APPENDIX G

Geotechnical Due Diligence Report

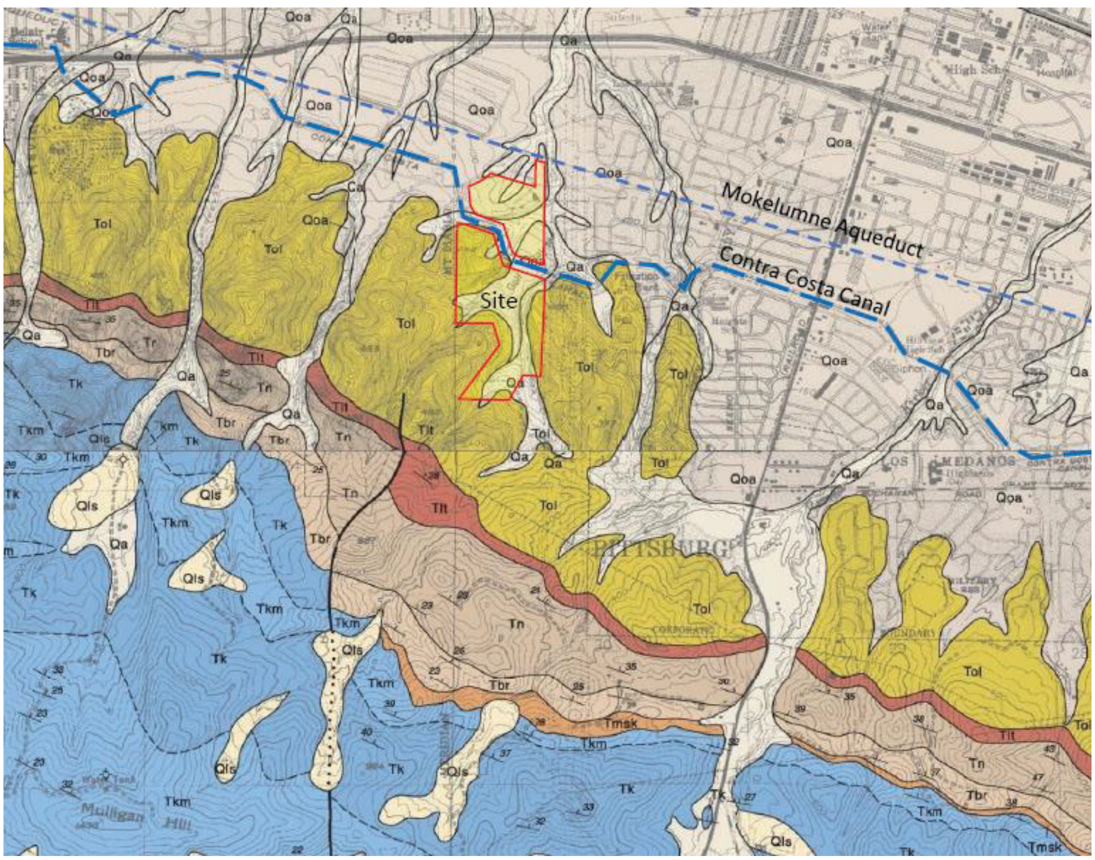
AVAIO CAPITAL

PITTSBURG TECHNOLOGY CENTER PITTSBURG, CALIFORNIA

GEOTECHNICAL DUE DILIGENCE REPORT

JANUARY 10, 2023

CONFIDENTIAL



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AVAIO CAPITAL

PITTSBURG TECHNOLOGY CENTER PITTSBURG, CALIFORNIA

GEOTECHNICAL DUE DILIGENCE REPORT

AVAIO CAPITAL

PROJECT NO. 31405786.000

DATE: JANUARY 2023

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January 10, 2023

Mr. John Malone, PhD
AVAIO Capital

Subject: Geotechnical Due Diligence Report for the Pittsburg Technology Center, Pittsburg, California.

Dear Mr. Malone:

The WSP Geotechnical and Tunneling Group is pleased to submit this Geotechnical Due Diligence Report for the proposed Pittsburg Technology Center site located in Pittsburg, California.

This report presents the results of our geotechnical due diligence investigation including field explorations, laboratory test results, conclusions, discussions and preliminary assessments for proposed earthwork and site improvements for the project.

Should you have any questions, please do not hesitate to contact us.

Sincerely,

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Matteo Montesi, PE, GE, PMP
Director, Geotechnical Engineer
Senior Technical Principal

Kenneth A. Johnson, PhD, CEG, PE
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Borehole Explorations	ConeTec, Inc. (San Leandro, CA)
Laboratory Testing	Inspection Services, Inc. (Berkley, CA)
Corrosion and Thermal Testing	Project X Corrosion Engineering, Inc. (Henrietta, CA)

TABLE OF CONTENT

TABLES

FIGURES

APPENDICES

ACRONYMS AND ABBREVIATIONS

AASHTO	American Association of Highway Transportation Officials
ACI	American Concrete Institute
APN	Assessor's Parcel Number
ASCE	American Society of Civil Engineers
ASTM	American Society for Testing and Materials
BMP	Best Management Practices
CBC	California Building Code
CBR	California Bearing Ratio
CEG	Certified Engineering Geologist
CH	High Plasticity Clay
CL	Low Plasticity Clay
CPT	Cone Penetrometer Test
DSHA	Probabilistic Seismic Hazard Analysis
EIT	Engineer-in-Training
EPA	Environmental Protection Agency
ET	Electrical Tomography
FEMA	Federal Emergency Management Agency
FHWA	Federal Highway Administration
FIRM	Flood Insurance Rate Maps
GE	Geotechnical Engineer
GPR	Ground Penetrating Radar
HSA	Hollow Stem Auger
IT	Information Technology
MASW	Multichannel Analysis of Surface Waves
MCE_G	Maximum Considered Earthquake Geometric Mean
MCE_R	Risk-Targeted Maximum Considered Earthquake
Mod-Cal	Modified California Sampler
Mw	Magnitude
NACE	National Association of Corrosion Engineers
NCHRP	National Cooperative Highway Research Program
NOAA	National Oceanic and Atmospheric Administration
OSHA	Occupational Safety and Health Administration
PCC	Portland Cement Concrete
PE	Professional Engineer
PGA	Peak Ground Acceleration
PSHA	Probabilistic Seismic Hazard Analysis
PL	Plastic Limit

S	Spectral Acceleration
SE	Sand Equivalent
SGRA	Site Ground Response Analysis
SPT	Standard Penetration Test
SSPWC	Standard Specifications for Public Works Construction
TI	Traffic Index
US	United States
USCS	Unified Soil Classification System
USDA	United States Department of Agriculture
USGS	United States Geological Survey
USACE	United States Army Corps of Engineers
$V_{s,30}$	Average Shear Wave Velocity

1 INTRODUCTION

1.1 BACKGROUND

WSP USA Inc (WSP) has been tasked to provide a Geotechnical Due Diligence Report for the Pittsburg Technology Center site located in Pittsburg, California. This document presents the results of WSP's review of available geologic information and initial geotechnical investigation including field explorations, laboratory test results, conclusions, discussions and preliminary assessments for proposed earthwork and site improvements for the project.

The proposed development would consist of several million square feet of building area in various individual data center buildings, plus appurtenant access roads, retaining walls, landscaping, ancillary support infrastructure, and open spaces. Each data center building would have data halls to house equipment necessary for information technology (IT) operations such as computers, servers, storage hardware, cables, racks, and communications equipment.

1.2 PROJECT SITE

The project site is approximately 105-acres in size and located in the former Delta View Golf Course owned by the City of Pittsburg. The site entrance is located at the east terminus of Golf Club Road just east of the intersection with West Leland Road. The site entrance is about 0.5 miles south of State Route SR-4 and about 0.6 miles west of Nevada Pacific Parkway. Pittsburg, California, is located on the southern shore of the Suisun Bay in the East Bay region of the San Francisco Bay Area within Contra Coast County. A site vicinity map showing the project site location is presented in Figure 1. The project site is located on Assessor's Parcel Number (APN) 095-150-032, 094-080-011, 095-160-001, and 095-160-002 and portions of APNs 094-090-001 and 094-080-002. Approximate centralized site coordinates as follows:

Latitude: 38.008°		Longitude: -121.912°

1.3 SCOPE OF WORK

The purpose of this geotechnical due diligence evaluation is to provide insight to key geologic features of the project site and preliminary geotechnical assessments of proposed improvements in support of Master Planning for the project. A preliminary conceptual development layout is presented as Figure 2.

The scope of the geotechnical work undertaken for this project can be summarized as follows:

- Review available geotechnical information and perform a site reconnaissance of the project area.
- Plan and execute a preliminary geotechnical exploration program including geophysical surveys, exploratory borings, and laboratory testing.
- Participate in coordination meetings with the project team and stakeholders.
- Prepare this Geotechnical Due Diligence Report to summarize the results of the review of available geotechnical and geologic information and identify geologic hazards and/or problematic soil conditions that could affect or impact the planned developments.

2 EXPLORATIONS AND TESTING

2.1 SITE RECONNAISSANCE

A general site reconnaissance was performed on November 18, 2022. Surface conditions of the site were observed, and planned field exploration locations were staked. A general topographic map indicating the field explorations for the subject investigation are presented in Figure 3.

2.2 EXPLORATORY BORINGS

Four (4) exploratory borings to a minimum depth of 40 feet, to be performed by ConeTec, Inc. were initially planned and scheduled for December 14 and 15, 2022 for this preliminary geotechnical investigation. However, heavy precipitation the week prior to and the week of planned drilling induced a saturated ground condition severely hindering and preventing practical and safe site access for the drilling equipment. Unfortunate equipment sinking and rutting of the terrain occurred in several areas of the project site. After numerous attempts to access the site without causing damage or mutilation to the ground, it was only possible to execute a single boring as originally planned. Additional explorations to shallower depth and additional geophysical surveys were inserted in the program to compensate for the lack of deep borings and to maintain project schedule, as described in the following sections.

The drilling method for the one deep boring consisted of 4-¼ inch diameter hollow stem auger (HSA) borings using a truck-mounted CME 75 drill rig. The top 5 feet of each boring was hand augered and large bulk samples of soil cuttings were secured. Drive Samples were obtained typically every 5 feet thereafter to a maximum depth of 41.5 feet below ground surface. The boring samples consisted of alternating Standard Penetration Test (SPT) split-spoon and Modified California (Mod-Cal) specimens to obtain both disturbed and relatively undisturbed soil samples, respectively. Groundwater was not encountered in the boring.

In lieu of deep borings, three (3) shallow hand auger borings to a depth of 5 feet and three (3) shallow shoveled holes to a depth of 3 feet were performed to obtain representative near-surface bulk grab samples. The hand auger borings were performed on relatively level ground whereas the shoveled holes were performed along the toe of hill sides in the project area.

A WSP geologist was present full time to log the explorations. Upon completion of the logging, the single deep boring was backfilled with a cement grout mix per the requirements of the boring permit from the County of Contra Costa Health Services Department (Environmental Health Division). The shallow hand auger borings and shoveled holes were backfilled with available cuttings and soils from the immediate area. Soil samples were transported to the laboratory of Inspection Services, Inc. (ISI) in Berkely, California for further evaluation and assignment tests. Exploration logs are presented in Appendix A.

2.3 MULTICHANNEL ANALYSIS OF SURFACE WAVES

Four (4) Multichannel Analysis of Surface Waves (MASW) geophysical survey lines were performed at the subject site by NorCal Geophysical Consultants, Inc. on December 19, 2022. The shear wave velocity analysis was completed via a combination of passive and active source refraction surveys. The active source surveys were performed with 10-foot geophone spacing for a line length of approximately 230 feet and data recorded to an approximate depth of 100 feet. The recorded surface waves were evaluated to develop a shear-wave velocity profile of the subject site to a depth of approximately 100 feet. The time-average shear wave velocity in the upper 100 feet was estimated to range from 936 to 1,018 ft/sec (average 981 ft/sec). Results of the MASW survey are presented in Appendix B.

2.4 ELECTRICAL RESISTIVITY

Four (4) Wenner 4-pin method electrical resistivity tests were performed at the subject site by NorCal Geophysical Consultants, Inc. on December 20, 2022, to estimate the grounding potential of the near-surface onsite soils. Electrical soil resistivity will be used for the design of the electrical grounding system of the proposed facilities. Electrode spacings of 1, 2, 4, 8, 15, 25, 50, 75, and 100 feet were performed. Each test was conducted in two orthogonal directions. Resistivity values ranged from 650 to 1,690 ohm-cm with an average on the order of 1,100 ohm-cm. Results of the electrical resistivity tests are presented in Appendix B.

2.5 LABORATORY TESTING

Geotechnical laboratory testing was performed by ISI on select soil samples that were collected from the explorations. The tests include determinations of 13 moisture contents, 4 dry densities, 6 sieve analyses, 6 hydrometers, 6 Atterberg limits, and 4 organic contents. Material compaction characteristics were evaluated by performing 4 Modified Proctor compaction tests.

Thermal resistivity and corrosion potential tests were performed by Project X Corrosion Engineering of Murietta, California on selected near-surface samples obtained from the explorations. Corrosion potential test suites included 4 pH, 4 electrical resistivity, 4 chlorides, and 4 sulfates tests. Likewise, 4 thermal resistivity tests (Rho) were performed near a relative compaction of 90 percent per ASTM D1557 (Modified Proctor).

All tests were performed in general accordance with the applicable ASTM test methods. The laboratory test results are presented on the boring logs (Appendix A) and in Appendix C. A summary of laboratory test results is presented in a table presented in Appendix D.

3 SITE CONDITIONS

3.1 SITE DESCRIPTION

The 105-acre site is bounded by an approximate 800-feet wide Pacific Gas & Electric (PG&E) transmission corridor to the east, West Leland Road to the north, other City owned property (Stoneman Park and additional former golf course land) to the west, and public open-space to the south. Developed residential areas exist further to the east, north and west. The open-channel Contra Costa Canal nearly passes through the northern third of the site in an east-west direction. The canal easement width is on the order of 100 feet. A portion of the canal along the east side of the site is believed to consist of a buried reinforced concrete box-shaped siphon structure (Figure 7). It's load carrying capacity is unknown for potential future loads (i.e., additional fill, pavement and vehicles) is unknown and therefore should be evaluated by a structural engineer.

A former water storage tank was located adjacent to the north side of the Contra Costa Canal. The east-west trending Mokelumne aqueduct is located adjacent to the northern property line. Another 130-foot diameter water storage tank is located about 800 feet south of the southern limit of the project area.

Originally part of the rifle range for the US Army's former Camp Stoneman, the land was given to the city by the federal government in 1947 and opened as a nine-hole golf course. It was ultimately expanded to an 18-hole public golf course. The golf course closed in 2018. The golf course facility has areas that used to occupy a clubhouse, pro shop, restaurant, maintenance building, parking lots, a water storage tank, cart paths, a driving range, practice areas, tees, fairways, greens, sand traps and water hazards. It is anticipated that the golf course has numerous buried water irrigation lines throughout the area. No habitable structures are currently present at the site (former buildings have been demolished). This ground is covered with low grass, trees, shrubs, and bare ground. Several random areas of accumulated debris and trash are present at the site.

Aerial photographs of the project area taken in June 2013 and June 2022 are presented in Figure 4. The limits of previous irrigation of the golf course and relatively recent wild grass burn areas are presented.

3.2 CLIMATE

Pittsburg is an industrial suburb located on the southern shore of the Suisun Bay in the East Bay region of the San Francisco Bay Area and is part of the Sacramento–San Joaquin River Delta area. Pittsburg experiences a hot summer Mediterranean climate bordering on semi-arid climate due to the Mt. Diablo rain shadow in East Contra Costa County. Winters are short, cold, wet, and partly cloudy.

Over the course of the year, the temperature typically varies from 39°F to 90°F and is rarely below 31°F or above 101°F. The hot season lasts about 4 (June through August) with an average daily high temperature above 83°F. The hottest month is July with an average high of 90°F and low of 58°F. The cool season lasts about 3 (mid-November through mid-February) with an average daily high temperature below 62°F. The coldest month is January with an average low of 40°F and high of 57°F.

The chance of wet days in Pittsburg varies throughout the year. The wet season lasts about 5 months (November through March). The wettest month is February. The dry season lasts 7 months (April 9 through October). The driest month is August. The area receives approximately 16 inches of rainfall annually. The average annual snowfall is zero (0) inches. On average, there are 265 sunny days per year in Pittsburg. On average, there is some precipitation about 60 days per year.

3.3 TOPOGRAPHY

The terrain within the former golf course can generally be described as relatively flat to slightly undulating. North of the Contra Costa Canal the lowest ground surface elevation is about +80 feet above mean sea level. The embankment crest elevation along the north side of the Contra Costa Canal is about +120 feet above mean sea level. South of the canal the ground gently rises and undulates within the former golf course fairways reaching elevations greater than +200 feet above mean sea level (Figure 3).

Hillsides along east, west, and south sides of the property reach elevations of about +230 to +250 feet above mean sea level, respectively. Within about ½ mile south and west of the site the ground surface quickly rises to elevations above +450 feet (and greater) above sea level. Further to the south, the Diablo Range's Los Medanos Hills reach elevations of approximately +1,300 feet above mean sea level. The project area is dissected by several natural short drainage courses emanating from the south and southwest. The inverts of these drainages coalesce and drain to the north toward small retention basins.

LiDAR based topography from the United States Geological Survey (USGS) with slope inclination intensity shading and highlights are presented on Figure 5 through Figure 9. These images are depicted with all vegetation and building features removed. In essence, the images are essentially of bare ground which allow for relatively clear visualization of ground conditions. Figure 5 presents a key map of the project area with unlabeled one meter contour intervals. Figure 10 through Figure 14 present enhanced details of the area including natural and man-made features; natural ground and existing cut slope inclinations; supply water and drainage structures; and ten (10) selected cross sections (A-A' through J-J').

Cross sections A-A' through J-J' are presented as Figure 10through Figure 14. These approximated cross sections with were developed using screen shots for the USGS Elevation Profile tool. Each of the developed cross sections has a vertical exaggeration scale which is unique to locations chosen. They include the relative location of surface features, slope conditions, and anticipated soils. The geologic formations presented therein are described in the following sections.

3.4 GEOLOGY

Contra Costa County is located east of San Francisco and extends from California's Great Valley geomorphic province in the east to the Diablo Range portion of the Coast Range geomorphic province to the west. The Great Valley geomorphic province is a deep basin filled with a thick sequence of Jurassic to Quaternary period alluvial deposits eroded from the eastern Sierra Nevada Mountain Range and western coastal mountain ranges. The thickness of these deposits varies from thin veneers along the valley edges to greater than 20,000 feet in the south and central portions of the valley. Tertiary and Cretaceous period outcrops border the central plain of the valley. A regional geologic map and legend for the project area are presented as Figure 15 and Figure 16, respectively. A site-specific geologic map of the project area is presented as Figure 17.

The project site is located along the northern portion of Contra Costa County which is adjacent to San Pablo Bay, Suisun Bay, and the Sacramento River from west to east, respectively. An unnamed creek with a series of dissecting ephemeral drainages traverses the site in a natural dendritic pattern with coalescing flow paths that generally tend north toward towards Suisun Bay. These drainage areas have deposited natural accumulations of alluvial soils which are located in the lower elevations of the project site which are mainly occupied by the former golf course footprint (Figure 5 through Figure 9). These alluvial deposits may include poorly consolidated sand, silt, and clay.

Rocks outcropping south of the project site within the northern tip of the Diablo Range include the Los Medanos Hills which consist of Tertiary-age (Miocene to Pliocene) sediments of the Oro Loma Formation that may be up to 300 feet thick and consists of moderately consolidated siltstone, sandstone, and claystone with interbedded pebble conglomerate.

3.5 TECTONIC SETTING AND HISTORIC SEISMICITY

The San Francisco Bay Area is located near the western edge of the North American Plate. The western edge of the North American Plate is generally defined by the San Andreas Fault zone, with the land west of the San Andreas fault zone considered part of the Pacific Plate. The crustal deformation related to this plate boundary is expressed by numerous faults within the San Andreas Fault system, and this system includes the Hayward Fault, Calaveras Fault, Concord Fault, Clayton Fault-Greenville Fault, and Napa Fault, among others. These Quaternary faults have varying degrees of seismic activity. However, they define a broad area susceptible to earthquake hazards. A regional fault map indicating historic activity in the San Francisco Bay area is presented as Figure 18.

In the state of California an “active fault” is defined as a fault that exhibits surface displacement having occurred during Holocene time (within the last 11,700 years). The definition of “potentially active” varies. A generally accepted definition is of a fault showing evidence of displacement that occurred between 11,700 years and 2.6 million years ago. However, “potentially active” is no longer used as a criterion for zoning by the California Geological Survey (CGS). The terms “sufficiently active” and “well-defined” are now used by the CGS as criteria for zoning faults under the Alquist-Priolo Earthquake Fault Zoning Act. A “sufficiently active” fault is one that shows evidence of Holocene surface displacement along one or more of its segments and branches. A “well-defined” fault is one whose trace is clearly detectable by a physical feature at or just below the ground surface. The definition “inactive” generally implies that a fault has not been subjected to seismic activity for more than 2.6 million years.

The project site is not located within an active Earthquake Fault Zone as defined by the CGS. However, many of the faults in the area are considered active but have not typically generated surface fault rupture. The location, historical seismicity, and maximum magnitudes for earthquakes in the vicinity are presented in Table 3.1. The project site may be subject to ground shaking from seismic events associated with the active and potentially active fault systems in the area. The intensity of ground shaking that occurs during an earthquake depends upon the magnitude of the earthquake, the location of the seismic source relative to the site, and the subsurface conditions.

Table 3.1 Project Vicinity Faults

FAULT	DISTANCE AND DIRECTION FROM PROJECT AREA	HISTORICAL SEISMICITY	MAXIMUM MOMENT MAGNITUDE EARTHQUAKE
□acaville-□irby Hills	26 miles northwest	Many <M 4	NA
Midland-Rio □ista	15 miles east	None within last 700,000 years	NA
Carneros-Franklin	11 miles west	M 6.4, 1898	6.4
West Napa	26 miles northwest	M 6.5, 2003	6.5
Clayton-Marsh Creek-Greenville	3.5 miles southwest	M 5.6, 1980	7.0
Concord/Green □alley	7 miles southwest	Historic Active Creep	6.9
Mt. Diablo Thrust	4 miles southwest	Holocene Active	6.7
Calaveras	12 miles southwest	M 5.6-6.4, 1861, M 4-4.5, 1970, 1990	6.8
Hayward	20 miles southwest	M 6.8, 1868, M 5.6, 1889, many <M 4.5	6.9
Rodgers Creek	30 miles northwest	Holocene Active	7.0
San Andreas	40 miles southwest	M 7.1, 1989, M 8.25, 1906, M 7.0, 1838, many <M 6	7.9

3.6 SURFICIAL SOILS

Much of the existing surficial soils at the former golf course site consist of man-placed fill soils or man-disturbed native soils. These materials consist of both fine (silts/clays) and coarse (sands) soils with highly variable organic content levels and porosity. In general, these soils may be considered poorly compacted. Topsoil could be up to several feet thick in some areas. These soils are deemed incompetent to support additional fill or settlement sensitive structures.

Based on available information from the United States Department of Agriculture (USDA) soil survey website, the lower elevations surficial soils are primarily characterized as "Capay Clay" (1 to 15 percent slopes) and "Rincon Clay Loam" (2 to 9 percent slopes) of the Hydrogeologic Soil Group "C" (slow infiltration rate. These clays have formed alluvial fans and stream terraces. The upper hillsides within the project area characterized by the USDA as "Altamont Clay" (15 to 30 percent slopes) and Altamont-Fontana Complex (30 to 50 percent slopes). These surficial soils are also considered Hydrogeologic Soil Group "C" (slow infiltration rate). The relative locations of these soil types are presented in Figure 19. The permeability of surficial soils is likely to be low.

Clayey native slopes surrounding the site show signs of extensive and variably deep desiccation cracking and ground fissuring.

3.7 GROUNDWATER

Groundwater levels at the site are subject to variations due to seasonal fluctuations, the presence of the Contra Costa Canal, and other artificial/natural influences. In general, groundwater levels at the project site may be considered at or slightly above the elevations of the natural drainages that cross the site. Groundwater table phreatic surface gradients are likely less than 2 percent emanating away from the natural drainages that cross the site. During the wet season, groundwater levels are expected to rise several feet. Isolated zones of perched groundwater may exist within the mass of the hillsides adjacent to the site albeit that there is little evidence such as lateral seeps or springs in the area.

4 GEOLOGIC HAZARDS

4.1 GENERAL

This section discusses common geologic hazards and their potential at the subject site. The evaluations presented herein are based on existing information, WSP's field explorations, laboratory testing, investigation interpretation and professional judgement.

4.2 FROST

Frost penetration depth or frost line is defined as the depth at which the ground moisture is expected to freeze during a sustained period of subfreezing ambient temperatures. Shallow foundations and buried utilities should be located below the frost line to reduce the impacts of ground deformation (heave) induced by groundwater freeze and thaw cycles. Pavements resting on frost-susceptible soils are subject to differential heaving, surface roughness and cracking, blocked drainage, and a reduction in strength during thaw periods.

Presence of frost-susceptible soils in combination with subfreezing temperatures in the soil and a source of water, form the conditions for the formation of frost. Soils are classified into general groups of frost susceptibility based on the fines content, either material passing the #200 sieve (NCHRP 1-37A, 2004) or material finer than 0.02 mm (USACE, 1965). Little to no frost action occurs in clean, free draining sands, gravels, crushed rock, and similar granular materials, under normal freezing conditions. Silts are highly susceptible, because of relatively small voids, high capillary action, and relatively high permeability (FHWA, 2006). Anticipated extreme depth of frost penetration ranges between 10 and 20 inches, based on the National Oceanic and Atmospheric Administration (NOAA) published relevant map (NOAA, 1978). A frost depth of less than 5 inches is suggested by the U.S. Department of Commerce for areas west of Stockton and Sacramento.

4.3 TSUNAMI, FLOOD, DEBRIS FLOW AND SEICHE

Tsunamis are large sea waves that are most often generated by displacements of the ocean floor along submarine faults. They can also develop in response to other events, such as submarine landslides. The site elevation is above +80 feet above mean sea level and the associated risk may be considered nil.

Other types of flooding may occur at the project site due to intense rainfall rates. Based on review of the Federal Emergency and Management Administration (FEMA) Flood Insurance Rate Maps (FIRM) (Map No. 06013C0118G, dated 9/30/2015), the site is not located within a mapped flood hazard zone.

The potential for debris flows including mudslides that may be brought on by intense and persistent periods of rain may exist within the offsite canyon areas to the west and south of the project site. Debris flows are fast moving flows of mud that may include rocks, vegetation, and other random materials. Once triggered, subsequent debris flows may become more frequent. Debris flows pose a hazard to life and property. Potential debris flow sources are presented in Figure 20. The quantity and intensity of debris flow volume has not been estimated.

Seiches are defined as oscillations in a closed body of water such as a lake or reservoir due to earthquake shaking or earthquake rupture. The subject site is not located near a large, enclosed body of water and therefore, the hazard to the project posed by seiches is considered nil.

4.4 SUBSIDENCE

Land subsidence occurs when extensive amounts of groundwater are withdrawn from aquifer systems or due to seismic event, and can damage buried utilities, structures, and generally infrastructure. Typically, fine-grained materials (clays and silts) are more susceptible to settling than coarse-grained materials when subjected to groundwater extraction. Subsidence can also occur in areas of shallow underground mines with incompetent overburden materials. No groundwater extraction or underground mines are known to be near the site. The risk of ground subsidence at the site may be considered low.

4.5 FAULT SURFACE RUPTURE

Ground surface displacement, or rupture, caused by an earthquake is a major consideration in the design of construction across active faults. The Alquist-Priolo Earthquake Fault Zoning Act requires the State Geologist to identify earthquake fault zones along traces of both recently active and potentially active major faults. CGS has not mapped any fault zones within the project area. While there is always a possibility of an unmapped fault crossing the project site, based on the available data, the possibility of fault ground rupture may be considered low.

4.6 GROUND MOTION

The time-average shear wave velocity ($V_{S,30}$) in the upper 100 feet (30 m) was estimated through the MASW geophysical survey to be equal to 981 ft/sec. Therefore, the project site can be classified as Seismic Site Class D per ASCE 7-16, as shown in Table 4.1.

Table 4.1 Seismic Site Classes per ASCE 7-16

SIESMIC SITE CLASS	$V_{S,30}$ (FEET/SEC)
A. HARD ROCK	> 5,000
B. ROCK	2,500 to 5,000
C. VERY DENSE SOIL AND SOFT ROCK	1,200 to 2,500
D. STIFF SOIL	600 to 1,200
E. SOFT CLAY SOIL(1)	< 600

(1) Any profile with more than 10 feet of soil with Plasticity Index (PI) greater than 20, moisture content greater than 40 percent, and undrained shear strength less than 500 psf is classified as Site Class E.

Seismic demand per ASCE 7-16 for the subject site can be determined from the Seismic Design Maps using the ASCE 7 Hazard Tool provided that certain code requirements are met (see discussion below and Section 11.4.8 of ASCE 7). Estimated preliminary seismic design parameters using a Seismic Site Class D are presented in Table 4.2. The proposed facilities may be classified as Risk Category III and IV, for single-story and multi-story buildings, respectively. Appendix D presents the ASCE 7 Hazard Tool Report.

Table 4.2 Preliminary Seismic Design Parameters

DESIGN PARAMETER	SYMBOL	VALUE
Site Class	--	D
Mapped MCE_R (5% damped) spectral acceleration for short periods (Site Class B/C)	S_S	1.891
Mapped MCE_R (5% damped) spectral acceleration for 1-second period	S_1	0.646
Short-period site coefficient (at 0.2 sec period)	Fa	1
Long-period site coefficient (at 1.0 sec period)	Fv	1.7*
MCE_R (5% damped) spectral acceleration at short periods adjusted for site class ($S_{MS}= F_aS_s$)	S_{MS}	1.891
MCE_R (5% damped) spectral acceleration at 1-second period adjusted for site class ($S_{M1} = F_vS_1$)	S_{M1}	1.10*
Design spectral response acceleration (5% damped) at short period (2/3*S_{MS})	S_{DS}	1.261
Design spectral response acceleration (5% damped) at 1-second period (2/3*S_{M1})	S_{D1}	0.732*
Mapped MCE_G Peak Ground Acceleration	PGA	0.784
Site Coefficient for Peak Ground Acceleration	F_{PGA}	1.1
MCE_G Peak Ground Acceleration adjusted for site class effects	PGA_M	0.862
Long-Period transition period (seconds)	T_L	8

* See discussion below for additional requirements for site-specific studies
Source: Based on ASCE 7-16, available at https://asce7hazardtool.online/
MCE_R: Risk-Targeted Maximum Considered Earthquake (2% probability of exceedance in 50 years)
MCE_G: Maximum Considered Earthquake Geometric Mean

Generally, ASCE 7-16 11.4.8 requires a Site-Specific Ground Motion Hazard Analysis for structures on Site Class D with $S_1 \geq 0.2$. ASCE 7-16 Supplement 3 (which is adopted by the 2022 California Building Code (CBC) and became effective as of January 1st, 2023) provides an exception to avoid a Site-Specific Ground Motion Hazard Analysis, as long as the value of the parameter S_{M1} reported in Table 4.2 (and, subsequently, S_{D1} as well) is increased by 50%.

A site-specific ground motion hazard analysis with or without a site response analysis may be considered in all cases to try to reduce the seismic demand and to generate, if needed, acceleration time histories.

4.7 LIQUEFACTION

Liquefaction is a phenomenon in which saturated, cohesionless soils lose their inherent shear strength and stiffness due to build-up of excess pore water induced by cyclic loading, such as that caused by an earthquake. Liquefaction potential depends on several factors, primarily the (a) relative density and type of soil, (b) the depth to the groundwater, (c) overburden pressures, and (d) the duration and intensity of seismic shaking (PGA). Loose, saturated granular materials (sands and low to non-plastic silts) are most susceptible to liquefaction. Cyclic softening is a phenomenon in which saturated silts and clays exhibit significant strains and strengths loss during cyclic loading.

The potential consequences of liquefaction to engineered structures include loss of bearing capacity, buoyancy forces on underground structures and utilities, ground oscillations or "cyclic mobility," increased lateral earth pressures on retaining walls, post-liquefaction settlement, lateral spreading/slope instability, and "flow failures" or lateral spreading in slopes.

The CGS has identified much of the low-lying zones of the project area as having the potential for earthquake induced liquefaction based on the wide presence of Quaternary age sediments that may have a shallow groundwater condition. An excerpt of the CGS Seismic Hazards Program Liquefaction Hazard Map is presented as Figure 21. This map is only presented as a guide for identifying areas that could have a perceived risk and potential for liquefaction that should be specifically investigated for such conditions if buildings for human occupancy are planned. Inasmuch, future geotechnical investigations for the project should include sufficient subsurface explorations (i.e., borings and CPTs) throughout the site with corresponding laboratory testing that would allow for a proper detailed assessment of liquefaction potential, adverse effects, and remediation (if necessary). Notwithstanding, due to the anticipated subsurface soil and groundwater conditions at the site, the liquefaction potential may be considered low to moderate.

4.8 LATERAL SPREADING

Lateral spreading is defined as the finite, lateral displacement of gently sloping ground because of pore pressure build-up or liquefaction in a shallow underlying deposit during an earthquake. Lateral spreading can impose lateral loads upon the foundations. The subject site is generally flat, without free faces (such as stream banks or slopes). Therefore, lateral spreading hazard may be considered low.

4.9 SEISMIC SETTLEMENT

Seismic settlement is a phenomenon in which loose, unsaturated coarse-grained soils tend to densify during earthquake. Given the anticipated predominant presence of granular soils and the deep groundwater table, seismic compaction might manifest at the site due to a seismic event. The estimated anticipated seismic settlement is on the order less than one to several inches. Minor remedial earthwork of the near-surface soils is expected to mitigate most of the surficial seismic settlement potential.

4.10 EXISTING SLOPES

The subject site is surrounded by rising hillsides to the west, south and east consisting of undisturbed natural ground with maximum inclinations on the order of 15 to 20 degrees from horizontal. These hillsides reach peak elevations outside of the project area on the order of +230 to +270 feet above mean sea level. Two existing northeast facing cut slopes associated with the construction of the Contra Costa Canal are located immediately adjacent to the canal along the west side of the site. These cut slopes are about 20 and 35 feet high with an inclination of about 1.5H:1V (Figure 6, 7 and 9). An existing north facing cut slope exists adjacent to the canal east of the project site boundary within the PG&E transmission corridor. This cut slopes is also about 30 feet high with an inclination of 1.5H:1V (Figure 6 and 9).

No existing landslides including landslips, escarpments, slumps, or other salient ground failures were observed in the project area slopes during the site reconnaissance and investigation activities that are directly within planned development areas. Existing slopes in the project area may be considered stable. However, the presence of desiccation cracks in existing slopes or their potential development in engineered cut slopes should be considered during final slope design.

4.11 EXPANSIVE SOILS

Expansive soils are materials that undergo significant volume changes in response to relative changes in water content (wetting and drying). Expansive soils have a significant amount of clay particles, which can absorb, release, and hold water. The amount of expansive clay minerals and the magnitude of water content change controls volumetric changes. Seasonal water content fluctuations might result in volume changes of surficial soils, exerting stress on pavements and shallow foundations bearing on them.

Lightly loaded structures are more susceptible to damage by expansive soil. Expansive soils can be highly plastic, stiff and overconsolidated with low natural water content and high natural dry unit weight. Simplified methods have been developed to identify expansive soils based on grain size and index properties. In general, soils meeting all four of the following provisions may be considered expansive:

- Plasticity Index (PI) > 15 percent
- Percent of fines (passing sieve #200) > 40 percent
- Percent of colloidal particles (<5 micron) > 20 percent
- Expansion Index > 20

Since the subject site is expected to be underlain by predominantly low to high plasticity clays (CL/CH), medium to high expansion potential is anticipated. The extensive desiccation cracks in the area also provide evidence of potentially expansive soils. Inasmuch, the potential hazard to the project due to expansive soils may be considered moderate to high.

4.12 COLLAPSIBLE SOILS

Collapsible soils can be defined as soils that have the potential to undergo rapid deformation when inundated with water under constant applied load. Typically, collapsible soils have a low dry density and low natural moisture content. Many collapsible soils have little to no plasticity and often classify as silts (ML) or lean clays (CL) (FHWA, 2017). Several criteria based on dry density, liquid limit, void ratio, and other index properties have been proposed for the indirect identification of collapsible soils. Those serve as indicators and do not account for soil properties, such as natural particle structure or cementation. The onsite soils are presumed to have low collapse potential.

4.13 SOIL CORROSIVITY

Corrosion testing (pH, sulfates, chloride, electrical resistivity) typically evaluates the presence of chemicals corrosive to concrete and ferrous materials in the subsurface soils.

The amount of dissolved inorganic solutes in soil is directly proportional to the corrosive potential. High sulfate content might be deleterious to concrete materials in foundation elements, while high chlorides content might be corrosive to ferrous materials. Sulfates and chlorides concentrations higher than 1,000 (parts per million) ppm and 500 ppm, respectively, may be indicative of corrosive environments. Similarly, pH values lower than 5.5 may generally be considered detrimental for concrete foundations. Tests soils at the site have a pH ranging from 8.1 to 8.7. Soluble sulfate test results ranged from 33 to 556 mg/kg. Soluble chloride test results ranged from 18 to 869 mg/kg. Minimum electrical resistivity test results ranged from 482 to 1,876 ohm-cm.

American Concrete Institute (ACI) 318-14, Table 19.3.1.1 classifies the soil environments based on the water-soluble sulfate concentrations into Exposure Categories, as shown in Table 4.3. Restrictions to the concrete types apply if the sulfates concentration indicates exposure category higher than S0.

Table 4.3 Exposure Categories Based on Sulfate Concentration

EXPOSURE CATEGORY	SULFATE CONCENTRATION
S0	$SO_4^{-2} < 0.10$ %
S1	$0.10\% \leq SO_4^{-2} < 0.20\%$
S2	$0.20\% \leq SO_4^{-2} < 2\%$
S4	$2\% > SO_4^{-2}$

In addition, per ACI 318, for non-prestressed concrete, the permitted maximum amount of water-soluble chloride ions incorporated into the concrete depends on the degree of exposure to an anticipated external source of moisture and chlorides. Additional information on the effects of chlorides on the corrosion of steel reinforcement are discussed in ACI 201.2R (providing guidance on concrete durability) and ACI 222R (providing guidance on factors impacting corrosion of metals in concrete). Initial evaluation of the chloride ion content of the concrete mixture can be obtained by testing individual concrete ingredients.

Resistivity is an indirect measurement of the soluble salt content in the soils, and generally varies with the soil moisture content, and is inversely proportional to the soil corrosive potential. The evaluation of corrosion potential of buried unprotected metal objects can be performed based on the commonly accepted correlation with the minimum soil resistivity per National Association of Corrosion Engineers (NACE, 1984), as shown in Table 4.4.

Table 4.4 Corrosion Potential Based on Electrical Resistivity

MINIMUM RESISTIVITY (OHM-CM)	CORROSION POTENTIAL
0 – 1,000	Severely Corrosive
1,000 – 2,000	Fairly Corrosive
2,000 – 10,000	Moderately Corrosive
>10,000	Mildly Corrosive

WSP opines that near-surface soils at the site are expected to be moderately to severely corrosive. It is recommended that the corrosion test results be reviewed and evaluated by the project designers considering the proposed improvements and project lifespan requirements. A qualified corrosion engineer can be contacted for detailed evaluation of corrosion potential with respect to construction materials at this site and review the proposed design.

4.14 RADON

Radon is a colorless, odorless, tasteless radioactive gas, produced as a natural decay produce of uranium. Radon can be encountered in different concentrations in subsurface materials and may seep from the ground into the atmosphere and in the built environment, especially in basements or ground floors. The radon concentration in the atmosphere is typically lower than 0.5 pCi/L (picocuries per liter of air). Remedial actions should be taken when radon concentrations exceed 4 pCi/L, per recommendations of US Environmental Protection Agency (EPA). Based on the available geohazards online database from the EPA, the subject site is mapped within a zone with a radon average of 2 to 4 pCi/L (moderate level). Monitoring the radon levels during the service life of the planned development may be warranted.

5 GEOTECHNICAL CONSIDERATIONS

WSP opines that there are no geologic hazards or problematic soil conditions that would prevent the planned development, provided that a design geotechnical investigation program is included in the next steps and that sound geotechnical engineering recommendations are implemented in the project design. Based on the results of our site reconnaissance, document review, field explorations, laboratory testing, assessments, and professional experience, it is our judgement that the construction of the proposed project is feasible from a geotechnical standpoint. A rigorous, robust, and rational geotechnical investigation following local, regional, and state guidelines and requirements is recommended. The following sections present considerations for geotechnical design for earthwork and structures.

5.1 EARTHWORK

Conventional earthwork and grading methods may be considered appropriate for the subject project. The delineation of potentially problematic areas that required special attention should be evaluated during the detailed geotechnical investigation phase. Grading plans prepared by the project Civil Engineer should be prepared in conjunction with the recommendations in a Geotechnical Design Report.

5.1.1 SITE PREPARATION

Prior to start of any earthwork, the site should be cleared of vegetation, debris, and trash. Buried obstructions, such as tree roots and abandoned utilities, should be removed. Deleterious materials including organics and other debris resulting from the clearing and grubbing operations should be removed from the site. Soils with organic content exceeding 2 percent may be considered "topsoil" and should not be used for engineered fill. Near-surface soils within the former golf course are anticipated to be variably loose and soft with low to high moisture content. Based on the anticipated subsurface conditions, mass grading can be accomplished using conventional heavy-duty earthmoving and compaction equipment. Large cobbles and boulders that would require special equipment or handling are not anticipated. All work should be performed in accordance with the latest approved editions of the Standard Specifications for Public Works Construction (SSPWC), Part 2 (Construction Materials) and Part 3 (Construction Methods) and the California Building Code (CBC) Appendix J.

5.1.2 REMEDIAL EARTHWORK

Most near-surface soils are anticipated to not be site suitable for direct support of proposed improvements. Inasmuch, some remedial earthwork and grading should be anticipated throughout the former golf course area. A specified level of soil overexcavation and subsequent recompaction may be required depending on planned site grades with respect to existing grades and the depth of existing incompetent materials. The depth and lateral extent of remedial earthwork should be determined based on the results of a thorough and comprehensive geotechnical investigation of the site. It is estimated that the depth of remedial earthwork could range from 5 to 15 feet over a significant portion of the site.

The remediation of near-surface expansive soils at the site may include their direct removal and replacement with low to non-expansive material to depth on the order of about 5 feet below finish grade in building areas and 3 feet in pavement areas. Alternatively, the use of lime stabilization treatment may be considered in order to reduce or eliminate the expansion potential of compacted soils. The use of engineered geotextile/geogrid reinforcement may also be considered for in areas where imposed loads may induce excessive shear stresses and differential settlement. The Geotechnical Design Report should provide area specific recommendations for remedial earthwork.

5.1.3 ENGINEERED FILL

In general, existing onsite soils may be reused as engineered fill within specified limits to be determined. In conventional earthwork terms, all engineered fill soils should be compacted to a minimum of 90 percent of maximum dry density as determined by ASTM D1557 (Modified Proctor), in loose lifts not exceeding 12 inches in thickness, moisture-conditioned to near optimum moisture content (±2%).

Areas including pavements, slab-on-grade for floors, walkways, and other hardscape/flatwork areas, the upper 12 inches of subgrade should be moisture conditioned near the optimum moisture content (±2%) and compacted to at least 95 percent relative compaction of the maximum laboratory dry density as determined by ASTM D 1557 (Modified Proctor). The maximum particle size in this zone should be limited to 1-½ inch.

The relative compaction of fills should be tested by a qualified geotechnical professional and construction services laboratory personnel.

5.1.4 ENGINEERED SLOPES

Conceptually, it is anticipated that the project may have engineered slopes consisting of excavations (cuts) and embankments (fill) on the order of 20 to 40 feet in maximum height. All slopes should have a maximum inclination of no greater than 2H:1V. Terraces at least 8 feet wide at not more than 30-foot vertical intervals on all cut and fill slopes should be provided to control surface drainage. Terraces should be provided with suitable access to allow for cleaning and maintenance. Where more than two terraces are required, one terrace, located at approximately mid-height, should be at least 12 feet wide. Swales or ditches should be provided on terraces. Brow ditches should be placed at the top of all slopes. Where existing ground is steeper than 5H:1V (20%) and the depth of fill exceeds 5 feet, benching should be performed in accordance with Figure J107.3 of the CBC Appendix J. A keyway should be provided which is at least 10 feet wide and 2 feet deep.

Slopes may be susceptible to shallow sloughing in periods of intense rainfall, heavy irrigation, and upslope runoff. Periodic slope maintenance may be required including rebuilding the slope face. Sloughing of fill slopes can be reduced by overbuilding and cutting back to the desired slope. To a lesser extent, sloughing can be reduced by backrolling slopes at frequent intervals during grading. At a minimum, all fill slopes should be trackwalked so that a dozer track covers all surfaces at least twice. All cut and fill slopes should be planted and maintained. Both cut and fill slopes may be subject to softening and creep movement, whether the slopes are natural or man-made.

Geologic and geotechnical observations should be performed during the excavation of planned cut slopes to document newly exposed material conditions and verify the presence of potentially adverse bedding conditions of the Oro Loma Formation, where present. Although not anticipated, if excavations of material that has groundwater seepage is observed, the excavation should be halted and appropriate mitigation measures should be implemented (i.e., install closely spaced horizontal drains).

All planned slopes should be properly analyzed and designed following conventional geotechnical engineering practice which include appropriate field explorations, disturbed/undisturbed sampling, laboratory testing, and limit equilibrium stability analyses for permanent, temporary, and seismic conditions. The analyses should account for potential variable groundwater conditions, imposed external loads and the presence of desiccation cracks.

5.1.5 TEMPORARY EXCAVATIONS

Temporary excavations should be laid back or shored in accordance with the U.S. Occupational Safety and Health Administration (OSHA) and any other applicable regulations. For planning purposes, all near-surface soils can be considered OSHA Type C soil. The actual OSHA soil type should be determined by the contractor's responsible person in the field at the time of construction. Type C soils may have up to 1½H:1V temporary construction excavation slopes up to 20 feet high. If stability of an excavation becomes questionable during construction, the excavation should be evaluated promptly by the geotechnical engineer. The vertical unbraced excavations are not recommended.

The soil classifications presented in this report may be used for the planning of temporary excavations in accordance with OSHA requirements. Construction personnel should be aware that soil conditions may change rapidly if soil moisture conditions change or if soils that have been disturbed by previous excavations are encountered. Measures should be taken to protect construction personnel from raveling of excavated slopes. All excavations should comply with current OSHA safety requirements.

No surcharge loads, such as the weight of heavy equipment, should be placed within 10 feet from the top of open excavations. Care should be taken during excavation to avoid removing support for any existing improvements, such as foundations, pavements, and buried utilities. The contractor is responsible for selecting, designing, and constructing temporary shoring systems (if needed) that adequately protect the existing structures, utilities, and other improvements.

5.1.6 EROSION CONTROL

The potential for soil erosion is largely impacted by local soil characteristics, vegetative cover, topographic relief, and the frequency and intensity of rainfall and wind. Removal of vegetation and disturbance to surficial soils by construction activities may result in local increases of erosion rates in unprotected areas. As a result, sedimentation may increase in local drainages and slope intersections. Uncontrolled diversion of storm water runoff from the site to unlined drainage channels could result in extensive erosion due to concentrated flow. This is particularly true during and immediately following site grading. Site development normally increases the amount of impervious area, thus increasing the volume of storm water runoff. Concentration of flow in drainage structures can result in increased flow velocities and erosion potential. Soils on slopes exposed by site development will be subject to erosion by wind and water. This can result in increased turbidity of runoff to the downstream area.

Erosion prevention and sedimentation control is a complex issue and is usually best addressed by sound planning and the use of Best Management Practices (BMPs). Erosion control BMPs are the "best" available technologies that are consistent with conventional local control practices. Implementation is dependent onsite conditions and applicability of proven cost-effective methods. The selection and implementation of construction BMPs is dependent on what existing features need to be protected.

BMPs for erosion and sediment control are selected to meet the specific objectives based on site conditions, serviceability, and cost. Various BMPs in combination or succession may be needed for a given area. Selection of erosion control BMPs should be based on minimizing disturbed areas, stabilizing disturbed areas, and protecting slopes and channels. It also should be based on retaining sediment on-site and controlling the site perimeter. All implemented BMPs should be regularly monitored and controlled after initial installation, as well as during and after any storm generating runoff, to determine maintenance requirements and the general condition of the installed system.

To reduce soil erosion and sediment transport, protective material such as gravel, crushed stone, pavement, and other effective erosion control materials should be used to stabilize exposed soils. Slopes should be provided with temporary drainage and erosion control measures during construction until permanent measures can be installed. Storm water runoff from construction areas should be conveyed to temporary diked detention areas for sediment deposition, then discharged to the existing natural drainage courses with velocities slow enough to prevent further erosion in the drainage courses.

Control of erosion and sedimentation on recently graded construction sites require both vegetative and structural measures. Vegetative species used to control erosion should be selected to accommodate the soil characteristics and climate at the site. Storm runoff control should be provided during and after completion of site grading by using diversion dikes and permanent drainage facilities. Sediment retention structures such as sediment basins, sediment traps or silt fences should be used to keep eroded material on the site. Straw bales used alone, or in combination with geotextiles, can be effective sediment retention structures when properly installed and maintained.

5.1.7 SITE DRAINAGE

Final elevations at the site should be planned so that positive drainage is established around structures such that surface water runoff is directed away from foundations and top of slopes and other proposed elements of the project. Positive site drainage is defined as a slope of 1 percent or more for a distance of 5 feet or more away from foundations.

5.1.8 STORMWATER INFILTRATION

The feasibility of a stormwater infiltration system is dependent on the geologic, hydrogeologic and geotechnical conditions of a site. In general, near-surface soils at the site are relatively impermeable. Based on our evaluation and experience, these near-surface soils are expected to have a slow infiltration rates less than 0.5 inch/hour. Based on our understanding of the overall site conditions and planned construction, the use of a stormwater infiltration system, which would permit wetting and saturation of both compacted engineered fill soils and natural undisturbed formational soils, should not be utilized in project design.

5.2 STRUCTURES

Building, retaining wall and bridging structures having ground supporting elements consisting of shallow footings, deep foundations, and slab-on-grade floors may be considered appropriate for the subject project. Structure plans prepared by the project Structural Engineer should be prepared in conjunction with the recommendations presented in a rigorously reviewed and approved Geotechnical Design Report

5.2.1 SHALLOW FOUNDATIONS

It is anticipated that conventional shallow spread and continuous foundations may be used for the project structures if supported on dense native soils or properly compacted fill. The Geotechnical Design Report should specify minimum dimensions for shallow foundations, maximum allowable soil bearing pressure, sliding/passive lateral resistance and estimated total/differential settlements. Shallow foundation dimensions and reinforcement should be determined by the project Structural Engineer.

5.2.2 DEEP FOUNDATIONS

Deep foundations may be considered when shallow foundations are deemed unsuitable for structure support. Deep foundations may include cast-in-place drilled holes (CIDH), driven steel or prestressed precast concrete piles, micropiles or special proprietary systems. Deep foundations may derive their downward axial resistance from end bearing and side friction along the shaft. However, end bearing resistance may be limited or neglected depending on the chosen installation method and groundwater conditions. Uplift resistance is principally derived from side friction along the shaft. An exception to this is if a CIDH shaft is used that has a specially designed belled end. Lateral resistance for deep foundations may be derived from passive resistance generated from adjacent soils when loads are applied. Deep foundations may be designed as groups in order to improve both axial and lateral capacity. Deep foundation dimensions and reinforcement should be determined by the project Structural Engineer.

5.2.3 SEISMIC DESIGN

Seismic design loads should be determined using the seismic design coefficients derived from ASCE 7-16 with applicable Supplements. Preliminary seismic design parameters are presented in Table 4.2. However, site-specific seismic hazard studies are permitted for design of any structure and are required in certain conditions. The objective of a site-specific ground motion evaluation is to determine ground motions for local conditions with a higher degree of confidence than is possible by using the general procedure presented in the code. In some conditions, such as Site Class D with S_1>0.2g (applicable to this project site) nor performing a site-specific Ground Motion Hazard Analyses will result in a penalty on the estimation of the long period (1-sec) spectral coefficients.

Site-specific procedures for computing earthquake ground motions include dynamic Site Ground Response Analysis (SGRA) and probabilistic and deterministic seismic hazard analysis (PSHA and DSHA, respectively). A seismic hazard analysis may consist of one of the following approaches:

- PSHA and, possibly, DSHA if the site is near an active fault
- PSHA/DSHA followed by SGRA
- SGRA only

A SGRA is not required by code for Site Class D sites but is always permitted. The first approach is applicable to bedrock or stiff soil conditions (not softer than Site Class D) and corresponds to the ASCE 7-16 requirement for a Ground Motion Hazard Analysis in Site Class D sites with S_1>0.2g. In this case, the response spectrum can either be computed directly at the ground surface with PSHA/DHSA for the applicable Site Class, or it can be computed for the bedrock using PSHA and DHSA and then transferred to the ground surface using the code-based site coefficients. The drawback of this approach is that the absence of a dynamic site-response analysis implies that acceleration time histories are not developed as part of the site-specific seismic hazard analysis.

The second approach is similar to the first one but it includes a dynamic site-response analysis as well. This makes it applicable to all Site Classes. In addition, acceleration time-histories are developed as part of the study. The third approach can be used if the bedrock spectrum is available either from other studies or if it is taken directly from the code.

There are advantages and disadvantages to each method. If bedrock is at a depth much greater than the extent of the site investigations (such as the case in this project) the direct approach of computing the ground surface motion with PSHA/DSHA may be more reasonable (with or without acceleration time histories developed). If acceleration time histories are needed, the base ground motions are usually obtained by searching available recorded ground motions for similar seismotectonic settings and Site Class (similar style of faulting, expected magnitude, source to site distance, etc.). The ground motions are then scaled and/or spectrally matched to the target spectrum.

In summary, a site-specific seismic hazard study may be considered for the project. The type and extent of the site-specific study may be determined jointly with the client but, in general, should include, as a minimum, PSHA and DSHA given the proximity to the numerous high-potential active faults in the project vicinity presented in Table 3.1. Dynamic site-response analyses may be beneficial to develop site-specific acceleration time histories, if needed. It is noted that a site-specific seismic hazard analysis may reduce the code-based response spectrum (reduction capped at 20% of the code-based spectral accelerations) but it may also increase the resulting response spectrum for certain spectral periods.

5.2.4 RETAINING WALLS

Various types of retaining walls may be considered for the project depending on location and function. Retaining walls in areas backfilled with compacted soil may consist of conventional cast-in-place (CIP) cantilever walls, mechanically stabilized earth (MSE) walls, modular block walls, counterfort walls, gravity walls, gabion walls, and other proprietary wall systems. Retaining walls in areas of excavation cuts may consist of soldier pile walls (with or without anchors), soil nail walls, tendon anchor walls, and other proprietary wall systems. Retaining walls should be designed in accordance with local and state guidelines, standards, procedures and specifications including those promulgated by Caltrans, AASHTO and FHWA. Retaining walls should be designed based on appropriate input from the Geotechnical Engineer including ultimate/allowable bearing pressures, lateral active/passive earth pressures, sliding resistance, seismic loads, and total/differential settlement. Retaining walls design may be performed by the project Civil or Structural Engineer.

5.2.5 SLAB-ON-GRADE FLOORS

This section pertains to recommendations for concrete slab-on-grade floors (including concrete mat foundations for liquid filled storage tanks and transformers) supported on uniformly compacted engineered fill. Subgrade soil supporting floor slabs should be prepared in accordance with the earthwork recommendations of this report. Heavily loaded slab-on-grade floors should be designed as structural mat foundations using a vertical modulus of subgrade reaction, k(v1), or other appropriate design methodology. All concrete placement, joint spacing, and curing operations be performed in accordance with the recommended guidelines of ACI. If expansive soils are present at a shallow depth, the use of post-tensioned floor slabs may be considered.

Subsurface moisture and vapor naturally migrate upward through the soil. Where the soil is covered by a building or pavement, this subsurface moisture will collect and transmit through the concrete slab-on-grade. Therefore, floor slabs should be underlain with appropriate layered underlays to provide a capillary moisture break, vapor barrier and uniform ground support. To reduce the impact of moisture, a polyolefin vapor barrier membrane (>15 mil thickness) with a very low water vapor permeance and high puncture resistance/strength, should be utilized between the prepared subgrade and the bottom of the slab-on-grade floor.

6 FUTURE INVESTIGATIONS

If the subject site is selected for the proposed development, it is highly recommended that a carefully considered and planned field program potentially consisting of shallow backhoe test pits, exploratory borings, cone penetrometer tests (CPT) soundings, and geophysical surveys be performed in areas of planned buildings, retaining walls, cut slopes, and fill embankments. Additional explorations should be performed in areas of special structures that may include bridges and culverts. The existing Contra Costa Canal siphon structure may require supplemental investigations using geophysical techniques such as ground penetrating radar (GPR) and electrical tomography (ET).

Laboratory testing may include additional conventional geotechnical tests to further characterize subsurface material physical and mechanical properties which may include but not be limited to drain/undrained strength, deformation resistance, elasticity parameters, plasticity, particle size distribution, CBR/R-value, permeability, compaction, organic content, expansion index, swell potential, clay minerology, organic content, corrosion potential, and thermal resistivity.

Site-specific analyses for seismic design of buildings and data center equipment may be warranted. These analyses include methods and procedures for computing earthquake ground motions such as dynamic SGRA, PSHA and DSHA as described in Section 5.2.3. The results of these analyses would also be utilized for evaluation of earthquake induced liquefaction, lateral spreading and seismic settlement potential. Recommendations for remedial earthwork and ground improvement should be provided as deemed appropriate.

The analyses and design of earthworks for cut/fill slopes and retaining systems should include long-term static, short-term construction, extreme seismic events and fluctuating groundwater conditions. Design recommendations for pavements should be provided. Estimates for total/differential ground settlement in areas of man-placed fill and existing alluvial soils should be addressed. Design recommendations for shallow/deep foundations and slab-on-grade floors should be provided. Recommendations for specific construction observation and testing should be provided.

The results of the geotechnical investigation should be presented in a Geotechnical Design Report following local and state adopted guidelines, codes and standards. The report should be signed by a licensed California professional Certified Engineering Geologist (CEG) and Geotechnical Engineer (GE). These professionals should also review and comment on the developed engineering plans and specifications for grading/earthwork, structure foundations, pavements and other project features as deemed appropriate.

7 LIMITATIONS

This Geotechnical Due Diligence Report has been prepared for the exclusive use of our client and their consultants for the evaluation of the subject project site. The findings, conclusions, discussions, and recommendations presented in this report are not for project design. No warranty, express or implied, is made.

The scope of services was limited to those described herein. It should be recognized that definition and evaluation of subsurface conditions are difficult. Judgments leading to conclusions and recommendations are generally made with incomplete knowledge of the subsurface conditions present due to the limitations of data from field studies. Additional field explorations, laboratory testing, and engineering analyses are required for the project.

WSP offers various levels of investigative and engineering services to suit the varying needs of different clients. Although risk can never be eliminated, more detailed and extensive studies yield more information, which may help understand and manage the level of risk. Since detailed study and analysis involves greater expense, our clients participate in determining levels of service, which provide information for their purposes at acceptable levels of risk. The client and key members of the design team should discuss the issues addressed in this report with WSP, so that the issues are understood and applied in a manner consistent with the owner's budget, tolerance of risk and expectations for future performance and maintenance.

Recommendations contained in this report are based on very limited field observations and subsurface explorations, laboratory tests, and our professional judgement. It is possible that soil or groundwater conditions could vary between or beyond the points explored. Our geotechnical scope of services did not include environmental assessments or evaluations regarding the presence or absence of wetlands or hazardous substances in the soil, surface water, or groundwater at this site.

This report may be used only by the client and only for the purposes stated, within a reasonable time from its issuance, but no later than one year from the date of the report. Land use, site conditions (both on site and off site) or other factors may change over time.

Any party, other than the client who wishes to use this report shall notify WSP of such intended use. Based on the intended use of this report and the nature of the new project, WSP may require that additional work be performed and that an updated report be issued. Non-compliance with any of these requirements by the client or anyone else will release WSP from any liability resulting from the use of this report by any unauthorized party and the client agrees to defend, indemnify, and hold harmless WSP from any claims or liability associated with such unauthorized use or non-compliance.

8 REFERENCES

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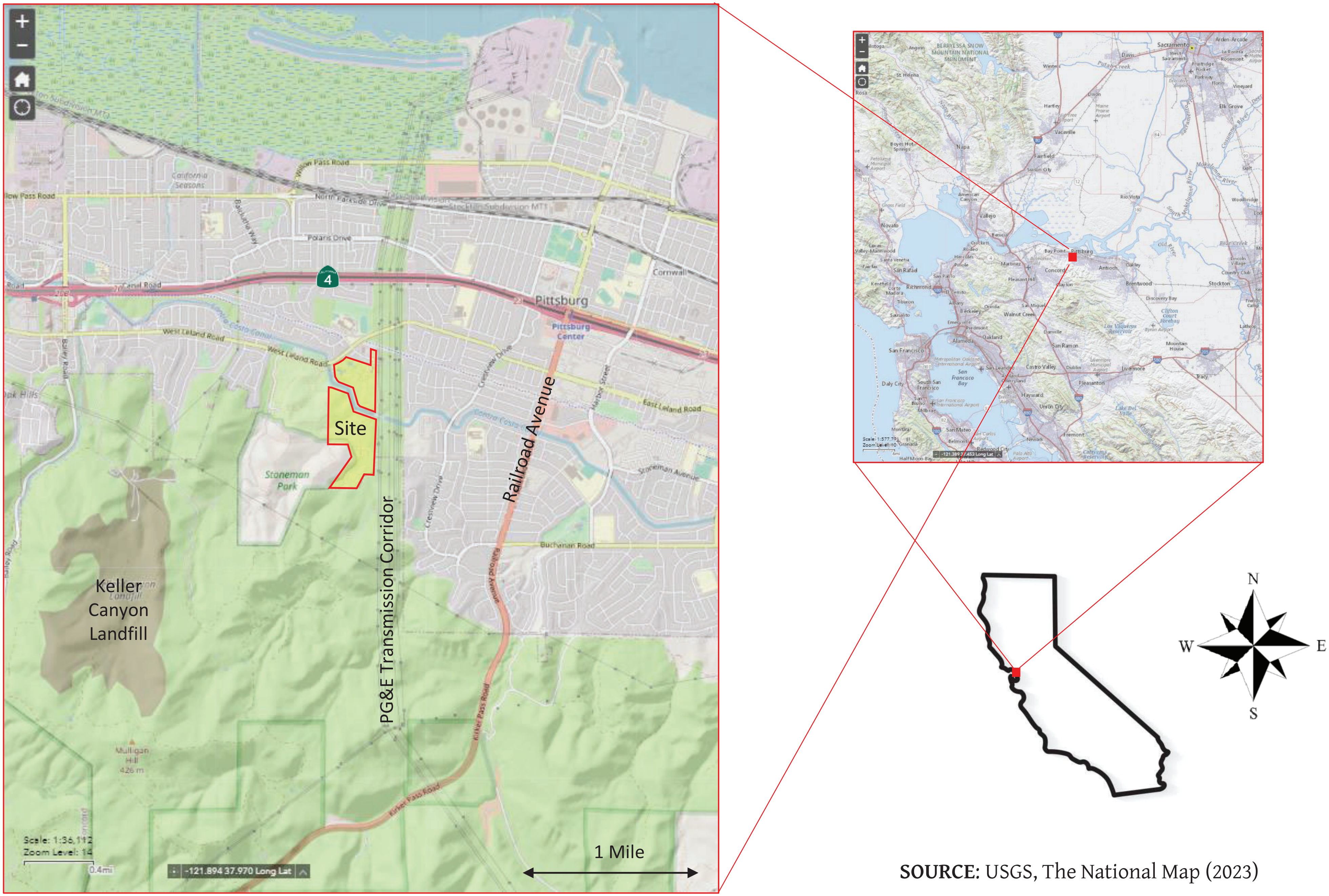
FIGURES



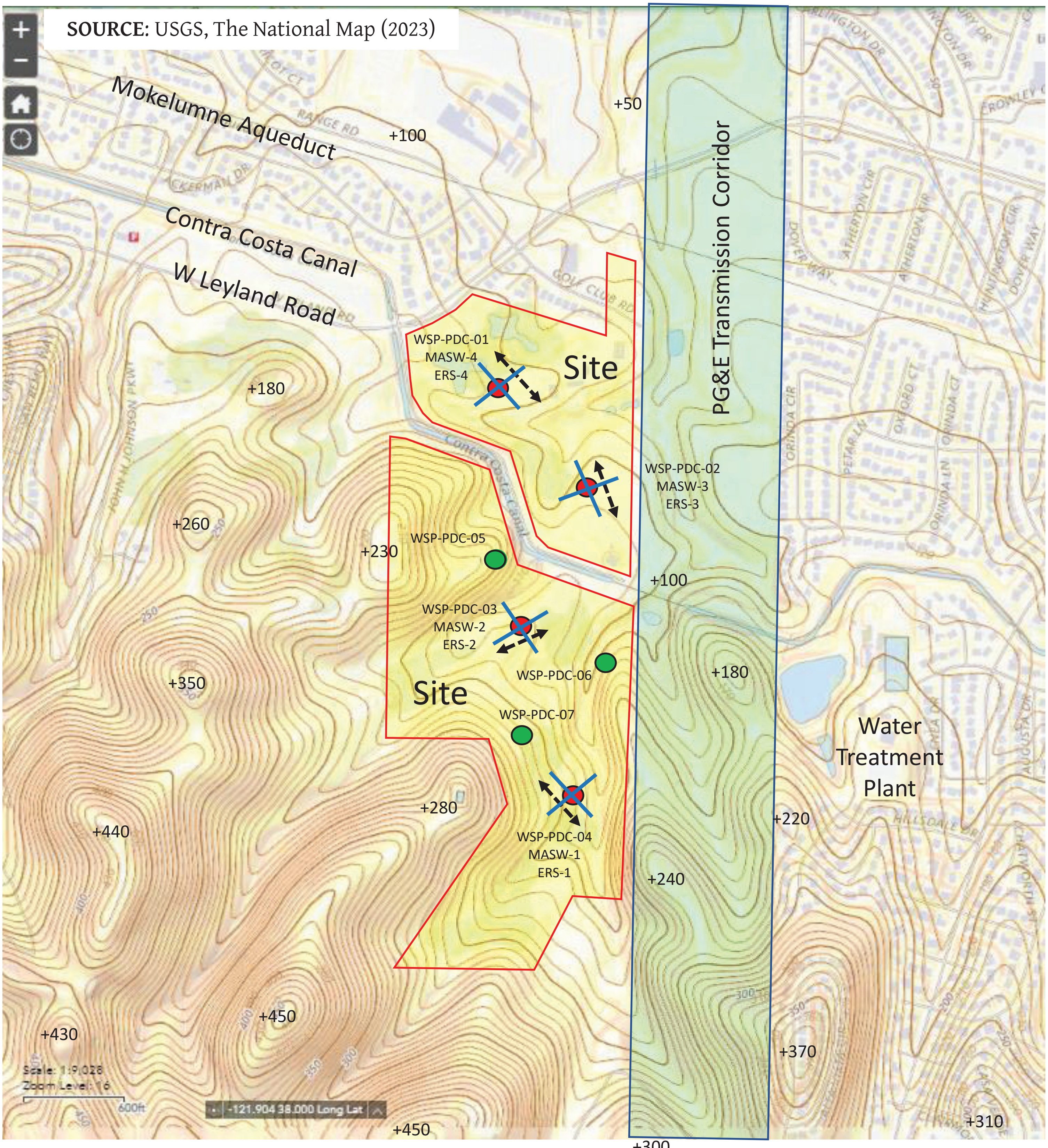
SOURCE: USGS, The National Map (2023)

WSP USA Wells Fargo Bank Building 401 B street, Suite 1650 San Diego, CA 92101-4245 Tel.: +1 619 338-9376	PROJECT NO.: 31405786.000 DRAWN BY: M. Arzamendi CHECKED BY: M. Montesi DATE: January 13, 2023	**Vicinity Map** Pittsburg Technology Center Pittsburg, California	**FIGURE** 1

SOURCE: Kimley Horn (2022)

Note: This drawing only represents one of many preliminary project concepts.
The actual quantity, size, location and elevation of buildings and features are to be determined.

	WSP USA Wells Fargo Bank Building 401 B street, Suite 1650 San Diego, CA 92101-4245 Tel.: +1 619 338-9376	PROJECT NO.: 31405786.000 DRAWN BY: M. Arzamendi CHECKED BY: M. Montesi DATE: January 13, 2023	**Preliminary Concept Layout** Pittsburg Technology Center Pittsburg, California	**FIGURE** **2**



1000 feet
Contour interval 10 feet

N
W E
S

- Approx. location of boring
- Approx. location of bulk sample
- Approx. location of MASW geophysical survey line
- Approx. location of electrical resistivity survey lines

wsp	WSP USA Wells Fargo Bank Building 401 B street, Suite 1650 San Diego, CA 92101-4245 Tel.: +1 619 338-9376	PROJECT NO: 31405786.000 DRAWN BY: M. Arzamendi CHECKED BY: M. Montesi DATE: January 13, 2023	**Topographic Map and Exploration Locations** Pittsburg Technology Center Pittsburg, California	FIGURE **3**

Imagery Date: 6/1/2013

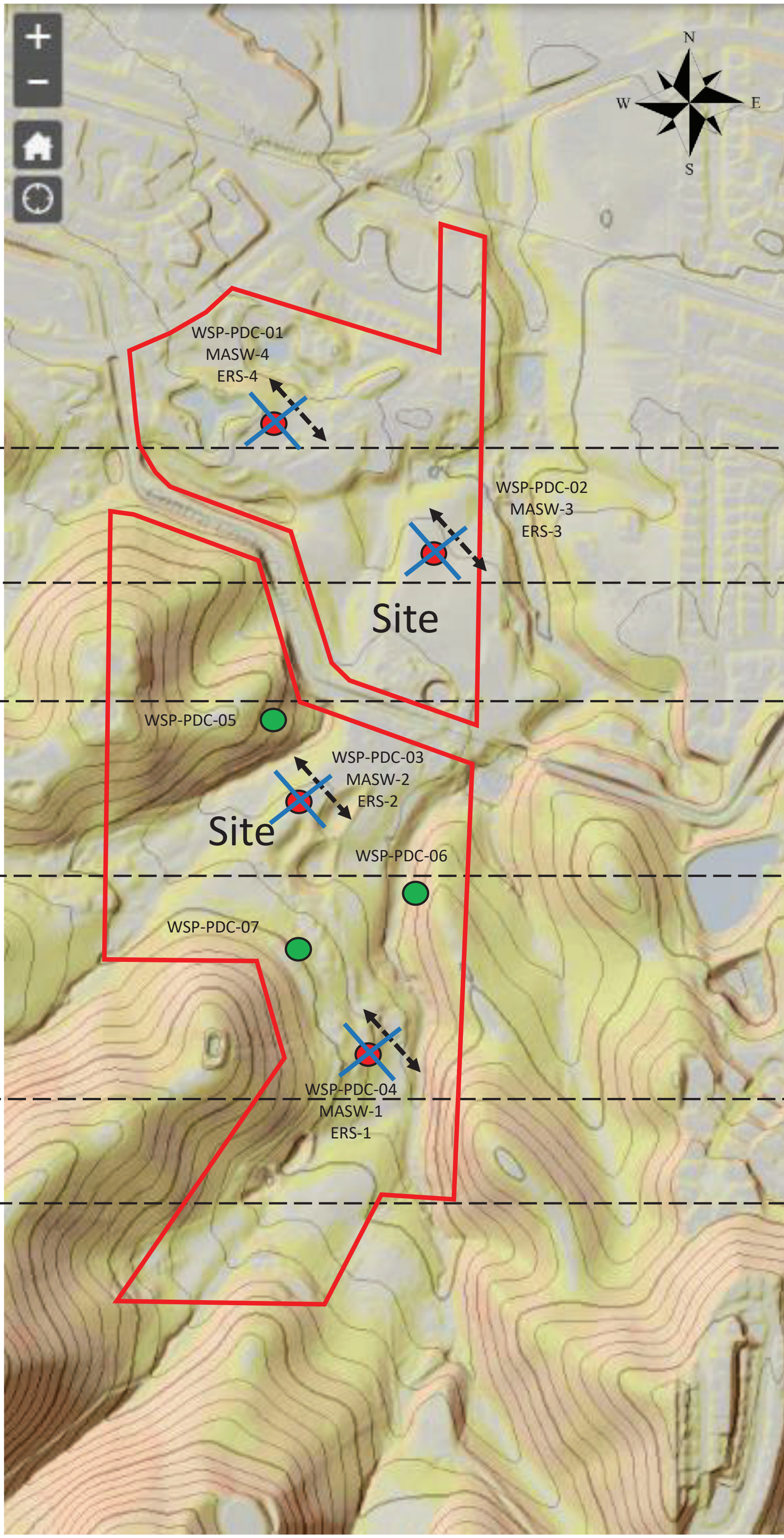
Imagery Date: 6/1/2022

1000 feet

N
W
E
S

SOURCE: Google Earth (2023)

	WSP USA Wells Fargo Bank Building 401 B street, Suite 1650 San Diego, CA 92101-4245 Tel.: +1 619 338-9376	PROJECT NO: 31405786.000	**Aerial Photographs**	**FIGURE**
		DRAWN BY: M. Arzamendi		
		CHECKED BY: M. Montesi	Pittsburg Technology Center Pittsburg, California	**4**
		DATE: January 13, 2023		



Boring
Bulk sample
MASW
Electrical Resistivity

North Section (Figure 11)

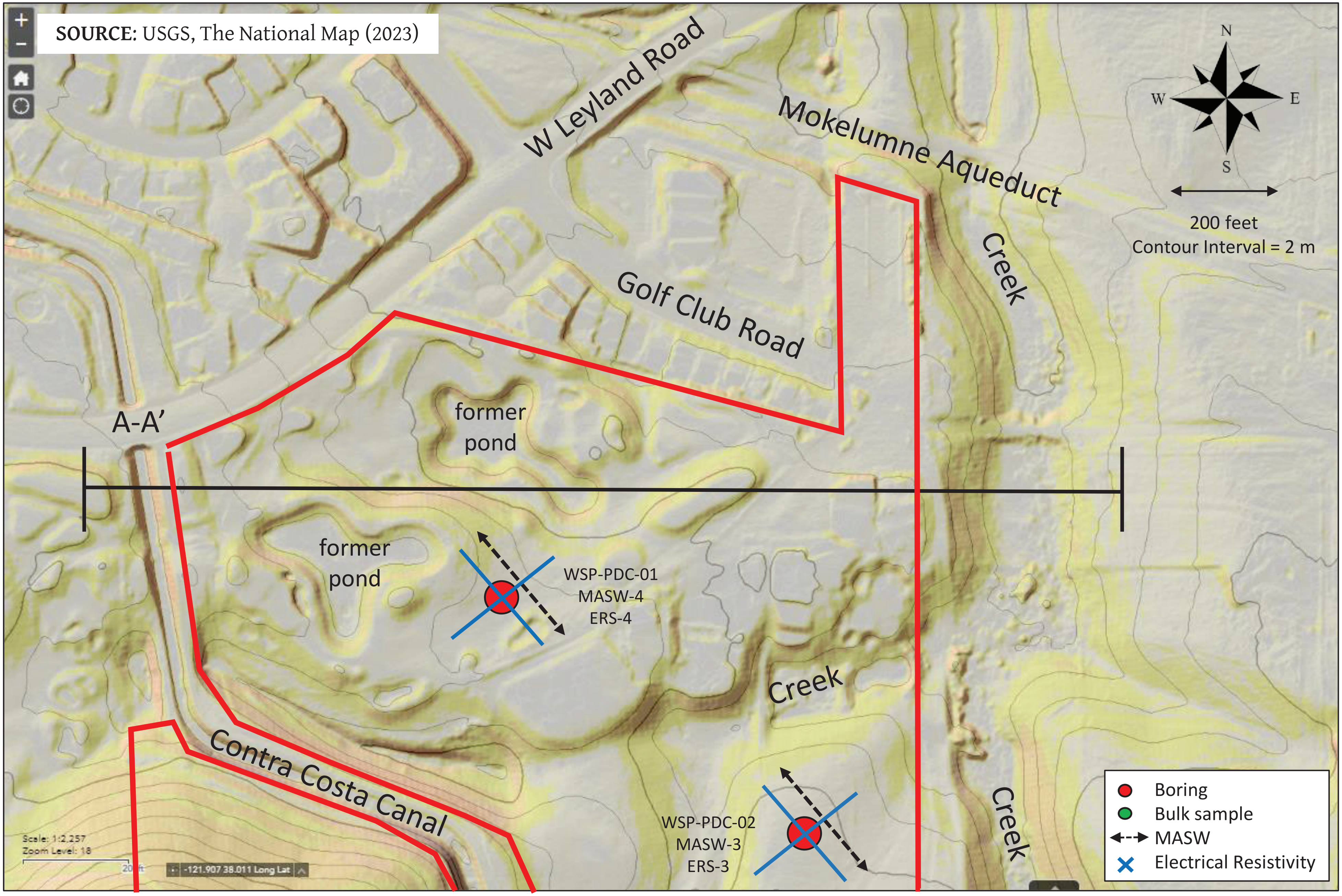
North-Central Section (Figure 12)

South Central Section (Figure 13)

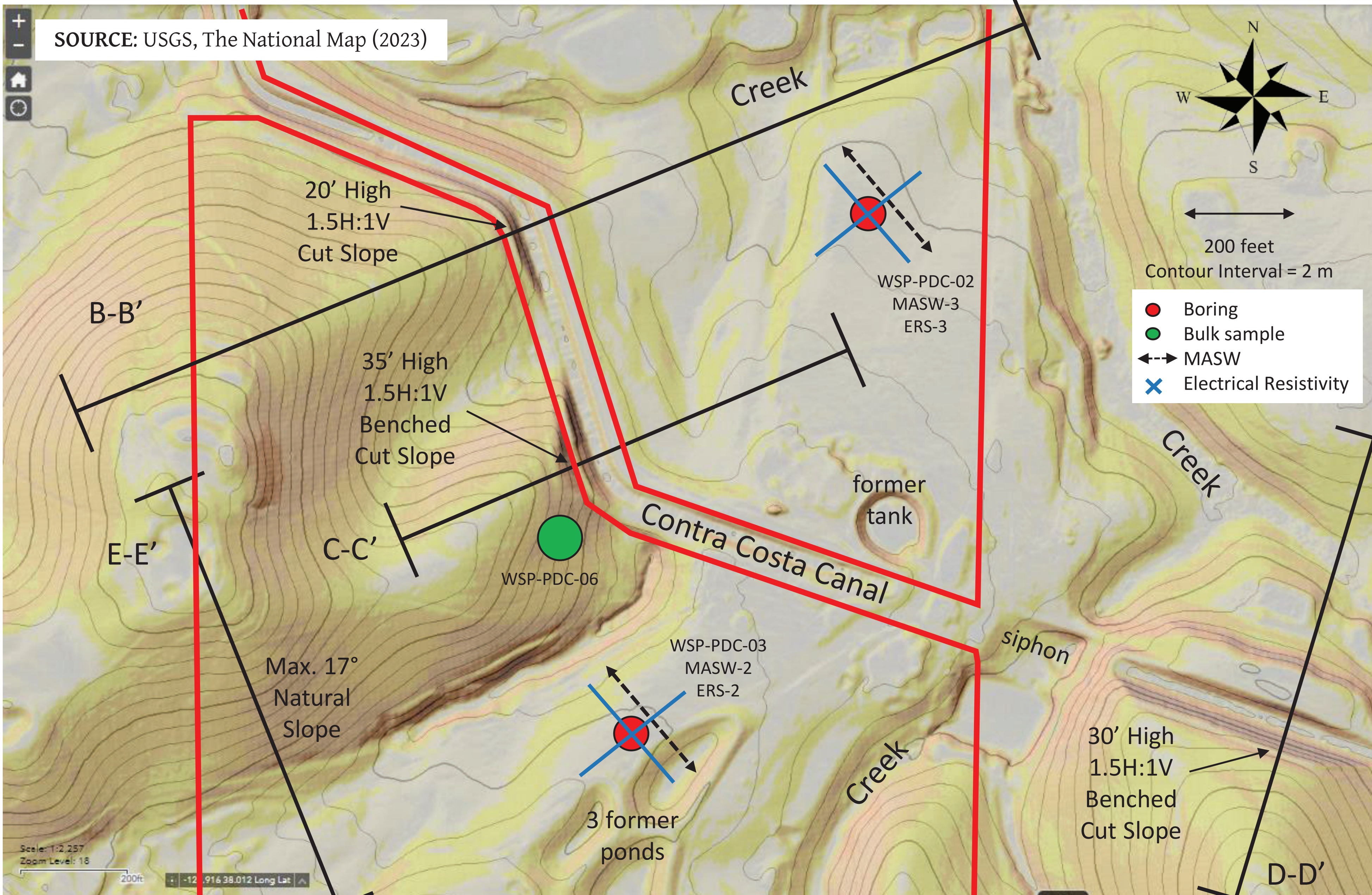
South Section (Figure 14)

1000 feet
Contour Interval = 5 m

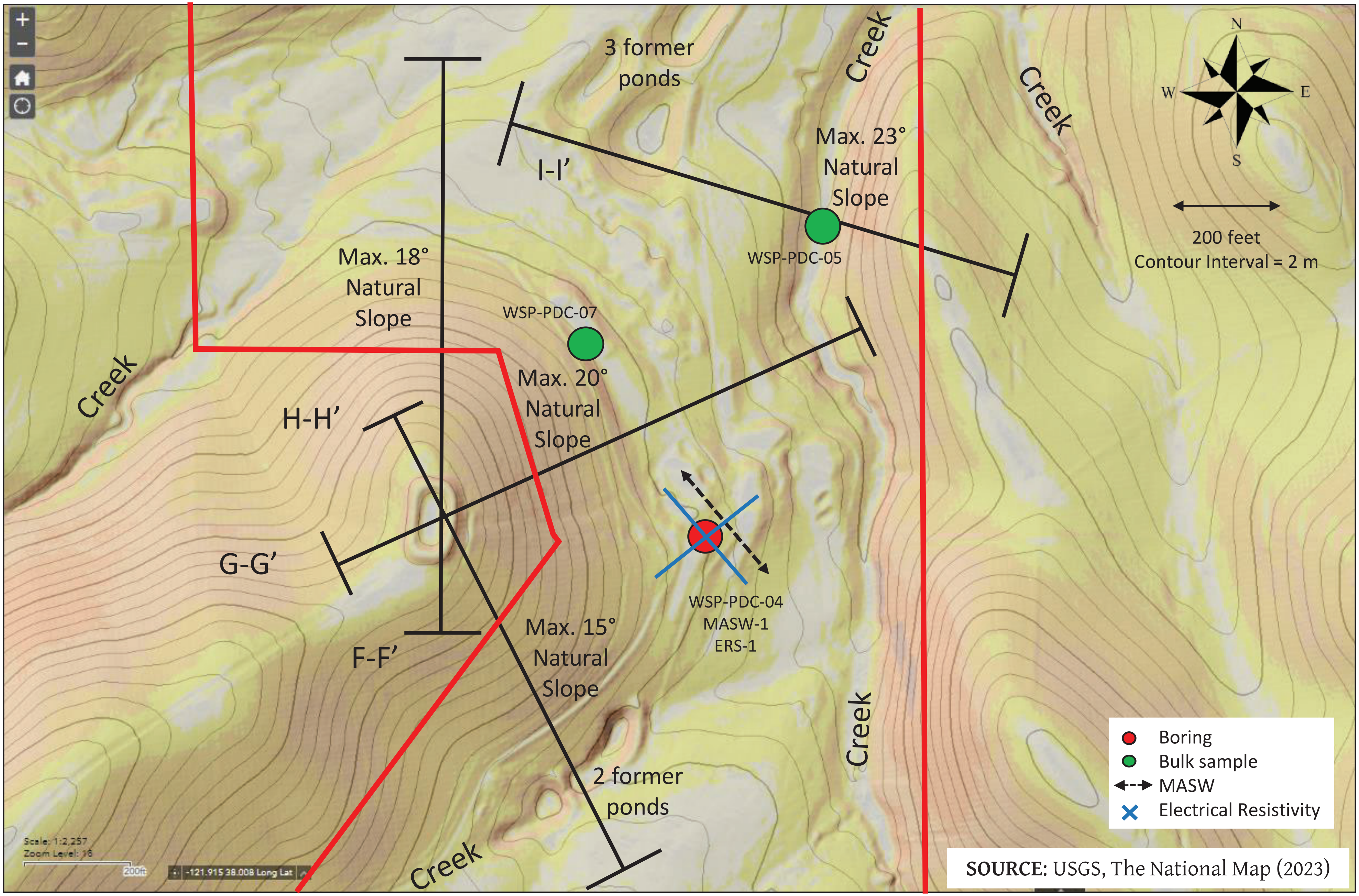
wsp	WSP USA Wells Fargo Bank Building 401 B street, Suite 1650 San Diego, CA 92101-4245 Tel.: +1 619 338-9376	PROJECT NO: 31405786.000 DRAWN BY: M. Arzamendi CHECKED BY: M. Montesi DATE: January 13, 2023	**LiDAR Based Topography and Slope Intensity Map** Pittsburg Technology Center Pittsburg, California	**FIGURE** **5**



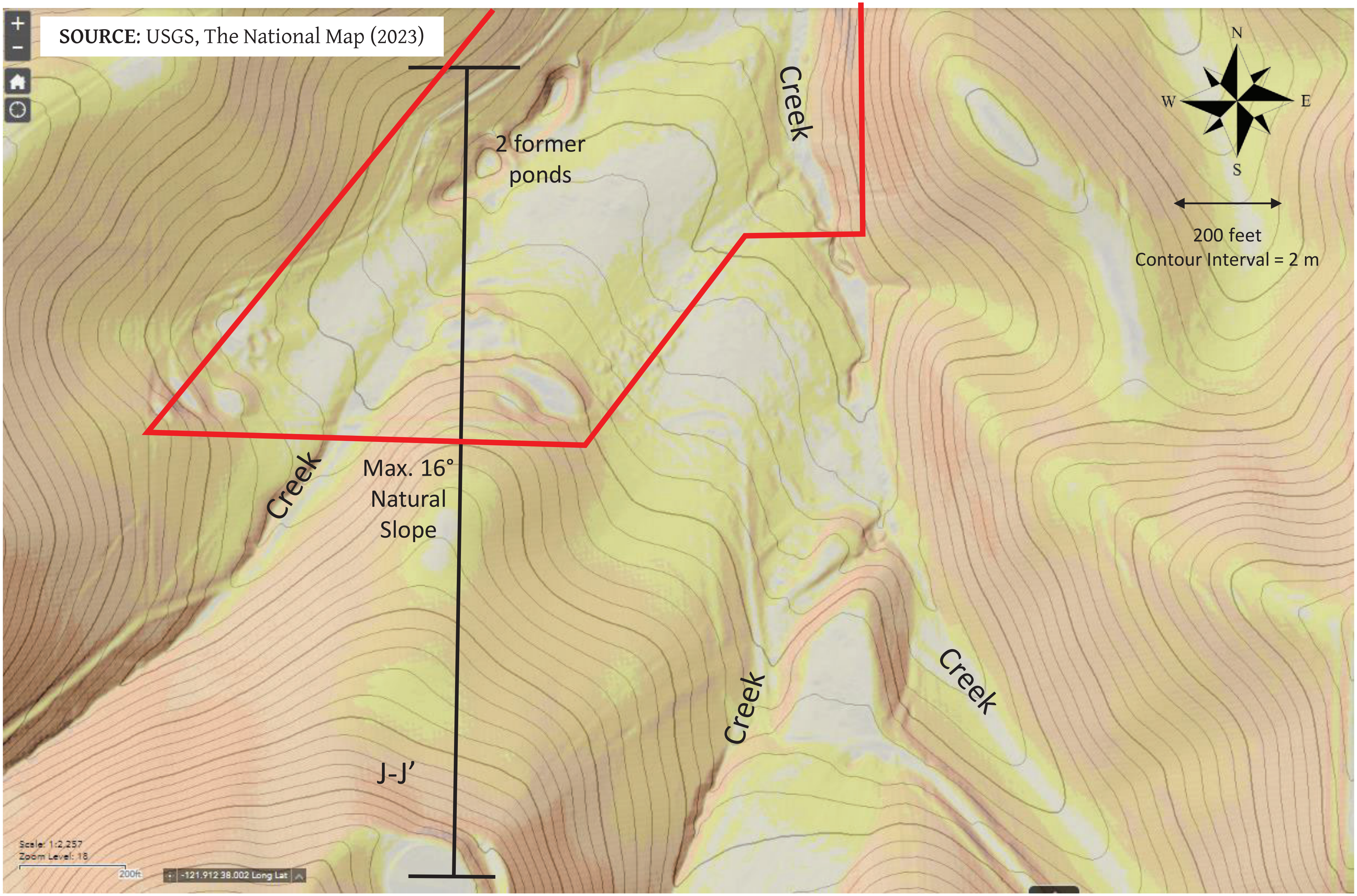
WSP USA Wells Fargo Bank Building 401 B street, Suite 1650 San Diego, CA 92101-4245 Tel.: +1 619 338-9376	PROJECT NO.: 31405786.000 DRAWN BY: M. Arzamendi CHECKED BY: M. Montesi DATE: January 13, 2023	**LiDAR Based Topography North Section** Pittsburg Technology Center Pittsburg, California	**FIGURE** **6**



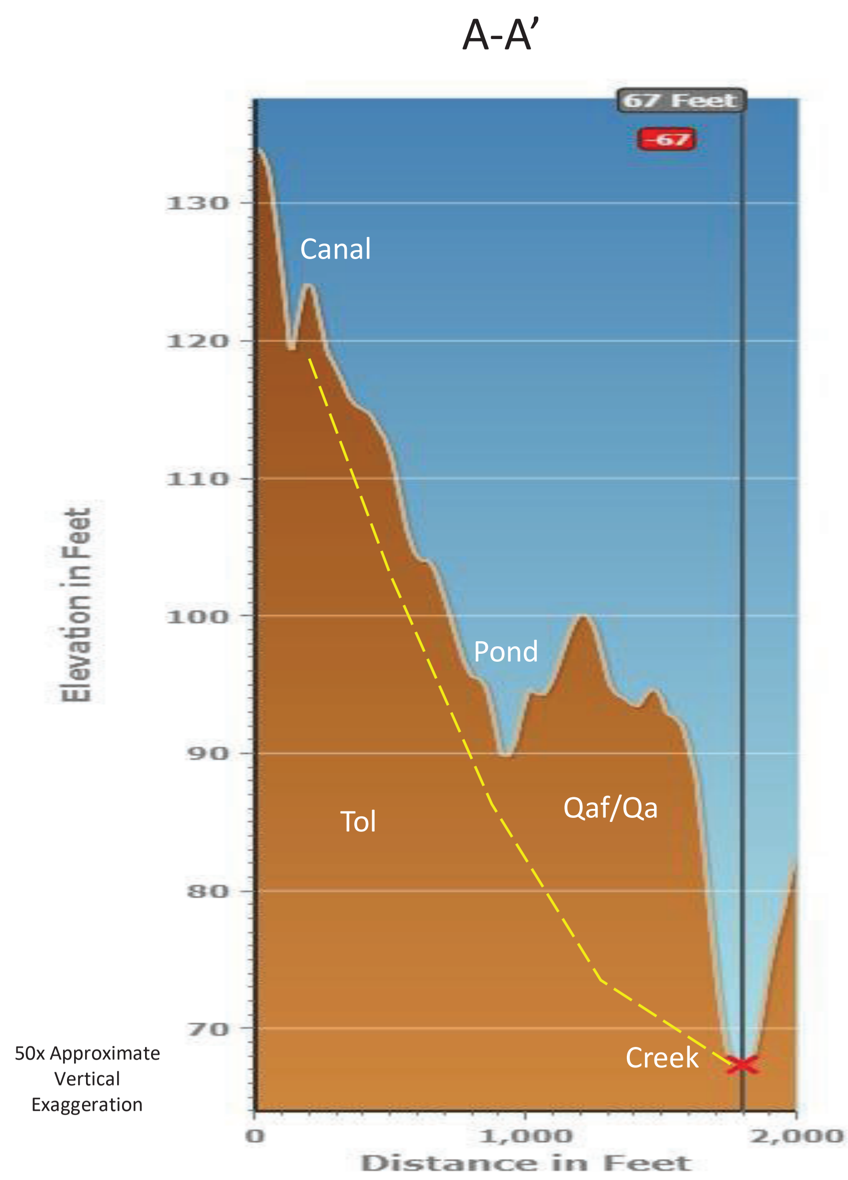
WSP	WSP USA Wells Fargo Bank Building 401 B street, Suite 1650 San Diego, CA 92101-4245 Tel.: +1 619 338-9376	PROJECT NO.: 31405786.000 DRAWN BY: M. Arzamendi CHECKED BY: M. Montesi DATE: January 13, 2023	**LiDAR Based Topography North-Central Section** Pittsburg Technology Center Pittsburg, California	**FIGURE** **7**

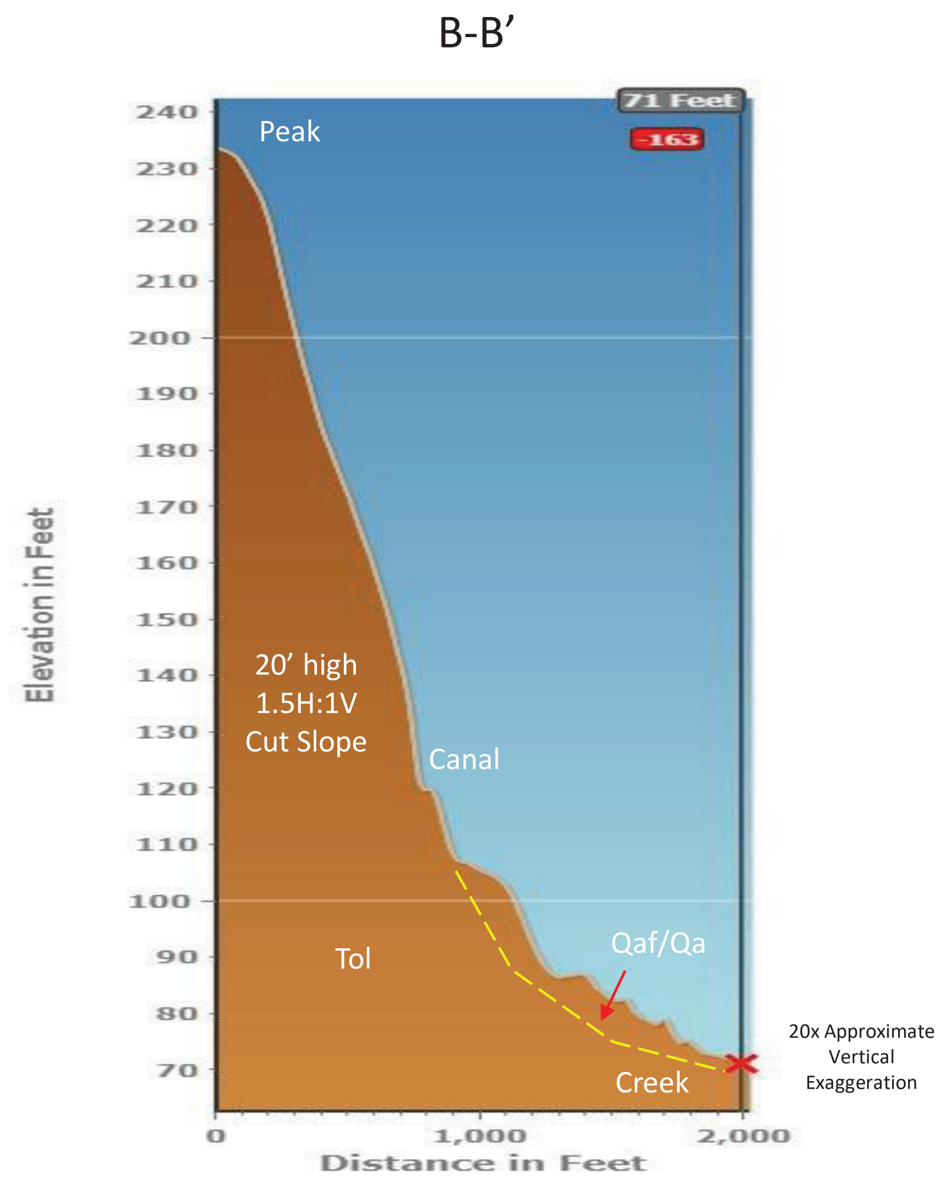


WSP	WSP USA Wells Fargo Bank Building 401 B street, Suite 1650 San Diego, CA 92101-4245 Tel.: +1 619 338-9376	PROJECT NO.: 31405786.000 DRAWN BY: M. Arzamendi CHECKED BY: M. Montesi DATE: January 13, 2023	**LiDAR Based Topography** **South Central Section** Pittsburg Technology Center Pittsburg, California	**FIGURE** **8**



WSP	WSP USA Wells Fargo Bank Building 401 B street, Suite 1650 San Diego, CA 92101-4245 Tel.: +1 619 338-9376	PROJECT NO.: 31405786.000 DRAWN BY: M. Arzamendi CHECKED BY: M. Montesi DATE: January 13, 2023	**LiDAR Based Topography** **South Section** Pittsburg Technology Center Pittsburg, California	**FIGURE** **9**

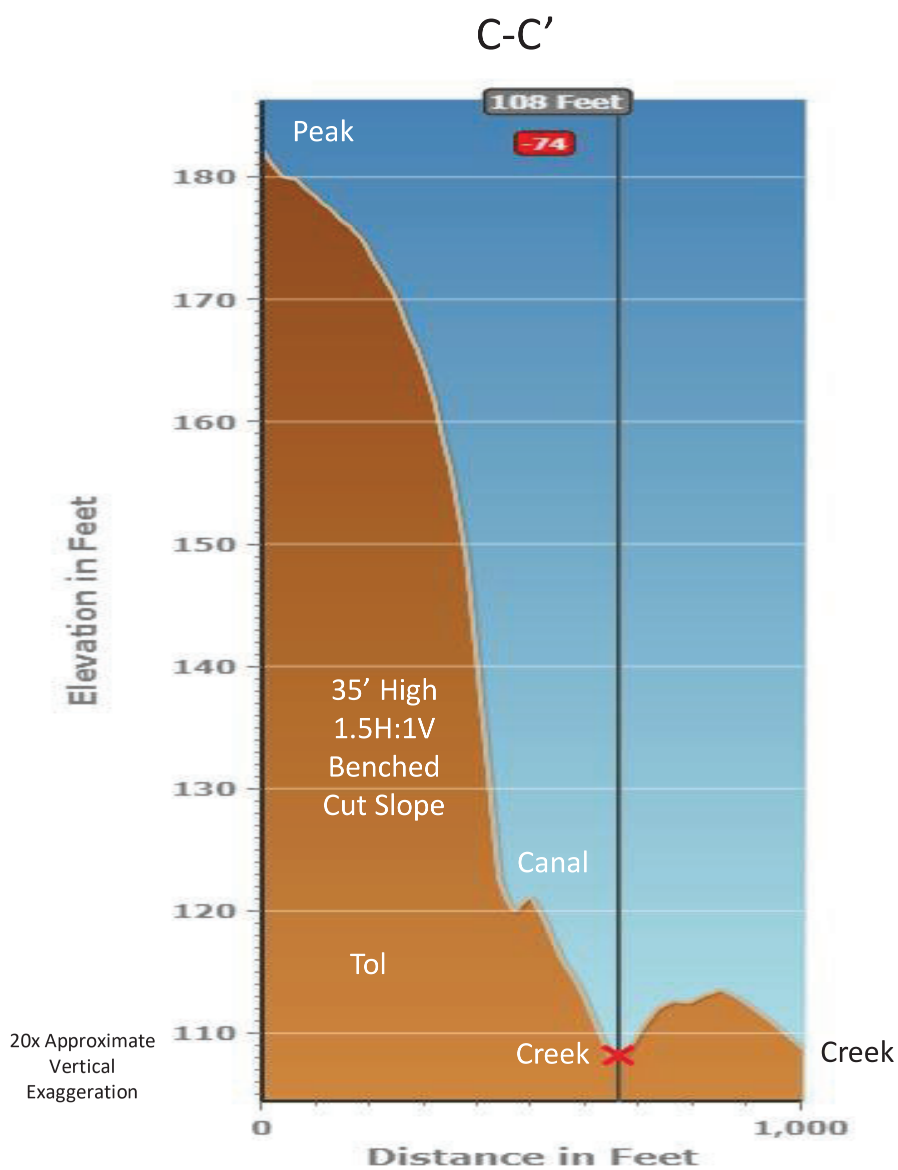


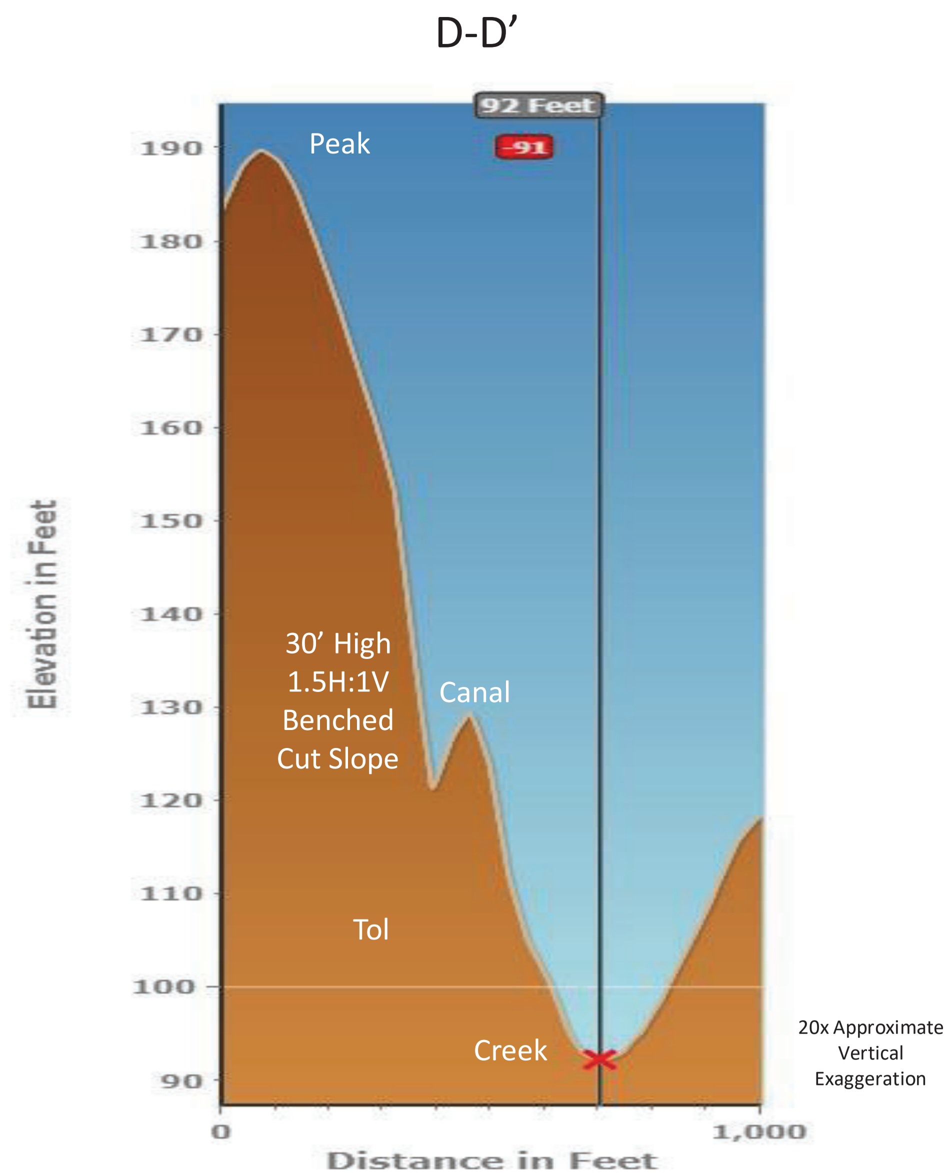


Qaf/Qa – Undifferentiated fill and alluvium
Tol – Oro Loma Formation

SOURCE: USGS, The National Map (2023)

WSP	WSP USA Wells Fargo Bank Building 401 B street, Suite 1650 San Diego, CA 92101-4245 Tel.: +1 619 338-9376	PROJECT NO.: 31405786.000 DRAWN BY: M. Arzamendi CHECKED BY: M. Montesi DATE: January 13, 2023	**Cross Sections A-A’ and B-B’** Pittsburg Technology Center Pittsburg, California	**FIGURE** **10**



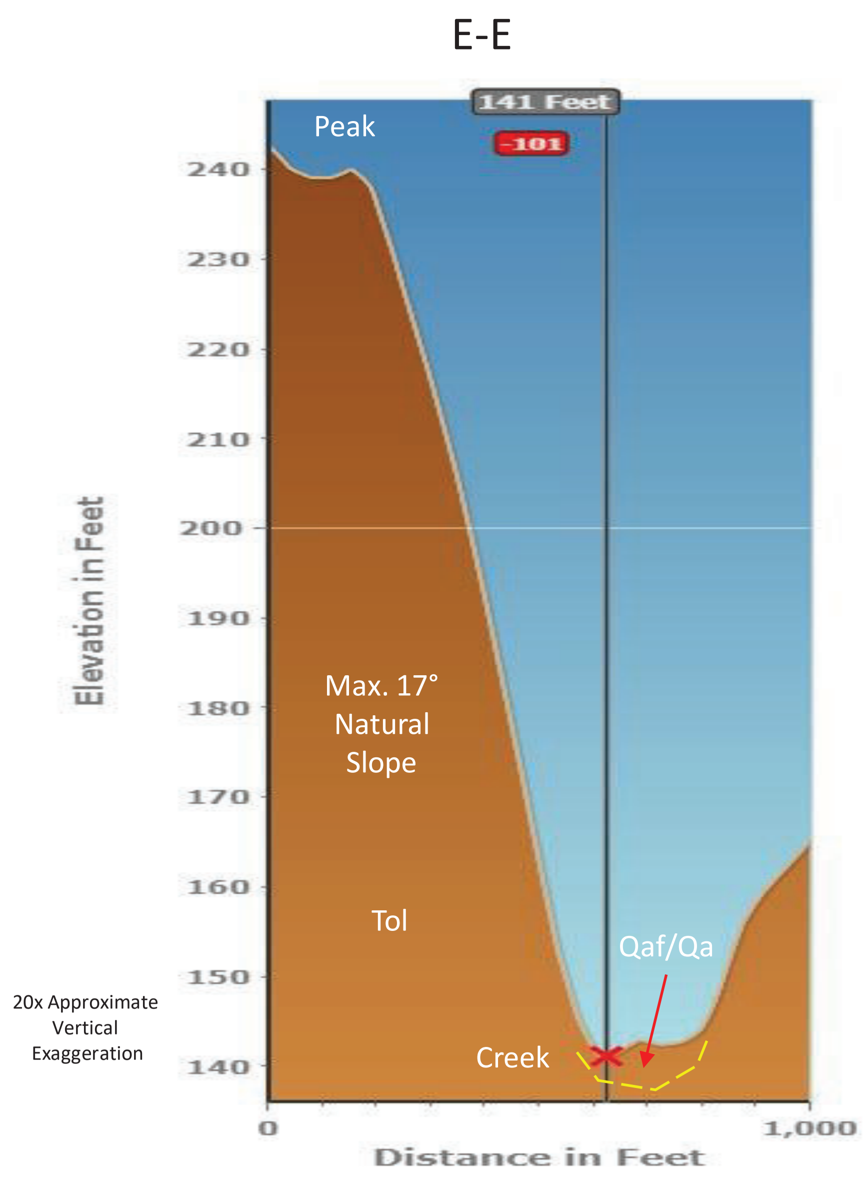


Qaf/Qa – Undifferentiated fill and alluvium
Tol – Oro Loma Formation

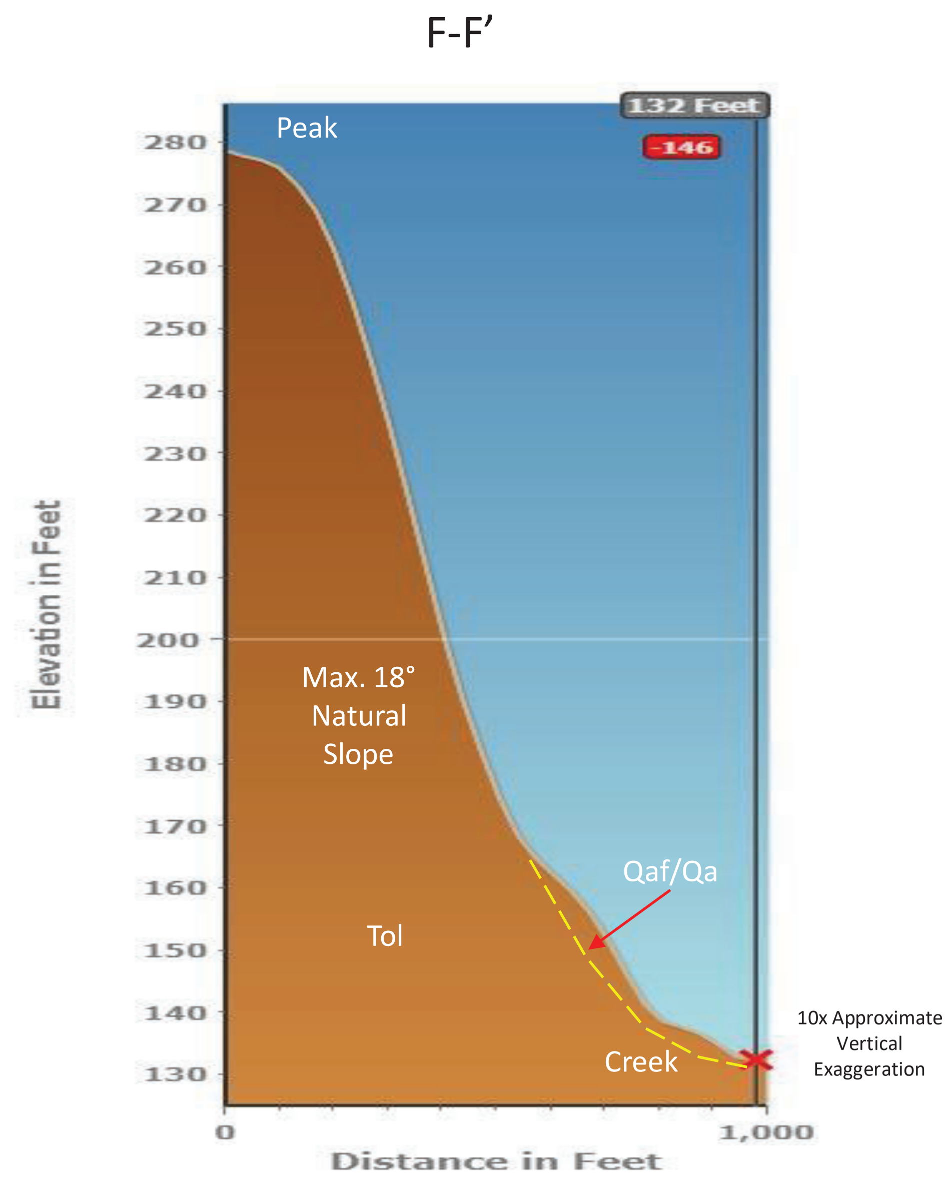
SOURCE: USGS, The National Map (2023)

wsp	WSP USA Wells Fargo Bank Building 401 B street, Suite 1650 San Diego, CA 92101-4245 Tel.: +1 619 338-9376	PROJECT NO.: 31405786.000 DRAWN BY: M. Arzamendi CHECKED BY: M. Montesi DATE: January 13, 2023	**Cross Sections C-C' and D-D'** Pittsburg Technology Center Pittsburg, California	**FIGURE** **11**

E-E



F-F'



Qaf/Qa – Undifferentiated fill and alluvium
Tol – Oro Loma Formation

SOURCE: USGS, The National Map (2023)

WSP USA
Wells Fargo Bank Building
401 B street, Suite 1650
San Diego, CA 92101-4245

Tel.: +1 619 338-9376

PROJECT NO.: 31405786.000

DRAWN BY: M. Arzamendi

CHECKED BY: M. Montesi

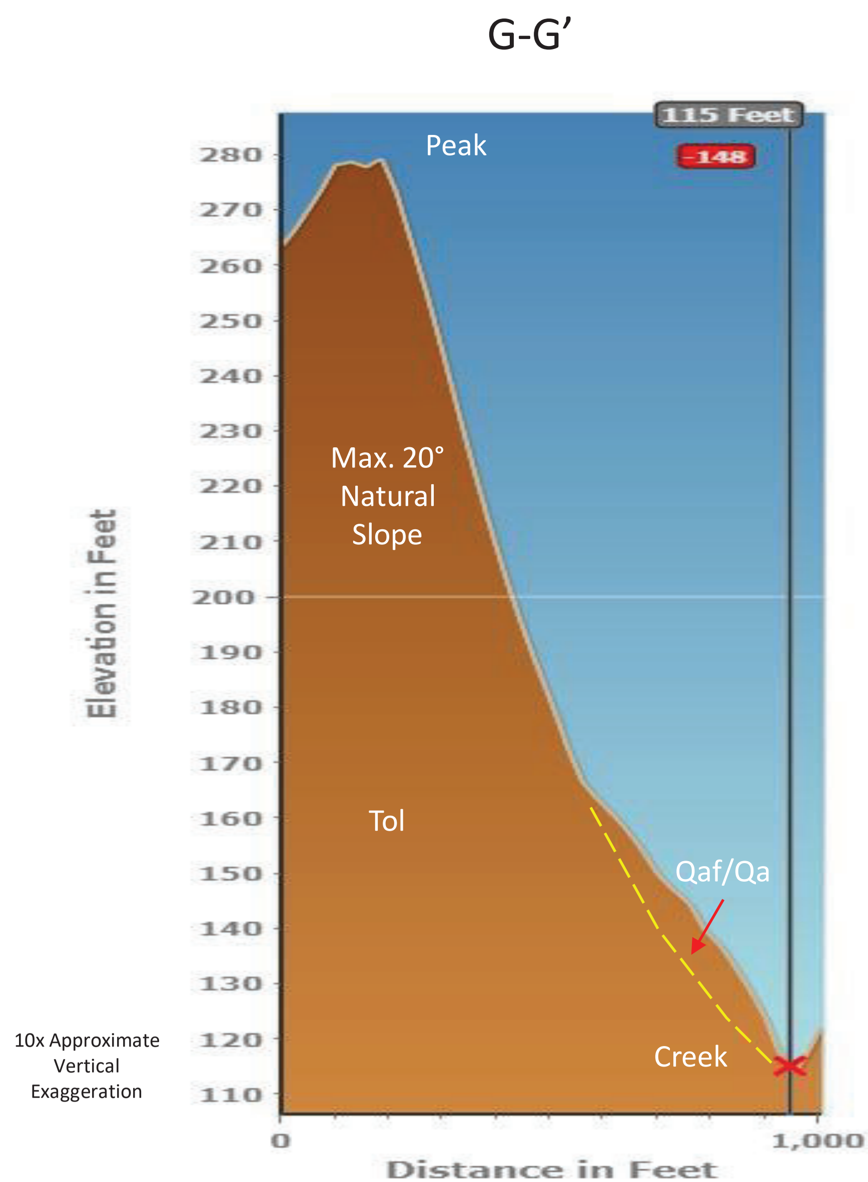
DATE: January 13, 2023

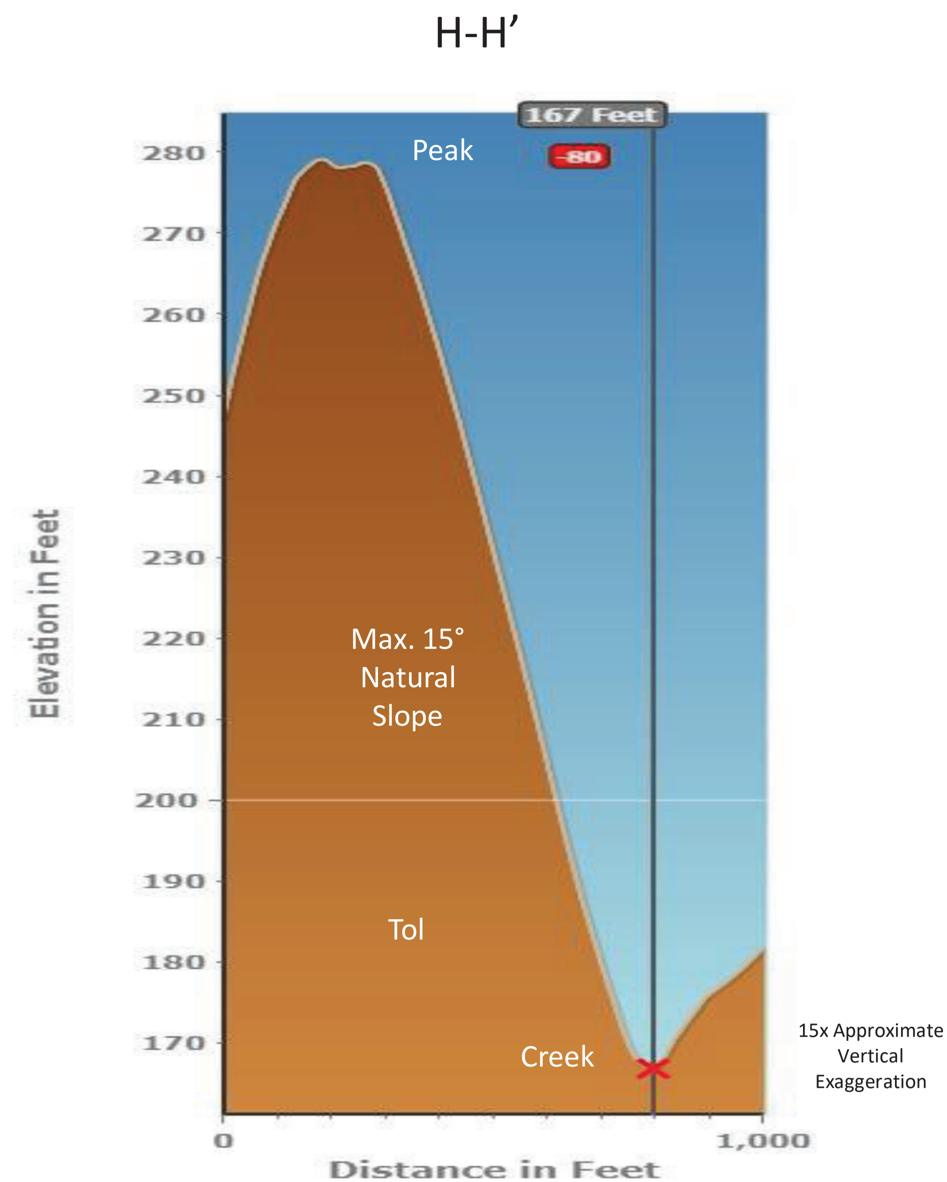
Cross Sections E-E' and F-F'

Pittsburg Technology Center
Pittsburg, California

FIGURE

12

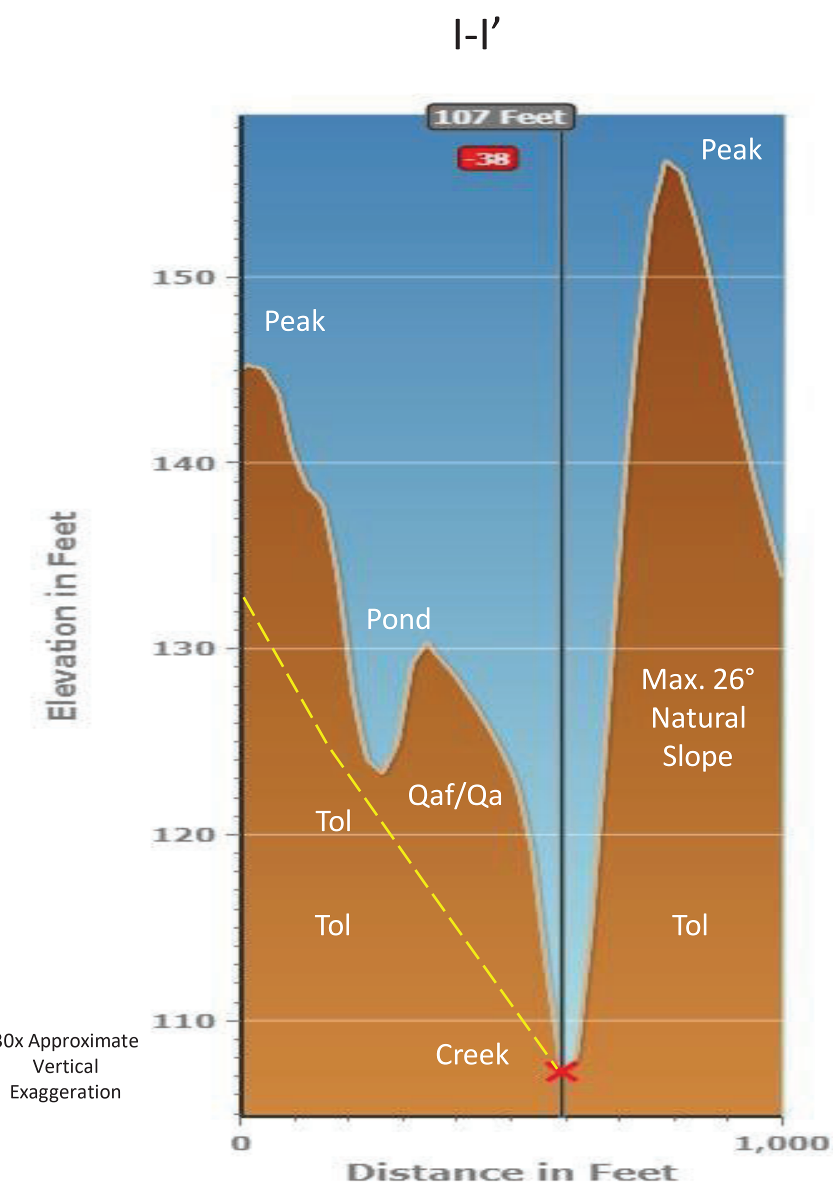


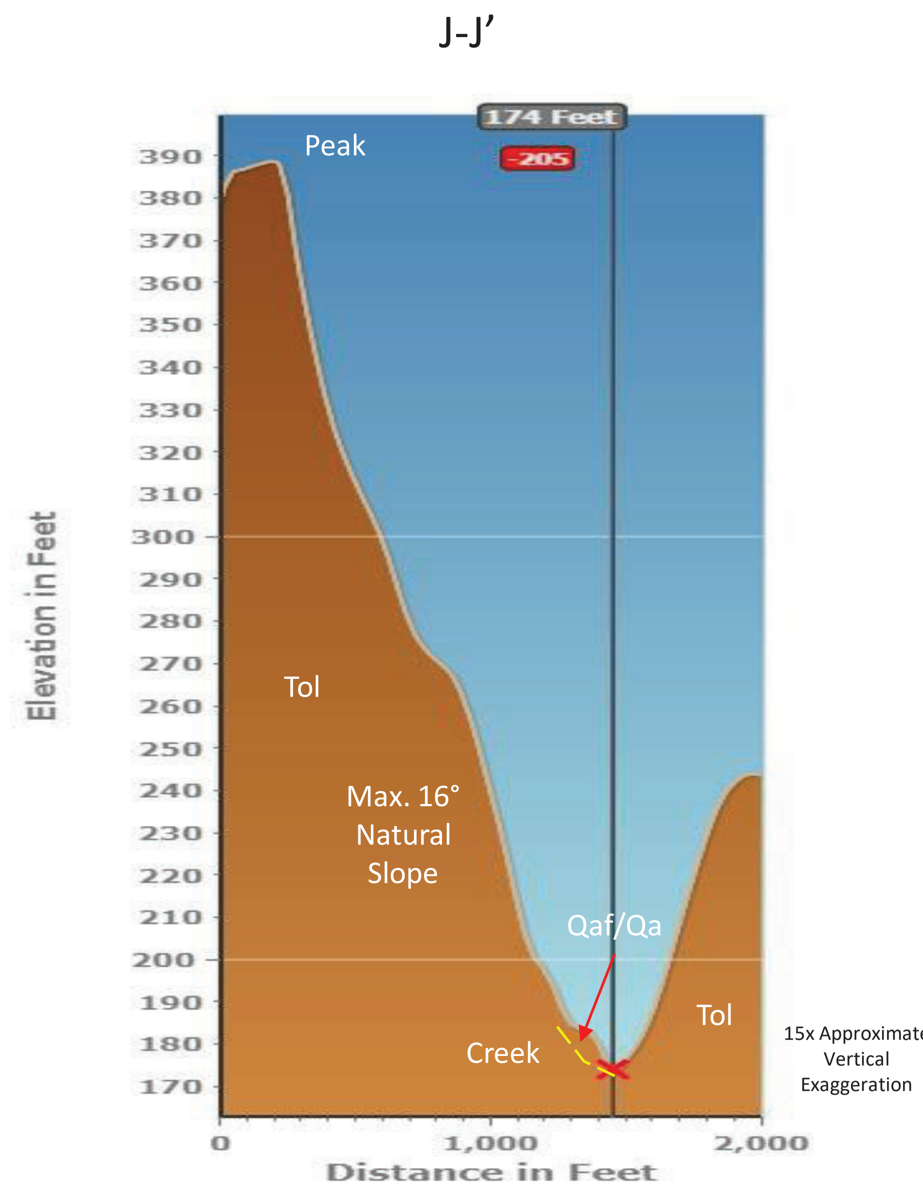


Qaf/Qa – Undifferentiated fill and alluvium
Tol – Oro Loma Formation

SOURCE: USGS, The National Map (2023)

WSP	WSP USA Wells Fargo Bank Building 401 B street, Suite 1650 San Diego, CA 92101-4245 Tel.: +1 619 338-9376	PROJECT NO.: 31405786.000 DRAWN BY: M. Arzamendi CHECKED BY: M. Montesi DATE: January 13, 2023	**Cross Sections G-G’ and H-H’** Pittsburg Technology Center Pittsburg, California	**FIGURE** **13**

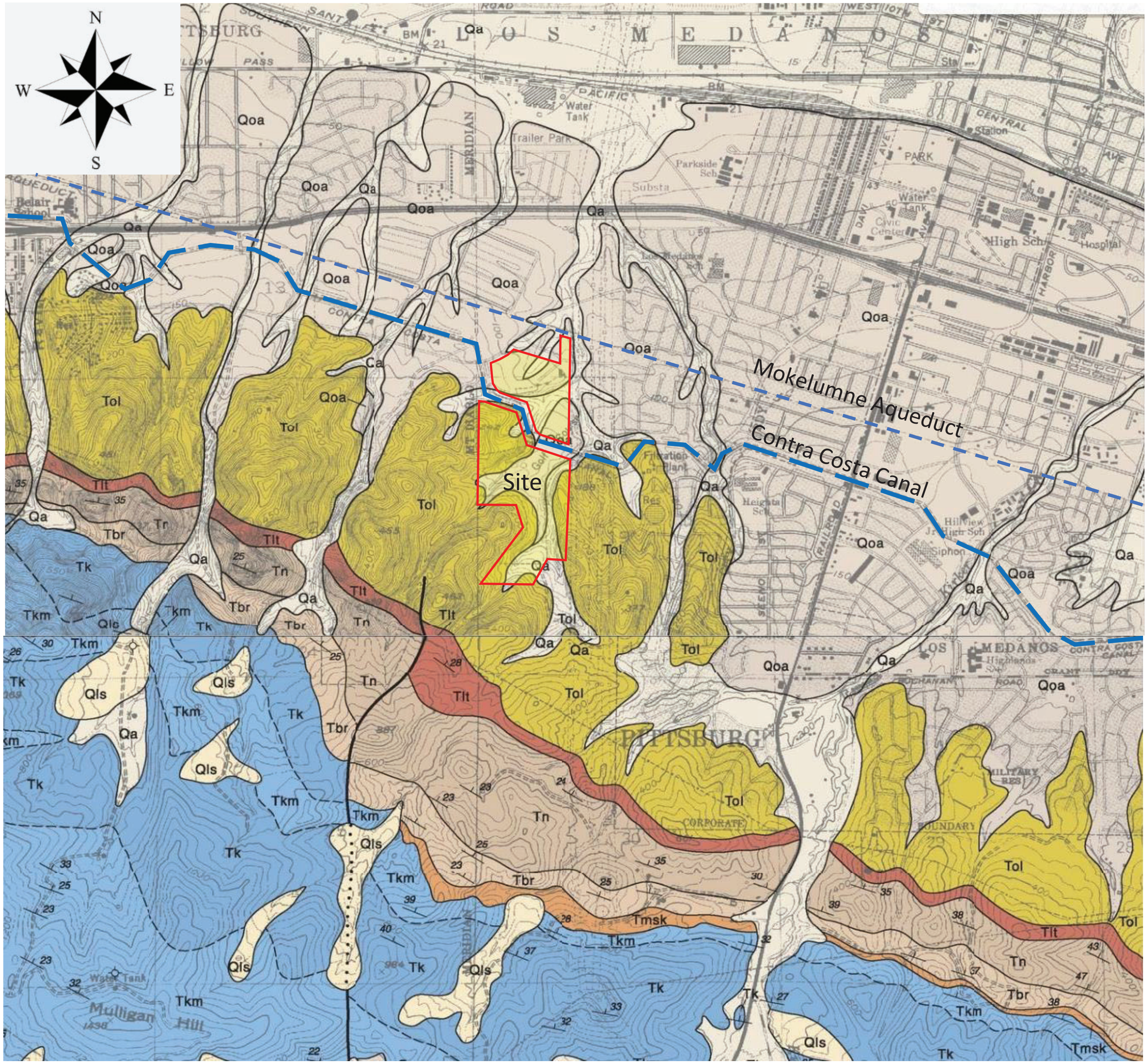




Qaf/Qa – Undifferentiated fill and alluvium
Tol – Oro Loma Formation

SOURCE: USGS, The National Map (2023)

WSP	WSP USA Wells Fargo Bank Building 401 B street, Suite 1650 San Diego, CA 92101-4245 Tel.: +1 619 338-9376	PROJECT NO.: 31405786.000 DRAWN BY: M. Arzamendi CHECKED BY: M. Montesi DATE: January 13, 2023	**Cross Sections I-I' and J-J'** Pittsburg Technology Center Pittsburg, California	**FIGURE** **14**



GEOLOGIC SYMBOLS

not all symbols shown on each map

FORMATION CONTACT dashed where inferred or indefinite, dotted where concealed

MEMBER CONTACT between units of a formation; Prominent bed

CONTACT BETWEEN SURFICIAL SEDIMENTS located only approximately in places

FAULT: Dashed where indefinite or inferred, dotted where concealed, queried where existence is doubtful. Parallel arrows indicate inferred relative lateral movement. Relative vertical movement is shown by U/D (U=upthrown side, D=downthrown side). Short arrow indicates dip of fault plane. Sawteeth are on upper plate of low angle thrust fault.

FOLDS: overturned; ANTICLINE; SYNCLINE

arrow on axial trace of fold indicates direction of plunge; dotted where concealed by surficial sediments

Strike and dip of sedimentary rocks: 18 inclined; 20 inclined (approximate); 80 overturned; horizontal; vertical

Strike and dip of metamorphic or igneous rock foliation or flow banding or compositional layers: 75 inclined; inclined (approximate); vertical; 80 overturned

OTHER SYMBOLS: Direction of landslide movement; outline of water bodies shown on map; water well; oil well; springs

See Figure 5 for Geologic Units Legend

GEOLOGIC MAP OF THE VINE HILL & HONKER BAY QUADRANGLES

CONTRA COSTA & SOLANO COUNTIES, CALIFORNIA

GEOLOGIC MAP OF THE CLAYTON QUADRANGLE

CONTRA COSTA COUNTY, CALIFORNIA

□□□□ BY THOMAS W. DIBBLEE, JR., 2006

EDITED BY JOHN A. MINCH

1 Mile

SOURCE: USGS, National Geologic Map Database (2023)

WSP USA
Wells Fargo Bank Building
401 B street, Suite 1650
San Diego, CA 92101-4245

Tel.: +1 619 338-9376

PROJECT NO.: 31405786.000	**Regional Geologic Map**
DRAWN BY: M. Arzamendi	
CHECKED BY: M. Montesi	Pittsburg Technology Center Pittsburg, California
DATE: January 13, 2023	

FIGURE

15

LEGEND

Qa

SURFICIAL SEDIMENTS

Qa *Alluvial gravel, sand and clay of valley areas*

Qls

LANDSLIDE RUBBLE

Qoa

OLDER SURFICIAL SEDIMENTS

Dissected alluvial fan deposits, undeformed, age late Pleistocene

— UNCONFORMITY —

Tol | Ttl

ORO LOMA FORMATION

Similar to Orinda Formation of Livermore basin.

Valley sediments, sublithified; age Pliocene to early Pleistocene

Tol *Pebble conglomerate, sandstone and claystone, interbedded, gray to greenish gray*

Ttl *Lawlor tuff at base, light gray pumaceous tuff breccia, radiometric age about 4 my or Pliocene*

Tb

BASALT

Tb *Black, massive, very fine grained intrusive into Markley Formation in NW area, age probably Pliocene*

Holocene ? Pleistocene ? Pliocene

QUATERNARY

Tn | Tbr | Tmc | Tms

MONTEREY AND RELATED FORMATIONS

Nonmarine and marine biogenic and clastic, lithified; age late to middle Miocene

Tn *Neroly Formation, gray to blue gray, medium to pebbly, Tmc claystone and some sandstone in SW corner, brackish marine*

Tbr *Sandstone, see above*

Tms *Sandstone, light gray to tan, medium grained, arkosic, basal unit locally called Sobrante, age middle to lower Miocene*

Tmc *Clay shale/siltstone, gray, vaguely bedded, argillaceous to sandy, includes fine grained sandstone*

Tmsk

KIRKER FORMATION

Marine clastic, lithified, age Oligocene

Tmsk *Tuffaceous sandstone in NE areas, tan-white, fine grained*

Tks | Tk | Tkm | Tkn

KREYENHAGEN FORMATION

Marine clastic, lithified; age late Eocene

Tks *Sandstone, tan, medium grained, arkosic*

Tk *Shale Member, gray, thin bedded, micaceous argillaceous to semi-siliceous, diatomaceous in upper part*

Tkm *Markley Sandstone Member, light gray to tan, fine to medium grained, arkosic, locally contains thin layers of coal*

Tkn *Nortonville Shale Member, gray to dark gray, micaceous, argillaceous*

Miocene — Oligocene

TERTIARY

CENOZOIC

SOURCE: USGS, National Geologic Map Database (2023)

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San Diego, CA 92101-4245

Tel.: +1 619 338-9376

PROJECT NO.: 31405786.000

DRAWN BY: M. Arzamendi

CHECKED BY: M. Montesi

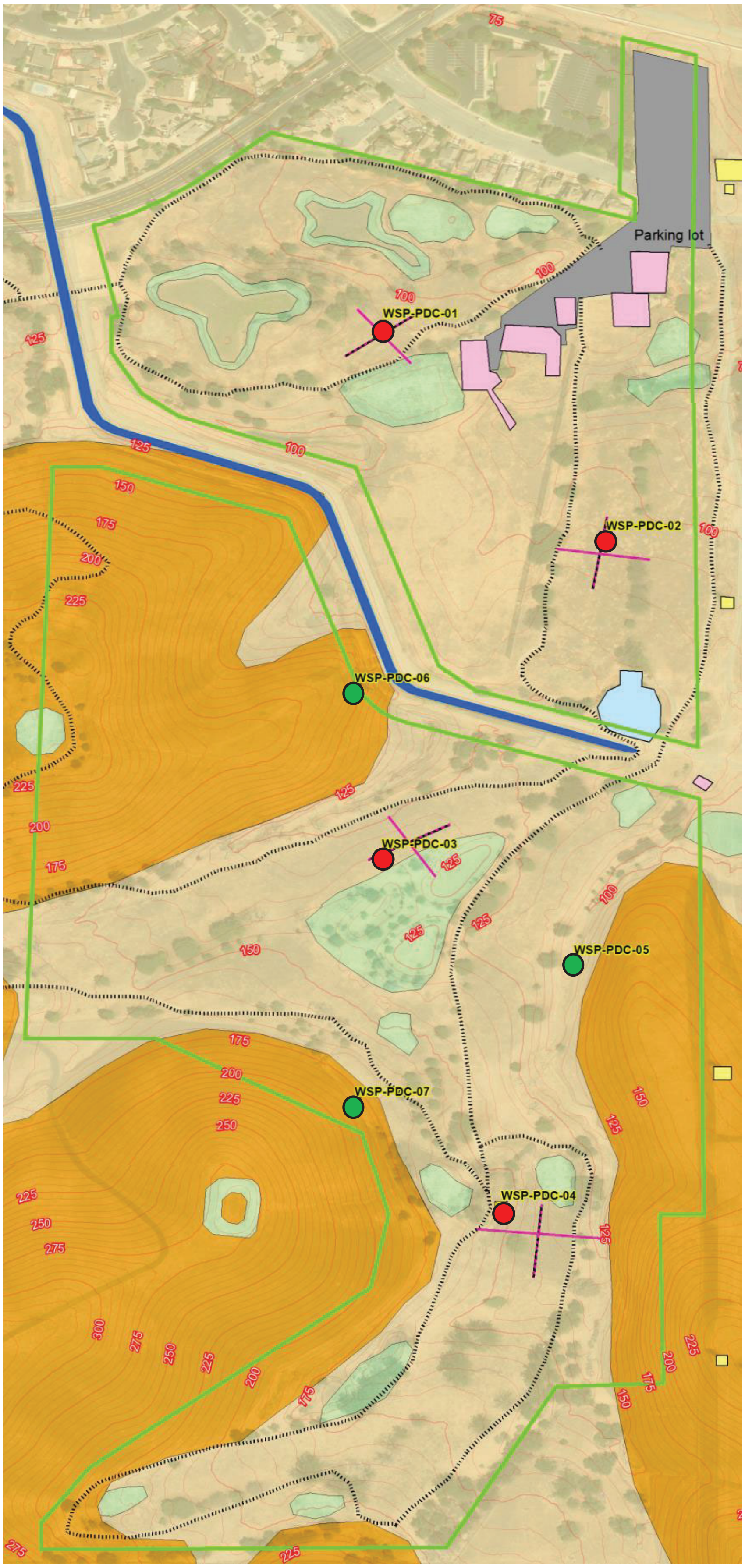
DATE: January 13, 2023

Regional Geologic Map Legend

Pittsburg Technology Center
Pittsburg, California

FIGURE

16



Legend

Boring
Bulk sample
Electrical resistivity
MASW
Project boundaries
Elevation contours

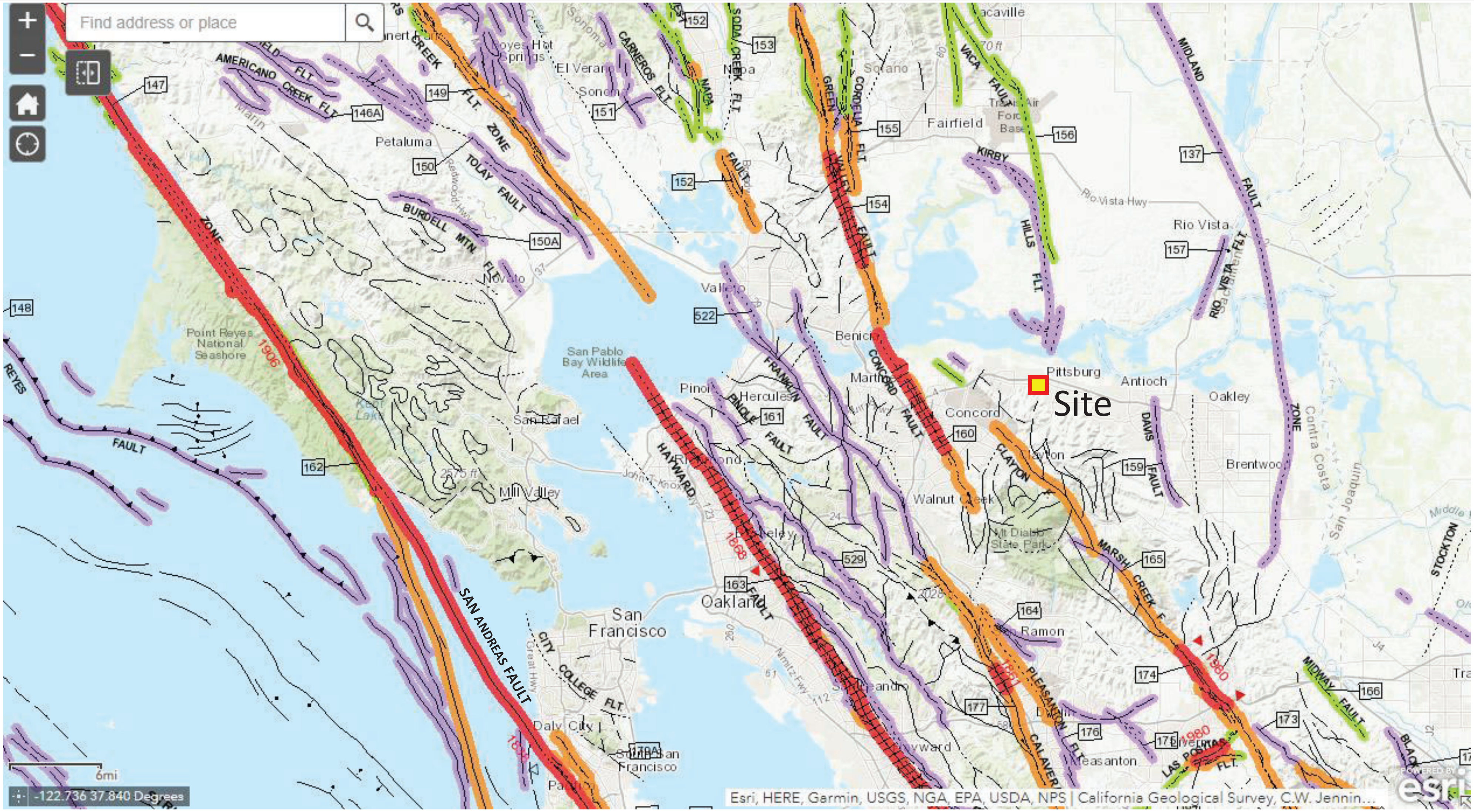
Surficial geology:

Qaf Artificial fill
Qa Alluvium
Tol Oro Loma Formation

Existing structures:

Contra Costa Canal
Foundation
PG&E Tower
Recycled water tank
Golf cart paths
Parking lot

WSP USA Wells Fargo Bank Building 401 B street, Suite 1650 San Diego, CA 92101-4245 Tel.: +1 619 338-9376	PROJECT NO: 31405786.000 DRAWN BY: E. O'Hara CHECKED BY: M. Arzamendi DATE: January 13, 2023	**Site Geologic Map** Pittsburg Technology Center Pittsburg, California	**FIGURE** **17**



Fault traces on land are indicated by solid lines where well located, by dashed lines where approximately located or inferred, and by dotted lines where concealed by younger rocks or by lakes or bays. Fault traces are queried where continuation or existence is uncertain. All offshore faults based on seismic reflection profile records are shown as solid lines where well defined, dashed where inferred, queried where uncertain.

Fault along which historic (last 200 years) displacement has occurred.

Holocene fault displacement (during past 11,700 years) without historic record.

Late Quaternary fault displacement (during past 700,000 years).

Quaternary fault (age undifferentiated).

Pre-Quaternary fault (older that 1.6 million years) or fault without recognized Quaternary displacement.

ADDITIONAL FAULT SYMBOLS

Bar and ball on downthrown side (relative or apparent).

Arrows along fault indicate relative or apparent direction of lateral movement.

Arrow on fault indicates direction of dip.

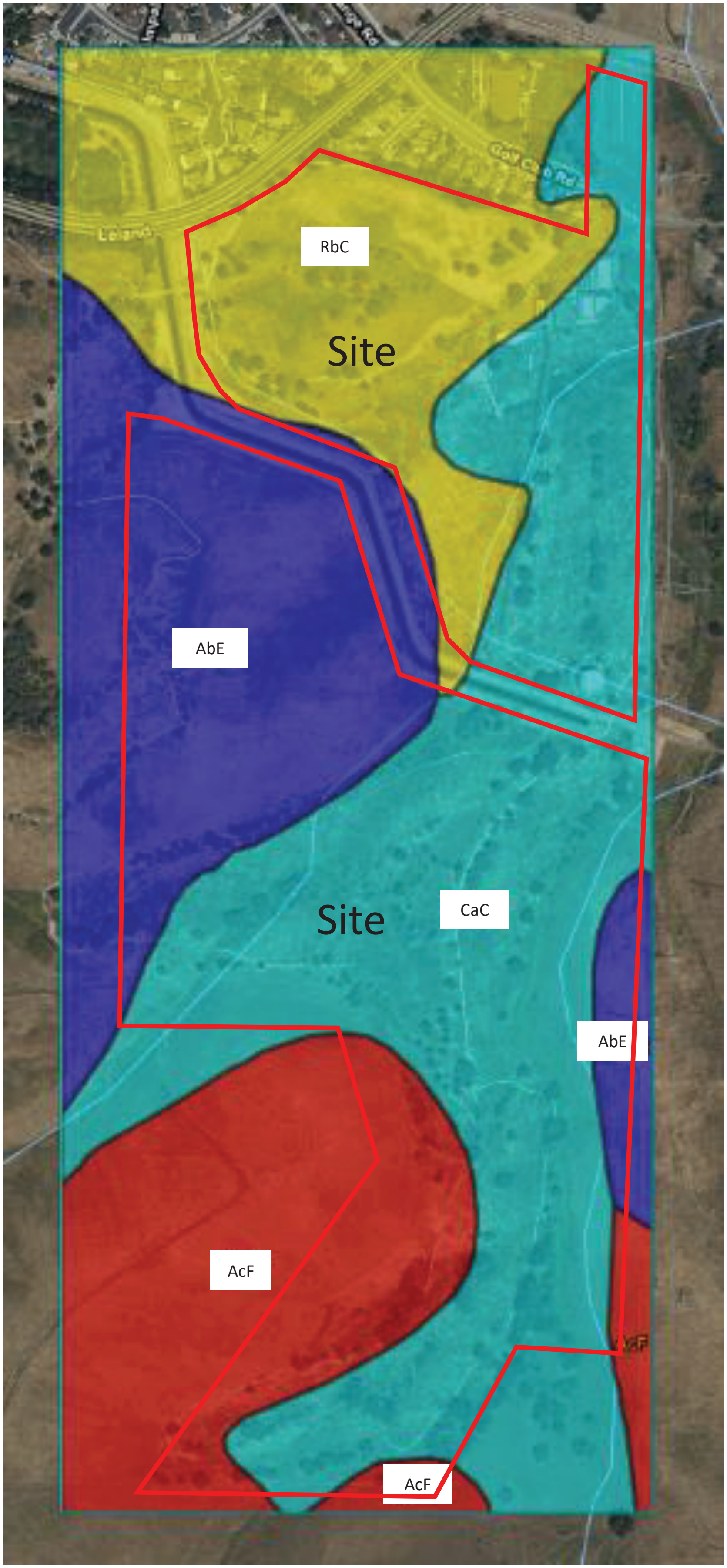
Low angle fault (barbs on upper plate).

10 miles

N E W

SOURCE: CGS, Fault Activity Map of California (2023)

wsp	WSP USA Wells Fargo Bank Building 401 B street, Suite 1650 San Diego, CA 92101-4245 Tel.: +1 619 338-9376	PROJECT NO.: 31405786.000 DRAWN BY: M. Arzamendi CHECKED BY: M. Montesi DATE: January 13, 2023	**Regional Fault Activity Map** Pittsburg Technology Center Pittsburg, California	**FIGURE** **18**



N
W E
S

Map Unit Legend

Contra Costa County, California (CA013)

Contra Costa County, California (CA013)

Map Unit Symbol	Map Unit Name	Acres in AOI	Percent of AOI
AbE	Altamont clay, 15 to 30 percent slopes, MLRA 15	40.6	22.4%
AcF	Altamont-Fontana complex, 30 to 50 percent slopes	33.0	18.2%
CaC	Capay clay, 1 to 15 percent slopes, MLRA 17	68.3	37.7%
RbC	Rincon clay loam, 2 to 9 percent slopes, MLRA 14	39.3	21.7%
Totals for Area of Interest		**181.3**	**100.0%**

1000 feet

SOURCE: USDA, Web Soil Survey (2023)

WSP USA Wells Fargo Bank Building 401 B street, Suite 1650 San Diego, CA 92101-4245 Tel.: +1 619 338-9376	PROJECT NO: 31405786.000 DRAWN BY: M. Arzamendi CHECKED BY: M. Montesi DATE: January 13, 2023	**Surficial Soils Map** Pittsburg Technology Center Pittsburg, California	**FIGURE** **19**

MAP SHOWING PRINCIPAL DEBRIS-FLOW SOURCE AREAS IN CONTRA COSTA COUNTY, CALIFORNIA

By

Stephen D. Ellen, Robert K. Mark, Gerald F. Wieczorek, Carl M. Wentworth, David W. Ramsey, and Thomas E. May
with digital cartographic assistance by Scott E. Graham, Gregg S. Beukelman, and Andrew D. Barron

1997

WSP USA Wells Fargo Bank Building 401 B street, Suite 1650 San Diego, CA 92101-4245 Tel.: +1 619 338-9376	PROJECT NO: 31405786.000 DRAWN BY: M. Arzamendi CHECKED BY: M. Montesi DATE: January 13, 2023	**Potential Debris Flow Sources** Pittsburg Technology Center Pittsburg, California	**FIGURE** **20**

CGS Seismic Hazards Program: Liquefaction Zones - Liquefaction Zones

1000 feet

N
W E
S

SOURCE: CGS, Seismic Hazards Program (2023)

WSP USA
Wells Fargo Bank Building
401 B street, Suite 1650
San Diego, CA 92101-4245

Tel.: +1 619 338-9376

PROJECT NO: 31405786.000
DRAWN BY: M. Arzamendi
CHECKED BY: M. Montesi
DATE: January 13, 2023

Liquefaction Hazard Map

Pittsburg Technology Center
Pittsburg, California

FIGURE
21

APPENDIX

A GEOPHYSICAL SURVEY RESULTS

Geophysical Report

Geophysical Investigation
Pittsburg Data Center
2232 Golf Club Road
Pittsburg, California

January 10, 2023
NORCAL JOB NO. NS225138

Prepared for:

wsp

2150 River Plaza Drive, Suite 400
Sacramento, California 95833

Prepared by:

NORCAL GEOPHYSICAL CONSULTANTS INC.
A Terracon COMPANY

January 10, 2023

2150 River Plaza Drive, Suite 400
Sacramento, California 95833

Subject: Geophysical Investigation
Pittsburg Data Center
2232 Golf Club Road
Pittsburg, California

NORCAL Project No. NS225138

Attention: Ms. Rachel Reardon

Dear Ms. Reardon,

This report presents the findings of a geophysical investigation consisting of the seismic multi-channel analysis of surface waves (MASW) and electrical resistivity sounding (ERS) survey methods. The work was performed by NORCAL Geophysical Consultants, Inc. a Terracon Company (NORCAL), for WSP USA Inc. (WSP) for the planned data center near 2232 Golf Club Road in Pittsburg, California. We understand that the results of the MASW survey will be used to aid in assessing the Seismic Site Class and the VES results will be used to help determine parameters for electrical grounding grids.

This work was authorized under a WSP Project-Specific Subcontractor Services Agreement dated December 13, 2022. Professional Geophysicist David T. Hagin (CA PGp No. 1033), Senior Geophysical Technician Travis W. Black and Staff Geophysicist Matthew LaRiviere performed the survey on December 19 and 20, 2022.

The scope of NORCAL's services for this project consisted of using geophysical methods to characterize the subsurface. The accuracy of our findings is subject to specific site conditions and limitations inherent to the techniques used. We performed our services in a manner consistent with the standard of care ordinarily exercised by members of the profession currently employing similar methods. No warranty, with respect to the performance of services or products delivered under this agreement, expressed or implied, is made by NORCAL.

We appreciate having the opportunity to provide our services for this project. If you have any questions or require additional geophysical services, please do not hesitate to call on us.

Sincerely,
NORCAL Geophysical Consultants, Inc.

David Hagin

David T. Hagin
California Professional Geophysicist
PGp 1033

PROFESSIONAL GEOPHYSICIST
DAVID T HAGIN
No. 1033
STATE OF CALIFORNIA

1-10-2023

Donald J. Kirker

Donald J. Kirker, Reviewer
California Professional Geophysicist
PGp No. 997

PROFESSIONAL GEOPHYSICIST
DONALD KIRKER
No. 997
STATE OF CALIFORNIA

1-10-2023

1.0 INTRODUCTION

It is our understanding that a new data center and other possible structures are planned for development at the subject site. For this investigation, two geophysical survey methods were used in support of the planning stages of this development. They are the seismic multi-channel analysis of surface waves (MASW) and electrical resistivity sounding (ERS) methods.

2.0 SCOPE OF WOR□

Our scope of work included acquiring MASW and ERS data at each of four locations, as determined by WSP. The MASW consisted of a single sounding and the ER comprised two soundings in a cross formation at each location. The MASW soundings are designated as MASW-1 through MASW-4 and the ER soundings as ERS-1 through ERS-4, as shown overlain on an aerial photographic image on **Plate 1 – Site Location Map**.

To provide documentation of our investigation, this report includes details of the instrumentation, data acquisition and processing, the layered one-dimensional (1D) MASW (shear-wave) and ERS (electrical resistivity) models as well as the site location map.

3.0 SITE CONDITIONS

The following description of site conditions is derived from our observations during the survey and a review of publicly available aerial photographs, geologic and topographic maps.

Item	Description
Site Information	The site is located at or near 2232 Golf Club Road in Pittsburg, California. The coordinates of the approximate center of the site are: (38°00'32.1"N 121°54'39.8"W).
Current Conditions	The site was previously a golf course, and remnants of the course are visible in many locations. Several pathways extend along the site in the north-south direction and the Contra Costa Canal roughly bisects the site into northern and southern portions. The geophysical surveys were conducted in areas covered with soil and moderate to tall dry grass.
Existing Topography	Based on our Trimble Geo-7X GPS and Google Earth, the survey area consists of gently rolling terrain with surface elevations ranging from roughly 90- to 150-ft (NAVD88).
Site Geology	Available geologic maps (USGS, 2003; CGS 2010) indicate that the shallow site geology consists of Quaternary age older alluvium.

4.0 MASW SURVEY

4.1 METHODOLOGY

The MASW survey determines the shear-wave velocities of the subsurface as a function of depth. The survey method is a sounding, producing 1D data that are presented in both tabular form and as a step-chart graph representing the layered shear-wave model produced. The location of an MASW sounding is considered to be the center of the geophone array. The MASW results are presented by the step-chart graphs on **Plates 2 through 5 – MASW Sounding**. Descriptions of the MASW methodology, our data acquisition and analysis procedures, and the instrumentation we employed are provided in **Appendix A – MASW Survey**.

4.2 RESULTS

The orientations of the seismic arrays for MASW-1 and -3 were S-N, as shown on Plate 1. MASW-2 and -4 were oriented SW-NE. The results of the MASW sounding survey are listed below in Tables A through D. The left columns contain the depth range for each layer (feet below ground surface) and the right columns comprise the associated shear (S-) wave values in feet per second (ft/sec). The results are also presented graphically by the step charts shown on **Plates 2 through 5 – MASW Sounding**. On each plate, the vertical axis represents depth below ground surface in feet. The horizontal axis indicates the shear-wave velocity in feet per second.

Table A : *MASW-1: Seismic S-Wave Velocity vs Depth*

DEPTH RANGE (FT)	S-WAVE VELOCITY (FT/SEC)
0 - 2	700
2 - 5	660
5 - 9	600
9 - 14	700
14 - 19	780
19 - 27	720
27 - 36	640
36 - 47	1,120
47 - 61	1,410
61 - 100	1,310

Vs(30) = 977 fps

Table B : *MASW-2: Seismic S-Wave Velocity vs Depth*

DEPTH RANGE (FT)	S-WA□E □ELOCITY (FT/SEC)
0 - 5	820
5 - 11	700
11 - 18	1,060
18 - 27	1,160
27 - 38	930
38 - 53	1,030
53 - 70	1,210
70 - 93	1,100
93 - 100	890

Vs(30) = 1018 fps

Table C : *MASW-3: Seismic S-Wave Velocity vs Depth*

DEPTH RANGE (FT)	S-WA□E □ELOCITY (FT/SEC)
0 - 4	730
4 - 8	570
8 - 14	670
14 - 20	1,060
20 - 29	1,050
29 - 40	880
40 - 53	1,130
53 - 70	1,300
70 - 92	1,050
92 - 100	1,140

Vs(30) = 993 fps

Table D : *MASW-4: Seismic S-Wave Velocity vs Depth*

Vs(30) = 936 fps

DEPTH RANGE (FT)	S-WAVE VELOCITY (FT/SEC)
0 - 3	840
3 - 10	850
10 - 15	780
15 - 22	710
22 - 30	730
30 - 40	920
40 - 52	1,070
52 - 68	830
68 - 100	1,210

The calculated Vs values from MASW- 1 through -4 range from a low of 570 ft/sec to a maximum of 1,310 ft/sec. The measured shear-wave values are relatively low. The values generally increase with increasing depth; however, velocity inversions (decreasing Vs with depth) are apparent on all four soundings.

The standard method of reporting MASW data is to consider the location of the 1D velocity vs. depth model as the center point of the MASW array. However, this does not mean that the measured velocity values represent materials solely beneath that location. In fact, the subsurface conditions underlying the entire length of the array, and for several tens of feet to either side, contribute to the measured velocity values.

5.0 ERS SURVEY

5.1 METHODOLOGY

The ERS survey, using the Wenner 4-Pin method, measures the Electrical Resistivity (ER) of the shallow sub-surface. The four "pins" (electrodes) are arranged in a collinear array. Current is transmitted between the outer two electrodes and the resulting voltage is measured across the inner two electrodes. Readings were taken with electrode separations (a-spacings) of 1-, 2-, 4-, 8-, 15-, 25-, 50-, 75- and 100-ft. More detailed descriptions of the ERS methodology, our data acquisition and analysis procedures, and the instrumentation we used are provided in **Appendix B – ERS Survey**.

5.2 RESULTS

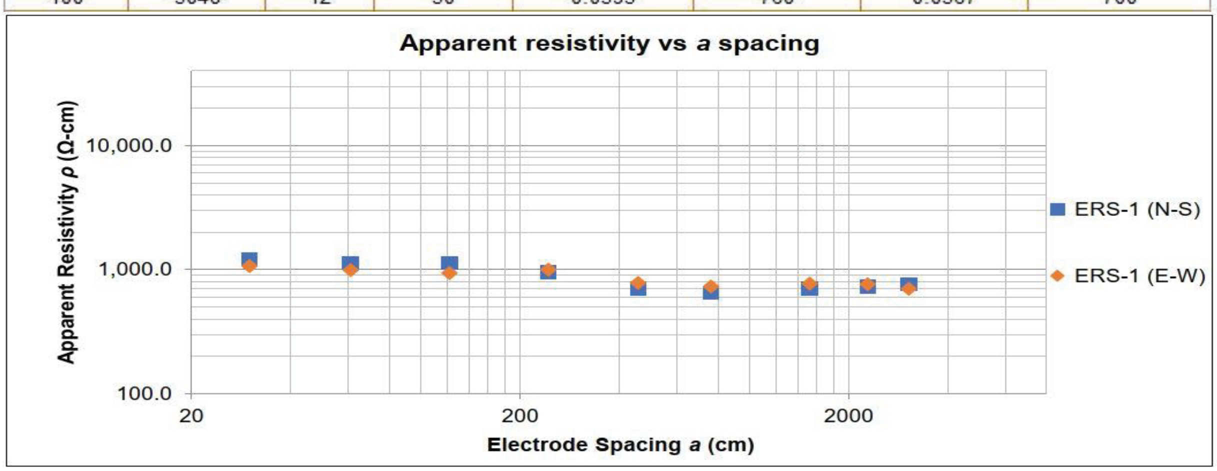
The VES survey results are presented on the Field Electrical Resistivity Data Sheets below. The data for each ER sounding were acquired along two perpendicular arrays with a common center point, oriented as specified on each data sheet. The apparent resistivity values are presented in units of ohm-centimeters.

FIELD ELECTRICAL RESISTIVITY DATA SHEET
WSP ■ Data Center - Pittsburg, California
December 20, 2022 ■ NORCAL Project No. NS225138

Array Loc.	**ERS-1**(38.005134°N, 121.910600°W)		
Instrument	AGI SuperSting R1	**Weather**	Sunny, High 58°F.
Serial #	SP0303161	**Ground Cond.**	clay & soil
Cal. Check	4/8/2022	**Tested By**	T. Black
Test Date	December 20, 2022	**Method**	Wenner 4-pin ASTM G57-06 (2012); IEEE 81-2012
Notes & Conflicts	Arrays oriented N-S and W-E with a common midpoint.		

Apparent resistivity ρ is calculated as : $\rho = \dfrac{4\pi a R}{1 + \dfrac{2a}{\sqrt{a^2+4b^2}} - \dfrac{a}{\sqrt{a^2+b^2}}}$

Electrode Spacing *a*		Electrode Depth *b*		ERS-1 (N-S)		ERS-1 (E-W)	
(feet)	(centimeters)	(inches)	(centimeters)	Measured Resistance *R* Ω	Apparent Resistivity ρ (Ω-cm)	Measured Resistance *R* Ω	Apparent Resistivity ρ (Ω-cm)
1	30	4	10	5.4870	1210	4.8710	1070
2	61	4	10	2.8130	1130	2.4830	990
4	122	4	10	1.4540	1130	1.2180	940
8	244	12	30	0.6018	950	0.6374	1000
15	457	12	30	0.2419	700	0.2679	780
25	762	12	30	0.1349	650	0.1527	730
50	1524	12	30	0.0729	700	0.0803	770
75	2286	12	30	0.0505	730	0.0532	760
100	3048	12	30	0.0395	760	0.0367	700



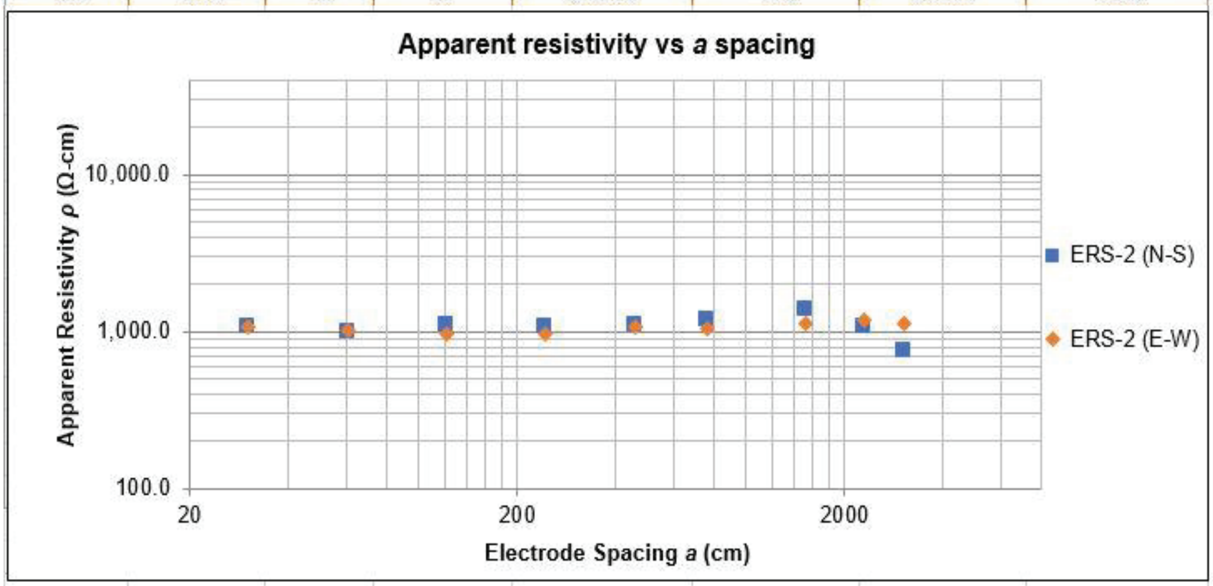
FIELD ELECTRICAL RESISTIVITY DATA SHEET

WSP ■ Data Center - Pittsburg, California
December 20, 2022 ■ NORCAL Project No. NS225138

Array Loc.	**ERS-2**(38.008145°N, 121.911540°W)		
Instrument	AGI SuperSting R1	**Weather**	Sunny, High 58°F.
Serial #	SP0303161	**Ground Cond.**	Clay and soil
Cal. Check	4/8/2022	**Tested By**	T. Black
Test Date	December 20, 2022	**Method**	Wenner 4-pin ASTM G57-06 (2012); IEEE 81-2012
Notes & Conflicts	Arrays oriented N-S and W-E with a common midpoint.		

Apparent resistivity ρ is calculated as : $\rho = \dfrac{4\pi a R}{1 + \dfrac{2a}{\sqrt{a^2 + 4b^2}} - \dfrac{a}{\sqrt{a^2 + b^2}}}$

Electrode Spacing *a*		**Electrode Depth *b***		**ERS-2 (N-S)**		**ERS-2 (E-W)**	
(feet)	(centimeters)	(inches)	(centimeters)	Measured Resistance *R* Ω	Apparent Resistivity ρ (Ω-cm)	Measured Resistance *R* Ω	Apparent Resistivity ρ (Ω-cm)
1	30	4	10	4.8560	1070	4.8900	1070
2	61	4	10	2.4530	980	2.5820	1030
4	122	4	10	1.4340	1110	1.2510	970
8	244	12	30	0.6888	1080	0.6099	960
15	457	12	30	0.3768	1090	0.3653	1060
25	762	12	30	0.2435	1170	0.2188	1050
50	1524	12	30	0.1432	1370	0.1179	1130
75	2286	12	30	0.0738	1060	0.0818	1180
100	3048	12	30	0.0390	750	0.0584	1120



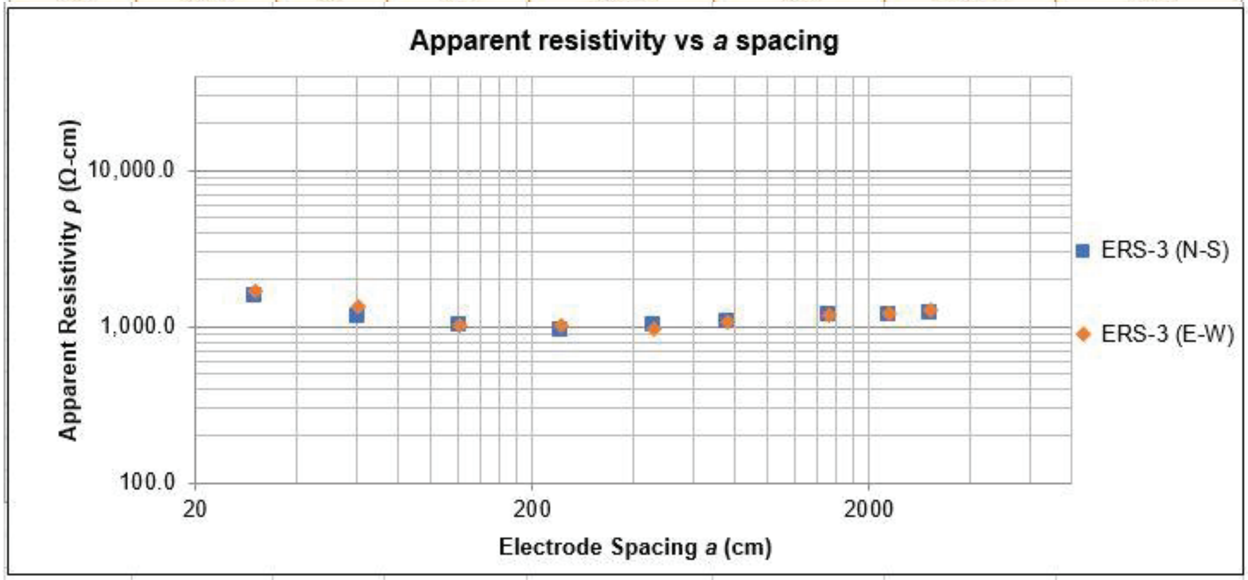
FIELD ELECTRICAL RESISTIVITY DATA SHEET

WSP ■ Data Center - Pittsburg, California
December 20, 2022 ■ NORCAL Project No. NS225138

Array Loc.	ERS-3(38.010177°N, 121.91097°W)		
Instrument	AGI SuperSting R1	**Weather**	Sunny, High 58°F.
Serial #	SP0303161	**Ground Cond.**	Clay and soil
Cal. Check	4/8/2022	**Tested By**	T. Black
Test Date	December 20, 2022	**Method**	Wenner 4-pin ASTM G57-06 (2012); IEEE 81-2012
Notes & Conflicts	Arrays oriented N-S and W-E with a common midpoint.		

Apparent resistivity ρ is calculated as : $\rho = \dfrac{4\pi a R}{1 + \dfrac{2a}{\sqrt{a^2+4b^2}} - \dfrac{a}{\sqrt{a^2+b^2}}}$

Electrode Spacing *a*		Electrode Depth *b*		ERS-3 (N-S)		ERS-3 (E-W)	
(feet)	(centimeters)	(inches)	(centimeters)	Measured Resistance *R*	Apparent Resistivity ρ	Measured Resistance *R*	Apparent Resistivity ρ
				Ω	(Ω-cm)	Ω	(Ω-cm)
1	30	4	10	7.1130	1560	7.6720	1690
2	61	4	10	2.8650	1150	3.3150	1330
4	122	4	10	1.3110	1020	1.3320	1030
8	244	12	30	0.5977	940	0.6491	1020
15	457	12	30	0.3482	1010	0.3352	970
25	762	12	30	0.2245	1080	0.2200	1060
50	1524	12	30	0.1226	1170	0.1240	1190
75	2286	12	30	0.0825	1190	0.0846	1220
100	3048	12	30	0.0634	1210	0.0672	1290



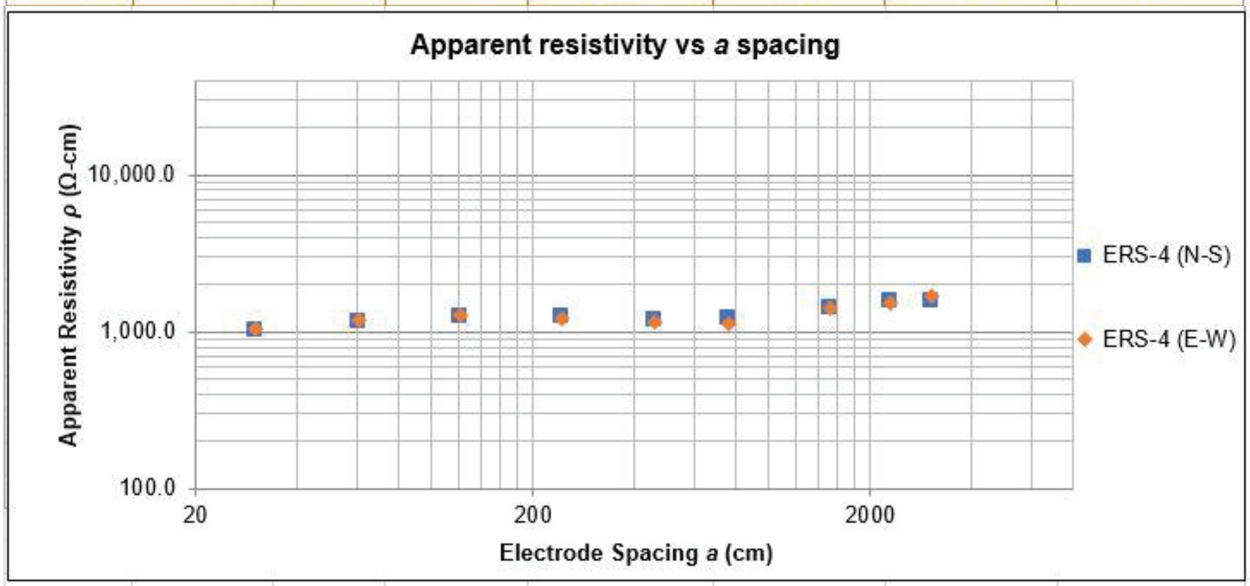
FIELD ELECTRICAL RESISTIVITY DATA SHEET

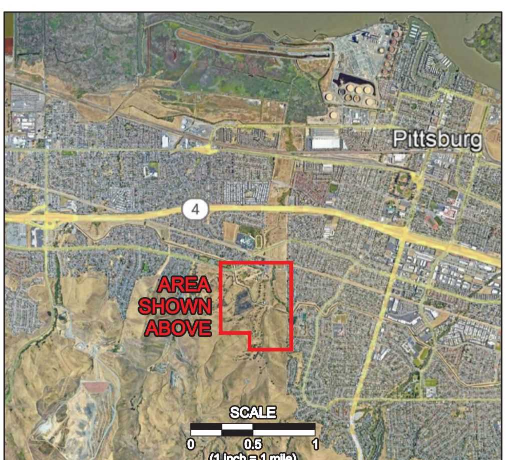
WSP ■ Data Center - Pittsburg, California
December 20, 2022 ■ NORCAL Project No. NS225138

Array Loc.	ERS-4(38.012018°N, 121.912299°W)		
Instrument	AGI SuperSting R1	Weather	Sunny, High 58°F.
Serial #	SP0303161	Ground Cond.	Clay and soil
Cal. Check	4/8/2022	Tested By	T. Black
Test Date	December 20, 2022	Method	Wenner 4-pin ASTM G57-06 (2012); IEEE 81-2012
Notes & Conflicts	Arrays oriented N-S and W-E with a common midpoint.		

Apparent resistivity ρ is calculated as : $\rho = \dfrac{4\pi aR}{1 + \dfrac{2a}{\sqrt{a^2+4b^2}} - \dfrac{a}{\sqrt{a^2+b^2}}}$

Electrode Spacing *a*		Electrode Depth *b*		ERS-4 (N-S)		ERS-4 (E-W)	
(feet)	(centimeters)	(inches)	(centimeters)	Measured Resistance *R* Ω	Apparent Resistivity ρ (Ω-cm)	Measured Resistance *R* Ω	Apparent Resistivity ρ (Ω-cm)
1	30	4	10	4.6170	1010	4.7270	1040
2	61	4	10	2.8790	1150	2.9830	1190
4	122	4	10	1.6040	1240	1.6620	1290
8	244	12	30	0.7991	1260	0.7673	1210
15	457	12	30	0.4092	1180	0.4009	1160
25	762	12	30	0.2491	1200	0.2367	1140
50	1524	12	30	0.1466	1400	0.1473	1410
75	2286	12	30	0.1081	1550	0.1075	1540
100	3048	12	30	0.0815	1560	0.0886	1700





VICINITY MAP

SCALE
0 150 300 600
(1 inch = 300 feet)

LEGEND

◆	MASW SOUNDING LOCATION	●	ER SOUNDING LOCATION
▬	MASW SOUNDING ARRAY	▬	ER SOUNDING ARRAY

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PROFESSIONAL GEOPHYSICIST
DAVID T HAGIN
No. 1033
STATE OF CALIFORNIA

SITE LOCATION MAP
MASW & ERS SURVEY
PITTSBURG DATA CENTER

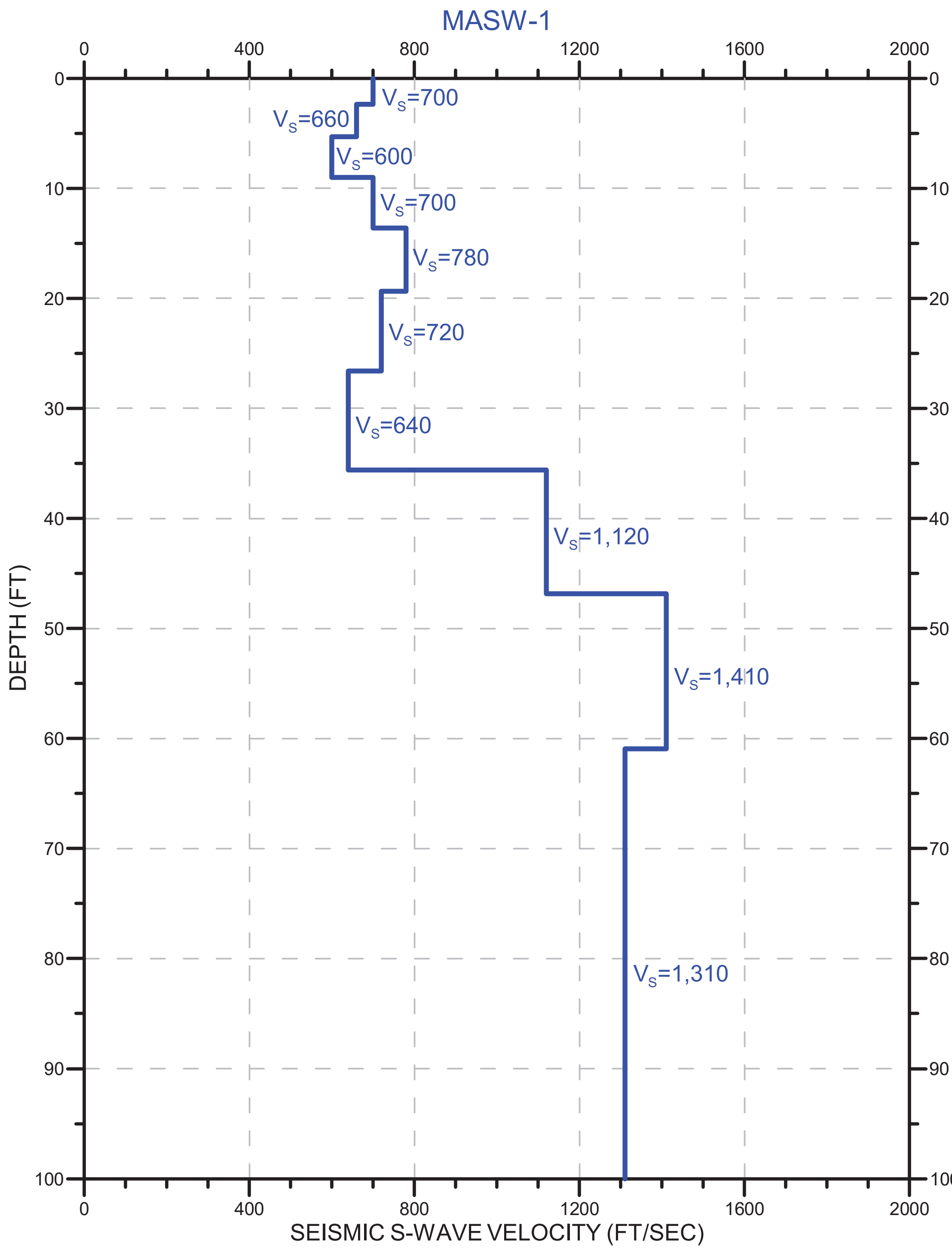
LOCATION: PITTSBURG, CALIFORNIA

CLIENT: WSP

JOB #. NS225138 | DATE: JANUARY 2023 | PLATE 1

DRAWN BY: H.PHILSON | APPROVED BY: DTH

David Hagin 1/5/2023



LEGEND	
	SEISMIC S-WAVE VELOCITY

Vs(30) = 977 fps

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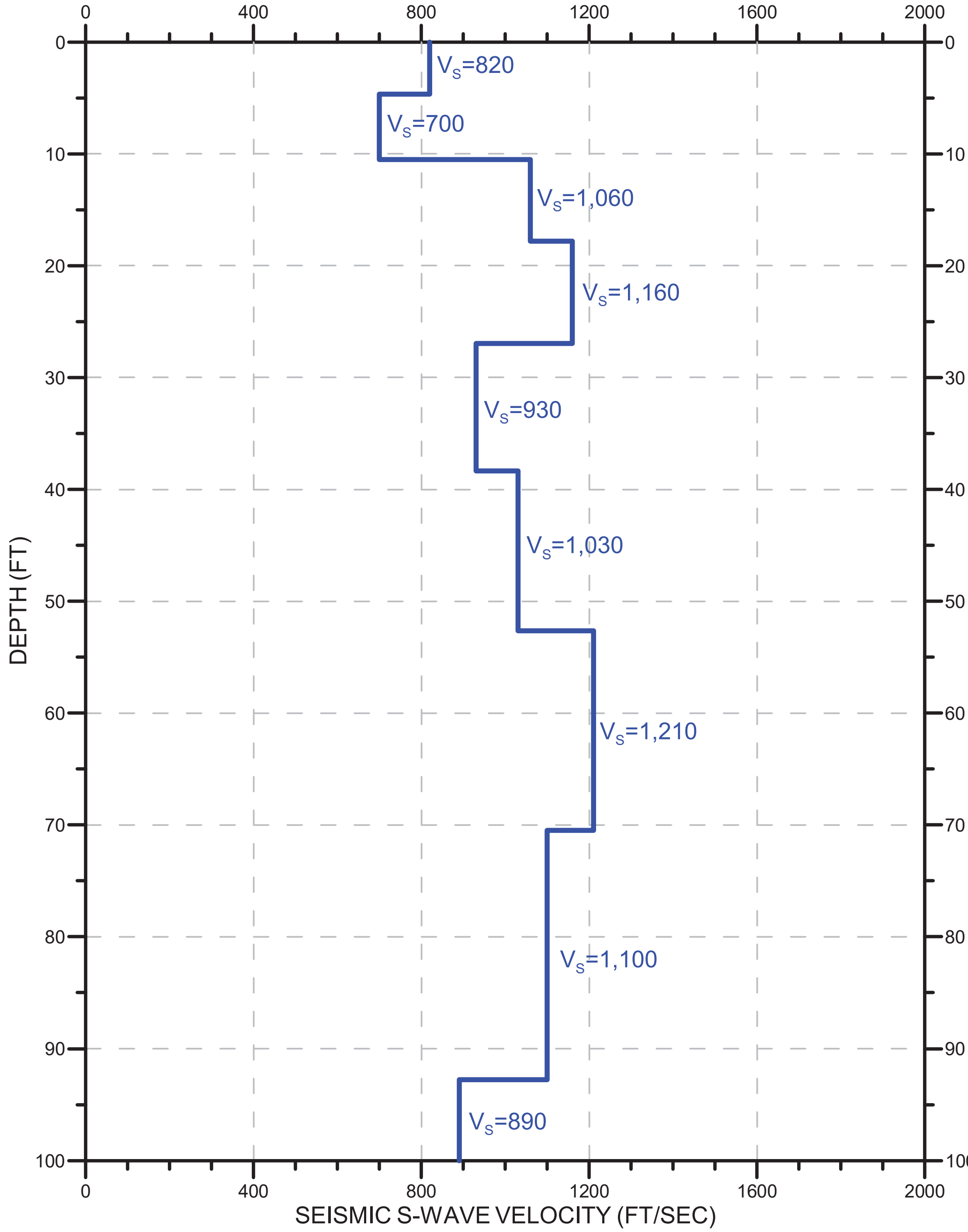
MASW SOUNDING
MASW-1
PITTSBURG DATA CENTER

LOCATION: PITTSBURG, CALIFORNIA		
CLIENT: WSP		
JOB #: NS225138	DATE: JANUARY 2023	PLATE
DRAWN BY: H.PHILSON	APPROVED BY: DTH	2
David Hagin	1/5/2023	

MASW-2



LEGEND	
	SEISMIC S-WAVE VELOCITY

Vs(30) = 1018 fps

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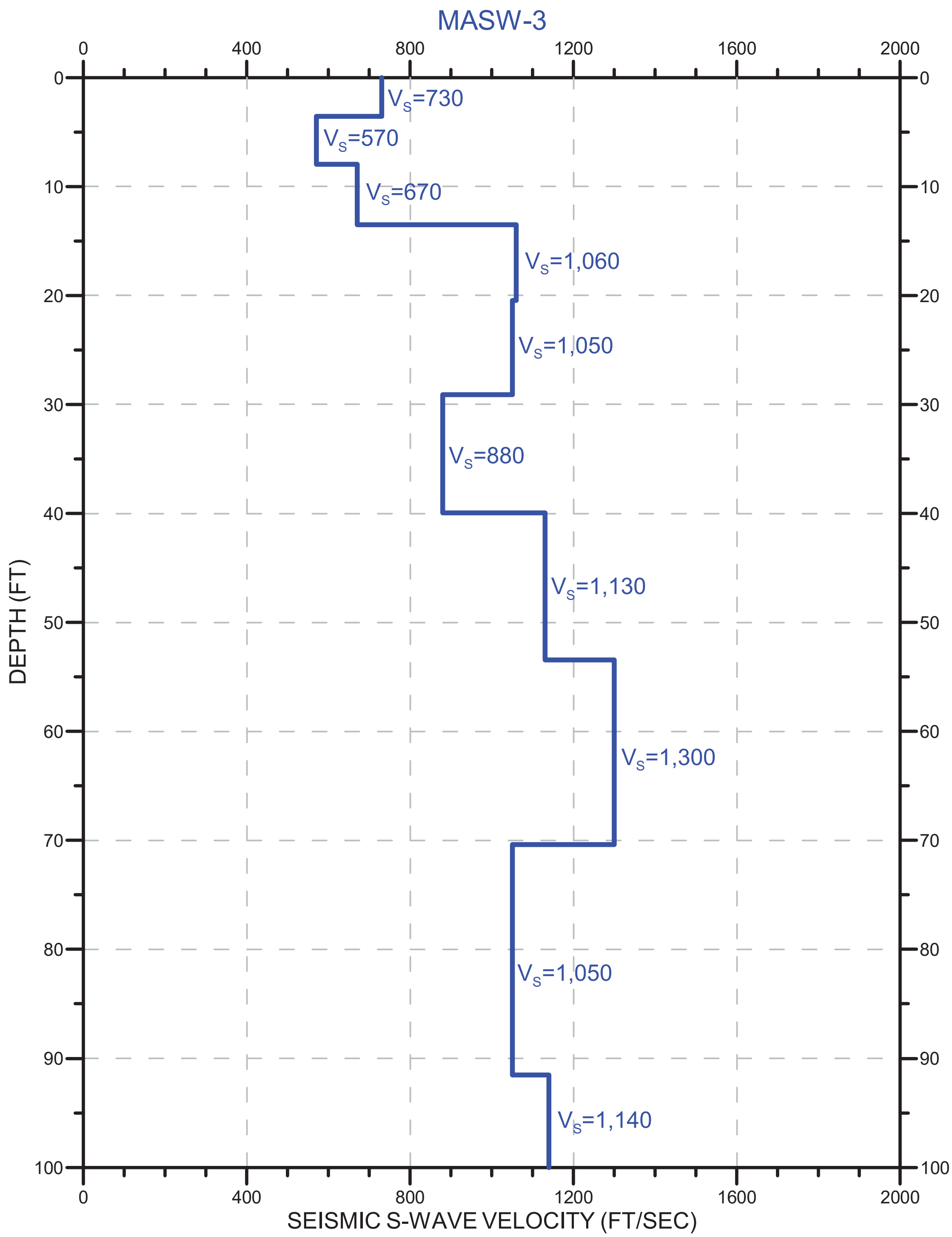
STATE OF CALIFORNIA

MASW SOUNDING
MASW-2
PITTSBURG DATA CENTER

LOCATION: PITTSBURG, CALIFORNIA		
CLIENT: WSP		
JOB #: NS225138	DATE: JANUARY 2023	PLATE
DRAWN BY: H.PHILSON	APPROVED BY: DTH	3
David Hagin	1/5/2023	



LEGEND	
———	SEISMIC S-WAVE VELOCITY

Vs(30) = 993 fps

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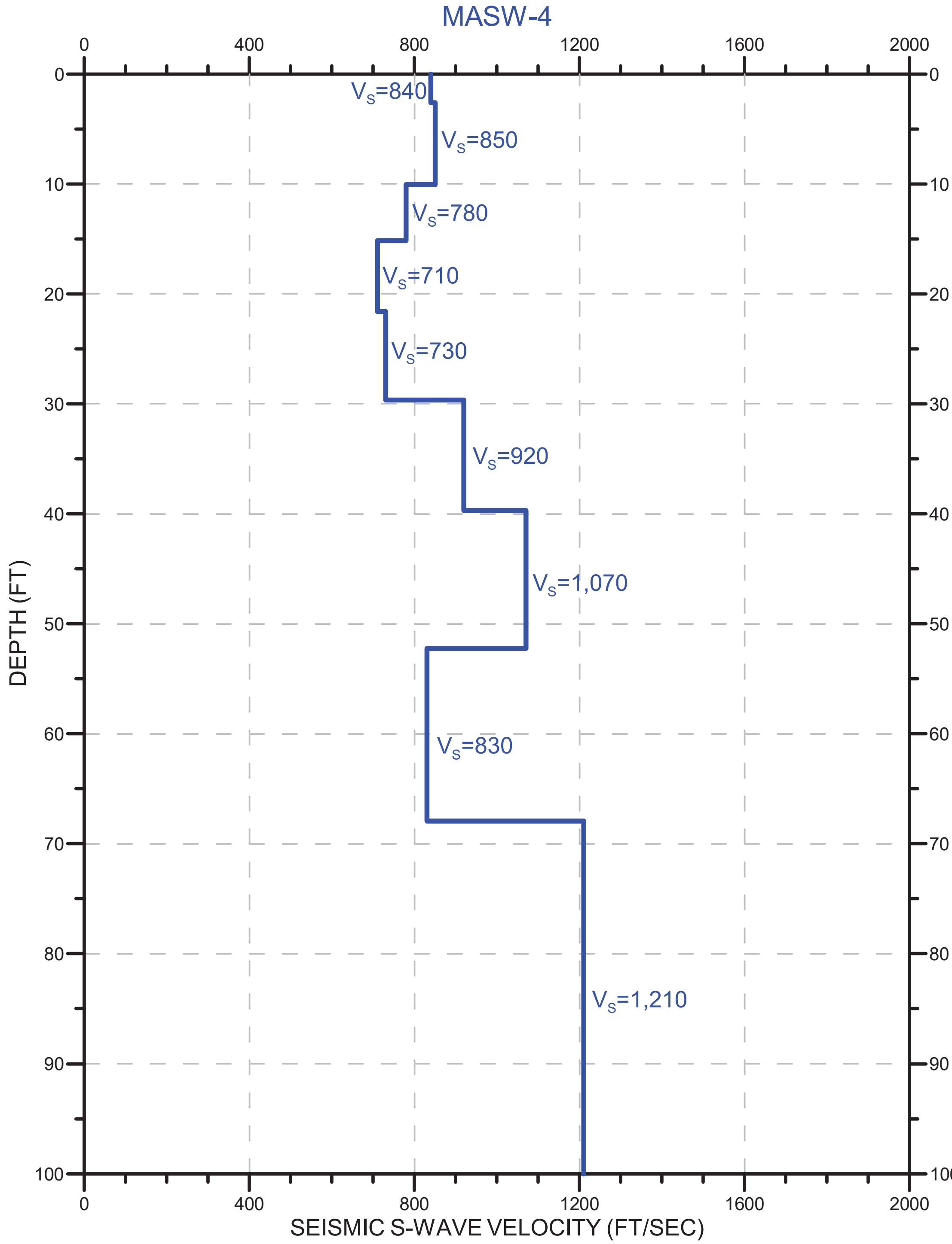
MASW SOUNDING
MASW-3
PITTSBURG DATA CENTER

LOCATION: PITTSBURG, CALIFORNIA		
CLIENT: WSP		
JOB #: NS225138	DATE: JANUARY 2023	PLATE
DRAWN BY: H.PHILSON	APPROVED BY: DTH	4
David Hagin	1/5/2023	



LEGEND	
(blue line)	SEISMIC S-WAVE VELOCITY

Vs(30) = 936 fps

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MASW SOUNDING
MASW-4
PITTSBURG DATA CENTER

LOCATION: PITTSBURG, CALIFORNIA		
CLIENT: WSP		
JOB #: NS225138	DATE: JANUARY 2023	PLATE
DRAWN BY: H.PHILSON	APPROVED BY: DTH	5
David Hagin	1/5/2023	

APPENDIX A:
MASW Sounding

APPENDI A:
MASW Sounding

1.0 METHODOLOGY

When seismic energy is generated at or near the ground surface, both body and surface waves are produced. Body waves expand omni-directionally throughout the subsurface. They consist of both compressional (P) and shear (S) waves. Surface waves (e.g., Rayleigh, Love, etc.) radiate along the ground surface at velocities that are proportional to shear wave velocity (Vs). Rayleigh waves are characterized by retrograde elliptical particle motion, and travel at approximately 0.9 times the velocity of S-waves.

If a vertical impact source is used, approximately two-thirds of the seismic energy that is produced is in the form of ground roll. As a result, surface waves are typically the most prominent signal on multi-channel seismic records. In addition, surface waves have dispersion properties that body waves lack. That is, different wavelengths have different penetration depths and, therefore, propagate at different velocities. By analyzing the dispersion of surface waves, it is possible to obtain an S-wave versus depth velocity profile. Since s-wave velocity is directly proportional to shear modulus, this provides a direct indication in the variation of stiffness (or rigidity) of subsurface materials.

Surface waves can be recorded and analyzed using a method referred to as Multichannel Analysis of Surface Waves (MASW). This method is used to collect surface wave data using a fixed array of geophones and shot points. This is referred to as a sounding, and results in a one-dimensional (1-D) model depicting variation in S-wave velocity versus depth beneath the center of the array. However, the subsurface conditions underlying the entire length of the array, and for several tens of feet to either side, contribute to the measured velocity values. The method requires an energy source that is capable of producing ground roll and geophones that are capable of detecting low frequencies (<10 Hz) signals.

2.0 DATA ACQUISITION

We acquired four MASW soundings, denoted MASW-1 through MASW-4, in locations determined by WSP personnel. The seismic arrays each consist of four-shot points and 24-geophones distributed at 6-ft intervals in a 210-ft long collinear array. A diagram of the configuration of each seismic array is shown below, in Figure 1.

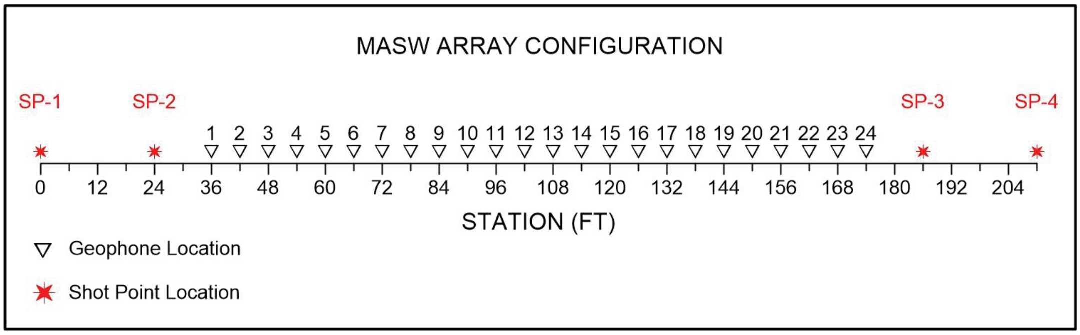

Figure 1: *MASW Array Configuration.*

Seismic energy was produced at each shot point using a 16-pound sledgehammer striking a metal plate on the ground surface. The resulting seismic waveforms were detected by Oyo ***Geospace*** geophones with a natural frequency of 4.5-Hz and recorded using a Geometrics ***Geode*** 24-channel distributed array engineering seismograph. The seismic waveforms were digitized, pre-processed and amplified by the Geode and transmitted via a ruggedized Ethernet cable to a field computer. The recorded data were archived for subsequent processing and displayed on the computers LCD screen in the form of seismograms for quality assurance purposes.

The positions of the MASW arrays are shown on Plate 1 by the red lines. The center points of the arrays, which are considered the sounding locations, are represented by the red diamonds.

3.0 DATA ANALYSIS

The seismic wave-traces (shot gathers) recorded at each shot point were analyzed using the computer program ***SURFSEIS*** developed by the Kansas Geological Survey (Version 5.0, 2016). This interactive program converts the data acquired from all four shot points in a given sounding into a dispersion curve representing phase velocity versus frequency. This curve is then inverted to produce a 1D model indicating S-wave velocity versus depth. The steps involved in this procedure are as follows:

1) The shot gathers are converted to KGS format.
2) Stations are assigned to the geophone and shot point locations.
3) The resulting records are viewed to determine their overall quality. If necessary, portions of the records are muted to remove interference from refractions, reflections and higher mode events.
4) For each formatted (and/or muted) record, the program produces what is referred to as an "overtone plot". This is a colored cross-section indicating phase velocity versus frequency and amplitude. The vertical axis represents phase velocity (increasing upward); the horizontal axis represents frequency (increasing to the right); and signal amplitude is

indicated by various colors, with the hottest colors (orange to red to dark brown) representing the greatest signal to noise ratio. Typically, the strongest signals align in a curved pattern with a symmetry with the shape of a “hockey stick” where the blade is pointing upward at the lower end of the frequency spectrum (higher velocity at greater depth) and the handle projects to the right in the direction of increasing frequencies indicating lower velocities.

5) The overtone plots compiled from the four shot points are reviewed to determine their overall quality and the best among them (possibly all) are merged to form a single overtone. This enhances the overall signal to noise ratio of the survey and incorporates data from both ends of the spread (if feasible).
6) The resulting overtone plot is used as a guide in deriving a dispersion curve representing phase velocity versus frequency. This is done by fitting the curve along the center of the hockey stick where the signal to noise ratio is highest.
7) The resulting dispersion curve is inverted through an iterative process to compute a 1D model representing S-wave velocity versus depth.

The velocities in each depth range for MASW-1 through MASW-4 are tabulated in Tables A through D in the main body of the report. The data are also depicted by the step-chart graphs on Plates 2 through 5.

APPENDIX B:
ERS Survey

APPENDIX B:

ERS Survey

1.0 METHODOLOGY

1.1 ELECTRICAL RESISTIVITY: DEFINITION AND APPLICATIONS

Electrical resistivity (ER) is the resistance of a volume of earth material to the flow of electrical current. The ER of sedimentary earth materials is directly affected by factors such as grain size, porosity, mineralogy, moisture content and groundwater salinity. However, it has been our experience through numerous ER surveys conducted throughout the Bay Area that, in unconsolidated materials, grain size seems to have the largest effect on ER of all these parameters. Specifically, fine grained materials such as clays and silts typically have relatively low ER whereas coarse grained materials such as sands and gravels have relatively high ER.

The ER of rock is affected primarily by mineralogy and the degree of weathering and fracturing. Rock formations that are deeply buried and not exposed to chemical weathering are generally impermeable, contain little water, and have a relatively high electrical resistivity. Conversely, highly weathered and fractured rock that contains moisture typically has lower resistivity values. Alternatively, some rocks contain conductive minerals that can result in the rock having relatively low ER.

Given the relationships described above, geophysical methods that measure subsurface ER can be used to determine the depth, thickness and lateral extent of groundwater aquifers, the depth to groundwater, the depth to rock, the depth, thickness and lateral extent of clay layers and the depth, thickness and lateral extent of sand/gravel deposits. ER measurements can also be used to evaluate soil corrosion potential and to provide parameters for the design of electrical grounding systems.

1.2 ELECTRICAL RESISTIVITY SOUNDING

Measuring the variation in ER versus depth beneath a fixed point is referred to as a vertical electrical sounding (ERS). This involves transmitting electrical current *(I)* into the ground between two electrodes, and measuring the resulting electrical potential or voltage drop (*V*) between two other electrodes. There are many different electrode configurations that can be used. The most common are the Wenner and Schlumberger arrays. With both techniques, the four electrodes are arranged in a collinear array. Current is transmitted between the outer two electrodes (referred to as A and B) and the resulting voltage is measured across the inner two electrodes (referred to as M and N). Readings are typically taken with many different electrode separations, ranging from less than one foot to several hundreds of feet. The larger the separation, the deeper the current is forced to flow to complete a circuit. The actual current flow occurs within a generally hemi-spherical volume of earth between the current electrodes. The readings obtained with each electrode separation are used to compute a value referred to as apparent resistivity *(ρ_a).* The term “apparent” is used because the value represents the resistivity of a volume of earth with varying resistivity values rather than a discrete layer with consistent resistivity. The location of the sounding is defined as the center of the electrode array.

For ER surveys involving the design of grounding systems, such as this survey, the Four Pin Wenner Array is typically used. With this array the electrode separation (a) is uniform between all four electrodes and increases from one reading to the next. The depth of the electrode (b) is also increased at greater a-spacings. The equation that is used to compute apparent resistivity values is presented on the Field Electrical Resistivity Data Sheets included in Appendix A.

2.0 INSTRUMENTATION

We collected ERS data using a ***SuperSting R1*** Resistivity Meter, manufactured by Advanced Geosciences Incorporated (AGI). The SuperSting is a self-contained unit that transmits current at outputs ranging from 1 to 2,000 milliamps (mA). The instrument measures the electrical potential drop (voltage) caused by the current influx and converts the data to values of resistance and apparent resistivity. The data are stored in internal memory and can be downloaded to a computer for subsequent processing and archiving.

3.0 DATA ACQUISITION

The ERS survey at each location consisted of two perpendicular electrode arrays. The arrays are denoted as ERS-1 through ERS-4, as shown on Plate 1. The ***SuperSting R1*** was connected to the four electrodes in the array using 14-gauge insulated single conductor wires. Once programmed with the a-spacing for a given measurement, the instrument transmitted electrical current through the outer electrodes (A and B) and measured the voltage drop across the inner pair (M and N). Each measurement was made twice, and the results compared to make sure that there was no more than 2% deviation between the measurements. The averaged readings were then saved for subsequent processing. This procedure was repeated for every prescribed a-spacing starting with small values and expanding with each subsequent measurement to the largest spacing. Measurements were acquired using a-spacings of 1-, 2-, 4-, 8-, 15-, 25-, 50-, 75- and 100-ft, as specified by WSP. The results of the ERS survey are presented in Section 5.2 of the main body of this report in units of ohm-centimeters.

4.0 LIMITATIONS

A common feature of all electrical methods is that the models derived from the electric imaging are not unique. That is, depending on the subsurface geo-electric structure, there may be many models that will produce essentially the same apparent resistivities. This is known as the *principal of equivalence*. To overcome this limitation, computer software programs include routines for evaluating the equivalence of a given model relative to the observed resistivity values, resulting in a model that provides the closest fit to the observed data. Additionally, if the ground surface is too resistive, the system may have problems transmitting current into the subsurface (this situation can be remedied through the application of salt water at the base of each electrode). Conversely, if the ground surface is highly conductive, the potentials measured become negligible, resulting in a very low signal-to-noise ratio and therefore unreliable data.

APPENDIX

B BORING LOGS

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
E. O'Hara	**12-15-22**	**12-15-22**	**38° 0' 43.92" / -121° 54' 43.2" WGS84**	**WSP-PDC-01**

DRILLING CONTRACTOR	BOREHOLE LOCATION DESCRIPTION	SURFACE ELEVATION
ConeTec	**Former Golf Course Fairway**	**114.0 ft NAVD88**
DRILLING METHOD	DRILL RIG	BOREHOLE DIAMETER
Hollow-Stem Auger	**CME 75 Truck Drill**	**4.25-inch**
SAMPLER TYPE(S) AND SIZE(S) (ID)	SPT HAMMER TYPE	HAMMER EFFICIENCY, ERi
Bulk, SPT (2"), Mod Cal (2.5")	**Automatic 140lb hammer 30in drop**	**92.5%**
BOREHOLE BACKFILL AND COMPLETION	GROUNDWATER READINGS — DURING DRILLING: **not encountered**; AFTER DRILLING (DATE): **not encountered**	TOTAL DEPTH OF BORING
Cement grout mix		**41.5 ft**

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	Dry Unit Weight (pcf)	Moisture Content (%)	Liquid Limit (%)	Plasticity Index (%)	Passing #4 (%)	Passing #200 (%)	Pocket Penetration (tsf)	Remarks
	0	Vegetation.	S-1					15	45	32	100	77		Top 5 ft: Hand Auger with 3" Diameter M, CP, CR, PA, PI OC = 3.4% pH = 8.1 Thermal Res.
	1	**ARTIFICIAL FILL (Qaf)** Lean CLAY with SAND (CL); light brown; moist; mostly low plasticity fines.												
112.00	2	- brown.												
	3	CLAYEY SAND (SC); medium dense; light brown; moist; mostly fine SAND; some high plasticity fines; weak cementation.												
110.00	4	**ALLUVIUM (Qa)** Lean CLAY (CL); hard; dark brown; moist; mostly low to medium plasticity fines; trace fine SAND.												
	5	- brown; little fine to medium SAND.	S-2	9 16 16	32	100	108	14					>5.0	No Dent with Pocket Penetration M, UW
108.00	6													
	7													
106.00	8													
	9													
104.00	10	Lean CLAY with SAND (CL); hard; light brown; moist; some fine SAND; mostly low plasticity fines.	S-3	11 23 29	52	56		11						No Dent with Pocket Penetration M
	11													
102.00	12													
	13													
100.00	14													
	15	- trace fine to coarse, subrounded GRAVEL.	S-4	15 36 75/5"	111/11	94	114	10						M, UW
98.00	16													
	17													
96.00	18													
	19													
94.00	20	Lean CLAY (CL); hard; brown; moist; mostly low plasticity fines; little fine SAND.	S-5	10 18 20	38	72		14					>5.0	No Dent with Pocket Penetration M
	21													
92.00	22													
	23													
90.00	24													
	25													

(continued)

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
E. O'Hara	**12-15-22**	**12-15-22**	**38° 0' 43.92" / -121° 54' 43.2" WGS84**	**WSP-PDC-01**

DRILLING CONTRACTOR	BOREHOLE LOCATION DESCRIPTION	SURFACE ELEVATION
ConeTec	**Former Golf Course Fairway**	**114.0 ft NAVD88**
DRILLING METHOD	DRILL RIG	BOREHOLE DIAMETER
Hollow-Stem Auger	**CME 75 Truck Drill**	**4.25-inch**
SAMPLER TYPE(S) AND SIZE(S) (ID)	SPT HAMMER TYPE	HAMMER EFFICIENCY, ERi
Bulk, SPT (2"), Mod Cal (2.5")	**Automatic 140lb hammer 30in drop**	**92.5%**
BOREHOLE BACKFILL AND COMPLETION	GROUNDWATER READINGS — DURING DRILLING: **not encountered**; AFTER DRILLING (DATE): **not encountered**	TOTAL DEPTH OF BORING
Cement grout mix		**41.5 ft**

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	Dry Unit Weight (pcf)	Moisture Content (%)	Liquid Limit (%)	Plasticity Index (%)	Passing #4 (%)	Passing #200 (%)	Pocket Penetration (tsf)	Drilling Method	Remarks
	25		Lean CLAY with SAND (CL); hard; brown; moist; some fine SAND; mostly low plasticity fines; some fine, rounded GRAVEL; strong cementation.		S-6	13 28 41	69	100	106	13					>5.0		No Dent with Pocket Penetration M, UW
88.00	26																
	27																
86.00	28																
	29																
84.00	30		- dense; little fines; weak cementation.		S-7	10 15 16	31	72		13							M
	31																
82.00	32																
	33																
80.00	34																
	35		- very dense; moderate cementation.		S-8	22 43 63	106	89	104	18							M, UW
78.00	36																
	37																
76.00	38																
	39																
74.00	40		- dense.		S-9	14 18 21	39	100		13							M
	41																
72.00	42		Bottom of borehole at 41.5 ft below ground surface.														
	43																
70.00	44																
	45																
68.00	46																
	47																
66.00	48																
	49																
	50																

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
E. O'Hara	**12-14-22**	**12-14-22**	**38° 0' 37.44" / -121° 54' 36" WGS84**	**WSP-PDC-02**

DRILLING CONTRACTOR **ConeTec**	BOREHOLE LOCATION DESCRIPTION **Former Golf Course Fairway**	SURFACE ELEVATION **107.0 ft NAVD88**
DRILLING METHOD **Hand Auger**	DRILL RIG **N/A**	BOREHOLE DIAMETER **3-inch**
SAMPLER TYPE(S) AND SIZE(S) (ID) **Bulk**	SPT HAMMER TYPE **N/A**	HAMMER EFFICIENCY, ERi **N/A**
BOREHOLE BACKFILL AND COMPLETION **Backfilled with Auger Cuttings**	GROUNDWATER READINGS — DURING DRILLING **not encountered** — AFTER DRILLING (DATE) **not encountered**	TOTAL DEPTH OF BORING **5.0 ft**

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	Dry Unit Weight (pcf)	Moisture Content (%)	Liquid Limit (%)	Plasticity Index (%)	Passing #4 (%)	Passing #200 (%)	Pocket Penetration (tsf)	Drilling Method	Remarks
	0		Vegetation.		S-1												Hand Auger to 5 feet
	1		**ARTIFICIAL FILL (Qaf)** Fat CLAY (CH); dark brown; moist; mostly high plasticity fines; trace fine SAND.							20	52	35	100	79			M, PA, PI OC = 1.2%
105.00	2																
	3		- white concretions.														
103.00	4		- brown; little fine SAND.														
	5		Bottom of borehole at 5.0 ft below ground surface.														
101.00	6																
	7																
99.00	8																
	9																
97.00	10																
	11																
95.00	12																
	13																
93.00	14																
	15																
91.00	16																
	17																
89.00	18																
	19																
87.00	20																
	21																
85.00	22																
	23																
83.00	24																
	25																

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
E. O'Hara	**12-14-22**	**12-14-22**	**38° 0' 28.8" / -121° 54' 42.84" WGS84**	**WSP-PDC-03**

DRILLING CONTRACTOR **ConeTec**	BOREHOLE LOCATION DESCRIPTION **Former Golf Course Fairway**	SURFACE ELEVATION **145.0 ft NAVD88**
DRILLING METHOD **Hand Auger**	DRILL RIG **N/A**	BOREHOLE DIAMETER **3-inch**
SAMPLER TYPE(S) AND SIZE(S) (ID) **Bulk**	SPT HAMMER TYPE **N/A**	HAMMER EFFICIENCY, ERi **N/A**
BOREHOLE BACKFILL AND COMPLETION **Backfilled with Auger Cuttings**	GROUNDWATER READINGS — DURING DRILLING: **not encountered**; AFTER DRILLING (DATE): **not encountered**	TOTAL DEPTH OF BORING **5.0 ft**

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	Dry Unit Weight (pcf)	Moisture Content (%)	Liquid Limit (%)	Plasticity Index (%)	Passing #4 (%)	Passing #200 (%)	Pocket Penetration (tsf)	Drilling Method	Remarks
	0		Vegetation.		S-1												Hand Auger to 5 feet
	1		**ARTIFICIAL FILL (Qaf)** SANDY lean CLAY (CL); very soft; dark brown; moist; mostly low plasticity fines; trace fine SAND.							19	33	8	99	55			M, CP, CR, PA, PI OC = 3.5% pH = 9.7 Thermal Res.
143.00	2		Well-graded SAND (SW); brown; moist; mostly fine to medium SAND; trace subangular GRAVEL; weak cementation.														
	3																
141.00	4		1" Diameter round gravel.														
	5		Bottom of borehole at 5.0 ft below ground surface.														
139.00	6																
	7																
137.00	8																
	9																
135.00	10																
	11																
133.00	12																
	13																
131.00	14																
	15																
129.00	16																
	17																
127.00	18																
	19																
125.00	20																
	21																
123.00	22																
	23																
121.00	24																
	25																

AVAIO CAPITAL

PREPARED BY: B. Arabzadeh

CHECKED BY: M. Arzamendi

Pittsburg Technology Center

Pittsburg, California

PROJECT NUMBER: 31405786.000

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San Diego, CA 92121

LOG OF BORING

WSP-PDC-03

Wl l Oy c. **E. O'Hara** / cOI T yaEO **12-14-22** / i v BhVOETvl yaEO **12-14-22**	cv#O1v VO Wi aETvl RV3gGe0d eul eugoG3kg30Hy 3gn; C **38° 0' 18.36" / -121° 54' 38.16" WGS84**	1v VOTy **WSP-PDC-04**
y#TWT l i v l E#ai Ev# **ConeTec**	cv#O1v VO Wi aETvl yO9i #ThETvl **Former Golf Course Fairway**	9U#/ai OOVOt aETvl **99.0 ft NAVD88**
y#TWT l BOE1vy **Hand Auger**	y#TV#T **N/A**	cv#O1v VO yTaBOEO# **3-inch**
9aBhVO# E. hOF9Cal y 9TZOF9CFTy C **Bulk**	9hE 1aBBO# E. hO **N/A**	1aBBO# O/ / T TOl i . , O#o **N/A**
cv#O1v VOcai K/ TWal y i v BhVOETvl **Backfilled with Auger Cuttings**	l #v Ul y WaEO# #OayT l 9 / yU#T l y#TWT l **not encountered** / a/ EO# y#TWT l FyaEOC **not encountered**	Ev EaV yOhE1 v/ cv#T l **5.0 ft**

OVOt aETvl FgC	y OhE1 FgC	B3gAu3b l u3mpsk	yO9i #ThETvl	93; mtA Ves3gee0	93; mtAl n; (Au	cbe) k mAu%0f	cbe) k mAureeg	#AseRAuwFq C	y uwU0eg WAedpg FmsrC	BeokgnuA i e0gA0gFq C	VoMhdHVo egFq C	h3kgsgw T0HAx Fq C	h3kkod 64 Fq C	h3kkod 6PNN Fq C	hesYAg hA0Ag3gee0 FgkrC	y uteod B AgpeH	#A; 3uYk
	N		t AdAg3gee0f		9S2												130HandAuge 7 rAAg
	2		**ARTIFICIAL FILL (Qaf)** VA30 i Va. Fi VOLH3uY (ue) 0L; eokgL; ekgbwbe) mb3kgsogwrd0AkLbogbA ro0A 9al y f							2-	52	-	DD	77			B, ha, hT vi = Nf5q
D-fNN	P																
	5		heeubwdu3HAH9al y F9h OLbodpg(ue) 0L; eokgL; ekgbw ro0A 9al y LbogbA ro0AkL; eHAu3gA ge) A3Y sA; A0g3gee0f														
D7fNN	4																
	7		cegge; er (euApebA 3g7fNrg(Abe) duen0Hknu3sAf														
D5fNN	%																
	-																
D2fNN	8																
	D																
8DfNN	2N																
	22																
8-fNN	2P																
	25																
87fNN	24																
	27																
85fNN	2%																
	2-																
82fNN	28																
	2D																
-DfNN	PN																
	P2																
--fNN	PP																
	P5																
-7fNN	P4																
	P7																

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
E. O'Hara	12-15-22	12-15-22	38° 0' 25.92" / -121° 54' 37.44" WGS84	WSP-PDC-05

DRILLING CONTRACTOR	BOREHOLE LOCATION DESCRIPTION	SURFACE ELEVATION
ConeTec	Adjacent to Creek	120.0 ft NAVD88
DRILLING METHOD	**DRILL RIG**	**BOREHOLE DIAMETER**
Other Drilling Method	N/A	N/A-inch
SAMPLER TYPE(S) AND SIZE(S) (ID)	**SPT HAMMER TYPE**	**HAMMER EFFICIENCY, ERi**
Bulk	N/A	N/A

BOREHOLE BACKFILL AND COMPLETION	GROUNDWATER READINGS	DURING DRILLING	AFTER DRILLING (DATE)	TOTAL DEPTH OF BORING
Backfilled with Shovel Cuttings		not encountered	not encountered	3.0 ft

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	Dry Unit Weight (pcf)	Moisture Content (%)	Liquid Limit (%)	Plasticity Index (%)	Passing #4 (%)	Passing #200 (%)	Pocket Penetration (tsf)	Drilling Method	Remarks
	0		Vegetation.		S-1												Shovel Sample in top 3 feet
	1		**ARTIFICIAL FILL (Qaf)** Fat CLAY (CH); dark brown; moist; mostly high plasticity fines; trace medium SAND.														
118.00	2		SANDY lean CLAY (CL); dark brown; moist; mostly high plasticity fines; little fine to medium SAND.														
	3		Bottom of borehole at 3.0 ft below ground surface.														
116.00	4																
	5																
114.00	6																
	7																
112.00	8																
	9																
110.00	10																
	11																
108.00	12																
	13																
106.00	14																
	15																
104.00	16																
	17																
102.00	18																
	19																
100.00	20																
	21																
98.00	22																
	23																
96.00	24																
	25																

AVAIO CAPITAL

PREPARED BY: B. Arabzadeh

CHECKED BY: M. Arzamendi

Pittsburg Technology Center
Pittsburg, California

PROJECT NUMBER: 31405786.000

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LOG OF BORING

WSP-PDC-05

YT/ / Db 6) E. O'Hara · 6D/ Gk b(I D 12-15-22 · LTPvYDI Gk b(I D 12-15-22	6TEDmTYD YTL(I Gk 38° 0' 33.48" / -121° 54' 43.56" WGS84	mTYD Gk WSP-PDC-06
bEGYYGk/ LTkIE(LITE ConeTec	6TEDmTYD YTL(I Gk bDSLEG I Gk Hillside Toe	SWEC(LD DYDi (I Gk 155.0 ft NAVD88
bEGYYGk/ PDI mTb Other Drilling Method	bEGYYEG N/A	6TEDmTYD bGPDI DE N/A-inch
S(PvYDEI)vDwGs(kb SGZDwGswGs Bulk	SvI m(PPDEI)vD N/A	m(PPDE DCOGGk L),DEt N/A
6TEDmTYD6(L4COGY(kb LTPvYDI Gk Backfilled with Shovel Cuttings	/ETWkb%(IDE ED(bGk/S · bWEGk/ bEGYYGk/ not encountered · (CIDE bEGYYGk/ wb(IDs not encountered	ITI(YbDvImTC6TEGk/ 3.0 ft

DYDi (I Gk	bDvI m	PgfAhtgu / hgr atNp	bDSLEG I Gk	Sge ruA Yn Ngftno	Sge ruA k qe yAh	6un; p r Ah3 toF	6un; p r AhVnf	EANhdAhc	b hc	P ntpfqhA LnofAof	Yt#qtB Yte ff	vugpftNfc GBAx	v gppto. CH	v gppto. C022	v nNAf v AoAfrgftno	b htuo. P AfanB	EAe ghMp
	1		i A AfgftnoF	S-1						0H	52	93	122	79			SandAuSge ruA to fnr 9 VAAf
			ARTIFICIAL FILL (Qaf) Cgf LY( ) ; tfa S(kb wLmsl yhn; ol e ntpfl e npfuc at. a rugpftNfc VtoApl ftrgNA VtoA S(kbF														P, Lv, LE, v(, vG rm= JF0 I aAhe guEApF
159F22	0		LY( ) D) S(kb wSLsl BgHMyhn; ol e ntpfl e npfuc VtoA fn e ABtqe S(kbl pne A e ABtqe rugpftNfc VtoApl %AgML Ae AofgftnoF														
	9		6nffne nVynhAanuA gf 9F2 VfyAun; . hnqoB pqhVgNAF														

Elevation scale: 159F22, 151F22, 1H8F22, 1H7F22, 1H5F22, 1H9F22, 1H1F22, 198F22, 197F22, 195F22, 199F22, 191F22
Depth scale: 2, 1, 0, 9, H, 5, 3, 7, J, 8, 12, 11, 10, 19, 1H, 15, 13, 17, 1J, 18, 02, 01, 00, 09, 0H, 05

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vEDv(EDb 6)K 6F(hgy: gBAa
LmDL4Db 6)K PF(h ge AoBt

v tffpyqh I ANaonun. c L AofAh
v tffpyqh , L gutVnhotg

vETzDLI kWP6DEK 91H257J3F222

wsp

%Sv WS(H755 Dgpf. gfA P guu SqtfA 152 Sgo btA n, L(80101

LOG OF BORING

WSP-PDC-06

CT/ / D; qY	qD/ G; ; LI D	FT6 vODI GT) ; LI D	qTEDbTOD CTFLI GT) kCAtUDH VH) Ht0UDApt Ars ; AtBu w	bTOD G;
E. O'Hara	12-15-22	12-15-22	38° 0' 21.96" / -121° 54' 43.56" WGS84	WSP-PDC-07

; EGDOG/ FT) I ELFI TE	qTEDbTOD CTFLI GT) ; D9FEG9I GT)	9WE. LFD DODf LI GT)
ConeTec	Hillside Toe	201.0 ft NAVD88
; EGDOG/ 6 DI bT;	; EGDCEG	qTEDbTOD ; G6 DI DE
Other Drilling Method	N/A	N/A-inch
9L6 vODE I YvDk9wL) ; 9GZDk9wkG; w	9vI bL6 6 DE I YvD	bL6 6 DE D. . GFGD) FY, DEe
Bulk	N/A	N/A
qTEDbTOD qLFK. GCL) ; FT6 vODI GT)	/ ETW) ; MLI DE EDL; G/ 9 — ; WEG/ ; EGDOG/: not encountered — L. I DE ; EGDOG/ k, LI Dw: not encountered	I TI LC; DvI b T. qTEG/
Backfilled with Shovel Cuttings		3.0 ft

DODf LI GT) kdw	DvI b kdw	6 AtnleAd / IAg0eAp	; D9FEG9I GT)	9Au gdh OHNAteH	9Au gdh) Bu hnl	qdH(p gnl Rei	qdH(p gnl dHH	EnNHanlc k# w	; lc W eMne/0t kgNbw	6 HeptBln FH tnrt k# w	GePBes Ceu el k# w	vdApteNetc Gsnx k# w	vApper V O4 k# w	vApper V O3%%k# w	vHNynt vnr ntlAteH kpow	; leete V6 nt0Hs	Enu Alyp
	%		f nVntAteHr i														
	-		**ARTIFICIAL FILL (Qaf)** . At FOLY (ee0 9L) ; kFbwmsAly hlH(r mu Heptmu Hptdc 0eV0 gdApteNetc oer npmtlANh oer n 9L) ; i	9S						35	52	21	- %%	72			90Hand9Au gdh er tHg 2 onnt 6 , Fv, FE, vL, vG gb = 1iR I 0nlu AdEnpi
- 88i%%	3																
	2		FOLYDY 9L) ; k9F wmlH(r mu Heptmu Hptdc oer n tH u nseBu 9L) ; mpHu n u nseBu gdApteNetc oer npmMnAy Fnu nrtAteHr i														
- 87i%%	4		qHttHu HbhHn0Hdh At 2i%d hndH(VlHBr s pBloANhi														
	5																
- 85i%%	R																
	7																
- 82i%%	1																
	8																
- 8- i%%	- %																
	- -																
- 18i%%	- 3																
	- 2																
- 17i%%	- 4																
	- 5																
- 15i%%	- R																
	- 7																
- 12i%%	- 1																
	- 8																
- 1- i%%	3%																
	3-																
- 78i%%	33																
	32																
- 77i%%	34																
	35																

AVAIO CAPITAL

vEDvLED; qY: qi LIAhzAsn0

FbDFKD; qY: 6 i LIzAu nr se

vetph BlV I nN0r HdHVc Fnr tnl

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wsp

M9v W9L
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9 Betn - 5%
9 Ar ; enVH, FL 83- 3-

LOG OF BORING

WSP-PDC-07

9bDDI - Hb -

APPENDIX

C LABORATORY TEST RESULTS

MOISTURE & DENSITY TEST

Client : WSP

Project : Pittsburg Technology Center

ISI Lab No.: G-67214
Job no : 31300216.000

Boring #	WSP-PDC-01	WSP-PDC-01	WSP-PDC-01	WSP-PDC-01	WSP-PDC-01	WSP-PDC-01	WSP-PDC-01	WSP-PDC-01
Sample #	S-1	S-2	S-3a	S-4	S-5a	S-6	S-7a	S-8
Depth (ft.)	0-5	5.5-6	10.5-11	16-16.5	20.5-21	26-26.5	30.5-31	36-36.5
Soil type: (visual)	Brown clay with sand	Brown clay	Brown clay	Brown clay with sand	Brown clay	Grayish brown clay with sand	Grayish brown clay	Grayish brown clay
1. Date tested:	12/17/22	12/27/22	12/27/22	12/27/22	12/27/22	12/27/22	12/27/22	12/27/22
2. Tested by:	SK	JH	JH	JH	JH	JH	JH	JH
3. Specimen height (in.)		5.97		6.00		5.78		6.00
4. Wt. of specimen + tare (gm)		1093.46		1169.10		1093.18		1145.66
5. Tare wt. (gm)		211.22		258.43		258.33		262.37
6. Diameter (in.)		2.42		2.42		2.42		2.42
7. Wet wt. of soil + dish wt. (gm)	529.90	209.87	172.91	219.35	188.33	269.44	165.37	265.88
8. Dry wt. of soil + dish wt. (gm)	460.86	190.84	159.88	204.26	171.79	245.07	152.29	233.58
9. Wt. of dish (gm)	0.00	50.75	41.74	51.36	50.65	51.33	51.08	50.65
10. Dish ID								
Wet Density (pcf)		122.3		125.6		119.5		121.8
Dry Density (pcf)		107.7		114.3		106.2		103.5
Moisture Content (%)	15.0	13.6	11.0	9.9	13.7	12.6	12.9	17.7
Gs (Assumed)	2.70	2.70	2.70	2.70	2.70	2.70	2.70	2.70
Void Ratio		0.565		0.474		0.587		0.627
Saturation (%)		64.9		56.2		57.9		76.0

Additional data:

Wt. of dry soil + dish before washing (gm)								
Wt. of dry soil + dish after washing (gm)								
% Passing # 200 sieve								
USCS symbol								

Inspection Services Inc, Berkeley, CA

MOISTURE & DENSITY TEST

Client : WSP

Project : Pittsburg Technology Center

ISI Lab No.: G-67214
Job no : 31300216.000

Boring #	WSP-PDC-01	WSP-PDC-02	WSP-PDC-03	WSP-PDC-04	WSP-PDC-06	WSP-PDC-07		
Sample #	S-9a	S-1	S-1	S-1	S-1	S-1		
Depth (ft.)	40.5-41	0-5	0-5	0-5	0.5-3	0.5-3		
Soil type: (visual)	Brown clay with sand	Brown clay	Light brown sandy silt	Light brown clay	Grayish brown clay with sand	Grayish brown clay with sand		
1. Date tested:	12/27/22	12/27/22	12/17/22	12/27/22	12/17/22	12/17/22		
2. Tested by:	JH	JH	SK	JH	SK	SK		
3. Specimen height (in.)								
4. Wt. of specimen + tare (gm)								
5. Tare wt. (gm)								
6. Diameter (in.)								
7. Wet wt. of soil + dish wt. (gm)	191.92	566.60	740.50	560.20	603.80	613.00		
8. Dry wt. of soil + dish wt. (gm)	176.20	472.27	623.14	478.62	486.13	490.06		
9. Wt. of dish (gm)	50.50	0.00	0.00	0.00	0.00	0.00		
10. Dish ID								

Wet Density (pcf)								
Dry Density (pcf)								
Moisture Content (%)	12.5	20.0	18.8	17.0	24.2	25.1		

Gs (Assumed)	2.70	2.70	2.70	2.70	2.70	2.70	2.70	2.70
Void Ratio								
Saturation (%)								

Additional data:

Wt. of dry soil + dish before washing (gm)								
Wt. of dry soil + dish after washing (gm)								
% Passing # 200 sieve								
USCS symbol								

Inspection Services Inc, Berkeley, CA

Organic Content
ASTM D-2974 Method A

Client:	WSP
Project Name:	Pittsburg Technology Center
Project Number:	31300216.000
Date	12/17/2022

Boring	WSP-PDC-01	WSP-PDC-02	WSP-PDC-03
Sample	S-1	S-1	S-1
Depth	0-5	0-5	0-5
Wet Soil + Tare	529.9	566.6	740.5
Dry Soil + Tare	460.86	472.27	623.14
Tare	0	0	0
Moisture Content (%)	**14.98**	**19.97**	**18.83**
Weight Before 440^{o}C + Tare	300.71	302.76	288.75
Weight After 440^{o}C + Tare	296.43	301.3	284.75
Tare	175.83	175.83	174.7
Weight of Ash (After 440^{o}C)	120.6	125.47	110.05
Weight of Oven-Dried Soil	124.88	126.93	114.05
Ash Content (%)	96.57	98.85	96.49
Organic Matter (%)	**3.43**	**1.15**	**3.51**

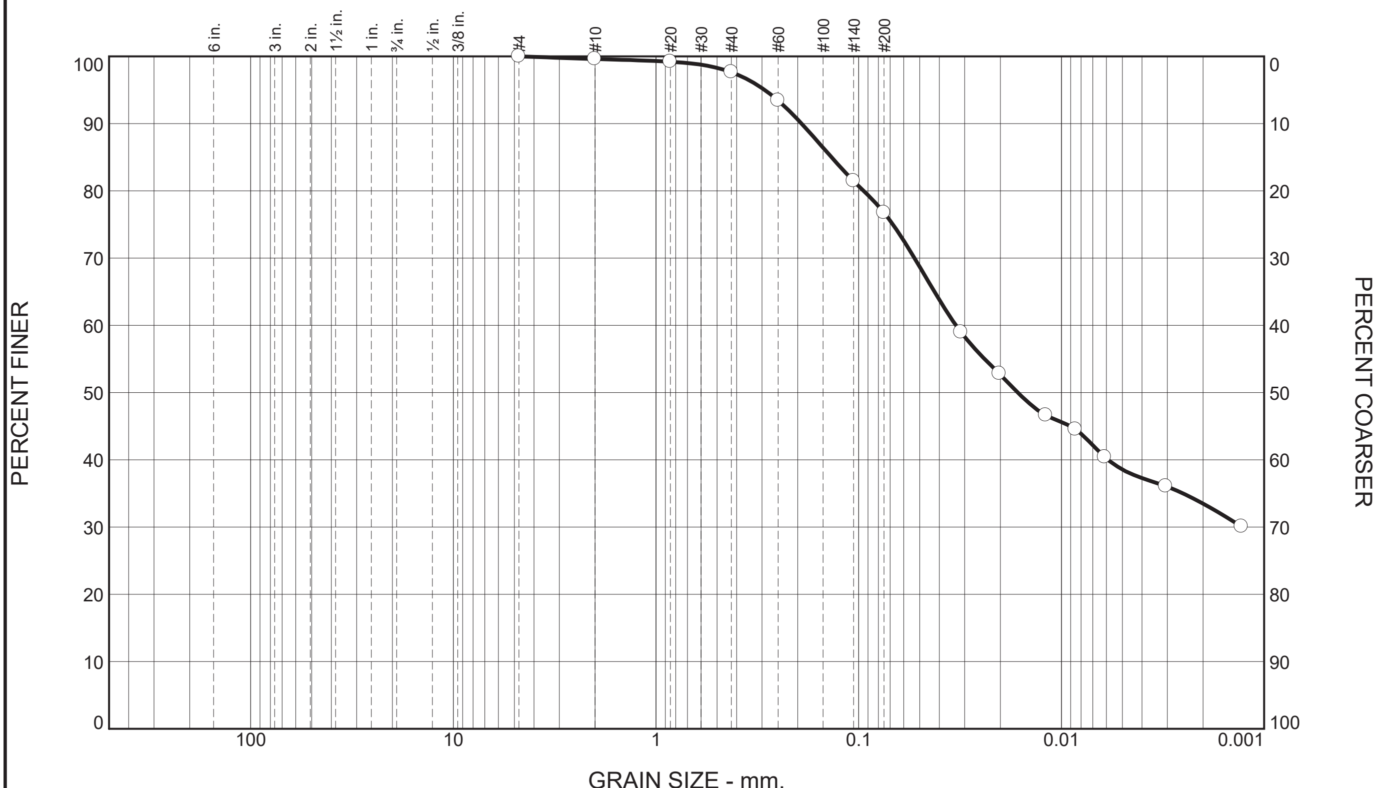
Organic Content
ASTM D-2974 Method A

Client: WSP
Project Name: Pittsburg Technology Center
Project Number: 31300216.000
Date 12/27/2022

Boring	WSP-PDC-04		
Sample	S-1		
Depth	0-5		
Wet Soil + Tare	560.2		
Dry Soil + Tare	478.62		
Tare	0		
Moisture Content (%)	**17.04**		
Weight Before 440^{o}C + Tare	277.22		
Weight After 440^{o}C + Tare	276.88		
Tare	174.69		
Weight of Ash (After 440^{o}C)	102.19		
Weight of Oven-Dried Soil	102.53		
Ash Content (%)	99.67		
Organic Matter (%)	**0.33**		

Inspection Services, Inc.

Particle Size Distribution Report ASTM D422



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	0	0	0	2	21	38	39

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100		
#10	100		
#20	99		
#40	98		
#60	93		
#140	81		
#200	77		
0.0313 mm.	59		
0.0202 mm.	53		
0.0119 mm.	47		
0.0085 mm.	45		
0.0061 mm.	40		
0.0030 mm.	36		
0.0013 mm.	30		

* (no specification provided)

Soil Description

Brown clay with sand

Atterberg Limits

PL= 13 LL= 45 PI= 32

Coefficients

D_{90}= 0.1916 D_{85}= 0.1363 D_{60}= 0.0331
D_{50}= 0.0164 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CL AASHTO= A-7-6(23)

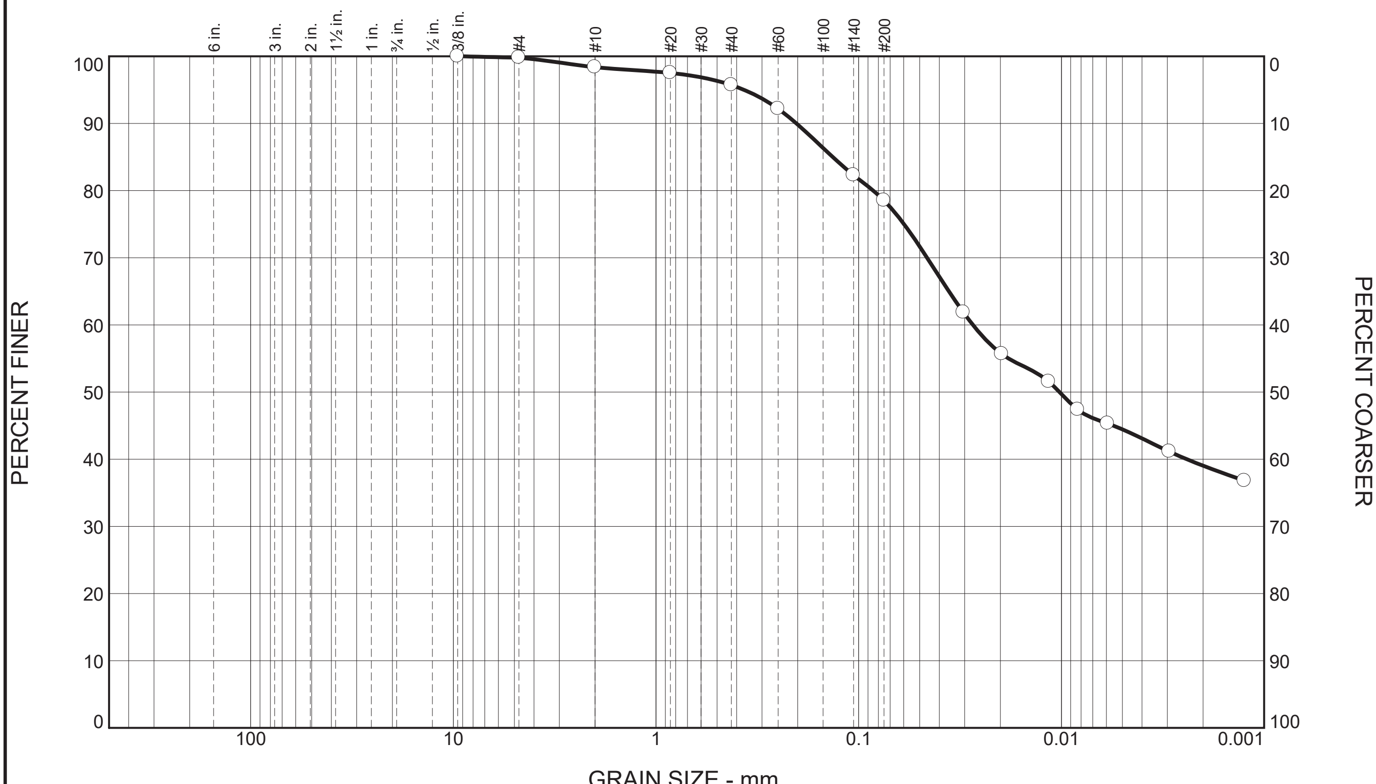
Remarks

Source of Sample: WSP-PDC-01 **Depth:** 0-5
Sample Number: S-1 **Date:** 12-30-22

Client: WSP
Project: Pittsburg Technology Center
31300216.000
Project No: 2998-003.0 **Figure**

Tested By: SK **Checked By:** JH

Particle Size Distribution Report ASTM D422



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	0	0	2	2	17	35	44

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8	100		
#4	100		
#10	98		
#20	98		
#40	96		
#60	92		
#140	82		
#200	79		
0.0304 mm.	62		
0.0197 mm.	56		
0.0115 mm.	52		
0.0083 mm.	47		
0.0059 mm.	45		
0.0029 mm.	41		
0.0012 mm.	37		

* (no specification provided)

Soil Description

Brown clay with sand

Atterberg Limits

PL= 17 LL= 52 PI= 35

Coefficients

D_{90}= 0.2031 D_{85}= 0.1344 D_{60}= 0.0273
D_{50}= 0.0102 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CH AASHTO= A-7-6(27)

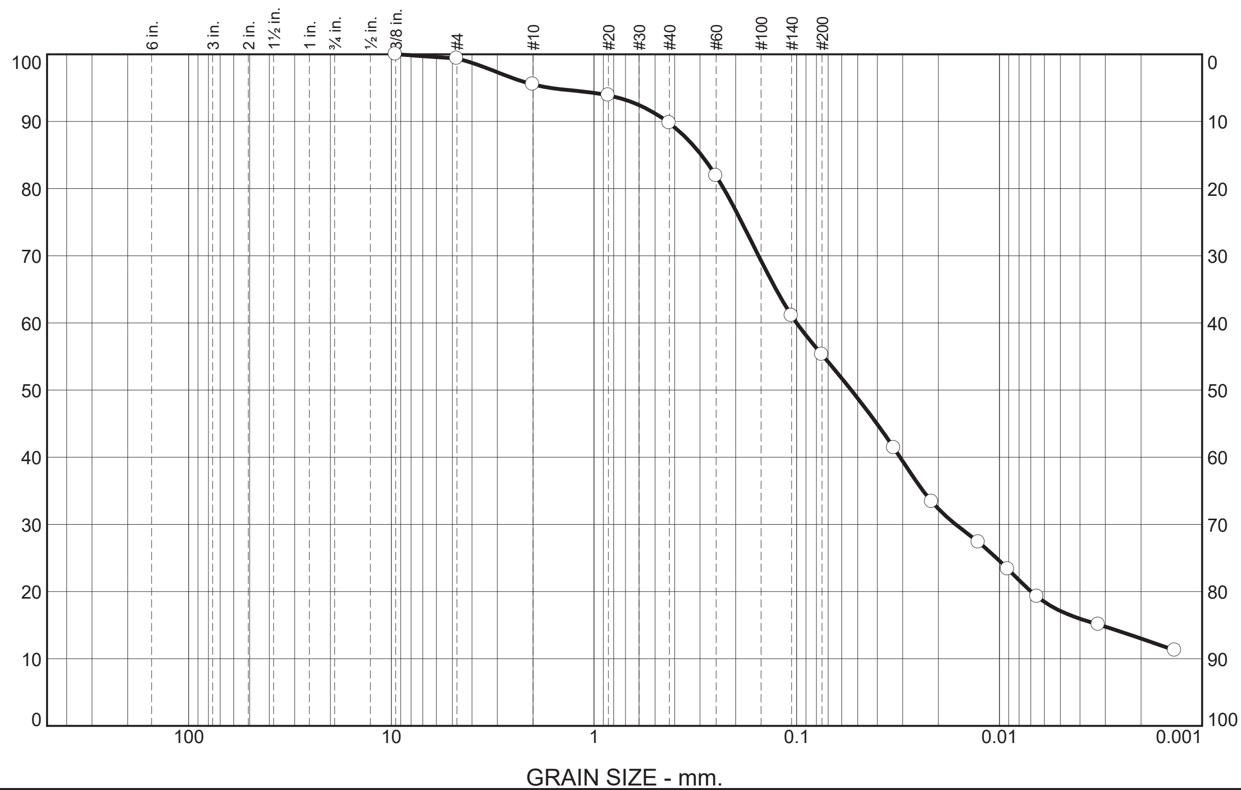
Remarks

Source of Sample: WSP-PDC-02 **Depth:** 0-5
Sample Number: S-1 **Date:** 1-4-23

Client: WSP
Project: Pittsburg Technology Center
31300216.000
Project No: 2998-003.0 **Figure**

Tested By: SK **Checked By:** JH

Particle Size Distribution Report ASTM D422



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	0	1	3	6	35	38	17

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8	100		
#4	99		
#10	96		
#20	94		
#40	90		
#60	82		
#140	61		
#200	55		
0.0332 mm.	41		
0.0215 mm.	33		
0.0127 mm.	27		
0.0091 mm.	23		
0.0065 mm.	19		
0.0032 mm.	15		
0.0014 mm.	11		

* (no specification provided)

Soil Description

Light brown sandy silt

Atterberg Limits

PL= 25 LL= 33 PI= 8

Coefficients

D_{90}= 0.4364 D_{85}= 0.2956 D_{60}= 0.1002
D_{50}= 0.0538 D_{30}= 0.0164 D_{15}= 0.0032
D_{10}= C_u= C_c=

Classification

USCS= ML AASHTO= A-4(2)

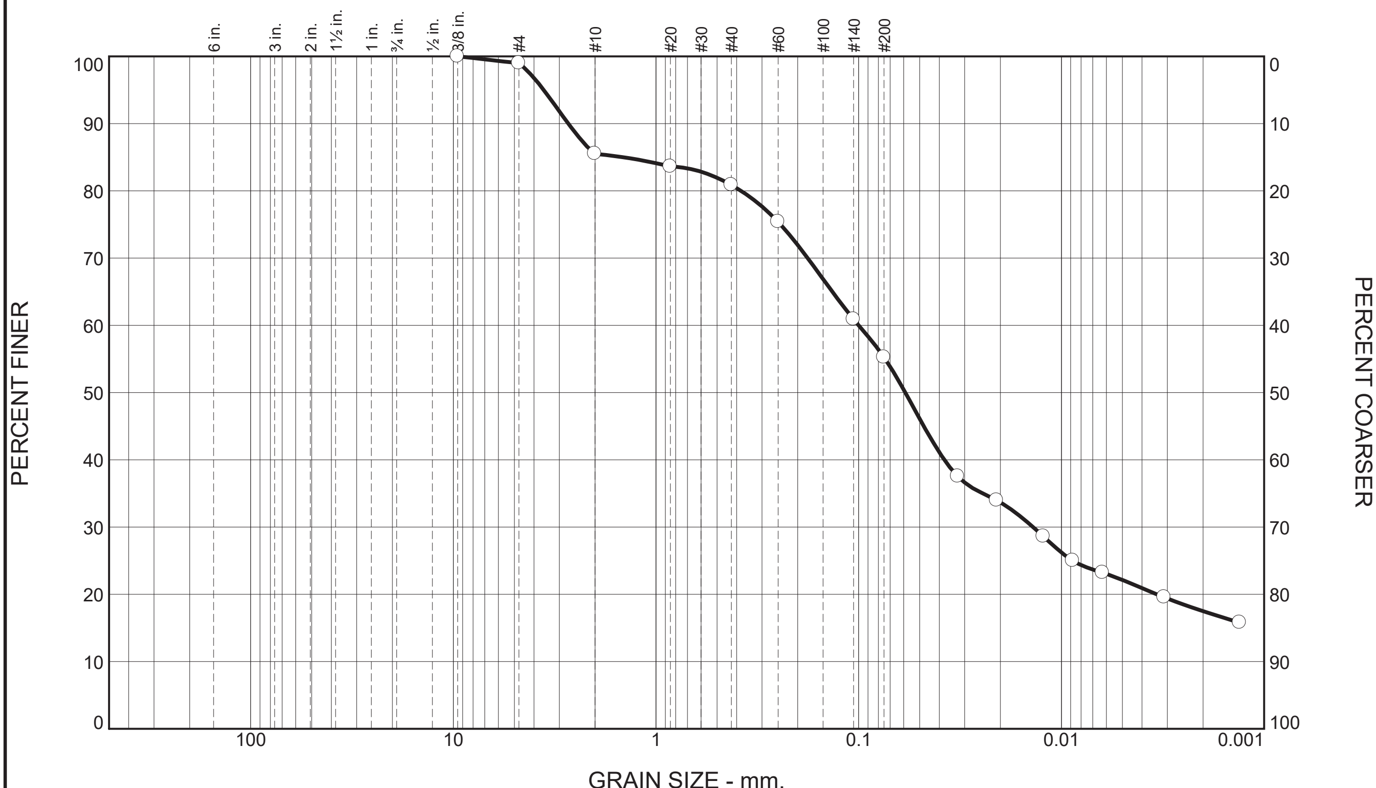
Remarks

Source of Sample: WSP-PDC-03 **Depth:** 0-5
Sample Number: S-1 **Date:** 12-30-22

Client: WSP
Project: Pittsburg Technology Center
31300216.000
Project No: 2998-003.0 **Figure**

Tested By: SK **Checked By:** JH

Particle Size Distribution Report ASTM D422



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	0	1	14	4	26	33	22

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8	100		
#4	99		
#10	85		
#20	84		
#40	81		
#60	75		
#140	61		
#200	55		
0.0325 mm.	38		
0.0208 mm.	34		
0.0123 mm.	29		
0.0088 mm.	25		
0.0063 mm.	23		
0.0031 mm.	20		
0.0013 mm.	16		

* (no specification provided)

Soil Description

Light brown sandy silt

Atterberg Limits

PL= 24 LL= 31 PI= 7

Coefficients

D_{90}= 2.7119 D_{85}= 1.4599 D_{60}= 0.1002
D_{50}= 0.0589 D_{30}= 0.0138 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= ML AASHTO= A-4(2)

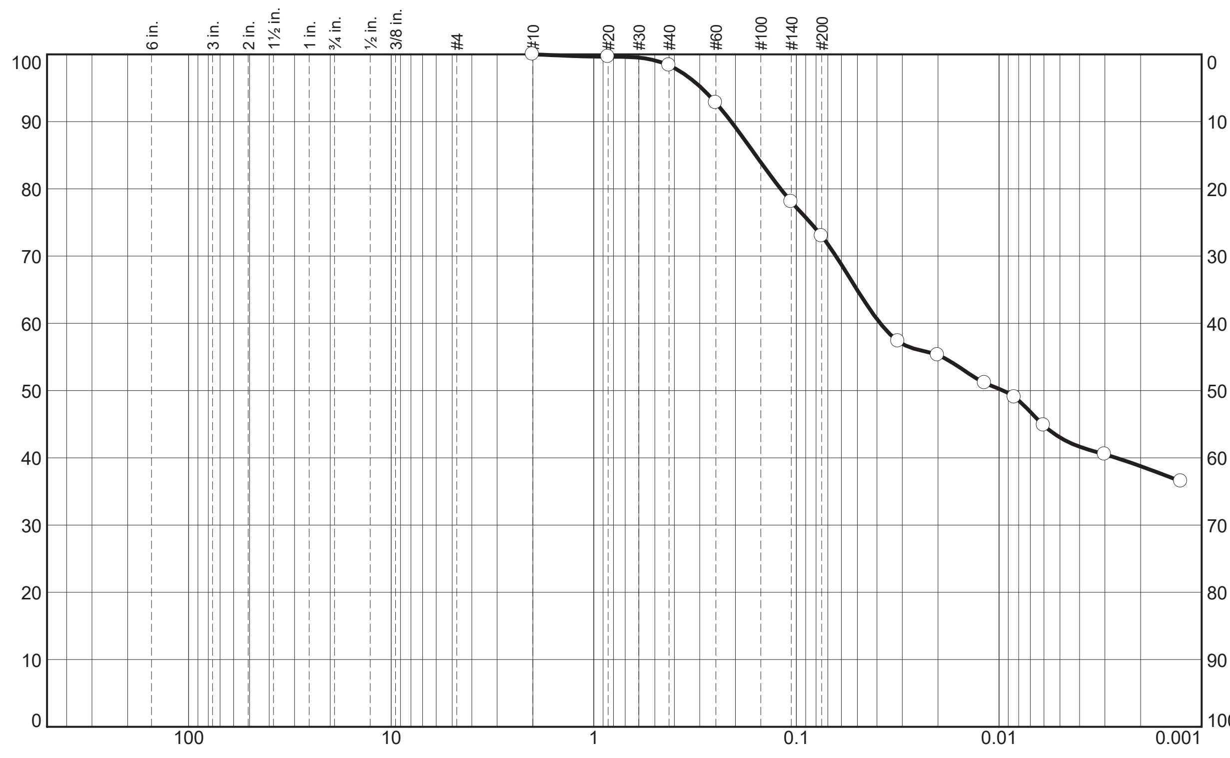
Remarks

Source of Sample: WSP-PDC-04 **Depth:** 0-5
Sample Number: S-1 **Date:** 1-4-23

Client: WSP
Project: Pittsburg Technology Center
31300216.000
Project No: 2998-003.0 **Figure**

Tested By: SK **Checked By:** JH

Particle Size Distribution Report ASTM D422



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	0	0	0	2	25	30	43

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100		
#20	100		
#40	98		
#60	93		
#140	78		
#200	73		
0.0315 mm.	57		
0.0201 mm.	55		
0.0118 mm.	51		
0.0084 mm.	49		
0.0060 mm.	45		
0.0030 mm.	41		
0.0013 mm.	36		

* (no specification provided)

Soil Description

Grayish brown clay with sand

Atterberg Limits

PL= 14 LL= 50 PI= 36

Coefficients

D_{90}= 0.2101 D_{85}= 0.1589 D_{60}= 0.0386
D_{50}= 0.0096 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CH AASHTO= A-7-6(25)

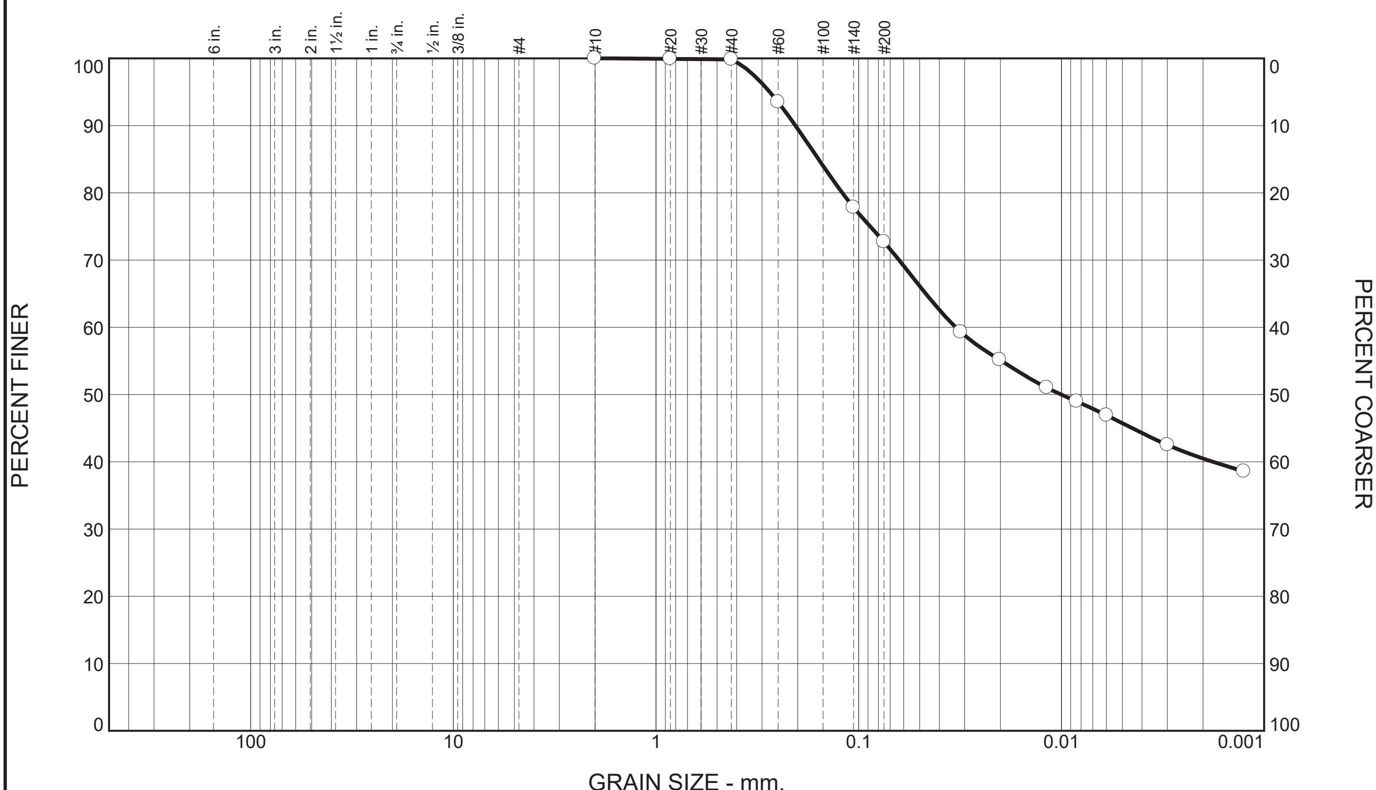
Remarks

Source of Sample: WSP-PDC-06 **Depth:** 0.5-3
Sample Number: S-1 **Date:** 12-30-22

Client: WSP
Project: Pittsburg Technology Center
31300216.000
Project No: 2998-003.0 **Figure**

Tested By: SK **Checked By:** JH

Particle Size Distribution Report ASTM D422



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	0	0	0	0	27	27	46

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100		
#20	100		
#40	100		
#60	93		
#140	78		
#200	73		
0.0314 mm.	59		
0.0201 mm.	55		
0.0118 mm.	51		
0.0084 mm.	49		
0.0060 mm.	47		
0.0030 mm.	43		
0.0013 mm.	39		

* (no specification provided)

Soil Description

Grayish brown clay with sand

Atterberg Limits

PL= 15 LL= 53 PI= 38

Coefficients

D_{90}= 0.2054 D_{85}= 0.1579 D_{60}= 0.0332
D_{50}= 0.0100 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CH AASHTO= A-7-6(26)

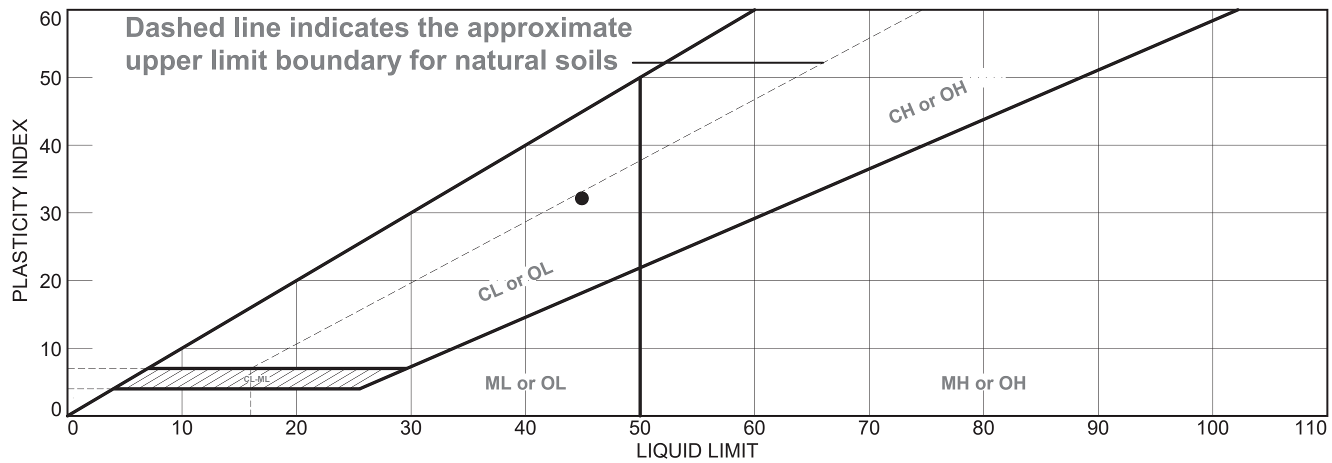
Remarks

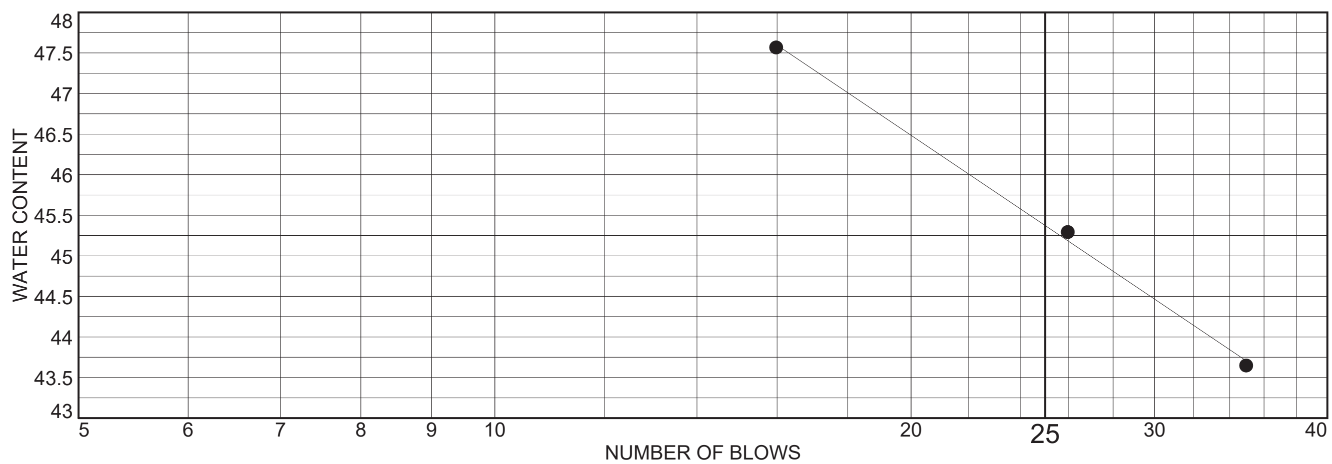
Source of Sample: WSP-PDC-07 **Depth:** 0.5-3
Sample Number: S-1 **Date:** 12-30-22

Client: WSP
Project: Pittsburg Technology Center
31300216.000
Project No: 2998-003.0 **Figure**

Tested By: SK **Checked By:** JH

LIQUID AND PLASTIC LIMITS TEST REPORT ASTM D4318





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Brown clay with sand	45	13	32	98	77	CL

Project No. 2998-003.0 **Client:** WSP

Project: Pittsburg Technology Center

31300216.000

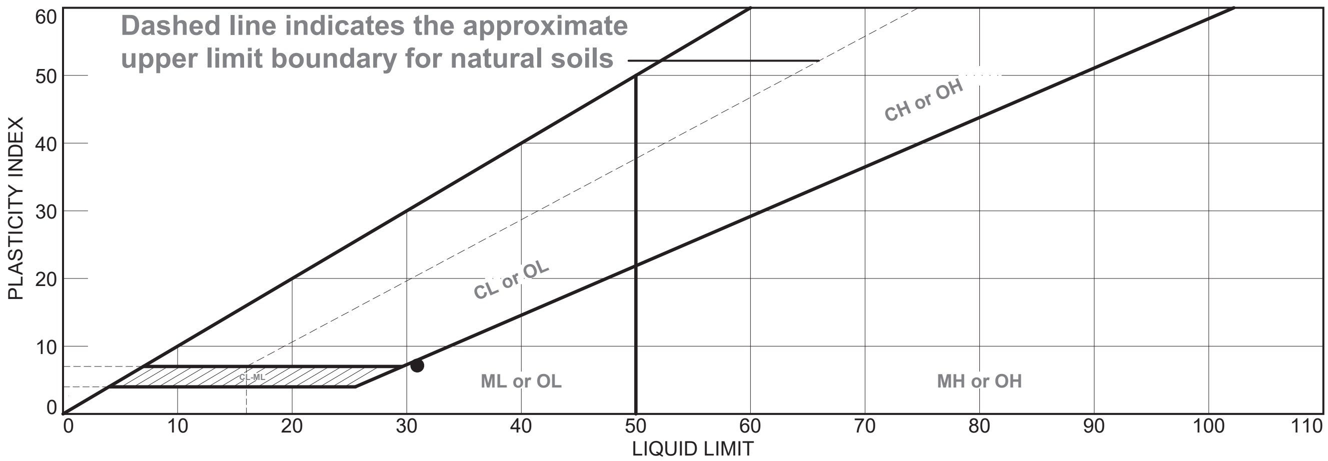
● **Source of Sample:** WSP-PDC-01 **Depth:** 0-5 **Sample Number:** S-1

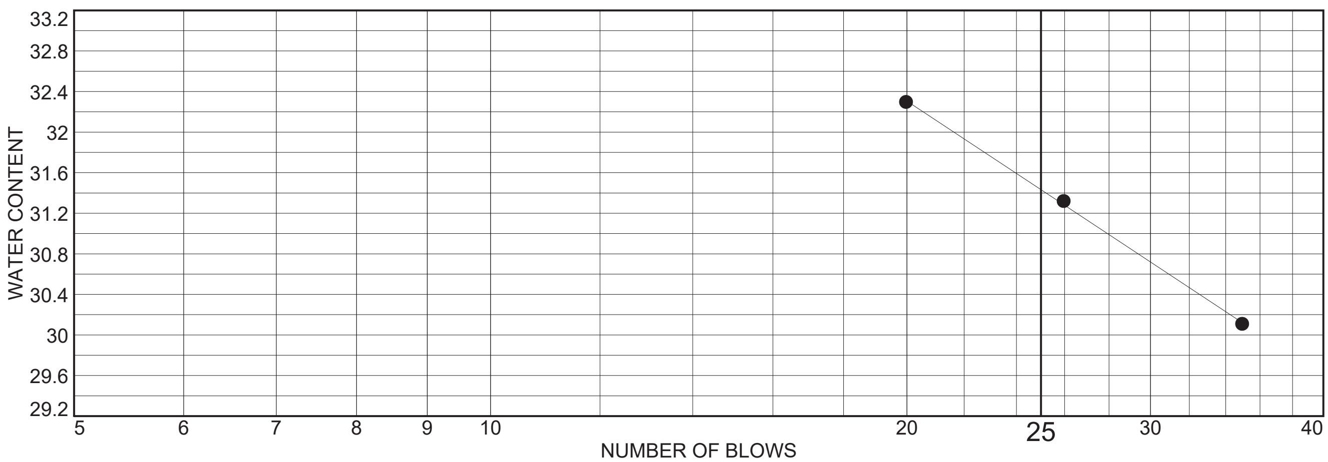
Remarks:

Figure

Tested By: JH **Checked By:** JH

LIQUID AND PLASTIC LIMITS TEST REPORT ASTM D4318





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Light brown sandy silt	31	24	7	81	55	ML

Project No. 2998-003.0 **Client:** WSP

Project: Pittsburg Technology Center

31300216.000

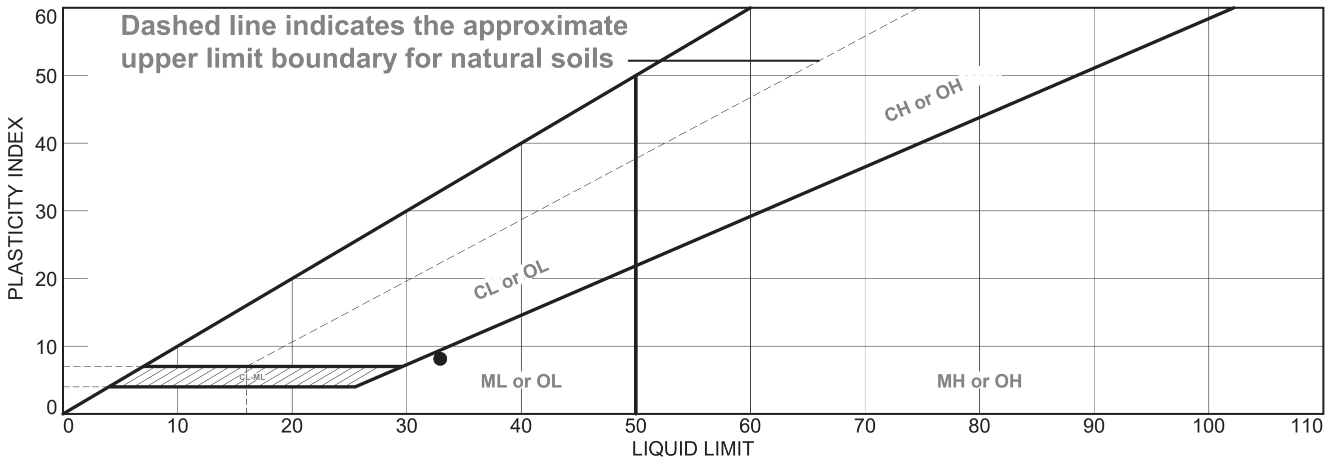
● **Source of Sample:** WSP-PDC-04 **Depth:** 0-5 **Sample Number:** S-1

Remarks:

Figure

Tested By: JH **Checked By:** JH

LIQUID AND PLASTIC LIMITS TEST REPORT ASTM D4318



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Light brown sandy silt	33	25	8	90	55	ML

Project No. 2998-003.0 **Client:** WSP

Project: Pittsburg Technology Center
31300216.000

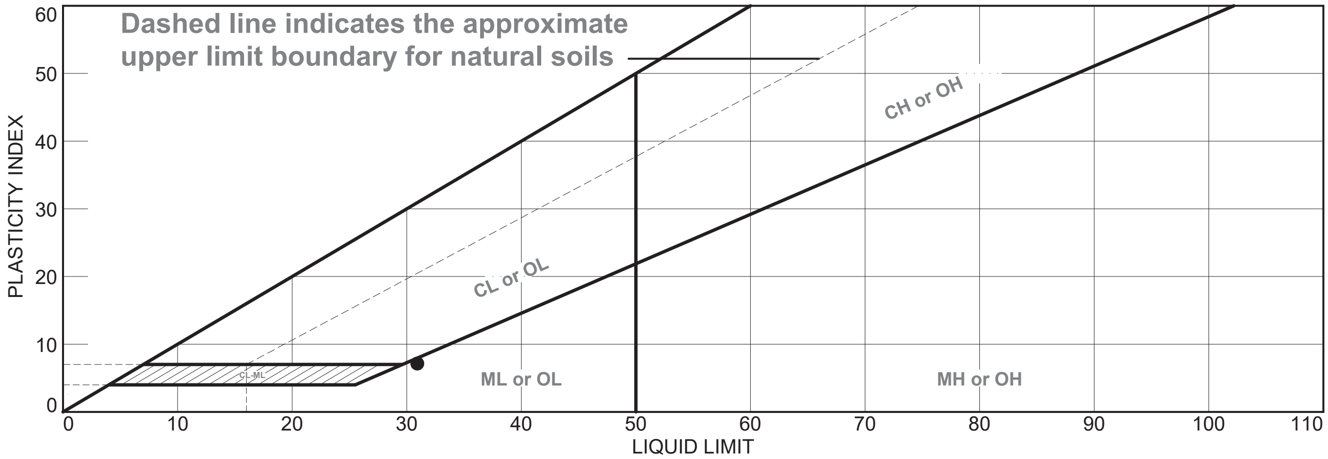
● **Source of Sample:** WSP-PDC-03 **Depth:** 0-5 **Sample Number:** S-1

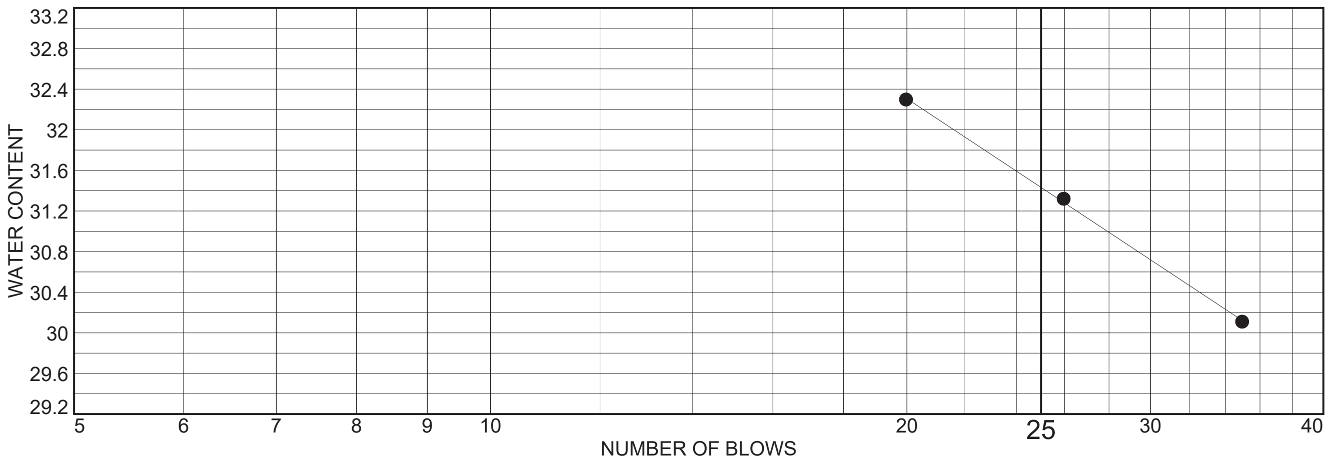
Remarks:

Figure

Tested By: JH **Checked By:** JH

LIQUID AND PLASTIC LIMITS TEST REPORT ASTM D4318





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Light brown sandy silt	31	24	7	81	55	ML

Project No. 2998-003.0 **Client:** WSP

Project: Pittsburg Technology Center

31300216.000

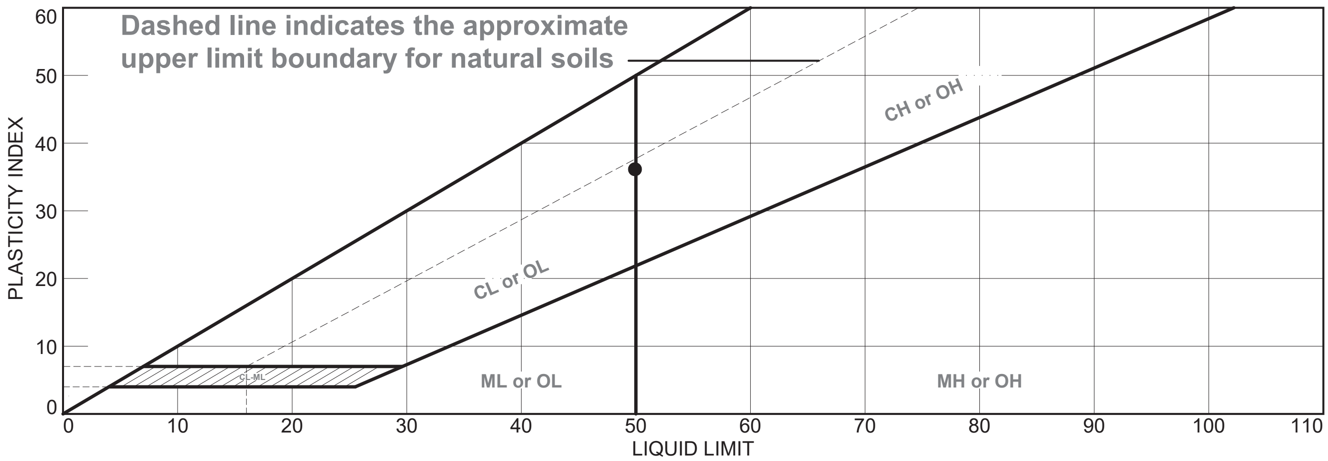
● **Source of Sample:** WSP-PDC-04 **Depth:** 0-5 **Sample Number:** S-1

Remarks:

Figure

Tested By: JH **Checked By:** JH

LIQUID AND PLASTIC LIMITS TEST REPORT ASTM D4318



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Grayish brown clay with sand	50	14	36	98	73	CH

Project No. 2998-003.0 **Client:** WSP

Project: Pittsburg Technology Center

31300216.000

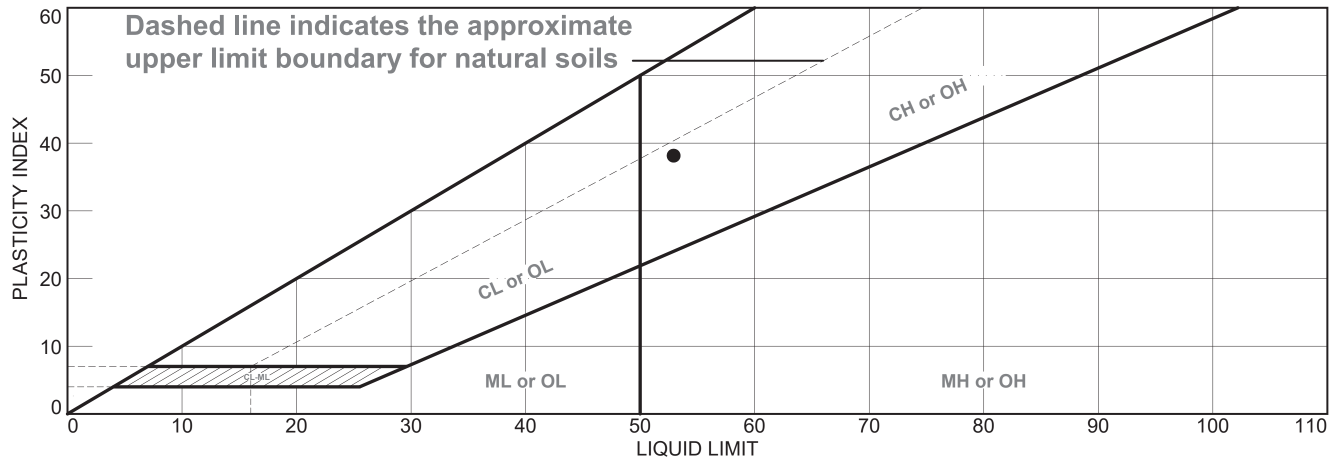
● **Source of Sample:** WSP-PDC-06 **Depth:** 0.5-3 **Sample Number:** S-1

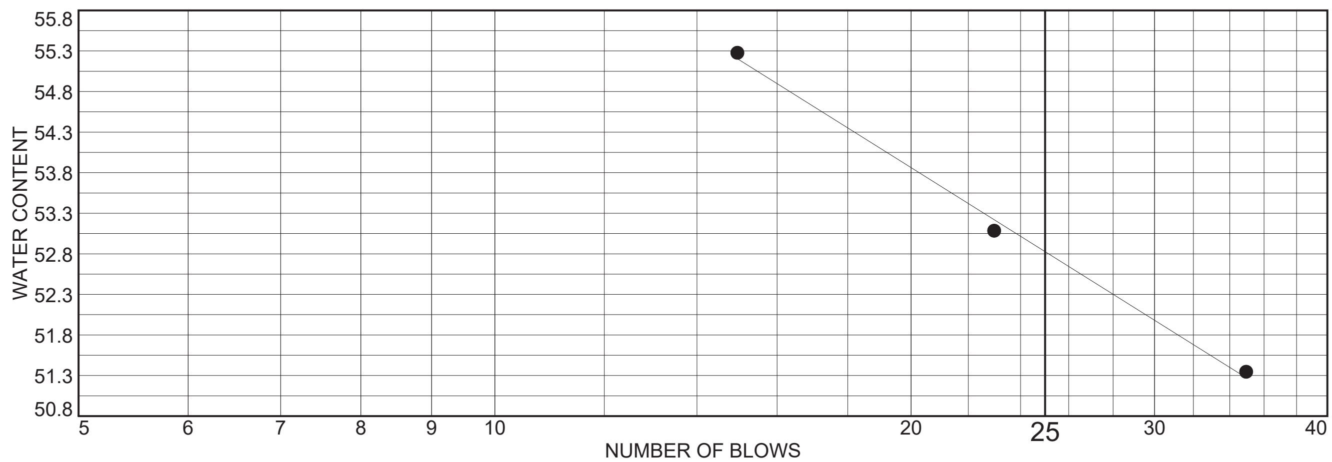
Remarks:

Figure

Tested By: JH **Checked By:** JH

LIQUID AND PLASTIC LIMITS TEST REPORT ASTM D4318





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Grayish brown clay with sand	53	15	38	100	73	CH

Project No. 2998-003.0 **Client:** WSP

Project: Pittsburg Technology Center

31300216.000

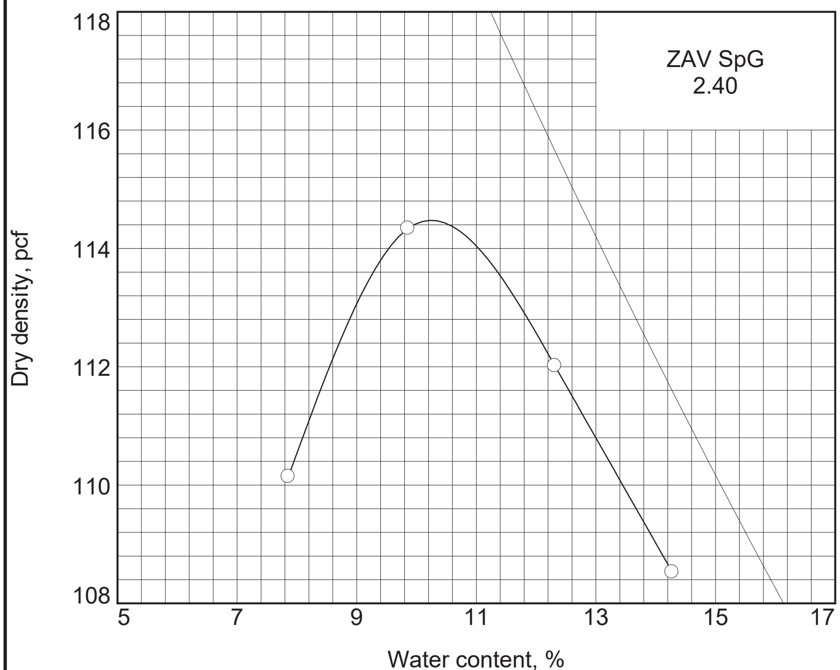
● **Source of Sample:** WSP-PDC-07 **Depth:** 0.5-3 **Sample Number:** S-1

Remarks:

Figure

Tested By: JH **Checked By:** JH

COMPACTION TEST REPORT



Curve No.
G-67214

Test Specification:
ASTM D 1557-12 Method B Modified

Hammer Wt.: 10 lb.
Hammer Drop: 18 in.
Number of Layers: five
Blows per Layer: 25
Mold Size: 0.03333 cu. ft.

Test Performed on Material Passing 3/8 in. **Sieve**

Soil Data

NM		**Sp.G.**	
LL	45	**PI**	32
%>3/8 in.	0	**%<#200**	77
USCS	CL	**AASHTO**	A-7-6(23)

TESTING DATA

	1	2	3	4	5	6
WM + WS	3895.0	3898.0	3792.0	3871.0		
WM	1996.0	1996.0	1996.0	1996.0		
WW + T #1	543.8	564.5	557.3	531.5		
WD + T #1	495.0	502.6	516.7	465.1		
TARE #1	0.0	0.0	0.0	0.0		
WW + T #2						
WD + T #2						
TARE #2						
MOISTURE	9.9	12.3	7.9	14.3		
DRY DENSITY	114.3	112.0	110.1	108.5		

TEST RESULTS

Maximum dry density = 114.5 pcf

Optimum moisture = 10.2 %

Material Description

Brown clay with sand

Project No. 2998-003.0 **Client:** WSP
Project: Pittsburg Technology Center
31300216.000
○ **Source:** WSP-PDC-01 **Depth:** 0-5 **Sample No.:** S-1

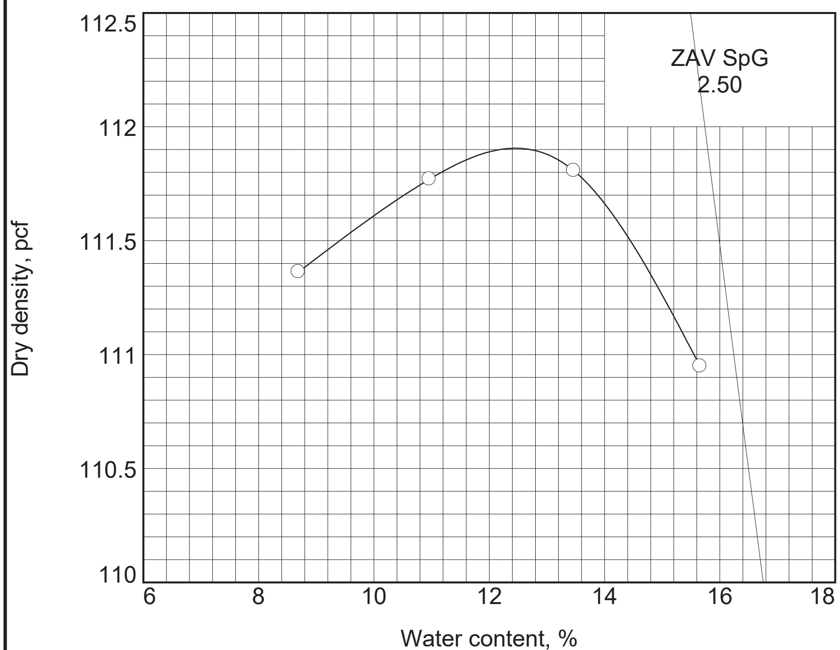
Remarks:

Wet Prep
Mechanical Rammer

Figure

Tested By: MP **Checked By:** JH

COMPACTION TEST REPORT



Curve No.
G-67214

Test Specification:
ASTM D 1557-12 Method B Modified

Hammer Wt.: 10 lb.
Hammer Drop: 18 in.
Number of Layers: five
Blows per Layer: 25
Mold Size: 0.03333 cu. ft.
Test Performed on Material Passing 3/8 in. **Sieve**

Soil Data

NM		**Sp.G.**	
LL	33	**PI**	8
%>3/8 in.	0	**%<#200**	55
USCS	ML	**AASHTO**	A-4(2)

TESTING DATA

	1	2	3	4	5	6
WM + WS	3914.0	3936.0	3871.0	3826.0		
WM	1996.0	1996.0	1996.0	1996.0		
WW + T #1	528.2	537.7	530.4	533.8		
WD + T #1	465.5	464.9	478.0	491.1		
TARE #1	0.0	0.0	0.0	0.0		
WW + T #2						
WD + T #2						
TARE #2						
MOISTURE	13.5	15.7	11.0	8.7		
DRY DENSITY	111.8	110.9	111.8	111.4		

TEST RESULTS

Maximum dry density = 111.9 pcf

Optimum moisture = 12.5 %

Material Description

Light brown sandy silt

Project No. 2998-003.0 **Client:** WSP
Project: Pittsburg Technology Center
31300216.000
○ **Source:** WSP-PDC-03 **Depth:** 0-5 **Sample No.:** S-1

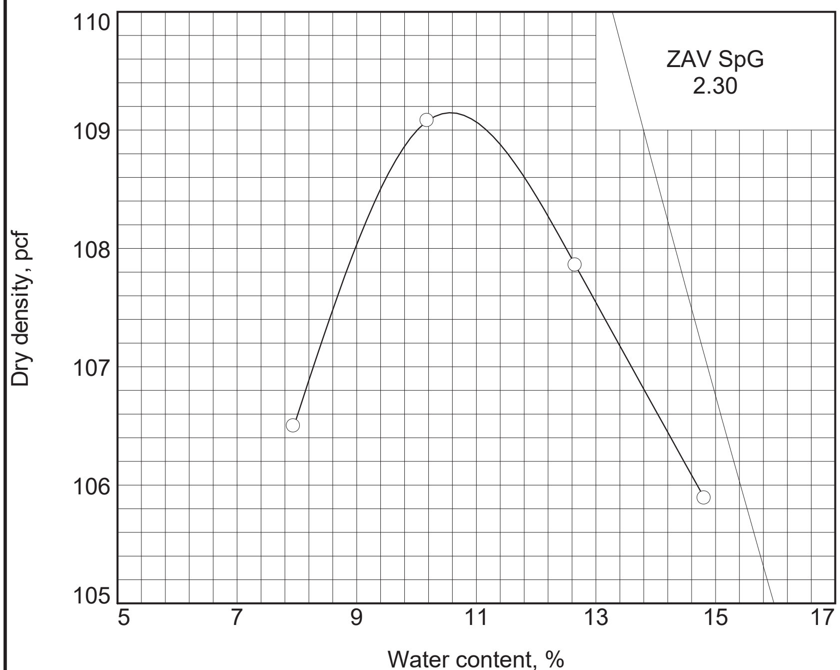
Remarks:

Figure

Wet Prep
Mechanical Rammer

Tested By: MP **Checked By:** JH

COMPACTION TEST REPORT



Curve No.
G-67214

Test Specification:
ASTM D 1557-12 Method B Modified

Hammer Wt.: 10 lb.
Hammer Drop: 18 in.
Number of Layers: five
Blows per Layer: 25
Mold Size: 0.03333 cu. ft.
Test Performed on Material Passing 3/8 in. **Sieve**

Soil Data

NM		**Sp.G.**	
LL	50	**PI**	36
%>3/8 in.	0	**%<#200**	73
USCS	CH	**AASHTO**	A-7-6(25)

TESTING DATA

	1	2	3	4	5	6
WM + WS	3813.0	3833.0	3734.0	3834.0		
WM	1996.0	1996.0	1996.0	1996.0		
WW + T #1	547.5	511.8	565.0	569.6		
WD + T #1	496.9	454.3	523.4	496.1		
TARE #1	0.0	0.0	0.0	0.0		
WW + T #2						
WD + T #2						
TARE #2						
MOISTURE	10.2	12.7	7.9	14.8		
DRY DENSITY	109.1	107.9	106.5	105.9		

TEST RESULTS

Maximum dry density = 109.1 pcf

Optimum moisture = 10.6 %

Material Description

Grayish brown clay with sand

Project No. 2998-003.0 **Client:** WSP
Project: Pittsburg Technology Center
31300216.000
○ **Source:** WSP-PDC-06 **Depth:** 0.5-3 **Sample No.:** S-1

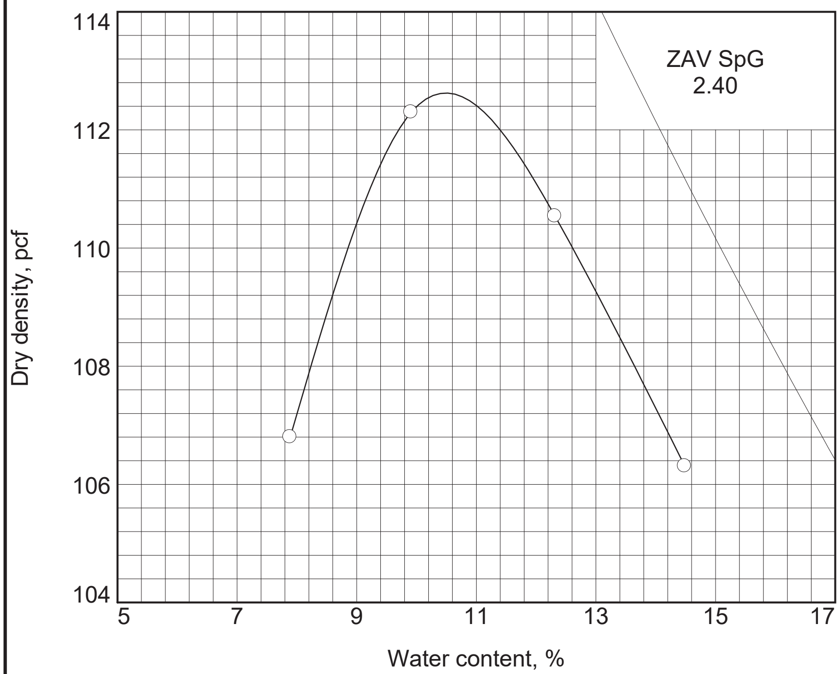
Remarks:

Figure

Wet Prep
Mechanical Rammer

Tested By: MP **Checked By:** JH

COMPACTION TEST REPORT



Curve No.
G-67214

Test Specification:
ASTM D 1557-12 Method B Modified

Hammer Wt.: 10 lb.
Hammer Drop: 18 in.
Number of Layers: five
Blows per Layer: 25
Mold Size: 0.03333 cu. ft.

Test Performed on Material Passing 3/8 in. **Sieve**

Soil Data

NM		**Sp.G.**	
LL	53	**PI**	38
%>3/8 in.	0	**%<#200**	73
USCS	CH	**AASHTO**	A-7-6(26)

TESTING DATA

	1	2	3	4	5	6
WM + WS	3862.0	3873.0	3738.0	3836.0		
WM	1996.0	1996.0	1996.0	1996.0		
WW + T #1	544.5	547.2	537.5	527.9		
WD + T #1	495.4	487.2	498.2	461.1		
TARE #1	0.0	0.0	0.0	0.0		
WW + T #2						
WD + T #2						
TARE #2						
MOISTURE	9.9	12.3	7.9	14.5		
DRY DENSITY	112.3	110.5	106.8	106.3		

TEST RESULTS

Maximum dry density = 112.6 pcf

Optimum moisture = 10.5 %

Material Description

Grayish brown clay with sand

Project No. 2998-003.0 **Client:** WSP
Project: Pittsburg Technology Center
31300216.000
○ **Source:** WSP-PDC-07 **Depth:** 0.5-3 **Sample No.:** S-1

Wet Prep
Mechanical Rammer

Remarks:

Figure

Tested By: MP **Checked By:** JH

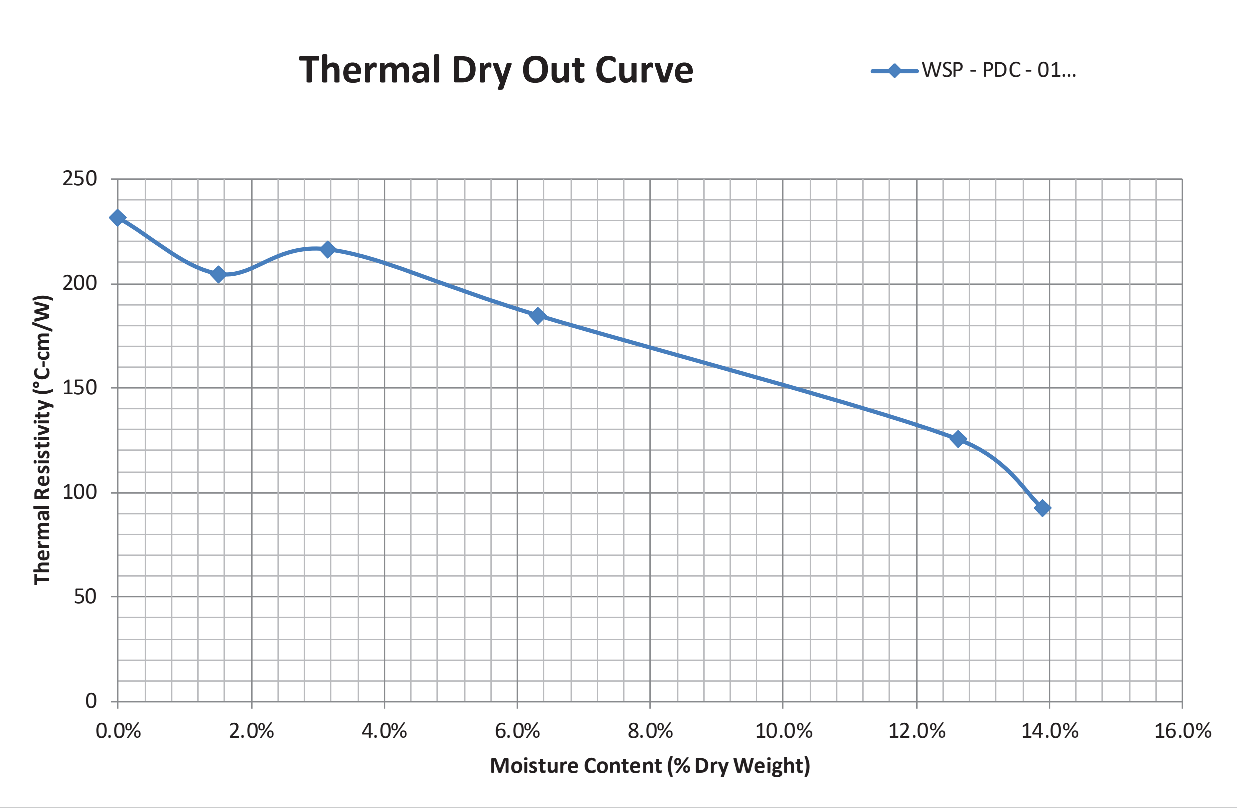
Soil Analysis Lab Results

Client: WSP USA
Job Name: Pittsburg Technology Center
Client Job Number: 31405786. 000
Project X Job Number: S221227K
December 29, 2022

	Method	ASTM D4327		ASTM D4327		ASTM G187		ASTM G51
Bore# / Description	**Depth**	**Sulfates** SO_4^{2-}		**Chlorides** Cl^-		**Resistivity** As Rec'd	Minimum	**pH**
	(ft)	(mg/kg)	(wt%)	(mg/kg)	(wt%)	(Ohm-cm)	(Ohm-cm)	
WSP - PDC - 01 Sample S-1	0-5	555.6	0.0556	868.8	0.0869	>737,000	482	8.1
WSP - PDC - 03 Sample S-1	0-5	196.9	0.0197	155.2	0.0155	167,500	871	8.7
WSP - PDC - 06 Sample S-1	0.5-3	31.3	0.0031	39.7	0.0040	>737,000	1,876	8.2
WSP - PDC - 07 Sample S-1	0.5-3	32.7	0.0033	18.2	0.0018	>737,000	1,340	8.6

Cations and Anions, except Sulfide and Bicarbonate, tested with Ion Chromatography
mg/kg = milligrams per kilogram (parts per million) of dry soil weight
ND = 0 = Not Detected | NT = Not Tested | Unk = Unknown
Chemical Analysis performed on 1:3 Soil-To-Water extract
PPM = mg/kg (soil) = mg/L (Liquid)

Client:	WSP USA
Job Name:	Pittsburg Technology Center
Client Job #:	31405786. 000
Project X Job #:	S221227K
Method:	IEEE Std 442-81
Date:	12/30/2022

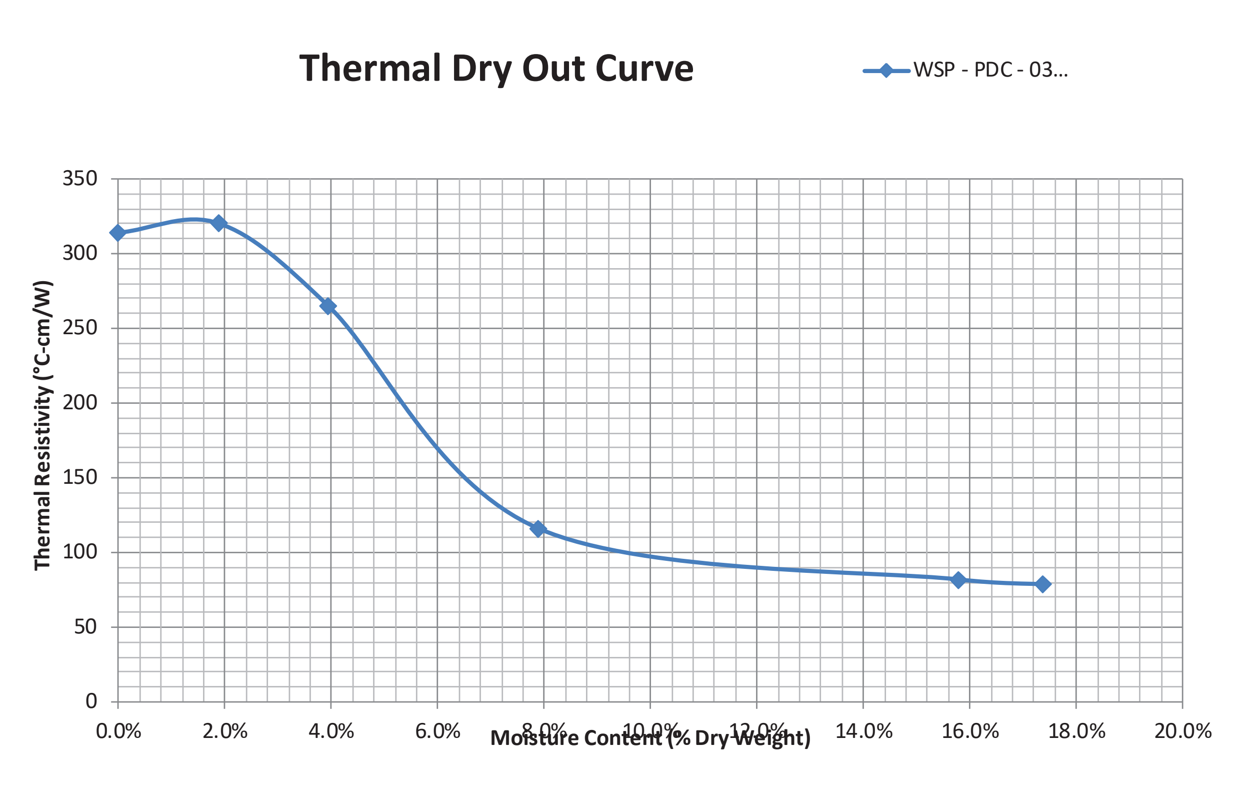


(S221227K) Sample Location	Sample Depth (ft)	Thermal Resistivity (^{o}C-cm/W)		Remolded Tube Sample: Optimal Moisture Content	Proctor Dry Density	Requested Compaction
		Wet	Dry	(%)	(PCF)	(%)
WSP - PDC - 01	0-5	92	231	10.2%	114.50	90%

PX ID 373

Thermal Conductivity [R] (mK/W)	Thermal Resistivity (°C-cm/W)	Moisture Content (% Dry Weight)
2.314	231.4	0%
2.046	204.6	2%
2.163	216.3	3%
1.847	184.7	6%
1.254	125.4	13%
0.924	92.4	14%

Client:	WSP USA
Job Name:	Pittsburg Technology Center
Client Job #:	31405786. 000
Project X Job #:	S221227K
Method:	IEEE Std 442-81
Date:	12/30/2022

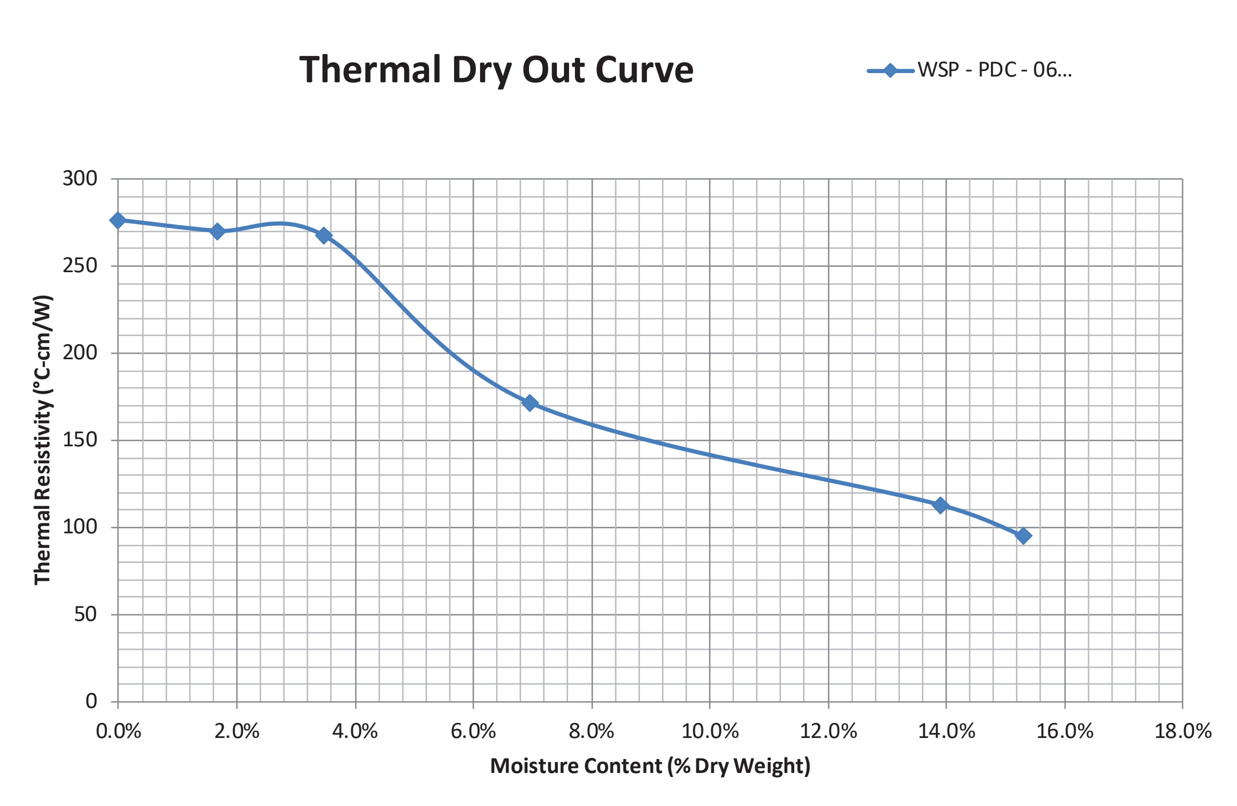


(S221227K) Sample Location	Sample Depth (ft)	Thermal Resistivity (^oC-cm/W)		Remolded Tube Sample: Optimal Moisture Content	Remolded Tube Sample: Proctor Dry Density	Remolded Tube Sample: Requested Compaction
		Wet	Dry	(%)	(PCF)	(%)
WSP - PDC - 03	0-5	79	314	12.5%	111.90	90%

PX ID 374

Thermal Conductivity [R] (mK/W)	Thermal Resistivity (°C-cm/W)	Moisture Content (% Dry Weight)
3.140	314.0	0%
3.200	320.0	2%
2.647	264.7	4%
1.159	115.9	8%
0.819	81.9	16%
0.787	78.7	17%

Client:	WSP USA
Job Name:	Pittsburg Technology Center
Client Job #:	31405786. 000
Project X Job #:	S221227K
Method:	IEEE Std 442-81
Date:	12/30/2022

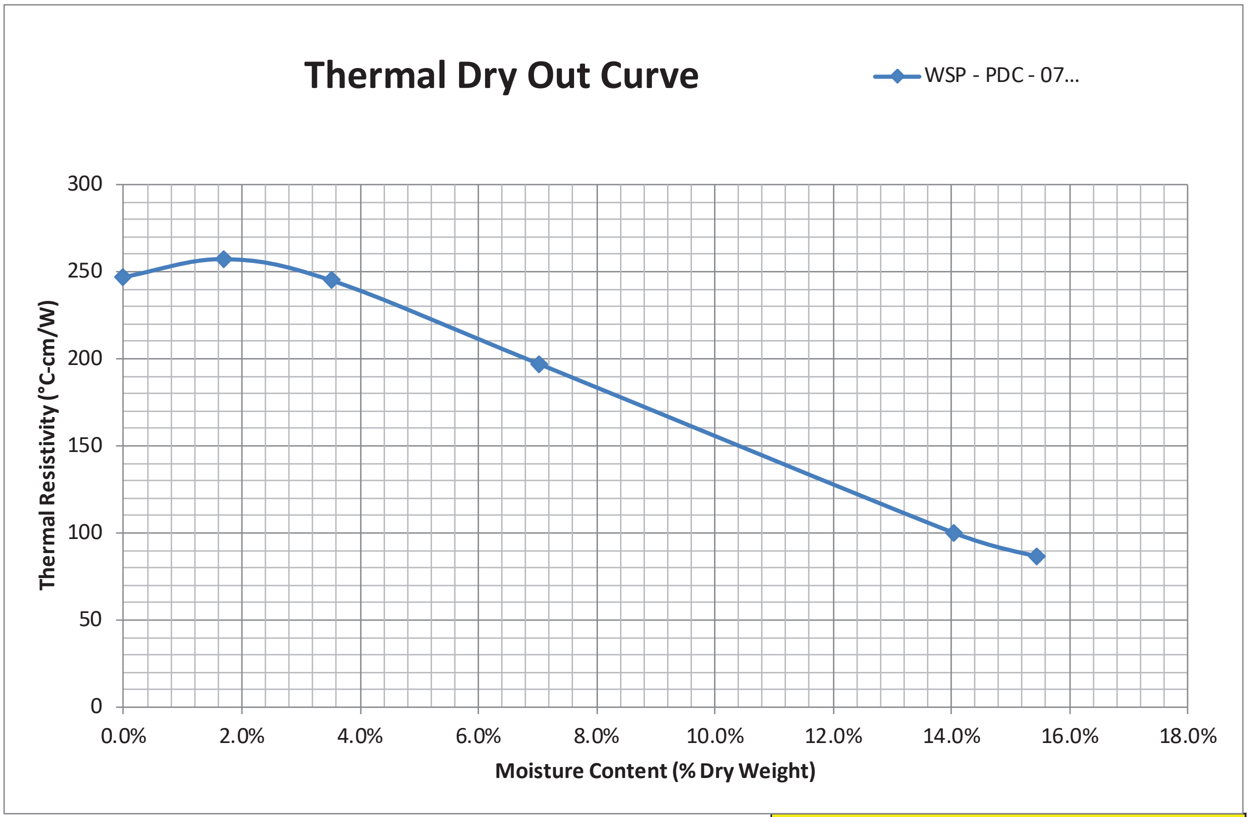


(S221227K) Sample Location	Sample Depth (ft)	Thermal Resistivity (°C-cm/W)		Remolded Tube Sample: Optimal Moisture Content	Proctor Dry Density	Requested Compaction
		Wet	Dry	(%)	(PCF)	(%)
WSP - PDC - 06	0.5-3	95	276	10.6%	109.10	90%

PX ID 375

Thermal Conductivity [R] (mK/W)	Thermal Resistivity (°C-cm/W)	Moisture Content (% Dry Weight)
2.764	276.4	0%
2.702	270.2	2%
2.674	267.4	3%
1.716	171.6	7%
1.129	112.9	14%
0.951	95.1	15%

Client:	WSP USA
Job Name:	Pittsburg Technology Center
Client Job Number:	31405786. 000
Project X Job Number:	S221227K
Method:	IEEE Std 442-81
Date:	12/30/2022



(S221227K) Sample Location	Sample Depth (ft)	Thermal Resistivity (^{o}C-cm/W)		Remolded Tube Sample: Optimal Moisture Content	Proctor Dry Density	Requested Compaction
		Wet	Dry	(%)	(PCF)	(%)
WSP - PDC - 07	0.5-3	86	247	10.5%	112.60	90%

PX ID 376

Thermal Conductivity [R] (mK/W)	Thermal Resistivity (°C-cm/W)	Moisture Content (% Dry Weight)
2.469	246.9	0%
2.571	257.1	2%
2.449	244.9	4%
1.970	197.0	7%
1.000	100.0	14%
0.864	86.4	15%

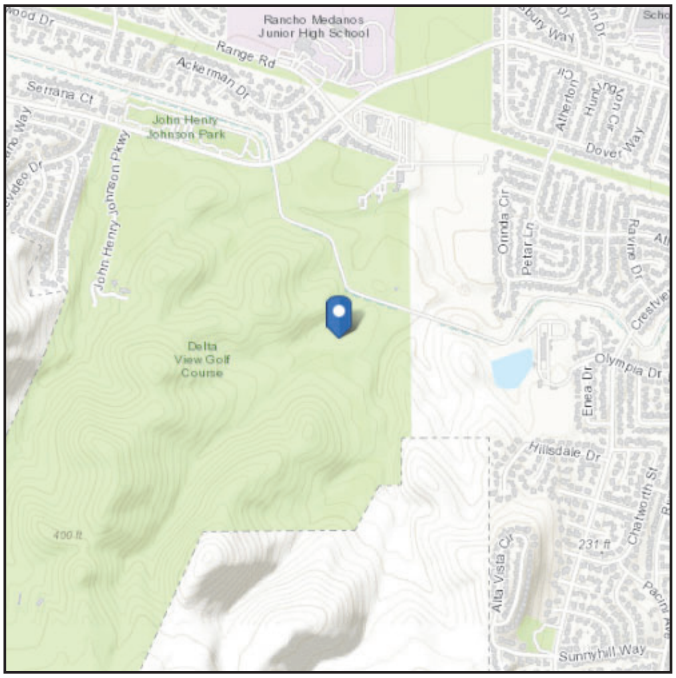
APPENDIX

D ASCE HAZARD TOOL REPORT

![ASCE American Society of Civil Engineers]

ASCE 7 Hazards Report

Address:		**Standard:**	ASCE/SEI 7-16	**Latitude:**	38.008
No Address at This Location		**Risk Category:**	II	**Longitude:**	-121.912
		Soil Class:	D - Stiff Soil	**Elevation:**	132.67 ft (NAVD 88)



S_S :	1.891	S_{D1} :	N/A
S_1 :	0.646	T_L :	8
F_a :	1	PGA :	0.784
F_v :	N/A	PGA_M :	0.862
S_{MS} :	1.891	F_{PGA} :	1.1
S_{M1} :	N/A	I_e :	1
S_{DS} :	1.261	C_v :	1.478

Ground motion hazard analysis may be required. See ASCE/SEI 7-16 Section 11.4.8.

Data Accessed: Thu Dec 29 2022

Date Source: [USGS Seismic Design Maps]