

**GEOTECHNICAL INVESTIGATION  
FOR PROPOSED NEW RETAINING WALLS,  
CABIN, AND PARKING AREA**

at the

**Healing Cultures Property**

10707 La Honda Road

Woodside, California

Report Prepared for:

**Healing Cultures, Inc.**

Report Prepared by:

**GeoForensics, Inc.**

August 2020

File: 220116  
August 19, 2020

Healing Cultures, Inc.  
10707 La Honda Road  
Woodside, CA 94061

Attention: Ms. McKenzie Brooks

Subject: **Healing Cultures Property  
10707 La Honda Road  
Woodside, California  
GEOTECHNICAL INVESTIGATION FOR  
PROPOSED NEW RETAINING WALLS,  
CABIN, AND PARKING AREA**

Dear Ms. Brooks:

In accordance with your authorization, we have performed a subsurface investigation into the geotechnical conditions present at the location of the proposed improvements. This report summarizes the conditions we measured and observed, and presents our opinions and recommendations for the design and construction of the proposed new retaining walls, cabin, and parking area.

## **Site Description**

The subject site is a gently to steeply sloping, irregularly-shaped parcel located on the west side of La Honda Road (at the approximate location shown on Figure 1). The property is bounded by other developed single family residential lots to the sides, forested land and creek to the rear, and La Honda Road to east.

The site is currently occupied by a two-story tall, wood-framed residence situated near the center of the lot. There is a detached garage east of the house. The wooden house floors are supported above crawlspace areas, while the garage has a concrete slab-on-grade floor. An asphalt driveway leads from the street down to the garage, forms a turnaround in front of the house, and extends down to a lower bench on the lot.

The ground surface in the site vicinity has an overall slope down towards the west (as shown on Figure 2). At the site, the ground slopes gently to steeply down towards the west. Surface gradients range from 20:1 to 2:1 (horizontal:vertical, H:V). During the original development of the property, it appears that at least 7 feet of cuts and fills were placed required in order to create the driveway down from La Honda Road, and the existing more level areas on the site.

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The grounds around the residence have been landscaped with a variety of small to medium sized bushes and shrubs, numerous small to large trees, and various other native plants and grasses. On the moderately steep slopes down from La Honda Road, there are 3 foot tall wood post and lagging walls. There is a wood lagging wall and gabion basket wall within an existing old slide (on the slope at the rear of the property).

### **Proposed Construction**

We understand that the current development for the site proposes the demolition of the existing gabion basket and wood lagging retaining walls within the existing slide, and the subsequent construction of new replacement retaining walls, new cabin, and parking area. New foundation loads are expected to be typical for the cabin structure (i.e. light). Excavation work at the site is expected to be limited to retaining wall landslide repair and cabin foundation excavations. Fills up to 15 feet thick may be included as part of the site development. Site retaining walls up to 8 feet tall will be required for the proposed construction.

### **INVESTIGATION**

#### **Scope and Purpose**

The purpose of our investigation was to determine the nature of the subsurface soil conditions so that we could provide geotechnical recommendations for the construction of the proposed new retaining walls, cabin, parking area, and associated improvements. In order to achieve this purpose, we have performed the following scope of work:

- 1 - visited the property to observe the geotechnical setting of the area to be developed;
- 2 - reviewed relevant published geotechnical maps;
- 3 - drilled four borings near the location of the proposed improvements;
- 4 - performed laboratory testing on collected soil samples;
- 5 - assessed the collected information and prepared this report.

The findings of these work items are discussed in the following sections of this report.

#### **Site Observations**

We visited the site on July 23, 2020 to observe the geotechnically relevant site conditions. During our visit, we noted the following conditions:

- A - We observed hairline to 1/8 inch wide cracks in the asphalt on the limited portions of the driveway we observed.

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- B - In general, the wood post and lagging site retaining walls were in fair condition with some tilt and rot noted along the driveway. The wood lagging and gabion basket walls within the slide mass were also in fair to poor condition, with notable tilt and offsets.
- C - We would characterize the drainage in the vicinity of the new retaining walls, and new cabin, and parking areas all to be sheet flow to the west.

### **Geologic Map Review**

We reviewed the *Geology of the Onshore Part of San Mateo County, California: Derived from the Digital Database Open-File 98-137*, by Earl E. Brabb, R.W. Graymer, and D.L. Jones (1998) and the *Geotechnical Hazards Synthesis Map for San Mateo County*, by Leighton and Associates (1976). The relevant portions of the Brabb, Graymer, and Jones map and Leighton map have been reproduced in Figures 3 and 3a.

The Brabb, Graymer, and Jones map indicates that the site is underlain by Lambert Shale and San Lorenzo Formation, Undivided (lower Miocene, Oligocene, and middle and upper Eocene) (map symbol "T1s"). These materials are described as "brown and dark-gray to gray, brown, and red mudstone, siltstone, and shale. Includes some beds of fine- to coarse-grained sandstone. Lambert Shale is generally more siliceous than San Lorenzo Formation, but the two units cannot be distinguished where out of stratigraphic sequence and without fossils."

The County map plots a queried landslide deposit just north of the subject property.

Our subsurface exploration (see below) encountered clay and sand materials which we judged to be consistent with the mapping.

The active San Andreas Fault is mapped approximately 4.0 miles (6.4 km) northeast of the site.

### **Subsurface Exploration**

On July 23, 2020 we drilled 4 borings at the site at the locations shown on Figure 4. The borings were drilled using a Mobile B-24 truck-mounted drilling rig equipped with 4.0 inch diameter, helical flight augers. Logs of the soils encountered during drilling record our observations of the cuttings traveling up the augers and of relatively undisturbed samples collected from the base of the advancing holes. The final boring logs are based upon the field logs with occasional modifications made upon further laboratory examinations of the recovered samples and laboratory test results. The final logs are attached in Appendix A.

The relatively undisturbed samples were obtained by driving a 3.0 inch (outer diameter) Modified California Sampler and a Standard Penetration Sampler (as noted on the logs) into the base of the advancing hole by repeated blows from a 140 pound hammer lifted 30 inches. On the logs, the number of blows required to drive the sampler the final 12 inches of the 18 inch drive, have been recorded as the Blow Counts. These blows have not been adjusted to reflect equivalent blows of any other type of sampler or hammer, or to account for the different samplers used.

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### **Subsurface Conditions**

Boring 1 (between the two retaining walls downslope of the slide headscarp) penetrated stiff to very stiff silty clay with varying amounts of organics and rock fragments down to the terminated boring depth of 19.5 feet. We estimated that the upper 6 feet of clay is old slide debris.

Boring 2 at the lower terrace (near the proposed retaining wall) first penetrated 7 feet of very stiff silty sandy clay with rock fragments. This was underlain by very stiff fine sandy silt from 7 to 13.5 feet. Below this was very stiff silty clay with roots down to the terminated boring depth of 14.5 feet. We interpret the clay encountered at 7 feet to be competent materials.

Boring 3 (near the proposed cabin) encountered 13.5 feet of firm to very stiff silty clay with varying amounts of sand and rock fragments. We interpret the clay encountered by 7 feet to be competent.

Boring 4 (near the entry gate) penetrated very stiff silty clay to 5.5 feet. Below this was hard siltstone/claystone down to the terminated boring depth of 14.5 feet.

Please refer to Appendix A for a more detailed description of each boring.

No free groundwater was encountered during the drilling of the holes. However, during periods of heavy rain or late in the winter, groundwater seepage may exist at shallower depths, most likely as perched water atop the stiffer clays/bedrock.

### **Laboratory Testing**

The relatively undisturbed samples collected during the drilling process were returned to the laboratory for testing of engineering properties. In the lab, selected soil samples were tested for moisture content, density, organics content, and plasticity. The results of the laboratory tests are attached to this report in Appendix B.

Analysis to determine the percent organics was performed on a sample of the deeper clay materials (Sample 2-3 @ 14 feet) showed the tested materials are composed of only 4.3 percent organic matter.

Plasticity Index (PI) testing performed on the site near surface clayey materials produced PI results of 35, 54, and 71. This testing indicated that the near surface clayey soil materials have high to very high plasticity and are highly to very highly expansive.

### **CONCLUSIONS AND RECOMMENDATIONS**

#### **General**

Based upon our investigation, we believe that the proposed improvements can be safely constructed. Geotechnical development of the site is controlled by the presence of expansive soils, steep slopes, and an existing old landslide.

Expansive soils derive their name from their propensity to change volume in response to changes in moisture content. When they are dry, they shrink; when they become wet, they swell. The pressures these soils can exert as they expand can be sufficiently high to move conventional retaining wall and building foundations. The foundation movement induced by the soil shifting can cause wall coverings to crack, doors and windows to stick, and floors to slope, walls to crack and tilt. Seasonal movements of expansive soils have caused such distress to countless structures in the Bay Area. When located on slopes, these soils will also tend to slowly creep downhill.

To combat seasonal expansive soil movements, it is necessary to utilize a foundation system which derives its support from the deeper, more stable soils. Typically, a drilled, cast-in-place pier foundation system is used to reach the more stable materials. Therefore, we have recommended that such foundation system be utilized at this site.

The recommendations in this report should be incorporated into the design and construction of the proposed new retaining walls, cabin, parking area, and associated improvements.

**Seismicity**

The greater San Francisco Bay Area is recognized by Geologists and Seismologists as one of the most active seismic regions in the United States. Several major fault zones pass through the Bay Area in a northwest direction which have produced approximately 12 earthquakes per century strong enough to cause structural damage. The faults causing such earthquakes are part of the San Andreas Fault System, a major rift in the earth's crust that extends for at least 700 miles along western California. The San Andreas Fault System includes the San Andreas, San Gregorio, Hayward, Calaveras Fault Zones, and other faults.

In 2014, seismologic and geologic experts convened by the U.S. Geological Survey, California Geological Survey, and the Southern California Earthquake Center concluded that there is a 72 percent probability for at least one “large” earthquake of magnitude 6.7 or greater to occur in the Bay Area before the year 2043. The northern portion of the San Andreas fault is estimated to have a 6 percent probability, while the Hayward and Calaveras faults are estimated to have a 14 and 7 percent probability of producing an earthquake of that magnitude or greater during that time period.

**Ground Rupture** - The lack of mapped active fault traces through the site, suggests that the potential for primary rupture due to fault offset on the property is low.

**Ground Shaking** - The subject site is likely to be subject to very strong to violent ground shaking during its life span due to a major earthquake in one of the above-listed fault zones. Current (2018) building code design may be followed by the structural engineer to minimize damages due to seismic shaking, using the following input parameters from the Structural Engineers Association of California (SEAOC) Calculator based upon ASCE 7-16 design parameters:

Site Class - C	$SM_S = 2.346$	$SM_I = 1.073$	$SD_S = 1.564$	$SD_I = 0.716$
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**Landsliding** - The subject site and the surrounding area are gently to steeply sloping. A landslide had occurred on the western slope due to the El Nino rains in 1998. Retaining walls were installed to arrest the progression upslope of the slide/scarp. However, these walls were not installed to sufficient depth to prevent continued movements. In addition, as with any slope, other minor sloughing of the steeper site slopes could occur during earthquake shaking. The proposed improvements should not be affected by shallow sloughing, as they will be supported by drilled piers into the bedrock materials at the site. Therefore, the hazard due to large-scale seismically-induced landsliding is, in our opinion, relatively low for the site.

**Liquefaction** - Liquefaction most commonly occurs during earthquake shaking in loose fine sands and silty sands associated with a high ground water table. These conditions were demonstrated to be absent down to the site bedrock. Based upon the subsurface investigation, the proposed building site is underlain by clayey soils then bedrock at shallow depths. Therefore, it is our opinion the liquefaction is unlikely to occur on the subject property.

**Ground Subsidence** - Ground subsidence may occur when poorly consolidated soils densify as a result of earthquake shaking. Since the proposed building site is underlain at shallow depths by resistant materials, the hazard due to ground subsidence is, in our opinion, considered to be low.

**Lateral Spreading** - Lateral spreading may occur when a weak layer of material, such as a sensitive or liquefiable soil, loses its shear strength as a result of earthquake shaking. Overlying blocks of competent material may be translated laterally towards a free face. The existing landslide headscarp will be repaired and supported by a new retaining wall, and these types of soils are not present proximate to or at the site, hence, the hazard due to lateral spreading is, in our opinion, considered to be low.

### **Site Preparation and Grading**

All debris resulting from the demolition of existing improvements should be removed from the site and may not be used as fill. Any existing underground utility lines to be abandoned should be removed from within the proposed building envelope and their ends capped outside of the building envelope.

Any vegetation and organically contaminated soils should be cleared from the building area. All holes resulting from removal of tree stumps and roots, or other buried objects, should be overexcavated into firm materials and then backfilled and compacted with native materials.

The placement of fills at the site is expected to include: utility trench backfill, retaining wall backfill, slide reconstruction, slab subgrade materials, and finished drainage and landscaping grading. These and all other fills should be placed in conformance with the following guidelines:

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Fills may use organic-free soils available at the site or import materials. Import soils should be free of construction debris or other deleterious materials and be non-expansive. *A minimum of 3 days prior to the placement of any fill, our office should be supplied with a 30 pound sample (approximately a full 5 gallon bucket) of any soil or baserock to be used as fill (including native and import materials) for testing and approval.*

All areas to receive fills should be stripped of organics and loose or soft near-surface soils and existing slide debris. Fills should be placed on level benches cut into competent materials, as identified by our field personnel, in lifts no greater than 6 inches thick (loose) and be compacted to at least 90 percent of their Maximum Dry Density (MDD), as determined by ASTM D-1557. If native expansive soils are used for fill at the site, then the soils should be placed at 3 to 5% over Optimum Moisture Content and be compacted to **between** 85 to 90 percent of their MDD. In pavement (concrete or asphalt) areas to receive vehicular traffic, all baserock materials should be compacted to at least 95 percent of their MDD. Also, the upper 6 inches of soil subgrade beneath any pavements should be compacted to at least 90 percent of its MDD.

All unretained fills to be placed on slopes steeper than 6 to 1 (horizontal to vertical, H:V) will need to be keyed and benched into competent native materials. Any retained fills will need to be benched into competent native materials, however, a formal keyway is not required. The entire base of any keyway should extend into competent weathered bedrock materials, located about 6 to 13 feet below grade. The entire bases of all benches should extend into or through competent colluvial soils, as identified in the field by representatives from our office. It should be anticipated that the outer edge of bench excavations will extend at least 5 feet below native grade. Keyways and benches should be sloped back into the hillside at a minimum 2% gradient.

For fills over 5 feet thick, or where deemed necessary by our personnel, a blanket drain should be provided within any keyway excavations, and chimney drains should be provided at the back of any benches identified by our office in the field. The blanket drain should cover the entire keyway and consist of a minimum 6 inch thick layer of clean crushed drain rock completely covered (top and sides) with filter fabric (Mirafi 140N or approved equivalent). Chimney drains should consist of a minimum 6 inch wide column of clean crushed drain rock, also wrapped with filter fabric, for at least half the height and for the full width of the bench. These systems should drain to 4 inch diameter perforated pipes, placed at the base of the drain rock. The pipes should consist of Schedule 40 PVC or SDR 35. No flexible, corrugated pipe may be used within any drainage system installed as part of this project. The bench drain pipes may connect to the keyway blanket drain pipe. A solid line should be used to convey the water to an appropriate discharge point. We note that *Caltrans Class 2 permeable rock* is an acceptable substitution for clean drain rock and filter fabric.

Temporary, dry-weather, vertical excavations should remain stable for short periods of time to heights of 5 feet. All excavations should be shored or sloped in accordance with OSHA standards.

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Permanent cut and/or fill slopes should be no steeper than 2:1 (H:V). However, even at this gradient, minor sloughing of slopes may still occur in the future. Positive drainage improvements (e.g. drainage swales, catch basins, etc.) should be provided to prevent water from flowing over the tops of cut and/or fill slopes.

### **Cabin Foundations – Piers**

Due to the presence of near surface site soils of marginal quality and highly expansive site soils, the cabin foundations will need to penetrate into the deeper, more stable soils. We recommend a pier and grade beam foundation system be used.

Piers should penetrate a minimum of 12 feet below lowest adjacent grade, and 5 feet into competent native materials, whichever is deeper. In sloping areas, the piers should penetrate a minimum of 12 feet below lowest adjacent grade, and 8 feet into competent weathered rock, whichever is deeper. It should be assumed that up to 13 feet of overburden will exist at the site, so nominal pier depths may range from 12 to 21 feet below lowest adjacent grade.

The piers should have a minimum diameter of 16 inches and be nominally reinforced with a minimum of four #4 bars vertically.

Actual pier depth, diameter, reinforcement, and spacing should be determined by the structural engineer based upon the following design criteria:

A friction value of 600 psf may be assumed to act on that portion of the pier below a depth of 7 feet. Lateral support may be assumed to be developed along the length of the pier below a depth of 7 feet, using a passive pressure of 400 pcf Equivalent Fluid Weight (EFW). Passive resistance may be assumed to act over 1.5 projected pier diameters. However, for passive resistance to start, the footing must be embedded so that there is a minimum of 10 feet of horizontal cover between the face of the pier and any adjacent, parallel slope. Above 7 feet, no frictional or lateral support may be assumed. These design values may be increased 1/3 for transient loads (i.e. seismic and wind).

Even though piers are designed to derive their vertical resistance through skin friction, the bases of the piers holes should be clean and firm prior to setting steel and pouring concrete. If more than 6 inches of slough exists in the base of the pier holes after drilling, then the slough should be removed. If less than 6 inches of slough exists, the slough may be tamped to a stiff condition. Piers should not remain open for more than a few days prior to casting concrete. In the event of rain, shallow groundwater, or caving conditions it may be necessary to pour piers immediately.

All perimeter piers, and piers under load-bearing walls, should be connected by concrete grade beams. Perimeter grade beams should penetrate a minimum of 6 inches below crawlspace grade (unless a perimeter footing drain is installed to intercept water attempting to enter around the perimeter). Interior grade beams do not need to penetrate below grade. All other isolated floor supports must also be pier supported to resist expansive soil uplift, however, they do not need to be connected by grade beams.

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In order to reduce any expansive soil uplift forces on the base of the grade beams, the beams should have either a uniform 4 inch void between their base and the soil, or should be constructed with a knife edge and triangular shaped void in a rectangular trench. The void can be created by the use of prefabricated cardboard void material (e.g. K-void, SureVoid, Carton-void), half a sonotube faced concave down, or other methods devised by the contractor and approved by our offices. *The use of Styrofoam is not acceptable for creating the void.*

All improvements connected directly to any pier supported structure, also need to be supported by piers. This includes, but is not limited to: porches, decks, entry stoops and columns, etc. If the designer does not wish to pier support these items, then care must be taken to structurally isolate them (with expansion joints, etc.) from the pier supported structure.

If the above recommendations are followed, total foundation settlements should be less than 1 inch, while differential settlements should be less than ½ inches.

### **Retaining Walls**

*New site retaining walls should not be structurally connected to the cabin or other residential structures. New site walls tied to the cabin, walls which are located on, or within 10 feet of the crest of, slopes steeper than 5:1 (H:V); and, walls for which expansive soil movements are undesirable, should utilize a pier and grade beam foundation system. Site retaining walls which are detached from the cabin and are located in level areas (flatter than 5:1, H:V) may be supported by drilled piers or by spread footings depending upon wall type. If spread footings are utilized, then some expansive soil movements of the walls may occur.*

**Wall Forces** - Any unrestrained retaining walls required for the proposed construction should be designed to resist an active pressure of 55 pcf Equivalent Fluid Weight (EFW) in supporting soils with retained slopes less than 4:1 (H:V). An active pressure of 75 pcf EFW should be utilized for retained slopes with an inclination of 2:1 (H:V). Where retained slopes are greater than 4:1, though less than 2:1, the designer should linearly interpolate between 55 and 75 pcf EFW.

Where a retaining wall is located within a horizontal distance less than twice the height of the lower retaining wall, the lower retaining wall will need to be designed for an additional surcharge pressure from the upper wall(s). Once the geometry of such walls has been determined, please provide our office with a cross-section so that we can determine the required surcharge.

Any restrained retaining walls required should be designed for the aforementioned active pressures with an additional uniform pressure of 8H psf, where H is the height of the wall in feet. We leave it to the design professional's judgment in determining whether a wall is restrained or not. An additional uniform force of 10H psf may be applied to account for seismic forces on the wall, although it is our opinion that such forces need not be applied.

All retaining walls should also be designed to resist a point load applied at the midpoint of the wall, equal to ½ the maximum applied surcharge.

**Drilled Piers** - Any wall which is structurally connected to the house, or that is located on, or within 10 feet of the crest of, slopes steeper than 5:1 (H:V) should utilize a drilled pier foundation system. Additionally, any site walls for which expansive soil shifting is unacceptable should use drilled piers. We note that pier-supported walls may not rely upon a toe footing to resist overturning forces. All vertical and lateral forces should be resisted by piers. This may require the use of a staggered, double row of piers, depending upon the wall height and any surcharges.

Please refer to the *Cabin Foundations – Drilled Piers* section of this report for the applicable pier design recommendations.

If drilled piers are utilized beneath a concrete or block wall, they will need to be connected by a concrete grade beam. No grade beam is required for a wood lagging wall.

**Spread Footings** - For detached site walls in level areas (flatter than 5:1, H:V), and where expansive soil shifting is acceptable, spread footings may be used. Footings should be designed using an allowable bearing pressure of 2500 psf (on soil) or 3500 psf (on bedrock), at a minimum depth of 36 inches below adjacent grade, or on competent materials as approved by our office in the field. Lateral pressures may be resisted by a passive pressure of 300 pcf EFW assumed to be acting against the sides of the footings (or shear keys, if required). Passive resistance may start at a depth of 1 foot below exterior grade. However, for passive resistance to start, the footing must be embedded so that there is a minimum of 10 feet of horizontal cover between the face of the footing and any adjacent, parallel slope. Alternatively, lateral pressures may be resisted by friction between the base of the footings and the ground surface. A friction coefficient of 0.35 (on soil) or 0.40 (on bedrock) may be assumed. Frictional and passive resistance may not be used in combination. The above values may be increased 1/3 for transient loads.

**Wall Drainage** - The above values have been provided assuming that back-of-wall drains will be installed to prevent build-up of hydrostatic pressures behind all walls. This drainage system may consist of a prefabricated drainage panel (i.e. Miradrain) or a gravel and filter fabric type system. We also recommend that any interior retaining walls, or walls through which efflorescence transmission would be undesirable, should be waterproofed. The waterproofing should be specified by the designer, though we suggest the use of Bituthene, Miradri, or other similar waterproofing membrane. Surface drainage above the wall should preclude overtopping of the wall, and should also preclude ponding on the ground surface above the wall. *Additionally, the ground surface above all walls should form a drainage swale to carry water to the sides of the wall and/or to area drain locations.*

The back-of-wall drain systems should be installed with a minimum 3-inch diameter perforated pipe placed a minimum of 4 inches below the top of the footing (preferably at the base of the footing heel). The pipe should not be placed on top of the heel of the wall footing unless seepage through the base of the wall is acceptable. Perforations should be placed face-down (at 5 and 7 o'clock). The perforated pipe should connect to a solid discharge line, which discharges away from the new structures. This solid line should not connect to surface water drain lines (i.e. downspout and area drain lines). If water transmission through the base of a wall is not a concern, then weep holes may be used in place of the pipe.

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If used, the gravel system should consist of a minimum 12 inch wide column of drain rock ( $\frac{3}{8}$  to  $\frac{3}{4}$  inch clean, crushed rock) extending the full width of the wall. The rock should continue to within 12 inches of finish grade. Prior to backfilling with the drain rock, a layer of filter fabric (Mirafi 140N or approved equivalent) should be placed against all soil surfaces to separate the rock and soil. The filter fabric should wrap over the top of the gravel and then a 12 inch thick cap of native soils should be placed at the top of the drain. If concrete flatwork is to directly overlay the back-of-wall drain, or if the drain is located in a crawlspace area, then the soil cap should be eliminated.

If prefabricated drainage panels are used, a packet of filter fabric-wrapped drain rock should be placed around the perforated collector pipe at the base of the panel. The tops of the panels should be sealed and secured in accordance with the manufacturer's recommendations. The base of the drainage panels should extend down below the top of the filter fabric-wrapped drain rock.

We note that Caltrans Class II permeable rock may be utilized in lieu of clean drain rock and filter fabric. The Class II permeable rock needs to be compacted into place, and needs to be certified by the quarry or rockery that it meets the Caltrans Class II permeable rock specifications.

### **Slabs-on-Grade**

The cabin floors should not consist of concrete slabs-on-grade, although they may consist of structural slabs (supported by drilled piers and grade beams, and have a 4-inch void under the slab). This is due to the expansive nature of the site soils which would cause deformations in a conventional slab-on-grade. However, any sidewalks or patios may consist of conventional concrete slabs-on-grade, though it should be expected that some seasonal/post-construction shifting of such slabs may occur. We have provided guidelines to help reduce post-construction movements, however, it is nearly impossible to economically eliminate all shifting.

To help reduce cracking, we recommend slabs be a minimum of 5 inches thick and be nominally reinforced with #5 bars at 12 inches on center, each way. Slabs which are thinner or more lightly reinforced may experience undesirable cosmetic cracking. However, actual reinforcement and thickness should be determined by the structural engineer based upon anticipated usage and loading.

In large non-interior slabs (e.g. patios, etc.), score joints should be placed at a maximum of 10 feet on center. In sidewalks, score joints should be placed at a maximum of 5 feet on center. All slabs should be separated from adjacent improvements (e.g. footings, porches, columns, etc.) with expansion joints. Interior floor slabs will experience shrinkage cracking. These cosmetic cracks may be sealed with epoxy or other measures specified by the architect.

It would be prudent (though not required) to underlay all slabs with at least 24 inches of non-expansive materials. This will help to reduce future expansive soil movements of the slabs. Slabs which are not underlain by this non-expansive material may undergo excessive seasonal shifting.

Slabs should be set back a minimum of 10 feet from the crest of any slope steeper than 5:1, and 5 feet from any flatter slope.

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All interior slabs should be underlain by a minimum of 4 inches of clean  $\frac{3}{4}$  inch crushed drain rock. The drain rock should be covered by a vapor barrier which conforms to ASTM E1745-97 (e.g. Stego Wrap or an approved equivalent). The architect or structural engineer should determine if sand is required over the vapor barrier.

Slabs which will be subject to light vehicular loads and through which moisture transmission is not a concern (e.g. driveway) should be underlain by at least 6 inches of compacted baserock, in lieu of any sand and gravel. The 6 inches of granular subgrade may be included as part of the 18 inches of non-expansive materials. Exterior landscaping flatwork (e.g. patios and sidewalks) may be placed directly on proof-rolled soil subgrade materials (e.g. no granular subgrade), however, they will be potentially subject to greater amounts of shifting and moisture transmission.

As stated previously, in pavement (concrete or asphalt) areas to receive vehicular traffic, all baserock materials should be compacted to at least 95 percent of their MDD. Also, the upper 6 inches of native soil subgrade beneath any pavements should be compacted to at least 90 percent of its MDD.

To reduce post-construction expansive soil movements (i.e. heave) of any slabs, care should be taken to keep the subgrade moist for an extended period of time prior to pouring the slabs. *Shrinkage cracks should not be allowed to develop in the soil beneath any proposed slabs.* Ideally, all slab areas and crawlspace subgrade areas should be sprayed, and covered with a vapor barrier and any granular materials as soon as exposed by grading.

## **Drainage**

**Surface Drainage** - Adjacent to any buildings, the ground surface should slope at least 5 percent away from the foundations within 5 feet of the perimeter. Impervious surfaces should have a minimum gradient of 2 percent away from the foundation.

Surface water should be directed away from all buildings into drainage swales, or into a surface drainage system (i.e. catch basins and a solid drain line). "Trapped" planting areas should not be created next to any buildings without providing means for drainage (i.e. area drains).

All new roof eaves should be lined with gutters. The downspouts may be connected to solid drain lines, or may discharge onto paved surfaces which drain away from the structure. The downspouts may be connected to the same drain line as any catch basins, but must not directly connect to any perforated pipe drainage system. If splash blocks are preferred, then a perimeter footing drain system **must** be installed.

**Footing Drain** - Due to the potential for changes to surface drainage provisions, it would be wise (though not required) to install a perimeter footing drain to intercept water attempting to enter the crawlspace. If a footing drain is not installed, some infiltration of moisture into the crawlspace may occur. Such penetration should not be detrimental to the performance of the structure, but can possibly cause humidity and mildew problems within the cabin.

The footing drain system, if installed, should consist of a 12 inch wide gravel-filled trench, *dug at least 12 inches below the elevation of the adjacent crawlspace*. The trench should be lined with a layer of filter fabric (Mirafi 140N or equivalent) to prevent migration of silts and clays into the gravel, but still permit the flow of water. Then 1 to 2 inches of drain rock (clean crushed rock or pea gravel) should be placed in the base of the lined trench. Next a perforated pipe (minimum 3 inch diameter) should be placed on top of the thin rock layer. The perforations in the pipe should be face down. The trench should then be backfilled with more rock to within 6 inches of finished grade. The filter fabric should be wrapped over the top of the rock. Above the filter fabric 6 inches of native soils should be used to cap the drain. If concrete slabs are to directly overlay the drain, then the gravel should continue to the base of the slab, without the 6 inch soil cap. This drain should not be connected to any surface drainage system.

**Drainage Discharge** - The surface drain lines should discharge at least 15 feet away from the cabin, at a location approved by our office. The discharge location(s) should be protected by energy dissipaters to reduce the potential for erosion. Care should be taken not to direct concentrated flows of water towards neighboring properties. This may require the use of multiple discharge points.

The footing drain (if installed) and any back-of-wall drain lines should discharge independently from the surface drainage system. A sump pump may be required for the footing drain discharge system. The surface and subsurface drain systems should not be connected to one another.

**Drainage Materials** - Drain lines should consist of hard-walled pipes (e.g. SDR 35 or Schedule 40 PVC). In areas where vehicle loading is not a possibility, SDR 38 or HDPE pipes may be used. Corrugated, flexible pipes may not be used in any drain system installed at the property.

Surface drain lines (e.g. downspouts, area drains, etc.) should be laid with a minimum 2 percent gradient ( $\frac{1}{4}$  inch of fall per foot of pipe). Any subsurface drain systems (e.g. footing drains) should be laid with a minimum 1 percent gradient ( $\frac{1}{8}$  inch of fall per foot of pipe).

### **Pavement**

The new driveway/parking areas may consist of concrete, interlocking pavers, or asphaltic concrete over Caltrans Class II aggregate base (baserock). The asphalt should have a minimum thickness of 2½ inches. The baserock should have a minimum thickness of 12 inches, though 18 inches is preferable due to the expansive nature of the near-surface site soils. All of the baserock should attain a minimum compaction of 95 percent of its MDD. Any fill below this layer should attain a minimum of 90 percent relative compaction.

### **Plan Review and Construction Observations**

The use of the recommendations contained within this report is contingent upon our being contracted to review the plans, and to observe geotechnically relevant aspects of the construction.

File: 220116  
August 19, 2020

We should be provided with a full set of plans to review at the same time the plans are submitted to the building/planning department for review. A minimum of one working week should be provided for review of the plans.

At a minimum, our observations should include: compaction testing of fills and subgrades; pier drilling; forming of the grade beams voids; slab and parking area subgrade preparation; installation of any drainage system (e.g. back-of-wall, footing, and surface), and final grading. A minimum of 48 hours notice should be provided for all construction observations.

### LIMITATIONS

This report has been prepared for the exclusive use of the addressee, and their architects and engineers for aiding in the design and construction of the proposed development. It is the addressee's responsibility to provide this report to the appropriate design professionals, building officials, and contractors to ensure correct implementation of the recommendations.

The opinions, comments and conclusions presented in this report were based upon information derived from our field investigation and laboratory testing. Conditions between or beyond our borings may vary from those encountered. Such variations may result in changes to our recommendations and possibly variations in project costs. Should any additional information become available, or should there be changes in the proposed scope of work as outlined above, then we should be supplied with that information so as to make any necessary changes to our opinions and recommendations. Such changes may require additional investigation or analyses, and hence additional costs may be incurred.

Our work has been conducted in general conformance with the standard of care in the field of geotechnical engineering currently in practice in the San Francisco Bay Area for projects of this nature and magnitude. We make no other warranty either expressed or implied. By utilizing the design recommendations within this report, the addressee acknowledges and accepts the risks and limitations of development at the site, as outlined within the report.

Respectfully Submitted;  
**GeoForensics, Inc.**



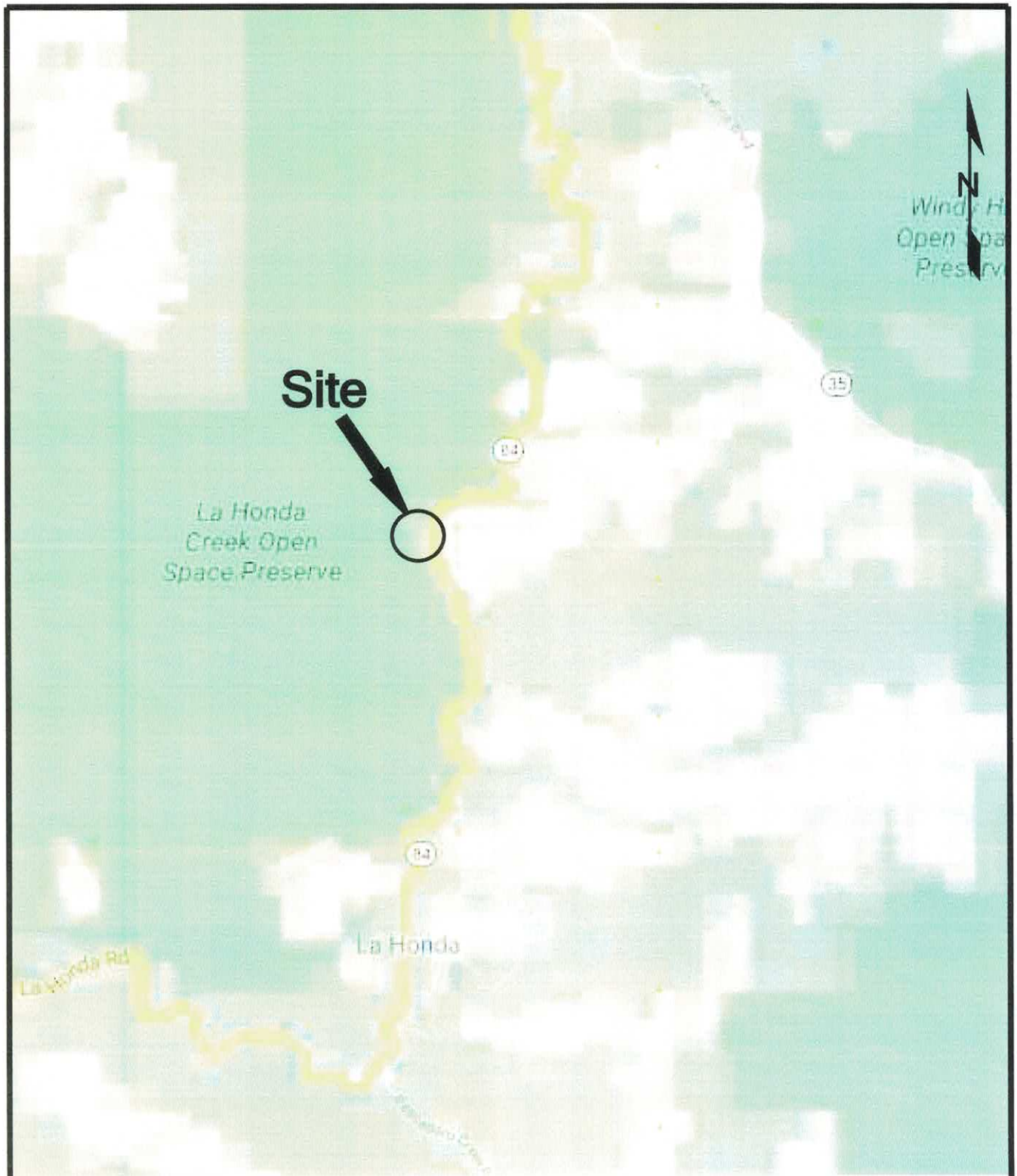
Daniel F. Dyckman, PE, GE  
Senior Geotechnical Engineer, GE 2145



Bernard A. Atendido  
Field Engineer



cc: 3 to addressee

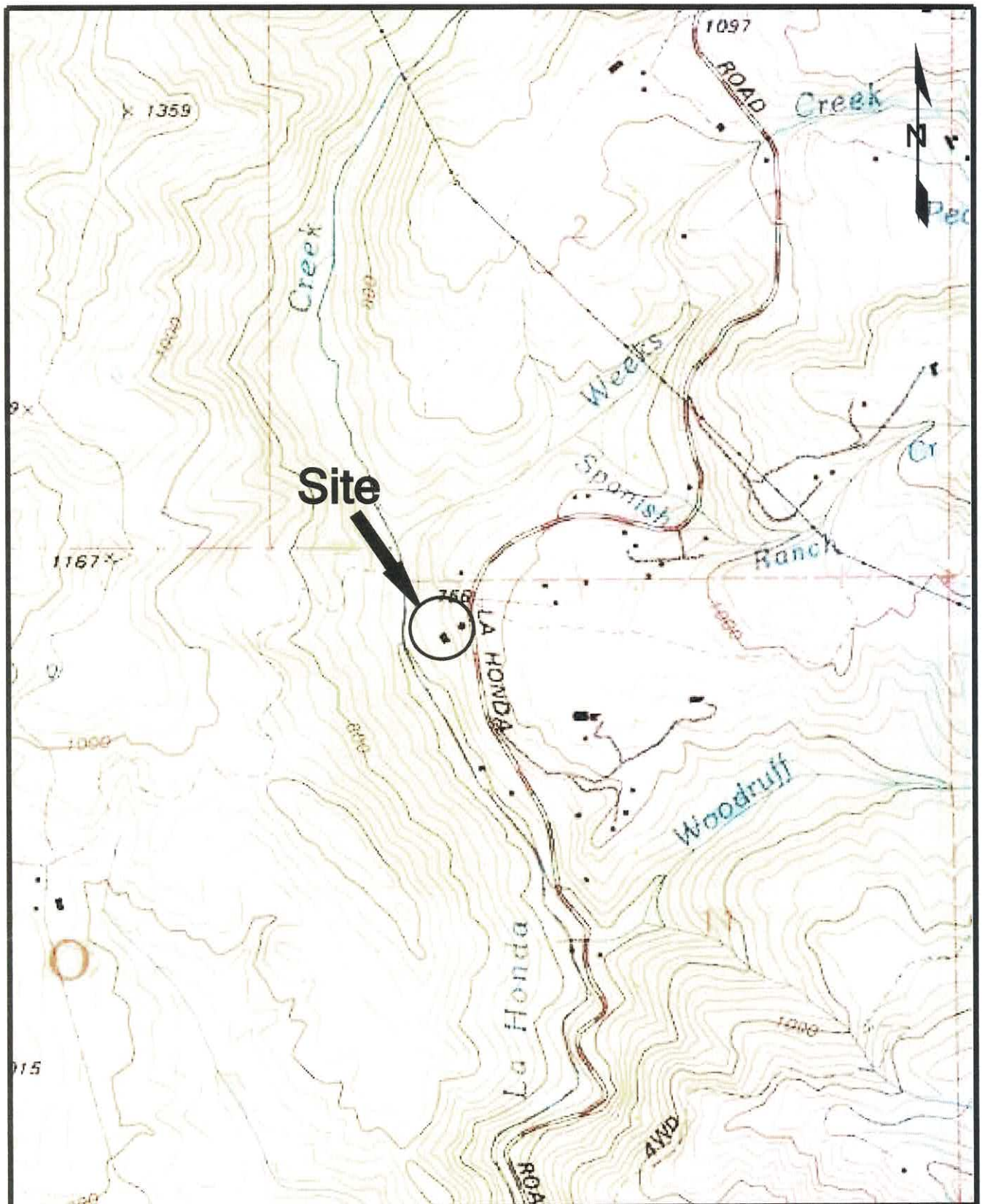


Source: Google Maps

**GEOFORENSICS, INC.**

303 Vintage Park Dr., Suite 220, Foster City, CA 94404  
Tel: (650) 349-3369 Fax: (650) 571-1878

Figure 1 - Site Location

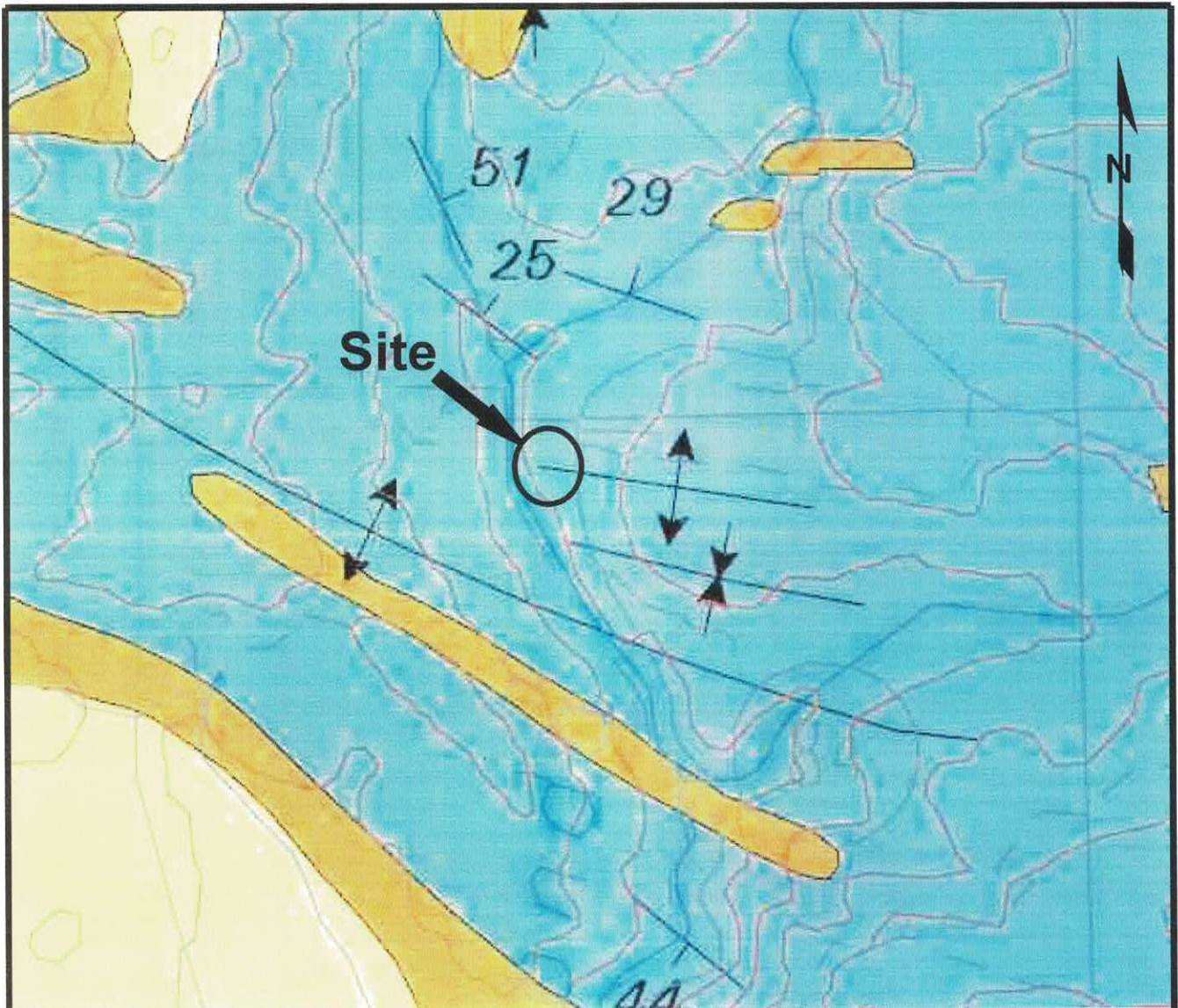


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Figure 2 - Vicinity Topography



**Lambert Shale and San Lorenzo Formation, Undivided (lower Miocene,**

**T1s**

Oligocene, and middle and upper Eocene) —Brown and dark-gray to gray, brown, and red mudstone, siltstone, and shale. Includes some beds of fine-to coarse-grained sandstone. Lambert Shale is generally more siliceous than San Lorenzo Formation, but the two units cannot be distinguished where out of stratigraphic sequence and without fossils

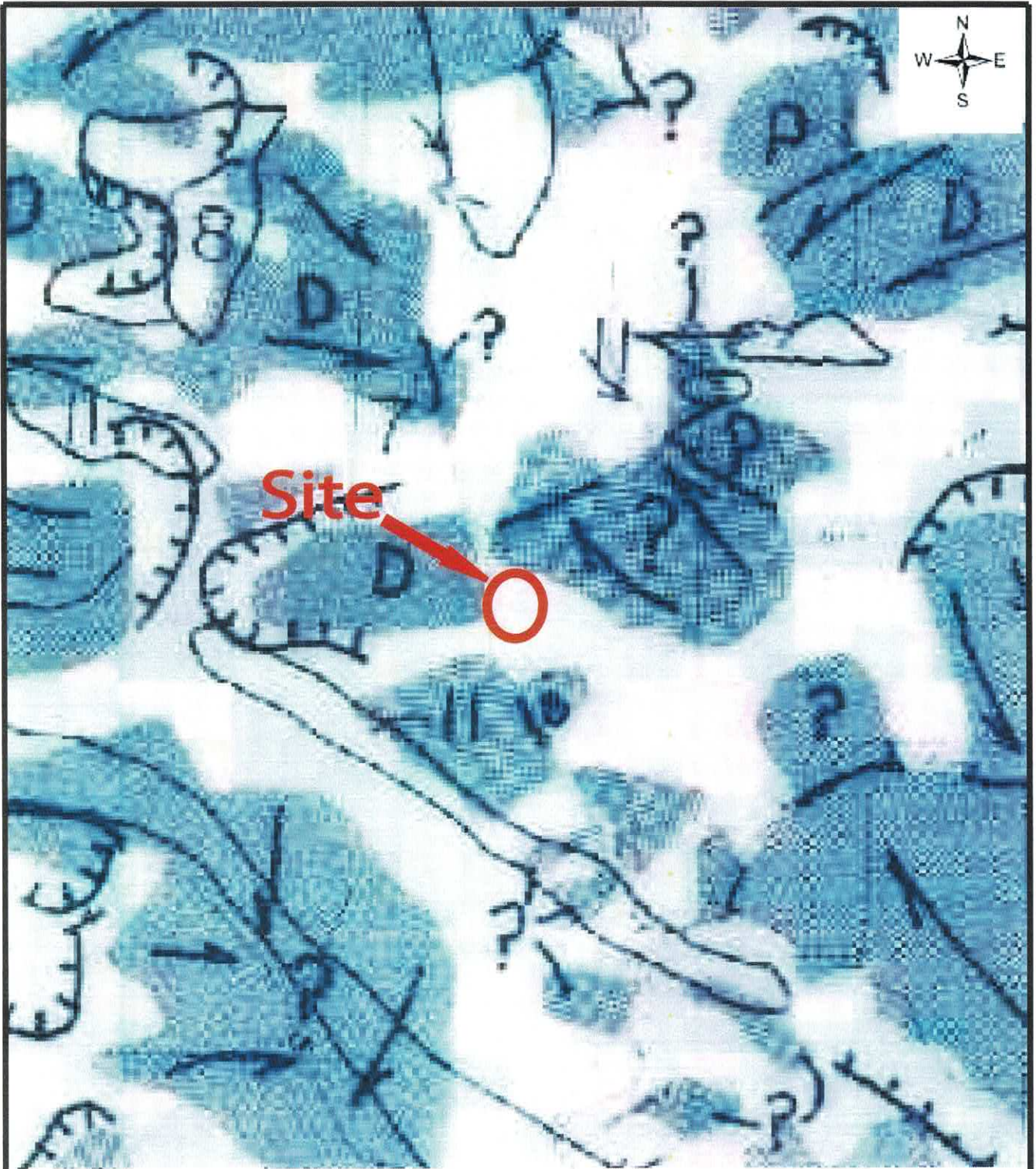
Source: Geology of the Onshore part of San Mateo County, California: derived from the digital database open-file 98-137. E.E. Brabb, R.W. Graymer, and D.L. Jones (1998)

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Figure 3 - Geologic Map



Source: Geolotechnical Hazards Synthesis Map for San Mateo County. Leighton and Associates (1976)

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Figure 3a - County of San Mateo Geologic Map



Base drawing provided by Google Maps  
No Scale on this drawing

 - Boring Locations

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Figure 4 - Site Photo with  
Approximate Boring Locations

## **APPENDIX A - BORING LOGS**

# LOG OF BORING

DEPTH (ft)	SAMPLE NUMBER	SAMPLE LOC.	BLOW COUNTS (12 inches)	MATERIAL DESCRIPTION	DRY DENSITY (pcf)	MOISTURE CONTENT (70)
5	1-1		28	silty CLAY; light brown; dry to slightly moist (CH)	91.0	21.4
				silty CLAY with gravels; brown; slightly moist; stiff (CH)		
10	1-2		50	silty CLAY with some organics; dark brown; slightly moist; very stiff (CH)	90.3	32.0
15	1-3		84	silty CLAY with rock fragments; dark brown; slightly moist; very stiff (CH)	90.9	30.2
20	1-4		33	silty CLAY with rock fragments; dark brown; slightly moist; very stiff (CH)	-	26.9
25				<b>Bottom of Boring at 19.5 feet</b>		
				<b>No Groundwater encountered</b>		
30						

Logged by: BA  
 Job# 220116  
 Drilled on 7/12/19

B-24 Truck Mounted Drilling Rig  
 140 Pound Hammer  
 No Groundwater encountered

Mod. Cal  
 Sampler  
 SPT Sampler

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Figure A1 - Log of Boring 1

# LOG OF BORING

DEPTH (ft)	SAMPLE NUMBER	SAMPLE LOC.	BLOW COUNTS (12 inches)	MATERIAL DESCRIPTION	DRY DENSITY (pcf)	MOISTURE CONTENT (70)
5	2-1		14	silty sandy CLAY with rock fragments; light brown; slightly moist; stiff (CH)  more sandy with depth	95.7	21.2
10	2-2		31	fine sandy SILT (near silty SAND); light brown; slightly moist; very stiff (ML)	89.2	24.0
15	2-3		29	silty CLAY with roots; dark brown; slightly moist; very stiff (CH) (organic smell)	-	-
20				<b>Bottom of Boring at 14.5 feet</b>  <b>No Groundwater encountered</b>		
25						
30						

Logged by: BA  
 Job# 220116  
 Drilled on 7/12/19

B-24 Truck Mounted Drilling Rig  
 140 Pound Hammer  
 No Groundwater encountered

Mod. Cal  
 Sampler  
 SPT Sampler

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Figure A2 - Log of Boring 2

# LOG OF BORING

DEPTH (ft)	SAMPLE NUMBER	SAMPLE LOC.	BLOW COUNTS (12 inches)	MATERIAL DESCRIPTION	DRY DENSITY (pcf)	MOISTURE CONTENT (70)
				2 inches of asphalt on grade		
	3-1		13	silty fine sandy CLAY; red brown and green brown; slightly moist; firm to stiff (CH)	82.3	30.8
5						
	3-2		40	silty CLAY with rock fragments; red brown with yellow brown; slightly moist; very stiff (CH)	89.9	28.5
10				organic smell in clays at 10 feet		
	3-3		25	silty CLAY (near clayey SILT) with siltstone fragments and some sand; dark gray; slightly moist; very stiff (CH)	86.5	26.0
15						
				<b>Bottom of Boring at 13.5 feet</b>		
20				<b>No Groundwater encountered</b>		
25						
30						

Logged by: BA  
 Job# 220116  
 Drilled on 7/12/19

B-24 Truck Mounted Drilling Rig  
 140 Pound Hammer  
 No Groundwater encountered

Mod. Cal  
 Sampler  
 SPT Sampler

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Figure A3 - Log of Boring 3

# LOG OF BORING

DEPTH (ft)	SAMPLE NUMBER	SAMPLE LOC.	BLOW COUNTS (12 inches)	MATERIAL DESCRIPTION	DRY DENSITY (pcf)	MOISTURE CONTENT (%)
5	4-1		36	silty sandy CLAY (near clayey SILT); red brown and gray; slightly moist; very stiff (CH)	56.7	41.4
10	4-2		87	fine sandy SILTSTONE/CLAYSTONE; red brown and orange brown ; slightly moist; hard (MH/CH)	87.3	28.4
15	4-3		41	fine sandy SILTSTONE/CLAYSTONE; red brown and orange brown ; slightly moist; hard (MH/CH)	-	25.5
20				<b>Bottom of Boring at 14.5 feet</b> <b>No Groundwater encountered</b>		
25						
30						

Logged by: BA  
 Job# 220116  
 Drilled on 7/12/19

B-24 Truck Mounted Drilling Rig  
 140 Pound Hammer  
 No Groundwater encountered

Mod. Cal  
 Sampler  
 SPT Sampler

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Figure A4 - Log of Boring 4

## **APPENDIX B - LABORATORY TEST RESULTS**



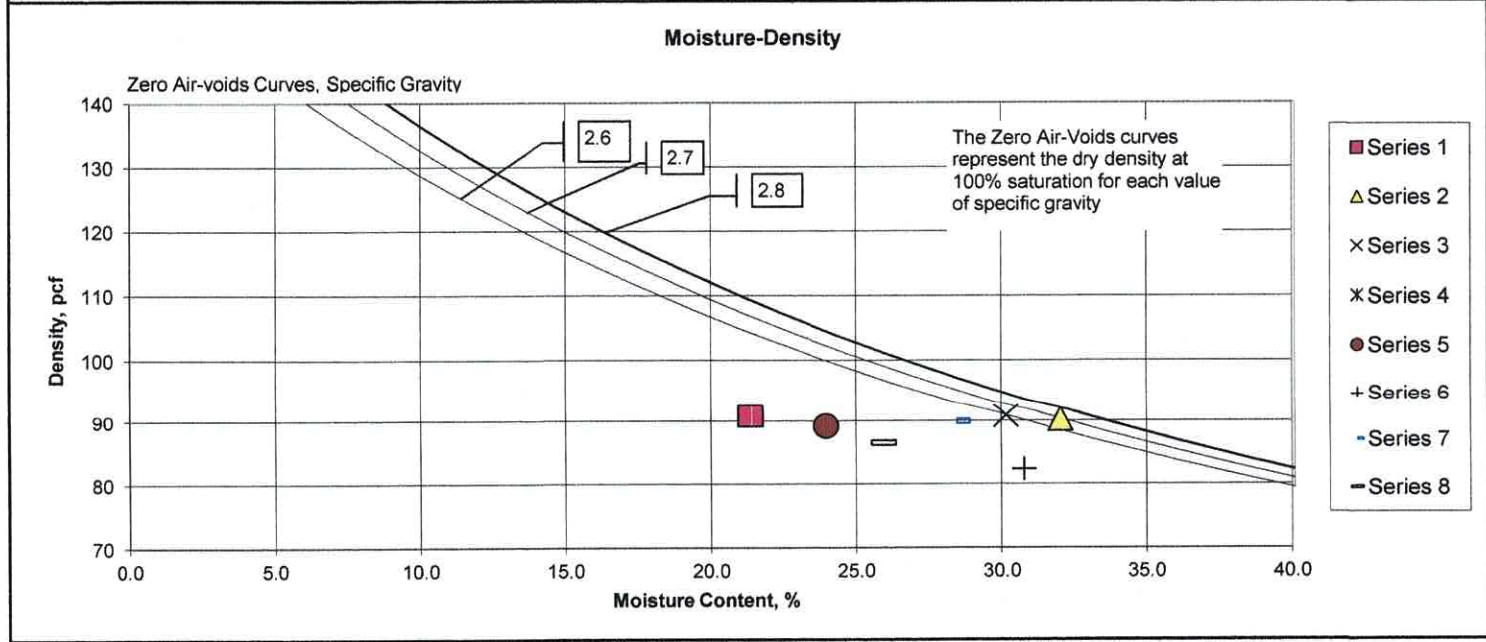
## Moisture-Density-Porosity Report

Cooper Testing Labs, Inc. (ASTM D7263b)

CTL Job No: <u>060-2882a</u>	Project No. <u>220116</u>	By: <u>RU</u>
Client: <u>GeoForensics</u>	Date: <u>08/06/20</u>	
Project Name: <u>Healing</u>	Remarks:	

Boring:	1-1	1-2	1-3	1-4	2-2	3-1	3-2	3-3
Sample:								
Depth, ft:								
Visual Description:	Dark Yellowish Brown Sandy CLAY w/ Gravel	Dark Brown Fat CLAY	Dark Brown CLAY	Dark Brown Sandy CLAY	Dark Brown Sandy CLAY (Claystone)	Yellowish Brown Sandy Elastic SILT	Dark Brown Sandy CLAY	Dark Brown CLAY w/ Sand
Actual $G_s$								
Assumed $G_s$	2.70	2.70	2.70		2.70	2.70	2.70	2.70
Moisture, %	21.4	32.0	30.2	26.9	24.0	30.8	28.5	26.0
Wet Unit wt, pcf	110.5	119.2	118.3		110.6	107.7	115.5	109.0
Dry Unit wt, pcf	91.0	90.3	90.9		89.2	82.3	89.9	86.5
Dry Bulk Dens.pb, (g/cc)	1.46	1.45	1.46		1.43	1.32	1.44	1.39
Saturation, %	67.6	99.6	95.1		72.6	79.3	87.8	73.8
Total Porosity, %	46.0	46.5	46.1		47.1	51.2	46.7	48.7
Volumetric Water Cont, $\theta_w$ , %	31.1	46.3	43.9		34.2	40.6	41.0	35.9
Volumetric Air Cont., $\theta_a$ , %	14.9	0.2	2.3		12.9	10.6	5.7	12.8
Void Ratio	0.85	0.87	0.86		0.89	1.05	0.88	0.95
Series	1	2	3	4	5	6	7	8

Note: All reported parameters are from the as-received sample condition unless otherwise noted. If an assumed specific gravity ( $G_s$ ) was used then the saturation, porosities, and void ratio should be considered approximate.





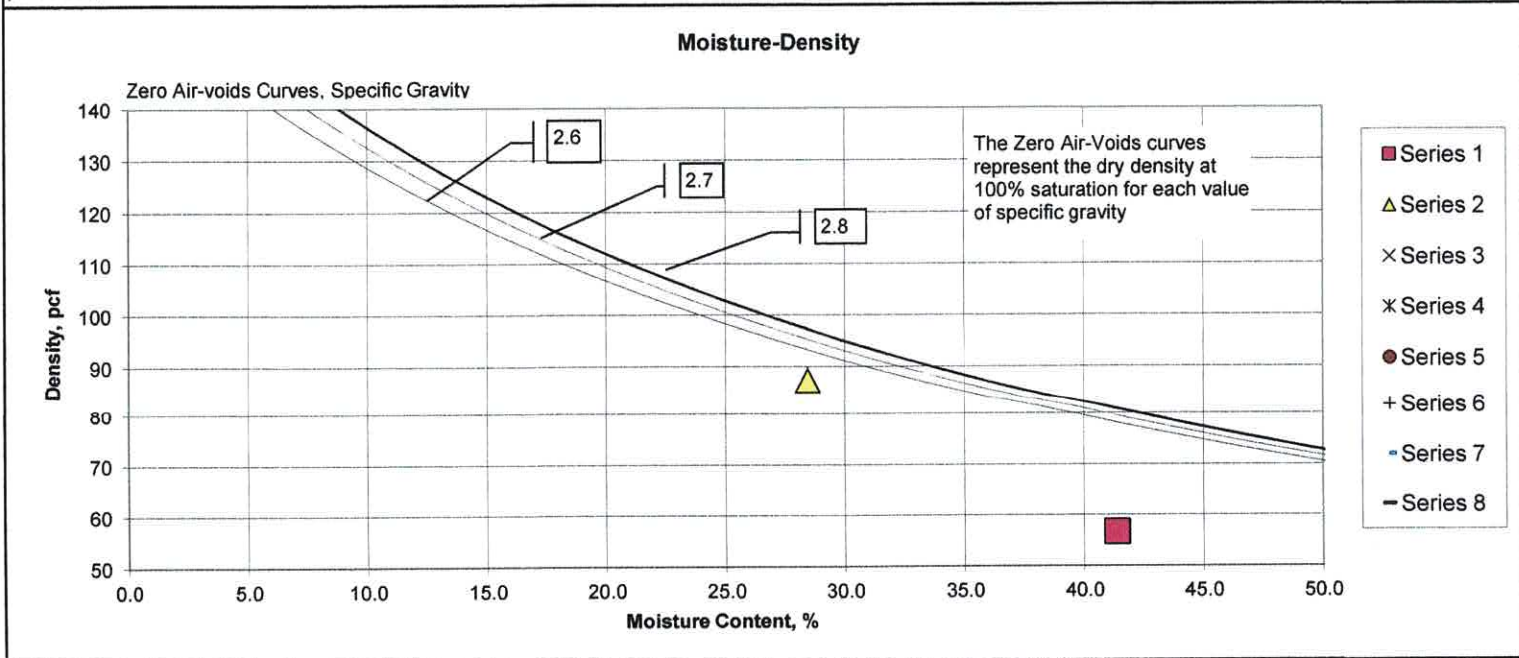
# Moisture-Density-Porosity Report

Cooper Testing Labs, Inc. (ASTM D7263b)

<b>CTL Job No:</b> 060-2882b	<b>Project No.</b> 220116	<b>By:</b> RU
<b>Client:</b> GeoForensics	<b>Date:</b> 08/06/20	
<b>Project Name:</b> Healing	<b>Remarks:</b> 4-3 - sample dsturbed; m/c only.	

Boring:	4-1	4-2	4-3					
<b>Sample:</b>								
<b>Depth, ft:</b>								
<b>Visual Description:</b>	Yellowish Brown Sandy Fat CLAY	Dark Reddish Brown Sandy CLAY (Claystone)	Dark Reddish Brown Sandy CLAY (Claystone)					
<b>Actual <math>G_s</math></b>								
<b>Assumed <math>G_s</math></b>	2.70	2.70						
<b>Moisture, %</b>	41.4	28.4	25.5					
<b>Wet Unit wt, pcf</b>	80.2	112.1						
<b>Dry Unit wt, pcf</b>	56.7	87.3						
<b>Dry Bulk Dens.pb, (g/cc)</b>	0.91	1.40						
<b>Saturation, %</b>	56.6	82.3						
<b>Total Porosity, %</b>	66.4	48.3						
<b>Volumetric Water Cont., <math>\theta_w</math>, %</b>	37.6	39.7						
<b>Volumetric Air Cont., <math>\theta_a</math>, %</b>	28.8	8.6						
<b>Void Ratio</b>	1.97	0.93						
<b>Series</b>	1	2	3	4	5	6	7	8

Note: All reported parameters are from the as-received sample condition unless otherwise noted. If an assumed specific gravity ( $G_s$ ) was used then the saturation, porosities, and void ratio should be considered approximate.



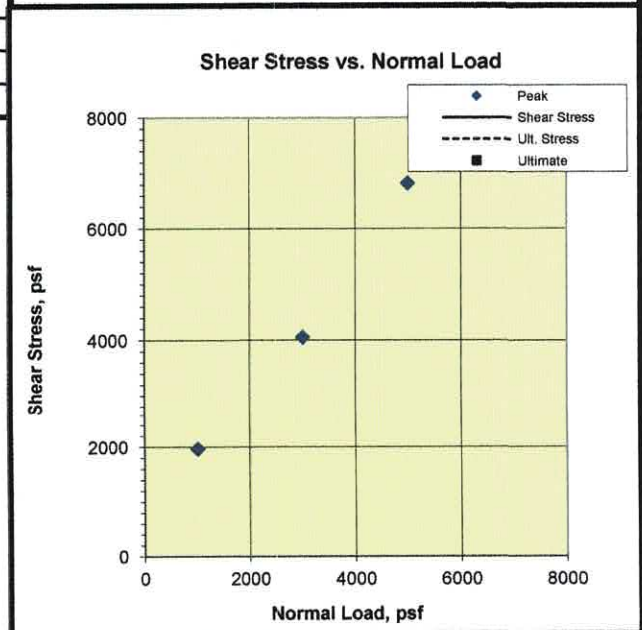
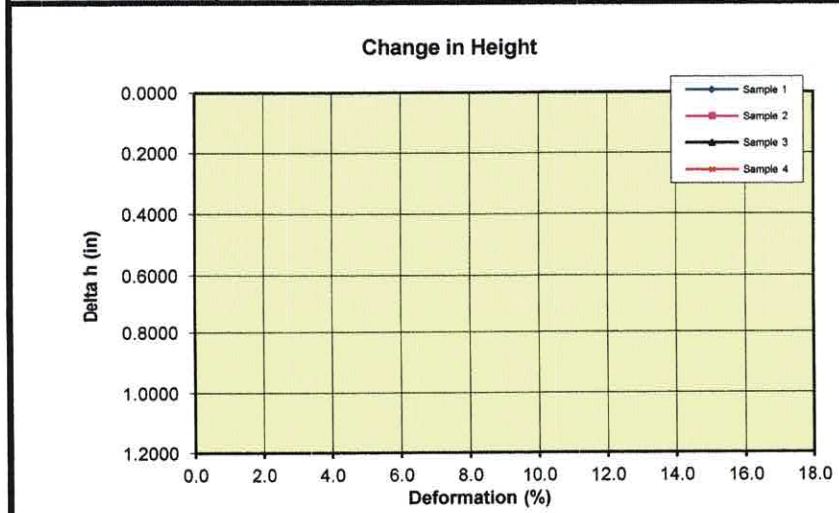
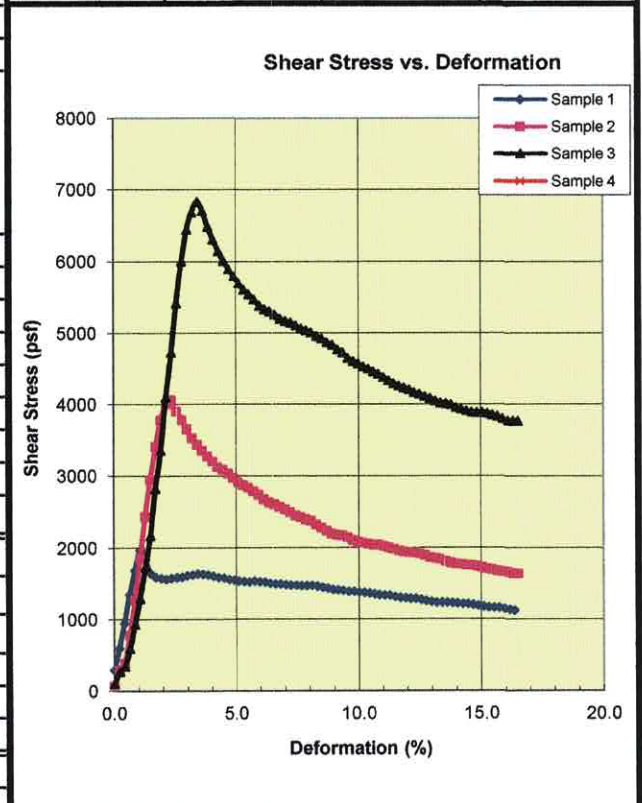


## Consolidated Undrained Direct Shear (ASTM D3080M)

CTL Job #: <u>060-2882</u>	Project #: <u>220116</u>	By: <u>MD</u>
Client: <u>GeoForensics</u>	Date: <u>8/4/2020</u>	Checked: <u>PJ</u>
Project Name: <u>Healing</u>	Remolding Info: _____	

Specimen Data			
	1	2	3
Boring:	2-1	2-1	2-1
Sample:			
Depth (ft):			
Visual Description:	Dark Brown Sandy CLAY (Claystone)	Dark Brown Sandy CLAY (Claystone)	Dark Brown Sandy CLAY (Claystone)
Normal Load (psf)	1000	3000	5000
Dry Mass of Specimen (g)	113.8	116.1	118.8
Initial Height (in)	1.01	1.00	1.01
Initial Diameter (in)	2.42	2.42	2.42
Initial Void Ratio	0.805	0.754	0.725
Initial Moisture (%)	21.8	20.6	21.3
Initial Wet Density (pcf)	113.7	115.8	118.5
Initial Dry Density (pcf)	93.4	96.1	97.7
Initial Saturation (%)	73.0	73.6	79.3
ΔHeight Consol (in)	-0.0006	0.0100	0.0107
At Test Void Ratio	0.806	0.737	0.706
At Test Moisture (%)	28.7	25.5	24.8
At Test Wet Density (pcf)	120.1	121.8	123.3
At Test Dry Density (pcf)	93.3	97.0	98.8
At Test Saturation (%)	96.1	93.4	94.9
Strain Rate (%/min)	1.1	1.1	1.1
Strengths Picked at	Peak	Peak	Peak
Shear Stress (psf)	1969	4061	6831
ΔHeight (in) at Peak			
Ultimate Stress (psf)			

Phi (deg)	Ult. Phi (deg)
Cohesion (psf)	Ult. Cohesion (psf)



**Remarks:** \*DS-CU\* A fully undrained condition may not be attained in this test. ΔH is not measured during undrained direct shear tests. Due to the high apparent phi angle, no phi or cohesion is reported. To add phi and cohesion to the report go to the "phi" tab and in cells G30, G31, H30, and H31 enter end points for a line through the 3 data points. The points plotted can be changed on the "Eng Values" tab using cells L6, A2, C2, and E2.



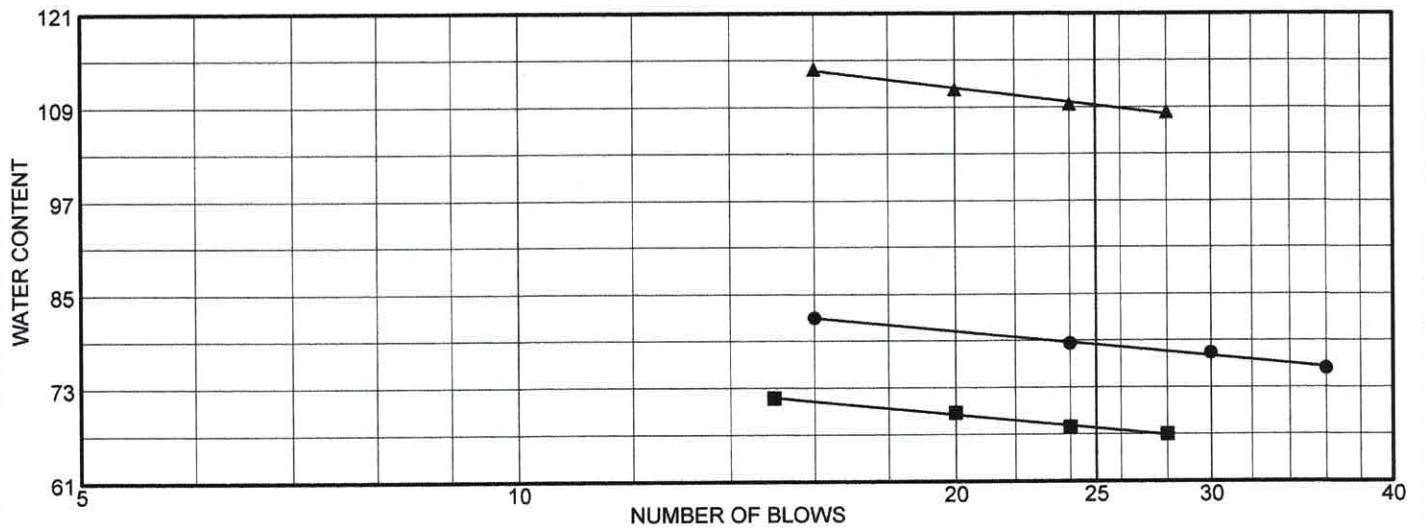
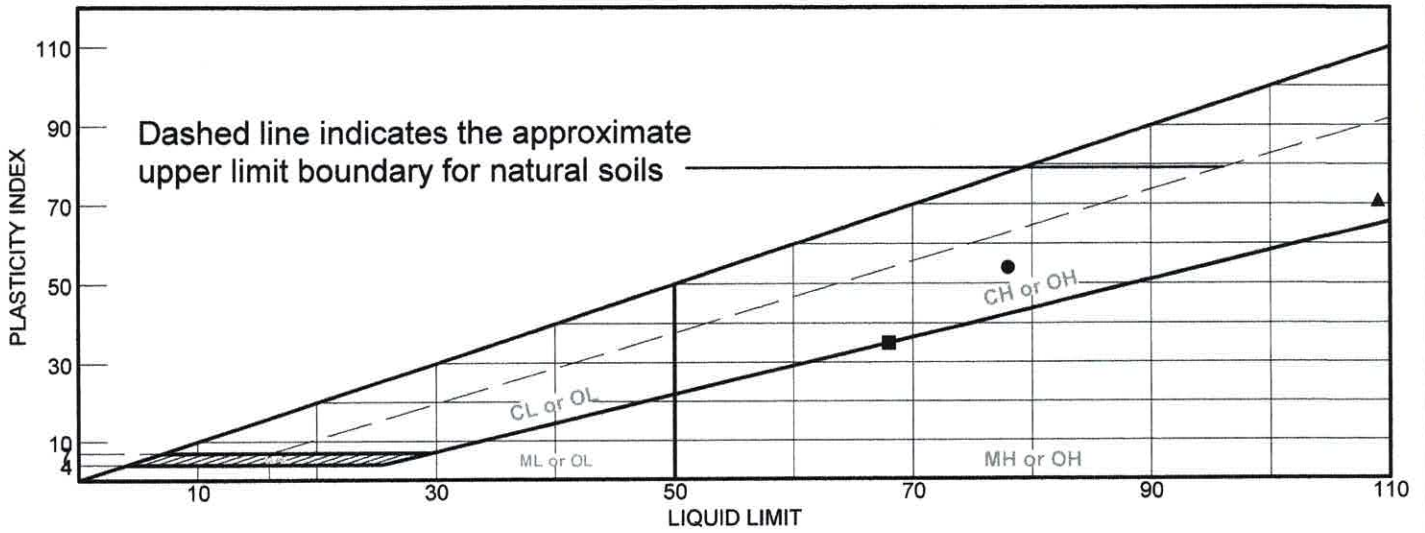
**Organic Content Test**  
**ASTM D 2974-00 (Method C - 440 °C)**

<b>CTL JOB NO.:</b>	060-2882	<b>PROJECT:</b>	Healing	<b>DATE:</b>	8/6/2020
<b>CLIENT :</b>	GeoForensics	<b>PROJECT NO.:</b>	220116	<b>BY:</b>	RU
<b>Boring :</b>	2-3				
<b>Sample :</b>					
<b>Depth (ft.):</b>					
<b>Visual Description:</b>	Dark Brown CLAY				
<b>Dish No.</b>					
<b>Dish wt., gm</b>	80.93				
<b>Soil, Org, Dish &amp; H<sub>2</sub>O, gm</b>	200.24				
<b>Oven Dry wt (105°C), gm</b>	170.00				
<b>Furnace Dry wt. (440°C), gm</b>	166.13				
<b>Moisture Content, % of Oven Dry Mass</b>	34.0				
<b>Organic Matter, %</b>	4.3				

**Note:** ASTM provides no guidelines for including information about the organic content of a sample in the description when the wet/dry liquid limit data is not available. CTL developed the following guidelines to fill this gap:

- 0-5%: The organics are either not mentioned or mentioned as being "trace".
- 5-15%: The soil is considered as inorganic and is classified, as per ASTM 2487, with "with organics" included in the desc
- 15-50%: The soil is considered as organic and is described, per ASTM 2487.
- > 50%: The soil is described as "Peat".

# LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Dark Brown Fat CLAY	78	24	54			
■	Yellowish Brown Sandy Elastic SILT	68	33	35			
▲	Yellowish Brown Sandy Fat CLAY	109	38	71			

**Project No.** 060-2882     **Client:** GeoForensics  
**Project:** Healing - 220116

● **Source:** 1-2  
 ■ **Source:** 3-1  
 ▲ **Source:** 4-1

**Remarks:**

●  
 ■  
 ▲