Silicon Valley Clean Water

Influent Connector Pipeline Project Planning Report

Task Order 2016-04

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Project Background

Silicon Valley Clean Water (SVCW) is a Joint Powers Authority (JPA) that owns and operates a regional wastewater treatment plant (WWTP) at the eastern end of Redwood Shores, within Redwood City, and related wastewater pumping and transmission facilities. SVCW treats the majority of the wastewater generated from the mid-peninsula of San Mateo County south of the San Mateo Bridge. The JPA members include the cities of Belmont, Redwood City, and San Carlos, and the West Bay Sanitary District (which provides sanitary sewer collection services to the cities of Menlo Park, Portola Valley, and portions of Atherton, Woodside, East Palo Alto, and unincorporated areas of San Mateo County).

The individual members of the JPA own and operate the sanitary sewer collection systems within their respective jurisdictions. West Bay Sanitary District (WBSD) also owns the existing flow equalization facility (FEF) that is leased to SVCW and used to store wastewater during wet weather conditions. SVCW owns and operates the WWTP and the sanitary sewer force main and pump stations that convey the wastewater from the member agency connections to the treatment plant.

SVCW is implementing a Capital Improvement Program (CIP) to improve the reliability of the conveyance system. The CIP will consist of the following elements: replacement of conveyance system pump stations; replacement of conveyance system force mains; and upgrades to SVCW's treatment facility. A Conveyance System Master Plan (CSMP) was issued in 2011 and initial steps of the Plan are being implemented. The CSMP identifies an influent connector pipeline (ICP) to transport 80 million gallons per day (mgd) of raw wastewater from the newly proposed Headworks Facility to the existing wastewater treatment plant (WWTP) upstream of the existing screening facility.

The CIP includes 17 related components, including improvements and upgrades throughout the conveyance system and SVCW's WWTP. The CIP includes 7 projects that upgrade the conveyance system along with 10 projects that improve the existing WWTP. The ICP Project is grouped with the 10 WWTP improvement components of the CIP.

The 7 conveyance system upgrades include the following:

- Gravity Pipeline,
- Belmont Force Main Rehabilitation,
- Belmont Pump Station Rehabilitation,
- San Carlos Pump Station Site Improvements,
- San Carlos Odor Control Facility,
- Redwood City Pump Station Replacement, and

- Menlo Park Pump Station Rehabilitation.

The 10 WWTP improvement components include the following:

- Receiving Lift Station (RLS),
- Headworks Facility,
- Odor Control Facility,
- Flow Diversion Structure,
- Nutrient Removal Facilities,
- Secondary Clarifiers,
- Stormwater Treatment Planters,
- Stormwater Pump Station,
- Civil Improvements for the Front of Plant area, and
- Influent Connector Pipeline (ICP), which is the subject of this report.

Report Purpose

This planning report presents the current thinking regarding the Influent Connector Pipeline Project, which is one of several projects included in an overall CIP being executed by SVCW. The purpose of this report is to:

- Provide information for SRF Planning Loan Compliance,
- Provide information for SRF Construction Loan Application,
- Document the work completed during the alternatives analysis, and
- Identify additional recommendations and outstanding issues.

Information provided summarizes the historical work in developing the ICP concept and is not intended to be final for work moving forward.

Project Benefits

The ICP Project offers many benefits, including but not limited to the following:

- Connection of the proposed Headworks Facility to the existing WWTP;
- Increased reliability over use of the existing influent forcemain;
- Redundancy for maintenance, inspection, and repairs;

- Gravity flow over the range of influent flow conditions, without excessive head loss or settling of solids;
- Improved structural and seismic performance; and
- Creation of a sealed joint-less pipe system which eliminates the leaks that are currently being experienced with the existing influent forcemain.

Site Location

The existing influent forcemain, constructed as part of the original treatment plant facility, is located to the south of the treatment plant property boundary line. The Influent Connector Pipeline will be located within SVCW's property at 1400 Radio Road, Redwood Shores, California.

Relationship to Other Projects

The ICP will convey wastewater from the new Headworks Facility to the existing WWTP.

Upon completion of all the CSMP Projects, wastewater will be conveyed by the new Gravity Pipeline to the RLS. The wastewater will then be pumped to the new Headworks Facility. From the new Headworks Facility the wastewater will flow by gravity through the ICP to the existing WWTP primary treatment processes.

Construction of the ICP can begin once the Civil Improvements soil stabilization, parking, and road access elements are complete. The Headworks Facility can start up once either the entire ICP is completed and the existing influent force main is abandoned, or the western segment of the ICP is completed with a temporary tie-in to the existing influent force main. The latter option is referred to as "Early Startup" and is described further in Section 2.5.2.

The ICP Project will be constructed under a single progressive design-build contract, along with the Receiving Lift Station, New Headworks Facility, Odor Control Facility, and Electrical Infrastructure.

Alternatives Analysis

An Alternatives Analysis was conducted to evaluate different ICP configurations and alignment alternatives. A total of 8 alternatives, summarized in Table ES-1 were analyzed and quantified to aid in selecting the preferred, recommended alternative.

Table ES-1 ICP Alignment Alternatives

Option Name	Description
Alternative A : Rehabilitation of Existing Pipeline	Alternative A includes installing 225 feet of new 63" HDPE pipe, using open cut construction, to connect the future headworks facility to the existing influent line and rehabilitating 575 feet of 54" RCP (Reinforced Concrete Pipe) and 175 feet of 60" RCP.
Alternative B: Replace Existing Influent Line	Alternative B requires the removal of a portion of the existing 54-inch and all of the 60-inch influent line in order to install, in its place, the 975 feet of 84-inch HDPE pipe from the new headworks facility to the treatment plant.
Alternative C: New Pipe Alignment	Alternative C includes installing 900 feet of new 84-inch HDPE pipe in a new alignment routed within the street right of way and plant property boundary.

Option Name	Description
Alternative D: Microtunnel in New Alignment	Alternative D includes installing 940 feet of new 84-inch HDPE pipe, with 600 of the 1100 feet microtunneled inside a new 90-inch steel casing. The alignment will follow nearly the same alignment as Alternative C.
Alternative E: CIPP + New Alignment	Alternative E combines Alternatives A and C to install 900 feet of new 66-inch HDPE pipe in the same alignment as Alternative C and connects to and rehabilitate the existing influent line as outlined in Alternative A.
Alternative F1: Parallel Pipes	Alternative F1 combines Alternatives B and C to install a total 1850 feet of new HDPE pipe in a parallel configuration. Nearly 900 feet of HDPE pipe will be routed along the alignment as outlined in Alternative B while 975 of HDPE pipe will be routed in the alignment as outlined in Alternative C.
Alternative F2: Parallel Pipes	Alternative F2 follows the Alternative B alignment but involves installing a total of 1900 feet of new HPDE pipe in a parallel configuration thereby introducing redundancy to Alternative B.
Alternative F3: Parallel Pipes	Alternative F3 follows the Alternative C alignment but involves installing a total of 1800 feet of new HDPE pipe in a parallel configuration thereby introducing redundancy to Alternative C.

Common to all eight alternatives was the use of HDPE pipe material. Therefore, the recommended alternative was selected based on other factors besides pipe material. HDPE was selected as an initial candidate because it is joint-less and flexible and offers excellent corrosion resistance. Of the 8 alternatives, Alignment F3 was chosen as the recommended alternative alignment because it:

- Provides needed capacity of 80 MGD with minimal headloss,
- Avoids San Francisco Bay Conservation and Development Commission (BCDC) jurisdiction and permitting requirements,
- Increases reliability and redundancy over other alternatives due to the dual pipeline arrangement,
- Allows the existing 54-inch RCP forcemain to remain in service during construction so no bypassing is required, and
- Appears comparable to other feasible alternatives in terms of cost, constructability, head loss, operational complexity, conflicts with existing utilities, and impacts to plant access and parking.

The selected alternative, Alternative F3 is comprised of a dual pipeline of two different diameters to accommodate the full range of diurnal and seasonal flow variation. The ICP is routed within the property boundary of the existing WWTP, eliminating the need for property acquisition. At the time of writing, it was assumed the ICP will be constructed using a joint-less piping system such as high-density polyethylene (HDPE).

Hydraulic Analysis

Hydraulic calculations were completed to analyze head loss along the pipeline.

In conducting the hydraulic analysis for wet weather flow, the following assumptions were made:

- 2040 peak wet weather flow (PWWF) is 80 MGD
- The entrance to both pipes is from a common hydraulic source (represented as a "reservoir" in the hydraulic model)
- Water surface elevation (WSE) in the Influent Mix Box (the downstream end of ICP) at 80 MGD is 111.69, based on the Stage 1 Influent Screening Drawings (Brown & Caldwell, 2014)

Based on these assumptions, and the head loss calculations detailed in Section 6.2, the WSE in Distribution Box 2 (the upstream end of ICP) will be 113.74 at PWWF. Since the WSE at the outlet of the Headworks Facility will be higher than 113.74 at PWWF (see Figure 6-1), the system can flow by gravity and does not require pumps.

In conducting the head loss analysis for dry weather flow, the following assumptions were made:

- 2015 peak dry weather flow (PDWF) is 22.5 MGD
- Only the 48-inch diameter pipe is engaged during flows under 22.5 MGD
- WSE in the Influent Mix Box at 30 MGD is 107.12 based on the Stage 1 Influent Screening Drawings (Brown & Caldwell, 2014), so the WSE at 22.5 MGD is no higher than 107.12

Based on these assumptions, and the head loss calculations detailed in Section 6.2, the WSE in Distribution Box 2 (the upstream end of ICP) will be 109.17 at PDWF. Since the elevation of the weir to Distribution Box 2 is 113.60, there will be a free discharge across this weir at 22.5 MGD, and no pumping is required.

In order to meet maximum head loss and minimum velocity criteria under both wet weather and dry weather flows, a combination of pipe sizes was chosen:

- 48-inch diameter pipe to convey dry weather flow
- 72-inch diameter pipe to engage in parallel with the 48-inch diameter pipe to convey wet weather flow

The 48-inch pipe will convey flows up to the PDWF of 22.5 mgd. Above 22.5 mgd, the gate at the upstream end of the 72-inch pipe will open, and both pipes will be in service to convey flows up to the PWWF of 80 mgd.

Additional Design Considerations

Additional design considerations that must be carried from this planning phase into design and construction of the ICP Project are listed below and summarized further in Section 7 of this report:

- Constructability in highly compressible soils
- Construction sequencing including the direction of the pipeline construction as well as coordination with other CSMP projects and the two progressive design-build projects (Gravity Pipeline and Front of Plant)

- Safe access considerations for plant staff, deliveries, and visitors as well as construction staff
- Corrosion protection as the ICP will be constructed in marine soils where ground water will be brackish and corrosive
- Operational plan of the two pipes in the ICP during wet weather and dry weather flows
- Differential settlement considerations between existing and new structures and the ICPs
- Power supply for all new motorized equipment included in the project
- Instrumentation and SCADA requirements for motor operated valves and pumps
- Interim operations and bypass requirements of the 18-inch Redwood Shore Forcemain
- Geotechnical considerations within the project area
- Environmental impacts to the visual environment, air quality, greenhouse gas emissions, and biological resources
- Staging and storage areas
- Excavation material disposal and reuse
- Dewatering of the larger diameter pipe when it is taken offline during dry weather flows

Life Cycle Cost

A life cycle cost analysis was performed for the selected alignment alternative, Alternative F3 described above. This life cycle cost for the SVCW ICP Project does not include Design costs but does include the following cost components:

- Capital Cost
- O&M Labor
- Power
- Equipment Rehabilitation and Replacement

The cost for each of the components listed above were developed for each year over a 75 year period between 2018 and 2093 in present day dollars. A 75 year period was chosen because, according to the Plastic Pipe Institute, HDPE has a useful life of 75 to 100 years. Because of the soil conditions at the existing project site and to be conservative, the 75-year life cycle was selected. Using these costs, the Net Present Value over the 75-year life was calculated and is presented in Table ES-2 below.

Table ES-2. Total Life Cycle Costs

	Cost
Construction Cost (2016 Dollars)[1]	
Construction Cost	$4,300,000
Capital Cost (2018 Dollars)[2]	
Base Market Fluctuation	$8,000,000
Low Market Fluctuation	$7,600,000
High Market Fluctuation	$8,500,000
Annual O&M Labor Costs	
Annual Labor Cost	$6,000
Annual Power Costs	
Annual Power Cost	$6,500
Rehabilitation and Replacement Costs	
Motorized Gate Repair Cost (every 5 years/Gate)	$3,500
Condition Assessment Inspection Cost (every 10 years/Pipe)	$11,500
Sump Pump Replacement Cost (every 10 years/Pump)	$400,000
Pipe Cleaning Cost (every 20 years/Pipe)	$18,100
Pipe Breakage Repair Cost (once per lifetime)	$500,000
75-Year Life Cycle Cost (LCC) for Influent Connector Pipe	
Capital Cost[3]	$7.6 - $8.5 million
NPV of Labor, Power, and Rehabilitation/Replacement	$3.7 million
75-year LCC (2022 Dollars)[2]	$11.3 - $12.2 million

[1] Raw Construction Cost in 2016 dollars (US) based on the construction cost included in the Alternatives Analysis TM presented to SVCW. This differs from the Construction Cost presented in the Opinion of Probable Cost of Construction TM, dated May 2016 ($4,424,000) due to the inclusion of different contingency costs.

[2] Capital Cost reflects the Raw Construction Cost with Project Contingency, Soft Costs, Market Fluctuations, and Escalation applied to the raw cost.

[3] Range based on market fluctuations from -5 to 15 percent.

Outstanding Issues to Carry Into Design

Outstanding issues to be carried over into the design phase of the Project include the following and are summarized in further detail in Section 11 of this report:

- Detailed design of the connection to the existing WWTP and associated valving;
- Construction Schedule;
  - Timing of construction of the ICP in relation to the new Headworks Facility and other Front of Plant projects;
- Control strategy for switching between one pipe and two pipe operations for wet weather flow;
  - Management of standing water in the wet weather pipe after use (draining or chemical dosing);
- Review of joint-less pipe technology, materials, and application to project;
- Site Survey, topographic survey, and examination of the property boundary;
- Utility location surveying/potholing;
- Review of constructability;

- Soil borings (supplemental sub-surface exploration);
- More detailed Project-specific hydraulic calculations;
- Buoyancy structure analysis and hydraulic mapping;
- Coordination with the storm water pollution prevention plan (SWPPP);
- Update of cost estimate based on preferred alignment, selected pipe sizes, trench design, backfill material, and pipe material;
- Determination of need and means of access inside the pipe; and
- Means of access for deliveries, plant operations, and visitors during construction of the Project.

Section 1

Introduction

1.1 Introduction and Project Purpose

Silicon Valley Clean Water (SVCW) completed a Conveyance System Master Plan (CSMP) in 2011. The CSMP identified recommended improvements for the reliability of the conveyance system and the wastewater treatment plant (WWTP). These improvements have been incorporated into SVCW's Capital Improvement Program (CIP) and are referred to collectively as the Wastewater Conveyance System and Treatment Plant Reliability Improvement Project (Reliability Improvement Project). The Reliability Improvement Project consists of the following elements: replacement of conveyance system pump stations; replacement of conveyance system force mains; and upgrades to SVCW's treatment facility.

The Program identifies an influent connector pipe. The purpose of the new Influent Connector Pipes (ICP) is to convey flow from a newly proposed Headworks Facility to the existing WWTP, to some point upstream of the existing screening facility. This interconnecting pipeline will transport up to 80 mgd of screened and degritted raw wastewater.

This report details project planning for the ICP project and will provide an overall summary of the project development phase of the ICP project.

1.2 Background

SVCW is a Joint Powers Authority (JPA) that owns and operates a regional wastewater treatment plant at the eastern end of Redwood Shores, within Redwood City, and related wastewater pumping and transmission facilities. SVCW treats the majority of the wastewater generated from the mid-peninsula of San Mateo County south of the San Mateo Bridge. The JPA members include the cities of Belmont, Redwood City, and San Carlos, and the West Bay Sanitary District (which provides sanitary sewer collection services to the cities of Menlo Park, Portola Valley, and portions of Atherton, Woodside, East Palo Alto, and unincorporated areas of San Mateo County).

The individual members of the JPA own and operate the sanitary sewer collection systems within their respective jurisdictions. West Bay Sanitary District (WBSD) also owns the existing flow equalization facility (FEF) that is leased to SVCW and used to store wastewater during wet weather conditions. SVCW owns and operates the wastewater treatment plant (WWTP) and the sanitary sewer force main and pump stations that convey the wastewater from the member agency connections to the treatment plant.

1.3 Existing Conveyance System

SVCW's existing conveyance system assets include four pump stations, one for each of the four member agencies, a wet weather booster station located in the San Carlos Pump Station, an influent lift station located at the WWTP, and an approximately nine-mile-long force main. SVCW leases from the WBSD a flow equalization facility, which is an integral part of SVCW's existing conveyance system.

1.4 History of SVCW and the Conveyance System

To understand the need for the Reliability Improvement Project, it is useful to know the history of SVCW, the assumptions used during the original design of the conveyance system, why the various components were built, and why at different times. This description of the history of SVCW will illustrate that the conveyance system is being operated in a manner different than its original design intent and, now, beyond its useful life.

Until the mid-1960's, the mid-peninsula cities had their own wastewater treatment plants (WWTP). Redwood City Sanitary District owned and operated the Redwood City Sewage Treatment Facility. Belmont and San Carlos owned and operated the Belmont/San Carlos Joint Sewage Treatment Facility. The developer of Redwood Shores (Mobil Land) owned the Redwood Shores Treatment Plant, and it was operated by Redwood City Sanitary District. The Redwood City and Belmont/San Carlos plants separately discharged effluent to San Francisco Bay. The Redwood Shores Plant consisted of oxidation ponds and had no discharge as all the wastewater was evaporated. The level of treatment provided by these three plants and the locations of their outfalls could not meet the new stricter wastewater treatment and disposal regulations being imposed and developed at the state (Porter-Cologne Act, 1969) and federal (Clean Water Act, 1972) levels.

The Regional Water Quality Control Board (Regional Board) ordered a 10-to-1 dilution requirement for San Francisco Bay discharges. With encouragement from the Regional Board, in June 1969, the three cities formed the Strategic Consolidation Sewerage Plan Joint Powers Authority (SCSP JPA) for the purpose of addressing the new water quality regulations on a regional basis. To meet the 10-to-1 dilution requirement as soon as possible, the SCSP JPA would build connecting pipelines and a deep-water outfall for discharging the effluent from the existing three small treatment plants in advance of constructing the regional treatment plant. The site of the regional treatment plant needed to be decided so design of the new outfall could begin. After considering several sites, the SCSP JPA selected the Redwood Shores Plant site at the mouth of Steinberger Slough for the regional plant.

The pipeline consisted of six miles of reinforced concrete pipe that connected the treatment plants to the deep-water outfall located at the mouth of Steinberger Slough[1]. This new conveyance system was designed as a low pressure force main. In 1969 designs were completed for the pipeline as well as for the Redwood City Pumping Plant and the San Carlos Pumping Plant. These pumping plants were built adjacent to the respective individual treatment plants. The pump stations, pipeline, and deep water outfall were put into service in 1971. The outfall, pipeline, and the Redwood City Pumping Plant (renamed Redwood City Pump Station) are still in use today.

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[1] It should be noted that reinforced concrete pipe was the pipe of choice when the pipeline was designed in the early 1970's. High density polyethylene (HDPE) pipe was not available in large diameters at that time. The highly corrosive nature of the Redwood Shores saline soils made steel a poor candidate for this alignment.

Concurrent with the SCSP JPA improvement plans, Belmont's capital plans anticipated needing a new pump station and a pipeline that would connect it to the Belmont/San Carlos Joint Plant until the regional plant was operational. By the time the regional plant was operational and the Belmont/San Carlos Joint Plant closed, Belmont would also need a direct connection to the new SCSP force main. Design for a new pump station and direct connection forcemain on the west side of U.S. Highway 101 finished in 1973. The force main consisted of two segments. The first was from the new Belmont pump station to the point of the future connection to the 54-inch force main. This section was 1200 feet of 24-inch wrapped and cement lined steel pipe. The second segment was downstream of the future connection point and terminated at the San Carlos/Belmont Joint Plant. In this segment the pipe size was reduced to 20-inches and the material changed to asbestos cement pipe. This change in size and material was likely due to the City wanting to reduce costs for this segment that would be used for less than 10 years.

In the mid-1970's, in response to Regional Board direction, the service area for the regional plant originally envisioned by the SCSP JPA expanded to include the West Bay Sanitary District service area. In November 1975 the members of the SCSP JPA and West Bay Sanitary District (previous named Menlo Park Sanitary District) founded South Bay System Authority (SBSA, renamed in 2014 to Silicon Valley Clean Water) JPA as the successor to the Strategic Consolidation Sewerage Plan JPA.

This addition necessitated expanding the conveyance system to connect WBSD. Design of a 2.7-mile-long 33-inch diameter reinforced concrete pipe force main between the Redwood City Pump Station and the future Menlo Park Pump Station site was completed in 1976. The pipe was put into service when the regional plant became operational in 1982. The addition of WBSD to the system required that a booster pump station be added to the force main system, as the additional WBSD flows were not anticipated in the original forcemain headloss and pressure calculations.

1.5 Reasons the Reliability Improvement Project is Needed

The SVCW Wastewater Conveyance System and Treatment Plant Reliability Improvement Project is necessary to eliminate ongoing reliability concerns and accommodate changes in wastewater flowrates. Replacement of the conveyance system is SVCW's highest priority due to its age and continual state of failure. The existing SVCW conveyance system components are beyond their useful life. The American Society of Civil Engineers published a report entitled "Failure to Act" with the purpose "to provide an objective analysis of the economic implications for the United States of its continued underinvestment in infrastructure." Table 1-1 lists the useful life for force mains and pump stations used in the ASCE report.

Table 1-1. Useful Lives of Wastewater Pump Stations and Force Mains

Component	Useful Life (years)
Force Mains	25
Pumping Stations – Concrete Structures	50
Pumping Stations – Mechanical and Electrical	15

Source: Table 5 of *Failure to Act, the economic impact of current investment trends in water and wastewater treatment infrastructure.* American Society of Civil Engineers. 2011.

1.5.1 Force Mains

SVCW's 46-year-old concrete force main is in poor condition and needs to be replaced. The pipeline suffers from several problems caused by the soils in which it is installed and the sewage characteristics. Problems have compounded, resulting in a history of numerous leaks. These leaks range from minor to the occasional catastrophic failure. Leaks require repairs along streets and in backyards and sometimes within biologically sensitive environments.

One section of the original force main that had the most leaks was replaced in 2015 with a fused-jointed high density polyethylene (HDPE) pipe. This was a 1.7-mile long portion of the 48-inch diameter force main from the Redwood City Pump Station to the north end of Inner Bair Island. The Reliability Improvement Project will replace the remaining original force main that begins where the 48-inch replacement project ended (the north end of Inner Bair Island) and terminates at the WWTP.

Much of the existing force main is buried in young bay mud soils that are poorly suited to the existing pipeline material and joint system. Young bay mud has two main problems; it is expansive and corrosive. Expansive soils are weak, unstable, have high shrink-swell potential, and settle over time. The pipeline consists of 12-foot-long reinforced concrete pipe sections that are connected to each other with single non-restrained "O-ring" joints. The young bay mud soil does not provide sufficient support for the reinforced concrete pipe and its joints. This results in pipe movement and separation at the joints and is the cause of the majority of the leak events.

The bay mud soil is highly corrosive to buried steel and concrete that comes into direct contact with the soil. The pipe is also subjected to microbiologically influenced corrosion (MIC) from sewer gases inside the pipe. Internal and external corrosion of the concrete and reinforcing steel leads to more significant leaks. When surges in flow occur (such as during a power outage) the resulting pressure and vacuum surge conditions have broken the weakened pipeline resulting in major sewage spills. These types of leaks tend to be catastrophic with the potential of uncontrollable discharge of untreated wastewater to the environment.

The frequency of pipeline leaks is expected to increase as the pipe ages, given the current poor condition of the pipelines, continued movement of weak soils, and acceleration of the internal and external corrosion.

In addition to the problems related to the soil, the existing pipeline was designed as a low-pressure force main pipeline and not for typical force main pressures. When WBSD was added to the conveyance system and as wet weather flows have risen, flows in the force main have grown higher than the original design anticipated. When the WBSD flows were added, a booster pump station, and later a flow equalization facility, were added to the system.

With Herculean efforts, SVCW maintains pressures and surges in the conveyance system to within the force main's pressure limits, though this approach comes with significant risk. SVCW must carefully manage the flow in the pipeline to minimize leaks by opening and closing valves, turning on and off pumps (including the booster and influent lift pumps), diverting flow to storage, and backing up sewage in member agency collection systems. During wet weather events, wastewater flows from the WBSD collection system are diverted to the WBSD flow equalization facilities. When flows subside, the WBSD wastewater is pumped from the flow equalization facilities through the Menlo Park Pump Station and to the treatment plant. Sometimes these pressure management efforts require using all available pumps and valves leaving limited or no backup equipment.

1.6 Proposed Conveyance System Project Overview

The Reliability Improvement Project includes 17 related components, including improvements and upgrades throughout the conveyance system and SVCW's WWTP. The Reliability Improvement Project proposes a combination of rehabilitating, repurposing, and decommissioning existing SVCW conveyance system assets, and the construction of replacement assets.

The conveyance system upgrades include the following seven (7) projects:

- Gravity Pipeline,
- Belmont Force Main Rehabilitation,
- Belmont Pump Station Rehabilitation,
- San Carlos Pump Station Site Improvements,
- San Carlos Odor Control Facility,
- Redwood City Pump Station Replacement, and
- Menlo Park Pump Station Rehabilitation.

Although the ICP project is included in the same CSMP as the above conveyance system upgrade projects, the ICP project is grouped with the other WWTP improvements components of the CSMP. These WWTP improvement components include the following 10 projects:

- Receiving Lift Station (RLS),
- Headworks Facility,
- Odor Control Facility,
- Flow Diversion Structure,
- Nutrient Removal Facilities,
- Secondary Clarifiers,
- Stormwater Treatment Planters,
- Stormwater Pump Station,
- Civil Improvements for the Front of Plant area, and
- Influent Connector Pipeline, which is the subject of this report.

1.6.1 New Headworks Facility

The new Headworks Facility will be constructed downstream of the receiving lift station to provide coarse screening and grit removal from the raw wastewater. This is a new treatment process being added to the WWTP treatment train. The ICP will be built to connect the headworks to the existing primary treatment process.

1.6.2 ICP

The existing influent forcemain conveys wastewater to the existing WWTP. However, once the new Gravity Pipeline, Receiving Lift Station, and Headworks Facility are constructed, a new influent connector pipeline is needed to connect the new Headworks Facility to the existing WWTP. The ICP project, as included in the CIP, is proposed to be this connecting link.

An Alternatives Analysis was conducted to evaluate different ICP configurations and alignment alternatives. The analysis identifies an alternative alignment that will replace the existing influent 54-inch RCP forcemain, adjacent to the WWTP. The selected alternative is comprised of two pipes of different sizes, to accommodate all weather conditions (both seasonal and diurnal flow ranges). The ICP are routed within the property boundary of the existing WWTP, eliminating property acquisition and easement requirements. At the time of the creation of this report, it was assumed the ICP will be constructed using a sealed joint-less piping system such as High Density Polyethylene (HDPE).

1.7 Project Benefits

The ICP Project offers many benefits, including but not limited to the following:

- Connection of the proposed Headworks Facility to the existing WWTP;
- Increased reliability over use of the existing influent forcemain;
- Redundancy for maintenance, inspection, and repairs;
- Minimal headloss, to reduce system pumping requirements;
- Accommodation of the full range of influent flows without excessive head loss and reduced potential for settling of solids;
- Superior structural/seismic performance; and
- Creation of a sealed joint-less pipe system by use of HDPE which eliminates the leaks that are currently being experienced with the existing influent forcemain.

Section 2

Site Location and Relationship to Other Projects

2.1 Site Location

The Influent Connector Pipeline will be located within SVCW's property at 1400 Radio Road, Redwood Shores, California as shown in Figure 2-1. The ICP begins in the area immediately adjacent to the west side of the existing WWTP facilities and is routed along the south side of the existing plant parking area within the WWTP fence boundary, and connects to the existing WWTP at its south east corner. The physical area is shown in Figure 2-2.

2.2 Summary of Field Investigations

A geotechnical investigation was conducted by CDM Smith during the planning phase of the Project. This recent geotechnical investigation by CDM Smith completed the following tasks:

- Reviewed historical and on-going geotechnical investigations, as-built drawings, and other construction records for other improvements in the project area. Relevant exploration logs and laboratory test results were extracted for inclusion in the geotechnical data report and to refine the project's geotechnical exploration program.
- Conducted a site visit to observe surface conditions and physical surface constraints to construction in the project site, as well as to identify and finalize the planned locations for the supplemental subsurface explorations.
- Performed the supplemental subsurface soil investigation for the project focusing on the site-specific conditions that may have an impact on the project design and construction. The investigation consisted of drilling four borings using mud-rotary approach with SPT and Shelby Tube sampling for depths up to 42 feet below ground surface and monitoring groundwater encountered within these borings.
- Performed laboratory testing of representative samples obtained from the exploration borings, which included: moisture content, dry density, grain-size, Atterberg limits, specific gravity, consolidation and direct shear (undrained) testing to establish undrained shear strength properties to supplement existing test data.

The data collected from the review of the available past geotechnical investigations at the site and the recent geotechnical investigation have been summarized in a Geotechnical Data Report (GDR) (CDM Smith 2017a). The geotechnical interpretations and recommendations developed have been presented in a Geotechnical Interpretive Report (GIR) (CDM Smith 2017b). Summaries of relevant information from these reports are briefly included in the corresponding sections of this report.

Figure 2-1. Vicinity Map

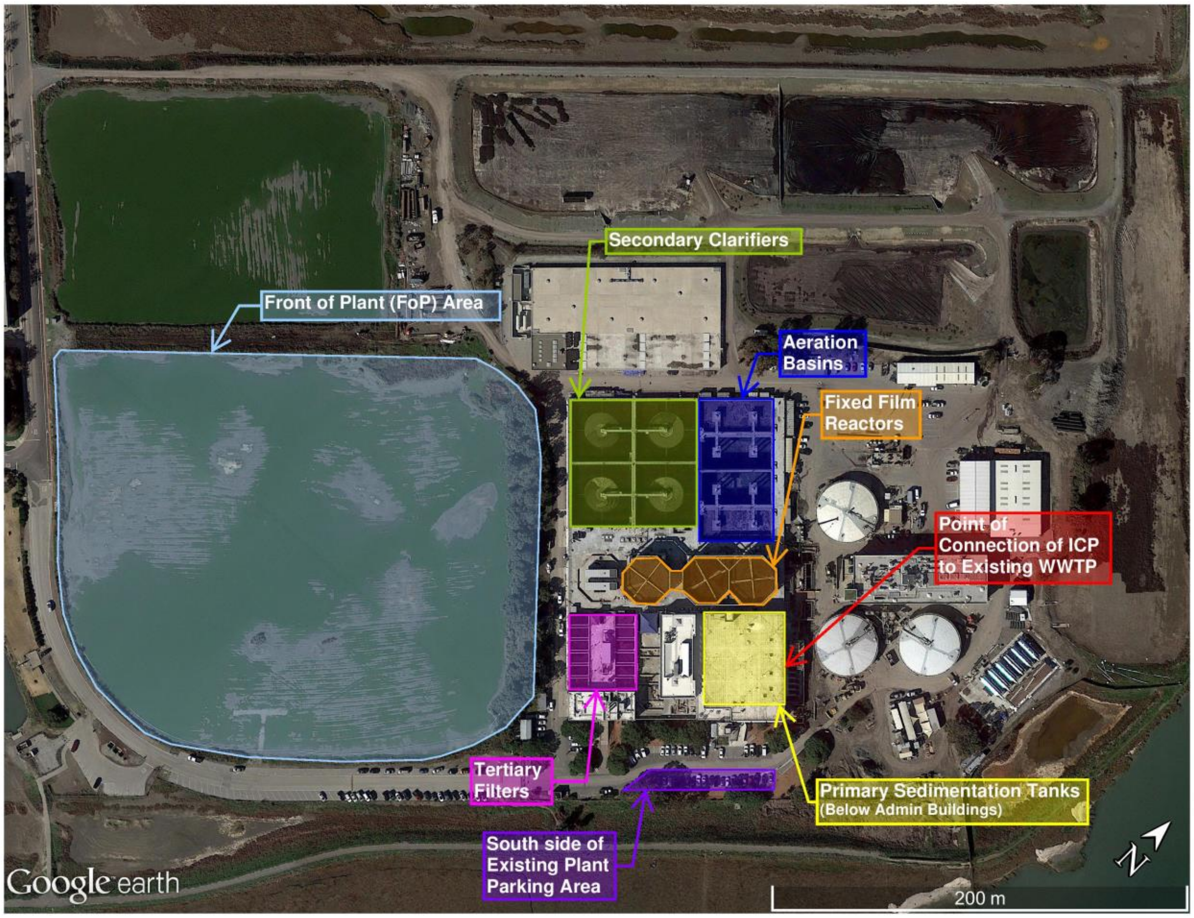

Figure 2-2. Physical Area

2.3 Site Features

This section details the features of the existing site on which the new dual influent connector pipeline will be routed. Some of the major site features are listed below:

- Gate/Fence line
- Underground Utility
- Paved Area/Planter Area
- Ornamental Pond Area
- Parking, driveways, access, etc.
- Staging and Storage Area
- Drainage area

2.3.1 Hydrologic, Geologic, and Topographic Features

No major surface water resources are impacted as part of the ICP project, though design documents will need to include direction to the progressive design-builder about the Storm Water Pollution Prevention Plan (SWPPP) as the ICP impacts the WWTP storm water infrastructure. The project will need to contain site runoff as part of the SWPPP.

Limited work has been done for hydrologic and topographic mapping as part of the CSMP's projects at the front of the existing WWTP. No hydrologic or topographic work has been performed directly related to the ICP project. Future design efforts will require topographic mapping and hydrologic review of the Project site. In general surface topography of the project site is flat with no distinct topographic features noted across the project site.

A geological review has been completed as part of the geotechnical investigation. In general, the geological review found the existing WWTP site was created by placing levees and fill over reclaimed marshland starting in about the 1950s (DCM|GeoEngineers 2009). The most recent fills were placed during the development of the site during late 1970s and early 1980s for the construction of SVCWTP facilities.

2.3.1.1 Site Geology

As part of the geotechnical investigation (CDM Smith 2017a,b), published USGS geology maps were reviewed to obtain geotechnical conditions along the pipeline alignment. Geologic mapping by U.S. Geologic Survey (USGS) (Brabb et al. 1998) indicates that the project site is underlain by bay mud locally referred to as Young Bay Mud (YBM). An earlier USGS map (Brebb and Pampeyan 1983) shows that portions of the project site with some areas of artificial fill, while majority of the site with YBM. The descriptions of these geologic units are as described below:

- **YBM**: Water-saturated estuarine mud, predominantly gray, green and blue clay and silty clay underlying marshlands and tidal mud flats of San Francisco Bay. The mud also contains few lenses of well-sorted, fine sand and silt, a few shelly layers (oysters), and peat.
- **Artificial Fill (af)**: Loose to very well consolidated gravel, sand, silt, clay, rock fragments, organic matter, and man-made debris in various combinations.

In this area, the af soil unit is typically underlain by YBM soil unit.

2.3.2 Subsurface Conditions

Subsurface conditions along the pipeline alignment at the project site were investigated by reviewing the results of the previous exploration programs that have been conducted by Cooper, Clark & Associates (1978a, 1978b, 1980 and 1981), Dames & Moore (1978), Fugro (2002), Fugro West Inc. (2004a, 2004b and 2004c), DCM|GeoEngineers (2009), and DCM Consulting (2014 and 2015) in the vicinity of the project site (Figure 2-2). In areas where sufficient subsurface information was not available, geotechnical borings were taken as part of the current investigation (CDM Smith 2017a and 2017b) to explore subsurface conditions and collect additional geotechnical data. The pipeline alignment and the selected exploration locations from the previous and current geotechnical investigations are shown on Figure 2-2. Exploration logs from the previous investigations in the vicinity of the pipeline alignment and relevant to this project and those logs from the current investigation are included in the Appendix A of the GDR. Similarly, the laboratory investigation results are pertinent to this project selected from previous investigations and those from the current investigations d are included in Appendix B of the GDR.

Based on the review of the geotechnical data explored, CDM Smith (2017b) noted that the site is underlain with fill and native YBM, which is consistent with the findings from previous explorations. Below is a brief description of the soils observed within the CDM Smith (2017a) soil borings; starting from the ground surface.

- ***Asphalt, Base Course Fill & Fabric***: Approximately 0.3 feet of asphalt pavement was observed in all of the borings underlain with base course material consisting of moist, silty Sand with gravel (SM)[2],well graded Sand with silt and gravel (SM-SW) and gravelly SAND (SP). The base course was observed to the approximate depth of 2 feet below ground surface. Based on laboratory data the gravel content ranged from 26 to 34 %, sand from 55 to 58 % and fines from 11 to 16 %. Filter fabric was observed underlying the base course at a depth of about 2 feet at CDM-04 only.

YBM: All of the borings encountered YBM to the depth of the borings. These soils consisted of Elastic Silt/or Fat Clay, wet, with scattered shells, occasional organics and trace amounts of sand. Within the upper 5 feet the consistency ranged from very stiff to very soft with trace amounts of gravel, below 5 feet the blows per foot were zero or very soft. Based on the laboratory testing the liquid limit (LL) ranged from 70 to 103%, the plastic limit (PL) from 34 to 40% and the plasticity index (LL-PL=PI) ranged from 36 to 66%. Moisture contents ranged from 76 to 103% and dry unit weight from 48 to 59 pounds per cubic foot (pcf). Groundwater was observed between 3 to 4 feet below ground surface in all of the borings.

2.3.3 Existing Utilities and Plant Access

The ICP Project will require either the relocation, replacement, and/or protection of other utilities. The following utilities are anticipated to be encountered over the length of the alignment that will need to be avoided or protected in place:

- 4-inch gas main
- Ferric Chloride Feedline
- 4-inch potable waterline along with the plant water booster pump facility and connecting utilities
- Plant electrical and communication
- Existing influent line
- 12 kV electrical feeder conduit

The following utilities are anticipated to need replacing and/or relocating:

[2] USCS Soil Classification Group Symbol

- 12-inch RCP storm drain and slit drain
- Automatic gate sensors
- 18-inch Redwood Shores sanitary sewer force main
- Lighting electrical conduit
- Landscape piping

Although the existing utilities above are believed to be within the project area, further investigation is required for design. Review of record drawings as well as field investigations, such as potholing, are required to confirm location and depth of utilities and reduce the risk of unanticipated construction costs due to unmarked or mismarked utilities.

2.4 Current and Projected Land Use of Project Site

The proposed alignment alternative will involve development of WWTP-related infrastructure within the existing WWTP site boundaries. The Project does not propose to introduce any new incompatible land uses to the site and does not propose to construct new infrastructure that would physically divide the community. Because the Project is within the existing WWTP site boundaries, there are currently no habitat or natural community conservation plans applicable to the Project area.

2.5 Relationship to Other Projects

Upon completion of all the CSMP Projects, wastewater will be conveyed by the new Gravity Pipeline to the RLS. The wastewater will then be pumped to the new Headworks Facility. From the new Headworks Facility, the screened and de-gritted wastewater will be conveyed by the ICP to the existing WWTP. A schematic showing the flow of wastewater from the Gravity Pipeline to the existing WWTP is shown in Figure 2-4. .

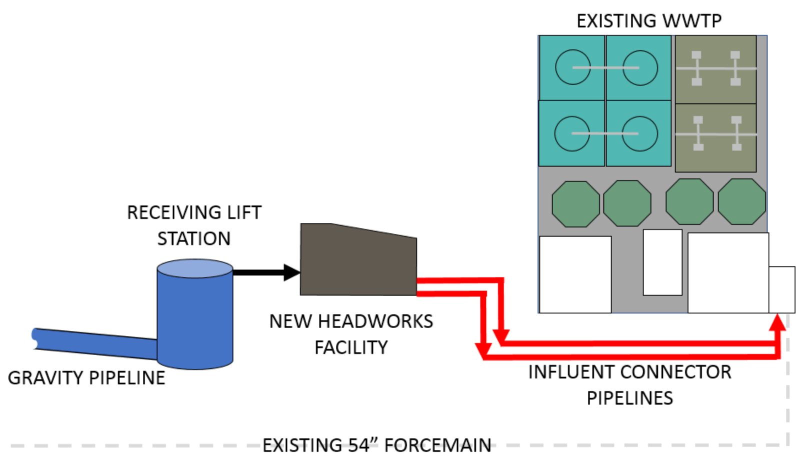

Figure 2-4 Flow Schematic of ICP Project and other Related CSMP Projects

Figure 2-5 presents a rendering of the completed FoP projects, as well as the future aeration basins, secondary clarifiers, and flow diversion (FD) structure

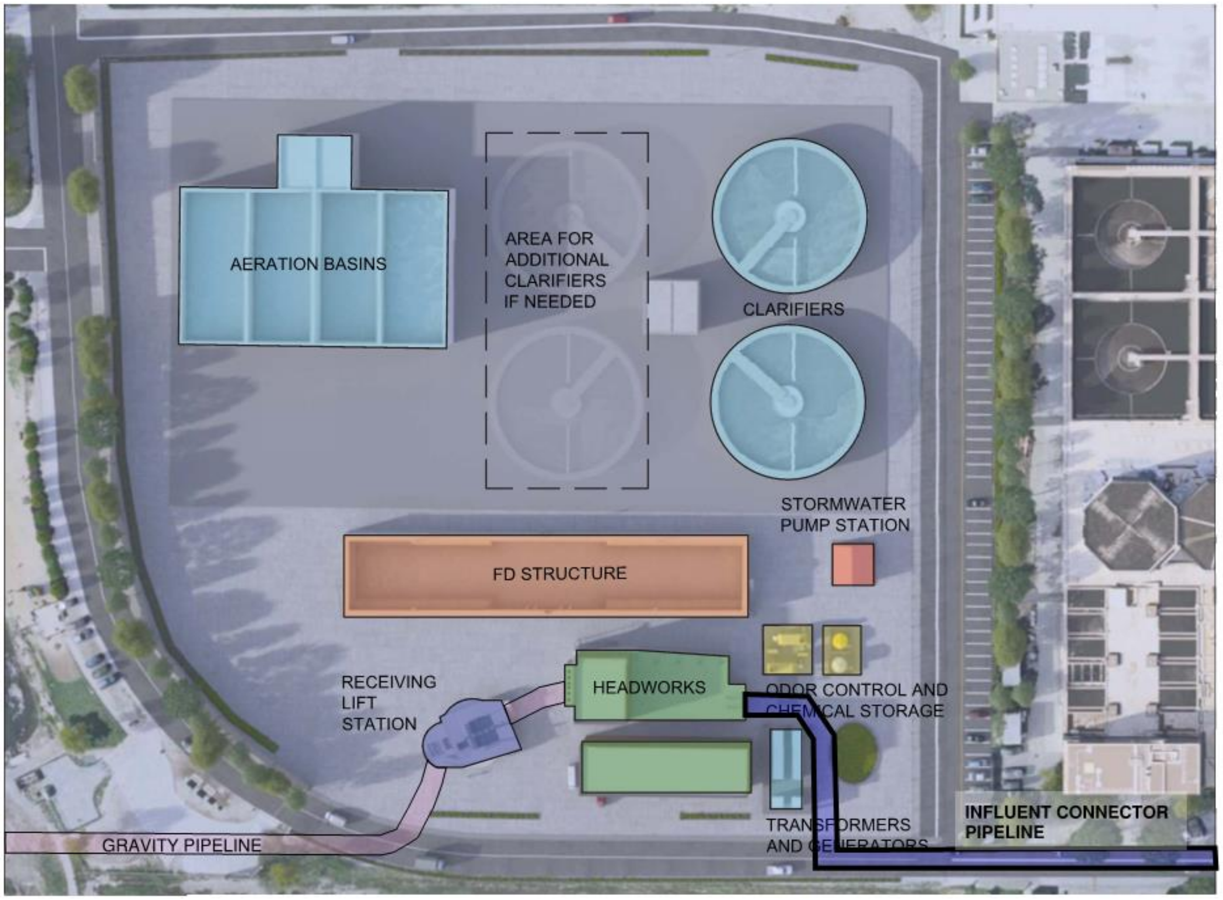

Figure 2-5 Rendering of Plant Area after Completion of Construction

Projects that will require coordination with the ICP are the Civil Improvements to the Front of Plant area and the new Headworks Facility.

2.5.1 Coordination with Civil Improvements

The Civil Improvements soil stabilization, parking, and road access elements must be completed prior to construction of the ICP to ensure there is an adequate plant parking area.

2.5.2 Coordination with Headworks Facility

SVCW is considering commissioning the Headworks Facility before construction of the Gravity Pipeline is complete. This implementation is referred to as Early Startup. If SVCW chooses to proceed with Early Startup, the west segment of the ICP (shown in green in Figure 2-6 below) would need to be completed and connected to the Headworks Facility prior to the Headworks Facility start-up. The west segment of the ICP would initially be connected to a tie-in point on the existing influent forcemain that will ultimately be abandoned once the east segment of the ICP alignment is completed and put into service. The interim configuration to support early startup of the Headworks Facility is shown in Figure 2-6. The flow schematic is shown in Figure 2-7, with the flow direction during early startup shown in blue.

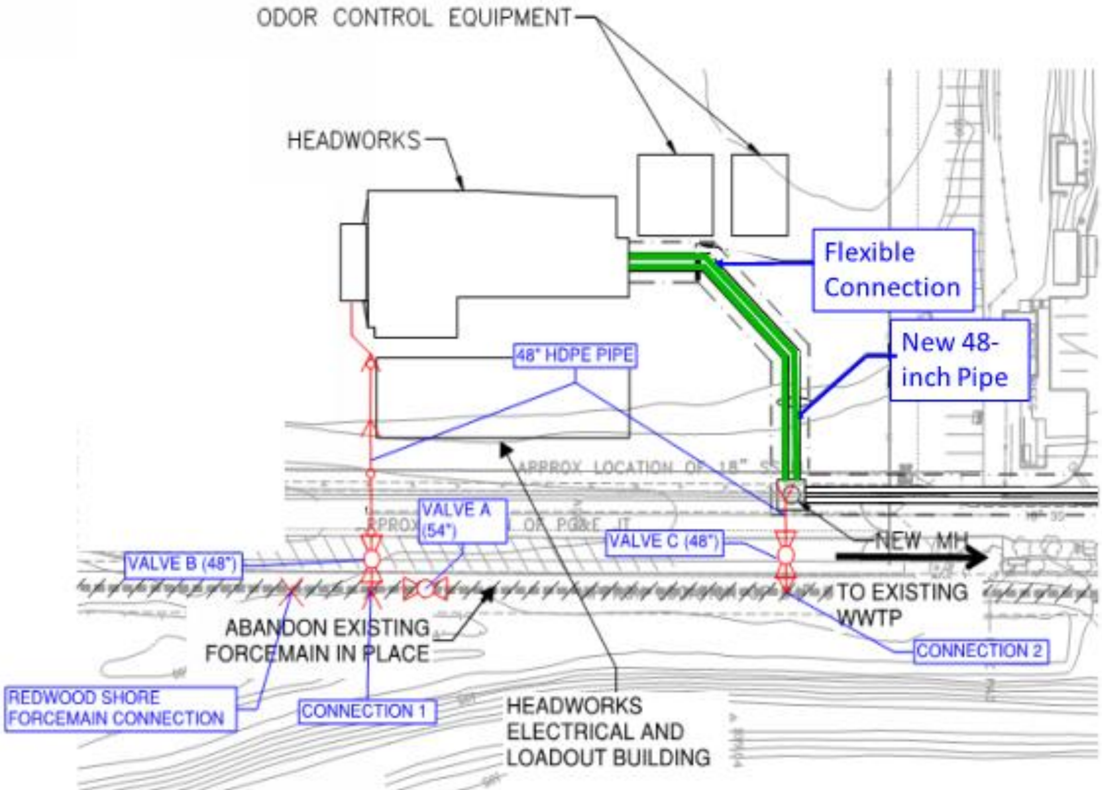

Figure 2-6 Interim Configuration during Early Startup of Headworks Facility

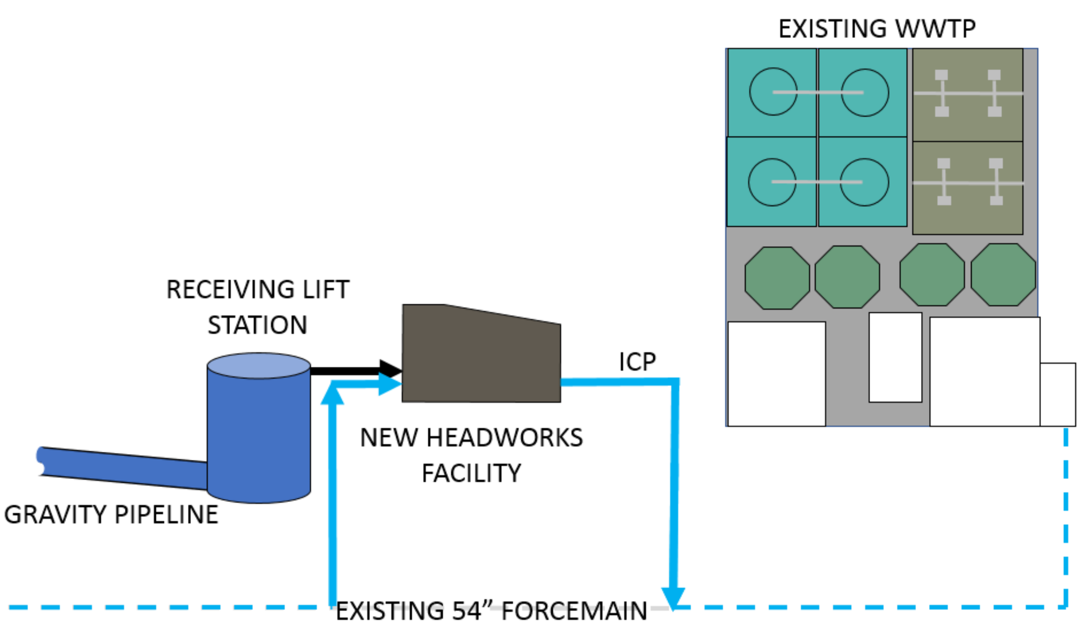

Figure 2-7 Flow Schematic with Early Startup of Headworks Facility

If SVCW chooses not to proceed with Early Startup, the entire ICP must be completed prior to Headworks Facility start-up. In this implementation, Valves A-C and Connections 1 and 2 in Figure 2-6 would not be constructed, and flow would follow the red arrows shown in Figure 2-8.

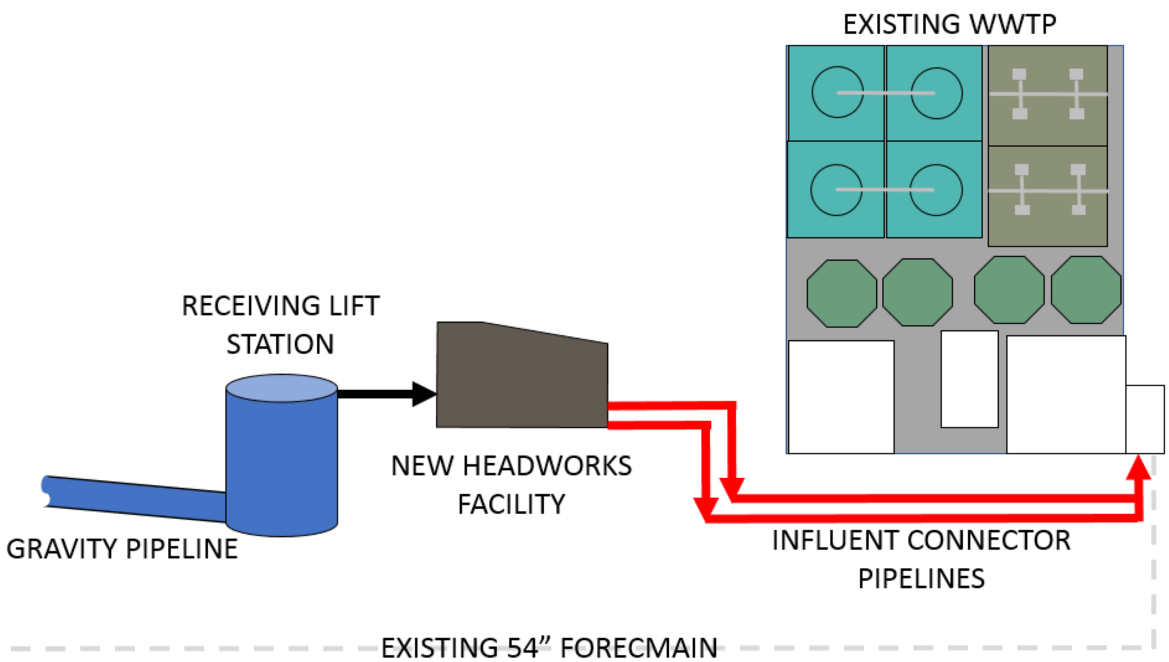

Figure 2-8 Flow Schematic without Early Startup of Headworks Facility

Because construction of the Headworks Facility has an expected duration of more than two years and construction of the ICP has an expected duration of nine months, it is not anticipated that the ICP would be on the critical path in any of these possible scenarios.

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Section 3

Planning and Design Parameters and Assumptions

3.1 Planning and Design Parameters and Assumptions

Pipe sizes were selected based on the following criteria:

- Dual pipeline consisting of one small pipe and one large pipe,
- Capacity to convey Peak Wet Weather Flow (PWWF) in both pipes,
- Maximum head loss of approximately two (2) feet,
- Minimum velocity of 1.5 feet per second during Average Dry Weather Flow (ADWF).

A dual pipeline is required, since minimum velocity criteria could not be met with a single pipeline given the limits to headloss. The first pipe was sized to meet the minimum velocity criterion, assuming the second pipe is offline when flows are at ADWF. The second pipe was sized to meet the maximum head loss criterion, assuming both pipes are operating in parallel when flows are at PWWF.

Based on the above criteria, comments by SVCW, and the recommended alignment alternative (see Sections 5 and 6), CDM Smith identified a 48-inch HDPE pipe (internal diameter of 44 inches) and a 72-inch HDPE pipe (internal diameter of 72 inches). The combined capacity of both pipes is 80 mgd. The 48-inch pipe has a capacity of 22.5 mgd, and provides acceptable velocities over the range of low flows and the average dry weather flows. There is also sufficient capacity to allow for the Gravity Pipeline to be flushed without needing to engage the wet weather pipe.

Final selection of the dry weather pipe diameter should also consider the final system operating conditions, such as:

- Peak flow attenuation and equalization in the Gravity Pipeline,
- Operating strategy for routinely generating scouring velocities in the Gravity Pipeline,
- Other RLS operating conditions, and
- The frequency of events that exceed the dry weather pipe's capacity requiring use of the wet weather pipe.

The dry weather pipe size should accommodate the range of dry weather flows while meeting headloss and velocity requirements.

Given that stagnant wastewater in the larger pipeline must be pumped out or treated with biocide each time it is taken offline, it is desirable to minimize the frequency this occurs. Reducing the use of the wet weather pipe can be achieved by increasing the flow capacity of the dry weather pipe. This could be accomplished by increasing the pipe's diameter while maintaining the same elevation of the New Headworks Facility. Alternately, the effective capacity of the dry weather pipe would be greater if final design of the conveyance system accommodates a higher dry weather water surface elevation in the New Headworks Facility, creating a higher driving head to push more flow through the dry weather pipe. Flow conditions have been provided to CDM Smith as shown in Table 3-1. Operational flow conditions have not been formalized and should be completed during detailed design.

At this time, it is assumed that an automated gate at the upstream end of the 72-inch pipe would open when flows exceed PDWF, switching the system from single-pipe to dual-pipe operation. Pipe sizes, design flows, and the method of splitting flow between the two pipes must be confirmed during design.

Table 3-1. Total Design Flows and Capacity for FoP Projects(1)

	Current			Projected			
FoP Project	MDWF OCT-2015 (hourly)(2)	PDWF OCT-2015 (hourly)(2)	ADWF OCT-2015 (daily)(2)	ADWF 2040 (daily)(3)	PWWF 10yr, 1 Stm, 2040 (hourly)(4)	PDWF 2040 (daily)(3)	PDWF 2040 (hourly)(5)
Tunnel and Gravity Pipeline	2.4	20.5	10.9	17.3	102.9(5)	22	33.9 (3)
Receiving Lift Station	2.4	20.5	10.9	17.3	75	22	33.9 (3)
Headworks (1)	2.7	22.5	11.8	17.9	80(4)	23	33.9 (3)
Interconnector Pipe (1)	**2.7**	**22.5**	**11.8**	**17.9**	**80**	**23**	**33.9 (3)**

1. Design flows SVCW to CDM Smith, 1/25/2017.
2. Minimum Dry Weather Flow (MDWF hourly), Average Dry Weather Flow (ADWF daily), and Peak Dry Weather Flow (PDWF hourly) for October 2015 based on flow data provided by SVCW SCADA output from each pump station.
3. ADWF 2040 (daily) and PDWF 2040 (daily) flow rates from Table 5-9 of TM 1 for Final Plant Capacity Study (Brown and Caldwell, 2013)
4. WWTP Capacity Flow Rates provided by SVCW
5. PDWF 2040 (hourly) flow rates from Member Agency Master Plans and CSMP

Section 4

Influent Connector Pipeline Alternatives Analysis

This section of the report summarizes the methodology, selection criteria, and alternatives analysis previously presented in the SVCW Alignment Alternative Analysis Report, included as an attachment to this report.

4.1 Alternatives Evaluation Methodology and Selection Criteria

Eight (8) alternatives were developed by expanding five (5) possible options to connect the new headworks to the existing WWTP while considering direction provided by SVCW.

SVCW provided instruction that the ICP use:

- Pipe material such as High Density Polyethylene (HDPE) to use a joint-less piping system,
- Construction methods compatible with highly compressible soils and high ground water, and
- Flexible couplings at connections to structures.

The review of the five possible options to connect the Headworks to the existing WWTP included:

- Rehabilitation of the existing pipeline
- Replacement of the existing influent pipeline with upgraded materials, such as HDPE
- Installation of a new influent line in an alignment different from the existing alignment
- Microtunneling of a new influent line
- A hybrid of the above options

Expanding the five possible approaches resulted in the eight (8) alternatives presented in Table 4-1 below.

Table 4-1 ICP Alignment Alternatives

Option Name	Description
Alternative A : Rehabilitation of Existing Pipeline	Alternative A includes installing 225 feet of new 63” HDPE pipe, using open cut construction, to connect the future headworks facility to the existing influent line and rehabilitating 575 feet of 54” RCP (Reinforced Concrete Pipe) and 175 feet of 60” RCP.
Alternative B: Replace Existing Influent Line	Alternative B requires the removal of a portion of the existing 54-inch and all of the 60-inch influent line in order to install, in its place, the 975 feet of 84-inch HDPE pipe from the new headworks facility to the treatment plant.
Alternative C: New Pipe Alignment	Alternative C includes installing 900 feet of new 84-inch HDPE pipe in a new alignment routed within the street right of way and plant property boundary.

Option Name	Description
Alternative D: Microtunnel in New Alignment	Alternative D includes installing 940 feet of new 84-inch HDPE pipe, with 600 of the 1100 feet microtunneled inside a new 90-inch steel casing. The alignment will follow nearly the same alignment as Alternative C.
Alternative E: CIPP + New Alignment	Alternative E combines Alternatives A and C to install 900 feet of new 66-inch HDPE pipe in the same alignment as Alternative C and connects to and rehabilitate the existing influent line as outlined in Alternative A.
Alternative F1: Parallel Pipes	Alternative F1 combines Alternatives B and C to install a total 1850 feet of new HDPE pipe in a parallel configuration. Nearly 900 feet of HDPE pipe will be routed along the alignment as outlined in Alternative B while 975 of HDPE pipe will be routed in the alignment as outlined in Alternative C.
Alternative F2: Parallel Pipes	Alternative F2 follows the Alternative B alignment but involves installing a total of 1900 feet of new HPDE pipe in a parallel configuration thereby introducing redundancy to Alternative B.
Alternative F3: Parallel Pipes	Alternative F3 follows the Alternative C alignment but involves installing a total of 1800 feet of new HDPE pipe in a parallel configuration thereby introducing redundancy to Alternative C.

Common to all eight (8) alternatives was the use of HDPE pipe material so that the best ranking alternative selected is based on factors other than selection of pipe material. HDPE was selected as an initial candidate because it is joint-less and flexible while offering excellent corrosion resistance and superior performance in highly compressive soils.

Each of the eight (8) influent connector pipeline alternatives were then quantified to aid in selecting preferred alternatives. A weighted selection matrix reflecting SVCW preferences was used to analyze the eight alternatives. Each alignment was assigned a score in eleven categories. Scores in each category were weighted based on their importance to SVCW and stakeholders. The following categories were utilized in the alignment ranking/selection process:

1. Constructability: Given the method of construction, each alternative was assigned a score from 0 to 5, with a 0 signaling that the alternative is easy to construct, a 1 to 4 signaling that the alternative is difficult yet possible to construct and a 5 signaling that the alternative is not constructible.

2. Head loss: A score of 0, 1, or 2 is assigned to each alternative, with 0 being assigned if the alternative is capable of handling 80 MGD of flow with a single pipeline, 1 being assigned if the alternative is capable of handling 80 MGD of flow with a dual pipeline arrangement, and 2 being assigned if the alternative does not provide 80 MGD capacity.

3. Number of Utilities to Relocate: The number of utilities requiring replacement or relocation due to each alternative was tallied up, with each utility resulting in a 1 point increase in score.

4. Influent Bypass Requirements: A score of 0 or 1 is assigned to each alternative, with 0 being assigned if the alternative does not require installation of an influent bypass pipeline and 1 being assigned if the alternative does.

5. Utilities to Protect in Place: The number of utilities that need to be protected in place due to each alternative was tallied up, with each utility resulting in a 1 point increase in score.

6. Plant Access Interference Level: A score of 1 to 5 is assigned to each alternative, with 1 being assigned if the alternative only interferes with plant access for a short period of time, a 2 to 4 if the alternative periodically or partially blocks access during construction, and 5 if the alternative unacceptably blocks access during construction.

7. Plant Parking Interference Level: A score of 0 or 1 is assigned to each alternative, with 0 being assigned if the alternative does not interfere with plant parking and 1 being assigned if the alternative does.

8. BCDC Permit Requirement: A score of 0 to 5 is assigned to each alternative, with 0 being assigned if there is no impact to schedule, 2 to 4 if there is a limited impact to schedule that may be mitigated, and 5 if the BCDC permit requirement impacts the construction schedule.

9. Process Impact: A score of 1 to 5 is assigned to each alternative to expresses the amount an alignment may affect the process of the plant, e.g. flow equalization, peaking flows, septic solids build up, odors, etc. 1=Minor; 2 to 4=Some Impact; 5=Unacceptable.

10. Operational Complexity: A score of 0 to 5 is assigned to each alternative to express the level of effort to operate each alternative (e.g. operation of large valves, seasonal operational changes, managing flows from dry to wet weather). A score of 0 represents that the operations of the alternative present no impact to level of effort; 2 to 4, some impacts; and 5, high impacts.

11. Cost: A score of 1, 3, or 5 is applied to all alternatives based on planning level cost estimates. A score of 1 is assigned to the alternative with the lowest cost, 3 is assigned to all alternatives with a mid-range cost, and 5 is assigned to the alternative with the highest cost.

A factor of importance of 5, 10, or 15 is assigned for each of the 11 categories with less important categories receiving a score of 5, more important categories receiving a score of 10, and the most important categories receiving a score of 15. Then, the individual scores in each of the 11 categories are multiplied by their respective factors of importance. The weighted score is summed across the categories to produce a total ranking score, as shown in Table 5-2. For this ranking system, the alternatives with higher scores are considered less favorable. In addition to the ranking score, each alternative was reviewed for any fatal flows to identify any alignments that are not feasible. Some of these fatal flaws are identified in the ranking as “Not Acceptable”.

4.2 Alternatives Analysis

Scoring of all alternatives considered is summarized in Table 4-2 below.

Of the eight alignment alternatives presented in Table 5-2, four (4) were considered viable: Alternatives E, F1, F2, and F3. The other four alignments were eliminated due to fatal flaws in constructability, operational, and/or functionality concerns. The ranking score for the preferred alignments are similar, ranging from 175 to 220. Opinion of Probable Construction Costs (OPCC) were also similar, ranging from $3,740,0,00 to $4,700,000. The OPCC is based on a Level V Planning Level Cost Estimate (-20%/+50%), and does not include engineering fees or construction contingencies.

Table 4-2 Alternative Alignments Scoring

Importance factor	5=Less Important to Avoid 10=Somewhat Important 15=More Important	15	15	5	5	5	10	5	1	15	15	15				
Alternative	**Description**	**Construct-ability**	**Head-loss**	**Utilities Relocate/ Replace**	**Influent Bypass**	**Utilities Protect in place**	**Plant Access**	**Plant Parking**	**BCDC permit**	**Process Impacts**	**Operational Complexity**	**Cost**	**Total Score**	**$ Amount (+50%/-30%)**	Fatal Flaw	**Fatal Flaw Description**
A	CIPP	0	2	0	1	4	1	0	5	1	0	1	130	$1,450,000	Y	Cannot meet head loss requirements.
B	Replace in Place	1	0	0	1	6	2	0	5	5	3	3	165	$3,040,000	Y	Cannot meet process requirements. Organic solids will settle under dry weather flows
C	Single Pipe in New Alignment	2	0	5	0	6	3	1	0	5	3	3	165	$2,820,000	Y	Cannot meet process requirements. Organic solids will settle under dry weather flows
D	Microtunnel in New Alignment	5	0	5	0	6	5	0	0	5	3	5	255	$11,670,000	Y	Is not constructible. Cannot meet process requirements. Organic solids will settle under dry weather flows.
E	CIPP + New Alignment	1	1	5	0	6	4	1	2	3	4	3	195	$3,740,000	N	
F1	Parallel Pipes New & Existing	2	1	5	0	6	4	1	3	3	4	3	220	$4,210,000	N	
F2	Parallel Pipes Existing Alignment	1	1	0	1	6	2	0	5	3	4	3	180	$4,700,000	N	
F3	Parallel Pipes All New Alignment	1	1	5	0	6	4	1	0	3	4	3	175	$4,300,000	N	

Constructability-Given method of construction, approximates a risk level, length of construction, and if progressive design-builder will encounter delays. 0=Ease of construction;1 to 4=Difficult construction; 5=Not Constructible

Head loss-Indicates the pipe size will allow for proper flow and with minimal head loss. 0=Provides 80 mgd alone; 1=Provides 80 mgd with second pipe; 2=Does not provide 80 mgd capacity

Utilities Relocate or Replace-Indicates the number of utilities (major and minor) requiring replacement or relocation.

Influent Bypass-Alignment will require bypass of main influent line to treatment plant. 0=No 1=Yes

Utilities Protect in Place-Indicates the number of utilities being crossed or disturbed but will not require replacement or relocating.

Plant Access-Indicates that plant access will be interrupted either at the entrance of the plant or near the ILS. 1=Minor (Access inconvenienced for short period); 2 to 4=Limited (Access blocked periodically or partially during construction); 5=Unacceptable (Access blocked throughout construction)

Plant Parking-Indicates that plant parking will be occupied during construction within the plant property. 1=Yes 0=No (Assumes new parking installed prior)

BCDC Permit-Indicates the alignment will require a BCDC permit and the added impact to overall construction for obtaining the permit. 0=No Impact (No Impact to Schedule); 2 to 4=Limited Impact (Can mitigate some impact to schedule); 5=High Impact (Impacts Construction Schedule)

Process Impact-Expresses considerations that may affect the process of the plant, e.g. flow equalization, septic solids build up, odors, etc. 1=Minor; 2 to 4=Some Impact; 5=Unacceptable.

Operational Complexity-Expresses an increased effort to operation, e.g. operation of large valves, seasonal operational changes, managing of flows from dry to wet weather. 0=No Impact; 2 to 4=Some Impact; 5=High Impact to Operations

Cost-Based on a Level V Planning Level Cost Estimate (-20%/+50%), does not include engineering fees or construction contingencies. Emphasizes highest and lowest cost. Mid-range costs are all within contingency factor. 1=Lowest Cost; 3=Mid-Range Costs; 5=Highest Cost

Total Score-Higher Scores indicate less favorable selections

Fatal Flaw-Review from a technical perspective if the project is not feasible, has major issues, or fails to meet the needs of the overall objective. Y=Yes; N=No

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F

Section 5

Selected Influent Connector Pipeline Alignment Alternative

5.1 Recommended Alignment Alternative

Based on the alternatives analysis presented in Section 4.2 and feedback received from SVCW staff during the January 27, 2016 Alignment Alternatives Presentation, Alignment F3 (shown in Figure 5-1) is recommended for implementation because it:

- Provides needed capacity of 80 MGD with minimal headloss,
- Avoids BCDC jurisdiction and permitting requirements,
- Increases reliability and redundancy due to the dual pipeline arrangement,
- Allows the existing 54-inch RCP forcemain to remain in service during construction so no bypassing is required, and
- Appears comparable to other feasible alternatives in terms of cost, constructability, head loss, operational complexity, conflicts with existing utilities, and impacts to plant access and parking.

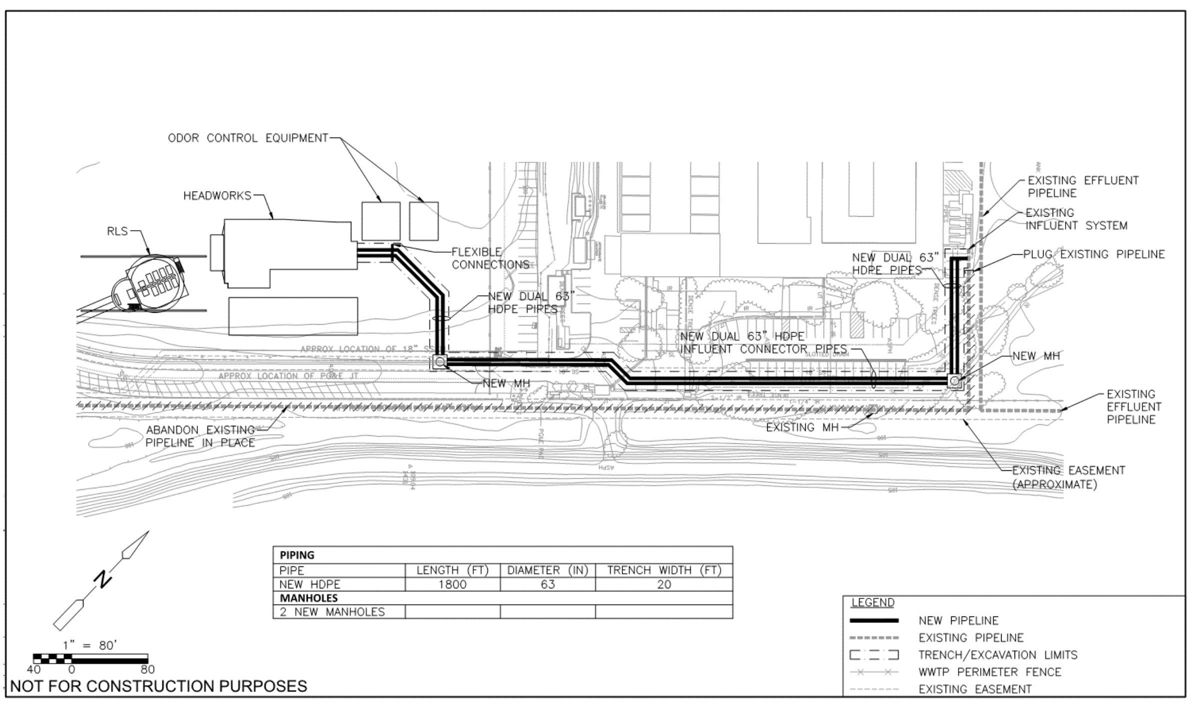

Figure 5-1. Alignment Alternative F3 Routing

5.1.1 Pipe Sizing

After completing the Alternatives Analysis described in Sections 4 and 5 of this report, CDM Smith reviewed additional pipe sizing options and recommended using two pipes of different diameters. In this configuration, only the smaller of the two pipes would be used during periods of dry weather, while both pipes would be used during periods of wet weather. The updated recommendation to use a 48-inch and a 72-inch diameter pipe was detailed in a Pipe Size TM provided to SVCW.

As discussed in Section 3, Final pipe sizes should be determined by the progressive design-builder based on the final piping configuration, allowable head loss, acceptable minimum velocity, equalization provided by the Gravity Pipeline, and planned operation of the RLS. Revisions to design flows or other criteria may impact the operating water surface elevation in the new Headworks Facility and/or the size of the ICP pipes.

5.1.2 Alternative F3

Level V Opinion of Probable Cost of Construction (does not include contingency): $4,300,000

Total Length: 1800 feet of HDPE Pipe in parallel configuration

Alternative F3 includes parallel 48-inch and 72-inch diameter pipes running in a joint trench from the proposed Headworks Facility, through the southern portion of the treatment plant property, to the existing WWTP at the existing Screening Facility. In combination, these pipes have 80 mgd (PWWF) capacity. It is assumed that the 48-inch diameter pipe would convey flows up to 22.5 mgd (PDWF), and both pipes would operate in parallel when flows exceed 22.5 mgd. The pipe sizing rationale is described further in Appendix B. These diameters should be confirmed during design.

This dual pipe configuration minimizes head loss while maintaining velocities at 1.5 feet per second during the ADWF of 11.8 mgd. Since this velocity will be experienced during the diurnal peak even when the daily flow is lower, the occurrence of solids settling within the pipe(s) will be reduced compared to what would occur in a single large pipe. Another advantage of a dual pipe configuration is that one pipe can be taken out for maintenance while the second pipe remains in service.

The recommended alignment eliminates the need for a BCDC permit by having the pipe remain on SVCW property, with some impact to access and parking along the alignment. Because the proposed alternative follows a different alignment than the existing influent pipeline, the existing influent pipeline can remain in service during construction, removing the need for costly bypassing.

Several utility conflicts are present along Alternative F3. Existing utilities will require field location and as-built drawings review to reduce the risk of impacts to the plant and construction personnel safety.

5.2 Major Project Components

Based on a planning-level analysis, the selected alternative for the project, shown in Figure 5-1 of this report, includes construction of the following:

- Two parallel pipes in a joint trench,
- A joint-less piping system, such as HDPE pipe,
- Smaller diameter pipe for dry weather conditions, and both the smaller and larger diameter pipes for wet weather conditions,
- Sheet piling for excavation stability and water infiltration control,
- A tremie slab concrete trench bottom for developing a firm base and water infiltration control, Use of light weight aggregate for pipe trench backfill,
- Potential manway access at two locations, and
- Pile supports at structures.

Pending final design of the ICP, these features will need to be confirmed and validated by the engineer of record. Other major components may include but are not limited to: valves, actuators and instrumentation, existing lift station demo, utility relocations, and corrosion control.

5.3 Design Criteria

Design criteria for the ICP are presented in Table 5-1.

Table 5-1. ICP Design Criteria

Criteria	Value	Units	Notes
ADWF (Oct 2015) – Hourly	11.8	MGD	
PDWF (Oct 2015) – Hourly	22.5	MGD	
PWWF (10 yr, Projected 2040) – Hourly	80	MGD	
Number of Pipes	2	--	One smaller dry weather pipe and a larger wet weather pipe such that when both wet and dry weather pipes are in operation, the ICP system can deliver the projected 2040 PWWF of 80 MGD.
Pipe Material	HDPE or similar	--	HDPE pipe was recommended for a joint-less, corrosion-resistant piping system.
Nominal Diameter of Dry Weather Pipe	48	inch	The 48-inch pipe will be able to convey flows up to the 2015 PDWF of 22.5 MGD.
Nominal Diameter of Wet Weather Pipe	72	Inch	
Maximum Head Loss at Design Flows	2	feet	See Section 3.1 for a full table of flow values.
Minimum Design Velocity	1.5	ft/sec	Minimum design velocity during 2015 ADWF of 11.8 MGD in the single 48-inch dry weather pipe.

In addition to the design criteria in Table 5-1, the following criteria need to be considered:

- Corrosion protection of appurtenances and structures.
- Piles constructed under new structures, such as manholes.
- Flexible couplings at any location of potential differential settlement, including connections to all structures.
- Minimize impacts to plant activities such as chemical deliveries and personnel access during construction.
- Provide a nominal slope for pipe drainage.

5.4 Useful Life of the Project

According to the Plastic Pipe Institute, the material used for the influent connector pipeline, HDPE, has a life cycle of 75 to 100 years with a 100 years being more acceptable. Because of the soil conditions that exist at the treatment plant and to be conservative, a 75-year life cycle for HDPE pipe was selected. Although the useful life of the project is 75 to 100 years, the planning horizon provided by SVCW for this project is the year 2040.

5.5 Site Layout

The ICP alignment, staging and storage area, and construction trench are shown in Figure 5-2 on the next page. For construction of the ICP Project, a 20-foot wide trench is assumed for installation of the dual pipeline. The trench will be positioned in approximately the middle of the road that leads to the main entrance. Once past the plant water booster pump station, it will jog south, keeping the trench wall approximately 5 to 7 feet off the fence/property line to avoid the gas main, chemical piping, and other utilities along that area. It will continue parallel to the property line until it turns north toward the plant. This route will keep approximately 20 feet clear between the end of maintenance ramp and the trench. This should provide sufficient space for vehicular access to the ramp.

5.6 Key Decisions

Pipeline alignment and connection to the new Headworks Facility and existing influent system will be determined during design. Chemical and utility lines currently hanging on SVCW's fence may need to be relocated during construction. Additional decisions to be made during design are

- Whether the new Headworks Facility will start up prior to the ICP (See Section 2.5.2)
    - Means of connecting to the existing WWTP on the interim
- Selection of material to meet design requirements of ICP (See Section 11.1.4)
- Means of accessing the ICP for inspection and maintenance and the need for and recommended location for manways, if any (See Section 11.1.9)

- Control of stagnant water in the larger pipe between wet weather events (See Section 11.1.3)
- Valving configuration and details of connection to the existing plant (See Section 11.1.1)

A complete list of decisions and issues is provided in Section 11 of this report.

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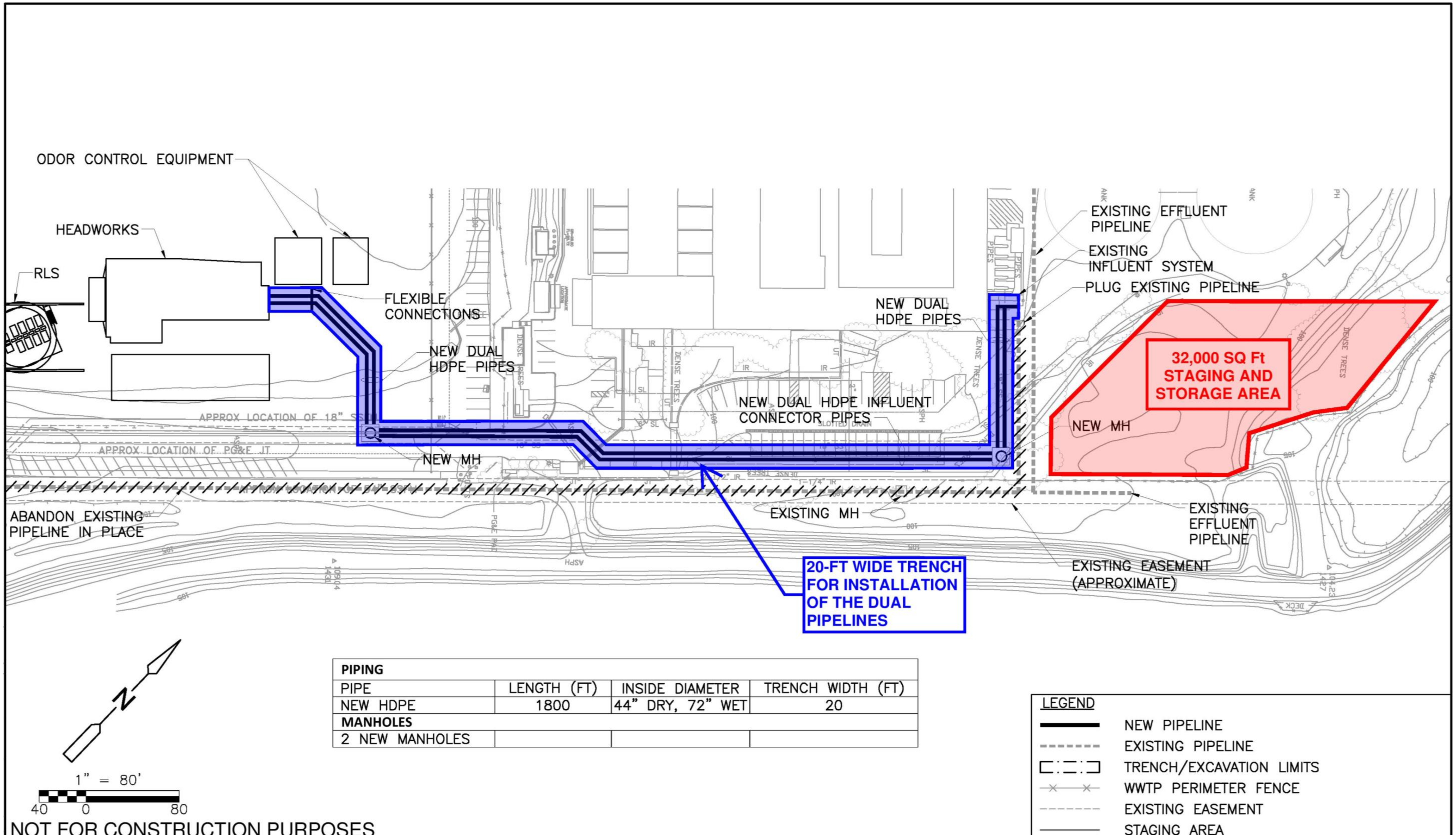

PIPING			
PIPE	LENGTH (FT)	INSIDE DIAMETER	TRENCH WIDTH (FT)
NEW HDPE	1800	44" DRY, 72" WET	20
MANHOLES			
2 NEW MANHOLES			

Figure 5-2 ICP Alignment, Staging and Storage Area, and Construction Trench

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Section 6

Hydraulic Analysis

6.1 Hydraulic Profile

Figure 6-1 shows the hydraulic profile for the Headworks and Influent Connector Pipeline projects. The hydraulic profile shows total head loss in the ICP to be nearly 2 feet from the point of connection at the headworks to the existing Influent Mix Box located just upstream of the WWTP existing screening facility. Supporting calculations are described in Section 6.2.

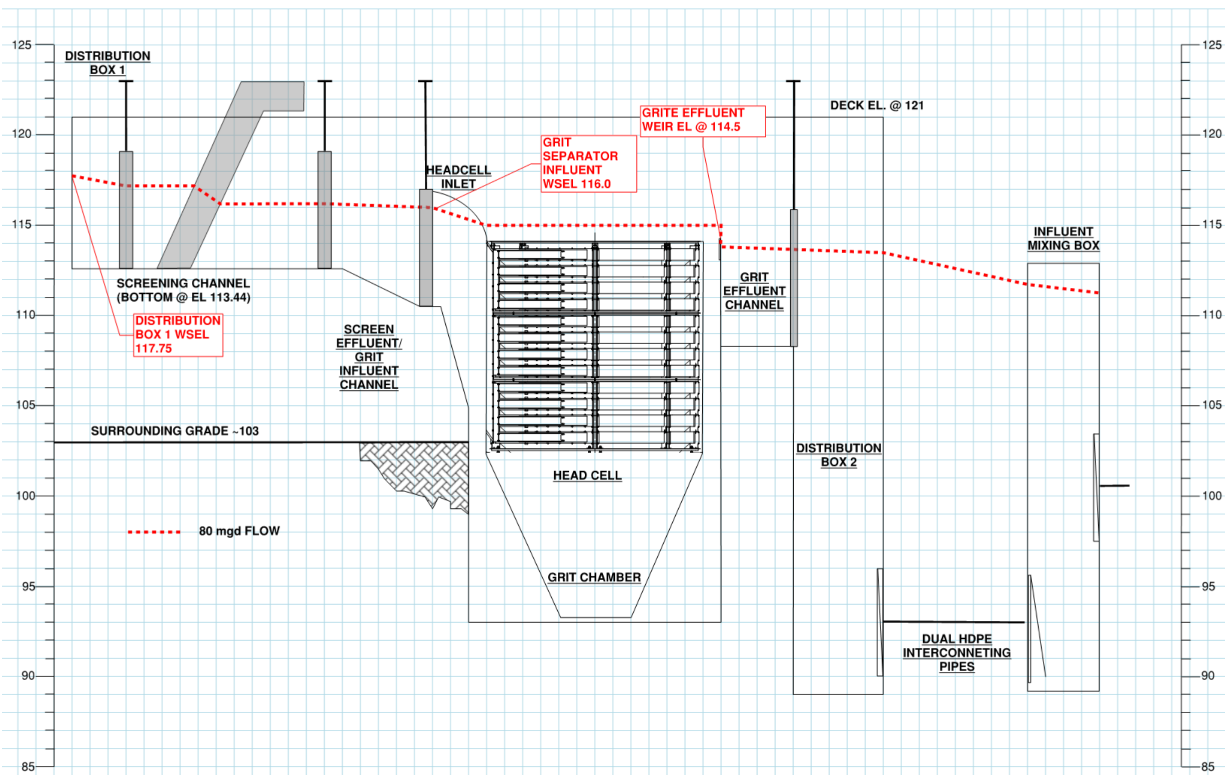

Figure 6-1. Hydraulic Profile for Headworks Project

6.2 Hydraulic Analysis

Losses from pipe fittings were approximated using K values assigned for each pipe. The assumed K values are detailed in Table 6-1. Minor losses and line losses were summed to determine the total head loss along the pipeline. A summary of the head loss calculations during PDWF using only the smaller of the pipes is included in Table 6-2 below, and a summary of the head loss calculations during PWWF using both pipes is included in Table 6-3 below.

Table 6-1. K Values Used to Calculate Head Loss from Pipe Fittings

		48-inch Pipe (44-inch ID)		72-inch Pipe	
Pipe Fitting	K Value	Quantity	Sum of K	Quantity	Sum of K
Entrance	0.5	3	1.5	3	1.5
Exit	1	3	3	3	3
Primary Settling Channel Sluice Gate	0.2	1	0.2	1	0.2
Swing Check Valve	2.5	0	0	1	2.5
Butterfly Valve	1.3	2	2.6	2	2.6
90-deg elbow	0.3	2	0.6	2	0.6
45-deg elbow	0.2	4	0.8	4	0.8
Through Tee	0.5	1	0.5	1	0.5
		TOTAL	**9.2**	**TOTAL**	**11.7**

Table 6-2. Head Loss Calculations during PDWF

Pipe ID	Flow (mgd)	Dia (ft)	v (ft/s)	A (ft^2)	V head	f Friction Factor	Sum K	Pipe L (ft)	Minor hL (ft)	Friction Losses (ft)	Total Losses (ft)	Total Losses (psi)
44	22.5	3.7	3.29	10.60	0.17	0.0115	9.2	950	1.5	0.50	2.05	0.89

Table 6-3. Head Loss Calculations during PWWF

Pipe ID	Flow (mgd)	Dia (ft)	v (ft/s)	A (ft^2)	V head	f Friction Factor	Sum K	Pipe L (ft)	Minor hL (ft)	Friction Losses (ft)	Total Losses (ft)	Total Losses (psi)
44	22.5	3.7	3.29	10.60	0.17	0.0115	9.2	950	1.5	0.50	2.05	0.89
72	57.46	6.0	3.14	28.27	0.15	0.0107	11.7	900	1.8	0.25	2.04	0.88

In conducting the hydraulic analysis for wet weather flow, the following assumptions were made:

- 2040 peak wet weather flow (PWWF) is 80 MGD
- The entrance to both pipes is from a common hydraulic source (represented as a “reservoir” in the hydraulic model)
- Water surface elevation (WSE) in the Influent Mix Box (the downstream end of ICP) at 80 MGD is 111.69, based on the Stage 1 Influent Screening Drawings (Brown & Caldwell, 2014)

Based on these assumptions, and the head loss calculations detailed in Section 6.2, the WSE in Distribution Box 2 (the upstream end of ICP) will be 113.74 at PWWF. Since the overflow weir to the grit effluent channel of the Headworks Facility is set above 114 (see Figure 6-1), the system can flow by gravity and does not require pumps.

In conducting the head loss analysis for dry weather flow, the following assumptions were made:

- 2015 peak dry weather flow (PDWF) is 22.5 MGD
- Only the 48-inch diameter pipe is engaged during flows under 22.5 MGD
- WSE in the Influent Mix Box at 30 MGD is 107.12 based on the Stage 1 Influent Screening Drawings (Brown & Caldwell, 2014), so the WSE at 22.5 MGD is no higher than 107.12

Based on these assumptions, and the head loss calculations detailed in Section 6.2, the WSE in Distribution Box 2 (the upstream end of ICP) will be 109.17 at PDWF. Since the elevation of the weir to Distribution Box 2 is 113.60, there will be a free discharge across this weir at 22.5 MGD, and no pumping is required.

In order to meet maximum head loss and minimum velocity criteria under both wet weather and dry weather flows, a combination of pipe sizes was chosen:

- 48-inch diameter pipe to convey dry weather flow (up to 22.5 MGD)
- 72-inch diameter pipe to engage in parallel with the 48-inch diameter pipe to convey wet weather flow (22.5 MGD to 80 MGD)

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Section 7

Additional Design Considerations

This section of the report details design components that will require further consideration as the project progresses.

7.1 Constructability

The ICP will be constructed in highly compressible soils with low shear strength and high groundwater. An approach similar to the construction approach used for the outfall replacement project, including sheet pile shoring and using a tremie seal base, should be reviewed in application to the ICP Project.

Additional constructability considerations include:

- Trench width for parallel pipes,
- Construction during dry season and impacts of over-runs into the rainy season,
- Recommendations for excavated material disposal and storage,
- Bypass of the 18-inch Redwood Shore Forcemain and other interim and bypass operations, and
- Rerouting and/or protection of major electrical feeders.

Refer to Section 11.1 for additional discussions concerning constructability of the ICP Project.

7.2 Construction Sequencing

Construction sequencing should consider the direction in which construction of the pipeline will occur. It is suggested that construction be planned to begin at the headworks, to allow for the use of the ICP during early startup of the headworks, as described below:

- The first segment would be constructed from the connection to the new Headworks Facility to first manhole downstream of the connection point, as shown in Figure 2-6.
- The second segment would be constructed from the manhole to the existing WWTP.

Sequencing should also seek to minimize impacts to plant access, processes, and maintenance activities.

7.3 Safety

Construction of the ICP will occur in the southern portion of the existing treatment plant, which is a highly-used area for plant staff, deliveries, and visitors. Open trench pipeline construction in such an area poses high risk of injury, making it critical to incorporate safety considerations into the design of the Project. Bid documents should include considerations for:

- Redirection of foot traffic,
- Clear markings of direction and location for deliveries,
- Alternate access points for emergency vehicles and personnel,
- Alternate routes for safe ingress and egress by staff and visitors,
- Means to control access to open trench construction,
- Barricades or other means to prevent falls during work and non-work hours,
- Typical and site-specific trench safety measures, and
- Typical and site-specific construction safety measures.

SVCW should review the construction zone to determine how construction of the ICP Project may impact internal safety and emergency procedures. For example, emergency meeting locations may be occupied by construction work and/or equipment. SVCW's safety policies should be reviewed and adjusted to accommodate construction activities. SVCW should inform staff by training and providing signage of any changes to safety policies.

Additionally, it is recommended that an SVCW liaison be appointed to attend progressive design-builder safety meetings to ensure work will be safe for both progressive design-builder and plant staff, and that construction activities do not conflict with plant activities. Scheduling of construction activities and early communication will keep all parties informed and aware of hazards during construction.

7.4 Corrosion Protection

Construction of the ICP will occur in marine soils where ground water may be brackish and corrosive. Corrosion protection means should be considered for:

- All ferric-based metallic fittings, such as flexible connections;
- Steel piling;
- Reinforced structures, such as manways; and
- Pipeline transitional pieces (e.g., HDPE to Steel coupling).

It is anticipated that the pipes themselves will be made of HDPE or a similar corrosion-resistant material and therefore will not require any corrosion protection.

7.5 Property Acquisition

Property Acquisition is not an anticipated activity for the ICP construction because all construction activities will occur on SVCW property.

7.6 Operational Plan

The operational plan will be closely tied to the design of the ICP. Because two pipes will be used (one for dry weather conditions and two for wet weather conditions), the flow at which the second pipe will engage must be determined. Once this operational point is determined, the pipe sizes may be confirmed. For the sake of this project planning process, engaging the second pipe for use was assumed to be for flows above 22.5 MGD which is the hourly PDWF. By establishing the size of the smaller pipe in this manner it then also allowed for maximum velocities at minimum (low) dry weather flows in order to avoid, to the extent possible, deposition of organic matter. The future designer will need to confirm this flow strategy with the latest operational plans and the time of design.

Switching between the two pipes is anticipated to require active control of automated gates within the new Headworks Facility and isolation at the point of connection to the existing treatment plant. Operations to isolate the wet weather pipe while it is offline will need to be developed once the design of the ICP connection to the existing treatment plant is further developed.

Since the larger pipe will only engage during high flows, stagnant water will develop between rain events. Water in the pipe can either be pumped out or dosed with a nitrate salt like Bioxide. Chemical dosing will require the handling and possible storage of an additional chemical, while pumping the pipe dry will require either the installation of a permanent pump dedicated to this activity or the use of a portable pump. Furthermore, if the standing water in the pipe is drained, the drained pipe would need to be filled with clean, recycled water, to counteract buoyant forces acting on the pipe in the Young Bay Mud present in the ICP Project site. However, if the pipes need to be drained for maintenance, the preferred method to do this would be through the use of a portable pump system.

In addition, a maintenance plan that includes exercising the ICP valves should be developed in accordance with the valve manufacturer’s recommendations.

7.7 Permits

No Federal, State, Regional, or Local permits are required because all construction is anticipated to occur on developed SVCW property. As part of the design effort, the designer shall confirm that no permits are required; permits will be the requirement of the selected progressive design-builder.

7.8 Ventilation and Odor Control

Ventilation and odor control are likely unnecessary because the ICP will be fully submerged during operation.

7.9 Security

There are no anticipated security concerns for this project.

7.10 Structural and Architectural

Structural design consideration should be given to differential settlement between existing structures, new structures, and the new pipeline. Selection of properly-sized flexible connections as points of connection will be required by the designer of record.

7.11 Electrical

A power supply will be required for any new motor operated valves. If a permanent pump is selected as the means of dewatering the larger pipe between rain events, this pump would also require a power supply.

7.12 Primary and Standby Power

At a minimum, the location of the power supply should be identified to ensure the existing electrical distribution equipment can accommodate the additional electrical loads. Any motor operated valves installed in the project should fail open and be fitted with means for manual operation in the event of power loss. Draining the pipes is not a critical operation, so standby power is not required for pumps (if any are determined to be required for the Project during design).

7.13 Surge Control

No surge control is anticipated for the ICP because the pipes flow under gravity and do not have any fast-acting valves.

7.14 Instrumentation and Controls/SCADA

Instrumentation and SCADA requirements for motor operated valves and pumps (if any) will be determined during design. Communication criteria and network capabilities also need to be reviewed to ensure the existing infrastructure is sufficient to handle any new input/output (I/O).

7.15 Interim Operations and Bypass Requirements

The ICP will impact the 18-inch Redwood Shore Forcemain. One potential solution to address the impacts to the 18-inch forcemain is to redirect the flows from the 18-inch forcemain to the existing 54-inch RCP forcemain. To confirm the feasibility of this bypass, a hydraulic analysis of the 54-inch forcemain and the 18-inch forcemain must be performed to confirm pumping and system capacity of each. Final design will have the 18-inch forcemain directed to the inlet of the new Headworks Facility. Potential interim operation of the new Headworks Facility will require redirection of both forcemains. The bypass of the 18-inch forcemain may be included as part of the interim operation layout.

Additionally, interim operation and bypass requirements of the existing Influent Lift Station will need to be developed as design progresses. Lastly, an in-depth utility review may identify additional interim operations or bypass requirements of previously unforeseen impacted utilities.

7.16 Geotechnical

Detailing of the trench excavation, how the pipe will be supported, and pipe backfill are among the most important design consideration. The ICP alignment is underlain by Young Bay Mud. This is a soft soil with limited bearing capacity that creates a more complicated design approach. The 2017 GIR by CDM Smith is to be reviewed for additional discussion of design recommendations. Of note, are the recommendation to:

- Use a pile support system and light weight fill as part of the design of the pipe, to minimize settlement.
- Use aggregate piles if manholes are included in the pipeline design, and
- Have dewatering to be limited to the trench, and to avoid external dewatering in order not to induce settlements.

Though a pile supported pipe is recommended in the GIR, an earth supported pipe was used in the Alternatives Analysis to complete the cost estimate, and was based on a recent outfall project at the WWTP. To reduce cost the design builder should evaluate how to limit the number of piles or have the pipe earth supported rather than pile supported to develop the most efficient and cost effective design. Excavation and shoring design require special attention, since the weak Young Bay Mud has the potential to cause trench failures.

7.17 Stakeholders

As the ICP project is anticipated to be located solely on SVCW property, no other stakeholders will be immediately impacted by this project.

7.18 Environmental Impacts

No additional design considerations are anticipated for environmental impacts beyond those identified in the Environmental Impact Report. These environmental impacts are summarized below:

- Visual Environmental Impacts during Construction: Construction of the ICP Project will require construction equipment and vehicles on-site, which will result in visual environmental impacts during construction. However, the FoP area where the ICP will be located is only visible from SVCW property and nearby walking trails. Therefore, the visual environmental impacts during construction of the ICP Project will be minimal. Because the ICP consists of a dual buried pipeline, there will be no visual environmental impacts during operation of the ICP.
- Air Quality Impacts during Construction: Construction and associated activities will result in temporary increases in air pollution emissions from construction equipment exhaust, earth disturbance, truck traffic, and construction-related vehicle trips to and from the site. According to the current program implementation schedule, the Project will be constructed in coordination with the Headworks Facility, which will be constructed between the years 2017 and 2018. A summary of annual emissions from construction-related activities for the Headworks Facility Project, inclusive of the ICP Project is presented in Table 7-1 below.

Because the Headworks Facility Project includes much more construction activities than the ICP Project, it may be assumed that the majority of the emissions presented in Table 7-1 are due to construction activities of the Headworks Facility Project. The ICP will have negligible impacts during operation because the project will require almost no new vehicle trips to the project area.

Table 7-1 Annual (tons) Emissions from Construction of the Headworks Facility and the ICP

Year	ROG	NOx	CO	SO_2	PM_{10}	$PM_{2.5}$
2018	0.24	2.60	1.90	3.56E-03	0.11	0.10
2019	0.12	1.26	0.84	1.59E-03	0.06	0.06
2020	5.72E-03	0.06	0.04	8.00E-05	2.77E-03	2.56E-03

- Greenhouse Gas Emissions during Construction: In addition to the above-mentioned air quality impacts during construction, there will be short term emissions of construction-related greenhouse gas emissions (GHG) during the period of construction mentioned above (2018-2020). Again, the estimates provided in Table 7-2 reflect the GHG emissions emitted by construction of the Headworks Facility, including the construction of the ICP Project. Therefore, it may again be assumed that the majority of the emissions presented in Table 7-2 are due to construction activities of the Headworks Facility Project. The Bay Area Air Quality Management District currently has no recommended significance threshold of GHG emissions resulting from construction projects. However, SVCW plans on implementing some of the practices listed below to reduce construction GHG emissions to less than significant levels:

  - Using alternative-fueled (e.g., biodiesel, electric) construction vehicles/equipment of at least 15 percent of the fleet, as feasible;
  - Using local building materials (within 100 miles) of at least 10 percent; and
  - Recycling at least 50 percent of construction waste or demolition materials.

Table 7-2 Annual (tons) GHG Emissions from Construction of the Headworks Facility and the ICP

Year	GHG
2018	317
2019	140
2020	7

- Impacts to Biological Resources during Construction: The construction of the ICP Project will have no anticipated impacts to biological resources.

Section 8

Life Cycle Cost Estimate

Since the Life Cycle Cost Analysis TM (Attachment E) was presented to SVCW on September 1, 2016, the following inputs to the LCC analysis were refined:

- Construction Cost from $4,424,000 to $4,300,000
- Annual O&M Labor Cost from $6,500 to $6,000
- Pipe Breakage Repair Cost of approximately $500,000

The refined LCC Analysis is presented in this section of the report.

8.1 Life Cycle Cost Estimate Assumptions

A LCC analysis was performed for the selected alignment alternative, Alternative F3 described in Section 6 above. The assumptions used for the analysis are detailed in Tables 8-1 and 8-2 below.

Table 8-1. Capital Cost Estimate Assumptions

Assumption	Value
Construction Cost (2016)[1]	$4,300,000
Midpoint of Construction[3]	2018
Escalation[2]	0.04
Project Contingency[2]	0.25
Soft Costs[2]	0.43
Market Fluctuations, Low	-0.05
Market Fluctuations, Base	0
Market Fluctuations, High	0.15

[1] Raw Construction Cost in 2016 dollars (US) based on the construction cost included in the Alternatives Analysis TM presented to SVCW. This differs from the Construction Cost presented in the Opinion of Probable Cost of Construction TM, dated May 2016 ($4,424,000) due to the inclusion of different contingency costs.

[2] Based on guidance in the Life Cycle Cost Analysis Guidelines TM, dated July 2016.

[3] Based on CIP Program Schedule Version #21, dated July 2016.

Table 8-2. O&M and Rehabilitation/Replacement Cost Estimate Assumptions

Assumption	Value
Discount Rate for Rehabilitation/Replacement	0.07
Discount Rate for O&M	0.03
Year of Beneficial Use	2022
Useful Life (years)	75
Power Cost (¢/kWh)	12.9

As design progresses and construction schedules are updated the Life Cycle Cost Estimate should be reviewed and updated.

8.2 Life Cycle Cost Breakdown

8.2.1 Overview

Using the assumptions listed in Section 8.1 above, a life cycle cost analysis was developed for the selected alignment alternative. This life cycle costs for the SVCW Influent Connector Pipes does not include Design Costs, but does include the following cost components:

- Capital Costs
- O&M Labor
- Power
- Equipment Rehabilitation and Replacement
- Pipeline repair

The cost for each of the components listed above were developed for each year over a 75 year period between 2018 and 2093 in present day dollars. The Net Present Value of the cash flow over that 75 year period was then calculated for all the cost components.

Per the Plastic Pipe Institute, HDPE has a useful life of 75 to 100 years. Because of the soil conditions that exist at the treatment plant vicinity and to be conservative, a 75-year life cycle for HDPE pipe was selected.

The results of the analysis are details in Table 8-3 below, with the breakdown of each of the costs in the Table described in further detail in Sections 8.2.2 to 8.2.7 of this report.

Table 8-3. Total Life Cycle Costs

	Cost
Capital Cost (2018 Dollars)[1]	
Base Market Fluctuation	$7,800,000
Low Market Fluctuation	$7,600,000
High Market Fluctuation	$8,500,000
Annual O&M Labor Costs	
Annual Labor Cost	$6,000
Annual Power Costs	
Annual Power Cost	$6,500
Rehabilitation and Replacement Costs	
Motorized Gate Repair Cost (every 5 years/Gate)	$3,500
Condition Assessment Inspection Cost (every 10 years/Pipe)	$11,500
Sump Pump Replacement Cost (every 10 years/Pump)	$400,000
Pipe Cleaning Cost (every 20 years/Pipe)	$18,100
Pipe Breakage Repair Cost (once per lifetime)	$500,000
75-Year Life Cycle Cost (LCC) for Influent Connector Pipe	
Capital Cost[2]	$7.6 - $8.5 million
NPV of Labor, Power, and Rehabilitation/Replacement	$3.7 million
75-year LCC (2022 Dollars)[1]	$11.3 - $12.2 million

[1] Capital Cost reflects the Raw Construction Cost included in Table 8-1 with Project Contingency, Soft Costs, Market Fluctuations, and Escalation applied to the raw cost.

[2] Range based on market fluctuations from -5 to 15 percent.

8.2.2 Capital Cost

The capital cost, in 2016 dollars, is calculated based on the project's raw construction cost, project contingency, soft costs, and market fluctuations, according to Equation 1, below. The result from Equation 1 is then escalated to the mid-point of construction.

Capital Cost = Construction Cost · (1+ Project contingency +∑Soft Costs + Market Fluctuations) [Equation 1]

The capital cost was determined to be between $7.6M to $8.5M depending on market fluctuations, as shown in Table 8-3 above. The raw construction cost used in the calculation is shown in Table 8-1 above.

8.2.3 Annual O&M Labor Cost

The annual operation and maintenance activities associated with the ICP project are summarized in Table 8-3 above, while the itemized labor costs associated with motorized gates and maintenance management are summarized in Table 8-4 below. The total number of labor hours was divided by 2,080 hours to determine the number of Full-Time Equivalents (FTE) of labor required. The cost associated with the labor was then calculated based on a cost of $150,000/FTE, per the Life Cycle Cost Guidance TM, July 2016.

Table 8-4. Itemized Annual Labor Costs

Activity	Staff	Frequency		Total Annual
	Hours	No.	Basis	Staff Hours
Motorized gates				
Inspection	0.5	2	per year /gate	2
Channel Cleaning	1	1	per week	52
Maintenance Management				
Generating Work Orders, Procurement, Tracking Work Progress	0.5	1	per week	26
			Total Staff Hours	80
			FTEs	0.04
			Total Labor Cost	$ 5769
			Rounded Labor Cost	$ 6000

8.2.4 Annual Power Cost

The power costs associated with the Influent Connector Pipes Project are summarized in Table 8-3 above and itemized in Table 8-5 below. Power costs for the project are determined by multiplying the estimated annual power usage of each type of equipment by the electrical cost. For the Influent Connector Pipes Project, the electric cost is $0.129 per kilowatt-hour used, per the Life Cycle Cost Guidance TM. Operation of gates and the sump pump are assumed to be 100 days per year because they are only used during wet weather events.

Table 8-5. Itemized Annual Power Costs

Equipment	Power Demand (Hp)	Total No. of Units	Average No. Operating	Total Power Use (kWh/yr)	Annual Power Cost
Gates					
Slide Gates	2	16	2	1560	$201
Pumps					
Sump Pump	20	2	1	46789	$6037
				Total Annual Power Cost	$ 6237
				Rounded Annual Power Cost	$ 6500

8.2.5 Rehabilitation and Replacement

The rehabilitation and replacement activities associated with the Influent Connector Pipes are summarized in Table 8-3 above and itemized in Table 8-6, below. The frequency and cost associated with each activity are also shown. Rehabilitation and replacement activities and costs were determined for the gates and pumps are based on typical equipment lifespan and costs. Pipe cleaning was assumed to occur at long intervals due to HDPE pipe's interior smooth surface, which allows minimal accumulation of Fats, Oils, and Grease. A three-man crew and light equipment would be required for cleaning. A pipe breakage repair estimate, though not anticipated is also included to account for a simple, incidental breakage. Anticipated costs for repair are an assumed value based on institutional knowledge and understanding.

Table 8-6. Itemized Rehabilitation and Replacement Costs

Equipment/Activity	No. of Units	Type of Rehabilitation	No.	Basis	Cost of Rehab
Motorized Gate	2	Repair	1	every 5 years /Gate	$3,500
Pipe Condition Assessment	2	Inspection	1	every 10 years/Pipe	$11,500
Sump Pump	2	Replacement	1	every 10 years /Pump	$400,000
Pipe Cleaning	2	Cleaning	1	Every 20 years/Pipe	$18,100
Pipe Breakage Repair	2	Repair	1	Once per lifetime[1]	$500,000

[1] Based on a single pipe break.

8.2.6 Net Present Value Analysis

The Net Present Value (NPV) of the cost components was calculated in two steps. First, the O&M costs for each year from 2018 to 2093 were developed by escalating the costs presented in Sections 8.1 through 8.2 to the year in which the cost would be incurred using Equation 2.

$$FV = PV \cdot (1+i)^{(Y_n - Y_{2016})} \quad \text{[Equation 2]}$$

where:

FV= Future Value
PV = Present Value

i = Escalation (4%)
Y_n = Year of Cost Occurrence
Y_{2016} = Present Year (2016)

The NPV of the escalated costs were then determined by discounting the costs to the Year of Beneficial Use, using Equation 3. For this LCC analysis, the Year of Beneficial Use was assumed to be 2022. Discounting was performed, according to Equation 3, on all future costs occurring after the Year of Beneficial Use. All costs incurred before the Year of Beneficial Use are considered "sunk costs" and are calculated using Equation 2 and then added to the sum of costs calculated with Equation 3 to determine the 75-year LCC at the Year of Beneficial Use.

$$Z_i = FV_i \cdot (1+r)^{-(Y_n-Y_b)} \qquad \text{[Equation 3]}$$

Where:

Z_i= Future Cost at Year of Beneficial Use
FV_i = Future Value, as calculated by Equation 1
r = Discount Rate (7% for rehab and replacement, 3% for all else)
Y_n = Year of Cost Occurrence
Y_b = Year of Beneficial Use

8.2.7 Conclusions

The 75-year LCC associated with the SVCW Influent Connector Pipeline, calculated as described above, is summarized in Table 8-7. As show, the total 75-year LCC is determined to be between $11.3 and $12.2 million dollars (in 2022 dollars), depending on market fluctuations.

Table 8-7. 75-Year Life Cycle Cost (LCC)

	2022
Capital Cost[1]	$7.6 – 8.5 million
NPV of Labor, Power, and Rehabilitation/Replacement	$3.7 million
75-year LCC (2022 dollars)[1]	$11.3 – $12.2 million

[1] Range based on market fluctuations from -5 to 15 percent.

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Section 9

Energy

9.1 Energy Analysis

The Energy Action Plan II is the State of California's principal energy planning and policy document, which includes the following energy efficiency action specific to WWTPs such as the proposed project:

- Identify opportunities and support programs to reduce electricity demand related to the water supply system during peak hours and opportunities to reduce the energy needed to operate water conveyance and treatment systems.

Therefore, an energy analysis is included in this section for the influent connector pipeline project. Anticipated operational energy usage for the various slide gates and sump pump associated with the ICP project is detailed in Table 9-1.

Table 9-1. Annual Operational Energy Usage

	Energy (kWh/yr)
Slide Gates	1,560
Sump Pump	46,789
TOTAL	**48,349**

9.2 Energy Efficiency Features

Minimizing head loss is a key criterion for the ICP Project. Pipeline sizes are based on limiting head loss to approximately 2 feet. Minimizing head loss results in a lower headworks hydraulic gradeline across the new Headworks Facility, which results in lower total dynamic head at the RLS pumps. Therefore, minimizing head loss was the primary energy efficiency feature of this project, and no additional energy efficiency features are anticipated to be required for this project.

The ICP normal mode of operation is by gravity flow, requiring no energy for pumping wastewater flow within the pipe. The construction of the dual pipeline within a single trench will reduce excavation cost which will translate to savings in energy utilization during construction (for excavation, material transport, backfill, and material disposal).

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Section 10

Preliminary Project Level Schedule

A preliminary project level schedule is shown in Figure 10-1 on the next page. This schedule assumes the Project for the construction of the ICP will be bid and constructed separately as a stand-alone project.

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SVCW
Capital Improvement Program
WWTP Improvements
Planning-Level - Construction Schedule - Influent Connector Pipeline (ICP) Project

listen. think. deliver.

Activity ID	Activity Name	Duration In Work Days
	SVCW Influent Connector Pipeline Project	183
	Construction (Begin Dry Season)	183
	Sitework	37
SW-1000	Issue Notice to Proceed (NTP)	0
SW-1010	Pipeline Contractor Mobilization	15
SW-1020	Locate / Relocate Existing Underground Utilities	10
SW-1030	Install Best Management Practices (BMPs)	5
SW-1040	Clearing & Grubbing	7
SW-1050	Install Dewatering System	5
	New Dual Influent Connector Pipeline (ICP)	146
ICP-2000	Excavation/Sheet Piles/Manway Installation	84
ICP-2010	Trench and Install Dual Pipes	64
ICP-2020	Backfill Trench Excavation	10
ICP-2025	Install Valves and Appurtenances	5
ICP-2030	Pressure Test Pipe & Accept	2
ICP-2035	Abandon Existing Pipes	5
ICP-2040	Cleanup & Site Restoration	25
ICP-2050	Project Complete	0

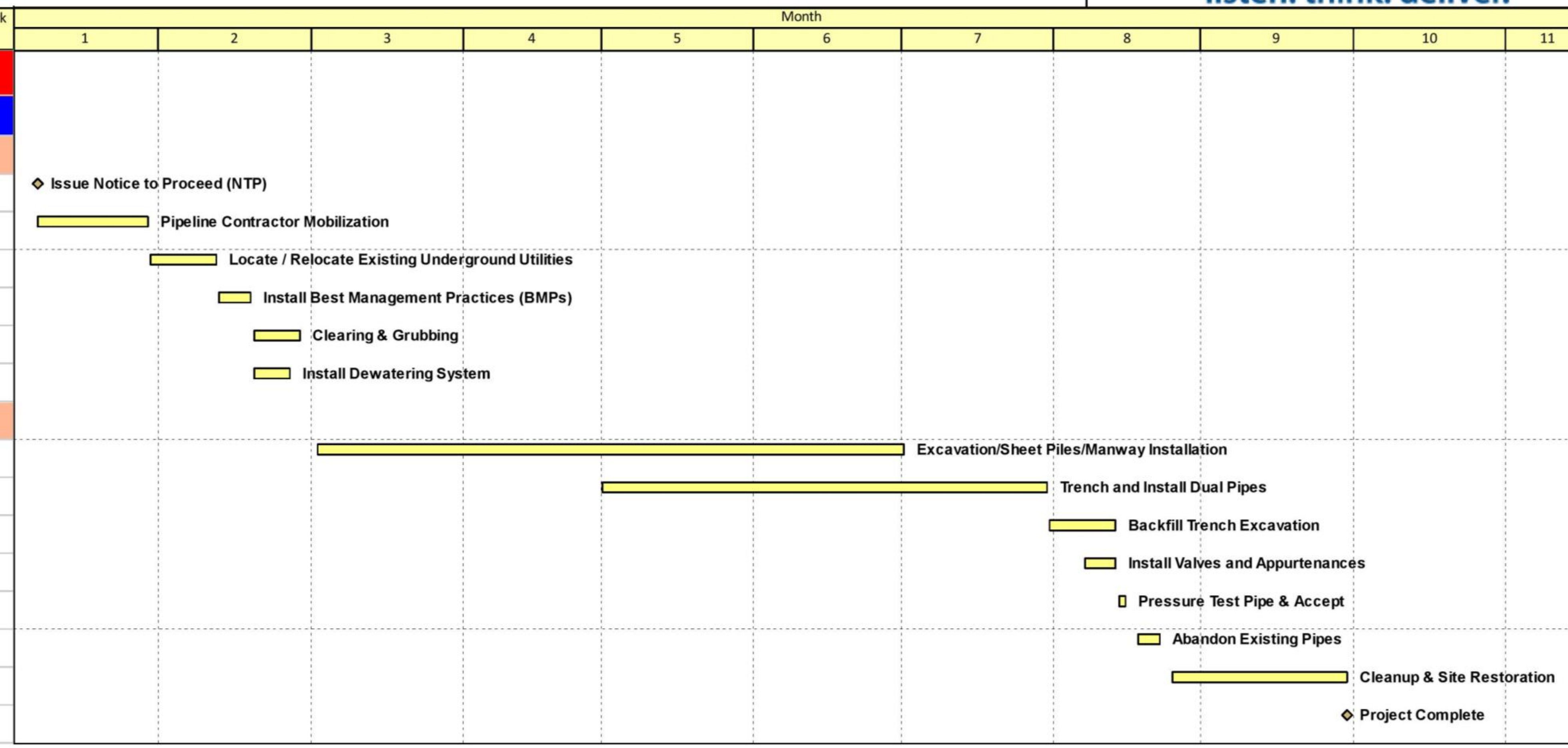

Figure 10-1 Preliminary Project Level Schedule

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Section 11

Outstanding Issues to Carry Into Design

11.1 Description of Key Issues

The following key issues require further consideration during design:

- Hydrologic mapping (See Section 2.3.1)
- Buoyancy/structural analysis (See Sections 2.3.2 and 11.1.3)
- Final pipe sizing (See Section 5.1.1)
- Detailed design of flexible connections between rigid structures and flexible pipeline (See Section 5.3)
- Hydraulic calculations (See Section 6)
- Trench design and pipe support based on geotechnical information (See Section 7.16)
- Valving configuration and details of connection to the existing plant (See Section 11.1.1)
- Construction Schedule (See Sections 11.1.2 and 11.5)
  - Sequencing of construction of the ICP in relation to the new Headworks Facility and other Front of Plant facilities
- Control strategy for switching between single pipe and dual pipe operations for wet weather flow (See Section 11.1.3)
  - Management of standing water in the wet weather pipe after use (draining or chemical dosing)
- Selection of joint-less pipe technology, materials, and application to project (See Section 11.1.4)
- Site survey, topographic survey, and examination of the property boundary (See Section 11.1.5)
- Utility location surveying/potholing (See Section 11.1.6)
- Review of constructability (See Section 11.1.7)
- Update of cost estimate based on preferred alignment, selected pipe sizes, trench design, backfill material, and pipe material (See Section 11.1.8)
- Determination of need and means of access inside the pipe (See Section 11.1.9)

- Need for and recommended location for manways, if any (See Section 11.1.9)
- Means of access for deliveries, plant operations, and visitors during construction (See Section 11.1.10)
- Seismic Importance Factor to be utilized by the agency based on either Risk Category 3 or 4 (See Section 11.1.11)
- Soil borings (supplemental subsurface exploration) (See Section 11.2)

11.1.1 Connection to the Existing WWTP

ICP project development and planning was focused primarily on identifying the general alignment of the ICP and did not make a final determination on the connection to the existing WWTP. Moving forward, the designer should consider the following:

- Point of connection(s) and piping arrangement including
    - A piping arrangement to allow bypassing of wet weather flows to the existing screening facility to reduce head loss at those flows;
- Bypassing of plant flows during tie-in including
    - Use of the existing bypass piping connection near the plant's Influent Lift Station;
- Phasing out and demolition of the Influent Lift Station; and
- Valving arrangement and type of valve for isolation of the wet weather pipe when not in operation including
    - WWTP's existing electrical and network capacity for new motor operated equipment.

11.1.2 Construction Schedule

A program level construction schedule has been developed during the ICP project development as shown in Section 10 of this report. All FoP facilities, including the ICP will be combined into a single progressive design-build contract. Further consideration needs to be given to how ICP will be sequenced relative to other FoP facilities. If desired, the progressive design-builder may sequence ICP construction at the beginning of the FoP project.

Depending on pending decisions regarding early startup of the new Headworks Facility, as discussed in Section 11.5 below, the ICP Project construction schedule is expected to be adjusted.

11.1.3 ICP Operational Control Strategies and Maintenance

Currently the pipes are sized to convey up to 22.5 mgd (2015 hourly PDWF) in a 48-inch dry weather pipe and the remaining flow of 57.5 mgd in the 72-inch wet weather pipe. Above 22.5 mgd, flow will be split between the 48-inch pipe and 72-inch pipe. Operational control strategies should be refined to minimize head loss, prevent solids settling, and account for buoyant forces across the entire range of flows.

The designer must select a method for handling stagnant water remaining in the ICP's wet weather pipe after it has been taken out of use. Possible approaches include:

- Do nothing;
- Chemical dosing (e.g. nitrate salts); or
- Pumping the standing water out of the pipe and refilling it with recycled water to combat the buoyant forces exerted by the soils in the area.

Each operation method needs to be evaluated for impacts to the plant process, need for new infrastructure, construction and operational costs, as well as Operations and Maintenance preference.

Maintenance procedures of the ICP need to be developed, including identifying regular inspections for condition assessment, potential dewatering and cleaning of the pipe, exercising of isolation valves, and replacement of valve components. These maintenance procedures, and others identified during design should be discussed further to ensure design of the ICP can accommodate the recommended maintenance activities.

11.1.4 Joint-less Pipe Technology Review

The ICP will be constructed from a joint-less pipe material, likely HDPE. Further consideration should be given to the type of HDPE pipe given the various sizes of the ICP. HDPE pipe 63-inches and less is manufactured by extruding the pipe creating a solid wall and is then heated and fused together in the field forming a "joint-less" pipe system. HDPE pipe sizes larger than 63-inches require HDPE pipe to be manufactured by spiral winding around a mandrel. This spiral wound HDPE pipe is also known as profiled wall pipe. The cross section of the pipe wall and the jointing method used differs between manufacturers. During design, the 72-inch HDPE profile wall pipe should further review of the jointing method, pressure rating, and flexibility of the profiled pipe to confirm the technology is being used correctly and meets design criteria. Availability of progressive design-builders qualified to install joint-less pipe and competition shall play a role in selecting the final pipe material and joinery requirements.

11.1.5 Site Survey

Currently no site survey (topographic survey) for the ICP Project has been done. A site survey will aid in producing a complete and accurate design of the project. In addition, it will be required to accurately delineate property boundaries to assure ICP construction stays within the SVCW property boundaries under all conditions.

11.1.6 Utility Location Survey

A utility location survey that includes potholing will be required to produce a complete and accurate design. Identifying correctly the location of all utilities allows for a more accurate and complete design while aiding in construction planning to minimize construction delays and increased cost due to unmarked or mismarked conflicting utilities.

11.1.7 Constructability Review

No constructability review of the ICP has been completed as part of planning phase project development. Design of the ICP should be reviewed to ensure construction methods are consistent with accomplishing the design within the assumed schedule. The following paragraphs discuss points requiring additional review consideration.

Currently, it is assumed that a trench 20-feet wide will be excavated to install both parallel pipes simultaneously. A sheet pile system and tremie seal base slab should be considered to control the excavation and allow for placement. Further constructability review should confirm the feasibility of this approach and reassure construction activities can occur within the confined construction area.

Review of the construction during dry and wet weather seasons should be considered. Any fatal flaws of construction during wet weather conditions should be identified to allow SVCW and the progressive design-builder to plan accordingly.

After the utility investigation, all bypass operations, relocation, or protection of utilities should be identified and reviewed (in particular, the bypass of the 18-inch Redwood Shore Forcemain). Currently it is assumed the flow from the 18-inch forcemain may be redirected into existing 54-inch RCP forcemain, which will be abandoned once the ICP is fully constructed and operational. Construction approaches need to be reviewed for feasibility, as well as technical issues including hydraulic impacts.

11.1.8 Cost Estimate Update

As part of the Alternative Analysis during planning phase project development, an opinion of probable construction cost was completed. Since then some assumed aspects have changed (e.g. pipe sizes changed from twin 63 inch pipes, to parallel 48-inch and 72-inch pipes). The cost estimate should be updated accordingly and include additional items as developed during detailed design.

11.1.9 ICP Access

From the Alternatives Analysis two points of access were identified for the ICP. Further consideration should be given to the correct number and location of the access points. Considering soil conditions and potential for differential settlement, there is risk of the structure settling on the non-rigid HDPE pipe. Additionally, manway access will be more expensive due to soil conditions and the need for a pile support system. When determining ICP access, it is suggested that the designer review if access can be achieved at the existing WWTP and/or the new Headworks Facility and if those access points meet the needs for maintenance activities. If it is determined that access may be provided in either of these locations, the need for manway access structures would be eliminated. Before elimination of manway access, a thorough evaluation must be completed including considerations for drainage/dewatering, safety, isolation, flexibility, and all other engineering and operations concerns.

11.1.10 Plant Access

Construction activities will temporarily block access through the main entrance to the WWTP as well as reduce access to the maintenance ramp into the plant’s storage and maintenance building.

As part of the Civil Site Improvements project, a new access will be provided on the northern side of the plant for truck deliveries and vehicle access and additional new parking outside the existing fence line of the WWTP. Detailed design and design documents should consider how to maintain safe access for plant staff and visitors, and reduce or eliminate impacts to the plant's maintenance ramp. Some options for pedestrian access include an elevated walkway routing foot traffic away from the construction area or designated paths at grade. A temporary gate should be provided for pedestrian access while the permanent gate is blocked due to construction.

11.1.11 Risk Category to Determine Seismic Importance Factor

Per the 2016 California Building Code Section 1604.5 a wastewater treatment plant shall be designed using a minimum Risk Category III. Based on CDM Smith preliminary geotechnical interpretive report, the Site Classification is most likely Site Class E for the pipe only. When determining the Risk Category for the ICP, it should be recognized as a critical part of the WWTP process and therefore should meet the same seismic design requirements as the rest of the WWTP. The ICP will use a minimum Risk Category III for design. A higher Risk Category of IV will be used for the ICP design if elected by SVCW. Selecting a higher Risk Category may have higher design and construction costs. The selected progressive design-builder will be required to use the identified risk category and site class, and gather additional seismic information to develop seismic design criteria for the ICP.

11.2 Further Field Investigations

Further field investigations will be required to progress design. It is recommended that at least two types of field surveys be performed:

- Site topographic survey/delineation of property boundary
- Utility survey/potholing

A site topographic survey should be performed to accurately identify the locations of surface features. Locations of surface features are imperative to determine a final ICP alignment. Secondly, a utility survey should be performed that includes potholing to confirm the alignment and vertical location of underground utilities. Confirming the location of underground utilities will inform selection of the final alignment and identify utilities requiring relocation, bypassing, protection in place, or other temporary services during construction of the ICP.

11.3 Further Alternatives Analysis

Alternatives Analyses for pipeline systems, valve, and pump type may be required as design dictates. However, no other major alternatives analyses are anticipated that have not been addressed in the Alignment Alternatives Analysis

11.4 SVCW Decisions

The following topics have been identified as key decisions requiring SVCW's attention that have impacts to the final design of the ICP:

- Early Startup of Headworks (See Section 2.5.2)

  - Means of connecting to the existing WWTP on interim
- Selection of material to meet design requirements of ICP (See Section 11.1.4)
- Means of accessing the ICP for inspection and maintenance and the need for and recommended location for manways, if any (See Section 11.1.9)
- Identifying the Seismic Risk Category to determine if the ICP seismic design should be based on Risk Category III or IV design criteria (see Section 11.1.11)
- Control of stagnant water in the larger pipe between wet weather events (See Section 11.1.3)
- Valving configuration and details of connection to the existing plant (See Section 11.1.1)

The ICP Project will be combined with the other FoP projects, including the Headworks Facility Project, under a single progressive design-build, which will offer economy of scale advantages and reduction in the number of progressive design-builders with some commercial advantages to the SVCW.

As described in Section 2.5 above, SVCW is considering using the new Headworks Facility prior to completion of other FoP facilities. Early startup of the Headworks Facility is accomplished by directing flow from the existing 54-inch RCP Forcemain to the Headworks Facility and then directing it back to the WWTP. Currently, it is assumed flow will be sent through a portion of the ICP to return it into the existing 54-inch RCP forcemain. SVCW will need to determine if:

1. Early startup of the Headworks will be implemented
2. And how flow will be conveyed.

Selection of material for the will most likely result in selection of large-diameter HDPE pipe, or similar pipe material. However, the designer will need to confirm that the jointing method is acceptable to SVCW's criteria of a "joint-less pipe". Additionally, there are limited manufactures of profiled wall pipe which may result in sole sourcing. SVCW should review the material and manufacture for acceptance to proceed with the design using a 72-inch HDPE pipe.

Access to the ICP should be allowed for operation, maintenance, and repair needs. SVCW needs to decide on what types of operation and maintenance activities need to be performed on the ICP. Once these operations have been identified, design can be adjusted to accommodate those needs. Additional decisions about access may include location of access points, number of access points, and locations for pipe drainage.

SVCW also needs to decide on the preferred means of handling stagnant water in the ICP. There are multiple approaches to handling water remaining in the wet weather pipe after use. The selected approach will affect what new systems need to be design.

11.5 Project Coordination

As mentioned in previous sections, additional coordination is required between the new Headworks Facility and the ICP. Major coordination is required with respect to both interim and final operations of the Headworks.

The approach and means of how to operate the new Headworks Facility prior to operating the new RLS pumps requires further thought and detailing. Once the interim approach is selected, the ICP design can be modified as needed to accommodate interim operating conditions. Changes may include additional fittings, valves, and piping to be included as part of the ICP design.

Final design of the new Headworks Facility will include details related to how the ICP will connect to the headworks. These details may include, but are not limited to: finalizing pipe size, selected pipe elevation, pipe support, pipe material, and flexible connections.

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Appendix A

References

1. Draft Environmental Impact Report – Silicon Valley Clean Water Wastewater Conveyance System and Treatment Plant Reliability Improvement Project: http://www.svcw.org/projects/Shared Documents/DEIR_SVCW Conveyance System Project_November 28.pdf

2. Capital Improvement Plan – 2015 Update; FY08-09 to FY22-23: http://www.svcw.org/programs/Shared%20Documents/Final%202015%20CIP%20Update%2007152015.pdf

3. 2011 Conveyance System Master Plan for the South Bayside System Authority: http://www.svcw.org/projects/63%20inch%20pipeline/CSMP_Aug_2011_Vol_2_Final.pdf

4. Geotechnical References:
   a. Brabb, E.E., Graymer, R.W., and Jones, D.L. (1998), Geology of the onshore part of San Mateo County, California: a digital database: U.S. Geological Survey, Open-File Report OF-98-137, scale 1:62,500.
   b. Brabb, E.E. and Pampeyan, E.H. (1983). Geologic map of San Mateo County, California: U.S. Geological Survey, Miscellaneous Investigations Series Map I-1257-A, scale 1:62,500.
   c. Cooper, Clark & Associates (1978a). "Foundation Investigation, Proposed Subregional Wastewater Works, Redwood City, California", Prepared for the 'South Bayside System Authority', February 15.
   d. Cooper, Clark & Associates (1978b). "Supplementary Subsurface Investigation and Laboratory Testing, SBSA Project Unit No. 1, Redwood City, California", Prepared for the 'South Bayside System Authority', October 18.
   e. Dames & Moore (1978). "Soil Investigation and Slope Stability Evaluations Construction Excavations, Subregional Wastewater Works, Redwood City, California" for South Bayside System Authority, December 22.
   f. Cooper, Clark & Associates (1980). "Progress Report: Installation and Observation of Groundwater Wells and Piezometers, proposed Main Structure, Redwood City, California", Prepared for the 'South Bayside System Authority', November 07.
   g. Cooper, Clark & Associates (1981). "Consultation: Re: Proposed Influent/Effluent Tie-In to Existing Force Main, Wastewater Treatment Plant, Redwood City, California", Prepared for the 'South Bayside System Authority', May 07.
   h. Fugro (2002). "Geotechnical Investigation and Data Report, SBSA SWTP Recycled Water System Storage, Redwood City, California" for South Bayside System Authority, October 17.

i. Fugro West, Inc. (2004a). "Recommended Su Profile for Shoring Design (Revised), South Bayside System Authority (SBSA), Redwood City, California", Prepared for the 'South Bayside System Authority', July 14.

j. Fugro West, Inc. (2004b). "Geotechnical Study: Recycled Water Storage and Disinfection Facilities, South Bayside System Authority, Redwood City, California", October 20.

k. Fugro West, Inc. (2004c). "Supplemental Geotechnical Recommendations, Recycled Water Storage and Disinfection Facilities, South Bayside System Authority (SBSA), Redwood City, California", Prepared for the 'South Bayside System Authority', November 08.

l. DCM|GeoEngineers (2009). "Technical Memorandum: New Administration and Plant Control Building Project, South Bayside System Authority Wastewater Treatment Plant, Redwood City, California", Prepared for South Bayside Authority, July 06.

m. DCM Consulting (2014). "Cone Penetrometer Test (CPT) Logs from the CPT Investigation at the SVCWTP site", Directed by David Mathy.

n. DCM Consulting (2015). "Supplemental Cone Penetrometer Test (CPT) Logs from the CPT Investigation at the SVCWTP site", Directed by David Mathy.

5. Geotechnical Data Report (CDM Smith, April 2017)

6. Geotechnical Interpretive Report (CDM Smith, April 2017)

Attachment A

Gravity Influent Connector Alignment Alternatives TM

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REPORT

Gravity Influent Connector Alignment Alternatives

Silicon Valley Clean Water

January 2016

CDM Smith

cdmsmith.com

Table of Contents

Executive Summary

Project Background

Silicon Valley Clean Water (SVCW) is implementing a Capital Improvement Program (CIP) to improve the reliability of the conveyance system. The CIP will consist of the following elements: replacement of conveyance system pump stations; replacement of conveyance system force mains; and upgrades to SVCW's treatment facility. A Conveyance System Master Plan (CSMP) was issued in 2011 and initial steps of the Plan are being implemented.

The CSMP identifies an influent connector pipeline to transport 80 million gallons per day (mgd) of raw wastewater from the future headworks facility to the influent side of the plant's existing primary treatment system. The presented project constitutes the installation of a raw sewage pipeline and/or the rehabilitation of a portion of the existing influent line.

Report Purpose

The purpose of the report is to provide a comprehensive overview of the features and design criteria of the gravity influent connector in its eight alignment alternatives and to summarize the engineering investigations and the evaluations, findings, and recommendations of the CDM Smith team.

The following report details all eight alignment alternatives, three of which are being recommended by the CDM Smith team. All alignment alternative sections include descriptions and considerations including soil conditions, headloss and velocity concerns, impacted utilities, impacts on plant access and activities, permits required, and alignment cost.

Alignment Alternatives

To provide a reliable influent connector pipeline from the new headworks, either a new pipeline will need to be installed and/or the existing influent line will need to be rehabilitated. The CDM Smith team has identified eight (8) alternatives, six (6) of which provide a new pipeline along various alignments, and two which connect to and rehabilitate the existing influent line. Table ES-1 below provides a brief description for each of the alternatives Table ES-2 provides a brief comparison of all these alternatives. In addition to the eight alternatives summarized below, CDM Smith has also investigated scenarios using the existing Influent Lift Station (ILS). SVCW through internal discussions has decided not to pursue ILS use and therefore are not included in this report. The marginal benefits gained do not compensate the high O&M and capital costs; especially when conveyance can be accomplished by gravity.

Table ES-1 Alignment Alternatives Summary Description

Option Name	Figure No.	Description
Alternative A : Rehabilitation of Existing Pipeline	ES-1	Alternative A includes installing 225 feet of new 63" HDPE pipe, using open cut construction, to connect the future headworks facility to the existing influent line and rehabilitating 575 feet of 54" RCP (Reinforced Concrete Pipe) and 175 feet of 60" RCP.
Alternative B: Replace Existing Influent Line	ES-2	Alternative B requires the removal of a portion of the existing 54-inch and all of the 60-inch influent line in order to install, in its place, the 975 feet of 84-inch HDPE pipe from the new headworks facility to the treatment plant.
Alternative C: New Pipe Alignment	ES-3	Alternative C includes installing 900 feet of new 84-inch HDPE pipe in a new alignment routed within the street right of way and plant property boundary.
Alternative D: Microtunnel in New Alignment	ES-4	Alternative D includes installing 940 feet of new 84-inch HDPE pipe, with 600 of the 940 feet microtunneled inside a new 90-inch steel casing. The alignment will follow nearly the same alignment as Alternative C.
Alternative E: CIPP + New Alignment	ES-5	Alternative E combines Alternatives A and C to install 900 feet of new 66-inch HDPE pipe in the same alignment as Alternative C and connects to and rehabilitate the existing influent line as outlined in Alternative A.
Alternative F1: Parallel Pipes	ES-6	Alternative F1 combines Alternatives B and C to install a total 1850 feet of new 63-inch HDPE pipe in a parallel configuration. Nearly 900 feet of 63-inch HDPE pipe will be routed along the alignment as outlined in Alternative B while 950 of 63-inch HDPE pipe will be routed in the alignment as outlined in Alternative C.
Alternative F2: Parallel Pipes	ES-7	Alternative F2 follows the Alternative B alignment but involves installing a total of 1900 feet of new 63-inch HPDE pipe in a parallel configuration thereby introducing redundancy to Alternative B.
Alternative F3: Parallel Pipes	ES-8	Alternative F3 follows the Alternative C alignment but involves installing a total of 1800 feet of new 63-inch HDPE pipe in a parallel configuration thereby introducing redundancy to Alternative C.
1. All measurements are approximate.		

Table ES-2 Comparison of Alignment Alternatives

Option Name	Description	Cost[1]	Bypass (Y/N)	BCDC Permit (Y/N)	Alignment Ranking Score
Alternative A	Rehabilitation of Existing Pipeline	$1,450,000	N	Y	130
Alternative B	Replace Existing Influent Line in Existing Alignment	$3,040,000	Y	Y	165
Alternative C	New Pipe Alignment	$2,820,000	N	N	165
Alternative D	Microtunnel in New Alignment	$11,670,000	N	N	255
Alternative E	CIPP + New Alignment (Alternatives A and C)	$3,740,000	N	Y	195
Alternative F1	Parallel Pipes (Alternatives B and C)	$4,210,000	N	Y	220

Option Name	Description	Cost[1]	Bypass (Y/N)	BCDC Permit (Y/N)	Alignment Ranking Score
Alternative F2	Parallel Pipes (Dual Alternative B)	$4,700,000	N	Y	180
Alternative F3	Parallel Pipes (Dual Alternative C)	$4,430,000	N	N	175

1. -30%/+50%Contingency

Preferred Alignment Alternatives

Of the eight presented alignment alternatives, four (4) are considered viable, Alternatives E, F1, F2, and F3; and are summarized in Table ES-3 below. The other four alignments are eliminated due to fatal flaws in constructability, operational, and/or functionality concerns. The ranking score for the preferred alignments are similar ranging from 175 to 220. OPCCs (Opinion of Probable Costs of Construction) are also similar differing within less than one million dollars ($1,000,000) of each other.

Table ES-3 Preferred Alignment Alternatives

Option Name	Description	Cost	Benefits of Alternative
Alternative E	CIPP + New Alignment (Alternatives A and C)	$3,740,000	▪ Minimal foot print. ▪ Lowest construction cost of dual pipes options ▪ Shortest construction time of dual pipe options ▪ Flexible construction schedule for BCDC permit ▪ Minimal impact to plant activities/access ▪ No influent bypassing required ▪ Redundancy
Alternative F1	Parallel Pipes (Alternatives B and C)	$4,210,000	▪ Flexible construction schedule for BCDC permit ▪ Increased reliability over Alternative E ▪ Minimal impact to plant activities/access ▪ No influent bypassing required ▪ Redundancy
Alternative F2	Parallel Pipes (Dual Alternative B)	$4,700,000	▪ Most utilities avoided by using existing alignment ▪ Least impact to plant activities/access of dual pipe options ▪ Increased reliability over Alternative E ▪ Redundancy
Alternative F3	Parallel Pipes (Dual Alternative C)	$4,430,000	▪ No BCDC permit ▪ Increased reliability over Alternative E ▪ No influent bypassing required ▪ Redundancy

Recommendation

Of the preferred alternatives identified in Table ES-3, alternative F3 has been identified because of its

- Low ranking score
- Medium cost
- Does not require a BCDC permit
- Provides increased reliability over Alternative E,
- Does not require influent bypassing,
- Provides redundancy.

Alternative F3 is parallel pipes running in a joint trench from the proposed headworks, through the southern portion of the treatment plant's property, and connects at or within the plant's existing ILS. The advantages of this configuration allows for minimizing headloss while providing adequate flow velocities throughout the range of flows experienced. By keeping velocities higher the occurrence of solids settling within the pipe(s) is reduced. It is estimated dual 63-inch pipes or some combination of a smaller and larger diameter pipe can be used to provide proper flows and velocities. Also in the event of maintenance the 2nd pipe can be used to allow for access to the other pipe without stopping flow to the plant.

The alignment reduces permitting requirements by having the pipe remain on SVCW property, with some impact to access and parking along the alignment. By constructing through the south side of the plant, the existing influent pipeline remains in service removing the need for costly bypassing. Additionally, since the existing influent pipeline remains in service, restrictions to construction schedules are more flexible to allow for early installation and reduced site congestion to other plant projects, like the proposed headworks.

Alternative F3 runs through the treatment plant, therefore extra consideration will have to be given to utilities as compared to some alignments. Existing utilities will require field location and as-built drawings review to reduce the risk of impacts to the plant and construction personnel safety.

Project Cost

Alternative F3 has a preliminary cost estimate of $4,430,000 and is less costly than most all of the other alternatives. Project cost includes for the construction of

- Two large diameter HDPE pipe, paralleled in a joint trench,
- Sheet pile shoring
- Concrete trench bottom and light weight backfill.
- Manway access at two locations.
- And pile supports at structures.

Conclusion

Design should move forward with the concept of having parallel pipes going through SVCW's property while trying to reduce impacts to plant access, existing utilities, and staff parking. Pipes' sizes, project cost, final alignment, and final connection approach will be developed further during design. Pipes should be sized to convey dry weather flow through one smaller diameter pipe and the remaining wet weather flow through a second larger diameter pipe. The pipe system needs to be designed against the high ground water and highly compressive clay soils found at the site. A review of record drawings and field locating by potholing or other means should be part of the design process to reduce potential costly changes during construction. The means of connecting to the existing plant to avoid using actuated valves, along with timing the decommissioning of the ILS, will require careful review and coordination. Lastly further discussion should occur for the relocating of existing utilities, such as those currently hanging on SVCW's property fence line; as the construction for the new pipe can accommodate these needs.

Section 1

1.1 Introduction

Silicon Valley Clean Water (SVCW) is implementing a Capital Improvement Program (CIP) to improve the reliability of the conveyance system. The CIP will consist of the following elements: replacement of conveyance system pump stations; replacement of conveyance system force mains; and upgrades to SVCW's treatment facility. A Conveyance System Master Plan (CSMP) was issued in 2011 and initial steps of the Plan are being implemented.

The CSMP identifies an influent connector pipeline to transport 80 million gallons per day (mgd) of raw wastewater from a future headworks facility to the existing screens just upstream of the primary treatment system. The presented project constitutes the installation of a raw sewage pipeline and/or the rehabilitation of a portion of the existing influent line. This interconnecting pipeline will transport up to 80 mgd of screened and degritted raw wastewater.

This report identifies a number of alignment alternatives to be used for the influent connector pipeline. Each alternative has been evaluated for a number of criteria to identify the most viable alternatives. A description of each alternative, its considerations, and recommended alternatives are provided in this report.

1.2 Influent Connector Pipeline Consideration

The design and construction of this conveyance system requires special consideration of the following:

- Constructing through very poor soils identified as "Young Bay Mud"
- Minimizing headloss to maintain a lower new headworks facility and in turn a smaller Receiving Lift Station (RLS)
- Maintaining in place and/or relocating existing utilities, including but not limited to: sewer, gas, irrigation, storm drain, electrical, and communication/data
- Providing bypass equipment and materials for plant influent flows.
- Minimizing impacts to plant activities and access (e.g. deliveries, staff parking and access, visitor access, etc.)
- Obtaining Special Permitting (San Francisco Bay Conservation and Development Commission, BCDC)

The pipeline alignment and method of construction to be used for the project include the following possible options to connect the new headworks and the existing primary sedimentation tanks:

- Rehabilitation of the existing pipeline
- Replacement of the existing influent pipeline with upgraded materials, such as HDPE (High Density Polyethylene)
- Installation of a new influent line in an alignment different from the existing alignment
- Microtunneling of a new influent line
- A hybrid of the above options

1.3 Procedure

Each of the alignment alternatives were evaluated for the following considerations:

- Soils Condition
- Headloss/Velocity
- Utilities
- Plant Access/Activities
- Permitting
- Operational Complexity
- Cost
- Fatal Flaws

Findings of each consideration against the alternative are discussed in detail below. In general each consideration was reviewed using input provided by SVCW, sound engineering judgment and assumptions, standard calculations, and record drawing reviews. A score as to the level of impact each consideration has been assigned to provide an overall rank to the alignment and help in evaluating which alternatives are the most viable.

1.3.1 Description of Cost Development

CDM Smith has prepared an Opinion of Probable Costs of Construction (OPCC) for each alternative identified. The computerized estimating system Sage Timberline Estimating System (TES) was used for preparing the OPCC. The system operates using CDM Smith's proprietary customized database. Current prevailing wage rates were used in the estimate to calculate labor based on the intended project construction bid period. Similarly, construction equipment pricing is based on Primedia Blue Book Equipment Rates adjusted for the bid period. Material pricing in the OPCC include pricing based on our TES Database in addition to bid and budget pricing we have obtained and adjusted to market conditions. The level of accuracy of the OPCC is consistent with the Association for the Advancement of Cost Engineering (AACE) best practice for a Class IV estimate which defines project definition between 1-15%. The expected level of accuracy of a Class IV OPCC ranges from -30% for the lower range of cost and +50% for the high range. The following assumptions and exclusions were used in preparation of the OPCC for each alternative described in the report:

- Bypass piping assumed to be dual lines that can sustain 80 mgd flow. Piping will be below grade where in roadways, otherwise it will be placed on grade.
- No cost has been added for hard dig or handling of hazardous materials.
- No ground improvements have been included in the estimate (i.e. driven piles or geo piers).
- No cost is included to abandon lines other than to bulkhead in existing or new manholes.

Furthermore, the OPCCs presented in the report do not include construction contingency, escalation, or engineering design fees or services during construction. The following markups are applied to the OPCCs as directed by SVCW:

- Building permits (% total cost) 1%
- Builder's Risk Insurance (% total cost) 1%
- General Liability Ins (% total cost) 1.5%
- GC Bonds (% total cost) 2%
- Sales Tax (Material) 9%
- General Conditions 10%
- Contractor OH&P 12%

1.4 Report Organization

The Gravity Influent Connector Alignment Alternative Report is organized into five sections as discussed below.

Executive Summary. The Executive Summary presents a summary of the project description, report purpose, alignment alternatives, preferred alignment alternatives, and the CDM Smith team's recommendations.

Section 1, Introduction. Section 1 describes the purpose, an introduction to evaluation approach, and organization of the Gravity Influent Connector Alignment Alternative Report.

Section 2, Development of Alignment Alternatives. Section 2 summarizes each of the eight alignment alternatives developed. Each of the alignment alternatives were described and evaluated for soils condition, headloss/velocity, utilities, plant access/activities, permitting, and cost. In addition, the pros and cons of each alternative were also considered.

Section 3, Evaluation of Alignment Alternatives. Section 3 describes how alignment alternatives are compared using a ranking procedure. The section includes a summary of the categories used in the alignment ranking/selection process and descriptions of importance factor assigned to each category. The section also presents a table complete with total ranking score of each of the alternatives and summarizes the ranking procedure. Lastly, the four viable alignment alternatives are presented and summarized in this section.

Section 4, Recommendation. Section 4 describes the CDM Smith's recommended alignment alternative. The section describes in detail why the recommended alignment alternative was chosen and presents a table summarizing the pros and cons of the alternative. The section also summarizes the project cost for the recommended alignment alternative.

Section 2

This section of the report presents the eight alignment alternatives developed to convey flow from the future headworks facility to the influent side of the plant's existing screens. Section 2.1 outlines the development of the alignment alternatives where all flow conditions will be under gravity forces.

Section 2.1 Development of Alignment Alternatives

Eight (8) alternatives were developed by expanding the above mentioned five (5) connection options to improve the reliability of the influent connector pipeline. Table 2-1 provides a brief description for each of the alternatives evaluated for full gravity flow. In addition to the eight alternatives summarized below, CDM Smith has investigated the use of the existing Influent Lift Station (ILS). Through internal discussions, SVCW has decided not to use the ILS as a means to convey peak flows through the new influent connector pipe. Therefore, these additional alternatives that consider the ILS for use are not included in this report for further discussion.

Table 2-1 Alignment Alternatives without ILS Summary Description

Option Name	Figure No.	Description
Alternative A : Rehabilitation of Existing Pipeline	2-1	Alternative A includes installing 225 feet of new 63" HDPE pipe, using open cut construction, to connect the future headworks facility to the existing influent line and rehabilitating 575 feet of 54" RCP (Reinforced Concrete Pipe) and 175 feet of 60" RCP.
Alternative B: Replace Existing Influent Line	2-2	Alternative B requires the removal of a portion of the existing 54-inch and all of the 60-inch influent line in order to install, in its place, the 975 feet of 84-inch HDPE pipe from the new headworks facility to the treatment plant.
Alternative C: New Pipe Alignment	2-3	Alternative C includes installing 900 feet of new 84-inch HDPE pipe in a new alignment routed within the street right of way and plant property boundary.
Alternative D: Microtunnel in New Alignment	2-4	Alternative D includes installing 940 feet of new 84-inch HDPE pipe, with 600 of the 1100 feet microtunneled inside a new 90-inch steel casing. The alignment will follow nearly the same alignment as Alternative C.
Alternative E: CIPP + New Alignment	2-5	Alternative E combines Alternatives A and C to install 900 feet of new 66-inch HDPE pipe in the same alignment as Alternative C and connects to and rehabilitate the existing influent line as outlined in Alternative A.
Alternative F1: Parallel Pipes	2-6	Alternative F1 combines Alternatives B and C to install a total 1850 feet of new 63-inch HDPE pipe in a parallel configuration. Nearly 900 feet of 63-inch HDPE pipe will be routed along the alignment as outlined in Alternative B while 975 of 63-inch HDPE pipe will be routed in the alignment as outlined in Alternative C.
Alternative F2: Parallel Pipes	2-7	Alternative F2 follows the Alternative B alignment but involves installing a total of 1900 feet of new 63-inch HPDE pipe in a parallel configuration thereby introducing redundancy to Alternative B.
Alternative F3: Parallel Pipes	2-8	Alternative F3 follows the Alternative C alignment but involves installing a total of 1800 feet of new 63-inch HDPE pipe in a parallel configuration thereby introducing redundancy to Alternative C.

2.1.1 Alternative A: Rehabilitation of Existing Pipeline

Cost: $1,450,000; Total Length: 975 feet (225 feet of 63-inch HDPE, 575 feet rehabbed 54-inch RCP (Reinforced Concrete Pipe), 175 feet rehabbed 60" RCP); Not a recommended gravity alternative due to high headloss.

This alternative, as shown in Figure 2-1, includes lining a portion of the existing influent pipe up to the existing screens upstream of primary sedimentation basin. It will also require approximately 225 feet of HDPE pipe to connect the new headworks facility to the existing influent line.

To be added: Figure 2-1. Alternative A: Rehabilitation of Existing Pipeline

Alignment Description

This alternative will start with a single 63-inch HDPE connecting pipe from the new headworks to the existing influent line. From this connection point the existing influent line will be rehabilitated to strengthen and seal the leaking pipeline.

Cured-in-place pipe (CIPP) liner is identified as a preferred method for rehabilitating the existing influent pipe. Other methods exist for rehabilitating deteriorated pipe (e.g. Linabond, Slip-lining), but CDM Smith feels a CIPP rehabilitation has added benefit over other methods. CIPP was selected due to the following advantages:

- Ease of installation
- Minimal to no need for man entry into the pipe
- Minimal construction foot print
- 360 degree repair
- Increased strength and resistance to settling of pipe
- Minimal reduction to inside diameter of pipe.

CIPP is a trenchless rehabilitation method using a felt liner, impregnated with resin (polyester, vinyl ester, or epoxy), that is inverted through the pipe using water or air, or is pulled into position. The liner is then cured in place with hot water, steam, or high intensity UV light. With an epoxy liner, hot water should be used to help maintain a better connection of the liner with the host pipe. An epoxy liner will result in one of the strongest and thinnest liners and will allow for the curing water to be disposed of into the sewer system. Other CIPP liner materials release styrene into the curing water, which, if discharged into the treatment plant, can disrupt the biological processes.

To install the liner, a minimum of two access pits and small staging areas for equipment will be required. These access pits will also be used for the construction of access man ways to allow for future access and maintenance. A third pit may also be required near the ILS if proper access is not allowed through the ILS vault. The first access point/man way is needed to install the liner for the length of 54-inch RCP and provide the point of connection for the HDPE pipe from the new headworks. This access point will also facilitate the installation of a new bypass connection. The second access pit will allow installation of the CIPP liner in the 60-inch RCP and termination of the 54-inch CIPP. The liner is anticipated to be one inch thick reducing the inside diameters to 52- and 58-inches. During installation and curing of the linear all plant flow will be bypassed using temporary pumps and pipes.

Alignment Considerations

Soils Condition

For the CIPP alternative, poor soils have to be accounted for; particularly in the new connecting pipe from the future headworks to the existing influent line. It is assumed approximately 225 feet of 63-inch solid wall HDPE pipe will be installed, with an invert of 11 to 13 feet below the average grade of 103 (plant datum). This short length of pipe, like other alternatives, have to consider young bay mud soil conditions; and each alternative is anticipated to require the following:

- Use of imported light weight backfill to minimize future settlement.
- HDPE-fused joint-less pipe to prevent leaks that would occur at joints due to settlement.
- Use of "burrito wrap" (geotextile wrapped drain rock) for pipeline bedding and embedment to prevent surface settlement due to migration of native fines into the porous bedding and embedment.
- Fully interlocked sheet pile shoring along the full length of the trench. The modular nature of the shoring allows for installation into almost any shape, and with the proper wales and struts, the excavation becomes very extensive. Sheet piles can be installed to provide a nearly watertight excavation in a variety of ground conditions. This makes them extremely useful for projects in loose, soft, or otherwise unstable soils with high ground water. All of which is descriptive of the Young Bay Mud soils for this project.
- Flexible connections at buried pipe connections to new and existing structures to allow for differential settlement.
- Permeation grouting of the existing pipeline bedding and backfill which historically has been found to be pea gravel to manage perched groundwater which is frequently stored in the porous existing pea gravel backfill.

The manhole and access pits for the CIPP alternative will also require mitigation for the poor soil conditions by using:

- Imported light-weight backfill to prevent future settlement of manhole structures.
- Fully interlocked sheet pile around access pits during construction.
- Flexible connections on pipe connections at manholes.

Headloss/Velocity

To maintain the headworks at a lower elevation, a pipeline design of 2 feet or less of headloss between the new headworks and the existing treatment plant has been selected. Preliminary hydraulic estimates show Alternative A will result in 8.5 feet of loss at 80 mgd of flow. By itself, CIPP is not a viable alternative under gravity flow making this a fatal flaw.

Utilities

Record drawings and Google Earth© overlays of utilities provided by SVCW have been reviewed to identify what utilities would be impacted or require relocation. Comparatively, very few utilities will be impacted using CIPP. The utilities that will be encountered are in the initial segment of pipe from the headworks to the existing influent pipe. No utilities are anticipated to be relocated under this alignment, though the following utilities will need to be protected in place:

- 4-inch plant gas main
- 18-inch sanitary sewer force main
- 16 kV electrical feeder conduit
- 4-inch potable water line

Plant Access/Activities

Plant access will intermittently be blocked while the first segment of pipe crosses Radio Road. It is not anticipated to have heavy or dramatic impacts to access. Construction in this area can be staged to accommodate everyday activities: both vehicle and foot traffic and emergency vehicle access. This alternative also has no impact to parking within the plant.

Permitting

CIPP activities will occur outside of the SVCW's property requiring a San Francisco Bay Conservation and Development Commission (BCDC) Permit. The permit will cover the excavation for access to the existing influent line and construction of manholes. Other construction activities may also require which can be part of one inclusive permit.

Cost

The OPCC of Alternative A is $1,450,000 (+50%/-30%). This estimated cost was determined using the information presented in the alignment description above and refined by comparing recent CIPP projects done by CDM Smith across the United States. Elements of the OPCC for this alternative included:

- Excavation, asphalt demolition, and replacement associated with the installation of the bypass
- Two new precast man ways
- Six (6) inch tremie seal trench slab
- Pipe construction including: trench shoring, dewatering, and pipe installation and materials

- Backfill using light weight material such as pumice

Piping Connection

Due to the poor soils, flexible connections will be used at all new structures to allow for differential settlement between pipe and structure. No new connections will be required at the existing plant as all existing connections and configurations can be retained.

Alignment Pros and Cons

A summary of the pros and cons has been listed in Table 2-2 below. The most significant issue with this selection is that it does not allow 80 mgd of flow while providing a low level of headloss as described above. Some of the most attractive features of this option are the trenchless construction approach and costs.

Table 2-2 CIPP Pros & Cons Summary

Pros	Cons
▪ Minimal foot print. ▪ Low construction cost ▪ Most utilities avoided. ▪ Least impact to plant activities/access. ▪ Shortest construction time.	▪ Full plant flow bypass. ▪ Liner decreases pipe diameter. • Headloss is greater than 8 feet. ▪ Requires BCDC permit. ▪ Slightly increases weight of pipe where settlement is of high concern.

2.1.2 Alternative B: Replace Existing Influent Pipe

Cost: $3,040,000; Total Length 975 feet of 84-inch HDPE Pipe.

This alternative is shown in Figure 2-2 and includes using open cut construction to remove a portion of the existing 54-inch and 60-inch RCP influent line and install a new HDPE pipe in place of the removed influent line.

To be added: Figure 2-2. Alternative B: Replace Existing Influent Pipe

Alignment Description

Like Alternative A, an HDPE pipe will be routed from the headworks (plant) south, to the easement where the existing RCP is located. A new manhole will be installed at this location which will also act as an access point for influent wastewater bypassing. The new manhole will provide for future access and maintenance. A second manhole can be installed at the east end of the alignment where the pipe turns east to connect with the existing plant. The existing 54-inch and 60-inch RCP force main between the two new manholes will be removed and a new 84-inch HDPE pipe along the same horizontal alignment will be installed using open cut construction. Interlocking steel sheet pile shoring will be required along the full length. The sheet pile shoring is required to maintain watertight conditions in the soft unstable soil along the open cut construction. Due to the routing, this alternative has similar impacts and advantages as stated in Alternative A.

Alignment Considerations

Soils Condition

This replacement alternative will also need to consider young bay mud soil conditions and has all of the same soil considerations as described in Alternative A, just on a larger scale. The pipe will be installed with an invert varying approximately 10 to 13 feet below ground and is anticipated to require the following for the entire length of pipe construction:

- Imported light weight backfill to prevent future settlement.
- HDPE fused joint-less pipe to prevent future joint leaks and settlement at the joints.
- Burrito wrap (geotextile wrapped drain rock) for pipeline bedding and embedment to prevent surface settlement due to migration of native fines into the porous bedding and embedment.
- Fully interlocked sheet pile shoring for the whole length of the trench and around new manhole locations, to provide a water tight excavation in this area of high groundwater and poor soils.
- Flexible connections at buried pipe connections to new and existing structures to allow for differential settlement.
- Permeation grouting of the existing pipeline bedding and backfill, which historically, has been found to be pea gravel to manage perched groundwater. Frequently, ground water is stored in the porous existing pea gravel backfill.

Headloss/Velocity

To minimize headloss at high flows (up to 80 mgd), a larger diameter pipe will be needed. The largest HDPE solid wall pipe commonly manufactured within the US at this time is a 63-inch pipe (outside diameter). Preliminary hydraulic estimates push headloss to approximately 7.5 feet at 80 mgd if a 63-inch HDPE pipe is used. Other alternative HDPE piping systems (spiral wound) are available in larger diameters up to 144-inches, e.g. Duramaxx, Spirolite, and Weholite. To reduce headloss in a single pipe to below 2 feet, an inside diameter of approximately 84- inches is required for this alternative.

However, with this size of pipe, the flow velocities for average dry and dry peak flows are low and allow for solids to settle within the pipe. Large quantities of solids may accumulate over time, and when a high enough flow does occur, the plant will be inundated by a high solids load. During these low flow periods the solids may also gone into a septic condition complicating treatment processes.

Utilities

Similar to Alternative A, very few utilities will be impacted using this alternative. The utilities that will be encountered are in the initial segment of pipe from the headworks to the existing influent pipe and where the pipe heads north to connect to the treatment plant. All influent flows will need to be bypassed during replacement of the existing pipeline. No major utilities are anticipated to be relocated under this alignment. Some minor utilities will need to be replaced such as irrigation and lighting conduit. Though the following utilities will need to be protected in place:

- 4-inch gas main
- 18-inch sanitary sewer force main
- 16 kV electrical feeder conduit
- 12-inch Storm Drain
- 4-inch potable water line
- Chemical feed line for sodium bisulfate

Plant Access/Activities

Plant access will be intermittently blocked while the first segment of pipe crosses Radio Road. It is not anticipated to have heavy or dramatic impacts to access. Construction in this area can be staged to accommodate everyday activities, both vehicle and foot traffic and emergency vehicle access. This alternative also has no impact to parking within the plant.

Access will also be intermittently blocked while the last segment of pipe heading north into the plant is installed. This portion will not have major impacts to access as plant activities can be detoured around the north side of the plant or construction can be phased to allow more direct detouring.

Permitting

Construction activities will occur outside of the SVCW's property, requiring a San Francisco Bay Conservation and Development Commission (BCDC) Permit.

Cost

The estimated OPCC of Alternative B is $3,040,000 (+50%/-30%). The Alternative B cost is inclusive of all work described in the alignment description. Elements of the OPCC for this alternative included:

- Excavation, asphalt demolition, and replacement associated with the installation of the bypass
- Two new precast man ways
- Six (6) inch tremie seal trench slab
- Pipe construction including: trench shoring, demolition of existing pipe, dewatering, pipe installation, and materials
- Backfill using light weight material such as pumice

Piping Connection

Due to the poor soils, flexible connections will be used at all structures to allow for differential settlement between pipe and structure. Connecting to the existing plant can conceptually be accomplished using a reducer and flexible connections to go from the 84-inch HDPE pipe to the existing 60 inch influent line

Alignment Pros and Cons

A summary of the pros and cons has been listed in Table 2-3 below. The most significant issue with this selection are the low velocity as described above, the risks associated with influent bypassing, and the need for a BCDC permit. Some of the most attractive features of this option are that the alignment maintains parking inside the plant and avoids major utilities.

Table 2-3 Replacement of Existing Influent Line Pros & Cons

Pros	Cons
▪ Sized to convey all flows. ▪ Most utilities avoided by using existing alignment (slightly more than CIPP alt.). ▪ Minimal impact to plant activities/access. ▪ Use of new materials increases reliability of pipe over CIPP.	▪ Full plant flow bypass during 3 to 6 months of construction. ▪ Low pipe velocity. • Settling of solids within large diameter pipe may occur. ▪ Requires BCDC permit. ▪ Short list of large diameter HDPE pipe manufacturers. ▪ Shoring and dewatering required ▪ Demolition of existing pipe

2.1.3 Alternative C: New Pipe Alignment

Cost: $2,820,000; Total Length 900 feet of 84-inch HDPE Pipe;

This alternative is shown in Figure 2-3 and includes the installation of approximately 900 feet of new 84-inch HDPE pipe from the headworks to the existing primaries. This alternative alignment will be routed within the street right of way and plant property boundary.

To be added: Figure 2-3. Alternative C: New Pipe Alignment

Alignment Description

This alternative is similar to the Alternative B but rather than extending the pipe to the existing influent line easement, the new pipe alignment will follow Radio Road into the plant and along the main access road through the plant, connecting at the existing influent pump station. Manholes may be installed in one or two locations for future access and maintenance. One may be installed at the turn in Radio Road where the pipe line transitions from south to east, while a second manhole may be positioned where the pipe turns north to connect into the main treatment plant.

The difficulty with this alignment is avoiding impacts to utilities and plant access. Of most importance in selecting an alignment for this alternative was to maintain access to the maintenance ramp at the southeast side of the plant. Second was to maintain access for vehicles, deliveries, foot traffic, and visiting personnel to the main office. Lastly was to consider avoiding major utilities. A 15-foot wide trench is assumed for installation of this pipe. The trench will be positioned in approximately the middle of the road that leads to the main entrance. Once past the plant water booster pump station, it will jog south, keeping approximately 5 to 7 feet off the fence/property line to avoid the gas main, chemical piping, and other utilities along that area. It will continue parallel to the property line until it turns north toward the plant. This route will keep approximately 20 feet clear between the maintenance ramp and the trench. This should provide sufficient space to access the ramp and to provide flow through traffic and some of the parking at the front of the plant.

Construction of this alignment will be open cut and use interlocking steel sheet pile shoring for the full length of installation. The sheet pile shoring is required for the trench to maintain watertight conditions in the soft unstable soil.

Alignment Considerations

Soils Condition

Assuming the pipe will be connected at the same elevation as the current influent line, the new pipe will be installed with an invert varying approximately between 10 and 13 feet below grade. Because of the open cut means of construction, the effects of the Young Bay Mud as described in Alternatives A and B will be mitigated for the length of this alignment.

Headloss/Velocity

To minimize headloss at high flows (up to 80 mgd), a larger diameter pipe will be needed. The largest HDPE solid wall pipe commonly manufactured within the US at this time is a 63-inch pipe (outside diameter). Preliminary hydraulic estimates push headloss to approximately 7.5 feet at 80 mgd if a 63-inch HDPE pipe is used. Other alternative HDPE piping systems (spiral wound) are available in larger diameters up to 144-inches, e.g. Duramaxx, Spirolite, and Weholite. To reduce headloss in a single pipe below 2 feet an inside diameter of 84- inches is required for this alternative. Again, as described in Alternative B, the difficulty with this size of pipe is the low velocities that are experienced for most of the year. These low velocities cause solids to settle within the pipe and the potential for a slug of solids to enter the plant from the pipe during a high flow event.

Utilities

Of all alternatives, this alignment has the largest impacts to utilities. This alternative requires the relocation of major utilities, replacement, and protection of others in place. The following utilities encountered throughout the length of the alignment will need to be protected in place:

- 4-inch gas main
- Ferric Chloride Feedline

- 4-inch potable waterline along with the plant water booster pump facility and connecting utilities
- Plant electrical and communication
- Existing influent line
- 16 kV electrical feeder conduit

The following utilities will need to be replaced and/or relocated:

- 12-inch RCP storm drain and slit drain
- Automatic gate sensors
- 18-inch sanitary sewer force main
- Lighting electrical conduit
- Landscaping piping

Plant Access/Activities

Access to the plant will be maintained but inconvenienced. During some stages of construction, access will be blocked and traffic will be routed around the north side of the plant. The south parking area will need to be shared between construction activities and plant activities. Approximately half of the parking area within the plant in at the south parking lot will be unavailable during the construction of this alternative.

Permitting

A BCDC permit is not required for this alignment as all construction will be within SVCW's property. For construction activities, special permissions may be required to allow for equipment to access or be staged outside of plant property.

Cost

The estimated OPCC of Alternative C is $2,820,000 (+50%/-30%). Costs for Alternative C incorporate the items listed in the alternative description. Elements of the OPCC for this alternative included:

- Abandonment of the existing influent line with a newly installed bulkhead.
- Two new precast man ways.
- Six (6) inch tremie seal trench slab
- Pipe construction including: trench shoring, demolition of existing pipe, dewatering, pipe installation, and materials,
- Backfill using light weight material such as pumice.

Piping Connection

Due to the poor soils flexible connections will be used at all structures to allow for differential settlement between pipe and structure. Connecting to the existing plant can conceptually be accomplished using a reducer and flexible connections to go from the 84-inch HDPE pipe to existing 60 inch influent line.

Alignment Pros and Cons

A summary of the pros and cons has been listed in Table 2-4 below. The most significant issue with this selection is that it does not allow 80 mgd of flow while providing a low level of headloss as described above, impact to parking lot inside the plant, and the amount of utilities to be accounted for. Some of the most attractive features of this option are the costs and no BCDC permit required.

Table 2-4 New Pipe Alignment Pros & Cons

Pros	Cons
▪ Costs less than Alt B. ▪ No BCDC permit. ▪ New materials Increase reliability compared to Alt A. ▪ Slightly shorter length of pipe required than Alt. B. ▪ No major demolition needed, except where connecting to existing plant.	▪ Requires replacement, relocation, and protection of plant utilities. ▪ Bypass or relocate 18-inch SSFM ▪ Low pipe velocity. • Settling of solids within large diameter pipe may occur. ▪ Short list of large diameter HDPE pipe manufacturers. ▪ Shoring and dewatering required.

2.1.4 Alternative D: Microtunnel in New Alignment

Cost: $11,670,000; Total Length 940 feet of 84-inch HDPE Pipe and 600 feet of 90-inch steel pipe. Not recommended due to cost and poor soil conditions.

This alternative is shown in Figure 2-4 and includes installing nearly 600 feet of new HDPE pipe inside a new steel casing in a similar alignment as Alternative C. The steel casing would be installed using microtunneling.

To be added: Figure 2-4. Alternative D: Microtunnel in New Alignment

Alignment Description

As described in previous alternatives, the first segment of this alignment will be open cut from the headworks to Radio Road where a launch pit of the microtunnel boring machine (MTBM) is installed. The MTBM launch pit will penetrate approximately 100 feet below grade to more stable soil conditions. The microtunnel portion of the pipeline would be for 800 feet paralleling the south property line. From the receiving pit, the pipe will emerge from the microtunnel and via open cut, routed parallel to the existing 60-inch influent pipe and connect at the treatment plant's influent pump station.

Alignment Considerations

Soils Condition

The Young Bay Mud does not have sufficient strength to support the tunnel machine on this horizontal alignment. Due to the very deep layer of Young Bay Mud (60-70+ feet), this alternative would either need to penetrate approximately a 100 foot depth to reach a slightly more stable soil or would require ground improvement along the alignment to prevent the tunnel machine from diving down deeper into the Young Bay Mud.

Headloss/Velocity

Headlosses will be overcome using a single large diameter pipe of 84 inches. See Alternative B, or C for a further description pertaining to velocities.

Utilities

Impacted utilities are the same as those described in Alternative B, except the 18 inch SSFM may need to be relocated/redirected to allow for the microtunnel pit

Plant Access/Activities

Access and plant activities will be hindered, but not completely blocked, by the access pits and open cut segments of the pipe. Plant access and activities will need to be rerouted around these obstacles along the north side of the plant.

Permitting

No permits required for this alignment as all activities will be within SVCW's property.

Cost

The estimated cost of Alternative D is $11,670,000 (+50%/-30%). Costs for this alternative are reflective of a deep shaft microtunneling project as outlined in the alternative description above. The drop shafts are assumed to be assumed to be roughly 30 feet in diameter and 100 feet in depth. Cost for shafting of this magnitude were received from one of CDM Smith's experts in tunneling. It was recommended that shafting cost of the given size ranged from $10,000-$15,000 per vertical foot of construction. It is also assumed that three large horsepower pumps will be needed to pump flow from the invert of the second shaft up to the existing screens. Electrical and instrumental allowances were added to this alternative for the powering and operation of the pumps.

Piping Connection

Piping connections are the same as those described in Alternative C.

Alignment Pros and Cons

A summary of the pros and cons are listed below in Table 2-5. As discussed and shown below the cons for the Alternative D far out way the benefits with cost, soils, and constructability concerns being the highest concerns.

Table 2-5 Microtunnel Pros & Cons

Pros	Cons

▪ Parking inside the plant is undisturbed. ▪ No BCDC permit required	▪ Most expensive ▪ Deep construction ▪ Launch pit in the middle of the road interrupting traffic ▪ Access to pipe is very difficult ▪ High risk construction ▪ Some utilities impacted

2.1.5 Alternative E: CIPP + New Alignment

Cost: $3,740,000; Total Length 225 feet of 63-inch HDPE Pipe, 900 feet of 66-inch HDPE Pipe, 575 feet CIPP of 54-inch RCP, and 175 feet CIPP of 60-inch RCP.

Alignment Description

Alternative E is a hybrid alternative that combines Alternatives A and C and is shown in Figure 2-5 below. Rather than a large single pipe, two smaller diameter pipes would carry the high flows. Two pipes will leave the head works from a diversion box. One pipe follows the alignment described in Alternative A, and the other follows that of Alternative C. Alternative E has the combined impacts, costs, and considerations of both alternatives, but also reduces others (e.g. eliminates bypass pumping, reduces headloss, and alleviates solid settling in the pipe). By providing a dual piping system, the headloss at high flows can be resolved. During dry weather flows, all flow would pass through a single pipe alignment. By having dry weather flows go through one pipe, velocities are high enough to prevent settling of solids. Once headloss becomes too high due to increased flow, the second pipe is opened. When flow decreases to a point where only one pipe is needed, flow will be restricted to one pipe. Either active or passive diversion between wet weather and dry weather flows will occur within the headworks facility and will be further developed during design. Wastewater remaining in the wet weather pipeline can be drained with small drainage pumps and, for example, can be discharged into the influent mix box. Another option to handle stagnant water from going septic in the wet weather pipe, is to treat it with bioxide calcium nitrate and leave the pipe full. This alternative also promotes redundancy. In the event one pipe is out of service due to failure, inspection, or maintenance; the second pipe may remain in service, requiring no disruption in flow (up to approximately 35 mgd) to the plant.

To be added: Figure 2-5. Alternative E: CIPP + New Alignment

Alignment Considerations

Soils Condition

See descriptions in Alternative A and C

Headloss/Velocity

Headloss will be reduced to approximately 2 feet for all flows up to 80 mgd. In order to maintain a headloss near 2 feet, a 63-inch solid wall pipe will be used for the new pipe in conjunction with the existing rehabbed influent line. The rehabbed pipe using CIPP can carry approximately 35 mgd of flow without going over 2 feet of headloss.

Utilities

See descriptions of Alternative A and C. The exception to this is influent flow bypassing may not be needed. Construction of one pipe can be completed and used to

Plant Access/Activities

See descriptions of Alternative A and C.

Permitting

A BCDC permit will be required for the CIPP work.

Cost

The estimated OPCC of Alternative E is $3,740,000 (+50%/-30%). For more information regarding cost development, see descriptions of Alternative A and C.

Piping Connection

Due to the poor soils, flexible connections will be used at all structures to allow for differential settlement between pipe and structures. Connecting to the existing plant will be accomplished using a tee and reducers with flexible connections that fit the new HDPE pipe to the rehabbed 60-inch influent line. Connecting to the existing plant will require flow control to prevent dry weather flows from entering back into the wet weather pipe when not in use. This can be accomplished using a check valve on the wet weather pipe or valving. Another approach can be to build a new, or modify the existing, influent mix box and use slide gates. How to best accomplish this connection to the existing screens will be developed further during design.

Alignment Pros and Cons

A summary of the pros and cons for Alternative E are summarized in Table 2-6 below. Alternative E is not the lowest cost option but provides increased reliability, operation,

Table 2-6 CIPP + New Alignment Pros & Cons

Pros	Cons
▪ No influent bypassing required ▪ Minimize headloss with good velocities ▪ No settling of solids in pipe ▪ No more utilities anticipated to be impacted than Alternative C. ▪ Redundancy	▪ Increased cost as compared to single pipe Alt. ▪ Increased construction time frame ▪ Bypass or relocate 18-inch SSFM ▪ Requires large diameter valves. ▪ Wet weather pipe has stagnate water after use • May require small dewatering pump system.

2.1.6 Alternative F: Parallel Pipes

Cost: $4,210,000 to 4,700,000; Total Length approximately 1900 feet of 63-inch HDPE Pipe;

Alignment Description

This alternative uses two new pipes to convey the flow. The alignment of these pipes have multiple options:

- Option F1: Alternative B and Alternative C combined, using smaller diameter pipe, shown in Figure 2-6.
- Option F2: Two pipes located along the existing influent line easement outside of SVCW property, shown in Figure 2-7.
- Option F3: Two pipes located within SVCW property boundaries, shown in Figure 2-8.

By providing a dual piping system, the headloss at high flows can be resolved to keep the headworks at a lower elevation under a gravity flow scenario. During dry weather flows, all flow will pass through a single pipe. By having dry weather flows flowing through one pipe, velocities are high enough to prevent settling of solids. Once headloss becomes too high due to increased velocity or flow, the second pipe is opened. When flow decreases to a point where only one pipe is needed, flow will be restricted to a single pipe. Either passive or active diversion between wet weather and dry weather flows can occur within the headworks facility and will be developed further during design. These alternative also promote redundancy. In the event one pipe is out of service due to failure or maintenance, the second pipe may remain in service, requiring no disruption in flow to the plant.

The stagnant wastewater in the second pipe during dry weather flows remains of concern. Design can accommodate a drainage line connected to a drainage pump to keep the unused line empty once no longer conveying peak wet weather flows. Other options to prevent septic solids buildup will be reviewed further during the design phase such as dosing the wet weather pipeline with bioxide calcium nitrate while leaving the pipe full.

For each parallel option, pipe size was estimated to be 63 inches with each pipe conveying roughly 40+/- mgd each. Pipe size was determined assuming a headloss near 2 feet. It should be noted that pipe size can be adjusted for a smaller diameter dry weather pipe, combined with a larger diameter wet weather pipe. Pipe sizing will be further developed during design.

Option F1

This alignment combines Alternatives B and C but with smaller diameter pipes. To convey 80 mgd of flow, two 63-inch HDPE pipe are needed. Two pipes are routed from the headworks with one pipe following the alignment presented in Alternative C, being routed through the main access road into the plant. The second pipe follows the alignment presented in Alternative B, being routed within the existing influent line easement. The two pipes will combine into a single pipe prior to connecting to the existing treatment plant. As smaller pipes are used, there is slightly more room for access, but does not change any impacts to utilities as described in Alternative C.

To be added: Figure 2-6. Option F1: Parallel Pipes

Option F2

This alignment follows Alternative B, but requires a larger trench of approximately 20 feet wide to fit the two 63-inch HDPE pipe. This option is not anticipated to have any more impacts on utilities or access as already described in Alternative B. Because this option requires removing the existing influent line, a bypass will be required. The advantage gained is the minimized impacts to plant access and parking and fewer utilities to handle as compared to Alternative B, F1 and F3.

To be added: Figure 2-7. Option F2: Parallel Pipes

Option F3

This alignment follows Alternative C, but requires a larger trench of approximately 20 feet wide to fit the two 63-inch HDPE pipe. This option is not anticipated to have any more impacts on utilities or access as already described in Alternative C. The reduced distance to the access ramp and the trench will remain at 15 feet to provide sufficient access to the ramp.

To be added: Figure 2-8. Option F3: Parallel Pipes

Alignment Considerations

Soils Condition

See descriptions of Alternatives B and C

Headloss/Velocity

Headloss will be reduced to approximately 2 feet for all flows up to 80 mgd. In order to maintain a headloss of nearly 2 feet, two 63-inch HDPE pipe will be used to convey the flow. Either pipe can carry approximately 40 mgd of flow without going over 2 feet of headloss.

Utilities

Option F1: See utilities impacted under Alternative B and C.

Option F2: See utilities impacted as described in Alternative B.

Option F3: See utilities impacted as described in Alternative C.

Plant Access/Activities

Option F1: See utilities impacted under Alternative B and C.

Option F2: See utilities impacted as described in Alternative B.

Option F3: See utilities impacted as described in Alternative C.

Permitting

Option F1: See utilities impacted under Alternative B and C.

Option F2: See utilities impacted as described in Alternative B.

Option F3: See utilities impacted as described in Alternative C.

Cost

Option F1: The estimated cost of Alternative F1 is $4,210,000 (+50%/-30%). For more information regarding cost development, see cost descriptions of Alternatives B and C.

Option F2: The estimated cost of Alternative F2 is $4,700,000 (+50%/-30%). For more information regarding cost development, see cost description of Alternative B.

Option F3: The estimated cost of Alternative F1 is $4,430,000 (+50%/-30%). For more information regarding cost development, see cost description of Alternative C.

Piping Connection

For all F alternatives, due to the poor soils, flexible connections will be used at all structures to allow for differential settlement between pipe and structures. Connecting to the existing plant will be accomplished using a tee and reducers with flexible connections that fit the new HDPE pipe to the existing 60-inch influent line outside of the ILS. Connecting to the existing plant will require flow control to prevent dry weather flows from entering back into the wet weather pipe when not in use. This can be accomplished using a check valve on the wet weather pipe or valving. Another approach can be to build a new, or modify the existing, influent mix box and use slide gates. How to best accomplish this connection to the existing screens will be developed further during design.

Alignment Pros and Cons

A summary of each F alternative is provided in the tables below. Most pros and cons are similar to their respective model alignments, as previously described. What is gained by the use of a parallel system is the increased reliability and performance with the tradeoff of increased costs and construction impacts. In general the F alternatives offer the most reliable and functional solutions as new materials are used, and capacity, headloss, and flow velocities are within acceptable ranges. Conversely the F alternatives, when compared to Alternative B and C alone, have the higher construction costs and impacts, due to larger foot prints. O&M costs are also anticipated to increase as the parallel system uses valves and potentially a drain pump or chemical treatment for the wet weather pipeline.

Option F1

Option F1 has nearly the combined pros and cons of Alternative B and C, and have been summarized in Table 2-7-F1.

Table 2-7-F1 Parallel Pipes

Pros	Cons
▪ No influent bypassing required ▪ Minimize headloss with good velocities preventing settling of solids ▪ No settling of solids in pipe ▪ Redundancy ▪ Flexible construction schedule to allow for BCDC permit. ▪ F alternatives offer most reliable alternatives	▪ Increased cost. ▪ Increased construction time frame. ▪ Wet weather pipe has stagnate water after use. • May require small dewatering pump system. ▪ BCDC permit required. ▪ Has most impact to utilities. ▪ Demolition of existing pipeline. ▪ Requires large diameter valves. ▪ Shoring and dewatering required.

Option F2

Option F2 has similar pros and cons as Alternative B, and have been summarized in Table 2-7-F2. The main benefit gained from moving Alternative B into a parallel pipe scenario is controlling the velocities to prevent solids settling. This in turn adds cost to construction due to the larger excavation and increased materials.

Table 2-7-F2 Parallel Pipes

Pros	Cons
▪ Minimize headloss with good velocities preventing settling of solids ▪ Most utilities avoided by using existing alignment (slightly more than CIPP alt.). ▪ Minimal impact to plant activities/access. ▪ Redundancy	▪ Full plant flow bypass during 3 to 6 months of construction. ▪ Requires BCDC permit. ▪ Short list of large diameter HDPE pipe manufacturers. ▪ Shoring and dewatering required ▪ Demolition of existing pipe ▪ Requires large diameter valves.

Option F3

Option F3 has similar pros and cons as Alternative C, and have been summarized in Table 2-7-F3. The main benefit gained from moving Alternative C into a parallel pipe scenario is controlling the velocities to prevent solids settling. This in turn adds cost to construction due to the larger excavation and increased materials.

Table 2-7-F3 Parallel Pipes

Pros	Cons
▪ No BCDC permit. ▪ Slightly shorter length of pipe required than Alt. F1 and F2. ▪ Minimize headloss with good velocities preventing settling of solids ▪ No Bypassing required ▪ Redundancy ▪ No major demolition needed, except where connecting to existing plant.	▪ Requires replacement, relocation, and protection of plant utilities. ▪ Bypass or relocate 18-inch SSFM ▪ Low pipe velocity. • Settling of solids within large diameter pipe may occur. ▪ Short list of large diameter HDPE pipe manufacturers. ▪ Shoring and dewatering required. ▪ Requires large diameter valves.

Section 3

3.1 Comparison of Alternatives

The considerations described in the previous section were quantified to aid in selecting preferred alternatives. A weighted selection matrix was used to analyze the eight alternatives. Each alignment was assigned a score in eleven categories. The categories in the matrix were determined to be a concern that has decisive merit, and which each alternative can be evaluated against. An importance factor was assigned to each category to weigh it as being more or less important to SVCW and stakeholder. The following categories were utilized in the alignment ranking/selection process along with their assigned scoring and the importance factor:

1. Constructability: Given the method of construction, each alternative was assigned a score from 0 to 5, with a 0 signaling that the alternative is easy to construct, a 1 to 4 signaling that the alternative is difficult yet possible to construct and a 5 signaling that the alternative is not constructible

2. Headloss: A score of 0, 1, or 2 is assigned to each alternative, with 0 being assigned if the alternative is capable of handling 80 MGD of flow with a single pipeline, 1 being assigned if the alternative is capable of handling 80 MGD of flow with a dual pipeline arrangement, and 2 being assigned if the alternative does not provide 80 MGD capacity.

3. Number of Utilities to Relocate: The number of utilities requiring replacement or relocation due to each alternative was tallied up, with each utility resulting in a 1 point increase in score.

4. Influent Bypass Requirements: A score of 0 or 1 is assigned to each alternative, with 0 being assigned if the alternative does not require installation of an influent bypass pipeline and 1 being assigned if the alternative does.

5. Utilities to Protect in Place: The number of utilities that need to be protected in place due to each alternative was tallied up, with each utility resulting in a 1 point increase in score.

6. Plant Access Interference Level: A score of 1 to 5 is assigned to each alternative, with 1 being assigned if the alternative only interferes with plant access for a short period of time, a 2 to 4 if the alternative periodically or partially blocks access during construction, and 5 if the alternative unacceptably blocks access during construction.

7. Plant Parking Interference Level: A score of 0 or 1 is assigned to each alternative, with 0 being assigned if the alternative does not interfere with plant parking and 1 being assigned if the alternative does.

8. BCDC Permit Requirement: A score of 0 to 5 is assigned to each alternative, with 0 being assigned if there is no impact to schedule, 2 to 4 if there is a limited impact to schedule that may be mitigated, and 5 if the BCDC permit requirement impacts the construction schedule.

9. Process Impact: A score of 1 to 5 is assigned to each alternative to expresses the amount an alignment may affect the process of the plant, e.g. flow equalization, peaking flows, septic solids build up, odors, etc. 1=Minor; 2 to 4=Some Impact; 5=Unacceptable.

10. Operational Complexity: A score of 0 to 5 is assigned to each alternative to express the level of effort to operate each alternative (e.g. operation of large valves, seasonal operational changes, managing flows from dry to wet weather). A score of 0 represents that the operations of the alternative present no impact to level of effort; 2 to 4, some impacts; and 5, high impacts.

11. Cost: A score of 1, 3, or 5 is applied to all alternatives based on planning level cost estimates. A score of 1 is assigned to the alternative with the lowest cost, 3 is assigned to all alternatives with a mid-range cost, and 5 is assigned to the alternative with the highest cost.

A factor of importance of 5, 10, or 15 is assigned for each of the 11 categories with less important categories receiving a score of 5, more important categories receiving a score of 10, and the most important categories receiving a score of 15. Then, the individual scores in each of the 11 categories are multiplied by its respective weighting factor, or factor of importance. The weighted score is summed across the categories to produce a total ranking score, as shown in Table 3-1. For this ranking system, as it is described, the alternatives with higher scores are considered less favorable. In addition to the ranking score each alternative was reviewed for any fatal flows to identify any alignments that are not feasible. Some of these fatal flaws are identified in the ranking as “Not Acceptable”.

Table 3-1 Alternative Alignments Scoring without ILS

Importance factor	5=Less Important to Avoid 10=Somewhat Important 15=More Important	15	15	5	5	5	10	5	1	15	15	15				
Alternative	**Description**	**Construct-ability**	**Head-loss**	**Utilities Relocate/ Replace**	**Influent Bypass**	**Utilities Protect in place**	**Plant Access**	**Plant Parking**	**BCDC permit**	**Process Impacts**	**Operational Complexity**	**Cost**	**Total Score**	**$ Amount (+50%/-30%)**	Fatal Flaw	**Fatal Flaw Description**
A	CIPP	0	2	0	1	4	1	0	5	1	0	1	130	$1,450,000	Y	Cannot meet headloss requirements.
B	Replace in Place	1	0	0	1	6	2	0	5	5	3	3	165	$3,040,000	Y	Cannot meet process requirements. Organic solids will settle under dry weather flows
C	Single Pipe in New Alignment	2	0	5	0	6	3	1	0	5	3	3	165	$2,820,000	Y	Cannot meet process requirements. Organic solids will settle under dry weather flows
D	Microtunnel in New Alignment	5	0	5	0	6	5	0	0	5	3	5	255	$11,670,000	Y	Is not constructible. Cannot meet process requirements. Organic solids will settle under dry weather flows.
E	CIPP + New Alignment	1	1	5	0	6	4	1	2	3	4	3	195	$3,740,000	N	
F1	Parallel Pipes New & Existing	2	1	5	0	6	4	1	3	3	4	3	220	$4,210,000	N	
F2	Parallel Pipes Existing Alignment	1	1	0	1	6	2	0	5	3	4	3	180	$4,700,000	N	
F3	Parallel Pipes All New Alignment	1	1	5	0	6	4	1	0	3	4	3	175	$4,430,000	N	

Constructability-Given method of construction, approximates a risk level, length of construction, and if contractor will encounter delays. 0=Ease of construction;1 to 4=Difficult construction; 5=Not Constructible	**BCDC Permit**-Indicates the alignment will require a BCDC permit and the added impact to overall construction for obtaining the permit. 0=No Impact (No Impact to Schedule); 2 to 4=Limited Impact (Can mitigate some impact to schedule); 5=High Impact (Impacts Construction Schedule)
Headloss-Indicates the pipe size will allow for proper flow and with minimal headloss. 0=Provides 80 mgd alone; 1=Provides 80 mgd with second pipe; 2=Does not provide 80 mgd capacity	**Process Impact**-Expresses considerations that may affect the process of the plant, e.g. flow equalization, septic solids build up, odors, etc. 1=Minor; 2 to 4=Some Impact; 5=Unacceptable.
Utilities Relocate or Replace-Indicates the number of utilities (major and minor) requiring replacement or relocation.	**Operational Complexity**-Expresses an increased effort to operation, e.g. operation of large valves, seasonal operational changes, managing of flows from dry to wet weather. 0=No Impact; 2 to 4=Some Impact; 5=High Impact to Operations
Influent Bypass-Alignment will require bypass of main influent line to treatment plant. 0=No 1=Yes	**Cost**-Based on Planning Level Cost Estimate. Emphasizes highest and lowest cost. Mid-range costs are all within contingency factor. 1=Lowest Cost; 3=Mid-Range Costs; 5=Highest Cost
Utilities Protect in Place-Indicates the number of utilities being crossed or disturbed but will not require replacement or relocating.	**Total Score**-Higher Scores indicate less favorable selections
Plant Access-Indicates that plant access will be interrupted either at the entrance of the plant or near the ILS. 1=Minor (Access inconvenienced for short period); 2 to 4=Limited (Access blocked periodically or partially during construction); 5=Unacceptable (Access blocked throughout construction)	**Fatal Flaw**-Review from a technical perspective if the project is not feasible, has major issues, or fails to meet the needs of the overall objective. Y=Yes; N=No
Plant Parking-Indicates that plant parking will be occupied during construction within the plant property. 1=Yes 0=No (Assumes new parking installed prior)	

A comparative summary of the alternatives is shown in Table 3-2.

Table 3-2 Comparison of Alignment Alternatives

Option Name	Description	Cost	Bypass (Y/N)	BCDC Permit (Y/N)	Alignment Ranking Score
Alternative A	Rehabilitation of Existing Pipeline	$1,450,000	N	Y	130
Alternative B	Replace Existing Influent Line in Existing Alignment	$3,040,000	Y	Y	165
Alternative C	New Pipe Alignment	$2,820,000	N	N	165
Alternative D	Microtunnel in New Alignment	$11,670,000	N	N	255
Alternative E	CIPP + New Alignment (Alternatives A and C)	$3,740,000	N	Y	195
Alternative F1	Parallel Pipes (Alternatives B and C)	$4,210,000	N	Y	220
Alternative F2	Parallel Pipes (Dual Alternative B)	$4,700,000	N	Y	180
Alternative F3	Parallel Pipes (Dual Alternative C)	$4,430,000	N	N	175

3.2 Preferred Alignment Alternatives without ILS

Of the eight presented alignment alternatives, four (4) are considered viable, Alternatives E, F1, F2, and F3; and are summarized in Table 3-3 below. The other four alignments are eliminated due to fatal flaws in constructability, operational, and/or functionality concerns. The ranking score for the preferred alignments are similar, ranging from 175 to 220. OPCCs are also similar differing within less than one million dollars ($1,000,000) of each other.

Table 3-3 Preferred Alignment Alternatives

Option Name	Description	Cost	Benefits of Alternative
Alternative E	CIPP + New Alignment (Alternatives A and C)	$3,740,000	▪ Minimal foot print. ▪ Lowest construction cost of dual pipes options ▪ Shortest construction time of dual pipe options ▪ Flexible construction schedule for BCDC permit ▪ Minimal impact to plant activities/access ▪ No influent bypassing required ▪ Redundancy
Alternative F1	Parallel Pipes (Alternatives B and C)	$4,210,000	▪ Flexible construction schedule for BCDC permit ▪ Increased reliability over Alternative E ▪ Minimal impact to plant activities/access ▪ No influent bypassing required ▪ Redundancy
Alternative F2	Parallel Pipes (Dual Alternative B)	$4,700,000	▪ Most utilities avoided by using existing alignment ▪ Least impact to plant activities/access of dual pipe options ▪ Increased reliability over Alternative E ▪ Redundancy
Alternative F3	Parallel Pipes (Dual Alternative C)	$4,430,000	▪ No BCDC permit ▪ Increased reliability over Alternative E ▪ No influent bypassing required (by constructing one pipeline at a time) ▪ Redundancy

Section 4

4.1 Recommendation

The alignment recommendation is based on the evaluation presented in this report and feedback from SVCW based on the January 27, 2016 Alignment Alternatives Presentation to SVCW's operations, maintenance, and engineering personnel. Of the preferred alternatives identified in Table 3-3, alternative F3 has been selected because of its

- Low ranking score
- Medium cost
- Does not require a BCDC permit
- Provides increased reliability over Alternative E,
- Does not require influent bypassing,
- Provides redundancy.

4.1.1 Alternative F3:

Cost: $4,430,000; Total Length 1800 feet of 63-inch HDPE Pipe in parallel configuration.

Alternative F3 is parallel pipes running in a joint trench from the proposed headworks, through the southern portion of the treatment plant's property, and connects at or within the plant's existing ILS. The advantages of this configuration allows for minimizing headloss while providing adequate flow velocities throughout the range of flows experienced. By keeping velocities higher the occurrence of solids settling within the pipe(s) is reduced. It is estimated dual 63-inch pipes or some combination of a smaller and larger diameter pipe can be used to provide proper flows and velocities. Also in the event of maintenance the 2nd pipe can be used to allow for access to the other pipe without stopping flow to the plant.

The alignment reduces permitting requirements by having the pipe remain on SVCW property, with some impact to access and parking along the alignment. By constructing through the south side of the plant, the existing influent pipeline remains in service removing the need for costly bypassing. Additionally, since the existing influent pipeline remains in service, restrictions to construction schedules are more flexible to allow for early installation and reduced site congestion to other plant projects, like the proposed headworks.

Alternative F3 runs through the treatment plant, therefore extra consideration will have to be given to utilities as compared to some alignments. Existing utilities will require field location and as-built drawings review to reduce the risk of impacts to the plant and construction personnel safety.

4.1.1.1 Alternative F3 Pipe Sizing

Alternative F3 identifies two 63-inch pipes to provide 80 mgd of flow, with approximately 40 mgd per pipe. A 63 inch pipe will provide good flow velocities (greater than 2 feet per sec) down to approximately 24.5 mgd and acceptable velocities (greater than 1 foot per sec) down to 12 mgd. Flows near 40 mgd and greater will need to be split between the two pipes during wet weather events. Further consideration during design should be given to using different sizes combination of pipe.

For example the following pipe sizes were estimated for the diameter when considering velocities, head loss , and using the current design flow assumptions (dated Nov 13 2015); with a Peak Dry Weather Flow of 23 MGD, a Daily Dry Weather Low Flow of 2.5 MGD, and a Peak Wet Weather Flow of 80 MGD into the plant.

Small Pipe- 44 Inch I.D. (48 OD) will move 22.2 mgd with a head loss of 2 feet. If 23 mgd is desired a very slight increase (perhaps less than 6 inches) of the headworks effluent channel would provide the 23 mgd. The pipe will experience a low flow (2.5 mgd) velocity of 0.36 fps. At 6.9 mgd, the velocity will remain above 1 fps. At flows below 6.9 mgd (velocities below 1 fps) some solids may temporarily settle out until flows rise once again over 6.9 mgd.

Large Pipe- A 72 inch I.D. pipe in conjunction with the 44 inch will be needed to move the total wet weather flow of 80 mgd.

Of note, if the proposed gravity tunnel is used for flow equalization and plant flows are regularly equalized between 10 and 15 MGD, estimated pipeline velocities will be between about 1.5 and about 2.4 fps for the 44 inch diameter pipe. Final pipe sizes should be selected based off of predicted future maximum flow and equalized flow, with consideration for at what point and how often the larger wet weather pipe is to be used.

4.1.2 Project Cost

Alternative F3 has a preliminary cost estimate of $4,430,000 and is less costly than most all of the other alternatives. Project cost includes for the construction of

- Two large diameter HDPE pipe, paralleled in a joint trench,
- Sheet pile shoring
- Concrete trench bottom and light weight backfill.
- Manway access at two locations.
- And pile supports at structures.

The OPCC provided for this alternative, as well as all others within the report, were developed as directed by SVCW and are reflective of a -30%/+50% accuracy. Furthermore, the OPCCs presented in the report do not include construction contingency, escalation, or engineering design fees or services during construction. As design progresses and further detail is added, project costs should be updated for better planning and budgeting.

4.1.3 Conclusion

Design should move forward with the concept of having parallel pipes going through SVCW's property while trying to reduce impacts to plant access, existing utilities, and staff parking. Pipes' sizes, project cost, final alignment, and final connection approach will be developed further during design. Pipes should be sized to convey dry weather flow through one smaller diameter pipe and the remaining wet weather flow through a second larger diameter pipe. The pipe system needs to be designed against the high ground water and highly compressive clay soils found at the site. A review of record drawings and field locating by potholing or other means should be part of the design process to reduce potential costly changes during construction. The means of connecting to the existing plant to avoid using actuated valves, along with timing the decommissioning of the ILS, will require careful review and coordination. Lastly further discussion should occur for the relocating of existing utilities, such as those currently hanging on SVCW's property fence line; as the construction for the new pipe can accommodate these needs.

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Attachment B

Dual Pipe Sizes for Influent Connector Pipe TM

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Technical Memorandum

To: *Kim Hacket, SVCW*

From: *Bob Allen & Dane Whitmer, CDM Smith*

Date: *14 March 2016*

Subject: *Dual Pipe Sizes for Influent Connector Pipe*

During the project development phase of the Influent Connector Pipe (ICP) CDM Smith presented on January 27, 2016 various alignments to Silicon Valley Clean Water (SVCW) for consideration. The conveyance systems presented are designed to convey flow up to 80 million gallons per day (mgd). To convey the flows and control velocity CDM Smith showed 3 alignments, among others, that use two parallel pipes 66 inch in size. The purpose of two pipes in parallel is not only to convey the flow of 80 mgd but to provide the needed velocity to prevent solids from settling in the pipe. A large single pipe will have slow moving water allowing solids to settle until a large event flushes the pipe potentially resulting in a large load of septic solids going into the plant. Additionally it is highly beneficial to keeping headloss to a minimum to prevent a higher headworks and in turn more power use for pumping. As presented using 66 inch HDPE pipes a preliminary estimate puts headloss through just the ICP at approximately 1.5 feet.

After the presentation Kim Hackett with SVCW asked if the pipes could be different sizes rather than two pipe of the same size? Using the current design flow assumptions (dated Nov 13 2015) of Peak Dry Weather Flow of 23 MGD, Daily Dry Weather Low Flow of 2.5 MGD and a PWWF of 80 MGD into the plant, the following is estimates were found:

Small Pipe- 44 Inch I.D. (48 OD) will move 22.2 mgd at 2 feet Head Loss. If SVCW desires 23 mgd a very slight increase (perhaps less than 6 inches) of the headworks effluent channel would get us to 23 mgd. The pipe will experience a low flow (2.5 mgd) velocity of 0.36 fps. At 6.9 mgd velocity will remain above 1 fps. At flows below 6.9 mgd (velocities below 1 fps) some solids may temporarily settle out until flows rise once again to / over 6.9 mgd - 1 fps.

Large Pipe- A 72 inch I.D. pipe in conjunction with the 44 inch will be needed to move the total wet weather flow of 80 mgd.

If the tunnel is used for flow equalization and plant flows are regularly equalized between 10 and 15 MGD velocities will be between ~1.5 and ~2.4 fps respectively.

If SVCW has further questions please call or email.

cc: Bob Donaldson, Collaborative Strategies Consulting

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Attachment C

Early Startup of Headworks Facility TM

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Technical Memorandum

To: *Bill Bryan, SVCW*

From: *Jan Davel*

Prepared By: *Dane Whitmer, CDM Smith*
Bill Schilling, CDM Smith

Date: *December 13, 2016*

Subject: *Headworks Facility Project - Early Startup of Headworks Facility*

1.0 Introduction

Silicon Valley Clean Water (SVCW) is implementing a Capital Improvement Program (CIP) to improve the reliability of their conveyance system and wastewater treatment plant (WWTP). The CIP includes rehabilitation and repurposing of several collection system pump stations and installation of the following facilities:

- Gravity Pipeline to replace the existing 54-inch forcemain that conveys wastewater to the treatment plant
- Receiving Lift Station (RLS) located on the treatment plant site at the end of the new Gravity Pipeline
- Headworks Facility to remove screenings and grit from influent wastewater
- Influent Connector Pipes (ICP) to convey flow from the Headworks Facility to the primary clarifiers
- Odor control facilities to treat foul air venting from the gravity tunnel, RLS and Headworks Facility, referred to as the Front of Plant (FoP) Odor Control Facilities

SVCW is evaluating the feasibility of constructing, testing and accepting the Headworks Facility approximately 18 months before the other facilities listed above. The purpose of this memo is to summarize the conceptual approach for an early startup of the Headworks Facility and to discuss the advantages challenges and costs of the early startup.

2.0 Existing Conditions

Figures 1 and 2 below, show the current configuration of the influent conveyance and preliminary treatment facilities at the SVCW WWTP. The influent conveyance and preliminary treatment facilities consist of a 54-inch reinforced concrete force main, an Influent Lift Station (ILS), an Influent Mix Box, and a Screen Facility. The Influent Mix Box is located at the outlet of the 54-inch force main and the suction pipes for the ILS pipe are connected to the 54-inch force main, just upstream of the Influent Mix Box. These facilities are also shown in Figure 5 at the end of this TM.

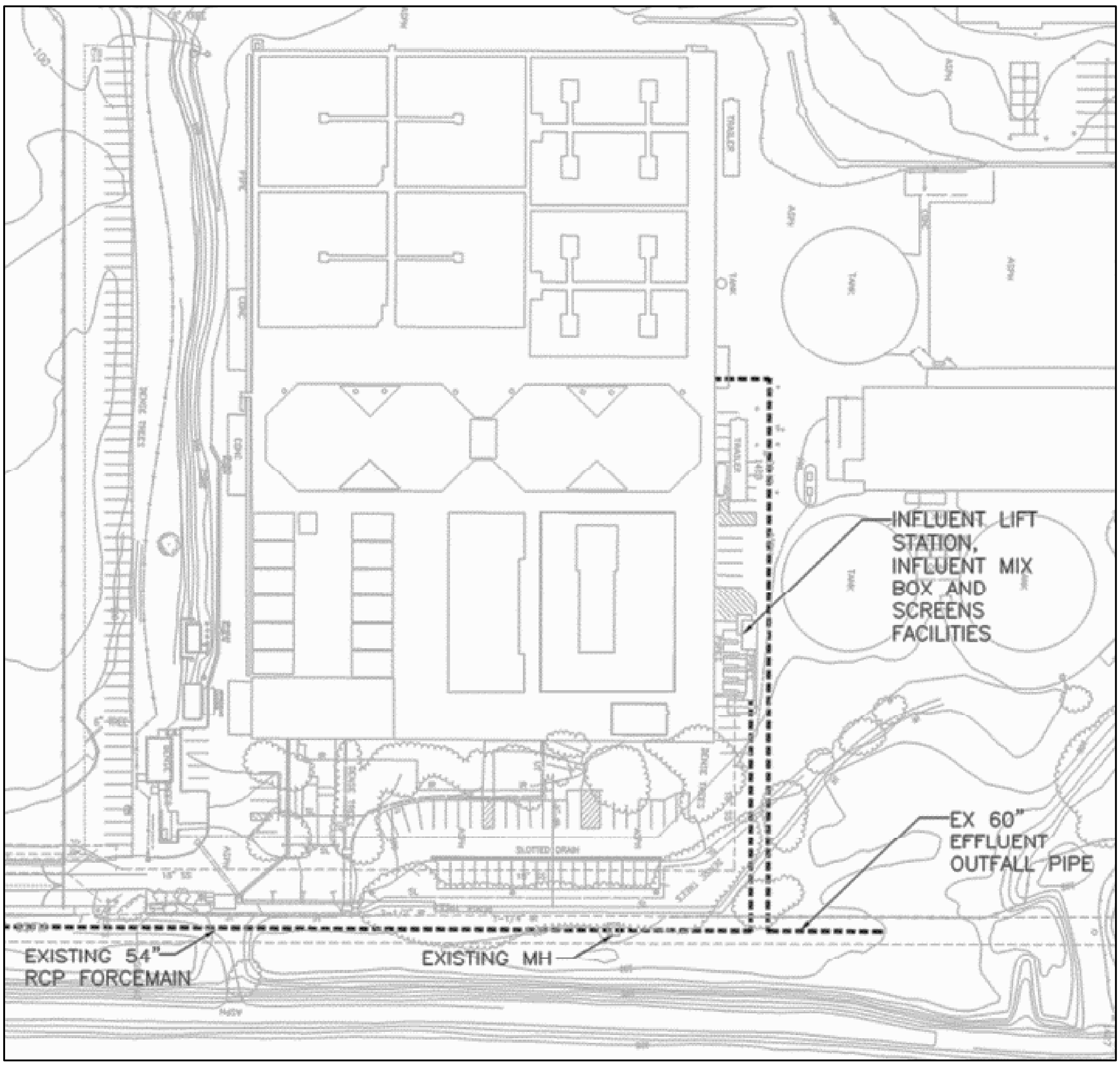

Figure 1
Existing SVCW Influent Conveyance Facilities Site Plan

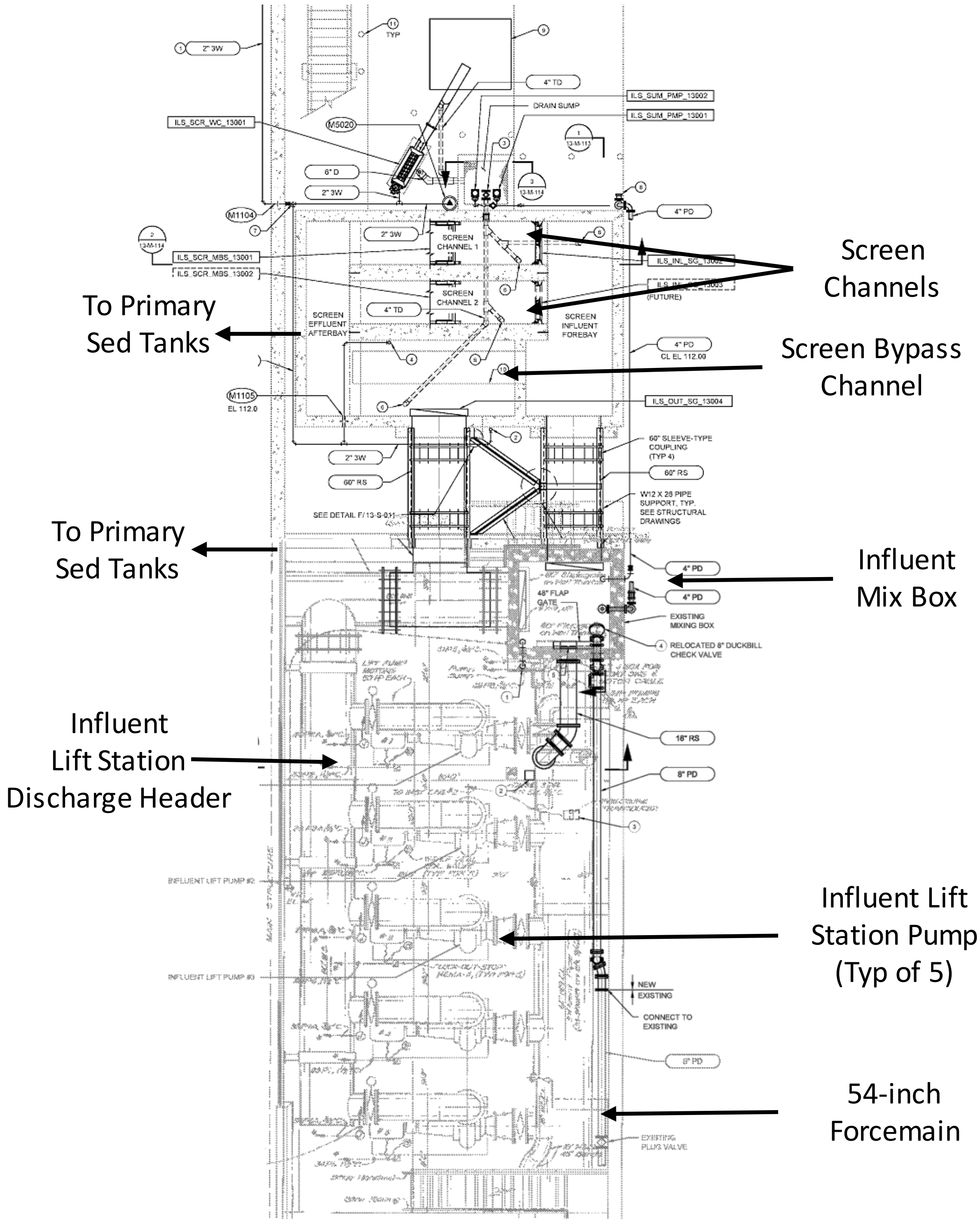

Figure 2
Existing SVCW Influent Conveyance Facilities Mechanical Plan

Under dry weather conditions, raw sewage is pumped through the existing 54-inch force main, past the suction pipes for the ILS pumps, which are normally off, directly to the existing Influent Mix Box. The Influent Mix Box then directs flow to either the Screen Facility or the Primary Settling Tanks. Flow is normally sent to the Screen Facility, but can be diverted to the Primary Settling Tanks when the Screen Facility needs to be shut down for maintenance, high flow wet weather events or other reasons.

Under wet weather conditions, the ILS pumps are started, causing a knuckle valve (flap gate) to be drawn closed inside the Influent Mix Box. Under these conditions, the ILS pumps withdraw sewage from the 54-inch force main and discharge it directly to the Primary Settling Tanks. The influent conveyance and preliminary treatment facilities are operated in this manner during wet weather conditions to reduce the pressure in the existing 54-inch force main. The ILS pumps are manually started and typically turned to protect the influent forcemain when the influent flow causes pressures in the existing forcemain to rise and typically are used to maintain influent pressures in the existing forcemain below 16 psig at the Redwood City Pump Station.

3.0 Proposed Improvements

As discussed in Section 1.0, SVCW requires several improvements to their influent conveyance and preliminary treatment facilities. Figure 3, below, shows the conceptual layout of these facilities including the RLS, Headworks Facility, FoP Odor Control Facility, and the ICP. After the facilities shown in Figure 3 are constructed, raw sewage will be conveyed through the Gravity Pipeline to the RLS, which will pump it up to the new Headworks Facility. The raw sewage will flow through the Headworks and the ICP to the existing WWTP. The existing 54-inch forcemain will no longer be needed and it will be abandoned in place. The proposed facilities are also shown in Figure 6 at the end of this TM.

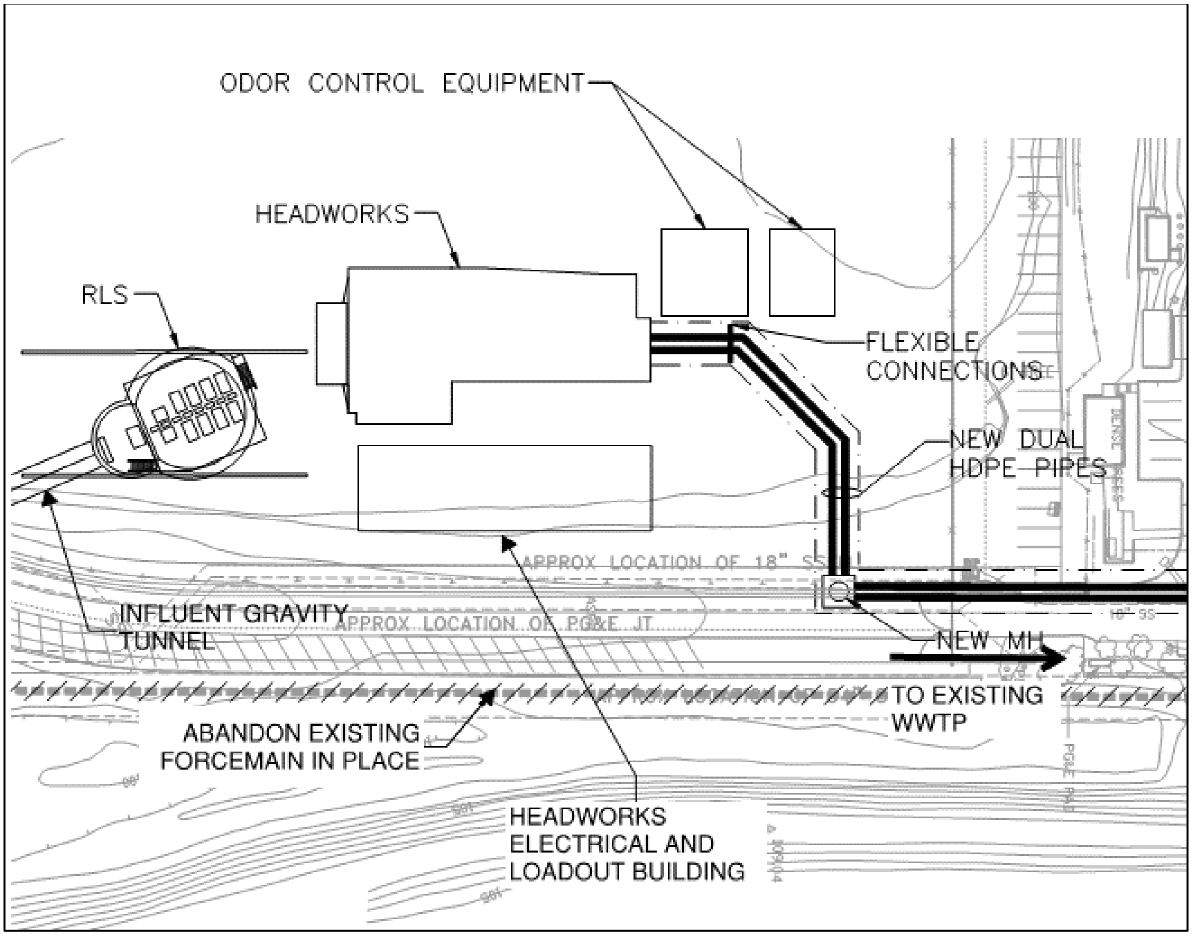

Figure 3
SVCW Proposed Conveyance System and Preliminary Treatment Improvements

4.0 Early Connection of Headworks

SVCW is considering constructing the Headworks Facility before construction of the Gravity Pipeline, RLS, and ICP is complete. This would allow SVCW to realize the benefits of improved screenings and grit removal much earlier than if construction of the Headworks Facility were delayed until after the Gravity Pipeline, RLS, and ICP are constructed. According to the latest CIP schedule, constructing the Headworks and FoP Odor Control Facilities prior to completing construction of the Gravity Pipeline, RLS, and ICP would allow the Headworks and FoP Odor Control Facilities to be constructed 18 months earlier.

Figure 4, below, shows a conceptual layout of the influent conveyance and preliminary treatment facilities under the scenario where the Headworks Facility is constructed and started up before the Gravity Pipeline, RLS, and ICP. The layout is also shown in Figure 7 at the end of this TM. The conceptual layout shown in Figure 4 and 7 is discussed in detail in Section 4.1. The capital costs and operational impacts associated with starting up the Headworks early are discussed in Sections 4.2 and 4.3, respectively.

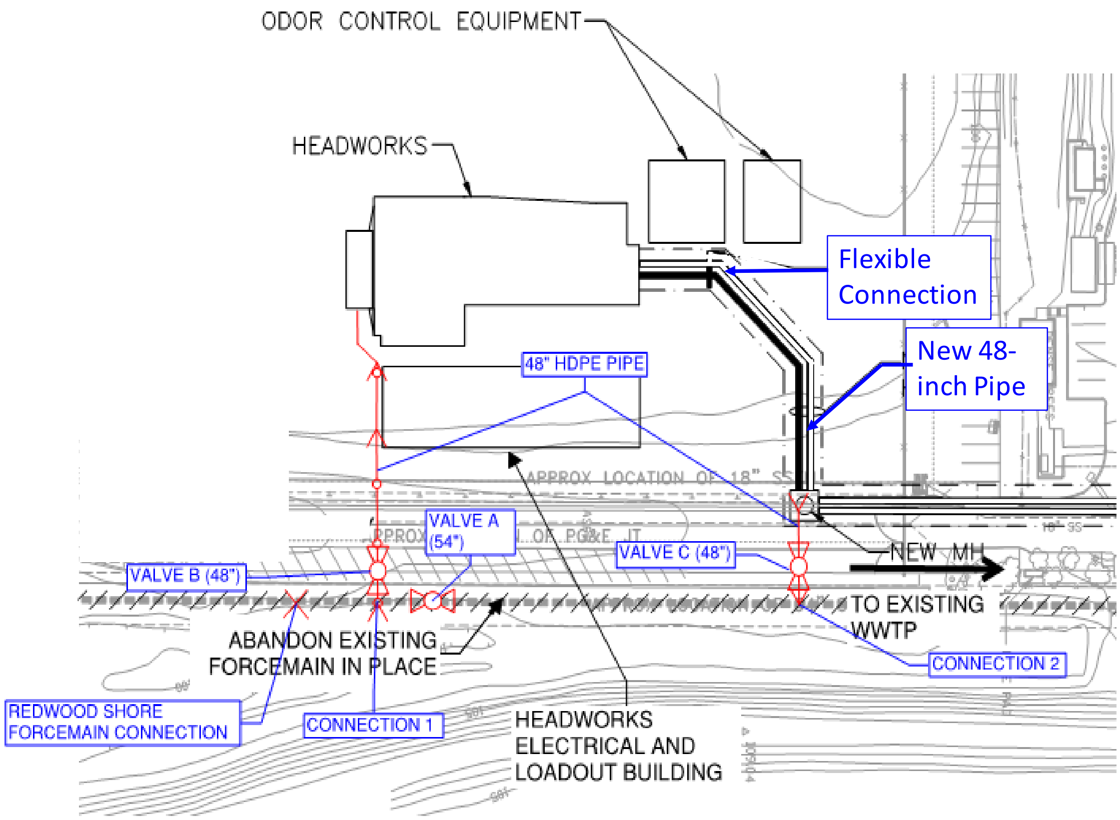

Figure 4
Conceptual Layout of Early Startup of Headworks and FoP Odor Control Facilities

4.1 Conceptual Layout

The conceptual layout shown in Figure 4 includes the following facilities:

- The proposed Headworks Facility and FoP Odor Control Facilities.
- A portion of one of the ICP between the Headworks Facility and a manhole located near the existing entrance gate to the plant.
- New piping to connect the 18-inch Redwood Shores forcemain to the existing 54-inch forcemain.
- A new 48-inch HDPE pipe to convey raw sewage from the existing 54-inch forcemain at Connection Point 1 to the influent channel of the Headworks Facility.

- A new 48-inch HDPE pipe to convey screened and de-gritted sewage from the manhole at the end of the ICP back into the existing 54-inch forcemain.

- Connection Point 1 – This connection point includes a new 54" x 54" x 48" tee, a 54-inch valve on the existing 54-inch forcemain (Valve A), and a new 48-inch valve on the new 48-inch pipe (Valve B). The new valves and tee will need to be pile-supported.

- Connection Point 2 – This connection point includes a new 54" x 54" x 48" tee, and a new 48-inch valve on the new 48-inch pipe (Valve C). The new valve and tee will need to be on a pile-supported concrete pad.

Under the configuration shown in Figure 4, the Headworks Facility would operate as follows:

- During dry weather conditions, raw sewage from the existing 54-inch forcemain will be diverted to the new Headworks Facility for preliminary treatment. Effluent from the Headworks will be sent back into the 54-inch forcemain using a portion of the ICP, where it will be conveyed to the Influent Mix Box. This will be accomplished by closing Valves A and opening Valves B and C.

- During wet weather conditions, raw sewage will not be diverted to the Headworks Facility. Since the Headworks Facility is at a higher elevation than the Influent Mix Box, sending wet weather flows to the Headworks Facility during interim operation would increase the pressure in the existing 54-inch force main most likely beyond its pressure rating. Therefore, wet weather flows will be conveyed through the existing 54-inch forcemain directly to the Influent Mix Box, bypassing the Headworks Facility. Under this scenario, operation of the influent conveyance and preliminary treatment facilities will match the existing operations. This will be accomplished by opening Valves A and closing Valves B and C.

Consideration was given to using the full length of the ICP to convey effluent from the Headworks Facility to the Influent Mix Box, rather than utilizing a portion of the existing 54-inch forcemain. This idea was eliminated from further consideration because it would require significant piping modifications at the Influent Mix Box and would require installation of several pieces of pipe and valves that would become obsolete after the Gravity Pipeline and RLS were constructed.

4.2 Capital Costs

The facilities shown in red in Figure 4 are only needed during the Headworks early start-up and operation period prior to construction of the Gravity Pipeline and the RLS. These facilities are referred to as Interim Facilities, and include the new 48-inch HDPE pipes and the fittings and valves required at Connection Point 1 and Connection Point 2. The other facilities shown in Figure 4 will remain functional after construction of the Gravity Pipeline and RLS.

The Level 5 Opinion of Probable Construction Cost associated with the interim facilities is summarized in Table 1, included at the end of this TM. As shown, the cost of constructing the interim facilities is estimated to be approximately $1,050,000 (+50%, -30%). The costs shown in

Table 1 were developed using aspects of the previously submitted OPCC for the ICP and Headworks Facility Projects. The following assumptions were made in developing the costs:

- Pipes will be constructed using open trench with sheet piling, similar to the approach for the outfall replacement project currently under construction
- Three plant shutdowns will be required to install new piping and valves

4.3 Operational Impacts and Costs

The operational impacts and costs associated with the configuration shown in Figure 3 and discussed above are as follows:

- The existing pump stations pumping flow to the plant will need to discharge to a higher elevation during dry weather operations after the new Headworks Facility is started up. This will increase the discharge pressure on the pumps and therefore increase the amount of energy required to operate the pumps. The water surface elevation in the new Headworks Facility will be approximately 117 feet during dry weather flows. The water surface elevation in the existing Influent Mix Box is approximately 109.0 feet at a dry weather flow of 12.8 mgd. Therefore, the discharge pressure on the pumps will be increased by 8 ft. The combined increased energy cost to operate the conveyance system pumps under the higher discharge pressure is approximately $25,000/year, assuming an energy cost of $0.13/kilowatt hour.
- Currently, the maximum pressure in the 54-inch force main occurs when influent flows to the plant are approximately 50 mgd and the ILS pumps are not operating. Under the configuration shown in Figure 3, the maximum pressure in the 54-inch force main will occur when peak dry weather flows (approximately 23 mgd) are being sent to the Headworks Facility. Based on a preliminary review of the hydraulic conditions under both of these scenarios, the maximum pressure in the 54-inch force main under the configuration shown in Figure 3 will be approximately 2.5 psi higher than the maximum system pressure under the current configuration.

5.0 Advantages and Disadvantages

The advantages of bringing the headworks online early include the following:

- The total project cost (construction cost plus contingency and soft costs) of the Headworks Facility and FoP Odor Control Facility is estimated to be $52,700,000 (see Headworks Facility Opinion of Probable Construction Cost TM). Constructing these facilities early eliminates approximately 18 months of escalation from the project. At annual escalation rate of 4.5%, this is a savings of approximately $3,700,000.
- Opens up space for other FoP projects that would have been occupied by the headworks construction contractor. This will significantly reduce congestion in the FoP area.

- Significantly eliminates complexity of startup by not having to go through concurrent testing of the proposed Receiving Lift Station and gravity tunnel at the same time.
- Provides 18 months of operation for plant staff to become familiar with the facility, fine tune equipment, and adjust operational procedures prior to the addition of even more complex issues of the gravity sewer storage and operation, and acceptance of the RLS.
- Provides the added process reliability of flow equalization at the plant by providing a connection to the drying beds. Currently, SVCW can only equalize a portion of the collection system flows at the Menlo Park Flow Equalization Facility. With this HW to Drying bed connection SVCW could extend its complete plant shutdown window from only several hours in the middle of the night to almost two days, which is an exceptional increase in repair and operational windows for in plant repairs.
- Provides an additional 18 months, and perhaps longer, of screening and grit removal, reducing impacts to downstream equipment and processes.

The disadvantages of bring the headworks online early include the following:

- Increases construction cost of approximately $1,050,000 (+50%/-30%)
- Increases annual system pumping cost by approximately $25,000 to pump wastewater to the elevation of the new headworks facility. (Assumes $0.13/kWh and average flow of 12.8 mgd)

In conclusion, the increased construction and O&M costs associated with early startup of the Headworks and FoP Odor Control Facilities will be offset by the savings realized by avoiding 18 months of escalation in construction costs. Therefore, there will be an overall net savings realized by bringing the Headworks Facility online early. The net savings will be approximately $2,612,500 ($3,700,000 - $1,050,000 – 1.5 yrs x $25,000/yr of increased electricity costs). This does not include the additional O&M savings associated with 18 additional months of improved screenings and grit removal.

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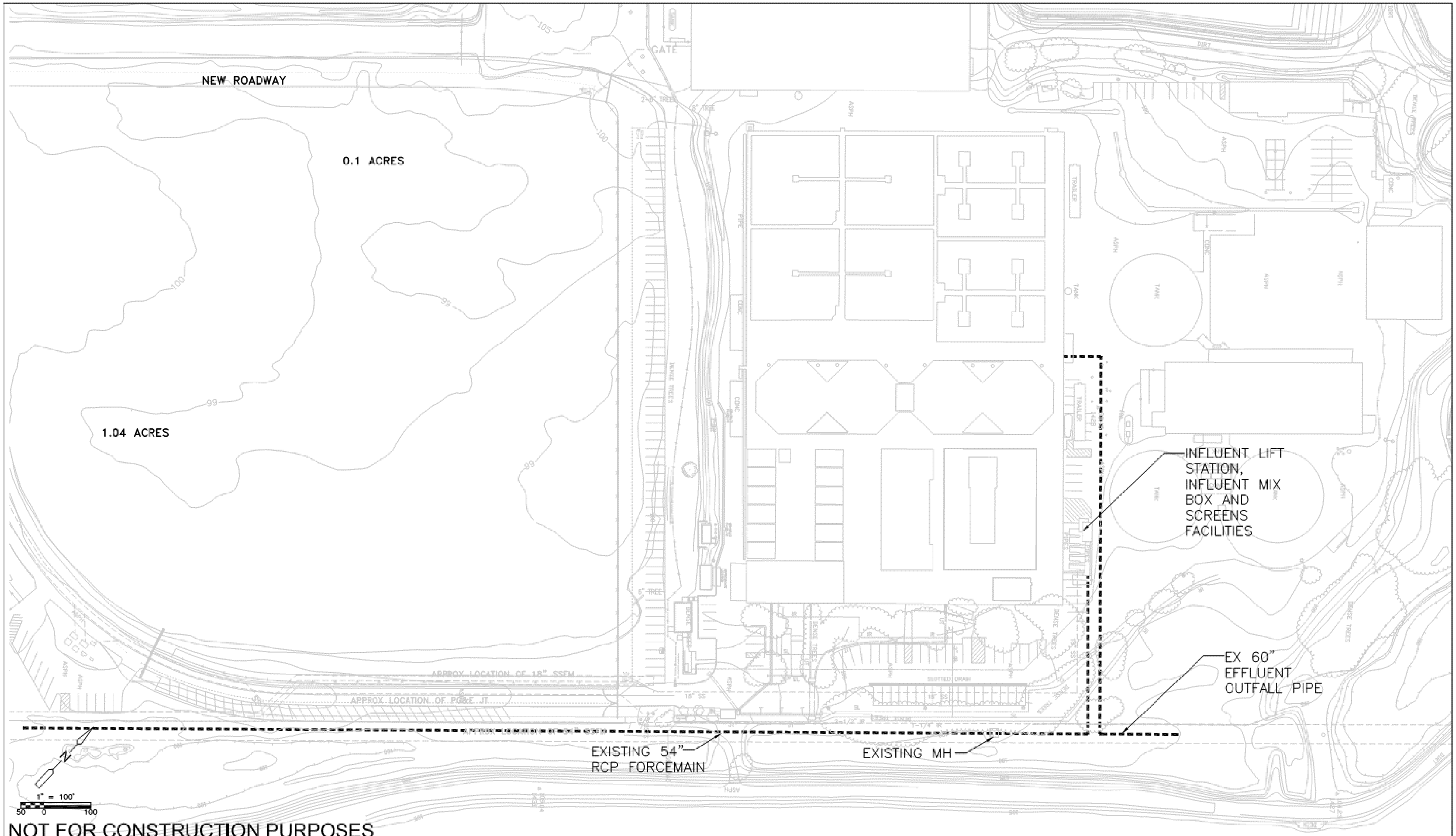

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FIGURE 5 EXISTING PLANT SITE PLAN

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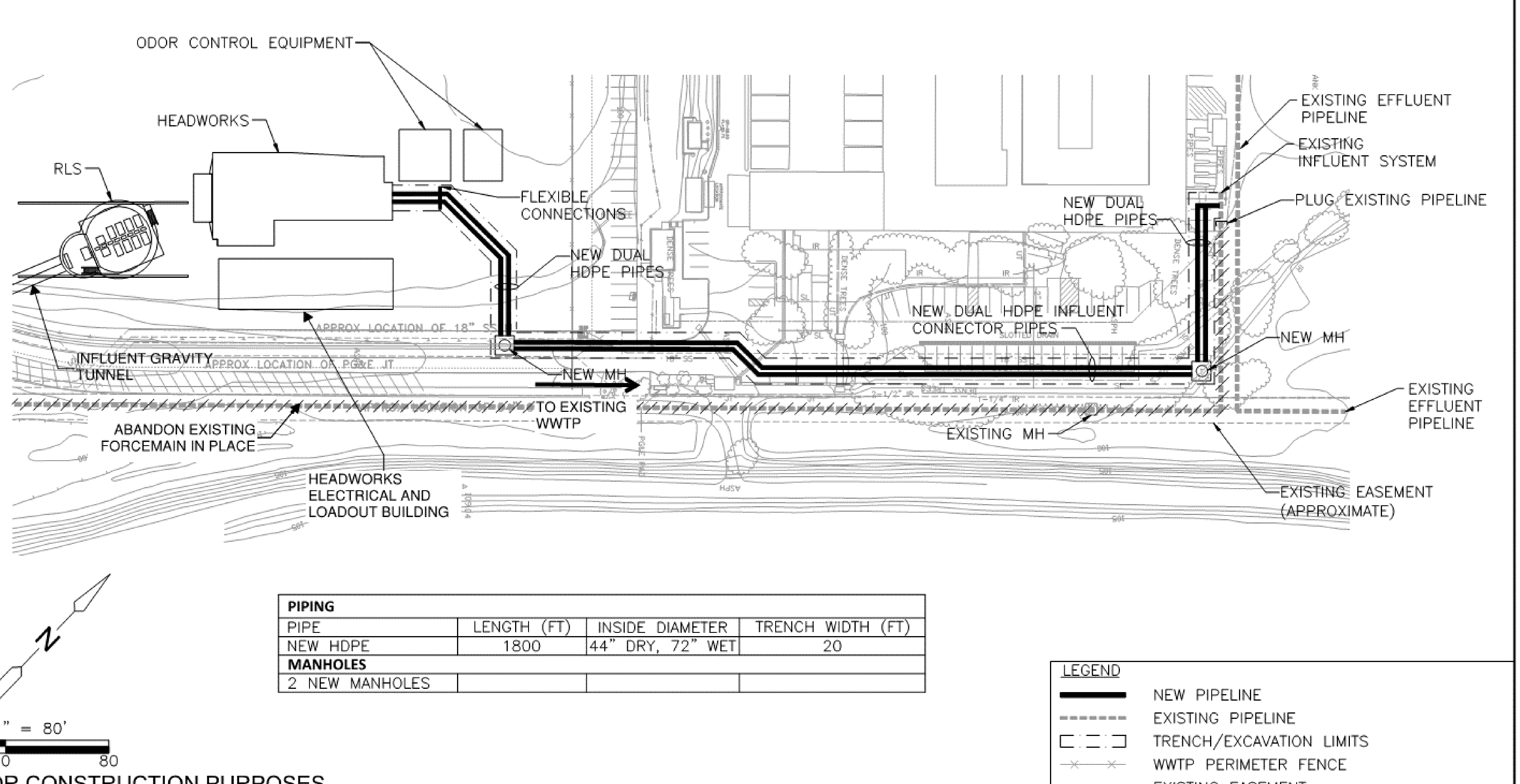

PIPING			
PIPE	LENGTH (FT)	INSIDE DIAMETER	TRENCH WIDTH (FT)
NEW HDPE	1800	44" DRY, 72" WET	20
MANHOLES			
2 NEW MANHOLES			

LEGEND
- NEW PIPELINE
- EXISTING PIPELINE
- TRENCH/EXCAVATION LIMITS
- WWTP PERIMETER FENCE
- EXISTING EASEMENT

NOT FOR CONSTRUCTION PURPOSES

CDM Smith

FIGURE 6 CONCEPTUAL FACILITY LAYOUT

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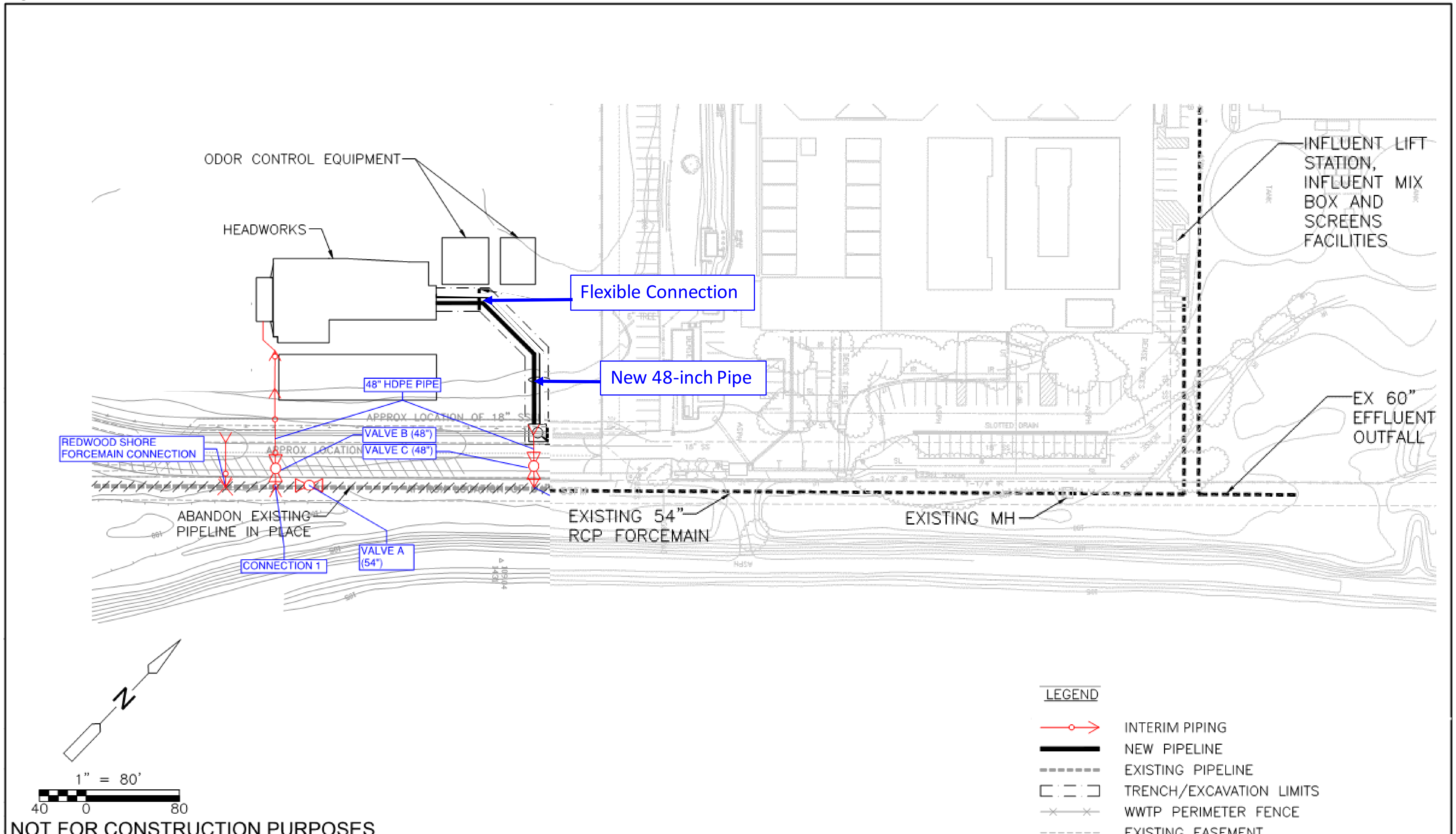

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FIGURE 7 HEADWORKS EARLY STARTUP TEMPORARY PIPING

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Table 1: SVCW Proposed Headworks Interim Piping OPCC					
ITEM	**UNITS**	**QUANTITY**	**UNIT COST**	**COST (Rounded)**	**COMMENTS**
48" HDPE Pipe and trench	lf	225	$775	$174,000	Unit cost taken from 02/24/2016 OPCC for dual 66" ICP of $775/lf. Excluding cost for restrained flexible couplings. Cost is conservative when compared to 48" HDPE pipe.
6" Tremie Seal Slab	cy	25	$175	$4,000	Unit cost taken from 02/24/2016 OPCC for dual 66" ICP.
Dresser Couplings (48"/54")	ls	6	$14,000	$84,000	Unit cost taken from 02/24/2016 OPCC for dual 66" ICP. Cost is for 60" dresser coupling and SS hardware.
Pipe Shoring	lf	225	$555	$125,000	Unit cost taken from 02/24/2016 OPCC for dual 66" ICP. Cost is for dual 66" pipes. Cost is similar.
Piles	-	16	$10,355	$166,000	Unit Cost taken from 04/04/2016 OPCC for Headworks. 135' of pile supported pipe and piles at interconnections/valves.
Valves (44"/54")	ls	2	$75,000	$150,000	Unit cost taken from 02/24/2016 OPCC for dual 66" ICP of $75,000 for 60 inch BFV. SVCW to provide 1 of 3 valves.
48" Connection to Existing 54" RCP	ea	2	$35,000	$70,000	Assumed that the connection will be made via concrete collar over new section of pipe.
	Sub Cost			**$770,000**	
Building Permits	1% of sub cost		$7,700		
Bldr's Risk Ins	1% of sub cost		$7,700		
Gen Liab Ins	1.5% of sub cost		$11,550		
GC Bonds	2% of sub cost		$15,400		
Sales Tax	9% of sub cost		$69,300		
	Total			**$882,000**	
GC General Conditions	10% of Total		$77,000		Excludes escalation and contingencies. Does not reflect any shut down costs carried by the district, night work, and engineering costs.
Contractor Total OH&P	12% of Total		$92,400		Relocation of Redwood Shores FM carried under ICP cost estimate
	Grand Total			**$1,050,000**	

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Attachment D

Draft Opinion of Probable Cost for Selected Alternative F3

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Draft Opinion of Probable Cost 2016 - Conceptual Design Level

Project name	Influent Connector Pipe CA
Estimator	S.M.
Labor rate table	CA16 San Francisco
Equipment rate table	00 15 Equip Rate BF
Notes	This is an Opinion of Probable Construction Cost only, as defined by the documents provided at the level of design indicated above. CDM has no control over the cost of labor, materials, equipment, or services furnished, over schedules, over contractor's methods of determining prices, competitive bidding (at least 3 each - both prime bidders and major subcontractors), market conditions or negotiating terms. CDM does not guarantee that this opinion will not vary from actual cost, or contractor's bids. There are not any costs provided for: Change Orders, Design Engineering, Construction Oversight, Client Costs, Finance or Funding Costs, Legal Fees, Land Acquisition or temporary/permanent Easements, Operations, or any other costs associated with this project that are not specifically part of the bidding contractor's proposed scope. Assumptions *Bypass piping assumed to be dual lines that can sustain 80mgd flow. Piping will be below grade where in roadways, otherwise it will be placeed on grade. *No cost has been added for hard dig or handling of hazardous material. *No cost is included to abandon lines other than to bulkhead in existing or new manholes.
Report format	Sorted by 'Area/95CSI Sctn/Element' 'Detail' summary Allocate addons

Spreadsheet Level	Takeoff Quantity	Labor Amount	Material Amount	Equip Amount	Sub Amount	Other Amount	Total Cost/Unit	Total Amount
008 Alternative F3: Parallel Pipes In New Alignment								
02220 Demolition								
02220.48802 Existing Surface Demo For New 66" Lines Into New Headworks								
Saw Cut Asphalt Pavement	2,500.00 lf	3,040	2,425	2,928	-	-	3.36 /lf	8,393
Demo Bituminous Pavement	11,205.00 sf	17,333	-	8,664	-	-	2.32 /sf	25,997
Load Off-site Haul	276.66 cy	209	-	285	-	-	1.78 /cy	494
Haul Demo/Off Site 18cy Rear Dump	26.00 load	2,473	-	2,780	-	-	202.03 /load	5,253
DemoTipping Fees	276.66 cy	-	-	-	20,443	-	73.89 /cy	20,443
02220.48802 Existing Surface Demo For New 66" Lines Into New Headworks	**11,205.00 sf**	**23,055**	**2,425**	**14,656**	**20,443**		**5.41 /sf**	**60,579**
02220.48804 Plug Existing 60" RCP								
Bulkhead Existing 60" INF Line	1.00 ea	1,031	850	720	-	-	2,600.66 /ea	2,601
02220.48804 Plug Existing 60" RCP	**1.00 ea**	**1,031**	**850**	**720**			**2,600.66 /ea**	**2,601**
02220 Demolition		**24,086**	**3,275**	**15,376**	**20,443**			**63,180**
02240 Dewatering								
02240.48802 Well Point Dewatering At Manhole Excavations								
Mobilize Dewatering Equipment	1.00 ea	-	-	-	1,180	-	1,180.00 /ea	1,180
Drill/Install 2' Diameter Casing w/3" Slotted PVC	80.00 vlf	-	-	-	5,098	-	63.72 /vlf	5,098
Install Collector Pipe/Fittings/Check Valve	160.00 lf	4,138	2,200	-	-	-	39.62 /lf	6,338
Install Dewatering Discharge Manifold/Header - 6"	200.00 lf	2,483	1,309	-	-	-	18.96 /lf	3,792
Dewatering Pump 1,000 gph (16 gpm/0.024 MGD)	4.00 ea	-	-	2,000	-	-	500.00 /ea	2,000
Standby Pump 1,000 gph (16 gpm/0.024 MGD)	1.00 ea	-	-	500	-	-	500.00 /ea	500
Maintain/Adjust Dewatering Pipe	1.00 mo	3,518	200	-	-	-	3,717.57 /mo	3,718
Remove Temporary & Dewatering Pipe	360.00 lf	1,862	-	-	-	-	5.17 /lf	1,862
02240.48802 Well Point Dewatering At Manhole Excavations	**2.00 ea**	**12,001**	**3,709**	**2,500**	**6,278**		**12,243.86 /ea**	**24,488**
02240.48804 Trench Dewatering For Dual 66" HDPE								
Drill and Install wells at 20' on center	90.00 ea	11,306	4,275	37,384	-	-	588.50 /ea	52,965
Install header piping	1,000.00 lf	5,173	5,000	-	-	-	10.17 /lf	10,173
Install Discharge Pipe- 10"	1,000.00 lf	19,657	5,500	-	-	-	25.16 /lf	25,157
Remove Discharge Pipe	1,000.00 lf	1,293	-	-	-	-	1.29 /lf	1,293
02240.48804 Trench Dewatering For Dual 66" HDPE	**1,800.00 lf**	**37,429**	**14,775**	**37,384**			**49.77 /lf**	**89,589**
02240 Dewatering		**49,430**	**18,484**	**39,884**	**6,278**			**114,076**
02250 Sheeting, Shoring & Bracing								
02250.48802 Steel Sheeting Trench Shoring For Dual 66" HDPE								
Install Beam and Plate	28,500.00 sf	84,314	342,000	35,158	-	-	16.19 /sf	461,471
Remove Beam and Plate	28,500.00 sf	44,967		18,751	-	-	2.24 /sf	63,718
02250.48802 Steel Sheeting Trench Shoring For Dual 66" HDPE	**950.00 lf**	**129,281**	**342,000**	**53,908**			**552.83 /lf**	**525,189**
02250.48804 Steel Sheeting Manhole Shoring								
Install Beam and Plate	1,200.00 sf	10,650	10,800	4,441	-	-	21.58 /sf	25,891
Remove Beam and Plate	1,200.00 sf	5,325		2,220	-	-	6.29 /sf	7,546
02250.48804 Steel Sheeting Manhole Shoring	**2.00 ea**	**15,975**	**10,800**	**6,661**			**16,718.31 /ea**	**33,437**
02250 Sheeting, Shoring & Bracing		**145,256**	**352,800**	**60,570**				**558,626**
02455 Driven Piles								
02455.48802 14" SQ Driven Concrete Piles @ Manhole Locations (100' In Depth)								
Layout Piles	28.00 ea	2,800	-	-	-	-	100.00 /ea	2,800
Pile Cutoff	28.00 ea	1,648	-	587	-	-	79.85 /ea	2,236
Pile Load Test	2.00 ea	21,300	-	11,081	-	-	16,190.45 /ea	32,381
Mobilize & Demobilize	1.00 ea	-	-	-	8,500	-	8,500.00 /ea	8,500
14" x 14" Prestressed Concrete Piles	2,800.00 vf	37,275	196,000	19,391	-	-	90.24 /vf	252,667
02455.48802 14" SQ Driven Concrete Piles @ Manhole Locations (100' In Depth)	**28.00 ea**	**63,024**	**196,000**	**31,059**	**8,500**		**10,663.69 /ea**	**298,583**
02455 Driven Piles		**63,024**	**196,000**	**31,059**	**8,500**		**/ea**	**298,583**
02530 Sanitary Sewerage								
02530.48802 Dual 66" HDPE From Existing Influent To New Headworks								
Pipe Alignment Survey	900.00 lf	2,554	494	-	-	-	3.39 /lf	3,048
Large Bore Pipe Excavation & Installation	1,800.00 lf	114,244	-	64,968	-	-	99.56 /lf	179,212
Large Bore Backfill Crew	1,800.00 lf	29,385	-	35,395	-	-	35.99 /lf	64,779
3/4 Stone Inside GeoTextile Wrap	3,044.00 cy	-	83,101	-	65,373	-	48.78 /cy	148,474
Pumice Stone For Backfill From 1' over pipe to 15" Below Grade	2,782.36 cy	-	60,043	-	35,426	-	34.31 /cy	95,469
Pipe Detectable/Non-Detectable Tape	1,800.00 lf	533	99	-	-	-	0.35 /lf	632
Pipe Interferences @ Trench Excavation	10.00 ea	2,670	1,000	1,153	-	-	482.35 /ea	4,824
Pipe Test	1,800.00 lf	33,111	4,050	-	-	-	20.65 /lf	37,161
Pipe Locates (Pot Hole)	10.00 ea	2,670	500	1,153	-	-	432.35 /ea	4,324
Trench Dewatering w/Sump Pump	1,800.00 lf	18,023	-	106	-	-	10.07 /lf	18,129
Geotextile Fabric	67,745.36 sf	71,699	7,245	-	-	-	1.17 /sf	78,944

Spreadsheet Level	Takeoff Quantity	Labor Amount	Material Amount	Equip Amount	Sub Amount	Other Amount	Total Cost/Unit	Total Amount
02530.48802 Dual 66" HDPE From Existing Influent To New Headworks								
Load Spoils from Stockpile	4,488.26 cy	4,649	-	5,704	-	-	2.31 /cy	10,353
Haul Spoils/Off Site	4,488.26 cy	32,448	-	37,493	-	-	15.58 /cy	69,941
60-0/0" FLG Coupling Adaptor- 150# Dresser Flex Connection	4.00 ea	6,692	38,240	-	-	-	11,232.92 /ea	44,932
Mileage & Per Diem for Tech	2.00 trip	-	-	-	1,062	600	831.00 /trip	1,662
PE Profile Wall Pipe (Thermal Weld) Pipe, 66"	1,800.00 lf	109,872	453,600	-	-		313.04 /lf	563,472
PE Profile Wall Pipe (Thermal Weld) Pipe 45 Bend, 66"	8.00 ea		46,400	-	-		5,800.00 /ea	46,400
02530.48802 Dual 66" HDPE From Existing Influent To New Headworks	**1,800.00 lf**	**428,551**	**694,772**	**145,972**	**101,861**	**600**	**762.09 /lf**	**1,371,755**
02530.48804 66" Tie In Connection to Existing Influent								
Large Bore Pipe Excavation & Installation	1.00 ea	7,140	-	4,060	-	-	11,200.74 /ea	11,201
Large Bore Backfill Crew	1.00 ea	1,837	-	2,212	-	-	4,048.70 /ea	4,049
3/4 Stone Inside GeoTextile Wrap	65.00 cy	-	1,775	-	1,396	-	48.78 /cy	3,170
Pumice Stone For Backfill From 1' over pipe to 15" Below Grade	45.00 cy	-	971	-	573	-	34.31 /cy	1,544
Pipe Locates (Pot Hole)	1.00 ea	267	50	115	-	-	432.35 /ea	432
Trench Dewatering w/Sump Pump	1.00 ea	1,413	-	42	-	-	1,454.59 /ea	1,455
60" CL 150 Motor Operated Flanged Butterfly Valve	1.00 ea		81,166	-	-	-	81,166.27 /ea	81,166
12.0' Depth- Cast Iron Valve Box (Top/Bottom/2-Extension/Lid + Base) & Appurtenances (Tag & Collar)	1.00 ea	310	448	-	-	-	758.37 /ea	758
60-0/0" FLG Coupling Adaptor- 150# Dresser Flex Connection	3.00 ea	5,019	28,680	-	-	-	11,232.92 /ea	33,699
60-0/0" 150# 316 SS Bolt Sets	2.00 ea	-	5,400	-	-	-	2,700.00 /ea	5,400
60-0/0" Full Faced Red Rubber (SBR) Gasket 1/8"	2.00 ea	-	86	-	-	-	43.25 /ea	86
Mileage & Per Diem for Tech	1.00 trip	-	-	-	531	300	831.00 /trip	831
PE Profile Wall Pipe (Thermal Weld) Pipe Tee, 60"	1.00 ea		4,580	-	-		4,580.00 /ea	4,580
PE Profile Wall Pipe (Thermal Weld) Pipe Reducer, 66"	2.00 ea		10,240	-	-		5,120.00 /ea	10,240
02530.48804 66" Tie In Connection to Existing Influent	**1.00 ea**	**15,986**	**133,396**	**6,430**	**2,500**	**300**	**158,611.75 /ea**	**158,612**
02530.48806 Remove & Relocate Existing 18" SSFM								
Pipe Excavation & Installation	800.00 lf	28,561	-	16,242	-	-	56.00 /lf	44,803
Backfill Crew	800.00 lf	4,897	-	5,899	-	-	13.50 /lf	10,797
3/4 Stone Bedding/Zone/Engineered Fill Material	206.90 cy	-	5,648	-	1,506	-	34.58 /cy	7,155
Trench Shield- 8x20	4.00 u/mo	-	-	6,976	-	-	1,744.00 /u/mo	6,976
Pipe Detectable/Non-Detectable Tape	800.00 lf	237	44	-	-	-	0.35 /lf	281
Pipe Interferences @ Trench Excavation	7.00 ea	1,869	700	807	-	-	482.35 /ea	3,376
Pipe Test	800.00 lf	4,138	640	-	-	-	5.97 /lf	4,778
Pipe Locates (Pot Hole)	7.00 ea	1,869	350	807	-	-	432.35 /ea	3,026
Trench Dewatering w/Sump Pump	800.00 lf	12,559	-	74	-	-	15.79 /lf	12,633
Load Spoils from Stockpile Cat 325 Excavator-32MT- 180hp	311.11 cy	215	-	264	-	-	1.54 /cy	478
Haul Spoils/Off Site 18cy Rear Dump 2 Load/Hour	311.11 cy	733	-	847	-	-	5.08 /cy	1,579
18-0/0" 150# 316 SS Bolt Sets	2.00 ea	-	800	-	-	-	400.00 /ea	800
18-0/0" Full Faced Red Rubber (SBR) Gasket 1/8"	2.00 ea	-	23	-	-	-	11.67 /ea	23
Mileage & Per Diem for Tech	1.00 trip	-	-	-	450	300	750.00 /trip	750
HDPE Fusion Machine & Tech 6"-18"	7.00 day	-	-	2,450	2,800	-	750.00 /day	5,250
HDPE DIPS, Butt-Fused Pipe, DR 11, 18"	800.00 lf		41,352	-	-		51.69 /lf	41,352
HDPE DIPS, Butt-Fused, 90 Bnd, DR 11, 18"	1.00 ea		1,325	-	-		1,324.65 /ea	1,325
HDPE DIPS, Butt-Fused 45 Bend, DR 11, 18"	4.00 ea		2,596	-	-		649.04 /ea	2,596
HDPE DIPS, Butt-Fused Tee, DR 11, 18"	1.00 ea		988	-	-		988.29 /ea	988
HDPE DIPS, Butt-Fused Flange Adapter, DR 11, 18"	2.00 ea		390	-	-		195.00 /ea	390
HDPE DIPS Backing Ring, PP Coated DI, 18"	2.00 ea	-	261	-	-		130.55 /ea	261
Flange Bolt-up, 18"	2.00 ea	1,379	-	-	-	-	689.45 /ea	1,379
HDPE DIPS, MJ Adapter, 18"	8.00 ea		4,680	-	-		585.00 /ea	4,680
HDPE Electrofusion Coupling, 18"	4.00 ea		4,924		-	-	1,231.00 /ea	4,924
02530.48806 Remove & Relocate Existing 18" SSFM	**800.00 lf**	**56,458**	**64,722**	**34,366**	**4,756**	**300**	**200.75 /lf**	**160,602**
02530.48808 6" Pump Line								
Pipe Excavation & Installation	1.00 ea	3,570	-	2,030	-	-	5,600.37 /ea	5,600
Bore Backfill Crew	1.00 ea	918	-	1,106	-	-	2,024.35 /ea	2,024
3/4 Stone Bedding/Zone/Engineered Fill Material	6.61 cy	-	180	-	48	-	34.58 /cy	229
Trench Shield- 8x20	4.00 u/mo	-	-	6,976	-	-	1,744.00 /u/mo	6,976
Pipe Interferences @ Trench Excavation	5.00 ea	1,335	500	577	-	-	482.35 /ea	2,412
Pipe Test	50.00 lf	259	40	-	-	-	5.97 /lf	299
Pipe Locates (Pot Hole)	5.00 ea	1,335	250	577	-	-	432.35 /ea	2,162
Trench Dewatering w/Sump Pump	50.00 lf	785	-	5	-	-	15.79 /lf	790
Load Spoils from Stockpile Cat 325 Excavator-32MT- 180hp	9.30 cy	166	-	203	-	-	39.69 /cy	369
Haul Spoils/Off Site 18cy Rear Dump 2 Load/Hour	9.30 cy	34	-	39	-	-	7.81 /cy	73
6-0/0" 150# 316 SS Bolt Sets	2.00 ea	-	100	-	-	-	50.00 /ea	100
6-0/0" Full Faced Red Rubber (SBR) Gasket 1/8"	2.00 ea	-	4	-	-	-	2.22 /ea	4
Welder with Rig	6.00 ea	569	-	270	-	125	160.66 /ea	964
6" AWWA C200 WSP CML,14Ga, (0.0747 in.)	50.00 lf		526	-	-	-	10.52 /lf	526
6" C208 Fab 90 Ell,14Ga, (0.0747 in.)	2.00 ea	-	78	-	-	-	39.00 /ea	78

Spreadsheet Level	Takeoff Quantity	Labor Amount	Material Amount	Equip Amount	Sub Amount	Other Amount	Total Cost/Unit	Total Amount
02530.48808 6" Pump Line								
Flange Class 150, 6"	2.00 ea	-	114	-	-	-	57.00 /ea	114
6" Interior/Joint Mortar Patch	6.00 ea	621	-	-	-	150	128.46 /ea	771
6" Exterior/Joint Tape Wrap	6.00 ea	186	-	-	-	144	55.04 /ea	330
Flange Bolt-up, 6"	2.00 ea	521	-	-	-	-	260.72 /ea	521
02530.48808 6" Pump Line	**50.00 lf**	**10,299**	**1,793**	**11,783**	**48**	**419**	**486.83 /lf**	**24,342**
02530.48810 20 HP Non Clog Pump								
Furnish Lubricants	1.00 gal	-	-	-	-	50	50.00 /gal	50
Unload/Protect Equip - Medium Equip	1.00 ea	264	-	69	-	-	332.76 /ea	333
Submersible Non-Clog Pump 20 HP	1.00 ea	731	21,575	168	-	-	22,474.21 /ea	22,474
Install Guide Rails	1.00 ea	163	-	-	-	-	162.77 /ea	163
Install Base Elbow	1.00 ea	163	-	-	-	-	162.77 /ea	163
Install Local Control Panel	1.00 ea	326	-	-	-	100	425.54 /ea	426
Grout Equip Base- Non Shrink, Non Matallic	5.33 cf	503	-	-	-	352	160.31 /cf	855
Test & Check	1.00 ea	651	-	-	-	-	651.09 /ea	651
Install Equipment- RT Crane 80 MT	6.00 ch	551	-	1,050	-	-	266.77 /ch	1,601
SS Anchor Bolts/Sleeve 3/4" x 8"	6.00 ea	114	-	-	-	90	34.06 /ea	204
02530.48810 20 HP Non Clog Pump	**1.00 ea**	**3,465**	**21,575**	**1,287**		**592**	**26,919.10 /ea**	**26,919**
02530 Sanitary Sewerage		**514,758**	**916,258**	**199,837**	**109,165**	**2,211**		**1,742,229**
02742 Pipeline Restoration								
02742.48802 Surface Restoration For Open Excavation Trenching								
Roadway- Bituminous Surface/Wearing Course 2.0"	1,245.00 sy	3,978	10,505	2,232	1,653	-	14.75 /sy	18,367
Roadway- Bituminous Binder/Intermediate Course 4"	1,245.00 sy	7,968	20,262	4,471	3,188	-	28.83 /sy	35,890
Roadway- Bituminous Base Course 10"	1,245.00 sy	15,926	8,404	8,937	8,264	-	33.36 /sy	41,531
Roadway- Tack Coat	1,245.00 sy	-	467	199	-	-	0.54 /sy	666
02742.48802 Surface Restoration For Open Excavation Trenching	**11,205.00 sf**	**27,872**	**39,638**	**15,839**	**13,104**		**8.61 /sf**	**96,454**
02742 Pipeline Restoration		**27,872**	**39,638**	**15,839**	**13,104**			**96,454**
02958 Bypass Systems								
02958.48802 Bypass System For 18" SSFM Relocate								
Site Pump Watch Labor	5.00 dy		-	-	13,452		2,690.40 /dy	13,452
Fuel Consumption (Avg 5 Gal Per Hour)	10.00 dy	-	-	-	2,832		283.20 /dy	2,832
Suction Hose 6" x 25'	10.00 dy	-	-		1,475	-	147.50 /dy	1,475
Bypass Setup	1.00 ea	-	-	-	4,484	-	4,484.00 /ea	4,484
Bypass Removal	1.00 ea	-	-	-	2,596	-	2,596.00 /ea	2,596
HDPE Bypass Piping	800.00 lf	-	-	-	8,000	-	10.00 /lf	8,000
6" X 10' Groove Hose	10.00 dy				101	-	10.15 /dy	101
500 Gal. Fuel Cell	10.00 dy				236	-	23.60 /dy	236
6" Sound Attenuated Trash Pump 24/7 Rate	10.00 dy			-	2,280	-	228.00 /dy	2,280
6" Sound Attenuated Trash Pump (Standby)	10.00 dy			-	1,270	-	127.00 /dy	1,270
Gate Valves	10.00 dy			-	295	-	29.50 /dy	295
Secondary Pump Containment Berm	10.00 dy			-	236	-	23.60 /dy	236
02958.48802 Bypass System For 18" SSFM Relocate	**800.00 lf**				**37,257**		**46.57 /lf**	**37,257**
02958 Bypass Systems					**37,257**			**37,257**
03000 CONCRETE								
03000.48802 Monolithic Concrete Slab Under Access Vaults								
ASTM D448 #357 Stone (2.00- No. 4)	30.00 cy	-	437	-	250	-	22.88 /cy	686
Slab-on-Grade Form Oil & Hdwre	160.00 sf	-	80	-	-	-	0.50 /sf	80
Slab-on-Grade Form Hoisting	160.00 sf	-	-	34	-	-	0.21 /sf	34
Hand Fine Grade SOG	800.00 sf	379	-	-	-	-	0.47 /sf	379
Slab-on-Grade < 12" 1 Form Use	160.00 sf	3,050	652	-	-	-	23.14 /sf	3,703
Chamfer Strip	160.00 lf	-	44	-	-	-	0.28 /lf	44
Rebar Accesories/Unload & Store	3.70 tn	106	37	25	-	-	45.34 /tn	168
SOG Rebar	3.70 tn	4,693	3,497	-	-	-	2,213.37 /tn	8,189
Pump Place Slab on Grade	30.00 cy	439	-	-	-	-	14.62 /cy	439
Trowel Finish @ SOG	800.00 sf	755	-	-	-	-	0.94 /sf	755
Water Base Non-Residual Cure	800.00 sf	122	53	-	-	-	0.22 /sf	175
SOG Concrete Pump- 92' Boom (28m)	30.00 cy	-	-	-	595	59	21.80 /cy	654
4500 psi Concrete- West Region	30.00 cy	-	4,410	-	-	-	147.00 /cy	4,410
6 Mil. Poly Vapor Barrier	800.00 sf	104	40	-	-	-	0.18 /sf	144
03000.48802 Monolithic Concrete Slab Under Access Vaults	**2.00 ea**	**9,648**	**9,249**	**59**	**845**	**59**	**9,929.60 /ea**	**19,859**
03000 CONCRETE		**9,648**	**9,249**	**59**	**845**	**59**		**19,859**
03315 Tremie Concrete Trench Slab								
03315.48802 6" Tremie Seal Slab For Dual 66" Pipe Trench								
Truck Place Trench Slab	667.00 cy	18,874	-	-	-	-	28.30 /cy	18,874

Spreadsheet Level	Takeoff Quantity	Labor Amount	Material Amount	Equip Amount	Sub Amount	Other Amount	Total Cost/Unit	Total Amount
03315.48802 6" Tremie Seal Slab For Dual 66" Pipe Trench								
Tremie Slab Concrete	667.00 cy	-	87,544	-	-	-	131.25 /cy	87,544
03315.48802 6" Tremie Seal Slab For Dual 66" Pipe Trench	**667.00 cy**	**18,874**	**87,544**				**159.55 /cy**	**106,418**
03315 Tremie Concrete Trench Slab		**18,874**	**87,544**					**106,418**
03410 Structural Precast Concrete								
03410.48802 Precast Concrete Accese Vaultes								
Manhole Excavate & Install Crew	2.00 ea	14,281	-	8,121	-	-	11,200.74 /ea	22,401
Manhole Backfill Crew	2.00 ea	1,837	-	2,212	-	-	2,024.35 /ea	4,049
3/4 Stone Structural Section	46.00 cy	-	1,256	-	988	-	48.78 /cy	2,244
Pumice Stone For Backfill	185.00 cy	-	3,992	-	2,355	-	34.31 /cy	6,348
Precast Vault 20'x20'x15	2.00 ea	8,103	100,058	9,183	-	-	58,672.25 /ea	117,344
03410.48802 Precast Concrete Accese Vaultes	**2.00 ea**	**24,220**	**105,306**	**19,516**	**3,343**		**76,193.06 /ea**	**152,386**
03410 Structural Precast Concrete		**24,220**	**105,306**	**19,516**	**3,343**			**152,386**
008 Alternative F3: Parallel Pipes In New Alignment		**877,169**	**1,728,554**	**382,141**	**198,935**	**2,269**		**3,189,069**

Estimate Totals

Description	Amount	Totals	Hours	Rate
Labor	877,169		12,046 hrs	
Material	1,728,554			
Subcontract	198,935			
Equipment	382,141		3,954 hrs	
Other	2,269			
	3,189,068	3,189,068		

Subtotal Direct Cost		3,189,068		
Building Permits(% total cost)	31,891			1.00 %
Bldr's Risk Ins (% total cost)	32,210			1.00 %
Gen Liab Ins (% total cost)	48,798			1.50 %
GC Bonds (% total cost)	66,039			2.00 %
Sales Tax	189,963			9.00 %
Subtotal Prior to OH&P	368,901	3,557,969		
GC General Conditions	318,907			10.00 %
Contractor Total OH&P	426,956			12.00 %
Subtotal with OH&P	745,863	4,303,832		
Construction Contingency				
Total Cost at:		4,303,832		
No Escalation Included				
		4,303,832		
Total		***4,303,832***		

"This Opinion of Probable Construction Cost is produced in accordance with CDM Smith's Firmwide Quality policies and best practices as described in CDM Smith's Estimating Manual Dated 01/03/12 Section 10 titled Quality Control. I hereby attest that the Cost Estimating policies and procedures were followed in preparation of the Opinion of Probable Cost"

Lead Estimator initials: S.M. *Date:*

Attachment E

Life Cycle Cost Analysis TM

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Memorandum

To: *Bob Donaldson, PM for SVCW*

From: *Bob Allen, CDM Smith*

Prepared by: *Dane Whitmer, CDM Smith*

Date: *September 1, 2016*

Subject: *Influent Connector Pipeline Project – Life Cycle Cost Analysis*

1.0 Introduction

This Technical Memorandum (TM) presents the 75-Year Life Cycle Cost (LCC) associated with the Influent Connector Pipes that will be installed as part of the Silicon Valley Clean Water (SVCW) Capital Improvement Program (CIP). The LCCs are for a 75 year period from the point of installation in 2018 to 2093. The LCCs were prepared in accordance with the Life Cycle Cost Analysis Guidelines TM, dated July 13, 2016. This work is being completed as part of the SVCW Influent Connector Pipeline Project.

2.0 Project Background and Purpose

SVCW is implementing a CIP to improve the reliability of their conveyance system and wastewater treatment plant (WWTP). The CIP includes rehabilitation and repurposing of several collection system pump stations and installation of the following facilities:

- Gravity Pipeline to replace the existing 54-inch force main that conveys wastewater to the treatment plant
- Receiving Lift Station (RLS) located on the treatment plant site at the end of the new Gravity Pipeline
- Headworks Facility to remove screenings and grit from influent wastewater
- Influent Connector Pipe to convey flow from the Headworks Facility to the primary clarifiers
- Odor control facilities to treat foul air venting from the RLS and Headworks Facility, referred to as the Front of Plant (FoP) Odor Control Facilities

An Environmental Impact Report Project Description (EIR Project Description) is currently being prepared for the CIP.

The Influent Connector Pipeline Project is being performed to support the development of the EIR Project Description by developing conceptual alignments to identify a preferred alignment. Another goal of the Influent Connector Pipelines Project is to develop a conceptual level cost estimate, including Life Cycle Costs, for the Influent Connector Pipes and the FoP Odor Control Facility.

3.0 LCC Analysis

3.1 Overview

This life cycle costs for the SVCW Influent Connector Pipes include the following cost components:

- Capital Costs
- O&M Labor
- Power
- Equipment Rehabilitation and Replacement

The cost for each of the components listed above were developed for each year over a 75 year period between 2018 and 2093 in present day dollars, as described in Section 3.2 through 3.6 below. The Net Present Value of the cash flow over that 75 year period was then calculated for all the cost components as described in Section 3.8.

According to the Plastic Pipe Institute, HDPE has a life cycle of 75 to 100 years with a 100 years being more acceptable. Because of the soil conditions that exist at the treatment plant and to be conservative a 75-year life cycle for HDPE pipe was selected.

3.2 Capital Cost

The capital cost, in 2016 dollars, is calculated based on the project's raw construction cost, project contingency, soft costs, and market fluctuations, according to Equation 1, below. The result from Equation 1 is then escalated to the mid-point of construction.

Capital Cost = Construction Cost · (1+ Project contingency +∑Soft Costs + Market Fluctuations) [Equation 1]

The calculation of the capital cost is summarized in Table 1 below. As shown, the capital cost was determined to be between $7.4M to $8.1M, depending on market fluctuations. The raw construction cost used in the calculation was based on the costs presented in the Opinion of Probable Cost of Construction, dated May 2016. The mid-point of construction was based on the latest CIP Program Schedule version #21, dated July 2016. All other values and assumptions were based on guidance in the Life Cycle Cost Analysis Guidelines TM, dated July 2016.

Table 1. SVCW Influent Connector Pipelines Capital Cost

	Rate
Raw Construction Cost (2016 Dollars)[1]	$4,424,000
Project Contingency[2]	25%
Soft Costs[2]	
CM, ESDC, Testing, Inspection	18%
Contract Change Orders (CCO)	5%
Planning	5%

Table 1. SVCW Influent Connector Pipelines Capital Cost

	Rate
Design	10%
Project Management	5%
Market Fluctuations	
Low	-5%
Base	0%
High	15%
Escalation[2]	4%
Mid-Point of Construction[3]	2018
Capital Cost (2018 Dollars)	
Low Market Fluctuation	$7,800,000
Base Market Fluctuation	$8,000,000
High Market Fluctuation	$8,800,00

[1]Based on the construction cost include in the Opinion of Probable Cost of Construction TM, dated May 2016

[2]Based on guidance in the Life Cycle Cost Analysis Guidelines TM, dated July 2016.

[2]Based on CIP Program Schedule Version #21, dated July 2016

3.3 Annual O&M Labor

The annual operation and maintenance activities associated with the Influent Connector Pipes are itemized in Table 2, below. The labor associated with each activity and the frequency of each activity are also included in Table 2. The total number of labor hours was divided by 2,080 hours to determine the number of Full-Time Equivalents (FTE) of labor required. The cost associated with the labor was then calculated based on a cost of $150,000/FTE, per the Life Cycle Cost Guidance TM.

Table 2. Itemized Labor Costs

Activity	Staff	Frequency		Total Annual
	Hours	No.	Basis	Staff Hours
Motorized gates				
Inspection	0.5	2	per year /gate	2
Channel Cleaning	1	1	per week	52
Maintenance Management				
Generating Work Orders, Procurement, Tracking Work Progress	0.5	1	per week	26
			Total Staff Hours	88
			FTEs	0
			Total Labor Cost	$ 6346

3.4 Power

The power costs associated with the Influent Connector Pipes Project are itemized in Table 3 below. Power costs for the project are determined by multiplying the estimated annual power usage of each type of equipment by the electrical cost. For the Influent Connector Pipes Project, the electric cost is $0.129 per kilowatt-hour used, per the Life Cycle Cost Guidance TM. Operation of gates and the sump pump are used only during wet weather events and is assumed to be 100 days out of the year.

Table 3. Power Costs for the SVCW Influent Connector Pipelines

Equipment	Power Demand (Hp)	Total No. of Units	Average No. Operating	Total Power Use (kWh/yr)	Annual Power Cost
Gates					
Slide Gates	2	16	2	1560	$201
Pumps					
Sump Pump	20	2	1	46789	$6037
				Total	$ 6237

3.5 Rehabilitation and Replacement

The rehabilitation and replacement activities associated with the Influent Connector Pipes are itemized in Table 4, below. The frequency and cost associated with each activity are also shown. Rehabilitation and replacement activities and costs were determined for the gates and pumps are based on typical equipment lifespan and costs. Pipe cleaning was assumed to occur at long intervals due to HDPE pipe's smooth surface, which allows minimal accumulation of Fats, Oils, and Grease. A three-man crew and light equipment would be required for cleaning. A pipe breakage, though not anticipated is also included to account for a possible catastrophic event. Anticipated costs for repair are based on SVCW's current cost to repair the influent force main.

Table 4. Rehabilitation and Replacement Costs for SVCW Influent Connector Pipes

Equipment	No. of Units	Type of Rehabilitation	No.	Basis	Cost of Rehab
Motorized Gate	2	Repair	1	every 5 years /Gate	$3,500
Condition Assessment	2	Inspection	1	every 10 years/Pipe	$11,500
Sump Pump	2	Replacement	1	every 10 years /Pump	$400,000
Pipe Cleaning	2	Cleaning	1	Every 20 years/Pipe	$18,100
Pipe Breakage Repair	2	Repair	1	Once per lifetime	$1,200,000

3.6 Net Present Value Analysis

The Net Present Value (NPV) of the cost components discussed in Sections 3.2 through 3.5 was calculated in two steps. First, the O&M costs for each year from 2018 to 2093 were developed by escalating the costs presented in Sections 3.2 through 3.5 to the year in which the cost would be incurred using Equation 2.

$$FV = PV \cdot (1+i)^{(Y_n - Y_{2016})} \quad \text{[Equation 2]}$$

where:

FV= Future Value
PV = Present Value
i = Escalation (4%)

Y_n = Year of Cost Occurrence
Y_{2016} = Present Year (2016)

The NPV of the escalated costs were then determined by discounting the costs to the Year of Beneficial Use, using Equation 3. For this LCC analysis, the Year of Beneficial Use was assumed to be 2022. Discounting was performed, according to Equation 3, on all future costs occurring after the Year of Beneficial Use. All costs incurred before the Year of Beneficial Use are considered "sunk costs" and are calculated using Equation 2 and then added to the sum of costs calculated with Equation 3 to determine the 75-year LCC at the Year of Beneficial

$Z_i = FV_i \cdot (1+r)^{-(Y_n - Y_b)}$ [Equation 3]

Where:

Z_i= Future Cost at Year of Beneficial Use
FV_i = Future Value, as calculated by Equation 1
r = Discount Rate (7% for rehab and replacement, 3% for all else)
Y_n = Year of Cost Occurrence
Y_b = Year of Beneficial Use

4.0 Conclusions

The 75-year LCC associated with the SVCW Influent Connector Pipelines, calculated as described above, is summarized in Table 5. As show, the total 75-year LCC is determined to be between $11.6M and $12.4 million dollars (in 2022 dollars), depending on market fluctuations. For the full results of the LCC analysis, see Appendix A.

Table 5. 75-Year Life Cycle Cost (LCC) for Influent Connector Pipes

	2022
Capital Cost[1]	$8 – 8.8 million
NPV of Labor, Power, and Rehabilitation/Replacement	$4.1 million
75-year LCC (2022 dollars) [1]	$11.9 – $12.8 million

[1] Range based on market fluctuations from -5 to 15 percent.

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Silicon Valley Clean Water

DRAFT Influent Connector Pipeline Geotechnical Data Report

Task Order 2015-03

March 2017

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Section 1

Introduction

This Geotechnical Data Report (GDR) summarizes the results of our investigation of geotechnical data for the influent connector pipeline project located within Silicon Valley Clean Water (SVCW) wastewater treatment plant (SVCWTP) located within Redwood City, California on the west side of San Francisco Bay, between San Mateo and Dumbarton Bridges. SVCWTP is situated at the end of a peninsula with Bay Slough to the north and Steinberger Slough to the south, as shown on Figure 1, Vicinity Map. The purpose of the geotechnical investigation was to obtain subsurface soil and groundwater information along the influent connector pipeline alignment.

1.1 Project Background

Silicon Valley Clean Water (SVCW), which was known prior to 2014 as South Bayside System Authority (SBSA), is currently implementing the initial steps of the 2011 Conveyance System Master Plan (CSMP) through its implementation of the Capital Improvement Program (CIP) to improve the reliability of the conveyance system. The steps identified in the CSMP consist of replacement of conveyance system pump stations; replacement of conveyance system force mains; and upgrades to SVCW's treatment facility and include the influent connector pipeline. Influent connector pipeline will be used to transport 80 million gallons per day (mgd) of raw wastewater from the future headworks facility to the influent side of the plant's existing primary treatment system**.** The proposed alignment for the influent connector pipeline was chosen based on an alignment alternatives analysis completed by CDM Smith (May 2016). Eight alignment alternatives (Alternatives A, B, C, D, E, F1, F2, and F3), consisting of different combinations of rehabilitating existing pipeline, constructing new pipeline(s) along the current alignment, and constructing new pipeline(s) along a new pipeline alignment(s) using open cut and microtunneling construction techniques were evaluated. Based on this evaluation, four preferred alternatives (Alternatives E, F1, F2, and F3) were identified and one recommended alternative (Alternative F3) was selected that uses open-cut method. The proposed alignment of Alternative F3 is shown on Figure 2.

1.2 Proposed Construction

The recommended alternative consists of the installation of two parallel HDPE pipelines, (1) a72-inch nominal diameter profile wall HDPE pipeline and (2) a 48-inch nominal diameter solid wall HDPE pipeline, from the future headworks facility to the influent side of the plant's existing primary treatment system. As shown on Figure 2, the alignment of the recommended alternative through the plant will be located within the street right-of-way (ROW) (i.e., Radio Road into the plant and along the main access road) and plant property boundary (parallel to the existing 54-inch RCP forcemain). The existing surface elevation along the alignments of the pipelines vary from El. 99 to El 103. The area in front of the headworks has been used as an ornamental pond and the surface elevations in this area is from El. 99 to El. 100. The entire area will be filled up to 4 feet in thickness to raise the finished grade elevation to about El. 103 to El. 104. Both pipelines will be installed to an invert elevation approximately 13-feet below ground surface, using open cut construction methods within a 15-foot wide trench excavated between interlocking steel sheet pile shoring walls installed for the full-length of the alignment.

Access manholes for each pipeline are being considered in two locations for future access and maintenance, as follows:

- At the turn in Radio Road where the pipeline transitions from south to east; and
- At the location where the pipeline turns north to connect into the main treatment plant.

1.3 Scope of Work

This geotechnical data report is prepared in fulfillment of Subtask 3.5.5C under Task Order No. 2015-03, dated September 25th, 2015. The scope primarily included the subsurface evaluation to estimate soil groundwater conditions along the pipeline alignment, and included the following tasks:

- Reviewed historical and on-going geotechnical investigations, as-built drawings, and other construction records for other improvements in the project area. Relevant exploration logs and laboratory test results were extracted for inclusion in the geotechnical data report and to refine the project's geotechnical exploration program.
- Conducted a site visit to observe surface conditions and physical surface constraints to construction in the project site, as well as to identify and finalize the planned locations for the supplemental subsurface explorations.
- Performed the supplemental subsurface soil investigation for the project focusing on the site-specific conditions that could have an impact on the project design and construction. The investigation consisted of drilling four borings using mud-rotary approach with SPT and Shelby Tube sampling for depths up to 42 feet below ground surface and monitoring groundwater encountered within these borings.
- Performed laboratory testing of representative samples obtained from the exploration borings, which included: moisture content, dry density, grain-size, Atterberg limits, specific gravity, consolidation and direct shear (undrained) testing to establish undrained shear strength properties to supplement existing test data.
- Prepared this geotechnical data report summarizing the data obtained from the above tasks.

Section 2
Surface Conditions

We reviewed the published geology maps to obtain geotechnical surface conditions along the pipeline alignment at the project site that are pertinent to the SVCW Influent Connector Pipeline Project. Specifically, we reviewed the following:

1. Brabb, E.E., Graymer, R.W., and Jones, D.L. (1998). Geology of the onshore part of San Mateo County, California: a digital database: U.S. Geological Survey, Open-File Report OF-98-137, scale 1:62,500.

2. Brabb, E.E. and Pampeyan, E.H. (1983). Geologic map of San Mateo County, California: U.S. Geological Survey, Miscellaneous Investigations Series Map I-1257-A, scale 1:62,500.

2.1 Site Conditions

The project area is located on the south eastern part of the SVCWTP site. The SVCWTP site was created by placing levees and fill over reclaimed marshland starting in about the 1950s (DCM|GeoEngineers 2009). The most recent fills were placed during the develop of the site during late 1970s and early 1980s for the construction of SVCWTP facilities north of the project site. During the construction of the SVCWTP facilities, the project site was reportedly used as construction staging area. Subsequent to the construction of the SVCWTP facilities, the area in front of the Plant has been used as an ornamental pond, and the surface elevations in this area range from El. 99 to El. 100. The surface elevation to the east and south of this ornamental pond area rise slightly up to El. 103 to El. 104. In general, the surface topography of the project site is relatively flat, and no distinct topographic features are noted across the project site.

2.2 Site Geology

Geologic mapping by U.S. Geologic Survey (USGS) (Brabb et al. 1998) indicates that the project site is underlain by bay mud locally referred to as Younger Bay Mud (YBM), as shown on Figure 3, Surface Geologic Map. An earlier USGS map Brebb and Pampeyan (1983) shows that portions of the project site with some areas of artificial fill, while majority of the site with YBM. Descriptions of these geologic units are as described below:

- **Bay Mud**: Water-saturated estuarine mud, predominantly gray, green and blue clay and silty clay underlying marshlands and tidal mud flats of San Francisco Bay. The mud also contains few lenses of well-sorted, fine sand and silt, a few shelly layers (oysters), and peat.

- **Artificial Fill (af)**: Loose to very well consolidated gravel, sand, silt, clay, rock fragments, organic matter, and man-made debris in various combinations.

In this area, the artificial fill soil unit is typically underlain by bay mud (YBM) soil unit.

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Section 3

Subsurface Conditions

Subsurface conditions along the pipeline alignment at the project site were investigated by reviewing the results of the previous exploration programs that have been conducted by Cooper, Clark & Associates (1978a, 1978b, 1980 and 1981), Dames & Moore (1978), Fugro (2002), Fugro West Inc. (2004a, 2004b and 2004c) DCM|GeoEngineers (2009), and DCM Consulting (2014 and 2015) in the vicinity of the project site (Figure 4). In areas where sufficient subsurface information was not available, geotechnical borings were drilled as part of the current investigation to explore subsurface conditions and collect additional geotechnical data. The pipeline alignment and the selected exploration locations from the previous and current geotechnical investigations are shown on Figure 4, Geotechnical Exploration In the Vicinity. The borehole logs from these previous investigations that are in the vicinity of the pipeline alignment and are pertinent to this project were selected and are included in Appendix A, Logs of Borings.

3.1 Previous Geotechnical Investigations

We reviewed the following previous geotechnical investigations to present the data summarized in this report:

1. Cooper, Clark & Associates (1978a). "Foundation Investigation, Proposed Subregional Wastewater Works, Redwood City, California", Prepared for the 'South Bayside System Authority', February 15.

2. Cooper, Clark & Associates (1978b). "Supplementary Subsurface Investigation and Laboratory Testing, SBSA Project Unit No. 1, Redwood City, California", Prepared for the 'South Bayside System Authority', October 18.

3. Dames & Moore (1978). "Soil Investigation and Slope Stability Evaluations Construction Excavations, Subregional Wastewater Works, Redwood City, California" for South Bayside System Authority, December 22.

4. Cooper, Clark & Associates (1980). "Progress Report: Installation and Observation of Groundwater Wells and Piezometers, proposed Main Structure, Redwood City, California", Prepared for the 'South Bayside System Authority', November 06.

5. Cooper, Clark & Associates (1981). "Consultation: Re: Proposed Influent/Effluent Tie-In to Existing Force Main, Wastewater Treatment Plant, Redwood City, California", Prepared for the 'South Bayside System Authority', May 07.

6. Fugro (2002). "Geotechnical Investigation and Data Report, SBSA SWTP Recycled Water System Storage, Redwood City, California" for South Bayside System Authority, October 17.

7. Fugro West, Inc. (2004a). "Recommended Su Profile for Shoring Design (Revised), South Bayside System Authority (SBSA), Redwood City, California", Prepared for the 'South Bayside System Authority', July 14.

8. Fugro West, Inc. (2004b). "Geotechnical Study: Recycled Water Storage and Disinfection Facilities, South Bayside System Authority, Redwood City, California", October 20.

9. Fugro West, Inc. (2004c). "Supplemental Geotechnical Recommendations, Recycled Water Storage and Disinfection Facilities, South Bayside System Authority (SBSA), Redwood City, California", Prepared for the 'South Bayside System Authority', November 08.

10. DCM|GeoEngineers (2009). "Technical Memorandum: New Administration and Plant Control Building Project, South Bayside System Authority Wastewater Treatment Plant, Redwood City, California", Prepared for South Bayside Authority, July 06.

11. DCM Consulting (2014). "Cone Penetrometer Test (CPT) Logs from the CPT Investigation at the SVCWTP site", Directed by David Mathy.

12. DCM Consulting (2015). "Supplemental Cone Penetrometer Test (CPT) Logs from the CPT Investigation at the SVCWTP site", Directed by David Mathy.

A summary of the results from the previous geotechnical investigations in the vicinity of the pipeline alignment is presented herein.

3.1.1 Cooper, Clark & Associates (1978a)

This geotechnical investigation was conducted in 1975, prior to the construction of the current Wastewater Treatment Plan (WWTP) to evaluate the subsurface soil and groundwater conditions for its design. This investigation included a field and laboratory investigation program. Locations of the borings applicable to this project are shown on Figure 4, and the logs of these selected borings are included in Appendix A. The laboratory investigation consisted of performing visual classification, moisture content and dry density tests, direct shear tests and consolidation tests. All laboratory test results, except from consolidation tests, were incorporated in the borehole logs presented in Appendix A. No consolidation test results were available from samples collected from the selected borings.

The relevant findings from this investigation, which are also summarized in DCM|GeoEngineers (2009), are included below:

- 2-feet thick YBM crust was noted in the borings drilled over the entire project area.
- YBM to 65 to 75 feet below ground surface (bgs) in borings explored.

3.1.2 Cooper, Clark & Associates (1978b)

This geotechnical investigation was conducted in 1978 to obtain additional subsurface soil and groundwater information and perform additional laboratory evaluations to obtain supplementary geotechnical data for the design and construction of the current Wastewater Treatment Plant (WWTP). This investigation included a field and laboratory investigation program. Locations of the borings applicable to this project are shown on Figure 4, and the logs of these selected borings are included in Appendix A.

The laboratory investigation consisted of performing visual classification, moisture content and dry density tests, direct shear tests, unconfined compression tests, and unconsolidated-undrained (UU) triaxial tests. All laboratory test results, except unconfined compression and UU triaxial tests, were incorporated in the borehole logs presented in Appendix A. The borehole logs included in Appendix A show "yield point" strength from the direct shear tests. Unconfined compression test results and UU triaxial test results corresponding to the selected borings are included in Appendix B, (Figures B16 and B17). The logs for the selected borings showing maximum strength from the direct shear tests are included in Appendix A.

The relevant findings from this investigation, which were also summarized in DCM|GeoEngineers (2009), are included below:

- Post-development borings show that between 1 and 4 feet of fill has been noted to have been placed over the YBM crust.
- Below the fill and YBM crust, highly compressible YBM extends to a depth of approximately 75 feet in the project area.
- Below the YBM, firm to stiff clay interlayered with dense sand extends to the maximum depth explored by Cooper, Clark & Associates of 200 feet.

3.1.3 Cooper, Clark & Associates (1980)

This geotechnical investigation was completed by Cooper, Clark & Associates in 1980, to investigate the groundwater elevation and its rate of rising below the WWTP main structure that was being constructed at that time.

The field investigation included drilling three shallow borings and installed two piezometers (PA and PB) in each boring. Locations of the borings applicable to this project are shown on Figure 4, and the logs of these selected borings are included in Appendix A.

The laboratory investigation consisted of performing visual classification, and moisture content and dry density tests. All laboratory test results were incorporated in the borehole logs presented in Appendix A. Groundwater levels recorded for the selected piezometers are included in Appendix A, Figure A3-1).

3.1.4 Cooper, Clark & Associates (1981)

This geotechnical investigation was completed by Cooper, Clark & Associates in 1980 to evaluate the feasibility of connecting a 60-inch diameter influent pipeline and a 66-inch diameter effluent pipeline, both cement-lined and coated welded steel pipelines, to an existing 54-inch diameter RCP forcemain located immediately within the existing perimeter levee southwest of the SVCWTP.

The field investigation included drilling four borings. Locations of the borings applicable to this project are shown on Figure 4, and the logs of these selected borings are included in Appendix A. The laboratory investigation consisted of performing visual classification, moisture content and dry density tests, direct shear tests, and a consolidation test on a representative sample of the YBM. All laboratory test results, except the consolidation test, were incorporated in the borehole logs presented in Appendix A. Results of the consolidation test are included in Appendix B (Figure B15).

The relevant findings from this investigation, which are also summarized in Cooper, Clark & Associates (1981) are included below:

- At the boring locations explored, the area is blanketed by fill ranging in thickness from 5 feet to 11-1/2 feet. The fill is underlain by YBM extending to depths explored.
- Groundwater was encountered at a depth of about 5 feet below the existing ground surface.

3.1.5 Dames & Moore (1978)

For the construction of the SVCWTP in 1977, temporary excavations extending to EL 68 were planned with side slopes of 2H:1V. However, during construction, slope failures occurred at several locations and it was decided to raise the plant by 7 feet (Fugro West 2004). Subsequent to the failures, Dames & Moore conducted a geotechnical investigation to evaluate Su profiles of soft Bay Mud that caused the failures of 2H:1V slopes during excavations. The Dames & Moore (1978) report was not available for our review, however, this evaluation was summarized in the Fugro West, Inc. (2004) report, as follows:

- Dames & Moore evaluated various factors that might affect the Su of the soft Bay Mud, such as: (a) the existence of former sloughs and non-slough areas; (b) the effect of construction traffic and fill loads; and (c) the effect of filling and excavation.
- Based on this evaluation Dames & Moore concluded that
  - There was a small but discernable difference in soil strength between the slough and non-slough areas; and
  - The effects of construction traffic and fill loads, as well as filling and excavation did not display a discernable difference in pre-construction and post-construction shear strength, especially if the excavation was conducted in relatively short construction period.

While the locations of these failures are approximately 400 to 800 feet north to northwest of our current project site, the shear strength data for the YBM is relevant. Therefore, the back-calculated Su profile for the Bay Mud by Dames & Moore is included in Appendix B (Figure B8).

3.1.6 Fugro (2002)

This geotechnical memorandum was not available for review. The geotechnical data collected were summarized in the Fugro West (2004a), as follows:

- Field geotechnical investigation completed include drilling four exploratory borings (B1 to B4), five cone penetration tests (CPT-1 to CPT-5) and three in-situ vane shear tests (FVST-1 to FVST-3).

- Su measured from the in-situ vane shear tests displayed lower strengths than those recommended by Dames & More (1978). The memorandum concluded that this discrepancy was due to equipment limitations during testing.

- Additional vane shear tests (Fugro West 2004a) were performed along with laboratory testing.

As noted in the previous section, the locations of the explorations are north of the project site, however, the shear strength data for the YBM is relevant to our project. Therefore, summaries of the relevant shear strength data collected for the Bay Mud by Fugro (2002) are included in Appendix B (Figure B13).

3.1.7 Fugro West, Inc. (2004a)

This geotechnical investigation was completed by Fugro West, Inc. in 2004 to evaluate the undrained shear strength (Su) profiles for the design of a disinfection facility, pump station, and two storage facilities at the WWTP site of the SVCW. Excavations as deep as 10 to 25 feet deep were anticipated for the disinfection facility construction. The geotechnical investigation included: compiling and reviewing available geotechnical data pertinent to the project; and conducting field explorations and laboratory investigations to supplement the available subsurface data. Fugro West (2004a) summarized geotechnical data from the following previous geotechnical reports:

- Cooper, Clark & Associates (1975)

- Dames & Moore (1978)

- Fugro (2002)

These reports were not available for us to review, but the Fugro West (2004a and 2004b) summaries have been provided earlier.

During this investigation, supplemental field and laboratory investigations were performed. The field investigation included: drilling two borings (B5 and B6, Torvane shear tests in the borings, and two field vane shear tests (B5A and B6A) at approximately 3 feet away from the corresponding borings. The in-situ shear strength data collected from this field investigation are included in Appendix B (Figure B14). The laboratory investigation consisted of visual inspection and classification, ten unconsolidated undrained (UU) and triaxial compression tests to determine the undrained shear strength of the YBM. The results obtained from these tests are included in Appendix B of this report. The findings from this investigation pertaining to this project are briefly summarized below:

- The subsurface conditions at locations explored consisted of undocumented fill of about 6 feet, underlain by 4 feet of stiff clay, identified as Bay Mud Crust. Below the crust, YBM was encountered and extended to a depth of about 61 feet, followed by stiff alluvium deposits (Old Bay Clay).

3.1.8 Fugro West, Inc. (2004b)

This geotechnical investigation was completed by Fugro West, Inc. in 2004 to evaluate the subsurface soil and groundwater conditions for the design of a disinfection facility, pump station, two storage facilities, and two 36-inch diameter pipelines at the SVCWTP. Excavations as deep as 10 to 25 feet deep were anticipated for the disinfection facility and 8 to 15 feet were anticipated for the pipeline constructions. Temporary shoring using retrievable sheet piles were considered in the report. The geotechnical investigation included: compiling and reviewing available geotechnical data pertinent to the project; conducting field explorations and laboratory investigations to supplement the available subsurface data; conducting a site specific seismic response analysis; and developing geotechnical recommendations and preparing geotechnical report. An earlier Fugro West report (2004a) provided a summary of the data reviewed from the previous reports.

For this geotechnical investigation, two phases of field and laboratory investigations were performed. The first phase of the field investigation included: four borings to a maximum depth of 130 feet, five CPTs to a maximum depth of 80 feet, and three field vane shear tests to a maximum depth of 80 feet. The second phase of the field investigation included: two exploratory borings and two field vane shear tests. The laboratory investigations consisted of performing visual inspection and classification tests, strength tests (unconsolidated, undrained (UU) triaxial compression tests), and consolidation tests. The locations of Phase 1 and Phase 2 explorations are about 400 to 800 feet north of the project area, however, the laboratory consolidation tests on YBM and undrained shear data collected on the YBM are pertinent for this project. Therefore, the undrained shear (Su) and consolidation data collected from this investigation on YBM are included in Appendix B (Figure B9) of this report.

3.1.9 DCM|GeoEngineers (2009)

This geotechnical investigation was conducted by DCM|GeoEngineers in 2009 to evaluate the subsurface soil and groundwater conditions for the design of the New Administration and Plant Control Building Project at SVCW's wastewater treatment plant. The investigation included: review of the earlier geotechnical reports at the site (Cooper, Clark & Associates 1975, 1978a, 1978b and 1980; and Fugro West Inc. 2004), as well as a field and laboratory investigation program.

The field investigation included drilling eight shallow borings and monitoring of groundwater levels during the drilling. Of these, the borings in the vicinity of the proposed pipeline alignment have been selected for inclusion in this data report, and their locations are shown on Figure 2, Geotechnical Exploration Plan. A summary of the subsurface data revealed from this field investigation has been summarized in DCM|GeoEngineers (2009) as a table, which is included in Appendix A. The laboratory investigation consisted of performing visual inspection and classification. The results obtained from these results were incorporated in the summary table (Table 1 of DCM|GeoEngineers (2009)) included in Appendix A of this report.

The findings from this investigation are summarized below:

- Geologic mapping by USGS indicate that YBM deposits underlie the entire site.

- The pre-WWTP development borings (Cooper, Clark & Associates 1975) reviewed showed about 2 feet of YBM crust over the project area. Post-WWTP development borings showed between 1 to 4 feet of fill over the YBM crust.
- Bellow the fill and YBM crust, soft, highly compressible YBM extends to a depth of approximately 75 feet in the project area. Below the YBM, firm to stiff clay interlayered with dense sand extends to the maximum depths explored, which is about 200 feet.
- Groundwater was measured in three of the eight shallow borings at depths between 1 and 3.5 feet at the end of drilling prior to backfilling.

3.1.10 DCM Consulting (2014 & 2015)

Based on previous borings and penetrometer tests (CPT), Dave Mathy of DCM Consulting Inc. mapped the depth of the YBM soils. Considering the data gaps existed for this project, DCM Consulting Inc. performed additional CPT investigations in 2014 and in 2015, and updated the depth to YBM map. Based on this updated map (DCM Consulting, Inc. 2015), the depth to YBM at the project site ranges from 60 feet at the southern corner of the facility (entrance gate) to 70 feet below ground surface to the north. Relevant logs of the CPT investigations performed by DCM Consulting Inc. (2014 and 2015) are included in Appendix A. The updated map showing depth to YBM contours are also included in Appendix A for information purposes only, and is not part of the GDR.

3.2 Current Geotechnical Investigation

Additional investigation was conducted in areas where sufficient subsurface data was not available for developing geotechnical recommendations for the design of the proposed dual large-diameter buried HDPE influent connector pipelines. This included: performing a site reconnaissance visit, field geotechnical explorations, and laboratory investigations. Brief descriptions of each of these tasks are presented in the following subsections.

3.2.1 Site Reconnaissance Visit

A site reconnaissance visit was performed by CDM Smith on March 8, 2016 to observe the surface conditions, physical surface constraints and locate boring locations.

Proposed CDM smith borings were located within SVCW boundaries, generally along Radio Road which is the main entrance to the wastewater treatment facility and provides parking for visitors and employees. Radio Road is also underlain with numerous utilities. During our site reconnaissance and subsurface explorations, construction to repair the plant outfall was on-going north of the existing influent system. Boring locations were adjusted in the field due to utilities, construction activities and to minimize impact to the facilities actives. Figure 2 presents the approximate locations of the current and pertinent previous explorations.

As part of our exploration planning, the utility locate network (Underground Service Alert North 811 or USA) and the SVCW were notified of our proposed boring locations. Members of the USA network located the underground utilities leading up to SVCW boundaries and SVCW located utilities within the facility.

3.2.2 Subsurface Exploration

To identify, evaluate and characterize the subsurface conditions at the project site, soil borings were drilled at the approximate locations identified on Figure 4, Geotechnical Exploration Plan. The following subsections present description of the subsurface explorations and associated results, which are presented in Appendix A.

3.2.2.1 Soil Borings

CDM Smith conducted a subsurface investigation along the proposed influent connector pipeline between March 16 and 18, 2016. The subsurface conditions were explored by advancing a total of four borings (CDM-1 to CDM-4) as shown on Figure 4.

Utility clearance prior to drilling was performed by either hand-excavating to a depth of 5 feet or by trenching. There was some uncertainty of the location of an 18-inch diameter influent pipeline. Under the direction of SVCW, two trenches were excavated to the depth of 11.5 feet just south of CDM-01 and CDM-3. These borings were not drilled or sampled until receiving approval from SVCW. Table 3-1 presents a summary of boring depths, dates drilled, and any comments.

Table 3-1 Soil Boring Summary

Boring Designation	Depth Drilled (Ft)	Date Drilled	Comment
CDM-1	42	3-17-2016	Utility trench excavated to locate 18-inch SSFM
CDM-2	42	3-16-2016	
CDM-3	41.4	3-16-2016	Utility trench excavated to locate 18-inch SS.
CDM-4	42	3-16-2016	

Soil borings were drilled with a modified, track-mounted B-57 drill rig, owned and operated by Woodward Drilling, Rio Vista, CA. Borings were advanced using mud rotary methods. The Boring Logs from the current and previous subsurface investigations along with a legend are presented in Appendix A.

3.2.2.2 Soil Sampling

Disturbed samples were obtained from the soil borings in general accordance with the Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils (ASTM D 1586) at 5-foot vertical intervals. Relatively undisturbed samples were also obtained by placing 1-inch diameter rings inside of the split-spoon sampler. SPT testing consists of driving a 2-inch outside diameter (OD), split-spoon sampler a total of 18 inches into the bottom of the boring with a 140-pound hammer, free falling 30 inches. The number of blows required to drive the sampler 18 inches through three 6-inch increments is recorded on the field logs. The SPT resistance, or SPT N-value, is the number of blows required to drive the sampler from 6 to 18 inches. The SPT N-values provide a means for evaluating the relative density or compactness of cohesionless (granular) soils, and consistency or stiffness of cohesive (fine-grained) soils.

Representative ring samples were capped with plastic caps and sealed with tape, soil retrieved from the split-barrel sampler were collected and stored in zip-loc freezer bags. These samples were shipped to our geotechnical laboratory in Bellevue, Washington, for review and testing. Undisturbed Shelby tube samples were also collected at selected intervals to be used for geotechnical analyses in accordance with ASTM D1587. Shelby tubes were sealed like the 1-inch rings samples by capping and sealing both ends. The Shelby tube samples were maintained in an upright position and protected from shock and temperature extremes during transportation to Copper Testing Labs, located in Palo Alto, CA.

3.2.2.3 Site Groundwater

As mentioned previously, the boreholes were hand excavated with an auger to the depth of 5 feet. Groundwater levels were estimated from the condition of the soils extracted from the borehole, observed water levels on the auger, and allowing the groundwater to stabilize within the borehole and measuring the depth. Depth to groundwater encountered has been incorporated into the borehole logs

3.2.3 Geotechnical Laboratory Testing

Soil samples collected from the SPT samples were delivered to our CDM Smith laboratory, and Shelby Tube samples were delivered to the Cooper Testing Laboratory (Cooper). Cooper performed Consolidation and Unconfined Triaxial tests, both labs performed Atterberg limits, and moisture content tests and CDM Smith performed particle size analysis and moisture content and dry density tests on representative samples.

Listed below is the geotechnical laboratory testing performed by both laboratories, in general accordance with the ASTM standards:

- Fifteen moisture content analyses for soil (ASTM D2216);
- Two grain size analyses including hydrometer analyses for soil (ASTM D422);
- Six Atterberg limits analyses for soil (ASTM D4318);
- Two, one-dimensional incremental load consolidation test for soil (ASTM D2435); and
- Two, Unconsolidated-Undrained Triaxial test (ASTM D2850).
- Twelve, Determination of Density (Unit Weight) of Soils (ASTM D263).

The test results are presented in Appendix B along with Summary Table B-1. Test results are also provided at the appropriate sample depths on the individual boring logs included in Appendix A.

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Section 4
Limitations

This GDR was prepared for the exclusive use of CDM Smith, Silicon Valley Clean Water, and their authorized agents for the Influent Pipeline Connector Project at the Silicon Valley Clean Water (SVCW) Wastewater Treatment Plant (SVCWTP), under a scope of work and level of effort determined by SVCW to be suitable for its objectives and purposes. This GDR presents the data collected from field explorations of subsurface conditions and laboratory investigations performed on samples collected from this investigation using the means and methods described in this report, as well as subsurface data collected and laboratory results obtained by others in earlier investigations. No other representation is made. Unanticipated soil conditions are commonly encountered and cannot be fully be determined by merely exploring at select locations. This report should be made available to prospective contractors for information on factual data only. Subsurface conditions interpreted from the data presented in this GDR may not be construed as a guarantee or warranty of such interpreted conditions. Depending on the design or construction approach adopted by the Contractor, and the intended means and methods of the Contractor, additional geotechnical data may be necessary.

CDM Smith

Sri Rajah, Ph.D., P.E.
Principal Engineer

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Figures

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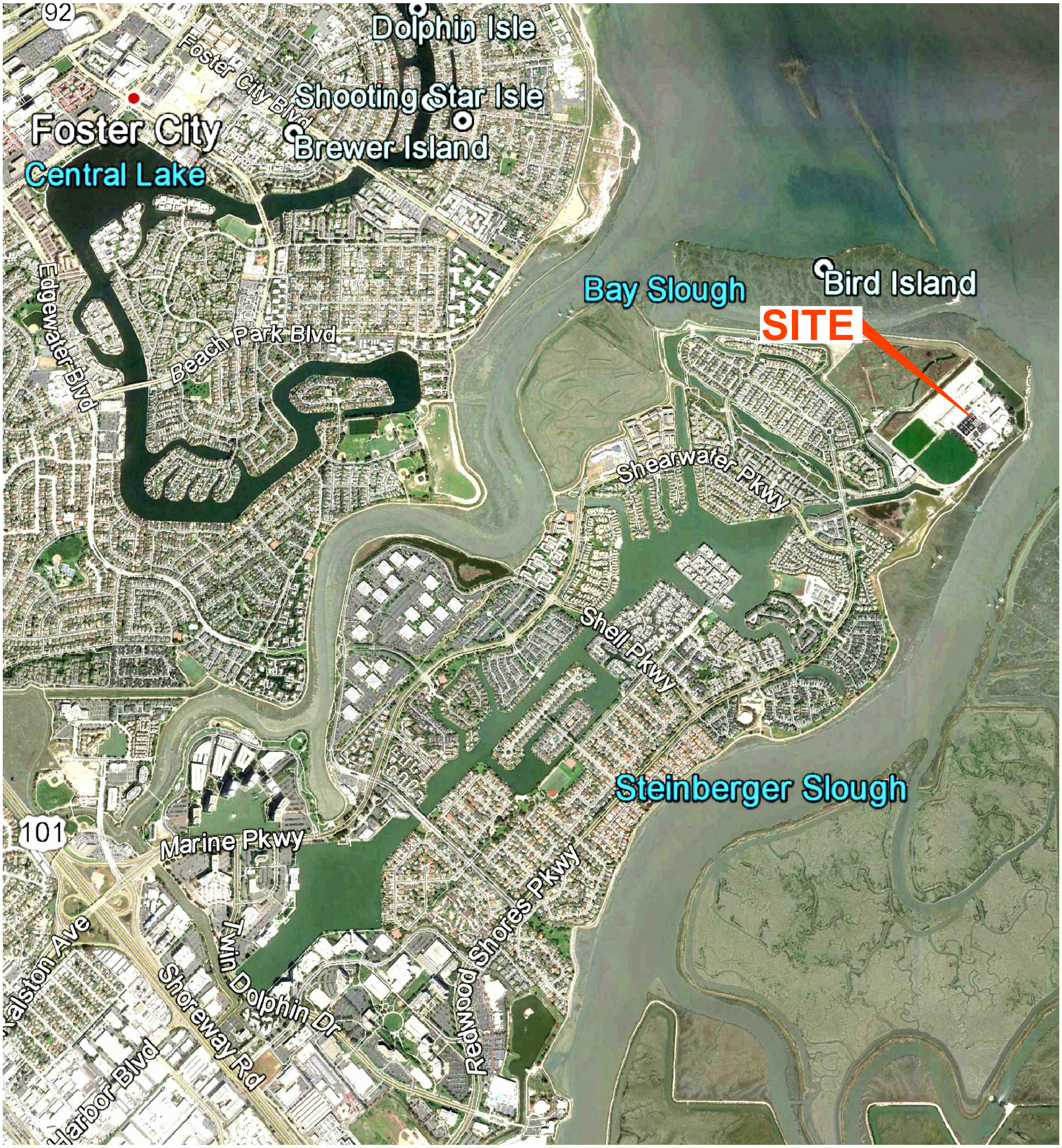

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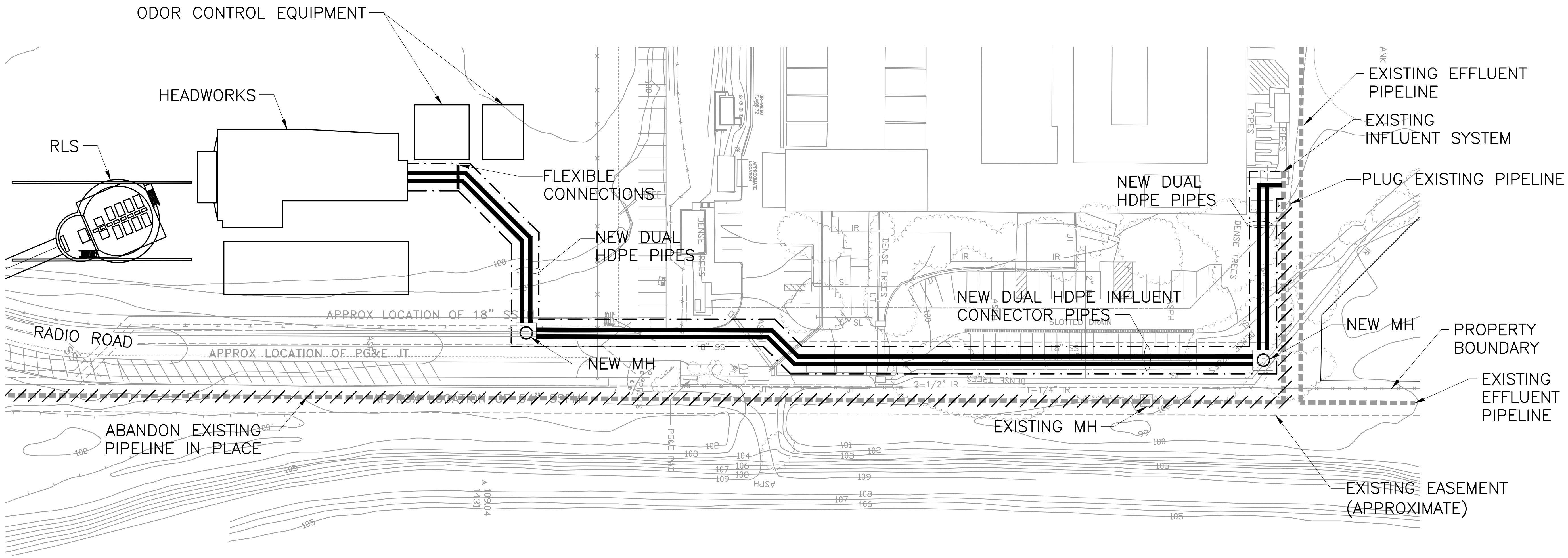
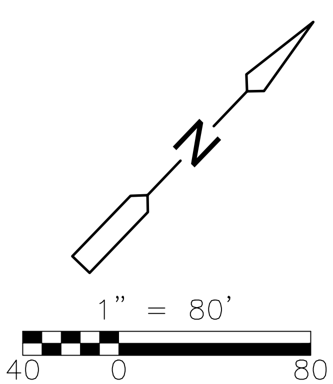
Scale in Feet

CDM Smith

Figure 1
Vicinity Map
WWTP Influence Connector
Silicon Valley Clean Water, Redwood City, California





PIPING			
PIPE	LENGTH (FT)	INSIDE DIAMETER	TRENCH WIDTH (FT)
NEW HDPE	1800	44" DRY, 72" WET	20
MANHOLES			
2 NEW MANHOLES			

LEGEND

- NEW PIPELINE
- EXISTING PIPELINE
- TRENCH/EXCAVATION LIMITS
- WWTP PERIMETER FENCE
- EXISTING EASEMENT

NOT FOR CONSTRUCTION PURPOSES

CDM Smith

Figure 2
Alternative F3 Site Plan
WWTP Influence Connector
Silicon Valley Clean Water, Redwood City, California

af **Artificial fill (Historic)**—Loose to very well consolidated gravel, sand, silt, clay, rock fragments, organic matter, and man-made debris in various combinations. Thickness is variable and may exceed 30 m in places. Some is compacted and quite firm, but fill made before 1965 is nearly everywhere not compacted and consists simply of dumped materials

Qhbm **Bay mud (Holocene)**—Water-saturated estuarine mud, predominantly gray, green and blue clay and silty clay underlying marshlands and tidal mud flats of San Francisco Bay, Pescadero, and Pacifica. The upper surface is covered with cordgrass (*Spartina* sp.) and pickleweed (*Salicornia* sp.). The mud also contains a few lenses of well-sorted, fine sand and silt, a few shelly layers (oysters), and peat. The mud interfingers with and grades into fine-grained deposits at the distal edge of Holocene fans, and was deposited during the post-Wisconsin rise in sea-level, about 12 ka to present (Imbrie and others, 1984). Mud varies in thickness from zero, at landward edge, to as much as 40 m near north County line

Source:
Brabb, E.E., Graymer, R.W., and Jones, D.L. (1998). Geology of the onshore part of San Mateo County, California: a digital database: U.S. Geological Survey, Open-File Report OF-98-137

NOT FOR CONSTRUCTION PURPOSES

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Figure 3
Surface Geologic Map (Brabb, Graymer and Jones 1998)
WWTP Influence Connector
Silicon Valley Clean Water, Redwood City, California

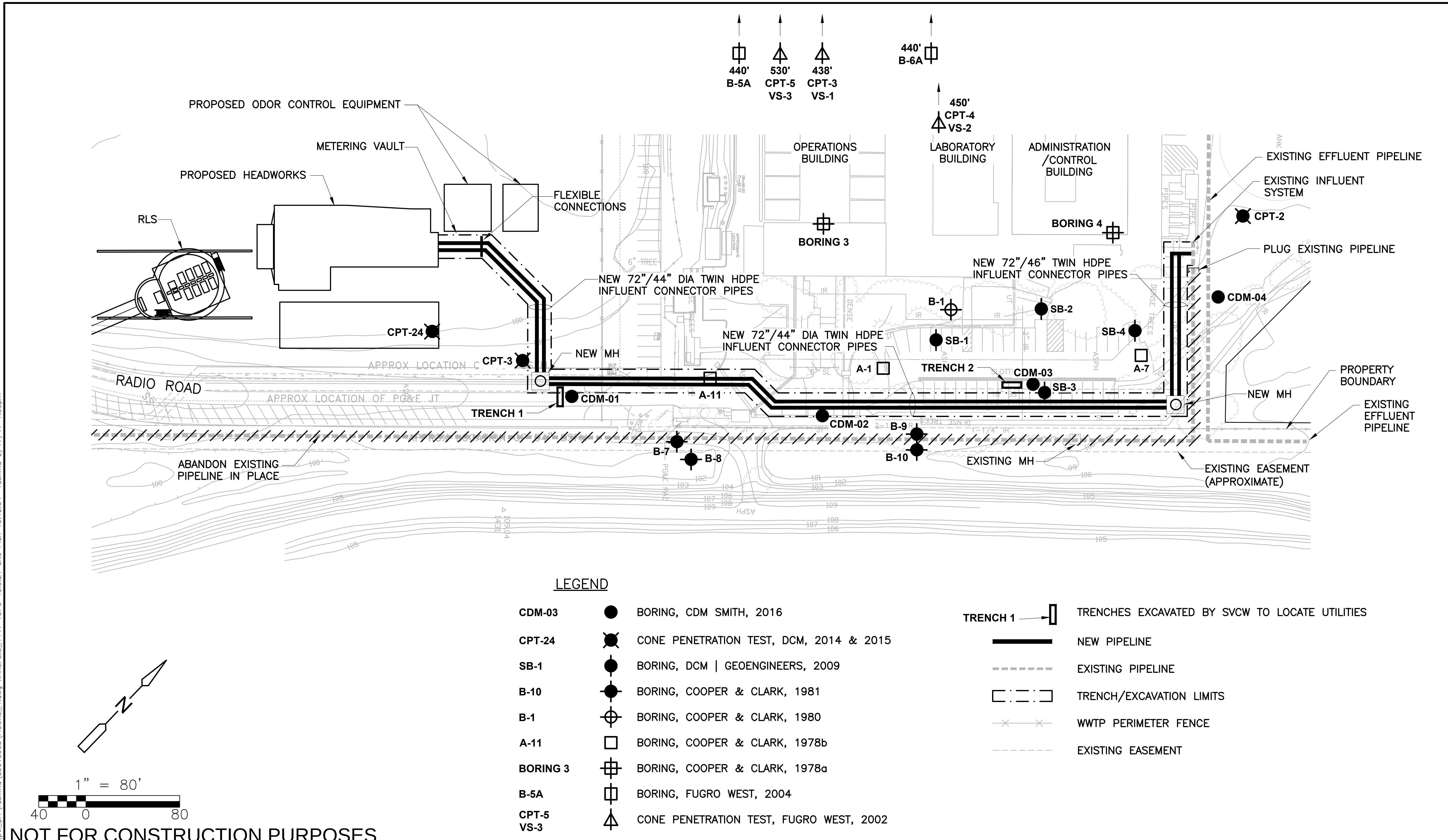

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Figure 4
Geotechnical Exploration In The Vicinity
WWTP Influence Connector
Silicon Valley Clean Water, Redwood City, California

Appendix A

Legends, Logs of Borings and Cone Penetrometer Tests

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Figure Number	Exploration Number	Data Source	Comment
A1-1	CDM Smith Legend	CDM Smith (2017)[1]	
A1-2	CDM-01	CDM Smith (2017)	
A1-3	CDM-02	CDM Smith (2017)	
A1-4	CDM-03	CDM Smith (2017)	
A1-5	CDM-04	CDM Smith (2017)	
A2-1	CPT-24	DCM Consulting (2015)[2]	
A2-2	CPT-3	DCM Consulting (2014)[3]	On Freyer & Laureta Younger Bay Mud Contour Map, CPT is referred to as CPT-6
A2-3	CPT-2	DCM Consulting (2014)	
A3-1	SB-1 through SB-4	DCM/GeoEngineers (2009)[4]	Table 1 – Summary of Subsurface Soil and Groundwater Conditions at Shallow Borings
A4-1	B-7	Cooper & Clark (1981)[5]	
A4-2	B-8	Cooper & Clark (1981)	
A4-3	B-9 & B-10	Cooper & Clark (1981)	
A4-4	Cooper & Clark Legend	Cooper & Clark (1981)	
A5-1	B-1	Cooper & Clark (1980)[6]	
A6-1	A-1	Cooper, Clark & Associates (1978) [7]	
A6-2	A-7	Cooper, Clark & Associates (1978)	
A6-3	A-11	Cooper, Clark & Associates (1978)	
A7-1	Cooper, Clark & Associates Legend	Cooper, Clark & Associates (1978)[8]	
A7-2	Boring 3	Cooper, Clark & Associates (1978)	
A7-3	Boring 4	Cooper, Clark & Associates (1978)	
A8-1	Updated CPT/ Younger Bay Mud (YBM) Mapping	DCM Consulting (2015)[9]	Provided for information only. Not part of the Geotechnical Data Report.

[1] this report.

[2] DCM Consulting (2015). "Supplemental Cone Penetrometer Test (CPT) Logs from the CPT Investigation at the SVCWTP site", Directed by David Mathy.

[3] DCM/ Consulting (2014). "Cone Penetrometer Test (CPT) Logs from the CPT Investigation at the SVCWTP site", Directed by David Mathy

[4] DCM/ GeoEngineers, (2009), "Technical Memorandum: New Administration and Plant Control Building Project South Bay Authority Wastewater Treatment Plant Redwood City, California" prepared for the South Bayside System Authority, July 6.

[5] Cooper & Clark (1981). "Consultation RE: Proposed Influent/Effluent Tie-in to Existing Force Main Wastewater Treatment Plant Redwood City, California for The South Bayside System Authority", May 7.

[6] Cooper & Clark (1980). "Progress Report of Installation and Observation of Groundwater Wells and Piezometers, Proposed Main Structure Redwood City, California for The South Bayside System Authority", November 06.

[7] Cooper, Clark & Associates (1978a), "Supplementary Subsurface Investigation and Laboratory Testing SBSA Project Unit 1, Redwood City, California" prepared for South Bayside System Authority, October 18.

[8] Cooper, Clark & Associates (1978b), "Supplementary Subsurface Investigation and Laboratory Testing SBSA Project Unit 1, Redwood City, California" prepared for South Bayside System Authority, February 14.

9 DCM Consulting (2015). "Supplemental Cone Penetrometer Test (CPT) Logs from the CPT Investigation at the SVCWTP site", Directed by David Mathy.

SOIL CLASSIFICATION LEGEND

MAJOR DIVISIONS				TYPICAL NAMES
COARSE GRAINED SOILS More than half is larger than No. 200 sieve	GRAVELS More than half coarse fraction is larger than No. 4 sieve size	Clean gravels with little or no fines	GW	Well graded gravels, gravel-sand mixtures
			GP	Poorly graded gravels, gravel-sand mixtures
		Gravel with over 12% fines	GM	Silty gravels, gravel-sand-silt mixtures
			GC	Clayey gravels, gravel-sand-clay mixtures
	SANDS More than half coarse fraction is smaller than No. 4 sieve size	Clean sands with little or no fines	SW	Well graded sands, gravelly sands
			SP	Poorly graded sands, gravelly sands
		Sands with over 12% fines	SM	Silty sand, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures
FINE GRAINED SOILS More than half is smaller than No. 200 sieve	SILTS AND CLAYS Liquid limit less than 50		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
			OL	Organic clays and organic silty clays of low plasticity
	SILTS AND CLAYS Liquid limit greater than 50		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic clays of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS			PT	Peat and other highly organic soils

SAMPLE TYPE SYMBOLS

- Disturbed bag or jar sample
- Std. Penetration Test (2.0" OD)
- Type U Ring Sampler (3.25" OD)
- California Sampler (3.0" OD)
- Undisturbed Tube Sample
- Grab Sample
- Core Run
- Non-standard Penetration Test (with split spoon sampler)

CONTACT BETWEEN UNITS

- Change in geologic unit
- Soil type change within geologic unit
- Obscure or gradational change

DESCRIPTORS FOR SOIL STRATA AND STRUCTURE (ENGLISH/METRIC)

General Thickness or Spacing

Parting:	less than 1/16 in. (1/6 cm)
Seam:	1/16 to 1/2 in. (1/6 to 1 1/4 cm)
Layer:	1/2 to 12 in. (1 1/4 to 30 1/2 cm)
Stratum:	> 12 in. (30 1/2 cm)
Scattered:	< 1 per ft. (30 1/2 cm)
Numerous:	> 1 per ft. (30 1/2 cm)

Structure

Pocket:	Erratic, discontinuous deposit of limited extent
Lens:	Lenticular deposit
Varved:	Alternating seams of silt and clay
Laminated:	Alternating seams
Interbedded:	Alternating layers

General Attitude

Near horizontal:	0 to 10 deg.
Low angle:	10 to 45 deg.
High angle:	45 to 80 deg.
Near Vertical:	80 to 90 deg.

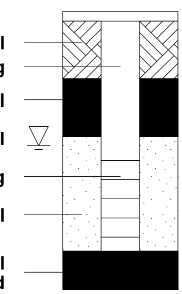
MOISTURE DESCRIPTION

- Dry - Free of moisture, dusty
- Moist - Damp but no visible free water
- Wet - Visible free water, saturated

STRUCTURE DESCRIPTION (cont.)

Fractured	Breaks easily along definite fractured planes
Slickensided	Polished, glossy, fractured planes
Blocky, Diced	Breaks easily into small angular lumps
Sheared	Disturbed texture, mix of strengths
Homogeneous	Same color and appearance throughout

WELL COMPLETIONS



RELATIVE DENSITY OR CONSISTENCY VS. SPT N-VALUE

COARSE GRAINED			FINE GRAINED		
Density	N (blows/ft)	Approx. Relative Density (%)	Consistency	N (blows/ft)	Approx. Undrained Shear Str. (psf)
Very Loose	0 to 4	0 - 15	Very Soft	0 to 2	<250
Loose	4 to 10	15 - 35	Soft	2 to 4	250 - 500
Medium Dense	10 to 30	35 - 65	Medium Stiff	4 to 8	500 - 1000
Dense	30 to 50	65 - 85	Stiff	8 to 15	1000 - 2000
Very Dense	Over 50	85 - 100	Very Stiff	15 to 30	2000 - 4000
			Hard	over 30	>4000

PHYSICAL PROPERTY TEST

- AL - Atterberg Limits
- FC - Fines Content
- GSD - Grain Size Distribution
- MC - Moisture Content
- MD - Moisture Content/Dry Density
- Comp - Compaction Test (Proctor)
- SG - Specific Gravity
- CBR - California Bearing Ratio
- RM - Resilient Modulus
- Perm - Permeability
- TXP - Triaxial Permeability
- Cons - Consolidation
- Chem - Analytical Chemical Analysis
- Corr - Corrosion
- VS - Vane Shear
- DS - Direct Shear
- UC - Unconfined Compression
- TX - Triaxial Compression
- UU - Unconsolidated, Undrained
- CU - Consolidated, Undrained
- CD - Consolidated, Drained

Notes:

1. Sample descriptions in this report are based on visual field and laboratory observations, which include density/consistency, moisture condition, grain size, and plasticity estimates, and should not be construed to imply field or laboratory testing unless presented herein. Visual-manual classification methods in accordance with ASTM D 2488 were used as an identification guide. Where laboratory data are available, soil classifications are in general accordance with ASTM D 2487.

2. Dual symbols are used to indicate gravel and sand units with 5 to 12 percent fines.

3. WOR = weight of rod.

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Silicon Valley Clean Water
Influent Connector
Redwood City, California

Project No: 76558-111593 Figure: A1-1

SOIL CLASSIFICATION/LEGEND SVCW.GPJ GINT STD US LAB.GDT 7/8/16 REV.

Boring Log CDM-1

Other Tests	Percent Passing # 200 Sieve	Liquid Limit.	Plastic Limit	Plasticity Index	Moisture Content	Dry Density (pcf)	Penetration Resistance (blows / 6 in.)	Depth (feet)	Sample	USCS	Symbol	DESCRIPTION	Elev. (feet)
												Asphalt	
	10.9				11				G	FILL		Well graded SAND with silt & gravel (SW-SM), gray, moist, subangular fine to coarse sand & fine gravel and crushed asphalt, asphalt base course (Fill).	
												No sample recovery.	100
							11 11 5	5					
													95
							0 0 0	10				No soil SPT sample, gravel in the tip of sampler.	
												Fat CLAY (CH), dark gray, very soft, wet, shattered shells (Younger Bay Mud).	90
							0 0 0	15					
													85
		89.7			91 103 88	50.2 57.6 48.7	0 0 0	20					
													80
							0 0 0	25		CH			
Con=1500 psf					92	48.4						Grading green-gray.	75
		95	34	61			0 0 0	30					
													70
							0 0 0	35					
												No sample recovery.	65
								40					
												Boring terminated at 42 feet below ground surface.	60

Northern/Easting: /
Surface Elevation: 103'
Logged By: DAW

Drill Rig: Track-mounted B-57
Equipment/Hammer: Mud Rotary/140 lbs
Date Completed: 3-17-16

Con: One Diamential Consolidation Test
UU: Unconfined-Undrained Triaxial Test

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Silicon Valley Clean Water
Influent Connector
Redwood City, California

Boring Log CDM-1
Project No: 76558-111593

Figure: A1-2

Boring Log CDM-2

Other Tests	Percent Passing # 200 Sieve	Liquid Limit.	Plastic Limit	Plasticity Index	Moisture Content	Dry Density (pcf)	Penetration Resistance (blows / 6 in.)	Depth (feet)	USCS	DESCRIPTION	Elev. (feet)
								0		Asphalt	103
	15.5				6				FILL	Silty SAND with gravel (SM), gray, moist, subangular fine to coarse sand & fine gravel and crushed asphalt, asphalt base course (Fill).	
							8 4 5		CH	Fat CLAY (CH), dark gray, stiff to very soft, wet, trace fine to coarse sand & fine gravel, occasional organcis, scattered shells, iron-oxide stains, (Younger Bay Mud).	100
							5 2 1	5		Grading soft, trace fine sand.	
							0 0 0	10		Grading very soft, trace gravel possibly sluff.	95
		103	37	66	80	52.7	0 0 0	15			90
UU=330 psf					87	50.1					85
							0 0 0	20			
					62	59.1	0 0 0	25			80
							0 0 0	30			75
UU=485 psf		70	34	36	80	52.5			MH	Elastic SILT (MH), gray, very soft, wet, trace fine sand, scattered shells (Younger Bay Mud).	70
							0 0 0	35			65
							0 0 0	40			
								41.5		Boring terminated at 41.5 feet below ground surface.	60

Northern/Easting: /
Surface Elevation: 103'
Logged By: DAW

Drill Rig: Track-mounted B-57
Equipment/Hammer: Mud Rotary/140 lbs
Date Completed: 3-16-16

Con: One Diamential Consolidation Test
UU: Unconfined-Undrained Triaxial Test

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Silicon Valley Clean Water
Influent Connector
Redwood City, California

Boring Log CDM-2 Figure: A1-3
Project No: 76558-111593 1 of 1

SVCW LOG OF BORING SVCW.GPJ GINT STD US LAB.GDT 7/8/16 REV.

Boring Log CDM-3

Other Tests	Percent Passing # 200 Sieve	Liquid Limit.	Plastic Limit	Plasticity Index	Moisture Content	Dry Density (pcf)	Penetration Resistance (blows / 6 in.)	Depth (feet)	USCS	DESCRIPTION	Elev. (feet)
								0		Asphalt	103
									FILL	Gravelly SAND (SP), gray, moist, subangular fine to coarse sand & fine gravel, asphalt base course (Fill).	
									CH	Fat CLAY (CH), dark gray, very stiff to very soft, wet (Younger Bay Mud).	100
							11 11 5	5			
											95
					87	50.1	0 0 0	10		Grading very soft.	
											90
		98	36	62	92		0 0 0	15		Occasional organics & scattered shells.	
											85
							0 0 0	20			
											80
							0 0 0	25			
											75
							0 0 0	30			
											70
							0 0 0	35			
											65
								40			
										Boring terminated at 41.5 feet below ground surface.	60

Northern/Easting: /
Surface Elevation: 103'
Logged By: DAW

Drill Rig: Track-mounted B-57
Equipment/Hammer: Mud Rotary/140 lbs
Date Completed: 3-18-16

Con: One Diamential Consolidation Test
UU: Unconfined-Undrained Triaxial Test

CDM Smith

Silicon Valley Clean Water
Influent Connector
Redwood City, California

Boring Log CDM-3 Figure: A1-4
Project No: 76558-111593 1 of 1

Boring Log CDM-4

Other Tests	Percent Passing # 200 Sieve	Liquid Limit.	Plastic Limit	Plasticity Index	Moisture Content	Dry Density (pcf)	Penetration Resistance (blows / foot)	Depth (feet)	Sample	USCS	Symbol	DESCRIPTION	Elev. (feet)
												Asphalt	
										FILL		Gravelly SAND (SP), gray, moist, subangular fine to coarse sand & fine gravel, asphalt base course, fabric at 2 feet (Fill).	
										CH		Fat CLAY (CH), dark gray, very soft, wet, occasional organics, scattered shells, (Younger Bay Mud). Trace fine gravel between 3 and 5 feet.	100
							0 0 0	5				Grading very soft.	
													95
							2 1 1	10				Grading slightly sandy	
										MH		Elastic SILT (MH), green-gray, very soft, wet, scattered shells.	90
Con=1375 psf		90	40	50	100	45.3							
							0 0 0	15					
													85
							0 0 0	20				Scattered shells.	
													80
UU=425 psf					77	54.9							
							0 0 0	25					
													75
							0 0 0	30					
													70
							0 0 0	35					
													65
							0 0 0	40					
												Boring terminated at 41.5 feet below ground surface.	60

Northern/Easting: /
Surface Elevation: 103'
Logged By: DAW

Drill Rig: Track-mounted B-57
Equipment/Hammer: Mud Rotary/140 lbs
Date Completed: 3-16-16

Con: One Diamential Consolidation Test
UU: Unconfined-Undrained Triaxial Test

CDM Smith

Silicon Valley Clean Water
Influent Connector
Redwood City, California

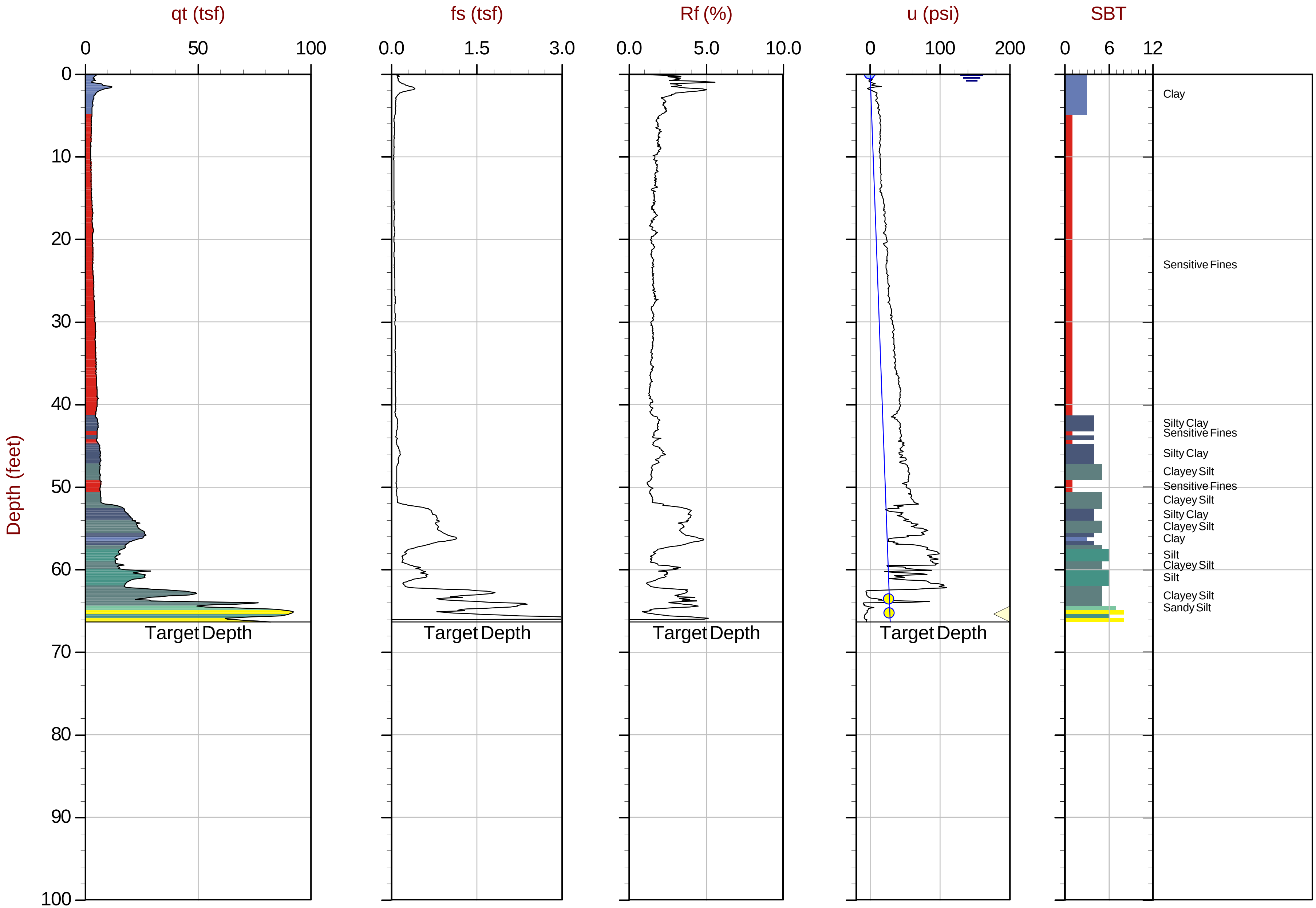
Boring Log CDM-4
Project No: 76558-111593

Figure: A1-5

CALIFORNIA PUSH TECHNOLOGIES INCORPORATED

DCM Consulting, Inc.

Job No: 15-56018
Date: 11:05:15 12:23
Site: SVCW Front of Plant Improvements
Sounding: CPT-24
Cone: 413:T375F10U200



Max Depth: 20.225 m / 66.35 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: 0.150 m

File: 15-56018_CP24.COR
Unit Wt: SBT Zones

SBT: Robertson and Campanella, 1986
Coords: WGS84 10S N: 4155434m E: 568091m
Sheet No: 1 of 1

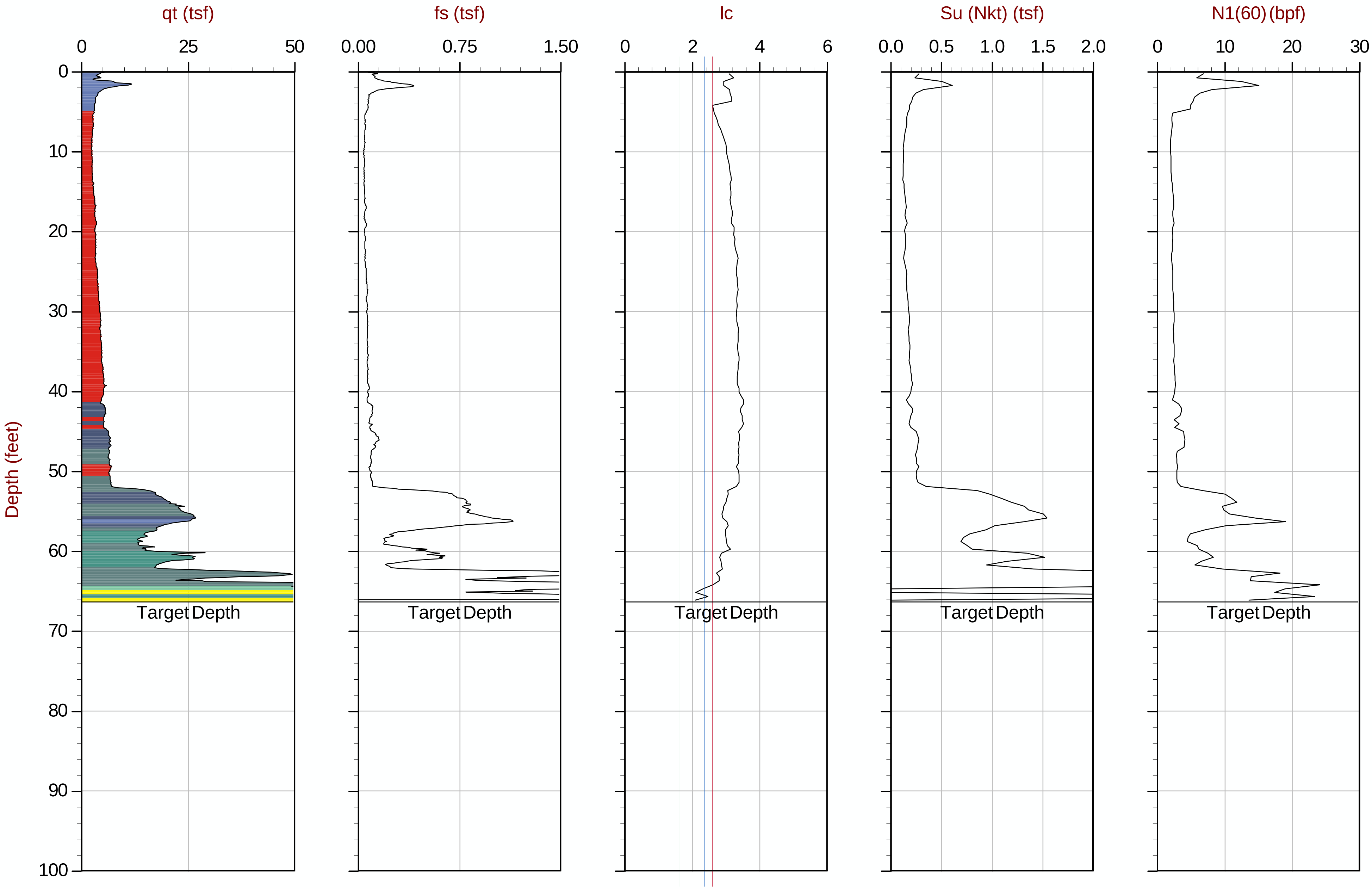
Equilibrium Pore Pressure (Ueq) Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

Figure A2-1

DCM Consulting, Inc.

Job No: 15-56018
Date: 11:05:15 12:23
Site: SVCW Front of Plant Improvements
Sounding: CPT-24
Cone: 413:T375F10U200



Max Depth: 20.225 m / 66.35 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: 0.150 m

File: 15-56018_CP24.COR
Unit Wt: SBT Zones
Su Nkt: 15.0

SBT: Robertson and Campanella, 1986
Coords: WGS84 10S N: 4155434m E: 568091m
Sheet No: 1 of 1

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

Figure A2-1
Page 2 of 2

DCM CONSULTING

Site: SVCW INFUENT

Sounding: CPT-3

Engineer: D.MATHY

Date: 5/9/2014 02:01

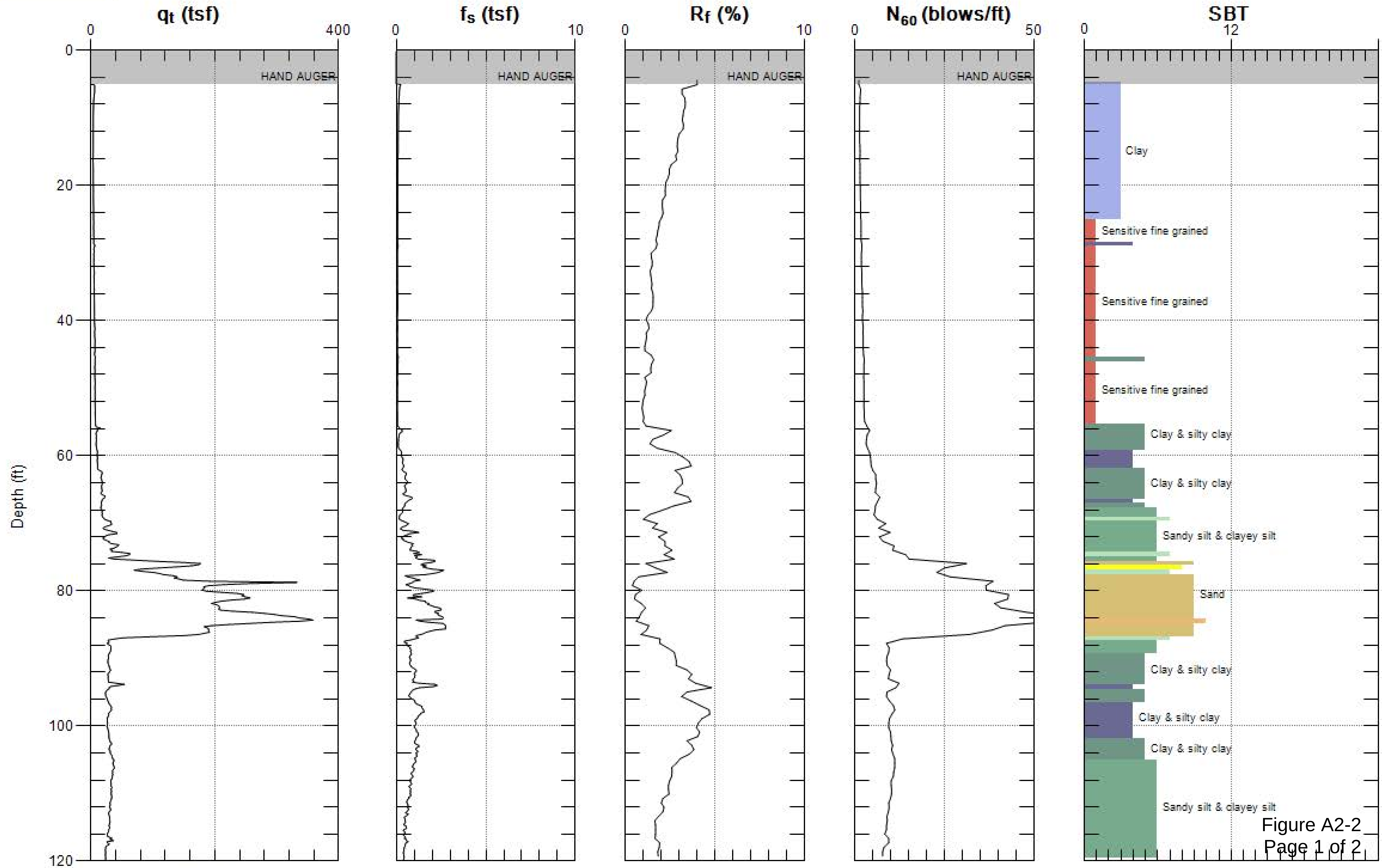


Figure A2-2
Page 1 of 2

Max. Depth: 120.079 (ft)
Avg. Interval: 0.656 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG DCM CONSULTING

Site: SVCW INFUENT

Sounding: CPT-3

Engineer: D.MATHY

Date: 5/9/2014 02:01

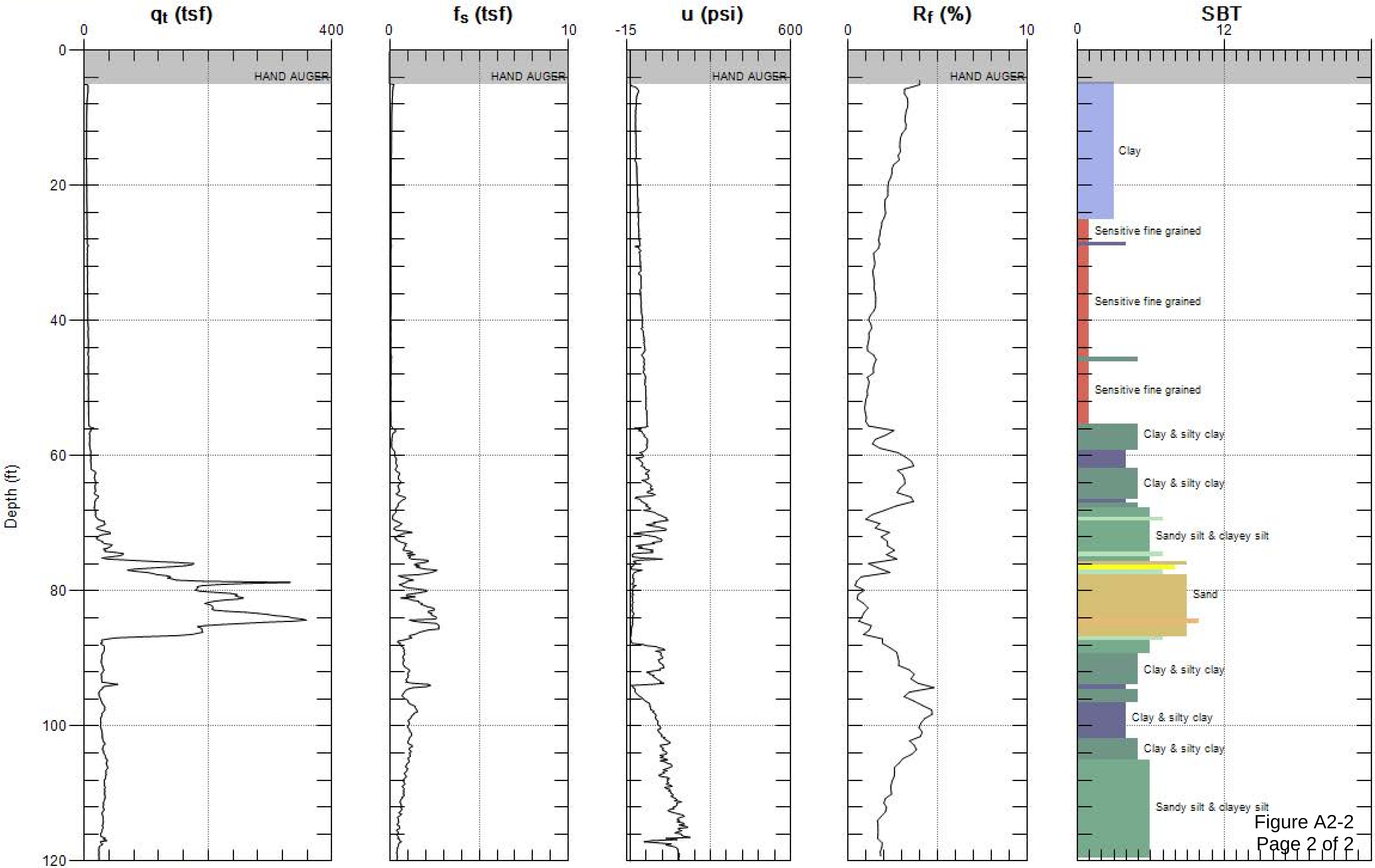


Figure A2-2

Page 2 of 2

Max. Depth: 120.079 (ft)

Avg. Interval: 0.656 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG

DCM CONSULTING

Site: SVCW INFUENT

Sounding: CPT-2

Engineer: D.MATHY

Date: 5/9/2014 10:57

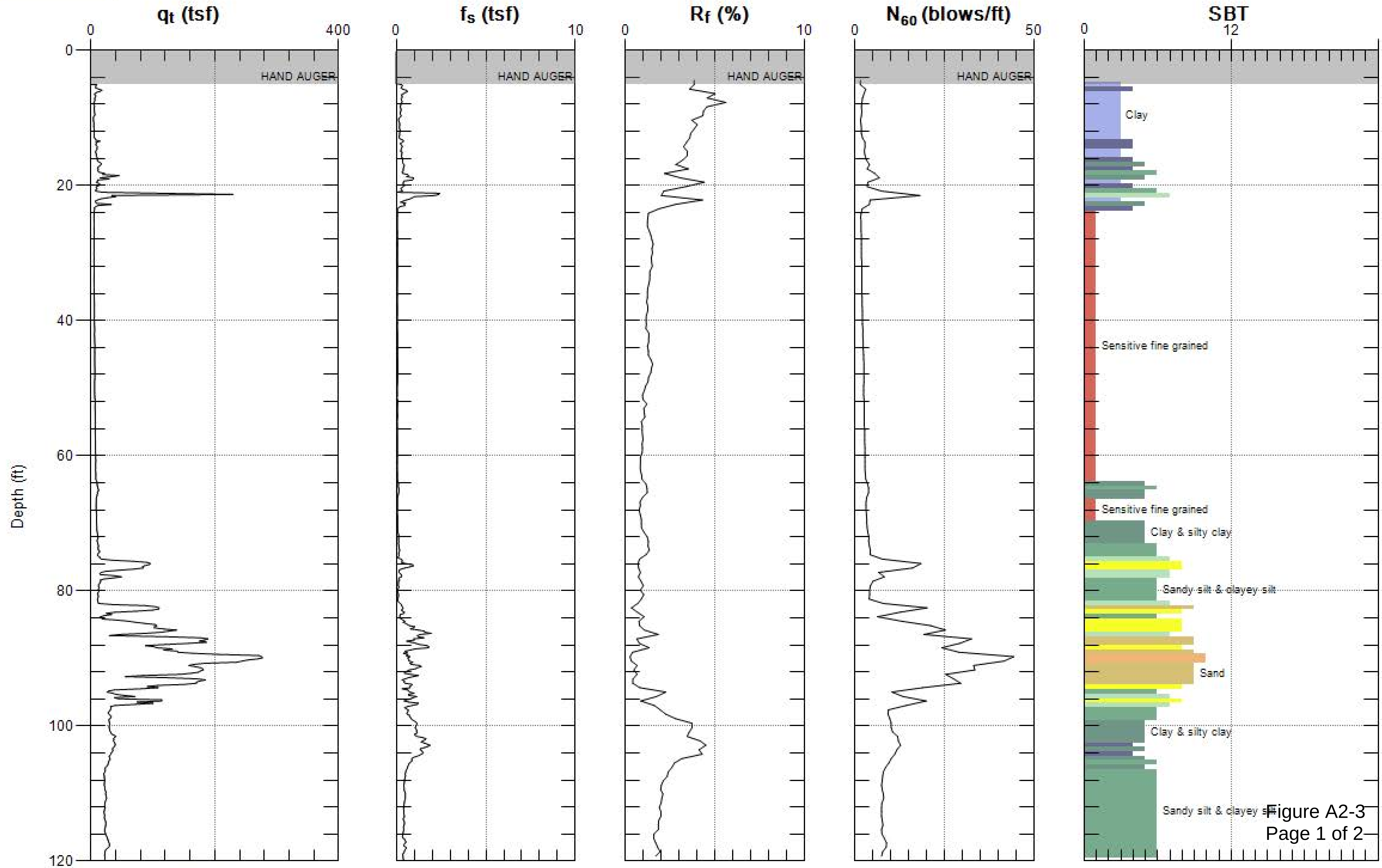


Figure A2-3
Page 1 of 2

Max. Depth: 120.079 (ft)
Avg. Interval: 0.656 (ft)

SBT: Soil Behavior Type (Robertson 1990)

DCM CONSULTING

Site: SVCW INFUENT

Sounding: CPT-2

Engineer: D.MATHY

Date: 5/9/2014 10:57

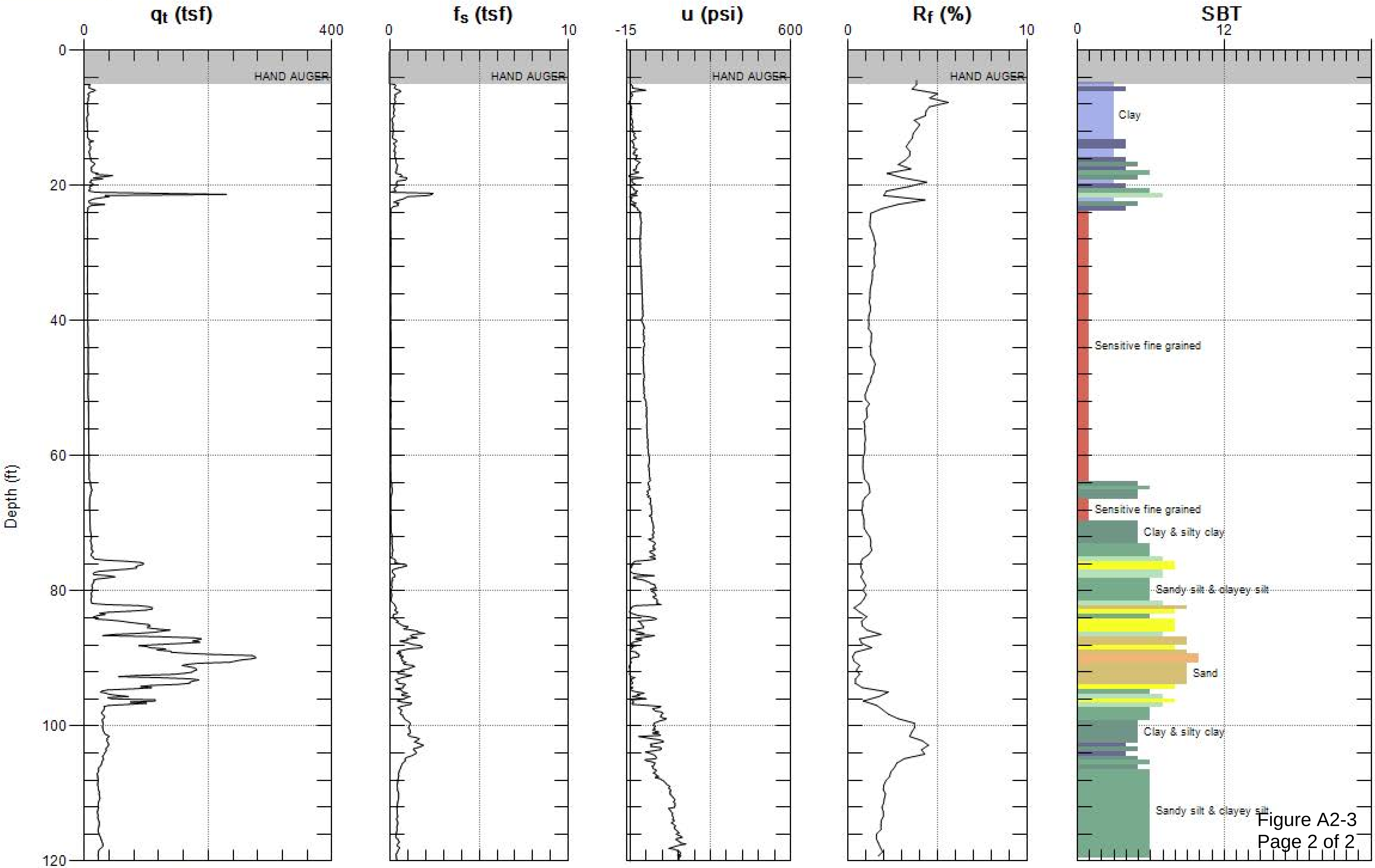


Figure A2-3
Page 2 of 2

Max. Depth: 120.079 (ft)
Avg. Interval: 0.656 (ft)

SBT: Soil Behavior Type (Robertson 1990)

Table 1 - Summary of Subsurface Soil and Groundwater Conditions at Shallow Borings

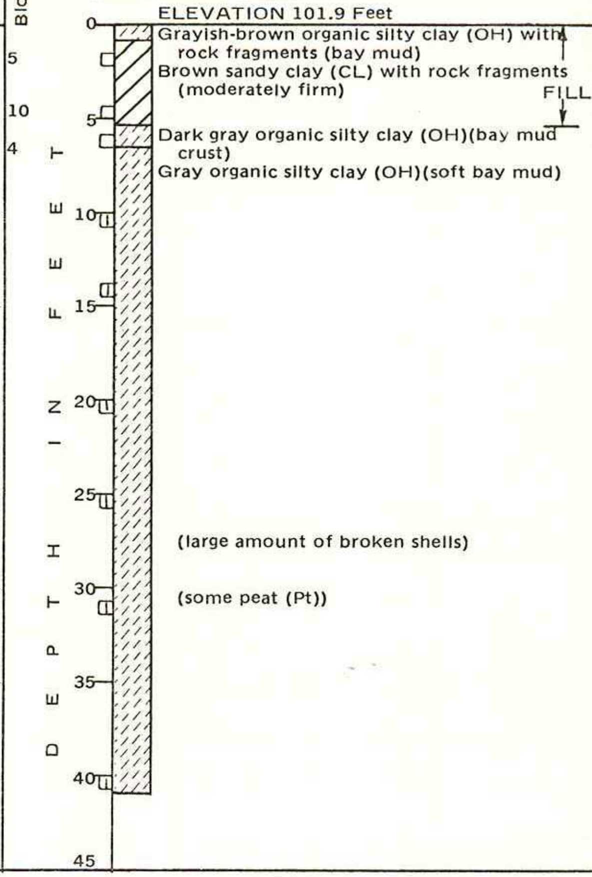
Shallow Boring #	Boring Depth	Asphalt Thickness	Fill			Young Bay Mud	Groundwater Depth[5]
			Thickness	USCS[3]	N_{SPT}[4]	N_{SPT}[4] (Depth)	
SB-1	5 ft	4 in	1.5 ft[1]	SC	NT[2]	15* (2½ ft) 11 (4 ft)	1.5 ft
SB-2	6.5 ft	-	4 ft	CL	6*	4 (4 ft) 3* (5½)	1 ft
SB-3	5 ft	4 in	1.5 ft	GW (AB)	NT	14* (2½ ft) 5 (4 ft)	NE
SB-4	5 ft	4 in	1.5 ft	SC	NT	15* (2½ ft) 2 (4 ft)	NE

[1] Thickness includes aggregate base rock.
[2] NT = Not taken.
[3] USCS – Unified Soil Classification System (see Method of Soil Classification in Appendix A).
[4] Standard Penetration Test Blow Count (*Modified California Sample blow count reduced by factor of 0.7).
[5] Borings were not open long enough to determine equilibrium groundwater level. NE = Not Encountered during duration of drilling.
[6] First attempt of drilling encountered possible obstruction at 3½ feet. Boring moved 7 feet to the northeast and redrilled.

BORING B-7

ELEVATION 101.9 Feet

CLASSIFICATION DATA: % FINES (-NO. 200)	LIQUID LIMIT	PLASTICITY INDEX	STRENGTH DATA: TYPE OF STRENGTH TEST	TEST SURCHARGE PRESSURE, LBS/SQ FT	TEST MOISTURE CONTENT, %	SHEAR STRENGTH, LBS/SQ FT	MOISTURE-DENSITY DATA: NATURAL MOISTURE CONTENT, %	Field Dry Density pcf	Blows/ft.
							15.6	134/116	5
			DS	500	Natural	600	30.6	120/92	10
			DS	600	Natural	500	67.7	99/59	4
			DS	700	Natural	270	102.3	89/44	
			DS	800	Natural	280	98.6	84/42	
			DS	1000	Natural	350	85.3	94/51	
			DS	1200	Natural	400	90.1	92/48	
			DS	1400	Natural	430	91.3	90/47	
							86.6	91/49	



FIELD NOTES:

1. Borings B-7 through B-10 were drilled on April 7 & 8, 1981 with truck-mounted, 5-inch-diameter, rotary-wash equipment.
2. The following symbol, ☐, denotes an undisturbed sample taken in a 2½-inch-diameter, split-tube barrel driven into the soil by 340-pound slip jars falling 15± inches inside the boring.
3. The following symbol, ☐, denotes an undisturbed sample taken in a 2½-inch-diameter, 16-gauge, Shelby tube pushed into the soil.
4. The following symbol, ☐, denotes a bulk (disturbed) sample.
5. Boring elevations are relative to MSL Datum.
6. The following symbol, ▽, denotes the groundwater level at the time and depth shown on the boring logs.

LABORATORY NOTES AND ABBREVIATIONS:

The tabulated shear strengths are yield point values.

DS = Strain controlled direct shear test at natural moisture content.

DSX = Strain controlled direct shear test after the sample had been submerged in water until movement ceased under a surcharge equal to the test surcharge.

Revisions: By___ Date___
By___ Date___
Location San Mateo, CA

By BSK Date 4/8/81
Checked By___
Job Number 1726-A3 Name SBSA

BORING LOG

3151808

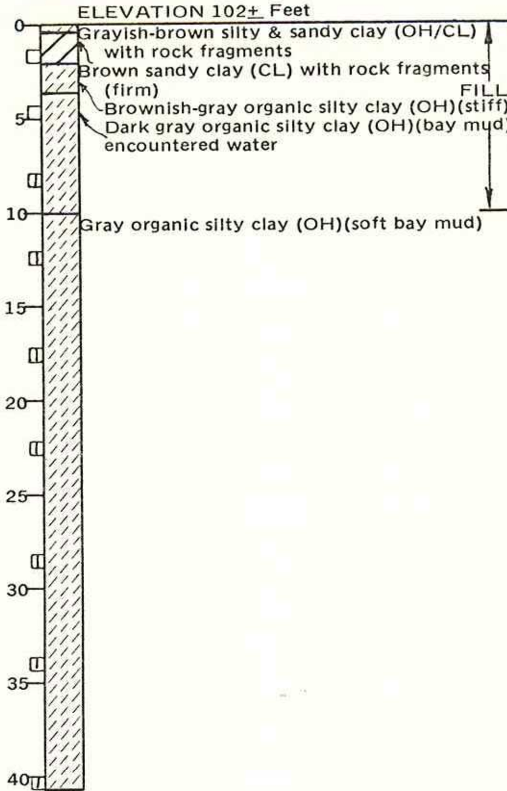
Cooper & Clark

BORING B-8

ELEVATION 102± Feet

Classification Data: % Fines (-No. 200)	Classification Data: Liquid Limit	Classification Data: Plasticity Index	Strength Data: Type of Strength Test	Strength Data: Test Surcharge Pressure, Lbs/Sq Ft	Strength Data: Test Moisture Content, %	Strength Data: Shear Strength, Lbs/Sq Ft	Moisture-Density Data: Natural Moisture Content, %	Moisture-Density Data: Field Dry Density pcf	Blows/ft.
			DS	300	Natural	850	34.8	109/81	11
			DS	500	Natural	440	54.8	99/64	2
			DS	600	Natural	500	56.6	100/64	
			DS	700	Natural	300	95.3	91/47	
			DS	900	Natural	290	91.8	90/47	
			DS	1000	Natural	340	88.5	93/49	
			DS	1200	Natural	440	83.7	90/49	
			DS	1400	Natural	470	84.5	95/51	
							89.4	91/48	

DEPTH IN FEET



Revisions: By___ Date___ By___ Date___ Location San Mateo, CA

By BSK Date 4/8/81 Checked By___ Job Number 1726-A3 Name SBSA

BORING LOG

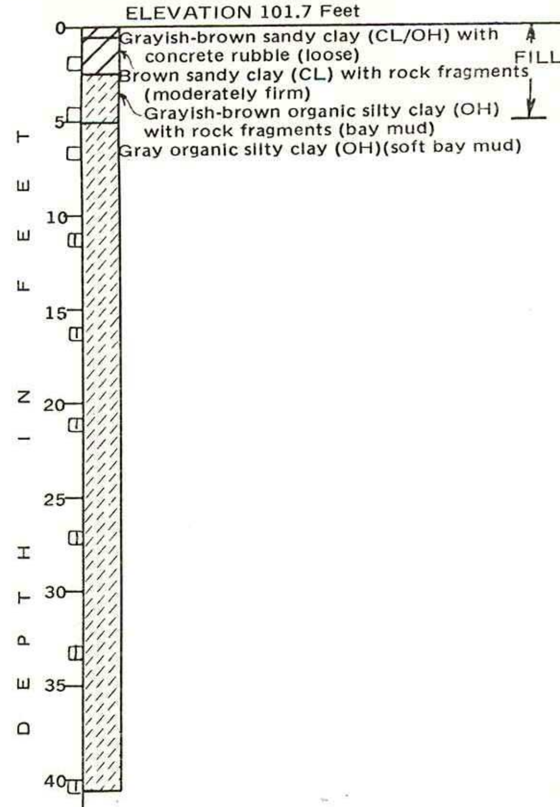
3151809

Cooper & Clark

BORING B-9

ELEVATION 101.7 Feet

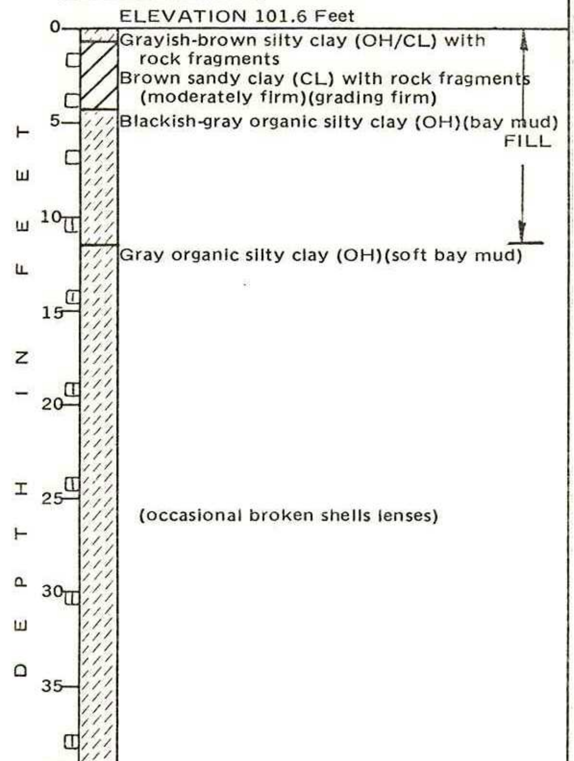
CLASSIFICATION DATA: % FINES (-NO. 200)	LIQUID LIMIT	PLASTICITY INDEX	STRENGTH DATA: TYPE OF STRENGTH TEST	TEST SURCHARGE PRESSURE, LBS/SQ FT	TEST MOISTURE CONTENT, %	SHEAR STRENGTH, LBS/SQ FT	MOISTURE-DENSITY DATA: NATURAL MOISTURE CONTENT, %	Field Dry Density pcf	Blows/ft.
							11.1	130/117	12
			DS	500	Natural	650	60.7	101/63	5
			DS	500	Natural	280	82.9	95/52	1
			DS	700	Natural	250	102.9	87/43	
			DS	800	Natural	310	95.1	91/46	
			DS	1000	Natural	310	94.1	92/48	
			DS	1200	Natural	470	82.7	95/52	
							80.9	92/51	
			DS	1600	Natural	600	79.4	94/52	



BORING B-10

ELEVATION 101.6 Feet

CLASSIFICATION DATA: % FINES (-NO. 200)	LIQUID LIMIT	PLASTICITY INDEX	STRENGTH DATA: TYPE OF STRENGTH TEST	TEST SURCHARGE PRESSURE, LBS/SQ FT	TEST MOISTURE CONTENT, %	SHEAR STRENGTH, LBS/SQ FT	MOISTURE-DENSITY DATA: NATURAL MOISTURE CONTENT, %	Field Dry Density pcf	Blows/ft.
			DS	300	Natural	550	13.5	137/121	6
			DS	400	Natural	600	32.4	112/85	6
			DS	600	Natural	600	57.0	99/63	5
			DS	700	Natural	400	63.2	96/59	
			DS	800	Natural	260	101.6	88/44	
			DS	1000	Natural	370	96.3	88/45	
			DS	1100	Natural	340	94.3	90/47	
			DS	1300	Natural	470	81.9	95/52	
			DS	1500	Natural	520	84.5	92/50	



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By____ Date____
By____ Date____
Location San Mateo, CA

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Checked By____
Job Number 1726-A3 Name SBSA

BORING LOGS

Figure 4-3
Page 1 of 1

3151810

Cooper & Clark

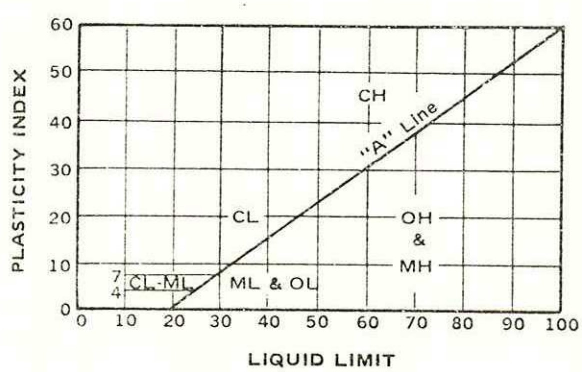
MAJOR DIVISIONS			SYMBOLS	TYPICAL NAMES
COARSE GRAINED SOILS (More than ½ of soil > no. 200 sieve size)	GRAVELS (More than ½ of coarse fraction > no. 4 sieve size)		GW	Well graded gravels or gravel-sand mixtures, little or no fines
			GP	Poorly graded gravels or gravel-sand mixtures, little or no fines
			GM	Silty gravels, gravel-sand-silt mixtures
			GC	Clayey gravels, gravel-sand-clay mixtures
	SANDS (More than ½ of coarse fraction < no. 4 sieve size)		SW	Well graded sands or gravelly sands, little or no fines
			SP	Poorly graded sands or gravelly sands, little or no fines
			SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures
FINE GRAINED SOILS (More than ½ of soil < no. 200 sieve size)	SILTS & CLAYS	LL < 50	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
			OL	Organic silts and organic silty clays of low plasticity
	SILTS & CLAYS	LL > 50	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic clays of medium to high plasticity, organic silty clays, organic silts
HIGHLY ORGANIC SOILS			Pt	Peat and other highly organic soils

CLASSIFICATION CHART

(Unified Soil Classification System)

CLASSIFICATION	RANGE OF GRAIN SIZES	
	U.S. Standard Sieve Size	Grain Size in Millimeters
BOULDERS	Above 12"	Above 305
COBBLES	12" to 3"	305 to 76.2
GRAVEL	3" to No. 4	76.2 to 4.76
coarse	3" to ¾"	76.2 to 19.1
fine	¾" to No. 4	19.1 to 4.76
SAND	No. 4 to No. 200	4.76 to 0.074
coarse	No. 4 to No. 10	4.76 to 2.00
medium	No. 10 to No. 40	2.00 to 0.420
fine	No. 40 to No. 200	0.420 to 0.074
SILT & CLAY	Below No. 200	Below 0.074

GRAIN SIZE CHART



PLASTICITY CHART

Revisions:
By___ Date___
By___ Date___
Location San Mateo, CA

By BSK Date___
Checked By___
Job Number 1726-A3 Name SBSA

151811

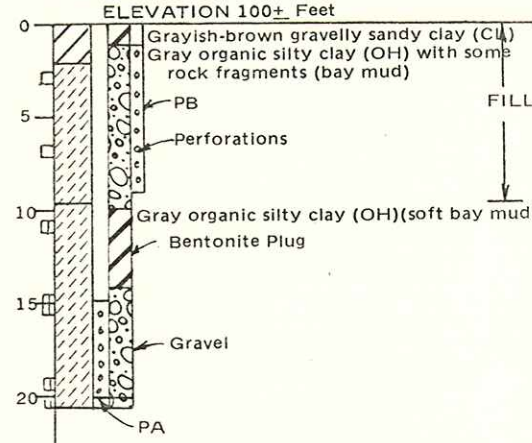
METHOD OF SOIL CLASSIFICATION

Cooper & Clark

GROUNDWATER LEVEL DATA			STRENGTH DATA				MOISTURE-DENSITY DATA	
DATE	ELEVATION PA	ELEVATION PB	TYPE OF STRENGTH TEST	TEST SURCHARGE PRESSURE, LBS/SQ FT	TEST MOISTURE CONTENT, %	SHEAR STRENGTH, LBS/SQ FT	NATURAL MOISTURE CONTENT, %	FIELD DRY DENSITY LBS/CU FT
8/2/80	97.8	Dry	DS	300	Natural	1450	31.2	114/87
8/26	97.5	Dry	DS	400	Natural	600	43.6	112/78
8/10	97.5	Dry						
9/22	97.6	Dry					86.3	93/50
9/29	97.6	Dry					93.5	91/47
10/13	97.5	Dry					99.0	90/45
							74.5	89/51
							77.9	94/53

BORING B-1

ELEVATION 100± Feet



FIELD NOTES:

1. The borings were drilled on August 8 and 11, 1980 with truck-mounted, power-driven, 5-inch-diameter, helical auger equipment.
2. The following symbol,☐, denotes an undisturbed sample taken in a 2½-inch-diameter, split-tube barrel driven into the soil by 245-pound slip jars falling 18± inches inside the boring.
3. The following symbol,◫, denotes an undisturbed sample taken in a 2½-inch-diameter, 16-gauge, Shelby tube pushed into the soil.
4. Boring elevations were estimated by interpolation between the plot plan spot elevations.

LABORATORY NOTES AND ABBREVIATIONS:

The tabulated shear strengths are yield point values.

DS = Strain controlled direct shear test at natural moisture content.

BORING LOG

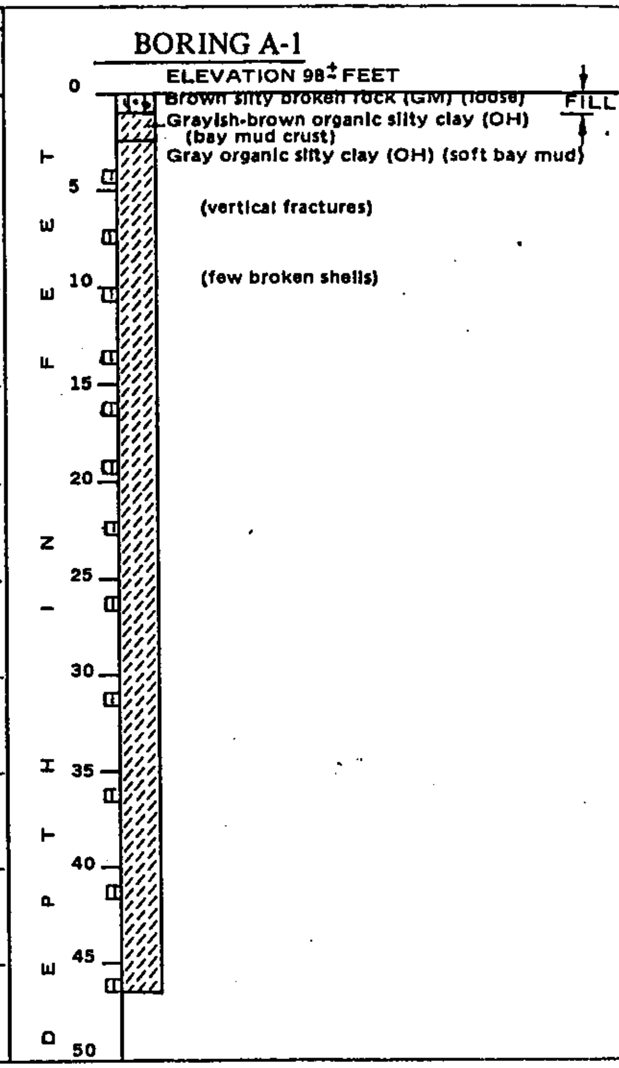
Revisions: By ____ Date ____ By ____ Date ____ Location Redwood City

By BSK Date ____ Checked By RK 11/5/80 Job Number 1726-A6 Name SBSA

3151831

Cooper & Clark

CLASSIFICATION DATA			STRENGTH DATA				MOISTURE-DENSITY DATA	
% FINES (-NO. 200)	LIQUID LIMIT	PLASTICITY INDEX	TYPE OF STRENGTH TEST	TEST SURCHARGE PRESSURE, LBS/SQ FT	TEST MOISTURE CONTENT, %	SHEAR STRENGTH, LBS/SQ FT	NATURAL MOISTURE CONTENT, %	DRY DENSITY, LBS/CU FT
			DS	300	Natural	300	82.1	51
			DS	400	Natural	250	96.2	46
			DS	500	Natural	250	103.0	44
			DS	600	Natural	340	90.5	47
							114.5	42
			DS	800	Natural	350	87.1	49
							90.6	46
			DS	900	Natural	320	100.2	45
			DS	1100	Natural	435	87.5	50
			DS	1200	Natural	455	83.4	53
							95.9	48
			DS	1500	Natural	615	71.6	56



FIELD NOTES

1. The borings were drilled on September 28 through 30 and October 2 through 6, 1978 with truck-mounted, 5-inch-diameter, rotary-wash equipment.
2. The following symbol, ◫ , denotes an undisturbed sample taken in 2½-inch-diameter, 16 gage, Shelby tube, pushed into the soil.
3. Boring elevations were estimated by interpolation between contours of photography dated 9/28/78.

LABORATORY NOTES AND ABBREVIATIONS

The tabulated shear strengths are maximum values.

DS = Strain controlled direct shear test at natural moisture content.

Revisions:
By ______ Date ______
By ______ Date ______
Location Redwood City, California

By BSK/te Date 10/18/78
Checked By [illegible] Date [illegible]
Job Number 1726-A5 Name South Bayside Authority

BORING LOG

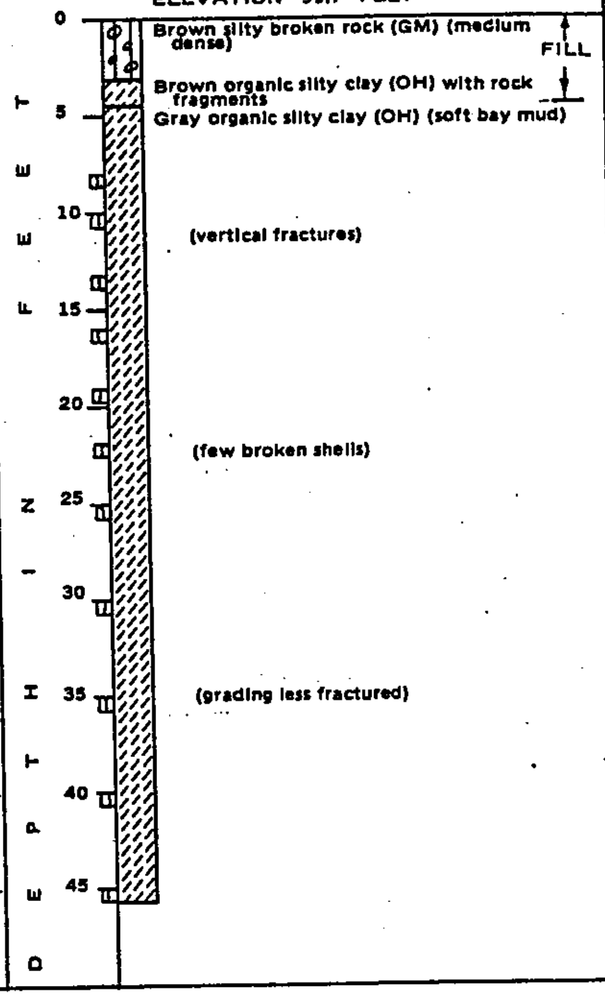
3152258

Figure A6-1
 [redacted]

CLASSIFICATION DATA			STRENGTH DATA				MOISTURE-DENSITY DATA	
% FINES (-NO. 200)	LIQUID LIMIT	PLASTICITY INDEX	TYPE OF STRENGTH TEST	TEST SURCHARGE PRESSURE, LBS/SQ FT	TEST MOISTURE CONTENT, %	SHEAR STRENGTH, LBS/SQ FT	NATURAL MOISTURE CONTENT, %	DRY DENSITY, LBS/CU FT
			DS	300	Natural	290	94.7	47
			DS	400	Natural	215	102.4	43
			DS	400	Natural	250	101.4	44
			DS	600	Natural	280	94.9	46
			DS	600	Natural	320	100.5	45
			DS	700	Natural	280	97.2	46
			DS	800	Natural	365	81.3	50
			DS	900	Natural	405	75.6	52
			DS	1000	Natural	435	79.9	53
			DS	1300	Natural	560	67.1	62
			DS	1300	Natural	645	70.5	58

BORING A-7

ELEVATION 99.7± FEET



BORING LOG

Revisions:
By ________ Date ________
By ________ Date ________

By BSK Date 10/18/78
Checked By ________
Job Number 1726-A5 Name South Bayside Systems Authority Location Redwood City, California

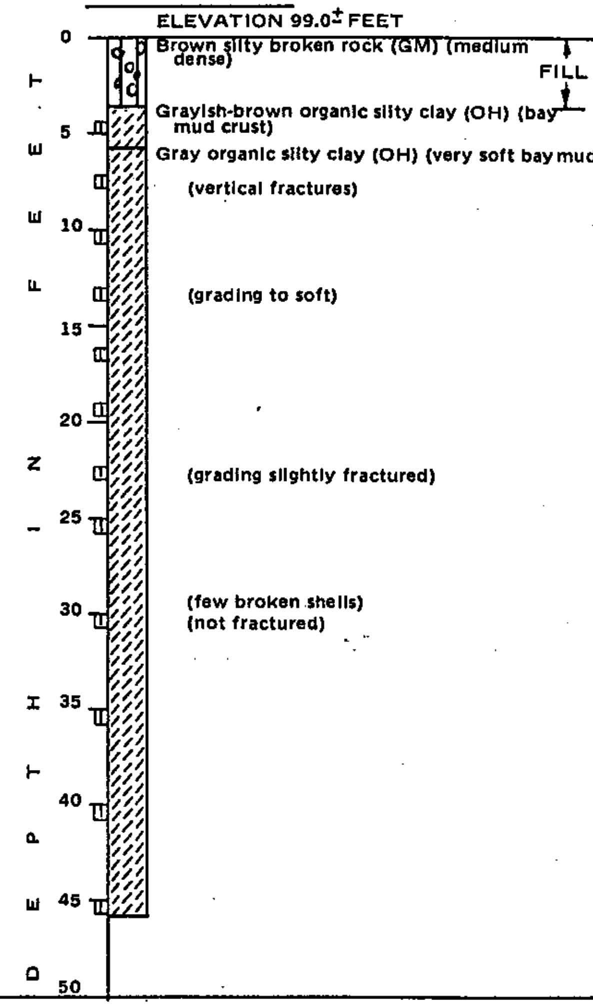
Figure A6-2
Page 1 of 1

COOPER · CLARK & ASSOCIATES

CLASSIFICATION DATA			STRENGTH DATA				MOISTURE-DENSITY DATA	
% FINES (-NO. 200)	LIQUID LIMIT	PLASTICITY INDEX	TYPE OF STRENGTH TEST	TEST SURCHARGE PRESSURE, LBS/SQ FT	TEST MOISTURE CONTENT, %	SHEAR STRENGTH, LBS/SQ FT	NATURAL MOISTURE CONTENT, %	DRY DENSITY, LBS/CU FT
			DS	300	Natural	780	53.9	66
			DS	300	Natural	115	101.3	45
			DS	400	Natural	240	110.1	43
			DS	400	Natural	250	101.2	44
							96.1	46
			DS	600	Natural	270	95.1	48
			DS	700	Natural	310	90.6	48
							88.8	48
			DS	1000	Natural	410	82.9	51
			DS	1300	Natural	470	82.9	50

BORING A-11

ELEVATION 99.0± FEET



Revisions:
By ______ Date ______
By ______ Date ______

By BSK Date 10/18/78
Checked By BK 10/18/78
Job Number 1726-A5 Name South Bayside Systems Authority Location Redwood City

BORING LOG

315225

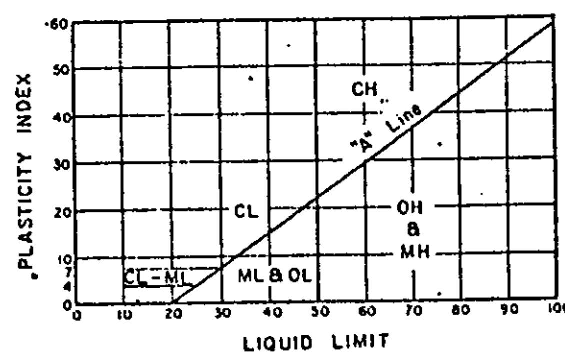
 [redacted]

MAJOR DIVISIONS		SYMBOLS	TYPICAL NAMES
COARSE GRAINED SOILS (More than 1/2 of soil > no. 200 sieve size)	GRAVELS (More than 1/2 of coarse fraction > no. 4 sieve size)	GW	Well graded gravels or gravel-sand mixtures, little or no fines
		GP	Poorly graded gravels or gravel-sand mixtures, little or no fines
		GM	Silty gravels, gravel-sand-silt mixtures
		GC	Clayey gravels, gravel-sand-clay mixtures
	SANDS (More than 1/2 of coarse fraction < no. 4 sieve size)	SW	Well graded sands or gravelly sands, little or no fines
		SP	Poorly graded sands or gravelly sands, little or no fines
		SM	Silty sands, sand-silt mixtures
		SC	Clayey sands, sand-clay mixtures
FINE GRAINED SOILS (More than 1/2 of soil < no. 200 sieve size)	SILTS & CLAYS LL < 50	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
		OL	Organic silts and organic silty clays of low plasticity
	SILTS & CLAYS LL > 50	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
		CH	Inorganic clays of high plasticity, fat clays
		OH	Organic clays of medium to high plasticity, organic silty clays, organic silts
HIGHLY ORGANIC SOILS		Pt	Peat and other highly organic soils

CLASSIFICATION CHART
(Unified Soil Classification System)

CLASSIFICATION	RANGE OF GRAIN SIZES	
	U.S. Standard Sieve Size	Grain Size in Millimeters
BOULDERS	Above 12"	Above 305
COBBLES	12" to 3"	305 to 76.2
GRAVEL	3" to No. 4	76.2 to 4.76
coarse	3" to 3/4"	76.2 to 19.1
fine	3/4" to No. 4	19.1 to 4.76
SAND	No. 4 to No. 200	4.76 to 0.074
coarse	No. 4 to No 10	4.76 to 2.00
medium	No. 10 to No. 40	2.00 to 0.420
fine	No. 40 to No. 200	0.420 to 0.074
SILT & CLAY	Below No. 200	Below 0.074

GRAIN SIZE CHART



PLASTICITY CHART

METHOD OF SOIL CLASSIFICATION

3151773

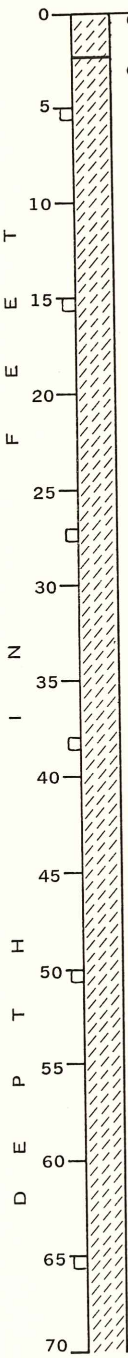
Figure A7-1
Page 1 of 1 [redacted]

PLATE A-2

BORING 3

ELEVATION 99.2 FEET

CLASSIFICATION DATA			STRENGTH DATA				MOISTURE-DENSITY DATA	
% FINES (-NO. 200)	LIQUID LIMIT	PLASTICITY INDEX	TYPE OF STRENGTH TEST	TEST SURCHARGE PRESSURE, LBS/SQ FT	TEST MOISTURE CONTENT, %	SHEAR STRENGTH, LBS/SQ FT	NATURAL MOISTURE CONTENT, %	DRY DENSITY, LBS/CU FT
			DS	300	Natural	260	97.6	43
			DS	600	Natural	350	83.9	48
							76.7	53
			DS	1300	Natural	570	79.9	52
			DS	1600	Natural	560	71.8	56
			DS	2100	Natural	900	32.1	87



Grayish-brown organic silty clay (OH) (bay mud crust)

Gray organic silty clay (OH) (soft bay mud)

Boring continued on Plate A-1G

Revisions:
By________ Date________
By________ Date________

By BSK Date 10/27/75
Checked By BK Date 10/29/75
Job Number 1726-A Name South Bayside System Authority Location Redwood City

BORING LOG

3151780

Figure A7-2
Page 1 of 3

Cooper & Clark

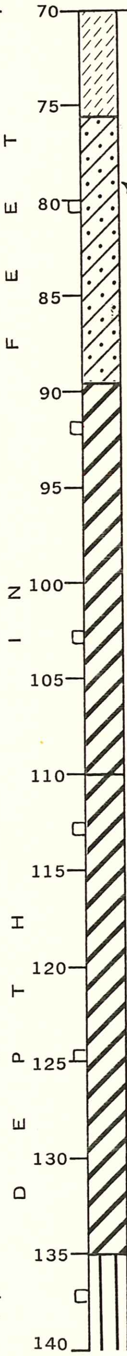
PLATE A-1F

(-74)

BORING 3 (Continued)

% FINES (-NO. 200)	LIQUID LIMIT	PLASTICITY INDEX	TYPE OF STRENGTH TEST	TEST SURCHARGE PRESSURE, LBS/SQ FT	TEST MOISTURE CONTENT, %	SHEAR STRENGTH, LBS/SQ FT	NATURAL MOISTURE CONTENT, %	DRY DENSITY, LBS/CU FT	BLOWS/FOOT
CLASSIFICATION DATA			STRENGTH DATA				MOISTURE-DENSITY DATA		
			DS DS	500 2700	Natural Natural	250 1450	20.5	100	
			DS	3000	Natural	1650	33.1	88	31
	72.2	46.0	DS	3000	Natural	2700	27.5	97	40
			DS	3000	Natural	1250	43.5	75	12
			DS	3000	Natural	1150	42.6	76	15
			DS DS	500 3000	Natural Natural	400 1650	22.8	102	34

DEPTH IN FEET



75 ft: Gray clayey sand (SC) (loose)
(Some organic)

80 ft: (Gas coming out from this depth)

90 ft: Grayish-brown silty clay (CH) (stiff)

110 ft: Gray silty clay (CH) (with some broken shells (firm)
(Old bay mud)

135 ft: Grayish-brown silt (ML) (firm)

Boring continued on Plate A-1H

BORING LOG

By BSK Date 10/27/75
Checked By RAH Date 10/29/75
Job Number 1726-A Name South Bayside System Authority Location Redwood City

Revisions:
By____ Date____
By____ Date____

3151782

Cooper & Clark

PLATE A-1G

BORING 3 (Continued)

% Fines (-No. 200)	Liquid Limit	Plasticity Index	Type of Strength Test	Test Surcharge Pressure, Lbs/Sq Ft	Test Moisture Content, %	Shear Strength, Lbs/Sq Ft	Natural Moisture Content, %	Dry Density, Lbs/Cu Ft	Blows/Foot
			DS	3000	Natural	2900	32.4	91	26
	79.5	52.4	DS	3000	Natural	2800	46.7	74	18
			DS	3000	Natural	2300	22.3	104	30
			DS DS	1000 3000	Natural Natural	800 1800	22.5	102	30
	39.7	22.4	DS	3000	Natural	2050	22.6	104	
			DS	3000	Natural	2700	26.1	98	35

Depth in feet: 140, 145, 150, 155, 160, 165, 170, 175, 180, 185, 190, 195, 200, 205

- Gray silty clay (CL) (stiff)
- Gray silty clay (CH) (stiff)
  (Old bay mud)
- Gray clayey sand (SC) (medium dense)
  (Few broken shells)
- Gray silty clay (CL) (stiff)
- Grayish-brown silty clay (CH) (stiff)

By BSK Date 10/27/75
Checked By BLC Date 10/27/75
Job Number 1726-A Name South Bayside System Authority Location Redwood City

Revisions:
By ______ Date ______
By ______ Date ______

BORING LOG

3151783

Figure A7-2
Page 3 of 3

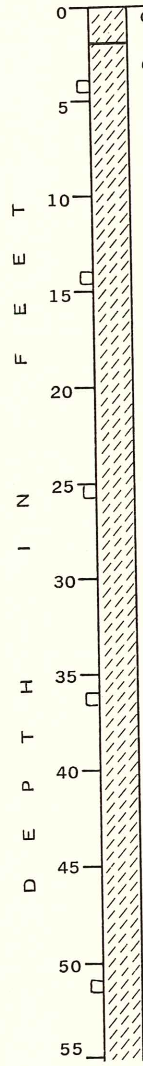
Cooper & Clark

PLATE A-1H

CLASSIFICATION DATA			STRENGTH DATA				MOISTURE-DENSITY DATA	
% FINES (-NO. 200)	LIQUID LIMIT	PLASTICITY INDEX	TYPE OF STRENGTH TEST	TEST SURCHARGE PRESSURE, LBS/SQ FT	TEST MOISTURE CONTENT, %	SHEAR STRENGTH, LBS/SQ FT	NATURAL MOISTURE CONTENT, %	DRY DENSITY, LBS/CU FT
			DS	300	Natural	340	90.9	48
			DS	600	Natural	300	102.8	43
							75.6	53
			DS	1200	Natural	520	83.0	49
			DS	1700	Natural	530	69.8	56

BORING 4

ELEVATION 99.2 FEET



Grayish-brown organic silty clay (OH) (bay mud crust)

Gray organic silty clay (OH) (soft bay mud)

Boring continued on Plate A-1J

Revisions:
By____ Date____
By____ Date____

By BSK Date 10/27/75
Checked By BM 10/27/75
Job Number 1726-A Name South Bayside System Authority Location Redwood City

BORING LOG

3151784

Figure A7-3
Page 1 of 2

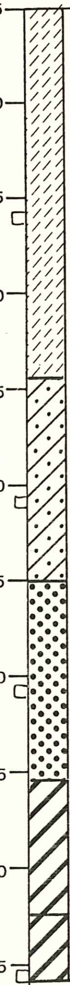
Cooper & Clark

PLATE A-1I

(4-74)

BORING 4 (Continued)

% Fines (-No. 200)	Liquid Limit	Plasticity Index	Type of Strength Test	Test Surcharge Pressure, lbs/sq ft	Test Moisture Content, %	Shear Strength, lbs/sq ft	Natural Moisture Content, %	Dry Density, lbs/cu ft	Blows/Foot	Depth in Feet	Description
										55	
										60	
			DS	2100	Natural	730	42.0	77		65	
										70	
										75	Gray clayey sand (SC) with bay mud (medium dense)
			DS	2700	Natural	750	23.6	102	12	80	
										85	Gray coarse sand (SP) (dense) (Piece of wood)
			DS	3000	Natural	3900	7.4	117	47	90	(Some gravel)
										95	Grayish-brown silty clay (CH) (stiff)
										100	Gray silty clay (CH) (firm)
			DS	3000	Natural	1100	40.0	78	40	105	(Old bay mud)
										110	



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By___ Date___

By BSK Date 10/27/75
Checked By BW Date 10/29/75
Job Number 1726-A Name South Bayside System Authority Location Redwood City

BORING LOG

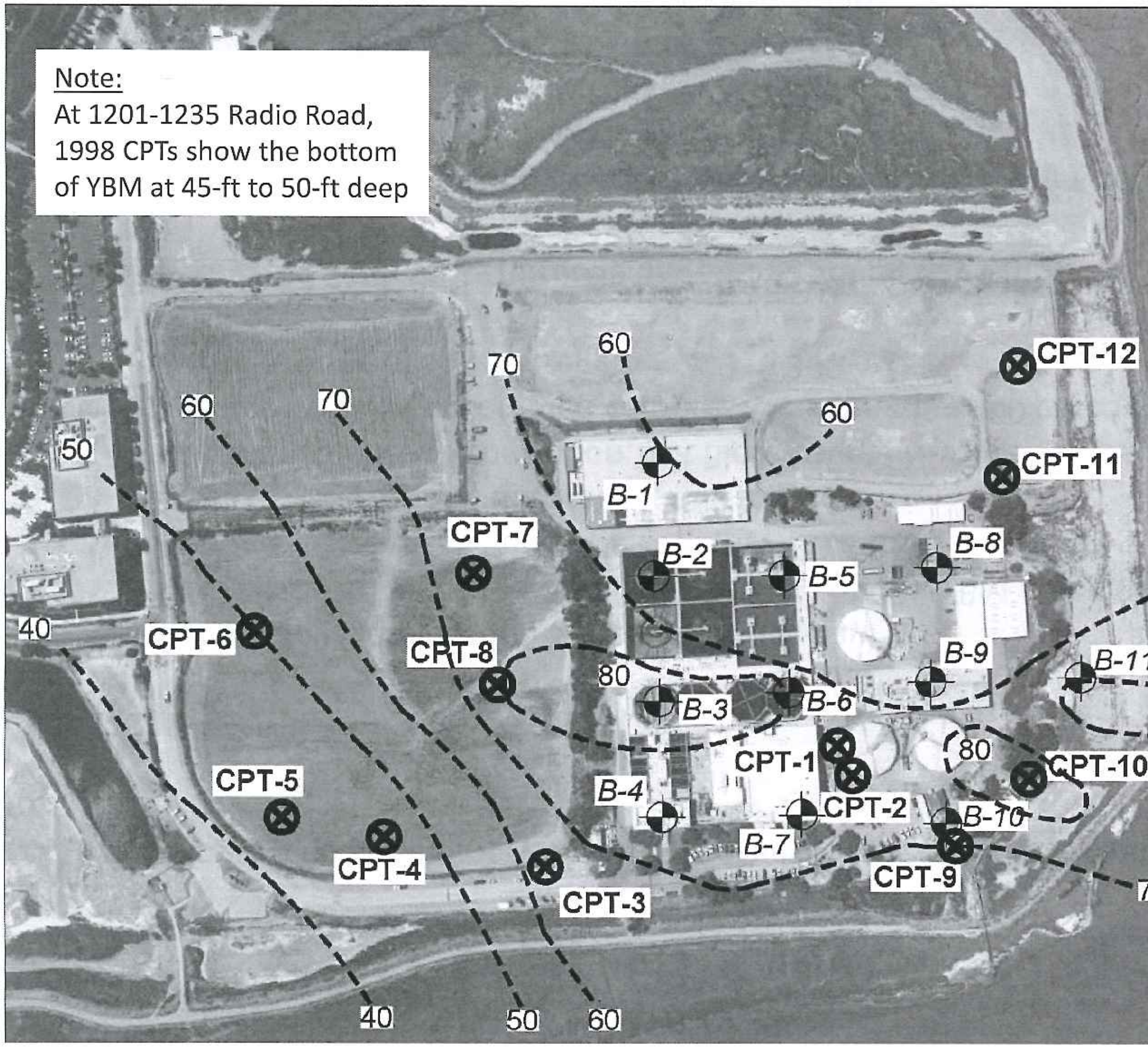
3151785

4-74)

Figure A7-3
Page 2 of 2

Cooper & Clark
PLATE A-IJ

Updated CPT/YBM Mapping



CPT Number	Approx Depth BGS to Bottom of YBM (ft)	Date of CPT
CPT-1	75	5/9/2014
CPT-2	75	5/9/2014
CPT-3	62	5/9/2014
CPT-4	45	7/15/2014
CPT-5	42	7/15/2014
CPT-6	50	7/15/2014
CPT-7	73	7/15/2014
CPT-8	80	7/15/2014
CPT-9	70	10/17/2014
CPT-10	84	10/17/2014
CPT-11	65	1/22/2015
CPT-12	63	1/22/2015

Boring Number	Approx Depth BGS to Bottom of YBM (ft)	Date of Boring
B-1	60	1975
B-2	65	1975
B-3	85	1975
B-4	75	1975
B-5	67	1975
B-6	80	1975
B-7	75	1975
B-8	65	1975
B-9	65	1975
B-10	70	1975
B-11	60	1975

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Appendix B

Geotechnical Laboratory Test Results

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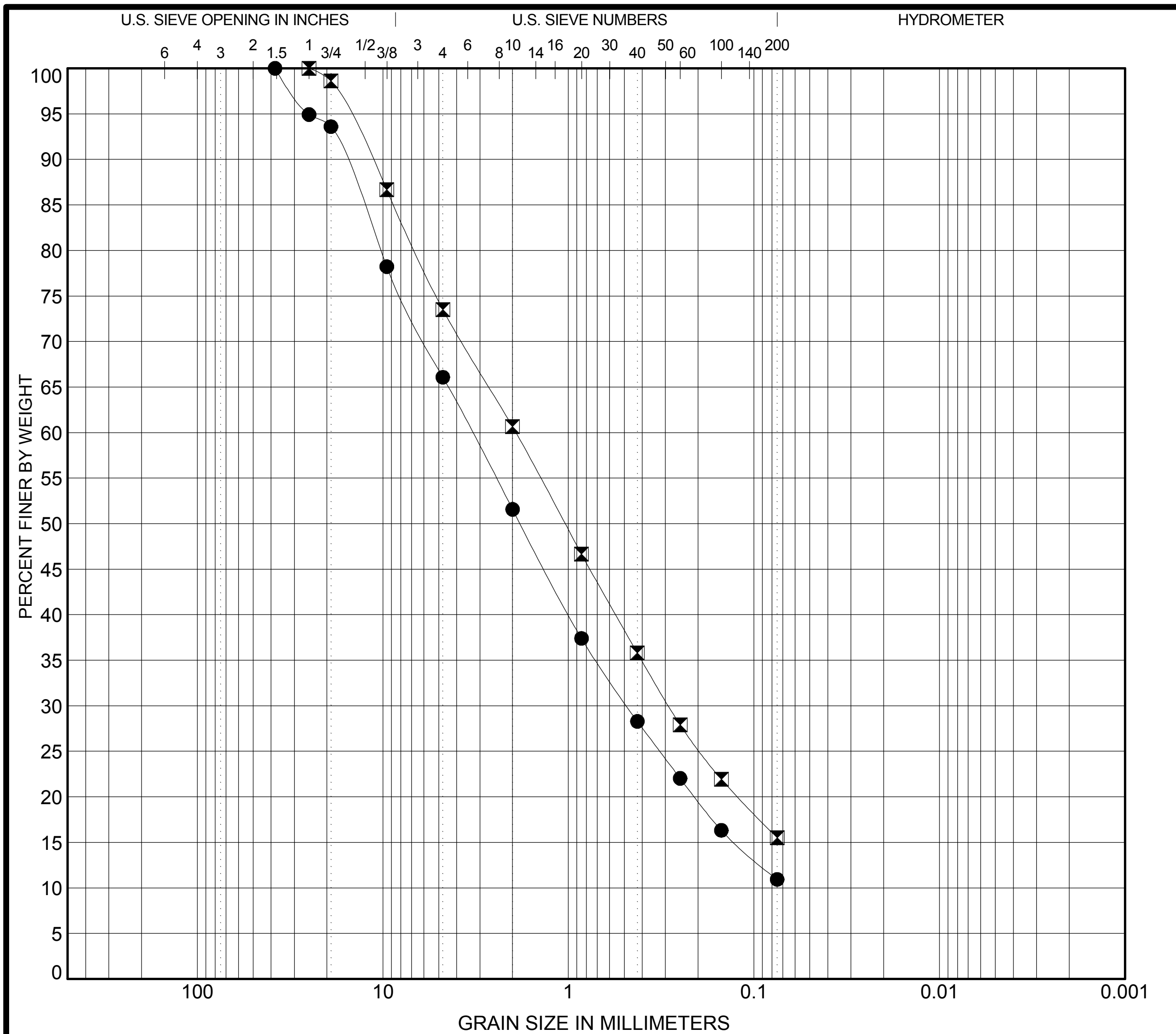
Figure Number	Summary Laboratory Test Results	Data Source	Comment
B1	CDM Smith Lab Test Summary Table B1	CDM Smith (2017)[1]	
B2	Grain Size Distribution	CDM Smith (2017)	
B3	Plasticity Chart	CDM Smith (2017)	
B4	Liquid and Plasticity Limits	CDM Smith (2017)	
B5	Consolidation Test CDM-1	CDM Smith (2017)	
B6	Consolidation Test CDM-4	CDM Smith (2017)	
B7	Unconsolidated-Undrained Triaxial Tests CDM-1, CDM-2 & CDM-4	CDM Smith (2017)	
B8	Undrained Shear Strength, Su	Fugro West (2004)[2]	Data reproduced from Dames and Moore report (1978).
B9	Undrained Shear Strength, Su, Comparison From All Tests	Fugro West (2004)	
B10	Moisture Content Profile (B-1 through B-6)	Fugro West (2004)	
B11	Dry Density Profile (B-1 throughB-6)	Fugro West (2004)	
B12	Recommended Su Profile (Outside Sludge Pond)	Fugro West (2004)	
B13	Comparison of Recommended Su Profiles	Fugro West (2004)	
B14	Comparison of 2002 and 2004 Vane Shear Tests	Fugro West (2004)	
B15	Consolidation Test Data	Cooper & Clark (1981)[3]	
B16	Summary of Unconfined Shear Tests	Cooper & Clark (1978)[4]	
B17	Summary of Triaxial Shear Tests	Cooper & Clark (1978)	

[1] This report.

[2] Fugro West, Inc. (2004a). "Recommended Su Profile for Shoring Design (Revised), South Bayside System Authority (SBSA), Redwood City, California", Prepared for the 'South Bayside System Authority', July 14.

[3] Cooper, Clark & Associates (1981). "Consultation: Re: Proposed Influent/Effluent Tie-In to Existing Force Main, Wastewater Treatment Plant, Redwood City, California", Prepared for the 'South Bayside System Authority', May 07.

[4] Cooper, Clark & Associates (1978b). "Supplementary Subsurface Investigation and Laboratory Testing, SBSA Project Unit No. 1, Redwood City, California", Prepared for the 'South Bayside System Authority', October 18.



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Boring	Depth	Classification	LL	PL	PI	Cc	Cu
●	**CDM-1**	**1.5**	Well graded SAND with silt & gravel (SW-SM)				**1.07**	**49.67**
⧗	**CDM-2**	**0.5**	Silty SAND with gravel (SM)					

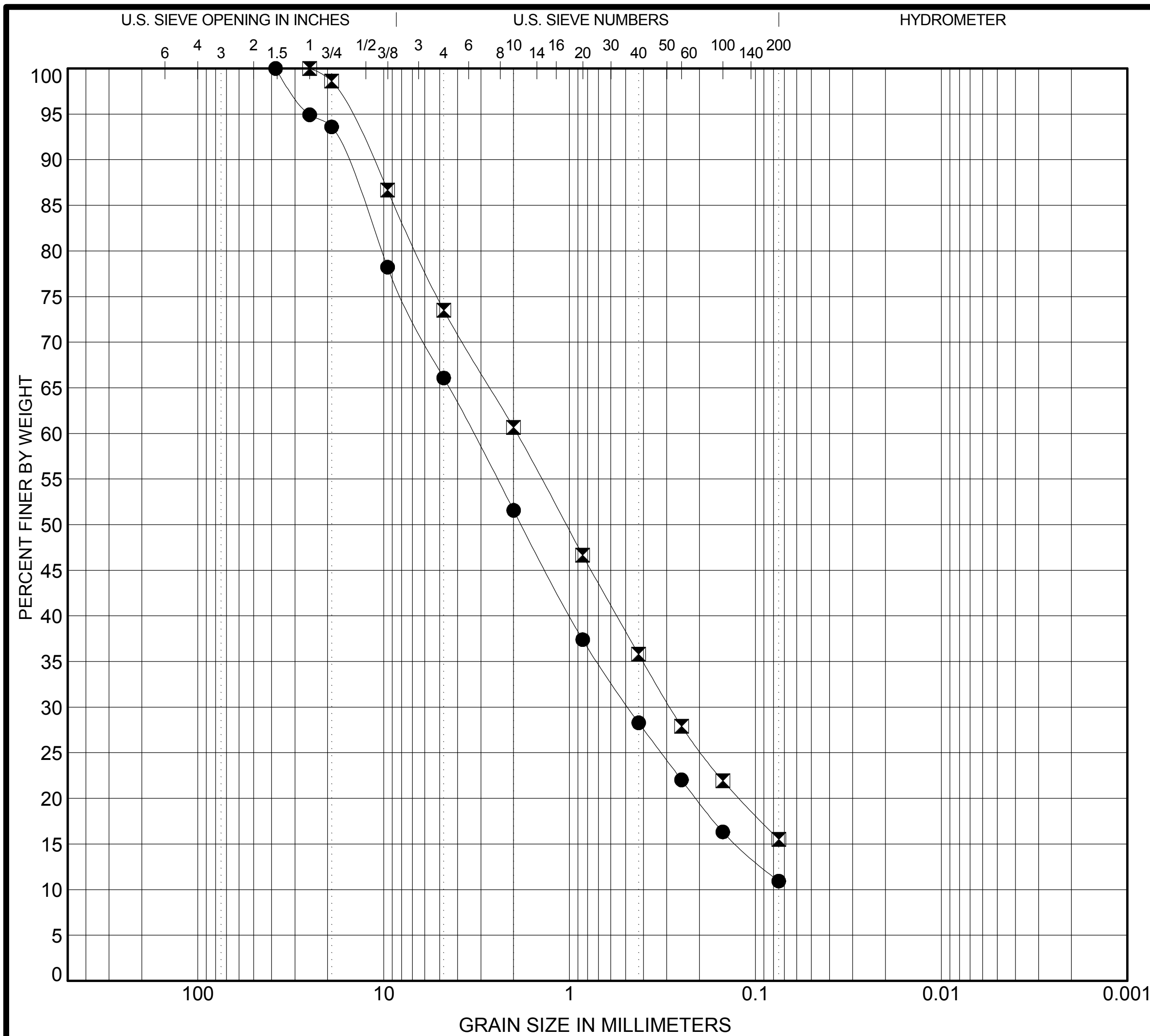
	Boring	Depth	D60	D30	D10	%Gravel	%Sand	%Fines
●	**CDM-1**	**1.5**	**3.31**	**0.48**		**33.9**	**55.2**	**10.9**
⧗	**CDM-2**	**0.5**	**1.92**	**0.29**		**26.5**	**58.0**	**15.5**

GRAIN SIZE DISTRIBUTION

Silicon Valley Clean Water
Influent Connector
Redwood City, California

Project No: 76558-111593 Figure: B2

CDM Smith



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Boring	Depth	Classification	LL	PL	PI	Cc	Cu
●	**CDM-1**	**1.5**	Well graded SAND with silt & gravel (SW-SM)				**1.07**	**49.67**
⧗	**CDM-2**	**0.5**	Silty SAND with gravel (SM)					

	Boring	Depth	D60	D30	D10	%Gravel	%Sand	%Fines
●	**CDM-1**	**1.5**	**3.31**	**0.48**		**33.9**	**55.2**	**10.9**
⧗	**CDM-2**	**0.5**	**1.92**	**0.29**		**26.5**	**58.0**	**15.5**

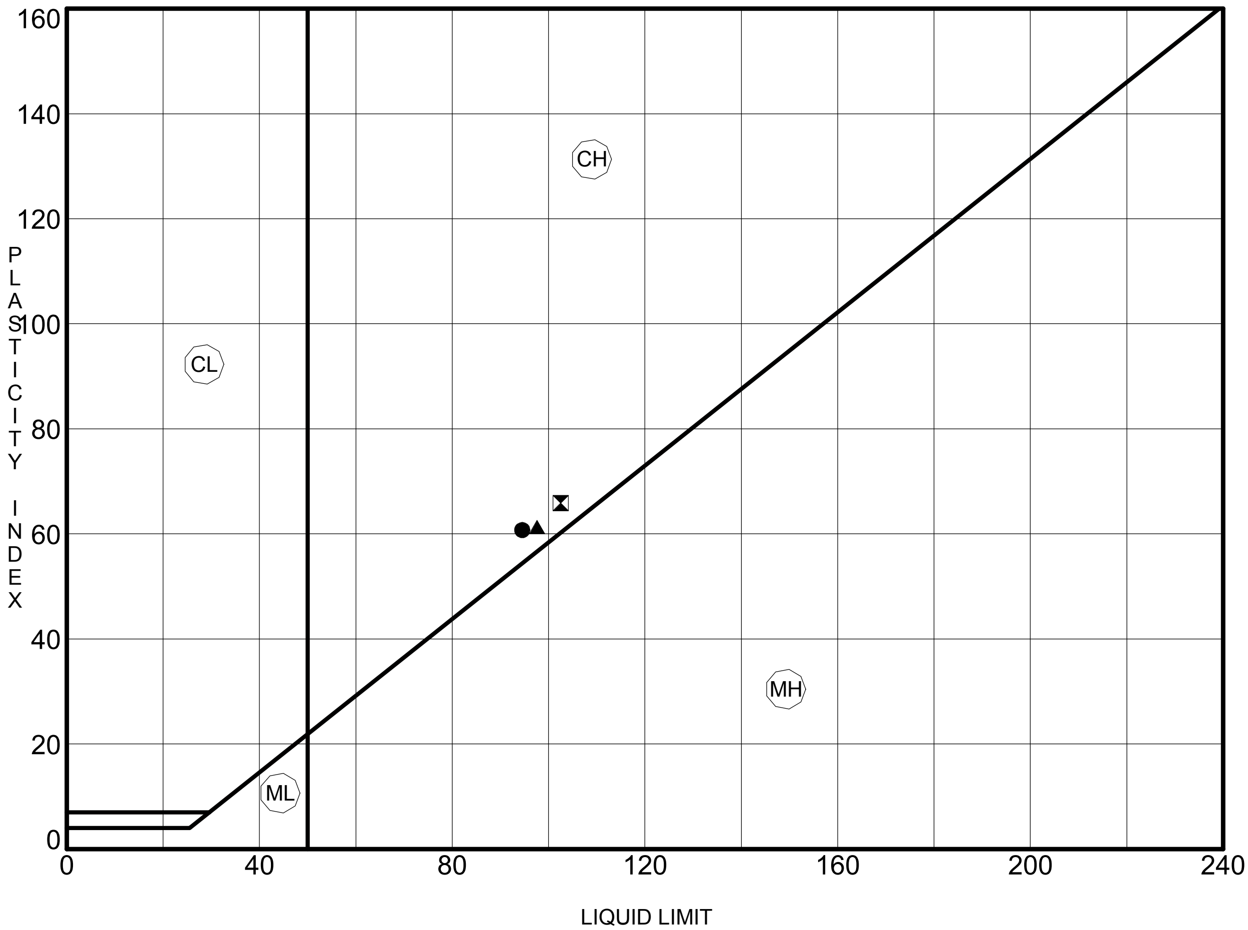
GSA_DEPTH SVCW.GPJ GINT STD US LAB.GDT 1/16/17 REV.

CDM Smith

GRAIN SIZE DISTRIBUTION

Silicon Valley Clean Water
Influent Connector
Redwood City, California

Project No: 76558-111593 Figure: B2



	Sample ID	Depth	M/C	LL	PL	PI	Classification	Gs
●	**CDM-1**	**30**		**95**	**34**	**61**	**Fat CLAY (CH)**	
⧗	**CDM-2**	**15**		**103**	**37**	**66**	**Fat CLAY (CH)**	
▲	**CDM-3**	**15**		**98**	**36**	**62**	**Fat CLAY (CH)**	

INDEX TEST RESULTS

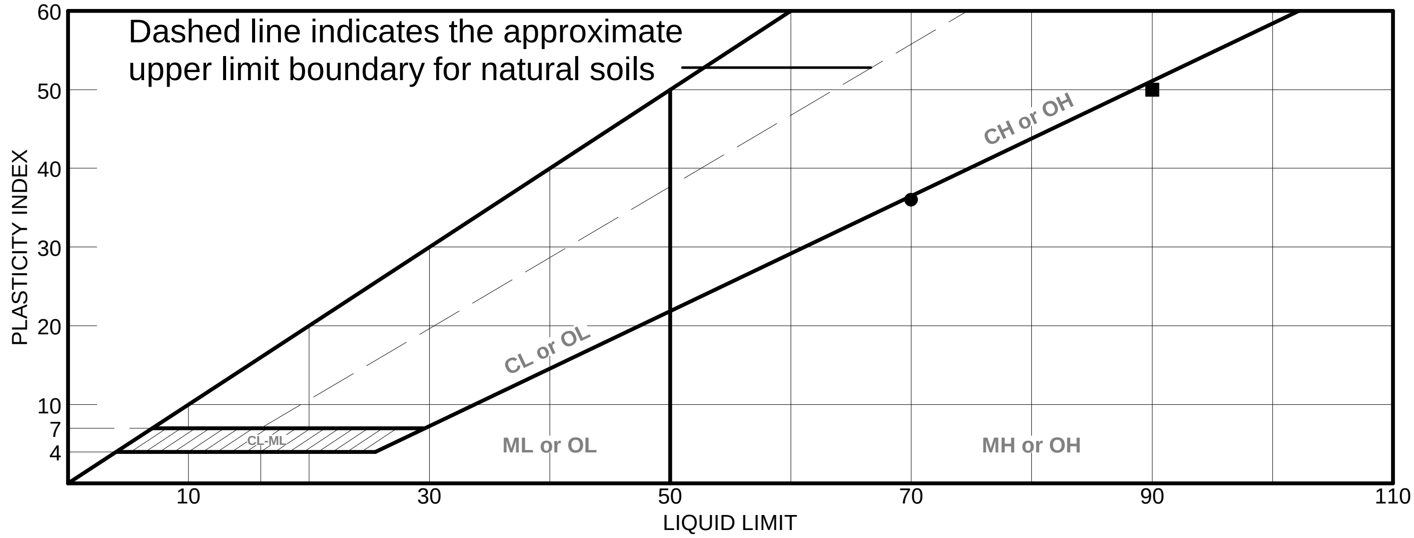
Silicon Valley Clean Water
Influent Connector
Redwood City, California

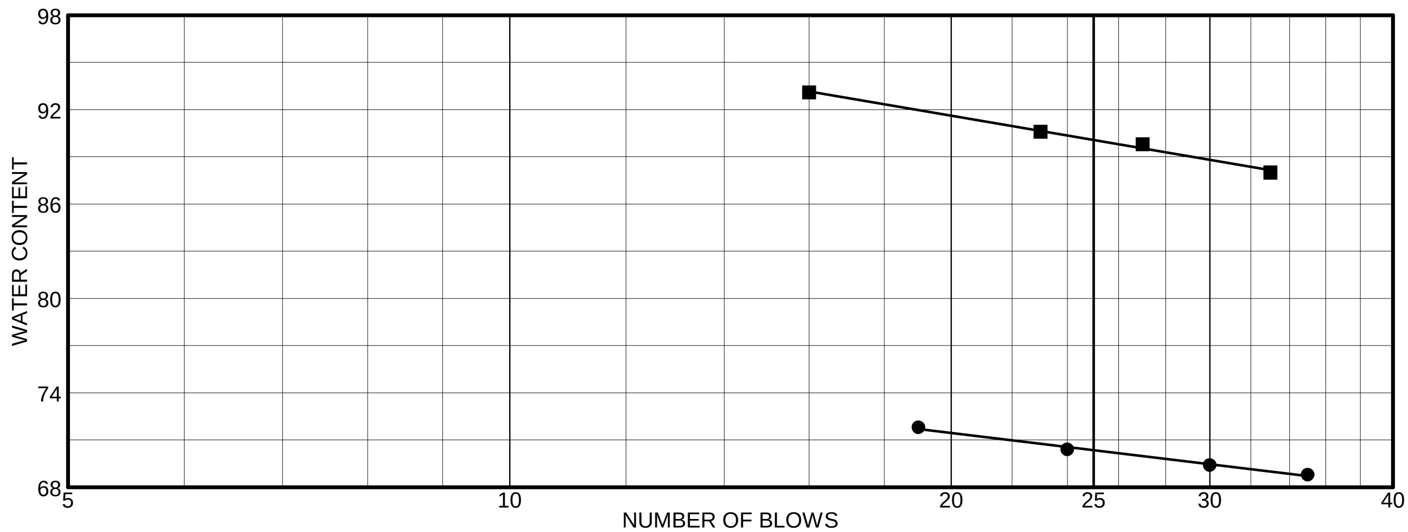
Project No: 76558-111593 Figure: B3

CDM Smith

ATTERBERG LIMITS HIGH LIMITS SVCW.GPJ GINT STD US LAB.GDT 1/16/17 REV.

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Gray Elastic SILT, trace Sand	70	34	36			
■	Dark Greenish Gray Elastic SILT (Bay Mud)	90	40	50			

Project No. 580-019 **Client:** CDM Smith

Project: SVCW - 76558-111593

● **Source:** CDM-2 **Elev./Depth:** 33-35'

■ **Source:** CDM-4 **Elev./Depth:** 13-15(Tip-4")

Remarks:

●

■

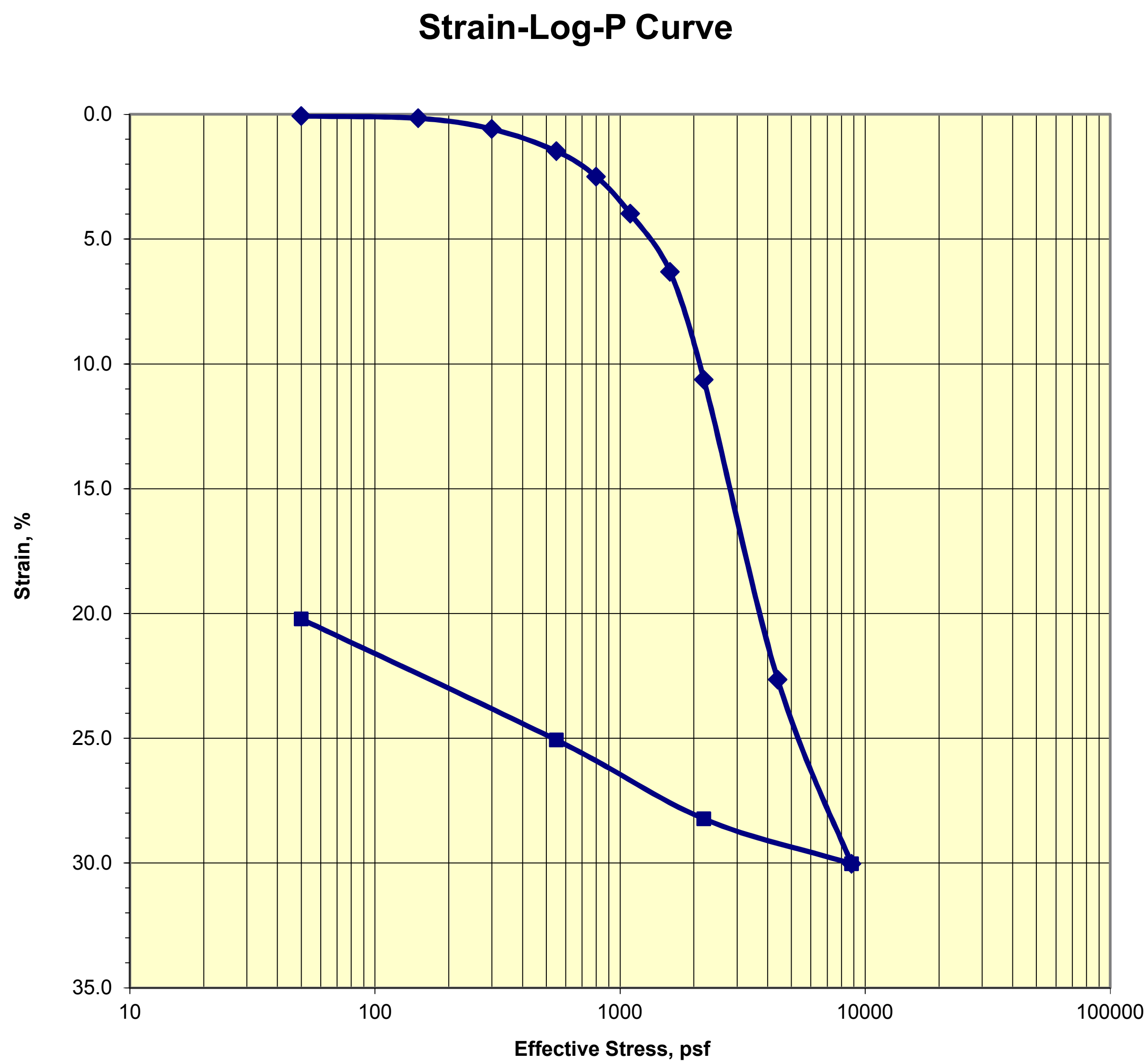
LIQUID AND PLASTIC LIMITS TEST REPORT

COOPER TESTING LABORATORY

Figure B4

Consolidation Test
ASTM D2435

Job No.:	580-019	**Boring:**	CDM-1	**Run By:**	MD
Client:	CDM Smith	**Sample:**		**Reduced:**	PJ
Project:	SVCW - 76558-111593	**Depth, ft.:**	28-30(Tip-4")	**Checked:**	PJ/DC
Soil Type:	Greenish Gray CLAY w/ shell fragments (Bay Mud)			**Date:**	4/12/2016



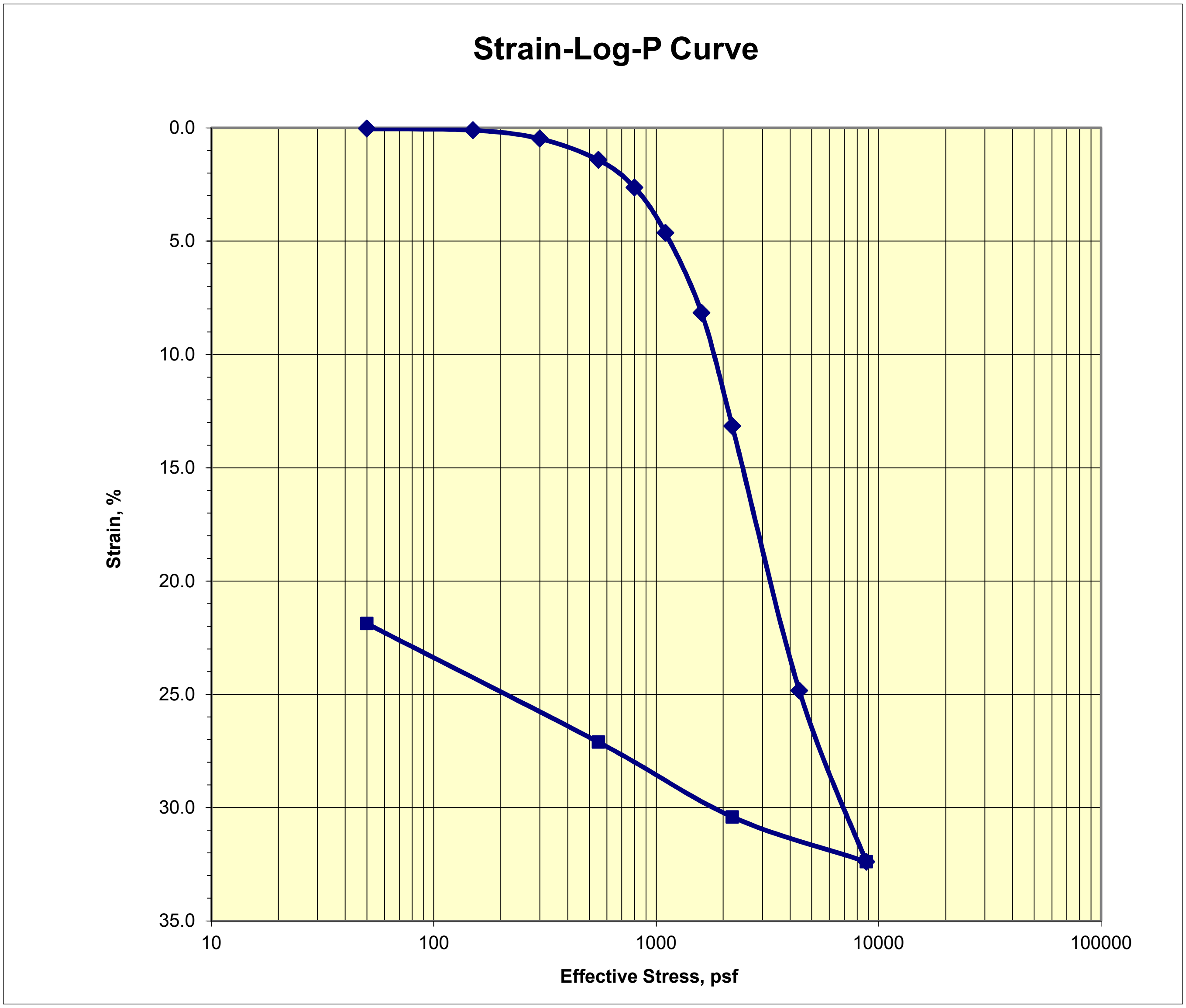
Assumed Gs 2.7	Initial	Final
Moisture %:	91.5	65.2
Dry Density, pcf:	48.4	61.1
Void Ratio:	2.479	1.759
% Saturation:	99.6	100.0

Remarks:

Figure B5

Consolidation Test
ASTM D2435

Job No.:	580-019	**Boring:**	CDM-4	**Run By:**	MD
Client:	CDM Smith	**Sample:**		**Reduced:**	PJ
Project:	SVCW - 76558-111593	**Depth, ft.:**	13-15(Tip-4")	**Checked:**	PJ/DC
Soil Type:	Dark Greenish Gray Elastic SILT (Bay Mud)			**Date:**	4/12/2016



Assumed Gs 2.65	**Initial**	**Final**
Moisture %:	100.0	69.6
Dry Density, pcf:	45.3	58.1
Void Ratio:	2.655	1.845
% Saturation:	99.8	100.0

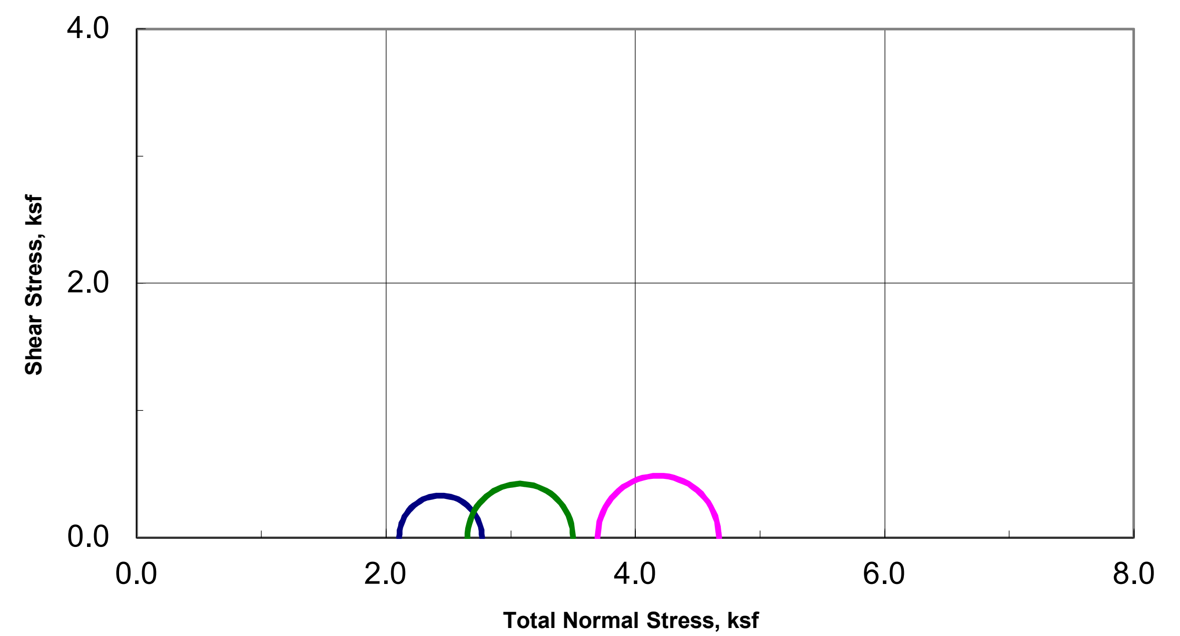
Remarks:

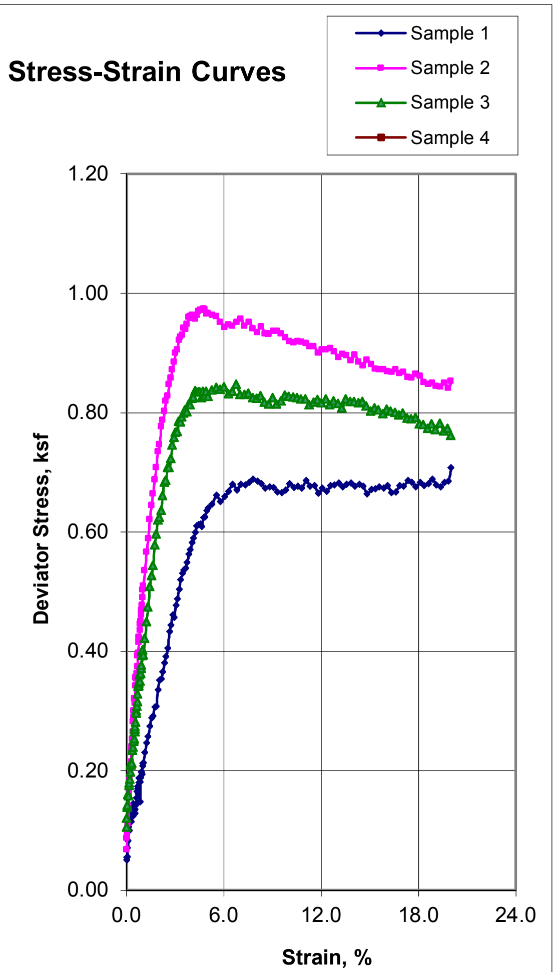
Figure B6

Cooper Testing Labs, Inc.
937 Commercial Street
Palo Alto, CA 94303

Unconsolidated-Undrained Triaxial Test

ASTM D2850





Sample Data	1	2	3	4
Moisture %	87.1	80.0	76.7	
Dry Den,pcf	50.1	52.5	54.9	
Void Ratio	2.362	2.211	2.072	
Saturation %	99.6	97.8	100.0	
Height in	6.08	6.10	6.14	
Diameter in	2.85	2.86	2.85	
Cell psi	14.6	25.7	18.4	
Strain %	15.00	4.74	6.78	
Deviator, ksf	0.664	0.974	0.847	
Rate %/min	1.00	1.00	1.00	
in/min	0.061	0.061	0.061	
Job No.:	**580-019**			
Client:	**CDM Smith**			
Project:	**SVCW - 76558-111593**			
Boring:	CDM-2	CDM-2	CDM-4	
Sample:				
Depth ft:	18-20	33-35	23-25	

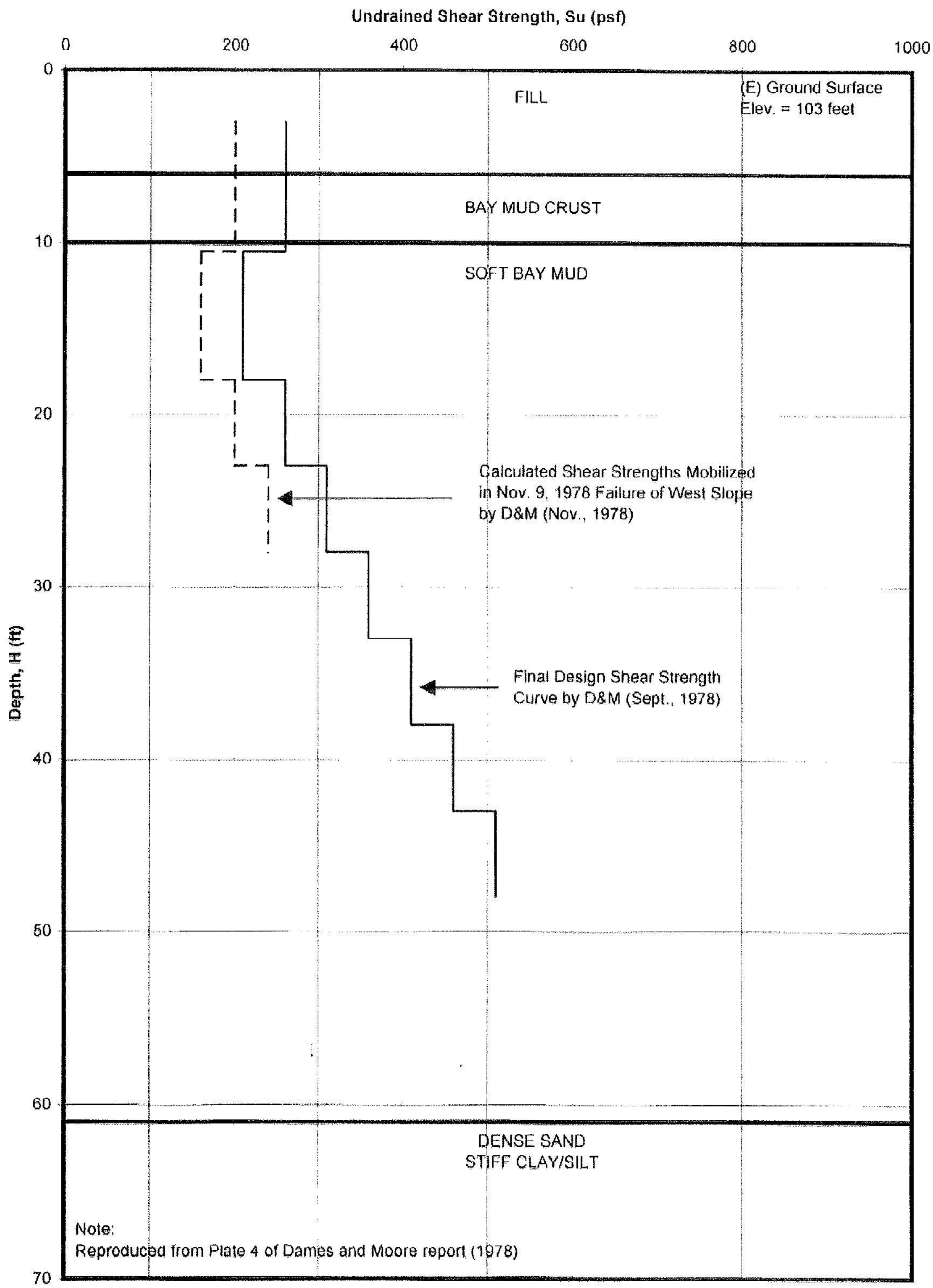
Visual Soil Description

Sample #	
1	Gray CLAY, trace Sand
2	Gray CLAY, trace Sand
3	Gray CLAY, trace Sand
4	

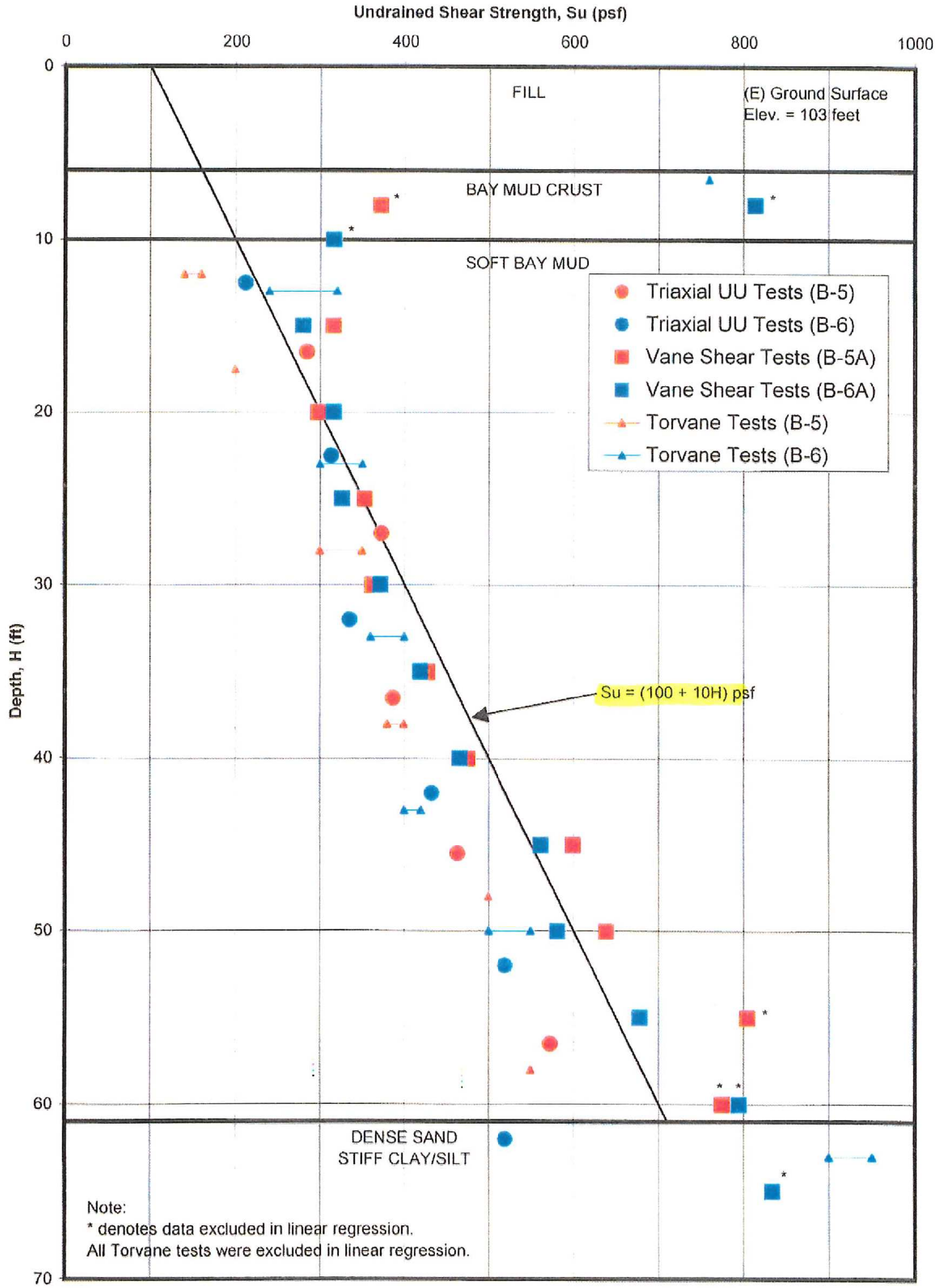
Remarks:

Figure B7

Note: Strengths are picked at the peak deviator stress or 15% strain which ever occurs first per ASTM D2850.

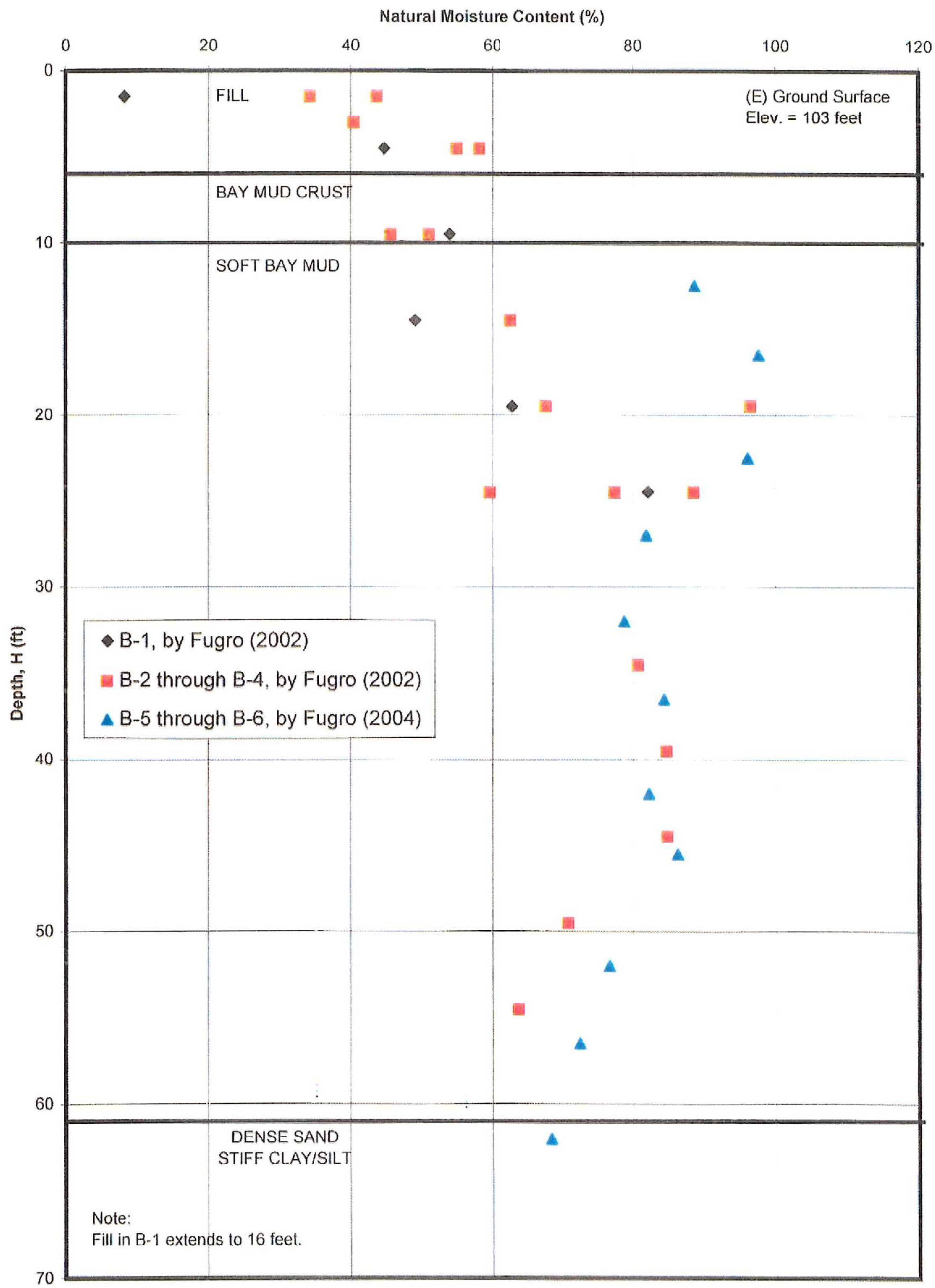


UNDRAINED SHEAR STRENGTH, Su, BY DAMES AND MOORE (1978) PLATE 2



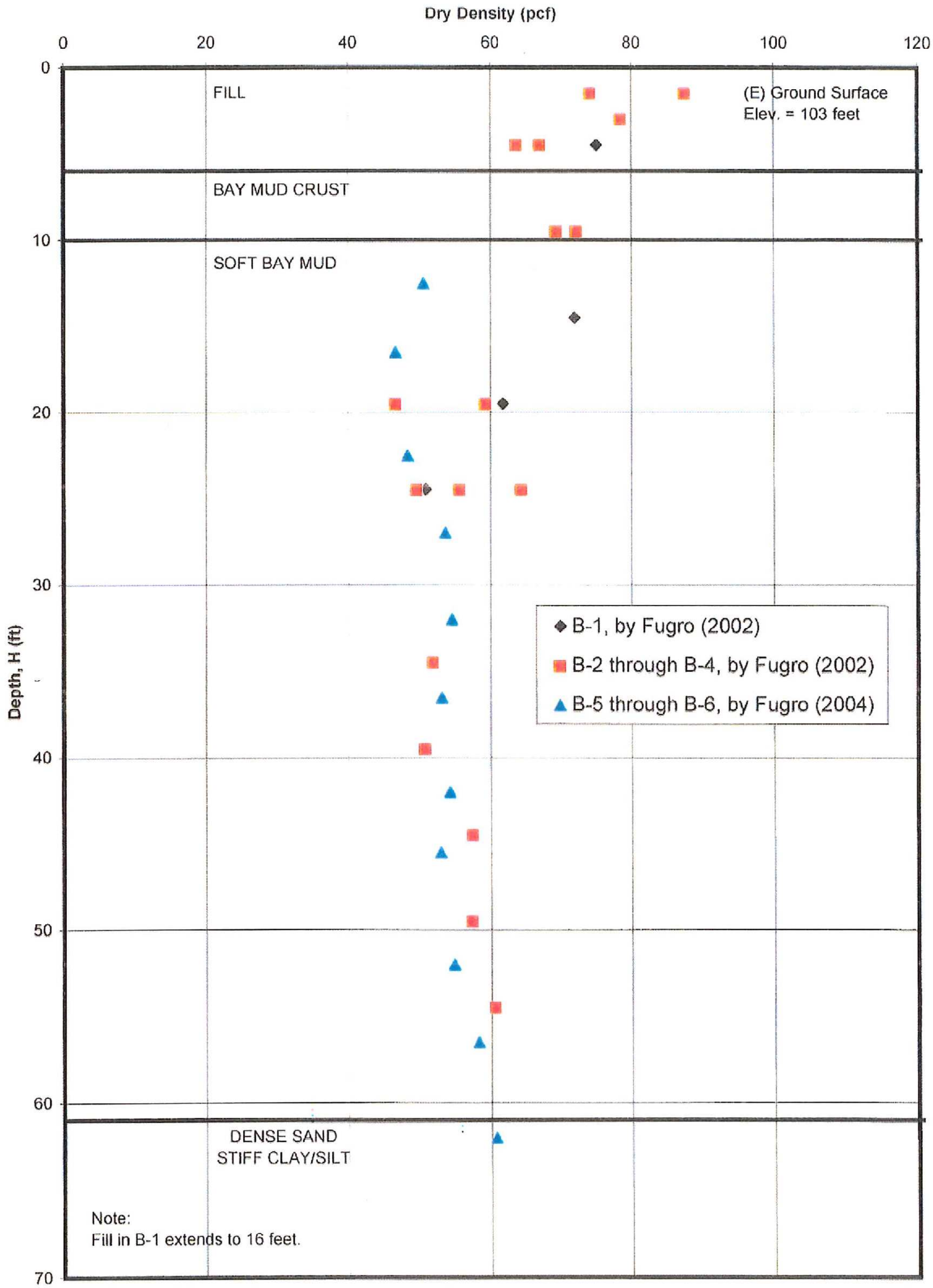
UNDRAINED SHEAR STRENGTH, Su, COMPARISON FROM ALL TESTS

PLATE 3



MOISTURE CONTENT PROFILE (B-1 THROUGH B-6)

PLATE 4

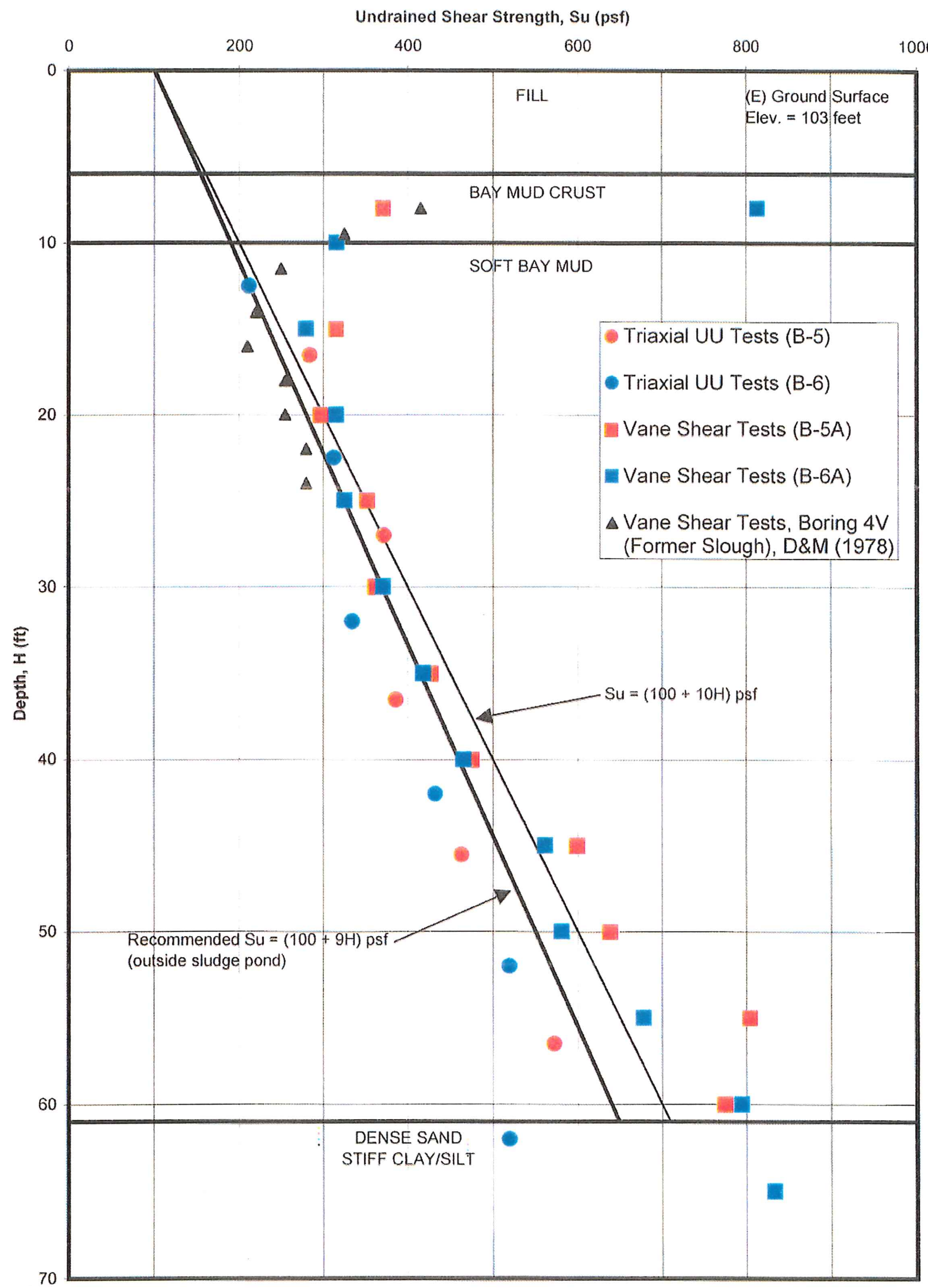


DRY DENSITY PROFILE (B-1 THROUGH B-6)

PLATE 5

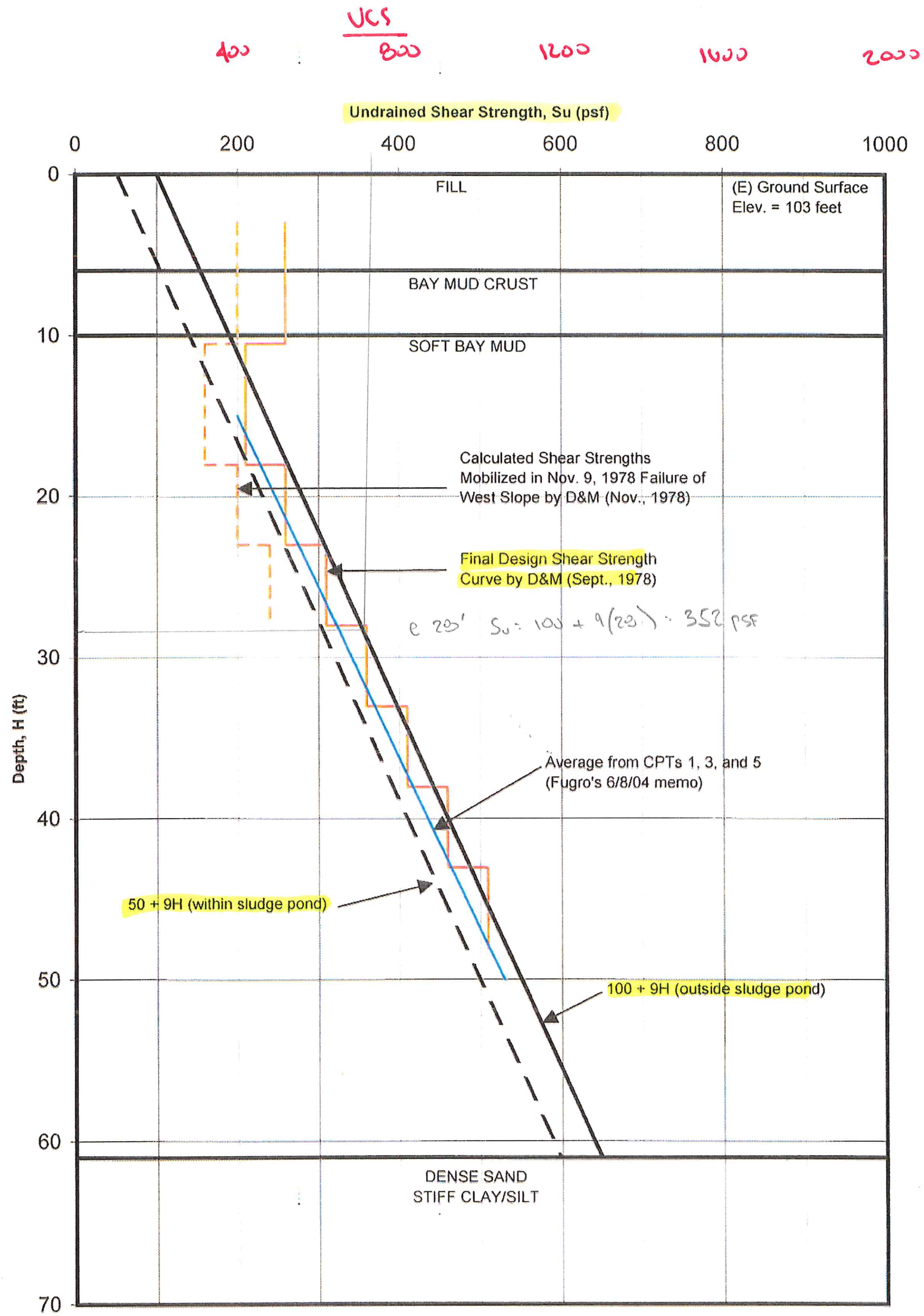
UCS, PSF

400 800 1200 1600 2000



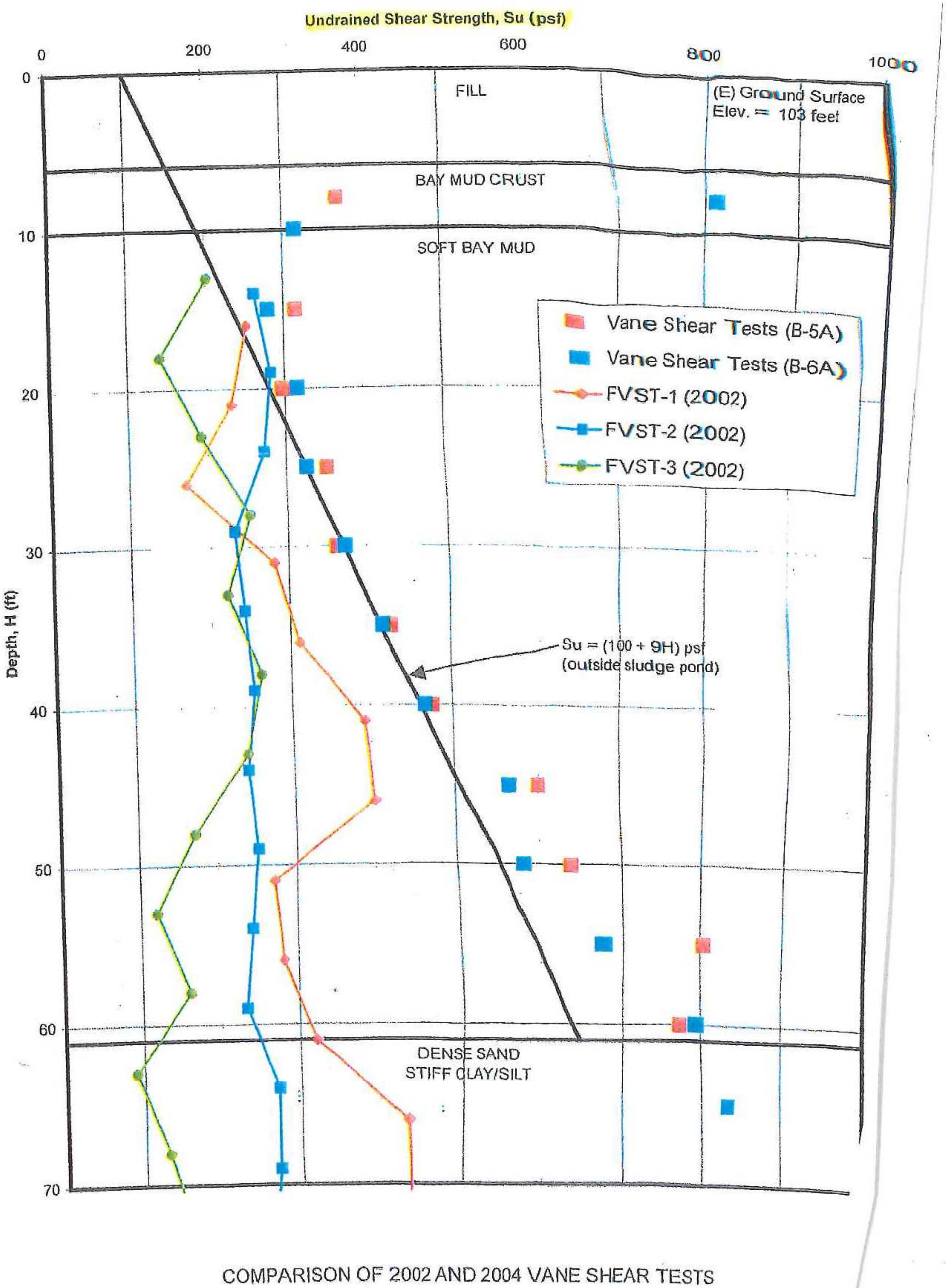
RECOMMENDED Su PROFILE (OUTSIDE SLUDGE POND)

PLATE 6

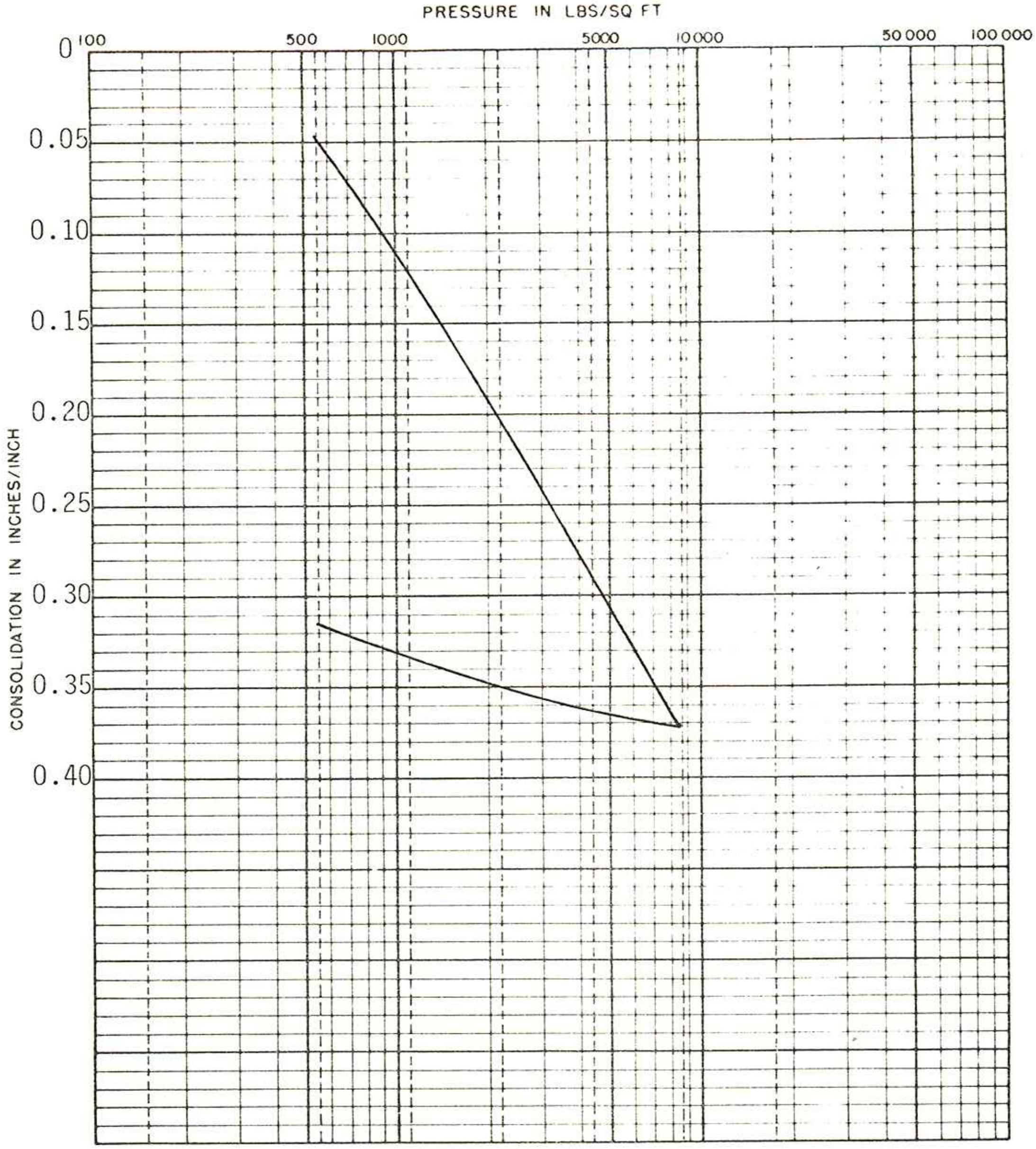


COMPARISON OF RECOMMENDED Su PROFILES

PLATE 7



COMPARISON OF 2002 AND 2004 VANE SHEAR TESTS



KEY	BORING	DEPTH (FT)	SOIL DESCRIPTION	SOIL TYPE	NATURAL MOISTURE CONTENT %	NATURAL DRY DENSITY (P.C.F.)	SPECIFIC GRAVITY
———	B-8	17	Gray organic silty clay (soft bay mud)	OH	92.2	48	

DRA... BS... DA... /22/...
CHECKED BY ... DATE ...

...NUMBER ... NAME ...SA
LOCATION Redwood City, CA

CONSOLIDATION TEST DATA

...51812

Figure B15
Page 1 of 1

Cooper & Clark

BORING NO.	DEPTH FEET	YIELD POINT SHEARING STRENGTH LBS/SQ. FT.	MOISTURE CONTENT PERCENT	DRY DENSITY LBS/CU. FT.
A-1	16	135	114.5	42
A-1	22	230	90.6	46
A-1	41	360	95.9	48
A-2	12	265	88.4	48
A-2	31	310	76.9	53
A-2	46	500	68.9	58
A-3	22	190	74.0	54
A-3	29	340	67.1	59
A-3	44½	490	57.2	66
A-4	8	355	80.2	53
A-4	14	145	92.4	47
A-4	35	385	80.1	52
A-5	14	245	90.0	49
A-5	30	360	72.6	56
A-5	40	470	66.8	60
A-6	14	270	82.6	51
A-6	20	285	87.1	49
A-6	40	310	74.4	55
A-8	25	300	88.7	49
A-8	35	280	83.7	50
A-8	45	315	70.5	58
A-9	18	190	90.0	48

SUMMARY OF UNCONFINED SHEAR TESTS

By BSK Date 10/18/78
Checked By BK Date 10/18/78
Job Number 1726-A5 Name South Bayside Systems Authority Location Redwood City, CA.
Revisions:
By ______ Date ______
By ______ Date ______

315228

Cooper & Clark

BORING NO.	DEPTH FEET	NORMAL PRESSURE LBS/SQ.FT.	YIELD POINT SHEARING STRENGTH LBS/SQ.FT.	MOISTURE CONTENT PERCENT	DRY DENSITY LBS/CU.FT.
A-10	12	850 1950	400 480	79.2	52
A-10	17	900 1950	430 500	70.3	56
A-10	16	700 1750	200 250	96.1	46
A-11	25	960 2320	290 350	88.8	48
A-11	35	1380 2440	400 460	68.9	57
A-11	45	1800 2550	540 600	67.3	59
A-12	7	650 1400	380 430	82.7	52
A-12	16	680 1750	200 270	99.7	44
A-12	25	1060 2420	380 440	73.4	55
A-12	40	1440 2500	470 520	63.9	61
A-13	13	800 1850	320 380	92.6	47
A-13	22	1000 2370	320 400	85.0	50
A-13	35	1450 2500	470 540	75.2	54
A-13	45	2050 3130	600 670	75.5	55
A-14	9	500 1250	230 270	93.5	47
A-14	15	800 1830	300 350	85.0	50
A-14	30	1400 2420	410 460	77.8	53
A-15	15	700 1240	210 240	79.7	52
A-15	30	1340 2400	360 420	72.2	56
A-15	45	2130 3200	650 730	70.5	56
A-16	11	440 1190	320 370	102.6	44
A-16	22	1000 1850	305 365	71.2	56
A-16	35	1450 2500	460 510	51.0	70

SUMMARY OF TRIAXIAL SHEAR TESTS

3152285

By BSK Date 10/18/78
Checked By [illegible] Date [illegible]
Job Number 1726-A5 Name South Bayside Systems Auth. Location Redwood City, California

Revisions:
By ____ Date ____
By ____ Date ____

Borehole Designation	Depth Below Ground Surface (feet)	In-Situ		Unconsolidated Undrained Triaxial Shear Strength (psf)	Atterberg Limits			Mechanical Analysis Percent Gravel		
		Dry Unit Weight (pcf)	Moisture Content (%)		Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Gravel (%)	Sand (%)	Silty & Clay (%)
CDM-1	0-2		Moisture Content					33.9	55.2	10.9
	20-20.3	50.2	90.7							
	20.3-20.7	57.6	103.1							
	20.7-21	48.7	87.6							
	28-30	48.4	91.5							
	30-31				95	34	61			
CDM-2	0.5-1		5.8					26.5	58.0	15.5
	15-15.3	52.7	79.9		103	37	66			
	18-20	50.1	87.1	330						
	26-26.3	59.1	62.3							
	33-35	52.5	80.0	485	70	34	36			
CDM-3	10-10.3	50.1	86.5							
	10.3-10.7	52.4	79.7							
	15-16.5		91.6		98	36	58			
CDM-4	13-15	45.3	100.0		90	40	50			
	23-25	54.9	76.7	425						

Figure B-1 ________________________

cdmsmith.com

Silicon Valley Clean Water

Draft Influent Connector Pipeline Geotechnical Interpretive Report

Task Order 2015-03

Silicon Valley Clean Water

April 2017

The information contained in the document titled "Draft Influent Connector Pipeline Geotechnical Interpretive Report" for Silicon Valley Clean Water in Redwood City, California, dated April 2017, has received appropriate technical review and approval. The conclusions and recommendations presented represent professional judgments and are based upon findings from the investigations and sampling identified in the report and the interpretation of such data based on our experience and background. This acknowledgement is made in lieu of all warranties, either express or implied. The activities outlined in this report were performed under the supervision of a California Registered Professional Engineer.

Reviewed and Approved by:

Osman Pekin, Ph.D., P.E., G.E.	Jeff Woon, P.E., G.E.
Principal Geotechnical Engineer	Sr. Geotechnical Engineer/ Reviewer

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Appendix

Section 1

Introduction

This Geotechnical Interpretive Report (GIR) is a planning level study which summarizes the results of our interpretation of the geotechnical data gathered during the current and previous geotechnical investigations for the influent connector pipeline project within Silicon Valley Clean Water (SVCW) wastewater treatment plant (WWTP). The subsurface soil and groundwater information obtained at the project site from this and previous investigations is presented by CDM Smith in a separate Geotechnical Data Report (GDR).

The purpose of this GIR, as defined in our overall project Task Order Authorization (September 25, 2015), is to prepare a geotechnical report interpreting available data to a preliminary level sufficient to provide initial direction for starting final design and construction. The agency recently decided on the project delivery method of Progressive Design Build for this and several other projects rolled into a single construction project called Front of Plant (FoP). This GIR also provides the next steps which should be taken for future geotechnical work. As such, this GIR is not intended to be a part of final design and construction documents until after further geotechnical input is provided and the next steps identified here are addressed.

The project site is located within Redwood City, California on the west side of San Francisco Bay, between San Mateo and Dumbarton Bridges. The WWTP is situated at the end of a peninsula with Bay Slough to the north and Steinberger Slough to the south, as shown on Figure 1, Vicinity Map.

1.1 Project Background

SVCW, which was known prior to 2014 as South Bayside System Authority (SBSA), is currently implementing the initial steps of the 2011 Conveyance System Master Plan (CSMP) through its implementation of the Capital Improvement Program (CIP) to improve the reliability of the conveyance system. The steps identified in the CSMP consist of replacement of conveyance system pump stations; replacement of conveyance system force mains; and upgrades to SVCW's treatment facility, including the influent connector pipeline. The influent connector pipeline will be used to transport up to 80 million gallons per day (mgd) of raw wastewater (projected peak wet weather flow) from the future headworks facility to the influent side of the plant's existing primary treatment system.

The proposed alignment for the influent connector pipeline was chosen based on an alignment alternatives analysis completed by CDM Smith (May 2016). Eight alignment alternatives (Alternatives A, B, C, D, E, F1, F2, and F3), consisting of different combinations of rehabilitating existing pipeline, constructing new pipeline(s) along the current alignment, and constructing new pipeline(s) along a new pipeline alignment(s) using open cut and microtunneling construction techniques were evaluated. The recommended alternative (Alternative F3), based on this evaluation, uses the open-cut method. Flow within the influent connector pipeline requires minimum flow velocities over a range of operating conditions to prevent unwanted settlement of solids. The proposed alignment of Alternative F3 is shown on Figure 2, Site Plan.

1.2 Proposed Construction

The recommended alternative consists of the installation of two parallel HDPE pipelines, (1) a72-inch nominal diameter profile wall HDPE pipeline and (2) a 48-inch nominal diameter solid wall HDPE pipeline, from the future headworks facility to the influent side of the plant's existing primary treatment system. Based on the current alternative, the approximate length of the influent connector pipeline is 850 feet, starting at the new Headworks building and terminating at the existing influent junction box to the east of the primary tanks. As shown on Figure 2, the alignment of the recommended alternative through the plant will be located within the street right-of-way (ROW) (i.e., Radio Road into the plant and along the main access road) and plant property boundary (parallel to the existing 54-inch RCP forcemain). The existing surface elevation along the alignments of the pipelines varies from El. 99 to El 103. The area in front of the headworks has been used as a pond; surface elevations in this area are between El. 99 and El. 100. The entire area will be filled up to 4 feet in thickness to raise the finished grade elevation to about El. 103 to El. 104. Both pipes will be installed to an invert elevation approximately 13-feet below ground surface, using open cut excavation construction methods within a 15-foot wide trench excavated between interlocking steel sheet pile shoring walls installed for the full-length of the alignment.

Access manholes for each pipeline are being considered in two locations for future access and maintenance, as follows:

- At the turn in Radio Road where the pipeline transitions from south to east; and
- At the location where the pipeline turns north to connect into the main treatment plant.

1.3 Scope of Work

This geotechnical interpretive report is prepared in fulfillment of Subtask 3.5.5D under Task Order No. 2015-03, dated September 25th, 2015. The scope primarily included a review of the GDR and performing necessary geotechnical evaluations and interpretations to develop the recommendations contained in this report. The following tasks have been performed:

- Review the GDR containing historical and recent geotechnical investigations and laboratory test data.
- Review and discuss the regional and local geologic and seismic conditions in the project area.
- Review available as-built drawings, and other construction records for other improvements in the Project area.
- Discuss potential geotechnical issues related to the open cut excavations and shoring and provide preliminary evaluation of geotechnical constraints.
- Perform preliminary geotechnical evaluation for pipe support including pile support, settlement, seismic considerations, support of excavations, shoring system alternatives, dewatering, vibration impacts, corrosion, and other relevant project constraints.

- Provide preliminary information relevant to settlement, piles, allowable bearing, lateral pipe support (soil modulus), lateral earth pressures, construction considerations and recommendations for temporary shoring and dewatering, and other geotechnical concerns.
- Provide preliminary recommendations for earthwork issues including disposal and selected reuse of on-site materials, import materials, special (lightweight) materials, and pavement repair considerations.
- Perform a preliminary evaluation of conditions at the pond area and identify potential impacts of differential settlement mitigation methods between new and old fill areas.
- Identify potential items of construction impact such as dewatering induced settlements and vibration induced damages, and provide preliminary recommendations for mitigative steps.
- Prepare this Draft GIR, including a discussion of regional and local geologic and seismic conditions, local soil and groundwater conditions, and preliminary geotechnical engineering evaluations and recommendations.
- Identify items requiring further evaluation.

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Section 2

Site and Subsurface Conditions

As summarized in the GDR (CDM Smith 2017a), we reviewed the published geology maps (Brabb et al. 1998; and Brabb and Pampeyan 1983) to obtain geotechnical surface conditions along the pipeline alignment and conducted a site visit to observe site conditions.

2.1 Site Conditions

The project area is located on the southeastern part of the WWTP site. The WWTP site was created by placing levees and fill over reclaimed marshland starting in about the 1950s (DCM|GeoEngineers 2009). The most recent fills were placed during the development of the site during late 1970s and early 1980s for the construction of WWTP facilities north of the project site. During the construction of the WWTP facilities, the project site was reportedly used as construction staging area. Subsequent to the construction of the WWTP facilities, the area in front of the Plant has been used as an ornamental pond, and the surface elevations in this area range from El. 99 to El. 100. The surface elevation to the east and south of this ornamental pond area rise slightly up to El. 103 to El. 104. In general, the surface topography of the project site is relatively flat, and no distinct topographic features are noted across the project site.

2.2 Site Geology

Geologic mapping by U.S. Geologic Survey (USGS) (Brabb et al. 1998) indicates that the project site is underlain by bay mud locally referred to as Younger Bay Mud (YBM), as shown on Figure 3, Geologic Map. An earlier USGS map Brebb and Pampeyan (1983) shows that portions of the project site with some areas of artificial fill, while majority of the site with YBM. Descriptions of these geologic units are as described below:

- **Bay Mud**: Water-saturated estuarine mud, predominantly gray, green and blue clay and silty clay underlying marshlands and tidal mud flats of San Francisco Bay. The mud also contains few lenses of well-sorted, fine sand and silt, a few shelly layers, and peat.

- **Artificial Fill (af)**: Loose to very well consolidated gravel, sand, silt, clay, rock fragments, organic matter, and man-made debris in various combinations.

In this area, the artificial fill soil unit is typically underlain by bay mud (YBM) soil unit.

2.3 Subsurface Conditions

A generalized subsurface profile is shown in Figure 4. Subsurface conditions along the pipeline alignment at the project site is described in the CDM Smith GDR (2017a). As presented in the GDR, CDM Smith drilled four soil borings (CDM-1 to CDM-4) along the proposed influent connector pipe line alignment. The GDR also presented subsurface information from previous exploration programs in the vicinity of the project. These included: Cooper, Clark & Associates (1978a, 1978b, 1980 and 1981), Dames & Moore (1978), Fugro (2002), Fugro West Inc. (2004a, 2004b and 2004c), DCM|GeoEngineers (2009), and DCM Consulting (2014 and 2015). These available documents provide information on deeper soil layers. Generally, the very soft YBM extends to approximately 70 feet bgs, with relatively more competent materials below that depth.

2.4 Generalized Subsurface Profile

Data from recent and historic subsurface explorations has been compiled in the GDR (CDM Smith, 2017a). Based on available information, a general description of the geologic units can be summarized as follows:

2.4.1 Fill

Approximately 0.3 feet of asphalt pavement was encountered in all the recent borings. The asphalt layer was underlain by base course material consisting of moist, silty Sand with gravel (SM)[1],well graded Sand with silt and gravel (SM-SW) and gravelly SAND (SP). The base course was observed to an approximate depth of 2 feet below ground surface. Based on laboratory data, the gravel content ranged from 26 to 34 %, sand from 55 to 58 % and fines from 11 to 16 %. Filter fabric was observed underlying the base course at a depth of about 2 feet at CDM-04 only.

2.4.2 Young Bay Mud Crust (YBM Crust)

Below the Fill is a layer of Young Bay Mud Crust with standard penetration test (SPT N values) ranging from 9 to 16 indicating stiff to very stiff consistency. This layer is presumably the upper, consolidated portions of the original soft YBM. The Crust is about 7.5 feet thick at boring CDM-1, thinning to the north to about 6 feet thick at boring CDM-3. From CDM-3 the crust pinches out and was not encountered at boring CDM-4.

2.4.3 Young Bay Mud – Very Soft (YBM)

Below the Fill and YBM Crust, soft, highly compressible very soft YBM with SPT N values of zero throughout most of the layer extending to a depth of approximately 65 to 70 feet in the project area (based on extrapolation from DCM Consulting 2015). The YBM consist of Elastic Silt/or Fat Clay, wet, with scattered shells, occasional organics and trace amounts of sand. Based on the laboratory testing the liquid limit (LL) ranged from 70 to 103%, the plastic limit (PL) from 34 to 40% and the plasticity index (LL-PL=PI) ranged from 36 to 66%. Moisture contents ranged from 76 to 103% and dry unit weight from 48 to 59 pounds per cubic foot (pcf).

2.4.4 Stiff Bay Mud/ Alluvium

Below approximately 70 feet bgs, the native materials are relatively more competent, with layers of sandy clay and stiff clay. These deeper materials are considered to be suitable for pile support. Additional data from available historic documents is provided in the GDR.

2.4.5 Groundwater

Groundwater was measured in four borings at depths between 3 and 4 feet at the end of drilling prior to backfilling. Groundwater should be expected throughout the site at depths as shallow as 1 to 3 feet below present ground surface.

[1] USCS Soil Classification Group Symbol

Section 3

Seismicity and Geotechnical Considerations

3.1 Faulting and Seismicity

The San Francisco Bay Area is one of the most seismically active regions in the United States. Significant earthquakes have occurred in the San Francisco Bay Area and are believed to be associated with crustal movements along a system of subparallel fault zones that generally trend in a northwesterly direction. The closest "active" fault to the project site is the San Andreas fault which is approximately 10.5 kilometers (6.5 miles) to the southwest. Other active faults include the Hayward and Calaveras faults are more than 10 miles to the northeast of the project alignment.

Earthquake intensities will vary throughout the Bay Area, depending on the magnitude of earthquake, distance from the causative fault, the types of materials underlying the site. During the useful life of the pipeline, the site will probably be subjected to at least one moderate to severe earthquake that will cause strong ground shaking.

3.1.1 Geologic Hazards

In addition to direct effects on structures and pipelines, strong ground shaking from earthquakes can also produce other side effects that include surface fault rupture, soil liquefaction, seismically induced settlement, lateral spreading, and earthquake-induced flooding and tsunami. Results of each of the secondary effects are discussed below:

3.1.1.1 Surface Fault Rupture

The project alignment is not located within a currently designated State of California Earthquake Fault Zone. Based on our review of existing geologic information, no known major surface fault crosses through or extends towards the site. The potential for surface rupture resulting from the movement of a previously unrecognized fault is not known with certainty, but the potential for ground rupture at the project site due to surface fault rupture is low.

3.1.1.2 Liquefaction

Soil liquefaction is a phenomenon primarily associated with saturated cohesionless soil layers, located within about 50 feet of the ground surface, lose strength during cyclic loading, as caused by earthquakes. During the loss of strength, the soil acquires "mobility" sufficient to permit both horizontal and vertical movements. Soils that are most susceptible to liquefaction are clean, loose, saturated, uniformly graded, fine-grained sands that lie below the groundwater table within a depth usually considered to be about 50 feet. The factors known to influence liquefaction potential include soil type and depth, grain size, density, groundwater level, degree of saturation, and both the intensity and duration of ground shaking.

Based on recent subsurface data as well as previous geotechnical information, the subsurface soils at the project site are predominantly soft clays (YBM) and firm to stiff clays, which are not susceptible to liquefaction.

3.1.1.3 Seismically-Induced Settlement

Earthquake induced settlement—compression of the underlying loose soils due to liquefaction or densification that occur during strong ground shaking —can cause uneven settlement of the ground surface. Since the subsurface conditions were predominantly clayey and have a low potential for liquefaction, the potential for seismically induced settlement is also considered low.

3.1.1.4 Lateral Spreading

Seismically induced lateral spreading involves lateral movement of earth materials during an earthquake due to ground failure of the subsurface layers. Lateral spreading is characterized by near vertical cracks with predominantly horizontal movement of the soil mass involved along potentially liquefiable layers.

Even though the site borders the Bay with shallow, unprotected earth slopes, the underlying materials are non-liquefiable. Some lateral bulging of slopes with soft clay materials may develop in seismic events but there is a reasonable buffer distance between the proposed pipeline location and the shoreline. Also, placing the pipeline on piles (as recommended in later sections of this report) will provide additional protection.

3.1.1.5 Seismically-Induced Flooding

Flooding may be caused by failure of dams, other water retaining structures, or an earthen levee due to an earthquake. The proposed pipeline alignment will be buried, the potential for seismically-induced flooding to affect the facility is considered low.

3.1.1.6 Tsunami

Being close to the shore and at low elevations, the site is likely to be impacted by a Tsunami event. Such events would be of more concern to the various facilities than direct impact to the subject underground pipeline project.

3.1.2 Seismic Coefficient

Based on the results of our recent and previous exploration data and laboratory testing, the site can be characterized as class E (Soft clay soil) as defined by the California Building Code (CBC), 2013. The seismic coefficients presented in Table 1, *Seismic Design Parameters*, are considered appropriate for structural design of the manhole and other structures associated with the pipeline.

Table 1 - Seismic Design Parameters

Design Parameters	Design Value
Site Class	E
Maximum Considered Earthquake SRA, Short Period (S_s)	1.50
Maximum Considered Earthquake SRA, 1-Second Period (S_1)	0.642
Design SRA, Short Period (S_{DS})	0.9
Design SRA, 1-Sec (S_{D1})	1.028
Site coefficient, Fa at S_s	0.9
Site coefficient, Fv at S_1	2.4

3.2 Geotechnical Considerations

The primary geotechnical considerations for the project are:

- Presence of soft and compressible Young Bay Mud,
- Differential settlement concerns between old and new fill areas (i.e. pond area versus other areas) along the pipe alignment,
- Ground shaking due to seismic events,
- High groundwater level,
- Temporary shoring and dewatering for the pipeline trench construction.

Of special concern is the area of the former pond. When fill is placed in that area, substantial settlement (more than 2 feet) is expected. These consolidation settlements can take a long time (more than 50 years) to due to very soft, thick and single-drained conditions (i.e. water from deeper layers has to squeeze out to the top since the bottom is also relatively impervious). A portion of the pipeline will be in this area, as shown in Figure 5.

The Headworks structure will presumably be on piles and this pipeline will be connecting to it. Even though flexible joints are considered, it is a concern to accommodate these potential high differential settlement magnitudes.

There may be other considerations for earthwork and preparation prior to foundations of the structures. For example, even though the structures will be on piles, it may not be desirable to have excessive ground settlement beyond and adjacent to the structures. There may be considerations to expedite and manage excess settlements in the area. One possibility is the use of wick drains (possibly with added temporary surcharge). The wick drains (and surcharge) help expedite consolidation settlement considerably since the drainage path for the water to squeeze out of the clay is reduced from 65-70 feet to a few feet. Since the rate of settlement is in proportion to the square of the drainage path length, the consolidation time can typically be reduced from decades to months. Some generic information on the wick drain concept is shown in Figure 6.

With methods like that discussed above, the ground within the pond boundary where the pipe will be located might be remediated and the pipe can be ground supported without excessive differential settlement concerns. Alternatively, the pipe can be pile supported, as is the structure it will connect to, and the differential settlements can be minimized in that way.

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Section 4

Conclusions and Design Recommendations

We conclude that the proposed pipeline project is feasible from a geotechnical engineering point of view, provided the recommendations presented in this report are incorporated into the project design and construction.

Based on the pipeline alignment considered, the geotechnical data reviewed, and the results from the limited geotechnical field and lab investigation conducted during this study, preliminary geotechnical recommendations were developed for and are summarized in the following subsections.

4.1 Soil and Groundwater Design Parameters

Soil and groundwater parameters used for design are presented in Table 2.

Table 2 - Soil and Groundwater Design Parameters

Stations	Design GWT depts bgs, ft	Soil Profiles				
		Depth ft	Soil Type	Unit Weight pcf	Cohesion psf	Friction Angle, deg
0+00 to 8+50	3.0	0-2	Fill (med. dense)	125	150	32
		2-7.5 to N.E	Young Bay Mud Crust (stiff)	115	500	0
		7.5-65 to 2-65	Young Bay Mud (very soft)	95	100	0

4.2 Buried Pipe

The entire the pipeline between stations 0+00 and 8+50 are buried. The geotechnical recommendations for the buried pipe are presented in the following subsections. Both ground support and pile support recommendations are being provided. Figure 7 provides a preliminary typical detail for the trench using ground support alternative. Figure 8 provides some generic information on pile and cradle supported pipe system. The ground support alternative is not recommended without controlling settlements first, as described in Section 3.2.

4.2.1 Allowable Bearing Pressure for Pipe Subgrade

Allowable bearing pressure for pipe subgrade was estimated to support the dead and live loads on the pipe and the weight of the pipe and its contents.

We recommend that the pipe be supported on bedding consisting of a 9-inch thick concrete tremie slab overlaying a minimum 12-inch thick ballast (select fill) layer. The concrete slab should meet the Caltrans Specifications in 19-3.02H: “Lean Concrete Backfill”. The ballast layer should be placed in 6 to 8-inch lifts and compacted to 95 percent of the maximum dry density as measured using ASTM D1557. Any wet and soft foundation subgrade materials encountered at the trench bottom should be over excavated and replaced with compacted angular and well graded aggregates. The extent of over-excavation should be at least 18 inches or as otherwise directed by the Geotechnical Engineer. We recommend placement of woven geotextile for separation and stability purposes at the trench bottom.

If the ground support alternative is selected, we recommend no net allowable bearing pressure increase within the pond area, due to the large anticipated settlement. With some ground improvement in the pond area, the net-zero load condition for the remainder can be used. In any alternative, it is recommended to provide pile support for the manholes.

4.2.2 Pile Support for Manhole Structures

Two (pressure) manhole structures will be located along the alignment. Due to potential settlement concerns, it is recommended to use driven piles to support these manhole structures. Preliminary pile design recommendations are as provided below.

4.2.3 Pile Support Alternative for Pipeline

The site soils are soft and subject to consolidation settlement when additional loads are applied, even when the added loads are relatively minor (e.g. a few feet of fill). Generally, it is plausible to maintain a 'net zero' load condition for the pipeline, by using lightweight fill as part of the trench backfill materials. However, portions of the pipeline near the Headworks area will be subject to long term ground settlement as fill is placed over the former pond area. Of the total approximate pipe length of 850 feet, about 200 feet of it at the southerly end, is in this area prone to settlements reportedly estimated to reach 2 to 2.8 feet (DCM, January 17, 2017). For vertical support of the pipe, designing for 'net zero' conditions could work in the remaining 650 feet, but the 200 feet section would require pile foundations to avoid damaging levels of differential settlement across the transition zone near the edge of the pond.

We considered the possibility of using a combination of piles at the southerly end and a 'net zero' bearing for the remainder of the pipe, with flexible joints in the transition zone. However, it is generally not good practice to mix foundation types as it may pose problems with different response behaviors under seismic events or other future loads or groundwater fluctuations. Also, lateral bulging concern due to seismic events at this near-shore location could be another consideration in favor of piles, however this is not considered an absolute requirement as there is some reasonable distance between the pipe and the shoreline slope. With these concerns and taking into account some other factors listed below, the use of piles for the entire pipeline alignment can be considered. Piles could have the following advantages:

- If a pile rig is needed anyway for certain portions of the work, providing additional piles for the remainder could be reasonably cost effective.
- Piles may be needed to support the manhole structures, in resisting thrust forces, and under seismic loads, with a remaining pile length in question of about 600 feet.
- The piles plus reinforced bottom slab/ cradle/ pile cap would add some cost but there could be reduction in other costs associated with not having to provide excavation bottom preparation, bottom slab, lightweight materials, special pipe joints, and possibly

reduced cost for the excavation/ backfill/ shoring if a narrower trench can be used with piles and cradle side supports.

With these considerations, we consider the use of pile foundations for the support of this pipe and related structures (manholes) to be a viable alternative. Following are the preliminary pile design recommendations:

1. Recommended pile type is 14” precast concrete driven piles.
2. The depth to more competent old Bay Mud is 70 feet bgs for design estimation purposes.
3. In the pond area, the soil in the upper 70 feet would be subject to negative skin friction as it settles. An adhesion value of -100 psf is recommended for negative skin friction.
4. Beyond the pond area, provided no new ground loads are introduced and presumably consolidation has already taken place, negative skin friction need not be accounted for.
5. If the pond area settlement were to be expedited by using wick drains and surcharge, plus substantial completion of that settlement were to be verified by measurements, then down drag can be eliminated.
6. The piles would derive their resistance from below 70 feet, predominantly in skin friction. Some end bearing would also apply in the sandy zone of limited thickness. However, there is a likelihood that this layer would be penetrated or be close to being penetrated with longer piles, below which is stiff clay. For preliminary design purposes, we recommend using 750 psf skin friction below 70 feet and neglecting end bearing for pile capacity and length calculations. These values would likely translate to about 100 feet long piles, depending on the design loads.
7. Lateral pile capacity should be determined using LPILE or similar program, based on actual design loads, allowable deflections, and type of support once these details are available.
8. Indicator piles with Pile Dynamic Analyzer (PDA) testing should be used to verify pile capacities and lengths prior to production piles.
9. Piles should be designed and fabricated to withstand a high corrosion environment.

4.2.4 Lateral Pipe Support

Soil modulus (E’) is one of the key parameters that characterize the level of available lateral pipe support. Areas mapped to have lower soil modulus values imply lower safety factors for the pipe. Soils with low CPT tip resistance and low SPT N values would have a low modulus.

In our opinion, the tabulated moduli ranges are suitable as factored values for use in preliminary design. These values are subject to verification during final design, after additional information is clarified (e.g. if surcharge, wick drains, etc. will be implemented or not).

Table 3 - Simplified Soil Categories for Design

Soil Category	q_c (tsf)	N_{eq}	E' (psi)
Fill (medium dense)	>10	> 4	400 - 800
Young Bay Mud Crust (stiff)	5 – 10	2 – 4	250
Young Bay Mud (very soft)	<5	<2	100

4.2.5 Thrust Blocks and Restraints

The allowable soil bearing values (lateral or vertical) recommended for thrust blocks and restraints is 300 psf where the contact soil is YBM, and 1,200 psf where the soil contact is stiff YBM crust or fill materials. When the thrust force is applied, some movement of the soil should be expected as the allowable soil bearing pressure is approached.

For the case of an upward thrust force, resistance may be provided using the weight of the pipe, the weight of a thrust block (if needed) and the weight of the soil directly above the pipe and thrust block. An adhesion value of 100 psf can be used between the YBM and concrete.

4.3 Materials

4.3.1 Backfill - Lightweight Backfill

Lightweight fill for use in the trench zone above the pipe zone may consist of expanded shale, clay and slate (ESCS) aggregate fill meeting the requirements of ASTM C330. Grading: 3/4" to No. 4 with compacted moist density in the range of 60 to 75 lb./ft^3

4.3.2 Backfill – Select Fill

Select fill is recommended for support of the tremie slab. Select fill may consist of an acceptable grading based on Caltrans Specification "26-1.02B, Class 2 Aggregate Base", 1-1/2-in. maximum or approved equal.

4.3.3 Pipe Bedding – 3/4-In. Drain Rock

Pipe bedding should consist of ¾-inch crushed rock and rock dust in accordance with Section 200-1.2 of the Standard Specifications for Public Works Construction.

4.3.4 Separation Geotextile

A layer of non-woven geotextile is recommended for separation between the granular bedding material and the adjacent clayey soil. Separation Geotextile should be in accordance with Section 88-1.02B of Caltrans Standard Specification.

4.3.5 Aggregate Base (AB)

Aggregate base (AB) for road base should conform to the requirements of crushed aggregate base in accordance with Caltrans Specification "26-1.02B, Class 2 Aggregate Base", 3/4-in. maximum.

4.4 Corrosion Potential

Due to close proximity of marine water and the height of the water table (shallow groundwater adjacent to the Steinberger Slough), it is anticipated that highly corrosive conditions will exist (both liquid below surface and airborne). The HDPE pipe material will not be subject to corrosion, but all metal fittings should be corrosion protected and concrete mix should be designed for high chlorine content. Further testing and consultation with a corrosion specialist is recommended during the design phase.

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Section 5

Construction Considerations

5.1 Trenching and Excavation Support

Based on the subsurface conditions encountered and our experience with similar materials, it is our opinion that the materials along the pipeline alignment can be excavated with ordinary backhoe equipment. However, very soft soil and groundwater will occur in most of the trench excavations. Trench shoring practices should conform to the OSHA and CALOSHA requirements.

5.2 Trench Shoring

The pipe trench excavation will require shoring support. Typical trench installations at the WWTP site have utilized sheet piles with struts and walers. Due to soft soils, bottom heave should be considered and pile embedment depths designed accordingly. Dewatering and shoring designs should be coordinated to have consistent earth pressure and hydrostatic pressure design values. Shoring design will be the responsibility of the contractor. Soil parameters provided in Table 2 should be used for shoring design and Contractor's proposed design pressures and pressure distributions consistent with their proposed system and bracing conditions should be submitted for review by the Engineer prior to submittal of detailed shoring calculations. Further considerations are provided in Section 5.5.

5.3 Dewatering

Throughout the pipeline alignment, groundwater will occur about 1 to 3 feet below the ground surface. Along the length of the pipeline alignment, the profile of the pipeline, as well as the ground surface, are relatively flat. The depth to invert of the pipeline will be about 15 feet below the existing ground surface. The soft, fine grained soils that will be encountered will require dewatering. We recommend that the dewatering system be designed to lower the groundwater at least 2 feet below the design elevation of the trench bottom. The potential adverse effects of lowering the groundwater must be considered in the dewatering system design. Further considerations are provided in Section 5.5. It is recommended that pipeline buoyancy mitigations be included in the construction of the pipeline due to the shallow groundwater conditions at the site. See Figure 7 for conceptual approach related to buoyancy.

5.4 Vibrations

The vibrations induced by the construction activities can compromise the structural integrity of nearby structures by causing structural damage directly via vibration amplitude and/or indirectly by settlement of the foundation soils. Vibration monitoring should be provided during driving of piles and sheet piles as well as during extraction of sheet piles. All vibrations should be kept below threshold values to be established by a vibration expert. CDM Smith can provide additional services in this area before and during construction upon request. Further considerations are provided in Section 5.5.

5.5 Additional Considerations for Temporary Construction

At a minimum, the following constraints are recommended for trenching and temporary open excavations. The Geotechnical Engineer should review the plans and specifications, to verify that these and other requirements are adequately incorporated into the bid documents.

1. Shoring Type: The shoring system should consist of continuous elements extending below the excavation bottom such as interlocking steel sheet piles or tangent piles. Intermittent systems such as soldier beams and lagging or systems without continuous toe penetration such as trench boxes or slide rail systems are not considered to be suitable for use on this project.

2. Shoring Design: The shoring system should be designed by a Shoring Engineer licensed in the State of California and with a minimum of 10 years of experience in shoring design and a minimum of 3 projects involving excavation in soft soils below the groundwater. The lead Shoring Engineer should submit a resume for approval prior to start of work.

3. Shoring Design Guidelines: The available geotechnical data should be interpreted by the Shoring Engineer in relation to the specifics of the proposed shoring and dewatering system. The design earth pressure distributions would vary based on the bracing configuration and other factors that will be proposed.

    a. Shoring design should comply with the latest version of the Caltrans Trenching and Shoring Manual.

    b. In addition to conventional earth support and shoring design calculations, the bottom heave should be considered. The sheet pile embedment depths should be calculated for the deeper of earth support and bottom heave considerations.

    c. Groundwater should be assumed to be at no deeper than 3 feet below ground surface. Due to close proximity of the Bay, there may be tidal influence and groundwater level fluctuations throughout the day. Due to some saline water influence, the water unit weight can be assumed at 64 pcf.

    d. Earth pressures should be calculated to be consistent with the proposed system (bracing, etc.) and based on the soil strength parameters and other information provided in this report and the GDR.

    e. Surcharge loads should be consistent with the proposed equipment loads, traffic, and any other structures within the influence zone. Surcharge load should not be less than an equivalent soil height of 2 feet.

4. Dewatering Restrictions:

    a. External Dewatering—Due to highly compressible soils, any uncontrolled external dewatering such as using well points outside the shoring system is subject to various restrictions. In general, each 2 feet of groundwater lowering induces about the same effect as placing 1 foot of earth fill on the ground surface. Studies by others in the same area (DCM Consulting, January 2017) have estimated that 4 feet of new fill could cause 2' to 2.8' of long term settlement, so any significant water level lowering could impact existing structures and ongoing construction in the vicinity. Should external dewatering be proposed, the Contractor's Dewatering Engineer and the Contractor's Geotechnical Engineer should submit estimated settlements based on estimated dewatering durations as well as contingency plans such as recharge wells where existing facilities are within the influence zone of a potential settlement crater (assumed proportional to the drawdown zone).

b. Internal Dewatering—Alternatively, using internal dewatering only would significantly reduce potential settlement impact compared to external dewatering. However, this comes at the added burden of having to design the shoring system for full hydrostatic pressure in addition to submerged earth pressures.

5. Dewatering Designer: All dewatering systems should be designed by a qualified Dewatering Engineer. Dewatering evaluation should take into account the depth of shoring systems and soil properties as documented in the GDR, to demonstrate no significant impact of dewatering to existing facilities, and mitigative measures if any impacts are suspected.

6. Contractor's Geotechnical Engineer: The Contractor should employ a professional engineer licensed in the State of California, specializing in geotechnical engineering, having a minimum of 10 years of experience in geotechnical construction consultation. Due to highly compressible soils and potential impact to adjacent facilities, items requiring geotechnical input during construction may include: stockpiling plans with size limits, offset distances, and estimated settlement impacts; similar evaluations for heavy equipment (crane, etc.) support and impact. Evaluation, mitigation planning, and monitoring planning for geotechnical engineering should be provided for temporary construction activities which may impact existing facilities, utilities, levees, and newly constructed elements.

7. No Open Excavations: The Young Bay Mud in the area is known to have failed in the past (Dames and Moore, 1978). No temporary open excavations deeper than 5 feet should be made unless submitted by the Contractor's geotechnical engineer and approved by the Engineer.

8. Vibration and Settlement Monitoring: Pre-construction condition survey (photo and/or video), settlement monitoring, and vibration monitoring during construction should be provided. A vibration monitoring expert should provide recommendations during sheet pile installation, extraction, and other vibration causing activities. Sheet pile driver vibration frequencies should be adjusted as required to minimize impact, based on monitoring data. Due to relatively deep, soft, and submerged soils, it is possible to induce vibration waves at considerable distances from the source. Owner-provided vibration monitoring is recommended for gathering information related to potential impact to existing facilities, and not intended for informing or directing the Contractor. The Contractor can separately perform settlement, vibration, and any other monitoring as needed to carry out the construction activities in a safe, informed, and competent manner.

Care should be taken to avoid excessive soil loss (soil plug/ void formation) during sheet pile removal.

9. Coordination among Contractor's Engineers: The design of shoring, dewatering, and other activities should be integrated with one another in the preparation of submittals and during implementation. Preferably, shoring, dewatering, and monitoring plans are to be submitted as one complete package with parameters coordinated and consistent across these inter-related disciplines. The various professional activities required of the Contractor as described above (shoring, dewatering, geotechnical, and other monitoring) does not imply separate professionals be retained for each discipline. Suitably qualified multi-disciplinary professional(s) can fulfill more than one role.

5.6 Construction Monitoring

It is recommended that a qualified Geotechnical Engineer be present during construction to confirm that the intent of these recommendations is complied with.

Section 6

Geotechnical Issues to Carry into Design-Build

Following are some geotechnical considerations that are not addressed at this stage of the project and will need to be addressed by the Design-Builder. The main issues revolve around excessive settlement, differential settlement, and settlement duration factors. This does not imply all geotechnical considerations are listed here, as this is a preliminary geotechnical document. The location of origination point and termination point of the influent connector pipeline may be subject to changes from the current concept, depending on the finalized location of the new Headworks. The Design-Builder would need to identify and address a range of issues which evolve as the project progresses, in accordance with the standards of geotechnical engineering practice throughout design and construction. As the FoP project, in its current scope, includes the new Headworks, Receiving Lift Station, Odor Control, Electrical Power Facilities and Civil Site Work; other geotechnical reports associated with these projects will need to be considered by the Design Builder and its geotechnical engineers.

- Outstanding issues regarding the further evaluation and selection of pipe support alternatives include deciding the desired support method amongst the following options:

    1. Support the first 200 feet from the Headworks plus the two manholes on piles and the rest of the pipe on ground with 'zero-net' load design.

    2. Support the full length of the pipe and the two manholes on piles.

    3. If verifiable settlement mitigation activities are completed within the 200' section in the former pond area, consider supporting the full length of the pipe with 'zero-net' load design, with only the manholes supported on piles.

- Outstanding issues regarding if and where pile founded and ground bearing sections are intermixed (e.g. Headworks to pipe or some parts of the pipe to other parts of the pipe):

    - Evaluate differential settlement concerns based on loads and other details developed for the project.

    - Evaluate differential seismic response between pile founded and ground supported segments and if warranted, performing site specific response analysis, finite element modeling or other suitable methods to address it.

    - In conjunction with the pipe designer, evaluate settlement tolerances and the suitability of pipe and any special joints and couplings under static and seismic loading conditions.

- Outstanding issues regarding the evaluation of settlement related sequencing and mitigation activities between the pipe project, the pond fill earthwork, and the construction of Headworks and other structures:

    - Evaluate possible use of mitigative methods such as wick drains and surcharge.

- Evaluate possible use of lightweight fill materials to reduce earth loads.
- Consider if it is preferred to use induced settlements plus piles to minimize pile down drag and future ground separation adjacent to pile supported areas.
- Evaluate if it is preferred to drive piles pre- or post-settlement.
- Consider if it is advantageous to allow for earth-induced vertical and lateral pile loads to take place prior to connecting the piles to the pile caps.
- Establish a program to monitor settlements and safeguarding against impacts to other facilities.
- Adjust any surcharge and waiting periods to fit the project's schedule requirements and proactively keeping other disciplines informed about the time-settlement durations from the onset of design process.
- Consider subsurface lateral forces on piles at the periphery of settlement craters or at the boundaries of partial fill placement sectors to minimize induced uneven subsurface lateral surcharge forces on piles.

Section 7

Limitations

Our services consist of professional opinions, conclusions, and recommendations that are made in accordance with generally accepted geotechnical engineering principles and practices. No other warranty expressed or implied is provided.

The analyses and recommendations contained in this report are based on the data obtained from the subsurface explorations conducted for this study. These explorations indicate subsurface conditions only at specific locations and times, and only to the depths penetrated. Variations may exist and conditions not observed or described in this report could be encountered during construction. Our conclusions and recommendations are based on our analysis of the observed conditions.

In the event of any changes to the project, the conclusions and recommendations contained in this report is not valid unless new information is reviewed and the conclusions and recommendations are modified by the Geotechnical Engineer.

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Section 8

References

Brabb, E.E., Graymer, R.W., and Jones, D.L. (1998). Geology of the onshore part of San Mateo County, California: a digital database: U.S. Geological Survey, Open-File Report OF-98-137, scale 1:62,500.

Brabb, E.E. and Pampeyan, E.H. (1983). Geologic map of San Mateo County, California: U.S. Geological Survey, Miscellaneous Investigations Series Map I-1257-A, scale 1:62,500.

Cooper, Clark & Associates (1978a). "Foundation Investigation, Proposed Subregional Wastewater Works, Redwood City, California", Prepared for the 'South Bayside System Authority', February 15.

Cooper, Clark & Associates (1978b). "Supplementary Subsurface Investigation and Laboratory Testing, SBSA Project Unit No. 1, Redwood City, California", Prepared for the 'South Bayside System Authority', October 18.

Cooper, Clark & Associates (1980). "Progress Report: Installation and Observation of Groundwater Wells and Piezometers, proposed Main Structure, Redwood City, California", Prepared for the 'South Bayside System Authority', November 07.

Cooper, Clark & Associates (1981). "Consultation: Re: Proposed Influent/Effluent Tie-In to Existing Force Main, Wastewater Treatment Plant, Redwood City, California", Prepared for the 'South Bayside System Authority', May 07.

Dames & Moore (1978). "Soil Investigation and Slope Stability Evaluations Construction Excavations, Subregional Wastewater Works, Redwood City, California" for South Bayside System Authority, December 22.

DCM|GeoEngineers (2009). "Technical Memorandum: New Administration and Plant Control Building Project, South Bayside System Authority Wastewater Treatment Plant, Redwood City, California", Prepared for South Bayside Authority, July 06.

DCM Consulting (2014). "Cone Penetrometer Test (CPT) Logs from the CPT Investigation at the SVCWTP site", Directed by David Mathy.

DCM Consulting (2015). "Supplemental Cone Penetrometer Test (CPT) Logs from the CPT Investigation at the SVCWTP site", Directed by David Mathy.

Fugro (2002). "Geotechnical Investigation and Data Report, SBSA SWTP Recycled Water System Storage, Redwood City, California" for South Bayside System Authority, October 17.

Fugro West, Inc. (2004a). "Recommended Su Profile for Shoring Design (Revised), South Bayside System Authority (SBSA), Redwood City, California", Prepared for the 'South Bayside System Authority', July 14.

Fugro West, Inc. (2004b). "Geotechnical Study: Recycled Water Storage and Disinfection Facilities, South Bayside System Authority, Redwood City, California", October 20.

Fugro West, Inc. (2004c). "Supplemental Geotechnical Recommendations, Recycled Water Storage and Disinfection Facilities, South Bayside System Authority (SBSA), Redwood City, California", Prepared for the 'South Bayside System Authority', November 08.

Figures

CDM Smith

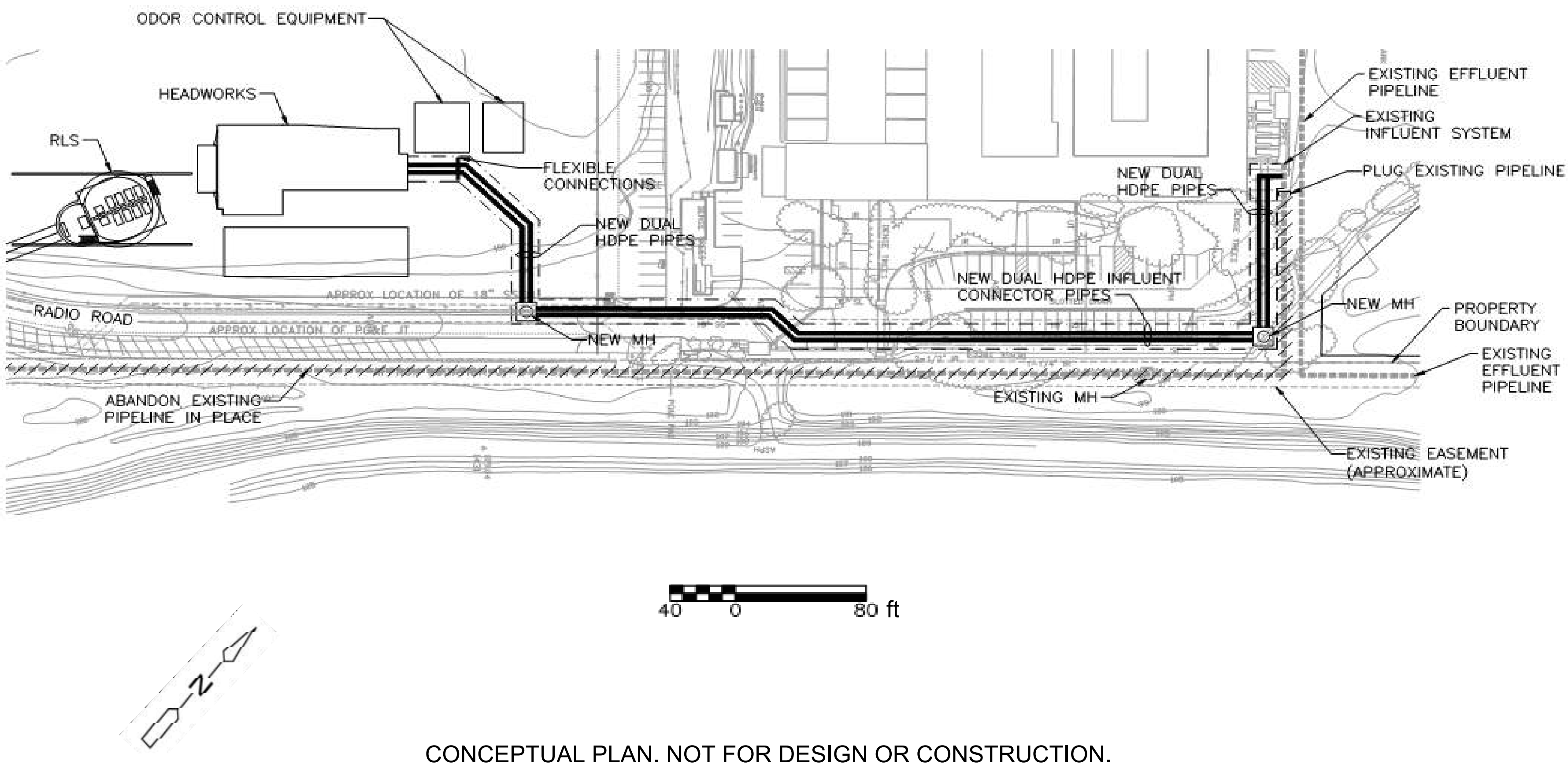
Figure 1

Vicinity Map

SVCW Gravity Influent Connector Geotechnical Interpretive Report

Project No. 111593-2015-03

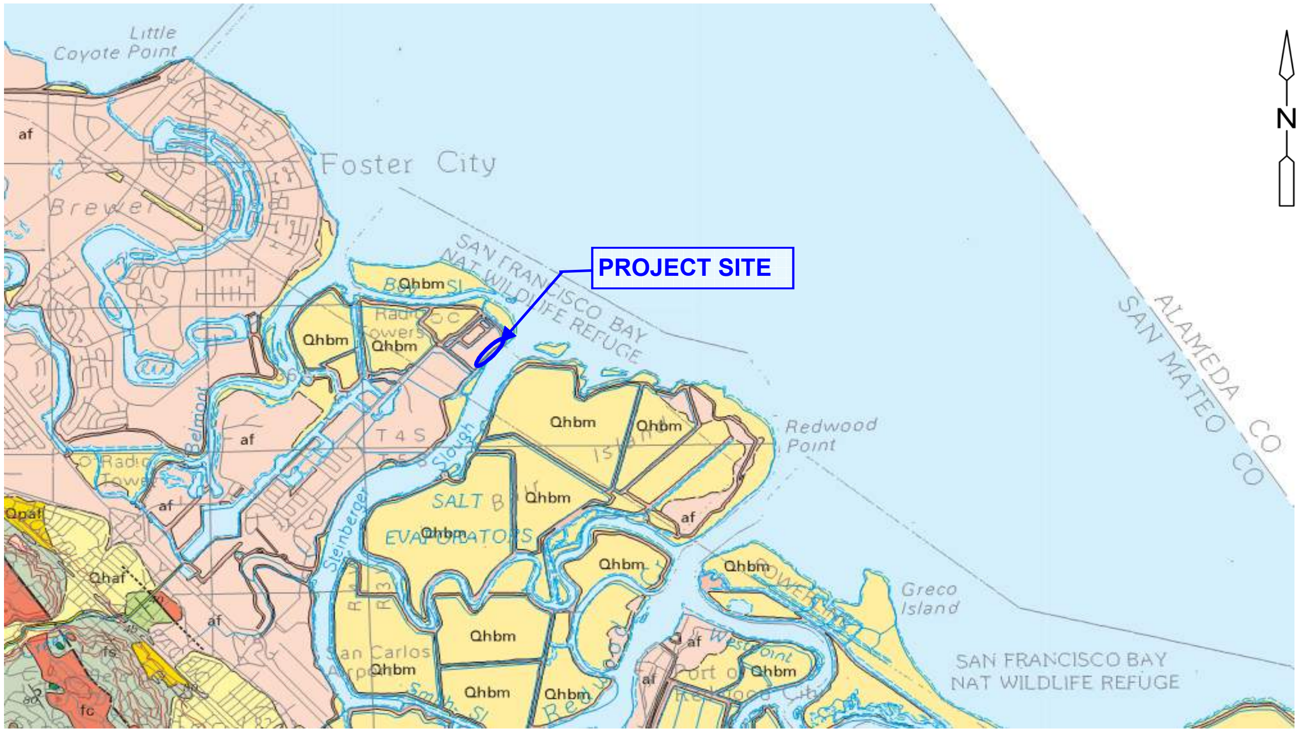
Silicon Valley Clean Water, Redwood City, California



CDM Smith

Figure 2
Site Plan

SVCW Gravity Influent Connector Geotechnical Interpretive Report
Project No. 111593-2015-03
Silicon Valley Clean Water, Redwood City, California



af

Artificial fill (Historic)—Loose to very well consolidated gravel, sand, silt, clay, rock fragments, organic matter, and man-made debris in various combinations. Thickness is variable and may exceed 30 m in places. Some is compacted and quite firm, but fill made before 1965 is nearly everywhere not compacted and consists simply of dumped materials

Qhbm

Bay mud (Holocene)—Water-saturated estuarine mud, predominantly gray, green and blue clay and silty clay underlying marshlands and tidal mud flats of San Francisco Bay, Pescadero, and Pacifica. The upper surface is covered with cordgrass (*Spartina* sp.) and pickleweed (*Salicornia* sp.). The mud also contains a few lenses of well-sorted, fine sand and silt, a few shelly layers (oysters), and peat. The mud interfingers with and grades into fine-grained deposits at the distal edge of Holocene fans, and was deposited during the post-Wisconsin rise in sea-level, about 12 ka to present (Imbrie and others, 1984). Mud varies in thickness from zero, at landward edge, to as much as 40 m near north County line

Ref. USGS Geologic Map (after Brabb and Pampeyan 1983)

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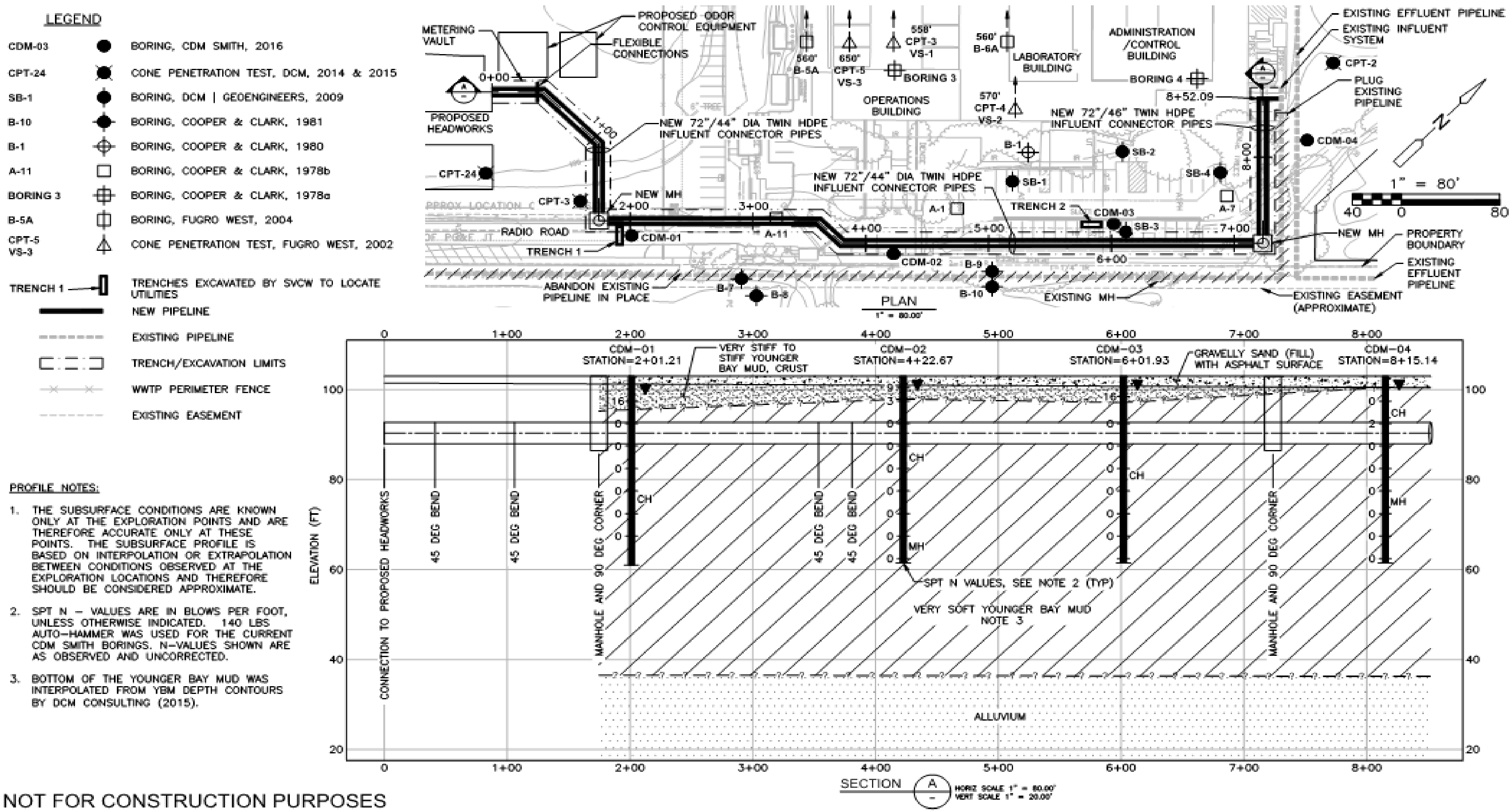
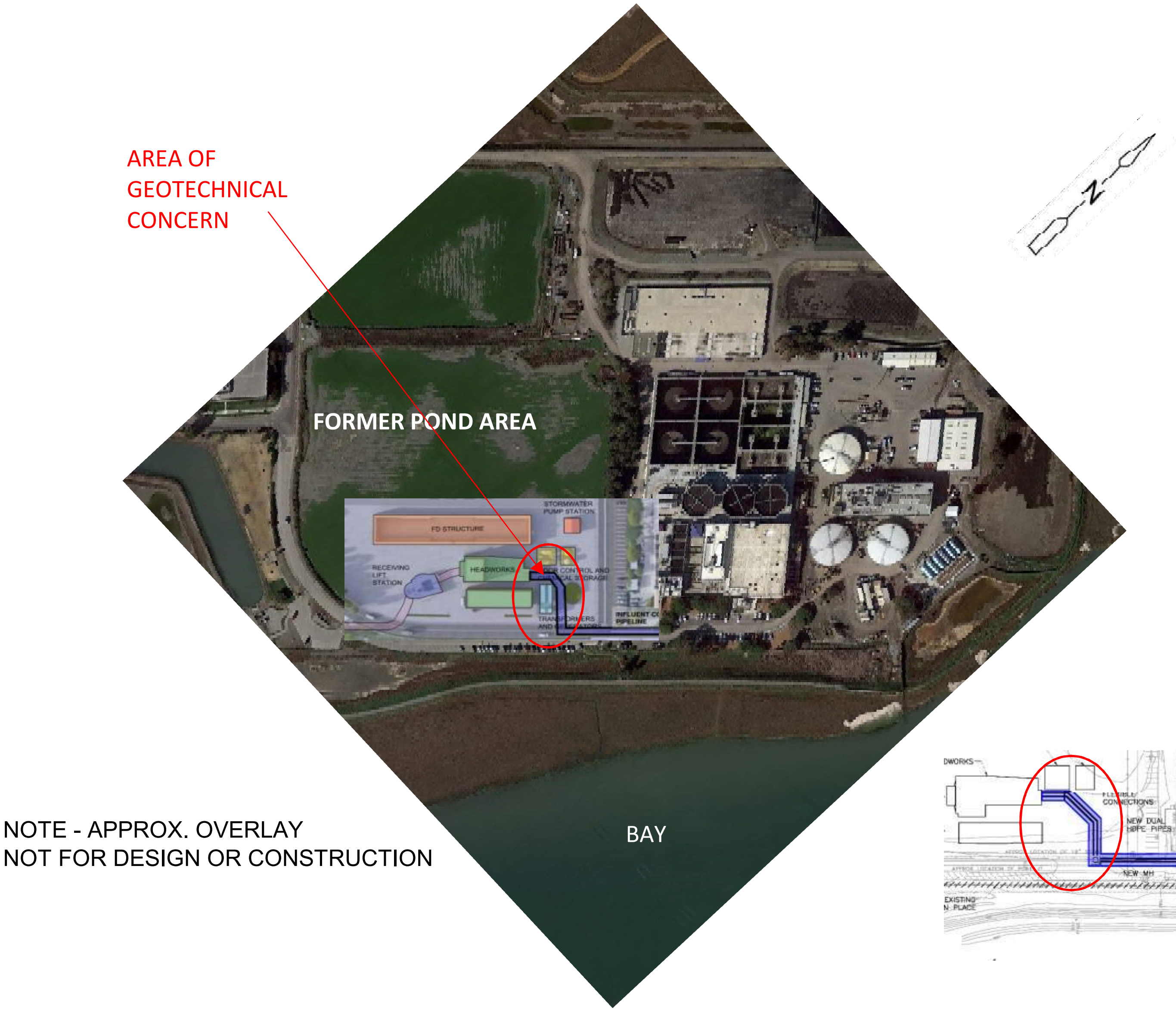
Figure 3
Geologic Map
SVCW Gravity Influent Connector Geotechnical Interpretive Report
Project No. 111593-2015-03
Silicon Valley Clean Water, Redwood City, California



NOT FOR CONSTRUCTION PURPOSES

Partial information shown. Refer to GDR for additional subsurface data including deeper historic borings and CPTs.
The report text is required for a proper interpretation. The GIR text is considered to be an integral part of this figure.

Figure 4
Generalized Subsurface Profile
SVCW Gravity Influent Connector Geotechnical Interpretive Report
Project No. 111593-2015-03
Silicon Valley Clean Water, Redwood City, California



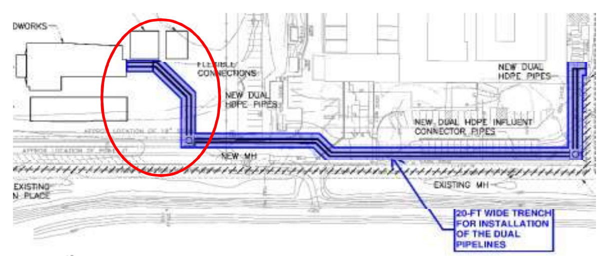


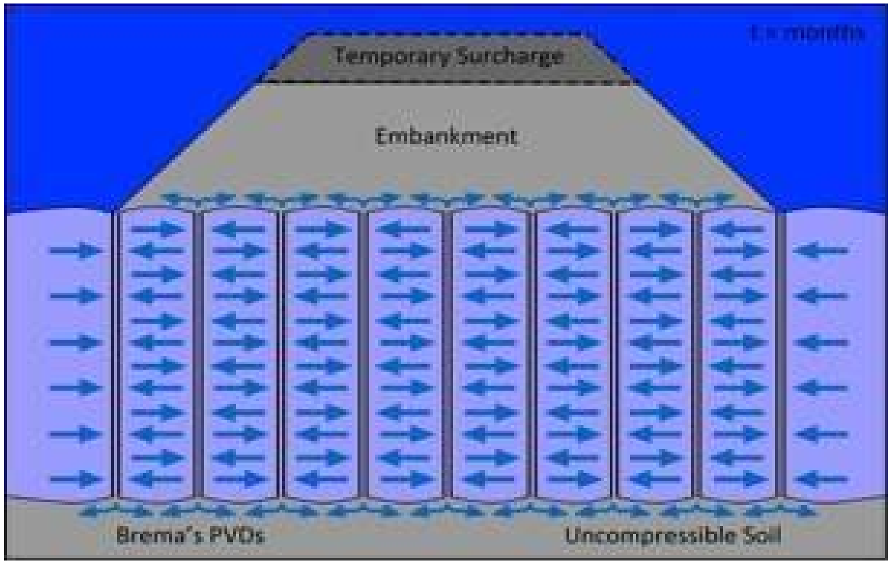
Figure 5

Area of Geotechnical Concern

SVCW Gravity Influent Connector Geotechnical Interpretive Report

Project No. 111593-2015-03

Silicon Valley Clean Water, Redwood City, California



Ref. http://www.brema-brata.com/brema/Geosynthetics/Entries/2009/7/13_PVD.html

NOTE - ONLY FOR CONCEPT OF APPLICATION. SUBJECT TO FURTHER GEOTECHNICAL EVALUATION AND COORDINATION WITH OTHER WORK IN THE FORMER POND AREA

CDM Smith

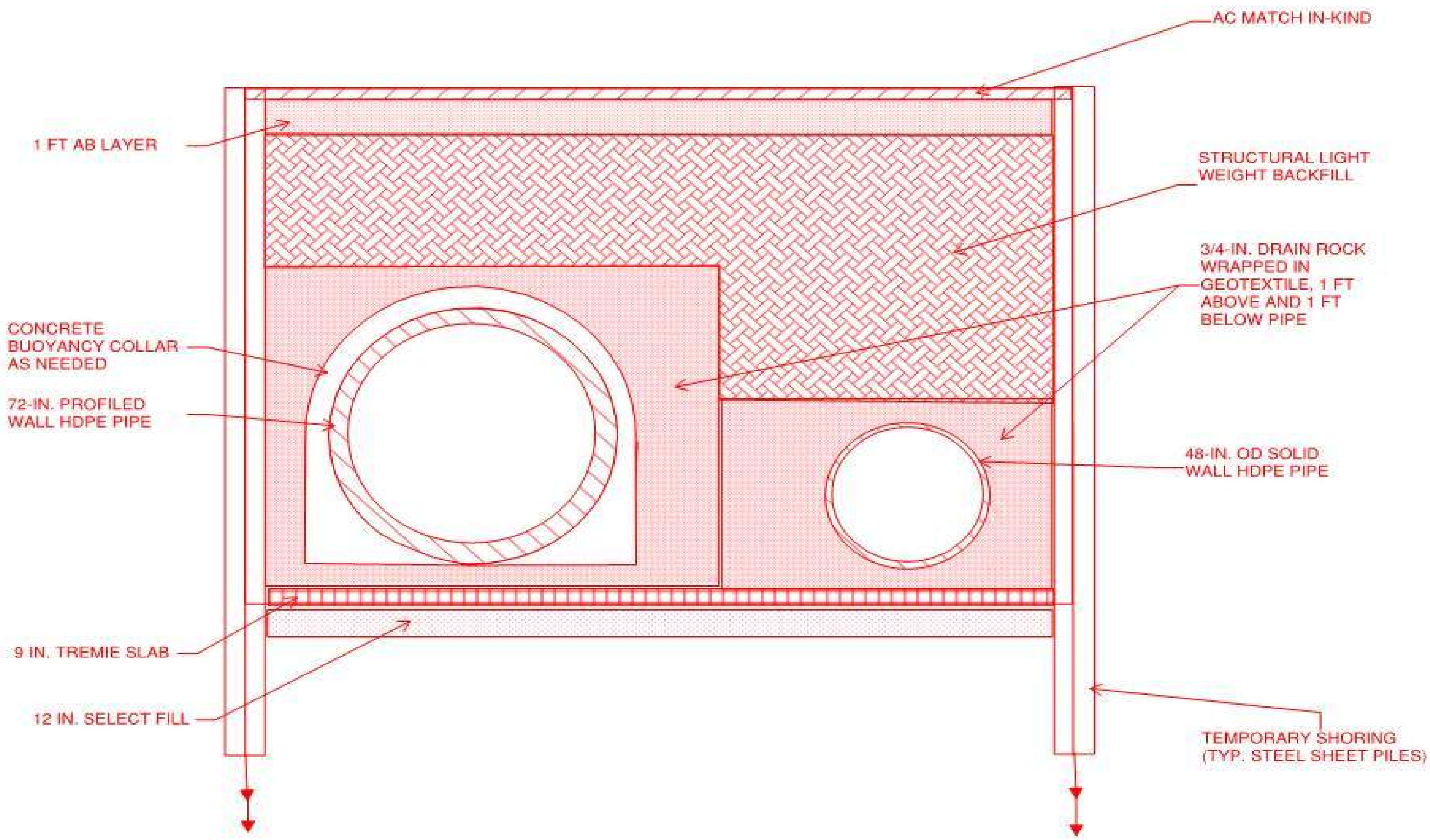
Figure 6

Typical Wick Drain Application

SVCW Gravity Influent Connector Geotechnical Interpretive Report

Project No. 111593-2015-03

Silicon Valley Clean Water, Redwood City, California



TYPICAL TRENCH DETAIL (N.T.S.) - NOT FOR DESIGN OR CONSTRUCTION
(APPLICABILITY OF GROUND SUPPORT VERSUS PILE SUPPORT tBD BASED ON FURTHER EVALUATION)

CDM Smith

Figure 7

Typical Trench Detail for Ground Support

SVCW Gravity Influent Connector Geotechnical Interpretive Report
Project No. 111593-2015-03
Silicon Valley Clean Water, Redwood City, California

Ref. Sample Photo of Pipe on Piles - www.LangleyConcreteGroup.com

PHOTO SHOWN FOR CONCEPT ILLUSTRATION ONLY, ACTUAL DESIGN WILL VARY
(APPLICABILITY OF GROUND SUPPORT VERSUS PILE SUPPORT TBD BASED ON FURTHER EVALUATION)

CDM Smith

Figure 8
Typical Pile and Cradle Supported Pipe
SVCW Gravity Influent Connector Geotechnical Interpretive Report
Project No. 111593-2015-03
Silicon Valley Clean Water, Redwood City, California

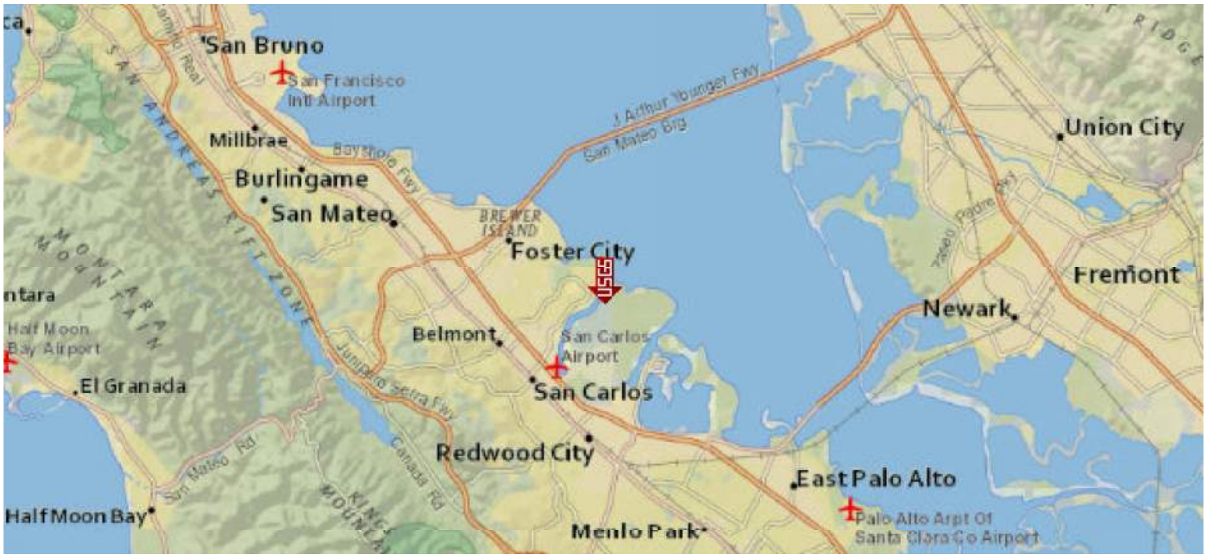
Appendix A

Seismic Analysis

Design Maps Summary Report

User–Specified Input

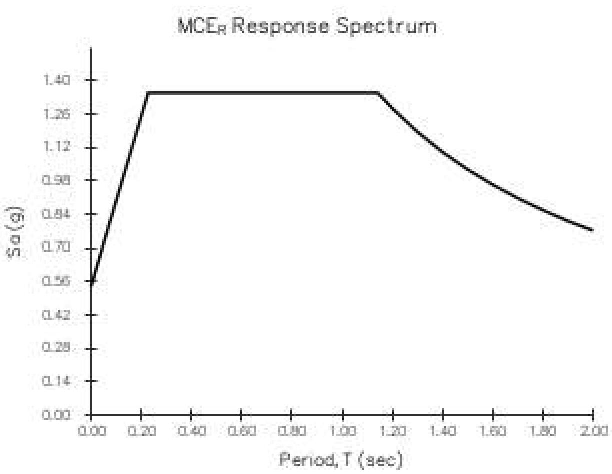
Report Title	SVCW Gravity Influent Connector Fri March 24, 2017 18:18:40 UTC
Building Code Reference Document	ASCE 7-10 Standard (which utilizes USGS hazard data available in 2008)
Site Coordinates	37.54371°N, 122.2278°W
Site Soil Classification	Site Class E – "Soft Clay Soil"
Risk Category	I/II/III

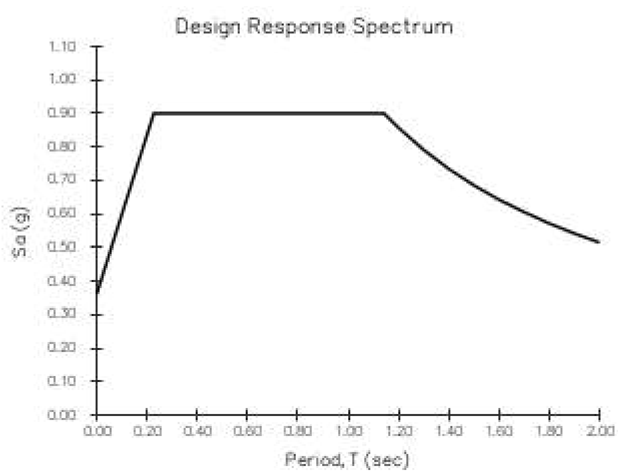


USGS–Provided Output

S_S =	1.500 g	S_{MS} =	1.350 g	S_{DS} =	0.900 g
S_1 =	0.642 g	S_{M1} =	1.542 g	S_{D1} =	1.028 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.





For PGA_M, T_L, C_{RS}, and C_{R1} values, please view the detailed report.

USGS Design Maps Detailed Report

ASCE 7-10 Standard (37.54371°N, 122.2278°W)

Site Class E – "Soft Clay Soil", Risk Category I/II/III

Section 11.4.1 — Mapped Acceleration Parameters

Note: Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain S_S) and 1.3 (to obtain S_1). Maps in the 2010 ASCE-7 Standard are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 11.4.3.

From Figure 22-1 [1] S_S = 1.500 g

From Figure 22-2 [2] S_1 = 0.642 g

Section 11.4.2 — Site Class

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class E, based on the site soil properties in accordance with Chapter 20.

Table 20.3–1 Site Classification

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard Rock	>5,000 ft/s	N/A	N/A
B. Rock	2,500 to 5,000 ft/s	N/A	N/A
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff Soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the characteristics: • Plasticity index *PI* > 20, • Moisture content *w* ≥ 40%, and • Undrained shear strength $\bar{s}_u$ < 500 psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

Section 11.4.3 — Site Coefficients and Risk–Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameters

Table 11.4–1: Site Coefficient F_a

Site Class	Mapped MCE_R Spectral Response Acceleration Parameter at Short Period				
	$S_S \le 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_S

For Site Class = E and S_S = 1.500 g, F_a = 0.900

Table 11.4–2: Site Coefficient F_v

Site Class	Mapped MCE_R Spectral Response Acceleration Parameter at 1–s Period				
	$S_1 \le 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \ge 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_1

For Site Class = E and S_1 = 0.642 g, F_v = 2.400

Equation (11.4–1):	$S_{MS} = F_aS_S = 0.900 \text{ x } 1.500 = 1.350$ g
Equation (11.4–2):	$S_{M1} = F_vS_1 = 2.400 \text{ x } 0.642 = 1.542$ g

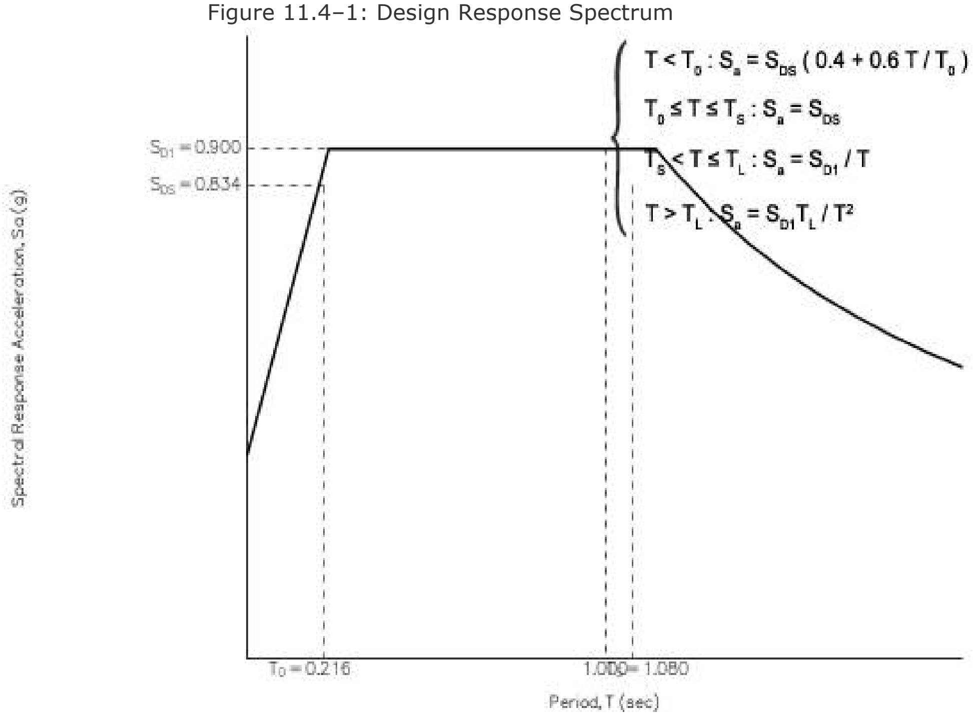
Section 11.4.4 — Design Spectral Acceleration Parameters

Equation (11.4–3):	$S_{DS} = ⅔\ S_{MS} = ⅔ \text{ x } 1.350 = 0.900$ g
Equation (11.4–4):	$S_{D1} = ⅔\ S_{M1} = ⅔ \text{ x } 1.542 = 1.028$ g

Section 11.4.5 — Design Response Spectrum

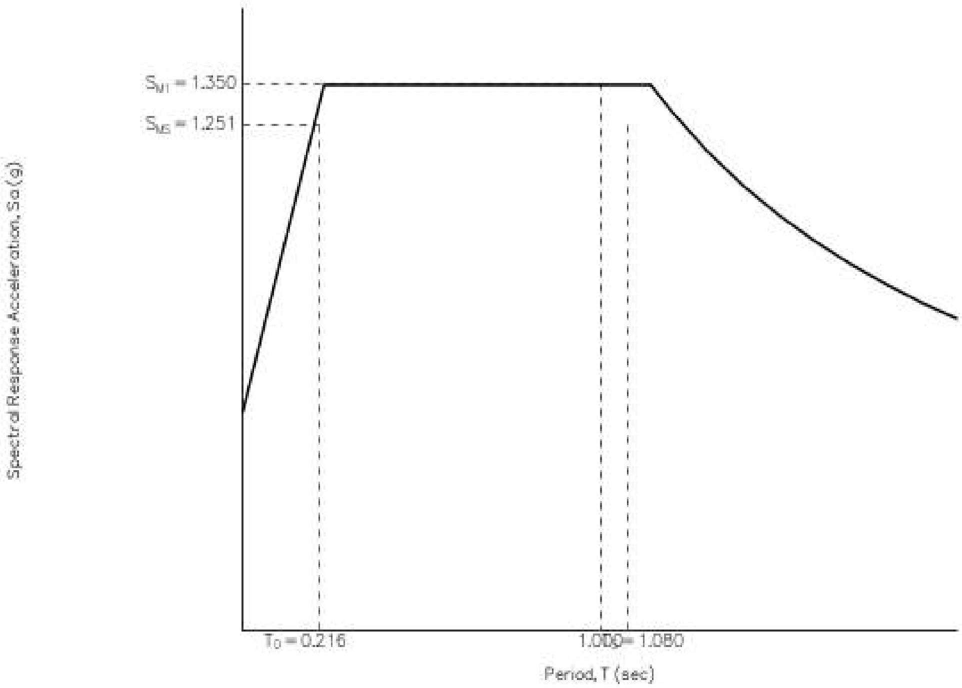
From Figure 22-12 [3]	T_L = 12 seconds

Figure 11.4–1: Design Response Spectrum



Section 11.4.6 — Risk-Targeted Maximum Considered Earthquake (MCE_R) Response Spectrum

The MCE_R Response Spectrum is determined by multiplying the design response spectrum above by 1.5.



Section 11.8.3 — Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F

From Figure 22-7 [4]	PGA = 0.570
Equation (11.8–1):	$PGA_M = F_{PGA}PGA = 0.900 \times 0.570 = 0.513$ g

Table 11.8–1: Site Coefficient F_{PGA}

Site Class	Mapped MCE Geometric Mean Peak Ground Acceleration, PGA				
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA ≥ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	$\boxed{0.9}$
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of PGA

For Site Class = E and PGA = 0.570 g, F_{PGA} = 0.900

Section 21.2.1.1 — Method 1 (from Chapter 21 – Site-Specific Ground Motion Procedures for Seismic Design)

From Figure 22-17 [5]	C_{RS} = 1.035
From Figure 22-18 [6]	C_{R1} = 0.978

Section 11.6 — Seismic Design Category

Table 11.6-1 Seismic Design Category Based on Short Period Response Acceleration Parameter

VALUE OF S_{DS}	RISK CATEGORY		
	I or II	III	IV
$S_{DS} < 0.167g$	A	A	A
$0.167g \le S_{DS} < 0.33g$	B	B	C
$0.33g \le S_{DS} < 0.50g$	C	C	D
$0.50g \le S_{DS}$	D	D	D

For Risk Category = I and S_{DS} = 0.900 g, Seismic Design Category = D

Table 11.6-2 Seismic Design Category Based on 1-S Period Response Acceleration Parameter

VALUE OF S_{D1}	RISK CATEGORY		
	I or II	III	IV
$S_{D1} < 0.067g$	A	A	A
$0.067g \le S_{D1} < 0.133g$	B	B	C
$0.133g \le S_{D1} < 0.20g$	C	C	D
$0.20g \le S_{D1}$	D	D	D

For Risk Category = I and S_{D1} = 1.028 g, Seismic Design Category = D

Note: When S_1 is greater than or equal to 0.75g, the Seismic Design Category is **E** for buildings in Risk Categories I, II, and III, and **F** for those in Risk Category IV, irrespective of the above.

Seismic Design Category ≡ “the more severe design category in accordance with Table 11.6-1 or 11.6-2” = D

Note: See Section 11.6 for alternative approaches to calculating Seismic Design Category.

References

1. *Figure 22-1*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-1.pdf
2. *Figure 22-2*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-2.pdf
3. *Figure 22-12*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-12.pdf
4. *Figure 22-7*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-7.pdf
5. *Figure 22-17*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-17.pdf
6. *Figure 22-18*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-18.pdf

USGS Design Maps Summary Report

User–Specified Input

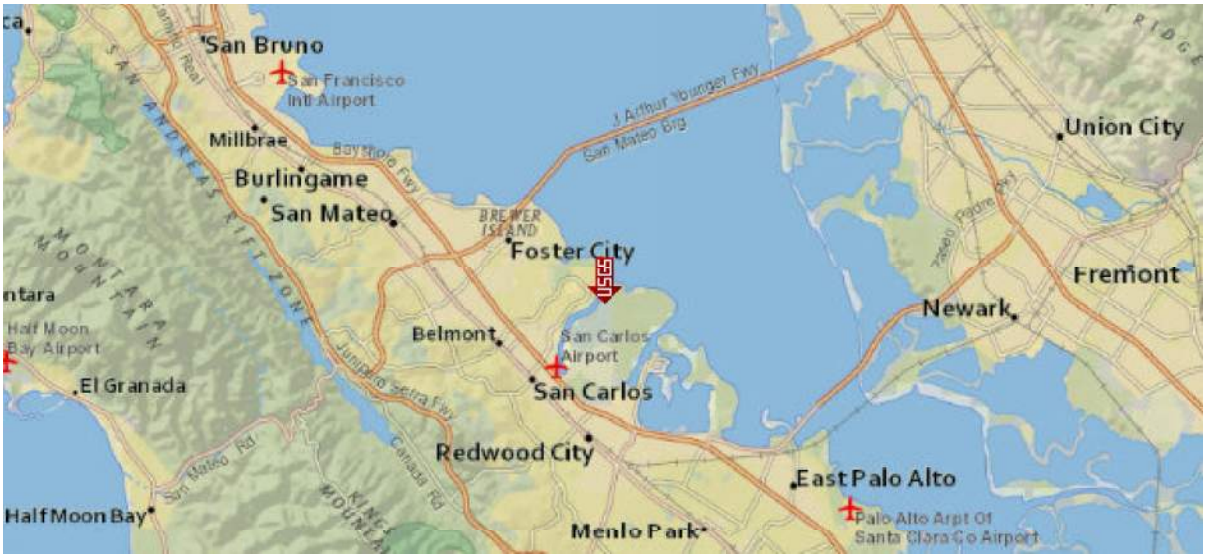
Report Title SVCW Gravity Influent Connector
Tue March 21, 2017 15:52:11 UTC

Building Code Reference Document ASCE 7-10 Standard
(which utilizes USGS hazard data available in 2008)

Site Coordinates 37.54371°N, 122.2278°W

Site Soil Classification Site Class E – "Soft Clay Soil"

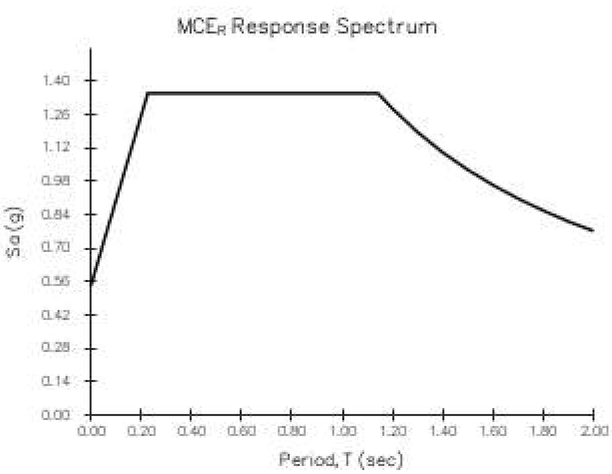
Risk Category IV (e.g. essential facilities)

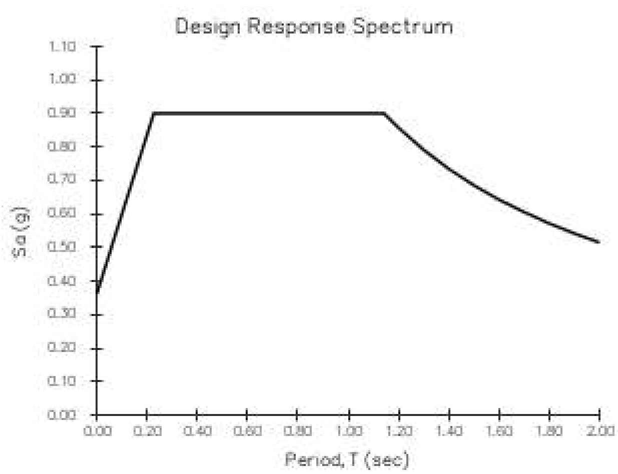


USGS–Provided Output

S_S =	1.500 g	S_{MS} =	1.350 g	S_{DS} =	0.900 g
S_1 =	0.642 g	S_{M1} =	1.542 g	S_{D1} =	1.028 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.





For PGA_M, T_L, C_{RS}, and C_{R1} values, please view the detailed report.

Design Maps Detailed Report

ASCE 7-10 Standard (37.54371°N, 122.2278°W)

Site Class E – "Soft Clay Soil", Risk Category IV (e.g. essential facilities)

Section 11.4.1 — Mapped Acceleration Parameters

Note: Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain S_S) and 1.3 (to obtain S_1). Maps in the 2010 ASCE-7 Standard are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 11.4.3.

From Figure 22-1 [1]	S_S = 1.500 g
From Figure 22-2 [2]	S_1 = 0.642 g

Section 11.4.2 — Site Class

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class E, based on the site soil properties in accordance with Chapter 20.

Table 20.3–1 Site Classification

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard Rock	>5,000 ft/s	N/A	N/A
B. Rock	2,500 to 5,000 ft/s	N/A	N/A
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff Soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the characteristics: • Plasticity index *PI* > 20, • Moisture content *w* ≥ 40%, and • Undrained shear strength $\bar{s}_u$ < 500 psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

Section 11.4.3 — Site Coefficients and Risk–Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameters

Table 11.4–1: Site Coefficient F_a

Site Class	Mapped MCE_R Spectral Response Acceleration Parameter at Short Period				
	$S_S \le 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	$\boxed{0.9}$
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_S

For Site Class = E and S_S = 1.500 g, F_a = 0.900

Table 11.4–2: Site Coefficient F_v

Site Class	Mapped MCE_R Spectral Response Acceleration Parameter at 1–s Period				
	$S_1 \le 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \ge 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	$\boxed{2.4}$
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_1

For Site Class = E and S_1 = 0.642 g, F_v = 2.400

Equation (11.4–1): $S_{MS} = F_aS_S = 0.900 \times 1.500 = 1.350$ g

Equation (11.4–2): $S_{M1} = F_vS_1 = 2.400 \times 0.642 = 1.542$ g

Section 11.4.4 — Design Spectral Acceleration Parameters

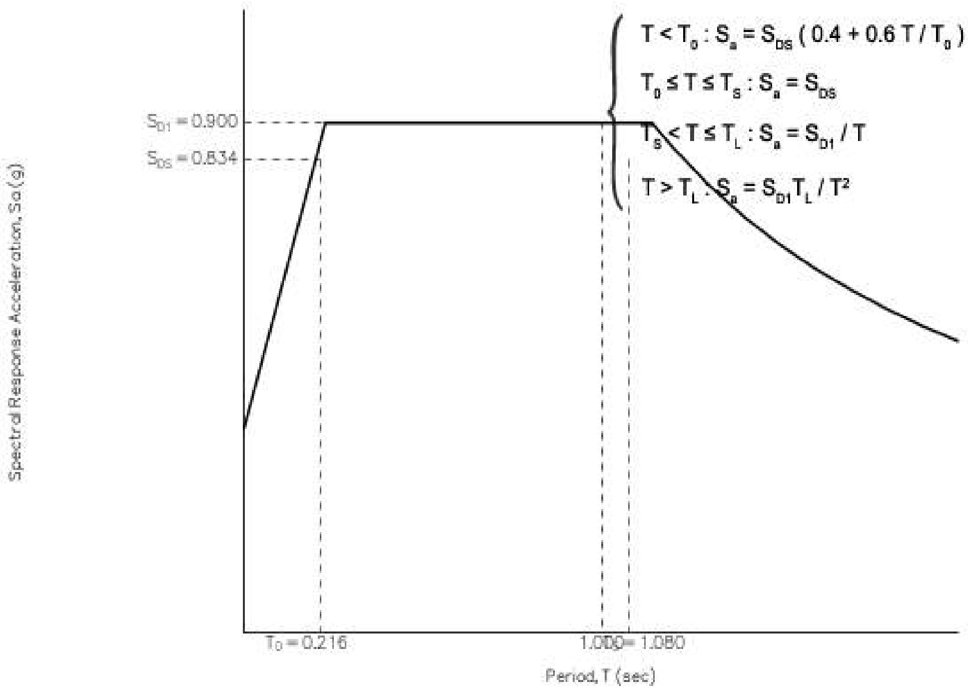
Equation (11.4–3): $S_{DS} = ⅔ S_{MS} = ⅔ \times 1.350 = 0.900$ g

Equation (11.4–4): $S_{D1} = ⅔ S_{M1} = ⅔ \times 1.542 = 1.028$ g

Section 11.4.5 — Design Response Spectrum

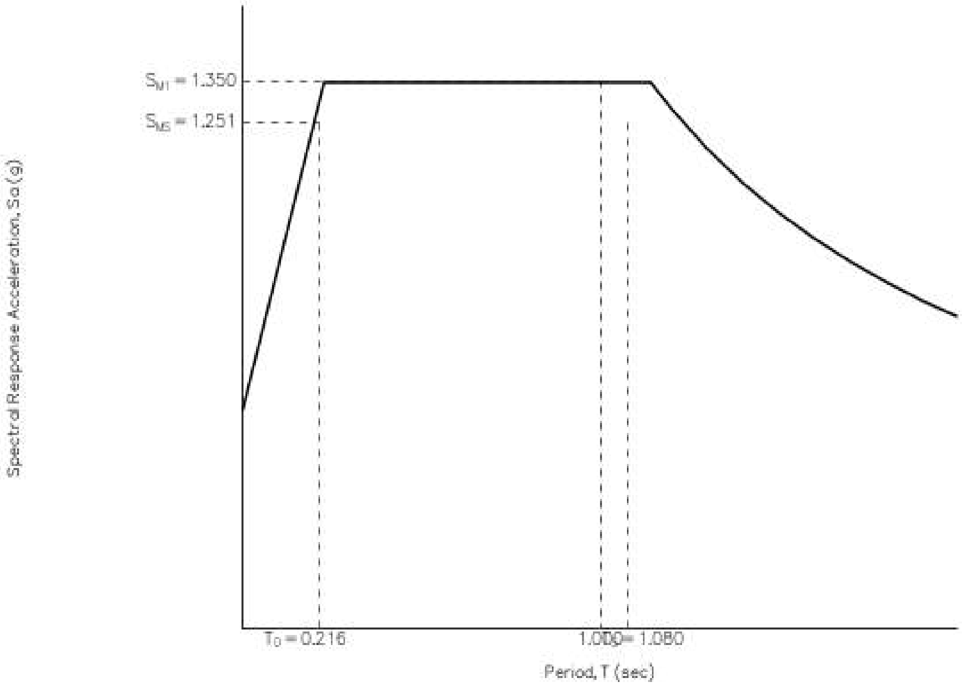
From Figure 22-12 [3] $T_L = 12$ seconds

Figure 11.4–1: Design Response Spectrum



Section 11.4.6 — Risk-Targeted Maximum Considered Earthquake (MCE_R) Response Spectrum

The MCE_R Response Spectrum is determined by multiplying the design response spectrum above by 1.5.



Section 11.8.3 — Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F

From Figure 22-7 [4] PGA = 0.570

Equation (11.8–1): $PGA_M = F_{PGA}PGA = 0.900 \times 0.570 = 0.513$ g

Table 11.8–1: Site Coefficient F_{PGA}

Site Class	Mapped MCE Geometric Mean Peak Ground Acceleration, PGA				
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA ≥ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of PGA

For Site Class = E and PGA = 0.570 g, F_{PGA} = 0.900

Section 21.2.1.1 — Method 1 (from Chapter 21 – Site-Specific Ground Motion Procedures for Seismic Design)

From Figure 22-17 [5] C_{RS} = 1.035

From Figure 22-18 [6] C_{R1} = 0.978

Section 11.6 — Seismic Design Category

Table 11.6-1 Seismic Design Category Based on Short Period Response Acceleration Parameter

VALUE OF S_{DS}	RISK CATEGORY		
	I or II	III	IV
$S_{DS} < 0.167g$	A	A	A
$0.167g \le S_{DS} < 0.33g$	B	B	C
$0.33g \le S_{DS} < 0.50g$	C	C	D
$0.50g \le S_{DS}$	D	D	D

For Risk Category = IV and S_{DS} = 0.900 g, Seismic Design Category = D

Table 11.6-2 Seismic Design Category Based on 1-S Period Response Acceleration Parameter

VALUE OF S_{D1}	RISK CATEGORY		
	I or II	III	IV
$S_{D1} < 0.067g$	A	A	A
$0.067g \le S_{D1} < 0.133g$	B	B	C
$0.133g \le S_{D1} < 0.20g$	C	C	D
$0.20g \le S_{D1}$	D	D	D

For Risk Category = IV and S_{D1} = 1.028 g, Seismic Design Category = D

Note: When S_1 is greater than or equal to 0.75g, the Seismic Design Category is **E** for buildings in Risk Categories I, II, and III, and **F** for those in Risk Category IV, irrespective of the above.

Seismic Design Category ≡ "the more severe design category in accordance with Table 11.6-1 or 11.6-2" = D

Note: See Section 11.6 for alternative approaches to calculating Seismic Design Category.

References

1. *Figure 22-1*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-1.pdf
2. *Figure 22-2*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-2.pdf
3. *Figure 22-12*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-12.pdf
4. *Figure 22-7*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-7.pdf
5. *Figure 22-17*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-17.pdf
6. *Figure 22-18*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-18.pdf

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