GEOTECHNICAL EVALUATION
GRADY RANCH PRECISE DEVELOPMENT PLAN
MARIN COUNTY, CALIFORNIA

Submitted to:

CSW/Stuber-Stroeh Engineering Group, Inc.,
Novato, California

Submitted by:

AMEC Geomatrix, Inc., Oakland, California

November 2008

Project 14648.000

AMEC Geomatrix

November 21, 2008

Project 14648.000

Ms. Emily Dean
CSW/Stuber-Stroeh Engineering Group, Inc.
45 Leveroni Court
Novato, California 94949

Subject: **Geotechnical Evaluation**
Grady Ranch Precise Development Plan
Marin County, California

Dear Ms. Dean:

AMEC Geomatrix, Inc. (AMEC Geomatrix) is pleased to submit this geotechnical evaluation report for the Precise Development Plan (PDP) for the Grady Ranch Development in Marin County, California. This report was developed in accordance with our revised proposal dated October 3, 2008 and our Professional Services Agreement with the CSW/Stuber-Stroeh Engineering Group, Inc., formally executed November 13, 2008. Our evaluation included compiling and reviewing existing data, performing a field reconnaissance, performing engineering evaluations, developing preliminary geotechnical recommendations, and preparing this report.

If you have any questions about this report, please do not hesitate to call any of the undersigned. It has been a pleasure working with you and we look forward to working with you on other future phases of the project.

Sincerely yours,
AMEC Geomatrix, Inc.

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1.0 INTRODUCTION

This report presents the results of the geological and geotechnical evaluation that AMEC Geomatrix, Inc. (AMEC) performed to support the Precise Development Plan (PDP) for the proposed Grady Ranch Development located in Marin County, California. The location of the project is shown on the attached Site Vicinity Map, Figure 1, and a site plan is presented in Drawing C1.1.

1.1 PURPOSE AND SCOPE

The purpose of this study was to provide a preliminary evaluation of the suitability of the site for the proposed development from a geotechnical engineering standpoint. Our scope of work to accomplish the stated purpose has included the following tasks:

1. Data Review: We compiled and reviewed available published and unpublished information and reports relevant to the geologic and geotechnical conditions at the Grady Ranch site.

2. Field Reconnaissance: We evaluated general geotechnical and geologic conditions at the Grady Ranch, and performed a supplemental field reconnaissance including geologic mapping to examine surface conditions or geotechnical/geologic features that may affect the design of the development.

3. Geotechnical Engineering Analyses and Reporting: We performed a preliminary geotechnical evaluation to develop preliminary geotechnical recommendation for the project, addressing slope stability, foundations, grading, retaining structures, and seismic considerations. We have prepared this report presenting the results of our evaluation and our preliminary recommendations.

Our scope of services to accomplish the above-state purposes was outlined in our revised proposal dated October 3, 2008.

The recommendations made in this report are based on the assumption that soil and groundwater conditions do not deviate appreciably from those disclosed in the exploratory borings drilled at this site. If any variations or undesirable conditions are encountered during future exploration or construction, the effects of these conditions on the recommendations presented herein should be evaluated and, if necessary, supplemental recommendations developed. The recommendations are made for the proposed Grady Ranch Project described

in this report. Significant changes in location, type, or embedment of the structures, or loading conditions should be evaluated as to their effects on the recommendations.

This report is preliminary in nature and is not intended to provide all of the subsurface information that will be needed by a contractor to construct the project. Additional subsurface exploration, laboratory testing, and engineering analyses will be necessary to develop final recommendations.

In the performance of our professional services, AMEC, its employees, and its agents comply with the standards of care and skill ordinarily exercised by members of our profession practicing in the same or similar localities. No other warranty, either expressed or implied, is made or intended in connection with the work performed by us, or by the proposal for consulting or other services, or by the furnishing of oral or written reports or findings. We are responsible for the conclusions and recommendations contained in this report, which are based on data related only to the specific project and locations discussed herein. In the event conclusions or recommendations based on these data are made by others, such conclusions and recommendations are not our responsibility unless we review and concur with such conclusions or recommendations in writing.

1.2 REPORT ORGANIZATION

A brief project description is presented in Section 2.0. Section 3.0 discusses the site evaluation methods performed for this study. A general description of the site conditions is provided in Section 4.0, and Section 5.0 discusses our evaluations and conclusions. Section 6.0 provides geotechnical recommendations for preliminary design. Section 7.0 presents the references.

This report includes an appendix that presents the logs of borings and test pits from previous investigations.

2.0 PROJECT DESCRIPTION

The site is currently undeveloped and is located immediately north of Lucas Valley Road. The location of the site is shown on the Site Vicinity Map (Figure 1). The major components of the project are shown on Drawing C1.1.

It is our understanding that the PDP is focusing on development of the Main Building, and that other buildings may be developed during future phases. The Main Building will include two large rooms referred to as Stage A and Stage B. For structural purposes, the Main Building, Stage A, and Stage B will be evaluated as three separate buildings and will be referred to as such in this report.

The Main Building is a 3-story, 65-foot tall, steel moment frame structure over the concrete parking level, with an anticipated fundamental period of 1.5 to 2 seconds. Stage A and Stage B are both one-story, 60-foot tall, steel braced frame buildings over the concrete parking level with an anticipated fundamental period of 0.4 to 1.0 seconds. All three structures are situated over a concrete parking garage consisting of a 12-inch flat plate slab supported by 30-inch by 30-inch concrete columns.

The following table provides unfactored column loads for the three distinct buildings that comprise the project.

Unfactored Column Loads		
Location of Column	**Dead (kips)**	**Live (kips)**
Main Bldg. Exterior	240	136
Main Bldg. Interior	522	421
Stage A Exterior	343.5	170
Stage A Interior	687	340
Stage B Exterior	196.5	105.5
Stage B Interior	393	211

In addition to the buildings, the project includes:

1. Access roads to the three buildings and to the west toward future areas of development
2. Site grading and retaining walls associated with the access roads and buildings
3. An entrance kiosk
4. Eight bridges
5. A wine cave

Additional elements in future development phases are not addressed in this proposal.

We understand that a Master Plan for the development was performed in 1993 and included results of geologic/geotechnical investigations by Harlan Miller Tait Associates (HMTA 1988) and Harlan Tait Associates (HTA 1993). These investigations included geologic mapping and field exploration consisting of exploratory borings and test pits. The work was performed by or under the direction of Dr. Robert Wright, Certified Engineering Geologist, who is now with AMEC. We have reviewed the original HMTA and HTA reports for information relevant to the project. Based on our review of the geologic map, select sheets from the Master Plan, and the previous boring and test pit logs, we understand that the building site is underlain by sandstone, shale, and mélange bedrock belonging to the Franciscan Complex. The surface soils overlying the bedrock consist of Quaternary deposits of colluvium and alluvium. Multiple landslides, identified as both dormant and active, exist throughout the Grady Ranch, which is similar to many other hillside areas of the San Francisco Bay Area. It is anticipated that any landslides that impinge upon the development will need to be stabilized as part of the project.

Miller Creek is located along the southern portion of the site and flows from west to east. A 50-foot setback between the top of the creek bank and the main building will be required. A 100-foot setback will be required for structures, roads, grading, and utilities in all other locations on site.

Based on a review of the preliminary floor plans and the preliminary grading plan developed by Urban Design Group (UDG), we understand that the main building will be constructed in the location of a small spur ridge that is flanked by two existing small drainage ravines, and it may intersect the footprint of several landslides. We anticipate that the cuts required for the construction of the main building will result in removal of some of these landslides, and the removal of only the lower portions of others of these landslides. Preliminary grading plans (see Drawing C1.1) indicate that excavations will be up to about 60 feet deep into the spur ridge. Proposed retaining walls are expected to have a maximum height of about 35 feet.

Earthwork at the site will include significant excavations and fills. It is our understanding that export of cut soils will minimized. Excavations will be made to develop the building pad, and the excavated material will be placed largely in a fill area along a spur ridge near the eastern side of the property, as shown on Drawing C1.1. Maximum fill thickness will be about 35 feet. Conceptual cross sections for fill placement and landslide repairs are shown on Drawing C1.2.

We understand that preliminary geotechnical recommendations in support of the development of the PDP are required at this time. Additional geotechnical studies will be required at a future time for the development of the construction documents.

3.0 SITE EVALUATION METHODS

3.1 REVIEW OF AVAILABLE HISTORIC GEOTECHNICAL / GEOLOGIC INVESTIGATIONS

Several engineering studies have been performed for the project site and included geotechnical investigations and geologic investigations (geotechnical investigations). These reports were obtained from other engineering firms who had been involved in past work for the project. We reviewed this information to obtain data relevant to our current study including subsurface information in the immediate vicinity of the project site.

Copies of subsurface information from the prior boring and test pit logs are included in Appendix A.

3.2 REVIEW OF PUBLISHED MATERIALS

A variety of published sources were reviewed to evaluate geotechnical data relevant to the subject parcel. These sources included geotechnical literature, reports, and maps published by various public agencies. Maps which we reviewed included topographic and geologic maps prepared by the United States Geological Survey, as well as geologic and fault maps prepared by the California Geological Survey (formerly the California Division of Mines and Geology). The purpose of this review was to assist with geologic and geotechnical characterization of the project site. Information obtained from our review of published documents is summarized in Section 4.0 of this report. A list of published documents reviewed for this investigation is presented in Section 7.0, References and Bibliography, at the end of this report.

3.3 AERIAL PHOTOGRAPH REVIEW

Six sets of black and white, stereo pair, aerial photographs were reviewed as part of our study. These photographs were taken during the period from 1958 to 2005 and ranged in scale from 1:1000 to 1:36000. A complete listing of all photographs reviewed for our study is included on the following table. The findings from the review of the aerial photographs are incorporated into the relevant portions of Section 4.0.

Photo Numbers	Scale	Date
SF-AREA-01-08 and -09	1:36000	03-01-58
AV-958-02-16 and -17	1:~11000	07-02-70
AV-1187-02-16 and -17	1:12000	04-17-75
AV-2860-09-14 and -15	1:12000	04-19-86
AV-4890-15-47 and -48	1:12000	08-09-95
KAV-9010-10-02, -03, and -04	1:10000	03-06-05

3.4 GEOLOGIC RECONNAISSANCE

On October 9, 2008, Mr. Todd Crampton, Senior Geologist with AMEC, performed a geologic reconnaissance and field mapping of the site and portions of the immediate surrounding properties to evaluate general geotechnical and geological conditions. The findings of our geologic reconnaissance and mapping are described in Section 4.0 below, as well as presented graphically on Drawing C1.1. In general, artificial fill is not mapped unless the fill is estimated to be more than about 5 feet thick.

4.0 SITE CONDITIONS

4.1 REGIONAL GEOLOGY AND SEISMICITY

The property is situated within the Coast Ranges geomorphic province. This province is characterized by northwest trending mountain ranges and intervening valleys controlled by folds and faults that resulted from the collision of the Farallon and North American plates, and subsequent translational shear along the San Andreas fault system. Most of the uplift in the Coast Ranges occurred by middle Miocene time (16 million years ago), with some uplift continuing through the Quaternary (past 2 million years). Bedrock in the region consists primarily of the Franciscan Complex, which also underlies the property. The Franciscan Complex consists of a diverse assemblage of sandstone, shale, greenstone, chert, and mélange, with lesser amounts of conglomerate, serpentine, calc-silicate rock, schist, and other metamorphic rocks. The gross structure of the Franciscan Complex consists of northwest-southeast trending fault-bounded units. A Regional Fault Map is presented as Figure 2. Outcrop structure ranges from sheared, weak materials, to massive, hard rock. Locally, alluvial and colluvial deposits and landslides mantle the bedrock. Figure 3 presents the regional geology developed by the CGS (Rice et al. 2002). A more detailed geologic map of the Grady Ranch property was developed by HMTA and is reproduced as Figure C1.3. This geology was further updated and refined within the Phase 1 building area, as shown on Drawing C1.1, and conceptual remedial approaches are presented on Drawing C1.2.

The property is located within the seismically active San Francisco Bay region, an area dominated by northwest-trending fault zones of the San Andreas Fault system. The San Andreas Fault zone, the closest known active fault zone, is located about 8 miles southwest of the property (Jennings 1994). The probably active Rodgers Creek fault zone is located about 11 miles northeast (Jennings 1994, and Pampeyan 1979). No active faults are known to traverse the property, and fault ground rupture is not considered to be a potential hazard. However, the property is likely to experience strong ground shaking resulting from an earthquake originating on one of the active faults in the region.

The U.S. Geological Survey (USGS) 2007 Working Group on California Earthquake Probabilities (WGCEP, 2008) estimated an approximately 63-percent probability that at least one major earthquake (with a moment magnitude $M_W \geq 6.7$) would occur in the San Francisco Bay Area before 2037.

4.2 LOCAL GEOLOGY

The property is underlain by Franciscan Complex that includes sandstone, shale, and mélange. The Franciscan Complex materials are generally mantled by shallow soils and surficial deposits. The geologic structure, bedrock, and surficial deposits are described below.

4.2.1 Geologic Structure

The gross geologic structure within the property consists of west-northwest-trending, fault-bounded blocks of alternating Franciscan Complex mélange and sandstone units (Figure 3). The mélange unit is characterized by a pervasively sheared shaley matrix enclosing "knockers" of hard, resistant rock. Mélange matrix is visible only in a very few exposure as it is generally mantled by soil and surficial deposits. The sandstone is characterized by generally closely spaced fractures of various orientations. A tendency exists for a platey to blocky joint system.

4.2.2 Bedrock

Bedrock at the site consists of Franciscan Complex melange and sandstone units. The melange unit consists of a mixture of rock types, including sandstone, greenstone and chert, in a matrix of sheared or pulverized rock material. Typically, the various rock types occur as hard, resistant masses called "knockers”, which may be a few feet to several tens of feet in smallest dimension. The "knockers” of sandstone are medium-grained, brown, moderately weathered, moderately hard, moderately cemented and closely fractured with a few thin clay seams. Brown highly weathered, slightly hard interbeds of shale and siltstone occur locally. The greenstone "knockers" are fine-grained, greenish-brown, moderately weathered, moderately hard, and hard and closely fractured.

Chert "knockers" are greenish-brown or reddish-brown, slightly weathered, hard and extremely fractured. The shaley matrix material is yellowish-brown (highly weathered) to greenish-black (slightly weathered) and extremely fractured; it ranges from moldable by finger pressure to slightly hard, and from plastic to friable. The sandstone unit consists of predominantly sandstone with some interbedded shale. The sandstone is generally a medium-grained arkose, but locally contains rock fragments similar to the greywacke sandstone common in the Franciscan. Typically, it is thickly bedded. Unweathered sandstone is generally gray, hard, and moderately fractured; weathered rock is light buff, moderately hard and moderately fractured. Exposures of the shale are generally weathered light buff, are slightly to moderately hard, and are closely fractured.

4.2.3 Surficial Deposits

Surficial deposits consist of colluvium, alluvium, landslides, and artificial fill. These deposits are described in more detail below.

4.2.3.1 Colluvium

The colluvium consists of unconsolidated slope wash and slope creep deposits which include a heterogeneous mixture of cobbles, gravel, sand, silt, and clay. Colluvium generally occupies

hillside swales and locally blankets the lower parts of hillslopes. It typically ranges from silty sand to greenish-black sandy clay, generally stratified, and locally easily eroded. Colluvial deposits grade downslope into and interfinger with alluvium. Estimated depths of colluvium range from less than 1 foot on the ridge crests to greater than 20 feet, although it is probably generally less than ten feet in swales and on the lower slopes.

4.2.3.2 Alluvium

The alluvium consists of crudely stratified stream deposits of sand, silt, clay, and gravel. Its on-site extent includes active stream channel deposits and terrace deposits along Miller Creek. It is also moderately to highly permeable and easily eroded. Alluvial deposits grade upslope into and interfinger with colluvium.

4.2.3.3 Landslides

Drawings C1.1 and C1.3 show the approximate distribution of landslides in the Grady Ranch study area. The landslides are classified by (1) state of activity; (2) certainty of identification; (3) type of movement; and (4) estimated thickness of deposit, in accordance with the explanation on Drawing C1.3. The landslides are generally located in swales and on slopes adjacent to drainages, and are primarily small, shallow (less than five feet deep) active slumps and earthflows. Some large slides occur on the upslope portions of the property. In addition to landslides, active creep is occurring locally in soils on steeper slopes and in some colluvium-filled swales. Active gullying of shallow colluvium, alluvium, and deeply weathered bedrock materials occurs along major drainages, and creek bank erosion and sloughing by undercutting is occurring along Miller Creek and the larger tributary drainages.

4.2.3.4 Artificial Fill

The artificial fill on the property consists of two general types: (1) moved soil and surficial deposits occurring locally along graded roads as berms and side cuts; and (2) broken concrete, rock, brick and metal dumped as bank protection along Miller Creek.

4.3 SUBSURFACE CONDITIONS

The subsurface materials encountered in the borings and test pits by HMTA and HTA include bedrock, colluvium, alluvium, and fill. Detailed descriptions of the materials encountered are presented on the boring and test pit logs in Appendix A.

4.4 GROUNDWATER

Groundwater levels vary throughout the study area, and appear to reflect the surface topography. In the 1984-1985 explorations by HMTA, groundwater was measured at depths

ranging from 5 to 13 feet in some holes; other holes were dry. Groundwater levels will vary seasonally, particularly in low lying areas, and adjacent to drainages.

5.0 EVALUATIONS AND CONCLUSIONS

5.1 GENERAL

Based on our review of the available information, it is our opinion the site is suitable for the construction of the proposed development from a geotechnical perspective. However, all of the conclusions and recommendations presented in this report should be incorporated in the design and construction of the project to minimize possible geotechnical problems.

The primary considerations for geotechnical design at the site are discussed in the following sections.

5.2 GEOTECHNICAL AND GEOLOGIC HAZARDS

Potential geotechnical/geologic hazards evaluated for the site include slope stability and landsliding, ground shaking, surface fault rupture, liquefaction, and possibly swelling or shrinking soils. The evaluation of these potential hazards is presented in this section. Discussion of site classification related to seismic analysis and design of the proposed development is presented in Section 6.2.

5.2.1 Slope Stability and Landsliding

The number of landslides and the potential for slope instability in the study area are comparable to other hillside areas in the San Francisco Bay Area. As described previously, there are various types of landslides and related features, both active and inactive (dormant). Areas of active and inactive landsliding, creep, and gullying identified during this and previous investigations are shown on Drawings C1.1 and C1.3.

We have reviewed the proposed floor elevations and grading shown on the grading and drainage plan prepared by CSW|ST2 and dated November 21, 2008. The planned grading will buttress some unstable areas where improvements are planned. In other areas, special foundations and/or conventional slope reconstruction, regrading, or buttressing will mitigate landslide hazards. Existing landslide and colluvial areas shown on Drawings C1.1 and C1.3 should be considered unstable or potentially unstable and they may need to be mitigated during development. Conceptual stability improvement methods are illustrated on Drawing C1.2.

5.2.2 Ground Shaking

As in other areas of the seismically active San Francisco Bay region, the proposed development will likely experience strong ground shaking from future major earthquakes on the San Andreas or other active faults. The expected motion characteristics of these earthquakes will depend on the characteristics of the generating fault, distance to the source

of energy release, the magnitude of the earthquake, as well as specific site geologic conditions. Localized stream bank sloughing, reactivation of existing and activation of new landslides, localized failure of cut and fill slopes, and ground settlement could occur as a result of ground shaking.

The adverse effects of ground shaking can be reduced by using modern seismic design methods. Structures designed and constructed in accordance with code requirements should provide adequate protection against major structural damage.

5.2.3 Surface Fault Rupture

No active or potentially-active faults have been identified in the immediate vicinity of the proposed site according to the California Geological Survey (e.g., Jennings 1994). The fact that an Earthquake Fault Rupture Zone has not been established for the site by the California Geological Survey indicates that the CGS does not consider there to be a significant likelihood that there are active faults in the vicinity of the site. Additionally, observations of the site and surrounding areas do not indicate the presence of geologic conditions, geomorphic features, or lineaments suggestive of active or inactive faults crossing the project site.

Based on this information, we consider that the potential for surface fault rupture at the Grady Ranch site is very low.

5.2.4 Liquefaction Potential

Liquefaction is a secondary effect of ground shaking and refers to the sudden and partial to complete loss of strength in saturated, loose to medium dense granular soils. Conditions where this phenomenon could occur are probably limited to areas of the recent terrace deposits and alluvium along Miller Creek. Given strong enough ground shaking, granular soils in these areas could liquefy if saturated.

5.2.5 Soil Swelling or Shrinkage Potential

The USDA (2008) indicates that surficial soils at the site are generally low to possibly moderate plasticity. Therefore, the shrink or swell potential of the surficial soils is likely to be low to moderate and it not likely to create major constraints on the project development. Future investigations should further evaluate the plasticity and shrink-swell potential of the site soils.

5.3 EROSION AND GULLYING

The potential for erosion is moderate where soil or deeply weathered bedrock is exposed in cut slopes or excavations. Erosion potential can be reduced by hydroseeding and landscaping, and providing interceptor drainage ditches near the top of cut slopes.

Active gullying is occurring within the intermittent streams that originate in the hills and flow south towards Miller Creek. The gullying ranges from minor down-cutting where hillside drainage is concentrated, to significantly eroded channels at the mouth of these streams where they flow into Miller Creek. Where necessary, the gullying process will be controlled by rock stabilization.

5.4 CREEK BANK STABILITY

In general, natural bank slopes of Miller Creek and its tributaries that are flatter than 2:1 (horizontal to vertical) can be considered stable. Bank slopes between 2:1 and 1:1 can be considered marginally stable, while slopes that are steeper than 1:1 and higher than about 10 feet should be considered unstable in accordance with current standards.

Scour has locally steepened and undercut creek banks and has created unstable slopes. Stream restoration for this project is addressed in the report titled *Hydrologic and Geomorphic Recommendations for Stream Conservation Areas at Grady Ranch*, prepared by Balance Hydrologics as part of the PDP submittal.

6.0 RECOMMENDATIONS

All of the conclusions and recommendations in this report are preliminary in nature based on limited subsurface information, previous and current geologic mapping, and our experience with similar geologic settings, and they are subject to refinement and modification as additional information becomes available.

6.1 EARTHWORK

Extensive earthwork is planned as part of the proposed development. The earthwork at the site is expected to include:

- Clearing and stripping of existing improvements, vegetation, and topsoil.
- Construction of the planned cut slope to be located west and north of the proposed building (cuts are anticipated to be up to about 60 feet deep).
- Construction of the planned fill slope to be located on the east side of the property (fill thickness is anticipated to be up to about 35 feet.
- Deep excavations for the building basement retaining walls pad and foundation areas.
- Miscellaneous cutting and filling to bring the site to grade.
- Preparation of areas to receive fill and site improvements.
- Placement of fill to backfill walls and shallow excavations.

6.1.1 Subgrade Preparation

Before fill is placed on any soil surface, organic-rich soils or other deleterious materials should be excavated and removed from the site. The upper 8 inches of any exposed soil surface upon which fill will be placed should be scarified, plowed, disked, and/or bladed until it is uniform in consistency and free of unbroken chunks and clods of soil greater than 4 inches in greatest dimension. The moisture content of the subgrade soil should then be adjusted to between optimum and 3 percent above optimum, and should be compacted with equipment suitable for the soil and site conditions. The subgrade soil should be compacted to not less than 90 percent of maximum dry density as determined using ASTM Method D1557.

6.1.2 Fill Materials

6.1.2.1 General Fill

All fill and backfill materials should be a soil or soil-rock mixture free of organic material, debris, and other deleterious substances. The fill should contain no particles larger than 4 inches in greatest dimension. In addition, no more than 15 percent of the fill particles should be larger than 2½ inches in greatest dimension. Native soil and earth fill obtained from on-site excavations may be suitable for use as fill, providing the materials are free of debris and organic matter. Minor amounts of concrete, asphalt concrete, or brick, if encountered in on-site

excavations, can be incorporated into the fill if broken into pieces not larger than 4 inches in size.

6.1.2.2 Select Fill

"Select" fill should have the following properties or characteristics:

- All fill particles should be less than 3 inches in size.
- Less than 30 percent of the material should be retained in the ¾-inch sieve.
- No less than 15 percent and no more than 50 percent of the material should pass the No. 200 sieve.
- The fines (i.e., material passing the No. 200 sieve) should have a plasticity index (PI) no greater than 15.
- The fill material should contain less than ½ percent by weight of organics and should be free of other objectionable materials (e.g., concrete, plastic, metal, and other wastes) or potentially hazardous substances.

6.1.3 Fill Placement and Compaction

Fill and backfill should be placed on the prepared subgrade in horizontal lifts that do not exceed 8 inches in thickness before compaction. The fill should be compacted with suitable equipment to the requirement listed below. The final surface of the compacted fill should be graded to promote good surface drainage, as described later below.

Any filling operations on slopes steeper than 5:1 should be keyed and benched into the weathered Franciscan Complex materials. Loose soils resulting from excavations should either be removed from the site or placed and compacted as engineered fill. All fill slopes should be overbuilt by at least 1 foot and then trimmed back to final grades. A keyway should be constructed for the new fill slope to be located on the eastern portion of the site. Section 6.1.4 provides keyway construction recommendations. Conceptual fill placement and keyway construction details are shown on Drawing C1.2.

During fill and backfill activities at the site, the degree of relative compaction (as determined by ASTM D 1557) should conform to the following minimum requirements:

Fill Location	Degree of Compaction (%)
General site fill	90
Structural fill (i.e., beneath structures)	95
Utility trench backfill	90
Pavement subgrade	95

Permanent slopes of compacted fill should be no steeper than 2:1. Wherever possible, permanent slopes should be graded to blend final ground surfaces into the adjacent topography. Exposed ground surfaces and fill slopes will be subject to wind and water erosion and local raveling if not adequately protected. Fill surfaces should be provided with erosion protection measures as soon as the final grades or cut and fill slopes are created.

Where space is limited, the use of geogrids or geotextiles may enable construction of steeper permanent reinforced earth fill slopes. Mechanically stabilized earth (MSE) walls may also be used for any retaining structures that retain engineered fill.

6.1.3.1 Weather Considerations

The time of year and weather conditions when foundation preparation and earthwork are undertaken will greatly affect the time and effort required to complete the work. Care should be taken to mitigate water access to the earthwork. Excavation, foundation preparation, and compaction of fill will be difficult during winter or early spring when weather conditions may render the foundation soil and fill materials saturated and wet. Also, the exposed foundation soil may become unstable. Therefore, to minimize delays in the project, the foundation construction and earthwork should be scheduled for late spring, summer, or early fall. If grading is to be performed during the rainy season, we recommend provisions be included in the construction contract for mitigating measures such as chemical stabilization (such as lime treatment) or use of geotextiles.

6.1.4 Keyway Construction

In general, where fills are to be placed over ground that is steeper than 5:1, the fill should be keyed and benched into competent material. Specifically, a keyway should be constructed at the downslope limits of the new fill slope to be located on the ridge near the east property line. In addition, the need for keyways below other fill areas should be evaluated following a detailed subsurface investigation to evaluate the properties of the existing surficial soils and the depth to competent ground.

Keyways should be at least 15 feet wide, and the base of keyways should extend a minimum of about 3 feet into competent material. In general, competent material should be comprised of undisturbed Franciscan Complex shale or sandstone. However, firm soils may be acceptable in come cases. Keyways should have a minimum slope of 2 percent into the hill. Benches should be excavated into the slope before placing the fill at vertical intervals of no more than 10 feet. These benches should be at least 10 feet wide and should have a minimum slope of 2 percent into the hill; however, the actual dimensions of the benches may be modified by the geotechnical engineer at the time of construction; in some cases, shallower “notching” of the fill into competent material may be acceptable.

A subdrain consisting of 4-inch diameter perforated PVC pipe should be installed at the rear of the keyway with the perforations facing down. A layer of Caltrans Class 2 Permeable Material (hereinafter referred to as “drain rock”) should be placed over the extent of the bottom of the keyway. The thickness of the drain rock should be at least 12 inches. The subdrain pipe should be bedded on at least 4 inches of drain rock. A vertical column of drain rock should be placed over the subdrain pipe and should extend up the back wall of the keyway. The drain rock column should be at least 12 inches wide.

Cleanouts should be installed at the ends of the subdrain and at distances no greater than 150 feet along the subdrain alignment. If there are turns within the subdrain alignment that are sharper than 45 degrees, a cleanout should be installed at each turn. The keyway should then be backfilled with engineered fill that is placed in accordance with the recommendations of this report.

A subdrain should also be installed at the rear of selected benches that are cut into the hillside. The subdrains for the benches should be constructed similar to the keyway subdrain except that a layer of drain rock over the entire bottom of each bench is not necessary.

A detail showing typical keyway and bench construction configurations is shown on Drawing C1.2.

6.1.5 Excavations

6.1.5.1 General

Excavation of colluvium and slide debris is likely to be relatively easy with conventional earthmoving equipment. Excavation of alluvium is also likely to be relatively easy, although the alluvium may contain a significant portion of large gravel, cobbles, and some boulders.

The quality of the Franciscan Complex bedrock is likely to vary across the site, and as a consequence the ease of excavation will vary as well. It may range from soil-like in its excavatability, to hard enough that it may require ripping.

Excavated material may be used as site fill. However, it may require processing and/or moisture conditioning to meet the requirements of select fill or general fill as described above in Section 6.1.2

6.1.5.2 Cut Slopes

For preliminary design, permanent cut slopes in soil or Franciscan Complex materials should be assumed to be no steeper than 2:1. Where possible, permanent slopes should be graded to blend gradually transition into final ground surfaces in the adjacent topography. Exposed

ground surfaces and cut slopes will be subject to wind and water erosion and local raveling if not adequately protected. Cut surfaces should be provided with erosion protection measures as soon as the final grades or cut and fill slopes are created.

6.1.5.3 Temporary Cut Slopes

The stability of temporary excavation slopes made at the site will depend on the depth of the excavation, the strength and character of the native soils and Franciscan Complex material exposed in the excavation, groundwater conditions, the construction schedule (i.e., the time of year and the length of time the excavation or cut is allowed to stand open), and the contractor's operations and equipment, among other factors. Because of the complex nature of the subsurface conditions at the site, the stability of temporary and permanent cut slopes is difficult to predict at this time. Adversely oriented beds, joints/fractures, and shears may exist almost anywhere cuts are made through these earth materials. As a consequence, slab, block, and wedge failures may occur randomly in excavation sidewalls. In addition, vibrations from excavation equipment (e.g., hydraulic hoe-ram) could open fractures and/or shake blocks loose from the rock faces and slopes.

For planning purposes and for preparing the engineer's construction cost estimates, temporary excavation slopes in soil and Franciscan Complex should be no steeper than 1:1 for slopes up to about 25 feet in height. For higher slopes, this inclination may be acceptable but it should be evaluated by an engineering geologist based on subsurface exploration during the final design. Even at this inclination, Franciscan Complex cuts could fail where adversely oriented rock discontinuities exist. Flatter slopes (or other measures) may be necessary if localized instability is observed during construction. Flatter side slopes also may be required (and should be anticipated) if the contractor intends to stockpile materials and/or use heavy equipment adjacent to the excavation. Review of the excavation conditions by an engineering geologist during future exploration or construction may provide information that would allow for use of steeper slopes than indicated above, resulting in cost savings.

If loose blocks/wedges/slabs of rock are exposed on excavation cut slopes, measures should be taken to prevent the blocks/wedges/slabs from falling/sliding down the slopes into work areas. Measures may include, but are not limited to, cleaning and barring the slopes of loose material, installing rock bolts, shotcrete and/or wire mesh, and constructing catchment fences along benches and/or the base of slopes.

The Franciscan Complex that will be encountered in excavations may slake/slough upon wetting and drying. Cut slopes in both soil and Franciscan Complex exposed for extended periods likely will ravel and require occasional cleanups. The Franciscan Complex and some site soils also will be prone to erosion where exposed to the elements.

Where slide debris or colluvium is exposed in an excavation, temporary cut slopes may need to be flatter than 2:1 to maintain slope stability. Where slide debris or colluvium will be exposed in permanent slopes, remedial measures may be needed to stabilize the ground.

Temporary excavations used in construction should be designed, planned, constructed, and maintained by the contractor and should conform to state and/or federal safety regulations and requirements. As is the case anywhere that excavations are made in soil and rock, unexpected caving of excavations, temporary cut slopes, or trench walls could occur at any time or place. Workers in excavations and trenches must be trained and adequately protected by appropriately inclining the excavation side walls or employing appropriate measures to support the ground.

6.1.6 Stabilization of Landslides and Colluvial Slopes

Where portions of landslides or colluvium will be removed during grading, especially where the material removal will occur lower on the slope, the loss of material is likely to reduce stability of the slope and lead to accelerated slope creep and/or sliding. In these cases, it will likely be necessary to stabilize the slopes.

The most straight-forward method of landslide repair or stabilization of colluvial slopes is to remove all of the slide debris or colluvium, excavate a key and benches into competent material, install subsurface drainage measures, and place engineered fill into the excavated area. A typical repair section is shown on Drawing C1.2. We recommend this method be used where there are discrete landslides that encroach on or immediately adjacent to areas to be developed (these slide areas are mapped as Qls on Drawing C1.1).

Typical depths of slides have been estimated based on geomorphic interpretation as shown on the cross section on Drawing C1.1. We recommend that the presence or absence of slides mapped as “queried” on Drawing C1.1 be evaluated during field investigations. In addition, final depths of all slides should be confirmed in the field at the time of construction by a representative of our firm.

All graded areas that will not otherwise be developed should be hydroseeded with low water, deep rooted, fast growing vegetation or otherwise planted with appropriate vegetation.

6.1.7 Dewatering Requirements for Groundwater

Although we did not observe springs within the project site, it is possible that groundwater will be encountered during site excavation. If and where groundwater is encountered, it should be carefully evaluated for the quantity of water that it may introduce into the excavation during

construction. Measures should be taken to collect groundwater from the excavation to prevent the excavated ground surfaces from becoming saturated and softened.

For long-term conditions, groundwater should be evaluated for anticipated hydrostatic conditions and flow characteristics, and appropriate subdrains should be implemented.

6.1.8 Surface Water Drainage and Erosion Control

Positive surface drainage should be provided adjacent to buildings to direct surface water away from the foundations into closed pipes that discharge downslope of the proposed building. In addition, surface water and rainwater collected on the roof of the building should be transported through gutters, downspouts, and closed pipes and routed to suitable discharge facilities. Ponding of surface water should not be allowed in any areas adjacent to the structure. Concentrated flows of water should not be allowed across site slopes as erosion or weakening of the slopes could occur.

During construction, the contractor should be responsible to provide adequate drainage control measures to prevent erosion and ponding of rainwater as well as any groundwater encountered during the excavation.

6.2 SEISMIC DESIGN

It is our understanding that planned building will be designed using ground motions developed in accordance with the 2007 California Building Code (CBC).

- Based on limited data from four borings performed on site in 1984 by HMT (presented in HMT 1988), the Standard Penetration Test (SPT) blow counts indicate that most portions of the site should be classified as Site Class D.
- In one boring from HMT (1993), located nearly 2000 feet to the west of the proposed main building location, the blow counts indicate a Site Class C. In our experience with Franciscan Complex materials such as are present at the site, SPT blow counts and/or direct measurements of shear wave velocities often indicate Site Class C conditions if overlying soils are relatively thin. It may be that portions of the site could be classified as Site Class C or even Site Class B based on future testing.
- Because most of the main building will be founded in areas of significant excavation, bedrock is expected to be exposed under much of the building foundation and it may be appropriate to reclassify the site to Site Class C (or even B, but this is less likely) on the basis of future subsurface exploration.
- In some locations with deep alluvium, such as along the stream channel where bridges are proposed, if future subsurface exploration indicates that the alluvium is loose to moderately dense, (and hence potentially liquefiable) the Site Class may be as poor as Site Class E in the current condition. However, in any location where this is the case, it will likely be appropriate to either implement some form of

liquefaction mitigation, or to construct foundations that extend through the potentially liquefiable zone in order to derive their support from underlying non-liquefiable soils or bedrock. This would likely result in a stiffening of the foundation, or improving the subsurface conditions, such that a Site Class D will be appropriate for design even in areas underlain with potentially looser soils.

The project structural has indicated that the fundamental period of the project buildings are as follows:

- The Main Building: anticipated period of 1.5 to 2 sec
- Stage A and Stage B: anticipated period of 0.4 to 1.0 sec

For structures with fundamental periods less than about 0.5 seconds, the spectral accelerations for Site Classes C and D are identical, and for these cases, distinguishing between Site Classes C and D may be moot.

For longer-period structures, the spectral accelerations will be significantly less for a Site Class C. In these cases, it may be possible to realize cost savings by re-evaluating the site class in order to potentially reduce the seismic demand by about 13 percent.

For these reasons, for the purposes of developing the response spectra during this stage of the design, we recommend using the following 2007 CBC seismic design parameters indicated for Site Class D. For comparison purposes, the values for a Site Class C and Site Class B are also shown in the following table and in Figure 4:

Description	**2007 CBC**		
Latitude	38.0421 N		
Longitude	122.6005 W		
	Recommended Values for Preliminary Design	For information only, showing sensitivity to Site Class Designation	
Site Class	D	C	B
Site Coefficient, F_a	1.0	1.0	1.0
Site Coefficient, F_v	1.5	1.3	1.0
Design Spectral Response Acceleration Parameter, S_{DS}	1.00	1.0	1.0
Design Spectral Response Acceleration Parameter, S_{D1}	0.60	0.52	0.40

6.3 RETAINING WALLS

There are currently no borings on site that extend more than a few feet into bedrock. Prior to final design it will be necessary to extend borings in the location of the deeper excavations at least 10 to 20 feet below any future foundation levels to evaluate the depth to Franciscan Complex and the properties and localized variability of this material. For this reason the following recommendations are subject to change as additional information becomes available.

6.3.1 Free-Standing Walls versus Building Walls

The major retaining walls for this project will be required to retain cuts up to about 60 vertical feet into the hillside, and these walls will become building and basement walls. We anticipate there will also be smaller walls to retain both fills as well as cuts.

Where retaining walls will be constructed as part of a building, it should be assumed that it will not be acceptable for the walls to deflect outward, and these walls should be designed as “restrained” or “non-yielding” walls that resist at-rest earth pressures. Where free-standing walls are structurally independent of buildings and it is acceptable for them deflect outward slightly, they may be assumed to be “yielding” walls that are designed to resist “active” earth pressures. Earth pressures for each of these conditions are discussed below.

Where basement walls or building retaining walls are more than a few to 10 feet high, it is likely that a cantilever wall system will not be stiff enough to resist at-rest earth pressures while limiting lateral deflections to acceptable levels. In these cases, a system of tiebacks or internal bracing will likely be needed to strengthen and stiffen the walls. If tiebacks will provide the permanent lateral load resistance, the tiebacks should be constructed with appropriate corrosion protection to provide long term performance.

Where retaining walls supporting cuts are free-standing (structurally independent from any building), it may be possible to use a soil nail system, which can be designed to be stable although the deflections may be larger than a tied-back systems.

6.3.2 Wall Construction Considerations

We anticipate that top-down construction of the larger walls will be necessary to protect the excavation during construction. Once excavation is complete, the original walls may be incorporated into the permanent walls, or a secondary wall may be constructed in front of the temporary wall. If a secondary wall is constructed, we recommend that it be poured directly against the temporary wall to avoid the need to backfill between the walls.

We anticipate that it will be most appropriate to construct these walls in a “top-down” fashion using one of the following general approaches. The final selection of systems will depend, in part, on the properties of the materials to be retained. Some of the more relevant properties include strength, joint orientation, weathering, and presence of groundwater.

1. Walls may be constructed below grade before excavation starts by using a secant wall method, in which a series of lean-concrete drilled piers are constructed with overlapping (secant) shafts such that a continuous wall is formed. Typically a steel beam is inserted into approximately every second pier hole while the concrete is still fresh. After the concrete has cured, excavation can begin adjacent to the wall.

   To minimize lateral deformations for all but fairly short walls, the walls should be laterally restrained by the construction of tiebacks. An upper row of tiebacks is typically installed after the excavation has extended just a few feet below the top of the wall. Tiebacks should be installed and tested to above their design loads, then locked off at the design load.

   Depending on the wall height, anticipated earth pressures, condition of materials to be retained, subsequent rows of tiebacks may also be needed.

   Once final grade has been reached, a permanent wall facing can be constructed that becomes part of the structural load-carrying wall. Alternatively, an architectural wall facing can be constructed if the temporary wall has been designed and constructed as a long-term structural system.

   It is likely that the temporary wall will have a low enough permeability that groundwater can build up behind it unless drainage features are installed. The permanent wall system may be designed either to resist hydrostatic pressures in addition to earth pressures, or drainage may be constructed to remove groundwater from behind the wall. Due to the difficulty in achieving satisfactory waterproofing for basement walls that retain hydrostatic water, we recommend constructing wall drainage as a preferred alternative, although either approach may be acceptable at the discretion of the design team.

2. A soldier pile and lagging wall system may be used. In this system, a series of lean-concrete drilled piers (or soldier piles) are constructed with a gap between each soldier pile. A steel beam is inserted in the hole while the concrete is still fresh; after the concrete has cured, excavation can begin adjacent to the wall. As the excavation progresses, lagging is inserted behind the flanges of the beams (or alternatively onto brackets welded onto the front of the beams) to retain the material behind the wall.

   Most of the above secant pile wall discussion regarding tieback and drainage is applicable for soldier pile and lagging walls.

3. Alternatively, depending on the properties of the materials to be retained, a similar system may be constructed by mixing soil and cement in-place to form a deep cement soil mixed (DCSM) wall. Most of the above discussion regarding secant walls is applicable for DCSM walls.

6.3.3 Earth Pressure and Anchor Considerations

Retaining walls should be designed to resist long-term static earth pressures (triangular pressure distribution) and seismic earth pressures as presented below in Section 6.3.4, “Lateral Earth Pressures”. For temporary loading conditions of tied-back walls, a uniform “apparent earth pressure” should be assumed, as presented below. Walls should be designed to adequately resist both loading conditions, but these long-term and temporary loads should not be considered to act concurrently.

Where a retaining wall will also be part of the building wall, at-rest earth pressures should be considered to limit the lateral wall movement. The capacity of and construction-phase shoring system should be checked during each stage of construction, as well as when the excavation is completed.

In addition to the lateral earth pressures, the retaining wall should be designed to resist surcharge pressures from construction activities. Construction surcharge pressures are dependent on the contractor's operations, such as placement of cranes and storage of materials, and should be determined by the contractor.

To limit lateral deflection of the shoring wall during construction, it is recommended that pre-stressed soil/rock anchors (tiebacks) be used to resist the lateral earth pressures. The anchors should develop load resistance beyond the imaginary plane shown on Drawing C1.2. Each anchor should be proof tested and locked off at a design load to be determined by the wall designer. If the anchors will be designed as permanent structural elements they should be corrosion-protected. Pressure grouting during installation and/or post grouting should be considered to improve performance and increase the bond stress. The upper row of anchors should be installed at a shallow depth, and the vertical distance between subsequent rows of anchors should determined by the wall designer but should not exceed 12 feet.

Groundwater may be present during installation. If the risk of subsidence due to caving-in of soil/rock is significant, the soil/rock anchor holes should be cased.

Additional resistance to lateral and surcharge pressures can be provided by passive earth pressure acting on shoring elements extending below the level of the excavation. The passive pressure given in Section 6.3.4, “Lateral Earth Pressures”, is for a continuous wall. If soldier piles bedded in concrete are used to shore the excavation, the passive pressure can be assumed to act on a width equal to twice the diameter of the concrete pier.

6.3.4 Lateral Earth Pressures

Below grade structures and any retaining walls should be designed to resist both lateral earth pressures (static and seismic) and any additional lateral loads caused by surcharge loads (such as traffic or adjacent structures) on the adjoining ground surface.

The recommended earth pressures for different loading conditions are listed in the following table:

LATERAL EARTH PRESSURES		
Loading Condition	**Equivalent Fluid Weight for Lateral Earth Pressure Calculations**	
	Level Ground	2:1 Back Slope
Active Earth Pressure[1,2]	35 pcf	50 pcf
At-Rest Earth Pressure[1,2]	55 pcf	80 pcf
Temporary Tied-Back Pressure	28H in psf[2,5]	40H in psf[2,5]
Seismic Increment, Active[1,3]	Uniform 30H in psf[5]	Uniform 45H in psf[5]
Seismic Increment, At-Rest[1,3]	Uniform 20H in psf [5]	Uniform 30H in psf [5]
Passive Earth Pressure in Competent Soil[4]	350 pcf	Not Applicable
Passive Earth Pressure in Competent Franciscan Complex material[4]	500 pcf	Not Applicable

Notes

1. **Active** pressure is typically used where the wall is unrestrained so that the top of the wall is free to laterally deflect by 0.4 percent of the wall height from the base of the heel to the top of the backfill above the heel. **At-rest** pressures should be used where the top of the wall is restrained (e.g., building or basement walls) so that deflections of this magnitude cannot occur.
2. Below water level, earth pressures may be assumed to be reduced by 50% and then combined with hydrostatic pressures.
3. When considering the seismic load case, the pressure increment should be distributed uniformly against the back of the wall and added to the static lateral earth pressure for Active or At-rest conditions. For calculating overall stability, the resultant of the seismic increment should be applied at a point 60 percent of the wall height above the base of the footing.
4. Ignore passive resistance for the upper 12 inches unless a rigid slab covers the ground surface. The pressures can be applied to 2 times the pier diameter and up to a maximum of 5000 psf.
5. H is in feet.

The above pressures are based on the assumption that sufficient drainage will be provided behind the walls to prevent the build-up of hydrostatic pressures from surface and subsurface water infiltration. Acceptable methods to provide adequate drainage will vary depending on the type of wall:

1. For walls that are constructed from the bottom up, made with cast-in-place concrete where both sides are formed, conventional drainage may be provided by gravel or drainage panels, as described below:

   Adequate drainage may be provided by a subdrain system consisting of a 4-inch diameter perforated pipe bedded in ¾-inch clean, open-graded rock. The entire rock/pipe unit should be wrapped in filter fabric. (As an alternative to open-graded rock wrapped in filter fabric, Caltrans Class 2 drain rock may be used without filter fabric.) The rock and fabric placed behind the wall should be at least one foot in width and should extend to within one foot of finished grade. The upper one foot of backfill should consist of compacted soils. Alternatively, prefabricated drainage panels may be used instead of drain rock, with the drainage panels connected to a 4-inch-diameter perforated pipe at the base of the wall. In either case, the subdrain pipe should be sloped to drain by gravity and be connected to a system of closed pipes that lead to suitable discharge facilities. In addition, the "high" end and all 90 degree bends of the subdrain pipe should be connected to a riser which extends to the surface and acts as a cleanout.

   If a free-standing wall is constructed in front of a temporary secant pile wall or a DCSM wall, groundwater may still build up behind the temporary wall and these hydrostatic forces may be transferred to the permanent wall. The permanent wall should be designed to resist hydrostatic forces, or drainage of the temporary wall should be provided as discussed below.

2. If a secant wall or DCSM wall is constructed as the permanent wall or integrally with the permanent wall, groundwater pressures are likely to build up unless a method to provide drainage is constructed; if no wall drainage is provided, the wall should be designed to resist hydrostatic pressures. If drainage is provided, its design should be reviewed by AMEC for adequacy both in allowing the groundwater to drain from behind the wall, and in collecting and transporting the water to an acceptable discharge facility.

6.4 FOUNDATIONS

As with all other portions of this report, the following recommendations are preliminary in nature based on the assumptions described herein, and they are subject to refinement and modification as additional subsurface data becomes available.

6.4.1 Portions of Building Underlain by Franciscan Complex Material

Based on the available subsurface exploration data, our geologic mapping, and our experience with similar geologic settings, we have developed the preliminary geologic map and cross section interpretations, which are shown on Drawings C1.1 and C1.3. The proposed building location is shown on the plan view on Drawing C1.1, and the finished floor levels are shown on the Drawing C1.1 cross sections. As can be seen in the cross sections, most of the building will be located in areas of relatively deep excavations, and it appears that in most of the area Franciscan Complex material will be exposed at the foundation level. This means that in these areas it will be possible to utilize shallow spread footings that bear on undisturbed Franciscan material. Although most Franciscan material is competent to provide relatively high allowable bearing pressures with little settlement, the Franciscan Complex is highly variable

and includes some material that may not provide reliable support for shallow spread footings. Therefore, future subsurface exploration should be designed to evaluate the quality (strength and compressibility) and variability of the Franciscan Complex material within the building footprint.

As an alternative to shallow spread footings, drilled piers may also be used that extend into the Franciscan material. In general, drilled piers will provide foundation support with less settlement that footings, but the cost will be significantly higher than footings.

6.4.2 Portions of the Building Underlain by Alluvium or Colluvium

In addition to the variability of the Franciscan Complex, as is indicated in the cross sections, it appears that portions of the building will be underlain by alluvium even after excavation has been performed to reach the building grade. There may also be portions underlain by colluvium.

In our opinion, the in situ alluvium and colluvium are not suitable for support of shallow spread footings. In areas underlain by alluvium or colluvium we consider the following foundation alternatives will be viable from a geotechnical perspective:

1. Remove and Replace. The alluvium and colluvium that are present beneath planned footings may be removed and replaced with engineered fill, and the building may be supported on shallow spread footings supported on the engineered fill. These footings will likely settle similar to or slightly more than footings on the adjacent Franciscan material, but the settlement is still likely to be less than about 1 to 1.5 inches.
2. Geopiers or Stone Columns. Geopiers or stone columns (described below) may be installed through the alluvium or colluvium to transmit a significant portion of the building loads from footings to the underlying Franciscan material. It is likely that static foundation settlement will be similar to that of footings on engineered fill. However, the possibility that alluvium or colluvium surrounding the Geopiers or stone columns could liquefy or densify during strong earthquake shaking should be evaluated, as well as the impact that this liquefaction or densification could have on the performance of the Geopiers or stone columns.
3. Drilled Piers. Drilled piers may be constructed to extend well into the Franciscan material so that building loads are supported by skin resistance between the sides of the pier and the surrounding material. It is likely that drilled pier foundation will settle less than footings, but they will be more expensive.

Estimates of foundation settlement should be revised during final design based on additional subsurface exploration at the site.

6.4.3 Bridge Foundations

It appears that bridge abutments will be located over alluvium. In these locations, foundation loads should be transferred to underlying Franciscan material through a deep foundation system such as drilled piers. Where abutments are located directly over competent Franciscan material, foundations may consist of either shallow spread footings supported on the firm Franciscan material, or drilled piers that extend into the Franciscan material and derive support from skin resistance between the sides of the pier and the surrounding material.

It will be most important to evaluate the potential for erosion and scour of material adjacent to the bridge foundations and to satisfactorily mitigate this hazard.

6.4.4 Summary of Foundation Alternatives

The following is a brief summary of the foundation alternatives and assessment based on foundation performance.

Structures	Foundation Alternatives	Performance
Main Building	A1: Where Franciscan material is exposed at the bottom of the building excavation, use shallow spread footings supported directly on Franciscan material.	Very Good
	In areas underlain by alluvium or colluvium, use:	
	• A1(a): Footings on engineered fill	Good
	• A1(b): Footings on Geopiers	Good
	• A1(c): Drilled Piers	Very Good
	A2: All Footings on engineered fill[(a)]	Good
	A3: All Footings on Geopiers	Good
	A4: All Drilled Piers	Very Good
Bridges	B1: Drilled Piers	Very Good
	B2: Footing Foundations	Good - Poor

(a) Assumes Franciscan material is over-excavated and replaced beneath the entire building foot print to a depth of about 3-5 feet beneath the bottom of footings.

Because of the difference of subgrade and/or foundation types across the site, a variation of total and differential settlement will occur. The performance of the foundations over the entire building should be evaluated and the final selection of the foundation system should be made after the final design level geotechnical investigation has been performed.

6.4.5 Shallow Spread Footings

As discussed above, shallow spread footings may be used to support the proposed building. Footings can be supported directly on Franciscan material or on recompacted engineered fill,

as well as on Geopiers or on recompacted engineered fill in areas that need mitigation. Recommendations for Geopiers are provided in the next section.

According to the grading and site plan (see Drawing C1.1), we understand that the finished floor elevation of the basement of the proposed building will be at approximately Elevation 240 feet. In order to create a level building pad, the slope and spur ridge currently existing within the building footprint will have to be cut to grade. The proposed building will require a retaining wall (up to approximately 35 feet high) around much of the building. A retaining wall will also be constructed north and west of the building to support the new cut slope.

Geologic mapping and previous boring logs indicate that the majority of the proposed building is underlain by stiff colluvium overlying the Franciscan Complex, with the southeast portion of the building being underlain by alluvium (see Drawing C1.1). The thickness of colluvium over the Franciscan Complex is not known, but we estimate that it typically ranges from about 2 to 10 feet but locally may be from 1 to 20 feet. We anticipate that the planned grading operations will likely result in the removal of the colluvium to expose Franciscan material at the proposed finished floor elevation over most of the building footprint.

The southeast portion of the building footprint is expected to be underlain by alluvium. Based on results of the geologic mapping and our experience in the area, it is likely that the alluvium is loose to moderately dense and may be susceptible to liquefaction and slope instability. In addition, the alluvium could settle if fill is placed directly on it. We recommend that the alluvium be over-excavated to expose Franciscan Complex and recompacted to at least 95 percent relative compaction as determined by ASTM D1557. Fill should be placed in accordance with the requirements listed in Section 6.1. We anticipate that the depth to Franciscan material will vary; the cross sections on Drawing C1.1 show inferred depths to Franciscan Complex in selected locations.

Footings constructed with the above recommendations will be supported on Franciscan material for most of the proposed building and will be supported on engineered fill or Geopiers for portions of the building footprint, such as the southeast part of the building. Footings bearing on Franciscan material, engineered fill, or Geopiers should have a minimum width of 2 feet and be founded at least 3 feet below adjacent finished grade. The horizontal distance between the edge at the bottom of a footing and the face of a permanent slope down should be at least 15 feet. Where they are not on a constant level, footings should be stepped in increments not exceeding 2 vertical feet. Footings that meet the foregoing requirements for bearing on Franciscan material or engineered fill may be preliminarily designed for the following net bearing pressures (in pounds per square foot, psf).

Loading Condition	Franciscan Complex	Engineered Fill
Dead load	4,000	2,500
Dead plus live loads	6,000	3,500
All loads, including wind and seismic	8,000	4,500

These bearing capacities are net values; therefore, the weight of the foundations and backfill above the footings can be neglected for design purposes.

Lateral loads can be resisted by a combination of passive resistance between the edge of the footings and the surrounding soil or Franciscan material and through friction between the footings and the subgrade material. The ultimate passive pressure acting on the face of the foundations can be estimated to be 350 pounds per cubic foot. We recommend that a coefficient of sliding resistance of 0.35 be used between the footings and the underlying recompacted soil; and 0.40 for footings directly on Franciscan Complex. The frictional resistance calculated using this factor corresponds to the peak (ultimate) static friction (i.e., factor of safety equal to 1.0).

Some settlement of the proposed building will occur given the compressibility of the engineered fill and Franciscan material underlying the building. The amount of settlement will depend on the foundation size and the magnitude of the applied loads. Based on preliminary design loads provided to us by the Crosby Group for spread footings designed in accordance with the recommendations presented above, we estimate that the maximum settlement will not exceed about 1 inch for footings founded on Franciscan material, or 1½ inches for footings founded on engineered fill or Geopiers. Differential settlement between an adjacent wall and/or column footings is not expected to exceed ½ inch.

We recommend that a representative from AMEC observe the bearing material exposed in footing excavations before reinforcing steel and concrete are placed. If loose or soft soils are exposed in any excavation, the footing should be deepened or the loose or soft soils excavated and replaced with engineered fill or lean concrete. Water should not be allowed to pond in the footing excavations. Water should be removed, along with soft or wet soil, before concrete is placed.

6.4.6 Footings on Geopiers or Stone Columns

Based on the preliminary subsurface conditions, portions of the southern end of the building footprint will be underlain by up to 10 to 20 feet of un-consolidated soils, consisting of alluvium and colluvium, that may not be competent to support the building. Based on the available information, we consider that one acceptable foundation option is to support the structure on stone columns or Geopiers that extend down into the Franciscan Complex. (The following

discussion focuses on Geopiers, but most of the recommendations regarding allowable bearing pressure, etc., are applicable to stone columns as well, with the primary difference being the details of the construction procedure.)

The Geopier Foundation System is a refinement on traditional methods for settlement control and bearing capacity improvement. Geopier elements are constructed by drilling a shaft to create a cavity, removing a volume of compressible soil, then building a bottom-bulb of clean, open-graded stone while vertically pre-stressing and pre-straining subsoil underlying the bottom bulb. The Geopier shaft is built on top of the bottom-bulb, using Caltrans ¾-inch Class 2 aggregate base rock placed in 1-foot rammed lifts above the groundwater level. For shaft portions that may exist within groundwater, up to 3 inch clean crushed aggregate is used. Densification of the bottom-bulb and of the undulating shaft lifts is accomplished by using impact ramming energy. The ramming energy from a modified 2,000 lb to 3,500 lb hydraulic hammer attached to a special tamper delivers 250,000 ft-lb to 1.5 million ft-lb of energy per minute. The beveled tamper consists of a special steel alloy shaft and a round beveled tamper head. The specially designed tamper head is beveled at 45 degrees on the bottom to transfer forces laterally during impact densification, resulting in pushing of aggregate into and against the confined walls of the cavity. Commonly a 30-inch drilled Geopier shaft will be 33 to 36-inches in diameter after the ramming action. In addition to increasing the soil's shear resistance at the Geopier element perimeter, the increased horizontal stress in the matrix soil improves load carrying capacity and makes the soil up to 2 diameters from the perimeter much stiffer. The stiff Geopier shafts control settlements right where the highest footing stresses occur, just under the footing; and reduce stresses transmitted to lower zones, thereby reducing settlement of the entire foundation system.

Typical allowable bearing capacities of Geopier-reinforced soil exceed 5 to 8 ksf. The stiffness of the Geopier columns is estimated to be on the order of 200 kips per inch of a vertical downward movement. We recommend the use of Geopier elements equipped with high grade steel anchors to resist uplift loads. Geopier Uplift elements can provide allowable uplift resistances between 25 to 40 tons depending on design depths (ultimate loads of 60 to 80 tons).

For preliminary design purposes, Geopier elements must occupy a minimum of 30 percent of each footing plan area for spread footings and should not be centered more than 12 feet apart for strip footings. The licensed Geopier design-builder, Geopier Foundation Company of Northern California (GFCNCA), would design and install Geopier foundations for this project. The stiffness compatibility with portions of the foundation that are directly supported on Franciscan material should be evaluated for acceptability with the structural engineer and project architect.

We recommend that Geopier construction operations be monitored by AMEC on a full-time basis as a Quality Assurance service, supplemented by a Geopier internal QC program. Together, the QA/QC program will monitor drill depths, Geopier shaft lengths, average lift thicknesses, installation procedures, aggregate quality, and densification of lifts. These items will be documented for each Geopier element installed to provide a complete installation report.

6.4.7 Drilled Piers

Drilled, cast-in-place piers should be designed to develop their vertical load carrying capacity through friction between the sides of the pier and the surrounding subsurface materials. Friction piers should have a minimum diameter of 24 inches, and there should be at least 3 diameters of soil between adjacent piers.

The piers should generally extend to a depth adequate to provide at least 10 feet of embedment into Franciscan material. For preliminary design the piers should be designed using an allowable skin friction values of 800 pounds per square foot (psf) for dead plus live loads and 1200 psf for all loads including wind or seismic.

For bridge foundations, we anticipate that the depth to competent material at the bridge locations could range between 10 and 20 feet; however, these depths would have to be confirmed by subsequent field exploration and may be deeper. Therefore, the piers should generally extend to a minimum depth of about 20 to 30 feet below the existing ground surface. The recommended design values above can be used starting at the depth where competent material is encountered. Because the anticipated loose to medium dense alluvium near the bridge abutments and piers may liquefy and/or densify and settle, this material should not be counted on to provide support to bridge foundation piers.

Lateral loads on the piers may be resisted by passive pressures acting against the sides of the piers. The allowable passive pressures equal to an equivalent fluid are presented in Section 6.3.4. These values can be assumed to be acting against two times the diameter of the individual pier shafts starting at the depth where competent soil or Franciscan material is encountered.

The satisfactory performance of cast-in-place piers depends on proper construction as well as proper design. Care must be taken during the drilling to prevent disturbing the foundation material that will surround the pier. Equipment or methods used for drilling should not cause quick soil conditions or cause scouring or caving of the shaft. After drilling, the pier should be constructed expeditiously in order to prevent deterioration of the surrounding foundation material from exposure to air or from the presence of water.

The length of the piers probably will have to be adjusted in the field at the time of drilling to account for variable site conditions. During construction, the soil encountered in pier shafts should be logged and the conditions of pier holes should be reviewed and documented before concrete is poured. The bottoms of pier holes should be dry and reasonably free of loose cuttings, slough, or debris before reinforcing steel is installed and concrete placed. If minor caving occurs, the loss of depth can be compensated for by increasing the lengths of the piers.

Steel casings may be required if the shafts cave. To minimize caving, concrete should be poured the same day the pier hole is drilled. If a casing is required, it should be withdrawn while placing concrete in the pier hole. The casing should be removed slowly so that the level of concrete at all times is at least 3 feet above the bottom of the casing.

The concrete of the pier should be dense and homogeneous. The methods used to place the concrete should prevent segregation. Concrete placed in dry, or dewatered drilled holes should not be permitted to fall from a height greater than 5 feet without the use of adjustable length pipes or tubes. Concrete should be vibrated into place.

If pier excavations are dry, concrete may be placed by discharging the mix into a hopper and connecting a rigid “elephant trunk” that directs the concrete down the pier shaft. It is important that falling concrete does not strike the sides of the shaft or the reinforcing steel cage, which would cause the concrete materials to segregate.

Groundwater may be encountered in pier holes and the potential for encountering groundwater will be increased if pier holes are drilled in the winter or spring. If possible, construction of drilled pier foundations should be undertaken in the dry summer and fall months to reduce the caving associated with groundwater and surface water (rain) inflows and to negate the special considerations required for placing concrete in water.

If groundwater is encountered in the pier shaft and cannot be pumped out to provide a dry excavation, the concrete should be placed by the tremie method, which involves pumping or pouring concrete through a long pipe to the bottom of the shaft. The end of the tremie pipe should be plugged so that water does not enter the pipe. Also, the bottom of the pipe should be embedded at least 3 feet into fresh concrete throughout the pour.

6.5 SLABS-ON-GRADE

Concrete floor slabs should be supported on a minimum of 6 inches of scarified and recompacted Franciscan material at discretion of the project geotechnical engineer, or a minimum of 18 inches re-compacted select fill placed on a prepared subgrade. The select fill should extend at least 5 feet beyond the perimeter of the building. In areas where dampness

of the floor slab would be undesirable, a layer of open-graded gravel, at least 4 inches thick, should be placed below the concrete slab to form a capillary break. A moisture-proof membrane should be installed over the gravel layer; at the discretion of the structural engineer, this membrane may be covered with 2 inches of sand to protect the membrane from damage during construction. The gravel and sand may be considered as the upper portion of select fill under the floor slabs. Gravel for use under the concrete floor slab should be clean, crushed rock meeting the following grading requirements:

Sieve Size	Percentage Passing Sieve
1 inch	100
3/4 inch	90-100
No. 4	0-10

Exterior slabs and sidewalks should be supported on a minimum of 6 inches of gravel or compacted aggregate base material. Control joints should be installed to control cracking of the slabs and walkways.

Use of a subdrain system may be appropriate in order to remove possible groundwater from under the slab. Details of the underdrain system, if considered necessary based on the final geotechnical exploration, will be provided in the final geotechnical report.

6.6 WINE CAVE

A wine cave is planned to extend through an existing ridge, from near the southwest corner of the main building and extending until it daylights on the southwest-facing hillside, as shown on Drawing C1.1. Tunneling methods and permanent support criteria should be developed following detailed site characterization along the length of the proposed alignment.

6.7 WATER TANKS

A pair of water tanks will be constructed to provide fire protection water to the project. These tanks will be founded on a graded pad located uphill to the north of the main buildings. Cut and fill operations should be performed in accordance with preliminary grading recommendations presented in Section 6.1 above. If retaining structures are needed, they should be developed in accordance with preliminary recommendations presented in Section 6.3 above. We anticipate that the tanks may be supported on shallow ringwall foundations, but this should be further evaluated during final design based on site specific data.

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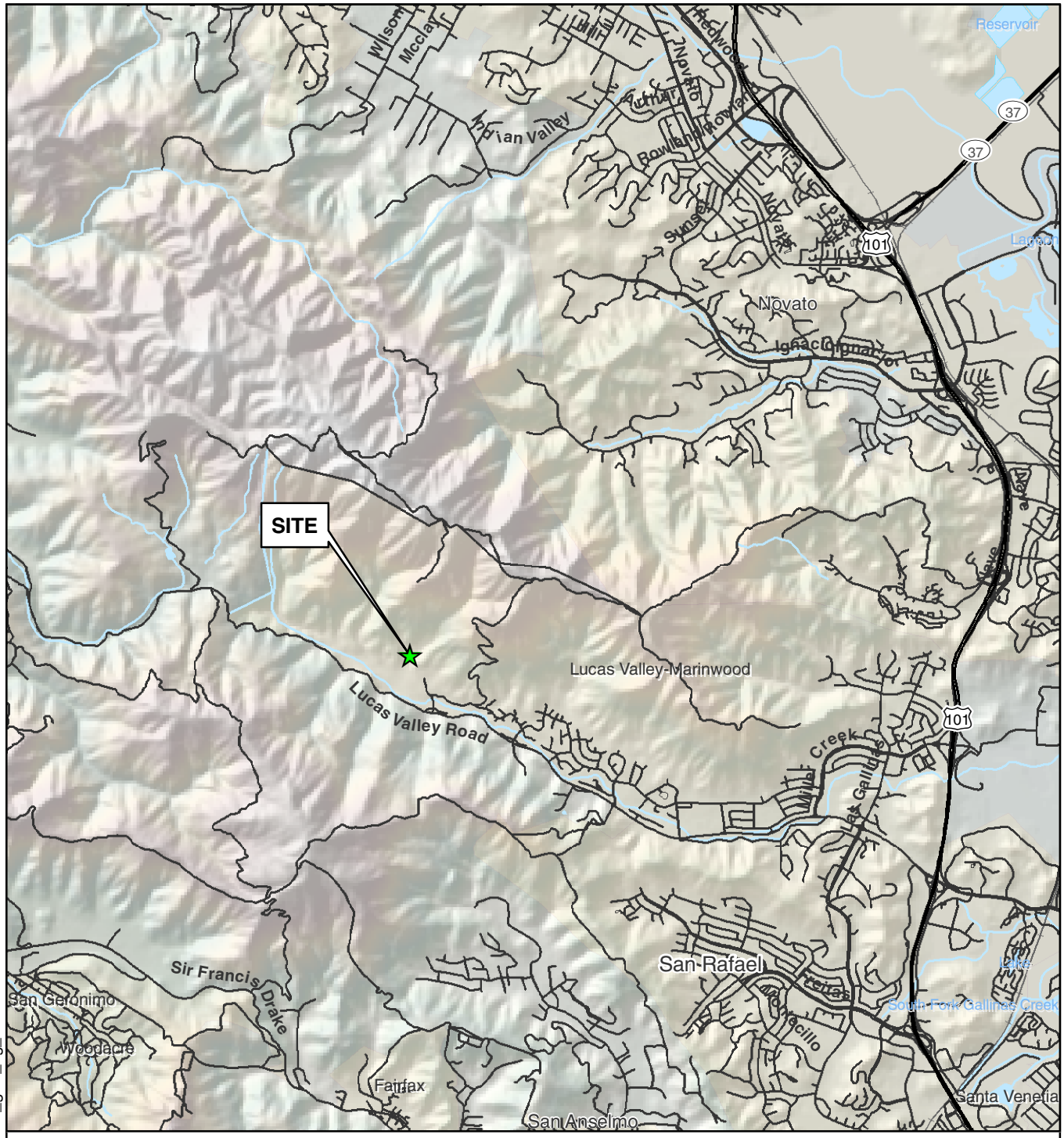
FIGURES



Explanation

Site location

N

0 2,000 4,000
Feet

SITE LOCATION MAP
Grady Ranch Precise Development Plan
Marin County, California

By: DMO	Date: 11/20/2008	Project No. 14648.000
AMEC Geomatrix		Figure 1

Shaded relief base derived from USGS 30m DEM.

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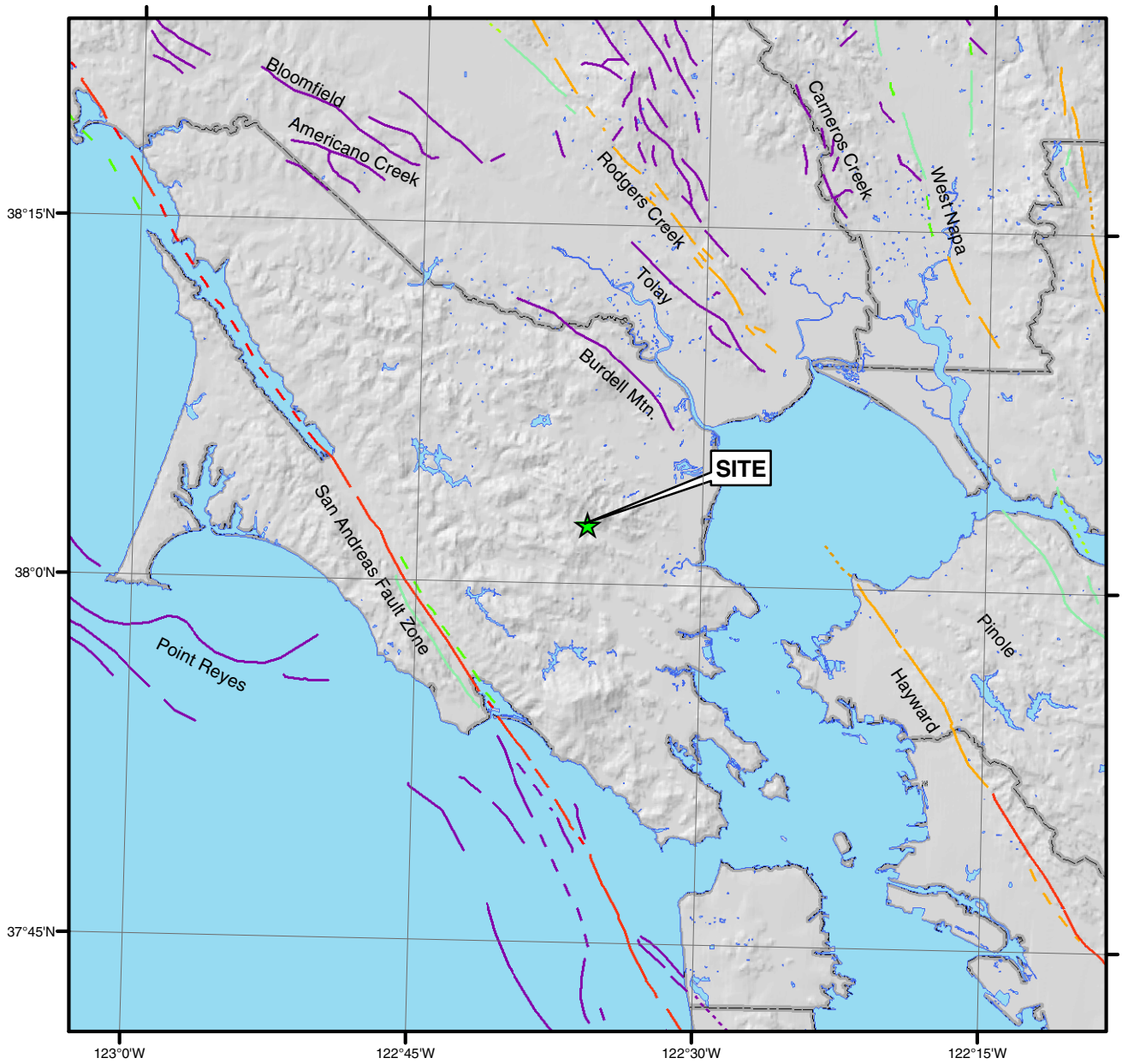

Explanation

Fault traces on land shown as solid where well located, dashed where uncertain or inferred, dotted where buried. Data from Jennings (1994).

- Faults with historic surface rupture/creep
- Faults that displace Holocene (~11 ka) or latest Pleistocene (~20 Ka) deposits or geomorphic surfaces.
- Faults that displace Late Quaternary (~ 780 ka) deposits or geomorphic surfaces.
- Quaternary faults, undifferentiated (1.8 Ma).

N

0 5 10 Miles

0 10 20 Kilometers

REGIONAL FAULT MAP
Grady Ranch Precise Development Plan
Marin County, California

By: DMO	Date: 11/14/2008	Project No. 14648.000
AMEC Geomatrix		Figure 2

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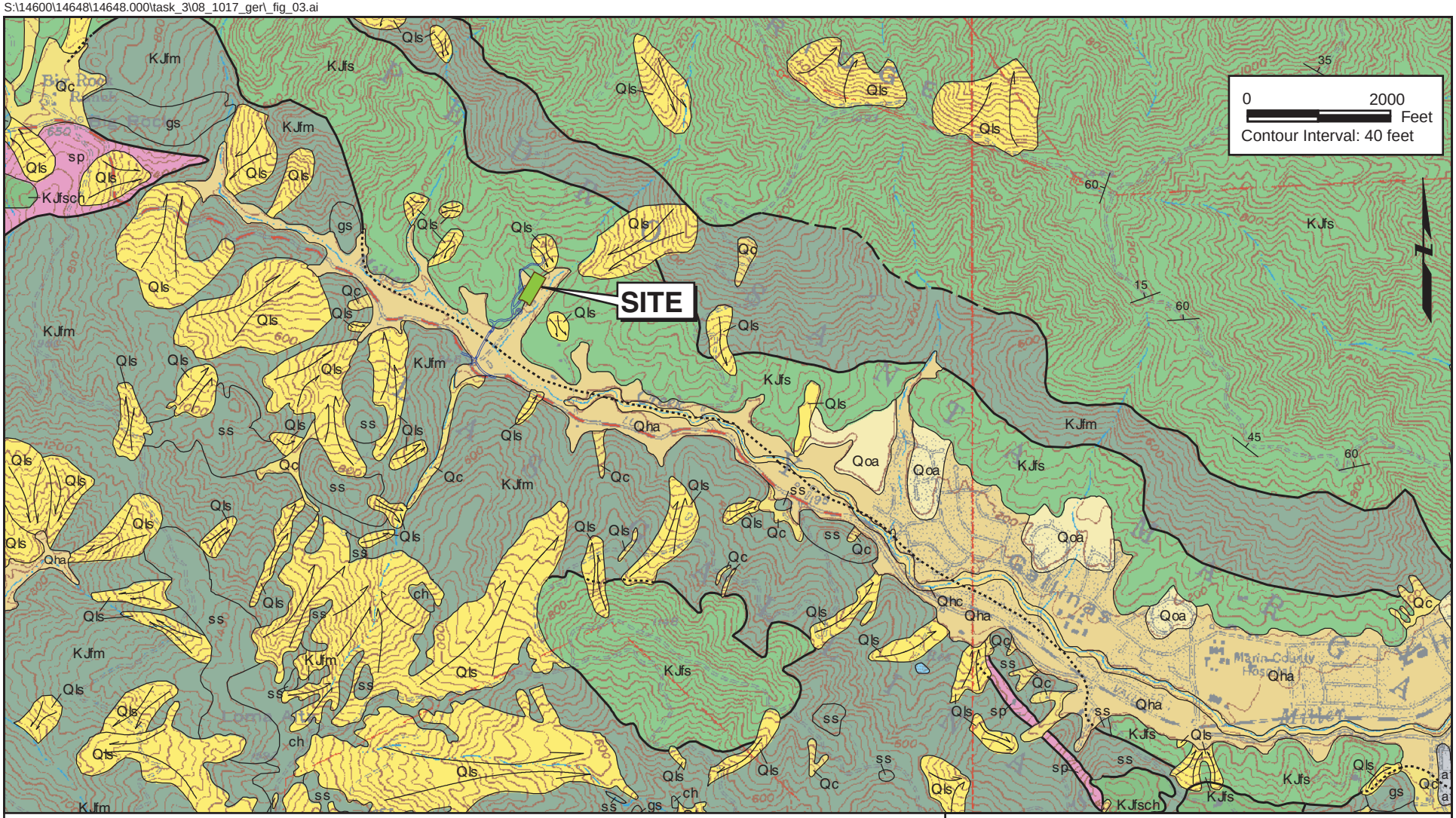


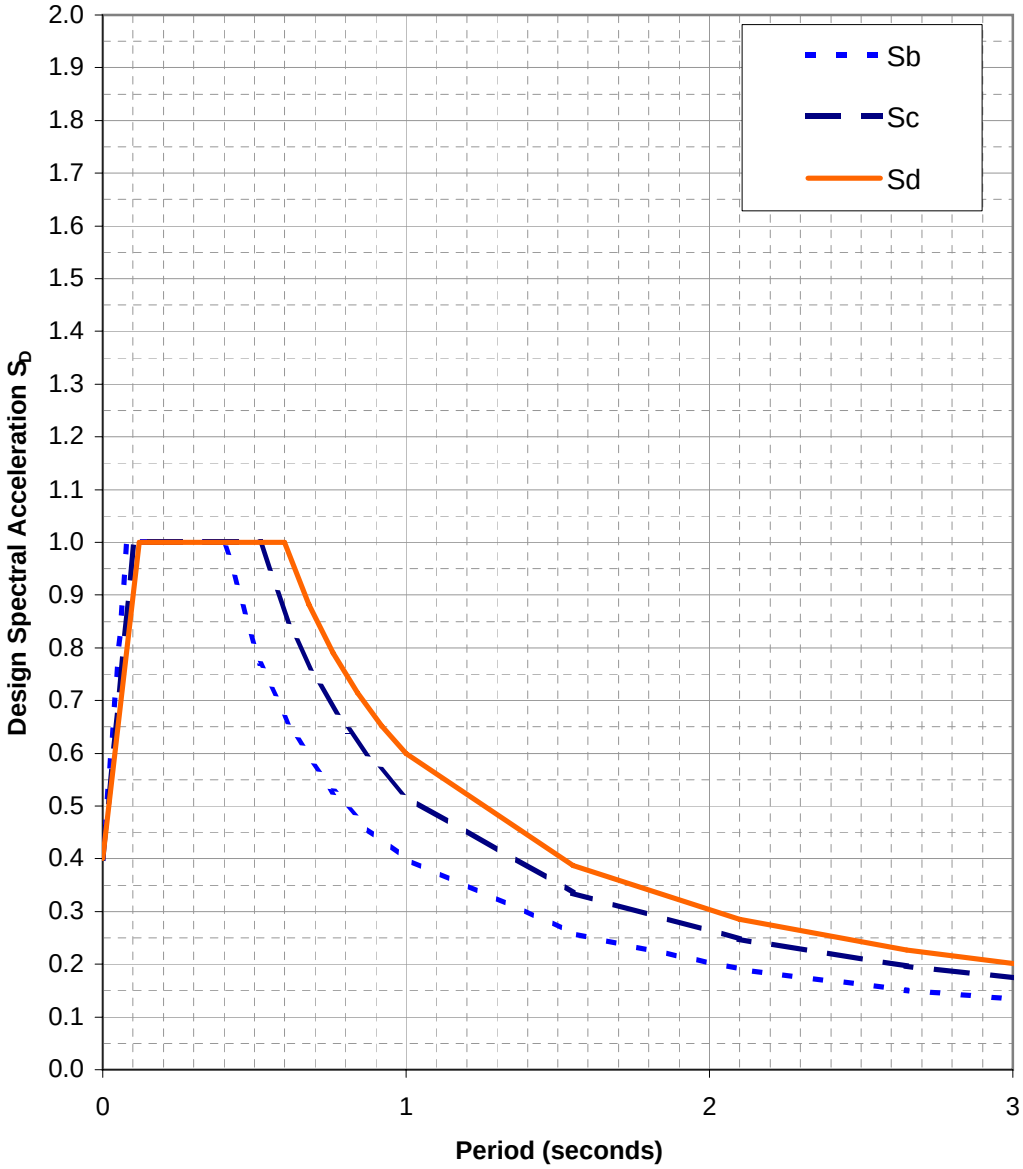
Explanation of Geologic Units

Qha	Holocene alluvium, undivided.
Qoa	Early to late Pleistocene deposits, undivided.
Qc	Colluvium.
Qls	Landslides.
KJfs	Franciscan Complex sandstone and shale.
KJfm	Franciscan Complex melange. Blocks within the melange are: ss, sandstone and shale; ch, chert; gs, greenstone.
KJfsch	Franciscan Complex schist, phyllite, and semischist
sp	Serpentinized ultramafic rock

Base: Rice et al. (2002)

REGIONAL GEOLOGIC MAP
Grady Ranch Precise Development Plan
Marin County, California

By: DMO	Date: 11/13/2008	Project No. 14846.000
AMEC Geomatrix		Figure 3



2007 CBC: Ss = 1.5; S1 = 0.6; Site Class Sb; TL = 12 sec; PGA= 0.4
2007 CBC: Ss = 1.5; S1 = 0.6; Site Class Sc; TL = 12 sec; PGA= 0.4
2007 CBC: Ss = 1.5; S1 = 0.6; Site Class Sd; TL = 12 sec; PGA= 0.4

2007 CBC DESIGN SPECTRA

Grady Ranch Precise Development Plan

FIGURE

4

AMEC Geomatrix

Marin County, California

JOB NUMBER	DRAWN	APPROVED	DATE	REVISED DATE
14648.000	JF		Nov 2008	

DRAWING C1.1

Site Geology, Cross Sections, & Slope Stabilization Plan (1 of 2)

DRAWING C1.2

Site Geology, Cross Sections, & Slope Stabilization Plan (2 of 2)

DRAWING C1.3

Geologic and Exploration Map

APPENDIX A

Boring and Test Pit Logs from
Geotechnical Reconnaissance
Lucasfllm/Grady Ranch
Lucas Valley Road
Marin County, California
by Harlan Tait, July 27, 1993

SOIL CLASSIFICATION CHART (ASTM D 2487)

MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES
COARSE-GRAINED SOILS MORE THAN 50% RETAINED ON NO. 200 SIEVE*	GRAVELS 50% OR MORE OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS	GW	WELL-GRADED GRAVELS AND GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
			GP	POORLY GRADED GRAVELS AND GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
		GRAVEL WITH FINES	GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES
			GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
	SANDS MORE THAN 50% OF COARSE FRACTION PASSES NO. 4 SIEVE	CLEAN SANDS	SW	WELL-GRADED SANDS AND GRAVELLY SANDS, LITTLE OR NO FINES
			SP	POORLY GRADED SANDS AND GRAVELLY SANDS, LITTLE OR NO FINES
		SANDS WITH FINES	SM	SILTY SANDS, SAND-SILT MIXTURES
			SC	CLAYEY SANDS, SAND-CLAY MIXTURES
FINE-GRAINED SOILS 50% OR MORE PASSES NO. 200 SIEVE*	SILTS AND CLAYS LIQUID LIMIT 50% OR LESS		ML	INORGANIC SILTS, VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
			OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50%		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDS OR SILTS, ELASTIC CLAYS
			CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY
HIGHLY ORGANIC SOILS			PT	PEAT, MUCK, AND OTHER HIGHLY ORGANIC SOILS

* Based on the material passing the 3-inch (75mm) sieve.

KEY TO TEST DATA AND LOGS

NOTATION	TEST TYPE
UC	UNCONFINED COMPRESSION
TX/UU	TRIAXIAL/UNCONSOLIDATED UNDRAINED
TX/CU	TRIAXIAL/CONSOLIDATED UNDRAINED
TX/CD	TRIAXIAL/CONSOLIDATED DRAINED
DS/UU	DIRECT SHEAR/UNCONSOLIDATED UNDRAINED
DS/CU	DIRECT SHEAR/CONSOLIDATED UNDRAINED
DS/CD	DIRECT SHEAR/CONSOLIDATED DRAINED
TV	TORVANE SHEAR
PS	PARTICLE SIZE ANALYSIS
LL	LIQUID LIMIT
PI	PLASTICITY INDEX
SG	SPECIFIC GRAVITY
CONSOL	CONSOLIDATION
PERM	PERMEABILITY

SYMBOL	DESCRIPTION
■	MODIFIED CALIFORNIA SAMPLER (3" O.D.): BLOW COUNT IS EQUIVALENT PENETRATION RESISTANCE, N., CONVERTED ACCORDING TO SOWERS, 1954.
◧	STANDARD PENETRATION TEST, ASTM D 1586: BLOW COUNT IS PENETRATION RESISTANCE, N.
◪	THIN-WALLED TUBE SAMPLE, ASTM D 1587
☒	DISTURBED (BULK) SAMPLE
▽	WATER LEVEL

STRENGTH TEST RESULTS* ARE INDICATED IN PSF AFTER THE TEST NOTATION. CONFINING OR NORMAL STRESS IS INDICATED IN PARENTHESES. TESTS ARE PERFORMED AT FIELD MOISTURE UNLESS OTHERWISE NOTED.

* UC: ONE HALF THE COMPRESSIVE STRENGTH
TX: ONE HALF THE PRINCIPAL STRESS DIFFERENCE
DS,TV: SHEAR STRENGTH

Harlan Miller Tait

SOIL CLASSIFICATION CHART/KEY TO DATA
ILM/GRADY RANCH
Marin County, California

Figure A1

Proj. No. 740.02 Date 7/7/88 App'd by

BORING 1

Equipment 6" diameter auger

Elevation 235' ± Date 12/12/84

Laboratory Tests	Blows/foot	Moisture Content (%)	Dry Density (pcf)	Depth (ft)	Sample	
	5			0	SM	DARK BROWN SILTY SAND - loose, damp, scatted rock fragments to 1" diameter, some organics and roots. (topsoil) -thin (1") sandy lenses.
	5			5		
	9				GM	YELLOW RED BROWN SILTY GRAVEL- medium dense, moist, subrounded gravels to 2" diameter, slight layering. (alluvium)
	16			10		-some clay and wet around gravels from 10-11.5'.
	6			15	SC	BROWN CLAYEY SAND - loose, saturated, layered, scattered subround and round hard fine gravels. (alluvium)
	34/6"			20	me	DARK BROWN GRAY MELANGE - moderately weathered, slightly hard (very hard consistency), moist, sheared, (hard fragments in a slickensided shale matrix). (bedrock)
				25		BOTTOM OF HOLE - 20.5'. -Water encountered at 13' ±. open tube piezometer installed to 19.3'.
				30		
				35		

Harlan Miller Tait

LOG OF BORING 1
GRADY RANCH
Marin County, California

Proj. No. 587.1 Date 12/14/84 App'd by

Figure A2

BORING 2

Equipment 6" diameter auger

Elevation 250' ± Date 12/12/84

Laboratory Tests	Blows/foot	Moisture Content (%)	Dry Density (pcf)	Depth (ft)	Sample	
	7			0	SM	DARK BROWN SILTY SAND - loose, moist, scattered roots, some organics. (topsoil) -scattered rootlet voids. -a few fine gravels.
	7			5		
	22			10	SC	BROWN CLAYEY SAND - medium dense, moist, mottled, a few fine gravels, slightly layered. (alluvium)
	34/6"			15	SC SS	YELLOW BROWN CLAYEY SAND - very dense, damp, scattered sandstone fragments. (colluvium) YELLOW BROWN SANDSTONE - fine grained, completely weathered, slightly hard (very hard consistency), extremely fractured. (bedrock) BOTTOM OF HOLE - 13.5'. -No water encountered. -hole backfilled.

Harlan Miller Tait

LOG OF BORING 2
GRADY RANCH
Marin County, California

Proj. No. 587.1 Date 12/14/84 App'd by

Figure A3

BORING 3

Equipment 6" solid stem auger

Elevation 243' ± Date 12/28/84

Laboratory Tests	Blows/foot	Moisture Content (%)	Dry Density (pcf)	Depth (ft)	Sample	
				0	CL	DARK BROWN SANDY CLAY - stiff, moist, scattered fine gravels, some organics. (colluvium ?)
	16					
				5		-change to damp and increase to very stiff, with scattered angular decomposed and a few moderately hard sandstone fragments below 5'.
	24				GC	BROWN CLAYEY GRAVEL - dense, moist, scattered angular fine gravels of sandstone. (colluvium)
	29			10		-a few coarse gravels below 9'.
					ss	BROWN SANDSTONE - fine grained, moderately weathered, moderately hard, moderately fractured, dry.
	34/3"			15		-change to brown gray, hard at 14' ±.
						-closely fractured from 17-17.5'.
	50/1"			20		-change to slightly weathered, hard, slightly fractured at 20'.
				25		-seam, approximately ½" assume closely fractured and weathered (recognized by increased drill rate) at 26'.
				30		BOTTOM OF HOLE - 29.5'. -Slope indicator installed to 29.0'. -Water encountered at 0.0' - 12/29/84 4.9' - 1/8/85
				35		

Harlan Miller Tait

LOG OF BORING 3
GRADY RANCH
Marin County, California

Proj. No. 587.1 Date 1/21/85 App'd by

Figure A4

BORING 4

Equipment 8" hollow stem auger

Elevation 250' ± Date 12/29/84

Laboratory Tests	Blows/foot	Moisture Content (%)	Dry Density (pcf)	Depth (ft)	Sample	Description
				0	GP	BROWN SANDY GRAVEL - medium dense, moist. (fill)
	10				SC	BROWN BLACK CLAYEY SAND - medium dense, moist, some organics. (topsoil)
	11			5	GC	YELLOW BROWN CLAYEY GRAVEL - medium dense, damp, scattered angular fine gravels. (colluvium)
	13				GC	RED BROWN CLAYEY GRAVEL - medium dense, moist, mottled, layered, rounded fine gravels. (alluvium)
				10	GP	BROWN SANDY GRAVEL - loose, saturated, rounded fine gravels. (alluvium)
	5					
				15		
	6					
				20		
					ss	BROWN SANDSTONE - fine grained, highly weathered, slightly hard, extremely fractured. -change to closely fractured at 23'. -change to gray brown, moderately weathered, moderately hard at 25'.
				25		BOTTOM OF HOLE - 25'. -Water encountered at 9'. -Observation well installed to 24.5'.
				30		
				35		

Harlan Miller Tait

LOG OF BORING 4
GRADY RANCH
Marin County, California

Proj. No. 587.1 Date 1/21/85 App'd by

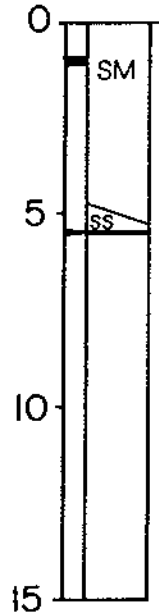
Figure A5

Laboratory Tests	Blows/foot	Moisture Content (%)	Dry Density (pcf)	Depth (ft)	Sample

BACKHOE PIT 1

Equipment: Extendahoe (24" bucket)

Elevation: 257 ± Date: 1/8/85



BROWN SILTY SAND - loose, moist, some organics in top 0.5'. (colluvium)
-change to medium dense at 1'.

-change to mottled, dense with scattered sandstone fragments at 3'.

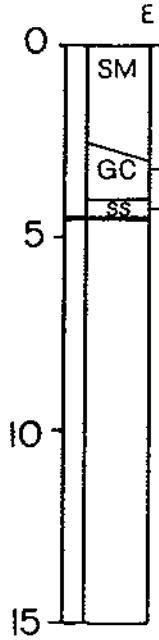
YELLOW BROWN SANDSTONE - fine grained, completely weathered, soft rock (very dense), extremely fractured, molds to clayey gravel with finger pressure.
-change to highly weathered, slightly hard at 5.5'.

BOTTOM OF PIT - 5.5'.
-no water encountered.
-backfilled.

BACKHOE PIT 2

Equipment: Extendahoe (24" bucket)

Elevation: 286 ± Date: 1/8/85



BROWN SILTY SAND - loose, moist, scattered roots, some organics to 1'. (landslide deposit)
-change to mottled yellow brown, medium dense with scattered angular sandstone fragments to 1.5' diameter.

BROWN CLAYEY GRAVEL - dense, moist, angular sandstone fragments to 1' diameter. (landslide deposit)

BROWN SANDSTONE - fine grained, highly weathered, moderately hard, moderately fractured.

BOTTOM OF PIT - 4.5'.
-some seepage at 4.0'.
-backfilled.

Harlan Miller Tait

LOG OF BACKHOE PITS 1 & 2
GRADY RANCH
Marin County, California

Figure A6

Proj. No. 587.1 Date 1/18/85 App'd by

LOG OF TEST PIT

EQUIPMENT

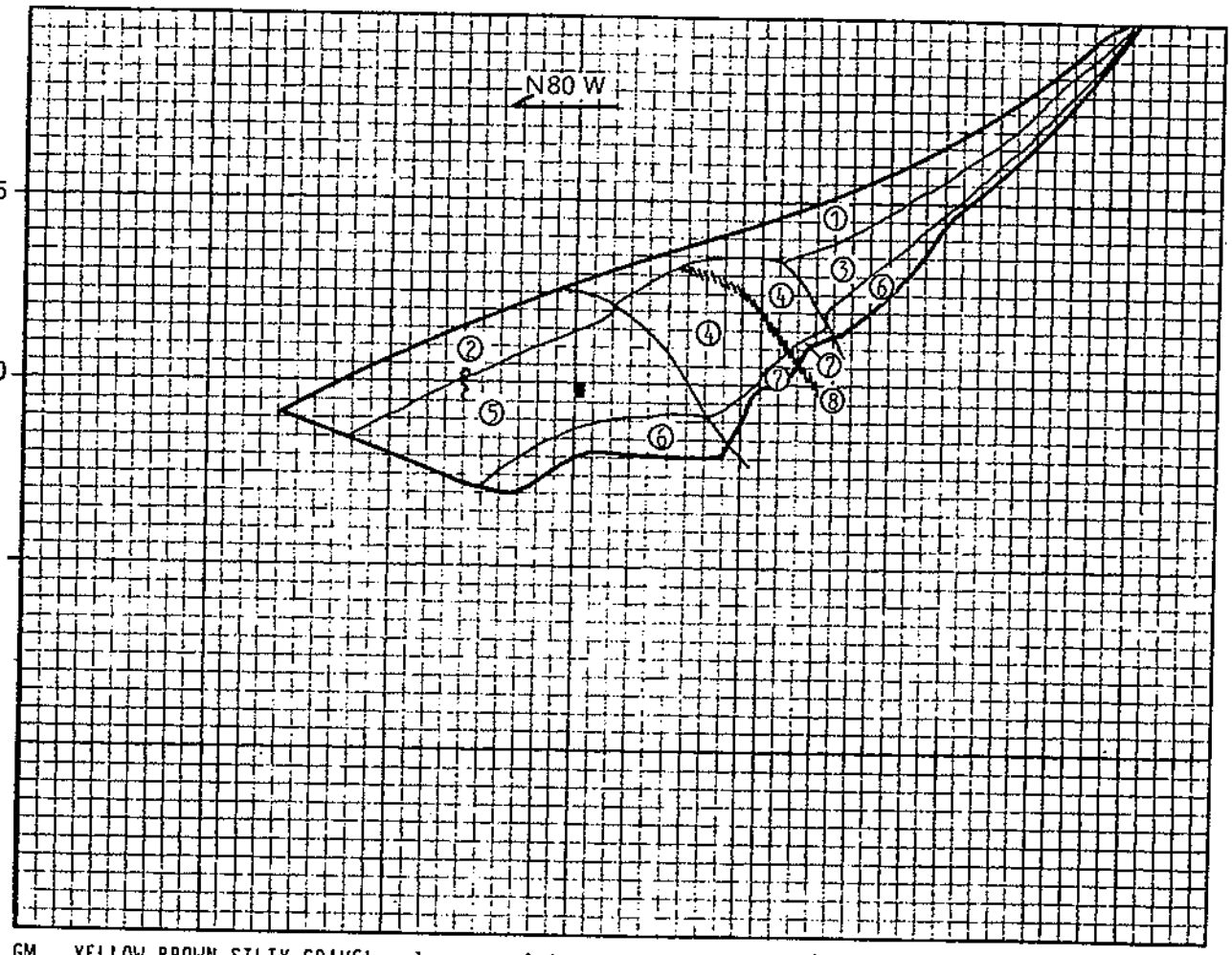
This summary applies only at the location of this test pit at the time of excavation. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

DEPTH IN FEET

ELEVATION 310' ±

DATE 1/8/85

SCALE: 1" = 5'



① GM YELLOW BROWN SILTY GRAVEL - loose, moist, scattered roots. (landslide deposit)
② SM BROWN SILTY SAND - loose, moist, local seepage, roots, some organics. (landslide deposit)
③ GC YELLOW BROWN CLAYEY GRAVEL - dense, moist. (colluvium)
④ GC DARK BLACK BROWN CLAYEY GRAVEL - dense, moist, angular sandstone fragments to 6" dia. (colluvium)
⑤ SC YELLOW BROWN CLAYEY SAND - dense, moist, scattered angular sandstone fragments. (colluvium)
⑥ ss BROWN SANDSTONE - fine grained, completely weathered, soft rock, extremely fractured - increase to highly weathered, moderately hard at bottom of pit.
⑦ me BLACK BROWN MELANGE - moderately weathered, soft rock, sheared and extremely fractured.
⑧ CH BROWN CLAY - medium stiff, wet, 1/2" seam N30 E65 SE into hill.

Harlan Miller Tait

LOG OF BACKHOE PIT BP-3

GRADY RANCH

Marin County, California

Proj. No. 587.1 Date 1/21/85 App'd by

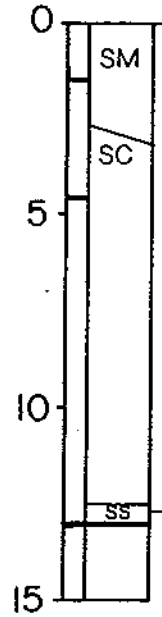
Figure A7

Laboratory Tests	Blows/foot	Moisture Content (%)	Dry Density (pcf)	Depth (ft)	Sample

BACKHOE PIT 4

Equipment: Extendahoe (24" bucket)

Elevation: 315 ± Date: 1/8/85



BROWN SILTY SAND - loose, wet, some organics, scattered roots. (landslide deposit)

YELLOW BROWN CLAYEY SAND- medium dense, moist, mottled, a few sandstone fragments (soft). (colluvium/ landslide deposit ?)

-change to dense at 6' - possible base of landslide deposit.

-gradual change to very dense at approximately 11'.

BROWN SANDSTONE - fine grained, highly weathered, slightly hard, closely fractured.

BOTTOM OF PIT - 13'.

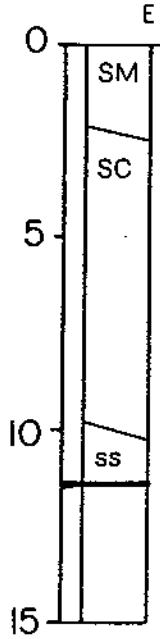
-no water encountered.

-backfilled.

BACKHOE PIT 5

Equipment: Extendahoe (24" bucket)

Elevation: 282 ± Date: 1/8/85



BROWN SILTY SAND - loose, wet, some organics. (landslide deposit)

YELLOW BROWN CLAYEY SAND - loose, moist, mottled. (colluvium/ landslide deposit ?)

-change to medium dense at 3'.

-change to dense at 6.5' - possible base of landslide deposit.

-scattered angular, highly weathered sandstone fragments at 9'.

BROWN SANDSTONE - fine grained, highly weathered, soft rock, exremely fractured.

-change to moderately weathered, slightly hard at 11'.

BOTTOM OF PIT - 11'.

-seepage encountered at 6.5'.

-backfilled.

Harlan Miller Tait

LOG OF BACKHOE PITS 4 & 5
GRADY RANCH
Marin County, California

Proj. No. 587.1 Date 1/18/85 App'd by

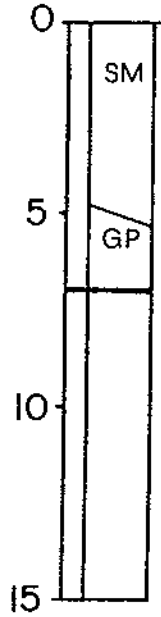
Figure A8

Laboratory Tests	Blows/foot	Moisture Content (%)	Dry Density (pcf)	Depth (ft)	Sample

BACKHOE PIT 6

Equipment: Extendahoe (24" bucket)

Elevation: 279 ± Date: 1/8/85



BROWN SILTY SAND - medium dense, moist, roots, organics, scattered angular rock fragments to 1' diameter. (colluvium with some alluvium intermixed)

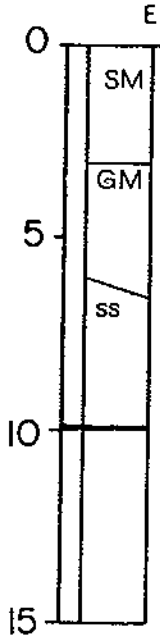
BROWN GRAVEL - loose, saturated, angular and subrounded gravels to 6" diameter. (alluvium with some colluvium intermixed)

BOTTOM OF PIT - 7.0'.
- -water encountered at 6.0'.
- -backfilled.

BACKHOE PIT 7

Equipment: Extendahoe (24" bucket)

Elevation: 289 ± Date: 1/8/85



BROWN SILTY SAND - loose, moist, roots, organics. (colluvium)
- -change to medium dense, scattered angular sandstone fragments at 2.5'.

LIGHT BROWN SILTY GRAVEL - dense, moist, a few roots, angular sandstone fragments to 6" diameter. (colluvium)

BROWN SANDSTONE - fine grained, completely weathered, soft rock, extremely fractured.
- -change to highly weathered, slightly hard at 8'.
- -change to moderately hard, closely fractured at 10'.

BOTTOM OF PIT - 10'.
- -no water encountered.
- -backfilled.

Harlan Miller Tait

LOG OF BACKHOE PITS 6 & 7
GRADY RANCH
Marin County, California

Proj. No. 587.1 Date 1/18/85 App'd by

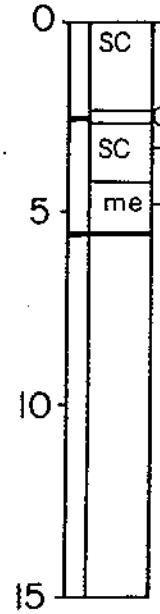
Figure A9

Laboratory Tests	Blows/foot	Moisture Content (%)	Dry Density (pcf)	Depth (ft)	Sample

BACKHOE PIT 8

Equipment: Extendahoe (24" bucket)

Elevation: 284 ± Date: 1/8/85



BROWN CLAYEY SAND - medium dense, moist, roots, organics. (landslide deposit)

BLUE GRAY SANDY CLAY - medium stiff, wet, scattered roots - slide plane ≅ 3" thick. (landslide plane)

BROWN CLAYEY SAND - very dense, moist, a few roots. (colluvium/ decomposed melange ?)

DARK GRAY MELANGE - slightly weathered, slightly hard, sheared and extremely fractured.

BOTTOM OF PIT - 5.5'.

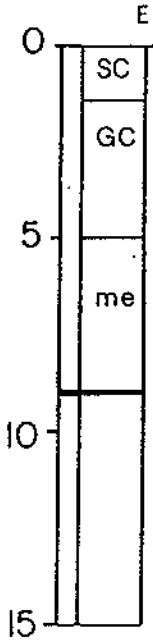
-no water encountered.

-backfilled.

BACKHOE PIT 9

Equipment: Extendahoe (24" bucket)

Elevation: 330 ± Date: 1/9/85



DARK GRAY CLAYEY SAND - loose, wet, organics. (landslide deposit)

BROWN CLAYEY GRAVEL - medium dense, wet, angular gravels (greenstone ?) to 6" diameter. (landslide deposit)

NOTE: Hole caves above 5'.
Slide plane not observed at 5'.

GRAY BROWN MELANGE - completely weathered, soft rock, sheared and extremely fractured, moist, molds to clayey sand.

-change to slightly weathered at 9' with large (6'+ diameter) hard boulder, dry.

BOTTOM OF PIT - 9'.

-seepage encountered at 5'.

-backfilled.

Harlan Miller Tait

LOG OF BACKHOE PITS 8 & 9
GRADY RANCH
Marin County, California

Proj. No. 587.1 Date 1/18/85 App'd by

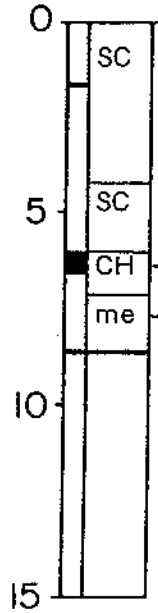
Figure A10

Laboratory Tests	Blows/foot	Moisture Content (%)	Dry Density (pcf)	Depth (ft)	Sample

BACKHOE PIT 10

Equipment: Extendahoe (24" bucket)

Elevation: 350 ± Date: 1/9/85



DARK GRAY BROWN CLAYEY SAND - loose, wet, roots, organics, scattered fine gravels. (landslide deposit)

YELLOW BROWN CLAYEY SAND - medium stiff, wet, mottled, scattered fine gravels. (landslide deposit)

GRAY CLAY - very stiff, moist, slickensides dip 15-20° downslope, thickness of clay varies 0.1-1.0'. (landslide plane)

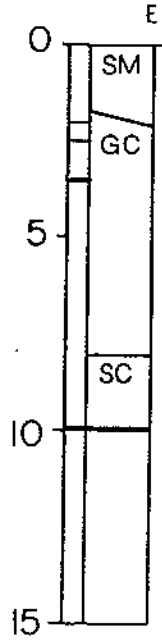
GRAY BROWN MELANGE - highly weathered, soft rock, sheared and extremely fractured.
-moderately hard and dry at 8.5'.

BOTTOM OF PIT - 8.5'.
-seepage encountered at 5.5'.
-backfilled.

BACKHOE PIT 11

Equipment: Extendahoe (24" bucket)

Elevation: 259 ± Date: 1/9/85



BROWN SILTY SAND - medium dense, moist, some organics. (topsoil)

YELLOW RED BROWN CLAYEY SAND - dense, moist, subangular to rounded gravels to 4" diameter. (terrace deposit)
-change to very dense at approximately 5'.

YELLOW BROWN CLAYEY SAND - very dense, damp. (residual soil/ completely weathered sandstone ?)

BOTTOM OF PIT - 10'.
-no water encountered.
-backfilled.

Harlan Miller Tait

LOG OF BACKHOE PITS 10 & 11
GRADY RANCH
Marin County, California

Proj. No. 587.1 Date 1/18/85 App'd by

Figure A11