Geotechnical Reports

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REPORT

GEOTECHNICAL STUDY
DUTTON PLACE
2975 DUTTON MEADOW
APN 043-121-006
SANTA ROSA, CALIFORNIA

Project Number: 1067.19.04.1

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INTRODUCTION

This report presents the results of our geotechnical study for the Dutton Place project to be constructed at 2975 Dutton Meadow in Santa Rosa, California. The property extends over relatively level terrain and is located on the west side of Dutton Meadow at the intersection with Mojave Avenue. A majority of the property is used for cattle grazing with a portion that is fenced off and contains a barn with stacks of supplies and debris. The barn is accessed by a gravel driveway off Dutton Meadow. Mature trees line the northern side of the driveway and a portion of the southern barn fence line. The site location is shown on Plate 1, Appendix A.

We understand it is proposed to construct 102 living units contained within 2- and 3-story buildings. The structures will be constructed on post-tensioned concrete slabs-on-grade.

Actual foundation loads are not known at this time. We anticipate the loads will be typical for the light to moderately heavy type of construction planned and that wall loads will range from about 1 to 2 kips per lineal foot.

Grading plans are not available, but we anticipate that the planned grading will be the minimum amount needed to construct level building pads and provide the building sites and paved areas with positive drainage, and could include cuts and fills on the order of 3 to 5 feet.

SCOPE

The purpose of our study, as outlined in our Professional Service Agreement dated November 7, 2003, was to generate geotechnical information for the design and construction of the project. Our scope of services included reviewing selected published geologic data pertinent to the site; evaluating subsurface conditions with test borings and laboratory tests; analyzing the field and laboratory data; and presenting this report with the following geotechnical information:

1. A brief description of soil and groundwater conditions observed during our study;

2. A discussion of seismic hazards that may affect the proposed development;

3. Conclusions and recommendations regarding:

    a. Primary geotechnical engineering concerns and mitigating measures, as applicable;

    b. Site preparation and grading including remedial grading of weak, porous, compressible and/or expansive surface soils;

    c. Foundation types, design criteria, and estimated settlement behavior;

    d. Lateral loads for retaining wall design;

    e. Support of concrete slabs-on-grade;

    f. Preliminary pavement thickness based on our experience with similar soils and projects;

    g. Utility trench backfill;

    h. Geotechnical engineering drainage improvements; and

i. Supplemental geotechnical engineering services.

STUDY

Site Exploration

We reviewed our previous geotechnical studies in the vicinity and selected geologic references pertinent to the site. The geologic literature reviewed is listed in Appendix B.

On December 9, 2003, we performed a geotechnical reconnaissance of the site and explored the subsurface conditions by drilling five test borings to depths ranging from about 12 to 15 feet. The borings were drilled with a truck-mounted drill rig equipped with 6-inch diameter, solid stem augers at the approximate locations shown on the Exploration Plan, Plate 2. The test boring locations were determined approximately by pacing their distance from features shown on the Exploration Plan and should be considered accurate only to the degree implied by the method used. Our field engineer located and logged the borings and obtained samples of the materials encountered for visual examination, classification and laboratory testing.

Relatively undisturbed samples were obtained from the borings at selected intervals by driving a 2.43-inch inside diameter, split spoon sampler, containing 6-inch long brass liners, using a 140-pound hammer dropping approximately 30 inches. The sampler was driven 12 to 18 inches. The blows required to drive each 6-inch increment were recorded and the blows required to drive the last 12 inches, or portion thereof, were converted to equivalent Standard Penetration Test (SPT) blow counts for correlation with empirical data.

The logs of the borings showing the materials encountered, groundwater conditions, converted blow counts and sample depths are presented on Plates 3 through 7. The soils are described in accordance with the Unified Soil Classification System, outlined on Plate 8.

The test boring logs show our interpretation of subsurface soil and groundwater conditions on the date and at the locations indicated. Subsurface conditions may vary at other locations and times. Our interpretation is based on visual inspection of soil samples, laboratory test results, and interpretation of drilling and sampling resistance. The location of the soil boundaries should be considered approximate. The transition between soil types may be gradual.

Laboratory Testing

The samples obtained from the borings were transported to our office and re-examined by the project engineer to verify soil classifications, evaluate characteristics, and assign tests pertinent to our analysis. Selected samples were laboratory tested to determine their water content, dry density, classification (Atterberg Limits, percent of silt and clay), particle size distribution, unconfined compressive strength, expansion potential (Expansion Index - EI), and corrosivity. The test results are presented on the test boring logs. Results of the classification, particle size distribution, unconfined compression strength, and corrosivity tests are presented on Plates 9 through 12.

SITE CONDITIONS

General

Sonoma County is located within the California Coast Range geomorphic province. This province is a geologically complex and seismically active region characterized by sub-parallel northwest-trending faults, mountain ranges and valleys. The oldest bedrock units are the Jurassic-Cretaceous Franciscan Complex and Great Valley sequence sediments originally deposited in a marine environment. Subsequently, younger rocks such as the Tertiary-age Sonoma Volcanics group, the Plio-Pleistocene-age Clear Lake Volcanics and sedimentary rocks such as the Guinda, Domengine, Petaluma, Wilson Grove, Cache, Huichica and Glen Ellen formations were deposited throughout the province. Extensive folding and thrust faulting during late Cretaceous through early Tertiary geologic time created complex geologic conditions that underlie the highly varied topography of today. In valleys, the bedrock is covered by thick alluvial soils.

Geology and Soils

The United States Geological Survey (USGS) geologic maps reviewed, Fox et al. (1973), indicate the property is underlain by alluvial fan deposits bordering uplands (Qof) and alluvial fan deposits grading headward to terrace deposits (Qyf). The alluvial fan deposits are shown to consist of moderately sorted fine sand and silt with gravel becoming more abundant toward fan heads.

Mapping by the U.S. Soil Conservation Service (Miller, 1990) has classified soil over the portion of this property proposed for development as belonging to the Clear Lake series and the Wright series. The Clear Lake series comprises two soil horizons. The topsoil is shown to be a clay that exhibits moderate to high plasticity (LL = 50-60; PI = 20-35) and high shrink-swell potential, and extends from a depth of 0 to 60 inches. The subsoil is shown to be a sandy clay loam that exhibits low plasticity (LL = 15-25; PI = 5-10) and moderate shrink-swell potential, and extends from a depth of 60 to 72 inches. Runoff over these soils is slow. The hazard of erosion is slight depending on slope. The risk of corrosion is given as high for uncoated steel.

The Wright series comprises two soil horizons. The topsoil is shown to be a loam and sandy clay loam that is non-plastic, exhibits low shrink-swell potential, and extends from a depth of 0 to 25 inches. The subsoil is shown to be a clay that exhibits moderate to high plasticity (LL = 40-50; PI = 20-30) and high shrink-swell potential, and extends from a depth of 25 to 62 inches. Runoff over these soils is very slow. The hazard of erosion is none to slight depending on slope. The risk of corrosion is given as high for uncoated steel. Corrosivity test results for the on-site near surface soils are presented on Plate 12.

Surface

The property extends primarily over relatively level terrain. The southeast corner of the property is about two feet higher than the surrounding area due to fill that was previously placed. The vegetation consists of seasonal grasses.

In general, the ground surface is soft and spongy. This is a condition generally associated with weak, porous surface soils. The surface soils are disturbed by randomly arrayed shrinkage cracks generally associated with expansive soils. Locally, expansive soils shrink and swell with the weather cycle. The cyclic shrinking and swelling tends to disturb the upper portion of the expansive clay. This zone is defined hereinafter as the active layer. Natural drainage consists of sheet flow over the ground surface that concentrates in drainage ditches.

Subsurface

Our borings and laboratory tests indicate that the portion of the site we studied is blanketed by up to 3 feet of weak, porous, compressible, clayey soils (topsoil). Porous soils appear hard and strong when dry but become weak and compressible as their moisture content increases towards saturation. These soils exhibit high plasticity (LL = 48-57; PI = 31-40) and moderate expansion potential (EI = 89). At the southeast corner of the property and in the barn area, the surface soils are covered by up to two feet of heterogeneous fill. Heterogeneous fill is a material with varying density, strength, compressibility and shrink-swell characteristic that often has an unknown origin and placement history. These surface materials (fill, topsoil) are underlain by very stiff to hard clay with varying sand content and dense to very dense sand with varying clay and gravel content. A detailed description of subsurface conditions found in our borings is given in Plates 3 through 7, Appendix A.

Groundwater

Free groundwater was detected in some of our borings at depths ranging from 7 to 11 feet below the ground surface at the time of drilling. Fluctuation in the groundwater level typically occurs because of a variation in rainfall intensity, duration and other factors such as flooding and periodic irrigation.

Flooding

Our review of the Sonoma County Multiple Listing Service Earthquake and Flood Zone Map for Sonoma County, California, indicates that the proposed building site is not located within Zone "A," the 100-year flood boundary as designated by the Federal Emergency Management Agency. Evaluation of flooding potential is typically the responsibility of the project civil engineer.

DISCUSSION AND CONCLUSIONS

Seismic Hazards

General

We did not observe subsurface conditions within the portion of the property we studied that would suggest the presence of materials that may be susceptible to seismically induced densification or lurching. Therefore, we judge the potential for the occurrence of these phenomena at the site to be low.

Seismicity

Data presented by the Working Group on California Earthquake Probabilities (2002) estimates the chance of one or more large earthquakes (Magnitude 6.7 or greater) in the San Francisco Bay region within the next 30 years to be approximately 62 percent. Therefore, future seismic shaking should be anticipated at the site. It will be necessary to design and construct the proposed project in strict adherence with current standards for earthquake-resistant construction.

Faulting

We did not observe landforms within the area that would indicate the presence of active faults and the site is not within a current Alquist-Priolo Earthquake Fault Zone. Therefore, we believe the risk of fault rupture at the site is low. However, the site is within an area affected by strong seismic activity. Several northwest-trending Earthquake Fault Zones exist in close proximity to and within several miles of the site (Brown, 1970; Helley and Herd, 1977; Bortugno, 1982). The shortest distances from the site to the mapped surface expression of these faults are presented below in Table 1.

TABLE 1

ACTIVE FAULT PROXIMITY

Fault	Direction	Distance-Miles
San Andreas	SW	17½
Healdsburg-Rodgers Creek	NE	3
West Napa	E	20
Maacama	N	13

Liquefaction

Liquefaction is a rapid loss of shear strength experienced in saturated, predominantly granular soils below the groundwater level during strong earthquake ground shaking due to an increase in pore water pressure. The occurrence of this phenomena is dependent on many complex factors including the intensity and duration of ground shaking, particle size distribution and density of the soil. The granular soils at the site located below the groundwater surface were generally dense to very dense and, where medium dense, were logged as clayey. Clayey soils are considered to have a low susceptibility to liquefaction (Marcuson, 1990). Therefore, we judge the potential for liquefaction at the site is low.

Geotechnical Issues

General

Based on our study, we judge the proposed project can be built as planned, provided the recommendations presented in this report are incorporated into its design and construction. The primary geotechnical concerns during design and construction of the project are:

1. The presence of up to 3 feet of highly expansive, weak, porous, compressible, surface clayey soils and localized areas with up to 2 feet of heterogeneous fill.

2. The detrimental effects of uncontrolled surface runoff on the long-term satisfactory performance of residential structures.

3. The strong ground shaking predicted to impact the site during the life of the project.

Heterogeneous Fill

Heterogeneous fills of unknown quality and unknown method of placement, such as those found at the Dutton Place site, can settle and/or heave erratically under the load of new fills, structures, slabs, and pavements. Footings, slabs, and pavements supported on heterogeneous fill could also crack as a result of such erratic movements. Thus, it will be necessary to remove the heterogeneous fill and replace it as an engineered fill if it is to be used for structural support.

Expansive Soil

Expansive surface soils, such as those found at the site, shrink and swell as they lose and gain moisture during the local weather cycle. The resulting volumetric changes can heave and crack lightly loaded foundations, slabs and pavements. The detrimental effects of these movements can be remediated by pre-swelling the expansive soils and covering them with a moisture fixing and confining blanket of properly compacted select fill, as subsequently defined. In building areas, the blanket thickness required depends on the expansion potential of the soils and the anticipated performance of the foundations and slabs. In order to effectively reduce foundation and slab heave given the expansion potential of the site's soils, a blanket thickness of 30 inches will be needed. In exterior slab and paved areas, the select fill blanket will need to be 12 inches thick.

As an alternative to the use of select fill in building areas, the structures can be supported on post-tensioned slabs supported on general engineered fill.

Foundation and Slab Support - Provided grading is performed as discussed above, satisfactory foundation support can be obtained from spread footings that bottom on the select engineered fill. Conventional interior slab-on-grade floors can also be satisfactorily supported on the select engineered fill. Post-tensioned slabs can be satisfactorily supported on general engineered fill.

Exterior Slabs and Pavements - Exterior slabs and pavements will heave and crack as the expansive soils shrink and swell through the yearly weather cycle. Slab and pavement cracking and distress is typically concentrated along edges where moisture content variation is more prevalent within subgrade soils. Slab and pavement performance and the incidence of repair can be reduced by covering the pre-swelled expansive soils with at least 12 inches of select fill (see "On-Site Soil Quality" section) prior to constructing the slab or pavement required to carry the anticipated traffic.

On-Site Soil Quality

All fill materials used in the upper 30 inches of the building areas, if conventional slabs are used, and the upper 12 inches of exterior slab and pavement subgrade must be select, as subsequently described in "Recommendations." We anticipate that, with the exception of organic matter and of rocks or lumps larger than 6 inches in diameter, some of the excavated material will be suitable for re-use as general and select fill. The expansive clayey soils, where encountered, will not be suitable for use as select fill unless stabilized with lime.

Select Fill

The select fill can consist of approved on-site soils, import materials with a low expansion potential, or lime stabilized on-site clayey soils. Lime stabilized soils may prevent the growth of landscape vegetation due to the inherent elevated pH level of the soil. The geotechnical engineer must approve the use of on-site soils as select fill during grading.

Settlement

If remedial grading is performed and the spread footings are installed in accordance with the recommendations presented in this report, we estimate that post-construction differential settlements across the building will be about 1 inch.

Surface Drainage

Surface runoff typically sheet flows over the ground surface but can be concentrated by the planned site grading, landscaping, and drainage. The surface runoff can pond against structures and cause deeper than normal soil heave and/or seep into the slab rock. Therefore, strict control of surface runoff is necessary to provide long-term satisfactory performance of residential projects. It will be necessary to divert surface runoff around improvements and provide positive drainage away from structures. This can be achieved by constructing the building pad several inches above the surrounding area and conveying the runoff into man made drainage ditches or natural swales that lead downgradient of the site.

RECOMMENDATIONS

Seismic Design

The site is within 1997 Uniform Building Code (UBC) (ICBO, 1997) seismic zone 4; therefore, a Seismic Zone Factor "Z" of 0.4 (Table 16-I), modified as necessary to conform with ordinance(s) adopted by the County of Sonoma, should be used. The soil profile at the site approximates type S_D (Table 16-J). The 1997 UBC has identified the locations of known active fault near-source zones in California and along the California/Nevada border. The purpose of these zones is to determine a near-source factor to be used in design for every site located within Seismic Zone 4. The near-source zones have been mapped considering the surface projection of the source (as opposed to the mapped surface expression of the fault) using the dip angle of the fault. For non-vertical faults, the dip of the fault is an important parameter because it determines the location of the fault at depth.

Active faults have been classified as A, B, or C in accordance with the criteria specified in the 1997 UBC (Table 16-U). Only faults classified as A or B are shown on the 1997 UBC maps (ICBO, 1998) because the UBC assumes faults classified as C do not increase the near-source factor. The distance from the site to the A and B faults that have the potential to control the near-source factors are listed in Table 2 below. The distances shown in Table 2 are not converted from Table 1, presented earlier. Each table measures different distances, as previously discussed.

TABLE 2

1997 UBC NEAR-SOURCE ZONE DISTANCES

Fault	Direction	Fault Type	Distance-km
Rodgers Creek	NE	A	4 ½
Maacama (south)	N	B	>15

Using Tables 16-S and 16-T of the 1997 UBC and Table 2, the near-source factors, N_a and N_v, for the site are 1.3 and 1.7, respectively. The project structural engineer should determine the appropriate seismic response coefficients (C_a and C_v) needed to determine total design lateral force in accordance with the UBC.

Grading

Site Preparation

Areas to be developed should be cleared of vegetation and debris including that left by the removal of obsolete structures. Trees and shrubs that will not be part of the proposed development should be removed and their primary root systems grubbed. Cleared and grubbed material should be removed from the site and disposed of in accordance with County Health Department guidelines. We did not observe septic tanks, leach lines or underground fuel tanks during our study. Any such appurtenances found during grading should be capped and sealed and/or excavated and removed from the site, respectively, in accordance with established guidelines and requirements of the County Health Department. Voids created during clearing should be backfilled with engineered fill as recommended herein.

Stripping

Areas to be graded should be stripped of the upper few inches of soil containing organic matter. Soil containing more than two percent by weight of organic matter should be considered organic. Actual stripping depth should be determined by a representative of the geotechnical engineer in the field at the time of stripping. The strippings should be removed from the site, or if suitable, stockpiled for re-use as topsoil in landscaping.

Excavations

Following initial site preparation, excavation should be performed as planned or recommended herein. Excavations extending below the proposed finished grade should be backfilled with suitable materials compacted to the requirements given below.

Within building areas, the heterogeneous fill and weak, porous soils should be excavated to within 6 inches of their entire depth (up to about 3 feet in our borings). If conventional slabs-on-grade are used, additional excavation should be performed, as necessary, to allow space for the installation of a blanket of select fill, at least 30 inches thick, beneath the building pad subgrade. The excavation of heterogeneous fill and expansive soils should also extend at least 12 inches below exterior slab and/or pavement subgrade to allow space for the installation of the select fill blanket discussed in the conclusions section of this report.

The excavation of heterogeneous fill and weak, porous, expansive surface materials should extend at least 5 feet beyond the outside edge of the exterior footings of the proposed buildings and 3 feet beyond the edge of exterior slabs and/or pavements. The excavated materials should be stockpiled for later use as compacted fill, or removed from the site, as applicable.

At all times, temporary construction excavations should conform to the regulations of the State of California, Department of Industrial Relations, Division of Industrial Safety or other stricter governing regulations. The stability of temporary cut slopes, such as those constructed during the installation of underground utilities, should be the responsibility of the contractor. Depending on the time of year when grading is performed, and the surface conditions exposed, temporary cut slopes may need to be excavated to 1¼:1, or flatter. The tops of the temporary cut slopes should be rounded back to 2:1 in weak soil zones.

Fill Quality

All fill materials should be free of perishable matter and rocks or lumps over 6 inches in diameter and must be approved by the geotechnical engineer prior to use. The upper 30 inches of fill (for conventional slabs-on-grade) beneath and within 5 feet of the building footprint and the upper 12 inches of fill beneath and within 3 feet of exterior slabs and/or pavement edges should be select fill. We judge that the on-site soils are generally suitable for use as general fill but will not be suitable for use as select fill unless they are stabilized with lime. Lime stabilized soils may prevent the growth of landscape vegetation due to the inherent elevated pH level of the soil.

Select Fill

Select fill should be free of organic matter, have a low expansion potential, and conform in general to the following requirements:

SIEVE SIZE	PERCENT PASSING (By Dry Weight)
6 inch	100
4 inch	90 - 100
No. 200	10 - 60

Liquid Limit - 40 Percent Maximum
Plasticity Index - 15 Percent Maximum
R-value - 20 Minimum

Expansive on-site soils may be used as select fill if they are stabilized with lime. In general, imported fill, if needed, should be select. Material not conforming to these requirements may be suitable for use as import fill; however, it shall be the contractor's responsibility to demonstrate that the proposed material will perform in an equivalent manner. The geotechnical engineer should approve imported materials prior to use as compacted fill.

Lime Stabilization

For preliminary planning purposes, we estimate that high calcium lime mixed at about 5 percent (dry weight) will stabilize the expansive site soils. The percentage of lime required needs to be verified prior to construction with laboratory R-value and/or pH testing.

The lime stabilization should be performed in accordance with Section 24 of the Caltrans Standard Specifications except that a curing seal will not be required, provided the moisture content of the lime-stabilized material is maintained at or above optimum moisture content until it is permanently covered with subsequent construction. Lime stabilized materials are not suitable for reuse as general fill, select fill or backfill after compaction has taken place.

Fill Placement

The surface exposed by stripping and removal of heterogeneous fill and weak, expansive, compressible surface soils should be scarified to a depth of at least 6 inches, uniformly moisture-conditioned to about 4 percent above optimum and compacted to at least 90 percent of the maximum dry density of the materials as determined by ASTM Test Method D-1557. In expansive soil areas, moisture conditioning should be sufficient to completely close all shrinkage cracks for their full depth within pavement, exterior slab and building areas. If grading is performed during the dry season, the shrinkage cracks may extend to a few feet below the surface. Therefore, it may be necessary to excavate a portion of the cracked soils to obtain the proper moisture condition and degree of compaction. Approved fill material should then be spread in thin lifts, uniformly moisture-conditioned to near optimum and properly compacted. All structural fills, including those placed to establish site surface drainage, should be compacted to at least 90 percent relative compaction. Expansive soils used as fill should be moisture-conditioned to about 4 percent above optimum. Only approved select materials should be used for fill within the upper 30 inches of interior slab subgrades (conventional slabs-on-grade) and within the upper 12 inches of exterior slabs and/or pavement subgrades.

Permanent Cut and Fill Slopes

In general, cut and fill slopes should be no steeper than 2:1. In expansive soil areas, cut and fill slopes should be no steeper than 3:1.

Wet Weather Grading

Generally, grading is performed more economically during the summer months when on-site soils are usually dry of optimum moisture content. Delays should be anticipated in site grading performed during the rainy season or early spring due to excessive moisture in on-site soils. Special and relatively expensive construction procedures, including dewatering of excavations and importing granular soils, should be anticipated if grading must be completed during the winter and early spring or if localized areas of soft saturated soils are found during grading in the summer and fall.

Foundation Support

Depending on the remedial grading performed at the site, the proposed structures can be supported on spread footings that bottom on select engineered fill or on post-tensioned slabs.

Spread Footings

Spread footings should be at least 12 inches wide and should bottom on select engineered fill at least 18 inches below pad subgrade. Because of the potential for uneven soil support, continuous footings (except for retaining walls) should have sufficient reinforcement to span, as a simple beam, an unsupported distance of approximately 10 feet.

The bottoms of all footing excavations should be thoroughly cleaned out or wetted and compacted using hand-operated tamping equipment prior to placing steel and concrete. This will remove the soils disturbed during footing excavations, or restore their adequate bearing capacity, and reduce post-construction settlements. Footing excavations should not be allowed to dry before placing concrete. If shrinkage cracks appear in soils exposed in the footing excavations, the soil should be thoroughly moistened to close all cracks prior to concrete placement. The moisture condition of the foundation excavations should be checked by the geotechnical engineer no more than 24 hours prior to placing concrete.

Bearing Pressures - Footings installed in accordance with these recommendations may be designed using allowable bearing pressures of 2000, 2500, and 3000 pounds per square foot (psf), for dead loads, dead plus code live loads, and total loads (including wind and seismic), respectively.

Lateral Pressures - The portion of spread footing foundations extending into firm natural soil or select engineered fill may impose a passive equivalent fluid pressure and a friction factor of 350 pcf and 0.35, respectively, to resist sliding. Because full mobilization of the passive resistance requires some horizontal movement (diminished frictional resistance), the frictional component should be reduced by 50 percent if both passive pressure and friction are used simultaneously. Passive pressure should be neglected within the upper 6 inches, unless the soils are confined by concrete slabs or pavements.

Post-Tension Slabs

A post tension (PT) slab should be a designed to accommodate edge moisture variation distances of 5 and 4 feet for edge and center lift conditions, respectively, a differential edge swell of 0.8 inches and a center swell of 0.9 inches. These parameters were developed using the Post-Tensioning Institute manual "Design and Construction of Post-Tensioned Slabs-On-Ground, Second Edition" (1996). A PT slab installed in accordance with the foregoing recommendations may be designed using allowable bearing pressures of 2000, 2500, and 3000 pounds per square foot (psf) for dead loads, dead plus code live loads, and total loads, including wind and seismic, respectively. We recommend a minimum slab thickness of 10 inches and a 12-inch-wide (minimum) perimeter thickened edge. Concentrated loads in the slab interior should also be supported by thickened beams within the slab.

The PT slab should be protected with an impervious membrane (10 mil visqueen or better) and 4 inches of slab rock similar to that recommended for conventional slabs-on-grade. The subgrade soils within and for a distance of 5 feet beyond the footprint of the buildings should be kept pre-swelled until the capillary moisture break is placed. The moisture content of the subgrade soils should be approved by the geotechnical engineer within 24 hours prior to placing the capillary moisture break.

Because PT slabs are designed to move with the expansive soils as they shrink and swell, structural elements that are attached to the structure, such as patio overhangs and stairwells, but have there own foundation should not be used or should be founded on the PT slab. Exterior flatwork and concrete walkway subgrades should be underlain by at least 12 inches of select fill and be pre-swelled by soaking prior to installation of the walkway. In addition, concrete walkways should be:

1) Cast separate from the PT slab to allow differential movement to occur without distressing the walkway;

2) Reinforced to reduce cracks; and

3) Grooved to induce cracking in a non-obtrusive manner.

The Post-Tensioning Institute states "Consideration should be given to 'artificial' effects, such as planter units adjacent to structural bearing areas. Tree roots can be a serious problem and cause volume reduction in limited areas, thus causing distress to the slab foundation. Trees that are planted closer to the foundation than half their ultimate height can be expected to cause significant differential movement."

Retaining Walls

Retaining walls constructed at the site must be designed to resist lateral earth pressures plus additional lateral pressures that may be caused by surcharge loads applied at the ground surface behind the walls.

Retaining walls free to rotate (yielding greater than 0.1 percent of the wall height at the top of the backfill) should be designed for active lateral earth pressures. If walls are restrained by rigid elements to prevent rotation, they should be designed for "at rest" lateral earth pressures.

Retaining walls should be designed to resist the following earth equivalent fluid pressures (triangular distribution):

Active Pressure (level backfill) .... 40 pcf
Active Pressure (3:1 or steeper backfill) ....60 pcf
At Rest Pressure..70 pcf

These pressures do not consider additional loads resulting from adjacent foundations or other loads. If these additional surcharge loadings are anticipated, we can assist in evaluating their effects. Where retaining wall backfill is subject to vehicular traffic, the walls should be designed to resist an additional surcharge pressure equivalent to two feet of additional backfill. Retaining walls will yield slightly during backfilling. Therefore, walls should be backfilled prior to building on or adjacent to the walls.

Backfill against retaining walls should be compacted to at least 90 and not more than 95 percent relative compaction. Over-compaction or the use of large compaction equipment should be avoided because increased compactive effort can result in lateral pressures higher than those recommended above.

Foundation Support

Retaining walls should be supported on spread footings or drilled piers, as applicable, designed in accordance with the recommendations presented in this report. Retaining wall foundations should be designed by the project civil or structural engineer to resist the lateral forces set forth in this section.

Drilled Piers - Drilled, cast-in-place, reinforced concrete piers should be used for retaining wall foundation support where grading is not used to remediate expansive soil heave. Drilled piers should be at least 12 inches in diameter and should extend at least 8 feet below planned pad elevation.

The portion of the piers extending below the active layer (3 feet) may be designed using an allowable skin friction of 500 psf for dead load plus long term live loads. This value can be increased by ⅓ for total loads, including wind or seismic forces. End bearing should be neglected because of the difficulty of cleaning out small diameter pier holes, and the uncertainty of mobilizing end bearing and skin friction simultaneously.

Lateral loads on piers will be resisted by passive pressure on the soil. An equivalent fluid pressure of 350 pcf acting on two pier diameters should be used. Confinement for passive pressure may be assumed from 3 feet below the lowest adjacent finished ground surface, as applicable.

The piers should be interconnected with grade beams to support wall loads and to redistribute stresses imposed by wind or earthquakes. The grade beams should be designed to span between the piers in accordance with structural requirements. The steel from the piers should extend sufficient distance into the grade beams to develop its full bond strength.

The piers and grade beams should be designed to resist uplift pressures imposed by expansive soils. The uplift pressure should be assumed to be 2,000 psf of grade beam surface contact.

We encountered groundwater and/or caving-prone soils within planned pier depth during our study. If groundwater is encountered during drilling, it may be necessary to de-water the holes and/or place the concrete by the tremmie method. If caving soils are encountered, it may be necessary to case the holes. The drilling subcontractor should review this report, become familiar with site conditions as they pertain to his operation and draw his own conclusions regarding drilling difficulty, suitable drill rigs and the need for casing and dewatering prior to bidding.

Concrete mix design and placement should be done in accordance with the current ADSC and/or ACI specifications. Concrete should not be allowed to mushroom at the top of the piers.

Wall Drainage and Backfill

Retaining walls should be backdrained as shown on Plate 13, Appendix A. The backdrains should consist of 4-inch diameter, rigid perforated pipe embedded in Class 2 permeable material. The pipe should be PVC Schedule 40 or ABS with SDR 35 or better, and the pipe should be sloped to drain to outlets by gravity. The top of the pipe should be at least 8 inches below lowest adjacent grade. The Class 2 permeable material should extend to within 1½ feet of the surface. The upper 1½ feet should be backfilled with compacted soil to exclude surface water. Expansive soils should not be used for wall backfill. Where expansive soils are present in the excavation made to install the retaining wall, the excavation should be sloped back 1:1 from the back of the footing or grade beam. The ground surface behind retaining walls should be sloped to drain. Where migration of moisture through retaining walls would be detrimental, retaining walls should be waterproofed.

Slab-On-Grade

Provided grading is performed in accordance with the recommendations presented herein, slabs-on-grade (non post-tensioned) should be underlain by select engineered fill as previously discussed in "Excavations". Slab-on-grade subgrade should be rolled to produce a dense, uniform surface. The future expansion potential of the subgrade soils should be reduced by thoroughly presoaking the slab subgrade prior to concrete placement. The moisture condition of the subgrade soils should be checked by the geotechnical engineer no more than 24 hours prior to placing the capillary moisture break. The slabs should be underlain with a capillary moisture break consisting of at least 4 inches of clean, free-draining crushed rock or gravel (excluding pea gravel) at least ¼-inch and no larger than ¾-inch in size. Interior slabs subject to vehicular traffic should be underlain by crushed rock. Class 2 aggregate base can be used for slab rock under exterior slabs. Where migration of moisture vapor through slabs would be detrimental, an impermeable membrane moisture vapor barrier should be provided between the drain rock and the slabs. Outlets should be provided for the interior slab rock to drain through foundation walls into a perimeter subdrain. As an alternative, living area slabs should be provided with an underdrain system.

The installation of these alternative subdrain systems is discussed in "Geotechnical Drainage Improvements."

Slabs should be at least 4 inches thick, and should be designed by the project civil or structural engineer to support the anticipated loads and reduce cracking. Garage slabs, if conventional slabs are used, should be carefully separated from foundations and framing elements with felt paper, mastic, or other positive and low friction materials.

Utility Trenches

The shoring and safety of trench excavations is solely the responsibility of the contractor. Attention is drawn to the State of California Safety Orders dealing with "Excavations and Trenches."

Unless otherwise specified by the City of Santa Rosa, on-site, inorganic soil may be used as general utility trench backfill. Where utility trenches support pavements, slabs and foundations, trench backfill should consist of aggregate baserock. The baserock should comply with the minimum requirements in Caltrans Standard Specifications, Section 26 for Class 2 Aggregate Base. Trench backfill should be moisture-conditioned as necessary, and placed in horizontal layers not exceeding 8 inches in thickness, before compaction. Each layer should be compacted to at least 90 percent relative compaction as determined by ASTM Test Method D-1557. The top 6 inches of trench backfill below vehicle pavement subgrades should be moisture-conditioned as necessary and compacted to at least 95 percent relative compaction. Jetting or ponding of trench backfill to aid in achieving the recommended degree of compaction should not be attempted.

Pavements

Provided the site grading is performed to remediate expansive soil heave, as recommended herein, the uppermost 12-inches of pavement subgrade soils will be either imported select fill with a minimum R-value of 20 or lime stabilized site soils that generally have an R-value of at least 50. Based on those R-values we recommend the pavement sections listed on Tables 3 and 4 be used.

TABLE 3
PAVEMENT SECTIONS
WITH IMPORTED SELECT FILL SUBGRADE

TI	THICKNESS (feet)		
	ASPHALT CONCRETE	CLASS 2 AGGREGATE BASE	IMPORTED SELECT FILL*
7.0	0.30	1.05	1.0
6.5	0.30	0.95	1.0
6.0	0.25	0.85	1.0
5.5	0.25	0.75	1.0
5.0	0.20	0.70	1.0
4.5	0.20	0.60	1.0

* R-value ≥ 20

TABLE 4
PAVEMENT SECTIONS
WITH LIME STABILIZED SELECT FILL SUBGRADE

TI	THICKNESS (feet)		
	ASPHALT CONCRETE	CLASS 2 AGGREGATE BASE	LIME STABILIZED* SELECT FILL
7.0	0.30	0.50	1.0
6.5	0.30	0.50	1.0
6.0	0.25	0.50	1.0
5.5	0.25	0.50	1.0
5.0	0.20	0.50	1.0
4.5	0.20	0.50	1.0

* R-value ≥ 50

Pavement thicknesses were computed using Method 301 F of the Caltrans Highway Design Manual and are based on a pavement life of 20 years. These recommendations are intended to provide support for the auto and light truck traffic represented by the indicated Traffic Indexes. They are not intended to provide pavement sections for heavy concentrated construction storage or wheel loads such as forklifts, parked truck-trailers and concrete trucks or for post-construction concentrated wheel loads such as self-loading dumpster trucks.

In areas where heavy construction storage and wheel loads are anticipated, the pavements should be designed to support these loads. Support could be provided by increasing pavement sections or by providing reinforced concrete slabs. Alternatively, paving can be deferred until heavy construction storage and wheel loads are no longer present. Loading areas for self-loading dumpster trucks should be provided with reinforced concrete slabs at least 6 inches thick, and reinforced with No. 4 bars at 12-inch centers each way. Alternatively, the asphalt concrete section should be increased to at least 8 inches in these areas.

Prior to placement of aggregate base, the upper 6 inches of the pavement subgrade soils (excluding lime stabilized soils) should be scarified, uniformly moisture-conditioned to near optimum, and compacted to at least 95 percent relative compaction to form a firm, non-yielding surface. Lime stabilized select fill subgrade soils should be compacted as specified in Section 24 of the Caltrans Standard Specifications.

Aggregate base materials should be spread in thin layers, uniformly moisture-conditioned, and compacted to at least 95 percent relative compaction to form a firm, non-yielding surface. The materials and methods used should conform to the requirements of the City of Santa Rosa and the current edition of the Caltrans Standard Specifications, except that compaction requirements should be based on ASTM Test Method D-1557. Aggregate used for the base course should comply with the minimum requirements specified in Caltrans Standard Specifications, Section 26 for Class 2 Aggregate Base.

Parking Lot Drainage

Water tends to migrate under pavements and collect in the aggregate courses at low areas on parking lot subgrade soils, such as around storm drain inlets and the thread of paved swales leading to inlets. The ponded water will soften subgrade soils and, under repetitive heavy-wheel loads, will induce inordinately high stresses on the subgrade and pavement components that could result in untimely maintenance. Under-pavement drainage can be improved and maintenance reduced by replacing a 12-inch wide strip (extending at least 15 feet on either side of the inlet) of the select subbase layer or subgrade soils with ¾-inch or 1½-inch free-draining Class 1 Permeable Material. The drain rock should be outletted into the storm drain inlet. Storm drain trenches can be made to serve as pavement subdrains. We should be consulted to verify the suitability of storm drain trenches as pavement subdrains in a case-specific basis.

Where pavements will abut landscaped areas, the pavement baserock layer and subgrade soils should be protected against saturation from irrigation and rainwater with a subdrain, similar to that previously discussed. The subdrain should extend to a depth of at least 18 inches below the bottom of the baserock layer.

Wet Weather Paving

In general, the pavements should be constructed during the dry season to avoid the saturation of the subgrade and base materials, which often occurs during the wet winter months. If pavements are constructed during the winter, a cost increase relative to drier weather construction should be anticipated. Unstable areas may have to be overexcavated to remove soft soils. The excavations will probably require backfilling with imported crushed (ballast) rock. The geotechnical engineer should be consulted for recommendations at the time of construction.

Geotechnical Drainage

Surface water should be diverted away from slopes, foundations and edges of pavements. Surface drainage gradients within 5 feet of building foundations should be constructed with a minimum slope of 2 percent for paved areas and 4 percent for unpaved areas. Where a flatter gradient is required to satisfy design constraints, area drains should be installed within the rear and side yard swales with a spacing no greater than about 20 feet. Roofs should be provided with gutters and the downspouts should empty onto splash blocks that discharge directly onto paved areas or be connected to closed (glued Schedule 40 PVC or better) conduits discharging well away from foundations, preferably onto paved areas or into the site's surface drainage system.

Where living area slab subgrades are less than 6 inches above adjacent exterior grade and where migration of moisture through the slab would be detrimental, slab underdrains should be installed to prevent surface runoff from entering the slab rock. Slab underdrains should consist of trenches that extend at least 6 inches below the bottom of the slab rock and slope to drain by gravity. The slab underdrain trenches should be at least 6 inches wide and spaced no further than 15 feet, both ways. Additional drain trenches should be installed, as necessary, to drain all isolated under slab areas. Four-inch diameter perforated pipe sloped to drain to outlets by gravity should be placed in the bottom of the trenches. Slab underdrain trenches should be backfilled to subgrade level with clean, free draining slab rock. An illustration of this system is shown on Plate 14. If slab underdrains are not used, it should be anticipated that water will enter the slab rock, permeate through the concrete slab and ruin floor coverings. Roof downspouts and surface drains must be maintained entirely separate from slab underdrains and retaining wall backdrains.

Water seepage or the spread of extensive root systems into the soil subgrade of footings, slabs or pavements could cause differential movements and consequent distress in these structural elements. Landscaping should be planned with consideration for these potential problems.

Maintenance

Periodic land maintenance will be required. Surface and subsurface drainage facilities should be checked frequently, and cleaned and maintained as necessary. A dense growth of deep-rooted ground cover must be maintained on all slopes to reduce sloughing and erosion. Sloughing and erosion that occurs must be repaired promptly before it can enlarge.

Supplemental Services

RGH Geotechnical and Environmental Consultants (RGH) recommends that we be retained to review the project plans and specifications to determine if they are consistent with our recommendations. In addition, we should be retained to observe construction, particularly site excavations, compaction of fills and backfills, foundation and subdrain installations, and perform field and laboratory testing.

If, during construction, we observe subsurface conditions different from those encountered during the explorations, we should be allowed to amend our recommendations accordingly. If different conditions are observed by others, or appear to be present beneath excavations, RGH should be advised at once so that these conditions may be evaluated and our recommendations reviewed and updated, if warranted. The validity of recommendations made in this report is contingent upon our being notified and retained to review the changed conditions.

If more than 18 months have elapsed between the submission of this report and the start of work at the site, or if conditions have changed because of natural causes or construction operations at, or adjacent to, the site, the recommendations made in this report may no longer be valid or appropriate. In such case, we recommend that we be retained to review this report and verify the applicability of the conclusions and recommendations or modify the same considering the time lapsed or changed conditions. The validity of recommendations made in this report is contingent upon such review.

These supplemental services are performed on an as-requested basis and are in addition to this geotechnical study. We cannot accept responsibility for items that we are not notified to observe or for changed conditions we are not allowed to review.

LIMITATIONS

This report has been prepared by RGH for the exclusive use of Cobblestone Homes and their consultants as an aid in the design and construction of the proposed Dutton Place project described in this report.

The validity of the recommendations contained in this report depends upon an adequate testing and monitoring program during the construction phase. Unless the construction monitoring and testing program is provided by our firm, we will not be held responsible for compliance with design recommendations presented in this report and other addendum submitted as part of this report.

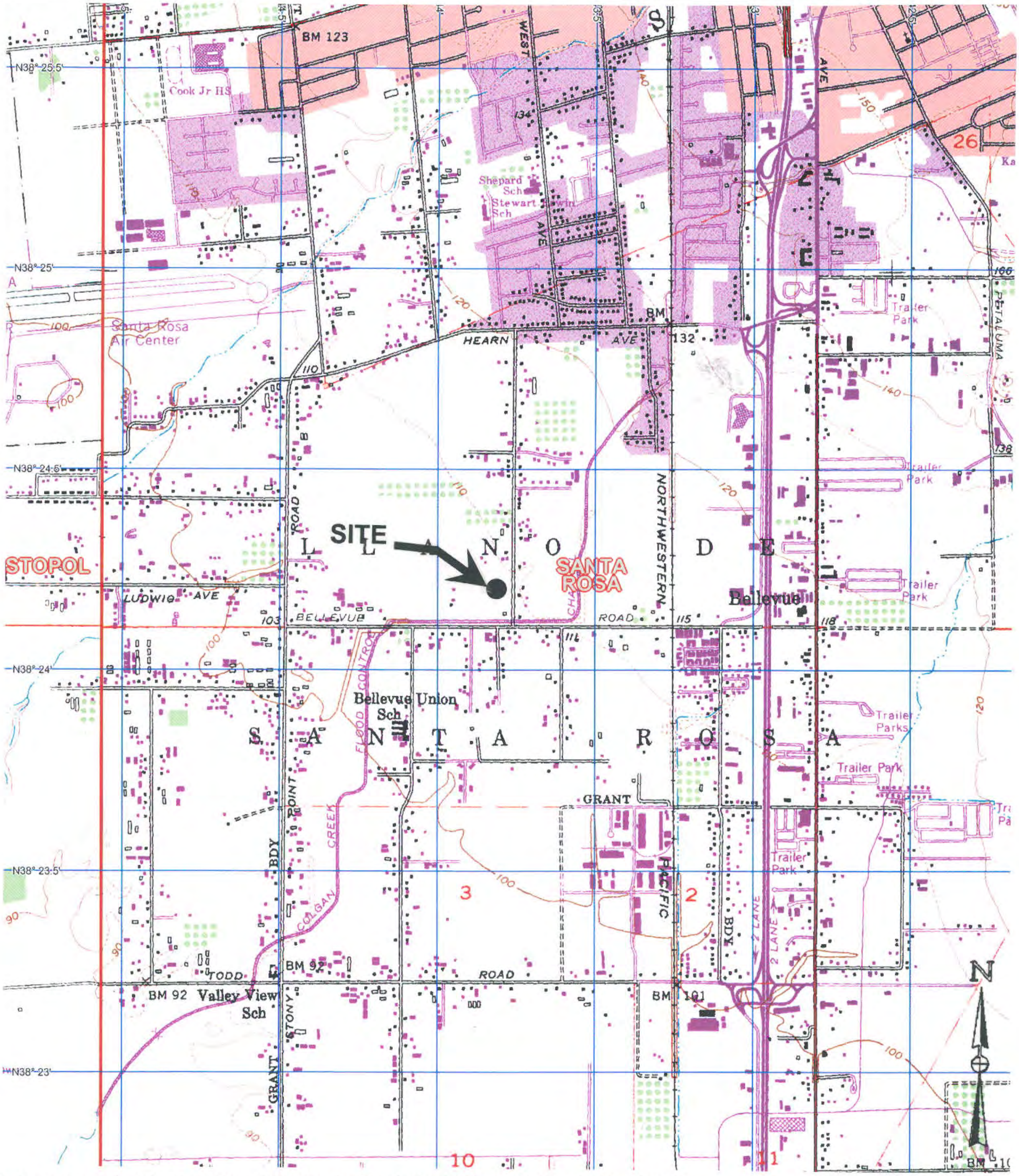
Our services consist of professional opinions and conclusions developed in accordance with generally accepted geotechnical engineering principles and practices. We provide no other warranty, either expressed or implied. Our conclusions and recommendations are based on the information provided to us regarding the proposed construction, the results of our field exploration, laboratory testing program, and professional judgment. Verification of our conclusions and recommendations is subject to our review of the project plans and specifications, and our observation of construction.

The test borings represent subsurface conditions at the locations and on the date indicated. It is not warranted that they are representative of such conditions elsewhere or at other times. Site conditions and cultural features described in the text of this report are those existing at the time of our field exploration on December 9, 2003, and may not necessarily be the same or comparable at other times.

The scope of our services did not include an environmental assessment or a study of the presence or absence of hazardous, toxic or corrosive materials in the soil, surface water, groundwater or air (on, below or around this site), nor did it include an evaluation or study for the presence or absence of wetlands.

APPENDIX A - PLATES

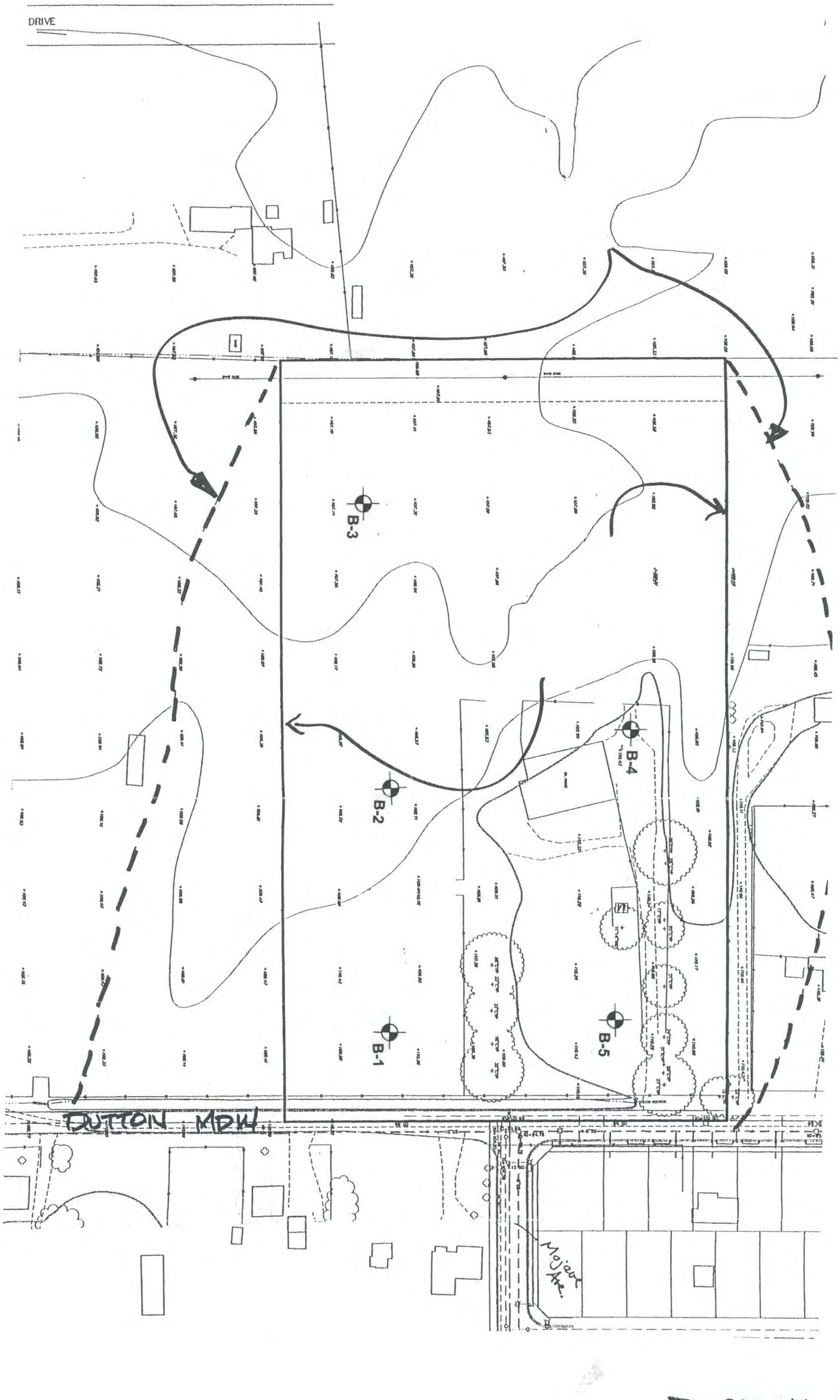
LIST OF PLATES



Reference: U.S.G.S. TopoQuad 7½-Minute Quad, Santa Rosa, California Quadrangle

Scale: 1" = 2000'

R G H Geotechnical and Environmental Consultants	Job No: 1067.19.04.1 Appr: JW Drwn: jj Date: January 2004	**SITE LOCATION MAP** **DUTTON PLACE** **2975 DUTTON MEADOW** Santa Rosa, California	PLATE **1**



R G H Geotechnical and Environmental Consultants	Job No: 1067.19.04.1 Appr: JAW Drwn: jj Date: February 2004	**EXPLORATION PLAN** **DUTTON PLACE** **2975 DUTTON MEADOW** Santa Rosa, California	PLATE **2**

EQUIPMENT: **Truck Mounted: 6" Solid Stem Auger**

LOGGED BY: **TLJ** START DATE: **12-9-03**

ELEVATION: ** FINISH DATE: **12-9-03**

Other Laboratory Tests	Dry Density (pcf)	Moisture Content (%)	% Passing #200 Sieve	Blows/foot*	DEPTH (FEET)	Description
Corrosivity (see Plate 12)				17	0	BLACK SANDY CLAY (CL), very stiff, wet (Fill).
	100	21.1		22	2	BLACK SANDY CLAY (CH), very stiff, wet.
					4	
	103	19.5		31		becomes hard
					6	LIGHT BROWN SANDY CLAY (CL), hard, wet.
					8	
						LIGHT BROWN CLAYEY SAND (SC), medium dense, wet.
	98	24.5		16	10	
					12	BLACK SANDY CLAY (CL), stiff, wet.
						BROWN SAND WITH GRAVEL (SP), very dense, wet.
					14	
				50+		

Bottom of Boring.
No free groundwater encountered.

* Converted to equivalent standard penetration blow counts.
** Existing ground surface.

RGH Geotechnical and Environmental Consultants

Job No: 1067.19.04.1
Appr: JAJ
Drwn: jj
Date: FEB 2004

LOG OF BORING 1

Dutton Place
2975 Dutton Meadow
Santa Rosa, California

PLATE 3

EQUIPMENT: **Truck Mounted: 6" Solid Stem Auger**

LOGGED BY: **TLJ** START DATE: **12-9-03**

ELEVATION: ** FINISH DATE: **12-9-03**

Other Laboratory Tests	Dry Density (pcf)	Moisture Content (%)	% Passing #200 Sieve	Blows/foot*	Sample	DEPTH (FEET)	Description
						0	BLACK CLAY WITH SAND (CH), stiff, wet, porous (Topsoil).
Corrosivity (see Plate 12)				14		2	
							BROWN SANDY CLAY WITH GRAVEL (CL), very stiff, wet.
				28		4	
				50+			BROWN GRAVELLY SAND (SP), very dense, wet.
						6	
						8	
							Increased amount of fines, dense.
				48			
						10	BLACK CLAYEY SAND (SC), dense, wet.
						12	
				50+			
							Bottom of Boring.

* Converted to equivalent standard penetration blow counts.

** Existing ground surface.

RGH Geotechnical and Environmental Consultants

Job No: 1067.19.04.1

Appr: JAW

Drwn: jj

Date: FEB 2004

LOG OF BORING 2

Dutton Place

2975 Dutton Meadow

Santa Rosa, California

PLATE **4**

Other Laboratory Tests	Dry Density (pcf)	Moisture Content (%)	% Passing #200 Sieve	Blows/foot*	Sample	DEPTH (FEET)
				14		0
						2
				22		
						4
				25		
						6
						8
				50+		10
						12
				50+		14

EQUIPMENT: **Truck Mounted: 6" Solid Stem Auger**

LOGGED BY: **TLJ** START DATE: **12-9-03**

ELEVATION: ** FINISH DATE: **12-9-03**

BLACK CLAY WITH SAND (CL), stiff, wet, porous to 3 feet.

very stiff

LIGHT BROWN SANDY CLAY (CH), very stiff, wet.

BROWN SAND WITH GRAVEL (SP), very dense, wet.

Bottom of Boring.

* Converted to equivalent standard penetration blow counts.

** Existing ground surface.

RGH Geotechnical and Environmental Consultants

Job No: 1067.19.04.1

Appr: JAJ

Drwn: jj

Date: FEB 2004

LOG OF BORING 3

Dutton Place

2975 Dutton Meadow

Santa Rosa, California

PLATE

5

EQUIPMENT: **Truck Mounted: 6" Solid Stem Auger**

LOGGED BY: **TW** START DATE: **12-9-03**

ELEVATION: ** FINISH DATE: **12-9-03**

Other Laboratory Tests	Dry Density (pcf)	Moisture Content (%)	% Passing #200 Sieve	Blows/foot*	Sample	DEPTH (FEET)	Description
						0	BROWN CLAYEY SAND (SC), medium dense, wet, with concrete chunks (Fill).
Corrosivity (see Plate 12)				30			
						2	BLACK CLAY (CH), very stiff, moist.
						4	
Classification (see Plate 9)		16.4		42			becomes hard
							OLIVE BROWN CLAY WITH SAND (CL), hard, very moist.
						6	
Classification (see Plate 9)		22.7		31			
						8	
							BROWN SAND WITH CLAY AND GRAVEL (SP), very dense, wet.
				50+		10	
						12	
				50+			
							Bottom of Boring.

* Converted to equivalent standard penetration blow counts.

** Existing ground surface.

RGH Geotechnical and Environmental Consultants

Job No: 1067.19.04.1

Appr: TW

Drwn: jj

Date: FEB 2004

LOG OF BORING 4

Dutton Place
2975 Dutton Meadow
Santa Rosa, California

PLATE 6

EQUIPMENT: **Truck Mounted: 6" Solid Stem Auger**

LOGGED BY: **TW** START DATE: **12-9-03**

ELEVATION: ** FINISH DATE: **12-9-03**

Other Laboratory Tests	Dry Density (pcf)	Moisture Content (%)	% Passing #200 Sieve	Blows/foot*	Sample	DEPTH (FEET)	Description
						0	DARK BROWN SANDY CLAY (CL), stiff, wet, porous (Topsoil).
Classification (see Plate 9)		19.7		10			
Classification (see Plate 9)				20		2	BLACK CLAY (CH), very stiff, moist.
		16.7					
						4	
				50+			LIGHT BROWN CLAYEY SAND (SC), very dense, moist.
						6	
				50+		8	
						10	
							BROWN CLAY WITH SAND (SP), hard, wet.
				50+		12	
							Bottom of Boring.

* Converted to equivalent standard penetration blow counts.

** Existing ground surface.

RGH Geotechnical and Environmental Consultants	Job No: 1067.19.04.1 Appr: JW Drwn: jj Date: FEB 2004	**LOG OF BORING 5** Dutton Place 2975 Dutton Meadow Santa Rosa, California	PLATE 7

MAJOR DIVISIONS				TYPICAL NAMES
COARSE GRAINED SOILS More Than Half is Larger Than #200 Sieve	GRAVELS More Than Half Coarse Fraction Is Larger Than No. 4 Sieve Size	Clean Gravels With Little or No Fines	GW	Well Graded Gravels, Gravel - Sand Mixtures
			GP	Poorly Graded Gravels, Gravel - Sand Mixtures
		Gravels With Over 12% Fines	GM	Silty Gravels, Poorly Graded Gravel - Silt - Silt Mixtures
			GC	Clayey Gravels, Poorly Graded Gravel - Sand - Clay Mixtures
	SANDS More Than Half Coarse Fraction Is Smaller Than No. 4 Sieve Size	Clean Sands With Little or No Fines	SW	Well Graded Sands, Gravelly Sands
			SP	Poorly Graded Sands, Gravelly Sands
		Sands With Over 12% Fines	SM	Silty Sands, Poorly Graded Sand - Clay Mixtures
			SC	Clayey Sands, Poorly Graded Sand - Clay Mixtures
FINE GRAINED SOILS More Than Half is Smaller Than #200 Sieve	SILTS AND CLAYS Liquid Limit 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
			OL	Organic Clays and Organic Silty Clays of Low Plasticity
	SILTS AND CLAYS Liquid Limit Greater Than 50		MH	Organic Silts, Micaceous or Diatomacious 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

UNIFIED SOIL CLASSIFICATION SYSTEM

Shear Strength, psf

Confining Pressure, psf

Consol - Consolidation
LL - Liquid Limit (In %)
PL - Plastic Limit (In %)
G_s - Specific Gravity
SA - Sieve Analysis
■ - "Undisturbed" Samples
☒ - Bulk or Disturbed Sample
◩ Standard Penetration Test
□ - Sample Attempt With No Recovery

Tx	320	(2600)	Unconsolidated Undrained Triaxial
Tx CU	320	(2600)	Consolidated Undrained Triaxial
DS	2750	(2000)	Consolidated Drained Direct Shear
FVS	470		Field Vane Shear
UC	2000		Unconfined Compression
LVS	700		Laboratory Vane Shear
SS	- Shrink Swell		
EXP	- Expansion		
P	- Permeability		

Note: All strength tests on 2.8" or 2.4" diameter sample unless otherwise indicated

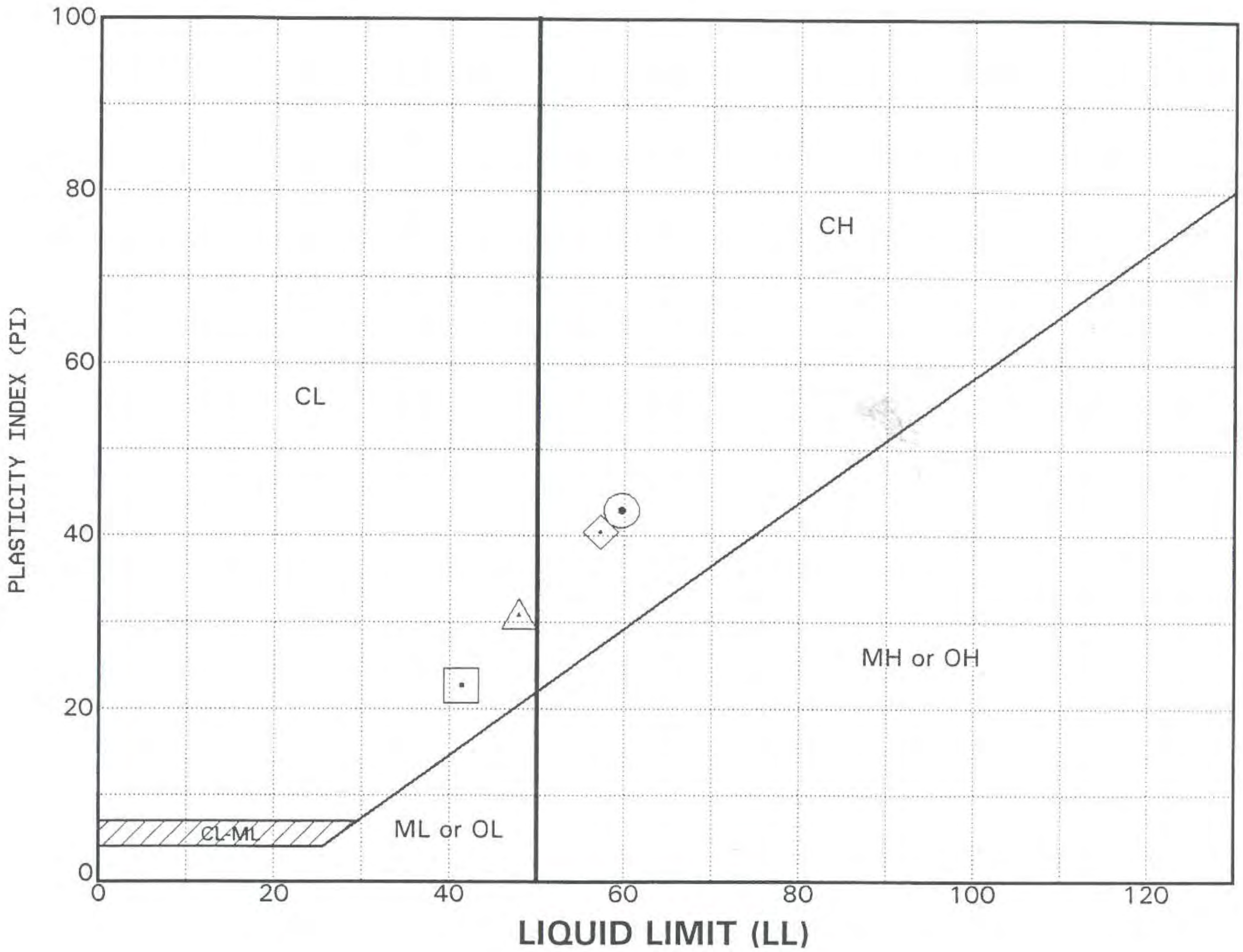
KEY TO TEST DATA

RGH
Geotechnical and Environmental Consultants

Job No: 1067.19.04.1
Appr: JW
Drwn: jj
Date: January 2004

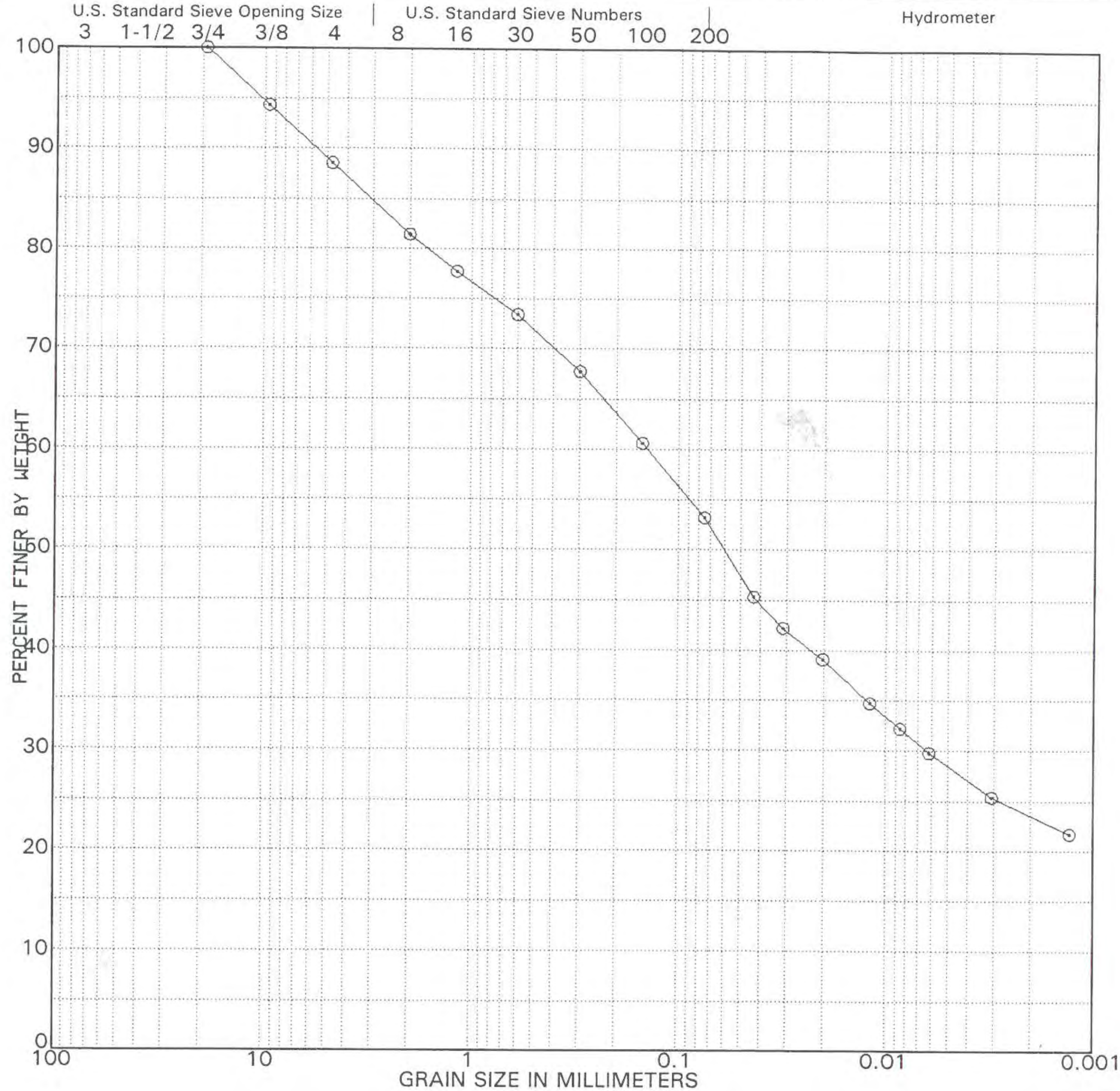
SOIL CLASSIFICATION CHART AND KEY TO TEST DATA
DUTTON PLACE
2975 DUTTON MEADOW
Santa Rosa, California

PLATE
8



SAMPLE SOURCE	CLASSIFICATION	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	% PASSING #200 SIEVE	UBC EXPANSION
⊙ B-4 @ 4.5'	Black Fat Clay (CH)	60	43		
⊡ B-4 @ 7.0'	Olive Lean Clay W/Sand (CL)	41	22		
△ B-5 @ 0.5'	Dark Brown Sandy Lean Clay (CL)	48	31	53	89
◇ B-5 @ 2.5'	Black Fat Clay (CH)	57	40		

RGH Geotechnical and Environmental Consultants	Job No: 1067.19.04.1 Appr: JAW Drwn: jj Date: January 2004	**CLASSIFICATION TEST DATA** **DUTTON PLACE** **2975 DUTTON MEADOW** Santa Rosa, California	PLATE **9**



Cobbles	GRAVEL		SAND			SILT	CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE		

SYMBOL	SAMPLE SOURCE	CLASSIFICATION
⊙	B-5 @ 0.5'	Dark Brown Sandy Lean Clay (CL)

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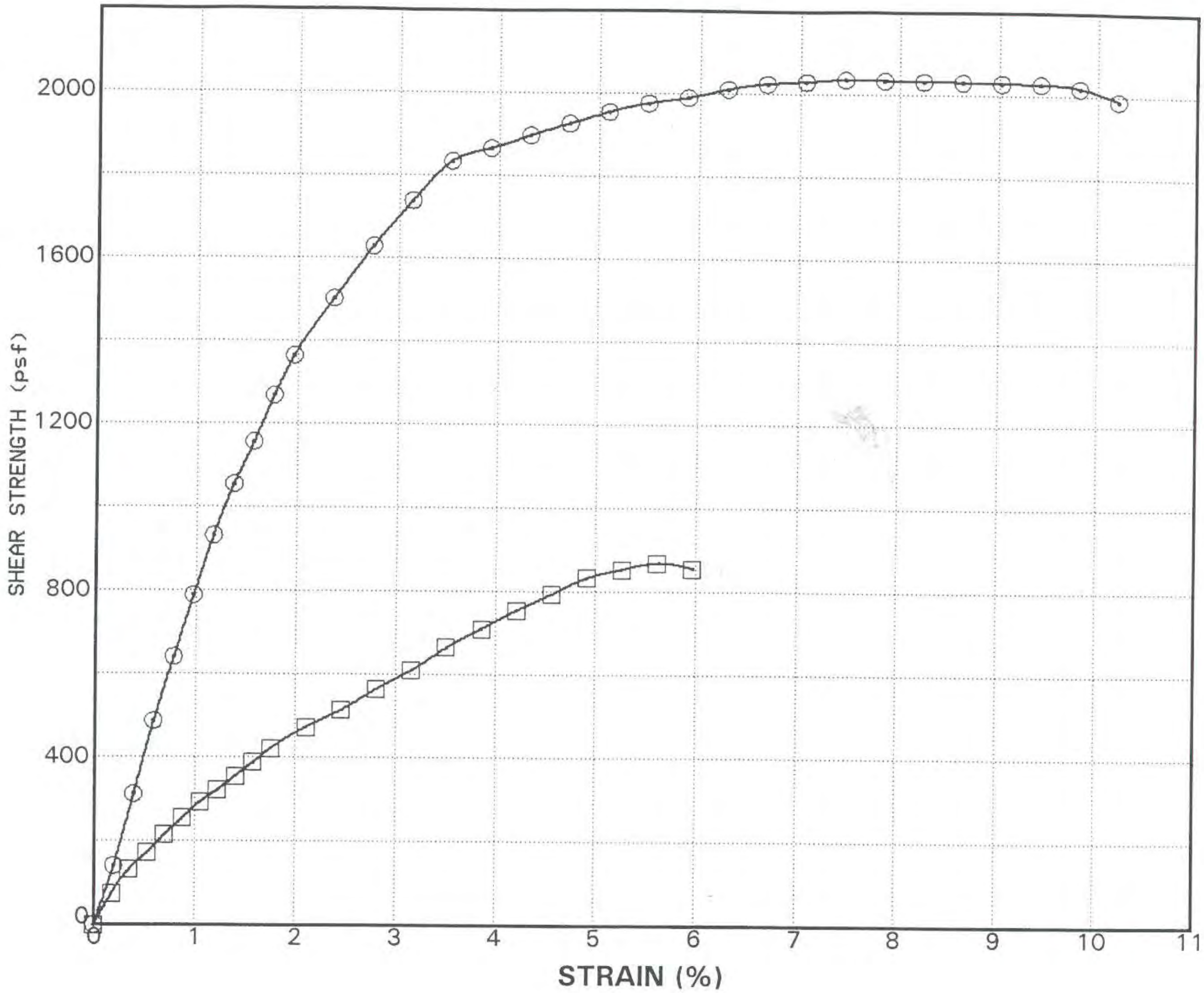
Job No: 1067.19.04.1
Appr: JAW
Drwn: jj
Date: January 2004

PARTICLE SIZE ANALYSIS
DUTTON PLACE
2975 DUTTON MEADOW
Santa Rosa, California

PLATE
10



Sample Source	Classification	Type of Test	Confinement Pressure (psf)	Ultimate Strength (psf)	Strain (%)	Dry Density (pcf)	Moisture Content (%)
⊙ B-1 @ 2.3'	Brown Sandy Fat Clay (CH)	UC		2034	7	100	21.1
⊡ B-1 @ 9.5'	Light Brown Clayey Sand (SC)	UC		868	6	98	24.5

RGH

Geotechnical and Environmental Consultants

Job No: 1067.19.04.1

Appr: JWJ

Drwn: jj

Date: January 2004

STRENGTH TEST DATA
DUTTON PLACE
2975 DUTTON MEADOW
Santa Rosa, California

PLATE
11

ETS — **Environmental Technical Services**

1343 Redwood Way
Petaluma, CA 94954
(707) 795-9605 / FAX 795-9384

-Soil, Water & Air Testing & Monitoring
-Analytical Labs
-Technical Support

Serving people and the environment so that both benefit.

COMPANY: RGH Geotech, 1305 N. Dutton Avenue, Santa Rosa, CA 95401	DATE RECEIVED	DATE of COMPLETION	ANALYST(S)	SUPERVISOR
ATTN: Eric Chase JOB SITE: Dutton Place, Santa Rosa, California JOB #: 1067.19.4.1	12/10/03	12/17/03	W. Zuo G. Hundt	D. Jacobson LAB DIRECTOR G.S. Conrad PhD

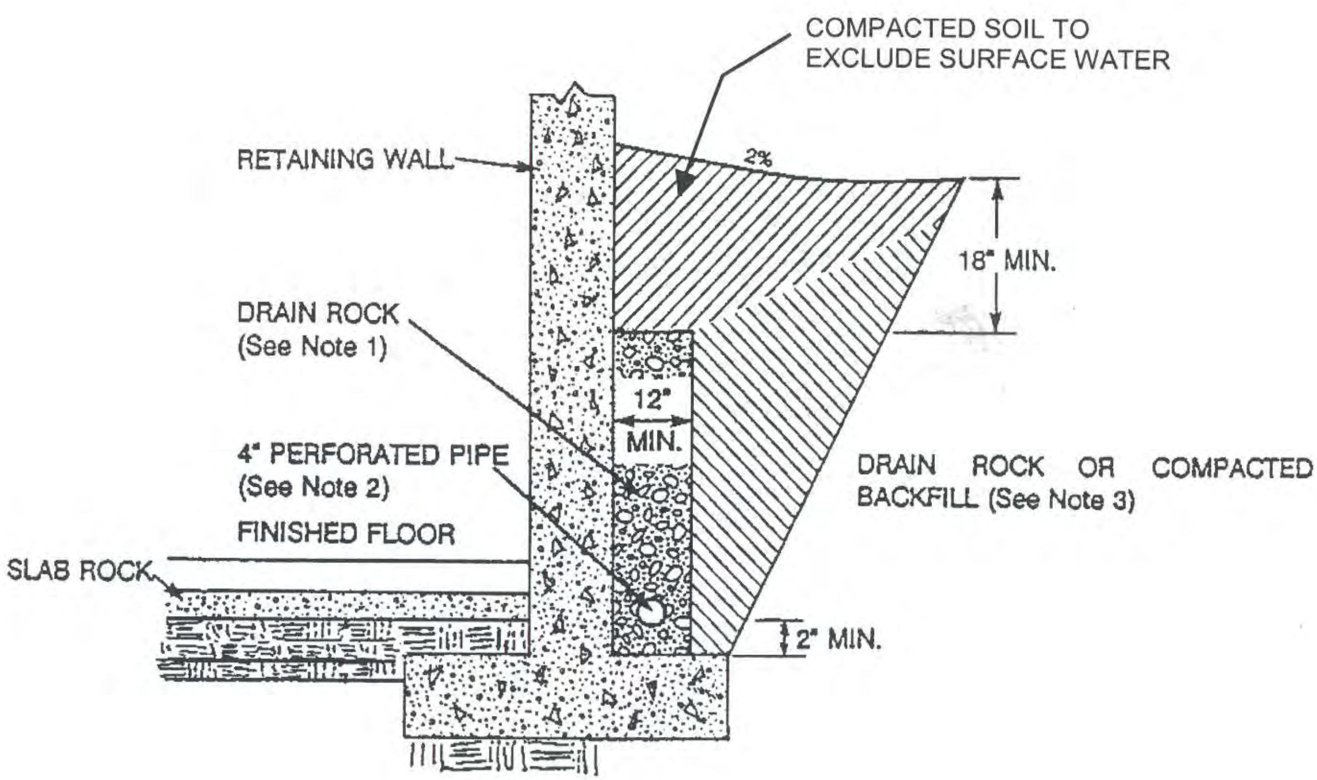
LAB SAMPLE NUMBER	SAMPLE ID	DESCRIPTION of SOIL and/or SEDIMENT	SOIL pH -log[H+]	NOMINAL RESISTIVITY ohm-cm	ELECTRICAL CONDUCTIVITY µmhos/cm	SULFATE SO4 ppm	CHLORIDE Cl ppm
00357-1	DP1/SR	B-1 @ 0.5'	5.46	1320	[760]	198	60
00357-2	DP2/SR	B-2 @ 1.5'	5.49	1210	[825]	195	72
00357-3	DP3/SR	B-4 @ 1.5'	7.24	1520	[606]	246	120
Method	Detection	Limits —>	—	1	0.1	1	1

LAB SAMPLE NUMBER	SAMPLE ID	DESCRIPTION of SOIL and/or SEDIMENT	SALINITY ECe mmhos/cm	SOLUBLE SULFIDES (S=) ppm	SOLUBLE CYANIDES (CN=) ppm	REDOX mV	PERCENT MOISTURE %
00357-1	DP1/SR	B-1 @ 0.5'				+753.0	
00357-2	DP2/SR	B-2 @ 1.5'				+776.0	
00357-3	DP3/SR	B-4 @ 1.5'				+728.2	
Method	Detection	Limits —>	—	0.1	0.1	1	0.1

****************************** COMMENTS ******************************

Resistivities are over 1000 ohm/cm which is fine, but two soil reactions (i.e., pHs) are moderately acidic which does not help; sulfates are mildly elevated, but chlorides are mostly low. The CalTrans times to perforation for these soils are as follows: for DP1 and18 ga steel the time is a very poor 8.4 yrs., and for 12 ga it is 18.5 yrs; for DP2 the respective times are 8.0 yrs & 17.6 yrs; but for DP3 times are much better at 26.0 yrs & 57.2 yrs. Sulfates are not high enough that they should have a direct adverse impact on concrete, grout mortar or cement; and chlorides are safely below the limit where they can adversely impact contained steel reinforcement. All three have very high redoxes (i.e., >+700 mV) which is very good. These soils are good candidates for lime treating as concerns exposed steel. Lime treatment of these soils to pH 7.5-8.5 is calculated to improve 18 ga times to the 25-30 years for DP1 & DP2; and to ≈30 years for DP3. For DP1 & DP2 this is greater improvement than upgrading to 12 ga steel alone, but for DP3 it would be somewhat less than the upgrade option. Obviously, some combination of upgrading exposed steel and lime treating would result in even greater improvements; &/or other options may be desireable. Last, as concerns these corrosivity results the standard concrete mix should be sufficient in these soils, even in the low pH materials as long as pH remains @ ≥5; (and assuming that if there is to be any slab coating [e.g. vinyl, epoxy, etc.], there will be a strong, tough vapor barrier).

\\\\NOTES: Methods are from following sources: extractions by Cal Trans protocols as per Cal Test 417 (SO4), 422 (Cl), and 532/643 (pH & resistivity); &/or by ASTM Vol. 4.08 & ASTM Vol. 11.01 (=EPA Meth Chem Anal, or Standard Methods); pH - ASTM G 51; Spec. Cond. - ASTM D 1125; resistivity - ASTM G 57; redox - Pt probe/ISE; sulfate - extraction Title 22, detection ASTM D 516 (=EPA 375.4); chloride - extraction Title 22, detection ASTM D 512 (=EPA 325.3); sulfides - extrac. Title 22, detection EPA 376.2 (=SMEWW 4500-S D); cyanides - extraction Title 22, detection ASTM D 4374 (=EPA 335.2).



Notes:

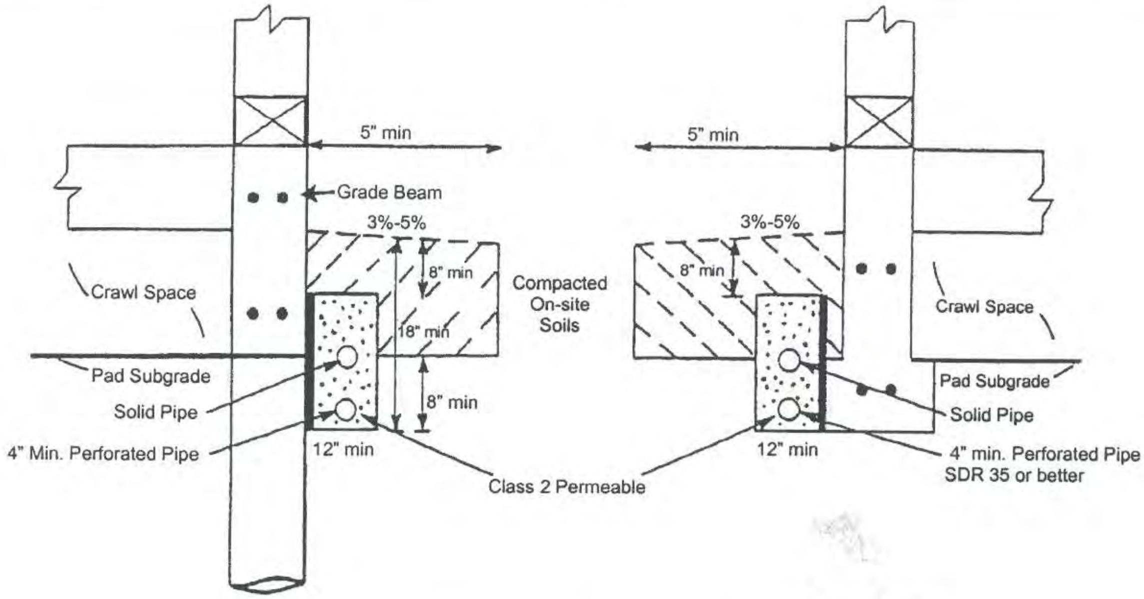
1. Drain rock should meet the requirements for Class 2 Permeable Material, Section 68, State of California "Caltrans" Standard Specification, latest edition. Drain rock should be placed to approximately three-quarters the height of the retaining wall.

2. Pipe should conform to the requirements of Section 68 of State of California "Caltrans" Standards, perforations placed down, sloped at 1% for gravity flow to outlet or sump with automatic pump. The pipe invert should be located at least 8 inches below the finished grade.

3. During compaction the contractor should use appropriate methods such as temporary bracing and/or light compaction equipment to avoid overstressing the walls.

Not to Scale

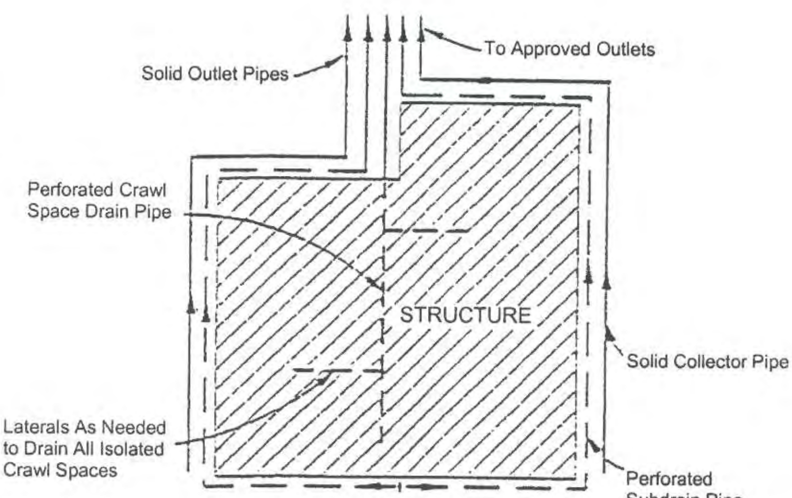
R G H Geotechnical and Environmental Consultants	Job No: 1067.19.04.1 Appr: JAW Drwn: jj Date: January 2004	**RETAINING WALL BACKDRAIN ILLUSTRATION** **DUTTON PLACE** **2975 DUTTON MEADOW** Santa Rosa, California	PLATE **13**



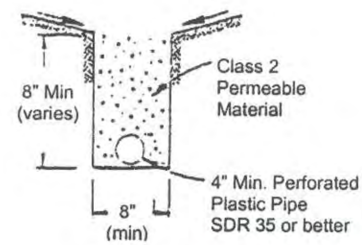
Pier and Grade Beam

Spread Footing

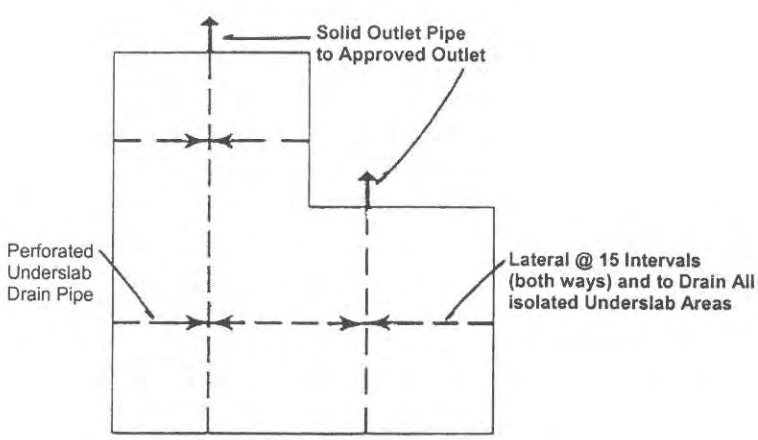
PERIMETER FOUNDATION DRAINS



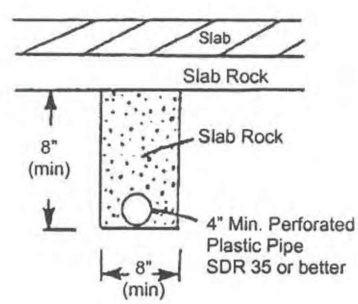
TYPICAL FOUNDATION DRAIN PLAN



CRAWL SPACE DRAIN



TYPICAL UNDERSLAB DRAIN PLAN



SLAB UNDERDRAIN

Not to Scale

R G H Geotechnical and Environmental Consultants	Job No: 1067.19.04.1 Appr: JAW Drwn: jj Date: January 2004	**TYPICAL SUBDRAIN DETAILS ILLUSTRATION** **DUTTON PLACE** **2975 DUTTON MEADOW** Santa Rosa, California	PLATE **14**

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APPENDIX C - DISTRIBUTION

Cobblestone Homes (5,1)
Attn: Frank Denney
1400 North Dutton Avenue, Suite 1
Santa Rosa, California 95401

TW:EGC:GWR:tw:jj(10671941.Rpt)

RGH CONSULTANTS
Experience is the difference

Santa Rosa Office
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Santa Rosa, CA 95401
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Napa Office
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Napa, CA 94559
P: 707-252-8105
F: 707-544-1082

Middletown Office
P.O. Box 852
Middletown, CA 95461
P: 707-987-4602
F: 707-987-4603

April 30, 2018

Burbank Housing Development Corporation
Attention: Karen Massey
790 Sonoma Avenue
Santa Rosa, CA 95404
kmassey@burbankhousing.org

Geotechnical Engineering Report Update
Lantana Homes
2979 Dutton Meadow
Santa Rosa, California

Project Number: 1259.08.12.1

Introduction

We have reviewed the geotechnical aspects of the project plans prepared by Tierney/Figueiredo for the subject project. Those plans indicate that the site will be developed with 48 single family homes constructed duet style where the units are attached at the garages. The homes will include 45 two-story and three single story wood-framed units. The project will include extending Mojave Avenue to the west off Dutton Meadow and constructing a loop road south off Mojave Avenue and Common Way at west end of Mojave Avenue. The homes will be accessed by driveways off Dutton Meadow, Mojave Avenue, Common Way and the new loop street. The results of our geotechnical study for a previous project planned at the site were presented in a report dated February 26, 2004 (attached). That report addressed a project that included construction of 102 residential units that were to be contained within two- and three-story buildings.

On April 26, 2018, we performed a brief reconnaissance of the site. The barn and stacks of supplies and debris noted in our report were no longer present, but the remainder of the site surface conditions have not changed significantly since our report was issued.

Conclusions and Recommendations

Based on our review and reconnaissance, it is our opinion that the recommendations in our report, with the updated criteria presented below, are valid for design and construction of the improvements.

Seismic Design

Seismic design parameters presented below are based on Section 1613 titled “Earthquake Loads” of the 2016 California Building Code (CBC). Based on Table 20.3-1 of American Society of Civil Engineers (ASCE) Standard 7-10, titled “Minimum Design Loads for Buildings and Other Structures” (2010), we have determined a Site Class of D should be used for the site. Using a site latitude and longitude of 38.4033°N and 122.7309°W, respectively, and the U.S. Seismic Design Maps from the United States Geological Survey (USGS) website (http://earthquake.usgs.gov/designmaps/us/application.php), we recommend that the following seismic design criteria be used for structures at the site.

2016 CBC Seismic Criteria	
Spectral Response Parameter	**Acceleration (g)**
S_S (0.2 second period)	1.781
S_1 (1 second period)	0.709
S_{MS} (0.2 second period)	1.781
S_{M1} (1 second period)	1.064
S_{DS} (0.2 second period)	1.187
S_{D1} (1 second period)	0.709

Post-Tensioned Slabs

A post tension (PT) slab should be a designed to accommodate edge moisture variation distances of 4.9 and 7.5 feet for edge and center lift conditions, respectively, a differential edge swell of 0.8 inch and a center swell of 1.1 inch. These parameters were developed using the Post-Tensioning Institute manual “Design and Construction of Post-Tensioned Slabs-On-Ground, Third Edition” (2004). A PT slab installed in accordance with the foregoing recommendations may be designed using allowable bearing pressures of 1,700, 2,500 and 3,300 pounds per square foot (psf) for dead loads, dead plus code live loads, and total loads, including wind and seismic, respectively. We recommend a minimum slab thickness of 10 inches and a 12-inch-wide (minimum) perimeter thickened edge. Concentrated loads in the slab interior should also be supported by thickened beams within the slab.

The PT slab should be underlain with a capillary moisture break consisting of at least 4 inches of clean, free-draining crushed rock or gravel (excluding pea gravel) at least ¼-inch and no larger than ¾-inch in size. The subgrade soil within and for a distance of 5 feet beyond the footprint of the building(s) should be kept pre-swelled until the capillary moisture break is placed. The moisture content of the subgrade soil should be approved by the geotechnical engineer within 24 hours prior to placing the capillary moisture break. Where migration of moisture vapor through slabs would be detrimental, a moisture vapor barrier should be provided.

Because PT slabs are designed to move with the expansive soil as it shrinks and swells, structural elements that are attached to the structure, such as porch overhangs, but have their own foundation should not be used or should be founded on the PT slab. Exterior flatwork and concrete walkway subgrades should be underlain by at least 12 inches of select fill and be pre-swelled by soaking prior to installation of the walkway. In addition, concrete walkways should be:

1. Cast separate from the PT slab to allow differential settlement to occur without distressing the walkway;

2. Reinforced to reduce cracks; and

3. Grooved to induce cracking in a non-obtrusive manner.

The Post-Tensioning Institute states "Consideration should be given to 'artificial' effects, such as planter units adjacent to structural bearing areas. Tree roots can be a serious problem and cause volume reduction in limited areas, thus causing distress to the slab foundation. Trees that are planted closer to the foundation than half their ultimate height can be expected to cause significant differential movement."

The recommendations presented herein are subject to the limitations set forth in our referenced report. We trust this provides the information you require at this time. If you have questions please call.

Very truly yours,
RGH Consultants

REGISTERED PROFESSIONAL ENGINEER
ERIC G. CHASE
No. 2628
GEOTECHNICAL
STATE OF CALIFORNIA

Eric G. Chase
Project Manager

EGC:JJP:ec:ejw
Electronically submitted

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Attachment: Geotechnical Study, Dutton Place, 2975 Dutton Meadow, APN 043-121-006, Santa Rosa, California, RGH Project No. 1067.19.04.1, dated February 26, 2004