November 16, 1999

Ms. Patty Mowrey
616 Colby Court
Walnut Creek, CA 94598

RE: Geotechnical Foundation Investigation
Proposed Residence on Rudgear Drive
Lot 6, Subdivision 7207 of Contra Costa County
Walnut Creek, California

Dear Ms. Mowrey:

Geolith Consultants has completed a geotechnical foundation investigation for the construction of your proposed residence on Lot 6 of Subdivision 7207, located along Rudgear Drive in Walnut Creek, California. This report presents the results of our research, field and laboratory data, geotechnical engineering analysis, and geotechnical review of the proposed construction.

ACCOMPANYING FIGURES AND APPENDICES

Figure 1 - Site Location Map
Figure 2 - Boring Location Map
Figure 3 – Geologic Cross Section A-A'
Appendix A – Subsurface Investigation (Boring Logs and Drilling Permit)
Appendix B – Laboratory Test Results (Moisture/Density and PI Data)

PURPOSE AND SCOPE OF WORK

The purpose of our geotechnical investigation was to evaluate the subsurface conditions at the site and provide appropriate geotechnical engineering recommendations for the proposed residential construction and foundation work. The scope of work performed for this investigation, as listed in our proposal dated August 17, 1999, included the following tasks:

X Review of selected published and unpublished geologic and soils maps and reports pertinent to the property.

X Review of historic stereopair vertical aerial photographs of the area and vicinity dating back to 1946.

X Compilation and analysis of the data collected from our in-house research.

X Subsurface exploration consisting of drilling three exploratory borings; 21, 24 and 25 feet deep. The boring logs are included in Appendix A.

X Laboratory testing of the soil and bedrock samples obtained from the exploratory borings. Test results are included in Appendix B.

X Evaluation and geotechnical engineering analysis of the collected field and laboratory data.

X Preparation of this summary report of our findings which includes geotechnical recommendations for the proposed construction; and the preparation of a site plan showing the locations of all subsurface exploratory borings.

Due to changes in the standards of practice and applicable building codes, we should note that our current recommendations supersede those presented in the original geotechnical report prepared by The Buller Group, dated January 9, 1989. Although not anticipated at this site, we should note that our investigation did not include the evaluation or assessment of any potential environmental hazards or groundwater contamination that may be present.

SITE DESCRIPTION

The site is located in Walnut Creek, Contra Costa County, California as shown on Figure 1. Access to the property is from Rudgear Road and Rudgear Drive. The property is part of a 7 lot subdivision originally developed in the late 1980s (Buller Group, Inc., 1989). To date, all of the lots are vacant of homes or structures. The subdivision improvements include retaining walls along Rudgear Road at the north end of the subdivision, and the construction of a central paved street with retaining walls. The site vegetation consists of seasonal grasses, scattered oak trees, and some brush.

Geomorphically, the proposed building site is on a northeasterly-facing slope along the flanks of Sugarloaf Hill. Topographically, the subject building site is situated below Rudgear Drive. The descending slope gradient varies from approximately 2:1 (50% slope) near Rudgear Drive to 3:1 (33% slope) near the downslope property boundary. The overall slope gradient is commonly 2.5:1 (40% slope). Spectacular views of Shell Ridge and Mount Diablo are visible toward the northeast.

PROPOSED CONSTRUCTION

We understand that construction of a two-story residence with attached two-car garage is proposed along the upper portion of the slope that descends from Rudgear Drive. It is desired that the upper story of the proposed home be positioned near the same approximate grade as Rudgear Drive to accommodate the driveway and garage at this elevation. A pier and grade beam foundation with an upper retaining wall is anticipated for the proposed construction. The upper retaining wall would be backfilled with soil to reach the desired driveway/front lawn grade.

SOILS, BEDROCK, AND BEDROCK STRUCTURE

Artificial Fill

An wedge of fill appears to have been placed near the upper portion of the subject lot, to support Rudgear Drive. This fill wedge is clearly shown along a recently graded roadway in the 1946 aerial photographs and suggests that the fill was placed by cut and cast grading methods common to that era. Along our geologic section line A-A' (see Figure 3), the toe of this fill wedge is located at a horizontal distance of approximately 24 feet from the Rudgear Drive right-of-way, at an elevation of approximately 330 feet. The maximum explored depth of artificial fill along cross section line A-A' is approximately 10 feet. The fill comprises a layered mixture of mottled light and dark brown, sandy silt and clayey silt with small bedrock fragments. The fill is moderately compact and has a moisture content that varies from dry to moist.

Native Soils

Native soils are present below the down hill portion of the artificial fill wedge and occur as a relatively thin veneer above the bedrock along the lower portions of the site. This thin mantle of soil is approximately 3 feet thick within the building envelope of Lot 6 (see Figure 3). The native soil recovered in exploratory borings No. 2 and No. 3 consisted of very dark grayish-brown to grayish-brown sandy to clayey-silt, with some small, scattered pebbles (see Boring Logs). The upper 1.5 feet of native soil is loose and becomes more firm below this depth. The soil is dry near the surface and is slightly moist below. Because of the steep slope at this location the native soil is vulnerable to down slope movement via soil creep.

Bedrock

According to published geologic maps by Crane (1988), Dibblee (1980), and Wagner (1978), the bedrock underlying the site is undifferentiated Pliocene and Upper Miocene age non-marine sedimentary beds that may include: pebble conglomerate, sandstone, claystone and siltstone. Bedrock was encountered within our exploratory borings at -8.75 feet in boring No.1, at -5.0 feet in boring No. 2 and at -2.75 feet in boring No. 3 (as measured below the existing ground surface). The decreasing depth-to-bedrock preceding downslope is likely due to the road fill wedge along the top of the slope, adjacent Rudgear Drive. The bedrock encountered within the exploratory borings was composed predominantly of interbedded silty-sandstone, sandstone and matrix supported pebble conglomerate. The lithologic correlation of the samples from the exploratory borings, shown in our geologic cross section A-A' (see Figure 3), appears to indicate that bedding is oriented roughly parallel to the existing natural slope. Furthermore, the variable thickness of individual beds may indicate that they are lensoidal in nature.

Bedrock Structure

The site and vicinity are located along the southwest limb of a northwest-striking syncline known as the Stone Valley Syncline (Crane, 1988). The axis of the syncline is mapped further to the northeast of the subject site. The bedrock underlying the site is thought to be dipping toward the northeast, or in the direction of the descending slope, at 25 to 45 degrees. A single bedding plane attitude taken along the road cut for Rudgear Drive, directly across from the site strikes approximately N40^{o} W and dips approximately 37 degrees to the northeast.

LANDSLIDES

According to published landslide maps by Nilsen (1975), there is a landslide feature mapped near the southeast corner of the property that extends upslope to near the ridge top. This mapping is based on the review of aerial photographs without the benefit of site specific data, such as subsurface borings. Our subsurface exploration, site observations, and review of aerial photographs dating back to 1946 does not support the presence of such a landslide within the site. However, the aerial photographs reviewed for this report suggest that there is likely a shallow slope failure located upslope to the southeast of the property, and that the toe of this landslide was removed during the original construction of Rudgear Drive (prior to 1946). Therefore, we conclude that the site is clear of landsliding that would adversely affect the proposed construction.

REGIONAL FAULTING AND SEISMICITY

According to the published geologic maps reviewed for this research, no faults have been mapped across the property. The active faults located closest to the site include the northwest trending Concord, Calaveras, and Hayward faults. The Calaveras fault is mapped closest to the site, approximately 1.4 miles to the south (ICBO, 1998). The northern Calaveras is capable of a maximum magnitude event of 7.0, and has an 18% probability of recurrence in the next 30 years. The Concord fault is located approximately 3.0 miles to the northeast. The Concord-Green Valley fault is considered capable of generating a maximum magnitude of 6.8, with 6% probability of occurrence in the next 30 years. Both the Calaveras and Concord are B-type faults, which means that special design provisions are mandated for structures located within 6 miles (10 km) of such features. The Hayward fault is located approximately 11.0 miles southwest (California State Geologist, 1993; and Jennings, 1994). The Northern Hayward fault is capable of generating a 6.6 magnitude earthquake (or 7.1 for both the north and south segments, slipping simultaneously). The Hayward fault has a 32% chance of occurrence over the next 30 years. It is an A-type fault, requiring that structures located within 9 miles be designed for specialized seismic accelerations.

Although the site itself is not located within a recognized Earthquake Fault Zone (CDMG, 1993, revised), the property can still be impacted by ground shaking from any one of the previously mentioned faults. Such zones are designated by the State of California for active faults as defined by the Alquist-Priolo Earthquake Fault Zoning Act of 1972 (Chapter 7.5, Division 2 of the California Public Resources Code).

Due to the proximity of these major faults, strong seismic ground shaking should be expected at the subject site during a moderate or major earthquake event, particularly on one of the nearby faults. The risk of ground lurching will be dependent on the epicentral distance from the source event and the stiffness of the earthen materials underlying the site.

The default seismic load factor decreed by the 1997 Uniform Building Code (UBC) requires consideration of near-field loading criteria, when sites are within 10 km of a known active fault(s), as described in: *Maps of Known Active Faults and Near-Source Zones in California and Adjacent Portions of Nevada*: Calif Div Mines & Geology, Structural Eng'rs Assn of California Seismology Committee, and the Int'l Conference of Building Officials (1998).

The controlling feature would appear to be the northern Calaveras fault, now considered a semi-continuous Type B fault. This right-lateral strike-slip fault system has been assigned a slip rate of at least 6 mm/year (+/- 3 mm/yr), capable of a maximum magnitude (M_L) 6.8 quake (M.D. Petersen, et al, 1996, *Probabilistic Seismic Hazard Assessment for the State of California*: Calif Div Mines & geology OFR 96-08 and US Geol Survey OFR 96-706). Chapter 16 in Volume 2 of the 1997 UBC spells out the various earthquake loading parameters that may be used absent a site specific dynamic response analysis, such as SHAKE 91.

The Seismic Zone Factor, **Z, is given as 0.40** (for Zone 4 coastal California). We would estimate that the subject site is underlain by alluvium, approximated by the UBC-designation **Soil Type S_c**, transmitting **shear wave velocities between 1,200 and 2,500 feet per second** could be expected. This assumes that any structure contemplated for this site will be founded on engineered fill or weathered bedrock. Using the parameters for dense soil and weathered bedrock, a series of default seismic load coefficients were determined from Vol. 2 of the 1997 UBC. These include: a **Seismic Coefficient C_a = 0.40N_a; Seismic coefficient C_v = 0.56N_v ; Near-Source Factor, N_a = 1.0; and Near-Source Factor, N_v = 1.2.** These values apply to Type B faults situated 2.3 km from sites being evaluated on soil type S_c in seismic zone (Z) 4.

SUBSURFACE INVESTIGATION

The subsurface conditions at the site were investigated by drilling three exploratory borings using a B-24 Mobil truck-mounted drilling rig. All three borings were drilled with a 4-inch solidstem auger. Drilling locations are shown on Figure 2. Boring No. 1 was drilled at the top of the slope, adjacent to Rudgear Drive to a depth of 23.0 feet and sampled to 24.0 feet. Boring No.2 was drilled approximately 35 feet northeast of boring No. 1, to a depth of 24.0 feet and sampled to 25.0 feet. Boring No. 3 was drilled approximately 90 feet northeast of boring No. 1 to a depth of 20 feet and sampled to 20.75 feet.

Samples of the subsurface materials at each drilling site were recovered at selected intervals using conventional drive samplers. The samples were sealed and transmitted to our soil testing laboratory for appropriate testing, as discussed in the following section and in Appendix B. Further details of the subsurface exploration program are discussed in Appendix A. The conditions with respect to the proposed development are summarized in the paragraphs that follow:

- Exploratory boring B-1, encountered approximately 8.75 feet of artificial fill. The artificial fill consisted of a mottled light and dark brown mixture of sandy silt and clayey silt with small fragments of bedrock. The fill was moderately compact with a moisture content that varied from dry to moist. The artificial fill was resting on bedrock that consisted of interbedded silty-sandstone, sandstone and conglomerate. At 8.75 feet the bedrock encountered comprised a mottled orange, matrix supported, poorly cemented conglomerate. At approximately 12 feet below the ground surface a moderate yellowish-brown silty fine sandstone to dark yellowish-orange fine to medium sandstone was encountered. At approximately 19 feet the bedrock comprised a pale yellowish-brown matrix-supported, poorly to moderately cemented conglomerate. (Boring log No. 1, Appendix A).

- Exploratory boring B-2, encountered approximately 3.25 feet of artificial fill identical to the fill encountered in B-1. The artificial fill was resting on approximately 1.75 feet of native soil. The native soil comprised very dark grayish-brown to grayish-brown slightly moist and firm sandy to clayey-silt with some small, scattered pebbles. The native soil was resting on moderate yellowish-brown, poorly cemented, silty fine sandstone. At approximately 14.75 feet below the ground surface, the bedrock changed to a grayish-orange, poorly cemented medium to coarse sandstone with some scattered pebbles. At approximately 20 feet the bedrock consisted of a moderate yellowish-brown, poorly cemented silty fine sandstone. At approximately 24.75 feet the bedrock changed to a mottled gray-brown, poorly sorted, matrix supported clay-rich conglomerate with angular to rounded clasts up to ½ inch in diameter (Boring log No.2, Appendix A).

- Exploratory boring B-3, encountered approximately 2.75 feet of native soil that comprised very dark grayish-brown to grayish brown sandy to clayey-silt. The upper 1.5 feet of native soil is loose and dry, but becomes firm and slightly moist with depth. The native soil is resting on poorly cemented, moderate yellowish-brown silty fine sandstone encountered approximately 2.75 feet below the surface. At 6.25 feet the bedrock consists of a pale yellowish-brown poorly sorted, poorly cemented, medium to coarse sandstone with some scattered pebbles. At approximately 15 feet, moderate yellowish-brown, poorly cemented, silty fine sandstone was encountered. At approximately 20 feet the bedrock changed to a mottled gray-brown, matrix supported, poorly sorted clay-rich conglomerate with angular to rounded clasts up to ½ inch in diameter (Boring log No.3, Appendix A).

Ground Water

Ground water was not encountered in any of the subsurface exploratory borings. It should be noted that ground water levels may change and can vary with seasonal rainfall patterns, long-term climate fluctuations and with the influence of local site conditions such as drainage patterns and the presence of groundwater barriers such as landslide shear surfaces or bedrock fault contacts.

LABORATORY TESTING

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

1. In-situ Moisture Content: The moisture content tests were used to evaluate variations in retained soil moisture. Moisture contents from native soil samples retrieved from the exploratory borings commonly ranged from 10 to 20 percent. Below the existing ground surface and beyond the surface drying profile, the native on-site soils typically retain soils moistures ranging from 18 to 20 percent. The bedrock soil moistures are highly variable depending on the type of rock sampled and range from 12 to 28 percent. Only one sample of the road fill was actually tested for moisture content (13.5 percent), but the fill ranged from dry to moist during the field exploration.

2. In-situ Dry Density: The dry density tests evaluate variations in the material's density and saturation, and are also useful for estimating relative soil strengths and for comparing densities of the in-place materials. The test results from native soil samples retrieved from our exploratory borings indicate that dry densities range from 100 pcf to 105 pcf, with the uppermost creep-prone soils being approximately 86 pcf. The dry densities within the bedrock vary from 83 pcf to 112 pcf depending on the type of bedrock sampled. A single sample of the road fill from Boring B-1 was 104 pcf.

3. Atterberg Limits: The Atterberg limits test was performed in order to help classify and evaluate the expansion potential of residual clayey y topsoils. One such sample was tested for expansiveness from B-3 at 1.0 to 1.5 feet. The results indicate a PI = 23, which is of medium expansion potential.

Further details regarding the laboratory-testing program are provided in Appendix B. The results of the laboratory tests are presented in Appendix A on the boring logs at the appropriate sample depths.

CONCLUSIONS

Based on the information gathered during the course of this subsurface investigation, it is our opinion that from a geotechnical standpoint, the property is suitable for the proposed residential development, provided our recommendations are incorporated into the final design of the project. The potential geotechnical constraints identified during this study include relatively steep descending slopes (up to 35% closest to Rudgear Drive) in combination with creep-prone, medium expansive surface soils, up to 3 feet in thickness across the site. Other construction constraints include structural setbacks from adjacent property lines, structural setbacks from the Rudgear Drive fee title right-of-way, and height limits for residential construction within hillside areas of Walnut Creek.

RECOMMENDATIONS

Fill Placement Recommendations

A. Site Preparation Prior to Engineered Fill Placement

The area of the proposed work should be clearly staked in the field. The work area should be cleared of all obstructions, including underground utility or irrigation lines, existing structures, minor brush, and wood debris. Construction debris from the removal of existing structures, trees to be removed from the site, and other demolition debris are unsuitable for use as fill materials and should be hauled off site. Geolith Consultants personnel should observe and approve all excavations prior to the placement of compacted fill (1997 UBC App Sec 3317.3 and 3317.4).

B. Placement and Compaction of Engineered Fill

The on-site soils and bedrock types are suitable for use as engineered fill provided that these soils contain less than 3 percent organic materials. Roots and other organic debris may have to be removed from the topsoil before this criteria is met. Fill material should be placed and compacted to a dry density not less than 90 percent of the maximum dry density at a field moisture content about 2 percent above the optimum moisture content, as determined by ASTM D 1557-92 (UBC App Sec 3305.1.1). Geolith Consultants personnel should be present to perform compaction testing during fill placement.

Each excavation can be filled with engineered fill consisting of appropriate on-site soils or similar import fill materials (UBC App Sec 3313.3). Any import fill (from off-site sources) should have a Plasticity Index (PI) of less than 15. All fill placed at the site, including on-site soils, should not contain rocks or lumps larger than about 6 inches in greatest dimension with not more than 15 percent larger than about 2 inches. Any import materials for use as fill should be approved by Geolith Consultants prior to use.

Fill materials should be spread in layers or lifts not exceeding 8 inches in uncompacted thickness. The surface of all fill slopes should be either back-rolled or compacted beyond the limits of the slope and cut back in order to achieve satisfactory compaction at the finished slope face. All fill slopes should have a maximum inclination of 2:1 (horizontal to vertical).

C. Foundation Recommendations

C.1. Residential Pier-and-Grade Beam Foundations

We recommend that the proposed home be constructed on a pier-and-grade beam foundation system with piers that extend through the soil/compacted fill and are embedded into the underlying bedrock. The floor system for the proposed home should derive all structural support from the interconnected pier-and-grade beam foundation. Interior slab-on-grade floors for living space areas are not recommended, unless you authorize our making an engineering evaluation of such systems, using the tenets of the 1997 Uniform Building Code for the design of post-tensioned slabs on expansive soils (1997 UBC Vol. 2 Sec. 1802.4 and 1804.2).

We recommend cast-in-place, straight-shaft concrete piers that are at least 24 inches in diameter, and a minimum depth of 25 feet. To determine whether these depths are adequate to carry the required structural loads, in native alluvium allowable skin friction values of 400 pounds per square foot for dead plus live loads and 600 pounds per square foot for all loads, including wind and seismic, beginning at a depth of -3 feet. Allowable skin friction values in fill areas can be taken as 500 psf for dead plus live loads and 750 psf for all loads, including wind and seismic, for well-compacted soils, beginning at a depth of -3 feet (to account for seasonal desiccation).

Lateral loads on the piers may be resisted by passive pressures acting against the sides of the piers. We recommend a passive pressure equal to an equivalent fluid weighing 800 pounds per cubic foot (pcf) per foot of depth to a maximum value of 4000 pcf. This value can be assumed to be acting against 1-½ times the diameter of the individual pier shafts, beginning 3 feet below the bottom of the grade beams, or 1 foot below the bottom of retaining wall footings.

The bottom of the pier excavations should be reasonably free of loose cuttings and soil/mud debris prior to installing reinforcing steel and pouring concrete. If the pier excavations encounter water or if water has accumulated in the piers, the concrete may be tremied to the bottom of the excavations. All pier holes should be securely covered after drilling to minimize subsequent cleaning and for safety reasons.

Maximum horizontal distance between the grade 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. In order to minimize the possible detrimental effect of the expansive soils, we recommend that all grade beams be designed to resist an uplift pressure of 1200 pounds per square foot, if placed in the upper 3 feet of the existing slope. For footings founded on top of or within the underlying bedrock, this value may be reduced to 300 psf uplift.

C.2. Garage and Exterior Slabs-On-Grade

We recommend that any slab-on-grade floors for the garages or other exterior hardscape areas, including concrete driveways, be supported directly on a layer of compacted non-expansive, import fill at least 8 inches thick. Immediately prior to construction of the slab(s), the subgrade surface (beneath the import fill) should be scarified to a depth of 6 inches, moisture conditioned to about 2 to 4 percent over optimum water content, and compacted to at least 90% relative compaction (UBC App Sec 3313.4).

The slab(s) should be entirely supported by the foundations system, or structurally independent from the perimeter foundations and “free-floating”. Score joints should be provided for “free-floating’ slabs at a maximum spacing of 10 feet in both directions. The slabs should be appropriately reinforced according to structural requirements using reinforcing bars only (instead of wire mesh); concentrated loads may require additional reinforcement.

In any slab area where minor floor wetness would be undesirable, 4 inches of free draining gravel should be placed beneath the floor slab to serve as a capillary barrier between the subgrade material and the slab (sufficiently sloped to drain). An impermeable membrane should be placed over the gravel, and the membrane should be covered with at least 2 inches of sand to protect it from puncture during construction.

Considerable heat of hydration may be developed within thickened concrete slabs during curing. This could lead to expulsion of moisture vapor of hydration, which could become trapped beneath impervious flooring elements, if they are placed before all the excess moisture has escaped from the slab. If the building is sealed up too quickly, more slab moisture tends to be trapped. We would recommend that the slabs be moisture tested prior to placement of impervious flooring materials, such as vinyl. Most floor contractors perform the ASTM-D 4263-83 *Standard Test Method for Indicating Moisture in Concrete by the Plastic Sheet Method*, sealing an 18-inch square 4 mil thickness clear polyethylene sheet over a portion of the floor for 16 hours and looking for evidence of moisture condensation. One test per 500 square feet of floor area is recommended.

Geolith Consultants, Inc. should be consulted on foundation designs for all proposed structures at the site. This includes retaining walls, swimming pools, and other amenities.

D. Retaining Walls

All retaining wall must be designed to resist lateral earth pressures, slope creep forces and additional lateral pressures that may be caused by the inclination of the slope, above the wall, and below the wall.

D.1. Active and At-Rest Pressures

We recommend unrestrained (cantilever) walls, 4 feet in height or less, with a slope surcharge of 2:1, be designed to resist an equivalent fluid pressure of 85 pounds per cubic foot, beginning at the ground surface, and increasing hydrostatically, with depth, to the pier bottom. The active (upslope) pressures should be calculated over the full-center-to-center distance, between piers. Any restrained walls or cantilever walls greater than 4 feet high with a slope surcharge of 2:1 should be designed assuming an equivalent fluid pressure of 90 pcf.

Unrestrained (cantilever) walls with level backfill may be designed for to resist an equivalent fluid pressure of 65 pcf, for heights up to 4 feet, and 75 pcf for heights greater than 4 feet. This assumed native soils are used in the engineered fill behind the walls, and that the walls are not resisting landslides (dormant or active).

D.2. Footings

Walls built on sloping ground are typically supported on cast-in-ground piers, integrally supporting (via moment connection) conventional spread footings. The minimum pier diameter we would recommend is 18 inches (1.50 feet) in diameter. This is because of the disparity between passive resistance acting over two pier diameters, as opposed to active forces, which act across the entire upslope face of the wall.

D.3. Bearing Capacity

Bearing capacity of soil exposed beneath footings will vary according to the soil type. For footings founded in the upper three feet, within residual clay soils, the allowable bearing capacity should be taken to be 1500 psf, increasing to 2,000 psf including wind and seismic loads. For footings founded deeper than –3 feet, within or upon the weathered bedrock, the allowable bearing capacity may be increased to 2500 psf, increasing to 3,325 psf for wind and seismic loads.

These allowable bearing pressures are net values; therefore, the weight of the footings can be neglected for design purposes. However, all footings should have a minimum embedment of 18-inches or as designated by U.B.C., which ever is greater.

Lateral loads may be resisted by: (1) friction between the foundation bottoms and the supporting materials and (2) passive pressures acting against the sides of the footings. We recommend a coefficient of friction of 0.38 and a passive pressure equal to an equivalent fluid weighing 1385 pcf, starting at the top of the footing for retaining walls when the finish grade surface is level for at least 3.2 times the footing depth away from the top of the footing, and only 200 pcf when adjacent to a 2:1 downslope.

Horizontal creep forces acting to a depth of 4 feet have been incorporated into the design load of 85 pcf.

D.4. Passive Resistance

On 2:1 downslopes the passive resistance acting against the embedded pier on the downslope side (pushing upslope) should be taken to be 200 pcf equivalent fluid pressure, taken over two pier diameters. On 3:1 downslopes, this value may be increased to 300 pcf taken over two pier diameters.
Bending moments incurred by active and passive forces should be resolved in order to determine the adequate depth of embedment, realizing that passive resistance can only be generated over a distance of two pier diameters, while active forces act over the entire post-to-post distance.

D.5 Retaining Wall Drainage

The above pressures assume that sufficient drainage will be provided behind the walls to prevent the build-up of hydrostatic pressures from surface and subsurface water infiltration. Adequate drainage may be provided by utilizing crushed rock as backfill behind the wall along with a 4-inch rigid perforated backdrain pipe placed near the base of the backfill. The backdrain pipe should be connected to a system of closed pipe(s) (non-perforated) that lead to the storm runoff discharge facilities. In addition, the "high" end and all 90 degree bends of the drain pipe should be connected to a riser pipe which extends to the surface and acts as a cleanout. The number of required cleanouts can be reduced by using "sweep"-90's or double 45-degree elbows at the corners.

All fill must be compacted to at least 90 percent relative compaction (ASTM D-1557-91), utilizing equipment that will not damage the wall. We do not anticipate there will be much difficulty in achieving 90% compaction using crushed rock backfill.

E. Perimeter Subdrain Recommendations

Because of the nature of medium expansive soils on this site and the proposed front lawn area, we recommend the employment of trench subdrains around the perimeter of the structures to retard seasonal soil moisture fluctuations beneath the supporting foundations. These perimeter subdrains are usually 3 to 5 feet deep adjacent to the structures, and are sloped to drain by gravity.

The perimeter subdrains should consist of perforated four-inch diameter collector pipes. Cleanout riser pipes should be provided at all changes in direction of the subdrainage system. Only solid wall PVC pipe with slots or perforations should be used and surrounded with ¾ inch, clean, open-graded drain rock. ABS pipe should be used in areas exposed to the sun.

FUTURE CONSTRUCTION OBSERVATION, TESTING, AND CONSULTATION

During any proposed grading of the project, and/or construction of the residence, or any other amenities at the project site, Geolith Consultants, Inc. personnel should be present for full and part-time observations and testing of the following aspects of the work, in accordance with UBC App Sec 3313. 3315.3 and 3317. Our costs for the following tasks are not included within the current budget, however, we can provide you with a cost estimate upon request.

- Observe and approve all excavations prior to fill placement.
- Observe the placement of all compacted fill and trench backfill.
- Perform compaction testing of all compacted fill.
- Observe any additional grading and any final surface drainage conditions.
- Observe and approve all foundation excavations, especially pier holes, prior to pouring concrete.
- Observe fine grading work to verify that sufficient surface drainage measures/improvements are completed as recommended.
- Observe placement of wall backdrains, perimeter trench subdrains, perforated collector pipes, and outfall lines.

CLOSURE AND LIMITATIONS

Although not anticipated for this site, we should note that our investigation did not include any evaluation or assessment of potential environmental hazards such as hazardous material or groundwater contamination.

We have employed accepted geotechnical engineering services and engineering geologic procedures, and our professional opinions and conclusions are made in accordance with generally accepted geotechnical engineering principles and practices. This warranty is in lieu of all other warranties, either expressed or implied.

The contents of this report are valid as of the date of preparation. However, changes in the condition of the site can occur over time as a result of either natural processes or human activity. In addition, advancements in the practice of geotechnical engineering and engineering geology may affect the validity of this report. Consequently, this report should not be relied upon after an elapsed period of three years without a review by Geolith Consultants for verification of validity.

We hope that this report provides you with the information you requested. If you have any questions, please do not hesitate to call us at (925) 682-7601.

Sincerely,

GEOLITH CONSULTANTS, INC.

Patrick L. Drumm, RG, CEG, CHG
Senior Engineering Geologist

Fred H. P. Chin, PhD, PE
Principal Engineer

J. David Rogers, Ph.D., R.G., C.E.G., C.HG.
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REFERENCES

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FIGURES

AERIAL PHOTOGRAPHS

TYPE	DATE	PHOTO	SCALE
Stereo Pair Black & White	7-26-46	GS-CP 2-3 GS-CP 2-4	1: 26,400
Stereo Pair Black & White	5-4-57	AV-253 19-20 AV-253 19-21	1: 12,000
Single Photo Black & White	7-17-63	1330-31-3662	1: 24,000
Single Photo Black & White	5-19-69	AV-904-01-03	1: 12,000
Single Photo Black & White	4-12-71	AV-994-07-04	1: 12,000
Single Photo Black & White	8-21-73	AV-1101-07-04	1: 12,000
Single Photo Black & White	5-26-76	AV-1250-07-04	1: 12,000
Single Photo Black & White	5-5-78	AV-1498-07-01	1: 12,000
Single Photo Black & White	5-2-80	AV-1860-07-04	1: 12,000
Single Photo Black & White	4-27-82	AV-2131-07-06	1: 12,000
Single Photo Black & White	5-7-84	AV-2460-07-05	1: 12,000
Single Photo Black & White	4-20-86	AV-2862-07-06	1: 12,000

APPENDIX A

SUBSURFACE INVESTIGATION

APPENDIX A – SUBSURFACE INVESTIGATION

Our subsurface exploration program was completed in one day. On November 4, 1999, a truck-mounted B-24 drill rig with solid stem augers drilled three, 4-inch diameter exploratory borings to depths of 24.0 feet, 25.0 feet and 20.75 feet below the existing ground surface. The approximate locations of the exploratory borings is shown on Figure 2. After drilling, the borings were backfilled with cement grout as directed by a representative from Contra Costa County Environmental Health Services (Permit No. 99-1185). A copy of this permit is included in this section of the report.

The materials encountered in the exploratory borings were logged in the field by a Geolith Consultants geologist. The soils are described in accordance with the Unified Soil Classification System (ASTM D-2487). The logs of the borings are included in this Appendix.

Artificial fill, soil and bedrock samples were recovered from the exploratory borings at various depths appropriate to the investigation. Samples were obtained using a 2.5-inch Modified California sampler fitted with 2.5 inch thin walled brass tubes. Representative samples were brought to our laboratory for evaluation and testing, as described in Appendix B.

Resistance blow counts were recorded during sampling by dropping a 140-pound hammer with 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.

When a 1-inch O.D. Split Spoon sampler is used in conjunction with a 140-pound hammer, the blow counts are considered Standard Penetration resistance (SPT or "N") values. However, when the larger diameter Modified California sampler is used, the recorded blow counts are converted to "equivalent" standard penetration values by multiplying by a factor of 0.56. Blow counts shown on the boring logs that have been converted to SPT blow counts are noted with an asterisk (*).

The information shown on boring logs are our interpretation of the subsurface conditions at these locations. This information may not be valid at other location throughout the site, and the materials encountered in the exploratory borings may change with time as result of normal landscape-forming processes.

APPENDIX B
LABORATORY TEST RESULTS

APPENDIX B – LABORATORY TEST RESULTS

The laboratory testing program was directed toward a quantitative and qualitative evaluation of the physical and mechanical properties of the subsurface material at the site.

In-situ Moisture Content

The natural, in-place soil moisture content was determined on samples of the material recovered from the borings in accordance with ASTM Test Designation D-2216. These water contents are recorded on the boring logs at the appropriate sample depths.

In-situ Dry Density

In-place dry density determinations were performed on relatively undisturbed samples of the subsurface materials from the borings to evaluate their physical properties. The results of these tests are also shown on the boring logs at the appropriate depths.

Atterberg Limits

The Atterberg limits were determined for samples of the clayey subsurface soils to estimate the range of moisture content over which these material exhibit plasticity. The Atterberg limits were determined in accordance with ASTM Test Designation D-4318. These values are used to classify the soils in accordance with Unified Soil Classification System and to evaluate the expansion potential and approximate strength. The results of these tests are presented on the following figures, and are also shown on the boring logs at the appropriate sample depths.