REPORT
GEOTECHNICAL INVESTIGATION
Planned Development at
490 South Van Ness Avenue
San Francisco, California

Prepared for:

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Prepared by:

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Project Number: 09-3406

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November 12, 2009

TABLE OF CONTENTS

TABLE OF CONTENTS, CONTINUED

INTRODUCTION

Purpose

A geotechnical investigation has been completed for the planned development at 490 South Van Ness Avenue 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 22, 2009. Our investigation included a reconnaissance of the site and surrounding vicinity; sampling and logging two test borings to a maximum depth of 51 feet below the ground surface; laboratory testing conducted on selected samples of the earth materials recovered from the borings; 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 locations of the test borings are depicted relative to site features on Plate 1, Boring Location Map. The logs of the test borings are displayed on Plates 2 and 3. Explanations of the symbols and other codes used on the logs are presented on Plate 4, 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 Project

It is our understanding that the project will consist of the design and construction of a mixed-use development at the subject site. The proposed structure will have 1 basement level and extend 6 floors above grade. Approximately 50 residential units will be housed in the building. No other project details are known at this time.

FINDINGS

Site Description

The subject site is located west of South Van Ness Avenue, between Adair and 16th Streets in San Francisco, California. The topography in the vicinity of the site slopes downward toward the east at an average inclination of about 60:1 (horizontal: vertical). At the time of our investigation, the subject site was occupied by an oil change/service station with parking 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. These units are Jurassic to Cretaceous in age and form the basement rocks in the region.

The site is located in the USGS San Francisco North Quadrangle (1993). Locally, Schlocker (1958) has mapped the site vicinity as being underlain by undivided Quaternary-age deposits. These deposits typically consist of sand-clay soil mixtures.

Mapping by DeLisle (1993) indicates that bedrock in the vicinity of the subject site is about 100 feet below the ground surface.

Background

We reviewed a proposal by Treadwell & Rollo dated June 16, 2009. In this proposal they reported the following:

"In September 1998, one 8,000-gallon gasoline underground storage tank (UST), two 6,000-gallon gasoline UST, one 550-gallon waste oil UST, associated piping were removed from the Site by SEMCO. During the removal activities, petroleum hydrocarbon contamination was noted within the soil and groundwater. Overexcavation was performed at the tank excavations and contaminated soil and groundwater were removed and properly disposed of at a regulated landfill."

Earth Materials

As described in the background section above, underground storage tanks and contaminated soil were removed from the site and presumably replaced with fill soil. We anticipate that the thickness of fill will vary across the site.

Our borings drilled at the subject site generally encountered sand-clay soil mixtures to the maximum depth explored of 51 feet. Boring 1 penetrated about 3 feet of very loose to loose, clayey sand with gravel overlying medium dense to very dense, clayey sand. At a depth of about 35 feet, Boring 1 encountered very dense, poorly graded sand with clay, underlain at a depth of about 40 feet by very dense, clayey sand. Boring 2 penetrated about 3 feet of very loose to loose, clayey sand with gravel underlain by medium dense, clayey sand. Detailed descriptions of the materials encountered as well as test results are shown on the Boring Logs, Plates 2 and 3.

Groundwater

Free groundwater was encountered in our borings at a depth of 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 upslope of the subject site.

CONCLUSIONS

General

On the basis of our site reconnaissance, field exploration program, and literature review, we conclude that the site is suitable for support of the proposed improvements. The primary geotechnical concerns are support of temporary slopes and adjacent improvements, groundwater considerations, founding the improvements in suitable materials, and seismic shaking and related effects during earthquakes. These items are addressed 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 adjacent improvements, it may be necessary to underpin those improvements. 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 our borings at a depth of about 10 feet. The depth to the free groundwater may require dewatering to excavate and construct the proposed basement level in the dry. In addition, the basement level may need to be constructed using water tight methods. Depending on the basement floor elevation relative to the groundwater, floor slabs and retaining walls may need to be designed for hydrostatic 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. Given the depth to the free groundwater level and the anticipated full basement, a mat slab may provide the most water-tight construction. 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 6.8 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 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 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 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 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).

We extended a boring to 51 feet below the ground surface. The earth materials encountered in our boring had relatively high plastic fines contents and/or high relative densities. We judge that due to the plastic fines contents and relatively high relative densities of the earth materials sampled, that there is a relatively low potential for damage to the proposed improvements from liquefaction at the site.

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. Because it is our opinion that the site has a low potential for liquefaction, 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 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

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. Excavations extending near or below the free groundwater depth encountered in our borings of 10 feet, may require dewatering so that construction may proceed in the dry.

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 uncompacted fill, 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 existing improvements, care should be taken to adequately support the adjacent 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 and construction 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 in soil 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

If the improvements are designed using the 2007 California Building Code, the following parameters apply:

Site Class D
$S_s = 1.500$, $S_1 = 0.693$
Fa = 1.0, Fv = 1.5
$SM_s = 1.500$, $SM_1 = 1.040$
$SD_s = 1.000$, $SD_1 = 0.693$

Foundations

General

It is our understanding that the planned structure will have a full basement level beneath the entire footprint. The foundation design criteria presented below assume that the bottoms of the foundation elements will be at least 10 feet below current site grades. If foundations are bottomed closer to the current ground surface, we should be retained to compute the appropriate foundation design criteria for the shallower depth.

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. Given the depth to the free groundwater level and the anticipated full basement, a mat slab may provide the most water-tight construction. 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 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 3,500 and 5,000 psf, respectively.

Resistance to lateral pressures can be obtained from passive earth pressures against the face of the footing 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. These values include a safety factor of 1.5 and may be used in combination without reduction. 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

Refer to the section below for Slabs On Grade for drainage and water proofing considerations. 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 3,500 psf for combined dead plus sustained live loads, and 5,000 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 7,500 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. These values include a safety factor of 1.5 and may be used in combination without reduction. 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 800 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, or designed to resist hydrostatic and earth pressures below a depth of about 10 feet. 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.

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 40 pcf where the backslope is level, and 60 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 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.

We recommend that retaining walls extending more than 10 feet below current site grades be designed to resist the lateral pressures presented above plus hydrostatic water pressure. The depth to free groundwater should be assumed at 10 feet below current site grades.

If retaining walls are designed using the 2007 California Building Code, 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 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.

If the basement floor slab is above the free groundwater level (about 10 feet below current site grades), additional protection against moisture seepage into the basement may be provided by installing a slab underdrain system. The slab underdrain system should consist of trenches, which are at least 12 inches deep and 6 inches wide, spaced no further than 10 feet apart beneath the floor slab. The bottoms of the trenches should slope to drain to a low-point by gravity. A 3-inch diameter, rigid perforated pipe should be placed near the bottom of the trench which is fully encapsulated in drain rock. The drainrock should be fully encapsulated in an approved filter fabric. The perforated pipes should be tied to closed conduits which outlet at appropriate discharge points.

Water tight construction should be considered, if the basement floor slab is at or below the free groundwater level (about 10 feet below current site grades). Slabs bottomed more than 10 feet below current site grades should be designed to resist hydrostatic uplift pressures assuming the free groundwater level is at a depth of 10 feet below current site grades.

Site Drainage

Positive drainage should be provided away from the improvements. Roof downspouts should 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 (retaining wall backdrains and underslab 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 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 JCN Developers, LLC and their 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.

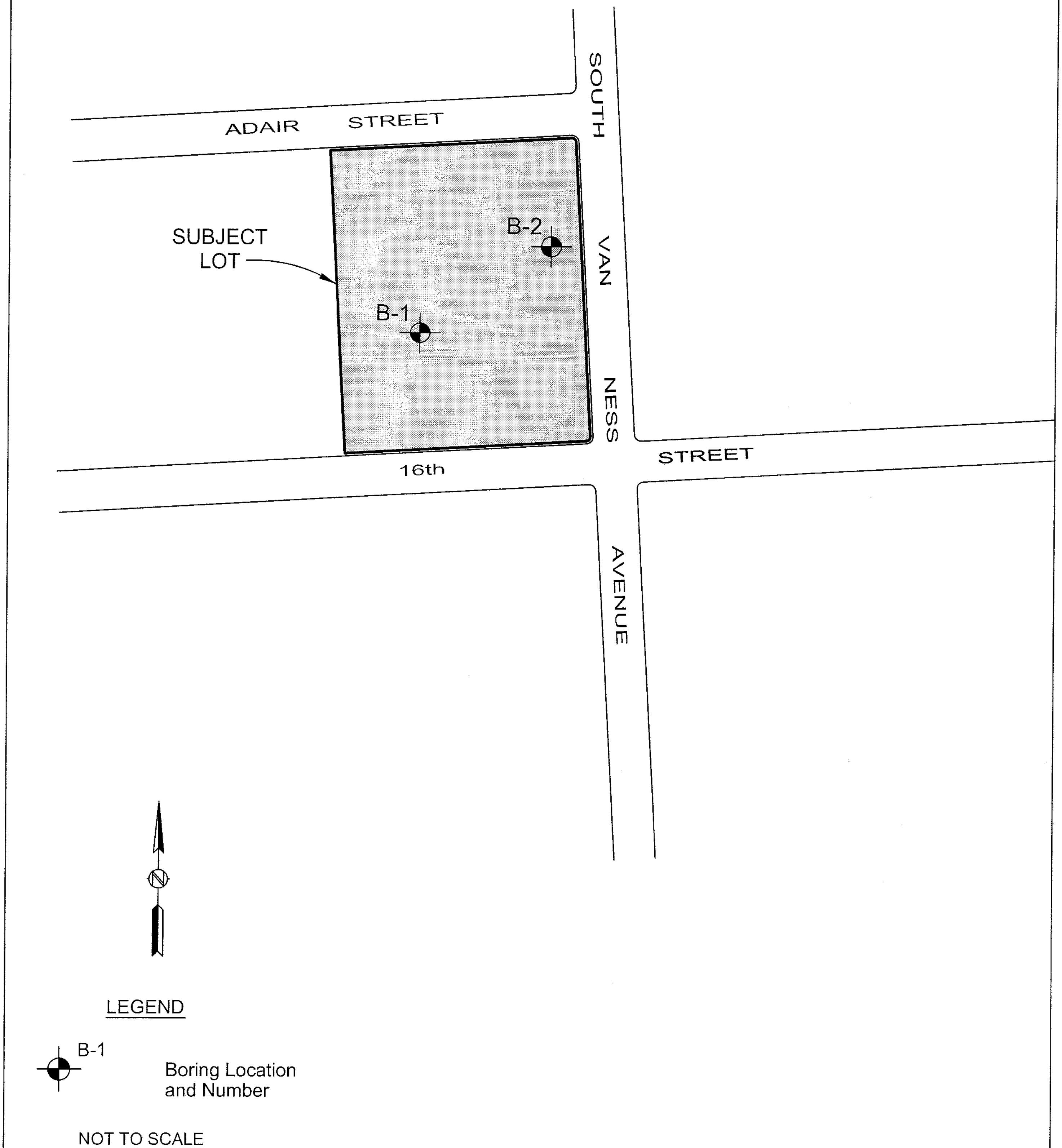
The test boring logs represent subsurface conditions at the locations and on the date indicated. It is not warranted that they are representative of such conditions elsewhere or at other times. Site conditions and cultural features described in the text of this report are those existing at the time of our field exploration, conducted on November 3, 2009, and may not necessarily be the same or comparable at other times.

The location of the test borings were 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: 09-3406 Appr: Drwn: LPDD Date: 11/9/09	**BORING LOCATION MAP** 490 South Van Ness Avenue San Francisco, California	PLATE 1

EQUIPMENT: **6" Flight Auger**
LOGGED BY: **A.K.**
ELEVATION: *
START DATE: **11-3-09**
FINISH DATE: **11-3-09**

Other Laboratory Tests	Pocket Penetrometer (ksf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200 sieve	Blows/Foot	Sample	DEPTH (FEET)	Description
							0	Asphalt and Base 3"
							0–4	Brown Clayey Sand with Gravel (SC), moist, very loose to loose, with brick and concrete fragments (Fill)
							4	Brown Clayey Sand (SC), moist, medium dense
		28.3		33	27	◣	5	
		20.9		18	27	◣	10	saturated, greenish-brown
		27.1		24	20	◣	15	- brown
		23.0		27	38	◣	20	- dense
		19.2		26	30	◣	25	- medium dense, orangish-brown
		22.1		15	73	◣	30	- very dense
		21.6		10	60	◣	35	Brown Poorly Graded Sand with Clay (SP-SC), saturated, very dense
		19.6		21	73	◣	40	Brown Clayey Sand (SC), saturated, very dense
		18.6		24	75	◣	45	
		20.9		18	64	◣	50	
							51	BOTTOM OF BORING = 51' Water @ 10'

* Existing ground surface.

Earth Mechanics
Consulting Engineers

Job No: 09-3406
Appr:
Drwn: LPDD
Date: NOV 2009

LOG OF BORING 1

490 South Van Ness Avenue
San Francisco, California

PLATE 2

EQUIPMENT: **6" Flight Auger** ELEVATION: *

LOGGED BY: **A.K.** START DATE: **11-3-09**

FINISH DATE: **11-3-09**

Other Laboratory Tests	Pocket Penetrometer (ksf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200 sieve	Blows/Foot	Sample	DEPTH (FEET)	Description
							0	Asphalt and Base 3"
							1–4	Brown Clayey Sand with Gravel (SC), moist, very loose to loose, with brick and rock fragments (Fill)
							4–5	Mottled Brown Clayey Sand (SC), moist, medium dense
		18.1		32	11		5–6	
		22.2		21	23		10–11	saturated, bluish-brown
		29.3		24	29		15–16	- brown

BOTTOM OF BORING = 16-1/2'
Water @ 10'

* Existing ground surface.

Earth Mechanics
Consulting Engineers

Job No: 09-3406
Appr:
Drwn: LPDD
Date: NOV 2009

LOG OF BORING 2

490 South Van Ness Avenue

San Francisco, California

PLATE

3

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: 09-3406 Appr: Drwn: LPDD Date: NOV 2009	**SOIL CLASSIFICATION CHART AND KEY TO TEST DATA** 490 South Van Ness Avenue San Francisco, California	PLATE **4**

APPENDIX B

List of References

1. California Department of Conservation, Division of Mines and Geology, 1998, *Maps of Known Active Fault Near-Source Zones in California and Adjacent Portions of Nevada.*

2. CDMG, 2000, State of California Seismic Hazards Zones, City and County of San Francisco, California Division of Mines and Geology, released November 17, 2000.

3. DeLisle, M., 1993, *Map Showing Areas of Exposed Bedrock and Contours on Bedrock Surface on a Portion of the San Francisco North 7.5' Quadrangle*, unpublished map by the California Division of Mines and Geology.

4. DeLisle, M., 1993, *Map Showing Generalized Contours on the Groundwater Surface on a Portion of the San Francisco North 7.5' Quadrangle*, unpublished map by the California Division of Mines and Geology.

5. Schlocker, J., 1958, Geology of the San Francisco North Quadrangle, California, United States Geological Survey Professional Paper 782, scale 1:24,000.

6. Seed, H. B., and Idriss, E., 1982, *Ground Motion and Soil Liquefaction During Earthquakes*, Earthquake Engineering Research Institute Monograph

7. United States Geological Survey, 1993, San Francisco North Quadrangle, 7.5 Minute Series, Scale 1:24,000.

APPENDIX C

Field Exploration

Our field exploration consisted of a geologic reconnaissance and subsurface exploration by means of two test borings logged by our Engineer on November 3, 2009. The test borings were drilled with a truck-mounted drill rig utilizing continuous flight, 6-inch-diameter augers. The borings were drilled at the approximate locations shown on Plate 1.

The logs of the test borings are displayed on Plates 2 and 3. Representative undisturbed samples of the earth materials were obtained from the test borings 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 Logs 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 Logs and referenced on Plate 4.

Laboratory Testing

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

APPENDIX D

Distribution

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