EARTH MECHANICS CONSULTING ENGINEERS

Geotechnical Engineering

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November 29, 2007
Project Number: 04-2202

Mr. Eddy Tsang
334 Kearny Street, Suite 300
San Francisco, CA 94108

Subject: Geotechnical Plan Review
Planned Development at
180 Jones Street (Previously 181-189 Turk Street)
San Francisco, California

Dear Mr.Tsang,

This letter presents the results of our geotechnical review of the plans for the planned development at 180 Jones Street (Previously 181-189 Turk Street) in San Francisco, California. Earth Mechanics Consulting Engineers performed a geotechnical investigation for the project and presented results in the report dated January 15, 2005.

We reviewed sheets A-0 through A-7, dated 2/23/05, revised 11/28/07, by Gabriel Y. Ng & Associates.

Based on our review, we conclude that the plans are in general conformance with the intent of the recommendations contained in our geotechnical report.

Please note the following:

1. The project has been revised from a 5-story wood frame building to an 8-story concrete building. It is our opinion that the project geotechnical report dated January 15, 2005, is valid for the revised project without modification.

2. All site grading, foundation excavations, backfill, and geotechnical construction should be performed in accordance with the recommendations set forth in the project geotechnical report prepared by Earth Mechanics Consulting Engineers, Oakland, CA, (510) 839-0765, dated January 15, 2005. The contractor should coordinate all such work with the Geotechnical Engineer so that the necessary tests and on-site construction reviews can be made. Earth Mechanics Consulting Engineers should be notified at least 48 hours prior to our required site observations of foundation excavations and geotechnical-related construction.

We appreciate the opportunity to be of continued service to you on this project. If you have any questions, please call me at (510) 839-0765.

Sincerely,
EARTH MECHANICS CONSULTING ENGINEERS

H. Allen Gruen

REGISTERED PROFESSIONAL ENGINEER
H. ALLEN GRUEN
GE 2147
EXP 12/31/07
GEOTECHNICAL
STATE OF CALIFORNIA

H. Allen Gruen, C.E., G.E.
Principal Engineer

cc: Gabriel Y. Ng & Associates (2 copies)
1375 Sutter Street, Suite 102
San Francisco, CA 94109

REPORT
GEOTECHNICAL INVESTIGATION
Planned Development
181-189 Turk Street
San Francisco, California

Prepared for:

Mr. Eddy Tsang
334 Kearney Street, Suite 300
San Francisco, CA 94108

Prepared by:

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360 Grand Avenue, Suite 262
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H. Allen Gruen

H. Allen Gruen
Registered Geotechnical Engineer No. 2147

REGISTERED PROFESSIONAL ENGINEER
H. ALLEN GRUEN
No. 2147
Exp. 12/31/05
GEOTECHNICAL
STATE OF CALIFORNIA

January 15, 2005

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INTRODUCTION

Purpose

A geotechnical investigation has been completed for the planned development to be constructed at 181-189 Turk Street in San Francisco, 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 October 30, 2004. Our investigation included a geologic reconnaissance of the site and surrounding properties; sampling and logging one test boring to a depth of 12 feet below existing grade; laboratory testing conducted upon selected samples of the earth materials recovered from the boring; a review of published geologic data pertinent to the project area; geotechnical interpretation and engineering analyses; and the preparation of this report.

This report contains the results of our investigation, including findings regarding site, soil, geologic and groundwater conditions; conclusions pertaining to site exposure to geologic hazards, potential settlement, and foundation alternatives; and recommendations for site preparation and grading, foundations, retaining walls, and slabs on grade.

Pertinent exhibits appear in Appendix A. The location of the test boring is depicted in relationship to existing site features on Plate 1, Boring Location Map. The Log of Boring is displayed on Plate 2. An explanation of the symbols and other codes used on the log is presented on Plate 3, Soil Classification Chart and Key to Test Data. Natural moisture content and percent passing the # 200 sieve obtained in the laboratory are posted on the Log of Boring.

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

Proposed Project

It is our understanding that the project will consist of the design and construction of a new development with between 20 and 30 residential units. The building will extend 5 stories above grade and have 1 basement level. No other project details are known at this time.

FINDINGS

Site Description

The subject lot is located in the southeast quadrant of the intersection of Turk Street and Jones Street in San Francisco, California. The subject lot is covered by asphalt paving and is currently used as a parking lot. The topography in the vicinity of the site slopes downward toward the southeast at an average inclination of about 70:1 (horizontal:vertical).

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. This unit is Jurassic to Cretaceous in age and forms the basement rocks in the region.

Locally, the site lies within the USGS San Francisco North Quadrangle. Schlocker, et. al. (1958) has mapped the area of the site as being underlain by Quaternary-age dune sand. This deposit consists of clean, well-sorted, fine to medium grained sand.

Mapping by DeLisle (1993) indicates that bedrock in the vicinity of the subject site is about 40 feet below the ground surface.

Earth Materials

Our boring at the subject site penetrated fill consisting primarily of loose to medium dense, dune sand with concrete debris to the maximum depth explored of 12 feet. At a depth of 12 feet, our boring encountered an obstruction which could not be drilled through. It is our opinion that the obstruction was a large piece of concrete debris.

Groundwater

Free groundwater was not encountered in our boring to the maximum depth explored of 12 feet. Mapping by DeLisle (1993) indicates that groundwater in the vicinity of the subject site is about 10 feet below the ground surface. We anticipate that the depth to the free water table will vary with time and that zones of seepage may be encountered near the ground surface following rain or irrigation near the subject site.

CONCLUSIONS

General

On the basis of our site reconnaissance and data review, we conclude that the site is suitable for support of the proposed improvements. The primary geotechnical concerns are founding improvements in competent earth materials, the potential for high groundwater, support of temporary slopes and adjacent improvements, and seismic shaking and related effects during earthquakes. These items are addressed below.

Foundation Support

It is our opinion that the planned improvements may be supported on a mat foundation. The Structural Engineer may choose to use drilled piers to support heavy, concentrated loads, 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.

Groundwater Considerations

Free groundwater was not encountered in our boring to the maximum depth explored of 12 feet. However, mapping by DeLisle (1993) indicates that groundwater in the vicinity of the subject site is about 10 feet below the ground surface. Our firm previously drilled a boring near the southeast corner of Turk and Hyde Streets, where water was encountered at a depth of about 30 feet below the street elevation. We also drilled a boring near the southeast corner of Leavenworth and McAllister Streets, where water was encountered at a depth of about 17 feet below the street elevation.

There is a potential that groundwater could be encountered in the planned site excavations, more than 10 feet below the street elevation. If groundwater were encountered, the excavation could be dewatered using well points or sump pits.

Temporary Slopes and Undermining of Existing Structures

Temporary slopes will be necessary during the planned site excavations. It is our opinion that the sand encountered in our boring at the site will not stand in vertical cuts more than a few feet in height. In order to safely develop the site, temporary slopes will need to be laid back at safe inclinations or temporary shoring will have to be installed. The contractor may choose to excavate test pits or trenches prior to construction to confirm subsurface conditions and evaluate 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 support to the adjacent improvements.

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.

Geologic Hazards

Faulting

The property does not lie within an Alquist-Priolo Earthquake Fault Zone as defined by the California Division of Mines and Geology. The closest mapped active fault in the vicinity of the site is the San Andreas Fault, located about 7.7 miles southwest of the site (Jennings, 1992). No active faults are shown crossing the site on reviewed published maps, nor did we observe evidence of active faulting during our investigation. Therefore we conclude that the potential risk for damage to improvements at the site due to surface rupture from faults to be 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 expect that the site will be exposed to strong earthquake shaking during the life of the improvements. The recommendations contained in the applicable edition of the San Francisco and Uniform Building Codes 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 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 site, as well as the majority of the area southeast of the site between the subject site and the San Francisco Bay lies within a liquefaction potential zone as mapped by the California Division of Mines and Geology for the City and County of San Francisco (CDMG, 2000).

It is our opinion that granular earth materials beneath the depth of our soil boring at the subject site, could be subject to liquefaction. We judge that the impact of liquefaction on the planned structure supported on a stiffened mat foundation would be limited to post liquefaction settlements. These post liquefaction settlements would affect the subject structure and the adjacent buildings, streets and appurtenant improvements for several blocks in the vicinity of the subject site. It is our opinion that a properly designed and constructed stiffened mat foundation would be sufficient to reduce total settlements from post liquefaction strain to less than about 2 inches with differential settlements of less than ½ inch over a 25-foot span. Due to the size and depth of the mat foundation, reduction in bearing capacity is not a concern; nor would sand boils injecting sand and water into the lower level of the structure be a concern due to the thickness of the mat slab.

Lateral Spreading

Lateral spreading or lurching is generally caused by liquefaction of marginally stable soils underlying gentle slopes and is usually accompanied by fissures. Lateral spreads involve lateral displacements of large, surficial blocks of soil as a result of liquefaction in a subsurface layer. Movement occurs in response to the combined gravitational and inertial forces generated by an earthquake. Lateral spreads generally develop on gentle slopes (most commonly between 0.3 and 3 degrees) and move toward a free face, such as an incised river channel or bank. Since the nearest free face is the San Francisco Bay located over 8,000 feet east of the site, we judge that there is a low risk of damage to the improvements from seismically induced lateral spreading.

Seismic Settlement and Differential Compaction

During earthquake shaking, loose granular soils above the groundwater may densify resulting in the settlement of the ground surface. We judge that there is a potential for densification and settlement of the fill materials at the subject site during earthquake shaking. Improvements founded near the ground surface may experience settlement during strong seismic shaking as the loose fill densifies. Based on data from Tokimatsu and Seed (1987), we estimate that the magnitude of settlement due to seismic densification would be less than 1 inch.

Landsliding

The geologic maps of the site vicinity reviewed for this study did not show landslides at the site or its immediate vicinity. In addition, a map prepared by the California Division of Mines and Geology for the City and County of San Francisco (CDMG, 2000) does not indicate that the subject site lies within an area of potential earthquake-induced landsliding. 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 damage to the improvements from slope instability at the site is low provided the recommendations presented in this report are incorporated into the design and construction of the project.

RECOMMENDATIONS

Site Preparation and Grading

Clearing

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 be less than about 2 inches. Deeper stripping may be required to remove localized concentrations of organic matter, such as 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

Existing loose, unstable soil should be overexcavated in areas designated for placement of future engineered fills or 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 unstable soils that should be removed and replaced as engineered fill. The depth and extent of overexcavation should be approved in the field by the geotechnical engineer prior to fill placement.

Subgrade Preparation

Exposed soils designated to receive engineered fill should be scarified to a minimum depth of 6 inches, 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.

General Engineered Fill

It is anticipated that the on-site soils will be suitable for reuse as general engineered 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.

General engineered 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 existing improvements, care should be taken to adequately support the 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. It is our opinion that the sand encountered in our boring at the site will not stand in vertical cuts more than a few feet in height. In order to safely develop the site, temporary slopes will need to be laid back at safe inclinations or temporary shoring will have to be installed. The contractor may choose to excavate test pits or trenches prior to construction to confirm subsurface conditions and evaluate the need for temporary shoring. 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.

Seismic Design

In accordance with the requirements of the Uniform Building Code, the closest seismic source is the San Andreas Fault located about 7.7 miles southwest of the subject site. The San Andreas Fault is a seismic source type A. The San Andreas Fault has a maximum moment magnitude of 7.9 and is about 12 kilometers from the subject site which results in near source factors of Na = 1.0 and Nv = 1.12. The site is within seismic Zone 4; therefore, a Seismic Zone Factor "Z" of 0.4 is appropriate. The soil profile at the site approximates type S_C.

Foundations

General

The planned improvements may be supported on a mat foundation. The Structural Engineer may choose to use drilled piers to support heavy, concentrated loads, or for shoring and underpinning, if required. Design criteria for each foundation type are presented below.

Mat Foundation

The planned improvements may be supported on a stiffened mat foundation, designed to tie structural elements together and to accommodate the anticipated settlements. A modulus of vertical subgrade reaction of 50 tons per cubic foot may be used for elastic analyses of the mat foundation. The bearing capacity of the mat foundation should be less than 3,000 psf for dead plus sustained live loads and 4,000 psf for total loads including wind or seismic loads. Localized increases in bearing pressures of up to 5,000 psf may be utilized. The weight of the mat may be ignored in computing allowable bearing pressures.

The foundation should be stepped as necessary to produce level tops and bottoms. A passive equivalent fluid pressure of 300 pounds per cubic foot and a friction factor of 0.4 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 for concentrated loads, or shoring excavation walls and underpinning adjacent improvements. Piers should be designed for a maximum allowable skin friction of 600 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 300 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 that are free to rotate at the top should be designed to resist active lateral earth pressures equivalent to those exerted by a fluid weighing 30 pcf where the backslope is level, and 50 pcf for backfill at a 2:1 (horizontal:vertical) slope. 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 (H:V) 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.

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•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.

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 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 foundations 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

Slab-on-grade floor subgrade should be proof rolled to provide a firm, unyielding surface for slab support. If moisture penetration through the slab would be objectionable, slabs should be underlain by a moisture vapor barrier membrane. The membrane may be covered with 2 inches of damp, clean sand to protect it during construction.

Surface Drainage

The site should be graded to provide positive drainage away from building areas. Roof runoff should be directed toward downspouts that discharge into closed conduits, or onto concrete slabs or asphalt pavements that drain away from the foundations and into the site storm drain system.

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 foundation excavations and fill compaction, as well as to perform appropriate field observations and laboratory tests.

If, during construction, subsurface conditions different from those encountered in the explorations 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. Eddy Tsang 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, the results of our field exploration and laboratory testing programs, 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.

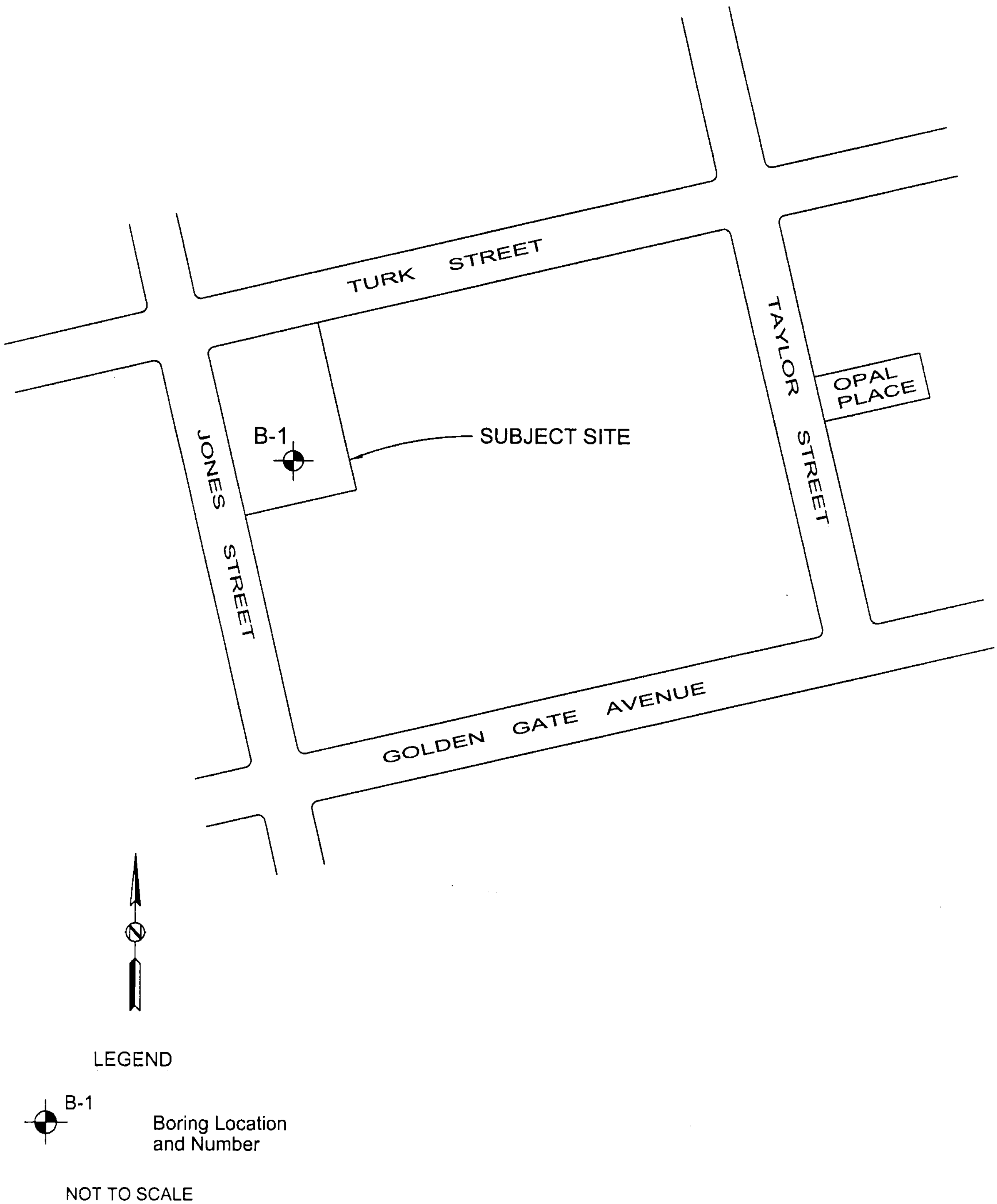
The test boring log represents subsurface conditions at the location and on the date indicated. It is not warranted that it is representative of such conditions elsewhere or at other times. Site conditions and cultural features described in the text of this report are those existing at the time of our field exploration, conducted on December 2, 2004, 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



Earth Mechanics Consulting Engineers	Job. No: 04-2202 Appr: Drwn: LPDD Date: 12/5/04	BORING LOCATION MAP 181-189 Turk Street San Francisco, California	PLATE 1

EQUIPMENT: **4" Flight Auger**

LOGGED BY: **A. Gruen**

ELEVATION: *

START DATE: 12-2-04

FINISH DATE: 12-2-04

Other Laboratory Tests	Pocket Penetrometer (ksf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200 sieve	Blows/Foot	DEPTH (FEET)	Description
					24	0	A.C. Paving
							Baserock
		3.1		2		1	Brown Poorly Graded Sand (SP), loose to medium dense, with concrete debris
					7	6	
		5.1		4		7	
					14	8	
		5.9		7		9	
						12	(FILL)

Refusal @ 12' in debris
No Free Water Encountered

* Existing ground surface.

Earth Mechanics
Consulting Engineers

Job No: 04-2202
Appr:
Drwn: LPDD
Date: DEC 2004

LOG OF BORING 1

181-189 Turk Street

San Francisco, 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, POOORLY 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

			Shear Strength, psf (Confining Pressure, psf)	
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

Earth Mechanics Consulting Engineers	Job No: 04-2202 Appr: Drwn: LPDD Date: DEC 2004	**SOIL CLASSIFICATION CHART AND KEY TO TEST DATA** 181-189 Turk Street San Francisco, California	PLATE 3

APPENDIX B

List of References

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14. Schlocker, J., Bonilla, M. G., and Radbruch, D, H., 1958, *Geology of the San Francisco North Quadrangle, California*, United States Geological Survey Miscellaneous Geologic Investigations Map I-272 (scale: 1:24,000).

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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 project engineer on December 2, 2004. The test boring was drilled with a hand carried, portable drill rig utilizing continuous flight, 4-inch-diameter augers. The boring was drilled at the approximate location shown on Plate 1.

The Log of 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 and a 2.5-inch inside diameter, modified California 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 or to refusal and the number of blows was recorded for each 6 inches of penetration or fraction thereof in the case of refusal. 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 classification is shown on the boring log and is referenced on Plate 3, Soil Classification Chart and Key to Test Data.

Laboratory Testing

Natural water content was determined on selected soil samples recovered from the test boring; the data are recorded at the appropriate sample depths on the boring log. Selected samples were washed through the # 200 sieve to determine the silt and clay content. The percent passing the #200 sieve shown on the boring log indicates the combined silt and clay content.

APPENDIX D

Distribution

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