



AGNEW CIVIL ENGINEERING
CIVIL AND GEOTECHNICAL CONSULTANTS

454 LAS GALLINAS AVE - SUITE 1047
SAN RAFAEL, CA 94903

TEL 415 868 5532
FAX 415 472 0603
ADMIN@AGNEWCIVIL.COM

May 7, 2020

John Beug and Cidney Vinall
11786 CA-1
Point Reyes Station, CA 94956

RE: Geotechnical Investigation
New Residence and Related Work
70 Fox Drive
Inverness, CA 94956

Dear Mr. Beug and Mrs. Vinall:

In accordance with your request, the undersigned engineer has completed a geotechnical investigation for the proposed construction of a new single-family residence with a detached garage, driveway, and a new onsite wastewater treatment system now being considered on your property in Inverness, California. The following report presents the results of my research, subsurface exploration, and geotechnical analyses, along with suitable recommendations, design parameters, and construction criteria for the geotechnical aspects of the proposed work.

1. PROJECT DESCRIPTION

Based on my conversation and a review of the preliminary design package by your designer Paul Discoe Design, it is my understanding that the proposed new work will primarily consist of a new single-family home with a detached garage. The new home will be built at or near existing grades with only moderate earthwork, mainly foundation and retaining wall excavation and terracing. All appropriate surface and subsurface drainage improvements and moisture-proofing measures should also be included, as required with the work.

2. PURPOSE AND SCOPE OF WORK

The purpose of the geotechnical investigation was to evaluate the soil and geologic conditions at the site to identify the relevant geotechnical design constraints as well as potential geologic hazards that may affect the proposed work. The subsurface information and laboratory test data have also been used to develop appropriate recommendations and design parameters for the geotechnical aspects of the proposed work, including grading and excavation, new building and wall foundations, retaining walls, slabs-on-grade, surface and subsurface drainage measures, and erosion control. Appropriate CBC seismic design criteria for this site have also been provided.

This investigation has been completed in general compliance with currently accepted standards for residential geotechnical reports in this area. As outlined in a proposal dated September 27, 2019, the scope of work performed for this investigation included the following tasks:

- A site reconnaissance to observe the current surface conditions at the site,

- A review of the published geologic and landslide maps of the area and related information, as well as available design information for the project. Evaluation of available satellite imagery on Google Earth was also examined.
- Subsurface exploration consisting of drilling and sampling with portable equipment a total of three exploratory test boring(s) to characterize the soil and bedrock materials at the site. The boring(s) were located near the perimeter and in the footprint of the new building area(s). The boring(s) were done in accordance with ASTM D1586.
- Laboratory testing of the soil and bedrock samples obtained from the boring(s).
- Evaluation and geotechnical engineering analysis of the collected data.
- Preparation of this detailed report to summarize the relevant findings and conclusions, with suitable recommendations, design parameters, and construction criteria for the geotechnical aspects of the project.

Although not anticipated here, it should be noted that this investigation did not include any assessment of environmental hazards or contamination that may be present. Building records and other historical or archival research was also beyond the scope of the investigation. In addition, no testing or evaluation for the presence of sulfates or other corrosive materials was completed as part of the investigation. Finally, due to changes in geotechnical engineering standards, county requirements and possible changes in the site conditions that could occur with time, this report should only be used for a period of about three years from the above date unless a written update or plan review letter is obtained from this office.

3. SITE DESCRIPTION AND TOPOGRAPHY

The USGS topographic map for this area, reproduced as Plate 1, indicates that the subject site is located in moderately sloping hillside terrain in the foothills of the residential area known as Inverness in western Marin County (USGS, 1954, rev. 1971). The site is situated on a minor spur ridge roughly above the 400 elevation contour. Natural slope inclinations at and near the site are typically on the order of about 5:1 (horizontal to vertical), with some of the surrounding slopes as steep as 4:1 and locally steeper on the adjacent ridgeline to the west. Drainage is by sheet flow runoff, mostly toward the major drainage swale to the southeast.

The subject parcel is a large irregularly-shaped lot, roughly about 20.0 acre in size. The lot is currently covered by numerous large and small oak trees, with occasional fir and bay trees, by grasses, weeds, and minor brush. No obvious indications of any recent or historic landslides were visible at the site.

As shown on Plate 2, the proposed building area for the new residence is located mostly in the gently sloping area near the top of the ridge.

4. GEOLOGY

The site is located in the central Coast Ranges, a geomorphic province of California typically characterized by steeply sloping hillside areas separated by incised drainages, creating a “knob and swale” topography in this region. The overall geologic structure of the area roughly parallels that of the San Andreas and Hayward faults and generally trends northwest-southeast.

The San Andreas Fault Zone is a dividing line between rocks of disparate origin and represents the classic boundary between the North American and Pacific plates—with the Point Reyes Peninsula residing on the Pacific Plate, and the rest of Marin County being part of the North American Plate. Olema Valley and the submerged valleys flooded by Tomales Bay and Bolinas Lagoon are part of the San Andreas Rift Valley.

The salient geologic feature of western Marin is the San Andreas fault zone, a fracture zone along which great segments of the earth's crust (the Pacific plate on the west and the North American plate on the east) are moving past one another. This plate movement has continued for millions of years at an average rate of about 2 cm per year. West of the fault Late Cenozoic marine and continental rocks rest upon Cretaceous granitic basement rock. East of the fault is a mélangé unit of the Franciscan Complex; this chaotic unit has undergone intense deformation for tens of millions of years. This unit is overlain in places by the Pleistocene Millerton Formation and other Quaternary deposits, such as terrace gravel, alluvium, and bay mud (see Plate 6).

Regional geologic mapping (Clark and Brabb, 1997) indicates that the site is underlain by Granodiorite and Granite of Inverness Ridge, where dikes and masses of aplite and alaskite are locally abundant.

The USDA Soil Survey (Kashiwagi, 1978) describes the surface soils at the site as Inverness loam. This map unit consists of deep, well-drained soil on the uplands.

5. LANDSLIDING AND SLOPE STABILITY

The preliminary landslide maps published for this area by the USGS (Wentworth, Wagner, 1977) indicates that the subject lot is underlain by bedrock. No landslide deposits are mapped by Wentworth on or near the site. It should be noted Wagner shows a large slide to the north on the other side of the ridge. Additionally, no debris-flows have historically been mapped at the site (Ellen 1997, Davenport 1982). The USGS slope stability map (Wagner 1977) classifies the site as Category 3, or marginally stable.

No obvious indications of any recent or historic landslides were evident on the lot during the current investigation. Fortunately, a review of recent satellite imagery revealed no indications of any recent slide activity in this area of the site, even during recent wet winters such as 2005-2006.

6. FAULTING AND SEISMICITY

The subject lot is located within the seismically active San Francisco Bay Area, where the risk of earthquake hazards is very real. However, the lot is not within a State of California Earthquake Fault Zone for active faults (Hart, et al, 1997). Since there are no known active faults on or immediately adjacent to the site, the potential for on-site ground rupture from faulting is considered to be very low.

Given the lack of any observed recent or historical landslides, the risk of seismically induced large-scale land sliding at the site also appears to be low. However, due to the steep terrain, there is some risk of raveling or minor sloughing of some of the steeper slope areas, especially if a strong earthquake occurs during wet conditions. Recommendations for appropriate construction measures to improve the stability of the steeper slope areas are provided in a later section.

Under the Alquist-Priolo Earthquake Fault Zoning Act. The California Division of Mines and Geology (now known as the California Geological Survey) produced 1:24,000 scale maps showing known active and potentially active faults and defining zones within which special fault studies are required. The nearest special study zone of the San Andreas Fault is located approximately 0.61 kilometers (2000 feet) northeast of the site. The approximate location of the 1906 break is about 3000 feet northeast of the site. While there are no

known measurements of offset on this specific trace, offsets were reported to be on the order of 20 feet near Olema, approximately 2.9 kilometers (1.8 miles) southeast of the site.

The site does not lie within the limits of an Alquist-Priolo Special Studies Zone, and no evidence to confirm active faulting was observed during our investigation. However, the site lies in immediate proximity to the active San Andreas Fault, and lies just west of a documented break from the 1906 earthquake. Therefore, we judge there is a low risk of surface rupture resulting from future earthquakes within the San Andreas Fault Zone.

Due to the proximity of the site to these and other major faults in the region, strong ground shaking should be expected in the event of a moderate or major earthquake, particularly on one of the closer faults. The most recent 2015 USGS Uniform California Earthquake Rupture Forecast (UCERF3) has estimated that there is at least a 72 percent chance of a major damaging earthquake (magnitude 6.7 or greater) on one of the active faults in the greater Bay Area by the year 2044, with the nearby Hayward fault assigned the highest probability for this event (USGS, 2015).

Therefore, the proposed new construction should be designed to meet or exceed the seismic design criteria as outlined in the latest revision of the CBC (2019) and in accordance with the ASCE 7-16 Standard. Given the proximity to the active San Andreas fault, the following criteria apply:

Table 1 - Seismic Design Criteria

Latitude: 38.05417731 ° N Longitude: -122.81857441 ° W	ASCE 7-16 Table/Figure	Factor/Coefficient	Value
Mapped Peak Ground Acceleration MCE _G	Figure 22-7	PGA	1.031g
Seismic Design Category	Table 11.6-1	SDC	E
Short-Period MCER at 0.2s	Figure 22-1	S _S	2.411g
1.0s Period MCER	Figure 22-2	S ₁	1.011g
Soil Profile Type	Table 20.3-1	Site Class	D (Stiff Soil)
PGA Site Coefficient	Table 11.8-1	F _{PGA}	1.1
Short Period Site Coefficient	Table 11.4-1	F _a	1.2
1.0s Period Site Coefficient	Table 11.4-2	F _v	1.7
Adjusted MC Spectral Response Parameters	Equation 11.8-1	PGA _M	1.134
	Equation 11.4-1	S _{MS}	2.411g
	Equation 11.4-2	S _{M1}	1.719g
Spectral Acceleration Parameters	Equation 11.4-3	S _{DS}	1.608g
	Equation 11.4-4	S _{D1}	1.146g
Long-Period Transition Period	Figure 22-12	T _L	12s

7. SUBSURFACE EXPLORATION PROGRAM

The subsurface conditions in the vicinity of the proposed construction were investigated by sampling a total of three exploratory boring(s) at the approximate locations shown on the Site Plan, Plate 2. All boring(s) had a maximum diameter of 3.5 inches. The boring(s) were advanced using portable equipment with solid flight augers and conventional sampling equipment with a 140-pound hammer.

Boring 1 was located near the south downslope portion of the building area, Boring 2 was located further north and upslope near the middle of the building area, and Boring 3 was located downslope near the proposed garage. The boring(s) extended to a maximum depth of about 21.5 feet and were terminated after bedrock penetration was confirmed.

Further details of the subsurface exploration program are provided in Appendix I. Samples of the surface and subsurface materials from the boring(s) were recovered at selected intervals using conventional drive samplers. The samples were sealed and transmitted to the laboratory for appropriate testing, as discussed in the following section. Detailed descriptions of the materials encountered in the boring(s) and the specific laboratory test results are presented on the attached boring log(s), which are also included in Appendix I. The subsurface conditions encountered in the boring(s) are also summarized in the paragraphs below.

7.1. SOILS

As expected, a natural soil profile underlain by highly weathered bedrock was discovered in all borings. The topsoil in all borings was described as stiff, very clayey, sandy Silt. The topsoil extended to a depth of 2.0 feet and was underlain by colluvium.

The colluvium in all borings was described as firm to stiff, very silty, sandy clay with numerous to abundant pebbles and decomposed granite rock fragments. The colluvium extended to a depth of about 5.0 to 7.0 feet and was underlain directly by weathered bedrock. The colluvium possessed a low plasticity and a low expansion potential on the basis of the Atterberg Limits test, see Plate 5.

7.2. BEDROCK

As noted above, generally competent weathered bedrock materials were encountered beneath the soil profile at all boring locations at depths of about 5.0 to 7.0 feet. The bedrock unit observed in the borings is described as a granodiorite and granite. These bedrock materials were mottled olive-brown and strong brown, very severely to pervasively weathered, very severely fractured, and very friable with typically a very soft hardness but no obvious localized shearing to suggest landslide debris. The bedrock typically became harder and denser with depth.

These relatively weak bedrock materials typically had low sample blow counts where severely weathered or decomposed, but with generally increasing blow counts with depth. The bedrock units observed in the borings were also consistent with the granitic rocks of Inverness Ridge, as described for this area on the published geologic maps discussed above and are generally similar to the bedrock materials seen in nearby cuts.

7.3. GROUNDWATER

Groundwater was only encountered in Boring 3 at a depth of 16 feet at the time of drilling and was still at 16 feet one hour later. The boring(s) were then backfilled with lean cement grout immediately after drilling per County Health requirements and no further groundwater measurements were possible. It should be noted

that temporary shallow groundwater and seepage are often a transient but expected condition, especially near the soil/bedrock contact, along secondary drainage swales, bay margins, seasonal stream channels, and particularly after heavy rains. In addition, groundwater levels may change over time and will vary with seasonal rainfall patterns, long-term climate fluctuations, and with the influence of irrigation or any landscape watering.

8. LABORATORY TESTING

The laboratory investigation was directed toward a quantitative and qualitative evaluation of the physical and mechanical properties of the in-place soil and bedrock materials at the subject site. The following tests were performed on selected samples from the field investigation in accordance with the standards of the American Society for Testing Materials (ASTM) or contemporary practices:

Table 2 - Laboratory Testing

Laboratory Test	Test Method	Number of Tests
In-situ Moisture Content	ASTM D-2216	19
In-situ Dry Density	ASTM D-2216	6
Atterberg Limits	ASTM D-4318	1
Wash Over 200 Sieve.	ASTM D-1140	1
Sieve Analysis	ASTM D-422	N/A
Unconfined Compression	ASTM D-1266	N/A

The moisture content tests determine the relative degree of soil saturation and are used to evaluate the moisture profile in accordance with ASTM Test Designation D-2216. These water contents are recorded on the boring log(s) at the appropriate sample depths.

The dry density tests evaluate variations in material density and the degree of compaction or consolidation. The dry density tests are also useful for comparing the densities of the in-place materials and their relative strengths with the sample blow count. The results of these tests are also shown on the boring log(s) at the appropriate depths.

The Atterberg limits tests help classify and evaluate the expansion potential of a representative sample of the clayey soils. The Atterberg limits were determined in accordance with ASTM Test Designations D-4318. These values are used to classify the soil in accordance with the Unified Soil Classification System and to evaluate the soil's expansion potential, compressibility, and approximate strength. The results are presented on the boring log(s) at the appropriate sample depth and on the attached Plate 5.

The results of the applicable laboratory tests are presented in Appendix I on the boring log(s) at the approximate sample depths.

The percent passing the No. 200 sieve was performed on a sample of the native soils to assist in the proper classification of this material as either predominantly granular (sand and gravel) or fine-grained (silt or clay) in terms of overall composition by measuring the percentage of fines as well as the sand and gravel fractions. This test was performed in accordance with ASTM Test Designation D-1140, and the results are shown on the boring log at the appropriate sample depth.

9. DISCUSSION AND CONCLUSIONS

9.1. GENERAL

Based on the results of this investigation, it is the opinion of the undersigned civil engineer that the site is suitable for the proposed new residence, garage, and related construction.

However, all of the conclusions and recommendations presented in this report should be incorporated into the design and construction of the project to minimize possible soil and foundation problems. The primary geotechnical concerns are discussed in the following paragraphs, and detailed recommendations for the geotechnical aspects of the proposed work are presented in the next section of this report.

9.2. GEOLOGIC HAZARDS, LANDSLIDES, AND SLOPE STABILITY

The primary geologic hazard for this site is the strong ground shaking expected during a major earthquake on one of the nearby faults in the region, a hazard present on virtually all sites in the greater Bay Area. Given the bedrock setting, no magnified or amplified shaking is expected compared to soft ground sites at similar epicentral distances. In addition, fault rupture hazards are considered low to very low at this site. The primary construction measures to address the risk of strong ground shaking include properly designed foundations and retaining walls in accordance with both current code guidelines and the recommendations of this report.

The risk of seismically-induced landslide hazards can also be considered low for this site provided appropriate construction measures are followed during the work, as outlined below. Fortunately, no recent landslide has been observed on the site. As with most steep hillside lots, the primary slope stability issues are related to the steep terrain, the relatively weak underlying native soils, and the depth to the fractured and weathered bedrock materials.

The risk of other possible ground movement or settlement problems associated with strong seismic shaking was also evaluated for this site, including ground settlement and soil liquefaction. The risk of soil liquefaction is considered low for the dense soils observed under the water table.

Any possible liquefaction that could occur at this site would be limited to relatively clean, saturated sand lenses with low relative density, typically below the water table. However, any liquefaction would be a highly localized phenomena with resulting minimal differential settlements expected to be well within structural tolerances for the deep foundations of the proposed new structures.

In order to maintain long-term slope stability and minimize the risk of future problems, and to reduce the likelihood of any earthquake-induced landslide, the following measures are considered applicable and should be suitably incorporated into the work on this lot:

- The weak and loose native soils can be adequately stabilized using drilled piers and retaining walls as part of the new building construction. The building site should be terraced to the degree possible so that the surface soils are directly supported by retaining walls or foundation grade beams within the building area, resulting in an overall reduction in the effective slope inclination across the new building area. In areas where shallow groundwater is not present, local excavation to remove some of the native soils is also recommended.
- Runoff from the hill above should be adequately controlled and not allowed to flow onto the lot in an uncontrolled and concentrated manner.

- Collected drainage at the site should be discharged with properly designed energy dissipation structures at the discharge points, preferably at least two.
- In other areas, maintaining proper erosion control measures and ground cover plantings for the surface soils as well as appropriate surface drainage controls above and around the building area will help prevent possible future sloughing, raveling or erosion of the steeper slope areas.
- Retaining walls or site walls supported by deep drilled piers, such as along the driveway, should also be considered as a way to provide additional terracing to reduce the overall steepness of the finish slopes around the proposed building area. Surface runoff should also be collected above these walls and along the driveway and routed to suitable discharge points, as noted above.

9.3. FOUNDATIONS AND RETAINING WALLS

The steeply sloping terrain, the weak surface soils, as well as the depth of any required excavation in the building area, are the primary geotechnical considerations for foundation design at the site.

In order to minimize long-term settlement problems or slope creep effects, most new foundations for the residence and for all site walls, driveway support or appurtenant structures in sloping areas should be extended through the relatively weak native soils using drilled piers that derive support by skin friction in the underlying competent bedrock.

Although wall cuts over 10 feet high are not expected and not recommended, if any required excavation along the uphill side of the building area exposes competent bedrock at the subgrade level, the main wall foundation could consist of a conventional footing bearing directly on the bedrock provided a nearly level pad or terrace is present for at least 10 feet on the downslope side of the footing per UBC requirements. This condition may be very difficult to meet on this lot. Due to the depth to bedrock, drilled pier support will likely be needed for most structural wall footings. In addition, it should be noted that drilled piers in wall footings may also be more suitable for lateral support in lieu of a shear key, which could be difficult to construct and maintain dimensional control in the fractured bedrock.

For any deeper wall cuts over about 5 feet high, the contractor should also consider special shoring or excavation measures to minimize potential problems such as undermining of the areas adjacent to the wall cut. These measures should include, but may not be limited to, rounding back the top of the cut in the upper 3 to 5 feet of fill or topsoil, keeping cut faces moist with misters and plastic sheeting, adding closely-spaced drilled piers (“stitch piers”) along the top of the cut in the most critical areas, and/or completing the wall(s) in sections. No wall cuts should be considered in areas of weak saturated surface soils.

Major excavation and site work should also be limited to the dry season, and ideally should not be attempted during the wet winter months to reduce the risk of problems during construction.

Finally, due to expected variations in the depth to bedrock, Special Inspection of all pier drilling and foundation excavations will be required by the County to verify that competent bedrock materials are encountered at the expected depths in all areas.

9.4. DRAINAGE AND EROSION CONTROL

As with all hillside homes, surface and subsurface drainage should be carefully controlled around and above the building area to minimize uncontrolled surface runoff, seepage problems, and any subsequent weakening or erosion of the surface soils.

As noted above, runoff should not be allowed to flow uncontrolled onto the site from the hill above. The collected drainage at the site should also be discharged directly to at least two properly designed energy dissipators at discharge points near the lower part of the lot. Appropriate waterproofing, seepage collection, and sub-drainage facilities should also be provided behind all structural walls for the home and for the site walls above the building area.

Both temporary and permanent erosion control measures should also be included as required with the work. Ideally, the grading and foundation construction should be pursued in the dry summer and fall months, with foundation excavation and retaining wall construction largely completed by October 15 unless special drainage controls and temporary erosion control measures are implemented.

The specific geotechnical recommendations for this site and the appropriate geotechnical design parameters and construction criteria for the project are presented in the following section.

10. RECOMMENDATIONS AND DESIGN PARAMETERS

10.1. GENERAL SITE GRADING

10.1.1. The civil engineer should be notified at least four (4) working days prior to any site clearing, grading, and foundation excavation so that the work in the field can be coordinated with the grading contractor and arrangements for testing and observation can be made. The recommendations of this report are based on the assumption that the civil engineer will perform the required testing and observation during grading and construction. It is the owner's responsibility to make the necessary arrangements for these required services. All of the following recommendations may not be applicable for the current project scope. However, some have still been included to address any changes in the project scope.

10.1.2. Areas to receive foundations or to be graded should be cleared of obstructions, vegetation, and other unsuitable material. Surface vegetation, roots, and organically contaminated soil should be removed from areas to be graded. A stripping depth of 8 inches is anticipated.

10.1.3. Where fill is planned to raise grade, area to receive engineered fill should be scarified 6 inches, moisture conditioned to about 1 to 2 percent over optimum moisture content and compacted to at least 90 percent relative compaction.

10.1.4. Engineered fill should be moisture conditioned to about 1 to 2 percent over optimum moisture content, placed in thin lifts not to exceed 8-inches in loose thickness and compacted to at least 90 percent relative compaction.

10.1.5. The relationship between moisture content and dry unit weight shall be based on ASTM Test Designation D1557-00. The relative density and moisture content of the compacted soil shall be based on ASTM D2922-04.

10.1.6. Soils used for engineered fill should be non-expansive, have a Plasticity Index less than 15, be free of organic material, and contain no rocks or clods greater than 6 inches in diameter, with no more than 15 percent larger than 4 inches.

10.1.7. At a minimum, the top 8 inches of subgrade soil below non-load bearing concrete slabs-on-grade should be moisture conditioned to about 1 to 2 percent over optimum moisture content and compacted to at least 90 percent relative compaction.

10.1.8. The upper 12 inches of subgrade below driveway pavements should be moisture conditioned to about 1 to 2 percent over optimum moisture content and compacted to at least 95 percent relative compaction. The aggregate base below pavements should also be compacted to at least 95 percent relative compaction.

10.1.9. Temporary cut slopes should be inclined no steeper than 0.5:1 (horizontal to vertical) during the summer dry season and no steeper than 1:1 during or immediately following the rainy season. Permanent cut slopes excavated into soil should be inclined less than 2:1 (horizontal to vertical).

10.1.10. Fill slopes should be inclined less than 2:1 (horizontal to vertical).

10.1.11. Fill slopes greater than 3 feet in height should be keyed and benched into firm native soil. Keys should be at least 6 feet wide and embedded at least 12 inches into firm native soil for fill slopes up to 6 feet in height. Keys should be at least 10 feet wide and embedded at least 18 inches into firm native soil for fill slopes more than 6 feet high. The keyway dimensions for fill slopes located near existing cut slopes should be evaluated on a case by case basis.

10.1.12. Subdrains should be used at the back of keys and fill slopes. Subdrains should consist of a minimum 12-inch wide column of Caltrans Class 1, Type A permeable material that extends 6 inches below the base of the key or granitic bedrock. A 4-inch perforated rigid collector pipe should be placed about 2 to 4 inches from the base of the gravel. The height of the gravel drain should be determined in the field during construction. The actual locations and depths of subdrains should be determined in the field by the civil engineer at the time of construction.

10.1.13. The face of cut and fill slopes should be groomed to remove any loose soil and create a firm, uniform slope surface.

10.1.14. Engineered fill should be observed and tested by our firm or another qualified professional. Engineered fill over 12 inches in thickness should be continuously observed during fill placement. In-place density tests should be performed as follows: one test for every 12 inches of material placed for fill slopes, in trenches or around structures; one test for every 1,000 square feet for relatively thin fill sections and one test whenever there is a definite suspicion of a change in the quality of moisture control or effectiveness in compaction. The actual testing schedule should be determined by a representative from our firm at the time of grading.

10.1.15. After the earthwork operations have been completed and the civil engineer has finished their observation of the work, no further earthwork operations shall be performed except with the approval of and under the observation of the civil engineer.

10.2. PIER AND GRADE BEAM FOUNDATIONS

10.2.1. All foundations required for the new residence in *proximity to sloping terrain or where deep colluvium is present*, as well as any new foundations for deck supports, site walls or possible future building additions outside the existing building footprint, should be supported on drilled, cast in place, straight shaft concrete piers that are designed to develop their load-carrying capacity through friction between the sides of the piers and the surrounding subsurface materials.

10.2.2. MINIMUM PIER DIAMETER

10.2.2.1. 16 inches

10.2.3. MINIMUM CENTER-TO-CENTER SPACING

10.2.3.1. House Piers: 5 Pier Diameters

10.2.3.2. Larger diameter piers and higher capacity grade beams may be useful in some instances, especially in at-grade areas, to reduce the overall number of piers. Depending on drilling operation difficulty, reducing the number of piers may be advantageous.

10.2.4. EMBEDMENT

10.2.4.1. 10 feet of embedment into competent bedrock at or near sloping areas (3:1 or steeper)

10.2.4.2. 8 feet of bedrock embedment for wall piers in level areas (4:1 or flatter)

10.2.4.3. Since competent bedrock was encountered at depths of about 10 feet, the piers will typically extend to maximum depths in the range of about 20 feet in most at-grade areas, unless substantial excavation is performed. In addition, deeper piers may be locally needed to satisfy structural requirements.

10.2.5. MINIMUM PIER DEPTH

10.2.5.1. Excavation in the proposed building areas would decrease the required pier depths. However, a minimum pier depth of 15 feet should be provided in all sloping areas except for retaining wall footings excavated into competent bedrock in level areas, where a minimum depth of 12 feet is acceptable, unless hard bedrock is encountered, where wall piers in level areas can be reduced to 10 feet.

10.2.6. SKIN FRICTION VALUES

10.2.6.1. Dead Load + Live Load: 750 pounds per square foot (psf)

10.2.6.2. Dead Load + Live Load + Wind or Seismic: 900 psf

10.2.6.3. Where harder bedrock is encountered, the reduced depths can be taken as roughly equivalent to the piers extended to full depth in other areas. Additional depth for the piers

may be needed to accommodate structural requirements. No contribution to skin friction should be used for the overlying weak native soils.

10.2.7. LATERAL LOADS

- 10.2.7.1. Lateral loads on the piers may be resisted by passive pressures acting against the sides of the piers. A passive pressure equal to an equivalent fluid weighing 300 pounds per square foot per foot of depth is recommended up to a maximum value of 3000 pounds per square foot. This value can be assumed to be acting against 1.5 times the diameter of the individual pier shafts starting 4 feet below the bottom of the grade beams when also at least 5 pier diameters from the slope face or from 1 foot below the footing subgrade for any wall piers in level cut areas where at least 10 feet from the slope. During earthquake shaking, these values can be increased by 20%.
- 10.2.7.2. The surficial soils may have a tendency to settle or shrink away from the sides of the piers, leaving them unsupported for the upper few feet. Therefore, the piers in soil areas should be designed as free-standing columns, in accordance with the requirements of the Uniform Building Code, for the top 2 feet. *In addition, to minimize damage resulting from potential slope creep effects, all foundation piers on sloping terrain or within 10 feet from the crest of any slope steeper than 4:1 (horizontal to vertical) be designed to resist a uniform lateral pressure of 300 pounds per square foot acting against the projected diameter of the pier to a depth of 4 feet below the ground surface.*
- 10.2.7.3. The bottom of pier excavations should be reasonably free of loose cuttings and soil fall-in prior to installing reinforcing steel and placing concrete. In addition, any accumulated water in pier excavations should be removed prior to placing reinforcing steel, and concrete (where minor) or the concrete should be placed by tremie methods to the bottom of the hole when substantial groundwater is present. Care should be taken during concrete placement to avoid "mushrooming" at the top of the pier because distress in the building may result from excess slope creep forces on the "mushroom caps". Finally, it is recommended that the contractor use equipment appropriately sized to perform the recommended work. In particular, deep piers extended at least 10 feet into bedrock will be required in difficult access areas of the site; equipment capable of performing in these conditions should be employed. Pier casing or other shaft stabilization measures could be required due to the noncohesive nature of some of the soils.
- 10.2.7.4. All friction piers should be tied together with grade beams tie-beams or wall footings in at least one direction. Maximum horizontal distance between the grade beams or tie-beams should be in the range of 15 feet. The grade beams should be designed to span between the piers in accordance with structural requirements. Due to competent bedrock available at depth, the allowable skin friction values given above should provide substantial support capacity for each pier such that a minimum number of piers can be used.
- 10.2.7.5. However, for greater structural redundancy to resist the effects of earthquake shaking, it is also recommended that the grade beams be designed to resist a nominal uplift pressure of at least 1000 plf. If the grade beams are to retain soil, they should be also designed to resist the appropriate lateral earth pressures provided in Section 10.4, "Retaining Walls". The floor system for the home will most likely be structurally

supported and derive support from the pier-and-grade beam foundations. Concrete interior slab-on-grade floors are only recommended for living space when located in bedrock cut areas.

- 10.2.7.6. Since the actual depth at each pier location will have to be verified in the field, it is important that the pier drilling operations be performed under the observation of the undersigned civil engineer or other qualified professional to confirm that they are constructed in accordance with the recommendations presented herein.

10.3. SPREAD FOOTING FOUNDATIONS

10.3.1. Any structural or non-structural retaining walls that support cuts excavated into competent bedrock can be founded on conventional, continuous footings provided a level terrace at least 8 feet wide is present on the downhill side of the wall and/or drilled pier support is provided on the downhill side of the structural wall footing.

10.3.2. The only exception is for modular block walls less than 4 feet high on cut terraces where the face block can be excavated to sufficient depth as to provide support in competent bedrock. For higher walls, steeply sloping conditions, and where bedrock support is not readily available, drilled pier support should be provided as outlined in the previous section. Most upslope walls along the roadway, where bedrock support could be expected at shallow depth, may be considered suitable for footings depending on the depth of excavation required.

10.3.3. If foundations are embedded into engineered fill, all the existing loose soil under the foundation should be removed and replaced as engineered fill, and there should be at least 12 inches of fill below all footings.

10.3.4. All new wall footings should extend at least 18 inches below the lowest adjacent finished grade and bear directly on competent bedrock. In addition, any footings located adjacent to utility trenches should also have their bearing surfaces below an imaginary 1:1 (horizontal to vertical) plane projected upward from the edge of the bottom of the adjacent trench.

10.3.5. All footings should have a minimum width of 16 inches and should be continuous and structurally interconnected with adjacent foundation or slab elements in at least one direction. Any column footings should have a minimum (least dimension) width of 24 inches and should also be interconnected with adjacent foundation and slab elements, where possible. Continuous footing elements should also have a minimum stem width of at least 8 inches and should be designed as grade beams or tie-beams capable of spanning at least 6 feet. All footings should be tied together with reinforcing steel according to structural requirements.

10.3.6. Foundations designed in accordance with the above may be designed for an allowable soil bearing pressure of 3,000 psf for footings embedded into bedrock and 2,000 psf for footings embedded into engineered fill or competent native soils. The allowable bearing capacities may be increased by 1/3 for short term seismic and wind loads.

10.3.7. Lateral load resistance for structures supported on footings may be developed in friction between the foundation bottom and the supporting subgrade. A friction coefficient of 0.40 is

considered applicable where footings are supported on firm native soil. A friction coefficient of 0.25 is considered applicable where footings are supported on engineered fill.

10.3.8. Where footings are poured neat against the adjacent subgrade, a passive lateral earth pressure of 250 pcf may be used. The top 12 inches of soil should be neglected in passive design. If the foundation is poured neat against the soil, friction and passive resistance may be used in combination. The above values can be increased by 20% during seismic shaking.

10.3.9. Footings located adjacent to other footings or utility trenches should have their bearing surfaces founded below an imaginary 1.5:1 plane projected upward from the bottom edge of the adjacent footings or utility trenches.

10.3.10. Prior to placing concrete, foundation excavations should be cleaned and observed by the civil engineer.

10.4. RETAINING WALL LATERAL PRESSURES

10.4.1. Retaining walls should be designed to resist both lateral earth pressures and any additional surcharge loads.

10.4.2. The following lateral pressures assume that the walls are fully drained to prevent hydrostatic pressure behind the walls. Drainage materials behind the wall should consist of Class 1, Type A permeable material (Caltrans Specification 68-1.025) or an approved equivalent. The drainage material should be at least 12 inches thick. The drains should extend from the base of the walls to within 12 inches of the top of the backfill. A perforated pipe should be placed (holes down) about 2 inches above the bottom of the wall and be tied to a suitable drain outlet. Wall back drains should be plugged at the surface with clayey material to prevent infiltration of surface runoff into the back drains. See Plate-3 Retaining Wall Detail, for more information.

10.4.3. Retaining walls up to 10 feet high may be designed using the following:

Table 3 - Retaining Wall Earth Pressure

Condition	Cut Slopes		Fill Slopes	
	4:1 ¹ or flatter	2:1 to 1.5:1	4:1 or flatter	2:1
Unrestrained	45 pcf ²	60 pcf	50 pcf	65 pcf
Restrained	55 pcf	75 pcf	60 pcf	80 pcf

¹ Inclination behind wall, horizontal to vertical.

² "pcf" signifies "pounds per cubic foot" equivalent fluid pressure.

- A linear interpolation should be used to determine design values for retaining walls where the slope behind the wall is between 4:1 and 2:1.

- For surcharge loads, increase design pressures behind the wall by an additional uniform pressure equivalent to one-half (for restrained condition) or one-third (for unrestrained condition) of the maximum anticipated surcharge load.
- During strong earthquake shaking, for basement or lower level walls and any walls over 6 feet high, the following should be used to calculate the additional resultant dynamic active thrust ΔP_{ae} per Mikola/Sitar 2013. $\Delta P_{ae} = k_h y H^2$, $k_h = \text{larger of } 0.375(S_{ds}/2.5) \text{ or } 0.375(\text{PGA})$, ΔP_{ae} moment should be applied at $0.33H$,

10.5. CONCRETE SLABS-ON-GRADE

10.5.1. At a minimum, the top 6 inches of subgrade soil below exterior non-load bearing concrete slabs-on-grade should be non-expansive import fill materials moisture conditioned to about 1 to 2 percent over optimum moisture content and compacted to at least 90 percent relative compaction.

10.5.2. At a minimum, the top 8 inches of subgrade soil below interior non-load bearing concrete slabs-on-grade should be non-expansive import fill materials moisture conditioned to about 1 to 2 percent over optimum moisture content and compacted to at least 90 percent relative compaction. See Plate-4, Subgrade Preparation Detail, for more information.

10.5.3. Prior to placement of any import fill, the subgrade surface in all slab areas should be scarified to depth of at least 6 inches, moisture conditioned to at least 2-3% above optimum water content, and then compacted to at least 90% relative compaction and should be proof rolled to provide a smooth, firm surface for slab support. See Plate-4, Subgrade Preparation Detail, for more information.

10.5.4. At a minimum, the upper 12 inches of subgrade below concrete driveway pavements should be moisture conditioned to about 1 to 2 percent over optimum moisture content and compacted to at least 95 percent relative compaction. If the pavement is located over existing loose soil, the top 2 feet of loose soil should be removed and replaced as compacted engineered fill. The aggregate base should be compacted to at least 95 percent relative compaction.

10.5.5. All slabs-on-grade can be expected to suffer some cracking and movement. However, thickened exterior edges, a well-prepared subgrade including thorough wetting prior to placing concrete, adequately spaced expansion joints and good workmanship should reduce cracking and movement.

10.5.6. In areas where floor wetness would be undesirable, an expert, experienced with moisture transmission and vapor barriers should be consulted. At a minimum, a blanket of 6 inches of free-draining gravel should be placed beneath the floor slab to act as a capillary break. In order to minimize vapor transmission, an impermeable membrane (minimum thickness 15 mil) should be placed over the gravel. The capillary gravel can be used as the required import fill below interior slab floors. See Plate-4, Subgrade Preparation Detail, for more information.

10.6. PAVEMENTS

10.6.1. To have the selected pavement sections perform to their greatest efficiency, the grading recommendations provided in this report should be closely followed. Subgrade preparation is very important to the life of pavement.

10.6.2. Only quality materials of the type and thickness (minimum) specified should be used. Baserock (R=78 minimum) should meet CALTRANS Standard Specifications for Class 2 Untreated Aggregate Base. Subbase (R=50 minimum) should meet CALTRANS Standard Specifications for Class 2 Untreated Aggregate Subbase.

10.6.3. Place concrete and asphalt only during periods of fair weather when the free air temperature is within prescribed limits.

10.6.4. Develop a maintenance program and perform routine maintenance.

10.6.5. Sufficient gradients should be provided for rapid runoff of stormwater and to prevent ponding water on or adjacent to the pavement.

10.7. UTILITY TRENCHES

10.7.1. Utility trenches placed parallel to structures should not extend within an imaginary 1.5:1 (horizontal to vertical) plane projected downward from the bottom edge of the adjacent footing (pier and grade beam ok).

10.7.2. Trenches should be shored in accordance with appropriate safety codes.

10.7.3. Trenches may be backfilled with compacted engineered fill placed in accordance with the grading section of this report. The backfill material should not be jetted in place.

10.7.4. The portion of utility trenches that extend below foundations should be sealed with 2-sack sand slurry (or equivalent) to prevent subsurface seepage from flowing under buildings

10.8. SITE DRAINAGE

10.8.1. Controlling surface and subsurface runoff is important to the performance of the project. Runoff from the slope above the building area and runoff from the building area should be collected and discharged in a controlled manner.

10.8.2. Runoff from slopes should be collected or diverted around the building area.

10.8.3. Surface drainage should include provisions for positive gradients so that surface runoff is not permitted to pond adjacent to foundations or other improvements. Where bare soil or pervious surfaces are located next to the foundation, the ground surface within 10 feet of the structure should be sloped at least 5 percent away from the foundation. Where impervious surfaces are used within 10 feet of the foundation, the impervious surface within 10 feet of the structure should be sloped at least 2 percent away from the foundation. Swales should be used to collect and remove surface runoff where the ground cannot be sloped the full 10-foot width away from the structure. Swales should be sloped at least 2 percent towards the discharge point.

10.8.4. Swales may be used to collect and remove surface runoff where the ground cannot be sloped the full 10-foot width away from the structure. Swales located within 10 feet of structures should be sloped at least 2 percent towards the discharge point.

10.8.5. Full roof gutters should be placed around the eaves of the structures. Discharge from the roof gutters should be conveyed away from the downspouts and discharged in a controlled manner.

10.8.6. Collected water should not be discharged back into the ground around the proposed structures. Water should also not be discharged into the ground above or onto graded slopes. It may be possible to discharge water back into the ground at the base of the site.

10.8.7. Both temporary and permanent erosion control measures should also be included as required with the work. Ideally, the grading and foundation construction should be pursued in the dry summer and fall months, with foundation excavation and retaining wall construction largely completed by October 15 unless special drainage controls and temporary erosion control measures are implemented.

10.9. PLAN REVIEW, CONSTRUCTION OBSERVATION, AND TESTING

10.9.1. Agnew Civil Engineering should be provided the opportunity for a general review of the final project plans prior to construction to evaluate if our geotechnical recommendations have been properly interpreted and implemented. If our firm is not accorded the opportunity of making the recommended review, we can assume no responsibility for misinterpretation of our recommendations. We recommend that our office review the project plans prior to submittal to public agencies, to expedite project review. Agnew Civil Engineering also requests the opportunity to observe and test grading operations and foundation excavations at the site. Observation of grading and foundation excavations allows anticipated soil conditions to be correlated to those actually encountered in the field during construction. Of particular importance on this project would be observations during foundation excavations and pier drilling to confirm the depth to competent bedrock materials in all areas.

11. CLOSURE AND LIMITATIONS

The recommendations of this report are based upon the assumption that the soil conditions do not deviate from those disclosed in the boring(s). If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that planned at the time, our firm should be notified so that supplemental recommendations can be given.

This report is issued with the understanding that it is the responsibility of the owner, or his representative, to ensure that the information and recommendations contained herein are called to the attention of the Architects and Engineers for the project and incorporated into the plans, and that the necessary steps are taken to ensure that the Contractors and Subcontractors carry out such recommendations in the field. The conclusions and recommendations contained herein are professional opinions derived in accordance with current standards of professional practice. No other warranty expressed or implied is made.

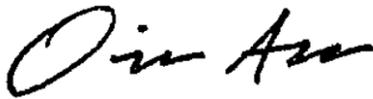
The findings of this report are valid as of the present date. However, changes in the conditions of a property can occur with the passage of time, whether they are due to natural processes or to the works of man, on this or adjacent properties. In addition, changes in applicable or appropriate standards occur whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated, wholly or partially, by changes outside our control. Therefore, this report should not be relied upon after a period of three years without being reviewed by a civil engineer.

All people who own hillside properties should realize that some future erosion, sliding or other detrimental ground movements are always a possibility, although generally, the likelihood is very low that damaging events will actually occur. The probability of severe ground movements or erosion is substantially reduced by the proper maintenance of adequate drainage and erosion control measures at the site. Therefore, the owner(s) should recognize their responsibility for performing adequate maintenance. A copy of this report should also be provided to all future homeowners of the properties to outline these maintenance requirements.

The undersigned engineer has employed accepted geotechnical and civil engineering procedures, and the professional opinions and conclusions have been made in accordance with generally-accepted principles and practices at the time this work was performed. This warranty is in lieu of all other warranties, either expressed or implied.

I hope this report provides you with the information you require at this time and appreciate the opportunity to be of service on this project. If you have any questions regarding this report, please feel free to call me at your earliest convenience.

Sincerely,



Orion W. Agnew CE
Principal Civil Engineer

OWA/ohs
C:\...\reports\Beug.rpt

Attachments:

Plate 1 - Site Vicinity Map and Topography
Plate 2 - Site Plan and Boring Location Map
Plate 3 - Retaining Wall Detail
Plate 4 - Subgrade Preparation Detail
Plate 5 - Liquid and Plastic Limits
Plate 6 - Geology Map

Appendix I - Subsurface Investigation

Copies: Addressee (5, 1 by e-mail)
William Clark P.E., Clark Civil Engineering (1 by e-mail)
Jenny, Paul Discoe Design (1 by e-mail)



PUBLISHED REFERENCES

Blake Jr., M.C., AU - Jones, D.L., AU - Graymer, R.W., AU - digital database by Soule, Adam TI, "Geologic map and map database of parts of Marin, San Francisco, Alameda, Contra Costa, and Sonoma counties, California", 2000.

Brown, R.D., Jr., and Wolfe, E.W., 1972, Map showing active breaks along the San Andreas Fault between Point Delgada and Bolinas Bay, California: U.S. Geological Survey Miscellaneous Geologic Investigations Map I-692, scale 1:24,000.

California Department of Conservation, Division of Mines and Geology, "Geology of the Tamales Bay Study Area", Marin County, California (1:12,000)", Open-File Report 77-15 S.F., Plate 2.

California Department of Conservation, Division of Mines and Geology (2000), "Digital Images of Official Maps of Alquist-Priolo Earthquake Fault Zones of California, Central Coast Region", DMG CD 2000-004.

California Department of Conservation, California Geologic Survey, "Preliminary Geologic Map of the Napa and Bodega Bay 30' x 60' Quadrangles, California (1:100,000)" David L. Wagner and Carlos I. Gutierrez 2017

California Department of Conservation, Miles Division of Mines and Geology, Digital Database of Faults from the Fault Activity Map of California and Adjacent Areas, CD 2000-006, California Department of Conservation, Division of Mines and Geology, 2000.

CDMG, 1997, "Active Fault Near-Source Zones," Sheet E-17 for use with 1997 UBC. Clark, Joseph C., and Brabb, Earl E., 1997, Geology of Point Reyes National Seashore and Vicinity, California: A Digital Database: U.S. Geological Survey Open-File Report 97-456.

Davenport, C.W., An analysis of slope failures in Eastern Marin County, California, resulting from the January 3 and 4, 1982 storm, California Division of Mines and Geology Open-File Report 84-22, various scales.

Ellen, S.D., Cannon, S.H., and Reneau, S.L., 1988, Distribution of debris flows in Marin County, plate 6, in Ellen, S.D., and Wicczorek, G.F., eds., Landslides, floods, and marine effects of the storm of January 3-5, 1982, in the San Francisco Bay region, California: U.S. Geological Survey Professional Paper 1434, 310 p., plate 6 at 1:24,000

Idriss, I.M. & Boulanger, R.W. "SPT-Based Liquefaction Triggering Procedures" Department of Civil and Environmental Engineering, College of Engineering, University of California at Davis, UCD/GCM-10/02, December 2010.

Knudson, K. L., Sowers, J. M., Witter, R. C., Wentworth, C. M., and Helley, E. J., "Preliminary Maps of Quaternary Deposits and Liquefaction Susceptibility, Nine-County San Francisco Bay Region" California: A Digital Database, Open-File Report 00-44, Online Version 1.0, U.S. Geological Survey, 2000.

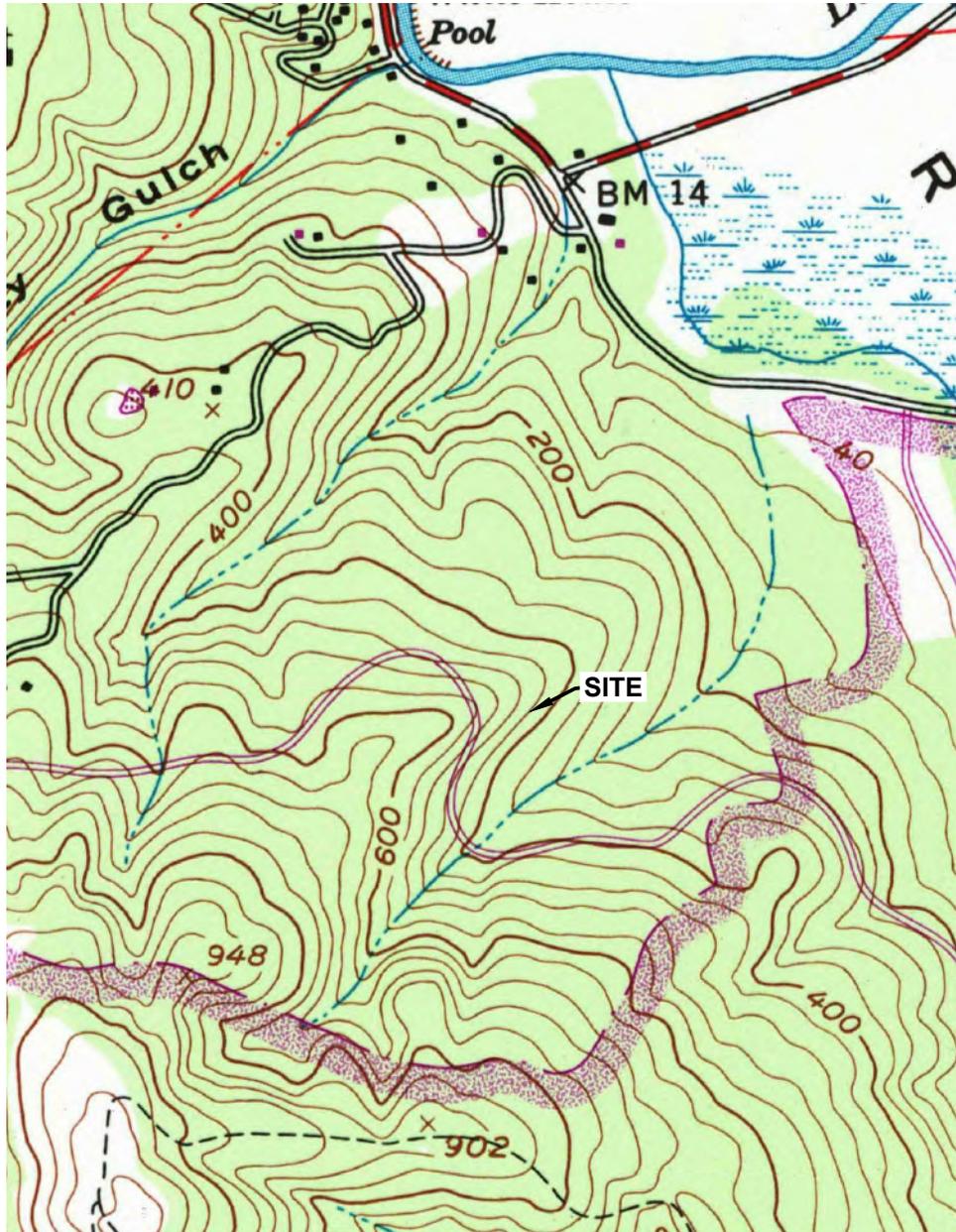
Tor Helge Nilsen, R.H. Wright, T.C. Vlastic, and W.E. Spangle "Relative slope stability and land-use planning in the San Francisco Bay region", California Professional Paper 944

United States Geological Survey (USGS), 2015, "UCERF3: A New Earthquake Forecast for California's Complex Fault System." USGS Fact Sheet 2015-3009, online version.

USGS, 1956, "Topographic Map of the Inverness 7.5' Quadrangle"

United States Geological Survey (USGS), 1997, Clark, Joseph C., Brabb, Earl E. "Geology of Point Reyes National Seashore and vicinity, California: A digital database" Open-File Report 97-456

Wagner, D.L., "Geology for planning in western Marin County", California: California Division of Mines and Geology Open-File Report 77-15, scale 1:12,000.

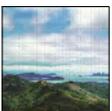


BASE: USGS 7 1/2' QUADRANGLE, INVERNESS 1954 REV. 1971.

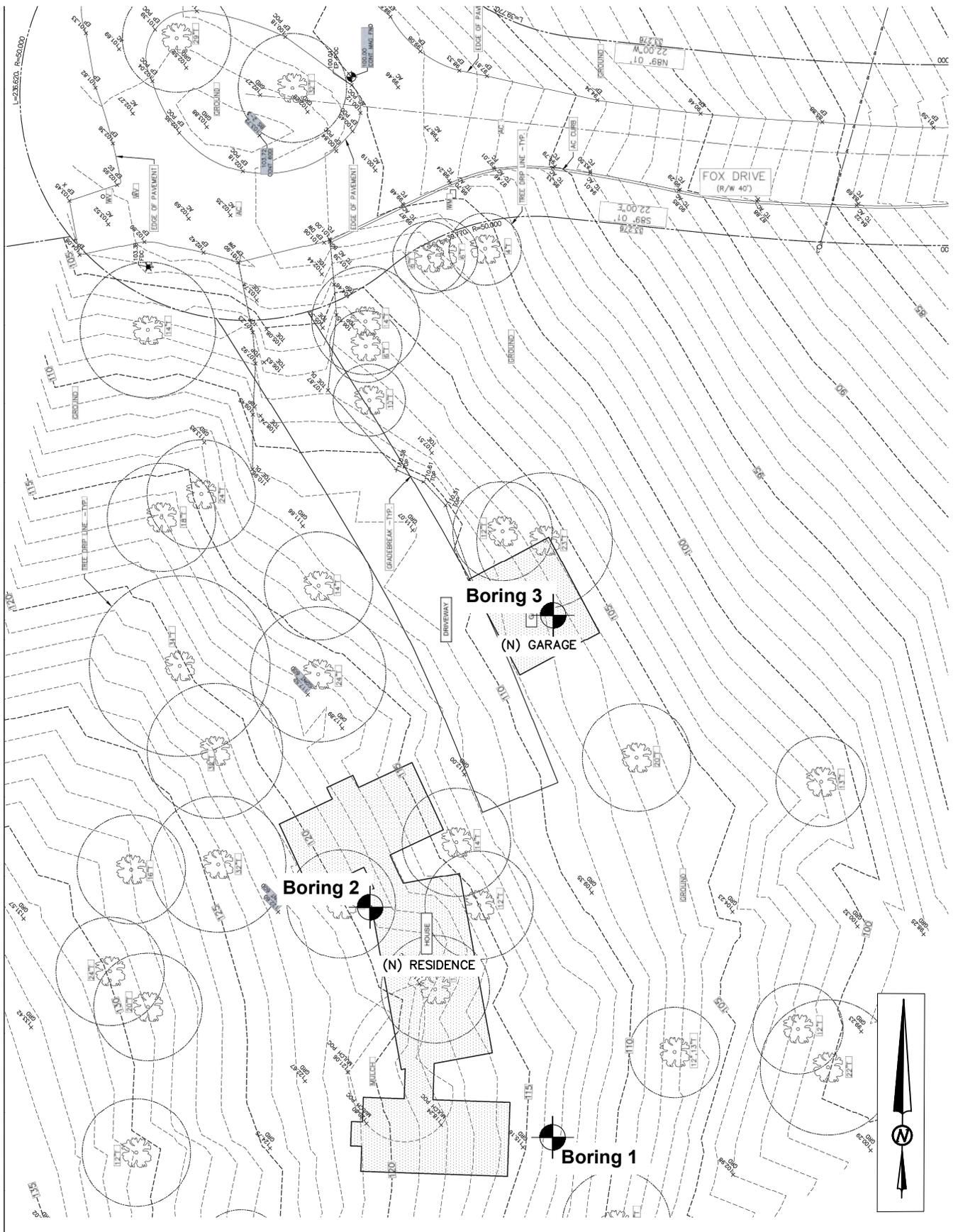
SITE VICINITY MAP AND TOPOGRAPHY

70 FOX DRIVE, INVERNESS, CA

PROJECT NO.	DATE	SCALE	PLATE NO.
107-1	MAY 2020	1"=1000'	1



AGNEW CIVIL ENGINEERING
CIVIL AND GEOTECHNICAL CONSULTANTS

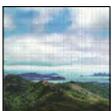


BASE: SITE PLAN BY PAUL DISCOE DESIGN

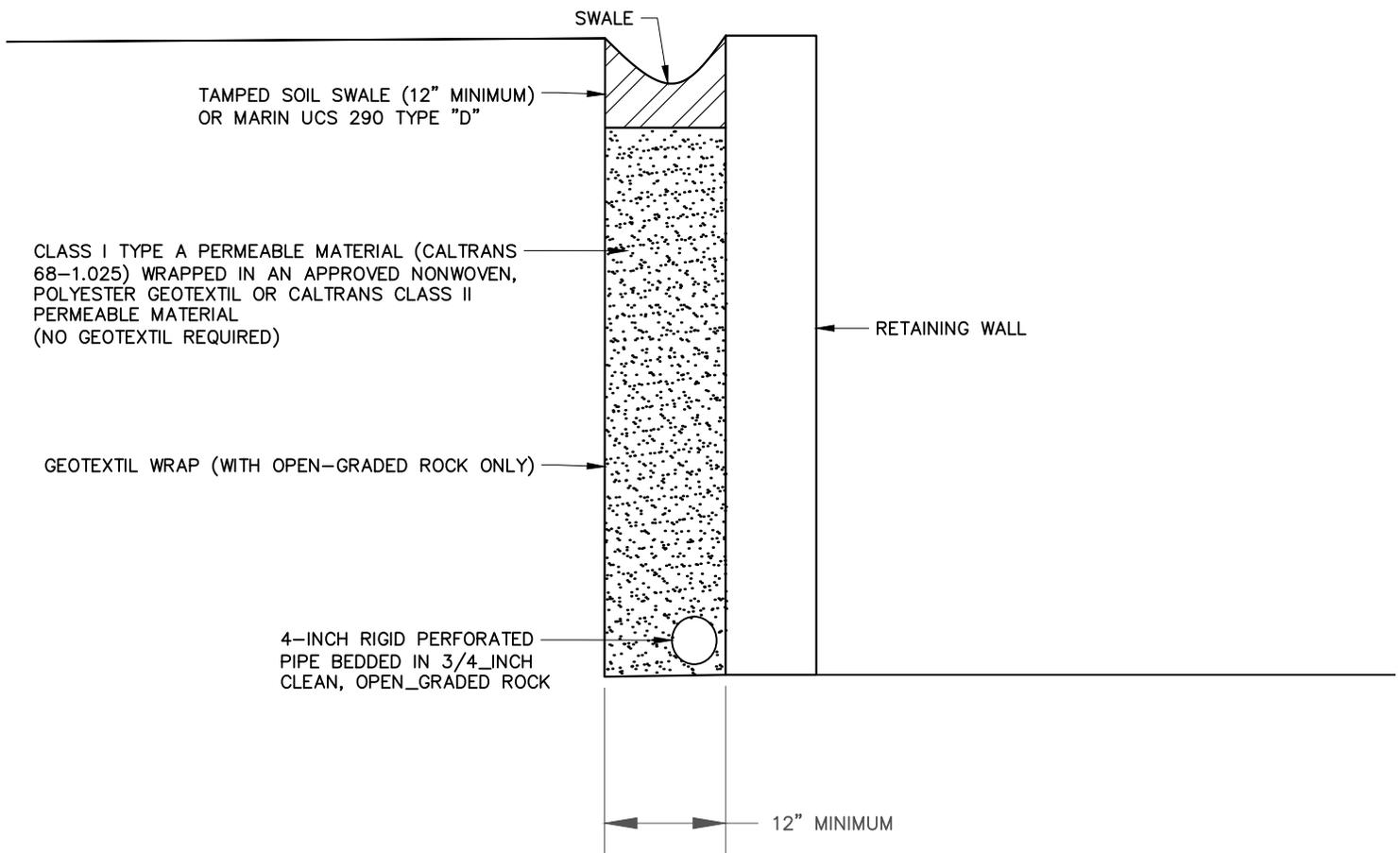
SITE PLAN AND BORING LOCATION MAP

70 FOX DRIVE, INVERNESS, CA

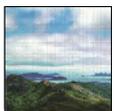
PROJECT NO.	DATE	SCALE	PLATE NO.
107-1	MAY 2020	1"=30'	2



AGNEW CIVIL ENGINEERING
CIVIL AND GEOTECHNICAL CONSULTANTS



RETAINING WALL DETAIL

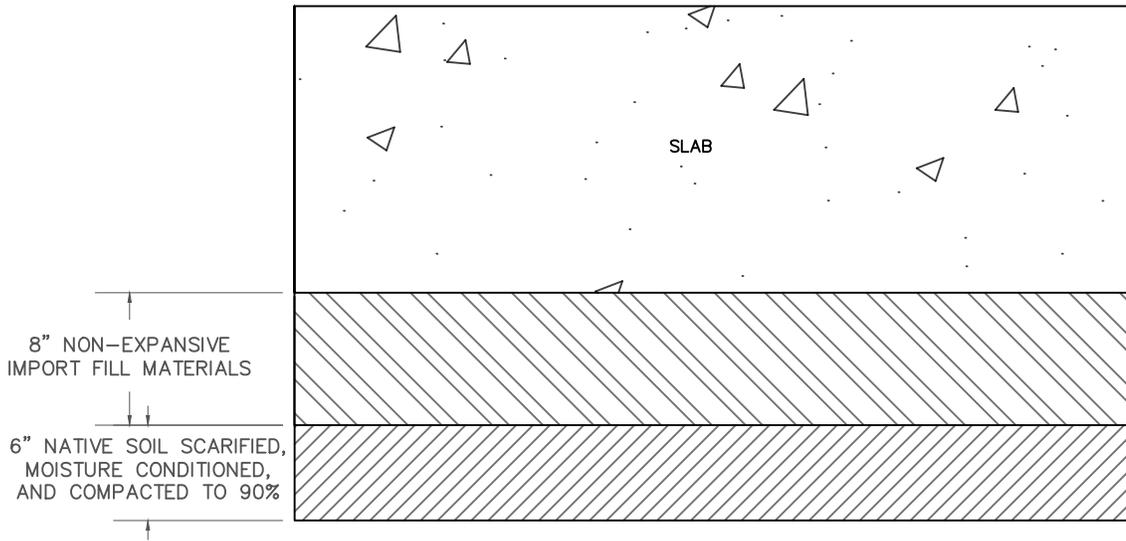


AGNEW CIVIL ENGINEERING
CIVIL AND GEOTECHNICAL CONSULTANTS

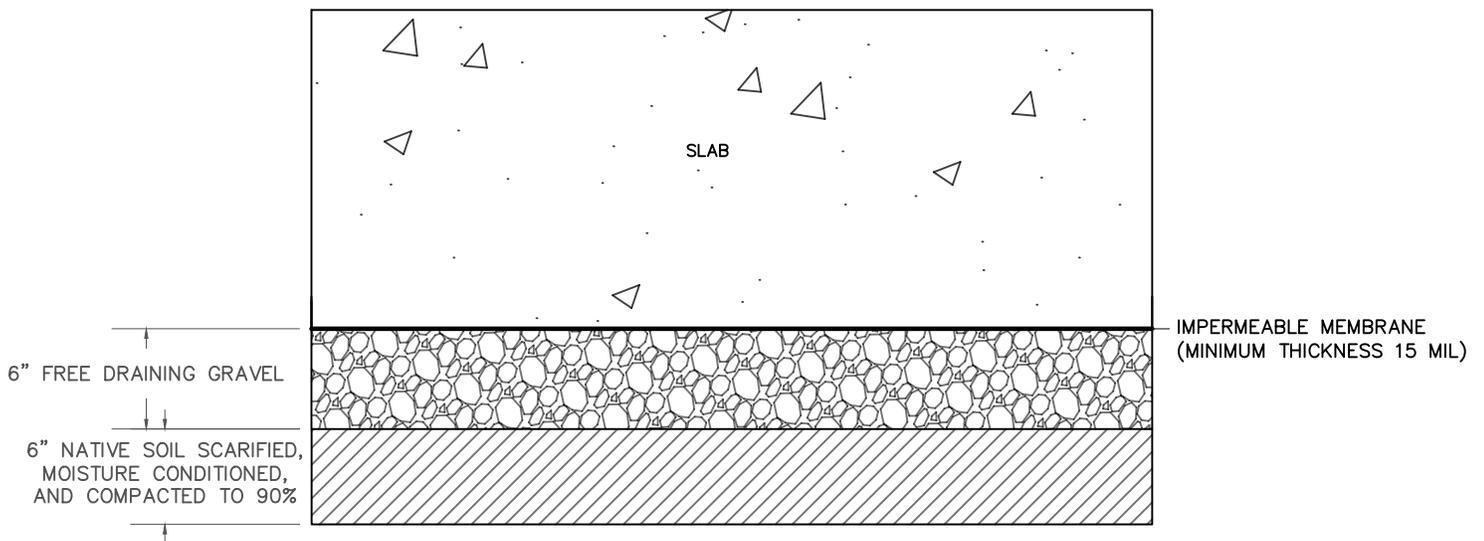
70 FOX DRIVE, INVERNESS, CA

PROJECT NO.	DATE	SCALE	PLATE NO.
107-1	MAY 2020	N/A	3

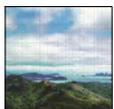
OPTION A - MINOR FLOOR WETNESS POSSIBLE



OPTION B - MINOR FLOOR WETNESS REDUCED



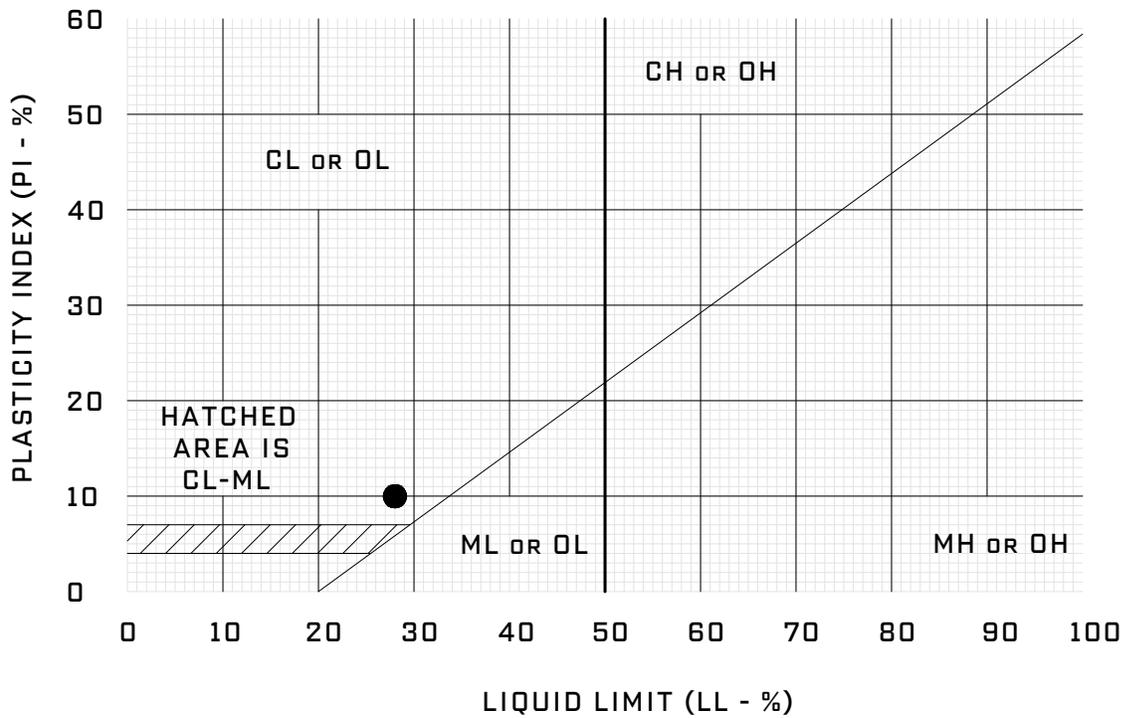
SUBGRADE PREPARATION DETAIL



AGNEW CIVIL ENGINEERING
CIVIL AND GEOTECHNICAL CONSULTANTS

70 FOX DRIVE, INVERNESS, CA

PROJECT NO.	DATE	SCALE	PLATE NO.
107-1	MAY 2020	N/A	4

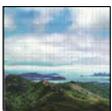


SAMPLE	SYMBOL	LL	PL	PI	-200	SOIL CLASSIFICATION
B-2: 2.0'-2.5'	●	28	18	10		CL

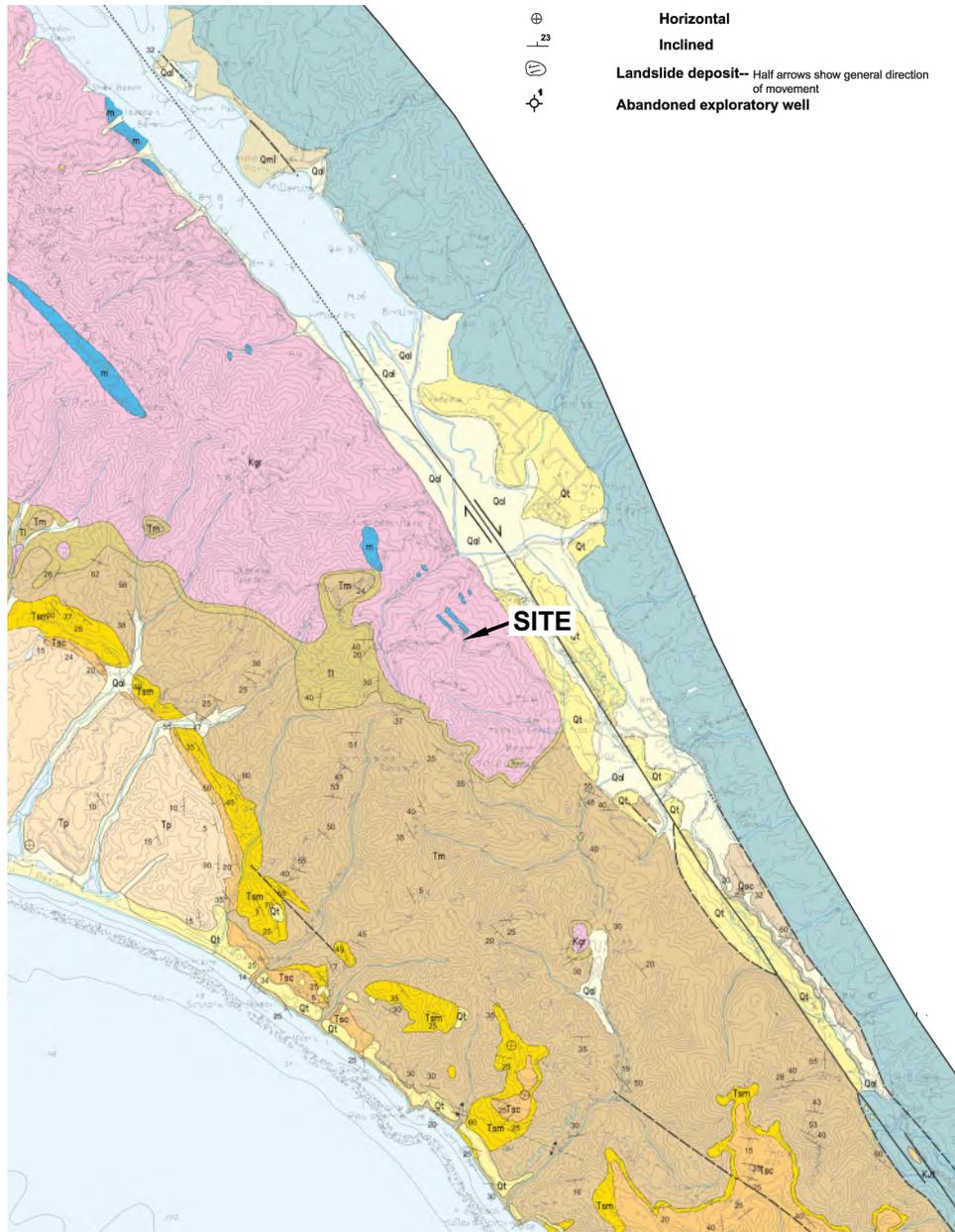
LIQUID AND PLASTIC LIMITS

70 FOX DRIVE, INVERNESS, CA

PROJECT NO.	DATE	SCALE	PLATE NO.
107-1	MAY 2020	NA	5



AGNEW CIVIL ENGINEERING
CIVIL AND GEOTECHNICAL CONSULTANTS

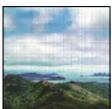


BASE: UNITED STATES GEOLOGICAL SURVEY (USGS), 1997, CLARK, JOSEPH C., BRABB, EARL E. "GEOLOGY OF POINT REYES NATIONAL SEASHORE AND VICINITY, CALIFORNIA: A DIGITAL DATABASE" OPEN-FILE REPORT 97-456

GEOLOGY MAP

70 FOX DRIVE, INVERNESS, CA

PROJECT NO.	DATE	SCALE	PLATE NO.
107-1	MAY 2020	1"=8000'	6



AGNEW CIVIL ENGINEERING
CIVIL AND GEOTECHNICAL CONSULTANTS

APPENDIX I - SUBSURFACE INVESTIGATION

The subsurface investigation was performed on April 16, 2020, using custom portable equipment for three exploratory boring(s) at the location(s) shown on the Site Plan. The boring(s) were advanced using a custom portable drill rig with continuous solid flight augers and conventional sampling equipment. The boring(s) were 3.5-inches in diameter and were extended to maximum depth(s) of 21.5 feet. The boring(s) were terminated in competent weathered bedrock.

The materials encountered in the boring(s) were continuously logged in the field by the civil engineer at the time of drilling. The soils are described in accordance with the Unified Soil Classification System (ASTM D-2487). The logs of the boring(s), as well as the key for classification of the soil, are included as a part of this Appendix.

Representative material samples were obtained from the exploratory boring(s) at selected depths. All samples were transmitted to the laboratory for evaluation and appropriate testing, as described in Section 8 of the report. The results of these tests are presented on the attached boring logs at the approximate sample depths. Relatively undisturbed samples were obtained using a series of 6-inch long, thin-walled brass liners inside a 3.0-inch O.D. Modified California sampler, and disturbed samples were obtained using the 2-inch O.D. Split Spoon samplers. The Modified California sampler is designated on the boring log(s) by a large "X" and the Split Spoon sampler is designated by one vertical line.

Resistance blow counts were obtained by dropping a 140-pound hammer through a 30-inch free fall. The sampler was driven 18 inches and the number of blows was recorded for each 6 inches of penetration. The blows per foot recorded on the boring logs represent the accumulated number of blows that were required to drive the last 12 inches and converted to standard Penetration Resistance ("N") values. When the 2.0-inch split spoon sampler is used in combination with a standard 140-pound hammer, these blow counts are the standard penetration resistance (SPT) or "N" values. However, when a larger diameter sampler is used, the blow counts recorded are not standard penetration resistance values. Accordingly, these values have been converted to equivalent SPT or N values on the attached log by multiplying by an appropriate factor.

The locations were obtained by rough field measurements using existing features at the site and plotted on the Grading and Drainage Plan produced by Clark Civil Engineering. The attached boring log(s) and related information show an interpretation of the subsurface conditions at the date and location indicated, and it is not warranted that they are representative of the conditions at other locations or times.

PRIMARY DIVISIONS			GROUP SYMBOL	SECONDARY DIVISIONS
COARSE GRAINED SOILS MORE THAN HALF OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVELS MORE THAN HALF OF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS (LESS THAN 5% FINES)	GW	WELL GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
		GRAVEL WITH FINES	GP	POORLY GRADED GRAVELS OR GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
			GM	SILTY GRAVELS, GRAVEL-SAND MIXTURES, NON-PLASTIC FINES
		GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES	
	SANDS MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE	CLEAN SANDS (LESS THAN 5% FINES)	SW	WELL GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
			SP	POORLY GRADED SANDS OR GRAVELLY SANDS, LITTLE OR NO FINES
		SANDS WITH FINES	SM	SILTY SANDS, SAND SILT MIXTURES, PLASTIC FINES
			SC	CLAYEY SANDS, SAND-SILT MIXTURE, NON-PLASTIC LINES
FINE GRAINED SOILS MORE THAN HALF OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS LIQUID LIMIT IS LESS THAN 50%	ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY	
		CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS	
		DL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY	
	SILTS AND CLAYS LIQUID LIMIT IS GREATER THAN 50%	MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS	
		CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS	
		OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS	
HIGHLY ORGANIC SOILS			PT	PEAT AND OTHER HIGHLY ORGANIC SOILS

DEFINITION OF TERMS

SILTS AND CLAYS	U.S. STANDARD SERIES SIEVE			CLEAR SQUARE SIEVE OPENINGS			COBBLES	BOULDERS
	200	40	10	4	3/4"	3"		
	SAND			GRAVEL				
	FINE	MEDIUM	COARSE	FINE	COARSE			

GRAIN SIZES

SANDS AND GRAVELS	BLOWS/FOOT+	SILTS AND CLAYS	STRENGTH*	BLOWS/FOOT+
VERY LOOSE	0 - 4	VERY SOFT	0 - 1/4	0 - 2
LOOSE	4 - 10	SOFT	1/4 - 1/2	2 - 4
MEDIUM DENSE	10 - 30	FIRM	1/2 - 1	4 - 8
DENSE	30 - 50	STIFF	1 - 2	8 - 16
VERY DENSE	OVER 50	VERY STIFF	2 - 4	16 - 32
		HARD	OVER 4	OVER 32

RELATIVE DENSITY

CONSISTENCY

* NUMBER OF BLOWS OF 140 POUND HAMMER FALLING 30 INCHES TO DRIVE A 2 INCH O.D. (1-3/8 INCH I.D.) SPLIT SPOON (ASTM D - 1586)
+ UNCONFINED COMPRESSIVE STRENGTH IN TONS/SQ. FT. AS DETERMINED BY LABORATORY TESTING OR APPROXIMATED BY THE STANDARD PENETRATION TEST (ASTM D - 1586), POCKET PENETROMETER, TORVANE, OR VISUAL OBSERVATION



AGNEW CIVIL ENGINEERING
CIVIL AND GEOTECHNICAL CONSULTANTS

KEY TO EXPLORATORY BORING LOGS
UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D-2487)

