

Appendix 3.7-1

Preliminary Geotechnical Evaluation

REPORT:

Preliminary Geotechnical Evaluation

Moffett Field NASA Research Park

Master Planning Phase

Mountain View, California

Langan Project No. 731762801

March 14, 2023

Minor refinements to the proposed project (e.g., pile driving is no longer proposed as part of the project) were made after the preparation of the Preliminary Geotechnical Evaluation for the Berkeley Space Center at NASA Research Park. These refinements do not change the conclusions and/or recommendations in the Preliminary Geotechnical Evaluation.

Langan

MOFFETT FIELD NASA RESEARCH PARK: STAGE 1

EXECUTIVE SUMMARY – GEOTECHNICAL

Anticipated Subsurface Conditions at the Site:

- Upper 5 to 15 feet of near-surface soil is stiff to hard clay that has a **high expansion** potential
- Underlying the near-surface soils are alluvial deposits consisting primarily of clay with sand and gravel interbeds
 - The clay is overconsolidated and moderately compressible
 - The sand and gravel interbeds are potentially liquefiable
- Preliminary design groundwater level at a depth of **5 feet** below the ground surface.

Potential Geotechnical Concerns at the Site:

- Highly expansive near-surface soil
- Shallow groundwater
- Providing adequate foundation support
- Potential for total and differential settlements under new building loads
- Potential for liquefaction and liquefaction-induced settlement at the site and adjacent areas during a major earthquake
- Excavation support and dewatering

Building Type	Construction Type	Range of Stories Above Grade	Estimated Building Loads Range (psf)	Estimated Static Settlement for Shallow Foundations (with no ground improvement)		Probable Foundation Type
				Total Settlement Range (inches)	Differential Settlement over 50 feet (inches)	
Lab/Office	Type I with Steel Framed Floors and Roof	2 to 4	620 to 1,010	1-1/2 to 2	1/4	Shallow Foundations (if static and seismic settlements are acceptable) -Or- Deep Foundations (driven PCPS concrete piles or DD piles) -Or- Shallow Foundations over Ground Improvement (DDCs)
	Type I with Steel Framed Floors and Roof	5 to 8	1,205 to 1,790	2 to 2-3/4	1/2	
Residential	Type V over Type I Podium	2 to 4	395 to 680	1 to 1-1/2	1/4	
	Type V over two Type I Podium	3 to 6	695 to 1,075	1-1/2 to 2	1/4	
Long-Span	Steel Framed Roof over Concrete Floor	1	500	1-1/2 to 2 (60-ft x 30-ft span to 60-ft x 60-ft span)	1/4	

Note: In addition to the static settlements, up to 2 inches of liquefaction-induced settlements could occur.

Building Type	Construction Type	Range of Stories Above Grade	Estimated Building Loads Range (psf)	Estimated Static Settlement for Buildings with One-Level Basement		Probable Foundation Type
				Total Settlement (inches)	Differential over 50 feet (inches)	
Lab/Office	Type I over Concrete Basement	2 to 4	1,370 to 1,760	1/8 to 1/2	1/4	Mat Foundation (if static and seismic settlements are acceptable)
	Type I over Concrete Basement	5 to 8	1,955 to 2,540	1/2 to 3/4	1/4	

- Notes:
- 1) Structural engineer to determine the need for tiedowns.
 - 2) Excavation anticipated to be 20 feet below existing grade
 - 3) In addition to the static settlements, up to 1 inch of liquefaction-induced settlements could occur.



14 March 2023

Ms. Kristina Covall
Moffett Properties, LLC
601 California Street, Suite 1310
San Francisco, California 94108

**SUBJECT: Preliminary Geotechnical Evaluation
Moffett Field NASA Research Park
Master Planning Phase
Mountain View, California
Langan Project No. 731762801**

Dear Kristina:

This letter report presents the results of our preliminary geotechnical evaluation for the proposed Moffett Field NASA Research Park development adjacent to Moffett Field in Mountain View, California. Our scope of services was performed in accordance with our proposal dated 16 November 2021. The information provided in this letter report is based on discussions with you and your team and review of data from previous investigations by Langan and others in the site vicinity.

The project site is approximately 36.2 acres and is located adjacent to Moffett Field. The project site location is shown on Figure 1. It is approximately triangular in shape and is bound by Wescoat Road to the northwest, Cody Road to the east, and a planned residential development (unaffiliated with this project) to the south. It is currently occupied by approximately 15 commercial and office buildings, surrounded by surface parking lots. Current plans indicate all the existing buildings, parking lots, and roads at the site will be demolished. We understand the project team is in the master planning phase and intends to explore opportunities to repurpose and redevelop the site to create an innovation hub.

Based on our discussions with the project team, we understand that the proposed mixed-use development may include the construction of lab/office and residential buildings, as well as long-span, warehouse-type structures. The lab/office buildings would be built either at-grade or over a one-level concrete basement. The residential buildings and long-span structures are planned to be built at-grade (no basements). At this time, the proposed structures have maximum height limits of approximately 130 feet.

According to our discussions with HOK Structure Engineers (HOK), we understand various floor levels and construction types (Type I and Type V) are being considered for the proposed structures during this master planning phase of the project. We understand that Type I is concrete and Type V refers to light framed (wood) construction. The proposed structures are considered to be built with column spacing of approximately 30 by 30 feet for lab/office and residential structures and either 60 by 30 feet or 60 by 60 feet for long-span structures. A summary of the

proposed construction types and building loads are presented in Tables 1, 2, and 3, respectively. The estimated building loads were calculated using the estimated floor loads provided by HOK.

TABLE 1
Proposed Lab/Office Construction Types and Estimated Building Loads

Building Type	Construction Type	Stories Above Grade	Basement Levels	Assumed Floor Layout			Estimated Total Building Loads (psf)
				Type	No. of Floors	Estimated Floor Load (psf ¹ /floor)	
Lab/Office	Type I with Steel Framed Floors and Roof	2 to 4	0	Transfer Level 1	1	300	620 to 1,010
				Typical Floor	1 to 3	195	
				Roof	1	125	
	Type I with Steel Framed Floors and Roof over Concrete Basement	2 to 4	1	Basement	1	750	1,370 to 1,760
				Transfer Level 1	1	300	
				Typical Floor	1 to 3	195	
				Roof	1	125	
	Type I with Steel Framed Floors and Roof	5 to 8	0	Transfer Level 1	1	300	1,205 to 1,790
				Typical Floor	4 to 7	195	
				Roof	1	125	
	Type I with Steel Framed Floors and Roof over Concrete Basement	5 to 8	1	Basement	1	750	1,955 to 2,540
				Transfer Level 1	1	300	
				Typical Floor	4 to 7	195	
				Roof	1	125	

¹ psf = pounds per square foot

TABLE 2
Proposed Residential Construction Type and Estimated Building Loads

Building Type	Construction Type	Stories Above Grade	Assumed Floor Layout			Estimated Total Building Loads (psf)
			Type	No. of Floors	Estimated Floor Load (psf/floor)	
Residential	Type V over Type I Podium	2 to 4	Podium	1	300	395 to 680
			Typical Floor	1 to 4	95	
	Type V over two Type I Podium	3 to 6	Podium	2	300	695 to 1,075
			Typical Floor	1 to 4	95	

Table 3
Proposed Long-Span Structure Construction Type and Estimated Building Loads

Building Type	Construction Type	Stories Above Grade	Assumed Floor Layout			Estimated Total Building Loads (psf)
			Type	No. of Floors	Estimated Floor Load (psf/floor)	
Long-Span	Steel Framed Roof over Concrete Slab	1	Concrete Slab	1	400	500
			Roof	1	100	

1.0 SCOPE OF SERVICES

Our scope of services was performed in accordance with our proposal dated 16 November 2021. The purpose of our services is to provide you with a preliminary evaluation of the geotechnical conditions at the site and potential geotechnical issues associated with the various proposed construction and building types. Our preliminary assessment is based on existing subsurface data and available geologic maps; additional field exploration was not part of this scope of services. We performed preliminary engineering analyses using the existing data to develop preliminary conclusions and recommendations regarding:

- soil and groundwater conditions
- preliminary design criteria for the recommended foundation type(s), consisting of values for vertical and lateral resistance and required foundation embedment
- estimated foundation settlement, including total and differential settlements
- appropriate ground improvement methods, including preliminary evaluation and commentary on the usefulness of a potentially site-wide surcharge (or other appropriate methodologies) program

- temporary shoring and dewatering
- site seismicity
- seismic hazards
- methods for mitigation of liquefaction potential, if appropriate
- site preparation and grading, including criteria for site stripping/clearing, fill quality and compaction
- temporary and permanent cut and fill slopes
- seismic design considerations
- construction considerations.

2.0 DOCUMENT REVIEW

To evaluate potential site conditions, we reviewed geotechnical reports performed by Langan and others in the vicinity of the site. Documents we reviewed include:

- Geomatrix. Preliminary Geotechnical Investigation, Proposed Commercial Development, NASA Research Park, Mountain View, California, Project Number 6534, dated 4 August 2000.
- Fugro West, Inc., Geotechnical Study, NASA Ames Water Storage Tank, Moffett Field, California, dated 19 November 2009.
- Langan Engineering and Environmental Services, Inc., Updated Geotechnical Investigation, Hanger 1, Moffett Federal Airfield, Mountain View, California, dated 23 September 2020.
- Langan Engineering and Environmental Services, Inc., Preliminary Geotechnical Investigation, NASA Research Park/Mountain View Housing Ventures LLC, Mountain View, California, dated 03 December 2018.
- Langan Engineering and Environmental Services, Inc., Preliminary Geotechnical Evaluations, North Bayshore Residential Phase 1, Mountain View, California, dated 12 May 2022.
- Langan Engineering and Environmental Services, Inc., *Draft Preliminary Geotechnical Investigation, 465 Fairchild Drive, Mountain View, California*, Project Number 770648801, dated 13 April 2018
- Langan Engineering and Environmental Services, Inc., Geotechnical Investigation, Site Improvements, 184 North Shoreline Boulevard, Mountain View, California, dated 22 March 2017.

- Langan Engineering and Environmental Services, Inc., Geotechnical Investigation, Pear Avenue, Phase 2 (South Parcel), Mountain View, California, dated 25 April 2018.
- Langan Engineering and Environmental Services, Inc., Geotechnical Investigation, Site Improvements, 184 North Shoreline Boulevard, Mountain View, California, dated 22 March 2017.
- Lowney Associates, Geotechnical Investigation, Pear Avenue Research and Development Building, Mountain View, California, dated 21 November 1997.
- TRC Lowney, Geotechnical Investigation, Loral Satellite Assembly, Integration and Test Facility, NASA Research Park, Mountain View, California, dated 4 August 2006.
- TRC Lowney, Geotechnical Investigation, Loral Satellite Assembly, Integration and Test Facility, Part 2, NASA Research Park, Mountain View, California, dated 29 August 2006.
- Treadwell & Rollo, A Langan Company, Geotechnical Investigation, Bay View Campus at NASA Ames, Off-site Improvements, Mountain View, California, Project Number 730467303, dated 17 August 2012.
- Treadwell & Rollo, A Langan Company, Geotechnical Investigation, 1350 Shorebird Way, Mountain View, California, dated 14 September 2012.
- Treadwell & Rollo, A Langan Company, Geotechnical Investigation, Solar Carports, 1565 Charleston Road, Mountain View, California, dated 22 March 2012.
- Treadwell & Rollo, Preliminary Geotechnical Investigation Report, Google Campus–Charleston East Property, Mountain View, California, dated 11 January 2008.

3.0 EXISTING SITE CONDITIONS

The site is currently occupied by 15 commercial and office buildings and surrounding at-grade parking lots and driveways within the NASA campus, as shown on Figure 2. Our review of historical aerial photographs and topographic maps (Historic Aerials, 2023) indicates the site was vacant until construction of the existing structures, which mostly occurred between 1956 and 1987. The project site is relatively level, with site grades ranging from approximate Elevations 20 and 30 feet², with a general slope downward from the southern boundary to the northern boundary of the site.

4.0 SUBSURFACE CONDITIONS

The site is in Santa Clara Valley, which is principally underlain by stream-deposited alluvium consisting of interbedded layers of clay, silt, sand, and gravel. The results of our document review indicate that the site is underlain by alluvial deposits that predominantly consist of medium stiff to hard clay and sandy clay with interbedded lenses and layers of medium dense to very dense

² All elevations reference World Geodetic System Datum of 1984 (WGS84) as shown on Google Earth.

sand and gravel with varying amounts and types of fines to the maximum depth explored of 100 feet below ground surface (bgs). The alluvial deposits are deep, estimated to be on the order of a thousand feet or more before encountering bedrock.

Based on our review of existing nearby borings, we anticipate the near-surface clay is stiff to hard silty clay, sandy clay and clay with silt and varies in thickness from about 5 to 15 feet. Atterberg limit test results indicate the near-surface clay has a high expansion potential³. Below the near-surface clay, the site is underlain by medium stiff to very stiff clays with variable amounts of sand, silt, and gravel, and interbedded layers of medium to very dense sand with variable amounts of silt, clay, and gravel. Lab results indicate the alluvial clay is overconsolidated⁴ and moderately compressible.

Groundwater was encountered during previous investigations at or near the site at depths ranging between about 6 to 13 feet bgs. However, these measurements may not reflect stabilized groundwater levels. Seasonal fluctuations in rainfall influence groundwater levels and may cause several feet of variation. Based on the California Geological Survey (CGS) Seismic Hazard Zone Report for the Mountain View Quadrangle (CGS, 2006), the historic high groundwater level is about 5 feet.

We preliminarily estimate a design groundwater level at a depth of 5 feet bgs. Based on our review of the site history (Envirostor, 2018), we understand surrounding industries have significantly contributed to solvent contamination of the groundwater in the region. The environmental study of the soil and groundwater at site is not included in this report and is being conducted by others.

5.0 SEISMICITY

The project site is in a seismically active region. Numerous earthquakes have been recorded in the region in the past, and moderate to large earthquakes should be anticipated during the service life of the proposed development. The Monte Vista-Shannon, San Andreas, Hayward, and Calaveras faults are the major faults closest to the site. These and other faults of the region are shown on Figure 3. For each of these faults, as well as other active faults within about 50 kilometers (km) of the site, the distance from the site and estimated mean Moment magnitude⁵ [2014 Working Group on California Earthquake Probabilities (WGCEP) (2015) and Uniform California Earthquake Rupture Forecast Version 3 (UCERF3) as detailed in the United States Geological Survey Open File Report 2013-1165] are summarized in Table 4. The mean Moment magnitude presented on Table 4 was computed assuming full rupture of the segment using Hanks and Bakun (2008) relationship.

³ Moderately expansive soil undergoes moderate volume changes in moisture content. Highly expansive soil undergoes large volume changes with changes in moisture content.

⁴ An overconsolidated clay has experienced a pressure greater than its current load.

⁵ Moment magnitude is an energy-based scale and provides a physically meaningful measure of the size of a faulting event. Moment magnitude is directly related to average slip and fault rupture area.

TABLE 4
Regional Faults and Seismicity

Fault Segment	Approx. Distance from fault (km)	Direction from Site	Mean Moment Magnitude
Monte Vista-Shannon	9	Southwest	7.0
San Andreas 1906 event	14	Southwest	8.1
Pilarcitos	15	Southwest	6.7
Total Hayward-Rodgers Creek Healdsburg	15	East	7.6
Mission (connected)	19	Northeast	6.1
Total Calaveras	21	East	7.5
Butano	24	South	6.7
Sargent	32	Southeast	6.8
Total San Gregorio	32	West	7.6
Mount Diablo Thrust	41	Northeast	6.6
Greenville	45	Northeast	7.1
Franklin	49	North	6.7
Contra Costa (Lafayette)	50	North	6.1
Contra Costa (Larkey)	50	North	6.0

Figure 3 also shows the earthquake epicenters for events with magnitude greater than 5.0 from January 1800 through August 2014. Since 1800, four major earthquakes have been recorded on the San Andreas fault. In 1836 an earthquake with an estimated maximum intensity of VII on the Modified Mercalli (MM) scale (Figure 4) occurred east of Monterey Bay on the San Andreas fault (Toppozada and Borchardt 1998). The estimated Moment magnitude, M_w , for this earthquake is about 6.25. In 1838, an earthquake occurred with an estimated intensity of about VIII-IX (MM), corresponding to an M_w of about 7.5. The San Francisco Earthquake of 1906 caused the most significant damage in the history of the Bay Area in terms of loss of lives and property damage. This earthquake created a surface rupture along the San Andreas fault from Shelter Cove to San Juan Bautista approximately 470 kilometers in length. It had a maximum intensity of XI (MM), an M_w of about 7.9, and was felt 560 kilometers away in Oregon, Nevada, and Los Angeles. The Loma Prieta Earthquake occurred on 17 October 1989 in the Santa Cruz Mountains with an M_w of 6.9, the epicenter of which is approximately 44 km from the site.

In 1868 an earthquake with an estimated maximum intensity of X on the MM scale occurred on the southern segment (between San Leandro and Fremont) of the Hayward fault. The estimated M_w for the earthquake is 7.0. In 1861, an earthquake of unknown magnitude (probably an M_w of about 6.5) was reported on the Calaveras fault. The most recent significant earthquake on this fault was the 1984 Morgan Hill earthquake ($M_w = 6.2$).

The 2016 U.S. Geologic Survey (USGS) predicted a 72 percent chance of a magnitude 6.7 or greater earthquake occurring in the San Francisco Bay Area in 30 years (Aagaard et al. 2016).

More specific estimates of the probabilities for different faults in the Bay Area are presented in Table 5.

TABLE 5
Estimates of 30-Year Probability (2014 to 2043) of a
Magnitude 6.7 or Greater Earthquake

Fault	Probability (percent)
Hayward-Rodgers Creek	33
Calaveras	26
N. San Andreas	22
Hunting Creek/ Berryessa/ Green Valley/ Concord/ Mt. Diablo/ Greenville	16
San Gregorio	6

6.0 SEISMIC HAZARDS

During a major earthquake on a segment of one of the nearby faults, strong to violent ground shaking is expected to occur at the project site. Strong ground shaking during an earthquake can result in ground failure such as that associated with soil liquefaction⁶, lateral spreading⁷, and seismic densification⁸. Each of these conditions has been preliminarily evaluated based on our literature review, field investigation and analyses, and is discussed in this section.

6.1 Liquefaction

When a saturated, cohesionless soil liquefies during a major earthquake, it experiences a temporary loss of shear strength caused by a transient rise in excess pore water pressure generated by strong ground motion. Flow failure, lateral spreading, differential settlement, loss of bearing, ground fissures, and sand boils are evidence of excess pore pressure generation and liquefaction.

⁶ Liquefaction is a transformation of soil from a solid to a liquefied state during which saturated soil temporally loses strength resulting from the buildup of excess pore water pressure, especially during earthquake-induced cyclic loading. Soil susceptible to liquefaction includes loose to medium dense sand and gravel, low-plasticity silt, and some low-plasticity clay deposits.

⁷ Lateral spreading is a phenomenon in which surficial soil displaces along a shear zone that has formed within an underlying liquefied layer. Upon reaching mobilization, the surficial blocks are transported downslope or in the direction of a free face by earthquake and gravitational forces.

⁸ Seismic densification is a phenomenon in which non-saturated, cohesionless soil is compacted by earthquake vibrations, causing ground surface settlement.

As shown on Figure 5, the site is mapped within a liquefaction hazard zone as designated by the California Geological Survey (CGS) seismic hazard zone map for the area titled State of California Seismic Hazard Zones, Mountain View Quadrangle, Official Map, released October 18, 2006. Specifically, the map shows the site is in an area "where historic occurrence of liquefaction, or local geological, geotechnical and groundwater conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required."

Based on results of our liquefaction analysis using a select number of projects by Langan where we have electronic cone penetration test (CPT) data, we conclude isolated layers of loose to medium dense sandy soil encountered in the borings and CPTs at random depths are susceptible to liquefaction. Assuming a design groundwater depth of 5 feet bgs, layers of medium dense saturated sand with varying amounts of clay and silts varying in thickness from approximately 2 to 6 feet were encountered below the groundwater level from depths of approximately 5 to 49 feet bgs. Based on our analyses, we conclude these layers could potentially liquefy during a major earthquake and may experience liquefaction-induced settlement.

Following the procedures presented in Boulanger and Idriss (2014), we evaluated the liquefaction potential at the site. The Boulanger and Idriss (2014) procedures are updates of the Idriss and Boulanger (2008) procedures and the simplified procedures developed by Seed and Idriss (1971) and later by the 1996 NCEER and the 1998 NCEER/NSF workshops on the Evaluation of Liquefaction Resistance of Soils (Youd and Idriss 2001). In our evaluation, we considered a peak ground acceleration (PGA_M) of 0.59g based on the Risk-Targeted Maximum Considered Earthquake (Section 7.10). Based on our evaluation, we estimate liquefaction-induced settlement of these layers could be on the order of ½ to 2 inches. Because the potentially liquefiable layers are discontinuous, we estimate that up to 1½ inch of differential settlement may occur during an earthquake.

Additional building-induced settlement could potentially occur under shallow foundations if a liquefiable layer is present at or near the bottom of footings. This will need to be assessed during the final geotechnical investigation.

6.2 Seismic Densification

Seismic densification can occur during strong ground shaking in loose, clean granular deposits above the water table, resulting in ground surface settlement. Considering the soil above the groundwater table is predominantly medium stiff to hard clay, we conclude the potential for ground settlement due to seismic densification is low. However, it is possible that there are pockets of soil susceptible to seismic densification at the site. We recommend additional exploration be performed during a final geotechnical investigation to further evaluate this hazard.

6.3 Lateral Spreading

Lateral spreading is a phenomenon in which a surficial soil displaces along a shear zone that has formed within an underlying liquefied layer. The surficial blocks are transported downslope or in the direction of a free face, such as a channel, by earthquake and gravitational forces.

Lateral spreading is generally the most pervasive and damaging type of liquefaction-induced ground failure generated by earthquakes. Researchers (Youd, Hansen and Bartlett, 2002) have concluded that potentially liquefiable soils with a corrected SPT blow count ($N_{1,60-CS}$) greater than 15 are not susceptible to lateral spreading.

The site is relatively flat and the nearest free face is Stevens Creek which is approximately 4,000 feet west of the site. Additionally, potentially liquefiable soils with approximate $(N_1)_{60,CS}$ blow counts less than 15 are not continuous across the site. In general, based on our document review and preliminary analysis, it appears the potential for lateral spreading at the site is low. However, we recommend further evaluation and additional exploration be performed during the final geotechnical investigation to assess the potential for lateral spreading.

6.3 Tsunami

Recent published maps (California Emergency Management Agency 2009) indicate the site is not within the tsunami inundation area; therefore, we conclude the potential risk by inundation from tsunami to be low for the site. However, the project civil engineer should evaluate the impact of sea level rise on the potential risk of inundation from a tsunami over the life of the building.

6.4 Fault Rupture

Historically, ground surface ruptures closely follow the traces of geologically young faults. The site is not within an Earthquake Fault Zone, as defined by the Alquist-Priolo Earthquake Fault Zoning Act and no known active or potentially active faults exist on the site. Therefore, we conclude the risk of fault offset at the site from a known active fault is low. In a seismically active area, the remote possibility exists for future faulting in areas where no faults previously existed; however, we conclude the risk of surface faulting and consequent secondary ground failure is low.

6.5 Flooding

Stevens Creek is located about 4,000 feet west of the site and salt ponds are located about 7,000 feet north of the site. The property is located within Federal Emergency Management Agency (FEMA) Flood Zone D, designated as "areas in which flood hazards are undetermined, but possible," (FEMA 2014). The project civil engineer should further evaluate the future effects of sea level rise and the potential for flooding at the project site.

7.0 PRELIMINARY CONCLUSIONS AND RECOMMENDATIONS

Our preliminary findings regarding potential geotechnical concerns for the site include:

- the presence of highly expansive near-surface soil
- shallow groundwater
- providing adequate foundation support
- potential for total and differential settlements under new building loads

- potential for liquefaction and liquefaction-induced settlement at the site and adjacent areas during a major earthquake
- excavation support and dewatering.

These concerns should be further evaluated during a detailed geotechnical investigation, particularly the extent of liquefaction potential at the site. Our preliminary conclusions regarding these concerns are discussed in the remainder of this report.

7.1 Expansive Soil Considerations

Based on the reports reviewed, the existing near-surface soil has high expansion potential. Moisture fluctuations in near-surface expansive soil could cause the soil to expand or contract resulting in movement and potential damage to improvements that overlie them. Potential causes of moisture fluctuations include drying during construction, and subsequent wetting from rain, capillary rise, landscape irrigation, poor drainage and type of plant selection.

The volume changes from expansive soils can cause cracking of foundations, floor slabs and exterior flatwork. New foundations, exterior slabs and concrete flatwork proposed in areas should be designed and constructed to resist the effects of the expansive soil. These effects can be mitigated by moisture conditioning the expansive soil, providing select, non-expansive fill below flatwork and other at-grade improvements, and providing additional reinforcing steel. New foundations can be embedded deeper than conventional footings to reduce the effects of expansive soil.

All fill placed beneath improvements should meet the following criteria:

- be free of organic matter
- contain no rocks or lumps larger than three inches in greatest dimension
- have a low expansion potential (defined by a liquid limit of less than 40 and plasticity index lower than 12)
- have a low corrosion potential⁹
- be approved by the geotechnical engineer.

We preliminarily recommend that at least 12 inches of select fill be placed above native soil in areas that will have concrete flatwork, and 24 inches of select fill beneath building slabs-on-grade. Materials for capillary break (sand and gravel) should not count as part of the select fill. The select

⁹ Low corrosion potential is defined as a minimum resistivity of 2,000 ohms-cm and maximum sulfate and chloride concentrations of 250 parts per million.

fill should extend at least five feet beyond building footprints. Select fill should meet the criteria previously presented in this section.

Alternatively, instead of placing select fill, the upper 24 inches of the existing surface soil in building pads may be lime treated to reduce the expansion potential and help winterize the site. We preliminarily recommend that at least 5 percent lime by weight of the soil be used to treat native soil for at-grade structures. The lime treatment contractor should evaluate the type and amount of lime to reduce the plasticity index of the soil to meet the select fill criteria. The lime treatment should extend at least five feet beyond building footprints where hardscape areas are planned; however, landscape areas should not be lime treated because the lime treated soil may make it difficult for the plants to survive. In landscape areas adjacent to building footprints and exterior slabs, select fill should be placed.

If the surface soil becomes wet, it may be difficult to compact during the winter unless it is dried. If required, the soil can be mixed with lime to aid in drying and compaction.

7.2 Shallow Foundations and Settlement

The primary considerations related to the selection of the foundation systems are whether the buildings will be constructed at-grade or with basements, the final building loads, the presence of highly expansive near-surface soil, liquefaction-induced settlements, and the anticipated building settlements resulting from the presence of moderately compressible soil. As previously outlined, we understand multiple building sizes, construction types, and basement levels (0 or 1) are being considered during this master planning phase.

Details for the probable foundation types are described in the following subsections.

7.2.1 At-Grade Buildings

The clay layers underlying the site are typically overconsolidated, and primary consolidation settlement of these layers is expected to be complete under existing fill and/or building loads. To evaluate the settlement of the site due to consolidation of the alluvial deposits under the weight of new building loads, we reviewed laboratory consolidation tests on relatively undisturbed samples of clay in the site vicinity. Test results from nearby sites indicate the alluvial clay generally has an overconsolidation ratio of 1.5 to 5. We assume there will be minimal grading at the site. However, if new fill is added to the site, on a preliminary basis we estimate about ¼ inch and 1/2 inch of consolidation settlement for one and two feet of new fill, respectively. This settlement would need to be considered in foundation selection and design.

Based on information provided by HOK, the proposed at-grade structures will impose average uniform pressures (dead plus live loads) on the order of 395 to 1,075 psf for residential, 620 to 1,790 psf for lab/office, and about 500 psf for long-span structures. Table 6 presents estimated total and differential static settlements of shallow foundations based on the building loading information provided, and an allowable bearing pressure of 3,500 psf.

Shallow foundations systems such as spread footings or a mat could be used provided the estimated settlements are tolerable. Footings should be at least 18 inches wide for continuous footings and 24 inches wide for isolated spread footings and can be preliminarily designed using an allowable bearing pressure of 3,500 psf with a one-third increase for wind or seismic loading. To reduce the effects of expansive soil, we recommend that perimeter footings be embedded at least 36 inches below the lowest adjacent final soil subgrade, and interior spread footings be embedded at least 30 inches below the lowest adjacent final soil subgrade. Footings adjacent to utility trenches should bear below an imaginary 1.5:1 (horizontal to vertical) plane projected upward from the bottom edge of the utility trench. The estimated differential settlements do not include the rigidity of the foundation system. A rigid mat system would reduce the differential settlements.

TABLE 6
Estimated Static Settlements for At-Grade Buildings

Building Type	Construction Type	Range of Stories Above Grade	Estimated Building Loads Range (psf)	Estimated Static Settlement for Shallow Foundations	
				Total Settlement Range (inches)	Differential over 50 feet (inches)
Lab/Office	Type I with Steel Framed Floors and Roof	2 to 4	620 to 1,010	1-1/2 to 2	1/4
	Type I with Steel Framed Floors and Roof	5 to 8	1,205 to 1,790	2 to 2-3/4	1/2
Residential	Type V over Type I Podium	2 to 4	395 to 680	1 to 1-1/2	1/4
	Type V over two Type I Podium	3 to 6	695 to 1,075	1-1/2 to 2	1/4
Long-Span	Steel Framed Roof over Concrete Floor	1	500	1-1/2 to 2 (60-ft x 30-ft span to 60-ft x 60-ft span)	1/4

Note:

1. In addition to the static settlements, up to 2 inches of liquefaction-induced settlements could occur.

Furthermore, as previously discussed, we have estimated 1/2 to 2 inches of liquefaction-induced settlement could occur during a major earthquake at the site; based on the expected variability,

liquefaction-induced differential settlement could be on the order of 1-1/2 inches over 50 feet. These settlements are in addition to the predicted static settlement. If these settlements are not tolerable then ground improvement or deep foundations will be required.

Lateral loads can be resisted by a combination of passive resistance acting against the vertical faces of footings and friction along the footing bases. On a preliminary basis, passive resistance can be calculated using an allowable uniform pressure (rectangular distribution) of 1,000 psf; the upper foot of soil should be ignored unless confined by a concrete slab or pavement. Frictional resistance should be computed using a base friction coefficient of 0.25, considering no waterproofing. If waterproofing is installed at the base of footings, a base friction coefficient of 0.15 should be used. Passive resistance and base friction values include a factor of safety of about 1.5 and may be used in combination without reduction.

Ground improvement methods can be used to mitigate the effects of liquefaction, improve bearing capacity of soils, and/or transfer the support of building loads through the elements to deeper, more competent soil. On the basis of our experience with different methods of ground improvement, we judge that the most appropriate methods to improve the ground at the site are drilled displacement columns (DDCs) and aggregate piers (APs). APs may not be suitable because the elements are backfilled with aggregate baserock that could provide a conduit for water to rise and may not be environmentally suitable.

If ground improvement is performed, the allowable bearing pressure would increase and settlements will decrease. Because ground improvement systems are installed by specialty design-build contractors we do not provide specific design recommendations. However, based on our experience typical allowable bearing capacities of 4,500 to 6,000 psf could be achievable. Because of environmental issues, we understand it would be desirable to limit ground improvement elements to lengths of about 20 feet. Using a length of 20 feet for the ground improvement elements, we preliminarily estimate static settlements would be about 1 inch for average uniform building loads less than about 900 psf and liquefaction-induced settlements would be about 1 inch; however, building settlements would need to be confirmed by the specialty contractor along with the spacing elements.

For at-grade structures with average uniform building loads greater than about 900 psf, deep foundations may be required to limit total settlement to less than 1 inch. Preliminary recommendations for deep foundations are presented in Section 7.3.

7.2.2 Buildings with Basements

An excavation of about 20 feet is anticipated for a one-level basement. Based on the available subsurface data, we anticipate the soil below the bottom of the excavation will generally consist of medium stiff to very stiff clay or medium dense to dense sand. We judge these soil layers can support moderate to heavy foundation loads without excessive static settlement, especially because of the removal of the overburden load of the soil within the excavation depths. Groundwater is anticipated as high as five feet bgs and the basement slabs and walls will need to account for the water and hydrostatic pressure. Therefore, we preliminarily conclude buildings

with basements can be supported on a mat foundation, provided the estimated settlements are tolerable.

Removing 20 feet of soil at 120 to 125 pcf per foot of soil will result in a net unloading of pressure of about 2,400 to 2,500 psf in the soil. For most of the building scenarios proposed, there will be a net unloading of the soil versus applied building loads, except for lab/office structures with 7 or 8 stories. However, because the clay layers are overconsolidated, the soil will be entirely in recompression under all of the expected building loads.

For buildings with basements, initially, the soil beneath the foundation level will rebound or swell as the weight of the soil above is removed during excavation. As the building loads are applied, the soil will recompress. Where there is a net unloading of soil, we anticipate the proposed structures will settle moderately due to recompression of the soil under the building loads. We estimate total static settlement should be less than one inch. Settlement of new building loads for scenarios will need to be analyzed using a subgrade modulus approach. In this approach differently loaded areas, e.g. highly loaded core areas versus more lightly loaded perimeter columns, for example, will have different subgrade moduli, depending on the anticipated settlement at each area, and an iterative analysis should be undertaken between us and the structural engineer for the mat design. We preliminary recommend a modulus of subgrade reaction on the order of 45 to 60 kips per cubic foot (kcf).

The majority of the anticipated settlement should occur during construction. Differential settlement will depend on the rigidity of the mat. On a preliminary basis, we estimate differential settlement could be on the order of about 1/4 inch between column locations. Additionally, seismically-induced settlement could potentially be up to 1 inch below the proposed basement level. The mat should span an unsupported area of 5 feet in diameter in any location within the interior. The evaluation for areas of non-support is purely empirical and is not meant to model actual site performance; the purpose is to establish mat stiffness to control differential settlement.

These settlements should be accounted for in the design of the mat and the structure, particularly at building entrances and other areas where there are structural or aesthetic connections between the surrounding areas outside of the building that should not experience static ground settlement. However, the adjacent sidewalks, streets, and underground utilities could experience seismically-induced settlements of up to two inches, and they may become damaged following a strong earthquake.

There is a possibility that liquefiable material will be present at the foundation level, in which case some additional excavation or ground improvement may be needed. These conditions will be evaluated during design-level geotechnical investigations.

Using a design groundwater at 5 feet below grade, a 16-foot-deep basement, and a 4-foot-thick mat (per HOK), the bottom of the mat will extend about 15 feet below groundwater. The building will need to be designed for hydrostatic pressure and the structural engineer should determine, if tiedowns are needed to resist hydrostatic uplift. However, the environmental impact of installing tiedowns should be evaluated.

Table 7 summarizes the anticipated probable foundation types and estimated static settlement for the proposed one-level basement building scenarios. Preliminary conclusions regarding the anticipated probable foundation systems for these building scenarios are described in the following sections.

TABLE 7
Estimated Static Settlements for Buildings with One-Level Basement

Building Type	Construction Type	Range of Stories Above Grade	Estimated Building Loads Range (psf)	Estimated Static Settlement	
				Total Settlement (inches)	Differential over 50 feet (inches)
Lab/Office	Type I over Concrete Basement	2 to 4	1,370 to 1,760	1/8 to 1/2	1/4
	Type I over Concrete Basement	5 to 8	1,955 to 2,540	1/2 to 3/4	1/4

Notes:

1. Structural engineer to determine the need for tiedowns.
2. Excavation anticipated to be 20 feet below existing grade
3. In addition to the static settlements, up to 1 inch of liquefaction-induced settlements could occur.

Lateral forces can be resisted by a combination of passive resistance against the vertical face of the mat foundation and friction along the base of the mat. Passive resistance may be calculated using lateral pressures corresponding to a uniform pressure of 1,000 psf. Friction along the bottom of the foundation will be affected by the type of waterproofing system used. The waterproofing manufacturer should provide a recommended friction coefficient based on testing of their product. For initial estimating, frictional resistance can be computed using a base friction coefficient of 0.15, which assumes a waterproofing membrane is placed below the mat. These values include a factor of safety of about 1.5 and may be used in combination without reduction. The mat should be designed to resist hydrostatic pressure based on the design groundwater level.

7.3 Deep Foundations

Because of the potential contamination issues at site we judge displacement piles would be appropriate to minimize soil cuttings. We judge appropriate deep foundations for the proposed project are driven precast, pre-stressed (PCPS) concrete piles, or Drilled Displacement (DD) piles. The use of the piles would need to be approved by the governing regulatory agencies. The piles will primarily gain capacity from friction in the interbedded clay and sand layers at the site. Piles will develop lateral resistance from the passive pressure acting on the upper portion of the piles and their structural rigidity.

We recommend a preliminary allowable skin friction of 700 psf for dead plus live loads in the interbedded sand and clay layers below the site. These preliminary skin friction values may be used in pricing and estimating; however, final axial pile capacities should be determined by the specialty design/build contractor after the pile type has been chosen, and should be verified by a test program.

Deep foundations should be able to be designed to settle less than one inch, with less than ½ inch of differential settlements between columns, under static conditions.

7.3.1 Driven Piles

If noise and vibrations are acceptable, driven PCPS concrete piles could be used for support of the structures. An indicator pile and static and dynamic load test programs should be performed to provide additional information regarding ease of installation and production pile lengths and capacities. Indicator piles should be installed with the same equipment that will be used to install the production piles. For planning purposes, the pile test program should consist of indicator piles being monitored during driving with a pile driving analyzer (PDA). All indicator piles should be retapped after the initial drive. A Case Pile Wave Analysis Program (CAPWAP) analysis should be performed on a representative blow during initial drive, near the end of driving, and near beginning of retap. Static load tests should be performed in accordance with ASTM D3689 and ASTM D1143 for tension and compression load tests, respectively.

7.3.2 Drilled Displacement Piles

DD piles are installed with a displacement tool that results in displacement and densification of the surrounding soil. They are also installed by drilling to the required depth and injecting grout as the tool is withdrawn. While the grout is still fluid, a steel reinforcing cage is inserted into the shaft. 16- and 18-inch diameter DD piles are typical.

An indicator pile and load test program should be performed for DD piles to provide additional information regarding ease of installation and production pile lengths and capacities. Indicator piles should be installed with the same equipment that will be used to install the production piles. For planning purposes, the load test program should consist of static load testing of at least two piles in compression and two piles in tension in accordance with ASTM D1143 and D3689, respectively, per building. Details regarding the recommended indicator pile and testing program will be presented in a design-level geotechnical investigation report after site investigation is performed.

7.4 Ground Improvement

Ground improvement can be performed to allow for support of the proposed building using shallow foundations on improved ground, if the total and differential settlements discussed in Section 7.2 are not tolerable. On the basis of our experience with different methods of improvement and considering the potential environmental concerns with APs, we judge that the most appropriate method for this site is drilled displacement columns (DDCs).

DDCs are constructed by using a displacement auger to create a soil shaft that is filled with grout as the displacement auger is withdrawn from the hole. DDCs typically vary between 18 to 30 inches in diameter. Installation of DDCs produces minimal soil cuttings because the soil is displaced during column installation.

This system is installed under design-build contracts by specialty contractors, and we do not provide specific design recommendations or settlement estimates for these systems; however, we are providing design guidelines that should be considered in the design of the ground improvement.

As discussed previously, ground improvement can be used to mitigate the effects of liquefaction, improve bearing capacity of soils, and transfer the support of building loads through the elements to deeper, more competent soil. Due to environmental concerns, we understand ground improvement element lengths may need to be limited to a maximum of 20 feet. For this area, we preliminarily estimate 20-foot elements would extend through a majority of the liquefiable zones at the site. There could be liquefiable soil below this depth with associated settlements of up to about 1 inch.

Based on our experience with sites with similar soil conditions, we anticipate static settlement of properly constructed footings supported on DDC-improved soil should be limited to 1 inch under the weight of the building loads, with about ½-inch of differential settlement between adjacent columns. The design-build ground improvement contractor should design the ground improvement to perform as specified by the structural engineer.

7.5 Tiedown Anchors

Tiedown anchors may be used where needed to resist hydrostatic and/or seismic uplift pressure. Tiedown anchors typically consist of relatively small-diameter, drilled, concrete or grout-filled shafts with steel bars or tendons embedded in the concrete or grout. The anchors develop their uplift resistance from friction between the sides of the shaft and the surrounding soil.

The center-to-center spacing of tiedown anchors should be at least four shaft diameters apart or 4 feet, whichever is greater. The ultimate bond strength between the anchor and soil will depend on the installation procedure. The actual bond strength should be estimated by the designer. For planning purposes, however, we preliminarily recommend using an ultimate skin friction of 1,500 psf for post-grouted tiedowns. Higher values may be obtained depending upon the techniques employed by the contractor and the results of pullout tests. A safety factor of 1.5 and 2.0 should be used for temporary loads (e.g. seismic) and permanent loads (e.g. hydrostatic), respectively.

Special attention should be given to waterproofing the connections between the tiedown anchors and the foundation. Because tiedowns will be permanent, we recommend they be double corrosion protected. The tiedowns will be installed below the water table; therefore, the contractor should use an installation method that prevents the holes from caving. If water is present in the shaft, concrete should be placed using a tremie system. High strength bars or

strands may be used as tensile reinforcement in the anchors. A minimum stressing length (free length) of 10 and 15 feet should be provided for bar and strand tendons, respectively.

The first two production tiedowns and two percent of the remaining tiedowns should be performance-tested to 1.5 or 2.0 times the design load, depending on whether temporary or permanent uplift governs. All other tiedowns should be proof-tested to either 1.5 or 2.0 times the design load. If indicated by the structural engineer, after testing, anchors should be loaded and locked off to a portion of their design load.

7.6 Temporary Shoring

During the excavation for the basement, the adjacent properties and improvements should be supported by temporary shoring. There are several key considerations in selecting a suitable shoring system. Those we consider to be primary concerns are:

- protection of surrounding improvements, including roadways, utilities, and adjacent structures,
- control of groundwater inflow and contaminated soil and groundwater
- penetration of shoring system into the dense/stiff soils below the bottom of the excavation,
- proper construction of the shoring system to reduce the potential for ground movement, and
- cost.

The considered basement level (16 feet basement height plus a four-feet-thick mat foundation) is anticipated to extend 20 feet below existing grades. During excavation for the proposed basement levels, shoring will be required to laterally restrain the sides of the excavation and limit the movement of adjacent improvements, such as public streets and sidewalks.

Soldier pile, and timber lagging, could be used to support the excavation; however, because of the presence of contaminated ground water, it may be more economical to use an impermeable cutoff wall such as a soil-cement-mixed wall to limit the amount of dewatering. For a soldier beam-and-lagging system, steel soldier piles would be placed in predrilled holes and backfilled with lean and/or full-strength concrete prior to site excavation. Wood lagging would be placed between the soldier beams as the excavation proceeds.

Soil-cement-mixed walls can be installed by advancing hollow-stem augers and pumping cement slurry through the tips of the augers during auger penetration. In one type of soil-cement-mixed walls, the walls are constructed by excavating grooves with a moving chain-saw cutter. The soil is mixed with the cement slurry in situ, forming continuous overlapping soil-cement columns or continuous walls. Steel beams are placed in the soil-cement columns or walls at pre-determined spacing to provide rigidity. Soil-cement walls are considered temporary; permanent walls are usually built in front of the walls. Both shoring system will likely require either grouted tiebacks

or internal bracing, depending on whether encroachment permits can be obtained to drill beneath the adjacent city streets and adjacent properties, if needed.

The selection, design, construction, and performance of the shoring system should be the responsibility of the contractor. The shoring should be designed by an engineer knowledgeable in this type of construction.

7.7 Groundwater and Dewatering Considerations

The preliminary design groundwater level provided does not account for the impact of sea level rise (SLR) at the site; the project civil engineer and design team should determine the level of SLR that the project design will consider over the life of the building. We can provide recommendations for the effects of this considered SLR on the design groundwater level, if requested. The groundwater level should be further evaluated in a final geotechnical investigation.

To construct the basement of the building, the groundwater level within the limits of the proposed basement wall should be maintained at least three feet below the planned maximum excavations until sufficient weight and/or uplift capacity is available to resist the hydrostatic uplift forces on the bottom of the structure. The project structural engineer should evaluate when there is sufficient load from the structure such that dewatering can be stopped.

Variables that significantly influence the performance of the dewatering system and the quantity of water produced include the number, depth, and positioning of the wells, the interval over which each well is screened, and the rate at which each well is pumped and the type of shoring system used. Different combinations of these variables can be used to dewater the site. The site dewatering should be designed and implemented by an experienced dewatering contractor. However, we should check the dewatering system proposed by the contractor prior to installation.

7.8 Excavation and Temporary and Permanent Slopes

The soil to be excavated from the site consists of materials that can be excavated with conventional earthmoving equipment such as loaders and backhoes. Footings, grade beams, and underground utilities for the existing building to be demolished may be encountered. During excavation, the shoring system is expected to yield and deform, which could cause surrounding improvements to settle slightly. Considering the size and depth of the excavation and the presence of adjacent facilities, we judge a monitoring program should be established to evaluate the effects of the construction on these facilities.

Excavations deeper than five feet that will be entered by workers should be shored or sloped for safety in accordance with the OSHA standards (29 CFR Part 1926). Inclinations of temporary slopes should not exceed those specified in local, state or federal safety regulations. As a minimum, the requirements of the current OSHA Health and Safety Standards for Excavations (29 CFR Part 1926) should be followed. The contractor should determine temporary slope

inclinations based on the subsurface conditions exposed at the time of construction. We recommend temporary slopes be no steeper than 1.5:1 (horizontal to vertical).

Temporary slopes should not be left open for an extended period of time. If temporary slopes are left open for extended periods of time, exposure to weather and rain could result in sloughing and erosion. Vehicles and other surcharge loads should be kept at least 10 feet from the tops of temporary slopes. Slopes should be protected from excessive drying or saturation during construction. If, during construction, different subsurface conditions are encountered, we recommend that we be contacted immediately to evaluate these conditions.

Where space does not permit a sloped excavation, the temporary excavation faces should be retained using soldier-pile-and-lagging wall with either tieback or internal bracing, as recommended in Section 7.6.

We preliminarily recommend permanent fill slopes may not be steeper than 2:1.

7.9 Basement Walls

Basement walls should be designed to resist the lateral pressures imposed by the adjacent soil and vehicles. If foundations of nearby buildings are present above an imaginary 30-degree plane drawn up from the bottom of the excavation, an additional surcharge to account for the weight of the buildings will need to be added to the wall pressures. In addition, because of the presence of potentially liquefiable soil, basement walls should be designed to resist the additional pressure due to liquefaction during an earthquake. Walls should be drained above groundwater table or designed for full hydrostatic pressure. Walls should be waterproofed and water stops should be placed at all construction joints.

7.10 Seismic Design

Potentially liquefiable material was encountered during previous investigations at the site; however, because of the relatively thin and discontinuous nature of the layers that could potentially generate excess pore pressure or liquefy, the seismic ground response at the site should not be significantly affected. Accordingly, we conclude the site should behave similar to a stiff soil site. Based on the subsurface conditions at the site, we preliminarily conclude the site should be classified as Site Class D, in accordance with the 2022 CBC and ASCE 7-16.

For seismic design in accordance with the provisions of 2022 CBC/ASCE 7-16, we recommend the following:

- Risk-Targeted Maximum Considered Earthquake (MCE_R) S_s and S_1 of 1.62g and 0.65g, respectively.
- Site Class D
- Site Coefficients F_a and F_v of 1.0 and 1.7, respectively, assuming the exceptions of Section 11.4.8 are met

- MCER spectral response acceleration parameters at short periods, S_{MS} , and at one-second period, S_{M1} , of 1.71g and 1.84g, respectively
- Design Earthquake (DE) spectral response acceleration parameters at short period, S_{DS} , and at one-second period, S_{D1} , of 1.14g and 0.65g, respectively.
- PGAM of 0.59g.

If the exceptions of Section 11.4.8 in ASCE 7-16 are not met, then site specific spectra will need to be developed, because the site is classified as Site Class D and S_1 is greater than 0.2g.

We caution that if significant liquefaction potential is determined based on a detailed geotechnical investigation, the site could be classified as Site Class F. If desired, ground improvement could be performed to mitigate a liquefaction hazard to change the site class from F to D; however, the ground improvement will be more costly because of the need for closer spacing or other ground improvement methods.

7.11 Construction Considerations

The near-surface soil is expected to consist of expansive clay. The shear strength and overall stability of the clayey soil is related to the moisture content of the soil; this type of soil loses strength when wet. Therefore, site preparation and grading may be difficult if performed during the rainy season. The grading contractor should be prepared to repair weak and wet subgrade, if required.

A monitoring program should be established to evaluate the effects of the construction on the nearby buildings and improvements. The contractor should install surveying points to monitor the movement and settlement of nearby structures and the ground surface during foundation installation. The monitoring should provide timely data, which can be used to modify construction activities, if needed.

The soil subgrade should be prepared and stabilized as needed to provide an adequate working platform for the anticipated construction equipment at grade and in the excavation. The contractor should evaluate the need for a working pad.

8.0 FUTURE STUDIES AND LIMITATIONS

The conclusions and recommendations presented in this letter are preliminary and should not be used to develop construction drawings. A detailed geotechnical investigation should be performed to address any new construction. The investigation should include field exploration and preparation of a design-level geotechnical report to address site geotechnical concerns. The design level investigation should establish the depth of groundwater, evaluate the presence near-surface expansive soil, potentially compressible and liquefiable subsurface materials, and develop final recommendations for foundations and other geotechnical aspects of the project. Additional data on the foundation systems for adjacent structures should be compiled and evaluated.

Sincerely yours,

Langan Engineering and Environmental Services, Inc.



Peter Brady, GE
Senior Project Engineer



Cary Ronan, GE
Associate Principal/Vice President



John Gouchon, GE
Principal/Vice President



Attachments: References

Figure 1 – Site Location Map

Figure 2 – Site Plan

Figure 3 – Map of Major Faults and Earthquake Epicenters in the San Francisco Bay Area

Figure 4 – Modified Mercalli Intensity Scale

Figure 5 – Regional Seismic Hazard Zones Map

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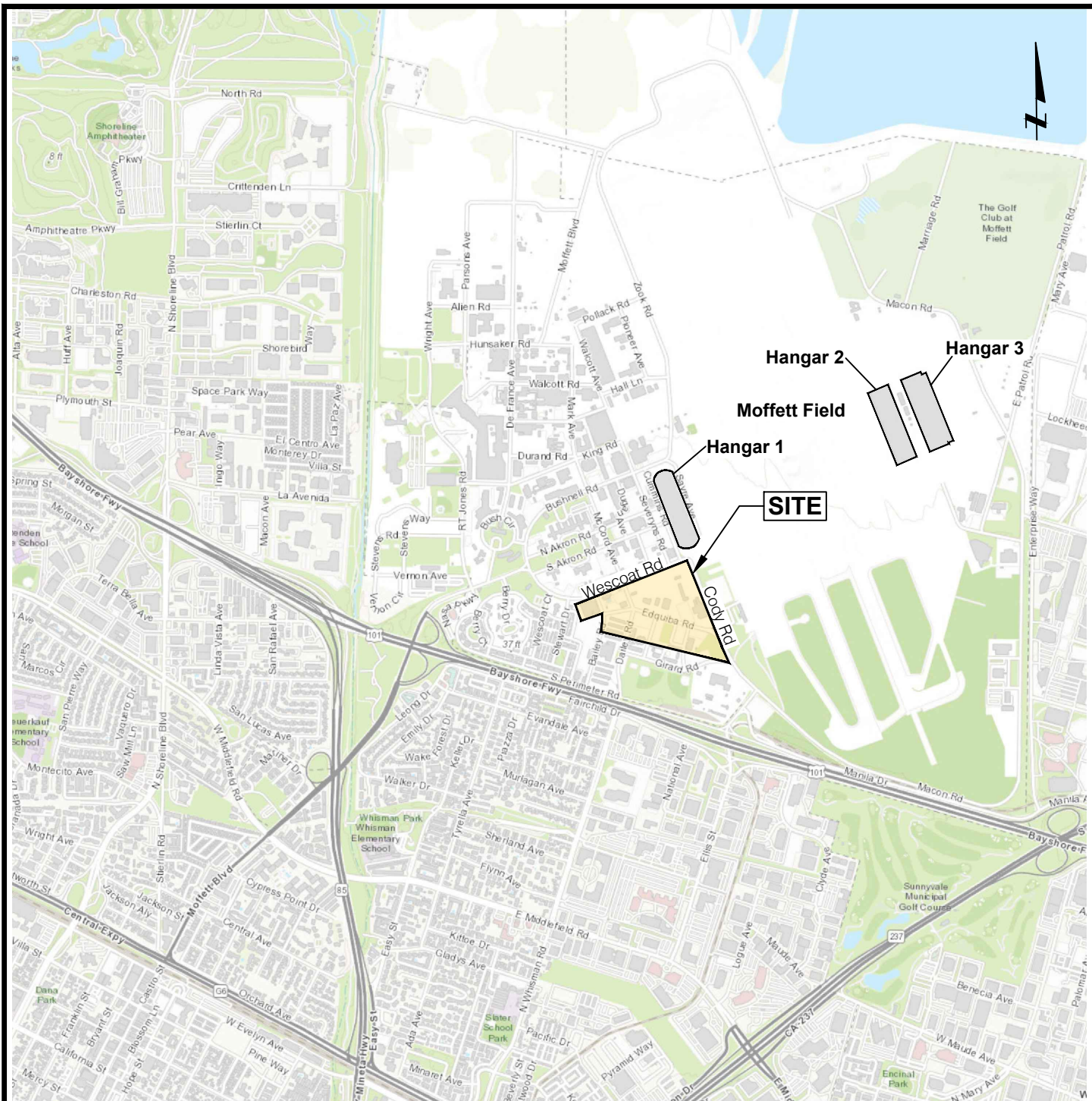
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FIGURES



Notes:

1. Topographic base map is provided through Langan's Esri Arc GIS software licensing and Arc GIS online, National Geographic Society, i-cubed.

0 2,000 Feet

LANGAN

Langan Engineering and
Environmental Services, Inc.
135 Main Street, Suite 1500
San Francisco, CA 94105

T: 415.955.5200 F: 415.955.5201 www.langan.com

Project

**MOFFETT FIELD
NASA RESEARCH PARK**

MOUNTAIN VIEW

SANTA CLARA COUNTY CALIFORNIA

Figure Title

**SITE LOCATION
MAP**

Project No.

731762801

Date

03/06/2023

Drawn By

AG

Checked By

CA

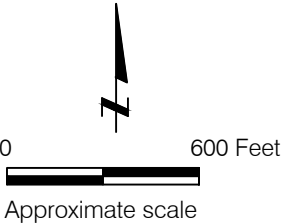
Figure

1

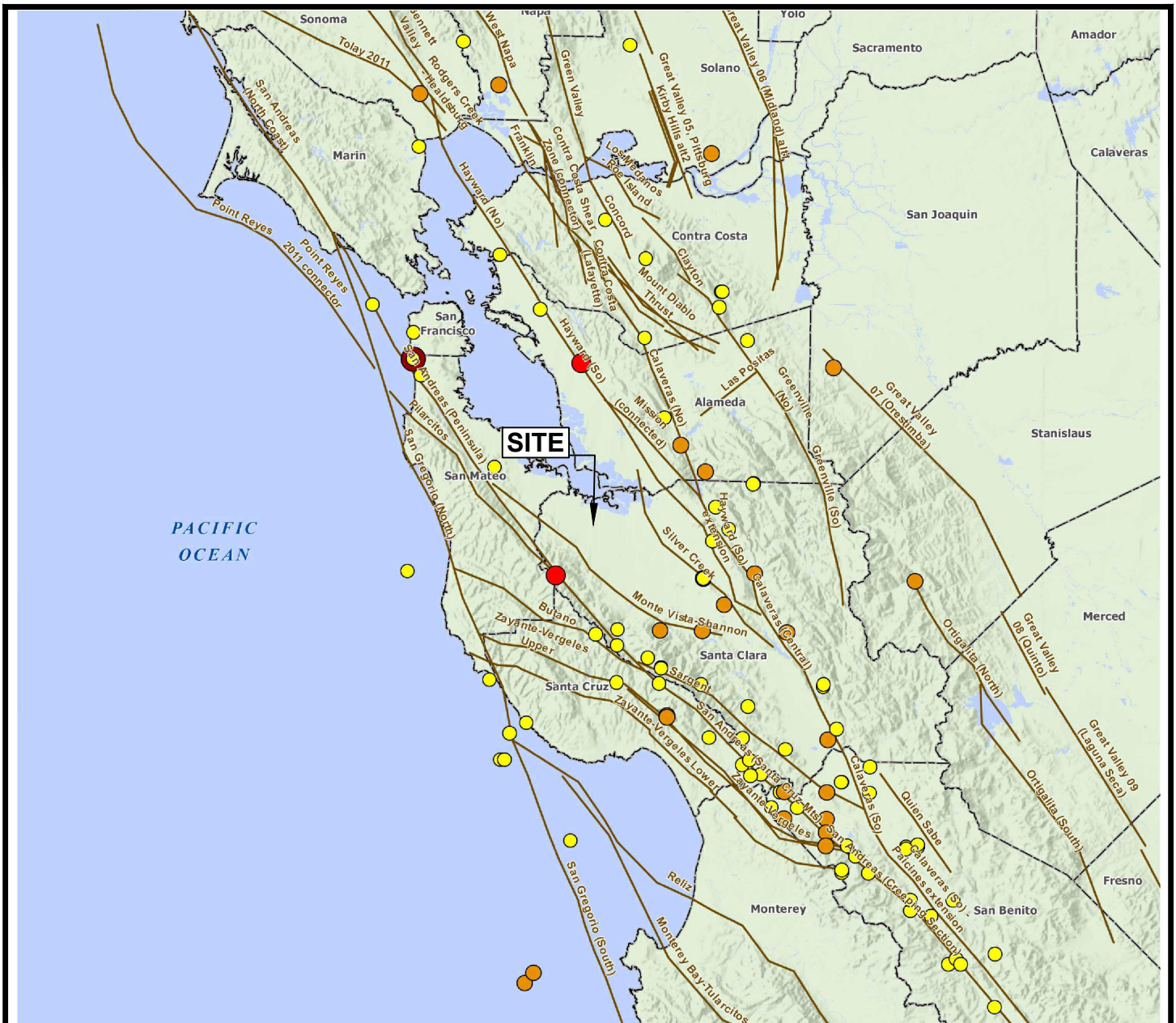


EXPLANATION
- - - Site boundary

Note:
1. Aerial by Google Earth Pro 2023.



LANGAN Langan Engineering and Environmental Services, Inc. 135 Main Street, Suite 1500 San Francisco, CA 94105 T: 415.955.5200 F: 415.955.5201 www.langan.com	Project MOFFETT FIELD NASA RESEARCH PARK MOUNTAIN VIEW SANTA CLARA COUNTY CALIFORNIA	Figure Title SITE PLAN		Project No. 731762801	2
				Date 03/06/2023	
				Drawn By AG	
				Checked By CA	



LEGEND



County Boundary



Fault

Earthquake Epicenter Magnitude

● Magnitude 5 to 5.9

● Magnitude 6 to 6.9

● Magnitude 7 to 7.4

● Magnitude 7.5 to 8

0 20 Miles



Approximate scale

Notes:

1. Quaternary fault data displayed are provided by the CGS Map Sheet 48: Fault based seismic sources used in the Uniform California Earthquake Rupture Forecast, Version (UCERF3).
2. The Earthquake Epicenter (Magnitude) data is provided by the U.S Geological Survey (USGS) and is current through 2015.
3. Basemap hillshade and County boundaries provided by USGS and California Department of Transportation.
4. Map displayed in California State Coordinate System, California (Teale) Albers, North American Datum of 1983 (NAD83), Meters.

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Langan Engineering and
Environmental Services, Inc.
135 Main Street, Suite 1500
San Francisco, CA 94105

T: 415.955.5200 F: 415.955.5201 www.langan.com

Project

**MOFFETT FIELD
NASA RESEARCH PARK**

MOUNTAIN VIEW

SANTA CLARA COUNTY CALIFORNIA

Figure Title

**MAP OF MAJOR
FAULTS AND EARTHQUAKE
EPICENTERS IN THE
SAN FRANCISCO BAY AREA**

Project No.

731762801

Date

03/06/2023

Drawn By

AG

Checked By

CA

Figure

3

I Not felt by people, except under especially favorable circumstances. However, dizziness or nausea may be experienced.

Sometimes birds and animals are uneasy or disturbed. Trees, structures, liquids, bodies of water may sway gently, and doors may swing very slowly.

II Felt indoors by a few people, especially on upper floors of multi-story buildings, and by sensitive or nervous persons.

As in Grade I, birds and animals are disturbed, and trees, structures, liquids and bodies of water may sway. Hanging objects swing, especially if they are delicately suspended.

III Felt indoors by several people, usually as a rapid vibration that may not be recognized as an earthquake at first. Vibration is similar to that of a light, or lightly loaded trucks, or heavy trucks some distance away. Duration may be estimated in some cases.

Movements may be appreciable on upper levels of tall structures. Standing motor cars may rock slightly.

IV Felt indoors by many, outdoors by a few. Awakens a few individuals, particularly light sleepers, but frightens no one except those apprehensive from previous experience. Vibration like that due to passing of heavy, or heavily loaded trucks. Sensation like a heavy body striking building, or the falling of heavy objects inside.

Dishes, windows and doors rattle; glassware and crockery clink and clash. Walls and house frames creak, especially if intensity is in the upper range of this grade. Hanging objects often swing. Liquids in open vessels are disturbed slightly. Stationary automobiles rock noticeably.

V Felt indoors by practically everyone, outdoors by most people. Direction can often be estimated by those outdoors. Awakens many, or most sleepers. Frightens a few people, with slight excitement; some persons run outdoors.

Buildings tremble throughout. Dishes and glassware break to some extent. Windows crack in some cases, but not generally. Vases and small or unstable objects overturn in many instances, and a few fall. Hanging objects and doors swing generally or considerably. Pictures knock against walls, or swing out of place. Doors and shutters open or close abruptly. Pendulum clocks stop, or run fast or slow. Small objects move, and furnishings may shift to a slight extent. Small amounts of liquids spill from well-filled open containers. Trees and bushes shake slightly.

VI Felt by everyone, indoors and outdoors. Awakens all sleepers. Frightens many people; general excitement, and some persons run outdoors.

Persons move unsteadily. Trees and bushes shake slightly to moderately. Liquids are set in strong motion. Small bells in churches and schools ring. Poorly built buildings may be damaged. Plaster falls in small amounts. Other plaster cracks somewhat. Many dishes and glasses, and a few windows break. Knickknacks, books and pictures fall. Furniture overturns in many instances. Heavy furnishings move.

VII Frightens everyone. General alarm, and everyone runs outdoors.

People find it difficult to stand. Persons driving cars notice shaking. Trees and bushes shake moderately to strongly. Waves form on ponds, lakes and streams. Water is muddied. Gravel or sand stream banks cave in. Large church bells ring. Suspended objects quiver. Damage is negligible in buildings of good design and construction; slight to moderate in well-built ordinary buildings; considerable in poorly built or badly designed buildings, adobe houses, old walls (especially where laid up without mortar), spires, etc. Plaster and some stucco fall. Many windows and some furniture break. Loosened brickwork and tiles shake down. Weak chimneys break at the roofline. Cornices fall from towers and high buildings. Bricks and stones are dislodged. Heavy furniture overturns. Concrete irrigation ditches are considerably damaged.

VIII General fright, and alarm approaches panic.

Persons driving cars are disturbed. Trees shake strongly, and branches and trunks break off (especially palm trees). Sand and mud erupts in small amounts. Flow of springs and wells is temporarily and sometimes permanently changed. Dry wells renew flow. Temperatures of spring and well waters varies. Damage slight in brick structures built especially to withstand earthquakes; considerable in ordinary substantial buildings, with some partial collapse; heavy in some wooden houses, with some tumbling down. Panel walls break away in frame structures. Decayed pilings break off. Walls fall. Solid stone walls crack and break seriously. Wet grounds and steep slopes crack to some extent. Chimneys, columns, monuments and factory stacks and towers twist and fall. Very heavy furniture moves conspicuously or overturns.

IX Panic is general.

Ground cracks conspicuously. Damage is considerable in masonry structures built especially to withstand earthquakes; great in other masonry buildings - some collapse in large part. Some wood frame houses built especially to withstand earthquakes are thrown out of plumb, others are shifted wholly off foundations. Reservoirs are seriously damaged and underground pipes sometimes break.

X Panic is general.

Ground, especially when loose and wet, cracks up to widths of several inches; fissures up to a yard in width run parallel to canal and stream banks. Landsliding is considerable from river banks and steep coasts. Sand and mud shifts horizontally on beaches and flat land. Water level changes in wells. Water is thrown on banks of canals, lakes, rivers, etc. Dams, dikes, embankments are seriously damaged. Well-built wooden structures and bridges are severely damaged, and some collapse. Dangerous cracks develop in excellent brick walls. Most masonry and frame structures, and their foundations are destroyed. Railroad rails bend slightly. Pipe lines buried in earth tear apart or are crushed endwise. Open cracks and broad wavy folds open in cement pavements and asphalt road surfaces.

XI Panic is general.

Disturbances in ground are many and widespread, varying with the ground material. Broad fissures, earth slumps, and land slips develop in soft, wet ground. Water charged with sand and mud is ejected in large amounts. Sea waves of significant magnitude may develop. Damage is severe to wood frame structures, especially near shock centers, great to dams, dikes and embankments, even at long distances. Few if any masonry structures remain standing. Supporting piers or pillars of large, well-built bridges are wrecked. Wooden bridges that "give" are less affected. Railroad rails bend greatly and some thrust endwise. Pipe lines buried in earth are put completely out of service.

XII Panic is general.

Damage is total, and practically all works of construction are damaged greatly or destroyed. Disturbances in the ground are great and varied, and numerous shearing cracks develop. Landslides, rock falls, and slumps in river banks are numerous and extensive. Large rock masses are wrenched loose and torn off. Fault slips develop in firm rock, and horizontal and vertical offset displacements are notable. Water channels, both surface and underground, are disturbed and modified greatly. Lakes are dammed, new waterfalls are produced, rivers are deflected, etc. Surface waves are seen on ground surfaces. Lines of sight and level are distorted. Objects are thrown upward into the air.

LANGAN

Langan Engineering and
Environmental Services, Inc.
135 Main Street, Suite 1500
San Francisco, CA 94105

T: 415.955.5200 F: 415.955.5201 www.langan.com

Project

**MOFFETT FIELD
NASA RESEARCH PARK**

MOUNTAIN VIEW

SANTA CLARA COUNTY CALIFORNIA

Figure Title

**MODIFIED
MERCALLI
INTENSITY SCALE**

Project No.

731762801

Date

03/06/2023

Drawn By

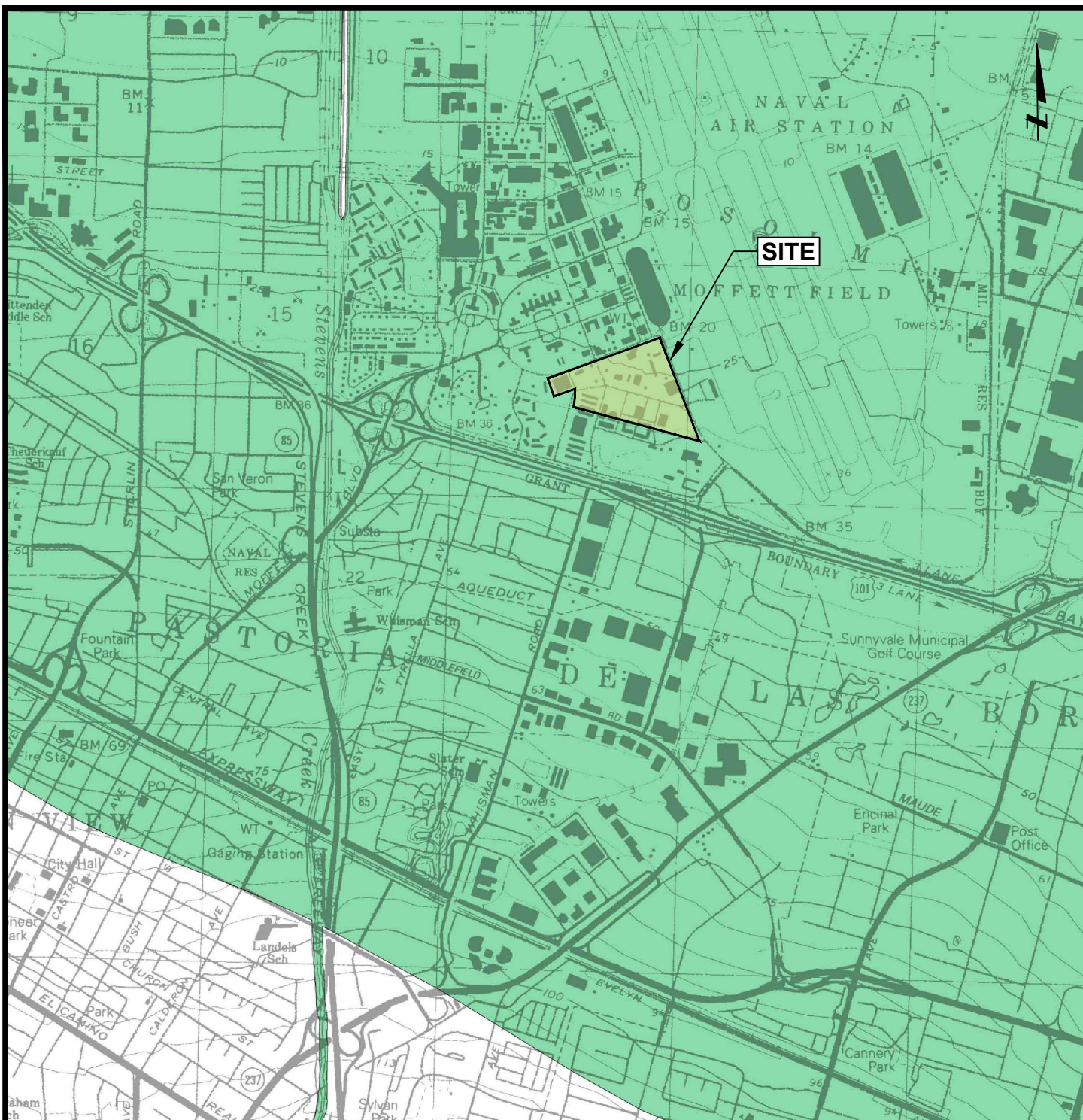
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Checked By

CA

Figure

4



Approximate site boundary

Liquefaction; Areas where historic occurrence of liquefaction, or local topographic, geological,

Notes:

1. Topographic base map is from "State of California, Seismic Hazard Zones Mountain View Quadrangle", dated October 18, 2006

0 2,000 Feet



Approximate scale

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Project

**MOFFETT FIELD
NASA RESEARCH PARK**

MOUNTAIN VIEW

SANTA CLARA COUNTY CALIFORNIA

Figure Title

**REGIONAL SEISMIC
HAZARDS MAP**

Project No.

731762801

Date

03/06/2023

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Figure

5