GEOTECHNICAL INVESTIGATION
FOR
VAN CRUYNINGEN RESIDENCE
835 WESTRIDGE DRIVE
PORTOLA VALLEY, CALIFORNIA

May 2006

Prepared for

Mr. Ike van Cruyningen
835 Westridge Drive
Portola Valley, California 94028

Project No. 1681-1

ROMIG ENGINEERS, INC.
GEOTECHNICAL & ENVIRONMENTAL SERVICES

May 31, 2006
1681-1

Mr. Ike van Cruyningen
835 Westridge Drive
Portola Valley, California 94028

**RE: GEOTECHNICAL INVESTIGATION
VAN CRUYNINGEN RESIDENCE
835 WESTRIDGE DRIVE
PORTOLA VALLEY, CALIFORNIA**

Dear Mr. Van Cruyningen:

In accordance with your request, we have performed a geotechnical investigation for your proposed residence at 835 Westridge Drive in Portola Valley, California. The accompanying report summarizes the results of our field exploration, laboratory testing, and engineering analysis, and presents our geotechnical recommendations for the proposed construction.

We refer you to the text of our report for specific recommendations.

Thank you for the opportunity to work with you on this project. If you have any questions or comments about the findings or recommendations from our investigation, please call.

Very truly yours,

ROMIG ENGINEERS, INC.

Coleman Ng

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GEOTECHNICAL INVESTIGATION
VAN CRUYNINGEN RESIDENCE
835 WESTRIDGE DRIVE
PORTOLA VALLEY, CALIFORNIA

PREPARED FOR:

MR. IKE VAN CRUYNINGEN

835 WESTRIDGE DRIVE

PORTOLA VALLEY, CALIFORNIA 94028

PREPARED BY:

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MAY 2006

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GEOTECHNICAL INVESTIGATION
FOR
VAN CRUYNINGEN RESIDENCE
835 WESTRIDGE DRIVE
PORTOLA VALLEY, CALIFORNIA

INTRODUCTION

This report presents the results of our geotechnical investigation for the proposed van Cruyningen residence to be constructed at 835 Westridge Drive in Portola Valley, California. The approximate location of the site is shown on the Vicinity Map, Figure 1. The purpose of this investigation was to evaluate subsurface conditions at the site and to provide geotechnical recommendations for design and construction of the proposed residence.

Project Description

The project consists of constructing a two-story, single-family residence with an attached garage in the western portion of the subject site. The proposed residence has a footprint area of approximately 3,800-square-feet and will have a partial basement below the southwestern half. A guest house is proposed northwest of the main residence. The site slopes moderately down to the east. A septic system and leech field will be located downslope (east) of the proposed residence. Structural loads are expected to be relatively light as is typical for this type of construction.

Scope of Work

Our scope of work for this investigation was presented in our agreement with Mr. Ike van Cruyningen dated March 29, 2006. In order to accomplish our investigation, we performed the following work.

- Review of geologic and seismic conditions in the site vicinity.

- Subsurface exploration consisting of drilling, sampling, and logging of four exploratory borings near the proposed residence and guest house.

- Laboratory testing of selected samples to aid in soil classification and to help evaluate their engineering properties.

- Engineering analysis and evaluation of surface and subsurface information to develop geotechnical design criteria for the proposed residence and guest house.
- Preparation of this report presenting our findings and geotechnical recommendations for the proposed project.

Limitations

This report has been prepared for the exclusive use of Mr. Ike van Cruyningen for specific application to developing geotechnical recommendations for the currently proposed residence at 835 Westridge Drive in Portola Valley, California. We make no warranty, expressed or implied, except that our services have been performed in accordance with geotechnical engineering principles generally accepted at this time and location. This report was prepared to provide engineering opinions and recommendations only. In the event there are any changes in the nature, design, or location of the residence, or if any future improvements are planned, the conclusions and recommendations presented in this report should not be considered valid unless 1) the changes are reviewed by us, and 2) the conclusions and recommendations presented in this report are modified or verified in writing.

The analysis, conclusions, and recommendations presented in this report are based on site conditions as they existed at the time of our investigation; the currently planned construction; review of previous reports relevant to the site conditions; and laboratory test results. In addition, it should be recognized that certain limitations are inherent in the evaluation of subsurface conditions and that certain conditions may not be detected during an investigation of this type. Changes in the information or data gained from any of these sources could result in changes in our conclusions or recommendations. If such changes occur, we should be advised so that we can review our report in light of those changes.

SITE EXPLORATION AND RECONNAISSANCE

Site reconnaissance and subsurface exploration were performed on April 27, 2006. Four exploratory borings were advanced to depths ranging from approximately 4.7 to 14 feet below ground surface using portable Minuteman drilling equipment. The approximate locations of the borings are shown on the Site Plan, Figure 2. Logs of these borings and the results of our laboratory tests on selected soil samples are attached in Appendices A and B, respectively.

Surface Conditions

The site is located in a residential area along the southeast side of Westridge Drive. At the time of our investigation, the site was occupied by a single-story, wood-frame residence that had a wood siding exterior. An attached two-car garage was located at the western wing of the residence. An asphalt concrete driveway extended from the garage to Westridge Drive. A brick walkway was located along the northwest side of the residence, and a wooden deck was located along the east side of the residence.

The existing residence was located on a relatively flat building pad southwest of the building area for the proposed residence. The ground surface north and east of the existing residence sloped down to the north and east, respectively, at inclinations ranging from about 3:1 to 8:1 (horizontal:vertical). The at-grade portions of the proposed residence and guest house will be located on a moderately sloping area. The site was landscaped with native grasses, small to large shrubs, and small to large trees.

The existing residence appeared to be supported on a conventional continuous and isolated spread footing foundation system. Where exposed, no obvious cracks or signs of distress were observed on the perimeter foundation stem walls. Several hairline to 1/8-inch-wide cracks were present on the asphalt concrete driveway, and vertical offsets up to about 1/2-inch were noted on portions of the walkway. The swimming pool appeared to have performed in a satisfactory manner. Roof downspouts generally discharged adjacent to the perimeter foundations of the residence.

Subsurface Conditions

At the location of Exploratory Boring EB-1, we encountered about 1 foot of stiff, sandy clay of low to moderate plasticity overlying soft, severely weathered claystone/shale and clayey sandstone bedrock of the Whiskey Hill Formation. Sampler refusal occurred in the clayey sandstone bedrock at a depth of about 4.7 feet below existing grade.

At the location of Boring EB-2, we encountered about 4 feet of fill consisting of firm to stiff, sandy clay of low to moderate plasticity overlying soft, severely weathered sandstone bedrock of the Whiskey Hill Formation.

At the locations of Borings EB-3 and EB-4, we encountered about 1 to 2 feet of firm to stiff, sandy clay of low to moderate plasticity overlying about 1 to 2 feet of residual soil consisting of stiff, sandy clay of low to moderate plasticity. These surface and near-surface soils were underlain by soft, severely weathered sandstone bedrock of the Whiskey Hill Formation. Sampler refusal occurred in Boring EB-3 at a depth of about 5.4 below existing grade.

A Liquid Limit of 35 and a Plasticity Index of 18 were measured on a sample of surface soil obtained from Boring EB-2. These tests results indicate the surface and near-surface soils on the site have a low to moderate potential for expansion.

A free-swell test performed on a sample of claystone/shale bedrock obtained from Boring EB-1 resulted in free-swell of 70 percent, indicating the claystone/shale bedrock at the site has moderate potential for expansion. Free-swell tests performed on samples of sandstone and clayey sandstone from the site resulted in free-swell in the range of 10 to 50 percent, indicating the sandstone and clayey sandstone at the site has relatively low potential for expansion.

Ground Water

Free ground water was not encountered in our borings during drilling and sampling. It should be noted, however, that perched water conditions could develop in the surface soils during periods of rainy weather and fluctuations in the level of ground water could occur due to variations in rainfall, landscaping, surface and subsurface drainage patterns, and other factors.

GEOLOGIC SETTING

As part of our investigation, we reviewed our local experience and geologic literature pertinent to the general area of the site. The information reviewed indicates the site is underlain by sandstone (Tbu) and shale (Tbs) bedrock of the Whiskey Hill Formation, which was formerly called Butano Formation (Town of Portola Valley Geologic Map, 1984). The Town of Portola Valley's Map titled "Movement Potential of Undisturbed Ground" (1984) characterizes the condition of the slopes in the area of the proposed residence and guest house as "relatively stable ground," Sbr. The Town's Movement Potential Map, which is presented on Figure 4, identifies an active landslide in the southern portion of the site: this area is characterized as an "area with significant potential for downslope movement of ground," Pd and Md. This potentially unstable area is at least 100 feet from the southern end of the proposed residence and will have no direct influence on the proposed constrcutoin.

The subject lot and the immediate site vicinity are located in an area that slopes down to the east (approximately 10 feet vertically per 50 feet laterally, although locally the topography may be steeper). Surface elevations in the proposed building area range from about elevation 600 to 620 feet above sea level.

Seismicity

The San Francisco Bay Area is located in an active seismic region. The faults most likely to produce large earthquakes in the area include the San Andreas, San Gregorio, Hayward, and Calaveras Faults. The San Andreas and San Gregorio Faults are located approximately 1.1 and 12 miles southwest of the site, respectively. The Hayward and Calaveras Faults are located approximately 18 and 22 miles northeast of the site, respectively. The estimated maximum magnitude of earthquakes along these faults, and selected historical earthquakes with an estimated magnitude greater than 6.0 that have been generated by these faults, are presented in Table 1 below. The site is not located within an Alquist-Priolo Fault Rupture Hazard Zone (formerly known as a Special Studies Zone), an area where the potential for fault rupture is considered probable. Thus, the main hazard from earthquakes is expected to be related to the strong ground shaking that is produced.

Table 1. Earthquake Magnitudes and Historical Earthquakes
Van Cruyningen Residence
Portola Valley, California

Fault	Maximum Magnitude	Historical Earthquakes	Estimated Magnitude
San Andreas	8.3	1989 Loma Prieta	6.9
		1906 San Francisco	8.3
		1865 N. of 1989 Loma Prieta Earthquake	6.5
		1838 San Francisco-Peninsula Segment	6.8
		1836 East of Monterey	6.5
Hayward	7.3	1868 Hayward	6.8
		1858 Hayward	6.8
Calaveras	7.3	1984 Morgan Hill	6.2
		1911 Morgan Hill	6.2
		1897 Gilroy	6.3
San Gregorio	7.3	1926 Monterey Bay	6.1

A panel of experts convened in 1999 by the U. S. Geological Survey concluded there is a 70 percent chance of at least one "large" earthquake of Magnitude 6.7 or larger in the Bay Area before 2030. They also concluded there could be more than one earthquake of this magnitude and numerous "moderate" earthquakes of about magnitude 6 during this same timeframe. The San Andreas Fault has the second highest likelihood of producing a large earthquake in the Bay Area, estimated as a 21 percent chance of a Magnitude 6.7 or larger earthquake, while the Hayward Fault has the highest likelihood of rupture (32 percent) during the next 30 years (Working Group, 1999).

Earthquake Design Parameters

The International Conference of Building Officials (ICBO) released the 1997 Uniform Building Code (UBC), which contained major revisions to the seismic design approach presented in earlier versions of the UBC. The main geotechnical related revision was that structural design must consider near-source effects for active faults (Holocene-age displacements in the past 11,000 years) located within 15 kilometers of the site. This can result in higher design lateral earthquake forces than in the previous code for structures located close to active faults. The 1997 UBC seismic design philosophy was also clarified under Division IV - Earthquake Design, Section 1626 - General. It reads: "1626.1 Purpose. The purpose of the earthquake provisions herein is primarily to safeguard against major structural failures and loss of life, not to limit damage or maintain function." If the proposed residence and guest house will be designed in accordance with the 1997 UBC, or the 2001 California Building Code, the following factors should be considered.

The site is located within Seismic Zone 4; therefore a Seismic Zone Factor, Z, of 0.40 applies to the site. Based on site geology and subsurface conditions encountered at the site, Soil Profile Type S_C (very dense soil and soft rock) applies to the site. Since the site is located about 1.8 kilometers from the active San Andreas Fault, Near-Source Factors of Na = 1.50 and Nv = 2.0 should be assumed for design.

Geologic Hazards

As part of our investigaiotn, we reviewed the potential for geologic hazards to impact the proposed residence and guest house considering the geologic setting and the soils encountered during our investigation. The results of our review are presented below.

- Fault Rupture - The site is not located in an Alquist-Priolo Fault Rupture Hazard Zone or zone where fault rupture is considered likely. Therefore, active faults are not believed to exist beneath the site, and the potential for fault rupture to occur at the site is considered low.

- Ground Shaking - The site is located in an active seismic area. Moderate to large earthquakes are probable along several active faults in the greater Bay Area over a 30 to 50 year design life. Severe ground shaking should be expected at the building site several times during the design life of the proposed residence. The residence and guest house should be designed and constructed in accordance with current earthquake resistance standards.

- Differential Compaction - Differential compaction occurs during moderate and large earthquakes when soft or loose, natural or fill soils are densified and settle, often unevenly across a site. None of the soils encountered in our borings are prone to significant differential compaction including the existing sandy clay fill in Boring EB-2. If the proposed residence and guest house are constructed on foundations supported on competent soil or weathered bedrock, and site grading is performed as recommended in this report, the potential for damage to the proposed structures and site improvements due to differential compaction during an earthquake is low.

- Liquefaction - Liquefaction occurs when loose to medium dense, saturated sandy soils lose strength during earthquake shaking. Ground settlement and lateral spreading can accompany liquefaction. Soils most susceptible to liquefaction are saturated, loose, silty sands, uniformly graded sands, and sandy silts. The stiff clays and bedrock encountered at the site have very low potential to liquefy even during severe ground shaking that will occur at the site during a major earthquake.

CONCLUSIONS

From a geotechnical viewpoint, the site is suitable for the proposed residence and guest house provided the recommendations presented in this report are followed during design and construction. The primary geotechnical concerns for the proposed residence and site improvements are the presence of up to about 4 feet of fill in portions of the site, the moderately sloping nature of the site, and the presence of potentially unstable slopes in areas mapped as Pd and Md, as shown on Figure 4. In order to reduce risk of future differential movements across the proposed residence and guest house, we recommend the guest house and at-grade portions of the residence be constructed on a pier and grade beam foundation system supported in weathered bedrock. Due to the possible presence of potentially unstable slopes in the portions of the property mapped as Pd and Md, we do not recommend locating a storm water dissipation trench east of the proposed residence, as shown on Figure 2 since this area is mapped as Pd. A preferred location for on-site storm water discharge is near the northern corner of the property. The narrow strip of land mapped as Ps along the eastern side of the property downslope of the preferred discharge location contains a shallow natural creek channel. Specific geotechnical recommendations for the project are presented in the following sections of this report.

Because subsurface conditions may vary from those encountered at the locations of our exploratory borings, and to observe that our recommendations are properly implemented, we recommend that we 1) review the project plans for conformance with the recommendations in this report and 2) observe and test during earthwork and foundation construction.

FOUNDATIONS

Pier and Grade Beam Foundation

The guest house and at-grade portions of the proposed residence should be supported on a pier and grade beam foundation system. Piers should have a diameter of at least 16 inches and should extend at least 8 feet below bottoms of the grade beams, and at least 8 feet into residual soil or weathered bedrock, whichever is deeper. The piers may be designed for an allowable skin friction value 550 pounds per square foot for dead plus live loads with a one-third increase allowed when considering additional short-term wind or seismic loading. Skin friction support against the upper 2 feet of the piers should be neglected in design. An allowable skin friction value of 400 pounds per square foot may be assumed when considering resistance to uplift loading of the piers. All piers should be reinforced in the vertical direction with at least four No. 5 bars.

Grade beams should be designed to span between the piers as determined by the structural engineer. Perimeter grade beams should extend at least 6-inches below slab subgrades and at least 6-inches below the surface of the crawl-space to help reduce the potential for seepage of water under the grade beams and into the crawl-space or into the soil subgrade below slabs-on-grade. All grade beams should be reinforced with the equivalent of at least two No. 4 bars, top and bottom, to provide structural continuity and to allow spanning of local irregularities.

Pier drilling should be observed by a member of our staff to confirm that the piers are constructed with at least the recommended minimum dimensions, are bearing in competent materials, and are properly cleaned of all loose and soft soil or debris. The minimum pier depths recommended above may require adjustment if differing conditions are encountered during drilling. Due to the variable hardness of the sandstone at the site, a drill bit equipped with carbide teeth or other suitable rock drilling equipment will be required.

Reinforcing steel and concrete should be placed in the pier holes as soon as practical after drilling of the pier holes is completed. Although it is unlikely, it is possible that ground water may seep into the pier holes during and/or after drilling. Any water that seeps into the pier holes should be pumped out prior to placement of concrete.

Thirty-year differential movement due to static loads is not expected to exceed approximately 1/2-inch across the recommended pier and grade beam foundation system.

We recommend the foundation piers be designed to resist an active soil pressure equal to an equivalent fluid pressure of 125 pounds per cubic foot, acting on the upper 2 feet of the piers, in the downhill direction. The active pressure may be assumed to act against one and a half pier diameters. The active load and other lateral loads may be resisted by passive earth pressure based upon an equivalent fluid pressure of 350 pounds per cubic foot, acting on 1.5 times the projected area of the pier below a depth of 2 feet. Passive resistance against the upper 2 feet of the piers should be neglected in design.

Basement Foundation

The basement and basement retaining walls should be supported on a mat foundation. The mat may be designed to impose pressures on foundation soils of up to 3,000 pounds per square foot for dead plus live loads with a one-third increase allowed when considering additional short-term wind or seismic loading. A vertical modulus of subgrade reaction of 40 pounds per cubic inch may be assumed for the mat subgrade.

The mat should be reinforced to provide structural continuity and to permit spanning of local irregularities. The bottom of the mat excavation should be cleaned of all loose and soft soil and debris. A member of our staff should observe the basement excavation to evaluate whether scarification and recompaction of the excavation bottom is needed.

A damp-proofing system designed by a specialist in damp-proofing systems should be installed below and around the edges of the mat foundation and behind the basement walls.

Thirty year differential movement due to static loads is not expected to exceed 1/2-inch across the basement structure supported on a mat foundation. Post-construction differential movements between the basement structure and portions of the main residence constructed at-grade are expected to be less than 1/2-inch.

Lateral loads will be resisted by friction between the bottom of mat foundation and the supporting subgrade. A coefficient of friction of 0.3 may be assumed for design. In addition to friction, lateral resistance will also be provided by passive soil pressure acting against the edge of the mat foundation where the mat is cast neat against the basement excavation or where the mat is backfilled with properly compacted structural fill. An allowable passive soil resistance based on an equivalent fluid pressure of 350 pounds per cubic foot may be used for design, where appropriate. Passive resistance provided by the upper 2 feet of soil or weathered bedrock should be neglected in design.

RETAINING WALLS

Retaining walls should be designed to resist lateral earth pressure from the adjacent soil and backfill. We recommend that walls with level backfill that are not restrained against lateral movement, such as the basement walls, be designed to resist an equivalent fluid pressure of 40 pounds per cubic foot plus an additional uniform lateral pressure of 8H pounds per square foot, where H is the height of the wall. Retaining walls with sloping backfill as steep as about 3:1 (horizontal:vertical), should be designed to resist an equivalent fluid pressure of at least 65 pounds per cubic foot, plus 8H psf as previously defined. Unrestrained retaining walls that are able to rotate laterally may be designed without the 8H surcharge pressure described above. Where walls will be subjected to surcharge loads, such as from foundations, adjacent residences, or construction loading, the walls should be designed for an additional uniform lateral pressure equal to one-half of the surcharge pressure.

A subsurface drainage system should be installed behind retaining walls to prevent buildup of water pressure from surface water infiltration and/or underground seepage. The drainage system should consist of a 4-inch diameter perforated pipe (perforations placed down) located at the base of the wall. The perforated pipe should be embedded in a section of 1/2- to 3/4-inch clean, crushed rock at least 12 inches wide. The free-draining 1/2- to 3/4-inch crushed rock should extend up the back of the retaining walls to within about 1.5 feet of finished grade. A filter fabric, such as Mirafi 140N, should be wrapped around the crushed rock to protect it from infiltration of native soil. The upper 1.5 feet of retaining wall backfill should consist of compacted on-site clay. The drainage pipe should slope to a free draining outlet and cleanouts should be provided. Damp-proofing of retaining walls should be included in areas such as the basement where wall dampness and efflorescence would be undesirable.

Miradrain, Enkadrain, or other drainage fabrics approved by our office may be used for wall drainage as an alternative to the gravel drainage system described above. If used, the drainage fabric should extend from a depth of 1 foot below the wall backfill down to the drainage pipe at the base of the wall. A 12-inch wide section of 1/2- to 3/4-inch crushed rock and filter fabric should be placed around the drain pipe, as recommended above.

Backfill placed behind the walls should be compacted to at least 90 percent relative compaction, using light compaction equipment. If heavy compaction equipment is used, the walls should be temporarily braced.

Basement retaining walls should be supported on a mat foundation designed in accordance with the recommendations presented previously.

SLABS-ON-GRADE

Concrete walkways and exterior flatwork should be at least 4 inches thick and should be constructed on at least 6 inches of Class 2 aggregate base. The soil subgrade and aggregate base should be prepared and compacted as recommended in the section of this report titled "Earthwork." In addition, considering the potential for relatively small movements of the surface and near-surface soils, we expect that reinforced slabs will perform better than unreinforced slabs. Consideration should be given to using a control joint spacing of no more than about 10 feet. It would be desirable to provide a thickened edge or moisture cutoff along the edges of slabs adjacent to landscape areas to limit infiltration of water below the edges of the slabs and into the underlying subgrade.

Concrete slab-on-grade floors for any at-grade portions of the residence and garage should be constructed on a layer of non-expansive fill at least 6 inches thick. Slab subgrades should be prepared and compacted as recommended in the section of this report titled "Earthwork."

In areas where floor dampness would be undesirable, such as in building interiors, concrete slabs-on-grade should be constructed on at least 4 inches of clean, free-draining gravel, such as 1/2- to 3/4-inch crushed rock with no more than 5 percent passing the ASTM No. 200 sieve, covered with a high quality, UV-resistant membrane vapor barrier. Pea gravel should not be used as the capillary break material. The vapor barrier may be covered with a 2-inch thick layer of sand to protect it during construction. If sand is used over the vapor barrier, the sand should be lightly moistened with water just prior to concrete placement. The sand and gravel layers may be considered as the non-expansive fill recommended above.

Although it is unlikely that ground water will rise to the level of the basement floor, it is our opinion that a sub-slab drainage system should be installed below the basement slab. The subslab drainage system should consist of a minimum 6-inch thick blanket of free-draining gravel, such as 1/2- or 3/4-inch crushed rock with no more than 5 percent passing the ASTM No. 200 sieve. The subgrade below the gravel blanket should be sloped at an inclination of about 2 percent to a subdrain pipe or pipes running the full length of the basement. The subdrain pipe(s) should consist of 4-inch diameter perforated PVC with perforations placed down. The pipe(s) should be sloped to discharge by gravity into a sump. A filter fabric, such as Mirafi 140N or equivalent, should be installed between the soil subgrade and the crushed rock. A damp-proofing system should be installed below the mat slab and around the basement walls to reduce the potential for dampness of the basement structure. A schematic sketch of the basement subslab drainage system described above is presented on Figure 5.

EARTHWORK

Clearing and Subgrade Preparation

All deleterious materials, such as vegetation, root systems, topsoil, fill, existing slabs and foundations, utiliries, etc., should be cleared from the areas of the site to be built on or paved. Excavations that extend below finished grade should be backfilled with structural fill compacted as recommended below.

After the site has been properly cleared, stripped, and excavated to the required grades, the exposed soil surfaces in areas to receive structural fill or slabs-on-grade should be scarified to a depth of 6 inches, moisture conditioned, and compacted as recommended for structural fill in the section titled "Compaction."

We do not anticipate that any significant fills will be placed on this site. However, if structural fill is to be placed and compacted on sloping portions of the site, the fill should be benched and keyed into underlying competent native soil or weathered bedrock. If significant fills are required, we can provide benching and keying criteria, as appropriate. If excess soil is to be disposed of in the "meadow" area northeast of the residence, the ground should be stripped of vegetation and organic material and the fill should be placed in thin lifts and at least nominally compacted to reduce the potential for instability and/or surface erosion.

If a temporary ramp is constructed to access the basement excavation, the ramp should be properly backfilled and compacted in accordance with our recommendations.

Material For Fill

All on-site soil containing less than 3 percent organic material by volume (ASTM D2974) should be suitable for use as structural fill. Structural fill placed at the site should not contain rocks or pieces larger than 6 inches in greatest dimension and no more than 15 percent larger than 2.5 inches. Imported non-expansive fill should have a Plasticity Index no greater than 15, should be predominately granular, and should have sufficient binder so as not to slough or cave into foundation excavations or utility trenches. Proposed import materials should be approved by a member of our staff prior to delivery to the site.

Finished Slopes

Finished slopes should be cut or filled no steeper than about 3:1 (horizontal:vertical). Exposed slopes may be subject to minor sloughing and erosion requiring periodic maintenance. We recommend that all slopes and soil surfaces disturbed during construction be planted with erosion resistant vegetation.

Temporary Slopes and Excavations

The contractor should be responsible for the design and construction of all temporary slopes and any required shoring. Shoring and bracing should be provided in accordance with all applicable local, state, and federal safety regulations, including current OSHA excavation and trench safety standards.

Temporary slopes less than 5 feet deep excavated in the native soils or weathered bedrock should be capable of standing near-vertical for short construction periods with minimal bracing. Field modification of temporary cut slopes may be required. Unstable materials encountered on slopes during and after excavation should be trimmed off even if this requires cutting the slopes back to a flatter inclination.

Protection of structures near cuts will also be the responsibility of the contractor. A preconstruction survey is generally performed to document existing conditions prior to construction, with intermittent monitoring of adjacent structures during construction.

Compaction

Scarified surface soils and all structural fill should be compacted in uniform lifts, no thicker than 8-inches in uncompacted thickness, conditioned to the appropriate moisture content, and compacted as recommended for structural fill in Table 2 below. The relative compaction and moisture content recommended in Table 2 is relative to ASTM Test D1557, latest edition.

Table 2. Compaction Recommendations
Van Cruyningen Residence
Portola Valley, California

General	**Relative Compaction***	**Moisture Content***
• Scarified subgrade in areas to receive structural fill.	90 percent	At least 2 percent above optimum
• Structural fill.	90 percent	At least 2 percent above optimum
Pavement Areas		
• Upper 6-inches of soil below aggregate base.	95 percent	Above optimum
• Aggregate base.	95 percent	Near optimum
Utility Trench Backfill		
• On-site soils	90 percent	Above optimum
• Imported sand	95 percent	Near optimum

* Relative to ASTM D1557, latest edition.

Surface Drainage

Finished grades should be designed to prevent ponding and to direct surface water away from foundations, edges of slabs and pavements, and toward suitable collection and discharge facilities. Slopes of at least 2.5 percent are recommended. At a minimum, splash blocks should be provided at the ends of downspouts to carry surface water at least 6 feet away from perimeter foundations. Preferably, downspout drainage should be collected in a closed pipe system that discharges into a storm drain system or other suitable discharge outlet.

Drainage facilities should be observed to verify that they are adequate and that no adjustments need to be made, especially during first two years following construction. We recommend an as-built plan showing the locations of surface and subsurface drain lines and clean-outs be prepared. Drainage facilities should be periodically checked to verify they are functioning properly and to be cleaned of silt and debris that mat collect in the lines.

FUTURE SERVICES

Plan Review

Romig Engineers should review the completed grading and foundation plans for conformance with the recommendations presented in this report. We should be provided with these plans as soon as possible upon completion in order to limit the potential for delays in the permitting process that might otherwise be attributed to our review process. In addition, it should be noted that many of the local building and planning departments now require "clean" geotechnical plan review letters prior to acceptance of plans for their final review. Since our plan reviews often result in recommendations for modification of the plans, our generation of a "clean" review letter often requires two iterations. At a minimum, we recommend the following note be added to the plans.

"All earthwork, slab subgrade preparation, pier drilling, foundation and slab construction, retaining wall construction and backfilling, and surface drainage should be performed in accordance with the geotechnical report prepared by Romig Engineers, Inc., dated May 31, 2006. Romig Engineers should be notified at least 48 hours in advance of any earthwork and foundation construction and should observe and test during earthwork and foundation construction as recommended in the geotechnical report."

Construction Observation and Testing

The earthwork and foundation phases of construction should be observed and tested by us to 1) confirm that subsurface conditions are compatible with those used in the analysis and design; 2) observe compliance with the design concepts, specifications, and recommendations; and 3) allow design changes in the event that subsurface conditions differ from those anticipated. The recommendations in this report are based on a limited number of borings. The nature and extent of variation across the site may not become evident until construction. If variations are exposed during construction, it will be necessary to reevaluate our recommendations.

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REFERENCES

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California Division of Mines and Geology, July 1, 1974, State of California Special Studies Zones Maps, Mindego Hill 7.5' Quadrangle.

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Schwartz, D.P., 1994, New Knowledge of Northern California Earthquake Potential, in Proceedings of Seminar on New Developments in Earthquake Ground Motion Estimation and Implications for Engineering Design Practice, Applied Technology Council.

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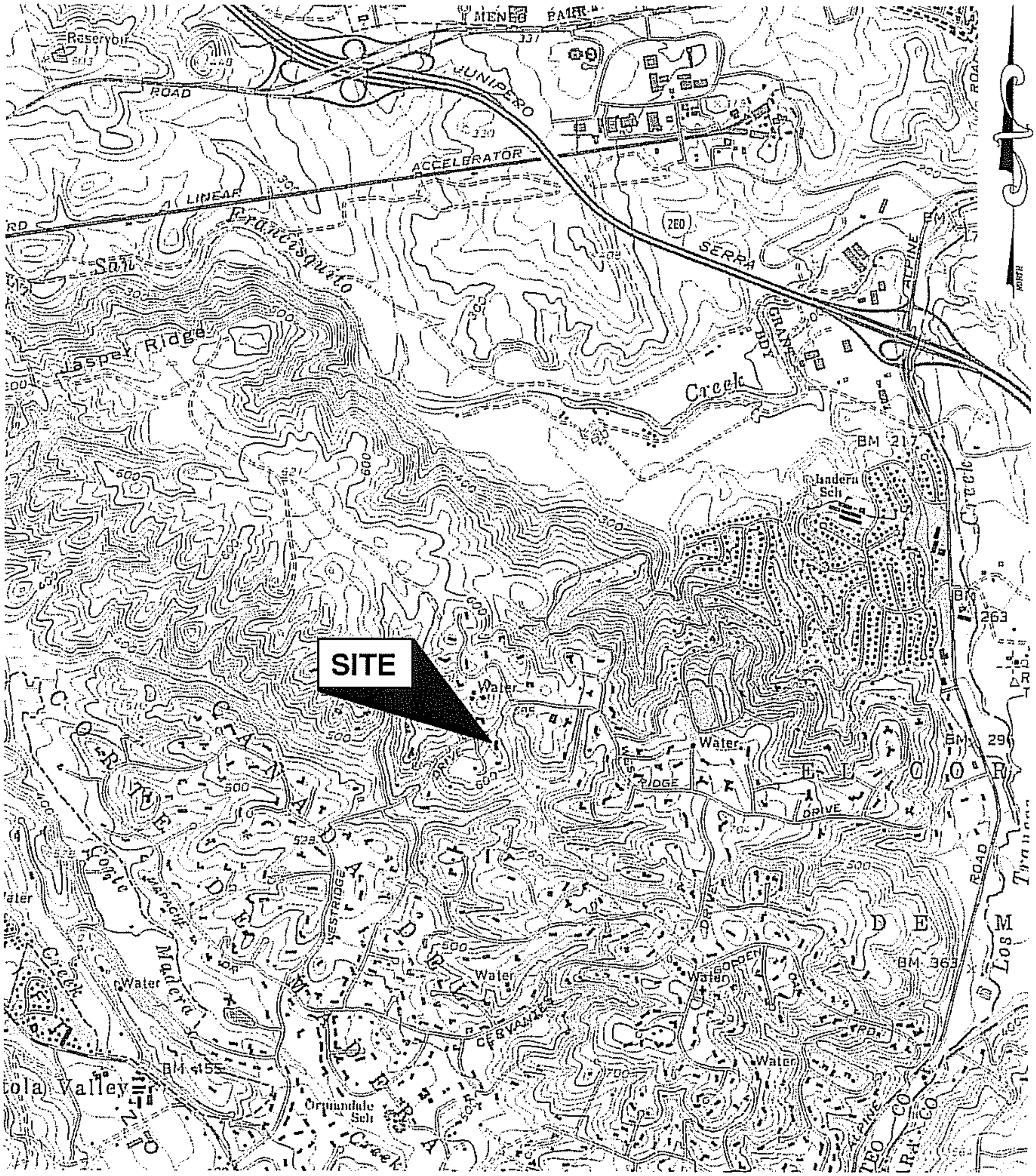
Town of Portola Valley, 1984, Movement Potential of Undisturbed Ground, Town of Portola Valley, San Mateo County, California.

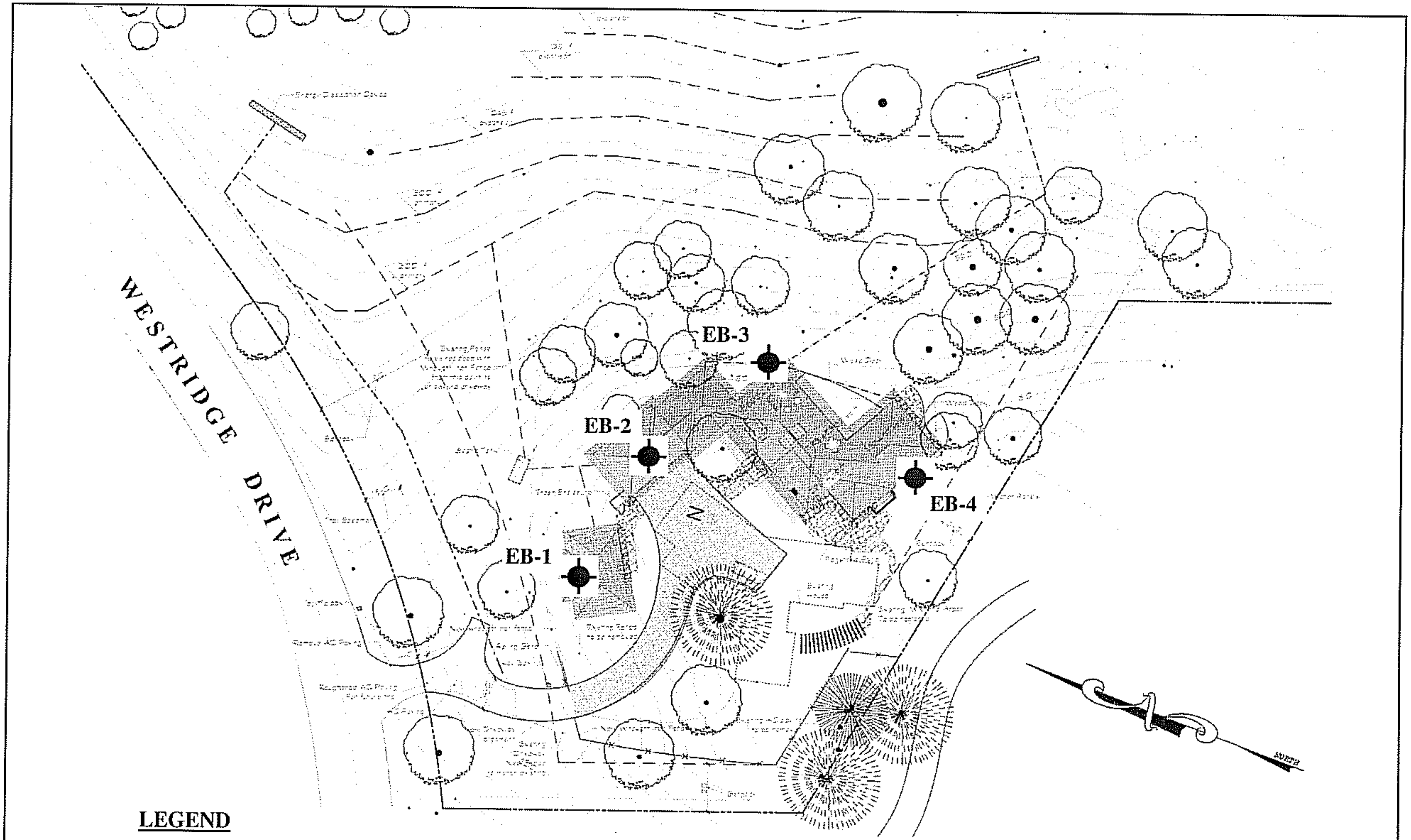
Working Group on California Earthquake Probabilities, 1999, Earthquake Probabilities in the San Francisco Bay Region: 2000 to 2030, U.S. Geological Survey Circular 1189.



Scale: 1 inch = 2000 feet
Base is USGS Palo Alto 7.5 Minute Palo Alto Quadrangle, dated 1961, photorevised 1968 and 1973.

VICINITY MAP
VAN CRUYNINGEN RESIDENCE
PORTOLA VALLEY, CALIFORNIA

FIGURE 1
MAY 2006



LEGEND

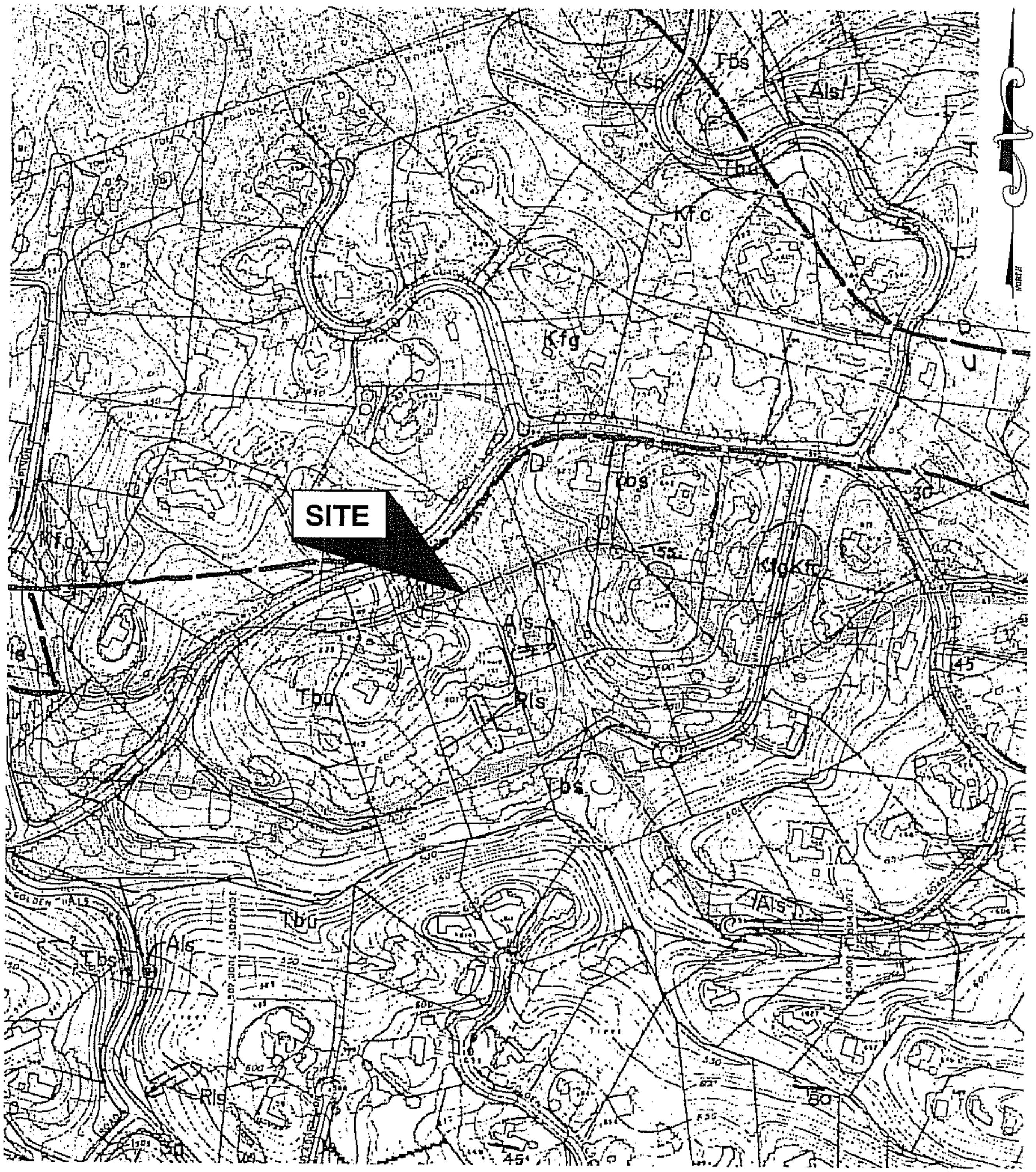
EB-1 — Approximate Location of Exploratory Boring.

Approximate Scale: 1 inch = 60 feet.

Base is site plan provided by F. John Richards, Architect, dated April 28, 2006.

SITE PLAN
VAN CRUYNINGEN RESIDENCE
PORTOLA VALLEY, CALIFORNIA

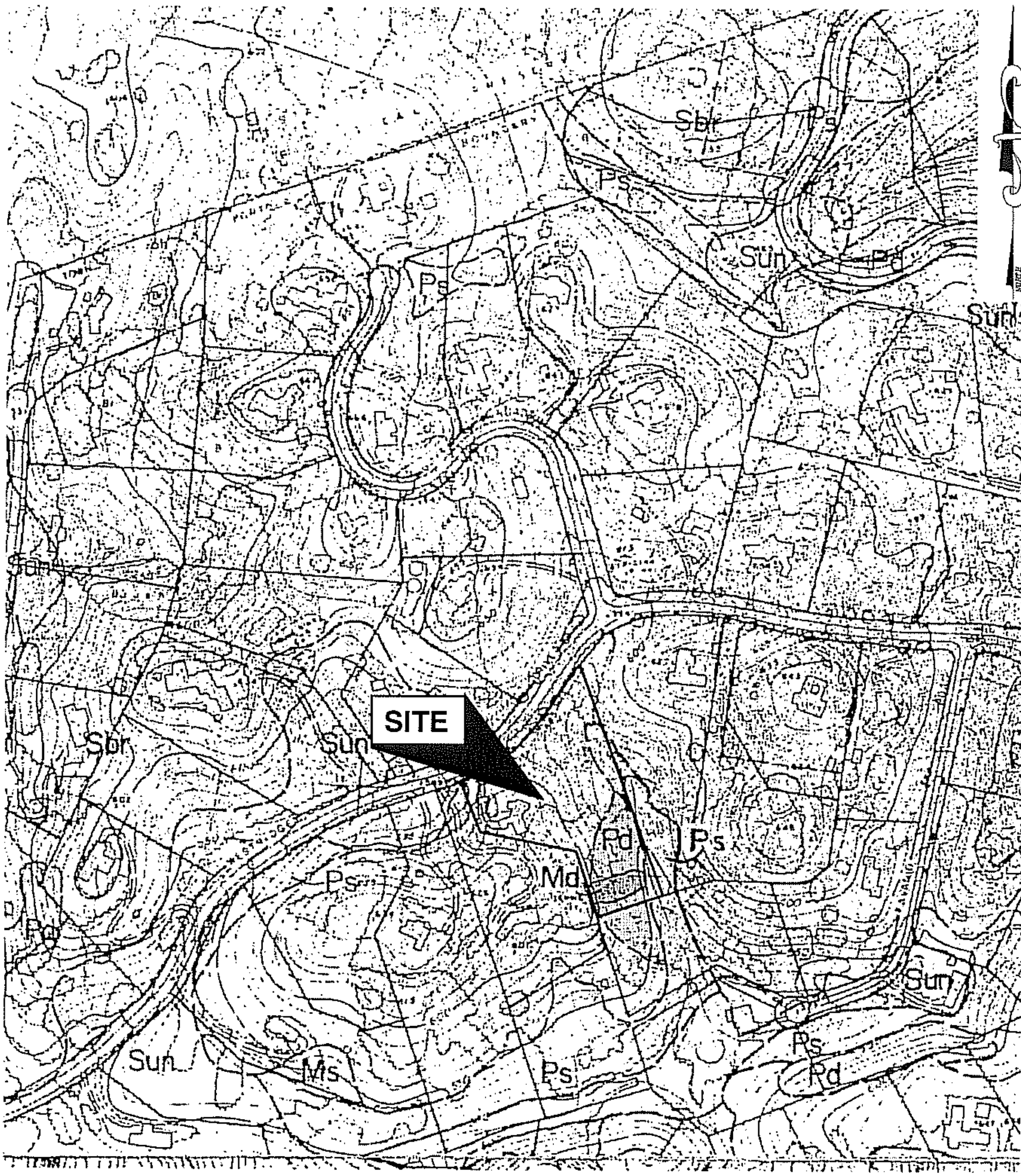
FIGURE 2
MAY 2006



Scale: 1 inch = 500 feet
Base is Town of Portola Valley Geologic Map , William Spangle & Associates, November 26, 1975.

VICINITY GEOLOGIC MAP
VAN CRUYNINGEN RESIDENCE
PORTOLA VALLEY, CALIFORNIA

FIGURE 3
MAY 2006

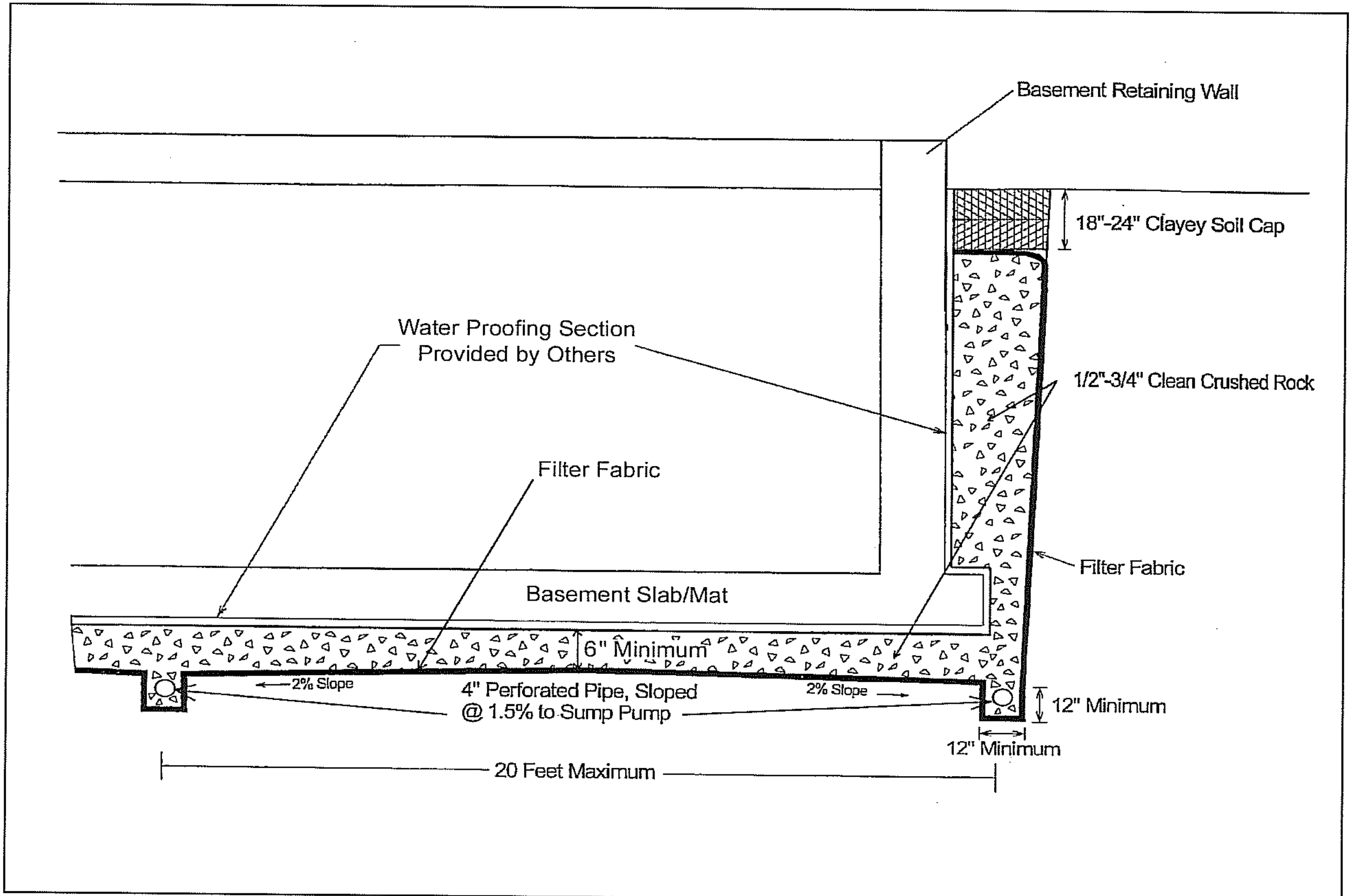


Scale: 1 inch = 400 feet
Base is Town of Portola Valley Movement Potential Map , William Spangle & Associates, November 26, 1975.

TOWN'S MOVEMENT POTENTIAL MAP
VAN CRUYNINGEN RESIDENCE
PORTOLA VALLEY, CALIFORNIA

FIGURE 4
MAY 2006

ROMIG ENGINEERS, INC.



SUBSLAB DRAINAGE DETAIL
VAN CRUYNINGEN RESIDENCE
PORTOLA VALLEY, CALIFORNIA

FIGURE 5
MAY 2006

APPENDIX A

FIELD INVESTIGATION

The soils encountered during drilling were logged by our representative and samples were obtained at depths appropriate to the investigation. The samples were taken to our laboratory where they were examined and classified in accordance with the Unified Soil Classification System. The logs of our borings and a summary of the soil classification system (Figure A-1) and bedrock descriptions used on the logs (Figure A-2) are attached.

Several tests were performed in the field during drilling. The standard penetration resistance was determined by dropping a 140-pound hammer through a 30-inch free fall, and recording the blows required to drive the 2-inch (outside diameter) sampler 18 inches. The standard penetration test (SPT) resistance is the number of blows required to drive the sampler the last 12 inches, and is recorded on the boring log at the appropriate depth. Soil samples were also collected using 2.5-inch and 3.0-inch O.D. drive samplers. The blow counts shown on the logs for these larger samplers do not represent SPT values and have not been corrected in any way.

The locations of the borings were determined by pacing using the site plan provided to us. The locations of the borings should be considered accurate only to the degree implied by the method used.

The boring logs and related information depict our interpretation of subsurface conditions only at the specific location and time indicated. Subsurface conditions and ground water levels at other locations may differ from conditions at the locations where sampling was conducted. The passage of time may also result in changes in the subsurface conditions.

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USCS SOIL CLASSIFICATION

PRIMARY DIVISIONS			SOIL TYPE	SECONDARY DIVISIONS
COARSE GRAINED SOILS (< 50 % Fines)	GRAVEL	CLEAN GRAVEL (< 5% Fines)	GW	Well graded gravel, gravel-sand mixtures, little or no fines.
			GP	Poorly graded gravel or gravel-sand mixtures, little or no fines.
		GRAVEL with FINES	GM	Silty gravels, gravel-sand-silt mixtures, non-plastic fines.
			GC	Clayey gravels, gravel-sand-clay mixtures, plastic fines.
	SAND	CLEAN SAND (< 5% Fines)	SW	Well graded sands, 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, non-plastic fines.
			SC	Clayey sands, sand-clay mixtures, plastic fines.
FINE GRAINED SOILS (> 50 % Fines)	SILT AND CLAY Liquid limit < 50%		ML	Inorganic silts and very fine sands, with slight plasticity.
			CL	Inorganic clays of low to medium plasticity, lean clays.
			OL	Organic silts and organic clays of low plasticity.
	SILT AND CLAY Liquid limit > 50%		MH	Inorganic silt, micaceous or diatomaceous fine sandy or silty soil.
			CH	Inorganic clays of high plasticity, fat clays.
			OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS			Pt	Peat and other highly organic soils.
BEDROCK			BR	Weathered bedrock.

RELATIVE DENSITY

SAND & GRAVEL	BLOWS/FOOT*
VERY LOOSE	0 to 4
LOOSE	4 to 10
MEDIUM DENSE	10 to 30
DENSE	30 to 50
VERY DENSE	OVER 50

CONSISTENCY

SILT & CLAY	STRENGTH^	BLOWS/FOOT*
VERY SOFT	0 to 0.25	0 to 2
SOFT	0.25 to 0.5	2 to 4
FIRM	0.5 to 1	4 to 8
STIFF	1 to 2	8 to 16
VERY STIFF	2 to 4	16 to 32
HARD	OVER 4	OVER 32

GRAIN SIZES

BOULDERS	COBBLES	GRAVEL		SAND			SILT & CLAY
		COARSE	FINE	COARSE	MEDIUM	FINE	

12 " 3" 0.75" 4 10 40 200

SIEVE OPENINGS U.S. STANDARD SERIES SIEVE

Classification is based on the Unified Soil Classification System; fines refer to soil passing a No. 200 sieve.

* Standard Penetration Test (SPT) resistance, using a 140 pound hammer falling 30 inches on a 2 inch O.D. split spoon sampler; blow counts not corrected for larger diameter samplers.

^ Unconfined Compressive strength in tons/sq. ft. as estimated by SPT resistance, field and laboratory tests, and/or visual observation.

KEY TO SAMPLERS

- Modified California Sampler (3-inch O.D.)
- Mid-size Sampler (2.5-inch O.D.)
- Standard Penetration Test Sampler (2-inch O.D.)

KEY TO EXPLORATORY BORING LOGS
VAN CRUYNINGEN RESIDENCE
PORTOLA VALLEY, CALIFORNIA

FIGURE A-1
MAY 2006

WEATHERING

Fresh
Rock fresh, crystals bright, few joints may show slight staining. Rock rings under hammer if crystalline.

Very Slight
Rock generally fresh, joints stained, some joints may show thin clay coatings, crystals in broken face show bright. Rock rings under hammer if crystalline.

Slight
Rock generally fresh, joints stained, and discoloration extends into rock up to 1 inch. Joints may contain clay. In granitoid rocks some occasional feldspar crystals are dull and discolored. Crystalline rocks ring under hammer.

Moderate
Significant portions of rock show discoloration and weathering effects. In granitoid rocks, most feldspars are dull and discolored; some are clayey. Rock has dull sound under hammer and shows significant loss of strength as compared with fresh rock.

Moderately Severe
All rock except quartz discolored or stained. In granitoid rocks, all feldspars dull and discolored and majority show kaolinization. Rock shows severe loss of strength and can be excavated with geologist's pick. Rock goes "clunk" when struck.

Severe
All rock except quartz discolored or stained. Rock "fabric" clear and evident, but reduced in strength to strong soil. In granitoid rocks, all feldspars kaolinized to some extent. Some fragments of strong rock usually left.

Very Severe
All rock except quartz discolored and stained. Rock "fabric" discernible, but mass effectively reduced to "soil" with only fragments of strong rock remaining.

Complete
Rock reduced to "soil". Rock fabric not discernible or discernible only in small scattered locations. Quartz may be present as dikes or stringers.

HARDNESS

Very hard
Cannot be scratched with knife or sharp pick. Hand specimens requires several hard blows of geologist's.

Hard
Can be scratched with knife or pick only with difficulty. Hard blow of hammer required to detach hand specimen.

Moderately Hard
Can be scratched with knife or pick. Gouges or grooves to 1/4 inch deep can be excavated by hard blow of point of a geologist's pick. Hard specimen can be detached by moderate blow.

Medium
Can be grooved or gouged 1/16 inch deep by firm pressure on knife or pick point. Can be excavated in small chips to pieces about 1 inch maximum size by hard blows of the point of a geologist's pick.

Soft
Can be gouged or grooved readily with knife or pick point. Can be excavated in chips to pieces several inches in size by moderate blows of a pick point. Small thin pieces can be brocken by finger pressure.

Very Soft
Can be carved with knife. Can be excavated readily with point of pick. Pieces 1 inch or more in thickness can be broken with finger pressure. Can be scratched readily by fingernail.

JOINT BEDDING AND FOLIATION SPACING

Spacing	Joints	Bedding and Foliation
Less than 2 in.	Very Close	Very Thin
2 in. to 1 ft.	Close	Thin
1 ft. to 3 ft.	Moderately Close	Medium
3 ft. to 10 ft.	Wide	Thick
More than 10 ft.	Very Wide	Very Thick

ROCK QUALITY DESIGNATOR (RQD)

RQD, as a percentage	Descriptor
Exceeding 90	Excellent
90 to 75	Good
75 to 50	Fair
50 to 25	Poor
Less than 25	Very Poor

KEY TO BEDROCK DESCRIPTIONS
VAN CRUYNINGEN RESIDENCE
PORTOLA VALLEY, CALIFORNIA

FIGURE A-2
MAY 2006

DRILL TYPE: Minuteman with 3-1/4" Continuous Flight Auger **LOGGED BY:** WAB

DEPTH TO GROUND WATER: Not Encountered. **SURFACE ELEVATION:** NA **DATE DRILLED:** 4/27/06

CLASSIFICATION AND DESCRIPTION	SOIL CONSISTENCY/ DENSITY or ROCK HARDNESS* (Figure A-2)	SOIL TYPE	SOIL SYMBOL	DEPTH (FEET)	SAMPLE INTERVAL	SPT RESISTANCE (Blows/ft)	WATER CONTENT (%)	SHEAR STRENGTH (TSF)*	UNCONFIN. COMP. (TSF)*
Topsoil: Medium to dark brown, Sandy Clay, moist, fine to coarse sand, low to moderate plasticity.	Stiff	CL		0			16	1.1	2.7
Whiskey Hill Formation: Medium to dark olive brown, Claystone/Shale, moist, severely weathered.	Soft	BR				15			
▲ Free Swell = 70%.					▲		16		4.0
Becoming light to medium olive brown, Clayey Sandstone.						86	12		3.7
						50/4"	20		
				5		50/6"	18		
Bottom of Boring at 4.7 Feet.									
				10					
				15					

Note: The stratification lines represent the approximate boundary between soil and rock types, the actual transition may be gradual.

*Measured using Torvane and Pocket Penetrometer devices.

EXPLORATORY BORING LOG EB-1 **BORING EB-1**

VAN CRUYNINGEN RESIDENCE MAY 2006

PORTOLA VALLEY, CALIFORNIA

DRILL TYPE: Minuteman with 3-1/4" Continuous Flight Auger **LOGGED BY:** WAB

DEPTH TO GROUND WATER: Not Encountered. **SURFACE ELEVATION:** NA **DATE DRILLED:** 4/27/06

CLASSIFICATION AND DESCRIPTION	SOIL CONSISTENCY/ DENSITY or ROCK HARDNESS* (Figure A-2)	SOIL TYPE	SOIL SYMBOL	DEPTH (FEET)	SAMPLE INTERVAL	SPT RESISTANCE (Blows/ft)	WATER CONTENT (%)	SHEAR STRENGTH (TSF)*	UNCONFIN. COMP. (TSF)*
Fill: Medium to dark brown mottled with orange brown, Sandy Clay, moist, fine to coarse sand, moderate plasticity. ■ Liquid Limit = 35%, Plasticity Index = 18%.	Firm to Stiff	CL		0			23	0.5	0.3
						4			
							16	0.3	0.3
Increasing fine gravel.						16	21	1.1	2.9
Whiskey Hill Formation: Light brown to orange brown, Sandstone, moist, very severely weathered.	Soft	BR		5		40	13		2.8
Becoming orange brown, Clayey Sandstone.						33	16		2.7
Note: The stratification lines represent the approximate boundary between soil and rock types, the actual transition may be gradual.						24	15		1.9
*Measured using Torvane and Pocket Penetrometer devices.				10					
▲ Free Swell = 40%.					▲	44	12		>4.5
Bottom of Boring at 12 Feet.									
				15					

EXPLORATORY BORING LOG EB-2
VAN CRUYNINGEN RESIDENCE
PORTOLA VALLEY, CALIFORNIA

BORING EB-2
MAY 2006

DRILL TYPE: Minuteman with 3-1/4" Continuous Flight Auger **LOGGED BY:** WAB

DEPTH TO GROUND WATER: Not Encountered. **SURFACE ELEVATION:** NA **DATE DRILLED:** 4/27/06

CLASSIFICATION AND DESCRIPTION	SOIL CONSISTENCY/ DENSITY or ROCK HARDNESS* (Figure A-2)	SOIL TYPE	SOIL SYMBOL	DEPTH (FEET)	SAMPLE INTERVAL	SPT RESISTANCE (Blows/ft)	WATER CONTENT (%)	SHEAR STRENGTH (TSF)*	UNCONFIN. COMP. (TSF)*
Topsoil: Medium to dark brown, Sandy Clay, moist, fine to coarse sand, low to moderate plasticity.	Firm to Stiff	CL		0			24	0.6	1.7
Residual Soil: Orange brown, Sandy Clay, moist, fine to coarse sand, low to moderate plasticity.	Stiff	CL				7	18	0.8	1.7
Whiskey Hill Formation: Orange brown, Sandstone moist, severely weathered.	Soft	BR				21	15		2.3
▲ Free Swell = 10%.				5	▲	50/5"	14		
						50/6"	7		
Bottom of Boring at 5.4 Feet.									
				15					

Note: The stratification lines represent the approximate boundary between soil and rock types, the actual transition may be gradual.

*Measured using Torvane and Pocket Penetrometer devices.

EXPLORATORY BORING LOG EB-3 **BORING EB-3**

VAN CRUYNINGEN RESIDENCE MAY 2006

PORTOLA VALLEY, CALIFORNIA

DRILL TYPE: Minuteman with 3-1/4" Continuous Flight Auger **LOGGED BY:** WAB

DEPTH TO GROUND WATER: Not Encountered. **SURFACE ELEVATION:** NA **DATE DRILLED:** 4/27/06

CLASSIFICATION AND DESCRIPTION	SOIL CONSISTENCY/ DENSITY or ROCK HARDNESS* (Figure A-2)	SOIL TYPE	SOIL SYMBOL	DEPTH (FEET)	SAMPLE INTERVAL	SPT RESISTANCE (Blows/ft)	WATER CONTENT (%)	SHEAR STRENGTH (TSF)*	UNCONFIN. COMP. (TSF)*
Topsoil: Medium to dark brown, Sandy Clay, moist, fine to coarse sand, low to moderate plasticity.	Firm to Stiff	CL		0		6	16	0.7	1.2
Residual Soil: Orange brown with gray and olive brown, Sandy Caly, fine to coarse sand, low to moderate plasticity.	Stiff	CL				16	17	0.9	1.8
Whiskey Hill Formation: Olive gray and olive brown, Clayey Sandstone, moist, severely weathered. ▲ Free Swell = 30%.	Soft	BR		5	▲	50	19	1.1	3.8
Note: The stratification lines represent the approximate boundary between soil and rock types, the actual transition may be gradual.						35	13		>4.5
▲ Free Swell = 50%.				10	▲	44	14		>4.5
Becoming orange brown and white, Sandstone, coarse grained.						42	9		>4.5
*Measured using Torvane and Pocket Penetrometer devices.						40	13		
Bottom of Boring at 14 Feet.				15					

APPENDIX B

LABORATORY TESTS

Samples collected during subsurface exploration were selected for tests to help evaluate the physical and engineering properties of the soils that were encountered. The tests that were performed are briefly described below.

The natural moisture content was determined in accordance with ASTM D2216 on nearly all samples recovered from the borings. This test determines the moisture content, representative of field conditions, at the time the samples were collected. The results are presented on the boring logs at the appropriate sample depth.

The Atterberg Limits were determined on one sample of soil in accordance with ASTM D4318. The Atterberg Limits are the moisture content within which the soil is workable or plastic. The results of these tests are presented on Figure B-1 and on the log of Boring EB-2 at the appropriate sample depth.

Free-swell tests were performed on five samples of bedrock encountered in the exploration borings. The results of these tests are presented on the boring logs at the appropriate sample depths.

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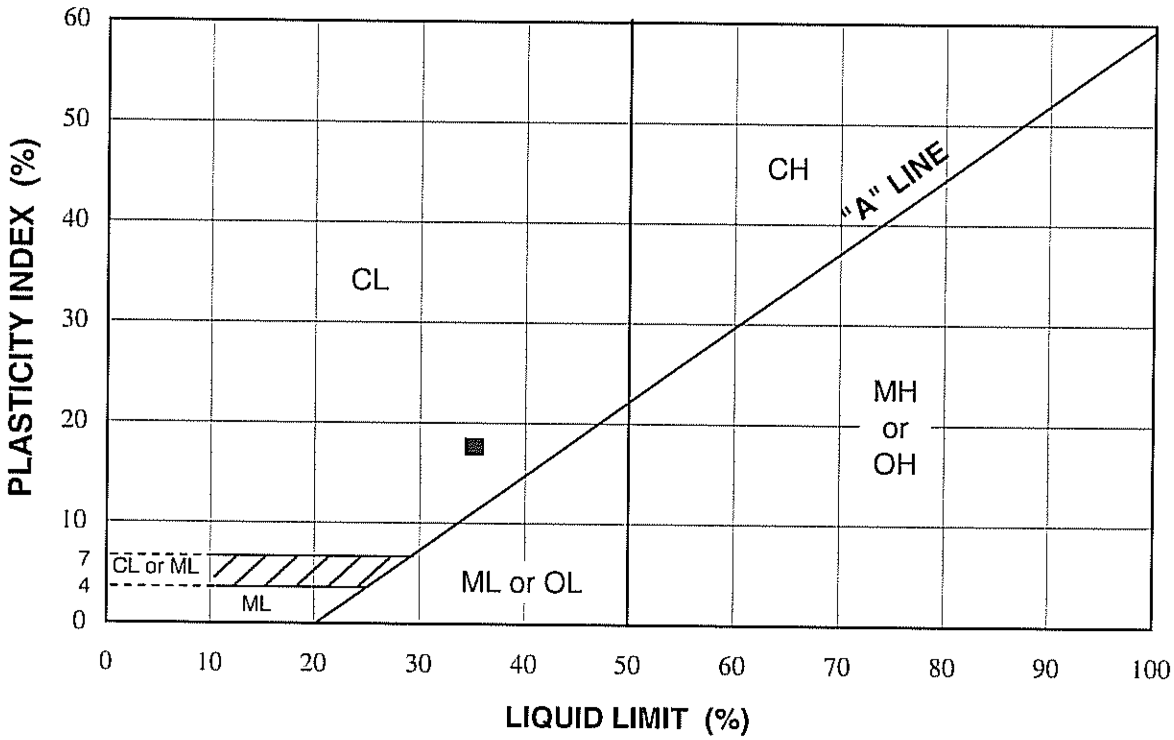

Chart Symbol	Boring Number	Sample Depth (feet)	Water Content (percent)	Liquid Limit (percent)	Plasticity Index (percent)	Liquidity Index (percent)	Passing No. 200 Sieve (percent)	USCS Soil Classification
■	EB-2	0-2	23	35	18	33		CL

PLASTICITY CHART
VAN CRUYNINGEN RESIDENCE
PORTOLA VALLEY, CALIFORNIA

FIGURE B-1
MAY 2006