Item No.5d
Town of Atherton

PARK AND RECREATION COMMITTEE – REGULAR AGENDA

TO: PARK AND RECREATION COMMITTEE

FROM: ROBERT OVADIA, PUBLIC WORKS DIRECTOR

DATE: MAY 2, 2018

SUBJECT: RECEIVE THE PRELIMINARY ENGINEERING DESIGN REPORT AND TAKE A VOTE TO SUPPORT OR NOT TO SUPPORT A WATER CAPTURE FACILITY IN HOLBROOK-PALMER PARK

RECOMMENDATION

Receive the Tetra Tech, Inc. Preliminary Engineering (20 percent) Design Report for the water capture facility project and take a vote to support or not to support the water capture project in Holbrook-Palmer Park.

BACKGROUND

The need for a stormwater detention facility was identified in the Town-wide Drainage Study Update prepared in 2015 to reduce flooding associated with the Atherton Channel. The update identified three possible sites for the detention facility including Las Lomitas Elementary School, Holbrook-Palmer Park, and the Menlo Park Circus Club (JL Dixon Stables). Due to the complexity of working with a privately-owned site at some distance from the Channel, the Town chose not to locate the project at the JL Dixon Stables. The Town next attempted to work with the Las Lomitas Elementary School District on a water capture project at the school. This opportunity ultimately proved unsuccessful after the Town and District were unable to come to agreement on funding and approval of a traffic signal at Walsh Road, continued maintenance, and the Town's ability to terminate the project prior to construction, if desired.

The remaining alternative was Holbrook-Palmer Park. The Town learned of a collaborative funding opportunity with Caltrans to not only provide the Town with the needed flood control project; but also, an opportunity to address the Town's requirements under the Regional Water Quality Control Permit. After being selected by Caltrans for the Project, the Town hired Tetra Tech Inc. to investigate the feasibility of installing a water capture facility in Holbrook-Palmer Park and to prepare preliminary concept designs.

DISCUSSION

The potential water capture project has been before the City Council numerous times since May 2017 and has been twice to the Parks & Recreation Committee. Below is a summary of the milestones of the potential project.

- On May 3, 2017 City Council authorized the City Attorney to review and the City Manager to sign a CIA with Caltrans to receive State funds to construct a stormwater capture facility at Las Lomitas Elementary School, and adopted a Resolution No. 17-14 for the Town of Atherton (Town) to enter in to a Cooperative Implementation Agreement (CIA) with the California Department of Transportation (Caltrans). The CIA was executed on May 24, 2017 between Caltrans and Town to construct a water capture facility in the Town of Atherton.

- On May 17, 2017 City Council authorized the City Attorney to prepare and the City Manager to execute an agreement amendment with Richard Watson & Associates (RWA), Inc. for a not to exceed fee of $85,000 to assist staff with: Development of a memorandum of understanding (MOU) to address project management, funding, and responsibilities between the Town and the Las Lomitas Elementary School District (District); and Development of a MOU with Menlo Park, Woodside, and Stanford University for ongoing operations and maintenance (O&M) for the proposed storm water capture facility located at Las Lomitas Elementary School; and Provide ongoing project management support during design and construction of the Las Lomitas School Water Capture facility.

- On July 19, 2017 City Council Authorize the City Attorney to review and the City Manager to sign a memorandum of understanding (MOU) with the Las Lomitas School District to address project management, funding, and responsibilities for the proposed Las Lomitas Elementary School storm water capture facility.

- On August 1, 2017 City Council reviewed the response from the Las Lomitas School District (District) for the proposed Las Lomitas storm water capture facility and directed staff to begin discussions with Caltrans to amend the Cooperative Implementation Agreement (CIA) to relocate the water capture facility project to an alternate location, i.e., Holbrook-Palmer Park.

- On September 6, 2017 City Council: Adopted a Resolution for the Town of Atherton to enter in to a Cooperative Implementation Agreement (CIA) with the California Department of Transportation (Caltrans); and Authorize the City Attorney to review and the City Manager to sign an amended CIA with Caltrans to receive State funds to construct a stormwater capture facility at Holbrook-Palmer Park; and Authorize staff to issue a request for qualifications (RFQ) for an engineering design consultant.

- On November 1, 2017 City Council authorized the City Attorney to prepare and the City Manager to execute a professional services agreement with Tetra Tech Inc., for a not to exceed fee of $550,000 to prepare only Phase I - Preliminary Engineering Concept Design and estimate services necessary for the Holbrook-Palmer Park Water Capture Facility; Approved a Task Order for Interwest Consulting Group to provide Project Management

Services through Preliminary Engineering Concept Design for a not to exceed fee of $136,200; and Authorized the City Attorney to prepare and the City Manager to execute any necessary amendments thereto.

- On January 10, 2018 staff presented the Water Capture Project to the Atherton Parks and Recreation Committee for feedback. Many of the committee members and residents in attendance were opposed to the project in the park for various reasons.

- On February 28, 2018 City Council reviewed, discussed and received a presentation on design of water capture facility projects in Lakewood, California. Agenda included: flights to and from LAX, visit to various water capture facilities including: Bolivar Park Water Capture Facility, Mayfair Park Water Capture Project Site, and Long Beach Airport Water Capture Project.

- On March 7, 2018 staff presented an update on the Water Capture project to the Atherton Parks and Recreation Committee. There were approximately 50 in attendance with many questions and mostly opposed to the project. The majority of the concerns were: the disruption of the park, hazardous materials coming into the park, on-going O&M costs, why this is needed for flood control, why not somewhere else, costs to decommission this facility, why not do green streets instead.

- On March 21, 2018 City Council discussed and provided feedback on the Special Meeting held on February 28, 2018, in Lakewood California, and provided direction that no additional public outreach for the Atherton Water Capture Facility Project was needed at this time pending the 20% concept plan. All the Councilmembers spoke very positively about the visit and how much they learned from the Lakewood projects. There were approximately 15 residents in attendance who voiced their concerns for the project in the Park. The majority of the concerns were: the disruption of the park, hazardous materials coming into the park, on-going O&M costs, why this is needed for flood control, why not somewhere else.

On April 18, 2018, the City Council received the Preliminary Engineering (20%) Design Report. Several residents were in attendance and voiced their concerns regarding the project. Following public comment, the Council directed staff to bring the Preliminary Engineering Design Report to the Park and Recreation Committee for a recommendation. Additionally, the Council requested additional information regarding water quality, project benefits related to the Bayfront Canal project, as well as additional information on potential site alternatives. The additional information and the Committee's recommendation were requested to be brought back to the Council at their May 16, 2018 meeting.

Staff is presenting the Preliminary Engineering 20% Design Report tonight and is requesting a recommendation to be brought back to the City Council.

ATTACHMENTS

Holbrook-Palmer Park Water Capture Project - Preliminary Engineering Design Report by Tetra Tech, Inc. dated: April 11, 2018

Holbrook-Palmer Park Water Capture Project Preliminary Engineering Design Report

April 11, 2018

Submitted to:
Town of Atherton
91 Ashfield Road
Atherton, CA 94027

Submitted by:
Tetra Tech, Inc.
9444 Balboa Ave, Suite 215
San Diego, CA 92103

HOLBROOK-PALMER PARK WATER CAPTURE PROJECT

PRELIMINARY ENGINEERING DESIGN REPORT

April 11, 2018

PRESENTED TO

Town of Atherton
91 Ashfield Road
Atherton, CA 94027

PRESENTED BY

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ACRONYMS/ABBREVIATIONS

Acronyms/Abbreviations	Definition
BAAQMD	Bay Area Air Quality Management District
BASMAA	Bay Area Stormwater Management Agencies Association
BGS	Below ground surface
BMP	Best Management Practice
C/CAG	City/County Association of Governments of San Mateo County
CDS	Continuous Deflective Separation
CEQA	California Environmental Quality Act
CIA	Cooperative Implementation Agreement
CFS	Cubic feet per second
CORS	Continuously Operating Reference Station
CPT	Cone penetration test
CSRC	California Spatial Reference Center
CWA	Clean Water Act
DSBB	Debris Separating Baffle Box
GHGs	Green House Gas Emissions
GI	Green infrastructure
GIS	Geographic Information Systems
GPS	Global Positioning System
H&H	Hydrologic & hydraulic
IS/MND	Initial Study/Mitigated Negative Declaration
JDS	Jensen Deflective Separator
LiDAR	Light Detection and Ranging
LSPC	Loading Simulation Program C++
MRP	Municipal Regional Stormwater Permit
MS4	Multiple Separate Storm Sewer System
NAD83	North American Datum 1988
NAVD88	North American Vertical Datum 1988
NLCD	National Land Cover Database
NPDES	National Pollution Discharge & Elimination System
NOI	Notice of Intent
NSBB	Nutrient Separating Baffle Box
O&M	Operations and maintenance
PCBs	Polychlorinated biphenyls

Acronyms/Abbreviations	Definition
PEDR	Preliminary Engineering Design Report
POCs	Pollutants of concern
PSF	Pounds per square foot
RAA	Reasonable Assurance Analysis
RTC	Real-time control
SFEI	San Francisco Estuary Institute
SRP	Stormwater Resources Plan
SUSTAIN	System for Urban Stormwater Treatment and Analysis Integration
SWPPP	Storm Water Pollution Prevention Plans
TMDL	Total Maximum Daily Load
TSS	Total Suspended Solids
USEPA	United States Environmental Protection Agency
WLA	Waste load allocation
WY	Water year

EXECUTIVE SUMMARY

This Preliminary Engineering Design Report (PEDR) for a storm water capture facility at Holbrook-Palmer Park (Park) was prepared for the Town of Atherton (Town) to highlight key project design components and to demonstrate how the project safely provides co-benefits of both water quality improvement and flood management. Additional site improvement components for the Park include landscaping, new pathways, an upgraded entry bridge rated for maintenance and emergency vehicles, and an improved irrigation system. The drainage area at the proposed point of diversion in the Atherton Channel is 2,843 acres and includes the Towns of Atherton and Woodside, City of Menlo Park, Stanford Lands and Unincorporated San Mateo County. The dominant land use within the watershed is residential, with a composite imperviousness of approximately 21%. The Park site provides a unique and regionally valuable location for stormwater capture because: (1) it is adjacent to a major flood control channel, owned and operated by the Town of Atherton, (2) has sufficient space for stormwater storage to meaningfully contribute towards multiple stormwater management goals, and (3) project configuration will allow for Park functionality to be maintained following construction, consistent with the Holbrook-Palmer Park Master Plan (Town of Atherton, 2015b).

The Park was identified as a priority site for a regional water capture project in both the Townwide Drainage Study Update (Town of Atherton, 2015a) and the San Mateo County Stormwater Resource Plan (SRP, 2017). The Townwide Drainage Study Update analyzes existing Town-level drainage network conditions and reported flooding complaints to present prioritized drainage system improvement project locations and preliminary design criteria. It was determined that approximately 10 ac-ft of storage would be required in the Town to alleviate flooding and allow for conveyance of the 10-year storm downstream of the Park at the Marsh Road culvert. At a County-wide planning assessment level, the SRP identifies pollutant load reduction strategies to comply with the San Francisco Bay Municipal Regional Stormwater NPDES Permit (MRP) and total maximum daily loads (TMDLs) for polychlorinated biphenyls (PCBs) and mercury (SFRWQCB, 2008, SFRWQCB, 2004, amended by SFRWQCB, 2006). The Park was identified in the SRP as an optimal location for regional water quality management due to the prime location, large contributing drainage area, and sizeable available footprint. The Town and the County, both having identified the Park as a priority location for safely improving water quality and flood prevention, coordinated with Caltrans via a Cooperative Implementation Agreement (CIA) to fund the design of the multi-benefit project.

The main objective of the project is to optimize the key performance components of the storm water capture facility (diversion rate, storage capacity, and outflow) for flood control per the Townwide Drainage Study Update, while also making progress towards regulatory pollutant load reduction targets. Several alternative configurations were assessed for compatibility with the Holbrook-Palmer Park Master Plan and to minimize disturbance to overall Park activities both during construction and for long term operations and maintenance (O&M). Conceptual layouts and cost estimates were prepared for the recommended capture facility configuration, supported by the detailed analyses included in this PEDR. On-site infiltration rates are restricted beneath the project footprint per the Geotechnical Exploration Report dated April 9, 2018 (ENGEO 2018), resulting in the recommendation to utilize cartridge filtration units (Kraken) to provide water quality treatment and outflow from the subsurface storage facility. Site configuration and underground infrastructure are amenable to managing diverted stormwater from the Atherton Channel by gravity piping to the storage/filtration/pump system, which will return cleaner stormwater back to the Atherton Channel at off-peak flow times. Key details of the optimal stormwater capture facility configuration are:

- diversion from the Atherton Channel at a rate of 100 cubic feet per second (cfs),
- construction of approximately 8.94 acre-feet of subsurface storage beneath the recreational area of the Park with a hard-bottomed vault with openings to allow for incidental infiltration. The vault would have a 10-foot ponded depth,
- and, a Kraken filtration unit with a treatment flow rate of 2.88 cfs.

The utilization of real-time controls (RTC) to target the peak runoff from storms would allow the facility to ***reduce the peak flowrate by 100 cfs*** for all flooding design storms simulated (e.g., 5-year, 7-year, 10-year, and 100-year, 24-hour storms). A long-term simulation using the most recent 15 years of rainfall data (62 storms total) was developed to assess the impacts on smaller, more frequently occurring storm events. The storage facility maintained peak flows downstream of the Park below 500 cfs, which is significantly lower than the conveyance capacity at the Marsh Road culvert (900 cfs). Additional benefits associated with this capture facility configuration include an estimated average annual capture volume of 194 ac-ft and load reduction of 6.42 g/year PCBs and 12.93 g/year mercury based on preliminary RAA baseline models (mercury time series were not provided for the preliminary RAA models at this time, so sediment as a proxy was used). Pre-treatment and removal of trash from the Atherton Channel would also assist the Town with complying with MRP trash reduction requirements.

The total estimated capital costs for this project are $13,513,168, with $2,064,370 for engineering costs and $11,448,798 for construction costs. The proposed stormwater capture facility would meet the budgetary constraints of the Caltrans CIA agreement for capital costs, while maximizing flood risk management and pollutant removal benefits to the extent feasible. The proposed schedule for the project will also meet the Caltrans CIA expenditure milestones, with a completion date of March 31, 2021.

1.0 PROJECT OBJECTIVES

The design and implementation of a storm water capture project at the Park is driven by multiple objectives:

1. Contribute towards the Town's water quality load reduction requirements per the MRP and Mercury and PCB TMDLs (see Section 3.0 for more information),
2. Alleviate local and/or downstream flooding through storm water detention, and
3. Minimize impacts to Park activities and circulation both during construction and following completion.

Additionally, the configuration and design of the storm water capture facility has been developed to maximize consistency with the Holbrook-Palmer Park Master Plan (Town of Atherton, 2015b).

The objective of this PEDR is to provide the Town with 20% design-level documents that will ultimately guide the development of the 100% detailed design documents. The preliminary design concepts presented in this PEDR will be optimized to meet the needs of the Town and Caltrans, as demonstrated by the supporting hydrologic and hydraulic (H&H) and water quality assessments. Quantification of the water quality and flood control benefits associated with the capture project at the Park are consistent with ongoing studies (e.g., the Reasonable Assurance Analysis (RAA) and Bayfront Canal and Atherton Channel Flood Management and Restoration Project) and past efforts (e.g., the 2015 Townwide Drainage Study Update and the SRP) to the maximum extent feasible.

2.0 EXISTING CONDITIONS

Holbrook-Palmer Park is located within the Town (**Figure 2-1**) adjacent to the Atherton Channel, which discharges into the Bayfront Canal and ultimately the San Francisco Bay. The diversion point for the storm water capture facility will divert runoff from the Town and upstream municipalities, addressing water quality and flood conditions caused by the contributing drainage area. This section will (1) characterize the contributing drainage area to the Park and (2) present the existing Park site conditions.

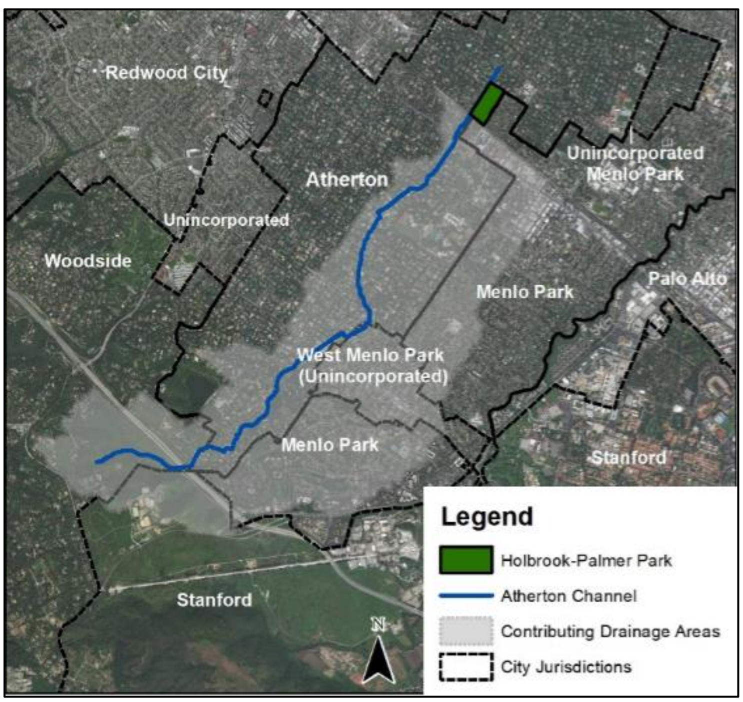

Figure 2-1. Project Vicinity

2.1 WATERSHED CONDITIONS

Upstream municipalities draining to the point of diversion for the Park include the Towns of Atherton and Woodside, the City of Menlo Park, Stanford Lands and Unincorporated San Mateo County. The total drainage area is 2,843 acres, of which there are 632 acres of impervious area based on the National Land Cover Database 2006 (NLDC 2006). Jurisdictional areas and impervious areas are presented in Table 2-1 and Figure 2-2.

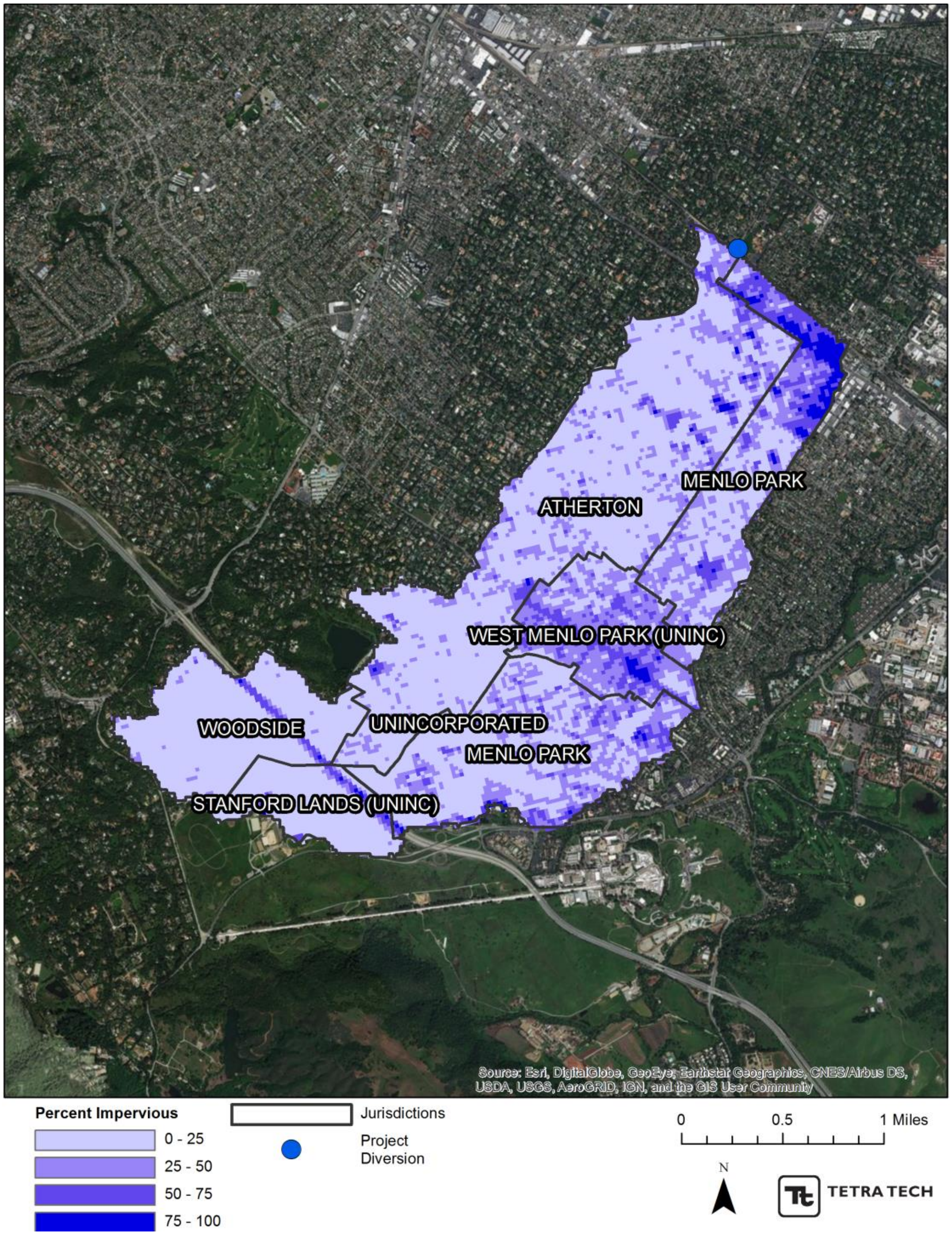

Figure 2-2. Imperviousness in the Park Drainage Area by Jurisdiction

Table 2-1. Tributary jurisdictions and impervious area

Jurisdiction	Area (ac)	Tributary Area Contribution	Impervious Area (ac)
Atherton	1,118	39%	218
Menlo Park	928	33%	297
Unincorporated	441	15%	95
Woodside	356	13%	22
Total	**2,843**	**100%**	**632**

The pollutants of concern (POCs) identified in the local TMDLs are PCBs and Mercury, which are strongly linked to historical land uses due to industrial activities that utilized these constituents before associated aquatic life and human health impacts were known. An assessment by BASMAA characterized six historical land use types in the Bay Area: (1) *Source Properties* represent known sites of use, release, and disposal of PCBs, (2) *Old Industrial* represents industrial areas present in 1968, (3) *Old Urban* represents urbanized areas developed by 1974, (4) *New Urban* represents areas urbanized after 1974*, (5) Open Space* represents undeveloped land, and (6) *Other* includes airport and military areas (BASMAA, 2017). Historical land use GIS layers for the project drainage area were provided by San Mateo County as part of their ongoing RAA. The project drainage area consists of primarily *Old Urban* areas, with a small portion *Open Space*, and an even smaller portion of *Old Industrial*. Current land uses were also classified into five categories, *Forest, Grass, Shrub, Urban,* and *Impervious* to develop hydrologic inputs (NLDC 2006). Drainage from the Town is predominantly from historical *Old Urban* (97.8%) areas and current *Urban* (78.3%) areas. Land uses are summarized in **Table 2-2.**

Table 2-2. Historical and current land uses in the Park drainage area by jurisdiction.

Jurisdiction	Historical Land Use Area (ac)			Current Land Use Area (ac)				
	Old Industrial	Old Urban	Open Space	Forest	Grass	Shrub	Urban	Impervious
Atherton	0	1,094	25	17	6	1	876	218
Menlo Park	8	802	118	7	28	2	595	296
Unincorporated (including Stanford Lands and West Menlo Park)	0	312	129	51	70	2	222	95
Woodside	0	54	302	114	112	9	99	22
Total	**9**	**2,261**	**573**	**189**	**216**	**15**	**1,792**	**632**

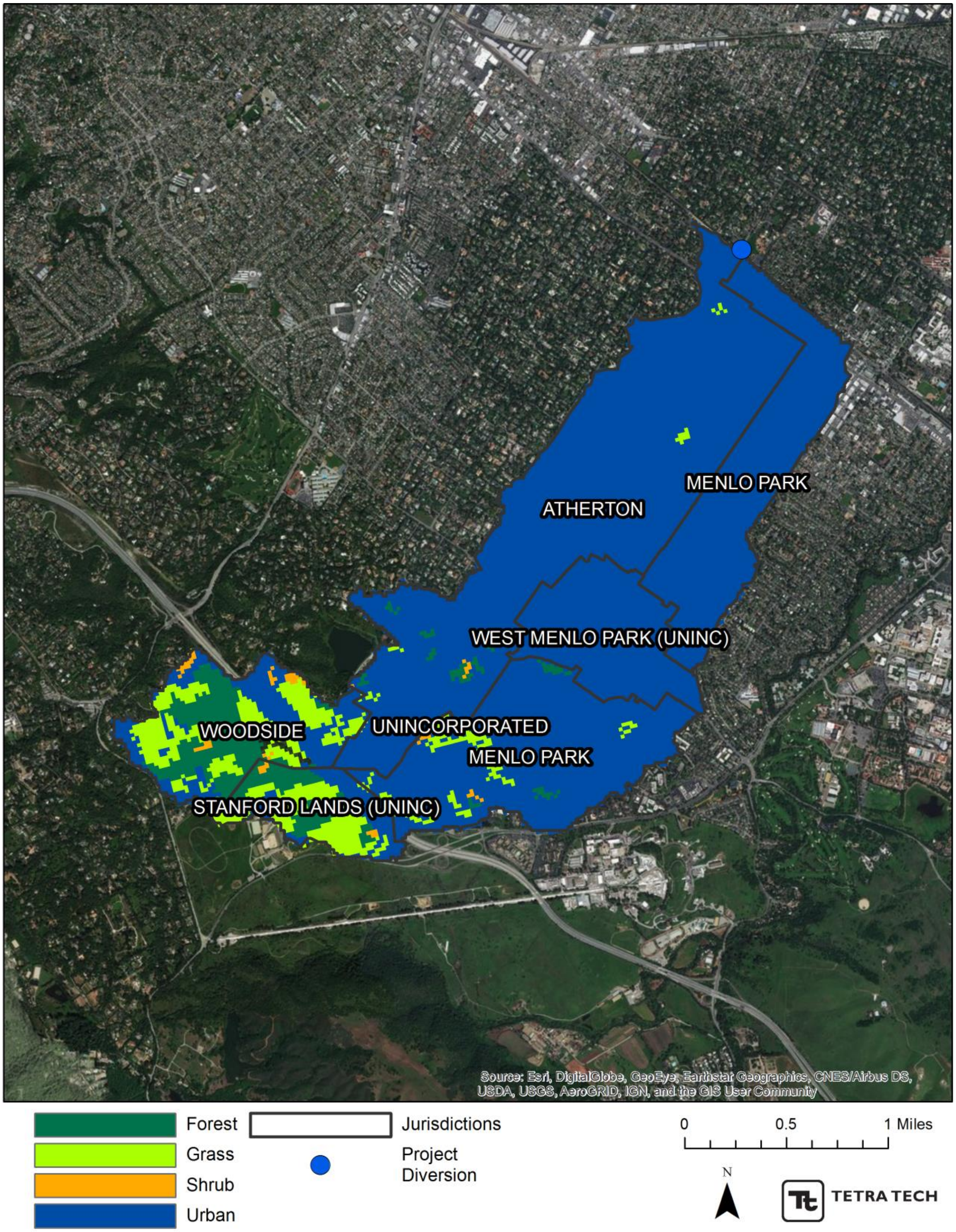

Figure 2-3. Current land use in the Park drainage area by jurisdiction

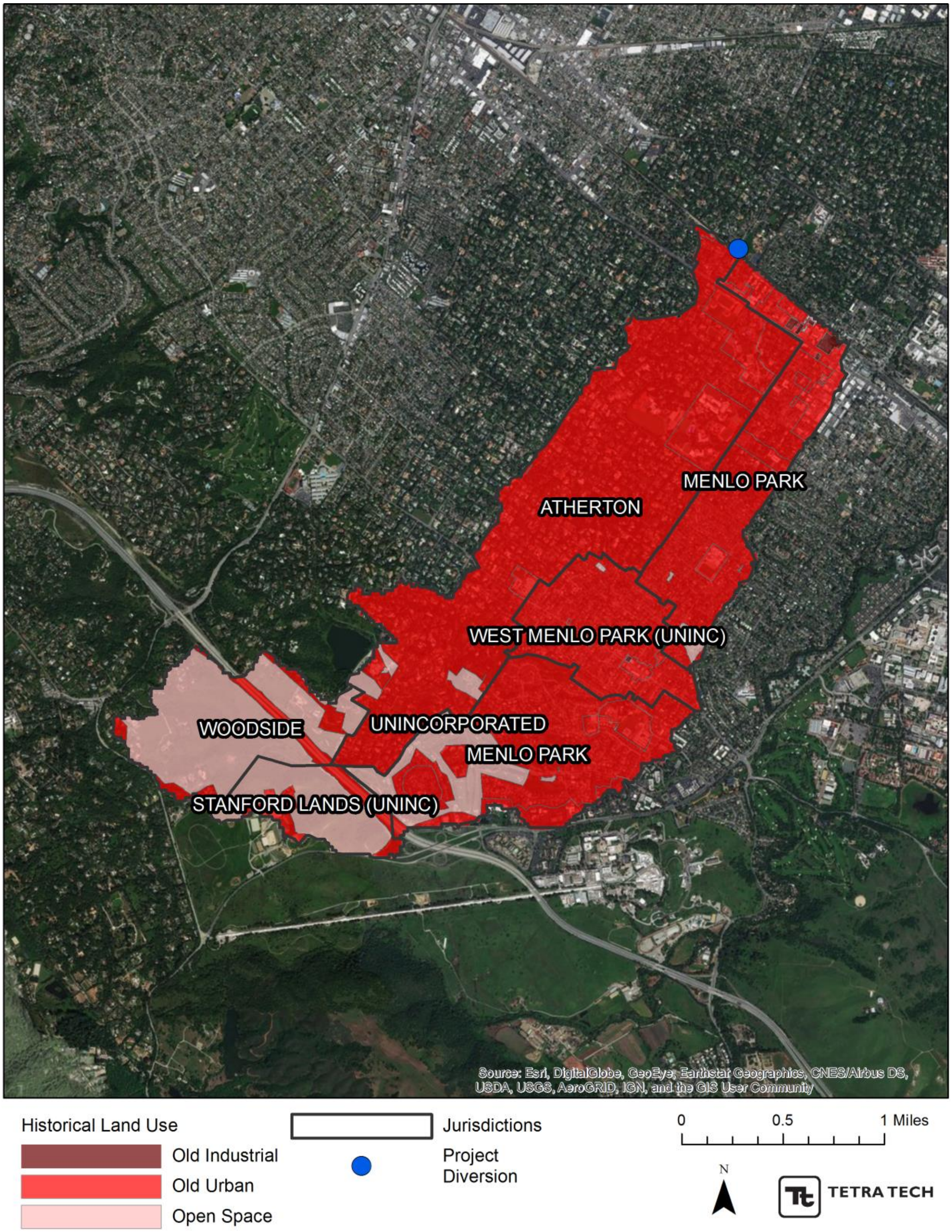

Figure 2-4. Historical land use in the Park drainage area by jurisdiction

2.2 PARK SITE CONDITIONS

The Park is a 22-acre recreational area owned by the Town consisting of mainly open space and other community amenities. Located at 150 Watkins Avenue, the Park is home to a baseball field, a large playing field, tennis courts, a playground, walking paths, restroom facilities, and multiple community gathering spaces. The Park is bounded by Caltrain to the southwest, residential properties to the southeast and northeast, and the Channel along Watkins Avenue to the northwest. There are many redwood trees throughout. There are also two bridges, which are the only vehicle access points to the Park. The Holbrook-Palmer Park Master Plan completed in 2015, outlines the improvements that have been made or are in the process of being made throughout the Park (see Figure 2-5 for the Master Plan layout). Surface improvements due to construction activities of the stormwater capture project will follow the Master Plan to the greatest extent feasible.

Figure 2-5. Holbrook-Palmer Park Master Plan (Source: Holbrook-Palmer Park Master Plan, 2015)

2.2.1 Utility Data Review, Survey, and Utility Mapping

The existing topographical survey information was prepared by Tetra Tech using a combination of sources: (1) an aerial survey that was performed on January 3, 2018 and (2) a conventional field survey that was performed on January 12, 2018 to provide supplemental information previously obscured to the aerial survey. Topographic information was georeferenced horizontally to the North American Datum of 1983 (NAD83) California Coordinate System, Zone 3, and vertically to the North American Vertical Datum of 1988 (NAVD88) using global positioning system (GPS) data collected on site and post-processed against horizontal and vertical values of nearby Continuously Operating Reference Stations (CORS) published by the California Spatial Reference Center (CSRC).

Several sources were utilized to locate the existing utilities in the Park area. The Town provided existing information regarding the wells and irrigation infrastructure at the Park, Channel information, and previous topographic surveys of the Park. Utility information was gathered from utility purveyors in the area to verify the locations of existing sewer, water, cable, telephone, and gas lines. The utility purveyors are tabulated in Table 2-3. The information obtained from aerial surveys, ground surveys, record drawings, and information from utility purveyors was then compiled to create a base map as can be seen in Figure 2-6. A full-size version of the existing utility plans and associated utility purveyor information can be found in Appendix C.

Table 2-3. Utility Purveyors

Utilities	Name of Company	Notified	Responded	Facility Presence
Storm Drain	City of Menlo Park	1/10/2018	1/11/2018	No
Storm Drain	City of Redwood	1/16/2018	1/16/2018	No
Water	California Water Service-Atherton	2/5/2018	2/5/2018	Yes
Communication	Level 3 Communications	1/16/2018	1/19/2018	No
Communication	MCI World Communications (Verizon)	1/10/2018	1/11/2018	Yes
Communication	Pacific Bell	1/10/2018	1/22/2018	Yes
Water	San Francisco Public Utilities Commission	1/11/2018	1/20/2018	Yes
Communication	Sprint	1/10/2018	2/23/2018	No
Sanitary Sewer	West Bay Sanitary District	1/11/2018	1/12/2018	Yes

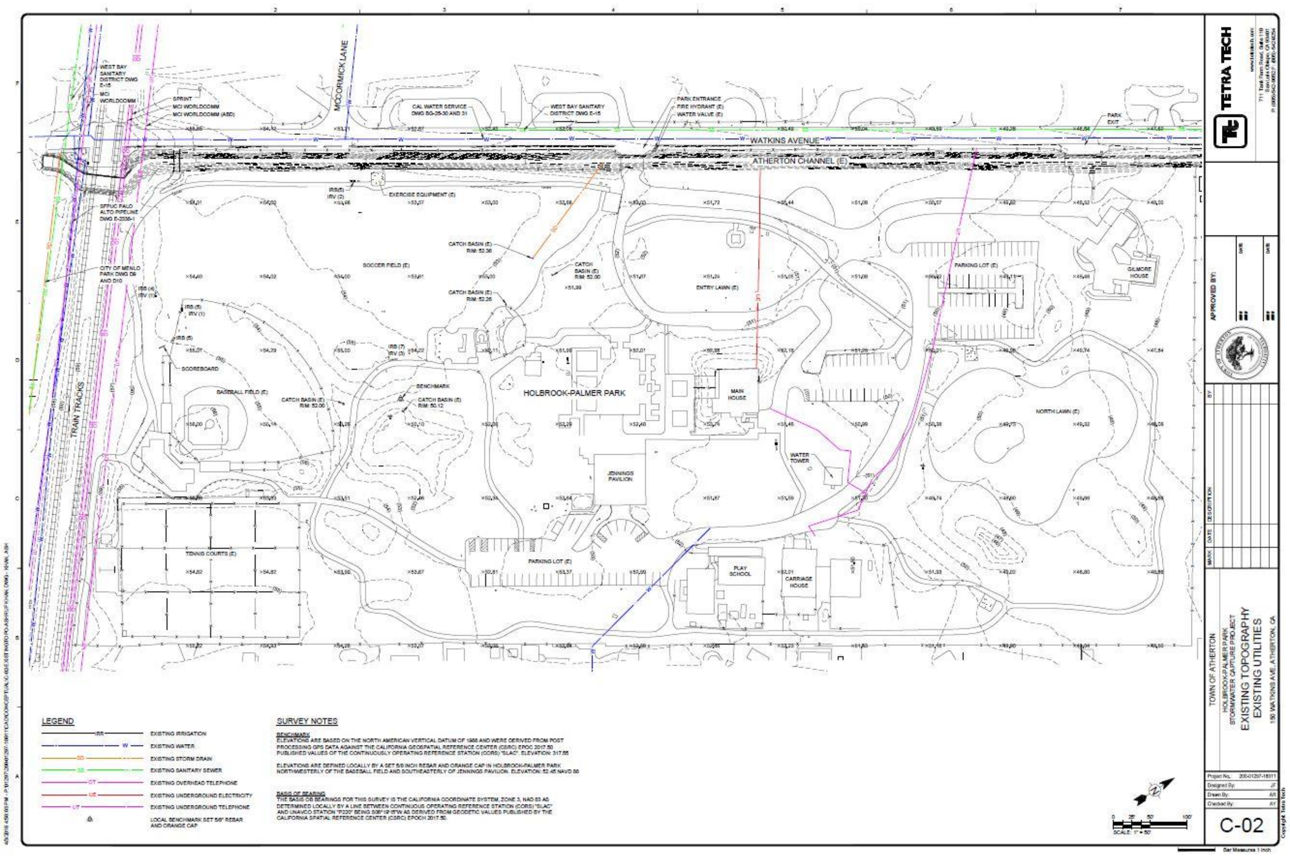

Figure 2-6. Map of existing site topography, including underground utility lines in the vicinity of the Park

2.2.2 Geotechnical Investigation

The following sections present a summary of the Draft Geotechnical Investigation Report conducted by ENGEO Incorporated and delivered on April 9, 2018 (ENGEO, 2018). The complete report is provided as Appendix E to this PEDR.

2.2.2.1 Existing Soil Types

Several soil types were encountered during the field exploration performed in March 2018 (see **Figure** 2**-**7 for testing locations). Stiff to hard clayey material to a depth of 20 to 25 feet below ground surface (bgs) were encountered at locations BH01/1-CPT01, BH02 / 1-CPT02, BH03 / 1-CPT03, and BH04 / 1-CPT4. At these locations, the upper 5 to 10 feet of soil consists of dark brown, high plasticity clay. Approximately 2 feet of the shallow surficial soil may be fill that was re-worked for agricultural and then recreational purposes. Beneath the fat clay, the soil is light yellowish brown with lower plasticity and varying amounts of silt and fine-grained sand. The lean clay extends to approximately 50 feet bgs at the location of BH02 / 1-CPT02.

Alternating layers of medium dense gravels, sands of varying fines content, and clays at a depth of 20 to 25 feet bgs were found at locations BH01 / 1-CPT01, BH03 / 1-CPT03, and BH04 / 1-CPT04.

Pavement section consists of 3 to 4 inches of asphalt underlain by 7 to 9 inches of aggregate base at locations BH05 / 1-CPT05 and BH06 / 1-CPT06. Below the pavement section, up to 1-1/2 feet of fill composed of mottled, very stiff lean clay was discovered.

Below the fill at location BH05 / 1-CPT05, light brown, very stiff lean clay to 16 feet bgs was encountered. Below the clayey material, medium dense sand appears to extend to a depth of approximately 35 feet bgs. The cone penetration test (CPT) logs indicate approximately 10 feet of clayey material below the medium dense sand.

Alternating layers of 2 to 4 feet of hard lean clay and medium dense sand with varying amounts of clay to a depth of approximately 13 feet bgs exist below the fill at location BH06 / 1-CPT06. Below this, the soil is hard/very dense and consists of approximately 5 feet of lean clay underlain by approximately 7 feet of clayey sand. Well-graded, dense sand exists at a depth of approximately 22 feet bgs.

Refer to Appendix E for the boring and CPT logs in the Draft Geotechnical Investigation Report.

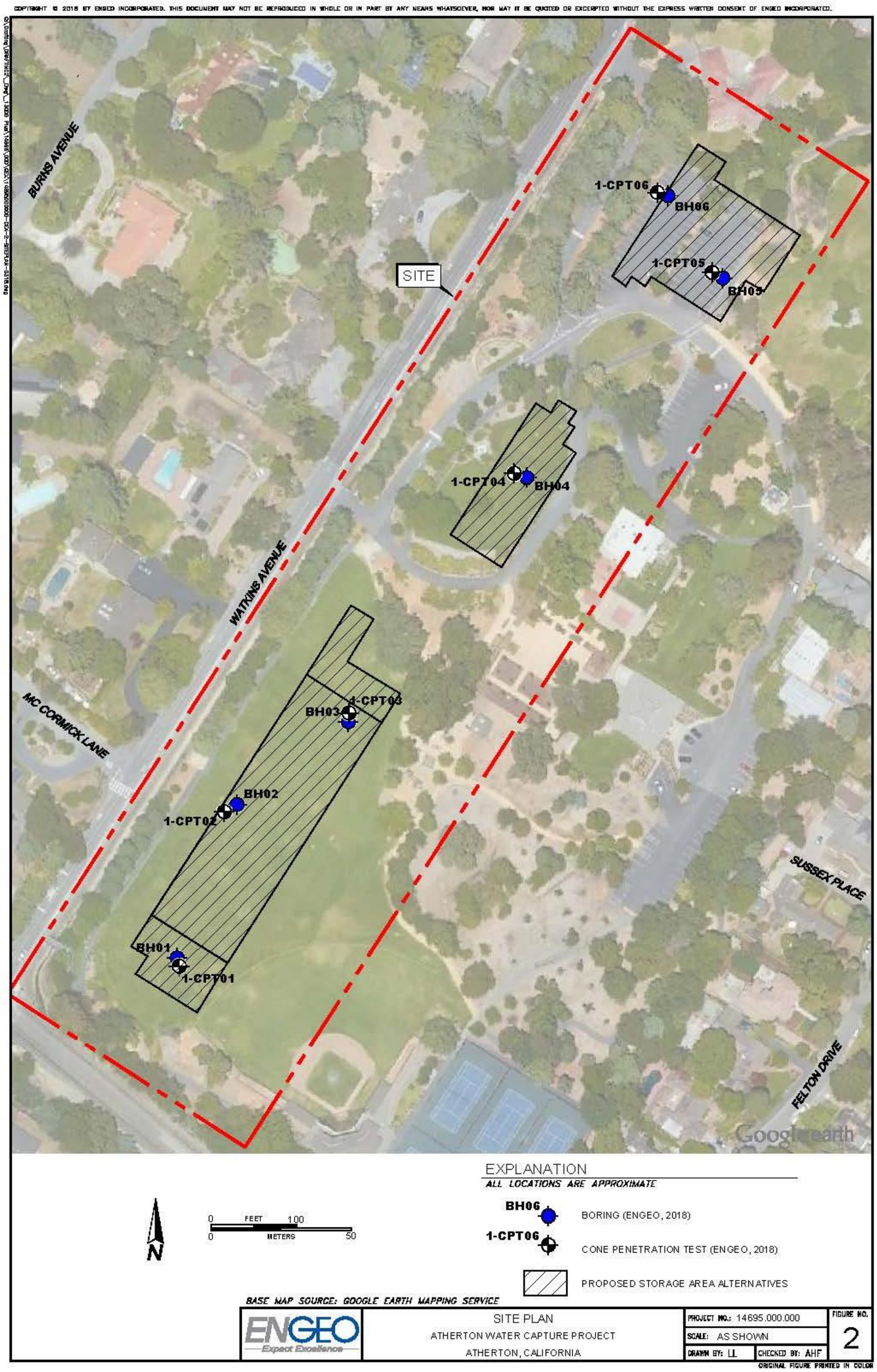

Figure 2-7. Boring location map (Source: ENGEO, 2018)

2.2.2.2 Groundwater

Static groundwater was observed in several of the subsurface explorations. The approximate observed groundwater depths ranged between 29 and 30 feet bgs, resulting in approximate groundwater elevations ranging between 14 and 17 feet. In addition to in-situ groundwater measurements, the historic high groundwater level was found between approximately 23 and 28 bgs from northeast to southwest.

2.2.2.3 Infiltration

The geotechnical explorations in the lawn and the entry lawn areas generally indicate clayey material to a depth of at least 20 to 25 feet bgs. The clayey material extends to approximately 50 feet bgs at the location of BH02 / 1-CPT02. The calculated infiltration rate of the clayey material varies between 0.1 in/hr and 0.4 in/hr. Based on results of percolation testing, it is not recommended to rely on infiltration in design at these locations.

At locations BH05 / 1-CPT05 and BH06 / 1-CPT06, sandy material below a depth of approximately 16 to 17 feet were encountered. At BH05, the sandy material encountered between 16 and 22 feet bgs was medium dense to dense, had low fines content, and demonstrated a calculated infiltration rate of approximately 18 in/hr during percolation testing. The clayey sand material at BH06 encountered from 17 to 22 feet bgs had higher fines content than at BH05 and was very dense. At these depths, the material demonstrated a calculated infiltration rate of approximately 0.2 in/hr. Below a depth of approximately 22 feet bgs are medium dense to dense sand with low fines content, like the material encountered at BH05 between 16 and 22 feet. If the parking lot option is considered for the water quality treatment facility and infiltration is incorporated into design, it is recommended to target the sandy material encountered at 17 feet bgs in BH05 and 22 feet bgs in BH06. This may require some over excavation depending on final design.

2.2.2.4 Liquefaction Susceptibility

Soil liquefaction results from loss of strength during cyclic loading, such as imposed by earthquakes. Soil most susceptible to liquefaction is clean, loose, saturated, uniformly graded, fine-grained sand. Empirical evidence indicates that loose to medium dense gravel, silty sand, low-plasticity silt, and some low-plasticity clay are also potentially liquefiable. For a soil to be potentially liquefiable, it must be saturated. For this site, the design groundwater depth was assumed to be at 23 feet, which corresponds with the historic high groundwater level.

The results of the liquefaction analysis indicate that the medium dense layers of clayey gravel and sand layers 25 to 30 feet bgs are potentially liquefiable. The liquefiable layers have a maximum thickness of approximately 8 feet. Based on the analysis, the site may experience up to 2 inches of total liquefaction-induced settlement. The site improvement should be designed to withstand a differential settlement of 1 inch over a 50-foot distance and perform as intended. Refer to the Draft Geotechnical Investigation Report for the detailed results of the liquefaction analysis.

In addition to the liquefaction analysis, the capping effect of any overlying non-liquefiable soils was evaluated. For liquefaction-induced ground failure to occur, the pore water pressure generated within the liquefied strata must exert a force sufficient to break through the overlying soil and vent to the surface resulting in sand boils or fissures. Proposed improvements founded at grade will have a thick enough layer of non-liquefiable soil above the liquefiable soil to provide a capping effect to prevent manifestation of liquefaction. The layers of non-liquefiable soil above the liquefiable soil are also thick enough to provide a capping effect to prevent manifestation of liquefaction at the base of the underground storage structure and wet well.

2.2.2.5 General Foundation Recommendations

It is recommended that the precast concrete structures be supported on continuous strip footings. The footings are recommended to be designed for an allowable bearing value of 4,000 pounds per square foot (psf) and be supported on competent non-yielding native material as determined by the Geotechnical Engineer or his/her representative in the field at the time of construction. This bearing capacity should be increased by one-third for the short-term effects of seismic loading. The footings are recommended to have a minimum width of 18 inches. The maximum allowable bearing pressure is a net value; the weight of the footing may be neglected for design purposes. All footings located adjacent to utility trenches should have their bearing surfaces below an imaginary 1:1 (horizontal: vertical) plane projected upward from the bottom edge of the trench to the footing. The bottom of the subsurface water quality structure between footing foundations should be underlain by a 12- to 18-inch-thick layer of 3/4- to 1-1/2-inch minus clean crushed rock underlain by an approved geotextile stabilization fabric (Mirafi 600X or approved equivalent).

A conventionally reinforced structural mat is recommended for the below-grade lift station. The mat foundation should be reinforced with top and bottom steel as determined by the structural engineer to provide structural continuity. The mat foundation should be designed for an allowable uniform soil pressure of 4,000 psf. Increase this bearing capacity by one-third for the short-term effects of seismic loading. The bottom of the mat should be underlain by a 12-inch-thick layer of 3/4-inch clean crushed rock underlain by an approved geotextile stabilization fabric (Mirafi 600X or approved equivalent). The stabilization fabric should be wrapped and placed above the bedding section. A layer of 6-ounce filter fabric may alternatively be used on top of the bedding section.

It is recommended that proposed at-grade structures be founded continuous footings with an interior slab-on-grade underlain by a minimum of 24 inches of non-expansive engineered fill. Continuous footings should have a minimum depth of 24 inches and a minimum width of 12 inches. Isolated footings should have a minimum depth of 24 inches and a minimum width of 18 inches. Continuous and isolated footings should be designed for a maximum allowable bearing pressure of 2,500 psf for dead-plus-live loads. Increase this bearing capacity by one-third for the short-term effects of wind or seismic loading. The maximum allowable bearing pressure is a net value; the weight of the footing may be neglected for design purposes. All footings located adjacent to utility trenches should have their bearing surfaces below an imaginary 1:1 (horizontal: vertical) plane projected upward from the bottom edge of the trench to the footing.

3.0 REGULATORY CONTEXT & PREVIOUS STUDIES

This section presents the regulatory requirements and previous studies that are driving the need for a regional stormwater facility at the Park, including the MRP, TMDLs, and flood control considerations.

3.1 MRP TMDLS

The Town is required to regulate pollutants in stormwater runoff from its municipal storm drain system in accordance with the MRP. Provisions of the MRP require each jurisdiction, including Atherton, to develop a Green Infrastructure (GI) Plan for stormwater that demonstrates a shift from traditional "gray" storm drain infrastructure – which channels polluted runoff directly into receiving waters without treatment – to a more resilient and sustainable storm drain system comprised of "green" infrastructure, which captures, stores, and treats stormwater.

The MRP requires compliance with TMDL requirements, specifically the San Francisco Bay Mercury TMDL (SFRWQCB, 2004, amended by SFRWQCB, 2006) and the San Francisco Bay PCB TMDL (SFRWQCB, 2008). Water quality objectives were established in each TMDL to protect beneficial uses of the impaired receiving water, mainly San Francisco Bay. The TMDLs apply to all Bay segments which includes Sacramento and San Joaquin Delta, Suisun Bay, Carquinez Strait, San Pablo Bay, Richardson Bay, Central Bay, Lower Bay, and South Bay. The primary beneficial use impaired by PCBs and Mercury in the Bay is COMM, which represents the commercial or recreational collection of fish, shellfish, or other organisms. Additional beneficial uses impaired include EST (Estuarine Habitat), RARE (Preservation of Rare and Endangered Species), and WILD (Wildlife Habitat), with significant bioaccumulation issues. PCB concentrations tend to be highest in sediments, thus typical bioaccumulation starts with bottom-feeding species and transfers along the food chain, with the additional issue of biomagnification (increased concentrations in organisms higher up on the food chain) known to occur with PCBs. Mercury is also strongly associated with sediments and builds up through bioaccumulation and biomagnification. Methylmercury, the organic form of Mercury, is of particularly concern due to the toxicity and increased bioavailability to aquatic organisms. Overall, the consumption of fish in San Francisco Bay is a threat to human health given the elevated levels of PCBs and Mercury in fish tissue. The legacy nature of the pollutants of concern and lack of knowledge on dispersion and degradation processes contribute to the uncertainty in future conditions.

The PCB TMDL established two water quality objectives; a fish tissue target of 10 µg/kg and a sediment target of 1 µg/kg based on a food web model developed by the San Francisco Estuary Institute (SFEI). To achieve these objectives, the total mass of PCBs in the active layer of the Bay must be reduced to 160 kilograms. Based on a mass budget model in the TMDL, external loads to the Bay must be reduced to 10 kilograms per year to achieve the required reduction within 30 years. Significant sources identified for PCBs loads to the Bay include direct atmospheric deposition (net removal through atmosphere), Central Valley Watershed (nonpoint source discharges entering the Bay via Sacramento and San Joaquin Rivers, expect to attain WLA through natural attenuation), Municipal Water Dischargers, Industrial Wastewater Dischargers, and stormwater runoff. Twenty percent (20%) of the allowable external load was allocated to urban stormwater runoff; therefore, the waste load allocation (WLA) for urban stormwater is 2 kilograms per year, which must be achieved by 2030. Allocations were further broken down by County based on respective Bay-side populations in year 2000, resulting in a WLA for San Mateo County of 0.2 kilograms per year. The baseline PCB load from stormwater runoff was estimated at 20 kilograms per year, based on grab samples from Water Year (WY) 2005, resulting in a required load reduction of 18 kilograms per year (90% reduction). MRP Permittees across the entire Bay Area are responsible for a load reduction of 14.4 kilograms per year, which must be achieved by 2030. The MRP specifies GI is expected to achieve a portion of the overall required load reduction. The current MRP term ends on June 30, 2020, which is prior to the load reduction compliance deadline, resulting in an interim load reduction goal of 3 kilograms per year by 2020. For all San Mateo County Permittees combined, the load reduction requirement by the end of the permit term for PCBs is 370 grams per year, of which 15 grams per year must be reduced through GI. The Town's contribution was identified using a population ratio (1.2%) in year 2000 (est. 7,196 people), when compared to the entire County (est. 600,000 people). The resulting total load reduction required for Atherton's contribution is 4.4 grams per year. If the same ratio is applied for GI load reduction, the Town should achieve a reduction of 0.2 grams per year by the end of the Permit term. Similarly, if the population based allocations are applied to Town, the WLA for Atherton is 2.5 grams per year. See Table 3-1 for a summary of compliance targets.

The Mercury TMDL established two water quality objectives; a fish tissue target applying to 60-centimeter-long striped bass of 0.2 mg/kg (to protect consumption of fish) and a fish target applying to 3 – 5-centimeter-long fish of 0.03 mg/kg (to protect aquatic organisms and wildlife). To achieve these objectives, the suspended sediment mercury concentration must be reduced to 0.2 mg/kg dry sediment. Sources of mercury loads to the Bay that were identified include bed erosion, Central Valley Watershed (nonpoint source discharges entering the Bay via Sacramento and San Joaquin Rivers, Central Valley Regional Water Quality Control Board developed mercury TMDLs for impaired water bodies draining to the Bay (CVRWQCB, 2010)), Guadalupe River Watershed (mining legacy, San Francisco Regional Water Quality Control Board developed the Guadalupe River Watershed Mercury TMDL (SFRWQCB, 2008)), atmospheric deposition (remote and local sources), non-urban stormwater runoff, wastewater (municipal and industrial), sediment dredging and disposal, and urban stormwater runoff. Accounting for all expected sources, the WLA for mercury in stormwater is 82 kilograms per year, of which San Mateo County has a WLA of 8.4 kilograms per year. The baseline/existing load from urban stormwater was estimated at 160 kilograms per year, based on box models for sediment and mercury corresponding to WY 2003. The required load reduction from stormwater is therefore 78 kilograms per year. MRP Permittees are responsible for a load reduction of 62 kilograms per year, to be achieved by 2028, of which San Mateo County is responsible for reducing 8.0 kilograms per year. The MRP further specifies that the load reduction achieved through GI is expected to be 10 kilograms per year. The MRP requires a total load reduction of 48 grams per year to be achieved by June 30, 2020 (the end of the permit term). The required load reductions through GI distributed by County were determined from the load reduction for a County required by the TMDL divided by the total load reduction required for the MRP Permittee area, multiplied by the total required 48 grams per year. San Mateo County Permittees must achieve a load reduction of 6 grams per year through GI by the end of the permit term. The required mercury load reduction achieved through GI by the end of the permit term for Atherton's contribution is 0.056 grams per year. This reduction value is based on the requirements of San Mateo County, based on the TMDL baseline model loads corresponding to WY 2003, and the relative area of Atherton (3224 acres) within San Mateo County (355,675 acres) (San Mateo County GIS Enterprise Data, 2016). Similarly, if area based allocations are applied, the WLA for Atherton is 78.5 grams per year. The required load reductions and WLA for the Town, based on area-weighting, are tentative at this time and a more precise calculation of the Town's requirements can be established after completion of the RAA watershed model. See Table 3-1 for a summary of compliance targets.

The MRP also requires an 80% reduction in trash load by 2019 and a 100% reduction, or no adverse trash impact, by 2022. Reducing trash generation rates to less than 5 gallons per acre per year satisfies attainment of the 80% reduction requirement. Non-compliance with the requirements outlined in the MRP permit will result in enforcement actions pursued against the Permittee(s) in violation.

Table 3-1. Summary of compliance targets from the MRP and TMDLs

RAA Recommendations to Achieve Final Compliance with BMPs	San Mateo County		Town of Atherton	
	Mercury (kg/yr)	PCBs (kg/yr)	Mercury (g/yr)	PCBs (g/yr)
Required load reduction by June 30, 2020 (Total)	N/A	0.37	N/A	4.4
Required load reductions by June 30, 2020 (GI)	0.006	0.015	0.056	0.2
TMDL WLA in urban stormwater discharges	8.4	0.2	78.5	2.5

3.2 SAN MATEO COUNTY SRP AND RAA

The City/County Association of Governments of San Mateo County (C/CAG) developed the Stormwater Resource Plan for San Mateo County (SRP) to provide a watershed-scale analysis of stormwater and dry weather capture projects throughout the county with the goals of reducing flooding and pollution associated with stormwater runoff, improve biological functioning of plants, soils, and other natural infrastructure, and provide community benefits through stakeholder engagement and education. The SRP identified the Park as a high priority regional stormwater capture project, and developed a very preliminary project concept and associated benefit estimates.

Additionally, the C/CAG is in the process of developing a RAA that will demonstrate that local agency GI Plans will achieve mandated PCB and mercury load reductions in accordance with MRP and TMDL provisions. The model inputs and timeseries generated as part of the draft RAA have been used for the quantification of water quality benefits in this preliminary engineering design report to ensure consistency with County-wide efforts going forwards.

3.3 FLOOD CONTROL DRIVERS

The Town completed the 2015 Townwide Drainage Study Update to develop a better understanding of the flooding concerns in the Town and the capacity of the existing infrastructure system. The study sorted projects into different tiers based on the potential flooding impact and time sensitivity. The top priority projects are called Tier- 1 and have the potential to cause safety issues and/or economic loss for the city and its residents if they are not addressed. The Park was identified as a Tier-1 priority in the study because the *"improvements mitigate flooding problems that can create significant life and safety issues"* (Town of Atherton 2015a). The flooding problems are due to the Atherton Channel being undersized. The Channel was determined to only have the capacity to safely convey the 7-year storm without causing distributed flooding and backups further in the Channel. The 2015 Townwide Drainage Study Update also determined that detaining approximately 10 acre-feet of stormwater runoff from the Channel would reduce the risk of flooding. Three locations were identified within the Town's jurisdiction as potential locations to implement a storage facility: (1) Las Lomitas School, (2) JL Dixon Stables, and (3) Holbrook-Palmer Park. Las Lomitas School was eliminated from consideration due to failed negotiations with the school board during the short time frame allowed to retain Caltrans funding. JL Dixon Stables was determined to be too far from the Channel and is on private property, which would increase the cost of the project, effectively reducing the project size, and thereby reducing the flood control benefit. Holbrook-Palmer Park was determined to be the best location in terms of feasibility and proximity to the Channel.

Additionally, the Town is also a party to the Bayfront Canal and Atherton Channel Flood Control Management and Restoration Project prepared by the County. The Town makes up 44% of the contributing watershed area and generates approximately 38% of the flows to the Bayfront Canal watershed, which is inclusive of the entire Park drainage area. A reduction in flow contributions to the Atherton Channel/Bayfront Canal per jurisdiction will be evaluated by San Mateo County if upstream detention and stormwater capture projects are implemented within the watershed and reduce flows tributary to the Restoration Project.

4.0 DECISION SUPPORT MODELING

The models developed to support the water capture project at the Park were configured to optimize the capture of stormwater to meet both water quality and flood risk mitigation goals. Several system configurations were modeled to represent the dynamic between diversion rate, storage capacity, and limited infiltration rates (meaning that practical hydraulic considerations like diversion structure design and outflow rates may substantially impact performance). Model results were analyzed to determine the most cost-effective, multi-benefit configuration for the Town. The following sections summarize the strategy used to simulate these real-world engineering constraints, while optimizing the performance of the system in terms of both water quality and flood control.

4.1 BASELINE CONDITIONS AND CONSTRAINTS

The following subsections summarize the compliance metrics, baseline runoff and pollutant loading, and groundwater considerations used to inform modeling.

4.1.1 Stormwater Compliance Metrics

Baseline stormwater pollutant loadings were determined in accordance with the Interim Accounting Methodology for TMDL loads reduced as discussed in the Bay Area RAA Guidance Document. The accounting system used for this methodology is based on assigned mercury and PCB loadings for different land use categories. The land use based yield is an accepted value for the estimate of the mass of contaminant contributed to an area of a particular land use per unit of time. These values are assigned to each land use type based on historical pollutant load contributions from the products, processes and legacy contaminants associated with each land type. Pollutant yields were developed based on studies which incorporated monitoring data of PCBs in San Francisco Bay and watersheds discharging to the Bay prior to 2014 and monitoring data of PCBs and Mercury from WY 2011 (Davis et al., 2014 and McKee et al., 2012). A regression analysis was conducted and coefficients associated with each historic land use were iteratively determined. Eighty-seven percent (87%) of the variability in the PCB yields was explained by land use and 76% of the variability in mercury yields was explained by land use, further justifying the methodology. Using correction factors, the total pollutant loads from the regression equations for each land use were normalized to the TMDL baseline loads. Pollutant yields by land use are summarized in **Table** 4-1 (BASMAA, 2017). As defined, *Source Properties* generate the largest PCB yield, with the next highest generated from *Old Industrial*. *Source Properties* and *Old Industrial* generate equivalent mercury yields.

Table 4-1. Estimated Land Use Based yields for PCBs and Mercury (BASMAA, 2017)

Land Use Category	Assumed Average PCBs Yield (mg/ac/yr)	Assumed Average Mercury Yield (mg/ac/yr)
Source Property	4,065	1,300
Old Industrial	86.5	1,300
Old Urban	30.3	215
New Urban	3.5	33
Other	3.5	26
Open Space	4.3	33

In lieu of using the baseline loading estimates established in the TMDLs, the baseline load may be recalculated using accepted methods outlined in the Bay Area RAA Guidance Document, one of which is using calibrated watershed models. The baseline load, based on a calibrated model, may be calculated using a long-term condition; the RAA guidance document recommends using water years 2000 – 2009. The baseline load may also be calculated using the representative water year as the critical condition, which is 2002 based on rainfall data within the MRP region. The baseline loads are driven by the land use based yields presented in Table 4-1 and determine required load reductions. As established in Section 2.1, most of the project drainage area is *Old Urban*, which generates a moderate load of both constituents. The Town has *Old Urban* as the primary historical land use, which directly impacts the baseline load generated from the Town and subsequently determines the Town's required load reduction (to achieve the WLAs established by the TMDLs). The preliminary RAA watershed model provided by the County was used for baseline loading estimates and is discussed in detail in the next section.

4.1.2 Watershed Characterization

Baseline watershed loading was estimated using the preliminary RAA watershed model developed from the Load Simulation Program in C++ (LSPC), provided by the San Mateo C/CAG. The timeseries provided flow and water quality (suspended sediment and PCB) estimates from the project drainage area, but is subject to change once the RAA is approved and finalized. Over the long-term period available (WY 2000 – WY 2009), no baseflow was simulated and hourly flows to the point of diversion maxed out at 320 cubic feet per second. During WY 2002 (critical year), the maximum hourly flow was 202 cubic feet per second. See Figure 4-1. for the full timeseries.

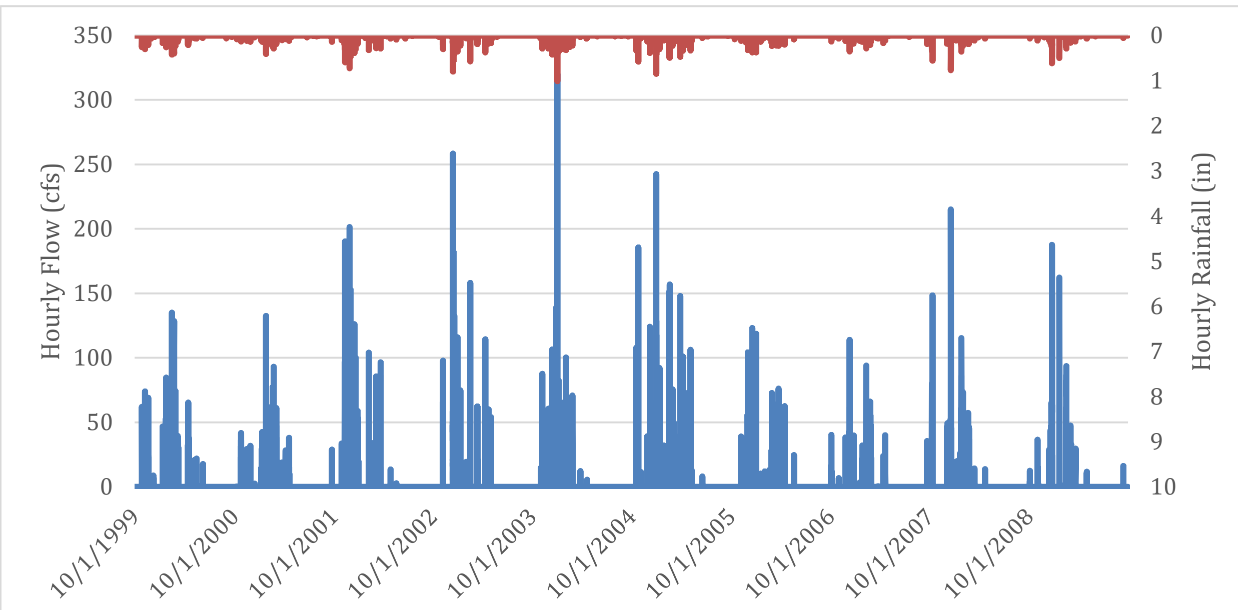

Figure 4-1. Hourly flows and rainfall at the point of diversion

Average annual suspended sediment and PCB loads are presented in Table 4-2, for both the long-term and critical WY (2002). Over the long-term simulation, annual watershed pollutant loadings ranged 73,592 – 154,168 kilograms suspended sediment per year and 6.7– 13.9 grams PCBs per year.

Table 4-2. Average Annual Runoff/Pollutant Loading

	Long-term (WY2000 - WY2009)	Critical WY (WY2002)
Runoff (ac-ft)	348	333
Suspended Sediment (kg/yr)	101,675	111,462
PCBs (g/yr)	9.2	10.1
Mercury (g/yr)	19.5	21.4

4.1.3 Groundwater Constraints

Results of the geotechnical investigation, including groundwater constraints and infiltration assumed below the facility, can be found in Section 2.2.2 and in Appendix E.

4.2 STORMWATER CAPTURE FACILITY MODELING

The primary design goals of this Project are to address local flooding concerns in the Town, as well as reduce long-term annual loading of PCBs and Mercury to the San Francisco Bay. The purpose of optimization modeling is to balance design components (storage volume, inflow diversion rates, treatment discharge rates) such that no one component limits the performance of the system. Optimization supports decision making throughout the design process by guiding selection of the most cost-effective system design through numerous model iterations. A summary of the general model assumptions and design parameters are included in Table 4-3 and generally shown in the Water Process Flow Schematic (**Figure** 4-2).

Table 4-3. Summary of key modeling parameters and assumptions

Site Parameter	Value(s) Used in Analysis	Information Source / Rationale
Diversion rate (cfs)	Variable; maximum 100 cfs	See Section 4.2.1 for water quality; See Section 4.2.2 for flood control
Storage footprint (ac)	0.894 (maximum footprint)	Proposed Site Plan (see Figure 5-3)
Storage depth (gravity inflow) (ft)	10	Groundwater constraint; geotechnical investigation
Outflow/treatment mechanism	Filtration; incidental infiltration	Infiltration not recommended as primary mechanism per geotechnical investigation (Appendix E)
Outflow rate (cfs)	1.82 – 5.76 modeled for water quality 5.00 modeled for flood control	See Section 4.2.1 for water quality; See Section 4.2.2 for flood control
Surface Depth to storage top elevation	4 feet	Proposed Water Process Flow Schematic (**Figure** 4-2)

Due to the different drivers, and associated performance metrics, for water quality and flood control, two optimization strategies were developed to inform the ultimate storm water capture project configuration. A summary of the water quality optimization methodology is presented in Section 4.2.1, the flood control optimization methodology is presented in Section 4.2.2, and the results are presented in Section 4.3.

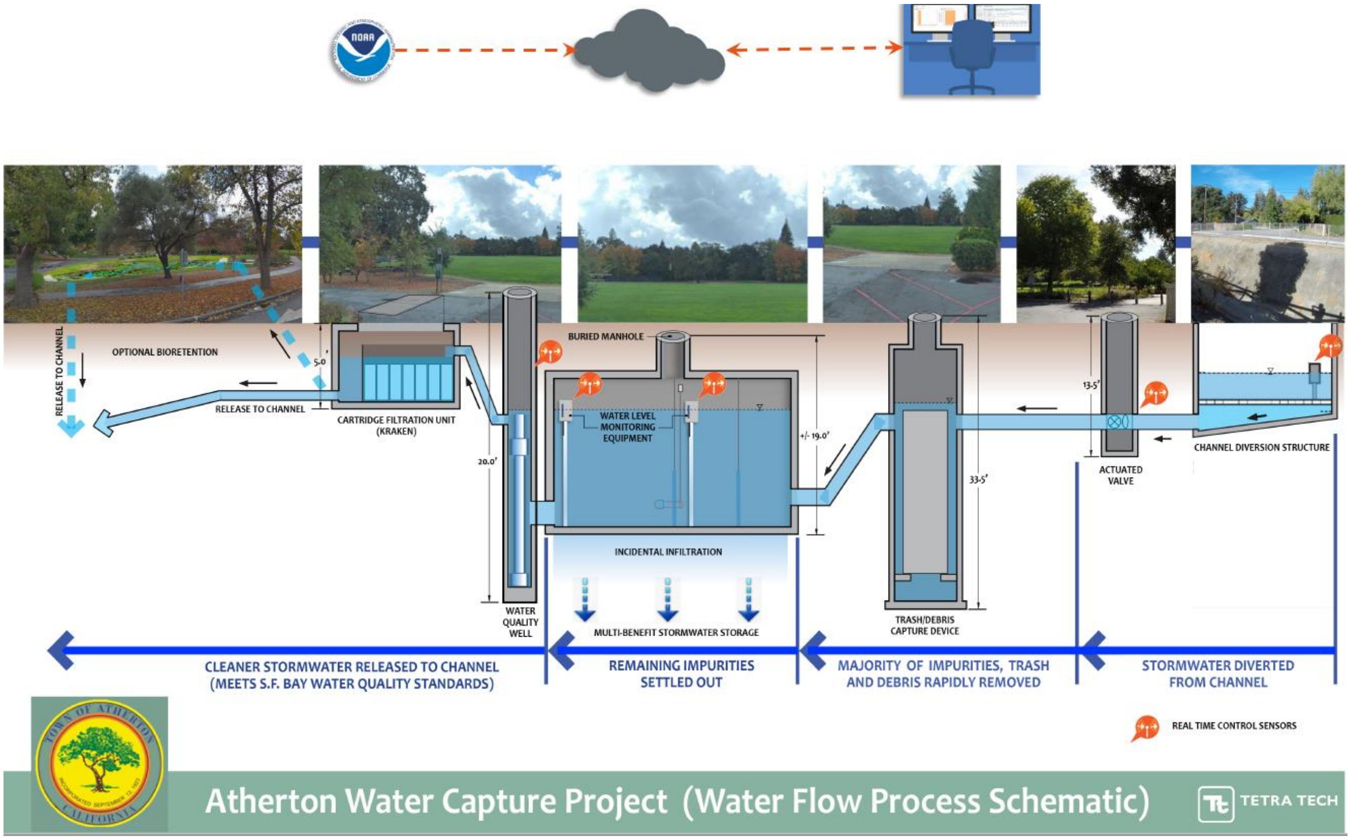

Figure 4-2. Water Process Flow Schematic

4.2.1 Water Quality Optimization Strategy

Several capture facility design parameters were varied as part of the optimization, including diversion inflow rates, storage volume, water quality filtration rates, and flood control discharge rates. The System for Urban Stormwater Treatment and Analysis Integration (SUSTAIN) model was used for this analysis due to the built-in optimization algorithms that automate the process of evaluating millions of different configurations to select a cost-effective solution. The model was run using continuous (10 years of hourly data coinciding with the long-term time period for the RAA) runoff and pollutant loading time-series data developed from LSPC, which were provided by San Mateo County as part of their RAA modeling efforts. These RAA timeseries are preliminary and are subject to change once the RAA is approved and finalized. The key decision-support modeling variables that were assessed include:

- Diversion rates, which were varied up to a maximum of 100 cfs from Atherton Channel to represent inflows to the capture facility that could be effectively pretreated. Variable diversion rates were used to identify points of declining water quality benefit to assist in final recommendations.
- Discharge, which occurs via filtration through the membrane filtration devices. Rates were varied according to throughput rates for commonly configurations of the Kraken filtration units.

Real-time controls (RTC) were also simulated to evaluate the additional benefit that could be realized by using predictive logic to meet water quality targets. Additional information on RTC is provided in Section 4.2.3.

These preliminary optimization model runs produced cost-effectiveness curves that relate each combination of design components to the respective pollutant or stormwater capture performance. Comparing the relative differences between various capture scenarios allowed for the determination of which recommended configuration to pursue.

4.2.2 Flood Control Optimization Strategy

The siting of the water capture facility at the Park is adjacent to Atherton Channel, which is undersized for design storms above the 7-year, 24-hour event per the 2015 Townwide Drainage Study Update (Town of Atherton, 2015a). Optimizing the diversion of peak flows from Atherton Channel to mitigate flooding impacts on adjacent and downstream areas is the primary target for the flood control optimization. The storage facility capacity is equivalent to the water quality facility volume (approximately 8.94 acre-feet). With the storage capacity maximized, optimization of flood control benefits is driven by the diversion rate and the strategic capture of peak flows using RTC logic.

Traditional flood control assessments are conducted using the design storm approach, wherein storm events of different return frequencies (e.g., 10-year, 24-hour storm) are modeled to (1) determine the location and magnitude of potential flooding (peak flows) and (2) to quantify the mitigation needs for the most impactful storm events that a given location might experience. In terms of optimization, the benefits of the stormwater capture facility will vary with the diversion rate and storage capacity commensurate to how much peak reduction for the flood event hydrograph can be attained. While design storms help to determine flood control for the largest events, long-term benefits can also be quantified (using the timeseries provided by San Mateo County for the RAA) to determine how various aspects of the optimization will affect the performance of the unit over a historical record of actual storm events. The long-term period from WY 2002 to 2009 was used to assess impacts on flooding, as it is more representative of actual storm conditions that occur more frequently.

To determine flood control targets and potential benefit, the 5-, 7-, 10-, and 100-year, 24-hours design storms were modeled using PCSWMM, a graphical interface for modeling and post-processing of the EPA's Stormwater Management Model (SWMM). A full hydraulic and hydrologic model was built for the Atherton Channel and the contributing drainage area that generates runoff to the point of diversion. This PCSWMM model leveraged the previous hydraulic model developed for the Townwide Drainage Study Update and the hydrology model was developed using municipal and national datasets for land cover, elevation, and drainage network with parameterization from SWMM guidance documentation. Design storm runoff at the point of diversion was determined from the model to assess the potential of the stormwater capture facility to reduce peak flowrates for flood events using both active and passive management for diversion and outflow (additional RTC information is included in Section 4.2.3.). Results were compared to findings from the Townwide Drainage Study Update to assess the efficacy of this project to address overall watershed flood control objectives.

4.2.3 RTC Considerations

A critical component of designing a stormwater capture facility that can meet multiple objectives (flood control and water quality) for marginal additional costs are RTC, which operate by relaying real-time monitoring data, such as water level and flow, from on-site sensors to cloud-based software (**Figure** 4-3). Based on the predicted rainfall forecast downloaded from National Weather Service climate models, the software automatically regulates on-site control hardware (such as valves and diversion structures) based on configurable and adaptable logic. Rapid, predictive, and responsive control provides "hard" infrastructure with flexibility and resiliency that could not otherwise be achieved through traditional static hydraulic structures, such as orifices, weirs, and manually operated gates.

RTC has been demonstrated nationwide to maximize water quality improvement (relative to passive systems) by actively and predictively managing the routing and storage of stormwater. Because RTC relies on real-time monitoring instruments, it provides a mechanism for rapidly analyzing the operation of stormwater capture facilities with respect to performance goals and adaptively modifying the control logic in response to observed functionality. The long-term monitoring datasets also allow maintenance to be proactively prescribed when facility functions underachieve target criteria (e.g. when filtration rates through engineered media drop below required thresholds).

This section discusses the RTC analyses and configurations relative to passively controlled stormwater capture facilities.

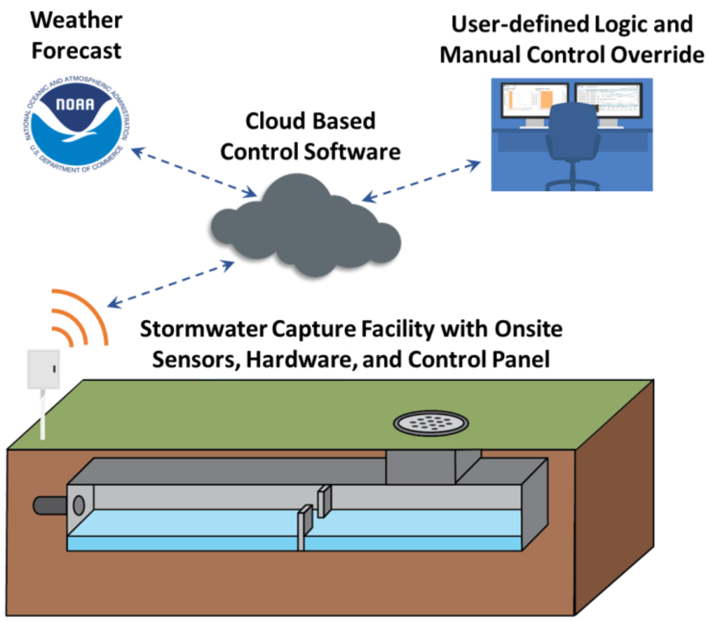

Figure 4-3. Conceptual Schematic of RTC Framework

4.2.4 RTC Configurations

The schematic below (Figure 4-4) displays the opportunities for real-time monitoring and RTC at the Park. The control points include the point of diversion and the outflow pathway, providing the ability to switch between the filtration unit or untreated flood control outflow depending on the appropriate management of runoff for maximum and balanced benefits. RTC at these points can:

- Maximize the diversion of the “first flush” of pollutants during the onset of wet weather flow (based on in-stream turbidity measurements or modeled pollutographs), while maintaining flood control capacity in the Channel,
- Maximize storage capacity by releasing stored water prior to predicted inflow, and
- Increase flood attenuation capacity by reducing flow to downstream areas during wet weather, delaying diversion of flows into the facility to coincide with peak flows.

The RTC software accomplishes these objectives using logic configured for the site that combines forecast information with on-site sensor data to determine the control point state (e.g., open/close valve, on/off pump, etc.). The software algorithms that determine the control point state provide the following basic logic:

- Determine forecasted rainfall from the National Weather Service
- Calculate forecasted runoff volume using site-specific model
- Check site conditions using sensor data (e.g., water level/volume, water quality status)
- Compare available storage with forecasted runoff volume
- Compare water quality status with water quality objectives
- Send signal to control point to manage water flow and level

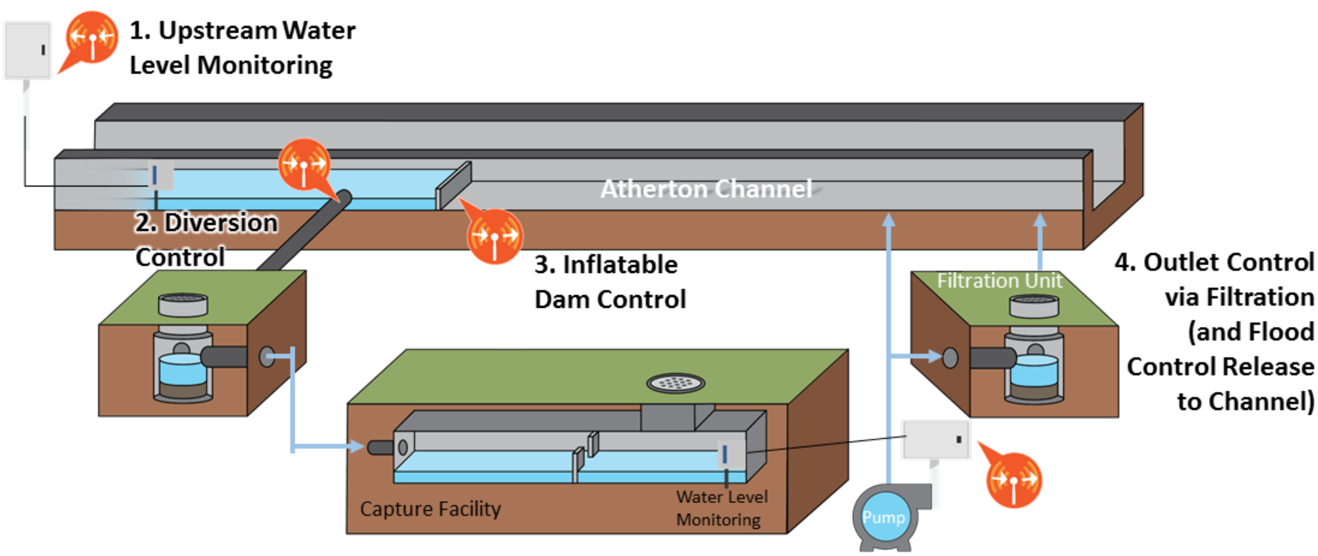

Figure 4-4. Potential RTC monitoring and control points

4.2.5 RTC Control Logic

RTC assessed for this project were focused on optimizing flood control benefits by capturing the peak runoff, as well as ensuring water quality benefits were achieved. The RTC modeling scope was limited to simulating the flow of water into the capture facility via the diversion **(**Figure 4-4, RTC #2) and between the outflow mechanisms for water quality filtration or flood control return in off-peak hours **(**Figure 4-4, RTC #4**).** Control logic in the RTC model was assumed to evaluate a forecast window 6 to 24 hours in the future and react to the predicted flow. Two control logic scenarios were assessed, as described in Table 4-4 below:

Table 4-4. RTC control logic scenarios

Control logic Scenario	Assessment
Shut Diversion When Full:	This scenario shuts off diversion to the stormwater capture facility when it reaches full capacity. Because there is no internal overflow mechanism (the system is completely offline), this feature is necessary to maintain the function of the pretreatment and capture facility and prevent backup of water in the diversion conduit. To account for flow time in the diversion pipe, the logic tracks how much water has been diverted from the main channel and compares it to the current volume in the facility to preemptively close the diversion and prevent backwater. This scenario would demonstrate added benefit by preventing "cleaner" water at the tail end of the storm from flushing out the "dirtier" water initially captured at first flush or during the peak of the storm event.
Predictive Diversion Control:	Similar to the "Shut Diversion When Full" scenario, this scenario prevents inflow when the capture facility has reached full capacity, but additionally prevents flow from entering the capture facility in *anticipation* of forecasted peak flows. This is accomplished by monitoring the predicted wet weather flow in the 24-hour forecast window and closing the diversion in time to increase storage in time for capture of peak runoff flows. Through effective forecasting and reactivity to current water levels during the storm event and how they change, the diversion can be opened just prior to the peak flow to enable the full diversion rate and facility storage to be used to reduce flooding concerns downstream off the facility.

The scenarios above represent predefined control logic that would be automatically implemented based on monitored site conditions and forecasted runoff; however, an additional level of performance improvement can be attained if *adaptive* RTC logic is implemented. By adjusting the control logic in real-time (instead of relying on predefined control rules), RTC software can best respond to actual conditions and prescribe the optimum control setpoints. Such optimization of control logic has been widely demonstrated for management of combined sewer systems. By applying the same principles for storm drain flow control and optimization, adaptive RTC offers the potential to boost performance of existing and/or constrained stormwater capital infrastructure.

4.2.6 Results Compared to Passive Alternative

The use of RTC represents the ability to apply more "active" management to the stormwater capture facility that can greatly enhance its benefit and functionality in addressing water quality goals and flood control needs. Sizing and configuration optimization have been performed assuming "passive" management so that the facility is designed under the most conservative estimate of benefits to ensure its efficacy to meet project performance goals. Further enhancement of this facility with real-time monitoring and control can improve water quality benefits by ensuring that the most polluted runoff is captured by the facility and filtered before return to the Channel. Additionally, flood control benefits can be greatly enhanced using RTC of the diversion and storage over passive management (Figure 4-5). Under passive management, the initial flows of a hydrograph are diverted into the facility and depending on the size of the event, may quickly fill the storage and require the diversion to be closed well before the peak flow occurs, resulting in ineffective flood control. However, given active control of the diversion to the facility, the peak flow for the event can be effectively reduced by the magnitude of the diversion rate, contributing to better flood control that would only otherwise be possible with a much larger facility that would not be cost-effective or feasible.

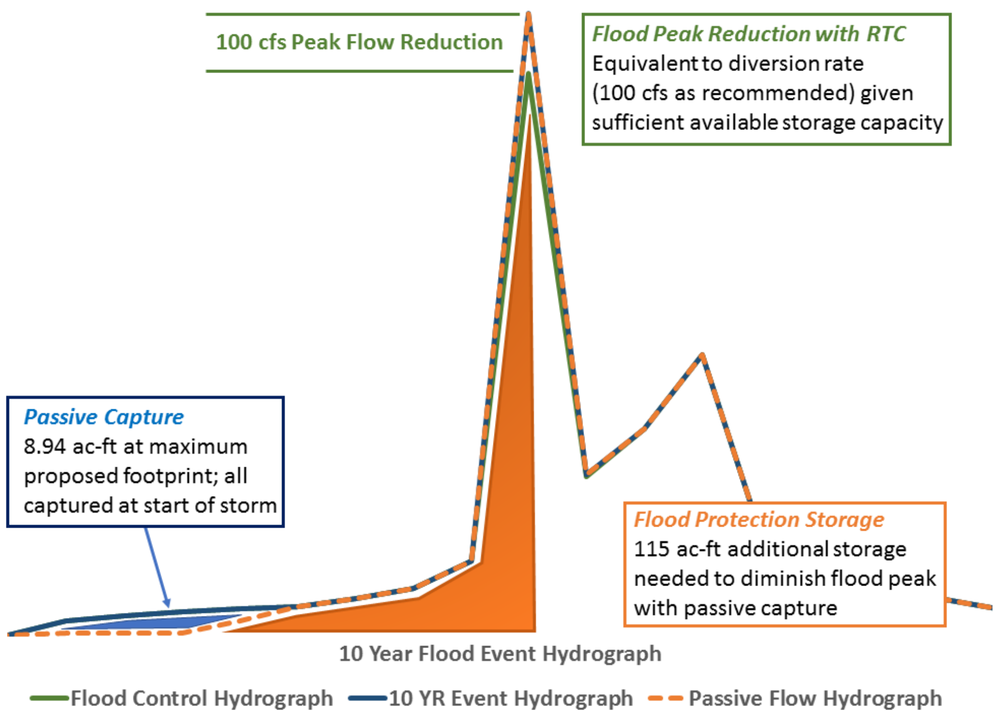

Figure 4-5. Differences in flow capture and timing under passive vs active (RTC) operation

4.3 STORMWATER CAPTURE FACILITY OPTIMIZATION MODELING RESULTS

The following subsections describe the results of the water quality and flood control optimization modeling used to support project design.

4.3.1 Water Quality Optimization Results

The water quality optimization results are based on the runoff volume captured by the facility and the associated PCB loads carried by the runoff from the preliminary RAA baseline timeseries data. These parameters were assessed for both the critical year (2002) and the long-term record (2002 – 2009). Storage volume was initially varied to determine if there was a diminishing return in water quality benefit as storage decreases from the maximum sited volume, but this was not the case and the maximum available footprint was selected. Due to these model results, as well as the emphasis on flood-control for this facility, the maximum storage volume (8.94 ac-ft) for this site was used to inform the remaining optimization scenarios with variable diversion rates and water quality filtration outflow rates. The diversion rates were varied between 50 and 100 cfs to reflect the capacity of single pretreatment units to provide effective capture of sediment prior to entry to the stormwater capture facility so that maintenance needs are minimized (50 cfs each). More details on pretreatment devices can be found in Table 5-1. Similarly, the filtration rates out of the storage facility were varied according to common capacities provided by Bioclean for the Kraken units. The results of these scenarios with associated PCB load reductions have been summarized for the different facility scenarios, as well as performance metrics, in **Table** 4**-**5.

Table 4-5. PCB load reduction with respect to outflow and diversion rates

Filtration	Unit Size	Outflow Rate	Avg. Ann. PCB Reduction (g)		Critical Year PCB Reduction (g)	
			50 cfs Diversion	100 cfs Diversion	50 cfs Diversion	100 cfs Diversion
Single Kraken Unit	8' x 14'	1.82 cfs	5.26	6.04	5.16	6.51
	8' x 16'	2.16 cfs	5.39	6.19	5.26	6.70
	10' x 16'	2.88 cfs	5.57	6.42	5.43	6.94
Multiple Kraken Units	(1) 8' x 12' and (1) 10' x 16'	4.36 cfs	5.71	6.58	5.75	7.41
	(2) 10' x 16'	5.76 cfs	6.12	7.11	5.79	7.52

PCB load reductions are slightly less for the critical year than the overall annual average performance period. Load reductions increase with increased diversion rate, as well as with the outflow rate for the treatment unit. These trends are displayed graphically in Figure 4-6**,** and this data taken together was used to identify the optimal configuration for the stormwater capture facility. Increases in diversion rate result in greater PCB load reduction since more runoff (e.g., more pollutant load) is diverted to the unit to be treated. Greater diversion rates also benefit flood control management because the reduction in peak flowrate is limited by how much runoff can be diverted into the facility. Increases in filtration outflow rates result in increased PCB load reduction due to the capacity to filter more water through the facility and free up capacity for additional flows. The increases due to filtration rate are more modest compared to those for increased diversion rate. Additionally, a higher filtration rate does not offer added flood control benefit since outflows during flood control events will be managed via the bypass (5 cfs) of the filtration unit. Filtration treatment units are very large, and because the site footprint is already constrained, the use of multiple units would possibly require a decrease in storage volume for the facility. These model results indicate that a marginal benefit would be realized for multiple filtration units; therefore, a facility with a single filtration unit with an outflow rate of 2.88 cfs and a diversion rate of 100 cfs will provide the most cost-effective and multi-benefit solution for the Town.

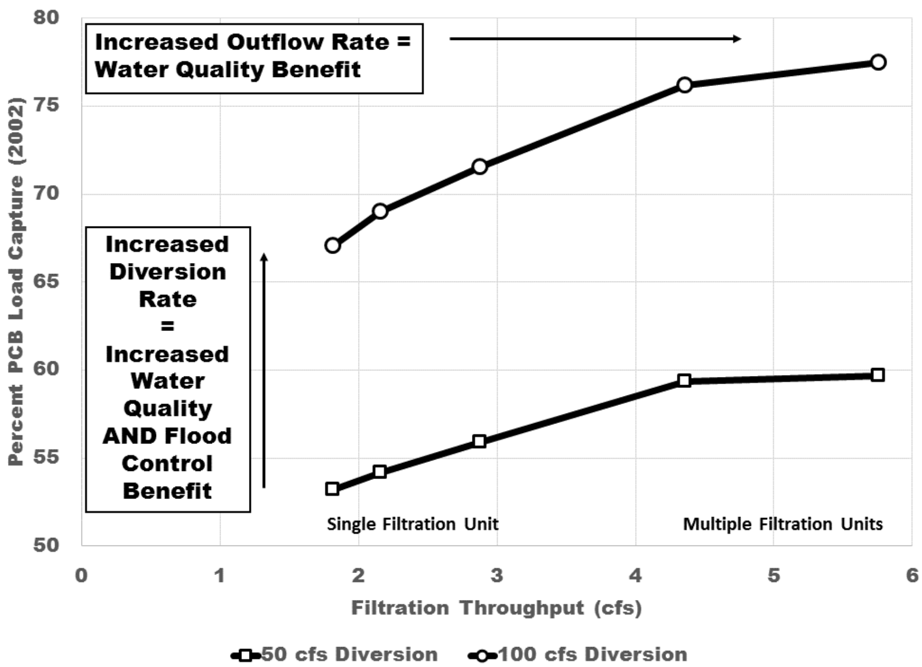

Figure 4-6. Water quality optimization results

4.3.2 Flood Control Optimization Results

Optimization of the stormwater capture facility for flood control was performed to determine the maximum possible peak flow reduction that could be attained given the site layout and design constraints. Results from the water quality optimization indicated that a diversion flowrate of 100 cfs would provide the greatest water quality and flood control benefits. If active controls are used effectively to open the diversion just before the peak flow for a given storm event, the potential peak flow reduction is commensurate to the diversion rate (e.g., 100 cfs diversion would result in a 100 cfs peak flow reduction). The peak flow reduction for a given diversion rate is limited by the available storage capacity in the facility prior to the diversion of flows. If RTC are utilized prior to the peak flowrate to close the diversion and empty any remaining water in the facility to maximize storage capacity, the 100 cfs diversion could be used to reduce higher flows by this full rate for slightly greater than one hour. This means that the peak flowrate for a flooding event could be reduced by 100 cfs during the period when flows in the Channel are greatest.

Long term analysis of flooding in the Channel was performed over the most recent 15 years of record from the preliminary RAA rainfall record (2001 – 2015). This rainfall time-series was modeled to determine the flooding in Atherton Channel resulting from the observed rainfall record to assess the effect that the stormwater capture facility might have on flood control over a longer time period and the fact that natural storms differ in magnitude and intensity from the statistically determined design storms. A subsection of the inflow (peak flows reaching the facility) and outflow (peak flows following diversion using a passive control system) timeseries are presented in Figure 4-7. Additional benefits (e.g., capture of exactly 100 cfs at the peaks) could be realized when using RTC; however, this assessment indicates that even passive controls has a significant impact on smaller, more frequently occurring storms events.

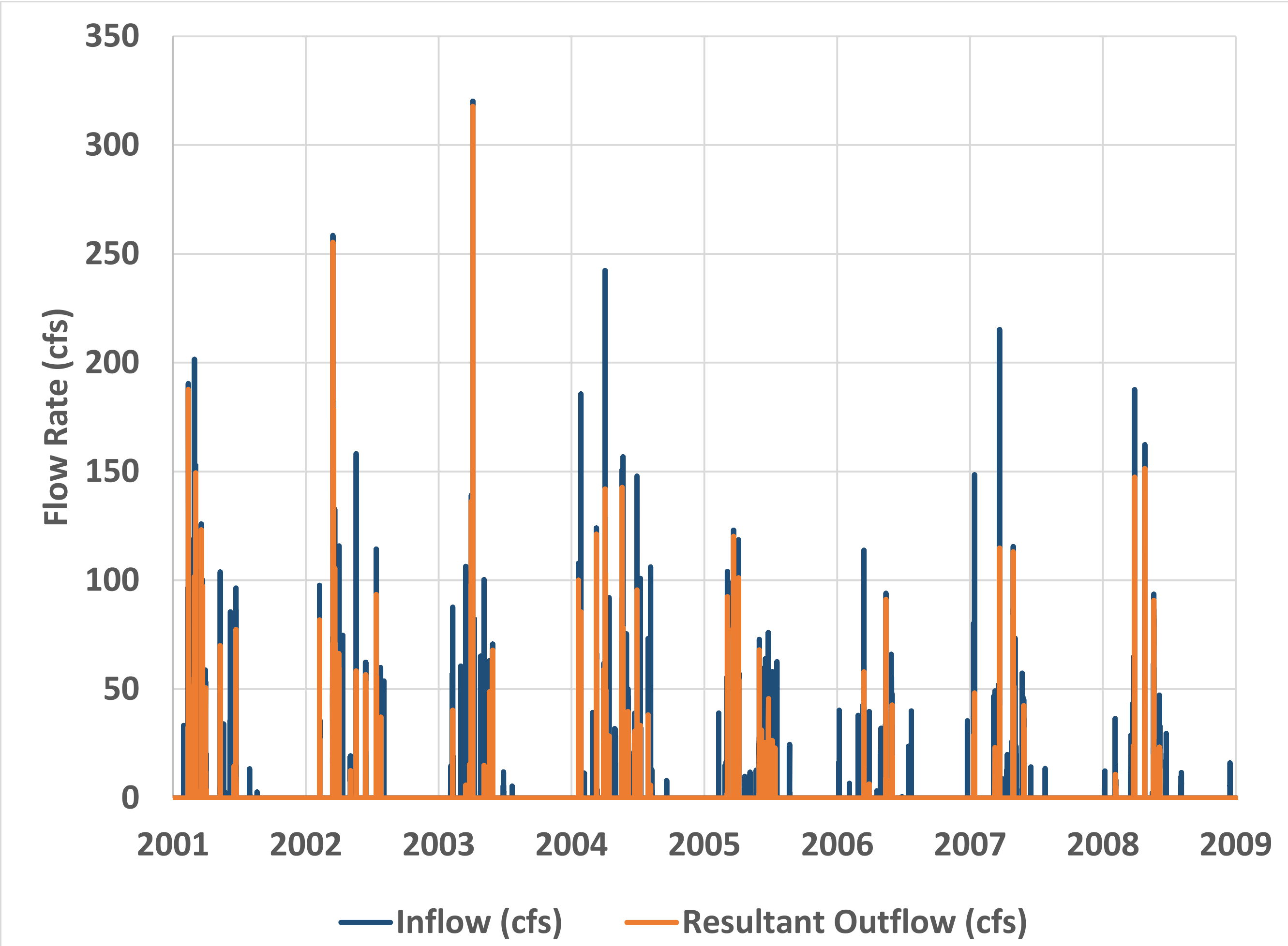

Figure 4-7. Inflow versus outflow from stormwater capture facility over long-term record WY2002-WY2009

During the longer 15-year period, there were 42 rain events greater than 1.0 inch in total rainfall, with the largest storm of 4.26 in. falling on 12/11/14. Peak flow rates for these 42 storms of greater than 1.0 inch ranged from 67 cfs to 573 cfs, with the storm event on 12/11/14 being associated with a peak flow rate of 546 cfs. Based on the PCSWMM modeling results and the capacity for the stormwater capture facility to capture peak flows with RTC, peak flows just downstream of the Park over the 15-year period of record would not top 500 cfs.

The Townwide Drainage Study Update (Town of Atherton, 2015a) indicated that the portion of Atherton Channel downstream of the Marsh Road culvert (just downstream of Holbrook-Palmer Park) is sized to convey flows only up to the 7-year, 24-hour design storm (900 cfs). This significant sizing constraint indicates that flood protection for Atherton Channel for much larger storms is possibly infeasible (i.e., 50- and 100-year storms). Flood management efforts have focused on flooding at the Marsh Road culvert to address drainage needs within the Town's boundary up to the 10-year, 24-hour design storm. Modeling results from the Townwide Drainage Study Update (Town of Atherton, 2015a) were based on the rational method and indicated that the 10-year design storm would result in a peak flowrate of 1,077 cfs at the Marsh Road culvert. PCSWMM results for the models developed as part of this effort indicate a peak flowrate at the diversion to the capture facility of 1,039 cfs for the 10-year storm. With passive management, stormwater would be diverted from the start of the event and flow reduction would only occur up until the facility storage was filled, which is often before peak flows arrive at the diversion point. For the 10-year, 24-hour storm event simulated, the 8.94 ac-ft storage volume would be filled during the first 8 hours of the hydrograph (Figure 4-8). Referencing Figure 4-8 below, it is demonstrated that the flood event peak does not occur until the 12th hour of the flood event. Thus, under passive management, runoff from this drainage area would be too large for the stormwater facility to provide any flood control without RTC.

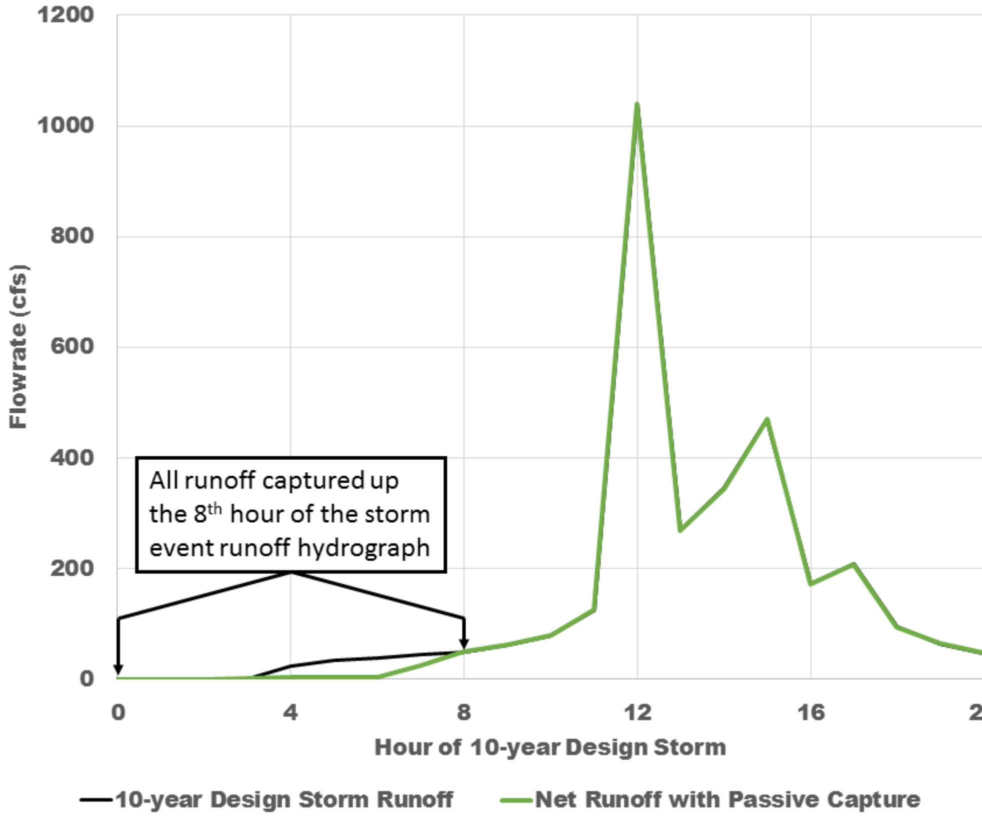

Figure 4-8. *PCSWMM results for 10-year flood event with passive capture*.

The importance for active RTC is critical for meeting flood control targets because the high runoff volumes associated with flooding events quickly exceed storage capabilities for regional capture facilities in the early stages of large rainfall events. Active controls are necessary to ensure that the facility is available to intercept flows when the peak flows do occur.

By operating the facility with RTC and opening the diversion just prior to the peak flowrate, the peak flowrates could effectively be reduced by 100 cfs to 939 cfs (Figure 4-9*).* Although this reduction would not be enough to fully meet the 900 cfs, 10-year design target at the Marsh Road culvert, it would contribute significantly to that management goal with a single project location. It might be feasible to meet the full flood control conveyance target with strategic surface runoff management distributed throughout the Town's drainage area at localized flooding locations in conjunction with the project at the Park.

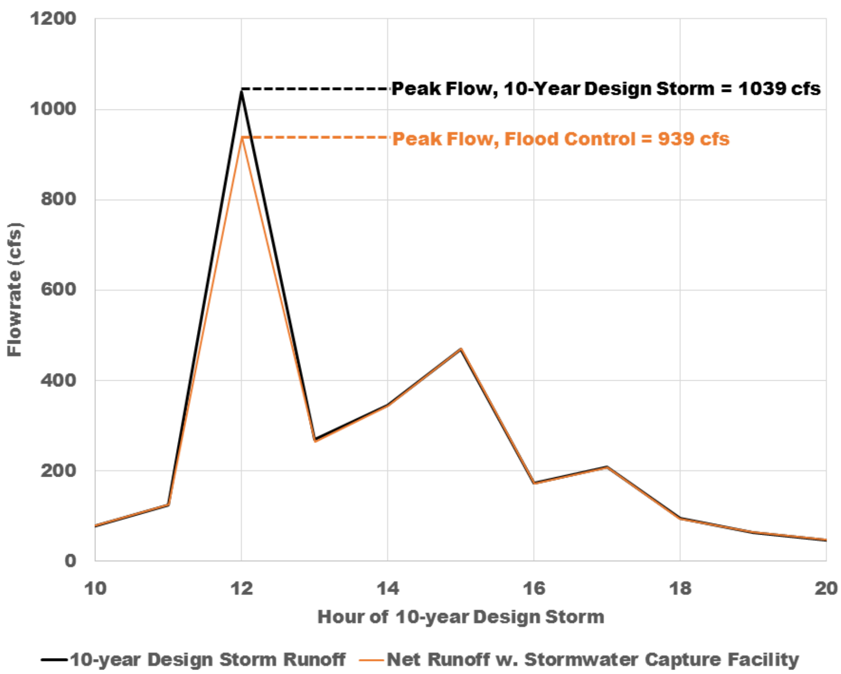

Figure 4-9. PCSWMM results for 10-year flood event with RTC.

Results of the PCSWMM flood modeling indicate flooding at several locations along the Atherton Channel for storms as small as the 5-year, 24-hour flood event (Figure 4-10). The location of many of these locations are consistent with reported flooding concerns in the Townwide Drainage Study Update (Town of Atherton, 2015a). A few flooding locations are also just downstream of the diversion point and could be alleviated when the diversion to the capture facility is open.

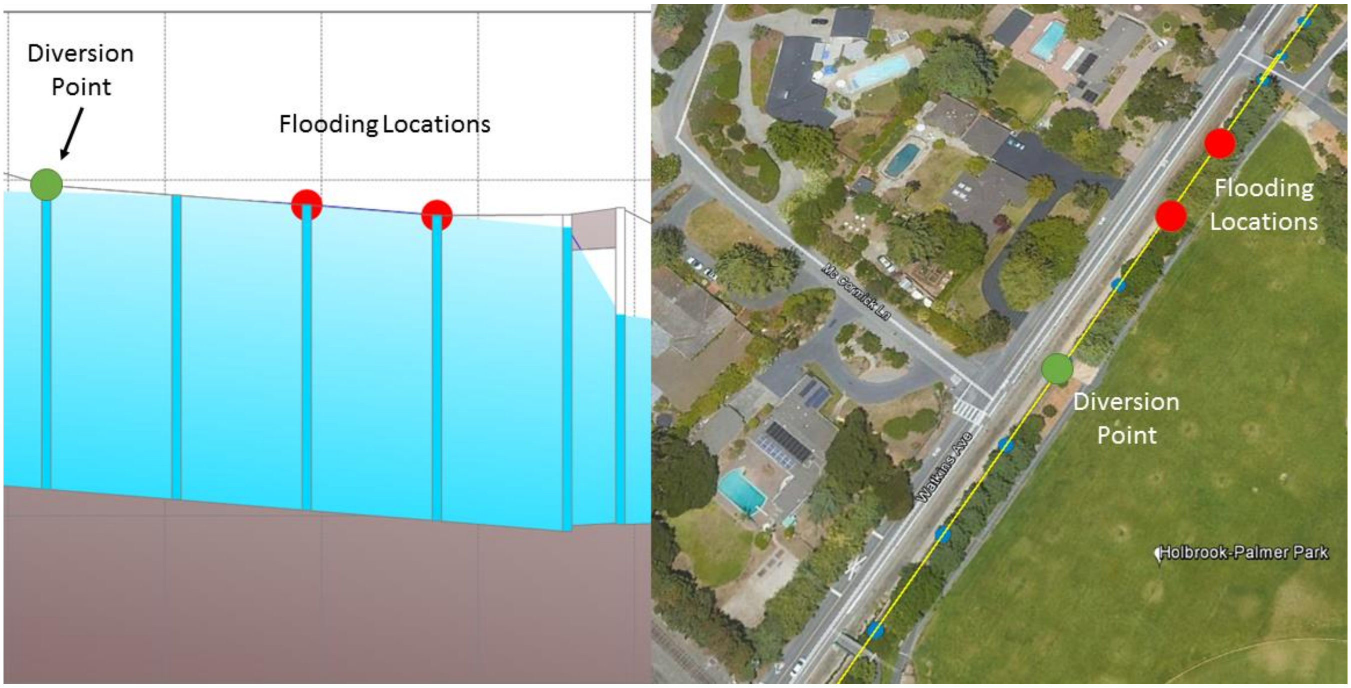

Figure 4-10. Reported flooding location with potential curb levee improvement

4.3.3 Optimal Stormwater Capture Facility Configuration

Based on the results of the optimization analysis and balancing the multiple objectives of water quality benefit and flood control, the following configuration and sizing recommendations for the stormwater capture facility are recommended:

- 8.94 ac-ft storage underground vault with a 10' ponding depth;
- 100 cfs gravity-fed diversion from the Atherton Channel to the stormwater capture facility;
- 2.88 cfs throughput Kraken filtration unit for water quality treatment

The recommended stormwater capture facility can be operated with RTC to provide valuable flood control protection for the Town, as well as water quality benefits detailed below.

4.3.4 Preliminary Compliance Analysis

- Indicate that RAA is in progress and that some revisions may be needed to reflect that modeling
- Preliminary assessment from optimization modeling

Holbrook-Palmer Park was previously selected as a high-priority project for the Town of Atherton to address stormwater management needs in the San Mateo County Stormwater Resource Plan (SRP, 2017). The potential benefits were estimated in the SRP, and these have been summarized below with those for the recommended stormwater capture facility described herein for comparison. It is shown that the recommended stormwater capture facility does not capture as much runoff or mercury as accounted for in the SRP, but it does result in a higher PCB load reduction. Some of these differences in magnitude are related to site constraints that were determined in the more detailed investigation at Holbrook-Palmer Park that has occurred since the publication of the SRP. Additionally, these results are based on preliminary RAA modeling which is still in progress, so some revisions to these results may be necessary once the final RAA model is disseminated. In terms of the MRP goals for PCB and mercury loads to be reduced by GI for the Town, this project would provide additional load reduction than is required by the Town as a percentage of overall load reductions requirements for the County. This assessment too is subject to RAA model revision and may be subject to change.

Table 4-6. Comparison of benefits for the Holbrook-Palmer Park project to the SRP values

Comparison of Benefits	Volume Reduced (ac-ft/yr)	PCB Reduced (mg/yr)	Mercury Reduced (mg/yr)
SRP Potential Benefits	242	3,769	26,746
Recommended Project Benefits	194	6,420	12,930

As specified previously, the flood control peak flow reduction for the optimal configuration could be up to 100 cfs when using RTC. The Townwide Drainage Study Update (Town of Atherton, 2015a) indicated that the capacity of Atherton Channel at the Marsh Road culvert was determined to be 900 cfs, which is approximately 177 cfs below the 10-year, 24-hour design storm simulated as part of that effort. The PCSWMM models developed as part of this PEDR use higher resolution input data and computation complexity and indicate a peak flowrate of 1,039 cfs for the 10-year storm. By operating the facility with RTC and opening the diversion just prior to the peak flowrate, the peak flowrates could effectively be reduced by 100 cfs to 939 cfs, which is notable progress towards the design flow rate of 900 cfs. This facility, in conjunction with distributed flood attenuation efforts, could effectively reduce flows so that they are properly conveyed through the Marsh Road culvert for storms up to the 10-year frequency. A storm of this size only fell twice in the 34-year period that RAA modeling covered, but historical rainfall records indicate that it is a more frequent climate event that should be managed for.

Additionally, flood control benefits related to the ongoing Bayfront Canal and Atherton Channel Flood Management and Restoration Project could be realized by the Town through coordination with the County of San Mateo.

5.0 BMP DESIGN COMPONENTS

This section presents the engineering and design components recommended for Holbrook-Palmer Park based on preceding decision support modeling to capture both dry weather and wet weather flows from the drainage network.

5.1 DIVERSION STRUCTURES

An inflatable rubber dam, like the gates manufactured by Obermeyer Hydro Inc. (see Figure 5-3) is proposed within the Atherton Channel to allow stormwater to be diverted and conveyed to the trash/debris capture device. The rubber dam is proposed to be 3-feet tall when fully inflated, extending the full width of the Channel. To optimize the rate of diversion, and to better match the geometry of the rubber dam diversion structure when fully inflated, the Atherton Channel will be modified from an existing trapezoidal section shape to a box Channel configuration approximately 10 feet on either side of the bridge abutments. The inflatable dam will be controlled by the RTC Control system described in Section 4.2.3 RTC Considerations. The components of this control system will be housed in the corporation yard on the east side of the Park. The air compressor responsible for filling the inflatable dam will also be housed in the corporation yard, hidden from the patrons of the park.

A grated drop inlet will be constructed upstream of the rubber dam and will be sized to capture 100 cfs, which results in dimensions that are 9 feet wide, and 2.8 feet long at a minimum. The diversion structure will funnel diverted water to a 54" pipe, which then sends runoff to the trash/debris capture device.

5.2 TRASH/DEBRIS CAPTURE DEVICE

Stormwater runoff transports sediment, metals, nutrients, trash, and debris that can compromise the performance of stormwater facilities and pollute receiving waters. Removing these, and other similar items, from stormwater prior to entering the proposed storage facility will be an integral part of the strategy to extend the life of the system. Effective capture of larger sediments before they enter the storage unit will allow a long-life expectancy (estimated at least 100 years) for the storage unit. Trash and debris screening will also reduce the maintenance frequency of the Park stormwater facilities and focus maintenance efforts to a concentrated area that is separate from ongoing Park activities.

The recommended diversion rate of 100 cfs was used for selecting an approximately-sized trash/debris capture device. Two types of capture devices are being considered for the project: hydrodynamic separators and a baffle box. Manufactured options for these devices include, but are not limited to, the following: the Jensen Deflective Separator (JDS), the Contech CDS (Continuous Deflective Separation) unit, the Stormceptor, Bio Clean Debris Separating Baffle Box (DSBB), or the Suntree Technologies Nutrient Separating Baffle Box (NSBB). These units are described in the following sections. Other similar units are also readily available and the final selection will be made during later design phases. Ultimately, the goal is to remove 80% of total suspended solids (TSS) that are 150 microns or greater that are diverted to the system.

5.2.1 Hydrodynamic Separators

A typical hydrodynamic separator collects stormwater runoff and directs the water into a separation chamber where water begins swirling, forcing the particles out of the runoff. One hundred percent of floatables and neutrally buoyant debris larger than the screen aperture (2400 microns or 2.4 mm), as well as hydrocarbons, are collected and trapped in the chamber by baffle walls. In addition, at least 90% of particles that are 1/3 the size of the screen aperture are removed for flows up to 100 cfs through the cyclonic motion of stormwater. Solids settle out of the water and fall into the sump chamber below, which is protected from the swirling motion above, eliminating the potential for scour and re-suspension of the solids.

With a proposed diversion rate of 100 cfs at the Park, a single 100 cfs Trash/debris capture unit approximately 18-feet by 18-feet in plan dimension can be employed to effectively remove sediment from the diverted stormwater flow. Figure 5-1 represents a typical Jensen JDS type hydrodynamic separator. The Stormceptor and the Contech CDS are other examples of hydrodynamic separators.

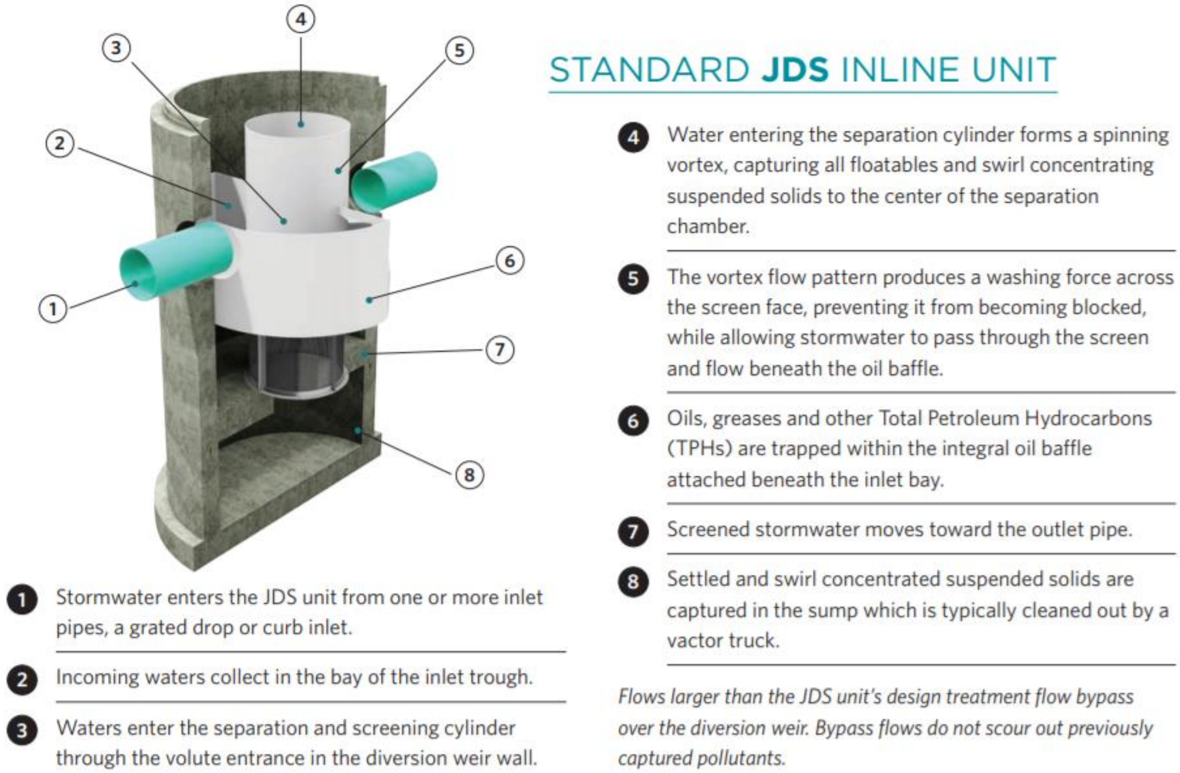

Figure 5-1. Typical Hydrodynamic Separator (Source: Jensen Engineered Systems)

5.2.2 Debris Separating Baffle Box

DSBB by Bio Clean Environmental Services and Suntree Technologies NSBB are also being considered as a trash/debris capture solution for the project. At a total flow rate of up to 50 cfs, DSBBs are available with screen pore sizes of 125, 150, or 250 microns. The DSBB systems use screens that are suspended above the sedimentation chambers that capture and store trash and debris. TSS is removed by routing the flows through a triple chambered system. An oil skimmer with hydrocarbon booms trap and absorb oil. Figure 5-2 illustrates the typical operation of a DSBB system. Two DSBB units, each with a 35’x12’ footprint, operating in parallel will be required to effectively remove sediment from the target flow rate.

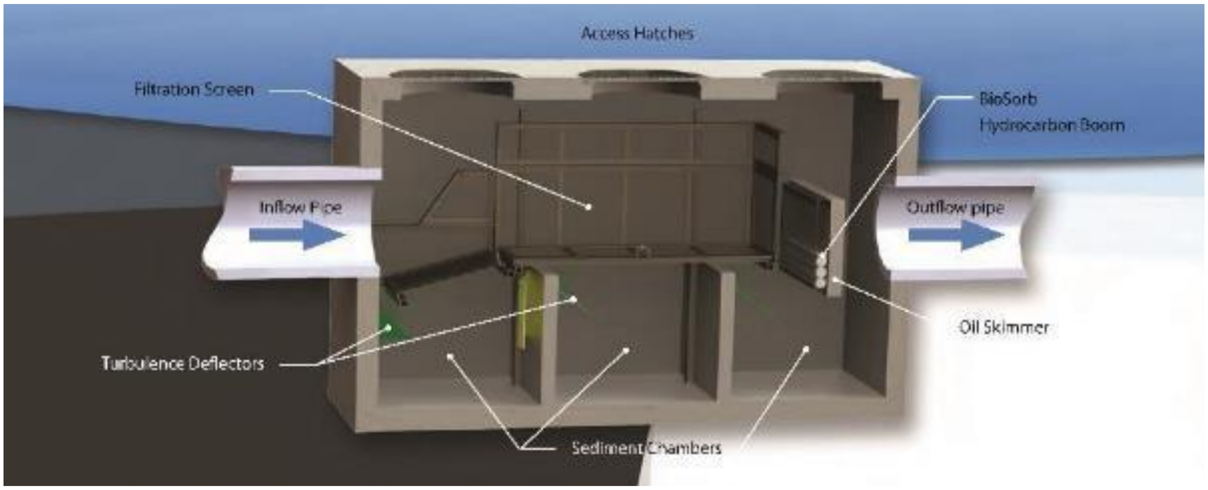

Figure 5-2. Typical DSBB System (Source: Bio Clean Environmental, Inc.)

A summary comparison of the five pretreatment devices is provided in Table 5-1**.**

Table 5-1. Comparison of Pretreatment Devices

	Contech CDS	Jensen Deflective Separator	Stormceptor	Bio Clean DSBB	Suntree Technologies NSBB
100% Gross Solids Removal (Full Capture Device)	Yes	Yes	No	No	No
Internal Bypass	Yes	Yes	Yes	Yes	Yes
Maximum Prefabricated Sediment Storage Sump Capacity	8.7 cy*	37.2 cy	> 70 cy	31.7 cy	> 30 cy
Single unit effective up to 100 cfs	No	Yes	No	No	No
Operate in parallel to achieve treatment of 100 cfs	Yes	Yes	Yes	Yes	Yes
* Contech CDS can be constructed deeper to accommodate greater sediment storage if needed					

5.3 PROPOSED STORMWATER CAPTURE ALTERNATIVES

Underground storage reservoirs can be designed to act as detention and/or retention basins depending on underlying infiltration capacity, and when paired with other treatment measures, make up regional water capture projects that serve to harvest, temporarily store, and treat stormwater runoff to meet a variety of stormwater management objectives. The storage facility at the Park has insufficient underlying infiltration rates to allow for full retention of captured flows, resulting in the necessity to utilize membrane filtration units to remove influent pollutants of concern prior to discharge back into the Channel.

Storage reservoirs are designed to capture a specified design volume and can be configured as online or offline systems. Online systems require an overflow device for managing elevated flow from larger storms. Offline systems require overflow systems or freeboard (the distance from the overflow device and the point where stormwater would overflow the system) to manage excess flow. The storage capture facility at the Park is proposed as an offline system due to the location of the site, as well as the significant flows that pass through the Channel during rain events, which far surpass the facility capacity.

5.3.1 Site Design Alternatives Analysis

Three different site design alternatives were assessed to identify the configuration that would provide the maximum benefit with the least impact to the Park and the patrons of the Park. Items considered for each of the site design alternatives were storage capacity, short and long-term system maintenance, excavation footprint and volume, surface improvements and modifications, Park access/safety impacts, and visibility of the proposed project components. The alternatives considered include:

- Alternative 1 (Figure 5-3) consisted of a single storage reservoir being placed beneath the play field adjacent to the Channel but outside of the baseball field area. The trash/debris capture device and the Kraken filter would be located near the entrance for easy access for maintenance activities.
- Alternative 2 (Figure 5-4) consisted of a two-part storage reservoir being placed beneath the entry lawn and the north parking lot, away from the field areas. The trash/debris capture device and the Kraken filter would be located near the entrance for easy access for maintenance activities.
- Alternative 3 (Figure 5-5) was combination of alternatives 1 and 2: The storage reservoir would be placed beneath the entry lawn and the play field, with a smaller footprint in the play field area. The trash/debris capture device and the Kraken filter would be located near the entrance for easy access for maintenance activities.

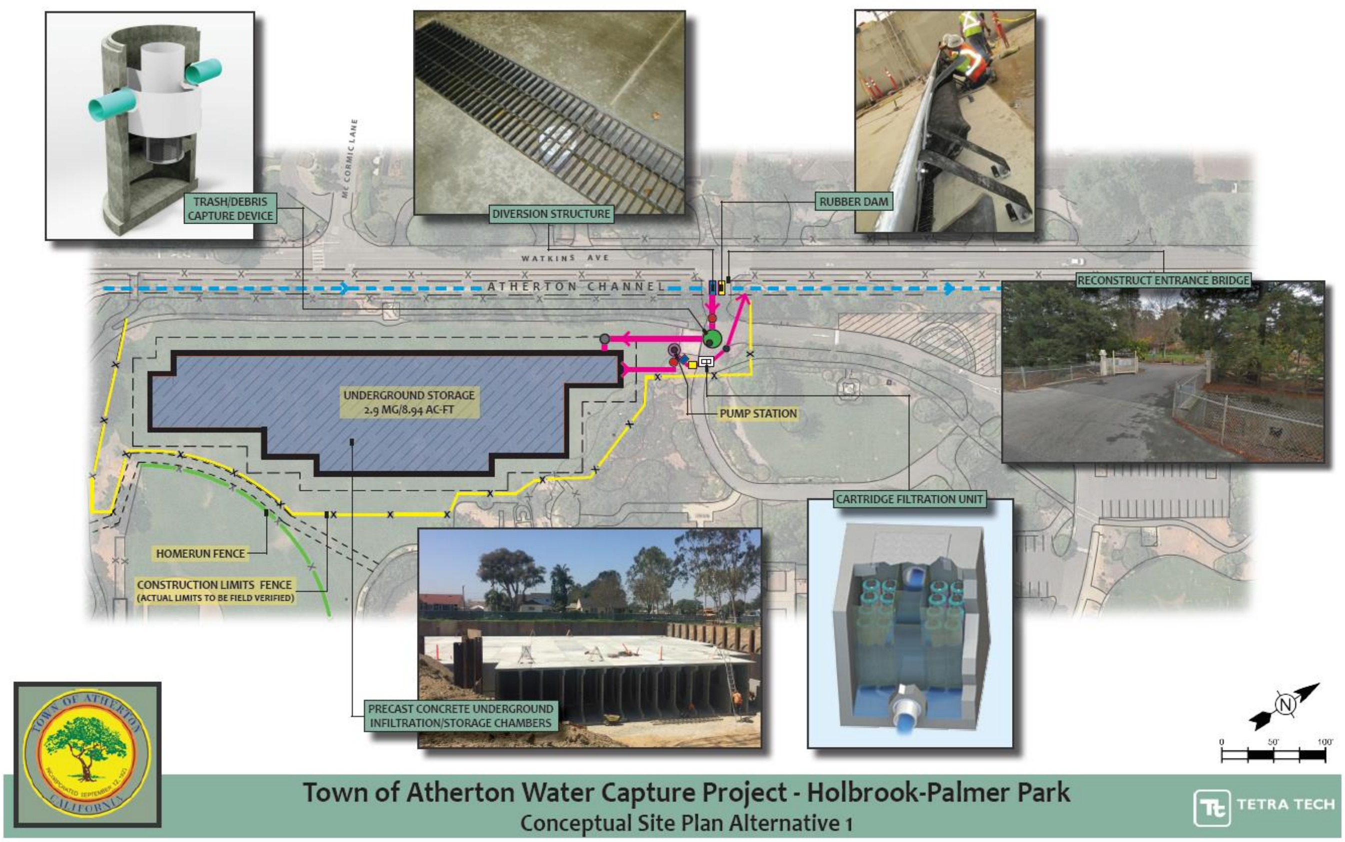

Figure 5-3. Site Alternative 1

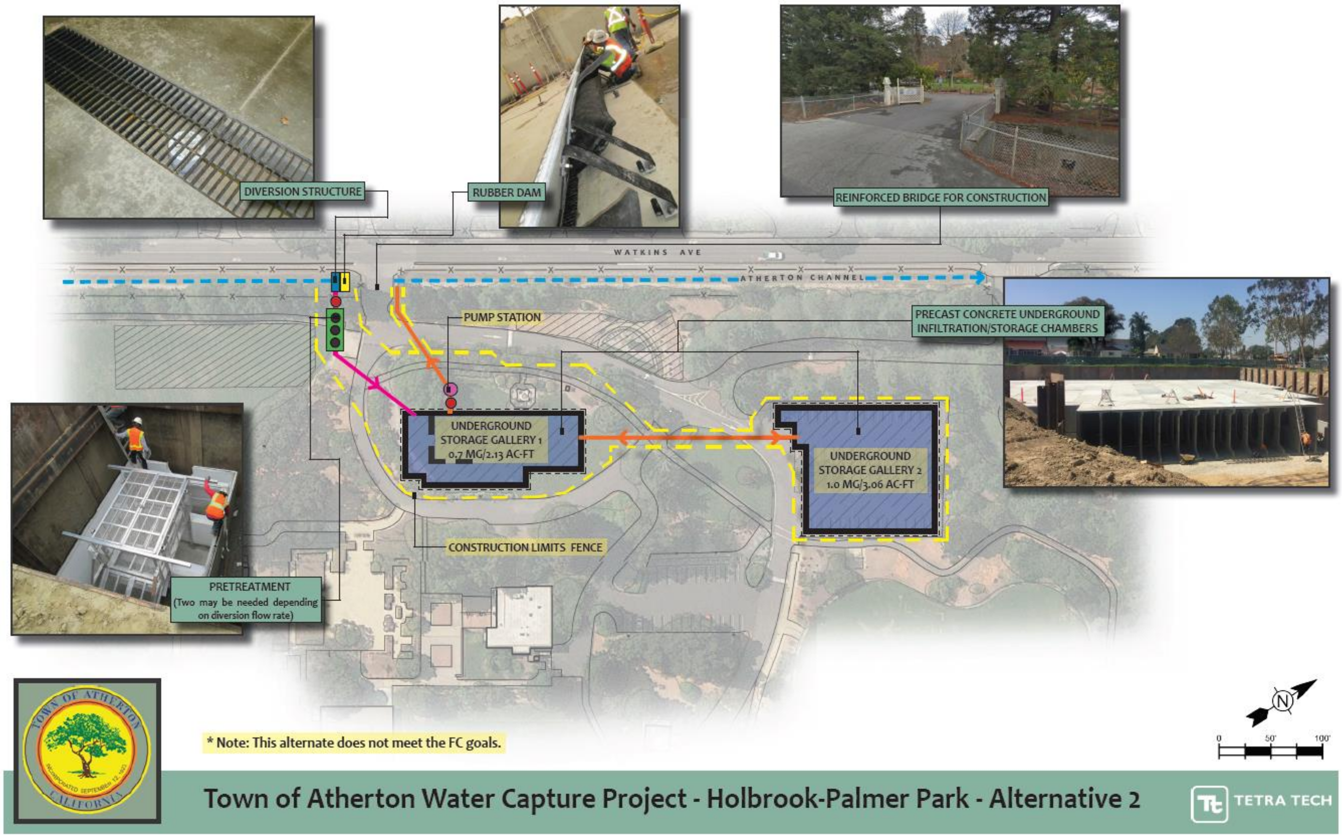

Figure 5-4. Site Alternative 2

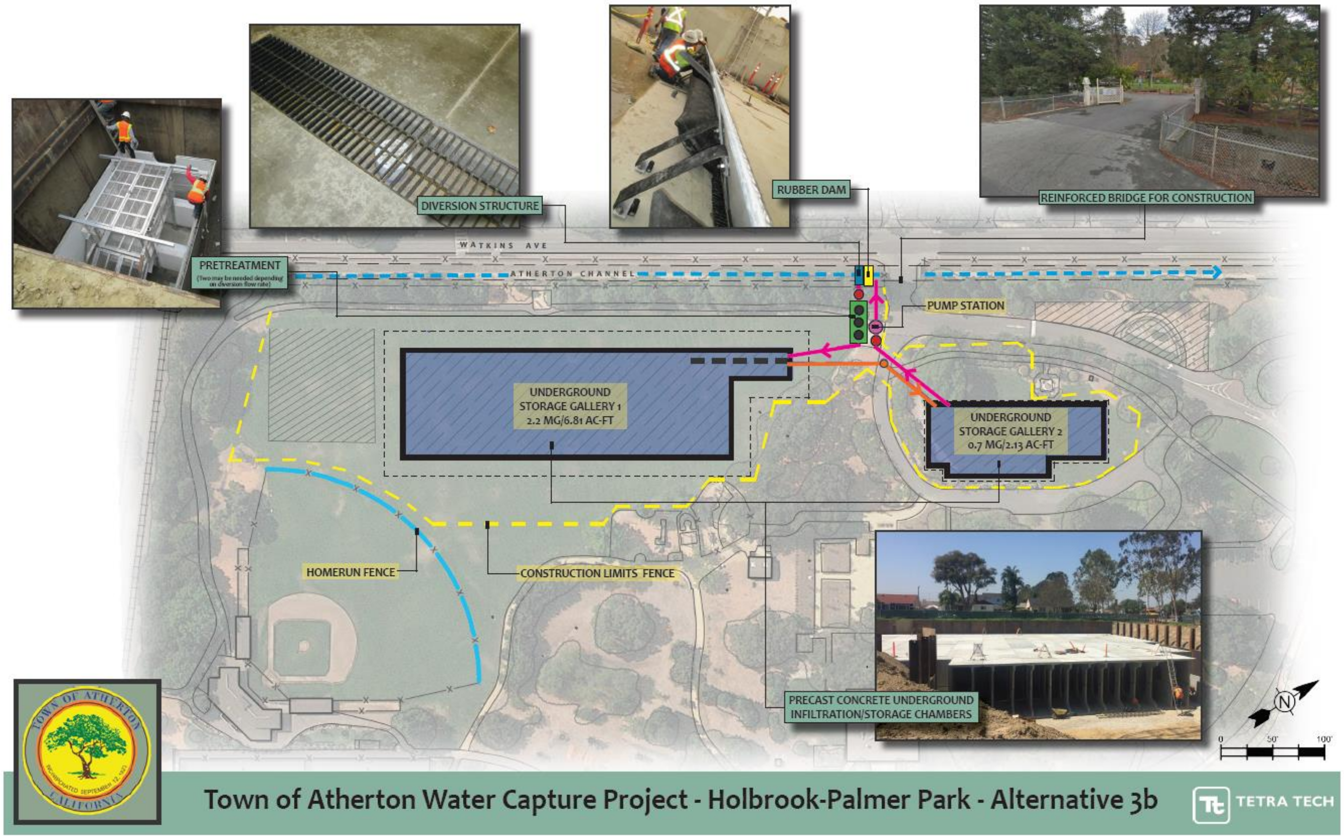

Figure 5-5. Site Alternative 3

A comprehensive pros and cons list for each alternative was developed for each alternative and is provided in Appendix A.

Other aspects of the project were explored as the project evolved and specific design and access considerations needed to be evaluated. Access to the project site is limited to two points of entry, both over bridge culverts spanning Atherton Creek. A structural inspection and assessment was performed by Tetra Tech, Inc. on January 29, 2018, which determined the existing entrance bridge to be undersized for the design loads typical of vactor trucks, construction equipment (hauling trucks, cranes, excavators), and emergency vehicles. Refer to Appendix D for structural assessment. Maintenance vehicles are frequently larger and heavier than passenger vehicles; therefore, they must only travel over bridges with a compatible load rating. Based on a Load Rating report provided by NV5 engineers in 2014, the recommended maximum loading at the bridges were the following (NV5, 2014):

- Single Axle only- maximum axle load of 8 tons (16,000 lbs.),
- Double Rear Axle- maximum load of 6.25 tons per axle for a total of 12.5 tons (25,000 lbs.) total.

To facilitate access of the vactor truck required to remove trash and debris from the pre-treatment device, the entrance bridge will need to be replaced or stabilized with a stronger bridge. This upgrade has additional benefits for the Town, whereas there will be an entry to the Park that is emergency vehicle rated and will allow for those vehicles to safely enter.

Due to the inability to infiltrate all of the stormwater diverted due to underlying soil conditions, several filtration and water reuse options were explored. The reuse of captured stormwater from the facility to offset the irrigation demand at the Park was considered; however, there are not sufficient dry weather flows in the Channel that would offset irrigation needs and justify the extensive aboveground improvements and increased O&M costs that would be associated. Ultimately, water reuse and advanced treatment was not included as part of the recommended water capture facility configuration. In lieu of reusing the captured stormwater onsite, a membrane filtration unit was identified to be the most cost-effective solution for removal of the MRP/TMDL required pollutants. The BioClean Kraken filter has been used for similar regional stormwater capture projects in California and was identified as a feasible option for managing the volumes and flow rates identified for this project. Additionally, the use of bioretention basins and bioswales was also explored as a means of bolstering the projects water quality performance and a way to beautify the Park (Figure 5-6).

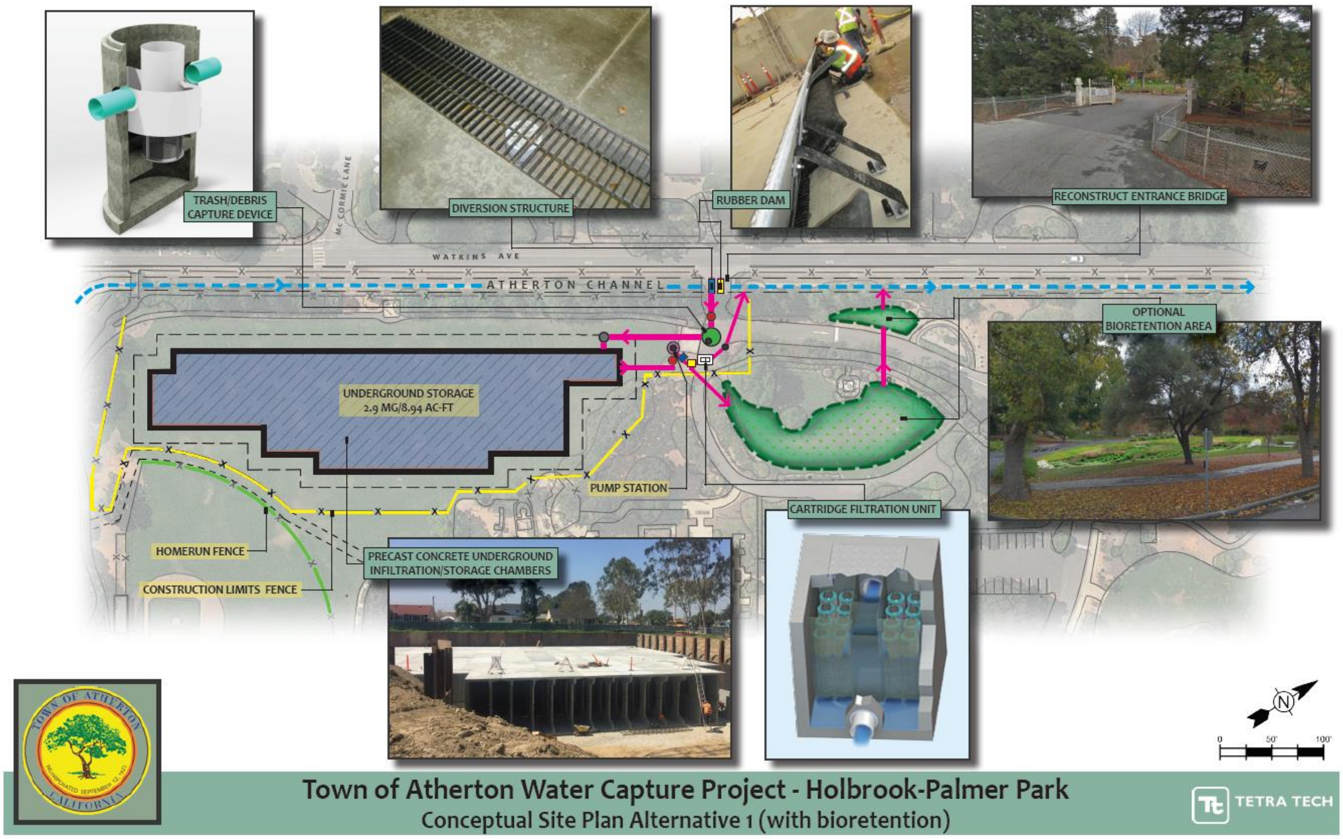

Figure 5-6. Site Alternative 1 with bioretention

Taking each of the site alternatives and design considerations into account, it was determined that the most cost-effective and least intrusive project design would site the stormwater facility storage gallery beneath the playfield (8.94 acre-feet), access hatches of the trash/debris capture device near the entrance of the Park where maintenance vehicles can access them quickly, and consolidate the locations of the submersible pump and a stormwater filter to achieve the water quality goals before discharging back to the Channel (Alternative 1). The building housing the active controls and air compressor will be in the existing corp yard, away from patrons of the Park. An upgraded bridge that is wider and capable of transporting emergency and maintenance vehicles will replace the existing entrance bridge.

5.3.2 Regional BMP Configuration

This PEDR includes an engineered evaluation of the stormwater capture system that has been recommended and features diversion and gravity-fed conveyance from the Atherton Channel to the 8.94 ac-ft underground storage reservoir followed by pumped conveyance from the storage tank to a Kraken filtration unit removing influent pollutants of concern. The pumps would be contained within a wet well and mounted on a rail system to provide some ease during maintenance. Dual submersible pumps could provide up to 2.88 cfs at 80 psi. The pumps would alternate during normal operations and will work together during high demand conditions to discharge 5 cfs from the storage gallery.

Appendix A provides detailed drawings and site layouts of the alternatives.

5.3.3 BMP Structure Alternatives

Underground storage galleries provide temporary or long term stormwater detention in urban areas where undeveloped land is rare and surface space cannot be sacrificed for large, open drainage basins.

Holbrook-Palmer Park is located adjacent to the Atherton Channel and has a large open field available for a sub-surface storage gallery. The preferred BMP site configuration accomplishes the task of providing the greatest flood control benefit and water quality improvements without sacrificing real estate or beneficial uses at the Park. By placing an underground storage facility below the main field at the Park as shown in Figure 5-3 the Town can accomplish both tasks with the smallest social and financial impact to the Town. This project location is optimal because of its proximity to the Channel, minimal impact on Park activities, and ability to potentially expand the footprint of the gallery to provide additional flood control benefits if watershed conditions or precipitation patterns drastically change. Additionally, this location will likely not need shoring, which is an expensive activity that can decrease the footprint of a construction zone but will add prolonged noise, vibration, and the presence of heavy machinery for installation and removal or the shoring. The proposed storage gallery has a volume of 8.94 acre-ft, and a footprint of approximately 1.1 acres.

The underground storage structures for Holbrook-Palmer Park will be designed to be consistent with all state building codes and seismic standards and California Building Code requirements. Specifically, that liquid containing structures shall withstand seismic forces per the loading scenarios described in ACI 350.3-06 “Seismic Design of Liquid-Containing Concrete Structures and Commentary” and the governing design code, such as the 2016 California Building Code. Design will be consistent with similar structures in the Bay Area and high seismic zone regions.

Options for underground storage reservoirs include modular designs and cast-in-place concrete structures. The following sections provide a comparison of such systems.

5.3.3.1 Modular Precast Design

Precast concrete storage systems, such as the StormTrap, Precon, Oldcastle and Jensen StormVault systems, made from durable, reinforced, and high-strength concrete would be the most appropriate modular unit for this project. They can be designed to exceed HS-20 loading, have varying depths of cover, and overcome buoyancy forces. Internal heights can vary to meet the desired storage volume, and underground space constraints. The resulting galleries can be modified to accomplish long term detention or infiltration, depending on soil conditions. A typical precast system can be seen in **Figure** 5**-**7*.*

Figure 5-7. Precast underground storage galleries

5.3.3.2 Cast-in-Place Design

For the cast-in-place option, the roof and walls for both storage tanks would be supported on a concrete mat foundation that would be structurally tied together and designed for HS-20 and other prescribed code loadings (see Figure 5-8**.** The construction of the storage tank, excluding the mobilization, excavation, and demobilization of the site, would take approximately 20 weeks to construct and cure. The main disadvantages of cast-in-place when compared to precast is the amount of time it takes between stages for the concrete to set and cure which directly increases the cost of the system. This is especially critical for backfilling, as the walls are required to reach 70 percent compressive strength before proceeding. The advantage of the cast-in-place structure is flexibility of design and watertight characteristics that may be limited with the precast option.

It is assumed that the precast option will be brought to the site in several pieces and connected in the field. This would be in the form of precast box culverts and/or rigid frames that are essentially linked together. Depending on the final configuration, a cast-in-place foundation may be required, along with the precast structure, to account for liquefaction and resist buoyant forces that the structure will experience. Although the precast option can be constructed with an accelerated schedule, the joints at which each segment would be connected are more susceptible to leakage, and special design consideration would need to be considered by the manufacturer to ensure a water tight seal. If a precast system is used, the construction schedule would be cut by 30% to 50%.

Figure 5-8. Example of cast-in-place underground storage vault construction

Increased construction time and funding allocation schedule are concerns related to the cast-in-place method due to the construction time for cast-in-place being almost double when compared to pre-cast installation. The project is funded through an agreement with Caltrans which requires the project to procure and invoice portions of the construction contract by specific milestones. The first milestone requires the contractor to procure or complete an invoice of $5.6 million by June of 2019. If a specific milestone is not met the project loses a portion and possibly all of funding allocated to the project. The prolonged schedule associated with the cast in place alternative makes this method potentially not practical for all projects. The precast option allows the contractor to preorder the units enabling a timely purchase order to be invoiced and meet the funding schedule. Funding is critical to the completion of the project and the precast units allow the allocation schedule to be met.

5.3.3.3 Reservoir Design Overview

The storage gallery at the Park will be designed to avoid trees, the baseball field and the Park entrance driveway. The depth of the reservoir will vary, but will be at least 10 feet deep and will be capable of storing 8.94 acre-feet during storm events. A baffle wall be installed to promote settling of any material still suspended in the influent following pretreatment. Geotechnical analysis of the soil below the facility location indicates that the soil is not suitable for infiltration as the primary means of removing water from the storage gallery (e.g., infiltration rate will not allow drawdown of the facility within an acceptable timeframe to prevent vector concerns). The storage gallery will be designed to allow for incidental infiltration, but will rely on filtration through Kraken units or the flood control bypass as the main outflow mechanisms.

5.4 IRRIGATION IMPROVEMENTS

Maintenance staff at the Park reported that the existing irrigation and water delivery system is prone to leaks and not very efficient. An updated system, such as the one that would be replaced as part of the stormwater capture project, would help the Park reduce overall water usage.

5.5 LANDSCAPING AND PARK IMPROVEMENTS

Landscaping of the Park after the stormwater capture facility construction is complete will return the Park to its existing condition, including new grass/sod and plantings. Surface improvements will be contained to the areas affected by construction. The walkways throughout the Park which are affected by the construction will be replaced per the Park Master Plan and per directions from the Parks and Recreation Commission.

In addition to the surface improvements in the construction zone, the entrance bridge will be replaced. The current bridge is not capable of supporting construction equipment or maintenance vehicles, so a new bridge will be installed prior to construction of the stormwater capture system. The Town will be able to use this updated bridge for emergency vehicle access, which is currently unavailable due to the structural capacity of the existing bridge.

5.6 OPERATIONS AND MAINTENANCE CONSIDERATIONS

Operations and maintenance of the water capture project at Holbrook-Palmer Park is critical to maintain the prolonged performance and associated water quality benefits. The following sections detail specific O&M design considerations specific to the site conditions at the Park location.

5.6.1 Maintenance Requirements

Maintenance requirements for stormwater projects often vary from traditional public works maintenance. To continually achieve the water quality and flood control objectives of the project certain maintenance will be required over time. The components of this project were selected to minimize maintenance needs, and consolidate maintenance access points to minimize impacts on Park activities. The primary maintenance activity is debris removal that will be performed at least quarterly using a vactor truck.

5.6.2 Maintenance Access

Both the access hatches and manholes that will be located near the entrance of the Park and the reconstructed bridge have limited impact on common access routes through the Park (see Figure 5-9). Maintenance vehicles entering the Park will be able to do so safely and with adequate spacing and turning radius to prevent disruption to Park activities. For ease of access, hatches and manholes have been clustered at a location outside of the existing Park roadway section and walking paths. A separate concrete drive aisle will be constructed to facilitate uninhibited access to inlet and outlet hatches for cleaning and maintenance purposes. Required maintenance will therefore be performed away from vehicular and pedestrian traffic. Maintenance will also be scheduled to minimize impacts on ongoing Park activities.

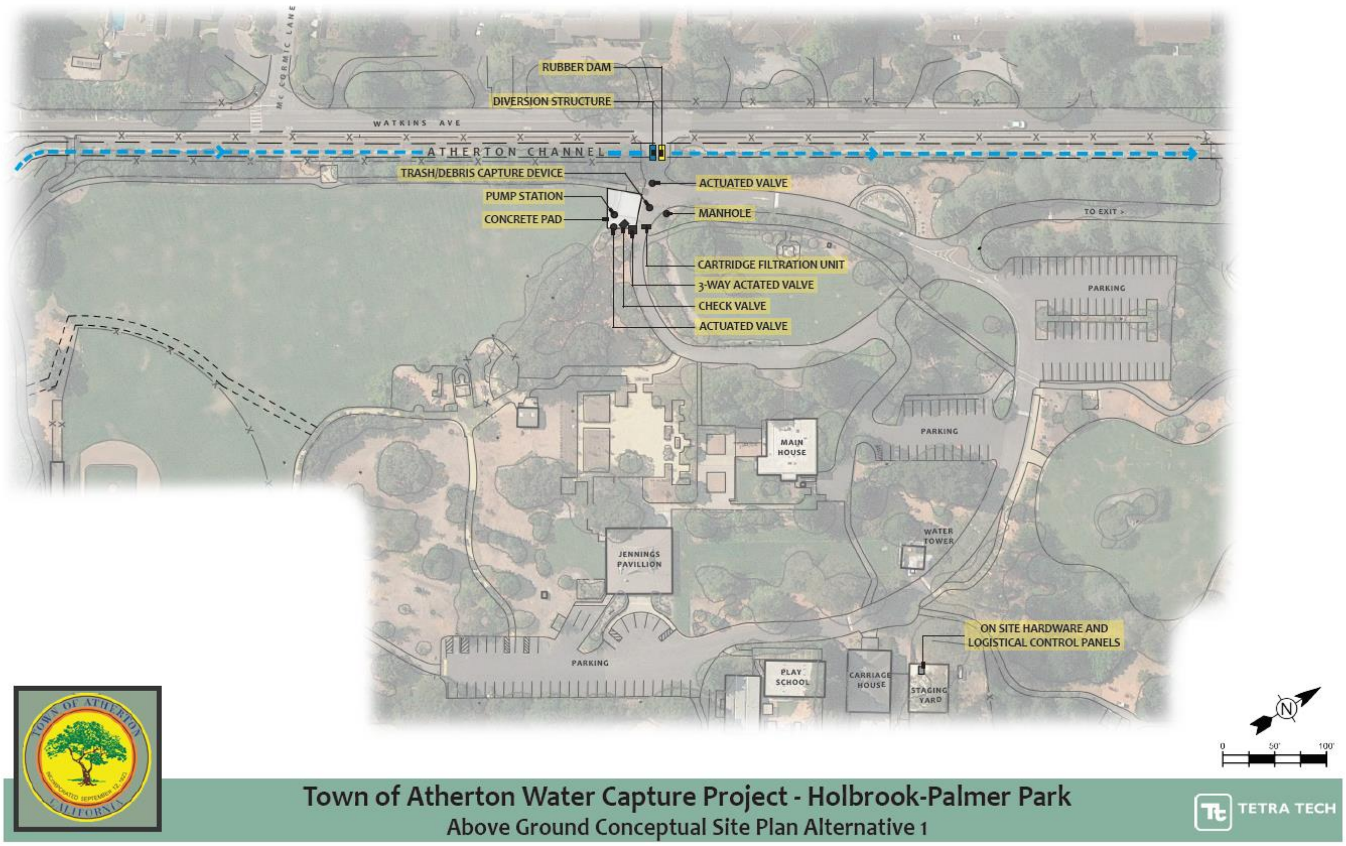

Figure 5-9. Access Hatches and Manhole Maintenance Locations

5.6.3 Maintenance Manual

A maintenance manual will be developed with detailed site schematics and task descriptions to streamline and tabulate future maintenance efforts. This manual will be tailored specifically to this project site to provide continuity of correct protocols between revolving town and maintenance staff, as well as efficiency of transferring knowledge between staff.

6.0 PERMITTING, COST ESTIMATE, AND SCHEDULE

Environmental documents and permits will be required for the completion of the water capture project at Holbrook-Palmer Park. Section 6.1 provides an overview of the different requirements that are applicable to this project, as well as any potential timeline implications.

The cost estimate and project schedule have been created to validate that the preliminary design may be built within the specified budget and within the time allocated to use the funds per the Caltrans CIA agreement.

6.1 ENVIRONMENTAL DOCUMENTS AND PERMITS

Consultation with regulatory agencies and acquisition of permits is required before the project components can be constructed. The following sections summarize regulatory permits and approvals relevant to the project.

6.1.1 CEQA Requirements

The California Environmental Quality Act (CEQA) requires that project proponents study and disclose a project's anticipated water quality and other environmental impacts and specify means to avoid or minimize those impacts. Any project needing state or local agency approval must comply with CEQA or indicate that its project is exempt from CEQA, pursuant to the exemptions described in CEQA regulations.

An Initial Study/Mitigated Negative Declaration (IS/MND) will be prepared by the design team based on CEQA requirements and State CEQA Guidelines, and any applicable guidance and procedures set by the Town. The IS/MND will be an informational document to be used in the local planning and decision-making process. By nature, the IS/MND does not provide recommendation for approval or denial of the Project. Instead, the IS/MND will describe the Project and its environmental setting including a discussion of each environmental impact category and supported by adequate documentation, including the Project area's existing conditions and applicable regulatory requirements.

Public participation is considered an essential part of the CEQA process and reflects a belief that citizens can make important contributions to environmental protections and notions of balanced decision-making through wide public involvement. The IS/MND will evaluate potential environmental impacts to the following resources:

- Air Quality: An air quality analysis will assess the potential air quality impacts that may arise from the implementation of the project, in compliance with the Bay Air Quality Management District (BAAQMD) CEQA Air Quality Guidelines.
- Biological Resources: Data regarding sensitive habitats and special-status species will be reviewed to determine if those protected resources occur or have the potential to occur with the project site.
- Cultural Resources: A phase I cultural resource assessment will be conducted.
- Greenhouse Gas Emissions: Greenhouse Gas Emissions (GHGs) from construction and operation activities will be estimated and analyzed in compliance with the BAAQMD CEQA Guidelines.
- Noise: Potential Noise and vibration impacts associated with the construction and operations activities of the project will be assessed.

6.1.2 United States Army Corps of Engineers and California Regional Water Quality Control Board, San Francisco

As part of the CEQA process, a wetlands specialist will perform a wetland delineation to determine the extents of the Waters of the U.S. and Waters of the State and prepare a report for submittal to the Army Corps of Engineers for verification in support of federal permitting. Because this project may result in a discharge of dredged and/or fill materials into waters of the United States, a Department of the Army Nationwide Permit may be required pursuant to Section 404 of the Clean Water Act (33 USC 1244; 33 CFR parts 323 and 330).

Additionally, the Federal Clean Water Act, in Section 401, specifies that states must certify that any activity subject to a permit issued by a federal agency, such as the Corps, meet all state water quality standards. In California, the State Board and the regional boards are responsible for taking certification actions for activities subject to any permit issued by the Corps pursuant to Section 404. As a non-discretionary special condition of a Nationwide Permit, a Section 401 Water Quality Certification may be required by the Regional Water Quality Control Board, San Francisco Bay Region, pursuant to Section 401 of the Clean Water Act and the Porter-Cologne Water Quality Control Act

6.1.3 Bay Area Air Quality Management District

The Bay Area Air Quality Management District (BAAQMD) is responsible for the issuance of air quality permits for stationary equipment in the Bay Area and the management of the resulting air emissions (pollutants). Nearly all stationary equipment that emits to the atmosphere requires an Air District permit. An air quality permit is a document that gives the permit holder authorization to build equipment and/or to operate that equipment. Each project is evaluated before a business can build and operate their equipment to ensure that all air quality requirements are met.

Efforts implemented during the CEQA phase described above will assess the potential air quality impacts that may arise from the implementation of the project. Included under CEQA efforts, the GHGs from construction and operation activities will be estimated and analyzed in compliance with the BAAQMD CEQA Guidelines.

Once constructed, the project will result in the displacement of atmospheric air from within the storage unit at a rate equal to the proposed stormwater diversion rate from the Channel. During design (phase 2), the BAAQMD will be notified of the proposed project, and CEQA findings will need to be discussed to determine whether the project is exempt from BAAQMD current rules or whether an Authorization To Construct and an Permit to Operate will be required for this project.

6.1.4 State Water Resources Control Board

Amendments to the federal Clean Water Act (CWA) require construction activities consisting of one acre or more to develop and implement Storm Water Pollution Prevention Plans (SWPPPs). In California, construction activities are regulated under the NPDES General Permit for stormwater discharges associated with construction activity adopted by the State Water Resources Control Board. The goal of the NPDES permit is to stop polluted discharges from entering the storm drain system and local coastal waters during construction activities.

The construction NPDES General Permit requires all permitted dischargers to:

- Develop and implement a SWPPP that specifies BMPs that will prevent construction pollutants from contacting stormwater with the intent of keeping products of erosion from moving off-site into receiving waters
- Eliminate or reduce non-stormwater discharges to storm sewer systems and other waters of the nation
- Perform inspections of all BMPs

Prior to the start of project construction, the City will submit a Notice of Intent (NOI) to obtain coverage under the General Permit and provide the permit fee and a site map. A SWPPP will be required for this project and the Construction Contractor will implement the SWPPP. The SWPPP must address the use of appropriately selected, correctly installed, and adequately maintained pollution reduction BMPs for compliance with the State Water Resources Control Board Construction NPDES General Permit.

6.1.5 California Department of Fish and Wildlife

Due to the rubber dam and grated drop inlet proposed to be constructed within Atherton Channel, this project will be subject to the notification requirement in Fish and Game Code Section 1602. Although unlikely, if upon review of the CEQA findings, the California Department of Fish and Wildlife determines that this project will result in substantial adverse effects to an existing state fish or wildlife resource, a Lake or Streambed Alteration Agreement will be needed.

6.1.6 Local Construction Permits

Depending on the selected concept, the Town of Atherton may require building and grading permits. Minor traffic control will be necessary during the construction activities, as well as for the hauling of export from the project during the excavation phase.

6.2 PRELIMINARY COST ANALYSIS

The cost analysis is utilized as a tool to ensure preliminary design is within the amount of funds available to the project. If the cost analysis indicates that the project is not feasible, then the design will need to be adjusted to bring it within the project budget while still meeting the project goals. The cost analysis was developed using various sources of information, as well as the Cost Estimator's judgment.

6.2.1 Construction Cost

The construction cost entails the various components of the project that a Contractor would construct for the City. Construction costs do not include items of work not directly performed by the Contractor, such as the City's construction management during construction. The construction costs were developed using various sources of cost information, such as Caltrans cost data, RS Means, vender and supplier provided costs and past bid data for similar projects. The estimated construction cost is $11,448,798 for the recommended optimal stormwater capture project configuration with a 20% design estimating contingency. Table 6-1 lists the respective breakdowns of the items required to complete the project.

Table 6-1. Estimated Construction Costs, Optimal BMP Configuration

Holbrook-Palmer Park Stormwater Capture Project	
Mobilization/Demobilization (3%) and SWPPP	$305,000
Channel Diversion and Pretreatment	$295,000
Wet Well and Conveyance	$460,000
Site Preparation and Demolition (Existing Park Area)	$80,000
Storage	$7,185,665
Kraken Filter	$155,000
Electrical Service, Controls, Instrumentation	$260,000
Landscape and Irrigation	$550,000
Site Amenities and Improvements	$190,000
Start-up, Testing, O&M Manuals, Record Drawings	$60,000
Subtotal	9,540,665
Design Estimating Contingency (20%)	$1,908,133
TOTAL	$11,448,798

6.2.2 Operations & Maintenance Costs

The operations and maintenance costs were developed on the basis that a service contractor would maintain the various components of the system. Estimated total annual operations and maintenance costs are presented in Table 6-2.

Table 6-2. Annual Estimated Operations & Maintenance Costs

Description	Frequency	No. of Times per Year	Unit Price	Total
Channel Diversion and Pretreatment				**$12,000**
Rubber Dam System – Inspection and Cleaning	Monthly	12	$250	$3,000
Diversion Structure – Inspection and Cleaning	Monthly	12	$250	$3,000
Pretreatment Device – Vacuum	Quarterly	4	$1,500	$6,000
Wet Well and Conveyance				**$19,621**
Dry Season Inspection and Cleaning (Vacuum)	Every other month	3	$750	$2,250
Wet Season Inspection and Cleaning (Vacuum)	As needed	6	$750	$4,500
Electrical Usage	Monthly	12	$300	$3,600
Valve Maintenance	As needed	1	$1,000	$1,000
Control Panel Maintenance	As needed	1	$1,000	$1,000
Pump Replacement	Every 20 Years	1/20	$25,425	$1,271
Kraken Filter	Annually	1	$6,000	$6,000
Storage				**$16,000**
Dry Season Inspection and Cleaning (Vacuum)	Quarterly	2	$4,000	$8,000
Wet Season Inspection and Cleaning (Vacuum)		2	$4,000	$8,000
Active Controls				**$25,000**
Continuous Monitoring and Adaptive Control	Continuous	1	$25,000	$25,000
Total O&M Costs				**$72,621**

6.2.3 Project Cost Summary

Project costs include all the necessary items to provide a finished product, including predesign, design, construction, construction management, and post construction work. The estimated project capital budget for the recommended configuration is $13.5 million, with a 10-year total project cost of $14.2 million as shown in Table 6-3**.**

Table 6-3. Comparison of total project capital costs and long-term operations and maintenance costs

Cost Component	Cost
Construction (With 20% design)	$11,448,798

Predesign (6% of construction)	$686,930
Design (7% of construction)	$805,000
Construction Management (5% of construction)	$572,440
Capital Cost Subtotal	**$13,513,168**
10-Year Operations and Maintenance Subtotal	**$726,210**
10-Year Total Project Cost	**$14,239,378**

6.3 FUNDING SOURCE AND IMPLEMENTATION SCHEDULE

The Town and the County had previously identified the Park as a priority project for safely improving water quality and flood prevention and have coordinated with Caltrans via a CIA for funding of the design and construction of the project. Caltrans property is primarily highways and typically composes only 2% of any one watershed within the State of California. Because of the high level of stormwater pollution contributions from Caltrans property, they are subject to meet stringent stormwater compliance targets throughout the state of California. In lieu of addressing stormwater compliance along their contributing land in a piecemeal, expensive, and multi-watershed method, a special TMDL attachment was adopted (Attachment IV, 2014) to create a more efficient and economical way to comply with pollutant reduction known as stormwater compliance units. These units are meant to create a cooperative implementation of stormwater capture projects throughout California by pairing Caltrans funding with previously identified stormwater capture projects. Unlike most infrastructure funding sources, those provided by Caltrans do not require matching from the Town, resulting in low capital costs for the Town to meet its flood control and water quality goals. There are constraints of the funding, however, which include that the funds be spent within three (3) Caltrans Fiscal years and that local agencies agree to undertake long term maintenance of the constructed facilities. The CIA agreement ensures that the project will be funded for design and construction, but also operated and maintained over time to continue benefiting the community and nearby San Francisco Bay.

As stated in the CIA, the Town is required to bill Caltrans for funding reimbursement on April 30, 2019 for the Caltrans Fiscal Year 2016-2017 funding allocation, April 30, 2020 for the Caltrans Fiscal Year 2017-2018 funding allocation, and April 30, 2021 for the Caltrans Fiscal Year 2018-2019 funding allocation. As a result, construction of the facility must be completed by March 31, 2021. The preliminary implementation schedule is provided in

Table 6-4**.** A preliminary funding allocation schedule for Caltrans is shown in Table 6-5**.**

Table 6-4. Preliminary Implementation Schedule

Description	Start Date	Finish Date
PHASE 1: Project Engineering Study Report	11/16/2017	5/11/2018
Environmental Documentation	5/28/2018	11/26/2018
PHASE 2a: Prepare Procurement Documents for Precast Modular Units	8/17/2018	9/27/2018
PHASE 2b: Procurement of Precast Modular Units	10/4/2018	2/28/2019
PHASE 2c: Detailed Design Documents / Bid and Award	5/28/2018	6/27/2019
PHASE 3: Construction Implementation	7/11/2019	3/31/2021

Table 6-5. Preliminary Caltrans Funding Allocation Schedule

Funding Period (Fiscal years)	Amount	Date
FY16-17	$5,600,000	June 2019
FY17-18	$1,900,000	June 2020
FY18-19	$3,000,000	June 2021
FY 19-20	$3,100,000	June 2022

7.0 CONCLUSIONS & RECOMMENDATIONS

This PEDR was prepared for the Town for a multi-benefit water capture facility at Holbrook-Palmer Park to mitigate flooding near the Atherton Channel, as well as advance progress towards compliance with MRP requirements. The existing site conditions, utilities, geotechnical conditions, hydrology, hydraulics, and water quality were characterized, and a subsequent optimization analysis was performed to determine the most cost-effective solution to provide multiple benefits to the Town. The modeling assessment has identified the recommended configuration:

- Diversion of 100 cfs from the Atherton Channel
- Storage of 8.94 ac-ft in a subsurface facility with a minimum ponding depth of 10 feet
- Water quality filtration through a membrane filtration unit (Kraken) with a treatment rate of 2.88 cfs
- Flood control bypass of the filtration unit using RTC with a rate of 5 cfs to generate additional capacity for peak flows, when needed
- Incidental infiltration through the base of the storage facility

This configuration will aid in the reduction of peak flows in the Channel up to 100 cfs and will alleviate local flooding concerns for smaller, more frequently occurring events. The predicted water quality performance will allow for the Town to make significant progress towards the MRP/TMDL PCB and Mercury load reduction requirements, with final assessments contingent on the updated regional RAA models. This recommended configuration satisfies the terms of the Caltrans funding agreement and is estimated to be within the funding allocation amounts provided in the CIA.

8.0 REFERENCES

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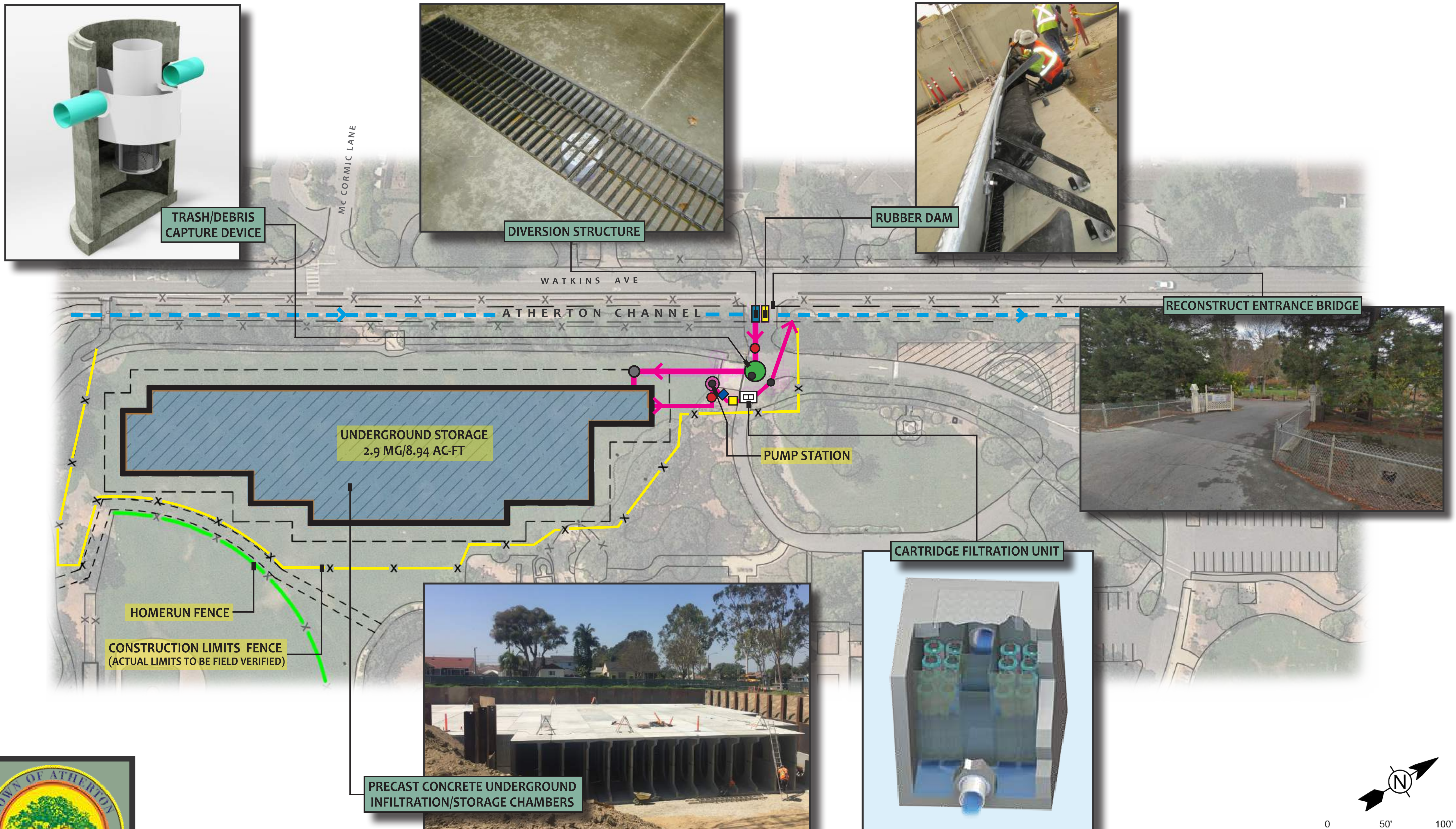
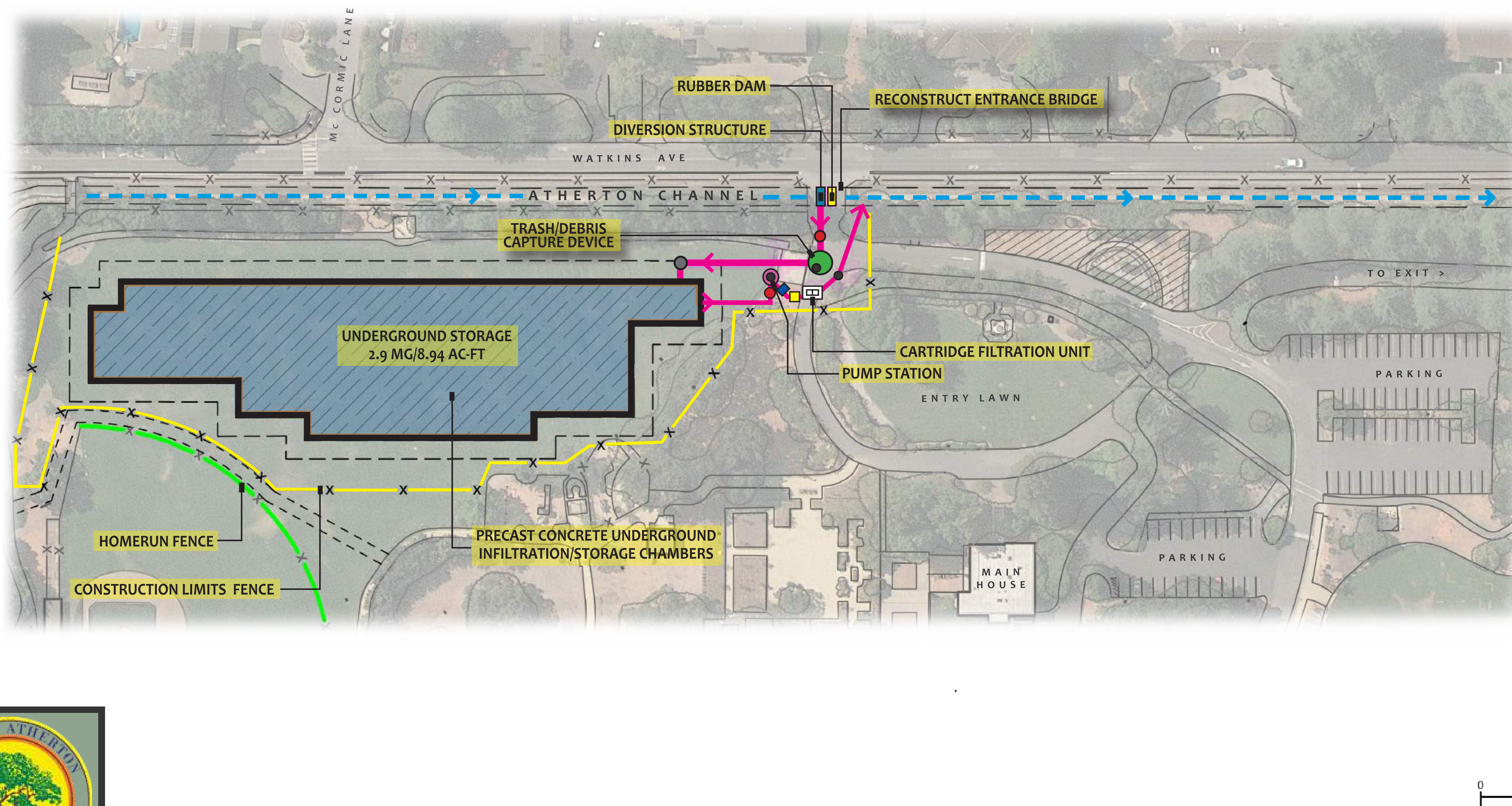
APPENDIX A: DETAILED DRAWINGS AND SITE LAYOUTS



Town of Atherton Water Capture Project - Holbrook-Palmer Park

Conceptual Site Plan Alternative 1

TETRA TECH



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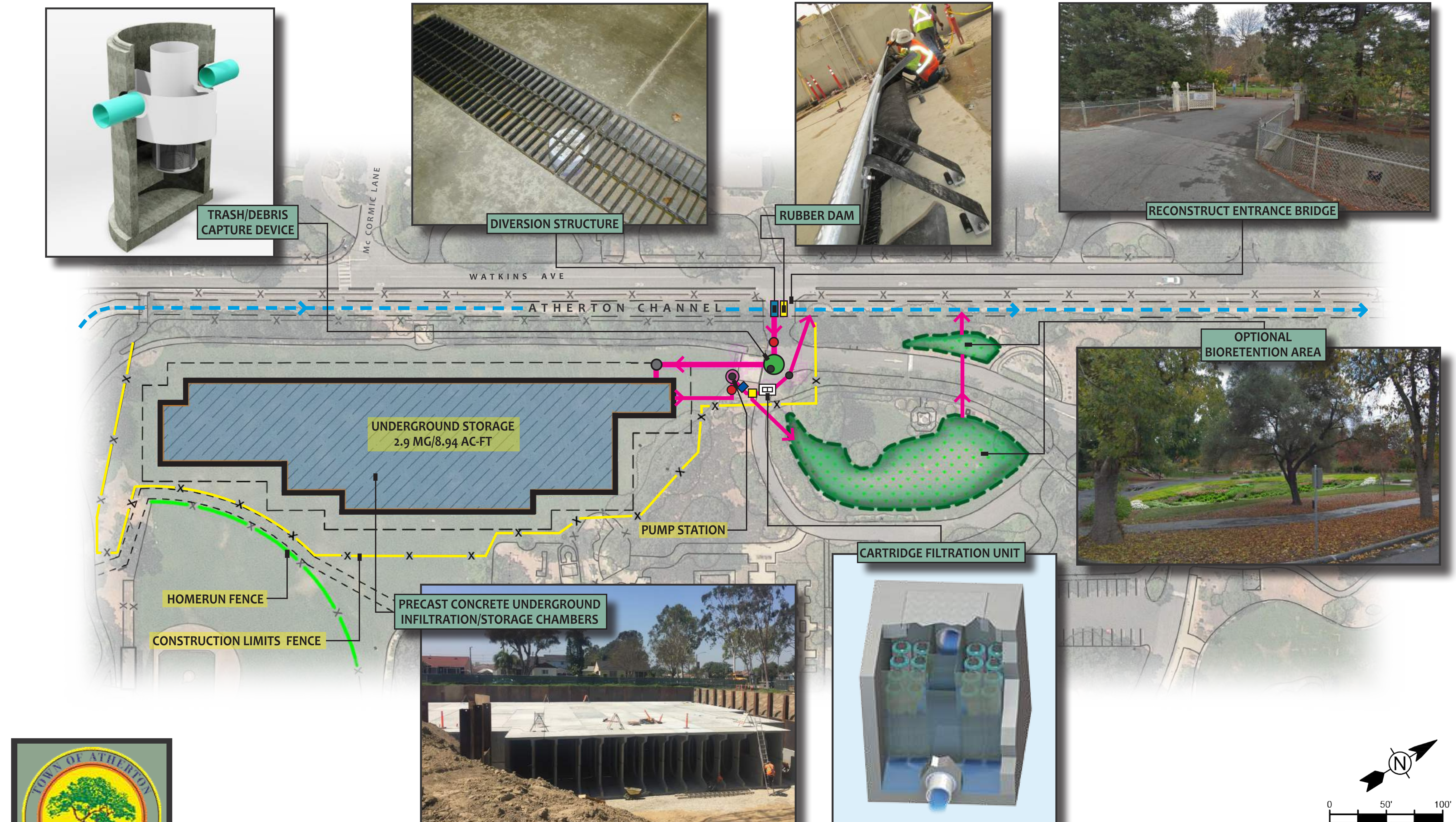
INCORPORATED SEPTEMBER 12, 1923
CALIFORNIA

Town of Atherton Water Capture Project - Holbrook-Palmer Park

Conceptual Site Plan Alternative 1

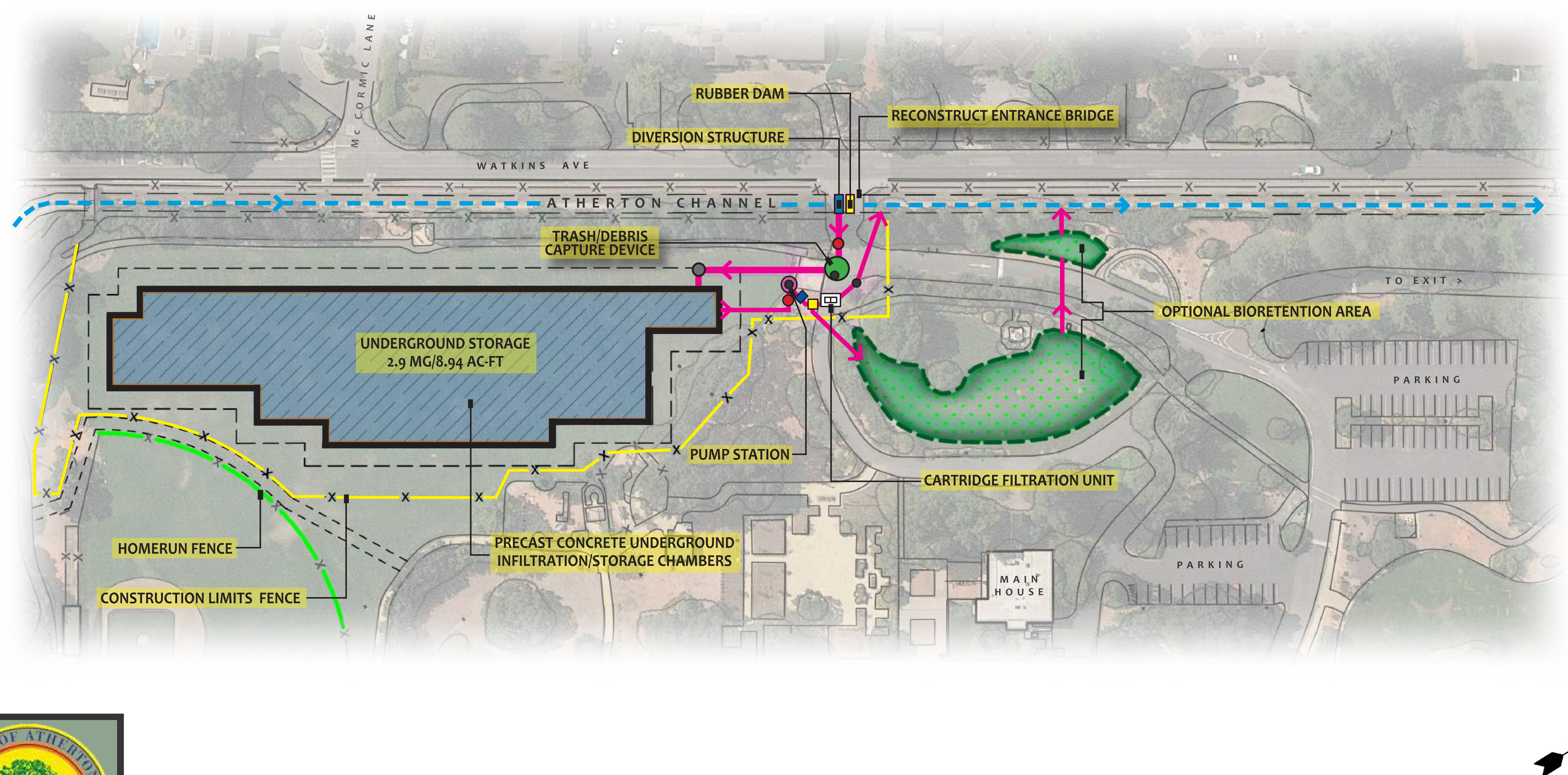
TETRA TECH



Town of Atherton Water Capture Project - Holbrook-Palmer Park

Conceptual Site Plan Alternative 1 (with bioretention)

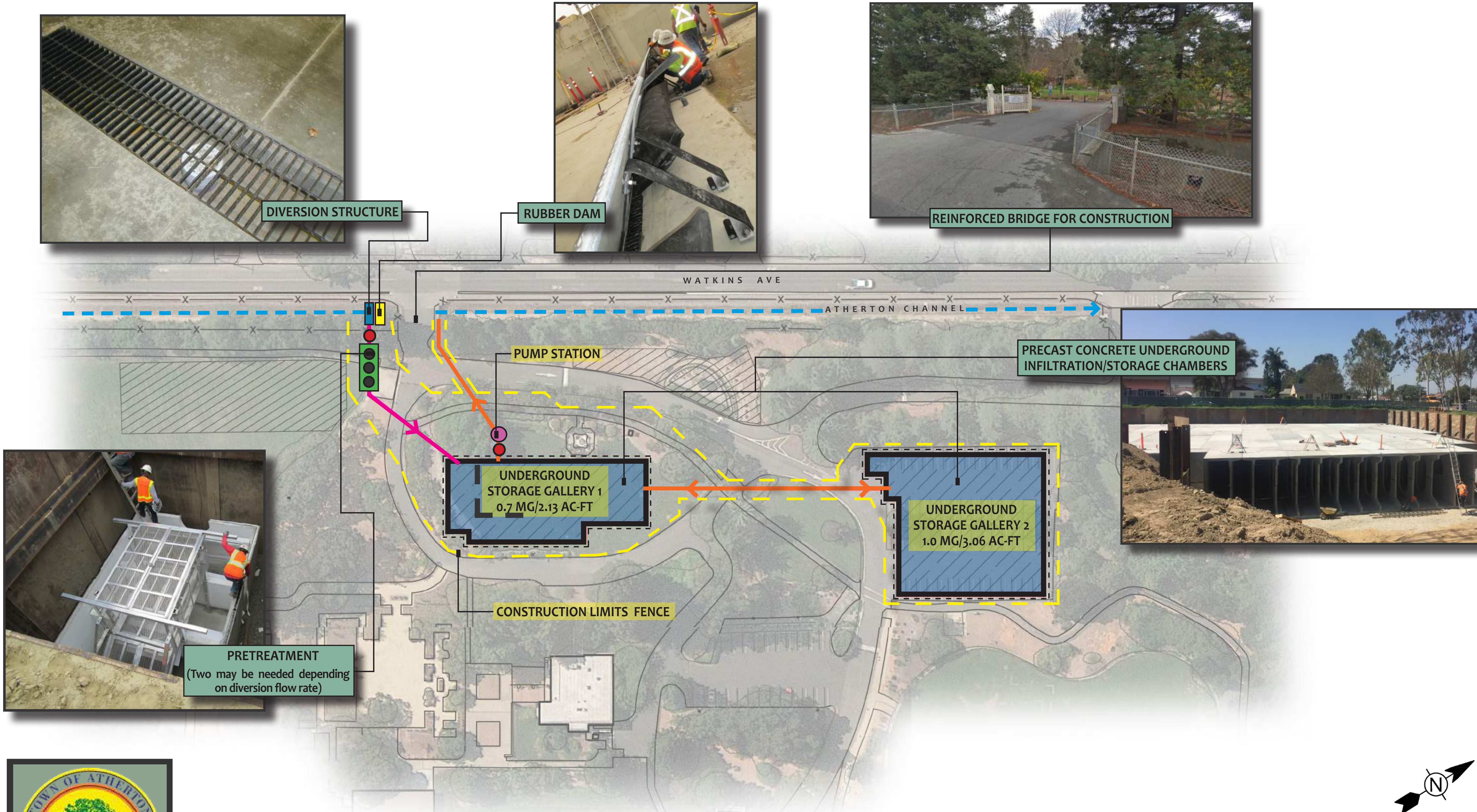
TETRA TECH



Town of Atherton Water Capture Project - Holbrook-Palmer Park

Conceptual Site Plan Alternative 1 (with bioretention)

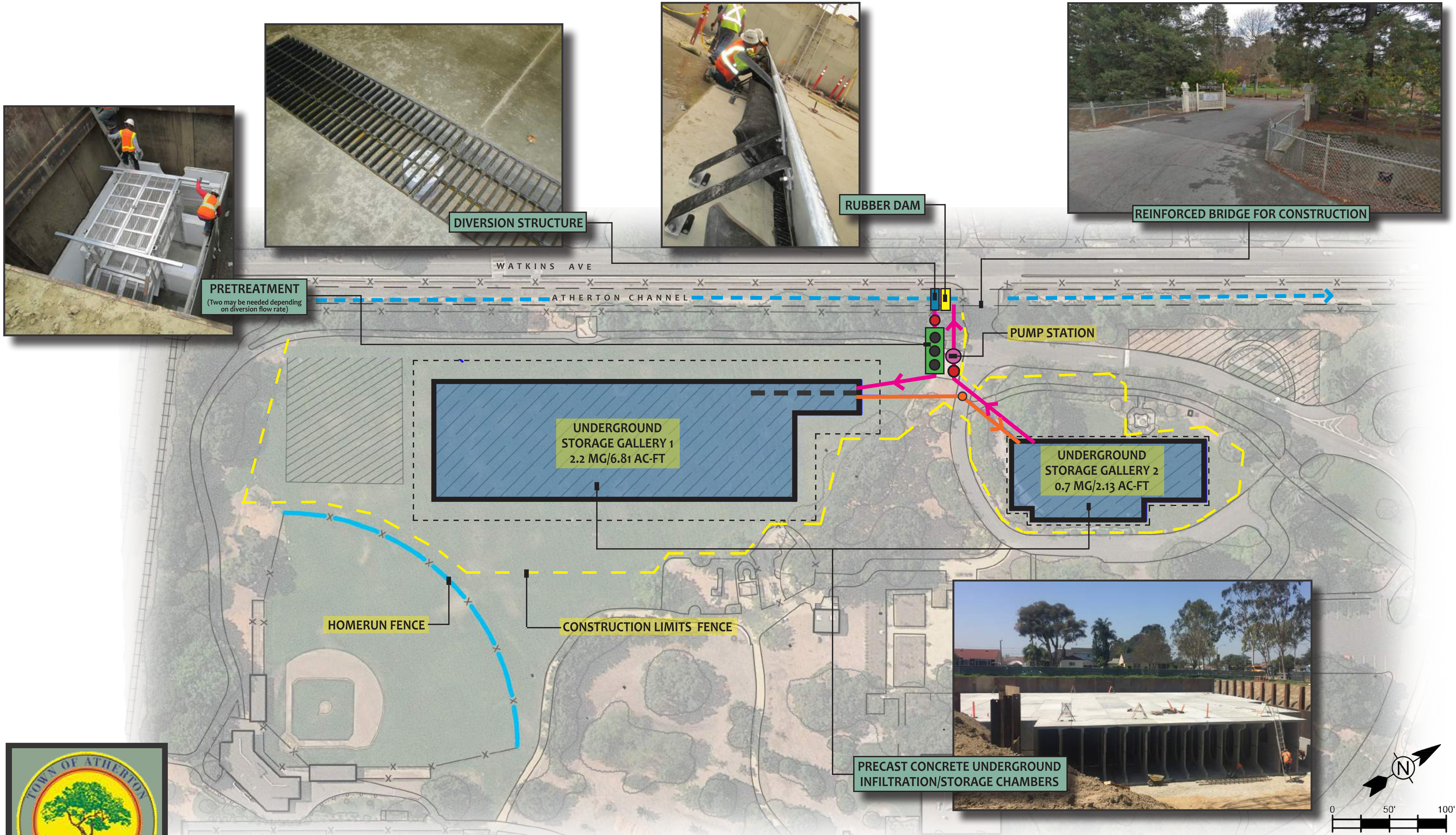
TETRA TECH



* Note: This alternate does not meet the FC goals.

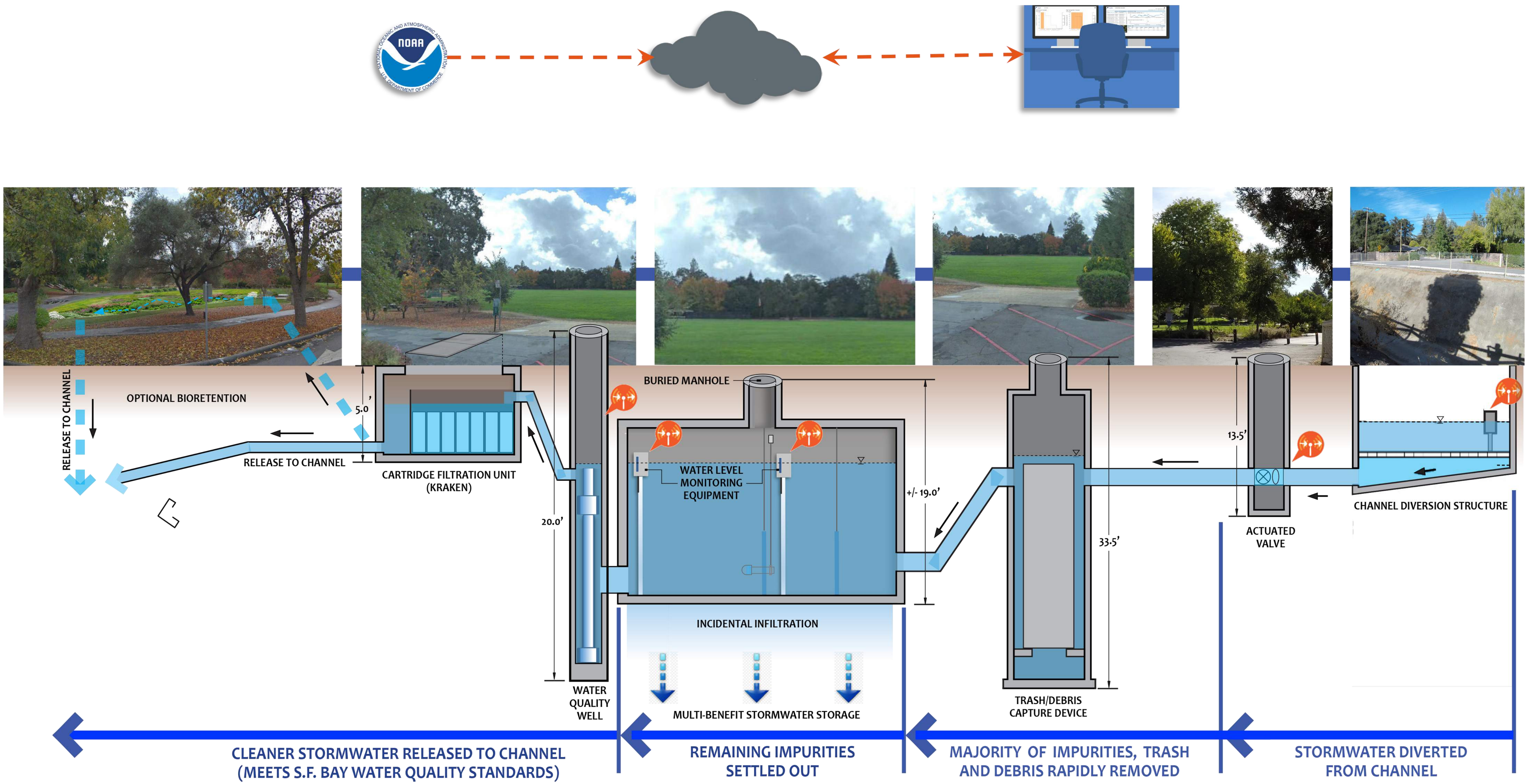
Town of Atherton Water Capture Project - Holbrook-Palmer Park - Alternative 2

TETRA TECH



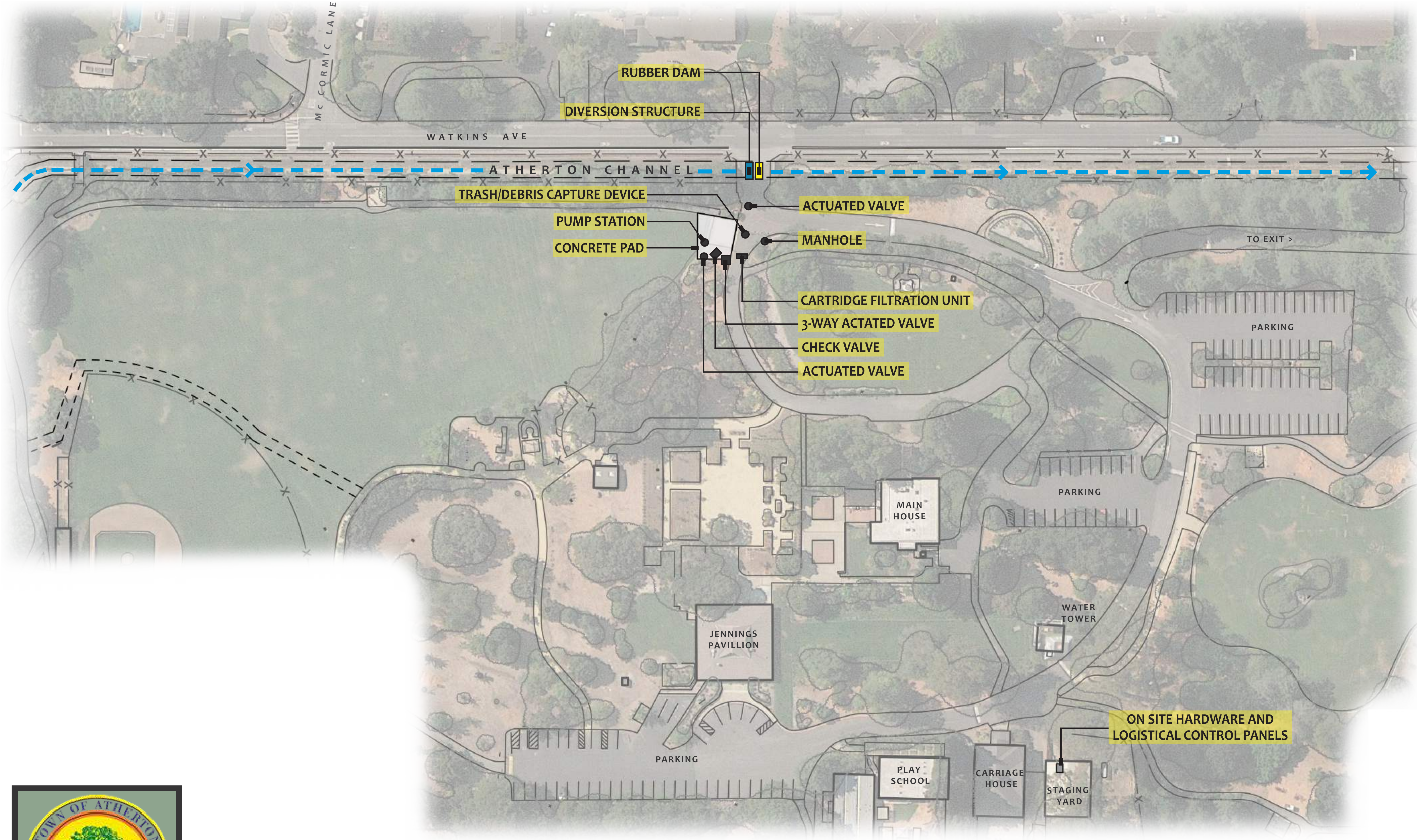
Town of Atherton Water Capture Project - Holbrook-Palmer Park - Alternative 3b

TETRA TECH



Atherton Water Capture Project (Water Flow Process Schematic)

TETRA TECH



Town of Atherton Water Capture Project - Holbrook-Palmer Park

Visual Infrastructure Alternative 1

TETRA TECH

Alternative 1

Diversion Alternative 1

	PRO's	CONS's
i		Soil hauling: 18989 cubic yards ~ 1583 trucks. Largest volume of exported soil accompanies largest volume of BMP benefit.
ii	Transportation of soil for stockpiling during phased construction will be convenient in large open space. Movement limited to one area of park.	
iii	-Scheduling operations involving large equipment, i.e. earthwork, gallery installation, will not interfere with park public access and use of the park because construction is confined to one area of the park. -Complete park closure will not be required.	-The field will be closed for duration of the project.
iv	-Construction/Public access will be separate. -Construction movements will be confined to one area of the park.	-Public entrance and exit will temporarily be the same route (the road is wide enough for 2 lanes).
v	-Vacuum truck will enter at either existing entrance bridge or (if installed) the new bridge near the diversion structure.	-Service truck to treatment system will need access to the upgraded path/walking area along the edge of the field.
vi	-3 trees to be removed	
vii	-Storage capacity is large with the possibility of expansion. 8.94 acre-ft.	
viii	-New bridge-to be locked when not in use -New field (same as existing) -Upgraded irrigation at field -Treatment system building -Traffic rated concrete path from vacuum truck entrance to field -Expanded asphalt path for service truck to treatment building -Removal of concrete pad near existing park entrance	
ix	-If new bridge is installed, no modifications to existing bridges. -If new bridge is not installed, bridges may require modifications/upgrades to accommodate vacuum truck.	
x	-8.94 acre-ft. storm water capture and/or infiltration. It is possible to expand this volume.	

xi	-Underdrain from Atherton Channel and return line from gallery confined to one area. Piping requirements are minimal because the gallery is close to the channel and there are no equalization pipes.	
xii	-Treatment system to be connected to irrigation system. New irrigation system for large portion of the field.	

Alternative 1

Diversion Alternative 2

	PRO's	CONS's
i		Soil hauling: 18989 cubic yards ~ 1583 trucks. Largest volume of exported soil accompanies largest volume of BMP benefit.
ii	Transportation of soil for stockpiling during phased construction will be convenient in large open space. Movement limited to one area of park.	
iii	-Scheduling operations involving large equipment, i.e. earthwork, gallery installation, will not interfere with park public access and use of the park because construction is confined to one area of the park. -Complete park closure will not be required.	- Field will be closed for duration of the project.
iv	-Construction/Public access will be separate. -Construction movements will be confined to one area of the park.	-Public entrance and exit will temporarily be the same route (the road is wide enough for 2 lanes).
v	-Vacuum truck will enter at existing entrance bridge	-Service truck to treatment system will need access to the upgraded path/walking area along the edge of the field.
vi	-2 trees to be removed	
vii	-Storage capacity is large with the possibility of expansion. 8.94 acre-ft.	
viii	-New field (same as existing) -Upgraded irrigation at field -Treatment system building -Traffic rated concrete path from vacuum truck entrance to field -Expanded asphalt path for service truck to treatment building. -Removal of concrete pad near existing park entrance	
ix	-Bridges may require modifications/upgrades to accommodate Vacuum truck pending structural analysis of existing bridge.	

x	-8.94 acre-ft. storm water capture and/or infiltration. It is possible to expand this volume.	
xi	-Underdrain from Atherton Channel and return line from gallery confined to one area. Piping requirements are relatively small because the gallery is close to the channel and there are no equalization pipes.	
xii	-Treatment system to be connected to irrigation system. New irrigation system for large portion of the field.	

Alternative 2

	PRO's	CONS's
i	Soil hauling: 11764 cubic yards ~ 981 trucks. Smallest volume of exported soil accompanies smallest volume of BMP benefit.	
ii		Transportation of soil for stockpiling during phased construction will require use of asphalt roads through park.
iii	-Baseball and soccer fields will not require closure during construction	-Scheduling operations involving large equipment, i.e. earthwork, gallery installation, will interfere with park public access to play school, southerly parking lot and field. -Northerly parking lot will be closed during construction.
iv	-Construction/Public access will be separate. -Park events at the baseball field and soccer will not be interfered with construction.	-Public entrance and exit will temporarily be the same route (the road is wide enough for 2 lanes). -Movements will require traffic control during park hours. -Access to park parking will be limited
v	-Vacuum truck will enter at existing entrance bridge. -Service truck for treatment system can use existing roads.	
vi		-7 trees to be removed
vii		-Storage capacity is small with no possibility of expansion. 5.18 acre-ft.
viii	-Upgraded irrigation at entry lawn. -Treatment system building -Removal of concrete pad near existing park entrance. -New parking lot.	-Irrigation will not be upgraded at field because it will not be damaged during construction.
ix	-Bridge may require modifications/upgrades to accommodate Vacuum truck pending structural analysis of existing bridge.	

x		-5.18 acre-ft. storm water capture and/or infiltration. It is not possible to expand this volume.
xi		-Underdrain from Atherton Channel to gallery, and return line from gallery to channel, and equalization pipes cross access roads. Scheduling will be required. -Piping requirements are relatively large because the gallery is farther from the channel and there are equalization pipes.
xii	-Treatment system to be connected to irrigation system. New irrigation system for entry lawn portion of the field.	

Alternative 3

Diversion Alternative 1

	PRO's	CONS's
i		Soil hauling: 18989 cubic yards ~ 1583 trucks. Largest volume of exported soil accompanies largest volume of BMP benefit.
ii	Transportation of soil for stockpiling during phased construction will be convenient in large open space. Movement limited to two areas of park.	-Closure of road around the entry lawn will eliminate traffic coordination with public.
iii	-Scheduling operations involving large equipment, i.e. earthwork, gallery installation, will not interfere with park public access and use of the park because construction is confined to one area of the park. -Complete park closure will not be required.	-The field will be closed for duration of the project. -Closure of road around the entry lawn.
iv	-Construction/Public access will be separate. -Construction movements will be confined to one area of the park.	-Public entrance and exit will temporarily be the same route (the road is wide enough for 2 lanes).
v	-Vacuum truck will enter at either existing entrance bridge or (if installed) the new bridge near the diversion structure.	-Service truck to treatment system will need access to the upgraded path/walking area along the edge of the field.
vi	-5 trees to be removed	
vii	-Storage capacity is large with the possibility of expansion. 8.94 acre-ft.	
viii	-New bridge-to be locked when not in use -New field (same as existing) -Upgraded irrigation at field and entry lawn -Treatment system building -Traffic rated concrete path from vacuum truck entrance to field -Expanded asphalt path for service truck to treatment building -Removal of concrete pad near existing park entrance	
ix	-If new bridge is installed, no modifications to existing bridges.	

	-If new bridge is not installed, bridges may require modifications/upgrades to accommodate Vacuum truck.	
x	-8.94 acre-ft. storm water capture and/or infiltration. It is possible to expand this volume.	
xi	-Underdrain from Atherton Channel and return line from gallery confined to one area.	-Piping requirements expanded because of equalization pipe.
xii	-Treatment system to be connected to irrigation system. New irrigation system for large portion of the field and the entry lawn.	

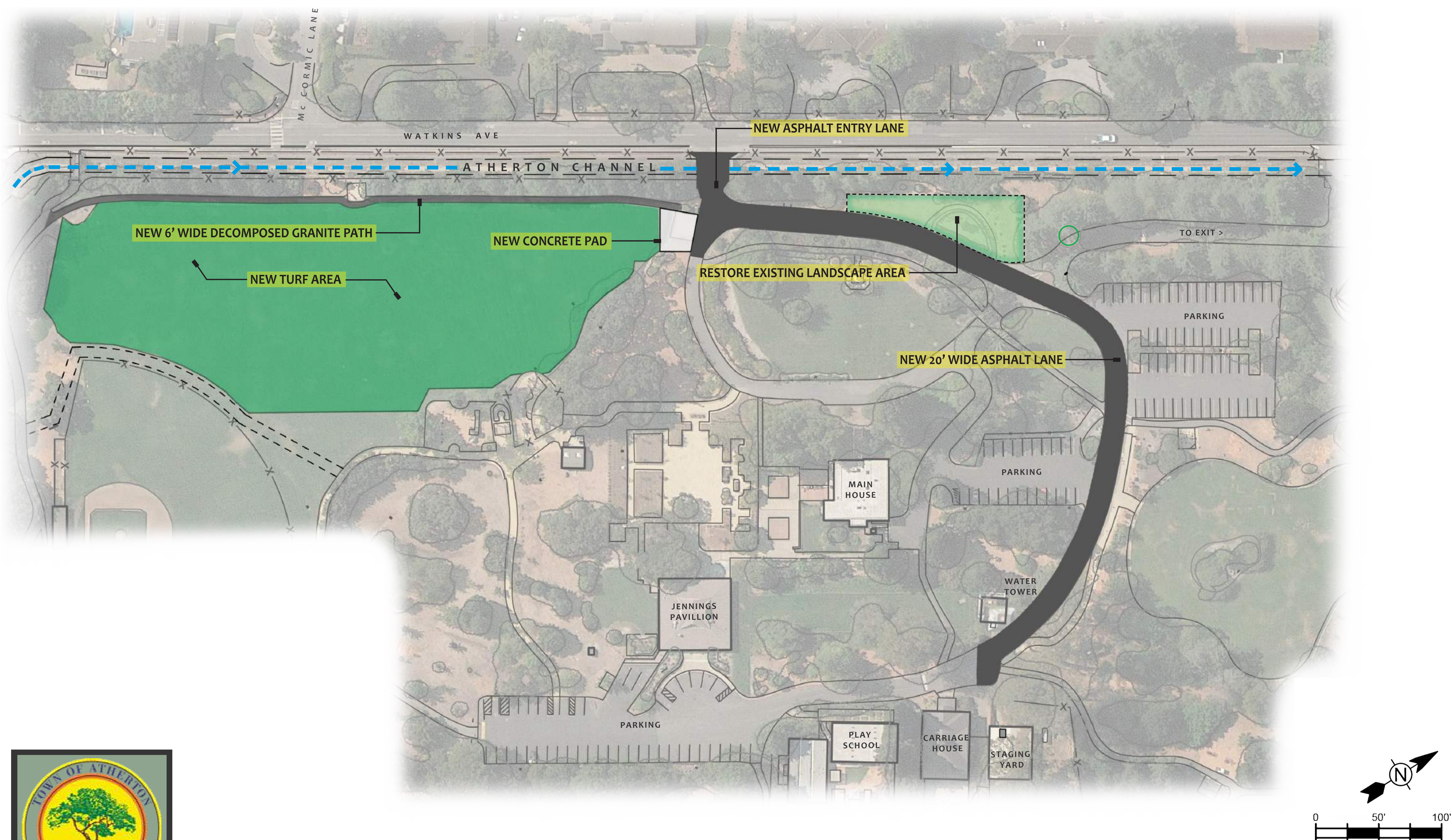
Alternative 3

Diversion Alternative 2

	PRO's	CONS's
i		Soil hauling: 18989 cubic yards ~ 1583 trucks. Largest volume of exported soil accompanies largest volume of BMP benefit.
ii	Transportation of soil for stockpiling during phased construction will be convenient in large open space. Movement limited to two areas of park.	-Closure of road around the entry lawn will eliminate traffic coordination with public.
iii	-Scheduling operations involving large equipment, i.e. earthwork, gallery installation, will not interfere with park public access and use of the park because construction is confined to one area of the park. -Complete park closure will not be required.	-The field will be closed for duration of the project. -Closure of road around the entry lawn.
iv	-Construction/Public access will be separate. -Construction movements will be confined to one area of the park.	-Public entrance and exit will temporarily be the same route (the road is wide enough for 2 lanes).
v	-Vacuum truck will enter at existing entrance bridge.	-Service truck to treatment system will need access to the upgraded path/walking area along the edge of the field.
vi	-4 trees to be removed	
vii	-Storage capacity is large with the possibility of expansion. 8.94 acre-ft.	
viii	-New field (same as existing) -Upgraded irrigation at field and entry lawn -Treatment system building -Expanded asphalt path for service truck to treatment building -Removal of concrete pad near existing park entrance	
ix	Bridges may require modifications/upgrades to accommodate Vacuum truck.	
x	-8.94 acre-ft. storm water capture and/or infiltration. It is possible to expand this volume.	
xi	-Underdrain from Atherton Channel and return line from gallery confined to one area.	-Piping requirements expanded because of equalization pipe.

xii	-Treatment system to be connected to irrigation system. New irrigation system for large portion of the field and the entry lawn.	

APPENDIX B: LANDSCAPE PLAN



Town of Atherton Water Capture Project - Holbrook-Palmer Park

Landscape Plan and New Hardscape

TETRA TECH

APPENDIX C: EXISTING UTILITY PLANS

ENGINEERING DEPARTMENT

GIS

BEAR GULCH DISTRICT

WATER SYSTEM

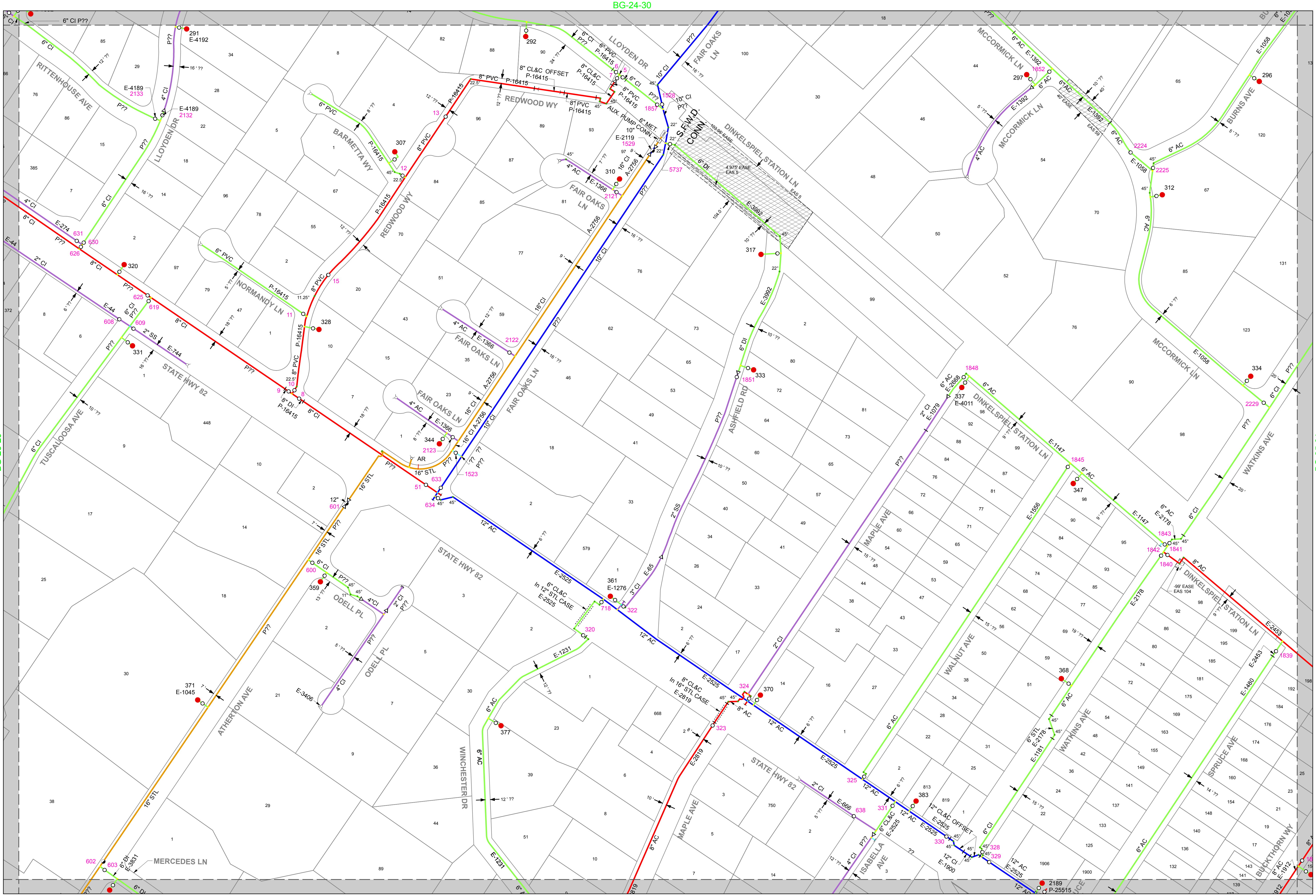
CONFIDENTIAL: Applicant hereby agrees that any plans or markings made by California Water Service (Cal Water) showing the estimated location of its underground facilities is done solely as an accommodation and without any warranties, representations, or guaranties of completeness or accuracy. Applicant acknowledges that said information is a suggestion as to possible locations, as would be necessary to protect Cal Water's property. Applicant accepts full responsibility for any damage to Cal Water's facilities. Applicant agrees that Cal Water is not liable for any direct or indirect damages arising out of the use of said information.

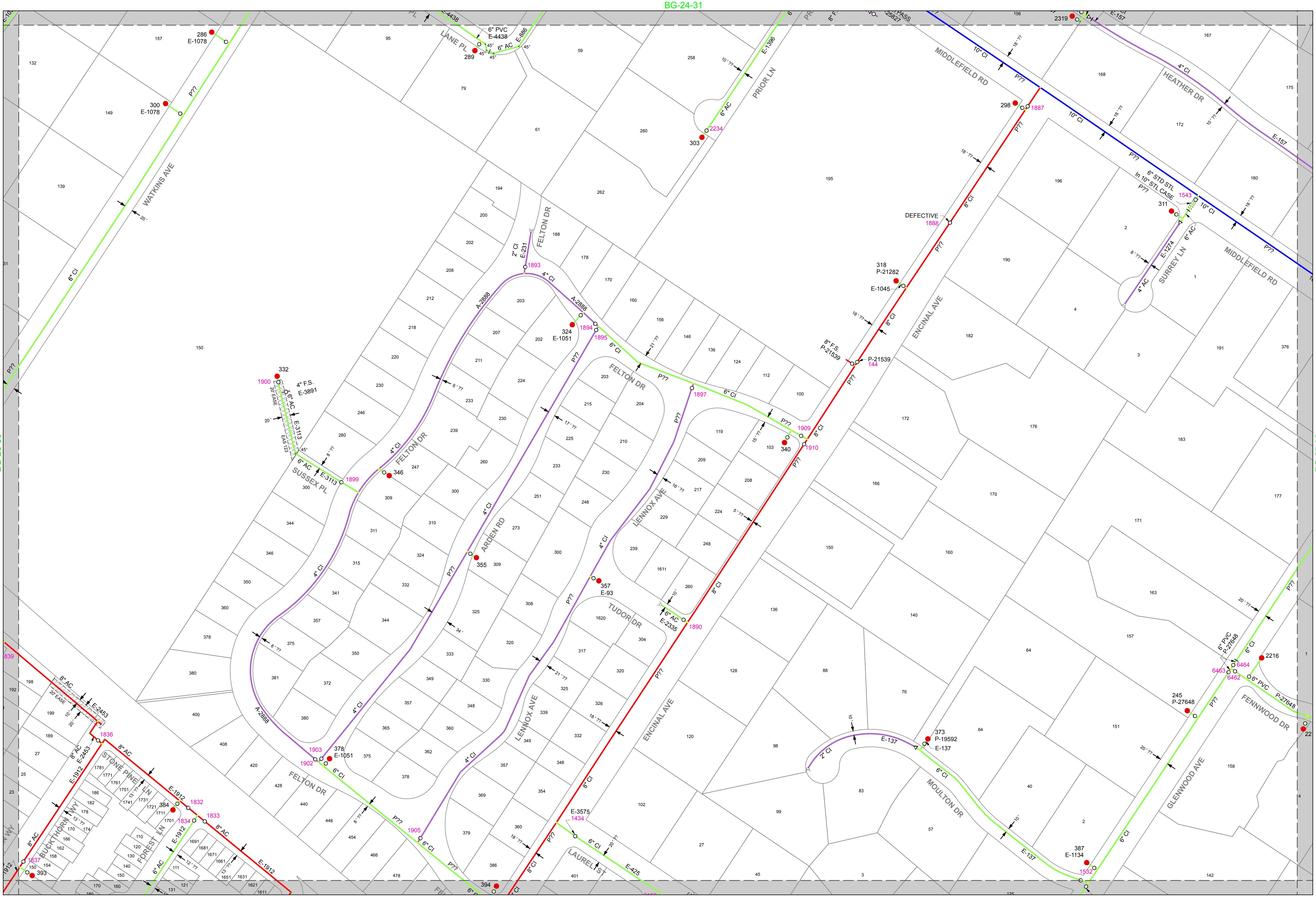
N

SCALE:
1" = 200'

Issued:
June 2015

Plat Sheet:
BG-25-30





ENGINEERING DEPARTMENT

GIS

BEAR GULCH DISTRICT
WATER SYSTEM

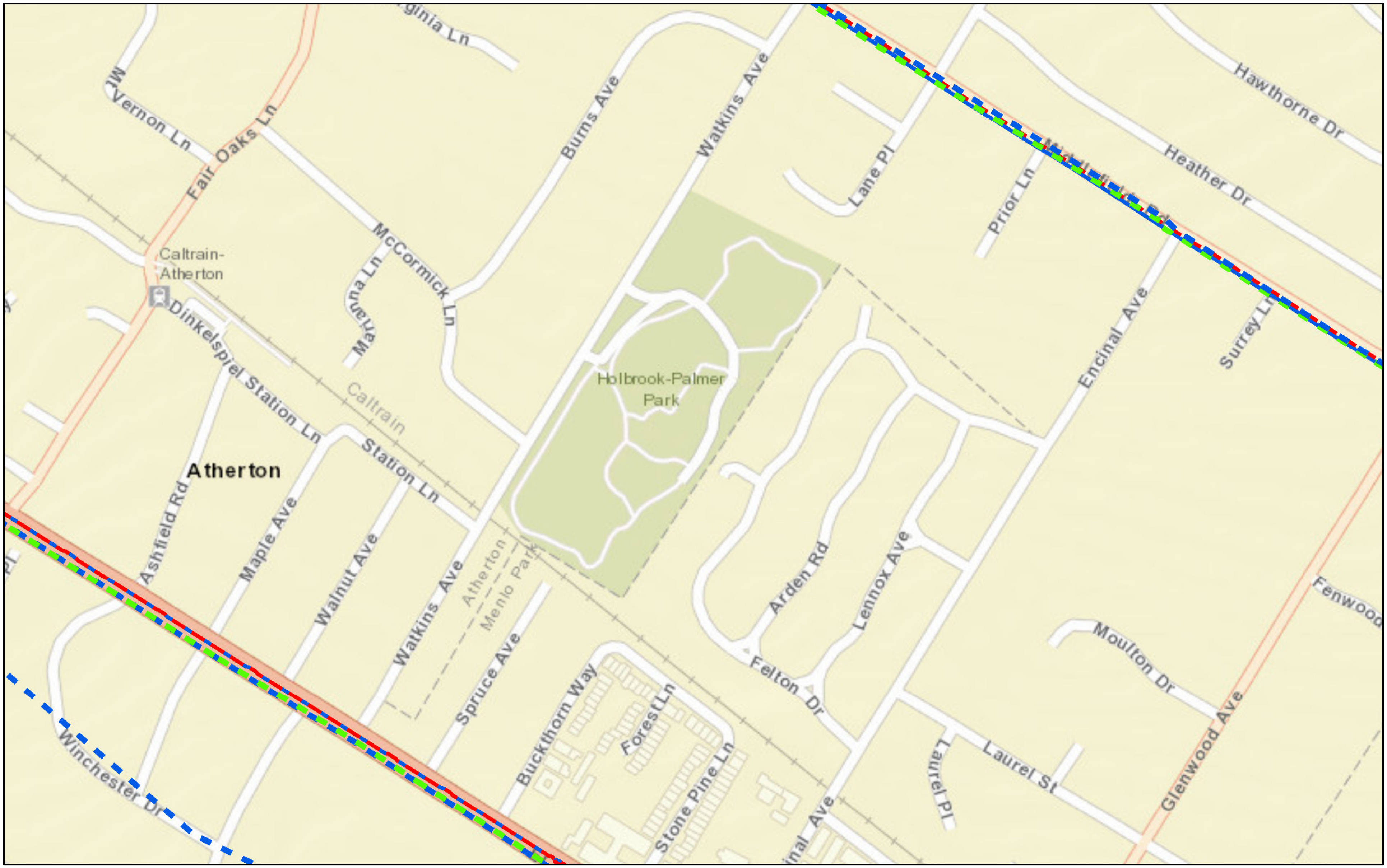
N

SCALE:
1" = 200'

Issued:
June 2015

Plat Sheet:
BG-25-31

CenturyLink and Level 3 Network



January 19, 2018

1:7,899

0 0.05 0.1 0.2 mi

0 0.1 0.2 0.4 km

All CTL National Routes

- Owned, Aerial
- Owned, Underground
- Leased, Aerial
- Leased, Underground

Level 3 Facilities

- Aerial
- Underground

Non Level 3 Facilities

- Aerial
- Underground

Level 3 Data is Highly Confidential and Proprietary.
Sources: Esri, HERE, DeLorme, USGS, Intermap, INCREMENT P, NRCan,

TETRA TECH

UTILITY INFORMATION REQUEST

FROM:	Tetra Tech, Inc. 711 Tank Farm Road, Suite 110 San Luis Obispo, CA 93401	**DATE:**	January 10, 2018
		JOB NO.:	
TO:	City of Menlo Park 701 Laurel St. Menlo Park, CA 94025	**PROJECT NAME:**	Holbrook-Palmer Park Water Capture Project
		CERTIFIED NO:	
ATTENTION:	Ivan Toews (650) 330-6712		

DESCRIPTION OF WORK: Tetra Tech, Inc. has been contracted by the Town of Atherton to prepare design plans for a southwestern portion of the existing Holbrook-Palmer Park located in the Town of Atherton. The proposed construction area is east of Watkins Avenue, between Middlefield Road and El Camino Real. This is a request for utility information in the form of prints, copies, etc. for the location given. Please check the appropriate response and forward the utility information and the original of this request to the above address.

☐ We have existing facilities in the location in question. All information has been forwarded.

☐ We will have facilities in the location in question in the future. All preliminary design or planning information has been forwarded.

☑ We have no facilities in the location in question.

☐ Other (please specify) ______________________

If you have any questions regarding this request, please contact Elva Pangilinan at (805) 542-9052 or at elva.pangilinan@tetratech.com.

Thank you for your assistance.

Elva Pangilinan

EP/am
Attachments

To be filled out by Utility Company

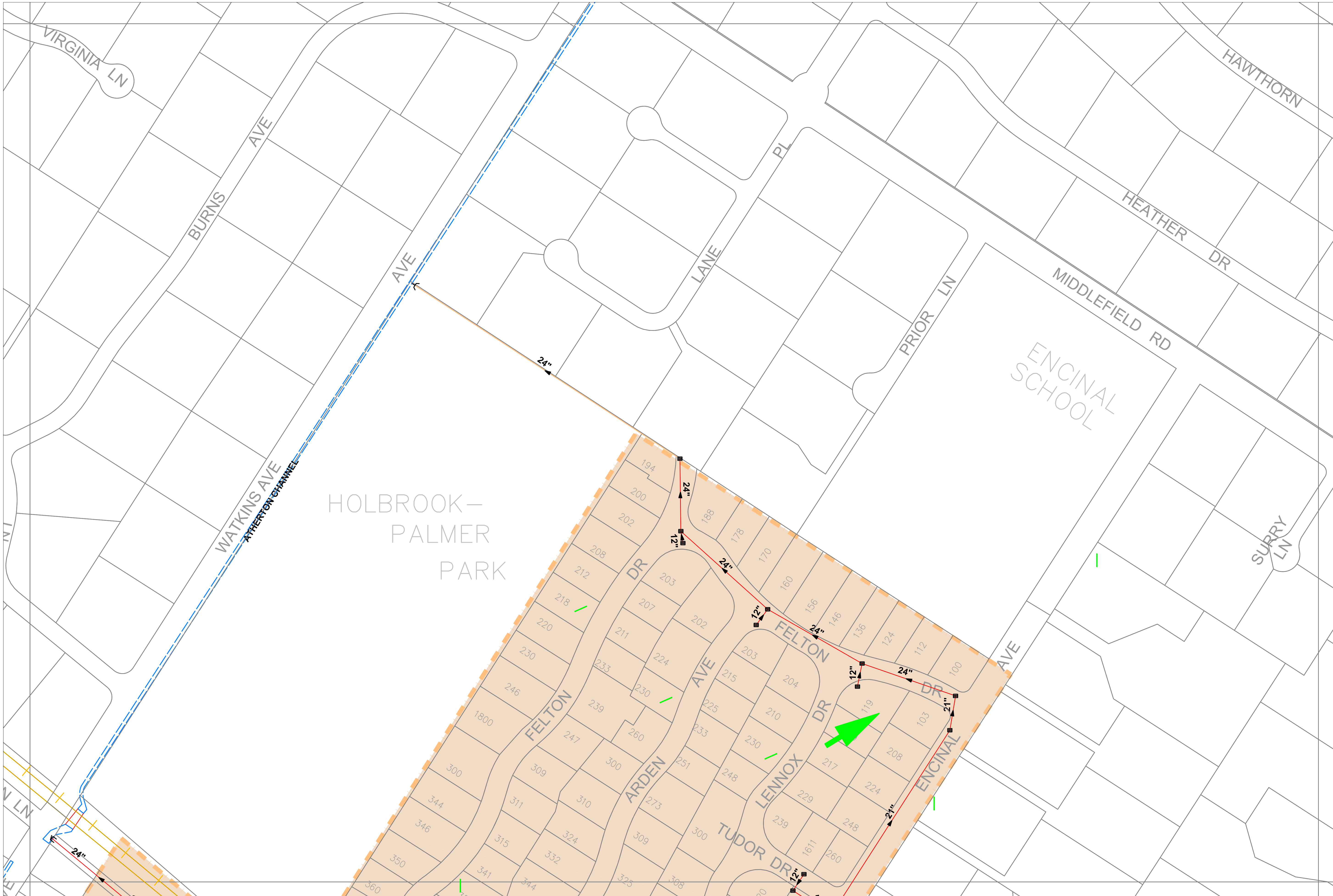
Utility Company Representative Signature

Ivan Toews
Typed Name of Representative

Date: 1/10/18

Phone: 650-330-6712

E-Mail: ijtoews@menlopark.org



Prepared by LYNX TECHNOLOGIES Nov 5, 2009

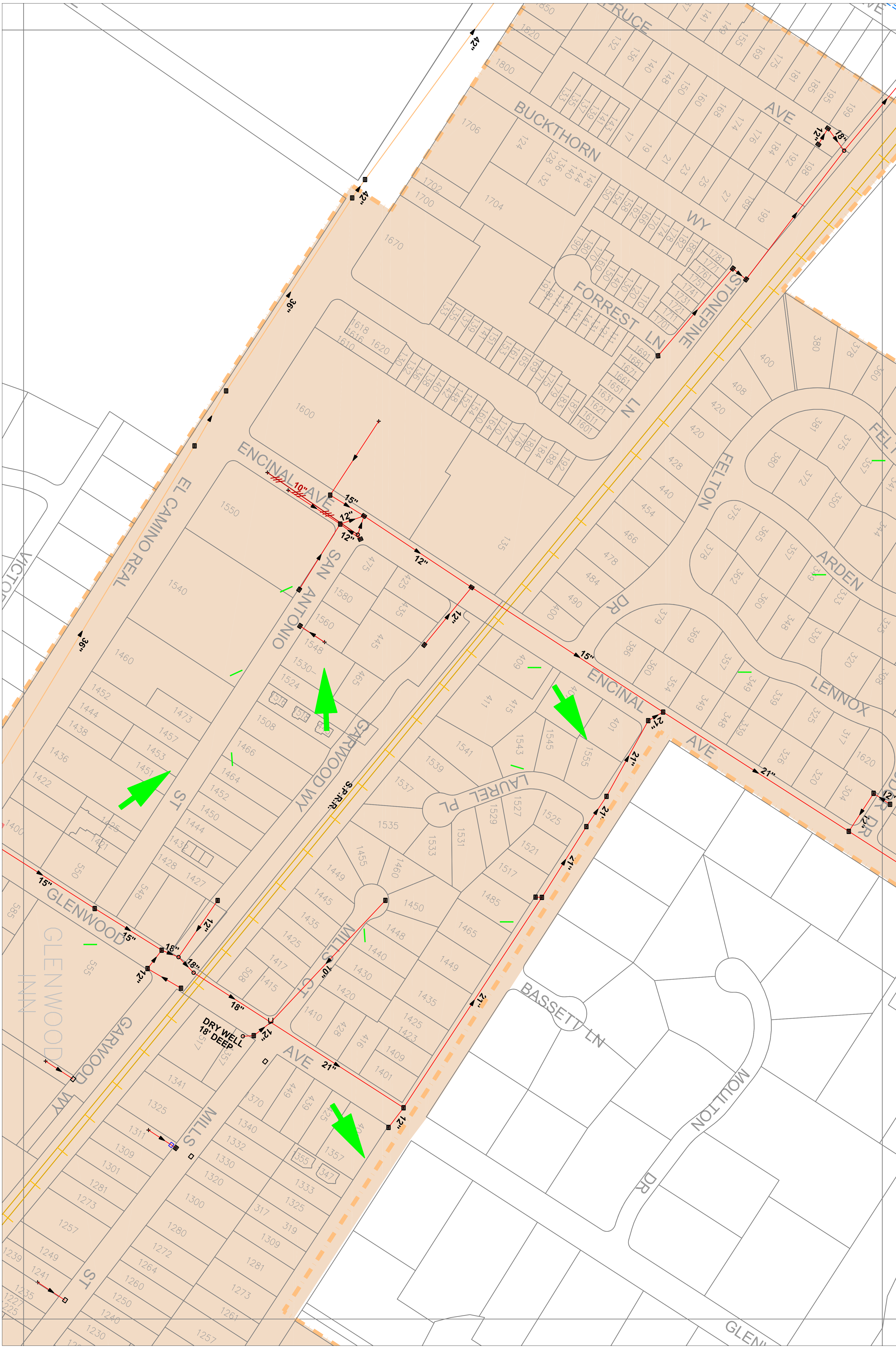
CITY OF MENLO PARK
SEWER/STORM DRAIN SYSTEM FACILITIES

GRAPHIC SCALE FEET 0 100 200 300

	D9	
C10	D10	E10

D9

THE CITY OF MENLO PARK ASSUMES NO LIABILITY FOR ERRORS OR OMISSIONS FOR UTILITY INFORMATION SHOWN ON THESE MAPS. ALL INFORMATION SHOULD BE FIELD VERIFIED.



D10

C10	D9
D11	D10
E11	E10

GRAPHIC SCALE 0 100 200 300 FEET

CITY OF MENLO PARK
SEWER\STORM DRAIN SYSTEM FACILITIES

CITY OF MENLO PARK

Prepared by LYNX TECHNOLOGIES Oct 20, 2009 at&t

January 22, 2018

TERA TECH, INC.
Attn: **AUSTIN McCOLLUM**

Subject: **WATKINS AVE, ATHERTON**
(THERE ARE NO OTHER UNDERGROUND FACILITIES OTHER THAN WHAT IS SHOWN ON MAPS)

Dear Austin,

AT&T has reviewed the plan map for the project mentioned above. We have determined that we **do have** existing underground facilities within the project limits as shown on the attached drawings.

It is your responsibility to review the attached drawings to determine whether or not our facilities conflict with your project. If you determine that a conflict exists, please notify AT&T in writing of the need to relocate its facilities well in advance of the commencement of the Project. If manhole or box adjustments are required, you must fax a letter of request 30 days in advance. My fax number is 408-945-1247.

The drawings indicate the approximate location of our existing facilities in the field. Please contact **UNDERGROUND SERVICE ALERT** on **1-800-227-2600** prior to any excavation work in these areas.

If you have further questions or concerns regarding this information, please call AT&T's Right of Way Manager, Bruno Czech at 408-635-8881.

Sincerely,
Kyeisha Warrick-Grant
Engineer Admin
408-635-8767
KW1512@att.com

Ref: # 33

Regarding as-builds, please be advised that all maps supplied are for reference purpose only. Maps only indicate approximate conduit routes and box/manhole locations. The maps **do not** indicate conduit size, type, overall structure size or depths of structure and **do not** guarantee exact location of underground facilities.

AT&T facilities may or may not be in conflict with your project. In order to determine this, AT&T facilities must be positive potholed for exact location and depths. This function is the responsibility of the requestor.

If a conflict is discovered, immediate coordination with AT&T must be made to determine what course of action can be taken to resolve said conflict. Please be advised that some resolutions may take several months to resolve. Furthermore, a conflict found during construction which should have been determined during the design stage could severely delay your project and the requestor of said records may be responsible for all cost incurred to resolve conflict.

AT&T does not maintain nor supply records for aerial facilities, facilities on private property and/or individual service conduits.

Prior to any construction activity, please call U.S.A. (Underground Service Alert) on 811, two working days in advance.

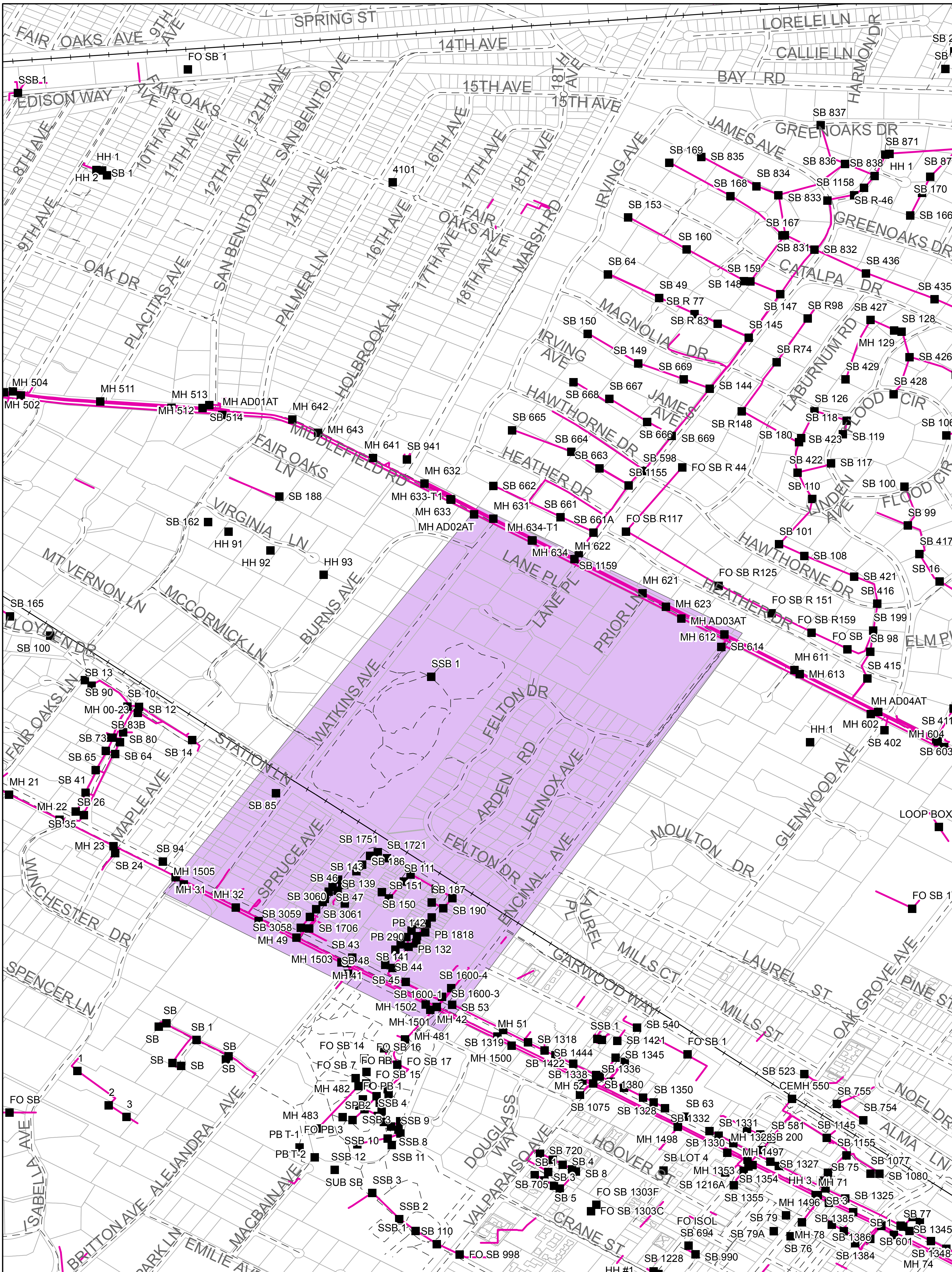
Maps/information supplied is Proprietary Information and is not for use or disclosure outside your firm and/or AT&T/SBC/Pacific Bell/Nevada Bell except under written agreement.

External Map Request for Tetra Tech

Date: 1/22/2018

Page Name: A469

Watkins Ave Project



1:8,411

N

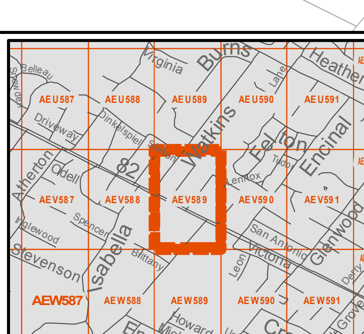
- Manholes
- Conduit
- Railroads
- Street Centerlines
- Parcels
- JobAOI

External Map Request for Tetra Tech

Watkins Ave Project

Date: 1/22/2018

Page Name: AEV589



1:1,830

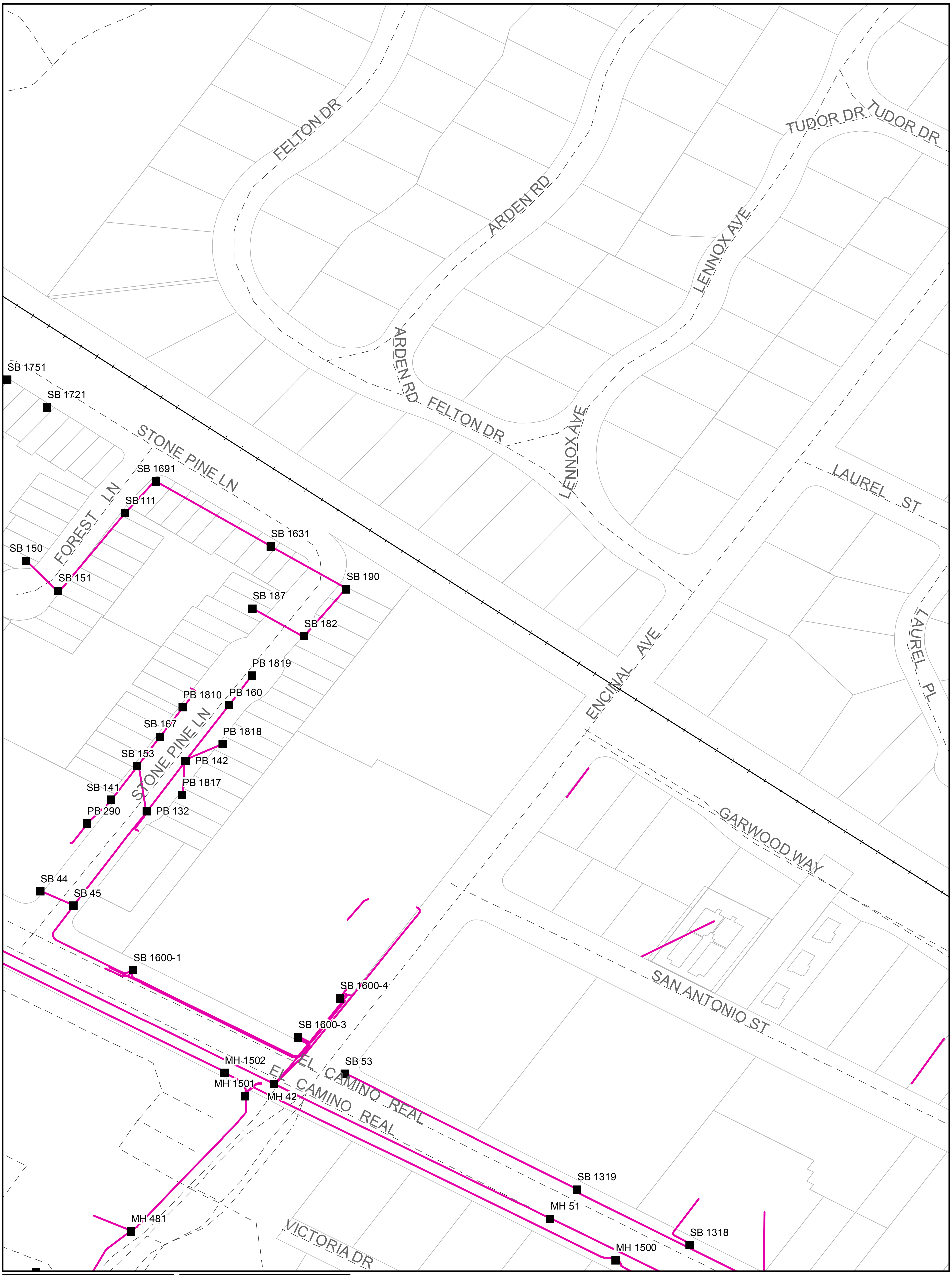
N

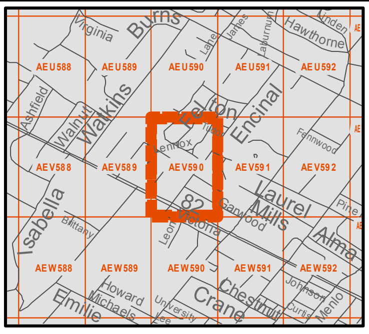
- Manholes
- Conduit
- Railroads
- Street Centerlines
- Parcels

External Map Request for Tetra Tech

Watkins Ave Project

Date: 1/22/2018
Page Name: AEV590





1:1,830

N

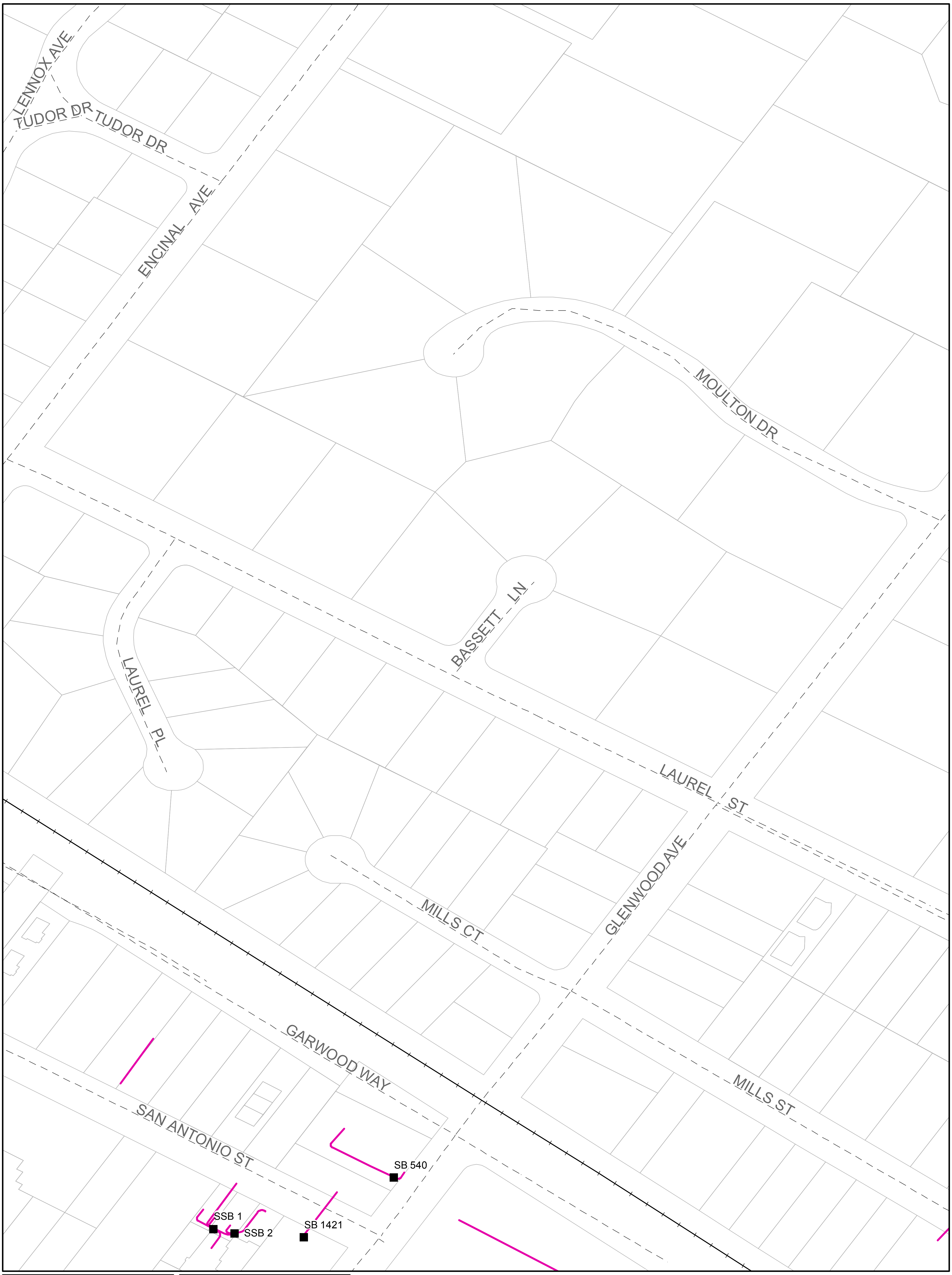
- Manholes
- Conduit
- Railroads
- Street Centerlines
- Parcels

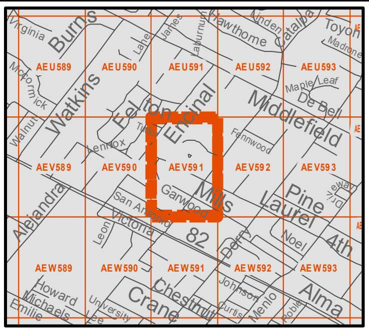
External Map Request for Tetra Tech

Date: 1/22/2018

Page Name: AEV591

Watkins Ave Project





1:1,830

N

- Manholes
- Conduit
- Railroads
- Street Centerlines
- Parcels

Redwood City GIS | Fair Oaks Ln & El Camin...

Not secure | igis.redwoodcity.org/gis/

Redwood City Internal GIS v. 4

El Camino Real AND Oakwood Dr

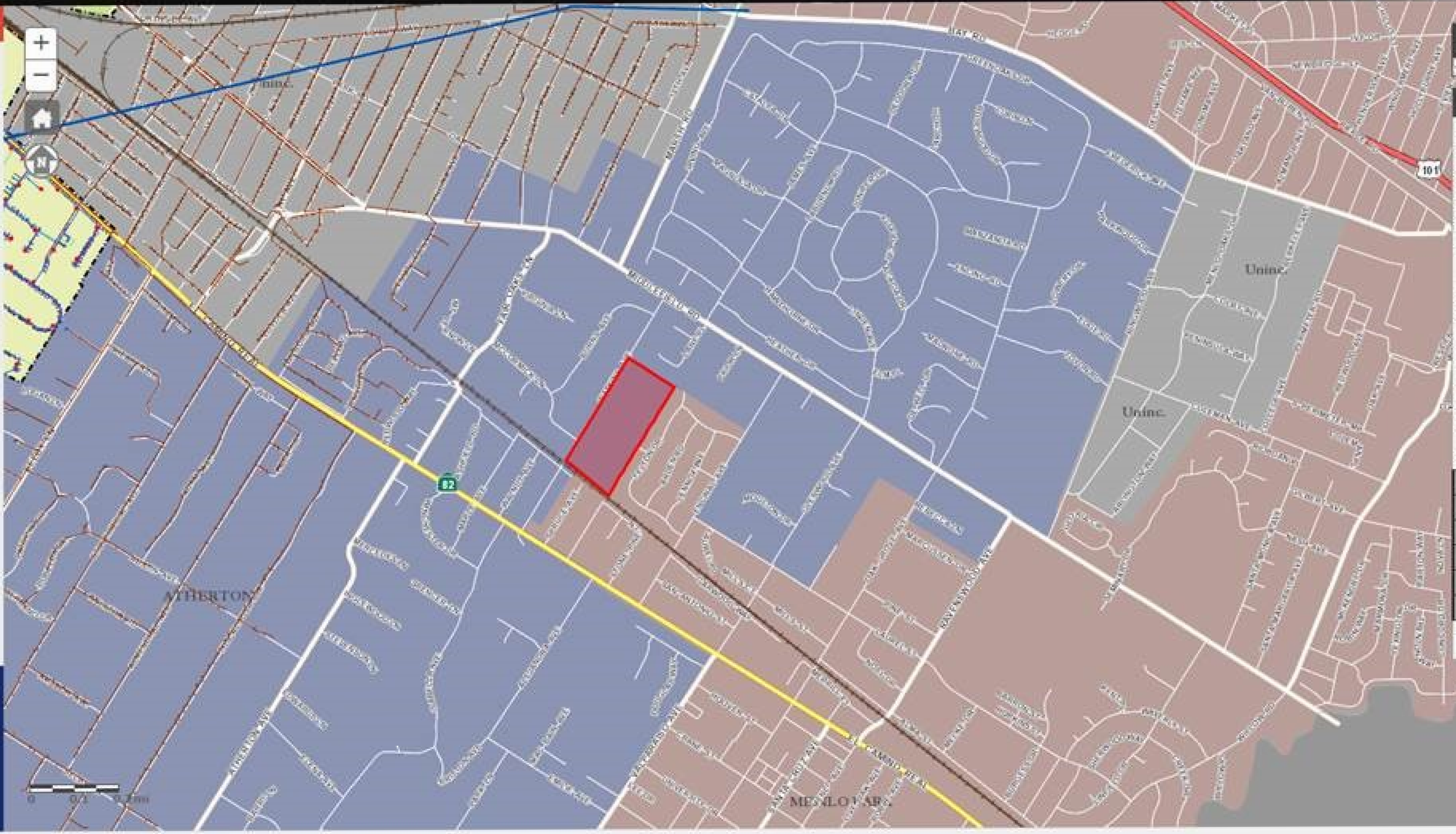
Layers/Legend

- General
  - Addresses
  - Road Labels
  - Parcels
  - Buildings
  - Zip Codes
  - Benchmarks
- Transportation
- Land Use
- City Projects
- Neighborhoods
- Schools & Childcare
- Public Service Facilities
- Natural Resources
- Emergency Services
- Hazards
- Historic
- 2010 Census
- Utilities
  - Water
  - Sewer
  - Storm
  - Recycled Water
  - Boundaries
- Other

Reset Layers to Default

Please send us Feedback!

Credits



1 Parcel Found

150 WATKINS AVE

APN: 061-310-100

Primary Address:
150 WATKINS AVE, 94027

Owner:
TOWN OF ATHERTON
91 ASHFIELD RD
ATHERTON, CA

Zoning	Tax Code	City	Year Built
N/A	1010	ATH	N/A

Lot Size *(Assessor)*	N/A
Parcel Area *(Estimated by GIS)*	948,007 sqft
Average Slope *(Estimated by GIS)*	N/A
Block Book:	N/A

- Tasks
- Land Use
- Emergency Services
- Hazards
- Building
- Other

Google Map | Street View | Bing Maps | County Info

Tony Robbins 3-15....pdf | I10 Buckeye Middl....pdf | H10 Buckeye Midd....pdf | Show all

Primary Phone · Standard · Online · Windows · Type a name or number

There are no active calls at this time

10:55 AM
1/16/2018

San Francisco
Water Power Sewer
Operator of the Hetch Hetchy Regional Water System

Water Supply and Treatment Division
1000 El Camino Real, Millbrae CA 94030
T (650)872-5900
F (650)872-5984

January 19, 2018

Ms. Elva Pangilinan
Tetra Tech, Inc.
711 Tank Farm Road, Suite 110
San Luis Obispo, CA 93401

Subject: Utility Information Request for 36" Palo Alto Pipeline
In Atherton

Dear Ms. Pangilinan,

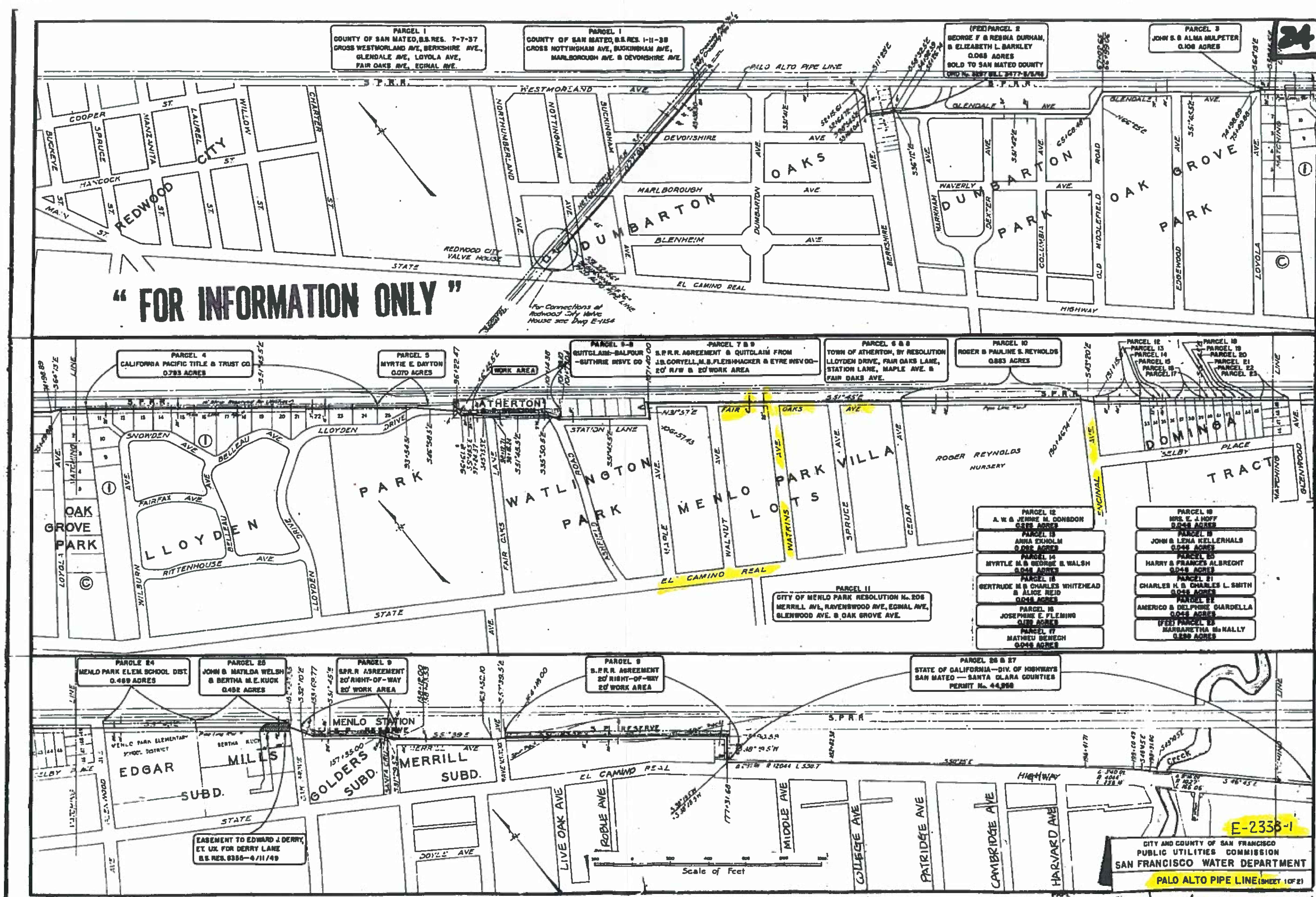
In response to your letter dated January 10, 2018 requesting for utility information within the proposed project area, I am attaching a copy of drawing E-2338-1 showing the location of SFPUC's Palo Alto Pipeline on our pipeline right-of-way. The drawing showing SFPUC pipeline sizes and approximate locations are for your reference only. **Exact location and depth of pipelines must be verified by potholing.**

If you need to work within the SFPUC rights-of-way, Please apply for a revocable permit or consent from Ms. Stacie Feng at (650) 871-2037.

Sincerely,

Jonathan Chow, P.E.
Manager, Maintenance Engineering

JC: SC
File



"FOR INFORMATION ONLY"
PARCEL 1 COUNTY OF SAN MATEO, B.S. RES. 7-7-37 CROSS WESTMORLAND AVE, BERKSHIRE AVE., GLENDALE AVE, LOYOLA AVE, FAIR OAKS AVE, EGINAL AVE.
PARCEL 1 COUNTY OF SAN MATEO, B.S. RES. 1-11-38 CROSS NOTTINGHAM AVE, BUCKINGHAM AVE, MARLBOROUGH AVE. & DEVONSHIRE AVE.
(FEE) PARCEL 2 GEORGE F. & REGINA DURHAM, & ELIZABETH L. BARKLEY 0.065 ACRES SOLD TO SAN MATEO COUNTY
PARCEL 3 JOHN S. & ALMA MULPETER 0.108 ACRES
PALO ALTO PIPE LINE
S.P.R.R.
REDWOOD CITY
DUMBARTON OAKS
DUMBARTON PARK
OAK GROVE PARK
REDWOOD CITY VALVE HOUSE
For Connections at Redwood City Valve House see Dwg E-1154
EL CAMINO REAL
STATE HIGHWAY
PARCEL 4 CALIFORNIA PACIFIC TITLE & TRUST CO. 0.793 ACRES
PARCEL 5 MYRTLE E. DAYTON 0.070 ACRES
PARCEL 5-8 QUITCLAIM-BALFOUR-GUTHRIE INVST CO
PARCEL 7 & 9 S.P.R.R. AGREEMENT & QUITCLAIM FROM J.B. CORYELL, M.B. FLEISHHACKER & EYRE INVST CO. 20' R/W & 20' WORK AREA
PARCEL 6 & 8 TOWN OF ATHERTON, BY RESOLUTION LLOYDEN DRIVE, FAIR OAKS LANE, STATION LANE, MAPLE AVE. & FAIR OAKS AVE.
PARCEL 10 ROGER & PAULINE S. REYNOLDS 0.883 ACRES
WORK AREA
ATHERTON
LLOYDEN PARK
WATLINGTON PARK
MENLO PARK VILLA LOTS
ROGER REYNOLDS NURSERY
DOMINGA TRACT
OAK GROVE PARK
PARCEL 11 CITY OF MENLO PARK RESOLUTION No. 206 MERRILL AVE, RAVENSWOOD AVE, EGINAL AVE, GLENWOOD AVE. & OAK GROVE AVE.
PARCEL 12 A. W. & JENNIE M. CONDON
PARCEL 13 ANNA EKHOLM
PARCEL 14 MYRTLE M. & GEORGE E. WALSH 0.046 ACRES
PARCEL 15 GERTRUDE M. & CHARLES WHITEHEAD & ALICE REID 0.046 ACRES
PARCEL 16 JOSEPHINE E. FLEMING
PARCEL 17 MATHIEU BENECH 0.046 ACRES
PARCEL 18 MRS. E. J. HOFF
PARCEL 19 JOHN & LENA KELLERHALS
PARCEL 20 HARRY & FRANCES ALBRECHT
PARCEL 21 CHARLES H. & CHARLES L. SMITH
PARCEL 22 AMERICO & DELPHINE CIARDELLA
(FEE) PARCEL 23 MARGARETHA McNALLY
PARCEL 24 MENLO PARK ELEM. SCHOOL DIST. 0.469 ACRES
PARCEL 25 JOHN & MATILDA WELSH & BERTHA M. E. KUCK 0.452 ACRES
PARCEL 9 S.P.R.R. AGREEMENT 20' RIGHT-OF-WAY 20' WORK AREA
PARCEL 9 S.P.R.R. AGREEMENT 20' RIGHT-OF-WAY 20' WORK AREA
PARCEL 26 & 27 STATE OF CALIFORNIA—DIV. OF HIGHWAYS SAN MATEO — SANTA CLARA COUNTIES PERMIT No. 44,958
MENLO STATION
EDGAR SUBD.
MILLS
GOLDERS SUBD.
MERRILL SUBD.
EASEMENT TO EDWARD J. DERRY, ET. UX. FOR DERRY LANE B.S. RES. 8355—4/11/49
LIVE OAK AVE
ROBLE AVE
MIDDLE AVE
COLLEGE AVE
PATRIDGE AVE
CAMBRIDGE AVE
HARVARD AVE
Scale of Feet
E-2338-1
CITY AND COUNTY OF SAN FRANCISCO PUBLIC UTILITIES COMMISSION SAN FRANCISCO WATER DEPARTMENT
PALO ALTO PIPE LINE (SHEET 1 OF 2)

Sprint
Together with NEXTEL

Sprint-Nextel Corp.

Russell Mix
Russell.mix@sprint.com
OSP Engineer
650-533-3438 cell

February 23, 2018

Austin McCollum
Tetra Tech
711 Tank Farm Road, Suite 110
San Luis Obispo, CA 93401

Subject: Holbrook-Palmer Park Water Capture Project

Mr. McCollum:

After careful review of the above subject project, I have determined that **Sprint has facilities** within your project area shown in the area map you sent in your request. Sprint facilities reside in the existing UPRR ROW within your project limits and may have conflict if any work is being done within the UPRR ROW. Per your email dated 1/10/2018 you stated that no work will be done within the UPRR Rail Road ROW despite your map showing your limits were included the ROW, if your statement is truly the case you should not have any conflict.

Due to property information Sprint does not send out drawings of its facilities, should a more accurate location be necessary for your design, or if your design enters into the RR ROW you may contact this office to arrange for a field meet and locate of our cable system.

Sprint is very concerned about any excavation near the fiber system, the fiber can be electronically located for the horizontal but the vertical can only be determined by potholing. No mechanized excavation within two feet is allowed. Potholing by vacuum truck or by hand is required to determine the exact vertical depth and horizontal location of Sprints facilities for design and before any construction actives start. Sprint requires that a Sprint inspector be on site when potholing or any construction activity is taking place near our facilities.

If you require any further information please feel free to contact me and any time. Email and phone are best as my mail stop is not my office and any response to standard mail maybe delayed.

Please remember to; "**CALL BEFORE YOU DIG 1-800-227-2600**", before potholing or any construction activity begins.

Sincerely,

Russell J Mix

Russell J Mix - Sprint Nextel Corporation
Facilities Engineer- Outside Plant-West
650-533-3438
russell.mix@sprint.com

LEGEND

VZB BURIED CABLE

Existing

Proposed

Abandoned

VZB DIRECT BURIED CABLE

Existing

Proposed

Abandoned

VZB AERIAL CABLE

Existing

Proposed

Abandoned

VZB FSRV (UNVERIFIED)

Aerial

Buried

VZB BURIED CONDUIT (SPAN)

Existing

Proposed

Abandoned

VZB SUBMARINE CABLE

Existing

Proposed

Abandoned

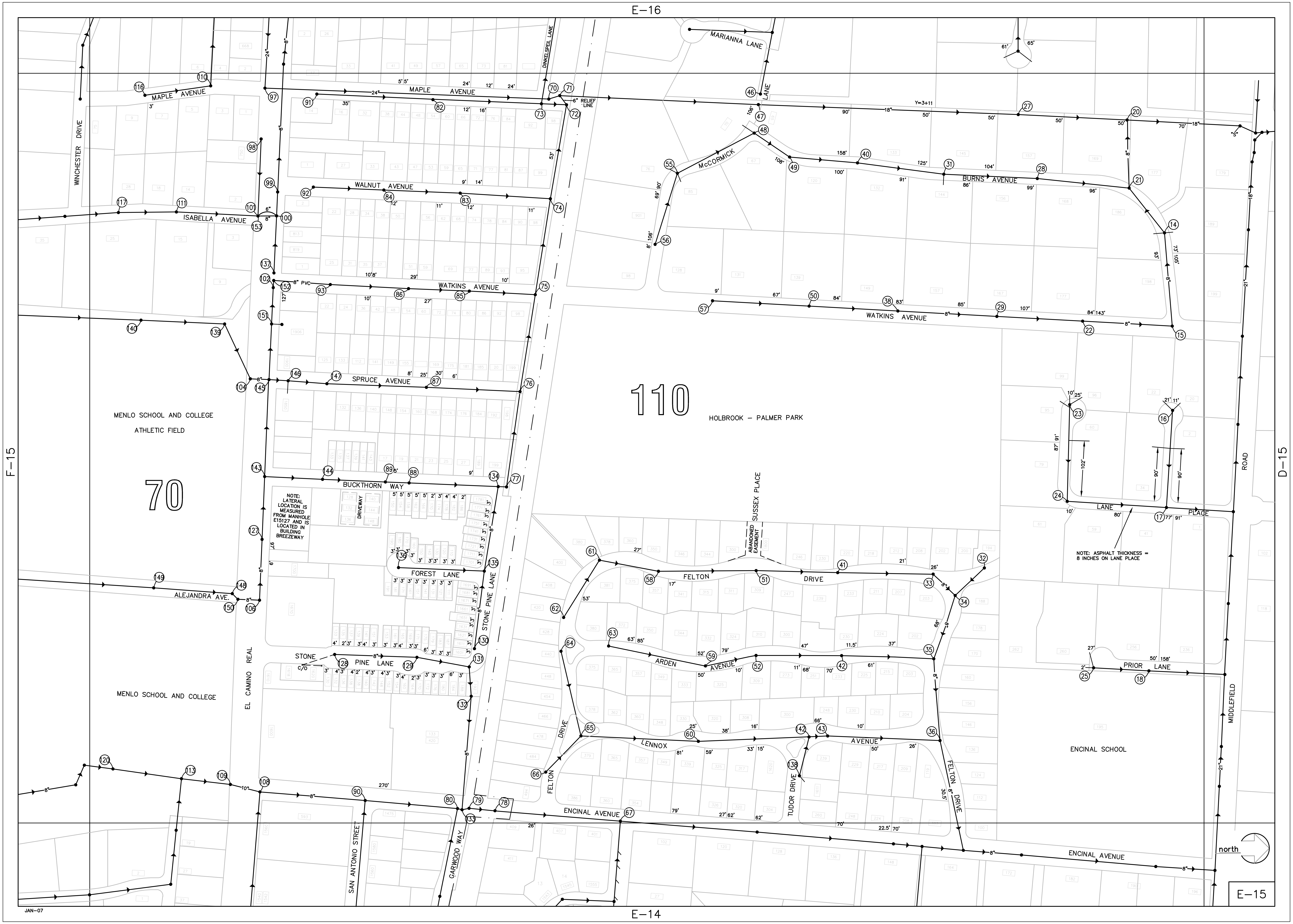
XO LONG HAUL AERIAL CABLE

XO METRO AERIAL CABLE

XO LONGHAUL BURIED CABLE

XO METRO BURIED CABLE


Holbrook-Palmer Park
Holbrook-Palmer Park
Atherton
Caltrain Atherton
Middlefield Rd
El Camino Real
Watkins Ave
Burns Ave
Fair Oaks Ln
Virginia Ln
Mt Vernon Ln
McCormick Ln
Marianna Ln
Lloyden Dr
Dinkelspiel Station Ln
Station Ln
Ashfield Rd
Maple Ave
Walnut Ave
Spruce Ave
Atherton Menlo Park
Felton Dr
Arden Rd
Lennox Ave
Tudor Dr
Encinal Ave
Glenwood Ave
Moulton Dr
Laurel St
Lane Pl
Prior Ln
James Ave
Heather Dr
Hawthorne Dr
Laburnum
Linden Ave
Surrey Ln
Redwood Way
Atherton Ave
Odell Pl
Winchester Dr
Buckthorn Way
Forest Ln
Stone Pine Ln
82
100m
300ft
No
Base Map


E-16
E-14
F-15
D-15
E-15
JAN-07
north
70
110
MENLO SCHOOL AND COLLEGE
ATHLETIC FIELD
MENLO SCHOOL AND COLLEGE
HOLBROOK - PALMER PARK
ENCINAL SCHOOL
MAPLE AVENUE
ISABELLA AVENUE
WALNUT AVENUE
WATKINS AVENUE
SPRUCE AVENUE
BUCKTHORN WAY
FOREST LANE
STONE PINE LANE
ALEJANDRA AVE.
EL CAMINO REAL
SAN ANTONIO STREET
GARWOOD WAY
ENCINAL AVENUE
WINCHESTER DRIVE
DINKELSPIEL LANE
MARIANNA LANE
McCORMICK LANE
BURNS AVENUE
LANE PLACE
PRIOR LANE
FELTON DRIVE
ARDEN AVENUE
LENNOX AVENUE
TUDOR DRIVE
SUSSEX PLACE
MIDDLEFIELD ROAD
ABANDONED EASEMENT
6" RELIEF LINE
NOTE: LATERAL LOCATION IS MEASURED FROM MANHOLE E15127 AND IS LOCATED IN BUILDING BREEZEWAY
NOTE: ASPHALT THICKNESS = 8 INCHES ON LANE PLACE
DRIVEWAY

APPENDIX D: BRIDGE ASSESSMENT

Full report begins on following page.

Technical Memorandum

Date:	03/09/2018
To:	Marty Hanneman - Town of Atherton
Cc:	Jason Fussel, PE, PLS (Project Manager – Tetra Tech)
From:	Michael Olsen, PE (Structural Project Engineer – Tetra Tech)
Project:	Atherton Water Capture Project
Subject:	Holbrook-Palmer Park Bridge Condition Assessment and Proposed Load Rating Memo

Statement of Purpose

A bridge condition assessment was performed by Tetra Tech to determine the appropriate Load Rating for the existing Holbrook-Palmer Park Bridges, and efficiently facilitate the transportation of materials and equipment for the Atherton Water Capture Project. The legal vehicle load rating refers to the AASHTO HL-93 design truck consisting of three axles, front and two rear axles with front axle weighing 8,000 lbs and two rear axles weighing 32,000 lbs each. The distance between the front and rear axle is fourteen feet, and the distance between the two rear axles can range anywhere from fourteen to thirty feet. If the required legal vehicle load rating cannot be satisfied by the existing condition of the bridges, alternative solutions will be proposed to ensure that the safe and efficient transportation of equipment is viable for the impending park improvements.

Site Investigation and Bridge Load Evaluation Results

Tetra Tech performed a site visit on January 29, 2018 in order to provide a visual condition assessment for both of the Holbrook Park Bridges and to verify the findings from the "Load Investigation and Proposed Load Rating for the Holbrook Park Bridges" report by NV5 Engineering performed in 2014. Tetra Tech utilized both the in-field investigation and review of the load rating calculations performed by NV5, to determine whether an increase to the proposed load rating found within that report may be allowed for the anticipated construction loads for the Atherton Water Capture Project. Figure 1, below, provides a visual representation of the current condition of the two existing culverts within the park.

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Tel 949.809.5000 **Fax** 949.809.5010 **tetratech.com**

Figure 1: Section of Bridge Culvert

From the in-field investigation, Tetra Tech confirmed the overall geometry of each of the bridges from the "As Constructed" plans provided in the aforementioned report. No significant damage or wear was observed at the bridge superstructure. It should be noted that at some sections of the culvert walls, minor concrete spalling and removal of concrete cover from weathering has exposed some of the rebar (See Figure 2). The damage to these walls should be addressed by the town in the near future to limit any further damage.

Figure 2: Exposed Rebar at Culvert Wall

The load rating proposed in the NV5 report utilized the LFD procedure, with reduced factors for the applied dead and live loads. Based on their initial investigation by NV5, the full legal load (i.e. HL-93/H-20) from the AASHTO LFD design procedures showed structural deficiencies in the existing system. The additional analysis to determine a loading less than HL-93/H-20 was also provided and the following are the results:

- Rear Single Axle – 8 tons (16,000 pounds)
- Double Rear Axle with axles 4 feet apart – 6.25 tons (12,500 pounds) or 12.5 tons (25,000) total for two axle combinations

Tetra Tech reviewed the structural calculations performed by NV5, as well as performed independent preliminary calculations (see Appendix A). Based on our findings, the aforementioned bridge load ratings from the NV5 report are in general agreement with maximum permissible loads conducted by Tetra Tech. Therefore, no increase in the bridge load rating may be allowed.

The following are the anticipated construction equipment loads, but are not limited to:

1) Concrete Modular Units = 30,000 to 40,000 lbs > Recommended Bridge Load Rating (NG)
2) Vactor Truck = 18,000 lbs front axle and 23,000 lbs rear axle > Recommended Bridge Load Rating (NG)
3) Fire Truck/H-20 = 32,000 lbs rear axle > Recommended Bridge Load Rating (NG)
4) Dump/Dirt Truck = 32,000 lbs rear axle > Recommended Bridge Load Rating (NG)

Alternatives and Rough Order of Magnitude Cost Estimates

Since the anticipated construction loads will exceed the recommended bridge load ratings for the existing bridges, the following alternatives are available to ensure safe passage of equipment during the construction phase of the Atherton Stormwater and Recapture Project:

A) Provide prefabricated temporary bridge.
B) Demolish and replace existing bridges with pre-fabricated metal bridge.
C) Install prefabricated metal bridge at a new location
D) Utilize crane to transfer equipment into and out of the park.

Option A provides a temporary solution that can be installed in a relatively short amount of time, given the availability and lead time. In order to install a prefabricated temporary bridge, a ramp-like structure and temporary structure would need to be constructed to elevate the temporary bridge without adding any additional load to the existing bridges. Depending on the duration required, purchasing a temporary bridge, as opposed to renting, may be a more advantageous option. The total construction cost for this approach would be approximately $117,000 (at one bridge location only). Please note that depending on availability and final bridge selection, there could be a higher risk of cost and increased lead time.

Options B and C provide a permanent solution that will allow heavy traffic load to enter and exit the park beyond the life of the project, alleviate any concerns going forward, and provide maintenance and emergency vehicle access. For each of these two options, the existing bridge slab would need to be demolished and new abutments at each end of the bridges would be required to support the bridge. If a new location is selected, then demolition costs would be reduced and lead to less disruption. The prefabricated bridge would be comprised of steel beams with a concrete deck or wood or composite decking. Weathered steel and various guardrail options could be added to the design at minimal cost to enhance aesthetics (See Figure 3). In regards to all bridge rental or replacement alternatives, careful examination of approaches on each side of the bridge must be performed to ensure proper turning radius is provided for larger vehicles. The cost for the replacement, as opposed to the bridge rental, are similar and the expected construction costs are $132,000, and $100,000, for Option B and Option C, respectively.

Figure 3: Prefabricated Steel Bridge by Big R

Option D would allow the existing bridges to remain in-place, but limitations regarding the types and weights of construction equipment required to enter and leave the park would be limited. Due to lack of feasibility, no cost estimate has been provided.

All cost estimates can be found in Appendix B.

General Recommendations

Given the site and construction constraints, a 24’-0” wide bridge replacement (two lane access per AASHTO) at a new location is the recommended alternative based on the following:

- No demolition of existing bridges and disruption to park operations.
- Reduces impacts to park operations and allows existing culvert bridges to be utilized.
- Reduced bridge width for project location will reduce cost.
- Construction schedule similar to Options A and B.

If there is no feasible location for a new bridge, Option B is recommended. Cost increases due to the demolition of the existing concrete slab are slightly higher than renting a bridge, but there are reduced risks regarding availability and price fluctuations. An assumed construction schedule is presented below for both Options B and C, for a total of 40 working days (6 to 8 weeks):

- 1-2 weeks for mobilization and site preparation
- 2 weeks for abutment installment
- 2 weeks for preparation and installation of bridge superstructure
- 1 week for miscellaneous work and
- 1 week for demobilization

TETRA TECH

Appendix A

Calculations

TETRA TECH

Preliminary Structural Calculations

for

Load Investigation and Proposed Load Rating

for

Holbrook Park Culverts

March 6, 2018

Prepared Under the Direction of

REGISTERED PROFESSIONAL ENGINEER
MICHAEL S. O. OLSEN
No. 81944
Exp. 3/31/18
CIVIL
STATE OF CALIFORNIA

Michael Olsen
Registered Civil Engineer, P.E. 81944

3/6/2018

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17885 Von Karman Avenue, Suite 500
Irvine, Ca. 92614
(949) 809-5000

200-01297-18011

Material Properties

Concrete Peak Compressive Strength	f'_c	=	3500	psi
Concrete Unit Weight	γ_c	=	150	pcf
Asphalt Unit Weight	γ_a	=	140	pcf
Reinforcing Steel Yield Strength	f_y	=	40	ksi

Loading Parameters

Span Length	=	11.625	ft
Depth of Concrete Deck	=	8.75	in
Depth of Asphalt Cover	=	2	in
Distance to center of bottom reinforcing bars	=	2.7	in
d	=	6.05	in
Height difference between two orthogonal rebars	=	0.66	in
Clear Concrete Cover	=	2	in
Impact Factor	=	1.2	LFD, (Reduced from 1.3 because of low speeds entering & exiting park)
Impact Factor	=	1.33	LRFD
Design Load Factor	=	1.3	LFD
$\varphi_{flexure}$	=	0.9	

Transverse Reinforcement

Transverse Rebar Distribution # 6 bars @ 5.5 in. O.C.

Required Distribution (%) $= \dfrac{100.00}{(11.625\ \text{ft})^{1/2}} = 29\%$

Rebar Area per foot $= \dfrac{(0.44\ \text{in}^2) \times 12}{5.5\ \text{in}} = 0.96\ \text{in}^2/\text{ft}$

Required Transverse Reinforcement per foot = 29.33% x 0.96 in^2/ft = 0.28 in/ft

Transverse Reinforcement PROVIDED per foot = 0.20 in/ft

Percentage of required Transverse Reinforcement provided $= \dfrac{0.20\ \text{in/ft}}{0.28\ \text{in/ft}} = 71\%$

Reduce distribution width by = 100% - 71% = 29%

Vertical Load Distribution Factors

Moment distributed over = 4 ft. + 0.06 x 11.63 ft = 4.70 ft (for LFD)

Reduced distribution length = 71% x 4.70 ft = 3.34 ft

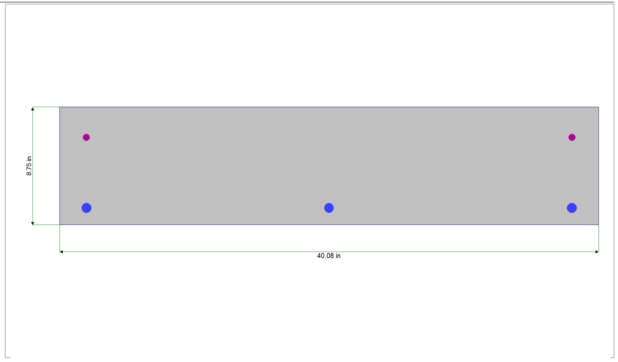
Description : Single Axle - Shear (LFD)

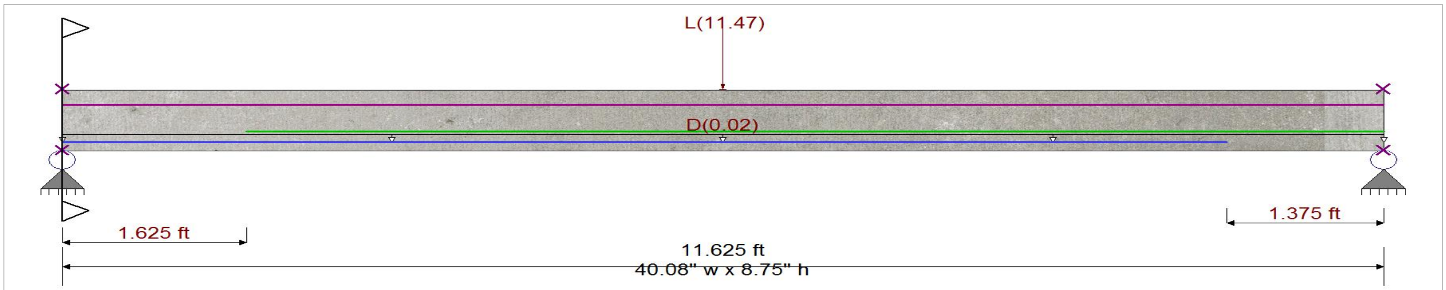
CODE REFERENCES

Calculations per ACI 318-14, IBC 2015, ASCE 7-10
Load Combination Set : IBC 2015

Material Properties

f'c	=	3.50 ksi	ϕ Phi Values	Flexure :	0.90
fr = $f'c^{1/2}$ * 7.50	=	443.706 psi		Shear :	0.750
ψ Density	=	145.0 pcf	β_1	=	0.850
λ LtWt Factor	=	1.0			
Elastic Modulus	=	3,122.0 ksi	Fy - Stirrups		40.0 ksi
fy - Main Rebar	=	40.0 ksi	E - Stirrups	=	29,000.0 ksi
E - Main Rebar	=	29,000.0 ksi	Stirrup Bar Size #		3
			Number of Resisting Legs Per Stirrup =		2





Cross Section & Reinforcing Details

Rectangular Section, Width = 40.080 in, Height = 8.750 in
Span #1 Reinforcing....
3-#6 at 1.250 in from Bottom, from 0.0 to 10.250 ft in this span
3-#6 at 2.750 in from Bottom, from 1.625 to 11.625 ft in this span
2-#4 at 2.250 in from Top, from 0.0 to 11.625 ft in this span

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loads
Load for Span Number 1
Uniform Load : D = 0.020 k/ft, Tributary Width = 1.0 ft, (Asphalt Surface)

Point Load : L = 11.470 k @ 5.813 ft, (Live Load)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.999** : 1	
Section used for this span	**Typical Section**	
Mu : Applied	51.450	k-ft
Mn * Phi : Allowable	51.510	k-ft
Location of maximum on span	5.823	ft
Span # where maximum occurs	Span # 1	

Maximum Deflection			
Max Downward Transient Deflection	0.179 in	Ratio =	781 >=36
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.273 in	Ratio =	511 >=18
Max Upward Total Deflection	0.000 in	Ratio =	999 <180

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2
Overall MAXimum	7.904	7.904
Overall MINimum	1.301	1.301
+D+H	2.169	2.169
+D+L+H	7.904	7.904
+D+Lr+H	2.169	2.169
+D+S+H	2.169	2.169
+D+0.750Lr+0.750L+H	6.470	6.470
+D+0.750L+0.750S+H	6.470	6.470

Concrete Beam

Description : Single Axle - Shear (LFD)

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2
+D+0.60W+H	2.169	2.169
+D+0.70E+H	2.169	2.169
+D+0.750Lr+0.750L+0.450W+H	6.470	6.470
+D+0.750L+0.750S+0.450W+H	6.470	6.470
+D+0.750L+0.750S+0.5250E+H	6.470	6.470
+0.60D+0.60W+0.60H	1.301	1.301
+0.60D+0.70E+0.60H	1.301	1.301
D Only	2.169	2.169
Lr Only		
L Only	5.735	5.735
S Only		
W Only		
E Only		
H Only		

Shear Stirrup Requirements

Entire Beam Span Length : Vu < PhiVc/2, Req'd Vs = Not Reqd 9.6.3.1, use stirrups spaced at 0.000 in

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Location (ft) in Span	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
MAXimum BENDING Envelope					
Span # 1	1	11.625	51.45	51.51	1.00
+1.30D+1.30Lr+1.30L					
Span # 1	1	11.625	51.45	51.51	1.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L+H	1	0.2726	5.813		0.0000	0.000

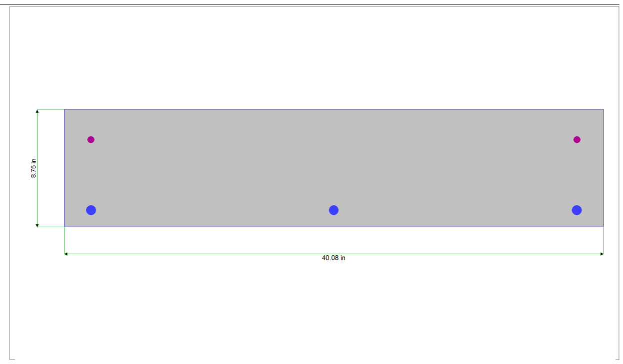
Description : Double Axle - Shear (LFD)

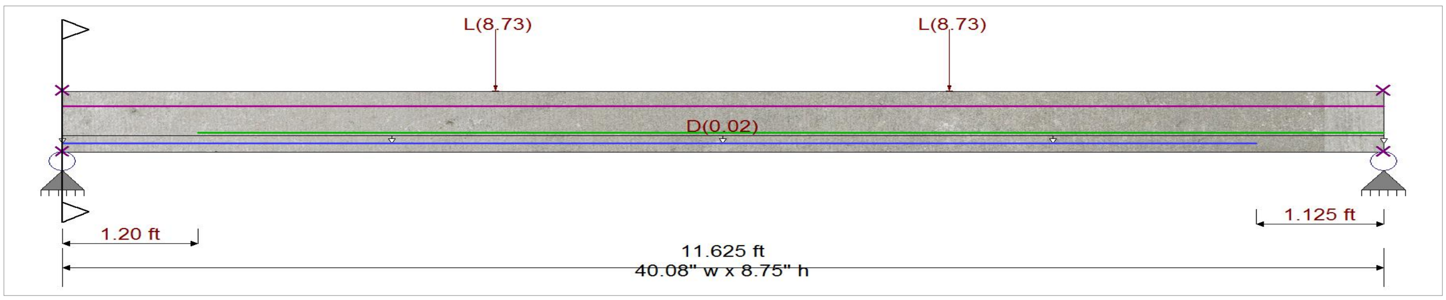
CODE REFERENCES

Calculations per ACI 318-14, IBC 2015, ASCE 7-10
Load Combination Set : IBC 2015

Material Properties

f'c	=	3.50 ksi	ϕ Phi Values	Flexure :		0.90
fr = $f'c^{1/2}$ * 7.50	=	443.706 psi		Shear :		0.750
ψ Density	=	145.0 pcf	β_1		=	0.850
λ LtWt Factor	=	1.0				
Elastic Modulus	=	3,122.0 ksi	Fy - Stirrups			40.0 ksi
fy - Main Rebar	=	40.0 ksi	E - Stirrups		=	29,000.0 ksi
E - Main Rebar	=	29,000.0 ksi	Stirrup Bar Size #			3
			Number of Resisting Legs Per Stirrup =			2





Cross Section & Reinforcing Details

Rectangular Section, Width = 40.080 in, Height = 8.750 in
Span #1 Reinforcing....
3-#6 at 1.250 in from Bottom, from 0.0 to 10.50 ft in this span
3-#6 at 2.750 in from Bottom, from 1.20 to 11.625 ft in this span
2-#4 at 2.250 in from Top, from 0.0 to 11.625 ft in this span

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loads
Load for Span Number 1
Uniform Load : D = 0.020 k/ft, Tributary Width = 1.0 ft, (Asphalt Surface)

Point Load : L = 8.730 k @ 3.813 ft, (Live Load)

Point Load : L = 8.730 k @ 7.813 ft, (Live Load)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.999** : 1	
Section used for this span	**Typical Section**	
Mu : Applied	51.462	k-ft
Mn * Phi : Allowable	51.510	k-ft
Location of maximum on span	5.823	ft
Span # where maximum occurs	Span # 1	

Maximum Deflection					
Max Downward Transient Deflection	0.291	in	Ratio =	479	>=36
Max Upward Transient Deflection	0.000	in	Ratio =	0	<360
Max Downward Total Deflection	0.390	in	Ratio =	357	>=18
Max Upward Total Deflection	0.000	in	Ratio =	999	<180

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2
Overall MAXimum	10.899	10.899
Overall MINimum	1.301	1.301
+D+H	2.169	2.169
+D+L+H	10.899	10.899
+D+Lr+H	2.169	2.169
+D+S+H	2.169	2.169

Description : Double Axle - Shear (LFD)

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2
+D+0.750Lr+0.750L+H	8.716	8.716
+D+0.750L+0.750S+H	8.716	8.716
+D+0.60W+H	2.169	2.169
+D+0.70E+H	2.169	2.169
+D+0.750Lr+0.750L+0.450W+H	8.716	8.716
+D+0.750L+0.750S+0.450W+H	8.716	8.716
+D+0.750L+0.750S+0.5250E+H	8.716	8.716
+0.60D+0.60W+0.60H	1.301	1.301
+0.60D+0.70E+0.60H	1.301	1.301
D Only	2.169	2.169
Lr Only		
L Only	8.730	8.730
S Only		
W Only		
E Only		
H Only		

Shear Stirrup Requirements

Entire Beam Span Length : PhiVc/2 < Vu <= PhiVc, Req'd Vs = Ht<=10", Not Reqd, use stirrups spaced at 0.000 in

Maximum Forces & Stresses for Load Combinations

Load Combination		Location (ft)	Bending Stress Results (k-ft)		
Segment Length	Span #	in Span	Mu : Max	Phi*Mnx	Stress Ratio
MAXimum BENDING Envelope					
Span # 1	1	11.625	51.46	51.51	1.00
+1.30D+1.30Lr+1.30L					
Span # 1	1	11.625	51.46	51.51	1.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L+H	1	0.3898	5.813		0.0000	0.000

Concrete Beam

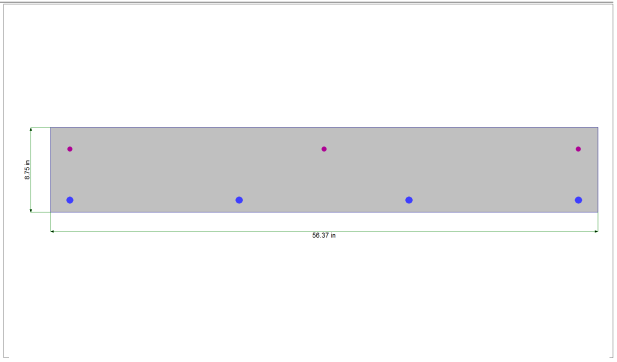
Description : Single Axle - Flexure (LFD)

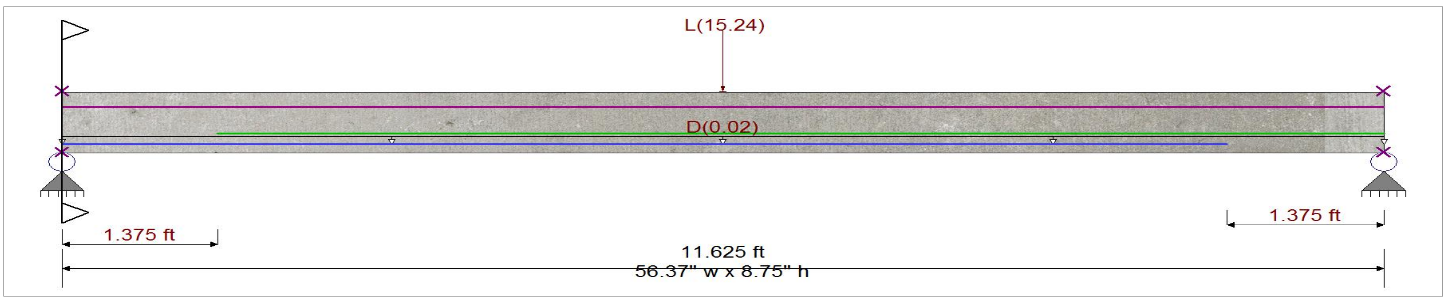
CODE REFERENCES

Calculations per ACI 318-14, IBC 2015, ASCE 7-10
Load Combination Set : IBC 2015

Material Properties

f'c	=	3.50 ksi	ϕ Phi Values	Flexure :	0.90
$fr = f'c^{1/2} * 7.50$	=	443.706 psi		Shear :	0.750
ψ Density	=	150.0 pcf	β_1	=	0.850
λ LtWt Factor	=	1.0			
Elastic Modulus	=	3,122.0 ksi	Fy - Stirrups		40.0 ksi
fy - Main Rebar	=	40.0 ksi	E - Stirrups	=	29,000.0 ksi
E - Main Rebar	=	29,000.0 ksi	Stirrup Bar Size #		3
			Number of Resisting Legs Per Stirrup =		2





Cross Section & Reinforcing Details

Rectangular Section, Width = 56.370 in, Height = 8.750 in
Span #1 Reinforcing....
4-#6 at 1.250 in from Bottom, from 0.0 to 10.250 ft in this span
4-#6 at 2.750 in from Bottom, from 1.375 to 11.625 ft in this span
3-#4 at 2.250 in from Top, from 0.0 to 11.625 ft in this span

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loads
Load for Span Number 1
Uniform Load : D = 0.020 k/ft, Tributary Width = 1.0 ft, (Asphalt Surface)

Point Load : L = 15.240 k @ 5.813 ft, (Live Load)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.999** : 1	
Section used for this span	**Typical Section**	
Mu : Applied	69.196	k-ft
Mn * Phi : Allowable	69.257	k-ft
Location of maximum on span	5.823	ft
Span # where maximum occurs	Span # 1	

Maximum Deflection			
Max Downward Transient Deflection	0.159 in	Ratio =	875 >=36
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.257 in	Ratio =	541 >=18
Max Upward Total Deflection	0.000 in	Ratio =	999 <180

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2
Overall MAXimum	10.723	10.723
Overall MINimum	1.862	1.862
+D+H	3.103	3.103
+D+L+H	10.723	10.723
+D+Lr+H	3.103	3.103
+D+S+H	3.103	3.103
+D+0.750Lr+0.750L+H	8.818	8.818
+D+0.750L+0.750S+H	8.818	8.818

Description : Single Axle - Flexure (LFD)

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2
+D+0.60W+H	3.103	3.103
+D+0.70E+H	3.103	3.103
+D+0.750Lr+0.750L+0.450W+H	8.818	8.818
+D+0.750L+0.750S+0.450W+H	8.818	8.818
+D+0.750L+0.750S+0.5250E+H	8.818	8.818
+0.60D+0.60W+0.60H	1.862	1.862
+0.60D+0.70E+0.60H	1.862	1.862
D Only	3.103	3.103
Lr Only		
L Only	7.620	7.620
S Only		
W Only		
E Only		
H Only		

Shear Stirrup Requirements

Entire Beam Span Length : Vu < PhiVc/2, Req'd Vs = Not Reqd 9.6.3.1, use stirrups spaced at 0.000 in

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Location (ft) in Span	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
MAXimum BENDING Envelope					
Span # 1	1	11.625	69.20	69.26	1.00
+1.30D+1.30Lr+1.30L					
Span # 1	1	11.625	69.20	69.26	1.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L+H	1	0.2574	5.813		0.0000	0.000

Concrete Beam

File = P:\01297\200-01297-18011\SupportDocs\Calcs\Proposed Bridge Loading Analyses.ec
ENERCALC, INC. 1983-2017, Build:10.17.12.10, Ver:10.17.12.1

Lic. # : KW-06002149 | **Licensee : TETRA TECH INC**

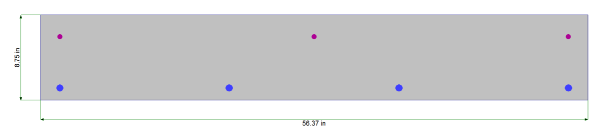
Description : Double Axle - Flexure (LFD)

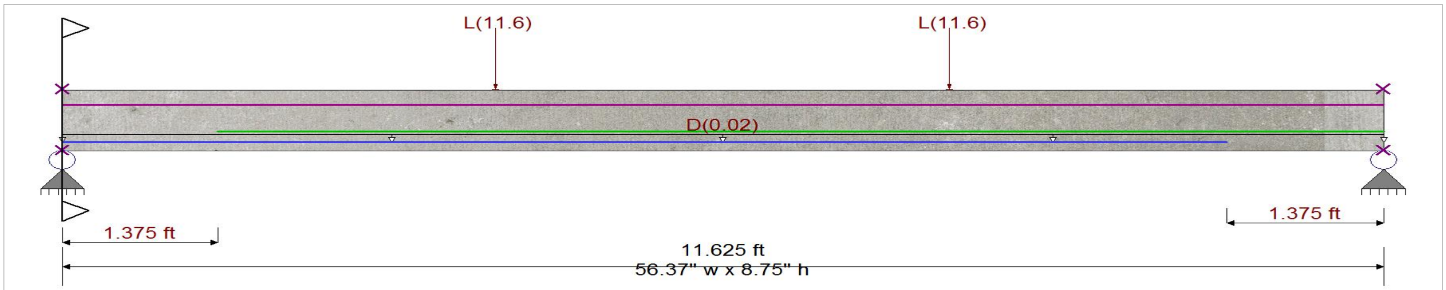
CODE REFERENCES

Calculations per ACI 318-14, IBC 2015, ASCE 7-10
Load Combination Set : IBC 2015

Material Properties

f'c	=	3.50 ksi	ϕ Phi Values	Flexure :		0.90
fr = $f'c^{1/2}$ * 7.50	=	443.706 psi		Shear :		0.750
ψ Density	=	150.0 pcf	β_1	=		0.850
λ LtWt Factor	=	1.0				
Elastic Modulus	=	3,122.0 ksi	Fy - Stirrups			40.0 ksi
fy - Main Rebar	=	40.0 ksi	E - Stirrups	=		29,000.0 ksi
E - Main Rebar	=	29,000.0 ksi	Stirrup Bar Size #			3
			Number of Resisting Legs Per Stirrup =			2





Cross Section & Reinforcing Details

Rectangular Section, Width = 56.370 in, Height = 8.750 in
Span #1 Reinforcing....
4-#6 at 1.250 in from Bottom, from 0.0 to 10.250 ft in this span
4-#6 at 2.750 in from Bottom, from 1.375 to 11.625 ft in this span
3-#4 at 2.250 in from Top, from 0.0 to 11.625 ft in this span

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loads
Load for Span Number 1
Uniform Load : D = 0.020 k/ft, Tributary Width = 1.0 ft, (Asphalt Surface)

Point Load : L = 11.60 k @ 3.813 ft, (Live Load)

Point Load : L = 11.60 k @ 7.813 ft, (Live Load)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.999** : 1	
Section used for this span	**Typical Section**	
Mu : Applied	69.215	k-ft
Mn * Phi : Allowable	69.257	k-ft
Location of maximum on span	5.823	ft
Span # where maximum occurs	Span # 1	

Maximum Deflection

Max Downward Transient Deflection	0.266 in	Ratio =	523 >=36
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.373 in	Ratio =	373 >=18
Max Upward Total Deflection	0.000 in	Ratio =	999 <180

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2
Overall MAXimum	14.703	14.703
Overall MINimum	1.862	1.862
+D+H	3.103	3.103
+D+L+H	14.703	14.703
+D+Lr+H	3.103	3.103
+D+S+H	3.103	3.103

Description : Double Axle - Flexure (LFD)

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2
+D+0.750Lr+0.750L+H	11.803	11.803
+D+0.750L+0.750S+H	11.803	11.803
+D+0.60W+H	3.103	3.103
+D+0.70E+H	3.103	3.103
+D+0.750Lr+0.750L+0.450W+H	11.803	11.803
+D+0.750L+0.750S+0.450W+H	11.803	11.803
+D+0.750L+0.750S+0.5250E+H	11.803	11.803
+0.60D+0.60W+0.60H	1.862	1.862
+0.60D+0.70E+0.60H	1.862	1.862
D Only	3.103	3.103
Lr Only		
L Only	11.600	11.600
S Only		
W Only		
E Only		
H Only		

Shear Stirrup Requirements

Entire Beam Span Length : Vu < PhiVc/2, Req'd Vs = Not Reqd 9.6.3.1, use stirrups spaced at 0.000 in

Maximum Forces & Stresses for Load Combinations

Load Combination Segment Length	Span #	Location (ft) in Span	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
MAXimum BENDING Envelope					
Span # 1	1	11.625	69.21	69.26	1.00
+1.30D+1.30Lr+1.30L					
Span # 1	1	11.625	69.21	69.26	1.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L+H	1	0.3733	5.813		0.0000	0.000

Concrete Beam

File = P:\01297\200-01297-18011\SupportDocs\Calcs\Proposed Bridge Loading Analyses.ec
ENERCALC, INC. 1983-2017, Build:10.17.12.10, Ver:10.17.12.1

Lic. # : KW-06002149 **Licensee : TETRA TECH INC**

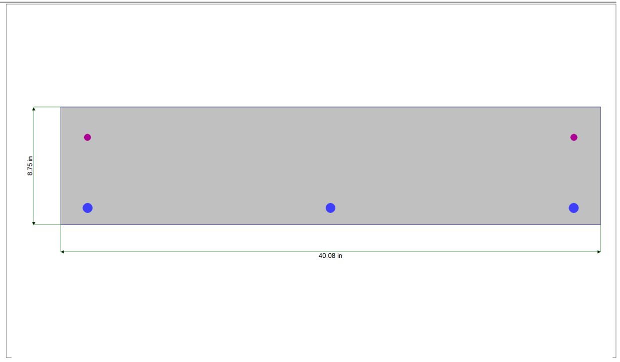
Description : Single Axle - Shear (AASHTO Strength I L.C.)

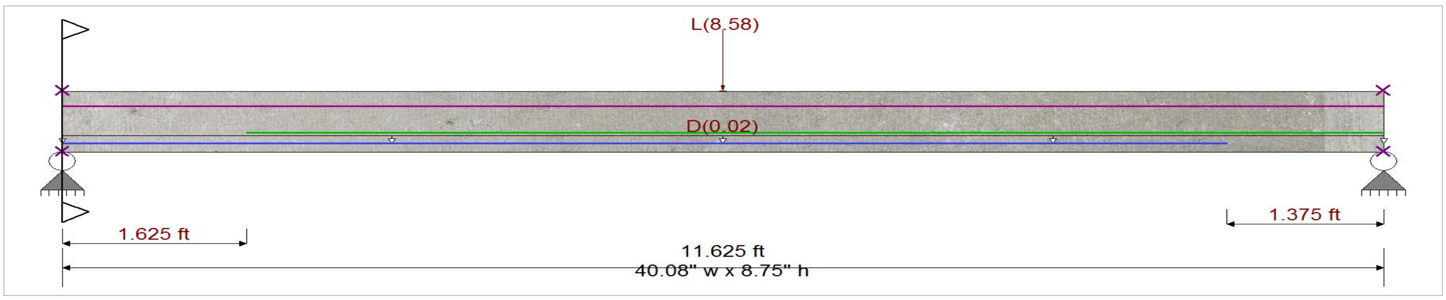
CODE REFERENCES

Calculations per ACI 318-14, IBC 2015, ASCE 7-10
Load Combination Set : IBC 2015

Material Properties

f'c	=	3.50 ksi	ϕ Phi Values	Flexure :	0.90
fr = $f'c^{1/2}$ * 7.50	=	443.706 psi		Shear :	0.750
ψ Density	=	145.0 pcf	β_1	=	0.850
λ LtWt Factor	=	1.0			
Elastic Modulus	=	3,122.0 ksi	Fy - Stirrups		40.0 ksi
fy - Main Rebar	=	40.0 ksi	E - Stirrups	=	29,000.0 ksi
E - Main Rebar	=	29,000.0 ksi	Stirrup Bar Size #		3
			Number of Resisting Legs Per Stirrup =		2





Cross Section & Reinforcing Details

Rectangular Section, Width = 40.080 in, Height = 8.750 in
Span #1 Reinforcing....
3-#6 at 1.250 in from Bottom, from 0.0 to 10.250 ft in this span
3-#6 at 2.750 in from Bottom, from 1.625 to 11.625 ft in this span
2-#4 at 2.250 in from Top, from 0.0 to 11.625 ft in this span

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loads
Load for Span Number 1
Uniform Load : D = 0.020 k/ft, Tributary Width = 1.0 ft, (Asphalt Surface)

Point Load : L = 8.580 k @ 5.813 ft, (Live Load)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.999** : 1	
Section used for this span	**Typical Section**	
Mu : Applied	51.437	k-ft
Mn * Phi : Allowable	51.510	k-ft
Location of maximum on span	5.802	ft
Span # where maximum occurs	Span # 1	

Maximum Deflection			
Max Downward Transient Deflection	0.089 in	Ratio =	1573 >=36
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.168 in	Ratio =	831 >=18
Max Upward Total Deflection	0.000 in	Ratio =	999 <180

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2
Overall MAXimum	6.459	6.459
Overall MINimum	1.301	1.301
+D+H	2.169	2.169
+D+L+H	6.459	6.459
+D+Lr+H	2.169	2.169
+D+S+H	2.169	2.169
+D+0.750Lr+0.750L+H	5.386	5.386
+D+0.750L+0.750S+H	5.386	5.386

Description : Single Axle - Shear (AASHTO Strength I L.C.)

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2
+D+0.60W+H	2.169	2.169
+D+0.70E+H	2.169	2.169
+D+0.750Lr+0.750L+0.450W+H	5.386	5.386
+D+0.750L+0.750S+0.450W+H	5.386	5.386
+D+0.750L+0.750S+0.5250E+H	5.386	5.386
+0.60D+0.60W+0.60H	1.301	1.301
+0.60D+0.70E+0.60H	1.301	1.301
D Only	2.169	2.169
Lr Only		
L Only	4.290	4.290
S Only		
W Only		
E Only		
H Only		

Shear Stirrup Requirements

Entire Beam Span Length : Vu < PhiVc/2, Req'd Vs = Not Reqd 9.6.3.1, use stirrups spaced at 0.000 in

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Location (ft) in Span	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
MAXimum BENDING Envelope					
Span # 1	1	11.625	51.44	51.51	1.00
+1.250D+1.750Lr+1.750L					
Span # 1	1	11.625	51.44	51.51	1.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L+H	1	0.1679	5.813		0.0000	0.000

Concrete Beam

Lic. # : KW-06002149 **Licensee : TETRA TECH INC**

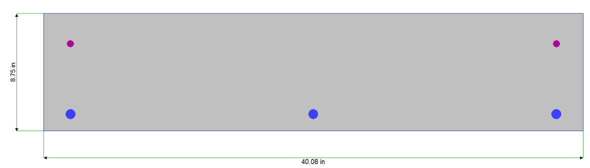
Description : Double Axle - Shear (AASHTO Strength I L.C.)

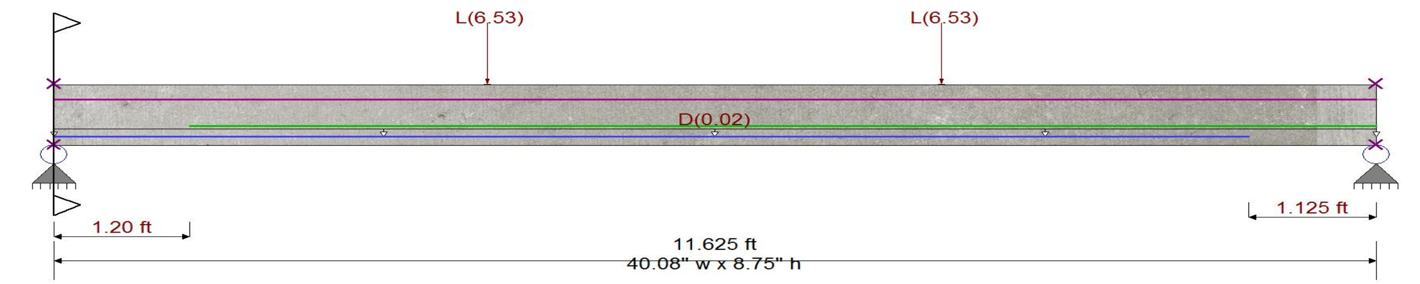
CODE REFERENCES

Calculations per ACI 318-14, IBC 2015, ASCE 7-10
Load Combination Set : IBC 2015

Material Properties

f'c	=	3.50 ksi	ϕ Phi Values	Flexure :		0.90
fr = $f'c^{1/2}$ * 7.50	=	443.706 psi		Shear :		0.750
ψ Density	=	145.0 pcf	β_1		=	0.850
λ LtWt Factor	=	1.0				
Elastic Modulus	=	3,122.0 ksi	Fy - Stirrups			40.0 ksi
fy - Main Rebar	=	40.0 ksi	E - Stirrups		=	29,000.0 ksi
E - Main Rebar	=	29,000.0 ksi	Stirrup Bar Size #			3
			Number of Resisting Legs Per Stirrup =			2





Cross Section & Reinforcing Details

Rectangular Section, Width = 40.080 in, Height = 8.750 in
Span #1 Reinforcing....
3-#6 at 1.250 in from Bottom, from 0.0 to 10.50 ft in this span
3-#6 at 2.750 in from Bottom, from 1.20 to 11.625 ft in this span
2-#4 at 2.250 in from Top, from 0.0 to 11.625 ft in this span

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loads
Load for Span Number 1
Uniform Load : D = 0.020 k/ft, Tributary Width = 1.0 ft, (Asphalt Surface)

Point Load : L = 6.530 k @ 3.813 ft, (Live Load)

Point Load : L = 6.530 k @ 7.813 ft, (Live Load)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.999** : 1	
Section used for this span	**Typical Section**	
Mu : Applied	51.446	k-ft
Mn * Phi : Allowable	51.510	k-ft
Location of maximum on span	5.823	ft
Span # where maximum occurs	Span # 1	

Maximum Deflection			
Max Downward Transient Deflection	0.150 in	Ratio =	931 >=36
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.252 in	Ratio =	553 >=18
Max Upward Total Deflection	0.000 in	Ratio =	999 <180

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2
Overall MAXimum	8.699	8.699
Overall MINimum	1.301	1.301
+D+H	2.169	2.169
+D+L+H	8.699	8.699
+D+Lr+H	2.169	2.169
+D+S+H	2.169	2.169

Concrete Beam

Description : Double Axle - Shear (AASHTO Strength I L.C.)

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2
+D+0.750Lr+0.750L+H	7.066	7.066
+D+0.750L+0.750S+H	7.066	7.066
+D+0.60W+H	2.169	2.169
+D+0.70E+H	2.169	2.169
+D+0.750Lr+0.750L+0.450W+H	7.066	7.066
+D+0.750L+0.750S+0.450W+H	7.066	7.066
+D+0.750L+0.750S+0.5250E+H	7.066	7.066
+0.60D+0.60W+0.60H	1.301	1.301
+0.60D+0.70E+0.60H	1.301	1.301
D Only	2.169	2.169
Lr Only		
L Only	6.530	6.530
S Only		
W Only		
E Only		
H Only		

Shear Stirrup Requirements

Entire Beam Span Length : PhiVc/2 < Vu <= PhiVc, Req'd Vs = Ht<=10", Not Reqd, use stirrups spaced at 0.000 in

Maximum Forces & Stresses for Load Combinations

Load Combination		Location (ft)	Bending Stress Results (k-ft)		
Segment Length	Span #	in Span	Mu : Max	Phi*Mnx	Stress Ratio
MAXimum BENDING Envelope					
Span # 1	1	11.625	51.45	51.51	1.00
+1.250D+1.750Lr+1.750L					
Span # 1	1	11.625	51.45	51.51	1.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L+H	1	0.2521	5.813		0.0000	0.000

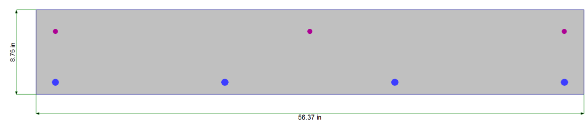
Description : Single Axle - Flexure (AASHTO Strength I L.C.)

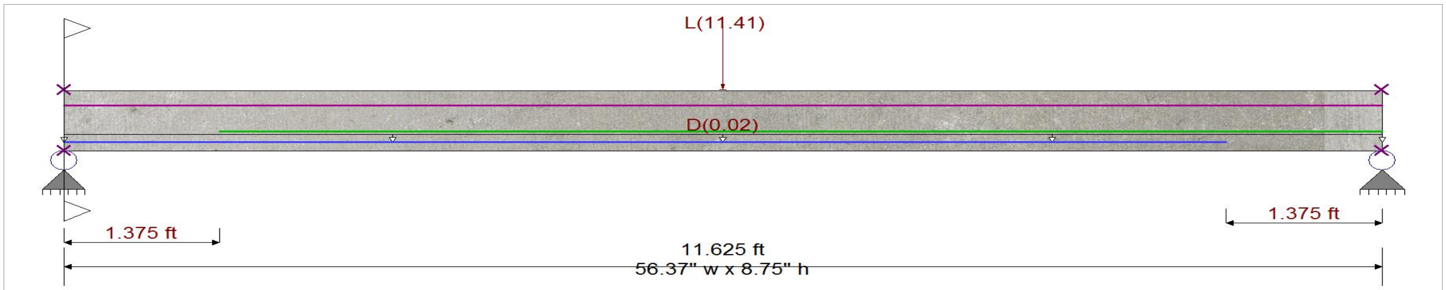
CODE REFERENCES

Calculations per ACI 318-14, IBC 2015, ASCE 7-10
Load Combination Set : IBC 2015

Material Properties

f'c	=	3.50 ksi	ϕ Phi Values	Flexure :	0.90
fr = $f'c^{1/2}$ * 7.50	=	443.706 psi		Shear :	0.750
ψ Density	=	150.0 pcf	β_1	=	0.850
λ LtWt Factor	=	1.0			
Elastic Modulus	=	3,122.0 ksi	Fy - Stirrups		40.0 ksi
fy - Main Rebar	=	40.0 ksi	E - Stirrups	=	29,000.0 ksi
E - Main Rebar	=	29,000.0 ksi	Stirrup Bar Size #		3
			Number of Resisting Legs Per Stirrup =		2





Cross Section & Reinforcing Details

Rectangular Section, Width = 56.370 in, Height = 8.750 in
Span #1 Reinforcing....
4-#6 at 1.250 in from Bottom, from 0.0 to 10.250 ft in this span
4-#6 at 2.750 in from Bottom, from 1.375 to 11.625 ft in this span
3-#4 at 2.250 in from Top, from 0.0 to 11.625 ft in this span

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loads
Load for Span Number 1
Uniform Load : D = 0.020 k/ft, Tributary Width = 1.0 ft, (Asphalt Surface)

Point Load : L = 11.410 k @ 5.813 ft, (Live Load)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.999** : 1	
Section used for this span	**Typical Section**	
Mu : Applied	69.196	k-ft
Mn * Phi : Allowable	69.257	k-ft
Location of maximum on span	5.823	ft
Span # where maximum occurs	Span # 1	

Maximum Deflection			
Max Downward Transient Deflection	0.078 in	Ratio =	1792 >=36
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.155 in	Ratio =	898 >=18
Max Upward Total Deflection	0.000 in	Ratio =	999 <180

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2
Overall MAXimum	8.808	8.808
Overall MINimum	1.862	1.862
+D+H	3.103	3.103
+D+L+H	8.808	8.808
+D+Lr+H	3.103	3.103
+D+S+H	3.103	3.103
+D+0.750Lr+0.750L+H	7.381	7.381
+D+0.750L+0.750S+H	7.381	7.381

Description : Single Axle - Flexure (AASHTO Strength I L.C.)

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2
+D+0.60W+H	3.103	3.103
+D+0.70E+H	3.103	3.103
+D+0.750Lr+0.750L+0.450W+H	7.381	7.381
+D+0.750L+0.750S+0.450W+H	7.381	7.381
+D+0.750L+0.750S+0.5250E+H	7.381	7.381
+0.60D+0.60W+0.60H	1.862	1.862
+0.60D+0.70E+0.60H	1.862	1.862
D Only	3.103	3.103
Lr Only		
L Only	5.705	5.705
S Only		
W Only		
E Only		
H Only		

Shear Stirrup Requirements

Entire Beam Span Length : Vu < PhiVc/2, Req'd Vs = Not Reqd 9.6.3.1, use stirrups spaced at 0.000 in

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Location (ft) in Span	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
MAXimum BENDING Envelope					
Span # 1	1	11.625	69.20	69.26	1.00
+1.250D+1.750Lr+1.750L					
Span # 1	1	11.625	69.20	69.26	1.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L+H	1	0.1553	5.813		0.0000	0.000

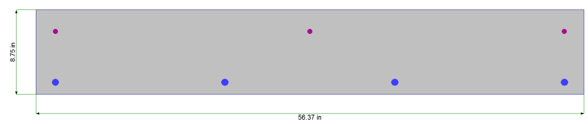
Description : Double Axle - Flexure (AASHTO Strength I L.C.)

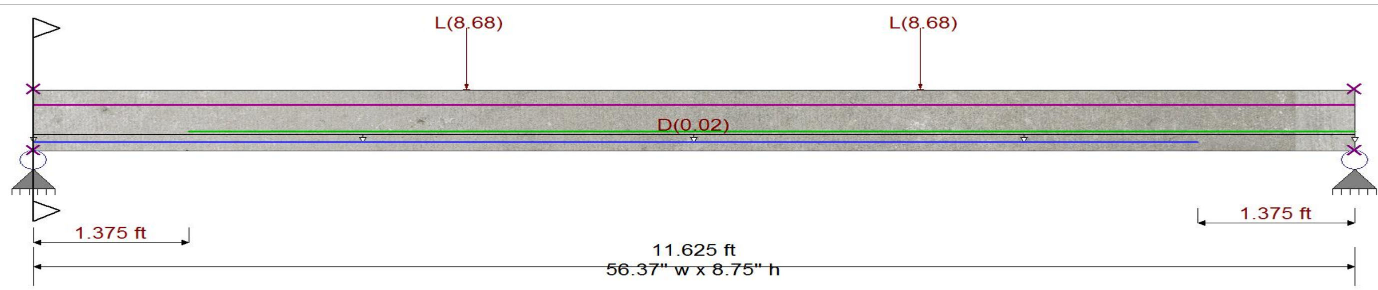
CODE REFERENCES

Calculations per ACI 318-14, IBC 2015, ASCE 7-10
Load Combination Set : IBC 2015

Material Properties

f'c	=	3.50 ksi	ϕ Phi Values		Flexure :	0.90
fr = $f'c^{1/2}$ * 7.50	=	443.706 psi			Shear :	0.750
ψ Density	=	150.0 pcf	β_1	=		0.850
λ LtWt Factor	=	1.0				
Elastic Modulus	=	3,122.0 ksi	Fy - Stirrups			40.0 ksi
fy - Main Rebar	=	40.0 ksi	E - Stirrups	=		29,000.0 ksi
E - Main Rebar	=	29,000.0 ksi	Stirrup Bar Size #			3
			Number of Resisting Legs Per Stirrup =			2





Cross Section & Reinforcing Details

Rectangular Section, Width = 56.370 in, Height = 8.750 in
Span #1 Reinforcing....
4-#6 at 1.250 in from Bottom, from 0.0 to 10.250 ft in this span
4-#6 at 2.750 in from Bottom, from 1.375 to 11.625 ft in this span
3-#4 at 2.250 in from Top, from 0.0 to 11.625 ft in this span

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loads
Load for Span Number 1
Uniform Load : D = 0.020 k/ft, Tributary Width = 1.0 ft, (Asphalt Surface)

Point Load : L = 8.680 k @ 3.813 ft, (Live Load)

Point Load : L = 8.680 k @ 7.813 ft, (Live Load)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.999** : 1	
Section used for this span	**Typical Section**	
Mu : Applied	69.183	k-ft
Mn * Phi : Allowable	69.257	k-ft
Location of maximum on span	5.823	ft
Span # where maximum occurs	Span # 1	

Maximum Deflection

Max Downward Transient Deflection	0.129 in	Ratio =	1077 >=36
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.235 in	Ratio =	592 >=18
Max Upward Total Deflection	0.000 in	Ratio =	999 <180

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2
Overall MAXimum	11.783	11.783
Overall MINimum	1.862	1.862
+D+H	3.103	3.103
+D+L+H	11.783	11.783
+D+Lr+H	3.103	3.103
+D+S+H	3.103	3.103

Description : Double Axle - Flexure (AASHTO Strength I L.C.)

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2
+D+0.750Lr+0.750L+H	9.613	9.613
+D+0.750L+0.750S+H	9.613	9.613
+D+0.60W+H	3.103	3.103
+D+0.70E+H	3.103	3.103
+D+0.750Lr+0.750L+0.450W+H	9.613	9.613
+D+0.750L+0.750S+0.450W+H	9.613	9.613
+D+0.750L+0.750S+0.5250E+H	9.613	9.613
+0.60D+0.60W+0.60H	1.862	1.862
+0.60D+0.70E+0.60H	1.862	1.862
D Only	3.103	3.103
Lr Only		
L Only	8.680	8.680
S Only		
W Only		
E Only		
H Only		

Shear Stirrup Requirements

Entire Beam Span Length : Vu < PhiVc/2, Req'd Vs = Not Reqd 9.6.3.1, use stirrups spaced at 0.000 in

Maximum Forces & Stresses for Load Combinations

Load Combination		Location (ft)	Bending Stress Results (k-ft)		
Segment Length	Span #	in Span	Mu : Max	Phi*Mnx	Stress Ratio
MAXimum BENDING Envelope					
Span # 1	1	11.625	69.18	69.26	1.00
+1.250D+1.750Lr+1.750L					
Span # 1	1	11.625	69.18	69.26	1.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L+H	1	0.2353	5.813		0.0000	0.000

Appendix B

Cost Estimates

TETRA TECH

DATE PREPARED: 3/6/2018

PREPARED BY: Brett Boehmke

CHECKED BY: M. Olsen

ENGINEER'S ESTIMATE OF PROBABLE CONSTRUCTION COST:

ITEM NO.	ITEM DESCRIPTION	UNIT OF MEASURE	ESTIMATED QUANTITY	UNIT COST	TOTAL COST
	Option A (Temporary Bridge)				
1	CLEARING AND GRUBBING	LS	1	$ 2,500.00	$ 2,500.00
2	DEMOLITION OF EXISTING SLAB ON CULVERT	LS	1	$ 7,500.00	$ 7,500.00
3	REPAIR APPROACHES	SF	693	$ 10.00	$ 6,930.00
4	EXCAVATION	CY	80	$ 130.00	$ 10,400.00
5	BACKFILL & COMPACTION	CY	80	$ 130.00	$ 10,400.00
6	TEMPORARY BRIDGE RENTAL	MONTH	5	$ 5,000.00	$ 25,000.00
7	STRUCTURAL CONCRETE	CY	27	$ 1,000.00	$ 27,000.00
				SUBTOTAL	$ 89,730.00
8	MOBILIZATION AND DEMOBILIZATION (NOT TO EXCEED 10% OF THE SUBTOTAL BASE BID)	LS	1	$ 8,973.00	$ 8,973.00
9	CONTIGENCY (20%)	LS	1	$ 17,946.00	$ 17,946.00
				GRAND TOTAL:	**$ 117,000.00**

Notes:

1. This is an estimate only, these figures are supplied as a guide. Tetra Tech, Inc. is not responsible for fluctuation in cost of material, labor or componenents, or unforseen contingincies.

2. Estimated costs for materials testing, staking, and construction management are not included.

TETRA TECH

DATE PREPARED: 3/6/2018

PREPARED BY: Brett Boehmke

CHECKED BY: M. Olsen

ENGINEER'S ESTIMATE OF PROBABLE CONSTRUCTION COST:

ITEM NO.	ITEM DESCRIPTION	UNIT OF MEASURE	ESTIMATED QUANTITY	UNIT COST	TOTAL COST
	Option B (Demolition and Replace)				
1	CLEARING AND GRUBBING	LS	1	$ 2,500.00	$ 2,500.00
2	DEMOLITION OF EXISTING SLAB ON CULVERT	LS	1	$ 7,500.00	$ 7,500.00
3	REPAIR APPROACHES	SF	693	$ 10.00	$ 6,930.00
4	EXCAVATION	CY	80	$ 130.00	$ 10,400.00
5	BACKFILL & COMPACTION	CY	80	$ 130.00	$ 10,400.00
6	PRE-FABRICATED STEEL BRIDGE	LS	1	$ 36,300.00	$ 36,300.00
7	STRUCTURAL CONCRETE	CY	27	$ 1,000.00	$ 27,000.00
				SUBTOTAL	$ 101,030.00
8	MOBILIZATION AND DEMOBILIZATION (NOT TO EXCEED 10% OF THE SUBTOTAL BASE BID)	LS	1	$ 10,103.00	$ 10,103.00
9	CONTIGENCY (20%)	LS	1	$ 20,206.00	$ 20,206.00
				GRAND TOTAL:	**$ 132,000.00**

Notes:

1. This is an estimate only, these figures are supplied as a guide. Tetra Tech, Inc. is not responsible for fluctuation in cost of material, labor or componenents, or unforseen contingincies.

2. Estimated costs for materials testing, staking, and construction management are not included.

TETRA TECH

DATE PREPARED: 3/6/2018

PREPARED BY: Brett Boehmke

CHECKED BY: M. Olsen

ENGINEER'S ESTIMATE OF PROBABLE CONSTRUCTION COST:

ITEM NO.	ITEM DESCRIPTION	UNIT OF MEASURE	ESTIMATED QUANTITY	UNIT COST	TOTAL COST
	Option C (Demolition and Replace in New Location)				
1	CLEARING AND GRUBBING	LS	1	$ 2,500.00	$ 2,500.00
2	DEMOLITION OF EXISTING SLAB ON CULVERT	LS	1	$ 7,500.00	$ 7,500.00
3	REPAIR APPROACHES	SF	504	$ 10.00	$ 5,040.00
4	EXCAVATION	CY	58	$ 130.00	$ 7,563.64
5	BACKFILL & COMPACTION	CY	58	$ 130.00	$ 7,563.64
6	PRE-FABRICATED STEEL BRIDGE	LS	1	$ 26,400.00	$ 26,400.00
7	STRUCTURAL CONCRETE	CY	20	$ 1,000.00	$ 19,636.36
				SUBTOTAL	$ 76,203.64
8	MOBILIZATION AND DEMOBILIZATION (NOT TO EXCEED 10% OF THE SUBTOTAL BASE BID)	LS	1	$ 7,620.36	$ 7,620.36
9	CONTIGENCY (20%)	LS	1	$ 15,240.73	$ 15,240.73
				GRAND TOTAL:	**$ 100,000.00**

Notes:

1. This is an estimate only, these figures are supplied as a guide. Tetra Tech, Inc. is not responsible for fluctuation in cost of material, labor or componenents, or unforseen contingincies.

2. Estimated costs for materials testing, staking, and construction management are not included.

APPENDIX E: DRAFT GEOTECHNICAL INVESTIGATION REPORT

ATHERTON WATER CAPTURE PROJECT

ATHERTON, CALIFORNIA

GEOTECHNICAL EXPLORATION

SUBMITTED TO
Mr. Jason Fussel, PE, LEED-AP
Tetra Tech, Inc.
152 N. Third St.
Ste. 201
San Jose, CA 95112

PREPARED BY
ENGEO Incorporated

April 9, 2018

PROJECT NO.
14695.000.000

Project No.
14695.000.000

April 9, 2018

Mr. Jason Fussel, PE, LEED-AP
Tetra Tech, Inc.
152 N. Third St.
Ste. 201
San Jose, CA 95112

Subject: Atherton Water Capture Project
150 Watkins Ave
Atherton, California

GEOTECHNICAL EXPLORATION

Dear Mr. Fussel:

ENGEO prepared this geotechnical report for Tetra Tech, Inc. as outlined in our agreements dated September 27, 2017 and January 26, 2018. We characterized the subsurface conditions at the site to provide the enclosed geotechnical recommendations for design.

Our experience and that of our profession clearly indicate that the risk of costly design, construction, and maintenance problems can be significantly lowered by retaining the design geotechnical engineering firm to review the project plans and specifications and provide geotechnical observation and testing services during construction. Please let us know when working drawings are nearing completion, and we will be glad to discuss these additional services with you.

If you have any questions or comments regarding this report, please call and we will be glad to discuss them with you.

Sincerely,

ENGEO Incorporated

Teresa Klotzback, EIT

Jonathan Boland, GE

Andrew Firmin, GE, QSD
tk/af/jk/WP

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TABLE OF CONTENTS (Continued)

1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

The purpose of this geotechnical report, as described in our proposals dated September 27, 2017 and January 26, 2018, is to provide geotechnical recommendations for design of the proposed infiltration system at the site. The scope of our services included the following:

- Reviewing published geologic maps and aerial photographs of the proposed site.
- Visiting the site to identify locations for exploratory borings and cone penetration test soundings.
- Retaining subcontractors to advance six Cone Penetration Tests (CPTs) and drill six exploratory borings, which were logged by a representative of our firm.
- Performing infiltration testing at the location of each boring.
- Performing laboratory testing of representative samples retrieved during the field exploration.
- Assessing and analyzing geotechnical hazards.
- Preparing this report identifying potential geotechnical hazards with recommendations for design.

We prepared this report for the exclusive use of our client and their consultants for evaluation of this project. In the event that any changes are made in the character, design or layout of the development, we must be contacted to review the conclusions and recommendations contained in this report to determine whether modifications are necessary. This document may not be reproduced in whole or in part by any means whatsoever, nor may it be quoted or excerpted without our express written consent.

1.2 SITE DESCRIPTION AND PROPOSED IMPROVEMENTS

The 21.7 acres site is located at Holbrook-Palmer Park at 150 Watkins Avenue (currently designated as APN 061-310-100) in Atherton, California. The site is bordered by Watkins Avenue to the north, residential structures to the east and south, and railroad tracks to the west (Figure 2). The site is a recreational park with a ball field, tennis courts, a playground, gardens, parking lots and walking paths. There are three structures on site used as community centers and a preschool. The site ground surface elevation is as low as approximately 50 feet (WGS84) toward the north end of the project and gradually rises to the south to approximately elevation 55 feet.

Based on our discussions with Tetra Tech and review of the concept plans prepared by Tetra Tech, dated February 14, 2018, we understand the project will include construction of the following:

- Below grade water quality treatment structure
- Wet well / lift station
- At-grade ancillary buildings
- Underground utilities
- Temporary and permanent bridge structures
- Pavement improvements

At this time, we understand that the underground storage will consist of a precast concrete structure and will most likely be located within the existing main field, extending to a depth of approximately 18 feet below ground surface (bgs). It is our understanding that the wet well / lift station will extend up to approximately 25 feet bgs, and will be approximately 96 inches in diameter.

The locations of the temporary and permanent bridges were not known at the time of this report.

2.0 FINDINGS

2.1 SITE BACKGROUND

Based on historic aerial photographs and data review, the site was previously used as a residence and farm in the early 1900's before being converted into a park in 1964. Prior to 1963, the site consisted of a manor house, carriage house, and water tower. The manor house burned down in 1963 and was replaced by what is now called the Main House. Based on review of aerial images, it appears that the site was regraded between 1968 and 1982, with construction of tennis courts, paved parking lots, and driveways. It seems that minor landscaping has occurred onsite since 1982.

A manmade channel (Atherton Channel) parallels Watkins Avenue along the northwestern boundary of the site. It appears that a stream coming from the foothills to the southwest of the site was channelized to this location between 1956 and 1961.

2.2 GEOLOGY AND SEISMICITY

2.2.1 Geology

The study area is located in the Coast Ranges geomorphic province of California. The Coast Ranges are characterized by a series of northwest-trending valleys and mountain ranges. West of the project site, the upland areas are bisected by the San Andreas fault. The bedrock in this region has been folded and faulted in a tectonic setting that is experiencing translational and compressional deformations of the earth's crust.

According to Dibblee and Minch, 2007, the site is underlain by Qa.1, which are surficial sediments consisting of alluvial sand, fine-grained, silt, and gravel; where differentiated from Qa, Qa.1 represents deposits at base of slopes and upper fan areas.

Geologic mapping by Brabb and Pampeyan, 1983, further differentiated the alluvial units. Specifically, the site is predominantly underlain by Qyf and bordered by Qof. Qyf and Qof were subsequently identified by Brabb, Graymer, and Jones, 1988, as Qhaf and Qpaf, respespectively. Qhaf is described as younger (inner) alluvial fan deposits (Holocene), consisting of unconsolidated fine- to coarse-grained sand, silt, and gravel, coarser grained at heads of fans and in narrow canyons. Qpaf is described as coarse-grained older alluvial fan and stream terrace deposits (Pleistocene), consisting of poorly consolidated gravel, sand and silt, coarser grained at heads of old fans in narrow canyons. The morphology of Qhaf mapped suggests that the deposits present at Holbrook-Palmer park may have originated from a drainage whose headwaters are located at Bear Gulch Reservoir in the foothills approximately three miles west of the site. Review of historical aerial photos dating back to 1930 suggest a stronger affinity for mature trees along areas mapped as Qhaf. Review of historical topographic maps show the presence of a drainage downstream of Bear Gulch Reservoir.

The subsurface exploration encountered both fine- and coarse-grained soils. The data collected from the site is generally consistent with the implied geologic history, given the transitory nature of alluvial fan deposition. In other words, alternating pulses of fine- and coarse-grained sediment are expected downstream of upland areas and affect locations such as this particular project site.

2.2.2 Faulting and Seismicity

The San Francisco Bay Area contains numerous active faults and is considered seismically active. An active fault is defined by the State as one that has had surface displacement within Holocene time (about the last 11,000 years).

Numerous small earthquakes occur every year in the San Francisco Bay Region, and larger earthquakes have been recorded and can be expected to occur in the future. The project site is not located within a State of California Earthquake Fault Zone for known active faults (State of California, 1982). Based on the 2010 USGS Quaternary Fault and Fold Database (QFFD), the nearest known active fault is the Monte Vista-Shannon fault, located about 3.5 miles south of the study area. The San Andreas fault is located approximately 5.1 miles west of the study area as well. Figure 5 shows the approximate location of active and potentially active faults and significant historic earthquakes mapped within the San Francisco Bay Region. The following table lists the closest mapped active faults and their proximity to the site.

TABLE 2.2.2-1: Regional Faults Site: Latitude = 37.463637; Longitude = -122.191352

FAULT NAME	APPROXIMATE DISTANCE (MILES)	ESTIMATE OF MAXIMUM MAGNITUDE (ELLSWORTH)
Monte Vista-Shannon	3.5	6.5
North San Andreas	5.1	7.9
Hayward-Rodgers Creek	14.0	7.3
San Gregorio Connected	14.6	7.5
Calaveras	19.3	7.0

The 2014 Working Group on California Earthquake Probabilities evaluated the regional seismicity of the Bay Area and published their results as The Uniform California Earthquake Rupture Forecast, Version 3 (UCERF 3). The Working Group periodically attempts to summarize seismic risk in California with time-dependent earthquake rupture forecasts, in which the probabilities of future events are conditioned upon the dates of previous earthquakes. According to UCERF 3, there is an aggregated 72 percent probability of a 6.7 M_W (Moment Magnitude) or greater earthquake on an active Bay Area fault over the next 30 years. The probability of a 6.7 M_W or greater earthquake on the Hayward fault, Calaveras and San Andreas faults are 14, 7, and 6 percent, respectively, over the next 30 years.

The study area is not located within a State of California Seismic Hazard Zones (CGS, 2006) for areas that may be susceptible to liquefaction (Figure 4).

2.3 FIELD EXPLORATION

Our field exploration included advancing six cone penetration tests (CPTs) and drilling six boreholes at various locations on the site. We performed our CPT exploration on March 2, 2018. We observed borehole drilling between March 6, 2018, and March 8, 2018.

We show the exploration locations on the Site Plan, Figure 2. The location and elevations of our explorations are approximate and we estimated them using commercial grade global positioning satellite (GPS) equipment and by pacing from features shown on the Site Plan in Figure 2; however, they should be considered accurate only to the degree implied by the method used.

As indicated on the Site Plan, exploration locations BH01 / 1-CPT01, BH02 / 1-CPT02, and BH03 / 1-CPT03 are associated with the main lawn. Exploration locations BH04 / 1-CPT04 are associated with the entry lawn, and locations BH05 / 1-CPT05 and BH06 / 1-CPT06 are associated with the northern parking lot.

2.3.1 Cone Penetration Tests

On March 2, 2018, we retained the services of a subcontractor with a CPT track rig to perform exploration to a maximum depth of about 50 feet in general accordance with ASTM D-5778. Measurements include the tip resistance to penetration of the cone (Qc), the resistance of the surface sleeve (Fs), and pore pressure (U) (Robertson and Campanella, 1988). We present the CPT data in Appendix B.

2.3.2 Borings

Between March 6, 2018, and March 8, 2018, we observed six borings drilled to a maximum depth of approximately 50 feet. We retained the services of a subcontractor with a truck-mounted drill rig and crew to advance the borings using 4-inch-diameter solid flight auger methods. We obtained bulk soil samples from drill cuttings and retrieved disturbed soil samples at various intervals within the borings using either a 2½-inch-inside-diameter (I.D.) California-type split-spoon sampler fitted with 6-inch-long brass liners or a 2-inch-outside-diameter (O.D.) Standard Penetration Test (SPT) split-spoon sampler. The split-spoon samplers were driven with a 140-pound hammer falling a distance of 30 inches using a rope and pulley system. The penetration of the samples into the soil materials was field recorded as the number of blows needed to drive the sampler 18 inches in 6-inch increments. The boring logs show the number of blows required for the last 1 foot of penetration, and the blow counts reported on the logs have not been converted using any correction factors. We used the field logs to develop the report boring logs presented in Appendix A.

2.4 LABORATORY TESTING

The laboratory test results are included on the borelogs in Appendix A. Individual test results are presented in Appendix C.

2.5 SUBSURFACE CONDITIONS

Based on the soil encountered in the borings and empirical correlations of the CPT data:

- We encountered stiff to hard clayey material to a depth of 20 to 25 feet bgs at locations BH01 / 1-CPT01, BH02 / 1-CPT02, BH03 / 1-CPT03, AND BH04 / 1-CPT4. At these locations, the upper 5 to 10 feet of soil consists of dark brown, high plasticity clay. Approximately 2 feet of the shallow surficial soil may be fill re-worked for agricultural and then recreational purposes. Beneath the fat clay, the soil is light yellowish brown with lower plasticity and varying amounts of silt and fine-grained sand. The lean clay extends to approximately 50 feet bgs at the location of BH02 / 1-CPT02.

- At locations BH01 / 1-CPT01, BH03 / 1-CPT03, and BH04 / 1-CPT04 we encountered alternating layers of medium dense gravels / sands of varying fines content and clays at a depth of 20 to 25 feet bgs.

- At locations BH05 / 1-CPT05 and BH06 / 1-CPT06, the pavement section consists of 3 to 4 inches of asphalt underlain by 7 to 9 inches of aggregate base. Below the pavement section, we encountered up to 1½ feet of fill composed of mottled, very stiff lean clay.

- Below the fill at location BH05 / 1-CPT05, we encountered light brown, very stiff lean clay to 16 feet bgs. Below the clayey material, we encountered medium dense sand that appears to extend to a depth of approximately 35 feet bgs. The CPT logs indicate approximately 10 feet of clayey material below the medium dense sand.

- Below the fill at location BH06 / 1-CPT06, we encountered alternating layers of 2 to 4 feet of hard lean clay and medium dense sand with varying amounts of clay to a depth of approximately 13 feet bgs. Below this, the soil is hard/very dense and consists of approximately 5 feet of lean clay underlain by approximately 7 feet of clayey sand. At a depth of approximately 22 feet bgs, we encountered well-graded, dense sand.

Consult the Site Plan (Figure 2), the boring logs in Appendix A, and the CPT logs in Appendix B for specific subsurface conditions at each exploration location. The boring logs contain the soil type, color, consistency, and visual classification in general accordance with the Unified Soil Classification System. The logs graphically depict the subsurface conditions encountered at the time of the exploration. The CPT logs provide profiles with depth of the total cone resistance (Qt), the resistance of the surface sleeve (Fs), the friction ratio (Rf), the pore pressure (U), and soil behavior type (SBT).

2.6 PERCOLATION TESTING

We performed percolation testing within our borings for the purpose of supporting stormwater facility design. Where the boring depth exceeded approximately 20 feet, we redrilled an adjacent boring to test the shallower soil profile. The percolation holes ranged in depth from10 feet and 22 feet bgs with a 4-inch diameter solid flight auger. At each percolation test location, a vertical 3-inch-diameter PVC drain pipe was temporarily set in place with the lowermost portion being perforated pipe. The annulus along the perforated interval was filled with well sand and the holes were presoaked with water up to the top of the boreholes up to 24 hours before testing. During percolation testing, we measured groundwater levels using a water level meter and/or a vibrating wire piezometer attached to an automated datalogger.

We generally followed the County of Los Angeles GMED guidelines for design, investigation, and reporting low impact development stormwater infiltration (GS200.2 Dated June 30, 2017). The tests were completed within 4 to 7 hours and generally followed the boring percolation test procedure with the exception of the percolation test at BH05, which followed the high flowrate percolation test procedure. Upon completion of testing, the standpipes were removed and the holes were backfilled with grout. The raw data and transducer calibration forms are included in Appendix D.

We processed the raw data following the procedures described in GS200.2. We applied reduction factors that account for horizontal infiltration, type of percolation test, and site variability. We did not apply a reduction factor for long-term maintenance, as described on Page 14 of GS200.2.

The adjusted infiltration rates and testing information are summarized in Table 2.6-1.

TABLE 2.6-1: Percolation Testing Results

Boring	Date Tested	Test Depth (ft, below ground surface)	Overall Duration (mins)	Height of Water at Start (ft)	Soil Type	Stabilized Infiltration Rate* (in/hr)
BH01	3/7/2018	12	140	10.6	lean clay	0.03
BH02	3/8/2018	18	229	7.6	lean clay	0.02
BH03	3/7/2018	10	318	10.1	clay, some gravel	0.05
BH04	3/7/2018	17	255	13.2	sandy clay	0.03
BH05	3/8/2018	21.5	129	3.2	sand, some gravel, minor clay	18
BH06	3/8/2018	20	358	4.8	clayey sand	0.2

*using calculations in Los Angeles County GS200.2 Dated June 30, 2017

2.7 GROUNDWATER CONDITIONS

We observed static groundwater in several of our subsurface explorations. We summarize our observations in the table below:

TABLE 2.7-1: Groundwater Observations

EXPLORATION LOCATION	APPROX. DEPTH TO GROUNDWATER (FEET)	APPROX. GROUNDWATER ELEVATION (FEET)**
1-CPT01*	30	17
BH01	Not measured	Not measured
1-CPT02*	29	17
BH02	29	17
1-CPT03*	29	17
BH03	Not measured	Not measured
1-CPT04*	29	15

EXPLORATION LOCATION	APPROX. DEPTH TO GROUNDWATER (FEET)	APPROX. GROUNDWATER ELEVATION (FEET)**
BH04	Not measured	Not measured
1-CPT05*	29	15
BH05	Not measured	Not measured
1-CPT06*	29	14
BH06	Not measured	Not measured

* Determined using Pore Pressure Dissipation test
** WGS84 datum

Fluctuations in the level of groundwater may occur due to variations in rainfall, irrigation practice, and other factors not evident at the time measurements were made.

In addition to in-situ groundwater measurements, we reviewed the historic high groundwater map published by CGS (Figure 6). The historic high groundwater level varies between approximately 23 and 28 feet bgs from northeast to southwest.

3.0 CONCLUSIONS

From a geotechnical engineering viewpoint, in our opinion, the proposed project may be designed as planned, provided the geotechnical recommendations in this report are properly incorporated into the design plans and specifications.

The primary geotechnical concerns that could affect development on the site are summarized in our conclusions below:

- Existing fill
- Expansive soil
- Seismic hazards, in particular liquefaction-induced settlement
- Deep excavations

We also evaluated the site for flooding hazard and soil corrosion potential, and provide 2016 CBC seismic design considerations below.

3.1 EXISTING FILL

As discussed in Section 2.5, the borings indicate that the main lawn, entry lawn and parking lot portions of the site are underlain by up to approximately 2 feet of non-engineered fill.

Non-engineered fill can undergo excessive settlement, especially under new fill or structural loads. Without proper documentation of existing fill placed on the site, we recommend complete removal and recompaction of the existing fill. It is our understanding that any existing fill within the footprint of the underground infiltration system will be removed as part of the excavation operations, however at-grade ancillary buildings and other surface improvements may be planned. We present fill removal recommendations in Section 4.0.

3.2 EXPANSIVE SOIL

Our laboratory testing indicates potentially expansive clays are located onsite. Plasticity Index (PI) data indicates site soils may exhibit moderate to high shrink/swell potential with variations in moisture content. Specifically, the near-surface soil (approximately upper 5 feet) is highly expansive with a PI of approximately 39. Soils below this depth are indicated to be moderately expansive with PI ranging between 11 to 18.

Expansive soil changes in volume with changes in moisture. They can shrink or swell and cause heaving and cracking of slabs-on-grade, pavements, and any other structures founded within a zone of wetting and drying. Damage to structures due to volume changes associated with expansive soil can be reduced by deepening the foundations to below the zone of moisture fluctuation, i.e. by using deep footings or drilled piers and/or founding structures on a level pad of non-expansive soil. We provide recommendations for foundation subgrade preparation and grading recommendations to reduce the expansion potential of the near-surface soil in Sections 4.0 and 5.0.

3.3 SEISMIC HAZARDS

Potential seismic hazards resulting from a nearby moderate to major earthquake can generally be classified as primary and secondary. The primary effect is ground rupture, also called surface faulting. The common secondary seismic hazards include ground shaking, soil liquefaction, and lateral spreading. These hazards are discussed in the following sections.

Based on topographic and lithologic data, the risk from regional subsidence or uplift, tsunamis, landslides and seiches is considered low at the site.

3.3.1 Ground Rupture

As previously discussed, the site is not located within a State of California Earthquake Fault Hazard Zone (1982). Fault rupture is unlikely within the limits of the project.

3.3.2 Ground Shaking

An earthquake of moderate to high magnitude generated within the San Francisco Bay Region, similar to those that have occurred in the past, could cause considerable ground shaking at the site. To mitigate the shaking effects, all structures should be designed using sound engineering judgment and the latest California Building Code (CBC) requirements as a minimum. Seismic design provisions of current building codes generally prescribe minimum lateral forces, applied statically to the structure, combined with the gravity forces of dead and live loads. The code-prescribed lateral forces are generally substantially smaller than the expected peak forces that would be associated with a major earthquake. Therefore, structures should be able to: (1) resist minor earthquakes without damage, (2) resist moderate earthquakes without structural damage but with some nonstructural damage, and (3) resist major earthquakes without collapse but with some structural as well as nonstructural damage. Conformance to the current building code recommendations does not constitute any kind of guarantee that significant structural damage would not occur in the event of a maximum magnitude earthquake; however, it is reasonable to expect that a well-designed and well-constructed structure will not collapse or cause loss of life in a major earthquake (SEAOC, 1996).

3.3.3 Liquefaction/Clay Soil Softening

Soil liquefaction results from loss of strength during cyclic loading, such as imposed by earthquakes. Soil most susceptible to liquefaction is clean, loose, saturated, uniformly graded, fine-grained sand. Empirical evidence indicates that loose to medium dense gravel, silty sand, low-plasticity silt, and some low-plasticity clay are also potentially liquefiable. In order for a soil to be potentially liquefiable, it must be saturated. For this site we considered the design groundwater depth to be at 23 feet, which corresponds with the historic high groundwater level described in Section 2.7.

We analyzed the potential for liquefaction using the CPT data with the software program CLiq (Version 1.7.6.34) applying the methodologies published by Idriss and Boulanger (2008). We used the peak ground acceleration (PGA) associated with the Maximum Considered Earthquake (MCE) event, 0.65g. We used a 8.0 Moment Magnitude earthquake, since the San Andreas fault controls the MCE earthquake shaking at the site. We present our liquefaction analysis in Appendix E of this report. The results generally indicate that the medium dense layers of clayey gravel and sand layers 25 to 30 feet below existing ground surface are potentially liquefiable. The liquefiable layers have a maximum thickness of approximately 8 feet.

Results of our analysis of the combined sand-like and clay-like settlements are provided in the table below. Based on our analysis, it is our opinion that for the MCE-level earthquake, the site may experience up to 2 inches of total liquefaction-induced settlement. Our results are presented in Appendix E of this report.

TABLE 3.3.3-1: Liquefaction-Induced Settlement

EXPLORATION DESIGNATION	ESTIMATED SETTLEMENT DUE TO LIQUEFACTION (inches)
1-CPT1	0.5
1-CPT2	0.1
1-CPT3*	0.2
1-CPT4	1.7
1-CPT5	1.8
1-CPT6*	0.2

* Evaluated to a depth of 30 feet. All other locations evaluated to a depth of 50 feet.

Based on the high end of the calculated total liquefaction settlements presented in the table above, the site improvements should be designed to withstand a differential settlement of 1 inch over a 50-foot distance and perform as intended.

3.3.4 Liquefaction-Induced Surface Rupture

In addition to the above liquefaction analysis, we also evaluated the capping effect of any overlying non-liquefiable soils. In order for liquefaction-induced ground failure to occur, the pore water pressure generated within the liquefied strata must exert a force sufficient to break through the overlying soil and vent to the surface resulting in sand boils or fissures. We based our analyses and review on guidelines provided by Ishihara (1985) and Youd and Garris (1995).

Proposed improvements founded at grade will have a thick enough layer of non-liquefiable soil above the liquefiable soil to provide a capping effect to prevent manifestation of liquefaction.

We understand that the underground storage will be founded at approximately 18 feet bgs, with the wet well / lift station a further five to seven feet below the storage structure. According to Ishihara (1985) and Youd and Garris (1995), the layers of non-liquefiable soil above the liquefiable soil are thick enough to provide a capping effect to prevent manifestation of liquefaction at the base of the underground storage structure.

The layers of non-liquefiable soil are also sufficiently thick to provide a cap at the base of the wet well / lift station at locations 1-CPT1 and 1-CPT2 at the main lawn. At location 1-CPT3, thin (< 4 inches) layers of sandy soil may liquefy at depths of 23 feet, 24 ½ feet, and 26 feet bgs. At location 1-CPT4, a 2-feet-thick layer of sandy soil may liquefy at a depth of 25 feet bgs. In the parking lot at location 1-CPT5, up to 1 foot of sandy soil may liquefy at a depth of 23 feet bgs, up to 2 feet of soil may liquefy at 25 feet bgs and up to 4 feet of soil may liquefy at 28 feet bgs. At 1-CPT 6, up to 1 foot of sandy soil may liquefy at 23 feet bgs and 24 feet bgs. We should further evaluate

3.3.5 Lateral Spreading

Lateral spreading involve lateral ground movements caused by seismic shaking. These lateral ground movements are often associated with a weakening or failure of an embankment or soil mass overlying a continuous layer of liquefied sand or weak soils.

The adjacent Atherton Channel may act as a free face. However, the potentially susceptible liquefiable layers appear discontinuous and are below the bottom of the Atherton Channel, Therefore, lateral spreading is considered a low risk in our opinion.

3.4 FLOODING

The project site is mapped within Zone X on the Federal Emergency Management Agency (FEMA 2015) Flood Hazard Map for Atherton, indicating that it is within an area determined to be outside the 0.2 percent annual chance floodplain. The Tsunami Inundation Map for the Redwood Point Quadrangle (CGS, 2009) maps the tsunami inundation line as close as 2 miles northwest of the project site. The project Civil Engineer should review pertinent information relating to possible flood levels for the subject site based on final pad elevations and provide appropriate design measures for development of the project, if necessary.

3.5 SOIL CORROSION POTENTIAL

As part of this study, we obtained two representative soil samples and submitted to a qualified analytical lab for determination of redox, pH, resistivity, sulfide sulfate, and chloride. The results are included in Appendix C and summarized in the table below.

TABLE 3.5-1: Corrosivity Test Results

SAMPLE LOCATION	DEPTH	REDOX (mV)	PH	RESISTIVITY (OHMS-CM)	SULFIDE (MG/KG)	CHLORIDE (MG/KG)	SULFATE (MG/KG)
BH03	0' – 2'	460	7.92	820	N.D.	28	20
BH05	8½' – 10'	450	8.19	1,200	N.D.	N.D.	N.D.

* pH according to ASTM D4972, Resistivity according to ASTM G57, Chloride and Sulfate according to ASTM D4327

** N.D., None detected

The 2016 CBC references the 2014 American Concrete Institute Manual, ACI 318-14, Section 19.3.1 for concrete durability requirements. ACI Table~19.3.1.1 provides the following exposure categories and classes, and Table 19.3.2.1 provides requirements for concrete in contact with soil based upon the exposure class.

TABLE 3.5-2: ACI Table 19.3.1.1: Exposure Categories and Classes

CATEGORY	SEVERITY	CLASS	CONDITION	
F Freezing and thawing	Not Applicable	F0	Concrete not exposed to freezing-and-thawing cycles	
	Moderate	F1	Concrete exposed to freezing-and-thawing cycles and occasional exposure to moisture	
	Severe	F2	Concrete exposed to freezing-and-thawing cycles and in continuous contact with moisture	
	Very Severe	F3	Concrete exposed to freezing-and-thawing cycles and in continuous contact with moisture and exposed to deicing chemicals	
			WATER- SOLUBLE SULFATE IN SOIL % BY WEIGHT*	**DISSOLVED SULFATE IN WATER MG/KG (PPM)****
S Sulfate	Not applicable	S0	$SO_4 < 0.10$	$SO_4 < 150$
	Moderate	S1	$0.10 \leq SO_4 < 0.20$	$150 \leq SO_4 \leq 1{,}500$ seawater
	Severe	S2	$0.20 \leq SO_4 \leq 2.00$	$1{,}500 \leq SO_4 \leq 10{,}000$
	Very severe	S3	$SO_4 > 2.00$	$SO_4 > 10{,}000$
			CONDITION	
P Requiring low permeability	Not applicable	P0	In contact with water where low permeability is not required.	
	Required	P1	In contact with water where low permeability is required.	
C Corrosion protection of reinforcement	Not applicable	C0	Concrete dry or protected from moisture	
	Moderate	C1	Concrete exposed to moisture but not to external sources of chlorides	
	Severe	C2	Concrete exposed to moisture and an external source of chlorides from deicing chemicals, salt, brackish water, seawater, or spray from these sources	

* Percent sulfate by mass in soil determined by ASTM C1580
** Concentration of dissolved sulfates in water in ppm determined by ASTM D516 or ASTM D4130

In accordance with the criteria presented in the above table, these soils are categorized as being within the F0 freeze-thaw class, S0 sulfate exposure class, P0 exposure class and C1 corrosion class. Cement type, water-cement ratio, and concrete strength, are not specified for these ranges.

Considering a 'Not Applicable' sulfate exposure, there is no requirement for cement type or water-cement ratio, however, a minimum concrete compressive strength of 2,500 psi is specified by the building code. For this sulfate range, we recommend Type II cement and a concrete mix design for foundations and building slabs-on-grade that incorporates a maximum water-cement ratio of 0.50. It should be noted, however, that the structural engineering design requirements for concrete may result in more stringent concrete specifications.

Based on the resistivity measurements, the soils are considered highly corrosive to extremely corrosive to buried metal piping. Values tested for chloride do not pose a significant impact to metals or concrete.

If desired to investigate this further, we recommend a corrosion consultant be retained to evaluate if specific corrosion recommendations are advised for the project. Note that ASTM Test Method D4327 was used in lieu of the ACI designated sulfate test methods as it provides better test results.

3.6 2016 CBC SEISMIC DESIGN PARAMETERS

The 2016 CBC utilizes design criteria set forth in the 2010 ASCE 7 Standard. Based on the subsurface conditions encountered, we characterized the site as Site Class D in accordance with the 2016 CBC. We provide the 2016 CBC seismic design parameters in Table 3.6-1 below, which include design spectral response acceleration parameters based on the mapped Risk Targeted Maximum Considered Earthquake (MCER) spectral response acceleration parameters.

TABLE 3.6-1: 2016 CBC Seismic Design Parameters, Latitude = 37.4636; Longitude = -122.1914

PARAMETER	VALUE
Site Class	D
Mapped MCE_R Spectral Response Acceleration at Short Periods, S_S (g)	1.60
Mapped MCE_R Spectral Response Acceleration at 1-second Period, S_1 (g)	0.74
Site Coefficient, F_A	1.00
Site Coefficient, F_V	1.50
MCE_R Spectral Response Acceleration at Short Periods, S_{MS} (g)	1.60
MCE_R Spectral Response Acceleration at 1-second Period, S_{M1} (g)	1.10
Design Spectral Response Acceleration at Short Periods, S_{DS} (g)	1.07
Design Spectral Response Acceleration at 1-second Period, S_{D1} (g)	0.74
Mapped MCE Geometric Mean (MCE_G) Peak Ground Acceleration, PGA (g)	0.65
Site Coefficient, F_{PGA}	1.00
MCE_G Peak Ground Acceleration adjusted for Site Class effects, PGA_M (g)	0.65
Long period transition-period, T_L	12 sec

3.7 INFILTRATION CONSIDERATIONS

The geotechnical explorations in the main lawn and the entry lawn areas generally indicate clayey material to a depth of at least 20 to 25 feet bgs. The clayey material extends to approximately 50 feet bgs at the location of BH02 / 1-CPT02. The calculated infiltration rate of the clayey material varies between 0.1 in/hr and 0.4 in/hr. Based on results of percolation testing, we do not recommend relying on infiltration in design at these locations.

At locations BH05 / 1-CPT05 and BH06 / 1-CPT06 we encountered sandy material below a depth of approximately 16 to 17 feet. At BH05, the sandy material encountered between 16 and 22 feet bgs was medium dense to dense, had low fines content and demonstrated a calculated infiltration rate of approximately 18 in/hr during percolation testing.

The clayey sand material at BH06 encountered from 17 to 22 feet bgs had higher fines content than at BH05 and was very dense. At these depths, the material demonstrated a calculated infiltration rate of approximately 0.2 in/hr. Below a depth of approximately 22 feet bgs, we encountered medium dense to dense sand with low fines content, similar to the material encountered at BH05 between 16 and 22 feet. If the parking lot option is considered for the water quality treatment facility and infiltration is incorporated into design, we recommend targeting the sandy material encountered at 17 feet bgs in BH05 and 22 feet bgs in BH06. This may require some overexcavation depending on final design.

4.0 EARTHWORK RECOMMENDATIONS

The following recommendations relate to site demolition and stripping, processing and compaction. Recommendations regarding temporary excavations and shoring are provided in Section 7.

The relative compaction and optimum moisture content of soil and aggregate base referred to in this report are based on the most recent ASTM D1557 test method. Compacted soil is not acceptable if it is unstable. It should exhibit only minimal flexing or pumping, as observed in the field by our representative.

As used in this report, the term “moisture condition” refers to adjusting the moisture content of the soil by either drying if too wet or adding water if too dry.

We define “structural areas” as any area sensitive to settlement of compacted soil. These areas include, but are not limited to building pads, sidewalks, pavement areas, and retaining walls.

4.1 DEMOLITION AND STRIPPING

Site improvement should commence with the removal of buried structures, including abandoned utilities and septic tanks and their leach fields, if any exist. All debris should be removed from any location to be graded, from areas to receive fill or structures, or those areas to serve as borrow. The depth of removal of such materials should be determined by a representative of our firm in the field at the time of grading.

Existing vegetation and pavements (asphalt concrete/concrete and underlying aggregate base) should be removed from areas to receive fill, or structures, or those areas to serve for borrow. Subject to approval by the Landscape Architect, strippings and organically contaminated soil can be used in landscape areas.

All excavations from demolition and stripping below design grades should be cleaned to a firm undisturbed soil surface determined by us. This surface should then be scarified to a depth of 12 inches, moisture conditioned, and backfilled with compacted engineered fill. The requirements for backfill materials and placement operations are the same as for engineered fill.

No loose or uncontrolled backfilling of depressions resulting from demolition and stripping is permitted.

4.2 NON-ENGINEERED FILL

We encountered up to 2 feet of existing non-engineered fill throughout the site. If existing fill is encountered during grading, it should be treated as unsuitable to remain below proposed structures and should be subexcavated to expose underlying competent native soil that is approved by a representative of our firm. The base of the excavations should be processed, moisture conditioned, as needed, and compacted in accordance with the subsequent recommendations for engineered fill.

4.3 EXPANSIVE SOIL MITIGATION

Successful performance of structures on expansive soil requires special attention during construction. It is imperative that exposed soils be kept moist prior to placement of concrete for foundation construction. It can be difficult to remoisturize clayey soil without excavation, moisture conditioning, and recompaction.

We provide specific grading recommendations for compaction of clay soil at the site. The purpose of these recommendations is to reduce the swell potential of the clay by compacting the soil at a high moisture content and controlling the amount of compaction.

4.4 OVER-OPTIMUM SOIL MOISTURE CONDITIONS

The contractor should anticipate encountering excessively over-optimum (wet) soil moisture conditions during winter or spring grading, during or following periods of rain, and below a depth of approximately 23 feet bgs. Wet soil can make proper compaction difficult or impossible. Wet soil conditions can be mitigated by:

1. Frequent spreading and mixing during warm dry weather.
2. Mixing with drier materials.
3. Mixing with a lime, lime-flyash, or cement product; or
4. Stabilizing with aggregate, geotextile stabilization fabric, or both.

Options 3 and 4 should be evaluated by ENGEO prior to implementation.

4.5 ACCEPTABLE FILL

Onsite soil material is suitable as fill material provided it is processed to remove concentrations of organic material, debris, and particles greater than 8 inches in maximum dimension.

Imported fill materials should meet the above requirements and have a plasticity index similar to onsite soil materials. We should be given the opportunity to sample and test proposed imported fill materials at least 5 days prior to delivery to the site.

4.6 FILL COMPACTION

4.6.1 Grading in Structural Areas

The contractor should perform subgrade compaction prior to fill placement, following cutting operations, and in areas left at grade as follows.

1. Scarify to a depth of at least 12 inches.
2. Moisture condition soil to at least 4 percentage points over the optimum moisture content; **and**
3. Compact the soil to 90 percent relative compaction. Compact the upper 6 inches of finish pavement subgrade to at least 92 percent relative compaction prior to aggregate base placement.

After the subgrade has been compacted, the contractor should place and compact acceptable fill as follows:

1. Spread fill in loose lifts that do not exceed 12 inches.
2. Moisture condition lifts to at least 4 percentage points over the optimum moisture content; **and**
3. Compact fill to 90 percent relative compaction; compact the upper 6 inches of fill in curb, gutter and driveway areas to at least 92 percent relative compaction prior to aggregate base placement.

The contractor should also compact the pavement Caltrans Class 2 Aggregate Base section to at least 95 percent relative compaction and moisture condition aggregate base to a minimum moisture content of optimum prior to compaction.

4.6.2 Underground Utility Backfill

4.6.2.1 General

The contractor is responsible for conducting trenching and shoring in accordance with CALOSHA requirements. Project consultants involved in utility design should specify pipe-bedding materials.

4.6.2.2 Structural Areas

The contractor should place and compact trench backfill as follows:

1. Trench backfill should have a maximum particle size of 6 inches.
2. Moisture condition trench backfill to at least 4 percentage points above the optimum moisture content. Moisture condition backfill outside the trench.
3. Place fill in loose lifts not exceeding 12 inches;
and
4. Compact fill to 90 percent relative compaction.

Jetting of backfill is not an acceptable means of compaction. We may allow thicker loose lift thicknesses based on acceptable density test results, where increased effort is applied to rocky fill or for the first lift of fill over pipe bedding.

4.6.3 Landscape Fill

The contractor should process, place and compact fill in accordance with the recommendations in Section 4.0 except compact to at least 85 percent relative compaction.

5.0 FOUNDATION RECOMMENDATIONS

We developed structural improvement recommendations using data obtained from our field exploration, laboratory test results, and engineering analysis. Foundations should be designed for 1 inch of differential movement over a distance of 50 feet for the seismic case, as described in Section 3.3.3.

It is our understanding that the below grade water quality treatment facility will consist of precast concrete structures, with the bottom open to allow incidental infiltration at the base. We assume that the bottom of the concrete structures will be located approximately 18 feet bgs.

We understand that a 96-inch-diameter wet well / lift station will also be constructed extending to a depth of 23 to 25 feet bgs.

We anticipate small, ancillary buildings such as a water quality treatment building may also be constructed at grade.

Recommendations for the temporary and permanent bridge structures are provided in Section 8.0.

5.1 FOUNDATION RECOMMENDATIONS

5.1.1 Subsurface Water Quality Structure

We recommend the precast concrete structures be supported on continuous strip footings. We recommend the footings be designed for an allowable bearing value of 4,000 pounds per square foot (psf) and be supported on competent non-yielding native material as determined by the Geotechnical Engineer or his/her representative in the field at the time of construction. Increase this bearing capacity by one-third for the short-term effects of seismic loading. We recommend footings have a minimum width of 18 inches.

The maximum allowable bearing pressure is a net value; the weight of the footing may be neglected for design purposes. All footings located adjacent to utility trenches should have their bearing surfaces below an imaginary 1:1 (horizontal:vertical) plane projected upward from the bottom edge of the trench to the footing.

The bottom of the subsurface water quality structure between footing foundations should be underlain by a 12- to 18-inch-thick layer of ¾- to 1½-inch minus clean crushed rock underlain by approved geotextile stabilization fabric (Mirafi 600X or approved equivalent).

Once a manufacturer of the precast concrete structures is determined, we should review the above recommendations. Additional recommendations and/or manufacturer recommendations may be incorporated

5.1.2 Wet Well / Lift Station

We understand that a 96-inch-diameter wet well / lift station will also be constructed extending to a depth of 23 to 25 feet bgs.

We recommend a conventionally reinforced structural mat be used for the below-grade sewer lift station. The mat foundation should be reinforced with top and bottom steel as determined by the structural engineer to provide structural continuity. The mat foundation should be designed for an allowable uniform soil pressure of 4,000 pounds per square foot (psf). Increase this bearing capacity by one-third for the short-term effects of seismic loading.

We recommend the bottom of the mat be underlain by a 12-inch-thick layer of ¾” clean crushed rock underlain by approved geotextile stabilization fabric (Miraf 600X or approved equivalent). The stabilization fabric should be wrapped and placed above the bedding section. A layer of 6-ounce filter fabric may alternatively be used on top of the bedding section.

5.1.3 At-Grade Ancillary Buildings

We recommend proposed at-grade structures be founded on continuous footings with an interior slab-on-grade underlain by a minimum of 24 inches of non-expansive engineered fill.

We provide minimum footing dimensions as follows in the table below.

TABLE 5.1.3-1

Minimum Footing Dimensions

Footing Type	*Minimum Depth (inches)	Minimum Width (inches)
Continuous	24	12
Isolated	24	18

*below lowest adjacent pad grade

Minimum footing depths shown above are taken from lowest adjacent finished grade. The cold joint between the exterior footing and slab-on-grade should be located at least 4 inches above adjacent exterior grade.

Design foundations recommended above for a maximum allowable bearing pressure of 2,500 pounds per square foot (psf) for dead-plus-live loads. Increase this bearing capacity by one-third for the short-term effects of wind or seismic loading.

The maximum allowable bearing pressure is a net value; the weight of the footing may be neglected for design purposes. All footings located adjacent to utility trenches should have their bearing surfaces below an imaginary 1:1 (horizontal:vertical) plane projected upward from the bottom edge of the trench to the footing.

5.2 FOUNDATION LATERAL RESISTANCE

Lateral loads may be resisted by friction along the base and by passive pressure along the sides of foundations. The passive pressure is based on an equivalent fluid pressure in pounds per cubic foot (pcf). We recommend the following allowable values for design:

- Passive Lateral Pressure: 300 pcf above groundwater level, 200 pcf below groundwater level
- Coefficient of Friction: 0.30

The above allowable values include a factor of safety of 1.5. Increase the above values by one-third for the short-term effects of wind or seismic loading.

6.0 SLABS-ON-GRADE

6.1 INTERIOR CONCRETE FLOOR SLABS

6.1.1 Minimum Design Section

We recommend the following minimum design:

1. Consider a minimum concrete thickness of 5 inches.

2. Consider minimum steel reinforcing of No. 3 rebar on 18-inch centers each way within the middle third of the slab to help control the width of shrinkage cracking that inherently occurs as concrete cures.

The structural engineer should provide final design thickness and additional reinforcement, as necessary, for the intended structural loads.

6.1.2 Slab Moisture Vapor Reduction

When buildings are constructed with concrete slab-on-grade, water vapor from beneath the slab will migrate through the slab and into the building. This water vapor can be reduced but not stopped. Vapor transmission can negatively affect floor coverings and lead to increased moisture within a building. When water vapor migrating through the slab would be undesirable, we recommend the following to reduce, but not stop, water vapor transmission upward through the slab-on-grade.

1. Construct a moisture retarder system directly beneath the slab-on-grade that consists of the following:

   a. Vapor retarder membrane sealed at all seams and pipe penetrations and connected to all footings. Vapor retarders shall conform to Class A vapor retarder per ASTM E 1745-97 “Standard Specification for Plastic Water Vapor Retarders used in Contact with Soil or Granular Fill under Concrete Slabs.” The vapor retarder should be **underlain by**

   b. 4 inches of clean crushed rock. Crushed rock should have 100 percent passing the ¾-inch sieve and less than 5 percent passing the No. 4 Sieve; **underlain by**

   c. 24 inches of non-expansive (PI<12) fill. The 4 inches of clean crushed rock can be considered part of the 24 inches of nonexpansive fill.

2. Use a concrete water-cement ratio for slabs-on-grade of no more than 0.50.

3. Provide inspection and testing during concrete placement to check that the proper concrete and water cement ratio are used.

4. Moist cure slabs for a minimum of 3 days or use other equivalent curing specific by the structural engineer.

The structural engineer should be consulted as to the use of a layer of clean sand or pea gravel (less than 5 percent passing the U.S. Standard No. 200 Sieve) placed on top of the vapor retarder membrane to assist in concrete curing.

6.2 SECONDARY SLABS

This section provides guidelines for secondary slabs such as exterior walkways, steps, and sidewalks. As a minimum, we provide the following considerations and recommendations.

- Secondary slabs-on-grade should be constructed structurally independent of adjacent foundation systems. This allows slab movement to occur with a reduced potential for foundation distress.

- Cracking of conventional slabs should be expected as a result of concrete shrinkage and the expansive soils at the site. Frequent control joints should be provided to control the cracking.

- More critical and/or heavily loaded slabs-on-grade should be reinforced for control of cracking. Such reinforcement should be designed by the Structural Engineer. In our experience, welded wire mesh may not be sufficient to control slab cracking.

- Slabs should have a minimum thickness of 4 inches and should slope away from the buildings at a slope of at least 2 percent to prevent water from flowing toward the building. More critical and/or heavily loaded slabs-on-grade may require additional thickness.

- It is critical that uniformity in soil moisture conditioning be achieved in subgrade soils in accordance with recommendations provided above for engineered fill, and that subgrade soils are not allowed to dry out prior to slab construction.

We recommend secondary slabs-on-grade be underlain by a 4-inch-thick layer of clean crushed rock or gravel. Aggregate base meeting CalTrans Class 2 requirements is also acceptable. Turned down free edges extending at least 2 inches beneath the crushed rock or gravel into compacted soil may be considered adjacent to landscape areas to reduce water infiltration into subgrade soils. Waterproof barriers may also be considered.

7.0 BELOW GRADE WALLS

As the water quality treatment structure and wet well / lift station are below ground, the walls of both structures act as retaining walls and should be designed to resist lateral earth pressures. We provide recommendations for design pressures, drainage and backfill for these structures below. Separate recommendations for temporary excavations and shoring are provided in Section 7.0.

7.1 LATERAL SOIL PRESSURES

Design proposed retaining walls to resist lateral earth pressures from adjoining natural materials and/or backfill and from any surcharge loads. Provided that adequate drainage is included as recommended below, design walls restrained from movement at the top to resist an equivalent fluid pressure of 60 pounds per cubic foot (pcf). In addition, design restrained walls to resist an additional uniform pressure equivalent to one-half of any surcharge loads applied at the surface.

Design unrestrained (yielding) retaining walls with adequate drainage to resist an equivalent fluid pressure of 40 pcf plus one-third of any surcharge loads.

The above lateral earth pressures assume level backfill conditions and sufficient drainage behind the walls to prevent any build-up of hydrostatic pressures from surface water infiltration and/or a rise in the groundwater level. If adequate drainage is not provided, we recommend that an additional equivalent fluid pressure of 40 pcf be added to the values recommended above for both restrained and unrestrained walls. Damp-proofing of the walls should be included in areas where wall moisture would be problematic.

Construct a drainage system, as recommended below, to reduce hydrostatic forces behind the retaining wall.

7.1.1 Wall Seismic Design

Seismic conditions should be considered in the design of the below grade retaining walls. Under seismic conditions, the active incremental seismic force along the face of a retaining wall should be added to the static active pressures, and can be calculated as follows:

$$\Delta P = 14 \times H^2$$

H is the design height of the wall (in feet) and ΔP is the active incremental seismic force in pounds per foot of wall. This force has a horizontal direction and should be applied at 1/3 x H from the base of the wall. This force should be combined with the appropriate active equivalent pressure, regardless if the wall is cantilevered or restrained.

7.2 RETAINING WALL DRAINAGE

Construct either graded rock drains or geosynthetic drainage composites behind the retaining walls to reduce hydrostatic lateral forces. For rock drain construction, we recommend two types of rock drain alternatives:

1. A minimum 12-inch-thick layer of Class 2 Permeable Filter Material (Caltrans Specification 68-2.02F) placed directly behind the wall, or

2. A minimum 12-inch-thick layer of washed, crushed rock with 100 percent passing the ¾-inch sieve and less than 5 percent passing the No. 4 sieve. Envelop rock in a minimum 6-ounce, nonwoven geotextile filter fabric.

For both types of rock drains:

1. Place the rock drain directly behind the walls of the structure.

2. Extend rock drains from the wall base to within 12 inches of the top of the wall.

3. Place a minimum of 4-inch-diameter perforated pipe (glued joints and end caps) at the base of the wall, inside the rock drain and fabric, with perforations placed down.

4. Place pipe at a gradient at least 1 percent to direct water away from the wall by gravity to a drainage facility.

ENGEO should review and approve geosynthetic composite drainage systems prior to use.

7.3 BACKFILL

Backfill behind retaining walls should be placed and compacted in accordance with Section 4.0. Use light compaction equipment within 5 feet of the wall face. If heavy compaction equipment is used, the walls should be temporarily braced to avoid excessive wall movement.

8.0 TEMPORARY EXCAVATIONS AND SHORING

We understand that the water quality treatment facility and wet well / lift station, with a depth up to 25 feet below ground level, are planned beneath the main lawn. The limits of the improvements may require excavation directly adjacent to the channel and parallel Watkins Avenue to the northwest and/or the train tracks to the southwest, and will require either temporary cut slopes or shoring.

Excavation, dewatering and shoring are temporary works that are typically the responsibility of the contractor to design, install, maintain and monitor, and should be in conformance with applicable OSHA Excavation and Trench Safety Standards. The Contractor should be familiar with applicable local, state, and federal regulations, including the current OSHA Excavation and Trench Safety Standards. It is the responsibility of the Contractor to provide stable, safe trench and construction slope conditions and to follow OSHA safety requirements. Since excavation procedures may be dangerous, it is also the responsibility of the Contractor to provide a trained "competent person" as defined by OSHA to supervise all excavation operations, ensure that all personnel are working in safe conditions and have thorough knowledge of OSHA excavation safety requirements.

8.1 TEMPORARY EXCAVATIONS

Shoring design should consider lateral earth pressures, surcharge due to adjacent buildings, vehicle loads, and construction-related activities. The design of the shoring should be sufficiently rigid to prevent detrimental movement of the temporary shoring and possible damage of adjacent structures, facilities or other improvements. Where possible, temporary construction slopes may be used. The soil at the site is considered to be "Type B" soil and, as such, temporary slopes should be no steeper than 1:1. The contractor should establish appropriate setback distances from the top of the slope for vehicles, equipment and spoil piles, and should establish appropriate protective measures for exposed slope faces.

8.2 TEMPORARY SHORING

We anticipate excavations up to 25 feet deep for the water quality treatment facility and wet well / lift station, potentially with temporary cut slopes up to 1:1 (horizontal:vertical) or flatter constructed above the shoring system to reduce the total shored height. At this time, we anticipate the temporary shoring will consist of drilled or driven soldier piles with lagging.

Applicable loading, including surcharges due to traffic, buildings, stockpiles, construction equipment, etc. should be incorporated into shoring design as a uniform, horizontal surcharge load (in units of pounds per square foot) of 50 percent of the vertical surcharge load. The horizontal load should be assumed to act below a 1:1 line of projection from the surcharge loading to the temporary shoring. Appropriate safety factors against overturning and sliding should also be incorporated into the design calculations.

We anticipate that the final temporary shoring design will be based on the contractor's means and methods of construction, including equipment and available shoring materials, as well as other general conditions defined by the project team. Recommendations for a temporary soldier pile and lagging shoring system are provided below.

8.2.1 Temporary Soldier Pile and Lagging System

We recommend the following design parameters be used.

TABLE 7.2.1-1: Temporary Soldier Pile and Lagging Shoring Design Parameters

Temporary Shoring Design Element	Design Parameter
Active Earth Pressure:	40 pcf (Level backfill conditions) Active earth pressures should be used where existing buildings and critical utilities are situated outside a 1:1 line of projection extending up from the bottom of the wall
At-Rest Earth Pressure:	60 pcf (Level backfill conditions) At-rest earth pressures should be used where existing buildings and critical utilities are situated within a 1:1 line of projection extending up from the bottom of the wall
Passive Earth Pressure:	300 pcf above design groundwater levels, and 200 pcf below design groundwater levels, acting on two times the pier diameter provided the soldier pile is backfilled with structural concrete, if drilled. This value may be increased by ⅓ when considering seismic loads.

8.3 DEWATERING CONSIDERATIONS

Groundwater was encountered at a depth of approximately 29 feet during our field exploration. Groundwater data in the site vicinity indicates a historic high groundwater depth of as shallow as approximately 23 feet. Temporary dewatering during construction may be necessary.

Assessment of dewatering and subsurface water migration and rates should be made during initial construction excavation procedures to determine the level of groundwater control and dewatering necessary.

We anticipate that groundwater may be removed using sumps, pumps, or other methods. The water level at excavation locations should be maintained at least 2 feet below the bottom of the excavations. The selection of equipment and method should be determined by the contractor. The dewatering system implemented should be selected so as to have minimal impact on the groundwater level surrounding the proposed excavations. The dewatering system should be designed to prevent pumping soil fines with the discharge water. Uncontrolled dewatering could cause settlement of the general area and affect existing improvements in the vicinity of the site.

Groundwater management including temporary storage in Baker tanks (or similar) and testing may be required by the Regional Water Quality Control Board or other agency(s) prior to discharge of generated water. Requirements of potential receiving facilities should be determined in advance of construction. Impacted groundwater may require discharge to a specialty facility.

9.0 PRELIMINARY BRIDGE STRUCTURAL SUPPORT

We understand a temporary bridge may be constructed to provide construction access to the park. We also understand a new permanent bridge may be built after construction of this project.

The recommendations below should be considered preliminary in nature. Once additional information including bridge location(s), foundation type(s), and loading are developed, we should revisit and update our recommendations. For the permanent bridge, we may recommend additional field exploration and laboratory testing be performed at abutment locations

9.1 TEMPORARY BRIDGE STRUCTURE

We anticipate temporary bridge crossings will consist of temporary shallow abutment bearing pads that will support temporary bridge crossings as shallow foundation systems. We recommend a capping layer of at least 1 foot of compacted Class 2 aggregate baserock be placed on the graded bearing pads. We recommend an allowable bearing capacity of 2,000 psf be assumed for temporary conditions.

9.2 PERMANENT BRIDGE STRUCTURE

For preliminary purposes we recommend the permanent bridge be supported on either 24-inch-diameter Cast in Drilled Hole (CIDH) piers or continuous strip footings at each abutment location. For the purposes of these recommendations, we have assumed that negligible scour will occur at the creek. If hydraulic study indicates that there is a potential for scour, the preliminary foundation recommendations contained herein should be modified as necessary.

9.2.1 Shallow Footings

A permissible net contact stress of 2,000 pounds per square foot (psf) may be considered at the vehicular bridge abutment locations. Where a shallow foundation is located within 10 feet from a slope, the footing should be provided with additional embedment such that a horizontal distance of 10 feet from the edge of the foundation to the slope face is achieved. The friction factor for sliding resistance may be assumed as 0.30 and passive pressures acting on footing foundations may be assumed as 300 pounds per cubic foot (pcf), starting at a depth of at least 2 feet below lowest adjacent grade or that depth necessary to achieve a horizontal distance of 10 feet between the outer base edge of the footing and the nearest free face, whichever is deeper.

9.2.2 CIDH Piles

We performed preliminary pile capacity analysis of 24-inch-diameter CIDH piles assuming they are friction-only piles. Our analysis is based on the beta and alpha methods and is in accordance with the guidelines provided in the Federal Highway Administration (FHWA) Publication FHWA NHI-10-016 (Brown et al, 2010).

TABLE 9.2.2-1
Preliminary Ultimate Compression Vertical Capacities,
24-Inch-Diameter CIDH

Pile Length (feet)	Capacity (kips)
20	104
30	173
40	225

9.2.3 ARS Curve

Based on the subsurface soil conditions encountered in the exploratory points and local seismic sources, we estimate the average shear wave velocity for the top 100 feet (30 meters) (Vs30) of soil materials to be approximately 270 meters/second.

To develop acceleration response spectrum (ARS) curves in accordance with the 2013 Caltrans SDC procedures, we performed site-specific seismic hazard analyses. The 2013 Caltrans SDC requires comparing and enveloping the ARS curves calculated from the following:

- Deterministic Criteria based on late-Quaternary faults in the 2012 fault database published by California Geology Survey.
- Probabilistic Criteria based on a 5 percent in 50 years probability of exceedance ground motion (975-year return period).
- Minimum Spectrum based on a M_w = 6.5 strike-slip event occurring at a distance of 12 km from site.

The deterministic criteria are controlled by an event on the San Andreas fault. The probabilistic criteria are based on source data from the 2008 United States Geological Survey (USGS) National Seismic Hazard Map.

The resulting ARS curve developed in accordance with the 2013 Caltrans SDC, include near-fault directivity effects and is contained in Figure 7.

10.0 PAVEMENT DESIGN

We developed the pavement recommendations based on the Caltrans Highway Design Manual design method, as this method is the preferred method for pavement design in California. Based on the soil conditions observed, we have assumed a Resistance Value (R-value) of 5 for a clayey subgrade and Traffic Indices (TI) provided by the Civil Engineer utilizing the methods contained in Chapter 630 of the Caltrans Highway Design Manual (including the asphalt factor of safety).

10.1 CHEMICALLY TREATED SUBGRADE

The pavement subgrade soil may be chemically treated to reduce the expansive index and provide additional stabilization during pavement construction. In addition, chemical treatment will increase the R-value of the pavement subgrade soil. The type of chemical treatment (lime, quicklime or cement) and percentage of chemical additive to be used should be based on testing of the actual subgrade soil after mass grading is completed.

Chemically treated soil generally can achieve a nominal R-value of 50. The preliminary pavement designs provided below provide a chemical treatment option. The actual R-value should be confirmed through testing on actual chemically treated subgrade soil.

Chemical treatment should be performed by a specialty contractor experienced in this type of work. In addition, excavations perform in chemically treated soils, such as for utility trenches, should be stockpiled and protected for reuse in the upper backfill area to match the section.

10.2 FLEXIBLE PAVEMENT

Based on the R-value of 5 and the provided TIs, we developed recommended pavement sections using Chapter 630 of the Caltrans Highway Design Manual (including the asphalt factor of safety), presented in the following table. We also provide recommended pavement sections for at least an 18-inch thickness of chemically treated subgrade.

TABLE 9.2-1: Recommended Asphalt Concrete Pavement Sections

TRAFFIC LOADING TRAFFIC INDEX	R-VALUE OF 5 (UNTREATED SUBGRADE)		R-VALUE OF 50 (CHEMICALLY TREATED SUBGRADE)	
	AC (INCHES)	AB (INCHES)	AC (INCHES)	AB (INCHES)
5	3	10	3	4
6	3.5	13	3.5	4
7	4	16	4	5
8	5	18	5	5
9	5.5	21	5.5	7

Notes: AC is asphalt concrete
AB is Class 2 aggregate base material with a minimum R-value of 78

Pavement construction and all materials should comply with the requirements of the Standard Specifications of the State of California Department of Transportation, Civil Engineer, and appropriate public agency.

10.3 RIGID PAVEMENT

Concrete pavement sections can be used to resist heavy loads and turning forces in areas such as fire lanes or trash enclosures. Final design of rigid pavement sections, and accompanying reinforcement, should be performed based on estimated traffic loads and frequencies. We recommend the following minimum design sections for rigid pavements:

- Use a minimum section of 6 inches of Portland Cement concrete over 12 inches of Caltrans Class 2 Aggregate Base for subgrade with an R-value of 5 assuming a traffic index of 7. For chemically treated subgrade with an R-value of 50, use a minimum section of 6 inches of Portland Cement concrete over 5 inches of Caltrans Class 2 Aggregate Base.
- Concrete pavement should have a minimum 28-day compressive strength of 3,500 psi.
- Provide minimum control joint spacing in accordance with Portland Cement Association guidelines.

10.4 SUBGRADE AND AGGREGATE BASE COMPACTION

The contractor should compact finished subgrade and aggregate base in accordance with Section 4.0. Aggregate Base should meet the requirements for ¾-inch maximum Class 2 AB in accordance with Section 26-1.02a of the latest Caltrans Standard Specifications.

10.5 CUT-OFF CURBS

Saturated pavement subgrade or aggregate base can cause premature failure or increased maintenance of asphalt concrete pavements. This condition often occurs where landscape areas directly abut and drain toward pavements. If desired to install pavement cutoff barriers, they should be considered where pavement areas lie downslope of any landscaped areas that are to be sprinklered or irrigated, and should extend to a depth of at least 4 inches below the base rock layer. Cutoff barriers may consist of deepened concrete curbs or deep-root moisture barriers.

If reduced pavement life and greater than normal pavement maintenance are acceptable to the owner, then the cutoff barrier may be eliminated.

11.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

This report presents geotechnical recommendations for design of the improvements discussed in Section 1.2 for the Atherton Water Capture Project. If changes occur in the nature or design of the project, we should be allowed to review this report and provide additional recommendations, if any. It is the responsibility of the owner to transmit the information and recommendations of this report to the appropriate organizations or people involved in design of the project, including but not limited to developers, owners, buyers, architects, engineers, and designers. The conclusions and recommendations contained in this report are solely professional opinions and are valid for a period of no more than 2 years from the date of report issuance.

We strived to perform our professional services in accordance with generally accepted geotechnical engineering principles and practices currently employed in the area; no warranty is expressed or implied. There are risks of earth movement and property damages inherent in building on or with earth materials. We are unable to eliminate all risks or provide insurance; therefore, we are unable to guarantee or warrant the results of our services.

This report is based upon field and other conditions discovered at the time of report preparation. We developed this report with limited subsurface exploration data. We assumed that our subsurface exploration data is representative of the actual subsurface conditions across the site. Considering possible underground variability of soil, rock, stockpiled material, and groundwater, additional costs may be required to complete the project. We recommend that the owner establish a contingency fund to cover such costs. If unexpected conditions are encountered, notify ENGEO immediately to review these conditions and provide additional and/or modified recommendations, as necessary.

Our geotechnical exploration did not include work to determine the existence of possible hazardous materials. If any hazardous materials are encountered during construction, notify the proper regulatory officials immediately.

This document must not be subject to unauthorized reuse, that is, reusing without written authorization of ENGEO. Such authorization is essential because it requires ENGEO to evaluate the document's applicability given new circumstances, not the least of which is passage of time.

Actual field or other conditions will necessitate clarifications, adjustments, modifications or other changes to ENGEO's documents. Therefore, ENGEO must be engaged to prepare the necessary clarifications, adjustments, modifications or other changes before construction activities commence or further activity proceeds. If ENGEO's scope of services does not include on-site construction observation, or if other persons or entities are retained to provide such services, ENGEO cannot be held responsible for any or all claims arising from or resulting from the performance of such services by other persons or entities, and from any or all claims arising from or resulting from clarifications, adjustments, modifications, discrepancies or other changes necessary to reflect changed field or other conditions.

We determined the lines designating the interface between layers on the exploration logs using visual observations. The transition between the materials may be abrupt or gradual. The exploration logs contain information concerning samples recovered, indications of the presence of various materials such as clay, sand, silt, rock, existing fill, etc., and observations of groundwater encountered. The field logs also contain our interpretation of the subsurface conditions between sample locations. Therefore, the logs contain both factual and interpretative information. Our recommendations are based on the contents of the final logs, which represent our interpretation of the field logs.

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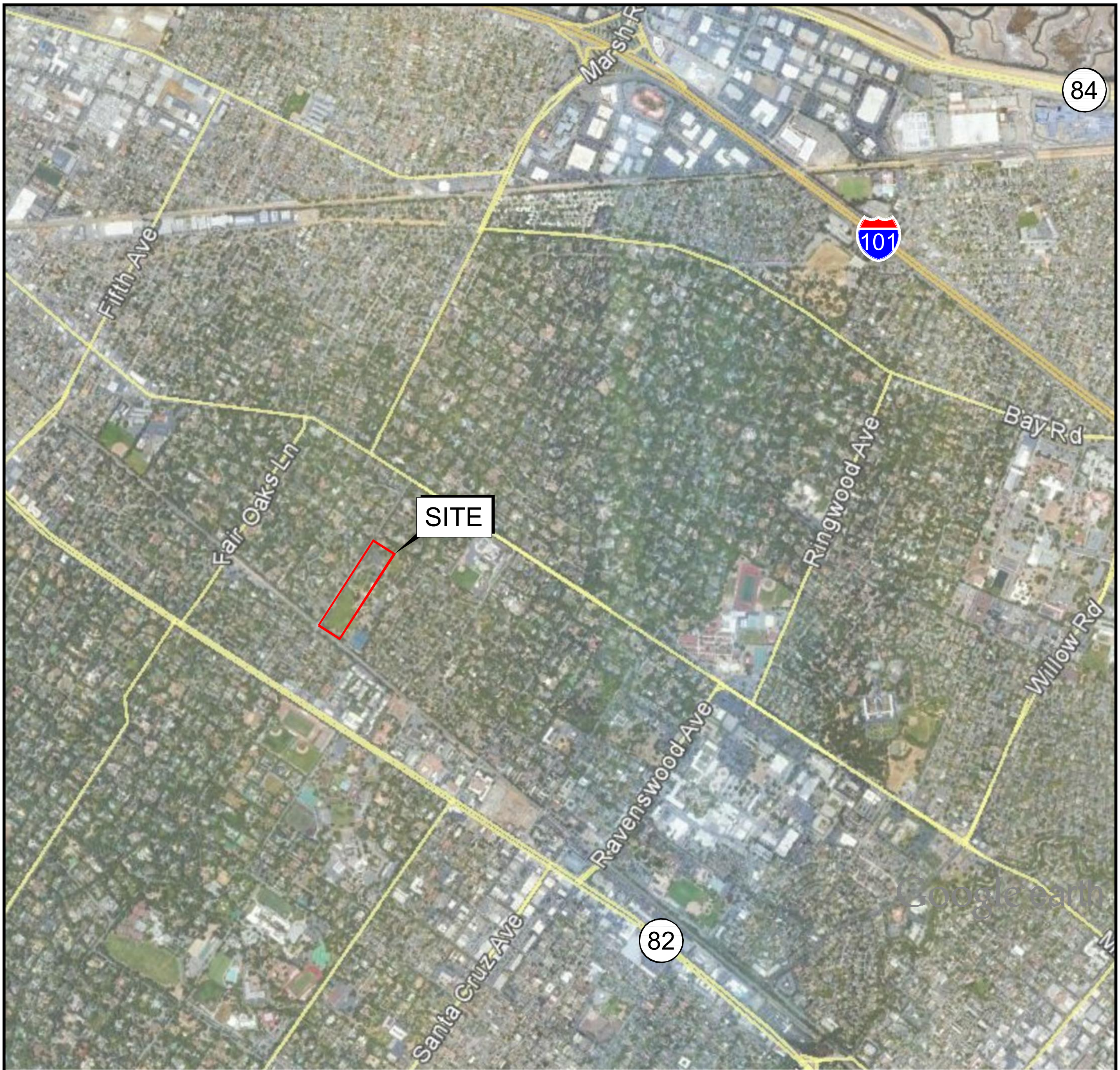
FIGURES

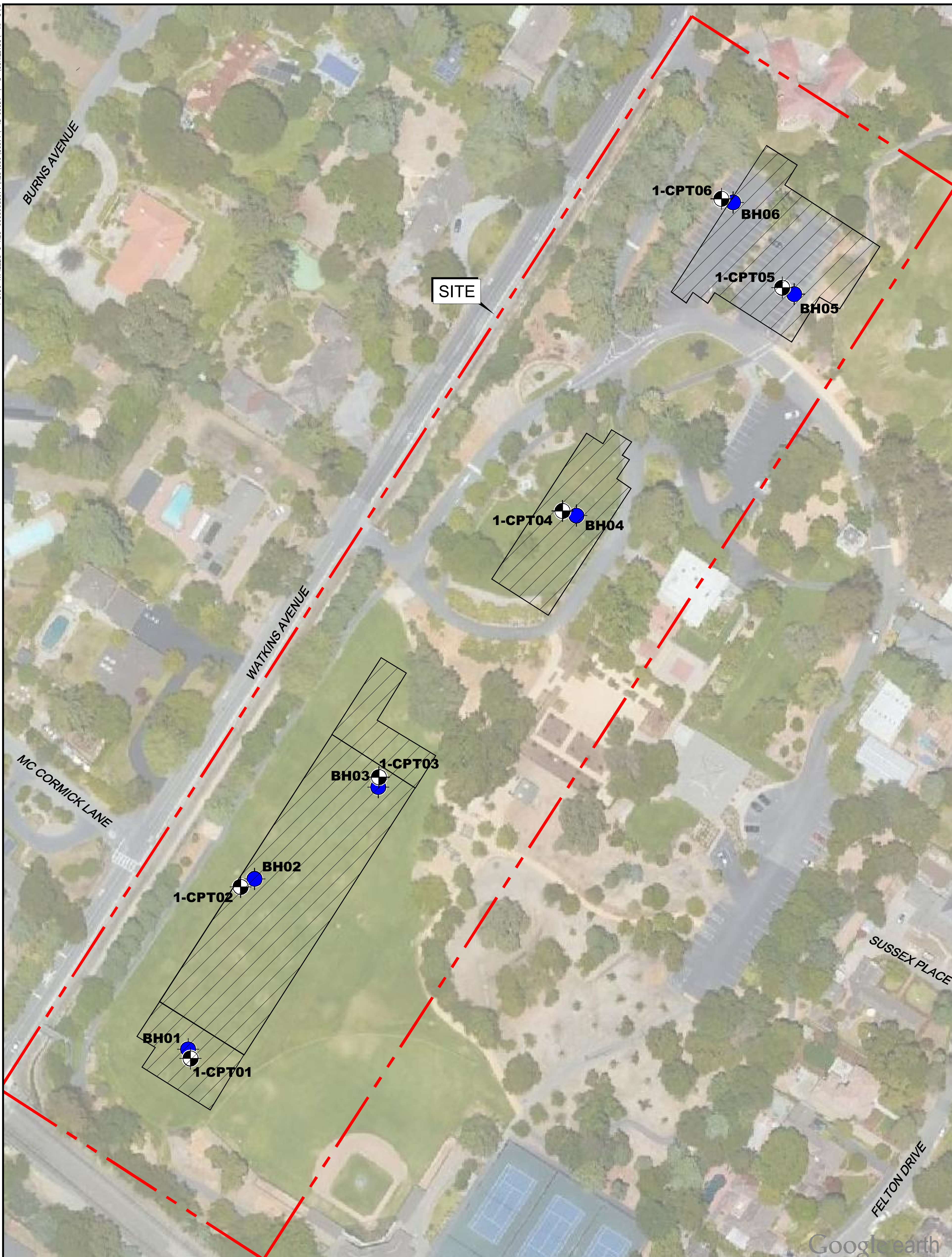


BASE MAP SOURCE: GOOGLE EARTH MAPPING SERVICE

ENGEO Expect Excellence	VICINITY MAP ATHERTON WATER CAPTURE PROJECT ATHERTON, CALIFORNIA	PROJECT NO.: 14695.000.000 SCALE: AS SHOWN DRAWN BY: LL CHECKED BY: AHF	FIGURE NO. 1

G:\Drafting\DRAFTING2_Dwg_13000 Plus\14695\000\GEX\14695000000-GEX-1-VICMAP-0318.dwg Plot Date: 3-27-18 llee

ORIGINAL FIGURE PRINTED IN COLOR



N

0 FEET 100

0 METERS 50

EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

BH06 — BORING (ENGEO, 2018)

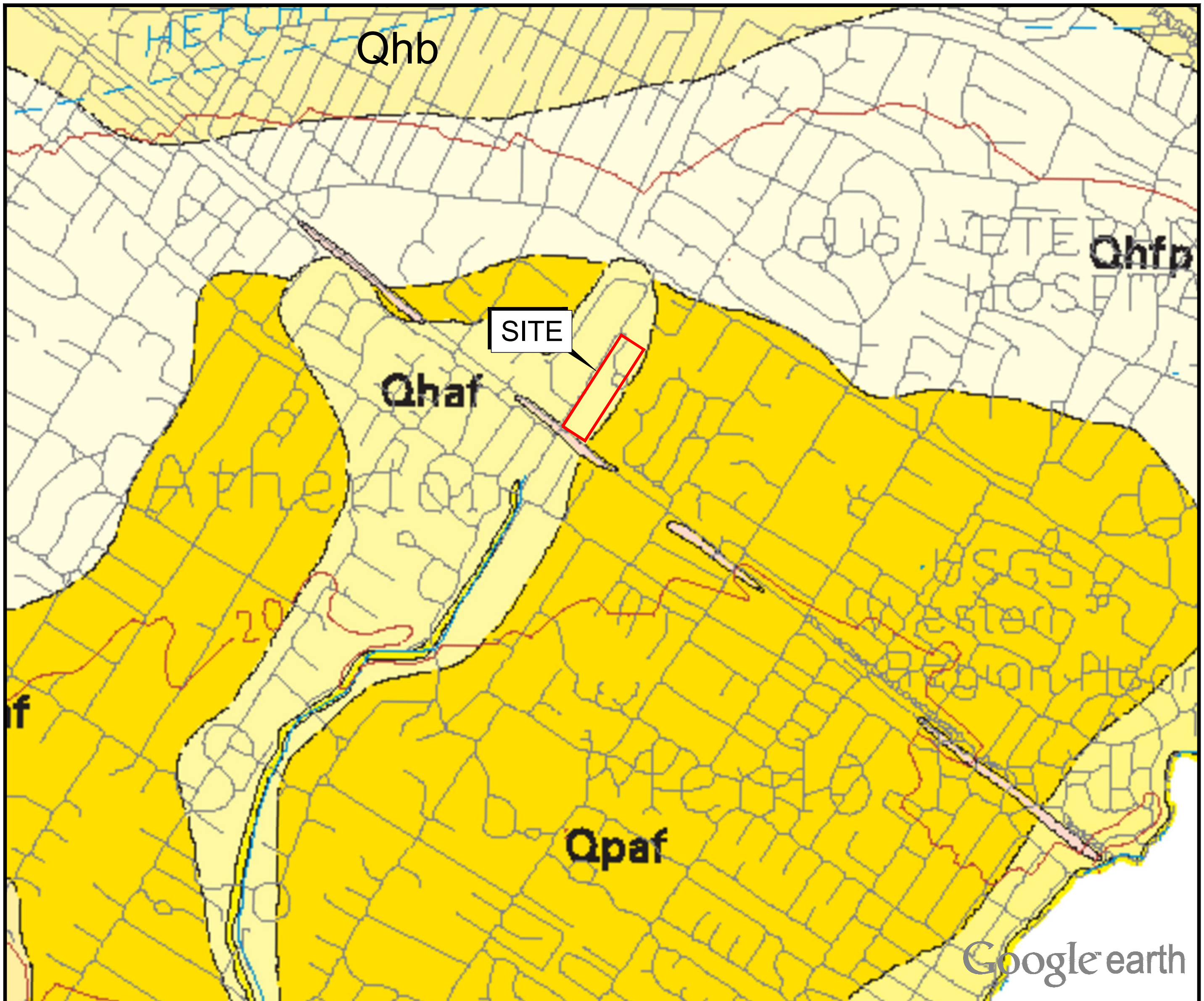
1-CPT06 — CONE PENETRATION TEST (ENGEO, 2018)

PROPOSED STORAGE AREA ALTERNATIVES

BASE MAP SOURCE: GOOGLE EARTH MAPPING SERVICE

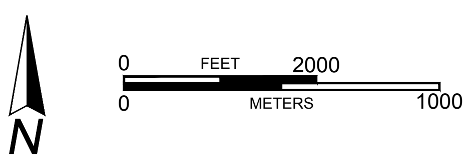
ENGEO Expect Excellence	SITE PLAN ATHERTON WATER CAPTURE PROJECT ATHERTON, CALIFORNIA	PROJECT NO.: 14695.000.000 SCALE: AS SHOWN DRAWN BY: LL / CHECKED BY: AHF	FIGURE NO. 2

ORIGINAL FIGURE PRINTED IN COLOR



EXPLANATION

Qhb	BASIN DEPOSITS (HOLOCENE)
Qhfp	FLOODPLAIN DEPOSITS (HOLOCENE)
Qhaf	ALLUVIAL FAN AND FLUVIAL DEPOSITS (HOLOCENE)
Qpaf	ALLUVIAL FAN AND FLUVIAL DEPOSITS (PLEISTOCENE)



BASE MAP SOURCE: BRABB, GRAYMER, AND JONES, 1998

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REGIONAL GEOLOGIC MAP
ATHERTON WATER CAPTURE PROJECT
ATHERTON, CALIFORNIA

PROJECT NO.: 14695.000.000
SCALE: AS SHOWN
DRAWN BY: LL
CHECKED BY: AHF
FIGURE NO. 3

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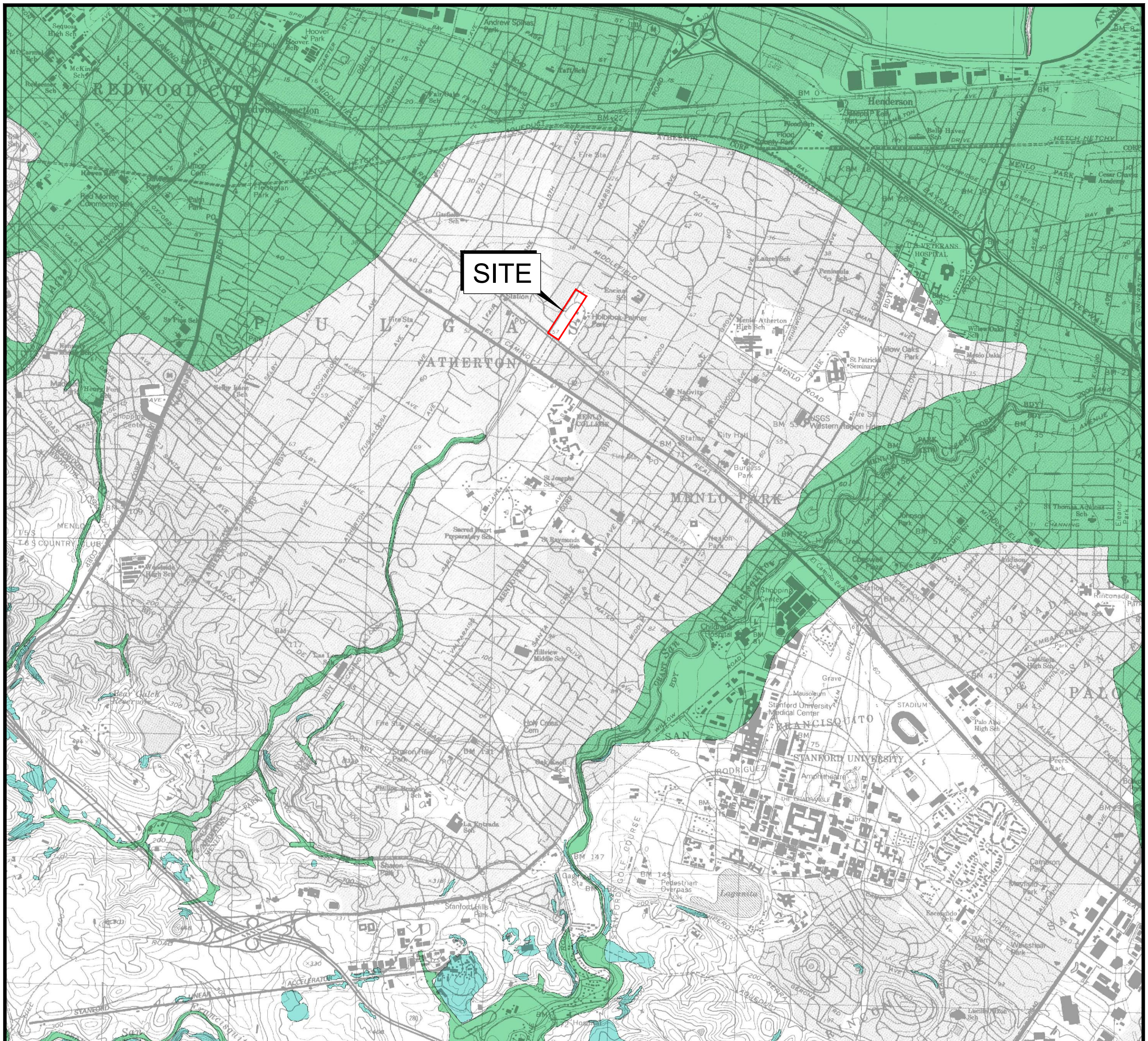
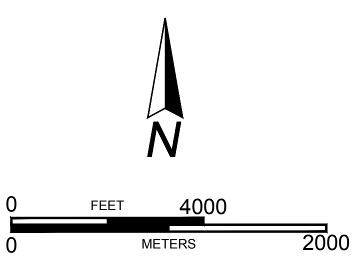
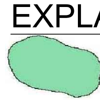


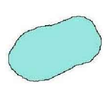


EXPLANATION



LIQUEFACTION

AREAS WHERE HISTORIC OCCURRENCE OF LIQUEFACTION, OR LOCAL GEOLOGICAL, GEOTECHNICAL AND GROUNDWATER CONDITIONS INDICATE A POTENTIAL FOR PERMANENT GROUND DISPLACEMENTS SUCH THAT MITIGATION AS DEFINED IN PUBLIC RESOURCES CODE SECTION 2693(c) WOULD BE REQUIRED



EARTHQUAKE-INDUCED LANDSLIDES

AREAS WHERE PREVIOUS OCCURRENCE OF LANDSLIDE MOVEMENT, OR LOCAL TOPOGRAPHIC, GEOLOGICAL, GEOTECHNICAL AND SUBSURFACE WATER CONDITIONS INDICATE A POTENTIAL FOR PERMANENT GROUND DISPLACEMENTS SUCH THAT MITIGATION AS DEFINED IN PUBLIC RESOURCES CODE SECTION 2693(c) WOULD BE REQUIRED

BASE MAP SOURCE: CALIFORNIA DEPARTMENT OF CONSERVATION, CALIFORNIA GEOLOGICAL SURVEY, 2006

ENGEO — Expect Excellence	SEISMIC HAZARD ZONES MAP ATHERTON WATER CAPTURE PROJECT ATHERTON, CALIFORNIA	PROJECT NO.: 14695.000.000 SCALE: AS SHOWN DRAWN BY: LL — CHECKED BY: AHF	FIGURE NO. 4

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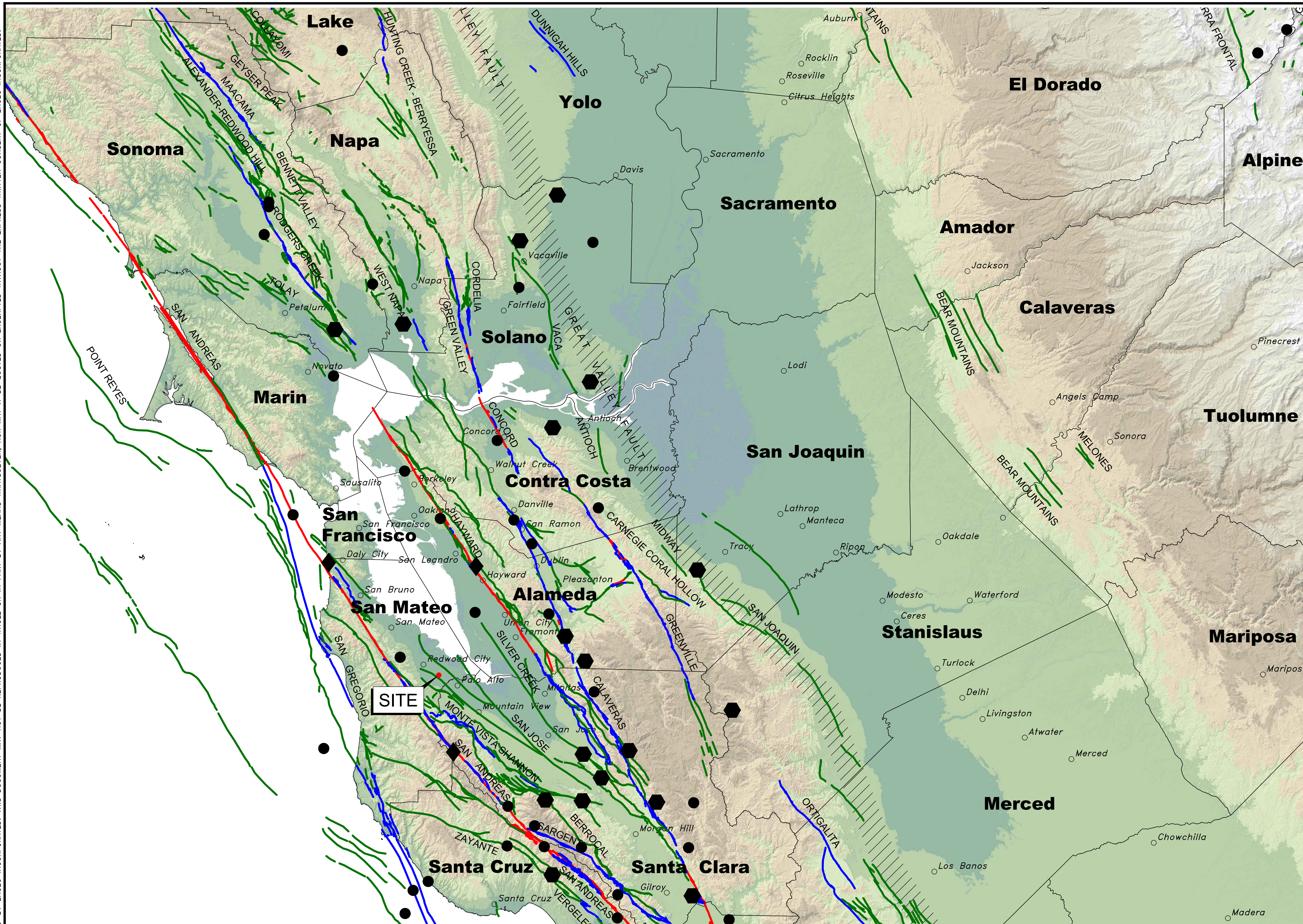

EXPLANATION

- MAGNITUDE 7+
- MAGNITUDE 6-7
- MAGNITUDE 5-6
- HISTORIC FAULT
- HOLOCENE FAULT
- QUATERNARY FAULT
- HISTORIC BLIND THRUST FAULT ZONE

BASE MAP SOURCE:
COLOR HILLSHADE IMAGE BASED ON THE NATIONAL ELEVATION DATASET (NED) AT 30 METER RESOLUTION
U.S.G.S. QUATERNARY FAULT DATABASE, NOVEMBER, 2010
U.S.G.S. HISTORIC EARTHQUAKE DATABASE (1800–2000)

BASE MAP SOURCE: BRABB, GRAYMER, AND JONES, 1998

ENGEO Expect Excellence	REGIONAL FAULTING AND SEISMICITY ATHERTON WATER CAPTURE PROJECT ATHERTON, CALIFORNIA	PROJECT NO.: 14695.000.000 SCALE: AS SHOWN DRAWN BY: LL CHECKED BY: AHF	FIGURE NO. 5

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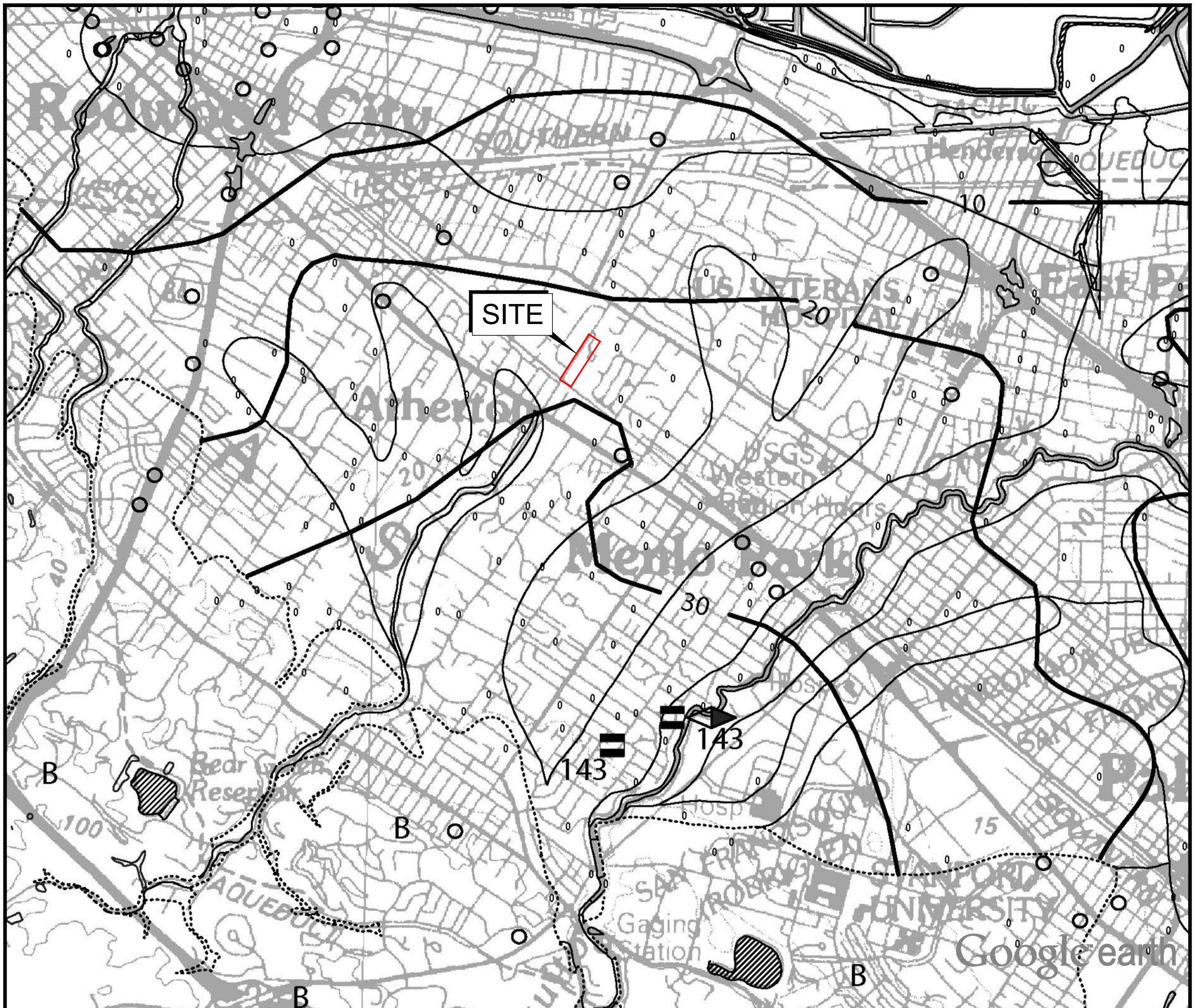
ORIGINAL FIGURE PRINTED IN COLOR



EXPLANATION

- Absence of ground failure noted
- Ground cracks not clearly associated with landslide, lateral spread, settlement, or primary movement
- Lateral spread
- B Bedrock
- Bedrock- Quaternary boundary
- 152 Number assigned to ground failure site (adapted from Youd and Hoose, 1978; and Tinsley and others,1998; by Knudsen and others, 2000)
- Water Body
- 10 Depth to ground water, in feet
- Geotechnical borings used in liquefaction evaluation

Ground-water level data provided by the State Water Resources Control Board.

N

0 FEET 4000
0 METERS 2000

BASE MAP SOURCE: STATE OF CALIFORNIA GEOLOGICAL SURVEY

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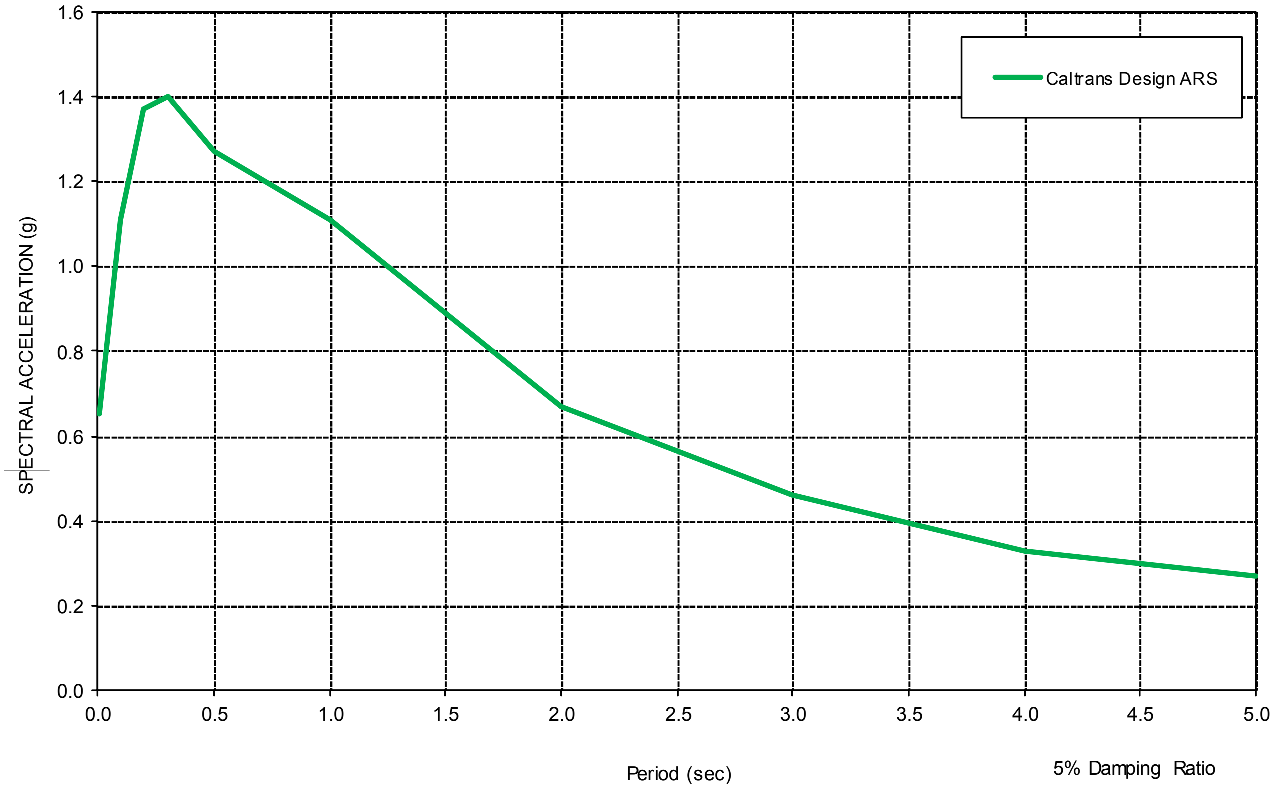
HISTORIC HIGH GROUNDWATER
ATHERTON WATER CAPTURE PROJECT
ATHERTON, CALIFORNIA

PROJECT NO.:	14695.000.000
SCALE:	AS SHOWN
DRAWN BY: LL	CHECKED BY: AHF
FIGURE NO.	6

C:\Users\llee\appdata\local\temp\AcPublish_9168\14695000000-GEX-6-HighGW-0318.dwg Plot Date: 3-27-18 llee

ORIGINAL FIGURE PRINTED IN COLOR



Peak Ground Acceleration (PGA)= 0.65g

Shear Wave Velocity ($V_{S,30}$) = 270 m/s

Spectral Coordinates

Period (s)	Acc. (g)
0.01	0.65
0.10	1.11
0.20	1.37
0.30	1.40
0.50	1.27
1.00	1.11
2.00	0.67
3.00	0.46
4.00	0.33
5.00	0.27

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ACCELERATION RESPONSE SPECTRUM

ATHERTON WATER CAPTURE PROJECT

ATHERTON, CALIFORNIA

PROJECT NO.: 14695.000.000

SCALE: AS SHOWN

DRAWN BY: SRP

CHECKED BY: AHF

FIGURE NO. 7

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ORIGINAL FIGURE PRINTED IN COLOR

APPENDIX A

BORING LOG KEY
EXPLORATION LOGS

KEY TO BORING LOGS

MAJOR TYPES			DESCRIPTION
COARSE-GRAINED SOILS MORE THAN HALF OF MAT'L LARGER THAN #200 SIEVE	GRAVELS MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE SIZE	CLEAN GRAVELS WITH LESS THAN 5% FINES	GW - Well graded gravels or gravel-sand mixtures GP - Poorly graded gravels or gravel-sand mixtures
		GRAVELS WITH OVER 12 % FINES	GM - Silty gravels, gravel-sand and silt mixtures GC - Clayey gravels, gravel-sand and clay mixtures
	SANDS MORE THAN HALF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE SIZE	CLEAN SANDS WITH LESS THAN 5% FINES	SW - Well graded sands, or gravelly sand mixtures SP - Poorly graded sands or gravelly sand mixtures
		SANDS WITH OVER 12 % FINES	SM - Silty sand, sand-silt mixtures SC - Clayey sand, sand-clay mixtures
FINE-GRAINED SOILS MORE THAN HALF OF MAT'L SMALLER THAN #200 SIEVE	SILTS AND CLAYS LIQUID LIMIT 50 % OR LESS		ML - Inorganic silt with low to medium plasticity CL - Inorganic clay with low to medium plasticity OL - Low plasticity organic silts and clays
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50 %		MH - Elastic silt with high plasticity CH - Fat clay with high plasticity OH - Highly plastic organic silts and clays
	HIGHLY ORGANIC SOILS		PT - Peat and other highly organic soils

For fine-grained soils with 15 to 29% retained on the #200 sieve, the words "with sand" or "with gravel" (whichever is predominant) are added to the group name.

For fine-grained soil with >30% retained on the #200 sieve, the words "sandy" or "gravelly" (whichever is predominant) are added to the group name.

GRAIN SIZES

U.S. STANDARD SERIES SIEVE SIZE — CLEAR SQUARE SIEVE OPENINGS

	200	40	10	4	3/4 "	3"	12"
SILTS AND CLAYS	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

RELATIVE DENSITY

SANDS AND GRAVELS	BLOWS/FOOT (S.P.T.)
VERY LOOSE	0-4
LOOSE	4-10
MEDIUM DENSE	10-30
DENSE	30-50
VERY DENSE	OVER 50

CONSISTENCY

SILTS AND CLAYS	STRENGTH*
VERY SOFT	0-1/4
SOFT	1/4-1/2
MEDIUM STIFF	1/2-1
STIFF	1-2
VERY STIFF	2-4
HARD	OVER 4

MOISTURE CONDITION

DRY	Dusty, dry to touch
MOIST	Damp but no visible water
WET	Visible freewater

SAMPLER SYMBOLS

- Modified California (3" O.D.) sampler
- California (2.5" O.D.) sampler
- S.P.T. - Split spoon sampler
- Shelby Tube
- Continuous Core
- Bag Samples
- Grab Samples
- NR No Recovery

LINE TYPES

- Solid - Layer Break
- Dashed - Gradational or approximate layer break

GROUND-WATER SYMBOLS

- Groundwater level during drilling
- Stabilized groundwater level

(S.P.T.) Number of blows of 140 lb. hammer falling 30" to drive a 2-inch O.D. (1-3/8 inch I.D.) sampler

* Unconfined compressive strength in tons/sq. ft., asterisk on log means determined by pocket penetrometer

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LOG OF BORING BH01

LATITUDE: 37.462287 LONGITUDE: 122.193166

Geotechnical Exploration
Atherton Water Capture
San Mateo County
14695.000.000

DATE DRILLED: 3/6/2018
HOLE DEPTH: Approx. 31½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (WGS84): Approx. 55 ft.

LOGGED / REVIEWED BY: T. Klotzback / A. Firmin
DRILLING CONTRACTOR: West Coast Exploration
DRILLING METHOD: Solid Flight Auger
HAMMER TYPE: 140 lb. Rope and Pulley

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
			FAT CLAY WITH SAND (CH), dark brown, moist, sand is fine-grained [FILL?]										
			LEAN CLAY WITH SAND (CL), light yellowish brown, hard, moist, sand is fine-grained, some orange staining										
5	50												
						27							4.5*
			LEAN CLAY WITH SILT (CL), light yellowish brown, stiff, moist, rootlets present										
						24							2.0* 1.39
10	45		LEAN CLAY WITH SILT (CL), light brown, stiff, moist, some fine rounded gravels present										
			SANDY LEAN CLAY (CL), light yellowish brown, hard, moist, fine rounded gravels present, some orange staining			22	34	16	18	68	18.8	105.9	1.5* 2.5*
			LEAN CLAY WITH SILT (CL), light yellowish brown, stiff, moist, fine subrounded to angular gravels present, some orange staining										
						29							1.5*
15	40												
			dark brown staining present			31							1.5*
			SANDY LEAN CLAY TO CLAYEY SAND (CL-SC), light yellowish brown to grayish brown, stiff, moist, sand is fine-grained, some orange and dark brown staining										
20	35												

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LOG OF BORING BH01

LATITUDE: 37.462287 LONGITUDE: 122.193166

Geotechnical Exploration
Atherton Water Capture
San Mateo County
14695.000.000

DATE DRILLED: 3/6/2018
HOLE DEPTH: Approx. 31½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (WGS84): Approx. 55 ft.

LOGGED / REVIEWED BY: T. Klotzback / A. Firmin
DRILLING CONTRACTOR: West Coast Exploration
DRILLING METHOD: Solid Flight Auger
HAMMER TYPE: 140 lb. Rope and Pulley

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
			LEAN CLAY WITH SAND (CL), light yellowish brown to grayish brown, stiff, moist, sand is fine-grained, some orange and dark brown staining			30							1.5*
25	30		LEAN CLAY WITH SAND (CL), light grayish brown, medium stiff to stiff, moist, sand is fine-grained, some fine angular gravels present, heavy orange staining, some dark brown staining			50							1.0* 1.5*
			WELL GRADED GRAVEL WITH SAND AND CLAY (GW), light gray to dark gray, medium dense, moist, gravel is fine to coarse and angular, sand is dark brown, some calcium carbonate nodules present			55							
30	25												
			more moisture visible, becomes dense no recovery			38							

Bottom of borehole at approximately 31 1/2 feet below ground surface.
No groundwater encountered 45 minutes after drilling.

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LOG OF BORING BH02

LATITUDE: 37.462767 LONGITUDE: 122.192943

Geotechnical Exploration
Atherton Water Capture
San Mateo County
14695.000.000

DATE DRILLED: 3/8/2018
HOLE DEPTH: Approx. 51½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (WGS84): Approx. 54 ft.

LOGGED / REVIEWED BY: T. Klotzback / A. Firmin
DRILLING CONTRACTOR: West Coast Exploration
DRILLING METHOD: Solid Flight Auger
HAMMER TYPE: 140 lb. Rope and Pulley

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
			FAT CLAY (CH), dark brown, hard, moist, organics and angular gravels present [FILL?]										
			FAT CLAY (CH), dark brown, hard, moist, organics and angular gravels present										
			some orange and black staining			55					21	105.4	4.5+*
	50												
5			LEAN CLAY WITH SILT (CL), light yellowish brown, hard, moist, trace fine angular to subrounded gravels, some orange staining			47							4.5* 4.0*
			LEAN CLAY WITH SILT (CL), light yellowish brown, very stiff, moist, some fine subgrounded gravels present			25					21.7	104.1	2.5* 3.0*
	45		becomes light brown mottled with light yellowish brown, some orange staining										
10													
			becomes stiff, some dark brown staining, some angular gravels present, higher silt content			33							3.0* 2.0*
			LEAN CLAY WITH SAND AND SILT (CL), light yellowish brown, stiff, moist, sand is fine-grained, orange and black staining, fine angular gravels present, higher silt content										
	40												
15													
						23				82	21.9	105.3	2.0*
	35												
20													

LOG OF BORING BH02

LATITUDE: 37.462767 LONGITUDE: 122.192943

Geotechnical Exploration
Atherton Water Capture
San Mateo County
14695.000.000

DATE DRILLED: 3/8/2018
HOLE DEPTH: Approx. 51½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (WGS84): Approx. 54 ft.

LOGGED / REVIEWED BY: T. Klotzback / A. Firmin
DRILLING CONTRACTOR: West Coast Exploration
DRILLING METHOD: Solid Flight Auger
HAMMER TYPE: 140 lb. Rope and Pulley

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
			LEAN CLAY WITH SAND AND SILT (CL), light yellowish brown, stiff, moist, sand is fine-grained, orange and black staining, fine angular gravels present, higher silt content becomes very stiff			30	36	18	18	82	20.2	105.9	3.5* 2.5*
	30		LEAN CLAY WITH SILT (CL), light brown mottled with light yellowish brown, stiff, moist										
25						16							1.5*
	25												
30			becomes medium stiff, some orange and black staining			32					29	97	1.0*
	20												
35			no recovery			24							
			becomes more moist, higher silt content, trace fine angular gravels			7					25.1		
	15												
40													

LOG - GEOTECHNICAL W/ELEV. 14695.000.000 GINT LOGS V1.GPJ ENGEO INC.GDT 4/9/18

ENGEO
Expect Excellence

LOG OF BORING BH02

LATITUDE: 37.462767 LONGITUDE: 122.192943

Geotechnical Exploration
Atherton Water Capture
San Mateo County
14695.000.000

DATE DRILLED: 3/8/2018
HOLE DEPTH: Approx. 51½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (WGS84): Approx. 54 ft.

LOGGED / REVIEWED BY: T. Klotzback / A. Firmin
DRILLING CONTRACTOR: West Coast Exploration
DRILLING METHOD: Solid Flight Auger
HAMMER TYPE: 140 lb. Rope and Pulley

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
			LEAN CLAY WITH SILT (CL), light brown mottled with light yellowish brown, stiff, moist			17							
	10		LEAN CLAY WITH SILT (CL), light gray, stiff, moist, cemented calcium carbonate deposits present										
45						12					30.1		
	5												
50			CLAYEY SAND WITH SILT (SC), gray, medium dense, moist, fine angular to subrounded gravels and calcium veins present, interbedded layers of sandy silt sand becomes more coarse			30							
			Bottom of borehole at approximately 51 1/2 feet below ground surface. Groundwater encountered at 29 1/2 feet below ground surface 20 minutes after drilling.										

LOG OF BORING BH03

ENGEO — Expect Excellence

LATITUDE: 37.463026 LONGITUDE: 122.192498

Geotechnical Exploration
Atherton Water Capture
San Mateo County
14695.000.000

DATE DRILLED: 3/6/2018
HOLE DEPTH: Approx. 22 ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (WGS84): Approx. 53 ft.

LOGGED / REVIEWED BY: T. Klotzback / A. Firmin
DRILLING CONTRACTOR: West Coast Exploration
DRILLING METHOD: Solid Flight Auger
HAMMER TYPE: 140 lb. Rope and Pulley

Depth in Feet	Elevation in Feet	DESCRIPTION	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
		FAT CLAY (CH), dark brown, stiff, moist, rootlets present [FILL?]								
		FAT CLAY (CH), dark brown, stiff, moist, rootlets present								
	50	becomes very stiff	35	57	18	39	90	23.2	102.8	2.0* 3.0*
5		SANDY LEAN CLAY (CL), light yellowish brown, hard, moist, sand is fine-grained, some black staining, drier than previous layer	31							4.5+*
	45	LEAN CLAY (CL), light yellowish brown, very stiff, moist, fine angular to subrounded gravels present	33					18.4	105.8	2.6 3.5*
10		LEAN CLAY WITH SILT (CL), light yellowish brown, very stiff, moist								
	40	LEAN CLAY (CL), brown, very stiff, moist, fine subrounded gravels present, light yellowish brown silt inclusions	27							2.5* 3.0*
		LEAN CLAY WITH SILT AND SAND (CL), light yellowish brown, very stiff, moist, sand is fine-grained, some orange staining								
15		LEAN CLAY (CL), light yellowish brown, very stiff, moist, some orange staining	30							2.5* 3.5*
	35	GRAVELLY LEAN CLAY (CL), light brown, very stiff, moist, some orange staining, gravels are fine to coarse	38				50	14.7	120.6	3.0* 4.0*
20										

ENGEO
Expect Excellence

LOG OF BORING BH03

LATITUDE: 37.463026 LONGITUDE: 122.192498

Geotechnical Exploration
Atherton Water Capture
San Mateo County
14695.000.000

DATE DRILLED: 3/6/2018
HOLE DEPTH: Approx. 22 ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (WGS84): Approx. 53 ft.

LOGGED / REVIEWED BY: T. Klotzback / A. Firmin
DRILLING CONTRACTOR: West Coast Exploration
DRILLING METHOD: Solid Flight Auger
HAMMER TYPE: 140 lb. Rope and Pulley

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
			GRAVELLY LEAN CLAY (CL), light brown, very stiff, moist, some orange staining, gravels are fine to coarse										
			CLAYEY GRAVEL (GC), light brown, moist, gravels are fine to coarse							14			
			Bottom of borehole at approximately 22 feet below ground surface. No groundwater encountered 10 minutes after drilling.										

LOG - GEOTECHNICAL W/ELEV. 14695.000.000 GINT LOGS V1.GPJ ENGEO INC.GDT 4/9/18

ENGEO
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LOG OF BORING BH04

LATITUDE: 37.46383 LONGITUDE: 122.191807

Geotechnical Exploration
Atherton Water Capture
San Mateo County
14695.000.000

DATE DRILLED:	3/6/2018	LOGGED / REVIEWED BY:	T. Klotzback / A. Firmin
HOLE DEPTH:	Approx. 21½ ft.	DRILLING CONTRACTOR:	West Coast Exploration
HOLE DIAMETER:	4.0 in.	DRILLING METHOD:	Solid Flight Auger
SURF ELEV (WGS84):	Approx. 52 ft.	HAMMER TYPE:	140 lb. Rope and Pulley

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
			FAT CLAY (CH), dark brown, stiff, moist, trace organics [FILL?]										
	50		FAT CLAY (CH), dark brown, stiff, moist, trace organics										
			becomes brown and very stiff			25					27.5	97.1	1.5* 2.5*
5													
			no recovery			22							2.5*
	45												
			LEAN CLAY WITH SILT (CL), light yellowish brown, medium stiff, moist, trace fine angular gravels, some orange staining										
10													
			LEAN CLAY (CL), dark brown, stiff, moist, trace fine subrounded gravels			16					23.6	101.7	0.5* 1.0*
	40												
			LEAN CLAY WITH SILT (CL), light brown, very stiff, moist, some orange and black staining										
15													
			becomes medium stiff			25							2.0* 1.0*
	35												
			SANDY LEAN CLAY WITH SILT (CL), light brown, stiff, wet, sand is fine-grained, some orange staining										
20													

LOG OF BORING BH04

ENGEO
Expect Excellence

LATITUDE: 37.46383 LONGITUDE: 122.191807

Geotechnical Exploration
Atherton Water Capture
San Mateo County
14695.000.000

DATE DRILLED:	3/6/2018	LOGGED / REVIEWED BY:	T. Klotzback / A. Firmin
HOLE DEPTH:	Approx. 21½ ft.	DRILLING CONTRACTOR:	West Coast Exploration
HOLE DIAMETER:	4.0 in.	DRILLING METHOD:	Solid Flight Auger
SURF ELEV (WGS84):	Approx. 52 ft.	HAMMER TYPE:	140 lb. Rope and Pulley

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
			SANDY LEAN CLAY WITH SILT (CL), light brown, stiff, wet, sand is fine-grained, some orange staining										
			becomes moist and very stiff			25	32	21	11	68	23.3	105.4	1.0 - 3.0* 2.5*
			Bottom of borehole at approximately 21 1/2 feet below ground surface. No groundwater encountered 15 minutes after drilling.										

LOG OF BORING BH05

LATITUDE: 37.464483 LONGITUDE: 122.191029

Geotechnical Exploration
Atherton Water Capture
San Mateo County
14695.000.000

DATE DRILLED: 3/7/2018
HOLE DEPTH: Approx. 22½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (WGS84): Approx. 51 ft.

LOGGED / REVIEWED BY: T. Klotzback / A. Firmin
DRILLING CONTRACTOR: West Coast Exploration
DRILLING METHOD: Solid Flight Auger
HAMMER TYPE: 140 lb. Rope and Pulley

Depth in Feet	Elevation in Feet	DESCRIPTION	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
		ASPHALT 4" AC over 7" AB								
		AGGREGATE BASE (AB)								
	50	LEAN CLAY (CL), dark brown mottled with light yellowish brown, very stiff, moist, [FILL], fine angular gravels present								
		LEAN CLAY (CL), light yellowish brown mottled with dark brown, very stiff, moist, some orange oxide staining, calcium carbonate nodules present	40					22.7	100.6	3.5* 4.0*
5	45	LEAN CLAY (CL), yellowish brown, very stiff, moist, fine subrounded gravels present	27							3.5*
		LEAN CLAY WITH SILT (CL), light yellowish brown, hard, moist, fine angular to subrounded gravels present 6" lens of light yellowish brown clay with silt [CL], very stiff, moist, fine angular gravels present	17	30	18	12				4.5* 3.5*
		becomes very stiff with orange staining	21							
10	40									
		LEAN CLAY WITH SILT (CL), light brown, very stiff, moist, some orange and black staining	29					22.8	99.6	3.5* 2.5 - 3.0*
15	35	WELL GRADED SAND WITH CLAY AND GRAVEL (SW-SC), gray and light gray, medium dense, moist, clay fraction is light yellowish brown, cobble found in shoe	35							
							11			
20										

LOG OF BORING BH05

LATITUDE: 37.464483 LONGITUDE: 122.191029

Geotechnical Exploration
Atherton Water Capture
San Mateo County
14695.000.000

DATE DRILLED: 3/7/2018
HOLE DEPTH: Approx. 22½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (WGS84): Approx. 51 ft.

LOGGED / REVIEWED BY: T. Klotzback / A. Firmin
DRILLING CONTRACTOR: West Coast Exploration
DRILLING METHOD: Solid Flight Auger
HAMMER TYPE: 140 lb. Rope and Pulley

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
	30		WELL GRADED SAND WITH CLAY AND GRAVEL (SW-SC), gray and light gray, medium dense, moist, clay fraction is light yellowish brown, cobble found in shoe										
			Bottom of borehole at approximately 22 1/2 feet below ground surface. No groundwater encountered 20 minutes after drilling.										

ENGEO
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LOG OF BORING BH06

LATITUDE: 37.464742 LONGITUDE: 122.19126

Geotechnical Exploration
Atherton Water Capture
San Mateo County
14695.000.000

DATE DRILLED: 3/8/2018
HOLE DEPTH: Approx. 31½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (WGS84): Approx. 50 ft.

LOGGED / REVIEWED BY: T. Klotzback / A. Firmin
DRILLING CONTRACTOR: West Coast Exploration
DRILLING METHOD: Solid Flight Auger
HAMMER TYPE: 140 lb. Rope and Pulley

Depth in Feet	Elevation in Feet	DESCRIPTION	Blow Count/Foot	Atterberg Limits: Liquid Limit	Plastic Limit	Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
		ASPHALT 3" AC over 9" AB								
		AGGREGATE BASE (AB)								
		LEAN CLAY (CL), dark brown mottled with light yellowish brown, [FILL]								
		WELL GRADED SAND (SW), yellowish brown, medium dense, moist, some clay	33							3.0*
5	45									
		LEAN CLAY (CL), dark brown, hard, moist, fine angular gravels present, trace fine-grained sand, some black staining, drier than previous sample	22					10.7	102.6	4.5+*
		CLAYEY SAND (SC), light yellowish brown, medium dense, moist, sand is poorly graded, some fine angular to subgrounded gravels present	22							4.5+*
10	40	POORLY GRADED SAND WITH CLAY (SP), light yellowish brown, medium dense, moist, sand is fine- to medium-grained, some fine angular to subgrounded gravels present								
		SANDY LEAN CLAY (CL), light brown, very stiff, moist, sand is fine-grained, some orange staining, calcium carbonate nodules present	16							3.5*
		SANDY LEAN CLAY (CL), light yellowish brown, hard, dry, sand is fine-grained, fine angular gravels and calcium carbonate nodules present								
15	35									
		higher silt content	50/6"					19.6	97.3	4.5+*
		CLAYEY SAND WITH SILT (SC), light brown, very dense, dry, sand is fine- to medium-grained, some fine angular to subrounded gravels present								
20	30									

LOG OF BORING BH06

ENGEO
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LATITUDE: 37.464742 LONGITUDE: 122.19126

Geotechnical Exploration
Atherton Water Capture
San Mateo County
14695.000.000

DATE DRILLED:	3/8/2018	LOGGED / REVIEWED BY:	T. Klotzback / A. Firmin
HOLE DEPTH:	Approx. 31½ ft.	DRILLING CONTRACTOR:	West Coast Exploration
HOLE DIAMETER:	4.0 in.	DRILLING METHOD:	Solid Flight Auger
SURF ELEV (WGS84):	Approx. 50 ft.	HAMMER TYPE:	140 lb. Rope and Pulley

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
			CLAYEY SAND WITH SILT (SC), light brown, very dense, dry, sand is fine- to medium-grained, some fine angular to subrounded gravels present			50/6"				42			4.5+*
			WELL GRADED SAND WITH CLAY (SW-SC), light yellowish brown, dense, dry to moist, some fine subrounded gravels present										
25	25												
						50				6			
30	20												
			coarse gravels present, more moisture visible			50							
			Bottom of borehole at approximately 31 1/2 feet below ground surface. No groundwater encountered 10 minutes after drilling.										

APPENDIX B

CONE PENETRATION TESTS (CPTs)

CALIFORNIA PUSH TECHNOLOGIES INCORPORATED

Job No: 18-56020
Client: ENGEO Inc.
Project: Atherton Storm Water Capture Project
Start Date: 2-Mar-2018
End Date: 2-Mar-2018

CONE PENETRATION TEST SUMMARY

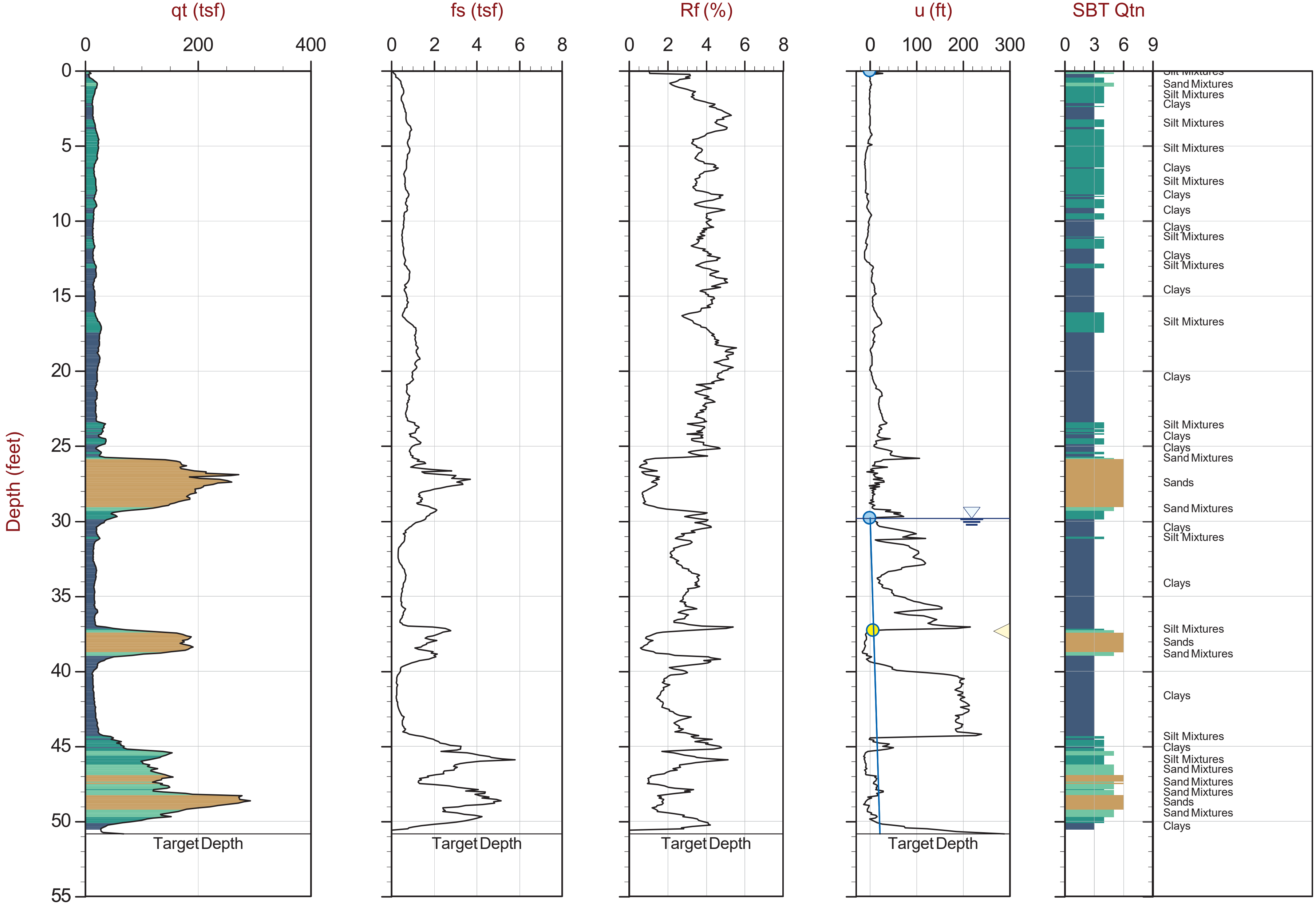
Sounding ID	File Name	Date	Cone	Assumed Phreatic Surface[1] (ft)	Final Depth (ft)	Northing[2] (m)	Easting (m)	Refer to Notation Number
1-CPT01	18-56020_CP01	2-Mar-2018	443:T1500F15U500	29.8	50.852	4146467	571354	
1-CPT02	18-56020_CP02	2-Mar-2018	443:T1500F15U500	29.3	50.688	4146528	571363	
1-CPT03	18-56020_CP03	2-Mar-2018	443:T1500F15U500	29.3	31.004	4146559	571404	3
1-CPT04	18-56020_CP04	2-Mar-2018	443:T1500F15U500	28.6	50.934	4146634	571470	
1-CPT05	18-56020_CP05	2-Mar-2018	443:T1500F15U500	29.0	50.770	4146703	571537	
1-CPT06	18-56020_CP06	2-Mar-2018	443:T1500F15U500	28.6	30.511	4146735	571518	

1. The assumed phreatic surface was based on pore pressure dissipation tests unless otherwise noted. Hydrostatic conditions were assumed for the calculated parameters.
2. The coordinates were acquired using consumer grade GPS equipment, datum: WGS 1984 / UTM Zone 10 North.
3. The assumed phreatic surface was based on equilibrium achieved from nearby sounding.

CALIFORNIA PUSH TECHNOLOGIES INCORPORATED

ENGEO Inc.

Job No: 18-56020
Date: 2018-03-02 07:49
Site: Atherton Storm Water Capture Project
Sounding: 1-CPT01
Cone: 443:T1500F15U500



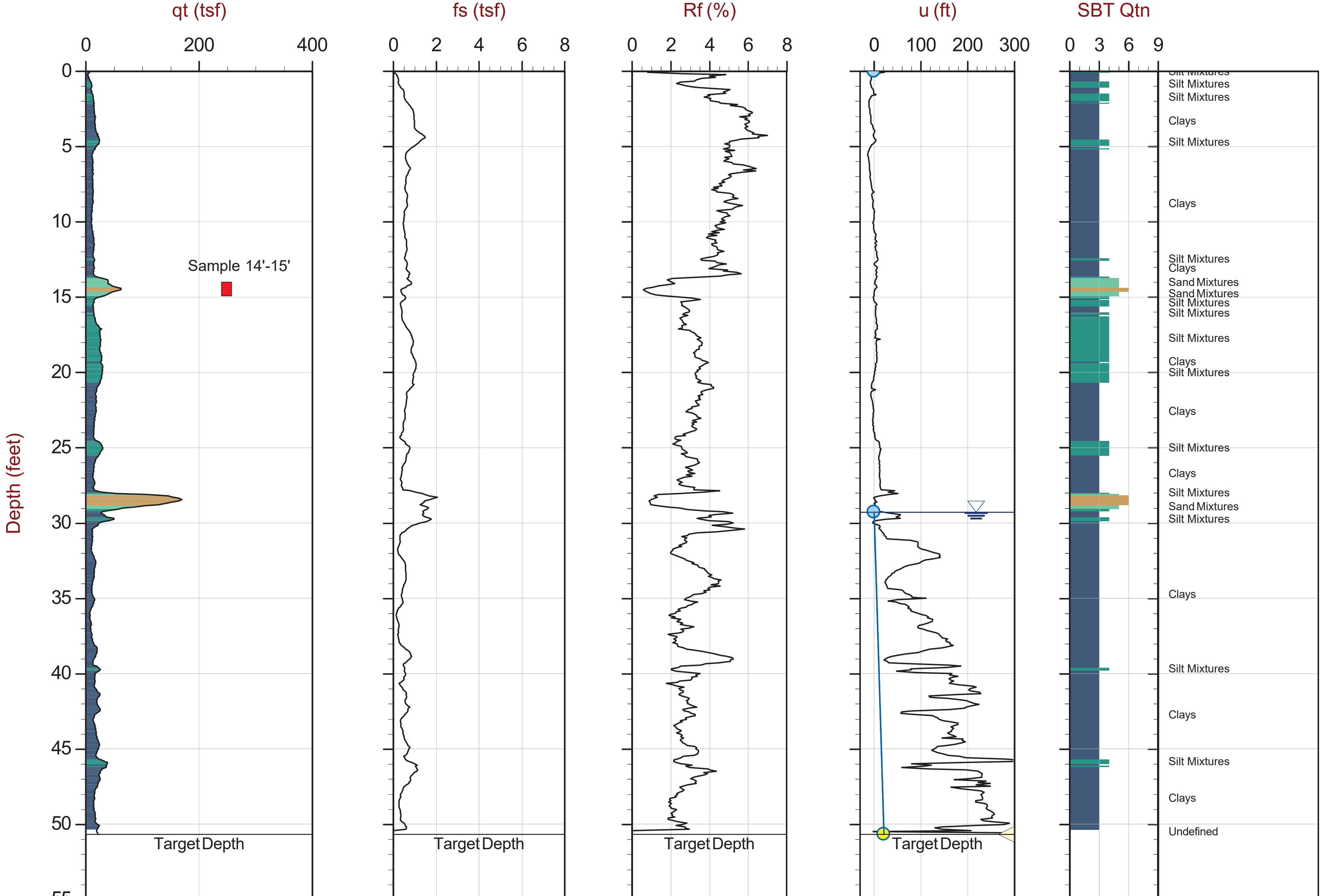
Max Depth: 15.500 m / 50.85 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point
File: 18-56020_CP01.COR
Unit Wt: SBTQtn (PKR2009)
SBT: Robertson, 2009 and 2010
Coords: UTM Zone 10 N: 4146467m E: 571354m
Page No: 1 of 1

Overplot Item: Assumed Ueq; Ueq; Dissipation, equilibrium achieved; Dissipation, equilibrium not achieved; Hydrostatic Line

CALIFORNIA PUSH TECHNOLOGIES INCORPORATED

ENGEO Inc.

Job No: 18-56020
Date: 2018-03-02 08:56
Site: Atherton Storm Water Capture Project
Sounding: 1-CPT02
Cone: 443:T1500F15U500



Max Depth: 15.450 m / 50.69 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point
File: 18-56020_CP02.COR
Unit Wt: SBTQtn (PKR2009)
SBT: Robertson, 2009 and 2010
Coords: UTM Zone 10 N: 4146528m E: 571363m
Page No: 1 of 1

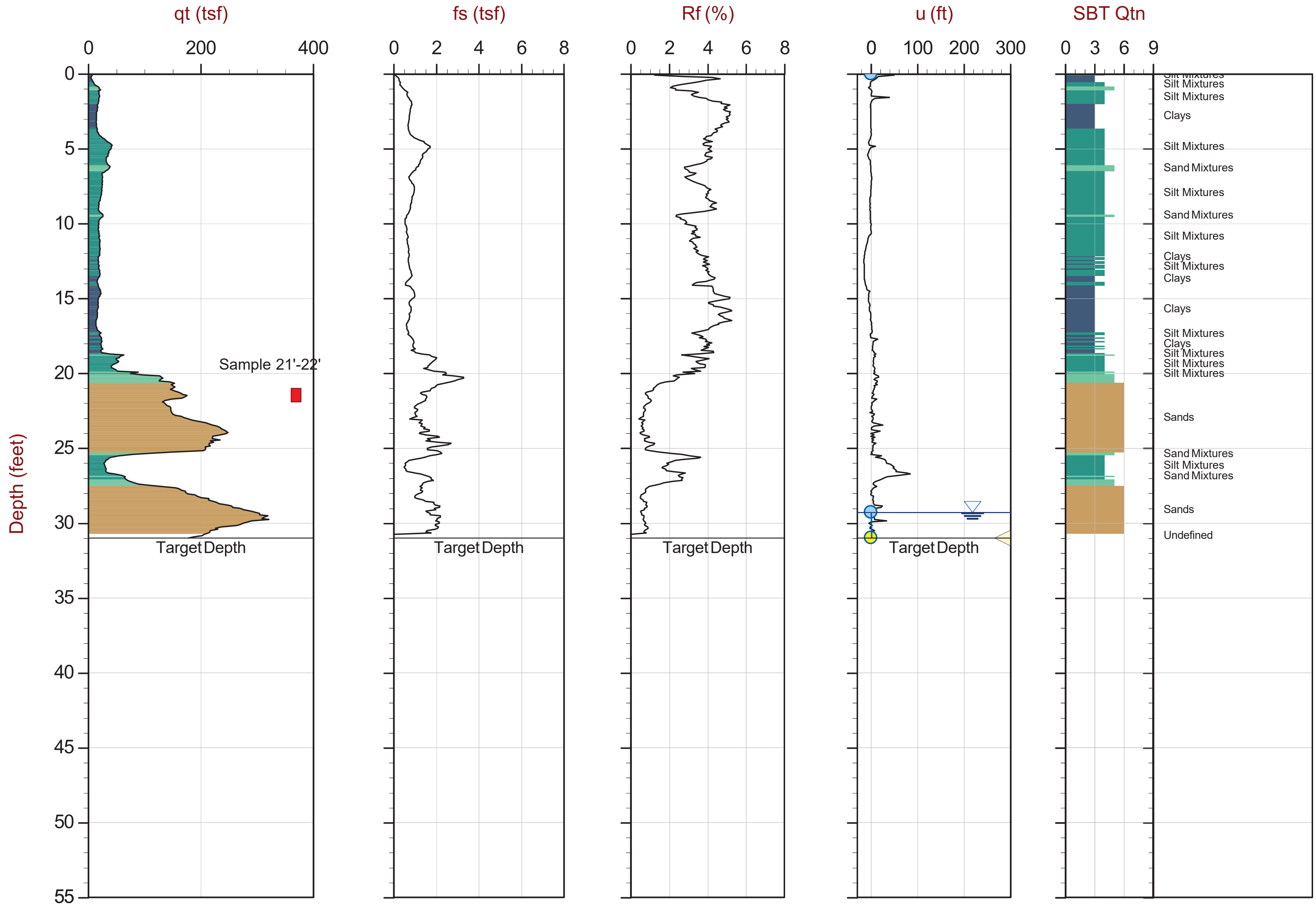
Overplot Item: Assumed Ueq; Ueq; Dissipation, equilibrium achieved; Dissipation, equilibrium not achieved; Hydrostatic Line; Sample

CALIFORNIA PUSH TECHNOLOGIES INCORPORATED

ENGEO Inc.

Job No: 18-56020
Date: 2018-03-02 10:17
Site: Atherton Storm Water Capture Project

Sounding: 1-CPT03
Cone: 443:T1500F15U500

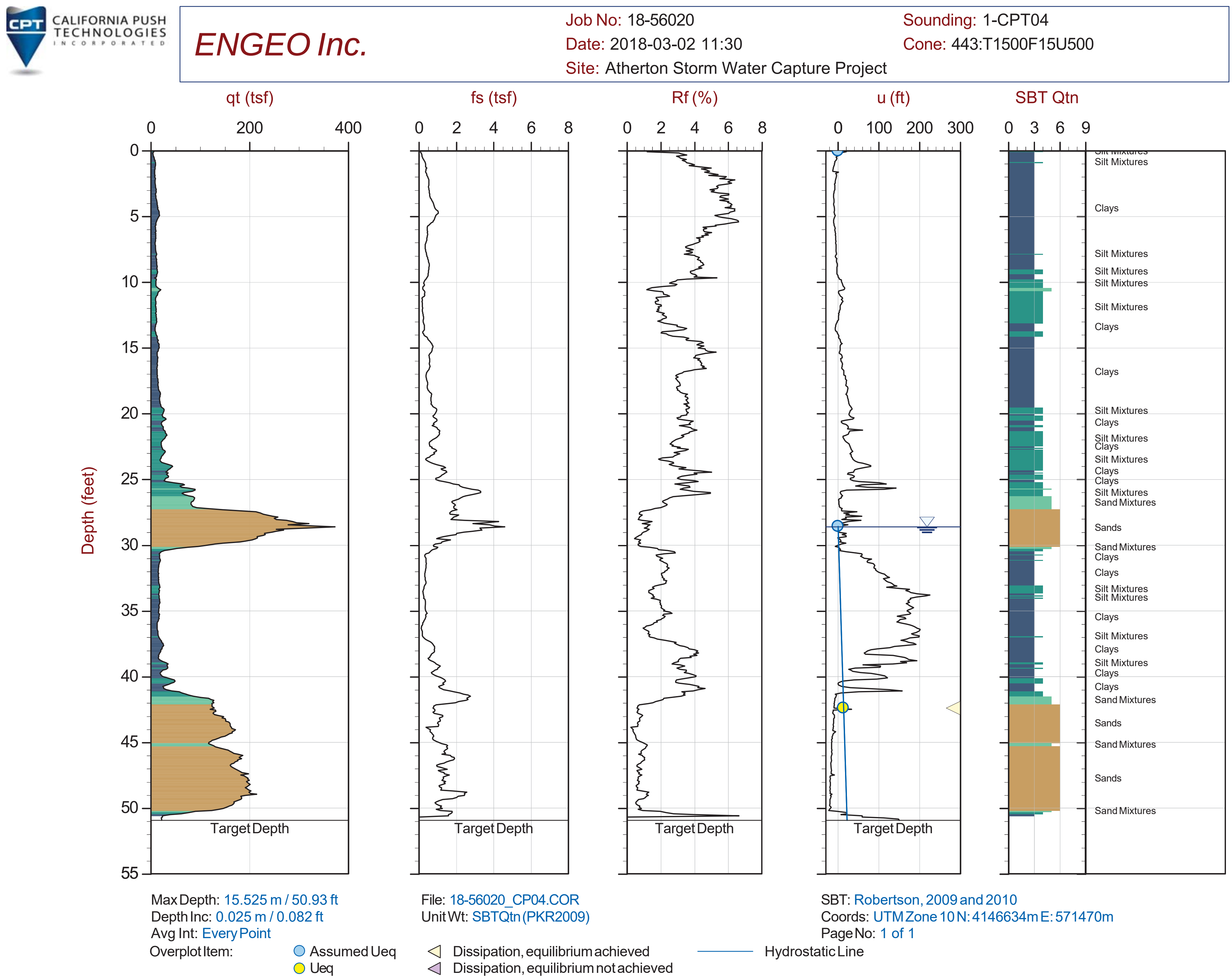


Max Depth: 9.450 m / 31.00 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 18-56020_CP03.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 10 N: 4146559m E: 571404m
Page No: 1 of 1

Overplot Item: Assumed Ueq; Ueq; Dissipation, equilibrium achieved; Dissipation, equilibrium not achieved; Hydrostatic Line; Sample

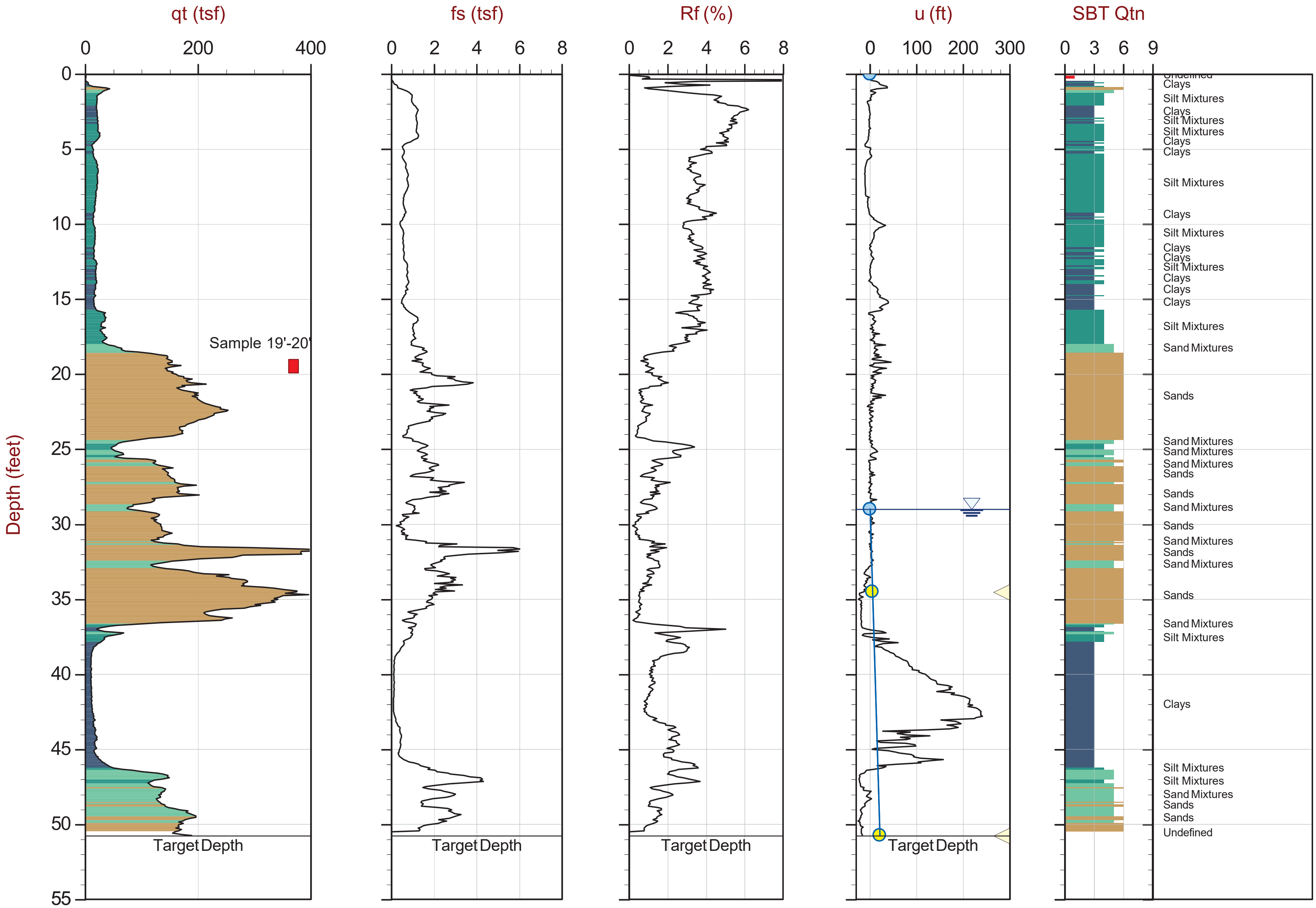


CALIFORNIA PUSH TECHNOLOGIES INCORPORATED
ENGEO Inc.
Job No: 18-56020
Date: 2018-03-02 11:30
Site: Atherton Storm Water Capture Project
Sounding: 1-CPT04
Cone: 443:T1500F15U500
qt (tsf)
fs (tsf)
Rf (%)
u (ft)
SBT Qtn
Depth (feet)
Target Depth
Silt Mixtures
Clays
Sands
Sand Mixtures
Max Depth: 15.525 m / 50.93 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point
File: 18-56020_CP04.COR
Unit Wt: SBTQtn (PKR2009)
SBT: Robertson, 2009 and 2010
Coords: UTM Zone 10 N: 4146634m E: 571470m
Page No: 1 of 1
Overplot Item:
Assumed Ueq
Ueq
Dissipation, equilibrium achieved
Dissipation, equilibrium not achieved
Hydrostatic Line

CALIFORNIA PUSH TECHNOLOGIES INCORPORATED

ENGEO Inc.

Job No: 18-56020
Date: 2018-03-02 13:06
Site: Atherton Storm Water Capture Project
Sounding: 1-CPT05
Cone: 443:T1500F15U500



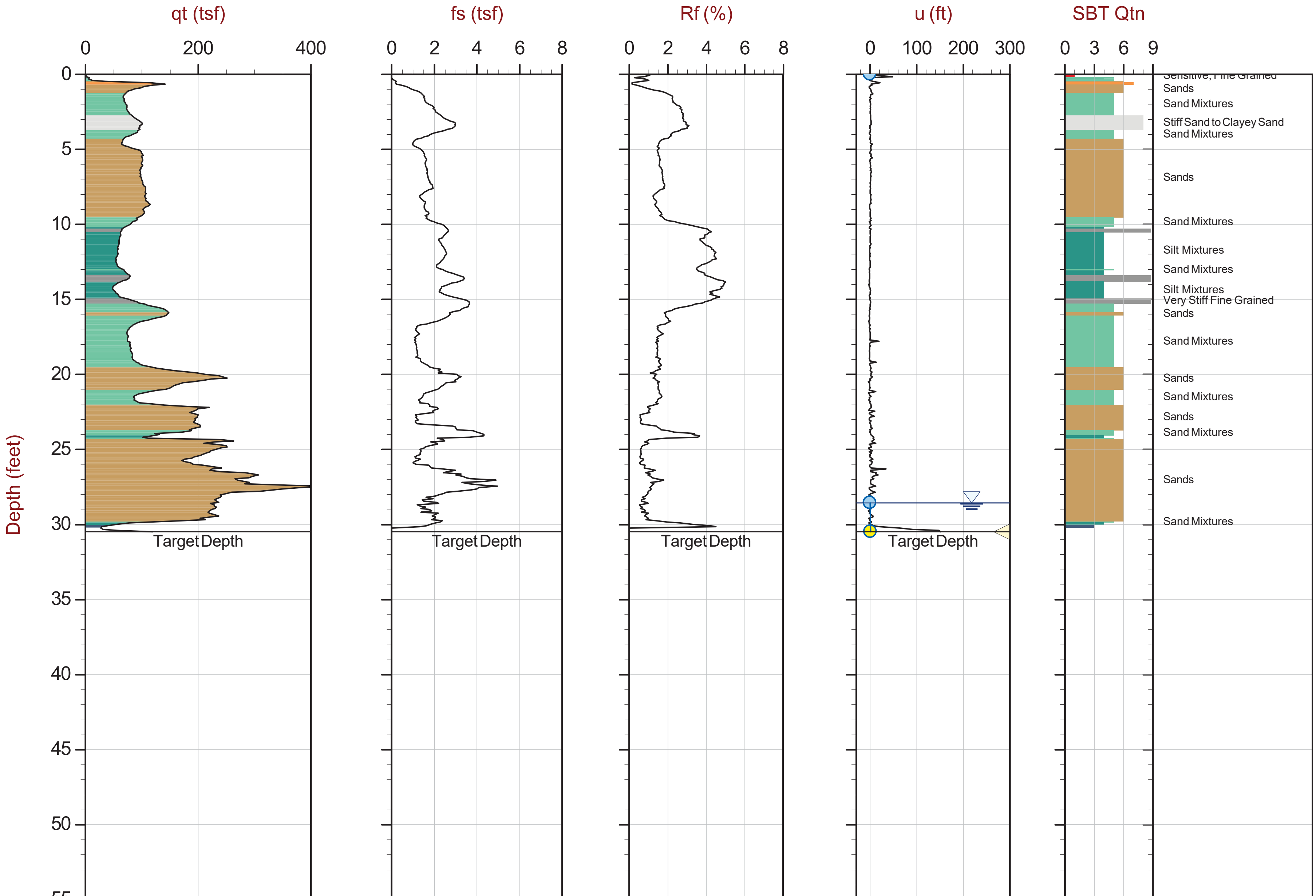
Max Depth: 15.475 m / 50.77 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point
Overplot Item: Assumed Ueq, Ueq, Dissipation, equilibrium achieved, Dissipation, equilibrium not achieved, Hydrostatic Line, Sample

File: 18-56020_CP05.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 10 N: 4146703m E: 571537m
Page No: 1 of 1

ENGEO Inc.

Job No: 18-56020
Date: 2018-03-02 15:17
Site: Atherton Storm Water Capture Project
Sounding: 1-CPT06
Cone: 443:T1500F15U500



Max Depth: 9.300 m / 30.51 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 18-56020_CP06.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 10 N: 4146735m E: 571518m
Page No: 1 of 1

Overplot Item: Assumed Ueq; Ueq; Dissipation, equilibrium achieved; Dissipation, equilibrium not achieved; Hydrostatic Line

APPENDIX C

LABORATORY TEST DATA

MOISTURE-DENSITY DETERMINATION
ASTM D7263

BORING ID:	BH01	BH02	BH02	BH02	BH02	BH02	BH02	BH02
DEPTH (ft.):	11	3	8	16	21	31	36.5-38	45-46.5
MOISTURE CONTENT (%):	18.8	21.0	21.7	21.9	20.2	29.0	25.1	30.1
DRY DENSITY (lbs/ft^3):	105.9	105.4	104.1	105.3	105.9	97.0		

BORING ID:	BH03	BH03	BH03	BH04	BH04	BH04	BH05	BH05
DEPTH (ft.):	3	8	18	3	10.5	20.5	3	13
MOISTURE CONTENT (%):	23.2	18.4	14.7	27.5	23.6	23.3	22.7	22.8
DRY DENSITY (lbs/ft^3):	102.8	105.8	120.6	97.1	101.7	105.4	100.6	99.6

BORING ID:	BH06	BH06						
DEPTH (ft.):	6	15.5						
MOISTURE CONTENT (%):	10.7	19.6						
DRY DENSITY (lbs/ft^3):	102.6	97.3						

PROJECT NAME: Atherton Water Capture Project
PROJECT NUMBER: 14695.000.000
CLIENT: Tetra Tech, Inc.
PHASE NUMBER: 001

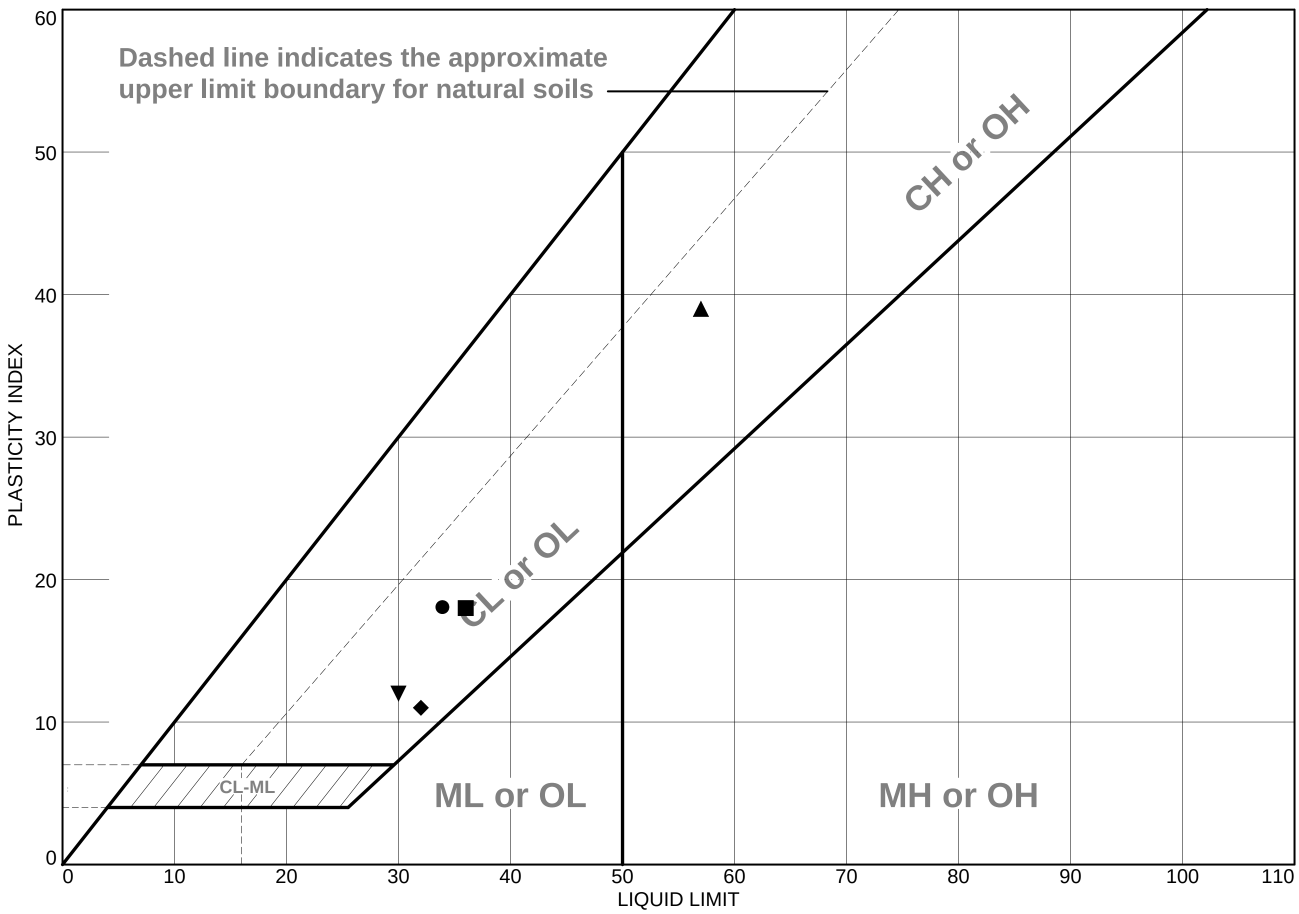
DATE: 03/29/18

ENGEO
Expect Excellence

Tested by: M. Bromfield   Reviewed by: G. Criste

ENGEO Incorporated 2010 Crow Canyon Place Suite 250 San Ramon, CA 94583 Laboratory address: 3420 Fostoria Way San Ramon, CA 94583

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	See exploration logs	34	16	18			
■	See exploration logs	36	18	18			
▲	See exploration logs	57	18	39			
◆	See exploration logs	32	21	11			
▼	See exploration logs	30	18	12			

Project No. 14695.000.000 **Client:** Tetra Tech, Inc.

Project: Atherton Water Capture Project

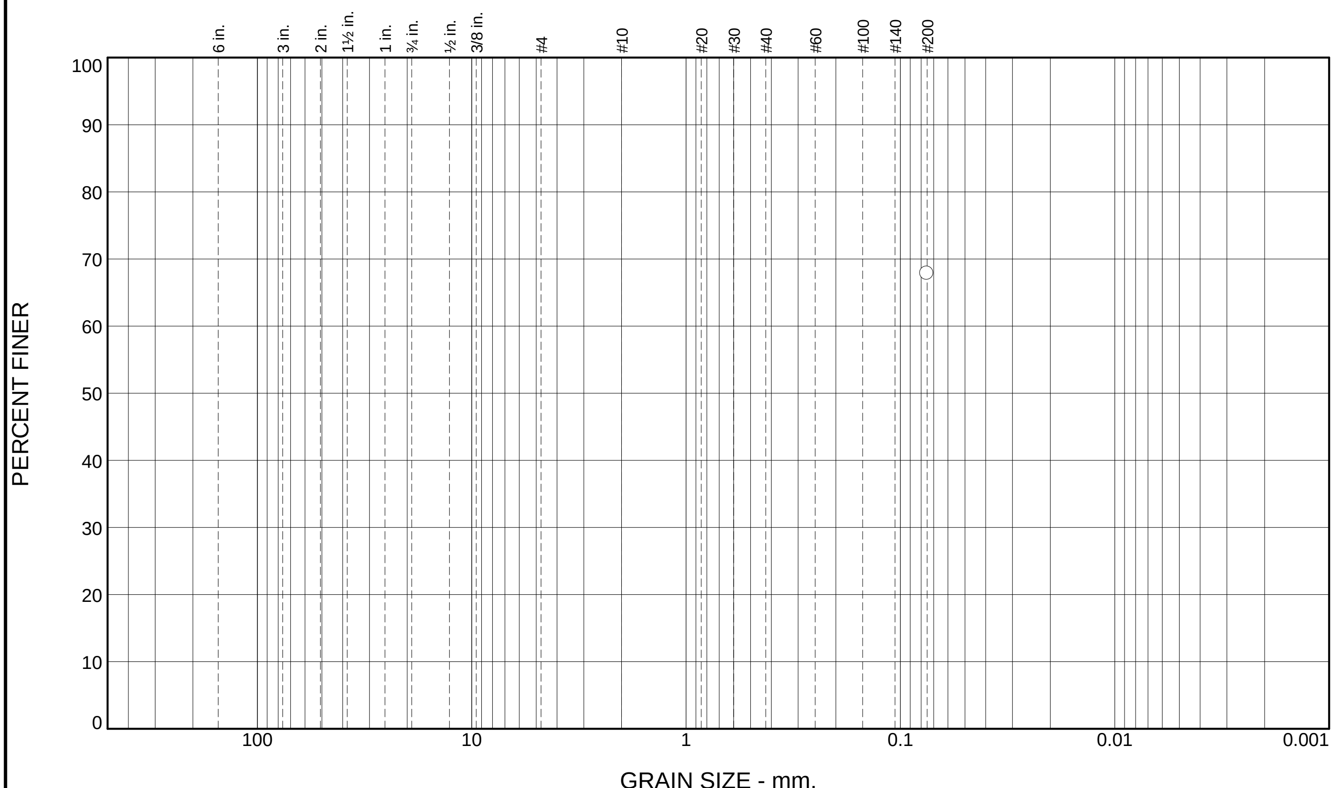
●**Depth:** 11.0 feet **Sample Number:** BH01 @ 11
■**Depth:** 20.5 feet **Sample Number:** BH02 @ 20.5
▲**Depth:** 3.0 feet **Sample Number:** BH03 @ 3
◆**Depth:** 21.0 feet **Sample Number:** BH04 @ 21
▼**Depth:** 7.5 feet **Sample Number:** BH05 @ 7.5

ENGEO INCORPORATED

Remarks:
●ASTM D4318, Wet method
■ASTM D4318, Wet method
▲ASTM D4318, Wet method
◆ASTM D4318, Wet method
▼ASTM D4318, Wet method

Tested By: M. Bromfield **Checked By:** G. Criste

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						67.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	67.8		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL= 16 LL= 34 PI= 18

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

PI: ASTM D4318, Wet method
GS: ASTM D1140, Method B
Dry sample weight = 245.89; Soak time = 2 hrs 10 mins

Sample Number: BH01 @ 11 **Depth:** 11.0 feet

Date: 3/29/18

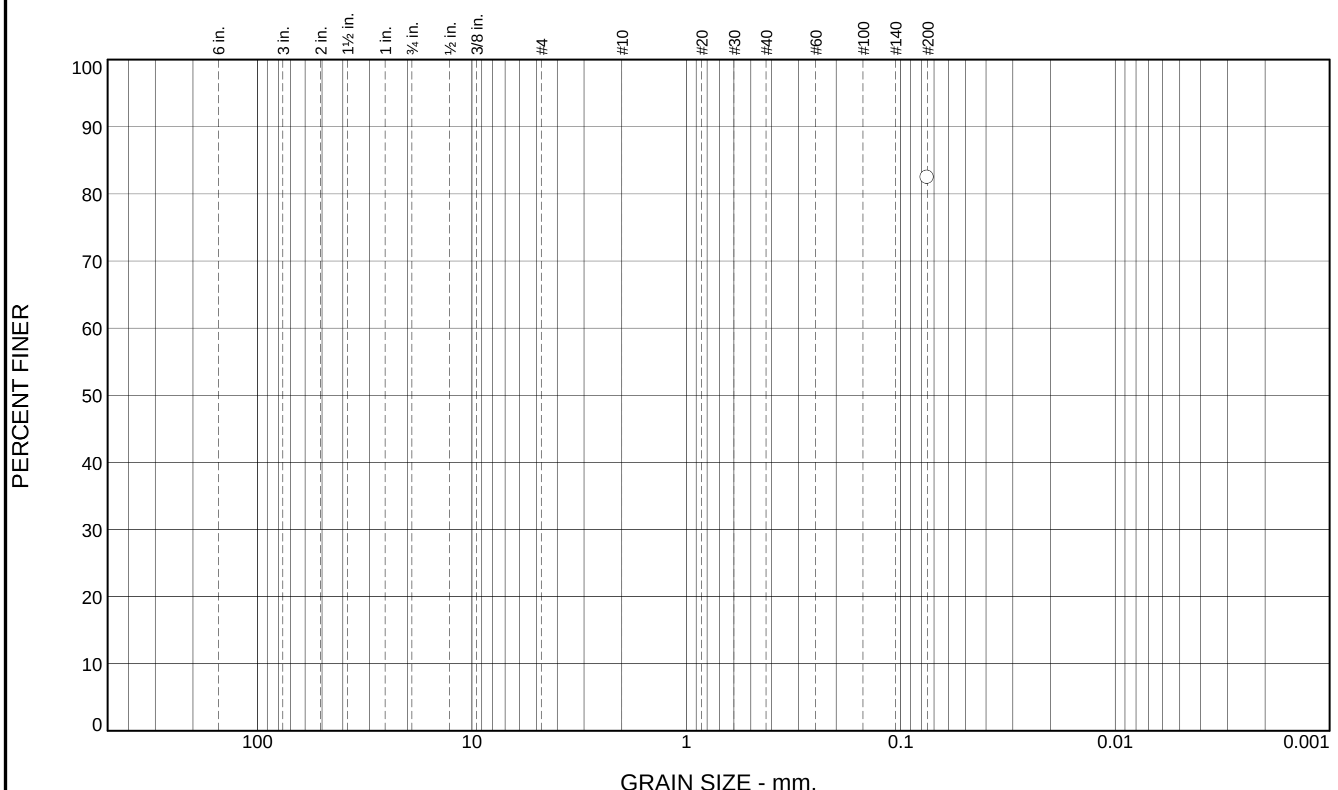
ENGEO INCORPORATED

Client: Tetra Tech, Inc.
Project: Atherton Water Capture Project

Project No: 14695.000.000

Tested By: M. Bromfield **Checked By:** G. Criste

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						82.4	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	82.4		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

ASTM D1140, Method B
Dry Sample Weight = 187.09; Soak Time = 2 hrs 10 mins

Sample Number: BH02 @ 16 **Depth:** 16 feet

Date: 3/29/2018

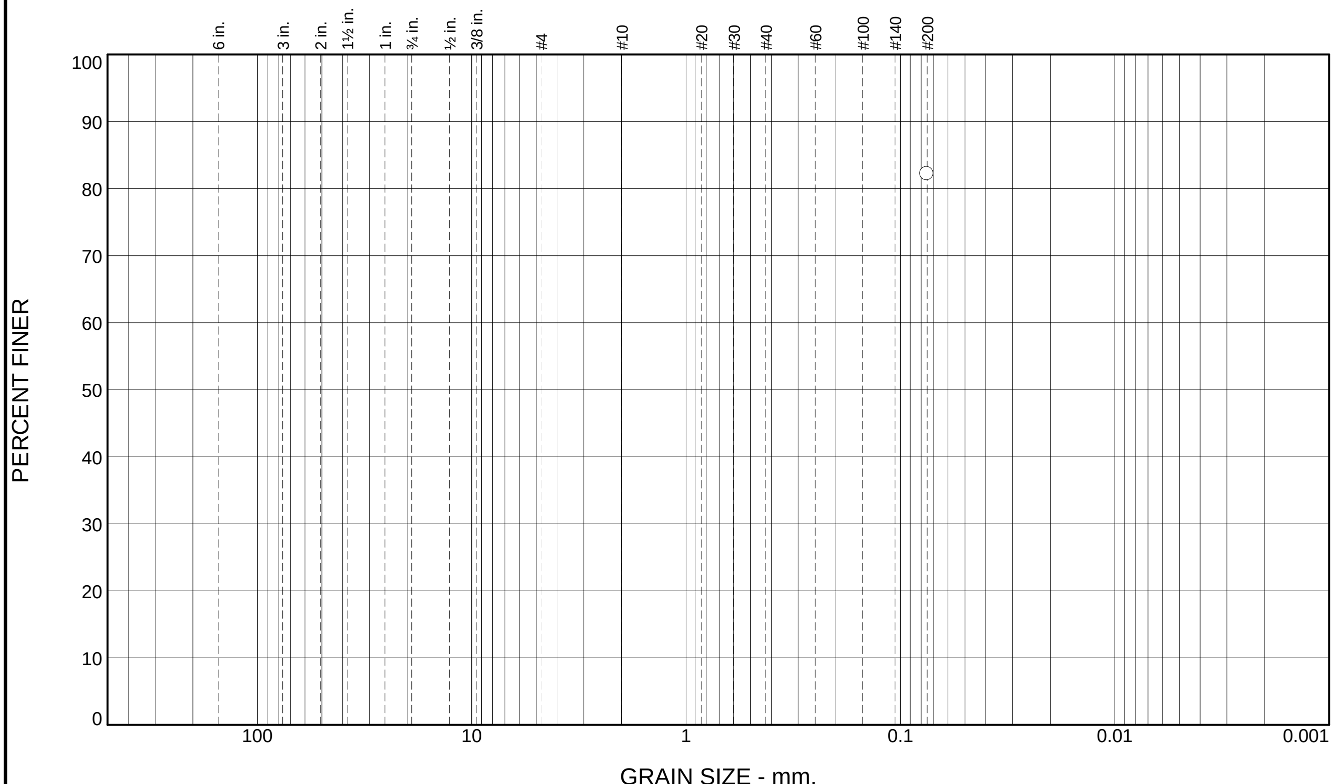
ENGEO INCORPORATED

Client: Tetra Tech, Inc.
Project: Atherton Water Capture Project

Project No: 14695.000.000

Tested By: M. Bromfield **Checked By:** G. Criste

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						82.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	82.2		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL= 18 LL= 36 PI= 18

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

PI: ASTM D4318, Wet method
GS: ASTM D1140, Method B
Dry Sample Weight = 268.82; Soak Time = 2 hrs 10 mins

Sample Number: BH02 @ 20.5 **Depth:** 20.5 feet

Date: 3/29/2018

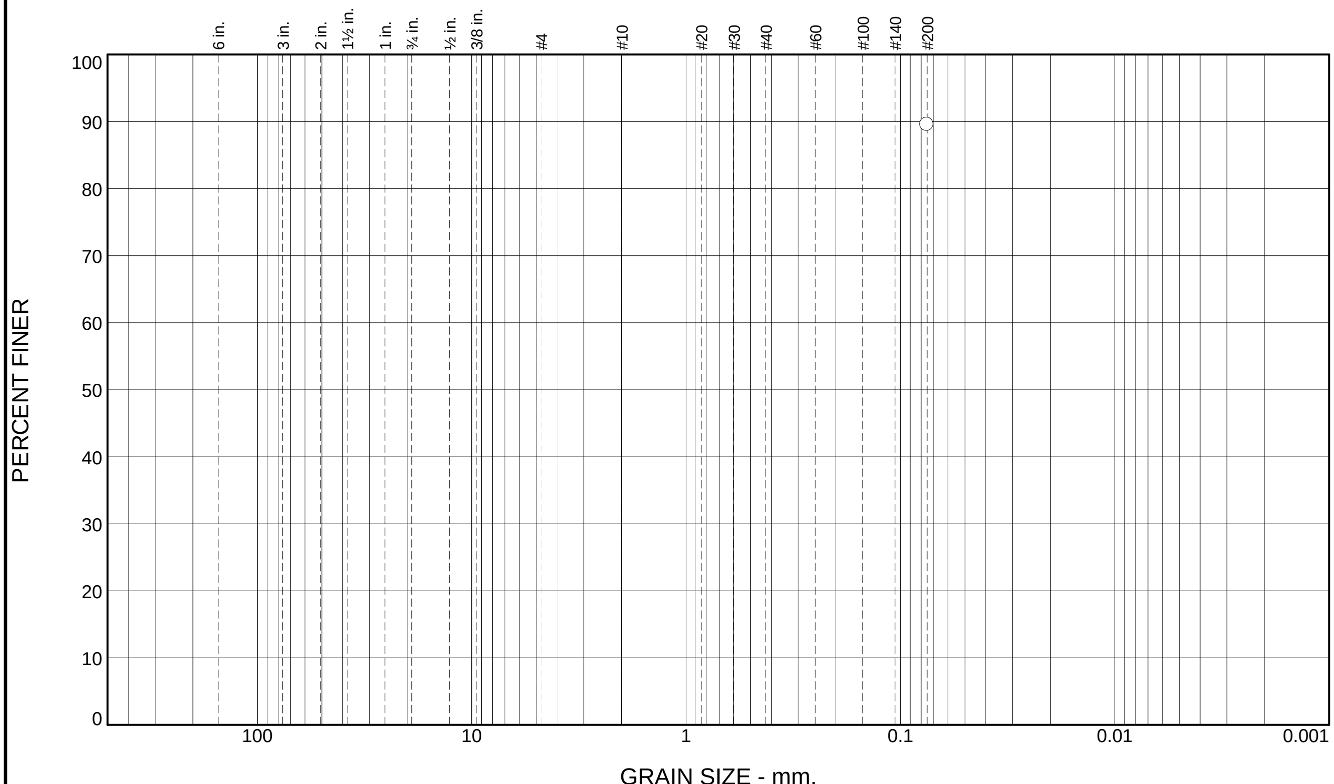
ENGEO INCORPORATED

Client: Tetra Tech, Inc.
Project: Atherton Water Capture Project

Project No: 14695.000.000

Tested By: M. Bromfield **Checked By:** G. Criste

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						89.5	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	89.5		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL= 18 LL= 57 PI= 39

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

PI: ASTM D4318, Wet method
GS: ASTM D1140, Method B
Dry Sample Weight = 258.2; Soak Time = 2 hrs 10 mins

Sample Number: BH03 @ 3 **Depth:** 3.0 feet

Date: 3/29/2018

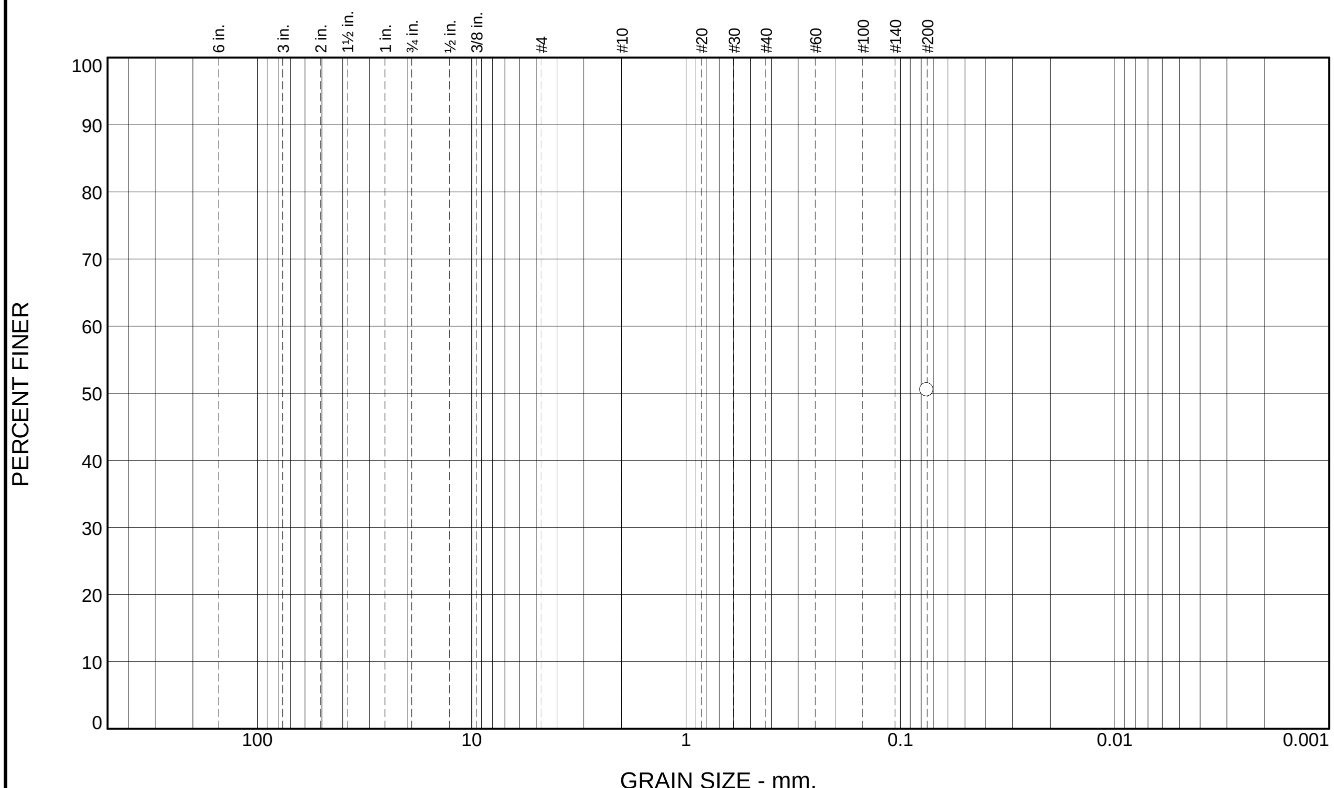
ENGEO INCORPORATED

Client: Tetra Tech, Inc.
Project: Atherton Water Capture Project

Project No: 14695.000.000

Tested By: M. Bromfield **Checked By:** G. Criste

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						50.4	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	50.4		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

ASTM D1140, Method A
Dry Sample Weight = 192.7; Soak Time = 2 hrs 10 mins

Sample Number: BH03 @ 18 **Depth:** 18 feet

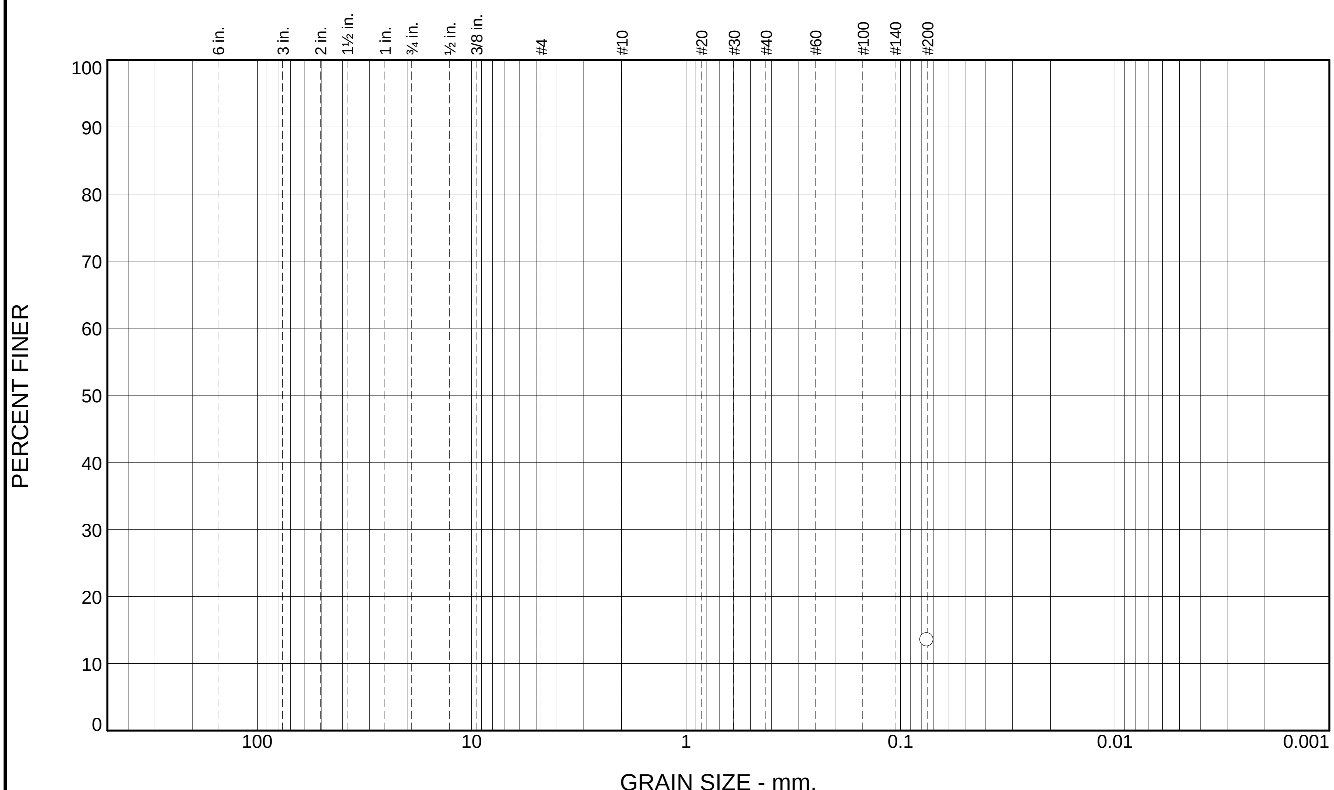
Date: 3/29/2018

ENGEO INCORPORATED

Client: Tetra Tech, Inc.
Project: Atherton Water Capture Project
Project No: 14695.000.000

Tested By: M. Bromfield **Checked By:** G. Criste

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						13.5	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	13.5		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

ASTM D1140, Method B
Dry Sample Weight = 201.38; Soak Time = 2 hrs 10 mins

Sample Number: BH03 @ 21-22 **Depth:** 21-22 feet

Date: 3/29/2018

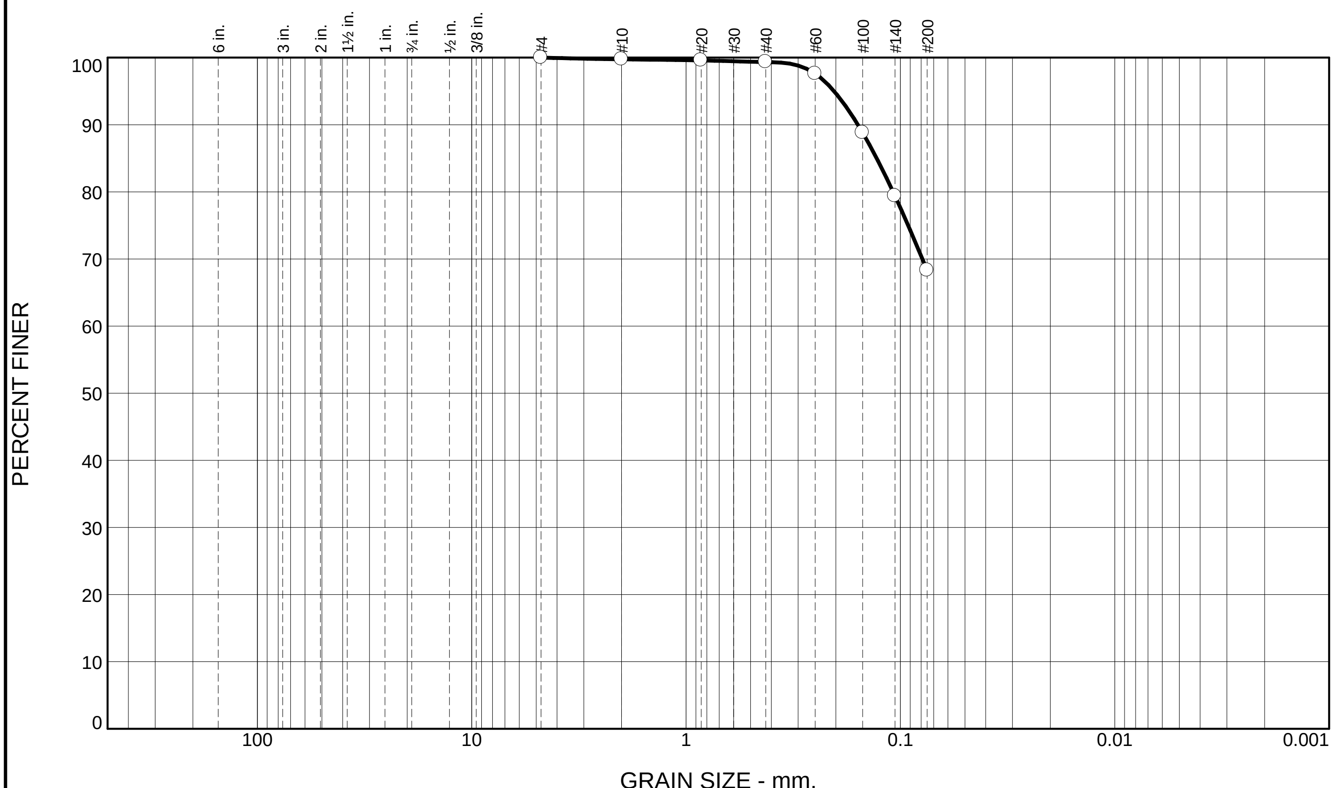
ENGEO INCORPORATED

Client: Tetra Tech, Inc.
Project: Atherton Water Capture Project

Project No: 14695.000.000

Tested By: M. Bromfield **Checked By:** G. Criste

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.3	0.4	31.0	68.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	99.7		
#20	99.6		
#40	99.3		
#60	97.6		
#100	88.8		
#140	79.4		
#200	68.3		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL= 21 LL= 32 PI= 11

Coefficients

D_{90}= 0.1579 D_{85}= 0.1291 D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CL AASHTO= A-6(6)

Remarks

PI: ASTM D4318, Wet method
GS: ASTM D6913, Method B
USCS: ASTM D2487

Sample Number: BH04 @ 21 **Depth:** 21.0 feet

Date: 3/29/2018

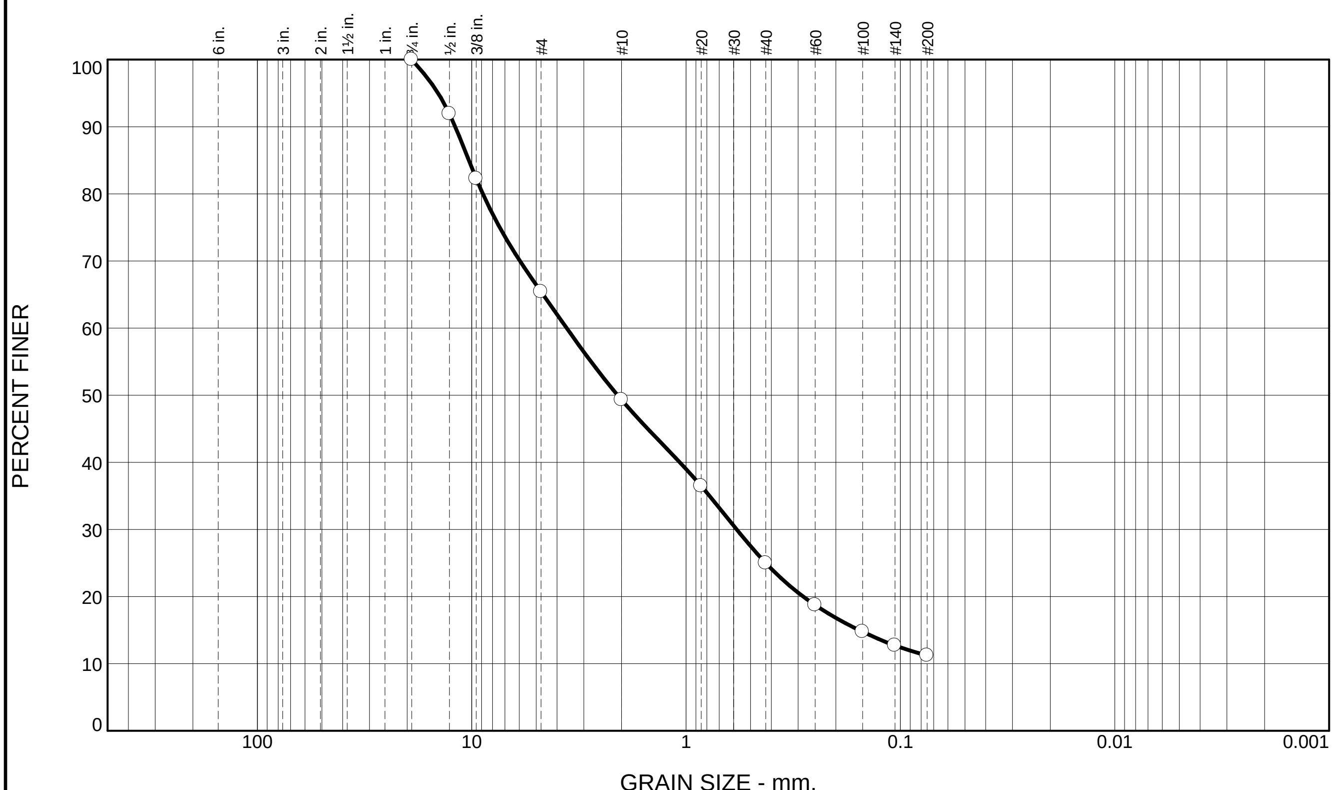
ENGEO INCORPORATED

Client: Tetra Tech, Inc.
Project: Atherton Water Capture Project

Project No: 14695.000.000

Tested By: M. Bromfield **Checked By:** G. Criste

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	34.6	16.1	24.3	13.8	11.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4	100.0		
1/2	91.9		
3/8	82.3		
#4	65.4		
#10	49.3		
#20	36.5		
#40	25.0		
#60	18.7		
#100	14.8		
#140	12.7		
#200	11.2		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 11.9314 D_{85}= 10.3153 D_{60}= 3.6081
D_{50}= 2.0864 D_{30}= 0.5805 D_{15}= 0.1556
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

ASTM D6913, Method A

Sample Number: BH05 @ 19-20 **Depth:** 19-20 feet

Date: 3/29/2018

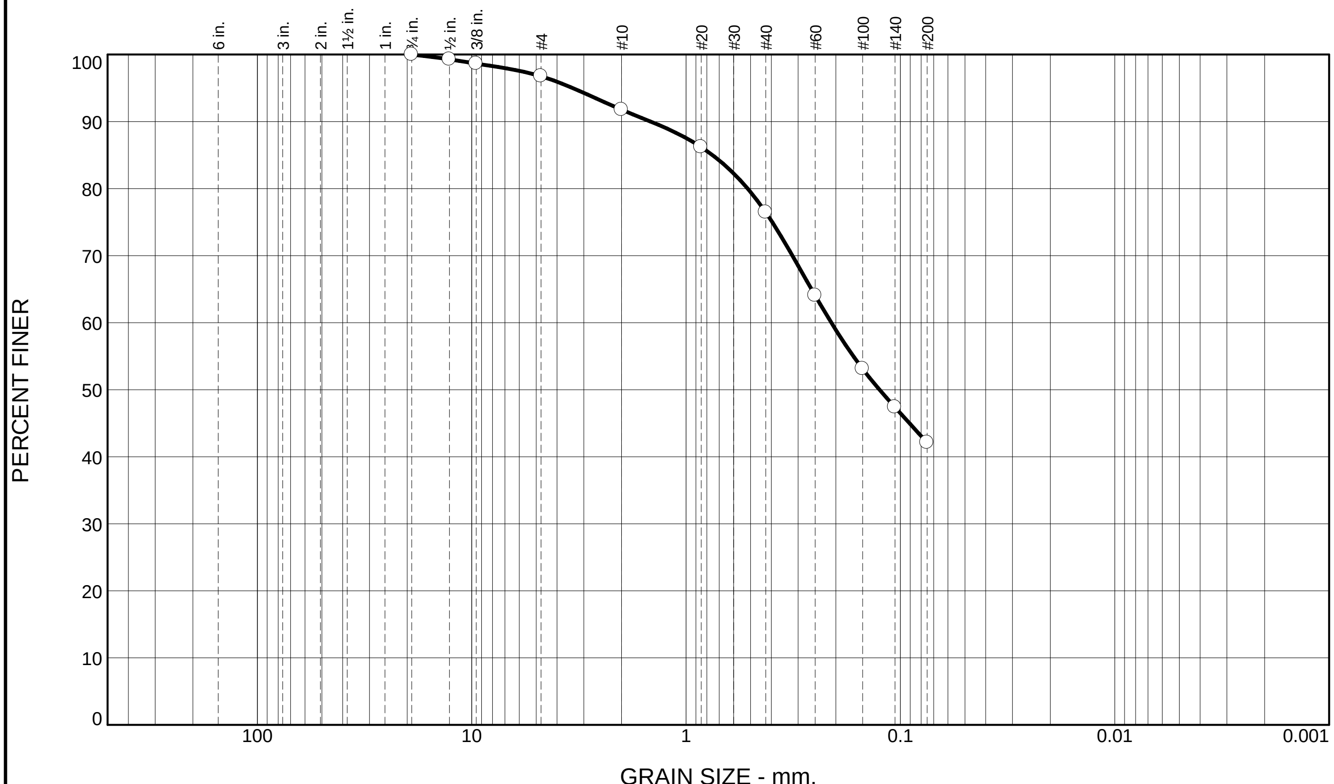
ENGEO INCORPORATED

Client: Tetra Tech, Inc.
Project: Atherton Water Capture Project

Project No: 14695.000.000

Tested By: M. Bromfield **Checked By:** G. Criste

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	3.2	5.0	15.3	34.4	42.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4	100.0		
1/2	99.3		
3/8	98.6		
#4	96.8		
#10	91.8		
#20	86.2		
#40	76.5		
#60	64.0		
#100	53.1		
#140	47.4		
#200	42.1		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 1.4624 D_{85}= 0.7524 D_{60}= 0.2101
D_{50}= 0.1252 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

ASTM D6913, Method A

Sample Number: BH06 @ 20 **Depth:** 20 feet

Date: 3/29/2018

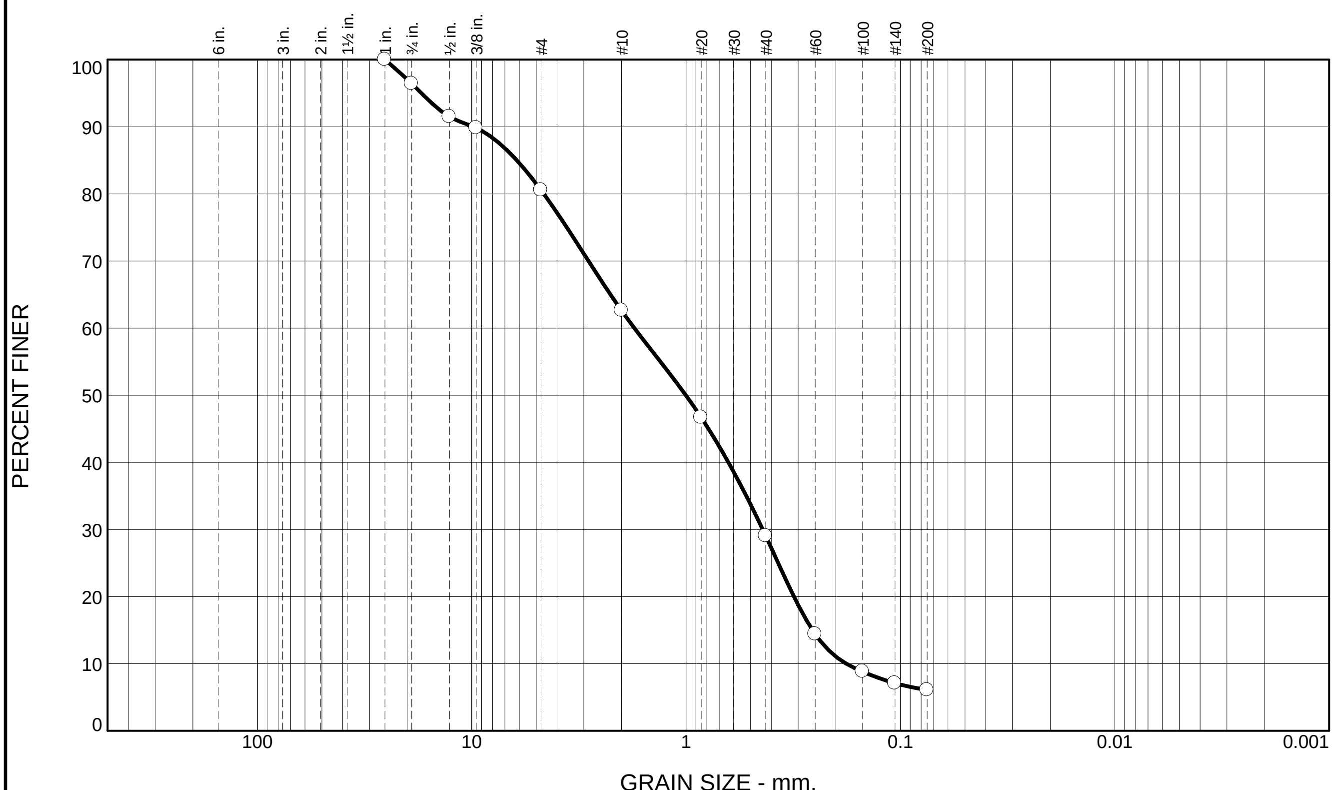
ENGEO INCORPORATED

Client: Tetra Tech, Inc.
Project: Atherton Water Capture Project

Project No: 14695.000.000

Tested By: M. Bromfield **Checked By:** G. Criste

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	3.6	15.8	18.0	33.6	22.9	6.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1	100.0		
3/4	96.4		
1/2	91.5		
3/8	89.8		
#4	80.6		
#10	62.6		
#20	46.7		
#40	29.0		
#60	14.4		
#100	8.8		
#140	7.1		
#200	6.1		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 9.8155 D_{85}= 6.1610 D_{60}= 1.7418
D_{50}= 1.0030 D_{30}= 0.4392 D_{15}= 0.2574
D_{10}= 0.1788 C_u= 9.74 C_c= 0.62

Classification

USCS= AASHTO=

Remarks

ASTM D6913, Method A

Sample Number: BH06 @ 25-26.5 **Depth:** 25-26.5 feet

Date: 3/29/2018

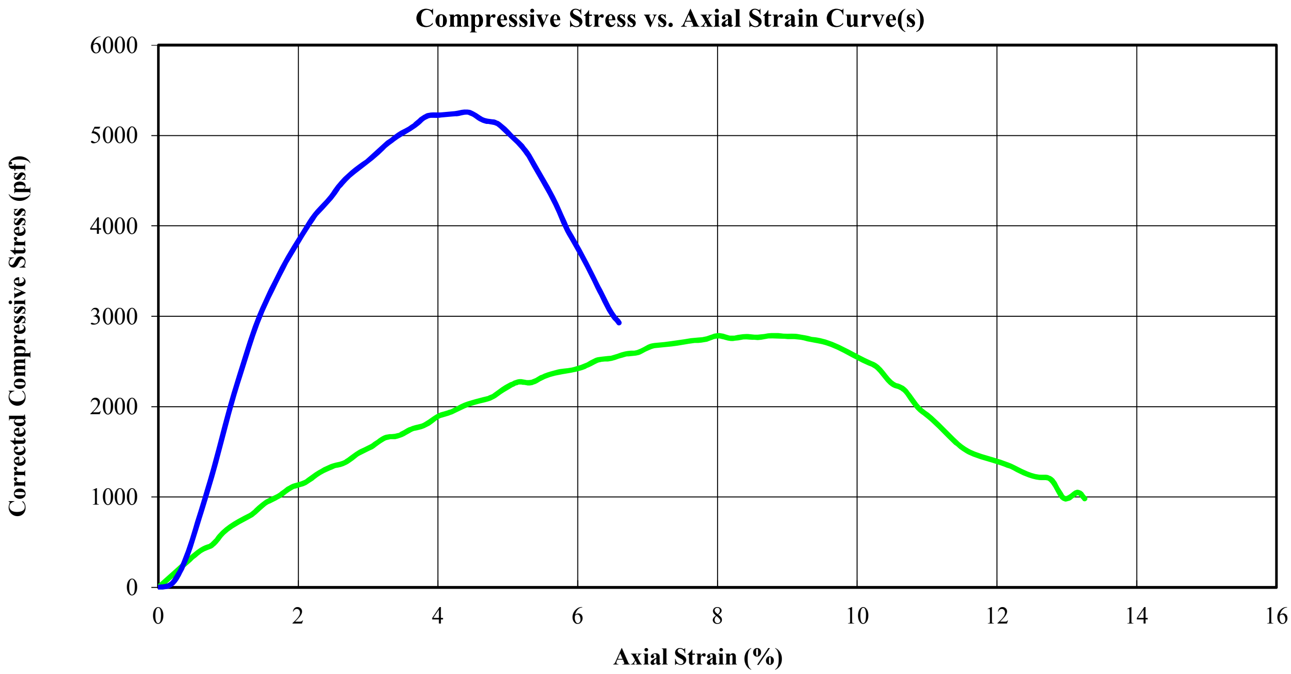
ENGEO INCORPORATED

Client: Tetra Tech, Inc.
Project: Atherton Water Capture Project

Project No: 14695.000.000

Tested By: M. Bromfield **Checked By:** G. Criste

UNCONFINED COMPRESSION TEST REPORT
(ASTM D2166)



BEFORE TEST	SPECIMEN BH01 @ 8	SPECIMEN BH03 @ 8
Moisture Content (%)	21.3	18.4
Dry Density (pcf)	105.4	105.8
Saturation (%)	100.0	100.0
Void Ratio	0.56	0.46
Diameter (in)	2.396	2.385
Height (in)	5.29	4.99
Height-To-Diameter Ratio	2.21	2.09
TEST DATA		
Unconfined Compressive Strength (psf)	2785	5256
Undrained Shear Strength (psf)	1393	2628
Strain Rate (in./min.)	0.05	0.05
Specific Gravity (Assumed)	2.650	2.650
Strain at Failure (%)	8.77	4.45
Liquid Limit		
Plastic Limit		
Test Remarks		

SPECIMEN	DESCRIPTION
BH01 @ 8	See exploration logs
BH03 @ 8	See exploration logs

PROJECT NAME: Atherton Water Capture Project
PROJECT NO: 14695.000.000
CLIENT: Tetra Tech, Inc.
LOCATION: Atherton, CA
PHASE NO: 001

Test Date: 03/30/18
Tested By: M. Bromfield
Reviewed By: G. Criste

ENGEO
Expect Excellence
3420 Fostoria Way, Suite E, San Ramon, CA 94583 | T (925) 355-9047 | F (888) 279-2698 | www.engeo.com

California State Certified Laboratory No. 2153

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 462 2771 Fax. 925 462 2775
www.cercoanalytical.com

Client: ENGEO Incorporated
Client's Project No.: 14695.000.000
Client's Project Name: Atherton Water Capture Project
Date Sampled: 22-Mar-18
Date Received: 23-Mar-18
Matrix: Soil
Authorization: Signed Chain of Custody

Date of Report: 28-Mar-2018

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Conductivity (umhos/cm)*	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
1803172-001	BH03 @ 0-2'	460	7.92	-	820	N.D.	28	20
1803172-002	BH05 @ 8.5-10'	450	8.19	-	1,200	N.D.	N.D.	N.D.

	Redox	pH	Conductivity	Resistivity	Sulfide	Chloride	Sulfate
Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Reporting Limit:	-	-	10	-	50	15	15
	27-Mar-2018	27-Mar-2018	-	27-Mar-2018	26-Mar-2018	27-Mar-2018	27-Mar-2018

* Results Reported on "As Received" Basis

N.D. - None Detected

Cheryl McMillen
Laboratory Director

Quality Control Summary - All laboratory quality control parameters were found to be within established limits

1803172 10169

CHAIN OF COSTODY RECORD

PROJECT NUMBER:	PROJECT NAME:
14695.000.000	Atherton Water Capture Project

SAMPLED BY: (SIGNATURE/PRINT) **Teresa Klotzback**

PROJECT MANAGER: **Andy Firmin**

ROUTING: E-MAIL afirmin@engeo.com Hard Copy **Andy Firmin**

	SAMPLE NUMBER	DATE	TIME	MATRIX	NUMBER OF CONTAINERS	CONTAINER SIZE	PRESERVATIVE	Redox	pH	Sulfate	Resistivity	Chloride	Sulfide	REMARKS REQUIRED DETECTION LIMITS
001	BH03 @ 0 - 2'	22-Mar-18		Soil	1	Baggie	None	x	x	x	x	x	x	ASTM Test Methods, no brief evaluation
002	BH05 @ 8.5 - 10	22-Mar-18		Soil	1	Baggie	None	x	x	x	x	x	x	ASTM Test Methods, no brief evaluation

RELINQUISHED BY: (SIGNATURE)	DATE/TIME	RECEIVED BY: (SIGNATURE)	RELINQUISHED BY: (SIGNATURE)	DATE/TIME	RECEIVED BY: (SIGNATURE)
[signature]	3/23/18 1215	[signature]	[signature]	3/23/18 1330	

RELINQUISHED BY: (SIGNATURE)	DATE/TIME	RECEIVED BY: (SIGNATURE)	RELINQUISHED BY: (SIGNATURE)	DATE/TIME	RECEIVED BY: (SIGNATURE)

RELINQUISHED BY: (SIGNATURE)	DATE/TIME	RECEIVED FOR LABORATORY BY: (SIGNATURE)	DATE/TIME	REMARKS
				STANDARD 10 DAY TURNAROUND

ENGEO INCORPORATED

2010 Crow Canyon Place Suite 250
San Ramon, CALIFORNIA 94583
925-866-9000 FAX (888) 279-2698
WWW.ENGEO.COM

DISTRIBUTION: ORIGINAL ACCOMPANIES SHIPMENT; COPY TO PROJECT FIELD FILES

APPENDIX D

PERCOLATION TESTING RESULTS

Atherton Water Capture

BH01 Water Level Meter Data

Date and Time	Inches of Water
3/7/2018 9:10	130.2
3/7/2018 9:15	127.8
3/7/2018 9:20	125.4
3/7/2018 9:25	123
3/7/2018 9:26	132.6
3/7/2018 9:28	131.4
3/7/2018 9:30	130.2
3/7/2018 9:35	127.2
3/7/2018 9:40	124.8
3/7/2018 9:45	122.4
3/7/2018 9:46	133.2
3/7/2018 9:50	130.5
3/7/2018 9:55	127.8
3/7/2018 10:55	108.6
3/7/2018 11:56	95.64
3/7/2018 12:56	85.2
3/7/2018 13:56	77.4
3/7/2018 14:56	70.8
3/7/2018 15:56	68.4

Atherton Water Capture
BH01 Vibrating Wire Piezometer Data

Date and Time	Inches of Water	Instrument Temp. (F)
3/7/2018 9:10	126.696	56.66
3/7/2018 9:11	125.904	55.94
3/7/2018 9:12	124.884	55.22
3/7/2018 9:13	124.068	54.68
3/7/2018 9:14	123.444	54.32
3/7/2018 9:15	122.748	54.14
3/7/2018 9:16	122.136	53.96
3/7/2018 9:17	121.416	53.6
3/7/2018 9:18	120.684	53.42
3/7/2018 9:19	119.928	53.42
3/7/2018 9:20	119.22	53.24
3/7/2018 9:21	118.608	53.24
3/7/2018 9:22	117.96	53.24
3/7/2018 9:23	117.444	53.24
3/7/2018 9:24	116.916	53.06
3/7/2018 9:25	116.388	53.06
3/7/2018 9:26	125.364	53.06
3/7/2018 9:27	124.692	53.06
3/7/2018 9:28	124.032	53.06
3/7/2018 9:29	123.336	53.06
3/7/2018 9:30	122.676	53.24
3/7/2018 9:31	122.1	53.24
3/7/2018 9:32	121.464	53.24
3/7/2018 9:33	120.816	53.24
3/7/2018 9:34	120.276	53.24
3/7/2018 9:35	119.7	53.24
3/7/2018 9:36	119.22	53.24
3/7/2018 9:37	118.68	53.24
3/7/2018 9:38	118.14	53.24
3/7/2018 9:39	117.732	53.42
3/7/2018 9:40	117.228	53.42
3/7/2018 9:41	116.724	53.42
3/7/2018 9:42	116.244	53.42
3/7/2018 9:43	115.776	53.42
3/7/2018 9:44	115.332	53.42
3/7/2018 9:45	114.876	53.42
3/7/2018 9:46	124.608	53.42
3/7/2018 9:47	124.128	53.6
3/7/2018 9:48	123.444	53.6
3/7/2018 9:49	122.736	53.6
3/7/2018 9:50	122.088	53.6
3/7/2018 9:51	121.428	53.6
3/7/2018 9:52	120.84	53.6
3/7/2018 9:53	120.24	53.78
3/7/2018 9:54	119.652	53.78
3/7/2018 9:55	119.112	53.78
3/7/2018 9:56	118.62	53.78
3/7/2018 9:57	118.08	53.78
3/7/2018 9:58	117.552	53.78
3/7/2018 9:59	117.084	53.78
3/7/2018 10:00	116.532	53.78
3/7/2018 10:01	116.088	53.96
3/7/2018 10:02	115.608	53.96
3/7/2018 10:03	115.236	53.96
3/7/2018 10:04	114.78	53.96
3/7/2018 10:05	114.36	53.96

Date and Time	Inches of Water	Instrument Temp. (F)
3/7/2018 10:06	113.88	53.96
3/7/2018 10:07	113.46	53.96
3/7/2018 10:08	113.052	53.96
3/7/2018 10:09	112.716	54.14
3/7/2018 10:10	112.332	54.14
3/7/2018 10:11	112.02	54.14
3/7/2018 10:12	111.72	54.14
3/7/2018 10:13	111.468	54.14
3/7/2018 10:14	111.096	54.14
3/7/2018 10:15	110.724	54.14
3/7/2018 10:16	110.34	54.14
3/7/2018 10:17	110.064	54.14
3/7/2018 10:18	109.8	54.14
3/7/2018 10:19	109.56	54.14
3/7/2018 10:20	109.32	54.32
3/7/2018 10:21	109.104	54.32
3/7/2018 10:22	108.852	54.32
3/7/2018 10:23	108.612	54.32
3/7/2018 10:24	108.36	54.32
3/7/2018 10:25	108.084	54.32
3/7/2018 10:26	107.844	54.32
3/7/2018 10:27	107.508	54.32
3/7/2018 10:28	107.184	54.32
3/7/2018 10:29	106.764	54.32
3/7/2018 10:30	106.344	54.32
3/7/2018 10:31	105.684	54.32
3/7/2018 10:32	105.108	54.5
3/7/2018 10:33	104.76	54.5
3/7/2018 10:34	104.448	54.5
3/7/2018 10:35	104.064	54.5
3/7/2018 10:36	103.752	54.5
3/7/2018 10:37	103.392	54.5
3/7/2018 10:38	103.056	54.5
3/7/2018 10:39	102.672	54.5
3/7/2018 10:40	102.228	54.5
3/7/2018 10:41	101.832	54.5
3/7/2018 10:42	101.388	54.5
3/7/2018 10:43	100.968	54.5
3/7/2018 10:44	100.632	54.68
3/7/2018 10:45	100.188	54.68
3/7/2018 10:46	99.732	54.68
3/7/2018 10:47	99.324	54.68
3/7/2018 10:48	98.988	54.68
3/7/2018 10:49	98.556	54.68
3/7/2018 10:50	98.292	54.68
3/7/2018 10:51	98.028	54.68
3/7/2018 10:52	97.692	54.68
3/7/2018 10:53	97.416	54.68
3/7/2018 10:54	97.176	54.68
3/7/2018 10:55	97.056	54.68
3/7/2018 10:56	96.852	54.68
3/7/2018 10:57	96.708	54.68
3/7/2018 10:58	96.552	54.68
3/7/2018 10:59	96.336	54.68
3/7/2018 11:00	96.204	54.86
3/7/2018 11:01	96.084	54.86

Atherton Water Capture
BH01 Vibrating Wire Piezometer Data

Date and Time	Inches of Water	Instrument Temp. (F)
3/7/2018 11:02	95.904	54.86
3/7/2018 11:03	95.652	54.86
3/7/2018 11:04	95.388	54.86
3/7/2018 11:05	95.076	54.86
3/7/2018 11:06	94.62	54.86
3/7/2018 11:07	94.08	54.86
3/7/2018 11:08	93.624	54.86
3/7/2018 11:09	93.132	54.86
3/7/2018 11:10	92.676	54.86
3/7/2018 11:11	92.22	54.86
3/7/2018 11:12	91.8	54.86
3/7/2018 11:13	91.392	54.86
3/7/2018 11:14	90.948	54.86
3/7/2018 11:15	90.504	54.86
3/7/2018 11:16	90.132	54.86
3/7/2018 11:17	89.784	55.04
3/7/2018 11:18	89.388	55.04
3/7/2018 11:19	88.98	55.04
3/7/2018 11:20	88.644	55.04
3/7/2018 11:21	88.32	55.04
3/7/2018 11:22	88.008	55.04
3/7/2018 11:23	87.672	55.04
3/7/2018 11:24	87.372	55.04
3/7/2018 11:25	87.072	55.04
3/7/2018 11:26	86.772	55.04
3/7/2018 11:27	86.52	55.04
3/7/2018 11:28	86.232	55.04
3/7/2018 11:29	85.944	55.04
3/7/2018 11:30	85.656	55.04
3/7/2018 11:31	85.356	55.04
3/7/2018 11:32	85.008	55.04
3/7/2018 11:33	84.684	55.04
3/7/2018 11:34	84.384	55.04
3/7/2018 11:35	84.048	55.04
3/7/2018 11:36	83.7	55.04
3/7/2018 11:37	83.508	55.22
3/7/2018 11:38	83.244	55.22
3/7/2018 11:39	82.956	55.22
3/7/2018 11:40	82.668	55.22
3/7/2018 11:41	82.44	55.22
3/7/2018 11:42	82.164	55.22
3/7/2018 11:43	81.912	55.22
3/7/2018 11:44	81.6	55.22
3/7/2018 11:45	81.384	55.22
3/7/2018 11:46	81.072	55.22
3/7/2018 11:47	80.868	55.22
3/7/2018 11:48	80.556	55.22
3/7/2018 11:49	80.328	55.22
3/7/2018 11:50	80.112	55.22
3/7/2018 11:51	79.836	55.22
3/7/2018 11:52	79.5	55.22
3/7/2018 11:53	79.188	55.22
3/7/2018 11:54	78.984	55.22
3/7/2018 11:55	78.756	55.22
3/7/2018 11:56	78.516	55.22
3/7/2018 11:57	78.24	55.22
3/7/2018 11:58	78.06	55.22

Date and Time	Inches of Water	Instrument Temp. (F)
3/7/2018 11:59	77.856	55.22
3/7/2018 12:00	77.604	55.22
3/7/2018 12:01	77.412	55.4
3/7/2018 12:02	77.112	55.4
3/7/2018 12:03	76.86	55.4
3/7/2018 12:04	76.608	55.4
3/7/2018 12:05	76.368	55.4
3/7/2018 12:06	76.068	55.4
3/7/2018 12:07	75.924	55.4
3/7/2018 12:08	75.672	55.4
3/7/2018 12:09	75.468	55.4
3/7/2018 12:10	75.192	55.4
3/7/2018 12:11	75.036	55.4
3/7/2018 12:12	74.784	55.4
3/7/2018 12:13	74.604	55.4
3/7/2018 12:14	74.412	55.4
3/7/2018 12:15	74.16	55.4
3/7/2018 12:16	74.028	55.4
3/7/2018 12:17	73.836	55.4
3/7/2018 12:18	73.584	55.4
3/7/2018 12:19	73.44	55.4
3/7/2018 12:20	73.2	55.4
3/7/2018 12:21	72.96	55.4
3/7/2018 12:22	72.732	55.4
3/7/2018 12:23	72.468	55.4
3/7/2018 12:24	72.288	55.4
3/7/2018 12:25	72.132	55.4
3/7/2018 12:26	72	55.4
3/7/2018 12:27	71.856	55.4
3/7/2018 12:28	71.712	55.58
3/7/2018 12:29	71.544	55.58
3/7/2018 12:30	71.268	55.58
3/7/2018 12:31	71.04	55.58
3/7/2018 12:32	70.848	55.58
3/7/2018 12:33	70.632	55.58
3/7/2018 12:34	70.344	55.58
3/7/2018 12:35	70.116	55.58
3/7/2018 12:36	69.972	55.58
3/7/2018 12:37	69.768	55.58
3/7/2018 12:38	69.444	55.58
3/7/2018 12:39	69.24	55.58
3/7/2018 12:40	69.024	55.58
3/7/2018 12:41	68.868	55.58
3/7/2018 12:42	68.688	55.58
3/7/2018 12:43	68.592	55.58
3/7/2018 12:44	68.34	55.58
3/7/2018 12:45	68.232	55.58
3/7/2018 12:46	68.004	55.58
3/7/2018 12:47	67.86	55.58
3/7/2018 12:48	67.764	55.58
3/7/2018 12:49	67.572	55.58
3/7/2018 12:50	67.476	55.58
3/7/2018 12:51	67.32	55.58
3/7/2018 12:52	67.176	55.58
3/7/2018 12:53	67.032	55.76
3/7/2018 12:54	66.9	55.76
3/7/2018 12:55	66.78	55.76

Atherton Water Capture
BH01 Vibrating Wire Piezometer Data

Date and Time	Inches of Water	Instrument Temp. (F)
3/7/2018 12:56	66.612	55.76
3/7/2018 12:57	66.456	55.76
3/7/2018 12:58	66.288	55.76
3/7/2018 12:59	66.096	55.76
3/7/2018 13:00	65.952	55.76
3/7/2018 13:01	65.82	55.76
3/7/2018 13:02	65.784	55.76
3/7/2018 13:03	65.82	55.76
3/7/2018 13:04	65.88	55.76
3/7/2018 13:05	65.952	55.76
3/7/2018 13:06	66.06	55.76
3/7/2018 13:07	66.324	55.76
3/7/2018 13:08	66.492	55.76
3/7/2018 13:09	66.684	55.76
3/7/2018 13:10	66.972	55.76
3/7/2018 13:11	67.356	55.76
3/7/2018 13:12	67.752	55.76
3/7/2018 13:13	68.076	55.76
3/7/2018 13:14	68.388	55.76
3/7/2018 13:15	68.616	55.76
3/7/2018 13:16	68.868	55.76
3/7/2018 13:17	69.072	55.76
3/7/2018 13:18	69.324	55.76
3/7/2018 13:19	69.516	55.76
3/7/2018 13:20	69.66	55.76
3/7/2018 13:21	69.792	55.76
3/7/2018 13:22	69.864	55.76
3/7/2018 13:23	70.02	55.94
3/7/2018 13:24	70.068	55.94
3/7/2018 13:25	70.176	55.94
3/7/2018 13:26	70.236	55.94
3/7/2018 13:27	70.26	55.94
3/7/2018 13:28	70.32	55.94
3/7/2018 13:29	70.32	55.94
3/7/2018 13:30	70.392	55.94
3/7/2018 13:31	70.38	55.94
3/7/2018 13:32	70.38	55.94
3/7/2018 13:33	70.416	55.94
3/7/2018 13:34	70.308	55.94
3/7/2018 13:35	70.272	55.94
3/7/2018 13:36	70.248	55.94
3/7/2018 13:37	70.248	55.94
3/7/2018 13:38	70.176	55.94
3/7/2018 13:39	70.164	55.94
3/7/2018 13:40	70.068	55.94
3/7/2018 13:41	70.056	55.94
3/7/2018 13:42	69.996	55.94
3/7/2018 13:43	69.936	55.94
3/7/2018 13:44	69.9	55.94
3/7/2018 13:45	69.828	55.94
3/7/2018 13:46	69.876	55.94
3/7/2018 13:47	69.78	55.94
3/7/2018 13:48	69.756	56.12
3/7/2018 13:49	69.708	56.12
3/7/2018 13:50	69.636	56.12
3/7/2018 13:51	69.576	56.12
3/7/2018 13:52	69.492	56.12

Date and Time	Inches of Water	Instrument Temp. (F)
3/7/2018 13:53	69.408	56.12
3/7/2018 13:54	69.348	56.12
3/7/2018 13:55	69.24	56.12
3/7/2018 13:56	69.168	56.12
3/7/2018 13:57	69.084	56.12
3/7/2018 13:58	69.012	56.12
3/7/2018 13:59	68.904	56.12
3/7/2018 14:00	68.82	56.12
3/7/2018 14:01	68.724	56.12
3/7/2018 14:02	68.652	56.12
3/7/2018 14:03	68.532	56.12
3/7/2018 14:04	68.424	56.12
3/7/2018 14:05	68.412	56.12
3/7/2018 14:06	68.304	56.12
3/7/2018 14:07	68.232	56.12
3/7/2018 14:08	68.1	56.12
3/7/2018 14:09	68.04	56.12
3/7/2018 14:10	67.932	56.12
3/7/2018 14:11	67.896	56.12
3/7/2018 14:12	67.788	56.12
3/7/2018 14:13	67.608	56.12
3/7/2018 14:14	67.512	56.12
3/7/2018 14:15	67.416	56.12
3/7/2018 14:16	67.26	56.12
3/7/2018 14:17	67.176	56.12
3/7/2018 14:18	66.984	56.12
3/7/2018 14:19	66.984	56.12
3/7/2018 14:20	66.876	56.3
3/7/2018 14:21	66.792	56.3
3/7/2018 14:22	66.648	56.3
3/7/2018 14:23	66.588	56.3
3/7/2018 14:24	66.444	56.3
3/7/2018 14:25	66.36	56.3
3/7/2018 14:26	66.264	56.3
3/7/2018 14:27	66.156	56.3
3/7/2018 14:28	66.036	56.3
3/7/2018 14:29	65.976	56.3
3/7/2018 14:30	65.916	56.3
3/7/2018 14:31	65.796	56.3
3/7/2018 14:32	65.664	56.3
3/7/2018 14:33	65.616	56.3
3/7/2018 14:34	65.556	56.3
3/7/2018 14:35	65.424	56.3
3/7/2018 14:36	65.268	56.3
3/7/2018 14:37	65.148	56.3
3/7/2018 14:38	65.088	56.3
3/7/2018 14:39	64.956	56.3
3/7/2018 14:40	64.812	56.3
3/7/2018 14:41	64.572	56.3
3/7/2018 14:42	64.452	56.3
3/7/2018 14:43	64.176	56.3
3/7/2018 14:44	63.9	56.3
3/7/2018 14:45	63.612	56.3
3/7/2018 14:46	63.3	56.3
3/7/2018 14:47	62.976	56.3
3/7/2018 14:48	62.568	56.3
3/7/2018 14:49	62.232	56.3

Atherton Water Capture
BH01 Vibrating Wire Piezometer Data

Date and Time	Inches of Water	Instrument Temp. (F)
3/7/2018 14:50	61.764	56.3
3/7/2018 14:51	61.32	56.48
3/7/2018 14:52	60.84	56.48
3/7/2018 14:53	60.288	56.48
3/7/2018 14:54	59.796	56.48
3/7/2018 14:55	59.376	56.48
3/7/2018 14:56	59.184	56.48
3/7/2018 14:57	59.208	56.48
3/7/2018 14:58	59.22	56.48
3/7/2018 14:59	59.304	56.48
3/7/2018 15:00	59.34	56.48
3/7/2018 15:01	59.34	56.48
3/7/2018 15:02	59.388	56.48
3/7/2018 15:03	59.388	56.48
3/7/2018 15:04	59.544	56.48
3/7/2018 15:05	59.616	56.48
3/7/2018 15:06	59.592	56.48
3/7/2018 15:07	59.472	56.48
3/7/2018 15:08	59.436	56.48
3/7/2018 15:09	59.316	56.48
3/7/2018 15:10	59.124	56.48
3/7/2018 15:11	59.04	56.48
3/7/2018 15:12	58.98	56.48
3/7/2018 15:13	58.836	56.48
3/7/2018 15:14	58.74	56.48
3/7/2018 15:15	58.764	56.48
3/7/2018 15:16	58.788	56.48
3/7/2018 15:17	58.872	56.48
3/7/2018 15:18	58.968	56.48
3/7/2018 15:19	59.148	56.48
3/7/2018 15:20	59.196	56.48
3/7/2018 15:21	59.076	56.48
3/7/2018 15:22	59.028	56.48
3/7/2018 15:23	58.992	56.48
3/7/2018 15:24	58.92	56.48
3/7/2018 15:25	58.992	56.66
3/7/2018 15:26	59.136	56.66
3/7/2018 15:27	59.508	56.66
3/7/2018 15:28	59.784	56.66
3/7/2018 15:29	60.048	56.66
3/7/2018 15:30	60.312	56.66
3/7/2018 15:31	60.504	56.66
3/7/2018 15:32	60.612	56.66
3/7/2018 15:33	60.756	56.66
3/7/2018 15:34	60.888	56.66
3/7/2018 15:35	61.08	56.66
3/7/2018 15:36	61.032	56.66
3/7/2018 15:37	60.876	56.66
3/7/2018 15:38	60.588	56.66
3/7/2018 15:39	60.228	56.66
3/7/2018 15:40	59.904	56.66
3/7/2018 15:41	59.532	56.66
3/7/2018 15:42	59.268	56.66
3/7/2018 15:43	59.064	56.66
3/7/2018 15:44	58.968	56.66
3/7/2018 15:45	59.088	56.66
3/7/2018 15:46	59.436	56.66

Date and Time	Inches of Water	Instrument Temp. (F)
3/7/2018 15:47	59.76	56.66
3/7/2018 15:48	60.036	56.66
3/7/2018 15:49	60.252	56.66
3/7/2018 15:50	60.516	56.66
3/7/2018 15:51	60.744	56.66
3/7/2018 15:52	60.924	56.66
3/7/2018 15:53	61.08	56.66
3/7/2018 15:54	61.248	56.66
3/7/2018 15:55	61.296	56.66
3/7/2018 15:56	61.464	56.66
3/7/2018 15:57	61.548	56.66
3/7/2018 15:58	61.656	56.66
3/7/2018 15:59	61.788	56.66
3/7/2018 16:00	61.836	56.66
3/7/2018 16:01	61.86	56.66
3/7/2018 16:02	61.848	56.66
3/7/2018 16:03	61.896	56.66
3/7/2018 16:04	61.968	56.84
3/7/2018 16:05	61.944	56.84
3/7/2018 16:06	61.968	56.84
3/7/2018 16:07	62.004	56.84
3/7/2018 16:08	62.028	56.84
3/7/2018 16:09	62.064	56.84

Atherton Water Capture

BH02 Water Level Meter Data

Date and Time	Inches of Water
3/8/2018 11:41	91.56
3/8/2018 11:42	90.48
3/8/2018 11:43	89.76
3/8/2018 11:44	88.8
3/8/2018 11:45	88.08
3/8/2018 11:50	85.8
3/8/2018 11:55	83.52
3/8/2018 12:00	81.6
3/8/2018 12:10	77.76
3/8/2018 12:20	73.92
3/8/2018 12:30	71.76
3/8/2018 13:00	66.48
3/8/2018 13:30	64.32
3/8/2018 14:00	63.6
3/8/2018 14:30	61.08
3/8/2018 15:00	60.12
3/8/2018 15:30	59.04

Atherton Water Capture
BH03 Water Level Meter Data

Date and Time	Inches of Water
3/7/2018 10:35	134.4
3/7/2018 10:36	132
3/7/2018 10:37	130.44
3/7/2018 10:38	128.4
3/7/2018 10:39	126.6
3/7/2018 10:40	126
3/7/2018 10:45	121.8
3/7/2018 10:50	119.64
3/7/2018 11:52	90.24
3/7/2018 12:53	80.76
3/7/2018 13:53	69.96
3/7/2018 14:53	60.6
3/7/2018 15:53	55.8

Atherton Water Capture
BH03 Vibrating Wire Piezometer Data

Date and Time	Inches of Water	Instrument Temp. (F)
3/7/2018 10:34	121.176	60.98
3/7/2018 10:35	118.068	59.9
3/7/2018 10:36	115.824	59
3/7/2018 10:37	114	58.46
3/7/2018 10:38	112.152	58.1
3/7/2018 10:39	110.724	57.92
3/7/2018 10:40	109.092	57.74
3/7/2018 10:41	107.604	57.56
3/7/2018 10:42	106.8	57.38
3/7/2018 10:43	105.768	57.2
3/7/2018 10:44	104.712	57.02
3/7/2018 10:45	104.532	57.02
3/7/2018 10:46	104.004	57.02
3/7/2018 10:47	103.512	57.02
3/7/2018 10:48	102.948	57.02
3/7/2018 10:49	102.6	56.84
3/7/2018 10:50	102.18	57.02
3/7/2018 10:51	101.856	57.02
3/7/2018 10:52	101.532	57.02
3/7/2018 10:53	101.232	57.02
3/7/2018 10:54	100.896	57.02
3/7/2018 10:55	100.776	57.02
3/7/2018 10:56	100.176	57.02
3/7/2018 10:57	99.888	57.02
3/7/2018 10:58	99.6	57.02
3/7/2018 10:59	99.24	57.02
3/7/2018 11:00	99.048	57.02
3/7/2018 11:01	98.688	57.02
3/7/2018 11:02	98.604	57.02
3/7/2018 11:03	98.7	57.02
3/7/2018 11:04	98.22	57.02
3/7/2018 11:05	96.42	57.02
3/7/2018 11:06	94.176	57.02
3/7/2018 11:07	91.86	57.2
3/7/2018 11:08	89.472	57.2
3/7/2018 11:09	87.84	57.2
3/7/2018 11:10	86.028	57.2
3/7/2018 11:11	84.828	57.2
3/7/2018 11:12	83.304	57.2
3/7/2018 11:13	82.2	57.2
3/7/2018 11:14	81.384	57.2
3/7/2018 11:15	80.904	57.2
3/7/2018 11:16	80.448	57.38
3/7/2018 11:17	79.932	57.38
3/7/2018 11:18	79.596	57.38
3/7/2018 11:19	79.032	57.38
3/7/2018 11:20	78.648	57.38
3/7/2018 11:21	78.348	57.38
3/7/2018 11:22	77.904	57.38
3/7/2018 11:23	77.544	57.56
3/7/2018 11:24	77.532	57.56
3/7/2018 11:25	77.1	57.56
3/7/2018 11:26	76.824	57.56
3/7/2018 11:27	76.5	57.56
3/7/2018 11:28	76.164	57.56
3/7/2018 11:29	76.164	57.56
3/7/2018 11:30	75.816	57.56

Date and Time	Inches of Water	Instrument Temp. (F)
3/7/2018 11:31	75.636	57.74
3/7/2018 11:32	75.528	57.74
3/7/2018 11:33	75.312	57.74
3/7/2018 11:34	74.796	57.74
3/7/2018 11:35	74.64	57.74
3/7/2018 11:36	74.556	57.74
3/7/2018 11:37	74.1	57.74
3/7/2018 11:38	73.86	57.74
3/7/2018 11:39	73.656	57.74
3/7/2018 11:40	73.44	57.92
3/7/2018 11:41	73.188	57.92
3/7/2018 11:42	72.936	57.92
3/7/2018 11:43	72.636	57.92
3/7/2018 11:44	72.468	57.92
3/7/2018 11:45	72.048	57.92
3/7/2018 11:46	71.856	57.92
3/7/2018 11:47	71.82	57.92
3/7/2018 11:48	71.292	57.92
3/7/2018 11:49	70.956	58.1
3/7/2018 11:50	71.016	58.1
3/7/2018 11:51	70.728	58.1
3/7/2018 11:52	70.452	58.1
3/7/2018 11:53	69.984	58.1
3/7/2018 11:54	69.96	58.1
3/7/2018 11:55	69.684	58.1
3/7/2018 11:56	69.384	58.1
3/7/2018 11:57	69	58.1
3/7/2018 11:58	68.856	58.1
3/7/2018 11:59	68.736	58.28
3/7/2018 12:00	68.304	58.28
3/7/2018 12:01	68.244	58.28
3/7/2018 12:02	67.908	58.28
3/7/2018 12:03	67.848	58.28
3/7/2018 12:04	67.488	58.28
3/7/2018 12:05	67.356	58.28
3/7/2018 12:06	67.02	58.28
3/7/2018 12:07	66.864	58.28
3/7/2018 12:08	66.804	58.28
3/7/2018 12:09	66.348	58.28
3/7/2018 12:10	66.504	58.28
3/7/2018 12:11	66.192	58.28
3/7/2018 12:12	65.808	58.46
3/7/2018 12:13	65.724	58.46
3/7/2018 12:14	65.676	58.46
3/7/2018 12:15	65.544	58.46
3/7/2018 12:16	65.364	58.46
3/7/2018 12:17	65.232	58.46
3/7/2018 12:18	64.992	58.46
3/7/2018 12:19	64.896	58.46
3/7/2018 12:20	64.74	58.46
3/7/2018 12:21	64.476	58.46
3/7/2018 12:22	64.176	58.46
3/7/2018 12:23	64.02	58.46
3/7/2018 12:24	63.78	58.46
3/7/2018 12:25	63.588	58.46
3/7/2018 12:26	63.732	58.64
3/7/2018 12:27	63.372	58.64

Atherton Water Capture
BH03 Vibrating Wire Piezometer Data

Date and Time	Inches of Water	Instrument Temp. (F)
3/7/2018 12:28	63.18	58.64
3/7/2018 12:29	62.964	58.64
3/7/2018 12:30	62.796	58.64
3/7/2018 12:31	62.7	58.64
3/7/2018 12:32	62.64	58.64
3/7/2018 12:33	62.244	58.64
3/7/2018 12:34	62.208	58.64
3/7/2018 12:35	62.088	58.64
3/7/2018 12:36	61.896	58.64
3/7/2018 12:37	61.68	58.64
3/7/2018 12:38	61.548	58.64
3/7/2018 12:39	61.452	58.64
3/7/2018 12:40	61.38	58.64
3/7/2018 12:41	61.104	58.64
3/7/2018 12:42	60.876	58.64
3/7/2018 12:43	60.744	58.82
3/7/2018 12:44	60.42	58.82
3/7/2018 12:45	60.252	58.82
3/7/2018 12:46	60.168	58.82
3/7/2018 12:47	59.82	58.82
3/7/2018 12:48	59.964	58.82
3/7/2018 12:49	59.844	58.82
3/7/2018 12:50	60	58.82
3/7/2018 12:51	59.736	58.82
3/7/2018 12:52	59.52	58.82
3/7/2018 12:53	59.364	58.82
3/7/2018 12:54	59.064	58.82
3/7/2018 12:55	58.86	58.82
3/7/2018 12:56	58.728	58.82
3/7/2018 12:57	58.512	59
3/7/2018 12:58	58.26	59
3/7/2018 12:59	58.164	59
3/7/2018 13:00	57.696	59
3/7/2018 13:01	57.684	59
3/7/2018 13:02	57.708	59
3/7/2018 13:03	57.66	59
3/7/2018 13:04	57.192	59
3/7/2018 13:05	57.336	59
3/7/2018 13:06	57.132	59
3/7/2018 13:07	56.82	59
3/7/2018 13:08	56.748	59
3/7/2018 13:09	56.736	59
3/7/2018 13:10	56.472	59
3/7/2018 13:11	56.208	59
3/7/2018 13:12	56.196	59
3/7/2018 13:13	55.92	59
3/7/2018 13:14	55.764	59
3/7/2018 13:15	55.56	59
3/7/2018 13:16	55.284	59
3/7/2018 13:17	55.2	59
3/7/2018 13:18	55.164	59.18
3/7/2018 13:19	55.092	59.18
3/7/2018 13:20	54.768	59.18
3/7/2018 13:21	54.564	59.18
3/7/2018 13:22	54.324	59.18
3/7/2018 13:23	54.264	59.18
3/7/2018 13:24	54.072	59.18

Date and Time	Inches of Water	Instrument Temp. (F)
3/7/2018 13:25	53.964	59.18
3/7/2018 13:26	53.7	59.18
3/7/2018 13:27	53.568	59.18
3/7/2018 13:28	53.256	59.18
3/7/2018 13:29	53.232	59.18
3/7/2018 13:30	53.016	59.18
3/7/2018 13:31	52.836	59.18
3/7/2018 13:32	52.392	59.18
3/7/2018 13:33	52.344	59.18
3/7/2018 13:34	51.996	59.18
3/7/2018 13:35	51.72	59.18
3/7/2018 13:36	51.504	59.18
3/7/2018 13:37	51.132	59.18
3/7/2018 13:38	50.904	59.18
3/7/2018 13:39	50.7	59.18
3/7/2018 13:40	50.484	59.18
3/7/2018 13:41	50.244	59.18
3/7/2018 13:42	50.16	59.18
3/7/2018 13:43	49.932	59.18
3/7/2018 13:44	49.704	59.36
3/7/2018 13:45	49.38	59.36
3/7/2018 13:46	49.26	59.36
3/7/2018 13:47	48.876	59.36
3/7/2018 13:48	48.588	59.36
3/7/2018 13:49	48.336	59.36
3/7/2018 13:50	48.408	59.36
3/7/2018 13:51	47.856	59.36
3/7/2018 13:52	47.796	59.36
3/7/2018 13:53	47.604	59.36
3/7/2018 13:54	47.112	59.36
3/7/2018 13:55	46.992	59.36
3/7/2018 13:56	46.596	59.36
3/7/2018 13:57	46.428	59.36
3/7/2018 13:58	46.332	59.36
3/7/2018 13:59	46.092	59.36
3/7/2018 14:00	45.96	59.36
3/7/2018 14:01	45.876	59.36
3/7/2018 14:02	45.516	59.36
3/7/2018 14:03	45.444	59.36
3/7/2018 14:04	45.144	59.36
3/7/2018 14:05	45.072	59.36
3/7/2018 14:06	44.568	59.36
3/7/2018 14:07	44.316	59.36
3/7/2018 14:08	44.46	59.36
3/7/2018 14:09	44.292	59.36
3/7/2018 14:10	43.968	59.36
3/7/2018 14:11	44.028	59.36
3/7/2018 14:12	43.68	59.36
3/7/2018 14:13	43.512	59.36
3/7/2018 14:14	43.74	59.54
3/7/2018 14:15	43.596	59.54
3/7/2018 14:16	43.404	59.54
3/7/2018 14:17	43.32	59.54
3/7/2018 14:18	43.032	59.54
3/7/2018 14:19	43.056	59.54
3/7/2018 14:20	42.792	59.54
3/7/2018 14:21	42.72	59.54

Atherton Water Capture
BH03 Vibrating Wire Piezometer Data

Date and Time	Inches of Water	Instrument Temp. (F)
3/7/2018 14:22	42.612	59.54
3/7/2018 14:23	42.3	59.54
3/7/2018 14:24	42.06	59.54
3/7/2018 14:25	41.964	59.54
3/7/2018 14:26	41.784	59.54
3/7/2018 14:27	41.712	59.54
3/7/2018 14:28	41.46	59.54
3/7/2018 14:29	41.376	59.54
3/7/2018 14:30	41.4	59.54
3/7/2018 14:31	41.304	59.54
3/7/2018 14:32	41.16	59.54
3/7/2018 14:33	40.896	59.54
3/7/2018 14:34	40.908	59.54
3/7/2018 14:35	40.596	59.54
3/7/2018 14:36	40.704	59.54
3/7/2018 14:37	40.392	59.54
3/7/2018 14:38	40.224	59.54
3/7/2018 14:39	40.044	59.54
3/7/2018 14:40	39.96	59.54
3/7/2018 14:41	39.744	59.54
3/7/2018 14:42	39.876	59.54
3/7/2018 14:43	39.576	59.54
3/7/2018 14:44	39.552	59.54
3/7/2018 14:45	39.396	59.54
3/7/2018 14:46	39.336	59.54
3/7/2018 14:47	39.012	59.54
3/7/2018 14:48	39.12	59.54
3/7/2018 14:49	39.156	59.54
3/7/2018 14:50	38.916	59.72
3/7/2018 14:51	39.06	59.72
3/7/2018 14:52	38.676	59.72
3/7/2018 14:53	38.688	59.72
3/7/2018 14:54	38.508	59.72
3/7/2018 14:55	38.556	59.72
3/7/2018 14:56	38.484	59.72
3/7/2018 14:57	38.304	59.72
3/7/2018 14:58	38.16	59.72
3/7/2018 14:59	38.076	59.72
3/7/2018 15:00	38.148	59.72
3/7/2018 15:01	38.028	59.72
3/7/2018 15:02	37.668	59.72
3/7/2018 15:03	37.68	59.72
3/7/2018 15:04	37.704	59.72
3/7/2018 15:05	37.62	59.72
3/7/2018 15:06	37.452	59.72
3/7/2018 15:07	37.212	59.72
3/7/2018 15:08	37.332	59.72
3/7/2018 15:09	37.164	59.72
3/7/2018 15:10	36.984	59.72
3/7/2018 15:11	36.96	59.72
3/7/2018 15:12	36.768	59.72
3/7/2018 15:13	36.816	59.72
3/7/2018 15:14	36.792	59.72
3/7/2018 15:15	36.744	59.72
3/7/2018 15:16	36.648	59.72
3/7/2018 15:17	36.516	59.72
3/7/2018 15:18	36.492	59.72

Date and Time	Inches of Water	Instrument Temp. (F)
3/7/2018 15:19	36.348	59.72
3/7/2018 15:20	36.36	59.72
3/7/2018 15:21	36.192	59.72
3/7/2018 15:22	36.276	59.72
3/7/2018 15:23	36.108	59.72
3/7/2018 15:24	36.072	59.72
3/7/2018 15:25	35.916	59.72
3/7/2018 15:26	35.808	59.72
3/7/2018 15:27	35.772	59.72
3/7/2018 15:28	35.4	59.72
3/7/2018 15:29	35.472	59.72
3/7/2018 15:30	35.352	59.72
3/7/2018 15:31	35.244	59.72
3/7/2018 15:32	35.28	59.72
3/7/2018 15:33	35.232	59.72
3/7/2018 15:34	35.076	59.72
3/7/2018 15:35	35.28	59.72
3/7/2018 15:36	35.112	59.72
3/7/2018 15:37	35.1	59.72
3/7/2018 15:38	34.944	59.72
3/7/2018 15:39	34.956	59.9
3/7/2018 15:40	34.824	59.9
3/7/2018 15:41	34.716	59.9
3/7/2018 15:42	34.416	59.9
3/7/2018 15:43	34.404	59.9
3/7/2018 15:44	34.524	59.9
3/7/2018 15:45	34.488	59.9
3/7/2018 15:46	34.224	59.9
3/7/2018 15:47	34.356	59.9
3/7/2018 15:48	34.452	59.9
3/7/2018 15:49	34.152	59.9
3/7/2018 15:50	34.308	59.9
3/7/2018 15:51	34.02	59.9
3/7/2018 15:52	34.02	59.9
3/7/2018 15:53	34.104	59.9
3/7/2018 15:54	33.804	59.9
3/7/2018 15:55	33.612	59.9
3/7/2018 15:56	33.468	59.9
3/7/2018 15:57	33.624	59.9
3/7/2018 15:58	33.144	59.9
3/7/2018 15:59	33.336	59.9
3/7/2018 16:00	33.12	59.9
3/7/2018 16:01	33.252	59.9
3/7/2018 16:02	33.3	59.9
3/7/2018 16:03	33.372	59.9
3/7/2018 16:04	33.24	59.9
3/7/2018 16:05	33.096	59.9
3/7/2018 16:06	33.048	59.9

Atherton Water Capture
BH04 Water Level Meter Data

Date and Time	Inches of Water
3/7/2018 11:35	157.8
3/7/2018 11:36	154.8
3/7/2018 11:38	148.44
3/7/2018 11:40	142.8
3/7/2018 11:42	137.16
3/7/2018 11:44	133.56
3/7/2018 11:46	128.4
3/7/2018 11:48	124.2
3/7/2018 11:50	121.08
3/7/2018 12:00	109.44
3/7/2018 12:10	104.4
3/7/2018 12:20	100.8
3/7/2018 12:30	99
3/7/2018 12:40	98.16
3/7/2018 12:50	96
3/7/2018 13:20	90
3/7/2018 13:50	86.88
3/7/2018 14:50	81
3/7/2018 15:50	78.12

Atherton Water Capture

BH05 Water Level Meter Data

Date and Time	Inches of Water
3/8/2018 9:28	
3/8/2018 10:20	19.8
3/8/2018 10:25	
3/8/2018 10:27	20.4
3/8/2018 10:35	
3/8/2018 10:37	
3/8/2018 10:38	28.8
3/8/2018 10:47	3.6
3/8/2018 11:00	98.4
3/8/2018 11:17	0

Atherton Water Capture
BH05 Vibrating Wire Piezometer Data

Date and Time	Inches of Water	Instrument Temp. (F)
3/8/2018 9:08	18.048	55.04
3/8/2018 9:09	18.06	55.04
3/8/2018 9:10	18.144	55.22
3/8/2018 9:11	18.096	55.22
3/8/2018 9:28	13.044	63.86
3/8/2018 9:29	13.032	63.86
3/8/2018 9:30	31.38	62.78
3/8/2018 9:31	19.812	61.52
3/8/2018 9:32	28.932	60.62
3/8/2018 9:33	13.92	59.72
3/8/2018 9:34	27.492	60.44
3/8/2018 9:35	38.652	59.9
3/8/2018 9:36	38.784	58.82
3/8/2018 9:37	40.248	58.1
3/8/2018 9:38	40.44	57.2
3/8/2018 9:39	41.148	56.48
3/8/2018 9:40	41.568	55.76
3/8/2018 9:41	41.58	55.4
3/8/2018 9:42	42.048	55.04
3/8/2018 9:43	42.396	54.68
3/8/2018 9:44	42.768	54.68
3/8/2018 9:45	42.852	54.5
3/8/2018 9:46	43.548	54.5
3/8/2018 9:47	43.884	54.5
3/8/2018 9:48	44.532	54.5
3/8/2018 9:49	44.748	54.5
3/8/2018 9:50	45.012	54.5
3/8/2018 9:51	45.612	54.5
3/8/2018 9:52	45.936	54.68
3/8/2018 9:53	47.592	54.68
3/8/2018 9:54	48.732	54.68
3/8/2018 9:55	49.692	54.68
3/8/2018 9:56	50.196	54.68
3/8/2018 9:57	51.072	54.5
3/8/2018 9:58	51.612	54.5
3/8/2018 9:59	52.332	54.5
3/8/2018 10:00	52.32	54.5
3/8/2018 10:01	52.836	54.5
3/8/2018 10:02	53.436	54.5
3/8/2018 10:03	53.676	54.5
3/8/2018 10:04	54.204	54.5
3/8/2018 10:05	54.384	54.5
3/8/2018 10:06	54.72	54.5
3/8/2018 10:07	55.272	54.5
3/8/2018 10:08	55.644	54.5
3/8/2018 10:09	56.016	54.5
3/8/2018 10:10	56.64	54.68
3/8/2018 10:11	56.7	54.68
3/8/2018 10:12	56.616	54.68
3/8/2018 10:13	56.772	54.68
3/8/2018 10:14	56.1	54.86
3/8/2018 10:15	55.86	54.86
3/8/2018 10:16	56.388	54.86
3/8/2018 10:17	56.496	55.04
3/8/2018 10:18	57.168	55.04
3/8/2018 10:19	56.976	55.04
3/8/2018 10:20	56.928	55.22

Date and Time	Inches of Water	Instrument Temp. (F)
3/8/2018 10:21	28.5	55.22
3/8/2018 10:22	21.888	55.4
3/8/2018 10:23	16.38	55.4
3/8/2018 10:24	11.172	55.4
3/8/2018 10:25	25.476	55.58
3/8/2018 10:26	62.496	56.48
3/8/2018 10:27	68.472	56.12
3/8/2018 10:28	28.896	55.94
3/8/2018 10:29	19.548	55.94
3/8/2018 10:30	12.372	55.76
3/8/2018 10:31	9.12	55.94
3/8/2018 10:32	8.232	55.94
3/8/2018 10:33	8.1	56.12
3/8/2018 10:34	7.98	56.12
3/8/2018 10:35	52.572	57.74
3/8/2018 10:36	64.596	57.02
3/8/2018 10:37	66.384	57.02
3/8/2018 10:38	28.956	56.84
3/8/2018 10:39	20.76	56.84
3/8/2018 10:40	13.188	56.84
3/8/2018 10:41	9.96	56.66
3/8/2018 10:42	9.408	56.84
3/8/2018 10:43	8.952	56.84
3/8/2018 10:44	8.892	57.02
3/8/2018 10:45	8.988	57.02
3/8/2018 10:46	8.94	57.2
3/8/2018 10:47	60.756	59
3/8/2018 10:48	69.288	58.1
3/8/2018 10:49	72.516	58.1
3/8/2018 10:50	76.572	57.92
3/8/2018 10:51	81.84	57.92
3/8/2018 10:52	85.428	57.92
3/8/2018 10:53	88.728	58.1
3/8/2018 10:54	92.46	58.1
3/8/2018 10:55	96.504	58.28
3/8/2018 10:56	99.744	58.46
3/8/2018 10:57	105.684	58.64
3/8/2018 10:58	124.584	59
3/8/2018 10:59	157.668	59.18
3/8/2018 11:00	86.292	59.36
3/8/2018 11:01	59.856	59.54
3/8/2018 11:02	42.696	59.9
3/8/2018 11:03	26.292	60.08
3/8/2018 11:04	18.792	60.26
3/8/2018 11:05	15.096	60.26
3/8/2018 11:06	13.344	60.44
3/8/2018 11:07	11.688	60.44
3/8/2018 11:08	10.704	60.44
3/8/2018 11:09	9.876	60.44
3/8/2018 11:10	9.396	60.44
3/8/2018 11:11	9.204	60.44
3/8/2018 11:12	9.216	60.44
3/8/2018 11:13	9.096	60.44
3/8/2018 11:14	8.928	60.44
3/8/2018 11:15	8.796	60.44
3/8/2018 11:16	8.772	60.44
3/8/2018 11:17	8.712	60.44

Atherton Water Capture

BH06 Water Level Meter Data

Date and Time	Inches of Water
3/8/2018 8:22	58.8
3/8/2018 8:23	54.6
3/8/2018 8:24	52.56
3/8/2018 8:25	50.28
3/8/2018 8:26	47.76
3/8/2018 8:27	45.96
3/8/2018 8:28	44.4
3/8/2018 8:29	42.96
3/8/2018 8:30	41.76
3/8/2018 8:35	36.36
3/8/2018 8:40	31.8
3/8/2018 8:45	28.56
3/8/2018 8:50	25.56
3/8/2018 8:55	23.28
3/8/2018 9:00	66
3/8/2018 9:45	76.8
3/8/2018 10:30	78
3/8/2018 11:05	42.84
3/8/2018 11:31	98.4
3/8/2018 12:15	36.6
3/8/2018 12:46	26.76
3/8/2018 12:50	70.8
3/8/2018 13:20	37.68
3/8/2018 13:50	27.24
3/8/2018 14:20	21.48

Atherton Water Capture
BH06 Vibrating Wire Piezometer Data

Date and Time	Inches of Water	Instrument Temp. (F)
3/8/2018 8:13	24.912	54.14
3/8/2018 8:14	24.996	54.14
3/8/2018 8:15	17.952	54.14
3/8/2018 8:21	58.848	55.76
3/8/2018 8:22	58.128	55.76
3/8/2018 8:23	55.056	55.76
3/8/2018 8:24	52.116	55.94
3/8/2018 8:25	49.632	56.12
3/8/2018 8:26	47.628	56.12
3/8/2018 8:27	45.456	56.3
3/8/2018 8:28	44.196	56.48
3/8/2018 8:29	43.236	56.48
3/8/2018 8:30	41.676	56.66
3/8/2018 8:31	40.788	56.66
3/8/2018 8:32	39.168	56.66
3/8/2018 8:33	37.944	56.84
3/8/2018 8:34	36.816	56.84
3/8/2018 8:35	35.772	56.84
3/8/2018 8:36	35.04	57.02
3/8/2018 8:37	34.032	57.02
3/8/2018 8:38	33.276	57.02
3/8/2018 8:39	32.292	57.2
3/8/2018 8:40	31.404	57.2
3/8/2018 8:41	30.612	57.38
3/8/2018 8:42	29.988	57.38
3/8/2018 8:43	29.34	57.56
3/8/2018 8:44	28.584	57.56
3/8/2018 8:45	27.864	57.56
3/8/2018 8:46	27.456	57.74
3/8/2018 8:47	26.568	57.74
3/8/2018 8:48	26.292	57.92
3/8/2018 8:49	25.536	57.92
3/8/2018 8:50	25.2	58.1
3/8/2018 8:51	24.564	58.1
3/8/2018 8:52	24.192	58.28
3/8/2018 8:53	23.724	58.46
3/8/2018 8:54	23.22	58.46
3/8/2018 8:55	23.244	58.46
3/8/2018 8:56	22.452	58.64
3/8/2018 8:57	22.668	58.64
3/8/2018 8:58	23.172	58.82
3/8/2018 8:59	25.02	59
3/8/2018 9:00	70.212	59.18
3/8/2018 9:01	66.096	59
3/8/2018 9:02	64.752	58.64
3/8/2018 9:03	61.956	58.46
3/8/2018 9:04	59.016	58.1
3/8/2018 9:05	56.736	58.1
3/8/2018 9:06	54.648	57.92
3/8/2018 9:07	52.908	57.92
3/8/2018 9:08	51.132	57.92
3/8/2018 9:09	49.584	57.92
3/8/2018 9:10	47.688	58.1
3/8/2018 9:11	46.692	58.1
3/8/2018 9:12	45.192	58.1
3/8/2018 9:13	43.956	58.1
3/8/2018 9:14	42.828	58.28

Date and Time	Inches of Water	Instrument Temp. (F)
3/8/2018 9:15	42.108	58.28
3/8/2018 9:16	41.076	58.28
3/8/2018 9:17	40.056	58.28
3/8/2018 9:18	38.844	58.28
3/8/2018 9:19	37.98	58.28
3/8/2018 9:20	36.924	58.28
3/8/2018 9:21	36.492	58.46
3/8/2018 9:22	35.256	58.46
3/8/2018 9:23	34.632	58.46
3/8/2018 9:24	33.948	58.46
3/8/2018 9:25	33.3	58.46
3/8/2018 9:26	32.364	58.46
3/8/2018 9:27	31.812	58.46
3/8/2018 9:28	31.224	58.46
3/8/2018 9:29	30.648	58.64
3/8/2018 9:30	30.084	58.64
3/8/2018 9:31	29.52	58.64
3/8/2018 9:32	28.644	58.64
3/8/2018 9:33	28.212	58.64
3/8/2018 9:34	27.732	58.64
3/8/2018 9:35	27.228	58.82
3/8/2018 9:36	26.748	58.82
3/8/2018 9:37	26.46	58.82
3/8/2018 9:38	26.028	58.82
3/8/2018 9:39	25.5	59
3/8/2018 9:40	25.272	59
3/8/2018 9:41	24.984	59.18
3/8/2018 9:42	25.02	59.36
3/8/2018 9:43	25.092	59.54
3/8/2018 9:44	24.9	59.72
3/8/2018 9:45	43.824	59.72
3/8/2018 9:46	73.776	59.9
3/8/2018 9:47	69.972	59.9
3/8/2018 9:48	68.004	60.08
3/8/2018 9:49	66.096	60.08
3/8/2018 9:50	64.308	60.08
3/8/2018 9:51	62.676	59.9
3/8/2018 9:52	60.396	59.9
3/8/2018 9:53	58.5	59.9
3/8/2018 9:54	56.892	59.9
3/8/2018 9:55	55.452	59.9
3/8/2018 9:56	54.072	60.08
3/8/2018 9:57	52.632	60.08
3/8/2018 9:58	51.372	60.08
3/8/2018 9:59	49.872	60.08
3/8/2018 10:00	48.924	60.08
3/8/2018 10:01	47.568	60.26
3/8/2018 10:02	46.56	60.26
3/8/2018 10:03	45.36	60.26
3/8/2018 10:04	44.244	60.26
3/8/2018 10:05	43.548	60.26
3/8/2018 10:06	42.864	60.44
3/8/2018 10:07	42.06	60.44
3/8/2018 10:08	41.268	60.44
3/8/2018 10:09	40.464	60.44
3/8/2018 10:10	39.72	60.44
3/8/2018 10:11	38.664	60.44

Atherton Water Capture
BH06 Vibrating Wire Piezometer Data

Date and Time	Inches of Water	Instrument Temp. (F)
3/8/2018 10:12	38.088	60.44
3/8/2018 10:13	37.572	60.44
3/8/2018 10:14	36.648	60.62
3/8/2018 10:15	36.492	60.62
3/8/2018 10:16	35.508	60.62
3/8/2018 10:17	35.184	60.62
3/8/2018 10:18	34.56	60.62
3/8/2018 10:19	34.08	60.8
3/8/2018 10:20	33.36	60.8
3/8/2018 10:21	33.108	60.8
3/8/2018 10:22	32.22	60.8
3/8/2018 10:23	31.992	60.8
3/8/2018 10:24	31.356	60.8
3/8/2018 10:25	30.888	60.98
3/8/2018 10:26	30.876	60.98
3/8/2018 10:27	30.732	61.16
3/8/2018 10:28	30.42	61.16
3/8/2018 10:29	30.444	61.16
3/8/2018 10:30	30.12	61.16
3/8/2018 10:31	34.56	61.34
3/8/2018 10:32	67.632	61.34
3/8/2018 10:33	65.076	61.52
3/8/2018 10:34	62.82	61.7
3/8/2018 10:35	61.02	61.88
3/8/2018 10:36	59.04	61.88
3/8/2018 10:37	57.192	61.88
3/8/2018 10:38	55.512	62.06
3/8/2018 10:39	53.616	62.06
3/8/2018 10:40	52.176	62.06
3/8/2018 10:41	51	62.06
3/8/2018 10:42	49.464	62.06
3/8/2018 10:43	48.156	62.06
3/8/2018 10:44	46.56	62.24
3/8/2018 10:45	45.372	62.24
3/8/2018 10:46	44.268	62.24
3/8/2018 10:47	43.332	62.42
3/8/2018 10:48	42.48	62.42
3/8/2018 10:49	41.436	62.42
3/8/2018 10:50	40.536	62.42
3/8/2018 10:51	39.684	62.42
3/8/2018 10:52	39.048	62.42
3/8/2018 10:53	37.92	62.6
3/8/2018 10:54	37.08	62.6
3/8/2018 10:55	36.54	62.6
3/8/2018 10:56	35.64	62.6
3/8/2018 10:57	35.028	62.6
3/8/2018 10:58	34.464	62.6
3/8/2018 10:59	34.128	62.78
3/8/2018 11:00	33.648	62.78
3/8/2018 11:01	33.528	62.78
3/8/2018 11:02	33.396	62.78
3/8/2018 11:03	33.288	62.96
3/8/2018 11:04	32.928	62.96
3/8/2018 11:05	33.192	62.96
3/8/2018 11:06	32.484	62.96
3/8/2018 11:07	31.968	62.96
3/8/2018 11:08	32.16	62.96

Date and Time	Inches of Water	Instrument Temp. (F)
3/8/2018 11:09	31.716	62.96
3/8/2018 11:10	31.344	62.96
3/8/2018 11:11	31.188	62.96
3/8/2018 11:12	31.32	62.96
3/8/2018 11:13	30.9	62.96
3/8/2018 11:14	30.708	62.96
3/8/2018 11:15	30.492	63.14
3/8/2018 11:16	30.3	63.14
3/8/2018 11:17	30.276	63.14
3/8/2018 11:18	29.82	63.14
3/8/2018 11:19	29.28	63.14
3/8/2018 11:20	28.98	63.14
3/8/2018 11:21	28.86	63.14
3/8/2018 11:22	28.416	63.14
3/8/2018 11:23	28.296	63.14
3/8/2018 11:24	28.476	63.14
3/8/2018 11:25	28.272	63.14
3/8/2018 11:26	28.26	63.14
3/8/2018 11:27	28.188	63.14
3/8/2018 11:28	27.876	63.14
3/8/2018 11:29	54.216	63.32
3/8/2018 11:30	87.804	63.68
3/8/2018 11:31	94.656	64.04
3/8/2018 11:32	90.36	64.22
3/8/2018 11:33	85.62	64.4
3/8/2018 11:34	81.684	64.58
3/8/2018 11:35	79.284	64.58
3/8/2018 11:36	77.016	64.58
3/8/2018 11:37	74.232	64.76
3/8/2018 11:38	71.796	64.76
3/8/2018 11:39	69.864	64.76
3/8/2018 11:40	67.896	64.76
3/8/2018 11:41	66.216	64.76
3/8/2018 11:42	64.392	64.76
3/8/2018 11:43	63.216	64.76
3/8/2018 11:44	61.92	64.76
3/8/2018 11:45	60.456	64.76
3/8/2018 11:46	59.22	64.76
3/8/2018 11:47	57.996	64.76
3/8/2018 11:48	56.412	64.76
3/8/2018 11:49	55.092	64.76
3/8/2018 11:50	53.544	64.76
3/8/2018 11:51	52.392	64.76
3/8/2018 11:52	51.132	64.76
3/8/2018 11:53	49.98	64.76
3/8/2018 11:54	48.672	64.76
3/8/2018 11:55	47.724	64.76
3/8/2018 11:56	46.68	64.76
3/8/2018 11:57	45.816	64.76
3/8/2018 11:58	44.88	64.76
3/8/2018 11:59	43.968	64.76
3/8/2018 12:00	43.164	64.76
3/8/2018 12:01	42.348	64.76
3/8/2018 12:02	41.46	64.76
3/8/2018 12:03	40.596	64.76
3/8/2018 12:04	39.804	64.76
3/8/2018 12:05	39.324	64.76

Atherton Water Capture
BH06 Vibrating Wire Piezometer Data

Date and Time	Inches of Water	Instrument Temp. (F)
3/8/2018 12:06	39.108	64.76
3/8/2018 12:07	38.82	64.76
3/8/2018 12:08	38.532	64.76
3/8/2018 12:09	38.196	64.94
3/8/2018 12:10	37.644	64.94
3/8/2018 12:11	37.572	64.94
3/8/2018 12:12	37.164	64.94
3/8/2018 12:13	37.056	64.94
3/8/2018 12:14	36.732	64.94
3/8/2018 12:15	36.576	64.94
3/8/2018 12:16	36.48	64.94
3/8/2018 12:17	35.94	64.94
3/8/2018 12:18	35.568	64.94
3/8/2018 12:19	35.808	64.94
3/8/2018 12:20	34.968	64.94
3/8/2018 12:21	34.692	64.94
3/8/2018 12:22	34.488	64.94
3/8/2018 12:23	33.876	64.94
3/8/2018 12:24	33.552	64.94
3/8/2018 12:25	33.216	64.94
3/8/2018 12:26	32.724	64.94
3/8/2018 12:27	32.52	64.94
3/8/2018 12:28	32.04	64.94
3/8/2018 12:29	31.524	64.94
3/8/2018 12:30	31.344	64.94
3/8/2018 12:31	31.14	64.94
3/8/2018 12:32	31.02	64.94
3/8/2018 12:33	30.864	64.94
3/8/2018 12:34	30.744	64.94
3/8/2018 12:35	30.72	64.94
3/8/2018 12:36	30.804	64.94
3/8/2018 12:37	30.684	64.94
3/8/2018 12:38	30.672	64.94
3/8/2018 12:39	30.396	64.94
3/8/2018 12:40	30.396	64.94
3/8/2018 12:41	30.432	64.94
3/8/2018 12:42	30.444	64.94
3/8/2018 12:43	30.612	64.94
3/8/2018 12:44	30.492	64.94
3/8/2018 12:45	30.564	64.94
3/8/2018 12:46	30.552	64.94
3/8/2018 12:47	30.156	64.94
3/8/2018 12:48	30.492	64.94
3/8/2018 12:49	30.348	64.94
3/8/2018 12:50	30.372	64.94
3/8/2018 12:51	35.688	65.12
3/8/2018 12:52	59.052	65.66
3/8/2018 12:53	64.98	65.84
3/8/2018 12:54	62.196	66.02
3/8/2018 12:55	59.736	66.2
3/8/2018 12:56	57.42	66.2
3/8/2018 12:57	55.044	66.2
3/8/2018 12:58	52.968	66.2
3/8/2018 12:59	51.204	66.2
3/8/2018 13:00	49.332	66.2
3/8/2018 13:01	47.532	66.2
3/8/2018 13:02	45.792	66.2

Date and Time	Inches of Water	Instrument Temp. (F)
3/8/2018 13:03	44.436	66.2
3/8/2018 13:04	43.32	66.2
3/8/2018 13:05	42.576	66.38
3/8/2018 13:06	41.52	66.56
3/8/2018 13:07	41.34	66.56
3/8/2018 13:08	41.064	66.74
3/8/2018 13:09	40.392	66.74
3/8/2018 13:10	40.26	66.74
3/8/2018 13:11	39.348	66.74
3/8/2018 13:12	39.324	66.74
3/8/2018 13:13	38.604	66.74
3/8/2018 13:14	38.604	66.74
3/8/2018 13:15	37.92	66.74
3/8/2018 13:16	37.704	66.74
3/8/2018 13:17	37.356	66.74
3/8/2018 13:18	36.984	66.56
3/8/2018 13:19	36.768	66.56
3/8/2018 13:20	36.48	66.56
3/8/2018 13:21	36.192	66.56
3/8/2018 13:22	35.58	66.56
3/8/2018 13:23	35.292	66.56
3/8/2018 13:24	34.884	66.56
3/8/2018 13:25	34.32	66.56
3/8/2018 13:26	33.852	66.56
3/8/2018 13:27	33.528	66.56
3/8/2018 13:28	33.204	66.56
3/8/2018 13:29	32.748	66.56
3/8/2018 13:30	32.268	66.56
3/8/2018 13:31	32.352	66.56
3/8/2018 13:32	32.124	66.38
3/8/2018 13:33	32.268	66.38
3/8/2018 13:34	32.352	66.38
3/8/2018 13:35	32.076	66.38
3/8/2018 13:36	31.98	66.38
3/8/2018 13:37	32.076	66.38
3/8/2018 13:38	32.028	66.38
3/8/2018 13:39	32.052	66.38
3/8/2018 13:40	31.74	66.38
3/8/2018 13:41	31.752	66.38
3/8/2018 13:42	31.62	66.38
3/8/2018 13:43	31.872	66.38
3/8/2018 13:44	31.656	66.38
3/8/2018 13:45	31.704	66.38
3/8/2018 13:46	31.728	66.2
3/8/2018 13:47	31.524	66.2
3/8/2018 13:48	31.464	66.2
3/8/2018 13:49	31.44	66.2
3/8/2018 13:50	31.212	66.2
3/8/2018 13:51	30.6	66.2
3/8/2018 13:52	30.696	66.2
3/8/2018 13:53	30.72	66.2
3/8/2018 13:54	30.576	66.2
3/8/2018 13:55	30.468	66.2
3/8/2018 13:56	30.564	66.2
3/8/2018 13:57	30.336	66.2
3/8/2018 13:58	30.516	66.2
3/8/2018 13:59	30.444	66.2

Atherton Water Capture

BH06 Vibrating Wire Piezometer Data

Date and Time	Inches of Water	Instrument Temp. (F)
3/8/2018 14:00	30.348	66.2
3/8/2018 14:01	30.324	66.2
3/8/2018 14:02	30.168	66.2
3/8/2018 14:03	30.264	66.2
3/8/2018 14:04	30.192	66.2
3/8/2018 14:05	30.06	66.2
3/8/2018 14:06	29.94	66.2
3/8/2018 14:07	29.904	66.02
3/8/2018 14:08	29.868	66.02
3/8/2018 14:09	29.784	66.02
3/8/2018 14:10	29.628	66.02
3/8/2018 14:11	29.352	66.02
3/8/2018 14:12	29.508	66.02
3/8/2018 14:13	29.508	66.02
3/8/2018 14:14	29.412	66.02
3/8/2018 14:15	29.124	66.02
3/8/2018 14:16	29.328	66.02
3/8/2018 14:17	29.28	66.02
3/8/2018 14:18	29.232	66.02
3/8/2018 14:19	29.172	66.02
3/8/2018 14:20	29.124	66.02
3/8/2018 14:21	28.872	66.02
3/8/2018 14:22	28.872	66.02
3/8/2018 14:23	28.776	66.02
3/8/2018 14:24	28.668	66.02
3/8/2018 14:25	28.536	66.02
3/8/2018 14:26	28.62	66.02
3/8/2018 14:27	28.524	66.02
3/8/2018 14:28	28.464	66.02
3/8/2018 14:29	28.332	65.84
3/8/2018 14:30	28.536	65.84
3/8/2018 14:31	28.572	65.84
3/8/2018 14:32	28.428	65.84
3/8/2018 14:33	28.452	65.84
3/8/2018 14:34	28.332	65.84
3/8/2018 14:35	28.272	65.84

GEOKON 48 Spencer St. Lebanon, NH 03766 USA

Vibrating Wire Pressure Transducer Calibration Report

Model Number:	4500ALV-70 kPa	Date of Calibration:	September 06, 2013
Serial Number:	1322864	Temperature:	22.8 °C
Calibration Instruction:	VW Pressure Transducers	Barometric Pressure:	1001.6 mbar
Cable Length:	50 feet	Technician:	KRogers

Applied Pressure (kPa)	Gage Reading 1st Cycle	Gage Reading 2nd Cycle	Average Gage Reading	Calculated Pressure (Linear)	Error Linear (%FS)	Calculated Pressure (Polynomial)	Error Polynomial (%FS)
0.0	9836	9837	9837	0.054	0.08	0.005	0.01
14.0	8935	8935	8935	13.98	-0.03	13.99	-0.02
28.0	8030	8030	8030	27.96	-0.05	27.99	0.00
42.0	7124	7124	7124	41.95	-0.06	41.99	-0.01
56.0	6214	6214	6214	56.01	0.02	56.02	0.03
70.0	5306	5306	5306	70.04	0.05	69.99	-0.02

(kPa) Linear Gage Factor (G): -0.01545 (kPa/ digit) **Regression Zero: 9840**

Polynomial Gage factors: **A: -1.724E-08** **B: -0.01519** **C:**

Thermal Factor (K): 0.01720 (kPa/ °C)

Calculate C by setting P=0 and R_1 = initial field zero reading into the polynomial equation

(psi) Linear Gage Factor (G): -0.002240 (psi/ digit)

Polynomial Gage Factors: **A: -2.5E-09** **B: -0.002203** **C:**

Thermal Factor (K): 0.002495 (psi/ °C)

Calculate C by setting P=0 and R_1 = initial field zero reading into the polynomial equation

Calculated Pressures:

Linear, $P = G(R_1 - R_0) + K(T_1 - T_0) - (S_1 - S_0)$*

Polynomial, $P = AR_1^2 + BR_1 + C + K(T_1 - T_0) - (S_1 - S_0)$*

*Barometric pressures expressed in kPa or psi. Barometric compensation is not required with vented transducers.

The above instrument was found to be in tolerance in all operating ranges.
The above named instrument has been calibrated by comparison with standards traceable to the NIST, in compliance with ANSI Z540-1.

GEOKON 48 Spencer St. Lebanon, N.H. 03766 USA

Vibrating Wire Pressure Transducer Calibration Report

Type: S

Serial Number: 09-3325

Pressure Range: 350 kPa

Date of Calibration: February 10, 2010

Temperature: 23.5 °C

†Barometric Pressure: 977.7 mbar

Calibration Instruction: VW Pressure Transducers

Technician: KLBellavance

HZ/1000

Applied Pressure (kPa)	Gage Reading 1st Cycle	Gage Reading 2nd Cycle	Average Gage Reading	Calculated Pressure (Linear)	Error Linear (%FS)	Calculated Pressure (Polynomial)	Error Polynomial (%FS)
0.0	8795	8795	8795	0.433	0.12	-0.127	-0.04
70.0	8162	8163	8163	70.13	0.04	70.29	0.08
140.0	7532	7533	7533	139.5	-0.13	140.0	0.01
210.0	6898	6899	6899	209.4	-0.17	209.9	-0.03
280.0	6259	6259	6259	279.9	-0.04	280.0	0.00
350.0	5617	5617	5617	350.6	0.17	350.1	0.01

(kPa) Linear Gage Factor (G): 0.1102 (kPa/ digit) **Regression Zero: 8799**

Polynomial Gage Factors: A: -4.167E-07 B: -0.1042 C: 948.39

Thermal Factor (K): 0.0283 (kPa/ °C)

(psi) Linear Gage Factor (G): 0.01598 (psi/ digit)

Polynomial Gage Factors: A: -6.04312E-08 B: -0.01511 C: 137.55

Thermal Factor (K): 0.00410 (psi/ °C)

Calculated Pressures: **Linear, $P = G(R_0 - R_1)+K(T_1 - T_0)-(S_1 - S_0)$****

Polynomial, $P = AR_1^2 + BR_1 + C + K(T_1 - T_0)-(S_1 - S_0)$**

†Barometric pressures are absolute. Barometric compensation is not required with vented and differential pressure transducers.

Factory Zero Reading:

GK-401 Pos. B or F(R_0): 8784 Temp(T_0): 23.1 °C †Baro(S_0): 977.3 mbar Date: February 10, 2010

*Initial zero readings must be established in the field following the procedures described in the Instruction Manual. If the Polynomial equation is used the field value of C must be calculated by plugging the initial zero reading into the polynomial equation with the value of P set to zero.

The above instrument was found to be in tolerance in all operating ranges.
The above named instrument has been calibrated by comparison with standards traceable to the NIST, in compliance with ANSI Z540-1.

APPENDIX E

CPT LIQUEFACTION ANALYSIS

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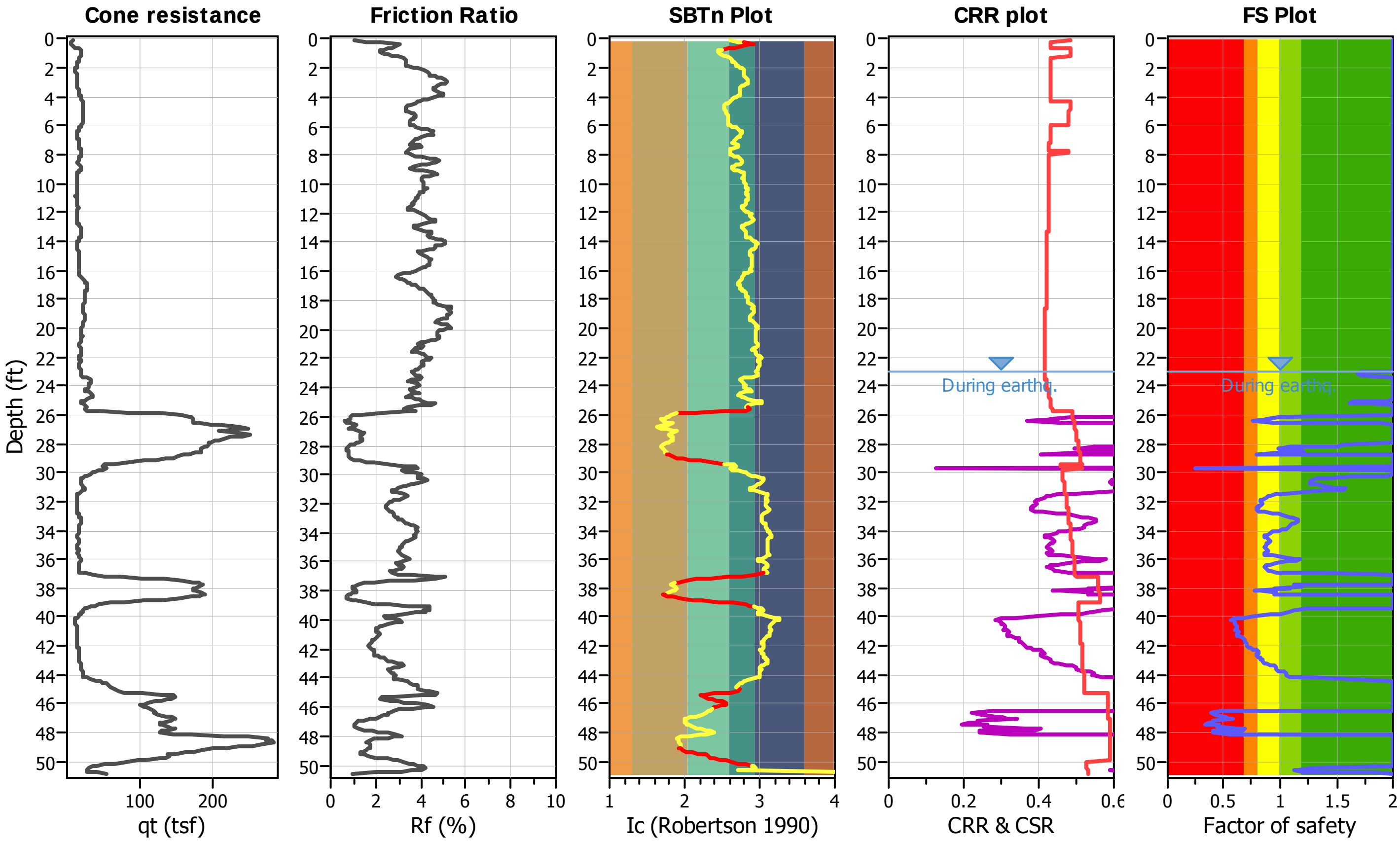
LIQUEFACTION ANALYSIS REPORT

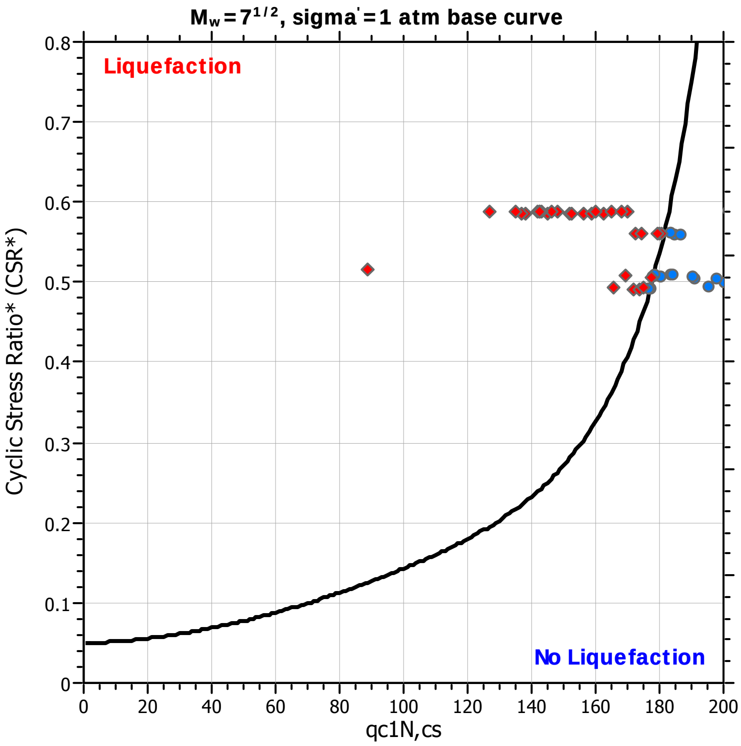
Project title : Atherton Water Capture Project **Location : 150 Watkins Ave, Atherton, CA**

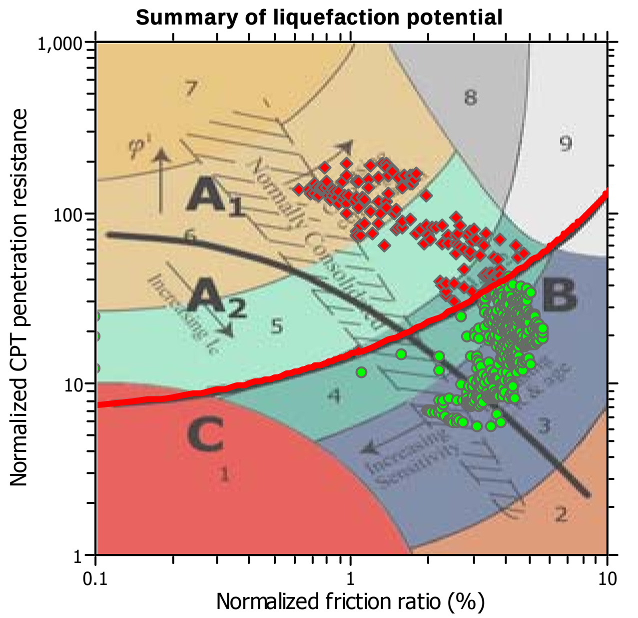
CPT file : 1-CPT01

Input parameters and analysis data

Analysis method:	I&B (2008)	G.W.T. (in-situ):	23.00 ft	Use fill:	No	Clay like behavior applied:	Sand & Clay
Fines correction method:	I&B (2008)	G.W.T. (earthq.):	23.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	8.00	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method
Peak ground acceleration:	0.65	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

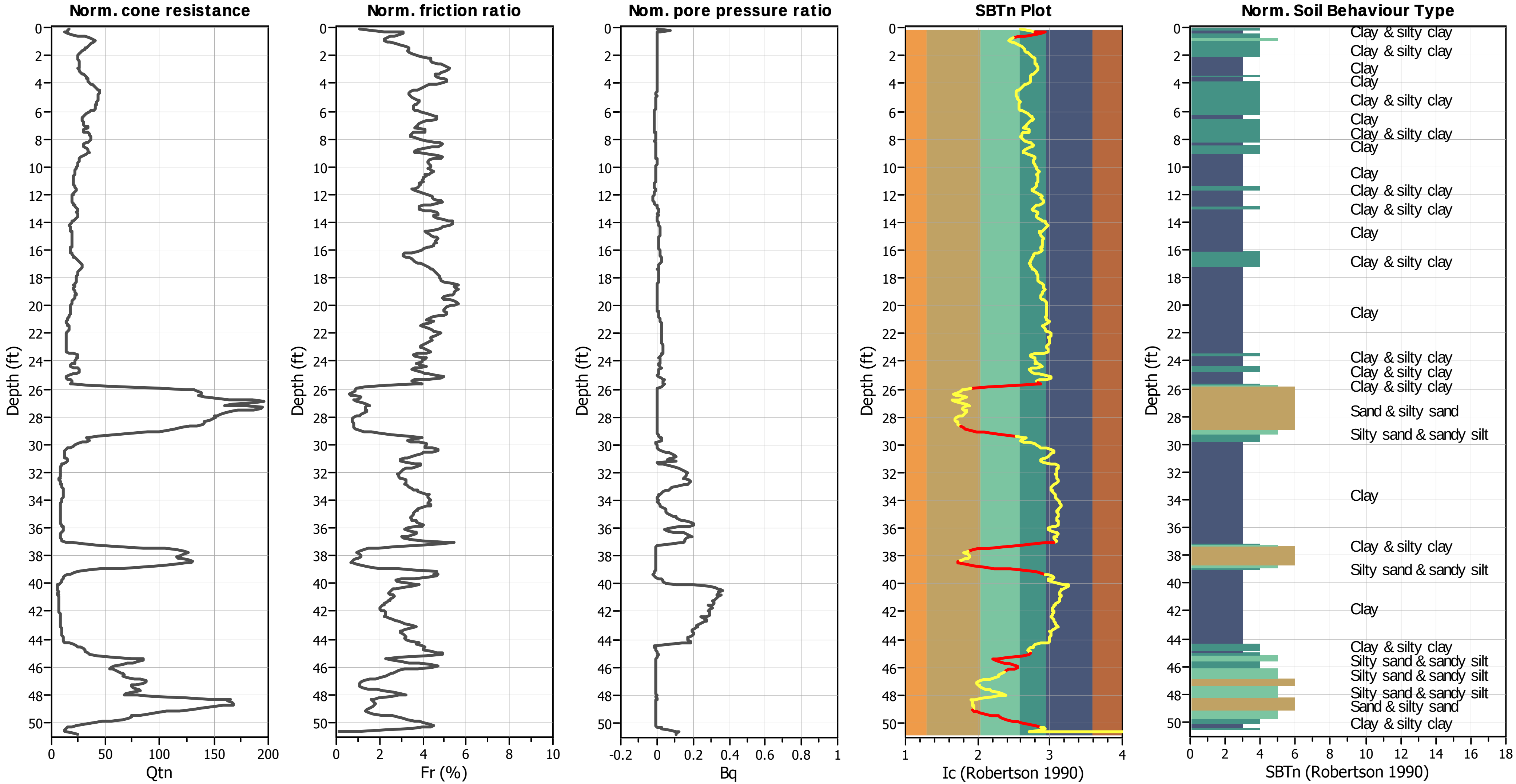






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots (normalized)



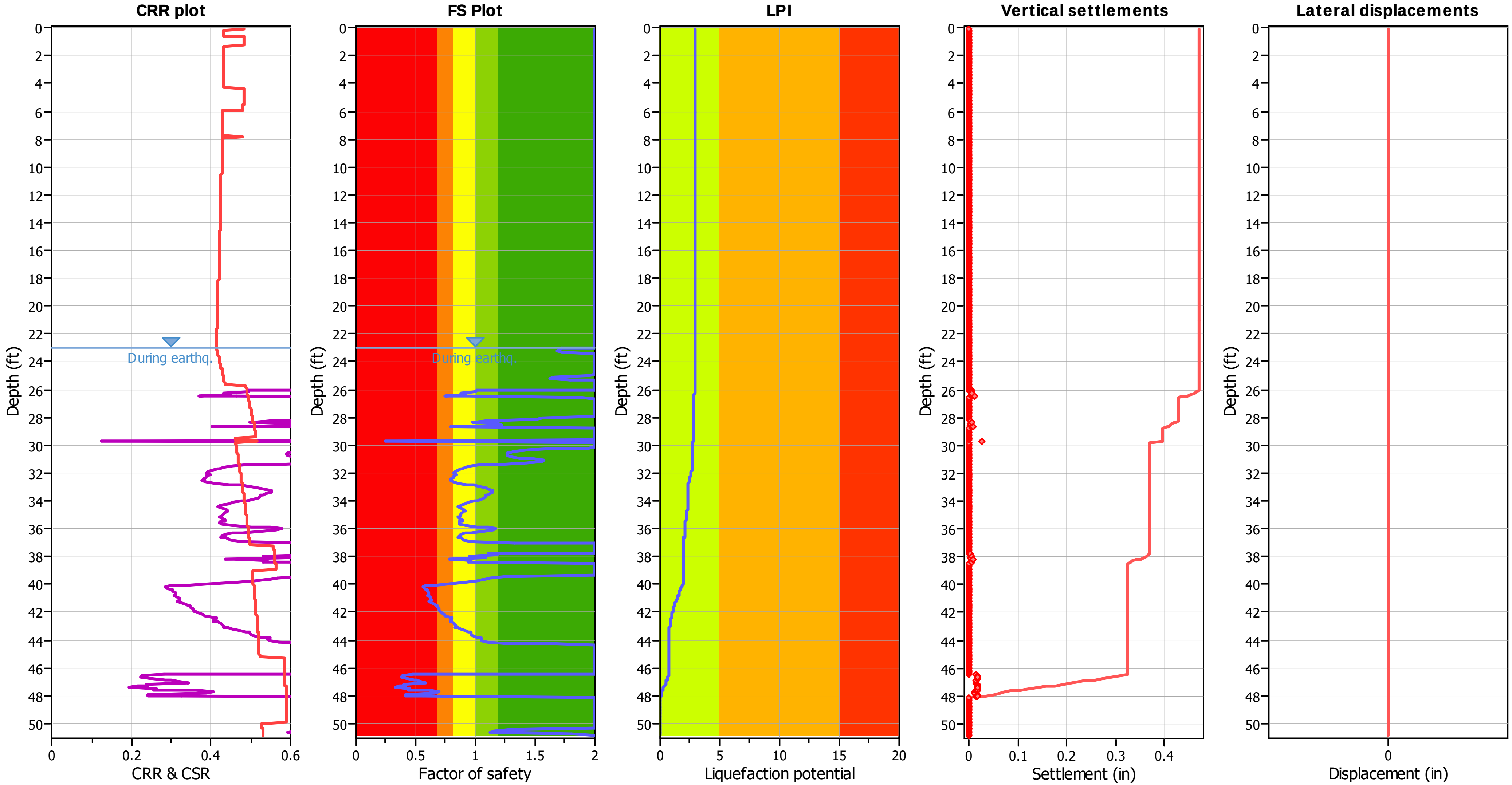
Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	23.00 ft	Fill weight:	N/A
Fines correction method:	I&B (2008)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.65	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	23.00 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to
9. Very stiff fine grained

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	23.00 ft	Fill weight:	N/A
Fines correction method:	I&B (2008)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.65	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	23.00 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

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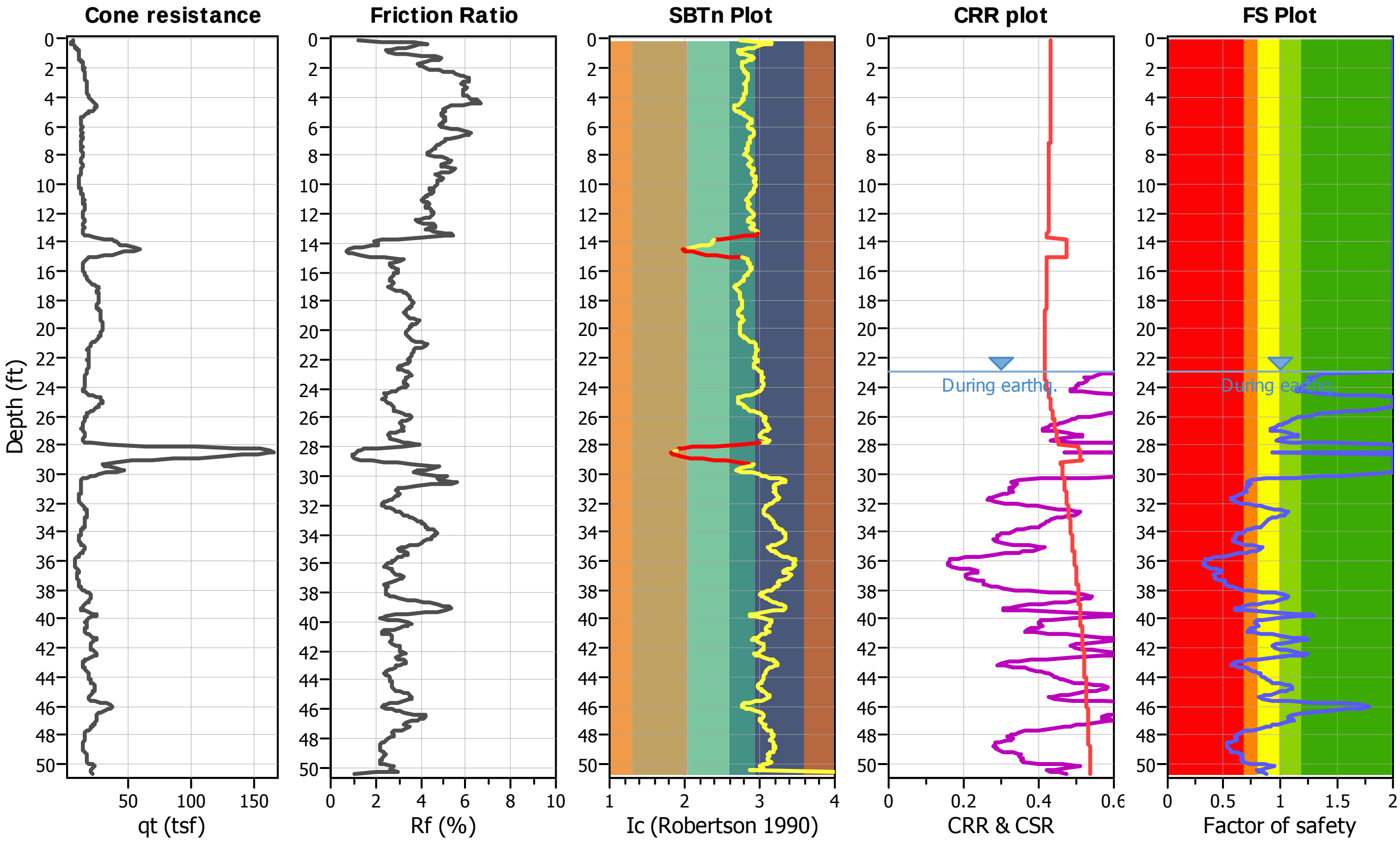
LIQUEFACTION ANALYSIS REPORT

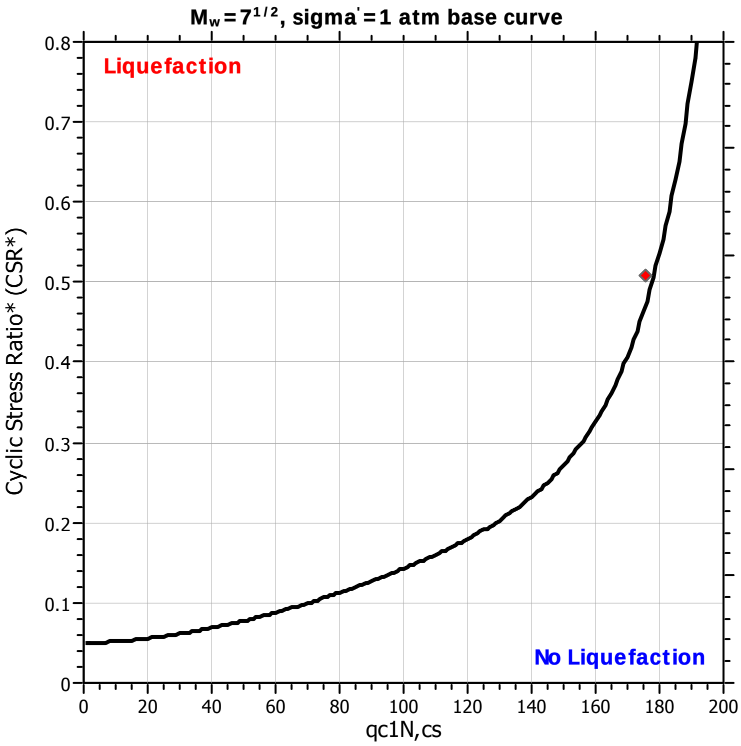
Project title : Atherton Water Capture Project **Location : 150 Watkins Ave, Atherton, CA**

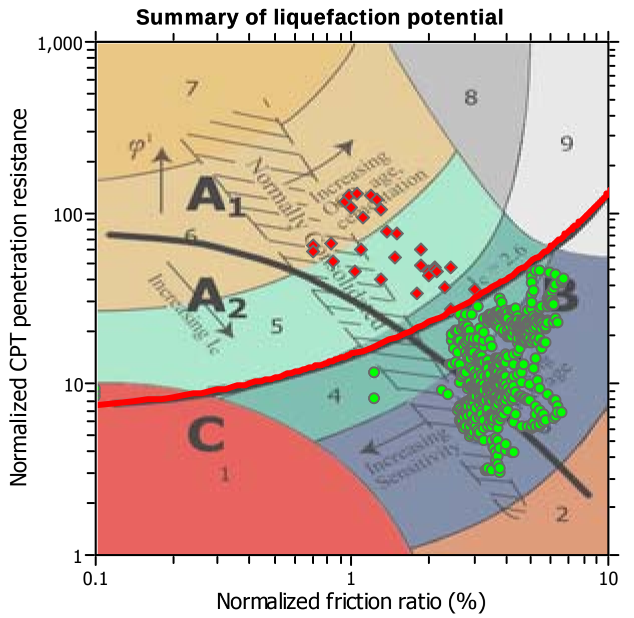
CPT file : 1-CPT02

Input parameters and analysis data

Analysis method:	I&B (2008)	G.W.T. (in-situ):	23.00 ft	Use fill:	No	Clay like behavior applied:	Sand & Clay
Fines correction method:	I&B (2008)	G.W.T. (earthq.):	23.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	8.00	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method
Peak ground acceleration:	0.65	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

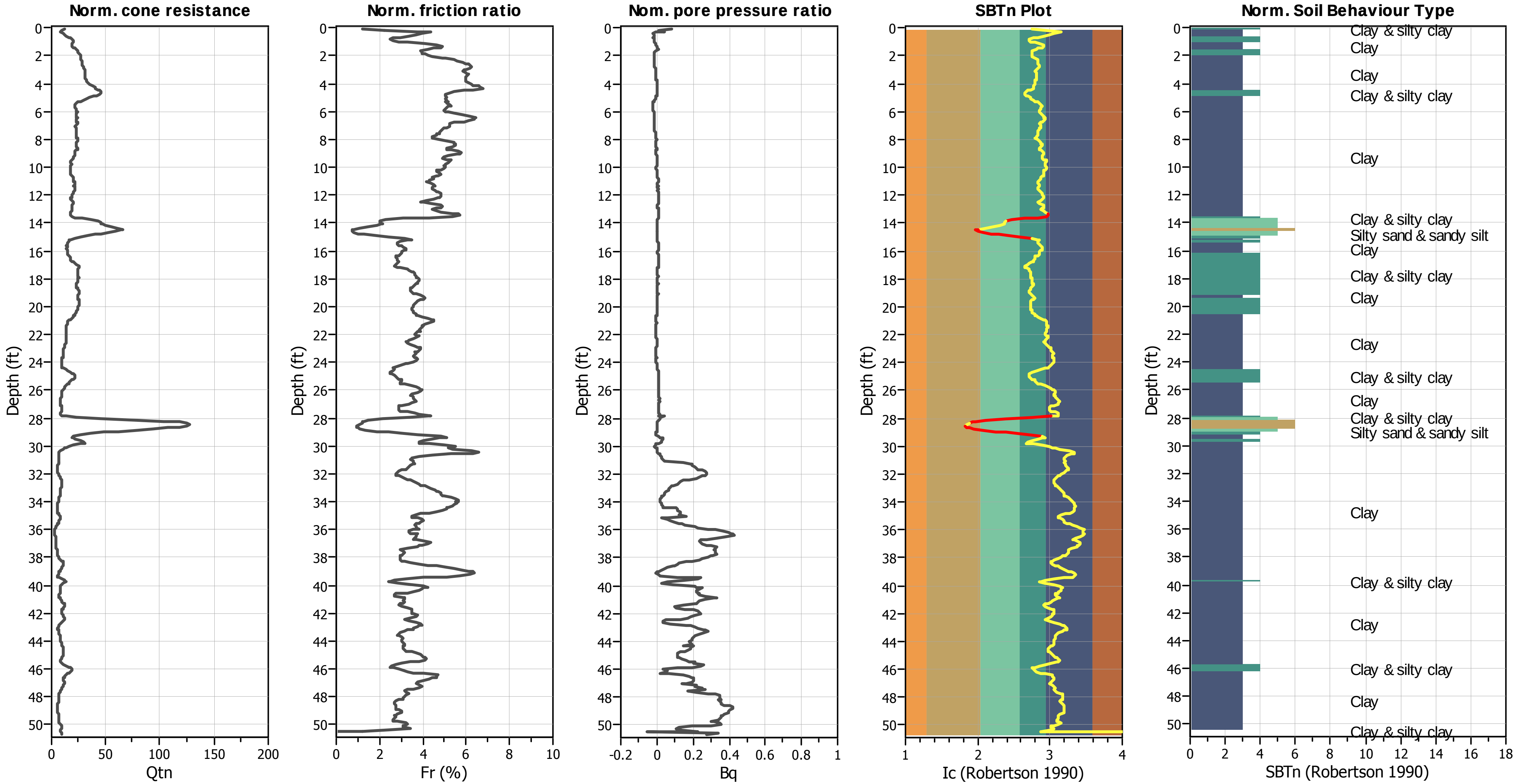






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots (normalized)



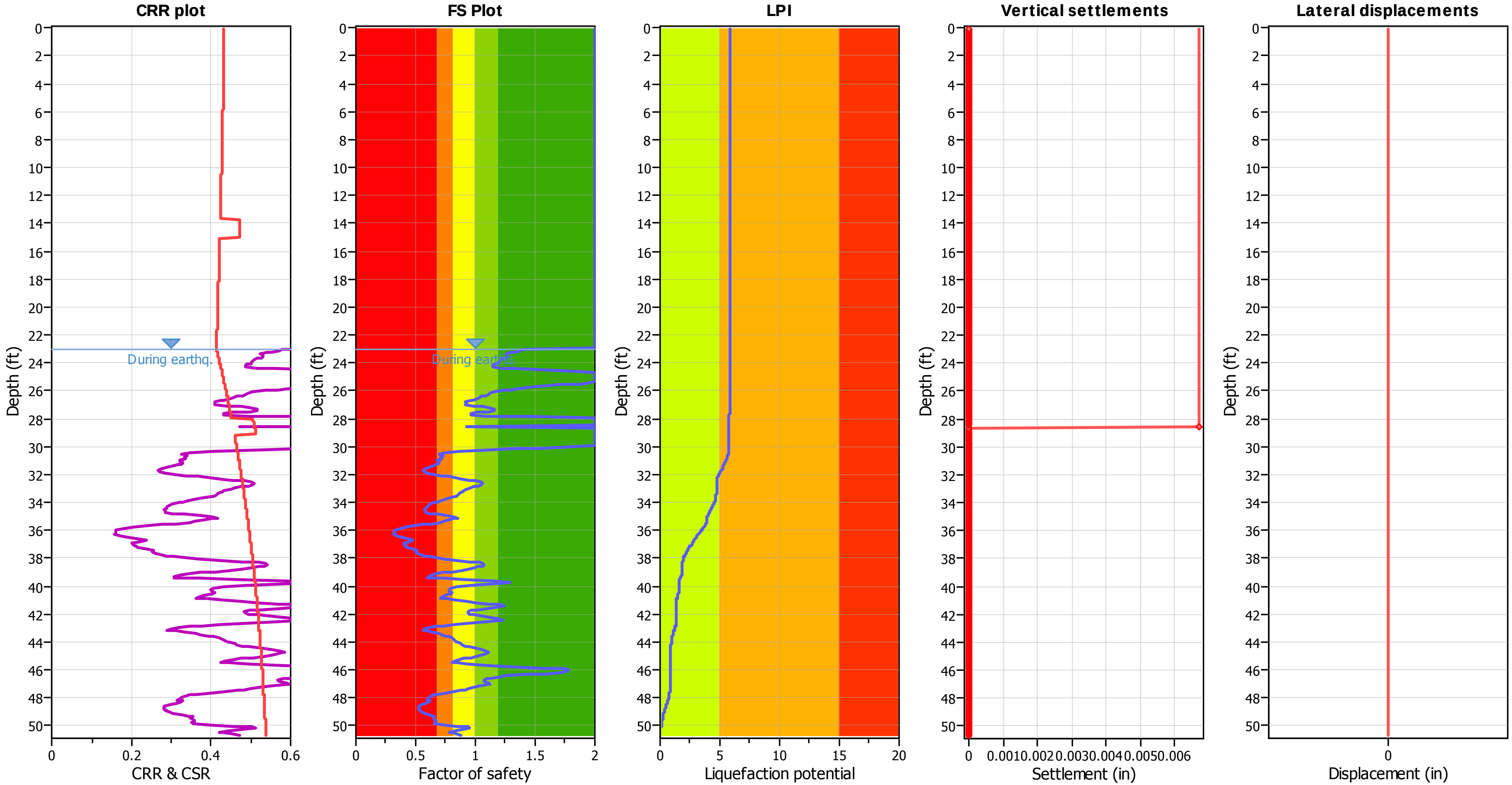
Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	23.00 ft	Fill weight:	N/A
Fines correction method:	I&B (2008)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.65	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	23.00 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

- 1. Sensitive fine grained
- 2. Organic material
- 3. Clay to silty clay
- 4. Clayey silt to silty
- 5. Silty sand to sandy silt
- 6. Clean sand to silty sand
- 7. Gravely sand to sand
- 8. Very stiff sand to
- 9. Very stiff fine grained

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	23.00 ft	Fill weight:	N/A
Fines correction method:	I&B (2008)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.65	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	23.00 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme
- Very high risk
- High risk
- Low risk

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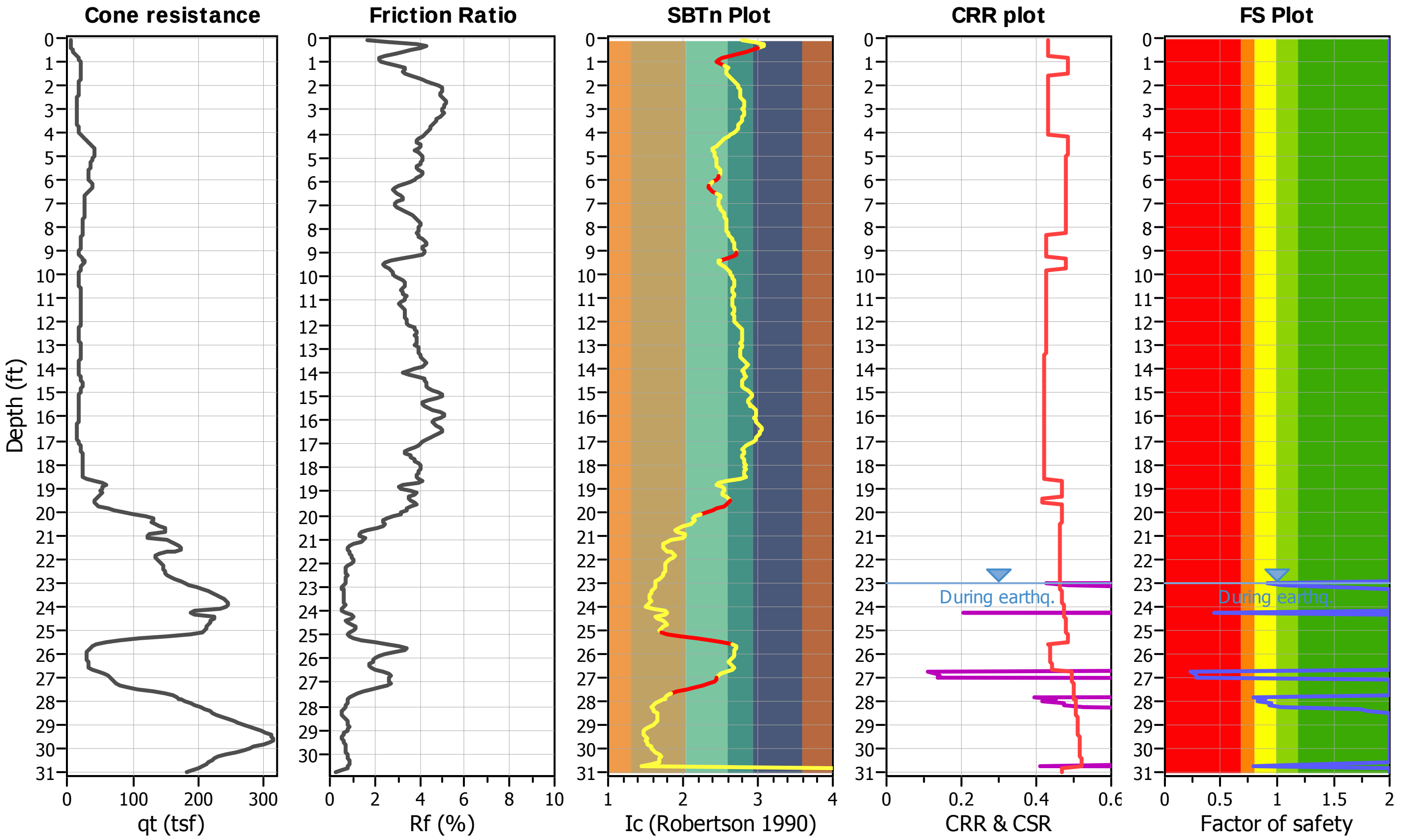
LIQUEFACTION ANALYSIS REPORT

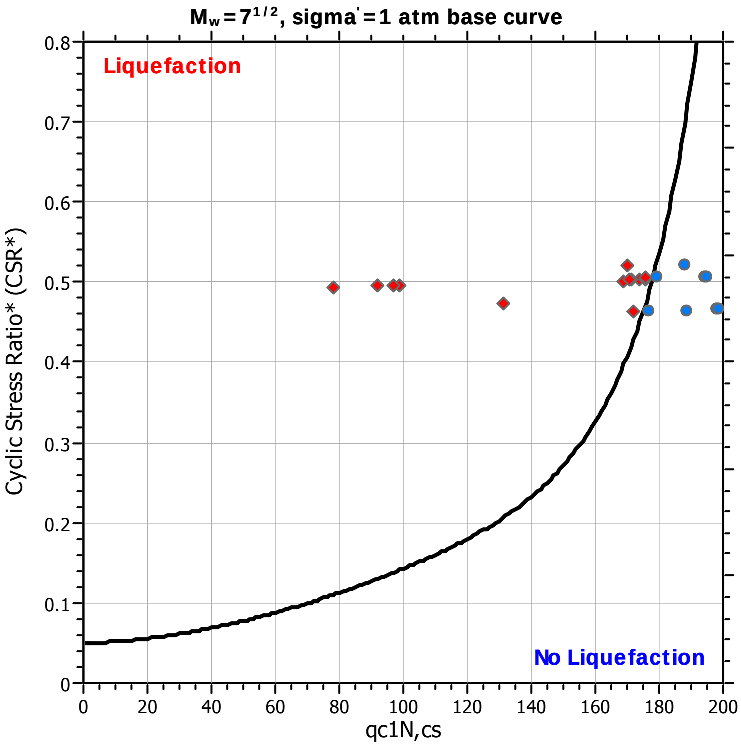
Project title : Atherton Water Capture Project **Location : 150 Watkins Ave, Atherton, CA**

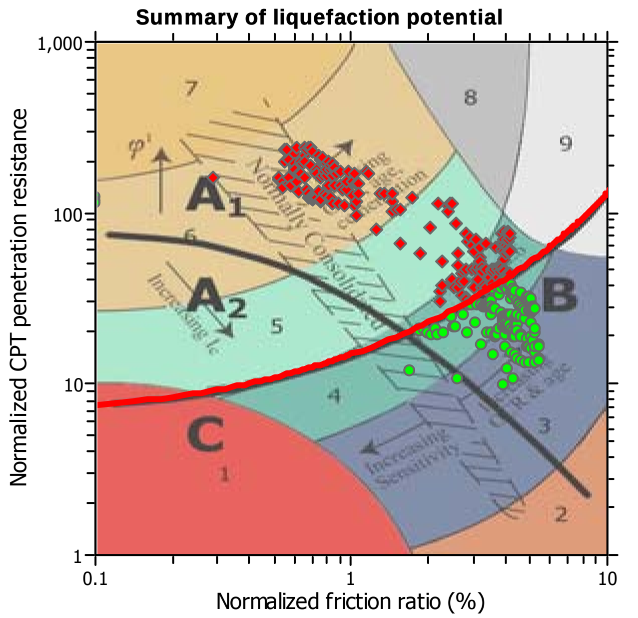
CPT file : 1-CPT03

Input parameters and analysis data

Analysis method:	I&B (2008)	G.W.T. (in-situ):	23.00 ft	Use fill:	No	Clay like behavior applied:	Sand & Clay
Fines correction method:	I&B (2008)	G.W.T. (earthq.):	23.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	8.00	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method
Peak ground acceleration:	0.65	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

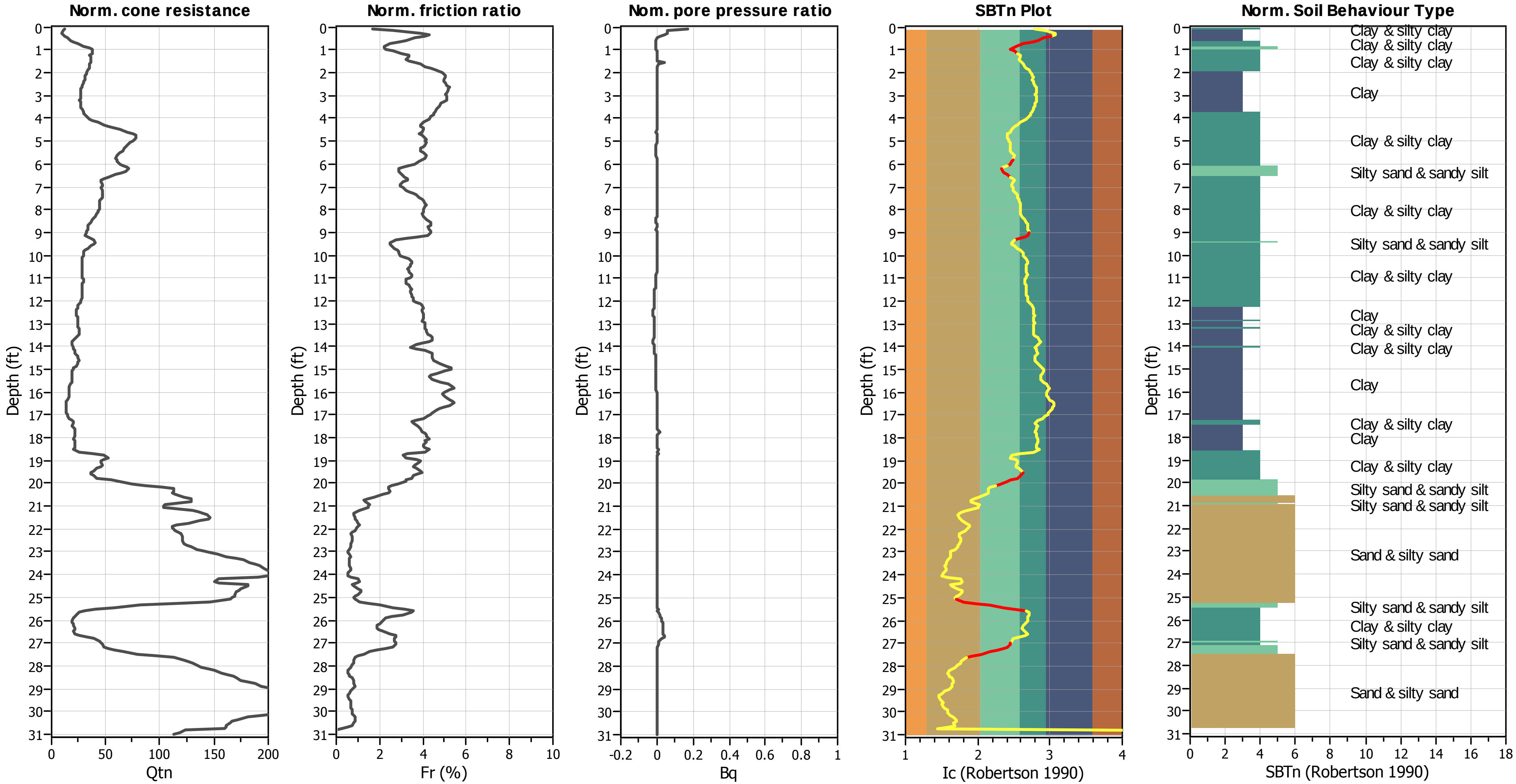






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots (normalized)



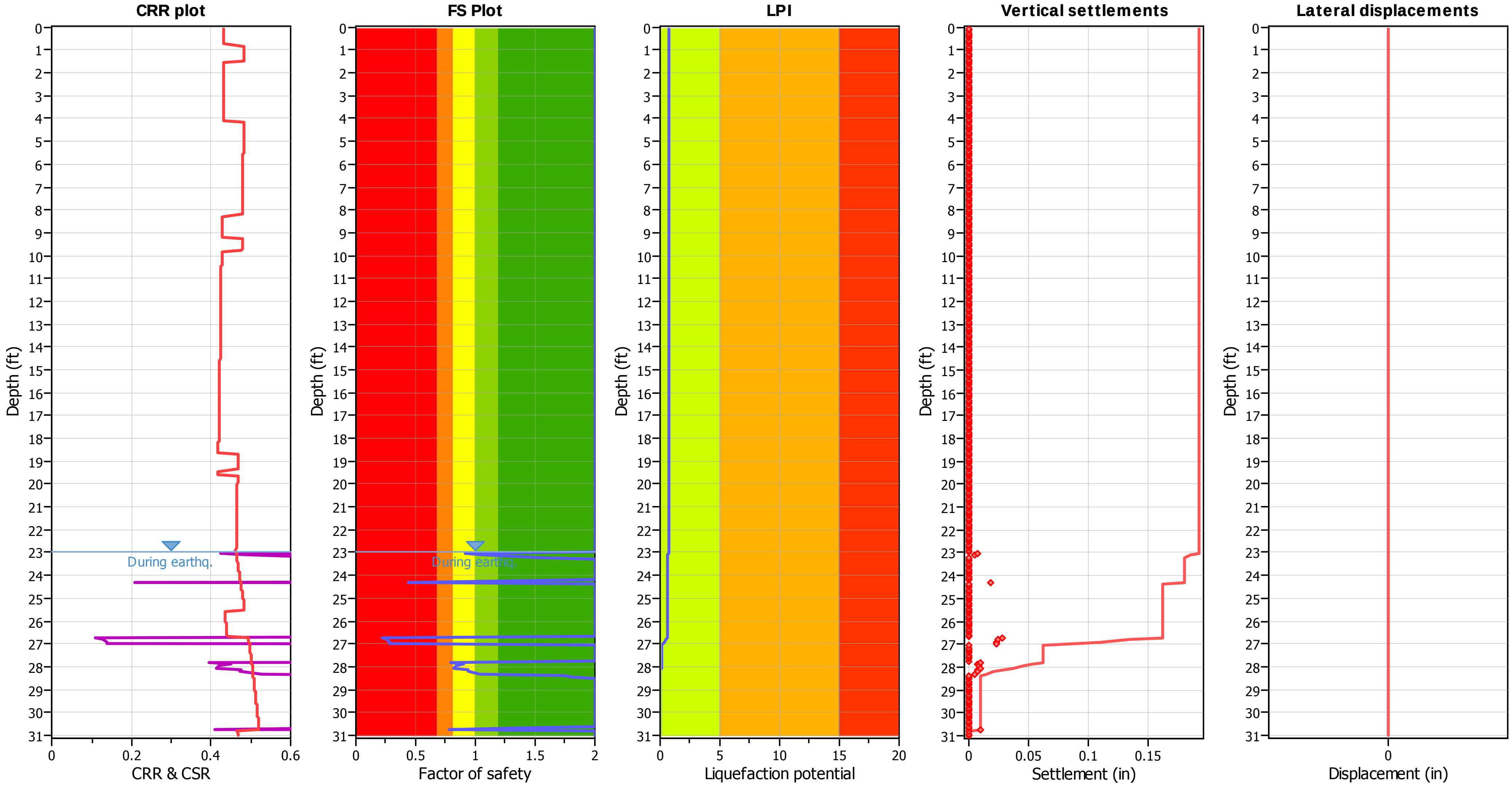
Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	23.00 ft	Fill weight:	N/A
Fines correction method:	I&B (2008)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.65	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	23.00 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to
9. Very stiff fine grained

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	23.00 ft	Fill weight:	N/A
Fines correction method:	I&B (2008)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.65	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	23.00 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

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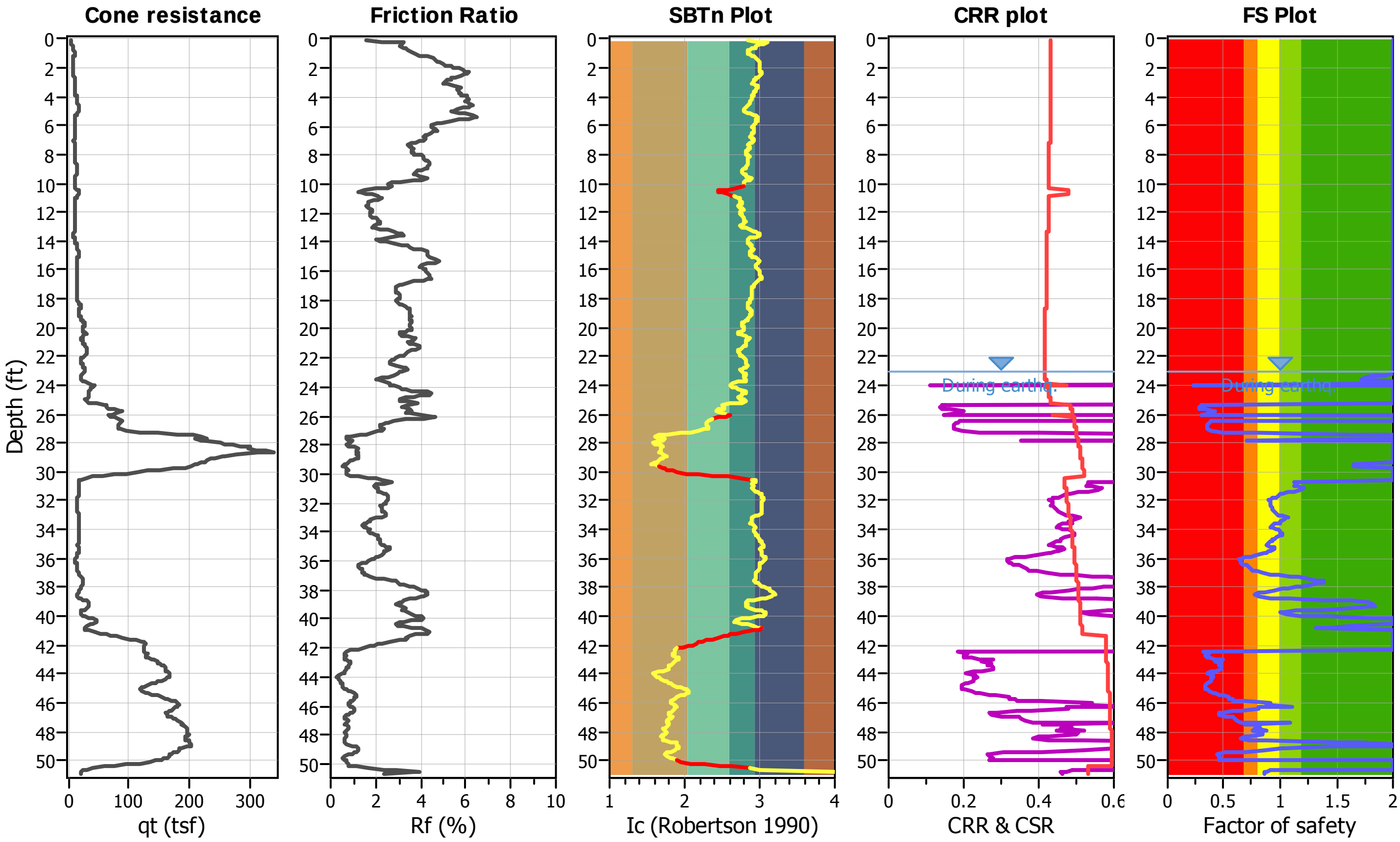
LIQUEFACTION ANALYSIS REPORT

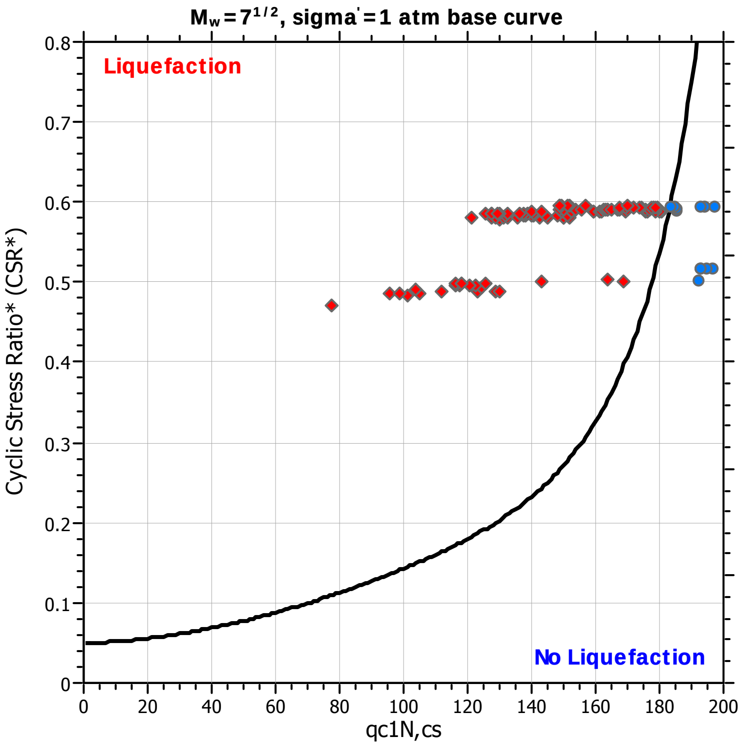
Project title : Atherton Water Capture Project **Location : 150 Watkins Ave, Atherton, CA**

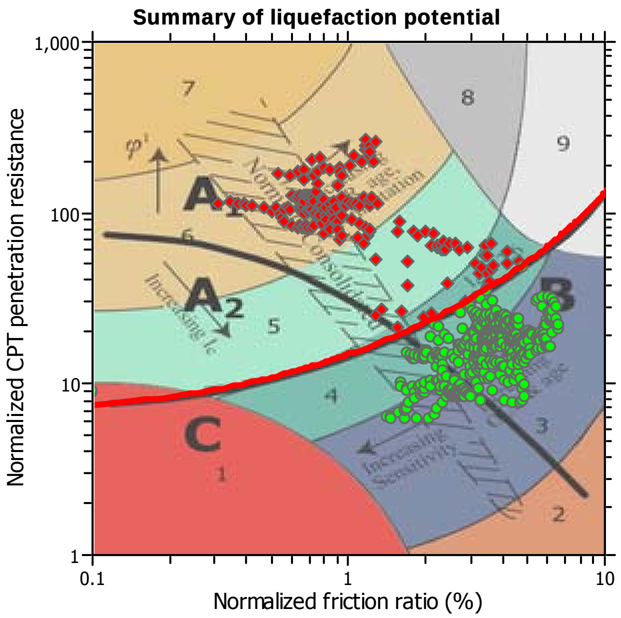
CPT file : 1-CPT04

Input parameters and analysis data

Analysis method:	I&B (2008)	G.W.T. (in-situ):	23.00 ft	Use fill:	No	Clay like behavior applied:	Sand & Clay
Fines correction method:	I&B (2008)	G.W.T. (earthq.):	23.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	8.00	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method
Peak ground acceleration:	0.65	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

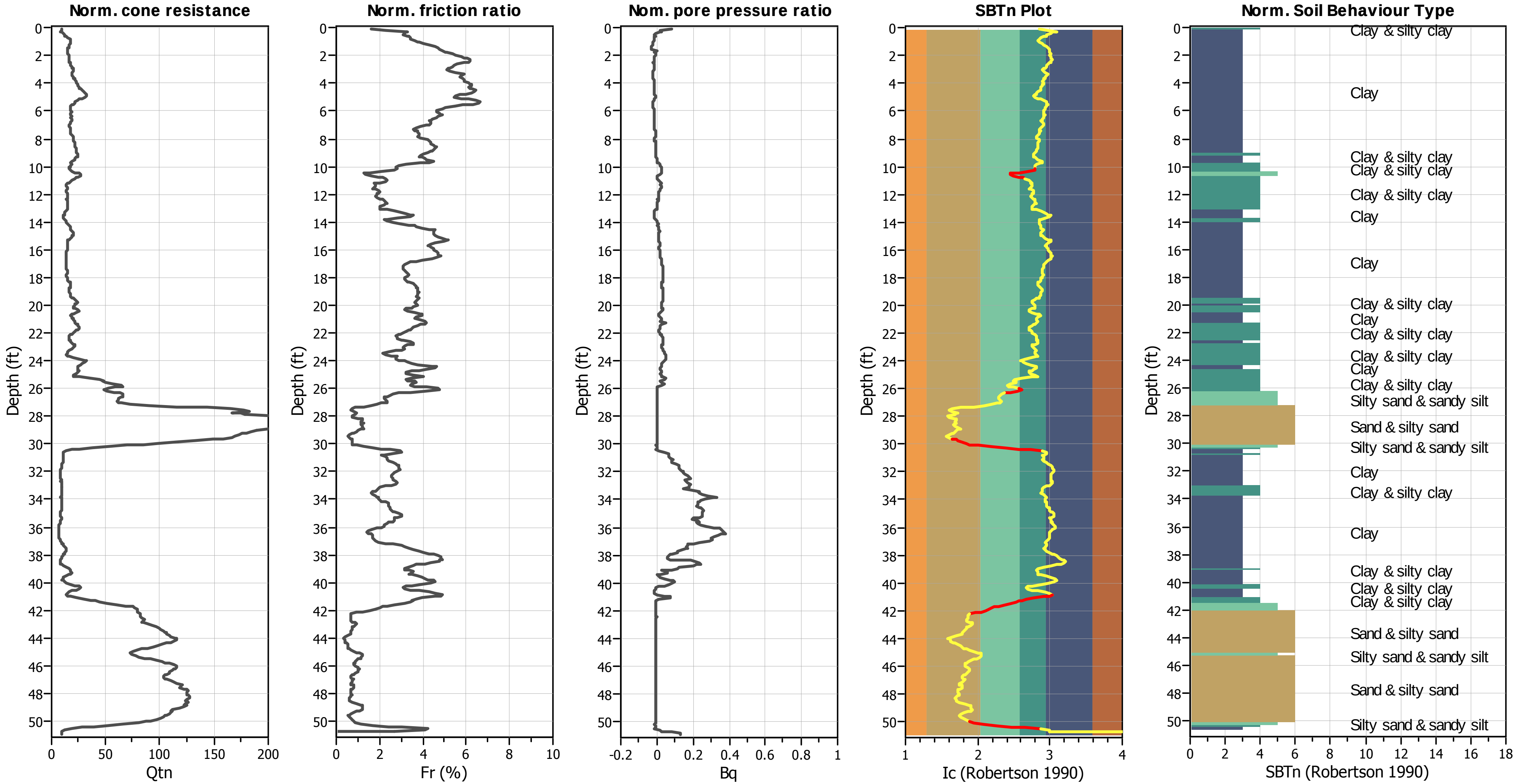






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots (normalized)



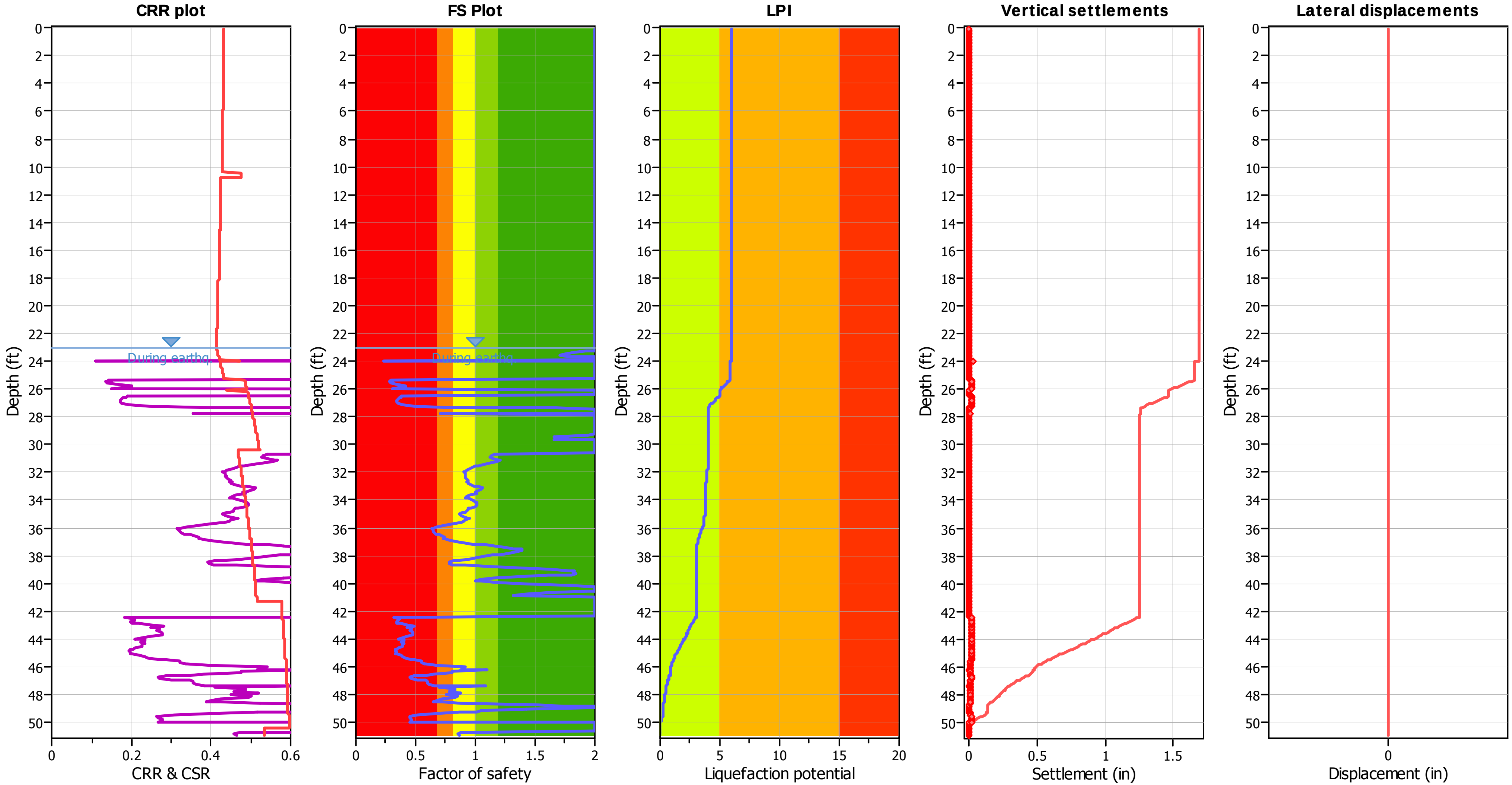
Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	23.00 ft	Fill weight:	N/A
Fines correction method:	I&B (2008)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.65	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	23.00 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

- 1. Sensitive fine grained
- 2. Organic material
- 3. Clay to silty clay
- 4. Clayey silt to silty
- 5. Silty sand to sandy silt
- 6. Clean sand to silty sand
- 7. Gravely sand to sand
- 8. Very stiff sand to
- 9. Very stiff fine grained

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	23.00 ft	Fill weight:	N/A
Fines correction method:	I&B (2008)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.65	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	23.00 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

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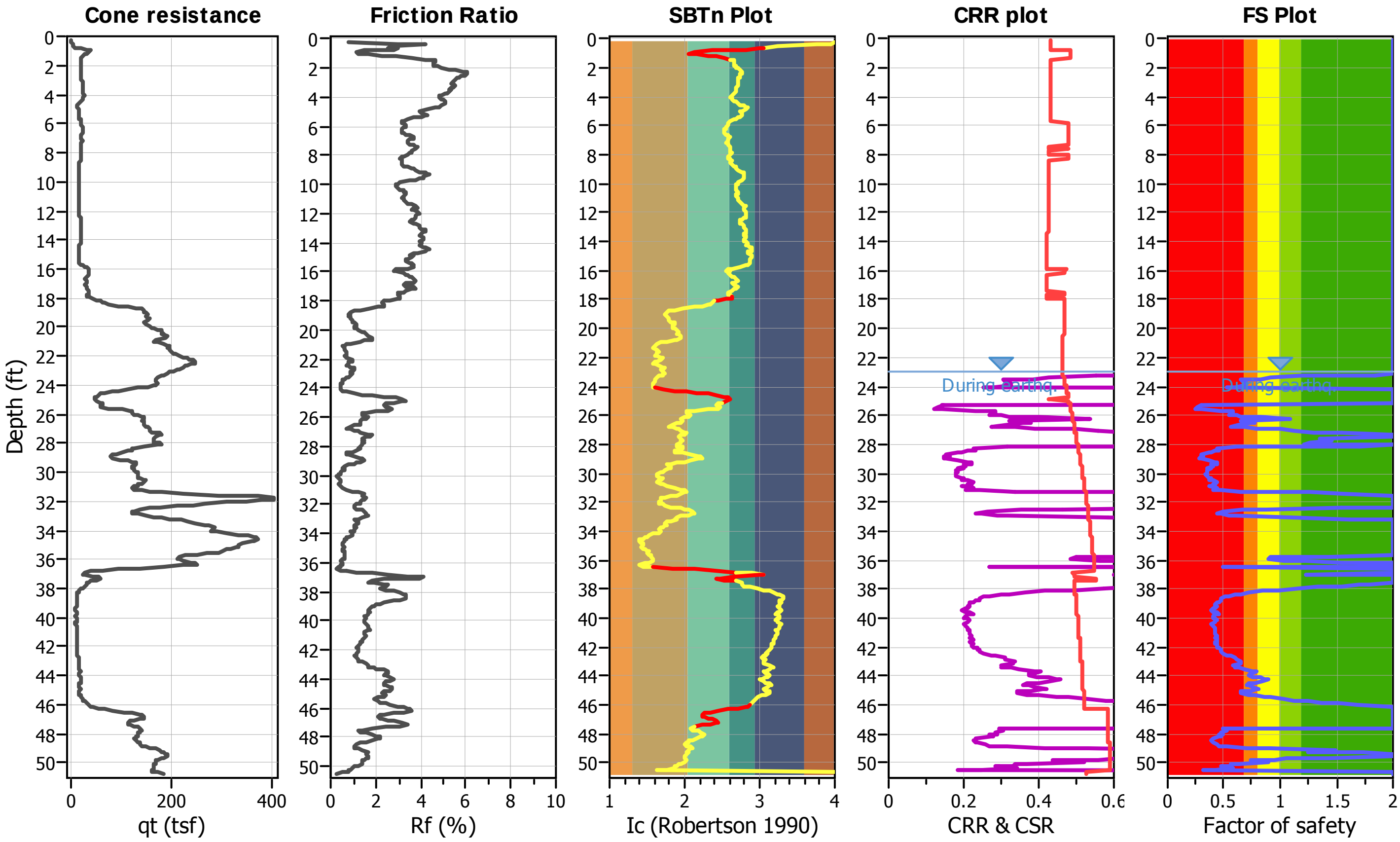
LIQUEFACTION ANALYSIS REPORT

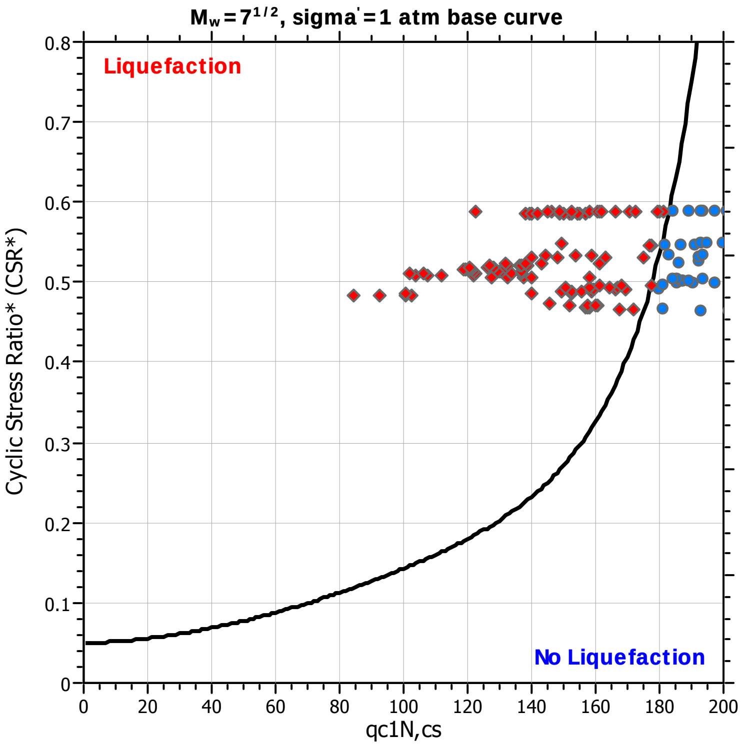
Project title : Atherton Water Capture Project **Location : 150 Watkins Ave, Atherton, CA**

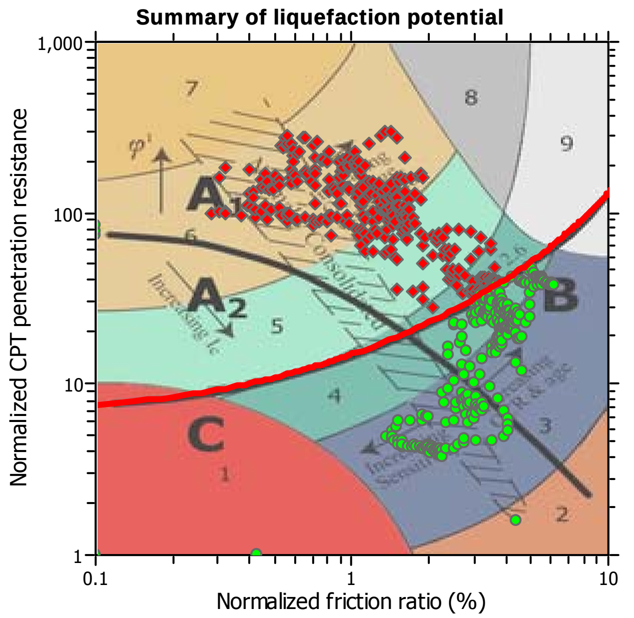
CPT file : 1-CPT05

Input parameters and analysis data

Analysis method:	I&B (2008)	G.W.T. (in-situ):	23.00 ft	Use fill:	No	Clay like behavior applied:	Sand & Clay
Fines correction method:	I&B (2008)	G.W.T. (earthq.):	23.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	8.00	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method
Peak ground acceleration:	0.65	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

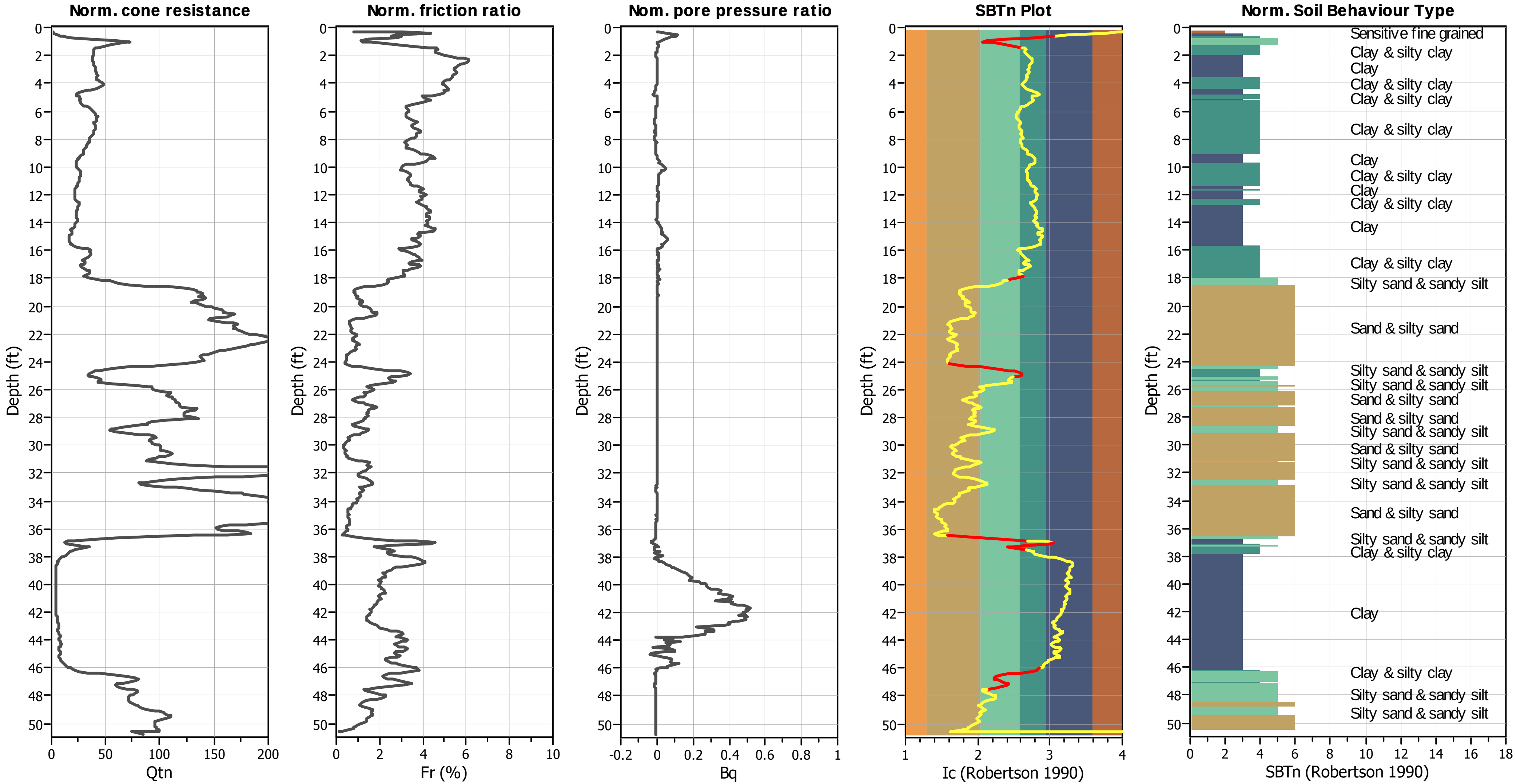






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots (normalized)



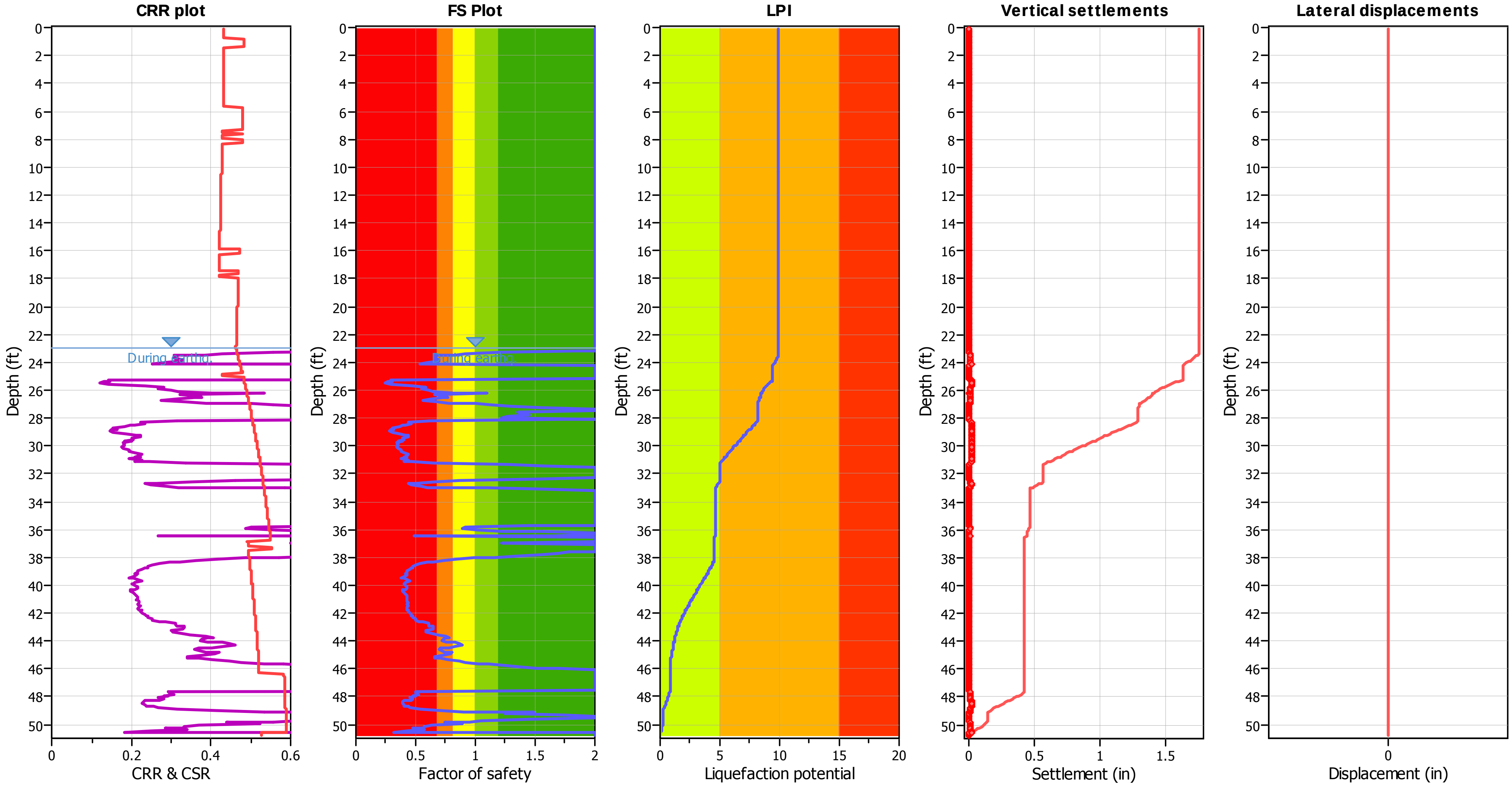
Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	23.00 ft	Fill weight:	N/A
Fines correction method:	I&B (2008)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.65	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	23.00 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to
9. Very stiff fine grained

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	23.00 ft	Fill weight:	N/A
Fines correction method:	I&B (2008)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.65	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	23.00 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

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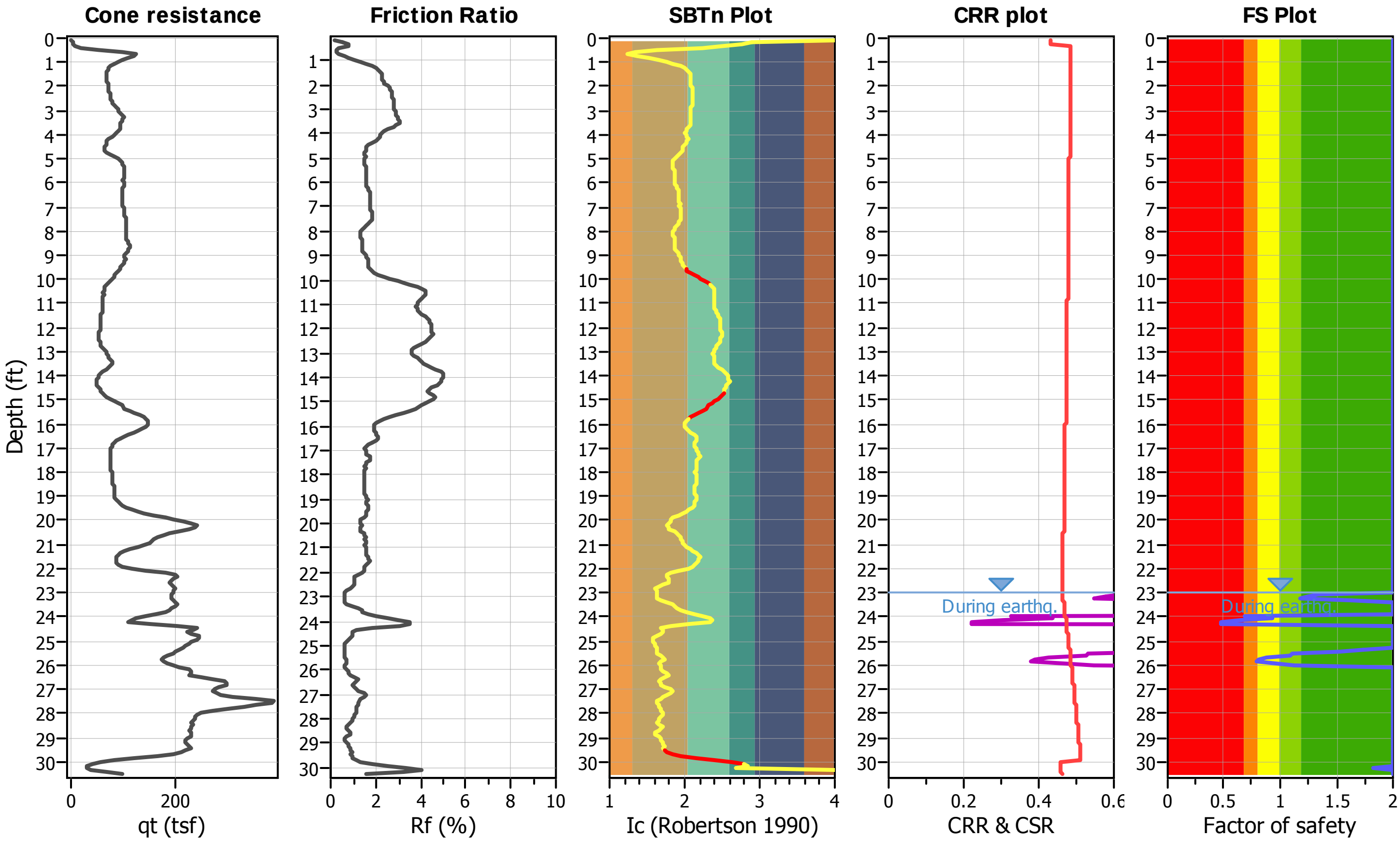
LIQUEFACTION ANALYSIS REPORT

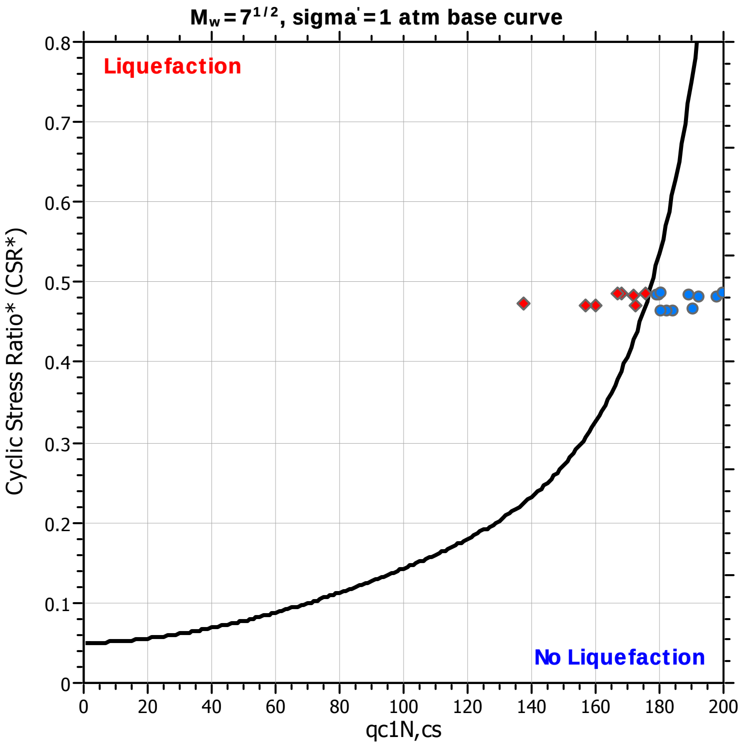
Project title : Atherton Water Capture Project **Location : 150 Watkins Ave, Atherton, CA**

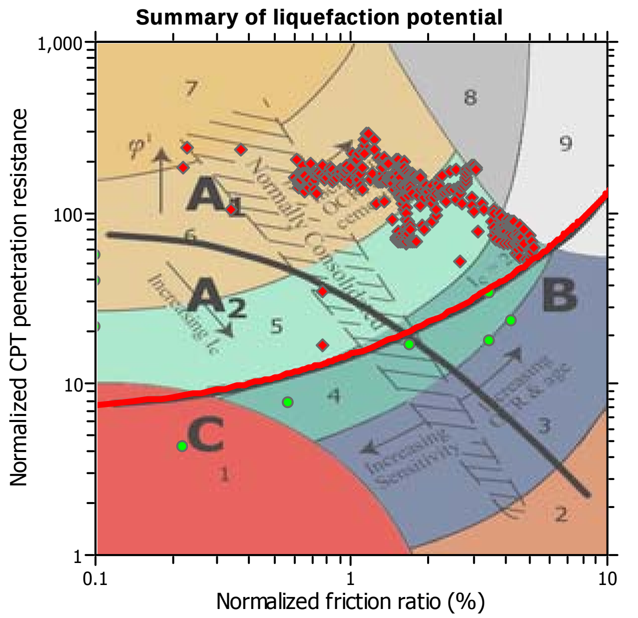
CPT file : 1-CPT06

Input parameters and analysis data

Analysis method:	I&B (2008)	G.W.T. (in-situ):	23.00 ft	Use fill:	No	Clay like behavior applied:	Sand & Clay
Fines correction method:	I&B (2008)	G.W.T. (earthq.):	23.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	8.00	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method
Peak ground acceleration:	0.65	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

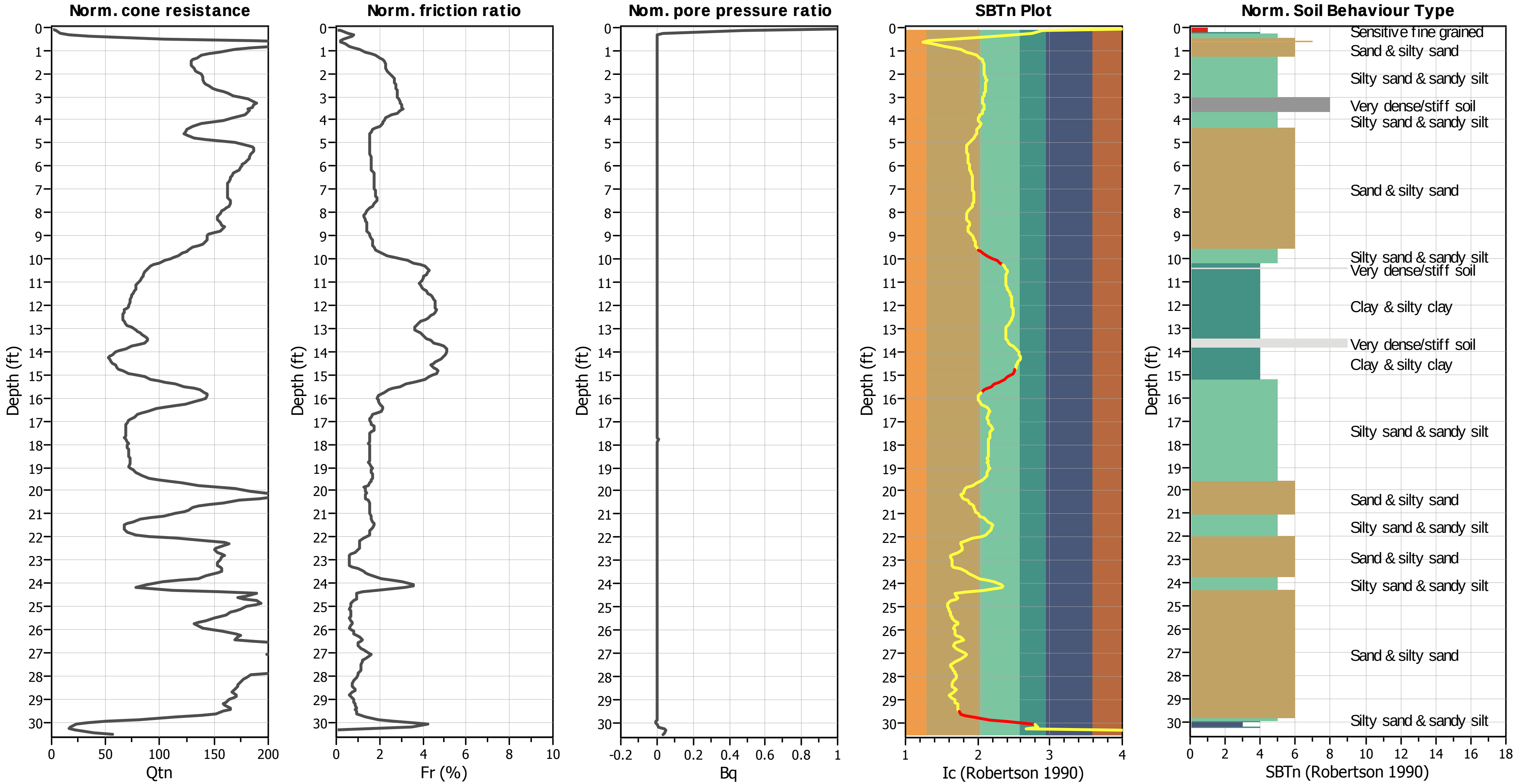






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots (normalized)



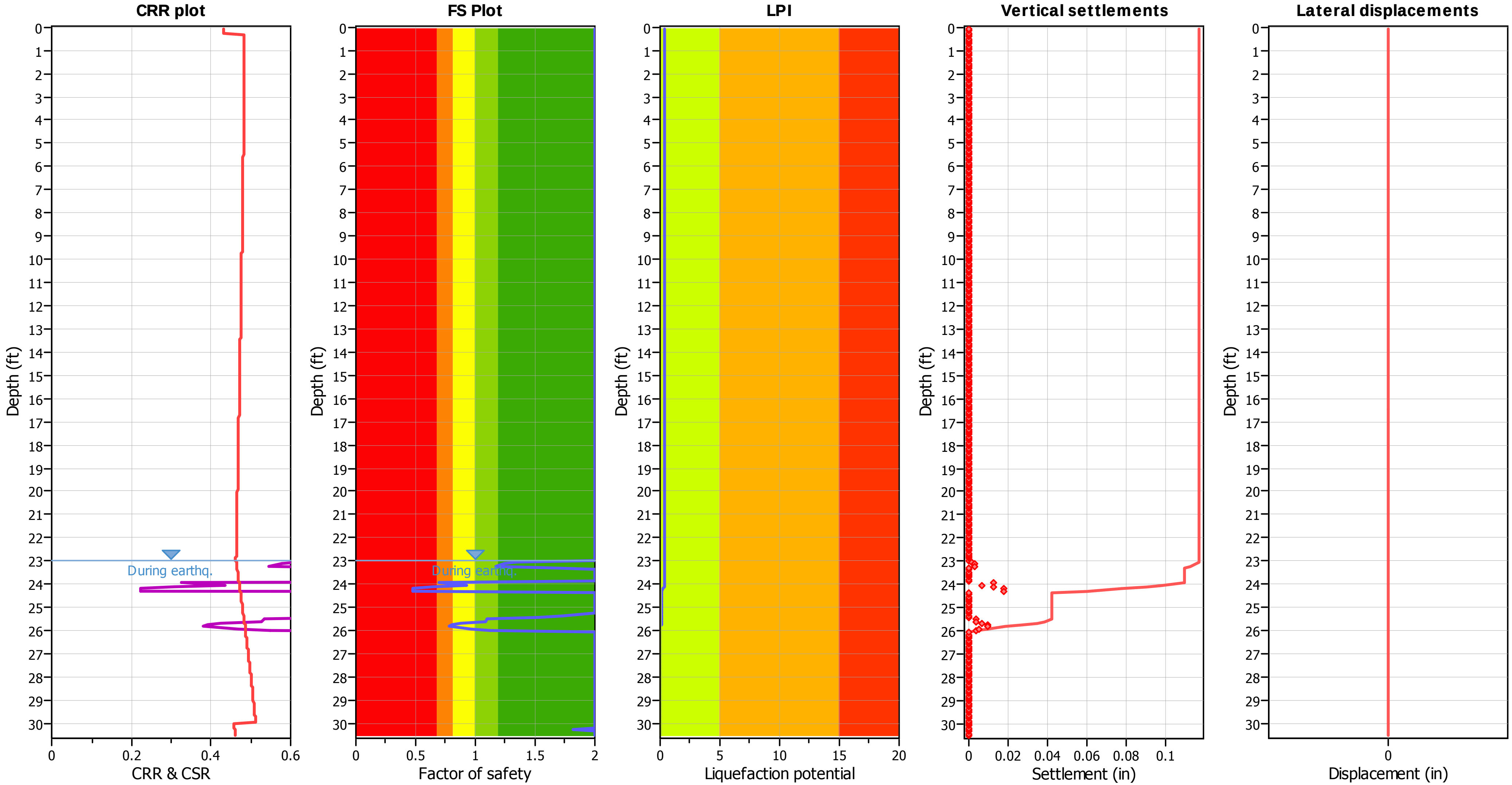
Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	23.00 ft	Fill weight:	N/A
Fines correction method:	I&B (2008)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.65	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	23.00 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

1. Sensitive fine grained	4. Clayey silt to silty	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	23.00 ft	Fill weight:	N/A
Fines correction method:	I&B (2008)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.65	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	23.00 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

SAN RAMON

SAN FRANCISCO

SAN JOSE

OAKLAND

LATHROP

ROCKLIN

SANTA CLARITA

IRVINE

CHRISTCHURCH

WELLINGTON

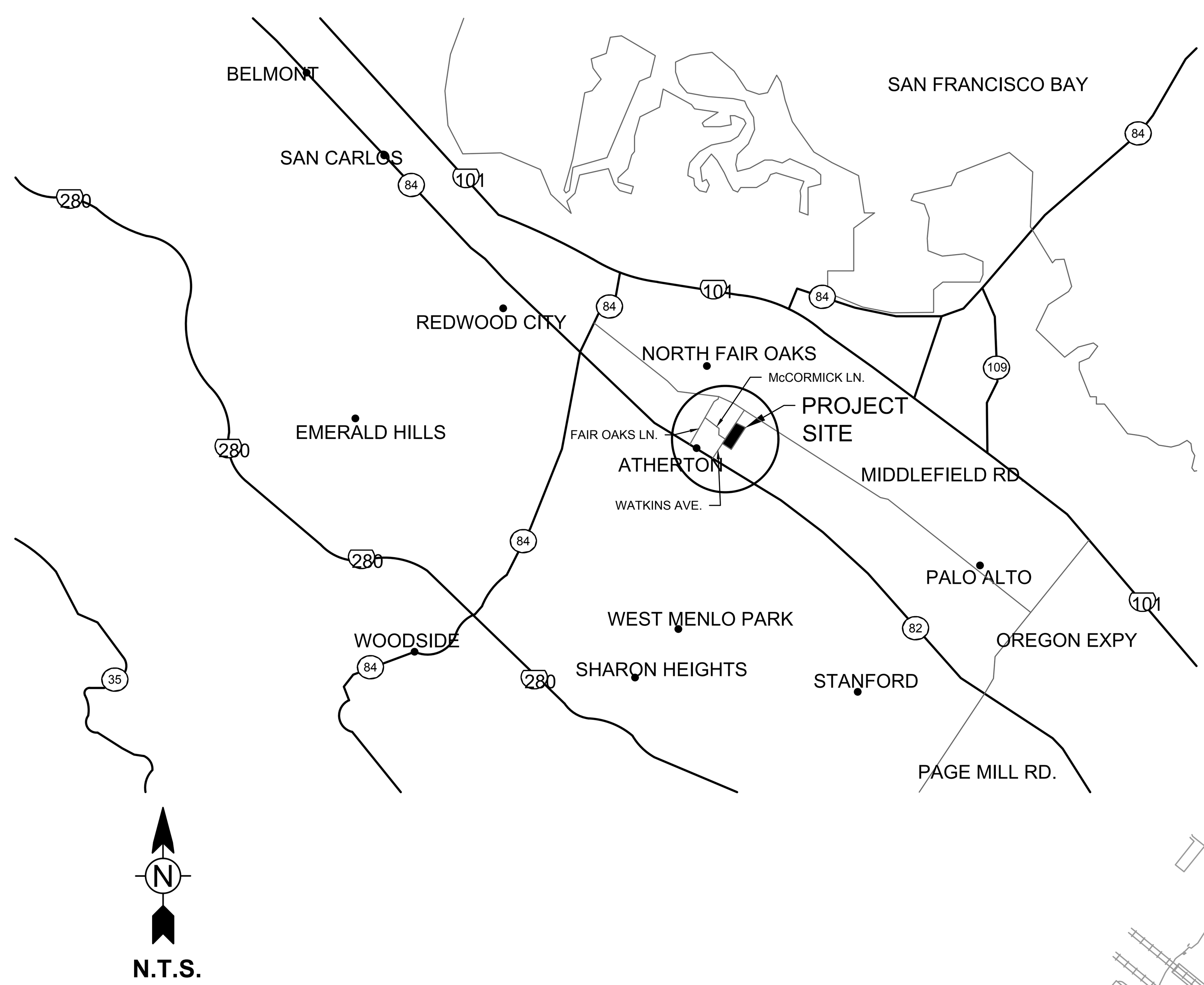
AUCKLAND

APPENDIX F: 20% DESIGN PLANS

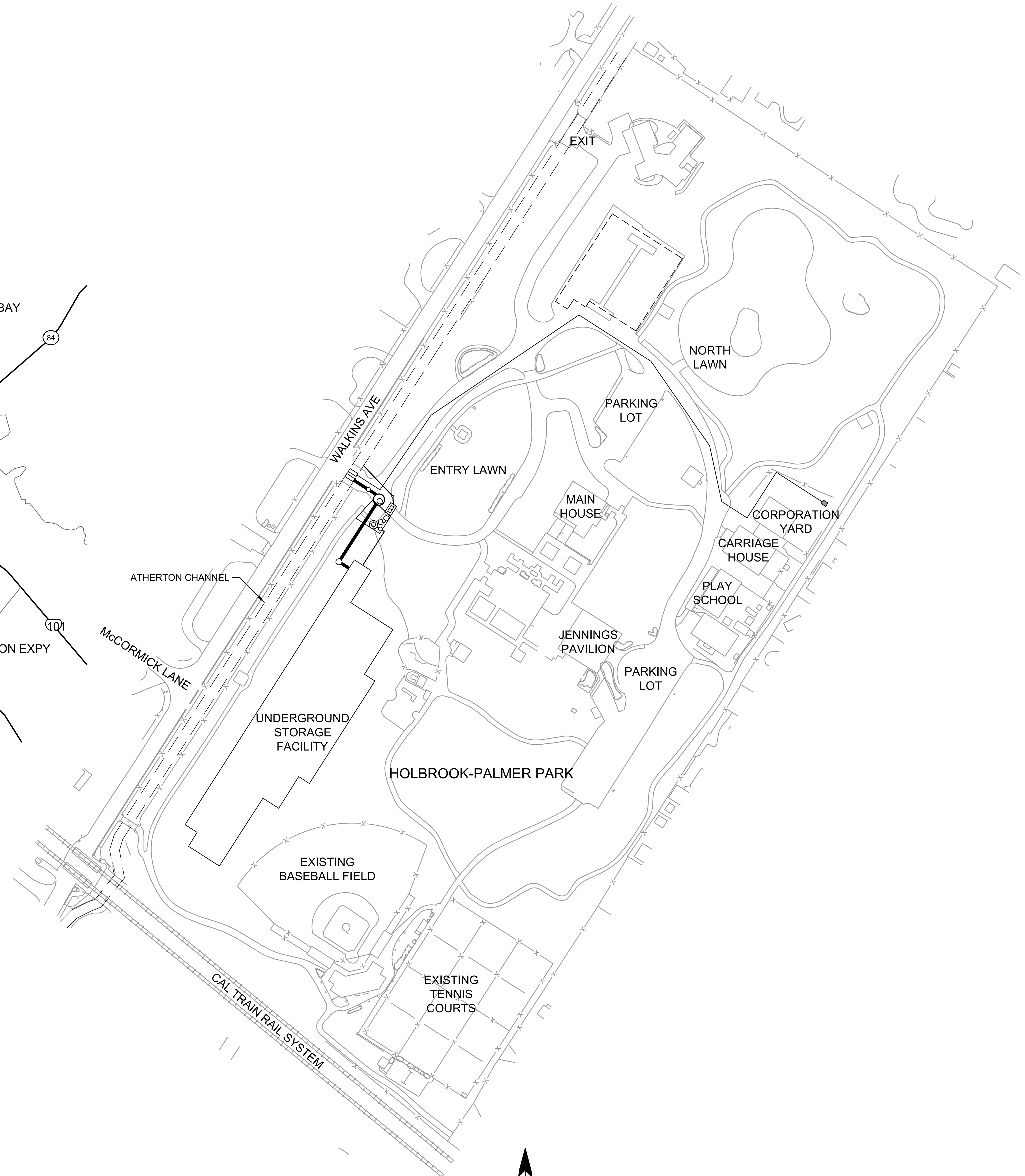
TOWN OF ATHERTON
HOLBROOK-PALMER PARK WATER CAPTURE PROJECT
SAN MATEO, COUNTY, CA

INDEX TO PROJECT PLANS

SHT. NO.	DRAWING NO.	SHEET TITLE
1	G-01	TITLE SHEET
2	G-02	GENERAL NOTES
3	C-01	SITE PLAN
4	C-02	EXISTING TOPOGRAPHY & UTILITIES
5	C-03	PLAN AND PROFILE
6	C-04	DETAILS
7	C-05	DETAILS
8	C-06	DETAILS
9	C-07	DETAILS
10	C-08	DETAILS



LOCATION MAP



VICINITY MAP

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SAN DIEGO, CA 92103
P: (858)-268-5746 F: (858)-268-5809

APPROVED BY: DATE DATE

MARK	DATE	DESCRIPTION	BY

TOWN OF ATHERTON
HOLBROOK-PALMER PARK WATER CAPTURE PROJECT
TITLE SHEET
150 WATKINS AVE, ATHERTON, CA

Project No.:	200-01297-18011
Designed By:	JF
Drawn By:	AR
Checked By:	AY

G-01

Bar Measures 1 inch
20% DESIGN 4/11/2018

NOTICE TO CONTRACTOR

GENERAL NOTES

1. ALL WORK SHALL CONFORM TO THE PROJECT TECHNICAL SPECIFICATIONS. WORK NOT COVERED BY THE PROJECT SPECIFICATIONS SHALL CONFORM TO THE STANDARD PLANS OF SAN MATEO COUNTY, THE CITY OF ATHERTON, AMERICAN PUBLIC WORKS ASSOCIATION AND CALTRANS STANDARD PLANS AND SPECIFICATIONS. (ALL LATEST EDITIONS).
2. THE CONTRACTOR SHALL NOTIFY THE CITY ENGINEER TWO (2) WORKING DAYS IN ADVANCE FOR INSPECTION PRIOR TO STARTING WORK.
3. THE CONTRACTOR SHALL NOTIFY THE UTILITY COMPANIES 48 HOURS IN ADVANCE PRIOR TO WORKING AROUND THEIR FACILITIES.
4. THE CONTRACTOR SHALL CALL UNDERGROUND SERVICE ALERT AT (800) 462-2444 48 HOURS PRIOR TO ANY EXCAVATION.
5. THE CONTRACTOR SHALL PROVIDE AND MAINTAIN AT ALL TIMES DURING CONSTRUCTION AMPLE MEANS AND DEVICES WITH WHICH TO PROMPTLY REMOVE AND PROPERLY DISPOSE OF ALL WATER FROM ANY SOURCE ENTERING THE EXCAVATION OR OTHER PARTS OF THE WORK.
6. ALL EXISTING IMPROVEMENTS, INCLUDING STRIPING, LANDSCAPING, MONUMENTS, BENCHES, REMOVED OR DAMAGED SHALL BE REPLACED "IN KIND", TO THE SATISFACTION OF THE CITY ENGINEER.
7. ANY IMPROVEMENTS DAMAGED DURING THE CONSTRUCTION SHALL BE REMOVED AND RECONSTRUCTED PER CITY OF ATHERTON STANDARD PLAN, STANDARD PLANS FOR PUBLIC WORKS CONSTRUCTION, AND TO THE SATISFACTION OF THE CITY ENGINEER.
8. THE EDGES OF ALL A.C. PATCH SHALL BE SEALED WITH ASPHALT EMULSION.
9. THE TRAFFIC CONTROL PLAN SHALL BE PREPARED BY THE GENERAL CONTRACTOR. THE TRAFFIC CONTROL AND CONSTRUCTION SIGNING SHALL BE PER WATCH HANDBOOK, OR THE MANUAL ON UNIFORM TRAFFIC CONTROL DEVICES (MUTCD) ALONG WITH THE CALIFORNIA SUPPLEMENT.
10. CONTRACTOR SHALL HAVE SUFFICIENT MATERIALS ON HAND AT ALL TIMES TO EXPEDITE ANY EMERGENCY REPAIRS.
11. WORKING HOURS SHALL BE BETWEEN 8AM AND 5PM MONDAY - FRIDAY ON ARTERIALS UNLESS OTHERWISE APPROVED BY THE CITY ENGINEER.
12. A CONSTRUCTION PERMIT IS REQUIRED FOR ANY WORK TO BE DONE IN THE PUBLIC RIGHT-OF-WAY.
13. A CONSTRUCTION BOND IS REQUIRED FOR ALL WORK TO BE DONE WITHIN THE PUBLIC RIGHT-OF-WAY.
14. CONTRACTOR SHALL PROVIDE PROOF OF WORKER'S COMPENSATION AND LIABILITY INSURANCE.
15. ALL SURVEY MONUMENTS DESTROYED AS A RESULT OF THE CONTRACTOR'S WORK SHALL BE RESET "IN KIND" AT THE CONTRACTORS EXPENSE BY A LICENSED SURVEYOR OR REGISTERED CIVIL ENGINEER. COPIES OF ALL TIES TO RESET MONUMENTS SHALL BE SUPPLIED TO THE CITY PRIOR TO RELEASE OF ANY SURETIES.
16. PARKING LOT AND AREAS ADJACENT TO PARKING LOT SHALL BE PROTECTED FROM HEAVY EQUIPMENT TO AVOID CRACKING AND/OR DAMAGING THE AREA. THE CONTRACTOR IS RESPONSIBLE FOR THE REPAIR OF ANY DAMAGES TO THIS AREA. AT MINIMUM THE CONTRACTOR SHALL GRIND AND OVERLAY, AND RE-STRIPE ANY PARKING LOT AREAS USED DURING CONSTRUCTION.
17. THE CONTRACTOR SHALL VISIT THE SITE AND BE KNOWLEDGEABLE OF CONDITIONS THEREON. THE CONTRACTOR SHALL INVESTIGATE, VERIFY AND BE RESPONSIBLE FOR ALL CONDITIONS OF THE PROJECT. THE CONTRACTOR SHALL NOTIFY THE CITY OF ANY CONDITIONS REQUIRING CORRECTION OR MODIFICATION PRIOR TO PROCEEDING WITH THE WORK.
18. THE CONTRACTOR SHALL BE RESPONSIBLE FOR THE ACCURATE PLACEMENT OF ALL WORK ON THE SITE.
19. ALL DIMENSIONS SHALL HAVE PREFERENCE OVER SCALE. DIMENSIONS SHALL BE VERIFIED IN THE FIELD PRIOR TO PROCEEDING WITH THE WORK. THE CITY SHALL BE NOTIFIED OF ANY CORRECTIONS REQUIRED.
20. NOTES AND TYPICAL DETAILS APPLY TO ALL DRAWINGS UNLESS NOTED OTHERWISE.
21. TRADE NAMES AND MANUFACTURERS REFERRED TO ARE FOR QUALITY STANDARDS ONLY. EQUALS WILL BE PERMITTED ONLY AS APPROVED BY THE CITY. SUBMITTALS AND/OR SHOP DRAWING ARE REQUIRED FOR ALL PRODUCTS OR ITEMS TO BE INSTALLED. SEE SPECIFICATIONS.
22. THE GENERAL CONTRACTOR AND ITS SUBCONTRACTORS SHALL FURNISH ALL LABOR, EQUIPMENT, AND MATERIALS; AND PERFORM ALL WORK NECESSARY, INDICATED, REASONABLY INFERRED OR REQUIRED BY ANY CODE WITH JURISDICTION. ALL CONTRACTORS SHALL COMPLETE THEIR SCOPE OF WORK FOR A COMPLETE AND PROPERLY FINISHED JOB.
23. THE GENERAL CONTRACTOR AND ALL SUBCONTRACTORS SHALL AT ALL TIMES KEEP THE PREMISES FREE FROM ACCUMULATION OF WASTE MATERIALS OR RUBBISH CAUSED BY THEIR WORK, AND AT THE COMPLETION OF THE WORK, SHALL REMOVE ALL RUBBISH FROM AND ABOUT THE JOB SITE, AND ALL THEIR TOOLS, SCAFFOLDING, AND SURPLUS MATERIALS, AND SHALL LEAVE THE JOB BROOM CLEAN.
24. THE PROJECT SITE SHALL BE KEPT NEAT AND ORDERLY. TRASH AND DEBRIS GENERATED BY CONSTRUCTION OPERATIONS SHALL BE REMOVED FROM THE PROJECT SITE ON A REGULAR BASIS.
25. THE CONTRACTOR SHALL PROVIDE PROPER ANCHORAGE OF ALL EQUIPMENT AND ASSEMBLIES SHOWN HEREIN IN ACCORDANCE WITH MANUFACTURER'S REQUIREMENTS, TRADE AND INDUSTRY STANDARDS, AND ALL GOVERNING CODES AND ORDINANCES.
26. WHERE SPECIFIC DIMENSIONS, DETAILS, OR DESIGN INTENT CANNOT BE DETERMINED, CONSULT THE ENGINEER BEFORE PROCEEDING WITH THE WORK.
27. ALL COSTS FOR PERMITS AND MISCELLANEOUS FEES SHALL BE THE RESPONSIBILITY OF THE CONTRACTOR.
28. THE CONTRACTOR SHALL LOCATE ALL EXISTING UTILITIES AND PROTECT THEM FROM DAMAGE AS REQUIRED. ALL EXISTING UTILITY CROSSINGS SHALL BE POTHOLED AND FIELD LOCATED. IN ANY INSTANCES WHERE THE CLEARANCE BETWEEN THE TOP OF THE EXISTING PIPE AND THE BOTTOM OF THE CROSSING PIPE OR CONDUIT IS LESS THAN 18", CONTRACTOR SHALL INSTALL BLANKET PROTECTION IN ACCORDANCE WITH THE STANDARD PLANS FOR PUBLIC WORKS CONSTRUCTION.
29. EXISTING ELEVATIONS AND LOCATIONS TO BE JOINED BY THE WORK SHALL BE VERIFIED BY THE CONTRACTOR BEFORE COMMENCING THE WORK. THE CONTRACTOR SHALL NOTIFY THE ENGINEER OF ANY DISCREPANCIES ON THE DRAWINGS.
30. ALL DISSIMILAR METALS SHALL BE SUFFICIENTLY ISOLATED FROM EACH OTHER TO PREVENT GALVANIC ACTION.
31. CONTRACTOR TO DESIGN AND PROVIDE ALL NECESSARY SHORING AND BRACING FOR REMOVAL ITEMS OR CONSTRUCTION OF IMPROVEMENTS.
32. CONTRACTOR SHALL BE RESPONSIBLE FOR THE COORDINATION OF ALL DRAWINGS WITH ITS SUBCONTRACTORS. ALL SUBCONTRACTORS SHALL BE FURNISHED WITH COMPLETE SETS OF PLANS AND SPECIFICATIONS BY THE CONTRACTOR.
33. NOTIFICATION: AT LEAST TEN (10) DAYS BEFORE THE START OF CONSTRUCTION, THE CONTRACTOR SHALL NOTIFY, IN WRITING, RESIDENTS ALONG WATKINS AVE , McCORMICK LANE, AND FELTON DRIVE OF THE PROPOSED CONSTRUCTION START DATE. A COPY OF SAID WRITTEN NOTIFICATION SHALL BE PROVIDED TO THE INSPECTOR FOR APPROVAL BEFORE THEY ARE DISTRIBUTED TO THE OCCUPANTS OF THE ABUTTING PROPERTY.
34. TRENCH COMPACTION AND SUBGRADE COMPACTION TESTS SHALL BE PERFORMED BY A LABORATORY DESIGNATED BY THE CITY, UTILIZING CALIFORNIA TEST METHOD NO. 216 AND 231. THE CITY WILL FURNISH FIRST TESTS, WHEN REQUESTED, AT NO COST TO THE CONTRACTOR. ANY RE-TESTING REQUIRED DUE TO FIRST TEST FAILURES SHALL BE MADE AT LOCATIONS ORDERED BY THE ENGINEER, AND SHALL BE AT THE CONTRACTOR'S EXPENSE - COST OF WHICH WILL BE DEDUCTED FROM THE CONTRACTOR'S FINAL PAYMENT.
35. ALL EXISTING IMPROVEMENTS, INCLUDING CURB AND GUTTERS, SIDEWALKS, ASPHALT CONCRETE OR P.C.C. PAVING WHICH ARE BEING JOINED OR MATCHED IN CONNECTION WITH THE PROJECT, SHALL BE JOINED, OR MATCHED IN A MANNER SATISFACTORY TO THE CITY ENGINEER, INCLUDING NECESSARY SAWCUTTING, REMOVAL, REPLACEMENT OR CAPPING. SAW CUTTING OF CONCRETE WALKS, CURBS, GUTTERS, AND PCC PAVEMENT SHALL OCCUR AT THE NEAREST JOINT UNLESS NOTED IN THE PLANS.
36. THE CONTRACTOR SHALL CONDUCT CONSTRUCTION OPERATIONS IN SUCH A MANNER THAT STORM, SEWER, IRRIGATION, OR OTHER WATERS MAY PROCEED UNINTERRUPTED ALONG THE STREET OR THEIR CONYENCE COURSES.
37. ALL BURIED VALVES SHALL BE PROVIDED WITH RISERS, VALVE BOXES AND COVERS. NUTS AND BOLTS SHALL BE 316 STAINLESS STEEL.

DEMOLITION

1. THE CONTRACTOR SHALL BE RESPONSIBLE FOR ALL DEMOLITION REQUIRED FOR PROPER INSTALLATION OF IMPROVEMENTS SHOWN IN THE PLANS AND SPECIFICATIONS. SEE CIVIL, ARCHITECTURAL, STRUCTURAL, ELECTRICAL, AND MECHANICAL DRAWINGS FOR ADDITIONAL DEMOLITION NOTES.
2. EXISTING ITEMS TO REMAIN THAT ARE DAMAGED DURING THE COURSE OF CONSTRUCTION SHALL BE RESTORED TO THEIR ORIGINAL CONDITION BY THE CONTRACTOR AT NO ADDITIONAL COST TO THE CITY.
3. SEE ADDITIONAL DEMOLITION NOTES THROUGHOUT THE PLANS.
4. PRIOR TO ANY CONCRETE REMOVAL, THE CONTRACTOR SHALL FIELD LOCATE AND STAKE OR MARK OUT DROP INLET LOCATION FOR LOS ANGELES COUNTY FLOOD CONTROL DISTRICT APPROVAL.

PATCHING NOTE

1. THE CONTRACTOR SHALL BE RESPONSIBLE FOR PATCHING ALL AREAS AT OR ADJACENT TO DEMOLISHED ITEMS SPECIFIED ELSEWHERE IN THESE PLANS. THE INTENT OF PATCHING IS TO RESTORE DEMOLISHED AREAS TO ORIGINAL CONDITION AS CLOSE AS REASONABLY POSSIBLE. PATCHING OR RESTORATION SHALL ALSO TAKE PLACE AT ALL AREAS TO REMAIN THAT RECEIVE DAMAGE DURING THE COURSE OF CONSTRUCTION UNDER THIS CONTRACT. THE CITY SHALL BE THE SOLE DETERMINANT AS TO THE REQUIREMENTS OR ACCEPTABILITY OF PATCHING OR RESTORATION WORK. ALL PATCHING MATERIALS, FINISHES, OR COLORS SHALL MATCH ADJACENT SURFACES AS CLOSELY AS POSSIBLE. SEE SPECIFICATIONS AND CITY OF ATHERTON STANDARD PLANS.

EXISTING FACILITY NOTE

1. THESE PLANS HAVE BEEN PREPARED FROM FIELD SURVEYS PERFORMED BY TETRA TECH AND RECORD DOCUMENTS FURNISHED BY THE CITY OF ATHERTON. ALL EXISTING INFORMATION SHOWN ON THE PLANS PREPARED BY TETRA TECH HAS BEEN TRANSFERRED FROM DOCUMENTS PROVIDED BY THE CITY OF ATHERTON. TETRA TECH HAS PROVIDED DRAFTING SERVICES ONLY WITH REGARD TO DEPICTING EXISTING IMPROVEMENTS AND CONDITIONS. TETRA TECH CANNOT GUARANTEE THE ACCURACY OR COMPLETENESS OF EXISTING INFORMATION SHOWN. IF CONTRACTOR NOTICES ANY DISCREPANCIES BETWEEN EXISTING INFORMATION SHOWN ON THESE PLANS AND ACTUAL CONDITIONS, THE CONTRACTOR SHALL GIVE NOTICE OF SUCH FACT TO THE CITY OR ITS REPRESENTATIVE WITHOUT DELAY.

JOB SITE RESPONSIBILITY

1. CONSTRUCTION CONTRACTOR AGREES THAT IN ACCORDANCE WITH GENERALLY ACCEPTED CONSTRUCTION PRACTICES, CONSTRUCTION CONTRACTOR WILL BE REQUIRED TO ASSUME SOLE AND COMPLETE RESPONSIBILITY FOR JOB SITE CONDITIONS DURING THE COURSE OF CONSTRUCTION OF THE PROJECT, INCLUDING SAFETY OF ALL PERSONS AND PROPERTY; AND THAT THIS REQUIREMENT SHALL BE MADE TO APPLY CONTINUOUSLY AND NOT BE LIMITED TO NORMAL WORKING HOURS.

SECURITY DURING CONSTRUCTION

1. THE CONTRACTOR SHALL COORDINATE WITH THE CITY AND PROVIDE CONTINUOUS SECURITY FOR THE CONSTRUCTION AREAS OF THIS FACILITY DURING THE CONSTRUCTION OF THE IMPROVEMENTS. THIS SECURITY MUST BE IN PLACE 24 HOURS EACH WORK DAY, HOLIDAY, AND WEEKEND DAY AS REQUIRED TO MEET THE SATISFACTION OF THE CITY. THE CONTRACTOR SHALL BE RESPONSIBLE TO THE CITY FOR ANY LOSS OF PROPERTY OR DAMAGE TO PROPERTY CAUSED BY THE CONTRACTOR'S FAILURE TO PROVIDE ADEQUATE SECURITY.

CORRELATION WITH SPECIFICATIONS

1. THE GENERAL NOTES ON THIS SHEET AND SHOWN ELSEWHERE ON THE PLANS ARE INTENDED TO BE COMPLEMENTARY WITH THE SPECIFICATIONS AND THE PROJECT MANUAL. SEE SPECIFICATIONS FOR PRECEDENCE OF DOCUMENTS. CONTRACTOR SHALL CAREFULLY REVIEW AND BECOME FAMILIAR WITH THE CONTENT OF ALL CONTRACT DOCUMENTS.

GRADING NOTES

1. THE LOCATION AND PROTECTION OF ALL UTILITIES IS THE RESPONSIBILITY OF THE CONTRACTOR.
2. THE LOCATION OF THE EXISTING UTILITIES AND SUBSTRUCTURES SHOWN HEREON HAVE BEEN TAKEN FROM RECORDS. THE ENGINEER DOES NOT WARRANT THE COMPLETENESS OR CORRECTNESS OF THE LOCATIONS. IT SHALL BE THE CONTRACTOR'S RESPONSIBILITY TO REVIEW PLANS, AS-BUILTS, AND LOCATE ALL UNDERGROUND FACILITIES, BEFORE COMMENCING WORK, AND PROTECT THEM FROM DAMAGE. SHOULD UTILITIES BE ENCOUNTERED THAT ARE NOT INDICATED, NOTIFY THE ENGINEER IMMEDIATELY. MARK HIGH VOLTAGE ELECTRICAL SERVICES WITH PROPER WARNING SIGNS AND BARRICADES.
3. ALL EXPORT OF MATERIALS FROM THE SITE MUST GO TO A PERMITTED SITE APPROVED BY THE BUILDING OFFICIAL OR A LEGAL DUMPSITE. RECEIPTS FOR ACCEPTANCE OF EXCESS MATERIAL BY A DUMPSITE ARE REQUIRED AND MUST BE PROVIDED TO THE BUILDING OFFICIAL UPON REQUEST.
4. A COPY OF THE GRADING PERMIT AND APPROVED GRADING PLANS MUST BE IN THE POSSESSION OF THE CONTRACTOR AND AVAILABLE AT THE SITE AT ALL TIMES.
5. SITE BOUNDARIES, EASEMENTS, DRAINAGE DEVICES, RESTRICTED USE AREAS SHALL BE LOCATED PER CONSTRUCTION STAKING BY FIELD ENGINEER OR LICENSED SURVEYOR. PRIOR TO GRADING, AS REQUESTED BY THE BUILDING OFFICIAL, ALL PROPERTY LINES, EASEMENTS, AND RESTRICTED USE AREAS SHALL BE STAKED.
6. IF GRADING AUTHORIZED BY THIS PLAN IS TO EXTEND THROUGH THE RAINY SEASON, NOVEMBER 1 THROUGH APRIL 15 OF THE FOLLOWING YEAR, SEPARATE UPDATED PLANS FOR EROSION CONTROL MUST BE SUBMITTED FOR APPROVAL PRIOR TO OCTOBER 1.
7. TRANSFER OF RESPONSIBILITY: IF THE CIVIL ENGINEER, THE SOIL ENGINEER, OR THE ENGINEERING GEOLOGIST OF RECORD IS CHANGED DURING GRADING, THE WORK SHALL BE STOPPED UNTIL THE REPLACEMENT AGREED IN WRITING TO ACCEPT THEIR RESPONSIBILITY WITHIN THE AREA OF TECHNICAL COMPETENCE FOR ARRIVAL UPON COMPLETION OF THE WORK. IT SHALL BE THE DUTY OF THE PERMITTEE TO NOTIFY THE BUILDING OFFICIAL IN WRITING OF SUCH CHANGE PRIOR TO THE RECOMMENCEMENT OF SUCH GRADING.
8. SHORING SHALL COMPLY WITH THE CALIFORNIA ADMINISTRATIVE CODE, TITLE 8; CONSTRUCTION SAFETY ORDERS, "INJURY AND ILLNESS PREVENTION - SAFETY MEASURES" OF THE GENERAL CONDITIONS AND "EXCAVATION SHORING, FORMS, & FALSE WORK" OF THE GENERAL REQUIREMENTS. SHORING PLANS SHALL BE SUBMITTED TO THE PROJECT ENGINEER FOR REVIEW AND APPROVAL. SHORING PLANS SHALL BE DESIGNED IN ACCORDANCE WITH RECOMMENDATIONS OF THE PROJECT GEOTECHNICAL ENGINEERING REPORT.
9. FOR TRENCHES OR EXCAVATIONS 5 FEET OR MORE IN DEPTH INTO WHICH A PERSON IS REQUIRED TO DESCEND, THE CONTRACTOR SHALL OBTAIN NECESSARY PERMITS FROM THE STATE OF CALIFORNIA DIVISION OF INDUSTRIAL SAFETY PRIOR TO THE ISSUANCE OF A BUILDING OR GRADING PERMIT.

ADDITIONAL NOTES TO CONTRACTOR ARE ON SHEET G-04.

INDEX TO STANDARD PLAN

STD PLAN	SPPWC TITLE
120-2	CURB AND GUTTER - BARRIER
224-2	SUPPORT FOR CONDUITS ACROSS TRENCHES
225-2	BLANKET PROTECTION FOR PIPES
320-2	MANHOLE - PIPE TO PIPE MAIN LINE ID = 36" (900mm) OR LARGER
324-2	MANHOLE SHAFT WITH ECCENTRIC REDUCER
331-3	JUNCTION STRUCTURE - PIPE TO PIPE INLET ID ≥ OR OD > 1/2 MAIN LINE ID
332-2	JUNCTION STRUCTURE - PIPE TO PIPE INLET ID ≤ 24" (600mm)
334-2	JUNCTION STRUCTURE - PIPE TO PIPE INLET ID < 30" (750mm)
342-2	TRANSITION STRUCTURE
633-4	36" (914mm) MANHOLE FRAME AND COVER
635-3	STEEL STEP

ABBREVIATIONS

ABBREVIATION	WORD OR WORDS	ABBREVIATION	WORD OR WORDS
AC	ASPHALT CONCRETE	MH	MANHOLE
AVE	AVENUE	PT	POINT
BLVD	BOULEVARD	R/W	RIGHT OF WAY
BM	BENCH MARK	S	SLOPE
CLF	CHAIN LINK FENCE	SD	STORM DRAIN
CMP	CORRUGATED METAL PIPE	SF	SQUARE FOOT
CONST	CONSTRUCTION	SPEC	SPECIFICATIONS
DIA	DIAMETER	SPPWC	STANDARD PLANS FOR PUBLIC WORKS CONSTRUCTION
DWG	DRAWING		
EC	END OF CURVE	SS	SANITARY SEWER
EL	ELEVATION	ST	STREET
E	EXISTING	STA	STATION
FL	FLOW LINE	STD	STANDARD
GC	GRADE CHANGE	W	WIDTH
HORIZ	HORIZONTAL		
INV	INVERT		
L	LENGTH		

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APPROVED BY: DATE DATE

MARK	DATE	DESCRIPTION	BY

TOWN OF ATHERTON
HOLBROOK-PALMER PARK
WATER CAPTURE PROJECT

GENERAL NOTES
150 WATKINS AVE, ATHERTON, CA

Project No.:	200-01297-18011
Designed By:	JF
Drawn By:	AR
Checked By:	AY

G-02

Bar Measures 1 inch

20% DESIGN 4/11/2018

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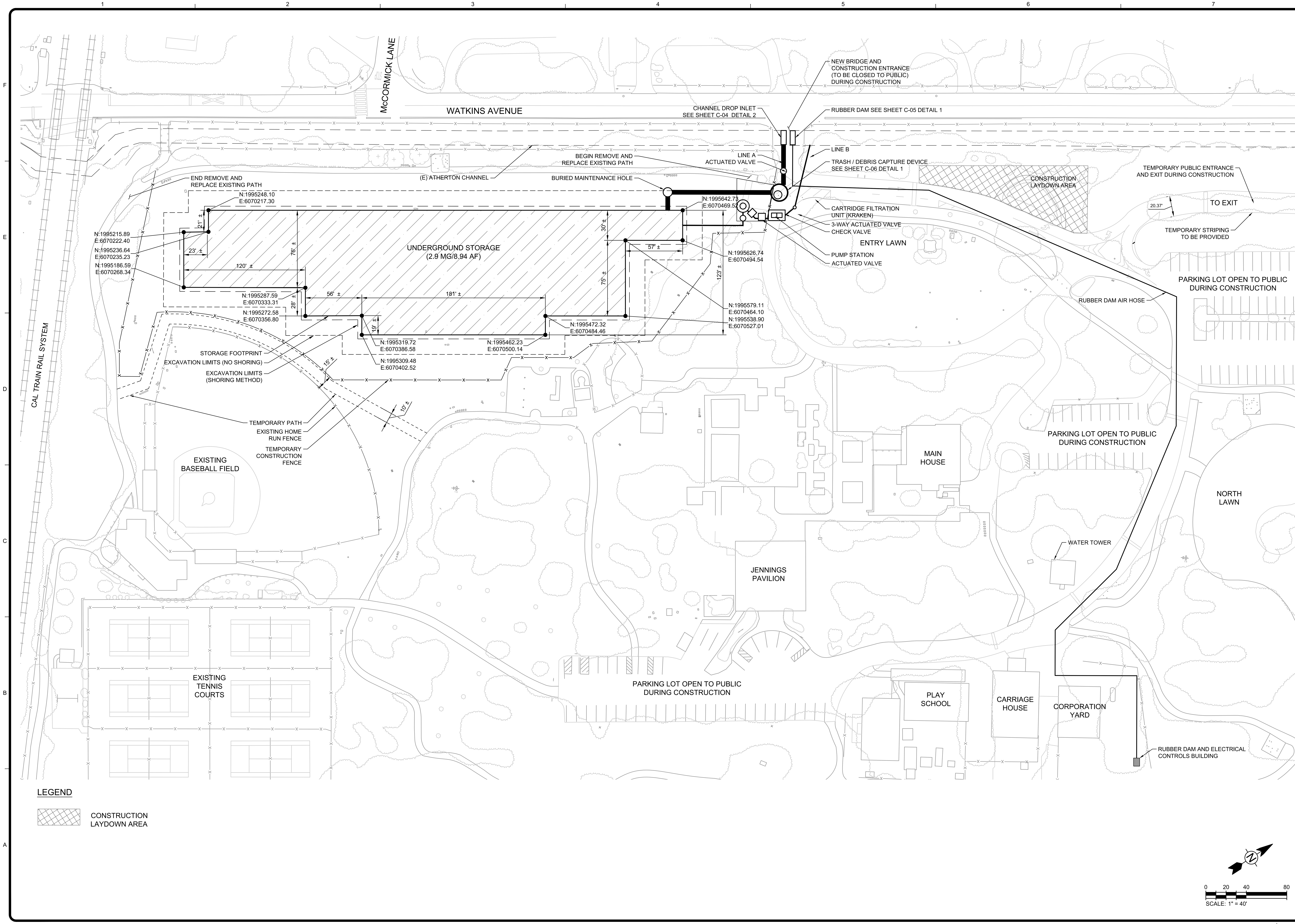

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SAN DIEGO, CA 92103
P: (858)-268-5746 F: (858)-268-5809

APPROVED BY:

DATE

DATE

MARK	DATE	DESCRIPTION	BY

TOWN OF ATHERTON
HOLBROOK-PALMER PARK
WATER CAPTURE PROJECT

SITE PLAN

150 WATKINS AVE, ATHERTON, CA

Project No.:	200-01297-18011
Designed By:	JF
Drawn By:	AR
Checked By:	AY

C-01

Bar Measures 1 inch

20% DESIGN 4/11/2018

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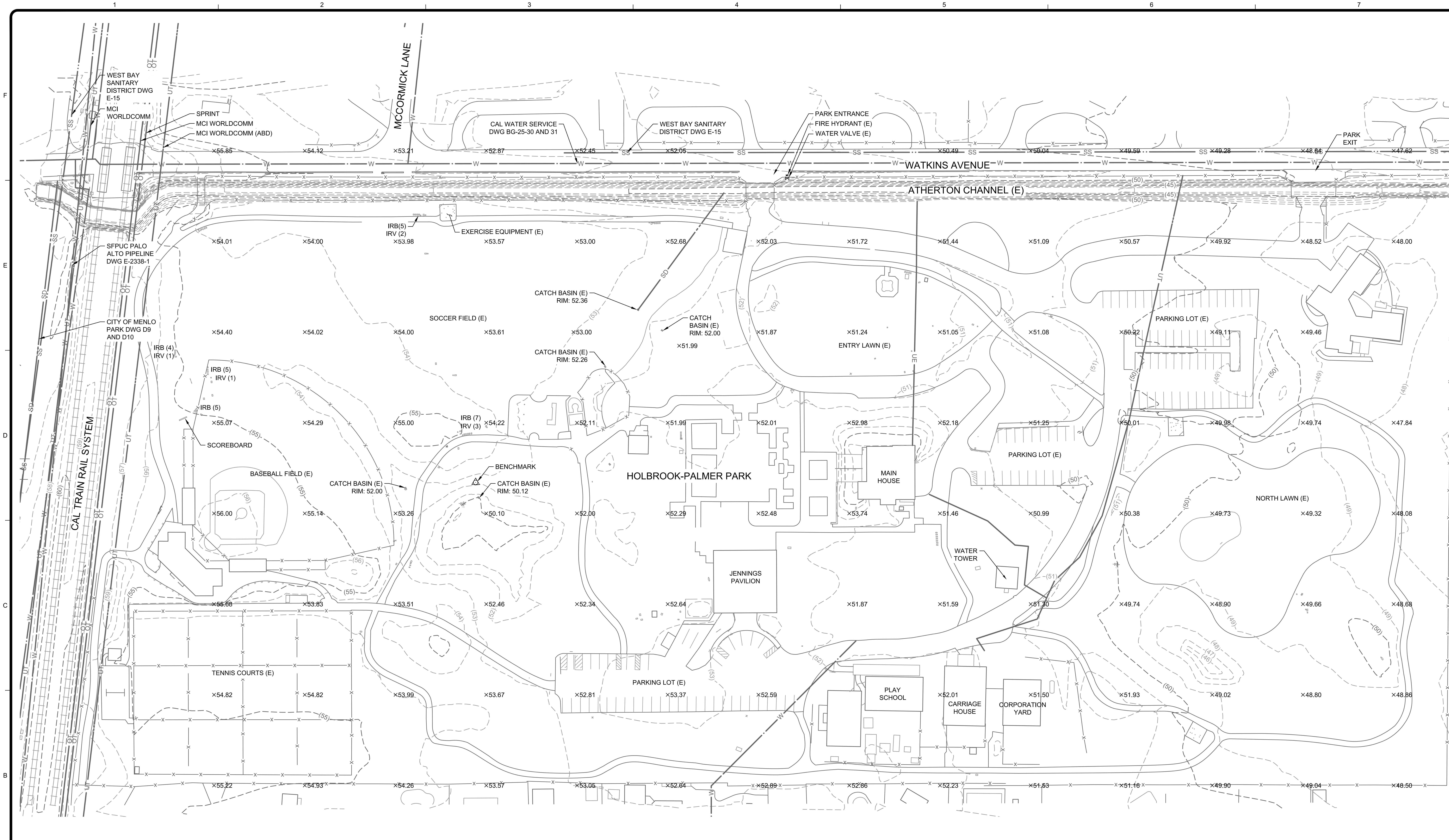

LEGEND

— W —	EXISTING WATER
— SD —	EXISTING STORM DRAIN
— SS —	EXISTING SANITARY SEWER
— OT —	EXISTING OVERHEAD TELEPHONE
— UE —	EXISTING UNDERGROUND ELECTRICAL
— UT —	EXISTING UNDERGROUND TELEPHONE
△	LOCAL BENCHMARK SET 5/8" REBAR AND ORANGE CAP

SURVEY NOTES

BENCHMARK
ELEVATIONS ARE BASED ON THE NORTH AMERICAN VERTICAL DATUM OF 1988 AND WERE DERIVED FROM POST PROCESSING GPS DATA AGAINST THE CALIFORNIA GEOSPATIAL REFERENCE CENTER (CSRC) EPOC 2017.50 PUBLISHED VALUES OF THE CONTINUOUSLY OPERATING REFERENCE STATION (CORS) "SLAC". ELEVATION: 317.55

ELEVATIONS ARE DEFINED LOCALLY BY A SET 5/8 INCH REBAR AND ORANGE CAP IN HOLBROOK-PALMER PARK NORTHWESTERLY OF THE BASEBALL FIELD AND SOUTHEASTERLY OF JENNINGS PAVILION. ELEVATION: 52.45 NAVD 88

BASIS OF BEARING
THE BASIS OB BEARINGS FOR THIS SURVEY IS THE CALIFORNIA COORDINATE SYSTEM, ZONE 3, NAD 83 AS DETERMINED LOCALLY BY A LINE BETWEEN CONTINUOUS OPERATING REFERENCE STATION (CORS) "SLAC" AND UNAVCO STATION "P220" BEING S06°19'15"W AS DERIVED FROM GEODETIC VALUES PUBLISHED BY THE CALIFORNIA SPATIAL REFERENCE CENTER (CSRC) EPOCH 2017.50.

0 25' 50' 100'
SCALE: 1" = 50'

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APPROVED BY: DATE DATE

MARK	DATE	DESCRIPTION	BY

TOWN OF ATHERTON
HOLBROOK-PALMER PARK
WATER CAPTURE PROJECT
EXISTING TOPOGRAPHY
EXISTING UTILITIES
150 WATKINS AVE, ATHERTON, CA

Project No.:	200-01297-18011
Designed By:	JF
Drawn By:	AR
Checked By:	AY

C-02

Bar Measures 1 inch
20% DESIGN 4/11/2018

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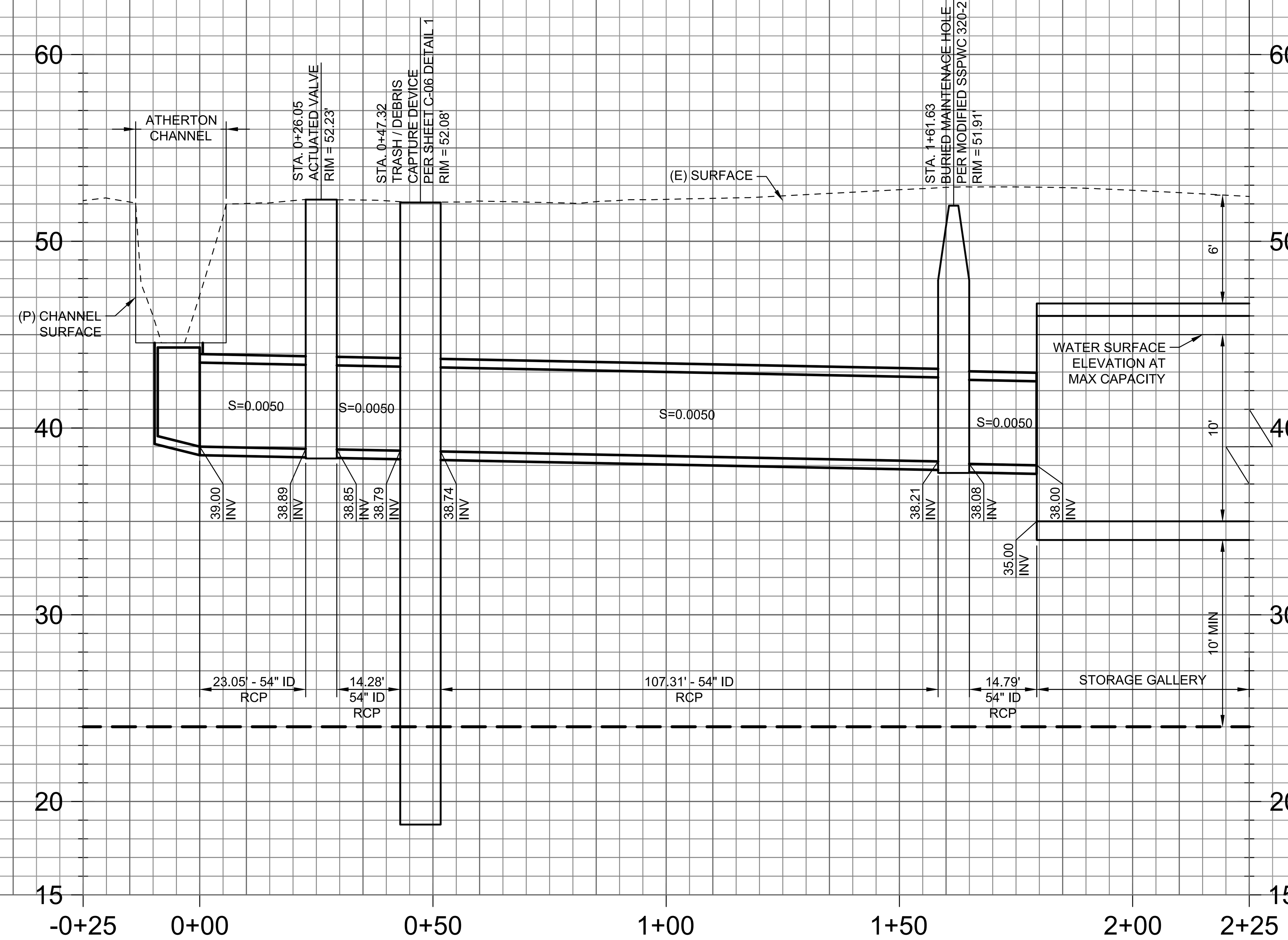
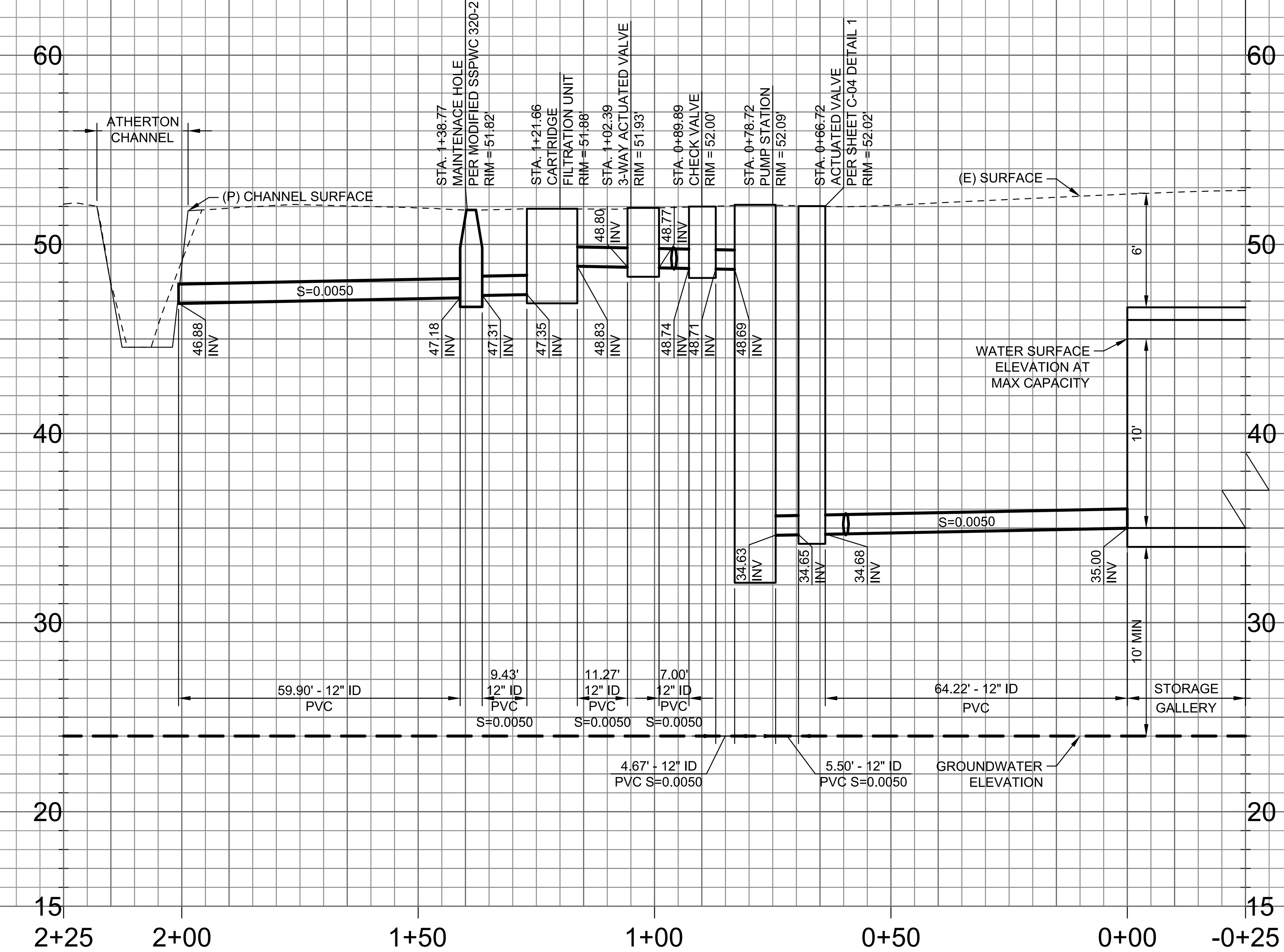
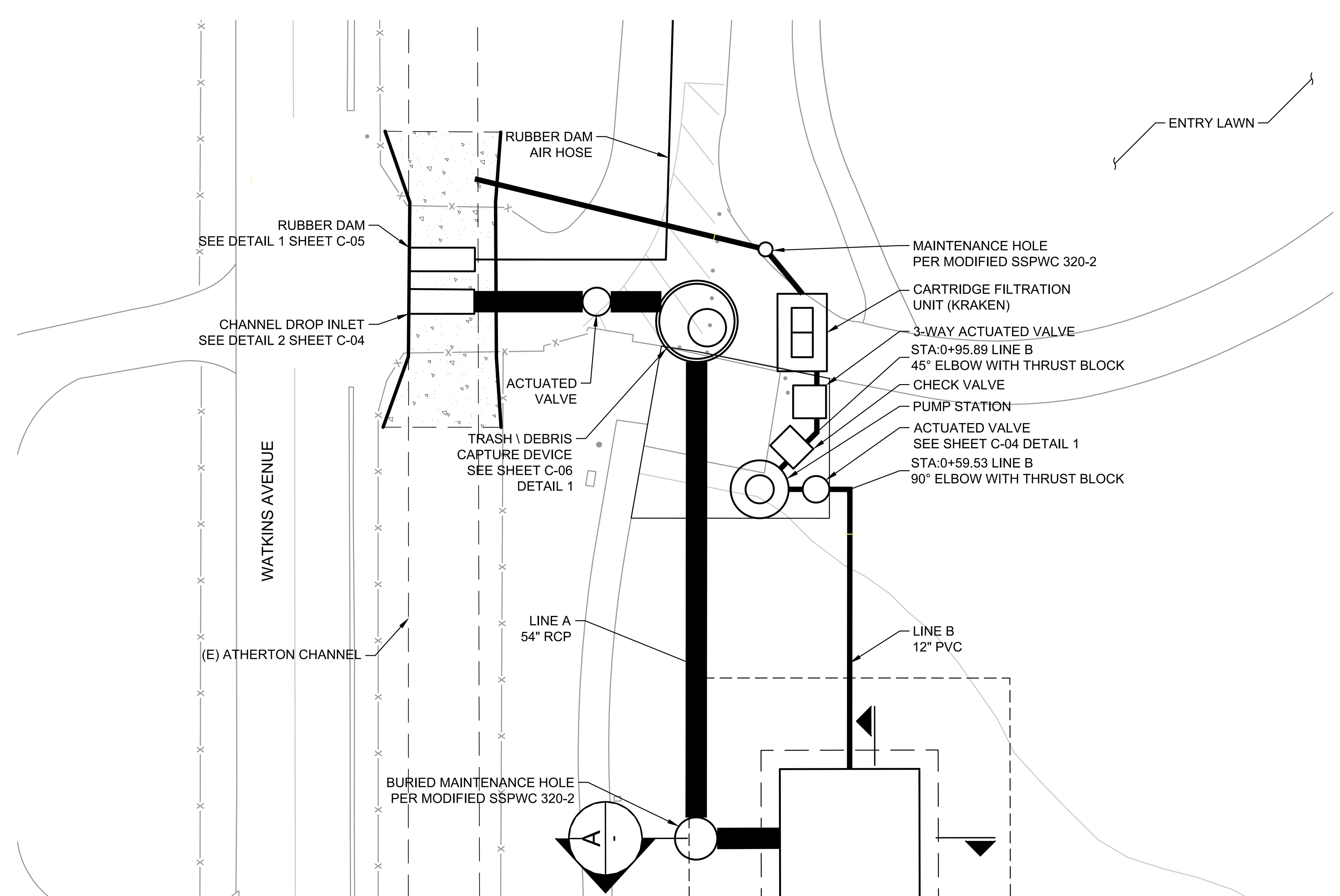


PROFILE LINE A
SCALE: HORIZ 1" = 20'
VERT 1" = 5'



PROFILE LINE B
SCALE: HORIZ 1" = 20'
VERT 1" = 5'



0 10 20 40
SCALE: 1" = 20'

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APPROVED BY:
DATE
DATE

MARK	DATE	DESCRIPTION	BY

TOWN OF ATHERTON
HOLBROOK-PALMER PARK
WATER CAPTURE PROJECT

PLAN AND PROFILE

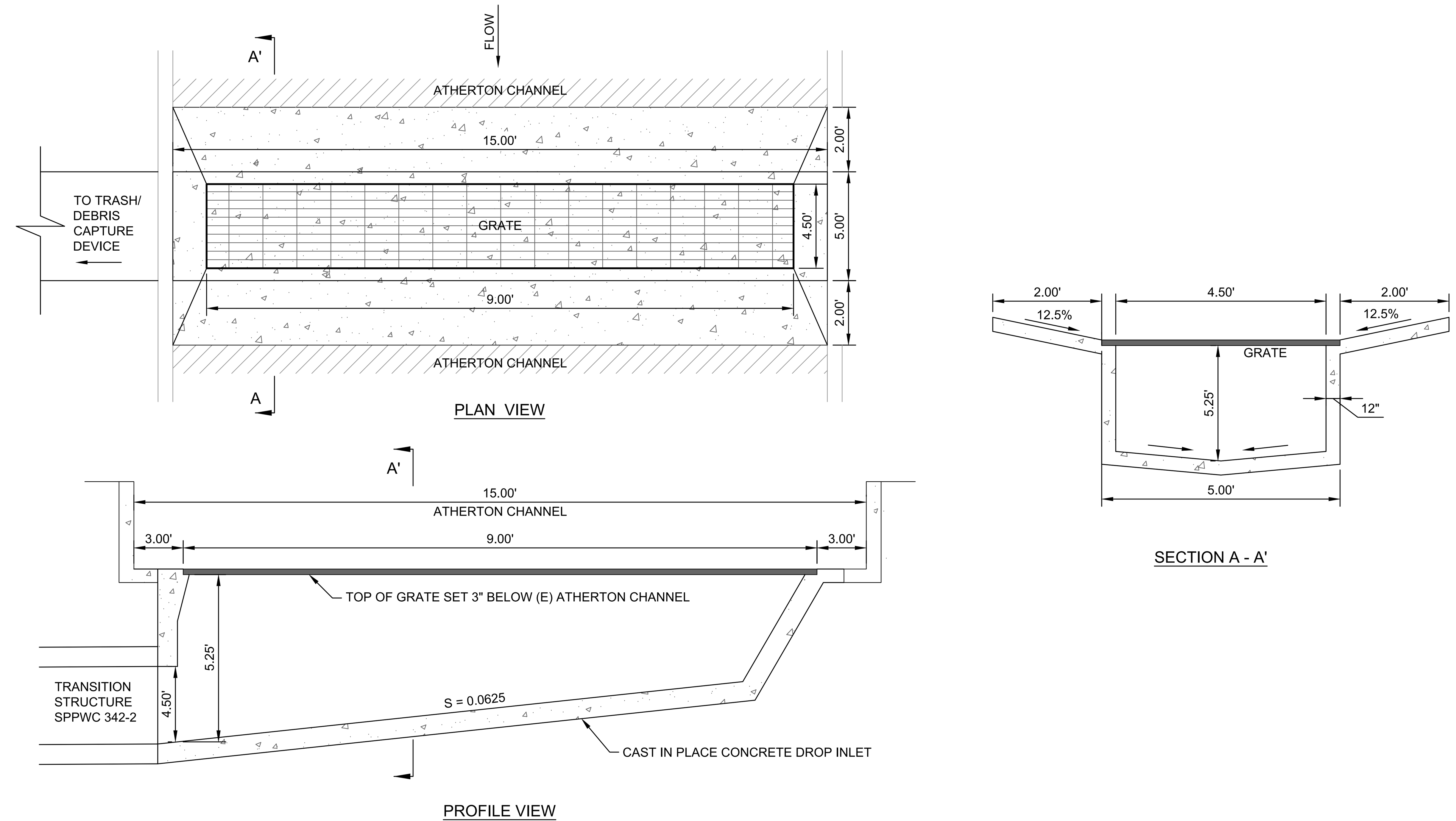
150 WATKINS AVE, ATHERTON, CA

Project No.:	200-01297-18011
Designed By:	JF
Drawn By:	AR
Checked By:	AY

C-03

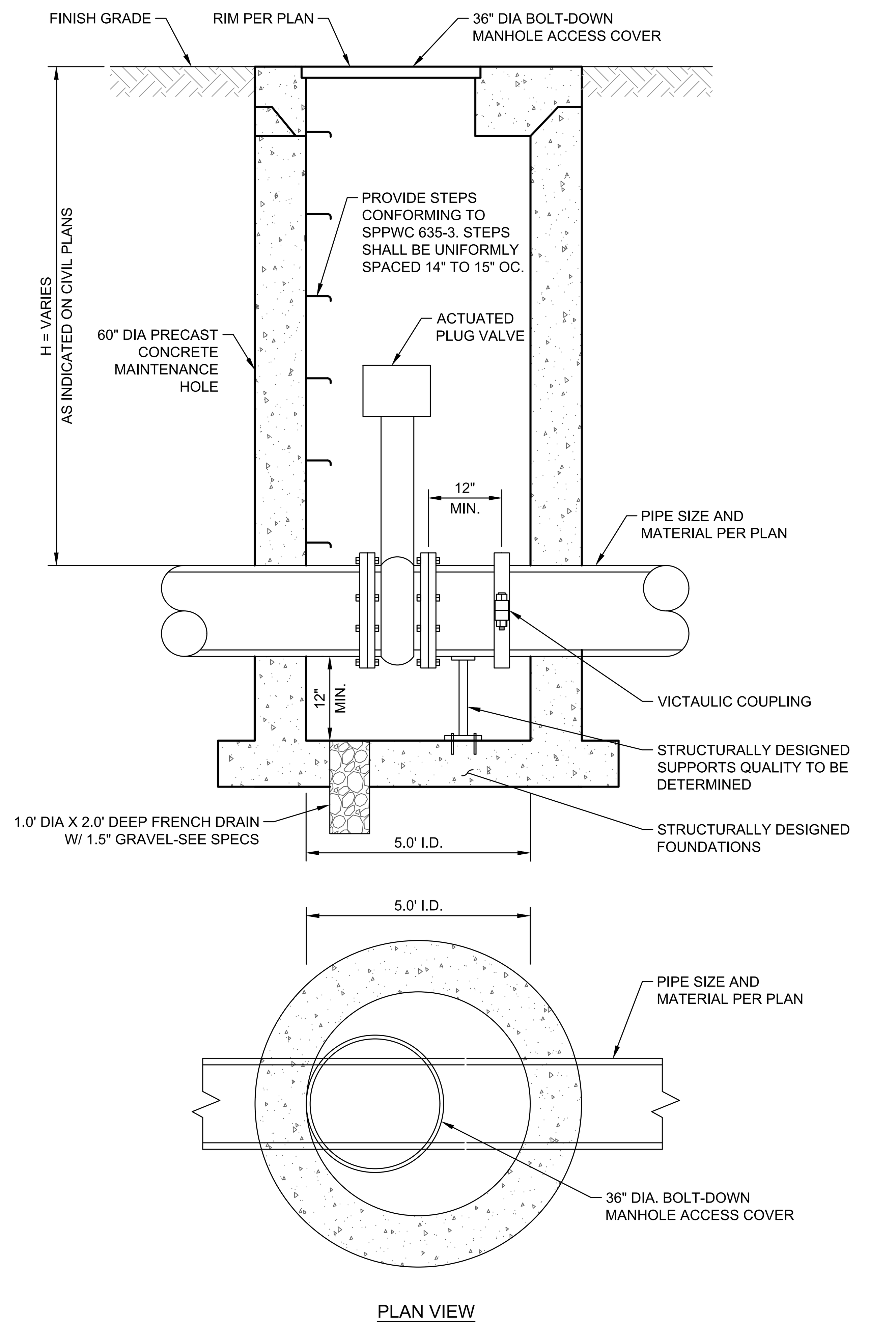
Bar Measures 1 inch
20% DESIGN 4/11/2018

4/10/2018 7:52:52 PM - P:\01297\200-01297-18011\CAD\SHEETFILES\C-03-PLAN&PROFILE.DWG - PETRICH, FORREST



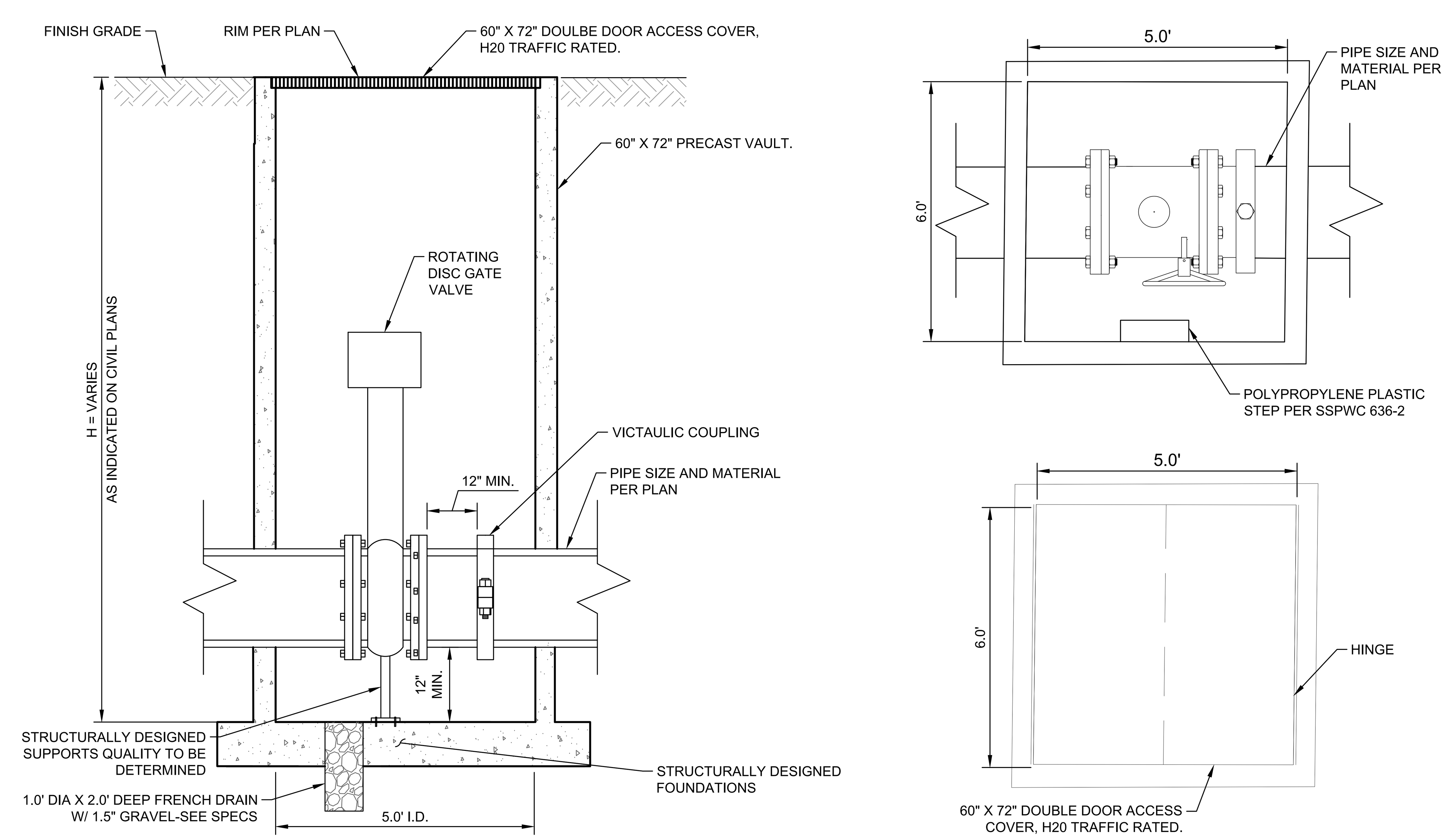
2 GRATED DROP INLET

NTS REFERENCE SHEETS C-01, C-03



1 ACTUATED VALVE STRUCTURE (≤12" DIA. PIPE)

NTS REFERENCE SHEETS C-03



3 ACTUATED VALVE STRUCTURE (>12" DIA. PIPE)

NTS REFERENCE SHEETS C-03

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TOWN OF ATHERTON

HOLBROOK-PALMER PARK WATER CAPTURE PROJECT

DETAILS

150 WATKINS AVE, ATHERTON, CA

Project No.:	200-01297-18011
Designed By:	JF
Drawn By:	AR
Checked By:	AY

C-04

Bar Measures 1 inch

20% DESIGN 4/11/2018

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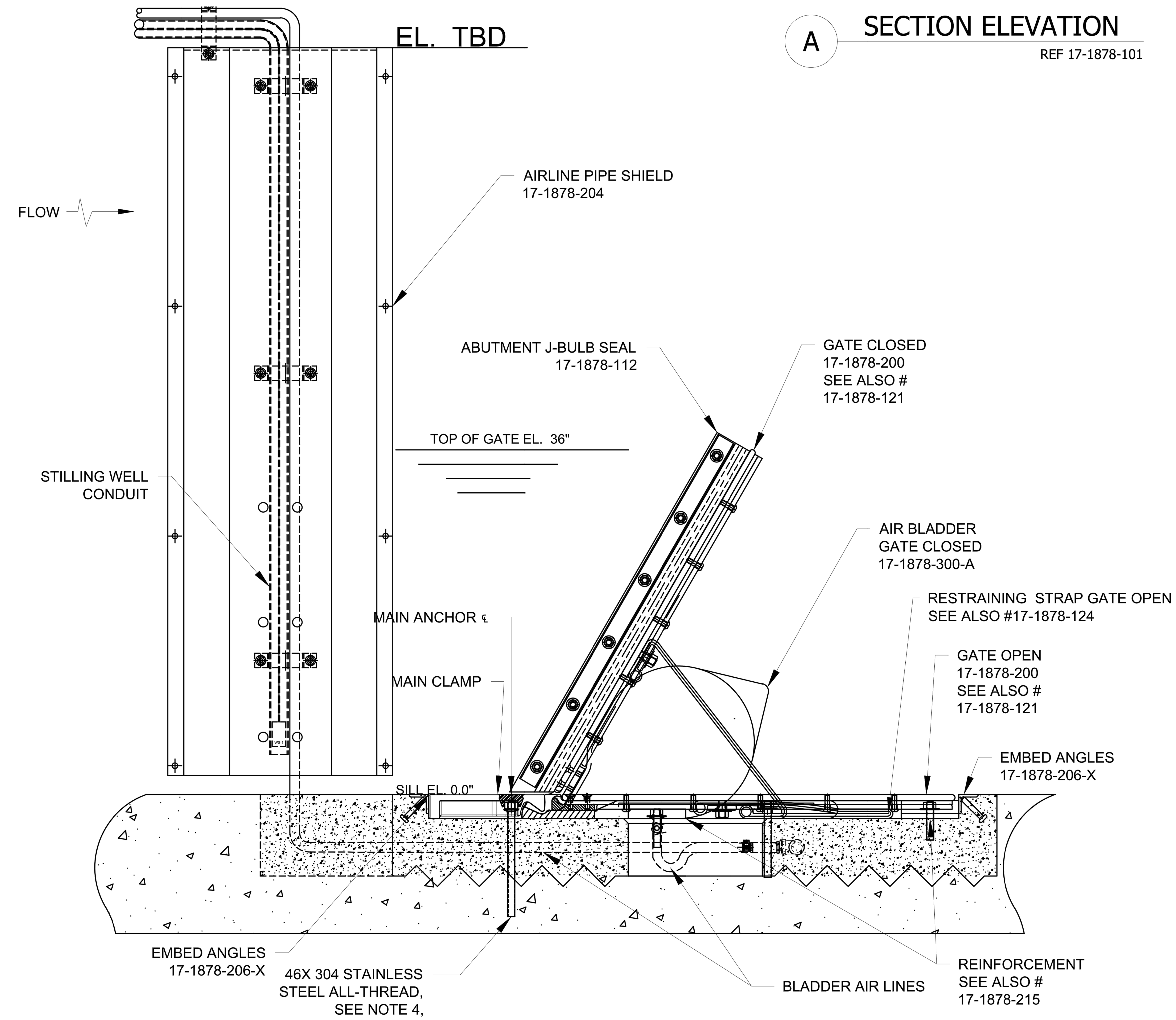
SECTION ELEVATION

A — REF 17-1878-101



NOTES:

1) CONCRETE FINISH: CONCRETE UNDER GATE SYSTEM **MUST** HAVE HIGH QUALITY TROWEL FINISH AND BE FLAT WITHIN ±1/8". A BROOM FINISH IS **NOT** ACCEPTABLE AND WILL RESULT IN ACCELERATED WEAR OF AIR BLADDERS.

2) THE HORIZONTAL CONCRETE CLAMPING SURFACE EXTENDING 6" DOWNSTREAM OF MAIN ANCHOR BOLT CENTERLINE AND 6" UPSTREAM OF MAIN ANCHOR BOLT CENTERLINE SHOULD BE LEVEL WITHIN ±1/4" FROM ABUTMENT TO ABUTMENT AND SHOULD NEVER VARY MORE THAN 1/8" PER FOOT.

3) ALL DIMENSIONS TO WITHIN ±1/8" OF TRUE POSITION UNLESS OTHERWISE DIMENSIONED OR NOTED ON PART DRAWING. SPACING OF ADJACENT MAIN ANCHOR BOLTS SHOULD NOT VARY MORE THAN 1/16" FROM NOMINAL SPACING.

4) ALL MAIN ANCHOR BOLTS TO BE PLUMB WITHIN ±1/8". MAIN ANCHOR BOLT CENTER LINE TO BE PERPENDICULAR TO ABUTMENT FACE ±0.1°.

5) DURING CONCRETE PLACEMENT, THE FORM WORK SUPPORTING THE ABUTMENT PLATES MUST BE HELD PLANAR, PLUMB AND PERPENDICULAR TO THE MAIN ANCHOR BOLTS WITHIN 1/8" PER FOOT AND WITHIN 1/4" OVERALL. RIGID STEEL BRACING OF FORMS IS RECOMMENDED. CONCRETE LIFTS SHOULD BE LIMITED SO AS TO NOT EXCEED ABOVE DEFLECTION LIMITS OF FORMS.

6) CONTRACTOR RESPONSIBLE FOR ALL CONCRETE CUTTING, GROUTING, GROUT, AIR PIPE, AIR PIPE FITTINGS, WELDING, VALVING, ELECTRICAL CONDUIT, CONDUIT FITTINGS, ASSOCIATED TOOLS, REBAR, ANCHORING AND SURFACE EPOXY, AND ANYTHING ELSE PERTAINING TO COMPONENTS NOT SUPPLIED BY OBERMEYER HYDRO, INC. OR INCLUDED IN THE OBERMEYER BILL OF MATERIALS.

7) THIS DOCUMENT AND ATTACHED DRAWINGS ARE PROPERTY OF OBERMEYER HYDRO, INC. AND IS NOT TO BE USED, DISCLOSED, REPRODUCED OR TRANSMITTED BY ELECTRONIC OR ANY OTHER MEANS WITHOUT PRIOR WRITTEN AUTHORIZATION.

8) ALL ANCHOR BOLTS, BOLTS, FASTENERS TO BE ACCURATELY TORQUED TO APPROPRIATE LEVEL AS DEFINED IN THE OBERMEYER INSTALLATION PROCEDURE.

9) THERE IS **NO WELDING ALLOWED** ON THE MAIN ANCHOR BOLTS. WELDING TO HEAT TREATED ANCHOR BOLTS CAN CAUSE FRACTURE AND CATASTROPHIC SYSTEM FAILURE.

10) ALL DIMENSIONS IN INCHES UNLESS OTHERWISE NOTED.

NO	DRAWN	APRV'D	DATE APRV'D	REVISION DESCRIPTION
1	BAH		21 July 2017	NEW DRAWING
2	JDM		17 AUG 2017	FLUSH GATE DESIGN
3	JDM		25 AUG 2017	ADDED STILLING WELL
4				
5				
6				
7				

OBERMEYER ideas that hold water™ HYDRO, INC.

DRAWING DESCRIPTION: SECTION ELEVATION DRIVE OVER GATE

SHEETS: 1 of 1 — SCALE: NO SCALE

JOB: Mayfair Park

DWG. NO. 17-1878-110

FOR REFERENCE ONLY

NOTE:

1. DRAWING IS CURRENTLY BEING GENERATED BY OBERMEYER. THIS DRAWING WILL BE REPLACED ONCE THE CORRECT DRAWING IS RECEIVED.
2. DAM WILL BE PROGRAMMED TO AUTOMATICALLY DEFLATE WHEN THE DEPTH OF FLOW IN THE CHANNEL SURPASSES THE HEIGHT OF THE DAM.

1 — RUBBER DAM (CONCEPTUAL)

NTS REFERENCE SHEET C-01, C-03

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APPROVED BY: DATE DATE

MARK	DATE	DESCRIPTION	BY

TOWN OF ATHERTON
HOLBROOK-PALMER PARK
WATER CAPTURE PROJECT

DETAILS

150 WATKINS AVE, ATHERTON, CA

Project No.:	200-01297-18011
Designed By:	JF
Drawn By:	AR
Checked By:	AY

C-05

Bar Measures 1 inch

20% DESIGN 4/11/2018

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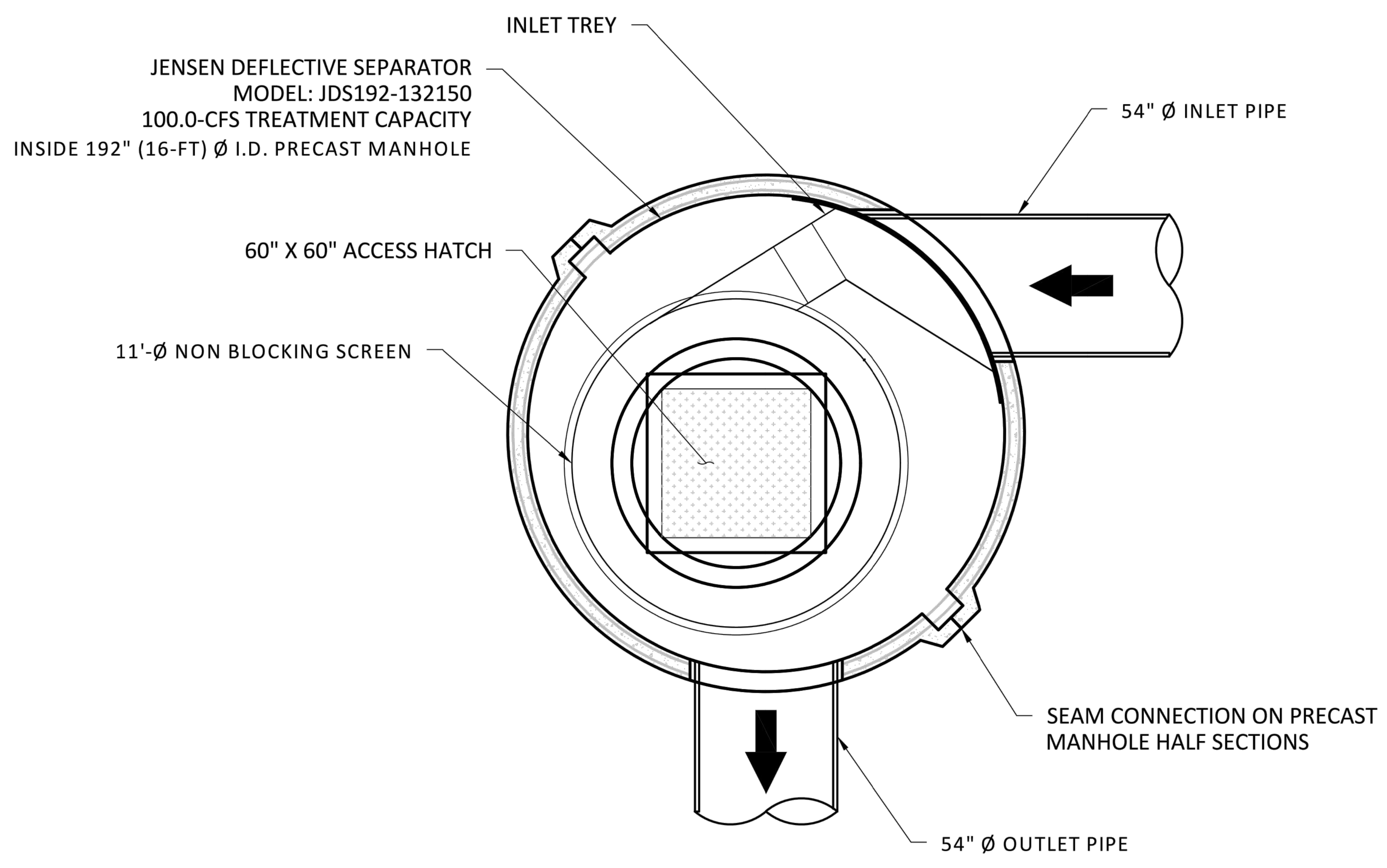
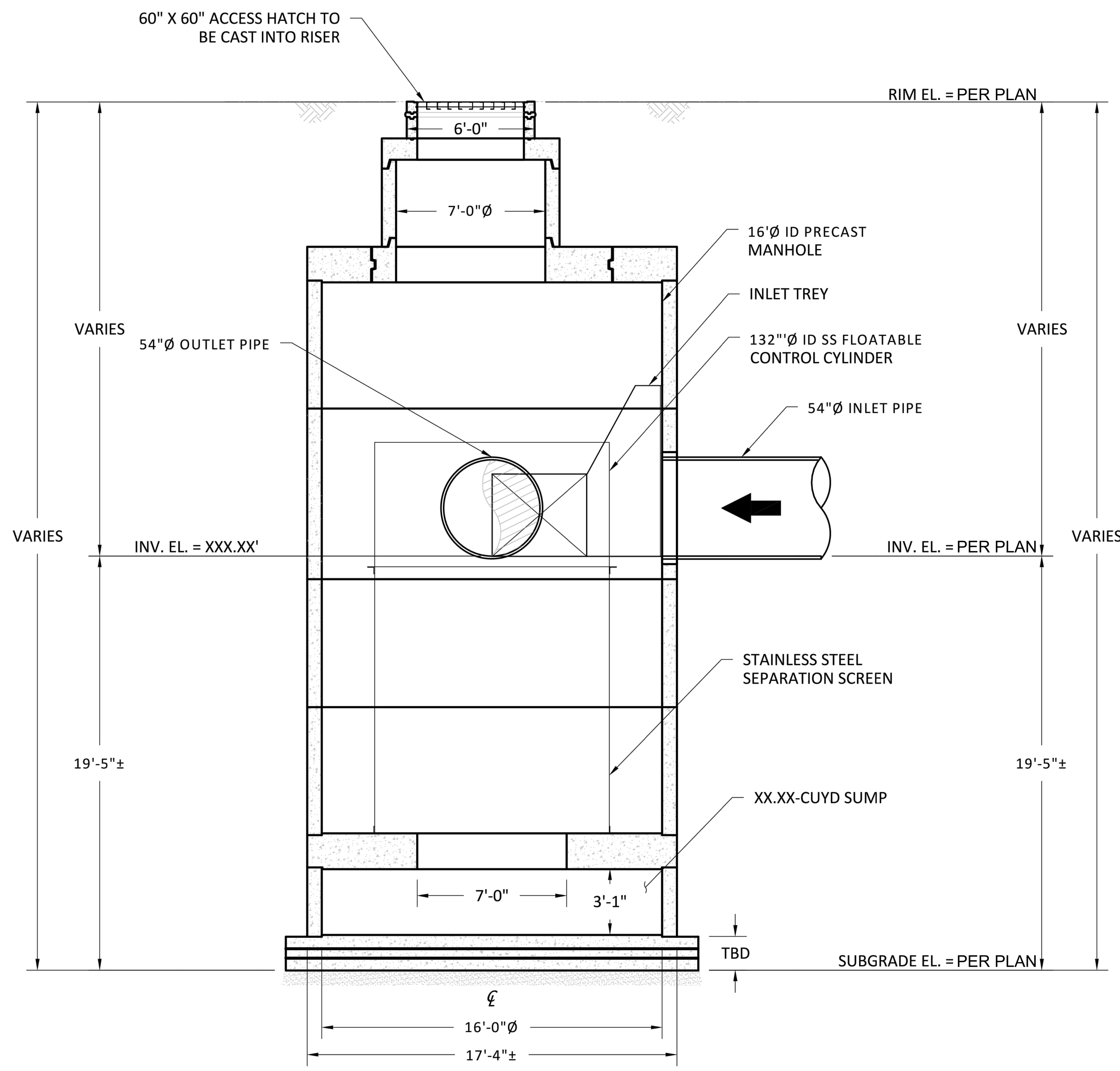


PLAN VIEW (MIRRORED)
SCALE= 1:100



ELEVATION VIEW
SCALE= 1:100

JENSEN DEFLECTIVE SEPARATOR
MODEL: JDS192-132150, 100.0-CFS CAPACITY

HYDRAULICS AND TREATMENT :

STORMWATER QUALITY DESIGN FLOW (SQDF)	100.0 - CFS
STORM DRAIN DESIGN CONVEYANCE FLOW	100.0 - CFS
RETURN FREQUENCY/PERIOD OF PEAK DESIGN CONVEYANCE FLOW	10 - YRS
SUGGEST MAXIMUM INTERNAL BYPASS FLOW	100.0 - CFS

STORMWATER TREATMENT UNIT EMPLOYS THE CONTINUOUS DEFLECTIVE SEPARATION TREATMENT PROCESS TO PRODUCE A NON-BLOCKING SCREENING SYSTEM WITH SWIRL CONCENTRATION.

TREATMENT :

NON-BLOCKING SCREENING AND SWIRL CONCENTRATING TREATMENT CAPACITIES.

1. 100% CAPTURE FOR TRASH & DEBRIS.
2. JENSEN DEFLECTIVE SEPARATOR IS A NON-BLOCKING SCREENING UNIT.
3. JENSEN DEFLECTIVE SEPARATOR DESIGNED TO PROCESS ENTIRE SQDF.
4. CAPTURE OF OTHER WATER QUALITY CONSTITUENTS INCLUDE:
   - TOTAL SUSPENDED SOLIDS (TSS)
   - HEAVY METALS
   - OIL & GREASE
   - NUTRIENTS
   - FECAL COLIFORM

FLOW SPLITTING NOTES :

1. FINAL FLOW SPLITTING STRUCTURE/VAULT DIMENSIONS TO BE DETERMINED (TBD) FROM ENGINEERING, HYDRAULIC AND ENERGY GRADELINE ANALYSIS OF THE PRETREATMENT AND BYPASS FLOW. THE DIMENSIONS SHOWN SHOULD BE CONSIDERED PRELIMINARY UNTIL COMPLETION OF HGL AND EGL ANALYSIS.

CONSTRUCTION NOTES :

1. CONTRACTOR TO VERIFY VERTICAL DIMENSIONS OF ALL PRECAST PIECES IN FIELD.
2. VERIFY SUBBASE ELEVATION BEFORE PLACING PRECAST COMPONENTS OR BACKFILLING.
3. APPLY BUTYL MASTIC AND/OR GROUT TO SEAL JOINTS OF MANHOLE STRUCTURE.
4. APPLY LOAD TO MASTIC SEAL IN JOINTS OF MH SECTIONS TO COMPRESS SEALANT IF NECESSARY. UNIT MUST BE WATER TIGHT, HOLDING WATER UP TO FLOWLINE INVERT (MINIMUM).
5. CONTRACTOR TO GROUT SEAL DISCHARGE PIPE TO INTERNAL SCREEN SUPPORT PEDESTAL AND PENETRATION THROUGH MANHOLE WALL.
6. ALL INTERNAL COMPONENTS INSTALLED BY MANUFACTURER.
7. BLOCK AND/OR GROUT PACK BENEATH FRAMES AND COVERS TO MATCH FINISHED GRADE.

MATERIALS & MANUFACTURING :

1. ALL DIMENSIONS ARE IN DECIMAL INCHES.
2. STRUCTURES DESIGNED FOR H-20 TRAFFIC AND SOIL LOADING PER ASTM C890 & C915 AND ASSHTO LOADING METHODS.
3. PRECAST VAULT MATERIALS AND MANUFACTURING METHODS SHALL CONFORM TO ASTM C-857 & C-478.
4. PRECAST MANHOLE MATERIALS AND MANUFACTURING METHODS SHALL CONFORM TO ASTM C-478 AND LACSD S-A-206, WHEN IN LA COUNTY.
5. CONCRETE SHALL HAVE A MINIMUM COMPRESSIVE STRENGTH F'c = 5,000-psi AT 28-DAYS.
6. THE PORTLAND CEMENT USED IN THE PRECAST SECTION SHALL MEET THE REQUIREMENTS OF TYPE II/V HIGH SULFATE RESISTANT CEMENT IN ACCORDANCE WITH ASTM CLASS M C-150.

MATERIAL LIST - PROVIDED WITH UNIT :

QTY.	COMPONENT DESCRIPTION	RESPONSIBLE PROVIDER	RESPONSIBLE INSTALLER
1	SS INLET TRAY	JENSEN	CONTRACTOR
1	2,400/4,700-MICRON SEPARATION SCREEN	JENSEN	CONTRACTOR
1	SS FLOATABLE CONTROL CYLINDER	JENSEN	CONTRACTOR
1	60" x 60" ACCESS HATCH	JENSEN	CONTRACTOR

SHEET NUMBER
1

MODEL:	PROJECT:	
JDS192-132150 **JENSEN DEFLECTIVE SEPARATOR**	HOLBROOK-PALMER PARK, ATHERTON, CA	JENSEN STORMWATER SYSTEMS 521 DUNN CIRCLE, SPARKS, NV 89431-6312 (877) 649-0095 FAX (775) 440-2013 www.jensenengineeredsystems.com

NON-BLOCKING CONTINUOUS DEFLECTIVE SEPARATION SCREENING & SWIRL CONCENTRATION PROCESS	ORG. DWG. DATE	REV. DWG. DATE	SCALE	SHEET SIZE	DRAWN BY	SHEET
	04/05/2018	XX/XX/XXXX	AS SHOWN	11" x 17"	S.T.	JDS192-132150

FOR REFERENCE ONLY

1 TRASH / DEBRIS CAPTURE DEVICE
NTS REFERENCE SHEET C-01, C-03

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APPROVED BY: DATE DATE

MARK DATE DESCRIPTION BY

TOWN OF ATHERTON
HOLBROOK-PALMER PARK
WATER CAPTURE PROJECT
DETAILS
150 WATKINS AVE, ATHERTON, CA

Project No.: 200-01297-18011
Designed By: JF
Drawn By: AR
Checked By: AY

C-06

Bar Measures 1 inch
20% DESIGN 4/11/2018

LEGEND:

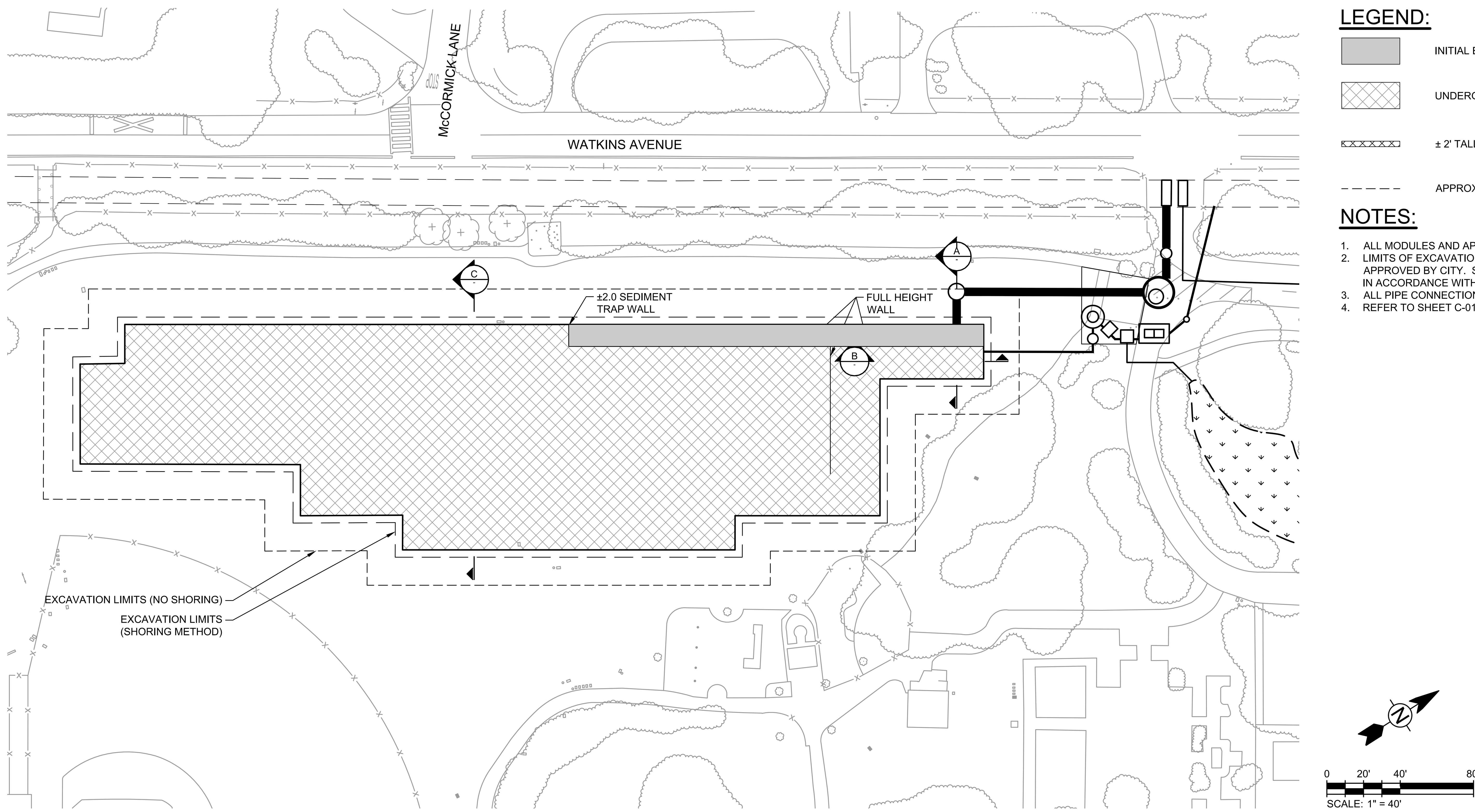
(shaded)	INITIAL ENERGY DISSIPATION/CONTAINMENT/SETTLEMENT AREA
(crosshatched)	UNDERGROUND STORAGE FACILITY
(xxxxxx)	± 2' TALL SEDIMENT TRAP WALL
(dashed)	APPROXIMATE RESERVOIR EXCAVATION LIMITS

RESERVOIR STATISTICS:

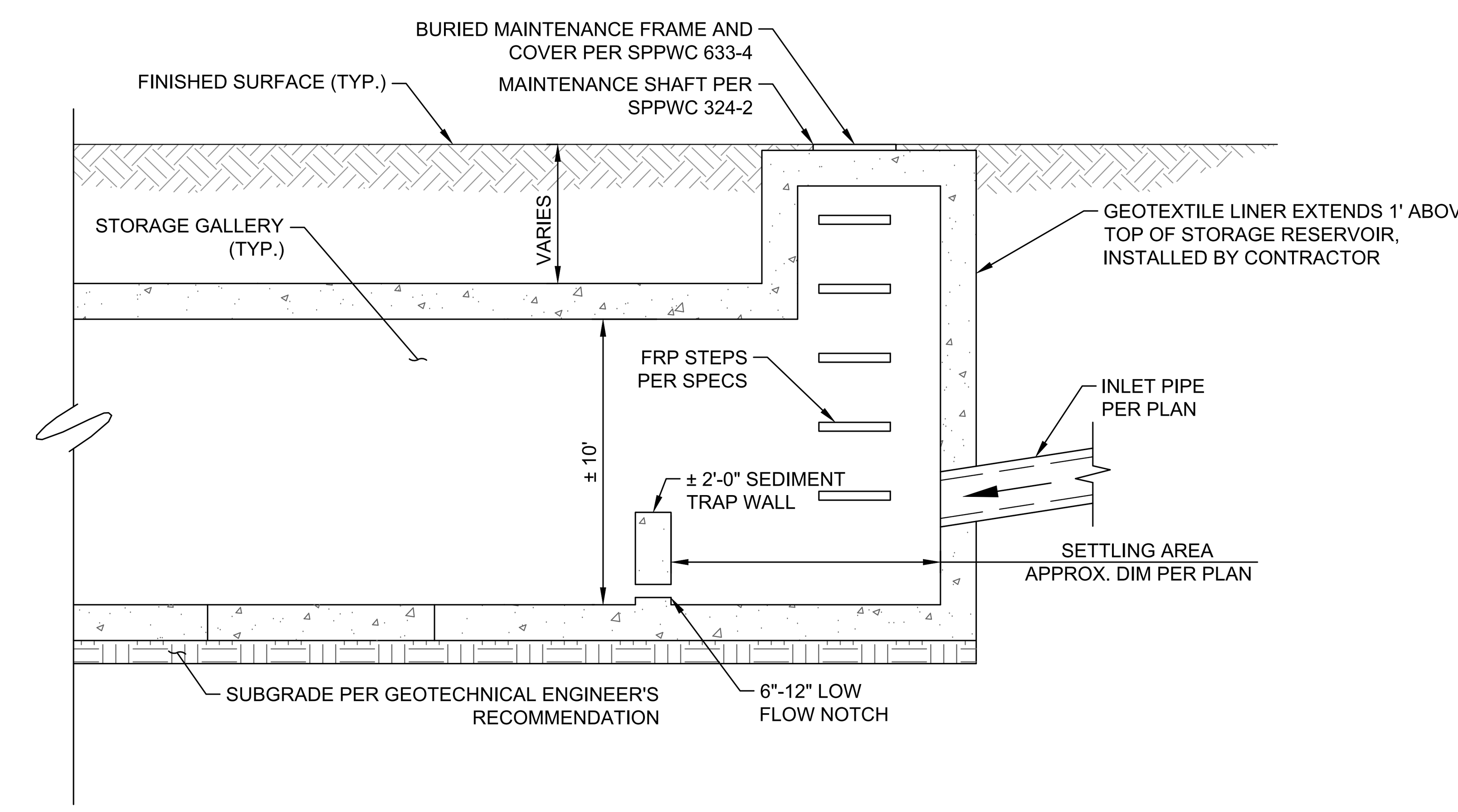
TOTAL AVAILABLE RESERVOIR AREA:	1.07 ACRE
TOTAL INITIAL AREA VOLUME:	8.94 ACRE-FEET

NOTES:

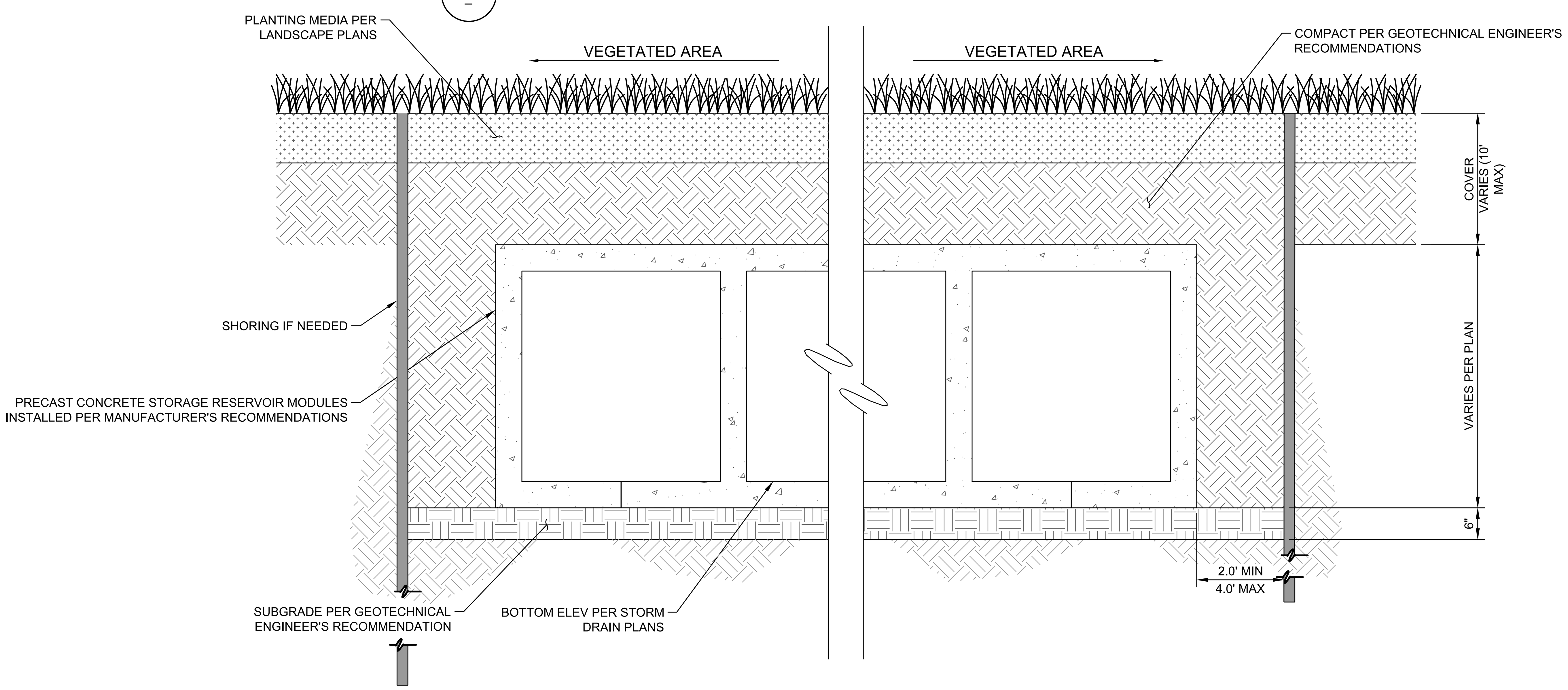
1. ALL MODULES AND APPURTENANCES SHALL BE INSTALLED PER MANUFACTURER'S RECOMMENDATIONS BY THE CONTRACTOR.
2. LIMITS OF EXCAVATION SHALL BE A 2' MINIMUM AND 4' MAXIMUM SETBACK FROM RESERVOIR LIMITS UNLESS OTHERWISE REQUIRED BY MANUFACTURER AND APPROVED BY CITY. SHORING LIMITS FOR ALL OTHER STRUCTURES OTHER THAN THE RESERVOIR SHALL BE DETERMINED AND DESIGNED BY THE CONTRACTOR IN ACCORDANCE WITH THE SPECIFICATIONS. SHORING SHALL NOT EXTEND PAST PROPERTY LINES OR INTO PUBLIC RIGHT-OF-WAY.
3. ALL PIPE CONNECTION WILL NEED TO BE EXTENDED/TRIMMED AS NEEDED.
4. REFER TO SHEET C-01 FOR DETAILED COORDINATES FOR STORAGE LIMITS.



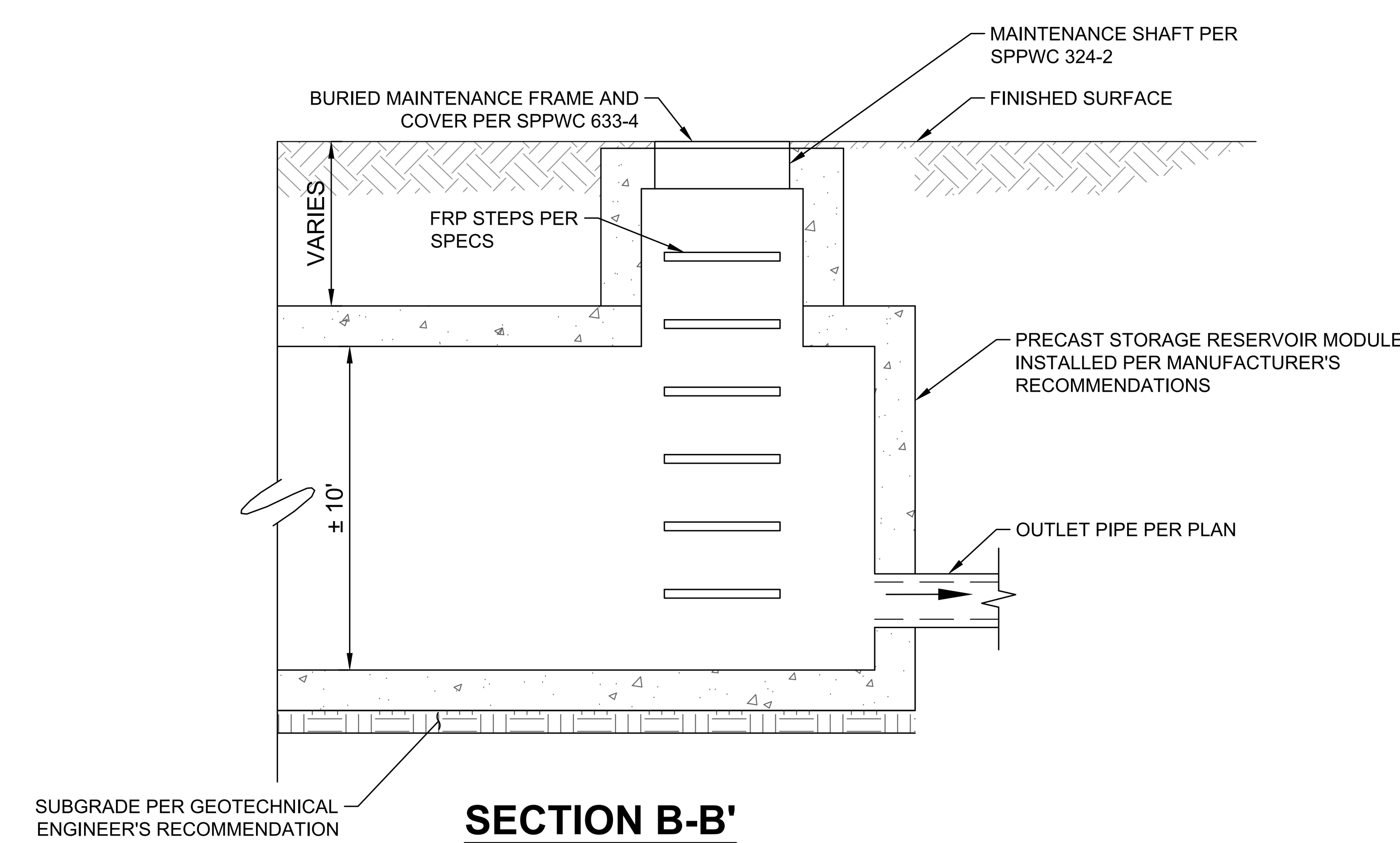
1 STORAGE RESERVOIR



SECTION A-A'
N.T.S.



SECTION C-C'
N.T.S.



SECTION B-B'
N.T.S.

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APPROVED BY: DATE DATE

MARK	DATE	DESCRIPTION	BY

TOWN OF ATHERTON
HOLBROOK-PALMER PARK
WATER CAPTURE PROJECT

DETAILS
150 WATKINS AVE, ATHERTON, CA

Project No.:	200-01297-18011
Designed By:	JF
Drawn By:	AR
Checked By:	AY

C-07

Bar Measures 1 inch
20% DESIGN 4/11/2018

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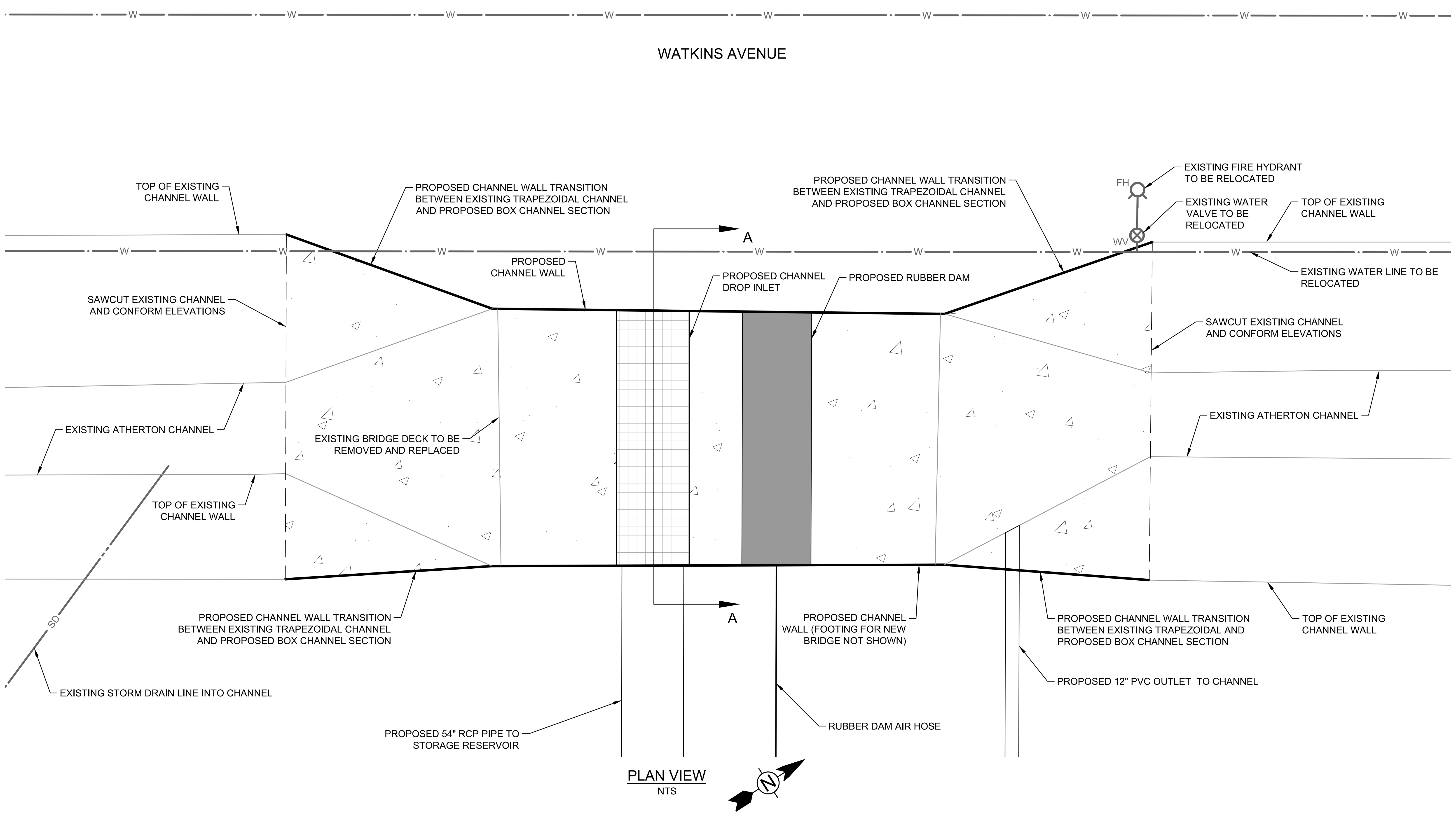
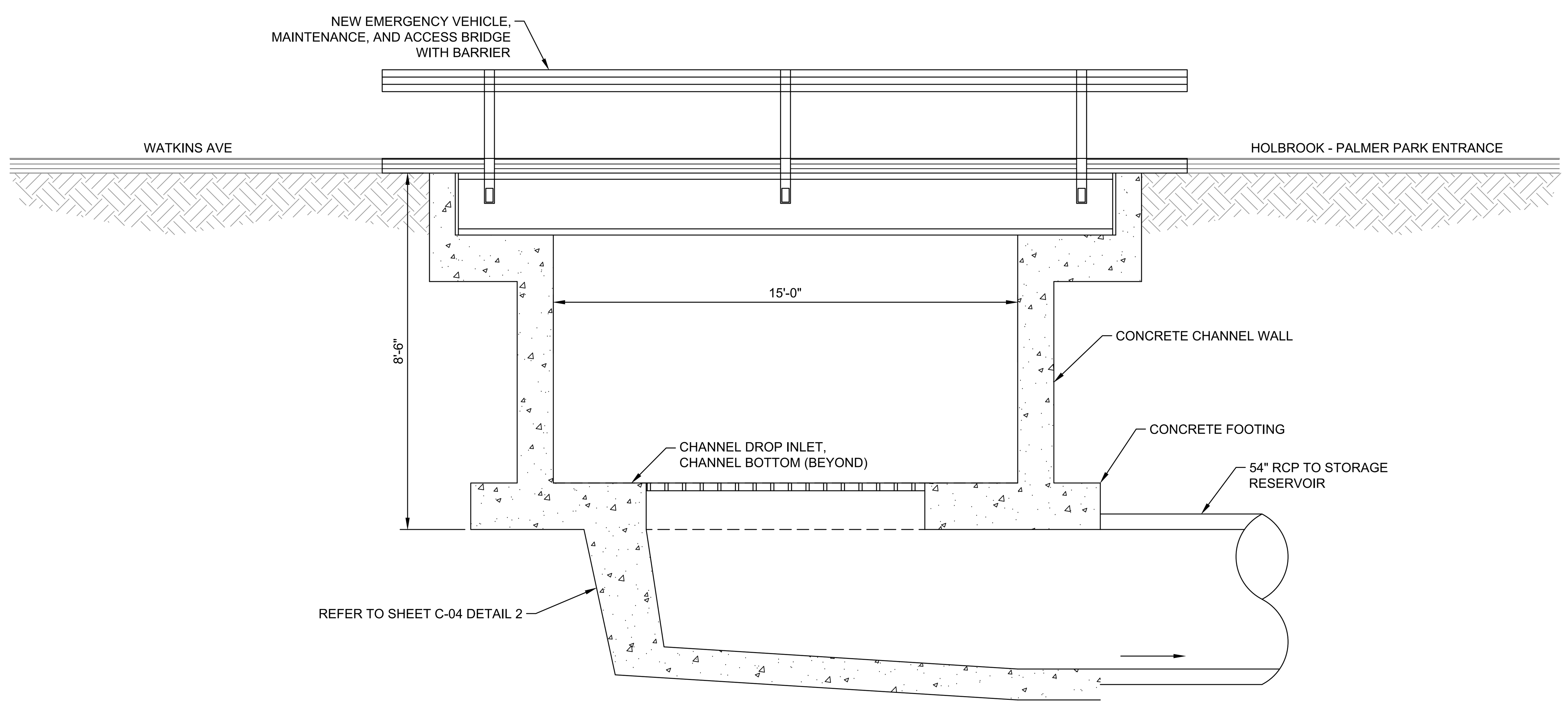


PLAN VIEW
NTS



SECTION A-A
NTS

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MARK	DATE	DESCRIPTION	BY

TOWN OF ATHERTON
HOLBROOK-PALMER PARK WATER CAPTURE PROJECT

DETAILS
150 WATKINS AVE, ATHERTON, CA

Project No.:	200-01297-18011
Designed By:	JF
Drawn By:	AR
Checked By:	AY

C-08

Bar Measures 1 inch
20% DESIGN 4/11/2018

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