JF Consulting, Inc.

Geotechnical Services

September 4, 2012
Project 1492

Mr. Andrew 'Boz' Bosworth
3 Greenfield Court
San Mateo, CA
94403

Subject: Geotechnical Investigation
New Residence and Swimming Pool
Lands of Bosworth
3 & 7 Greenfield Court
San Mateo, California

Dear Mr. Bosworth:

This report presents the results of our geotechnical investigation of your property located at 3 & 7 Greenfield Court in San Mateo, California. At present, each of these parcels supports a single-family residence. These structures will be demolished and a single, new residence with a swimming pool constructed. The existing swimming pool in the front yard will be decommissioned. Figure 1 of this report is a map showing the location of the property in relationship to nearby landmarks.

INFORMATION PROVIDED

We were provided with a site plan showing the location of the existing structures on the two parcels and the 'footprint' of the proposed residence and swimming pool. These plans were prepared by Oculus Architecture entitled "Site Plan - 3/7 Greenfield Court - Bosworth Residence - APN 042-251-260, dated June 18, 2012. We used this plan to create our 'Site Plan & Geologic Map', Figure 2 appended to this report. Our Figure 3 is a cross-section through the main body of the new residence, showing the existing structure and the extent of proposed construction at the rear of the new residence.

SCOPE OF WORK

We performed the followings items of work for this study:

1. Reviewed available geologic maps and reports pertaining to the area.

2. Studied aerial photographs of the site and surrounding areas.

3. Advanced five soil borings in the approximate locations shown on our site plan, using a portable, Minute Man Drill Rig.

4. Logged, sampled and classified the soil/bedrock materials encountered in each boring.

5. Performed laboratory testing on selected, recovered samples to measure their pertinent engineering characteristics.

6. Prepared this report presenting our findings and recommendations, including:

    a. a discussion of the potential geologic and geotechnical hazards that might affect the property;
    b. the suitability of the site for the proposed construction;
    c. recommendations for site preparation, grading and compaction;
    d. recommendations for foundation piers and grade beams;
    e. recommendations for construction of the new swimming pool;
    f. recommendations for the decommissioning of the existing pool;
    g. recommendations for construction of new parking areas and driveways;
    h. recommendations for the construction of underground utility trenches;
    i. design parameters for the basement walls and floor slab; and,
    j. a discussion of the control of site surface drainage.

FINDINGS

Site Description

The irregularly shaped parcels (#3 and #7 Greenfield) lie on the north side of Greenfield Court. The existing residence at 3 Greenfield Court is a two-story, wood-framed residence with the front on level ground. The rear of the house extends over the break in slope with a lower level. The house has a stucco finish that exhibits little evidence of settlement or movement. A small swimming pool is located in the front yard, to the left side of the residence. There are large outcrops of Franciscan Chert formation bedrock in the front yard. The 'cuts' made for construction of Greenfield Court also expose the same bedrock formation. We believe that this bedrock will be difficult to excavate and to advance pier holes. Powerful heavy equipment will probably be required to make the required excavations for the lower level of the proposed construction and to drill the required piers for the foundations of the new house and associated retaining walls.

The residence at 7 Greenfield Court has a garage at street level, with the balance of the house constructed on the slope behind the garage. Both residences will be removed from the site to make room to construct the new home.

The existing swimming pool will be removed and the excavation filled with engineered fill.

Vegetation on the top of the site is minimal, mostly decorative bushes, flowers and small trees. One larger oak tree is in the front yard.

The land below the house slopes to the north at an inclination of about 2.5:1 (horizontal to vertical). The ground cover consists of ivy, native weeds and bushes and oak trees. Evidence of 'soil creep' is apparent in the more mature trees.

Drainage on the site can be characterized as "sheet flow" from the upper levels to the rear yard. Minor erosion was noted at the termination of a rain gutter leader, down near the end of the property.

We noted two small deposits of man-made fill on the slope behind #3 Greenfield Court. The fills are small and maybe a maximum of 4 feet deep. These will be removed during preparation of the building pad.

Figure 2 of this report is our Site Plan & Geologic Map and shows the site's mapped geologic formation, the 'footprints' of the existing structure and the 'footprint' of the proposed structure.

Site Geology and Surficial Soils

Figure 3 of this report is a portion of a geologic map of the area. The map shows the site to be underlain by deposits of Franciscan age chert (fc). This was confirmed by our borings and analysis of the bedrock outcrops in the front yard and in the road cut for Greenfield Court. The attitude of the bedrock 'bedding' is nearly horizontal which is a favorable orientation to the site slopes (See Figure 3 - Cross Section).

Our borings 1-4 were advanced on the slope behind the existing residence and in front of the residence at #7 Greenfield Court using a Minute Man, portable rig. In general, the upper two and one-half to three feet of site soils are colluvium consisting of non-plastic, very sandy CLAY to slightly clayey SAND. Highly weathered bedrock was encountered below the colluvium, with the bedrock being moderately dense, then very dense at around eight feet. Refusal was encountered in all borings between eight and ten feet.

Boring 5 was in the front yard and the small drill rig could not advance more than 6 inches (refusal).

No ground water was encountered in any of the borings.

Based upon our site drilling, we are of the opinion that the colluvium and the upper horizons of the highly weathered bedrock will be prone to significant erosion, if concentrated water is discharged on the slope. Due to the dense nature of the bedrock below, permeability is likely to be very poor.

A sample of the surficial soil was collected and tested in accordance with ASTM D4318 and was found to be non-plastic.

The logs of our borings are appended to this report. Laboratory test results appear on the logs and on other sheets.

Landslide Susceptibility

Figure 4 of this report is a map that includes the subject site. This map was prepared to identify the susceptibility of landsliding on lands underlain by rock types in San Mateo County. In summary, the subject site is located at the margin of lands identified as being within the best category, I and the next best, category II. We are of the opinion that deep-seated landsliding is not a hazard to this property.

Seismicity

The site is located in the seismically active San Francisco Bay Area but is not located within a Special Studies Zone as established by the Alquist-Priolo Earthquake Fault Hazard Zones fault zoning act of 1972. Figures 4 & 6 show the locations of faults with respect to the site.

The faults that might affect the site are:

San Andreas Fault	5.8 km to the southwest
Hayward Fault	25.7 km to the northeast
Calaveras Fault	38.6 km to the northeast

No faults are known, or suspected, to pass through the site. The excavation and logging of a fault exploration trench was not a part of our scope of work.

Considering the seismic history of the Bay Area, we feel it is likely that the site will be shaken by several earthquakes of Richter Magnitude 6.5, or greater during the next 30 years, and by at least one earthquake of Richter Magnitude 7.5, or greater. The severity of the ground shaking at the site will be determined by the size of the earthquake and the distance from the site.

Should a large earthquake occur near to the site, ground shaking at the site will be both severe and prolonged.

The following general site seismic parameters may be used for design of the structure in accordance with Section 1613 of the California Building Code.

Site Class: C (bedrock)

Mapped Acceleration Parameters: Ss (for short periods) = 1.882 g

S1 (for 1-second period) =0.980 g

Site Coefficient: Fa (for short periods) = 1.0
Fv (for 1-second period) =1.3

Adjusted Maximum Considered EQ Spectral Response Acceleration Parameters:
Sms = Fa*Ss = 1.882 g
Sm1 = Fv*S1 =1.273 g

Design Spectal Response Acceleration Parameters:

Sds = 2/3*Sms = 1.255 g
Sd1= 2/3*Sma = 0.849 g

Seismic Design Category: **E**

We should point out that the structural seismic design is not intended to eliminate damage to a structure. The goal of the design system is to minimize the loss of human life. It is unlikely that any structure can be designed to withstand the forces of a great earthquake without any damage at all.

<u>Potential Geologic and Geotechnical Hazards</u>

There are several potential geologic and geotechnical hazards that can affect any given site. They are discussed below, along with any required mitigation measures.

Ground Rupture - In our opinion, this is not a hazard to this site. No mitigation is required.

Ground Shaking - This hazard is common to all properties in California. Mitigate by proper structural design and by following the recommendations presented in this report.

Lurching and Lateral - Spreading Such seismically generated movements are induced in areas with weak soils near open cuts or slopes. Such conditions do not exist on this site. No mitigation is required.

Liquefaction- In our opinion, liquefiable soils are not present on this site. No mitigation is required.

Landsliding -	We do not believe that deep-seated landsliding is a hazard to this site. The bedrock formation underlying the site is almost horizontal. The surficial colluvium and upper portions of the highly weathered bedrock could move downslope in shallow 'skin failures' if activated by concentrated water discharge. Mitigate by strict control of collected surface drainage water.
Compressible Soils-	Such soils are present on this site. Mitigate by supporting the new structures and retaining walls on deep piers.
Expansive Soils -	Such soils are not present on this site. No special mitigation is required.
Erosion -	The site soils are easily eroded. Mitigate by controlling the discharge of concentrated water, both during and after construction.

RECOMMENDATIONS

General

In our opinion, the site is suitable for the proposed development, provided the recommendations given in this report are followed.

Site Preparation, Grading and Compaction

1. It is the responsibility of the Civil Engineer to provide sufficient grade stakes with appropriate information to guide the earthwork contractor.

2. The Soils Engineer must be given at least 48 hours notice of any required presence at the site for any inspection, testing, or observation services. Call JF Consulting, Inc. at (408) 867-6321. JF Consulting, Inc. cannot be held responsible for any delays in construction caused by lack of proper notice.

3. The Soils Engineer can at any time make additional recommendations or change the recommendations given in this report, in order to allow for discovered conditions found during grading. Should our recommendations not be followed, then the owner/builder shall be responsible.

4. The site is underlain by bedrock at shallow depth. This bedrock rock may be difficult or impossible to excavate or to drill foundation pier holes by

conventional equipment. It may be necessary to use pneumatic tools or other special equipment to make such excavations.

The presence of rock at shallow depth below this site should specifically be brought to the attention of the project design team and to the attention of contractors whose work may be affected by the presence of rock.

5. All portions of the site to be covered with cut, fill, buildings, foundations, vehicle pavements or slabs-on-grade should be stripped of surface vegetation (including any major root systems). Any structure, underground utility, etc. within these areas should be removed and disposed of off site. Strippings can be used for later use in landscape areas, but cannot be used as structural fill.

6. Stripped areas should be scarified to a depth of about 6", water-conditioned to bring the soils water content to about 4% above the optimum, and compacted to a density equivalent to at least 90% of the maximum dry density of the soil according to ASTM D1557 (latest Edition). Subgrades and aggregate base rock for driveways should both be compacted to a minimum relative density of 95%.

7. Fill should be placed in thin (8" loose), horizontal layers and then water-conditioned and compacted, as given above.

8. Should import soil be required, a sample should be approved by our office prior to its delivery to the site. In general, any import fill should be granular and non-expansive in nature, and should be free of debris or rocks larger than about 6" in diameter.

9. All fill must be tested and approved by the Soils Engineer.

10. The earthwork contractor should be made aware that the site soils will tend to slough and cave when vertical or steep cuts are made. All excavation work should follow the current OSHA guidelines.

Decommissioning of Existing Swimming Pool

The existing swimming pool in the front yard will be removed. It lies in an area of the future driveway. The following steps should be employed in the decommissioning.

1. Remove the gunite shell and all associated plumbing.

2. Bring the resulting excavation to design grade with either Class 2 Aggregate Baserock or quarry fines, placed in thin (say 6" loose lifts) with each lift compacted to a minimum of 95% relative density, using a vibratory compactor.

3. As the fill approaches design grade by about 2 feet, notch into the sidewalls and

continue placing and compacting the fill to the required degree of compaction.

New Swimming Pool

The new swimming pool will be constructed between new site retaining walls. The excavation for the entire pool should be advanced until hard bedrock is exposed. If this results in 'over-excavation' in some areas, then lean concrete should be placed to form the contour of the pool bottom. The entire bottom of the excavation should be covered with at least 6" of clean, free-draining, 3/4" diameter drain rock. All water that might collect in the drain rock must be collected and discharged by gravity to an acceptable location (probably a sump with a pump).

The pool walls should be designed in accordance to the values appearing in the next section of this report (Retaining Walls)

Lower Level Retaining Walls

The basement walls may be designed according to the following design values.

1. The average bulk density of material placed on the backfill side of the wall will be 120 pcf.

2. The vertical plane extending down from the ground surface to the bottom of the heel of the wall will be subject to pressure that increases linearly with depth as follows.

Condition	Design Pressure
Active, drained	45 pcf
At-rest, drained	65 pcf

   The above values are for non-seismic conditions. Active pressures should only be used for walls that are not restrained to move. Basement walls should be designed for at-rest pressure.

3. The effects of earthquakes may be simulated by applying a horizontal line load surcharge to the stem of the wall at a rate of 16 H^2 lb/horizontal foot of wall, where H is the height of the surface of the backfill above the base of the wall. This surcharge should be applied at a height of 0.6H above the base of the wall.

4. A coefficient of "friction" of 0.3 may be used to calculate the ultimate resistance to horizontal sliding of the wall base over the ground beneath the base.

5. An equivalent fluid pressure of 300 psf/ft may be used to calculate the ultimate passive resistance to lateral movement of the ground in front of the toe of the wall and in front of any "key" beneath the toe or stem of the wall.

6. Foundations for the retaining wall should be supported on reinforced concrete piers. Reinforced concrete piers should be designed using recommendations presented in the building foundation section of this report.

A zone of drainage material at least 18 inches wide should be placed on the backfill side of walls designed for drained condition. This zone should extend up the back of the wall to about 18 inches down from the proposed ground surface above. The upper 18 inches or so of material above the drainage material should consist of native, clayey soil.

The drainage material and the clayey soil cap should be placed in layers about 6 inches thick and moderately compacted by hand-operated equipment to eliminate voids and to minimize post-construction settlement. Heavy compaction should not be applied; otherwise, the design pressure on the wall may be exceeded.

The drainage material should consist of either Class 2 Permeable Material complying with Section 68 of the CALTRANS Standard Specifications, latest edition, or 3/4 to 1½ inch clean, durable coarse aggregate. If the coarse aggregate is chosen as the drainage material, it should be separated from all adjacent soil by Mirafi 700X or a similar filter fabric approved by the project Soil Engineer.

Any water that may accumulate in the drainage material should be collected and discharged by a 4-inch-diameter, perforated pipe placed "holes down" near the bottom of the drainage material. The perforated pipe should have holes no larger than 1/4-inch diameter.

Foundations for Residence & Retaining Walls

The foundations for the new residence and all retaining walls should be supported on reinforced, drilled and cast in place concrete piers and embedded grade beams. We believe that drilling the piers will be difficult, so properly sized equipment must be used.

The piers will derive their support from "skin friction" or adhesion. Piers should be at least 18 inches in diameter and should extend a minimum of 10 feet into the underlying, fresh bedrock. The borings encountered up to 6 feet of weak colluvium and weathered bedrock above the top of the 'fresh' bedrock, therefore, the piers could range in depth from 16 - 20 feet in depth, as measured from the existing ground surface.

We recommend that the piers on the slopes should be advanced to such a depth that the separation of the pier and the slope be a horizontal distance of at least 15 feet. The pier should then extend another 8 feet below this horizontal line. So for a 2:1 (horizontal to vertical slope) the total estimated depth of piers will at least 15 feet, providing competent bedrock has been encountered for a minimum of 10 feet.

Piers should be spaced at least 3 pier diameters apart (center to center) but no more than 8 feet apart. The allowable load carrying capacity (dead plus live loads) of each pier may be calculated assuming "skin friction" or adhesion of 400 psf between the shaft of the pier and adjacent soil/bedrock, but ignoring the upper 5 feet of embedment of the pier below the lowest adjacent grade. End bearing of the pier should be ignored.

The depth of embedment of piers should be also be designed to resist a lateral pressure equivalent to 50 pounds per cubic foot acting on the top 5 feet of piers and across at least 2.5 pier diameters. A passive resistance of 200 pounds per cubic foot may be used. The actual depth of embedment of the piers should be decided by the Soils Engineer in the field at the time of drilling of the pier holes. For planning purposes, however, it may be assumed that the average required embedment will be 20 feet below existing grade for piers.

The allowable foundation pressures given previously may be increased by one-third when considering additional short-term wind or seismic loading.

Reinforced concrete foundation beams should be embedded at least 12 inches below the lowest adjacent grade and should be designed to safely transmit all imposed loads to the supporting perimeter and interior piers. The uphill and downhill perimeters of the proposed house should be connected with reinforced concrete cross beams and should be no more than 20 feet apart.

A member of our staff must be present during the drilling of the piers to judge when the pier hole has encountered the top of the 'fresh' bedrock.

Concrete should only be placed in foundation excavations (piers & grade beams) that have been kept moist, are free of drying cracks and contain no loose or soft soil or debris.

Concrete Slabs-On-Grade

Concrete slabs-on-grade used for living areas should be placed over a capillary break section consisting of a minimum of 5 inches of 3/4 inch diameter drain rock overlaid by a 15 mil thick moisture barrier, such as Stego-wrap. Concrete can be cast directly on this product. If another moisture barrier is chosen, then the membrane should be covered with 1-2 inches of sand.

Exterior slabs, such as for patios and pool decking, should be cast upon a bed of Class 2 Aggregate Baserock at least 6” in thickness and compacted to at least 90% relative density according to ASTM Test 1557. Conventional construction using wire mesh is not acceptable. Slabs will perform longer with less cracking if the following is done.

1. Reinforce slabs using a minimum of #4 bars at 18" centers, both directions.

2. Keep slab sections small, on the order of 8' to 10'. Slabs without construction Joints at frequent spacing are more likely to crack in an uncontrolled manner.

3. Dowel adjacent slab sections together.

4. Slope slabs away from the building foundations and towards appropriate collection and discharge points.

The above items of work will greatly extend the lifetime of the concrete flatwork. Cracking may still appear, but it should be minimal and acceptable in appearance (i.e. along construction joints). The subgrades for all concrete flat work should be inspected by our office to verify that the soils possess adequate moisture content and are free of drying cracks. It may be necessary to re-hydrate the subgrade soils for up to one week in advance of placing aggregate baserock.

Driveway and Parking Areas

The subgrade for the new driveway should be cut to grade, scarified and compacted to a minimum of 95% relative density, based upon ASTM D1557 (latest edition). Class 2 Aggregate Base rock should be placed in two lifts with a total section thickness of 8 inches. Each lift should be compacted to a minimum of 95% relative density. The base rock should be 'virgin' rock and not recycled baserock.

If asphalt concrete pavement is used, then the asphalt concrete must be at least 4" thick and should be placed in two lifts. If concrete is used for the surface, then it must be at least 5" thick, reinforced with #4 bars placed 18" on center, both directions.

If pavers are used, then the base rock section should be the thickness prescribed by the manufacturer of the pavers, but not less than 8".

Site Drainage

Surface drainage gradients should be planned to prevent ponding and to promote drainage of surface water away from building foundations, slabs, edges of pavements and sidewalks, and towards suitable collection and discharge facilities.

Water seepage or the spread of extensive root systems into the soil subgrades of foundations, slabs, or pavements, could cause differential movements and consequent distress in these structural elements. This potential risk should be given due consideration in the design and construction of landscaping.

Providing adequate surface and subsurface drainage is of great importance, as most structures are generally prone to drainage problems. All site drainage water should be handled and discharged in a legal, prudent, reasonable and proper manner so as not to create a nuisance, risk or hazard to this property or adjoining properties.

We generally recommend that structures be equipped with roof gutters and downspouts. All runoff waters including all downspouts, patio, parking and driveway drainage, and all other drainage should be collected in closed solid pipes with periodic cleanouts and discharged into legally approved area storm drain systems or dissipater pits. The dissipater pits should be located as far away from structures with raised floors as the site will allow.

If the above is not totally practical or feasible, then all site drainage waters should be discharged well away from edge of pavements and all building foundation areas. Care should be used so that drainage waters are not concentrated and discharged on adjacent properties. Site drainage waters should be well dispersed in as natural a manner as possible and should not be discharged in a concentrated manner if a legally-approved storm drain system is not present.

It should be noted that moisture is usually present under most structures with raised floors as surface and subsurface waters flow from higher surrounding elevations. To minimize the amount of moisture under a structure, a sub-surface drainage system may be constructed around the perimeter of the structure. The building designer and contractor should very carefully consider and provide for drainage waters that might flow into and be trapped in the foundation crawl space area and also consider potential higher humidity and very good cross-ventilation.

The above site drainage recommendations are general in nature and should be carried out by the house designer, contractor, owner, and future owners to the fullest possible extent. However, from many years of soil engineering experience within Northern California, we have found that water and moisture below most structures is relatively common. Therefore, we suggest that if the owner desires assurance with respect to site drainage, an expert in the field of hydrology and drainage should be retained to prepare specific recommendations.

For this site the water collected from roof gutters and area drains should be disposed of in areas away from the new construction near the western property line.

Additional Geotechnical Work

We recommend that JF Consulting, Inc. be retained to provide the following work:

1. Work with project designers to prepare the plans and specifications.
2. Review all project plans and specifications prior to the job going to bid.
3. Observe and advise during site preparation.
4. Observe, test and advise during all phases of earthwork construction.
5. Observe the excavations for foundation piers;

6. Observe and advise during the excavation for the new pool;
7. Observe and advise during the excavation for foundations for the new retaining walls;
7. Observe, test and advise during preparation of driveway subgrade & baserock;
8. Observe and advise during the installation of all sub- drains and surface drains.
9. Perform a final walk-through near the end of construction with the Architect, the Builder and the Owner.

LIMITATIONS

This report has been prepared in accordance with the accepted standards of the engineering profession. Our recommendations have been based upon our findings at the site and on our understanding of the proposed work. Should the proposed work change, or should unexpected conditions come to light during construction, then it may be necessary to alter or add to our recommendations.

Our recommendations should not be used for any other site, or for any construction on this site not specifically described in this report.

Our recommendations have been made with the assumption that JF Consulting, Inc. will be retained to provide the required inspections. Should this not be the case, then JF Consulting, Inc. cannot be held responsible for the recommendations given in this report or for the resulting construction. Another Soils Engineer can be retained to provide these services, provided that they accept this report as their own recommendations and they then become entirely responsible for the work performed.

At any time during the construction process, the owner may terminate our services. At this time, we will place our files in order for storage and the owner will be responsible for all our time to date. Should at any time there be a disagreement between the owner/builder and JF Consulting, Inc., we retain the right to cease our services and to notify the appropriate jurisdictions that we are no longer associated with the project. Should any reader of this report be unclear on our meaning, our office should be contacted for clarification.

This report was prepared by:
JF CONSULTING, INC.

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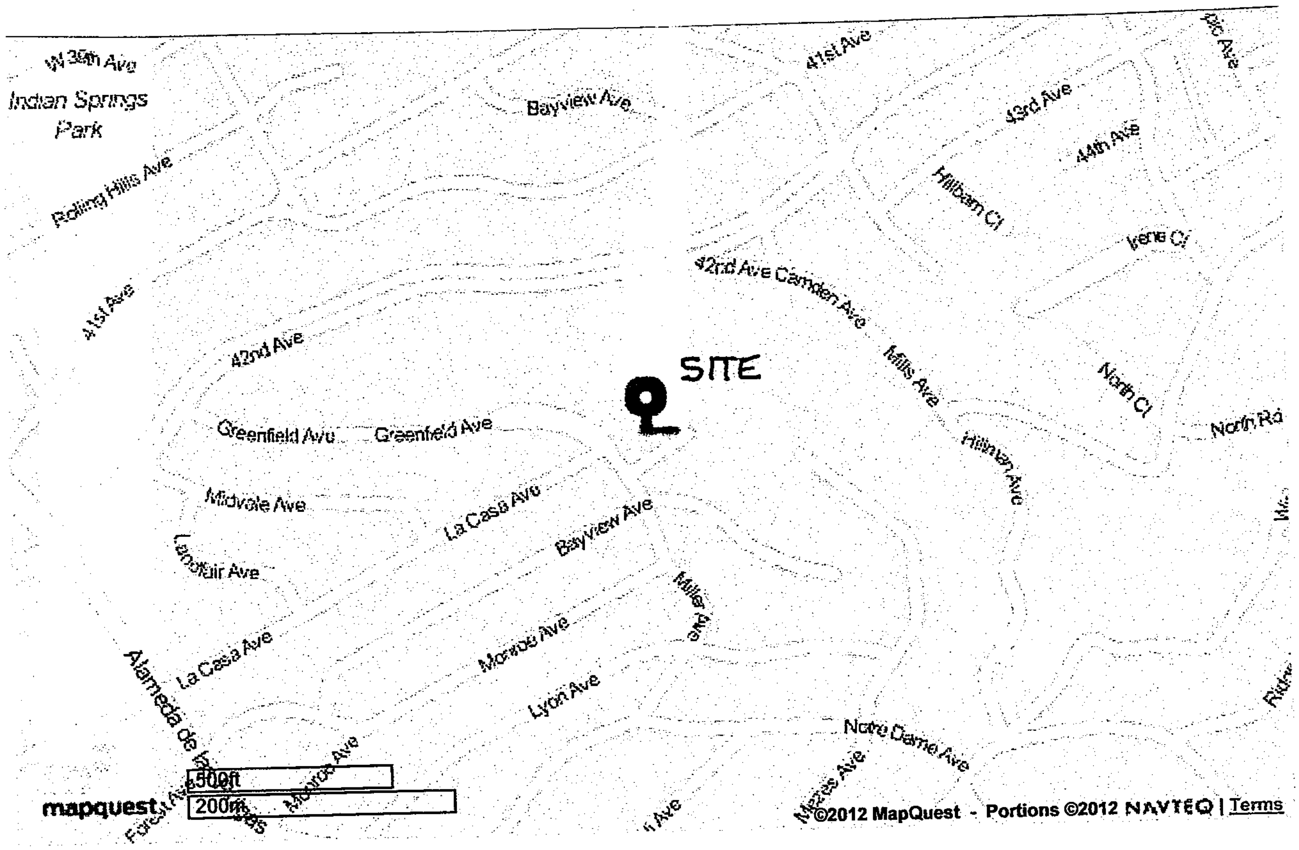
FIGURE 1



LOCATION MAP
LANDS OF BOSWORTH
3/7 GREENFIELD COURT
SAN MATEO, CALIFORNIA
Scale: as shown

MAP SOURCE: "MAPQUEST, 2012"

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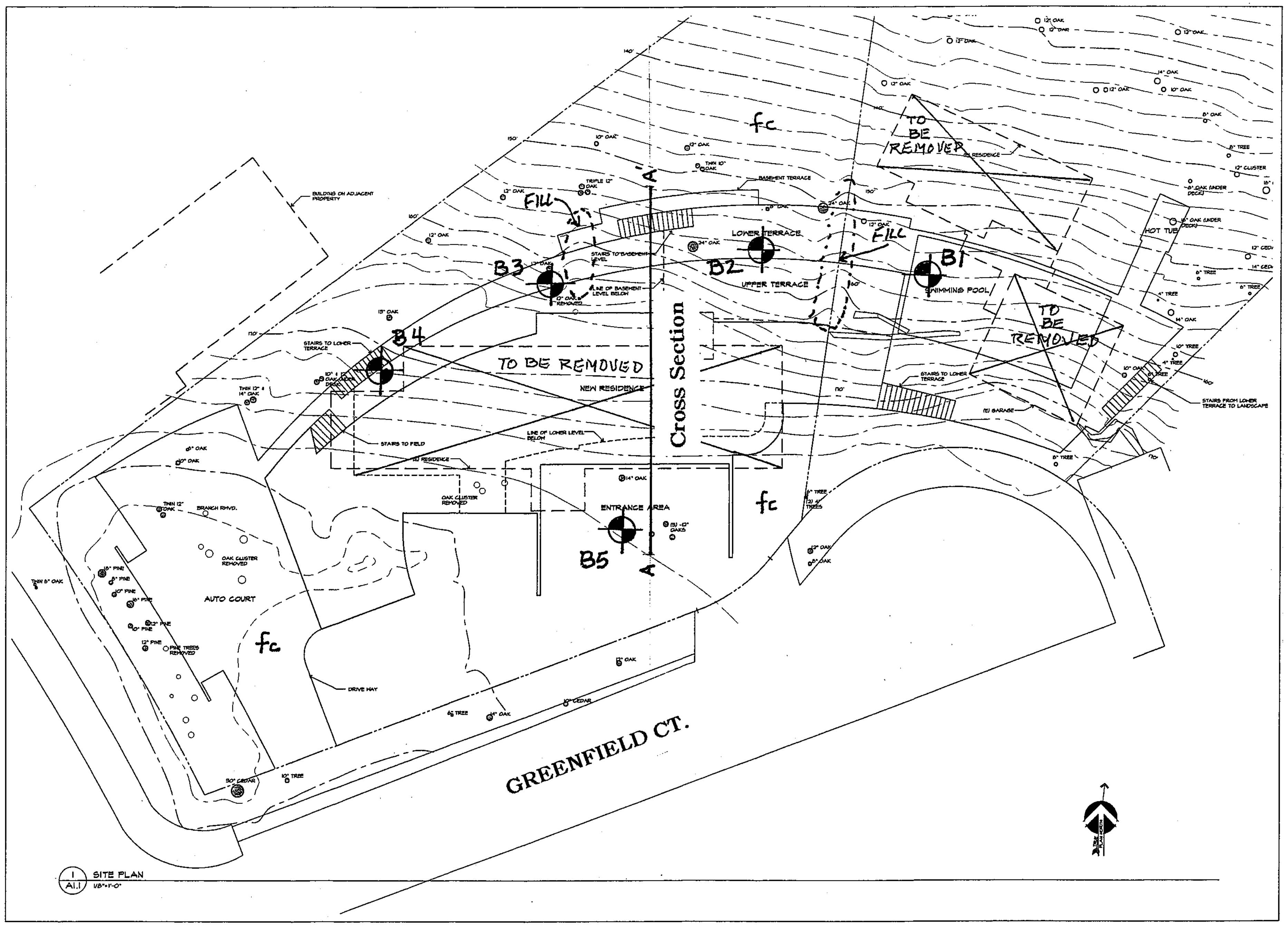
PROJECT 1492
AUGUST, 2012

FIGURE 2



KEY

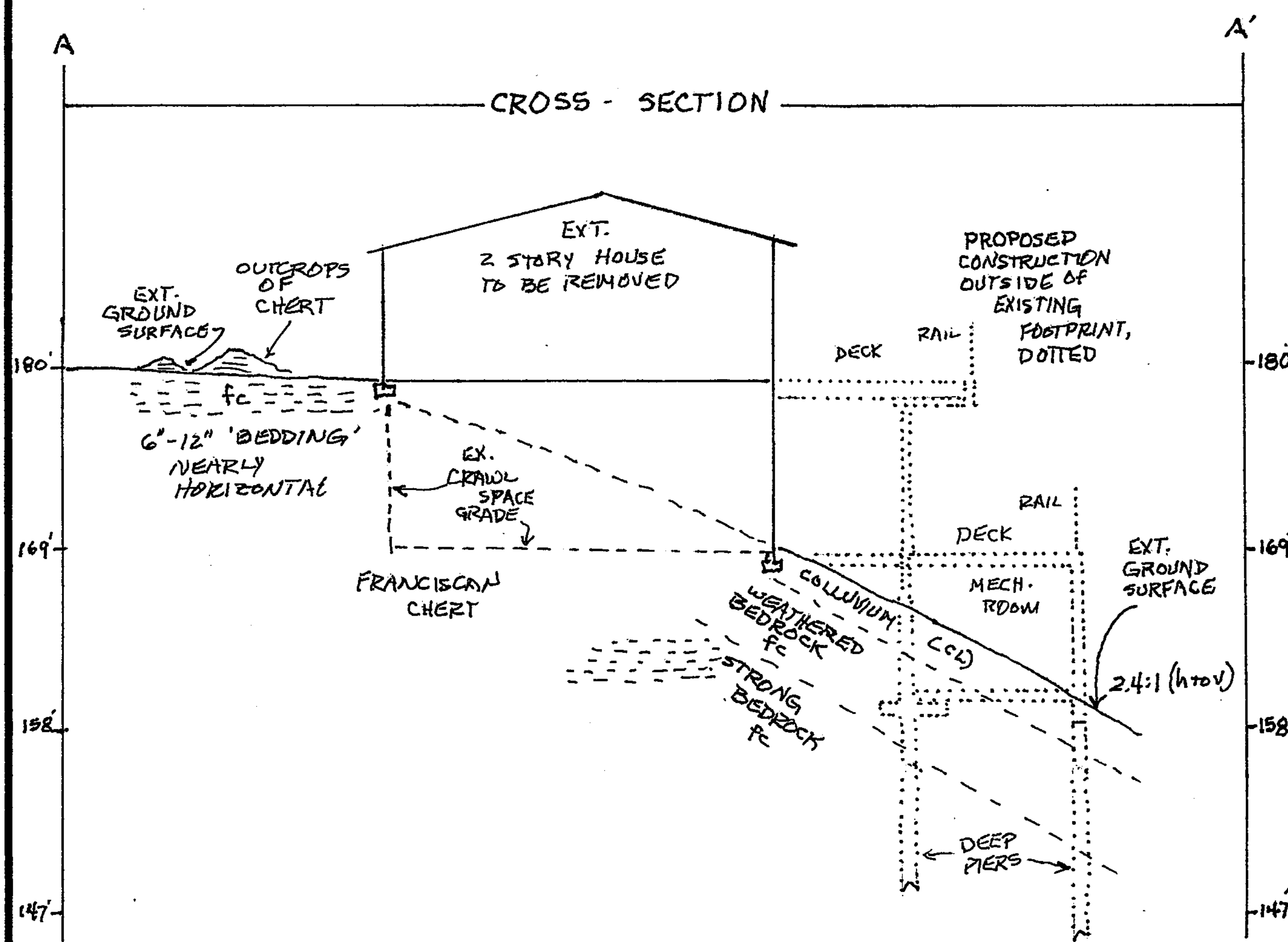
LOCATION OF BORING

fc FRANCISCAN CHERT

MAP SOURCE: 'SITE PLAN 3/7 GREENFIELD COURT BOSWORTH APN 042-251-260 by OCULUS ARCHITECTS 6/18/12

SITE PLAN & GEOLOGIC MAP		
SCALE: 1" ≅ 27'	APPROVED BY:	DRAWN BY JEF
DATE: AUG. 2012		REVISED
LANDS OF BOSWORTH 3 & 7 GREENFIELD COURT SAN MATEO, CA		
JF CONSULTING, INC.		DRAWING NUMBER PROJECT 1492

FIGURE 3



CROSS-SECTION
BOSWORTH RESIDENCE
3/7 GREENFIELD COURT
SAN MATEO, CALIFORNIA
Scale: 1" = 11 feet (h & v)

MAP SOURCE: "Proposed Section at Living Room - A New Residence at: 3/7 Greenfield Court, San Mateo, California - Bosworth - A.P.N. 042-251-260" by Oculus Architects, June 15, 2012.

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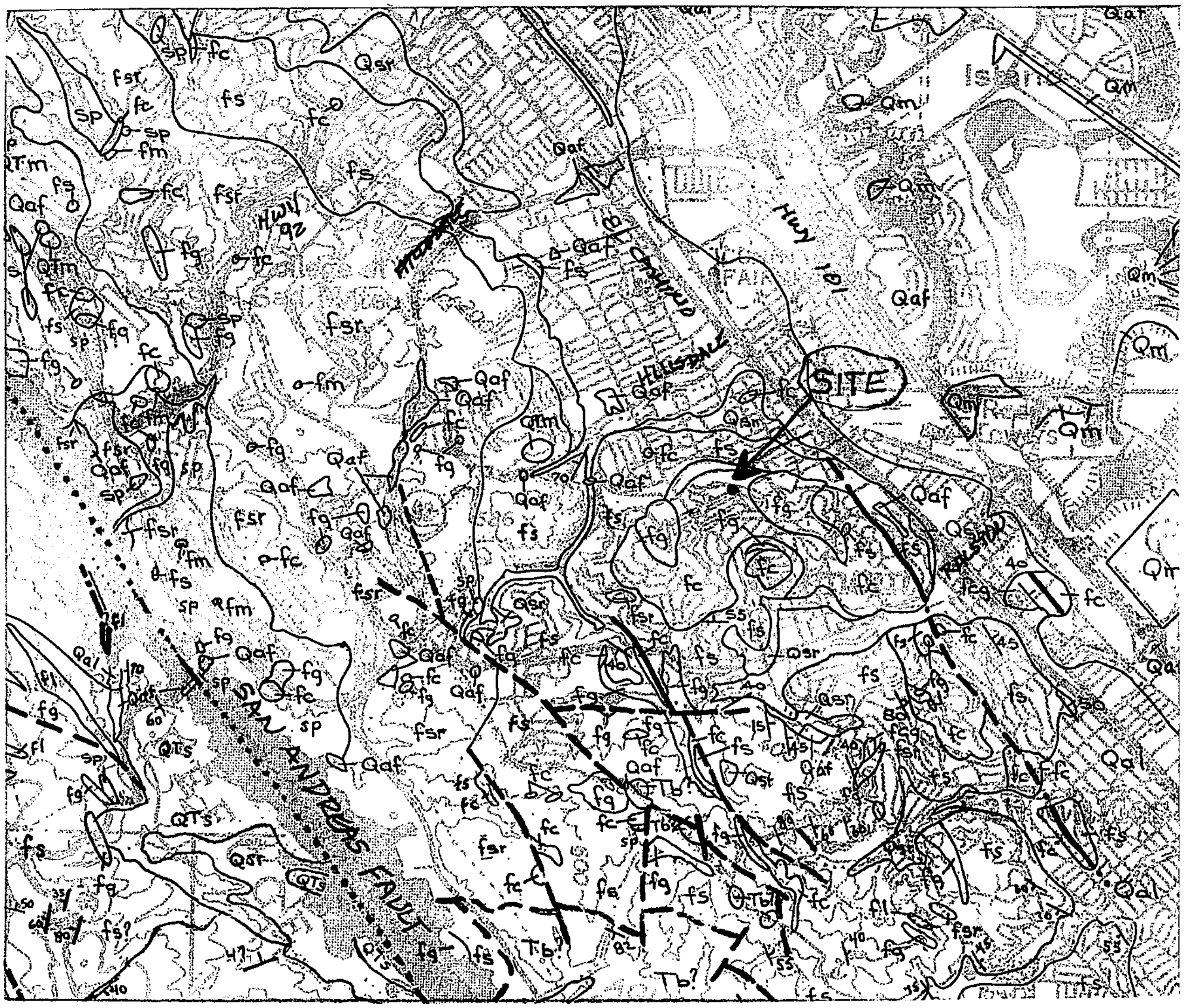
FIGURE 4



GEOLOGIC MAP
LANDS OF BOSWORTH
3/7 GREENFIELD COURT
SAN MATEO, CALIFORNIA
Scale: 1" = 1 mile

MAP SOURCE: "Preliminary Geologic Map of San Mateo County, California", USGS Basic Data Contribution 41, by Earl E. Brabb and Earl H. Pampeyan, 1972.

KEY

Qsr	Slope wash/ravine fill
fs	Franciscan Sandstone
fg	Franciscan Greenstone
fc	Franciscan Chert

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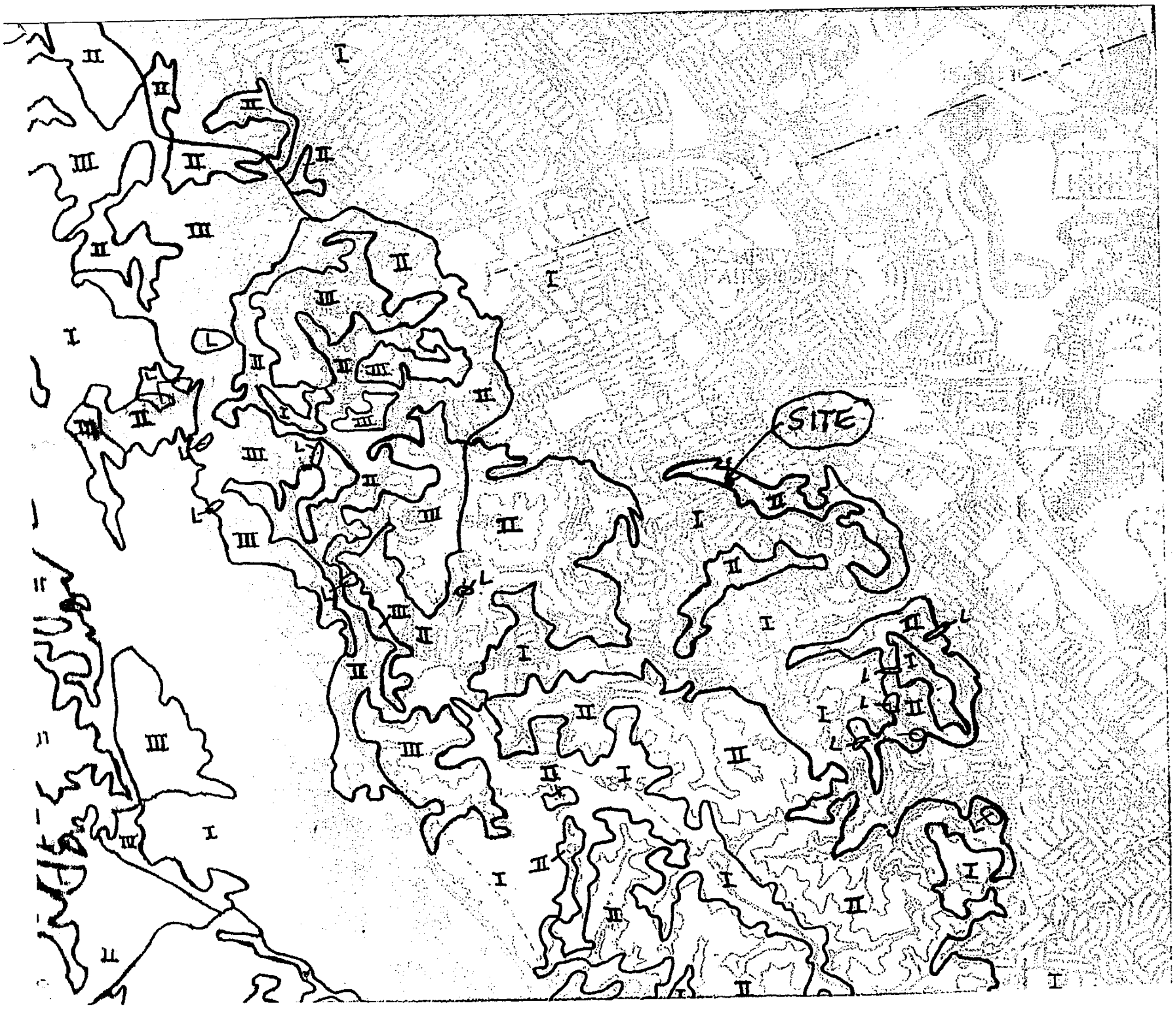
FIGURE 5



LANDSLIDE SUSCEPTIBILITY MAP
LANDS OF BOSWORTH
3/7 GREENFIELD COURT
SAN MATEO, CALIFORNIA
Scale: 1" = 1 mile

MAP SOURCE: "Landslide Susceptibility in San Mateo County, California" USGS Basic Data Contribution 43, by Earl E. Brabb Earl H. Pampeyan and Manual G. Bonilla, 1972.

KEY

I	0-1% of the rock unit has failed
II	2-8% of the rock unit has failed
III	9-25% of the rock unit has failed
IV	26-42% of the rock unit has failed
V	43-53% of the rock unit has failed
VI	54-70% of the rock unit has failed
L	100% of the rock unit has failed (Landslide Deposits)

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FIGURE 6

NOTES: This map was made by reducing a portion of "Faults that are Historically Active or that show Evidence of Geologically Young Surface Displacement, San Francisco Bay Region, A Progress Report: Oct 1970", by Brown, Robert D. Jr., USGS Open File Map

JF CONSULTING, INC. has shown all faults as solid lines, and has added many other faults not shown on the original map.

This map should not be used to determine whether or not a given property lies on a fault line. Its only purpose is to give the reader of this report a feeling for the number of faults and their distance from the site.

Not all known faults are shown on the map. Not all faults shown on the map are known to be active, and some potentially active faults may not be shown.

Where fault lines pass under alluvium (valley fill) their location may be estimated, even though the fault is shown as a solid line.

This map was prepared in December 1993, REVISED 2004.

The United States Geologic Survey cannot be held responsible for any information on this map, or for any individuals interpretation of this information.

The approximate location of the site is shown by a black dot, indicated by a black arrow.

GENERALIZED FAULT MAP OF THE BAY AREA

Scale 1" = 8 Miles

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PROJECT NO. 1492

DATE: AUGUST, 2012

3/7 GREENFIELD CT.

KEY TO EXPLORATORY BORING LOGS

SOIL CLASSIFICATIONS

PRIMARY DIVISIONS			GROUP1 SYMBOL	SECONDARY DIVISIONS
COARSE GRAINED SOILS More than half of material is larger than No. 200 sieve size	GRAVELS More than half coarse fraction is larger than No.4 sieve	Clean Gravels (less than5% fines*)	GW	Well graded gravels, gravel-sand mixtures, little or no fines
			GP	Poorly graded gravels, gravel-sand mixtures, little or no fines
		Gravel with fines*	GM	Silty gravels, gravel-sand-silt mixtures, non-plastio fines
			GC	Clayey gravels, gravel-sand-clay mixtures, plastio fines
	SANDS More than half 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, silt-sand mixtures, non-plastio fines
			SC	Clayey sand, sand-clay mixtures, plastio fines
FINE GRAINED SOILS More than half of material is smaller than No. 200 sieve size	SILTS AND CLAYS Liquid limit is less than 35		ML	Inorganic silts, clayey silts, rock flour, silty very fine sands
			CL	Inorganic clays of low plasticity, gravelly clay of low plasticity
			OL	Organic silts and organic silty clays of low plasticity
	SILTS AND CLAYS Liquid limit is between35 and 50		MI	Inorganic silts, clayey silts and silty fine sand with intermediate plasticity
			CI	Inorganic clays, gravely clays, sandy clays and silty clays of intermediate plasticity
			OI	Inorganic clays and silty clays of intermediate plasticity
	SILTS AND CLAYS Liquid limit is greater than 50		MH	Inorganic silts, clayey silts, elastic silts, micaceous or diatomaceous silty or fine sandy soil
			CH	Inorganic clays of high plasticity
			OH	Organic clays and silts of high plasticity
HIGHLY ORGANIC SOILS			Pt	Peat, meadow mat, highly organic soils

GRAIN SIZES

U.S. STANDARD SERIES SIEVE: 200, 40, 10, 4
CLEAR SQUARE SIEVE OPENINGS: ¾", 3", 12"

Silts and Clays	SAND			GRAVEL		Cobbles	Boulders
	Fine	Medium	Coarse	Fine	Coarse		
Boundaries	200	40	10	4	¾"	3"	12"

RELATIVE DENSITY

SANDS, GRAVELS AND NON-PLASTIC SILTS	BLOWS/FOOT*
VERY LOOSE	0 – 4
LOOSE	4 – 10
MEDIUM DENSE	10 – 30
DENSE	30 – 50
VERY DENSE	OVER 50

CONSISTENCY

CLAYS AND PLASTIC SILTS	UNCONFINED SHEAR STRENGTH (PSF)	BLOWS/FOOT*
VERY SOFT	0 – 250	0 – 2
SOFT	250-500	2 – 4
FIRM	500-1000	4 – 8
STIFF	1000-2000	8 – 16
VERY STIFF	2 000– 4000	16 – 32
HARD	>4000	OVER 32

SYMBOLS

▽	Initial Ground Water Level
▼	Final Ground Water Level
*	Standard Penetration Sampler
X	Modified California Sampler
D	Dames & Moore Sampler

NOTES

*BLOWS per FOOT – Resistance to advance the soil sampler in number of blows of a 140-pound hammer falling 30 inches to drive a split spoon sampler.

Stratification lines on the logs represent the approximate boundary between soil types, and the transition may be gradual.

Modified California Sampler – 2 ½ O.D. (1 7/8 Inch I.D.) sampler

Standard Penetration Sampler – 2 inch O.D. (1 3/8 Inch I.D.) split spoon sampler (ASTM D1586).

Dames & Moore Sampler – 3 inch O.D. (2.5 inch I.D.) sampler

BORING LOG

No. 1

PROJECT 3/7 Greenfield Court | DATE May 7,'12 | LOGGED BY JEF

DRILL RIG Minute Man | HOLE DIA. 3" | SAMPLER X - Modified California; * - S.P.T

GROUND WATER DEPTH INITIAL na | FINAL na | HOLE ELEVATION 158'+/-

DESCRIPTION	SOIL TYPE	DEPTH	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	TORVANE (tsf)	LIQUID LIMIT (%)	WATER CONTENT (%)	PLASTIC LIMIT (%)	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
Dark Brown, dry, firm, very sandy CLAY -	CL/											
clayey SAND	SC	1		8								
(Colluvium)			X					6.8		92		
		2										
				12								
Reddish-Tan, dry, stiff, highly weathered	fc	3	X					3.6		102		
Franciscan Chert												
(Bedrock)		4		18								
			X					3.3		106		
		5										
Reddish-Tan, dry, stiff, less weathered	fc	6										
Franciscan Chert												
		7										
harder to advance on stronger bedrock		8		36								
			X					5.6		118		
		9										
REFUSAL			*				SPT sampler 'bouncing'					
BOTTOM OF BORING, NO WATER		10										
		11										
		12										
		13										
		14										
		15										
		16										
		17										
		18										
		19										
		20										

BORING LOG

No. 2

PROJECT 3/7 Greenfield Court DATE May 7,'12 LOGGED BY JEF

DRILL RIG Minute Man HOLE DIA. 3" SAMPLER X - Modified California; * - S.P.T

GROUND WATER DEPTH INITIAL na FINAL na HOLE ELEVATION 156'+/-

DESCRIPTION	SOIL TYPE	DEPTH	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	TORVANE (tsf)	LIQUID LIMIT (%)	WATER CONTENT (%)	PLASTIC LIMIT (%)	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
Brown, damp, firm-stiff, very sandy CLAY	CL											
(Colluvium)		1		7								
			X					7.4		99		
		2										
				14								
Reddish-Brown, dry, stiff, highly weathered	fc	3	X					3.6		107		
Franciscan Chert												
		4										
becomes less weathered				44								
and harder to drill		5	X					6.2		111		
(Bedrock)		6										
				36								
		7	X					3.1		117		
REFUSAL		8	*	30/0"				SPT 'bouncing'				
BOTTOM OF BORING, NO WATER												
		9										
		10										
		11										
		12										
		13										
		14										
		15										
		16										
		17										
		18										
		19										
		20										

BORING LOG

No. 3

PROJECT 3/7 Greenfield Court | DATE May 7,'12 | LOGGED BY JEF

DRILL RIG Minute Man | HOLE DIA. 3" | SAMPLER X - Modified California; * - S.P.T

GROUND WATER DEPTH INITIAL na | FINAL na | HOLE ELEVATION 166'+/-

DESCRIPTION	SOIL TYPE	DEPTH	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	TORVANE (tsf)	LIQUID LIMIT (%)	WATER CONTENT (%)	PLASTIC LIMIT (%)	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
Brown, moist, stiff, very sandy CLAY/clayey SAND (Colluvium)	CL/ SC	1		14								
			X					4.7		102		
		2										
Reddish-Brown,damp, stiff, highly weathered Franciscan Chert	fc			15								
		3	X					4.3		101		
		4		33								
becomes less weathered			X					2		132		
and harder to drill		5										
		6										
(BEDROCK)												
		7										
		8	*									
			*	50+								
REFUSAL		9	*					3.3				
BOTTOM OF BORING, NO WATER												
		10										
		11										
		12										
		13										
		14										
		15										
		16										
		17										
		18										
		19										
		20										

BORING LOG

No. 4

PROJECT 3/7 Greenfield Court | DATE May 7,'12 | LOGGED BY JEF

DRILL RIG Minute Man | HOLE DIA. 3" | SAMPLER X - Modified California; * - S.P.T

GROUND WATER DEPTH INITIAL na | FINAL na | HOLE ELEVATION 171'+/-

DESCRIPTION	SOIL TYPE	DEPTH	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	TORVANE (tsf)	LIQUID LIMIT (%)	WATER CONTENT (%)	PLASTIC LIMIT (%)	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
Brown, moist, stiff, very sandy,silty CLAY	CL											
(Colluvium)		1		12								
			X					16.3		88		
		2										
				12								
		3	X					3		89		
Reddish-Brown, dry, highly weathered Franciscan Chert	fc											
		4										
		5		40								
becomes less weathered and harder to drill			X					3.6		121		
		6										
		7										
			*									
REFUSAL		8	*	30/0"								
BOTTOM OF BORING, NO WATER												
		9										
		10										
		11										
		12										
		13										
		14										
		15										
		16										
		17										
		18										
		19										
		20										

BORING LOG

No. 5

PROJECT 3/7 Greenfield Court | DATE May 7,'12 | LOGGED BY JEF

DRILL RIG Minute Man | HOLE DIA. 3" | SAMPLER X - Modified California; * - S.P.T

GROUND WATER DEPTH INITIAL na | FINAL na | HOLE ELEVATION 178'+/-

DESCRIPTION	SOIL TYPE	DEPTH	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	TORVANE (tsf)	LIQUID LIMIT (%)	WATER CONTENT (%)	PLASTIC LIMIT (%)	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
Reddish-Brown, hard, fresh Franciscan Chert Bedrock REFUSAL	fc				Could not advance drill							
		1										
		2										
		3										
		4										
		5										
		6										
		7										
		8										
		9										
		10										
		11										
		12										
		13										
		14										
		15										
		16										
		17										
		18										
		19										
		20										

Moisture / Density

Project Name 3/7 Greenfield Court
Project Number 1492
Date May 10,'12
Tested By JEF

Sample	Depth	Cup #	Cup Tare	Sample Height	Wet Wt. with Tare	Dry Wt. with Tare	Weight of Water	Dry Weight. - Tare	% Water Content	Dry Density	Description of Sample
	feet		grams	inches	grams	grams	grams	grams		lbs per cu. ft	
DH-1	1.0	36	32.8	3.6	298.0	281.0	17.0	248.2	6.8%	91.7	
	2.5	35	32.2	3.6	319.0	309.0	10.0	276.8	3.6%	102.2	
	4.0	5	37.0	2.3	227.0	221.0	6.0	184.0	3.3%	106.4	
	8.0	32	32.4	2.0	220.0	210.0	10.0	177.6	5.6%	118.0	
DH-2	1.0	3	31.4	4.0	351.0	329.0	22.0	297.6	7.4%	98.9	
	2.5	44	31.1	3.8	347.0	336.0	11.0	304.9	3.6%	106.67	
	4.5	31	31.0	2.7	270.0	256.0	14.0	225.0	6.2%	110.78	
	6.5	17	37.4	2.6	273.0	266.0	7.0	228.6	3.1%	116.88	
DH-3	1.0	15	37.7	3.6	327.0	314.0	13.0	276.3	4.7%	102.03	
	2.5	8	37.4	3.4	306.0	295.0	11.0	257.6	4.3%	100.72	
	4.0	45	32.3	4.0	437.0	429.0	8.0	396.7	2.0%	131.84	
DH-4	1.0	22	38.0	4.0	345.0	302.0	43.0	264.0	16.3%	87.74	
	2.5	36	32.8	3.5	273.0	266.0	7.0	233.2	3.0%	88.58	
	5.0	21	38.0	3.7	387.0	375.0	12.0	337.0	3.6%	121.08	