



INDUSTRIAL PARKWAY WEST
HAYWARD, CALIFORNIA

PRELIMINARY GEOTECHNICAL EXPLORATION

SUBMITTED TO
Mr. Kevin Lao
CenterPoint Properties
1111 Broadway, Suite 2130
Oakland, CA 94607

PREPARED BY
ENGEO Incorporated

January 31, 2023

PROJECT NO.
21194.000.001

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CenterPoint Properties
1111 Broadway, Suite 2130
Oakland, CA 94607

Subject: Industrial Parkway West
29469 Ruus Road
Hayward, California

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Dear Mr. Lao:

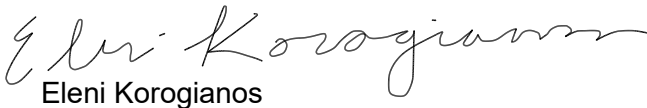
With your authorization, we prepared this preliminary geotechnical exploration report for the proposed warehouse building development at 29469 Ruus Road in Hayward, California, as outlined in our agreement dated November 18, 2022. The accompanying report contains our findings and our preliminary conclusions and recommendations for the geotechnical aspects of the proposed development.

Our preliminary findings indicate that the project site is suitable for the proposed warehouse project provided the preliminary recommendations provided in this report are incorporated into the project planning. Additional design-level exploration services will be needed to develop recommendations for grading, pavement design, retaining walls, and design of foundations for the proposed project. We are pleased to have been of service to you 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 report, please call and we will be glad to discuss them with you.

Sincerely,

ENGEO Incorporated


Eleni Korogianos


Jeff Fippin, GE




Csilla Kenny, PE

ek/ck/jaf/ca

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1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

We prepared this preliminary geotechnical exploration report for the proposed warehouse building at 29469 Ruus Road in Hayward, California. We prepared this report as outlined in our agreement dated November 18, 2022. CenterPoint Properties authorized us to conduct our scope of services, which included the following.

- Available geologic and geotechnical information review
- Preliminary subsurface field exploration
- Soil laboratory testing
- Preliminary data analysis and conclusion/recommendation development
- Preliminary report preparation

We prepared this report for the exclusive use of CenterPoint Properties and their consultants for the preliminary design of this project. For our use, we were provided preliminary project plans dated June 24, 2022, and discussed the project with you. In the event that any changes are made in the character, design, or layout of the development, we must be contacted to review the conclusions and recommendations contained in this report to evaluate whether modifications are necessary.

1.2 PROJECT LOCATION

The project site is located at 29469 Ruus Road in Hayward, California as shown in Figure 1. The overall site encompasses approximately 7.6 acres and is bounded to the north by various industrial and storage facilities, to the west by single-family residential homes and a commercial building, to the east by auto-related facilities, and to the south by Industrial Parkway West. The approximate boundary of the site is shown on the site plan, Figure 2.

The site is relatively level with ground surface elevations ranging between approximately Elevation 8.5 feet and Elevation 12.5 feet (Datum: NVGD29). There are currently several structures across the site. The remainder of the site is primarily paved with asphalt concrete and contains stacked wood pallets, scrap metal and other debris, and vehicles.

1.3 PROPOSED DEVELOPMENT

Based on the site plan and our discussions with you, we understand the proposed development will consist of construction of a new 103,000-square-foot warehouse building with associated offices, paved parking area, driveway, and landscaping elements. We anticipate minor grading will be required. Structural plans and loading are yet to be established; however, we anticipate the proposed building will be one story of concrete tilt-up construction and assume that structural loads will be representative for this type of construction. We also assume the proposed building will not include below-grade construction.

2.0 FINDINGS

2.1 GEOLOGY AND SEISMICITY

2.1.1 Regional Geology

The site is located in the northern portion of the California Coast Ranges geomorphic province, which is dominated by northwest-trending faults and folds. The Coast Ranges are a complex series of linear mountain ranges that lie more-or-less parallel to the coast and to the San Andreas Fault System. The Coast Ranges are composed primarily of Jurassic and Cretaceous rocks that accumulated on the sea floor and were later scraped off when the ocean plate on which they originated was subducted beneath North America. These older rocks include a tectonic mix of sandstone, chert, altered basalt referred to as greenstone, and serpentinite, collectively referred to as the Franciscan Complex. While Franciscan Complex bedrock is exposed in the hills and cliffs of the San Francisco Bay Area, the flanks of the hills are blanketed with thin to thick layers of colluvium and alluvium (weathered material washed downslope from the bedrock exposures). Valleys within this area of the San Francisco Bay Area are filled with water-laid stream deposits.

2.1.2 Local Geology

Regional geologic mapping by Dibblee (2005) indicates that the site is underlain by Holocene alluvium (Qa), which generally consists of gravel, sand, and clay of valley areas and includes gravel and sand of major stream channels. A geologic map of the site is shown in Figure 3.

2.1.3 Seismicity

The San Francisco Bay Area contains numerous active faults. The major active faults in the area include the San Andreas and San Gregorio to the west and the Hayward and Calaveras to the east. An active fault is defined by the State Mining and Geology Board as one that has experienced surface displacement within Holocene time (about the last 11,000 years) (Hart, 1997).

Numerous small earthquakes occur every year in the San Francisco Bay Region, and larger earthquakes have been recorded and can be expected to occur in the future. Figure 4 shows the approximate locations of mapped active faults and significant historic earthquakes recorded within the San Francisco Bay Region. The following table lists several of the mapped active faults and their proximity to the site.

TABLE 2.1.3-1: Active Faults Capable of Producing Significant Ground Shaking in the Site Vicinity - Latitude: 37.61839°, Longitude: -122.06537°

FAULT NAME	APPROXIMATE DISTANCE FROM PROJECT SITE (MILES)	MAXIMUM MOMENT MAGNITUDE
Hayward (So) [5]	4.7	6.9
Hayward (So) [4]	4.7	6.7
Calaveras	14.4	7.1
Mission (connected) [0]	2.6	6.0
Hayward (So) [3]	8.2	6.5
San Andreas (Peninsula) [6]	27.3	7.8

The Uniform California Earthquake Rupture Forecast (UCERF3, 2015) evaluated the 30-year probability of a Moment Magnitude 6.7 or greater earthquake occurring on the known active fault systems in the Bay Area. The UCERF3 estimated an overall probability of 72 percent for the San Francisco Region as a whole.

The site is not located within a currently designated Earthquake Fault Special Study Zone and no known surface expression of active faults is believed to exist within the site. Fault rupture through the site; therefore, is not likely.

2.2 FIELD EXPLORATION

We performed our field exploration on December 8, 2022. To characterize the subsurface conditions at the site, we advanced three cone penetration tests (CPTs) and sampled continuously at two locations using direct push methods at the locations shown on the Site Plan in Figure 2. The locations of our explorations are approximate as we estimated them using consumer-grade GPS and their proximity to existing site features; they should be considered accurate only to the degree implied by the method used.

2.2.1 Cone Penetration Tests

We retained the services of a subcontractor operating a truck-rig to push CPTs at three locations to depths ranging between 50 and 100 feet in general accordance with ASTM D-5778. Measurements include the tip resistance to penetration of the cone (Q_c), the resistance of the surface sleeve (F_s), and pore pressure (U) (Robertson and Campanella, 1988). We also obtained shear-wave velocity measurements at 1-SCPT3 using seismic cone penetration testing methods (Robertson et al. 1986). We permitted and backfilled the CPTs in accordance with the requirements of the Alameda County Public Works Agency. The CPT logs are presented in Appendix A.

2.2.2 Direct Push Sampling

We observed continuous direct push sampling at the locations shown on the Site Plan, Figure 2. We retained the services of a subcontractor operating a tracked geoprobe rig to perform continuous direct push sampling at the two locations to approximate depths of 50 feet below the ground surface (bgs). An engineer from our firm observed the sampling and logged the subsurface conditions at these locations. We permitted and backfilled the borings in accordance with the requirements of the Alameda County Public Works Agency.

2.3 SUBSURFACE CONDITIONS

During our direct push sampling, we encountered approximately 2 inches of asphalt concrete pavement over approximately 2 inches of aggregate base. Below the aggregate base, we encountered up to 6 feet of existing fill over the native soil deposits (1-DP2). The existing fill encountered generally consists of yellowish brown to dark gray stiff to very stiff lean clay with varying amounts of sand.

Below the existing fill, our exploration generally encountered heterogeneous alluvium deposits consisting primarily of medium stiff to very stiff lean clay with varying amounts of silt and sand to approximately 50 feet bgs. We encountered some medium stiff clay between 12 and 20 feet bgs. In both direct push borings, we encountered a sand lens that consisted of clayey sand and silty sand extending from approximately 21 feet bgs to 24 feet bgs. Below the silty sand, we

encountered stiff to very stiff lean clay with sand interbedded with sandy silt that extended to the maximum depth explored within our direct push sampling of 50 feet below existing grade.

The interpretations of the CPT data generally support the observations of our direct push sampling. Below 50 feet, 1-SCPT3 indicated interbedded layers of clay and silt to the maximum depth explored of 100 feet. The average measured shear-wave velocity in 1-SCPT3 is approximately 750 feet per second. Plasticity indices in the upper 6 feet ranged between 32 and 41, indicating a high shrink/swell potential with variation in moisture content.

2.4 GROUNDWATER CONDITIONS

Based on pore pressure dissipation tests performed during our CPTs, groundwater was indicated at depths of approximately 10 to 13 feet bgs.

The mapped historical shallowest groundwater depth is approximately 10 feet bgs for the site (Seismic Hazard Zone report, 2000). For the purposes of preliminary planning and design of the project, we recommend considering an estimated design groundwater depth of 10 feet bgs.

Fluctuations in the level of groundwater may occur due to variations in rainfall, irrigation practices, and other factors, which may result in groundwater levels that differ from the levels measured during our exploration.

2.5 LABORATORY TESTING

We performed geotechnical laboratory testing on select soil samples recovered during our field exploration to evaluate their physical index properties and characteristics. We performed moisture content and Atterberg limits testing for this study. The test results are included in Appendix B.

3.0 PRELIMINARY CONCLUSIONS

Based on our findings, we preliminarily conclude that the proposed development is feasible from a geotechnical standpoint provided the anticipated geotechnical hazards and associated recommendations included in this report are incorporated in the design and construction of the project. The primary geotechnical considerations for this project are summarized in the following sections.

3.1 EXISTING FILL

As stated previously, we encountered fill extending to depths up to approximately 6 feet bgs during our field exploration. The existing fill present at the site is likely associated with the historical development dating back to the 1960s. Due to the era of construction, any fill was likely not placed in an engineered manner appropriate for the planned development and should be considered non-engineered. Non-engineered fill beneath the building pad may result in differential settlement due to the variable soil properties within the fill. We provide recommendations for treatment of existing fill in Section 4.3.

3.2 EXPANSIVE SOIL

We observed potentially expansive clay near the surface of the site. Laboratory testing indicates that this soil exhibits a high shrink/swell potential with variations in moisture content.

Expansive soil is susceptible to shrink and swell that could affect structures and other improvements constructed on the ground (such as pavement and flatwork). Potential mitigation measures to reduce adverse effects of expansive soil for this project may include one or several of the following.

- Deepening foundations to below the zone of moisture fluctuation, such as using deep embedded footings
- Placing a layer of low-expansion “select” import fill or use of lime treatment of the building pad

Successful performance of structures on expansive soil requires special attention during construction. It is imperative that exposed soil be kept moist prior to placement of concrete for foundation construction. It is extremely difficult to re-moisturize clayey soil without excavation, moisture conditioning, and re-compaction.

3.3 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 ground lurching. The following sections present a discussion of these hazards as they apply to the site.

3.3.1 Ground Rupture

Since there are no known active faults crossing the property and the site is not located within an Earthquake Fault Special Study Zone, it is our opinion that ground rupture is unlikely at the subject property.

3.3.2 Ground Shaking

An earthquake of moderate to high magnitude generated within the San Francisco Bay Region could cause considerable ground shaking at the site, similar to that which has occurred in the past. To mitigate the shaking effects, all structures should be designed using sound engineering judgment and the latest California Building Code (CBC) requirements, at a minimum.

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 comparable 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).

3.3.3 Liquefaction and Liquefaction-Induced Settlement

As shown in Figure 5, the site is mapped within a State of California Seismic Hazard Zone (CGS, 2006) 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 most susceptible to

liquefaction is clean, loose, saturated, uniformly graded, fine-grained sand. Empirical evidence indicates that loose to medium dense gravel, silty sand, low plasticity silt, and some low plasticity clay are also potentially liquefiable.

We analyzed the CPTs to estimate the potential for liquefaction using the software program Clig applying the methodology published by Boulanger & Idriss (2008). We used the Mapped Maximum Considered Earthquake (MCE) Geometric Mean peak ground acceleration (PGA_M) of 0.9g, as outlined in ASCE 7-16, and a moment magnitude of 7.3, based on the characteristic magnitude of the nearby Hayward Fault. We evaluated the liquefaction potential for the soil encountered below the design groundwater level of 10 feet bgs.

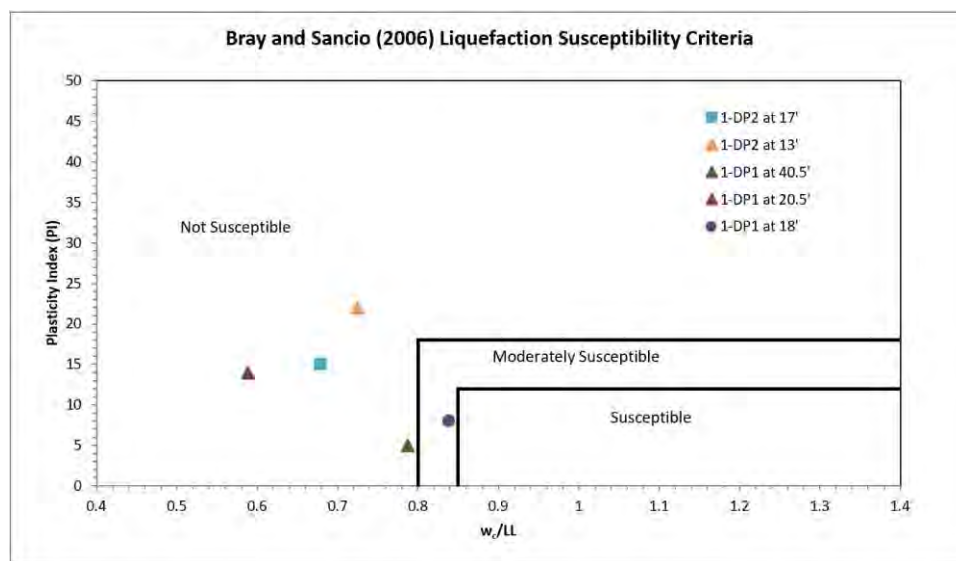
We also considered the criteria presented by Bray and Sancio (2006) to assess the potential for liquefaction triggering of the site soil. Bray and Sancio (2006) discusses the susceptibility to liquefaction as a function of liquid limit (LL), plasticity index (PI), and water content (w_c). Table 3.3.3-1 displays the liquefaction criteria.

TABLE 3.3.3-1: Bray and Sancio (2006) Criteria

LIQUEFACTION SUSCEPTIBILITY	w_c/LL	PI
Susceptible to Liquefaction	> 0.85	< 12
Moderately Susceptible to Liquefaction	> 0.80	12 to 18
Not Susceptible to Liquefaction	< 0.80	> 18

We performed laboratory testing on five samples selected from our direct push probes to assess their susceptibility to liquefaction. The results of our assessment are shown in Exhibit 3.3.3-1 below. We used these findings to establish a site-specific I_c (soil behavior index) cutoff of 2.5 based on the average I_c of the soil that is considered “not susceptible” to liquification. The results of our liquefaction analyses using this I_c cutoff value indicate there are layers of potentially liquefiable soil, primarily the silty sand layer at approximately 20 to 25 feet and interbedded sand layers at approximately 40 to 45 feet below the ground surface.

EXHIBIT 3.3.3-1: Bray and Sancio (2006) Liquefaction Susceptibility Criteria



3.3.4 Liquefaction-Induced Settlement

Deformation of the ground surface is a common result of liquefaction. Vertical settlement may result from densification of the deposit or volume loss from venting to the ground surface. Densification occurs as excess pore pressures dissipate, resulting in vertical settlement at the ground surface.

We calculated potential liquefaction-induced settlement estimates using the program Cliq. We estimate the total seismic-induced settlements to be approximately 2 inches for an MCE-level seismic event. We estimate that differential seismic settlement over a span of 50 feet could be up to approximately 1½ inches.

3.3.5 Lateral Spreading

Lateral spreading involves lateral ground movements caused by seismic shaking. These lateral ground movements are often associated with a weakening or failure of an embankment or soil mass over a layer of liquefied sand or weak soil. The effects of lateral spreading are often amplified by a “free face,” such as slopes found near rivers or creek banks. Based on the site conditions, the potential of lateral spreading is low.

3.3.6 Ground Lurching

Ground lurching is a result of the rolling motion imparted to the ground surface during energy released by an earthquake. Such rolling motion can cause ground cracks to form in weaker soil. The potential for the formation of these cracks is considered greater at contacts between deep alluvium and bedrock. Such an occurrence is possible at the site as in other locations in the Bay Area region, but based on the site location the offset, if any, would be minor.

3.4 SHALLOW GROUNDWATER

Based on the groundwater levels previously discussed, it appears that shallow groundwater beneath the site could affect the proposed development. Shallow groundwater can:

1. Impede grading activities, especially excavation and backfill of deep utility trenches.
2. Require construction dewatering.
3. Cause moisture damage to sensitive floor coverings.
4. Transmit moisture vapor through slabs causing excessive mold/mildew build-up, fogging of windows, and damage to computers and other sensitive equipment.

4.0 PRELIMINARY EARTHWORK RECOMMENDATIONS

4.1 DEMOLITION AND SITE CLEARING

The proposed development area should be cleared of obstructions, including existing buildings, foundations, and buried utilities. Existing underground utilities at the site should be identified and either properly abandoned or relocated.

If loose and/or deleterious materials are present within the existing fill, the materials should be removed, and the area should be backfilled with suitable material compacted to the recommendations presented in Section 4.5. In addition, if existing deep foundation elements are encountered within the footprint of the proposed building, these elements should be removed down to at least 5 feet below the bottom of the proposed finished grade. All backfill should be observed and tested by a representative of our firm.

No loose or uncontrolled backfilling of depressions resulting from demolition and stripping should be performed.

4.2 SUBGRADE PREPARATION

Following demolition or stripping of surface soil, areas to receive fill or new development should be scarified to a depth of 8 inches, moisture conditioned, and compacted to the requirements for engineered fill presented in Section 4.5. The finished subgrade should be firm and non-yielding under the weight of the compaction equipment.

4.3 EXISTING FILL OVEREXCAVATION

We recommend any existing fill within the building footprint be overexcavated to native material as observed by our representative.

The bottom of the overexcavated areas should be scarified, mixed, moisture conditioned to a depth of 8 inches, and compacted. The condition of the base of the overexcavated areas should be evaluated by our representative prior to backfilling. The depth of overexcavation may need to be increased if unstable conditions such as large voids or debris/organics are observed by our representative at the time of grading.

The overexcavated areas should be backfilled with engineered fill compacted to the requirements in Section 4.5. The excavated existing fill may be suitable for use as engineered fill provided it is processed to remove concentrations of organic material, debris, and particles greater than 6 inches in maximum dimension and meets the requirements in Section 4.4 below.

4.4 ACCEPTABLE FILL MATERIAL

On-site soil material and existing fill material may be suitable as fill material provided it is processed to remove concentrations of organic material, debris, and particles greater than 6 inches in maximum dimension. Once the site is stripped to expose the underlying soil, an evaluation of the quality of the fill will be conducted by a representative of our firm.

Imported fill materials should have a plasticity index less than 25 and have at least 20 percent passing the No. 200 sieve. We should be allowed to sample and test proposed imported fill materials at least 72 hours prior to delivery to the site.

If desired to reuse asphalt or Portland cement concrete as engineered fill, the recycled asphalt or concrete should be ground down to less than 6 inches in greatest dimension, but not pulverized. It should be noted that materials derived from crushed concrete and asphalt are generally not suitable for supporting plant growth. The landscape designer should be consulted for additional information.

4.5 FILL COMPACTION

4.5.1 Grading in Structural Areas

After preparing the site, as recommended in Sections 4.1, 4.2, and 4.3, the contractor should mechanically compact the subgrade in accordance with the recommendations in this section. The loose lift thickness should not exceed 8 inches or the depth of penetration of the compaction equipment used, whichever is less. Engineered fill should be compacted and moisture conditioned in accordance with the requirements presented in Table 4.5.1-1.

TABLE 4.5.1-1: Engineered Fill Compaction and Moisture Content Requirements

MATERIALS	RELATIVE COMPACTION (%)	MINIMUM MOISTURE CONTENT (percentage points above optimum)
General Fill	Between 87 and 92	4
Non-expansive Fill	Minimum 90	2
Aggregate Base	Minimum 95	0

4.5.2 Underground Utility Backfill

The contractor is responsible for conducting all trenching and shoring in accordance with CALOSHA requirements. Project consultants involved in utility design should specify pipe-bedding materials. The contractor should place and compact trench backfill in accordance with the recommendations in Section 4.5.1.

Where utility trenches cross below buildings, we recommend that a plug be placed within the trench backfill to help prevent the normally granular bedding materials from acting as a conduit for water to enter beneath the building. The plug should be constructed using a sand cement slurry (minimum 28-day compressive strength of 500 psi) or relatively impermeable native soil for pipe bedding and backfill. We recommend that the plug extend for a distance of at least 3 feet in each direction from the point where the utility enters the building perimeter.

Jetting of backfill without mechanical compaction is not an acceptable means of compaction. We may allow thicker loose lift thicknesses based on acceptable density test results for the first lift of fill over pipe bedding.

4.6 OVER-OPTIMUM SOIL MOISTURE CONDITIONS

The contractor should anticipate excessively over-optimum (wet) soil moisture conditions during winter or spring grading, or during or following periods of rain. As previously mentioned, we recommend a design groundwater depth of 10 feet; overly wet conditions may be encountered in excavations that extend below this depth. Wet soil can make proper compaction difficult or impossible. Wet soil conditions can be mitigated by:

1. Frequent spreading and mixing during warm dry weather,
2. Mixing with drier materials,
3. Mixing with a lime, lime-flyash, or cement product, or
4. Stabilizing with aggregate, geotextile stabilization fabric, or both.

We should evaluate Options 3 and 4 prior to implementation.

4.7 SLOPES

Permanent slopes should be constructed with gradients 2:1 (horizontal:vertical) or flatter. The contractor is responsible to construct temporary construction slopes in accordance with CALOSHA requirements.

4.8 SITE DRAINAGE

The project civil engineer is responsible for designing surface drainage improvements. With regard to geotechnical engineering issues, we recommend that finish grades be sloped away from buildings and pavements to the maximum extent practical. The latest California Building Code Section 1804.4 specifies minimum slopes for pervious surfaces of 5 percent away from foundations for a minimum distance of 10 feet measured perpendicular to the wall of the structure. Where lot lines or surface improvements restrict meeting this slope requirement, we recommend that specific drainage requirements be developed. At a minimum, we recommend the following.

1. Roof downspouts should discharge into closed conduits and direct away from foundations to appropriate drainage devices.
2. Water should not be allowed to pond near foundations, pavements, or exterior flatwork.

4.9 STORMWATER INFILTRATION AND BIORETENTION AREAS

Due to the high clay content of near-surface soil, we expect the site soil will have low permeability for stormwater infiltration. Therefore, best management practices should assume that stormwater infiltration will be limited unless an engineered system is designed. If bioretention areas are implemented, we recommend that a subdrain or other storm drain system be incorporated to collect and convey water to an approved outlet, considering the low permeability of site soil. When practical, bioretention areas should be planned a minimum of 5 feet away from structural improvements, such as buildings, streets, and sidewalks/driveways. When this is not practical, bioretention areas located within 5 feet of structural improvements should either:

1. Be constructed with structural side walls capable of withstanding the loads from the adjacent improvements, or
2. Incorporate filter material compacted to between 85 and 90 percent relative compaction and a waterproofing system designed to reduce the potential for moisture transmission into the subgrade soil beneath the adjacent improvement.

In addition, one of the following options should be followed.

1. We recommend that bioretention design incorporate a waterproofing system lining the bioswale excavation and a subdrain, or other storm drain system, to collect and convey water to an approved outlet. The waterproofing system should cover the bioretention area excavation in such a manner as to reduce the potential for moisture transmission beneath the adjacent improvements.
2. Alternatively, and with some risk of movement of adjacent improvements, if infiltration is desired, we recommend the perimeter of the bioretention areas be lined with an HDPE tree root barrier that extends at least 1 foot below the bottom of the bioretention areas/infiltration trenches.

Site improvements located adjacent to bioretention areas that are underlain by base rock, sand, or other imported granular materials, should be designed with a deepened edge that extends to the bottom of the imported material underlying the improvement.

If the surface of the bioretention area is depressed, the slope gradient should follow the slope guidelines described in earlier section(s) of this document. In addition, although not recommended, if trees are to be planted within bioretention areas, HDPE tree boxes that extend below the bottom of the bioretention system should be installed to reduce potential impact to subdrain systems that may be part of the bioretention area design. For this condition, the waterproofing system should be connected to the HDPE tree box with a waterproof seal.

Given the nature of bioretention systems and possible proximity to improvements, we recommend we be retained to review design plans and provide testing and observation services during the installation of linings, compaction of the filter material, and connection of designed drains.

It should be noted that the contractor is responsible for conducting all excavation and shoring in a manner that does not cause damage to adjacent improvements during construction and future maintenance of the bioretention areas. As with any excavation adjacent to improvements, the contractor should reduce the exposure time such that the improvements are not detrimentally impacted.

4.10 LANDSCAPING CONSIDERATION

Because the near-surface soil is highly expansive, we recommend restricting the amount of surface water infiltration near structures, pavements, flatwork, and slabs-on-grade. This may be accomplished by:

- Selecting landscaping that requires little or no watering, especially within 5 feet of structures or pavements.
- Using low precipitation sprinkler heads.
- Regulating the amount of water distributed to lawn or planter areas by installing timers on the sprinkler system.
- Providing surface grades to drain rainfall or landscape watering to appropriate collection systems and away from structures or pavements.
- Preventing water from draining toward or ponding near foundations, slabs-on-grade, or pavements.
- Avoiding open planting areas within 3 feet of the building perimeter.

We recommend that these items be incorporated into the landscaping plans.

5.0 PRELIMINARY FOUNDATION RECOMMENDATIONS

Major considerations in foundation design for the planned building at this site are the potential presence of existing fill, near-surface expansive soil, and liquefaction-induced settlement. In our opinion, the most effective foundation system for the proposed building is a shallow spread footing foundation system with implementation of specific mitigation measures.

The proposed structure may be supported by a conventional footing system provided the footings are deepened and the building pad is capped with 8 inches of non-expansive fill to reduce the effects of shallow expansive soil.

For preliminary purposes the following design parameters can be used for footing design. Additional soil samples should be obtained during the design-level study to further evaluate the allowable bearing capacity and validate these preliminary design parameters.

Minimum Width	18 inches
Minimum Depth*	36 inches
Allowable Bearing Pressure.....	2,000 pounds per square foot (psf)
* Depth below lowest adjacent finish grade	

For preliminary purposes, the foundation system should be designed to accommodate up to 2 inches of liquefaction-induced settlement (1½ inches of differential settlement over a distance of 50 feet) when evaluating building performance during a seismic event. Resistance to lateral loads may be provided by frictional resistance between the foundation concrete and the subgrade soil and by passive earth pressure acting against the side of the foundation.

The interior slabs-on-grade should be underlain by a layer of non-expansive fill at least 12 inches thick. The non-expansive fill may consist of imported select fill or lime treated soil. Consideration should be given to constructing the concrete slab-on-grade floor with a minimum thickness of 5 inches; however, the final slab thickness and reinforcing should be designed by the structural engineer based on the intended use and loading of the slab. The slab should be sufficiently rigid to transfer floor loads to the spread footings. Additional static settlement may be induced if the slab is not sufficiently rigid and loads are transferred to the soil by the slab.

If a two-pour system is used for footings and slab, the cold joint between the exterior footing and slab-on-grade should be located at least 4 inches above adjacent finish exterior grade. If this is not done, then we recommend the addition of a waterstop between the two pours to reduce moisture penetration through the cold joint and migration under the slab. Use of a monolithic pour would eliminate the need for either of these mitigations.

5.1 SLAB MOISTURE VAPOR REDUCTION

When buildings are constructed with concrete slab-on-grade, 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 recommend the following to reduce, but not stop, water vapor transmission upward through the slab-on-grade.

1. A vapor retarder membrane should be installed directly beneath the slab. The vapor retarder should be sealed at all seams and pipe penetrations. Vapor retarders should conform to Class A vapor retarder in accordance with ASTM E 1745, latest edition, "Standard Specification for Plastic Water Vapor Retarders used in Contact with Soil or Granular Fill under Concrete Slabs."
2. A 4-inch-thick layer of clean crushed rock should be placed between the vapor retarder membrane and the building pad soil if the footing and slab-on-grade option is selected.
3. The concrete should have a concrete water-cement ratio of no more than 0.50.

4. Inspection and testing should be provided during concrete placement to check that the proper concrete and water cement ratio are used.
5. Slabs should be moist cured for a minimum of 3 days or use other equivalent curing specified by the structural engineer.

The structural engineer should be consulted as to the use of a layer of clean sand or pea gravel (less than 5 percent passing the U.S. Standard No. 200 Sieve) placed on top of the vapor retarder membrane to assist in concrete curing.

5.2 2022 CBC SEISMIC DESIGN PARAMETERS

The 2022 CBC utilizes seismic design criteria established in the ASCE/SEI Standard "Minimum Design Loads and Associated Criteria for Buildings and Other Structures," (ASCE 7-16). Based on the subsurface conditions encountered and an anticipated building period of less than 0.5 second, we characterized the site as Site Class D. ASCE 7-16 requires a site-specific ground-motion hazard analysis at Site Class D sites with a mapped S_1 value greater than or equal to 0.2. However, Section 11.4.8 provides exceptions to these requirements. In Table 5.2-1 below, we provide the CBC seismic parameters based on the United States Geological Survey's (USGS') Seismic Design Maps for your use and consideration assuming the building will be designed using the exceptions in Section 11.4 of ASCE 7-16. If requested by the project structural engineer, we can perform a site-specific seismic-hazard analysis (SHA) following the guidelines of ASCE 7-16 under a separate scope of work

TABLE 5.2-1: 2022 CBC Seismic Design Parameters, Latitude: 37.61839° Longitude: 122.06537°

PARAMETER	VALUE
Site Class	D
Mapped MCE_R Spectral Response Acceleration at Short Periods, S_S (g)	1.99
Mapped MCE_R Spectral Response Acceleration at 1-second Period, S_1 (g)	0.76
Site Coefficient, F_a	1.0
Site Coefficient, F_v	1.7*
MCE_R Spectral Response Acceleration at Short Periods, S_{MS} (g)	1.99
MCE_R Spectral Response Acceleration at 1-second Period, S_{M1} (g)	1.30*
Design Spectral Response Acceleration at Short Periods, S_{DS} (g)	1.33
Design Spectral Response Acceleration at 1-second Period, S_{D1} (g)	0.87*
MCE_G Peak Ground Acceleration adjusted for Site Class effects, PGA_M (g)	0.92

*These parameters should only be used for calculation of T_s , determination of Seismic Design Category, and, calculations of S_{M1} and S_{D1} using the exceptions under item 1 of ASCE 7-16 Section 11.4.8. The latter will result in 50 percent increase in above listed S_{D1} and S_{M1} . (Supplement Number 3 <https://ascelibrary.org/doi/epdf/10.1061/9780784414248.sup3>).

6.0 PRELIMINARY PAVEMENT DESIGN

6.1 FLEXIBLE PAVEMENT

Without laboratory testing, we conservatively designed for a Resistance (R-Value) of 5. The design sections may be reduced based on R-Value testing of samples collected from actual pavement subgrade during a design-level exploration. Using a range of traffic indexes, we developed the following preliminary pavement sections using Chapter 630 of the 6th Edition of the Caltrans Highway Design Manual (including the asphalt factor of safety), presented in the table

below. During a design-level study, the project civil engineer should provide suitable traffic indices based on the estimated traffic loads and frequencies.

TABLE 6.1-1: Recommended Asphalt Concrete Pavement Sections

TRAFFIC INDEX	SECTION	
	HOT MIX ASPHALT (INCHES)	CLASS 2 AGGREGATE BASE (INCHES)
4.5	2½	10
5	3	10
6	3½	13
7	4	16

6.2 RIGID PAVEMENT

Concrete pavement sections can be used to resist heavy loads and turning forces in areas such as loading docks, 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.
- As an additional measure to improve performance, rigid pavements may be constructed with thickened edges extending at least beneath the aggregate base and 2 inches into compacted soil to reduce water infiltration, where adjacent to landscaped areas.

6.3 EXTERIOR FLATWORK

Exterior flatwork includes items such as concrete sidewalks, steps, and outdoor courtyards exposed to foot traffic only. We recommend flatwork be designed with a minimum section of 5 inches of concrete over 4 inches of aggregate base. The contractor should compact the aggregate base to at least 90 percent relative compaction. The flatwork edges should be thickened to at least 8 inches to help control moisture variations in the subgrade and place rebar within the middle third of the slab to help control the width and offset of cracks. Construct control and construction joints in accordance with current Portland Cement Association Guidelines.

7.0 DESIGN LEVEL GEOTECHNICAL EXPLORATION

We recommend that a design-level geotechnical exploration be performed once more detailed plans are available. The design-level exploration should include additional subsurface exploration including soil borings and additional cone penetration tests and laboratory testing to refine the preliminary conclusions and recommendations contained in this report. In addition a design-level geotechnical report should address the following.

- Assessment of potentially liquefiable and compressible soil layers, as necessary.

- Conceptual measures to mitigate hazards and geotechnical constraints such as loose/soft surface soil, existing fill, compressible soil, expansive soil, liquefiable soil, and lateral spreading, as necessary, based on field exploration results.
- A site-specific seismic-hazard analysis if required by the project structural engineer.
- Foundation concrete strength recommendations based on sulfate testing.
- Site grading recommendations, including fill placement recommendations, utility backfill, and recommendations for site drainage.
- Secondary slab-on-grade (flatwork) recommendations.
- Pavement recommendations for hot mix asphalt and Portland cement concrete.

The design-level geotechnical report should include a summary of the surface and subsurface conditions, seismicity, laboratory test data, exploration logs, and a site plan showing previous exploratory locations and improvement limits.

8.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

This preliminary report is issued with the understanding that it is the responsibility of the owner to transmit the information and recommendations of this report to developers, contractors, buyers, architects, engineers, and designers for the project so that the necessary steps can be taken by the contractors and subcontractors to carry out such recommendations in the field. The preliminary conclusions and recommendations contained in this preliminary report are solely professional opinions. We strive to perform our professional services in accordance with generally accepted principles and practices currently employed in the area; there is no warranty, either express or implied. There are risks of earth movement and property damages inherent in land development. We are unable to eliminate all risks; therefore, we are unable to guarantee or warrant the results of our work.

This preliminary report is based upon conditions discovered at the time of preparation of our work. This document must not be subject to unauthorized reuse, that is, reuse 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. Actual field or other conditions will necessitate clarifications, adjustments, modifications or other changes to our work. Therefore, we must be engaged to prepare the necessary clarifications, adjustments, modifications or other changes before construction activities commence or further activity proceeds. If our scope of services does not include on-site construction observation, or if other persons or entities are retained to provide such services, we cannot be held responsible for any or all claims, including, but not limited to claims arising from or resulting from the performance of such services by other persons or entities, and any or all claims arising from or resulting from clarifications, adjustments, modifications, discrepancies or other changes necessary to reflect changed field or other conditions.

SELECTED REFERENCES

- American Society of Civil Engineers. 2016. Minimum Design Loads for Buildings and other Structures ASCE 7-16, Reston: American Society of Civil Engineers.
- American Concrete Institute. 2008. Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary (ACI 318R-08).
- Boulanger, R. W. and Idriss, I. M. 2014. CPT and SPT Based Liquefaction Triggering and Procedures, Center for Geotechnical Modeling, Report No. UCD/CGM-14/01.
- California Building Standards Commission. 2019. California Building Code, Volumes 1 and 2, Sacramento, California.
- California Geologic Survey. 2006. Earthquake Zones of Required Investigation, Newark Quadrangle.
- California Geologic Survey. 2008. Special Publication 117A, Guidelines for Evaluating and Mitigating Seismic Hazards in California.
- Dibblee, Thomas W. 2005. Geologic Map of the Newark Quadrangle, Contra Costa & Alameda Counties, California.
- Division of Mines and Geology. 1997. Special Publication 117, Guidelines for Evaluation and Mitigating Seismic Hazards in California. Adopted March 13.
- Field, E.H., and 2014 Working Group on California Earthquake Probabilities, 2015, UCERF3: A new earthquake forecast for California's complex fault system: U.S. Geological Survey 2015–2009, 6 p., <https://dx.doi.org/10.3133/fs20153009>.
- Graymer, R. W. 2000. Geological Map and Map Database of the Oakland Metropolitan Area, Alameda, Contra Costa, and San Francisco Counties, California.
- Ishihara, K. 1985. Stability of Natural Deposits During Earthquakes, Proc 11th International Conference on Soil Mechanics and Foundation Engineering, Vol 1, A. A. Balkema, Rotterdam, The Netherlands, 321-376.
- Robertson, P. K. and Campanella, R. G. 1988. Guidelines for Geotechnical Design Using CPT and CPTU Data.
- Robertson, P. K. 2009. Performance based earthquake design using the CPT, Gregg Drilling and Testing, Inc.
- Robertson, P. K. and Lisheng, S. 2010. Estimation of Seismic Compression in dry soils using the CPT. Fifth International Conference on Recent Advances in Geotechnical Earthquake.
- SEAOC. 1996. Recommended Lateral Force Requirements and Tentative Commentary.
- United States Geologic Survey (USGS). 2008. National Seismic Hazard Maps Fault Parameters.

SELECTED REFERENCES (Continued)

- Youd, T. L. and I. M. Idriss. 2001. Liquefaction Resistance of Soils, Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshop on Evaluation of Liquefaction Resistance of Soils.
- Zhang, G. Robertson. P.K, Brachman, R. 2002. Estimating Liquefaction Induced Ground Settlements from the CPT, Canadian Geotechnical Journal, 39: pp 1168-1180.



FIGURES

FIGURE 1 – Vicinity Map

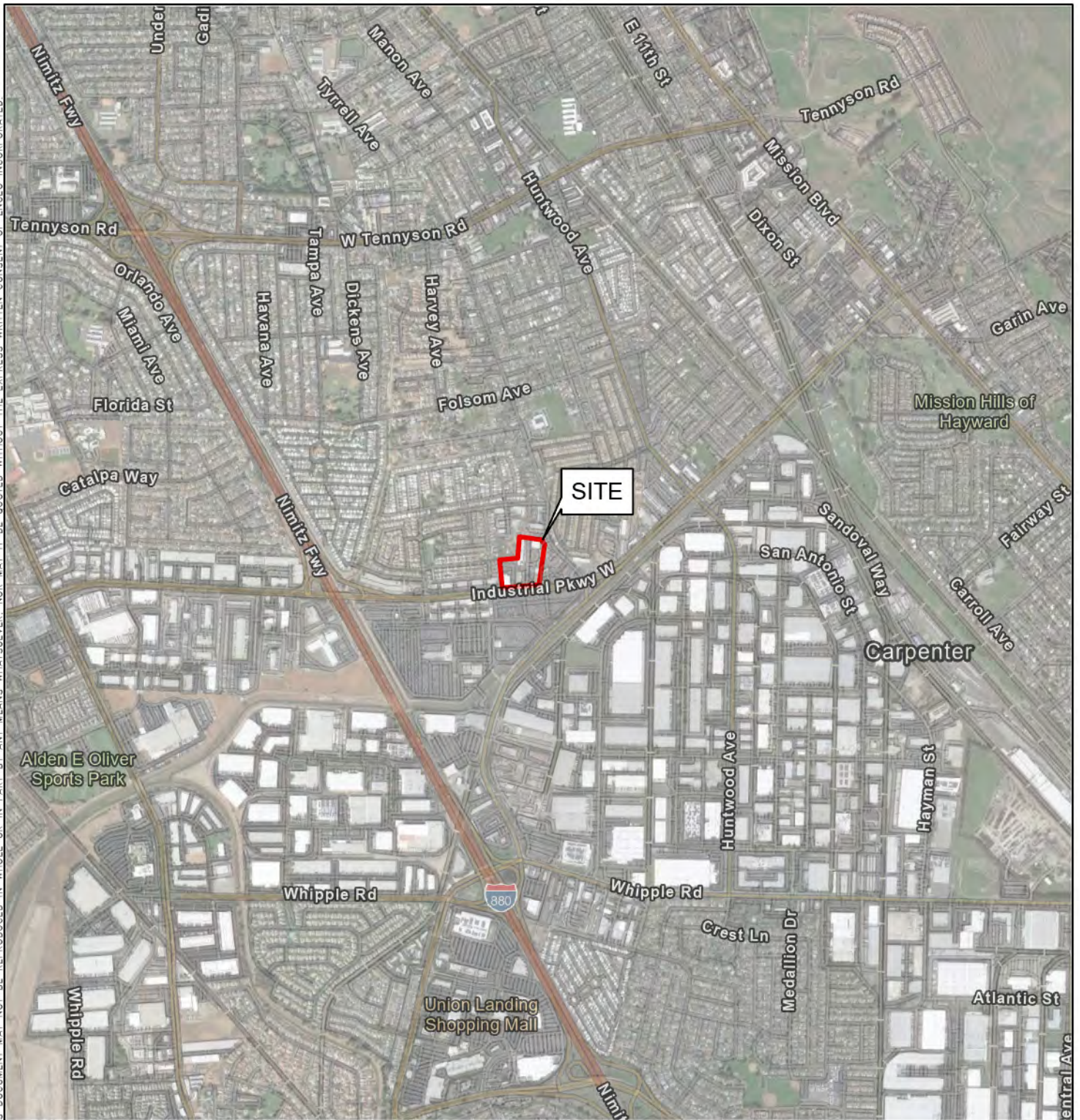
FIGURE 2 – Site Plan

FIGURE 3 – Regional Geologic Map

FIGURE 4 – Regional Faulting and Seismicity

FIGURE 5 – Seismic Hazard Zones Map

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0 1,000 2,000
FEET

BASEMAP SOURCE: GOOGLE EARTH MAPPING SERVICE 2022



VICINITY MAP
INDUSTRIAL PARKWAY WEST
HAYWARD, CALIFORNIA

PROJECT NO. : 21194.000.001

SCALE: AS SHOWN

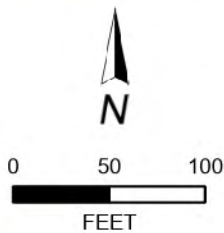
DRAWN BY: QRL

CHECKED BY: JAF

FIGURE NO.

1

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EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

- PROJECT SITE
- PROPOSED BUILDING FOOTPRINT
- CONE PENETRATION TEST (ENGEO, 2022)
- DIRECT PUSH SAMPLE (ENGEO, 2022)

BASEMAP SOURCE: GOOGLE EARTH MAPPING SERVICE 2021



SITE PLAN

INDUSTRIAL PARKWAY WEST
HAYWARD, CALIFORNIA

PROJECT NO. : 21194.000.001

SCALE: AS SHOWN

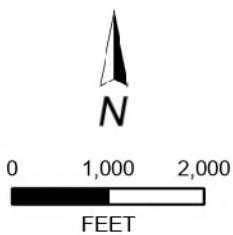
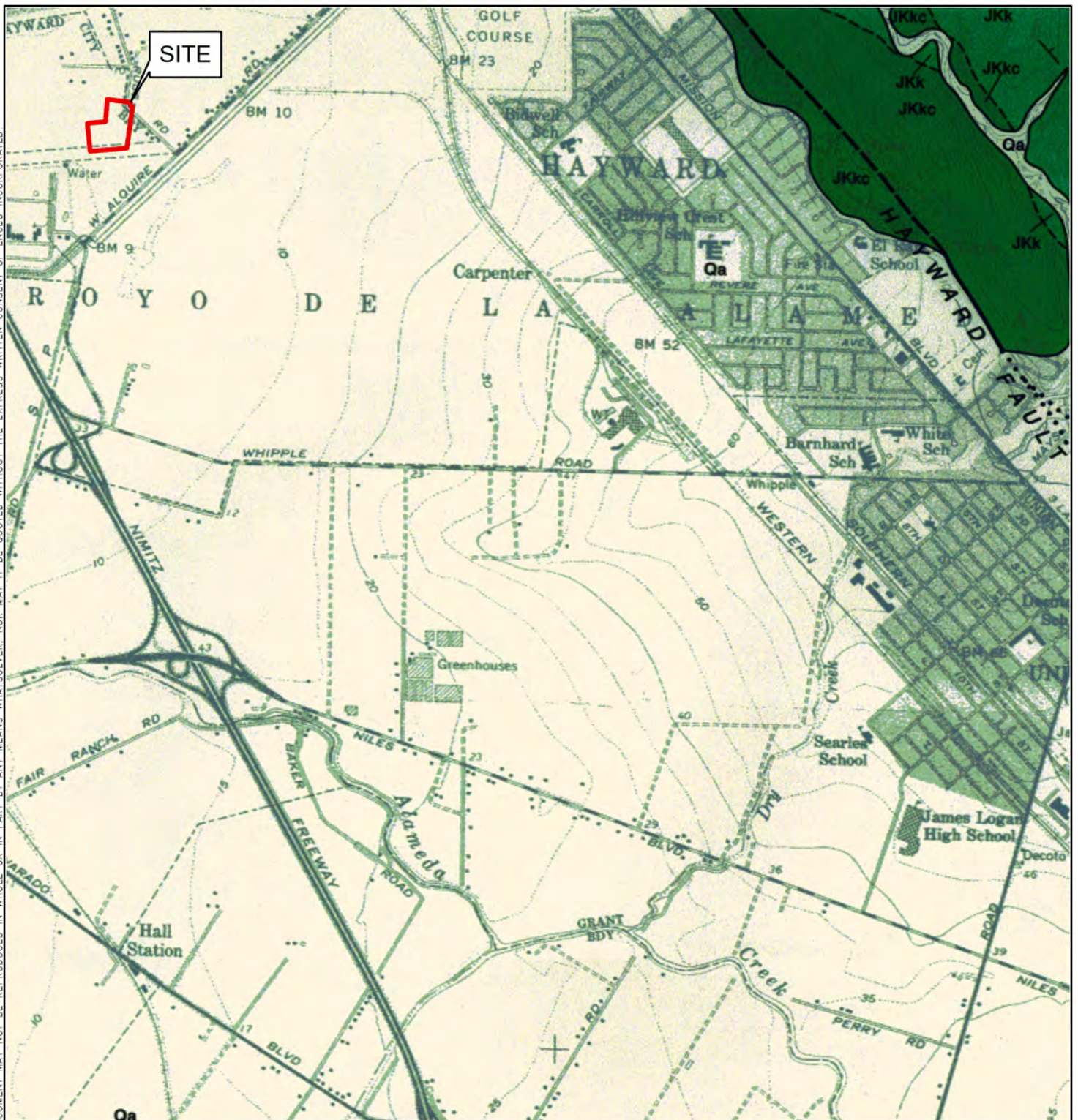
DRAWN BY: QRL

CHECKED BY: JAF

FIGURE NO.

2

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EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

- | | | |
|---|------|--|
|  | Qa | ALLUVIAL GRAVEL, SAND AND CLAY OF VALLEY AREAS |
|  | JKk | CLAYSHALE OR CLAYSTONE |
|  | JKkc | CONGLOMERATE |

BASEMAP SOURCE: DIBBLEE 2005



REGIONAL GEOLOGIC MAP

INDUSTRIAL PARKWAY WEST
HAYWARD, CALIFORNIA

PROJECT NO. : 21194.000.001

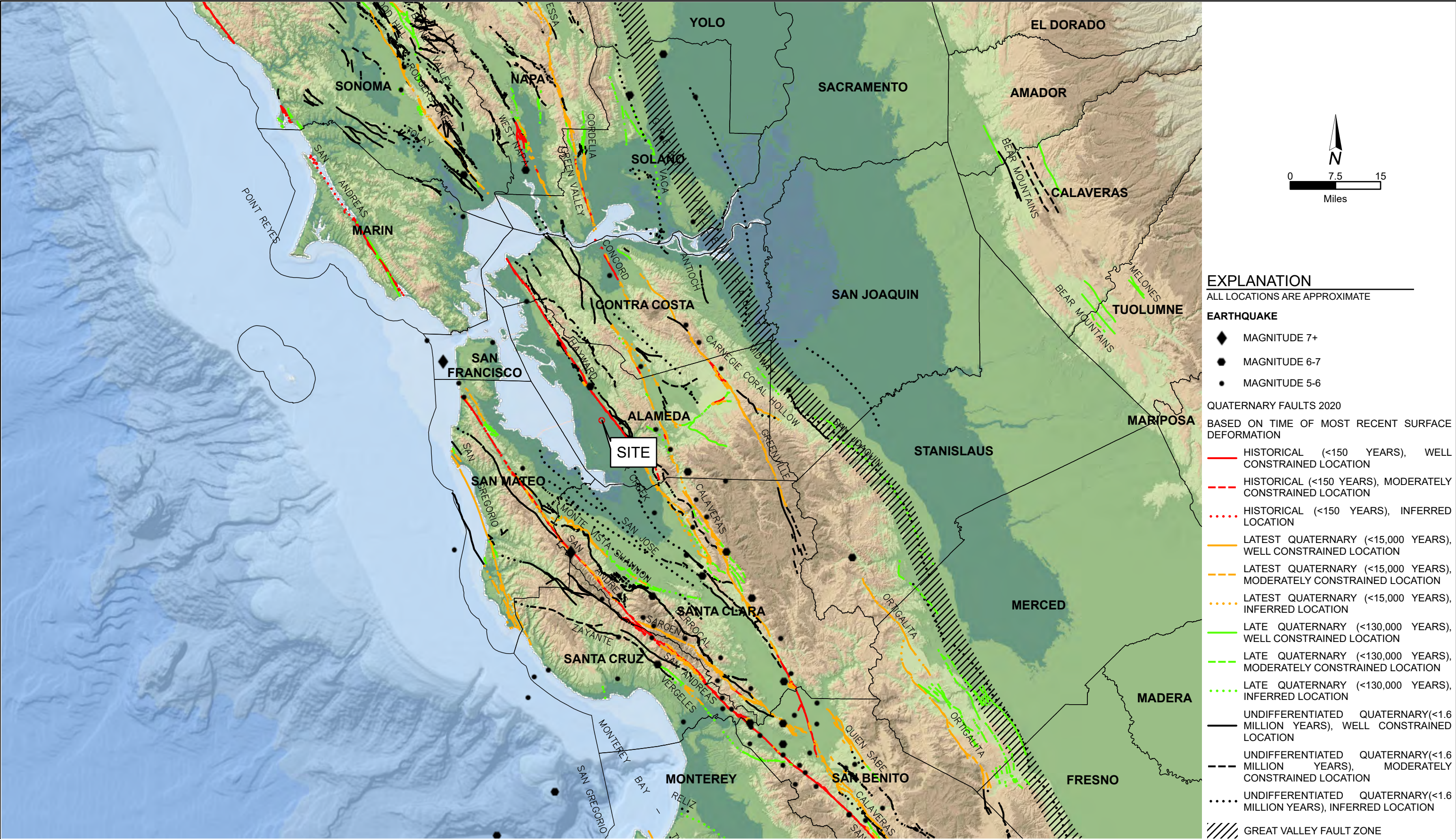
SCALE: AS SHOWN

DRAWN BY: QRL

CHECKED BY: JAF

FIGURE NO.

3



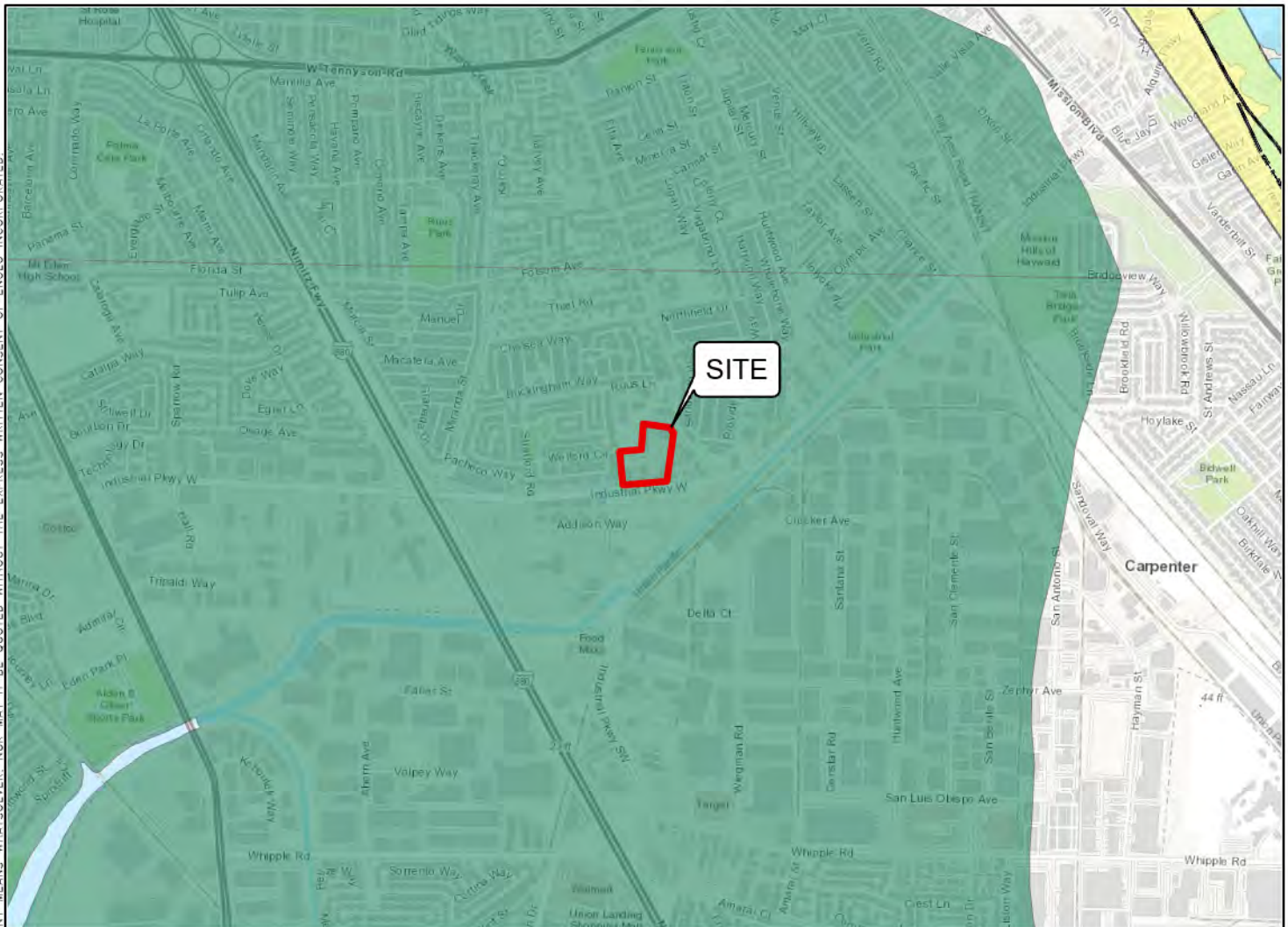
BASE MAP SOURCE
ESRI, GEBCO, DELORME, NATURALVUE
COLOR HILLSHADE IMAGE BASED ON THE NATIONAL ELEVATION DATA SET (NED) AT 30 METER RESOLUTION
U.S.G.S. QUATERNARY FAULT DATABASE, 2020
U.S.G.S. HISTORIC EARTHQUAKE DATABASE (1800-PRESENT)
U.S.G.S OPEN-FILE REPORT 96-705



REGIONAL FAULTING AND SEISMICITY
INDUSTRIAL PARKWAY WEST
HAYWARD, CALIFORNIA

PROJECT NO. : 21194.000.001		FIGURE NO. 4
SCALE: AS SHOWN		
DRAWN BY: SPPE	CHECKED BY: JAF	

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EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

— ACCURATELY LOCATED

- - - APPROXIMATELY LOCATED

EARTHQUAKE FAULT ZONE

ZONE BOUNDARIES ARE DELINEATED BY STRAIGHT-LINE SEGMENTS; THE BOUNDARIES DEFINE THE ZONE ENCOMPASSING ACTIVE FAULTS THAT CONSTITUTE A POTENTIAL HAZARD TO STRUCTURES FROM SURFACE FAULTING OR CREEP SUCH THAT AVOIDANCE AS DESCRIBED IN PUBLIC RESOURCES CODE SECTION 2621.5(A) WOULD BE REQUIRED



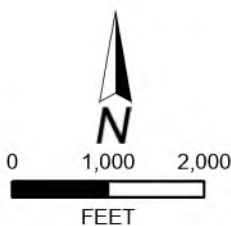
EARTHQUAKE-INDUCED LANDSLIDE ZONES

AREAS WHERE THE PREVIOUS OCCURRENCE OF LANDSLIDE MOVEMENT, OR LOCAL TOPOGRAPHIC, GEOLOGICAL, GEOTECHNICAL AND SUBSURFACE WATER CONDITIONS INDICATE A POTENTIAL FOR PERMANENT GROUND DISPLACEMENTS SUCH THAT MITIGATION AS DEFINED IN PUBLIC RESOURCES CODE SECTION 2693(C) WOULD BE REQUIRED.



LIQUEFACTION ZONE

AREAS WHERE THE HISTORICAL OCCURRENCE OF LIQUEFACTION, OR LOCAL GEOLOGICAL, GEOTECHNICAL AND GROUND WATER CONDITIONS INDICATE A POTENTIAL FOR PERMANENT GROUND DISPLACEMENTS SUCH THAT MITIGATION AS DEFINED IN PUBLIC RESOURCES CODE SECTION 2693(C) WOULD BE REQUIRED



BASEMAP SOURCE: ESRI MAPPING SERVICE
CALIFORNIA DEPARTMENT OF CONSERVATION, CALIFORNIA GEOLOGICAL SURVEY



SEISMIC HAZARDS ZONE MAP

INDUSTRIAL PARKWAY WEST
HAYWARD, CALIFORNIA

PROJECT NO. : 21194.000.001

SCALE: AS SHOWN

DRAWN BY: QRL

CHECKED BY: JAF

FIGURE NO.

5



APPENDIX A

CPT Report



Job No: 22-56-25168
Client: ENGEO Incorporated
Project: Industrial Park West
Start Date: 08-Dec-2022
End Date: 08-Dec-2022

CONE PENETRATION TEST SUMMARY

Sounding ID	File Name	Date	Cone	Cone Area (cm ²)	Assumed Phreatic Surface ¹ (ft)	Final Depth (ft)	Northing ²	Easting ²	Elevation ³ (ft)	Refer to Notation Number
1-CPT1	22-56-25168_CP01	08-Dec-2022	EC746:T1500F15U35	15	10.5	52.00	4163898	582420	14	4
1-CPT2	22-56-25168_CP02	08-Dec-2022	EC746:T1500F15U35	15	10.4	50.52	4163873	582447	13	
1-SCPT3	22-56-25168_SP03	08-Dec-2022	EC746:T1500F15U35	15	10.6	100.80	4163882	582544	13	

1. The assumed phreatic surface was based off the shallowest pore pressure dissipation tests performed within or nearest the sounding. Hydrostatic conditions were assumed for the calculated parameters.

2. The coordinates were collected using consumer grade GPS equipment. EPSG number: 32610 (WGS84 / UTM Zone 10S).

3. Elevations are referenced to the ground surface and were acquired from the Google Earth Elevation for the recorded coordinates.

4. The phreatic surface is based on the pore pressure dissipation test at nearby soundings.



ENGEO

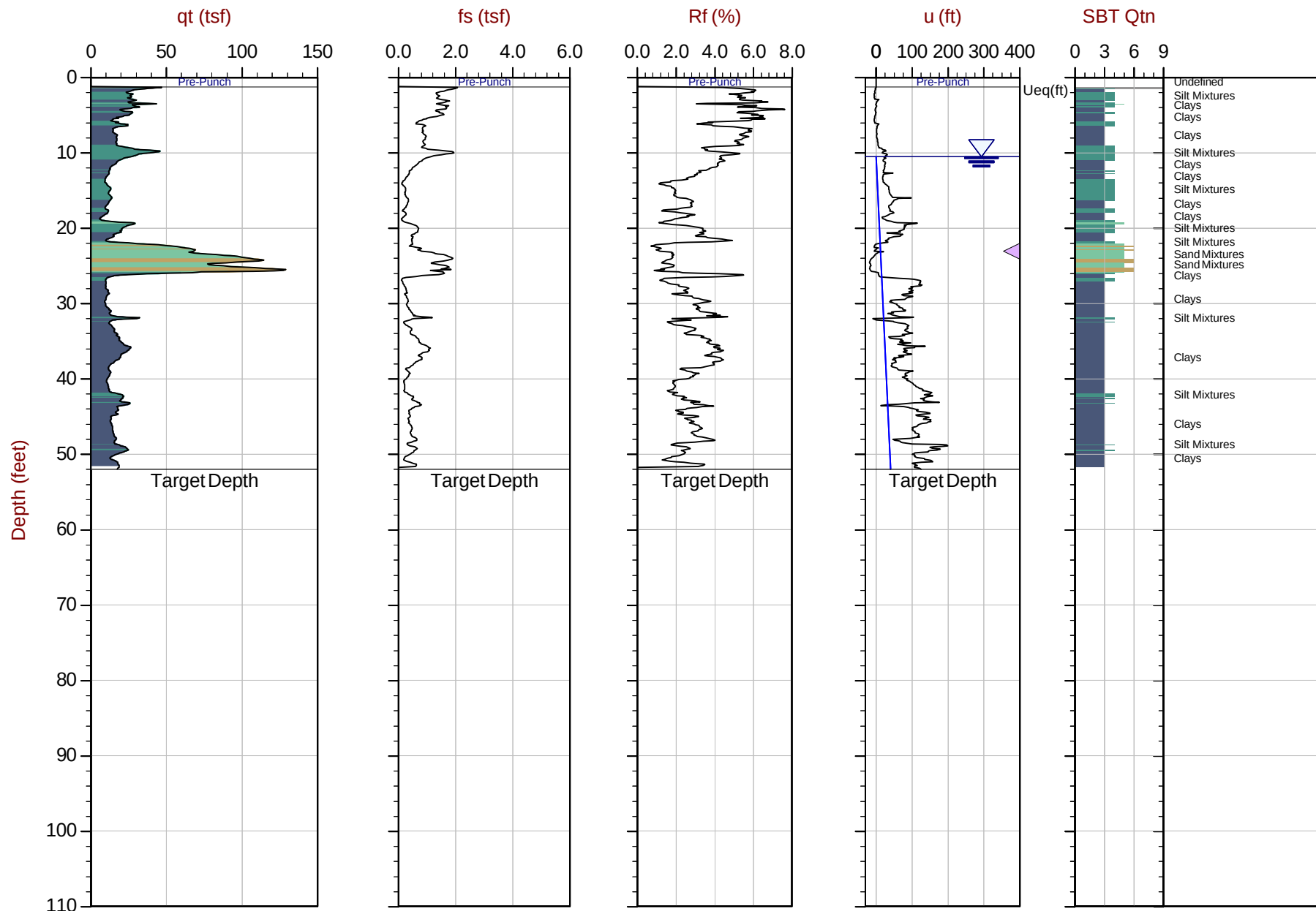
Job No: 22-56-25168

Date: 2022-12-08 11:34

Site: Industrial Parkway West

Sounding: 1-CPT1

Cone: 746:T1500F15U35



Max Depth: 15.850 m / 52.00 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 22-56-25168_CP01.COR
Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4163898m E: 582420m

● Equilibrium Pore Pressure (Ueq) ● Assumed Ueq ▲ Dissipation, Ueq achieved ▼ Dissipation, Ueq not achieved — Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



ENGEO

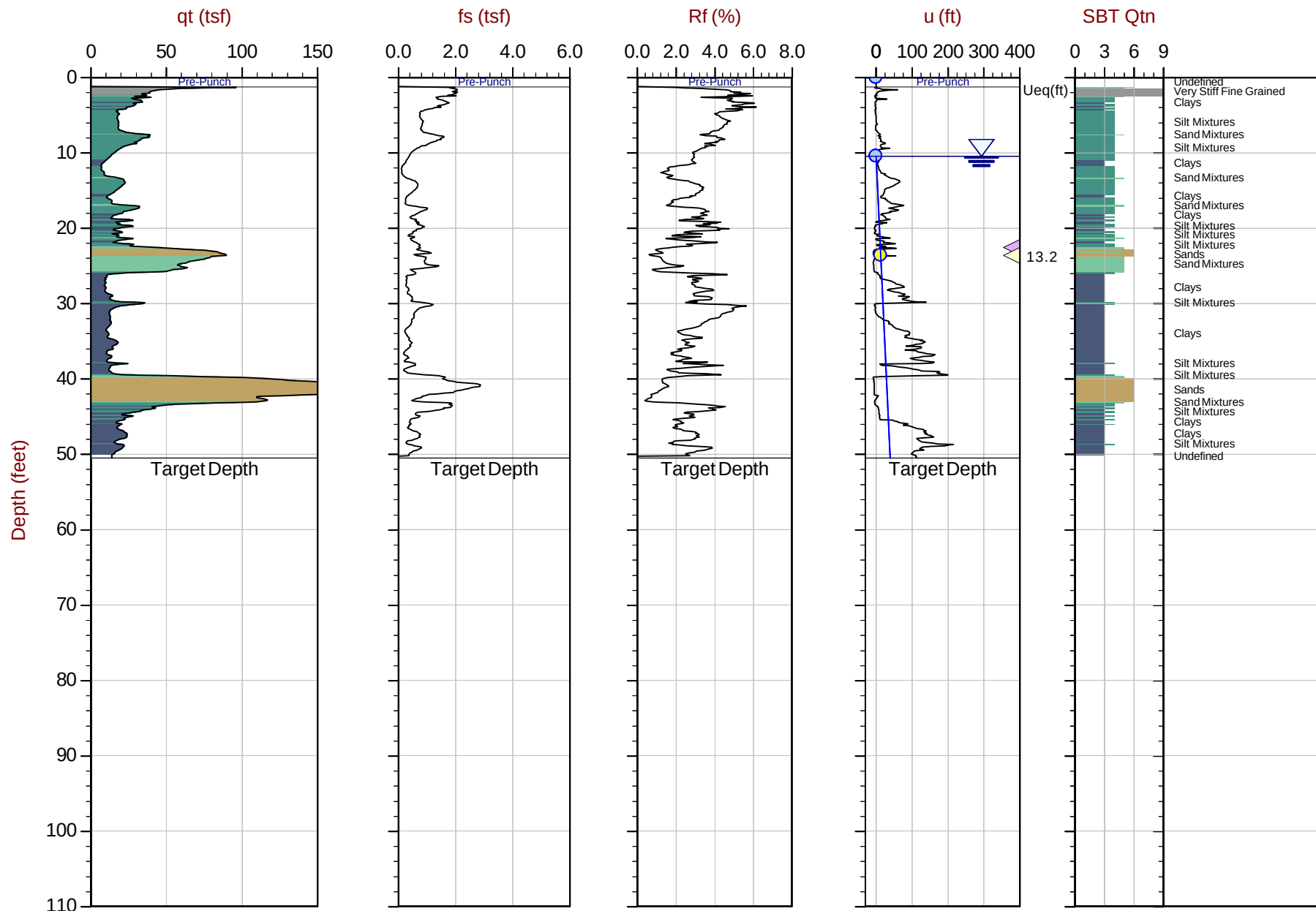
Job No: 22-56-25168

Date: 2022-12-08 09:43

Site: Industrial Parkway West

Sounding: 1-CPT2

Cone: 746:T1500F15U35



Max Depth: 15.400 m / 50.52 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 22-56-25168_CP02.COR
Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4163873m E: 582447m

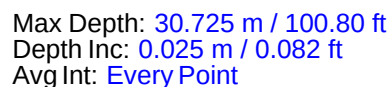
● Equilibrium Pore Pressure (Ueq) ● Assumed Ueq ▲ Dissipation, Ueq achieved ▼ Dissipation, Ueq not achieved — Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



Site: Industrial Parkway West

Cone: 746:T1500F15U35



File: 22-56-25168_SP03.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4163882m E: 582544m

● Equilibrium Pore Pressure (Ueq) ● Assumed Ueq ◀ Dissipation, Ueq achieved ◀ Dissipation, Ueq not achieved — Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

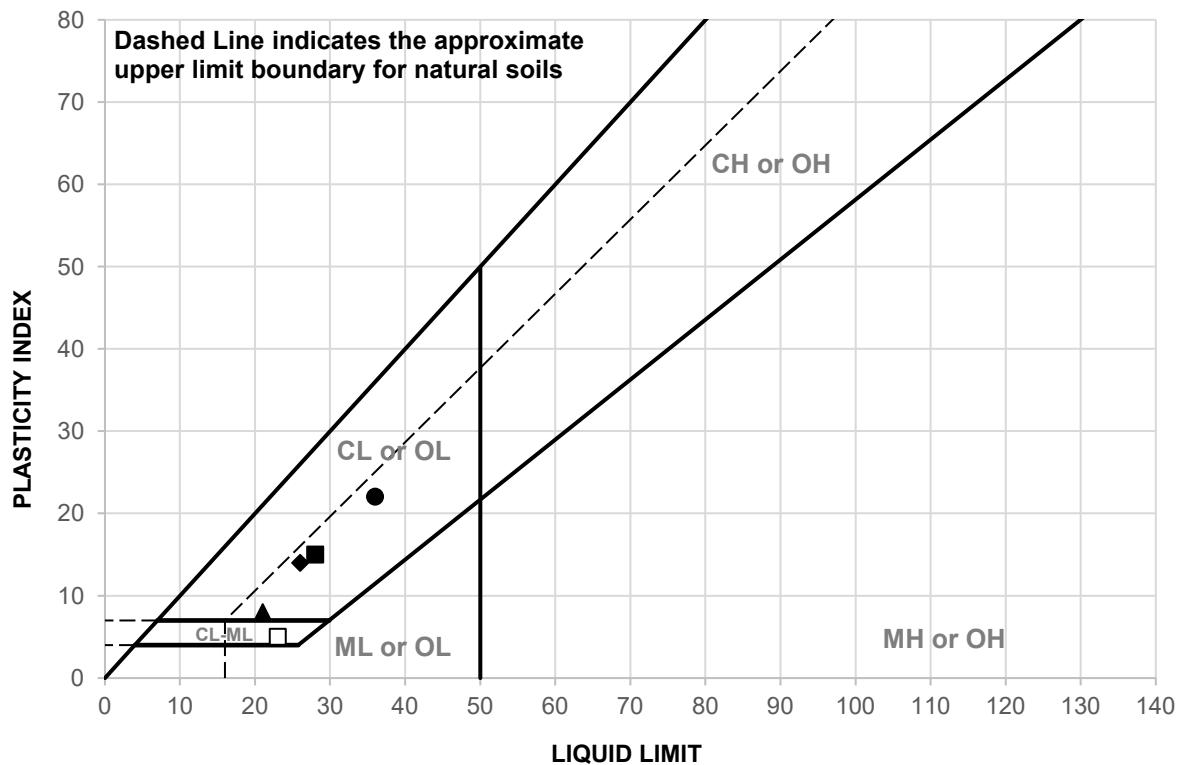


APPENDIX B

Laboratory Test Data

LIQUID AND PLASTIC LIMITS TEST REPORT

ASTM D4318



	SAMPLE ID	DEPTH (ft)	MATERIAL DESCRIPTION	LL	PL	PI
▲	1-DP1@18-18.5	18.0-18.5	See exploration logs	21	13	8
◆	1-DP1@20.5-21	20.5-21.0	See exploration logs	26	12	14
□	1-DP1@40.5-41	40.5-41.0	See exploration logs	23	18	5
●	1-DP2@13-13.5	13.0-13.5	See exploration logs	36	14	22
■	1-DP2@17-17.5	17.0-17.5	See exploration logs	28	13	15

	SAMPLE ID	TEST METHOD	REMARKS
▲	1-DP1@18-18.5	PI: ASTM D4318, Wet Method	
◆	1-DP1@20.5-21	PI: ASTM D4318, Wet Method	
□	1-DP1@40.5-41	PI: ASTM D4318, Wet Method	
●	1-DP2@13-13.5	PI: ASTM D4318, Wet Method	
■	1-DP2@17-17.5	PI: ASTM D4318, Wet Method	



CLIENT: CenterPoint Properties

PROJECT NAME: Industrial Parkway West

PROJECT NO: 21194.000.001 PH001

PROJECT LOCATION: Hayward, California

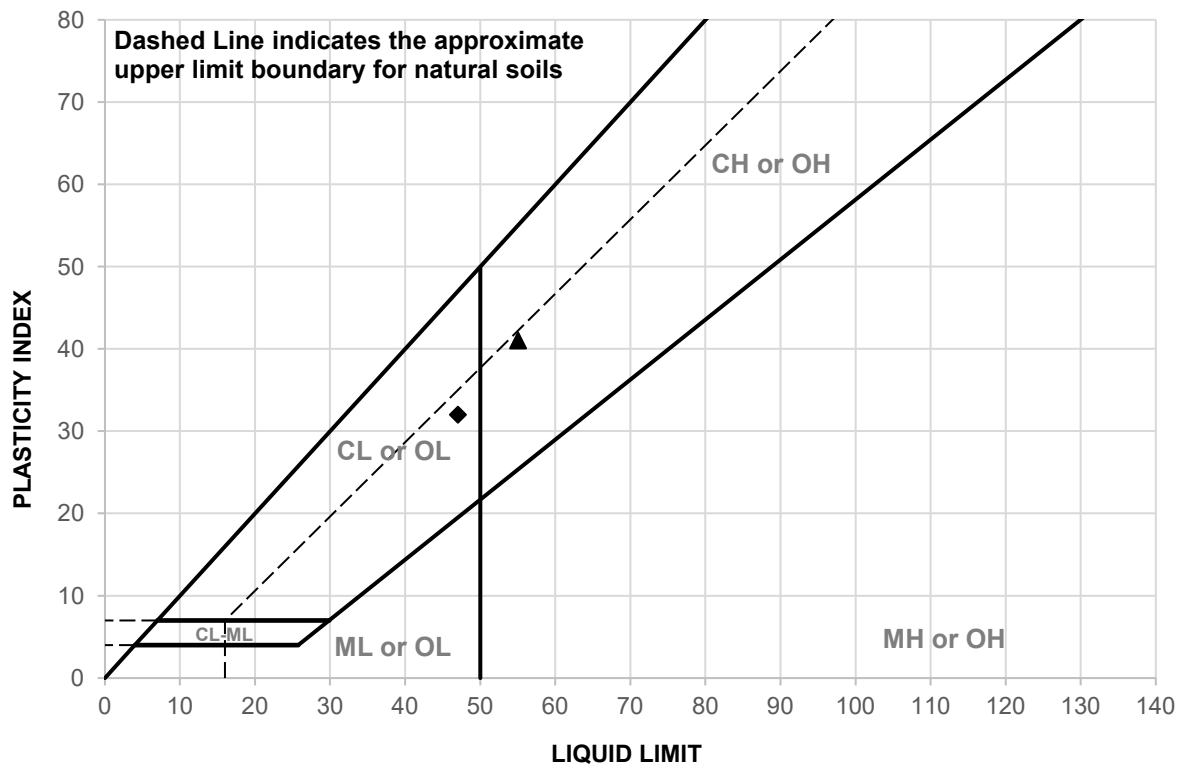
REPORT DATE: 1/4/2023

TESTED BY: G. Criste

REVIEWED BY: D. Seibold

LIQUID AND PLASTIC LIMITS TEST REPORT

ASTM D4318



	SAMPLE ID	DEPTH (ft)	MATERIAL DESCRIPTION	LL	PL	PI
▲	1-DP1@6	6	See exploration logs	55	14	41
◆	1-DP2@0.5	0.5	See exploration logs	47	15	32

	SAMPLE ID	TEST METHOD	REMARKS
▲	1-DP1@6	PI: ASTM D4318, Wet Method	
◆	1-DP2@0.5	PI: ASTM D4318, Wet Method	



CLIENT: CenterPoint Properties

PROJECT NAME: Industrial Parkway West

PROJECT NO: 21194.000.001 PH001

PROJECT LOCATION: Hayward, California

REPORT DATE: 1/20/2023

TESTED BY: G. Criste

REVIEWED BY: D. Seibold

MOISTURE CONTENT REPORT

ASTM D2216

SAMPLE ID	1-DP1@18-18.5	1-DP1@20.5-21	1-DP1@40.5-41	1-DP2@13-13.5	1-DP2@17-17.5			
DEPTH (ft.)	18-18.5	20.5-21	40.5-41	13-13.5	17-17.5			
METHOD A OR B	B	B	B	B	B			
MOISTURE CONTENT (%)	17.6	15.3	18.1	26.1	19.0			

SAMPLE ID								
DEPTH (ft.)								
METHOD A OR B								
MOISTURE CONTENT (%)								

SAMPLE ID								
DEPTH (ft.)								
METHOD A OR B								
MOISTURE CONTENT (%)								

SAMPLE ID								
DEPTH (ft.)								
METHOD A OR B								
MOISTURE CONTENT (%)								

SAMPLE ID								
DEPTH (ft.)								
METHOD A OR B								
MOISTURE CONTENT (%)								

SAMPLE ID								
DEPTH (ft.)								
METHOD A OR B								
MOISTURE CONTENT (%)								



CLIENT: CenterPoint Properties

PROJECT NAME: Industrial Parkway West

PROJECT NO: 21194.000.001 PH001

PROJECT LOCATION: Hayward, California

REPORT DATE: 1/3/2022

TESTED BY: M. Quasem

REVIEWED BY: W. Miller

