Engineering Geologic and Geotechnical Investigation Report

Proposed Water System Improvements for the Garberville Sanitary District, Humboldt County, California

Prepared for:

Garberville Sanitary District

August 2023

022067

SHN

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Reference: 022067

Table of Contents

Appendices

List of Illustrations

Figures **Follows Page**

Tables **Page**

Abbreviations and Acronyms

Units of Measure

Term	Definition
μm	micrometers
H	height of the wall
H:V	horizontal to vertical
mg/kg	milligrams per kilogram
mm	millimeters
mV	millivolts
ohms-cm	ohms-centimeter
pcf	pounds per cubic foot
pci	pounds per cubic inch
ppm	parts per million
psf	pounds per square foot
psi	pounds per square inch

Additional Terms

Term	Definition
AB	aggregate base
APN	Assessor's parcel number
ASCE	American Society of Civil Engineers
ASTM	ASTM-International
BGS	below ground surface
CBC	California Building Code
CEQA	California Environmental Quality Act
CGS	California Geologic Survey
GSD	Garberville Sanitary District
I.D.	internal diameter
MTJ	Mendocino Triple Junction
NAVD88	North American vertical datum, 1988
OSHA	Occupational Safety and Health Administration
PGA_M	Site modified peak ground acceleration
redox	reduction and oxidation potential
SE	sand equivalent
SPT	standard penetration test

Introduction

This report presents the results of SHN's investigation of geologic and geotechnical site conditions for proposed water distribution system improvements for the Garberville Sanitary District (GSD), in Garberville, Humboldt County, California. Improvements to the water distribution system are proposed to improve stability and reliability of the existing piping. Proposed new water storage tanks are intended to increase the water storage capacity for potable water and fire suppression for the community of Garberville.

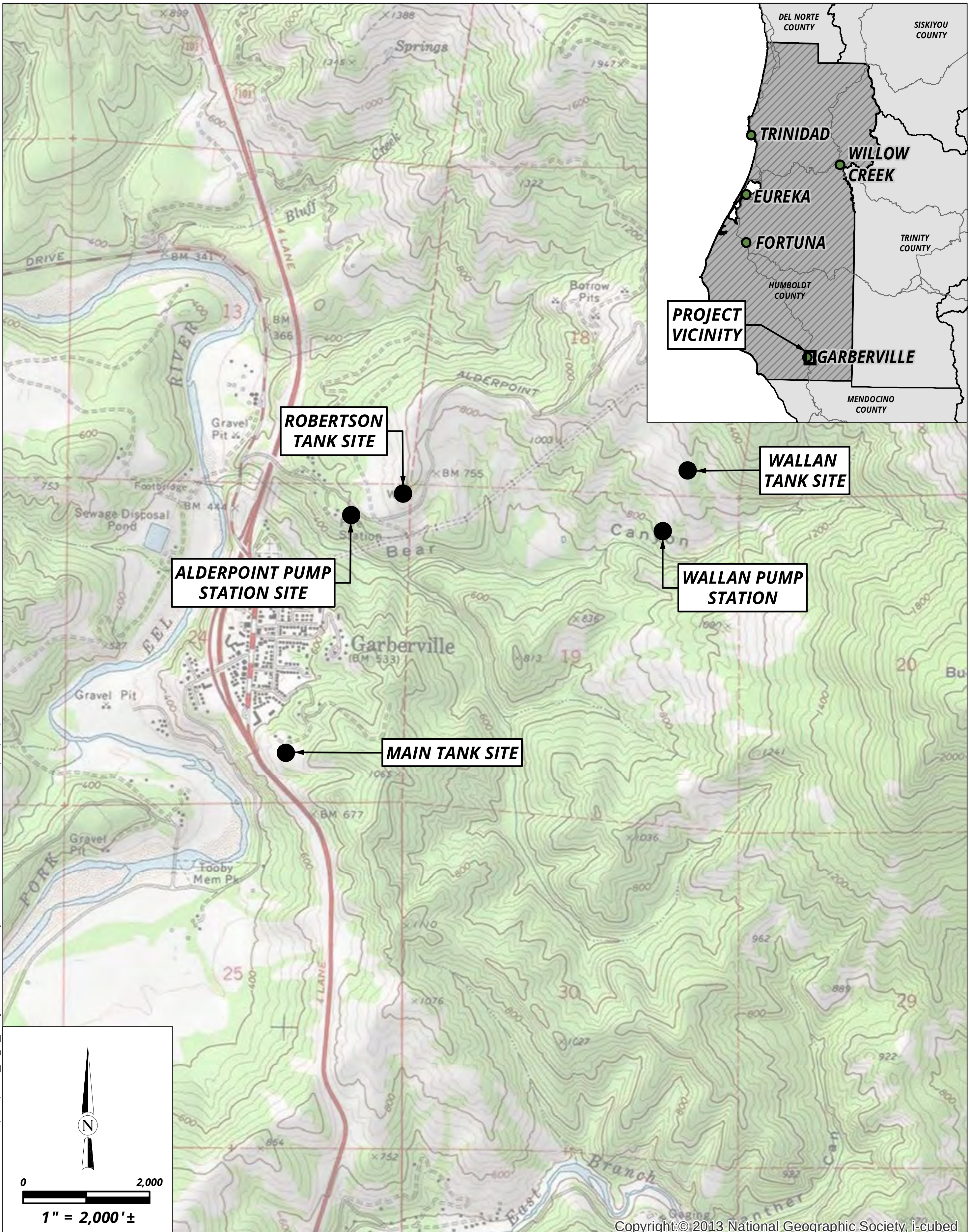
Our geotechnical investigation was completed to inform the project design team and to provide the necessary background information for Humboldt County and California Environmental Quality Act (CEQA) permitting. Our assessment focuses on characterization of the geologic conditions (geohazards) at the proposed water tanks, water lines, and pump station sites, and development of geotechnical recommendations relative to the construction of new water storage tanks and associated infrastructure. This report is intended to address all the items on the "Soils Engineering/Engineering Geology Report Checklist" provided on the Humboldt County Planning and Building Department's website (Humboldt County, 2008).

Project Location and Description

Garberville Sanitary District serves the unincorporated town of Garberville and surrounding area with sewer, wastewater, and water services. GSD owns, operates, maintains, and manages the public drinking water system, which includes two drinking water sources, water treatment facilities, three finished water storage tanks currently in service, multiple pumping stations, and a distribution piping network. GSD's service area covers 581 acres, and the water system serves approximately 1,200 people in the Garberville community. The area is topographically rugged, and the water system crosses a variety of terrain. The project elements requiring geotechnical consideration occur at five locations in the Garberville vicinity referred to as "Main Tank," "Wallan Tank," "Alderpoint Pump Station," "Robertson Tank," and "Wallan Pump Station" (Figure 1).

Specifically, elements of the project requiring geotechnical consideration include the following:

- Construction of a partially buried, approximately 550,000-gallon water storage tank (Main Tank), pump station (Maple Lane Pump Station), generator, and waterlines
- Installation of a buried waterline at the Main Tank site
- Replacement of the Wallan Tank with an aboveground steel tank
- Construction of a new pump station (Alderpoint Pump Station) across Alderpoint Road from the existing Arthur Road Pump Station. The new Alderpoint Pump Station will replace the existing Arthur Road Pump Station
- Visual evaluation of the stability of the Wallan Pump Station
- Demolition of the Robertson Tank



Garberville Sanitary District
Garberville Water System Improvements
Garberville, California

Project Location Map

August 2023 - 022067

Figure 1

Scope of Work

The scope of SHN's services included reviewing available geologic and subsurface information; field reconnaissance; overseeing the advancement of geotechnical borings; performing laboratory testing on selected soil samples; and providing engineering geologic and geotechnical recommendations to aid in project planning, design, and construction.

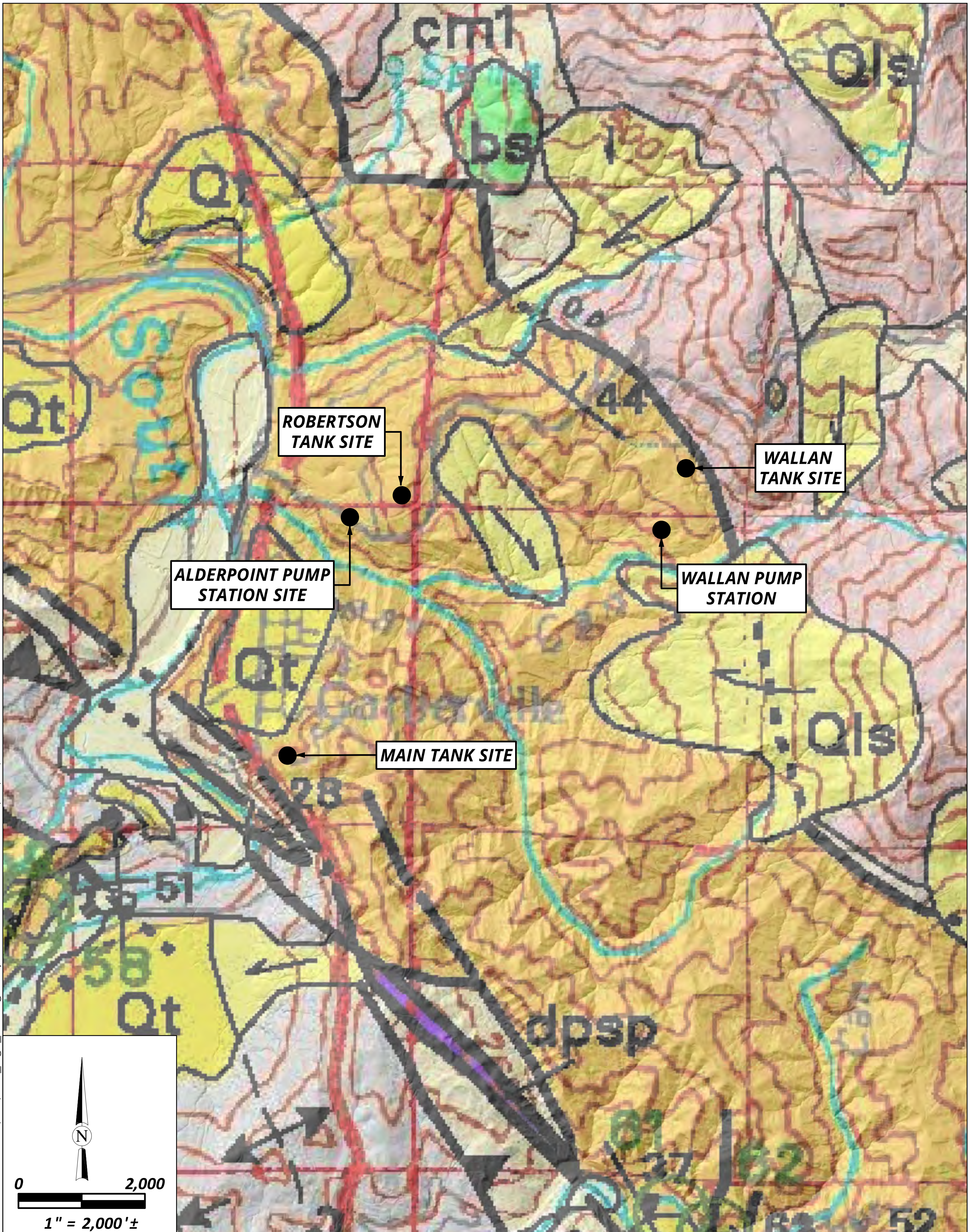
Specifically, the following information, recommendations, and design criteria are presented in this report:

- description of site terrain and local geology;
- engineering geologic assessment of sites where there are stability concerns;
- description of soil and groundwater conditions at the proposed water tank and pump station sites, based on our field exploration, laboratory testing, and review of existing geotechnical information;
- logs of the exploratory geotechnical borings at the proposed water tank and pump station sites (Appendix 1) and the results of laboratory tests conducted for this investigation (Appendix 2);
- assessment of potential earthquake-related geologic/geotechnical hazards (for example, strong earthquake ground shaking, surface fault rupture, liquefaction, settlement);
- seismic design parameters in accordance with the applicable portions of the 2022 California Building Code (CBC) and American Society of Civil Engineers (ASCE) 7-16 Standard, including site soil classification, seismic design category, and spectral response accelerations;
- recommendations for site improvements, including site and subgrade preparation, fill material, placement, and compaction requirements;
- recommendations for foundation type and design criteria, including bearing capacity, along with provisions to mitigate the effects of adverse soil conditions, as appropriate;
- expected total and differential settlement; and
- recommendations for observation of foundation installation, materials testing and inspection, and other construction considerations.

Geologic Setting

The project area is located within the western portion of the Coast Range Geomorphic Province in southern Humboldt County, California. The site is located in a complex and dynamic geologic environment, approximately 40 miles southeast of Cape Mendocino. Cape Mendocino marks the intersection of three crustal plates known as the Mendocino Triple Junction (MTJ) and is characterized by active tectonic deformation and high rates of seismicity.

Geologic mapping of the area (Figure 2) shows that the water system is underlain by bedrock associated with the Quaternary-Tertiary-aged undifferentiated Wildcat Group (Spittler, 1984). Bedrock associated with the Broken Formation of the Cretaceous-Jurassic aged Franciscan Complex is located directly east of the Wallan Tank in the northeastern part of the project area. The two geologic units are separated along a northwest-trending contact, which is interpreted as a relict bedrock fault. Portions of the project vicinity are underlain by alluvial terrace deposits associated with the ancestral Eel River (shown by Qt on the Figure 3). These alluvial terraces typically consist of an abrasion platform cut across Wildcat



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Garberville Sanitary District Garberville Water System Improvements Garberville, California	**Geologic Map** **McLaughlin, 2000** August 2023 - 022067	**Figure** **2**

QUATERNARY AND TERTIARY OVERLAP DEPOSITS

Qt

Undifferentiated nonmarine terrace deposits (Holocene and Pleistocene)- Dissected and (or) uplifted gravel, sand, silt, and clay, deposited in fluvial settings. In western Eureka quadrangle (Sheet 1) unit includes minor shallow marine intertongues and warped and tilted beds of late Pleistocene Hookton and Rohnerville Formations of Ogle (1953), in addition to younger late Pleistocene and Holocene fluvial terrace units a few feet to a few tens of feet higher than normal modern high-water level

Qls

Landslide deposits (Holocene and Pleistocene)-Unsorted clay- to boulder-size debris and broken rock masses that have moved downslope in debris flows, earth flows, and as more-or-less intact rotational or translational blocks, largely from Pleistocene to present. Only large landslides, occupying tens to hundreds of acres, are depicted here.

QTw

Marine and nonmarine overlap deposits (late Pleistocene to middle Miocene)-Thin-bedded to massive, weakly lithified siltstone, fine- to medium-grained sandstone, silty to diatomaceous mudstone and locally soft, scaly mudstone. Locally includes lenses of pebble to boulder conglomerate, carbonate concretions, abundant molluscan fossils, woody debris, and horizons of rhyolitic volcanic ash that are greater than 1 meter thick in some areas. Includes the Wildcat Group (Ogle, 1953), the Bear River beds (Haller, 1980), and related outlier Neogene deposits isolated along faults near Briceland, Garberville, Benbow, Piercy, Bridgeville and northeast of Weott. Unit also includes minor fault-bounded blocks along or near the coast between Bear River and the Mattole River that are incorporated into melange of the Coastal terrane; the Neogene Falor Formation northeast of Eureka (Manning and Ogle, 1950); and equivalent deposits in the offshore area deposited in shelf, slope, and slope basin settings. A few poorly exposed erosional remnants of shallow marine to brackish water strata mapped along high ridge crests overlying the Franciscan Complex in the 1:24,000 Zenia quadrangle are tentatively assigned to this unit. South of this map, unit correlates with valley-fill, perched gravel and shallow marine to nonmarine coal-bearing sedimentary rocks of Quaternary and Tertiary age in the Round Valley area of Covelo 1:100,000 quadrangle (Jayko and others, 1989)

COAST RANGES PROVINCE

FRANCISCAN COMPLEX

Coastal Belt

Yager terrane (Eocene to Paleocene?)

Sedimnetary rocks of the Yager terrane (Eocene to Paleocene?)-Argillite and arkosic sandstone rhythmically interbedded, thin to medium bedded; massive to thickly bedded arkosic sandstone with minor interbeds of argillite; and minor lenses of polymict boulder to pebble conglomerate. Southwest of Garberville, unit highly folded, but locally may be penetratively sheared or broken. Argillite and interbedded fine-grained sandstone is commonly calcareous and may have abundant plant debris in places. Sandstone characteristically contains prominent detrital muscovite. Based on fossil dinoflagellates and on spores and pollen from carbonate concretions in argillite, age of terrane is late to middle Eocene. Locally the lower beds of the terrane may be as old as Paleocene (McLaughlin and others, 1994). The Yager terrane is divided into 3 subunits based principally on topographic expression in aerial photographs and outcrop data:

y1

Sheared and highly folded mudstone-Includes minor rhythmically interbedded sandstone, locally with lenses of conglomerate. Exhibits irregular topography lacking a well-incised system of sidehill drainages

Central belt

Melange of the Central belt (early Tertiary to Late Cretaceous)- Consists of a matrix of clayey, penetratively sheared argillite and fine-grained sandstone, locally with intercalated green tuff and hard elliptical carbonate concretions armored with scaly black argillite. Includes blocks up to several kilometers across, of diverse lithologies and ages. Age range of the Central belt is based on the paleontologic and isotopic age range of rocks in the melange and on inferred range in age of penetrative shearing, boudinage, and related deformation that occurred during melange formation. Components of the Central belt melange include:

Unnamed Metasandstone and meta-argillite (Late Cretaceous to Late Jurassic)-Arkosic lithic metasandstone and meta-argillite, reconstituted to textural zones 1 to 2A (Jayko and others, 1989) and metamorphosed to pumpellyite and lawsonite grade, with less than 1/2 percent K-feldspar (fig. 5). Unit locally includes cobble- to pebble-size polymict conglomerate or monomict chert-pebble conglomerate. Locally, the metasandstone and meta-argillite depositionally overlie radiolarian chert in composite melange blocks. In some places in blocks metasandstone is imbricated or structurally interleaved with chert and basalt. Fossils from unnamed metasandstone and meta-argillite range in age from Late Cretaceous to Late Jurassic. Carbonate concretions and local, thin, silty, hemipelagic chert beds and lenses in melange matrix contain radiolaria and dinoflagellates ranging in age from Late Jurassic to Early Cretaceous (Tithonian to Aptian or Albian). Some metasandstone and conglomerate in composite blocks depositionally overlie chert with a Late Cretaceous (Cenomanian) radiolarian assemblage. The unnamed metasandstone and meta-argillite is divided into subunits of melange and broken formation based principally on topographic expression in aerial photographs and outcrop data:

cm1

Melange-Predominantly penetratively sheared, locally tuffaceous, scaly meta-argillite and less abundant blocks of metasandstone. Exhibits rounded, poorly incised, lumpy and irregular topography

cb1

Broken formation-Consists of bedded to massive, locally folded, rarely conglomeratic metasandstone and meta-argillite, with only minor amounts of highly sheared rocks. Exhibits sharp-crested topography with regular, well-incised sidehill drainages

bs

Basaltic rocks (Cretaceous and Jurassic)-Includes pillowed and non-pillowed flows, flow breccias, submarine tuff, and diabase. Basalt commonly is alkalic (high TiO_2 content). Basalt may be overlain by radiolarian chert or foraminiferal limestone. Age of locally overlying limestone indicates some basalt to be as young as Middle Cretaceous (Aptian); where overlain by radiolarian chert, basalt is no younger than Early Jurassic. Basalt is metamorphosed to low greenschist grade

dsp

Serpentinite melange (Jurassic?)-Partially to completely serpentinized ultramafic rocks (harzburgite, dunite), locally highly sheared, and includes minor masses of cumulate gabbro, diabase or basalt. Present beneath diabase and (or) basalt of the Benbow and Bear Buttes areas (Sheet 3). Contact with overlying ophiolitic rocks probably is an attenuation fault. Unit is partially equivalent to some serpentinite interspersed with and assigned to Central belt of Franciscan Complex sediments, with terrace sediments consisting of alluvial deposits (sand, silt, and gravel; Spittler, 1984). The Main Tank site is underlain by one such terrace more than 400 feet above the modern Eel River.

Bedrock of the undifferentiated Wildcat group is described as mudstone, shale, sandstone, siltstone, and minor amounts of conglomerate with highly variable degrees of consolidation. Specific descriptions of the geologic units within the project vicinity are presented on Figure 2a.

Geologic mapping by McLaughlin and others (2000) and Spittler (1984) show areas of landsliding (Qls on Figure 2; McLaughlin and others, 2000) in the project vicinity; these occur as translational/rotational and earthflow slides. Spittler (1984) shows areas of "disrupted ground," throughout the project vicinity, which is described as:

> "Irregular ground surface caused by complex landsliding processes resulting in features that are indistinguishable or too small to delineate individually at the map scale; also may include areas affected by downslope creep, expansive soils, and/or gully erosion; boundaries are usually indistinguishable."

The water distribution system is within the Garberville-Briceland fault zone. According to Kelsey and Carver (1988), the Garberville-Briceland fault zone is a discontinuous series of north-northwest trending lineaments that extend south-southeast from Bull Creek, through Garberville, to just north of Laytonville. There is no documented recent (Holocene) activity on the Garberville fault, nor are there mapped faults crossing the water system. The Garberville-Briceland fault zone is not zoned as active by the State of California (CGS, 2018).

Geologic Hazards

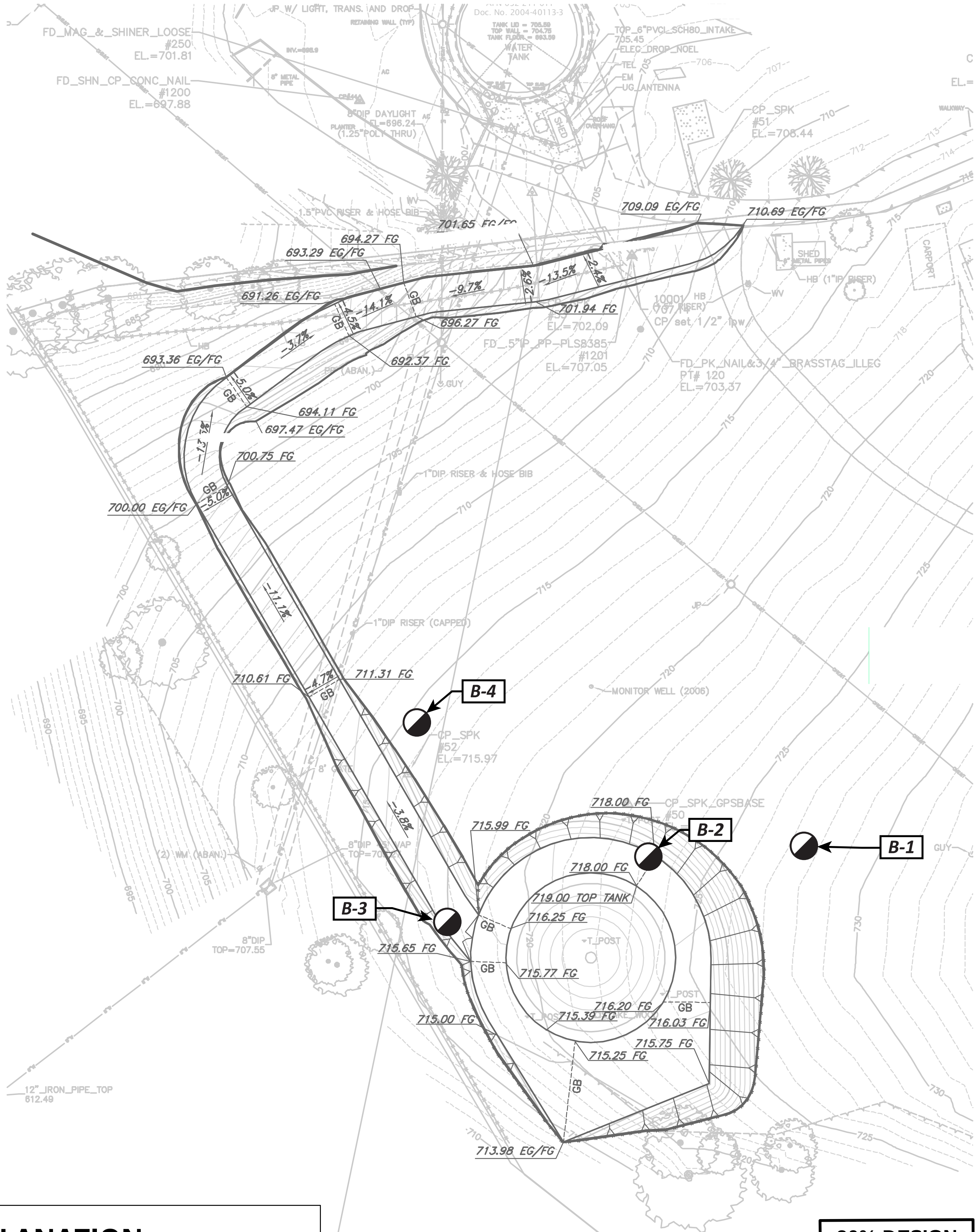
Potential geologic/geotechnical hazards common to the local area include seismic ground shaking, surface fault rupture, and slope instability. The assessment of these potential hazards is presented below.

Seismic Ground Shaking

The project site is in a seismically active area with the potential for strong earthquakes and strong ground shaking. As stated above, the water distribution system is within the Garberville-Briceland fault zone. This fault zone is not considered active by the State of California (CGS, 2018). The site is located approximately 15 miles northeast of the northern most extent of the San Andreas fault. Strong seismic ground shaking should be expected during the lifespan of the proposed water storage tanks and associated infrastructure.

Surface Fault Rupture

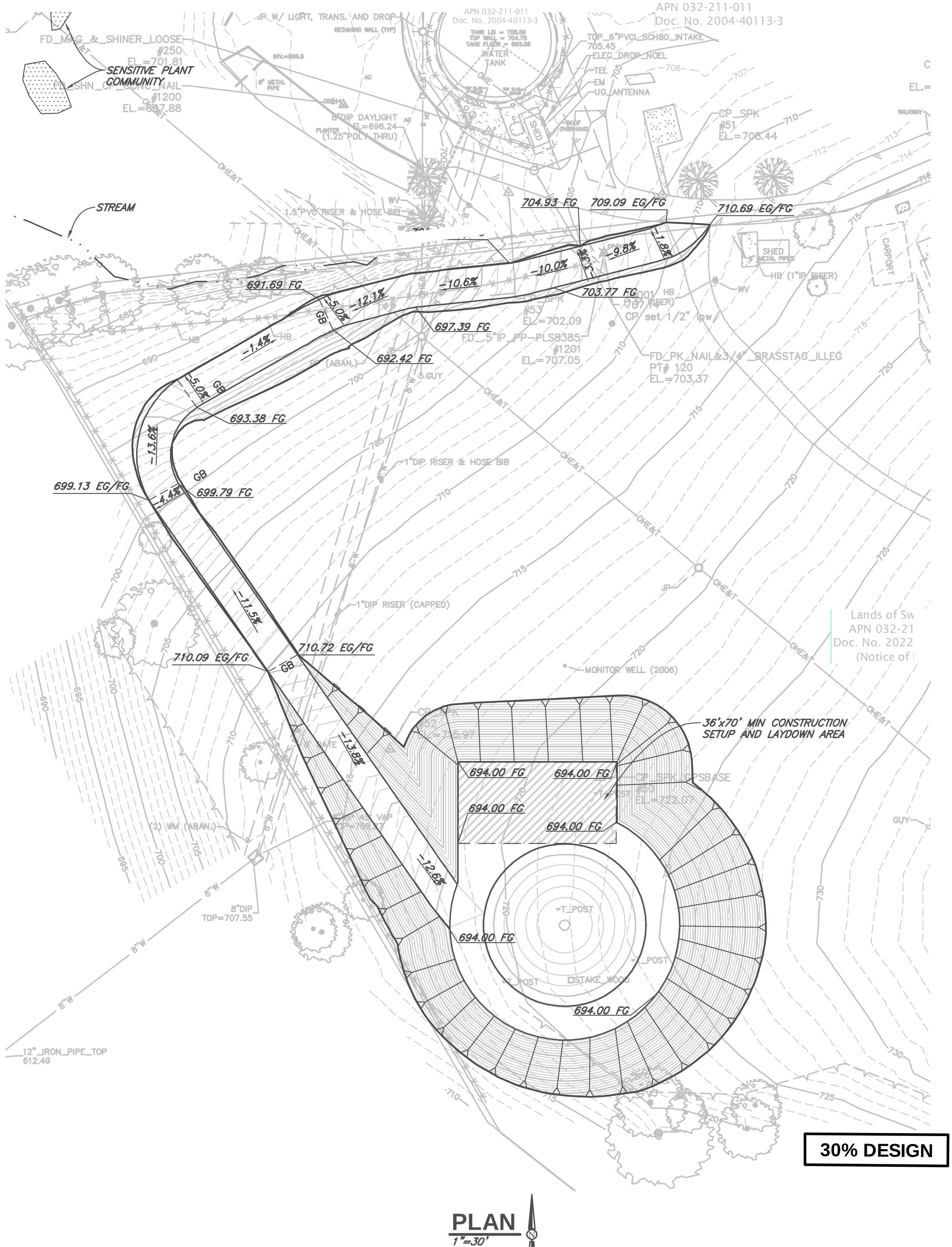
The project site is not located in a state-mandated Earthquake Fault Zone (CGS, 2018). The nearest known active fault is the San Andreas fault, which is approximately 15 miles southwest of the project site. The San Andreas fault is a northwest-trending strike-slip fault. Surface ruptures associated with 1906 San Francisco earthquake were identified at Shelter Cove (Lawson, 1908). During our field visit, we did not observe any geomorphic evidence suggesting recent surface rupture in the project area.



EXPLANATION

GEOTECHNICAL BORING

Garberville Sanitary District Garberville Water System Improvements Garberville California	**Geotechnical Boring Locations** **Main Tank Site** August 2023 - 022067	**Figure** **3**



Garberville Sanitary District
Garberville Water System Improvements
Garberville, California

Main Tank Excavation Plan

August 2023 - 022067

Figure 3A

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Liquefaction

Liquefaction is a soil behavior phenomenon in which soil located below the groundwater table temporarily loses strength during and immediately after a seismic event because of strong earthquake ground motions. Recently deposited and geologically young Holocene age sediments consisting of relatively loose, saturated, non-cemented granular soil are most susceptible.

As all the sites discussed in this report are located in upland settings on bedrock or older alluvial soils, there is a negligible potential for soil liquefaction to impact improvements related to this project.

Slope Stability

Numerous landslides and areas of unstable ground are shown on available geologic maps (Spittler, 1983; McLaughlin, 2000). The type and concentration of landsliding is relative to the underlying bedrock; more slides are mapped in areas underlain by Broken Formation bedrock, which does not underlie the improvement sites. Relatively few are mapped (or observed) in areas underlain by Wildcat Group sediments. We did not observe any features related to recent landsliding (tension cracks, seeps, springs, rills, or gullies) at the proposed new infrastructure sites, although unstable ground is mapped in the site vicinity. Localized landsliding adjacent to the Robertson Tank demolition site is noted below. Failures occur along roads within the service area (Alderpoint Road, for example), but these appear related to construction methods (unsupported sidecast fills on steep slopes) rather than underlying slope instability in the native soils.

Due to the site location in a seismically active area and the potential for strong seismic ground shaking to occur at the site, there is an ongoing potential for localized co-seismic landsliding to occur along steep slopes throughout the project area.

Field Investigation and Laboratory Testing

An engineering geologist from SHN conducted site reconnaissance on May 25, 2023, prior to each subsurface investigation to observe existing site conditions. A project geologist visited the Main Tank and Alderpoint Pump Station sites on June 8, 2023, and the Wallan Tank site on June 21, 2023, to oversee the advancement of geotechnical exploratory borings. The borings at each site were drilled and sampled by Taber Drilling of Sacramento, California, using a CME 75 track-mounted drill rig with solid-flight augers. Upon completion of drilling, the borings were backfilled with cement grout and soil cuttings. Field sampling and observation, and laboratory testing methods are described in the following paragraphs. Subsurface investigations specific to each site are described in the following sections.

Representative samples were obtained during drilling using standard penetration test (SPT; 1.375-inch internal diameter [I.D.]) and modified California (2.5-inch I.D.) split-spoon samplers. The samplers were driven 18 inches into the soil/rock using a 140-pound auto-hammer with a 30-inch drop. The number of "blows," or hammer drops, required for each 6-inch increment of sampler drive was recorded. The blow counts for each 6-inch drive and the sampler types are noted on the boring logs (Appendix 1).

Visual classifications of the earth materials encountered were made in the field in general accordance with the Manual-Visual Classification Method (ASTM-International [ASTM] D 2488). The final boring logs, presented in Appendix 1, were prepared based on the field logging, examination of samples in the laboratory, and the results of laboratory testing.

Selected soil samples were tested in SHN's certified soils-testing laboratory in Eureka, California, to determine selected index properties of the subsurface materials. Samples were tested for in-place moisture content and dry density, percent fines (passing the number 200 sieve), and plasticity index (Atterberg Limits). Results of the laboratory tests are provided at the corresponding sample locations on the geotechnical boring logs (Appendix 1) and included as Appendix 2.

Main Tank Site

At the main tank site, four exploratory geotechnical borings (B-1-LH through B-4-LH) were advanced to depths of 51.5 feet (B-1-LH and B-2-LH), 26.5 feet (B-3-LH), and 16.5 feet (B-4-LH). Three of the four borings were drilled in the planned vicinity of the partially buried water tank (based on an early conceptual tank footprint) and one boring (B-4-LH) was drilled in the planned location of the buried water line. Boring locations are shown on Figure 3.

Wallan Tank Replacement

At the Wallan Tank site, two exploratory geotechnical borings (B-1-W and B-2-W) were advanced to depths of 16.5 feet below grade. The geotechnical borings were placed on opposite sides of the existing Wallan Tank. Boring locations are shown on the site plan on Figure 4.

Wallan Pump Station

Field reconnaissance of the site and vicinity was completed on May 25, 2023. Geotechnical conditions at the site were determined based on surface geological exposures. Subsurface investigation at the site is not relevant to this effort and was not part of the work scope.

Alderpoint Pump Station

At the new Alderpoint Pump Station site, one exploratory geotechnical boring (B-1-APS) was advanced to 16.5 feet below grade. The geotechnical boring was placed near the edge of the proposed building footprint, at the bottom of the vegetated slope. The boring location is shown on the site plan on Figure 5.

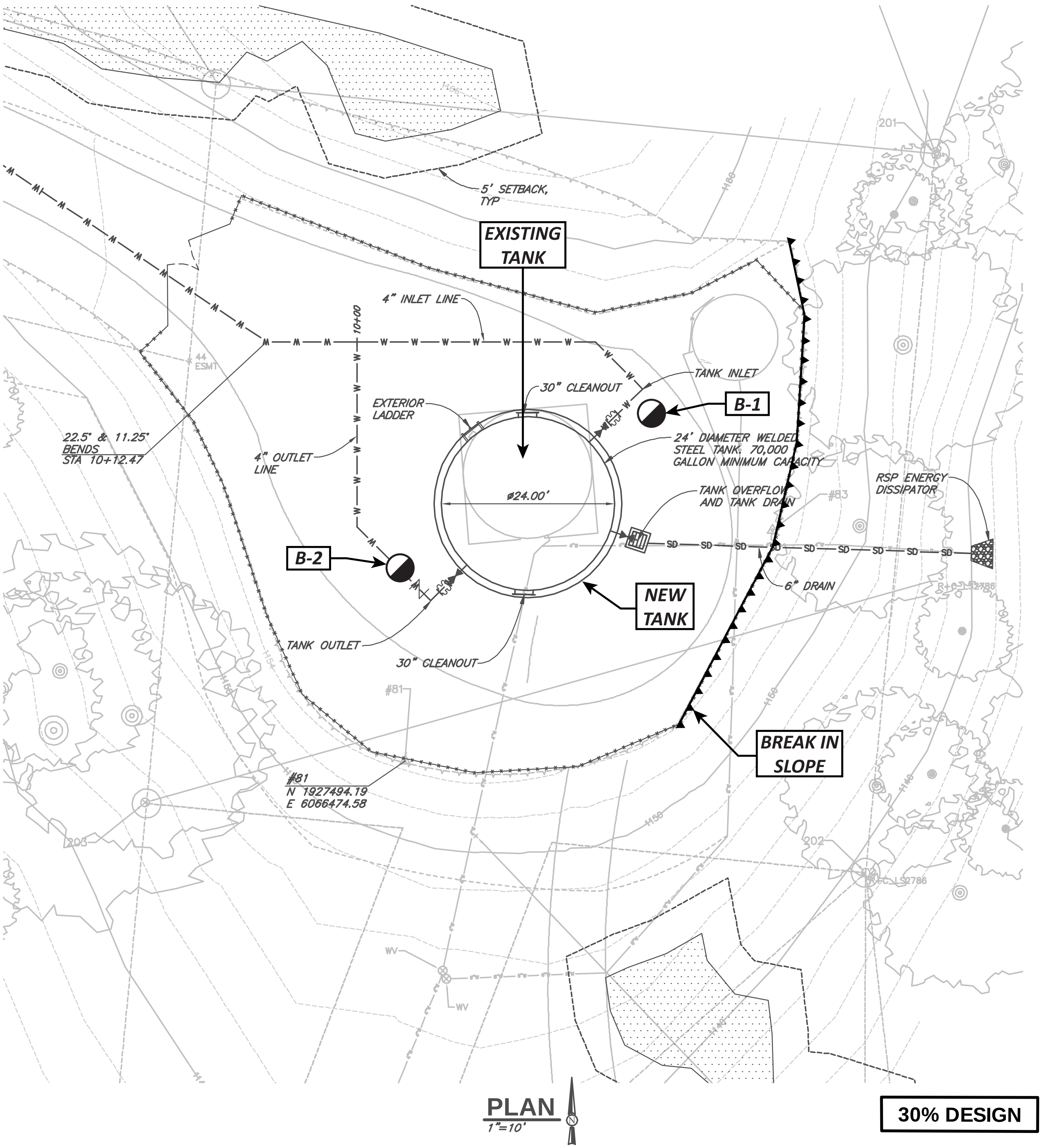
Robertson Tank Demolition

SHN Geosciences staff visited the site on May 25, 2023, and completed field reconnaissance of the site and vicinity. Geotechnical conditions at the site were determined based on surface geological exposures. Subsurface investigation at the site is not relevant to this effort and was not part of the work scope.

Project Location and Description

Main Tank Site

The Main Tank site is located southeast of the town of Garberville on Humboldt County Assessor's parcel number (APN) 032-211-021. Improvements at this site requiring geotechnical consideration consist of the construction of a partially buried, approximately 550,000-gallon concrete water tank, a pump station and generator adjacent to the tank (Maple Lane Pump Station), and a buried water line. We understand that the pump station and generator will be constructed on the engineered fill-pad surrounding the



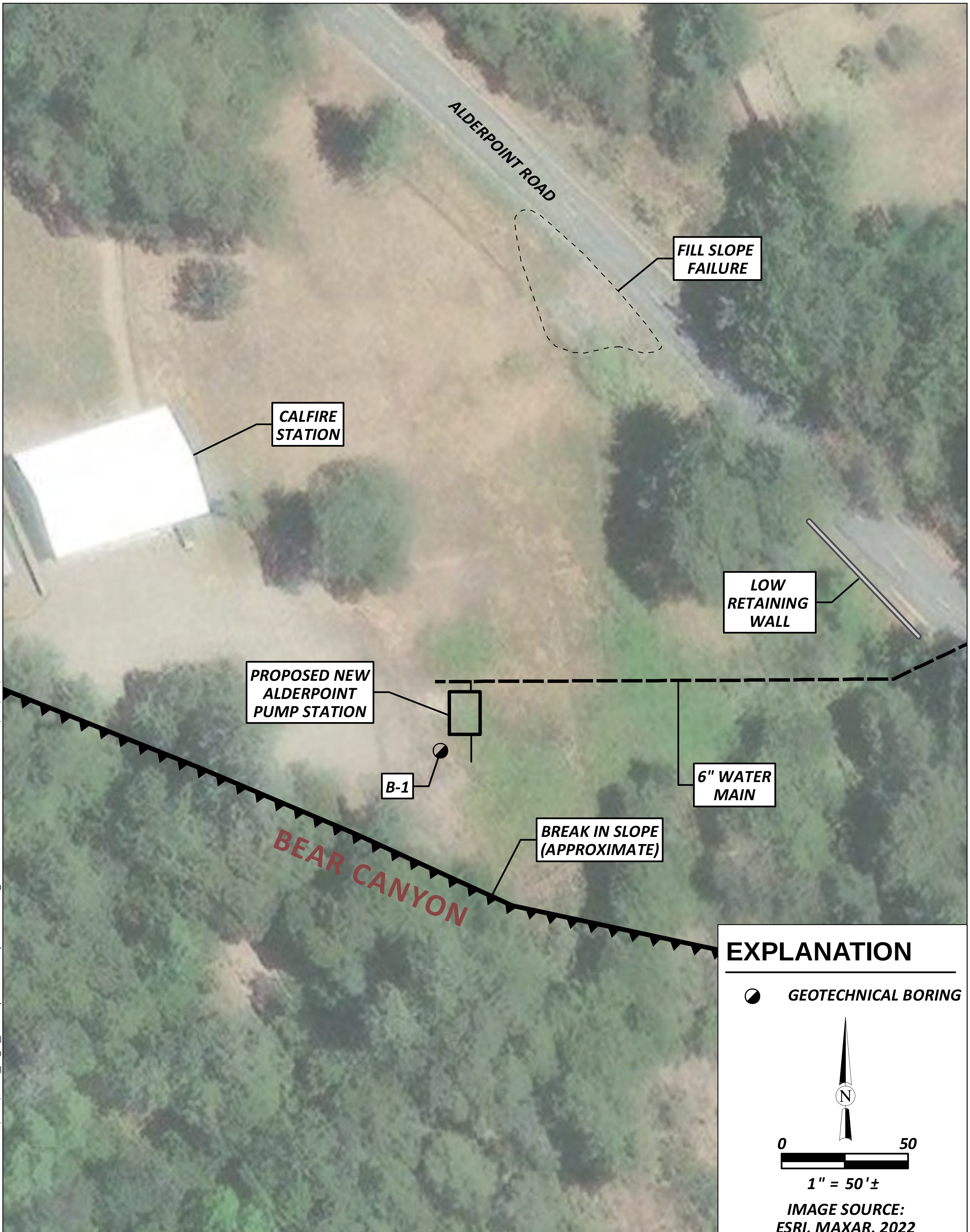
EXPLANATION

GEOTECHNICAL BORING

Garberville Sanitary District
Garberville Water System Improvements
Garberville, California

Geotechnical Boring Locations
Wallan Tank Site
August 2023 - 022067

Figure 4



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Garberville Sanitary District
Garberville Water System Improvements
Garberville, California

Geotechnical Boring Location
Proposed New Alderpoint Pump Station
August 2023 - 022067

Figure 5

partially buried tank, and the water main will exit at the bottom elevation of the buried tank and be routed across the field and to the northwest in a trench up to 25 feet deep. The finished site configuration shown on the 30% site plan with geotechnical boring locations is shown on Figure 3.

The site is situated on a gently to moderately sloping ancient alluvial terrace surface, with elevations ranging from approximately 690 to 740 feet (North American vertical datum, 1988 [NAVD88]). The partially buried tank is to be sited in the southern portion of the property, in the southwestern corner of the terrace remnant. Gentle slopes at the tank site slope toward the west and southwest. The site is vegetated by grasses; there are no trees or large plants within the area of proposed improvements (although the site is adjacent to the tree line). The southwestern border of the property at the tank site is marked by a wire fence near the top of a southwest-facing cut slope leading to Highway 101 that is densely vegetated with dense shrubs and mature hardwood trees. The slope south of the site is a forested valley wall slope associated with a natural, west-flowing stream.

The partially buried, 72-foot-wide concrete tank is to be installed at a depth 30+ feet below existing grade. The excavation will be up to 180 feet wide and will include a construction setup and laydown area, surrounded by temporary cut slopes up to about 34 feet high (see Figure 3a). After the tank is installed, the excavation will be backfilled with 20+ feet of engineered fill (varying thickness around the perimeter of the tank). The large temporary construction excavation will be filled upward from the bottom (it will become shallower) and inward from the edges (it will become narrower). The finished excavation would be up to about 150 feet wide. The engineered fill pad surrounding the tank will support the new Maple Lane Pump Station, a generator, and a service road. The Maple Lane Pump Station has a planned footprint of 20 by 15 feet and is to be sited on the southeast side of the buried tank.

Permanent slopes surrounding the tank and service road area will be fill slopes up to about 10 feet tall. The tank overflow pipe will consist of a subdrain to exit the tank on the south side that will daylight on the slope southeast of the tank area. Finished configuration will result in a service road within the partially filled water-main excavation and extending around the partially buried tank (refer to the site plan on Figure 3). The proposed service road surfaces are planned with gentle cross-slopes; the access road to the northwest will drain to the northwest, and the service road surrounding the tank will drain to the slope south of the tank area.

The water main exiting near the tank base elevation on the west will be routed to the northwest. The line will be installed up to about 25 feet below grade near the tank, and shallow to the northwest. The method of installation for the deepest section of the line is not yet determined, but could either consist of a temporary shored trench, or horizontally drilled directional bore.

Wallan Tank Replacement

The Wallan Tank site is located approximately 1 mile northeast from the town of Garberville, on Humboldt County APN 223-191-006. The site is situated at an approximate elevation of 1,100 feet (NAVD88) on an east facing ridge that divides two tributary drainages to the Eel River. From Wallan Road, access to the site is by a narrow, unpaved, steep road in a rural residential area. Improvements at this site requiring geotechnical consideration consist of the demolition of the existing aboveground redwood water tank and construction of a new aboveground steel water tank in approximately the same position. The new tank is expected to have an approximate capacity of 70,000 gallons. A site plan with geotechnical boring locations is shown on Figure 4.

The existing tank to be replaced occupies a relatively level, unpaved pad cut into a moderate gradient southwest-facing slope. The site is bordered by an 8- to 10-foot-high cut bank along its northern side and a steep, forested slope to the east; the site is accessed from the west by an unpaved driveway.

Wallan Pump Station

The Wallan Pump Station is located along the outboard edge of Wallan Road along the approach to the Wallan Tank. A small facility with a limited footprint, the pump station structure is inset into the road shoulder on a narrow sidehill bench several feet below the grade of Wallan Road. We understand that improvements at this site are limited to minor infrastructure upgrades that will not require structural modifications to the existing building or its foundation.

Alderpoint Pump Station

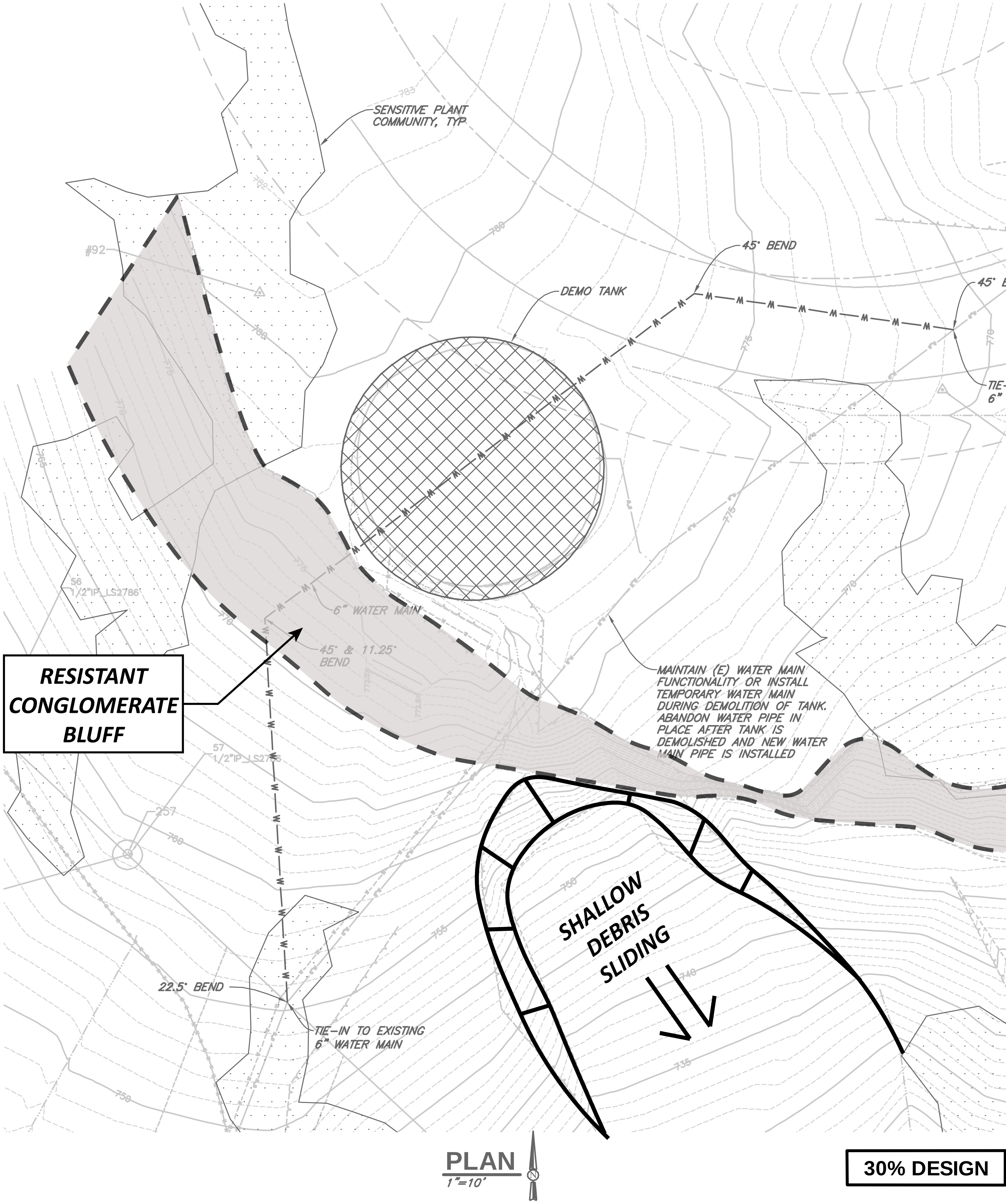
The proposed new Alderpoint Pump Station site is located approximately 1 mile from the town center of Garberville, adjacent to Alderpoint Road (to the north), at the east end of Humboldt County APN 223-183-003, which is currently partially occupied by a California Department of Forestry and Fire Protection (CAL FIRE) station. The new pump station is planned to replace the existing Arthur Road Pump Station that is located just to the east, across Alderpoint Road. The new pump station has a planned footprint of approximately 20 feet by 13 feet and includes two 8-inch water lines that will connect to the existing system. A site plan with geotechnical boring locations is shown on Figure 5.

The site is situated at an approximate elevation of 550 feet (Google Earth), on a gentle to moderate slope. The proposed new pump station footprint is sited approximately 50 feet from the edge of Bear Canyon, which occurs as a very steep slope, densely forested by oak and other hardwood trees. The footprint is generally located in an area that has been partially graded to provide a gravel-surfaced turnaround for CAL FIRE Station vehicles.

Isolated unstable areas occur along the outboard edge of Alderpoint Road adjacent to the site. These shallow failures appear related to unstable fill soils rather than instability of the underlying native soils.

Robertson Tank Demolition

The Robertson Tank is an existing structure located north of Garberville, along the north side of Alderpoint Road, about 650 feet northeast of the Arthur Road Pump Station (Figure 6). The existing tank lies mostly below grade along the water main connecting the existing Arthur and Wallan pump stations. The Robertson Tank is underlain by Wildcat Group bedrock. The site is located at the crest of a steep, linear, south-facing slope that exposes cemented cobble conglomerate. Slopes (bluffs) of this type are relatively common features associated with resistant areas within the Wildcat Group, several of which occur near the site. The bluff is by nature, a resistant landform with low erosion and mass wasting potential. Debris shed from the bluff through minor, periodic rockfall accumulates at the base of the slope and forms a shallow debris slide slope south of the bluff.



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Garberville Sanitary District
Garberville Water System Improvements
Garberville, California

Robertson Tank Demolition

August 2023 - 022067

Figure 6

Subsurface Conditions

Main Tank Site

The native materials encountered in the geotechnical borings are consistent with the known geologic conditions identified in previous geologic mapping (Spittler, 1984; Pleistocene age river terrace sediments and/or the older Wildcat Group). The soil profile generally consists of stiff to hard/medium dense to dense interbedded sandy lean clay and clayey sand observed to the maximum depth explored (51.5 feet in B-1-LH and B-2-LH). An interval of hard sandy lean clay with gravel was observed at a depth of 10 feet in B-2-LH, and an interval of loose clayey sand was observed at a depth of 5 feet in B-3-LH. Specific descriptions of the soils encountered are shown on the boring logs included in Appendix 1.

Groundwater was encountered below 35 feet in boring B-1-LH. In borings B-2-LH through B-4-LH, soils were dry in the upper 15 feet of the borings, with increased moisture observed near 15 feet below ground surface (BGS). Groundwater levels at the time of our investigation (early June) in this region would be approaching a seasonal low, which we would expect to occur in late summer or early fall.

Mottling was observed as shallow as 5 to 10 feet deep in the geotechnical borings. Groundwater levels fluctuate seasonally and can be expected to be higher during periods of intense precipitation.
Based on the groundwater conditions in the borings and the proposed excavation depths for the project, groundwater should be anticipated 10 feet below current grade during periods of prolonged heavy precipitation. Groundwater seepage may occur during grading and construction for the proposed new partially buried water tanks and related infrastructure, especially if sandy materials are encountered.

Wallan Tank Replacement

Published geologic mapping indicates the site is located immediately west of a significant geologic contact between the older Franciscan Bedrock (to the east) and younger Wildcat Group bedrock (to the west; McLaughlin et. al., 2000). Based on our subsurface observations, we interpret the materials in the geotechnical borings to be Wildcat Group, which is consistent with the mapping. To a depth of 5 feet beneath the site, we encountered deeply weathered conglomerate, which occurs as stiff silty clay with varying amounts of fine sand and gravels. Below 5 feet, we encountered highly weathered bedrock, consisting of highly fractured, moderately soft fine sandstone to siltstone. Specific descriptions of the materials encountered are shown on the boring logs provided in Appendix 1.

Groundwater was not encountered in the geotechnical borings during the time of our investigation (in late June). Groundwater levels can be expected to be higher during periods of intense precipitation, however, based on localized topography and the underlying soil conditions. We do not anticipate that groundwater will be a significant factor during construction of shallow improvements.

Alderpoint Pump Station

The native materials encountered in our geotechnical boring are consistent with previous geologic mapping (Wildcat Group). We encountered medium dense silty and clayey fine sand to a depth of 10 feet, with a layer of loose silty sand approximately 3 feet BGS. Between 10 and 15 feet below grade, we encountered stiff sandy lean clay overlying clayey sand with gravel to the maximum depth explored (16.5 feet).

Groundwater was not encountered in the geotechnical boring at the time of our investigation (in early June). Groundwater levels can be expected to be higher during periods of intense precipitation, however, based on localized topography and the underlying soil conditions. We don't anticipate groundwater will be a factor during construction of shallow improvements, provided construction occurs during the dry season.

Seismic Design Parameters

Based on the subsurface conditions encountered at our exploration locations, laboratory test results, and our interpretation of soil conditions within 100 feet of the ground surface, we classify the sites as a Site Class D consisting of a "Stiff Soil" in accordance with Chapter 20 of ASCE 7-16. On this basis, the mapped and design spectral response accelerations were determined using the ASCE 7 Hazard Tool (ASCE, 2022) in conjunction with the site class and site coordinates 40.094667 ^{0}, -123.793008^{0} (Main Tank Site); 40.107731^{0}, -123.770436^{0} (Wallan Tank Replacement); and 40.105182^{0}, -123.789514^{0} (Alderpoint Pump Station) at the location of the proposed tanks and structures. Calculated values for ASCE 7-16 are presented in the tables below.

Table 1a. ASCE/SEI 7-16 Spectral Acceleration Parameters (Main Tank Site)

Parameter	0.2 Second	1 Second
Maximum Considered Earthquake Spectral Acceleration (MCE_R)	S_S = 1.773	S_1 = 0.845
Site Class	D	
Site amplification factor	F_a = 1	F_v = N/A
Site-modified spectral acceleration	S_{MS} = 1.773	S_{M1} = N/A
Numeric seismic design value	S_{DS} = 1.182	S_{D1} = N/A
MCE_G peak ground acceleration (PGA)	0.74	
Site amplification factor at PGA (F_{PGA})	1.1	
Site modified peak ground acceleration (PGA_M)	0.814	

Table 1b. ASCE/SEI 7-16 Spectral Acceleration Parameters (Wallan Tank Replacement)

Parameter	0.2 Second	1 Second
Maximum Considered Earthquake Spectral Acceleration (MCE_R)	S_S = 1.662	S_1 = 0.849
Site Class	D	
Site amplification factor	F_a = 1	F_v = N/A
Site-modified spectral acceleration	S_{MS} = 1.662	S_{M1} = N/A
Numeric seismic design value	S_{DS} = 1.108	S_{D1} = N/A
MCE_G peak ground acceleration (PGA)	0.749	
Site amplification factor at PGA (F_{PGA})	1.1	
Site modified peak ground acceleration (PGA_M)	0.824	

Table 1c. ASCE/SEI 7-16 Spectral Acceleration Parameters (Alderpoint Pump Station)

Parameter	0.2 Second	1 Second
Maximum Considered Earthquake Spectral Acceleration (MCE_R)	S_S = 1.709	S_1 = 0.854
Site Class	D	
Site amplification factor	F_a = 1	F_v = N/A
Site-modified spectral acceleration	S_{MS} = 1.709	S_{M1} = N/A
Numeric seismic design value	S_{DS} = 1.139	S_{D1} = N/A
MCE_G peak ground acceleration (PGA)	0.749	
Site amplification factor at PGA (F_{PGA})	1.1	
Site modified peak ground acceleration (PGA_M)	0.824	

Geotechnical Conclusions and Recommendations

Based on the results of our field and laboratory investigation, it is our opinion that construction of the water storage tanks and pump stations at the project sites are feasible from a geohazard and geotechnical standpoint, if our recommendations are implemented during design and construction. The major geotechnical considerations for development of the proposed water storage tanks and pump stations are the potential for strong seismic ground shaking and the proximity to steep, locally unstable slopes.

The sites are likely to experience strong seismic ground shaking resulting from earthquakes on active faults in the region during the design life of the proposed water tanks and associated infrastructure. The intensity of ground shaking from earthquakes will depend on several factors, including the distance from the site to the earthquake focus, the magnitude and duration of the earthquake, and the response of the underlying soil. At a minimum, it will be necessary to design and construct the proposed structures in accordance with the earthquake-resistant provisions of the governing code.

All geotechnical-related work should be performed in accordance with the recommendations of the Geotechnical Engineer-of-Record during construction. Where the recommendations of this report and the cited sections of Title 24 are in conflict, the Owner or Engineer should request clarification from the Geotechnical Engineer-of-Record. The recommendations in this report should not be waived without the consent of the Geotechnical Engineer-of-Record for the project. The following subsections present recommendations for the geotechnical-related work.

Below we provide site-specific discussion and recommendations for each site, followed by general geotechnical recommendations for site preparation, grading, wet weather considerations, engineered fills, soil corrosivity, foundations, and so on.

Main Tank Site

The development of the main tank will require excavation of a large semi-circular area up to about 30 feet deep to accommodate the buried tank (Figures 3 and 3a). The water main will extend northwestward from the tank, exiting the tank bottom (25 feet below grade) and following an increasingly shallow alignment. The current plans show the water main constructed via trench, although we understand the final construction method will be determined by the contractor.

The tank excavation, as currently planned, will be 180 feet wide during construction; it is acceptable to use 1:1 temporary construction slopes for this excavation, although the contractor is responsible for the stability of the final excavation configuration based on the materials and conditions encountered at the time of excavation. Following construction, the excavation will be partially backfilled, supporting the pump station and a service road around the tank. The finished configuration will entail a smaller circular area, as the construction excavation is filled both upward and inward; finished fill slopes surrounding the finished configuration should be associated with a 2:1 slope gradient.

Following construction of the concrete tank, backfill placed in the excavation to achieve final grade around the tank and along the water main should be placed following the recommendations provided below in the “Site Preparation and Grading” and “Select Engineered Fill” sections. Fill placed against the native soils along the outside of the temporary tank excavation or along the service road should be benched, as prescribed below. Finished grade will result in the partial burial of the water tank; the water main excavation will be partially filled and retained as a service road.

The finished configuration will result in surface drainage around the partially buried water tank flowing toward an outlet at the south edge that flows onto the native hillside, while drainage along the service road (which will be a through cut) will be directed toward the northwest. Drainage from the area surrounding the tank will discharge at a single point above the adjacent hillslope and appropriate energy dissipation will be required.

Based on the results of our subsurface investigation, we believe that the proposed main tank can be supported on either a conventional spread footing foundation or mat slab foundation. Recommendations for both foundation types are provided in subsequent sections below in “Foundations.” Note that the walls of the buried tank are considered below-grade retaining walls and attention is directed to the “Below Grade Tank Walls and Retaining Walls” section.

Wallan Tank Replacement

The removal and replacement of the Wallan tank is to occur on a pre-existing graded pad with no change to the existing condition. The proposed replacement tank will occupy much of the footprint of the existing tank. Following demolition of the existing tank, treat the disturbed areas per the recommendations in the “Site Preparation and Grading” and “Select Engineered Fill” discussions below. We assume the replacement tank will be developed on a ring-wall foundation, discussed below in “Foundations.”

Based on the results of our site investigation, we did not encounter fill materials in either boring location. We interpret that the enlarged tank footprint will remain in the native cut surface. However, if any fill or other unsuitable materials are encountered during excavation/preparation of the site, they should be removed and our recommendations for general site and subgrade preparation should be adhered to.

Drainage onto the steep slope east of the site should include a significant energy dissipation feature.

Wallan Pump Station

We understand that the proposed improvements at the Wallan Pump Station are minor and will not require soil disturbance surrounding the facility. In its current condition, the pump station is within about 5 feet of a steep slope. The current site condition exhibits no evidence of slope instability or erosion that is affecting the small buffer strip adjacent to the pump station. Relative to the proposed infrastructure upgrades at the site, the primary objective is to maintain existing conditions without disturbing soils or vegetation surrounding the facility. Concentrated runoff should not be directed towards the steep slope outboard of the pump station.

Alderpoint Pump Station

Construction of the Alderpoint pump station is to occur on an undeveloped site with a slab-on-grade foundation. The geotechnical recommendations below regarding "Site Preparation and Grading," "Select Engineered Fill," and "Concrete Structural Slabs-on-Grade" are relevant.

The Alderpoint pump station and associated infrastructure will be located adjacent to the crest of the high valley wall slope of Bear Canyon. Although no recent landsliding is apparent on the slope adjacent to the site, it is prudent to maintain a reasonable setback to accommodate future potential geologic change. We recommend a minimum setback from the crest of the slope of 30 feet.

Robertson Tank Site

As the Robertson Tank demolition occurs atop a resistant bluff comprised of cemented cobble conglomerate, there is a low potential for impacts related to demolition and backfilling of the tank. The ground should be resistant to disturbance and have low erosion potential. Care should be taken during the demolition of the tank to avoid disturbance of the soil between the tank footprint and the top of the bluff directly south.

Once the aboveground portions of the tank have been removed, the side walls should be demolished to a minimum of 4 feet below grade and the debris removed from the excavation. Break a minimum of four 4-foot diameter holes through the tank floor to provide drainage through the tank; the debris from creating these holes may be retained in the holes. Any remaining voids in the holes in the tank floor should be filled with drain rock and the remainder of the excavation should be backfilled following the recommendations for "Select Engineered Fill" below. Treat the ground surface, as appropriate, to receive vegetation or other erosion control, as appropriate to meet project goals.

We understand the realigned water line will be routed through the footprint of the demolished Robertson Tank, in order to increase its setback from the bluff crest, which we agree is appropriate. We expect the depth of burial to be shallower than 4 feet (the depth of the remaining tank walls). The existing water line should be abandoned in place.

General Geotechnical Recommendations

Site Preparation and Grading

Following demolition of any remaining concrete and asphalt (where required), areas to be graded should be cleared of any rubbish or debris, organics, organic topsoil, loose soil and/or soft bedrock, and any other unsuitable material. Site preparation operations should extend at least 5 feet beyond the limits of improvements. We anticipate that stripping to a depth of less than 1 foot will be required to remove the organics and topsoil, where encountered. Deeper stripping may be locally required to remove concentrations of vegetation, such as brush and tree roots. Where the removal of large trees is required, it will be necessary to remove all major root systems, then fill the excavations with properly placed engineered fill compacted to at least 90 percent relative compaction[1].

Any vegetation and organic topsoil with more than 2 percent organic material by dry weight should be removed. The Geotechnical Engineer should observe and approve the prepared site prior to any excavation, subgrade preparation, and placement of fill or improvements.

All areas to receive engineered fill should be stripped of loose and/or soft surface soils and vegetation and benched into firm soil/rock. If zones of weak or saturated soils are encountered during site preparation, they should be removed by further excavation to expose firm natural soil/rock and replaced with engineered fill.

Fill placed in swales and drainage channels should be benched into firm soils along the bottom and sides to provide a firm level surface on which to place new engineered fill. In areas where proposed structures will be supported on spread footings and are located partially on cut and partially on fill, the cut portion should be over-excavated and replaced as engineered fill in order to provide at least 12 inches of engineered fill below all of the footings to provide uniform support for the entire foundation.

Non-engineered fill that may be present within the limits of grading should be identified and excavated to expose firm natural ground. In areas intended to support new water storage tanks and engineered fill, and for a distance of at least 5 feet beyond the limits of these improvements, topsoil and loose native soils should be excavated to expose firm, undisturbed native soil. The resulting surface created by removal of non-engineered fill and loose soils should be checked by the Geotechnical Engineer or qualified representative to determine whether further excavation is required to remove any loose or unsuitable materials. The approved surface may then be brought to pad grade with placement of engineered fill.

Permanent cut and fill slopes up to 5 feet in height should be placed no steeper than 1.5H:1V and 2H:1V (horizontal to vertical), respectively. Higher or steeper slopes should be reviewed by the Geotechnical Engineer or qualified representative for stability during construction. We understand that temporary construction slopes related to the development of the Main Tank may be as steep as 1H:1V. It is the contractor's responsibility to monitor the stability of temporary cut slopes. Additional recommendations are provided below in the "Excavations and Temporary Shoring" section.

Site grading during and shortly after the wet season is typically difficult and/or uneconomical. Onsite soils will have moisture contents well above optimum and will require greater than normal spreading, mixing, and/or aeration to achieve a near-optimum moisture content suitable for required compaction.

Engineered fill placed on slopes that are steeper than 5H:1V should be keyed and benched into supportive material to provide a firm, stable surface on which to support the fill. Prior to fill placement on slopes steeper than 5H:1V, a construction keyway should be excavated at the toe of the fill.

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1 Relative compaction refers to the in-place dry density of a soil expressed as a percentage of the maximum dry density of the same soil, as determined by the ASTM D1557 compaction test procedure. Optimum moisture is the water content (percentage by dry weight) corresponding to the maximum dry density.

The keyway should be a minimum of 8 feet wide or of a width equal to half the height of the fill slope, whichever is greater. The keyway should be excavated a minimum of 2 feet into bedrock or competent support material, as measured on the downhill side of the excavation. The depth to supportive material should be determined by this office in the field during construction. The base of the keyway excavation should have a nominal slope of approximately 2 percent dipping toward the back (uphill side) of the key. Subsequent construction benches should be excavated to remove any non-supportive surficial soil and should also have a nominal slope of approximately 2 percent dipping in the uphill direction. Our representative should observe the completed keyway and bench excavations to confirm they are founded in materials with sufficient supporting capacity.

Engineered fill placed as backfill, following construction of the main tank, should be benched into the surrounding temporary cutslope. Backfill material should be brought up uniformly around the below-grade structure (that is, backfill should be at the same elevation all around the structure as the backfill is placed and compacted). The elevation difference of the backfill surface around the structure should not be greater than 2 feet.

The area at both the top and toe of fill slopes should be graded or provided with a lined berm or V-ditch, to provide good surface drainage away from the slope to protect against erosion. All slope surfaces should be planted with fast-growing, erosion-resistant vegetation immediately after grading. Should erosion channels develop, they should be repaired immediately to prevent progressive undermining or sloughing of the slope surface.

Wet Weather Subgrade Protection

The near-surface soils consist of loose, non-cohesive, fine-grained granular materials and/or soft fine-grained silts. We expect that both light and heavy construction equipment will have difficulty operating on the near-surface soils if grading commences during and/or immediately following the wet season. Contractors should expect high soil moisture conditions in the near-surface soils throughout the wet season and into the late spring months following a typical winter wet season. The wet season in coastal northern California generally begins in the month of November and continues through May. Heavy rains are also not uncommon during the months of October and June. Beginning construction activities and earthwork immediately prior to the onset of the wet season is not advised and will likely lead to delays if measures are not taken to stabilize and protect the exposed subgrade.

Soils that have been disturbed during site preparation activities, or unsuitable areas identified during proof-rolling or probing, should be removed to firm ground and replaced with stabilization material and compacted structural fill.

Protection of the subgrade is the responsibility of the contractor. Track-mounted excavating equipment may be required during and following wet weather. The contractor will be responsible for constructing an all-weather access road and staging area. The thickness of the haul road to access the currently undeveloped portions of the site for construction and staging areas will depend on the amount and type of construction traffic. The materials used for haul roads or site access drives should be stabilization material consisting of pit or quarry run rock that is well-graded, angular, crushed rock consisting of 4- to 6-inch minus material with less than 5 percent passing the U.S. Standard No. 4 Sieve. The material should be free of organic matter and other deleterious material. A minimum 6- to 12-inch-thick mat of stabilization material should be used for light staging areas. The stabilization material for haul roads and areas with repeated heavy construction traffic will likely need to be increased to between 12 to 18 inches. The actual thickness of haul roads and staging areas should be based on the contractor's approach to site work and the amount and type of construction traffic and is the contractor's responsibility. The stabilization material should be placed in one lift over the prepared, undisturbed subgrade and compacted using a smooth-drum, non-vibratory roller. Additionally, a geotextile fabric should be placed as a barrier between the subgrade and stabilization material. The geotextile should meet specifications for soil separation and stabilization, such as Mirafi 600X or equivalent.

Select Engineered Fill

Fill placed in areas to support proposed water tank and pump station foundations should meet the requirements for select engineered fill. Select engineered fill should have less than 2 percent by dry weight of vegetation and deleterious material and should meet the gradation requirements presented in Table 2.

Table 2. Fill Gradation Criteria

Sieve Designation	Percent Passing by Dry Weight
3-inch (50 mm)[i]	100
2½-inch (37.5 mm)	85 minimum
¾-inch (19 mm)	70 minimum
No. 4 (4.75 mm)	60 minimum
No. 200 (75 µm) [ii]	5 minimum, 30 maximum

[i] mm: millimeters
[ii] µm: micrometers

We anticipate that onsite soils will be suitable for reuse as select engineered fill following removal of debris, organics, and any other unsuitable material. Fine-grained soil with a liquid limit greater than 40 and a plasticity index greater than 15 should not be used as select engineered fill. If clayey soils do not meet the plasticity requirements, mixing of the clayey soils with sandier soils may be required. Crushing and/or removal of rock particles greater than 3 inches in size will be required. Select engineered fill should have a low corrosion potential, which is defined as a minimum resistivity of 2,000 ohms per centimeter and maximum sulfate and chloride concentrations of 250 parts per million (ppm).

In addition, we do not recommend using river-run material as select engineered fill; crushed, angular material should have at least 50 percent of the material (as determined by the material's dry weight) containing a minimum of two fractured faces.

Engineered fill should be placed in loose lifts not exceeding 8 inches in thickness and compacted to a minimum of 90 percent relative compaction. The Geotechnical Engineer should approve all fill prior to placement.

A qualified field technician should be present to observe fill placement and perform field density tests in accordance with ASTM D 6938 at random locations throughout each lift to verify that the specified compaction is being achieved.

Samples of proposed import fill materials should be submitted to SHN for approval at least three business days prior to use at the site.

Excavations and Temporary Shoring

The contractor shall be responsible for the stability of all temporary excavations. Excavations should be made in accordance with and should comply with applicable Occupational Safety and Health Administration (OSHA) specifications and regulations. The contractor should periodically monitor all open cuts for evidence of incipient stability failures.

Excavations deeper than 4 feet below ground surface (or shallower if excavations appear unsafe) should be laid back to a safe slope inclination or supported by an appropriate shoring system. It should be noted the contractor is solely responsible for site safety and safe working conditions during construction. A temporary or permanent shoring system should be installed in a configuration that will allow vertical side slopes for deep excavations where laying back the excavation is impractical. Recommendations are presented below for the design and construction of a soldier pile wall for permanent shoring.

Excavated soils should be placed a minimum of 10 feet away from the edge of the below-grade excavation to reduce surcharge loads on the temporary cut slopes. If shoring systems are used, the effects of the soil stockpile on the shoring system should be taken into account during design if the soils are placed in the area between the top of the excavation and a 1H:1V (horizontal to vertical) projection from the toe of the excavation, to reduce the potential of a shoring failure.

Similarly, heavy equipment should be operated in a safe manner and should be kept an adequate distance from unshored or unbraced excavation sidewalls to prevent a cut slope stability hazard. If shoring is used, surcharge loads from heavy equipment should be considered in the design calculations to prevent a surcharge failure during construction. For an unshored excavation, a heavy equipment exclusionary zone should be established based on soil type, depth of excavation, presence of groundwater, and configuration of the open cut. As a general guideline, heavy equipment should be excluded from a zone located between the top of the excavation and a 1H:1V projection from the bottom toe of the adjacent excavation sidewall.

Utility Trench Backfill

New utility trenches excavated parallel to spread footing foundations should be set back from the footings such that the trench bottoms lie outside a projected hypothetical 1.5H:1V (horizontal to vertical) line extending downward from the footing bottom.

Unless concrete bedding is required around utilities, bedding should consist of sand having a sand equivalent (SE) of at least 30. The bedding should extend from 6 inches below to 1 foot above the conduit or pipe. Sand bedding should not be jetted or ponded into place and should be mechanically compacted to a minimum of 90 percent relative compaction.

In areas to support improvements (such as new slabs) and adjacent to structure foundations, backfill placed above the bedding in utility trenches should be properly placed and adequately compacted to minimize settlement and provide a stable subgrade.

In areas to support improvements such as slabs and pavements and adjacent to structure foundations, backfill placed above the bedding in utility trenches should be properly placed and adequately compacted to minimize settlement and provide a stable subgrade. If possible, the trench backfill should be compacted following rough grading but prior to final grading and compaction. Onsite inorganic soils meeting the requirements for engineered fill may be used as trench backfill. Backfill consisting of onsite soils should be placed in layers not exceeding 8 inches in loose thickness, water-conditioned, and compacted to at least 90 percent relative compaction as described for engineered fill. Trench backfill need only be compacted to 85 percent relative compaction in landscape areas or in areas more than 5 feet beyond the limits of buildings, pavements, concrete slabs-on-grade, sidewalks, or other flatwork. The upper 6 inches of trench backfill under pavements should be surface compacted to at least 95 percent relative compaction.

Where utility trenches cross underneath buildings, we recommend that a plug be placed within the trench backfill to minimize the normally granular backfill from acting as a conduit for water to enter beneath the building. The plug should be constructed using a sand cement slurry (minimum 28-day compressive strength of 500 pounds per square inch [psi]) or relatively impermeable native soil for pipe bedding or backfill. We recommend the plug extend a distance of at least 3 feet in each direction from the point where the utility enters the building perimeter.

Soil Corrosion Potential

As part of the investigation at the proposed partially buried concrete tank (Main Tank), laboratory corrosivity tests were performed on composited soil samples collected from boring B-1-LH at 25.5 to 26 feet BGS and from B-2-LH at 15 to 16.5 feet BGS. Tests were performed to evaluate the reduction and oxidation potential (redox), pH, resistivity, and concentrations of chloride and sulfate, of/in the soil that would be in contact with the Main Tank foundation elements and underground piping. The results of the soil corrosivity tests are included in Appendix 3 and are summarized in Table 3.

Table 3. Soil Corrosivity Test Results

Parameter	Composite Sample
Redox (mV)[a,b]	340
pH	6.72
Resistivity (100% Saturation) (ohms-cm)[c]	8,800
Chloride (mg/kg)[d]	<15
Sulfate (mg/kg)	<15

[a] Redox: oxidation-reduction potential
[b] mV: millivolts
[c] ohms-cm: ohms-centimeter
[d] mg/kg: milligrams per kilogram

- The redox potential is indicative of potentially slightly corrosive soils resulting from anaerobic soil conditions.
- The pH of the soil reportedly does not present corrosion problems for buried iron, steel, mortar-coated steel, and reinforced concrete structures.
- Based upon the resistivity measurement, the soil samples are classified as mildly corrosive. All buried iron, steel, cast iron, ductile iron, galvanized steel, and dielectric-coated steel or iron should be properly protected against corrosion. All buried metallic pressure piping such as ductile iron firewater pipelines should be protected against corrosion.
- The chloride ion and sulfate ion concentrations are below the detection limits of 15 milligrams per kilogram (mg/kg).

Foundations

1) Conventional Foundations for Water Storage Tanks and Pump Stations

Based on our geotechnical investigation, we conclude that the proposed new water storage tanks and pump station structures may be supported by concrete spread footings embedded at least 18 inches below the lowest adjacent grade in firm native soil/rock or properly compacted engineered fill. SHN defines lowest adjacent grade as the tank bottom, or exterior soil subgrades, whichever results in a deeper footing. Footing thicknesses and widths should meet the minimum requirements in the 2022 CBC. Footings founded in firm native soil/rock or properly compacted engineered fill should be designed using a maximum allowable bearing capacity of 2,500 pounds per square foot (psf) for dead plus normal duration live loads. These allowable bearing capacities may be increased by one-third for total load conditions, including wind and seismic.

Base friction resistance may be calculated using an ultimate friction coefficient of 0.35 for firm native soil/rock. If crushed aggregate base (AB) is used as engineered fill beneath the new water tanks, an ultimate base friction coefficient of 0.45 may be used. Passive resistance may be calculated using an equivalent fluid unit weight of 300 pounds per cubic foot (pcf). The recommended passive resistance is reduced by a factor of about 1.5 from the ultimate value to reduce deflections to tolerable amounts. The recommended passive pressure and friction coefficients may be combined, without reduction, for calculating total lateral resistance. The passive resistance contributed by soils within 1 foot of the ground surface should be neglected unless these soils are protected and confined by a slab-on-grade or pavement. Gaps between the footing and adjacent ground should be completely backfilled using engineered fill, concrete, or lean cement slurry with a 28-day unconfined compressive strength of at least 100 pounds per square inch (psi).

The ring-wall footing should be reinforced to resist hoop stresses within the wall. Hoop stresses may be calculated by assuming outward lateral pressure acting on the foundation equal to 0.45 times the vertical pressure imposed on the subgrade within the ring-wall. Lateral soil pressures acting on buried vaults that may be constructed adjacent to the tank should likewise be calculated using a lateral soil pressure equal to 0.45 times the vertical pressure acting on the adjacent subgrade.

Steel tank bottoms are typically domed upward from the perimeter to the center to allow differential settlement to occur without overstressing the tank bottom in tension. The settlement is anticipated to be greater at the center than at the perimeter. The imposed loads under full hydrostatic pressure may result in some settlement of the underlying engineered fill. Post-construction vertical settlement due to full hydrostatic loading is estimated at ½ inch near the center of the tank.

We recommend that a representative of the Geotechnical Engineer observe all foundation excavations prior to the placing of reinforcing steel. This inspection should be conducted to ensure that the bottoms and sides of all foundation excavations are level or suitably benched and are free of loose or soft soil, ponded water, and debris. If any loose pockets are encountered in the bottom of the foundation excavations, they should be over-excavated, and the base of the excavation should be backfilled with lean concrete. It is important that foundation excavations be clean and free of loose or soft soils, water, or other debris at the time concrete is placed.

2) Mat Foundation

The partially buried main water tank (Main Tank) may be supported on a mat foundation bearing on a minimum of 12 inches of Class 2 aggregate base compacted to a minimum of 95 percent relative compaction. The foundation should be designed using an allowable bearing capacity of 4,000 psf for dead plus long-term live loads. This allowable bearing capacity may be increased by one-third for total load conditions, including wind and seismic.

The mat foundation should be reinforced with grids of reinforcing steel bars. The project structural engineer should determine actual mat reinforcing based on anticipated loading and the design criteria presented in this report.

Subgrade Modulus for Mat Design. For mat design, we recommend using the following equation to estimate the subgrade modulus:

K_s (pounds per cubic inch [pci]) = $k_1/B*(1+B/L/2)/1.5$

Where: k_1= coefficient of subgrade reaction for 1-foot square plate = 250 pci
B = width of foundation bearing area beneath column or bearing wall in feet
L = length of foundation bearing area beneath column or bearing wall in feet

The equation is empirical and the k_1 units are pci (provided the values of B and L used are in feet). The values of B and L and the corresponding K_s value should be consistent with the calculated deflected shape of the mat.

Lateral Resistance. Base friction resistance for the mat foundation may be calculated using a friction coefficient of 0.35 (allowable value for concrete on engineered fill material). Passive resistance may be calculated using an equivalent fluid unit weight of 300 pcf. This value is reduced by a factor of 1.5 from the ultimate value to limit movement required to mobilize ultimate passive pressure. Both the ultimate base friction and allowable passive pressure may be combined in calculating total lateral resistance. The passive resistance contributed by fill material within 1 foot of the ground surface should be neglected unless these materials are protected and confined by a slab-on-grade or pavement.

The mat foundation should be cast neat against the engineered fill to develop the design passive resistance. Alternatively, any gap between the foundation and the adjacent ground should be completely backfilled using lean concrete.

Settlement. Maximum total settlement for the mat foundation due to static loads at the center of a tank designed and constructed in accordance with the recommendations presented in this report is estimated to be less than approximately 1 inch. Total differential settlement is estimated to be less than ½ inch. The majority of the total settlement is expected to occur during construction and initial filling of the tank.

Concrete Structural Slabs-on-Grade

Concrete slabs-on-grade should be supported by engineered fill prepared in accordance with our recommendations for earthwork.

A minimum of 4 inches of Class 2 Aggregate Base rock, compacted to a minimum of 90 percent relative compaction, should be provided beneath exterior flatwork and other slabs-on-grade.

It is important that the subgrade be moist and free of desiccation cracks at the time the slab is cast. Recommendations for slab reinforcement, strength, thickness, control and construction joints, etc., should be provided by others. Although cracks in concrete slabs are common and should be expected, the following measures may help to reduce cracking of slabs.

- Slabs should be cast using concrete with a maximum slump of 4 inches or less.
- Add a water reducing agent or plasticizer to the concrete to increase slump while maintaining a low water-cement ratio to reduce concrete shrinkage. (Concrete having a high water-cement ratio is a major cause of concrete cracking.)
- Control joints should be provided at appropriate intervals to control the location of shrinkage cracks.

Below Grade Tank Walls and Retaining Walls

Below-grade walls (including the tank walls) should be designed to resist both static lateral earth pressures and lateral pressures caused by earthquakes. We recommend permanent below-grade walls be designed for the more critical of either at-rest pressures or assumed static active pressure and a dynamic component.

For restrained backfill conditions, use an at-rest equivalent fluid pressure of 60 pcf above the design groundwater level and 95 pcf below, plus a traffic surcharge where the wall is adjacent to access roads or streets. Active earth pressures may be used for design of unrestrained retaining walls, if required, where the top of the wall is free to translate or rotate. To develop active earth pressures, the walls should be capable of deflecting by at least 0.004H (where H is the height of the wall). Cantilever walls retaining level engineered fill may be designed for active lateral earth pressures of 36 pcf, plus a traffic surcharge where the wall is adjacent to access roads.

If retaining wall (or tank wall) backfill will be subject to passenger vehicle or light truck traffic loading within a distance of H/2 from the top of the wall (where H is the wall height), the wall should be designed to resist an additional uniform lateral pressure of 72 psf applied to the back of yielding walls (active conditions), or 124 psf applied to the back of non-yielding walls (at-rest conditions). Surcharge loads imposed by greater loads or unusual loads within a distance of H of the back of the wall should be considered on a case-by-case basis.

In addition to the active or at-rest lateral soil pressures, retaining walls should be designed to resist additional dynamic earth pressures during earthquake loading. The additional dynamic pressure increment may be calculated using an additional equivalent fluid pressure of 16 pcf. The dynamic pressure increment should be applied to the wall as a triangular distribution so the resultant force acts at a distance of 0.33H above the base of the wall (where H is the height of the wall). Under the combined effects of static and dynamic loading, a safety factor of 1.1 against sliding or overturning is acceptable.

The dynamic component of the lateral earth pressure was calculated using the Mononabe-Okabe equation and, therefore, assumes that sufficient deformation of the wall will occur during seismic loading to develop active soil conditions. As previously discussed, we recommend permanent below-grade walls be designed for the more critical of either at-rest pressures or assumed static active pressure and a dynamic component.

Closure

The analyses, conclusions, and recommendations contained in this report are based on site conditions that we observed at the time of our investigation, data from our subsurface explorations, our current understanding of proposed project elements, and on our experience with similar projects in similar geotechnical environments. We have assumed that the information obtained from our subsurface explorations is representative of subsurface conditions throughout the areas of proposed improvements addressed in this report.

We have assumed, in preparing our recommendations, that SHN will be retained to review those portions of the plans and specifications that pertain to soil-related work. The purpose of this review is to confirm that our earthwork recommendations have been properly interpreted and implemented during design. If we are not provided with this opportunity for review of the plans and specifications, our recommendations could be misinterpreted.

We recommend a representative of our firm confirm site conditions during the construction phase. If subsurface conditions differ significantly from those disclosed by our investigation, we should be given the opportunity to re-evaluate the applicability of our conclusions and recommendations. Some alteration of recommendations may be appropriate. If the scope of the proposed construction changes from that described in this report, our recommendations should also be reviewed.

Limitations

The recommendations provided in this report are based on the assumption that we will be retained to provide the construction monitoring described above in order to evaluate compliance with our recommendations. If we are not retained for these services, SHN cannot assume any responsibility for any potential claims that may arise during or after construction as a result of misuse or misinterpretation of this report by others. Furthermore, if another geotechnical consultant is retained for follow-up service to this report, SHN will at that time cease to be the Geotechnical Engineer-of-Record.

The opinions presented in this report are valid as of the present date for the property evaluated. Changes in the condition of a property can occur with the passage of time, whether due to natural processes or the works of man, on this or adjacent properties. In addition, changes in applicable standards of practice can occur, whether from legislation or the broadening of knowledge. Accordingly, the opinions presented in this report may be invalidated, wholly or partially, by changes outside of our control. Therefore, this report is subject to review and should not be relied upon after a period of 2 years. In addition, this report should not be used and is not applicable for any property other than that evaluated.

Our conclusions and interpretations are also based on conditions at the time of our work. We cannot preclude changes that may occur in the future that could alter site conditions. This is especially true in Humboldt County, which is located in a dynamic geologic environment subject to large scale, catastrophic events (such as great earthquakes and large storms).

Lastly, this report applies only to the site described above. Because of the high degree of variability in geology in this region, it is not possible to extrapolate the results described herein to any other site. This report is to be considered in its entirety. No part, section, paragraph, sentence, or phrase is to be quoted, evaluated, or otherwise used without considering its context and relationship to the entire report.

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Geotechnical Boring Logs

1

BORING NUMBER B-1-LH

CLIENT	Garberville Sanitary District	**PROJECT NAME**	Main Tank
PROJECT NUMBER	022067.400	**PROJECT LOCATION**	APN 032-211-021, Humboldt County
DATE STARTED	6/8/23 **COMPLETED** 6/8/23	**GROUND ELEVATION**	**HOLE SIZE** 4"
DRILLING CONTRACTOR	Taber Drilling	**GROUNDWATER DEPTH**	
DRILLING METHOD	Solid Flight Augers	∇ **AT TIME OF DRILLING**	---
LOGGED BY	A. Troia **CHECKED BY** G. Simpson		
NOTES			

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		(CL-ML) SILT-CLAY, dry, brown, trace very fine sand, organics.										
		(CL-ML) SILTY LEAN CLAY, stiff to very stiff, dry, strong brown, low to medium plasticity, strong cementation, difficult to break samples; trace very fine sand.	SPT S1	100	5-7-8 (15)	>4.5						
		UC Test Undrained shear strength = 3613 psf	MCS S2, S3	100	8-12-13 (25)	>4.5	103	19				
5		(CL) Grades to SANDY LEAN CLAY, very stiff, dry to moist, strong brown/olive mottling; medium plasticity; very fine sand.	SPT S4	100	4-7-9 (16)				36	21	15	
		(SC) CLAYEY SAND-SANDY LEAN CLAY, medium dense/very stiff, dry to moist, strong brown, medium plasticity fines, strong cementation; occasional fine to coarse well-rounded silicious gravel, (WILDCAT FM?). **UC Test** Undrained shear strength = 1591 psf	MCS S5, S6	100	7-11-15 (26)	>4.5	107	20				
10		(SC) CLAYEY SAND, medium dense, dry to moist, strong brown; very fine sand; strong cementation; weak mottling and iron-oxide staining.	SPT S7	100	4-7-9 (16)							
15		Fine, cemented root casts at ~15'. Layer or lense of strong cementation. **UC Test** Undrained shear strength = 1191 psf	MCS S8, S9	100	6-10-11 (21)	3.5	104	22				
		(CL) Grades to SANDY LEAN CLAY, medium dense, moist, light brown to olive; medium plasticity; very fine sand; iron-oxide staining.										
20												

(Continued Next Page)

CLIENT Garberville Sanitary District

PROJECT NAME Main Tank

PROJECT NUMBER 022067.400

PROJECT LOCATION APN 032-211-021, Humboldt County

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
20		(SC) CLAYEY SAND, medium dense, moist, dark yellowish-brown and tan (mottled); moderate to strong cementation, medium plasticity fines; thin clayey interbeds, thinly bedded sand with alternationg oxidized beds.	SPT S10	100	6-8-9 (17)							49
25		(SP-SC) POORLY GRADED SAND with CLAY, dense, moist, strong brown; strong cementation; fine sand with occasional coarse well-rounded sand; mottled.	MCS S11, S12	100	11-13-27 (40)	3.5						
30		No sample recovery; driller notes possible groundwater.										
35		(SC) CLAYEY SAND; dense, wet, olive; strong cementation; fine sand with interbedded medium to coarse subangular sand.	SPT S13	100	9-16-16 (32)							46
40												

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CLIENT Garberville Sanitary District
PROJECT NAME Main Tank
PROJECT NUMBER 022067.400
PROJECT LOCATION APN 032-211-021, Humboldt County

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
		(SC) CLAYEY SAND; dense, wet, olive; strong cementation; fine sand with interbedded medium to coarse subangular sand. *(continued)*										
45		Becomes gray, medium dense, moist; very fine to find sand with trace medium sand; strong cementation, strong cohesion with medium plasticity fines; no dilatency.	SPT S14	100	6-9-10 (19)							
50		(CL) SANDY LEAN CLAY; very stiff, moist, gray to dark gray; very fine to fine sand; strong cementation with moderate toughness; sand is interbedded.	SPT S15	100	7-9-13 (22)							

Bottom of borehole at 51.5 feet.

BORING NUMBER B-2-LH

CLIENT Garberville Sanitary District
PROJECT NAME Main Tank
PROJECT NUMBER 022067.400
PROJECT LOCATION APN 032-211-021, Humboldt County
DATE STARTED 6/8/23 **COMPLETED** 6/8/23
GROUND ELEVATION **HOLE SIZE** 4"
DRILLING CONTRACTOR Taber Drilling
GROUNDWATER DEPTH
DRILLING METHOD Solid Flight Augers
AT TIME OF DRILLING ---
LOGGED BY A. Troia **CHECKED BY** G. Simpson
NOTES

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		(CL-ML) SILT-CLAY, stiff, dry, strong brown, moderate cementation, low plasticity fines, very fine sand, very fine roots/organics.	SPT S1	100	5-6-7 (13)							
		(CL-ML) SILTY LEAN CLAY, stiff, dry, strong brown; low to medium plasticity, moderate to strong cementation; slight mottling; <10% very fine sand, (WILDCAT FM.) **UC Test** Undrained shear strength = 1475 psf	MCS S2, S3	100	6-7-7 (14)	4.25	99	19				
5		(CL) SANDY LEAN CLAY, very stiff, dry, strong brown; very fine sand with occasional coarse sand.	SPT S4	100	4-6-11 (17)							66
10		(CL) SANDY LEAN CLAY with GRAVEL, hard, dry, strong brown; mottling/iron oxide staining; medium plasticity fines; fine well-rounded gravel and coarse sand.	MCS S5, S6	100	13-17-26 (43)	>4.5	103	23				
15		(SC) CLAYEY SAND, dense, moist, strong brown; strong cementation, weakly stratified, no dilatency; fine sand.	SPT S7	100	8-11-13 (24)							
20												

(Continued Next Page)

CLIENT Garberville Sanitary District **PROJECT NAME** Main Tank

PROJECT NUMBER 022067.400 **PROJECT LOCATION** APN 032-211-021, Humboldt County

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
20		(SC) CLAYEY SAND, dense, moist, strong brown; strong cementation, weakly stratified, no dilatency; fine sand. *(continued)*	SPT S8	100	9-10-12 (22)							40
25												
30		Becomes medium dense, moist, strong cementation and cohesion; iron-oxide staining.	MCS S9, S10	100	7-10-12 (22)		109	20				
35												
40		(CL) SANDY LEAN CLAY; very stiff, gray to bluish gray, low to medium plasticity, fine sand.	MCS S11, S12	100	6-11-15 (26)	2.75	111	20				70

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CLIENT Garberville Sanitary District **PROJECT NAME** Main Tank

PROJECT NUMBER 022067.400 **PROJECT LOCATION** APN 032-211-021, Humboldt County

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
45		(SC) CLAYEY SAND, dense, moist, strong brown; strong cementation, weakly stratified, no dilatency; fine sand. *(continued)*										
50		(CL) LEAN CLAY, very stiff, dry, gray to dark gray; strong cementation, difficult to break with knife; low plasticity; occasional completely weathered, rounded, medium gravels.	MCS S13, S14	100	13-18-23 (41)	>4.5						

Bottom of borehole at 51.5 feet.

BORING NUMBER B-3-LH

CLIENT Garberville Sanitary District
PROJECT NAME Main Tank
PROJECT NUMBER 022067.400
PROJECT LOCATION APN 032-211-021, Humboldt County
DATE STARTED 6/8/23 **COMPLETED** 6/8/23
GROUND ELEVATION **HOLE SIZE** 4"
DRILLING CONTRACTOR Taber Drilling
GROUNDWATER DEPTH
DRILLING METHOD Solid Flight Augers
▽ **AT TIME OF DRILLING** ---
LOGGED BY A. Troia **CHECKED BY** G. Simpson
NOTES

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		(SC) CLAYEY SAND to SANDY CLAY, loose/stiff, moist, strong brown; very fine to fine sand; low to medium plasticity fines; moderate cementation; slightly mottled, fine roots/organics in upper 6" and occasional to 10'.										
5			SPT S1	100	2-4-5 (9)							
10		(CL) SANDY CLAY, hard, dry, strong brown; mostly fine sand with medium to coarse sand; strong cementation; occasional rounded fine to medium gravels, (WILDCAT FM.) **UC Test** Undrained shear strength = 1605 psf	MCS S2, S3	100	10-21-21 (42)	4.0	103	22				
15			SPT S4	100	4-7-8 (15)							
20												

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CLIENT Garberville Sanitary District **PROJECT NAME** Main Tank

PROJECT NUMBER 022067.400 **PROJECT LOCATION** APN 032-211-021, Humboldt County

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	FINES CONTENT (%)
20		(CL) SANDY CLAY, hard, dry, strong brown; mostly fine sand with medium to coarse sand; strong cementation; occasional rounded fine to medium gravels, (WILDCAT FM.) *(continued)*	MCS S5, S6	100	10-21-24 (45)		101	20				54
25		Becomes moist.	SPT S7	100	12-12-16 (28)							

Bottom of borehole at 26.5 feet.

BORING NUMBER B-4-LH

CLIENT Garberville Sanitary District
PROJECT NAME Main Tank
PROJECT NUMBER 022067.400
PROJECT LOCATION APN 032-211-021, Humboldt County
DATE STARTED 6/8/23 **COMPLETED** 6/8/23
GROUND ELEVATION **HOLE SIZE** 4"
DRILLING CONTRACTOR Taber Drilling
GROUNDWATER DEPTH
DRILLING METHOD Solid Flight Augers
▽ **AT TIME OF DRILLING** ---
LOGGED BY A. Troia **CHECKED BY** G. Simpson
NOTES

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		(CL) SANDY LEAN CLAY, stiff, dry to moist, strong brown; moderate cementation, low plasticity; very fine sand with occasional coarse sand; organics in upper ~6".										
5			SPT S1	100	3-4-5 (9)							66
10		Becomes very stiff; mottled.	SPT S2	100	3-7-10 (17)				31	24	7	
15		(SC) CLAYEY SAND, medium dense, moist, brown; fine to medium sand.	SPT S3	100	5-10-13 (23)							42

Bottom of borehole at 16.5 feet.

BORING NUMBER B-1-APS

CLIENT Garberville Sanitary District
PROJECT NUMBER 022067.400
DATE STARTED 6/21/23 **COMPLETED** 6/21/23
DRILLING CONTRACTOR Taber Drilling
DRILLING METHOD Solid Flight Augers
LOGGED BY A. Troia **CHECKED BY** G. Simpson
NOTES

PROJECT NAME Alderpoint Pump Station
PROJECT LOCATION APN 223-183-003, Humboldt County
GROUND ELEVATION 550 ft (approx.) **HOLE SIZE** 4"
GROUNDWATER DEPTH
∇ **AT TIME OF DRILLING** --- Not encountered.

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		(ML) SILT with GRAVELS, dry, dark brown, mottled at base; subrounded, fine to medium gravels, (TOPSOIL/WILDCAT FM.)										
		(SM) SILTY SAND, medium dense, moist, strong brown and gray (mottled); very fine to fine sand, (WILDCAT FM.)	MCS		7-10-11 (21)		116	13				44
		Becomes loose; same as above with occasional fine to medium gravels.	MCS		3-4-5 (9)		112	16				
5		Becomes medium dense with moderate cementation; increased gravel content.	MCS		6-8-10 (18)		112	16				
		(SC) CLAYEY SAND, medium dense, dry to moist, strong brown; strong cementation; very fine to fine sand, occasional rounded fine to medium gravel; moderately cohesive, low to medium plasticity fines; mottled.	SPT		4-6-11 (17)							
10		(CL) SANDY LEAN CLAY, very stiff, dry to moist, strong brown; strong cementation; medium plasticity fines; very fine to fine sand, mottled.	SPT		5-8-10 (18)							
15		(SC) CLAYEY SAND with GRAVEL, dense, dry to moist, strong brown; strong cementation, fine to medium subrounded gravels, fine sand.	SPT		15-19-30 (49)							

Bottom of borehole at 16.5 feet.

GEOTECH BH COLUMNS - DATA TEMPLATE FOR TESTING.GDT - 8/9/23 15:44 - \\EUREKA\GEOGROUP\GINT\LIBRARY\BENTLEY\GINTCL\PROJECTS\PROJECT_FILES\2022\022067_GSD_ALDERPOINTPS.GPJ

BORING NUMBER B-1-W

CLIENT Garberville Sanitary District

PROJECT NUMBER 022067.400

DATE STARTED 6/21/23 **COMPLETED** 6/21/23

DRILLING CONTRACTOR Taber Drilling

DRILLING METHOD Solid Flight Augers

LOGGED BY A. Troia **CHECKED BY** G. Simpson

NOTES

PROJECT NAME Wallan Tank

PROJECT LOCATION APN 223-191-006, Humboldt County

GROUND ELEVATION 1153 ft NAVD88 **HOLE SIZE** 4"

GROUNDWATER DEPTH

∇ **AT TIME OF DRILLING** --- Not Encountered.

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		(ML) SILT with GRAVEL, dry, brown, fine to medium subrounded gravel.										
		SANDSTONE BOULDER, dry, strong brown; fine sand; highly fractured, requires light effort to break sample with knife.	MCS	100	10-15-34 (49)		120	22				
		(CL) LEAN CLAY, stiff, dry, brown, moderate cementation, medium plasticity; trace fine micaceous sand; occasional angular, well-graded gravels.	SPT	100	10-7-6 (13)				33	19	14	
5		SANDSTONE BEDROCK; highly weathered, moderately soft, highly fractured, fine sand; oxidized; zones of silty-clay throughout.	MCS	100	10-16-27 (43)		106	19				
		Becomes dark brown.	SPT	100	14-15-15 (30)							
10			MCS	100	16-13-18 (31)							
15		Same as above.	SPT	100	12-12-15 (27)							

Bottom of borehole at 16.5 feet.

BORING NUMBER B-2-W

CLIENT Garberville Sanitary District

PROJECT NAME Wallan Tank

PROJECT NUMBER 022067.400

PROJECT LOCATION APN 223-191-006, Humboldt County

DATE STARTED 6/21/23 **COMPLETED** 6/21/23

GROUND ELEVATION 1153 ft NAVD88 **HOLE SIZE** 4"

DRILLING CONTRACTOR Taber Drilling

GROUNDWATER DEPTH

DRILLING METHOD Solid Flight Augers

AT TIME OF DRILLING --- Not Encountered.

LOGGED BY A. Troia **CHECKED BY** G. Simpson

NOTES

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		(ML) SILT with GRAVEL, dry, brown, fine to medium subrounded gravels.										
		(CL-ML) SILTY CLAY with SAND, stiff, dry, brown; very fine sand, medium angular gravels, moderate cementation. **UC Test** Undrained shear strength = 1498 psf	MCS	100	10-9-11 (20)							
		(GW-GM) SILTY GRAVEL with SAND, medium dense, dry, brown; angular, well-graded gravels.	MCS	100	10-11-16 (27)		104	16				
5		SILTY SANDSTONE, moderately soft, dry, strong brown and gray, highly fractured.	SPT	100	8-7-13 (20)		117	11				
			SPT	44	7-12-10 (22)							
10		Slightly increased cementation.	MCS	100	11-13-14 (27)							
15		Same as above.	MCS	100	17-16-15 (31)							

Bottom of borehole at 16.5 feet.

Laboratory Results 2

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DENSITY BY DRIVE- CYLINDER METHOD (ASTM D2937)

Project Name:	**GSD-APS**	**Project Number:**	**022067.400**
Performed By:	**JMA**	**Date:**	**7/12/2023**
Checked By:	**KEW**	**Date:**	**7/18/2023**
Project Manager:	**JSO**		

Lab Sample Number	23-671	23-673	23-675		
Boring Label	B-1-APS	B-1-APS	B-1-APS		
Sample Depth (ft)	2-2.5'	4-4.5'	6-6.5'		
Diameter of Cylinder, in	2.42	2.42	2.42		
Total Length of Cylinder, in.	6.00	6.00	6.00		
Length of Empty Cylinder A, in.	0.00	0.00	0.00		
Length of Empty Cylinder B, in.	0.90	0.80	0.52		
Length of Cylinder Filled, in	5.10	5.20	5.48		
Volume of Sample, in^3	23.46	23.92	25.21		
Volume of Sample, cc.	384.41	391.94	413.05		

Pan #	SS8	SS3	SS12		
Weight of Wet Soil and Pan	1000.0	1013.0	1051.1		
Weight of Dry Soil and Pan	904.8	898.3	934.7		
Weight of Water	95.2	114.7	116.4		
Weight of Pan	192.9	197.0	194.2		
Weight of Dry Soil	711.9	701.3	740.5		
Percent Moisture	13.4	16.4	15.7		
Dry Density, g/cc	1.85	1.79	1.79		
Dry Density, lb/ft^3	115.6	111.7	111.9		

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PERCENT PASSING # 200 SIEVE (ASTM - D1140)

Project Name:	GSD-APS	Project Number:	022067.400
Performed By:	JMA	Date:	7/14/2023
Checked By:	KH	Date:	7/18/2023
Project Manager:	JSO		

Lab Sample Number	23-670				
Boring Label	B-1-APS				
Sample Depth	1.5-2.0'				
Pan Number	ss8				
Dry Weight of Soil & Pan	569.8				
Pan Weight	193.5				
Weight of Dry Soil	376.3				
Soil Weight Retained on #200&Pan	402.5				
Soil Weight Passing #200	167.3				
Percent Passing #200	**44**				

Lab Sample Number					
Boring Label					
Sample Depth					
Pan Number					
Dry Weight of Soil & Pan					
Pan Weight					
Weight of Dry Soil					
Soil Weight Retained on #200&Pan					
Soil Weight Passing #200					
Percent Passing #200					

Garberville Sanitary District Water Improvements - Geotech
Lower Hurlbutt site (one of three sites)

p1 of 3

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MATERIALS TESTING LABORATORY RECEIVING AND SCHEDULING OF TESTS

PROJECT NAME GSD WATER LH | Date Sampled 6/07/2023 | Sampled by ART

JOB NUMBER 022067.400 | Date Received | Results to ART

PROJECT MANAGER ART (J. O'BARR PM) | Date Recorded | ☐ Lab Billing Program Submitted

↳ TASK MANAGER FOR GEOTECH

TOTAL NUMBER OF SAMPLES BAGS ____ BUCKETS ____ SHELBY TUBES ____ BRASS LINERS ____

SAMPLE CONDITION: INTACT ☐ DAMAGED ☐ COMPOSITE ☐ UNDISTURBED ☐

Client Information:

SAMPLE NO. & DEPTH	MOISTURE DENSITY	UNCONFINED COMPRESSION	USDA TEXTURAL ANALYSIS	COARSE SIEVE ANALYSIS 3" to No.4	FINE SIEVE ANALYSIS No.4 to No. 200	% PASSING 200	SAND EQUIVALENT	SPECIFIC GRAVITY	PLASTICITY INDEX	COMPACTION CURVE (ASTM-1557)	CAL TRANS CURVE (CT-216)	COMPACTION CHECK POINT	% ORGANICS	LA RATTLER	DURABILITY	SULFATE SOUNDNESS	RICE SPECIFIC GRAVITY	R VALUE	CONSOLIDATION *		DIRECT SHEAR **	TRIAXIAL SHEAR **	LAB SAMPLE NUMBER
B-1-LH 1-2.5																							
3.5-4																							
4-4.5																							
5-6.5																							
7.5-8																							
8-8.5																							
10-11.5																							
15.5-16																							
16-16.5																							
20-21.5'																							
25.5-26																							
26-26.5																							
35-36.5'																							
45-46.5'																							
50-51.5'																							
B-2-LH 1-2.5																							
3.5-4																							
TOTAL																							

COMMENTS:

Samples will be retained for 90 days after completion of the testing program. If samples need to bre retained beyond 90 days, indicate how long to retain this sample program ____

* Indicate The Following: Consolidation Loads: ____ | Consolidated Drained: ☐ | note all points to be saturated
Consolidated Undrained: ☐ | Residual Cycles ____

** Indicate The Following: Confining loads: ____ | Unconsolidated Undrained: ☐

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MATERIALS TESTING LABORATORY RECEIVING AND SCHEDULING OF TESTS

PROJECT NAME GSD Water LH | Date Sampled 6/7/23 | Sampled by ART

JOB NUMBER 022067.406 | Date Received | Results to ART

PROJECT MANAGER ART/JSO | Date Recorded | ☐ Lab Billing Program Submitted

TOTAL NUMBER OF SAMPLES BAGS ____ BUCKETS ____ SHELBY TUBES ____ BRASS LINERS ____

SAMPLE CONDITION: INTACT ☐ DAMAGED ☐ COMPOSITE ☐ UNDISTURBED ☐

Client Information:

SAMPLE NO. & DEPTH	MOISTURE DENSITY	UNCONFINED COMPRESSION	USDA TEXTURAL ANALYSIS	COARSE SIEVE ANALYSIS 3" to No.4	FINE SIEVE ANALYSIS No.4 to No. 200	% PASSING 200	SAND EQUIVALENT	SPECIFIC GRAVITY	PLASTICITY INDEX	COMPACTION CURVE (ASTM-1557)	CAL TRANS CURVE (CT-216)	COMPACTION CHECK POINT	% ORGANICS	LA RATTLER	DURABILITY	SULFATE SOUNDNESS	RICE SPECIFIC GRAVITY	R VALUE	CONSOLIDATION *		DIRECT SHEAR **	TRIAXIAL SHEAR **	LAB SAMPLE NUMBER
B-2-LH 4-4.5		■																					
5-6.5'						■																	
10.5-11																							
11-11.5	■																						
15-16.5																							
20-21.5						■																	
30.5-31																							
31-31.5	■																						
40.5-41						■																	
41-41.5	■																						
50.5-51																							
51-51.5																							
B-3-LH 5-6.5									■														
10.5-11																							
11-11.5		■																					
15-16.5																							
20.5-21						■																	
TOTAL																							

COMMENTS:

Samples will be retained for 90 days after completion of the testing program. If samples need to bre retained beyond 90 days, indicate how long to retain this sample program ________

* Indicate The Following: Consolidation Loads: ________ Consolidated Drained: ☐ note all points to be saturated
Consolidated Undrained: ☐ Residual Cycles ________
** Indicate The Following: Confining loads: ________ Unconsolidated Undrained: ☐

(LOWER HURLBUTT)

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MATERIALS TESTING LABORATORY RECEIVING AND SCHEDULING OF TESTS

PROJECT NAME GSD WATER LH | Date Sampled 6/7/23 | Sampled by ART

JOB NUMBER 022067.400 | Date Received | Results to ART

PROJECT MANAGER ART / JSO | Date Recorded | ☐ Lab Billing Program Submitted

TOTAL NUMBER OF SAMPLES | BAGS ____ | BUCKETS ____ | SHELBY TUBES ____ | BRASS LINERS ____

SAMPLE CONDITION: INTACT ☐ DAMAGED ☐ | COMPOSITE ☐ UNDISTURBED ☐

Client Information:

SAMPLE NO. & DEPTH	MOISTURE DENSITY	UNCONFINED COMPRESSION	USDA TEXTURAL ANALYSIS	COARSE SIEVE ANALYSIS 3" to No.4	FINE SIEVE ANALYSIS No.4 to No. 200	% PASSING 200	SAND EQUIVALENT	SPECIFIC GRAVITY	PLASTICITY INDEX	COMPACTION CURVE (ASTM-1557)	CAL TRANS CURVE (CT-216)	COMPACTION CHECK POINT	% ORGANICS	LA RATTLER	DURABILITY	SULFATE SOUNDNESS	RICE SPECIFIC GRAVITY	R VALUE	CONSOLIDATION *		DIRECT SHEAR **	TRIAXIAL SHEAR **	LAB SAMPLE NUMBER
B-3-LH 21-21.5	■																						
25-26.5'																							
B-4-LH 5-6.5'						■																	
10-11.5'									■														
15-16.5'						■																	
TOTAL																							

COMMENTS:

Please indicate the total quantity of brass linners to be cleaned, a unit fee of $3 per sample will be applied ____

Samples will be retained for 90 days after completion of the testing program. If samples need to bre retained beyond 90 days, indicate how long to retain this sample program ____

* Indicate The Following: Consolidation Loads: ____ | Consolidated Drained: ☐ | note all points to be saturated

Consolidated Undrained: ☐ | Residual Cycles ____

** Indicate The Following: Confining loads: ____ | Unconsolidated Undrained: ☐

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DENSITY BY DRIVE- CYLINDER METHOD (ASTM D2937)

Project Name:	**GSD Water- Main Tank**	**Project Number:**	**022067.400**
Performed By:	**KEW**	**Date:**	**6/28/2023**
Checked By:	**KEW**	**Date:**	**7/10/2023**
Project Manager:	**JOB**		

Lab Sample Number	23-598	23-602	23-604	23-612	
Boring Label	B-2-LH	B-2-LH	B-2-LH	B-3-LH	
Sample Depth (ft)	11-11.5'	31-31.5'	41-41.5'	21-21.5'	
Diameter of Cylinder, in	2.40	2.40	2.40	2.40	
Total Length of Cylinder, in.	6.00	6.00	6.00	6.00	
Length of Empty Cylinder A, in.	0.00	0.00	0.00	0.00	
Length of Empty Cylinder B, in.	0.60	0.65	0.80	1.00	
Length of Cylinder Filled, in	5.40	5.35	5.20	5.00	
Volume of Sample, in^3	24.43	24.20	23.52	22.62	
Volume of Sample, cc.	400.32	396.61	385.49	370.67	

Pan #	SS14	SS2	SS6	SS1	
Weight of Wet Soil and Pan	1000.6	1025.3	1015.7	915.9	
Weight of Dry Soil and Pan	850.6	886.9	882.2	795.2	
Weight of Water	150.0	138.4	133.5	120.7	
Weight of Pan	192.7	193.4	195.9	195.1	
Weight of Dry Soil	657.9	693.5	686.3	600.1	
Percent Moisture	22.8	20.0	19.5	20.1	
Dry Density, g/cc	1.64	1.75	1.78	1.62	
Dry Density, lb/ft^3	102.6	109.2	111.1	101.1	

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PERCENT PASSING # 200 SIEVE (ASTM - D1140)

Project Name:	**GSD Water-Main Tank**	**Project Number:**	**022067.400**
Performed By:	**KEW**	**Date:**	**6/28/2023**
Checked By:	**KEW**	**Date:**	**7/10/2023**
Project Manager:	**JOB**		

Lab Sample Number	23-587	23-590	23-596	23-600	23-603
Boring Label	B-1-LH	B-1-LH	B-2-LH	B-2-LH	B-2-LH
Sample Depth	20-21.5'	35-36.5'	5-6.5'	20-21.5'	40.5-41'
Pan Number	SS8	SS10	SS15	SS3	SS12
Dry Weight of Soil & Pan	362.1	366.1	359.6	365.3	360.1
Pan Weight	192.9	195.3	194.3	197.0	193.9
Weight of Dry Soil	169.2	170.8	165.3	168.3	166.2
Soil Weight Retained on #200&Pan	279.6	287.5	250.7	297.8	243.5
Soil Weight Passing #200	82.5	78.6	108.9	67.5	116.6
Percent Passing #200	**49**	**46**	**66**	**40**	**70**

Lab Sample Number	23-611	23-614	23-616		
Boring Label	B-3-LH	B-4-LH	B-4-LH		
Sample Depth	20.5-21'	5-6.5'	15-16.5'		
Pan Number	SS5	SS7	S8		
Dry Weight of Soil & Pan	367.5	360.9	319.5		
Pan Weight	196.0	193.6	158.8		
Weight of Dry Soil	171.5	167.3	160.7		
Soil Weight Retained on #200&Pan	275.4	250.5	252.5		
Soil Weight Passing #200	92.1	110.4	67.0		
Percent Passing #200	**54**	**66**	**42**		

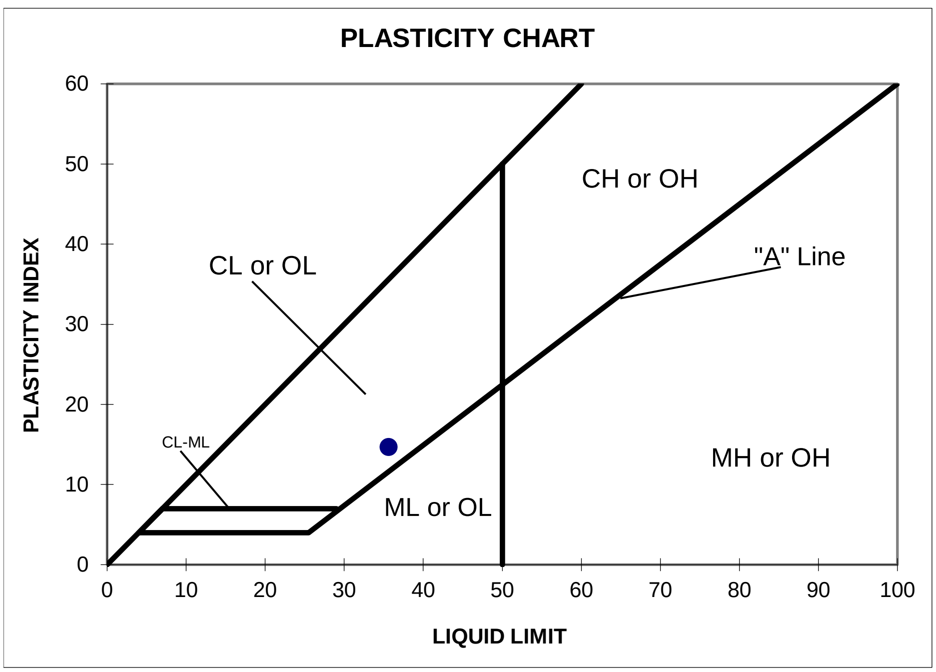
ENGINEERS & GEOLOGISTS, INC.
812 W. Wabash Eureka, CA 95501-2138 Tel: 707/441-8855 FAX: 707/441-8877 E-mail: shninfo@shn-engr.com

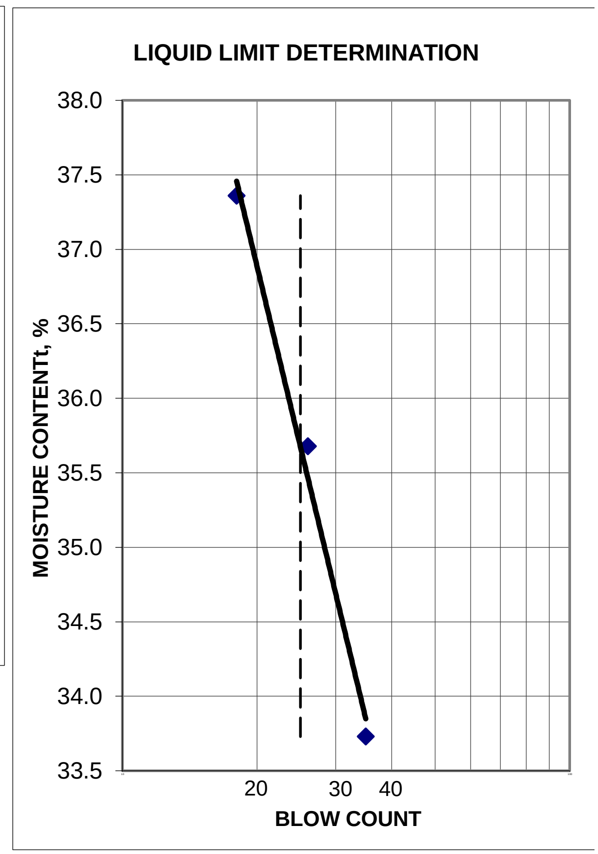
LIQUID LIMIT, PLASTIC LIMIT, and PLASTICITY INDEX (ASTM-D4318)

JOB NAME:	**GSD Water-Main Tank**	JOB #:	**022067.400**	LAB SAMPLE #:	**23-581**
SAMPLE ID:	**B-1 5-6.5**	PERFORMED BY:	**KEW**	DATE:	**7/7/2023**
PROJECT MANAGER:	**JOB**	CHECKED BY:	**KEW**	DATE:	**7/10/2023**

LINE NO.		TRIAL NO. 1	TRIAL NO. 2	TRIAL NO. 1	TRIAL NO. 2	TRIAL NO. 3
A	PAN #	17	18	7	8	9
B	PAN WT. (g)	20.260	20.170	28.900	29.040	28.610
C	WT. WET SOIL & PAN (g)	28.060	28.330	34.530	34.440	34.750
D	WT. DRY SOIL & PAN (g)	26.710	26.920	33.110	33.020	33.080
E	WT. WATER (C-D)	1.350	1.410	1.420	1.420	1.670
F	WT. DRY SOIL (D-B)	6.450	6.750	4.210	3.980	4.470
G	BLOW COUNT	--	--	35	26	18
H	MOISTURE CONTENT (E/F*100)	20.9	20.9	33.7	35.7	37.4

LIQUID LIMIT	**PLASTIC INDEX**	PLASTIC LIMIT
36	**15**	21





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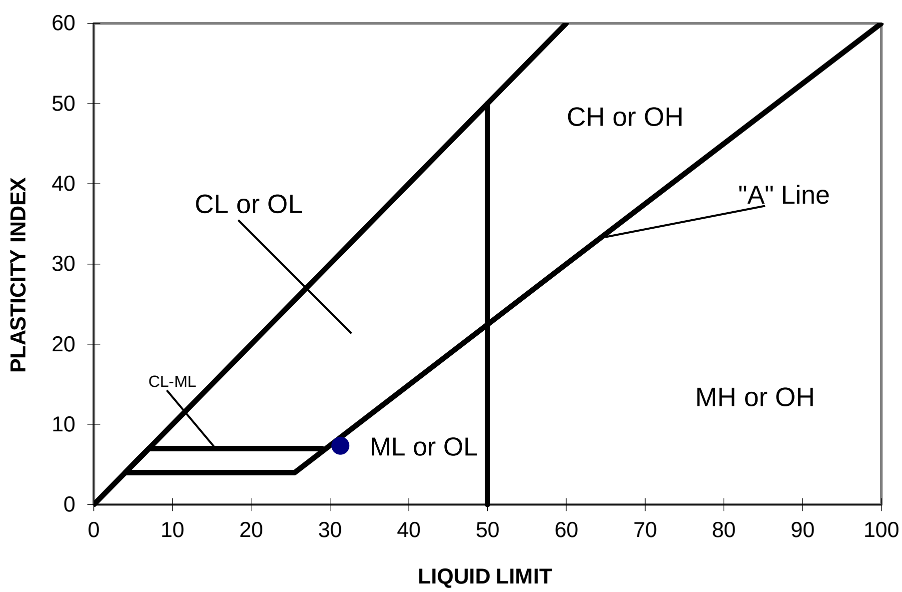
LIQUID LIMIT, PLASTIC LIMIT, and PLASTICITY INDEX (ASTM-D4318)

JOB NAME:	**GSD Water-Main Tank**	JOB #:	**022067.400**	LAB SAMPLE #:	**23-615**
SAMPLE ID:	**B-4 10-11.5'**	PERFORMED BY:	**KEW**	DATE:	**7/7/2023**
PROJECT MANAGER:	**JOB**	CHECKED BY:	**KEW**	DATE:	**7/10/2023**

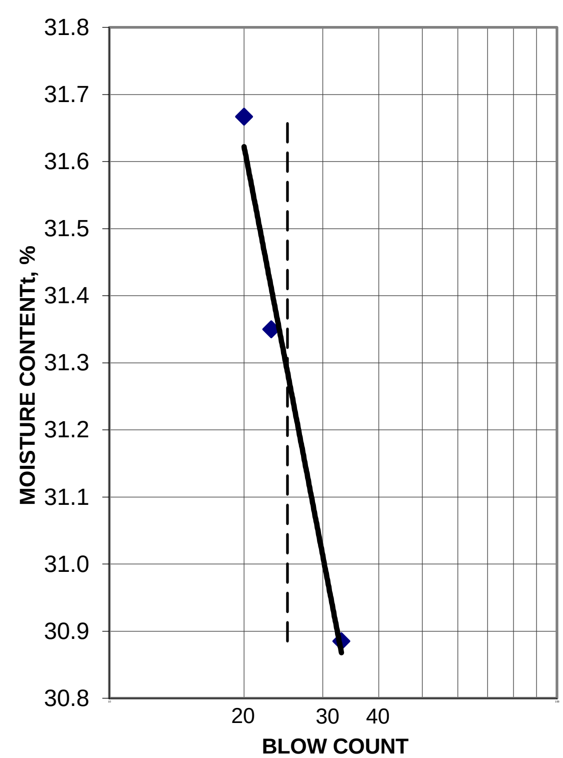
LINE NO.		TRIAL NO. 1	TRIAL NO. 2	TRIAL NO. 1	TRIAL NO. 2	TRIAL NO. 3
A	PAN #	13	14	1	2	3
B	PAN WT. (g)	21.940	20.130	29.580	28.940	28.970
C	WT. WET SOIL & PAN (g)	30.530	28.510	36.530	37.990	35.290
D	WT. DRY SOIL & PAN (g)	28.870	26.890	34.890	35.830	33.770
E	WT. WATER (C-D)	1.660	1.620	1.640	2.160	1.520
F	WT. DRY SOIL (D-B)	6.930	6.760	5.310	6.890	4.800
G	BLOW COUNT	--	--	33	23	20
H	MOISTURE CONTENT (E/F*100)	24.0	24.0	30.9	31.3	31.7

LIQUID LIMIT	**PLASTIC INDEX**	PLASTIC LIMIT
31	**7**	24

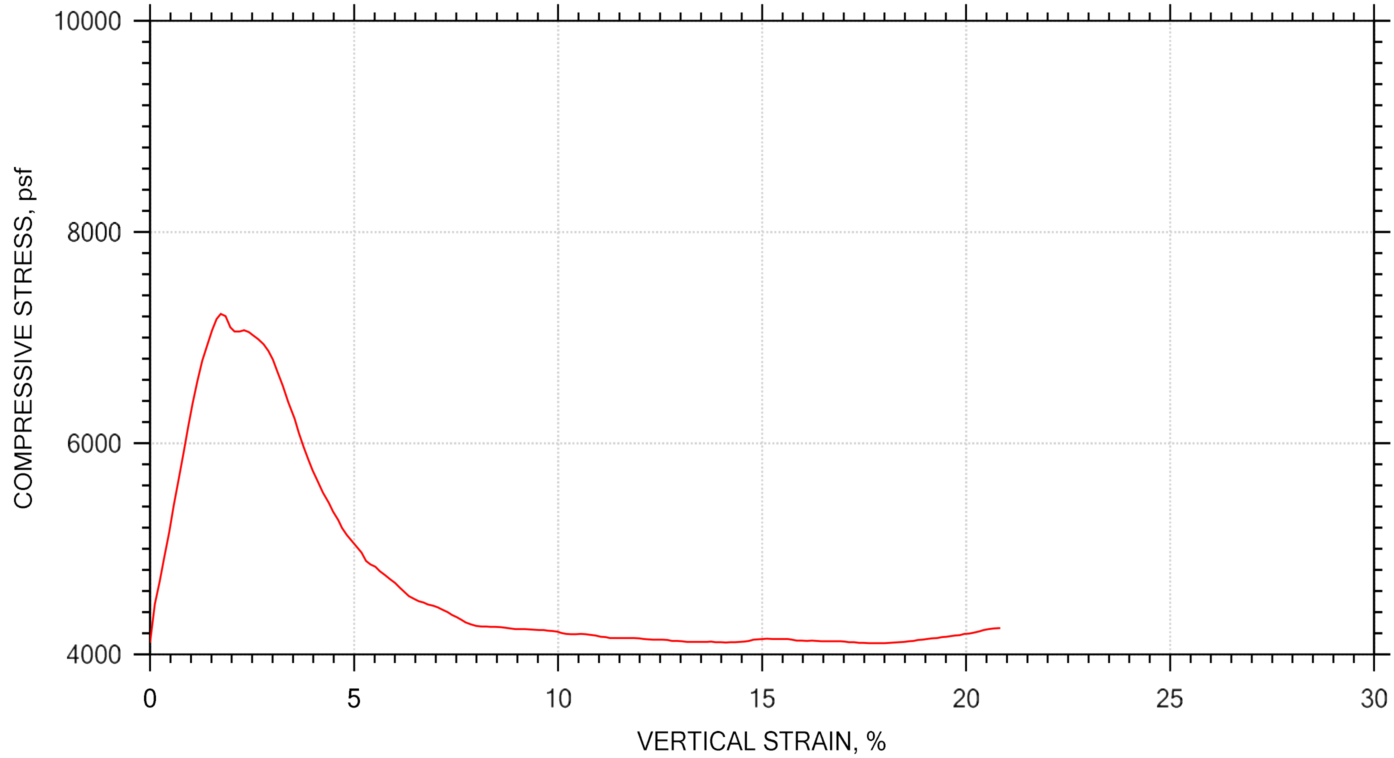
PLASTICITY CHART



LIQUID LIMIT DETERMINATION



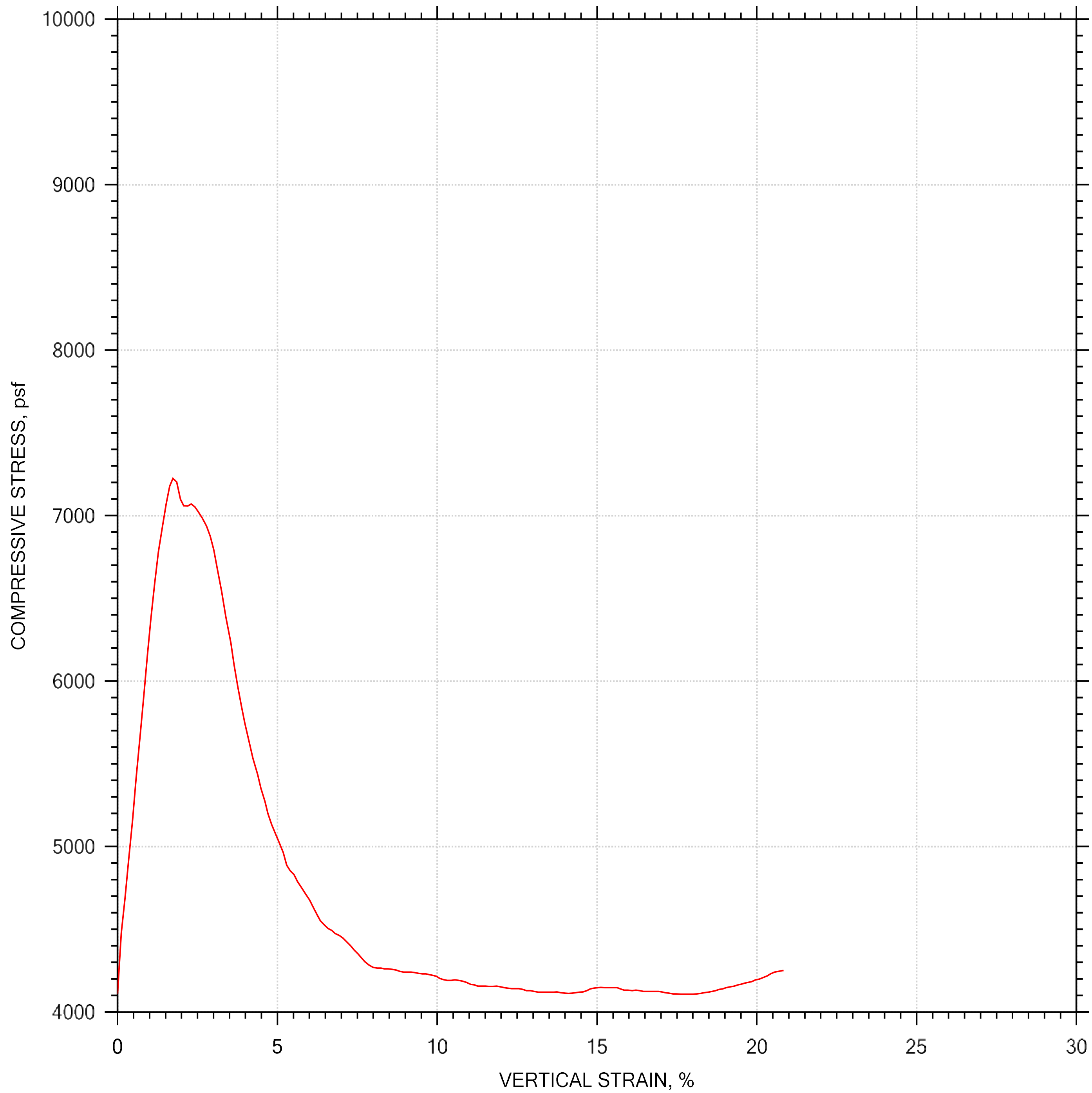
UNCONFINED COMPRESSION TEST REPORT



Symbol					
Test No.		23-580			
Initial	Diameter, in	2.4			
	Height, in	4.8			
	Water Content, %	19.19			
	Dry Density, pcf	103.1			
	Saturation, %	84.16			
	Void Ratio	0.604			
Unconfined Compressive Strength, psf		7225			
Undrained Shear Strength, psf		3613			
Time to Failure, min		1.5025			
Strain Rate, %/min		0.01			
Estimated Specific Gravity		2.65			
Liquid Limit		---			
Plastic Limit		---			
Plasticity Index		---			
Failure Sketch					

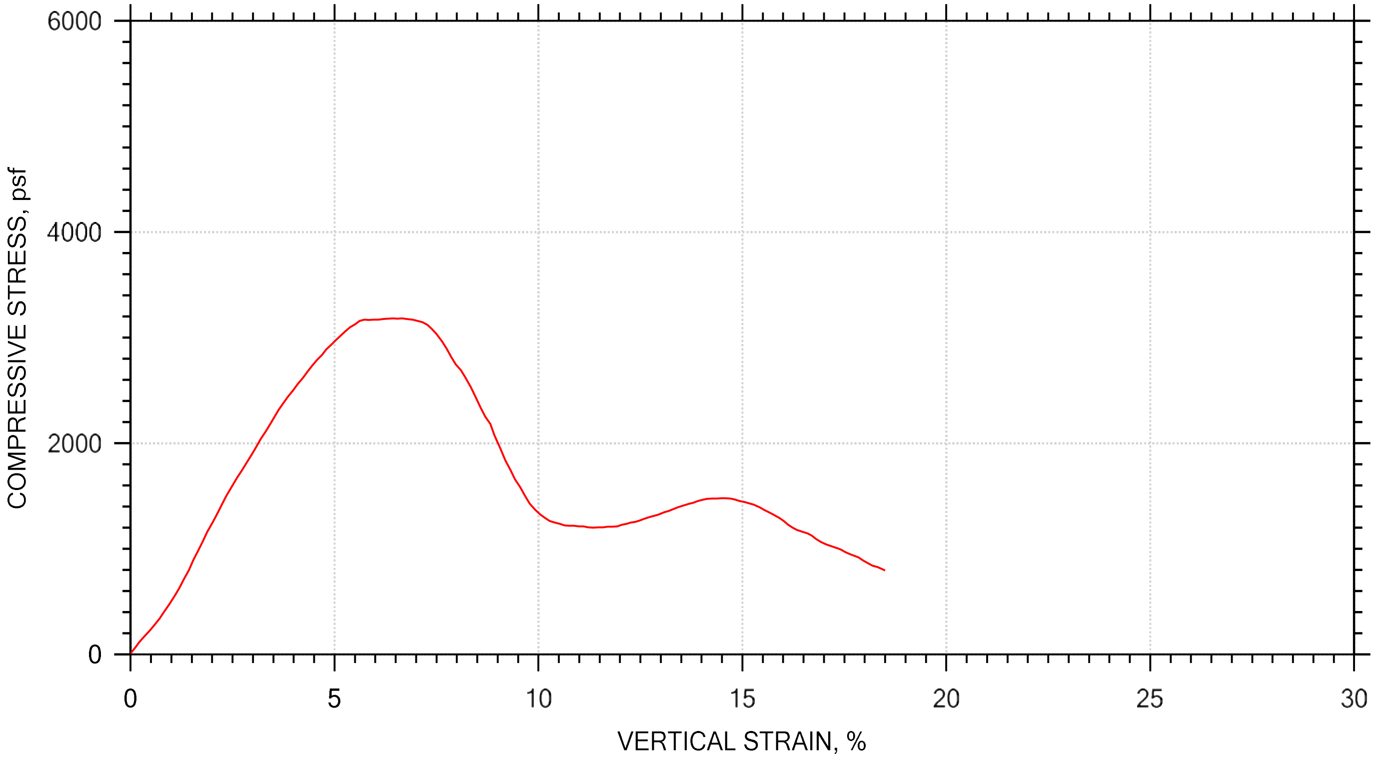
Project: GSD Water	Location: Garberville	Project No.: 022067.400
Boring No.: B-1-LH	Tested By: KEW	Checked By:
Sample No.: 3	Test Date: 6/29/23	Elevation:
Test No.: 23-580	Preparation: Undisturbed	Depth: 4-4.5'
Description:		
Remarks:		

UNCONFINED COMPRESSION TEST REPORT



	Project: GSD Water	Location: Garberville	Project No.: 022067.400
	Boring No.: B-1-LH	Tested By: KEW	Checked By:
	Sample No.: 3	Test Date: 6/29/23	Elevation:
	Test No.: 23-580	Preparation: Undisturbed	Depth: 4-4.5'
	Description:		
	Remarks:		

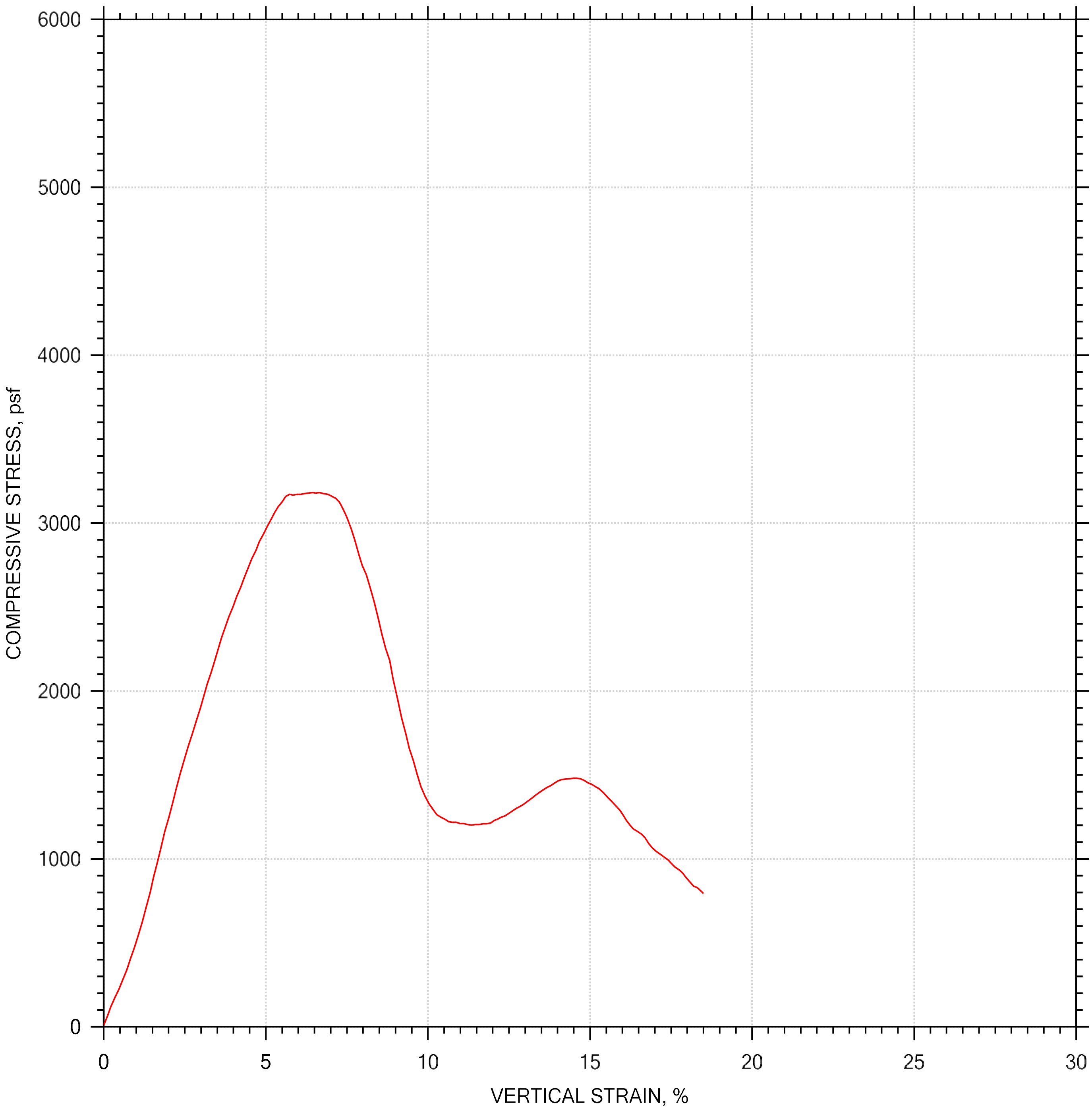
UNCONFINED COMPRESSION TEST REPORT



Symbol					
Test No.		23-583			
Initial	Diameter, in	2.4			
	Height, in	4.9			
	Water Content, %	20.16			
	Dry Density, pcf	106.7			
	Saturation, %	96.93			
	Void Ratio	0.551			
Unconfined Compressive Strength, psf		3182			
Undrained Shear Strength, psf		1591			
Time to Failure, min		5.7042			
Strain Rate, %/min		0.01			
Estimated Specific Gravity		2.65			
Liquid Limit		---			
Plastic Limit		---			
Plasticity Index		---			
Failure Sketch					

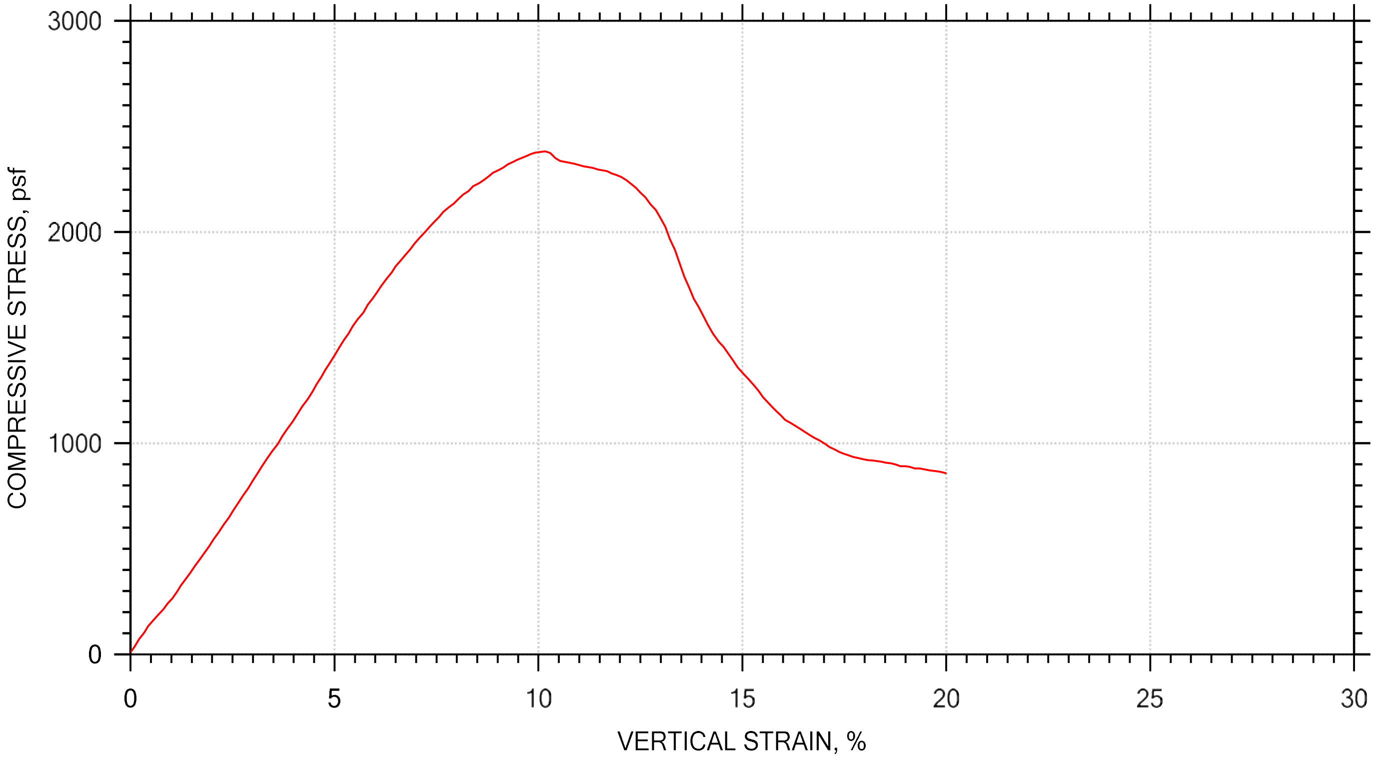
Project: GSD Water	Location: Garberville	Project No.: 022067.400
Boring No.: B-1-LH	Tested By: KEW	Checked By: KEW
Sample No.: 6	Test Date: 6/29/23	Elevation:
Test No.: 23-583	Preparation: Undisturbed	Depth: 8-8.5'
Description:		
Remarks:		

UNCONFINED COMPRESSION TEST REPORT



	Project: GSD Water	Location: Garberville	Project No.: 022067.400
	Boring No.: B-1-LH	Tested By: KEW	Checked By: KEW
	Sample No.: 6	Test Date: 6/29/23	Elevation:
	Test No.: 23-583	Preparation: Undisturbed	Depth: 8-8.5'
	Description:		
	Remarks:		

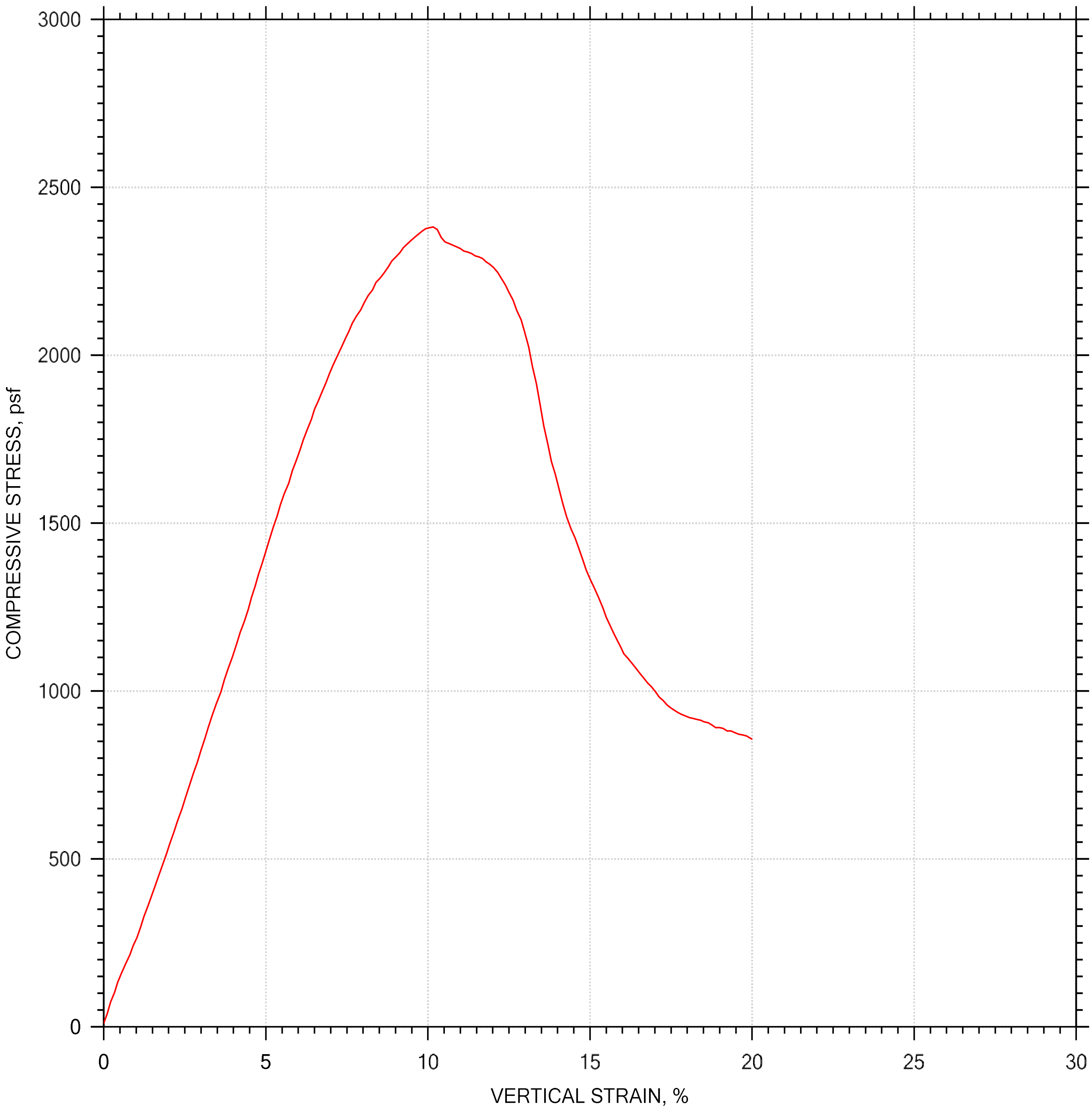
UNCONFINED COMPRESSION TEST REPORT



Symbol					
Test No.		23-586			
Initial	Diameter, in	2.4			
	Height, in	5			
	Water Content, %	21.96			
	Dry Density, pcf	103.8			
	Saturation, %	97.95			
	Void Ratio	0.594			
Unconfined Compressive Strength, psf		2382			
Undrained Shear Strength, psf		1191			
Time to Failure, min		8.7029			
Strain Rate, %/min		0.01			
Estimated Specific Gravity		2.65			
Liquid Limit		---			
Plastic Limit		---			
Plasticity Index		---			
Failure Sketch					

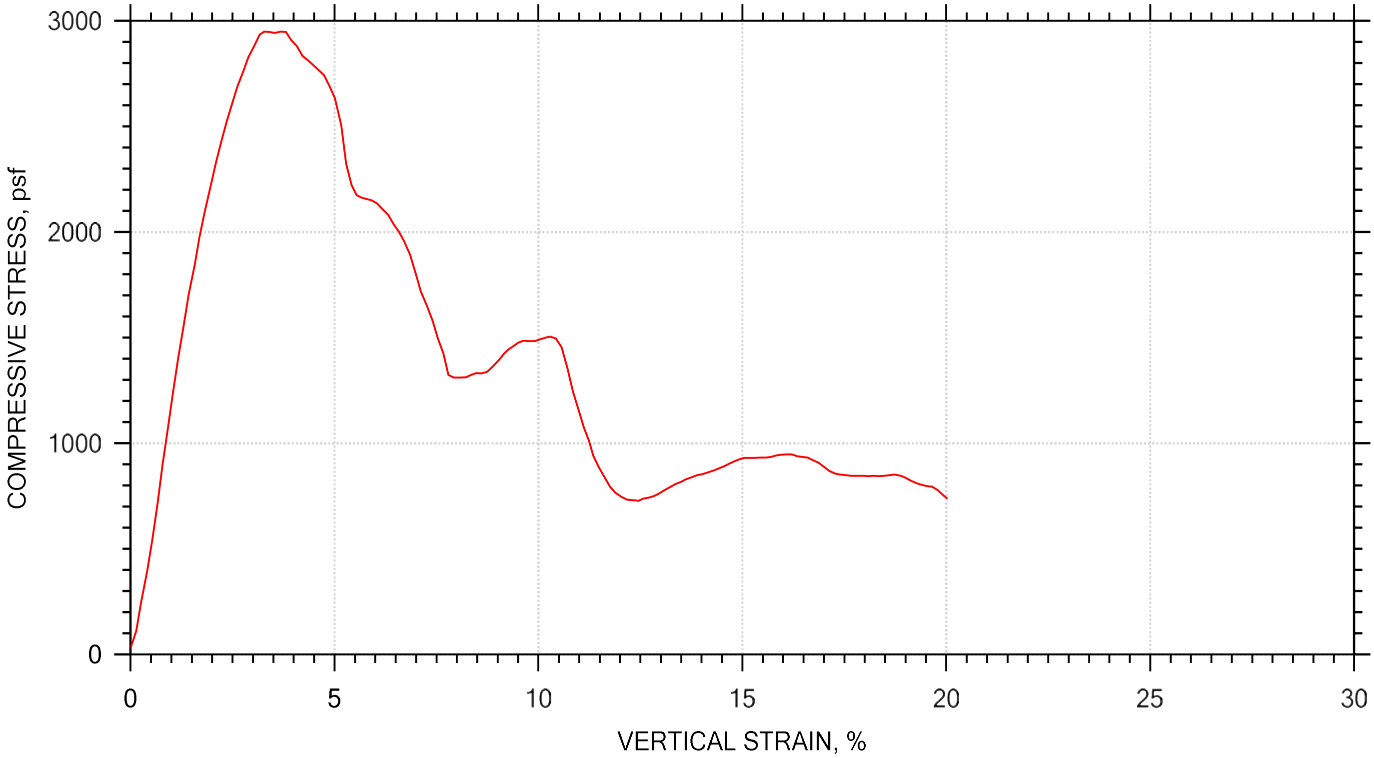
Project: GSD Water	Location: Garberville	Project No.: 022067.400
Boring No.: B-1-LH	Tested By: KEW	Checked By: KEW
Sample No.: 9	Test Date: 6/29/23	Elevation:
Test No.: 23-586	Preparation: Undisturbed	Depth: 16-16.5'
Description:		
Remarks:		

UNCONFINED COMPRESSION TEST REPORT



SHN	Project: GSD Water	Location: Garberville	Project No.: 022067.400
	Boring No.: B-1-LH	Tested By: KEW	Checked By: KEW
	Sample No.: 9	Test Date: 6/29/23	Elevation:
	Test No.: 23-586	Preparation: Undisturbed	Depth: 16-16.5'
	Description:		
	Remarks:		

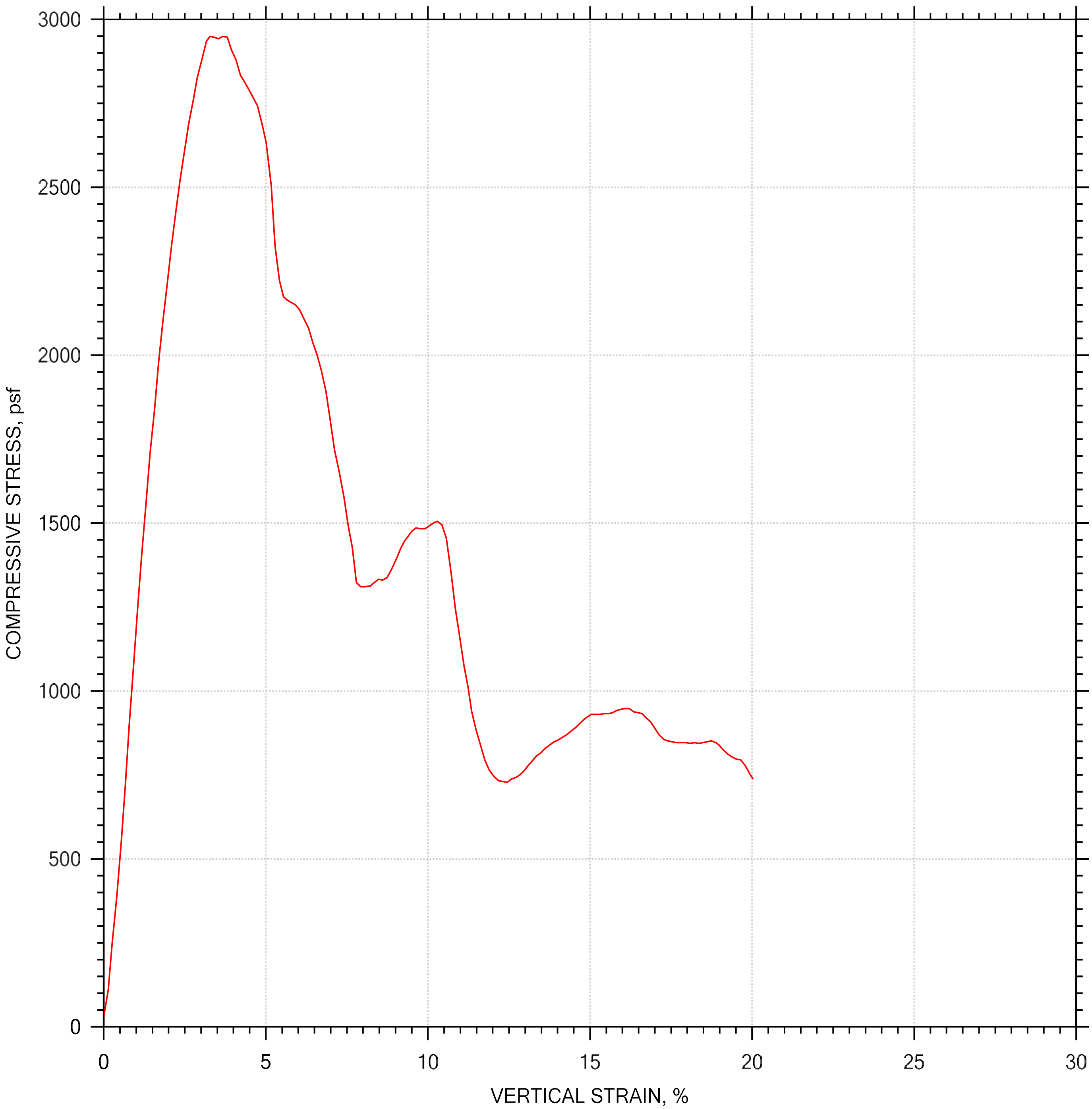
UNCONFINED COMPRESSION TEST REPORT



Symbol					
Test No.		23-595			
Initial	Diameter, in	2.4			
	Height, in	4.45			
	Water Content, %	19.27			
	Dry Density, pcf	99.48			
	Saturation, %	77.02			
	Void Ratio	0.663			
Unconfined Compressive Strength, psf		2950			
Undrained Shear Strength, psf		1475			
Time to Failure, min		2.8042			
Strain Rate, %/min		0.01			
Estimated Specific Gravity		2.65			
Liquid Limit		---			
Plastic Limit		---			
Plasticity Index		---			
Failure Sketch					

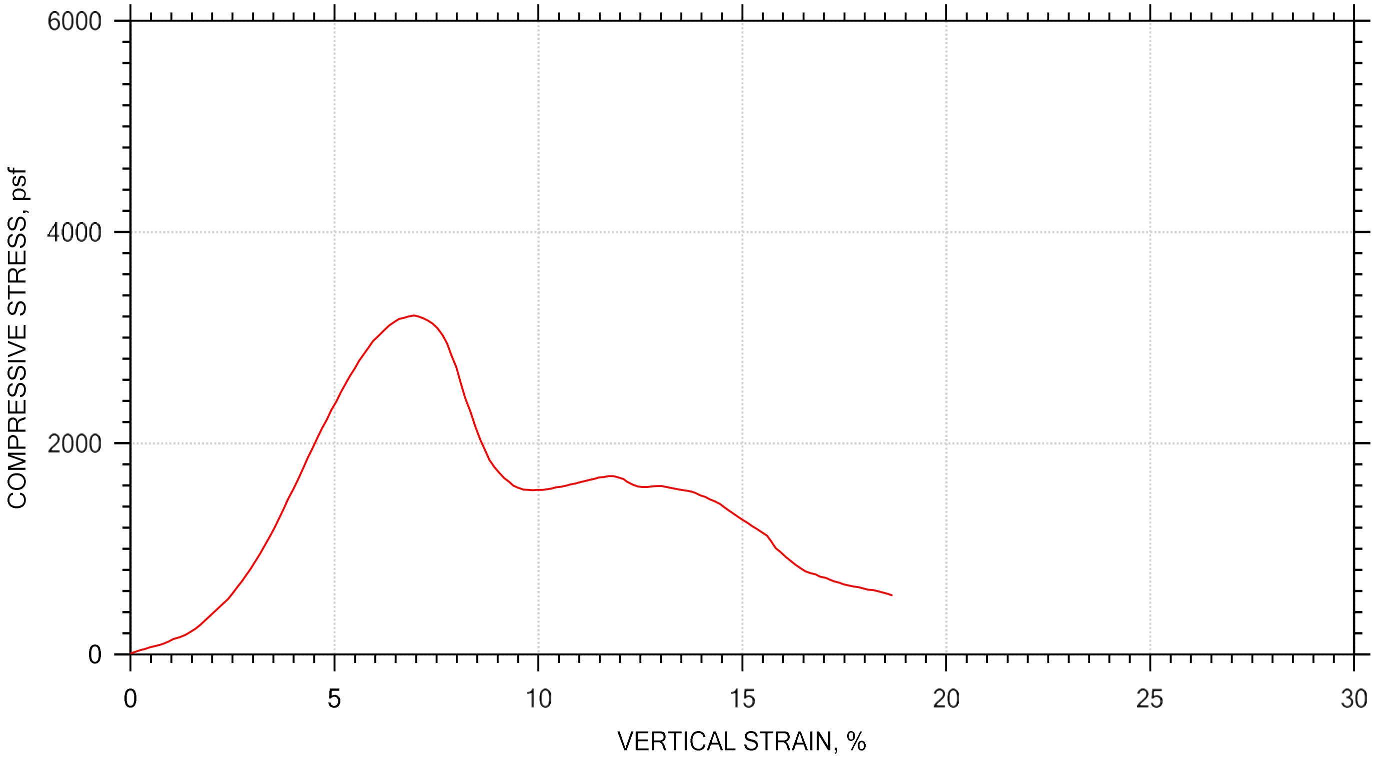
Project: GSD Water	Location: Garberville	Project No.: 022067.4
Boring No.: B-2-LH	Tested By: KEW	Checked By: KEW
Sample No.: 18	Test Date: 6/29/23	Elevation:
Test No.: 23-595	Preparation: Undisturbed	Depth: 4-4.5'
Description:		
Remarks:		

UNCONFINED COMPRESSION TEST REPORT



SHN	Project: GSD Water	Location: Garberville	Project No.: 022067.4
	Boring No.: B-2-LH	Tested By: KEW	Checked By: KEW
	Sample No.: 18	Test Date: 6/29/23	Elevation:
	Test No.: 23-595	Preparation: Undisturbed	Depth: 4-4.5'
	Description:		
	Remarks:		

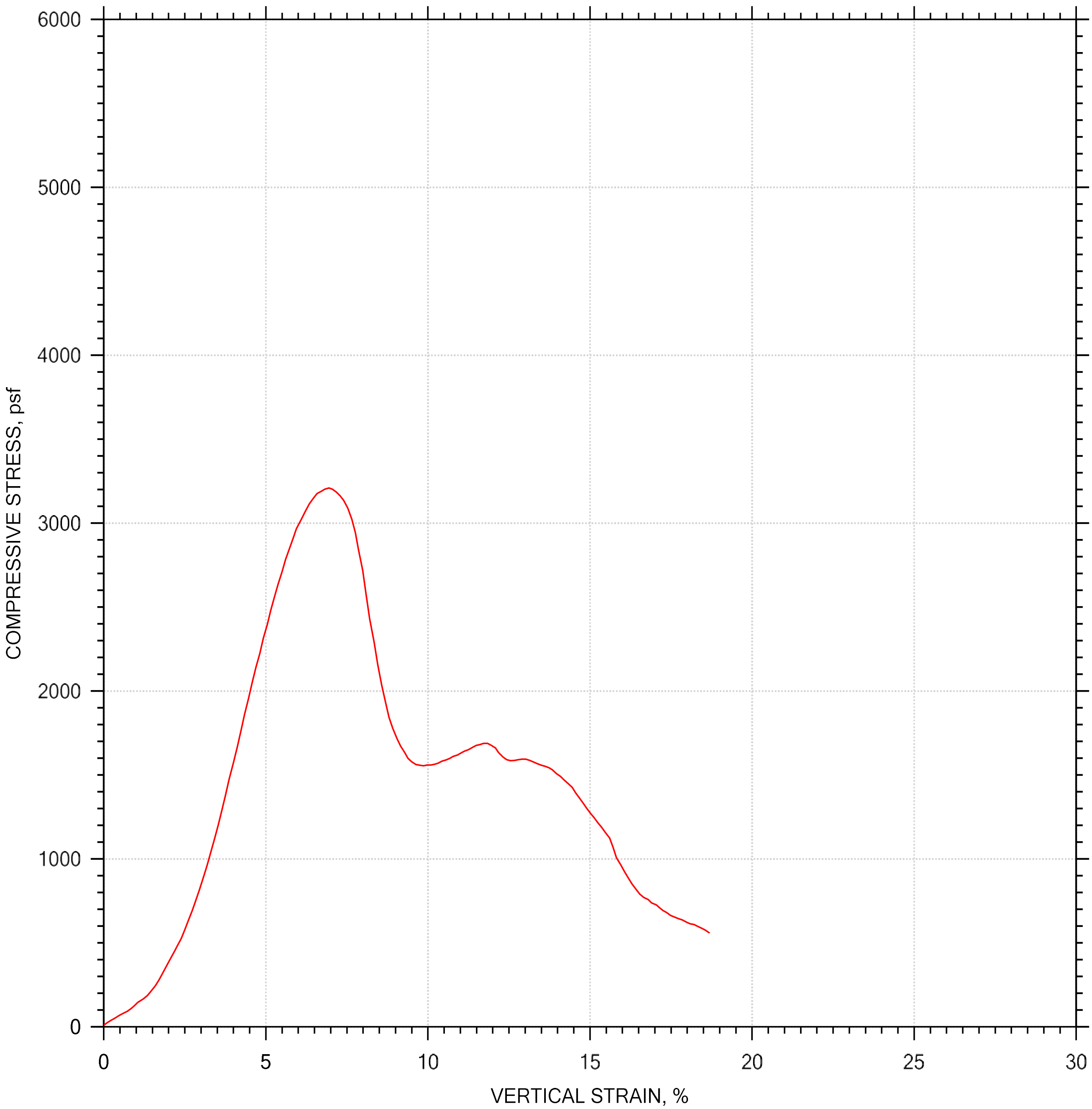
UNCONFINED COMPRESSION TEST REPORT



Symbol					
Test No.		23-609			
Initial	Diameter, in	2.4			
	Height, in	5			
	Water Content, %	22.53			
	Dry Density, pcf	103.2			
	Saturation, %	98.90			
	Void Ratio	0.604			
Unconfined Compressive Strength, psf		3209			
Undrained Shear Strength, psf		1605			
Time to Failure, min		5.9012			
Strain Rate, %/min		0.01			
Estimated Specific Gravity		2.65			
Liquid Limit		---			
Plastic Limit		---			
Plasticity Index		---			
Failure Sketch					

Project: GSD Water	Location: Garberville	Project No.: 022067.4
Boring No.: B-3-LH	Tested By: KEW	Checked By: KEW
Sample No.: 32	Test Date: 6/29/23	Elevation:
Test No.: 23-609	Preparation: Undisturbed	Depth: 11-11.5'
Description:		
Remarks:		

UNCONFINED COMPRESSION TEST REPORT



SHN	Project: GSD Water	Location: Garberville	Project No.: 022067.4
	Boring No.: B-3-LH	Tested By: KEW	Checked By: KEW
	Sample No.: 32	Test Date: 6/29/23	Elevation:
	Test No.: 23-609	Preparation: Undisturbed	Depth: 11-11.5'
	Description:		
	Remarks:		

Phone: (707) 441-8855 Email: info@shn-engr.com Web: shn-engr.com
812 W. Wabash Avenue, Eureka, CA 95501-2138

DENSITY BY DRIVE- CYLINDER METHOD (ASTM D2937)

Project Name:	**GSD Wallan Tank**	**Project Number:**	**022067.400**
Performed By:	**JMA**	**Date:**	**7/12/2023**
Checked By:	**KEW**	**Date:**	**7/18/2023**
Project Manager:	**JOB**		

Lab Sample Number	23-653	23-656	23-664		
Boring Label	B-1-W	B-1-W	B-2-W		
Sample Depth (ft)	2-2.5'	6-6.5'	4-4.5'		
Diameter of Cylinder, in	2.42	2.42	2.42		
Total Length of Cylinder, in.	6.00	6.00	6.00		
Length of Empty Cylinder A, in.	0.00	0.00	0.82		
Length of Empty Cylinder B, in.	0.25	1.72	0.53		
Length of Cylinder Filled, in	5.75	4.28	4.65		
Volume of Sample, in^3	26.45	19.69	21.39		
Volume of Sample, cc.	433.40	322.60	350.49		

Pan #	ss7	ss5	ss10		
Weight of Wet Soil and Pan	1211.3	850.9	924.7		
Weight of Dry Soil and Pan	1027.9	744.2	853.6		
Weight of Water	183.4	106.7	71.1		
Weight of Pan	192.9	195.3	195.3		
Weight of Dry Soil	835.0	548.9	658.3		
Percent Moisture	22.0	19.4	10.8		
Dry Density, g/cc	1.93	1.70	1.88		
Dry Density, lb/ft^3	120.3	106.2	117.3		

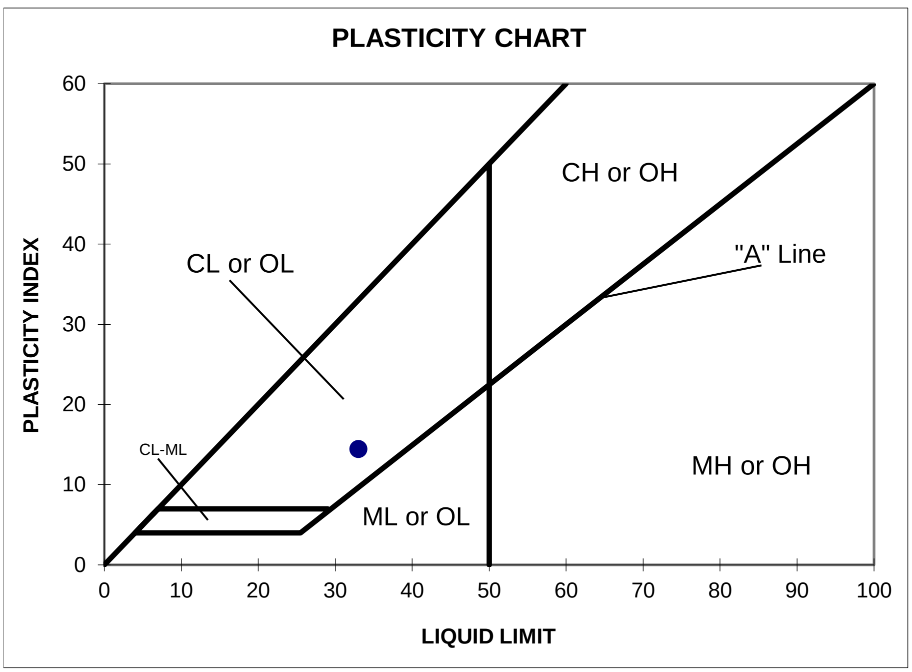
ENGINEERS & GEOLOGISTS, INC.
812 W. Wabash Eureka, CA 95501-2138 Tel: 707/441-8855 FAX: 707/441-8877 E-mail: shninfo@shn-engr.com

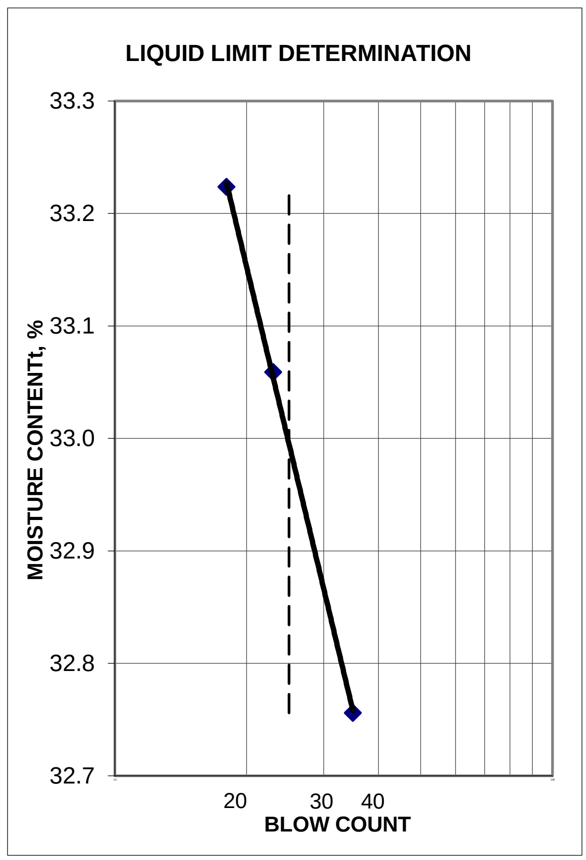
LIQUID LIMIT, PLASTIC LIMIT, and PLASTICITY INDEX (ASTM-D4318)

JOB NAME:	GSD Wallan Tank	JOB #:	022067.400	LAB SAMPLE #:	23-654
SAMPLE ID:	B-1-W @ 3-4.5'	PERFORMED BY:	JMA/SC	DATE:	7/14/2023
PROJECT MANAGER:	JOB	CHECKED BY:	KEW	DATE:	7/18/2023

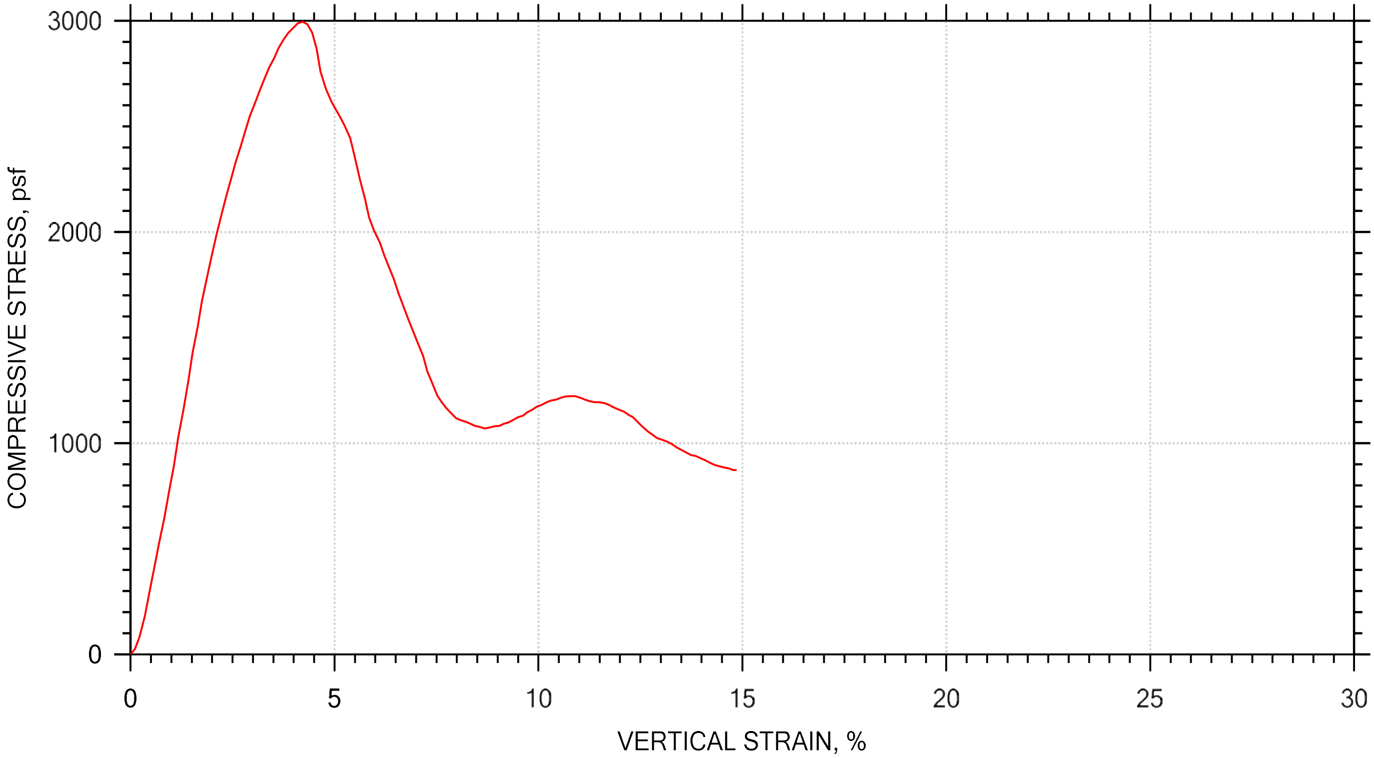
LINE NO.		TRIAL NO. 1	TRIAL NO. 2	TRIAL NO. 1	TRIAL NO. 2	TRIAL NO. 3
A	PAN #	1	2	3	13	14
B	PAN WT. (g)	29.560	28.920	28.970	21.970	20.140
C	WT. WET SOIL & PAN (g)	35.760	35.110	37.400	31.670	28.240
D	WT. DRY SOIL & PAN (g)	34.800	34.130	35.320	29.260	26.220
E	WT. WATER (C-D)	0.960	0.980	2.080	2.410	2.020
F	WT. DRY SOIL (D-B)	5.240	5.210	6.350	7.290	6.080
G	BLOW COUNT	--	--	35	23	18
H	MOISTURE CONTENT (E/F*100)	18.3	18.8	32.8	33.1	33.2

LIQUID LIMIT	PLASTIC INDEX	PLASTIC LIMIT
33	**14**	19





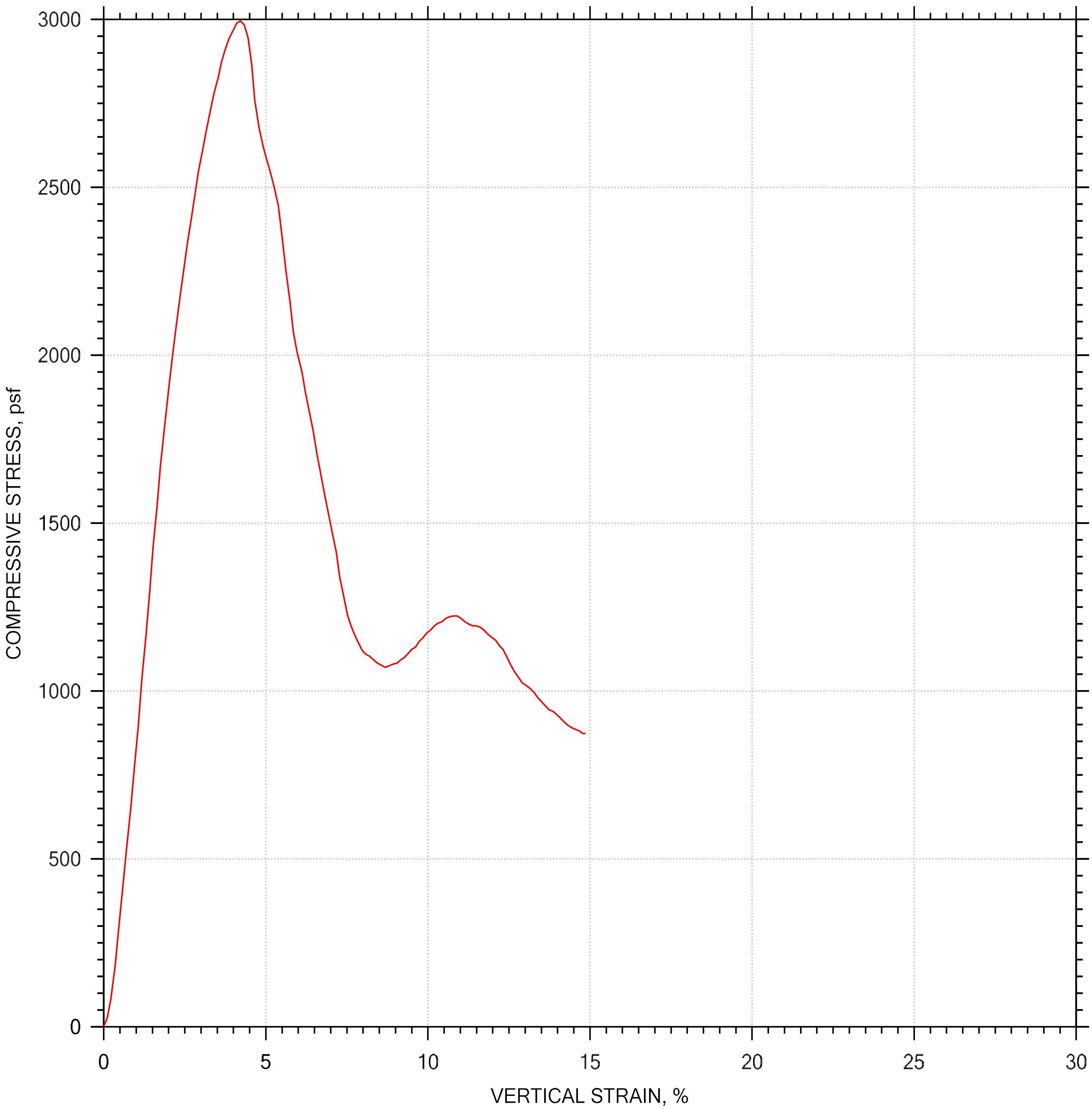
UNCONFINED COMPRESSION TEST REPORT



Symbol					
Test No.		23-662			
Initial	Diameter, in	2.42			
	Height, in	5.41			
	Water Content, %	16.44			
	Dry Density, pcf	103.9			
	Saturation, %	73.61			
	Void Ratio	0.592			
Unconfined Compressive Strength, psf		2996			
Undrained Shear Strength, psf		1498			
Time to Failure, min		3.6015			
Strain Rate, %/min		0.01			
Estimated Specific Gravity		2.65			
Liquid Limit		---			
Plastic Limit		---			
Plasticity Index		---			
Failure Sketch					

	Project: GSD Wallan Tank	Location: Garberville	Project No.: '022067.400
	Boring No.: B-2-W	Tested By: JMA	Checked By: KEW
	Sample No.: 2	Test Date: 7/14/23	Elevation:
	Test No.: 23-662	Preparation: Undisturbed	Depth: 2-2.5'
	Description: Medium stiff reddish brown SILT		
	Remarks:		

UNCONFINED COMPRESSION TEST REPORT



Project: GSD Wallan Tank	Location: Garberville	Project No.: '022067.400
Boring No.: B-2-W	Tested By: JMA	Checked By: KEW
Sample No.: 2	Test Date: 7/14/23	Elevation:
Test No.: 23-662	Preparation: Undisturbed	Depth: 2-2.5'
Description: Medium stiff reddish brown SILT		
Remarks:		

Corrosion Test Results

3

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 462 2771 Fax. 925 462 2775
www.cercoanalytical.com

19 July 2023

Job No. 2307017
Cust. No. 11258

Ms. Alyssa Troia
SHN Consulting Engineers and Geologists
812 W. Wabash Avenue
Eureka, CA 95501

Subject: Project No.: 022067.400
Project Name: Lower Hurlbutt, GSD Water Improvements Project, Lower Hurlbutt Site
Corrosivity Analysis – ASTM Test Methods with Brief Evaluation

Dear Ms. Troia:

Pursuant to your request, CERCO Analytical has analyzed the soil sample submitted on July 12, 2023. Based on the analytical results, this brief corrosivity evaluation is enclosed for your consideration.

Based upon the resistivity measurement, this composite sample is classified as "mildly corrosive". All buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron should be properly protected against corrosion depending upon the critical nature of the structure. All buried metallic pressure piping such as ductile iron firewater pipelines should be protected against corrosion.

The chloride ion concentration reflects none detected with a reporting limit of 15 mg/kg.

The sulfate ion concentration reflects none detected with a reporting limit of 15 mg/kg.

The pH of the soil is 6.72 which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

The redox potential is 340-mV and is indicative of potentially "slightly corrosive" soils resulting from anaerobic soil conditions.

This corrosivity evaluation is based on general corrosion engineering standards and is non-specific in nature. For specific long-term corrosion control design recommendations or consultation, please call *JDH Corrosion Consultants, Inc. at (925) 927-6630.*

We appreciate the opportunity of working with you on this project. If you have any questions, or if you require further information, please do not hesitate to contact us.

Very truly yours,
CERCO ANALYTICAL, INC.

J. Darby Howard, Jr., P.E.
President

JDH/jdl
Enclosure

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 462 2771 Fax. 925 462 2775
www.cercoanalytical.com

Client: SHN Consulting Engineers & Geologists
Client's Project No.: 022067.400
Client's Project Name: Lower Hurlbutt - GSD Water Improvements Project, Lower Hurlbutt Site
Date Sampled: 8-Jun-23
Date Received: 12-Jul-23
Matrix: Soil
Authorization: Signed Chain of Custody

Date of Report: 19-Jul-2023

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Conductivity (umhos/cm)*	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
2307017-001AB	B-2-LH @ 15-16.5' and B-1-LH @ 25.5-26'	340	6.72	-	8,800	-	N.D.	N.D.

	Redox	pH	Conductivity	Resistivity	Sulfide	Chloride	Sulfate
Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Reporting Limit:	-	-	10	-	50	15	15
Date Analyzed:	12-Jul-2023	13-Jul-2023	-	12-Jul-2023	-	18-Jul-2023	18-Jul-2023

* Results Reported on "As Received" Basis

N.D. - None Detected

Cheryl McMillen

Cheri McMillen
Chemist

<u>Quality Control Summary</u> - All laboratory quality control parameters were found to be within established limits

2307017

Chain of Custody

GSD Water Improvements Project, Lower Hurlbutt Site.

Page 1 of 1

1100 Willow Pass Court
Concord, CA 94520-1006
925 462 2771
Fax: 925 462 2775

CERCO analytical

Job No.	CU#		Client Project I.D.	Schedule		Date Sampled	Date Due
022067.400	11758		Lower Hurlbutt	Analyte			7/28/23

Full Name Alyssa Troia

Phone 707 441 8855

Fax

Company and/or Mailing Address SHN - 812 W Wabash Ave Eureka

Cell 5306134819 ☒

Sample Source SOIL

ANALYSIS / ASTM

Lab No.	Sample I.D.	Date	Time	Matrix	Contain.	Size	Preserv.	Qty.	Redox Potential	pH	Sulfate	Chloride	Resistivity-100% Saturated			Brief Evaluation						
001AB	B-2-LH 15-16.5' / B-1-LH 25.5-26' → composite (please combine)	6/8/23		S	PL			1	X	X	X	X	X			X						

MATRIX: DW - Drinking Water, GW - Ground Water, SW - Surface Water, WW - Waste Water, Water, SL - Sludge, S - Soil, Product

ABBREVIATIONS: HB - Hosebib, PV - Petcock Valve, PT - Pressure Tank, PH - Pump House, RR - Restroom, GL - Glass, PL - Plastic, ST - Sterile

SAMPLE RECEIPT

Total No. of Containers	
Rec'd Good Cond/Cold	
Conforms to Record	
Temp. a t Lab -°C	
Sampler	

	Date	Time
Relinquished By: Alyssa Troia	7/10/2023	12:00PM
Received By: Nicole Driep	7/12/23	12:14PM
Relinquished By:		
Received By:		
Relinquished By:		
Received By:		

Comments:

THERE IS AN ADDITIONAL CHARGE FOR EXTRUDING SOIL FROM METAL TUBES

Email Address: atroia@SHN-ENGR.com

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 462 2771 Fax. 925 462 2775
www.cercoanalytical.com

Attention: Accounts Payable
SHN Consulting Engineers & Geologists
812 W. Wabash Avenue
Eureka, CA 95501

sholler@shn-engr.com
kpryor@shn-engr.com

Project No.: 022067.400
Project Name: Lower Hurlbutt, GSD Water Improvements
Project, Lower Hurlbutt Site
Date Sampled: 06/08/23
Date Received: 07012/23
Matrix: Soil
Authorization: Signed Chain of Custody

19 July 2023
Invoice/Job No. 2307017
Sample No. 001AB
Cust. No. 11258

INVOICE FOR ANALYTICAL SERVICES

Analyte	Amount
Corrosivity Analysis – ASTM Test Methods with Brief Evaluation* One (1) Sample @ $270.00/Sample	$270.00
Composite Charge Two (2) Samples @ $20.00/Sample	$40.00
Disposal Charge One (1) Sample @ $10.00/Sample	$10.00
TOTAL AMOUNT DUE THIS INVOICE	**$320.00**

* Includes Redox, pH, sulfate, resistivity (100% saturation), and chloride

Invoices are due and payable within 30 days from receipt. All overdue accounts are subject to a 1.5% interest charge per month.

REMINDER
FOR IMMEDIATE PROCESSING OF YOUR REMITTANCE PLEASE INCLUDE THE ABOVE REFERENCED JOB NUMBER ON YOUR CHECK

cc: Ms. Alyssa Troia

Eureka, CA | Arcata, CA | Redding, CA | Willits, CA | Fort Bragg, CA | Coos Bay, OR | Klamath Falls, OR