ARAGÓN GEOTECHNICAL, INC.
Consultants in the Earth & Material Sciences

**PRELIMINARY GEOTECHNICAL INVESTIGATION
BUILDING 2, "FREEWAY 215 & NATWAR LANE" PROJECT
APN 294-180-032
CITY OF PERRIS, RIVERSIDE COUNTY, CALIFORNIA**

**FOR
FIRST INDUSTRIAL REALTY TRUST, INC.
898 N. PACIFIC COAST HWY., SUITE 175
EL SEGUNDO, CALIFORNIA 90245**

**PROJECT NO. 4673-SFLI
DECEMBER 21, 2020**

16801 Van Buren Blvd., Bldg. B
Riverside, CA 92504
Tel: 951.776.0345 ● **Fax:** 951.776.0395
www.aragongeo.com

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December 21, 2020
Project No. 4673-SFLI

First Industrial Realty Trust, Inc.
898 N. Pacific Coast Highway, Suite 175
El Segundo, California 90245

Attention: Mr. Matt Pioli

Subject: Preliminary Geotechnical Investigation Report
Proposed Building 2, "Freeway 215 & Natwar Lane" Project
APN 294-180-032
City of Perris, Riverside County, California.

Mr. Pioli:

In accordance with our proposal dated October 27, 2020 and your authorization, Aragón Geotechnical Inc. (AGI) has completed preliminary geotechnical and geological assessments for the above-referenced project. The attached report presents in detail the findings, opinions, and recommendations developed as a result of surface inspections, subsurface exploration and field tests, laboratory testing, and quantitative analyses. Our scope included an infiltration feasibility study for stormwater BMPs, but excluded environmental research and materials testing for contaminants in soil, groundwater, or air at the site. Infiltration-related findings are presented in a separate report for the designer's use in formulating a required water quality management plan.

Four exploratory borings were sited within the 4.99-acre parcel to characterize local soil units and potential influences from groundwater. The locality is fundamentally a deep alluvium site. Drilled intervals encountered massive Pleistocene-age alluvial strata comprising silty sand, clayey sand, and fines-poor sand with silt as majority classifications within 26.5 feet of existing grades. Shallow horizons of silty sand alluvium less than 10 feet thick cap much older, partly cemented, and very dense silty sand alluvium. Ground surfaces have been loosened by past agricultural tilling and burrowing fauna, and are classified compressible within roughly 3 feet of existing grades. AGI did not find evidence for pre-existing fill. Perched groundwater was encountered in two borings at depths between 23 and 24 feet.

Geologic constraints to development will require inclusion of structural measures to mitigate the high likelihood of strong earthquake ground motions at the site. However, risks from other natural hazards including liquefaction, surface fault rupture, excessive settlement, gross instability or landsliding, seiching, induced flooding, and tsunami appear to range from extremely low to zero.

Findings indicate the site should be suitable for the proposed concrete panel-wall industrial building. AGI recommends that shallow porous alluvium be removed and replaced as compacted engineered fill for adequate support of the structure and new pavements. An acceptable remedial grading "bottom" below the building outline should occur about 3 feet below existing grade. All site soils should be acceptable for reuse in compacted fills. AGI guidance is recommended to institute selective grading to place non-expansive soils near pad subgrades to the maximum extent feasible.

It is AGI's preliminary conclusion that properly designed and constructed conventional shallow footings should provide adequate building support. Overexcavation is recommended when or if needed to supply at least 24 inches of engineered fill below all shallow spread and continuous footings. Pavement areas should be partly stripped and partly processed-in-place to create recompacted depths of approximately 30 to 36 inches. Paved areas in cuts deeper than two feet should require only soil processing in place.

In addition to foundation design guidelines, including preliminary recommended design values for both vertical and lateral loads, this report presents recommendations for site earthwork, prescriptive code values for use in seismic groundshaking mitigation, concrete mix designs, and construction observation. It is recommended that grading and foundation plan reviews be performed by AGI prior to construction.

Thank you very much for this opportunity to be of service. Please do not hesitate to call our Riverside office if you should have any questions.

Very truly yours,
Aragón Geotechnical Inc.

Mark Doerschlag

Mark G. Doerschlag, CEG 1752
Engineering Geologist

C. Fernando Aragón

C. Fernando Aragón, P.E., M.S.
Geotechnical Engineer, G.E. No. 2994

MGD/CFA:mma

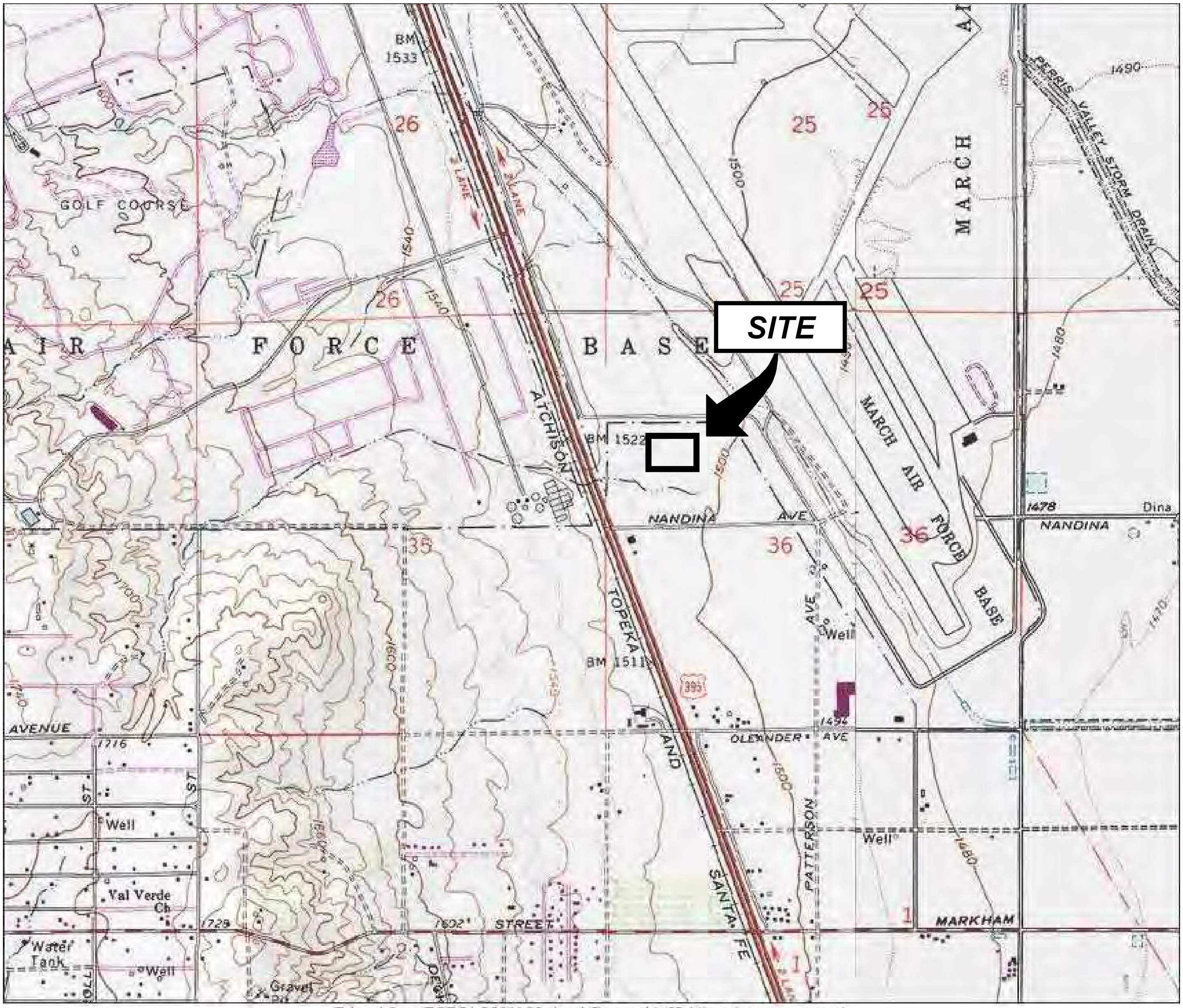
Distribution: (4) Addressee

1.0 INTRODUCTION

This report presents the results of preliminary soils engineering and geologic evaluations conducted by Aragón Geotechnical, Inc. (AGI) for the noted project, located north of the intersection of Natwar Lane at Nandina Avenue, Perris, California. The rectangular project site (APN 294-180-032) encompasses 4.99 acres. Map coordinates are approximately 33.87006°N x 117.25741°W at the northeastern corner of the project's main occupancy improvement, a proposed 133,060-square-foot logistics or light manufacturing building. These coordinates were selected for seismological analyses. Situs per the Public Lands Survey System places the project in the NW¼ of Section 36, Township 4 South, Range 3 West (San Bernardino Baseline and Meridian). The accompanying Site Location Map (Figure 1, next page) depicts the general location of the project on a 1:24,000-scale topographic quadrangle map. Although out-of-date with respect to the rapid urbanization of the surrounding Perris Valley area, the older map series was selected for clearer depictions of ground slope and drainage patterns.

The proposed warehouse building would be a smaller companion to a neighboring 23.2-acre project, Building 1, investigated by AGI in mid-June, 2019. We have made liberal use of data and interpretations from the previous study (AGI, 2019), although this report is intended to serve for entitlement purposes and as part of a stand-alone specification for Building 2. Similar construction is envisioned. Plans suggest that heavy truck ingress and egress will be possible from both Natwar Lane, an industrial cul-de-sac, and from Western Way, a new public street along the eastern side of the land parcel.

The primary objectives of our preliminary investigation were to determine the nature and engineering properties of the subsurface materials underlying the project area, in order to confirm general site suitability for the building and to provide *preliminary* foundation design, grading, and surfacing recommendations. Accordingly, our scope included site reconnaissance, aerial photo interpretation, geologic literature research, subsurface exploration, recovery of representative soil samples, laboratory soils testing, and geotechnical analyses. Authorized services included field tests to characterize water infiltration potential for a required BMP installation (e.g., basin or chamber array). An infiltration feasibility report has been issued by AGI under separate cover for the design civil engineer's use in formulating a required water quality management plan.



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Reference: U. S. Geological Survey 7½-Minute Series Topographic Map, Steele Peak Quadrangle (1978).

SITE LOCATION MAP		
BUILDING 2, FWY 215 & NATWAR LANE PROJECT, CITY OF PERRIS, CA.		
PROJECT NO. 4673-SFLI	DATE: 12/21/20	**FIGURE 1**

Geological assessments focused on risks posed by active earthquake faults, strong ground motion, liquefaction or other secondary seismic hazards, and groundwater. These were evaluated using published resources and site-specific data, plus conclusions drawn from previously reported quantitative analyses completed by AGI for Building 1. However, environmental research, Phase I or Phase II environmental site assessments, monitoring well construction, or contaminant testing of air or groundwater found in the site were beyond the scope of this geotechnical investigation.

2.0 PROPOSED CONSTRUCTION

A conceptual site development plan originating from the Irvine firm of HPA Architecture was referenced for property information and proposed construction. The conceptual plan lacked labeled finish grades or depictions of certain standard project features such as site walls. Lot line adjustments would place certain parts of the former Building 1 project within the Building 2 zone, and would remove an outlier or remnant bit of land east of Western Way as surplus. One major change noted from the previous Building 1 study was the potential extension of Van Buren Boulevard from a dead-end terminus near the March Field Air Museum to a connection with Western Way. The road construction would be entirely within March Air Reserve Base property, and was not part of the Building 1 or Building 2 investigation scopes.

Consistent with regional practice, AGI anticipated that Building 2 would feature concrete panel tilt-up walls with parapet heights of possibly 45 to 55 feet, resting on shallow continuous footings. Roof loads would be supported by arrays of interior columns. A concrete slab-on-grade industrial floor is expected. Fifteen dock doors would be included in the structure. Office areas with mezzanine levels would occupy the southwestern and southeastern building corners. Moderate foundation loads would be predicted. Basements or other subterranean construction were not shown on the drawing.

Surrounding the building, concrete paving is expected in truck areas while lighter-duty asphalt sections could be substituted in automobile driveways and stalls. We think concrete site walls will define the project perimeter. Live sewer, water, and gas utilities exist next to the property, and would presumably connect with the new industrial building via buried service laterals.

Future grading would be a cut-and-fill operation. Assuming that a level floor is designed for Building 2, we speculate that cuts and fills should still remain under 5 feet, referenced to existing grades. Grading quantities can be expected to increase after accounting for ground preparation measures for the building pad and for paved areas. Manufactured slopes or retaining walls are not expected on the grossly flat site.

3.0 FIELD INVESTIGATION AND LABORATORY TESTING

Subsurface geotechnical site characterization comprising 4 exploratory soil borings was completed by AGI on December 1, 2020. The site did not have access impediments posed by very soft soils, vegetation, or existing structures. The Building 2 investigation was programmed on the basis of an older development plan by a different engineering firm that did not include a warehouse building. Nonetheless, the borehole count and localities were judged sufficient for the more-intensive proposed use. The previous plan included elevation contours we referenced for this study.

AGI-selected drill sites were cleared of utility interference issues by notification to the 811 DigAlert service in advance of AGI's work. Although the soil boring sites were preferentially sited based on older plans, they still met customary goals of targeting the proposed structural envelope, truck-and-trailer drive aisles, and the site's predicted location for a stormwater management system. Borings were also spaced to meet a goal of spanning the parcel area to gauge the degree of geotechnical site variability. Drilled locations were deemed adequate to obtain data concerning (1) Soil material classifications, water contents, in-place densities, and settlement potential in light of local geological interpretations; (2) Presence or absence of groundwater; (3) Continuity of layers or units across the property; and (4) Unit geological origins and a derivation of site "stiffness" for earthquake engineering purposes.

The soil borings were drilled with a truck-mounted hollow-stem auger rig capable of driving and retrieving soil sample barrels. Borehole termination depths ranged from 10.5 to 26.5 feet. None of the borings encountered bedrock or were halted by machine refusal. As expected, all borings encountered deep sediments that were amenable to drive-tube sampling, performed at 2-foot to 5-foot depth increments. At shallow depths where soil bearing capacity and settlement potential would be the main items of concern, relatively undisturbed soil samples were recovered by driving a 3.0-inch-diameter "California modified" split-barrel sampler lined with brass rings. Deeper horizons in most borings included Standard Penetration Tests (SPTs) conducted using an unlined 2.0-inch O.D. split-barrel spoon. All sampler driving was done using rods and a mechanically actuated automatic 140-pound hammer free-falling 30 inches. Bulk samples of auger cuttings representative of shallow native materials found at several borings were bagged. All geotechnical samples were brought to AGI's Riverside laboratory for assigned soils testing.

Drill cuttings and each discrete sample were visually/manually examined and classified according to the Unified Soil Classification System, and observations made concerning relative density, constituent grain size, visible macro-porosity, plasticity, and past or present groundwater conditions. Continuous logs of the subsurface conditions encountered were recorded by an AGI civil engineer, and the results are presented on the Field Boring Logs in Appendix A. The approximate locations of the borehole explorations are illustrated on the Geotechnical Map (Plate No. 1 foldout), located at the back of this report.

"Undisturbed" samples were tested for dry density and water content. One-dimensional consolidation tests were conducted on selected barrel samples in order to evaluate settlement or collapse potential. Collapsible soils undergo rapid, irreversible compression when brought close to saturation while also subjected to loads such as from buildings or fill. The recovered bulk soil samples were evaluated for index and engineering properties such as shear strength, compaction criteria, expansion potential, and corrosivity characteristics. Discussions of the laboratory test standards used and the test results are presented in Appendix B.

4.0 SITE GEOTECHNICAL CONDITIONS

4.1 <u>Previous Site Uses</u>

AGI's scope included limited historical research to ascertain changes to surficial conditions through time, and address known or possible geotechnical impacts to project design or construction. Some findings were based on work already done by AGI for the adjacent Building 1 logistics center. Resources used for the previous study included the Riverside County Flood Control and Water Conservation District Stereoscopic aerial photograph archives in Riverside, California. The archive has been closed indefinitely due to the coronavirus pandemic. We re-checked older monoscopic pictures downloaded from the U.C. Santa Barbara Aerial Collections web application for evidence of past land uses and for geological assessments of active faulting potential and geomorphic history. Finally, the U.S. Geological Survey Historical Map Collection website was accessed for digital scans of topographic quadrangle sheets pre-dating the referenced base map used for Figure 1. Reviewed historical sources are listed under “References” at the end of this report.

For decades beginning before 1938 and up until at least the mid-1970's, the site was used for dry-farmed grain crops and irrigated alfalfa. Buildings have never existed within the project limits. There were no confirmed past uses for stock raising, poultry ranching, feedlot, or dairying operations.

By 1984 a property a few hundred feet to the southwest had been developed into a construction equipment yard. A paved stub of future Natwar Lane extended a short distance north of Nandina Avenue. The project site was no longer tilled. Other neighboring businesses started at about the same time. Natwar Lane was not fully completed as a dedicated improved street until mid-2012, when a cardboard recycling facility was completed next to the cul-de-sac street terminus.

Within just the last 15 months, the formerly undeveloped alignment for Western Way and a new industrial building have approached completion along the eastern side of the proposed Building 2 development. No work has yet commenced for Building 1.

4.2 <u>Surface Conditions</u>

Project limits are demarcated by undeveloped parts of the Building 1 collection of land parcels to the west and north [the north strip to be accreted to the Building 2 project], the previously mentioned cardboard recycling business with a bordering tall block wall to the south, and Western Way to the east. The latter street encompasses an easement for a major buried water transmission pipeline (the 96-inch-diameter welded-steel Perris Valley Pipeline). The Metropolitan Water District pipe was installed in 2009. Images showed that although almost the entire street R/W was excavated or disturbed in some fashion, no work seems to have encroached into the subject project site. No grading or dumping of fill soils occurred. Since the 1980s, the parcel has been periodically disced for weed abatement.

The site features a very low-gradient slope of just under one percent toward the east-southeast according to older site plans. Relief within the project area is only about 6½ feet. Disturbed soil surfaces dominate the property. It appears that most incident rainfall is absorbed by the loosened surface horizons. Excess water runoff can move unimpeded as sheetflow eastward and would ultimately be intercepted by Western Way. The site does not have mapped permanent or intermittent stream courses or even noticeable lines of concentrated surface flow.

4.3 Subsurface Conditions

AGI soil borings penetrated vertically heterogeneous native alluvial soil sequences dominated by medium dense silty sand with at least a few percent clay (Unified Soil Classification System classification SM) within 7 to 9 feet of existing grades. This "upper" sequence sometimes included a basal fluvial-sand zone marked by low cohesion and low proportions of silt. Soils within the uppermost 3 feet or so have been "churned" by burrowing fauna in additional to mechanical tilling. A weakly developed clayey "B" horizon could be interpreted near the bottom of the churned layer. Past and present explorations in the area have indicated the surficial unit deepens northward and eastward toward Western Way and the eastern property limits. No signs of man-made fill were detected onsite.

Laboratory tests on near-surface soils collected from the warehouse building site produced an expansion index of 56 (categorically a "medium" expansion potential). Surficial materials were also characterized by a high achievable maximum dry density of 134 pounds per cubic foot based on the modified Proctor standard method. Consolidation tests showed that surficial alluvium has a slight susceptibility to collapse when saturated under load, but generally low overall compressibility. Pinhole voids or vesicles were noted in one tested sample, but were infrequently seen. Vesicular textures are reliable indicators for detecting collapsible soils in the Inland area.

Below 7 to 9 feet, massive to sometimes stratified deposits of generally dense to very dense silty sand, clayey sand, and uncommon fines-poor sand with silt extended to the deepest-completed hole depth of 26.5 feet. The sometimes cemented deeper materials were interpreted to be far older than the surficial sequence. In AGI's 2019 soil borings for the adjacent Building 1 site, the deeper soils could be described as a fining-up sequence within 20 to 30 feet of existing grade, although this characteristic was not readily seen at the Building 2 site. The contact between surficial silty sand and very old soils was usually fairly abrupt and typical of an erosional surface.

Visible macro-porosity was uniformly absent in the deeper soils. Penetration resistance was high for soil sampling tools, with uncorrected SPT N-values ranging from 24 to 78 blows per one-foot increment (mean: 37.5 bpf) for sample depths between 15 and 25 feet. Bedrock was not encountered. Section 5.2 (Local Geologic Conditions) and the drill logs in Appendix A contain considerable additional descriptions and interpretations of soil conditions in the project area.

4.4 Groundwater

Slow groundwater inflows were observed in two exploratory borings. Stable water levels of 23.8 feet and 23.1 feet below grade were measured. The saturated zones appeared to be thin. Shallower and deeper soil samples were not mottled with iron oxide staining, a common proxy for detecting past historical high groundwater. All other (shallower) soil borings remained dry. Site findings mirrored the detection of similar thin perched-water layers in 2019 at the larger Building 1 industrial site.

The project site is within the West San Jacinto groundwater subbasin. According to many years of off-site environmental well hydrographs reviewed through the State GeoTracker website, groundwater within a radius of about a half-mile from the property becomes shallower to the west and north, with minimum measured depths occasionally under 20 feet. Groundwater gradients steepen near the site. The hydrogeologic regime is complex due to the heterogeneity of the alluvial basin fill, substantial erosional relief of buried bedrock surfaces under the northern Perris Valley, and municipal groundwater pumping. There is a well-documented record for rising groundwater levels inside of March ARB next to the site. Rising water levels are attributed to changing land uses in the Perris Plain vicinity, such as the cessation of formerly widespread agricultural pumping and introduction of irrigated suburban tracts, golf courses, wastewater treatment plants, and the Riverside National Cemetery near the project.

Under current and predicted future conditions, *we judge that groundwater should remain at or below the minimum-measured 23-foot depth*. We think that an unlined treated-wastewater effluent basin located west of the Interstate 215 freeway could be the source of water detected in year-2019 and 2020 borings around Natwar Lane. Shallow cemented hardpans probably impede contributions from ordinary seasonal rainfall. Groundwater should not influence building design or construction. Any open excavation or shaft deeper than 23 feet, however, could encounter saturated ground and water inflows. Future fluctuations in shallow water elevations are considered possible due to variations in precipitation, temperature, consumptive uses, or land use changes in Perris which were not present at the time observations were made.

5.0 ENGINEERING GEOLOGIC ANALYSES

5.1 Regional Geologic Setting

All of western Riverside County lies within the Peninsular Ranges Physiographic Province, one of 11 continental provinces recognized in California. The physiographic provinces are topographic-geologic groupings of convenience based primarily on landforms, characteristic lithologies, and late Cenozoic structural and geomorphic history. The Peninsular Ranges encompass southwestern California west of the Imperial-Coachella Valley trough and south of the escarpments of the San Gabriel and San Bernardino Mountains. Most of the province lies outside of California, where it comprises much of the Baja California Peninsula. The province is characterized by youthful, steeply sloped, northwest-trending elongated ranges and intervening valleys.

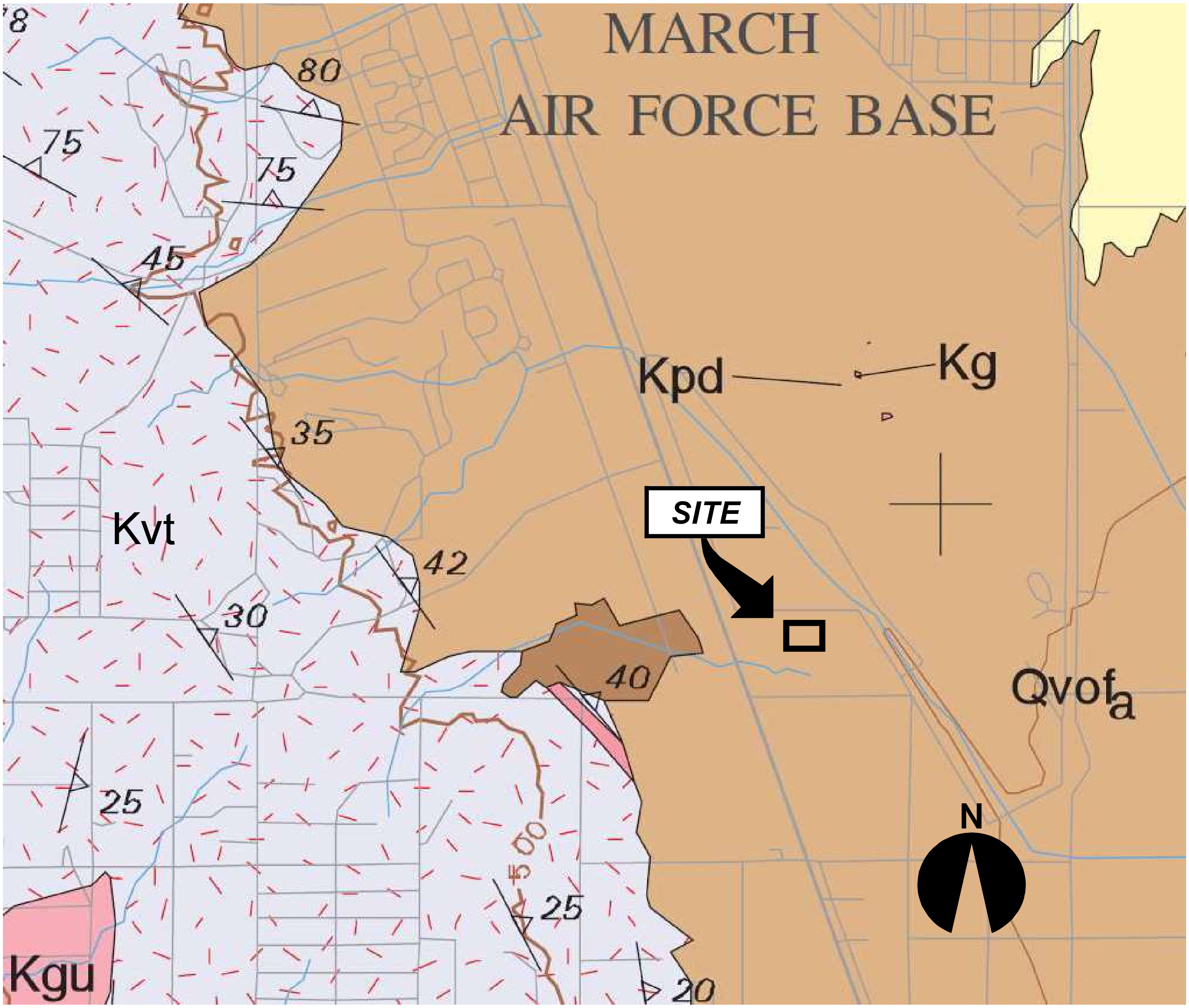
Structurally, the Peninsular Ranges province in California is composed of a number of relatively stable, elongated crustal blocks bounded by active faults of the San Andreas transform system. Although some folding, minor faulting, and random seismic activity can be found within the blocks, intense structural deformation and large earthquakes are mostly limited to the block margins. Exceptions are most notable approaching the Los Angeles Basin, where compressive stress gives rise to increasing degrees of vertical offset along the transform faults and a change in deformation style that includes young folds and active thrust ramps. Perris is located in the central portion of the Perris tectonic block, the longest sides of which are bounded by the San Jacinto fault zone to the northeast and the Elsinore and Chino fault systems to the southwest.

The Peninsular Ranges structural blocks are dominated by the presence of intrusive granitic rock types similar to those in the Sierra Nevada, although the province additionally contains a diverse array of metamorphic, sedimentary, and extrusive volcanic rocks. In general, the metamorphic rocks represent the highly altered host rocks for the episodic emplacement of Mesozoic-age granitic masses of varying composition. Parts of the province include thick sequences of younger marine and non-marine clastic sedimentary rocks of Mesozoic and Tertiary age, ranging from claystones to conglomerate. Pre-Quaternary sedimentary rocks are conspicuously absent from most of the Perris Block, however, which is dominated by crystalline basement materials.

5.2 Local Geologic Conditions

Bounded by sometimes bold mountainous terrain to the east and west, the Perris Plain is entirely underlain by massive to crudely bedded alluvium. Morton and Miller (2006) assign an early to middle Pleistocene age for very old alluvium (unit $Qvof_a$, Figure 2) that composes the majority of the topographical valley floor. Regional maps generally omit thin veneers of younger sediments that are frequently found near the edges of the Perris Plain. AGI interprets surficial silty sand in the Building 2 site to be representative of younger (but probably still pre-Holocene age) alluvium derived from elevated granitic bedrock terrain west of the Interstate 215 freeway. However, we have lumped the thin surficial soils into unit "$Qvof_a$" on site map and drill log exhibits. Most of Moreno Valley and the Perris Plain where the project is located are considered part of the "Paloma" depositional surface of Woodford et al. (1971), typified by strongly developed illuvial clay and calcic horizons atop the older parent materials.

The alluvium buries and conceals several deep erosional channels carved into granitic basement bedrock that can be considered tributaries to an ancestral San Jacinto River. The maximum depth of the $Qvof_a$ unit at the site is not known with certainty, but may be approximately 550 feet based on geophysical survey data (AECOM, 2013). Basement rock rises rapidly toward the Interstate 215 freeway alignment, where it is possibly only 50 to 70 feet deep. Granitic bedrock consisting of weakly foliated quartz diorite (Val Verde tonalite) crops out at the surface only about 4,200 feet west of the project site.



Selected vicinity units:

0 0.5 1.0 mi.

Qvof$_a$ Very old sandy alluvial-fan deposits (middle to early Pleistocene)

Kvt
Kgu
Kpd
Kg

Granitic basement rocks composed of weakly foliated tonalite, leucocratic granite, and minor pegmatite in March ARB (Cretaceous and older)

Reference: Modified after Morton and Miller (2006). Scale is approximate.

AGI	**VICINITY GEOLOGIC MAP**	
	BUILDING 2, FWY 215 & NATWAR LANE PROJECT, CITY OF PERRIS, CA.	
PROJECT NO. 4673-SFLI	DATE: 12/21/20	**FIGURE 2**

5.3 Slope Stability

The low-relief site is free of natural features associated with gross instability of slopes. The property is also distant from mountainous slopes surrounding Perris Valley. We judge landslide risks to be nil.

5.4 Flooding

All project areas are accorded a status of flood zone X, or outside of delineated "100-year" or 1% annual chance flood zones (FEMA, 2008). The site receives essentially zero off-site runoff. Post-development flood and debris flow risks should be extremely low.

5.5 Faulting and Regional Seismicity

The project is situated in region of active and potentially active faults, as is all of metropolitan Southern California. Active faults present several potential risks to structures and people. Hazards associated with active faults include strong earthquake ground shaking, soil densification and liquefaction, mass wasting (landsliding), and surface rupture along active fault traces. Generally, the following four factors are the principal determinants of seismic risk at a given location:

- Distance to seismogenically capable faults.
- The maximum or "characteristic" magnitude earthquake for a capable fault.
- Seismic recurrence interval, in turn related to tectonic slip rates.
- Nature of earth materials underlying the site.

5.5.1 Fault Rupture Potential

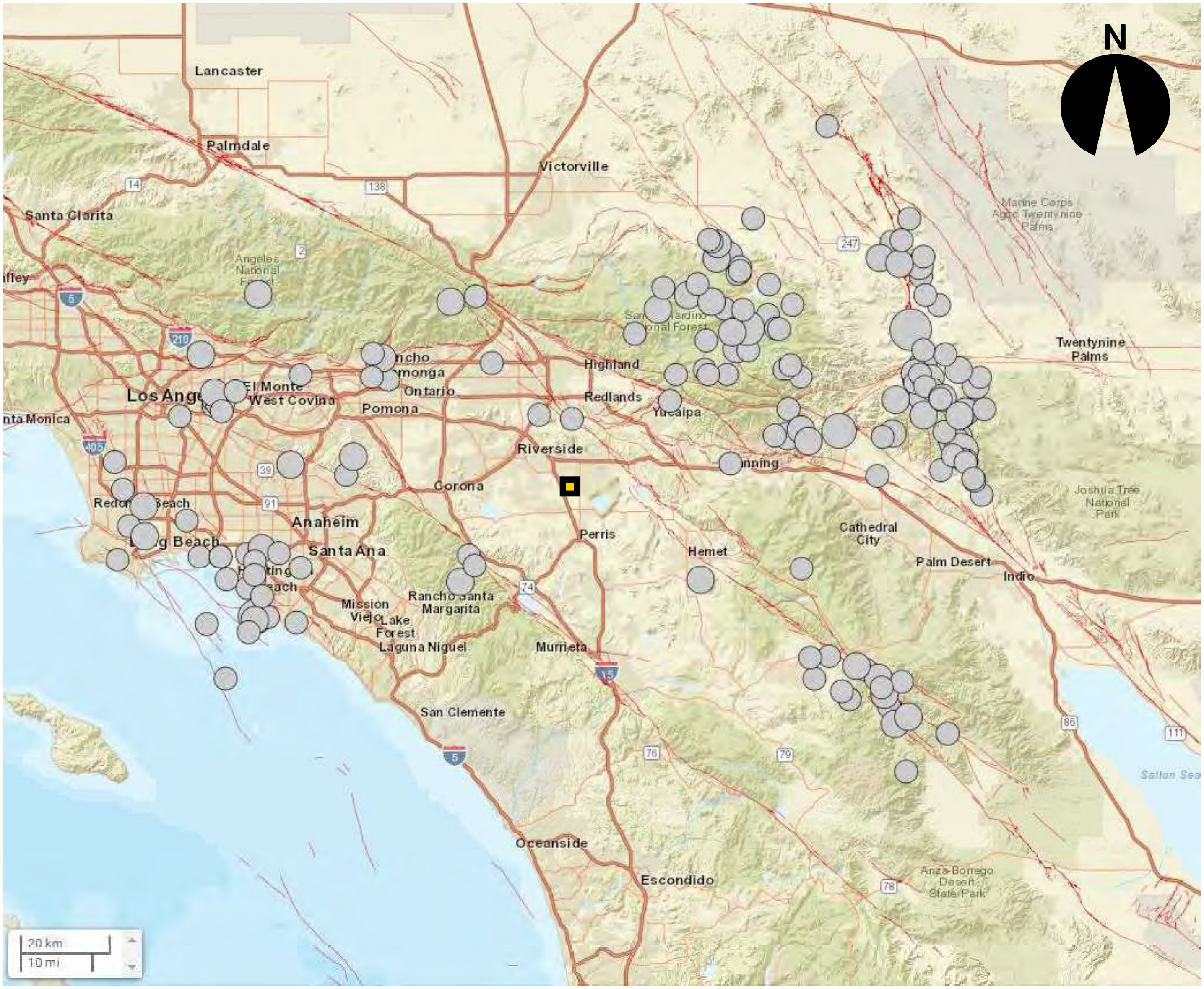
Surface rupture presents a primary or direct potential hazard to structures built across an active fault trace. Reviews of official maps delineating State of California Earthquake Fault Zones and Riverside County Fault Hazard Management zones indicated the project site is not located in a zone of required investigation for active faulting. The closest known active regional fault traces are associated with the San Jacinto Fault east of Moreno Valley, about 8.4 miles away at closest approach. Aerial photographic interpretations did not suggest visible lineaments or manifestations of fault topography related to active fault traces on or adjacent to the site. Accordingly, chances for direct surface fault rupture affecting the project are judged to be extremely low.

5.5.2 Strong Motion Potential

All Southern California construction is considered to be at high risk of experiencing strong ground motion during a structure's design life. In addition to the previously mentioned San Jacinto fault zone, the San Andreas Fault can be considered a potentially significant sources of lower-frequency and longer-duration shaking at the project. Other, more-distant regional faults are very unlikely to cause shaking as intense as that caused by rupture of one of the two listed faults. Probabilistic risk models for the Perris-Moreno Valley area fundamentally assign the highest seismic risks from large characteristic seismic events along the San Jacinto fault system. The mode-magnitude event for peak ground acceleration at a 2% in 50-year exceedance risk is a multi-segment M_w8.1 earthquake on the San Jacinto fault (U.S. Geological Survey, 2020b; dynamic conterminous U.S. 2014 model).

The searchable ANSS Comprehensive Earthquake Catalog indicates about 183 events of local magnitude M4.5 or greater have occurred within 100 kilometers of the project since instrumented recordings started in 1932 (Figure 3, next page). Clusters of epicenters are associated with the 1992 Landers and triggered Big Bear Lake events. These and other notable historical earthquakes in southern California over the last 30 years (e.g., Northridge, Hector Mine) were far away. They produced estimated peak ground accelerations well under 0.20g in the City of Perris area. Interestingly, earthquakes larger than the selected M4.5 intensity threshold have been rare along the northern San Jacinto fault and the San Andreas fault, even though both have among the fastest slip rates and shortest mean recurrence intervals among all California faults.

San Jacinto Fault: The San Jacinto fault constitutes a set of *en-échelon* or right- and left-stepping fault segments stretching from near Cajon Pass to the Imperial Valley region. The primary sense of slip along the zone is right-lateral, although many individual fault segments show evidence of at least several thousand feet of vertical displacement. The San Jacinto fault zone has been very active, producing possibly eight historical earthquakes of local magnitude 6.0 or greater. The communities of Hemet and San Jacinto were heavily damaged in 1918 and again in 1923 from events on the San Jacinto Fault. Pre-instrumental



Reference: U. S. Geological Survey (2020c) real-time earthquake epicenter map. Plotted are 183 epicenters of instrument-recorded events from 1932 to present (12/11/20) of local magnitude M4.5 or greater within a radius of ~62 miles (100 kilometers) of the site. Location accuracy varies. The site is indicated by the gold square. The red lines indicate the approximate surface traces of Quaternary active faults. The selected magnitude corresponds to a threshold intensity value where light damage potential begins. These events are also generally widely felt by persons. Notable Southern California historical events with epicenters just beyond the selected search radius would include the Northridge earthquake [San Fernando Valley], and the Hector Mine event in the Mojave Desert north of Yucca Valley.

AGI

SIGNIFICANT EVENT EPICENTER EXHIBIT		
BUILDING 2, FWY 215 & NATWAR LANE PROJECT, CITY OF PERRIS, CA.		
PROJECT NO. 4673-SFLI	DATE: 12/21/20	**FIGURE 3**

interpreted magnitudes for these events were $M_L6.8$ and $M_L6.3$, respectively. The historical record suggests each discrete segment *usually* reacts to tectonic stress more or less independently from the others, and to have its own characteristic large earthquake with differing maximum magnitude potential and recurrence interval. Researchers and code development authorities now model the fault with potential for multi-segment rupture, however, with consequent increases in calculated risk to structures.

San Andreas Fault: For most of its over-550-mile length, the San Andreas Fault can be clearly defined as a narrow, discrete zone of predominantly right-lateral shear. The southern terminus is close to the eastern shore of the Salton Sea, where it joins a crustal spreading center marked by the Brawley Seismic Zone. To the northwest, a major interruption of the otherwise relatively simple slip model for the San Andreas fault is centered in the San Gorgonio Pass region. Here, structural complexity resulting from a 15-kilometer left step in the fault zone has created (or reactivated) a myriad of separate faults spanning a zone 5 to 7 kilometers wide (Matti, et al., 1985; Sieh and Yule, 1997; 1998). Continuing research is refining speculation that propagation of ruptures from other portions of the San Andreas Fault might not be impeded through the Pass region. New data suggest the San Bernardino and Coachella Valley segments of the fault may experience concurrent rupture roughly once out of every three to four events. Multi-segment cascade rupture is currently considered in all 2008 and later State of California seismic hazard models (Petersen, 2008; Working Group, 2013), and has been adopted as a scenario event for emergency response training such as the annual ShakeOut drill.

Source characteristics for the two regional active fault zones with the highest contributions to site risks are listed in the following table. Fault data have been summarized from WGCEP (2013) as implemented for the latest California fault model. Magnitudes are based on a probabilistic recurrence interval of 2,475 years for each source, binned to nearest 0.05 magnitude decrement. The reference magnitudes usually reflect cascade ruptures.

Regional Seismic Source Parameters

Fault Name (segment)	Distance from Site (km)	Length (km)	Geologic Slip Rate (mm/yr)	Magnitude @ 2% in 50 Yr. Prob., M_W
San Jacinto (w/ stepovers)	11.2	25	14.0	8.1
San Andreas (Coachella→Mojave South)	26.5	302	10.0 to 32.5	8.25

Version 3 of the Uniform California Earthquake Rupture Forecast (UCERF3) is the reference fault source model for the current California building codes and insurance risk analyses. Utilizing knowledge of tectonic slip rates and last historical or constrained paleoseismic event dates, UCERF3 includes *time-dependent* rupture probabilities for many major California faults. For the San Jacinto fault zone (stepovers combined) between Hemet and Moreno Valley, the model ascribed a 13.8% chance for an earthquake of $M \geq 6.7$ in the next 30 years beginning in 2015 (Field et al., 2015). The conditional probability for an earthquake of magnitude $M_W \geq 6.7$ somewhere along the southern San Andreas Fault was calculated at 57 percent in 30 years. These probabilities will increase each year for successive 30-year windows. Most researchers peg the southern San Andreas as “overdue” for a very large earthquake.

Earthquake shaking hazards are quantified by deterministic calculation (specified source, specified magnitude, and a distance attenuation function), or probabilistic analysis (chance of intensity exceedance considering all sources and all potential magnitudes for a specified exposure period). With certain special exceptions, today’s engineering codes and practice generally utilize (time-independent) probabilistic hazard analysis. Prescribed parameter values calculated for the latest 2014 U.S. national hazard model indicate the site has a 10 percent risk in 50 years of peak ground accelerations (pga) exceeding approximately 0.47g, and 2 percent chance in 50-year exposure period of exceeding .75g (U.S. Geological Survey, 2020b). The reported pga values were linearly interpolated from 0.01-degree gridded data and include soil correction

(NEHRP site class D; local shear wave velocity estimate $V_{s30} \approx 280$ m/sec). Calculated peak or spectral acceleration values should never be construed as representing exact predictions of site response, however. *Actual* shaking intensities from any seismic source may be substantially higher or lower than estimated for a given earthquake event, due to complex and unpredictable effects from variables such as:

- Near-source directivity of horizontal shaking components
- Fault rupture propagation direction, length, and mode (strike-slip, normal, reverse)
- Depth and consistency of unconsolidated sediments or fill
- Topography
- Geologic structure underlying the site
- Seismic wave reflection, refraction, and interference (basin effects)

5.5.3 Secondary Seismic Hazards

Secondary hazards include landsliding or mass wasting, liquefaction, flooding (from ruptured tanks or canals, inundation following dam collapse, surface oscillations in enclosed water bodies, or tsunami), and combined saturated-unsaturated soil subsidence as a result of dynamic soil densification. All of these induced hazards are consequences of earthquake ground motion given the right set of initial conditions.

Flooding. AGI categorically rules out tsunami and seiche hazards. The project site is inland and not adjacent to lakes or open reservoirs. The site is beyond mapped dam breach inundation zones that could be affected by loss of Lake Perris. Induced flooding risks from domestic water storage tanks or vessels in the municipal water treatment plant west of the Interstate 215 freeway are considered insignificant.

Liquefaction. Riverside County classifies the site as "moderate" liquefaction potential. The site is not within State-delineated "Zones of Required Investigation" for either liquefaction potential or landsliding (California Department of Conservation, 2020b). Opportunity is present, as evidenced by groundwater locally less than 30 feet deep. However, our investigation findings for the subject site and the larger Building 1 project area are that the Natwar Lane area lacks liquefaction-susceptible materials. The sedimentary layers are geologically old and have very high relative densities. Field tests demonstrated that older alluvium universally has corrected SPT $N_{1(60)cs}$ values exceeding 30. Worldwide empirical data have demonstrated that liquefaction triggering is extremely unlikely whenever saturated soils meet a criterion of corrected $N \geq 30$. The site *passes* regulatory screening criteria used to differentiate sites with liquefaction hazard from those that have minimal hazard (California Department of Conservation, 2008). Related permanent ground deformation phenomena such as ground fissuring, ejection of pressurized sand-water mixtures from shallow liquefied layers (sand boils), flow slides, and lateral spreading have also been ruled out as hazards.

Subsidence. AGI finds that surface settlements from saturated and dry-sand volumetric changes should be trivial assuming that very shallow soils are treated by remedial grading for structural support. Calculated total surface settlements from a liquefaction model analysis are of extremely low magnitude (approximately 0.1 inch). Differential settlements would be even less. We think the tiny calculated differential settlement potentials are reasonable engineering assumptions for this site. Both the total and differential settlements are far lower than typical allowable maximum deflections for concrete panel-wall construction on continuous foundations.

Landslides. Section 5.3 notes that the site is flat and far from steep or boulder-strewn mountain slopes. Earthquake-induced hazards from slope instability or tumbling rocks are believed to be zero.

6.0 CONCLUSIONS AND RECOMMENDATIONS

6.1 General

Based on the results of our field exploration and laboratory tests, engineering analyses, local experience, and judgment, it is our professional opinion that the project site should be suitable from a geotechnical viewpoint for the proposed project. Geological hazards imposed on the industrial building would appear to be limited to strong ground motion due to earthquake. Geotechnical constraints include surficial lower-density natural materials judged mildly susceptible to hydrocollapse and compression under building loads. Alluvium below 3 to 4 feet, however, is demonstrably at or above 90 percent relative compaction and partly cemented. Some near-surface zones are clayey and categorized as expansive, though.

Prescriptive mitigation for the hazard of strong ground motion is nominally provided structural design adherence to local adopted building codes. Section 6.7 contains recommended short- and long-period design spectral accelerations for the project.

Soil excavation and compaction to create dense engineered fill are recommended to mitigate unsuitable alluvial deposits and mechanically disturbed horizons that would otherwise be present below shallow structural foundations, pavements, and inferred site walls. Listed below are the recommended earthwork actions for existing soil conditions impacting site development:

(1) Remedial grading should replace all loose, disturbed silty sand horizons as compacted engineered fill beside and below the warehouse building envelope, and any future CMU or concrete-panel site wall foundations. Based on the exploration logs, expected structural "removal" depths from existing grades should be approximately 3 feet. The contact line between unsuitable soils and competent bottom materials should be fairly obvious during mass grading, with an abrupt change expected from looser and crumbly soils to cohesive, hard sediments. Structural "removal" limits should be at least 5 feet beyond foundation concrete.

(2) Overexcavations should be deepened, if required, so that at least 24 inches of engineered fill is created beneath all future continuous or spread footings. If the project ultimately utilizes concrete site walls rather than fences around the property perimeter, wall footings should also bear on a minimum of 24 inches of engineered fill. Lateral excavation limits at final bottom elevations should be at least 5.0 feet beyond footing edges.

(3) At least 18 to 24 inches of soil stripping before placement of compacted engineered fill is recommended in all future new flexible and rigid pavement areas. The upper end of the stripping range is predicted toward the north property line. The remaining

10 to 12 inches may be processed and compacted in place. The intent is to recompact loose, heavily bioturbated, and mechanically tilled soils. Should pavement subgrades be planned more than 18 to 24 inches below current surfaces, in-place processing is recommended to create at least 12 inches of engineered soil fill below flexible or rigid pavement structural sections.

Careful staging of earthwork is urged to help minimize chances for placing expansive soils in the upper foot of industrial floor slab subgrades. Pre-project consultations between AGI and earthwork contractors would be encouraged to formulate plans for initial stockpiling and “round-robin” excavations and fills. Clay contents generally increase with depth. A goal of planning would be to devise schemes to keep excavated clayey soils only in the deeper portions of fills, and selectively retain shallow non-expansive materials typically found in the north half of the site for use in pad finishing. If Building 1 and Building 2 are mass-graded concurrently, we would encourage transfer of cut soils from the Building 1 truck and trailer yard between the warehouse and the freeway into the Building 2 pad outline. These soils are expected to have very low expansion potential and will be preferred materials for floor slab subgrades in both buildings.

6.2 Site Grading

The general guidelines presented below should be included in the project construction specifications to provide a basis for quality control during grading. It is recommended that all compacted fills be placed and compacted under continuous engineering observation and in accordance with the following:

- Demolition and removal of any and all abandoned buried improvements including foundations, slabs, irrigation pipes, tanks, or cables. None are expected.

- Clearing and disposal of weeds, shrubs, and foreign objects should be initiated prior to grading. If necessary in the opinion of the Geotechnical Engineer, the grading contractor must be prepared to supply personnel to pick woody debris or foreign objects from engineered fill during the grading operations.

- Excavation of fill, disturbed or porous native soil, or other unsuitable material as determined at the time of grading by the Geotechnical Engineer shall be performed as discussed in Section 6.1 for support of compacted engineered fill, structures, and improvements. Bottom acceptance will be by geological observation, probing, and density testing in alluvium. Competent soils below buildings shall demonstrate in-place dry densities of 85% or greater of the laboratory-determined maximum dry density to be accepted, and exhibit insignificant macro-porosity. All of the site soils appear to be acceptable for re-use in new engineered compacted fill if free from organic debris and trash. Final determinations of removal depths shall be made during grading based upon conditions encountered during earthwork activities.

- Observation and acceptance of all stripped areas by the Geotechnical Engineer and/or Engineering Geologist and/or their designated representative shall be done prior to placing fill.

- Shallow scarification of exposed bottoms to depths of 4 to 6 inches (structural envelope), or to planned processing depths (pavement and other engineered fill areas), moisture-conditioning by adding moisture or drying back to desired moisture contents as described below, and recompaction to at least 90 percent of the maximum dry density as determined by the ASTM D1557-12 test standard.

- Fill soils should be uniformly moisture-conditioned by mixing and blending to optimum water content or higher, and placed in lifts having thicknesses commensurate with the type of compaction equipment used, but generally no greater than 6 to 8 inches. Pre-watering of the site is recommended in advance of earthwork (depending upon seasonal conditions) to moisten the upper 36 inches of material. This will help reduce fugitive dust, and more importantly allow for easier mixing and clod crushing. Care will be needed to avoid overwatering the deeper clayey horizons and creating sticky, muddy, impassable conditions. *Fill water contents below the recommended minimum water content shall constitute a basis for non-acceptance of the fill irrespective of measured relative compaction, and at the discretion of the Geotechnical Engineer may require the* *fill be reworked to produce uniform water contents at or over the desired 100% of optimum moisture.*

- The contractor should utilize means and methods that result in uniform compaction of engineered fill meeting at least 90 percent of the laboratory maximum dry density determined by the ASTM D1557-12 standard. Sheepsfoot rollers and/or a Rex compactor are recommended for mixing and kneading action that will be needed to distribute water in clayey fill soils and break down clods. AGI recommends the uppermost 12 inches of pavement or base-course subgrade material achieve at least 95 percent relative compaction for soil classifications SM, SP-SM, SC, or related USCS coarse-grained classifications predicted for this site.

- Rocks or other similar irreducible inert particles larger than about 3 inches in diameter should be excluded from engineered structural fills on this site. Rocks should be very rare or absent.

- Field observation and testing shall be performed to verify that the recommended compaction and soil water contents are being uniformly achieved. Where compaction of less than 90 percent is indicated (95 percent in identified subgrade zones as previously noted), additional compaction effort, with adjustment of the water content as necessary, should be made until at least minimum-accepted compaction is obtained. Field density tests should be performed at frequencies not less than one test per 2-foot rise in fill elevation and/or per 1,000 cubic yards of fill placed and compacted at this site.

- Import soils, if required, should consist of predominantly granular material with low or negligible expansion potential and be free of deleterious organic matter and large rocks. The borrow site and import soils must be reviewed and accepted by the Geotechnical Engineer prior to use. Geotechnical acceptance will only be predicated on meeting certain engineering criteria, and would not address any environmental testing or clearances required by local agencies or by the proposed end use.

- Practices and design features that reduce chances for cyclical water content changes and correlative soil movement due to expansive clays should be considered. Finish surface contours should everywhere result in drainage being directed away from building foundations to swales, area drains, or water quality BMPs. The use of descending ramps to proposed dock doors should be discouraged; a better approach is an elevated building finish floor and exterior pavement surfaces sloping away from the dock doors. Roof runoff may be directed to planter strips or bioswales located at least 25 feet from the building. Landscape beds should not be placed next to the proposed structure unless xeriscape and micro-irrigation design practices can be enforced.

- It is recommended that expansion index and soluble sulfate content tests (3 each, or as needed to encompass all as-built soil types, whichever is greater) be performed upon completion of rough grading in the warehouse building pad. Atterberg limits testing to help qualify soil activity is recommended in the event an as-built expansion index greater than 20 is calculated.

6.3 Earthwork Volume Adjustments

Removal and recompaction of the unsuitable surficial alluvium will result in material volume loss. The calculation of earth balance factors for the site as a whole is subject to some uncertainty, based on imprecise estimates of shallow soil density from 0 to 2 feet (tilled zone), and the future achieved degrees of compaction. We believe that civil designers should make allowances for at least 12 to 15 percent shrinkage in the building removal area. Exterior paved areas may shrink closer to 20 percent from 0 to 2 feet. Bottom subsidence from heavy equipment is predicted to be almost undetectable in the cemented soils, and would conservatively not even reach 0.05 foot.

6.4 Slopes

Slopes are not depicted on the conceptual drawing, and should not be needed on the flat site. Nonetheless, designs can change or temporary boundary slopes may be required until adjacent projects are built. Cut slopes in the project would encounter natural silty sand. Slope design should in general conform to the following recommendations:

- Cut and fill slopes should be constructed at maximum slope inclinations of 2:1 (horizontal:vertical).
- The surfaces of all fill slopes should be compacted as generally recommended under Site Grading, and should be free of slough or loose soils in their finished condition. The desired result should be 90 percent relative compaction to the slope face.
- The fill portion of any fill-over-cut slopes should maintain a minimum horizontal thickness of 5 feet or one-half the remaining fill slope height (whichever is greater), and be adequately benched into undisturbed competent materials. Cut slopes in local native surficial alluvium are preliminarily judged feasible up to 5 feet high without needs for stabilization fills. Taller slopes, and any slope exposing low-cohesion soils such as clean or nearly clean sand should be reconstructed as stabilization fill slopes with the same minimum horizontal dimensions.
- Erosion control measures should be implemented for all slopes as soon as practicable after slope completion, per applicable City ordinances.

6.5 Foundation Design

Although information regarding anticipated foundation loads was not available for this report, the predicted construction type implies moderate imposed soil loads. Foundation plans, once they become available, must be evaluated by this firm for compatibility with the preliminary recommendations presented below.

Conventional shallow continuous or spread footings embedded entirely within compacted engineered fill appear feasible for the light industrial building. Structural loads may be supported on continuous or isolated spread footings at least 18 inches wide. *All* footings including site wall foundations should be bottomed a minimum of 24 inches below the lowest adjacent final grade. The recommended maximum allowable bearing value is limited to 3,000 pounds per square foot (FS $\geq$3.0). Bearing values may be increased by one-third when considering short-duration seismic or wind loads.

Lateral load resistance will be provided by friction between the supporting materials and building support elements, and by passive pressure. A friction coefficient of 0.39 may be utilized for foundations and slabs constructed atop structural fill composed of granular site materials. A passive earth pressure of 250 pounds per square foot, per foot of depth, may be used for the sides of footings. When combining passive pressure and frictional resistance, the passive pressure component should be reduced by one-third.

Any exterior isolated building footings should be tied in at least two perpendicular directions by grade beams or tie beams to reduce the potential for lateral drift or differential distortion. The base of the grade beams should enter the adjoining footings at the same depth as the footings (viewed in profile). The grade beam steel should be continuous at the footing connection. Footings should either be continuous across large openings, such as loading docks or main entrances, or be tied with a grade beam or tie beam.

Interior columns should be supported on spread footings or integrated footing and grade beam systems. Column loads should not be supported directly by slabs. When designing the interior building footings, the structural engineer should consider utilizing grade beams to control lateral drift of isolated column footings, if the combination of friction and passive earth pressure will not be sufficient to resist lateral forces.

Minimum foundation reinforcement should consist of four No. 5 bars, two near the top and two near the bottom (viewed in cross-section), or as dictated by loading conditions. However, footing and grade beam reinforcement specified by the project structural engineer shall take precedence over the latter guidelines.

Provided that AGI's recommendations for engineered fill depths below footings are incorporated into final design and construction, foundation settlements should be of low magnitude. Much of the anticipated foundation settlement is expected to occur during construction. Maximum consolidation settlements are not expected to exceed a ½-inch and should occur below the heaviest loaded columns. Differential settlement is not expected to exceed approximately ¼ to ½ of an inch between similarly loaded elements in a 30-foot span.

6.6 Floor Slab Design

Concrete slab-on-grade industrial floor construction is planned. The following recommendations are presented as options for minimum design parameters for the slabs, accounting for soil expansive pressures and measured soil strengths only. The minimum design parameters do not account for concentrated loads (e.g., machinery, pallet racks, etc.) and/or the installation of freezers or heating boxes.

The information and recommendations presented in these sections are not meant to supersede design by the project structural engineer. We have conceptualized options based on an as-built subgrade having an expansion index between 20 and 50, and plasticity index of 10 or less. This condition would be reflective of some degree of selective grading to actively exclude the site's more-clayey soils from floor subgrades. It should be possible to achieve expansion indices of under 20. Generally, the indicated dimensions or materials may be varied by the structural engineer to produce acceptable performance for heavy or point loads, or to reduce section thicknesses. Final verification of the applicability of these or any modified recommendations must be confirmed by expansion index testing at the conclusion of pad precise grading.

Lightly Loaded Floor Slabs. Commercial/office slabs in areas which will receive relatively light live loads (i.e., less than approximately 125 psf) may be a minimum of 4.5 inches thick if reinforced with No. 3 reinforcing bars at 18 inches on-center in two horizontally perpendicular directions. Reinforcing should be properly supported on chairs or blocks to ensure placement near the vertical midpoint of the slab. "Hooking" of the reinforcement is not considered an acceptable method of positioning the steel. The recommended minimum compressive strength of concrete in this application is 3,000 pounds per square inch (psi).

Transverse and longitudinal control joints are advised to isolate slab cracking due to concrete shrinkage or expansion. If utilized in lieu of added reinforcement or concrete additives, crack control joints should be spaced no more than 12 feet on center and constructed to a minimum depth of T/4, where "T" equals the slab thickness in inches.

Construction joints between pours should utilize dowel baskets to control vertical deflections from either interior loads or soil uplift pressures.

Highly Loaded Floor Slabs. The project structural engineer should design slabs in the event of expected high loads (i.e., machinery, forklifts, storage racks, etc.). Designs utilizing the modulus of subgrade reaction (k-value) may be used. A k-value of 150 pounds per square inch per inch may conservatively used for on-site soils. Recommended R-value tests for final pavement section design, and/or plate load tests, may be used to verify the subgrade modulus after completion of grading.

For live loads of up to 250 psf, plain concrete slabs should be at least 5½ inches thick. The concrete used in slab construction should conform to Class 560-C-3250. Transverse and longitudinal crack control joints (if utilized) should be spaced no more than 12 feet on center and constructed to a minimum depth of T/4, where "T" equals the slab thickness in inches. Construction joints between pours should utilize dowel baskets to control vertical deflections from either interior loads or soil uplift pressures. These suggested design factors can be altered as long as comparable stiffness and strength objectives can be achieved.

Moisture Protection. Ground-floor office portions of the warehouse building slab would be expected to have interior floor finishes (wood, vinyl, carpet) potentially sensitive to subgrade moisture or water vapor. Code requirements specify a minimum 6-mil-thick plastic vapor retarder installed per manufacturer guidelines with all laps/openings sealed. The barrier may be situated atop as-built subgrades if reasonably free of large stones. We advise that optional thicker 10-mil vapor retarders (e.g., StegoWrap® or Viper Vaporcheck® engineered membranes) should be favored due to greater damage resistance and even lower transmissivity. Protected areas should be separated from any areas that are not similarly protected. The separation may be created by a concrete cut-off wall extending at least 24 inches into the subgrade soil.

Subgrade Pre-Saturation. Pre-saturation is recommended for all pad soil and pedestrian walkway subgrades demonstrating post-grading expansion indices exceeding 20. A benefit of selective grading would be reduced chances for code-based categorization as an “expansive” pad. As already detailed in other sections, substitution and/or pad finishing with soils from the shallowest horizons and most-northern site areas (or cut areas west of Building 1) could bring expansion induces under 20. For as-built expansion indices under 20, AGI would recommend that soil water contents at least approach optimum soil water contents determined from ASTM D1557-12 to a depth of at least 12 inches prior to vapor retarder installation or commercial slab concrete placement. Extremely dry soils can pull water from wet concrete by capillary action and potentially affect hydration of cement pastes. Construction sequencing that helps preserve grading water should be encouraged. Soils with pad as-built expansion indices in the range of 20 to 50 should be at or over 110 percent of optimum water content to a depth of 12 inches; 120 percent of optimum or higher will be required if soils lie in the “medium” range of 50 to 90. Subgrade soil water contents should be checked and verified as suitable by AGI technical staff no more than 48 hours prior to concrete placement.

6.7 2019 California Building Code Seismic Criteria

Prescriptive mitigation for the hazard of strong ground motion is nominally provided by structural design adherence to local adopted building codes. The 2019 California Building Code (CBC), based on the 2018 *International Building Code*, maintains a “look-up” code convention for seismic engineering, using as primary inputs the site’s location and the assigned site class. The latter is a measure of shallow-earth elastic resistance determined by borehole tests, depth to bedrock, and/or geophysical methods. The updated 2019 code quantifies seismic risk based on the newer probabilistic 2014 National Seismic Hazard model. Design coefficients are ultimately functions of distance to active faults, fault activity, and measured or correlated mean shear wave velocity within 30 meters (~100 feet) of the ground surface. The tabulated criteria presented below were derived in accordance with the rules of Section 1613 of the 2019 CBC and ASCE/SEI Standard 7-16. The northeast building corner was conservatively selected for minimum site-to-source distance.

Table 6.7-1
2019 CBC Seismic Design Factors and Coefficients
(Lat. 33.87006°N, Long. 117.25741°W)

2019 CBC Section #	Seismic Parameter	Indicated Value or Classification
1613.2.1	Mapped Acceleration MCE_R S_s	1.500g (Note 1)
	Mapped Acceleration MCE_R S_1	0.589g (Note 1)
1613.2.2	Site Class	D (Note 2)
1613.2.3	Site Coefficient F_a	1.0
	Site Coefficient F_v	1.7 (Note 3)
1613.2.3	Adjusted MCE_R Spectral Response S_{MS}	1.500g
	Adjusted MCE_R Spectral Response S_{M1}	1.001g
1613.2.4	Design Spectral Response S_{DS}	1.000g (Note 4)
	Design Spectral Response S_{D1}	0.667g (Note 4)

Notes

(1) Interpolated from 0.01-degree gridded data in the probabilistic 2014 National Seismic Hazard Model (SEAOC, 2020), 2% in 50-year exceedance probability.

(2) Determinate classification, based on minimal site grading, borehole SPT data, known depth to bedrock >>30 meters, and estimated V_{s30} ≈260 m/sec. Soft clay horizons are absent.

(3) Provided that equivalent lateral force procedures are used to determine seismic resisting elements of the structures, and the seismic response coefficient C_s is determined in accordance with ASCE 7-16 §12.8.1.1.

(4) Defined by 2019 CBC §1613.1 and ASCE/SEI 7-16 §11.4.5. A *site-specific* MCE_R response spectral acceleration at any period shall be taken as the lesser of the probabilistic or deterministic spectral response accelerations, with the latter subject to lower-limit values. The design spectral response accelerations are calculated as ⅔ of the MCE_R value.

Based on ASCE 7-16 and CBC §1613.2.5, a Seismic Design Category of **D** for risk category I-III buildings/structures is assigned for buildings sited where $S_{D1} > 0.20g$ and $S_1 < 0.75g$. The option for alternative seismic design category determination based on a structure's fundamental period and CBC Table 1613.2.5(1) is allowed. The site-modified zero-period MCE_R ground motion estimate PGA_M is 0.557g. Seismic response coefficients determined by the USGS tool from Figures 22-18A and 22-19A of ASCE 7-16 would be:

$C_{RS} = 0.935$ $C_{R1} = 0.912$

It should be understood that the 2019 CBC and most other building codes define minimum criteria needed to produce acceptable life-safety performance. Code-compliant structures can still suffer damage. Project owners should be aware that structures can be designed to further limit earthquake damage, sometimes for modest cost premiums. Ultimately, final selection of design coefficients should be made by the structural consultant based on local guidelines and ordinances, expected structural response, and desired performance objectives.

6.8 Pavements

Depending upon budget, aesthetics, life-cycle costs, and proposed end use, Portland cement concrete (PCC) pavement or a mix of PCC and lighter-duty asphalt surfaces could be specified for the project. Customarily, truck driveways and trailer stalls use PCC pavement. Street pavements are no longer being considered since the Western Way street extension is nearing completion by others. Design assumptions explicitly assume that the uppermost porous and mechanically tilled topsoils in areas that will support new asphalt or PCC pavements, curbs and gutter, sidewalks, or other flatwork will be removed and recompacted as recommended in Section 6.1.

For a commonly accepted truck-traffic index of 8.0, equivalent maximum single-axle loads of 13,000 pounds, an R-value of 34 or greater as expected for local soils, and assumed concrete modulus of rupture of 500 psi, the recommended preliminary PCC design section includes 6.5 inches of un-reinforced (plain) concrete, over 12 inches of granular site soil compacted to not less than 95 percent relative compaction. Concrete used for pavement should have a minimum 28-day compressive strength f_c of 3,500 pounds per square inch. The structural engineer could consider alternative sections that include reinforcement or different-strength concrete mixes in the event of a different design traffic index, special conditions including ESALs exceeding 13,000 pounds, or requests for a thinner concrete section.

The following table presents *preliminary* recommended structural sections for standard-duty and heavy-duty hot mix asphalt pavements based upon Caltrans design methods, a 20-year pavement lifetime, and a representative soil R-value obtained from the Western Way alignment as reported in AGI (2019). The tabulated dimensions for standard-duty pavement (passenger automobile loads) represent the minimum-recommended vehicular structural section. Final recommended sections may change and should be based on expected loading, desired pavement lifetime, and recommended R-value tests on soils collected from as-built subgrades.

Table 6.8-1
Preliminary Conventional Asphalt Pavement Designs

Pavement End Use	Traffic Index	R-Value	A.C. Thickness	Base Thickness
Standard Duty Passenger Auto Aisles & Parking	5.5	34	3.0"	6.5"
Heavy Duty Semi Truck Loading	8.0	34	4.0" 5.0"	11.0" 9.5"

It is recommended that concrete curbs and ribbon gutters be poured neat against compacted soil subgrades in advance of pavement subgrade excavation and base course placement. It is especially critical that drainage pathways from tree wells or nearby landscaped areas not be created by inadvertent construction of curbs atop permeable base course layers.

Generally, subexcavation of pavement areas should not exceed that needed to mitigate compressible surficial soils described in Section 6.1. Granular subgrades should be processed and compacted to a minimum of 95 percent of the laboratory maximum dry density determined by ASTM D1557-12 to depths of at least 12 inches. Base course should meet materials specifications for Caltrans Class 2 aggregate base material or better, and should be placed and fully compacted in lifts no greater than 6 inches thick to a minimum dry density of 95 percent of the laboratory maximum dry density per the ASTM D1557-12 standard. Pavement gradients should be designed to allow rapid and unimpaired flows of runoff water, and concrete gutters should be provided at all flow lines.

Owners, designers, and general contractors should be aware that Class 2 base material may be composed of virgin natural stone (“crushed aggregate base” or CAB), or *reclaimed materials* such as crushed concrete and pulverized asphalt (crushed miscellaneous base, CMB). Reclaimed base containing concrete has been the source of unsatisfactory pavement performance at multiple Southern California projects due to unintended contamination with reactive aluminum metal fragments. Surface distress manifests as permanent pavement "bumps" or "pimples". It is not clear at this time that the problem is limited to only certain suppliers, or whether local suppliers can provide warranties for delivered product. The most conservative option is to specify only "CAB" for flexible pavement base courses, in our opinion.

6.9 Retaining Walls

Available plans did not depict retaining walls, and the limited site relief suggests walls may be avoidable except possibly for dock door areas. Preliminary recommended earth pressure values for walls are shown below. AGI assumes that a well-drained, select granular on-site or import material such as locally available decomposed granite sand with a sand equivalent value of 30 or better will be utilized for backfill. Very silty sand or clayey site soils are not recommended for wall backfill. Live loading (e.g., forklifts) must be added to the stated values. Seismic inertial loads must be added to static earth loads for any wall structure retaining more than 6 feet of material. Seismic loads for any Building 2 walls may be based on a design peak ground acceleration of 0.371g [⅔PGA_M] and MCE mode-magnitude event M_w8.1. Other expected site conditions such as drained, granular backfill soils (moist compacted γ_{soil} = 130 pcf is reasonable) appear to be consistent with the assumptions of the widely used Mononobe-Okabe method or similar later variations of rigid plastic methods for finding force magnitudes on walls. Standard reduction factors for pga (e.g., 0.5 for M-O method) may thus be implemented.

Table 6.9-1
Preliminary Retaining Wall Fluid Pressure

Inclination of Retained Material	**Equivalent Fluid Pressure (psf)**	
	Unrestrained	Restrained
Level	36	56

It is recommended preliminary wall designs be reviewed by AGI for locality-specific modifications and/or needs for additional soil tests before construction. The same recommended maximum foundation bearing value of 3,000 psf for structures may also be assumed for retaining walls and site walls founded atop engineered fill. Granular wall backfill at dock doors should be mechanically compacted to a minimum of 95 percent relative compaction; 90 percent or greater is sufficient where not subject to live loads. Density testing is recommended to verify the adequacy of compaction. Substitution with crushed or pit-run clean rock materials in wall panel pour strip backfills is allowable, but should also be accompanied by mechanical densification with plate compactors, ramming tampers, or concrete vibrators.

Exterior walls retaining more than 3 feet of soil should be provided with a means of drainage to prevent hydrostatic forces. Drainage provisions may be based on the wall height, wall length, and any irrigated land uses next to the improvement. Typical approaches would be a continuous perforated subdrain line embedded in open-graded crushed rock placed at the inside bottom of the wall, or through-the-wall options such as weepholes, or open head joints for CMU structures.

6.10 Temporary Excavations

Excavations at the site would be expected to encounter massive, cohesive sequences of very old alluvium, and/or engineered fill after mass grading. Excavations up to 5 feet in depth in these materials should stand vertically for temporary periods. Trenches open for any extended period of time, trenches placed in disturbed native ground, and all excavations greater than 5 feet in depth should be properly sloped or shored. Where sufficient space is available for a sloped excavation and the cut will be open for 24 hours or less, the side slopes should be inclined to no steeper than ½:1 (horizontal to vertical) per current rules for excavation material Type A and an excavation depth of 12 feet or less in unsaturated soil. Preliminary information is that slot-type trenches may be feasible with typical aluminum “speed shores” for laborer protection. The exposed earth materials in the excavation side slopes should be observed and verified as suitable by a geotechnical engineer. The exposed slope faces should be kept moist and not allowed to dry out.

Surcharge loads should not be permitted within five feet from the top of excavations, unless the cut or trench is properly shored. Contractors are ultimately responsible for verifying that slope height, slope inclination, excavation depths, and shoring design are in compliance with Cal-OSHA safety regulations (Title 8, Section 1540-1543 et seq.), or successor regulations.

6.11 Trench Backfill

All soil-backfilled utility trenches on this site should be backfilled in lifts and mechanically compacted to at least 90 percent of the laboratory maximum dry density. Utility purveyors may specify a greater degree of compaction in streets (e.g., lateral connections into Natwar Lane) than this stated minimum. Flooded or jetted backfill is not recommended except for densification of select imported granular bedding materials placed directly around utility lines. The local soils are deemed unsuitable to serve as pipe bedding materials due to silt and clay content. Density testing is recommended to verify the adequacy of compaction efforts.

6.12 Soil Corrosivity

Chemical analyses were performed to provide a general evaluation of the corrosivity of the native soils and included soluble sulfates, soluble chlorides, soil pH, and minimum saturated resistivity. Findings indicated the site soils should not be highly aggressive to concrete, but could be corrosive to buried metal. Analytic tests reported a soluble sulfate content of only 0.0071 weight percent in a sample from the proposed building site. Saturated resistivity was 4,020 ohm-cm. The latter roughly corresponds to a “moderate” potential for electrolytic corrosion of iron and unprotected mild steel. We encourage the owner to engage a qualified corrosion engineer for a more in-depth evaluation of risks to buried ferrous objects and for specification of special corrosion protection features that may be required. Fire protection lines should be keyed upon.

The categorically “negligible” sulfate concentrations indicate that normal Type I-II cement should be suitable for concrete mix designs utilized for this project, based on American Concrete Institute (ACI) 318 Table 4.3.1. Type V cement may optionally be used for any site concrete mix, and would be mandatory for measured sulfate concentrations exceeding 0.20 weight percent. It is recommended that all concrete which will come in contact with on-site soil materials be selected, batched, and placed in accordance with the latest California Building Code and ACI technical recommendations.

6.13 Construction Observation

The preliminary foundation recommendations presented in this report are based on the assumption that all foundations will bear entirely within properly compacted engineered fill approved by this office. It is recommended that all engineered fill placement operations be performed under continuous engineering observation and testing by AGI personnel. Engineered fill shall constitute any load-bearing soil placements, irrespective of yardage quantity or depth. Continuous observation is a 2019 CBC requirement for engineered fill. Continuous or periodic fill observation and testing may be suitable for trench backfills depending mostly on trench depth and contractor production. Verification testing of completed soil-subgrade expansion potential, soluble sulfate content, soil plasticity index, and pre-saturation (if required) is recommended at appropriate points in the construction time line. All foundation excavations should be observed prior to placing reinforcing steel to verify that foundations are embedded within satisfactory materials and that excavations are free of loose or disturbed soils and made to the recommended depths.

6.14 Investigation Limitations

The present findings and recommendations are based on the results of the field exploration combined with interpolations of soil and groundwater conditions between a limited number of subsurface excavations. The nature and extent of variations beyond or between the explorations may not become evident until construction. If conditions encountered during construction vary significantly from those indicated by this report, then additional geotechnical tests, analyses, and recommendations could be required from this office. Because this report has also incorporated assumed conditions or characteristics of the proposed development where specific information was not available, foundation plan and grading plan reviews by this firm are recommended prior to construction in order to evaluate the proposed facilities from a geotechnical viewpoint and allow modifications to the preliminary recommendations developed to date.

We recommend that the project engineer incorporate this report and subsequent plan review reports into the Building 2 project specification by title and date references on final drawings. Lastly, a pre-construction meeting with the owner, grading contractor, and civil engineer is strongly encouraged to present, explain, and clarify geotechnical concerns, uncertainties, and recommendations for the site.

7.0 CLOSURE

This report was prepared for the use of First Industrial Realty Trust, Inc. and their designates, in cooperation with this office. All professional services provided in connection with the preceding report were prepared in accordance with generally accepted professional engineering principles and local practice in the fields of soil mechanics, foundation engineering, and engineering geology, as well as the general requirements of Riverside County and the City of Perris in effect at the time of report issuance. We make no other warranty, either expressed or implied. We cannot guarantee acceptance of the final report by regulating authorities without needs for additional services.

We appreciate the opportunity to help engineer your planned business improvements in the Inland Empire. Consultations concerning our findings or alternative design approaches are invited. Please direct your inquiries to our Riverside office at (951) 776-0345, or via the convenience of email at www.aragongeo.com.

Respectfully submitted,
Aragón Geotechnical, Inc.

PROFESSIONAL GEOLOGIST
MARK G. DOERSCHLAG
No. 1752
Exp. 9/30/22
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

12/18/2020

Mark G. Doerschlag, CEG 1752
Engineering Geologist

REGISTERED PROFESSIONAL ENGINEER
CARLOS FERNANDO ARAGON
No. 2994
Exp: 06/30/22
GEOTECHNICAL
STATE OF CALIFORNIA

C. Fernando Aragón
Geotechnical Engineer, G.E. No. 2994

Signed 12-19-20

MGD/CFA:mma

Attachments: Appendices A and B
Geotechnical Map, Plate No. 1 (foldout)

Distribution: (4) Addressee

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AERIAL PHOTOGRAPHS

RCFCWCD Aerial Photography Collection, Riverside

Date Flown	Flight Number	Scale	Frame Numbers
1-28-62	Fairchild #24244	1:24,000	Line 1, Nos.83-84
5-24-74	1974 County	1:24,000	Nos. 307-308
2-4-84	1984 County	1:19,200	Nos. 1148-1149
1-21-90	1990 County	1:19,200	Line 6, Nos. 26-27
1-30-95	1995 County	1:19,200	Line 6, Nos. 25-26
3-11-00	2000 County	1:19,200	Line 6, Nos. 26-27
4-14-05	2005 County	1:19,200	Line 6, Nos. 25-26
3-14-10	2010 County	1:19,200	Line 6, Nos. 25-26

U.C. Santa Barbara Aerial Image Collections

Date Flown	Flight Number	Scale	Frame Numbers
6-7-38	AXM-1938A	1:20,000	Line 35, #73
8-28-53	AXM-1953B	1:20,000	Line 5K, #14
5-15-67	AXM-1967	1:12,000	3HH-120
3-8-04	EAG RV 04	1:21,000	616

Google Earth Pro Historical Image Archive

Image dates as shown in application:

5/31/94	1/3/06	2/9/16
5/21/02	4/27/06	10/21/16
12/30/02	5/24/09	2/9/18
10/25/03	11/15/09	8/10/18
11/13/03	3/9/11	8/24/18
1/4/04	6/17/12	
10/10/05	11/12/13	
12/2005	4/27/14	

APPENDIX A

A P P E N D I X A

MAP EXPLANATION & SUBSURFACE EXPLORATION LOGS

The Geotechnical Map (Plate No. 1, foldout at the back of this report) was prepared based upon information supplied by the client, or others, along with Aragón Geotechnical's field measurements and observations. Field exploration locations illustrated on the map were derived from taped and paced measurements of distance to surrounding improvements, and should be considered approximate. The selected boring locations were deemed sufficient by AGI for characterizing the possible range of subsurface conditions occurring at the site. Building 1 exploration site data may be found in AGI (2019).

The Field Boring Logs on the following pages schematically depict and describe the subsurface (soil and groundwater) conditions encountered at the specific exploration locations on the date that the explorations were performed. Unit descriptions reflect predominant soil types; actual variability may be much greater. Unit boundaries may be approximate or gradational. Text information often incorporates the field investigator's interpretations of geologic history, origin, diagenesis, and unit identifiers such as formation name or time-stratigraphic group. Additionally, soil conditions between recovered samples are based in part on judgment. Therefore, the logs contain both factual and interpretive information. Subsurface conditions may differ between exploration locations and within areas of the site that were not explored. The subsurface conditions may also change at the exploration locations over the passage of time.

The investigation scope and field operations were conducted in general accordance with the procedures recommended by the American Society for Testing and Materials (ASTM) standard D420-98 entitled "Site Characterization for Engineering Design and Construction Purposes" and/or other relevant specifications. Soil samples were preserved and transported to AGI's Riverside laboratory in general accordance with the procedures recommended by ASTM standard D4220 entitled "Standard Practices for Preserving and Transporting Soil Samples". Brief descriptions of the sampling and testing procedures are presented below:

Ring-Lined Barrel Sampling – ASTM D3550-01

In this procedure, a thick-walled barrel sampler constructed to receive thin-wall liners (either a stack of 1-inch-high brass rings or 6-inch stainless steel tubes for environmental testing) is used to collect soil samples for classification and laboratory tests. Samples were collected from selected depths in all 6 hollow-stem auger borings. The drilling rig was equipped with a 140-pound mechanically actuated automatic driving hammer operated to fall 30 inches, acting on rods. A 12-inch-long sample barrel fitted with 2.50-inch-diameter rings and tubes plus a waste barrel extension was subsequently driven a distance of 18 *inches or to practical refusal (considered to be ≥50 blows for 6 inches). The raw blow counts for each 6-inch increment of penetration (or fraction thereof) were recorded and are shown on the Field Boring Logs. An asterisk (*) marks refusal within the initial 6-inch seating interval. The hammer weight of 140 pounds and fall of 30 inches allow rough correlations to be made (via conversion factors that normally range from 0.60 to 0.65 in Southern California practice) to uncorrected Standard Penetration Test N-values, and thus approximate descriptions of consistency or relative density could be derived. The method provides relatively undisturbed samples that fit directly into laboratory test instruments without additional handling and disturbance.*

Standard Penetration Tests – ASTM D1586-11

In deeper boreholes, Standard Penetration Tests were performed to recover disturbed samples suitable for classification, and to provide baseline data for liquefaction susceptibility analysis and site class for seismic design. A split-barrel sampler with a 2.0-inch outside diameter is driven by successive blows of a 140-pound hammer with a vertical fall of 30 inches, for a distance of 18 inches at the desired depth. The drill rig used for this investigation was equipped with an automatic trip hammer acting on drilling rods. The total number of blows required to drive the sampler the last 12 inches of the 18-inch sample interval is defined as the Standard Penetration Resistance, or "N-value". Penetration resistance counts for each 6-inch interval and the raw, uncorrected N-value for each test are shown on the Field Boring Logs. Drive efficiencies for automatic hammers are higher than older rope-and-cathead systems, which are disappearing from practice. Where practical refusal was encountered within a 6-inch interval, defined as penetration resistance ≥50 blows per 6 inches, the raw blow count was recorded for the noted fractional interval; an asterisk (*) marks refusal within the initial 6-inch seating interval. The N-value represents an index of the relative density for granular soils or comparative consistency for cohesive soils.

Bulk Sample

A relatively large volume of soil is collected with a shovel or trowel. The sample is transported to the materials laboratory in a sealed plastic bag or bucket.

Classification of Samples

Bulk auger cuttings and discrete soil samples were visually-manually classified based on texture and plasticity, utilizing the procedures outlined in the ASTM D2487-11 standard. The assignment of a group name to each of the collected samples was performed according to the Unified Soil Classification System (ASTM D2488-09). The plasticity reported on field logs refers to soil behavior at field moisture content unless noted otherwise. Site material classifications are reported on the Field Boring Logs.

FIELD LOG OF BORING B - 1

Sheet 1 of 2

Project: ***BLDG. 2, NATWAR LANE INDUSTRIAL PROJECT***

Location: ***CITY OF PERRIS, RIVERSIDE COUNTY, CALIF.***

Date(s) Drilled:	**12/1/20**	Logged By:	**L. Arguello**
Drilled By:	**GP Drilling**	Total Depth:	**26.5 Ft.**
Rig Make/Model:	**Mobile B-61**	Hammer Type:	**Automatic trip**
Drilling Method:	**Hollow-Stem Auger**	Hammer Weight/Drop:	**140 Lb./30 In.**
Hole Diameter:	**8 In.**	Surface Elevation:	**± 1507.3 Ft. AMSL per site plan**

Comments: Located at west end of warehouse structure.

DEPTH (ft.)	ELEVATION (MSL DATUM)	SAMPLE INTERVALS: TYPE, "N" or (Blows/ft.)	USCS	GEOTECHNICAL DESCRIPTION	DRY DENSITY (pcf)	WATER CONTENT (%)	OTHER TESTS
0			SM	Silty Sand: Yellowish brown; loose 0-1', becoming medium dense below; slightly moist; fine to coarse grained sand in very silty matrix (estimated 40% fines at top). Some clay. Does not have particularly strong pedogenic soil development -- interpreted slightly younger fan sediments over Paloma-age alluvium. [Very old alluvium]			BULK: MAX, EI, SHEAR, SULFATE, CHLORIDE, pH, RESISTIVITY
	1505	RING 11 14 19 (33)	SM/SC		122.6	5.9	CONSOL
5		RING 13 15 16 (31)	SM	← Silty sand, grades light brown, not high clay content and now slightly porous.	120.8	6.5	CONSOL
		RING 9 11 18 (29)	SM	← Silty sand, slightly cemented and cohesive, fine-grained, not visibly porous.	111.8	4.3	
	1500		SP-SM	Sand with Silt: Yellowish brown; medium dense; slightly moist; primarily medium to coarse grained and low cohesion with under 15% fines; not visibly porous. Basal erosional surface. [Very old alluvium]			
			SM	Silty Sand: Yellowish brown to strong brown; dense to very dense; slightly moist; mostly fine to medium grained; clay-cemented and with spotty MnO films; cohesive. Firm drilling. [Very old alluvium]			
10		RING 30 50/3"	SM		114.4	7.9	
	1495						
15							

Continued on next sheet.

FIELD LOG OF BORING B - 1

Sheet 2 of 2

Project: ***BLDG. 2, NATWAR LANE INDUSTRIAL PROJECT***

Location: ***CITY OF PERRIS, RIVERSIDE COUNTY, CALIF.***

DEPTH (ft.)	ELEVATION (MSL DATUM)	SAMPLE INTERVALS: BULK	SAMPLE INTERVALS: DRIVE	TYPE, "N" or (Blows/ft.)	GRAPHIC LOG	USCS	GEOTECHNICAL DESCRIPTION	DRY DENSITY (pcf)	WATER CONTENT (%)	WELL COMPLETION	OTHER TESTS
15				SPT 9 15 21 N=36		SM	Silty Sand: Yellowish brown to brown; dense; slightly moist; typically fine to medium grained sand in variably clayey and silty matrix; not visibly porous. Stratification more pronoiunced with depth. [Very old alluvium]				
	1490										
20				SPT 22 22 24 N=46		SP-SC, SP-SM	← Sand with silt and silty sand with clay, layered, former is low cohesion, medium to coarse-grained gritty immature textures with <10% fines.				
	1485										
25				SPT 12 15 17 N=32		SM, CL	← Silty sand, with sandy clay bed ~6" thick, generally cohsive and not visibly porous.				

Bottom of boring at 26.5 ft.
Perched groundwater stabilized at 23.8 ft.
Boring backfilled with compacted soil cuttings.

FIELD LOG OF BORING B - 2

Sheet 1 of 1

Project: ***BLDG. 2, NATWAR LANE INDUSTRIAL PROJECT***

Location: ***CITY OF PERRIS, RIVERSIDE COUNTY, CALIF.***

Date(s) Drilled:	**12/1/20**	Logged By:	**L. Arguello**
Drilled By:	**GP Drilling**	Total Depth:	**10.5 Ft.**
Rig Make/Model:	**Mobile B-61**	Hammer Type:	**Automatic trip**
Drilling Method:	**Hollow-Stem Auger**	Hammer Weight/Drop:	**140 Lb./30 In.**
Hole Diameter:	**8 In.**	Surface Elevation:	**± 1505.5 Ft. AMSL per site plan**

Comments: Truck & trailer drive aisle.

DEPTH (ft.)	ELEVATION (MSL DATUM)	SAMPLE INTERVALS: BULK	SAMPLE INTERVALS: DRIVE	TYPE, "N" or (Blows/ft.)	GRAPHIC LOG	USCS	GEOTECHNICAL DESCRIPTION	DRY DENSITY (pcf)	WATER CONTENT (%)	WELL COMPLETION	OTHER TESTS
0	1505					SM	Silty Sand: Yellowish brown; loose 0-1', becoming medium dense below; slightly moist; fine to coarse grained sand in very silty matrix (estimated 40% fines at top). Some clay. Interpreted slightly younger fan sediments over Paloma-age alluvium. [Very old alluvium]				
				RING 8 15 50 (65)		SM	← Silty sand, becomes dense and grades light brown, carbonate-cemented, trace of fine weathered gravel.	130.4	8.5		
5	1500			RING *50/6"		SM	Silty Sand: Yellowish brown; very dense; slightly moist; generally fine to medium grained and slightly clayey, with carbonate cementation; friable. Top may be erosional contact. [Very old alluvium].	101.5	8.0		
				RING *50/6"		SM		115.3	6.9		
10	1495			RING 26 41 50/5"		SM	← Silty sand, as above.	132.7	7.4		

Bottom of boring at 10.5 ft.
No groundwater encountered.
Boring backfilled with compacted soil cuttings.

FIELD LOG OF BORING B - 3

Project:	*BLDG. 2, NATWAR LANE INDUSTRIAL PROJECT*
Location:	*CITY OF PERRIS, RIVERSIDE COUNTY, CALIF.*

Date(s) Drilled:	**12/1/20**	Logged By:	**L. Arguello**
Drilled By:	**GP Drilling**	Total Depth:	**26.5 Ft.**
Rig Make/Model:	**Mobile B-61**	Hammer Type:	**Automatic trip**
Drilling Method:	**Hollow-Stem Auger**	Hammer Weight/Drop:	**140 Lb./30 In.**
Hole Diameter:	**8 In.**	Surface Elevation:	**± 1503.2 Ft. AMSL per site plan**

Comments: Exploration boring at proposed BMP site (basin or chamber array).

DEPTH (ft.)	ELEVATION (MSL DATUM)	SAMPLE INTERVALS: BULK	DRIVE	TYPE, "N" or (Blows/ft.)	GRAPHIC LOG	USCS	GEOTECHNICAL DESCRIPTION	DRY DENSITY (pcf)	WATER CONTENT (%)	WELL COMPLETION	OTHER TESTS
0						SM	Silty Sand: Yellowish brown; loose 0-1', becoming medium dense below; slightly moist; fine to coarse grained sand in very silty matrix (estimated 40% fines at top); massive. Tilled and bioturbated at top 3 feet. Interpreted slightly younger fan sediments over Paloma-age alluvium. [Very old alluvium]				
	1500										
5			■	SPT 4 6 6 N=12		SM	← Silty sand, grades yellowish brown, uncemented, predominantly fine-grained.				
	1495						Drill rate slows; interpreted contact.				
						SM	Silty Sand: Yellowish brown; very dense; slightly moist; fine to coarse-grained sand with weathered grains; slightly clayey, with carbonate cementation; not visibly porous. Top may be erosional contact. [Very old alluvium].				
10			■	SPT 19 32 46 N=78							
	1490										
15											

Continued on next sheet.

FIELD LOG OF BORING B - 3

Sheet 2 of 2

Project: ***BLDG. 2, NATWAR LANE INDUSTRIAL PROJECT***

Location: ***CITY OF PERRIS, RIVERSIDE COUNTY, CALIF.***

DEPTH (ft.)	ELEVATION (MSL DATUM)	SAMPLE INTERVALS: BULK	DRIVE	TYPE, "N" or (Blows/ft.)	GRAPHIC LOG	USCS	GEOTECHNICAL DESCRIPTION	DRY DENSITY (pcf)	WATER CONTENT (%)	WELL COMPLETION	OTHER TESTS
15				SPT 13 21 32 N=53		SM	Silty Sand: Dark yellowish brown; very dense; slightly moist; fine to coarse-grained sand with weathered grains; slightly clayey and cohesive; not visibly porous and not distinctly stratified. [Very old alluvium].				
	1485										
20				SPT 10 12 12 N=24		SM	← Silty sand, becomes medium dense, mostly fine to medium grained, massive, common MnO spots.				
	1480										
25				SPT 6 12 19 N=31		SM, SC	← Layered silty sand and clayey sand, continues mostly fine to medium grained.				

Bottom of boring at 26.5 ft.
Perched groundwater stabilized at 23.1 ft.
Boring backfilled with compacted soil cuttings.

FIELD LOG OF BORING B - 4

Project: ***BLDG. 2, NATWAR LANE INDUSTRIAL PROJECT***

Location: ***CITY OF PERRIS, RIVERSIDE COUNTY, CALIF.***

Date(s) Drilled:	**12/1/20**	Logged By:	**L. Arguello**
Drilled By:	**GP Drilling**	Total Depth:	**11.5 Ft.**
Rig Make/Model:	**Mobile B-61**	Hammer Type:	**Automatic trip**
Drilling Method:	**Hollow-Stem Auger**	Hammer Weight/Drop:	**140 Lb./30 In.**
Hole Diameter:	**8 In.**	Surface Elevation:	**± 1504.6 Ft. AMSL per site plan**

Comments: Located in NE corner of warehouse structure.

DEPTH (ft.)	ELEVATION (MSL DATUM)	SAMPLE INTERVALS: TYPE, "N" or (Blows/ft.)	GRAPHIC LOG	USCS	GEOTECHNICAL DESCRIPTION	DRY DENSITY (pcf)	WATER CONTENT (%)	WELL COMPLETION	OTHER TESTS
0				SM	Silty Sand: Yellowish brown; loose 0-1', becoming medium dense below; slightly moist; fine to coarse grained sand in very silty matrix (estimated 40% fines at top); massive. Tilled and bioturbated at top 3 feet. Interpreted slightly younger fan sediments over Paloma-age alluvium. [Very old alluvium]				
		RING 7 8 12 (20)		SM	⟵ Silty sand, grades yellowish brown, with trace of clay and cohesive.	116.5	2.6		
5	1500	RING 16 18 20 (38)		SM	⟵ Silty sand, slightly porous,	120.0	6.3		
		RING 13 17 22 (39)		SM	⟵ Silty sand, becomes brown and stronger cohesion with traces of clay, massive.	124.0	11.0		
10	1495	RING 11 25 50/5"		SM	Silty Sand: Dark yellowish brown; very dense; slightly moist; mostly fine to medium-grained sand with weathered grains; slightly clayey and cemented; not visibly porous. Top may be erosional contact. [Very old alluvium].	126.9	10.1		

Bottom of boring at 11.5 ft.
No groundwater encountered.
Boring backfilled with compacted soil cuttings.

APPENDIX B

A P P E N D I X B

LABORATORY TESTING

Water Content - Dry Density Determinations – ASTM D2216-10

The dry unit weight and field moisture content were determined for each of the recovered barrel samples. The moisture-density information provides a gross indication of soil consistency and can assist in delineating local variations. The information can also be used to correlate soils and define units between individual exploration locations on the project site, as well as with units found on other sites in the general area.

Measured dry densities ranged from approximately 101.5 to 132.7 pounds per cubic foot. Water contents in ring samples ranged from 2.6 to 11.0 percent of dry unit weight. Sample locations and the corresponding test results are illustrated on the Field Boring Logs.

Modified Effort Compaction Test – ASTM D1557-12

One bulk soil sample was collected from a locality inside the planned building envelope. The representative future fill material was tested to determine the maximum dry density and optimum water content per the Method B procedure in the noted ASTM standard. The test method uses 25 blows of a 10-pound hammer falling 18 inches on each of 5 soil layers in a 1/30 cubic foot cylinder. Soil samples were prepared at varying moisture contents to create a curve illustrating achieved dry density as a function of water content. The test results are listed below and shown graphically on page B-4.

Maximum Density - Optimum Water Content Determination

Soil Description	Location	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
Silty Sand (SM), some clay [Very old alluvium]	B - 1 @ 0 - 4 ft.	134.0	8.0

Shear Strength Tests – ASTM D3080-11

Direct shear tests were performed on soils prepared to represent future compacted fill derived from surficial native site alluvium. We expect mass grading operations should produce soil masses with roughly equivalent strengths. "Fill" test samples were remolded to approximately 90 percent of the maximum dry density, at optimum water content as determined from a compaction test. All samples were initially saturated, consolidated and drained of excess moisture, and tested in a direct shear machine of the strain control type. Test samples are initially prepared and/or retained within standard one-inch-high brass *rings. Samples were tested at increasing normal loads to determine the Mohr-Coulomb shear strength parameters illustrated on page B-5. Peak and ultimate shear strength values are illustrated on the plot.*

Expansion Index Test – ASTM D4829-11

A laboratory clay expansion test of typical near-surface materials expected to be incorporated into structural compacted fill was performed in general accordance with the 1994 Uniform Building Code Standard 18-2 and subsequent modern ASTM adoption. A remolded sample is compacted in two layers in a 4-inch I.D. mold to a total compacted thickness of about 1.0 inch, using a 5.5-pound hammer falling 12 inches at 15 blows per layer. The sample is initially at a saturation between 49 and 51 percent. After remolding, the sample is confined under a normal load of 144 pounds per square foot and allowed to soak for 24 hours. The resulting volume change due to increase in moisture content within the sample is recorded and the Expansion Index (EI) calculated.

Expansion Index Test Results

Soil Description	Location	Expansion Index	Expansion Classification
Silty Sand (SM), some clay [Very old alluvium]	B - 1 @ 0 - 4 ft.	56	Medium

Consolidation Tests – ASTM D2435M-11

Natural alluvium was checked for collapse susceptibility and overall compressibility within predicted removal intervals and in probable competent materials. A series of cumulative vertical loads are applied to a small, laterally confined soil sample. The apparatus is designed to accept a one-inch-high brass ring containing an undisturbed or remolded soil sample. During each load increment, vertical compression (consolidation) of the sample is measured and recorded at selected time intervals. Porous stones are placed in contact with both sides of the specimen to permit the ready addition or release of water. Undisturbed samples are initially at field moisture content, and are subsequently inundated to determine soil behavior under saturated conditions. The test results are plotted graphically on pages B-6 and B-7.

Soil Corrosivity

A soil sample representative of future mass-graded fill in future contact with concrete or ferrous metals was tested in the laboratories of Project X Corrosion Engineers, Murrieta, California, to determine the tabulated data on the next page. The submitted soil sample was tested in general accordance with ASTM Standard Methods listed at the top of the table. Soluble-species quantitative determinations were based on 1:3 water-to-soil extracts.

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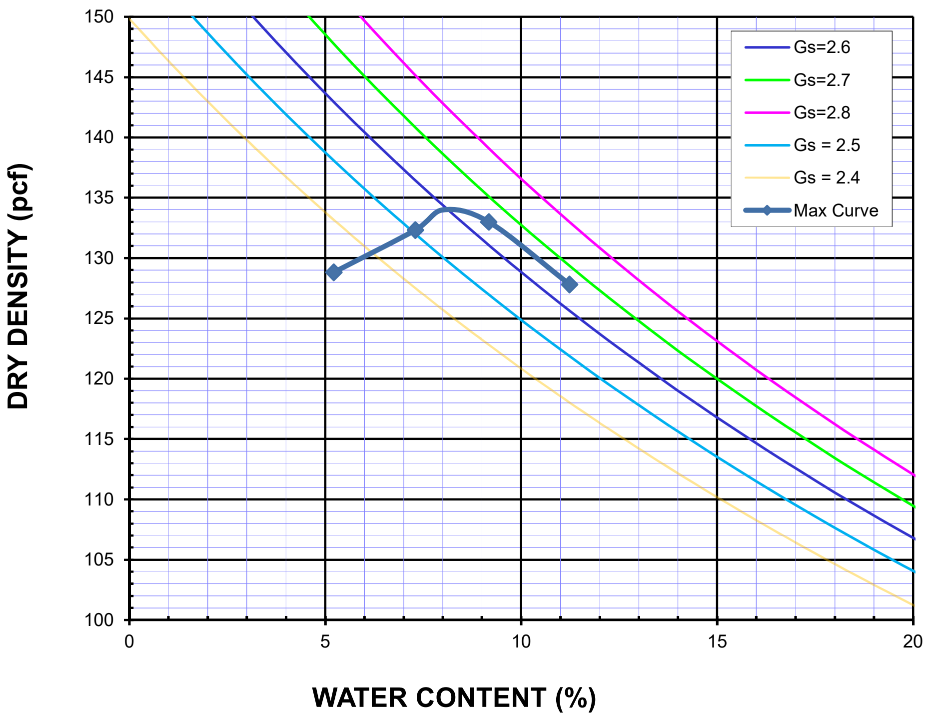
Maximum Density Test

Client:	First Industrial Realty Trust, Inc. 898 N. Pacific Coast Highway, Suite 175 El Segundo, CA 90245	**Project Name:**	Natwar Building 2 City of Perris, California
Project No.:	4673-SFLI	**Report Date:**	December 15, 2020
Sampled By:	Luis Arguello III	**Lab ID No.:**	20-2492
Date of Sampling:	December 1, 2020		

Information provided by Technician

X	Performed at Laboratory	X	Moist Preparation
	Performed at Jobsite		Dry Preparation

Tested By:	Cesar Lopez	**Date Tested:**	December 3, 2020
Sample Location:	B-1	**Source:**	Native — **Depth/Elev:** 0 - 4 ft
Sample Description:	Silty sand (SM) fine to coarse grained. [Very old alluvium]		



B	METHOD USED (A,B or C)
3/8-in.	SIEVE NUMBER
Mechanical	TYPE OF RAMMER
5.2%	AS REC'D MOISTURE
0.0%	PERCENT RETAINED
2.65	ASSUMED SPECIFIC GRAVITY
134.0	**MAXIMUM DENSITY [PCF]**
8.0	**OPTIMUM MOISTURE [%]**
-	**CORRECTED MAXIMUM DENSITY [PCF]**
-	**CORRECTED OPTIMUM MOISTURE [%]**

Remarks: No modifications made to test method, followed exact test procedure.

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AASHTO/ASTM/CTM Standards Used:	Unless noted, material was sampled in accordance with AASHTO T2/ASTM D75/CTM 125. Sample tested in accordance with ASTM D2216, D1557 & D4718.

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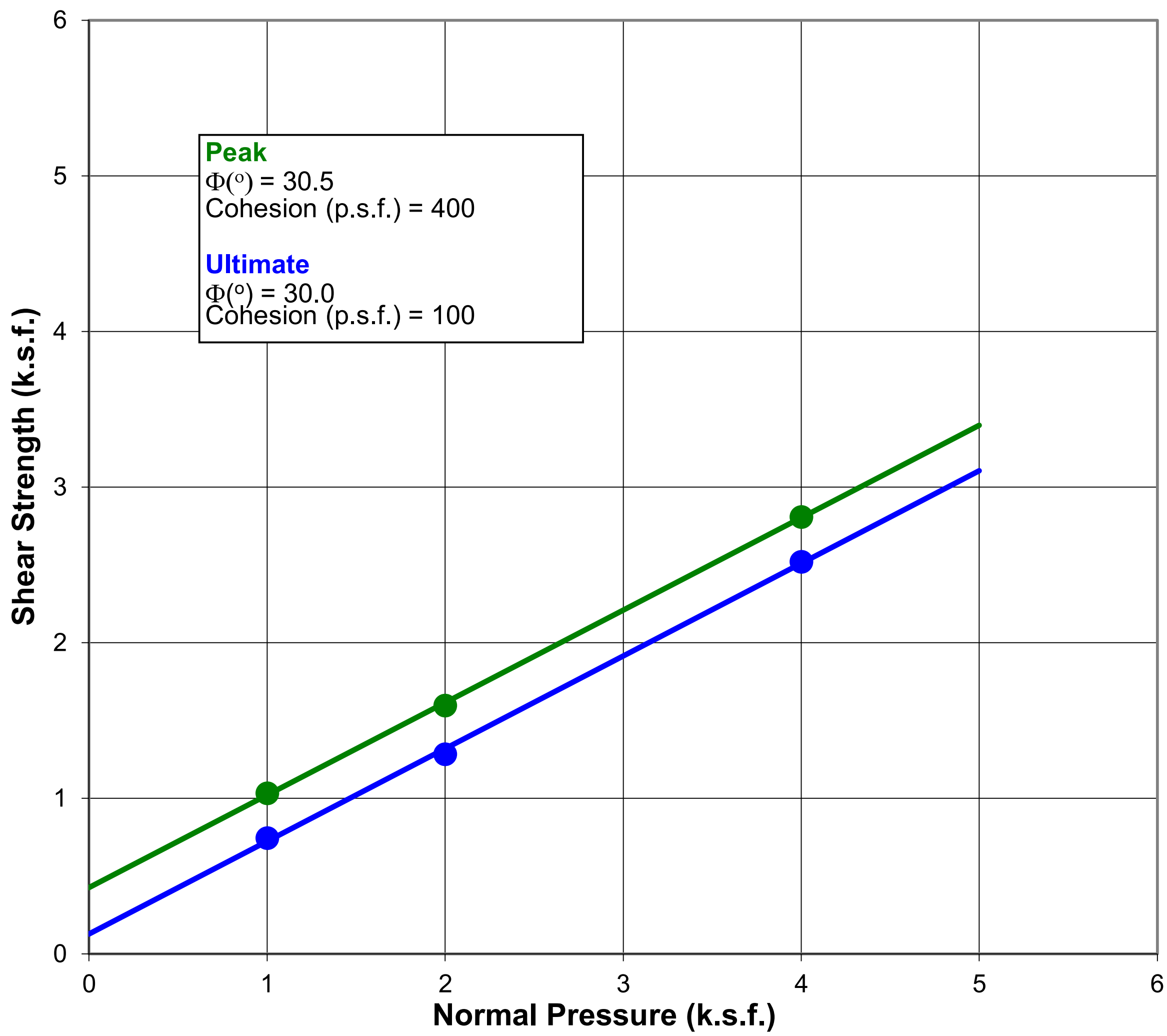
Direct Shear Test Diagram

Project Name:	Natwar Building 2, City of Perris, California		
Project Number:	4673-SFLI	Tested by:	Cesar Lopez
Sample Location:	B-1	Date Tested:	December 15, 2020
Sampled by:	Luis Arguello III	Depth (ft):	0.0 - 4.0
Date Sampled:	December 1, 2020	Lab I.D. No.:	20-2492
Test Condition:	Remolded, Consolidated, Drained.		
Sample Description:	Silty sand (SM), fine to coarse grained. [Very old alluvium]		



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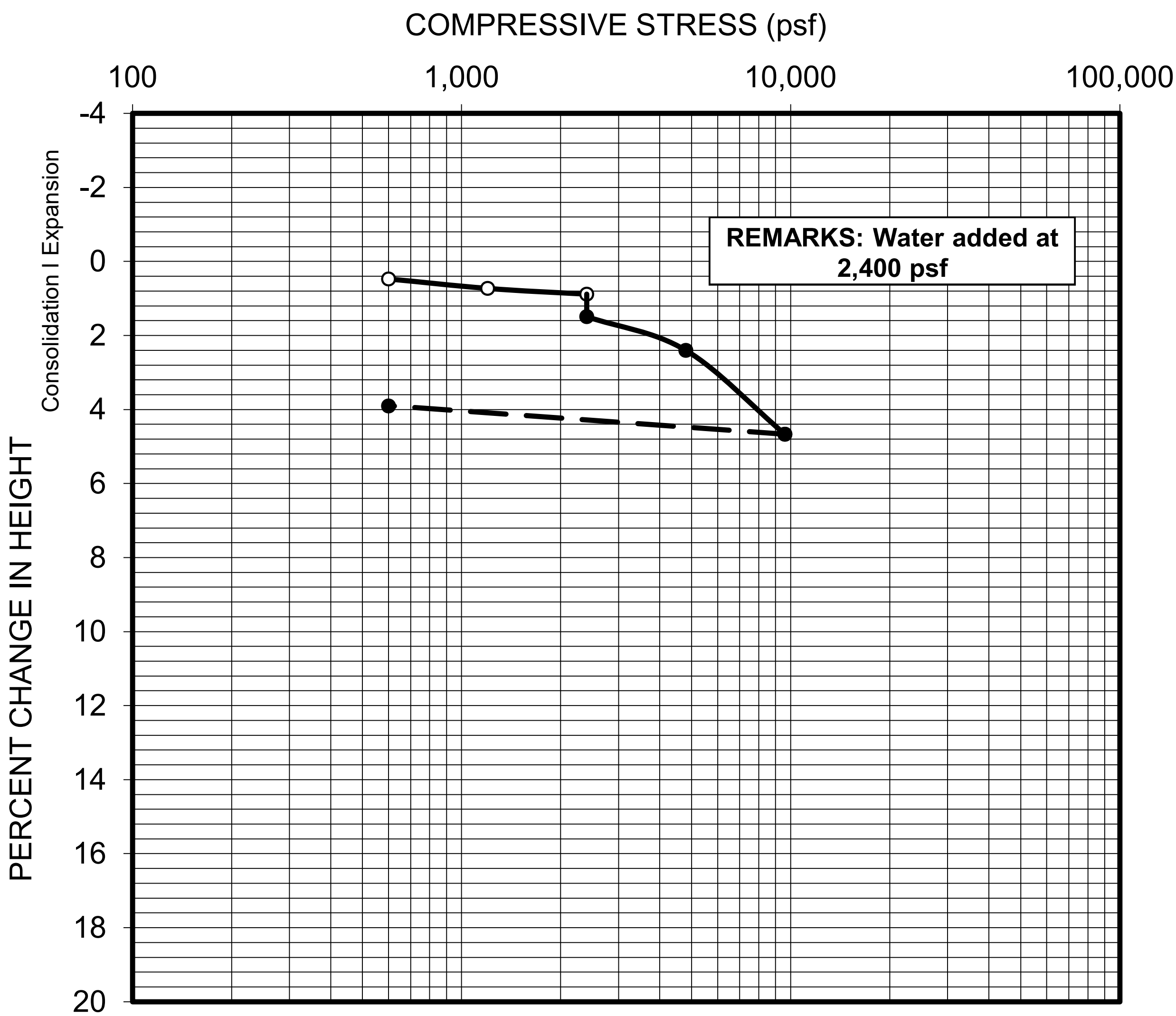
Consolidation Curve

Project Name:	Natwar Building 2, City of Perris, California		
Project Number:	4673-SFLI	Tested by:	Cesar Lopez
Sample Location:	B-1	Date Tested:	December 7, 2020
Sampled by:	Luis Arguello III	Depth (ft):	2.0
Date Sampled:	December 1, 2020	Moisture %:	5.9
Dry Density (pcf):	122.6	Saturation %:	42.5
Sample Description:	Silty sand (SM), fine to coarse grained. [Very old alluvium]		



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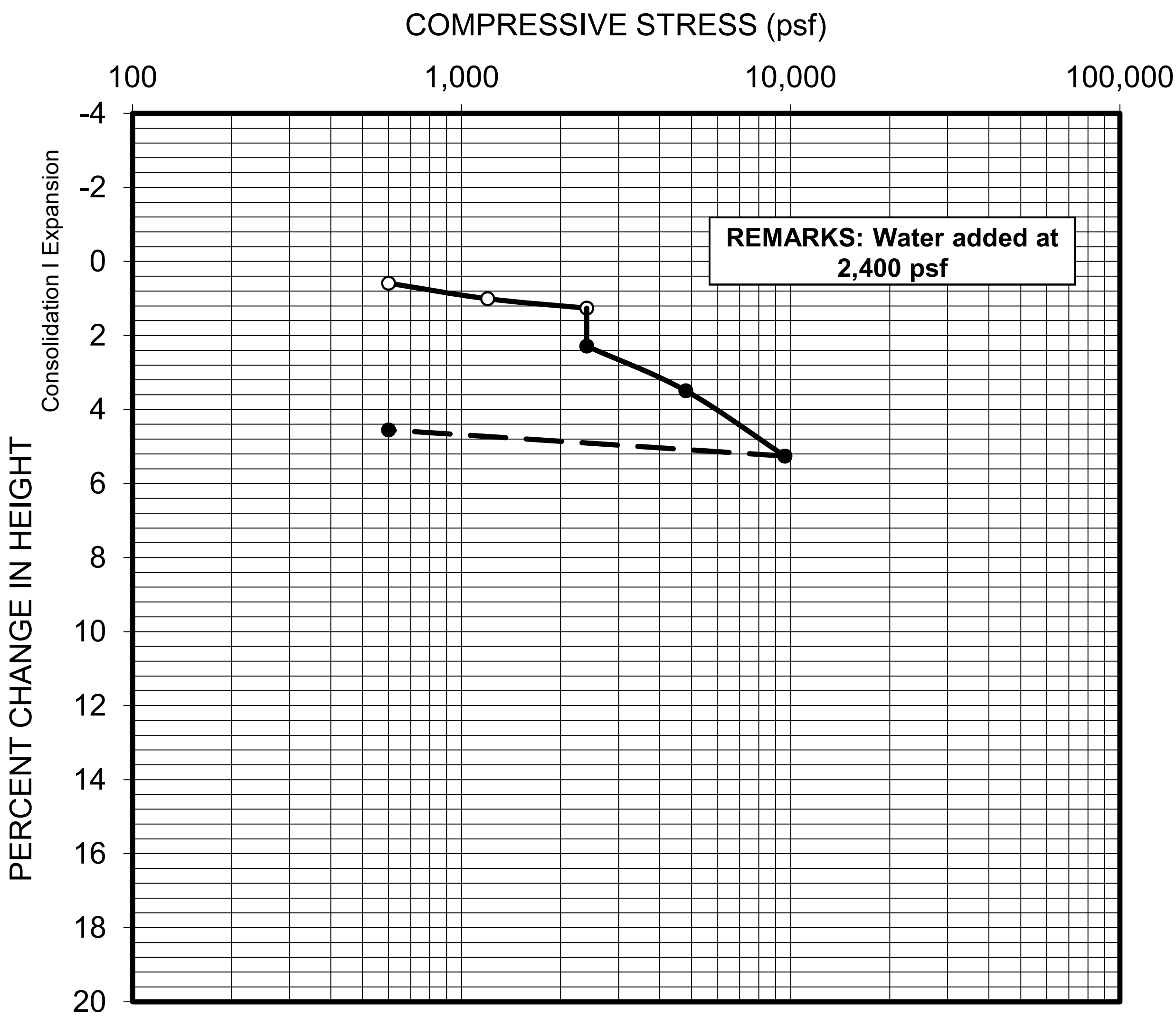
Consolidation Curve

Project Name:	Natwar Building 2, City of Perris, California		
Project Number:	4673-SFLI	Tested by:	Cesar Lopez
Sample Location:	B-1	Date Tested:	December 7, 2020
Sampled by:	Luis Arguello III	Depth (ft):	4.0
Date Sampled:	December 1, 2020	Moisture %:	6.5
Dry Density (pcf):	120.8	Saturation %:	44.4
Sample Description:	Silty sand (SM), fine to coarse grained, some clay. [Very old alluvium]		



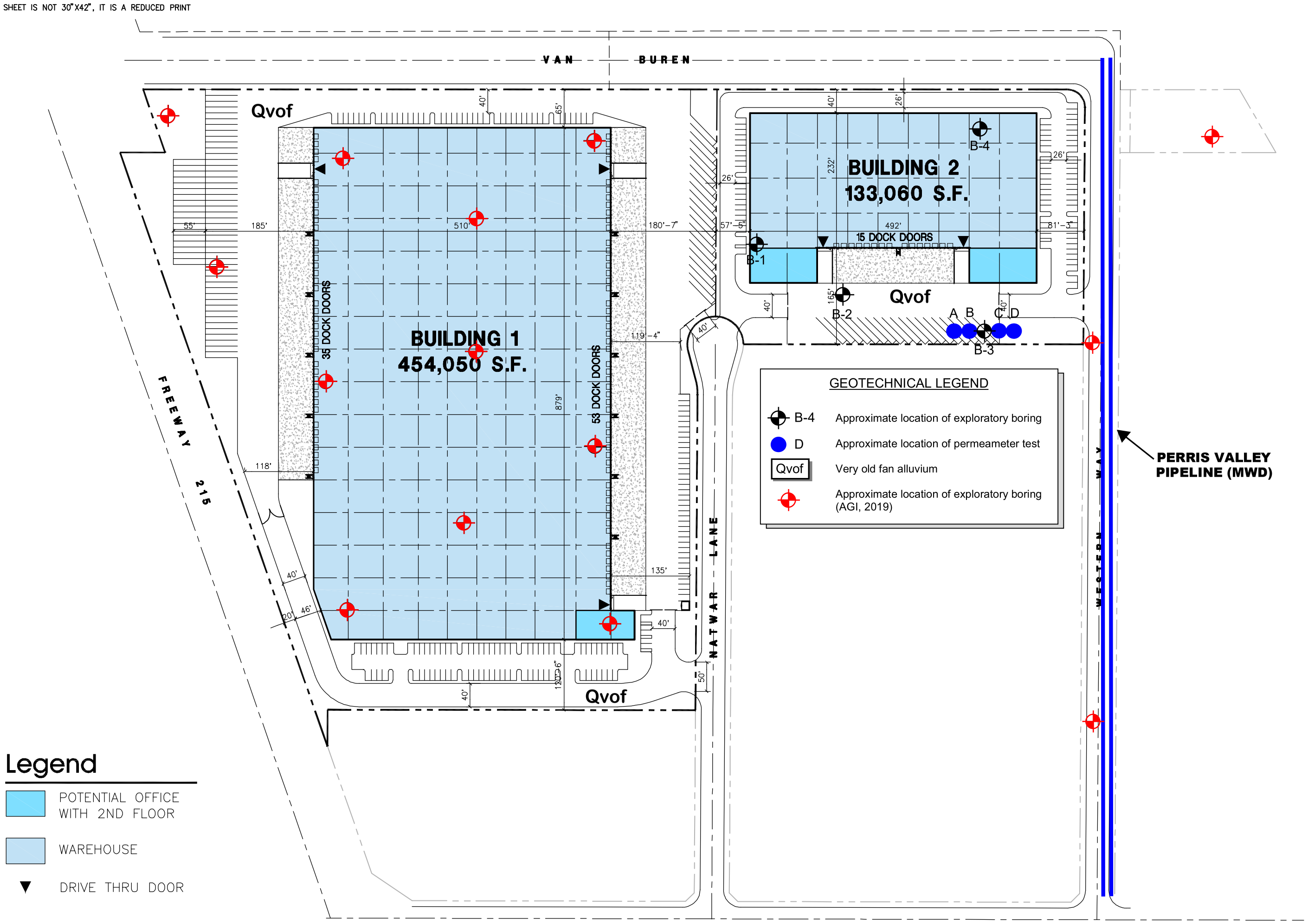
Soil Analysis Lab Results

Client: Aragon Geotechnical, Inc.
Job Name: First-Industrial
Client Job Number: 4673-SFLI
Project X Job Number: S201207A
December 9, 2020

	Method	ASTM D4327		ASTM D4327		ASTM G187		ASTM D4972
Bore# / Description	**Depth**	**Sulfates** SO_4^{2-}		**Chlorides** Cl^-		**Resistivity** As Rec'd | Minimum		**pH**
	(ft)	(mg/kg)	(wt%)	(mg/kg)	(wt%)	(Ohm-cm)	(Ohm-cm)	
20-2492 B-1	0-2	70.5	0.0071	47.6	0.0048	>737,000	4,020	8.1

Cations and Anions, except Sulfide and Bicarbonate, tested with Ion Chromatography
mg/kg = milligrams per kilogram (parts per million) of dry soil weight
ND = 0 = Not Detected | NT = Not Tested | Unk = Unknown
Chemical Analysis performed on 1:3 Soil-To-Water extract

CAUTION: IF THIS SHEET IS NOT 30"X42", IT IS A REDUCED PRINT



Legend

- POTENTIAL OFFICE WITH 2ND FLOOR
- WAREHOUSE
- ▼ DRIVE THRU DOOR

Note: This is a conceptual plan. It is based on preliminary information which is not fully verified and may be incomplete. It is meant as a comparative aid in examining alternate development strategies and any quantities indicated are subject to revision as more reliable information becomes available.

Aerial Map

Tabulation

	BLDG.1	BLDG.2	TOTAL
SITE AREA			
in sq. ft.	899,036	273,655	1,172,691 s.f.
in acres	20.6	6.3	26.9 ac
BUILDING AREA			
Office 1st floor	5,000	5,000	10,000 s.f.
Office 2nd floor	5,000	5,000	10,000 s.f.
Warehouse	444,050	123,060	567,110 s.f.
TOTAL	454,050	133,060	587,110 s.f.
COVERAGE	50.5%	48.6%	50.1%
AUTO PARKING REQUIRED			
1st 20K @ 1/1,000 sf	20	20	40 stalls
2nd 20K @ 1/2,000 sf	10	10	20 stalls
Over 40K @ 1/5,000 sf	83	19	102 stalls
TOTAL	113	49	162 stalls
AUTO PARKING PROVIDED			
standard (9'x19')	160	74	234 stalls
TRAILER PARKING PROVIDED			
trailer (12' x 55')	84	19	103 stalls
Zoning Ordinance for City			
Zoning Designation - Perris Valley Commercial Center SP (PVCC-SP) - Light Industrial			
MAXIMUM FLOOR AREA RATIO			
F.A.R. - .75			
MAXIMUM LOT COVERAGE			
Coverage - 50%			
SETBACKS			
Front Yard / Street side	Side Yard		
Local / Collector St. - 10'	Adjoining non-residential - 0'		
Arterials - 15'	Adjoining residential - 20'		
Expressway/Freeway - 20'			
Rear Yard			
Adjoining non- residential - 0'			
Adjoining residential - 20'			
LANDSCAPE REQUIRED			
Percentage	12%		
LANDSCAPE REQUIRED			
Percentage (base on net)	10.1%	12.7%	10.7%
in sq. ft.	91,050	34,867	125,917 s.f.

HPA architecture

AGI	GEOTECHNICAL MAP		
	BUILDING 2, NATWAR LANE INDUSTRIAL PROJECT, PERRIS, CALIFORNIA		
	PROJECT NO. 4673-SFLI	DATE: 12/21/20	PLATE NO. 1

Conceptual Site Plan

Freeway 215 & Natwar Lane

City of Perris, CA

FIRST INDUSTRIAL REALTY TRUST

18831 Bardeen Ave. - Ste. #100
Irvine, CA 92612
(949) 863-1770
www.hparchs.com

October 12, 2020/ Job #19100

Scheme 12