GEOTECHNICAL INVESTIGATION
COLLEGE OF MARIN
INDIAN VALLEY CAMPUS
POMO TRANSPORTATION TECHNOLOGY COMPLEX
NOVATO, CALIFORNIA

September 11, 2007

Project 739.12

Prepared For:
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CERTIFICATION

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GEOTEchnical investigation
College of Marin
Indian Valley Campus
Pomo Transportation Technology Complex
Novato, California

I. Introduction

This report summarizes our geotechnical investigation regarding the planned structure to connect two existing buildings of the Pomo Transportation Technology Center at the College of Marin, Indian Valley Campus (IVC) in Novato, California. The purpose of our geotechnical investigation is to evaluate site conditions and provide geotechnical recommendations for the design and construction of the project. This report is intended for the exclusive use of the College of Marin, CSW/Stuber-Stroeh Engineering Group and the project design team for this project and specific site. No other use is authorized without our written consent.

The scope of our geotechnical investigation is outlined in our Agreement between Consultant and Subconsultant dated August 10, 2007 and includes the following:

- Review of existing geotechnical and geologic data,
- Description of subsurface conditions based on exploration with 2 soil borings,
- Geologic hazards evaluation and recommended mitigation measures,
- Seismicity evaluation,
- Recommended foundation type and geotechnical design criteria,
- Concrete slab-on-grade recommendations,
- Criteria for site grading and trench backfill,
- Geotechnical drainage recommendations
- Presentation of our investigation in a geotechnical report.
II. PROJECT DESCRIPTION

The College of Marin - IVC Pomo Transportation Technology Center project includes a new structure connecting two existing buildings on the Indian Valley campus. The Indian Valley Campus is located at 1800 Ignacio Boulevard, Novato, Marin County, California. The site location and coordinates are presented on Figure 1, Site Map.

Detailed plans regarding the proposed construction have not been prepared, however it is our understanding the project includes constructing a one-story, wood frame, approximately 2,000 square foot office and connector structure with light to moderate foundations loads. We anticipate other ancillary items such as concrete flat work and utilities will be constructed. Minor site grading is anticipated.

The design team includes the College of Marin (Owner), HKIT Architects of Oakland, California, CSW/Stuber-Stroeh of Novato, California (Civil Engineer), and Bluestone Engineering of Walnut Creek, California (Structural Engineer).
III. SITE CONDITIONS

A. Regional Geology
The site is located within the Coast Ranges Geomorphic Province of California. The regional bedrock geology consists of complexly folded, faulted, sheared, and altered sedimentary, igneous, and metamorphic rock of the Franciscan Complex of Jurassic-Cretaceous age (65-190 million years ago). The Franciscan Complex is characterized by a diverse assemblage of greenstone, sandstone, shale, chert, and mélange, with lesser amounts of conglomerate, calc-silicate rock, schist and other metamorphic rocks.

The regional topography is characterized by northwest-southeast trending mountain ridges and intervening valleys that were formed by movement between the North American and the Pacific Plates. Continued deformation and erosion during the late Tertiary and Quaternary Age (the last several million years) formed the prominent Marin coastal ridges and the inland depression that is now the San Francisco Bay. The more recent seismic activity within the Coast Range Geomorphic Province is concentrated along the San Andreas Fault zone, a complex group of generally north to northwest trending faults.

Regional geologic mapping by the California Geological Survey (CGS 2002) indicates the site is underlain by Holocene age (present to 11,000 years ago) alluvial deposits of gravel, sand and silt that are poorly to moderately sorted (well-graded), and Franciscan Complex sandstone and shale primarily comprised of thick bedded, arkosic sandstone with interbeds of shale. The regional geology in the vicinity of the project site is shown on Figure 2.

B. Seismicity
Active Faults in the Region — The site is located within the seismically active San Francisco Bay Region and will therefore experience the effects of future earthquakes. Such earthquakes could occur on any of several active faults within the region. The active faults are classified into two types. Type A faults are capable of large magnitude earthquakes and have a high rate of seismic activity. Type B faults are capable of large magnitude earthquakes with a low rate of seismic activity or are smaller faults with a high rate of seismic activity. These faults are shown on the Active Fault Map, Figure 3.
Historic Fault Activity – Numerous earthquakes have occurred in the region within historic times. The results of our computer database search indicate that 57 earthquakes (Richter Magnitude 5.0 or larger) have occurred within 100 kilometers of the site area between 1735 and 2005. Using empirical attenuation relationships, the maximum historic ground acceleration (median peak) within the study area is approximately 0.18g. The five most significant historic earthquakes to affect the project site are summarized in Table A.

<table>
<thead>
<tr>
<th>Epicenter (Latitude, Longitude)</th>
<th>Historic Richter Magnitude</th>
<th>Year</th>
<th>Distance</th>
<th>Maximum Peak Site Acceleration</th>
</tr>
</thead>
<tbody>
<tr>
<td>37.70, -122.50</td>
<td>8.3</td>
<td>1906</td>
<td>42 km</td>
<td>0.18 g</td>
</tr>
<tr>
<td>38.20, -122.40</td>
<td>6.2</td>
<td>1898</td>
<td>20 km</td>
<td>0.14 g</td>
</tr>
<tr>
<td>37.80, -122.20</td>
<td>6.8</td>
<td>1836</td>
<td>44 km</td>
<td>0.10 g</td>
</tr>
<tr>
<td>37.60, -122.40</td>
<td>7.0</td>
<td>1838</td>
<td>54 km</td>
<td>0.09 g</td>
</tr>
<tr>
<td>37.70, -122.10</td>
<td>6.8</td>
<td>1868</td>
<td>59 km</td>
<td>0.08 g</td>
</tr>
</tbody>
</table>


The calculated site accelerations should only be considered as reasonable estimates. Many factors (soil conditions, orientation to the fault, etc.) can influence the actual ground surface accelerations. Significant deviations from the values presented are possible due to geotechnical and geologic variations from the typical conditions used in the empirical correlations.

Probability of Future Earthquakes – The historical records do not directly indicate either the maximum credible earthquake or the probability of such a future event. To evaluate earthquake probability in this region, the USGS has assembled a group of researchers into the "Working Group on California Earthquake Probabilities" to estimate the probabilities of earthquakes on active faults. Potential sources were analyzed considering fault geometry, geologic slip rates,
geodetic strain rates, historic activity, and micro-seismicity, to arrive at estimates of probabilities of earthquakes with a Moment Magnitude greater than 6.7 by 2032.

The probability studies focus on seven “fault systems” within the Bay Area. Fault systems are composed of different, interacting fault segments capable of producing earthquakes within the individual segment or in combination with other segments of the same fault system. The probabilities for the individual fault segments in the San Francisco Bay Area are presented on Figure 3.

In addition to the seven fault systems, the studies included probabilities of “background earthquakes.” These earthquakes are not associated with the identified fault systems and may occur on lesser faults (i.e., West Napa) or previously unknown faults (i.e., the 1989 Loma Prieta and 2000 Napa/Mt. Veeder Earthquake). When the probabilities on all seven fault systems and the background earthquakes are combined mathematically, there is a 62 percent chance for a magnitude 6.7 or larger earthquake to occur in the Bay Area by the year 2032. Smaller earthquakes (between magnitudes 6.0 and 6.7), capable of considerable damage depending on proximity to urban areas, have about an 80 percent chance of occurring in the Bay Area by 2032 (USGS, 2003).

Additional studies by the USGS regarding the probability of large earthquakes in the Bay Area are on going. These current evaluations include data from additional active faults and updated geological data.

C. Surface Conditions
The project site is currently developed as a community college. The college campus contains 1 and 2-story structures, asphalt walkways, and landscaped areas. The proposed building site is located between Auto Technology Buildings 1 and 2 in an existing landscape area. The existing site conditions are shown on Figure 4. Prior to subsurface exploration, underground service alert (USA) was notified in order to identify any active underground utilities. Sewer, storm drains, water, gas and electrical lines exist in the project vicinity. No other underground utilities were marked at the site. Additionally, we reviewed plans provided by COM that revealed the approximate location of existing utilities.
D. **Field Exploration and Laboratory Testing**

We explored subsurface conditions with two auger borings drilled with track mounted equipment on July 26, 2007 at the locations shown on Figure 4. The borings were logged by our field engineer and select soil samples were collected for laboratory testing to determine their pertinent engineering properties. The borings were drilled to depths between 35.5 and 45 feet below the ground surface. A soil classification chart and a rock classification chart are presented along with the boring logs on Figures A-1 through A-7. Soil samples tested in our laboratory to obtain pertinent engineering properties including moisture content, dry density, percent passing No. 200 sieve, particle size analysis (sieve analysis), and unconfined strength. The results of moisture content, dry density and unconfined strength tests are presented on the boring logs. The sieve analyses are presented on Figure A-8. Soil samples of the soils in the upper 5 were composited for corrosion tests. The corrosion test results are presented on Figure A-9. The subsurface exploration program is discussed in more detail and presented with the boring logs and laboratory testing results in Appendix A.

E. **Subsurface Conditions**

The subsurface conditions are generally consistent with the mapped geology (CGS 2002) and consistent of alluvial flood-plain materials, which are deposited as ancient rivers meandered across Pacheco Valley. Due to the variable nature of the alluvial soils, the determination of the various soil layers are based on similar soil type, consistency, and engineering properties. The subsurface conditions encountered in the borings consist of interbedded, variable medium dense clayey sand and stiff sandy clay underlain by Franciscan Complex sandstone and shale and shale mélange bedrock.

Boring 1 encountered loose, alluvial clayey sand in the upper 3 feet which grades to dense clayey sand with depth. At approximately 25 feet, the alluvial soils are predominately medium stiff to stiff sandy clay with occasional interbedded sand with gravel layers. Franciscan Complex sandstone and shale mélange was encountered at about 39 feet below the existing ground surface.

Boring 2 encountered a very stiff sandy clay fill layer that extends to a depth of about 5 feet. Underlying the fill layer is medium dense to dense, clayey sand (alluvial soils) that extends to a
depth of about 25 feet. Below the clayey sand is a very stiff clay layer that transitions into Franciscan Complex shale mélange at 30 feet. The interpreted subsurface conditions across the project site are shown of the geologic cross section presented on Figure 5.

Groundwater was observed in Boring 1 at approximately 25 feet below the ground surface and laboratory testing of Boring 2 samples at 25 feet indicate elevated moisture conditions. These observations may not represent the stabilized groundwater level. Groundwater levels will fluctuate with the seasons and may be nearer to the ground surface during periods of intense rainfall or for a period of time after significant rainfall.
IV. GEOLOGIC HAZARDS

A. General
The primary geologic hazards identified at the site are very strong seismic shaking, liquefaction, and seismically induced settlement. Other hazards, such as fault rupture, expansive soils or slope instability are not considered significant at the site. A brief description of each geologic hazard and, if needed, mitigation measures is listed in the following sections.

B. Fault Surface Rupture
Under the Alquist-Priolo Earthquake Fault Zoning Act\(^1\), the California Division of Mines and Geology (CDMG, now known as the California Geological Survey) produced 1:24,000 scale maps showing all known active faults and defining zones within which special fault studies are required. The project site is not located within an Alquist-Priolo Earthquake Fault Zone. The closest active faults are the San Andreas Fault located 11.1 miles (17.9 km) to the west and the Rodgers Creek fault located 7.1 miles (11.4 km) to the east. The potential for fault surface rupture is therefore remote.

*Evaluation:* No significant impact.
*Mitigation:* No mitigation measures are required.

C. Seismic Shaking
The site will likely experience seismic ground shaking similar to other areas in the seismically active San Francisco Bay Area. Earthquakes along several active faults in the region, as shown on Figure 3, could cause moderate to strong ground shaking at the site.

**Deterministic Seismic Hazard Analysis** – Deterministic Seismic Hazard Analysis (DSHA) predicts the intensity of earthquake ground motions by analyzing the characteristics of nearby faults, distance to the faults and rupture zones, earthquake magnitudes, earthquake durations, and site-specific geologic conditions. Empirical relations developed for rock sites (Abrahamson and Silva, 1997) modified to account for the stiff subsurface conditions (Seed et. al. 1997) provide

\(^1\) The Alquist Priolo Earthquake Fault Zoning Act prohibits placing most structures for human occupancy across traces of active faults. Maps of these fault zones are issued by the Department of Conservation Division of Mines and Geology.
approximate estimates of median peak site accelerations. A summary of the principal active faults affecting the site, their closest distance, moment magnitude of characteristic earthquake and probable peak ground accelerations, which an earthquake on the fault could generate at the site are shown in Table B.

<table>
<thead>
<tr>
<th>Fault</th>
<th>Moment Magnitude for Characteristic Earthquake</th>
<th>Closest Estimated Distance (kilometers)</th>
<th>Median Peak Ground Acceleration (g)¹</th>
</tr>
</thead>
<tbody>
<tr>
<td>San Andreas</td>
<td>7.9</td>
<td>18</td>
<td>0.25</td>
</tr>
<tr>
<td>Rodgers Creek</td>
<td>7.1</td>
<td>11</td>
<td>0.29</td>
</tr>
<tr>
<td>Hayward</td>
<td>7.1</td>
<td>12</td>
<td>0.24</td>
</tr>
<tr>
<td>San Gregorio</td>
<td>7.2</td>
<td>34</td>
<td>0.14</td>
</tr>
<tr>
<td>West Napa</td>
<td>6.5</td>
<td>30</td>
<td>0.11</td>
</tr>
</tbody>
</table>

(¹) Average of three attenuation relationship for stiff soil sites

Probabilistic Seismic Hazard Analysis – Probabilistic Seismic Hazard Analysis (PSHA) analyzes all possible earthquake scenarios while incorporating the probability of each individual event to occur. The probability is determined in the form of the recurrence interval, which is the average rate at which an earthquake of some size will be exceeded. Therefore, the design earthquake is not solely dependent on the fault with the closest distance to the site and/or the largest magnitude, but rather the probability of given seismic events of occurring.

The Division of State Architects (DSA) requires two probabilistic seismic hazard ground motions to be utilized for the project design. The first ground motion is the Upper-Bound Earthquake Ground-Motion (PGA_{UBE}) and is caused by an earthquake with 10% chance of exceedance in 100-years. The second ground motion defined by DSA is the Design-Basis Earthquake Ground-Motion (PGA_{DBE}) and is caused by an earthquake with a 10% chance of exceedance in 50-years. Because the PGA_{UBE} has a longer return period and larger earthquakes, larger ground motions can be expected. DSA requires the more conservative PGA_{UBE} to be utilized when determining the sites susceptibility to liquefaction and the PGA_{DBE} to be utilized for structure design.
Utilizing the FRISKSP program we recommend utilizing the $\text{PGA}_{UBE}$ and $\text{PGA}_{DBE}$ listed below in Table C. The ground motions given by the FRISKSP program were for rock sites. As with the DSHA, we modified the given rock motions to account for the soil conditions. Peak ground accelerations up to 0.5g on rock sites will typically amplify on soil sites, while rock motions greater than 0.5g will typically de-amplify.

<table>
<thead>
<tr>
<th>Chance of Exceedance</th>
<th>Statistical Return Period</th>
<th>Peak Ground Acceleration, Rock</th>
<th>Peak Ground Acceleration, Stiff Soil</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\text{PGA}_{UBE}$</td>
<td>10% in 100 years</td>
<td>949 years</td>
<td>0.50g</td>
</tr>
<tr>
<td>$\text{PGA}_{DBE}$</td>
<td>10% in 50 years</td>
<td>475 years</td>
<td>0.41g</td>
</tr>
</tbody>
</table>

Reference: FRISKSP, Seed et al, 1997
Project Coordinates: latitude 38.0748°, longitude -122.5759°

The potential for strong seismic shaking at the project site is high. Due to their close proximity, the San Andreas, Hayward, and Rogers Creek Faults present the highest potential for severe ground shaking. The most significant adverse impact associated with strong seismic shaking is potential damage to structures and improvements.

**Evaluation:** Less than significant with mitigation.

**Mitigation:** Mitigation for seismic shaking includes designing the structures in accordance with the most recent version of the California Building Code (CBC, 2001). Recommended seismic coefficients are provided in Section V of this report.

D. **Liquefaction Potential**
Liquefaction refers to the sudden, temporary loss of soil shear strength during strong ground shaking. Liquefaction-related phenomena include liquefaction-induced settlement, flow failure, and lateral spreading. These phenomena can occur where there are saturated, loose, granular deposits. The project site is mapped in a zone of moderate liquefaction susceptibility as shown
on Figure 6. Some layers of medium dense, granular deposits were observed below the groundwater elevation. Therefore, the risk of liquefaction at the site is low to moderate.

To evaluate soil liquefaction, the seismic energy from an earthquake is compared with the ability of the soil to resist pore pressure generation. The earthquake energy is termed the cyclic stress ratio (CSR) and is a function of the maximum credible earthquake peak ground acceleration, PGA and depth. The soil resistance is based on the relative density, and the amount and plasticity of the fines (clayey and silty materials that pass the #200 sieve). The relative density of cohesionless soil is correlated with Standard Penetration Test (SPT) blow count data and corrected for, hammer efficiency, overburden and percent fines \(N_{63.5,CS}\) measured during exploration. Liquefaction analyses (Seed et. al., 2003) indicate that intermittent layers of the loose saturated granular sand layers of variable thickness will likely liquefy during a maximum credible seismic event. The results of our liquefaction analyses are presented on Figure 7.

The principal adverse affect of liquefaction occurring at the site is post liquefaction settlement. The magnitude of the settlement at the surface is dependant on the thickness of the liquefiable layer and non-liquefiable layer. Based on our analyses (Youd and Garris, 1995), post liquefaction settlement is not expected at the site because the thickness of the non-liquefiable surface layer (20-25 feet) significantly exceeds the thickness if the potentially liquefiable layer (0-4 feet).

*Evaluation:* No significant impact.

*Mitigation:* No mitigation measures are required.

E. **Seismic Induced Ground Settlement**
Seismic ground shaking can induce settlement of unsaturated, loose, granular soils. Settlement occurs as the loose soil particles rearrange into a denser configuration when subjected to seismic ground shaking. Varying degrees of settlement can occur throughout such a deposit and could result in differential settlement of structures founded on such deposits. We did not encounter soils susceptible to significant seismically induced ground settlement during our exploration. Therefore, the risk of seismically induced settlement is low during an upper bound seismic event.
Evaluation: No significant impact.
Mitigation: No mitigation measures are required.

F. Lateral Spreading, Lurching and Ground Cracking
Lurching and associated ground cracking can occur during strong ground shaking. The ground cracking generally occurs along the tops of slopes where stiff soils are underlain by soft deposits or along steep slopes or channel banks. Since these conditions do not exist at or near the project site, the potential for lurching and ground cracking hazards are considered to be low. Lateral spreading involves large blocks of earth sliding towards a free face on an inclined, continuous, liquefied soil layer. Potentially liquefiable soil layers are present at the project site and the potential for lateral spreading does exist. However, based on the subsurface exploration, the liquefiable layers occur at differing elevations and do not appear continuous. Thus the potential for damage from lateral spreading appears to be low.

Evaluation: No significant impact.
Mitigation: No mitigation measures are required.

G. Erosion
Sandy soils on moderate slopes or clayey soils on steep slopes are susceptible to erosion when exposed to concentrated surface water flow. The potential for erosion is increased when established vegetation is disturbed or removed. A gentle slope exists in the vicinity of the planned structure and there are no drainage paths directed towards the project site, therefore the risk of significant erosion at the site is low.

Evaluation: No significant impact.
Mitigation: The project Civil Engineer should design the site drainage to collect surface water into storm drain systems and discharge water at appropriate locations. Re-establishing vegetation on disturbed areas will minimize erosion. Erosion control measures during and after construction should conform to the most recent version of the Erosion and Sediment Control Field Manual (California Regional Water Quality Control Board, 2002).
H. Seiche and Tsunami
Seiche and tsunamis are short duration, earthquake-generated water waves in large enclosed bodies of water and the open ocean, respectively. The extent and severity of a seiche would be dependent upon ground motions and fault offset from nearby active faults. The site is located several miles west of San Pablo Bay and is at an elevation of approximately 170 feet above sea level. Therefore, the risk of seiche or tsunami at the site is remote.

Evaluation: No significant impact.
Mitigation: No mitigation measures are required.

I. Flooding
The adverse impact from flooding is water damage to structures and furnishings. Although located near Pacheco Creek, the site is not located within the 100 year or 500 year flood zones as delineated by the Federal Emergency Management Agency (FEMA) flood maps, as shown on Figure 8. Therefore the risk of large scale flooding is low.

Evaluation: Less than significant with mitigation.
Mitigation: The surface grades need to be designed to account for surface drainage from surrounding areas. Site grading and drainage design is generally performed by the project Civil Engineer.

J. Settlement/Subsidence
Significant settlement can occur when new loads are placed at sites that are located over soft compressible clays, such as bay mud, or from compression of loose soils. Soft compressible clays or loose soils were not encountered in the subsurface exploration. The potential for significant settlement at the site is low.

Evaluation: No significant impact.
Mitigation: No mitigation measures are required.

K. Expansive Soil
Expansive soil occurs when clay particles interact with water causing volume changes in the clay soil. The clay soil may swell when saturated and shrink when dried. This phenomenon generally decreases in magnitude with increasing confinement pressure at depth. These
volume changes may damage lightly loaded foundations, flatwork, and pavement. Expansive soil also causes soil creep on sloping ground. Expansive soils were not observed during our site inspection or in the subsurface exploration. The potential for expansive soil damage is low.

Evaluation: No significant impact.
Mitigation: No mitigation measures are required.

L. Slope Instability
Weak soils and bedrock on moderate to steep slopes can move downslope due to gravity. Slope instability is often initiated or accelerated from soil saturation and groundwater pressure. Slope movement can vary from slow, shallow soil creep to large, sudden debris flows. Landslides can cause significant damage to structures and improvements, and sudden landslides can result in loss of life. A gentle slope exists in the vicinity of the planned structure, therefore the potential for landsliding at the project site is low. As shown on the Geologic Map, Figure 2, there are no landslides on or near the project site which can impact the project.

Evaluation: No significant impact.
Mitigation: No mitigation measures are required.

M. Soil Corrosion
Corrosive soil can damage buried metallic structures, cause concrete spalling and deteriorate rebar reinforcement. Laboratory testing was performed on representative samples of the near-surface site soils to evaluate pH, electrical resistivity, chloride and sulfate contents. These laboratory test results are presented on Figure A-9.

The results of our corrosivity testing indicate the upper soil layer has a pH of about 5.09, resistivity of approximately 13,900 ohm-cm, a chloride content about 2.4 mg/kg, and a soluble sulfate content about 12 mg/kg. Per Caltrans (2003) criteria, pH levels less than 5.5 classify as corrosive. Soil resistivity above 10,000 Ohm-cm classifies as mildly corrosive. Additionally, chloride concentrations less than 500 mg/kg and sulfate concentrations less than 2000 mg/kg are classified per Caltrans (2003) as being non-corrosive.

Evaluation: Less than significant with mitigation.
Mitigation: Based on the corrosion tests, we recommend a minimum of 400 kg/m³ of cementitious material consisting of 75% by mass Type II Modified or Type V cement plus 25% by mass mineral admixture conforming to ASTM Designation C618 and Section 90-4.02 of Clatrans Standard Specifications. The maximum water-to-cementitious material ratio shall be 0.40 for concrete structures. As is normally practiced, we recommend a minimum of 3 inches concrete coverage over steel reinforcing where concrete is exposed to the ground.
V. CONCLUSIONS AND RECOMMENDATIONS

A. Conclusions
Based on the results of our site investigation and evaluation, we conclude that from a geotechnical standpoint the site conditions are suitable for the planned development. The primary geotechnical concerns for design and construction is strong seismic ground shaking. Specific recommendations addressing design and construction issues are presented in the subsequent sections of this report.

B. Seismic Design
The site will experience strong ground shaking similar to other areas of the seismically active San Francisco Bay Region. Mitigation of strong ground shaking may include structural design in conformance with the provisions of the most recent version of the California Building Code (CBC) or site specific accelerations presented in the following sections.

California Building Code
Based on the interpreted subsurface conditions and closest fault type and distance, we recommend the seismic coefficients and site values shown in Table D to calculate the design base shear of the new construction.
TABLE D  
SEISMIC DESIGN FACTORS  
College of Marin – Pomo Transportation Technology Complex  
Novato, California

2001 California Building Code

<table>
<thead>
<tr>
<th>Factor Name</th>
<th>Coefficient</th>
<th>CBC Table</th>
<th>Site Specific Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Seismic Zone Factor</td>
<td>Z</td>
<td>16A-I</td>
<td>0.40</td>
</tr>
<tr>
<td>Soil Profile Type</td>
<td>$S_{A,B,C,D,E, or F}$</td>
<td>16A-J</td>
<td>$S_D$</td>
</tr>
<tr>
<td>Near Source Factor</td>
<td>$N_a$</td>
<td>16A-S</td>
<td>1.00</td>
</tr>
<tr>
<td>Near Source Factor</td>
<td>$N_i$</td>
<td>16A-T</td>
<td>1.14</td>
</tr>
<tr>
<td>Seismic Coefficient</td>
<td>$C_a$</td>
<td>16A-Q</td>
<td>0.44</td>
</tr>
<tr>
<td>Seismic Coefficient</td>
<td>$C_v$</td>
<td>16A-R</td>
<td>0.73</td>
</tr>
<tr>
<td>Seismic Source Type</td>
<td>A, B or C</td>
<td>16A-U</td>
<td>A</td>
</tr>
</tbody>
</table>

2007 California Building Code

<table>
<thead>
<tr>
<th>Factor Name</th>
<th>Coefficient</th>
<th>CBC Table</th>
<th>Site Specific Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Site Class</td>
<td>$S_{A,B,C,D,E, or F}$</td>
<td>1613.5.2</td>
<td>$S_D$</td>
</tr>
<tr>
<td>Spectral Acc. (short)</td>
<td>$S_a$</td>
<td>1613.5.1</td>
<td>1.50 g</td>
</tr>
<tr>
<td>Spectral Acc. (1-sec)</td>
<td>$S_i$</td>
<td>1613.5.1</td>
<td>0.60 g</td>
</tr>
<tr>
<td>Site Coefficient</td>
<td>$F_a$</td>
<td>1613.5.3 (1)</td>
<td>1.0</td>
</tr>
<tr>
<td>Site Coefficient</td>
<td>$F_v$</td>
<td>1613.5.3 (2)</td>
<td>1.5</td>
</tr>
</tbody>
</table>

(1) Soil Profile/Site Class Type $S_D$: Description: Stiff Soil Profile, Shear Wave Velocity between 600 and 1200 feet per second, Standard Penetration Test N values between 15 and 50, and Undrained Shear Strength between 1,000 psf and 2,000 psf.

(2) The project Structural Engineer should determine if the Near Source Factor ($N_a$) and Seismic Coefficient ($C_a$) may be reduced based on building type and structure irregularities (CBC 1629A.4.2 and 1630A.2.3.2).

(3) Seismic Source Type A: Faults that are capable of producing large magnitude events and have a high rate of seismic activity.

C. Site Grading

Based on the current project conditions, only minor site grading is anticipated. Site preparation and grading should conform to the following recommendations and criteria:

1. **Surface Preparation** – Clear all grass, brush, roots, over-sized debris and organic material from areas that will be within the new project work area. Excavate loose soil to expose firm natural soils. Any landscaping vegetation within the building areas should be scraped from the surface, stockpiled for reuse in landscaping or removed from the site. Any construction debris or
abandoned utilities encountered during site grading should be removed from the site. Utilities could also be abandoned in place in many cases provided cement grout completely fills any void in the utility. Rocks or concrete pieces larger than 6 inches encountered during subgrade preparation or site grading should be removed from the site.

2. **Materials** – Clean non-expansive soil and rock mixtures generated from on-site excavations may be suitable for use as fill provided the material is well mixed, maximum particle sizes are less than 6 inches, and have a maximum PI of 20. Materials used for retaining wall backfills should also be smaller than 4 inches. Processing will include removal and/or crushing of rock, mixing and moisture conditioning as described below.

If imported fill is required, the material shall consist of soil and rock mixtures that: (1) are free of organic material, (2) have a Liquid Limit less than 40 and a Plasticity Index of less than 20, and (3) have a maximum particle size of 4 inches. Any imported fill material needs to be tested to determine its suitability for use as fill material.

3. **Compacted Fill** – Where fills or structures are planned, the subgrade surface should be scarified to a depth of 8 inches, moisture conditioned to near optimum moisture content and compacted to a minimum of 90 percent relative compaction. Relative compaction refers to the ratio in percent of the in-situ dry density to the maximum laboratory density. The maximum laboratory dry density and optimum moisture content of fill materials should be determined in accordance with ASTM Test Method D-1557, "Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using a 10-lb. Rammer and 18-in. Drop." New fill or backfill should be moisture conditioned to within 3 percent of the optimum moisture content. Properly moisture conditioned and cured on-site materials should subsequently be placed in loose horizontal lifts of 8 inches thick or less, and uniformly compacted to at least 90 percent relative compaction. In areas where new asphalt pavement will be installed, the upper 8 inches should be further compacted to 95 percent relative compaction to provide a firm and unyielding surface.

D. **Foundation Design**

It is our understanding that the existing structures are supported on a drilled pier foundations system with drilled piers that extend 10 to 15 feet below grade. To maintain relatively uniform
support and performance for the Pomo transportation complex, we recommend the new structure should also be supported on a drilled pier foundation. Shallow foundations could be utilized for exterior structures or retaining walls. The foundations should be designed using the criteria presented in Table E.

### TABLE E

**FOUNDATION DESIGN CRITERIA**

Client: Marin – Pomo Transportation Technology Complex

**Novato, California**

<table>
<thead>
<tr>
<th>Drilled Piers:</th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Minimum diameter:</td>
<td>18 inches</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Minimum penetration:</td>
<td>10 feet</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Skin Friction (dead plus live loads) (^{1,2,3}):</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Alluvial Soils (upper 25 feet):</td>
<td>1500 psf</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Weathered bedrock:</td>
<td>2500 psf</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Lateral passive resistance (^4):</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Alluvial Soil (upper 25 feet):</td>
<td>300 pcf</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Weathered bedrock:</td>
<td>400 pcf</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Shallow Spread Footings:</th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Minimum depth: (^5)</td>
<td>18 inches</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Allowable bearing capacity: (^1,6)</td>
<td>2000 psf</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Base friction coefficient:</td>
<td>0.35</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Lateral passive resistance: (^4)</td>
<td>300 pcf</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

1. May increase design values by 1/3 for total design loads.
2. Uniform pressure distribution.
3. Use 75% of skin friction values for uplift resistance.
4. Equivalent fluid pressure, ignore upper 12 inches of passive support. For piers, apply values over effective width of two pier diameters.
5. Adjust footing depth on slopes to maintain at least 10 feet horizontal distance from base of footing to face of slope.
6. Foundation to bear on firm soil. Size foundations to maintain uniform bearing pressures.

Should the piers be left open for extended periods of time, they may partially fill with water. The water could either be pumped out and the piers cleaned before placement of concrete or the concrete conveyed to the bottom of the hole using a tremie pipe. The bottom of the drilled piers shall be clear of loose material and debris before concrete placement.
E. Retaining Walls

We understand that retaining may be incorporated into the project. Retaining walls that can deflect at the top can be designed using the unrestrained criteria shown in Table F. Walls that are structurally connected and not allowed to deflect are restrained. Restrained conditions are commonly designed using an uniform earth pressure distribution rather than an equivalent fluid pressure. Lateral support can be obtained from either passive soil resistance (i.e. keyways) or frictional sliding resistance of spread footings. In addition to the soil loads, the retaining walls should be designed to resist temporary seismic loads.

<table>
<thead>
<tr>
<th>TABLE F</th>
</tr>
</thead>
<tbody>
<tr>
<td>RETAINING WALL DESIGN CRITERIA</td>
</tr>
<tr>
<td>College of Marin – Pomo Transportation Technology Complex</td>
</tr>
<tr>
<td>Novato, California</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Foundations:</th>
<th>See Table E</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lateral Earth Pressure:</td>
<td></td>
</tr>
<tr>
<td>Level ground</td>
<td>Restricted^1,2</td>
</tr>
<tr>
<td>2:1 slope</td>
<td>30^H psf</td>
</tr>
<tr>
<td></td>
<td>40^H psf</td>
</tr>
<tr>
<td>Seismic Surcharge^3</td>
<td>12^H psf^2</td>
</tr>
</tbody>
</table>

Notes:

1. Interpolate earth pressures for intermediate slopes.
2. Rectangular pressure distribution. H equals exposed wall height in feet.
3. Equivalent fluid pressure.
4. Resultant force acts at a distance of 0.6^H feet above the base. The factor of safety for short-term seismic conditions can be reduced to 1.1 or greater.

All walls over 3 feet high require drainage to prevent the build-up of hydrostatic pressure. Either Caltrans Class 1B permeable material within filter fabric, Caltrans Class 2 permeable material or drainage panels can be used. The seepage should be collected in a 4-inch perforated PVC drain line at the base of the wall. The permeable material shall extend at least 12 inches from the back of the wall and be continuous from the bottom of the wall to within 12 inches of the ground surface. The upper 12 inches of backfill for exterior walls shall be impermeable, compacted soil.
Seepage collected in the drain line should be conveyed in a solid pipe by gravity to the storm drainage system or discharged through weep holes in the wall. The pipe shall have a minimum slope to drain of 1 percent. To maintain the wall drainage system, clean outs should be installed at the upstream end and at all major changes in direction in the drain line. A typical retaining wall drain and backfill detail is shown on Figure 9.

F. Concrete Slabs-on-Grade
Concrete slab-on-grade floors should be designed to be structurally connected to and span between foundation supports and grade beams. Interior reinforced concrete slab-on-grade floors should be at least 5 inches thick and reinforced with steel reinforcing bars (not wire mesh). Unless otherwise recommended by the Structural Engineer, we recommend crack control joints in both directions and that the reinforcing bars extend through the control joints. The Structural Engineer should design the concrete slab floors. Some notes for the structural design of concrete slabs are presented in Appendix B.

To improve interior moisture conditions, a 4-inch layer of clean, free draining, 3/4-inch angular gravel or crushed base rock should be placed beneath the interior concrete slabs to form a capillary moisture break. The base rock must be placed on a properly moisture conditioned and compacted subgrade that has been approved by the Geotechnical Engineer. A plastic membrane vapor barrier, 10 mils or thicker, should be placed over the compacted base rock. The vapor barrier shall meet the ASTM E 1745 Class A requirements and be installed per ASTM 1643. Eliminating the capillary moisture break and/or plastic vapor barrier may result in excess moisture intrusion through the floor slabs resulting in poor performance of floor coverings, mold growth, or other adverse conditions. A 2-inch layer of dry sand should be placed over the vapor barrier to prevent puncture of the plastic membrane and aid in slab curing.

Exterior reinforced concrete slabs-on-grade can be placed directly on a properly prepared subgrade as described in Section V.C. To improve slab performance, we recommend thickening exterior slabs to 5 inches and reinforcing with steel bars. Exterior slabs may be underlain with 4 inches or more of Caltrans Class 2 Aggregate Base compacted to at least 92 percent relative compaction. Some movement should be expected for exterior concrete slabs as the underlying soils react to seasonal moisture changes.
G. Site and Foundation Drainage

The site is relatively flat and there is a possibility that new grading or buildings could result in adverse drainage patterns and water ponding around buildings. Careful consideration should therefore be given to design of finished grades at the site. We recommend that the building areas be raised slightly and that the adjoining landscaped areas be sloped downward at least 0.25 feet for 5 feet (5 percent) from the buildings. Where hard surfaces, such as concrete or asphalt adjoin foundations, slope these surfaces at least 0.10 feet in the first 5 feet (2 percent).

Site drainage should be discharged away from the building area and outlets should be designed to reduce erosion of the soils immediately down slope. Site drainage improvements should be connected into the existing City storm drainage system if possible.

H. Utility Trench Backfills

Excavations for utilities will be in loose to medium stiff alluvial soil. Bedrock is not likely to be encountered. Trench excavations having a depth of 5 feet or more must be excavated and shored in accordance with OSHA regulations. Pursuant to OSHA classifications, on-site clay and sandy soils may be considered as Type C. Bedding materials for utility pipes should be well graded, non-corrosive sand with 90 to 100 percent of particles passing the No. 4 sieve and no more than 5 percent finer than the No. 200 sieve. Provide the minimum bedding beneath the pipe in accordance with the manufacturer’s recommendation, typically 3 to 6 inches. Trench backfill may consist of on-site soils, moisture conditioned, and placed in thin lifts and compacted to at least 90 percent relative compaction. The upper 12 inches of backfill for trenches within areas of asphalt paving should consist of properly moisture conditioned Caltrans Class 2 Aggregate Base compacted to at least 95 percent relative compaction. Use equipment and methods that are suitable for work in confined areas without damaging utility conduits.

I. Pavement Design

If needed, subgrade preparation for asphalt-paved areas should follow the recommendations in the site preparation and grading section of this report. Additionally, the subgrade should be smooth and unyielding under a moving, fully-loaded water truck. The subgrade should also be maintained at near optimum moisture content prior to placement of aggregate baserock. Areas of soft or saturated soils encountered during construction should be excavated and replaced with
properly moisture conditioned fill or aggregate base. During construction, the subgrade material must be examined and tested, if need be, to confirm that the R-value exceeds the assumed preliminary design value.

We have calculated the pavement section for the asphalt entrance driveway in accordance with Caltrans procedures for flexible pavement design (1990). We have estimated R-value of 10 for the preliminary pavement design and a range of Traffic Indices (T.I.). The recommended pavement section is presented on Table G below.

<table>
<thead>
<tr>
<th>T.I.</th>
<th>Asphalt Concrete</th>
<th>Aggregate Base</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.0</td>
<td>2.0 inches</td>
<td>6.0 inches</td>
</tr>
<tr>
<td>4.0</td>
<td>3.0 inches</td>
<td>7.0 inches</td>
</tr>
<tr>
<td>5.0</td>
<td>3.0 inches</td>
<td>10.0 inches</td>
</tr>
</tbody>
</table>

(1) The asphalt concrete should conform to the criteria for asphalt presented in Section 39 of the Caltrans Standard Specifications. The asphalt concrete shall be placed in layers not exceeding 2.5 inches in thickness and compacted to at least 95 percent relative compaction.

(2) To reduce the asphalt pavement section the “general rule of thumb” of 1 inch of asphalt equals 2 inches of aggregate base applies.

The aggregate base material should conform to Class 2 Aggregate Base in the current edition of Caltrans Standard Specifications. The aggregate base should be moisture conditioned to near optimum moisture content and compacted in layers not exceeding 6 inches to at least 95 percent relative compaction.
VI. SUPPLEMENTAL GEOTECHNICAL SERVICES

We must review the plans and specifications for the project when they are nearing completion to confirm that the intent of our geotechnical recommendations has been incorporated and provide supplemental recommendations, if needed.

During construction, we need to observe and/or test site preparation and surface drainage. We also need to observe foundation excavations for the structures and associated improvements to confirm that the soils encountered during construction are consistent with the design criteria.
LIST OF REFERENCES


California Department of Transportation (Caltrans), Standard Specifications, 2000.


Garcia, A.W., Houston, J.R., "Type 16 Flood Insurance Study: Tsunami Predictions for Monterey and San Francisco Bays and Puget Sound," U.S. Army Engineer Waterways Experiment Station, 1975


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Seed, R.B. it all, "Recent Advances in soil of Liquefaction Engineering, a united and consistent framework," 26th Annual ASCE Los Angeles Geotechnical Spring Seminar, Long Beach, California, April 30, 2003.

Southern California Earthquake Center, University of Southern California, 1999, Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction Hazards in California.


SITE LOCATION

PROJECT SITE COORDINATES:
LATITUDE, 38.0748°
LONGITUDE, -122.5759°

REFERENCE: DeLorme 3D TopoQuads, 1999; Based on USGS 7.5 Minute Novato Quadrangle
COPYRIGHT 2007, MILLER PACIFIC ENGINEERING GROUP
FILE: SiteMap.dwg

Miller Pacific
ENGINEERING GROUP

SITE LOCATION MAP
IVC Pomo Transportation Technology Center
Novato, California

Project No. 739.12 Date 8/20/07 Approved By:
Explanations:

Qhb: Holocene estuarine deposits - silts, clays and fine sands
Qoa: Early to late Pleistocene deposits, undivided - includes alluvial fan, terrace, basin and channel deposits
Qha: Holocene alluvium - poorly to moderately sorted gravel, sand and silt
Qls: Landslide - including debris flow and block slump landslides
KJfs: Franciscan Complex sandstone and shale - thick bedded, arkosic sandstone and interbedded shale
KJfm: Franciscan Complex melange - tectonic mixture of masses of resistant rock types embedded in a sheared, shaley matrix
REFERENCES:
SECTION A-A'

Explanation

- Alluvial deposits consisting of interbedded medium stiff to stiff, clay with sand and loose to medium dense clayey sand, occasional gravel

- Franciscan Complex shale with sandstone interbeds, sandstone, and shale melange

- Approximate groundwater elevation

SCALE

0 15 30 60 FEET

NO SCALE
Probabilistic SPT-Based Liquefaction Triggering Analysis

LIQUEFIABLE

NON-LIQUEFIABLE

Percentage lines denote probability of liquefaction.
SITE

URBANIZED AREA, OUTSIDE OF 100 YEAR AND 500 YEAR FLOOD ZONES


NO SCALE

Miller Pacific
ENGINEERING GROUP

FLOOD MAP
IVC Pomo Transportation Technology Center
Novato, California

Project No. 739.12 Date 8/20/07

Approved By:
NOTES:

1. Wall drainage should consist of clean, free draining 3/4 inch crushed rock (Class 1B Permeable Material) wrapped in filter fabric (Mirafi 140N or equivalent) or Class 2 Permeable Material. Alternatively, a pre-fabricated drainage panels (Miradrain G100N or equivalent) installed per the manufactures recommendations, may be used in lieu of drain rock and fabric.

2. All retaining walls adjacent to interior living spaces shall be water/vapor proofed as specified by the project architect or structural engineer.

3. Perforated pipe shall be SCH 40 or SDR 35 for depths less than 20 feet. Use SCH 80 or SDR 23.5 perforated pipe for depths greater than 20 feet. Place pipe perforations down and sloped at 1% to a gravity outlet. Alternatively, drainage can be outlet through 3" diameter weep holes spaced approximately 20' apart.

4. Clean outs should be installed at the upslope end and at significant direction changes of the perforated pipe. Additionally, all angled connectors shall be long bend sweep connections.

5. During compaction, the contractor should use appropriate methods (such as temporary bracing and/or light compaction equipment) to avoid over stressing the walls. Walls shall be completely backfilled prior to construction in front of or above the retaining wall.

6. Refer to the geotechnical report for lateral soil pressures.

7. All work and materials shall conform with Section 68, of the latest edition of the State of California Standard Specifications (Caltrans).
APPENDIX A

SUBSURFACE EXPLORATION AND LABORATORY TESTING

1.0 Subsurface Exploration – Auger Borings

We explored subsurface conditions at the site by drilling two test borings on July 26, 2007 utilizing a track mounted drilling rig and 4-inch solid flight augers. The boring locations are shown on Figure 4. Test borings were drilled to depths between 35.5 and 45 feet below the ground surface.

We obtained "undisturbed" samples using a 3-inch (75-mm) diameter, split-barrel California sampler with 2.5 by 6-inch brass tube liners. The 2-inch Standard Penetration Test (SPT) split-barrel sampler was intermittently used to aid in soil property indexing, identification, and liquefaction analysis. The samplers were driven with a 140-pound (63.5-kg) hammer falling 30 inches (760 mm). The number of blows required to drive the samplers 18 inches (460 mm) was recorded and is reported on the boring logs as blows per foot for the last 12 inches (305 mm) of driving. The samples obtained were examined in the field, sealed to prevent moisture loss, and transported to our laboratory.

The soils encountered were logged and identified in general accordance with ASTM Standard D 2487, "Field Identification and Description of Soils (Visual-Manual Procedure)." This standard is briefly explained on Figure A-1, Soil Classification Chart and Figure A-2, Rock Classification Chart. The exploratory boring logs are presented on Figures A-3 to A-7.

2.0 Laboratory Testing

We conducted laboratory tests on selected intact and bulk samples to verify field identifications and to evaluate engineering properties. The following laboratory tests were conducted in general accordance with the ASTM standard test method cited:

- Laboratory Determination of Water (Moisture Content) of Soil, Rock, and Soil-Aggregate Mixtures, ASTM D 2216;
- Density of Soil in Place by the Drive-Cylinder Method, ASTM D 2937;
- Unconfined Compressive Strength of Cohesive Soil, ASTM D 2166;
- Amount of Material in Soils Finer Than the No. 200 (75 μm) Sieve, ASTM D 1140;
- Particle Size Analysis of Soils, ASTM D 422;
- pH in soil, EPA 9040;
- Resistivity in Soil, SM 2510; and
- Anions in soil (sulfate and chloride), EPA 300.

The moisture content, dry density, finer than #200 sieve, and unconfined compression test results are shown on the exploratory boring logs, Figures A-3 through A-7 and the result of the particle size analyses and corrosion results are summarized on Figures A-8 and A-9.

The exploratory boring logs, description of soils encountered and the laboratory test data reflect conditions only at the location of the boring at the time they were excavated or retrieved. Conditions may differ at other locations and may change with the passage of time due to a variety of causes including natural weathering, climate and changes in surface and subsurface drainage.
SOIL CLASSIFICATION CHART

MAJOR DIVISIONS | SYMBOL | DESCRIPTION
--- | --- | ---
CLEAN GRAVEL | GW | Well-graded gravels or gravel-sand mixtures, little or no fines
GP | Poorly-graded gravels or gravel-sand mixtures, little or no fines
GRAVEL with fines | GM | Silty gravels, gravel-sand-silt mixtures
GC | Clayey gravels, gravel-sand-clay mixtures
CLEAN SAND | SW | Well-graded sands or gravelly sands, little or no fines
SP | Poorly-graded sands or gravelly sands, little or no fines
SAND with fines | SM | Silty sands, sand-silt mixtures
SC | Clayey sands, sand-clay mixtures
SILT AND CLAY liquid limit <50% | ML | Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity
CL | Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
OL | Organic silts and organic silt-clays of low plasticity
SILT AND CLAY liquid limit >50% | MH | Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts
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
ROCK | | Undifferentiated as to type or composition

KEY TO BORING AND TEST PIT SYMBOLS

CLASSIFICATION TESTS
AL ATTERBERG LIMITS TEST
SA SIEVE ANALYSIS
HYD HYDROMETER ANALYSIS
P200 PERCENT PASSING NO. 200 SIEVE
P4 PERCENT PASSING NO. 4 SIEVE

STRENGTH TESTS
TV FIELD TORVANE (UNDRAINED SHEAR)
UC LABORATORY UNCONFINED COMPRESSION
TXCU CONSOLIDATED UNDRAINED TRIAXIAL
TXUU UNCONSOLIDATED UNDRAINED TRIAXIAL
UC, CU, UU = 1/2 Deviator Stress

SAMPLER TYPE

| SYMBOL | DESCRIPTION |
--- | --- |
UNDISTURBED CORE SAMPLE: MODIFIED CALIFORNIA OR HYDRAULIC PISTON SAMPL | STANDARD PENETRATION TEST SAMPLE
X DISTURBED OR BULK SAMPLE | ROCK OR CORE SAMPLE

NOTE: Test boring and test pit logs are an interpretation of conditions encountered at the location and time of exploration. Subsurface rock, soil and water conditions may differ in locations and with the passage of time. Lines defining interface between differing soil or rock description are approximate and may indicate a gradual transition.

FILE: 739.12 BL.dwg
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SOIL CLASSIFICATION CHART
IVC Pomo Transportation Technology Center
Novato, California

Project No. 739.12 Date 08/15/2007 Approved By: — Figure
# FRACTURING AND BEDDING

<table>
<thead>
<tr>
<th>Fracture Classification</th>
<th>Spacing</th>
<th>Bedding Classification</th>
</tr>
</thead>
<tbody>
<tr>
<td>Crushed</td>
<td>less than 3/4 inch</td>
<td>Laminated</td>
</tr>
<tr>
<td>Intensely fractured</td>
<td>3/4 to 2-1/2 inches</td>
<td>Very thinly bedded</td>
</tr>
<tr>
<td>Closely fractured</td>
<td>2-1/2 to 8 inches</td>
<td>Thinly bedded</td>
</tr>
<tr>
<td>Moderately fractured</td>
<td>8 to 24 inches</td>
<td>Medium bedded</td>
</tr>
<tr>
<td>Widely fractured</td>
<td>2 to 6 feet</td>
<td>Thickly bedded</td>
</tr>
<tr>
<td>Very widely fractured</td>
<td>greater than 6 feet</td>
<td>Very thickly bedded</td>
</tr>
</tbody>
</table>

# HARDNESS

<table>
<thead>
<tr>
<th>Degree</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Low</td>
<td>Carved or gouged with a knife</td>
</tr>
<tr>
<td>Moderate</td>
<td>Easily scratched with a knife, friable</td>
</tr>
<tr>
<td>Hard</td>
<td>Difficult to scratch, knife scratch leaves dust trace</td>
</tr>
<tr>
<td>Very hard</td>
<td>Rock scratches metal</td>
</tr>
</tbody>
</table>

# STRENGTH

<table>
<thead>
<tr>
<th>Degree</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Friable</td>
<td>Crumbles by rubbing with fingers</td>
</tr>
<tr>
<td>Weak</td>
<td>Crumbles under light hammer blows</td>
</tr>
<tr>
<td>Moderate</td>
<td>Indentations &lt;1/8 inch with moderate blow with pick end of rock hammer</td>
</tr>
<tr>
<td>Strong</td>
<td>Withstands few heavy hammer blows, yields large fragments</td>
</tr>
<tr>
<td>Very strong</td>
<td>Withstands many heavy hammer blows, yields dust, small fragments</td>
</tr>
</tbody>
</table>

# WEATHERING

<table>
<thead>
<tr>
<th>Degree</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Complete</td>
<td>Minerals decomposed to soil, but fabric and structure preserved</td>
</tr>
<tr>
<td>High</td>
<td>Rock decomposition, thorough discoloration, all fractures are extensively coated with clay, oxides or carbonates</td>
</tr>
<tr>
<td>Moderate</td>
<td>Fracture surfaces coated with weathering minerals, moderate or localized discoloration</td>
</tr>
<tr>
<td>Slight</td>
<td>A few stained fractures, slight discoloration, no mineral decomposition, no affect on cementation</td>
</tr>
<tr>
<td>Fresh</td>
<td>Rock unaffected by weathering, no change with depth, rings under hammer impact</td>
</tr>
</tbody>
</table>

NOTE: Test boring and test pit logs are an interpretation of conditions encountered at the location and time of exploration. Subsurface rock, soil and water conditions may differ in other locations and with the passage of time.
<table>
<thead>
<tr>
<th>OTHER TEST DATA</th>
<th>UNDRAINED SHEAR STRENGTH psf (1)</th>
<th>BLOWS PER FOOT</th>
<th>MOISTURE CONTENT (%)</th>
<th>DRY UNIT WEIGHT psf (2)</th>
<th>DEPTH (feet)</th>
<th>SAMPLE SYMBOL</th>
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<td>30</td>
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<td>40.5% P200</td>
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**CLAYEY SAND (SC)**

Mottled yellow-red-orange, dry, loose, ~60% fine to medium-grained sand, ~40% low plasticity clay, predominately fine sand, occasional rounded to subangular chert fragments, at 3 feet grades moist, dense (Alluvium)

Grades moist, medium dense, ~75% fine to medium-grained sand, ~25% low plasticity clay

Grades ~60% fine to medium-grained sand, ~40% low plasticity clay

**NOTES:**

1. METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
2. METRIC EQUIVALENT DRY UNIT WEIGHT kN/m³ = 0.1571 x DRY UNIT WEIGHT (pcf)
3. GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY
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<th>OTHER TEST DATA</th>
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<th>BLOWS PER FOOT</th>
<th>MOISTURE CONTENT (%)</th>
<th>DRY UNIT WEIGHT (pcf)</th>
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**BORING 1 (CONTINUED)**

**CLAYEY SAND (SC)**
mottled red-orange-brown, wet, medium dense, ~85% fine to coarse-grained sand, ~15% low plasticity clay (Alluvium)

**CLAY WITH SAND (CL)**
mottled orange-brown-gray, saturated, medium stiff to stiff, ~80% low to medium plasticity clay, ~20% fine-grained sand, wood fragment (Alluvium)

**SILTY SAND WITH GRAVEL (SM)**
mottled dark gray and brown, saturated, medium dense, ~50% fine to coarse-grained sand, ~30% subangular gravel, ~20% medium plasticity clay (Alluvium)

**SANDY CLAY (CL)**
orange and brown, saturated, stiff, ~60% clay, ~35% fine-grained sand, ~5% fine-grained gravel (Alluvium)

**CLAY WITH SAND (CL)**
mottled light brown-orange-gray, moist, very stiff, ~85% low to medium plasticity clay, ~15% fine to medium-grained sand, sample displays thin discontinuous laminations (Residual Soil)

**SANDSTONE**
mottled gray and orange, dry, low hardness, weak, highly weathered, fracture surfaces heavily coated with oxidation stains

**NOTES:**
(1) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m³ = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY
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<tr>
<td></td>
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<td></td>
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<td>40</td>
</tr>
<tr>
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<td>50/1&quot;</td>
<td>15.8</td>
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<td>45</td>
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</table>

**SANDSTONE**
mottled gray and orange, dry, low hardness, weak, highly weathered, fracture surfaces heavily coated with oxidation stains

**SHALE MELANGE**
dark gray, moist, crushed, low hardness, friable to weak, completely to highly weathered

Bottom of boring at 45.1 feet
Groundwater observed at 25 feet during drilling

**NOTES:**
1. METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
2. METRIC EQUIVALENT DRY UNIT WEIGHT kN/m² = 0.1571 x DRY UNIT WEIGHT (pcf)
3. GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY
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<th>DRY UNIT WEIGHT (pcf) (2)</th>
<th>DEPTH (feet)</th>
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<td></td>
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<td></td>
<td>dark red-brown, moist to wet, medium dense, ~65% fine-grained sand, ~35% low plasticity clay (Alluvium)</td>
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<tr>
<td></td>
<td></td>
<td>50</td>
<td>22.7</td>
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<tr>
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<td></td>
<td></td>
<td></td>
<td>mottled light brown-orange, saturated, very stiff, ~85% low to medium plasticity clay, ~15% fine to medium-grained sand, discontinuous laminations (Residual Soil)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>50/6&quot;</td>
<td>15.5</td>
<td></td>
<td>-30</td>
<td>SHALE MELANGE</td>
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<tr>
<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td>mottled brown and gray, moist, low hardness, crushed, friable, completely weathered</td>
</tr>
<tr>
<td></td>
<td></td>
<td>60/6&quot;</td>
<td>9.4</td>
<td></td>
<td>-11</td>
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<td></td>
<td></td>
<td>Groundwater not encountered during drilling</td>
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NOTES:
(1) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(2) METRIC EQUIVALENT DRY UNIT WEIGHT KN/m³ = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY
U.S. STANDARD SIEVE NUMBERS

PERCENT FINER BY WEIGHT

GRAIN SIZE IN MILLIMETERS

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<th>CLASSIFICATION</th>
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<td>Boring 1 at 30.5 feet</td>
<td>SILTY SAND WITH GRAVEL (SM)</td>
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<tr>
<td></td>
<td></td>
<td>mottled dark gray and brown</td>
</tr>
<tr>
<td>–■–</td>
<td>Boring 1 at 31 feet</td>
<td>SANDY CLAY (CL)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>mottled brown and orange</td>
</tr>
<tr>
<td>–▲–</td>
<td>Boring 2 at 15 feet</td>
<td>CLAYEY SAND (SC)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>mottled yellow-red-orange</td>
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</table>

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FILE: SteveAnalyzes.doc

Miller Pacific
ENGINEERING GROUP

SIEVE ANALYSES
IVC Pomo Transportation Technology Center
Novato, California

Figure

Project No. 739.12 Date 8/20/07 Approved By: A-8
### pH

<table>
<thead>
<tr>
<th>Lab#</th>
<th>Sample ID</th>
<th>Compound Name</th>
<th>Result (pH Units)</th>
<th>RDL (pH Units)</th>
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<td>pH</td>
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Date Sampled: 08/23/07  
Date Received: 08/23/07  
Date Analyzed: 09/05/07  
Method: EPA 9040  
QC Batch: B003010

### Resistivity in Soil

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<th>Compound Name</th>
<th>Result (MOhm-cm)</th>
<th>RDL (MOhm-cm)</th>
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<td>739.12 (0-6')</td>
<td>Resistivity</td>
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Date Sampled: 08/23/07  
Date Received: 08/23/07  
Date Analyzed: 09/05/07  
Method: SM 2510  
QC Batch: B003010

### Anions in Soil

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<th>RDL (mg/kg)</th>
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<td>Sulfate as SO4</td>
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Date Sampled: 08/23/07  
Date Received: 08/23/07  
Date Analyzed: 09/05/07  
Method: EPA 300.0  
QC Batch: B003003

Lab Project#: 7082320  
CA Lab Accreditation #: 2303
APPENDIX B
DESIGN NOTES FOR CONCRETE SLABS-ON-GRADE

These "design notes" are for the general guidance of the project Civil Engineer or Architect who is responsible for the actual design of concrete slabs-on-grade for the project.

Recommendations are given in the body of the report for the subgrade support of concrete flatwork. However, concrete slabs-on-grade often perform poorly for various non-geotechnical reasons. These notes are offered as a reminder of factors which can influence slab performance. The designer should refer to the recommendations and design guidelines published by the Portland Cement Association, The American Concrete Institute and the Northern California Cement Promotion Group.

THICKNESS and STRENGTH. For residential walks, automobile driveways and garage floors, it is normal practice to use 4-inch thick slabs of 2500 psi concrete. For improved performance, the design may be upgraded to 5-inch thick slabs of 3000 psi concrete. Concrete streets and driveways subjected to truck traffic and concrete residential floors should be designed for the specific loads and job conditions.

SHRINKAGE. All concrete shrinks as it cures. Shrinkage will amount to 1/16 to 1/8 inch per 20-foot length. A concrete mix with a high water/cement ratio results in increased shrinkage and greater shrinkage cracking. A low water/cement ratio will reduce shrinkage and cracking.

REINFORCEMENT. It is normal to use non-reinforced concrete for residential concrete slabs. However, wire mesh or light steel reinforcement will minimize crack width and resist vertical offset across cracks.

CRACK CONTROL JOINTS. Crack control joints are used to control the location of the inevitable shrinkage cracks. Crack control joints should extend to a depth of 1/4 to 1/3 of the thickness of the slab and be spaced about 20 to 30 times the slab thickness (i.e., for a 4-inch thick slab, joints should be spaced 6 to 10 feet apart). If mesh is used in the slab, it should be continuous through the joints. If reinforcing steel is used, No. 3 or No. 4 bars should be used and only every other bar should extend across the joint. Keyed joints or dowels may also be considered. To be effective, the joints must be tooled into the fresh concrete or saw cut within 4 to 12 hours of the pour, while the concrete is still green.

ISOLATION JOINTS AND EXPANSION JOINTS. Isolation joints should be provided where vertical or horizontal movement is expected. The joints should extend for the full slab thickness and contain a compressible joint filler. Mesh and reinforcement should not extend across the joint. Joints used to accommodate expansion should be spaced about 60 feet apart.

CURING. Where aggregate base and a vapor barrier are placed under the slab, 2 inches or more of moist sand should directly underlay the slab to aid in more uniform curing between top and bottom. The slab should be cured with wet curing methods or moisture retention curing compounds. Particular care should be taken in hot and windy weather.
Important Information About Your Geotechnical Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes. The following information is provided to help you manage your risks.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects
Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared solely for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. And no one — not even you — should apply the report for any purpose or project except the one originally contemplated.

Read the Full Report
Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors
Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client’s goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:
- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:
- the function of the proposed structure, as when it’s changed from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, always inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.

Subsurface Conditions Can Change
A geotechnical engineering report is based on conditions that existed at the time the study was performed. Do not rely on a geotechnical engineering report whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. Always contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions
Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report’s Recommendations Are Not Final
Do not over rely on the construction recommendations included in your report. Those recommendations are not final, because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual
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- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, always inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.

Subsurface Conditions Can Change

A geotechnical engineering report is based on conditions that existed at the time the study was performed. Do not rely on a geotechnical engineering report whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. Always contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report’s Recommendations Are Not Final

Do not overrely on the construction recommendations included in your report. Those recommendations are not final, because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual
subsidiary conditions revealed during construction. The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should never be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, but recognize that separating logs from the report can elevate risk.

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, but preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. Be sure contractors have sufficient time to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations" or "risks of site conditions," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. Read these provisions closely. Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a geoenvironmental study differ significantly from those used to perform a geotechnical study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. Unanticipated environmental problems have led to numerous project failures. If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. Do not rely on an environmental report prepared for someone else.

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the express purpose of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infections, a number of mold prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant. None of the services performed in connection with the geotechnical engineer’s study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.

Rely on Your ASFE-Member Geotechnical Engineer for Additional Assistance

Membership in ASFE/ THE BEST PEOPLE ON EARTH exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with your ASFE-member geotechnical engineer for more information.

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