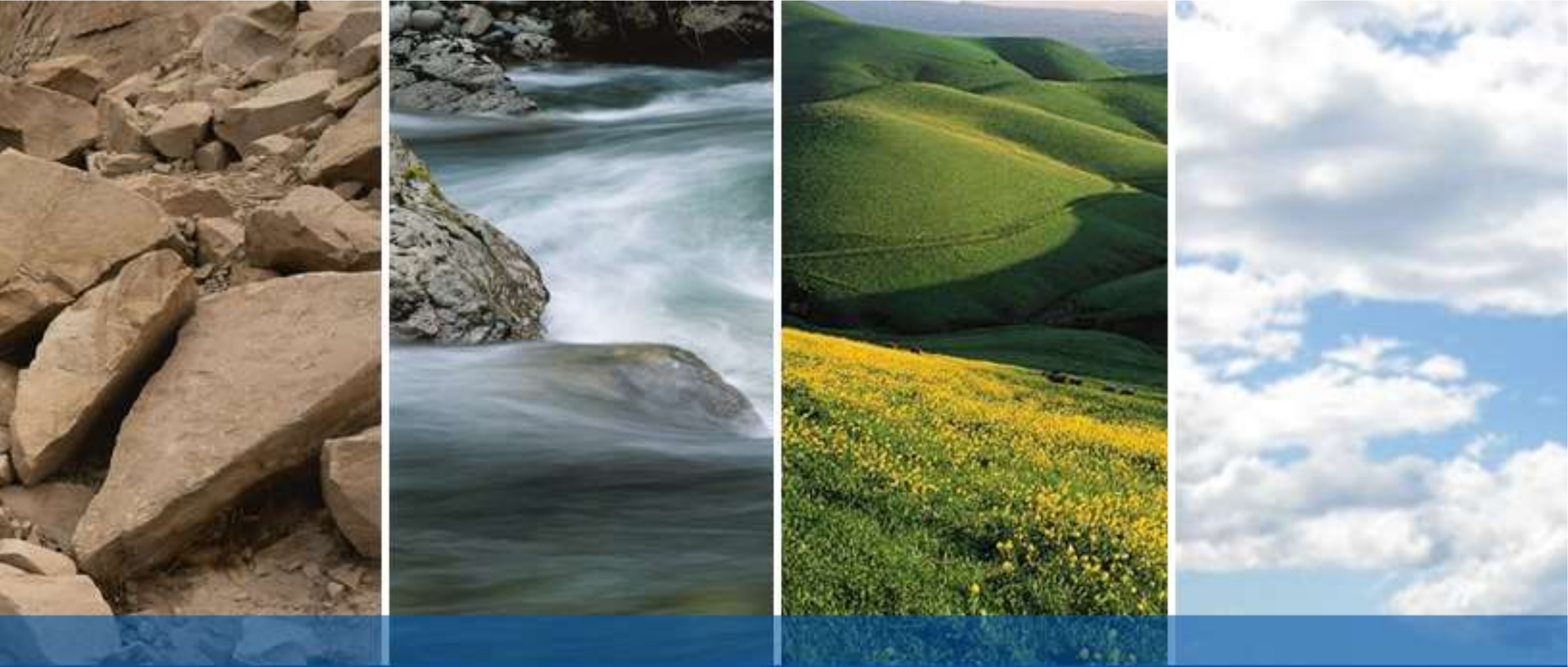


420 EAST 3RD STREET DEVELOPMENT
PITTSBURG, CALIFORNIA

GEOTECHNICAL FEASIBILITY EVALUATION

SUBMITTED TO
Mr. Kevin Fryer
IL Housing Solutions, LLC
888 San Clemente Drive, Suite 100
Newport Beach, CA 92660

PREPARED BY
ENGEO Incorporated

April 1, 2021

PROJECT NO.
18562.000.001

ENGEO
Expect Excellence

Project No.
18562.000.001

April 1, 2020

Mr. Kevin Fryer
IL Housing Solutions, LLC
888 San Clemente Drive, Suite 100
Newport Beach, CA 92660

Subject: 420 East 3rd Street Development
Pittsburg, California

GEOTECHNICAL FEASIBILITY EVALUATION

Dear Mr. Fryer:

We are pleased to present this geotechnical feasibility evaluation of the 420 East 3rd Street site in Pittsburg, California. The accompanying report presents our findings and preliminary recommendations regarding the proposed development.

Based on the findings of our feasibility evaluation, we opine that the proposed development is feasible from a geotechnical standpoint, provided the preliminary recommendations provided in this report are implemented during project planning. The scope of this report was limited to an initial study. A design-level exploration including laboratory testing and detailed engineering analyses should be conducted to develop design recommendations once building types and loading conditions are available.

We are pleased to have been of service on this project and are prepared to consult further with you and your design team as the project progresses. If you have any questions or comments regarding this preliminary report, please call and we will be glad to discuss them with you.

Sincerely,

ENGEO Incorporated

Alex Light, PE

REGISTERED PROFESSIONAL ENGINEER
ALEXANDER PATRICK LIGHT
No. 92370
CIVIL
STATE OF CALIFORNIA

Jeff Fippin GE

REGISTERED PROFESSIONAL ENGINEER
JEFFREY A. FIPPIN
No. 2631
GEOTECHNICAL
STATE OF CALIFORNIA

al/jaf/cjn

TABLE OF CONTENTS

1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

We prepared this report to evaluate the geotechnical feasibility of the proposed residential project at the site in Pittsburg, California. We were authorized to perform the following scope of services.

- Evaluation of suitability of the site for the proposed development.
- Preliminary assessment of geological hazards at the site and in the general project area.
- Discussion of potential geotechnical constraints such as loose/soft surface soil, existing fill, compressible soil, expansive soil, liquefiable soil, and lateral spreading, as necessary.
- Presentation of conceptual measures to mitigate hazards, geotechnical constraints, as appropriate.
- Discussion of anticipated foundation types and California Building Code (CBC) seismic design criteria.
- Preliminary pavement recommendations for hot mix asphalt and Portland cement concrete.

We were engaged to perform this evaluation based on existing data. No project-specific subsurface exploration was performed in support of this evaluation.

For our use in preparation of this report, we received the following:

1. Treadwell and Rollo; Preliminary Geotechnical Investigation, Proposed Harbor Bay Development, Pittsburg, California; October 17, 2005; Project No. 4245.01.
2. Treadwell & Rollo; Geotechnical Investigation, Proposed Harbor Park Development, Pittsburg, California; June 13, 2006; Project No. 4245.02.
3. Treadwell & Rollo; Geotechnical Consultation, Selected Approach for Remedial Mass Grading and Revised Earthen Embankment Construction, Harbor Park Development, Pittsburg, California; July 31, 2007; Project No. 4245.02.
4. Treadwell & Rollo; Final Report, Geotechnical Services during Remedial Mass Grading, The Proposed Hardbor Park Development, Pittsburg, California; July 17, 2008; Project No. 4245.02.
5. Sandis; Topographic Survey, 415 & 420 East 3rd Street, Pittsburg, California; March 13, 2017; Project No. 617005.
6. Rockridge Geotechnical; Geotechnical Investigation, Proposed New Campus & Sports Complex, Making Waves Academy, 959 E 3rd Street, Pittsburg, California; May 11, 2018, Project No. 18-1477.
7. Urban Arena; 420 E 3rd Street, Pittsburg, California; February 18, 2021; Project No. 20-067.

We prepared this report for the exclusive use of IL Housing Solutions, LLC and its consultants for the project. This document may not be reproduced in whole or in part by any means whatsoever, nor may it be quoted or excerpted without our express written consent.

1.2 SITE LOCATION AND PROPOSED DEVELOPMENT

The property at 420 East 3rd Street encompasses approximately 20.5 acres, identified as Assessor's Parcel Number (APN) 073-050-001-4. The site is located on the southwest side of the intersection of Harbor Street and East 3rd Street. Harbor Street and East 3rd Street border the north and east edges of the site, respectively, while the south side of the site is bordered by a landscaped area and an existing residential development borders the site to the west. The project site area lies downslope of the adjacent residential and landscaped areas; the slope appears to reach a maximum height of approximately 20 feet with a gradient of approximately 1½:1 (horizontal:vertical). The developable site envelope is relatively level, with an approximately 10-to-14-foot-high soil berm along the eastern boundary of the site adjacent to Harbor Street.

Based on the preliminary site plan, we understand that the future development will include approximately 236 lots for residential use. We anticipate that the development will additionally incorporate paved drive aisles and parking, underground utilities, secondary slabs on grade such as sidewalks, ancillary structures, landscaping, and stormwater basins.

Structural loads and grading are yet to be determined; however, we assume that structural loads will be consistent with similar construction.

2.0 FINDINGS

2.1 AERIAL PHOTOGRAPH INTERPRETATION

We reviewed historic aerial photographs and topographic maps available on www.historicaerials.com. We understand that the site was developed for industrial use in the 1920s. Based on our review of historic photographs, structures and railway lines associated with this work occupy the majority of the site and remained relatively unchanged between 1959 and 2005. By 2009, the existing structures and railways appear to have been demolished and the berm has been constructed in the eastern portion of the site. Conditions appear relatively unchanged since then.

2.2 EXISTING SITE CONDITIONS

We performed a reconnaissance of the site on March 29, 2021, to observe current site conditions. The site is currently vacant and is secured by a perimeter fence on all sides. The site consists of relatively level, vegetated open space.

PHOTO 2.2-1: Northern Area of Site (facing South)

PHOTO 2.2-2: Southern Area of Site (facing North)

2.3 REGIONAL GEOLOGY AND SEISMIC SETTING

The site is situated in the Coast Ranges geomorphic province of California, which is characterized by a series of parallel, northwesterly trending, folded and faulted mountain ranges and valleys. The site was originally tidal marshland prior to the current development. According to published geologic maps, the majority of the site is covered by existing fill deposits of variable thickness placed over alluvial deposits (Bibblee, 2006).

The site is not located within a currently designated Alquist-Priolo Earthquake Fault Zone and no known surface expression of active faults is believed to exist within the site. An active fault is defined by the State Mining and Geology Board as one that has had surface displacement within Holocene time (about the last 11,000 years) (Bryant and Hart, 2007).

Because of the presence of nearby active faults, the Bay Area Region is considered seismically active. Numerous small earthquakes occur every year in the region, and large (greater than Moment Magnitude 7) earthquakes have been recorded and can be expected to occur in the future. Based on the United States Geological Survey's (USGS) 2008 National Seismic Hazard Maps, the closest active faults in the area are the Great Valley fault and Green Valley fault, which are approximately 1.3 miles northwest and 9.5 miles west of the site, respectively.

2.4 PREVIOUS FIELD EXPLORATIONS

The site was explored in 2005 and again in 2006 by Treadwell and Rollo (T&R) with a total of 61 cone penetration tests (CPTs) and 13 drilled borings throughout the site. Locations of the borings and CPTs locations are presented in Figure 2.

The previous borings ranged in depth between 19 and 51½ feet and the CPTs ranged in depth between 30 and 50 feet. The exploration logs and associated laboratory testing results from the 2005 and 2006 exploration are included in Appendix A.

2.5 SUBSURFACE CONDITIONS

The 2006 geotechnical investigation encountered approximately 2 to 15 feet of man-made fill consisting of loose to medium dense sand and gravel with variable silt and clay throughout the site. In the southeastern portion of the site, an approximately 3 to 43-foot-thick layer of soft to medium stiff compressible clay was encountered below the fill. The approximate lateral extent of the compressible material is shown on Figure 2. Below the compressible soil (where encountered), the previous exploration penetrated medium stiff to stiff sandy silt and clay, medium dense to very dense sand, and dense to very dense sand and gravelly sand.

2.6 GROUNDWATER CONDITIONS

Groundwater measurements during previous explorations at the site generally encountered groundwater 4½ to 15½ feet below ground surface (bgs).

The Seismic Hazard Zone Report for the Honker Bay Point 7.5 Minute Quadrangle (CGS, 2019) does not provide a map of historical high groundwater; the document states that depths to groundwater are typically between 0 to 10 feet. Fluctuations in the level of groundwater are expected to occur due to the proximity of the site to the New York Slough, as well as variations in rainfall, irrigation practice, and other factors not in evidence at the time of the subsurface exploration.

2.7 PREVIOUS GEOTECHNICAL WORK

In addition to the site investigations performed in 2005 and 2006, we understand that T&R provided supplemental recommendations regarding to remedial site grading, earthen embankment construction and repair, surcharging, and wick drain installation.

The 2008 T&R report summarizes geotechnical testing and observation of select fill locations within the site. This work appears to have been completed in November and December of 2007. The report indicates elevation and relative compaction of engineered backfill placed within local "pond" excavations within the southeastern and northern portions of the site. Based on the topographic survey included with this report, it appears the site has been further graded sometime between 2008 and 2021 to level the site and remove stockpiles within the site.

The 2008 report indicates that utilities located within the site were removed and backfilled with engineered fill under the observation of T&R in 2006. Observations performed by T&R during wick drain installation are contained in a separate 2008 letter. These reports were not available for our review.

3.0 GEOLOGIC AND GEOTECHNICAL HAZARDS

3.1 SEISMIC HAZARDS

Potential seismic hazards resulting from a nearby moderate to major earthquake can generally be classified as primary and secondary. The primary effect is ground rupture, also called surface faulting. The common secondary seismic hazards include ground shaking and liquefaction. The following sections present a discussion of these and other hazards as they apply to the site. Based on topographic and lithologic data, the risk of regional subsidence or uplift, lurching, or landslides, is low to negligible at the site.

3.1.1 Ground Rupture

As previously noted, the site is not located within a designated Alquist-Priolo Earthquake Fault Zone and no known surface expression of active faults is believed to exist within the site; as such, the risk of fault rupture through the site is considered low.

3.1.2 Ground Shaking

Seismic design provisions of current building codes generally prescribe minimum lateral forces, applied statically to the structure, combined with the gravity forces of dead and live loads. The code-prescribed lateral forces are generally considered to be substantially smaller than the actual forces that would be associated with a major earthquake. Therefore, structures should be able to: (1) resist minor earthquakes without damage, (2) resist moderate earthquakes without structural damage but with some nonstructural damage, and (3) resist major earthquakes without collapse, but with some structural as well as nonstructural damage. Conformance to the current building code recommendations does not constitute any kind of guarantee that significant structural damage would not occur in the event of a maximum magnitude earthquake; however, it is reasonable to expect that a well-designed and well-constructed structure will not collapse or cause loss of life in a major earthquake (SEAOC, 1996). California Building Code (CBC, 2019) seismic design parameters are presented later in this report.

3.1.3 Liquefaction / Cyclic Softening

The site is located within a mapped State of California Seismic Hazard Zone (CGS, 2019) for areas that may be susceptible to liquefaction.

Soil liquefaction results from loss of strength during cyclic loading, such as imposed by earthquakes. The soil considered most susceptible to liquefaction is clean, loose, saturated, uniformly graded fine sand below the groundwater table. Empirical evidence indicates that loose silty sand is also potentially liquefiable. When seismic ground shaking occurs, the soil is subjected to cyclic shear stresses that can cause excess hydrostatic pressures to develop. If excess hydrostatic pressures exceed the effective confining stress from the overlying soil, it is said to have liquefied. If the sand consolidates or vents to the surface during and following liquefaction, ground settlement and surface deformation may occur. In addition to liquefaction of sandy materials, clayey soil can also undergo “cyclic-softening” or strength loss as a result of cyclic loading.

Liquefaction hazard maps are created using a variety of information, including regional geologic mapping, regional groundwater level and observances of historic liquefaction. They should not be considered site-specific predictors of liquefaction hazard but instead areas where regional information indicates that the potential for liquefaction is higher.

The existing subsurface data does not provide sufficient information for performing a rigorous liquefaction hazard evaluation. Analysis of boring and CPT data included in the 2006 T&R and 2018 Rockridge investigations indicate up to 5½ inches of liquefaction-induced settlement in localized areas. Based on our review of the existing data using preliminary 2019 CBC seismic parameters, we anticipate that an updated estimate of liquefaction-induced settlement based on the requirements of the building code will be greater.

Liquefaction potential and settlement analyses should be performed during design-level exploration using updated site-specific seismic parameters in accordance with the 2019 CBC. The design-level assessment should focus on identifying the lateral and vertical extent of isolated layers of potentially liquefiable clayey sand and gravel that were encountered below groundwater elevation at some of exploration locations. The cyclic-softening hazard for fine-grained materials, if this type of soil is encountered, should be evaluated using the criteria presented by Bray and Sancio (2006). The liquefaction assessment during design-level study should also consider groundwater depth based on proposed site grades, when known.

3.1.4 Liquefaction-Induced Surface Rupture

In order for liquefaction-induced ground failure to occur, the pore water pressure generated within the liquefied strata must exert a force sufficient to break through the overlying soil and vent to the surface resulting in sand boils or fissures. Based on our review of the available exploration data, the sandy materials susceptible to liquefaction are generally interlayered with clay and silt that may not liquefy. Based on the current data, the potentially liquefiable soil are overlain by a non-liquefiable cap sufficient to prevent surface rupture, however, further study is necessary to determine if the clayey and silty interbedded soil will liquefy. If future evaluation shows an insufficient thickness of non-liquefiable soil cap, building pads may require construction with engineered fill and reinforcement such as geogrid to reduce the risk of surface rupture such as sand boils from occurring.

3.1.5 Lateral Spreading

Lateral spreading is a failure within a nearly horizontal soil zone (possibly due to liquefaction) that causes the overlying soil mass to move toward a free face or down a gentle slope. Based on the site location and distance from New York Slough, we anticipate that the risk for lateral spreading is low, however this risk should be further evaluated as part of future design-level study considering liquefaction potential.

3.2 NON-ENGINEERED FILL

As discussed, we identified that there is likely artificial fill approximately 2 to 15 feet deep throughout the site. The fill generally consists of loose to medium dense sand and gravel with variable silt and clay. Based on the 2006 and 2008 reports, we understand the majority of the fill throughout the site was placed following demolition of the on-site structures and improvements and was done so without geotechnical engineering controls. Based on conversations with you, we understand that drilling borings and digging test pits to determine the extents and thickness of the artificial fill is not feasible due to environmental constraints. We may recommend removal, processing and replacement of this existing non-engineered fill if not restricted by environmental constraints; all construction debris and any other unsuitable material should be removed from the fill during processing. After removal and recompaction of the fill underneath the building, shallow foundations can be used for structural support. If existing fill is left in place in portions of the site that are being developed with walkways or other improvements that are not sensitive to settlement, on-going maintenance should be anticipated. Areas receiving fill during site grading included in the 2008 T&R report may be excluded from a fill over excavation recommendation.

3.3 COMPRESSIBLE SOIL

The 2005 and 2006 investigations encountered an approximately 3 to 43-foot-thick layer of soft to medium stiff, compressible clay in the southeastern portion of the site and at isolated areas in the northwestern portion of the site. Laboratory testing during previous exploration indicates soft clay in the southeastern portion of the site is normally to lightly overconsolidated. Structural loads and grading are yet to be determined; however, we anticipate that the compressible clay will experience moderate settlement under new fill loads if site grades are raised, with the magnitude of settlement depending on the building loads and height of fill.

Preliminarily, based on the previous laboratory testing and our experience on nearby projects, we anticipate settlement within the southwestern portion of the site approximately ½ inch per foot of additional fill may be possible; this settlement will primarily occur over up to 25 years after fill placement. Raising of site grades and construction of structures may result in several inches of settlement in areas underlain by compressible material. We recommend mitigating this settlement with a surcharge program with vertical wick drains and monitoring to confirm surcharge settlement is essentially complete before construction of the buildings.

Given the time passed since the construction of the earthen berm, we anticipate that the primary settlement of the underlying compressible material below the berm has been completed.

3.4 FLOODING

Federal Emergency Management Agency (FEMA) Flood Insurance Maps indicate that portions of the site are within a flood hazard area subject to inundation by the 0.2-percent-annual-chance flood event, or 1-percent-annual-chance flood event with average depth less than one foot or with drainage areas of less than one square mile. The Civil Engineer should review pertinent information relating to possible flood levels for the subject site based on final pad elevations and provide appropriate design measures for development of the project.

3.5 SOIL CORROSION POTENTIAL

One near-surface soil sample collected during the 2006 exploration was tested for corrosion potential. Based on the resistivity and chloride ion measurements, the soil samples are considered ‘highly corrosive’ with respect to corrosion of buried cast/ductile iron and steel structures according to the Caltrans Corrosion Guidelines (2018). Sulfate ion testing was not performed as a part of this investigation. We recommend performing additional testing during design-level study and retaining a corrosion consultant to provide specific long-term corrosion protection recommendations for buried metal and concrete pipes and foundations.

4.0 PLANNING CONSIDERATIONS

From a geotechnical engineering viewpoint, the site is suitable for the proposed development, provided the preliminary geotechnical considerations in this report are properly addressed in a design-level study. Based on our research and preliminary exploration, the main geotechnical concerns at the site include:

- Liquefaction potential and related secondary effects
- Shallow groundwater
- Compressible soil
- Existing fill
- Corrosive soil

In order to reduce the effects of the above geotechnical concerns, the foundations should be sufficiently stiff to move as a rigid units within tolerable differential movements. Foundation alternatives and combinations to be considered include structural mat foundation systems founded on ground improvement and deep foundation systems. We preliminarily discuss foundation considerations in the following sections.

4.1 2019 CBC SEISMIC DESIGN PARAMETERS

Based on the existing site data, we classified the site as Site Class E in accordance with the 2019 California Building Code (CBC). The 2019 CBC is based on the 2016 edition of the American Society of Civil Engineers document titled “Minimum Design Loads and Associated Criteria for Buildings and Other Structures” (ASCE 71-16).

We provide the 2019 CBC seismic design parameters in Table 4.1-1 below, which include design spectral response acceleration parameters based on the mapped Risk Targeted Maximum Considered Earthquake (MCE_R) spectral response acceleration parameters.

TABLE 4.1-1: 2019 CBC Seismic Design Parameters

PARAMETER	VALUE
Site Class	E
Mapped MCE_R Spectral Response Acceleration at Short Periods, S_S (g)	1.789
Mapped MCE_R Spectral Response Acceleration at 1-second Period, S_1 (g)	0.604
Site Coefficient, F_A	*
Site Coefficient, F_V	*
MCE_R Spectral Response Acceleration at Short Periods, S_{MS} (g)	*
MCE_R Spectral Response Acceleration at 1-second Period, S_{M1} (g)	*
Design Spectral Response Acceleration at Short Periods, S_{DS} (g)	*
Design Spectral Response Acceleration at 1-second Period, S_{D1} (g)	*
MCE_G Peak Ground Acceleration adjusted for Site Class effects, PGA_M (g)	0.821
Long-period transition-period, T_L (sec)	8

*Requires site-specific ground motion hazard analysis
Latitude: 38.030244 degrees, **Longitude:** -121.878868 degrees

As noted in Table 4.1-1, a ground motion hazard analysis needs to be performed in accordance with Section 11.4.8 of ASCE 7-16, for structures on Site Class E.

Based on our experience with similar developments at nearby sites, performing a Non-Ergodic Site Response Analysis may result in a reduction of ground motions. The reduction in turn can result in significant project cost savings. Assessing the benefit of performing the non-ergodic site response analysis versus the baseline site-specific hazard analysis will require collaboration with the Structural Engineer once the design is further developed. The analysis can be conducted during or after the design-level study.

4.2 FOUNDATIONS

The primary hazards at the site with regard to foundation design include compressible soft clay, liquefaction potential, and existing non-engineered fill. Other considerations that should be further assessed as to their impacts to foundations as part of the design-level study include shallow groundwater and corrosion potential.

Depending on the final site development concept and structural loads, we anticipate the proposed residential structures can be supported by a post-tensioned mat foundation; we recommend assuming building pads will comprise geogrid reinforced and engineered fill. Within areas of compressible soil deposits, we recommend surcharging to reduce the risk of static load settlement.

Preliminary recommendations based on the current concept plans are provided in the following sections.

4.2.1 Post-Tensioned Mat Foundation

We recommend that the proposed residential structures be supported on post-tensioned (PT) mat foundations bearing on competent soil or engineered fill. On a preliminary basis, we recommend assuming that PT mats are a minimum of 10 inches thick. The Structural Engineer should determine the actual PT mat thickness using geotechnical recommendations in a design-level geotechnical report.

PT mats are typically underlain by a moisture reduction system as recommended below. In addition, the building pad subgrade is typically moisture conditioned such that the subgrade soil is at a moisture content at least 3 to 5 percentage points above optimum immediately prior to foundation construction. The subgrade should not be allowed to dry prior to concrete placement.

Based on the results of liquefaction-induced settlement presented in the 2006 and 2018 investigation reports, PT slabs should be designed for up to 3 inches of post-liquefaction differential settlement. As discussed in Section 3.1.4, liquefaction-induced surface rupture will be further evaluated in future studies. For planning purposes, we recommend assuming mitigation of these hazards with construction of reinforced building pads comprising engineered fill and geogrid.

Future collaboration with the structural engineer is important to develop specific foundation recommendations once final structural details and loading are developed.

4.2.2 Slab Moisture Vapor Reduction

When buildings are constructed with a concrete slab-on-grade, including post-tensioned mats, water vapor from beneath the slab will migrate through the slab and into the building. This water vapor can be reduced but not stopped. Vapor transmission can negatively affect floor coverings and lead to increased moisture within a building. When water vapor migrating through the slab would be undesirable, we typically recommend a moisture retarder system to reduce, but not stop, water vapor transmission upward through the slab-on-grade. This generally involves installing a Class A vapor retarder membrane (ASTM E1745, latest edition). A layer of 4 inches of clean crushed rock may be provided below the concrete slab-on-grade floors. Lastly, we typically recommend a concrete water-cement ratio for slabs-on-grade of no more than 0.50, special inspections during concrete placement, and moist curing slabs for a minimum of 3 days (or other equivalent curing specified by the structural engineer).

4.2.3 Soft Soil Surcharge

Based on our understanding of the future structures on site, we estimate that buildings within the compressible soil deposits discussed in Section 3.3 may undergo several inches of static settlement following construction if not mitigated. To reduce the risk of static settlement within this area, we recommend installation of vertical wick drains through the compressible material and placement of a soil surcharge. For planning purposes, we estimate that wicks will be placed throughout the extents of the compressible soft clay deposits. We estimate a surcharge consisting of 5 feet fill above the final site grade will be sufficient to complete the primary settlement of underlying compressible soil. Based on our experience with similar projects, we anticipate the combination of wick drains spaced at 4 to 5 feet apart in addition to soil surcharge can reduce the time needed to complete primary settlement to approximately 4 to 6 months depending on thickness of compressible soil. Wick drain spacing, height of surcharge, and duration of settlement will be included in the design-level study.

5.0 PRELIMINARY PAVEMENT

5.1 FLEXIBLE PAVEMENT

Based on our experience with nearby developments, we assume an R-value of 5 for the site soil for preliminary pavement design. Using an assumed R-value of 5, and in accordance with the design methods contained in Topic 630 of Caltrans Highway Design Manual, we developed the preliminary flexible pavement sections presented in Table 5.1-1 below.

TABLE 5.1-1: Preliminary Pavement Sections

TRAFFIC INDEX	HOT MIX ASPHALT (inches)	CLASS 2 AGGREGATE BASE (inches)
5	3	10
6	3½	12½
7	4	15½

The above preliminary pavement sections are provided for estimating only. The Civil Engineer should determine the appropriate traffic indexes for parking areas, entry/exit drives, and fire/maintenance roads based on anticipated vehicle loading and frequencies.

5.2 RIGID PAVEMENT

Concrete pavement sections can be used to resist heavy loads and turning forces in areas such as fire lanes or trash enclosures. Final design of rigid pavement sections, and accompanying reinforcement, should be performed based on estimated traffic loads and frequencies. We recommend the following minimum design sections for rigid pavements.

- Use a minimum section of 6 inches of Portland Cement concrete over 6 inches of Caltrans Class 2 Aggregate Base.
- Concrete pavement should have a minimum 28-day compressive strength of 3,500 psi.
- Provide minimum control joint spacing in accordance with Portland Cement Association guidelines.

6.0 DESIGN-LEVEL GEOTECHNICAL REPORT

This report presents preliminary geotechnical findings, conclusions and recommendations intended for preliminary planning purposes only. A design-level geotechnical exploration and assessment should be performed when development plans are available. The design-level geotechnical report should further discuss topics presented in this report and address the following items.

- Additional field exploration and laboratory testing to support design-level recommendations
- Design-level analyses related to geologic and geotechnical hazards
- Site-specific seismic hazard analysis based on CBC 2019 (ASCE 7-16)
- Design recommendations for structural mat foundation systems and engineered building pads
- Design-level wick drain installation and soil surcharge recommendations
- Design-level earthwork and improvement design and construction recommendations

7.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

This report presents geotechnical recommendations for design of the improvements discussed in Section 1.2 for the 420 East 3rd Street project. If changes occur in the nature or design of the project, we should be allowed to review this report and provide additional recommendations, if any. It is the responsibility of the owner to transmit the information and recommendations of this report to the appropriate organizations or people involved in design of the project, including but not limited to developers, owners, buyers, architects, engineers, and designers. The conclusions and recommendations contained in this report are solely professional opinions and are valid for a period of no more than 2 years from the date of report issuance.

We strived to perform our professional services in accordance with generally accepted principles and practices currently employed in the area; no warranty is provided, either express or implied. There are risks of earth movement and property damages inherent in building on or with earth materials.

This report is based upon field and other conditions discovered at the time of report preparation. We developed this report with limited subsurface exploration data. No project-specific subsurface exploration was performed so actual subsurface conditions could vary from those assumed in this report. An exploration should be performed prior to site design. We recommend that the owner establish a contingency fund to cover such costs. If unexpected conditions are encountered, we must be notified immediately to review these conditions and provide additional and/or modified recommendations, as necessary.

Our services did not include excavation sloping or shoring, soil volume change factors, flood potential, or a geohazard exploration. In addition, our geotechnical exploration did not include work to determine the existence or extent of possible hazardous materials. If any hazardous materials are encountered during construction, the proper regulatory officials must be notified immediately.

This document must not be subject to unauthorized reuse, that is, reusing without our written authorization. Such authorization is essential because it requires us to evaluate the document's applicability given new circumstances, not the least of which is passage of time.

SELECTED REFERENCES

American Concrete Institute, (2014), Building Code Requirements for Structural Concrete (ACI 318-14).

Bray, J. D., & Sancio, R. B, (2006), "Assessment of the liquefaction susceptibility of fine-grained soils," Journal of Geotechnical and Geoenvironmental Engineering, 132(9), 1165-1177.

Bryant, W. and Hart, E. (2007). Special Publication 42, "Fault-Rupture Hazard Zones in California", Interim Revision 2007, California Department of Conservation.

California Building Code, 2019.

California Geological Survey; 2019, Seismic Hazard Zone Report, Seismic Hazard Zone Report for the Honker Bay 7.5-Minute Quadrangle, Contra Costa County County, California.

California Geologic Survey, 2008, Special Publication 117A, Guidelines for Evaluating and Mitigating Seismic Hazards in California.

Dibblee, 2006, Geologic Map of the Vine Hill & Honker Bay Quadrangles, Contra Costa and Solano Counties, California.

Historical Aerials, www.historicaerials.com.

Jennings, C.W. and Bryant, W.A., 2010, Fault Activity Map of California, State of California, Department of Conservation, California Geologic Survey.

Rockridge Geotechnical; Geotechnical Investigation, Proposed New Campus & Sports Complex, Making Waves Academy, 959 E 3rd Street, Pittsburg, California; May 11, 2018, Project No. 18-1477.

SEAOC, 1996, Recommended Lateral Force Requirements and Tentative Commentary.

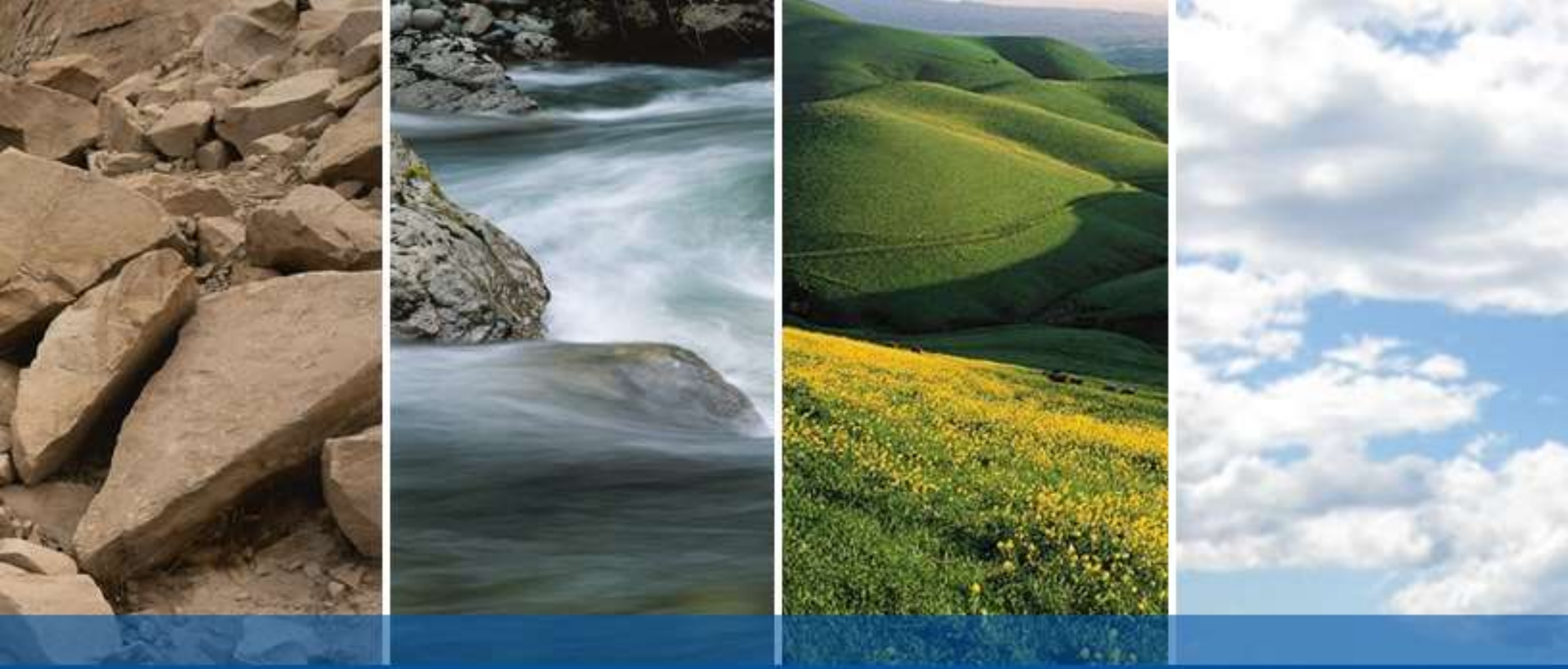
Treadwell and Rollo; Preliminary Geotechnical Investigation, Proposed Harbor Bay Development, Pittsburg, California; October 17, 2005; Project No. 4245.01.

Treadwell & Rollo; Geotechnical Investigation, Proposed Harbor Park Development, Pittsburg, California; June 13, 2006; Project No. 4245.02.

Treadwell & Rollo; Supplemental Geotechnical Consultation, Settlement for the Proposed Earthen Embankment, Proposed Harbor Park Development, Pittsburg, California; June 13, 2006; Project No. 4245.02.

Treadwell & Rollo; Geotechnical Consultation, Selected Approach for Remedial Mass Grading and Revised Earthen Embankment Construction, Harbor Park Development, Pittsburg, California; July 31, 2007; Project No. 4245.02.

Treadwell & Rollo; Final Report, Geotechnical Services during Remedial Mass Grading, The Proposed Hardbor Park Development, Pittsburg, California; July 17, 2008; Project No. 4245.02.



FIGURES

N

0 2000
FEET

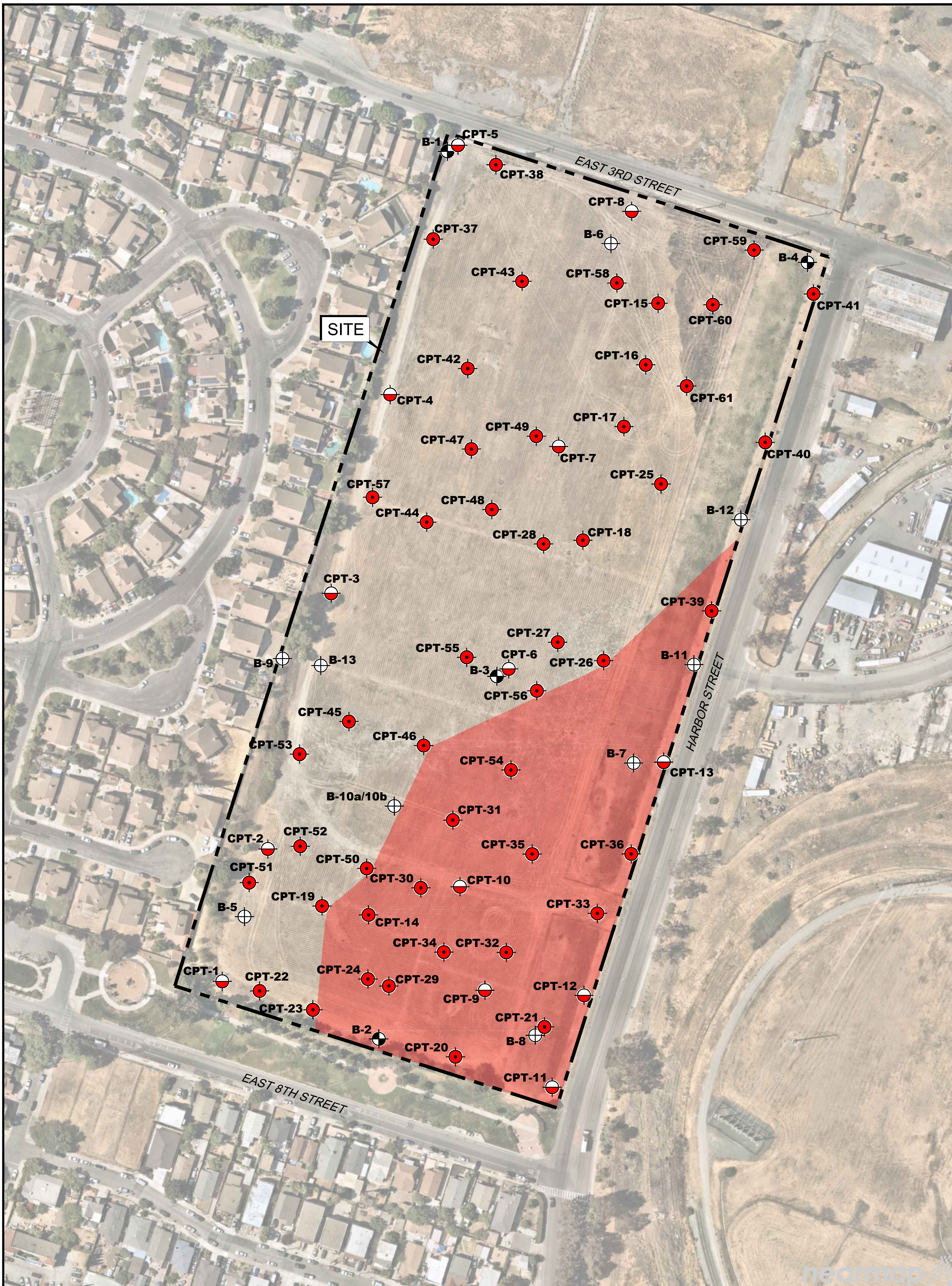
BASE MAP SOURCE: GOOGLE EARTH MAPPING SERVICE

ENGEO Expect Excellence	VICINITY MAP 420 EAST 3RD STREET DEVELOPMENT PITTSBURG, CALIFORNIA	PROJECT NO.: 18562.000.001 SCALE: AS SHOWN DRAWN BY: CC CHECKED BY: AL	FIGURE NO. 1

ORIGINAL FIGURE PRINTED IN COLOR

FILE PATH: G:\Drafting\PROJECTS_18000 to 19999\18562\18562000001\18562000001_FR_VM_1_0321.dwg SAVE DATE: 3/31/2021 12:07:43 PM SAVED BY: CCulbertson

FILE PATH: G:\Drafting\PROJECTS_18000 to 19999\18562\18562000001\18562000001_FR_SP_2_0321.dwg SAVE DATE: 3/31/2021 1:54:54 PM SAVED BY: CCulbertson



EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

EXTENTS OF COMPRESSIBLE MATERIAL

B-4 BORING (TREADWELL & ROLLO, INC., 8/06)

B-13 BORING (TREADWELL & ROLLO, INC., 12/05 AND 2/06)

CPT-13 CPT (TREADWELL & ROLLO, INC., 8/06)

CPT-61 CPT (TREADWELL & ROLLO, INC., 12/05 AND 2/06)

N

0 150 FEET

BASE MAP SOURCE: NEARMAP MAPPING SERVICE

ENGEO Expect Excellence

SITE PLAN
420 EAST 3RD STREET DEVELOPMENT
PITTSBURG, CALIFORNIA

PROJECT NO.: 18562.000.001	FIGURE NO. 2
SCALE: AS SHOWN	
DRAWN BY: CC	CHECKED BY: AL

ORIGINAL FIGURE PRINTED IN COLOR

EXPLANATION

GEOLOGIC CONTACT-DASHED WHERE GRADATIONAL OR APPROXIMATELY LOCATED

FAULT-DASHED WHERE INFERRED, DOTTED WHERE CONCEALED, QUERIED WHERE EXISTENCE IS DOUBTFUL. SAWTEETH ARE ON UPPER PLATE OF LOW ANGLE THRUST FAULT

Qa	ALLUVIUM
Qoa	OLDER ALLUVIUM
Qbm	BAY MUD

N

0 2000
FEET

BASE MAP SOURCE: DIBBLEE, 2006

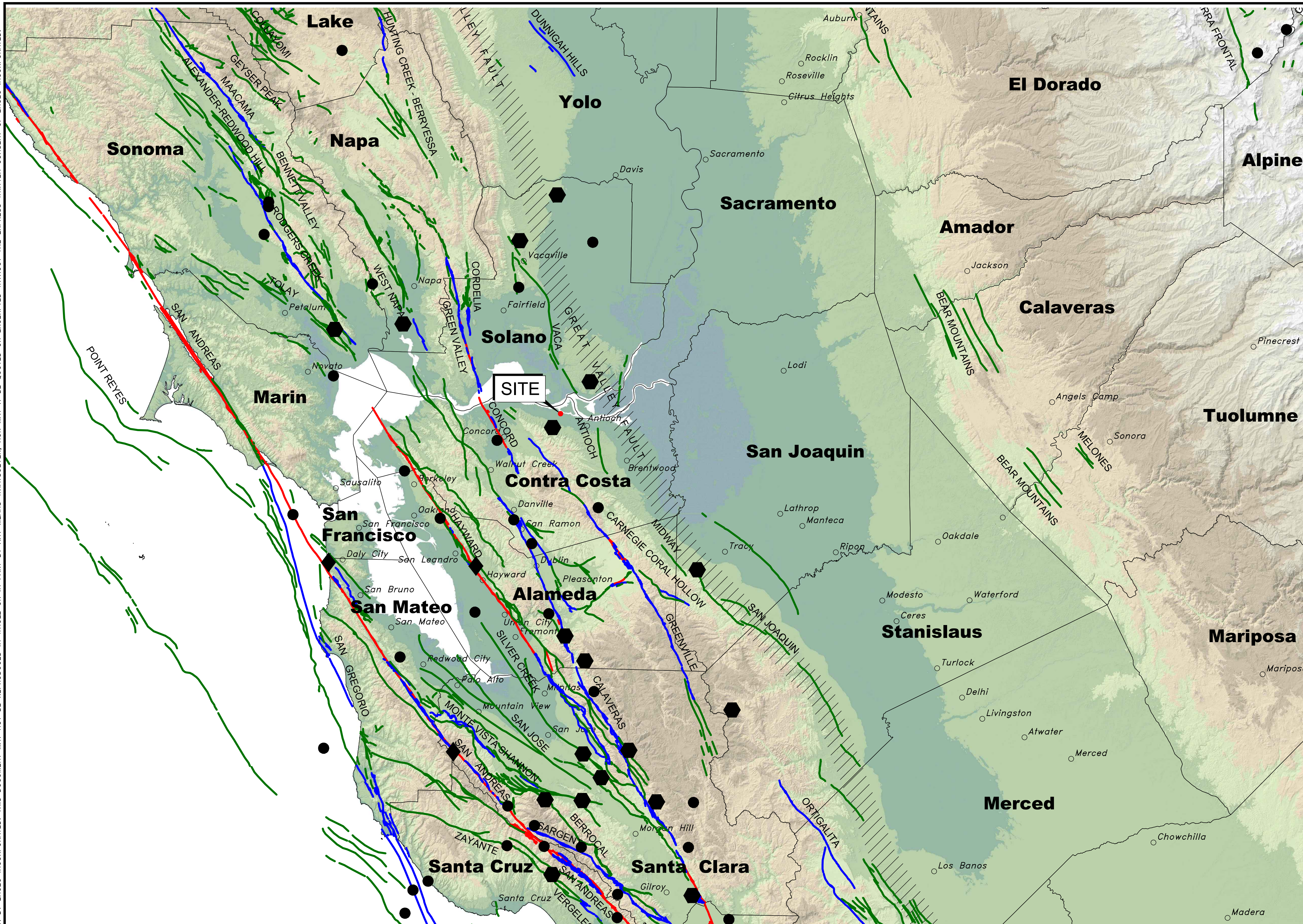
ENGEO
Expect Excellence

REGIONAL GEOLOGIC MAP
420 EAST 3RD STREET DEVELOPMENT
PITTSBURG, CALIFORNIA

PROJECT NO.: 18562.000.001		FIGURE NO. 3
SCALE: AS SHOWN		
DRAWN BY: CC	CHECKED BY: AL	

ORIGINAL FIGURE PRINTED IN COLOR

FILE PATH: G:\Drafting\PROJECTS_18000 to 19999\18562\185620000001\185620000001_FR_RGM_3_0321.dwg SAVE DATE: 3/31/2021 12:07:37 PM SAVED BY: CCulbertson



N

0 MILES 15
0 KILOMETERS 30

EXPLANATION

- MAGNITUDE 7+
- MAGNITUDE 6-7
- MAGNITUDE 5-6
- HISTORIC FAULT
- HOLOCENE FAULT
- QUATERNARY FAULT
- HISTORIC BLIND THRUST FAULT ZONE

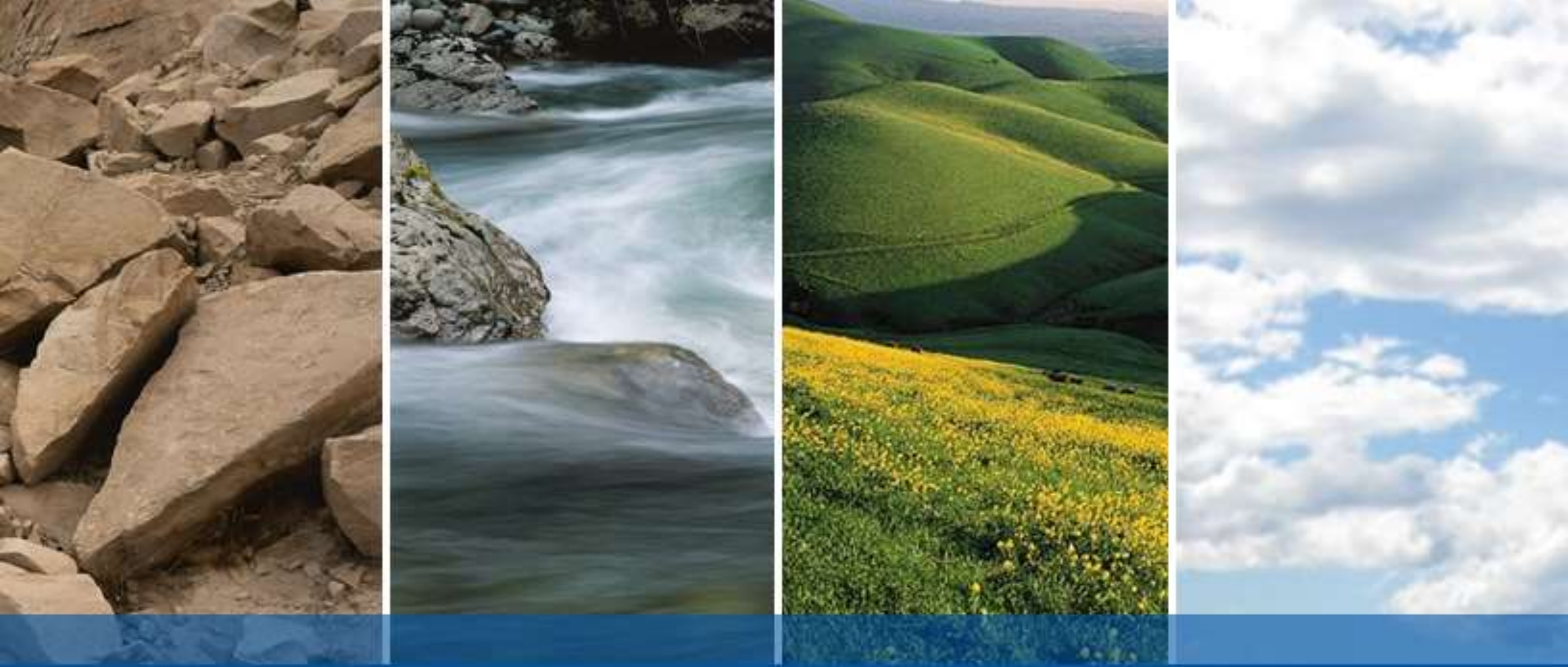
BASE MAP SOURCE:
COLOR HILLSHADE IMAGE BASED ON THE NATIONAL ELEVATION DATASET (NED) AT 30 METER RESOLUTION
U.S.G.S. QUATERNARY FAULT DATABASE, NOVEMBER, 2010
U.S.G.S. HISTORIC EARTHQUAKE DATABASE (1800–2000)

ENGEO
Expect Excellence

REGIONAL FAULTING AND SEISMICITY 420 EAST 3RD STREET DEVELOPMENT PITTSBURG, CALIFORNIA	PROJECT NO.: 18562.000.001		FIGURE NO. 4
	SCALE: AS SHOWN		
	DRAWN BY: CC	CHECKED BY: AL	

FILE PATH: G:\Drafting\PROJECTS_18000 to 19999\18562\18562000001\18562000001_FR_RFM_4_0321.dwg SAVE DATE: 3/31/2021 12:07:34 PM SAVED BY: CCulbertson

ORIGINAL FIGURE PRINTED IN COLOR



ENGEO
Expect Excellence