ALAN KROPP
& ASSOCIATES, INC.
GEOTECHNICAL CONSULTANTS

ALAN KROPP, CE, GE
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March 23, 2021
501-4D, L-32322

Mr. Lani Su
Sulh626@gmail.com

RE: Update of Geotechnical Investigation
1201 San Pablo Avenue Mixed-Use Development
Berkeley, California

Dear Mr. Su:

As authorized, we have performed a geotechnical update for the proposed mixed-use retail/residential building to be constructed on the currently vacant parcel located at 1201 San Pablo Avenue in Berkeley, California.

The following geotechnical report and supplemental reports/letters were prepared by our office for the previously proposed development on the property.

- Geotechnical Investigation Report, 1201 San Pablo Avenue Development, Berkeley, California, dated December 17, 2004;

- Update of Geotechnical Investigation Report, 1201 San Pablo Avenue Development, Berkeley, California, dated March 2, 2005;

- Response to Peer Review Comments, 1201 San Pablo Avenue Development, Berkeley, California, dated May 31, 2005; and

- Supplemental Geotechnical Investigation, 1201 San Pablo Avenue Development, Berkeley, California, dated August 7, 2018.

This update has been prepared to address the currently proposed mixed-use development project and to provide supplemental/updated geotechnical project recommendations as may be appropriate.

PROPOSED PROJECT

Since the issuance of our December 17, 2004 report, several multi-story, mixed-use projects with slightly different configurations have been proposed for the site. However, the site has remained vacant. Our most recent supplemental investigation, which was conducted in 2018, included the drilling of three additional exploratory borings on the site (supplementing information from the two exploratory borings drilled in 2004) and a complete update of the project geotechnical recommendations. The project under consideration in 2018 was a 5-story, mixed-use building encompassing almost the entire site, with one level of below-grade parking beneath the entire building footprint. Our review of preliminary architectural drawings by Trachtenberg Architects indicates that the currently proposed project is a similar mixed-use development; however, the building is to be up to 6 stories in height and will be constructed at grade. The building will occupy the entire site, except for a 5-foot-wide maintenance-only landscape strip along the eastern side of the site.

SCOPE OF UPDATE SERVICES

The scope of our update services included the following:

- A site reconnaissance visit to confirm current site conditions;
- A review of our files from our previous site studies;
- Review and update of our site liquefaction calculations in order to address increased ground motions, as required by the current building code (CBC 2019); and
- Preparation of this update letter, providing supplemental project recommendations, as appropriate, to address the current site development plan.

As with our previous geotechnical design services, the scope of our services did not include an environmental assessment or investigation for the presence of hazardous or toxic materials in the soil, groundwater, or air on, below, or around this site. An evaluation of the potential presence of sulfates in the soil or other possibly corrosive, naturally occurring elements was also beyond our scope.

SITE RECONNSISANCE

A site reconnaissance visit was conducted on March 7, 2021, in order to evaluate current site conditions.

The subject site is roughly rectangular in shape and has maximum plan dimensions of about 100 feet by 130 feet. The property is bordered by San Pablo Avenue and Harrison Street on the west and north, respectively, existing residences to the east, and an apartment complex to the south. The site is relatively level and consists of a vacant lot covered with a low growth of weeds. An approximately 2- to 3-foot-high concrete wall is located along the eastern property boundary.

Based on our observations, it does not appear that the site has changed significantly since the time of our 2004 and 2018 investigations, and no obvious geotechnical concerns were observed during our site reconnaissance visit.

UPDATED LIQUEFAFACTION ASSESSMENT

As outlined in our 2018 report, we evaluated soil liquefaction potential and liquefaction-induced settlement in general accordance with the guidelines published in SP117A (CGS, 2008). As discussed in SP117A, the basic procedure most often used in engineering practice to assess site liquefaction potential is the “Simplified Procedure,” originally developed by Seed and Idriss (1970) and subsequently modified by Seed and various other researchers. This method involves assessing the level of earthquake-induced stress required to induce liquefaction in each soil layer and comparing this value to an estimate of earthquake-induced stress that the layer would experience in a design-level earthquake. The data required to establish the amount of stress needed to induce liquefaction in a soil layer are typically obtained from borings and/or cone penetration tests.

We utilized the recommendations presented in the NCEER/NSF summary report by Youd and Idriss (2001) to evaluate liquefaction susceptibility of coarse-grained soils. In these procedures, liquefaction susceptibility of coarse-grained soils is directly correlated to "corrected" Standard Penetration Test (SPT) blow counts, and coarse-grained soils with corrected SPT blow counts of less than approximately 30 are considered susceptible to liquefaction.

More recent research has shown that fine-grained soils (silts and even clays) can also experience liquefaction-type behavior in response to earthquake shaking. We utilized the recommendations presented in the paper by Bray and Sancio (2006) to evaluate liquefaction susceptibility of fine-grained soils. These recommendations correlate liquefaction susceptibility of fine-grained soils to their Plasticity Index (PI), Liquid Limit (LL), and natural water content (w_c) as follows:

- Soils with a PI less than 12 with a w_c greater than 85 percent of the LL are considered susceptible to liquefaction.

- Soils with a PI between 12 and 18 and a w_c greater than 80 percent of the LL are considered potentially susceptible to liquefaction.

- Soils with a PI greater than 18 or a w_c less than 80 percent of the LL are considered NOT susceptible to liquefaction.

Within potentially liquefiable soils layers identified, we estimated the magnitude of liquefaction-induced settlement using the method outlined by Tokimatsu & Seed (1987). Levels of ground shaking used in our analyses were updated from our 2018 calculations and based on an earthquake moment magnitude (M_W) of 7.5 (increased from 7.1) with a peak ground acceleration of 0.944g (increased from 0.819g). An assumed depth to groundwater of 10 feet was used in our analyses. Updated liquefaction calculations are appended to this report (Appendix A). While the updated calculations (with higher earthquake magnitude and peak ground acceleration) did result in a slightly lower factor of safety against liquefaction in layers that were previously identified as at risk for liquefaction, no additional subsurface layers were identified at risk for liquefaction and no increase in predicted liquefaction-induced settlement was predicted. Therefore, the conclusions in regard to liquefaction issues as outlined in our 2018 report are still considered valid. For the convenience of the reader, these conclusions are repeated below.

The subsurface material as encountered in our site borings within the upper 8 to 14 feet predominately consisted of stiff to hard highly to critically plastic clays, which were judged not to be susceptible to liquefaction. However, underlying the surficial clays we encountered layers of loose to medium dense well graded to clayey gravels, and loose to medium dense clayey sands, which were judged to be potentially liquefiable. Starting at a depth of approximately 26 to 38 feet, stiff to hard clay with varying amounts of silt, sand, and gravel were encountered that we also judge are not susceptible to liquefaction.

The results of our analyses found that ground shaking at the site during a design level earthquake would be large enough to induce liquefaction in the well graded to clayey gravel layers and some of the clayey sand layers underlying the site. In Boring 2004-1, we calculated approximately 2.9 inches of settlement within the well graded to clayey gravel layers and clayey sand layer encountered between a depth of 10 and 24 feet. In Boring 2004-2, we calculated approximately 2.4 inches of settlement within the well graded to clayey gravel layer and clayey sand layer encountered between a depth of 11 and 26 feet. In

Boring 2018-2, we calculated approximately 0.7 inches of settlement within the clayey sand layer encountered between a depth of 27 and 32 feet.

The above estimates suggest approximately 1.0 to 3.0 inches of total liquefaction-induced settlement could occur in portions of the site following an earthquake. We note that these amounts of settlement occurring at the ground surface would likely be partially mitigated by 10 or more feet of overlying non-liquefiable material. Comparisons between the settlement estimates for each boring suggest that between 1.0 and 2.0 inches of differential settlement could occur between the boring locations. With a judgement based maximum liquefaction-induced differential settlement of up to 2.0 inches between borings (spaced roughly 50 to 60 feet apart), we would expect potentially 1.0 inch of differential settlement to occur over a horizontal distance of 20 to 30 feet (angular distortion of 1/240 to 1/360). In order to account for possible differential settlement of the proposed building during a design level earthquake, we recommend the building be supported on a stiffened foundation system comprised of a structural mat designed to tolerate this magnitude of differential settlement.

We judge there to be a generally low risk that large-scale lateral spreading will occur at the site as a result of the design-level earthquake. This judgment is based to a large degree on the observation that the site is situated in an area of relatively level surface topography without a nearby "free-face". However, we anticipate that some lateral deformation of the ground surface may occur as a result of liquefaction-induced movements. In our opinion, it is reasonable to assume that the magnitude of these localized lateral deformations will likely not significantly exceed the values calculated for liquefaction-induced settlement.

EVALUTION AND CONCLUSIONS

Based on our update evaluation, it is our opinion that the site is suitable, from a geotechnical viewpoint, for the proposed mixed-use development. Recommendations as given in our 2018 report are considered applicable, except where superseded by the supplemental recommendations given in this current update letter in order to address the currently proposed at-grade building construction. For completeness of the record, a copy of our 2018 report is appended to this update letter (Appendix B).

SUPPLEMENTAL RECOMMENDATIONS

Structural Mat Foundation

It remains our opinion that a structural mat foundation designed to effectively tolerate the anticipated potential liquefaction-related settlement and span and cantilever over local irregularities—while preventing excessive structural distress and/or collapse—is appropriate for the proposed mixed-use building. Recommendations as given in our 2018 report for support of the building on a 24-inch-minimum-thickness reinforced mat foundation, with the base of the mat at least 30 inches below the adjacent ground surface, is considered applicable. However, with at-grade construction, consideration of potential hydrostatic uplift forces and incorporation of an under-slab drainage system is not required. It is instead recommended that the mat be underlain by a minimum of 18 inches of compacted non-expansive material (Class II aggregate baserock or approved equivalent), placed over a scarified, moisture-conditioned and recompacted soil subgrade. The scarified subgrade soil should be moisture conditioned to approximately 2% to 5% above optimum moisture content and recompacted to a minimum relative degree of compaction of 90%, based on ASTM D1557-latest edition. The non-expansive layer (Class II aggregate baserock or approved equivalent), should be compacted to a minimum relative degree of compaction of 95%, based on ASTM D1557-latest edition.

In order to minimize vapor transmission, a vapor-retardant membrane (StegoWrap 15 mil or equivalent) should be placed over the compacted non-expansive material. The membrane should be placed with appropriate overlaps and taping (including taping at plumbing penetrations), in accordance with the manufacture's recommendations. We also recommend that the specifications for the mat require moisture-emission tests to be performed on the mat prior to the installation of the flooring. No flooring should be installed until safe moisture-emission levels are recorded for the type of flooring to be used.

California Building Code Seismic-Design Parameters

This section provides seismic-design parameters based on the 2019 California Building Code (CBC) and ASCE 7-16. The reported seismic-design parameters—as presented in Table 1, below—were developed utilizing the ASCE 7 online hazard report tool and are based on the site coordinates, site class (geology), and the assumed risk category of the building.

Table 1: Ground Motion Parameters Based on ASCE 7-16
(Site Coordinates: Latitude 37.88214°, Longitude -122.29607°)

Parameter	Value	ASCE 7-16 Reference
Risk Category (Assumed)	II	Table 1.5-1
Site Class	D	Table 20.3-1
S_s	2.042g	Figure 22-1
S_1	0.784g	Figure 22-2
F_a	1.0	Table 11.4-1
F_v	N/A*	Table 11.4-2
S_{MS}	2.042g	Equation 11.4-1
S_{M1}	N/A*	Equation 11.4-2
S_{DS}	1.361g	Equation 11.4-3
S_{D1}	N/A	Equation 11.4-4
C_V	1.5	Table 11.9-1
PGA	0.858g	Figure 22-9
F_{PGA}	1.1	Table 11.8-1
PGA_M	0.944g	Equation 11.8-1
T_L	8 Seconds	Figure 22-14

*Value not provided by online tool but may be determined by structural designer, based on building characteristics and/or structural design methodology, in accordance with exceptions given in Section 11.4.8.

PLAN REVIEW AND CONSTRUCTION OBSERVATION

We recommend our firm be provided the opportunity to review the geotechnical aspects of the plans and specifications for the work at this site in order to check that our geotechnical recommendations are properly interpreted and implemented. If our firm is not accorded the privilege of making the recommended review, we can assume no responsibility for misinterpretation of our recommendations.

We also recommend our firm be retained to provide geotechnical engineering services during the earthwork, foundation construction, and drainage phases of the work. This is to check compliance with the design concepts, specifications, and recommendations and to allow design changes, in the event that subsurface conditions differ from those anticipated prior to the start of construction.

We trust that this provides the information that you require at this time. If you have questions or comments regarding this letter, please call.

Very truly yours,

James R. Lott, G.E.
Associate Engineer

JRL/ab

Copies: Addressee (PDF)

501-4D 1201 San Pablo Ave-Mixed Use-2021 Update

APPENDIX A
UPDATED (2021) LIQUEFACTION MATERIALS

TABLE 1 - LIQUEFACTION ANALYSIS
(Youd et al. 2001)

PROJECT NAME: 1201 San Pablo Avenue
PROJECT NUMBER: 501-4D
DATE: 3/14/2021
BY: JRL

Highlighted information must be entered depending upon site specific conditions.

C_R = Correction Factor for Rod Length; 10-13' = 0.75; 13-20' = 0.85; 20-33' = 0.95; 33-99' = 1.0
C_S = Correction Factor for Sampling Method; SPT Sampler without liners = 1.2; Modified California Sampler = 0.63
C_B = Correction Factor for Borehole Diameter; For Hollow Stem, use ID.; 2.5-4.5" = 1.0; 5.9" = 1.05; 7.9" = 1.15
C_E - Correction Factor for Hammer Energy Ratio; Automatic Trip Hammer = 1.2, Rope and Cathead = 1.0
C_N - Correction Factor for Overburden Pressure

References:
Seed, R.B., et al., 2003, "Recent Advances in Soil Liquefaction Engineering: A Unified and Consistent Framework," American Society of Civil Engineers, Specialty Conference, April 30, 2003.
Youd, T. Leslie, et al.,2001, "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils."
Tokimatsu and Seed, 1987, Evaluation of settlements in sand due to earthquake shaking, Journal of Geotechnical Engineering, Vol. 113, No.8. (pg 405 in geotechnical earthquake engineering textbook)

Design M_w = 7.5
Water table = 10.0 ft below ground surface

Boring 2004-1

Youd, 2001																													Tokimatsu and Seed (1987)		
Depth at Center of Layer (ft)	*Unified Soil Class.*	*Layer Thickness (ft)*	*% Fines*	*N*	C_R	C_S	C_B	C_E	N_{60}	γ_m	σ_v	σ_v'	C_N	$N_{1,60}$	$N_{1,60cs}$			R_D	a_{max}	$(\tau_{max})_r$	$CSR_{,Mw}$	*MSF*	$CSR_{,M=7.5}$	$CRR_{,M=7.5}$	FS	Liq	$N_{1,60,cs}$	ε (%)	Settlement (in)		
12.5	GW-GC	5	12	17	0.85	0.63	1.00	1.00	9	120	1500	1344	1.25	11	--	13	--	0.97	0.944	1379	0.667	1.00	0.667	0.140	0.209	yes	13	2.1	1.26		
17.5	SC	5	38	17	0.85	0.63	1.00	1.00	9	120	2100	1632	1.14	10	--	--	17	0.96	0.944	1909	0.760	1.00	0.761	0.181	0.238	yes	17	1.8	1.08		
22	GW-GC	4	16	34	0.95	0.63	1.00	1.00	20	120	2640	1891	1.06	21	--	25	--	0.95	0.944	2371	0.815	1.00	0.815	0.290	0.356	yes	25	1.2	0.58		

Total Settlement (in): 2.92

Boring 2004-2

Youd, 2001																													Tokimatsu and Seed (1987)		
Depth at Center of Layer (ft)	*Unified Soil Class.*	*Layer Thickness (ft)*	*% Fines*	*N*	C_R	C_S	C_B	C_E	N_{60}	γ_m	σ_v	σ_v'	C_N	$N_{1,60}$	$N_{1,60cs}$			R_D	a_{max}	$(\tau_{max})_r$	$CSR_{,Mw}$	*MSF*	$CSR_{,M=7.5}$	$CRR_{,M=7.5}$	FS	Liq	$N_{1,60,cs}$	ε (%)	Settlement (in)		
14	SC	6	46	14	0.85	1.20	1.00	1.00	14	120	1680	1430	1.22	17	--	--	25	0.97	0.944	1539	0.700	1.00	0.700	0.300	0.429	yes	25	1.2	0.86		
19	GW-GC	4	15	18	0.85	1.20	1.00	1.00	18	120	2280	1718	1.11	20	--	23	--	0.96	0.944	2065	0.781	1.00	0.781	0.264	0.338	yes	23	1.4	0.67		
23.5	GW-GC	5	15	16	0.95	1.20	1.00	1.00	18	120	2820	1978	1.03	18	--	21	--	0.95	0.944	2521	0.828	1.00	0.829	0.233	0.281	yes	21	1.5	0.90		

Total Settlement (in): 2.44

TABLE 1 - LIQUEFACTION ANALYSIS
(Youd et al. 2001)

PROJECT NAME: 1201 San Pablo Avenue
PROJECT NUMBER: 501-4D
DATE: 3/14/2021
BY: JRL

Highlighted information must be entered depending upon site specific conditions.

C_R = Correction Factor for Rod Length; 10-13' = 0.75; 13-20' = 0.85; 20-33' = 0.95; 33-99' = 1.0
C_S = Correction Factor for Sampling Method; SPT Sampler without liners = 1.2; Modified California Sampler = 0.63
C_B = Correction Factor for Borehole Diameter; For Hollow Stem, use ID.; 2.5-4.5" = 1.0; 5.9" = 1.05; 7.9" = 1.15
C_E - Correction Factor for Hammer Energy Ratio; Automatic Trip Hammer = 1.2, Rope and Cathead = 1.0
C_N - Correction Factor for Overburden Pressure

References:

Seed, R.B., et al., 2003, "Recent Advances in Soil Liquefaction Engineering: A Unified and Consistent Framework," American Society of Civil Engineers, Specialty Conference, April 30, 2003.
Youd, T. Leslie, et al.,2001, "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils."
Tokimatsu and Seed, 1987, Evaluation of settlements in sand due to earthquake shaking, Journal of Geotechnical Engineering, Vol. 113, No.8. (pg 405 in geotechnical earthquake engineering textbook)

Design M_w = 7.5
Water table = 10.0 ft below ground surface

Boring 2018-1

Youd, 2001																											Tokimatsu and Seed (1987)		
Depth at Center of Layer (ft)	*Unified Soil Class.*	*Layer Thickness (ft)*	*% Fines*	*N*	*C_R*	*C_S*	*C_B*	*C_E*	N_{60}	*γ_m*	σ_v	σ_v'	C_N	$N_{1,60}$		$N_{1,60cs}$		R_D	*a_{max}*	$(\tau_{max})_r$	$CSR_{,Mw}$	*MSF*	$CSR_{,M=7.5}$	$CRR_{,M=7.5}$	FS	Liq	$N_{1,60,cs}$	*ε (%)*	Settlement (in)
11	SC	6	25.2	22	0.75	1.20	1.15	1.20	27	120	1320	1258	1.30	35	--	43	--	0.98	0.944	1217	0.629	1.00	0.629	--	--	no	43		0.00
17	SC	6	13.3	31	0.85	1.20	1.15	1.20	44	120	2040	1603	1.15	50	--	54	--	0.96	0.944	1857	0.753	1.00	0.753	--	--	no	54		0.00
35.5	SC	5	27.4	19	1.00	1.20	1.15	1.20	31	120	4260	2669	0.89	28	--	36	--	0.89	0.944	3567	0.869	1.00	0.869	--	--	no	36		0.00

Total Settlement (in): 0.00

Boring 2018-2

Youd, 2001																											Tokimatsu and Seed (1987)		
Depth at Center of Layer (ft)	*Unified Soil Class.*	*Layer Thickness (ft)*	*% Fines*	*N*	*C_R*	*C_S*	*C_B*	*C_E*	N_{60}	*γ_m*	σ_v	σ_v'	C_N	$N_{1,60}$		$N_{1,60cs}$		R_D	*a_{max}*	$(\tau_{max})_r$	$CSR_{,Mw}$	*MSF*	$CSR_{,M=7.5}$	$CRR_{,M=7.5}$	FS	Liq	$N_{1,60,cs}$	*ε (%)*	Settlement (in)
16	SC	8	28.6	23	0.85	1.20	1.15	1.20	32	120	1920	1546	1.17	37	--	47	--	0.97	0.944	1752	0.737	1.00	0.737	--	--	no	47		0.00
21.75	SM	3.5	15.6	42	0.95	1.20	1.15	1.20	66	120	2610	1877	1.06	70	--	76	--	0.95	0.944	2346	0.812	1.00	0.813	--	--	no	76		0.00
25.25	SM	3.5	15.6	39	0.95	1.20	1.15	1.20	61	120	3030	2078	1.01	61	--	67	--	0.94	0.944	2691	0.842	1.00	0.842	--	--	no	67		0.00
29.5	SC	5	40.9	12	0.95	1.20	1.15	1.20	19	120	3540	2323	0.95	18	--	--	27	0.92	0.944	3085	0.863	1.00	0.863	0.328	0.380	yes	27	1.1	0.68

Total Settlement (in): 0.68

Boring 2018-3

Youd, 2001																											Tokimatsu and Seed (1987)		
Depth at Center of Layer (ft)	*Unified Soil Class.*	*Layer Thickness (ft)*	*% Fines*	*N*	*C_R*	*C_S*	*C_B*	*C_E*	N_{60}	*γ_m*	σ_v	σ_v'	C_N	$N_{1,60}$		$N_{1,60cs}$		R_D	*a_{max}*	$(\tau_{max})_r$	$CSR_{,Mw}$	*MSF*	$CSR_{,M=7.5}$	$CRR_{,M=7.5}$	FS	Liq	$N_{1,60,cs}$	*ε (%)*	Settlement (in)
20.5	SC	5	12.6	45	0.95	1.20	1.15	1.20	71	120	2460	1805	1.08	76	--	80	--	0.96	0.944	2219	0.799	1.00	0.799	--	--	no	80		0.00
25.5	SC	5	12.6	25	0.95	1.20	1.15	1.20	39	120	3060	2093	1.01	39	--	42	--	0.94	0.944	2715	0.843	1.00	0.844	--	--	no	42		0.00

Total Settlement (in): 0.00

APPENDIX B
2018 SUPPLEMENTAL GEOTECHNICAL INVESTIGATION

SUPPLEMENTAL GEOTECHNICAL INVESTIGATION
1201 SAN PABLO AVENUE DEVELOPMENT
BERKELEY, CALIFORNIA

ALAN KROPP
& ASSOCIATES, INC.
GEOTECHNICAL
CONSULTANTS

40th YEAR ANNIVERSARY

ALAN KROPP, CE, GE
JAMES R. LOTT, CE, GE
JEROEN VAN DEN BERG, CE
THOMAS M. BRENCIC, CE
SANG HEE CHO

August 7, 2018
501-4C, L-31462

JianYi Investment & Development, Inc.
4500 Great American Parkway
Santa Clara, CA 95054

RE: Supplemental Geotechnical Investigation
1201 San Pablo Avenue Development
Berkeley, California

Dear Mr. Mason and Jing:

In accordance with your authorization, this firm has performed a supplemental geotechnical investigation for the proposed mixed-use retail/residential building to be constructed on the currently vacant parcel located at 1201 San Pablo Avenue in Berkeley, California. This location is shown on the attached Vicinity Map, Figure 1. The approximate site coordinates are Latitude: 37.8821 degrees North; Longitude: 122.2959 degrees West.

The following geotechnical report and supplemental reports/letters were prepared by our office for the previously proposed development on the property.

- Geotechnical Investigation Report, 1201 San Pablo Avenue Development, Berkeley, California, dated December 17, 2004;
- Update of Geotechnical Investigation Report, 1201 San Pablo Avenue Development, Berkeley, California, dated March 2, 2005; and
- Response to Peer Review Comments, 1201 San Pablo Avenue Development, Berkeley, California, dated May 31, 2005.

1.00 PROPOSED CONSTRUCTION

Based on our conversations with you and our review of the preliminary architectural drawings, we understand that the currently proposed project will consist of the construction of a 5-story mixed-use retail/residential building on the property. The new building will encompass almost the entire site and have one level of below grade parking beneath the entire building footprint that will extend to a depth of approximately 9 feet below the existing site grade. The ground floor of the building will consist of retail space fronting on San Pablo Avenue, a residential lobby, a multi-purpose room, a community room, a waste management room, and bike parking. The below grade parking will be accessed by a ramp from Harrison Street. Site improvements will consist of a walkway along the eastern side of the building.

2.00 PURPOSE

The purpose of our supplemental geotechnical investigation was to evaluate the general geotechnical suitability of the site for the proposed mixed-use building from a geotechnical engineering standpoint, and to provide geotechnical engineering recommendations for the proposed work.

3.00 SCOPE OF SERVICES

3.01 Special Report Requirement – Soil Liquefaction Evaluation

Review of the official seismic hazard map for this area prepared by the California Geological Survey (CGS, 2003) indicates that the site is within a mapped zone for which an evaluation of soil liquefaction is required. The California Geological Survey (CGS) has developed official guidelines for conducting geologic/geotechnical investigations within state-defined seismic hazard zones, which are presented in CGS Special Publication 117A (CGS, 2008). Essentially, CGS Special Publication 117A (SP117A) provides for two levels of investigation in evaluating potential soil liquefaction hazards:

- A *Screening Investigation of Liquefaction Potential* is used to determine whether a given site has "obvious indicators of a low potential for liquefaction failure". If this is the case, then a screening-level investigation is often sufficient to satisfy the site-specific soil liquefaction evaluation requirement.

- A *Quantitative Evaluation of Liquefaction Resistance* is required when the less rigorous screening investigation cannot demonstrate the absence of seismic hazards at a site. The quantitative evaluation typically involves determining whether soil liquefaction will occur, evaluating potential liquefaction-related effects and providing recommendations for their mitigation.

We note that SP117A presents "guidelines" and does not prescribe specific requirements for site investigations. The guidelines presented are therefore subject to interpretation by the owner's geotechnical engineer, local officials, and third-party reviewers involved in the project approval process.

3.02 Scope of Services

As outlined in our proposal dated June 1, 2018, the scope of our work to accomplish the stated purpose included the following:

1. A reconnaissance of the site in order to confirm the current site conditions so that we may address any significant change in conditions from the time of our 2004/2005 project reports;

2. A review of relevant published geologic materials that may have been published since the time of our 2004/2005 project reports/letters;

3. A supplemental subsurface exploration program consisting of the drilling of three exploratory test borings to further evaluate the subsurface materials and groundwater level(s) at the site;

4. Laboratory index, classification, and strength tests on surface and subsurface samples from the site, as required, to evaluate the properties of the materials recovered;

5. Geotechnical engineering analyses of the collected data including an updated evaluation of liquefaction potential at the site to address current code requirements;

6. Review of our previous project recommendations in light of current (2016) California Building Code requirements and development of supplemental geotechnical project recommendations as may be appropriate; and

7. Preparation of this supplemental geotechnical investigation report for the site which presents the results of our studies and provides geotechnical design and construction criteria for the geotechnical aspects of the proposed project.

The scope of our services did not include an environmental assessment or investigation for the presence of hazardous or toxic materials in the soil, groundwater, or air on, below, or around this site. An evaluation of the potential presence of sulfates in the soil, or other possibly corrosive, naturally occurring elements was also beyond our scope.

4.00 SITE INVESTIGATION

4.01 Surface Reconnaissance Visit and Existing Site Conditions

A surface reconnaissance visit was performed on June 13, 2018, to make observations of the surficial conditions present, to check the site for drilling access, to mark the proposed boring locations, and to note whether any obvious geotechnical concerns were exposed.

The subject site is roughly rectangular in shape and has maximum plan dimensions of about 100 feet by 130 feet. The property is bordered by San Pablo Avenue and Harrison Street on the west and north, respectively, existing residences to the east, and an apartment complex to the south. The site is relatively level and consists of a vacant lot covered with a low growth of weeds. An approximately 2- to 3-foot-high concrete wall is located along the eastern property line.

Based on our observations, it does not appear that the site has changed significantly since the time of our 2004/2005 investigation and no obvious geotechnical concerns were observed during our site reconnaissance visit.

4.02 Geologic and Seismic Setting

The site is located in the Coast Ranges Geomorphic Province of Northern California, which is characterized by a series of generally northwest-trending faults and folds. The Bay Area experienced uplift and faulting in several episodes during late Tertiary time (about 25 to 2 million years ago) producing the San Francisco Peninsula, the Berkeley Hills, and the intervening structural depression that is now San Francisco Bay. The geology of the area is strongly influenced by the San Andreas fault system, which includes a number of northwest-trending right-lateral faults. Deposition from the Pleistocene (1.85 million years ago) to the present was closely related to global fluctuations in sea level primarily related to glaciations, but also related to local tectonic movement.

The San Francisco Bay Region is recognized as being seismically active. Principal active faults in the Bay Area include the San Andreas, Hayward-Rogers Creek, Calaveras, San Gregorio, Concord, Green Valley and Mount Diablo (blind thrust) faults. Earthquakes occurring along these faults are capable of generating strong ground shaking at the site. The faults capable of producing the strongest ground shaking at the site are the San Andreas and the Hayward-Rogers Creek. The site is about 1.7 miles southwest of the Hayward fault and 17.3 miles northeast of the San Andreas fault.

Since 1800, five earthquakes have occurred in the Bay Area having estimated Moment Magnitudes greater than 6. In 1836, an earthquake with a Moment Magnitude of approximately 6.3 occurred east of Monterey Bay. In 1838, an earthquake having a Moment Magnitude of about 7.5 occurred on the San Andreas Fault. In 1868, an earthquake occurred on the southern section of the Hayward fault that had an estimated Moment Magnitude of 6.8. The San Francisco Earthquake of 1906 with an estimated Moment Magnitude of about 7.9 created a 250-mile-long surface rupture along the San Andreas fault extending from San Juan Bautista to Shelter Cove. The most recent earthquake to significantly affect the Bay Area was the 1989 Loma Prieta event with an estimated Moment Magnitude of 6.9. Smaller earthquakes that would have been felt at the site include the 1957 Daly City earthquake with an estimated magnitude of about 5.4.

Studies by the United States Geological Survey's Working Group on California Earthquake Probabilities (Aagaard et al., 2016) have estimated a 72 percent probability of at least one magnitude 6.7 or greater earthquake occurring in the San Francisco Bay Region before the year 2043. As part of their prediction, they estimated the probability to be 33 percent for a magnitude 6.7 or greater earthquake to occur on the Hayward-Rodgers Creek fault, 22 percent for a magnitude 6.7 or greater earthquake to occur on the Northern San Andreas fault, and 16 percent for a magnitude 6.7 or greater earthquake to occur on the Concord fault during the same period.

4.03 Existing Geotechnical Data Review

A variety of published sources were reviewed to evaluate geotechnical data relevant to the subject parcel. These sources included geotechnical literature, reports, and maps published by various public agencies. Maps which were reviewed included topographic and geologic maps prepared by the United States Geological Survey, as well as geologic and fault maps prepared by the California Geological Survey (formerly the California Division of Mines and Geology). A list of the published sources used in our investigation is presented at the end of this report.

A historic map of the area (Thompson and West, 1878) shows former marshlands at the Bay margin extending only as far inland as 1st or 2nd Street (about 10 blocks to the west of the site), and shows Cordonices Creek crossing San Pablo Avenue about one block to the north of the site. The recent creek and watershed map of Oakland and Berkeley published by the Oakland Museum (Sowers, 1993) also shows Cordonices Creek in approximately the same location as mapped by Thompson and West; however, the map indicates that an underground culvert/storm drain is located to the north of the site, between Cornell Avenue and San Pablo Avenue, suggesting that the creek has been rerouted into a culvert/storm drain and the former creek channel has been filled.

The topographic map for this area (the Richmond Quadrangle), prepared by the United States Geological Survey, indicates the site is located at an elevation of approximately 45 feet above mean sea level on the flatlands between the San Francisco Bay and Berkeley Hills.

A widely used geologic map of the area (Diblee, 1980) indicates the site is underlain by alluvial deposits of Quaternary age (younger than 1.85 million years). A more recent geologic map by Helley and Graymer (1997) maps the site as being underlain by alluvial fan deposits of Holocene age (younger than 11,000 years). The map indicates the material consists of medium dense to dense, gravely sand or sandy gravel that generally grades upward to sandy or silty clay.

Regional mapping of faults (USGS/CGS, 2006) indicates the site is approximately 1.7 miles southwest of the nearest active trace of the Hayward fault, 14.0 miles northwest of the nearest active trace of the Calaveras fault, 15.4 miles southwest of the nearest active trace of the Concord fault, and 17.3 miles to the northeast of the nearest active trace of the San Andreas fault. The site is not located within any Alquist-Priolo Earthquake Fault Zone designated by the State of California.

The California Geological Survey has released a map of this area which indicates areas that may be prone to earthquake-induced ground failure (landsliding and/or liquefaction) during a major earthquake. The map (CGS, 2003) indicates that sufficient concern exists in the designated areas to merit a site-specific evaluation, not necessarily that the hazard is actually present. The subject site is located within a mapped zone for which an evaluation of soil liquefaction is required. The eastern margin of the mapped earthquake-induced liquefaction zone is approximately one block to the east of the site. A liquefaction susceptibility map prepared by the Association of Bay Area Governments (ABAG, 2004) indicates that the site is located within an area of “moderate” susceptibility.

A professional paper on earthquake damage from the 1906 San Francisco earthquake (Youd and Hoose, 1978) suggests that there are no documented cases of liquefaction-induced ground failures having occurred in Berkeley as a result of the 1906 event. A 1990 report on the geotechnical aspects of the Loma Prieta Earthquake (Seed, et al.) suggests that documented occurrences of soil liquefaction in Berkeley associated with the Loma Prieta Earthquake were confined to areas that are underlain by fill (the Berkeley Marina and areas surrounding Highway 80 west of Aquatic Park). In general, the references that we reviewed do not include any reported incidents of liquefaction in the general project vicinity

4.04 Subsurface Exploration

On November 30, 2004, and June 20, 2018, we explored subsurface conditions at the project site by drilling a total of five exploratory test borings at the approximate locations shown on the attached Site Plan, Figure 2. The exploratory borings on Figure 2 are labeled by the initials of our firm followed by the year the borings were drilled and the original exploratory identification number. For example, our Boring 1 drilled in 2004 is labeled AKA 2004-1.

The test borings were drilled using a truck-mounted drill rig equipped with 6½-inch (2004) and 8-inch (2018) outside diameter (O.D.) hollow stem augers. During drilling, our field representative monitored the advancement of the augers and made notes of any changes in drilling conditions that were observed or commented on by the driller. Prior to advancing the augers, a plug was inserted to the bottom of the auger to prevent soil from entering the hollow stem. Special care was exercised when removing the plug prior to sampling. During sampling, the depth to the bottom of the hole was checked to verify that soil did not enter the bottom of the hollow stem after the plug was withdrawn (a phenomenon commonly referred to as heaving). Soil samples were obtained using 2-inch O.D. Standard Penetration Test (SPT) and 3-inch O.D. Modified California samplers. The samplers used during the drilling were driven using a 140-pound hammer falling 30 inches using an automatic-trip hammer. The hammer blows required to drive the sampler the final 12 inches of each 18-inch drive are presented on the attached boring logs. Approximate measurements of the unconfined strength of selected soil samples were performed during the drilling operations using a pocket penetrometer testing device.

Detailed descriptions of the materials encountered in the borings are found on the boring logs presented at the end of this report. A Key to Exploratory Boring Logs, Figure 5, is also presented. The attached logs and related information depicts subsurface conditions only at the specific locations shown on the Site Plan and on the particular date designated on the logs. The logs may have been modified from the original logs recorded during drilling as a result of further study of the collected samples, laboratory tests, or other efforts. Also, the passage of time may result in changes in the subsurface conditions due to environmental changes. The locations of the boring were approximately determined by hand tape measurement from existing field landmarks and the ground surface elevations at each boring location were approximately determined by interpolation of topographic map contours. The location and elevation should be considered accurate only to the degree implied by the method used.

A brief summary of the subsurface materials underlying the site that were encountered during our 2004 and 2018 subsurface explorations is provided below.

4.04.1 2004 Subsurface Exploration

On November 30, 2004, we initially explored the subsurface conditions at the project site by drilling two exploratory test borings. Boring 2004-1 was drilled along the north central portion of the site to a depth of approximately 50½ feet below the existing site grade. Boring 2004-2 was drilled along the south central portion of the site to a depth of approximately 33½ feet below the existing site grade. The 2004 boring logs are included in the attached Appendix A.

The surficial materials encountered in the 2004 exploratory borings generally consisted of stiff, lean to fat, silty and sandy clay that extended to depths of between 10 and 11 feet. We did not observe foreign materials or layering within the near-surface soils that would suggest these materials are fill. Underlying the surficial clayey soils, alternating layers of loose to medium-dense clayey sands and loose to medium-dense clayey gravels were encountered that extended to depths of approximately 24 to 26 feet. Below the clayey gravel and sands, stiff to hard clayey soils that extended to the maximum depth explored was encountered in the borings.

4.04.2 2018 Subsurface Exploration

On June 20, 2018, we performed our supplemental subsurface exploration of the site by drilling three additional test borings to depths of approximately 41½ feet below the existing site grades. Boring 2018-1 was drilled along the east central portion of the site, Boring 2018-2 was drilled near the southwest corner of the site, and Boring 2018-3 was drilled near the northwest corner of the site. The 2018 boring logs are included in the attached Appendix B.

The surficial materials encountered in the 2018 exploratory borings generally consisted of stiff to hard, lean to fat clay with varying amounts of sand and gravel that extended to depths of between 8 and 14 feet. We did not observe foreign materials or layering within the near-surface soils that would suggest these materials are fill. Underlying the surficial clayey soils, alternating layers of medium dense to dense, clayey and silty sand and stiff, sandy clay were encountered that extended to depths of approximately 32 to 38 feet. Below the clayey and silty sand and sandy clay, stiff to hard clayey soils that extended to the maximum depth explored of 41.5 feet was encountered in the borings.

Monitoring during drilling by our field engineer and comments by the driller did not provide any indications that significantly thick layers of clean sand or gravel were encountered during drilling between sampling intervals in either the 2004 or 2018 borings.

4.05 Groundwater Conditions

During our 2004 subsurface exploration, groundwater was encountered in Borings 2004-1 and 2004-2 at a depth of approximately 14 feet below the existing site grade at the time of drilling. Immediately after drilling, groundwater was measured in Boring 2004-2 at a depth of 9 feet below grade. Boring 2004-1 was left open following drilling to allow groundwater levels to stabilize prior to taking a final measurement. Groundwater was present at a depth of approximately 9 feet 6 inches in Boring 2004-1 approximately 4 hours after drilling.

At the time of our recent subsurface exploration, groundwater was encountered at a depth of 20 feet in Boring 2018-1, Boring 2018-2, and Boring 2018-3 at the time of drilling. Slotted PVC pipes were installed within the three boreholes following the completion of drilling to prevent caving and/or sloughing of the sides of the borehole and to facilitate monitoring of the groundwater level(s) at the site overnight. Upon our return to the site the following day, a second groundwater measurement was made prior to removing the casing and backfilling of the borings. At the time of the second groundwater measurement, water was observed at a depth of 9 feet 6 inches in Boring 2018-1 (23 hours later), at a depth of 9 feet in Boring 2018-2 (21 hours later), and at a depth of 9 feet in Boring 2018-3 (19 hours later).

In accordance with drilling requirements for the City of Berkeley, all exploratory borings were grouted with lean cement following the completion of drilling and groundwater level measurements. During the backfilling of the 2018 borings, the PVC pipes installed within the boreholes was used as a tremie pipe for the lean cement and was removed during the grouting operations. It should be noted that groundwater measurements in the borings may have been made prior to allowing a sufficient period of time for the equilibrium groundwater conditions to become established. In addition, fluctuations in the groundwater level may occur due to variations in rainfall, temperature, and other factors not evident at the time the measurements were made.

Based on our experience on similar projects in the site vicinity, the groundwater levels in the area generally ranged between 5 and 15 feet below the ground surface. Historical groundwater contours shown on one of the accompanying maps of the Seismic Hazard Zone Report (CGS, 2003) indicate the approximate groundwater level at site is at a depth of 5 to 10 feet. Fluctuations in the groundwater level at the site may occur daily, seasonally, or over several years due to changes in the weather, and water levels in nearby creeks, and/or culverts.

4.06 Laboratory Testing

Our geotechnical laboratory testing program was directed toward a quantitative and qualitative evaluation of the physical and mechanical properties of the soils underlying the site. The following geotechnical laboratory tests were performed on selected soil samples:

- Water content per ASTM Test Designation D-2216;
- Atterberg Limits per ASTM Test Designation D-4318; and
- Percent passing No. 200 sieve per ASTM Test Designation D-1140.

The tests were conducted in general accordance with the current edition of the referenced standards at the time the tests were performed. The water content tests were performed to evaluate the variations in soil moisture. The Atterberg Limits tests were performed to evaluate the soil’s expansive potential. The results of the percentage passing the No. 200 sieve tests were used to aid in the classification of the soils. The water content, Atterberg Limits, and percentage passing the No. 200 sieve test results were also used in our evaluation of soil liquefaction potential. The results of these tests are presented on the boring logs at the appropriate sample depths.

Atterberg Limits analyses were conducted on samples of the surficial clay soils in Borings 2004-1, 2004-2, 2018-1, and 2018-2. These analyses indicate that the near surface clay material have Liquid Limits (LL) of 45 to 64 and a Plasticity Index (PI) of 28 to 44. This is indicative of highly to critically expansive materials with a corresponding high to critical potential for shrink/swell behavior with changes in moisture content.

The results of grain-size analyses conducted on soil samples underlying the surficial clay materials indicated that the well graded to clayey gravels have between 12 and 16 percent finer than a #200 sieve, the clayey sands have between 12 and 46 percent finer than a #200 sieve, the silty sand has 16 percent finer than a #200 sieve, and the sandy clay has between 53 and 67 percent finer than a #200 sieve.

Atterberg Limits analyses conducted on a sample of the clayey gravels from Boring 2004-1 resulted in a LL of 27 and a PI of 12, while analyses conducted on a sample of the sandy clay from Boring 2018-3 resulted in a LL of 30 and a PI of 12.

5.00 EVALUATIONS AND CONCLUSIONS

5.01 General Site Suitability

Based on our investigation, it is our opinion the site is suitable for the construction of the proposed mixed-use building from a geotechnical standpoint. However, all of the conclusions and recommendations presented in this report should be incorporated in the design and construction of the project to minimize possible geotechnical problems.

The primary considerations for geotechnical design at the site are:

- Seismic considerations;
- Soil liquefaction potential and effects;
- Groundwater considerations;
- Expansive soils;
- Foundation type; and
- Excavations and temporary shoring.

Each of these conditions is discussed individually below.

5.02 Seismic Considerations

The subject site is located in a seismically active region of California. Significant earthquakes in the Bay Area have been associated with movements along well-defined fault zones. Earthquakes occurring along the San Andreas, Hayward, or any of a number of other Bay Area faults have the potential to produce strong ground shaking at the site. During strong earthquakes, various forms of ground failure can occur, such as surface fault rupture, landsliding, and/or seismically-induced liquefaction.

The proposed mixed-use building will very likely experience strong ground shaking during a major earthquake in the life of the structure. The California Building Code has adopted provisions for incorporation of strong ground shaking into the design of all structures. Our recommendations for geotechnical parameters to be used in the structural seismic design of the building are presented in Section 6.05, "California Building Code Seismic Design Parameters."

The site is not within an Alquist-Priolo Special Study Zone (CDMG, 1982) and the referenced geologic and seismic hazard maps do not show any active mapped fault traces or extensions of nearby mapped fault traces that would pass through the site. The closest mapped active fault is the Hayward, which is located near the base of the Berkeley Hills about 1.7 miles northeast of the site. We therefore judge that the likelihood of surface fault rupture directly below the site is considered to be remote. Due to the relatively level topography on the site and in the site vicinity, earthquake-induced landsliding is also not considered a significant site hazard.

Review of the official seismic hazard map for this area prepared by the California Geological Survey (CGS, 2003) indicates that the site is within a mapped zone for which an evaluation of soil liquefaction is required. Liquefaction is the transformation of a deposit of cohesionless soils from a solid state to a liquefied state as a consequence of increased pore pressure and reduced effective stress. Often, this transformation results from the cyclic loading of an earthquake and the soil acquires "mobility" sufficient to permit both horizontal and vertical movements. Soils that are most susceptible to liquefaction are clean, loose, saturated, uniformly graded, fine-grained sands. As noted in SP117A: (1) The vast majority of liquefaction hazards are associated with sandy soils and silty soils of low plasticity, and (2) cohesive soils are generally not considered susceptible to soil liquefaction. The presence of loose and medium dense sands and gravels within the test borings required that we conduct a more detailed evaluation of soil liquefaction potential and effects, as described in the following section.

5.03 Soil Liquefaction Potential and Effects

We evaluated soil liquefaction potential and liquefaction-induced settlement in general accordance with the guidelines published in SP117A (CGS, 2008). As discussed in SP117A, the basic procedure most often used in engineering practice to assess site liquefaction potential is the "Simplified Procedure" originally developed by Seed and Idriss (1970) and subsequently modified by Seed and various other researchers. This method involves assessing the level of earthquake-induced stress required to induce liquefaction in each soil layer and comparing this value to an estimate of earthquake-induced stress that the layer would experience in a design-level earthquake. The data needed to establish the amount of stress needed to induce liquefaction in a soil layer are typically obtained from borings and/or cone penetration tests.

We utilized the recommendations presented in the NCEER/NSF summary report by Youd and Idriss (2001) to evaluate liquefaction susceptibility of coarse-grained soils. In these procedures, liquefaction susceptibility of coarse-grained soils is directly correlated to "corrected" Standard Penetration Test (SPT) blow counts and coarse-grained soils with corrected SPT blow counts of less than approximately 30 are considered susceptible to liquefaction.

More recent research has shown that fine-grained soils (silts and even clays) can also experience liquefaction-type behavior in response to earthquake shaking. We utilized the recommendations presented in the paper by Bray and Sancio (2006) to evaluate liquefaction susceptibility of fine-grained soils. These recommendations correlate liquefaction susceptibility of fine-grained soils to their Plasticity Index (PI), Liquid Limit (LL), and natural water content (w_c) as follows:

- Soils with a PI less than 12 with a w_c greater than 85 percent of the LL are considered susceptible to liquefaction.

- Soils with a PI between 12 and 18 and a w_c greater than 80 percent of the LL are considered potentially susceptible to liquefaction.

- Soils with a PI greater than 18 or a w_c less than 80 percent of the LL are considered NOT susceptible to liquefaction.

Within potentially liquefiable soils layers identified, we estimated the magnitude of liquefaction-induced settlement, using the method outlined by Tokimatsu & Seed (1987). Levels of ground shaking used in our analyses were based on an earthquake moment magnitude (M_W) of 7.1 with a peak ground acceleration of 0.819g. An assumed depth to groundwater of 10 feet was used in our analyses.

The subsurface material as encountered in our site borings within the upper 8 to 14 feet predominately consisted of stiff to hard highly to critically plastic clays, which were judged not to be susceptible to liquefaction. However, underlying the surficial clays we encountered layers of loose to medium dense well graded to clayey gravels, and loose to medium dense clayey sands, which were judged to be potentially liquefiable. Starting at a depth of approximately 26 to 38 feet, stiff to hard clay with varying amounts of silt, sand, and gravel were encountered that we also judge are not susceptible to liquefaction.

The results of our analyses found that ground shaking at the site during a design level earthquake would be large enough to induce liquefaction in the well graded to clayey gravel layers and some of the clayey sand layers underlying the site. In Boring 2004-1, we calculated approximately 2.9 inches of settlement within the well graded to clayey gravel layers and clayey sand layer encountered between a depth of 10 and 24 feet. In Boring 2004-2, we calculated approximately 2.4 inches of settlement within the well graded to clayey gravel layer and clayey sand layer encountered between a depth of 11 and 26 feet. In Boring 2018-2, we calculated approximately 0.7 inches of settlement within the clayey sand layer encountered between a depth of 27 and 32 feet. The results of our liquefaction analyses are presented in the attached Appendix B.

The above estimates suggest approximately 1.0 to 3.0 inches of total liquefaction-induced settlement could occur in portions of the site following an earthquake. We note that these amounts of settlement occurring at the ground surface would likely be partially mitigated by 10 or more feet of overlying non-liquefiable material. Comparisons between the settlement estimates for each boring suggest that between 1.0 and 2.0 inches of differential settlement could occur between the boring locations. With a judgement based maximum liquefaction-induced differential settlement of up to 2.0 inches between borings (spaced roughly 50 to 60 feet apart), we would expect potentially 1.0 inch of differential settlement to occur over a horizontal distance of 20 to 30 feet (angular distortion of 1/240 to 1/360). In order to account for possible differential settlement of the proposed building during a design level earthquake, we recommend the building be supported on a stiffened foundation system comprised of a structural mat designed to tolerate this magnitude of differential settlement.

We judge there to be a generally low risk that large-scale lateral spreading will occur at the site as a result of the design-level earthquake. This judgment is based to a large degree on the observation that the site is situated in an area of relatively level surface topography without a nearby "free-face". However, we anticipate that some lateral deformation of the ground surface may occur as a result of liquefaction-induced movements. In our opinion, it is reasonable to assume that the magnitude of these localized lateral deformations will likely not significantly exceed the values calculated for liquefaction-induced settlement.

5.04 Groundwater Considerations

Groundwater was observed in the exploratory borings drilled at the site at a depth of roughly 9 feet below the existing site grades. Historical groundwater contours shown on one of the accompanying maps of the Seismic Hazard Zone Report (CGS, 2003) indicate an approximate groundwater level at site of 5 to 10 feet. Based upon the information obtained from these sources, as well as our experience working on similar projects in the surrounding area, we judge that a design groundwater level of 5 feet below the ground surface would be appropriate.

With a planned finished below-grade parking floor elevation of approximately 9 feet, we would anticipate that the depth of temporary site excavations during construction may be on the order of 12 to 15 feet below existing grade. At this depth, it appears that groundwater will be encountered in the bottom of the excavation for the below grade parking garage and it should be assumed that temporary construction dewatering will be required for the project. The design and implementation of temporary construction dewatering is considered the sole responsibility of the contractor and/or their dewatering consultant/subcontractor.

It should be assumed that over the life of the below-grade parking garage structure there may be times when groundwater levels temporarily rise above the garage floor. Available options for the design of the parking garage that extend below the groundwater table include: (1) designing the structure to resist lateral and uplift hydrostatic pressures and installing waterproofing along the exterior of these structures, or (2) providing a system of subdrains, underdrains, and pumps that consistently maintain water levels below the bottom of the structure. With a drained system that relies on pumping, there is always the possibility of a pump failure. Therefore, unless the parking area walls and floor are designed to withstand full hydrostatic pressures, we recommend that a pressure relief system be provided such that if there is a pump failure, the garage is allowed to partially flood in order to avoid development of full, unbalanced hydrostatic forces on the structure. Water removed as a result of dewatering during or after construction should be disposed of in accordance with City and County regulations, as well as other governing entities.

The groundwater conditions at the site also may result in excessive moisture transmission through the parking structure walls and slabs and we recommend that appropriate waterproofing of the walls and the floor be installed. A waterproofing expert should be consulted to provide recommendations.

5.05 Expansive Soils

As previously noted, the surficial clayey soils encountered in the borings extended to depths of approximately 8 to 14 feet. The results of three Atterberg Limits determinations conducted on samples of the near-surface material indicate these soils are highly to critically expansive. Expansive soils shrink and swell in response to changes in moisture and have the potential to damage improvements that are supported on them. The proposed mixed-use building will have one level of below-grade parking beneath the entire footprint of the building with the building foundation supported on materials below the zone of shrink/swell behavior.

However, as stated previously, the surface soils can shrink and swell with changes in moisture and can damage near surface elements located directly on them, such as perimeter walkways and/or other exterior flatwork. To help minimize tilting and/or cracking of exterior slabs-on-grade for the project, we recommend these elements be directly underlain by a layer of imported, non-expansive material. Recommendations for exterior slabs-on-grade are presented in Section 6.06, “Exterior Slabs-on-Grade.”

5.06 Foundation Type

We do not believe that geotechnical mitigation involving standard ground improvement methodologies (e.g. removal and replacement, grouting, dynamic densification, etc.) are warranted, give the limited amount of potential liquefaction-induced settlement anticipated. However, we judge that a structural mitigation scheme involving foundations designed to withstand the predicted settlements and lateral deformations can be used to reduce the risk to acceptable levels. Alternate foundation types that can be used to mitigate the potentially damaging effects of liquefaction-induced settlement include: (1) stiffened interconnected spread footings; (2) a structural mat foundation; and (3) deep foundations (drilled piers or piles) that gain support below the zone of liquefiable soils.

As previously stated, in our opinion, a structural mat foundation designed to effectively tolerate the anticipated settlement, span, and cantilever over local irregularities, while preventing excessive structural distress and/or collapse is appropriate for the proposed mixed-use building. Our recommendations for a foundation system comprised of a structural mat are presented in Section 6.03, “Structural Mat Foundation.”

5.07 Excavations and Temporary Shoring

Excavations on the order of 12 to 15 feet below the existing site grades are anticipated as part of this project. Temporary shoring or underpinning may need to be implemented by the contractor to protect adjacent improvements during site excavations. Existing improvements that may be affected by the proposed development include, but are not limited to, adjacent structures, sidewalks, curbs, pavements, and underground utilities. If temporary shoring or underpinning is needed to protect adjacent structures, the contractor is responsible for installation and performance of such measures. However, we recommend that we be given a chance to review the temporary shoring plans for possible interaction with the permanent foundation systems. We also suggest that the contractor thoroughly document the condition of nearby buildings, streets, storm drains, and sewer by video or other measures prior to commencement of site excavations. The contractor should also perform regular surveys during the excavation and construction to monitor and document any observed settlement or deflection of nearby streets, flatwork, and/or structures.

6.00 RECOMMENDATIONS

It is the responsibility of you or your representative to confirm that the recommendations presented in this report are called to the attention of the contractor, subcontractors, and any governmental body which may have jurisdiction and that these recommendations are carried out in the field.

6.01 Site Preparation and Earthwork

6.01.1 Site Clearing and Preparation

The site of the proposed mixed-use building should initially be stripped to sufficient depth to remove surface vegetation and weeds; these materials should be removed from the site. Any excavations required for the removal of any existing fill (such as in an old filled pit or basement) and/or old foundations that are below the planned finished site elevations should be backfilled with engineered fill that is placed and compacted in accordance with the recommendations as contained in Section 6.01.5, "Compaction." Underground utilities that interfere with the proposed construction should be re-routed or abandoned.

6.01.2 Excavations, Temporary Dewatering, and Temporary Shoring

After the site of the mixed-use building has been cleared, the excavation for the below grade parking garage should be made. The proposed mixed-use building will encompass most of the site and vertical or near vertical excavations of 12 to 15 feet in depth are anticipated to create the underground parking structures. Due to the depth of the anticipated excavations and the presence of relatively shallow groundwater at the site, it is anticipated that temporary dewatering and shoring will be required for this project.

Temporary shoring should be used as required to prevent the movement of materials exposed in the face of the excavations. Temporary dewatering should also be installed in order to draw down the ground water to at least 2 to 3 feet below the planned mat slab subgrade. We recommend the excavation and subsequent parking structure construction be continuous in order to minimize the length of time the excavation is exposed. However, since we have no control over the methods and timing used by the contractor, the stability of any excavations is solely the responsibility of the contractor. It is, however, recommended that our firm review the temporary dewatering and temporary shoring plans in order to evaluate the potential interaction between the temporary installations and the permanent structure.

It is anticipated that the majority of earthwork on this project will involve excavation and export of material from the site. However, fill placement operations, such as backfill of utility trench excavations and parking garage retaining wall backfill may be required. The excavated materials can be selectively stockpiled for these uses; however, all excess materials derived from the excavation should be removed from the site.

6.01.3 Subgrade Preparation

The subgrade surface in those areas to receive structural fill (including excavations created from the removal of existing structures), slabs-on-grade or pavements should be confirmed by the project soil engineer to be firm, non-yielding materials. Areas beneath exterior slabs-on-grade and pavements that are to receive non-expansive, select fill should be over-excavated as necessary to accommodate the recommended select fill layer. The exposed soils in those areas receiving non-expansive, select fill or structural fill should be scarified to a depth of 6 inches or the full depth of any existing shrinkage cracks, whichever is deeper. The scarified soils should then be moisture conditioned to 2 to 5 points above optimum water content and compacted to the specified relative compaction indicated in Section 6.01.5, “Compaction.” In areas to receive select fill, the moisture conditioned subgrade should be covered as soon as possible to prevent drying of the subgrade soils.

6.01.4 Material for Fill

All onsite soils having an organic content of less than 3 percent by volume are suitable for use as fill. However, all fill placed at the site, including onsite soil, should not contain rocks or lumps greater than 6 inches in greatest dimension with not more than 15 percent larger than 2.5 inches. Non-expansive select fill, where specified, should meet the requirements for general fill and should be predominantly granular with a plasticity index of 12 or less. All import material should be evaluated by our firm prior to importation to the site.

6.01.5 Compaction

All fill should be placed on a firm, unyielding base surface in lifts not exceeding 8 inches in uncompacted thickness. The fill should be compacted to at least 90 percent relative compaction by mechanical means only as determined by ASTM Test Designation D1557-latest revision. Granular fill should be compacted to at least 95 percent relative compaction. It is possible that exposed soils may be excessively wet or dry depending on the moisture content at the time of construction. If the soils are too wet, then they may be dried by aeration or mixing with drier materials. If the soils are too dry, then they may be wetted by the addition of water or by mixing with wetter materials.

6.01.6 Trench Backfill

Utility trenches should be backfilled with fill placed in lifts not exceeding 8 inches in uncompacted thickness. Native backfill materials should be compacted to at least 90 percent relative compaction (ASTM D1557-latest revision) and granular material should be compacted to at least 95 percent relative compaction (ASTM D1557-latest revision). These compaction recommendations assume a reasonable “cushion” layer around the pipe. If City and/or utility company specifications require more stringent backfill requirements, then those specifications should be followed.

If granular soil is used, sufficient water should be added during the trench backfilling operations to prevent the soil from “bulking” during compaction. All compaction operations should be performed by mechanical means only. We recommend against jetting. If granular backfill is used for utility trenches, we recommend an impermeable plug or mastic sealant be used where utilities enter the building to minimize the potential for free water or moisture to enter below the building.

6.02 Waterproofing

A number of waterproofing methods and details are available that are capable of producing a relatively water-tight below grade structure. Some of the methods that we are aware of include bentonite banels, sprayed-on bentonite, membranes, concrete admixtures, hot-mopped asphalt, and other liquid waterproofing materials. A number of considerations are involved in the design and detailing of a successful waterproofing system. We recommend that a waterproofing consultant be retained to consult regarding the selection, design, and implementation of an appropriate waterproofing system.

6.03 Structural Mat Foundation

We recommend that the foundation for the proposed mixed-use building consist of a reinforced concrete mat slab foundation system. The mat slab should be a minimum of 24 inches thick and the base of the mat slab should extend at least 30 inches below the adjacent ground surface. The mat can be designed assuming the following bearing pressures.

Load Case	Bearing Capacity
Dead Loads Allowable	1,500 psf
Dead + Live Loads Allowable	2,250 psf
Total Allowable	3,000 psf

The above “allowable” values include factors of safety of 3.0, 2.0, and 1.5 for dead loads, dead + live loads, and total loads, respectively. This allowable bearing pressure is a net value; therefore, the weight of the mat can be neglected for design purposes.

The mat foundation should be integrally connected to all portions of the structure so the entire foundation system acts as a unit. The mat should be reinforced with top and bottom steel in both directions to allow the foundation to span local irregularities that may result from potential differential settlement. As a minimum, we recommend that the mat be reinforced with sufficient top and bottom steel to span an edge cantilever of 5 feet and a random interior clear span of 10 feet. A modulus of subgrade reaction of 30 kips per cubic foot can be assumed for mat design.

Lateral loads on the structure may be resisted by passive pressures acting against the sides of the mat. We recommend an allowable passive pressure equal to an equivalent fluid weighing 300 psf per foot of depth (factor of safety ~ 2). Alternatively, an allowable friction coefficient of 0.30 (factor of safety ~ 2) can be used between the bottom of the mat and the subgrade soils. If the perimeter of the mat is poured neat against the soils, the passive pressure and friction coefficient may be used in combination.

Unless the underground garage is designed for hydrostatic uplift pressures, the below-grade parking garage slab/mat should be provided with an underdrain system. Typical recommended details for an underdrain system are illustrated on the attached Figure 3. The underslab drain system should consist of an 8-inch-thick layer of ¾-inch, clean, open-graded drainrock placed below the slab. A series of shallow trenches with perforated collector pipes should be placed at the base of the drainrock layer. The drainrock should be compacted by mechanical means, with care taken not to damage the collector pipes during the compaction efforts. The collector pipes should slope to one or more sump locations where sump pumps are installed. The drainrock should be separated from the native material by an approved non-woven, polyester geotextile such as Mirafi 140N or 140NL, or a 4-ounce equivalent. Prior to construction of the slab, the gravel subgrade should be proof-rolled to provide a smooth, well-compacted surface for slab support.

If the below grade parking garage will be provided with retaining wall and under-slab drainage, the parking garage slab should still be designed for hydrostatic uplift forces (assuming a design groundwater level of 5 feet below surface grade), unless there is a pressure relief system provided for the under-slab drainage system, such that if there is a pump failure, the garage is allowed to partially flood in order to avoid development of full, unbalanced hydrostatic forces.

As previously indicated, it is recommended that a waterproofing consultant should be retained to specify details of the wall and under-slab waterproofing systems. However, where the mat will be surfaced with flooring material, we recommend that the specifications for the mat require moisture emission tests to be performed on the mat prior to the installation of the flooring. No flooring should be installed until safe moisture emission levels are recorded for the type of flooring to be used.

6.04 Retaining Walls

Retaining walls for the underground parking structures should be designed to resist both ultimate (non-factored) lateral earth pressures and any additional lateral loads caused by surcharge loads on the adjoining ground surface. If the below grade parking garage is designed to resist hydrostatic pressures, we recommend walls be designed to resist the equivalent fluid pressure (not including surcharge loading) of 50 pounds per cubic foot (pcf) above the groundwater level and 90 pcf below the groundwater level in order to account for undrained conditions. For retaining wall design, we recommend assuming a design groundwater depth of 5 feet below the currently existing grade. If the below grade parking garage will be provided with retaining wall and under-slab drainage, the walls may be designed to resist an equivalent fluid pressure (not including surcharge loading) of 50 pcf for the full height, assuming that a pressure relief system is provided at the base of the wall, such that if there is a pump failure, the garage is allowed to partially flood in order to avoid development of full, unbalanced hydrostatic forces.

Building walls greater than 6 feet in height should also be designed for a temporary seismic load. The temporary seismic load can be modeled as a uniform lateral pressure applied over the height of the wall of 12H psf, where H is the wall height in feet.

The values given above assume level backfill behind the wall with no surcharge loads. Surcharge loads due to traffic behind the wall can be calculated using a uniform lateral pressure of 100 pounds per square foot (psf) applied over the top 10 feet of the wall. For additional surcharge loads, the design pressure behind the wall should be increased by an additional uniform pressure equivalent to one-half of the maximum anticipated surcharge load applied to the surface behind the wall.

If subsurface drainage is installed behind the parking garage retaining walls, we recommend the installation of either a geo-composite drainage system or a conventional gravel drainage system that meets the minimum requirements outlined below. If a geo-composite drainage system is used, it should consist of a pre-fabricated drainage composite system such as CCW MiraDRAIN 6000/6200 with CCW QuickDRAIN, or an approved equivalent installed in accordance with the manufacturer’s recommendations. If a drainage composite system is to be used, the manufacturer’s specifications for the system should be submitted to our office for review in order to confirm that it is appropriate for the intended use. A pre-fabricated drainage composite system is typically used when shoring is used as the backside form for the wall. However, an acceptable alternative system where there is sufficient space for installation behind the wall would be a gravel subdrain system (see Typical Retaining Wall Subdrain Detail, Figure 4) consisting of a 4-inch, rigid, perforated pipe, bedded in ¾-inch, clean, open-graded rock. The perforated drainpipe should be sloped to drain with a minimum positive gradient of 1 percent. The entire rock/pipe unit should be wrapped in an approved, non-woven, polyester geotextile such as Mirafi 140N or 140NL, or a 4-ounce equivalent. The rock and fabric placed behind the wall should be at least one foot in width and should extend to within one foot of finished grade. The upper one foot of backfill (6 inches for walls less than 5 feet in height) should consist of onsite, compacted, relatively impervious soils (an impermeable plug).

We should note flexible, perforated pipe (flexline), 2000-Pound Crush, Leachfield, and ASTM F810 pipe are not acceptable for use in the subdrain because of the likelihood of damage to the pipe during installation and the difficulty of future cleaning with mechanical equipment without damaging the pipe. We recommend the use of Schedule 40 PVC drainpipe, or equivalent for the drain system. The subdrain pipe should be connected to a system of closed pipes (non-perforated) that lead to suitable discharge facilities. At the location where the perforated subdrain pipe connects with the solid discharge drainpipe, drainrock backfill should be discontinued. A "clay plug" should be constructed out of relatively impervious soils to direct collected water into the perforated pipe and minimize the potential for water collecting around the solid drainpipe and saturating the adjacent soils. We recommend waterproofing be applied to any proposed retaining walls where applicable. The specification of the type of waterproofing and the observation of its installation should be performed by a qualified waterproofing consultant.

In addition to the drainage details noted above, the "high" end and all 90-degree bends of the subdrain pipe should be connected to a riser which extends to the surface and acts as a cleanout. The number of cleanouts can be reduced by installing "sweep" 90-degree bends or pairs of 45-degree bends in succession instead of using "tight" 90-degree bends. "Sweep" 90-degree bends are similar to those used in sanitary sewer pipe connections.

Structural backfill placed behind the retaining walls should be compacted in accordance with the requirements provided in Section 6.01, "Site Preparation and Earthwork." Retaining walls should be supported on the structural mat foundations designed in accordance with Section 6.03, "Structural Mat Foundation."

6.05 California Building Code Seismic Design Parameters

Based on our review of the site location, geology, and the 2016 California Building Code (CBC), we recommend the following parameters be used for seismic design of the building:

- Latitude = 37.8821 Degrees North
- Longitude = 122.2959 Degrees West
- Risk Category = II (assumed)
- Site Class = D
- Mapped Spectral Acceleration for Short Period ($S_{S,}$ Site Class B) = 2.131g
- Mapped Spectral Acceleration for 1-Second Period ($S_{1,}$ Site Class B) = 0.877g
- Maximum Considered Earthquake Spectral Response Acceleration for Short Period ($S_{MS,}$ Site Class D) = 2.131g
- Maximum Considered Earthquake Spectral Response Acceleration for 1-Second Period ($S_{M1,}$ Site Class D) = 1.315g
- Design Spectral Response Acceleration for Short Period ($S_{DS,}$ Site Class D) = 1.420g
- Design Spectral Response Acceleration for 1-Second Period ($S_{D1,}$ Site Class D) = 0.877g

6.06 Exterior Slabs-on-Grade

Due to the highly to critically expansive near-surface soils underlying the site, we recommend any exterior slabs-on-grade be supported on a minimum of 12 inches of imported, compacted, non-expansive fill.

In order to minimize volume change of the subgrade soils, these materials should be scarified to a depth of 6 inches, moisture conditioned to slightly above optimum water content, and compacted to the requirements for structural fill. Prior to the construction of the slabs, the subgrade surface should be proof-rolled to provide a smooth, firm surface for slab support.

The slabs should be structurally independent from the perimeter foundation of the building, and should be free-floating. Score cuts or construction joints should be provided at a maximum spacing of 10 feet in both directions. The slabs should be appropriately reinforced according to structural requirements; concentrated loads may require additional reinforcing. Minor movement of the concrete slab with resulting cracking should be expected. The recommendations presented above, if properly implemented, should help minimize the magnitude of this cracking.

It has been our experience that the installation of wire mesh for slab reinforcement has often not been performed properly during construction of the slab. As a result, we recommend that steel bar reinforcement be used to reinforce any proposed slabs.

6.07 Surface Drainage

We recommend that rainwater collected on the roof of the building be transmitted through gutters and downspouts to closed pipes that connect to a suitable discharge facility. Flexible drain pipe (flexline), 2000 pound crush pipe, leachfield, and ASTM F810 pipe are not recommended for use in these drainage systems because of the likelihood of damage to the pipe during installation due to the weak strength of these pipes. In addition, these drainpipes are sometimes difficult to clean with mechanical equipment without damaging the pipe. We recommend the use of Schedule 40 PVC or SDR 35 PVC drainpipe, or equivalent for the drain system.

Positive surface gradients of at least 2 percent should be provided adjacent to the structures to direct water away from foundations and slabs toward suitable discharge facilities. Ponding of surface water should not be allowed adjacent to the structure or on pavements. Planter areas located next to the building should be avoided. If they are necessary, each planter should contain an area drain and allow for the collection of water.

Provisions for on-site water retention should be developed by the civil engineer to conform to current regulations. Such measures may include retention basins, grassy swales, or other provisions, which may allow some of the water to eventually flow onto the street and into the nearby inlet leading to the storm drain system. We should note that suitable discharge facilities do not include so called “dry wells” and these should be avoided.

7.00 ADDITIONAL GEOTECHNICAL SERVICES

7.01 Supplemental Design Recommendations

If it is decided to design the below grade parking structure to resist hydrostatic pressures and the weight of the building does not offset the hydrostatic uplift forces, tie-downs may need to be considered. Geotechnical recommendations for design of tie-downs were not within our scope of services for this phase of the project. However, we can provide these recommendations (as well as recommendations for other issues that may arise during the design phase) as a future phase of the project, if requested.

7.02 Plan Review

We recommend that our firm be provided the opportunity for a general review of the geotechnical aspects of the design and specifications for the subject work at this site in order that the geotechnical recommendations may be properly interpreted and implemented in the design and specifications. If our firm is not accorded the privilege of making the recommended review, we can assume no responsibility for misinterpretation of our recommendations.

7.03 Construction Observation

The analyses and recommendations submitted in this report are based in part upon the data obtained from the exploratory borings drilled at the site. The nature and extent of variations across the site may not become evident until construction. If variations then become apparent, it will be necessary to re-examine the recommendations of this report.

We recommend our firm be retained to provide geotechnical engineering services during the earthwork, foundation construction, and drainage phases of the work. This is to observe compliance with the design concepts, specifications, and recommendations and to allow design changes in the event that subsurface conditions differ from those anticipated prior to the start of construction.

In order to effectively accomplish our observations during the project construction, we recommend that a pre-construction meeting be held to develop a mechanism for proper communications throughout the project. We also request that the client or client's representative (the contractor) contact our firm at least two working days prior to the commencement of any of the items listed above.

7.04 Wet Weather Construction

Although it is possible for construction to proceed during or immediately following the wet winter months, a number of geotechnical problems may occur which may increase costs and cause project delays. The water content of onsite soils may increase during the winter and rise significantly above optimum moisture content for compaction of subgrade or backfill materials. If this occurs, the contractor may be unable to achieve the recommended levels of compaction without using special measures and would likely have to:

- Wait until the materials are dry enough to become workable; and
- Dispose of the wet soils and import dry soils.

If utility trenches or excavations are open during winter rains, then caving of the trenches or excavations may occur. Also, if the excavations fill with water during construction, or if saturated materials are encountered at the anticipated bottom of the excavations, they may need to be extended to greater depths to reach adequate support capacity than would be necessary if dry weather construction took place.

We should also note that it has been our experience that increased clean-up costs will occur, and greater safety hazards will exist, if the work proceeds during the wet winter months. Furthermore, engineering costs to observe construction are increased because of project delays, modifications, and rework.

8.00 REPORT LIMITATIONS AND CLOSURE

This report has been prepared for the exclusive use of you and your consultants for specific application to the proposed project in accordance with generally accepted geotechnical engineering practices. No other warranty, either expressed or implied, is made. In the event the nature, design, or location of the project differs significantly from what has been noted above, the conclusions and recommendations contained in this report should not be considered valid unless the changes are reviewed and the conclusions of this report modified or verified in writing.

The findings of this report are valid as of the present date. However, the passing of time will likely change the conditions of the existing property due to natural processes or the works of man. In addition, due to new legislation or the broadening of knowledge, changes in applicable or appropriate standards may occur. Accordingly, the findings of this report may be invalidated, wholly or partly, by changes beyond our control. Therefore, this report should not be relied upon after three years without being reviewed by this office.

We trust that this provides the information that you require at this time. If you have any questions concerning this report, please feel free to contact us.

Very truly yours,

REGISTERED PROFESSIONAL ENGINEER THOMAS BRENCIC C66347 EXP. 6/30/20 CIVIL STATE OF CALIFORNIA

Thomas M. Brencic, C.E.
Project Engineer

James R. Lott, G.E.
Associate Engineer

REGISTERED PROFESSIONAL ENGINEER JAMES R. LOTT GE 2137 EXP. 12/31/19 GEOTECHNICAL STATE OF CALIFORNIA

JRL/TB/jc

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Attachments:
Figure 1 - Vicinity Map
Figure 2 - Site Plan
Figure 3 - Typical Retaining Wall Subdrain Detail
Figure 4 - Typical Drain Beneath Slab-On-Grade Detail
Figure 5 - Key To Exploratory Boring Logs

Appendix A - 2004 Exploratory Boring Logs 2004-1 and 2004-2
Appendix B - 2018 Exploratory Borings Logs 2018-1, 2018-2, and 2018-3
Appendix C - Liquefaction Analysis

501-4C 1201 San Pablo Ave - GI Report.docx

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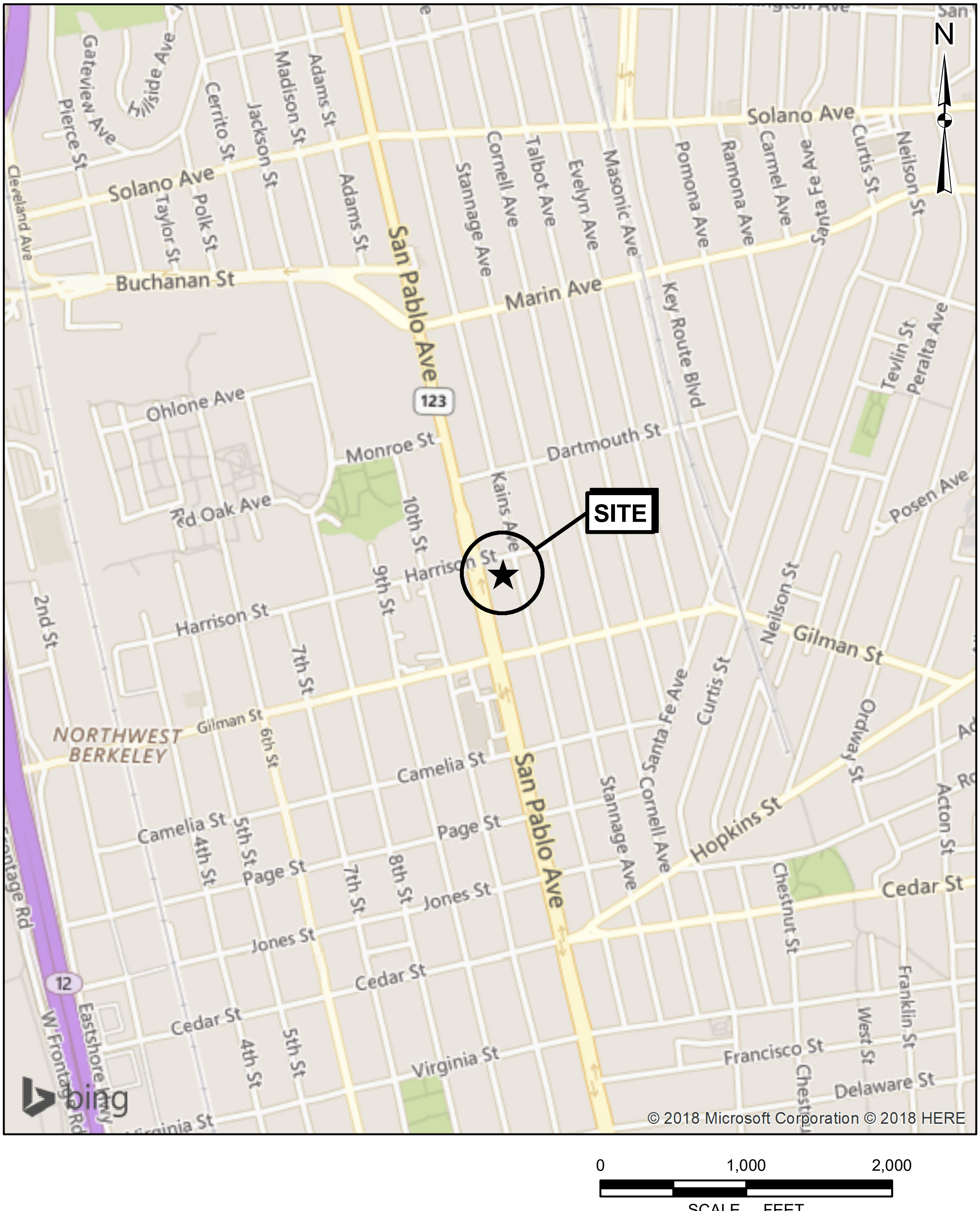

Original figure produced in color.

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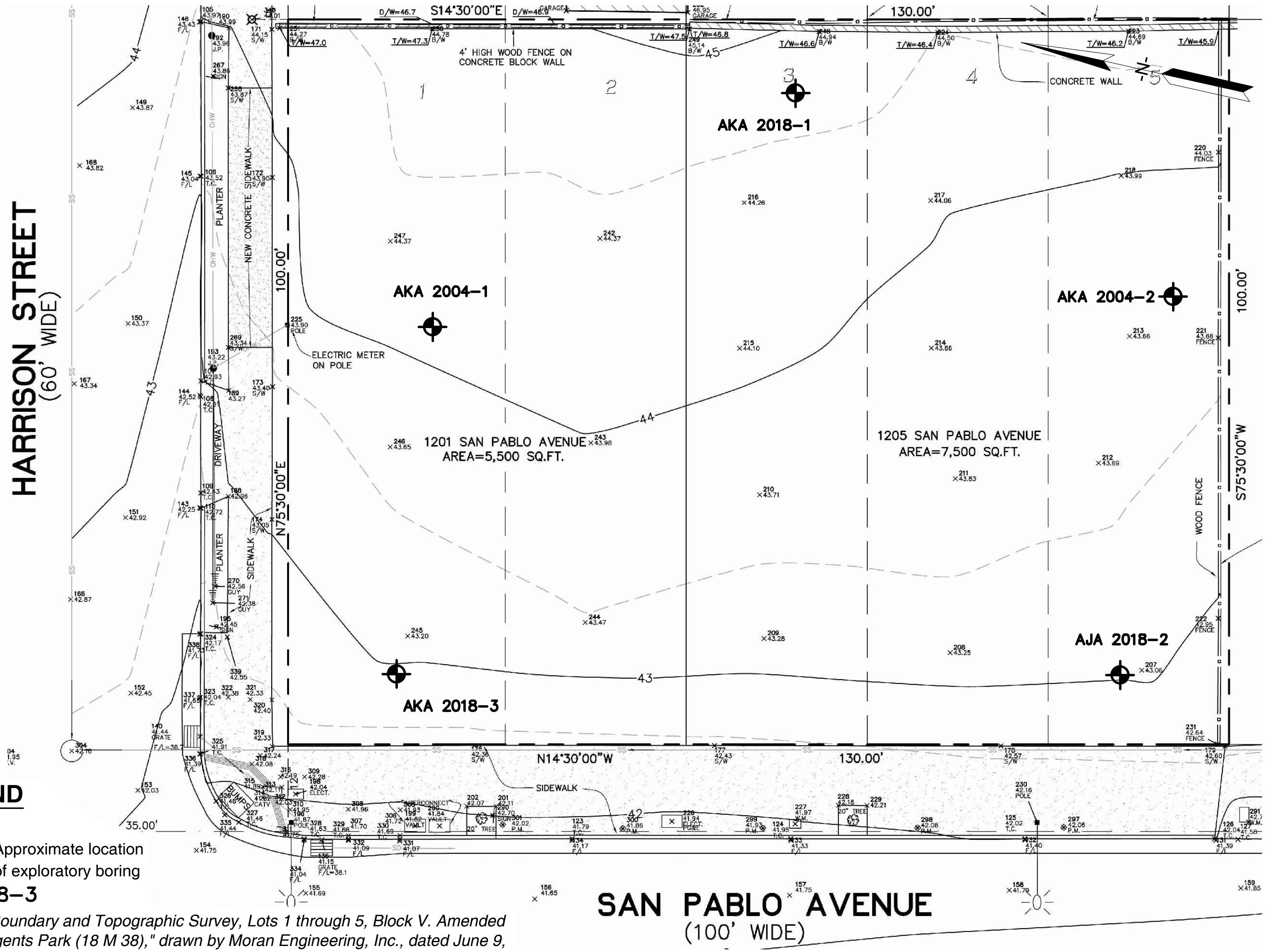
VICINITY MAP		
1201 SAN PABLO AVENUE DEVELOPMENT Berkeley, California		
PROJECT NO.	DATE	FIGURE 1
501-4C	**August 2018**	



LEGEND

Approximate location of exploratory boring

AKA 2018-3

Source: "Boundary and Topographic Survey, Lots 1 through 5, Block V. Amended Map of Regents Park (18 M 38)," drawn by Moran Engineering, Inc., dated June 9, 2016.

10 0 20 40

APPROXIMATE SCALE FEET

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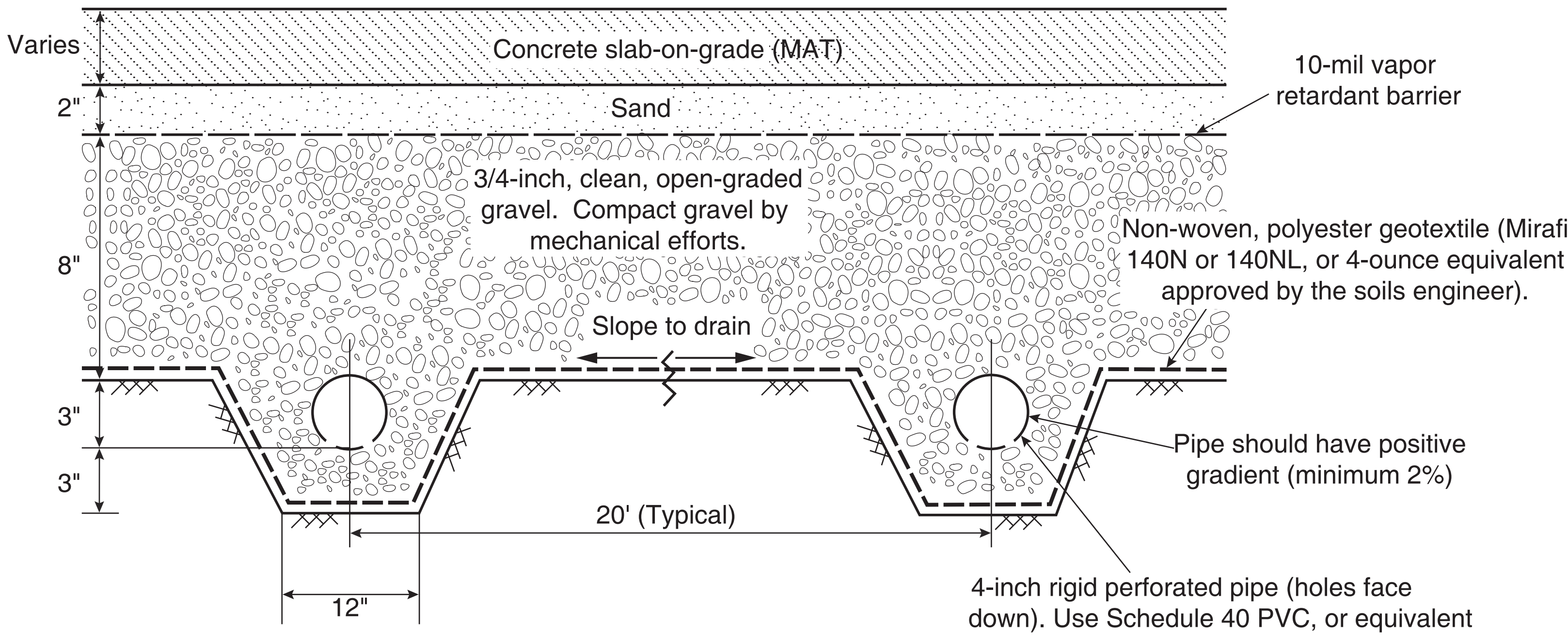
SITE PLAN

1201 SAN PABLO AVENUE DEVELOPMENT
Berkeley, California

PROJECT NO.	DATE	FIGURE 2
501-4C	August 2018	



Notes:
1. Not a structural drawing.
2. All dimensions should be considered minimums.
3. Sand and vapor barrier recommended in areas where minor floor wetness is considered undesirable. For applications where minor floor wetness is acceptable, sand and vapor barrier may be eliminated.
4. This diagram is not intended as a waterproofing barrier. A waterproofing expert should be consulted for waterproofing recomendations and details as required for below groundwater installations.
5. See text for further disscussion of detail.

NOT TO SCALE

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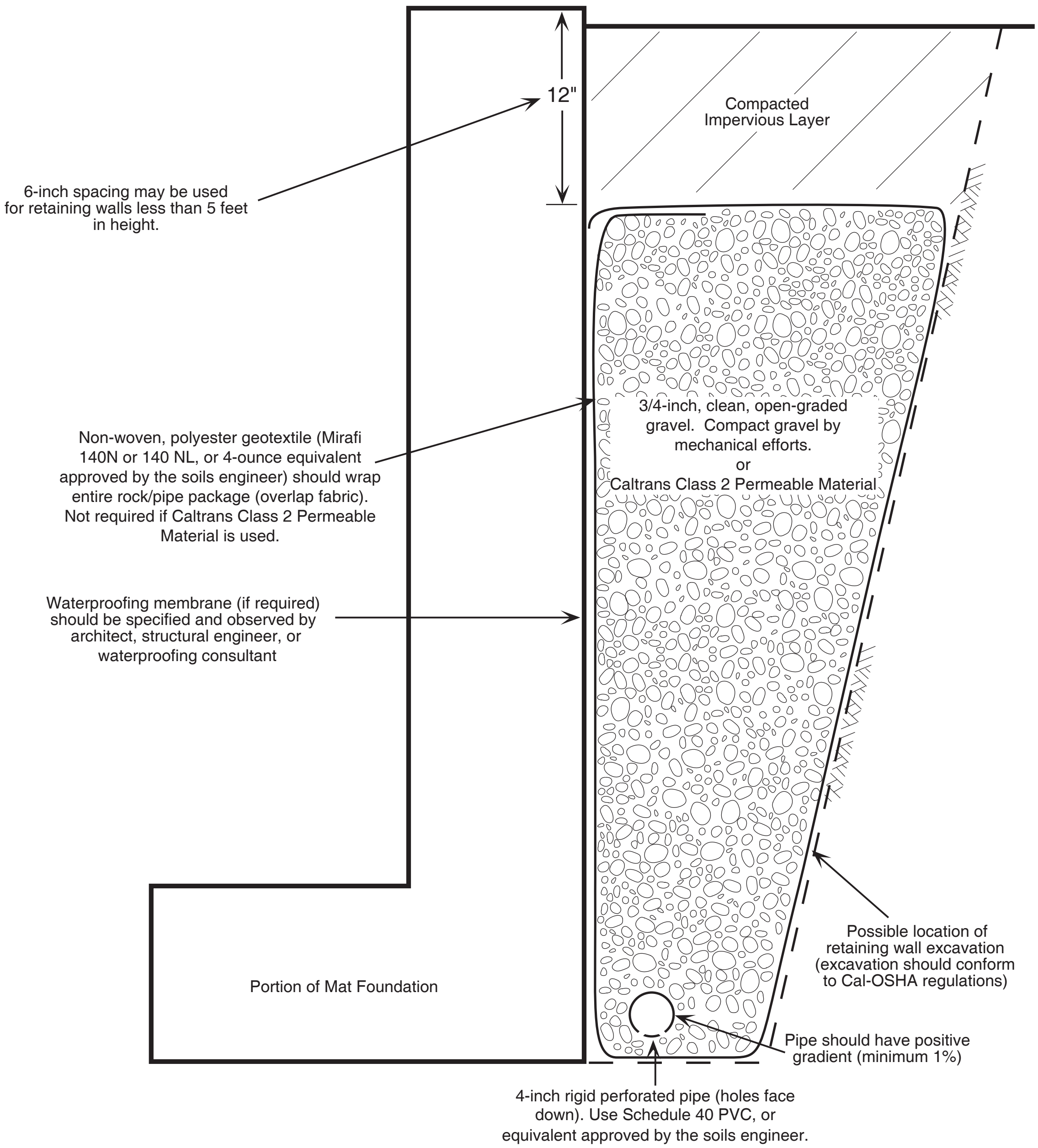
TYPICAL DRAIN BENEATH SLAB-ON-GRADE DETAIL

1201 SAN PABLO AVENUE DEVELOPMENT
Berkeley, California

PROJECT NO.	DATE	FIGURE 3
501-4C	**August 2018**	



NOT TO SCALE

NOTES:

1. Not a structural drawing
2. All dimensions should be considered minimums
3. See text for further discussion of detail

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TYPICAL RETAINING WALL SUBDRAIN DETAIL

1201 SAN PABLO AVENUE DEVELOPMENT
Berkeley, California

PROJECT NO.	DATE	FIGURE **4**
501-4C	**August 2018**	

SOIL CLASSIFICATION CHART

PRIMARY DIVISIONS			SECONDARY DIVISIONS: CRITERIA *	GROUP SYMBOL	GROUP NAME
COARSE-GRAINED SOILS MORE THAN 50% RETAINED ON NO.200 SIEVE	GRAVELS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO.4 SIEVE	CLEAN GRAVELS LESS THAN 5% FINES	$Cu \geq 4$ AND $1 \leq Cc \leq 3$ *A*	GW	Well-graded gravel
			$Cu < 4$ AND/OR $1 > Cc > 3$	GP	Poorly-graded gravel
		GRAVELS WITH FINES - MORE THAN 12% FINES	FINES CLASSIFY AS ML OR MH	GM	Silty gravel
			FINES CLASSIFY AS CL OR CH	GC	Clayey gravel
	SANDS 50% OR MORE OF COARSE FRACTION PASSES NO. 4 SIEVE	CLEAN SANDS LESS THAN 5% FINES	$Cu \geq 6$ AND $1 \leq Cc \leq 3$	SW	Well-graded sand
			$Cu < 6$ AND/OR $1 > Cc > 3$	SP	Poorly-graded sand
		SANDS WITH FINES - MORE THAN 12% FINES	FINES CLASSIFY AS ML OR MH	SM	Silty sand
			FINES CLASSIFY AS CL OR CH	SC	Clayey sand
FINE-GRAINED SOILS 50% OR MORE PASSES THE NO.200 SIEVE	SILTS AND CLAYS LIQUID LIMIT LESS THAN 50%	INORGANIC	PI > 7 AND PLOTS ON OR ABOVE "A" LINE	CL	Lean clay
			PI < 4 OR PLOTS BELOW "A" LINE	ML	Silt
		ORGANIC	$\frac{\text{LIQUID LIMIT - OVEN DRIED}}{\text{LIQUID LIMIT - NOT DRIED}} < 0.75$	OL	Organic Clay & Organic Silt
	SILTS AND CLAYS LIQUID LIMIT 50% OR MORE	INORGANIC	PI PLOTS ON OR ABOVE "A" LINE	CH	Fat clay
			PI PLOTS BELOW "A" LINE	MH	Elastic silt
		ORGANIC	$\frac{\text{LIQUID LIMIT - OVEN DRIED}}{\text{LIQUID LIMIT - NOT DRIED}} < 0.75$	OH	Organic Clay & Organic Silt
	HIGHLY ORGANIC SOILS		PRIMARILY ORGANIC MATTER, DARK IN COLOR, AND ORGANIC ODOR	PT	Peat

REFERENCE: Unified Soil Classification System (ASTM D2487-11)

* Criteria may be done on visual basis, not necessarily based on lab testing

A – $C_u = D_{60}/D_{100}$ & $C_c = (D_{30})^2 / (D_{10} \times D_{60})$

GRAIN SIZES

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

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

ABBREVIATIONS

INDEX TESTS

LL - Liquid Limit (%) (ASTM D4318-17)

PI - Plasticity Index (%) (ASTM D4318-17)

-200 - Passing No. 200 Sieve (%) (ASTM D1140-17)

STRENGTH TESTS

PP - Field Pocket Penetrometer test of unconfined compressive strength (tsf)

TV - Field Torvane test of shear strength (psf)

UC - Laboratory unconfined compressive strength (psf) (ASTM D2166/2166M-16)

TXUU - Laboratory unconsolidated, undrained triaxial test of undrained shear strength (psf) (ASTM D2850-15)

MISCELLANEOUS

ATOD - At time of drilling

psf/tsf - pounds per square foot / tons per square foot

psi - pounds per square inch (indicates relative force required to advance Shelby tube sampler)

SYMBOLS

Standard Penetration Test Split Spoon (2-inch O.D.)

Modified California Sampler (3-inch O.D.)

Thin-walled Sampler Tube (either Pitcher or Shelby) (3-inch O.D.)

Rock Core

Bag Sample

Groundwater Level during drilling

Groundwater Level after drilling

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KEY TO EXPLORATORY BORING LOGS

1201 SAN PABLO AVENUE DEVELOPMENT
Berkeley, California

PROJECT NO.	DATE	FIGURE 5
501-4C	**August 2018**	

APPENDIX A
2004 BORING LOGS

DRILL RIG: Continuous Hollow Flight Auger	SURFACE ELEVATION: Not Measured	LOGGED BY: TB
DEPTH TO GROUNDWATER: See Notes	BORING DIAMETER: 6-1/2 inches	DATE DRILLED: 11-30-04

DESCRIPTION AND REMARKS	COLOR	CONSISTENCY	SOIL TYPE	DEPTH (feet)	SAMPLER	BLOWS / FT	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	OTHER TESTS
CLAY, silty, sandy, with abundant fine to coarse subangular to subrounded gravel, and some roots, moist	Very Dark Gray	Stiff	CH						
				1					
				2					
	Dark Grayish Brown to Dark Olive Brown					[14]			PP = 3.5 ksf
- less sandy with few fine subangular gravel and trace roots				3					LL = 64 PI = 44
				4		[17]	30		-200 = 89 PP = 2.5 ksf
				5					
				6					
				7					
				8					
- sandy, with some fine subangular to subrounded gravel	Brown with some Yellowish Brown			9					~ 4 hours later
				10		[24]			PP = 4.5 ksf
GRAVEL (fine to coarse, subangular to subrounded, well graded), sandy, with some intermittent pockets of clay, moist	Yellowish Brown to Dark Grayish Brown	Medium Dense	GW-GC	11					
				12					
				13					
- moist to saturated	Dark Grayish Brown	Loose		14			19		ATOD -200 = 12
SAND (fine-to-medium-grained), clayey, silty, with some fine to coarse subangular to subrounded gravel, moist	Brown to Yellowish Brown	Loose	SC	15		[17]	16		-200 = 38
				16					
				17					
				18					
GRAVEL (fine to coarse, subangular to subrounded, well graded), clayey, silty, sandy, moist (continued on next sheet)	Yellowish Brown to Grayish Brown	Medium Dense	GW-GC	19 20		[34]	15		

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EXPLORATORY BORING LOG

1201 SAN PABLO AVENUE DEVELOPMENT
Berkeley, California

PROJECT NO.	DATE	SHEET	BORING NO.
501-4	December 2004	1 of 3	1

DRILL RIG: Continuous Hollow Flight Auger	SURFACE ELEVATION: Not Measured	LOGGED BY: TB
DEPTH TO GROUNDWATER: See Notes	BORING DIAMETER: 6-1/2 inches	DATE DRILLED: 11-30-04

DESCRIPTION AND REMARKS	COLOR	CONSISTENCY	SOIL TYPE	DEPTH (feet)	SAMPLER	BLOWS / FT	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	OTHER TESTS
GRAVEL (fine to coarse, subangular to subrounded, well graded), clayey, silty, sandy, moist (continued from previous sheet)	Yellowish Brown to Grayish Brown	Medium Dense	GW-GC	21					LL = 27 PI = 12 -200 = 16
				22					
				23					
CLAY, silty, slightly sandy, with trace fine subangular gravel, moist	Mottled Gray and Yellowish Brown	Stiff	CL	24					
				25		11	25		-200 = 80
				26					
				27					
				28					
				29					
- siltier, with no gravel	Mottled Yellowish Brown and Gray			30		15			
				31					
				32					
GRAVEL (fine to coarse, subangular to subrounded, well graded), clayey, silty, sandy, moist	Very Dark Gray	Medium Dense	GC	33					
				34					
CLAY, silty, slightly sandy, with few fine subangular gravel, moist	Mottled Dark Yellowish Brown and Gray	Very Stiff	CL	35		16			
				36					
				37					
				38					
- some fine to coarse subangular to subrounded gravel	Mottled Yellowish Brown and Brown	Hard		39					
(continued on next sheet)				40		44			

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EXPLORATORY BORING LOG

1201 SAN PABLO AVENUE DEVELOPMENT
Berkeley, California

PROJECT NO.	DATE	SHEET	BORING NO.
501-4	**December 2004**	**2 of 3**	1

DRILL RIG: Continuous Hollow Flight Auger	SURFACE ELEVATION: Not Measured	LOGGED BY: TB
DEPTH TO GROUNDWATER: See Notes	BORING DIAMETER: 6-1/2 inches	DATE DRILLED: 11-30-04

DESCRIPTION AND REMARKS	COLOR	CONSISTENCY	SOIL TYPE	DEPTH (feet)	SAMPLER	BLOWS / FT	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	OTHER TESTS
CLAY, silty, sandy, with some fine to coarse subangular to subrounded gravel, moist (continued from previous sheet)	Mottled Yellowish Brown and Brown	Hard	CL	41, 42, 43					
- few fine subangular gravel	Mottled Yellowish Brown and Gray	Very Stiff		44, 45	▯	31			
				46, 47, 48					
	Gray with some Yellowish Brown			49, 50	▯	32			

Bottom of boring at 50 feet and 6 inches

NOTES

1. Groundwater was encountered at approximately 14 feet at the time of drilling, and was at a depth of about 9 feet 6 inches 4 hours after drilling. (See text for discussion.)

2. Stratification lines represent the approximate boundaries between material types and the transitions may be gradual.

3. Penetration resistance values (blow counts) enclosed in brackets ([ ]) were recorded with a 3.0-inch O.D. Modified California sampler; these are not standard penetration resistance values.

4. Approximate unconfined compressive strength values were recorded in the field using a pocket penetrometer. These values are shown on the logs and are preceded by the symbol "PP".

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EXPLORATORY BORING LOG

1201 SAN PABLO AVENUE DEVELOPMENT
Berkeley, California

PROJECT NO.	DATE	SHEET	BORING NO.
501-4	**December 2004**	**3 of 3**	1

DRILL RIG: Continuous Hollow Flight Auger	SURFACE ELEVATION: Not Measured	LOGGED BY: TB
DEPTH TO GROUNDWATER: See Notes	BORING DIAMETER: 6-1/2 inches	DATE DRILLED: 11-30-04

DESCRIPTION AND REMARKS	COLOR	CONSISTENCY	SOIL TYPE	DEPTH (feet)	SAMPLER	BLOWS / FT	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	OTHER TESTS
CLAY, silty, sandy, with some fine subangular to subrounded gravel and some roots, moist	Very Dark Gray	Stiff	CL	1					
				2	X				LL = 45 PI = 28 -200 = 59
				3	X		19		PP = 3.5 ksf
CLAY, silty, slightly sandy, with few fine subangular gravel, moist;	Dark Grayish Brown	Stiff	CH			[25]			PP > 9.0 ksf
				4					
				5					
				6					
				7	X				
- sandier with some fine subangular gravel	Mottled Brown, Yellowish Brown and Dark Brown			8	X	[26]			PP = 5.0 ksf PP = 2.5 ksf
				9					▽ ~ 1.5 hours later
				10					
SAND (fine-to-medium-grained), clayey, silty, with trace fine subangular gravel, moist	Brown with some Gray	Medium Dense	SC	11					
				12	|				
				13	|	14	19		-200 = 46
				14					▽ ATOD
- some fine subangular gravel	Grayish Brown			15					
				16					
				17	|				
GRAVEL (fine to coarse, subangular to subrounded, well graded), clayey, silty, sandy, moist	Brown to Grayish Brown	Medium Dense	GW-GC	18	|	18	14		-200 = 15
				19					
(continued on next sheet)				20					

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EXPLORATORY BORING LOG

1201 SAN PABLO AVENUE DEVELOPMENT
Berkeley, California

PROJECT NO.	DATE	SHEET	BORING NO.
501-4	**December 2004**	**1 of 2**	2

DRILL RIG: Continuous Hollow Flight Auger	SURFACE ELEVATION: Not Measured	LOGGED BY: TB
DEPTH TO GROUNDWATER: See Notes	BORING DIAMETER: 6-1/2 inches	DATE DRILLED: 11-30-04

DESCRIPTION AND REMARKS	COLOR	CONSISTENCY	SOIL TYPE	DEPTH (feet)	SAMPLER	BLOWS / FT	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	OTHER TESTS
GRAVEL (fine to coarse, subangular to subrounded, well graded), clayey, silty, sandy, moist (continued from previous sheet)	Brown to Grayish Brown	Medium Dense	GW-GC	21					
				22					
				23		16			
				24					
				25					
CLAY, silty, sandy, with some fine subangular to subrounded gravel, moist	Mottled Brown, Redish Brown, and Gray	Stiff	CL	26					
				27					
				28		15	25		-200 = 62
				29					
				30					
				31					
	Yellowish Brown with some Dark Brown			32					
				33		[25]			

Bottom of boring at 33 feet and 6 inches

NOTES

1. Groundwater was encountered at approximately 14 feet at the time of drilling, and was at a depth of about 9 feet 1 hour after drilling. (See text for discussion.)

2. Stratification lines represent the approximate boundaries between material types and the transitions may be gradual.

3. Penetration resistance values (blow counts) enclosed in brackets ([ ]) were recorded with a 3.0-inch O.D. Modified California sampler; these are not standard penetration resistance values.

4. Approximate unconfined compressive strength values were recorded in the field using a pocket penetrometer. These values are shown on the logs and are preceded by the symbol "PP".

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EXPLORATORY BORING LOG

1201 SAN PABLO AVENUE DEVELOPMENT
Berkeley, California

PROJECT NO.	DATE	SHEET	BORING NO.
501-4	**December 2004**	**2 of 2**	2

APPENDIX B
2018 BORING LOGS

DRILL RIG: CME 75, Continous Hollow Stem Auger	SURFACE ELEVATION: 44.5 feet (see notes)	LOGGED BY: SC
DEPTH TO GROUNDWATER: 9.5 feet (see notes)	BORING DIAMETER: 8 inches	DATE DRILLED: 6/20/18

DESCRIPTION AND REMARKS	COLOR	CONSISTENCY	SOIL TYPE	DEPTH (ft)	SAMPLER TYPE	SAMPLER BLOW COUNTS	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	OTHER TESTS
CLAY, Lean - with some fine to medium grained sand, moist	Brown to Dark Brown	Stiff	CL						
				1					
				2					
				3					
				4					
-with fine to medium grained sand and trace fine gravel, dry to moist	Dark Brown	Very Stiff to Hard		5					
				6		33	18		LL = 45 PI = 29 -200 = 70.1%
				7					
SAND, Clayey - fine to coarse grained sand with fine to coarse subrounded gravel, moist	Brown to Yellowish Brown with some Very Dark Gray	Medium Dense	SC	8					
				9					▼
				10					
				11		22	16		-200 = 25.2%
				12					
				13					
				14					
-fine to coarse grained sand with fine to coarse subangular gravel, moist	Brown to Grayish Brown	Dense		15					
				16		31	13		-200 = 13.3%
				17					
				18					
				19					▽

(Continued on Next Page)

DESCRIPTION AND REMARKS	COLOR	CONSISTENCY	SOIL TYPE	DEPTH (ft)	SAMPLER TYPE	SAMPLER BLOW COUNTS	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	OTHER TESTS
(Continued from Previous Page)									
CLAY, Lean, Sandy - fine to coarse grained sand with silt and trace fine subangular gravel, wet	Light Brown with some Gray	Stiff	CL	21		10	23		
				22					
				23					
				24					
-with silt and fine to medium grained sand, wet	Mottled Light Brown, Gray and Yellowish Brown			25					
				26		10	26		-200 = 67.3%
				27					
				28					
				29					
				30					
				31		11	25		
				32					
SAND, Clayey - with fine to coarse grained sand and some fine subangular gravel, wet	Mottled Grayish Brown, Brown and Yellowish Brown	Medium Dense	SC	33					
				34					
				35					
				36		19	18		-200 = 27.4%
				37					
CLAY, Lean - with silt and some fine grained sand, moist	Mottled Yellowish Brown and Gray	Very Stiff	CL	38					
				39					
				40					
				41		20			

Bottom of boring at 41.5 feet.

(Continued on Next Page)

DESCRIPTION AND REMARKS	COLOR	CONSISTENCY	SOIL TYPE	DEPTH (ft)	SAMPLER TYPE	SAMPLER BLOW COUNTS	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	OTHER TESTS

(Continued from Previous Page)

NOTES:

1. Groundwater was encountered at approximately 20 feet at the time of drilling and was at a depth of about 9.5 feet 23 hours after drilling. The boring was grouted following drilling in accordance with City of Berkeley requirements. (See report)

2. Stratification lines represent the approximate boundaries between material types and the transitions may be gradual.

3. Elevations were estimated from "Boundary and Topographic Survey, Lots 1 Through 5, Block V, Located at 1201 & 1205 San Pablo Avenue, Berkeley, Alameda County, California," drawn by Moran Engineering, Inc., dated June 9, 2016.

DRILL RIG: CME 75, Continous Hollow Stem Auger	SURFACE ELEVATION: 43 feet (see notes)	LOGGED BY: SC
DEPTH TO GROUNDWATER: 9 feet (see notes)	BORING DIAMETER: 8 inches	DATE DRILLED: 6/20/18

DESCRIPTION AND REMARKS	COLOR	CONSISTENCY	SOIL TYPE	DEPTH (ft)	SAMPLER TYPE	SAMPLER BLOW COUNTS	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	OTHER TESTS
CLAY, Lean/Fat, Sandy - fine to coarse grained sand with silt and trace subrounded gravel, moist	Brown	Stiff	CL-CH	1 – 4					
-fine to medium grainded sand, with trace coarse grained sand, and fine to coarse gravel, dry to moist		Very Stiff		5 – 9		26	19		LL = 50 PI = 34 -200 = 56.8%
-fine to medium grained sand, moist	Grayish Brown to Brown with some Very Dark Gray			10 – 11		20	16		
SAND, Clayey - fine to coarse grained sand with some fine to coarse subangular gravel, moist	Grayish Brown to Brown with some Yellowish Brown	Medium Dense	SC	12 – 14					
	Mottled Gray, Grayish Brown and Reddish Brown			15 – 19		23	20		-200 = 28.6%

(Continued on Next Page)

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EXPLORATORY BORING LOG
1201 SAN PABLO AVENUE DEVELOPMENT
Berkeley, California

PROJECT NO.	DATE	SHEET	BORING NO.
501-4C	**August 2018**	**1 of 3**	**2**

DESCRIPTION AND REMARKS	COLOR	CONSISTENCY	SOIL TYPE	DEPTH (ft)	SAMPLER TYPE	SAMPLER BLOW COUNTS	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	OTHER TESTS
(Continued from Previous Page)									
SAND, Silty - fine to coarse grained sand with fine to coarse subangular gravel, wet	Dark Grayish Brown to Brown	Dense	SM	21					-200 = 15.6%
				22		42	14		
				23					
				24					
				25					
				26		39	15		
SAND, Clayey - fine to medium grained sand, wet	Mottled Light Brown and Gray	Medium Dense	SC	27					
				28					
				29					
				30					
				31		12	24		-200 = 40.9%
CLAY, Lean - with some silt and trace fine grained sand, moist	Reddish Brown with some Gray	Very Stiff	CL	32					
				33					
				34					
				35					
				36		20	31		
				37					
CLAY, Lean - with silt, fine to coarse grained sand and trace fine subangular gravel, moist	Mottled Light Brown, Yellowish Brown and Grayish Brown	Hard	CL	38					
				39					
				40					
				41		41			

Bottom of boring at 41.5 feet.

(Continued on Next Page)

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EXPLORATORY BORING LOG

1201 SAN PABLO AVENUE DEVELOPMENT
Berkeley, California

PROJECT NO.	DATE	SHEET	BORING NO.
501-4C	August 2018	2 of 3	2

DESCRIPTION AND REMARKS	COLOR	CONSISTENCY	SOIL TYPE	DEPTH (ft)	SAMPLER TYPE	SAMPLER BLOW COUNTS	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	OTHER TESTS

(Continued from Previous Page)

NOTES:

1. Groundwater was encountered at approximately 20 feet at the time of drilling and was at a depth of about 9 feet 21 hours after drilling. The boring was grouted following drilling in accordance with City of Berkeley requirements. (See report)

2. Stratification lines represent the approximate boundaries between material types and the transitions may be gradual.

3. Elevations were estimated from "Boundary and Topographic Survey, Lots 1 Through 5, Block V, Located at 1201 & 1205 San Pablo Avenue, Berkeley, Alameda County, California," drawn by Moran Engineering, Inc., dated June 9, 2016.

DRILL RIG: CME 75, Continous Hollow Stem Auger	**SURFACE ELEVATION:** 43 feet (see notes)	**LOGGED BY:** SC
DEPTH TO GROUNDWATER: 9 feet (see notes)	**BORING DIAMETER:** 8 inches	**DATE DRILLED:** 6/20/18

DESCRIPTION AND REMARKS	COLOR	CONSISTENCY	SOIL TYPE	DEPTH (ft)	SAMPLER TYPE	SAMPLER BLOW COUNTS	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	OTHER TESTS
CLAY, Lean/Fat - with trace fine to coarse grained sand, moist	Dark Brown	Stiff	CL-CH	1					
				2					
				3					
				4					
	Dark Brown to Brown	Very Stiff		5					
				6		20	23		
				7					
CLAY, Lean - with trace fine grained sand, moist	Mottled Light Brown and Gray, with some Very Dark Gray	Verry stiff	CL	8					
				9					▼
				10					
				11		18	21		
				12					
				13					
CLAY, Lean, Sandy - fine to medium grained sand with trace coarse grained sand and fine to coarse subrounded gravel, moist	Mottled Light Brown and Gray	Stiff	CL	14					
				15					
				16		14			-200 = 53.5%
				17					
SAND, Clayey - fine to coarse grained sand with fine to coarse subangular gravel, wet	Grayish Brown	Dense	SC	18					
				19					▽

(Continued on Next Page)

(Continued from Previous Page)

DESCRIPTION AND REMARKS	COLOR	CONSISTENCY	SOIL TYPE	DEPTH (ft)	SAMPLER TYPE	SAMPLER BLOW COUNTS	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	OTHER TESTS
SAND, Clayey - fine to coarse grained sand with fine to coarse subangular gravel, wet	Grayish Brown	Dense	SC	21		45	12		-200 = 12.6%
				22					
				23					
				24					
		Medium Dense		25					
				26		25			
				27					
CLAY, Lean, Sandy - fine grained sand with silt, moist	Mottled Yellowish Brown and Gray	Stiff	CL	28					
				29					
				30					
				31		14	22		LL = 30 PI = 12 -200 = 55.8%
				32					
CLAY, Lean - with some fine to coarse subangular gravel, moist	Grayish Brown to Gray with Light Brown Mottling	Very Stiff	CL	33					
				34					
				35					
				36		16	26		
				37					
				38					
				39					
	Mottled Gray and Light Brown	Stiff		40					
				41		12			

Bottom of boring at 41.5 feet.

(Continued on Next Page)

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EXPLORATORY BORING LOG
1201 SAN PABLO AVENUE DEVELOPMENT
Berkeley, California

PROJECT NO.	DATE	SHEET	BORING NO.
501-4C	**August 2018**	**2 of 3**	**3**

DESCRIPTION AND REMARKS	COLOR	CONSISTENCY	SOIL TYPE	DEPTH (ft)	SAMPLER TYPE	SAMPLER BLOW COUNTS	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	OTHER TESTS
(Continued from Previous Page)									

NOTES:

1. Groundwater was encountered at approximately 20 feet at the time of drilling and was at a depth of about 9 feet 19 hours after drilling. The boring was grouted following drilling in accordance with City of Berkeley requirements. (See report)

2. Stratification lines represent the approximate boundaries between material types and the transitions may be gradual.

3. Elevations were estimated from "Boundary and Topographic Survey, Lots 1 Through 5, Block V, Located at 1201 & 1205 San Pablo Avenue, Berkeley, Alameda County, California," drawn by Moran Engineering, Inc., dated June 9, 2016.

APPENDIX C
LIQUEFACTION ANALYSIS

TABLE 1 - LIQUEFACTION ANALYSIS
(Youd et al. 2001)

PROJECT NAME: 1201 San Pablo Avenue
PROJECT NUMBER: 501-4C
DATE: 7/24/2018
BY: TB

Highlighted information must be entered depending upon site specific conditions.

C_{R} = Correction Factor for Rod Length; 10-13' = 0.75; 13-20' = 0.85; 20-33' = 0.95; 33-99' = 1.0
C_{S} = Correction Factor for Sampling Method; SPT Sampler without liners = 1.2; Modified California Sampler = 0.63
C_{B} = Correction Factor for Borehole Diameter; For Hollow Stem, use ID.; 2.5-4.5" = 1.0; 5.9" = 1.05; 7.9" = 1.15
C_{E} - Correction Factor for Hammer Energy Ratio; Automatic Trip Hammer = 1.2, Rope and Cathead = 1.0
C_{N} - Correction Factor for Overburden Pressure

References:
Seed, R.B., et al., 2003, "Recent Advances in Soil Liquefaction Engineering: A Unified and Consistent Framework," American Society of Civil Engineers, Specialty Conference, April 30, 2003.
Youd, T. Leslie, et al.,2001, "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils."
Tokimatsu and Seed, 1987, Evaluation of settlements in sand due to earthquake shaking, Journal of Geotechnical Engineering, Vol. 113, No.8. (pg 405 in geotechnical earthquake engineering textbook)

Design M_w = 7.1
Water table = *10.0* ft below ground surface

Boring 2004-1

Youd, 2001																											Tokimatsu and Seed (1987)		
Depth at Center of Layer (ft)	***Unified Soil Class.***	***Layer Thickness (ft)***	***% Fines***	***N***	C_R	C_S	C_B	C_E	N_{60}	γ_m	σ_v	σ_v'	C_N	$N_{1,60}$	$N_{1,60cs}$			R_D	a_{max}	$(\tau_{max})_r$	$CSR_{,Mw}$	*MSF*	$CSR_{,M=7.5}$	$CRR_{,M=7.5}$	**FS**	**Liq**	$N_{1,60,cs}$	*ε (%)*	**Settlement (in)**
12.5	*GW-GC*	*5*	*12*	*17*	*0.85*	*0.63*	*1.00*	*1.00*	9	*120*	1500	1344	1.25	11	--	13	--	0.97	*0.819*	1196	0.579	*1.15*	0.503	0.140	0.278	yes	13	*2.1*	*1.26*
17.5	*SC*	*5*	*38*	*17*	*0.85*	*0.63*	*1.00*	*1.00*	9	*120*	2100	1632	1.14	10	--	--	17	0.96	*0.819*	1656	0.660	*1.15*	0.574	0.181	0.315	yes	17	*1.8*	*1.08*
22	*GW-GC*	*4*	*16*	*34*	*0.95*	*0.63*	*1.00*	*1.00*	20	*120*	2640	1891	1.06	21	--	25	--	0.95	*0.819*	2057	0.707	*1.15*	0.615	0.290	0.472	yes	25	*1.2*	*0.58*

Total Settlement (in): 2.92

Boring 2004-2

Youd, 2001																											Tokimatsu and Seed (1987)		
Depth at Center of Layer (ft)	***Unified Soil Class.***	***Layer Thickness (ft)***	***% Fines***	***N***	C_R	C_S	C_B	C_E	N_{60}	γ_m	σ_v	σ_v'	C_N	$N_{1,60}$	$N_{1,60cs}$			R_D	a_{max}	$(\tau_{max})_r$	$CSR_{,Mw}$	*MSF*	$CSR_{,M=7.5}$	$CRR_{,M=7.5}$	**FS**	**Liq**	$N_{1,60,cs}$	*ε (%)*	**Settlement (in)**
14	*SC*	*6*	*46*	*14*	*0.85*	*1.20*	*1.00*	*1.00*	14	*120*	1680	1430	1.22	17	--	--	25	0.97	*0.819*	1336	0.607	*1.15*	0.528	0.300	0.569	yes	25	*1.2*	*0.86*
19	*GW-GC*	*4*	*15*	*18*	*0.85*	*1.20*	*1.00*	*1.00*	18	*120*	2280	1718	1.11	20	--	23	--	0.96	*0.819*	1792	0.678	*1.15*	0.589	0.264	0.449	yes	23	*1.4*	*0.67*
23.5	*GW-GC*	*5*	*15*	*16*	*0.95*	*1.20*	*1.00*	*1.00*	18	*120*	2820	1978	1.03	18	--	21	--	0.95	*0.819*	2187	0.719	*1.15*	0.625	0.233	0.373	yes	21	*1.5*	*0.90*

Total Settlement (in): 2.44

TABLE 1 - LIQUEFACTION ANALYSIS
(Youd et al. 2001)

PROJECT NAME: 1201 San Pablo Avenue
PROJECT NUMBER: 501-4C
DATE: 7/24/2018
BY: TB

Highlighted information must be entered depending upon site specific conditions.

C_{R} = Correction Factor for Rod Length; 10-13' = 0.75; 13-20' = 0.85; 20-33' = 0.95; 33-99' = 1.0
C_{S} = Correction Factor for Sampling Method; SPT Sampler without liners = 1.2; Modified California Sampler = 0.63
C_{B} = Correction Factor for Borehole Diameter; For Hollow Stem, use ID.; 2.5-4.5" = 1.0; 5.9" = 1.05; 7.9" = 1.15
C_{E} - Correction Factor for Hammer Energy Ratio; Automatic Trip Hammer = 1.2, Rope and Cathead = 1.0
C_{N} - Correction Factor for Overburden Pressure

References:

Seed, R.B., et al., 2003, "Recent Advances in Soil Liquefaction Engineering: A Unified and Consistent Framework," American Society of Civil Engineers, Specialty Conference, April 30, 2003.
Youd, T. Leslie, et al.,2001, "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils."
Tokimatsu and Seed, 1987, Evaluation of settlements in sand due to earthquake shaking, Journal of Geotechnical Engineering, Vol. 113, No.8. (pg 405 in geotechnical earthquake engineering textbook)

Design M_{w} = 7.1
Water table = *10.0* ft below ground surface

Boring 2018-1

Youd, 2001																											Tokimatsu and Seed (1987)		
Depth at Center of Layer (ft)	***Unified Soil Class.***	***Layer Thickness (ft)***	***% Fines***	***N***	***C_R***	***C_S***	***C_B***	***C_E***	**N_{60}**	***γ_m***	**σ_v**	**σ_v'**	**C_N**	**$N_{1,60}$**		**$N_{1,60cs}$**		**R_D**	***a_{max}***	**$(\tau_{max})_r$**	**$CSR_{,Mw}$**	***MSF***	**$CSR_{,M=7.5}$**	**$CRR_{,M=7.5}$**	**FS**	**Liq**	**$N_{1,60,cs}$**	***ε (%)***	**Settlement (in)**
11	*SC*	*6*	*25.2*	*22*	*0.75*	*1.20*	*1.15*	*1.20*	27	*120*	1320	1258	1.30	35	--	43	--	0.98	*0.819*	1056	0.546	*1.15*	0.475	--	--	no	43		*0.00*
17	*SC*	*6*	*13.3*	*31*	*0.85*	*1.20*	*1.15*	*1.20*	44	*120*	2040	1603	1.15	50	--	54	--	0.96	*0.819*	1611	0.653	*1.15*	0.568	--	--	no	54		*0.00*
35.5	*SC*	*5*	*27.4*	*19*	*1.00*	*1.20*	*1.15*	*1.20*	31	*120*	4260	2669	0.89	28	--	36	--	0.89	*0.819*	3095	0.754	*1.15*	0.655	--	--	no	36		*0.00*

Total Settlement (in): 0.00

Boring 2018-2

Youd, 2001																											Tokimatsu and Seed (1987)		
Depth at Center of Layer (ft)	***Unified Soil Class.***	***Layer Thickness (ft)***	***% Fines***	***N***	***C_R***	***C_S***	***C_B***	***C_E***	**N_{60}**	***γ_m***	**σ_v**	**σ_v'**	**C_N**	**$N_{1,60}$**		**$N_{1,60cs}$**		**R_D**	***a_{max}***	**$(\tau_{max})_r$**	**$CSR_{,Mw}$**	***MSF***	**$CSR_{,M=7.5}$**	**$CRR_{,M=7.5}$**	**FS**	**Liq**	**$N_{1,60,cs}$**	***ε (%)***	**Settlement (in)**
16	*SC*	*8*	*28.6*	*23*	*0.85*	*1.20*	*1.15*	*1.20*	32	*120*	1920	1546	1.17	37	--	47	--	0.97	*0.819*	1520	0.639	*1.15*	0.556	--	--	no	47		*0.00*
21.75	*SM*	*3.5*	*15.6*	*42*	*0.95*	*1.20*	*1.15*	*1.20*	66	*120*	2610	1877	1.06	70	--	76	--	0.95	*0.819*	2035	0.705	*1.15*	0.613	--	--	no	76		*0.00*
25.25	*SM*	*3.5*	*15.6*	*39*	*0.95*	*1.20*	*1.15*	*1.20*	61	*120*	3030	2078	1.01	61	--	67	--	0.94	*0.819*	2335	0.730	*1.15*	0.635	--	--	no	67		*0.00*
29.5	*SC*	*5*	*40.9*	*12*	*0.95*	*1.20*	*1.15*	*1.20*	19	*120*	3540	2323	0.95	18	--	--	27	0.92	*0.819*	2676	0.749	*1.15*	0.651	0.328	0.503	yes	27	*1.1*	*0.68*

Total Settlement (in): 0.68

Boring 2018-3

Youd, 2001																											Tokimatsu and Seed (1987)		
Depth at Center of Layer (ft)	***Unified Soil Class.***	***Layer Thickness (ft)***	***% Fines***	***N***	***C_R***	***C_S***	***C_B***	***C_E***	**N_{60}**	***γ_m***	**σ_v**	**σ_v'**	**C_N**	**$N_{1,60}$**		**$N_{1,60cs}$**		**R_D**	***a_{max}***	**$(\tau_{max})_r$**	**$CSR_{,Mw}$**	***MSF***	**$CSR_{,M=7.5}$**	**$CRR_{,M=7.5}$**	**FS**	**Liq**	**$N_{1,60,cs}$**	***ε (%)***	**Settlement (in)**
20.5	*SC*	*5*	*12.6*	*45*	*0.95*	*1.20*	*1.15*	*1.20*	71	*120*	2460	1805	1.08	76	--	80	--	0.96	*0.819*	1925	0.693	*1.15*	0.603	--	--	no	80		*0.00*
25.5	*SC*	*5*	*12.6*	*25*	*0.95*	*1.20*	*1.15*	*1.20*	39	*120*	3060	2093	1.01	39	--	42	--	0.94	*0.819*	2356	0.732	*1.15*	0.636	--	--	no	42		*0.00*

Total Settlement (in): 0.00