100 WOOD HOLLOW DRIVE
NOVATO, CALIFORNIA

PRELIMINARY GEOTECHNICAL EXPLORATION

SUBMITTED TO
Mr. Logan Williams
Align Real Estate
255 California, Suite 525
San Francisco, CA 94111

PREPARED BY
ENGEO Incorporated

August 16, 2024

PROJECT NO.
26288.000.001

Expect Excellence

ENGEO
Expect Excellence

GEOTECHNICAL
ENVIRONMENTAL
COASTAL/MARITIME
WATER RESOURCES
CONSTRUCTION SERVICES

Project No.
26288.000.001

August 16, 2024

Mr. Logan Williams
Align Real Estate
255 California, Suite 525
San Francisco, CA 94111

Subject: 100 Wood Hollow Drive
Novato, California

PRELIMINARY GEOTECHNICAL EXPLORATION

Dear Mr. Williams:

We are pleased to present this preliminary geotechnical report for the conceptual planning of your proposed 65 single-family residential development at 100 Wood Hollow Drive, California. This report presents our preliminary geotechnical findings, conclusions, and recommendations.

Based on our initial assessment, it is our opinion that development at the project site is feasible from a geotechnical standpoint. The primary geotechnical and geologic considerations for the project are the presence of landslides, expansive soil, undocumented fill, and shallow perched groundwater. A design-level geotechnical exploration should be conducted prior to site development once information is available regarding the structural loads and proposed grading.

We are pleased to have been of service on this project and are prepared to consult further with you and your design team as the project progresses. If you have any questions or comments regarding this, please give us a call.

Sincerely,

ENGEO Incorporated

Vlad Zasmolin, PE

Todd Bradford, GE

REGISTERED PROFESSIONAL ENGINEER
TODD BRADFORD
No. 3200
GEOTECHNICAL
STATE OF CALIFORNIA

Robert H. Boeche, CEG

CERTIFIED ENGINEERING GEOLOGIST
ROBERT BOECHE
No. 2318
STATE OF CALIFORNIA

vz/tb/rb

2010 Crow Canyon Place, Suite 250 • San Ramon, CA 94583 • (925) 866-9000 • Fax (888) 279-2698
www.engeo.com

TABLE OF CONTENTS

TABLE OF CONTENTS (Continued)

1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

We prepared this preliminary geotechnical report for the proposed residential development at 100 Wood Hollow Drive (Site) in Novato, California. Our scope of services is outlined in our agreement dated July 12, 2024, and included the following.

- Review of published maps, previous reports, and historical information
- Review of aerial images
- Exploration of subsurface conditions
- Laboratory testing of soil samples
- Preparation of this report summarizing our preliminary conclusions and recommendations for the proposed development

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

1.2 PROJECT LOCATION, DESCRIPTION, AND PROPOSED DEVELOPMENT

For our use, we received the following documents.

- TWM Architects & Planners. (2000). Wood Hollow San Marin Business Park, Novato, CA, June 22, 2000.
- CBG. (2024). Overall Site Plan; 100 Wood Hollow Drive, Novato, CA, March 1, 2024.

The proposed residential development is located at the northern edge of Novato, California on a 13-acre parcel identified as Assessor's Parcel Number (APN) 125-202-017, as shown in Figures 1 and 2. The area is currently occupied by an office structure with its associated paved parking lot and landscape areas. The parcel is bounded by Wood Hollow Drive to the south, Meadow Crest Road to the west, and a vegetated Mt. Burdell to the north and east.

To facilitate the proposed development, the existing structure and its improvements will be demolished. As illustrated in the Site plan (CBG, 2024), 65 single-family residential houses, paved driving lanes, sidewalks, landscape areas, underground utilities, and bioretention basins are preliminarily planned.

1.3 SITE HISTORY

We reviewed historical aerial photographs of the Site available on Google Earth, UCSB Frame Finder, and www.historicaerials.com. Aerial photographs as early as 1952 show the Site occupied by vegetation and trees at the top of Mt. Burdells' slopes. The nearby US-101 and Redwood Boulevard were realigned sometime between 1969 and 1975 to curve into the current configuration that is seen in present day images. Between 1982 and 1987, the Fireman's Fund Office campus was constructed to the south, as was Meadow Creek Road, to the east of the Site.

In 2000, the existing office structure was built. The Site has remained relatively unchanged between the 2000 photograph and our exploration in 2024.

1.4 PREVIOUS REPORTS

We reviewed past geotechnical reports that we and other geotechnical consultants prepared for projects in immediate proximity or within the limits of the project Site. These reports include:

1. Cooper-Clark and Associates. 1978. Preliminary Geotechnical Investigation, Proposed 54-Acre Industrial Development, Nunes Ranch, Novato, California. July 7, 1978. Job No. 1950-C.
2. Herzog & Associates. 1982. Geotechnical Investigation, San Marin Commerce Park, Novato, California. April 28, 1982. Job No. 134.41.
3. Kleinfelder. 1998. Preliminary Geotechnical Investigation; Parcels 1 and 2, Wood Hollow Drive and Redwood Boulevard, Novato, California. September 24, 1998. Job No. 41-7449-01.
4. Kleinfelder. 1999. Geotechnical Investigation Report, Area “E”, San Marin Business Park, Redwood Boulevard, Novato, California. September 8, 1999. Job No. 41-7449-02.
5. Herzog & Associates. 2001. Geotechnical Review, Deformation Analyses, San Marin Business Park, Area E, Novato, California. October 24, 2001. Project No. 728-01-00.
6. ENGEO. 2023. Geotechnical Exploration Report, Valley Oaks, Novato California. January 12, 2023, Revised January 20, 2023. Project No. 15401.002.000.

2.0 FINDINGS

2.1 GEOLOGY AND SEISMICITY

2.1.1 Regional and Local Geology

The Site is located within the Coast Ranges geomorphic province of California. The Coast Ranges province is typified by a system of northwest-trending, fault-bounded mountain ranges, and intervening alluvial valleys. Bedrock in the Coast Ranges consists of igneous, metamorphic, and sedimentary rocks that range in age from Jurassic to Pleistocene. The present physiography and geology of the Coast Ranges are the result of deformation and deposition along the tectonic boundary between the North American plate and the Pacific plate. Plate boundary fault movements are largely concentrated along the well-known fault zones, which in the area include the San Andreas, Hayward, and Calaveras faults, as well as other lesser-known faults.

According to a published geologic map by Wagner and Guiterrez, 2017 (Figure 3), the Site is mapped to a Holocene alluvial fan and landslide deposits, and the surrounding Mt. Burdell consists of schist and semi-schist bedrock. Rice, 1974, corroborates these findings as he mapped the Site as having an existing rock debris flow landslide that has moved downslope by flow or creep processes, colluvium, and alluvium. Rice mentions that the landslides on this part of Mt. Burdell are roughly planar and occur parallel to the slope surface. Figure 4 shows historical landslide region depicted by Rice and the smaller mapped debris flow by Kleinfelder (1999).

2.1.2 Regional Faulting And Seismicity

The Site is located in a seismically active area that contains numerous faults. Small earthquakes occur every year in the San Francisco Bay Area and larger earthquakes have been recorded and can be expected to occur in the future. Active faults are cataloged and mapped by the United States Geological Survey (USGS) in the Quaternary Fault and Fold Database of the United States. An active fault is defined by the California Geological Survey as one that experienced surface displacement within Holocene time (about the last, 7,000 years) (CGS, 2018). Figure 5 shows the approximate locations of known active faults, along with other Quaternary faults based on the USGS Quaternary Fault and Fold Database, as well as significant historical earthquakes recorded within the San Francisco Bay Area region.

To identify nearby faults that are capable of generating strong seismic ground shaking at the Site, we utilized the USGS Earthquake Hazard Toolbox and the 2018 National Seismic Hazard Model (NSHM) to perform a disaggregation of the seismic hazard at the peak ground acceleration (PGA) and at spectral periods up to 5 seconds for a return period of 2,475 years. The resulting faults are listed in Table 2.1.2-1.

TABLE 2.1.2-1: Faults Considered Capable of Producing Strong Ground Shaking at the Site*

SOURCE NAME	RUPTURE DISTANCE, R_{RUP} (mi)	MOMENT MAGNITUDE, M_W
Hayward (North) (0)	6.3	7.40
San Andreas (North Coast) (3)	13.1	7.96
Rodgers Creek - Healdsburg (1)	7.5	6.89
Bennett Valley (0)	8.8	6.5

*Based on USGS Earthquake Hazard Toolbox: NSHM Conterminous U.S. 2018

These results represent known fault sources contributing at least 1 percent to the seismic hazard at the Site based on the disaggregation discussed above. The rupture distances (R_{RUP}) and mean moment magnitudes (M_W) listed are based on values assigned according to the 2018 NSHM, and the numbers in parentheses after the fault names correspond to fault subsections assigned by the NSHM. Note that the above fault table is not an exhaustive list and other faults in the region may generate seismic shaking at the project Site.

2.2 FIELD EXPLORATION

Prior to conducting the field exploration, we notified Underground Service Alert and retained the services of a private utility locator to clear the exploratory locations of existing utilities. Additionally, we obtained drilling permits from Marin County.

On July 25, 2024, we performed a preliminary geotechnical investigation that included the logging of an excavated test pit and drilled borings. Figure 2 shows the approximate location of our explorations.

2.2.1 Test Pit

We retained the services of a subcontractor to excavate one test pit northeast of the proposed development, as shown in Figure 2. A rubber-tired backhoe with a 2-foot-wide bucket was used to excavate the test pit. We logged the soil type, location, and uniformity of the subsurface conditions. The test pit was excavated to a maximum depth of approximately 4 feet below existing grade. We obtained bulk soil samples from the test pit using hand-sampling techniques.

We used the field logs to develop the report logs in Appendix A. The logs depict subsurface conditions at the exploration locations for the date of exploration; however, subsurface conditions may vary with time. Following field logging and sample collection, the test pit excavation was loosely backfilled with the excavated material.

2.2.2 Borings

We retained the services of a subcontractor to advance three boring with a CME-75 truck-mounted drill using solid-flight auger and mud-rotary methods. We advanced the borings until bedrock was encountered.

We collected soil samples at frequent depth intervals using either a 3-inch outside-diameter (O.D.) California-type split-spoon sampler fitted with 6-inch-long liners, or a 2-inch O.D. standard penetration test (SPT) split-spoon sampler. We advanced the samplers with a 140-pound hammer with a 30-inch drop, employing an automatic-trip hammer system. We recorded the penetration of the sampler as the number of blows needed to drive the sampler 18 inches in 6-inch increments. The boring log shows the number of blows required for the last foot of penetration, or the number of blows per depth of penetration for samples that met driving refusal. We did not convert the blow counts depicted on the boring log using any correction factors.

The boring logs are included in Appendix B. The log depicts interpreted subsurface conditions within the boring at the time the exploration was conducted. The stratification lines on our log represent the approximate boundaries between soil types and the transitions may be gradual. Subsurface conditions at other locations may differ from the conditions noted at these boring locations.

2.3 SURFACE CONDITIONS

The Site is currently occupied by a three-story, steel-framed office building constructed in 2000 and a parking lot with landscaped areas. The built area is in a low-lying valley that is bordered by Mt. Burdell to the east, west, and north. There has been historical grading associated with the existing development. There is approximately 24 feet of topographic relief (Elevation 44 to Elevation 20 (WGS, 1984)) trending from the northwest to the southeast of the Site (TWM Architects & Planners, 2000).

The steepness of the adjacent hills appears to be approximately 3:1 (horizontal:vertical) in the immediate vicinity of the Site. A retaining wall borders the northern portion of the building and was constructed for “slope stabilization” purposes according to TWM Architects & Planners (2000). Directly behind the retaining wall, there is a lined concrete subdrain located on the drainage easement that directs water away from the historically identified debris flow to Meadow Creek Road and Wood Hollow Drive.

2.4 SUBSURFACE CONDITIONS

The main geologic units encountered in our subsurface exploration are described below. Consult the Site Plan and exploration logs for specific subsurface conditions at each location.

We encountered 4 to 7 feet of undocumented fill throughout the parking lot in our borings. The fill consists of stiff to hard gravelly lean clay with varying amounts of sand. In TP-1, we only observed approximately a 1-foot-thick layer of fill consisting of sandy lean silt with gravel.

Below the fill, we encountered landslide deposits from historic slope instability and colluvium deposits from slope-washed sediments up to approximately 15 feet bgs in Borings 1-B1 and 1-B2. In 1-B3, we also encountered alluvial deposits at 20 feet bgs extending down to the top of rock. The colluvium and alluvium stratum generally consist of medium stiff to hard sandy lean clay with gravel and gravelly lean clay with sand; however, the alluvial deposits tended to have more rounded coarse-grained constituents.

Underlying the soil deposits, we encountered weak moderately to highly weathered sandstone and greenstone. While the area is geologically mapped to semi-schist, phyllite, and schist, we have encountered similar bedrock material in our previous explorations within the vicinity of Mt. Burdell (ENGEO, 2023).

2.5 GROUNDWATER

We did not observe static groundwater in any of our subsurface explorations. We observed perched groundwater conditions in Borings 1-B2 and 1-B3 at a depth of 5 feet bgs. Perched groundwater likely has infiltrated the fill layer and ponded on native clay layers.

TABLE 2.5-1: Groundwater Observations

SOURCE	APPROXIMATE DEPTH TO PERCHED GROUNDWATER (feet)
1-B1	N/A
1-B2	5
1-B3	5

Review of monitoring wells using GeoTracker (ca.gov), wells approximately 1000 feet to the south measured a static groundwater table approximately 9 ½ feet bgs.

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

3.0 PRELIMINARY DISCUSSIONS AND CONCLUSIONS

We evaluated the Site with respect to known geologic and other hazards common to the area. The primary hazards and the risks associated with these hazards with respect to the planned development are discussed in the following sections of this report which include landslides, compressible soil, undocumented existing fill, perched groundwater, and expansive soil.

3.1 LANDSLIDES

Several areas adjacent to the parcel have been evaluated by USGS, Rice (1974), and Kleinfelder (1999) as susceptible to debris flows and landslides, as shown in Figure 4. According to Kleinfelder (1999), there has been evidence that portions of the overall sliding mass have been re-activated in the last 200 years. This is characterized by abrupt eroded ground breaks, concavities, and hummocky topography. It is likely that landslides and debris flows will continue to be shed from the steep slopes above the project into the low-lying areas. Although slope instability may be a significant hazard, it can generally be mitigated through proper grading procedures.

Some feasible repair concepts for landslides include:

- Partial or full landslide removal and replacement with a keyway-supported engineered slope.
- Filling along toe of the slope to create a buttress and catchment area.
- Catchment retaining walls to dissipate raveled landslide debris.
- Structural solutions to retain or strengthen weak landslide material.

In general, it is possible to reduce construction risk by taking measures to stabilize the slope throughout construction using methods such as dewatering the slope, buttressing the landslide toe, and unloading the landslide crest. In contrast, construction methods that decrease slope stability may increase construction risk, such as excavating cuts near the landslide toe, adding mass to the landslide crest, or allowing additional water to enter the slope.

Landslide mitigation measures implemented for the existing Site improvements (e.g., retaining wall, subdrains, and lined concrete surface drains) may be inadequate for the purpose of the proposed residential development. If reuse of these implementations is desired, we can further evaluate their integrity and efficacy in the design-level geotechnical report. Additionally, a supplemental investigation for this development is required to determine the approximate extents of historical and existing landslides that may impact the structures.

Section 4.6 outlines the requirements for possible cut and fill slopes if a catchment berm or keyway is constructed.

3.2 COMPRESSIBLE SOIL (COLLUVIUM, LANDSLIDE DEBRIS)

Compressible soil is subject to settlement when a new loading scenario is introduced by structures, earthworks, or equipment. The amount of settlement is dependent on the magnitude and duration of the applied load, the shape and size of the applied load area, depth, thickness, and stress history of the compressible soil. The time required for primary settlement to occur is highly dependent on the mode of settlement, moisture content, and/or stiffness of the deposit. Consequently, sandy soil will settle almost immediately, whereas clayey soil will settle much more slowly.

Based on review of the boring logs, the current development placed fill material directly over native slide debris and colluvium. The subsurface consists of predominantly lean clay with sand mixture that is primarily stiff to very stiff. Based on our knowledge and experience, it is our opinion that a portion of settlement under new fill or structural loads will occur during construction and that the remaining settlement can be accommodated by designing the structural foundation to withstand some differential settlement.

To minimize settlement of the softer colluvium due to building loads or engineered fill loads, corrective grading measures that include the removal of compressible materials down to a non-yielding material or bedrock may be recommended. Laboratory testing and additional analysis should be performed in the design-level exploration to confirm the magnitude and extent of potentially compressible material and the potential settlement.

3.3 UNDOCUMENTED EXISTING FILL

The existing Site improvement grading plans indicate fills approximately 10 feet thick underly the Site. Our field explorations identified areas of the Site underlain by existing undocumented fills at least 7 feet thick. The fills were encountered in all borings and test pits. Poorly compacted undocumented fills are susceptible to excessive total and differential settlement, especially under new fill or building loads. We include recommendations for fill removal and recompaction in Section 4.0.

3.4 SHALLOW PERCHED GROUNDWATER

Based on groundwater levels previously discussed in Section 2.5., we encountered a shallow perched groundwater table in some of the borings. This perched water has the potential to impact temporary construction, excavations, and other underground construction. Regional dewatering operations may not be necessary and industrial sump pumps may be adequate to locally pump subsurface water encountered during excavation operations. We recommend that the contractor installing the subsurface utilities plan to perform some localized potholing prior to construction to determine if the groundwater will affect their improvements. Moreover, we provide recommendations in Section 5.4 to mitigate excessively wet ground at the base of excavations, if encountered.

In addition to the above considerations, shallow perched groundwater can:

- Cause moisture damage to sensitive floor coverings
- Transmit moisture vapor through slabs causing excessive mold/mildew build-up, fogging of windows, and damage to computers and other sensitive equipment.
- Cause premature pavement failure if hydrostatic pressures build-up beneath the section.

We provide recommendations to reduce these effects in Section 5.1.2.

3.5 EXPANSIVE SOIL

Based on the laboratory test results of soil at the Site, the clayey deposits have a plasticity index in the low to high 20s, which is an indication of moderately expansive behavior when wetted.

Expansive soil shrinks and swells as a result of moisture changes, which can cause heaving and cracking of slabs-on-grade, pavements, and structures founded on shallow foundations. Conventional grading operations, incorporating fill placement specifications tailored to the expansive characteristics of the soil, and use of a post-tensioned mat foundation are common, generally cost-effective measures to address the expansive potential of the near-surface soil. Preliminary grading recommendations for compaction of expansive soil at the Site are included in Section 4.0. Preliminary foundation design recommendations are provided in Section 5.0.

Additionally, successful construction on expansive soil requires special attention during grading. It is imperative to keep exposed soil moist with occasional sprinkling. If the soil is dry, it is extremely difficult to remoisturize without excavation, moisture conditioning, and recompaction.

3.6 SEISMIC HAZARDS

Potential seismic hazards resulting from a nearby moderate to major earthquake include ground rupture, also called surface faulting, ground shaking, soil liquefaction, lateral spreading, and densification. Based on topographic and lithologic data, risk from regional subsidence/uplift is negligible at the Site. The following sections present a discussion of these hazards as they apply to the Site.

3.6.1 Ground Rupture

The Site is not located within a State of California Earthquake Fault Hazard Zone and no known faults cross the Site. Therefore, ground rupture is unlikely at the property.

3.6.2 Ground Shaking

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

3.6.3 Liquefaction

Soil liquefaction results from loss of strength during cyclic loading, such as imposed by earthquakes. Soil most susceptible to liquefaction is clean, loose, saturated, uniformly graded, fine-grained sand. According to our exploration data, the soil on Site consists of sandy lean clay with gravel or gravelly lean clay with sand over bedrock. Based on these conditions, the potential for liquefaction at the Site is negligible during seismic shaking.

3.6.4 Lateral Spreading

Lateral spreading is a failure within a nearly horizontal soil zone (due to liquefaction) that causes the overlying soil mass to move toward a free face or down a gentle slope. Generally, effects of lateral spreading are most significant at the free face or the crest of a slope and diminish with distance from the slope. Because of the negligible potential for liquefaction at the Site, the potential for lateral spreading at the Site is also negligible.

3.6.5 2022 California Building Code (CBC) Seismic Parameters

The 2022 CBC utilizes seismic design criteria established in the ASCE/SEI Standard “Minimum Design Loads and Associated Criteria for Buildings and Other Structures,” (ASCE 7-16). Based on the subsurface conditions encountered, we preliminarily characterized the Site as Site Class C in accordance with Chapter 20 of ASCE 7-16.

In Table 3.5.5-1 below, we provide the CBC seismic parameters based on the United States Geological Survey’s (USGS) Seismic Design Maps for your use.

TABLE 3.5.5-1: 2022 CBC Seismic Information Latitude: 38.1250; Longitude: -122.5670

PARAMETER	DESIGN VALUE
Site Class	C
Mapped MCE_R spectral response accelerations for short periods, S_S (g)	1.5
Mapped MCE_R spectral response accelerations for 1-second periods, S_1 (g)	0.6
Site Coefficient, F_a	1.2
Site Coefficient, F_v	1.4
MCE spectral response accelerations for short periods, S_{MS} (g)	1.80
MCE spectral response accelerations for 1-second periods, S_{M1} (g)	0.84
Design spectral response acceleration at short periods, S_{DS} (g)	1.2
Design spectral response acceleration at 1-second periods, S_{D1} (g)	0.56
MCE Geometric Mean Peak Ground Acceleration, PGA_M (g)	0.68
Long period transition-period, T_L (sec)	12 sec

We recommend that we collaborate with the Structural Engineer-of-Record to further evaluate the effects of taking the exception on the structural design and identify the need for performing a Site-specific ground-motion hazard analysis. We can prepare a proposal for a Site-specific ground-motion hazard analysis, if requested.

4.0 PRELIMINARY EARTHWORK RECOMMENDATIONS

The following preliminary recommendations are for initial land planning and preliminary estimating purposes. Final recommendations regarding Site grading and foundation construction will be provided after design-level exploration has been undertaken.

4.1 SITE PREPARATION

Underground structures, such as buried pipes, septic tanks, and leach fields, if any, should be removed from the project Site entirely. All existing undocumented fill, vegetation, and soft or compressible soil should be removed, as necessary, for project requirements. The depth of removal of these materials should be determined by the geotechnical engineer’s qualified representative in the field at the time of grading. Evaluation of unsuitable deposits should be performed during grading by sampling and laboratory analyses.

Areas to receive fill or structures and those areas that serve as borrow for fill should be stripped of existing vegetation. In general, topsoil is estimated to be from 3 to 6 inches in thickness depending on location. Tree roots should be removed to a depth of at least 3 feet below finished grade in cut areas and 3 feet below original grade in fill areas. Subject to approval by the landscape architect, stripping's and organically contaminated soil that are not suitable for use as engineered fill may be used in landscape areas. Any topsoil that will be retained for future use in landscape areas should be stockpiled in areas where it will not interfere with the mass grading.

Within the development areas, excavations resulting from demolition and stripping that extend below final grades should be cleaned to firm undisturbed soil, as determined by the geotechnical engineer's representative. Following clearing and grubbing, all depressions in areas to be filled should be scarified, moisture conditioned, and backfilled with compacted engineered fill in accordance with Section 4.5.

4.2 UNDOCUMENTED FILL

We anticipate that existing undocumented fill is present within the footprint of the proposed development. We encountered the fill to depths up to 7 feet below the ground surface (bgs), likely related to the current development, as discussed in Section 3.3. The existing fill is unconsidered for support of structural and should be overexcavated, removed, and recompacted in accordance with Section 4.5.

4.3 ACCEPTABLE FILL

We anticipate native Site soil and existing on-site soil may be suitable as general engineered fill material, provided it is processed to remove concentrations of organic material, debris, and particles greater than 6 inches in maximum dimension then compacted in accordance with Section 4.5. Unsuitable materials and debris, including trees with their roots, should be removed from the project Site.

4.4 OVER-OPTIMUM SOIL MOISTURE CONDITIONS

The contractor should anticipate encountering excessively over-optimum (wet) soil moisture conditions due to the perched groundwater table, during winter or spring grading, or during or following periods of rain. Wet soil can make proper compaction difficult or impossible. Wet soil conditions can be mitigated by:

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

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

4.5 FILL COMPACTION

4.5.1 Grading in Structural Areas

After removal of soft soil and loose fill, the exposed non-yielding surface of all areas to receive minor fill, secondary slabs-on-grade, or pavements should be scarified to a depth of 8 inches, moisture conditioned, and recompacted to General Fill compaction specifications provided in Table 4.5.1-1, providing adequate bonding with the initial lift of fill. All fill should be placed in thin compacted lifts that do not exceed 10 inches or the depth of penetration of the compaction equipment used, whichever is less. Test procedures should be determined in accordance with ASTM D1557. We will collect additional samples during Site grading and transport them to our laboratory for compaction curve testing. We recommend the following compaction and moisture content requirements for the placement and compaction of engineered fills.

TABLE 4.5.1-1: Compaction Specifications

LOCATION	MINIMUM RELATIVE COMPACTION	MINIMUM MOISTURE CONTENT (percent above optimum)
General Fill (Site Soil)	90	4
Low Plasticity Import Fill	90	2
Aggregate Base (AB)	95	0

Pavement subgrade soil should be in a stable, non-pumping condition at the time aggregate base is placed and compacted. Proof-rolling with a heavy wheel-loaded piece of construction equipment should be implemented. Yielding materials should be appropriately mitigated, with suitable mitigation measures developed in coordination with the client, contractor, and our representative.

4.5.2 Underground Utility Backfill

Project consultants involved in utility design should specify pipe bedding materials. We recommend that utility trench backfilling be done under our observation. Trench backfill in structural areas should be placed and compacted in accordance with Table 4.5.1-1.

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

Jetting of backfill is not an acceptable means of compaction. Controlled density fill is also suitable for pipe zone and trench zone backfill.

4.5.3 Landscape Fill

We recommend processing, placing, and compacting fill in landscaped areas in accordance with the “General Fill” material in Table 4.5.1-1, except it should be compacted to at least 85 percent relative compaction.

4.6 GUIDELINES FOR GRADED SLOPES

In general, the following slope gradient guidelines may be applied for preliminary grading design of both permanent cut and fill slopes. The contractor is responsible to construct temporary construction slopes in accordance with Cal/OSHA requirements. Slopes steeper than 3:1 (horizontal:vertical) should be constructed with drainage benches at widths and intervals as recommended in the current California Building Code.

TABLE 4.6-1: Slope Specifications

ALLOWABLE SLOPE GRADIENT (horizontal:vertical)	MAXIMUM ALLOWABLE SLOPE HEIGHT (feet)	
	GENERAL FILL	BEDROCK CUT
2:1	10	10
2½:1	15	20
3:1	>15	>20

Depending on materials used to construct fill slopes or rebuild cut slopes, it may be necessary to incorporate additional slope stabilization techniques such as the use of geogrid reinforcement within the slope to enhance long-term stability. Graded cut and fill slopes exceeding 30 feet in height should include benches and/or concrete ditches, as designed by a civil engineer.

4.7 SITE DRAINAGE

The project civil engineer is responsible for designing surface drainage improvements. With regard to geotechnical engineering issues, we recommend that finish grades be sloped away from buildings and pavements to the maximum extent practical to reduce the potentially damaging effects of expansive soil. The latest California Building Code Section 1804.4 specifies minimum slopes of 5 percent away from foundations. Where lot lines or surface improvements restrict meeting this slope requirement, we recommend that specific drainage requirements be developed. As a minimum, we recommend the following.

1. Discharge roof downspouts into closed conduits and direct away from foundations to appropriate drainage devices.
2. Consider the use of rear lot surface drainage collection systems to reduce overland surface drainage from back to front of lot.

4.8 STORMWATER BIORETENTION AREAS

Infiltration testing was not included in our scope as part of this geotechnical exploration. Based on Site soil and the groundwater table encountered, the Site is likely unsuitable for infiltration.

If bioretention areas are planned at grade, we recommend that, when practical, they be placed a minimum of 5 feet away from the building and other structural Site improvements, such as streets, retaining walls, and sidewalks/driveways. When this is not practical, bioretention areas located within 5 feet of structural Site improvements can either:

1. Be constructed with structural side walls capable of withstanding the loads from the adjacent improvements, or
2. Incorporate filter material compacted in accordance with Section 4.5. and a waterproofing system designed to reduce the potential for moisture transmission into the subgrade soil beneath the adjacent improvement.

In addition, Site improvements located adjacent to bioretention areas that are underlain by baserock, sand, or other imported granular materials, should be designed with a deepened edge that extends to the bottom of the imported material underlying the improvement.

Where adjacent Site improvements include streets steeper than 3 percent or design elements that will experience lateral loads (such as from impact or traffic), additional design considerations may be required. In addition, although not recommended, if trees are to be planted within bioretention areas, HDPE Tree Boxes that extend below the bottom of the bioretention system should be installed to reduce potential impact to subdrain systems that may be part of the bioretention area design. For this condition, the waterproofing system should be connected to the HPDE Tree Box with a waterproof seal.

5.0 PRELIMINARY FOUNDATION DESIGN

In order to accommodate the potentially expansive soil, we recommend the residential buildings be supported on a post-tensioned mat, as discussed below.

5.1 POST-TENSIONED MAT FOUNDATION

We recommend that the proposed residential structures be supported on post-tensioned (PT) mat foundations bearing on engineered fill. On a preliminary basis, we recommend that PT mats be a minimum of 10 to 12 inches thick or greater and have a thickened edge at least 2 inches greater than the mat thickness. The structural engineer should determine the actual PT mat thickness using the geotechnical recommendations in the design-level geotechnical report. We recommend that the thickened edge be at least 12 inches wide.

PT mats may be designed for an average allowable bearing pressure of up to 1,000 pounds per square foot (psf) for dead-plus-live loads with maximum localized bearing pressures of 1,500 psf at column or wall loads. Allowable bearing pressures can be increased by one-third for wind or seismic loads.

5.1.1 Building Pad Preparation

The building pads should be uniform. For planning purposes, we recommend the pad surface should be moisture conditioned and compacted in accordance with General Fill in Section 4.5 prior to foundation construction; this moisture conditioning and compaction should be checked by our representative. The subgrade should not be allowed to dry prior to concrete placement.

5.1.2 Slab Moisture Vapor Reduction

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

1. Install a vapor retarder membrane directly beneath the slab. Seal the vapor retarder at all seams and pipe penetrations. Vapor retarders shall conform to Class A vapor retarder in accordance with ASTM E1745, latest edition, “Standard Specification for Plastic Water Vapor Retarders used in Contact with Soil or Granular Fill under Concrete Slabs.”

2. Concrete shall have a concrete water-cement ratio of no more than 0.50.
3. Provide inspection and testing during concrete placement to check that the proper concrete and water-cement ratio are used.

5.1.3 Shallow Foundation Lateral Resistance

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

- Passive Lateral Resistance: 250 pcf
- Coefficient of Friction: 0.30

5.2 PRELIMINARY RETAINING WALL RECOMMENDATIONS

Unrestrained walls constructed on level and sloped foregrounds should be designed for active lateral fluid pressure as provided below.

TABLE 5.2-1: Active Earth Pressure (Drained)

BACKFILL SLOPE CONDITION	ACTIVE PRESSURE (pcf)
Level	40
3:1	45
2:1	50

Passive pressures acting on foundations and shear keys may be assumed as 250 pcf, provided that the area in front of the retaining wall is level for a distance of at least 10 feet or three times the depth of foundation and keyway, whichever is greater. The upper 1 foot of soil should be excluded from passive pressure computations, unless it is confined by pavement or a concrete slab. The friction factor for sliding resistance may be assumed as 0.30. On a preliminary basis, the retaining wall footings may be planned using an allowable bearing pressure of 2,000 psf in firm native materials or fill. The footings should be at least 24 inches below lowest adjacent grades.

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

5.2.1 Retaining Wall Drainage

Either graded rock drains or geosynthetic drainage composites should be constructed behind the retaining walls to reduce hydrostatic lateral forces. For rock drain construction, we recommend two types of rock drain alternatives.

1. A minimum 12-inch-thick layer of Class 2 Permeable Filter Material (Caltrans Specification 68-2.02F) placed directly behind the wall, or
2. A minimum 12-inch-thick layer of washed, crushed rock with 100 percent passing the ¾-inch sieve and less than 5 percent passing the No. 4 sieve. Envelop rock in a minimum 6-ounce, non-woven geotextile filter fabric.

For both types of rock drains:

1. The rock drain should be placed directly behind the walls of the structure.
2. The rock drains should extend from the wall base to within 12 inches of the top of the wall.
3. A minimum of 4-inch-diameter perforated pipe (glued joints and end caps) should be placed at the base of the wall, inside the rock drain and fabric, with perforations placed down.
4. The pipe should be placed at a gradient at least 1 percent to direct water away from the wall by gravity to a drainage facility.

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

5.2.2 Backfill

Backfill behind the retaining walls should be placed and compacted in accordance with Section 4.5 as low-plasticity import fill. Use light compaction equipment within 5 feet of the wall face. If heavy compaction equipment is used, the walls should be temporarily braced to avoid excessive wall movement.

6.0 PRELIMINARY PAVEMENT DESIGN

6.1.1 Flexible Pavement

For preliminary planning purposes, a resistance value (R-value) of 5 was selected. We developed the following recommended pavement sections using Topic 633 of the 2021 Caltrans Highway Design Manual (including the asphalt factor of safety), and traffic indices varying from 5 to 7, presented in the table below.

TABLE 6.1.1-1: Preliminary Hot Mix Asphalt Concrete Pavement Sections

TRAFFIC INDEX	SECTION	
	HOT MIX ASPHALT (inches)	CLASS 2 AGGREGATE BASE (inches)
5	3	10
6	3 ½	13
7	4	16

The civil engineer should determine the appropriate traffic indexes based on the estimated traffic loads and frequencies.

These sections are for estimating purposes only. Actual sections to be used should be based on R-value tests performed on samples of actual subgrade materials recovered at the time of grading. Pavement construction and all materials should comply with the requirements of the Standard Specifications of the State of California Department of Transportation, civil engineer, and appropriate public agency.

7.0 DESIGN-LEVEL GEOTECHNICAL REPORT

This report presents findings, conclusions, and preliminary geotechnical recommendations intended for planning purposes only. Future design-level geotechnical explorations should be performed when development plans are finalized. We anticipate the design-level geotechnical report will include:

- Additional borings and test pits with soil sample collection to support design-level recommendations.
- Additional laboratory testing, including, but not limited to, moisture content, unit weight, plasticity index, gradation, strength, and corrosivity testing.
- Design-level assessment of slope stability for cut and fill slopes.
- Design recommendations for foundations.
- Design-level earthwork, improvement design, and construction recommendations.
- Development of corrective grading plans and cross sections.

8.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

This report presents geotechnical recommendations for preliminary design of the 100 Wood Hollow Drive residential project located in Novato, California. If changes occur in the nature or design of the project, we should be allowed to review this report and provide additional recommendations, if any. It is the responsibility of the owner to transmit the information and recommendations of this report to the appropriate organizations or people involved in design of the project, including but not limited to developers, owners, buyers, architects, engineers, and designers. The preliminary conclusions and recommendations contained in this report are solely professional opinions and are valid for a period of no more than 2 years from the date of report issuance.

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

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

Our services did not include excavation sloping or shoring, soil volume change factors, flood potential, or a geohazard exploration. In addition, our geotechnical exploration did not include work to determine the existence of possible hazardous materials. If any hazardous materials are encountered during construction, notify the proper regulatory officials immediately.

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

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

SELECTED REFERENCES

American Society of Civil Engineers (ASCE). 2016. Minimum Design Loads for Buildings and Other Structures, ASCE Standard, ASCE/SEI 7-16.

American Society of Testing and Materials (ASTM). Annual Book of Standards.

California Building Code. 2022.

California Geologic Survey. 2008. Special Publication 117A, Guidelines for Evaluating and Mitigating Seismic Hazards in California.

California Geological Survey. 2018. Earthquake Fault Zones – A Guide for Government Agencies, Property Owners / Developers, and Geoscience Practitioners for Assessing Fault Rupture Hazards in California: Special Publication 42 (SP42). Available at https://www.conservation.ca.gov/cgs/Documents/Publications/Special-Publications/SP_042.pdf

Cooper-Clark and Associates. 1978. Preliminary Geotechnical Investigation, Proposed 54-Acre Industrial Development, Nunes Ranch, Novato, California. July 7, 1978. Job No. 1950-C.

Division of Mines and Geology. 1997. Special Publication 117, Guidelines for Evaluation and Mitigating Seismic Hazards in California. Adopted March 13.

ENGEO. (2023). Geotechnical Exploration Report, Valley Oaks, Novato California. January 12, 2023, Revised January 20, 2023. Project No. 15401.002.000.

Herzog Geotechnical Consulting Engineers. 2001. Geotechnical Review, Deformation Analyses, San Marin Business Park, Area E Novato, California. October 24, 2001.

Kleinfelder. 1999. Geotechnical investigation, Office and Hotel Project, Eastern Portions of Parcel Adjacent to Redwood Boulevard, Novato, California. March 9, 1999.; Job No. 41-7475-01.

Kleinfelder. 1998. Preliminary Geologic Investigation, Parcels 1 and 2, Wood Hollow Drive and Redwood Boulevard, Novato, California. September 24, 1998. Project No. 41-7449-01.

Kleinfelder. 1999. Final Geotechnical Investigation, Office Building Project, Corner of Wood Hollow Drive and Redwood Boulevard, Novato, California. January 20, 1999. Project No. 41-7449-03.

Kleinfelder. 2001. Deformation Analyses, Area E, San Marin Business Park, Redwood Boulevard, Novato, California. October 19, 2001. Project No. 41-7449-02.

Post-Tensioning Institute. 2004. Design of Post-Tensioned Slabs-on-Ground, Third Edition.

Historical Aerials, www.historicaerials.com.

Structural Engineers Association of California (SEAOC), 1996, Recommended Lateral Force Requirements and Tentative Commentary.

SELECTED REFERENCES (Continued)

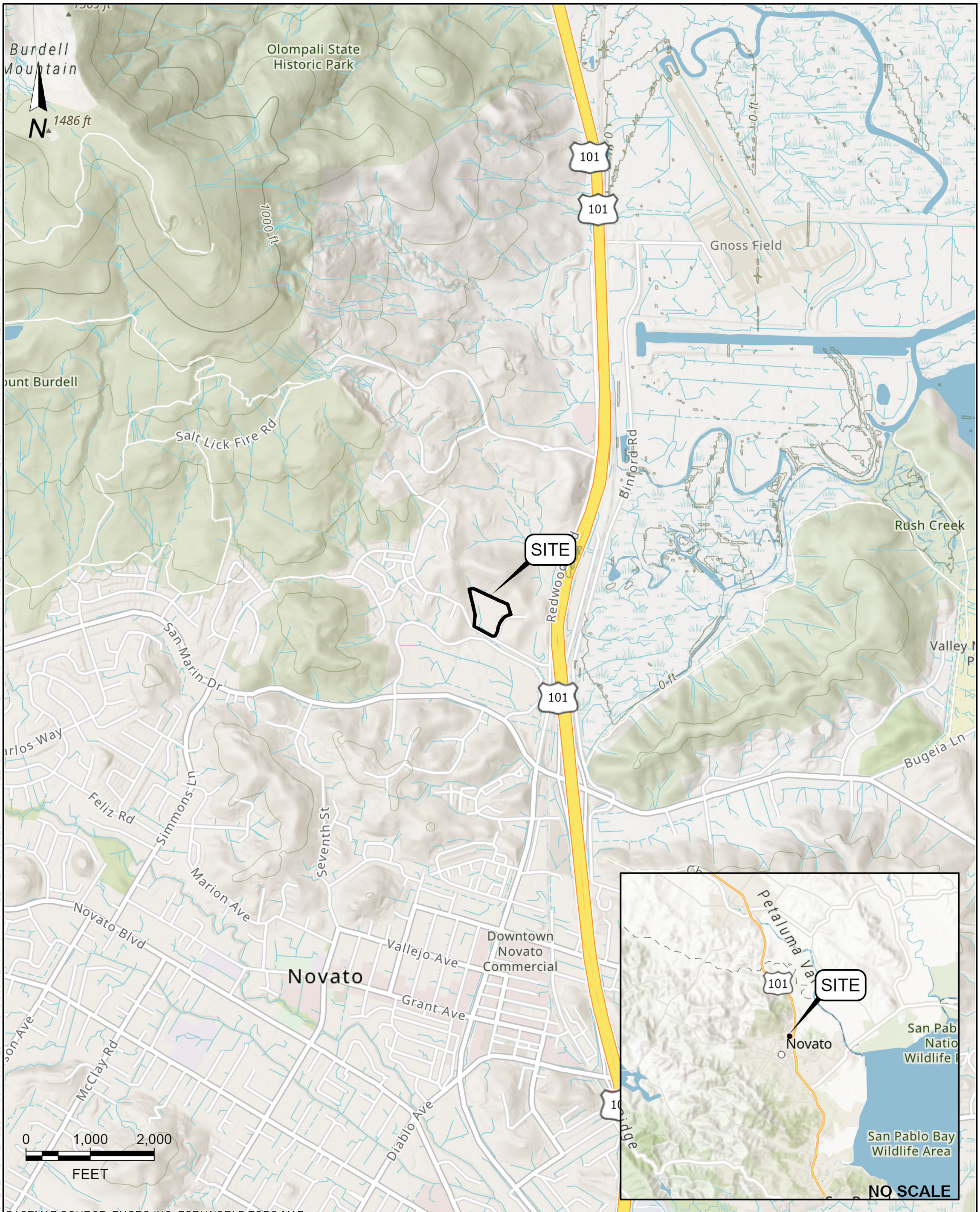
Rice, S.J., Smith, T.C., Strand, R.G., Wagner, D.L., Randolph-Loar, C.E., Witter, R.C., and Clahan, K.B., 2002, Geologic map of the Novato 7.5-minute quadrangle, Marin and Sonoma counties, California: A digital database., California Geological Survey, Preliminary Geologic Maps PGM-02-02, 1:24,000

U.S. Geological Survey (USGS). 2008. National Seismic Hazard Maps Fault Database.

USGS. 2017. Quaternary fault and fold database of the United States: U.S. Geological Survey website, https://earthquakes.usgs.gov/hazards/qfaults, accessed 01/19/2023 11:21 AM.

Wagner, D.L., Rice, S.R., Bezore, S.P., Randolph-Loar, C.E., Allen, J., and Witter, R.C., 2002, Geologic map of the Petaluma River 7.5-minute quadrangle, Marin and Sonoma counties, California: A digital database., California Geological Survey, Preliminary Geologic Maps PGM-02-04, 1:24,000

FIGURES



BASEMAP SOURCE: ENGEO INC, ESRI WORLD TOPO MAP

ENGEO
Expect Excellence

VICINITY MAP
100 WOOD HOLLOW DRIVE
NOVATO, CALIFORNIA

PROJECT NO. : 26288.000.001		FIGURE NO.
SCALE: AS SHOWN		1
DRAWN BY: NWC	CHECKED BY: TTB	

PATH: G:\DRAFTING\PROJECTS_26000 TO 27999\26288\26288000001\GEOTECHNICAL\GEX\26288000001_100WOODHOLLOWDRIVE – GEX.APRX
LAYOUT: 01. VICINITY 8.5 X 11 PORTRAIT USER: NCLOUGH

ORIGINAL FIGURE PRINTED IN COLOR

ENGEO Expect Excellence

SITE PLAN
100 WOOD HOLLOW DRIVE
NOVATO, CALIFORNIA

PROJECT NO.: 26288.000.001
SCALE: AS SHOWN
DRAWN BY: NWC
CHECKED BY: TTB
FIGURE NO. 2

PATH: G:\DRAFTING\PROJECTS_26000 TO 27999\26288\26288000001\GEOTECHNICAL\GEX\26288000001_100WOODHOLLOWDRIVE - GEX.APRX
LAYOUT: 02. SITE PLAN 8.5 X 11 PORTRAIT USER: NCLOUGH

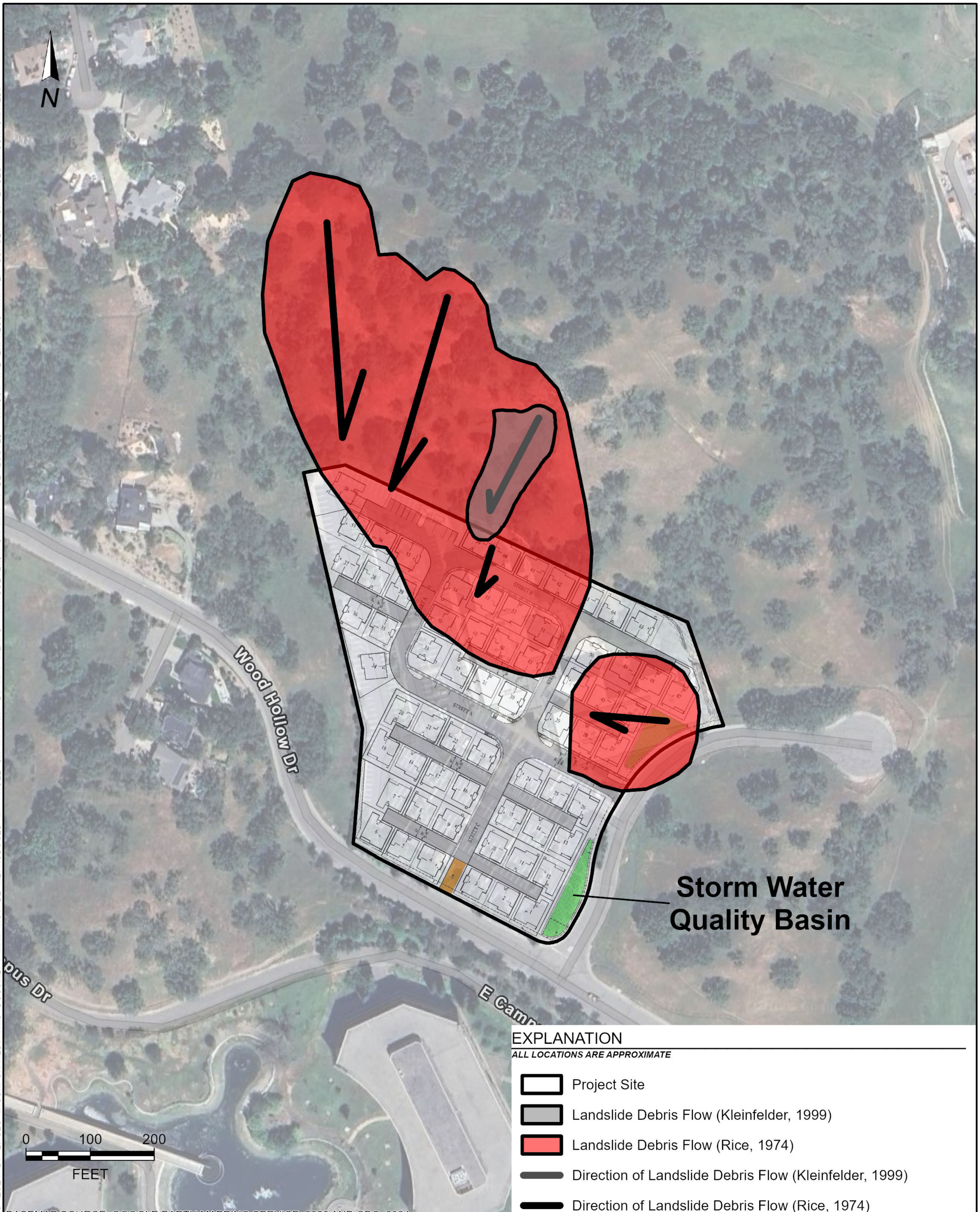
ORIGINAL FIGURE PRINTED IN COLOR

ENGEO
Expect Excellence

REGIONAL GEOLOGIC MAP
100 WOOD HOLLOW DRIVE
NOVATO, CALIFORNIA

PROJECT NO.: 26288.000.001
SCALE: AS SHOWN
DRAWN BY: NWC
CHECKED BY: TTB
FIGURE NO. 3

PATH: G:\DRAFTING\PROJECTS_26000 TO 27999\26288\26288000001\GEOTECHNICAL\GEX\26288000001_100WOODHOLLOWDRIVE - GEX.APRX
LAYOUT: 03. GEOLOGIC 8.5 X 11 PORTRAIT USER: NCLOUGH
ORIGINAL FIGURE PRINTED IN COLOR



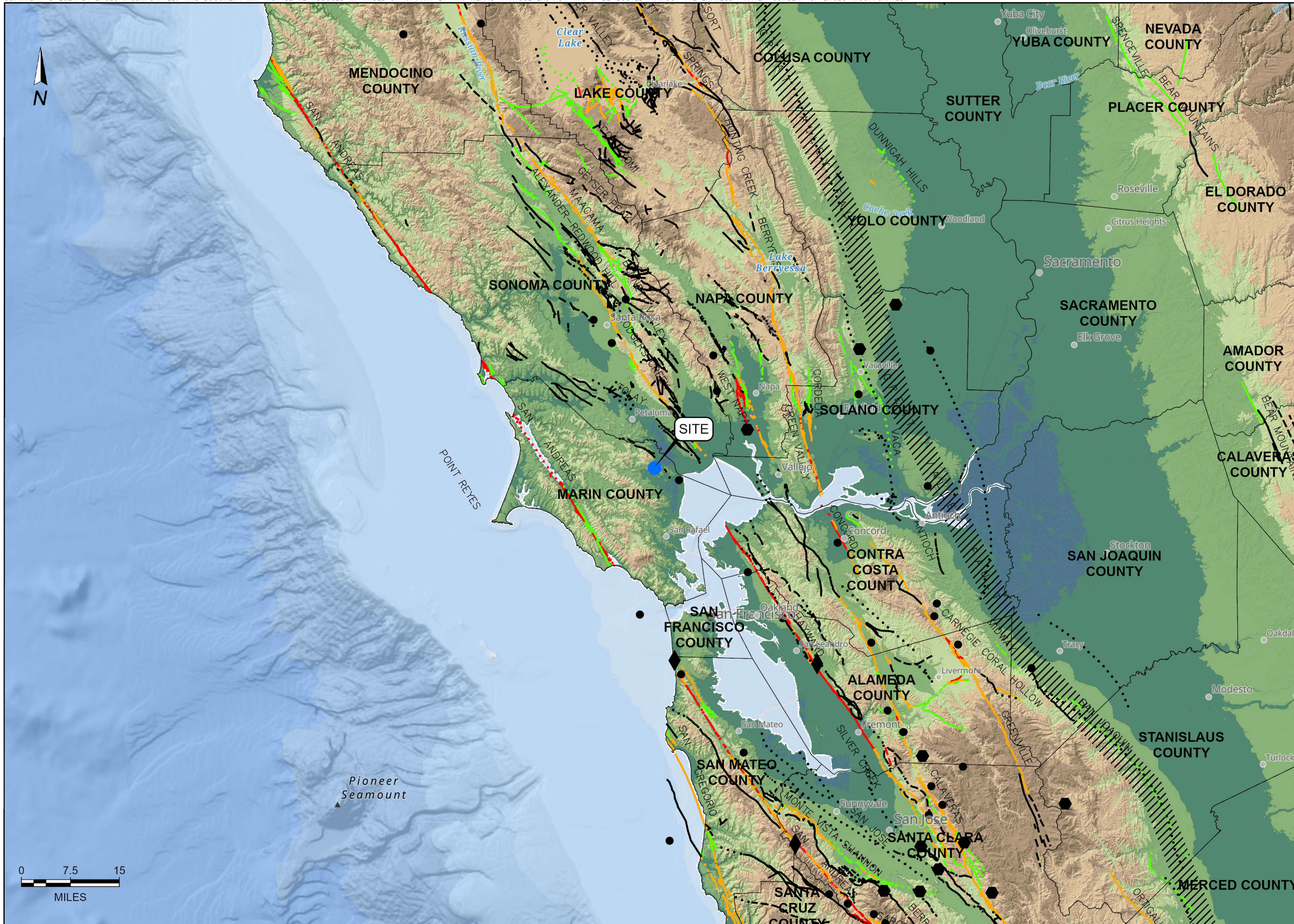
ENGEO
Expect Excellence

LANDSLIDE MAP
100 WOOD HOLLOW DRIVE
NOVATO, CALIFORNIA

PROJECT NO. : 26288.000.001	FIGURE NO. 4
SCALE: AS SHOWN	
DRAWN BY: NWC	CHECKED BY: TTB

PATH: G:\DRAFTING\PROJECTS_26000 TO 27999\26288\26288000001\GEOTECHNICAL\GEX\26288000001_100WOODHOLLOWDRIVE - GEX.APRX
LAYOUT: 04. LANDSLIDE MAP 8.5 X 11 PORTRAIT USER: NCLOUGH

ORIGINAL FIGURE PRINTED IN COLOR



EXPLANATION

- Project Site

HISTORIC EARTHQUAKE EPICENTERS

- Magnitude 5-6
- Magnitude 6-7
- Magnitude 7+
- Historic Blind Thrust Fault Zone

QUATERNARY FAULTS 2020

Based on time of most recent surface deformation

- Historical (<150 Years), Well Constrained Location
- Historical (<150 Years), Moderately Constrained Location
- Historical (<150 Years), Inferred Location
- Latest Quaternary (<15,000 Years), Well Constrained Location
- Latest Quaternary (<15,000 Years), Moderately Constrained Location
- Latest Quaternary (<15,000 Years), Inferred Location
- Latest Quaternary (<15,000 Years), Inferred Location
- Late Quaternary (<130,000 Years), Moderately Constrained Location
- Late Quaternary (<130,000 Years), Inferred Location
- Middle And Late Quaternary (<750,000 Years), Well Constrained Location
- Middle And Late Quaternary (<750,000 Years), Moderately Constrained Location
- Middle And Late Quaternary (<750,000 Years), Inferred Location
- Undifferentiated Quaternary(<1.6 Million Years), Well Constrained Location
- Undifferentiated Quaternary(<1.6 Million Years), Moderately Constrained Location
- Undifferentiated Quaternary(<1.6 Million Years), Inferred Location
- Class B (Various Age), Well Constrained Location
- Class B (Various Age), Moderately Constrained Location
- Class B (Various Age), Inferred Location

BASE MAP SOURCE:
CSUMB, ESRI, GARMIN, NATURALVUE, COUNTY OF MARIN, CALIFORNIA STATE PARKS, ESRI, TOMTOM, GARMIN, SAFEGRAPH, GEOTECHNOLOGIES, INC, METI/NASA, USGS, BUREAU OF LAND MANAGEMENT, EPA, NPS, USDA, USFWS, ESRI, GEBCO, GARMIN, NATURALVUE, COUNTY OF MARIN, CALIFORNIA STATE PARKS, ESRI, TOMTOM, GARMIN, SAFEGRAPH, FAO, METI/NASA, USGS, BUREAU OF LAND MANAGEMENT, EPA, NPS, USFWS
COLOR HILLSHADE IMAGE BASED ON THE NATIONAL ELEVATION DATA SET (NED) AT 30 METER RESOLUTION
U.S.G.S. QUATERNARY FAULT DATABASE, 2020
C.G.S. HISTORIC EARTHQUAKE DATABASE

ENGEO
Expect Excellence

REGIONAL FAULTING AND SEISMICITY MAP
100 WOOD HOLLOW DRIVE
NOVATO, CALIFORNIA

PROJECT NO.: 26288.000.001		FIGURE NO. 5
SCALE: AS SHOWN		
DRAWN BY: NWC	CHECKED BY: TTB	

APPENDIX A

TEST PIT LOGS

TEST PIT LOG 1-TP1

100 Hollow Wood Novato, CA 26288.000.001	Logged/Reviewed By: NI/BR Logged Date: 7/25/2024 Equipment: Backhoe	Lat: 38.125591 Long: -122.569083

Depth (Feet)	Description
0 – 1	SANDY LEAN SILT with GRAVEL (ML), light brown, dry, medium stiff, sand is fine-grained, gravel is subangular to angular and coarse PP: 1.0 – 1.75 [Fill]
1 – 4	GRAVELLY LEAN CLAY with SAND (CL), reddish brown to red, moist, very stiff to hard, low plasticity, somewhat cemented, sand is fine-grained, gravel is fine and subangular, extensive iron oxide staining [Residual Soil bedrock]
3	Becomes well cemented.
	End of test pit. No groundwater encountered.

APPENDIX B

KEY TO BORING LOGS
BORING LOGS

KEY TO BORING LOGS

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

GRAIN SIZES

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

	200		40		10		4		3/4 "		3"		12"	

SILTS AND CLAYS	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

RELATIVE DENSITY

SANDS AND GRAVELS	BLOWS/FOOT (S.P.T.)[1]
VERY LOOSE	0-4
LOOSE	4-10
MEDIUM DENSE	10-30
DENSE	30-50
VERY DENSE	OVER 50

CONSISTENCY

SILTS AND CLAYS	APPROXIMATE SHEAR STRENGTH (PSF)[2]
VERY SOFT	0-250
SOFT	250-500
MEDIUM STIFF	500-1,000
STIFF	1,000-2,000
VERY STIFF	2,000-4,000
HARD	> 4,000

MOISTURE CONDITION

DRY	Absence of moisture, dusty, dry to touch
MOIST	Damp but no visible water
WET	Visible freewater

LINE TYPES

Solid - Layer Break

Dashed - Gradational or approximate layer break

SAMPLER SYMBOLS

- Modified California (3-inch O.D.) Sampler
- California (2.5-inch O.D.) Sampler
- S.P.T. Split Spoon (2-inch O.D.) Sampler
- Shelby Tube
- Continuous Core
- Bag Samples
- Grab Samples
- No Recovery

GROUND-WATER SYMBOLS

- Groundwater level during drilling
- Stabilized groundwater level

NOTES

1. Standard Penetration Tests (S.P.T.) number of blows for a 140-pound hammer falling 30 inches to drive a 2-inch O.D. (1-3/8-inch I.D.) sampler, assuming 60% hammer efficiency
2. Approximate shear strength measured in field at time of drilling in units of pounds per square foot

ENGEO
Expect Excellence

100 Wood Hollow Drive

Soil Boring: 1-B1

Project Location:	100 Meadow Crest Rd, Novato, CA	Date Drilled:	07/25/2024	Logged By:	N. Inserra
Project No:	26288.000.001	Hole Depth:	15.5 ft	Reviewed By:	T. Bradford
Lat / long:	38.125638, -122.570438	Boring Diameter:	4 in	Drilling Contractor:	Hanlon Subsurface Exploration
Surface Elevation:	43 ft	Method:	Mud Rotary	Hammer Type:	140 lb. Auto Trip
Elevation Datum:	WGS84				

Depth (ft)	Elevation (ft)	Sample Graphic	Visual Classification and Remarks	Graphic Log	Water Levels	Uncorrected N-Value	Lab: Atterberg Limits (LL-PL-PI)	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Shear Strength (psf)	Compressive Strength (psf)	Pocket Penetrometer (tsf)	Torvane (psf)
0	43		**GRAVELLY LEAN CLAY WITH SAND** (**CL**), brown, moist [FILL]											
	40													
4			**SANDY LEAN CLAY WITH GRAVEL** (**CL**), reddish brown, hard, moist, fine-grained sand, angular fine-grained gravel [COLLUVIUM/LANDSLIDE DEBRIS]											
5						31		73	21.71	103.1		6064	4.5+	0.4
	35													
10														
			More cemented			86			11.7	112.9				
	30													
			GREENSTONE, dark greenish gray with brown, weak, moderately weathered, massive											
15						50/5"								

End of boring at approximately 15-1/2 ft below ground surface. Groundwater was not encountered during our exploration.

ENGEO
Expect Excellence

100 Wood Hollow Drive

Soil Boring: 1-B2

Project Location:	100 Meadow Crest Rd, Novato, CA	Date Drilled:	07/25/2024	Logged By:	N. Inserra
Project No:	26288.000.001	Hole Depth:	31 ft	Reviewed By:	T. Bradford
Lat / long:	38.125012, -122.570091	Boring Diameter:	4 in	Drilling Contractor:	Hanlon Subsurface Exploration
Surface Elevation:	34 ft	Method:	Mud Rotary	Hammer Type:	140 lb. Auto Trip
Elevation Datum:	WGS84				

Depth (ft)	Elevation (ft)	Sample Graphic	Visual Classification and Remarks	Graphic Log	Water Levels	Uncorrected N-Value	Lab: Atterberg Limits (LL-PL-PI)	Lab: Fines (%)	Lab: Moisture Content (%)	Lab: Dry Density (pcf)	Lab: Shear Strength (psf)	Lab: Compressive Strength (psf)	Lab: Pocket Penetrometer (tsf)	Lab: Torvane (psf)
			GRAVELLY LEAN CLAY WITH SAND (**CL**), brown, very stiff, moist, fine-grained sand, angular coarse gravel [FILL]											
						36	41-20-21						2.5	0.2
	30													
5			**GRAVELLY SILT WITH SAND** (**ML**), brown, very stiff, moist, fine-grained sand, angular coarse gravel		▽									
						24							2.5	0.6
			SANDY LEAN CLAY WITH GRAVEL (**CL**), reddish brown, very stiff, moist, 15% fine-grained sand, 10% gravel, iron-oxide staining [COLLUVIUM/LANDSLIDE DEBRIS]											
	25													
10						18			15.7					
			GRAVELLY LEAN CLAY WITH SAND (**CL**), reddish brown with olive gray, hard, moist, cemented, 10% fine-grained sand, 20% gravel											
	20													
15						61							4.5+	0.5
	15													

100 Wood Hollow Drive

Soil Boring: 1-B2

ENGEO Expect Excellence

Project Location:	100 Meadow Crest Rd, Novato, CA	Date Drilled:	07/25/2024	Logged By:	N. Inserra
Project No:	26288.000.001	Hole Depth:	31 ft	Reviewed By:	T. Bradford
Lat / long:	38.125012, -122.570091	Boring Diameter:	4 in	Drilling Contractor:	Hanlon Subsurface Exploration
Surface Elevation:	34 ft	Method:	Mud Rotary	Hammer Type:	140 lb. Auto Trip
Elevation Datum:	WGS84				

Depth (ft)	Elevation (ft)	Sample Graphic	Visual Classification and Remarks	Graphic Log	Water Levels	Uncorrected N-Value	Lab: Atterberg Limits (LL-PL-PI)	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Shear Strength (psf)	Compressive Strength (psf)	Pocket Penetrometer (tsf)	Torvane (psf)
			GRAVELLY LEAN CLAY WITH SAND (**CL**), reddish brown with olive gray, hard, moist, cemented, 10% fine-grained sand, 20% gravel			64							4.5	0.5
	10													
25						81							4.5+	1
			SANDSTONE, dark yellowish-brown, extremely weak to very weak, intensely weathered, massive			62							4	
	5													
30						50/5"								

End of boring at approximately 31 ft below ground surface. Ground water was encountered at 5 feet during our exploration.

100 Wood Hollow Drive

Soil Boring: 1-B3

ENGEO Expect Excellence

Project Location:	100 Meadow Crest Rd, Novato, CA	Date Drilled:	07/25/2024	Logged By:	N. Inserra
Project No:	26288.000.001	Hole Depth:	46 ft	Reviewed By:	T. Bradford
Lat / long:	38.124342, -122.569933	Boring Diameter:	4 in	Drilling Contractor:	Hanlon Subsurface Exploration
Surface Elevation:	27 ft	Method:	Mud Rotary	Hammer Type:	140 lb. Auto Trip
Elevation Datum:	WGS84				

Depth (ft)	Elevation (ft)	Sample Graphic	Visual Classification and Remarks	Graphic Log	Water Levels	Uncorrected N-Value	Lab: Atterberg Limits (LL-PL-PI)	Lab: Fines (%)	Lab: Moisture Content (%)	Lab: Dry Density (pcf)	Lab: Shear Strength (psf)	Lab: Compressive Strength (psf)	Lab: Pocket Penetrometer (tsf)	Lab: Torvane (psf)
			GRAVELLY LEAN CLAY WITH SAND (**CL**), black, very stiff, moist, fine-grained sand, angular to subangular fine- to medium grained gravel [FILL]											
	25													
						21	44-15-29		15.87	113.4		3595	2.5	1
5					▽	14							2	
	20		**SANDY LEAN CLAY WITH GRAVEL** (**CL**), reddish brown, hard, moist, fine-grained sand, iron oxide staining, calcite lenses [COLLUVIUM]			54							4.5+	
10			stiff, less cemented			27			17.3				2	0.5
	15													
			GRAVELLY LEAN CLAY WITH SAND (**CL**), reddish brown, hard, moist, fine-grained sand, subrounded fine- to medium grained gravel, iron oxide staining, calcite lenses											
15						45							4.5+	
	10													
			SANDY LEAN CLAY WITH GRAVEL (**CL**), reddish brown, hard, moist, coarse-grained sand, rounded fine-grained gravel [ALLUVIUM]											

100 Wood Hollow Drive

Soil Boring: 1-B3

Project Location:	100 Meadow Crest Rd, Novato, CA	Date Drilled:	07/25/2024	Logged By:	N. Inserra
Project No:	26288.000.001	Hole Depth:	46 ft	Reviewed By:	T. Bradford
Lat / long:	38.124342, -122.569933	Boring Diameter:	4 in	Drilling Contractor:	Hanlon Subsurface Exploration
Surface Elevation:	27 ft	Method:	Mud Rotary	Hammer Type:	140 lb. Auto Trip
Elevation Datum:	WGS84				

Depth (ft)	Elevation (ft)	Sample Graphic	Visual Classification and Remarks	Graphic Log	Water Levels	Uncorrected N-Value	Lab: Atterberg Limits (LL-PL-PI)	Lab: Fines (%)	Lab: Moisture Content (%)	Lab: Dry Density (pcf)	Lab: Shear Strength (psf)	Lab: Compressive Strength (psf)	Lab: Pocket Penetrometer (tsf)	Lab: Torvane (psf)
			SANDY LEAN CLAY WITH GRAVEL (**CL**), reddish brown, hard, moist, coarse-grained sand, rounded fine-grained gravel [ALLUVIUM]			61							4.5+	0.2
	5													
25			decrease in sand content			46			17.1				4.5+	1.5
	0													
			LEAN CLAY (**CL**), reddish brown with olive gray, hard, moist, trace rounded fine-grained gravel											
30						50							4.5+	0.9
	-5													
			GRAVELLY LEAN CLAY (**CL**), olive gray, hard, moist, subrounded fine to coarse gravel											
35						45							4	2
	-10													

100 Wood Hollow Drive

Soil Boring: 1-B3

Project Location:	100 Meadow Crest Rd, Novato, CA	Date Drilled:	07/25/2024	Logged By:	N. Inserra
Project No:	26288.000.001	Hole Depth:	46 ft	Reviewed By:	T. Bradford
Lat / long:	38.124342, -122.569933	Boring Diameter:	4 in	Drilling Contractor:	Hanlon Subsurface Exploration
Surface Elevation:	27 ft	Method:	Mud Rotary	Hammer Type:	140 lb. Auto Trip
Elevation Datum:	WGS84				

Depth (ft)	Elevation (ft)	Sample Graphic	Visual Classification and Remarks	Graphic Log	Water Levels	Uncorrected N-Value	Lab: Atterberg Limits (LL-PL-PI)	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Shear Strength (psf)	Compressive Strength (psf)	Pocket Penetrometer (tsf)	Torvane (psf)
			GRAVELLY LEAN CLAY (**CL**), olive gray, hard, moist, subrounded fine to coarse gravel			44							3	0.5
	-15													
45			**SANDSTONE**, yellowish brown, intensely to moderately weathered, massive			100/9"								

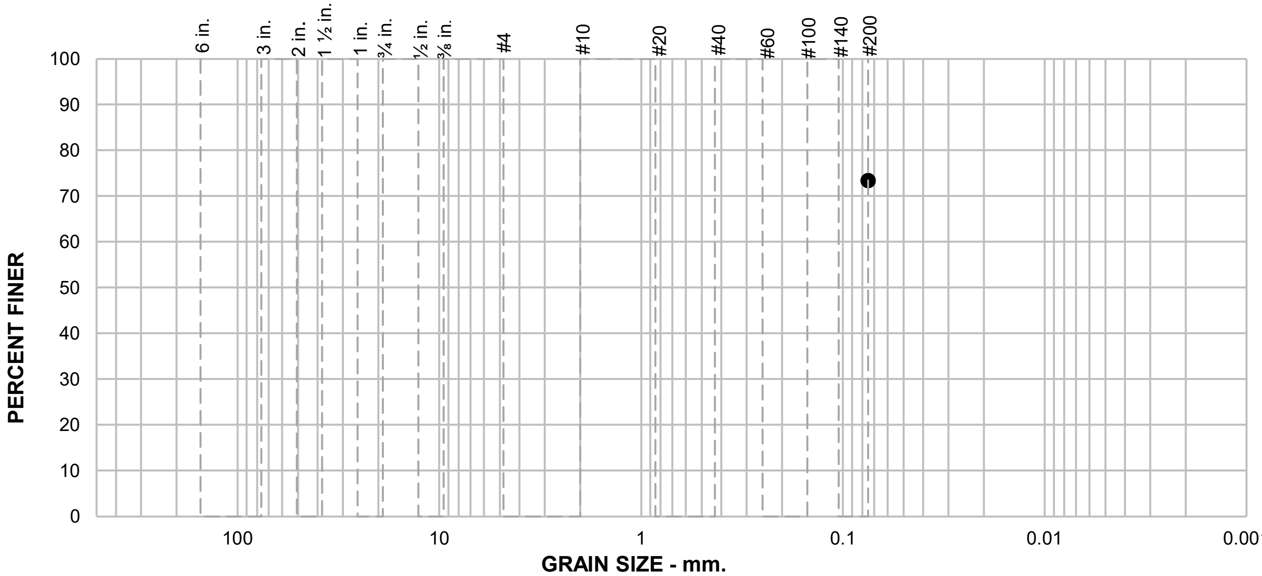
End of boring at approximately 46 ft below ground surface. Groundwater was encountered at 5 feet during our exploration.

APPENDIX C

LABORATORY TESTING

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D1140, Method B



SAMPLE ID: 1-B1@6-6.5

DEPTH (ft): 6-6.5

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
						73	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	73		

SOIL DESCRIPTION

See exploration logs

ATTERBERG LIMITS

PL = LL = PI =

COEFFICIENTS

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

CLASSIFICATION

USCS =

REMARKS

Soak time = 180 min
Dry sample weight = 171.6 g
Largest particle size ≥ No. 4 Sieve

* (no specification provided)

CLIENT: Newmark Management
PROJECT NAME: 100 Wood Hollow
PROJECT NO: 26288.000.001 PH001
PROJECT LOCATION: Novato, CA
REPORT DATE: 8/12/2024
TESTED BY: Y. Cabrales
REVIEWED BY: G. Criste

ENGEO Expect Excellence

MOISTURE CONTENT REPORT
ASTM D2216

SAMPLE ID	1-B2 @9-10.5	1-B3 @11-11.5	1-B3 @25.5-26					
DEPTH (ft.)	9-10.5	11-11.5	25.5-26					
METHOD A OR B	B	B	B					
MOISTURE CONTENT (%)	15.7	17.3	17.1					

ENGEO
Expect Excellence

CLIENT: Newmark Management
PROJECT NAME: 100 Wood Hollow
PROJECT NO: 26288.000.001 PH001
PROJECT LOCATION: Novato, CA
REPORT DATE: 8/8/2024
TESTED BY: Y. Cabrales
REVIEWED BY: G. Criste

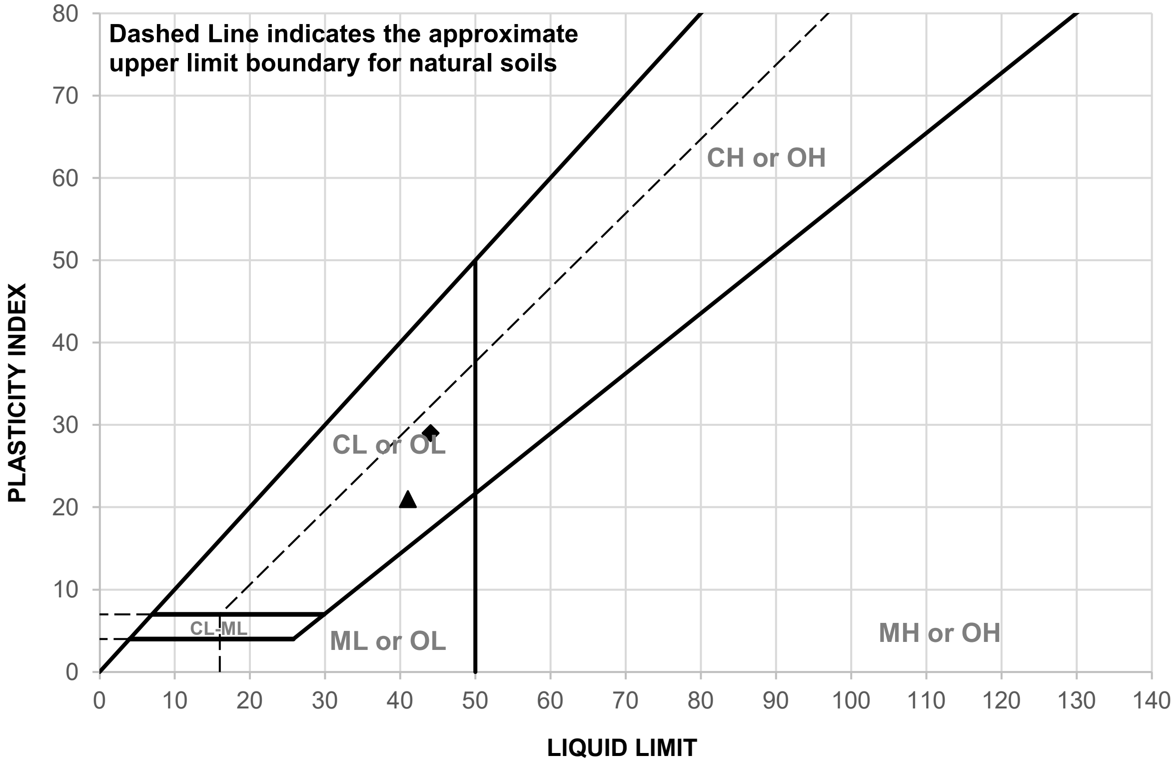
MOISTURE-DENSITY DETERMINATION REPORT
ASTM D7263

SAMPLE ID	1-B1 @12-12.5							
DEPTH (ft.)	12-12.5							
METHOD A OR B	B							
MOISTURE CONTENT (%)	11.7							
DRY DENSITY (pcf)	112.9							

ENGEO
Expect Excellence

CLIENT: Newmark Management
PROJECT NAME: 100 Wood Hollow
PROJECT NO: 26288.000.001 PH001
PROJECT LOCATION: Novato, CA
REPORT DATE: 8/8/2024
TESTED BY: Y. Cabrales
REVIEWED BY: G. Criste

LIQUID AND PLASTIC LIMITS TEST REPORT
ASTM D4318



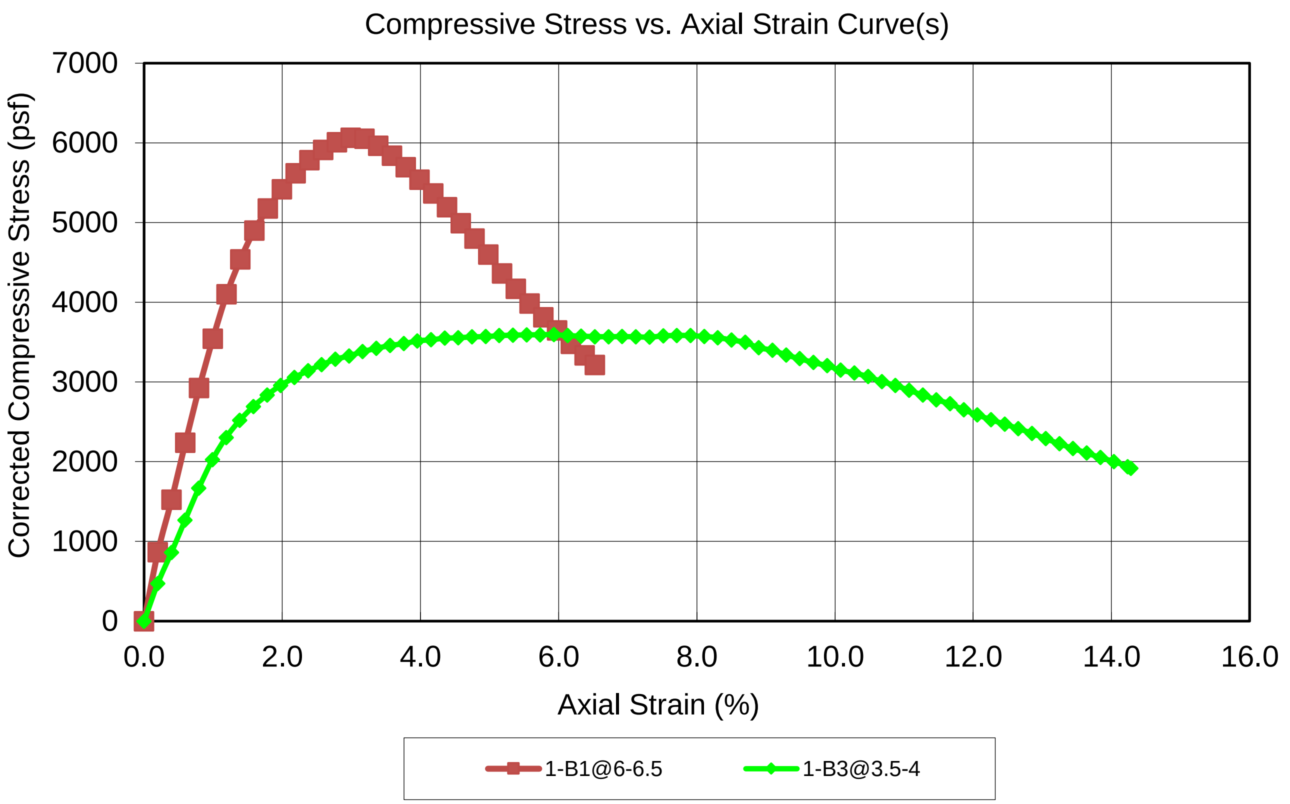
	SAMPLE ID	DEPTH (ft)	MATERIAL DESCRIPTION	LL	PL	PI
▲	1-B2@2.5-3	2.5-3	See exploration logs	41	20	21
◆	1-B3@3.5-4	3.5-4	See exploration logs	44	15	29

	SAMPLE ID	TEST METHOD	REMARKS
▲	1-B2@2.5-3	PI: ASTM D4318, Wet Method	
◆	1-B3@3.5-4	PI: ASTM D4318, Wet Method	

ENGEO
Expect Excellence

CLIENT: Newmark Management
PROJECT NAME: 100 Wood Hollow
PROJECT NO: 26288.000.001 PH001
PROJECT LOCATION: Novato, CA
REPORT DATE: 8/13/2024
TESTED BY: G. Criste
REVIEWED BY: D. Seibold

UNCONFINED COMPRESSION TEST REPORT
(ASTM D2166)



BEFORE TEST	SPECIMEN 1-B1@6-6.5	SPECIMEN 1-B3@3.5-4
Test Moisture Content (%)	21.71	15.87
Dry Density (pcf)	103.1	113.4
Saturation (%)	91.4	86.9
Void Ratio	0.65	0.50
Diameter (in)	2.411	2.398
Height (in)	5.020	5.058
Height-To-Diameter Ratio	2.08	2.11
TEST DATA		
Unconfined Compressive Strength (psf)	6064.46	3594.67
Undrained Shear Strength (psf)	3032.23	1797.33
Strain Rate (in/min)	0.050	0.051
Specific Gravity (ASSUMED)	2.720	2.720
Strain at Failure(%)	2.99	5.93
Test Remarks		

SPECIMEN	DESCRIPTION
1-B1@6-6.5	See exploration logs
1-B3@3.5-4	See exploration logs

PROJECT NAME: 100 Wood Hollow		**Test Date:** 8/8/24
PROJECT NO: 26288.000.001 PH001		**Tested By:** Y. Cabrales
CLIENT: Newmark Management		**Reviewed By:** G. Criste
LOCATION: Novato, CA		

ENGEO
Expect Excellence

ENGEO
Expect Excellence