of the trash/debris racks will require ongoing work within the creek and require regulatory agency permitting. If permitting is required, it is recommended that a five-year (or longer if possible) programmatic permit be obtained for the work.

9.2 Conclusions

1. The microtunneling option may be feasible. From a technical standpoint, Black & Veatch suggests that microtunneling may worth considering as an option since it could provide up to 39% of the additional flow capacity needed for the 70-year storm protection. In conjunction with other previously identified improvements, a diversion could provide capacity for the 70-year storm and partially close the gap between the 70-year storm and the 100-year storm. A double tunnel, or two tunnels on separate alignments, would provide additional capacity but with additionaladditional cost and impacts.

2. Additional study would be needed to confirm that microtunneling will work and can achieve the desired level of flood protection.protection. This includes determination if there is available space within the public rights-of-way for the shafts, identification of utility conflicts, detailed hydraulic studies, geotechnical investigation, and environmental review.

3. Microtunneling will have significant impacts on adjoining neighborhoods during construction, with lesser ongoing post-construction impacts. Tunnelling will have significant operations and maintenance requirements and costs. The benefits of tunneling should be weighed against the impacts and compared to other previously identified options for flood risk reduction.

9.2.1 Potential Information Needs for Further Study

For any alignments and methods considered suitable for further study, additional steps could include the following, generally in this order:

1. Determine which of the creek improvements listed in the WRA Report should be considered in conjunction with the diversion tunnel to achieve the 70-year and 100-year storm capacity (Alternatives 2 thru 7).
2. Hydraulic Modeling: The existing hydraulic model for Reach 2 would need to be modified to include the selected tunnel alignments developed for the selected alignments. The model would presume that the selected tunnel alignment and the selected WRA Report options are in place. The model should include two alternatives: (1) The downstream end the improvements would be at Newell Road (floodwalls remain in place) and (2) improvements continue for Newell Road downstream to West Bayshore Road culvert (floodwalls are removed or lowered). This would allow for any positive impacts to the water surface elevation upstream of Newell Road based on downstream improvements with the flood walls removed.
3. A detailed alignment should be developed for each alignment to be considered. This would include review of available staging for shaft construction and a preliminary review of existing utilities that could conflict with the shaft construction. It should also include a preliminary design of the intake and discharge structures to determine the size and layout of the structures and impacts to the creek bank and vegetation.

4. Discussion with regulatory agencies (California Department of Fish and Wildlife, San Francisco Bay Regional Water Quality Control Board, US Army Corps of Engineers) should be considered to cover impacts to steelhead passage and other environmental concerns.
5. Sediment Characterization and Sediment Transport Modeling: A review of available information of the sediment particles in the creek that could enter the tunnel would be performed. With the available information, a sediment transport model would be developed (likely based on the hydraulic model described above and watershed soil sampling) to estimate the quantity of sediment that could accumulate in the tunnel and possible ways to clean it.
6. Additional geotechnical investigations will be required to characterize the ground conditions along each alignment and confirm the suitability of the proposed tunneling method.
7. Utility surveys will be required along each alignment to identify any potential utility conflicts with the tunnel and finalize the depth of the alignment.

10.0 REFERENCES

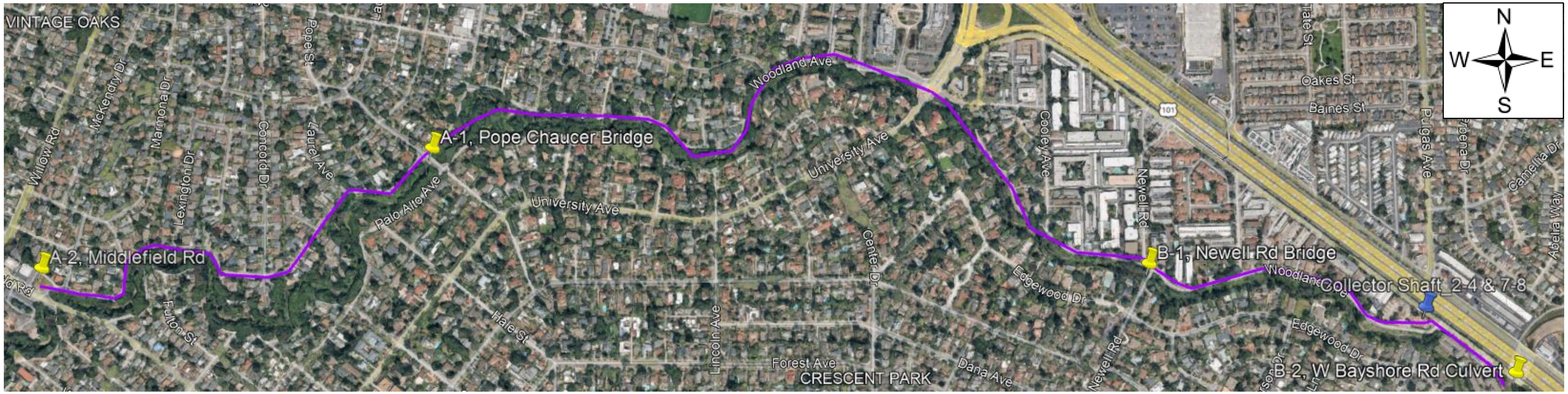
1. San Francisquito Creek Joint Powers Authority (SFCJPA). (2021). Basis of Design for San Francisquito Creek Reach 2 Project, Appendix E - Geotechnical Memorandum/Preliminary Foundation Report Newell Road Bridge (Replace), Page 38 of 852.
2. WRA Environmental Consultants. (2025). Alternatives Evaluation Technical Report – Urban Reach 2 Project, Public Draft.
3. San Francisquito Creek Joint Powers Authority (SFCJPA). (2008). Early Implementation Project (EIP) Preliminary Analysis Document, Draft.
4. ASCE 36-15 – Standard Design and Construction Guidelines for Microtunneling.
5. ASCE MOP 08 – Pipeline Design For Installation by Horizontal Directional Drilling – Third Edition

11.0 APPENDICES

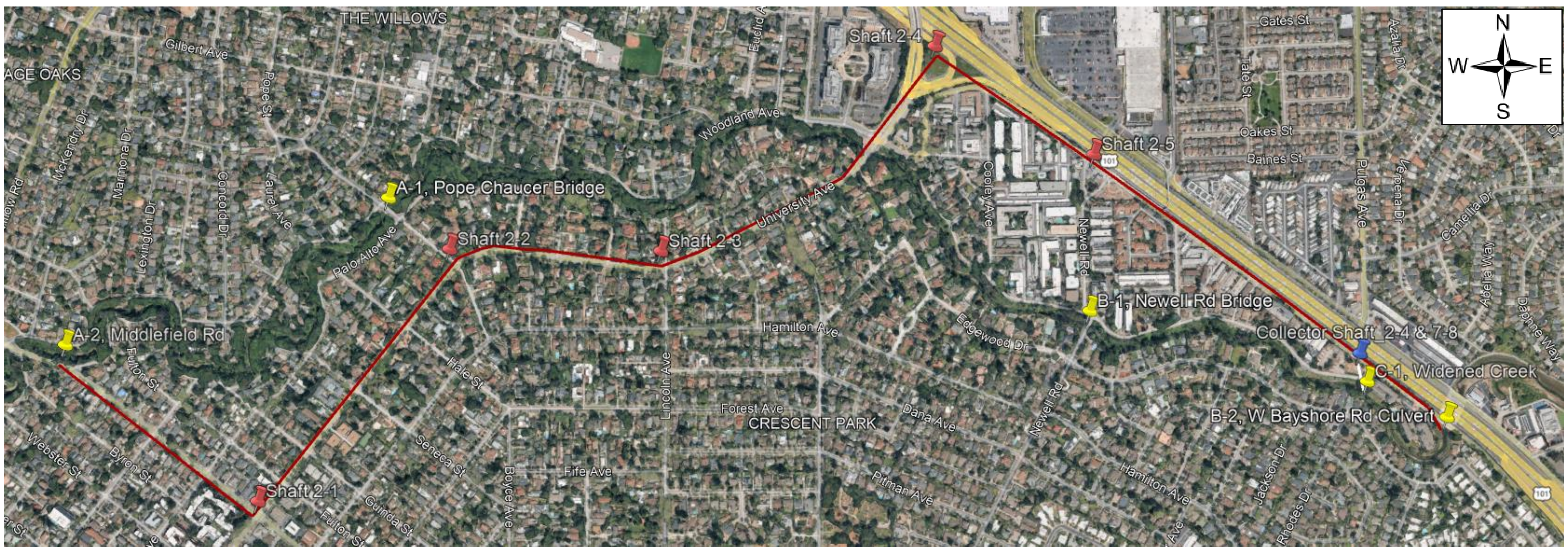
11.1 Appendix A – Conceptual Plan and Profile Drawings

11.2 Appendix B – Summary of Cost Estimates

11.3 Appendix C – Aerial Views of Considered Tunnel Alignments

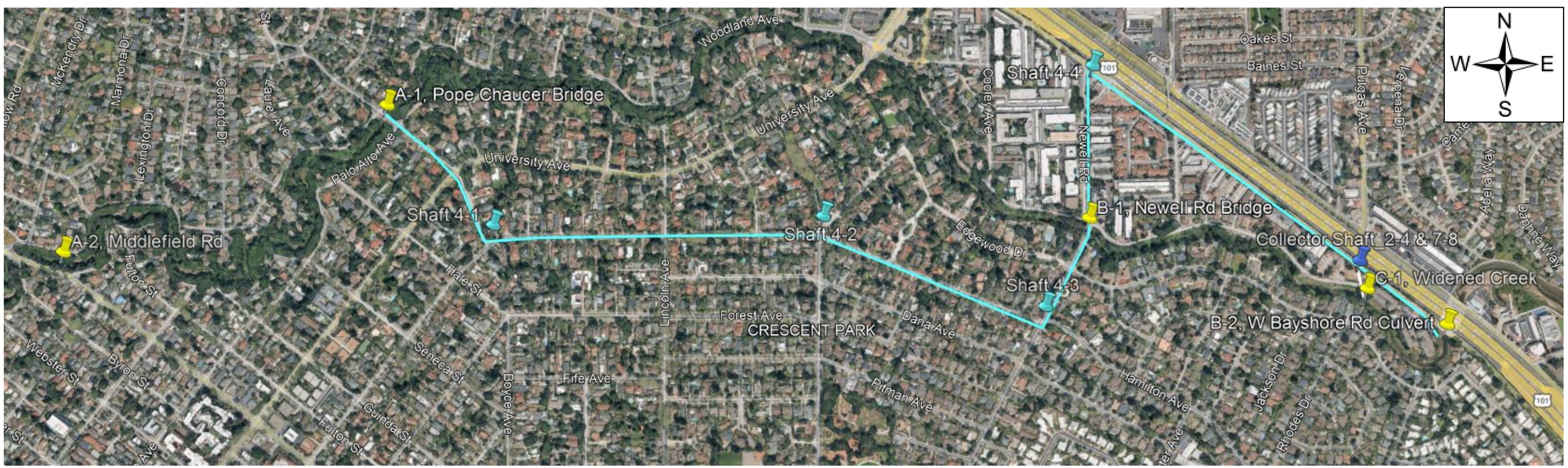


Alternative 1

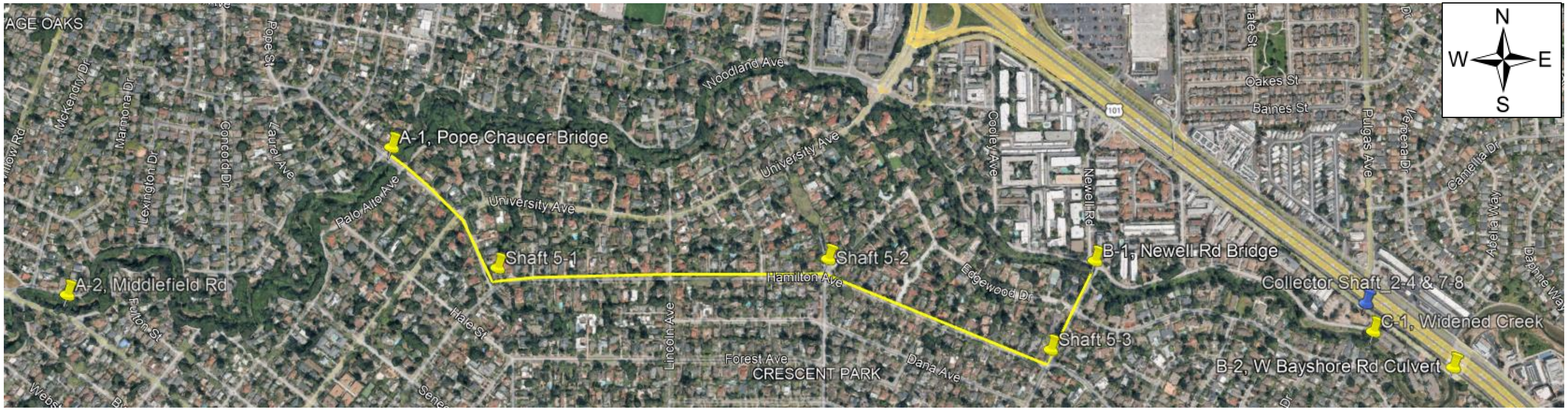


Alternative 2

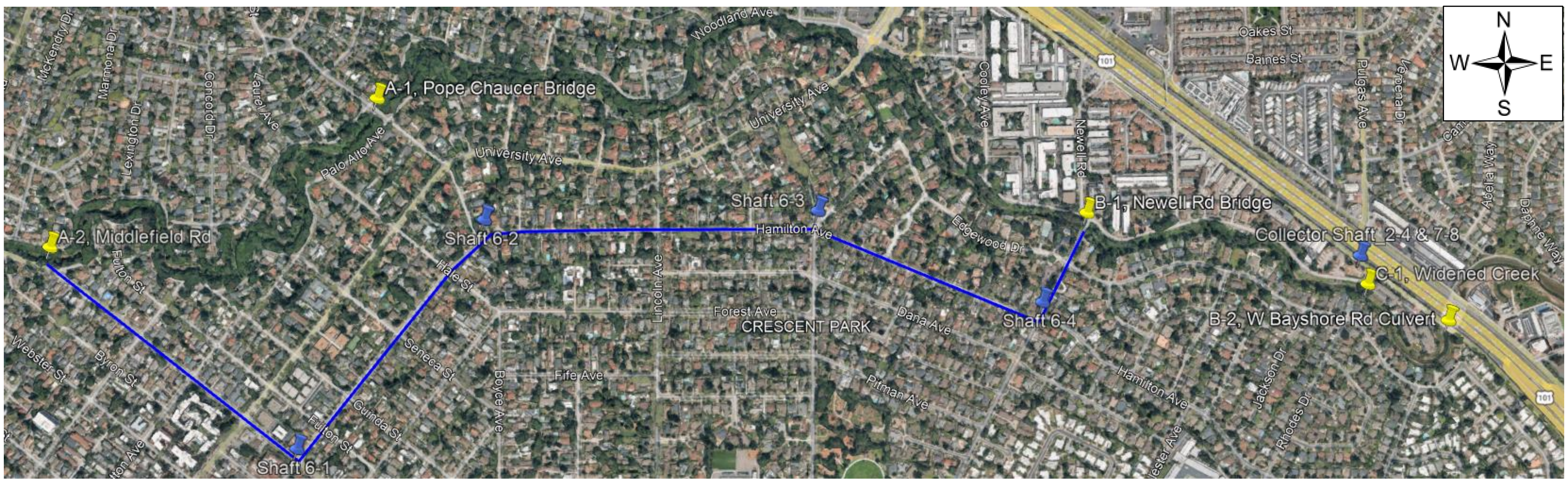
Alternative 3



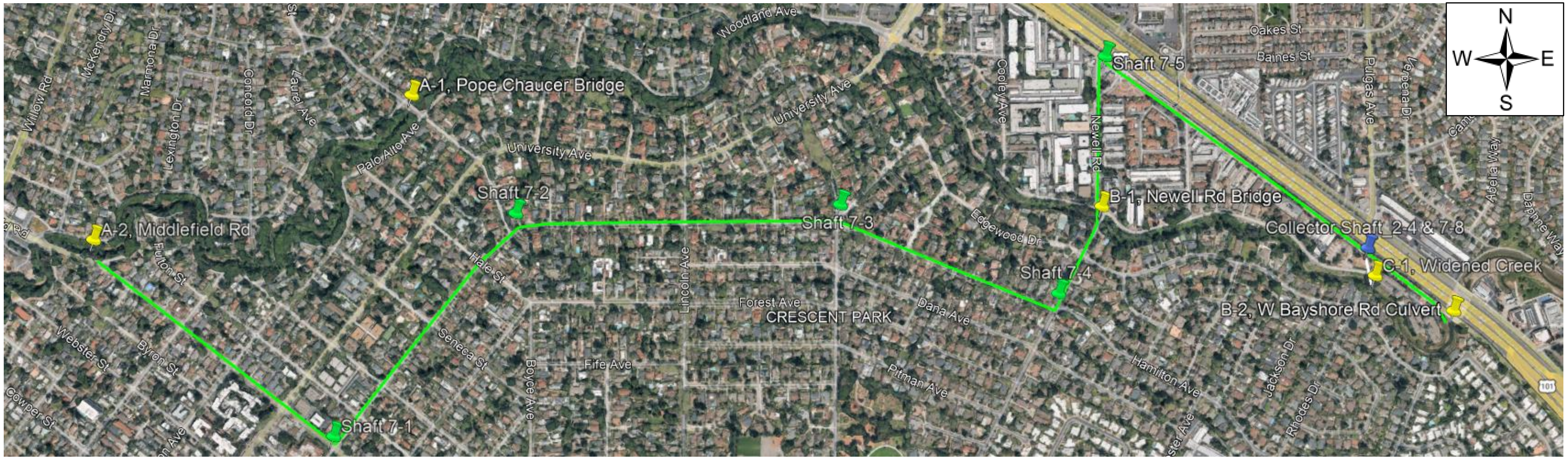
Alternative 4



Alternative 5



Alternative 6



Alternative 7