APPENDIX E

Geology and Soils Background Information

GEOTECHNICAL INVESTIGATION
NEW RESIDENCE
1170 SIGNAL HILL ROAD
PEBBLE BEACH, CALIFORNIA

for

Ms. Massy Mehdipour
1425 Dana Avenue
Palo Alto, CA 94301

by

Cleary Consultants, Inc.
900 N. San Antonio Road
Los Altos, California 94022

March 2010

CLEARY CONSULTANTS, INC.
Geotechnical Engineers and Geologists

J. Michael Cleary, CEG, GE
Christophe A. Ciechanowski, GE
Grant F. Foster, GE

March 31, 2010
Project No. 1301.1
Ser. 2880

Ms. Massy Mehdipour
1425 Dana Avenue
Palo Alto, CA 94301

RE: GEOTECHNICAL INVESTIGATION
NEW RESIDENCE
1170 SIGNAL HILL ROAD
PEBBLE BEACH, MONTEREY COUNTY, CALIFORNIA

Dear Ms. Mehdipour:

As authorized, we have performed a geotechnical investigation for your planned new home on the property at 1170 Signal Hill Road in Pebble Beach, Monterey County, California. The accompanying report presents the results of our field investigation, laboratory testing, and engineering analyses. The site and subsurface conditions are discussed and recommendations for the geotechnical engineering aspects of the project design are presented. The recommendations presented in this report are contingent upon our review of the grading and foundation plans and observation/testing of the earthwork and foundation installation phases of the construction.

Please refer to the text of the report for detailed recommendations. If you have any questions concerning our findings, please call.

Very truly yours,

CLEARY CONSULTANTS, INC.

J. M. CLEARY
No. 222
EXP. 9-30-11

GRANT FOSTER
No. 2662
EXP. 6-30-11

Grant Foster
Geotechnical Engineer 2662

J. Michael Cleary
Engineering Geologist 352
Geotechnical Engineer 222

GF/JMC:cm
Copies: Addressee (2)
Bill Bernstein AIA (4) Attn: William Bernstein
Whitson Engineers (1) Attn: Ken Whitson

J. M. CLEARY
No. 352
Exp 11/30/11

900 N. SAN ANTONIO ROAD • LOS ALTOS, CALIFORNIA 94022 • (650) 948-0574 • FAX (650) 948-7761
www.clearyconsultantsinc.com

TABLE OF CONTENTS

TABLES

DRAWINGS

INTRODUCTION

This report presents the results of our geotechnical investigation for the planned new residence on the property at 1170 Signal Hill Road in the Pebble Beach area of Monterey County, California. The general location of the site is shown on the Site Vicinity Map, Drawing 1. The purpose of this investigation was to explore the soil conditions in the planned new home area and develop recommendations for the geotechnical engineering aspects of the project design.

As indicated on the preliminary architectural plans prepared by Bill Bernstein AIA, November 2009, a new two level home with a basement will be constructed in the area primarily downslope of the existing home, which will be demolished. We understand that the new home will encompass approximately 14,000 square feet and will have a lower floor (basement) Elevation of 87.0 feet on the south portion and a lower floor Elevation of 98.5 feet on the north portion. A garage is planned at Elevation 107.0 feet on the front, or east side, of the residence. Building loads are expected to be typical of two story wood-frame residential construction.

Basement cuts will range up to about 17 feet in height, and new fills of up to about six feet in height are planned.

New driveway and exterior walkways/patios are anticipated for the property, as well as low landscaping walls.

SCOPE

As presented in our proposal agreement dated February 3, 2010, the scope of our services for this investigation has included:

1. A site reconnaissance by our engineer and review of published and unpublished geological information for this area.

2. Subsurface investigation consisting of seven (7) exploratory borings.

3. Laboratory testing of samples obtained from the borings.

4. Engineering analysis of the field and laboratory data.

5. Preparation of this geotechnical investigation report for use in the project design and construction. The report includes findings and recommendations for the following:

    a) Site soil conditions, geologic and seismic setting, and 2007 CBC criteria for seismic design, including liquefaction and dry settlement analysis, and mitigation measures, as required.

    b) Groundwater table, as encountered in the borings.

    c) Site preparation and grading.

    d) New residence foundation type(s), minimum foundation dimensions, and allowable soil engineering design criteria.

    e) Estimated foundation settlements.

f) Lateral earth pressures and equivalent fluid pressures for basement walls, landscape walls and recommendations for retaining wall backdrainage.

g) Driveway pavement section.

h) Support of concrete slabs-on-grade.

i) Surface drainage.

j) Any other unusual design or construction conditions encountered in the investigation.

This report has been prepared for the specific use of Ms. Massy Mehdipour and her consultants in accordance with generally accepted soil and foundation engineering principles and practices. No other warranty, either expressed or implied, is made. In the event that any substantial changes in the nature, design or location of the new residence are planned, the conclusions and recommendations of this report shall not be considered valid unless such changes are reviewed and the conclusions of this report modified or verified in writing. Any use or reliance of this report or the information herein by a third party shall be at such party's sole risk.

It should also be recognized that the passage of time may result in significant changes in technology, building code requirements, state of the practice, economic conditions, or site variations which would render the report inaccurate. Accordingly, neither the owners, nor any other party, should rely on the information or conclusions contained in this report after three years from its date of issuance without the express written consent of Cleary Consultants, Inc.

METHOD OF INVESTIGATION

A site reconnaissance and the subsurface exploration were performed on February 19, 2010, under the guidance of our engineer. Seven borings were drilled to a maximum depth of 31.0 feet at the locations shown on Drawing 3, Site Plan, using a track mounted hollow-stem auger drill rig. A key describing the soil classification system and soil consistency terms used in this report is presented on Drawing 6 and the soil sampling procedures are described in Drawing 7. Logs of the borings are presented on Drawings 10 through 18.

The borings were located in the field by pacing and interpolation of the features shown on the Site Plan provided us. These locations should be considered accurate only to the degree implied by the methods used. The elevations shown on the boring logs were taken from the topographic plan provided us.

Samples of the soil materials from the borings were returned to our laboratory for classification and testing. The results of moisture content, dry density, percent finer than No. 200 sieve, gradation, free swell, corrosion and plasticity index determinations are shown on the boring logs. Additional information on the plasticity index, corrosion, gradation testing is presented on Drawings 19 through 21.

A list of references consulted during this investigation is included at the end of the text.

GEOLOGY AND SEISMICITY

The subject property is located in the Cypress Point area of Pebble Beach, approximately 600 feet inland of Fan Shell Beach and the Pacific Ocean (See Drawing 1). This area is characterized by shoreline bluffs and low cliffs which are generally capped by recent (Holocene age) dune sand deposits, underlain by eroded granodiorite bedrock. The site is about 100 feet above sea level.

Drawing 2, Local Geologic Map, shows the site vicinity, extending for a distance of about 2000 feet inland, to be underlain by dune sand deposits (Qd). These deposits (Dupre, 1990) are up to 25 meters thick, unconsolidated, and consist of well drained medium to coarse grained loose sand with a poorly developed or absent organic soil horizon. The dune sand is subject to "accelerated erosion … in areas where vegetation (is) disturbed or removed".

Porphyritic granodiorite (Kgdp) is the underlying bedrock type in the Cypress Point area, forming resistant coastal bluffs and rocky outcrops. The granodiorite (Clark et al, 1997) is "light gray to moderately pink and medium grained with orthoclase phenocrysts ranging from three to ten centimeter long." The granodiorite is variably weathered, ranging from highly decomposed (d.g. materials) to fresh to slightly weathered crystalline rock.

The major controlling active faults in this region are the San Andreas fault located 29.5 miles northeast of the site, the San Gregorio-Palo Colorado fault which lies 3.5 miles offshore to the southwest and the Monterey Bay-Tularcitos fault which lies approximately 5.0 miles northeast of the site (Blake, 2000). In addition to the above active faults, the Cypress Point fault, considered potentially active, is mapped (Clark et al, 1997) about 1000 feet southwest of the site as a concealed trace beneath coastal terrace deposits (Qct). Therefore, as with the rest of the Monterey Bay area, the property is in a region of high seismic activity.

SITE CONDITIONS

A. Surface

As indicated on the Site Plan, Drawing 3, the new home will be built on an irregular previously graded and terraced site, which has an overall fall of about 20 feet from east to west across the new building footprint. The upper portion of the site includes a two level residence which appears to have been cut into the slope, with the lower level at Elevation 95 and the upper portion approximately ten feet higher, (roughly at street grade). The backyard has been terraced with a 50 to 75 foot wide gently sloping to flat area at Elevation 80 to 85, marking the outer/downhill limits of the planned new home. Further west, the dune sand terrain falls away at an overall gradient of approximately 25 percent toward 17 Mile Drive and the ocean.

Grasses, small shrubs and scattered trees were present on the property at the time of our investigation, however the backyard and terraced areas below the existing structure were largely un-vegetated dune sand. Several hard granodiorite bedrock outcrops are present on the parcel, including one at the bedroom wing of the proposed home (see Drawing 3 for general location). As measured in the field, the bedrock jointing strikes moderately to the northwest and dips strongly southward.

B. Subsurface

The exploratory borings encountered approximately eight to 14 feet of predominantly loose, medium to fine grained, slightly moist to dry cohesionless clean sand overlying one to five feet of loose to medium dense silty to clayey sand, in turn overlying very dense weathered granodiorite bedrock to 31.0 feet, the maximum depth explored. Refusal of the CME 55 auger drill rig was encountered at depths of 13.0, 31.0, 13.5 and 18.5 feet in EB-1, EB-3, EB-4 and EB-6.

The upper clean sand is non-plastic and non-expansive (plasticity index and free swells = zero) while the underlying silty to clayey sand has a low to moderate expansion potential (plasticity index = 17 percent and free swells of zero to 50 percent) based on the test data.

The attached boring logs and related information depict subsurface conditions only at the specific locations shown on Drawing 3 and on the particular date designated on the logs. Soil conditions at other locations may differ from conditions occurring at these locations. Also, the passage of time may result in a change of conditions at the boring locations due to environmental changes.

Subsurface profiles A-A', B-B', and C-C' depicting interpreted subsurface conditions through the building site are presented on Drawings 4 and 5.

C. Groundwater

Free water was encountered at depths of 9.5, 16 and 10.5 feet in EB-1, EB-2 and EB-7 during drilling; free water was not encountered in the remaining exploratory borings during the investigation. The borings were only open for a period of a few hours, however, and this may not have been a sufficiently long enough period to establish the stabilized water table conditions. It should also be noted that fluctuations of localized perched groundwater can be expected to occur due to such factors as variations in rainfall, temperature, runoff, irrigation, and other factors not evident at the time our measurements were made and reported herein.

GEOLOGIC AND SEISMIC HAZARDS EVALUATION

A. Fault Offset Hazard

Based on the findings of this investigation, we conclude that there are no known active or potentially active faults crossing the proposed building site. The site is also not within an Earthquake Fault Zone as defined by the State of California Alquist-Priolo Earthquake Fault Zoning Act. Therefore, the hazard resulting from surface fault rupture or fault offset at the site is considered very low.

B. Ground Shaking Hazards

1. Strong Ground Shaking

Strong ground shaking is likely to occur during the lifetime of the planned new home as a result of movement along one or more of the regional active faults discussed above. The new home and other improvements will need to be designed and constructed in accordance with current standards of earthquake-resistant construction.

Ground shaking during an earthquake could cause furnishings which are not rigidly attached to undergo movement with respect to the building. Design measures that minimize such potential movement and also minimize the adverse effects of such movement where they cannot be prevented should be utilized.

2. Soil Liquefaction

Liquefaction is a phenomenon in which saturated, essentially cohesionless soils lose strength during strong seismic shaking and may experience horizontal and vertical movements. Soils that are generally most susceptible to liquefaction are clean, loose, saturated, uniformly graded, fine-grained sands and silts that lie within roughly 50 feet of the ground surface.

The site is shown to lie within a moderate to low susceptibility for liquefaction zone as shown on the liquefaction susceptibility map for Monterey County (Dupre, 1990).

Our investigation found that the homesite is underlain by predominantly non-saturated loose to medium dense clean sand and silty sand underlain by granodiorite bedrock. Based on these conditions, we conclude that the likelihood of soil liquefaction during strong ground shaking at the site is low; however, the silty sand layer encountered below the observed groundwater table of 10.5 feet in EB-7 was conservatively analyzed for liquefaction-induced settlement using the LiquefyPro computer program (Version 5.0).

LiquefyPro evaluates liquefaction potential and calculates the settlement of saturated and unsaturated deposits due to seismic loads using SPT blowcount, total unit weight, fines content, peak horizontal acceleration and earthquake moment magnitude data. The program is based on the most recent publications of the NCEER Workshop and SP117 Implementation.

Based on the results of our analysis, the theoretical liquefaction-induced settlement is approximately one-half inch at the site using the calculated peak ground acceleration (S_{DS}/2.5) for the site as specified in Item Number 23 of CGS Note 48 and the Tokimatsu and Seed calculation method with magnitude scaling correction. The results and supporting data for the liquefaction analysis are included in Appendix A of this report.

3. **Soil Densification**

The recognized procedures for evaluation of seismically-induced settlement in dry sandy soils (Tokimatsu and Seed, 1987; Pradel, 1998) are considered most applicable to non-cohesive loose clean sands with less than 5 percent fines (Day, 2002). The loose to medium dense clean sand, silty sand and clayey sand layers encountered in EB-5 and EB-7 were analyzed for seismically-induced settlement using the LiquefyPro computer program.

The maximum calculated earthquake induced dry soil settlement for these layers is approximately three and one-half inches using the calculated peak ground acceleration (S_{DS}/2.5) for the site as specified in Item Number 23 of CGS Note 48. As subsequently recommended, the home will be supported on a structural slab with drilled caissons extending into granodiorite bedrock. Based on the above, the likelihood that the new home will experience distress as a result of earthquake-induced soil densification is very low.

The results and supporting data for the dry settlement analysis are included in Appendix A of this report.

4. Other Seismic Hazards

We have also considered the possibility of other seismically induced hazards at the site. Because the sandy soils overlying the granodiorite are unsaturated, with the exception of local perched water, soil lurching and lateral spreading are considered unlikely.

Ground cracking may be caused by any of the phenomena discussed above. Since there is a low potential for liquefaction-induced settlement and lateral spreading of the soils underlying the site, it is also considered unlikely that significant ground cracking will occur at the site.

Based on the findings of our investigation and review of published geologic maps, the site is not underlain by any known landslides.

C. Flooding

The site is outside of the runup zone resulting from a seismically generated tsunami as shown on the Tsunami Inundation Map for Emergency Planning, State of California, County of Monterey, July 1, 2009. This map shows the tsunami inundation limits to be roughly the route of 17 Mile Drive in the vicinity of Signal Hill Road, approximately 400 feet west of the planned homesite.

CONCLUSIONS AND RECOMMENDATIONS

From a geotechnical engineering standpoint, we conclude that the property can be developed as proposed provided the recommendations contained in this report are incorporated into the design and construction of the project. The new home will be built in an area that is underlain by loose dune sand of variable thickness and low bearing capacity, and these materials could experience differential settlement beneath building foundations and slabs. Accordingly, we recommend that a structural slab that is supported on drilled pier foundations obtaining skin friction support in the granodiorite bedrock be used for the new home. In our opinion, the above foundation system will provide a high degree of structural rigidity under the anticipated building and retaining wall loads with minimal risk of settlement.

Heavy duty drilling equipment in good condition will be required to achieve the required penetration into granodiorite bedrock, as discussed further in the report. Portions of the dune sand may require the use of casing prior to installing steel reinforcement and placing concrete. Any seepage encountered in the pier holes should be pumped out prior to concrete placement.

The southeast corner of the home, in the area of the two bedroom wings, is an area of resistant granodiorite bedrock outcropping, and difficult excavation requiring the use of jackhammers or a hoe ram may be required to achieve basement grade in this area. Consideration should be given to relocating the basement slightly to the west to avoid the outcrop. Difficult excavation may also be encountered in other portions of the basement (See Subsurface Profiles A-A' and B-B') in resistant granodiorite rock.

Although only intermittent water was encountered in the exploratory borings, indicating perched water conditions, some surface water infiltration from the surrounding soils at basement level is likely, particularly during peak winter storms. A drainage blanket should be installed beneath the basement structural slab to collect and remove water which may seep into this area. The retaining wall back drainage and basement foundation drain blanket should be drained to a sump and removed with a sump pump system, or to gravity drainage if feasible.

Basement excavations for retaining walls along the uphill side of the home are anticipated to range up to 17 feet in height. It is anticipated that temporary excavations can be made at a 2:1 gradient provided they are protected (winterized) prior to the wet season; however the final design, stability and safety of temporary excavations should be the responsibility of the contractor.

Site retaining walls i.e. those required for driveway and patio areas, that are three feet or less in height can be supported on spread footing foundations after reworking of the underlying loose soil.

Final cut and fill slopes should be no steeper than 3:1 (horizontal to vertical) in dune sand materials. Areas disturbed by grading should be planted prior to the initial winter to minimize erosion and downcutting in the sand.

Detailed recommendations for use in design and construction of the project are presented in the remainder of this report. These recommendations are contingent on our review of the earthwork and foundation plans for the project and our observation of the earthwork and foundation installation phases of construction.

A. Earthwork

1. Clearing and Site Preparation

Areas to be graded should be cleared of existing foundations, slabs, AC pavement, grass, shrubs, trees not designated to remain, and other vegetation as well as any other obstructions including root bulbs, stumps and debris. Holes resulting from the removal of underground obstructions, including tree root bulbs that extend below the planned finished grade, should be cleared of loose soil and backfilled with suitable material compacted to the requirements given below for engineered fill.

After clearing, areas to receive fill should be stripped to a sufficient depth to remove the surface vegetation, wood chips and organic laden topsoil. A stripping depth of two to four inches is anticipated. Strippings should be removed from the property, or stockpiled for later use in landscaped areas, if desired.

2. Recompaction of Surface Soils

After the areas to be graded have been cleared and required excavations have been made, the surface soils within areas to be filled should be recompacted. This work should consist of ripping the upper 12 inches, moisture conditioning the soils to optimum, and compacting them to at least 95 percent relative compaction as determined by ASTM Test Designation D1557. Compaction should be performed using heavy compaction equipment such as a self propelled vibratory smooth-drum roller. Significant addition of water will be required in the in the clean sands, which were dry to slightly moist at the time of our investigation, to achieve the required compaction.

3. Slope Gradients

Permanent cut and fill slopes should be no steeper than 3:1 (horizontal to vertical). Cut and fill slopes should be planted to minimize erosion and surface runoff should be diverted away from the top of slopes and carried to a suitable drainage collection system.

Temporary slopes are anticipated to be reasonably stable at an inclination of 2:1 (horizontal to vertical) provided they are winterized prior to the wet season. However, the contractor should be solely responsible for designing and constructing stable temporary excavations and should shore, slope or bench the excavations as required to maintain their stability and comply with all applicable safety standards, including CAL-OSHA requirements.

4. Fill Placement and Compaction

On-site soils having an organic content of less than three percent by volume can be used as fill. Any imported fill required at the site should be predominantly granular with a plasticity index of 6 or less and should not contain rocks or lumps greater than six inches in greatest dimension with not more than 15 percent larger than 2.5 inches.

Engineered fill should be compacted to at least 95 percent relative compaction, as determined by ASTM Test Designation D1557, including the upper 12 inches of subgrade under new AC pavements. Fill material should be spread and compacted in lifts not exceeding eight inches in uncompacted thickness. The moisture content of on-site soils utilized as fill should be adjusted to their optimum moisture content. Compaction should be performed using heavy compaction equipment such as a self-propelled smooth drum vibratory roller.

In order to achieve satisfactory compaction in the subgrade and fill soils, it may be necessary to adjust the soil moisture content at the time of construction. This may require that water be added and thoroughly mixed into any soils which are too dry or that scarification and aeration be performed in any soils which are too wet.

5. Trench Backfill

Utility trenches should be backfilled with engineered fill placed in lifts not exceeding eight inches in uncompacted thickness, except thicker lifts may be used with the approval of our representative provided satisfactory compaction is achieved. If on-site clean sand soil is used, the material should be compacted to at least 90 percent relative compaction by mechanical means only. Imported sand can also be used for backfilling trenches provided it is also compacted to at least 90 percent relative compaction. In slab and pavement areas, the upper three feet of trench backfill should be compacted to at least 95 percent relative compaction for on-site soils and imported sand.

Water jetting to achieve the required level of compaction should not be permitted.

6. Surface Drainage

Positive surface gradients should be provided away from the top of cutslopes and fillslopes, or surface swales should be installed to divert water from the face of the slope. Ponding of surface water should not be permitted on or adjacent to the building pad, flatwork or new driveway areas.

Positive surface gradients of at least two percent on porous surfaces and one percent on paved surfaces should be maintained away from the new home so that water does not collect in the vicinity of the building foundations. Area drains should be used to promote positive drainage in landscaped and paved areas around the new residence.

Water from roof downspouts should be collected in closed pipes and carried to suitable discharge.

7. Construction Observation

The grading and foundation installation phases of the project should be observed and tested by our representative for conformance with the project plans/specifications and our recommendations. This work includes site preparation and grading, selection of satisfactory fill materials, and placement and compaction of the subgrade, fill and baserock materials. Sufficient notification prior to commencement of earthwork operations is essential to make certain that the work will be properly observed and tested.

B. Structural Slab and Drilled Pier Foundation System

To provide uniform support and settlement performance, we recommend that the new home and garage be supported on a structural slab underlain by drilled piers obtaining skin friction support in the granodiorite bedrock.

The drilled pier foundations should consist of cast-in-place, straight shaft friction piers. The drilled piers should extend through any fill material and the existing native loose sandy soils, and at least six feet into the underlying granodiorite bedrock encountered in the borings at depths of eight to 14 feet. Piers should be spaced no closer than about three diameters center to center with maximum spacing to be determined by the structural engineer. The drilled piers should have a minimum diameter of 24 inches.

The portion of the drilled piers in granodiorite bedrock materials can be designed on the basis of 750 psf skin friction with a 50 percent increase for wind and seismic conditions. Point bearing resistance should generally be neglected, however any piers meeting refusal short of their design depth should be evaluated by our representative for end bearing support (suitability for end bearing will require satisfactory clean out of the pier bottom). For resistance to lateral loads, a uniform passive equivalent fluid pressure of 250 pcf in sand and 500 pcf in granodiorite, up to 4000 psf maximum, can be assumed to act over 1.5 times the projected area of the individual pier shaft. The passive pressure can be assumed to start one foot below the bottom of the structural slab.

Groundwater was encountered in several of the borings during our investigation, and any accumulated water in the pier holes should be removed prior to concrete placement. It is recommended that reinforcing steel and concrete be placed as soon as practical after drilling to minimize drying of the sidewalls and caving. The contractor should be prepared to install steel casing if caving of the pier holes is encountered.

The bottom of the pier excavations should be dry and relatively free of loose soil or fall-in prior to installing reinforcing steel and placing concrete. Since the actual lengths of the piers will depend on the subsurface conditions encountered in the field, the excavation of piers should be performed under the observation of our representative. Heavy duty drilling equipment in good working condition should be used to drill the pier holes. Difficult drilling is anticipated in the less weathered granodiorite portion of the drilled pier excavations.

Drilled piers can be eliminated under the structural slab where competent granodiorite bedrock is encountered at final basement subgrade. It is recommended that additional exploratory borings be performed during the foundation design phase to more precisely determine areas where this is feasible. A vertical modulus of subgrade reaction of 275 pci, or alternatively 2000 psf allowable bearing pressure, can be used for slab design in competent granodiorite.

Reinforcement of the drilled piers should be provided for their full length. Minimum pier reinforcement should consist of four No. 5 bars tied in a cage. Additional reinforcement may be required as determined by the structural engineer.

The structural slab should have a minimum thickness of 12 inches with 18 inches deep by 12 inches wide downturned edges, as a minimum.

Post-construction settlements under the anticipated building loads are expected to be within tolerable limits for the proposed construction.

Moisture vapor transmission can occur upward through the soil resulting in the collection of moisture under slabs and pavements. In any areas where moisture transmission may be detrimental, current industry practice for concrete slabs is to place a vapor retarder, such as a minimum 15 mil thick membrane or an integrally bonded vapor barrier such as Florpruf, or equivalent, on six inches of clean rock, such as ¾ inch crushed drain rock. While vapor barrier systems are the standard of practice for the industry, Cleary Consultants, Inc. does not practice in the field of moisture vapor transmission evaluation or mitigation, and we recommend that a qualified consultant in this field be retained to evaluate any specific moisture vapor transmission issues associated with the project.

To facilitate removal of transient infiltration beneath the basement slab, we recommend that the basement excavation beneath the six inch drain rock section be sloped at least 0.5 percent to a low point and drained either by gravity flow, if feasible, or by a sump pump, into a suitable discharge facility. The sump pump, if required, should be installed on the outside of the home to eliminate concern about the noise from the pump operation.

C. Seismic Design Parameters

Seismic design values for the project were determined using the USGS Earthquake Ground Motion Parameter Java Application, and subsurface information obtained from the exploratory borings was used for determining the site classification. Using the site Latitude (36.5817°N) and Longitude (121.9657°W) and Site Classification C as input, the computer application provides Seismic Hazard Curve information, Site Coefficients and Uniform Hazard Response Spectra for both "short" (0.2 seconds) and "long period" (1-second) durations as detailed in the 2007 CBC.

Based on the results of our investigation, the tables provided in Section 1613 of the 2007 CBC, and our analysis using the USGS Earthquake Ground Motion Parameter Java Application, the following seismic design parameters can be used in lateral force analyses at this site:

Site Class C – Very Dense Soil and Soft Rock with Standard Penetration Test Values >50 blows/foot

Site Coefficient $F_a = 1.0$

Site Coefficient $F_v = 1.3$

Maximum Considered Earthquake Spectral Response (Short Period);
$S_{MS} = (F_a)(S_S) = 1.658$

Maximum Considered Earthquake Spectral Response (1-Second Period);
$S_{M1} = (F_v)(S_1) = 0.939$

Design Spectral Response Acceleration (Short Period);
$S_{DS} = 2/3\ S_{MS} = 1.105$

Design Spectral Response Acceleration (1-Second Period);
$S_{D1} = 2/3\ S_{M1} = 0.626$

Seismic Design Category – D

D. Slabs-on-Grade

Concrete slabs-on-grade are anticipated for new patio and walkway areas. We recommend that following subgrade preparation as previously discussed, exterior concrete flatwork be supported on at least six inches of Class 2 aggregate base. The aggregate base should be compacted to at least 90 percent relative compaction.

E. Retaining Walls

All retaining walls required for the project must be designed to resist lateral earth pressures and any additional lateral loads caused by surcharge loading. Attached retaining walls for the new residence should be supported on the mat slab and drilled pier foundation system designed in accordance with the recommendations provided in Section B. Foundations.

Detached walls three feet or less in height can be supported on spread footings bearing on at least 24 inches of recompacted soil. Spread footings should be a minimum of 1.5 feet wide and bear at a minimum depth of 1.5 feet below the ground surface. Detached retaining wall spread footings bearing on reworked sand can be designed using an allowable bearing pressure of 1500 psf. Lateral loads can be resisted by friction between the foundation bottoms and the supporting subgrade. A friction coefficient of 0.30 is considered applicable. As an alternative, a passive pressure equal to an equivalent fluid pressure of 250 pcf can be taken against the sides of footings poured neat.

Unrestrained walls with either level or sloping backfills no steeper than 3:1 (horizontal to vertical) can be designed to resist an equivalent fluid pressure of 35 pcf and restrained walls can be designed to resist an equivalent fluid pressure of 35 pcf plus an additional uniform lateral pressure of six H psf where H = height of backfill above wall foundation in feet. Where backfill slope gradients exceed 3:1, an additional one and one-half pcf per degree of slope gradient exceeding 18° should be added to the above active pressure distribution. Wherever walls will be subjected to surcharge loads, they should be designed for an additional lateral pressure equal to one-third or one-half the anticipated surcharge load depending on whether the wall is unrestrained or restrained, respectively.

The preceding pressures assume that sufficient drainage is provided behind the walls to prevent the build-up of hydrostatic pressures from surface or subsurface water infiltration. Adequate drainage may be provided by means of a one foot wide vertical drain blanket placed behind the wall. The drain should consist of ¾-inch clean crushed gravel enclosed in a filter fabric, such as Mirafi 140, and a four-inch diameter perforated Schedule 40 or SDR 35 pipe placed at the base of the wall. The gravel should be capped with at least 18 inches of compacted native soil. The perforated pipe should be tied into a closed pipe that discharges to a suitable discharge facility.

Backfill placed behind retaining walls should be non-expansive and compacted to at least 90 percent relative compaction using light weight compaction equipment. If heavy compaction equipment is used, the walls must be appropriately braced to avoid overstressing or failure of the wall.

F. Driveway Pavement Section

The minimum flexible pavement section for new driveways should consist of two and one-half inches asphaltic concrete over six inches Class II aggregate base. The upper 12 inches of soil subgrade and the Class II aggregate base should be compacted to at least 95 percent relative compaction. Class II aggregate base should have an R-Value of at least 78 and conform to the requirements of Section 26, State of California "CALTRANS" Standard Specifications, latest edition.

The asphaltic concrete should conform to and be placed in accordance with the requirements of Section 39 in the State of California "CALTRANS" Standard Specifications.

G. Soil Corrosivity

Laboratory resistivity, pH, chloride and sulfate testing was performed on a soil sample obtained from the upper five feet of the borings during our geotechnical investigation for this project. The testing was performed by Cooper Testing Laboratory for the purpose of evaluating the soils' corrosion potential for use in the design of underground utilities and embedded concrete on this project.

In summary, the test results indicated a minimum resistivity of 16,497 Ohm-Cm, a PH of 6.7, a chloride content of 4 ppm, and water soluble sulfate content of <5 ppm. Soils with chloride contents of less than 500 ppm and sulfate contents of less than <5 ppm are considered to be of "low" corrosivity. Additionally, based on the resistivity testing, the soils are considered to be "progressively less corrosive."

Table 1 below shows the general correlation between resistivity and corrosion potential.

Table 1 - Correlation Between Resistivity and Corrosion Potential (c)

Soil Resistivity (Ohm-Cm)	Soil Classification
Below 500	Very Corrosive
500 to 1,000	Corrosive
1,000 to 2,000	Moderately Corrosive
2,000 to 10,000	Mildly Corrosive
Above 10,000	Progressively Less Corrosive

(c) National Association of Corrosion Engineers.

This condition combined with the slightly acidic condition of the soils encountered at the site could result in a reduced life span of buried steel piping for this project. Thicker gauge pipelines would have greater life spans. For example, the life spans for 18, 16 and 14 gauge steel culverts with a soil resistivity of 16,500 Ohm-Cm and a pH of 6.7 are estimated to be roughly 31, 40 and 50 years, respectively (California Division of Highways, 1993).

For the purposes of design of concrete in contact with the soil, there are no restrictions on types of cementitious materials to be used based on the resistivity and sulfate testing.

PLAN REVIEW AND CONSTRUCTION OBSERVATION

We should be provided the opportunity to review the foundation and grading plans and the specifications for the project when they are available. We should also be retained to provide soil engineering observation and testing services during the grading and foundation installation phases of the project. This will provide the opportunity for correlation of the soil conditions found in our investigation with those actually encountered in the field, and thus permit any necessary modifications in our recommendations resulting from changes in anticipated conditions.

****

LIST OF REFERENCES

Blake, Thomas, 2000, EQFAULT Program for Determination of Ground Acceleration Values for Earthquakes in California.

Blake, Thomas, 2000, New Fault-Model Files For EQFAULT, Modified from California Division of Mines and Geology fault Database for 183 Late-Quaternary California Faults.

Bowen, O.E., 1965, Stratigraphy, Structure, and Oil Possibilities in Monterey and Salinas Quadrangles, California: American Association of Petroleum Geologist 40th Annual Meeting, Special Publication of the Pacific Section, pp. 48-67.

Bryant, W.A., 1985, Faults in the Southern Monterey Bay Area, Monterey County, California: California Division of Mines and Geology, FER 167.

California Building Code, 2007.

Civiltech Software, Liquefy Pro Program, Version 5.0.

Clark, J.C., Dibblee, T.W., Green, H.G., Bowen, O.E., 1974, Preliminary Geologic Map of the Monterey and Seaside 7.5 Minute Quadrangles, Monterey County, California with Emphasis on Active Faults: U.S. Geological Survey, Misc. Field Studies Map MF-577, scale 1:24,000.

Clark, J. C., Dupre, W.R., and Rosenberg, L.I., 1997, Geologic Map of the Monterey and Seaside 7.5 Minute Quadrangles, Monterey County. USGS OFR 97-30.

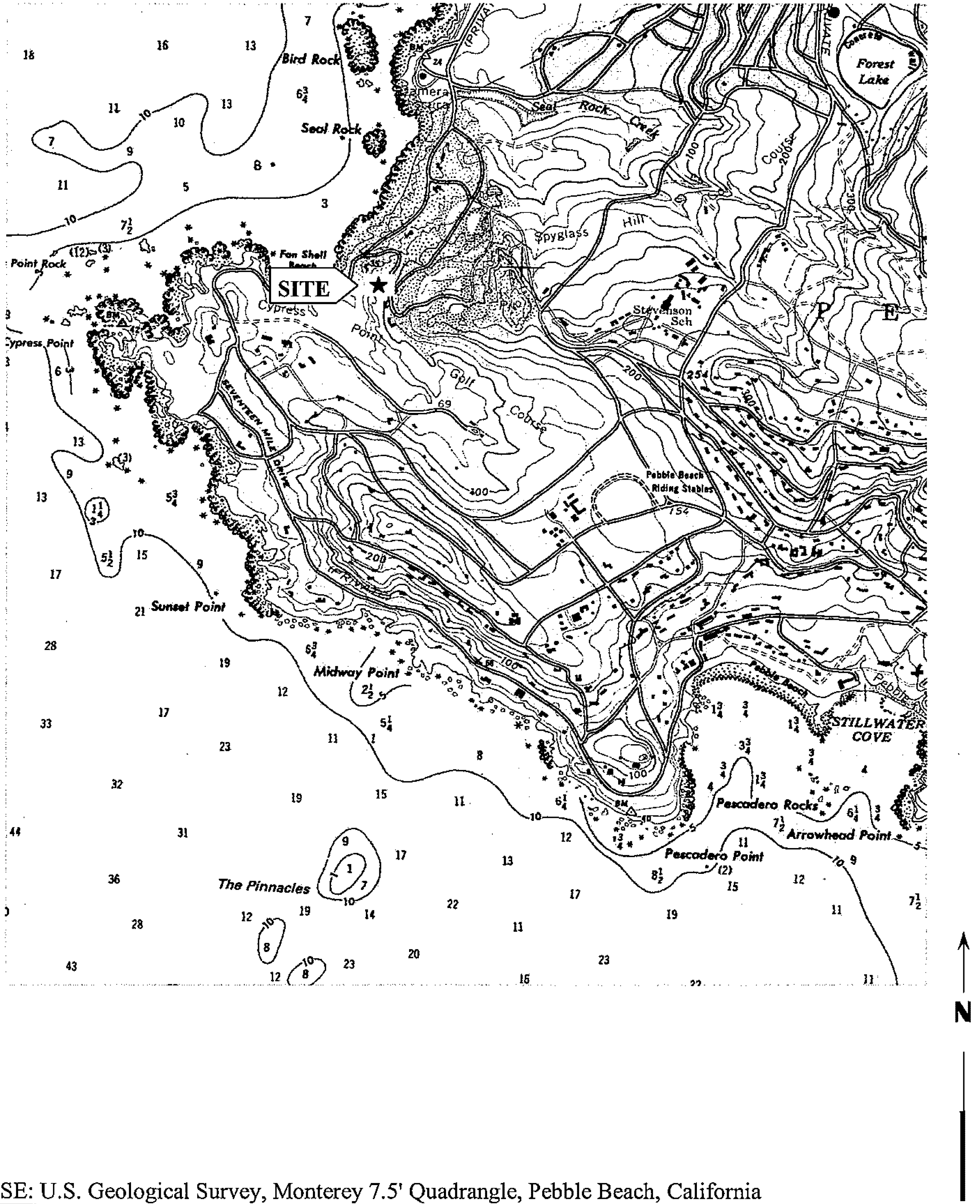
Dearman, W.F., et al, 1978, Engineering Grading of Weathered Granite, Engineering Geology Volume 12 (1978) page 345-374, Elsevier Scientific Publishing Co., Amsterdam.

Dupre, William R., 1990, Maps Showing Geology and Liquefaction Susceptibility of Quaternary Deposits in the Monterey, Seaside, Spreckels, and Carmel Valley Quadrangles, Monterey County, California, USGS MF 2096.

Jennings, C.W., 1994, Fault Activity Map of California and Adjacent Areas with Locations and Ages of Recent Volcanic Eruptions, California Division of Mines and Geology, Geologic Data Map No. 6.

State of California, July 1, 2009, Tsunami Inundation Map for Emergency Planning, County of Monterey, Monterey Quadrangle.

U. S. Geological Survey, Monterey 7 ½' Quadrangle Map, Monterey County, California.



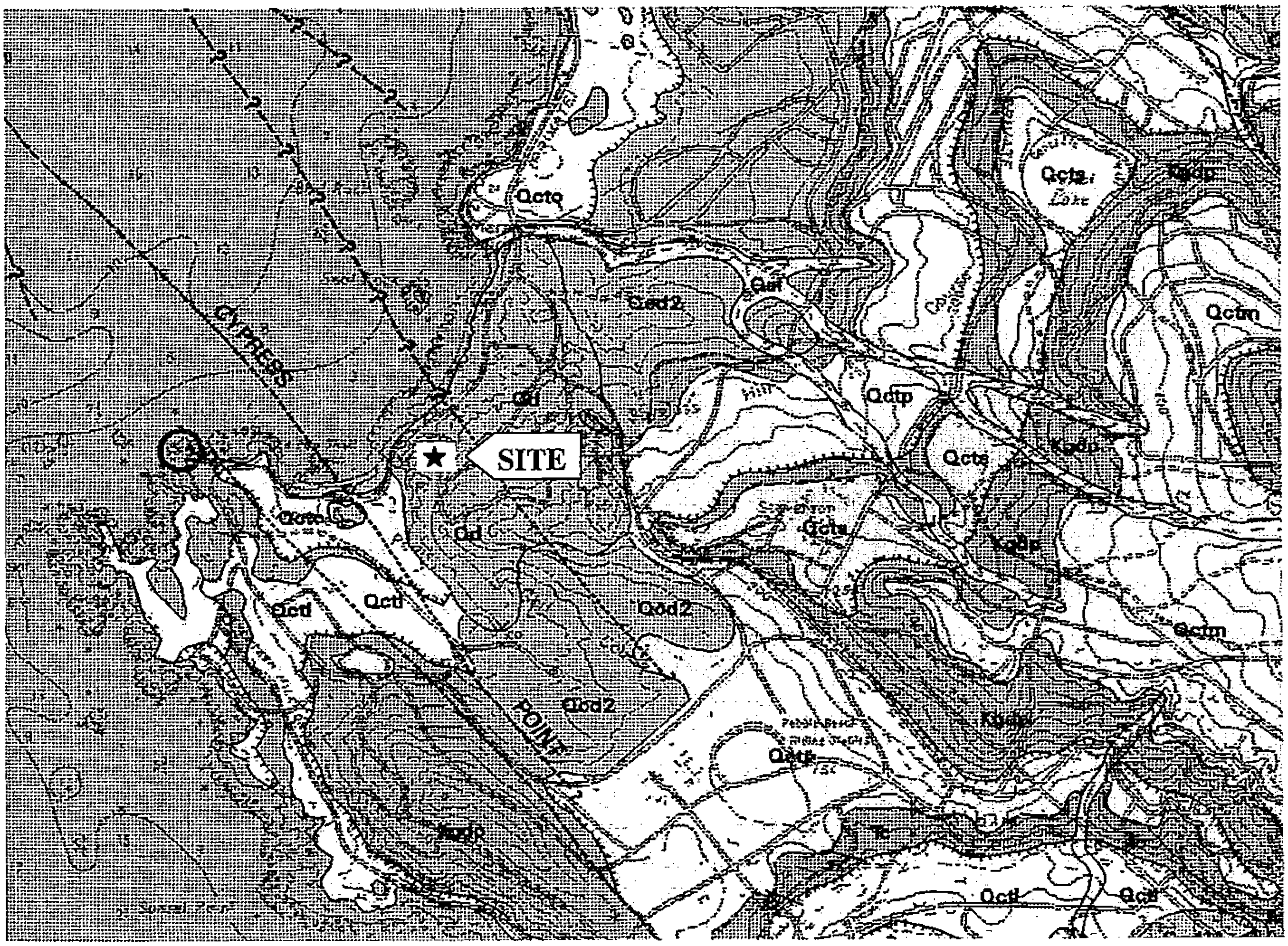
BASE: U.S. Geological Survey, Monterey 7.5' Quadrangle, Pebble Beach, California

SITE VICINITY MAP

CLEARY CONSULTANTS, INC.
Geotechnical Engineers and Geologists

NEW RESIDENCE
1170 Signal Hill Road
Pebble Beach, California

APPROVED BY	SCALE	PROJECT NO.	DATE	DRAWING NO.
JMC	1" = 2000'	1301.1	March 2010	1



EXPLANATION

Qaf	Artificial Fill
Qd	Dune Sand Deposits
Qod2	Older Dune Deposits
Qct	Coastal Terrace Deposits
Qcto	Ocean View Costal Terrace
Qctl	Lighthouse Coastal Terrace
Qctp	Peninsula College Coastal Terrace
Qcts	Sylvan Coastal Terrace
Qctm	Monte Vista Coastal Terrace
Tc	Carmelo Formation of Bowen
Kgdp	Porphyritic Granodiorite of Monterey of Ross

Fault, dashed where inferred, dotted where concealed

N

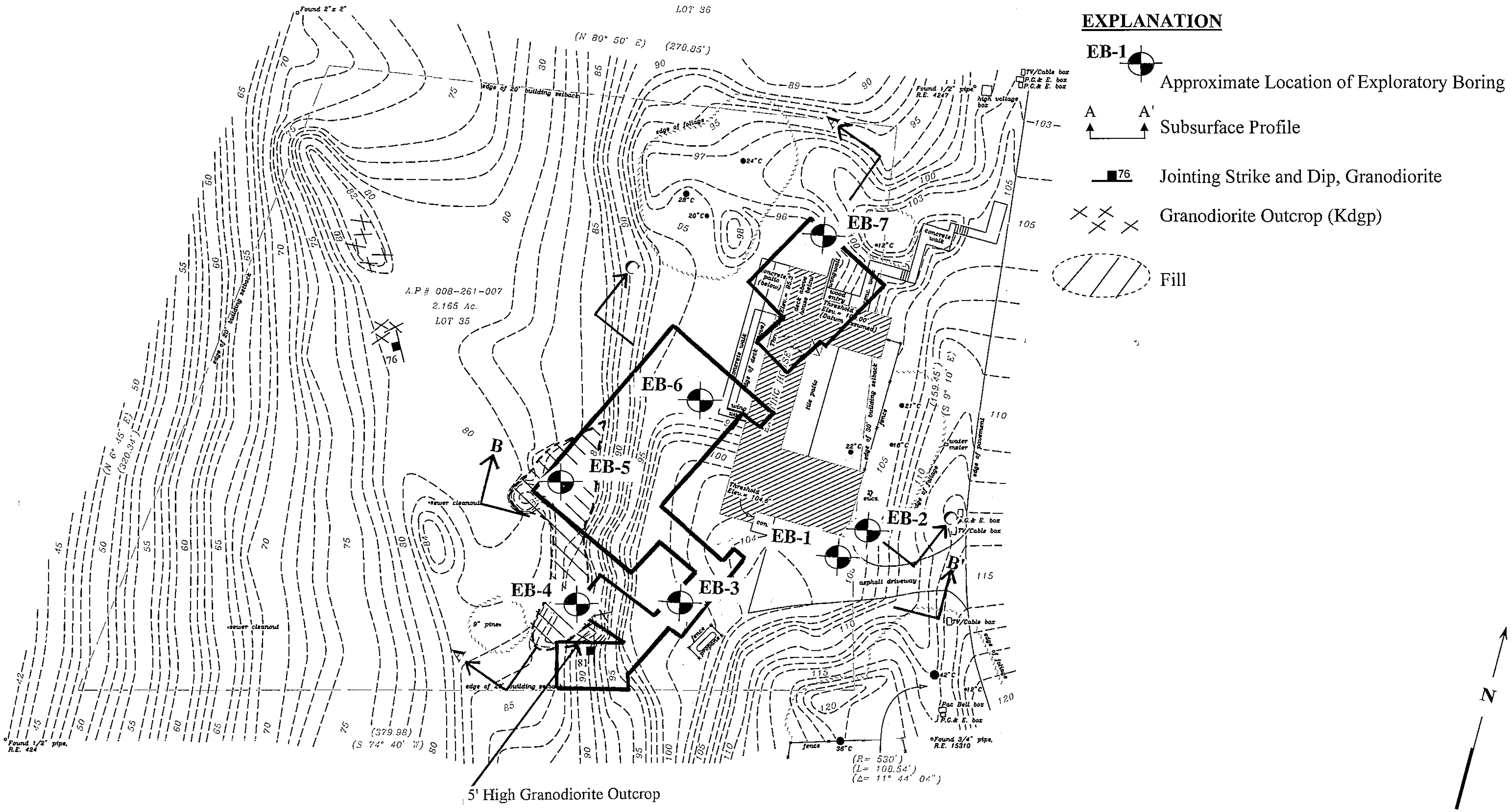
BASE: J.C. Clark, W.R. Dupre and L.I. Rosenberg, Geologic Map of the Monterey and Seaside 7.5 Minute Quadrangles, Monterey County, California, OFR 97-30

LOCAL GEOLOGIC MAP

CLEARY CONSULTANTS, INC.
Geotechnical Engineers and Geologists

NEW RESIDENCE
1170 Signal Hill Road
Pebble Beach, California

APPROVED BY	SCALE	PROJECT NO.	DATE	DRAWING NO.
JMC	1" = 2000'	1301.1	March 2010	2



BASE: Topographic Survey (September 2009) provided by Bernstein Architects

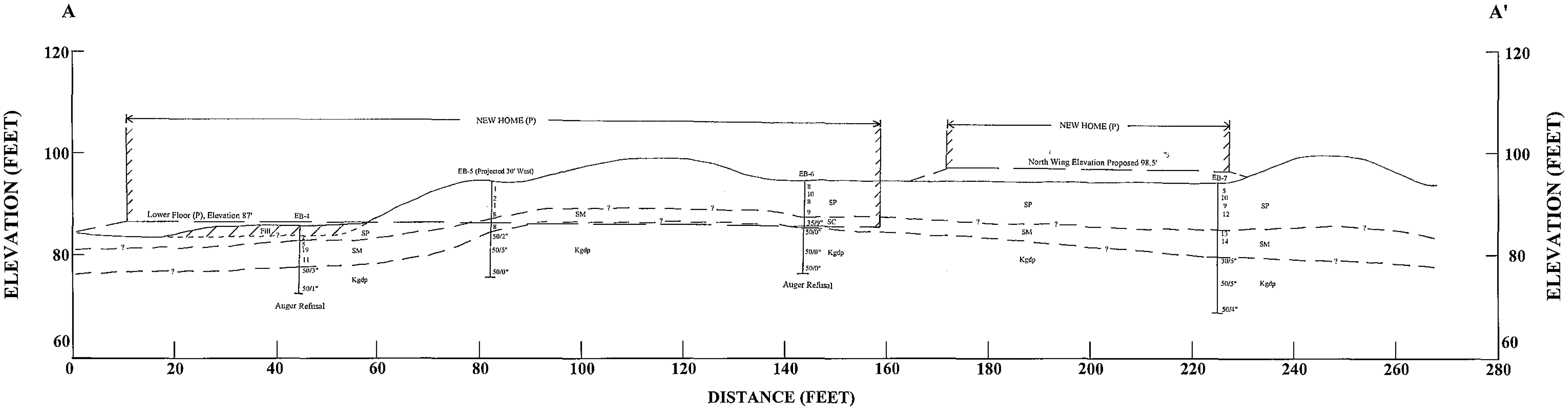
NOTE: Heavy lines show general limits of basement for new home. (Bernstein Architects, Basement Floor Plan, November 2009)

SITE PLAN

CLEARY CONSULTANTS, INC.
Geotechnical Engineers and Geologists

NEW RESIDENCE
1170 Signal Hill Road
Pebble Beach, California

APPROVED BY	SCALE	PROJECT NO.	DATE	DRAWING NO.
JMC	1" = 40' ±	1301.1	March 2010	3



LEGEND

SP	Clean Sand, Loose to Medium Dense (Dune Sand Deposit)
SM	Silty to Clayey Sand, Loose to Medium Dense
Kgdp	Granodiorite Bedrock, Very Dense
▽	Water Level as Encountered During Drilling

NOTE: 1) Assumed elevation datum per Topographic Survey of site (updated September, 2009)

2) Standard Penetration Resistance values shown on right side of exploratory borings

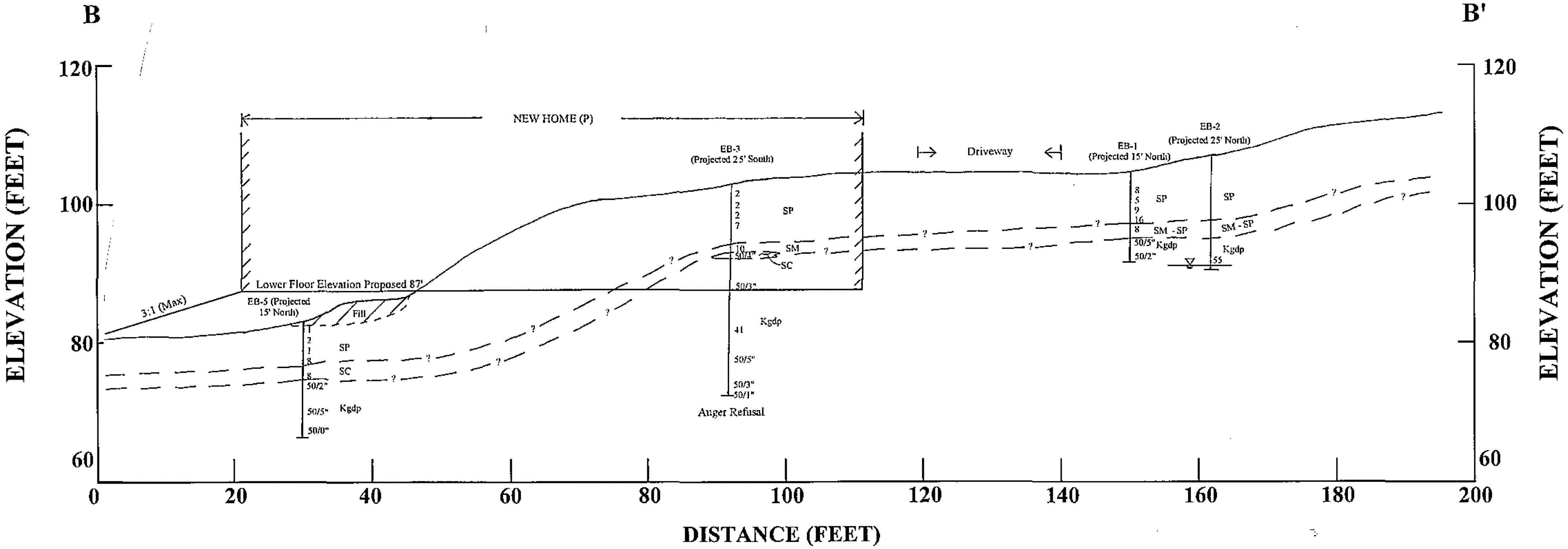
3) The indicated stratum lines are based on interpolation between widely spread borings and other data, and may not represent actual subsurface conditions.

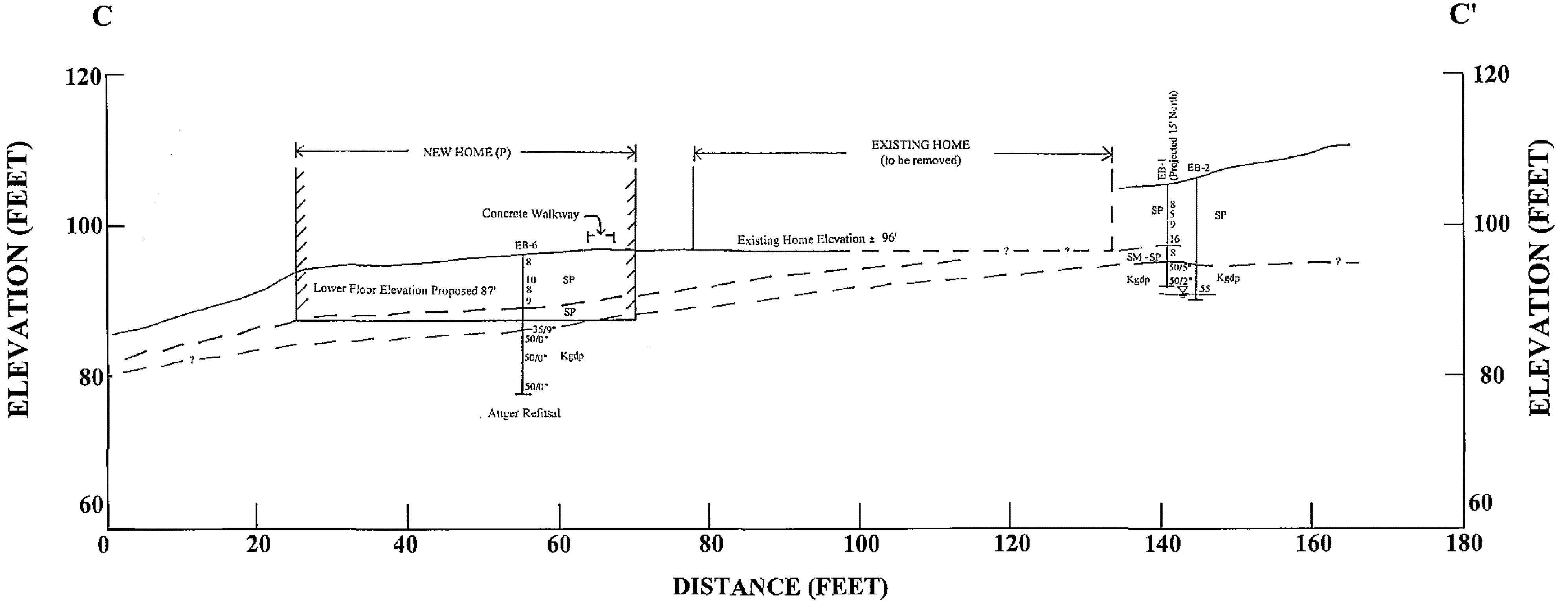
SUBSURFACE PROFILE A-A'

CLEARY CONSULTANTS, INC.
Geotechnical Engineers and Geologists

NEW RESIDENCE
1170 Signal Hill Road
Pebble Beach, California

APPROVED BY	SCALE	PROJECT NO.	DATE	DRAWING NO.
JMC	1" = 20'	1301.1	March 2010	4





LEGEND

Sp	Clean Sand, Loose to Medium Dense (Dune Sand Deposit)
SM	Silty Sand, Loose to Medium Dense
Kgdp	Granodiorite Bedrock, Very Dense
▽	Water Level as Encountered During Drilling

NOTE: 1) Assumed elevation datum per Topographic Survey of site (updated September, 2009)
2) Standard Penetration Resistance values shown on right side of exploratory borings
3) The indicated stratum lines are based on interpolation between widely spread borings and other data, and may not represent actual subsurface conditions.

SUBSURFACE PROFILE B-B' AND C-C'

CLEARY CONSULTANTS, INC.
Geotechnical Engineers and Geologists

NEW RESIDENCE
1170 Signal Hill Road
Pebble Beach, California

APPROVED BY	SCALE	PROJECT NO.	DATE	DRAWING NO.
JMC	1" = 20'	1301.1	March 2010	5

PRIMARY DIVISIONS			GROUP SYMBOL	SECONDARY DIVISION
COARSE GRAINED SOILS MORE THAN HALF OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVELS MORE THAN HALF OF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS (LESS THAN 5% FINES)	GW	Well graded gravels, 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, non-plastic fines
			GC	Clayey gravels, gravel-sand-clay mixtures, plastic fines
	SANDS MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE	CLEAN SANDS (LESS THAN 5% FINES)	SW	Well graded sands, gravelly sands, little or no fines
			SP	Poorly graded sands or gravelly sands, little or no fines
		SANDS WITH FINES	SM	Silty sands, sand-silt mixtures, non-plastic fines
			SC	Clayey sands, sand-clay mixtures, plastic fines
FINE GRAINED SOILS MORE THAN HALF OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS LIQUID LIMIT IS LESS THAN 50%		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
			OL	Organic silts and organic silty clays of low plasticity
	SILTS AND CLAYS LIQUID LIMIT IS GREATER THAN 50%		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic clays of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS			Pt	Peat and other highly organic soils

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D-2487)

U.S. STANDARD SERIES SIEVE | CLEAR SQUARE SIEVE OPENINGS

200 | 40 | 10 | 4 | 3/4" | 3" | 12"

SILTS AND CLAYS	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

GRAIN SIZES

SANDS AND GRAVELS	BLOWS/FOOT✢
VERY LOOSE	0 - 4
LOOSE	4 - 10
MEDIUM DENSE	10 - 30
DENSE	30 - 50
VERY DENSE	OVER 50

RELATIVE DENSITY

SILTS AND CLAYS	STRENGTH ☆	BLOWS/FOOT ✢
VERY SOFT	0 - 1/4	0 - 2
SOFT	1/4 - 1/2	2 - 4
FIRM	1/2 - 1	4 - 8
STIFF	1 - 2	8 - 16
VERY STIFF	2 - 4	16 - 32
HARD	OVER 4	OVER 32

CONSISTENCY

✢ Number of blows of 140 pound hammer falling 30 inches to drive a 2 inch O.D. (1-3/8 inch I.D.) split barrel (ASTM D-1586).

☆ Unconfined compressive strength in tons/sq.ft. as determined by laboratory testing or approximated by the standard penetration test (ASTM D-1586), pocket penetrometer, torvane, or visual observation.

CLEARY CONSULTANTS, INC.
Geotechnical Engineers and Geologists

KEY TO EXPLORATORY BORING LOGS

NEW RESIDENCE
1170 Signal Hill Road
Pebble Beach, California

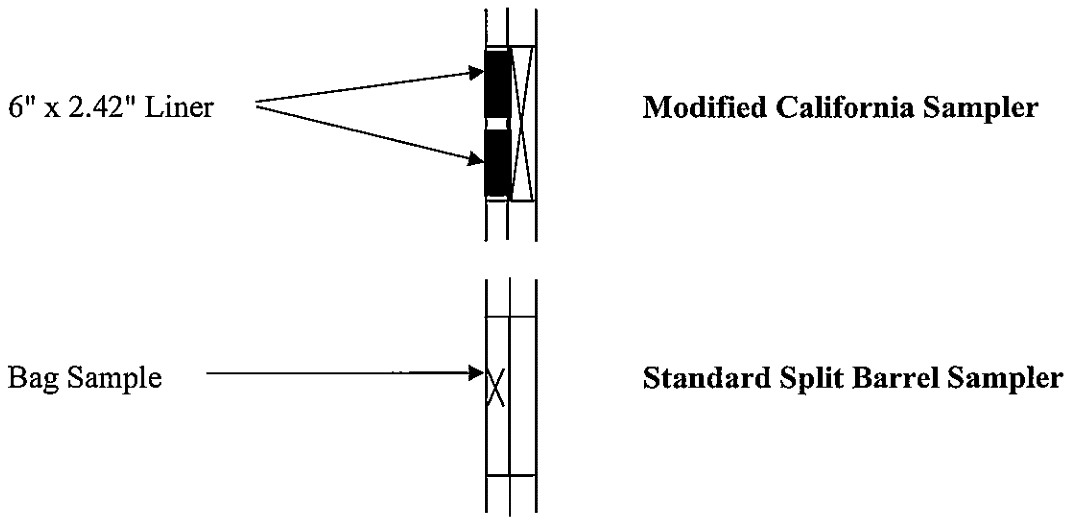
PROJECT NO.	DATE	DRAWING NO.
1301.1	March 2010	6

FIELD SAMPLING PROCEDURES

The soils encountered in the borings were continuously logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D-2487).

Representative soil samples were obtained from the borings at selected depths appropriate to the soil investigation. All samples were returned to our laboratory for classification and testing.

In accordance with the ASTM D1586 procedure, the standard penetration resistance was obtained by dropping a 140 pound hammer through a 30-inch free fall. The 2-inch O.D. Standard split barrel sampler was driven 18 inches or to practical refusal and the number of blows were recorded for each 6-inch penetration interval. The blows per foot recorded on the boring logs represent the accumulated number of blows, or N-value, required to drive the penetration sampler the final 12 inches. In addition, 3.0 inch O.D. x 2.42 inch I.D. drive samples were obtained using a Modified California Sampler and 140 pound hammer. Blow counts for the Modified California Sampler were converted to standard penetration resistance by multiplying by 0.6. The sample type is shown on the boring logs in accordance with the designation below.



Where obtained, the shear strength of the soil samples using either Torvane (TV) or Pocket Penetrometer (PP) devices is shown on the boring logs in the far right hand column.

CLEARY CONSULTANTS, INC.
Geotechnical Engineers and Geologists

SUMMARY OF FIELD SAMPLING PROCEDURES

NEW RESIDENCE
1170 Signal Hill Road
Pebble Beach, California

PROJECT NO.	DATE	DRAWING NO.
1301.1	March 2010	7

LABORATORY TESTING PROCEDURES

The laboratory testing program was directed toward a quantitative and qualitative evaluation of the physical and mechanical properties of the soils underlying the site.

The natural water content was determined on 79 samples of the materials recovered from the borings in accordance with the ASTM D2216 Test Procedure. These water contents are recorded on the boring logs at the appropriate sample depths.

Dry density determinations were performed on 20 samples to measure the unit weight of the subsurface soils in accordance with the ASTM D2937 Test Procedure. The results of these tests are shown on the boring logs at the appropriate sample depths.

Two Atterberg Limits determinations were performed on representative samples of the subsurface soils in accordance with the ASTM D4318 Test Procedure to determine the range of water contents over which the materials exhibited plasticity. The Atterberg Limits are used to classify the soils in accordance with the Unified Soil Classification System and to evaluate the soil's expansion potential. The results of these tests are presented on the boring logs.

The percent soil fraction passing the #4 and #200 sieves were determined on 13 and 22 samples of the subsurface soils in accordance with the ASTM D1140 Test Procedure to aid in the classification of the soils. The results of these tests are shown on the boring logs at the appropriate sample depths.

Free swell tests were performed on six samples of the soil materials to evaluate the swelling potential of the soil. The free swell tests were performed by slowly pouring 10 ml of air dried soil passing the No. 40 sieve into a 100 ml graduated cylinder filled with approximately 90 ml of distilled water. The suspension was stirred repeatedly to ensure thorough wetting of the soil specimen. The graduated cylinder was then filled with distilled water to the 100 ml mark and allowed to settle until equilibrium was reached (approximately 24 hours). The free swell volume of the soil was then noted. The percent free swell was calculated by subtracting the initial soil volume from the free swell volume, dividing the difference by the initial volume, and multiplying the result by 100 percent. The results of these tests are presented on the boring logs.

Two unconfined compression tests were performed in accordance with the ASTM D2166 Test Procedure on undisturbed samples of the subsurface soils to evaluate the undrained shear strength of the materials. The unconfined tests were performed on samples having a diameter of 2.43 inches and a height-to-diameter ratio of at least two. Failure was taken at the peak normal stress or at five percent strain, whichever occurred first. The results of these tests are presented on the boring logs at the appropriate sample depths.

DRAWING NO. 8

LABORATORY TESTING PROCEDURES CONTINUED

Corrosion testing was performed by Cooper Laboratory on a sample of the soil materials from EB-6 at a depth of one to five feet. Testing included resistivity, pH, chloride and sulfate testing performed in accordance with ASTM G57, ASTM G51, Caltrans 422(modified) and Caltrans 417(modified), respectively. The results of these tests are presented on Drawing 20 and are discussed in Section G. Soil Corrosivity.

Grain size distribution tests were performed on two samples of the sand materials in accordance with the ASTM D 422 Test Procedure to aid in the classification. The results of these tests are presented on Drawing 21.

EQUIPMENT	8" Diameter Hollow Stem Auger*	ELEVATION	105'±	LOGGED BY	TD
DEPTH TO GROUNDWATER	Not Det.	DEPTH TO BEDROCK	9.5'±	DATE DRILLED	2/19/2010

DESCRIPTION AND CLASSIFICATION									
DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (feet)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	SHEAR STRENGTH (KSF)
Driveway: 2.5" AC Over 6" AB SAND, dry, fine angular to subrounded sand, cohesionless	Whitish Tan	Loose	SP	1					
@2.5': Finer than #4 = 100% Finer than #200 = 1%				2 3		8	0 0	97	
				4		5	1		
@5.0': dark gray, fine to coarse sand laminations, moist, upper five feet caved as augers were removed from hole Finer than #4 = 100% Finer than #200 = 2%				5		9	2 1	90 80	
		Medium Dense		6					
@7.0': slightly moist, limited cohesion				7		16	1		
SILTY SAND, wet, fine to medium grained sand, roots up to 3/4" diameter	Brown	Loose	SM-SP	8			12 21	102	PP=1.0
@8.5': Finer than #4 = 100% Finer than #200 = 3%				9		8	17	116	
@9.5': wet Finer than #4 = 97% Finer than #200 = 28% Free Swell = 20%		(Very Dense)	(SM)	10		50/5"	13		
DECOMPOSED GRANODIORITE, slightly moist, highly weathered	Tan to Whitish Gray			11					
@11.0': driller reported hard drilling				12					
@13.0': fresh, no weathering, drilling refusal				13		50/2"	1		
Bottom of Boring = 13.0'				14 15 16 17 18					
* Drilled with a CME-55 Track Mounted Rig PP = Pocket Penetrometer				19 20					

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL

CLEARY CONSULTANTS, INC.
Geotechnical Engineers and Geologists

LOG OF EXPLORATORY BORING NO. 1

NEW RESIDENCE
1170 Signal Hill Road
Pebble Beach, California

APPROVED BY	SCALE	PROJECT NO.	DATE	DRAWING NO.
JMC	----	1301.1	March 2010	10

EQUIPMENT	8" Diameter Hollow Stem Auger*	ELEVATION	106'±	LOGGED BY	TD
DEPTH TO GROUNDWATER	16.0'±	DEPTH TO BEDROCK	12.0'±	DATE DRILLED	2/19/2010

DESCRIPTION AND CLASSIFICATION				DEPTH (feet)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	SHEAR STRENGTH (KSF)
DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE						
Landscape Area SAND, dry, fine to medium grained, angular to subrounded	Whitish Tan	Very Loose	SP						
				1					
				2					
				3					
				4					
				5					
				6					
				7					
				8					
				9					
				10					
				11					
				12					
DECOMPOSED GRANODIORITE, moist to wet	Orange-Gray		SM-SC						
				13					
				14					
	Gray-White	(Very Dense)							
				15					
@16.0': trace clay Finer than #200 = 16%				16	X	55	11		▼
Bottom of Boring = 16.5'									
				17					
				18					
* Drilled with a CME-55 Track Mounted Rig ▼ Water level as measured 0.25 hours after drilling				19					
				20					

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL

CLEARY CONSULTANTS, INC.
Geotechnical Engineers and Geologists

LOG OF EXPLORATORY BORING NO. 2

NEW RESIDENCE
1170 Signal Hill Road
Pebble Beach, California

APPROVED BY	SCALE	PROJECT NO.	DATE	DRAWING NO.
JMC	----	1301.1	March 2010	11

EQUIPMENT 8" Diameter Hollow Stem Auger*	ELEVATION 101'±	LOGGED BY TD	
DEPTH TO GROUNDWATER Not Enc.	DEPTH TO BEDROCK 10.5'±	DATE DRILLED 2/19/2010	

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (feet)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	SHEAR STRENGTH (KSF)
Landscape Area SAND, slightly moist to moist, fine to medium angular to subrounded sand, occasional 1/4" diameter rootlets, cohesionless	Whitish Tan	Very Loose	SP	0–2		2	2 1	95	
				3		2	1		
@4.0': slight cohesion				4–5		2	1 1	91	
		Loose		5–6		7	2		
SAND, moist, fine to medium grained @9.5': Finer than #4 = 100% Finer than #200 = 1% Free Swell = 0%	Dark Gray	Loose	SP	9–10		10	6 5 4	99 100	PP=0.25
CLAYEY SAND, very moist, fine to medium grained sand, completely weathered granodiorite	Dark Brown	Very Dense	SC (SM)	10.5		50/4"	16		
GRANODIORITE, slightly moist, highly weathered and decomposed	Tan to Whitish Gray	(Very Dense)		12					
@15.0': little or no weathering, fresh rock				15		50/3"	3		
				20					

* Drilled with a CME-55 Track Mounted Rig
PP = Pocket Penetrometer

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL

CLEARY CONSULTANTS, INC.
Geotechnical Engineers and Geologists

LOG OF EXPLORATORY BORING NO. 3

NEW RESIDENCE
1170 Signal Hill Road
Pebble Beach, California

APPROVED BY	SCALE	PROJECT NO.	DATE	DRAWING NO.
JMC	----	1301.1	March 2010	12

EQUIPMENT 8" Diameter Hollow Stem Auger*	ELEVATION 101'±	LOGGED BY TD
DEPTH TO GROUNDWATER Not Enc.	DEPTH TO BEDROCK 10.5'±	DATE DRILLED 2/19/2010

DESCRIPTION AND CLASSIFICATION

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (feet)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	SHEAR STRENGTH (KSF)
GRANODIORITE, slightly moist, continued	Tan to Whitish Gray	(Very Dense)	(SM)						
@20.5': highly weathered, iron staining, moist to wet Finer than #200 = 19%				21		41	17 11		
	Yellow Red			22					
				23					
		(Very Dense)		24					
@25.0': decomposed, friable granodiorite				25		50/5"	14		
				26					
				27					
				28					
				29					
@30.0': fresh granodiorite zones				30		50/3"			
@31.0': hard, slightly weathered granodiorite, drilling refusal	Gray-White			31		50/1"	18		
Bottom of Boring = 31.0'									
				32					
				33					
				34					
				35					
				36					
				37					
				38					
				39					
* Drilled with a CME-55 Track Mounted Rig				40					

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL

CLEARY CONSULTANTS, INC.
Geotechnical Engineers and Geologists

LOG OF EXPLORATORY BORING NO. 3

NEW RESIDENCE
1170 Signal Hill Road
Pebble Beach, California

APPROVED BY	SCALE	PROJECT NO.	DATE	DRAWING NO.
JMC	----	1301.1	March 2010	13

EQUIPMENT 8" Diameter Hollow Stem Auger*	ELEVATION 87'±	LOGGED BY TD
DEPTH TO GROUNDWATER Not Enc.	DEPTH TO BEDROCK 8.0'±	DATE DRILLED 2/19/2010

DESCRIPTION AND CLASSIFICATION

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (feet)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	SHEAR STRENGTH (KSF)
Landscape Area SAND, slightly moist, fine to medium angular to subrounded sand	Whitish Tan	Very Loose	SP	1		2	3 4		
@2.5': Finer than #200 = 1%		Loose		2 3		5	5 17		
SILTY SAND, very moist, fine to medium grained sand, occasional weathered granodiorite gravels	Dark Brown	Loose Medium Dense	SM	4					
@4.5': Finer than #4 = 94% Finer than #200 = 33% Free Swell = 0%				5		19	5 9	117 126	PP>4.5 **2.2ksf@ 2.2%strain
@6.0': possibly completely decomposed granodiorite Finer than #4 = 97% Finer than #200 = 34% Free Swell = 0%				6 7		11	12		
GRANODIORITE, slightly moist, partially weathered @8.0': driller reported hard drilling	Whitish Gray	(Very Dense)	(SM)	8 9 10 11 12		50/3"			
@13.5': drilling refusal				13		50/1"	1		
Bottom of Boring = 13.5'				14 15 16 17 18 19 20					

* Drilled with a CME-55 Track Mounted Rig
** Unconfined Compressive Strength
PP = Pocket Penetrometer

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL

CLEARY CONSULTANTS, INC.
Geotechnical Engineers and Geologists

LOG OF EXPLORATORY BORING NO. 4

NEW RESIDENCE
1170 Signal Hill Road
Pebble Beach, California

APPROVED BY	SCALE	PROJECT NO.	DATE	DRAWING NO.
JMC	----	1301.1	March 2010	14

EQUIPMENT 8" Diameter Hollow Stem Auger*	ELEVATION 84'±	LOGGED BY TD
DEPTH TO GROUNDWATER Not Det.	DEPTH TO BEDROCK 10.0'±	DATE DRILLED 2/19/2010

DESCRIPTION AND CLASSIFICATION

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (feet)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	SHEAR STRENGTH (KSF)
Landscape Fill	Whitish Tan	Very Loose	SP						
SAND, slightly moist, fine to medium angular to subrounded sand				1					
@1.5': Finer than #200 = 0%						1	6 4		
				2					
@3.0': no recovery				3		2			
				4		1	1 5		
				5					
		Loose							
				6		8	3		
				7					
CLAYEY SAND, wet, fine to coarse angular to subrounded sand	Dark Gray	Loose	SC	8					
@9.5': free water Liquid Limit = 31% Plasticity Index = 17% Finer than #4 = 100% Finer than #200 = 26% Free Swell = 50%				9		8	8 10	123 118	PP=3.25 **1.2ksf@ 3.7%strain
GRANODIORITE, slightly moist to moist, weathered and decomposed, iron staining	Yellowish to Whitish Gray	(Very Dense)	(SM)	10		50/2"	6		
				11					
				12					
				13		50/5"	3		
				14					
				15					
				16					
				17					
				18		50/0"			
Bottom of Boring = 18.5' * E-55 Track Mounted Rig ** Unconfined Compressive Strength PP = Pocket Penetrometer				19					
				20					

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL

CLEARY CONSULTANTS, INC.
Geotechnical Engineers and Geologists

LOG OF EXPLORATORY BORING NO. 5

NEW RESIDENCE
1170 Signal Hill Road
Pebble Beach, California

APPROVED BY	SCALE	PROJECT NO.	DATE	DRAWING NO.
JMC	----	1301.1	March 2010	15

EQUIPMENT	8" Diameter Hollow Stem Auger*	ELEVATION	96'±	LOGGED BY	TD
DEPTH TO GROUNDWATER	Not Det.	DEPTH TO BEDROCK	10.0'±	DATE DRILLED	2/19/2010

DESCRIPTION AND CLASSIFICATION									
DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (feet)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	SHEAR STRENGTH (KSF)
Landscape Area SAND, slightly moist, fine to medium angular to subrounded sand, roots up to 2.5" diameter	Whitish Tan	Loose	SP	1			6	88	
@1.5': Finer than #200 = 1%						8	3	92	
		Medium Dense		2					
				3		10	3		
		Loose		4			6		
@4.5': Finer than #200 = 10%						8	5	96	
				5					
				6		9	4		
				7					
SAND, very moist, fine to medium angular to subrounded	Dark Brownish Gray	Medium Dense	SP	8					
@9.0': Liquid Limit = Non-Plastic Plasticity Index = Non-Plastic Finer than #4 = 100% Finer than #200 = 3% Free Swell = 0%		Very Dense		9			6		
						35/9"	17	84	
GRANODIORITE, slightly moist, fresh to slightly weathered	Tan to Whitish Gray	(Very Dense)	(SM)	10		50/0"	7		
				11					
				12					
				13					
						50/0"	9 (Shoe)		
				14					
				15					
				16					
				17					
				18					
@18.5': drilling refusal						50/0"	5 (Shoe)		
Bottom of Boring = 18.5'				19					
* Drilled with a CME-55 Track Mounted Rig				20					

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL

CLEARY CONSULTANTS, INC.
Geotechnical Engineers and Geologists

LOG OF EXPLORATORY BORING NO. 6

NEW RESIDENCE
1170 Signal Hill Road
Pebble Beach, California

APPROVED BY	SCALE	PROJECT NO.	DATE	DRAWING NO.
JMC	----	1301.1	March 2010	16

EQUIPMENT	8" Diameter Hollow Stem Auger*	ELEVATION 95'±	LOGGED BY TD
DEPTH TO GROUNDWATER	10.5'±	DEPTH TO BEDROCK 14.0'±	DATE DRILLED 2/19/2010

DESCRIPTION AND CLASSIFICATION

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (feet)	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	SHEAR STRENGTH (KSF)
Landscape Area SAND, slightly moist to moist, fine to medium angular to subrounded sand, rootlets up to 0.25" diameter @1.5': Finer than #200 = 0%	Whitish Tan	Loose	SP	1	5	3 3	87	
		Medium Dense		2 3	10	5		
@4.5': Finer than #200 = 0%		Loose		4	9	5 4	101	
		Medium Dense		5 6	12	4		
				7 8 9				
SILTY SAND, wet to saturated, fine to medium grained sand @9.5': Finer than #200 = 2% @10.5': free water @11.0': Finer than #4 = 100% Finer than #200 = 26%	Dark Gray	Medium Dense	SM	10 11 12 13	13 14	19 ▽ 12	92	
GRANODIORITE, slightly moist, highly weathered, iron stained @19.0': fresh, little to no weathering Finer than #4 = 95% Finer than #200 = 14%	Gray	Very Dense	(SM)	14 15 16 17 18 19 20	30/5" 50/5"	14 11 6		

* Drilled with a CME-55 Track Mounted Rig

▽ Water level as encountered during drilling

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL

CLEARY CONSULTANTS, INC.
Geotechnical Engineers and Geologists

LOG OF EXPLORATORY BORING NO. 7

NEW RESIDENCE
1170 Signal Hill Road
Pebble Beach, California

APPROVED BY	SCALE	PROJECT NO.	DATE	DRAWING NO.
JMC	----	1301.1	March 2010	17

EQUIPMENT	8" Diameter Hollow Stem Auger*	ELEVATION	95'±	LOGGED BY	TD
DEPTH TO GROUNDWATER	10.5'±	DEPTH TO BEDROCK	14.0'±	DATE DRILLED	2/19/2010

DESCRIPTION AND CLASSIFICATION									
DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (feet)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	SHEAR STRENGTH (KSF)
GRANODIORITE, slightly moist, continued	Gray	Very Dense	(SM)						
				21					
				22					
				23					
				24					
@25.0': weathered decomposed granodiorite, clayey zones				25	X	50/4"	11		
Bottom of Boring = 25.5'				26					
				27					
				28					
				29					
				30					
				31					
				32					
				33					
				34					
				35					
				36					
				37					
				38					
				39					
* Drilled with a CME-55 Track Mounted Rig				40					

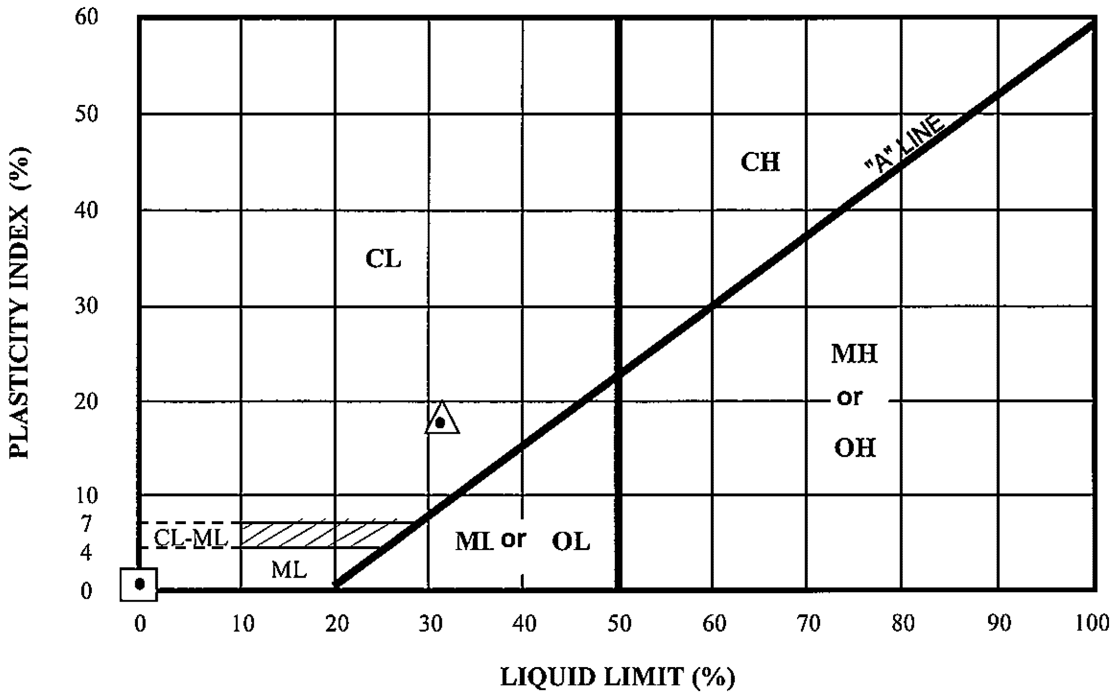
THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL

CLEARY CONSULTANTS, INC.
Geotechnical Engineers and Geologists

LOG OF EXPLORATORY BORING NO. 7

NEW RESIDENCE
1170 Signal Hill Road
Pebble Beach, California

APPROVED BY	SCALE	PROJECT NO.	DATE	DRAWING NO.
JMC	----	1301.1	March 2010	18



KEY SYMBOL	BORING NO.	SAMPLE DEPTH (feet)	NATURAL WATER CONTENT %	LIQUID LIMIT %	PLASTICITY INDEX %	PASSING NO. 200 SIEVE %	LIQUIDITY INDEX	UNIFIED SOIL CLASSIFICATION SYMBOL
◬	5	9.5	10	31	17	26	-0.2	SC*
⊡	6	9.0	6	---	---	3	---	SP*

*Classified as coarse-grained soil since less than 50% passes #200 sieve

CLEARY CONSULTANTS, INC.
Geotechnical Engineers and Geologists

PLASTICITY CHART

NEW RESIDENCE
1170 Signal Hill Road
Pebble Beach, California

PROJECT NO.	DATE	DRAWING NO.
1301.1	March 2010	19

COOPER
TESTING LABORATORY

Corrosivity Test Summary

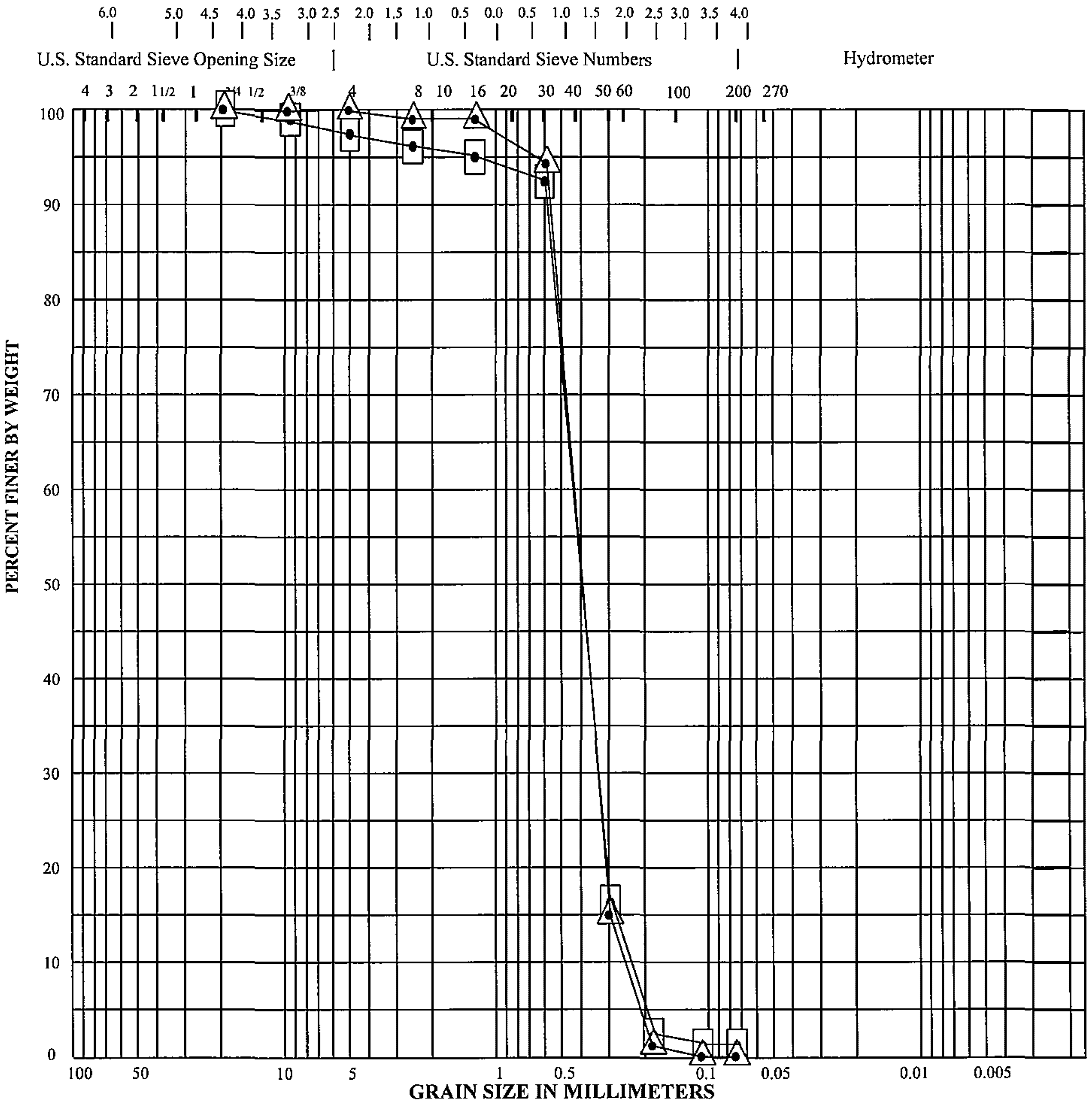
CTL # 018-524 **Date:** 2/26/2010 **Tested By:** PJ **Checked:** PJ

Client: Cleary Consultants **Project:** 1170 Signal Hill Rd. Pebble Beach, CA **Proj. No:** 1301.1

Remarks:

Sample Location or ID			Resistivity @ 15.5 °C (Ohm-cm)			Chloride	Sulfate-(water soluble)		pH	ORP	Sulfide	Moisture	Soil Visual Description
Boring	Sample, No.	Depth, ft.	As Rec.	Minimum	Saturated	mg/kg	mg/kg	%		(Redox)	Qualitative	%	
						Dry Wt.	Dry Wt.	Dry Wt.		mv	by Lead	At Test	
			ASTM G57	Cal 643	ASTM G57	Cal 422-mod.	Cal 417-mod.	Cal 417-mod.	ASTM G51	SM 2580B	Acetate Paper	ASTM D2216	
6	-	1-5	-	-	16,497	4	<5	<0.0005	6.7	166	-	3.9	Light Brown SAND

DRAWING NO. 20



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

SYMBOL	SAMPLE SOURCE	CLASSIFICATION
—△—	Boring 5 at 6'	Light Brown Sand (SP)
—□—	Boring 6 at 1' to 5'	Light Brown Sand (SP)

CLEARY CONSULTANTS, INC.

Geotechnical Engineers and Geologists

GRADATION TEST DATA

NEW RESIDENCE
1170 Signal Hill Road
Pebble Beach, California

PROJECT NO.	DATE	DRAWING NO.
1301.1	March 2010	21

APPENDIX A

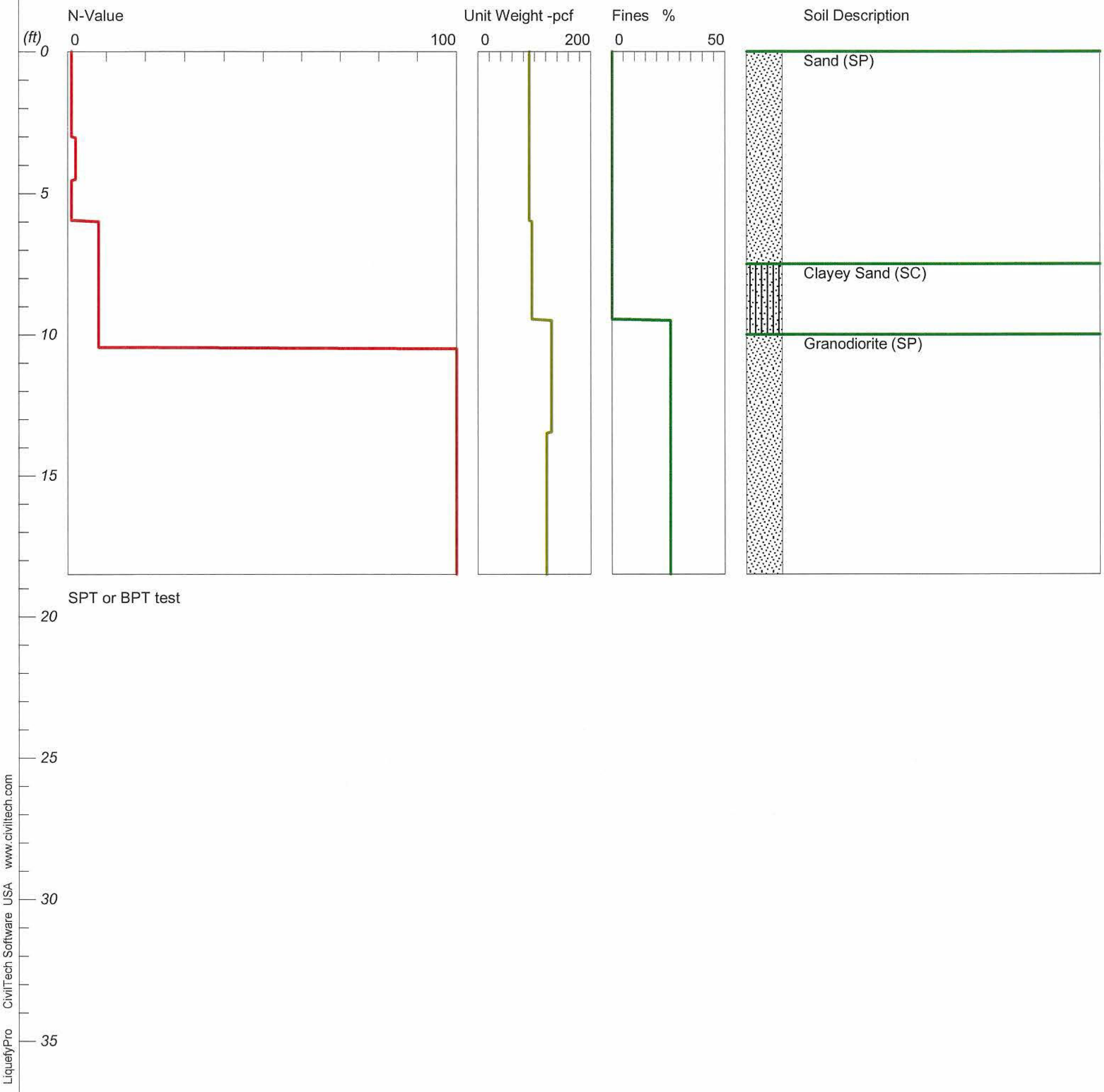
New Residence, 1170 Signal Hill Road, Liquefaction
and Dry Settlement Calculations, EB-5 and EB-7,
Drilled February 19, 2010

Liquefaction and Dry Settlement Analysis

New Residence 1170 Signal Hill Road Pebble Beach,

Hole No.=EB-5

Magnitude=8.5
Acceleration=0.442g

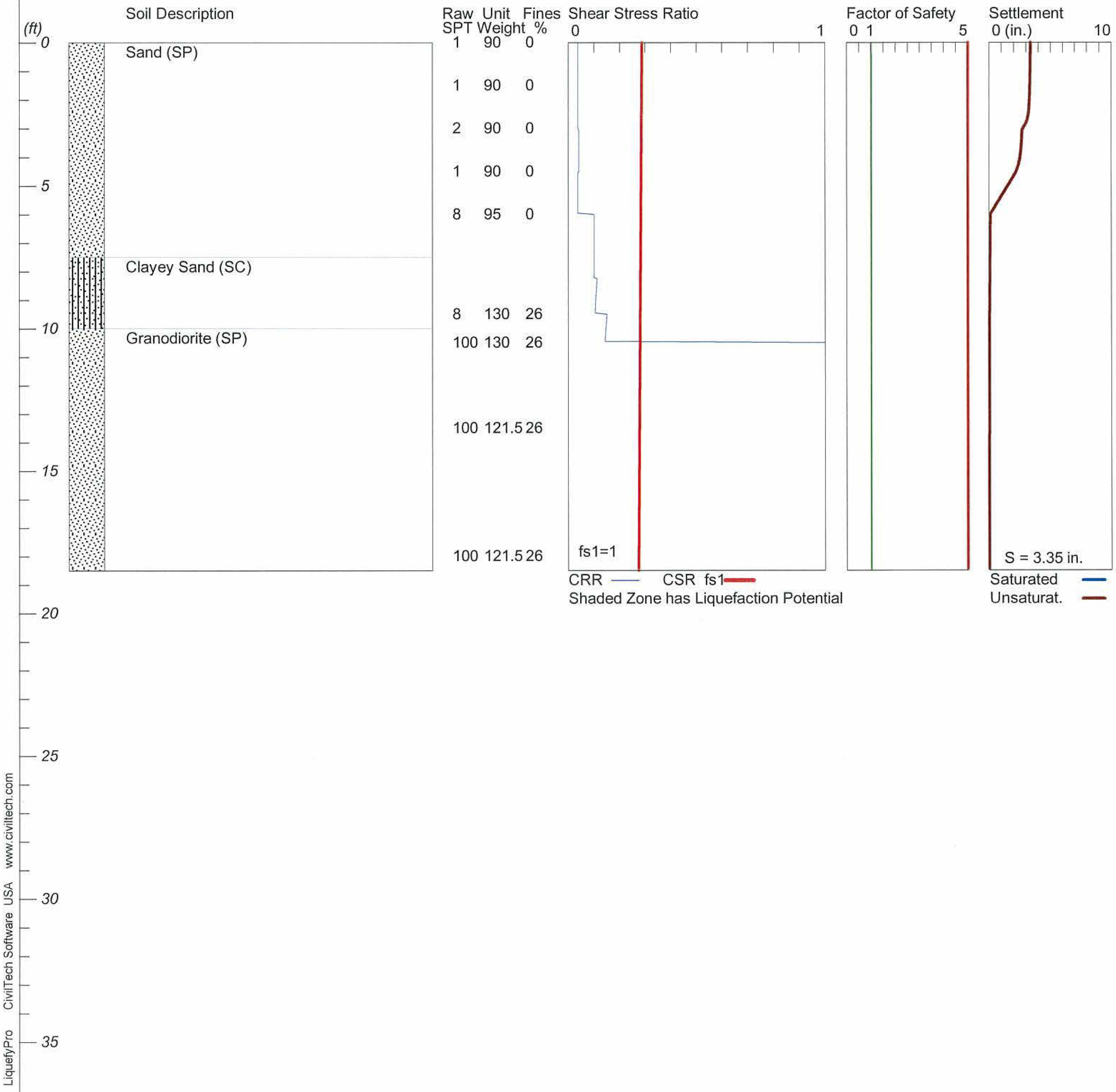


Liquefaction and Dry Settlement Analysis

New Residence 1170 Signal Hill Road Pebble Beach,

Hole No.=EB-5

Magnitude=8.5
Acceleration=0.442g



CLEARY CONSULTANTS, INC.

```
************************************************************************************
*******************
                                    LIQUEFACTION ANALYSIS CALCULATION DETAILS

                                          Copyright by CivilTech Software
                                               www.civiltechsoftware.com


************************************************************************************
*******************
        Font: Courier New, Regular, Size 8 is recommended for this report.
        Licensed to ,   3/19/2010       4:22:23 PM

        Input File Name: C:\Liquefy5\1170 Siganl Hill Road EB5.liq
        Title:  New Residence 1170 Signal Hill Road Pebble Beach,
        Subtitle:

 Input Data:
        Surface Elev.=
        Hole No.=EB-5
        Depth of Hole=18.50 ft
        Water Table during Earthquake= 999.00 ft
        Water Table during In-Situ Testing= 999.00 ft
        Max. Acceleration=0.44 g
        Earthquake Magnitude=8.50
        No-Liquefiable Soils:   Based on Analysis
        1. SPT or BPT Calculation.
        2. Settlement Analysis Method: Tokimatsu, M-correction
        3. Fines Correction for Liquefaction: Idriss/Seed
        4. Fine Correction for Settlement: During Liquefaction*
        5. Settlement Calculation in: All zones*
        6. Hammer Energy Ratio,                                   Ce = 1.25
        7. Borehole Diameter,                                         Cb= 1
        8. Sampling Method,                                          Cs= 1
        9. User request factor of safety (apply to CSR) ,   User= 1
           Plot one CSR curve (fs1=1)
        10. Average two input data between two Depths: No
        * Recommended Options

        In-Situ Test Data:
        Depth   SPT     Gamma   Fines
        ft              pcf     %
        __________________________________
        0.00    1.00    90.00   0.00
        1.50    1.00    90.00   0.00
        3.00    2.00    90.00   0.00
        4.50    1.00    90.00   0.00
        6.00    8.00    95.00   0.00
        9.50    8.00    130.00  26.00
        10.50   100.00  130.00  26.00
        13.50   100.00  121.50  26.00
        18.00   100.00  121.50  26.00
        __________________________________


 Output Results:
        Calculation segment, dz=0.050 ft
        User defined Print Interval, dp=1.00 ft

        Peak Ground Acceleration (PGA), a_max = 0.44g

        CSR Calculation:
```

```
          Depth    gamma    sigma    gamma'   sigma'   rd       mZ       a(z)     CSR      x
fs1       =CSRfs
          ft       pcf      atm      pcf      atm               g        g

_____________________________________________________________________________________________
-
          0.00     90.00    0.000    90.00    0.000    1.00     0.000    0.442    0.29     1.00
0.29
          1.00     90.00    0.043    90.00    0.043    1.00     0.000    0.442    0.29     1.00
0.29
          2.00     90.00    0.085    90.00    0.085    1.00     0.000    0.442    0.29     1.00
0.29
          3.00     90.00    0.128    90.00    0.128    0.99     0.000    0.442    0.29     1.00
0.29
          4.00     90.00    0.170    90.00    0.170    0.99     0.000    0.442    0.28     1.00
0.28
          5.00     90.00    0.213    90.00    0.213    0.99     0.000    0.442    0.28     1.00
0.28
          6.00     95.00    0.255    95.00    0.255    0.99     0.000    0.442    0.28     1.00
0.28
          7.00     95.00    0.300    95.00    0.300    0.98     0.000    0.442    0.28     1.00
0.28
          8.00     95.00    0.345    95.00    0.345    0.98     0.000    0.442    0.28     1.00
0.28
          9.00     95.00    0.390    95.00    0.390    0.98     0.000    0.442    0.28     1.00
0.28
          10.00    130.00   0.443    130.00   0.443    0.98     0.000    0.442    0.28     1.00
0.28
          11.00    130.00   0.504    130.00   0.504    0.97     0.000    0.442    0.28     1.00
0.28
          12.00    130.00   0.566    130.00   0.566    0.97     0.000    0.442    0.28     1.00
0.28
          13.00    130.00   0.627    130.00   0.627    0.97     0.000    0.442    0.28     1.00
0.28
          14.00    121.50   0.687    121.50   0.687    0.97     0.000    0.442    0.28     1.00
0.28
          15.00    121.50   0.744    121.50   0.744    0.97     0.000    0.442    0.28     1.00
0.28
          16.00    121.50   0.802    121.50   0.802    0.96     0.000    0.442    0.28     1.00
0.28
          17.00    121.50   0.859    121.50   0.859    0.96     0.000    0.442    0.28     1.00
0.28
          18.00    121.50   0.916    121.50   0.916    0.96     0.000    0.442    0.28     1.00
0.28

_____________________________________________________________________________________________
-
          CSR is based on water table at 999.00 during earthquake

          CRR Calculation from SPT or BPT data:
          Depth    SPT      Cebs     Cr       sigma'   Cn       (N1)60   Fines    d(N1)60
(N1)60f   CRR7.5
          ft                                  atm                        %

_____________________________________________________________________________________________
-
          0.00     1.00     1.25     0.75     0.000    1.70     1.59     0.00     0.00     1.59
0.05
          1.00     1.00     1.25     0.75     0.043    1.70     1.59     0.00     0.00     1.59
0.05
          2.00     1.00     1.25     0.75     0.085    1.70     1.59     0.00     0.00     1.59
0.05
          3.00     1.00     1.25     0.75     0.128    1.70     1.59     0.00     0.00     1.59
```

```
0.05
          4.00      2.00      1.25      0.75      0.170     1.70      3.19      0.00      0.00      3.19
0.06
          5.00      1.00      1.25      0.75      0.213     1.70      1.59      0.00      0.00      1.59
0.05
          6.00      8.00      1.25      0.75      0.255     1.70      12.75     0.00      0.00
12.75     0.14
          7.00      8.00      1.25      0.75      0.300     1.70      12.75     0.00      0.00
12.75     0.14
          8.00      8.00      1.25      0.75      0.345     1.70      12.75     0.00      0.00
12.75     0.14
          9.00      8.00      1.25      0.85      0.390     1.60      13.61     0.00      0.00
13.61     0.15
          10.00     8.00      1.25      0.85      0.443     1.50      12.77     26.00     5.95
18.72     0.20
          11.00     100.00    1.25      0.85      0.504     1.41      149.60    26.00     22.72
172.32    2.00
          12.00     100.00    1.25      0.85      0.566     1.33      141.24    26.00     21.70
162.94    2.00
          13.00     100.00    1.25      0.85      0.627     1.26      134.15    26.00     20.83
154.98    2.00
          14.00     100.00    1.25      0.85      0.687     1.21      128.21    26.00     20.10
148.32    2.00
          15.00     100.00    1.25      0.95      0.744     1.16      137.66    26.00     21.26
158.92    2.00
          16.00     100.00    1.25      0.95      0.802     1.12      132.64    26.00     20.65
153.28    2.00
          17.00     100.00    1.25      0.95      0.859     1.08      128.13    26.00     20.09
148.22    2.00
          18.00     100.00    1.25      0.95      0.916     1.04      124.05    26.00     19.59
143.64    2.00
_____________________________________________________________________________________________________
_
          CRR is based on water table at 999.00 during In-Situ Testing

          Factor of Safety,  - Earthquake Magnitude= 8.50:
          Depth     sigC'     CRR7.5    x Ksig    =CRRv     x MSF     =CRRm     CSRfs
F.S.=CRRm/CSRfs
          ft        atm
          _____________________________________________________________________________________
          0.00      0.00      0.05      1.00      0.05      0.73      0.04      0.29      5.00
          1.00      0.03      0.05      1.00      0.05      0.73      0.04      0.29      5.00
          2.00      0.06      0.05      1.00      0.05      0.73      0.04      0.29      5.00
          3.00      0.08      0.05      1.00      0.05      0.73      0.04      0.29      5.00
          4.00      0.11      0.06      1.00      0.06      0.73      0.04      0.28      5.00
          5.00      0.14      0.05      1.00      0.05      0.73      0.04      0.28      5.00
          6.00      0.17      0.14      1.00      0.14      0.73      0.10      0.28      5.00
          7.00      0.20      0.14      1.00      0.14      0.73      0.10      0.28      5.00
          8.00      0.22      0.14      1.00      0.14      0.73      0.10      0.28      5.00
          9.00      0.25      0.15      1.00      0.15      0.73      0.11      0.28      5.00
          10.00     0.29      0.20      1.00      0.20      0.73      0.15      0.28      5.00
          11.00     0.33      2.00      1.00      2.00      0.73      1.45      0.28      5.00
          12.00     0.37      2.00      1.00      2.00      0.73      1.45      0.28      5.00
          13.00     0.41      2.00      1.00      2.00      0.73      1.45      0.28      5.00
          14.00     0.45      2.00      1.00      2.00      0.73      1.45      0.28      5.00
          15.00     0.48      2.00      1.00      2.00      0.73      1.45      0.28      5.00
          16.00     0.52      2.00      1.00      2.00      0.73      1.45      0.28      5.00
          17.00     0.56      2.00      1.00      2.00      0.73      1.45      0.28      5.00
          18.00     0.60      2.00      1.00      2.00      0.73      1.45      0.28      5.00
          _____________________________________________________________________________________
          * F.S.<1: Liquefaction Potential Zone.  (If above water table: F.S.=5)
          ^ No-liquefiable Soils or above Water Table.
```

```
(F.S. is limited to 5,  CRR is limited to 2,    CSR is limited to 2)


CPT convert to SPT for Settlement Analysis:
Fines Correction for Settlement Analysis:
Depth    Ic        qc/N60  qc1      (N1)60  Fines    d(N1)60 (N1)60s
ft                         atm              %
_________________________________________________________________
0.00     -         -       -        1.59    0.00     0.00    1.59
1.00     -         -       -        1.59    0.00     0.00    1.59
2.00     -         -       -        1.59    0.00     0.00    1.59
3.00     -         -       -        1.59    0.00     0.00    1.59
4.00     -         -       -        3.19    0.00     0.00    3.19
5.00     -         -       -        1.59    0.00     0.00    1.59
6.00     -         -       -        12.75   0.00     0.00    12.75
7.00     -         -       -        12.75   0.00     0.00    12.75
8.00     -         -       -        12.75   0.00     0.00    12.75
9.00     -         -       -        13.61   0.00     0.00    13.61
10.00    -         -       -        18.72   26.00    0.00    18.72
11.00    -         -       -        100.00  26.00    0.00    100.00
12.00    -         -       -        100.00  26.00    0.00    100.00
13.00    -         -       -        100.00  26.00    0.00    100.00
14.00    -         -       -        100.00  26.00    0.00    100.00
15.00    -         -       -        100.00  26.00    0.00    100.00
16.00    -         -       -        100.00  26.00    0.00    100.00
17.00    -         -       -        100.00  26.00    0.00    100.00
18.00    -         -       -        100.00  26.00    0.00    100.00
_________________________________________________________________
        (N1)60s has been fines corrected in liquefaction analysis, therefore
d(N1)60=0.
        Fines=NoLiq means the soils are not liquefiable.


        Settlement of Saturated Sands:
        Settlement Analysis Method: Tokimatsu, M-correction
        Depth    CSRsf    / MSF*  =CSRm    F.S.     Fines    (N1)60s Dr       ec       dsz
dsp     S
        ft                                          %                %        %        in.
in.     in.

____________________________________________________________________________________
_________
        No Settlement of Saturated Sands

____________________________________________________________________________________
_________
        Settlement of Saturated Sands=0.000 in.
        qc1 and (N1)60 is after fines correction in liquefaction analysis
        dsz is per each segment, dz=0.05 ft
        dsp is per each print interval,  dp=1.00 ft
        S is cumulated settlement at this depth

        Settlement of Unsaturated Sands:
        Depth    sigma'   sigC'    (N1)60s CSRsf    Gmax     g*Ge/Gm  g_eff    ec7.5    Cec
ec      dsz      dsp      S
        ft       atm      atm                       atm                        %
%       in.      in.      in.

____________________________________________________________________________________
__________________________________
        18.45    0.94     0.61     100.00  0.27     1622.00  1.6E-4   0.0237   0.0075   1.25
0.0094  1.12E-4  0.000    0.000
        18.00    0.92     0.60     100.00  0.28     1599.61  1.6E-4   0.0233   0.0074   1.25
```

```
0.0092   1.11E-4 0.001   0.001
         17.00   0.86    0.56    100.00   0.28    1548.69 1.5E-4   0.0225   0.0071   1.25
0.0089   1.07E-4 0.002   0.003
         16.00   0.80    0.52    100.00   0.28    1496.03 1.5E-4   0.0216   0.0068   1.25
0.0085   1.02E-4 0.002   0.005
         15.00   0.74    0.48    100.00   0.28    1441.46 1.4E-4   0.0207   0.0066   1.25
0.0082   9.84E-5 0.002   0.007
         14.00   0.69    0.45    100.00   0.28    1384.74 1.4E-4   0.0199   0.0063   1.25
0.0079   9.42E-5 0.002   0.009
         13.00   0.63    0.41    100.00   0.28    1323.47 1.3E-4   0.0190   0.0060   1.25
0.0075   8.99E-5 0.002   0.011
         12.00   0.57    0.37    100.00   0.28    1257.00 1.3E-4   0.0180   0.0057   1.25
0.0071   8.53E-5 0.002   0.013
         11.00   0.50    0.33    100.00   0.28    1186.81 1.2E-4   0.0198   0.0063   1.25
0.0078   9.39E-5 0.002   0.015
         10.00   0.44    0.29    18.72    0.28    636.64  2.0E-4   0.0384   0.0412   1.25
0.0514   6.17E-4 0.007   0.022
         9.00    0.39    0.25    13.61    0.28    537.08  2.0E-4   0.0400   0.0648   1.25
0.0809   9.71E-4 0.016   0.038
         8.00    0.34    0.22    12.75    0.28    494.31  2.0E-4   0.0388   0.0682   1.25
0.0853   1.02E-3 0.019   0.057
         7.00    0.30    0.20    12.75    0.28    461.02  1.8E-4   0.0351   0.0617   1.25
0.0771   9.25E-4 0.019   0.076
         6.00    0.26    0.17    12.75    0.28    425.14  1.7E-4   0.0314   0.0552   1.25
0.0689   8.27E-4 0.017   0.094
         5.00    0.21    0.14    1.59     0.28    194.19  3.1E-4   1.0000   4.6774   1.25
5.8467   7.02E-2 1.403   1.497
         4.00    0.17    0.11    3.19     0.28    218.78  2.2E-4   0.2650   1.2394   1.25
1.5493   1.86E-2 1.005   2.502
         3.00    0.13    0.08    1.59     0.29    150.42  2.4E-4   1.0000   4.6774   1.25
5.8467   7.02E-2 0.237   2.739
         2.00    0.09    0.06    1.59     0.29    122.82  2.0E-4   0.0741   0.3468   1.25
0.4335   5.20E-3 0.530   3.269
         1.00    0.04    0.03    1.59     0.29    86.85   1.4E-4   0.0265   0.1239   1.25
0.1548   1.86E-3 0.055   3.324
         0.00    0.00    0.00    1.59     0.29    1.33    2.2E-6   0.0010   0.0048   1.25
0.0059   7.13E-5 0.024   3.348
```

```
         Settlement of Unsaturated Sands=3.348 in.
         dsz is per each segment, dz=0.05 ft
         dsp is per each print interval,  dp=1.00 ft
         S is cumulated settlement at this depth

         Total Settlement of Saturated and Unsaturated Sands=3.348 in.
         Differential Settlement=1.674 to 2.209 in.

         Units:  Unit: qc, fs, Stress or Pressure = atm (1.0581tsf); Unit Weight =
pcf; Depth = ft; Settlement = in.
```

```
         1 atm (atmosphere) = 1.0581 tsf(1 tsf = 1 ton/ft2 = 2 kip/ft2)
         1 atm (atmosphere) = 101.325 kPa(1 kPa = 1 kN/m2 = 0.001 Mpa)
         SPT             Field data from Standard Penetration Test (SPT)
         BPT             Field data from Becker Penetration Test (BPT)
         qc              Field data from Cone Penetration Test (CPT) [atm (tsf)]
         fs              Friction from CPT testing [atm (tsf)]
         Rf              Ratio of fs/qc (%)
         gamma           Total unit weight of soil
         gamma'          Effective unit weight of soil
         Fines           Fines content [%]
```

```
        D50             Mean grain size
        Dr              Relative Density
        sigma           Total vertical stress [atm]
        sigma'          Effective vertical stress [atm]
        sigC'           Effective confining pressure [atm]
        rd              Acceleration reduction coefficient by Seed
        a_max.          Peak Ground Acceleration (PGA) in ground surface
        mZ              Linear acceleration reduction coefficient X depth
        a_min.          Minimum acceleration under linear reduction, mZ
        CRRv            CRR after overburden stress correction, CRRv=CRR7.5 * Ksig
          CRR7.5                Cyclic resistance ratio (M=7.5)
          Ksig          Overburden stress correction factor for CRR7.5
        CRRm            After magnitude scaling correction CRRm=CRRv * MSF
          MSF           Magnitude scaling factor from M=7.5 to user input M
        CSR             Cyclic stress ratio induced by earthquake
        CSRfs           CSRfs=CSR*fs1 (Default fs1=1)
          fs1           First CSR curve in graphic defined in #9 of Advanced page
          fs2           2nd CSR curve in graphic defined in #9 of Advanced page
        F.S.            Calculated factor of safety against liquefaction
F.S.=CRRm/CSRsf
        Cebs            Energy Ratio, Borehole Dia., and Sampling Method Corrections
        Cr              Rod Length Corrections
        Cn              Overburden Pressure Correction
        (N1)60          SPT after corrections, (N1)60=SPT * Cr * Cn * Cebs
        d(N1)60         Fines correction of SPT
        (N1)60f         (N1)60 after fines corrections, (N1)60f=(N1)60 + d(N1)60
        Cq              Overburden stress correction factor
        qc1             CPT after Overburden stress correction
        dqc1            Fines correction of CPT
        qc1f            CPT after Fines and Overburden correction, qc1f=qc1 + dqc1
        qc1n            CPT after normalization in Robertson's method
        Kc              Fine correction factor in Robertson's Method
        qc1f            CPT after Fines correction in Robertson's Method
        Ic              Soil type index in Suzuki's and Robertson's Methods
        (N1)60s         (N1)60 after settlement fines corrections
        CSRm            After magnitude scaling correction for Settlement
calculation  CSRm=CSRsf / MSF*
          CSRfs                 Cyclic stress ratio induced by earthquake with user
inputed fs
          MSF*                  Scaling factor from CSR, MSF*=MSF, based on Item 2
of Page C.
          MSF           Magnitude scaling factor from M=7.5 to user input M
        ec              Volumetric strain for saturated sands
        dz              Calculation segment, dz=0.050 ft
        dsz             Settlement in each segment, dz
        dp              User defined print interval
        dsp             Settlement in each print interval, dp
        Gmax            Shear Modulus at low strain
        g_eff           gamma_eff, Effective shear Strain
        g*Ge/Gm         gamma_eff * G_eff/G_max,        Strain-modulus ratio
        ec7.5           Volumetric Strain for magnitude=7.5
        Cec             Magnitude correction factor for any magnitude
        ec              Volumetric strain for unsaturated sands, ec=Cec * ec7.5
        NoLiq           No-Liquefy Soils

        References:
```

```
        1. NCEER Workshop on Evaluation of Liquefaction Resistance of Soils. Youd,
T.L., and Idriss, I.M., eds., Technical Report NCEER 97-0022.
           SP117. Southern California Earthquake Center. Recommended Procedures for
Implementation of DMG Special Publication 117, Guidelines for
           Analyzing and Mitigating Liquefaction in California. University of
```

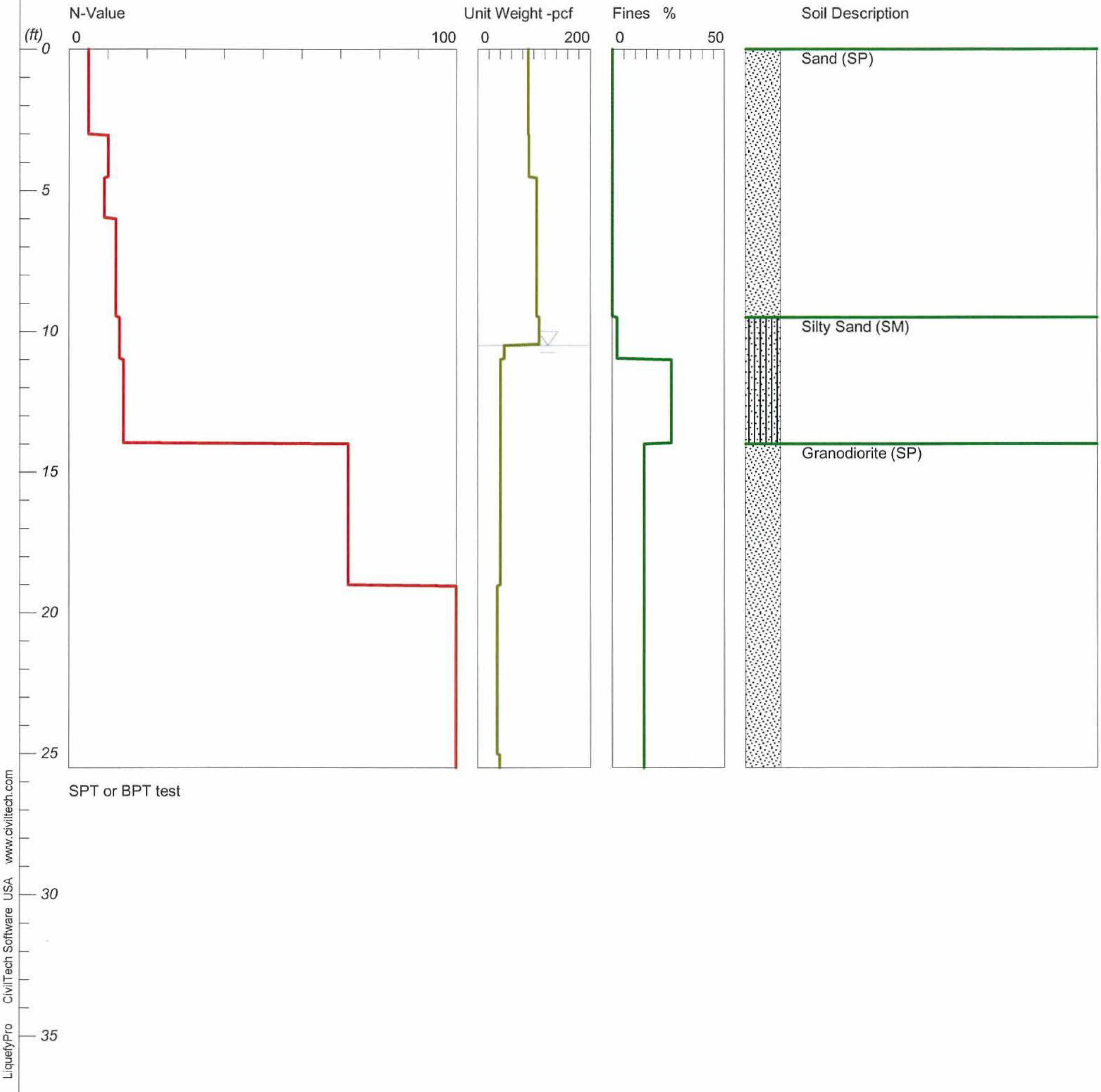
```
Southern California. March 1999.
        2. RECENT ADVANCES IN SOIL LIQUEFACTION ENGINEERING AND SEISMIC SITE
RESPONSE EVALUATION, Paper No. SPL-2, PROCEEDINGS: Fourth
           International Conference on Recent Advances in Geotechnical Earthquake
Engineering and Soil Dynamics, San Diego, CA, March 2001.
        3. RECENT ADVANCES IN SOIL LIQUEFACTION ENGINEERING: A UNIFIED AND
CONSISTENT FRAMEWORK, Earthquake Engineering Research Center,
           Report No. EERC 2003-06 by R.B Seed and etc. April 2003.

        Note: Print Interval you selected does not show complete results. To get
complete results, you should select 'Segment' in Print Interval (Item 12, Page C).
```

Liquefaction and Dry Settlement Analysis

New Residence 1170 Signal Hill Road Pebble Beach,

Hole No.=EB-7   Water Depth=10.5 ft

Magnitude=8.5
Acceleration=0.442g

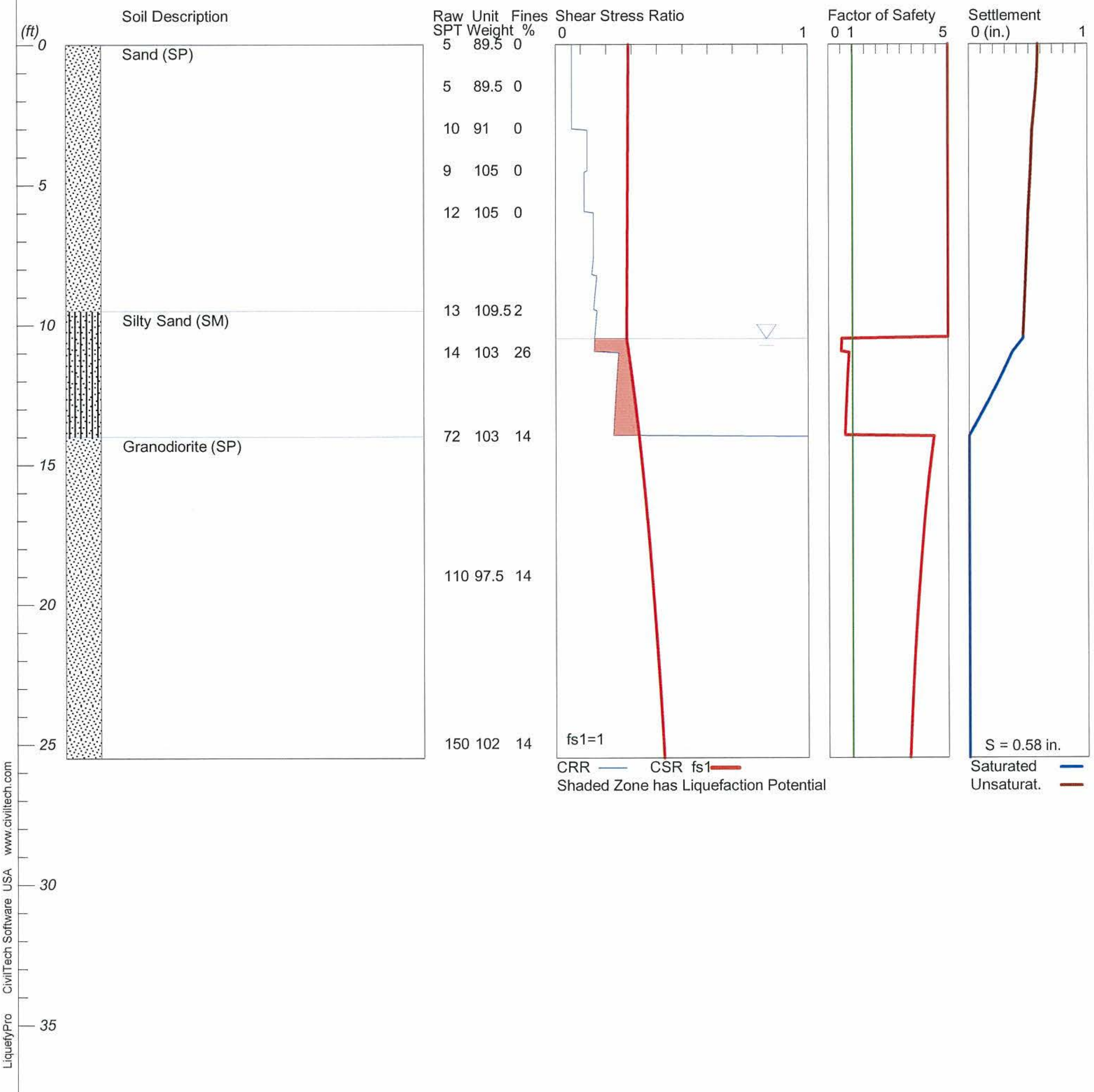


Liquefaction and Dry Settlement Analysis

New Residence 1170 Signal Hill Road Pebble Beach,

Hole No.=EB-7 Water Depth=10.5 ft

Magnitude=8.5
Acceleration=0.442g



```
************************************************************************************
*******************
                                   LIQUEFACTION ANALYSIS CALCULATION DETAILS

                                         Copyright by CivilTech Software
                                              www.civiltechsoftware.com


************************************************************************************
*******************
        Font: Courier New, Regular, Size 8 is recommended for this report.
        Licensed to ,    3/19/2010       4:14:45 PM

        Input File Name: C:\Liquefy5\1170 Siganl Hill Road EB7.liq
        Title:  New Residence 1170 Signal Hill Road Pebble Beach,
        Subtitle:

 Input Data:
        Surface Elev.=
        Hole No.=EB-7
        Depth of Hole=25.50 ft
        Water Table during Earthquake= 10.50 ft
        Water Table during In-Situ Testing= 10.50 ft
        Max. Acceleration=0.44 g
        Earthquake Magnitude=8.50
        No-Liquefiable Soils:   Based on Analysis
        1. SPT or BPT Calculation.
        2. Settlement Analysis Method: Tokimatsu, M-correction
        3. Fines Correction for Liquefaction: Idriss/Seed
        4. Fine Correction for Settlement: During Liquefaction*
        5. Settlement Calculation in: All zones*
        6. Hammer Energy Ratio,                                    Ce = 1.25
        7. Borehole Diameter,                                          Cb= 1
        8. Sampling Method,                                            Cs= 1
        9. User request factor of safety (apply to CSR) ,   User= 1
           Plot one CSR curve (fs1=1)
        10. Average two input data between two Depths: No
        * Recommended Options

        In-Situ Test Data:
        Depth    SPT      Gamma    Fines
        ft                pcf      %
        __________________________________
        0.00     5.00     89.50    0.00
        1.50     5.00     89.50    0.00
        3.00     10.00    91.00    0.00
        4.50     9.00     105.00   0.00
        6.00     12.00    105.00   0.00
        9.50     13.00    109.50   2.00
        11.00    14.00    103.00   26.00
        14.00    72.00    103.00   14.00
        19.00    110.00   97.50    14.00
        25.00    150.00   102.00   14.00
        __________________________________


 Output Results:
        Calculation segment, dz=0.050 ft
        User defined Print Interval, dp=1.00 ft

        Peak Ground Acceleration (PGA), a_max = 0.44g
```

CSR Calculation:

Depth	gamma	sigma	gamma'	sigma'	rd	mZ	a(z)	CSR	x	fs1 =CSRfs
ft	pcf	atm	pcf	atm		g	g			
0.00	89.50	0.000	89.50	0.000	1.00	0.000	0.442	0.29	1.00	0.29
1.00	89.50	0.042	89.50	0.042	1.00	0.000	0.442	0.29	1.00	0.29
2.00	89.50	0.085	89.50	0.085	1.00	0.000	0.442	0.29	1.00	0.29
3.00	89.50	0.127	89.50	0.127	0.99	0.000	0.442	0.29	1.00	0.29
4.00	91.00	0.170	91.00	0.170	0.99	0.000	0.442	0.28	1.00	0.28
5.00	105.00	0.216	105.00	0.216	0.99	0.000	0.442	0.28	1.00	0.28
6.00	105.00	0.265	105.00	0.265	0.99	0.000	0.442	0.28	1.00	0.28
7.00	105.00	0.315	105.00	0.315	0.98	0.000	0.442	0.28	1.00	0.28
8.00	105.00	0.365	105.00	0.365	0.98	0.000	0.442	0.28	1.00	0.28
9.00	105.00	0.414	105.00	0.414	0.98	0.000	0.442	0.28	1.00	0.28
10.00	109.50	0.465	109.50	0.465	0.98	0.000	0.442	0.28	1.00	0.28
11.00	103.00	0.517	40.60	0.502	0.97	0.000	0.442	0.29	1.00	0.29
12.00	103.00	0.565	40.60	0.521	0.97	0.000	0.442	0.30	1.00	0.30
13.00	103.00	0.614	40.60	0.540	0.97	0.000	0.442	0.32	1.00	0.32
14.00	103.00	0.663	40.60	0.560	0.97	0.000	0.442	0.33	1.00	0.33
15.00	103.00	0.711	40.60	0.579	0.97	0.000	0.442	0.34	1.00	0.34
16.00	103.00	0.760	40.60	0.598	0.96	0.000	0.442	0.35	1.00	0.35
17.00	103.00	0.809	40.60	0.617	0.96	0.000	0.442	0.36	1.00	0.36
18.00	103.00	0.857	40.60	0.636	0.96	0.000	0.442	0.37	1.00	0.37
19.00	103.00	0.906	40.60	0.655	0.96	0.000	0.442	0.38	1.00	0.38
20.00	97.50	0.952	35.10	0.672	0.95	0.000	0.442	0.39	1.00	0.39
21.00	97.50	0.998	35.10	0.689	0.95	0.000	0.442	0.40	1.00	0.40
22.00	97.50	1.044	35.10	0.705	0.95	0.000	0.442	0.40	1.00	0.40
23.00	97.50	1.091	35.10	0.722	0.95	0.000	0.442	0.41	1.00	0.41
24.00	97.50	1.137	35.10	0.739	0.94	0.000	0.442	0.42	1.00	0.42
25.00	97.50	1.183	35.10	0.755	0.94	0.000	0.442	0.42	1.00	0.42

CSR is based on water table at 10.50 during earthquake

```
        CRR Calculation from SPT or BPT data:
        Depth   SPT     Cebs    Cr      sigma'  Cn      (N1)60  Fines   d(N1)60
(N1)60f CRR7.5
        ft                              atm                     %

_____________________________________________________________________________________
_
        0.00    5.00    1.25    0.75    0.000   1.70    7.97    0.00    0.00    7.97
0.09
        1.00    5.00    1.25    0.75    0.042   1.70    7.97    0.00    0.00    7.97
0.09
        2.00    5.00    1.25    0.75    0.085   1.70    7.97    0.00    0.00    7.97
0.09
        3.00    5.00    1.25    0.75    0.127   1.70    7.97    0.00    0.00    7.97
0.09
        4.00    10.00   1.25    0.75    0.170   1.70    15.94   0.00    0.00
15.94   0.17
        5.00    9.00    1.25    0.75    0.216   1.70    14.34   0.00    0.00
14.34   0.16
        6.00    12.00   1.25    0.75    0.265   1.70    19.13   0.00    0.00
19.13   0.21
        7.00    12.00   1.25    0.75    0.315   1.70    19.13   0.00    0.00
19.13   0.21
        8.00    12.00   1.25    0.75    0.365   1.66    18.63   0.00    0.00
18.63   0.20
        9.00    12.00   1.25    0.85    0.414   1.55    19.81   0.00    0.00
19.81   0.21
        10.00   13.00   1.25    0.85    0.465   1.47    20.26   2.00    0.00
20.26   0.22
        11.00   14.00   1.25    0.85    0.502   1.41    20.99   26.00   6.96
27.96   0.34
        12.00   14.00   1.25    0.85    0.521   1.39    20.60   26.00   6.91
27.52   0.33
        13.00   14.00   1.25    0.85    0.540   1.36    20.24   26.00   6.87
27.10   0.32
        14.00   72.00   1.25    0.85    0.560   1.34    102.27  14.00   6.54
108.81  2.00
        15.00   72.00   1.25    0.95    0.579   1.31    112.39  14.00   6.97
119.36  2.00
        16.00   72.00   1.25    0.95    0.598   1.29    110.57  14.00   6.89
117.46  2.00
        17.00   72.00   1.25    0.95    0.617   1.27    108.84  14.00   6.82
115.66  2.00
        18.00   72.00   1.25    0.95    0.636   1.25    107.19  14.00   6.75
113.93  2.00
        19.00   72.00   1.25    0.95    0.655   1.24    105.61  14.00   6.68
112.29  2.00
        20.00   110.00  1.25    0.95    0.672   1.22    159.32  14.00   8.96
168.28  2.00
        21.00   110.00  1.25    0.95    0.689   1.20    157.39  14.00   8.88
166.27  2.00
        22.00   110.00  1.25    0.95    0.705   1.19    155.53  14.00   8.80
164.33  2.00
        23.00   110.00  1.25    0.95    0.722   1.18    153.74  14.00   8.72
162.46  2.00
        24.00   110.00  1.25    0.95    0.739   1.16    152.00  14.00   8.65
160.65  2.00
        25.00   110.00  1.25    0.95    0.755   1.15    150.32  14.00   8.58
158.90  2.00

_____________________________________________________________________________________
_
```

```
        CRR is based on water table at 10.50 during In-Situ Testing

        Factor of Safety,  - Earthquake Magnitude= 8.50:
        Depth   sigC'   CRR7.5  x Ksig  =CRRv   x MSF   =CRRm   CSRfs
F.S.=CRRm/CSRfs
        ft      atm
        _______________________________________________________________________
        0.00    0.00    0.09    1.00    0.09    0.73    0.06    0.29    5.00
        1.00    0.03    0.09    1.00    0.09    0.73    0.06    0.29    5.00
        2.00    0.05    0.09    1.00    0.09    0.73    0.06    0.29    5.00
        3.00    0.08    0.09    1.00    0.09    0.73    0.06    0.29    5.00
        4.00    0.11    0.17    1.00    0.17    0.73    0.12    0.28    5.00
        5.00    0.14    0.16    1.00    0.16    0.73    0.11    0.28    5.00
        6.00    0.17    0.21    1.00    0.21    0.73    0.15    0.28    5.00
        7.00    0.20    0.21    1.00    0.21    0.73    0.15    0.28    5.00
        8.00    0.24    0.20    1.00    0.20    0.73    0.15    0.28    5.00
        9.00    0.27    0.21    1.00    0.21    0.73    0.16    0.28    5.00
        10.00   0.30    0.22    1.00    0.22    0.73    0.16    0.28    5.00
        11.00   0.33    0.34    1.00    0.34    0.73    0.25    0.29    0.86 *
        12.00   0.34    0.33    1.00    0.33    0.73    0.24    0.30    0.79 *
        13.00   0.35    0.32    1.00    0.32    0.73    0.23    0.32    0.74 *
        14.00   0.36    2.00    1.00    2.00    0.73    1.45    0.33    4.41
        15.00   0.38    2.00    1.00    2.00    0.73    1.45    0.34    4.26
        16.00   0.39    2.00    1.00    2.00    0.73    1.45    0.35    4.13
        17.00   0.40    2.00    1.00    2.00    0.73    1.45    0.36    4.01
        18.00   0.41    2.00    1.00    2.00    0.73    1.45    0.37    3.91
        19.00   0.43    2.00    1.00    2.00    0.73    1.45    0.38    3.82
        20.00   0.44    2.00    1.00    2.00    0.73    1.45    0.39    3.74
        21.00   0.45    2.00    1.00    2.00    0.73    1.45    0.40    3.66
        22.00   0.46    2.00    1.00    2.00    0.73    1.45    0.40    3.60
        23.00   0.47    2.00    1.00    2.00    0.73    1.45    0.41    3.53
        24.00   0.48    2.00    1.00    2.00    0.73    1.45    0.42    3.48
        25.00   0.49    2.00    1.00    2.00    0.73    1.45    0.42    3.42
        _______________________________________________________________________
        * F.S.<1: Liquefaction Potential Zone.  (If above water table: F.S.=5)
        ^ No-liquefiable Soils or above Water Table.
        (F.S. is limited to 5,  CRR is limited to 2,    CSR is limited to 2)


        CPT convert to SPT for Settlement Analysis:
        Fines Correction for Settlement Analysis:
        Depth   Ic      qc/N60  qc1     (N1)60  Fines   d(N1)60 (N1)60s
        ft                      atm             %
        _______________________________________________________________
        0.00    -       -       -       7.97    0.00    0.00    7.97
        1.00    -       -       -       7.97    0.00    0.00    7.97
        2.00    -       -       -       7.97    0.00    0.00    7.97
        3.00    -       -       -       7.97    0.00    0.00    7.97
        4.00    -       -       -       15.94   0.00    0.00    15.94
        5.00    -       -       -       14.34   0.00    0.00    14.34
        6.00    -       -       -       19.13   0.00    0.00    19.13
        7.00    -       -       -       19.13   0.00    0.00    19.13
        8.00    -       -       -       18.63   0.00    0.00    18.63
        9.00    -       -       -       19.81   0.00    0.00    19.81
        10.00   -       -       -       20.26   2.00    0.00    20.26
        11.00   -       -       -       27.96   26.00   0.00    27.96
        12.00   -       -       -       27.52   26.00   0.00    27.52
        13.00   -       -       -       27.10   26.00   0.00    27.10
        14.00   -       -       -       100.00  14.00   0.00    100.00
        15.00   -       -       -       100.00  14.00   0.00    100.00
        16.00   -       -       -       100.00  14.00   0.00    100.00
        17.00   -       -       -       100.00  14.00   0.00    100.00
        18.00   -       -       -       100.00  14.00   0.00    100.00
```

```
        19.00   -       -       -       100.00  14.00   0.00    100.00
        20.00   -       -       -       100.00  14.00   0.00    100.00
        21.00   -       -       -       100.00  14.00   0.00    100.00
        22.00   -       -       -       100.00  14.00   0.00    100.00
        23.00   -       -       -       100.00  14.00   0.00    100.00
        24.00   -       -       -       100.00  14.00   0.00    100.00
        25.00   -       -       -       100.00  14.00   0.00    100.00
        _______________________________________________________________
        (N1)60s has been fines corrected in liquefaction analysis, therefore
d(N1)60=0.
        Fines=NoLiq means the soils are not liquefiable.


        Settlement of Saturated Sands:
        Settlement Analysis Method: Tokimatsu, M-correction
        Depth   CSRsf   / MSF*  =CSRm   F.S.    Fines   (N1)60s Dr      ec      dsz
dsp     S
        ft                                      %               %       %       in.
in.     in.

_____________________________________________________________________________________

__________
        25.45   0.43    0.73    0.59    3.40    14.00   100.00  100.00  0.000
0.0E0   0.000   0.000
        25.00   0.42    0.73    0.58    3.42    14.00   100.00  100.00  0.000
0.0E0   0.000   0.000
        24.00   0.42    0.73    0.58    3.48    14.00   100.00  100.00  0.000
0.0E0   0.000   0.000
        23.00   0.41    0.73    0.57    3.53    14.00   100.00  100.00  0.000
0.0E0   0.000   0.000
        22.00   0.40    0.73    0.56    3.60    14.00   100.00  100.00  0.000
0.0E0   0.000   0.000
        21.00   0.40    0.73    0.55    3.66    14.00   100.00  100.00  0.000
0.0E0   0.000   0.000
        20.00   0.39    0.73    0.53    3.74    14.00   100.00  100.00  0.000
0.0E0   0.000   0.000
        19.00   0.38    0.73    0.52    3.82    14.00   100.00  100.00  0.000
0.0E0   0.000   0.000
        18.00   0.37    0.73    0.51    3.91    14.00   100.00  100.00  0.000
0.0E0   0.000   0.000
        17.00   0.36    0.73    0.50    4.01    14.00   100.00  100.00  0.000
0.0E0   0.000   0.000
        16.00   0.35    0.73    0.48    4.13    14.00   100.00  100.00  0.000
0.0E0   0.000   0.000
        15.00   0.34    0.73    0.47    4.26    14.00   100.00  100.00  0.000
0.0E0   0.000   0.000
        14.00   0.33    0.73    0.45    4.41    14.00   100.00  100.00  0.000
0.0E0   0.000   0.000
        13.00   0.32    0.73    0.44    0.74    26.00   27.10   83.86   1.031
6.2E-3  0.126   0.126
        12.00   0.30    0.73    0.42    0.79    26.00   27.52   84.71   0.984
5.9E-3  0.121   0.247
        11.00   0.29    0.73    0.40    0.86    26.00   27.96   85.62   0.876
5.3E-3  0.112   0.358
        10.50   0.28    0.73    0.39    0.55    2.00    19.71   70.02   1.507
9.0E-3  0.091   0.449

_____________________________________________________________________________________

__________
        Settlement of Saturated Sands=0.449 in.
        qc1 and (N1)60 is after fines correction in liquefaction analysis
        dsz is per each segment, dz=0.05 ft
        dsp is per each print interval,  dp=1.00 ft
```

```
        S is cumulated settlement at this depth

        Settlement of Unsaturated Sands:
        Depth   sigma'  sigC'   (N1)60s CSRsf   Gmax    g*Ge/Gm g_eff   ec7.5   Cec
ec      dsz     dsp     S
        ft      atm     atm                     atm                     %
%       in.     in.     in.

_______________________________________________________________________________________
____________________________
        10.45   0.49    0.32    19.77   0.28    680.53  2.0E-4  0.0391  0.0391  1.25
0.0489  5.86E-4 0.001   0.001
        10.00   0.46    0.30    20.26   0.28    669.53  1.9E-4  0.0382  0.0370  1.25
0.0463  5.56E-4 0.005   0.006
        9.00    0.41    0.27    19.81   0.28    627.31  1.9E-4  0.0356  0.0355  1.25
0.0444  5.32E-4 0.011   0.017
        8.00    0.36    0.24    18.63   0.28    576.64  1.8E-4  0.0335  0.0362  1.25
0.0453  5.43E-4 0.010   0.027
        7.00    0.32    0.20    19.13   0.28    540.69  1.6E-4  0.0300  0.0313  1.25
0.0392  4.70E-4 0.010   0.037
        6.00    0.27    0.17    19.13   0.28    496.29  1.5E-4  0.0269  0.0280  1.25
0.0350  4.20E-4 0.009   0.046
        5.00    0.22    0.14    14.34   0.28    406.63  1.5E-4  0.0296  0.0448  1.25
0.0560  6.71E-4 0.013   0.059
        4.00    0.17    0.11    15.94   0.28    373.61  1.3E-4  0.0239  0.0317  1.25
0.0396  4.75E-4 0.011   0.070
        3.00    0.13    0.08    7.97    0.29    256.36  1.4E-4  0.0267  0.0808  1.25
0.1010  1.21E-3 0.010   0.080
        2.00    0.08    0.05    7.97    0.29    209.32  1.2E-4  0.0246  0.0745  1.25
0.0931  1.12E-3 0.022   0.103
        1.00    0.04    0.03    7.97    0.29    148.02  8.2E-5  0.0145  0.0439  1.25
0.0549  6.59E-4 0.018   0.121
        0.00    0.00    0.00    7.97    0.29    2.28    1.3E-6  0.0010  0.0031  1.25
0.0038  4.62E-5 0.007   0.128

_______________________________________________________________________________________
____________________________
        Settlement of Unsaturated Sands=0.128 in.
        dsz is per each segment, dz=0.05 ft
        dsp is per each print interval,  dp=1.00 ft
        S is cumulated settlement at this depth

        Total Settlement of Saturated and Unsaturated Sands=0.577 in.
        Differential Settlement=0.289 to 0.381 in.

        Units:  Unit: qc, fs, Stress or Pressure = atm (1.0581tsf); Unit Weight =
pcf; Depth = ft; Settlement = in.

_______________________________________________________________________________________
_____________
        1 atm (atmosphere) = 1.0581 tsf(1 tsf = 1 ton/ft2 = 2 kip/ft2)
        1 atm (atmosphere) = 101.325 kPa(1 kPa = 1 kN/m2 = 0.001 Mpa)
        SPT             Field data from Standard Penetration Test (SPT)
        BPT             Field data from Becker Penetration Test (BPT)
        qc              Field data from Cone Penetration Test (CPT) [atm (tsf)]
        fs              Friction from CPT testing [atm (tsf)]
        Rf              Ratio of fs/qc (%)
        gamma           Total unit weight of soil
        gamma'          Effective unit weight of soil
        Fines           Fines content [%]
        D50             Mean grain size
        Dr              Relative Density
        sigma           Total vertical stress [atm]
```

```
        sigma'          Effective vertical stress [atm]
        sigC'           Effective confining pressure [atm]
        rd              Acceleration reduction coefficient by Seed
        a_max.          Peak Ground Acceleration (PGA) in ground surface
        mZ              Linear acceleration reduction coefficient X depth
        a_min.          Minimum acceleration under linear reduction, mZ
        CRRv            CRR after overburden stress correction, CRRv=CRR7.5 * Ksig
          CRR7.5                Cyclic resistance ratio (M=7.5)
          Ksig          Overburden stress correction factor for CRR7.5
        CRRm            After magnitude scaling correction CRRm=CRRv * MSF
          MSF           Magnitude scaling factor from M=7.5 to user input M
        CSR             Cyclic stress ratio induced by earthquake
        CSRfs           CSRfs=CSR*fs1 (Default fs1=1)
          fs1           First CSR curve in graphic defined in #9 of Advanced page
          fs2           2nd CSR curve in graphic defined in #9 of Advanced page
        F.S.            Calculated factor of safety against liquefaction
F.S.=CRRm/CSRsf
        Cebs            Energy Ratio, Borehole Dia., and Sampling Method Corrections
        Cr              Rod Length Corrections
        Cn              Overburden Pressure Correction
        (N1)60          SPT after corrections, (N1)60=SPT * Cr * Cn * Cebs
        d(N1)60         Fines correction of SPT
        (N1)60f         (N1)60 after fines corrections, (N1)60f=(N1)60 + d(N1)60
        Cq              Overburden stress correction factor
        qc1             CPT after Overburden stress correction
        dqc1            Fines correction of CPT
        qc1f            CPT after Fines and Overburden correction, qc1f=qc1 + dqc1
        qc1n            CPT after normalization in Robertson's method
        Kc              Fine correction factor in Robertson's Method
        qc1f            CPT after Fines correction in Robertson's Method
        Ic              Soil type index in Suzuki's and Robertson's Methods
        (N1)60s         (N1)60 after settlement fines corrections
        CSRm            After magnitude scaling correction for Settlement
calculation  CSRm=CSRsf / MSF*
          CSRfs                 Cyclic stress ratio induced by earthquake with user
inputed fs
          MSF*                  Scaling factor from CSR, MSF*=MSF, based on Item 2
of Page C.
          MSF           Magnitude scaling factor from M=7.5 to user input M
        ec              Volumetric strain for saturated sands
        dz              Calculation segment, dz=0.050 ft
        dsz             Settlement in each segment, dz
        dp              User defined print interval
        dsp             Settlement in each print interval, dp
        Gmax            Shear Modulus at low strain
        g_eff           gamma_eff, Effective shear Strain
        g*Ge/Gm         gamma_eff * G_eff/G_max,        Strain-modulus ratio
        ec7.5           Volumetric Strain for magnitude=7.5
        Cec             Magnitude correction factor for any magnitude
        ec              Volumetric strain for unsaturated sands, ec=Cec * ec7.5
        NoLiq           No-Liquefy Soils

        References:
```

---

1. NCEER Workshop on Evaluation of Liquefaction Resistance of Soils. Youd, T.L., and Idriss, I.M., eds., Technical Report NCEER 97-0022.

SP117. Southern California Earthquake Center. Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for

Analyzing and Mitigating Liquefaction in California. University of Southern California. March 1999.

2. RECENT ADVANCES IN SOIL LIQUEFACTION ENGINEERING AND SEISMIC SITE RESPONSE EVALUATION, Paper No. SPL-2, PROCEEDINGS: Fourth

International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, San Diego, CA, March 2001.

3. RECENT ADVANCES IN SOIL LIQUEFACTION ENGINEERING: A UNIFIED AND CONSISTENT FRAMEWORK, Earthquake Engineering Research Center, Report No. EERC 2003-06 by R.B Seed and etc. April 2003.

Note: Print Interval you selected does not show complete results. To get complete results, you should select 'Segment' in Print Interval (Item 12, Page C).

CLEARY CONSULTANTS, INC.
Geotechnical Engineers and Geologists

J. Michael Cleary, CEG, GE
Christophe A. Ciechanowski, GE
Grant F. Foster, GE

June 22, 2011
Project No. 1301.1
Ser. 3300

Ms. Massy Mehdipour
1425 Dana Avenue
Palo Alto, CA 94301

RE: GEOLOGICAL ANALYSIS OF SITE ERODABILITY
NEW RESIDENTIAL PROJECT
1170 SIGNAL HILL ROAD
PEBBLE BEACH, MONTEREY COUNTY, CALIFORNIA

Dear Ms. Mehdipour:

As requested by Monterey County Planning, December 8, 2010, we have prepared this analysis and review of the potential site erodability and mitigation measures for your new residential project at 1170 Signal Hill Road in Pebble Beach. Our geotechnical investigation report for this project was submitted March 31, 2010. Our analysis included review of the following drawings:

- Site Plan (A-1.0), Ground Floor/Basement Plan (A-3.0) and First Floor Plan (A-3.1) for Casa Pebble Beach, 1170 Signal Hill Road, Pebble Beach, California prepared by Bill Bernstein AIA and Legorretta and Legoretta Architects, dated June 3, 2011, May 23, 2011 and May 27, 2011.

- Grading and Drainage Plans, C0.2 and C1.1, Single Family Residence and Driveway, 1170 Signal Hill Drive, Monterey County, Prepared by Whitson Engineers, June 20, 2011.

The grading and drainage plans indicate that the proposed development area within the designated "Limits of Developed/Disturbed Dune" will be cut down five feet maximum in the backyard, resulting in a berm at approximately Elevation 98 behind the home, and the front yard will be raised with up to about five feet of fill in the area of the garage driveway and front entry. The front portion of the home, excluding the garage, will be set into the slope, requiring cuts of up to about nine feet. Runoff from most of the front yard portion of the site will be directed to area drains connected to a storm drain tightline and carried to a new rip rap stilling basin for infiltration into the sandy soils in the southwest corner of the developed area. (Roof leaders on the south side of the home will be tied into this system). Runoff in the backyard will sheet flow to the contained level area (Elevation 94) located in the northwest portion of the backyard.

The runoff from the landscaped northerly one-third portion of the front yard will be directed around the north side of the home toward the contained low area in the northwest portion of the backyard.

We understand that the final location of the roof downspout leaders has not been determined at this time, however as discussed with Michael Baldi with Whitson Engineers, roof runoff will be tied into tightline disposal where practical or discharged into dry wells located at least three to five feet out from the residence.

The proposed cut and fill slopes within the area to be developed are shown at a 3:1 (horizontal to vertical) gradient, and these slopes will be vegetated in accordance with the recommendations of the project biologist and landscape architect.

Based on the above, it is our opinion that the planned residential project at 1170 Signal Hill Road as currently designed will mitigate the potential for erosion at the site. This applies to the construction period as well since we understand construction activities will be confined to the limits of the undisturbed dune line specified for the development, and disturbed areas and temporary slopes will be winterized as recommended in the geotechnical report.

We have provided our services in accordance with generally accepted geotechnical engineering principles and practice. No other warranty is implied.

We appreciate the opportunity to have been of continued service to you on this project. If you have any questions regarding this letter, please call.

Very truly yours,

CLEARY CONSULTANTS, INC.

J. Michael Cleary
Engineering Geologist 352
Geotechnical Engineer 222

J.M. CLEARY
No. 222
EXP. 9-30-11

J. M. CLEARY
No. 352

JMC:cm
Copies: Addressee (1)
Bill Bernstein AIA (3) Attn: William Bernstein
Whitson Engineers (1) Attn: Michael Baldi

CLEARY CONSULTANTS, INC.
Geotechnical Engineers and Geologists

J. Michael Cleary, CEG, GE
Christophe A. Ciechanowski, GE
Grant F. Foster, GE

November 23, 2011
Project No. 1301.1
Ser. 3456

Ms. Massy Mehdipour
1425 Dana Avenue
Palo Alto, CA 94301

RE: DRILLING OF SOIL BORINGS FOR GEOTECHNICAL INVESTIGATION
NEW RESIDENCE
1170 SIGNAL HILL ROAD
PEBBLE BEACH, MONTEREY COUNTY, CALIFORNIA

Dear Ms. Mehdipour:

This is to confirm that the soil borings drilled in February, 2010 for your planned new residence did not result in disturbance to the dune. The borings were drilled with a track-mounted auger rig requiring no grading or removal of vegetation; and were backfilled with the native sandy soil.

Please contact our office if you have any further questions regarding this matter.

Yours very truly,
CLEARY CONSULTANTS, INC.

Grant Foster
Geotechnical Engineer 2662

GRANT FOSTER
No. 2662

J. Michael Cleary
Geotechnical Engineer 222

J. M. CLEARY
No. 222

GF/JMC:pf
Copies: Addressee (1)
Bill Bernstein AIA (2) Attn: William Bernstein