GEOTECHNICAL INVESTIGATION
COPIA REDEVELOPMENT
501 1st STREET
NAPA, CALIFORNIA

May XX, 2021

Job No. 2489.002

Prepared For:
Harvest Properties
Attn: Colin Mitchell
180 Grand Avenue
Oakland, California 94612

CERTIFICATION

This document is an instrument of service, prepared by or under the direction of the undersigned professionals, in accordance with the current ordinary standard of care. The service specifically excludes the investigation of polychlorinated byphenols, radon, asbestos or any other hazardous materials. The document is for the sole use of the client and consultants on this project. No other use is authorized. If the project changes, or more than two years have passed since issuance of this report, the findings and recommendations must be updated.

MILLER PACIFIC ENGINEERING GROUP
(a California corporation)

REVIEWED BY

Rusty Arend
Geotechnical Engineer No. 3031
(Expires 6/30/21)

Mike Morisoli
Geotechnical Engineer No. 2541
(Expires 12/31/22)
# TABLE OF CONTENTS

1.0 INTRODUCTION.................................................................................................................. 1
2.0 PROJECT DESCRIPTION........................................................................................................... 1
3.0 SITE CONDITIONS.................................................................................................................. 2
   3.1 Regional Geology .............................................................................................................. 2
   3.2 Seismicity ........................................................................................................................ 3
       3.2.1 Regional Active Faults ......................................................................................... 3
       3.2.2 Historic Fault Activity ....................................................................................... 3
       3.2.3 Probability of Future Earthquakes ....................................................................... 3
   3.3 Surface Conditions .......................................................................................................... 4
   3.4 Field Exploration and Laboratory Testing ...................................................................... 5
   3.5 Geotechnical Reference Data ........................................................................................ 5
   3.6 Subsurface Conditions and Groundwater ...................................................................... 5
4.0 GEOLOGIC HAZARDS ....................................................................................................... 6
   4.1 Fault Surface Rupture ..................................................................................................... 6
   4.2 Seismic Shaking ............................................................................................................... 7
   4.3 Liquefaction and Related Effects .................................................................................. 8
       4.3.1 Liquefaction Evaluation .................................................................................... 8
       4.3.2 Estimated Post-Liquefaction Settlement ......................................................... 9
       4.3.3 Lateral Spreading ............................................................................................. 9
   4.4 Seismic Densification ...................................................................................................... 11
   4.5 Expansive Soil ................................................................................................................ 11
   4.6 Settlement ........................................................................................................................ 11
   4.7 Slope Instability/Landslides ........................................................................................ 12
   4.8 Erosion ............................................................................................................................ 12
   4.9 Tsunami/Seiche .............................................................................................................. 13
   4.10 Flooding ........................................................................................................................ 13
5.0 CONCLUSIONS AND RECOMMENDATIONS ................................................................... 13
   5.1 Seismic Design .............................................................................................................. 13
   5.2 Ground Improvement ..................................................................................................... 14
   5.3 Site Grading ................................................................................................................... 16
       5.3.1 Site Preparation ............................................................................................... 16
       5.3.2 Excavations ...................................................................................................... 16
       5.3.3 Fill Materials, Placement and Compaction .................................................... 17
   5.4 Foundation Design ......................................................................................................... 17
   5.5 Interior Concrete Slabs .................................................................................................. 18
   5.6 Exterior Concrete Slabs .................................................................................................. 19
   5.7 Site and Foundation Drainage ....................................................................................... 19
   5.8 Underground Utilities .................................................................................................. 19
   5.9 Pavements ...................................................................................................................... 20
6.0 SUPPLEMENTAL GEOTECHNICAL SERVICES .............................................................. 20
7.0 LIMITATIONS .................................................................................................................. 21
8.0 LIST OF REFERENCES ........................................................................................................ 21
FIGURE 1 SITE LOCATION MAP
FIGURE 2 SITE PLAN
FIGURE 3 REGIONAL GEOLOGIC MAP
FIGURE 4 ACTIVE FAULT MAP
FIGURE 5 HISTORIC EARTHQUAKE MAP
FIGURE 6 LIQUEFACTION SUSCEPTIBILITY MAP
FIGURE 7 FEMA FLOOD INSURANCE RATE MAP
FIGURE 8 TUNNELMANS GROUND CLASSIFICATION FOR SOIL

TABLE 1 DETERMINISTIC PEAK GROUND ACCELERATIONS FOR ACTIVE FAULTS
TABLE 2 PROBABILISTIC PEAK GROUND ACCELERATIONS FOR ACTIVE FAULTS
TABLE 3 RESULTS OF LIQUEFACTION AND LATERAL SPREADING ANALYSES
TABLE 4 2019 CALIFORNIA BUILDING CODE SEISMIC DESIGN CRITERIA
TABLE 5 MINIMUM PERFORMANCE REQUIREMENTS FOR GROUND IMPROVEMENT
TABLE 6 SHALLOW FOUNDATION DESIGN CRITERIA
TABLE 7 PRELIMINARY ASPHALT-CONCRETE PAVEMENT SECTIONS

APPENDIX A: SUBSURFACE EXPLORATION AND LABORATORY TESTING
APPENDIX B: GEOTECHNICAL REFERENCE DATA
1.0 INTRODUCTION

This report presents the results of our Geotechnical Investigation for the proposed Copia development at 501 1st Street in Napa, California. As shown on the Site Location Map, the site is located within the Oxbow area of downtown Napa. Our work was performed in accordance with our Agreement for Professional Services authorized on March 24, 2021. The purpose of our investigation was to explore subsurface conditions within the proposed project area and to develop geotechnical recommendations and criteria for use in design and construction of the project. The scope of our services includes:

1. Reviewing published geologic and geotechnical background information, including data from our preliminary investigations which were completed in 2012 and 2016 for a previously proposed development at the same site.

2. Supplemental subsurface exploration consisting of 11 cone penetration tests located within the general vicinity of the planned improvements.

3. Evaluating relevant geologic hazards including seismic shaking, liquefaction and lateral spreading, expansive soils, seismic induced ground settlement, flooding and other hazards.

4. Engineering analyses to develop geotechnical recommendations and design criteria related to building foundations, site grading, seismic design and other geotechnical-related items.

5. Preparation of this Geotechnical Investigation report which summarizes the subsurface exploration and laboratory testing programs, evaluation of relevant geologic hazards, and geotechnical recommendations and design criteria.

Issuance of this report completes our Phase 1 and 2 services outlined in our Agreement. Subsequent phases of work should include supplemental geotechnical consultation and plan review and observation and testing of geotechnical-related work items during construction.

2.0 PROJECT DESCRIPTION

Based on our review of preliminary drawings, the proposed development includes constructing approximately 55 townhomes and three buildings for commercial and residential use. The townhomes will be three-story structures configured in “clusters” along the waterfront and central portion of the site. The five-story commercial/residential buildings are planned near the south and
north end of the site and will provide ground level retail and residential amenity space and four floors of residential flats above. A fourth building is planned near the northwest end of the site and its intended use is currently not specified. The buildings will be constructed near existing grades and no underground parking is anticipated at this time. While building types are not known, we anticipate the new townhome structures will be wood framed while the commercial/residential buildings may utilize reinforced concrete or metal framing.

Ground improvement is anticipated beneath the new structures to reduce risks associated with liquefaction and lateral spreading and will likely consist of a combination of vibro-replacement stone columns and deep soil mixing. Ancillary improvements will likely include new pedestrian trails and communal areas, landscaping and exterior hardscape, underground utilities, pavements for new roadways and parking areas, site drainage and other improvements typical of such developments. Site grading is expected to include cuts and fills of a few feet as required to reduce flood risk and achieve the design grades for the new roadways, parking areas, and building pads. The proposed buildings and other new improvements are shown on the Site Plan, Figure 2.

3.0 SITE CONDITIONS

3.1 Regional Geology

The site is located within the Coast Ranges Geomorphic Province of California. Topographically, the Province is characterized by northwest-southeast trending mountain ranges of moderate relief, with intervening deep canyons, or narrow stream valleys. The province is known for its active seismicity, high rainfall, and susceptibility to erosion and landslide development in steep terrain. Within the Province there are occasional larger, alluvium-filled, basin-shaped valleys. These include the Santa Rosa Plain and Sonoma, Bennett, Napa and Knights Valleys. Most of these valleys are associated with known or suspected active faults and have formed in part by past fault displacement and crustal folding.

The Franciscan Complex is the baserock of the Province and it consists of a diverse assemblage of rock units, including sandstone, shale, greenstone (altered, submarine volcanic rocks), chert, and lesser amounts of conglomerate and metamorphic rocks. The Franciscan was deposited during the Jurassic-Cretaceous Period (65-190 million years ago) (Huffman & Armstrong, 1980). Of these rock types, the most prevalent is graywacke sandstone, which is massively bedded and has occasional shale interbeds. Masses of serpentinite of various dimensions are locally present. The serpentinite has been intruded and faulted into the Complex during long and ongoing tectonic processes. Overlying much of the Franciscan rock in Napa County are the Sonoma Volcanics. This rock sequence is the result of volcanism in the Pliocene Epoch (1.6 to 5 million years ago) that extends from Mt. St. Helena in the north to Vallejo in the south (Wagner & Bortugno, 1982).

The project site is located on relatively level to gently sloping terrain in the southeastern portion of the “oxbow” of the Napa River. Regional geologic mapping by the California Geological Survey indicates the site is underlain by Holocene-age alluvial deposits (CGS, 2002). The alluvium is generally characterized as stream terraces deposited as point bar and overbank deposits
primarily composed of moderately sorted clayey sand and sandy clay with gravel. A Regional Geologic Map and descriptions of the various geologic units are presented on Figure 3.

3.2 Seismicity
The project site is located within the seismically active San Francisco Bay Area and will therefore experience the effects of future earthquakes. Earthquakes are the product of the build-up and sudden release of strain along a “fault” or zone of weakness in the earth's crust. Stored energy may be released as soon as it is generated or it may be accumulated and stored for long periods of time. Individual releases may be so small that they are detected only by sensitive instruments, or they may be violent enough to cause destruction over vast areas.

Faults are seldom single cracks in the earth's crust but are typically comprised of localized shear zones which link together to form larger fault zones. Within the Bay Area, faults are concentrated along the San Andreas Fault zone. The movement between rock formations along either side of a fault may be horizontal, vertical, or a combination and is radiated outward in the form of energy waves. The amplitude and frequency of earthquake ground motions partially depends on the material through which it is moving. The earthquake force is transmitted through hard rock in short, rapid vibrations, while this energy becomes a long, high-amplitude motion when moving through soft ground materials, such as Bay Mud.

3.2.1 Regional Active Faults
The California Geological Survey (previously known as the California Division of Mines and Geology), defines a “Holocene-active fault” as one that had surface displacement within Holocene time (the last 11,700 years). CGS mapped various faults in the region as part of their Fault Activity Map of California (CGS, 2010). Many of these faults are shown in relation to the project site on the attached Active Fault Map, Figure 4. The nearest known Holocene-active faults are the West Napa and Green Valley Faults which are located approximately 2.7 kilometers (1.7 miles) west and 8.5 kilometers (5.6 miles) east of the site, respectively.

3.2.2 Historic Fault Activity
Numerous earthquakes have occurred in the region within historic times. The results of our USGS earthquake search catalogue indicates that at least ten earthquakes with a Richter Magnitude of 5.0 or larger have occurred within 100 kilometers (62 miles) of the site between 1900 and 2021. The approximate locations of earthquakes which occurred between 1985 and 2014 are shown on the Historic Earthquake Map, Figure 5.

3.2.3 Probability of Future Earthquakes
The site will likely experience moderate to strong ground shaking from future earthquakes originating on any of several active faults in the San Francisco Bay region. The historical records do not directly indicate either the maximum credible earthquake or the probability

---

1 Distances to faults estimated using USGS fault overlays on Google, accessed May 18, 2021.
of such a future event. To evaluate earthquake probabilities in California, the USGS has assembled a group of researchers into the “Working Group on California Earthquake Probabilities” (USGS 2003, 2008, 2013) to estimate the probabilities of earthquakes on active faults. These studies have been published cooperatively by the USGS, CGS, and Southern California Earthquake Center (SCEC) as the Uniform California Earthquake Rupture Forecast, Versions 1, 2, and 3. In these studies, potential seismic sources were analyzed considering fault geometry, geologic slip rates, geodetic strain rates, historic activity, micro-seismicity, and other factors to arrive at estimates of earthquakes of various magnitudes on a variety of faults in California.

Conclusions from the most recent UCERF3 and USGS indicate the highest probability of an earthquake with a magnitude greater than 6.7 originating on any of the active faults in the San Francisco Bay region by 2043 is assigned to the Hayward/Rodgers Creek Fault system. The Rodgers Creek Fault is located approximately 20.6 kilometers (12.8 miles) southwest of the site and is assigned a probability of 33 percent. The San Andreas Fault, located approximately 55.3 kilometers (33.1 miles) southwest of the site, is assigned a 22 percent probability of an earthquake with a magnitude greater than 6.7 by 2043. Additional studies by the USGS regarding the probability of large earthquakes in the Bay Area are ongoing. These current evaluations include data from additional active faults and updated geological data.

3.3 Surface Conditions

The project area is within a river "oxbow" that is bounded to the north by 1st Street, Vernon Street and Water Street, to the west by the Napa Valley Wine Train alignment, and to the east and south by the Napa River. The riverbank slopes generally slope at inclinations ranging from about 0.5:1 to 2:1 (horizontal:vertical). A relatively small erosion failure is evident along the creekbank at the northermmost point of the oxbow, and this “failure” damaged the existing asphalt-paved pedestrian pathway along the northern side of the existing Copia Building. From 2012 discussions with Art Ferretti, Copia Facility Manager, this erosion occurred during the New Year 2006 storm (flood) event. Portions of the creekbank surrounding the site are protected with rock rip-rap, including the sharp bend at the western end of the existing Copia Building.

The relatively level ground surface in the proposed redevelopment area is improved with asphalt-paved parking lots and driveways, landscape/garden areas, an outdoor kitchen and a water feature that begins near the existing Copia Building to the north and extends beyond First Street to the south and into the project site. First Street, a two-lane paved roadway, is located along the northwest side of the project area and this roadway crosses the Napa River on a multi-span concrete bridge that was reconstructed in 2006. North of the project area (the northern half) of the oxbow area, development includes asphalt-paved parking lots and the two-story Copia building. Development immediately west of the project area includes a collection of food/retail shops and single-family homes. The existing two-story Copia building and associated parking lots and landscaping north of First Street are not included in the planned redevelopment work.
3.4 Field Exploration and Laboratory Testing

We explored subsurface conditions in 2012 and 2016 with three cone penetration tests (CPTs) and four borings as part of our field explorations for previously proposed developments. We completed an additional 11 CPTs on April 8 and 9, 2021 to provide supplemental subsurface data for the currently proposed development. The approximate locations of the borings and CPTs are shown on the Site Plan, Figure 2. The borings were drilled to depths of about 45.5 to 48.5 feet below ground surface whereas the CPTs were advanced to depths of about 50 feet with the exception of CPT-13-21. This CPT encountered refusal at about four feet despite several attempts to relocate and advance the CPT to the target depth. The soil borings were excavated using truck-mounted drilling equipment. The borings were logged by our Field Geologist and samples were obtained for classification and laboratory testing. We prepared boring logs based on soil descriptions in the field as well as visual examination and testing of the soil samples in our laboratory. The boring logs are presented in Appendix A.

Cone penetration testing (CPT) is an exploration technique that provides a continuous profile of data throughout the depth of exploration. CPTs are particularly useful in defining stratigraphy, relative soil strength and in assessing liquefaction potential. This test method has been standardized and is described in detail by the ASTM D3441. The method generally consists of pushing an instrumented cone-tipped probe through the ground at a controlled rate and measuring tip resistance (at the front of the cone) and frictional resistance (along the sides of the cone). The data is processed by a computer to approximate engineering properties such as soil type, relative density, and strength. A CPT Soil Interpretation Chart and CPT plots of interpreted subsurface conditions are shown in Appendix A.

Laboratory testing of soil samples from the exploratory borings included determination of moisture content, dry density and the percentage of particles passing a No. 200 sieve. The results of our laboratory tests are presented on the boring logs. Our laboratory testing program is discussed in greater detail in Appendix A.

3.5 Geotechnical Reference Data

Previous field investigations were performed by the US Army Corp of Engineers and Fugro as part of previous flood control improvements and site development. The locations of three of these reference borings which were located near the proposed development are shown on Figure 2. Laboratory test data available from these reference borings includes gradation, Atterberg Limits, strength and other data. The boring logs and laboratory testing from these previous investigations are included in Appendix B.

3.6 Subsurface Conditions and Groundwater

Our exploratory borings, CPTs and reference borings by others generally confirm the regional geologic mapping. Within the existing Copia parking lots on the south side of First Street, the asphalt section consisted of about three inches of asphalt concrete over about 12 inches of aggregate baserock. Six inches of topsoil was encountered in the landscape/garden area at the location of our Boring B-3-16. About 2.5 to 4.5 feet of medium stiff to very stiff, sandy, gravelly
clay fill was encountered beneath the baserock or topsoil layers. Below the fill, alluvial deposits, also consisting of mixtures of clays, silts, sands and gravels were encountered to the maximum explored depth of about 50 feet.

Borings B-1-16 and B-3-16 were located closest to the top of the riverbank. The underlying alluvium consists of predominately loose to medium dense sand and gravelly sand. Boring B-1-16 and Boring B-3-16 were terminated at about 45.5 feet and 48.5 feet, respectively. Boring B-4-16 was located about 200 feet from the river and encountered predominately loose to medium dense, clayey, gravelly sand. The boring was terminated at 45.5 feet. Boring B-2-16 was drilled in the northwestern corner of the project area. Alluvium at this location consisted of soft to medium stiff, low to medium plasticity, clayey sand to sandy clay. Boring B-2-16 was terminated at 48.5 feet.

Groundwater was measured at about 15 to 20 feet below ground surface in the project borings and reference borings, and was estimated at about three to ten feet below ground surface based on pore pressure dissipation tests conducted at CPTs 6-21, 12-21, 15-21 and 16-21. Because the borings and CPTs were not left open for an extended period of time, a stabilized depth to groundwater may not have been observed at the boring and CPT locations. Groundwater elevations fluctuate seasonally and with changes in river elevations, and higher groundwater levels may be present during periods of intense rainfall and/or high tide.

4.0 GEOLOGIC HAZARDS

The principal geologic hazards which could potentially affect the project site are strong seismic shaking from future earthquakes in the San Francisco Bay Region along with subsequent post liquefaction settlements and lateral spreading. Flooding is also a potential hazard at the site. Other geologic hazards such as fault rupture, traditional landsliding, seiche and tsunami are not considered significant at the site. Geologic hazards, their impacts, and potential mitigation measures are discussed below.

4.1 Fault Surface Rupture

Under the Alquist-Priolo Earthquake Fault Zoning Act, the California Division of Mines and Geology (now known as the California Geological Survey) produced 1:24,000 scale maps showing known active and potentially active faults and defining zones within which special fault studies are required. The nearest known Holocene-active faults are the West Napa and Green Valley Faults which are located approximately 2.7 kilometers (1.7 miles) west and 8.5 kilometers (5.6 miles) east of the site, respectively. The site is not located within an Alquist-Priolo Special Studies Zone. We therefore judge the potential for fault surface rupture in the development area to be low.

Evaluation: Less than significant. No mitigation measures are required.
4.2 Seismic Shaking

The site will likely experience seismic ground shaking similar to other areas in the seismically active Bay Area. The intensity of ground shaking will depend on the characteristics of the causative fault, distance from the fault, the earthquake magnitude and duration, and site-specific geologic conditions. Estimates of peak ground accelerations are based on either deterministic or probabilistic methods.

Deterministic methods use empirical attenuation relations that provide approximate estimates of median peak ground accelerations. A summary of the active faults that could most significantly affect the planning area, their maximum credible magnitude, closest distance to the center of the planning area, and probable peak ground accelerations are summarized in Table 1. The calculated accelerations should only be considered as reasonable estimates. Many factors (soil conditions, orientation to the fault, etc.) can influence the actual ground surface accelerations.

Table 1 – Deterministic Peak Ground Accelerations for Active Faults

<table>
<thead>
<tr>
<th>Fault</th>
<th>Moment Magnitude for Characteristic Earthquake</th>
<th>Closest Estimated Distance (km)</th>
<th>Median Peak Ground Acceleration (g)</th>
<th>Median PGA +1 Std Dev (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>West Napa</td>
<td>6.6</td>
<td>1.7</td>
<td>0.49</td>
<td>0.78</td>
</tr>
<tr>
<td>Green Valley</td>
<td>6.8</td>
<td>8.5</td>
<td>0.32</td>
<td>0.54</td>
</tr>
<tr>
<td>Cordelia</td>
<td>6.5</td>
<td>12.3</td>
<td>0.23</td>
<td>0.41</td>
</tr>
<tr>
<td>Rodgers Creek</td>
<td>7.3</td>
<td>20.6</td>
<td>0.22</td>
<td>0.38</td>
</tr>
<tr>
<td>Great Valley 4B</td>
<td>6.7</td>
<td>25.5</td>
<td>0.15</td>
<td>0.26</td>
</tr>
</tbody>
</table>

Reference: Abrahamson & Silva, Boore & Atkinson, Campbell & Bozorgnia, and Chiou & Youngs 2008 NGA models using $V_{s30} = 260$ m/s.

Probabilistic Seismic Hazard Analysis analyzes all possible earthquake scenarios while incorporating the probability of each individual event to occur. The probability is determined in the form of the recurrence interval, which is the average time for a specific earthquake acceleration to be exceeded. The design earthquake is not solely dependent on the fault with the closest distance to the site and/or the largest magnitude, but rather the probability of given seismic events occurring on both known and unknown faults.

We calculated the peak ground acceleration for two separate probabilistic conditions, including the two percent chance of exceedance in 50 years (2,475-year statistical return period) and the ten percent chance of exceedance in 50 years (475-year statistical return period). The peak ground acceleration values were calculated utilizing the USGS Unified Hazard Tool. The results of the probabilistic analyses are presented below in Table 2.
Table 2 – Probabilistic Peak Ground Accelerations for Active Faults

<table>
<thead>
<tr>
<th>Probability of Exceedance</th>
<th>Statistical Return Period</th>
<th>Magnitude</th>
<th>Peak Ground Acceleration (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2% in 50 years</td>
<td>2,475 years</td>
<td>6.7</td>
<td>0.87</td>
</tr>
<tr>
<td>10% in 50 years</td>
<td>475 years</td>
<td>6.7</td>
<td>0.53</td>
</tr>
</tbody>
</table>


Ground shaking can result in structural failure and collapse of structures or cause non-structural building elements (such as light fixtures, shelves, cornices, etc.) to fall, presenting a hazard to building occupants and contents. Compliance with provisions of the most recent version of the California Building Code (2019 CBC) should result in structures that do not collapse in an earthquake. Damage may still occur and hazards associated with falling objects or non-structural building elements will remain.

The potential for strong seismic shaking at the project site is high. Due to their proximity and historic rate of activity, the West Napa and Green Valley Faults present the highest potential for severe ground shaking. The significant adverse impact associated with strong seismic shaking is potential damage to structures and improvements.

Evaluation: Less than significant with mitigation.
Recommendation: Design new structures in accordance with the provisions of the 2019 California Building Code or subsequent codes in effect when final design occurs. Recommended seismic design coefficients and spectral accelerations are presented in Section 5.1 of this report.

4.3 Liquefaction and Related Effects

Liquefaction refers to the sudden, temporary loss of soil shear strength during strong ground shaking. Liquefaction-related phenomena include liquefaction-induced settlement, flow failure, and lateral spreading. While these phenomena typically occur where there are saturated, loose, granular deposits, recent advances in liquefaction studies indicate that liquefaction can occur in soils with a relatively high fines content provided the fines exhibit a plasticity less than about seven. As shown on Figure 6, regional liquefaction hazard maps indicate the site is mapped within a zone of “high” susceptibility to liquefaction (Association of Bay Area Governments, 2021). Our subsurface exploration encountered interlayered loose to medium dense sandy soils within the alluvial soils that underly the site. We evaluated potential for liquefaction of these soils as described in the following sections.

4.3.1 Liquefaction Evaluation

To evaluate soil liquefaction, the seismic energy from an earthquake is compared with the ability of the soil to resist pore pressure generation, known as the Cyclic Resistance Ratio (CRR). The earthquake energy is termed the cyclic stress ratio (CSR) and is a function of the
maximum considered earthquake peak ground acceleration and depth. Soil resistance to liquefaction is primarily based on its relative density and the amount and plasticity of the fine-grained soil included in the soil matrix. The relative density of cohesionless soil is correlated with the Standard Penetration Test (SPT) blow count data measured in the field and corrected for hammer efficiency, overburden and percent fines to determine the \((N_{1})_{60,CS}\) value. Cone Penetration Test data, corrected for overburden, can also be utilized to determine the relative density of soils and its resistance to liquefaction.

We analyzed the potential for liquefaction utilizing CLiq software by Geologismiki. Our analysis utilizes the data from our CPTs and the procedures outlined by Idriss and Boulanger (2008, 2010 & 2014). For our evaluation, we considered a magnitude 7.0 earthquake producing a PGA of 0.87 g, which corresponds to the PGA\(_M\) value as defined in the ASCE 7-16 Section 11.8.3 and presented in Section 5.1. A groundwater depth of ten feet below ground surface was assumed for our analysis. This groundwater depth was estimated based on quarterly monitoring data of several groundwater wells installed during environmental monitoring for 644 1st Street located roughly 300 feet northwest of the site (BCS, 2011). The results of our liquefaction analyses indicate many of the sandy alluvial soil layers encountered in our boring and CPTs are susceptible to liquefaction under the estimated peak ground acceleration.

4.3.2 Estimated Post-Liquefaction Settlement

We estimated the amount of post-liquefaction settlement that may occur during the design ground motion utilizing the CLiq software and procedures outlined by Idriss and Boulanger. These procedures generally consist of calculating a factor of safety for liquefaction occurrence with potential liquefaction occurring where the factor of safety is less than 2.0. The results of our post-liquefaction settlement analyses are presented in Table 3 below and indicate liquefaction-induced settlements of up to about seven inches may occur during the design ground motion. Given that the soils encountered in our subsurface exploration vary laterally and vertically over short distances, the magnitude of liquefaction-induced settlements may also vary significantly throughout the site.

Additionally, we utilized procedures outlined by Ozocak and Sert (2010) to calculate the Liquefaction Potential Index (LPI), which is a gauge to determine if liquefiable layers will impact the ground surface. LPI is a function of the thickness, depth, and factor of safety of liquefiable layers within a soil column. The resulting LPI value corresponds to a relative potential for deformation impacting the ground surface. Typically, an LPI value less than 5 indicates a low risk of surface manifestation while values between 5 and 15 indicate a moderate risk. LPI values greater than 15 indicate a major or high probability of liquefaction impacting the ground surface. The results of our analyses are presented in Table 3 and indicate the LPI in most of our CPTs are higher than 15 which indicates a “major” probability of surface manifestation.

4.3.3 Lateral Spreading

Lateral spreading is the finite, lateral movement of gently to steeply sloping ground caused by a flow failure of underlying liquefying soil deposits. This phenomenon can also occur near
bodies of water where a “free-face” exists along banks. The southern and eastern portions of the site are located along the northern/western bank of the Napa River with channel elevations about 20 to 30 feet below the surrounding improvements. The alluvial soils include layers of potentially liquefiable soils which could result in lateral spreading toward the river. We estimated potential lateral displacements utilizing the Lateral Displacement Index (LDI) results calculated by CLiq and the procedures outlined by Zheng, et al (2004). The results of our analyses are presented in Table 3 below.

Table 3 – Results of Liquefaction and Lateral Spreading Analyses

<table>
<thead>
<tr>
<th>Location</th>
<th>Estimated Liquefaction-Induced Settlement(2)</th>
<th>Liquefaction Potential Index</th>
<th>Estimated Lateral Spreading (inches)</th>
<th>Lateral Displacement Index</th>
</tr>
</thead>
<tbody>
<tr>
<td>CPT-3-12</td>
<td>4.0</td>
<td>16</td>
<td>1</td>
<td>2</td>
</tr>
<tr>
<td>CPT-4-12</td>
<td>4.0</td>
<td>&gt;20</td>
<td>25</td>
<td>20</td>
</tr>
<tr>
<td>CPT-5-12</td>
<td>4.5</td>
<td>20</td>
<td>50</td>
<td>30</td>
</tr>
<tr>
<td>CPT-6-21</td>
<td>6.0</td>
<td>&gt;20</td>
<td>36</td>
<td>25</td>
</tr>
<tr>
<td>CPT-7-21</td>
<td>7.0</td>
<td>&gt;20</td>
<td>33</td>
<td>20</td>
</tr>
<tr>
<td>CPT-8-21</td>
<td>5.0</td>
<td>&gt;20</td>
<td>19</td>
<td>20</td>
</tr>
<tr>
<td>CPT-9-21</td>
<td>7.0</td>
<td>&gt;20</td>
<td>48</td>
<td>50</td>
</tr>
<tr>
<td>CPT-10-21</td>
<td>2.0</td>
<td>7</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td>CPT-11-21</td>
<td>4.0</td>
<td>20</td>
<td>25</td>
<td>30</td>
</tr>
<tr>
<td>CPT-12-21</td>
<td>2.0</td>
<td>8</td>
<td>7</td>
<td>5</td>
</tr>
<tr>
<td>CPT-14-21</td>
<td>1.0</td>
<td>2</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>CPT-15-21</td>
<td>3.5</td>
<td>17</td>
<td>31</td>
<td>15</td>
</tr>
<tr>
<td>CPT-16-21</td>
<td>1.5</td>
<td>5</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

(1) CPT-13-21 encountered refusal due to apparent obstruction at about four feet and was not included in our analyses.
(2) Estimated liquefaction-induced settlements were rounded to nearest half-inch.

Based on our analyses, it is our opinion that liquefaction and lateral spreading present a high risk of damage to the planned improvements.

*Evaluation:* Less than significant with mitigation.

*Mitigation:* Mitigation measures should include utilizing ground improvement to treat potentially liquefiable soils and to reduce the magnitude of liquefaction-induced settlement and lateral spreading.
4.4 Seismic Densification
Seismic ground shaking can induce settlement of unsaturated, loose, granular soils. Settlement occurs as the loose soil particles rearrange into a denser configuration when subjected to seismic ground shaking. Varying degrees of settlement can occur throughout a deposit, resulting in differential settlement of structures founded on such deposits. Loose, granular soils were encountered above the groundwater table in our subsurface exploration. However, ground improvement is anticipated beneath new structures to mitigate liquefaction and lateral spreading risk. Therefore, we judge the risk of seismic densification and resultant settlements affecting new structures is low. A moderate risk of seismic densification will exist in areas where ground improvement is not utilized.

Evaluation: Less than significant with mitigation.
Mitigation: Mitigation measures summarized in Section 4.3 apply are appropriate for reducing risks associated with seismic densification.

4.5 Expansive Soil
Expansive soils will shrink and swell with fluctuations in moisture content and are capable of exerting significant expansion pressures on building foundations, interior floor slabs and exterior flatwork. Distress from expansive soil movement can include cracking of brittle wall coverings (stucco, plaster, drywall, etc.), racked door and/or window frames, uneven floors, and cracked slabs. Flatwork, pavements, and concrete slabs-on-grade are particularly vulnerable to distress due to their low bearing pressures. Expansive soils also cause soil creep on sloping ground.

Existing structures appear to be performing relatively well and we did not observe significant heave or distress to site pavements and flatwork, curb and gutter or other surfaces which would suggest surface soils are highly expansive. Visual classification of the near-surface soils encountered in our borings indicate the soils are of low plasticity which suggests a low expansion potential. Therefore, the risk of expansive soil affecting the proposed improvements is considered low.

Evaluation: Less than significant with mitigation.
Recommendation: Fill materials used for site grading should conform to the recommendations outlined in Section 5.3.3. Mitigation measures should include using non-expansive soils within the upper three feet below the new building and other improvements. Soils should be moisture conditioned to slightly above the optimum moisture content during site grading and maintained at this moisture content until imported aggregate base and/or surface flatwork is completed.

4.6 Settlement
Significant settlement can occur when new loads are placed over soft, compressible clays or loose granular soils. Our borings encountered deep deposits of alluvium consisting of interlayered loose
to medium dense sandy and gravelly soils and soft to stiff clayey and silty soils which vary vertically and laterally throughout the site. New structural and fill loads could induce settlement in the soft/loose alluvial soils. However, we anticipate that ground improvement will be used to mitigate risks associated with liquefaction and lateral spreading. Ground improvement will improve the strength and stiffness of the underlying soils and reduce the risk of settlement under new fill and structural loads. Therefore, the risk of settlement is considered low.

**Evaluation:** Less than significant with mitigation.

**Recommendation:** Mitigation measures summarized in Section 4.3 apply are appropriate for reducing risks associated with seismic densification.

### 4.7 Slope Instability/Landslides

Slope instability generally occurs in relatively steep slopes and/or on slopes underlain by weak materials. The project site is relatively flat with minor grade changes and there are no known landslide deposits in the vicinity that could affect the site. Therefore, traditional (hillside) slope stability/landslides are not considered a geologic hazard at the project site. However, as previously noted, lateral spreading toward the riverbank could be considered a form of instability which could impact the project site.

**Evaluation:** Less than significant. No mitigation measures required.

### 4.8 Erosion

Sandy soils on moderate slopes or clayey soils on steep slopes are susceptible to erosion when exposed to concentrated water runoff. Erosion along the river banks, such as what occurred at the north end of the oxbow in 2006, should be anticipated at the site. We note that erosion rates are typically more significant on the outer bends of the river which would be the “opposite” bank, but erosion is still possible at the perimeter of the site. Improvements very near to the river will be more prone to erosion damage and setbacks from the top of river for the planned buildings will reduce risks. Erosion risks can be further reduced by vegetating or placing rip rap within areas prone to erosion and promptly repairing eroded damage to prevent more rapid and widespread problems.

**Evaluation:** Less than significant with mitigation.

**Recommendation:** Mitigation measures should be evaluated as project planning advances. Erosion risks should be greatest immediately adjacent to the top of riverbank and there are many “bioengineering” (i.e. vegetation) and “armoring” (i.e. rip-rap) options that could be considered to reduce risks. Careful control of stormwater discharges from the site should also be planned to further reduce risks of erosion. Future erosion should also be repaired and/or closely monitored so small areas of distress are less likely to become major areas of distress.
4.9 Tsunami/Seiche

Seiche and tsunamis are short duration, earthquake-generated water waves in large enclosed bodies of water and the open ocean, respectively. The extent and severity of a seiche or tsunami would be dependent upon ground motions and fault offset from nearby active faults. Regional tsunami hazard mapping indicates the site is not located within an area that is susceptible to tsunami inundation. Therefore, the likelihood of inundation by seiche or tsunami is low.

Evaluation: Less than significant. No mitigation measures are required.

4.10 Flooding

Existing site elevations range from about 15 feet near the top of the riverbank to about 20 feet throughout most of the site. As shown on Figure 7, Flood Insurance Rate Maps prepared by the Federal Emergency Management Agency (FEMA, 2010) indicate the northern and easternmost portions of the site adjacent to the shoreline are mapped within a special flood hazard area characterized as “Zone AE”. This designation corresponds to a FEMA special flood hazard area with a base flood elevation of about 22 feet (NAVD 88). Based on the FEMA mapping, the risk of future flooding at the site is high. We understand that site grading is expected include placing a few feet of fill so that finished grades are above the base flood elevation.

Evaluation: Less than significant with mitigation.
Recommendation: Design finished floor elevations above flood level and conform to any flood mitigation measures required by the City of Napa. The project Civil Engineer should design the site drainage system to accommodate anticipated runoff and should consider the potential for flooding and ponding of water during design of site finished grades.

5.0 CONCLUSIONS AND RECOMMENDATIONS

Based on the results of our subsurface exploration, we judge that construction of the proposed development is feasible from a geotechnical standpoint. Primary geotechnical considerations for the project will include providing uniform foundation support for the new structures, implementing ground improvement to reduce the magnitude of liquefaction-induced settlement and lateral spreading to levels which are appropriate for the new structures, and designing new structures to resist strong seismic ground shaking. Additional discussion and recommendations addressing these and other considerations are presented in the following sections.

5.1 Seismic Design

Minimum mitigation of ground shaking includes seismic design of new structures in conformance with the provisions of the most recent edition (2019) of the California Building Code. The magnitude and character of these ground motions will depend on the particular earthquake and

---

2 Site elevations are based on those shown on “Preliminary Site Plan – Development Limits (Sheet C1.0) by BC Engineering Group, Inc., dated March 2021. No vertical datum is specified.
the site response characteristics. Based on the interpreted subsurface conditions and close proximity of several nearby faults, we recommend the CBC coefficients and site values shown in Table 2 be used to calculate the design base shear of the new construction.

<table>
<thead>
<tr>
<th>Table 4 – 2019 California Building Code Seismic Design Criteria</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Parameter</strong></td>
</tr>
<tr>
<td>Site Class</td>
</tr>
<tr>
<td>Site Latitude</td>
</tr>
<tr>
<td>Site Longitude</td>
</tr>
<tr>
<td>Spectral Response (short), Ss</td>
</tr>
<tr>
<td>Spectral Response (1-sec), S1</td>
</tr>
<tr>
<td>Site Coefficient, Fa</td>
</tr>
<tr>
<td>Site Coefficient, Fv</td>
</tr>
<tr>
<td>Spectral Response (Short), SsMS</td>
</tr>
<tr>
<td>Spectral Response (1 sec), SsM1</td>
</tr>
<tr>
<td>Design Spectral Response (short), SsDS</td>
</tr>
<tr>
<td>Design Spectral Response (1 sec), SsD1</td>
</tr>
<tr>
<td>MCEG PGA Adjusted, PGA</td>
</tr>
<tr>
<td>Seismic Design Category</td>
</tr>
</tbody>
</table>

Due to the presence of sandy and silty soils that are potentially liquefiable, we judge the site soil classifies as Site Class F per the 2019 California Building Code. However, we anticipate the ground improvement will improve the strength and stiffness of the subsurface soils and we have therefore provided seismic design criteria for Site Class D for preliminary design purposes. As discussed in Section 5.2, additional subsurface exploration should be performed after ground improvement is complete to confirm the assumed Site Class is appropriate based on post-improvement conditions.

5.2 Ground Improvement

As previously discussed, the alluvial soils which underly the site include interlayered sandy soils which are susceptible to liquefaction and lateral spreading under future seismic events. Mitigation measures for liquefaction should include utilizing ground improvement to treat potentially liquefiable soils and reduce the magnitude of liquefaction-induced settlement and lateral spreading. For foundation support applications, ground improvement is generally defined as the alteration of site foundation conditions to provide better performance under design and/or operation loading conditions (USACE, 1999). The primary advantages of ground improvement include increasing the soil’s bearing capacity, shear strength, stiffness and resistance to liquefaction and thereby reducing the potential for settlement and lateral deformations under static and seismic conditions.
The project team is currently evaluating various ground improvement alternatives with a specialized ground improvement Contractor. While these evaluations are ongoing, we understand the work will be performed as a design-build contract. The preferred improvement concept is generally includes using a combination of a deep soil mixing “wall” constructed along the riverbank areas surrounding the eastern and southern site perimeter and vibro-replacement stone columns to improve vertical support beneath the new structures. Deep soil mixing generally consists of blending the soil with cementitious and/or other materials which are injected through hollow, rotated mixing shafts with specialized cutting tools. Vibro-replacement stone columns is the partial replacement of unsuitable soils with aggregate columns which are densified by the use of vibratory equipment. Regardless of which methods are implemented, we recommend that the ground improvement be designed and constructed by a qualified design-build Contractor to meet the minimum requirements summarized in Table 5.

Table 5 – Minimum Performance Requirements for Ground Improvement

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Design Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Peak Ground Acceleration</td>
<td>0.87 g</td>
</tr>
<tr>
<td>Allowable Bearing Capacity(^1)</td>
<td>XXXX pounds per square foot</td>
</tr>
<tr>
<td>Maximum Total Settlement(^2)</td>
<td>XX inches</td>
</tr>
<tr>
<td>Maximum Differential Settlement(^2,3)</td>
<td>XX inches over XX feet</td>
</tr>
<tr>
<td>Maximum Lateral Spreading Displacement(^2)</td>
<td>XX inches</td>
</tr>
</tbody>
</table>

(1) To be confirmed using plate load testing with a maximum vertical deflection of XX inches and a factor of safety of 3.0.

(2) Liquefaction-induced settlement and lateral spreading displacements shall be estimated using post-treatment CPT data and the peak ground acceleration noted above.

(3) Differential settlement to be estimated over a horizontal distance of XX feet.

We generally recommend that the design-build specifications include requirements for preproduction testing to confirm that the proposed methods will meet the minimum performance criteria outlined above. We anticipate this would include establishing representative test areas which are constructed in accordance with the proposed design and performing CPTs within the test areas to confirm the minimum performance requirements are achieved. CPTs should also be performed following completion of the ground improvement to confirm that the work conforms to the specifications. Additionally, the design-build Contractor should develop a quality assurance/quality control program which is implemented during construction to monitor and document the work and demonstrate it is completed in accordance with the project requirements.

We note that construction of the vibro-replacement stone columns is expected to generate ground-borne vibrations. The design-build Contractor should evaluate potential impacts to existing improvements as a result of ground-borne vibrations as part of their design. If deemed appropriate by the designer, construction of the vibro-replacement stone column should incorporate vibration monitoring to confirm the vibrations are within levels that are considered appropriate for the site.
5.3 Site Grading

Site grading is expected to include cuts and fills of a few feet as required to reduce flood risk and achieve the design grades for the new roadways, parking areas, and building pads. Site grading should be performed in accordance with the recommendations and criteria outlined in the following sections.

5.3.1 Site Preparation

Clear pavements, old foundations, over-sized debris, and organic material from areas to be graded. Debris, rocks larger than six inches, and vegetation are not suitable for structural fill and should be removed from the site. Trees that are located within the building areas should be removed and the root systems excavated. Existing foundations and utilities which are to be abandoned as part of the work should be removed from structural areas. In non-structural areas, utilities could be abandoned in place in many cases provided cement grout completely fills any void in the utility.

Where fills or other structural improvements are planned on level ground, the subgrade surface should be scarified to a depth of eight inches, moisture conditioned to slightly above the optimum moisture content and compacted to at least 90 percent relative compaction. Fills that are greater than five-feet-thick should be compacted to at least 95 percent relative compaction. Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density, as determined by ASTM D1557. Subgrade preparation should extend a minimum of five feet beyond the planned building envelope in all directions. The subgrade should be firm and unyielding when proof-rolled with heavy, rubber-tired construction equipment. If soft, wet or otherwise unsuitable materials are encountered at subgrade elevation during construction, we will provide supplemental recommendations to address the specific condition.

5.3.2 Excavations

In areas where ground improvement is not implemented, site excavations for new foundations, utilities, and other improvements will generally encounter loose to medium dense sandy soils and medium stiff to very stiff clayey soils. Excavations located within areas in which vibro-replacement stone columns are used will encounter dense to very dense gravel intermixed with native alluvial soils. In areas where deep soil mixing is used, excavations will encounter cemented soils. In unsupported excavations, stone columns and untreated sandy soils will be susceptible to flowing below the groundwater table and running to fast raveling above the groundwater table. Untreated clayey soils and soils treated by deep soil mixing are expected to exhibit firm behavior in unsupported excavations. Definitions of these various ground behaviors are presented in the Tunnelman’s Ground Classification for Soils, Figure 8.

Temporary (steeper) cut slopes may be required during construction. For planning purposes, cut slopes into soils in which deep soil mixing was used may be designed for an OSHA Type “B” soil whereas cut slopes into alluvial soils and soils improved with vibro-
replacement stone columns should be designed for an OSHA Type “C” soil. Permanent cut slopes should be inclined no steeper than 2:1. Based on our subsurface exploration, we judge the majority of site excavation can be performed with conventional equipment, such as medium-size dozers and excavators. However, soils which are improved with deep soil mixing may require specialized techniques or equipment to excavate (e.g., jackhammers or hydraulic breakers). Therefore, we recommend inclusion of a line item and clear definition for excavation in these areas as part of the project bid documents.

5.3.3 Fill Materials, Placement and Compaction

Fill materials should consist of non-expansive materials that are free of organic matter, have a Liquid Limit of less than 40 (ASTM D 4318), a Plasticity Index of less than 20 (ASTM D 4318), and a minimum R-value of 20 (California Test 301). The fill material should contain no more than 50 percent of particles passing a No. 200 sieve and should be well graded with a maximum particle size of four inches. Onsite soils may be suitable for use as fill provided they meet the criteria specified above. Any imported fill material needs to be tested to determine its suitability.

Fill materials should be uniformly moisture conditioned to within two percent of the optimum moisture content prior to compaction. Properly moisture conditioned fill materials should subsequently be placed in loose, horizontal lifts of eight-inches-thick or less and uniformly compacted to at least 90 percent relative compaction. Fill materials should be compacted to at least 92 percent relative compaction in areas where fill thicknesses exceed five feet. In pavement areas subjected to vehicle loads, the upper 12 inches of fill or natural soil should be compacted to at least 95 percent relative compaction and produce a firm and unyielding condition. The maximum dry density and optimum moisture content of fill materials should be determined in accordance with ASTM D1557.

5.4 Foundation Design

Provided that ground improvement is incorporated as discussed in Section 5.2, we judge the new structures could be supported on a relatively rigid shallow foundation system consisting of spread footings with interconnected grade beams or a thickened mat slab. Shallow foundations should be designed using the criteria provided in Tables 5 and 6.
### Table 6 – Shallow Foundation Design Criteria

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Design Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Minimum Embedment1</td>
<td>18 inches</td>
</tr>
<tr>
<td>Minimum Width</td>
<td>18 inches</td>
</tr>
<tr>
<td>Allowable Bearing Pressure2, 3</td>
<td>XX psf</td>
</tr>
<tr>
<td>Base Friction Coefficient</td>
<td>0.XX</td>
</tr>
<tr>
<td>Lateral Passive Resistance4</td>
<td>XX pcf</td>
</tr>
</tbody>
</table>

**Notes:**
1. Maintain minimum of seven feet of horizontal distance between the outer edge of footing and face of nearest adjacent slope/river bank.
2. Design shallow foundations to similar bearing pressures (i.e. size footing widths to maintain relatively uniform bearing loads).
3. Increase design values by 33 percent for total design loads including seismic.
4. Equivalent fluid pressure, not to exceed 3,000 psf. Neglect upper 12 inches unless confined by concrete.

### 5.5 Interior Concrete Slabs

Reinforced concrete slab floors are judged to be appropriate for the new structures provided the building pads are prepared in accordance with our recommendations. The concrete slab floors may be poured monolithically or separated with a cold joint at the Structural Engineer’s discretion. We recommend that interior concrete slabs have a minimum thickness of five inches and be reinforced with steel reinforcing bars (not mesh). Slabs should be placed on a moist subgrade to reduce potential for future expansive behavior. The project Structural Engineer should specifically design the concrete slabs, including locations of crack control joints.

To reduce the potential for moisture to move upward through the slab, a four-inch-thick layer of clean, free draining, ¾-inch angular gravel should be placed beneath interior concrete slabs to form a capillary moisture break. The gravel must be placed on a properly moisture conditioned and compacted subgrade that has been approved by the Geotechnical Engineer. A plastic membrane vapor barrier, 15 mils or thicker, should be placed over the free draining gravel. The vapor barrier should meet the ASTM E1745 Class A requirements and be installed per ASTM E1643. Eliminating the capillary moisture break and/or plastic vapor barrier may result in excess moisture intrusion through the floor slabs resulting in poor performance of floor coverings, mold growth, or other adverse conditions.

We note that over time, placing sand between the vapor barrier and concrete is becoming less common because of elevated interior moisture contents. If sand is used, it should be dry, and if it is not used, the slab should be carefully designed with a lower water-cement ratio since eliminating the sand can cause cracking or “curling” of the new concrete. For slabs that are not sensitive to moisture vapor, we recommend at least four inches of Class 2 Aggregate Base (Caltrans, 2015) compacted to at least 95 percent relative compaction.
5.6 Exterior Concrete Slabs

Exterior concrete walkway slabs and other concrete slabs that are not subjected to vehicle loads should be a minimum of four-inches-thick and underlain with four inches or more of Class 2 Aggregate Base. The aggregate base should be moisture conditioned to near optimum and compacted to at least 95 percent relative compaction. The upper eight inches of subgrade on which aggregate base is placed should be prepared as previously discussed under Section 5.3.

Where improved performance is desired (i.e. reduced risks of cracking or offsets due to seasonal shrink/swell or liquefaction-induced movements), exterior slabs can be thickened to five inches and reinforced with steel reinforcing bars (not welded wire mesh). Driveways and slabs subject to vehicle loads should be a minimum of five-inches-thick and designed to resist traffic loading. We recommend crack control joints no farther than six feet apart in both directions and that the reinforcing bars extend through the control joints. Some movement or offset at sidewalk joints should be expected as the underlying soils expand and shrink from seasonal moisture changes.

5.7 Site and Foundation Drainage

New grading could result in adverse drainage patterns causing water to pond around the structure. Careful consideration should be given to design of finished grades at the site. We recommend that the building area be raised slightly and that the adjoining landscaped areas be sloped downward at least 0.25 feet for five feet (five percent) from the perimeter of the structure. Where hard surfaces, such as concrete or asphalt adjoin foundations, slope these surfaces at least 0.10 feet in the first five feet (two percent).

Roof gutter downspouts may discharge onto the pavements but should not discharge onto landscaped areas immediately adjacent to the structure. Provide area drains for landscape planters adjacent to the structure and collect downspout discharges into a tight pipe collection system that discharges well away from the building. Site drainage should be discharged away from the building areas and outlets should be designed to reduce erosion. Site drainage improvements should be connected into an established storm drainage system.

5.8 Underground Utilities

Excavations for utilities will be in loose to medium dense sandy soils, medium stiff to very stiff clayey soils and improved ground and may encounter groundwater at shallow depths if wintertime or early spring work is performed. Trench excavations having a depth of five feet or more must be excavated and shored in accordance with OSHA regulations. We note that ground improvement is currently only anticipated beneath new structures and some liquefaction-induced settlement may occur, particularly in portions of the site where ground improvement is not used. Therefore, damage to utilities could occur if differential settlements are induced as a result of liquefaction. We generally recommend that flexible pipe materials and utility connections to buildings be used to reduce the potential for damage under seismic conditions.

Bedding materials for utility pipes should be poorly graded sand with 90 to 100 percent of particles passing the No. 4 sieve and no more than five percent finer than the No. 200 sieve. Crushed rock
or pea gravel may also be considered for pipe bedding. Provide the minimum bedding beneath the pipe in accordance with the manufacturer's recommendation, typically three to six inches. Trench backfill may consist of on-site soils, moisture conditioned and placed in thin lifts and compacted to at least 90 percent. Use equipment and methods that are suitable for work in confined areas without damaging utility conduits.

5.9 **Pavements**

We have calculated thicknesses for new asphalt pavements for the new roads in accordance with Caltrans procedures for flexible pavement design. Our calculations assume an R-value of ten for subgrade soils and a range of traffic indices from four to seven depending on the expected traffic loads for a twenty-year design life. The Civil Engineer should be responsible for determining the appropriate traffic index for use in design. In general, areas expected to experience loading from heavy vehicles should be designed using the higher traffic index, while parking areas and other lightly loaded areas can utilize a thinner pavement section based on the lower traffic index. The recommended pavement sections are presented in Table 7.

<table>
<thead>
<tr>
<th>Traffic Index</th>
<th>Asphalt Concrete (inches)</th>
<th>Aggregate Base (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.0</td>
<td>3.0</td>
<td>7.0</td>
</tr>
<tr>
<td>5.0</td>
<td>3.5</td>
<td>8.0</td>
</tr>
<tr>
<td>6.0</td>
<td>5.0</td>
<td>8.5</td>
</tr>
<tr>
<td>7.0</td>
<td>5.0</td>
<td>13.0</td>
</tr>
</tbody>
</table>

(1) Traffic Index for final pavement design to be determined by the project Civil Engineer.

In pavement areas, the upper 12 inches of subgrade should be compacted to at least 95 percent relative compaction. The aggregate base and asphalt-concrete should conform to the most recent version of Caltrans Standard Specifications and should be compacted to at least 95 percent relative compaction. Additionally, the subgrade and aggregate base should be firm and unyielding under heavy, rubber-tired construction equipment.

6.0 **SUPPLEMENTAL GEOTECHNICAL SERVICES**

As project plans are nearing completion, we should review them to confirm that the intent of our geotechnical recommendations has been incorporated. We can also consult with project team to supplement or clarify geotechnical recommendations, if needed. During construction, we should be present intermittently to observe ground improvement, foundation excavations, moisture conditioning of soils, fill placement and compaction, utility trench backfill, subgrade preparation and compaction in new pavement and flatwork areas, and other geotechnical-related work items. The purpose of our observation and testing is to confirm that site conditions are as anticipated, to
adjust our recommendations and design criteria if needed, and to confirm that the Contractor’s work is performed in accordance with the project plans and specifications.

7.0 LIMITATIONS

We believe this report has been prepared in accordance with generally accepted geotechnical engineering practices in the San Francisco Bay Area at the time the report was prepared. This report has been prepared for the exclusive use of Harvest Properties and/or their assignees specifically for this project. No other warranty, expressed or implied, is made. Our evaluations and recommendations are based on the data obtained during our subsurface exploration program and our experience with soils in this geographic area. Our approved scope of work did not include a detailed environmental assessment of the site. We recommend that an environmental consultant be retained to evaluate environmental-related issues.

The evaluations and recommendations do not reflect variations in subsurface conditions that may exist between boring locations or in unexplored portions of the site. Should such variations become apparent during construction, the general recommendations contained within this report will not be considered valid unless Miller Pacific is given the opportunity to review such variations and revise or modify our recommendations accordingly. No changes may be made to the general recommendations contained herein without the written consent of Miller Pacific.

We recommend that this report, in its entirety, be made available to project team members, contractors, and subcontractors for informational purposes and discussion. We intend that the information presented within this report be interpreted only within the context of the report as a whole. No portion of this report should be separated from the rest of the information presented herein. No single portion of this report shall be considered valid unless it is presented with and as an integral part of the entire report.

8.0 LIST OF REFERENCES


US Army Corps of Engineers, “Logs of Explorations, Third Street to Lincoln Avenue, Plate 97 and 98”.


22
SITE LOCATION

SITE COORDINATES
LAT. 38.3009°
LON. -122.2802°

REGIONAL GEOLOGIC MAP
(NO SCALE)

LEGEND

Qhty STREAM TERRACE DEPOSITS (Latest Holocene)
Composed of moderately sorted clayey sand and sandy clay with gravel

Qht STREAM TERRACE DEPOSITS (Holocene < 10,000 years)
Composed of moderately to well-sorted and bedded sand, gravel, silt, and minor clay

DATA SOURCE:
DATA SOURCE:
REFERENCE: MTC/ABAG Hazard Viewer Map (https://mtc.maps.arcgis.com), accessed May 18, 2021
LEGEND

SPECIAL FLOOD HAZARD AREAS SUBJECT TO INUNDATION BY THE 1% ANNUAL CHANCE FLOOD

The 1% annual flood (100-year flood), also known as the base flood, is the flood that has a 1% chance of being equaled or exceeded in any given year. The Special Flood Hazard Area is the area subject to flooding by the 1% annual chance flood. Areas of Special Flood Hazard include Zones A, AE, AH, AQ, AR, AO, V, and VE. The Base Flood Elevation is the water-surface elevation of the 1% annual chance flood.

ZONE A
- No Base Flood Elevations determined.

ZONE AE
- Base Flood Elevations determined.

ZONE AH
- Flood depths of 1 to 3 feet (usually areas of ponding); Base Flood Elevations determined.

ZONE AO
- Flood depths of 1 to 3 feet (usually sheet flow on sloping terrain); average depths determined. For areas of annual flood, vehicles also determined.

ZONE AR
- Special Flood Hazard areas formerly protected from the 2% annual chance flood by a flood control system that was subsequently deteriorated. Zone AR indicates that the former flood control system is being restored to provide protection from the 1% annual chance or greater flood.

ZONE AV
- Areas to be protected from 1% annual chance flood by a Federal flood protection system under construction; no Base Flood Elevations determined.

ZONE V
- Coastal flood zone with velocity hazard (wave action); Base Flood Elevations determined.

ZONE VE
- Coastal flood zone with velocity hazard (wave action); Base Flood Elevations determined.

FLOODWAY AREAS IN ZONE AE
- The floodway is the channel of a stream plus any adjacent floodplain areas that must be kept free of encroachment so that the 1% annual chance flood can be carried without substantial increases in flood height.

OTHER FLOOD AREAS

ZONE X
- Areas of 0.2% annual chance flood; areas of 1% annual chance flood with average depths of less than 1 foot or with drainage areas less than 1 square mile, and areas protected by levees from 1% annual chance flood.

ZONE X
- Areas determined to be outside the 0.2% annual chance floodplain.

ZONED
- Areas in which flood hazards are undetermined, but possible.

COASTAL BARRIER RESOURCES SYSTEM (CBRS) AREAS
- CBRS areas are normally located within or adjacent to Special Flood Hazard Areas.

OTHERWISE PROTECTED AREAS (OPA)
- Boundary dividing Special Flood Hazard Areas and boundary dividing Special Flood Hazard Areas of different Base Flood Elevations, flood depths or flood velocities.

- Limit of Moderate Wave Action
- Base Flood Elevation line and value; elevation in feet
- Base Flood Elevation value where uniform within zone; elevation in feet

* Referred to the North American Vertical Datum of 1988

TRANSPORTATION NETWORKS
- Cross section line
- Transect line
- Geographic coordinates referenced to the North American Datum of 1983 (NAD 83), Western Hemisphere
- North American Universal Transverse Mercator grid values, zone 11
- 660,000 feet
- 660000 Feet
- California State Plane coordinate system, zone D (PPS2000), Lambert Conformal Conic projection
- Bench mark (see explanation in Notes to Users section of this FIRM map)
- Revised by: RCA
- Checked by: Miller Pacific

FEMA FLOOD INSURANCE RATE MAP

Copia Redevelopment
Napa, California

Project No. 2489.002
Date: 5/24/2021
# Tunnelman's Ground Classification for Soils

<table>
<thead>
<tr>
<th>Classification</th>
<th>Behavior</th>
<th>Typical Soil Types</th>
</tr>
</thead>
<tbody>
<tr>
<td>Firm</td>
<td>Heading can be advanced without initial support, and final lining can be constructed before ground starts to move.</td>
<td>Loess above water table; hard clay, marl, cemented sand and gravel when not highly overstressed.</td>
</tr>
<tr>
<td>Raveling</td>
<td>Chunks or flakes of material begin to drop out of the arch or walls sometime after the ground has been exposed, due to loosening or to over-stress and “brittle” fracture (ground separates or breaks along distinct surfaces, opposed to squeezing ground). In fast raveling ground, the process starts within a few minutes, otherwise the ground is slow raveling.</td>
<td>Residual soils or sand with small amounts of binder may be fast raveling below the water table, slow raveling above. Stiff fissured clays may be slow or fast raveling depending upon degree of overstress.</td>
</tr>
<tr>
<td>Squeezing</td>
<td>Ground squeezes or extrudes plastically into tunnel, without visible fracturing or loss of continuity, and without perceptible increase in water content. Ductile, plastic yield and flow due to overstress.</td>
<td>Ground with low frictional strength. Rate of squeeze depends on degree of overstress. Occurs at shallow to medium depth in clay of very soft to medium consistency. Stiff to hard clay under high cover may move in combination of raveling at excavation surface and squeezing at depth behind surface.</td>
</tr>
<tr>
<td>Running</td>
<td>Granular materials without cohesion are unstable at a slope greater than their angle of repose (+/− 30° − 35°). When exposed at steeper slopes they run like granulated sugar or dune sand until the slope flattens to the angle of repose.</td>
<td>Clean, dry granular materials. Apparent cohesion in moist sand, or weak cementation in any granular soil, may allow the material to stand for a brief period of raveling before it breaks down and runs. Such behavior is cohesive-running.</td>
</tr>
<tr>
<td>Flowing</td>
<td>A mixture of soil and water flows into the tunnel like a viscous fluid. The material can enter the tunnel from the invert as well as from the face, crown, and walls, and can flow for great distances, completely filling the tunnel in some cases.</td>
<td>Below the water table in silt, sand, or gravel. Without enough clay content to give significant cohesion and plasticity. May also occur in highly sensitive clay when such material is disturbed.</td>
</tr>
<tr>
<td>Swelling</td>
<td>Ground absorbs water, increases in volume, and expands slowly into the tunnel.</td>
<td>Highly preconsolidated clay with plasticity index in excess of about 30, generally containing significant percentages of montmorillonite.</td>
</tr>
</tbody>
</table>

---

1 Modified by Heuer (1974) from Terzaghi (1950)
APPENDIX A
SUBSURFACE EXPLORATION AND LABORATORY TESTING

A. SUBSURFACE EXPLORATION

We explored subsurface conditions in 2012 and 2016 with three cone penetration tests (CPTs) and four borings as part of our field explorations for previously proposed developments. We completed an additional 11 CPTs on April 8 and 9, 2021 to provide supplemental subsurface data for the currently proposed development. The approximate locations of the borings and CPTs are shown on the Site Plan, Figure 2. The exploration was conducted under the technical supervision of our Field Geologist who examined and logged the soil materials encountered in our borings and obtained samples. The subsurface conditions encountered in the test borings and CPTs are summarized and presented on the Boring and CPT Logs, Figures A-1 through A-28.

Relatively “undisturbed” samples were obtained using a three-inch diameter, split-barrel Modified California Sampler with 2.5 by six-inch tube liners or a Standard Penetration Test (SPT) Sampler. The samplers were driven by a 140-pound hammer at a 30-inch drop. The number of blows required to drive the samplers 18 inches was recorded and is reported on the boring logs as blows per foot for the last 12 inches of driving. The samples obtained were examined in the field, sealed to prevent moisture loss, and transported to our laboratory.

B. LABORATORY TESTING

We conducted laboratory tests on selected intact samples to classify soils and to estimate engineering properties. The following laboratory tests were conducted in general accordance with the ASTM standard test method cited:

- Laboratory Determination of Water (Moisture Content) of Soil, Rock, and Soil-Aggregate Mixtures, ASTM D 2216
- Density of Soil in Place by the Drive-Cylinder Method, ASTM D2937
- Unconfined Compressive Strength of Cohesive Soil, ASTM D2166
- Amount of Material in Soils Finer than No. 200 (75-μm) Sieve, ASTM D1140

The laboratory test results are shown on the exploratory boring logs. The exploratory boring logs, description of soils encountered and the laboratory test data reflect conditions only at the location of the boring at the time they were excavated or retrieved. Conditions may differ at other locations and may change with the passage of time due to a variety of causes including natural weathering, climate and changes in surface and subsurface drainage.
<table>
<thead>
<tr>
<th>MAJOR DIVISIONS</th>
<th>SYMBOL</th>
<th>DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>COARSE GRAINED SOILS</strong> over 50% sand and gravel</td>
<td>GW</td>
<td>Well-graded gravels or gravel-sand mixtures, little or no fines</td>
</tr>
<tr>
<td></td>
<td>GP</td>
<td>Poorly-graded gravels or gravel-sand mixtures, little or no fines</td>
</tr>
<tr>
<td></td>
<td>GM</td>
<td>Silty gravels, gravel-sand-silt mixtures</td>
</tr>
<tr>
<td></td>
<td>GC</td>
<td>Clayey gravels, gravel-sand-clay mixtures</td>
</tr>
<tr>
<td></td>
<td>SW</td>
<td>Well-graded sands or gravelly sands, little or no fines</td>
</tr>
<tr>
<td></td>
<td>SP</td>
<td>Poorly-graded sands or gravelly sands, little or no fines</td>
</tr>
<tr>
<td></td>
<td>SM</td>
<td>Silty sands, sand-silt mixtures</td>
</tr>
<tr>
<td></td>
<td>SC</td>
<td>Clayey sands, sand-clay mixtures</td>
</tr>
<tr>
<td><strong>FINE GRAINED SOILS</strong> over 50% silt and clay</td>
<td>ML</td>
<td>Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity</td>
</tr>
<tr>
<td></td>
<td>CL</td>
<td>Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays</td>
</tr>
<tr>
<td></td>
<td>OL</td>
<td>Organic silts and organic silt-clays of low plasticity</td>
</tr>
<tr>
<td></td>
<td>MH</td>
<td>Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts</td>
</tr>
<tr>
<td></td>
<td>CH</td>
<td>Inorganic clays of high plasticity, fat clays</td>
</tr>
<tr>
<td></td>
<td>OH</td>
<td>Organic clays of medium to high plasticity</td>
</tr>
<tr>
<td><strong>HIGHLY ORGANIC SOILS</strong></td>
<td>PT</td>
<td>Peat, muck, and other highly organic soils</td>
</tr>
</tbody>
</table>

**SOIL CLASSIFICATION CHART**

- **Classification Tests**
  - PI: Plasticity Index
  - LL: Liquid Limit
  - SA: Sieve Analysis
  - HYD: Hydrometer Analysis
  - P200: Percent passing No. 200 Sieve
  - P4: Percent passing No. 4 Sieve

- **Strength Tests**
  - TV: Field Torvane (Undrained Shear)
  - UC: Laboratory Unconfined Compression
  - TXCU: Consolidated Undrained Triaxial
  - TXUU: Unconsolidated Undrained Triaxial

- **Sampler Driving Resistance**
  Modified California and Standard Penetration Test samplers are driven 18 inches with a 140-pound hammer falling 30 inches per blow. Blows for the initial 6-inch drive seat the sampler. Blows for the final 12-inch drive are recorded onto the logs. Sampler refusal is defined as 50 blows during a 6-inch drive. Examples of blow records are as follows:
  - 25 sampler driven 12 inches with 25 blows after initial 6-inch drive
  - 85/7" sampler driven 7 inches with 85 blows after initial 6-inch drive
  - 50/3" sampler driven 3 inches with 50 blows during initial 6-inch drive or beginning of final 12-inch drive

**NOTE:** Test boring and test pit logs are an interpretation of conditions encountered at the excavation location during the time of exploration. Subsurface rock, soil or water conditions may vary in different locations within the project site and with the passage of time. Boundaries between differing soil or rock descriptions are approximate and may indicate a gradual transition.
### BORING B-1-16

**EQUIPMENT:** Truck mounted Mobile B53 drill rig with 6-in Ø hollow-stem augers  
**DATE:** 8/30/16  
**ELEVATION:** 17 - feet*  
**REFERENCE:** Google Earth, 2016

<table>
<thead>
<tr>
<th>DEPTH (feet)</th>
<th>SAMPLE SYMBOL (4)</th>
<th>CONTENT</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>3&quot; Asphalt Concrete</td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>12&quot; Aggregate Base</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Sandy Gravelly CLAY (CL)</td>
<td>Brown, moist, very stiff, low plasticity clay, ~10-15% fine to coarse grained sand, ~5-10% sub-rounded gravel. [Fill]</td>
</tr>
<tr>
<td>5</td>
<td>SAND (SW)</td>
<td>Brown, moist, medium dense, fine to medium grained sand, sub-rounded to rounded, well graded. [Alluvium]</td>
</tr>
<tr>
<td>10</td>
<td>3&quot; Asphalt Concrete</td>
<td></td>
</tr>
<tr>
<td>15</td>
<td>12&quot; Aggregate Base</td>
<td></td>
</tr>
<tr>
<td>16</td>
<td>Sandy Gravelly CLAY (CL)</td>
<td>Brown, moist, very stiff, low plasticity clay, ~10-15% fine to coarse grained sand, ~5-10% sub-rounded gravel. [Fill]</td>
</tr>
<tr>
<td>17</td>
<td>SAND (SW)</td>
<td>Brown, moist, medium dense, fine to medium grained sand, sub-rounded to rounded, well graded. [Alluvium]</td>
</tr>
<tr>
<td>20</td>
<td>3&quot; Asphalt Concrete</td>
<td></td>
</tr>
</tbody>
</table>

NOTES:  
(1) UNCORRECTED FIELD BLOW COUNTS  
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m³ = 0.1571 x DRY UNIT WEIGHT (pcf)  
(3) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)  
(4) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

**BORING LOG**  
Copia Redevelopment  
Napa, California  
Project No. 2489.002  
Date: 9/1/16

A CALIFORNIA CORPORATION, © 2021, ALL RIGHTS RESERVED  
FILE: 2489.002 Boring Log.dwg
<table>
<thead>
<tr>
<th>Depth (feet)</th>
<th>Sample</th>
<th>Symbol</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>8</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>9</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>11</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>12</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**SAND (SP)**
Dark gray, saturated, loose, medium to coarse grained sand. [Alluvium]

**CLAY (CH)**
Dark gray, saturated, soft, high plasticity clay. [Alluvium]

**SAND (SP)**
Medium red brown, saturated, loose, medium to coarse grained sand. [Alluvium]

**Gravelly SAND (SW)**
Brown, saturated, loose, medium to coarse grained sand, sub-angular to rounded grains, ~25% gravel. [Alluvium]

Grades to medium dense, 30-35% gravel

Very stiff drilling at 39.0-feet

---

**NOTES:**
1. UNCORRECTED FIELD BLOW COUNTS
2. METRIC EQUIVALENT DRY UNIT WEIGHT kN/m² = 0.1571 x DRY UNIT WEIGHT (pcf)
3. METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
4. GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY
<table>
<thead>
<tr>
<th>Depth (feet)</th>
<th>Sample Symbol</th>
<th>Boring Terminated Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>13</td>
<td></td>
<td>Gravelly SAND (SW)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Brown, saturated, loose, medium to coarse sand, sub-angular to rounded grains, ~25% gravel. [Alluvium]</td>
</tr>
<tr>
<td>45</td>
<td></td>
<td>Gravelly SAND with Clay (SW)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Dark gray, saturated, dense, medium to coarse grained sand, ~30% gravel, ~10-15% medium plasticity clay. [Alluvium]</td>
</tr>
<tr>
<td>45.5</td>
<td></td>
<td>Boring terminated at 45.5 feet. Groundwater measured at 17.0 feet upon completion of exploration.</td>
</tr>
</tbody>
</table>

**NOTES:**
1. UNCORRECTED FIELD BLOW COUNTS
2. METRIC EQUIVALENT DRY UNIT WEIGHT kN/m³ = 0.1571 x DRY UNIT WEIGHT (pcf)
3. METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
4. GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY
**BORING B-2-16**

**EQUIPMENT:** Truck mounted Mobile B53 drill rig with 6-in Ø hollow-stem augers

**DATE:** 8/31/16

**ELEVATION:** 18 - feet*

**REFERENCE:** Google Earth, 2016

<table>
<thead>
<tr>
<th>DEPTH</th>
<th>SYMBOL</th>
<th>BLOWS / FOOT (1)</th>
<th>DRY UNIT WEIGHT pcf (2)</th>
<th>MOISTURE CONTENT (%)</th>
<th>SHEAR STRENGTH psf (3)</th>
<th>OTHER TEST DATA</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>9</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>11</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>13</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>15</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>17</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>19</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>20</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**NOTES:**
1. UNCORRECTED FIELD BLOW COUNTS
2. METRIC EQUIVALENT DRY UNIT WEIGHT kN/m² = 0.1571 x DRY UNIT WEIGHT (pcf)
3. METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
4. GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

**Sandy Gravelly CLAY (CL)**
Brown, moist, medium stiff, low plasticity clay, ~5-10% fine to medium grained sand, ~5-10% sub-rounded gravel. [Fill]

**Sandy CLAY (CL)**
Dark brown, moist, medium stiff, low to medium plasticity clay, ~20-30% fine grained sand, trace gravel. [Alluvium]

No recovery, cobble in front of sampler

Grades to medium yellow brown, ~25% fine to medium grained sand.

Stiff drilling at 12.0-feet, cuttings show stiff clay, brown, moist, medium to high plasticity

**Silty CLAY (CL)**
Mottled brown and gray, moist, soft, low to medium plasticity clay and silt. [Alluvium]
### BORING B-2-16
(CONTINUED)

<table>
<thead>
<tr>
<th>DEPTH (meters/feet)</th>
<th>SAMPLING SYMBOL (4)</th>
<th>BLOWS / FOOT (1)</th>
<th>DRY UNIT WEIGHT pcf (2)</th>
<th>MOISTURE CONTENT (%)</th>
<th>SHEAR STRENGTH psf (3)</th>
<th>OTHER TEST DATA</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7</td>
<td></td>
<td>3</td>
<td></td>
<td>33.2</td>
<td>P200</td>
<td>85.2%</td>
</tr>
<tr>
<td>8</td>
<td>Silty CLAY (CL)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>8</td>
<td></td>
<td></td>
<td>3</td>
<td>33.2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>Gravels present at 33.0-feet</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>SAND with Clay (SP)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>11</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>12</td>
<td>Large gravels present at 40.5-feet (approx. 3.0-inches in diameter)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**NOTES:**
- (1) UNCORRECTED FIELD BLOW COUNTS
- (2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m² = 0.1571 x DRY UNIT WEIGHT (pcf)
- (3) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
- (4) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

---

**Silty CLAY (CL)**
Mottled brown and gray, moist, soft, low to medium plasticity clay and silt. [Alluvium]

**Silty CLAY (CL)**
Dark grey, wet-saturated, soft, medium to high plasticity, ~20-30% silt

Gravels present at 33.0-feet

SAND with Clay (SP)
Medium brown, saturated, soft, ~15-20% medium plasticity clay, sand is medium grained, rounded to sub-rounded grains

Slow, stiff drilling at 36.0-feet

Large gravels present at 40.5-feet (approx. 3.0-inches in diameter)

---

**BORING BORING LOG**

A CALIFORNIA CORPORATION, © 2021, ALL RIGHTS RESERVED

FILE: 2489.002 Boring Log.dwg

---

**Copia Redevelopment**

**Napa, California**

**Project No. 2489.002**

**Date: 9/1/16**

---

**A-6**

**FIGURE**
<table>
<thead>
<tr>
<th>Depth</th>
<th>Sample</th>
<th>Symbol</th>
<th>Bows / Foot</th>
<th>Dry Unit Weight (pcf)</th>
<th>Moisture Content (%)</th>
<th>Shear Strength (psf)</th>
<th>Other Test Data</th>
<th>Other Test Data</th>
</tr>
</thead>
<tbody>
<tr>
<td>40</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>13</td>
<td></td>
<td></td>
<td>7</td>
<td>35.7</td>
<td>40.2%</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>45</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>14</td>
<td></td>
<td></td>
<td>15</td>
<td>30.3</td>
<td>32.0%</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>15</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>50</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>16</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>55</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>17</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>60</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**NOTES:**

1. UNCORRECTED FIELD BLOW COUNTS
2. METRIC EQUIVALENT DRY UNIT WEIGHT kN/m² = 0.1571 x DRY UNIT WEIGHT (pcf)
3. METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
4. GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

**BORING LOG**

Copia Redevelopment
Napa, California

**A-7**

504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

A CALIFORNIA CORPORATION, © 2021, ALL RIGHTS RESERVED
FILE: 2489.002 Boring Log.dwg
**BORING B-3-16**

**EQUIPMENT:** Truck mounted Mobile B53 drill rig with 6-in Ø hollow-stem augers

**DATE:** 9/1/16

**ELEVATION:** 19 - feet*

**REFERENCE:** Google Earth, 2016

<table>
<thead>
<tr>
<th>DEPTH (feet)</th>
<th>SYMBOL</th>
<th>SAMPLE</th>
<th>ELEVATION</th>
<th>DATE</th>
<th>EQUIPMENT</th>
<th>OTHER TEST DATA</th>
<th>OTHER TEST DATA</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**NOTES:**

1. **UNCORRECTED FIELD BLOW COUNTS**
2. **METRIC EQUIVALENT DRY UNIT WEIGHT kN/m$^3$ = 0.1571 x DRY UNIT WEIGHT (pcf)**
3. **METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)**
4. **GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY**

---

**Topsoil (6")**

Sandy Gravelly CLAY (CL)

- Brown, moist, medium dense, low plasticity, lots of fill debris present (brick, asphalt, etc.) [Fill]

**SAND (SP)**

- Light brown, dry, loose, fine grained, poorly graded [Alluvium]

Gravel present from 10.5-feet to 12.5-feet

As above, trace gravel present

Stiff/dense drilling at 16.0-feet

Gravel present at 17.5-feet

Groundwater present at 20.0-feet

---

**BORING LOG**

**Copia Redevelopment**

**Napa, California**

**Project No. 2489.002**

**Date:** 9/1/16

---

**FILE:** 2489.002 Boring Log.dwg

---

**504 Redwood Blvd.**

**Suite 220**

**Novato, CA 94947**

**T 415 / 382-3444**

**F 415 / 382-3450**

www.millerpac.com
### BORING B-3-16
(CONTINUED)

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Sample</th>
<th>Symbol</th>
<th>BLOWS / FOOT (1)</th>
<th>DRY UNIT WEIGHT pcf (2)</th>
<th>MOISTURE CONTENT (%)</th>
<th>SHEAR STRENGTH psf (3)</th>
<th>OTHER TEST DATA</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>25</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>8</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>23</td>
<td></td>
<td></td>
<td>23</td>
<td>15.2</td>
<td>P200</td>
<td>3.6%</td>
<td></td>
</tr>
<tr>
<td>24.0</td>
<td></td>
<td>SAND (SW)</td>
<td>Brown, wet-saturated, medium dense, medium to coarse grained sand, grains are sub-angular to rounded</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Decrease in gravel size at 28.5-feet</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Sucked up 5.5-feet of gravel into auger at 30.0-feet</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>30</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>35</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>38.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Large gravels present at 38.0-feet (approx. 3.0-inches in diameter)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**NOTES:**
1. UNCORRECTED FIELD BLOW COUNTS
2. METRIC EQUIVALENT DRY UNIT WEIGHT kN/m² = 0.1571 x DRY UNIT WEIGHT (pcf)
3. METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
4. GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

---

**BORING LOG**

Copia Redevelopment
Napa, California

A CALIFORNIA CORPORATION, © 2021, ALL RIGHTS RESERVED

FILE: 2489.002 Boring Log.dwg

504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

Project No. 2489.002  Date: 9/1/16
### BORING B-3-16
(CONTINUED)

<table>
<thead>
<tr>
<th>Depth (feet)</th>
<th>Sample</th>
<th>Symbol (4)</th>
<th>Blows / Foot (1)</th>
<th>Dry Unit Weight pcf (2)</th>
<th>Moisture Content (%)</th>
<th>Shear Strength psf (3)</th>
<th>Other Test Data</th>
<th>Other Test Data</th>
</tr>
</thead>
<tbody>
<tr>
<td>40</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>13</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>45</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>14</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>15</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>50</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>16</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>17</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>18</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>55</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>18</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>60</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**NOTES:**
1) UNCORRECTED FIELD BLOW COUNTS
2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m² = 0.1571 x DRY UNIT WEIGHT (pcf)
3) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
4) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

- Dense drilling at 42.5-feet, gravels scraping on auger
- Large gravel present at 46.0-feet
- End of boring at 48.5-feet
- Groundwater encountered at 18.0-feet
BORING B-4-16

EQUIPMENT: Truck mounted Mobile B53 drill rig with 6-in Ø hollow-stem augers
DATE: 8/30/16
ELEVATION: 17 - feet*
REFERENCE: Google Earth, 2016

3" Asphalt Concrete
12" Aggregate Base

3" Asphalt Concrete
12" Aggregate Base

Sandy Gravelly CLAY (CL)
Dark brown, moist, very stiff, urban debris (concrete and brick fragments, etc.). [Fill]
Concrete fragment in tip of sampler

CLAY (CL/CH)
Dark gray, moist, soft, medium to high plasticity clay. [Alluvium]
Grades ~20% low to medium plasticity clay.
Stiff drilling from 13.5-feet

Groundwater encountered @ 16.0-feet
Sand (SP)
Dark gray, saturated, loose, poorly graded sand. [Alluvium]

NOTES: (1) UNCORRECTED FIELD BLOW COUNTS
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m³ = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(4) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY
<table>
<thead>
<tr>
<th>DEPTH (feet)</th>
<th>SYMBOL (4)</th>
<th>SAND (SP)</th>
<th>Dark gray, saturated, loose, poorly graded [Alluvium]</th>
<th>Gravelly SAND (SW)</th>
<th>Dark gray, saturated loose, well graded, fine to medium grained sand</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7</td>
<td></td>
<td>Clayey Sandy GRAVEL (GC)</td>
<td>Medium brown, saturated, dense, ~20% sand, ~15% clay. [Alluvium]</td>
<td></td>
<td></td>
</tr>
<tr>
<td>8</td>
<td></td>
<td>Drilling eased from 30.0 to 31.5 feet</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>9</td>
<td></td>
<td>Gravelly Clayey SAND (SW)</td>
<td>Dark gray, saturated, loose to medium dense, ~20% gravel, 16% clay, coarse grained sand, sub-angular to sub-rounded. [Alluvium]</td>
<td></td>
<td></td>
</tr>
<tr>
<td>30</td>
<td></td>
<td>Stiffer drilling at 37 feet</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>35</td>
<td></td>
<td>Gravelly Clayey SAND (SW)</td>
<td>Yellow brown with multi colored gravels, saturated, very dense, ~30% gravel, ~15-20% clay, medium to coarse sand. [Alluvium]</td>
<td></td>
<td></td>
</tr>
<tr>
<td>40</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Notes:**
1. UNCORRECTED FIELD BLOW COUNTS
2. METRIC EQUIVALENT DRY UNIT WEIGHT kN/m² = 0.1571 x DRY UNIT WEIGHT (pcf)
3. METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
4. GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY
Gravelly Clayey SAND (SW)
Yellow brown with multi colored gravels, saturated, very dense, ~30% gravel, ~15-20% clay, medium to coarse sand. [Alluvium]

Boring terminated at 45.5 feet. Groundwater measured at 16.0 feet upon completion of exploration.

(*) Overconsolidated or Cemented
CPT-11-21

Copia Redevelopment
Napa, California

Project No. 2489.002
Date: 5/19/2021

CPT DATA

Net Area Ratio & D

Cone Size 15cm squared

1 - sensitive fine grained organic material
dirt

2 - clay

3 - silty clay to clay

4 - silty clay to dry clay

5 - dry clay to clayey silt

6 - dry clay to silty clay

7 - silty clay to sandy silt

8 - silty sand to sandy silt

9 - sand to silty sand

10 - gravelly sand to sand

11 - very stiff fine grained sand

12 - sand to clayey sand

S-Soil behavior type and SPT based on data from UBC-1983

Depth (m)

10 0

16.0

Friction test

P'N

200

50

20

10

200

10

Net Area Ratio (m)

50

10

0 5 10 15 20 25 30 35 40 45 50

0

50

100

150

200

250

300

Net Area Ratio (m)

50

100

150

200

250

300

Net Area Ratio (m)

50

100

150

200

250

300

Net Area Ratio (m)

50

100

150

200

250

300

Net Area Ratio (m)
APPENDIX B

GEOTECHNICAL REFERENCE DATA
# BORING LOG OAK

**G:\ENGINEER\GINT\PROJECTS\3303-002.GPJ**  
**LIBRARY_092906OAK.GLB**  
**10/18/06 08:54 a**

**LOCATION:** STA:2+47  
**SURFACE EL:** 18.7 ft +/-  (rel. NAVD88 datum)

### MATERIAL DESCRIPTION

- **Pavement:** 6" AC  
- **Aggregate:** 12" AB  
- **Clayey SAND with gravel (SC):** medium dense, brown, fine- to coarse-grained  
- **Lean CLAY with sand (CL):** stiff, brown, moist, some fine-grained sand  
- **Well-graded SAND with silt (SW-SM):** dense, brown to reddish brown, wet, fine- to coarse-grained, trace fine gravel  
- **Sandy SILT (ML):** soft, brown, moist to wet, fine-grained sand

### OTHER TESTS

- **PLASTICITY INDEX:** 2.0 P  
- **LIQUID LIMIT:** 2.3 P

---

**BORING DEPTH:** 21.5 ft  
**DEPTH TO WATER:** 19.0 ft  
**BACKFILL:** Grout  
**COMPLETION DATE:** April 22, 2004  
**NOTES:** 1. Terms and symbols defined on Plate A-1.
<table>
<thead>
<tr>
<th>ELEVATION, ft</th>
<th>DEPTH, ft</th>
<th>MATERIAL SYM.</th>
<th>SAMPLER TYPE</th>
<th>BLOW COUNT/ PRESSURE, psi</th>
<th>SURFACE EL: 20.2 ft +/- (rel. NAVD88 datum)</th>
</tr>
</thead>
<tbody>
<tr>
<td>121.5</td>
<td>Surface</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Material Description**

- **Pavement**: 12" AC
- **Aggregate**: 12" AB
- **Sandy Lean CLAY with gravel (CL)**: stiff, brown, moist, fine to coarse-grained sand, fine gravel - FILL
  - brick fragment at 6½ feet
- **Clayey SAND (SC)**: loose, brown, moist, fine-grained
- **Sandy Lean CLAY to Clayey SAND (CL-SC)**: soft, brown, wet, sandy (fine-grained)
- **Lean CLAY (CL)**: firm, gray, wet, trace organics
- **Silty SAND (SM)**: medium dense, olive brown, moist, fine- to medium-grained
- **Clayey SAND (SC)**: medium dense, gray, moist, fine- to medium-grained, trace fine gravel
- **Well-graded SAND with silt (SW-SM)**: dense, olive-brown, wet, fine- to coarse-grained, trace fine-gravel
- **Gravelly Lean CLAY with sand (CL)**: hard, yellowish brown, moist, fine to coarse gravel, coarse-grained sand
  - Lean CLAY (CL): hard, yellowish brown with brown and black mottling, moist, locally gravelly

**Water Content, %**

- **WATER CONTENT, %**
- **% PASSING #200 SIEVE**
- **DRY UNIT WEIGHT, pcf**
- **PLASTICITY LIMIT, %**
- **LIQUID LIMIT, %**
- **PLASTICITY INDEX**
- **UNGRADED MATERIALS**

<table>
<thead>
<tr>
<th>MATERIAL</th>
<th>SYMBOL</th>
<th>SAMPLER</th>
<th>BLOW COUNT/ PRESSURE, psi</th>
<th>OTHER TESTS</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**continued**

**Notes**: 1. Terms and symbols defined on Plate A-1.

**Logging Details**

- **DRILLING METHOD**: 8-in. dia. Rotary Wash
- **HAMMER TYPE**: Rope and Cathead
- **DRILLED BY**: Pitcher Drilling Co.
- **LOGGED BY**: R Storesund

**Location**: STA:3+80

**Project**: First Street Bridge Over Napa River

**City**: Napa, California
### MATERIAL DESCRIPTION

<table>
<thead>
<tr>
<th>ELEVATION, ft</th>
<th>DEPTH, ft</th>
<th>MATERIAL</th>
<th>SYMBOL</th>
<th>LOCATION: STA:3+80</th>
</tr>
</thead>
<tbody>
<tr>
<td>-35</td>
<td>55</td>
<td>Lean CLAY (CL): - continued</td>
<td>(50)</td>
<td>SORFACE EL: 20.2 ft +/- (rel. NAVD88 datum)</td>
</tr>
<tr>
<td>-40</td>
<td>60</td>
<td>- with medium- to coarse-grained sand at 56 feet</td>
<td>(55)</td>
<td></td>
</tr>
<tr>
<td>-45</td>
<td>65</td>
<td>Clayey SAND (SC): very dense, yellowish brown, wet, fine- to coarse-grained, trace fine gravel</td>
<td>(85)</td>
<td></td>
</tr>
<tr>
<td>-50</td>
<td>70</td>
<td>Lean CLAY (CL): very stiff, yellowish brown with gray mottling, moist, trace fine-grained sand</td>
<td>(51)</td>
<td></td>
</tr>
<tr>
<td>-55</td>
<td>75</td>
<td>Clayey SAND with gravel (SC): very dense, olive gray, moist, fine- to coarse-grained, some fine to coarse gravel</td>
<td>(39)</td>
<td></td>
</tr>
<tr>
<td>-60</td>
<td>80</td>
<td></td>
<td>41</td>
<td></td>
</tr>
<tr>
<td>-65</td>
<td>85</td>
<td></td>
<td>57</td>
<td></td>
</tr>
<tr>
<td>-70</td>
<td>90</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>-75</td>
<td>95</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**LOCATION:** First Street Bridge Over Napa River  
Napa, California
**LOG OF BORING NO. B-2**
First Street Bridge Over Napa River
Napa, California

<table>
<thead>
<tr>
<th>ELEVATION, ft</th>
<th>DEPTH, ft</th>
<th>MATERIAL SYMBOL</th>
<th>SAMPLE NO.</th>
<th>SAMPLER TYPE</th>
<th>BLOW COUNT/ PRESSURE, psi</th>
<th>WATER CONTENT, %</th>
<th>% PASSING #200 SIEVE</th>
<th>PLASTICITY INDEX</th>
<th>UNDRAINED SHEAR STRENGTH, Su, ksf</th>
<th>OTHER TESTS</th>
</tr>
</thead>
<tbody>
<tr>
<td>50/3&quot;</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

- **Clayey SAND with gravel (SC)**: - continued

- **Lean CLAY with sand (CL)**: hard, yellowish brown and olive gray, moist, fine- to medium-grained sand

**MATERIAL DESCRIPTION**

**LOCATION:** STA:3+80

**SURFACE EL:** 20.2 ft +/- (rel. NAVD88 datum)

**BORING DEPTH:** 121.5 ft
**DEPTH TO WATER:** Not Measured
**BACKFILL:** Grout
**COMPLETION DATE:** May 6, 2004
**NOTES:** 1. Terms and symbols defined on Plate A-1.

**BORING LOG OAK**

**G:\ENGINEER\GINT\PROJECTS\3303-002.GPJ**
**LIBRARY_092906OAK.GLB**
**10/18/06  08:55 a**