GEOTECHNICAL ENGINEERING STUDY

TOMATO STAND FISH PASSAGE PROJECT
PINOLE CREEK WATERSHED CULVERT REPLACEMENT
CONTRA COSTA COUNTY, CALIFORNIA 94553
ATLAS PROJECT NO. 91-68538-PW

PREPARED FOR:

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March 4, 2025

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RESTORATION DESIGN GROUP INC.
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Subject: **Geotechnical Engineering Study**
Tomato Stand Fish Passage Project
Pinole Creek Watershed Culvert Replacement
Contra Costa County, California 94553

Dear Mr. Walkling:

Per our approved proposal, **Atlas Technical Consultants (Atlas)** has completed this Geotechnical Engineering Study for the proposed Tomato Stand Fish Passage project in Martinez, California. Transmitted herewith are the results of our findings, conclusions, and recommendations for the design and construction of proposed foundations, site grading and drainage, and temporary trench slope stability. In general, the proposed improvements at the site are considered feasible from a geotechnical standpoint provided the recommendations of this report are implemented in the design and construction of the project.

Should you or members of the design team have questions or need additional information, please contact either of the undersigned at manuel.zea@oneatlas.com. We greatly appreciate the opportunity to be of service to you, and to be involved in the design of this project.

Respectfully submitted,
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GEOTECHNICAL ENGINEERING STUDY

1. INTRODUCTION

1.1 Purpose and Scope

The purposes of this study were to evaluate the subsurface conditions at the site and prepare geotechnical recommendations for the proposed project. This study provides geotechnical recommendations for the foundation design of the proposed fish passage structure. The study was performed in accordance with the scope of work outlined in our approved proposal dated February 2, 2024.

The scope of this study included field exploration, laboratory testing of selected samples, engineering analysis, and preparation of this report. The conclusions and recommendations presented in this report are based on the data acquired during this study, and on prudent engineering judgment, and experience. This study did not include an assessment of potentially toxic or hazardous materials that may be present on or beneath the site.

1.2 Site Description

The Pinole Creek Tomato Stand Fish Passage project site is located on the fire road approximately 50 feet from Alhambra Valley Road in Martinez, California, at the locations shown on Plate 1.

The existing creek crossing consists of a 30-foot long by 6-foot diameter corrugated metal culvert. The surface of the road crossing the culvert is covered with asphalt concrete pavement that appears to be very old and deteriorated. Based on the topographic plan prepared by Bellecci Surveying, we understand that the road elevation across the culvert is about +301 feet, and the bottom of the creek is about +292 feet on the upstream side and about +283 feet on the downstream side of the culvert. The average geographic coordinates of the project site were 37.9698 degrees north latitude and 122.2156 degrees west longitude.

1.3 Proposed Development

The new crossing will consist of a prefabricated, single-span steel bridge supported on prefabricated steel and concrete abutments. The proposed bridge is about 52 feet long, 16 feet wide, and has deck depth of about 3 feet. The project will also involve channel improvements to improve fish passage by removing the existing culvert and the fill material within the proposed bridge footprint, realign the channel bed to eliminate the scour pool on the culvert downstream side, rock protection on the 2:1 slopes of the bridge abutments, and pavement for access roadway from the Alhambra Valley Road to the new crossing structure. The water surface in the improved Pinole Creek channel is estimated to be about 2 feet above the invert elevation.

1.4 Validity of Report

This report is valid for three years after publication. If construction begins after this time, Atlas should be contacted to confirm that the site conditions have not changed significantly. If the proposed development or structural design differs considerably from that described above, Atlas should be notified to determine if additional recommendations are required. Additionally, if Atlas is not involved during the geotechnical aspects of construction, this report may become wholly or in part invalid, since Atlas' geotechnical personnel need to confirm that the subsurface conditions anticipated when preparing this report are consistent with the subsurface conditions revealed during construction. Atlas' subsequent involvement should include foundation and grading plan review; and during project construction, observation of foundation excavations, and grading observation and testing.

2. PROCEDURES AND RESULTS

2.1 Literature Review

Pertinent geologic and geotechnical literature pertaining to the site area was reviewed. These included various publications and maps issued by the United States Geological Survey (USGS), California Geological Survey (CGS), online resources, and other applicable government and private publications and maps, as listed in the References section.

2.2 Field Exploration

In order to characterize the subsurface conditions beneath the proposed improvement area, a field exploration program was conducted, which consisted of drilling two test borings, designated as B-1 and B-2, on July 11, 2024. The borings were located to satisfy the project requirements and to facilitate an interpretation of the subsurface soil profile across the project site. The location of the borings relative to the proposed improvements are shown on Plate 2.

The borings were drilled to total depths of 30 feet below the existing ground surface using a truck mounted, B-24 drill rig equipped with 4-inch diameter solid flight augers. Following completion of drilling, the boreholes were backfilled using a cement grout mix and the upper 4-inches were repaired with asphalt patch where applicable.

An Atlas representative visually classified the materials encountered in the borings according to the Unified Soil Classification System. Relatively undisturbed soil samples were recovered at selected intervals using a 3-inch outside diameter Modified California split spoon sampler containing 6-inch-long brass liners, and a 2-inch outside diameter Standard Penetration Test (SPT) sampler. The samplers were driven by means of a 140-pound automatic trip hammer with an approximate 30-inch fall. Resistance to penetration was recorded as the number of hammer blows required to drive the sampler the final foot of an 18-inch drive. All of the field blow counts recorded using Modified California (MC) split spoon sampler were converted in the final logs to equivalent SPT blow counts using appropriate modification factors suggested by Burmister (1948), i.e., a factor of 0.65 with inner diameter of 2.5 inches. Therefore, all blow counts shown on the final boring logs are either directly measured (SPT sampler) or equivalent SPT (MC sampler) blow counts.

The boring logs with descriptions of the various materials encountered in each boring, a key to the boring symbols, and select laboratory test results are included in Appendix A. Ground surface elevations indicated on the soil boring logs were estimated to the nearest foot using the provided topographic plan prepared by Bellecci Surveying.

2.3 Laboratory Testing

Laboratory tests were performed on select samples to determine some of the physical and engineering properties of the subsurface soils. The results of the laboratory testing are presented in the boring logs and in Appendix B. The following soil tests were performed for this study:

<u>Dry Density and Moisture Content (ASTM D2216 and ASTM 2937)</u> – In-situ dry density and/or moisture tests were conducted on select samples to measure the in-place dry density and moisture content of the subsurface materials.

<u>Atterberg Limits (ASTM D4318 and CT204)</u> - Atterberg Limits tests were performed on select samples of cohesive soils encountered at the site.

Particle Size Analysis (Wet and Dry Sieve, ASTM D6913, D1140, and CT202) – Sieve analysis testing, including fines content measurements, was conducted on select samples to measure the soil particle size distribution and/or the total percentage of fines (i.e., percent passing the USCS No. 200 sieve).

3. GEOLOGIC AND SEISMIC OVERVIEW

3.1 Geologic Setting

The site is located in the central portion of the Coast Ranges geomorphic province of California. The Coast Ranges extend from the Transverse Ranges in southern California north to the Oregon border and are comprised of a northwest-trending series of mountain ranges and intervening valleys that reflect the overall structural grain of the province. The ranges consist of a variably thick veneer of Cenozoic volcanic and sedimentary deposits overlying a Mesozoic basement of sedimentary, metamorphic, and basic igneous Franciscan Formation and primarily marine sedimentary rocks of the Great Valley Sequence. East-dipping sedimentary rocks of the Coast Ranges are flanked on the east by sedimentary rocks of the Great Valley geomorphic province (CGS Note 36, 2002).

Locally, the site is located east of San Francisco Bay in the Briones Hills area on Costa Peak, within the western area of Briones Regional Park. The geologic structure of the East Bay is primarily controlled by strike-slip movement along the major northwest trending faults of the area. Similarly, the geologic units underlying the Pinole Creek watershed are faulted and folded in association with the movement along the Hayward, Pinole, Franklin Canyon, and other unnamed and active strike-slip fault systems. The rocks present in the watershed are primarily Miocene to Eocene sandstones, shales and volcanic tuffs (Pearce et al., 2005). According to Dibblee and Minch (2005) mapping shown on Plate 3, the site is in an area underlain by Holocene-epoch Surficial Sediments consisting of alluvial gravel, sand, and clay of valley areas.

3.2 Regional Faulting and Tectonics

The site is in a seismically active region. Major active faults include the Hayward Fault located approximately 5.5 miles southwest of the site; the Concord Fault, located approximately 8.5 miles northeast of the site; and the San Andreas Fault located approximately 25.7 miles southwest of the site. Local inactive faults identified by CGS including the Pinole and Frankling Faults are mapped as passing in close proximity to the site, but their specific locations are poorly defined, and currently not considered to potentially induce fault rupture. The site location relative to these and other active and potentially active faults in the southern San Francisco Bay Area is shown on Plate 4.

3.3 Historic Seismicity

The San Francisco Bay Area is subject to a high level of seismic activity. Within the period of 1800 to 2000 there were an estimated 20 earthquakes exceeding a Richter magnitude of 6.0 within a 100-mile radius of the site, seven exceeding 6.5, four exceeding 7.0 and one exceeding 7.5. There have been six major Bay Area earthquakes since 1800. Those were in 1836 and 1868 on the Hayward-Rodgers Creek Fault, in 1861 on the Calaveras Fault, and in 1838, 1906, and 1989 on the San Andreas Fault (CGS, 2024).

The site is reported to have experienced shaking from 57 earthquakes of magnitude 5.5 or greater during the period of 1800 to 2000, occurring at various distances away from the site. Of those, 17 were greater than Magnitude 6.0, seven exceeded 6.5, four exceeded 7.0 and one was greater than 7.5. The most significantly known ground shaking affecting the site since the 1868 Hayward earthquake is likely the 1906 San Francisco earthquake, as well as the 1989 Loma Prieta earthquake.

4. FIELD AND LABORATORY FINDINGS

Subsurface conditions below the project site were interpreted based on the results of the test borings performed for this study, as well as the results of our laboratory testing. Detailed descriptions of the various subsurface soil units encountered during subsurface explorations are described in the following paragraphs.

4.1 Subsurface Soil Conditions

During our subsurface exploration program, we investigated the subsurface soils and evaluated soil conditions to a maximum depth of about 30 feet below the existing ground surface as performed for this study. Based on our collected data, the area within the footprint of the proposed bridge structure is generally underlain by alluvial soils generally consisting of stiff to very stiff fat clay, medium dense silty sand with varying amounts of gravel, and very dense clayey gravel with sand to the maximum depth explored of 30 feet below the ground surface.

Atterberg Limit test results performed on three samples of near-surface soils with significant amount of fines content recovered from the upper 5 feet in the borings resulted in measured Liquid Limits (LL) of 59 and corresponding respective Plasticity Indices (PI) of 19 and 35, indicating a high plasticity and moderate to high expansion (shrink/swell) potential. Laboratory test summaries are presented in Appendix B.

4.2 Groundwater Conditions

Free groundwater was encountered in the borings at a depth of 13.5 feet below existing site grades. We note that the borings may not have been left open for a sufficient period of time to establish equilibrium groundwater conditions. Groundwater levels can vary in response to seasonal rainfall as well as time of year; well pumping, irrigation, and alterations to site drainage. A detailed investigation of local groundwater conditions was not performed and is beyond the scope of this study.

4.3 Generalized Ground Profile

A generalized ground profile was developed based on the subsurface conditions encountered in borings B-1 and B-2. The interpreted profile generally consists of an upper layer of medium to high plasticity clays, a lower layer of very dense sand and gravel, and a mixture of sand and clay in a transition layer between the upper and the lower layers. Table 1 summarizes the engineering parameters for the soil layers in the generalized ground profile.

Table 1: Engineering Parameters

Layer	Elevation (ft)		USCS	Description	Unit Weight (pcf)	Effective Shear Strength[1]		Undrained Shear Strength[1] (psf)
	From	To				Cohesion (psf)	Friction (°)	
1	301	288	CH/CL	Fat and lean clays	135	-	-	2500
2	288	281	CL/SM	Clay and silty sand	130	-	-	1500
3	281	240	SP/GP	Sand and Gravel	140	0	40	-

Notes:
[1] Strength parameter were estimated based on the Standard Penetration Test (SPT) N-values.

5. GEOLOGIC AND SEISMIC HAZARDS

5.1 Geologic Hazards

5.1.1 Fault Ground Rupture and Fault Creep

The State of California adopted the Alquist-Priolo (A-P) Earthquake Fault Zone Act of 1972 (Chapter 7.5, Division 2, Sections 2621 – 2630, California Public Resources Code), which regulates development near active faults for the purpose of preventing surface fault rupture hazards to structures for human occupancy. In accordance with the Alquist-Priolo (A-P) Act, the California Geological Survey established boundary zones or A-P "Earthquake Fault Zones" surrounding faults or fault segments judged sufficiently active, well defined and mapped for some distance. These zones generally extend at least 500 feet on each side of a mapped or inferred trace of an active fault. Structures for human occupancy within designated Earthquake Fault Zone boundaries are not permitted unless surface fault rupture and fault creep hazards are adequately addressed in a site-specific evaluation of the development site.

The site is not within a designated Earthquake Fault Zone as defined by the State (Hart and Bryant, 1997). The closest Earthquake Fault Zone is that of the Hayward Fault, which is located about 5.5 miles southwest of the site. Since the site is not within an Earthquake Fault Zone and no faults are known to be present that are within or toward the project site, the potential for fault ground rupture and surface manifestations from fault creep is judged to be minimal.

5.1.2 Landsliding

Landslides can occur along pre-existing zones of weakness within bedrock on over-steepened slopes, where siltstone beds, weak clay laminations, or clay beds are present on weak planes that dips unsupported out-of-slope. Landslide also can occur on anti-dip slopes along other planes of weakness, such as faults or joints. Factors such as undercutting by stream erosion, weakening of shallow slope materials by seepage, wetting and drying, over-loading and ground shaking are generally considered important contributors to slope instability and landsliding.

Based on the field reconnaissance and review of the available geologic maps, there are no known landslides mapped near or at the proposed structure site. Therefore, the potential for landsliding is considered low.

5.2 Seismic Hazards

Seismic hazards resulting from the effects of an earthquake generally include ground shaking, liquefaction, lateral spreading, dynamic settlement, and fault ground rupture and fault creep. The site is not necessarily impacted by all these potential seismic hazards. These potential seismic hazards are discussed and evaluated below.

5.1.1 Ground Shaking

The site will likely experience strong ground shaking from a major earthquake originating from a number of significant faults in the San Francisco Bay Area, including the Hayward, Concord, and San Andreas Faults. Earthquake intensities vary throughout the Bay Area depending upon the magnitude of the earthquake, the distance of the site from the causative fault, the type of materials underlying the site and other factors.

5.1.2 Liquefaction Induced Phenomena

Research and historical data indicate that soil liquefaction generally occurs in saturated, loose granular soil (primarily fine to medium-grained, clean sand deposits) during or after strong seismic ground shaking and is typified by a loss of shear strength in the affected soil layer, thereby causing the soil to flow as a liquid. However, as discussed, site soils generally consist of clays with some sand and gravel; and sands with high amounts of fines (i.e., more than 25% passing the No. 200 sieve).

The site is located within a zone delineated for liquefaction hazard by the California Geological Survey (CGS), as shown on Plate 5. However, due to the primarily fine-grained soil and high amounts of fines, and very dense soil conditions in the coarse-grained soils encountered during our exploration, the liquefaction hazard at the site is considered low.

5.1.3 Dynamic Compaction (Settlement)

Dynamic compaction is a phenomenon where loose, sandy soil located above the water table densified from vibratory loading, typically from seismic shaking or vibratory equipment. The site is generally underlain by lean clays. Based on the lack of loose, clean sands encountered in the borings, the site is not susceptible to dynamic densification.

5.1.4 Lateral Spreading

Lateral spreading involves both vertical and lateral ground movement, with some vertical component, occurring as a result of liquefaction. In addition to liquefaction, a free face or slope is necessary in most cases for lateral spreading to occur. Lateral spreading can occur on relatively flat sites with slopes less than 2 percent under certain circumstances and manifest itself at the ground surface in the form of cracking and settlement. Lateral spreading can occur in areas located within close proximity to an open face which are supported by underlying liquefiable soil under or close to the open face. Under a lateral spreading condition, soils which liquefy lose strength and the slope moves towards the open face. Any structures or improvements located within close proximity to the slope can also move and possibly be destabilized. As mentioned above, the project site is not susceptible to liquefaction, and therefore, not susceptible to lateral spreading.

5.3 Other Hazards

Potential geologic hazards other than those caused by a seismic event generally include consolidation settlement and expansive soils. These are discussed and evaluated as follows.

5.3.1 Consolidation Settlement

Consolidation occurs as a result of water being squeezed out from a saturated soil as internal pore water pressures induced by an external load are dissipated over time. As the water moves out from the soil, the solid particles re-align into a denser configuration causing settlement. Consolidation typically occurs as a result of the construction of a new structure or engineered fills, but consolidation can also occur from groundwater withdrawal. Consolidation of clayey soils is usually a long-term process, whereby the water is squeezed out of the soil matrix with time. Sandy soils consolidate relatively rapidly with the introduction of a load. Consolidation of soft and loose soil layers and lenses can cause settlement of the ground surface or structures. Potentially compressible cohesive layers were encountered at the site from the ground surface to depths of approximately 13.5 to 23.5 feet below existing site grades. Therefore, consolidation settlement may affect the proposed development if the relatively heavy structure is supported on shallow foundations near the top of the embankments.

5.3.2 Expansive Soils

Visual observation of select samples and laboratory tests conducted for this study indicated the near-surface soils are generally of high plasticity and high expansion potential. Expansive soils may impact the performance of foundations and site flatwork, as expansive soil pressures may develop that can manifest primarily as seasonal heaving and settlement effects. Mitigation of the effects of expansive soils should be considered in the foundation design as briefly discussed below.

6. CONCLUSIONS AND RECOMMENDATIONS

The following conclusions and recommendations are based upon the analysis of the information gathered during this study and our understanding of the proposed improvements.

6.1 Conclusions

The site is considered suitable from a geologic and geotechnical standpoint for the proposed improvements provided the recommendations of this report are incorporated into the project design and implemented during construction. The predominant geotechnical and geological issues that need to be addressed at this site are summarized below.

Seismic Ground Shaking – The site is located within a seismically active region, and subject to potentially very strong ground shaking during the life of the structure and should be designed to consider the effects of seismic activity.

Abutment Slopes – The foundation elements located at the top of the abutment slopes, if any, will need to be deepened to provide sufficient resistance to lateral loads and protection from possible erosion of the nearby creek slopes.

Scour Potential – The proposed structure will be crossing an active creek channel located within the FEMA special flood hazard area Zone A. Based on the preliminary scour analysis, a total scour of 3 feet were estimated for the abutments located outside the active streambed of the creek channel. Impacts of scour on the foundation capacities should be considered in accordance with Caltrans standard of practice.

Corrosion Potential – Corrosivity test results are not available. However, since the structure site is in close proximity to the creek channel, high corrosion potential should be considered in the design of structure elements. Therefore, concrete and steel should be designed to withstand potential corrosion impacts.

Expansive Soils – Near-surface soils at the site were found to be potentially highly expansive. Foundations placed on or within the active zone of expansive soils should be designed to resist differential volume changes and to prevent structural damage to the supported structure. To mitigate the effects of volume changes in the active zone in the near surface soils, a minimum of 4 feet below the bottom of the bridge abutments should be removed and replaced with structure backfill.

Groundwater – Groundwater was encountered at a depth of 13.5 feet during the field investigation. Therefore, except for unanticipated shallow seepage or locally perched groundwater zones, groundwater should not make significant impacts for near-surface construction.

6.2 Seismic Design Parameters

If applicable, the proposed project should be designed to resist the seismic forces generated by earthquake shaking in accordance with the AASHTO LRFD Bridge Design Specifications, 2020 and the requirements of Caltrans Seismic Design Criteria (Caltrans SDC, 2019). Caltrans current practice is to use the Safety Evaluation Earthquake (SEE) design acceleration response spectrum (ARS) developed per SDC v2.0 to characterize design ground motions for embankments, earth retaining structures, and slopes with a 5 percent probability of exceedance in 50 years (i.e. 975-years return period). The SEE design ARS is evaluated based on the United States Geological Survey's (USGS) 2014 National Seismic Hazard (2014 NSHM) data for the 975-year return period using the ARS Online v3.1 web tool (Caltrans, 2024) with adjustment factors due to the basin effects and/or near-fault effects per the SDC V2.0.

Based on the subsurface conditions encountered in the borings, evaluation of the site geology, and extrapolating the subsurface conditions to 100 feet, Soil Profile Type D based on the time-averaged shear wave velocity (V_{s30}) for the upper 100 feet is considered appropriate for characterizing potential earthquake ground shaking conditions and seismic design considerations. The geographic coordinates of the site were 37.9698 degrees north latitude and 122.2156 degrees west longitude. The recommended site parameters for seismic design are as follows:

- Time-average shear wave velocity V_{s30} of 850 feet per second for the design subsurface soil profile.
- Horizontal Peak Ground Acceleration (HPGA) of 0.74 g.
- De-aggregated mean earthquake moment magnitude M_w of 6.9 with site-to-source distance of about 9 kilometers for the HPGA.

According to Caltrans SDC, the standard ARS curve should be adjusted to account for the near-source rupture directivity effect since the proposed structure site is within 9.4 miles from the rupture plane of an active fault. As such, the adjustment factors are fully applied as follows:

- No change in spectral values for periods less than 0.5 second.
- 20 percent increase in spectral values for periods equal to or greater than 1.0 second.
- Linearly interpolate spectral values for periods between 0.5 and 1 second.

Table 2 presents the recommended SEE design ARS values for the structure periods up to 5 seconds adjusted with the near-fault amplification factors for the structure site.

Table 2: Recommended Design Acceleration Response Spectrum (ARS)

Period(s)	NSHM SA (g)	Adjustment Factors		Design SA (g)
		Basin	Near Fault	
PGA	0.74	1	1	0.74
0.1	1.24	1	1	1.24
0.2	1.67	1	1	1.67
0.3	1.86	1	1	1.86
0.5	1.77	1	1	1.77
0.75	1.44	1	1.1	1.58
1.0	1.17	1	1.2	1.41
2.0	0.62	1	1.2	0.74
3.0	0.39	1	1.2	0.47
4.0	0.27	1	1.2	0.33
5.0	0.20	1	1.2	0.24

6.3 Foundations

6.3.1 Design Criteria

The geotechnical criteria for the foundation design requires the geotechnical nominal resistance of the foundation to be at least two times the Load and Resistance Factor Design (LRFD) service-I loading.

6.3.2 Structure Backfill

The specification for the structure backfills material should conform to California Department of Transportation (Caltrans) Standard Specifications, Section 19-3.02C, in 2023 Edition.

6.3.3 Type Selection

Two design schemes for the creek crossing were considered viable options for the fish passage structure. One design considered the structure as a prefabricated bridge while the other considered bottomless arch culvert. In both designs, the existing corrugated metal culvert and the fill soils around it will be removed to provide a wider channel bed for the fish passage structure. The feasibility of the foundation types for the two design schemes were assessed in previous submittal (Atlas, 2024), and the driven steel H-piles was the recommended foundation type to support the abutments of the prefabricated bridge structure at this site.

6.3.4 Axial Resistance

The installation of the foundation for the abutments of the selected design scheme will require cuts and fills into the existing slopes of Pinole channel. To avoid disturbing the structure backfill placed to mitigate expansive soils, pre-drilled holes 16-inch in diameter and 4 feet deep are likely to be required, as such, the contribution to the foundation's axial resistance from the engineered fills are ignored. Using the effective stress methods applied to the clay and the sand layers (AASHTO, 2020) to compute the unit nominal side and tip resistances, and considering a resistance factor of 0.7, the estimated pile length to provide 90 kips (service load) is about 30 feet for H14x89 driven steel piles. Table 3 summarizes the axial resistance design data for the recommended pile type. The pile length and the drivability of the steel H-piles can be re-evaluated as more site-specific information becomes available.

Table 3: Foundation Design Data – Axial Resistance

Support Location	Pile Type	Cut-off Elevation[1] (ft)	Required Nominal Resistance (Strength Limit State[2]) (kips)		Tip Elevation (ft)
			Compression	Tension	
Abutment 1	H14x89	291	130	-	261
Abutment 2	H14x89	291	130	-	261

Notes:
[1] Assumed based on bridge deck thickness of 3 feet
[2] Resistance factor $\phi_{qs} = \phi_{qp} = 0.7$

6.3.5 Lateral Resistance

The lateral resistance of the H14x89 steel pile is computed using the beam-column model supported by nonlinear lateral springs (*p-y* curves). The experimentally developed *p-y* curves for clays (Reese1972, 1975) and for sand (Reese 1974) were utilized to compute the lateral resistance of a single pile at the specified deflections for a fixed pile head condition. Table 4 summarizes the recommended lateral pile resistance for free- and fixed-head pile condition to be selected based on the appropriate pile head connection detail with the abutment of the prefabricated bridge.

Table 4: Foundation Stiffness – Lateral Resistance

Parameter		Pile Head Condition	
		Free	Fixed
Shear Force (kips)	0.5-inch	53	95
	1.0-inch	69	107
	2.0-inch	77	107
Bending Moment[1] (kips-ft)	0.5-inch	143	358
	1.0-inch	216	429
	2.0-inch	289	429
Depth to Maximum Bending Moment [2] (ft)	0.5-inch	5.5	0
	1.0-inch	6.6	0
	2.0-inch	9.0	0

Notes:
[1] Maximum bending moment
[2] Depth below the pile cut-off elevation.

To reduce the lateral demand on piles, more than one row of pile at each abutment may be required. In addition, an average ultimate passive soil pressure of 7.7 kips per square foot can be fully mobilized for the structure backfill lateral resistance behind an 8-foot-high abutment wall at a displacement of one percent the abutment height and should be reduced linearly in proportion to the abutment wall height for other wall heights.

6.4 Additional Site Investigation

The soil borings drilled at the site provided very limited subsurface information and do not provide support-specific information and the soil resistance at the estimate pile tip elevation. Therefore, it is recommended to drill a total of two additional soil borings or cone penetration tests at the proposed channel crossing to a depth of about 60 feet below the existing ground surface. The exact locations of the proposed borings should be selected within the footprint of the bridge abutments. Further, additional soil samples should be tested for corrosivity.

7. LIMITATIONS AND UNIFORMITY OF CONDITIONS

The recommendations of this report are based upon the soil and conditions encountered in the borings drilled at the project site. This report is issued with the understanding that it is the responsibility of the owner or his representatives to see that the information and recommendations contained herein are called to the attention of the other members of the design team and incorporated into the plans and specifications.

The findings and recommendations presented in this report are preliminary and valid as of the present time for the development as currently proposed. However, changes in the conditions of the property or adjacent properties may occur with the passage of time. In addition, changes in applicable or appropriate standards may occur through legislation or the broadening of knowledge. Accordingly, the findings and recommendations presented in this report may be invalidated, wholly or in part, by changes outside our control. In addition, if the currently proposed design scheme as noted in this report is altered, Atlas should be provided the opportunity to review the changed design and provide supplemental recommendations as needed.

This report was prepared upon your request for our services, and in accordance with currently accepted geotechnical engineering practice. No warranty based on the contents of this report is intended, and none shall be inferred from the statements or opinions expressed herein.

The scope of our services for this report did not include an environmental assessment or investigation for the presence or absence of wetlands or hazardous or toxic materials in the soil, surface water, groundwater or air, on, below or around this site. Any statements within this report or on the attached Plates, logs or records regarding odors noted or other items or conditions observed are for the information of our client only.

8. REFERENCES

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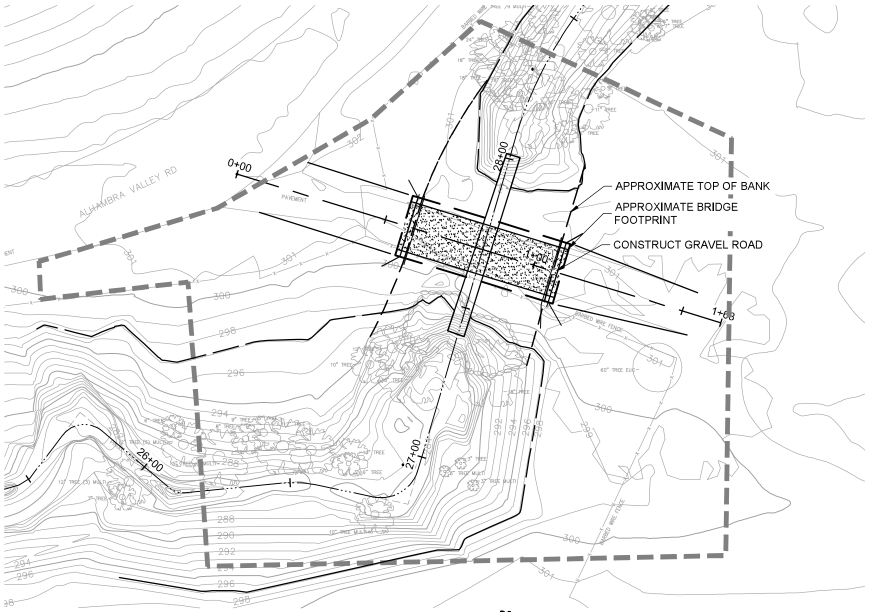
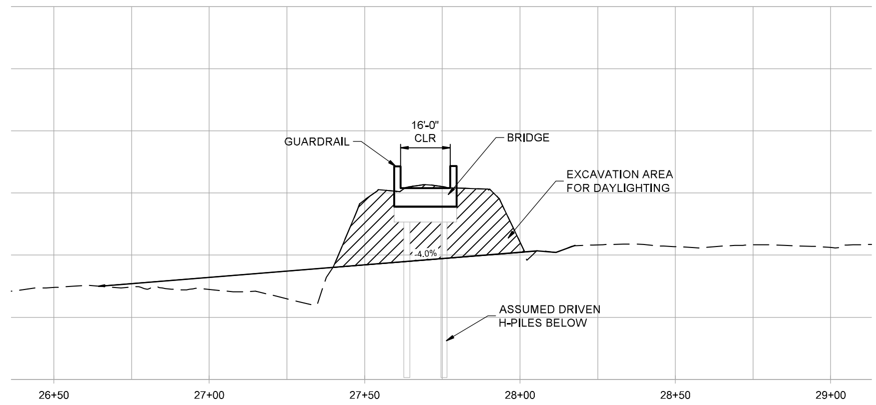
Publications may have been used as general reference and not specifically cited in the report text.

FIGURES

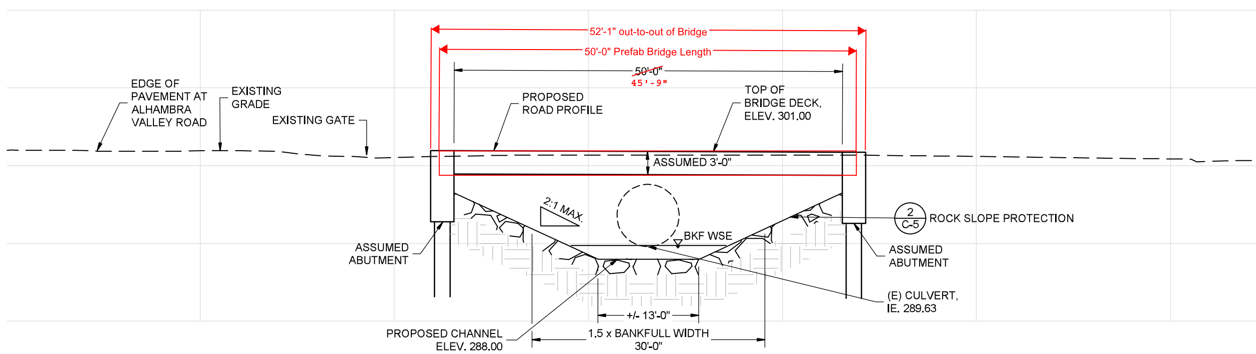
Figure 1 - Concept Design Section



PLAN
1" = 20'



PROFILE OF PINOLE CREEK AT NEW CROSSING
1" = 20' HORIZ.
1" = 10' VERT



PROFILE OF NEW CROSSING (1)
SCALE: 1/8" = 1'-0"

Figure 1
Concept Design Section

NO.	DATE	DESCRIPTION	APPR

REVISIONS

TOMATO STAND FISH PASSAGE

PROFILES

65% DESIGN

RDG
Restoration Design Group, Inc.
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Berkeley, CA 94710
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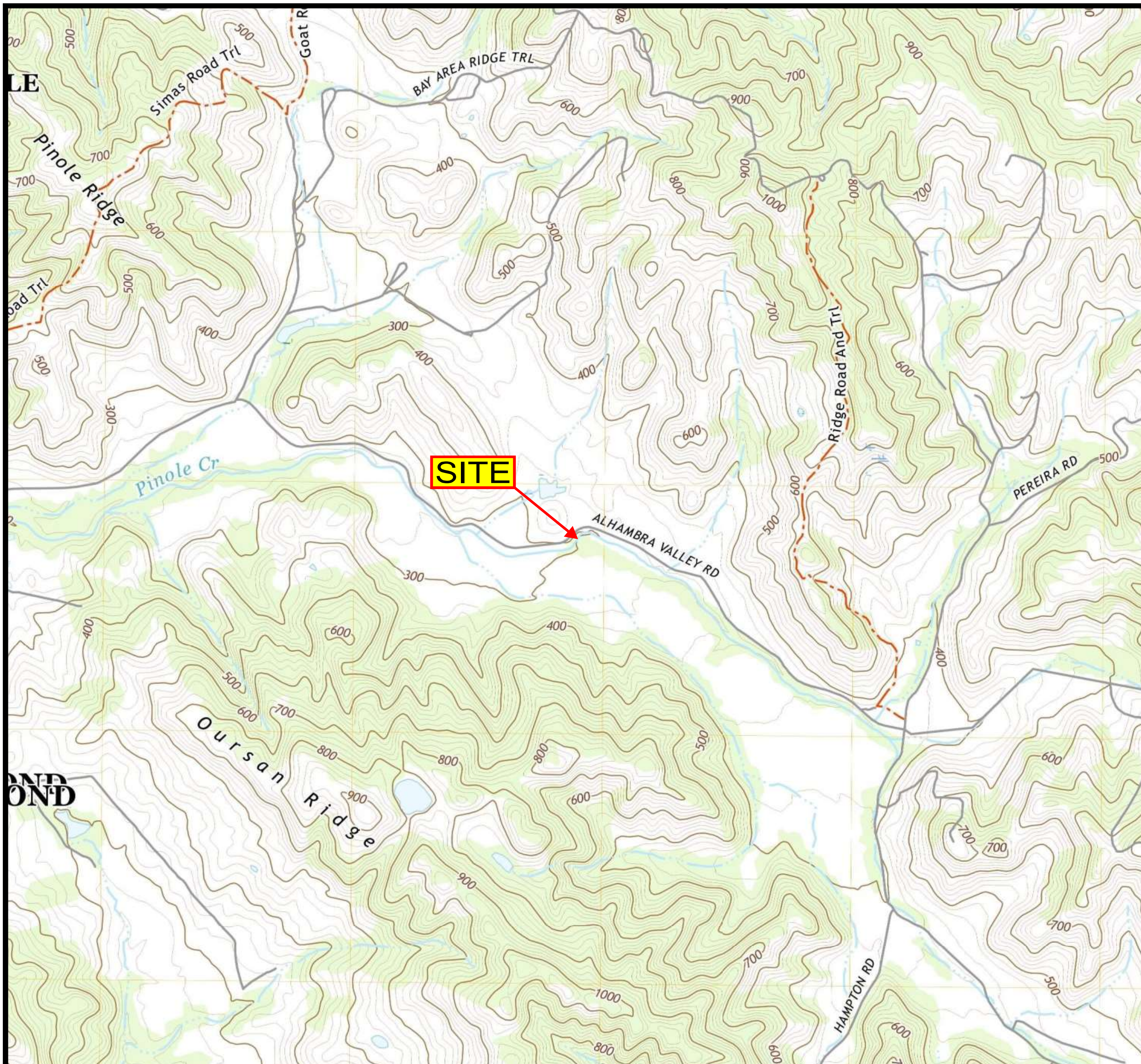
www.restorationdesigngroup.com

LICENSED PROFESSIONAL ENGINEER
MATTHEW J THOMAS
C 79249
CIVIL
STATE OF CALIFORNIA

DESIGN BY	MT, ES
DRAWN BY	JH, HH
CHECKED BY	MT, ES
SCALE	AS NOTED
DATE	NOVEMBER 8, 2024
SHEET	C-3
	OF XX

C:\USERS\HANSEN\RESTORATION DESIGN GROUP\PROJECTS - DOCUMENTS\TOMATO STAND FISH PASSAGE\04 AUTOCAD\WORKING DRAWINGS\SHEETS\TOM-L-S-PLAN-PROFILE.DWG PLOTTED ON: 11/1/2024

PLATES



N

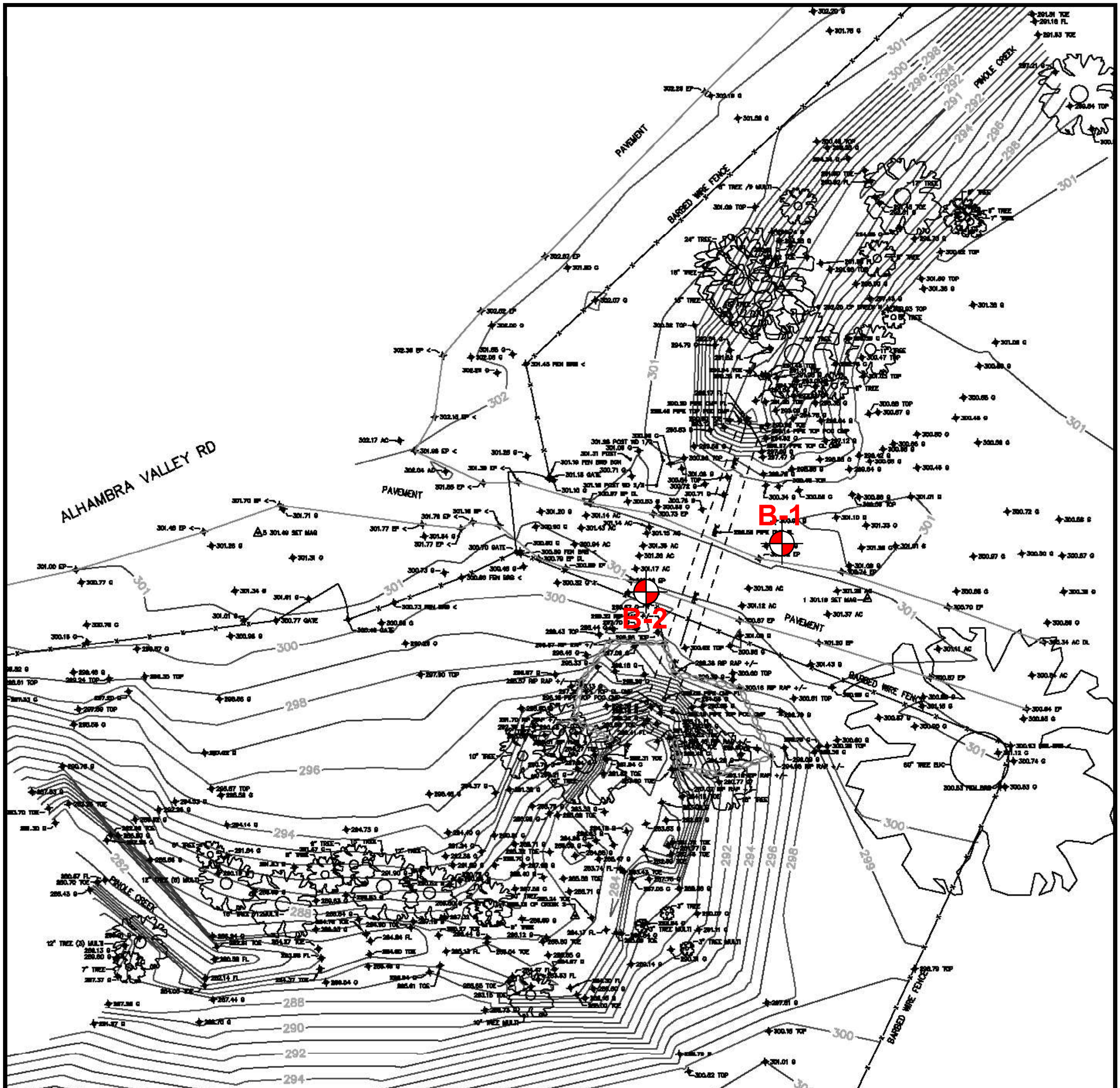
2000 ft.

Scale

Contour Interval 20 ft.

Reference: USGS Briones Valley Quadrangle California 7.5-Minute Series (2021)

ATLAS	Job No.: 91-68538-PW Approved: MZ Date: 08.07.2024	**VICINITY MAP** Tomato Stand Fish Passage Pinole Creek Watershed Culvert Replacement Contra Costa County, California 94553	**Plate** 1



B-1 - Approximate Boring Location

N

30 ft.
Scale

Topo Plan: Bellecci & Associates Inc. (07/09/2024)

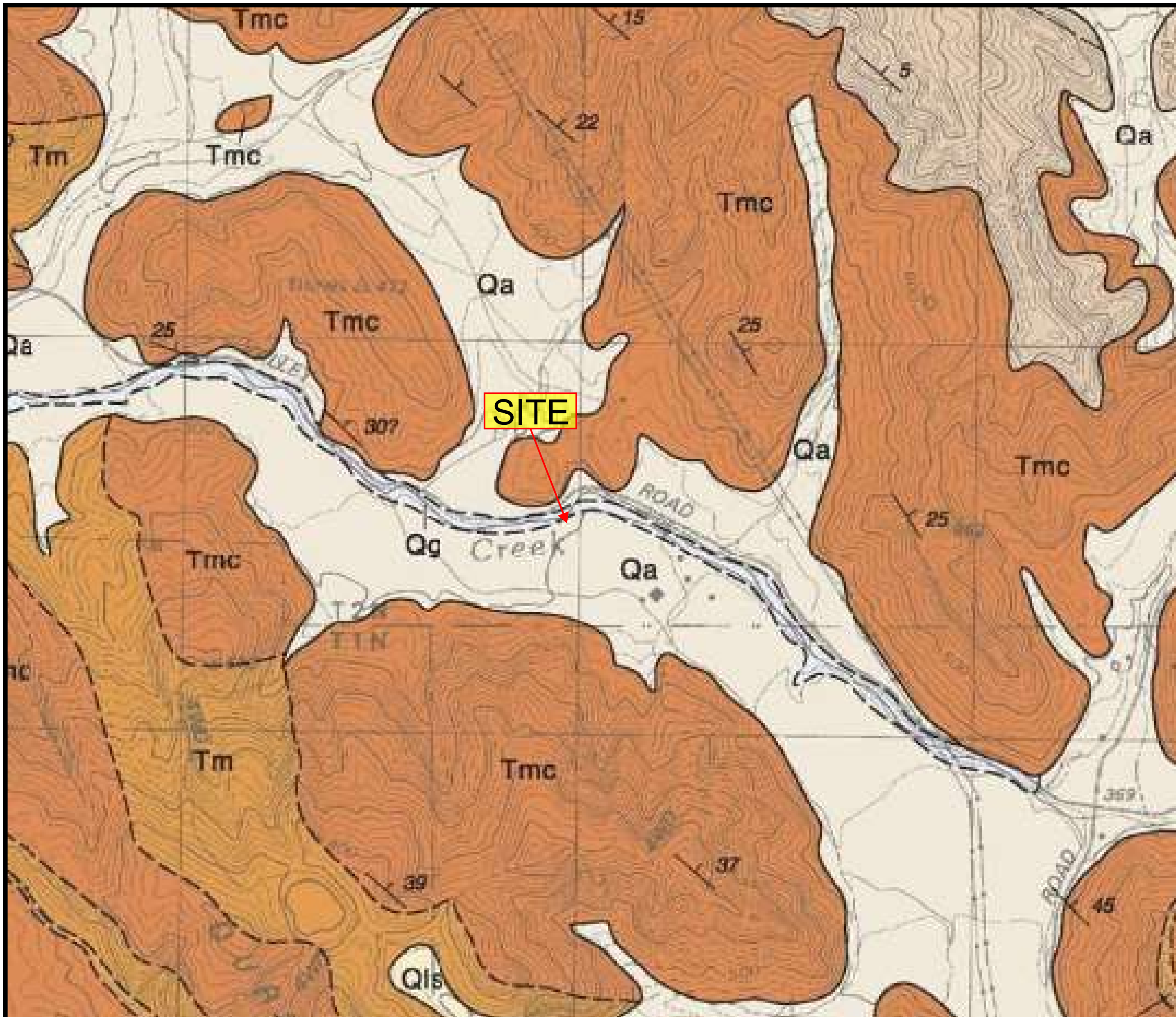
ATLAS

Job No.:	91-68538-PW
Approved:	MZ
Date:	08.07.2024

SITE PLAN

Tomato Stand Fish Passage Project
Pinole Creek Watershed Culvert Replacement
Contra Costa County, California 94553

Plate
2



EXPLANATION

Units

Qa: Surficial Sediments: Alluvial gravel, sand, and clay of valley areas (Holocene)

Qg: Stream Gravel: Sand to boulder size particles (Holocene);

Tmc: Monterey Formation: Claystone, silstone, and fined grained sandstone, gray, massive to vaguely bedded, unfossiferous, locally includes thin layers of sandstone (Miocene);

Tm: Monterey Formation: Siliceous Shale; white weathered, thin bedded, locally cherty and brittle.

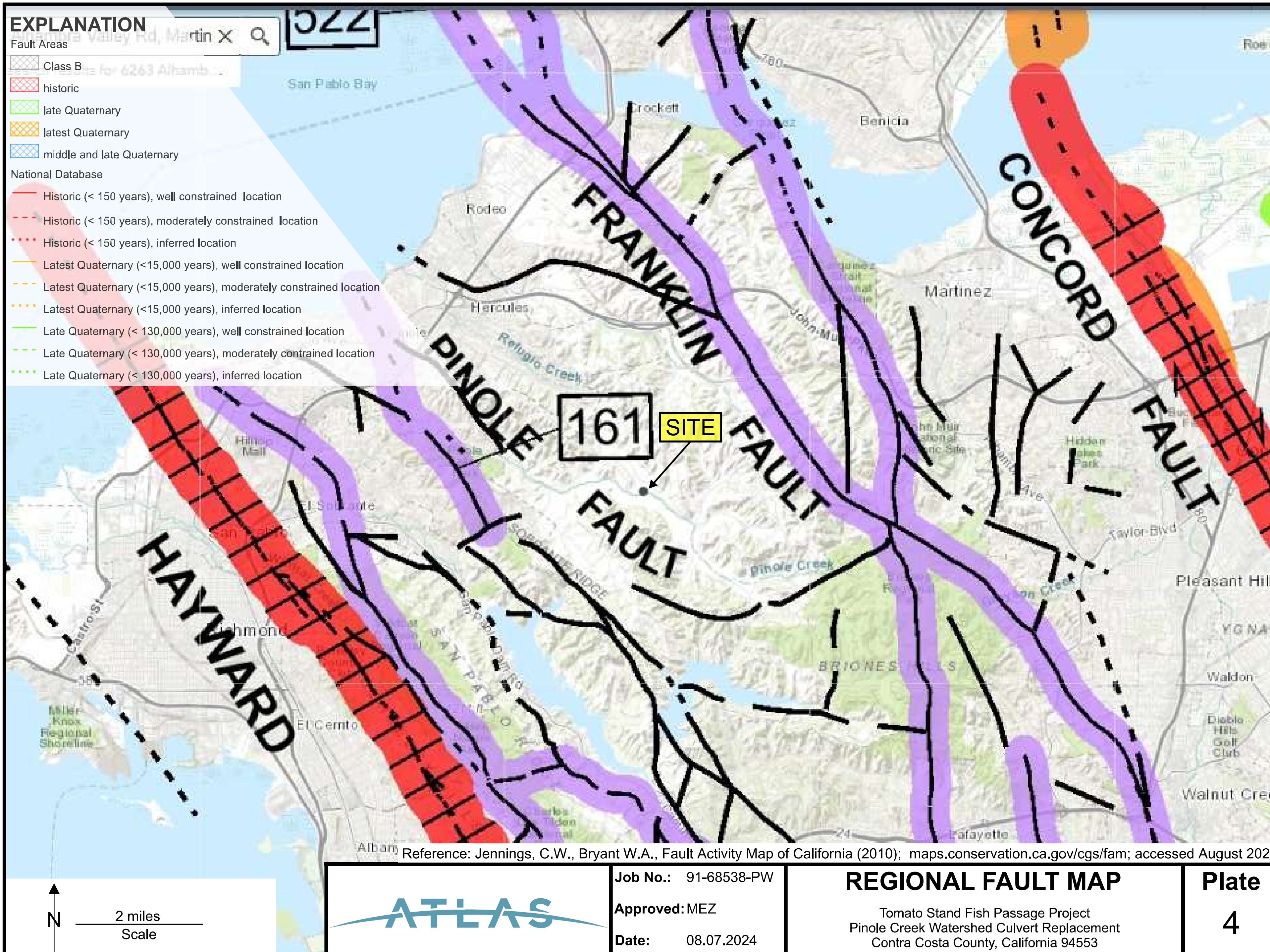
Symbols

Formation - dashed where indefinite or inferred dotted where concealed.

N

2000 feet
Scale
Reference: (Dibblee and Minch 2005)

ATLAS	Job No.: 91-68538-PW Approved: MZ Date: 08.07.2024	**REGIONAL GEOLOGIC MAP** Tomato Stand Fish Passage Project Pinole Creek Watershed Culvert Replacement Contra Costa County, California 94553	**Plate** 3



Reference: Jennings, C.W., Bryant W.A., Fault Activity Map of California (2010); maps.conservation.ca.gov/cgs/fam; accessed August 2024

Job No.:	91-68538-PW
Approved:	MEZ
Date:	08.07.2024

REGIONAL FAULT MAP

Tomato Stand Fish Passage Project
Pinole Creek Watershed Culvert Replacement
Contra Costa County, California 94553

Plate
4

EXPLANATION

Fault Traces

- Accurately Located
- Approximately Located
- Approximately Located, Queried
- Inferred
- Inferred, Queried
- Concealed
- Concealed, Queried
- Aerial Photo Lineament

Fault Zone

Liquefaction Zone

Landslide Zone

Parcels

- Parcel is in an Earthquake Fault Zone, a Liquefaction Zone, and a Landslide Zone
- Parcel is in an Earthquake Fault Zone and a Liquefaction Zone
- Parcel is in an Earthquake Fault Zone and a Landslide Zone
- Parcel is in an Earthquake Fault Zone
- Parcel is in a Liquefaction Zone and Landslide Zone
- Parcel is in a Liquefaction Zone
- Parcel is in a Landslide Zone
- Parcel is not in a zone or has not been evaluated

0 0.5 1.0

APPROXIMATE SCALE (MILES)

Reference: Reference: Esri Community Maps Contributors, California State Parks, GeoTechnologies, Inc. (2025)

ATLAS

Job No.: 91-68538-PW

Approved: MEZ

Date: 3-3-2025

EARTHQUAKE ZONES OF REQUIRED INVESTIGATION

Tomato Stand Fish Passage Project

Pinole Creek Watershed Culvert Replacement

Contra Costa County, California 94553

Plate

5

APPENDIX A

FIELD EXPLORATION

Key to Borings
Boring Logs

BORING NUMBER B-1

ATLAS

CLIENT Restoration Design Group Inc
PROJECT NAME Tomato Stand Fish Passage Project
PROJECT NUMBER 91-68538-PW
PROJECT LOCATION Contra Costa County, California
DATE STARTED 7/11/24 **COMPLETED** 7/11/24
GROUND ELEVATION 301 ft **HOLE SIZE** 4"
DRILLING CONTRACTOR West Coast Exploration
GROUND WATER LEVELS:
DRILLING METHOD Mobile B-24 Solid Flight Auger
▽ **AT TIME OF DRILLING** 13.50 ft / Elev 287.50 ft
LOGGED BY MZ **CHECKED BY** MZ
▼ **AT END OF DRILLING** 16.00 ft / Elev 285.00 ft
NOTES Elevations obtained from Google Earth, 2023
AFTER DRILLING ---

DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	Penetration Rate (sec./ft.)	SPT BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	FINES CONTENT (%)
0	**TOPSOIL 6"** :										
	Qa: FAT CLAY (CH) : Very stiff, dark brown, moist, with trace sand (Possible backfill).	MC 1-1		8-12-16 (28)		93	22				
5		MC 1-2		8-13-14 (27)		94	22	59	19	40	
		MC 1-3		10-13-10 (23)							
10	**Qa: FAT CLAY (CH)** : Very stiff, dark brown, moist.	MC 1-4		7-8-10 (18)		85	33				91
15	▽ Stiff, wet ▼	MC 1-5		7-5-5 (10)							
20	Very stiff	SPT 1-6		5-7-8 (15)			36				
25	**Tmc: CLAYEY GRAVEL WITH SAND (GC)** : Very dense, brown and gray, wet (Possible Monterey Formation).	SPT 1-7		50/4"							
30		SPT 1-8		50/3"							

Bottom of borehole at 30.0 feet.

BORING NUMBER B-2

CLIENT Restoration Design Group Inc
PROJECT NAME Tomato Stand Fish Passage Project
PROJECT NUMBER 91-68538-PW
PROJECT LOCATION Contra Costa County, California
DATE STARTED 7/11/24 **COMPLETED** 7/11/24
GROUND ELEVATION 301 ft **HOLE SIZE** 4"
DRILLING CONTRACTOR West Coast Exploration
GROUND WATER LEVELS:
DRILLING METHOD Mobile B-24 Solid Flight Auger
▽ **AT TIME OF DRILLING** 13.50 ft / Elev 287.50 ft
LOGGED BY MZ **CHECKED BY** MZ
▼ **AT END OF DRILLING** 16.00 ft / Elev 285.00 ft
NOTES Elevations obtained from Google Earth, 2023
AFTER DRILLING ---

DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	Penetration Rate (sec./ft.)	SPT BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0	**ASPHALT 2"** :										
	Qa: FAT CLAY (CH) : Very stiff, dark brown, moist, with trace sand (Possible backfill).	MC 2-1		5-5-8 (13)		89	23				85
5		MC 2-2		6-8-10 (18)							
	Qa: FAT CLAY (CH) : Very stiff, dark brown, moist.	MC 2-3		13-16-18 (34)		87	22	59	24	35	
10		MC 2-4		10-11-12 (23)							
15	▽ **Qa: SILTY SAND (SM)** : Medium dense, gray, wet, with trace to some gravel. ▼	MC 2-5		8-11-12 (23)		90	25				48
20	**Tmc: CLAYEY GRAVEL WITH SAND (GC)** : Very dense, brown and gray, wet (Possible Monterey Formation).	MC 2-6		8-22-35 (57)							
25		MC 2-7		50/5"			23				
30		MC 2-8		50/4"							

Bottom of borehole at 30.0 feet.

APPENDIX B

LABORATORY TEST RESULTS

Atterberg Limits Test Report
Particle Size Distribution

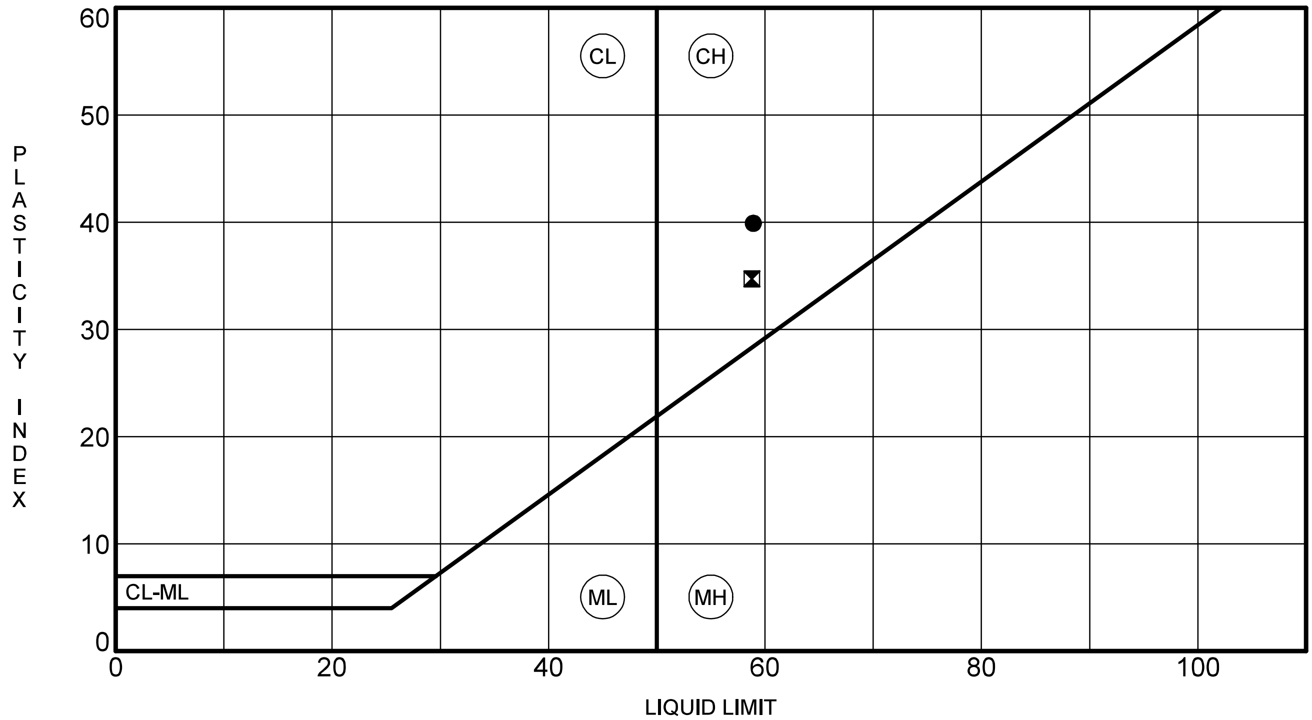
ATTERBERG LIMITS RESULTS

CLIENT Restoration Design Group Inc

PROJECT NAME Tomato Stand Fish Passage Project

PROJECT NUMBER 91-68538-PW

PROJECT LOCATION Contra Costa County, California

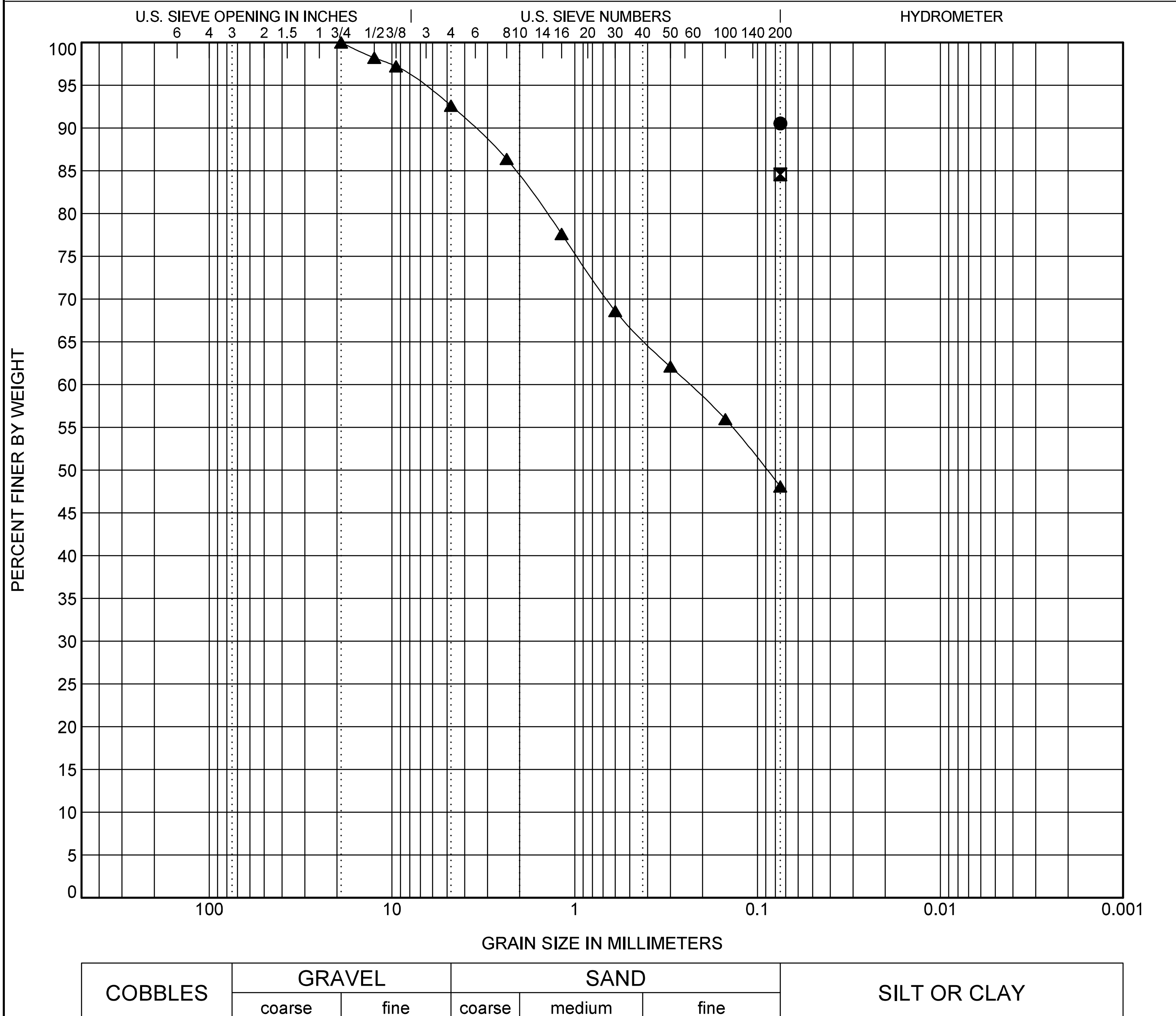


	Specimen Identification		LL	PL	PI	Fines	Classification
●	**B-1**	**4.3**	**59**	**19**	**40**		**FAT CLAY (CH)**
⊠	**B-2**	**6.8**	**59**	**24**	**35**		**FAT CLAY (CH)**

GRAIN SIZE DISTRIBUTION

CLIENT Restoration Design Group Inc **PROJECT NAME** Tomato Stand Fish Passage Project

PROJECT NUMBER 91-68538-PW **PROJECT LOCATION** Contra Costa County, California



Specimen Identification		Classification	LL	PL	PI	Cc	Cu
● **B-1**	**9.3**	**FAT CLAY (CH)**					
⊠ **B-2**	**1.8**	**FAT CLAY (CH)**					
▲ **B-2**	**14.3**	**SILTY SAND (SM)**					

Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
● **B-1**	**9.3**	**0.075**						**90.5**	
⊠ **B-2**	**1.8**	**0.075**						**84.6**	
▲ **B-2**	**14.3**	**19**	**0.237**			**7.4**	**44.5**	**48.1**	