[redacted] [redacted]

GEOTECHNICAL INVESTIGATION
PROPOSED TWO SINGLE FAMILY RESIDENCES ON A PARCEL SPLIT
APN 946-[redacted]
[redacted]
PLEASANTON, CALIFORNIA

PREPARED FOR:
[redacted]

By

[redacted]
Geotechnical Consultants

Project No. [redacted]
December 23, 2020

[redacted]

Geotechnical Consultants

Project No. [redacted]
December 23, 2020

Mr. [redacted]
Pleasanton, CA 94588

Subject: Proposed Two Single Family Residences on a Parcel Split
APN 946-[redacted]
[redacted]
Pleasanton, California
GEOTECHNICAL INVESTIGATION

Dear Mr. [redacted]

In accordance with your authorization, we have completed our geotechnical investigation for subdividing the subject site into two parcels and constructing single family residence on each of the new parcels. This report contains our findings pertaining to the geotechnical conditions of the site. Based on the data obtained, conclusions and recommendations were made for the design and construction of the project.

With regard to geotechnical considerations, the site is suitable for the proposed parcel split and constructing the proposed residences provided the recommendations in this report are incorporated into the design and followed during construction. Please refer to the accompanying report as to our findings, conclusions and recommendations.

Should you have any questions, please contact our office at your convenience.

[redacted]

Yours Sincerely
[redacted]

Principal Geotechnical Engineer

Copies: [redacted]

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GEOTECHNICAL INVESTIGATION

Purpose and Scope

This report presents the results of a Geotechnical Investigation conducted by [redacted] for [redacted] in the City of Pleasanton, California into two parcels and constructing a [redacted] each of the new parcels. The purpose of the investigation was to explore and evaluate the geotechnical conditions at the site and provide geotechnical recommendations for the proposed residences. A preliminary Architectural Site Plan for the parcel split was made available for our use in this investigation. Based on the data obtained, conclusions and recommendations were made for the design and construction of the project.

Our investigation included the following:

a) Review of available published geologic maps for the site and vicinity;

b) Field reconnaissance by the Geotechnical Engineer;

c) Drilling and sampling of the subsurface soils;

d) Laboratory testing of the soil samples retrieved from the exploratory borings;

e) Analysis of the data and formulation of conclusions and recommendations;

f) Preparation of this written report.

Site Location and Description

The approximately [redacted] site is located at [redacted] the City of Pleasanton, California as shown on Figure 1 in the Appendix. A neighboring residence at the corner of [redacted] borders the site on the east and north. On the south and [redacted] the site is open [redacted]

There are no building structures on the site. At the time of our field exploration the site was covered with very short weeds. Topographically, the site is virtually flat.

This description of the site was based on a site reconnaissance by our Geotechnical Engineer and review of available plans for the site and vicinity.

Proposed Project

The proposed project will consist of subdividing the subject site into two parcels and constructing a residence on each of the new parcels. One parcel faces [redacted] the other parcel faces [redacted] as shown on Figure 6 in the Appendix.

Site Geology

The reviewed geologic maps for the Livermore [redacted] ndicated that the site is mapped as underlain by alluvial deposits. [redacted] described these deposits as "*unconsolidated sand, silt, gravel, and clay deposits generally subject to redistribution by fluvial processes*" of the Holocene - Pleistocene Age. [redacted] escribed these deposits as "*alluvial gravel, sand and clay of valley areas*". The maps by [redacted] e shown on Figures 3 and 4 in the Appendix, respectively.

Also, we reviewed a "Seismic Hazard Zone Report for the [redacted] Quadrangle, Alameda County, California" published by the California Geological Survey [redacted] This CGS report included a Quaternary Geologic Map. Based on this map (Figure 5 in the Appendix) the site is mapped as underlain by alluvial fan deposits of the [redacted]

Faults and Seismicity

The subject site is located within the seismically active San Francisco Bay Area and may be subject to strong ground shaking during expected lifetime of the proposed residence. According to the California Division [redacted] no known active faults exist within or adjacent to the site, and the site is not within the Alquist-Priolo Earthquake Fault Hazard Zones. The nearest known active fault to the site is the Calaveras fault, mapped about 3.2 miles (5 kilometers) west of the site. Other nearby faults include the Pleasanton, Verona, and Las Positas faults, mapped by the [redacted] northwest, [redacted] f the site, respectively. These three faults (Pleasanton, [redacted] source of seismic activity. Other major faults of the region include the Greenville fault, mapped about 8.3 miles northeast, Hayward fault, about 10 miles southwest, Concord fault, 16 miles northwest, and the San Andreas fault, about 28 miles southwest of the site.

Any of these or other major faults in the region could produce significant shaking at the site. The U.S. Geological Survey Working Group on Earthquake Probabilities (2014) has estimated that the probability of a large earthquake in the region during a span of 30 years at 63 percent. Furthermore, they indicated that the most probable source of producing such an event is the northern segment of the Hayward fault. The Calaveras fault could produce more intense shaking at the site than the Hayward fault due to its closer proximity to the site. The International Conference of Building Officials (ICBO) (1998) estimated maximum moment magnitudes of 6.8 on the Calaveras fault, 7.1 on the Hayward fault, 6.9 on the Greenville fault, 6.7 on the Concord fault, and 7.9 on the San Andreas fault. The peak site accelerations that can be produced by these faults are addressed later in the "Seismic Hazard" section of this report.

Field Investigation

The field investigation was performed on December 7, 2020 and included the drilling of four (4) exploratory borings (2 on each of the new parcels) at the approximate locations shown on Figure 6 in the Appendix. The drilling was performed with a truck-mounted drill rig using power-driven, 4.0-inch solid flight augers. Visual classifications were made from auger cuttings and the samples in the field. As the drilling proceeded, undisturbed samples were obtained by means of 2-1/2 inch O.D., split-tube sampler. The sampler was driven 18 inches into the in-situ soils under the impact of a 140-pound hammer having a free fall of 30 inches. The number of blows for the last 12 inches of penetration was adjusted to the standard penetration resistance (N-Value).

The stratification of the soils, descriptions, location of undisturbed soil samples, and standard penetration resistance are shown on the "Exploratory Boring Logs" contained in the Appendix.

Laboratory Testing

The laboratory testing program was directed towards providing sufficient information for the determination of the engineering characteristics of the site soils so that the recommendations outlined in this report could be formulated. The tests included moisture content, dry unit-weight, plasticity index, sieve analysis, and unconfined compression tests. The laboratory test results are presented on Table 1 and on the boring logs in the Appendix.

Subsurface Conditions

The soils encountered consisted of alluvial deposits that extended to the bottom of borings. The upper layer(s) consisted of tan sandy silty clay of low plasticity or of sandy silt with occasional minor traces of gravel and extended to approximate depth of 18 feet in Borings 1 and 2 and between 13.5 and 15 feet in Borings 3 and 4. Below these upper layers in Borings 3 and 4 tannish brown sandy clay stiff layer about 3 feet thick was encountered. In all four borings a very stiff to hard layer consisting of dark gray silty clay was encountered at approximate depth of 17 to 18 feet and extended to the bottom of borings that ranged between 19.5 and 21.5 feet.

Groundwater was not encountered in any of the borings. It should be noted that fluctuations in the groundwater conditions do occur with variations in seasonal rainfall.

It is to be noted that the groundwater level contours in the Seismic Hazard Zone Report for the Livermore Quadrangle by the California Geological Survey indicated that the possible high groundwater level in the proximity of the site is at approximate depth between 40 and 50 feet below the existing ground surface.

A more detailed description of the soil conditions is presented on the boring logs in the Appendix.

Seismic Hazards

Ground Rupture

Since there are no known active faults within the site, the possibility of ground rupture from faulting is negligible.

Liquefaction Evaluation

Liquefaction occurs primarily in loose cohesionless soil below the groundwater when subjected to shock loads such as ground shaking during an earthquake. The site is mapped outside the zone of required investigation for liquefaction as shown on Figure 2 in the Appendix. Based on the soils encountered and the possible high groundwater at approximate depth 40 to 50 feet, the potential of seismically induced liquefaction at the site is low.

Ground Shaking

Any of the major faults in the region, particularly the Calaveras Fault, can produce significant shaking at the site. Following is a comparative matrix of peak ground acceleration values (PGA) for the Calaveras, Hayward, Greenville, Concord, and San Andreas faults assuming a maximum magnitude earthquake at the point on the respective fault that is closest to the subject site. The PGA's shown in Table A below were determined using the attenuation relationships by Boore et al., 1997.

TABLE A
Deterministic Acceleration Analysis (Boore et al., 1997)

Fault	Distance (Kilometers)	Moment Magnitude	Peak Ground Acceleration (g)	
			Mean	Mean + 1 Standard
Calaveras	5	6.8	0.41	0.68
Greenville	13	6.9	0.26	0.44
Hayward	16	7.1	0.25	0.42
Concord	26	6.7	0.14	0.24
San Andreas	45	7.9	0.18	0.30

The above analysis indicate that the Calaveras fault could produce the most intense shaking at the site and if a maximum credible earthquake ruptures this fault at its nearest proximity to the site, peak site accelerations could typically range between 0.41g and 0.68g. Therefore, proposed structures should be designed to accommodate ground shaking without experiencing significant structural damage by employing as a minimum the Building Code requirements.

Seismic Design Criteria

In reference to the currently applicable 2019 California Building Code (CBC) and the geotechnical data in this report, the parameters shown below should be used in the seismic design. It is to be noted that these parameters were obtained from the Structural Engineers Association of California (SEAOC) website in reference to the American Society of Civil Engineers, ASCE7-16 design procedure.

S_S (g)	S_1 (g)	Site Class	F_a	F_v	S_{MS} (g)	S_{M1} (g)	S_{DS} (g)	S_{D1} (g)
1.597	0.6	D	1.0	1.7	1.597	1.02	1.065	0.68

Where:

S_S: The 0.2-second spectral response acceleration for Site Class B.
S_1: The 1.0-second spectral response acceleration for Site Class B.
Site Class: Depends on the soil profile.
F_a: Site coefficient for adjusting S_S to the specific Site Class.
F_v: Site coefficient for adjusting S_1 to the specific Site Class.
S_{MS}: Adjusted spectral response acceleration for 0.2-second = $F_a \times S_S$.
S_{M1}: Adjusted spectral response acceleration for 1.0-second = $F_v \times S_1$.
S_{DS}: Design spectral response acceleration for 0.2 seconds = 2/3 S_{MS}.
S_{D1}: Design spectral response acceleration for 1.0 seconds = 2/3 S_{M1}.

DISCUSSION AND CONCLUSIONS

Our findings indicate that the site with regard to geotechnical considerations is suitable for the proposed parcel split and constructing a proposed residence on each of the new parcels provided the recommendations in this report are incorporated into the design and followed during construction.

There are no geologic hazards that preclude the site from development. The most prominent geologic hazard at the site is the seismic shaking which is typical in the seismically active San Francisco Bay Area. The seismic design criteria as presented earlier in this report should be considered in the design of the proposed residences.

The geotechnical conditions for the site are favorable for development. The near surface and foundation soils at the site have low potential for expansion. This is a favorable condition for foundation support and for the performance of concrete slabs-on-grade and pavement. Site grading including over-excavation and densification of the near surface soils will depend on the type of foundation system to be used for the proposed residence. The foundation system may consist of one of the following: 1) pier and grade beam foundation system, 2) structural concrete slab-on-grade (mat foundation), or 3) conventional shallow continuous and spread footings. For a pier and grade beam foundation system over-excavation and densification of the near surface soils may be limited to within the top foot. For the mat foundation and conventional shallow foundations, the upper 3 feet of the building pad should be over-excavated and the excavated soil be placed back in accordance with the engineered fill requirements. Recommendations for these foundation systems are presented in this report.

RECOMMENDATIONS

Site Preparation and Grading

The site in the proposed grading and structural areas should be stripped for removing near surface soils containing excessive organics. It is estimated that a stripping depth of about 4 inches may be necessary. Stripped material may not be used in engineered fill but may be stockpiled and used later for landscaping purposes.

The near surface soils should be over-excavated. The over-excavation depth will depend on the type of the foundation system to be used. For a pier and grade beam foundation system with raised wood floor, minimal grading can be performed mainly in the top one foot. For a mat foundation or conventional shallow foundations, the top 3 feet below existing ground surface or below pad grade whichever is deeper should be over-excavated. It is to be noted that the actual depth of over-excavation should be as determined by our Soil Engineer during grading. Grading of the building pad should extend a minimum of 5 feet laterally beyond the building envelope.

The exposed subgrade at the bottom of subexcavation should be scarified to approximate depth of 8 inches, moisture conditioned, and compacted to a minimum relative compaction of 90% at 2% above optimum moisture as determined by ASTM D1557-91. After compacting the subgrade fill can then be placed in lifts not exceeding 8 inches in thickness and compacted as recommended above until design grades are achieved.

Should import material be required for the purpose of establishing design grades, the import material should be approved by the Soil Engineer before it is brought to the site and should have equal or better engineering characteristics than the on-site soils.

Surface Drainage

All finish grades should be sloped away from all foundation and graded areas. The roof downspouts may be connected to a closed pipe system. As alternate, splash blocks may be used provided positive gradient at least five feet away from the foundations is maintained at all times and any backfill next to the foundations is properly compacted in accordance with the grading recommendations. Surface water should not be allowed to pond on the pad or migrate beneath the foundations. The outlet(s) of the drainage system should be discharged to a controlled drainage area. Surface drainage should be provided as designed by the project Civil Engineer and maintained by the property owners at all times.

Where landscaping and planter areas cannot be avoided next to the building and pavement, these areas should be properly drained in a manner to prevent irrigation and/or storm water from migrating beneath the foundations and pavement.

Foundations

[redacted]

The proposed residence may be supported on a pier and grade beam foundation system. The piers should have a minimum diameter of 16 inches and should extend to a minimum depth of 10 feet. The bottoms of the pier holes should be cleaned from excessive loose material as observed and approved by our Soil Engineer.

The piers may be designed based on skin friction along the pier shaft in the supporting soil, ignoring support in the top 1.5 feet. An allowable skin friction of 500 pounds per square foot in the supporting soil may be used in the design and may be increased by one-third to resist total loads which include wind or seismic forces.

To resist lateral loads passive resistance of 250 pounds per cubic foot in the supporting soil may be used. Passive resistance may be assumed to act on a projected area of twice the pier diameter times the depth of pier in the supporting soil.

No soil uplift soil pressure is necessary to be considered in the design. To resist uplift pressures if any induced by structural loads, the dead load of the structure and an adhesion value of 400 pounds per square foot along the pier shaft below a depth of 1.5 feet may be used.

Reinforcing steel should be provided as necessary for structural support and continuity of pier and grade beam. Spacing should be as required by the load distribution, but minimum spacing should not be less than 3 pier diameters, center-to-center.

Provided the foundations are designed and constructed as recommended above, the total and differential settlements within the building structure should be negligible.

[redacted]

Provided the building pad (in the top 3 feet) is over-excavated and re-constructed with engineered fill, a mat foundation for the proposed residence may be used. The mat may be designed as conventionally reinforced concrete slab-on-grade or post-tensioned slab. The slab should be structurally designed strong enough for the support of structural loads and to accommodate the potential of differential movement.

Conventionally Reinforced Structural Concrete Slab-on-Grade

One approach of designing the slab may be based on the "Design of Slab-on-Ground Foundations" procedure by the Wire Reinforcing Institute (WRI). The onsite soils have low potential for expansion and the design of slab to resist the effects of expansive soil does not control the design of the slab. We recommend the slab to be designed for assumed differential settlement of 1.0-inch within the building structure or 0.5 inches within a length of 20 feet whichever controls. For design simplification the slab may be designed assuming a cantilevered length (loss of support) of 4.5 feet over a span of 9.0 feet.

[redacted]

The post-tensioned slab-on-grade may be designed following the Post Tensioning Institute (PTI) Standard Requirements. As noted above the design of slab to resist the effects of expansive soil does not control the design of the slab. The slab should be designed for the assumed differential settlement as recommended above for the regularly reinforced slab.

For the above mat foundation systems (both types of slabs) an allowable bearing capacity of 1,500 pounds per square foot may be used in the design. The actual thickness and reinforcement of the slab should be as determined by the structural Engineer, but the slab should not be less than 10 inches thick.

The slab may be constructed over 2 inches of sand placed over vapor barrier, over the prepared subgrade. A 4-inch thick ¾ - inch crushed rock layer or open graded gravel material (optional) may be placed over the prepared subgrade. The slab should be thickened at the edges in order to contain the sand layer and gravel layer if applicable for minimizing the potential of surface water from migrating beneath the slab.

[redacted]

Provided the building pad (in the top 3 feet) is over-excavated and re-constructed with engineered similarly as noted above for the mat foundations, the proposed residence may be supported on conventional shallow foundations. The foundations should be excavated to a minimum depth of 18 inches below the top of subgrade and should have a minimum width of 15 inches. At this depth, an allowable bearing pressure of 2,200 p.s.f. due to dead load and 2,500 p.s.f. due to dead plus live loads may be used in the design. These values may be increased by one-third for the support of combined loads that include wind and seismic forces.

The footings should be reinforced as determined by the Structural Engineer, but as a minimum, the reinforcement in the continuous footings should not be less than four no. 4 bars, two near the top and two near the bottom of footing.

To resist lateral loads, one-half of the passive resistance (1/2 times 250 p.c.f.) of 125 p.c.f. below a depth of 12 inches and a coefficient of friction of 0.35 at the base of the footing may be used in the design.

Concrete Slab-on-Grade in Conjunction with Conventional Footings

The concrete slab-on-grade in the living areas (if applicable) should be structurally connected to the foundations. The thickness and steel reinforcement of the slab should be as determined by the Structural Engineer, depending on the structural requirements, but as a minimum the slab should not be less than 6 inches thick, reinforced with a minimum of no. 4 bars at 16 inches on-center both ways. The slab should be underlain with a minimum of 4 inches of ¾ - inch crushed rock or open graded gravel material for providing capillarity break. In addition, a waterproof membrane should be placed on top of the gravel layer, overlain with a minimum of two-inch thick sand layer. The subgrade should be kept in moist condition prior to constructing the slab.

Non-Structural Concrete Slabs-on-Grade

Non-structural concrete slabs-on-grade (garage slab other than mat foundations and exterior slabs) should be constructed as free floating slabs independent of the foundations and should be underlain with a minimum of 4 inches of ¾ - inch crushed rock. Reinforcing steel should be provided as determined by the Structural Engineer. If cut-fill transition condition occurs, the cut portion should be over-excavated to achieve uniform fill thickness. The subgrade should be prepared in accordance with grading requirements as previously recommended. Measures against condensation moisture in the garage floor may be provided (optional at the discretion of the designer and owner) by placing waterproof membrane on top of the gravel layer. In this case, a minimum of two-inch thick sand layer may be placed over the membrane at the discretion of the structural engineer.

Retaining Walls

The retaining walls if any should be designed in accordance with the following:

Active Pressure: 40 p.c.f. for flat backfill surface
55 p.c.f. for 3:1 back slope
65 p.c.f. for 2:1 back slope

Coefficient of friction at base of footing: 0.35
Passive resistance: 250 p.c.f. as previously recommended.

The active pressures shown above are for unrestrained walls. For restrained walls, the active pressures should be increased by one-third. If applicable, surcharge loads should be considered in the design.

A drainage blanket should be placed behind the wall. The blanket may consist of Class 2 permeable material per Caltrans specifications and should be a minimum of 12 inches thick and should extend to within 12 inches of the surface. A 4-inch diameter, perforated pipe should be installed in the bottom of the blanket on 2 to 4 inches bedding material (Class 2 permeable). As alternate, the blanket and bedding may consist of ¾-inch drain rock with filter fabric (Mirafi 140N or equivalent). The fabric should be placed at the interface between the drain rock and the soil. The upper 12 inches should be capped with on-site material and compacted.

Utility Trenches

The trench backfill should be placed in thin lifts and compacted as recommended earlier in the grading section of this report and in accordance with the City of Pleasanton requirements. All applicable safety standards and the requirements set forth by the City of Pleasanton should be followed.

Where utility trenches cross the building exterior, the bedding material can convey water beneath the structure. Therefore, it is recommended that such trenches next to the building structure be sealed with impermeable material or concrete for a distance of 2 feet on the exterior side of the foundation.

Pavement

The pavement for the driveways may consist of 3.0 inches of asphaltic concrete (AC) over 8.0 inches of Class 2 aggregate base (AB), or of 5.0 inches cement concrete over 6.0 inches Class 2 aggregate base. Steel reinforcement and details of the slab should be as determined by the Structural Engineer. If interlocking concrete pavers were used, we recommend the aggregate base to be a minimum of 9.0 inches thick.

The upper 8 inches of the subgrade should be scarified, moisture conditioned, and compacted to a minimum relative compaction of 90% at 2% above optimum moisture as determined by ASTM D1557-91. The aggregate base should be compacted to a minimum relative compaction of 95% at moisture content near optimum moisture.

ADDITIONAL GEOTECHNICAL SERVICES

The grading and foundation plans for the proposed project should be reviewed by the Soil Engineer prior to the construction phase.

The Soil Engineer should be notified to test and/or observe during the grading and foundation excavations requiring a minimum notice of two working days.

LIMITATIONS

The recommendations of this report are based upon the assumption that the soil conditions do not deviate from those encountered in the exploratory borings. It is not uncommon that different soil conditions might occur between the borings. Should any variations or undesirable conditions be encountered during construction, *GFK & ASSOCIATES, INC.* will provide supplemental recommendations as dictated by the field conditions.

This report was prepared in accordance with generally accepted procedures of geotechnical engineering practices and current standard of care. No other warranty is either expressed or implied.

This report is subject to our review after a period of three years, or if significant changes were made in the proposed project.

The use of this report by others presumes that they have verified all information and assume full responsibility for the total project.

REFERENCES

[redacted] Preliminary Geologic Map of the Livermore Quadrangle, Contra Costa County. USGS Open-[redacted] and [redacted] Foundation Map DF-196 (edited by John Minch).

California Division of Mines and Geology, 1982; Alquist-Priolo Earthquake Fault Zones Map for the Dublin and Livermore Quadrangles.

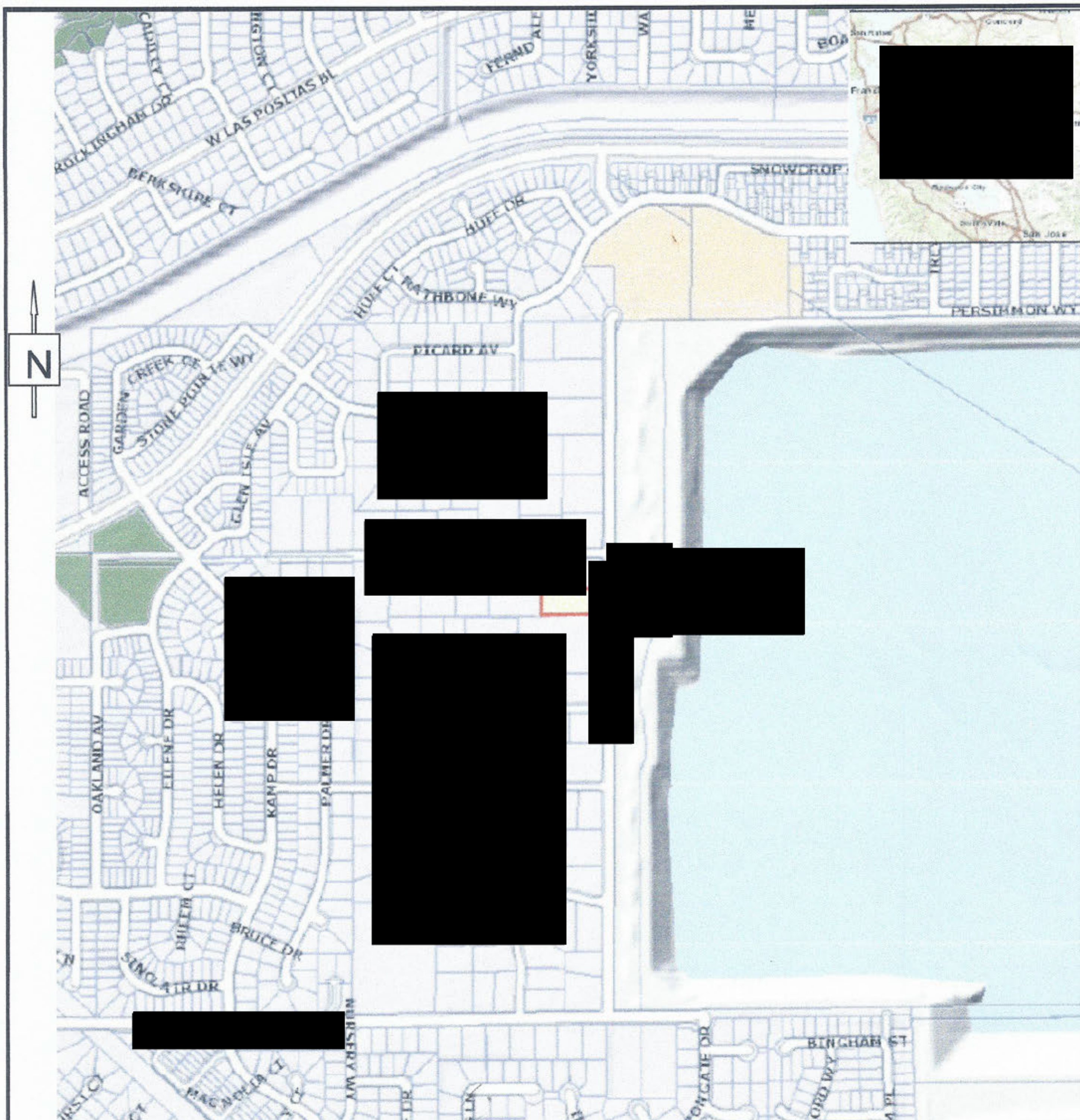
California Geological Survey, 2008; Seismic Hazard Zone Report for the Livermore 7.5-Minute Quadrangle, Alameda County, California.

[redacted] Hazards In The Livermore Valley and Vicinity, Alameda and Contra Costa Counties. California, DMG Open-File [redacted]

[redacted] Preliminary Photointerpretation Map of Landslide and Other Surficial Deposits of The Livermore 7 1/2' [redacted] California, U S Geological Survey [redacted]

[redacted] Earthquake Probabilities in the San Francisco Bay Region, U.S. Geological Survey.

APPENDIX



Source: Alameda County Parcel Viewer

Scale: As shown

[redacted]
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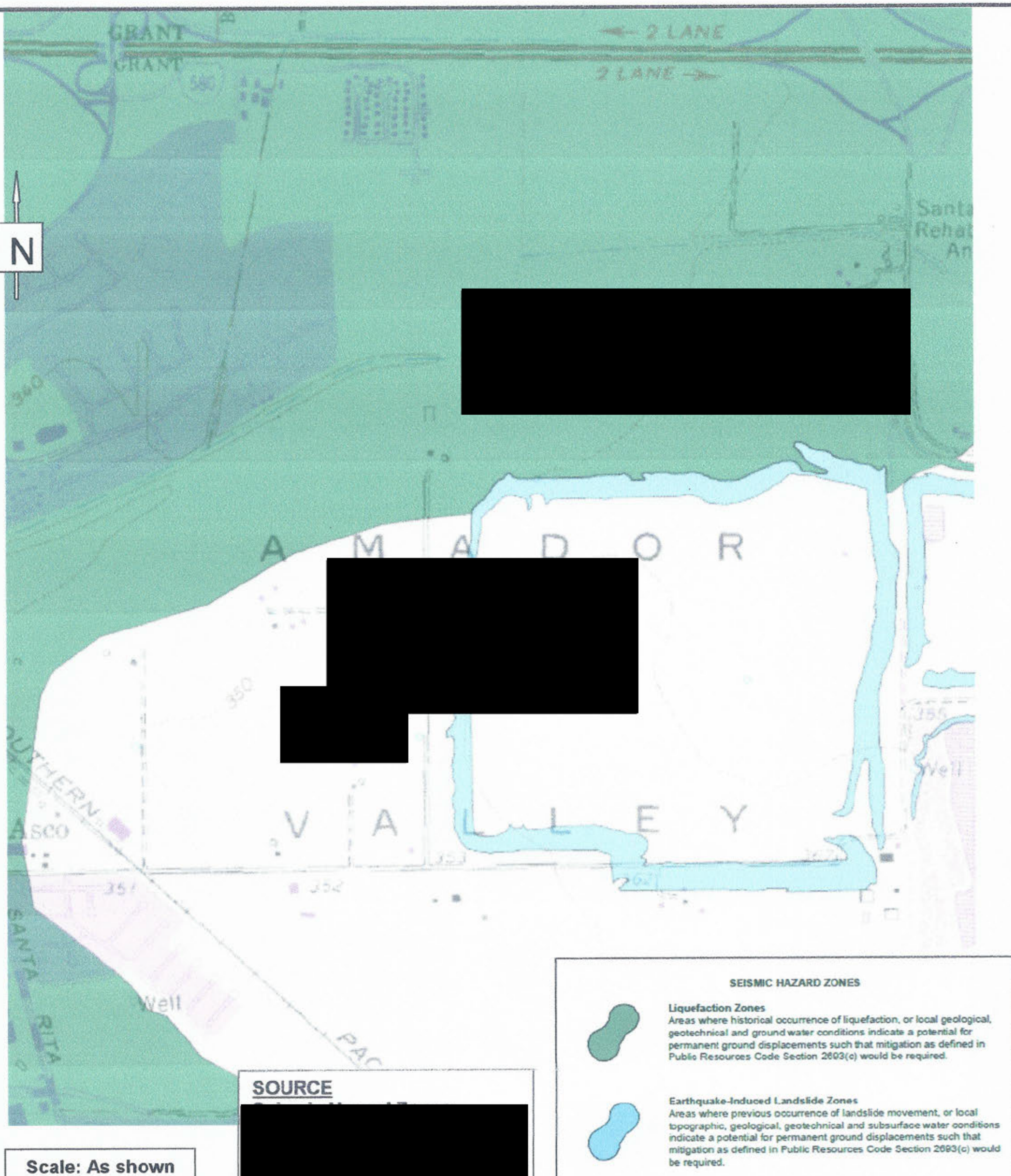
SITE LOCATION MAP

Proposed [redacted]
[redacted] Pleasanton, CA

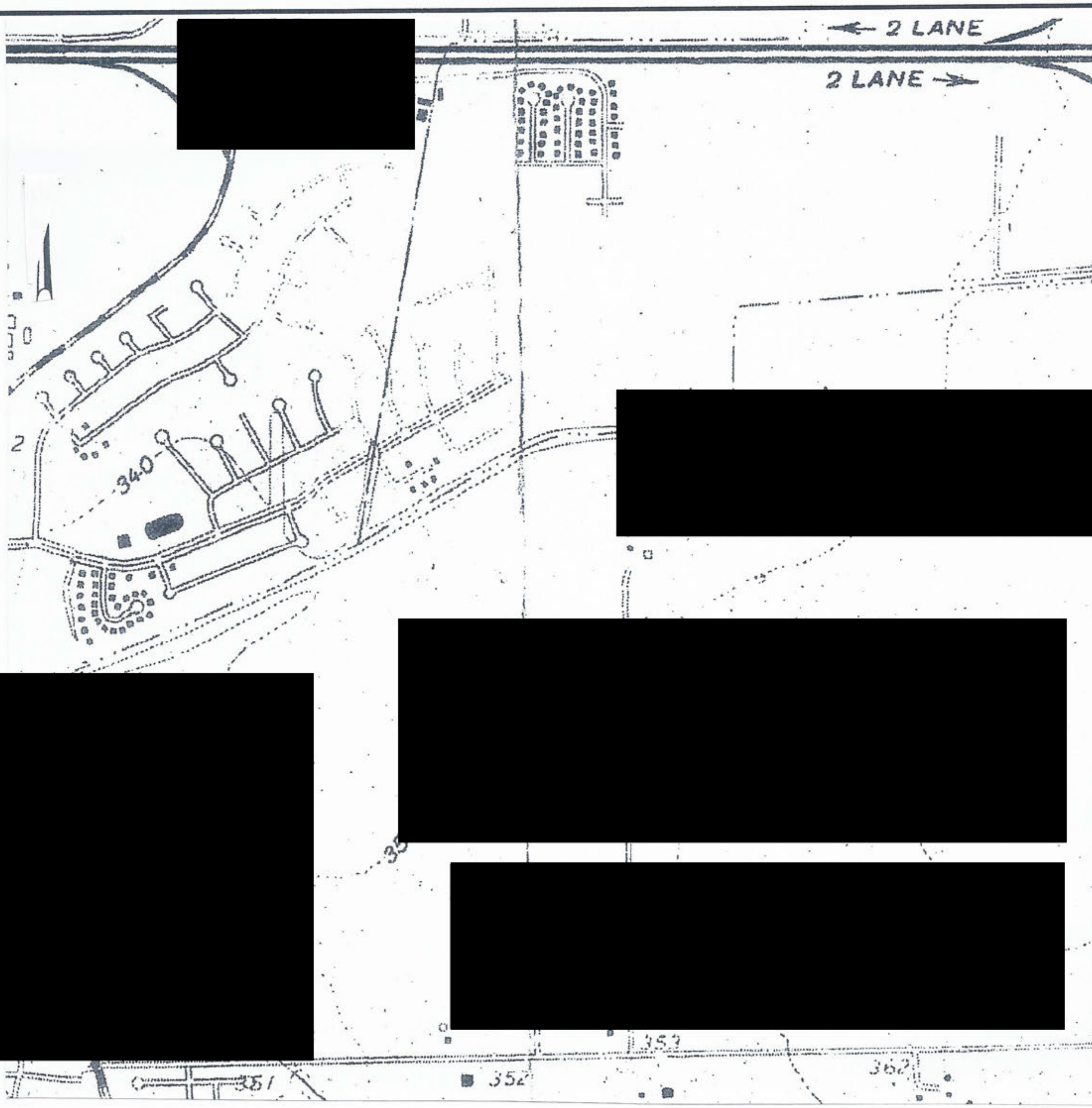
Project No.	Date	FIGURE
[redacted]	December 23, 2020	1



Scale: As shown

SOURCE
[redacted]

[redacted] Geotechnical Consultants	SEIZMIC HAZARD ZONES		
	Proposed [redacted] Pleasanton, CA		
	Project No. [redacted]	Date December 23, 2020	FIGURE 2



Explanation

[redacted]

SOURCE

[redacted]

[redacted]

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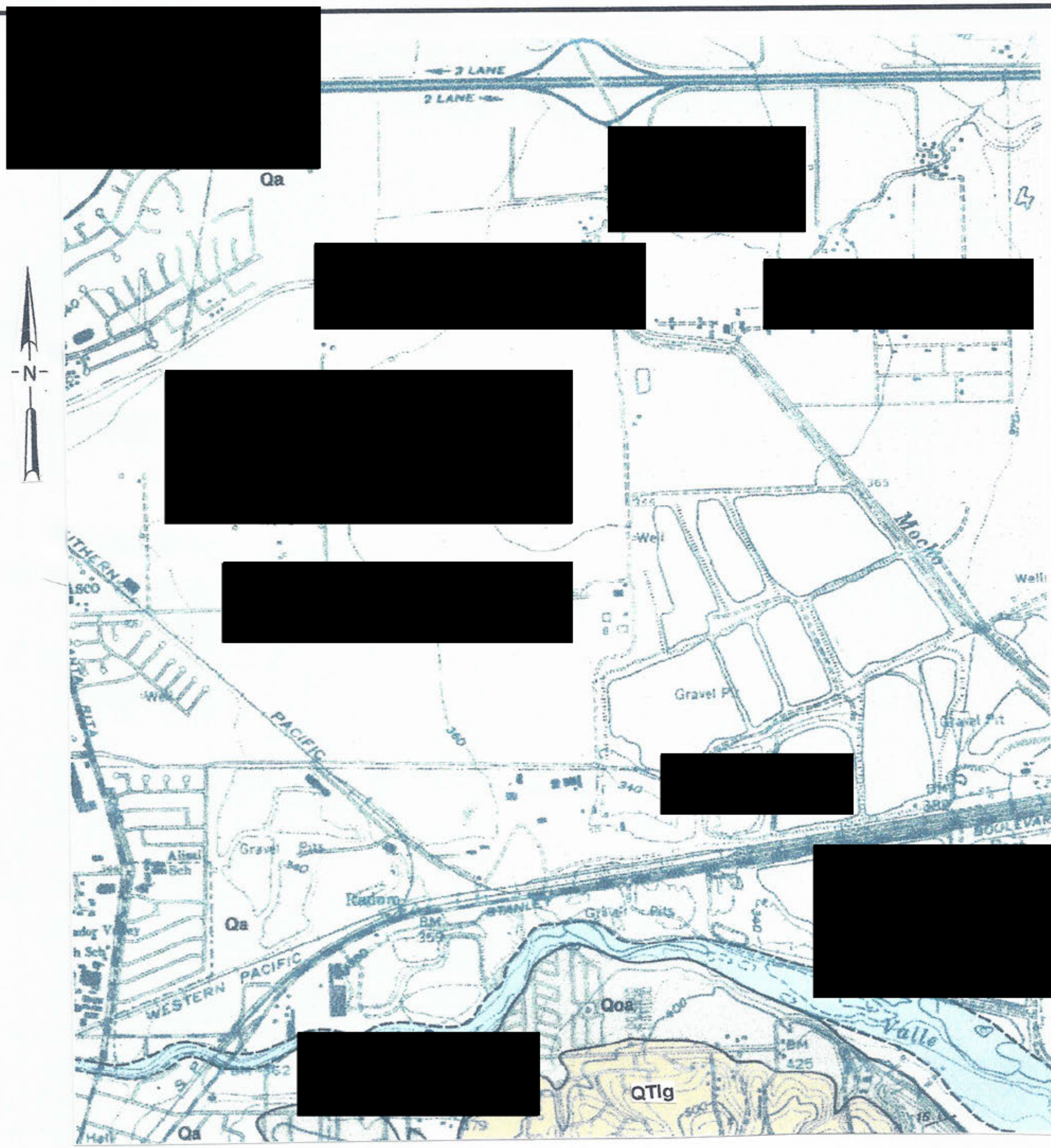
GEOLOGIC MAP (Majmundar)

Proposed [redacted]

[redacted] Pleasanton, CA

Project No.	Date	FIGURE
[redacted]	December 23, 2020	3



EXPLANATION

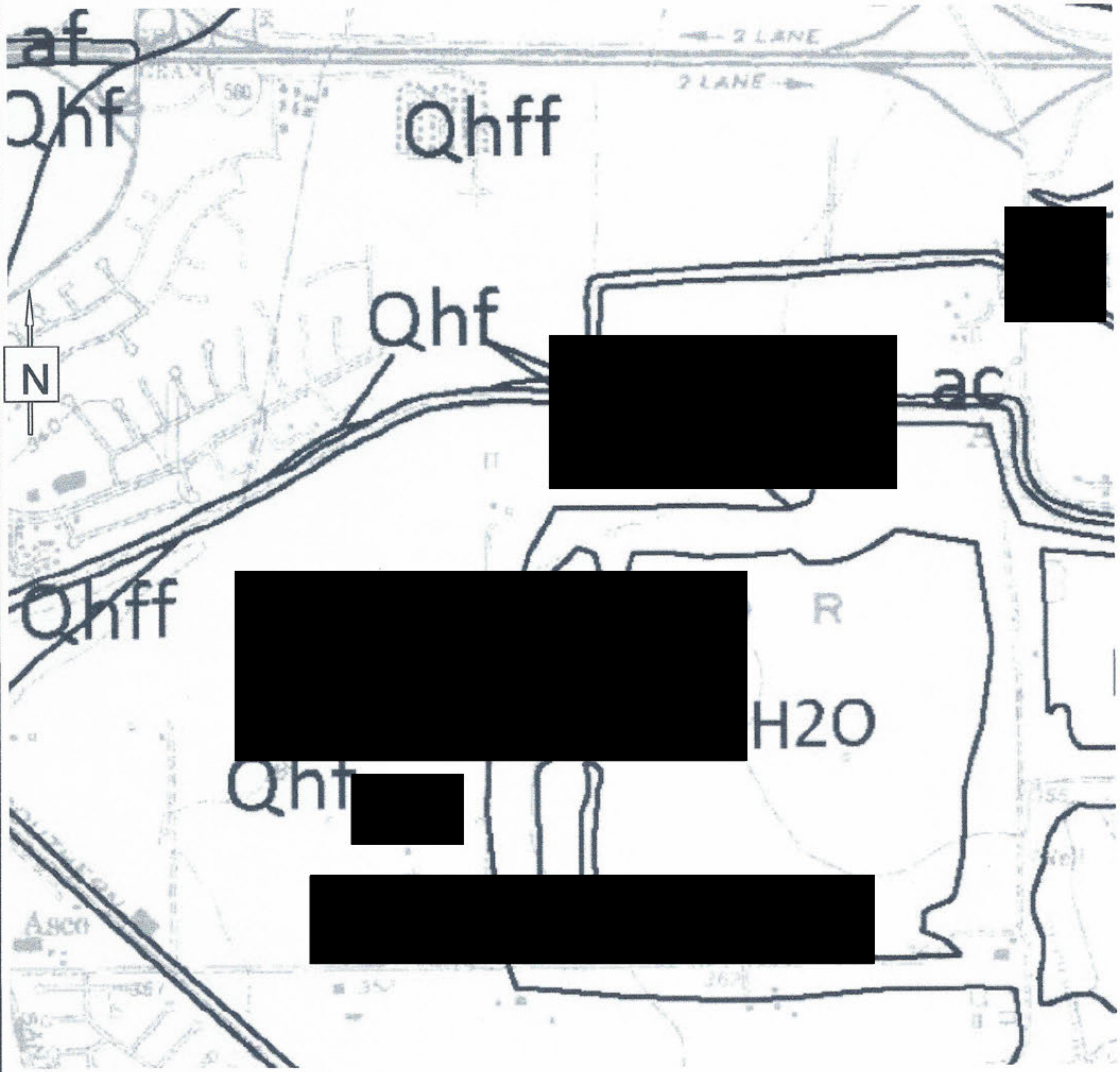
[redacted]

SOURCE

[redacted]

[redacted]
Geotechnical Consultants

GEOLOGIC MAP (Dibblee)		
Proposed [redacted] Drive, Pleasanton, CA		
Project No. [redacted]	Date December 23, 2020	**FIGURE 4**



PARTIAL EXPLANATION

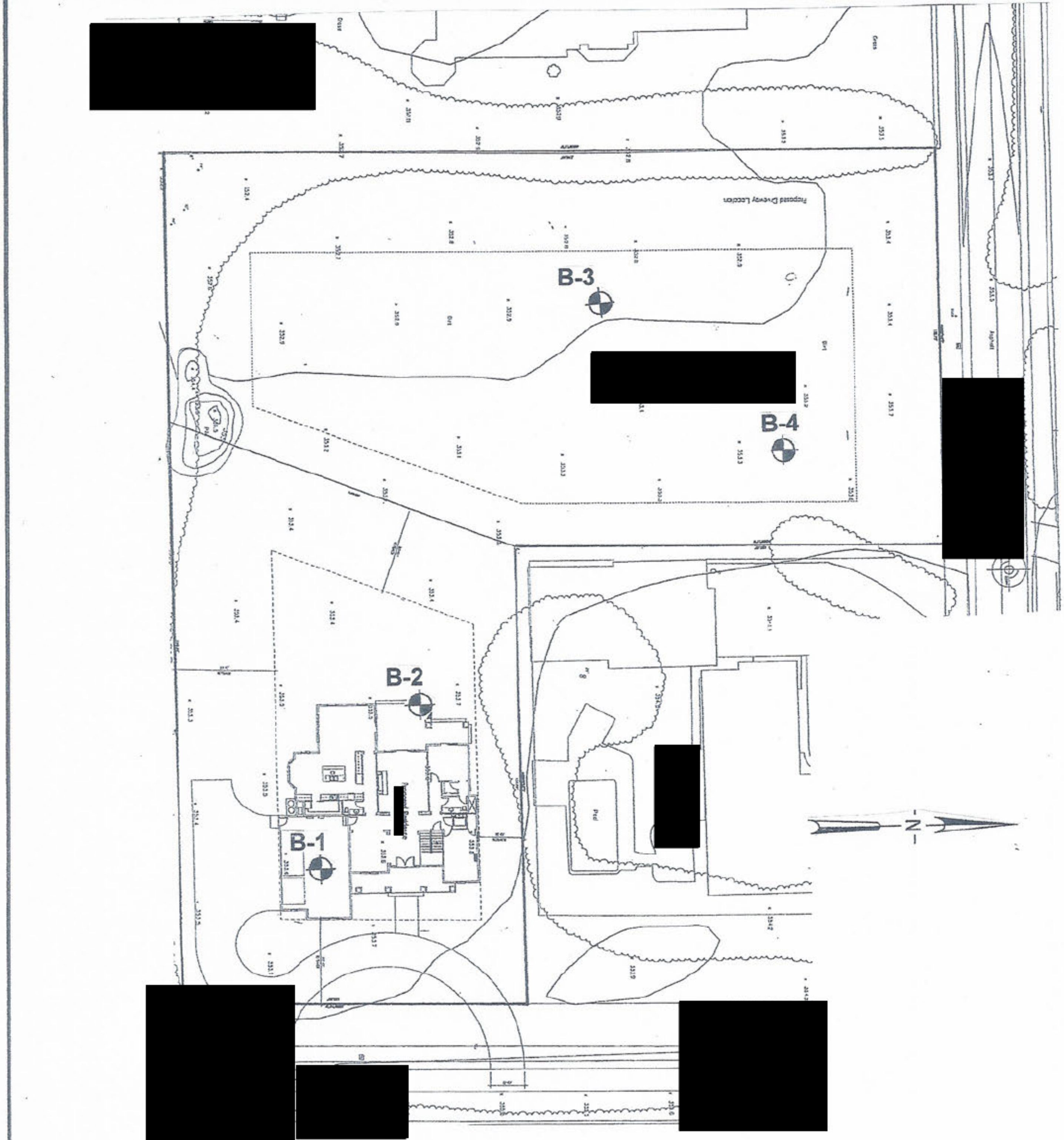
[redacted]

SOURCE

[redacted]

[redacted] Geotechnical Consultants	**GEOLOGIC MAP (Witter & Others)**
	Proposed [redacted]
	[redacted] Pleasanton, CA

Project No.	Date	FIGURE
[redacted]	December 23, 2020	5



LEGEND
Approximate location of exploratory borings

[redacted]

[redacted]
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SITE PLAN AND BORING LOCATION MAP

Proposed [redacted]
[redacted] Pleasanton, CA

Project No.	Date	FIGURE
[redacted]	December 23, 2020	6

LOGGED BY: [redacted]	BORING DIAMETER: [redacted]	BORING NO.: 1
DRILL RIG: [redacted]	SURFACE ELEVATION: [redacted]	DATE DRILLED: 12/07/2020

DEPTH (FT.)	SAMPLE NO.	SAMPLE TYPE	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION	USCS CLASSIFICATION	BLOWS/FT ADJUSTED TO SPT	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	REMARKS
	1-1			[redacted]	CL-ML	7	90 93	9.3 6.4	[redacted]
5	1-2			Same (slightly darker in color due to moisture)		14	94	12.8	
10	1-3			Same		13	94	12.8	
15	1-4			[redacted]	CL-ML	18	88	14.0	<u>Sieve Analysis</u> Sand 5% Fines 95%
20	1-5			[redacted]	CL	37	110	19.1	
				[redacted]					
25									
30									
35									

[redacted]

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EXPLORATORY BORING LOG

Proposed [redacted] Pleasanton, CA

PROJECT NO.	DATE	FIGURE
[redacted]	December 23, 2020	7

LOGGED BY: [redacted]	BORING DIAMETER: [redacted]	BORING NO.: [redacted]	
DRILL RIG: [redacted]	SURFACE ELEVATION: [redacted]	DATE DRILLED:	12/07/2020

DEPTH (FT.)	SAMPLE NO.	SAMPLE TYPE	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION	USCS CLASSIFICATION	BLOWS/FT ADJUSTED TO SPT	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	REMARKS
	2-1			[redacted]	CL-ML	7	75 83	5.7 5.9	[redacted]
5	2-2				ML	12	92 99	7.2 7.5	
10	2-3			Same		12	99	7.0	
15	2-4			[redacted]	CL-ML	9	96	13.9	
20	2-5			[redacted]	CL	18	103	24.2	
				[redacted]					
25									
30									
35									

[redacted]

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EXPLORATORY BORING LOG

Proposed [redacted]

PROJECT NO.	DATE	FIGURE
[redacted]	December 23, 2020	8

LOGGED BY:	[redacted]	BORING DIAMETER:	[redacted]	BORING NO.:	3
DRILL RIG:	[redacted]	SURFACE ELEVATION:	[redacted]	DATE DRILLED:	12/07/2020

DEPTH (FT.)	SAMPLE NO.	SAMPLE TYPE	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION	USCS CLASSIFICATION	BLOWS/FT ADJUSTED TO SPT	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	REMARKS
				[redacted]	[redacted]-ML				
	3-1			[redacted]	SM	11	108	2.9	
5	3-2				CL-ML	17	97	6.3	[redacted]
10	3-3			[redacted]		12	97	10.6	
15	3-4			[redacted]	CL	13	99	14.2	
20	3-5			Dark gray Silty Clay, moist, very stiff to hard	CL	26	113	16.7	
				Bottom of boring at 21.5 feet Groundwater not encountered					
25									
30									
35									

[redacted]

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EXPLORATORY BORING LOG

Proposed [redacted]

[redacted] Pleasanton, CA

PROJECT NO.	DATE	FIGURE
[redacted]	December 23, 2020	9

LOGGED BY: [redacted]	BORING DIAMETER: [redacted]	BORING NO.: 4
DRILL RIG: [redacted]	SURFACE ELEVATION: [redacted]	DATE DRILLED: 12/07/2020

DEPTH (FT.)	SAMPLE NO.	SAMPLE TYPE	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION	USCS CLASSIFICATION	BLOWS/FT ADJUSTED TO SPT	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	REMARKS
				[redacted]	CL-ML				
	4-1					10	86 88	4.8 5.0	LL=23% PI=4
	4-2					10	101	6.9	Sieve Analysis Gravel 1% Sand 33% Fines 66%
5									
	4-3			Same, no gravel		12	99	7.5	
10									
	4-4			[redacted]	CL	8	99	23.7	Q_u= 3,417 psf Strain 10.3%
15									
	4-5			[redacted]	CL	28	114	16.7	Q_u= 11,895 psf Strain 10.8%
20				Bottom of boring at 19.5 feet Groundwater not encountered					
25									
30									
35									

[redacted]
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EXPLORATORY BORING LOG

Proposed [redacted] Pleasanton, CA

PROJECT NO.	DATE	FIGURE
[redacted]	December 23, 2020	10

TABLE 1
SUMMARY OF LABORATORY TEST RESULTS

Sample No.	Depth (ft.)	Dry Density (p.c.f.)	Moisture Content (%)	Atterberg Limits		Unconfined Compressive Strength	Sieve Analysis
				Liquid Limit (%)	Plasticity Index		
1-1a	2.0	90	7.3	23	4		Sand 34% Fines 66%
1-1b	2.5	93	6.4				
1-2a	6.0	94	12.8				
1-3	11.0	94	12.8				
1-4	15.0	88	14.0				Sand 5% Fines 95%
1-5	21.0	110	19.1				
2-1a	1.5	75	5.7	26	6		
2-1b	2.0	83	5.9				Sand 30% Fines 70%
2-2a	3.5	92	7.2				
2-2b	4.0	99	7.5				
2-3	9.0	99	7.0				
2-4	14.0	96	13.9				
2-5	19.0	103	24.2				
3-1	3.0	108	2.9				
3-2	6.0	97	6.3				Gravel 2% Sand 31% Fines 67%
3-3	11.0	97	10.6				
3-4	16.0	99	14.2				
3-5	21.0	113	16.7				
4-1a	2.0	86	4.8				
4-1b	3.0	88	5.0				

TABLE 1, Continued
SUMMARY OF LABORATORY TEST RESULTS

Sample No.	Depth (ft.)	Dry Density (p.c.f.)	Moisture Content (%)	Atterberg Limits		Unconfined Compressive Strength	Sieve Analysis
				Liquid Limit (%)	Plasticity Index		
4-2	4.0	101	6.9	23	4		Gravel 1% Sand 33% Fines 66%
4-3	9.0	99	7.5				
4-4	14.0	99	23.7			3,417 psf Strain 10.3%	
4-5	19.0	114	16.7			11,895 psf Strain 10.8%	