

APPENDIX D
GEO TECHNICAL INVESTIGATION

**REPORT
GEOTECHNICAL INVESTIGATION
Planned Residential Development At
556 El Camino Real
Burlingame, California**

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April 14, 2013

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INTRODUCTION

Purpose

A geotechnical investigation has been completed for the proposed residential development at 556 El Camino Real in Burlingame, California. The purposes of this study have been to gather information on the nature, distribution, and characteristics of the earth materials at the site, assess geologic hazards, and to provide geotechnical design criteria for the planned improvements.

Scope

The scope of our services is outlined in our Proposal and Professional Service Agreement dated April 12, 2012. Our investigation included a reconnaissance of the site and surrounding vicinity; sampling and logging one test boring to a depth of 51-½ feet below the ground surface; laboratory testing conducted on selected samples of the earth materials recovered from the boring; a review of published geotechnical and geologic data pertinent to the project area; geotechnical interpretation and engineering analyses; and preparation of this report.

This report contains the results of our investigation, including findings regarding site, soil, geologic, and groundwater conditions; conclusions pertaining to geotechnical considerations such as weak soils, settlement, and construction considerations; conclusions regarding exposure to geologic hazards, including faulting, ground shaking, liquefaction, lateral spreading, and slope stability; and geotechnical recommendations for design of the proposed project including site preparation and grading, foundations, retaining walls, slabs on grade, and geotechnical drainage.

Pertinent exhibits appear in Appendix A. The location of the test boring is depicted relative to site features on Plate 1, Boring Location Map. The log of the test boring is displayed on Plate 2. Explanations of the symbols and other codes used on the log are presented on Plate 3, Soil Classification Chart and Key to Test Data.

References consulted during the course of this investigation are listed in Appendix B. Details regarding the field exploration program appear in Appendix C.

Proposed Development

It is our understanding that the project will consist of the design and construction of a new 4-story, 22-unit residential development with 1 full basement level. We have reviewed the preliminary plans by Triad/Holmes Associates. No other project details are known at this time.

FINDINGS

Site Description

The project site is located northeast of El Camino Real, between Floribunda Avenue and Bellevue Avenue, in Burlingame, California. The topography in the vicinity of the site slopes downward toward the northeast at an average inclination of about 80:1 (horizontal:vertical). At the time of our investigation, the subject site was occupied by a residential complex with appurtenant flatwork and yard areas.

Geologic Conditions

The site is within the Coast Ranges Geomorphic Province, which includes the San Francisco Bay and the northwest-trending mountains that parallel the coast of California. Tectonic forces resulting in extensive folding and faulting of the area formed these features. The oldest rocks in the area include sedimentary, volcanic, and metamorphic rocks of the Franciscan Complex, and sandstone, shale, and conglomerate of the Great Valley Sequence. These units are Jurassic to Cretaceous in age and form the basement rocks in the region.

The area has been mapped by Pampeyan (1994) as being underlain by older alluvium.

Earth Materials

The boring drilled for this investigation encountered alluvial deposits consisting predominately of sand-clay mixtures with varying amounts of gravel. Detailed descriptions of the materials encountered as well as test results are presented on the Boring Log, Plate 2.

Groundwater

Free groundwater was encountered in the boring drilled at the subject site at a depth of about 5 feet below the ground surface. We anticipate that this represents surface water infiltration that has become perched on a clayey soil lense, and not the phreatic groundwater surface. We anticipate that the depth to groundwater will vary depending on precipitation, and possible other factors. Seepage may be encountered into excavations near the ground surface following rainfall or irrigation in the vicinity of the subject site.

CONCLUSIONS

General

It is our opinion that the site is suitable for support of the planned improvements provided the recommendations contained in this report are followed during the design and construction of the project. The primary geotechnical concerns at the site are support of temporary slopes and adjacent improvements, groundwater considerations, supporting improvements in competent earth materials, and seismic shaking and related effects during earthquakes. These items are discussed below.

Temporary Slopes and Undermining of Existing Structures

Temporary slopes will be necessary during the planned site excavations. In order to safely develop the site, temporary slopes will need to be laid back in conformance with OSHA standards at safe inclinations, or temporary shoring will have to be installed. The contractor may choose to excavate test pits to evaluate site soils and the need for temporary shoring.

If excavations undermine or remove support from the existing or adjacent structures, it may be necessary to underpin those structures. Care should be taken to provide adequate shoring or underpinning to support the affected improvements as a result of the loss of support.

Temporary slopes and support of structures during construction are the responsibility of the contractor. Earth Mechanics Consulting Engineers is available to provide geotechnical consultation regarding stability of excavations and support of improvements.

Groundwater Considerations

Free groundwater was encountered in the boring drilled at the subject site at a depth of about 5 feet below the ground surface. We anticipate that this represents surface water infiltration that has become perched on a clayey soil lense, and not the phreatic groundwater surface. A piezometer may be installed at the site and monitored to determine if the phreatic groundwater table will be within the depth of planned excavations at the subject site. If the free groundwater table is within the depth of excavations, dewatering and water tight construction may be required for the proposed residential development. In addition, if the planned basement extends below the free groundwater table, we should be consulted to provide revised lateral earth pressures on subsurface walls due to hydrostatic water pressure and buoyant earth pressures.

Foundation Support

It is our opinion that the planned improvements may be supported on a conventional spread footing foundation bearing in competent earth materials. If the spread footings would cover a substantial portion of the building area, a mat foundation may be used as an alternative to reduce forming and steel bending costs. The Structural Engineer may also choose to use drilled piers to support improvements, or for shoring and underpinning, if required. Detailed foundation design criteria are presented later in this report.

We estimate that improvements supported on foundations designed and constructed in accordance with our recommendations will experience post-construction total settlements from static loading of less than 1 inch with differential settlements of less than ½ inch over a 50-foot span.

Geologic Hazards

Faulting

The property does not lie within an Alquist-Priolo Earthquake Fault Zone. The closest mapped active fault in the vicinity of the site is the San Andreas Fault located about 6 miles to the southwest. No active faults are shown crossing the site on reviewed published maps, nor did we observe evidence of faulting during our reconnaissance. Therefore we conclude that the potential for damage to the improvements from faulting is low.

Earthquake Shaking

Earthquake shaking results from the sudden release of seismic energy during displacement along a fault. During an earthquake, the intensity of ground shaking at a particular location will depend on a number of factors including the earthquake magnitude, the distance to the zone of energy release, and local geologic conditions. We judge that the site will be exposed to strong earthquake shaking during the life of the improvements. The recommendations contained in the applicable Building Code should be followed for reducing potential damage to the improvements from earthquake shaking.

Liquefaction

Liquefaction results in a loss of shear strength and potential volume reduction in saturated, loose, granular soils below the groundwater level from earthquake shaking. The occurrence of this phenomenon is dependent on many factors, including the intensity and duration of ground shaking, soil density and particle size distribution, and position of the groundwater table (Seed and Idriss, 1982).

The soil encountered in our boring consisted of cohesive clay-sand mixtures that generally had a relatively low potential for liquefaction. However, given the alluvial nature of the soil deposits at the site, there is a potential that liquefiable soils could exist. It is our opinion that if potentially liquefiable deposits exist beneath the subject site, they are contained in discrete pockets of limited vertical and lateral extent. We judge that the impact of potentially liquefiable deposits at the subject site would be limited to post-liquefaction settlements of less than 1 inch. Therefore, it is our opinion that there is a relatively low potential for damage to the planned improvements from liquefaction.

Lateral Spreading

Lateral spreading or lurching is generally caused by liquefaction of marginally stable soils underlying gentle slopes. In these cases, the surficial soils move toward an unsupported face, such as an incised channel, river, or body of water. We judge that if potentially liquefiable deposits exist beneath the subject site, they would be contained in isolated pockets of limited lateral and vertical extent, which would not be conducive to lateral spreading. Therefore, we judge that there is a low risk for damage of the improvements from seismically-induced lateral spreading.

Densification

Densification can occur in clean, loose granular soils during earthquake shaking, resulting in seismic settlement and differential compaction. It is our opinion that earth materials subject to seismic densification do not exist beneath the site in sufficient thickness to adversely impact the planned improvements.

Landsliding

The subject site is relatively level. The geologic maps of the site vicinity reviewed for this study did not indicate the presence of landslides at the site or its immediate vicinity. During our site reconnaissance, we did not observe evidence of active slope instability at the site or its immediate vicinity. Therefore, it is our opinion that the potential for landsliding at the site under static or seismic loading is low provided the recommendations in this report are incorporated into the plans and specifications for the planned improvements.

RECOMMENDATIONS

Site Preparation and Grading

General

We assume that the planned improvements will be constructed at or below existing site grades. If site grades are raised by filling more than about 1 foot, we should be retained to calculate the impact of filling on slope stability, site settlements, and foundations.

Clearing

Following removal of existing improvements to be demolished, areas to be graded should be cleared of debris, deleterious materials, and vegetation, and then stripped of the upper soils containing root growth and organic matter. We anticipate that the required depth of stripping will generally be less than 2 inches. Deeper stripping may be required to remove localized concentrations of organic matter, such as tree roots. The cleared materials should be removed from the site; strippings may be stockpiled for reuse as topsoil in landscaping areas or should be hauled off site.

Overexcavation

Loose, porous soils and topsoil, if encountered, should be overexcavated in areas designated for placement of future engineered fill or support of improvements. Difficulty in achieving the recommended minimum degree of compaction described below should be used as a field criterion by the geotechnical engineer to identify areas of weak soils that should be removed and replaced as engineered fill. The depth and extent of excavation should be approved in the field by the geotechnical engineer prior to placement of fill or improvements.

Subgrade Preparation

Exposed soils designated to receive engineered fill should be cut to form a level bench, scarified to a minimum depth of 6 inches, brought to at least optimum moisture content, and compacted to at least 90 percent relative compaction, in accordance with ASTM test designation D 1557.

Material for Fill

It is anticipated that the on-site soil will be suitable for reuse as fill provided that lumps greater than 6 inches in largest dimension and perishable materials are removed, and that the fill materials are approved by the geotechnical engineer prior to use.

Fill materials brought onto the site should be free of vegetative mater and deleterious debris, and should be primarily granular. The geotechnical engineer should approve fill material prior to trucking it to the site.

Compaction of Fill

Fill should be placed in level lifts not exceeding 8 inches in loose thickness. Each lift should be brought to at least the optimum moisture content and compacted to at least 90 percent relative compaction, in accordance with ASTM test designation D 1557.

Underpinning

During excavations adjacent to improvements, care should be taken to adequately support the existing improvements. When excavating below the level of foundations supporting existing structures, some form of underpinning may be required where excavations extend below an imaginary plane sloping at 1:1 downward and outward from the edge of the existing footings. All temporary underpinning design and construction are the responsibility of the contractor. Earth Mechanics is available to provide consultation regarding underpinning adjacent improvements.

Temporary Slopes

Temporary slopes will be necessary during the planned site excavations. In order to safely develop the site, temporary slopes will need to be laid back in conformance with OSHA standards at safe inclinations, or temporary shoring will have to be installed. All temporary slopes and shoring design are the responsibility of the contractor. Earth Mechanics is available to provide consultation regarding stability and support of temporary slopes during construction.

Finished Slopes

In general, finished cut and fill slopes should be constructed at an inclination not exceeding 2:1 (horizontal:vertical). Routine maintenance of slopes should be anticipated. The tops of cut slopes should be rounded and compacted to reduce the risk of erosion. Fill and cut slopes should be planted with vegetation to resist erosion, or protected from erosion by other measures, upon completion of grading. Surface water runoff should be intercepted and diverted away from the tops and toes of cut and fill slopes by using berms or ditches.

Seismic Design

The following seismic design parameters apply:

Site Class C

$S_s = 1.988$, $S_1 = 1.057$

$F_a = 1.0$, $F_v = 1.3$

$SM_s = 1.988$, $SM_1 = 1.374$

$SD_s = 1.325$, $SD_1 = 0.916$

Foundations

General

It is our opinion that the planned improvements may be supported on a conventional spread footing foundation bearing in competent earth materials. If the spread footings would cover a substantial portion of the building area, a mat foundation may be used as an alternative to reduce forming and steel bending costs. The Structural Engineer may also choose to use drilled piers to support improvements, or for shoring and underpinning, if required. Design criteria for each foundation type are presented below.

Spread Footings

Spread footings should be at least 12 inches wide and extend at least 18 inches below lowest adjacent exterior grade. If soft or unstable soil areas are encountered at the bottom of the footings, localized deepening of the footing excavation will be necessary. Footings should be stepped to produce level tops and bottoms and should be deepened as necessary to provide at least 7 feet of horizontal clearance between the portions of footings designed to impose passive pressures and the face of the nearest slope or retaining wall. Spread footings can be designed to impose dead plus code live load bearing pressures and total design load bearing pressures of 3,000 and 4,500 psf, respectively.

Resistance to lateral pressures can be obtained from passive earth pressures against the face of the footings and soil friction along the base of footings. We recommend that an allowable passive equivalent fluid pressure of 280 pcf and a friction factor of 0.3 times the net vertical dead load be used for design. Passive pressures should be disregarded in areas with less than 7 feet of horizontal soil confinement and for the uppermost 1-foot of foundation depth unless confined by concrete slabs or pavements.

Mat Foundation

A mat foundation may be used to support the planned improvements. The mat can be designed for an average bearing pressure over the entire mat of 3,000 psf for combined dead plus sustained live loads, and 4,500 psf for total loads including wind or seismic forces. The weight of the mat extending below current site grade may be neglected in computing bearing loads. Localized increases in bearing pressures of up to 5,000 psf may be utilized. For elastic design, a modulus of subgrade reaction of 50 kips per cubic foot may be used.

A passive equivalent fluid pressure of 280 pounds per cubic foot and a friction factor of 0.3 may be used to resist lateral forces and sliding. Passive pressures should be disregarded in areas with less than 7 feet of horizontal soil confinement and for the uppermost 1-foot of foundation depth unless confined by concrete slabs or pavements.

Drilled Piers

Drilled, cast-in-place, reinforced concrete piers may be used to support improvements, or shoring excavation walls and underpinning adjacent improvements. Piers designed to resist lateral loads from retaining walls should extend below grade a minimum of 8 times the pier diameter or twice the height of the retaining wall, whichever is less. Piers should be designed for a maximum allowable skin friction of 500 psf for combined dead plus sustained live loads. The above values may be increased by one-third for total loads, including the effect of seismic or wind forces. The weight of the foundation concrete extending below grade may be disregarded.

Resistance to lateral displacement of individual piers will be generated primarily by passive earth pressures acting against two pier diameters. Passive pressures should be assumed equivalent to those generated by a fluid weighing 280 pcf. Passive pressures should be disregarded in areas with less than 7 feet of horizontal soil confinement and for the uppermost 1-foot of foundation depth unless confined by concrete slabs or pavements.

Where groundwater is encountered during pier shaft drilling, it should be removed by pumping, or the concrete must be placed by the tremie method. If the pier shafts will not stand open, temporary casing may be necessary to support the sides of the pier shafts until concrete is placed. Concrete should not be allowed to free fall more than 5 feet to avoid segregation of the aggregate.

Retaining Walls

Retaining walls should be fully backdrained. The backdrains should consist of at least a 3-inch-diameter, rigid perforated pipe surrounded by a drainage blanket, or equivalent, such as a high profile drain. The pipe should be sloped to drain by gravity to appropriate outlets. Accessible subdrain cleanouts should be provided and maintained on a routine basis. The drainage blanket should consist of clean, free-draining crushed rock or gravel, wrapped in a filter fabric such as Mirafi 140N. Alternatively, the drainage blanket could consist of Caltrans Class 2 "Permeable Material" or a prefabricated drainage structure such as Mirafi Miradrain. The top of the drainpipe should be at least 8 inches below lowest adjacent grade. The drainage blanket should be at least 1 foot in width and extend to within 1 foot of the surface. The uppermost 1-foot should be backfilled with compacted native soil to exclude surface water.

Yielding retaining walls free to rotate at least 0.1 percent of the wall height at the top of the backfill could be subjected to active lateral earth pressures equivalent to those exerted by a fluid weighing 40 pounds per cubic foot (pcf) where the backslope is level; or 60 pcf where the backslope is inclined at 2:1 (horizontal:vertical). For intermediate slopes, interpolate between these values. In addition to lateral earth pressures, retaining walls must be designed to resist horizontal pressures that may be generated by surcharge loads applied at or near the ground surface. Where an imaginary 1:1 plane projected downward from the outermost edge of a surcharge load or foundation intersects a retaining wall, that portion of the wall below the intersection should be designed for an additional horizontal thrust from a uniform pressure equivalent to one-third the maximum anticipated surcharge load. In some cases, this value yields a conservative estimate of the actual lateral pressure imposed. We should be contacted if a more precise estimate of lateral loading on the retaining wall from surcharge pressures is desired.

Rigid retaining walls constrained against such movement could be subjected to "at-rest" lateral earth pressures equivalent to those exerted by the fluid pressures listed above plus a uniform load of $6 \bullet H$ pounds per square foot, where H is the height of the backfill above footing level. Where an imaginary 1:1 (H:V) plane projected downward from the outermost edge of a surcharge load or foundation intersects a lower retaining wall, that portion of the constrained wall below the intersection should be designed for an additional horizontal thrust from a uniform pressure equivalent to one-half the maximum anticipated surcharge load. In some cases, this value yields a conservative estimate of the actual lateral pressure imposed. We should be contacted if a more precise estimate of lateral loading on the retaining wall from surcharge pressures is desired.

A seismic pressure increment equivalent to a rectangular pressure distribution of $5H$ in psf may be used, where H is the height of the soil retained in feet.

Wall backfill should consist of soil that is spread in level lifts not exceeding 8 inches in thickness. Each lift should be brought to at least the optimum moisture content and compacted to not less than 90 percent relative compaction, per ASTM test designation D 1557. Retaining walls may yield slightly during backfilling. Therefore, walls should be properly braced during the backfilling operations.

Where migration of moisture through retaining walls would be detrimental or undesirable, retaining walls should be waterproofed as specified by the project architect or structural engineer.

Retaining walls should be supported on footings designed in accordance with the recommendations presented above. A minimum factor of safety of 1.5 against overturning and sliding should be used in the design of retaining walls.

Slabs on Grade

The subgrade soil in slab and flatwork areas should be proof rolled to provide a firm, non-yielding surface. If moisture penetration through the slab would be objectionable, slabs should be underlain by a capillary moisture break consisting of at least 4 inches of clean, free-draining crushed rock or gravel graded such that 100 percent will pass the 1-inch sieve and none will pass the No. 4 sieve. Further protection against slab moisture penetration can be provided by means of a moisture vapor barrier membrane, placed between the drain rock and the slab. The membrane may be covered with 2 inches of damp, clean sand to protect it during construction.

Site Drainage

Positive drainage should be provided away from the improvements. Roof runoff should be directed toward downspouts that discharge into closed conduits that drain into the site storm drain system. Surface drainage facilities (roof downspouts and drainage inlets) should be maintained entirely separate from subsurface drains. Drains should be checked periodically, and cleaned and maintained as necessary to provide unimpeded flow.

Supplemental Services

Earth Mechanics recommend that we be retained to review the project plans and specifications to determine if they are consistent with our recommendations. In addition, we should be retained to observe geotechnical construction, particularly site excavations, placement of retaining wall backdrains, fill compaction, and excavation of foundations, as well as to perform appropriate field observations and laboratory tests.

If, during construction, subsurface conditions different from those described in this report are observed, or appear to be present beneath excavations, we should be advised at once so that these conditions may be reviewed and our recommendations reconsidered. The recommendations made in this report are contingent upon our notification and review of the changed conditions.

If more than 18 months have elapsed between the submission of this report and the start of work at the site, or if conditions have changed because of natural causes or construction operations at or adjacent to the site, the recommendations of this report may no longer be valid or appropriate. In such case, we recommend that we review this report to determine the applicability of the conclusions and recommendations considering the time elapsed or changed conditions. The recommendations made in this report are contingent upon such a review.

These services are performed on an as-requested basis and are in addition to this geotechnical investigation. We cannot accept responsibility for conditions, situations or stages of construction that we are not notified to observe.

LIMITATIONS

This report has been prepared for the exclusive use of Mr. Roman Knop and his consultants for the proposed project described in this report. Our services consist of professional opinions and conclusions developed in accordance with generally-accepted geotechnical engineering principles and practices. We provide no other warranty, either expressed or implied. Our conclusions and recommendations are based on the information provided us regarding the proposed construction, our site reconnaissance, review of published data, and professional judgment. Verification of our conclusions and recommendations is subject to our review of the project plans and specifications, and our observation of construction.

The test boring log represents subsurface conditions at the location and on the date indicated. It is not warranted that it is representative of such conditions elsewhere or at other times. Site conditions and cultural features described in the text of this report are those existing at the time of our field exploration, conducted on April 5, 2013, and may not necessarily be the same or comparable at other times.

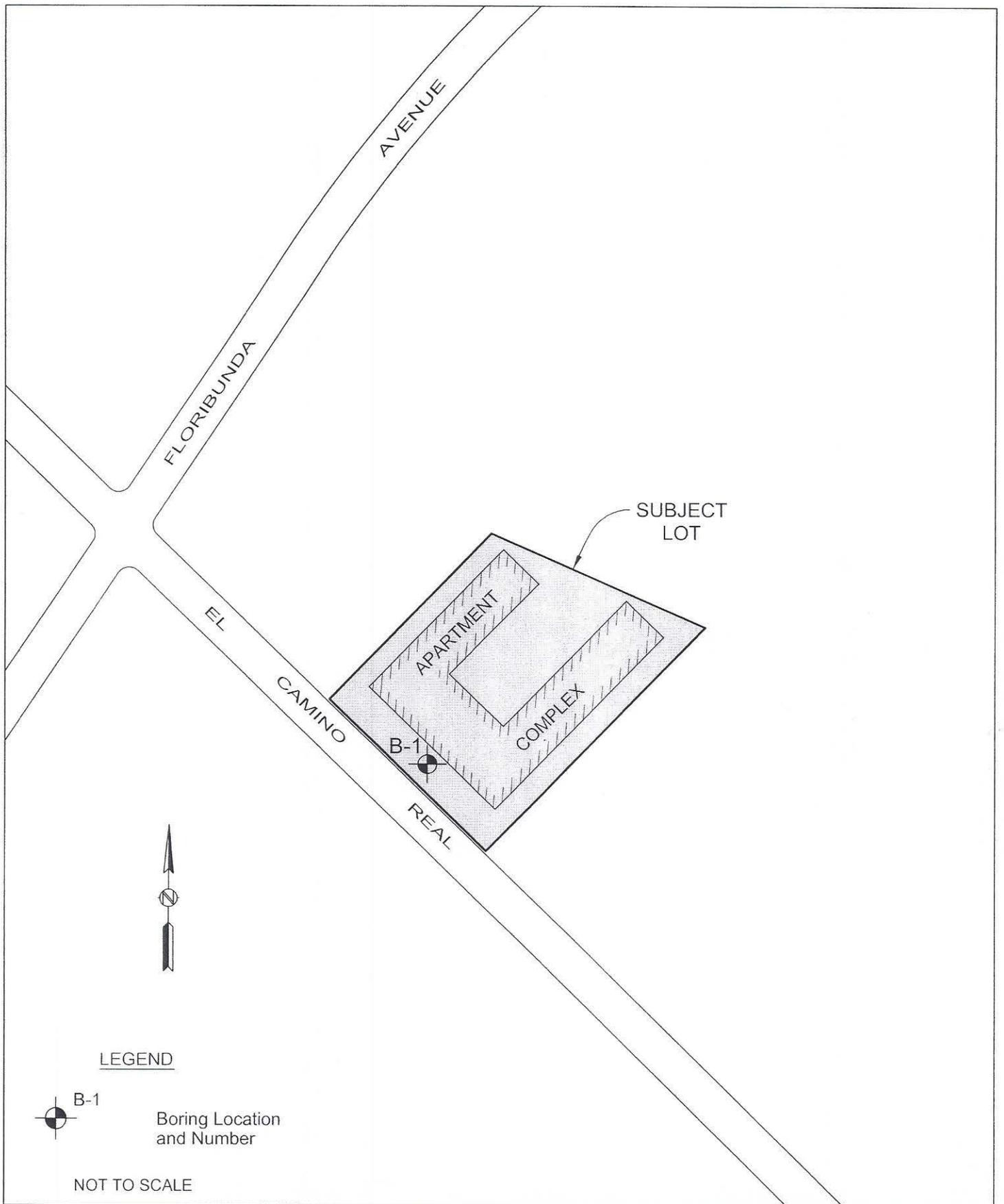
The location of the test boring was established in the field by reference to existing features and should be considered approximate only.

The scope of our services did not include an environmental assessment or an investigation of the presence or absence of hazardous, toxic, or corrosive materials in the soil, surface water, groundwater or air, on or below, or around the site, nor did it include an evaluation or investigation of the presence or absence of wetlands.

APPENDIX A

List of Plates

- | | | |
|---------|---|--|
| Plate 1 | - | Boring Location Map |
| Plate 2 | - | Log of Boring 1 |
| Plate 3 | - | Soil Classification Chart and Key to Test Data |



Earth Mechanics
 Consulting Engineers

Job. No: 12-3798
 Appr:
 Drwn: LPDD
 Date: 4/8/13

BORING LOCATION MAP
 556 El Camino Real
 Burlingame, California

PLATE
1

Other Laboratory Tests	Pocket Penetrometer (ksf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200 sieve	Blows/Foot Sample	DEPTH (FEET)	EQUIPMENT: 4" Flight Auger LOGGED BY: A.K.	ELEVATION: * START DATE: 4-5-13 FINISH DATE: 4-5-13
		20.7		59	5	0	Asphalt 3"	
						1	Dark Brown Sandy Silty Clay (CL-ML), moist, firm	
		20.6		76	13	5	Brown Silty Clay with Sand (CL-ML), saturated, stiff	
						6		
		13.4		22	29	10	Mottled Brown Clayey Sand with Gravel (SC), moist, medium dense	
						11		
		12.7		11	29	15	Mottled Brown Poorly Graded Gravel with Clay and Sand (GP-GC), moist, medium dense	
						16		
		13.0		17	28	20	Mottled Light Brown Clayey Sand with Gravel (SC), moist, medium dense	
						21		
		21.4		73	17	25	Brown Lean Clay with Sand (CL), moist, very stiff	
						26		
		15.0		30	26	30	Brown Clayey Sand (SC), moist, medium dense	
						31		
		18.1		66	25	35	Bluish Grey Sandy Silty Clay (CL-ML), moist, very stiff	
						36		
		11.9		47	71	40	Orangish Brown Clayey Sand (SC), moist, very dense	
						41		
		16.6		37	46	45	- brown with light grey, dense	
						46		
		13.0		52	45	50	Light Orangish Brown Sandy Lean Clay (CL), moist, hard	
						51		

Bottom of Boring = 51-1/2'
Water @ 5'

* Existing ground surface.

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Date: APR 2013

LOG OF BORING 1
556 El Camino Real
Burlingame, California

PLATE
2

MAJOR DIVISIONS			TYPICAL NAMES	
COARSE GRAINED SOILS More than Half > #200 sieve	GRAVELS MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS WITH LITTLE OR NO FINES	GW 	WELL GRADED GRAVELS, GRAVEL-SAND
			GP 	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES
		GRAVELS WITH OVER 12% FINES	GM 	SILTY GRAVELS, POORLY GRADED GRAVEL-SAND-SILT MIXTURES
			GC 	CLAYEY GRAVELS, POORLY GRADED GRAVEL-SAND-CLAY MIXTURES
	SANDS MORE THAN HALF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE	CLEAN SANDS WITH LITTLE OR NO FINES	SW 	WELL GRADED SANDS, GRAVELLY SANDS
			SP 	POORLY GRADED SANDS, GRAVELLY SANDS
		SANDS WITH OVER 12% FINES	SM 	SILTY SANDS, POORLY GRADED SAND-SILT MIXTURES
			SC 	CLAYEY SANDS, POORLY GRADED SAND-CLAY MIXTURES
FINE GRAINED SOILS More than Half < #200 sieve	SILTS AND CLAYS LIQUID LIMIT 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 CLAYS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY	
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50	MH 	INORGANIC SILTS, MICACEOUS OR DIATOMACIOUS 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

Consol	Consolidation	Tx	2630 (240)	Unconsolidated Undrained Triaxial
LL	Liquid Limit (in %)	Tx sat	2100 (575)	Unconsolidated Undrained Triaxial, saturated prior to test
PL	Plastic Limit (in %)	DS	3740 (960)	Unconsolidated Undrained Direct Shear
PI	Plasticity Index	TV	1320	Torvane Shear
Gs	Specific Gravity	UC	4200	Unconfined Compression
SA	Sieve Analysis	LVS	500	Laboratory Vane Shear
	Undisturbed Sample (2.5-inch ID)	FS	Free Swell	
	2-inch-ID Sample	EI	Expansion Index	
	Standard Penetration Test	Perm	Permeability	
	Bulk Sample	SE	Sand Equivalent	

KEY TO TEST DATA

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Job No: 12-3798
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Drwn: LPDD
Date: APR 2013

SOIL CLASSIFICATION CHART AND KEY TO TEST DATA

556 El Camino Real
Burlingame, California

PLATE

3

APPENDIX B

List of References

1. Brabb, E.E., and Pampeyan, E.H., 1983, *Geologic Map of San Mateo County, California*, United States Geological Survey Miscellaneous Investigations Series Map I-1257A, Scale 1:62,500.
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APPENDIX C

Field Exploration

Our field exploration consisted of a geologic reconnaissance and subsurface exploration by means of one test boring logged by our Engineer on April 5, 2013. The test boring was drilled with truck mounted equipment utilizing continuous flight, hollow stem, 8-inch-diameter augers. The boring was drilled at the approximate location shown on Plate 1.

The log of the test boring is displayed on Plate 2. Representative undisturbed samples of the earth materials were obtained from the test boring at selected depth intervals with a 1.4-inch inside diameter, split-barrel Standard Penetration Test (SPT) sampler.

Penetration resistance blow counts were obtained by dropping a 140-pound hammer through a 30-inch free fall. The sampler was driven 18 inches and the number of blows was recorded for each 6 inches of penetration. The blows per foot recorded on the Boring Log represent the accumulated number of blows that were required to drive the sampler the last 12 inches or fraction thereof.

The soil classifications are shown on the Boring Log and referenced on Plate 3.

Laboratory Testing

Natural water contents and percentages of gravel, sand, and fines were determined on selected soil samples recovered from the test boring. The data are recorded at the appropriate sample depths on the Boring Log.

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Project Number: 12-3798
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April 14, 2013

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APPENDIX D

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