DOCKETED

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Document Title:	SVY03A SPPE Application Appendices C-E - Part IV of VI
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Submitter Role:	Applicant Representative
Submission Date:	9/14/2023 2:10:58 PM
Docketed Date:	9/14/2023

<u>APPENDIX C</u>

Arborist Report

EDEN LANDING RD
Arborist Report 2023

Prepared For:

Verity Commercial Chicago

Attn: Leo Hoerdemann

Site: 26062 EDEN LANDING ROAD
HAYWARD, CA 94545

Submitted by:

David Beckham
Certified Arborist
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Date: 7/7/2023
Updated: 8/10/2023

Attn: Verity Commercial Chicago
Site: 26062 Eden Landing Road, Hayward, CA 94545

Subject Re: Tree removal and replacement for new building construction at 26062 Eden Landing Road, Hayward, CA 94545

Dear Verity Commercial Chicago,
At your request, Kielty Arborists Services LLC has visited the property referenced above to evaluate the trees present with respect to the proposed construction project. The report below contains the analysis of the site visit.

SUMMARY

There are 50 trees on the property, 47 of which are protected (#1-18 & 20-48). All of the trees surveyed are proposed for removal to facilitate the proposed construction. 32 of the trees surveyed are in poor condition (#2-5, 8, 10, 13, & 25-48). Trees (#1, 6-7, 9, 11, 12, 14-16, 20-24, 49, and 50). No trees were given good or excellent tree condition ratings. A large grove of Blue Gum Eucalyptus trees were observed at the center of the property (#25-48). Recent eucalyptus tree failures were observed in the grove. All of the eucalyptus trees have been topped and are recommended to be removed regardless of the proposed construction as they are hazardous to the site. The site at Eden Landing Road looks to be in fair condition; various trees have not been well maintained in the past. The topography of the land is flat. All of the trees will need to be replaced per the appraised values of the trees. **(Picture showing topped eucalyptus trees)**

ASSIGNMENT

At the request of Verity Commercial Chicago, Kielty Arborists Services LLC conducted a site visit on 7/5/2023 to prepare a comprehensive Tree Inventory Report for the proposed construction project. This report is a requirement when submitting plans to the City of Hayward. The analysis in this report is based on the land title survey dated 3/8/21, and concept plan & design presentation dated 6/8/23 provided by Slack Infrastructure. .

The primary focus of this report is as follows:

- Identification and assessment of trees on the construction site that may be affected by the proposed development.
- Determination of potential impacts on tree health and stability, considering factors such as root damage and crown damage.
- Provision of recommendations for tree protection and preservation measures during the construction process to mitigate potential impacts.
- Ensuring compliance with local regulations pertaining to tree preservation, protection, and removal within the construction plans.

Please note that the report will provide specific details regarding tree assessments, impacts, and preservation measures.

INTRODUCTION

According to our past communications with city staff, the City of Hayward requires the following tree reporting elements for development projects:

1. Inventory of all trees measuring 4" in diameter or larger.
2. Map of tree locations.
3. Tree protection, removal, or replacement recommendations for all trees eight inches in diameter or greater than 54 inches above the ground; or certain native species that are four inches in diameter or greater. Appraised values for each tree are also to be provided.

LIMITS OF THE ASSIGNMENT

As part of this assessment, it is important to note that Kielty Arborist Services LLC did not conduct an aerial inspection of the upper crown, a detailed root crown inspection, or a plant tissue analysis on the subject trees. Therefore, the information presented in this report does not include data obtained from these specific methods.

Furthermore, it is essential to clarify that no tree risk assessments were completed as part of this report unless stated otherwise. The focus of this assessment primarily centers on tree identification, general health evaluation, and the potential impacts of the proposed construction.

While the absence of these specific assessments limits the scope of the analysis, the findings and recommendations provided within this report are based on available information and observations made during the site visit.

PURPOSE & USE OF THE REPORT

This report informs tree management decisions for the construction project and provides recommendations to maximize tree survival. It serves as a valuable resource for stakeholders, facilitating informed discussions and sustainable tree management practices.

TESTING & ANALYSIS

In order to assess the trees, a thorough examination was conducted using a variety of methods. For trees with accessible trunks, precise measurements of the Diameter at Breast Height (DBH) were taken using a specialized diameter tape measure. In cases where the trunks were not readily accessible, visual estimations were employed to determine the DBH. As part of the inventory process, all trees exceeding a specific DBH threshold of 4-inches were included.

To evaluate the health of the trees, multiple factors were considered, including their overall appearance and our team's extensive experiential knowledge of each species. This holistic approach ensured a comprehensive understanding of the trees' well-being.

To accurately document the location of each tree, a GPS smartphone application was utilized during the data collection process. This enabled us to create detailed maps that are included in this report. However, it is important to note that despite our efforts to minimize errors, inherent limitations of GPS data collection, coupled with slight discrepancies between GPS data and CAD drawings, may result in approximate tree locations depicted on the map.

To perform this assessment, a site visit was conducted on 7/5/2023. During this visit, meticulous observations and high-quality photographs were obtained to provide a comprehensive analysis. The findings and recommendations presented in this report are based on the construction plans titled "SVY03 Stack Hayward Data Center" by HKS Inc. These plans were electronically provided to us via email and are dated 6/8/2023. By thoroughly analyzing these plans in conjunction with our field observations, we have developed an accurate and reliable assessment of the tree conditions.

METHOD OF INSPECTION

The inspections were conducted from the ground without climbing the trees. No tissue samples or root crown inspections were performed. The trees under consideration were identified based on the provided site plan. To assess the trees, their diameter at 54 inches above ground level (DBH or diameter at breast height) was measured using a D-Tape. Additionally, the protected trees were evaluated for their health, structure, form, suitability for preservation with the following explanation of the ratings:

Tree Health Ratings:

- Good: The tree displays vigorous growth with normal-sized, shaped, and colored foliage. The canopy density is between 90-100%, with minimal to no dead wood, minor or no pest infestation, and little to no decay. The tree is expected to have a natural lifespan.
- Fair: The new growth shoots may be shorter than expected, and the canopy density ranges from 60-90%. Some small branch dieback, noticeable pest infestation, and/or decay may be present. Although the tree is not currently in decline, external factors such as construction impacts, increased pest pressure, or drought may affect its health.
- Poor: The tree exhibits little to no new growth and significant dieback. The foliage may be undersized, distorted, yellowed, or display abnormal colors. The canopy density is 20-60% or

less, with substantial dead wood, pest infestation, or decay. The tree is not expected to reach its natural lifespan.

Tree Structure Ratings:

- Good: Minor structural flaws can be addressed through pruning. The tree has an upright trunk with a single leader or can be easily trained to have one. Scaffold branches are smaller than the leader, attached to the trunk at angles approaching 45 degrees, and well-spaced vertically and radially. No included bark or signs of previous branch failures. Foliage is evenly distributed on the limbs, and the canopy is symmetrical or mostly symmetrical.
- Fair: Some structural flaws cannot be corrected through pruning. The tree may have multiple trunks or leaders, a slight lean, branches attached at angles less than 30 to 10 degrees, and/or crowding on the trunk. Included bark, previous branch failures, or end-heavy limbs may be present, and some asymmetry in the canopy may be observed.
- Poor: Significant structural flaws that cannot be addressed through pruning are evident. There may be significant dead wood or decay, multiple trunks or leaders, crowded branches on the trunk, significantly included bark, previous branch failures, and/or asymmetry. The tree may also exhibit a precipitous lean, indicating potential hazard.

Tree Form Ratings:

- Good: The tree's form is nearly ideal for its species, with minor asymmetries or deviations that do not compromise function or aesthetics. It aligns with the intended use and is consistent with the landscape.
- Fair: The tree's form displays major asymmetries or deviations from the species norm and/or intended use. This compromises function and/or aesthetics.
- Poor: The tree's form is largely asymmetric or abnormal, significantly detracting from the intended use and aesthetics. It is visually unappealing and provides little to no function in the landscape.

Suitability for Preservation (for protected trees only):

This rating is based solely on the tree itself, irrespective of potential construction impacts.

- Good: The tree is currently an asset to the landscape and can be expected to survive minor to moderate construction impacts with adequate protection.
- Fair: The tree contributes to the landscape and may benefit from pruning or other maintenance activities. It should survive minor construction impacts with adequate protection, and implementing protective measures is recommended unless construction impacts are extensive.
- Poor: The tree does not contribute to the landscape and is in poor health, potentially posing hazards. It is not expected to survive any construction impacts. Some trees with poor viability may be retained if they will not be impacted by construction.

Overall Condition Ratings:

The trees were assigned a condition rating based on a combination of existing tree health (50%) and tree structure (50%) using the following scale:

- 1-29: Very Poor
- 30-49: Poor
- 50-69: Fair
- 70-89: Good
- 90-100: Excellent

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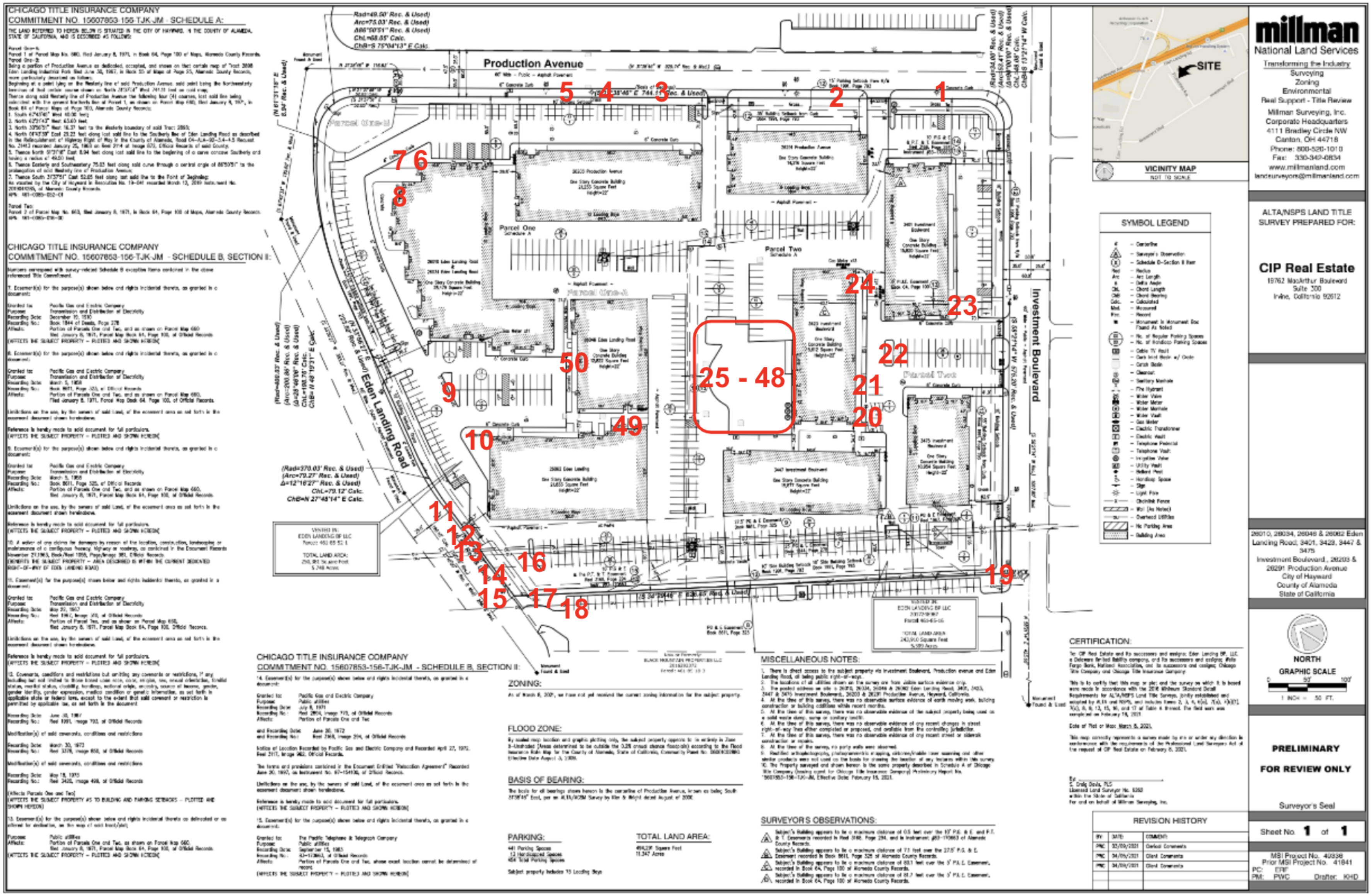
Tree Location Map



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OBSERVATIONS

Tree Tag #	Tree Picture #1	Common Name / Scientific Name	Trunk 1(in.)	Protected Tree	Preserve or Remove	Height (ft.) / Canopy Spread (ft.)	Health Rating	Structural Rating	Form Rating	Suitability for Preservation	Condition Rating (0-100%)	Appraised Value	Comments
1		RAYWOOD ASH ***Fraxinus angustifolia 'Raywood'***	21.3	Yes	(R)	40/35	Fair	Fair	Fair	Fair	55%	**$8,660**	Deadwood at top of canopy
2		RAYWOOD ASH ***Fraxinus angustifolia 'Raywood'***	22.3	Yes	(R)	40/30	Poor	Poor	Fair	Poor	30%	**$5,500**	Excessive deadwood at top of the canopy
3		RAYWOOD ASH ***Fraxinus angustifolia 'Raywood'***	16.7	Yes	(R)	35/30	Poor	Fair	Fair	Poor	45%	**$4,750**	Excessive deadwood at top of canopy
4		RAYWOOD ASH ***Fraxinus angustifolia 'Raywood'***	15.7	Yes	(R)	30/30	Poor	Fair	Fair	Poor	45%	**$4,290**	Excessive deadwood at top of canopy
5		RAYWOOD ASH ***Fraxinus angustifolia 'Raywood'***	19.5	Yes	(R)	40/30	Poor	Fair	Fair	Poor	30%	**$4,390**	Circling roots, excessive deadwood at top of canopy
6		COAST REDWOOD ***Sequoia sempervirens***	33.7	Yes	(R)	65/25	Good	Good	Good	Fair	60%	**$12,720**	Surrounded by hardscape, close to structure, small root area for species
7		COAST REDWOOD ***Sequoia sempervirens***	36.2	Yes	(R)	65/25	Good	Good	Good	Fair	60%	**$14,560**	Surrounded by hardscape, close to structure, small root area for species
8		COAST REDWOOD ***Sequoia sempervirens***	27.5	Yes	(R)	60/15	Poor	Poor	Fair	Poor	40%	**$5,040**	In decline, stressed, top of tree dead, excessive deadwood
9		EVERGREEN PEAR ***Pyrus kawakamii***	11	Yes	(R)	15/25	Good	Fair	Fair	Fair	65%	**$2,180**	Over extended, limb rust
10		COAST REDWOOD ***Sequoia sempervirens***	29.7	Yes	(R)	50/20	Fair	Poor	Fair	Fair	45%	**$6,360**	Top failed in past, close to structure
11		EVERGREEN PEAR ***Pyrus kawakamii***	14.1	Yes	(R)	20/25	Fair	Fair	Fair	Fair	60%	**$3,410**	Rust on leaves

Tree Tag #	Tree Picture #1	Common Name / Scientific Name	Trunk 1(in.)	Protected Tree	Preserve or Remove	Height (ft.) / Canopy Spread (ft.)	Health Rating	Structural Rating	Form Rating	Suitability for Preservation	Condition Rating (0-100%)	Appraised Value	Comments
12		EVERGREEN PEAR ***Pyrus kawakamii***	11.3	Yes	(R)	20/20	Fair	Fair	Fair	Fair	60%	**$2,480**	Rust on leaves
13		EVERGREEN PEAR ***Pyrus kawakamii***	10.7	Yes	(R)	20/15	Poor	Poor	Poor	Poor	40%	**$1,800**	Previous limb loss, deadwood
14		EVERGREEN PEAR ***Pyrus kawakamii***	12.3	Yes	(R)	15/15	Fair	Poor	Fair	Fair	60%	**$2,790**	Co-dominant at 8 ft, rust on leaves
15		EVERGREEN PEAR ***Pyrus kawakamii***	12.5	Yes	(R)	16/16	Fair	Fair	Fair	Fair	60%	**$2,850**	Rust on leaves
16		EVERGREEN PEAR ***Pyrus kawakamii***	8.8	Yes	(R)	12/15	Fair	Fair	Fair	Fair	60%	**$1,610**	Co-dominant at 6 feet, in small planting strip, rust on leaves
17*		BRADFORD PEAR ***Pyrus calleryana***	8.7	Yes	-	25/20	Good	Fair	Fair	Fair	65%	**-**	Co-dominant at 7 feet
18*		EVERGREEN PEAR ***Pyrus kawakamii***	11	Yes	-	12/20	Fair	Poor	Poor	Poor	30%	**-**	Fell in the past, laying on the ground, co-dominant at 1 ft
19*		PURPLE-LEAF PLUM ***Prunus cerasifera***	6.5	No	-	15/10	Fair	Poor	Fair	Fair	50%	**-**	Topped in past
20		POWHATAN CRAPE MYRTLE ***Lagerstroemia indica***	10.6	Yes	(R)	35/20	Good	Fair	Good	Fair	65%	**$2,330**	Surrounded by hardscape, close to buildings
21		POWHATAN CRAPE MYRTLE ***Lagerstroemia indica***	10.2	Yes	(R)	35/25	Good	Fair	Good	Fair	65%	**$2,220**	Surrounded by hardscape, close to buildings
22		POWHATAN CRAPE MYRTLE ***Lagerstroemia indica***	11.8	Yes	(R)	35/25	Good	Fair	Good	Fair	65%	**$2,700**	Surrounded by hardscape

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23		POWHATAN CRAPE MYRTLE ***Lagerstroemia indica***	11.8	Yes	(R)	18/12	Good	Fair	Good	Fair	65%	**$2,700**	Co-dominant at grade, close to building
24		POWHATAN CRAPE MYRTLE ***Lagerstroemia indica***	10.5	Yes	(R)	20/14	Good	Fair	Good	Fair	65%	**$2,300**	Co-dominant a grade, close to building
25		BLUE GUM ***Eucalyptus globulus***	31.2	Yes	(R)	60/20	Poor	Poor	Poor	Poor	30%	**$2,840**	Topped in past, species high risk of future limb failure
26		BLUE GUM ***Eucalyptus globulus***	26.2	Yes	(R)	60/20	Poor	Poor	Poor	Poor	30%	**$2,240**	Topped in past, species high risk of future limb failure
27		BLUE GUM ***Eucalyptus globulus***	50.4	Yes	(R)	60/20	Poor	Poor	Poor	Poor	30%	**$6,130**	Topped in past, species high risk of future limb failure
28		BLUE GUM ***Eucalyptus globulus***	36.7	Yes	(R)	60/20	Poor	Poor	Poor	Poor	30%	**$3,630**	Topped in past, species high risk of future limb failure
29		BLUE GUM ***Eucalyptus globulus***	41.2	Yes	(R)	60/20	Poor	Poor	Poor	Poor	30%	**$4,360**	Topped in past, species high risk of future limb failure
30		BLUE GUM ***Eucalyptus globulus***	26.7	Yes	(R)	60/20	Poor	Poor	Poor	Poor	30%	**$2,300**	Topped in past, species high risk of future limb failure
31		BLUE GUM ***Eucalyptus globulus***	25	Yes	(R)	60/20	Poor	Poor	Poor	Poor	30%	**$2,110**	Topped in past, species high risk of future limb failure
32		BLUE GUM ***Eucalyptus globulus***	23.1	Yes	(R)	60/20	Poor	Poor	Poor	Poor	30%	**$1,920**	Topped in past, species high risk of future limb failure
34		BLUE GUM ***Eucalyptus globulus***	31	Yes	(R)	60/20	Poor	Poor	Poor	Poor	30%	**$2,820**	Topped in past, species high risk of future limb failure

Tree Tag #	Tree Picture #1	Common Name / Scientific Name	Trunk 1(in.)	Protected Tree	Preserve or Remove	Height (ft.) / Canopy Spread (ft.)	Health Rating	Structural Rating	Form Rating	Suitability for Preservation	Condition Rating (0-100%)	Appraised Value	Comments
33		BLUE GUM ***Eucalyptus globulus***	26.1	Yes	(R)	60/20	Poor	Poor	Poor	Poor	30%	**$2,230**	Topped in past, species high risk of future limb failure
35		BLUE GUM ***Eucalyptus globulus***	23.2	Yes	(R)	60/20	Poor	Poor	Poor	Poor	30%	**$1,930**	Topped in past, species high risk of future limb failure
36		BLUE GUM ***Eucalyptus globulus***	26	Yes	(R)	60/20	Poor	Poor	Poor	Poor	30%	**$2,220**	Topped in past, species high risk of future limb failure
37		BLUE GUM ***Eucalyptus globulus***	29.7	Yes	(R)	60/20	Poor	Poor	Poor	Poor	30%	**$2,650**	Topped in past, species high risk of future limb failure
38		BLUE GUM ***Eucalyptus globulus***	30.2	Yes	(R)	60/20	Poor	Poor	Poor	Poor	30%	**$2,720**	Topped in past, species high risk of future limb failure
39		BLUE GUM ***Eucalyptus globulus***	35.7	Yes	(R)	60/20	Poor	Poor	Poor	Poor	30%	**$3,480**	Topped in past, species high risk of future limb failure
40		BLUE GUM ***Eucalyptus globulus***	21.2	Yes	(R)	60/20	Poor	Poor	Poor	Poor	30%	**$1,740**	Topped in past, species high risk of future limb failure
41		BLUE GUM ***Eucalyptus globulus***	24.4	Yes	(R)	60/20	Poor	Poor	Poor	Poor	30%	**$2,050**	Topped in past, species high risk of future limb failure
42		BLUE GUM ***Eucalyptus globulus***	35	Yes	(R)	60/20	Poor	Poor	Poor	Poor	30%	**$3,370**	Topped in past, species high risk of future limb failure
43		BLUE GUM ***Eucalyptus globulus***	25	Yes	(R)	60/20	Poor	Poor	Poor	Poor	30%	**$2,110**	Topped in past, species high risk of future limb failure
44		BLUE GUM ***Eucalyptus globulus***	24.7	Yes	(R)	60/20	Poor	Poor	Poor	Poor	30%	**$2,080**	Topped in past, species high risk of future limb failure

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Tree Tag #	Tree Picture #1	Common Name / Scientific Name	Trunk 1(in.)	Protected Tree	Preserve or Remove	Height (ft.) / Canopy Spread (ft.)	Health Rating	Structural Rating	Form Rating	Suitability for Preservation	Condition Rating (0-100%)	Appraised Value	Comments
45		BLUE GUM ***Eucalyptus globulus***	32.3	Yes	(R)	60/20	Poor	Poor	Poor	Poor	30%	**$2,990**	Topped in past, species high risk of future limb failure
46		BLUE GUM ***Eucalyptus globulus***	49	Yes	(R)	60/20	Poor	Poor	Poor	Poor	30%	**$5,840**	Topped in past, species high risk of future limb failure
47		BLUE GUM ***Eucalyptus globulus***	33.1	Yes	(R)	60/20	Poor	Poor	Poor	Poor	30%	**$3,100**	Topped in past, species high risk of future limb failure
48		BLUE GUM ***Eucalyptus globulus***	34.7	Yes	(R)	60/20	Poor	Poor	Poor	Poor	30%	**$3,330**	Topped in past, species high risk of future limb failure
49		POWHATAN CRAPE MYRTLE ***Lagerstroemia indica***	5.4	No	(R)	20/12	Good	Fair	Fair	Fair	65%	**$1,210**	Close to building, surrounded by hardscape
50		POWHATAN CRAPE MYRTLE ***Lagerstroemia indica***	4.1	No	(R)	12/10	Good	Fair	Fair	Fair	65%	**$1,040**	Close to building, surrounded by hardscape

Species List:
50 trees were surveyed on this property. The surveyed species are comprised of the following:

- Raywood ash, redwood, evergreen pear, Bradford pear, purple-leaf plum, crepe myrtle, and blue gum.

Tree Removal For Proposed Development:
'protected' Size Trees: Total = 47 (#1-16, 20-48)
'unprotected' Size Trees: Total = 3 (#49, 50)

Heritage And Protected Trees:

As defined by the City Of Hayward Municipal Code: *SEC. 10-15.13 - PROTECTED TREES. The following trees, when located on properties to which this Ordinance applies as set forth in Section 10-15.11 above, shall be Protected Trees:*

1.Trees having a minimum trunk diameter of eight inches measured 54" above the ground. When measuring a multi-trunk tree, the diameters of the largest three trunks shall be added together.

2.Street trees or other required trees such as those required as a condition of approval, Use Permit, or other Zoning requirement, regardless of size.

3.All memorial trees dedicated by an entity recognized by the City, and all specimen trees that define a neighborhood or community.

4.Trees of the following species that have reached a minimum of four inches diameter trunk size:
a.Big Leaf Maple Acer macrophyllum
b.California Buckeye Aesculus californica
c.Madrone Arbutus menziesii
d.Western Dogwood Cornus nuttallii
e.California Sycamore Platanus racemosa
f.Coast Live Oak Quercus agrifolia
G. Canyon Live Oak Quercus chrysolepis
h.Blue Oak Quercus douglassii
I. Oregon White Oak Quercus garryana
J. California Black Oak Quercus kelloggi
K. Valley Oak Quercus lobata
L. Interior Live Oak Quercus wislizenii
m.California Bay Umbellularia californica

5.A tree or trees of any size planted as a replacement for a Protected Tree. Trees located on a developed single-family residential lot that cannot be further subdivided are exempt unless they have been required or protected as a condition of approval.

Removed Trees Replacement Program:

As per City of Hayward Landscape Design Checklist:

•All trees and large shrubs on the site should be shown on a salvage/demolition plan. Trees to be preserved, trimmed, or removed must be indicated on the plan. Trees in good health that are proposed to be removed shall be replaced with a tree of equal size and value.
Comment: Indicate location, trunk diameter, species, and approximate dripline of trees. Retain significant trees and native vegetation that are in good condition, and avoid grading and paving within the dripline of the trees. The City Landscape Architect may require an arborist report.

•Tree protection measures shall be noted on the grading, site, and landscaping plans, if applicable.
Comment: See below for recommended minimum tree protection measures.

•A Tree removal permit must be obtained prior to removing any tree designated as a protected tree.
Comment: Replacement trees are typically required for trees authorized for removal, which will be specified by City Landscape Architect based on condition, size, species, and location of tree(s) to be removed. Show required replacement trees on planting plan.

•Street Trees – minimum one 24" box tree provided for every 20 to 40 lineal feet of street frontage, depending on tree species and as directed by City Landscape Architect.

•Parking Lot Landscaping – minimum one 15-gallon tree for every six parking stalls; tree wells or landscape medians minimum 5' wide; parking rows capped with landscape medians.

Tree mitigation summary chart:

The Reproduction Cost Method, Trunk Formula Technique was used for this section. This methodology was taken from the Guide For Plant Appraisal 10th Edition, by The Council Of Tree & Landscape Appraisers. This methodology is widely used for tree inventories, preconstruction, bonding, and some insurance claims. Reproduction cost is the cost to replicate or duplicate the item being appraised. Generally, this means estimating the cost of replacing the landscape item with one that is close to identical and thereby providing most or all of the characteristics and benefits of the original. When depreciation is applied to a reproduction cost, the result is termed a depreciated reproduction cost.

Depreciated Reproduction Cost:

Detailed charts listed on pp. 16-18 of this document. Total reproduction cost of trees being removed = $168,080

Project Features:

New construction of a data center and associated power facilities is proposed. Site plans SVY03 Stack Hayward Data Center, pp. #1-7, dated 6/8/2023, and Alta Survey - Eden Landing, sheet 1 of 1, dated 2/1/2021 were reviewed for the initial tree survey. All trees present on the construction site are proposed for removal.

CONCLUSIONS

Given that all trees are being removed to facilitate the proposed construction, it is necessary to replace trees that fall within the *Landscape Design Checklist,* as defined by the City of Hayward. By doing so, the developer will be in compliance with ordinance set forth by the City of Hayward. Following these steps ensure the property owners can maintain a healthy tree population that will add value to the property and benefit the environment.

ASSUMPTIONS AND LIMITING CONDITIONS

- **Legal Descriptions and Titles:** The consultant/arborist assumes the accuracy of any legal description and titles provided. No responsibility is assumed for any legal due diligence. The consultant/arborist shall not be held liable for any discrepancies or issues arising from incorrect legal descriptions or faulty titles.
- **Compliance with Laws and Regulations:** The property is assumed to be in compliance with all applicable codes, ordinances, statutes, or other government regulations. The consultant/arborist is not responsible for identifying or rectifying any non-compliance.
- **Reliability of Information:** Though diligent efforts have been made to obtain and verify information, the consultant/arborist is not responsible for inaccuracies or incomplete data provided by external sources. The client accepts full responsibility for any decisions or actions taken based on this data.
- **Testimony or Court Attendance:** The consultant/arborist has no obligation to provide testimony or attend court regarding this report unless mutually agreed upon through separate written agreements, which may incur additional fees.
- **Report Integrity:** Unauthorized alteration, loss, or reproduction of this report renders it invalid. The consultant/arborist shall not be liable for any interpretations or conclusions made from altered reports.
- **Restricted Publication and Use:** This report is exclusively for the use of the original client. Any other use or dissemination, without prior written consent from the consultant/arborist, is strictly prohibited.
- **Non-disclosure to Public Media:** The client is prohibited from using any content of this report, including the consultant/arborist's identity, in any public communication without prior written consent.
- **Opinion-based Report:** The report represents the independent, professional judgment of the consultant/arborist. The fee is not contingent upon any pre-determined outcomes, values, or events.
- **Visual Aids Limitation:** Visual aids are for illustrative purposes and should not be considered precise representations. They are not substitutes for formal engineering, architectural, or survey reports.
- **Inspection Limitations:** The consultant/arborist's inspection is limited to visible and accessible components. Non-invasive methods are used. There is no warranty or guarantee that problems will not develop in the future.

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ARBORIST DISCLOSURE STATEMENT

Arborists specialize in the assessment and care of trees using their education, knowledge, training, and experience.

- **Limitations of Tree Assessment:** Arborists cannot guarantee the detection of all conditions that could compromise a tree's structure or health. The consultant/arborist makes no warranties regarding the future condition of trees and shall not be liable for any incidents or damages resulting from tree failures.
- **Remedial Treatments Uncertainty:** Remedial treatments for trees have variable outcomes and cannot be guaranteed.
- **Considerations Beyond Scope:** The consultant/arborist's services are confined to tree assessment and care. The client assumes responsibility for matters involving property boundaries, ownership, disputes, and other non-arboricultural considerations.
- **Inherent Risks:** Living near trees inherently involves risks. The consultant/arborist is not responsible for any incidents or damages arising from such risks.
- **Client's Responsibility:** The client is responsible for considering the information and recommendations provided by the consultant/arborist and for any decisions made or actions taken.

The client acknowledges and accepts these Assumptions and Limiting Conditions and Arborist Disclosure Statement, recognizing that reliance upon this report is at their own risk. The consultant/arborist disclaims all warranties, express or implied.

CERTIFICATION

I hereby certify that all the statements of fact in this report are true, complete, and correct to the best of my knowledge and belief, and are made in good faith

David Beckham

INTERNATIONAL SOCIETY OF ARBORICULTURE
DAVID BECKHAM
WE-10724A
CERTIFIED ARBORIST

David Beckham - July 7th, 2023

Signature of Consultant

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Tree Tag #	Common Name / Scientific Name	Appraised Value	Diameter	Condition rating	Functional Limitations Rating (0-100%)	External Limitations Rating (0-100%)	Cross-sectional area (Diameter)2 × 0.7854	Replacement Tree Cost	Unit tree cost (Replacement Tree Cost / RPAC)	RPAC	Basic Reproduction Cost (Cross-sectional area × Unit tree cost)	Depreciated reproduction cost (Basic reproduction cost × Condition rating × Functional limitations × External limitations)	Total additional costs (Cleanup & Replacement tree installation)	Total reproduction cost (Depreciated reproduction cost + Total additional costs)	Comments
1	RAYWOOD ASH ***Fraxinus angustifolia 'Raywood'***	**$8,660**	21.3	55.00%	60.00%	60.00%	356.33	$350	$111	3.14	$39,698	$7,860	$800.00	$8,660	Deadwood at top of canopy
2	RAYWOOD ASH ***Fraxinus angustifolia 'Raywood'***	**$5,500**	22.3	30.00%	60.00%	60.00%	390.57	$350	$111	3.14	$43,513	$4,699	$800.00	$5,499	Excessive deadwood at top of the canopy
3	RAYWOOD ASH ***Fraxinus angustifolia 'Raywood'***	**$4,750**	16.7	45.00%	60.00%	60.00%	219.04	$350	$111	3.14	$24,403	$3,953	$800.00	$4,753	Excessive deadwood at top of canopy
4	RAYWOOD ASH ***Fraxinus angustifolia 'Raywood'***	**$4,290**	15.7	45.00%	60.00%	60.00%	193.59	$350	$111	3.14	$21,568	$3,494	$800.00	$4,294	Excessive deadwood at top of canopy
5	RAYWOOD ASH ***Fraxinus angustifolia 'Raywood'***	**$4,390**	19.5	30.00%	60.00%	60.00%	298.65	$350	$111	3.14	$33,272	$3,593	$800.00	$4,393	Circling roots, excessive deadwood at top of canopy
6	COAST REDWOOD ***Sequoia sempervirens***	**$12,720**	33.7	60.00%	40.00%	50.00%	891.97	$350	$111	3.14	$99,373	$11,925	$800.00	$12,725	Surrounded by hardscape, close to structure, small root area for species
7	COAST REDWOOD ***Sequoia sempervirens***	**$14,560**	36.2	60.00%	40.00%	50.00%	1029.22	$350	$111	3.14	$114,664	$13,760	$800.00	$14,560	Surrounded by hardscape, close to structure, small root area for species
8	COAST REDWOOD ***Sequoia sempervirens***	**$5,040**	27.5	40.00%	40.00%	40.00%	593.96	$350	$111	3.14	$66,172	$4,235	$800.00	$5,035	In decline, stressed, top of tree dead, excessive deadwood
9	EVERGREEN PEAR ***Pyrus kawakamii***	**$2,180**	11	65.00%	40.00%	50.00%	95.03	$350	$111	3.14	$10,588	$1,376	$800.00	$2,176	Over extended, limb rust
10	COAST REDWOOD ***Sequoia sempervirens***	**$6,360**	29.7	45.00%	40.00%	40.00%	692.79	$350	$111	3.14	$77,183	$5,557	$800.00	$6,357	Top failed in past, close to structure
11	EVERGREEN PEAR ***Pyrus kawakamii***	**$3,410**	14.1	60.00%	50.00%	50.00%	156.15	$350	$111	3.14	$17,396	$2,609	$800.00	$3,409	Rust on leaves
12	EVERGREEN PEAR ***Pyrus kawakamii***	**$2,480**	11.3	60.00%	50.00%	50.00%	100.29	$350	$111	3.14	$11,173	$1,676	$800.00	$2,476	Rust on leaves
13	EVERGREEN PEAR ***Pyrus kawakamii***	**$1,800**	10.7	40.00%	50.00%	50.00%	89.92	$350	$111	3.14	$10,018	$1,002	$800.00	$1,802	Previous limb loss, deadwood
14	EVERGREEN PEAR ***Pyrus kawakamii***	**$2,790**	12.3	60.00%	50.00%	50.00%	118.82	$350	$111	3.14	$13,238	$1,986	$800.00	$2,786	Co-dominant at 8 ft, rust on leaves
15	EVERGREEN PEAR ***Pyrus kawakamii***	**$2,850**	12.5	60.00%	50.00%	50.00%	122.72	$350	$111	3.14	$13,672	$2,051	$800.00	$2,851	Rust on leaves
16	EVERGREEN PEAR ***Pyrus kawakamii***	**$1,610**	8.8	60.00%	40.00%	50.00%	60.82	$350	$111	3.14	$6,776	$813	$800.00	$1,613	Co-dominant at 6 feet, in small planting strip, rust on leaves
17*	BRADFORD PEAR ***Pyrus calleryana***	-	8.7	-	-	-	-	-	-	-	-	-	-	-	Co-dominant at 7 feet
18*	EVERGREEN PEAR ***Pyrus kawakamii***	-	11	-	-	-	-	-	-	-	-	-	-	-	Fell in the past, laying on the ground, co-dominant at 1 ft
19*	PURPLE-LEAF PLUM ***Prunus cerasifera***	-	6.5	-	-	-	-	-	-	-	-	-	-	-	Topped in past
20	POWHATAN CRAPE MYRTLE ***Lagerstroemia indica***	**$2,330**	10.6	65.00%	40.00%	60.00%	88.25	$350	$111	3.14	$9,832	$1,534	$800.00	$2,334	Surrounded by hardscape, close to buildings
21	POWHATAN CRAPE MYRTLE ***Lagerstroemia indica***	**$2,220**	10.2	65.00%	40.00%	60.00%	81.71	$350	$111	3.14	$9,104	$1,420	$800.00	$2,220	Surrounded by hardscape, close to buildings
22	POWHATAN CRAPE MYRTLE ***Lagerstroemia indica***	**$2,700**	11.8	65.00%	40.00%	60.00%	109.36	$350	$111	3.14	$12,184	$1,901	$800.00	$2,701	Surrounded by hardscape
23	POWHATAN CRAPE MYRTLE ***Lagerstroemia indica***	**$2,700**	11.8	65.00%	40.00%	60.00%	109.36	$350	$111	3.14	$12,184	$1,901	$800.00	$2,701	Co-dominant at grade, close to building

Tree Tag #	Common Name / Scientific Name	Appraised Value	Diameter	Condition rating	Functional Limitations Rating (0-100%)	External Limitations Rating (0-100%)	Cross-sectional area (Diameter)2 × 0.7854	Replacement Tree Cost	Unit tree cost (Replacement Tree Cost / RPAC)	RPAC	Basic Reproduction Cost (Cross-sectional area × Unit tree cost)	Depreciated reproduction cost (Basic reproduction cost × Condition rating × Functional limitations × External limitations)	Total additional costs (Cleanup & Replacement tree installation)	Total reproduction cost (Depreciated reproduction cost + Total additional costs)	Comments
24	POWHATAN CRAPE MYRTLE ***Lagerstroemia indica***	**$2,300**	10.5	65.00%	40.00%	60.00%	86.59	$350	$111	3.14	$9,647	$1,505	$800.00	$2,305	Co-dominant a grade, close to building
25	BLUE GUM ***Eucalyptus globulus***	**$2,840**	31.2	30.00%	20.00%	40.00%	764.54	$350	$111	3.14	$85,176	$2,044	$800.00	$2,844	Topped in past, species high risk of future limb failure
26	BLUE GUM ***Eucalyptus globulus***	**$2,240**	26.2	30.00%	20.00%	40.00%	539.13	$350	$111	3.14	$60,064	$1,442	$800.00	$2,242	Topped in past, species high risk of future limb failure
27	BLUE GUM ***Eucalyptus globulus***	**$6,130**	50.4	30.00%	20.00%	40.00%	1995.04	$350	$111	3.14	$222,264	$5,334	$800.00	$6,134	Topped in past, species high risk of future limb failure
28	BLUE GUM ***Eucalyptus globulus***	**$3,630**	36.7	30.00%	20.00%	40.00%	1057.85	$350	$111	3.14	$117,853	$2,828	$800.00	$3,628	Topped in past, species high risk of future limb failure
29	BLUE GUM ***Eucalyptus globulus***	**$4,360**	41.2	30.00%	20.00%	40.00%	1333.17	$350	$111	3.14	$148,526	$3,565	$800.00	$4,365	Topped in past, species high risk of future limb failure
30	BLUE GUM ***Eucalyptus globulus***	**$2,300**	26.7	30.00%	20.00%	40.00%	559.90	$350	$111	3.14	$62,378	$1,497	$800.00	$2,297	Topped in past, species high risk of future limb failure
31	BLUE GUM ***Eucalyptus globulus***	**$2,110**	25	30.00%	20.00%	40.00%	490.88	$350	$111	3.14	$54,688	$1,313	$800.00	$2,113	Topped in past, species high risk of future limb failure
32	BLUE GUM ***Eucalyptus globulus***	**$1,920**	23.1	30.00%	20.00%	40.00%	419.10	$350	$111	3.14	$46,691	$1,121	$800.00	$1,921	Topped in past, species high risk of future limb failure
34	BLUE GUM ***Eucalyptus globulus***	**$2,820**	31	30.00%	20.00%	40.00%	754.77	$350	$111	3.14	$84,088	$2,018	$800.00	$2,818	Topped in past, species high risk of future limb failure
33	BLUE GUM ***Eucalyptus globulus***	**$2,230**	26.1	30.00%	20.00%	40.00%	535.02	$350	$111	3.14	$59,606	$1,431	$800.00	$2,231	Topped in past, species high risk of future limb failure
35	BLUE GUM ***Eucalyptus globulus***	**$1,930**	23.2	30.00%	20.00%	40.00%	422.73	$350	$111	3.14	$47,096	$1,130	$800.00	$1,930	Topped in past, species high risk of future limb failure
36	BLUE GUM ***Eucalyptus globulus***	**$2,220**	26	30.00%	20.00%	40.00%	530.93	$350	$111	3.14	$59,150	$1,420	$800.00	$2,220	Topped in past, species high risk of future limb failure
37	BLUE GUM ***Eucalyptus globulus***	**$2,650**	29.7	30.00%	20.00%	40.00%	692.79	$350	$111	3.14	$77,183	$1,852	$800.00	$2,652	Topped in past, species high risk of future limb failure
38	BLUE GUM ***Eucalyptus globulus***	**$2,720**	30.2	30.00%	20.00%	40.00%	716.32	$350	$111	3.14	$79,804	$1,915	$800.00	$2,715	Topped in past, species high risk of future limb failure
39	BLUE GUM ***Eucalyptus globulus***	**$3,480**	35.7	30.00%	20.00%	40.00%	1000.98	$350	$111	3.14	$111,518	$2,676	$800.00	$3,476	Topped in past, species high risk of future limb failure
40	BLUE GUM ***Eucalyptus globulus***	**$1,740**	21.2	30.00%	20.00%	40.00%	352.99	$350	$111	3.14	$39,326	$944	$800.00	$1,744	Topped in past, species high risk of future limb failure
41	BLUE GUM ***Eucalyptus globulus***	**$2,050**	24.4	30.00%	20.00%	40.00%	467.60	$350	$111	3.14	$52,094	$1,250	$800.00	$2,050	Topped in past, species high risk of future limb failure
42	BLUE GUM ***Eucalyptus globulus***	**$3,370**	35	30.00%	20.00%	40.00%	962.12	$350	$111	3.14	$107,188	$2,573	$800.00	$3,373	Topped in past, species high risk of future limb failure
43	BLUE GUM ***Eucalyptus globulus***	**$2,110**	25	30.00%	20.00%	40.00%	490.88	$350	$111	3.14	$54,688	$1,313	$800.00	$2,113	Topped in past, species high risk of future limb failure
44	BLUE GUM ***Eucalyptus globulus***	**$2,080**	24.7	30.00%	20.00%	40.00%	479.16	$350	$111	3.14	$53,383	$1,281	$800.00	$2,081	Topped in past, species high risk of future limb failure
45	BLUE GUM ***Eucalyptus globulus***	**$2,990**	32.3	30.00%	20.00%	40.00%	819.40	$350	$111	3.14	$91,288	$2,191	$800.00	$2,991	Topped in past, species high risk of future limb failure
46	BLUE GUM ***Eucalyptus globulus***	**$5,840**	49	30.00%	20.00%	40.00%	1885.75	$350	$111	3.14	$210,088	$5,042	$800.00	$5,842	Topped in past, species high risk of future limb failure

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47	BLUE GUM ***Eucalyptus globulus***	**$3,100**	33.1	30.00%	20.00%	40.00%	860.49	$350	$111	3.14	$95,866	$2,301	$800.00	$3,101	Topped in past, species high risk of future limb failure
48	BLUE GUM ***Eucalyptus globulus***	**$3,330**	34.7	30.00%	20.00%	40.00%	945.69	$350	$111	3.14	$105,358	$2,529	$800.00	$3,329	Topped in past, species high risk of future limb failure
49	POWHATAN CRAPE MYRTLE ***Lagerstroemia indica***	**$1,210**	5.4	65.00%	50.00%	50.00%	22.90	$350	$111	3.14	$2,552	$415	$800.00	$1,215	Close to building, surrounded by hardscape
50	POWHATAN CRAPE MYRTLE ***Lagerstroemia indica***	**$1,040**	4.1	65.00%	50.00%	50.00%	13.20	$350	$111	3.14	$1,471	$239	$800.00	$1,039	Close to building, surrounded by hardscape
	Total Value	$168,080													

APPENDIX D

Cultural Resources Report

Filed Under Separate Request For Confidentiality

APPENDIX E

Preliminary Geotechnical Report

CORNERSTONE EARTH GROUP

TYPE OF SERVICES Preliminary Geotechnical Investigation

PROJECT NAME Eden Landing Business Park Redevelopment

LOCATION 3401-3475 Investment Boulevard, 26203-26291 Production Avenue, and 26010-26062 Eden Landing Road
Hayward, California

CLIENT Kimley-Horn and Associates, Inc.

PROJECT NUMBER 234-49-1

DATE August 4, 2022

CORNERSTONE EARTH GROUP

Type of Services	**Preliminary Geotechnical Investigation**
Project Name	**Eden Landing Business Park Redevelopment**
Location	**3401-3475 Investment Boulevard, 26203-26291 Production Avenue, and 26010-26062 Eden Landing Road Hayward, California**
Client	**Kimley-Horn and Associates, Inc.**
Client Address	**4637 Chabot Drive, Suite 300 Pleasanton, California**
Project Number	**234-49-1**
Date	**August 4, 2022**

Prepared by

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Quality Assurance Reviewer

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CORNERSTONE EARTH GROUP

Type of Services	**Preliminary Geotechnical Investigation**
Project Name	**Eden Landing Business Park Redevelopment**
Location	**3401-3475 Investment Boulevard, 26203-26291 Production Avenue, and 26010-26062 Eden Landing Road** **Hayward, California**

SECTION 1: INTRODUCTION

This preliminary geotechnical report was prepared for the sole use of Kimley-Horn and Associates, Inc. for the Eden Landing Business Park Redevelopment project in Hayward, California. The location of the site is shown on the Vicinity Map, Figure 1. For our use, we were provided with the following documents:

- A conceptual site plan labeled "Test Fit OPTDC 3S 15-pod," prepared by Kimley Horn, dated June 15, 2022.

1.1 PROJECT DESCRIPTION

We understand the project is still in the early planning stages and specific details are not available. However, based on the information provided, the project will include redeveloping the 11½-acre site (Eden Landing Business Park) for a new data center and substation. We anticipate the data center building will be on the order of two to three stories with one to two levels of data storage. The remaining levels will likely consist of office space. Appurtenant parking, utilities, retaining walls, landscaping, and other improvements necessary for site development will likely be included.

Building loads are anticipated to range up to 1770 kips based on the documents provided to us. Grading plans are not available at this time; however, we assume grading will consist of cuts and fills on the order of 3 to 5 feet.

1.2 SCOPE OF SERVICES

Our scope of services was presented in our proposal dated June 20, 2022 and consisted of field and laboratory programs to evaluate physical and engineering properties of the subsurface soils, engineering analysis to prepare recommendations for site work and grading, building foundations, flatwork, retaining walls, and pavements, and preparation of this report. Brief descriptions of our exploration and laboratory programs are presented below.

1.3 EXPLORATION PROGRAM

Field exploration consisted of two borings drilled on July 13, 2022 with truck-mounted, hollow-stem auger drilling equipment and three Cone Penetration Tests (CPTs) advanced on July 8, 2022. The borings were drilled to depths of 40 to 80 feet; the CPTs were advanced to depths of 80 to 125 feet. Seismic shear wave velocity measurements were collected from CPT-2. One of the borings (Boring EB-2) was advanced adjacent to CPT-2 for direct evaluation of physical samples to correlated soil behavior.

The borings and CPTs were backfilled with cement grout in accordance with local requirements; exploration permits were obtained as required by local jurisdictions.

The approximate locations of our exploratory borings are shown on the Site Plan, Figure 2. Details regarding our field program are included in Appendix A.

1.4 LABORATORY TESTING PROGRAM

In addition to visual classification of samples, the laboratory program focused on obtaining data for foundation design and seismic ground deformation estimates. Testing included moisture contents, dry densities, grain size analyses, washed sieve analyses, Plasticity Index tests, consolidation test, and triaxial compression tests. Details regarding our laboratory program are included in Appendix B.

1.5 CORROSION EVALUATION

Two samples from our borings from depths from 3 to 5½ feet were tested for saturated resistivity, pH, and soluble sulfates and chlorides. In general, the on-site soils can be characterized as corrosive to severely corrosive to buried metal, and non-corrosive to buried concrete. Please refer to Section 3 for further information.

1.6 THERMAL RESITIVITY EVALUATION

We performed four exploratory borings on July 11, 2022 to depths of about 6 feet for the purpose of collecting soil samples for laboratory testing. Our borings were performed at the locations and depths provided by AWS. One sample from each boring at depths of 5½ feet were submitted to a laboratory and tested for moisture content, dry density, and thermal resistivity. The approximate locations of our supplemental exploratory borings are shown on the Site Plan, Figure 2. The laboratory results, including dry-out curves, provided by GeoTherm USA are included in Appendix C. Please refer to that report for results and recommendations.

1.7 ENVIRONMENTAL SERVICES

Environmental services were not requested for this project. If environmental concerns are determined to be present during future evaluations, the project environmental consultant should review our geotechnical recommendations for compatibility with the environmental concerns.

SECTION 2: REGIONAL SETTING

2.1 GEOLOGICAL SETTING

The site is located approximately 2 miles east of the San Francisco Bay. Based on our review of explorations at the site and in the site vicinity and review of recent geologic maps of the area (Graymer 2000; Helley and Graymer, 1997), the site is underlain by Holocene age fan or basin deposits (Qhaf and Qhb). The fan deposits (Qhaf) are generally described by Graymer (2000) as medium dense to dense, gravelly sand or sandy gravel, grading upward to sandy or silty clay. It may contain localized layers, lenses and stringers of silt and sand. The basin deposits (Qhb) are generally very fine silty clays and clays deposited near the distal edge of alluvial fans and adjacent to Bay Mud, which may extend partially onto the western or southern edge of the site. The young sediments are generally underlain by older alluvial fan deposits collectively referred to Older Bay Mud or Old Bay Clay. These older alluvial soils generally consist of clays, sands, silts and localized gravel layers.

2.2 REGIONAL SEISMICITY

While seismologists cannot predict earthquake events, geologists from the U.S. Geological Survey have recently updated (in 2015) earlier estimates from their 2014 Uniform California Earthquake Rupture Forecast (Version 3; UCERF3) publication. The estimated probability of one or more magnitude 6.7 earthquakes (the size of the destructive 1994 Northridge earthquake) expected to occur somewhere in the San Francisco Bay Area has been revised (increased) to 72 percent for the period 2014 to 2043 (Aagaard et al., 2016). The faults in the region with the highest estimated probability of generating damaging earthquakes between 2014 and 2043 are the Hayward (33%), Calaveras (26%), and San Andreas Faults (22%). In this 30-year period, the probability of an earthquake of magnitude 6.7 or larger occurring is 22 percent along the San Andreas Fault and 33 percent for the Hayward Fault.

The faults considered capable of generating significant earthquakes are generally associated with the well-defined areas of crustal movement, which trend northwesterly. The table below presents the State-considered active faults within 25 kilometers of the site.

Table 1: Approximate Fault Distances

Fault Name	Distance	
	(miles)	(kilometers)
Hayward (Total Length)	3.7	5.9
Calaveras	11.0	17.7
San Andreas (1906)	14.6	23.5
Monte Vista-Shannon	14.8	23.8

A regional fault map is presented as Figure 3, illustrating the relative distances of the site to significant fault zones.

SECTION 3: SITE CONDITIONS

3.1 SURFACE DESCRIPTION

The approximately 11½-acre site is located at Eden Landing Business Park and includes addresses 3401-3475 Investment Boulevard, 26203-26291 Production Avenue, and 26010-26062 Eden Landing Road in Hayward, California. The site is bounded by Eden Landing Road to the northwest, Production Avenue to the northeast, Investment Boulevard to the south and industrial development to the west. The site is located just south of Highway 92.

Eden Landing Business Park consists of 10 one- to two-story industrial buildings totaling 195,044 square feet. The buildings are concrete tilt-up and include 120 suites with both office and warehouse space. At-grade asphalt concrete (AC) pavement drive aisles and parking stalls surround the existing buildings. Portland Cement Concrete (PCC) sidewalks were observed along the outside of the buildings. A grove of trees was observed in the center of the site and 2 to 3 foot high landscape berms were observed along the perimeter of the site along Eden Landing Road, Investment Boulevard, and Production Avenue. Overhead electrical lines and towers were observed along the western edge of the site.

Surface pavements at our exploratory borings generally consisted of 4 inches of asphalt concrete over 4 inches of aggregate base. Based on visual observations, areas of the existing AC pavement appear to have been recently painted or a top coat applied. However, in general, the pavements appear to be in poor condition with areas of significant alligator cracking, longitudinal cracking, and pavement patching. In addition, vertical curbs were observed to be significantly cracked, offset, and lifted. Based on visual observation, the existing PCC flatwork and pavements appear to be in fair to poor condition with areas of significant cracking and signs of uneven settlement in sidewalks (vertical offset cracks and lifting).

3.2 SUBSURFACE CONDITIONS

Below the surface pavements, our Exploratory Boring EB-2 encountered undocumented fill consisting of very stiff sandy lean clay to a depth of about 2 feet below existing grades.

Beneath the fills or surface pavements, our exploratory borings generally encountered very stiff lean clay with varying amounts of sand to depths of about 8½ to 9 feet underlain by medium stiff lean clay with varying amounts of sand to a depth of about 12 to 15½ feet. Boring EB-1 encountered a thin layer of loose silty sand at a depth of about 8 feet. Beneath the medium stiff clay, Boring EB-1 encountered a layer of loose clayey sand to a depth of about 14½ feet underlain by medium stiff clay to a depth of about 17 feet. Beneath the medium stiff clays, Boring EB-1 encountered stiff to very stiff lean clays with varying amounts of sand to a depth of 32 feet underlain by medium depth clayey sand to a depth of 33 feet, loose well-graded sand with silt and gravel to a depth of about 38 feet, and stiff lean clay with varying amounts of sand to the terminal boring depth of 40 feet. Beneath the medium stiff clays, Boring EB-2 encountered stiff to very stiff clays with varying amounts of sand to a depth of about 73½ feet underlain by loose to medium dense poorly graded sand to a depth of about 79½ feet and stiff lean clay with varying amounts of sand to the terminal boring depth of 80 feet. Beneath the ground surface, our CPTs generally encountered primarily medium stiff to very stiff clays and silts with thin, interbedded layers of loose to dense sands to a depth of about 65 to 70 feet underlain by interbedded layers of very stiff to hard clays and silts and medium dense to dense sands and gravels to the maximum depth explored of about 125 feet.

3.2.1 Plasticity/Expansion Potential

We performed two Plasticity Index (PI) tests on representative samples. Test results were used to evaluate expansion potential of surficial soils. The results of the surficial PI tests indicated PIs ranging from 21 to 31, indicating moderate to high expansion potential to wetting and drying cycles.

3.2.2 In-Situ Moisture Contents

Laboratory testing indicated that the in-situ moisture contents within the upper 10 feet range from about 3 to 15 percent over the estimated laboratory optimum moisture.

3.3 GROUNDWATER

Groundwater was encountered in our exploratory borings at depths ranging from about 8 to 9 feet below current grades and inferred from pore pressure dissipation tests in our Cone Penetration Tests (CPTs) at depths ranging from about 7 to 10 feet below existing grades. All measurements were taken at the time of drilling and may not represent the stabilized levels that can be higher than the initial levels encountered. Maps published by the California Geologic Survey (CGS, 2003) indicate historic high groundwater depth at approximately 8 to 10 feet below the ground surface.

We also reviewed groundwater data available online from the website GeoTracker, https://geotracker.waterboards.ca.gov/. Nearby monitoring well data indicates that groundwater has been measured at depths of approximately 13 to 15 feet at wells located at 25800 Clawiter to the north of the site on from 2008 to 2010 and at depths of approximately 2½ to 7 feet at wells located at 4150 Point Eden Way to the southwest of the site from 2009 to 2012.

Based on the above, we recommend a preliminary design groundwater depth of 5 feet. Fluctuations in groundwater levels occur due to many factors including seasonal fluctuation, underground drainage patterns, regional fluctuations, and other factors.

3.4 CORROSION SCREENING

We tested two samples collected at depths of 3 and 5½ feet for resistivity, pH, soluble sulfates, and chlorides. The laboratory test results are summarized in Table 2A.

Table 2A: Summary of Corrosion Test Results

Sample Location	Depth (feet)	Soil pH[1]	Resistivity[2] (ohm-cm)	Chloride[3] (mg/kg)	Sulfate[4,5] (mg/kg)
EB-1	5.5	6.9	1,096	12	58
EB-2	3.0	6.5	607	91	75

Notes: [1]ASTM G51
[2]ASTM G57 - 100% saturation
[3]ASTM D3427/Cal 422 Modified
[4]ASTM D3427/Cal 417 Modified
[5]1 mg/kg = 0.0001 % by dry weight

Many factors can affect the corrosion potential of soil including moisture content, resistivity, permeability, and pH, as well as chloride and sulfate concentration. Typically, soil resistivity, which is a measurement of how easily electrical current flows through a medium (soil and/or water), is the most influential factor. In addition to soil resistivity, chloride and sulfate ion concentrations, and pH also contribute in affecting corrosion potential.

3.4.1 Preliminary Soil Corrosion Screening

Based on the laboratory test results summarized in Table 2A and published correlations between resistivity and corrosion potential, the soils may be considered severely to very severely corrosive to buried metallic improvements (Chaker and Palmer, 1989).

In accordance with the 2019 CBC Section 1904.1, alternative cementitious materials for different exposure categories and classes shall be determined in accordance with ACI 318-19 Table 19.3.1.1, Table R19.3.1, and Table 19.3.2.1. Based on the laboratory sulfate test results, a cement type restriction is not required, although, in our opinion, it is generally a good idea to include some sulfate resistance and to maintain a relatively low water-cement ratio. We have summarized applicable exposure categories and classes from ACI 318-19, Table 19.3.1.1 below in Table 2B.

Table 2B: ACI 318-19 Table 19.3.1.1 Exposure Categories and Classes

Freezing and Thawing (F)	Sulfate (S, soil)	In Contact with Water (W)	Corrosion Protection of Reinforcement (C)
F0[1]	S0[2]	W0[3]	C0[4]

1 (F0) "Concrete not exposed to freezing-and-thawing cycles" (ACI 318-19)
2 (S0) "Water soluble sulfate in soil, percent by mass" (ACI 318-19)
3 (W0) "Concrete dry in service" (ACI 318-19)
4 (C0) "Concrete dry or protected from moisture" (ACI 318-19)

We recommend the structural engineer and a corrosion engineer be retained to confirm the above information and provide additional recommendations, as needed.

SECTION 4: GEOLOGIC HAZARDS

4.1 FAULT SURFACE RUPTURE

As discussed above several significant faults are located within 25 kilometers of the site. The site is not located within a State-designated Alquist Priolo Earthquake Fault Zone. As shown in Figure 3, no known surface expression of fault traces is thought to cross the site; therefore, fault surface rupture hazard is not a significant geologic hazard at the site.

4.2 ESTIMATED GROUND SHAKING

Moderate to severe (design-level) earthquakes can cause strong ground shaking, which is the case for most sites within the Bay Area. A peak ground acceleration (PGA_M) was estimated following the ground motion hazard analysis procedure presented in Chapter 16 and 18 and Appendix J of the 2019 California Building Code (CBC) and Chapter 21, Section 21.2 of ASCE 7-16 and Supplement No. 1. For our analysis we used a PGA_M of 0.74g which was determined in accordance with Section 21.5 of ASCE 7-16.

4.3 LIQUEFACTION POTENTIAL

The site is within a State-designated Liquefaction Hazard Zone (CGS, Hayward Quadrangle, 2003). Our field and laboratory programs addressed this issue by testing and sampling potentially liquefiable layers to depths of at least 50 feet, performing visual classification on sampled materials, evaluating CPT data, and performing various tests to further classify soil properties.

4.3.1 Background

During strong seismic shaking, cyclically induced stresses can cause increased pore pressures within the soil matrix that can result in liquefaction triggering, soil softening due to shear stress loss, potentially significant ground deformation due to settlement within sandy liquefiable layers as pore pressures dissipate, and/or flow failures in sloping ground or where open faces are present (lateral spreading) (NCEER 1998). Limited field and laboratory data is available regarding ground deformation due to settlement; however, in clean sand layers settlement on the order of 2 to 4 percent of the liquefied layer thickness can occur. Soils most susceptible to liquefaction are loose, non-cohesive soils that are saturated and are bedded with poor drainage, such as sand and silt layers bedded with a cohesive cap.

4.3.2 Analysis

As discussed in the “Subsurface” section above, several sand layers were encountered below the preliminary design groundwater depth of 5 feet. Following the liquefaction analysis framework in the 2008 monograph, *Soil Liquefaction During Earthquakes* (Idriss and Boulanger, 2008), incorporating updates in *CPT and SPT Based Liquefaction Triggering Procedures* (Boulanger and Idriss, 2014), and in accordance with CDMG Special Publication 117A guidelines (CDMG, 2008) for quantitative analysis, these layers were analyzed for liquefaction triggering and potential post-liquefaction settlement. These methods compare the ratio of the estimated cyclic shaking (Cyclic Stress Ratio - CSR) to the soil’s estimated resistance to cyclic shaking (Cyclic Resistance Ratio - CRR), providing a factor of safety against liquefaction triggering. Factors of safety less than or equal to 1.3 are considered to be potentially liquefiable and capable of post-liquefaction re-consolidation (i.e. settlement).

The CSR for each layer quantifies the stresses anticipated to be generated due to a design-level seismic event, is based on the peak horizontal acceleration generated at the ground surface discussed in the “Estimated Ground Shaking” section above, and is corrected for overburden and stress reduction factors as discussed in the procedure developed by Seed and Idriss (1971) and updated in the 2008 Idriss and Boulanger monograph.

The soil’s CRR is estimated from the in-situ measurements from CPTs and laboratory testing on samples retrieved from our borings. SPT “N” values obtained from hollow-stem auger borings were not used in our analyses, as the “N” values obtained are less reliable in sands below groundwater. The tip pressures are corrected for effective overburden stresses, taking into consideration both the groundwater level at the time of exploration and the design groundwater level, and stress reduction versus depth factors. The CPT method utilizes the soil behavior type index (I_C) to estimate the plasticity of the layers.

The results of our CPT analyses (CPT-1 through CPT-3) are presented on Figures 4A through 4C of this report.

4.3.3 Summary

Our analyses indicate that several layers could potentially experience liquefaction triggering that could result in post-liquefaction total settlement at the ground surface ranging from less than ¼-inch up to about ⅓-inch based on the Yoshimine (2006) method. As discussed in SP 117A, differential movement for level ground sites over deep soil sites will be up to about two-thirds of the total settlement between independent foundation elements. In our opinion, differential settlements are anticipated to be on the order of ¼-inch or less over a horizontal distance of 30 to 60 feet.

4.3.4 Ground Deformation and Surficial Cracking Potential

The methods used to estimate liquefaction settlements assume that there is a sufficient cap of non-liquefiable material to prevent ground deformation or sand boils. For ground deformation to occur, the pore water pressure within the liquefiable soil layer will need to be great enough to break through the overlying non-liquefiable layer, which could cause significant ground deformation and settlement. The work of Youd and Garris (1995) indicates that the 6-foot thick layer of non-liquefiable cap is sufficient to prevent ground deformation and significant surficial cracking; therefore, the above total settlement estimates are reasonable.

4.4 LATERAL SPREADING

Lateral spreading is horizontal/lateral ground movement of relatively flat-lying soil deposits towards a free face such as an excavation, channel, or open body of water; typically, lateral spreading is associated with liquefaction of one or more subsurface layers near the bottom of the exposed slope. As failure tends to propagate as block failures, it is difficult to analyze and estimate where the first tension crack will form.

There are no open faces within a distance considered susceptible to lateral spreading; therefore, in our opinion, the potential for lateral spreading to affect the site is low.

4.5 SEISMIC SETTLEMENT/UNSATURATED SAND SHAKING

Loose unsaturated sandy soils can settle during strong seismic shaking. As the soils encountered at the site were predominantly stiff to very stiff clays and medium dense to dense sands, in our opinion, the potential for significant differential seismic settlement affecting the proposed improvements is low.

4.6 TSUNAMI/SEICHE

The terms tsunami or seiche are described as ocean waves or similar waves usually created by undersea fault movement or by a coastal or submerged landslide. Tsunamis may be generated at great distance from shore (far field events) or nearby (near field events). Waves are formed, as the displaced water moves to regain equilibrium, and radiates across the open water, similar to ripples from a rock being thrown into a pond. When the waveform reaches the coastline, it quickly raises the water level, with water velocities as high as 15 to 20 knots. The water mass, as well as vessels, vehicles, or other objects in its path create tremendous forces as they impact coastal structures.

Tsunamis have affected the coastline along the Pacific Northwest during historic times. The Fort Point tide gauge in San Francisco recorded approximately 21 tsunamis between 1854 and 1964. The 1964 Alaska earthquake generated a recorded wave height of 7.4 feet and drowned eleven people in Crescent City, California. For the case of a far-field event, the Bay area would have hours of warning; for a near field event, there may be only a few minutes of warning, if any.

A tsunami or seiche originating in the Pacific Ocean would lose much of its energy passing through San Francisco Bay. Based on the mapping of tsunami inundation potential for the San Francisco Bay Area by CGS (conservation.ca.gov/cgs/tsunami/maps), areas most likely to be inundated are marshlands, tidal flats, and former bay margin lands that are now artificially filled, but are still at or below sea level, and are generally within 1½ miles of the shoreline. The site is approximately 1¾ miles inland from the San Francisco Bay shoreline and is approximately 16 to 20 feet above mean sea level. In addition, the site is located outside of the tsunami inundation area, according to the Tsunami Inundation Maps for Emergency Planning by the California Geologic Survey. Therefore, the potential for inundation due to tsunami or seiche is considered low.

4.7 FLOODING

Based on our internet search of the Federal Emergency Management Agency (FEMA) flood map public database, the site is located within Zone X, described as “Areas determined to be outside the 0.2% annual chance floodplain.” We recommend the project civil engineer be retained to confirm this information and verify the base flood elevation, if appropriate.

The California Division of Safety of Dams has compiled an interactive map showing Dam Failure Breach Inundation Maps. Based on our review of these maps, the site does not appear to be within a dam inundation zone. We recommend the project civil engineer be retained to confirm this information.

SECTION 5: CONCLUSIONS

5.1 SUMMARY

From a geotechnical viewpoint, the project is feasible provided the concerns listed below are addressed in the project design. The preliminary recommendations that follow are intended for conceptual planning and preliminary design. A design-level geotechnical investigation should be performed once site development plans are prepared indicating where proposed structures are planned. The design-level investigation findings will be used to confirm the preliminary recommendations and develop detailed recommendations for design and construction. Descriptions of each geotechnical concern with brief outlines of our preliminary recommendations follow the listed concerns.

- Potential for significant static settlements
- Redevelopment considerations
- Shallow groundwater
- Presence of moderately to highly expansive soils
- Presence of undocumented fill
- Soil corrosion potential

5.1.1 Potential for Significant Static Settlements

As noted above and discussed in the "Foundations" section of this report, structural loads are anticipated to range from about 220 kips to 1,770 kips for typical dead plus live loads for interior column footings. As such, we estimate large static and long-term consolidation settlements to occur over the design life of the structure. In addition, based on the medium stiff clays encountered starting at depths of about 8½ feet, we anticipate low allowable bearing pressures would require large at-grade spread footings. Based on our engineering judgement, experience with similar projects, and the subsurface conditions, on a preliminary basis we anticipate the proposed data center structure may need to be supported on augercast piles or shallow foundations over ground improvement. Based on our preliminary analysis, if the proposed building is on the lower range of provided loads (on the order of 220 to 250 kips), we can perform additional analysis to determine if shallow foundations without ground improvement are feasible. Foundation settlement and alternatives should be further evaluated during the design-level geotechnical investigation and once final footing loads are confirmed.

5.1.2 Redevelopment Considerations

As discussed, the site is currently occupied by existing buildings and appurtenant flatwork, site fixtures, and landscaping. We understand that all existing improvements will be demolished for the construction of the building additions. Potential issues that are often associated with redeveloping sites include demolition of existing improvements, abandonment of existing utilities, and undocumented fills. Please refer to the "Earthwork" section below for further recommendations.

5.1.3 Shallow Groundwater

Shallow groundwater was measured at depths ranging from approximately 7 to 10 feet below the existing ground surface. As discussed above, we recommend a design groundwater depth of 5 feet. Our experience with similar sites in the vicinity indicates that shallow groundwater could significantly impact grading and underground construction. These impacts typically consist of potentially wet and unstable pavement subgrade, difficulty achieving compaction, and difficult underground utility installation. Dewatering and shoring of utility trenches may be required in some isolated areas of the site.

5.1.4 Presence of Moderately to Highly Expansive Soils

Moderately to highly expansive surficial soils were encountered in our borings and appear to generally blanket the site. Expansive soils can undergo significant volume change with changes in moisture content. They shrink and harden when dried and expand and soften when wetted. To reduce the potential for damage to the planned structures, slabs-on-grade should have sufficient reinforcement and be supported on a layer of non-expansive fill; footings should extend below the zone of seasonal moisture fluctuation. In addition, it is important to limit moisture changes in the surficial soils by using positive drainage away from buildings as well as limiting landscaping watering. We recommend that a plug of low-permeability clay soil, sand-cement slurry, or lean concrete be placed within trenches just outside where the trenches pass into building and pavement areas.

5.1.5 Undocumented Fill

As discussed above, we encountered about 2 feet of undocumented fill in our borings. The exact fill quality, placement, and compaction is not known at this time. However, even well compacted fill can still settle due to the weight of new fill and building loads. If determined feasible and structures are supported on shallow foundations, we recommend the fill be removed from within the building footprints and replaced as engineered fill. Undocumented fill should be further evaluated as a part of the design-level geotechnical investigation.

5.1.6 Soil Corrosion Potential

Preliminary soil corrosion tests on samples of the near-surface soil indicate the corrosion potential for buried metallic structures, such as metal pipes, to be severely to very severely corrosive. Therefore, special requirements for corrosion control should be made to protect metal pipes. As the preliminary soil corrosion screening was based on the results of limited sampling, consideration may be given to collecting and testing additional samples to confirm our findings. We recommend a corrosion engineer be retained to review the corrosion test results and provide corrosion mitigation design services, as needed.

5.2 DESIGN-LEVEL GEOTECHNICAL INVESTIGATION

The preliminary recommendations contained in this preliminary report were based on limited site development information, limited exploration, and our experience in the area with similar projects. As site conditions may vary significantly between the small-diameter borings performed during this investigation, we also recommend that we be retained to 1) perform a design-level geotechnical investigation, once detailed site development plans are available; 2) to review the geotechnical aspects of the project structural, civil, and landscape plans and specifications, allowing sufficient time to provide the design team with any comments prior to issuing the plans for construction; and 3) be present to provide geotechnical observation and testing during earthwork and foundation construction.

SECTION 6: EARTHWORK

6.1 ANTICIPATED EARTHWORK MEASURES

On a preliminary basis, we recommend that any existing foundations, debris, slabs, and/or abandoned underground utilities be removed entirely and the resulting excavations backfilled with engineered fill. Additionally, any native soils that are disturbed during demolition of the existing improvement should also be removed and replaced as engineered fill.

As discussed above, we encountered up to about 2 feet of undocumented fill in our borings. In addition, due to the current site development, we anticipate additional fills could be present at the site. For preliminary planning purposes, we recommend all undocumented fill be completely removed from within building areas and to a lateral distance of at least 5 feet beyond the building footprint. We anticipate the fills may be reused when backfilling the excavations. The actual lateral extent and depth of undocumented fill should be confirmed during the design-level investigation.

On-site soils below the paved surface layer appear to be suitable for use as fill at the site. As discussed in the "Subsurface" section in this report, the in-situ moisture contents are up to about 3 to 15 percent over the estimated laboratory optimum in the upper 10 feet of the soil profile. The contractor should anticipate drying the soils prior to reusing them as fill, and this includes the material from the basement excavation. In addition, repetitive construction loading may de-stabilize the soils which is why subgrade stabilization at the bottom of the basement excavation is recommended.

Imported fill material for use as general fill should be predominantly granular with a Plasticity Index of 15 or less, and not contain recycled asphalt concrete where it will be used within habitable building areas. All fill as well as scarified soils in those areas to receive fill or slabs-on-grade should be compacted to at least 90 percent relative compaction as determined by ASTM Test Designation D-1557, latest edition; and be at least 2 percent above optimum. Areas of fill placed behind basement or retaining walls where surface improvements are planned and/or where improvements will transition from on-grade support to overlying the basements should be compacted to 95 percent. The upper 6 inches of subgrade in pavement areas and all aggregate base materials should be compacted to at least 95 percent relative compaction (ASTM D-1557, latest edition). Utility trench backfill should be compacted to at least 95 percent relative compaction (ASTM D-1557, latest edition) by mechanical means only.

Surface water runoff should not be allowed to pond adjacent to the building foundations, slabs-on-grade, or pavements. Hardscape surfaces should slope at least 2 percent towards suitable discharge facilities; landscape areas should slope at least 3 percent away from buildings. Bio-treatment basins should be kept at least 10 feet away from buildings and, where possible, at least 3 feet away from pavements and flatwork.

SECTION 7: 2019 CBC SEISMIC DESIGN CRITERIA

7.1 SEISMIC DESIGN CRITERIA

We developed site-specific seismic design parameters in accordance with Chapter 16, Chapter 18 and Appendix J of the 2019 California Building Code (CBC) and Chapters 11, 12, 20, and 21 and Supplement No. 1 of ASCE 7-16.

7.1.1 Site Location and Provided Data For 2019 CBC Seismic Design

The project is located at latitude 37.62666° and longitude -122.119579°, which is based on Google Earth (WGS84) coordinates at the approximate center of the Eden Landing Business Park in Hayward, California. We have assumed that a Seismic Importance Factor (I_e) of 1.00 has been assigned to the structure in accordance with Table 1.5-2 of ASCE 7-16 for structures classified as Risk Category II. The building period has not been provided by the project structural engineer.

7.2 2019 CBC SEISMIC DESIGN CRITERIA

As discussed in the "Subsurface" of our report, our CPT and exploratory borings generally encountered loose to dense sands and medium stiff to hard clay deposits to a depth of 125 feet, the maximum depth explored. Shear wave velocity (V_S) measurements were performed while advancing CPT-2, resulting in a time-averaged shear wave velocity for the top 30 meters (V_{S30}) of 153 meters per second (502 feet per second), for the upper 100 feet.

7.2.1 2019 CBC Seismic Design

As our shear wave velocity for the upper 30 meters was less than 600 feet per second, per section 20.3.2 of ASCE 7-16, we have classified the site as Soil Classification E, which is described as a "soft soil" profile. Because we used site specific data from our explorations and laboratory testing, the site class should be considered as "determined" for the purposes of estimating the seismic design parameters from the code. Our site-specific ground motion hazard analysis considered a V_{S30} of 153 m/s (502 ft/s).

In accordance with Section 11.4.8 of ASCE 7-16, we performed a ground motion hazard analysis following Chapter 21, Section 21.2 of ASCE 7-16. We evaluated both Probabilistic MCE_R Ground Motions in accordance with Method 1 and Deterministic MCE_R Ground Motions to generate our recommended design response spectrum for the project. The recommended design spectral accelerations and associated periods are provided in Appendix D.

SECTION 8: FOUNDATIONS

8.1 SUMMARY OF RECOMMENDATIONS

Due to the estimated high structural loads, potential for significant static settlement, and low bearing clayey soil, on a preliminary basis, the proposed at-grade data center structures may be supported on a shallow foundation system (such as a rigid mat or conventional footings), provided they are underlain by ground improvement elements designed to mitigate post-construction settlement to tolerable levels. As an alternative, the buildings can be supported on a deep foundation, such as augercast piles, provided the ground floor building slab is designed as a structural slab capable of spanning unsupported between pile caps and grade beams. Preliminary foundation recommendations are presented in the following sections of this report.

7.3 SHALLOW FOUNDATION OVER GROUND IMPROVEMENT

On a preliminary basis, we anticipate differential static settlement greater than 2 inches for higher column loads and on the order of 1 to 1½ inches for the lower column loads, the buildings could potentially be supported on shallow footings or a rigid mat foundation provided they are underlain by ground improvement elements designed to reduce total and differential settlement to tolerable levels. In general, footings would likely need to be at least 18 to 24 inches wide and extend at least 24 to 36 inches below the lowest adjacent grade. Lowest adjacent grade is defined as the deeper of the following: 1) bottom of the adjacent interior slab-on-grade, or 2) finished exterior grade, excluding landscaping topsoil.

Bearing pressures will be dependent on the final ground improvement technique and spacing; however, substantial improvement in bearing capacity would be expected. On a preliminary basis, we expect allowable bearing pressures on the order of 4,000 to 5,000 psf for combined dead plus live loads would be feasible.

Ground improvement should be designed to reduce total settlement due to static and seismic conditions to tolerable levels. As discussed in the “Ground Improvement” section below, the ground improvement design should be such that the total foundation settlement (static and seismic) are reduced to 1 to 1½ inches or less, with no more than ¾ inch for either the static or seismic component.

7.4 GROUND IMPROVEMENT

7.4.1 Ground Improvement Requirements

If considered, ground improvement should consist of densification techniques to improve the ground’s resistance to liquefaction, reduce static settlement, and improve bearing capacity and seismic performance. Densification techniques could potentially consist of vibratory (vibro) replacement (i.e. stone columns), granular compaction piles (i.e. rammed aggregate), grouted displacement columns (i.e. CLSM), or similar densification techniques. The intent of the ground improvement design would be to increase the density of potentially liquefiable sands by laterally displacing and/or densifying the in-place soils. The degree to which the density is increased will depend on the improvement method and spacing. Ground improvement can also be used to reduce static settlements and increase bearing capacity.

Vibro replacement and granular compaction piles are similar in that a probe is vibrated into the ground to the design depth and a compacted open-graded gravel column is constructed from the bottom up. The surrounding soils are densified by the displacement of soil as well as vibrations from consolidating and expanding the gravel column laterally. Disadvantages of densification pile types include noise and vibration (and sometimes dust) produced during construction. Vibrations during installation may cause noise and vibrations that can be heard or felt off-site. Pre-drilling through surficial materials may reduce noise and vibration and should be anticipated for improvement areas adjacent to the site that may be sensitive to vibrations.

CLSM columns are formed in displaced soil cavities, which displace liquefiable and compressible soil with cemented Controlled Low Strength Material. CLSM column ground improvement can mitigate liquefaction and settlement of heavy foundations and slabs. CLSM columns are ideal for sensitive project sites such as those near critical structures that require low noise and no vibration construction methods, such as unreinforced masonry walls, occupied offices, sensitive soil (e.g. Bay Mud), and hazardous/contaminated soil sites where deep ground improvement is required.

The CLSM columns are separated from the bottom of the footing using a minimum 6-inch layer of crushed rock or other material “cushion”. No connectivity of the CLSM columns and overlying structural element is allowed. In some cases, a Ground Anchor may be used in a higher strength column to resist uplift forces. Lateral resistance is provided by footing, mat, or slab bottom friction at the concrete to cushion layer interface or passive resistance of the side walls. The target strengths of the CLSM are usually between 500 to 1,000 psi at 28 days, depending on load demands. The CLSM strength is tested using standard concrete sampling and loading methods.

Based on the chosen ground improvement technique, the upper 1 to 3 feet or more of the working pad will likely need to be re-compacted after ground improvement installation, due to surface disturbance and potential ground heave. For this reason, we do not recommend preparation of the final pad, placement of non-expansive fill, or the construction of utilities prior to ground improvement.

Contractors to perform recommended ground improvement should have adequate experience for the proposed methods to address the requirements herein. All construction quality control and quality assurance records should be supplied to the design team for review on completion of the ground improvement. Adequate quality control readings must be available at the time of installation so that real time oversight can be provided. The instrumentation provided will depend on the ground improvement method chosen. Once a method is chosen, the geotechnical engineer should modify the project design guideline specification for the appropriate method.

8.3 DEEP FOUNDATIONS

8.3.1 Augercast Piles

On a preliminary basis, as an alternative to shallow foundation over ground improvement elements, the proposed buildings may also be supported on a deep foundation system that derives support from the underlying dense or stiff alluvial soils underlying the site. Based on our experience with similar projects, we anticipate feasible deep foundations will consist of 16- to 18-inch diameter drilled and cast-in-place augercast piles. Augercast piles are concrete piles that are cast-in-place using a hollow-stem auger that drills to the design depth and then sand-cement grout (4,000 to 6,000 psi grout) is pumped through the hollow-stem as the drill stem is extracted. Two types of augercast piles are available: Auger Pressure Grouted (APG) piles, which like piers, remove the soil column and replace it with grout; and drilled displacement (APGD) piles, which displace the soil column as the drill stem is advanced, similar to driven piles, prior to pumping the grout. Recommendations for displacement augercast piles are not included for this project because dense sand layers encountered during our investigation may prevent the displacement auger from advancing to the necessary depth. Augercast piles are a low noise and vibration installation compared to driven piles. Various types of steel reinforcing, including rebar cages or H-piles may be installed into the still-wet grout after drilling to satisfy bending moment requirements. If APG piles are considered for this project, and the soil column is removed during drilling, considerable quantity of drill spoils will be generated. Therefore, disposal and/or removal of drill spoils from the site should be expected and planned for. The characterization and removal of drill spoils should be coordinated with the Project Environmental Engineer.

For reference, vertical and lateral load capacities of augercast piles will be similar to driven piles based on the pile surface area. For example, a 14-inch square pre-cast driven pile has a similar surface area as a 16-inch round augercast pile, and a 16-inch square pile will be similar to an 18-inch round pile.

On a preliminary basis, we recommend that at least two field pile load tests should be performed at locations within the building area recommended by the geotechnical engineer. Additional discussion on pile testing will be included in a design-level investigation.

Adjacent pile centers should be spaced at least three diameters apart. A reduction for vertical group effects may still be required. Grade beams should span between piles and/or pile caps in accordance with structural requirements.

If this option is desired, additional information, including vertical and lateral pile capacities can be provided in a design-level report.

SECTION 9: CONCRETE SLABS AND PEDESTRIAN PAVEMENTS

9.1 INTERIOR SLABS-ON-GRADE WITH SPREAD FOOTINGS

Due to the moderate to high expansion potential of the surficial soils, the proposed slabs-on-grade should be supported on at least 18 inches of non-expansive fill (NEF) to reduce the potential for slab damage due to soil heave. The NEF layer should be constructed over subgrade prepared in accordance with the recommendations in the “Earthwork” section of this report. If moisture-sensitive floor coverings are planned, the recommendations in the “Interior Slabs Moisture Protection Considerations” section below may be incorporated in the project design if desired. If significant time elapses between initial subgrade preparation and slab-on-grade [NEF] construction, the subgrade should be proof-rolled to confirm subgrade stability, and if the soil has been allowed to dry out, the subgrade should be re-moisture conditioned to at least 3 percent over the optimum moisture content.

The structural engineer should determine the appropriate slab reinforcement for the loading requirements and considering the expansion potential of the underlying soils. For unreinforced concrete slabs, ACI 302.1R recommends limiting control joint spacing to 24 to 36 times the slab thickness in each direction, or a maximum of 18 feet.

9.2 INTERIOR SLABS MOISTURE PROTECTION CONSIDERATIONS

The following general guidelines for concrete slab-on-grade construction where floor coverings are planned are presented for the consideration by the developer, design team, and contractor. These guidelines are based on information obtained from a variety of sources, including the American Concrete Institute (ACI) and are intended to reduce the potential for moisture-related problems causing floor covering failures, and may be supplemented as necessary based on project-specific requirements. The application of these guidelines or not will not affect the geotechnical aspects of the slab-on-grade performance.

- Place a minimum 15-mil vapor retarder conforming to ASTM E 1745, Class C requirements or better directly below the concrete slab; the vapor retarder should extend to the slab edges and be sealed at all seams and penetrations in accordance with manufacturer's recommendations and ASTM E 1643 requirements. A 4-inch-thick capillary break, consisting of crushed rock should be placed below the vapor retarder and consolidated in place with vibratory equipment. The mineral aggregate shall be of such size that the percentage composition by dry weight as determined by laboratory sieves will conform to the following gradation:

Sieve Size	Percentage Passing Sieve
1"	100
¾"	90 – 100
No. 4	0 – 10
No. 200	0 – 5

  The capillary break rock may be considered as the upper 4 inches of the non-expansive fill previously recommended.

- The concrete water:cement ratio should be 0.45 or less. Mid-range plasticizers may be used to increase concrete workability and facilitate pumping and placement.

- Water should not be added after initial batching unless the slump is less than specified and/or the resulting water:cement ratio will not exceed 0.45.

- Polishing the concrete surface with metal trowels is not recommended.

- Where floor coverings are planned, all concrete surfaces should be properly cured.

- Water vapor emission levels and concrete pH should be determined in accordance with ASTM F1869-98 and F710-98 requirements and evaluated against the floor covering manufacturer's requirements prior to installation.

9.4 EXTERIOR FLATWORK

Exterior flatwork, such as pedestrian walkways, patios, driveways, and sidewalks, may experience seasonal movement due to the native expansive soils; therefore, some cracking or vertical movement of conventional slabs should be anticipated where imported fill is not planned in flatwork areas. There are several alternatives for mitigating the impacts of expansive soils beneath concrete flatwork. We are providing recommendations to reduce distress to concrete flatwork that includes moisture conditioning the subgrade soils, using non-expansive fill, and providing adequate construction and control joints to control cracks that do occur. It should be noted that minor slab movement or localized cracking and/or distress could still occur.

- The minimum recommendation for concrete flatwork constructed on moderately to highly expansive soils is to properly prepare the clayey soils prior to placing concrete. This is typically achieved by scarifying, moisture conditioning, and re-compacting the subgrade soil. Subgrade soil should be moisture conditioned to at least 3 percent over the laboratory optimum and compacted using moderate compaction effort to a relative compaction of 87 to 92 percent (ASTM Test Method D1557). Since the near surface soils may have been previously compacted and tested, the subgrade soils could possibly be moisture conditioned by gradually wetting the soil, depending on the time of year slab construction occurs. This should not include flooding or excessively watering the soil, which would likely result in a soft, unstable subgrade condition, and possible delays in the construction while waiting for the soil to dry out. In general, the subgrade should be relatively firm and non-yielding prior to construction.

- Concrete flatwork, excluding pavements that would be subject to wheel loads, should be at least 4 inches thick and underlain by at least 9 inches of non-expansive fill. Non-expansive fill may include aggregate base, crushed rock, or imported soil with a PI of 15 or less. Non-expansive fill should be compacted to at least 90 percent relative compaction. Flatwork that will be subject to heavier or frequent vehicular loading should be designed in accordance with the recommendations in the "Vehicular Pavements" section below.

- We recommend a maximum control joint spacing of about 2 feet in each direction for each inch of concrete thickness and a construction joint spacing of 10 to 12 feet. Construction joints that abut the foundations or garage slabs should include a felt strip, or approved equivalent, that extends the full depth of the exterior slab. This will help to reduce the potential for permanent vertical offset between the slabs due to friction between the concrete edges. We recommend that exterior slabs be isolated from adjacent foundations.

At the owner's option, if desired to reduce the potential for vertical offset or widening of concrete cracks, consideration should be given to using reinforcing steel, such as No. 3 rebar spaced at 18 inches on center each direction.

SECTION 10: VEHICULAR PAVEMENTS

10.1 ASPHALT CONCRETE

The following asphalt concrete pavement recommendations tabulated below are based on the Procedure 608 of the Caltrans Highway Design Manual, estimated traffic indices for various pavement-loading conditions, and on a design R-value of 5. The design R-value was chosen based on engineering judgement considering the variable and expansive soil conditions. Additionally, due to the presence of moderate to highly expansive soils, we have also included an option for lime-treated subgrade soils using an estimated design R-value of 50.

Table 3: Preliminary Asphalt Concrete Pavement Recommendations

Design Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base* (inches)	Total Pavement Section Thickness (inches)
4.0	2.5	7.5	9.5
4.5	2.5	8.5	11.0
5.0	3.0	9.0	12.0
5.5	3.0	10.5	13.5
6.0	3.5	11.5	15.0
6.5	4.0	12.5	16.5
7.0	4.0	14.0	18.0
7.5	4.5	15.5	20.0
8.0	5.0	17.0	21.5

[1] Caltrans Class 2 aggregate base; minimum R-value of 78; subgrade R-value of 5

Table 4: Asphalt Concrete Pavement Recommendations, Lime Treated Subgrade

Design Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base* (inches)	Total Pavement Section Thickness (inches)
4.0	2.5	4.0	6.5
4.5	2.5	4.0	6.5
5.0	3.0	4.5	7.5
5.5	3.0	4.5	7.5
6.0	3.5	4.5	8.0
6.5	3.5	5.0	8.5
7.0	4.0	5.0	9.0
7.5	4.5	5.5	10.0
8.0	4.5	6.5	11.0

[1]Caltrans Class 2 aggregate base; minimum R-value of 78; subgrade R-value of 50

Frequently, the full asphalt concrete section is not constructed prior to construction traffic loading. This can result in significant loss of asphalt concrete layer life, rutting, or other pavement failures. To improve the pavement life and reduce the potential for pavement distress through construction, we recommend the full design asphalt concrete section be constructed prior to construction traffic loading. Alternatively, a higher traffic index may be chosen for the areas where construction traffic will use the pavements.

Asphalt concrete pavements constructed on expansive subgrade where the adjacent areas will not be irrigated for several months after the pavements are constructed may experience longitudinal cracking parallel to the pavement edge. These cracks typically form within a few feet of the pavement edge and are due to seasonal wetting and drying of the adjacent soil. The cracking may also occur during construction where the adjacent grade is allowed to significantly dry during the summer, pulling moisture out of the pavement subgrade. Any cracks that form should be sealed with bituminous sealant prior to the start of winter rains. One alternative to reduce the potential for this type of cracking is to install a moisture barrier at least 24 inches deep behind the pavement curb.

10.2 PORTLAND CEMENT CONCRETE

The Portland Cement Concrete (PCC) pavement recommendations outlined below are based on methods presented in American Concrete Pavement Association (ACPA, 2006). We have provided a few pavement alternatives as an anticipated Average Daily Truck Traffic (ADTT) was not provided. The following table presents minimum PCC pavements thicknesses for various traffic loading categories and the anticipated maximum Average Daily Truck Traffic (ADTT).

Table 5: PCC Pavement Recommendations

Traffic Category	Minimum PCC Thickness[1] (inches)	Class 2 Aggregate Base (inches)
Maximum ADTT = 10	5.5	6.0
Maximum ADTT = 20	6.0	6.0
Maximum ADTT = 50	6.0	6.0

[1]Subgrade design R-Value = 5

The PCC thicknesses above are based on a concrete compressive strength of at least 3,500 psi, and laterally restraining the PCC with curbs or concrete shoulders. Adequate expansion and control joints should be included. Consideration should be given to limiting the control joint spacing to a maximum of about 2 feet in each direction for each inch of concrete thickness. Due to the expansive surficial soils present, we recommend that the construction and expansion joints be dowelled.

10.2.1 Stress Pads for Trash Enclosures

Pads where trash containers will be stored, and where garbage trucks will park while emptying trash containers, should be constructed on Portland Cement Concrete. We recommend that the trash enclosure pads and stress (landing) pads where garbage trucks will store, pick up, and empty trash be increased to a minimum PCC thickness of 8 inches. The compressive strength, underlayment, and construction details should be consistent with the above recommendations for PCC pavements.

10.3 PAVEMENT CUTOFF

Surface water penetration into the pavement section can significantly reduce the pavement life, due to the native expansive clays. While quantifying the life reduction is difficult, a normal 20-year pavement design could be reduced to less than 10 years; therefore, increased long-term maintenance may be required.

It would be beneficial to include a pavement cut-off, such as deepened curbs, redwood-headers, or "Deep-Root Moisture Barriers" that are keyed at least 4 inches into the pavement subgrade. This will help limit the additional long-term maintenance.

SECTION 11: RETAINING WALLS

11.1 STATIC LATERAL EARTH PRESSURES

The structural design of any site retaining wall should include resistance to lateral earth pressures that develop from the soil behind the wall, any undrained water pressure, and surcharge loads acting behind the wall. Provided a drainage system is constructed behind the wall to prevent the build-up of hydrostatic pressures as discussed in the section below, we recommend that the walls with level backfill be designed for the following pressures:

Table 6: Recommended Lateral Earth Pressures

Wall Condition	Lateral Earth Pressure*	Additional Surcharge Loads
Unrestrained – Cantilever Wall	45 pcf	⅓ of vertical loads at top of wall
Restrained – Braced Wall	45 pcf + 8H** psf	½ of vertical loads at top of wall

* Lateral earth pressures are based on an equivalent fluid pressure for level backfill conditions
** H is the distance in feet between the bottom of footing and top of retained soil

If adequate drainage cannot be provided behind the wall, an additional equivalent fluid pressure of 40 pcf should be added to the values above for both restrained and unrestrained walls for the portion of the wall that will not have drainage. Damp proofing or waterproofing of the walls may be considered where moisture penetration and/or efflorescence are not desired.

11.2 SEISMIC LATERAL EARTH PRESSURES

11.2.1 Site Walls

The 2019 CBC states that lateral pressures from earthquakes should be considered in the design of basements and retaining walls. At this time, we are not aware of any retaining walls for the project. However, minor landscaping walls (i.e. walls 6 feet or less in height) may be proposed. In our opinion, design of these walls for seismic lateral earth pressures in addition to static earth pressures is not warranted.

11.3 WALL DRAINAGE

11.3.1 At-Grade Site Walls

Adequate drainage should be provided by a subdrain system behind all walls. This system should consist of a 4-inch minimum diameter perforated pipe placed near the base of the wall (perforations placed downward). The pipe should be bedded and backfilled with Class 2 Permeable Material per Caltrans Standard Specifications, latest edition. The permeable backfill should extend at least 12 inches out from the wall and to within 2 feet of outside finished grade. Alternatively, ½-inch to ¾-inch crushed rock may be used in place of the Class 2 Permeable Material provided the crushed rock and pipe are enclosed in filter fabric, such as Mirafi 140N or approved equivalent. The upper 2 feet of wall backfill should consist of compacted on-site soil. The subdrain outlet should be connected to a free-draining outlet or sump.

Miradrain, Geotech Drainage Panels, or equivalent drainage matting can be used for wall drainage as an alternative to the Class 2 Permeable Material or drain rock backfill. Horizontal strip drains connecting to the vertical drainage matting may be used in lieu of the perforated pipe and crushed rock section. The vertical drainage panel should be connected to the perforated pipe or horizontal drainage strip at the base of the wall, or to some other closed or through-wall system such as the TotalDrain system from AmerDrain. Sections of horizontal drainage strips should be connected with either the manufacturer's connector pieces or by pulling back the filter fabric, overlapping the panel dimples, and replacing the filter fabric over the connection. At corners, a corner guard, corner connection insert, or a section of crushed rock covered with filter fabric must be used to maintain the drainage path.

Drainage panels should terminate 18 to 24 inches from final exterior grade. The Miradrain panel filter fabric should be extended over the top of and behind the panel to protect it from intrusion of the adjacent soil.

11.4 BACKFILL

Where surface improvements will be located over the retaining wall backfill, backfill placed behind the walls should be compacted to at least 95 percent relative compaction using light compaction equipment. Where no surface improvements are planned, backfill should be compacted to at least 90 percent. If heavy compaction equipment is used, the walls should be temporarily braced.

11.5 FOUNDATIONS

In general, conventional at-grade site retaining walls may be supported on a continuous conventional footing. Strip footings should bear on natural, undisturbed soil or entirely on engineered fill, and extend at least 24 inches below the lowest adjacent grade.

On a preliminary basis, footings constructed to the above dimensions are capable of supporting maximum allowable bearing pressures of 1,700 psf for dead loads, 2,550 psf for combined dead plus live loads, and 3,400 psf for all loads including wind and seismic. These pressures are based on factors of safety of 3.0, 2.0, and 1.5 applied to the ultimate bearing pressure for dead, dead plus live, and all loads, respectively. These pressures are net values; the weight of the footing may be neglected for the portion of the footing extending below-grade (typically, the full footing depth). Top and bottom of mats of reinforcing steel should be included in continuous footings to help span irregularities and differential settlement.

SECTION 12: LIMITATIONS

This report, an instrument of professional service, has been prepared for the sole use of Kimley-Horn and Associates, Inc. specifically to support the design of the Eden Landing Business Park Redevelopment project in Hayward, California. The opinions, conclusions, and recommendations presented in this report have been formulated in accordance with accepted geotechnical engineering practices that exist in Northern California at the time this report was prepared. No warranty, expressed or implied, is made or should be inferred.

Recommendations in this report are based upon the soil and groundwater conditions encountered during our subsurface exploration. If variations or unsuitable conditions are encountered during construction, Cornerstone must be contacted to provide supplemental recommendations, as needed.

Kimley-Horn and Associates, Inc. may have provided Cornerstone with plans, reports and other documents prepared by others. Kimley-Horn and Associates, Inc. understands that Cornerstone reviewed and relied on the information presented in these documents and cannot be responsible for their accuracy.

Cornerstone prepared this report with the understanding that it is the responsibility of the owner or his representatives to see that the recommendations contained in this report are presented to other members of the design team and incorporated into the project plans and specifications, and that appropriate actions are taken to implement the geotechnical recommendations during construction.

Conclusions and recommendations presented in this report are valid as of the present time for the development as currently planned. Changes in the condition of the property or adjacent properties may occur with the passage of time, whether by natural processes or the acts of other persons. In addition, changes in applicable or appropriate standards may occur through legislation or the broadening of knowledge. Therefore, the conclusions and recommendations presented in this report may be invalidated, wholly or in part, by changes beyond Cornerstone's control. This report should be reviewed by Cornerstone after a period of three (3) years has elapsed from the date of this report. In addition, if the current project design is changed, then Cornerstone must review the proposed changes and provide supplemental recommendations, as needed.

An electronic transmission of this report may also have been issued. While Cornerstone has taken precautions to produce a complete and secure electronic transmission, please check the electronic transmission against the hard copy version for conformity.

Recommendations provided in this report are based on the assumption that Cornerstone will be retained to provide observation and testing services during construction to confirm that conditions are similar to that assumed for design, and to form an opinion as to whether the work has been performed in accordance with the project plans and specifications. If we are not retained for these services, Cornerstone cannot assume any responsibility for any potential claims that may arise during or after construction as a result of misuse or misinterpretation of Cornerstone's report by others. Furthermore, Cornerstone will cease to be the Geotechnical-Engineer-of-Record if we are not retained for these services.

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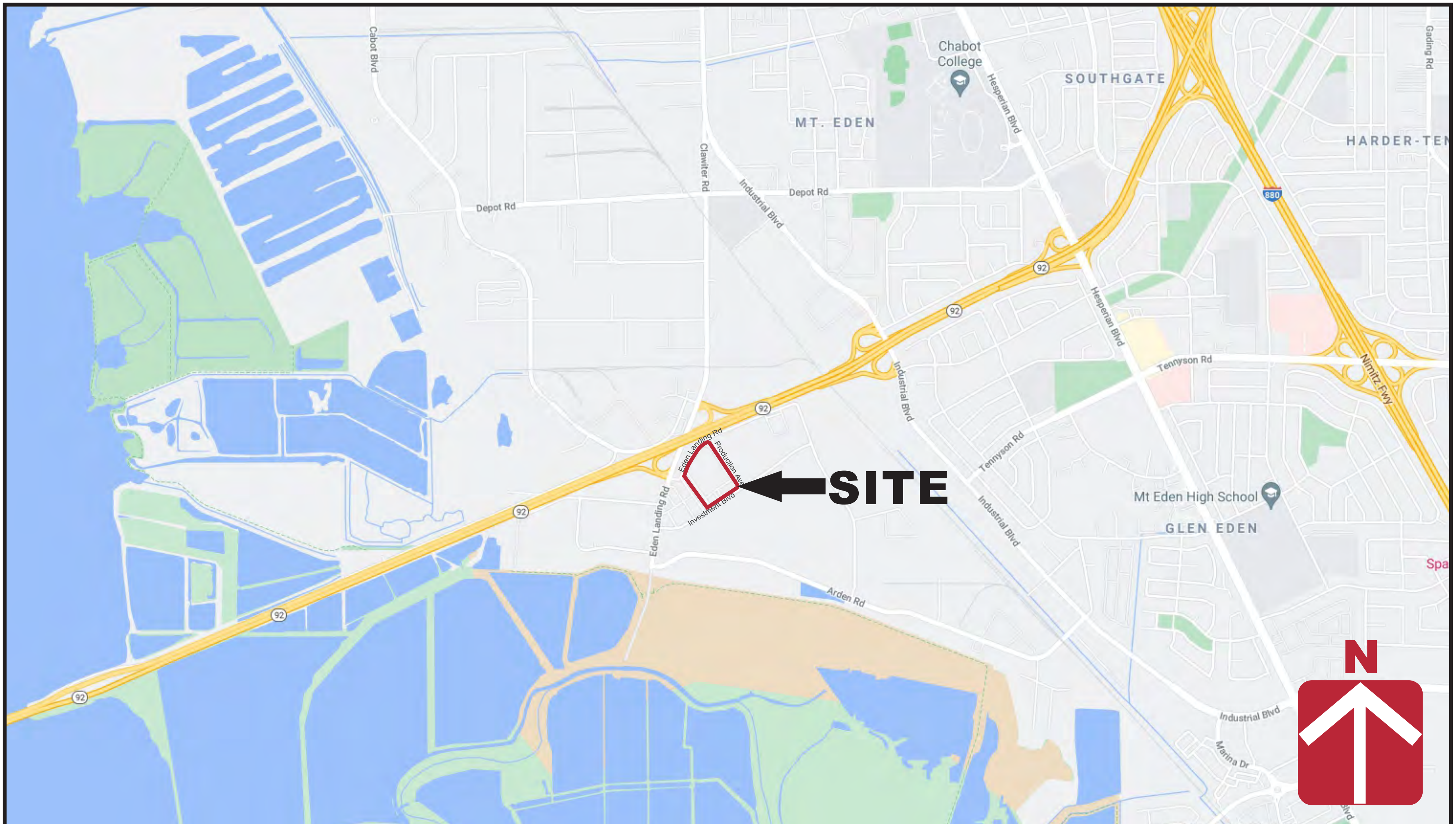

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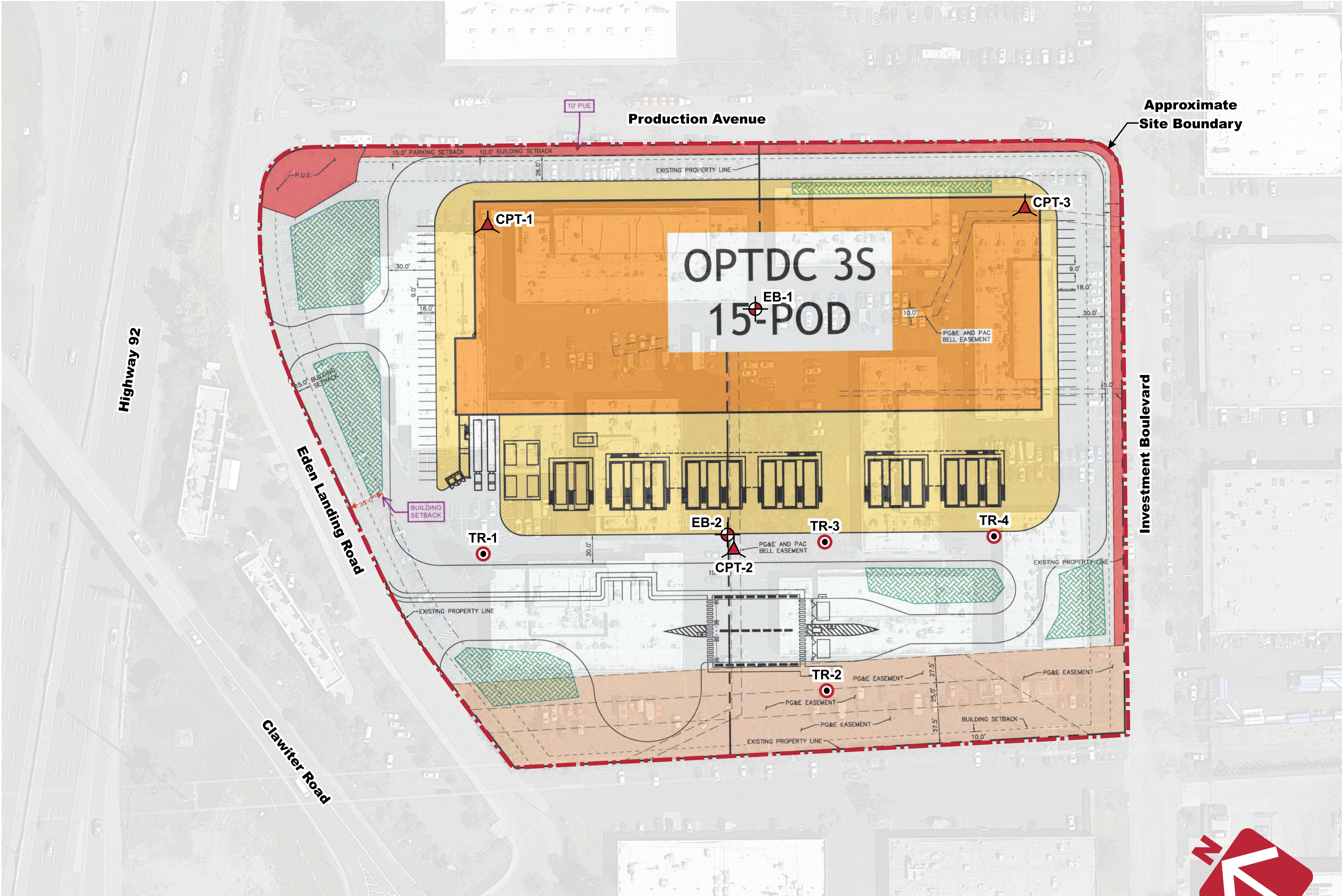
Vicinity Map

Eden Landing Business Park
Hayward, CA

Project Number	234-49-1
Figure Number	Figure 1
Date	July 2022
Drawn By	RRN



Legend

- Approximate location of exploratory boring (EB)
- Approximate location of cone penetration test (CPT)
- Approximate location of thermal resistivity sample (TR)

0 100 200
APPROXIMATE SCALE (FEET)

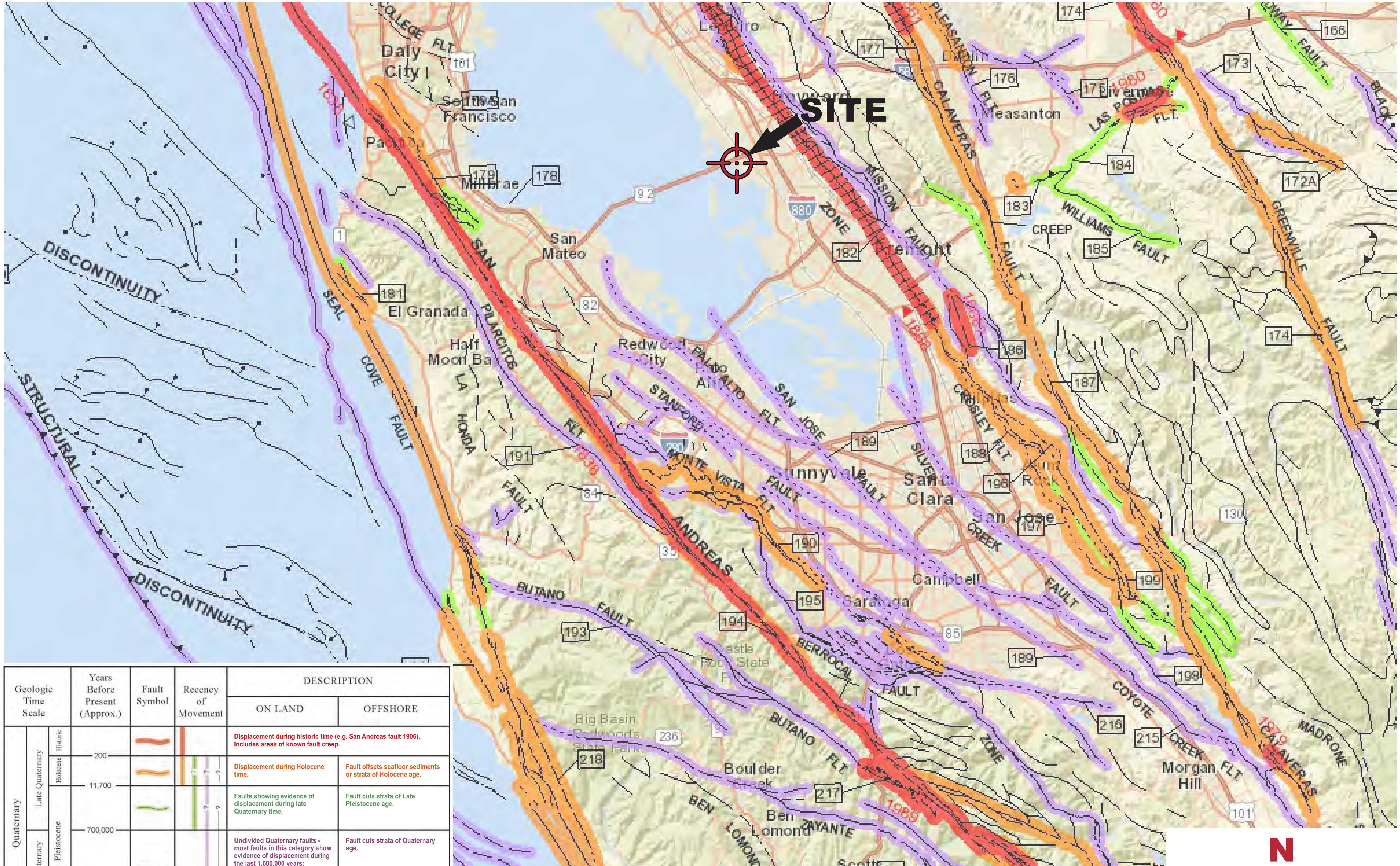
Base by Google Earth, dated 02/24/2021
Overlay by Kimbley Horn, Test Fit 15 Pod, dated 06/15/2022

Project Number 234-49-1
Figure Number Figure 2
Date July 2022
Drawn By RRN

Site Plan

Eden Landing Business Park
Hayward, CA

CORNERSTONE EARTH GROUP



Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement	DESCRIPTION: ON LAND	DESCRIPTION: OFFSHORE
Quaternary	Late Quaternary	Historic	200			Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
		Holocene	11,700			Displacement during Holocene time.	Fault offsets seafloor sediments or strata of Holocene age.
		Pleistocene	700,000			Faults showing evidence of displacement during late Quaternary time.	Fault cuts strata of Late Pleistocene age.
	Early Quaternary		1,600,000			Undivided Quaternary faults - most faults in this category show evidence of displacement during the last 1,600,000 years; possible exceptions are faults which displace rocks of undifferentiated Pilo-Pleistocene age.	Fault cuts strata of Quaternary age.
Pre-Quaternary			4.5 billion (Age of Earth)			Faults without recognized Quaternary displacement or showing evidence of no displacement during Quaternary time. Not necessarily inactive.	Fault cuts strata of Pliocene or older age.

Base by California Geological Survey - 2010 Fault Activity Map of California (Jennings and Bryant, 2010)

Regional Fault Map

Eden Landing Business Park
Hayward, CA

Project Number 234-49-1

Figure Number Figure 3

Date July 2022

Drawn By RRN

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FIGURE 4A

CPT NO. 1

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PROJECT/CPT DATA

Project Title	**Eden Landing Redevelopment**
Project No.	**234-49-1**
Project Manager	**MFR**

SEISMIC PARAMETERS

Controlling Fault	**Hayward (HN + HS + HE)**
Earthquake Magnitude (Mw)	**7.36**
PGA (Amax)	**0.74** (g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet)	**10.1**
Design Water Depth (feet)	**5**
Ave. Unit Weight Above GW (pcf)	**120**
Ave. Unit Weight Below GW (pcf)	**125**

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 5 **FEET**

0.00 **(Inches)**

LIQUEFACTION SETTLEMENT FROM 50 **FEET**

0.27 **(Inches)**

TOTAL SEISMIC SETTLEMENT 0.3 **INCHES**

POTENTIAL LATERAL DISPLACEMENT

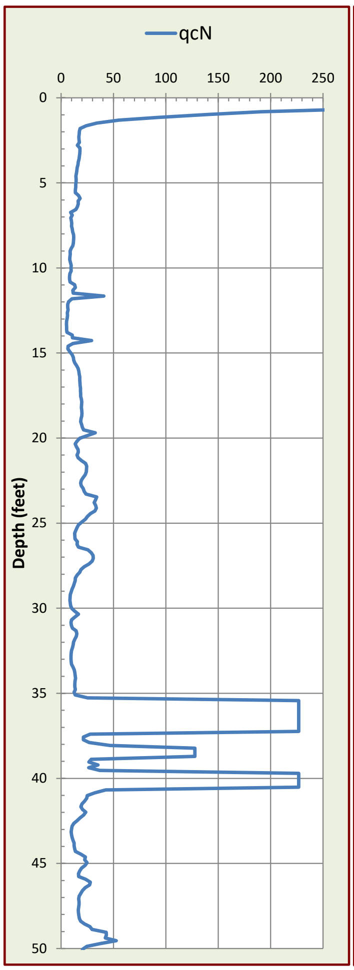
LDI[2] 0.00 **L/H** 1000.0

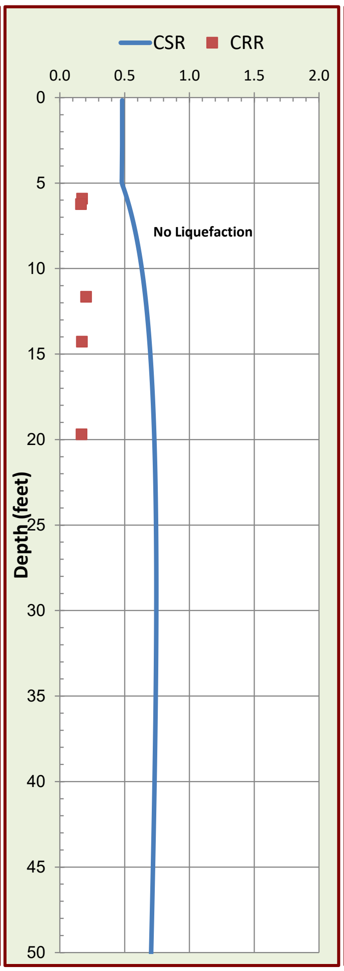
LDI[1]Corrected for Distance 0.00 (4 < L/H < 40)

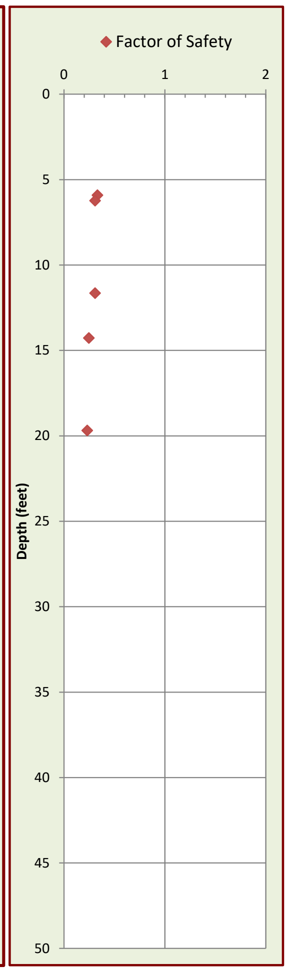
EXPECTED RANGE OF DISPLACEMENT

0.0 to **0.0** feet

[1]Not Valid for L/H Values < 4 and > 40.
[2]LDI Values Only Summed to 2H Below Grade.







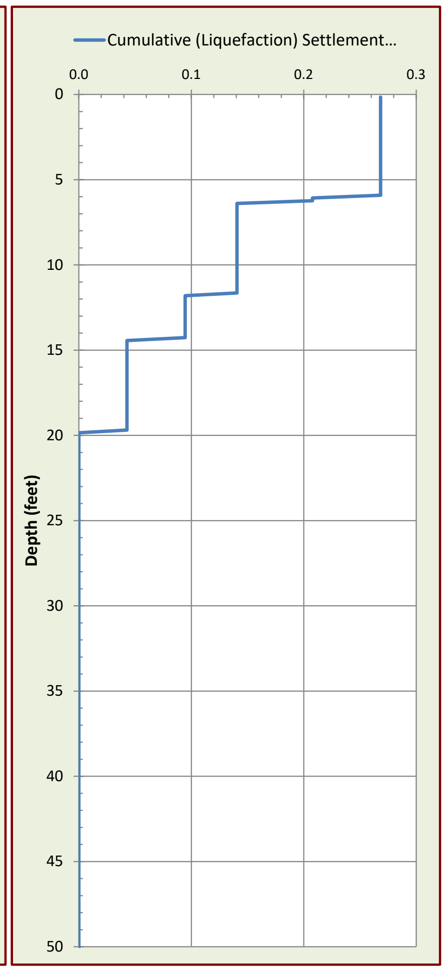


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FIGURE	4B
CPT NO.	2

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PROJECT/CPT DATA

Project Title	Eden Landing Redevelopment
Project No.	234-49-1
Project Manager	MFR

SEISMIC PARAMETERS

Controlling Fault	Hayward (HN + HS + HE)	
Earthquake Magnitude (Mw)	7.36	
PGA (Amax)	0.74	(g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet)	10.4
Design Water Depth (feet)	5
Ave. Unit Weight Above GW (pcf)	120
Ave. Unit Weight Below GW (pcf)	125

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM	5	FEET
	0.00	(Inches)
LIQUEFACTION SETTLEMENT FROM	50	FEET
	0.29	(Inches)
TOTAL SEISMIC SETTLEMENT	0.3	INCHES

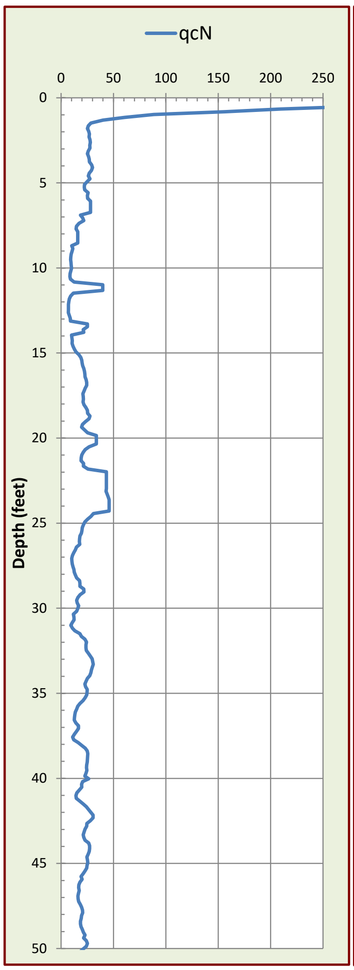
POTENTIAL LATERAL DISPLACEMENT

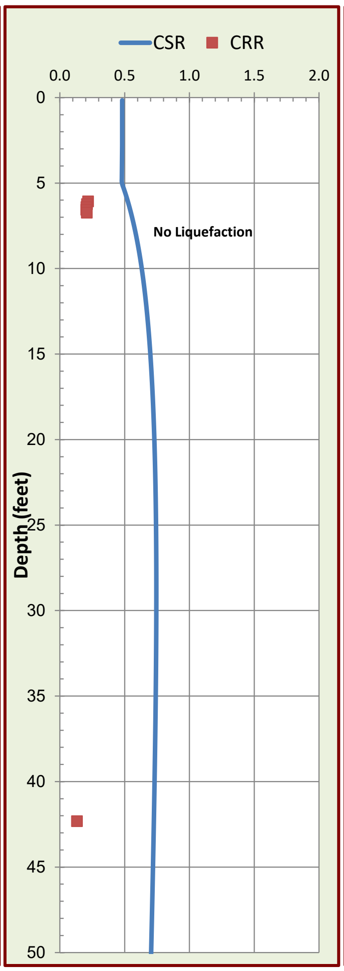
LDI[2]	0.00	L/H	1000.0
LDI[1] Corrected for Distance	0.00	(4 < L/H < 40)	

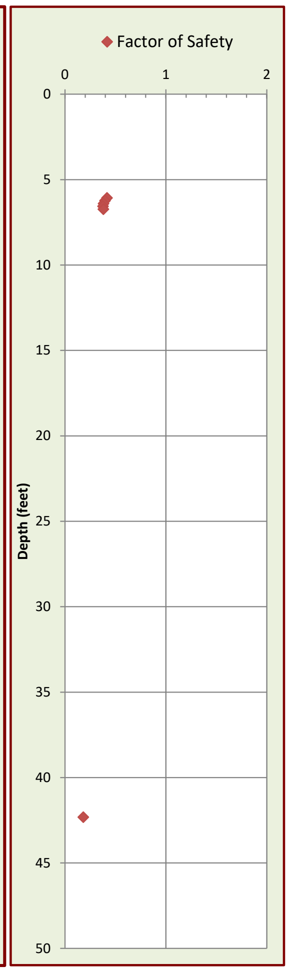
EXPECTED RANGE OF DISPLACEMENT

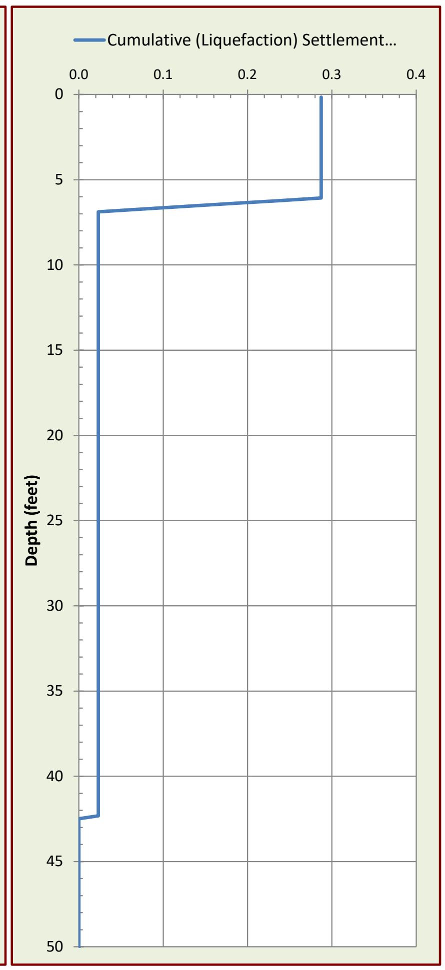
0.0 to 0.0 feet

[1]Not Valid for L/H Values < 4 and > 40.
[2]LDI Values Only Summed to 2H Below Grade.









CORNERSTONE EARTH GROUP

FIGURE 4C

CPT NO. 3

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PROJECT/CPT DATA

Project Title	**Eden Landing Redevelopment**
Project No.	**234-49-1**
Project Manager	**MFR**

SEISMIC PARAMETERS

Controlling Fault	**Hayward (HN + HS + HE)**
Earthquake Magnitude (Mw)	**7.36**
PGA (Amax)	**0.74** (g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet)	**10.4**
Design Water Depth (feet)	**5**
Ave. Unit Weight Above GW (pcf)	**120**
Ave. Unit Weight Below GW (pcf)	**125**

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 5 **FEET**

0.00 **(Inches)**

LIQUEFACTION SETTLEMENT FROM 50 **FEET**

0.11 **(Inches)**

TOTAL SEISMIC SETTLEMENT 0.1 **INCHES**

POTENTIAL LATERAL DISPLACEMENT

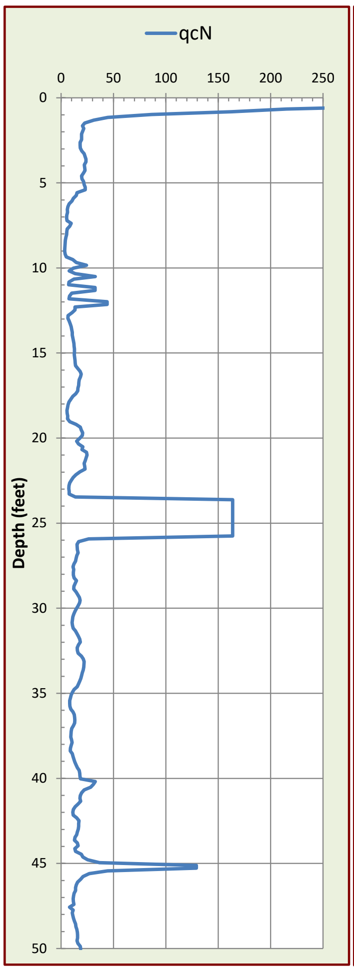
LDI[2] 0.00 **L/H** 1000.0

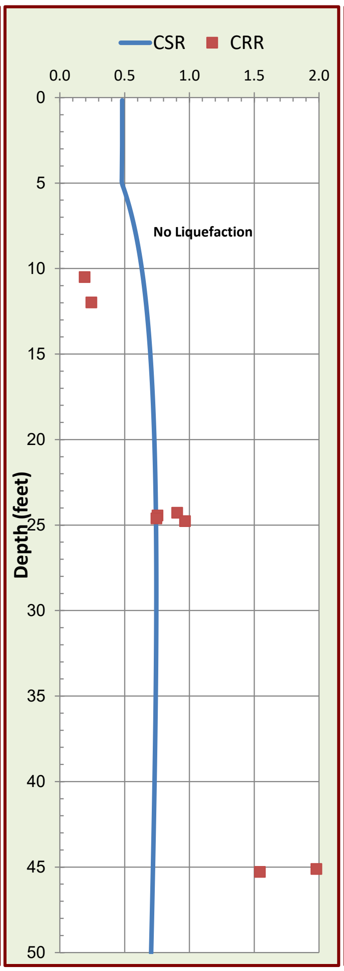
LDI[1] Corrected for Distance 0.00 (4 < L/H < 40)

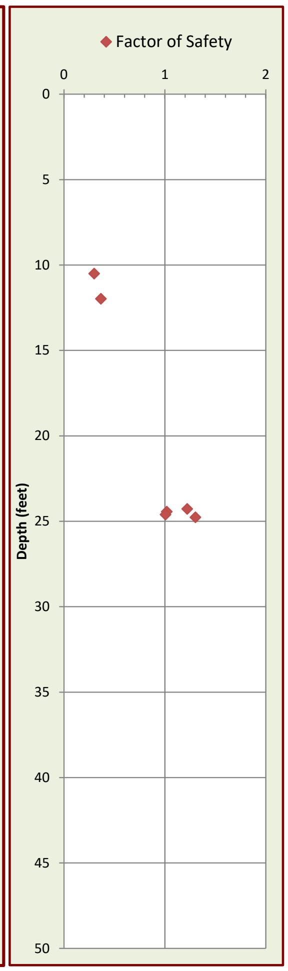
EXPECTED RANGE OF DISPLACEMENT

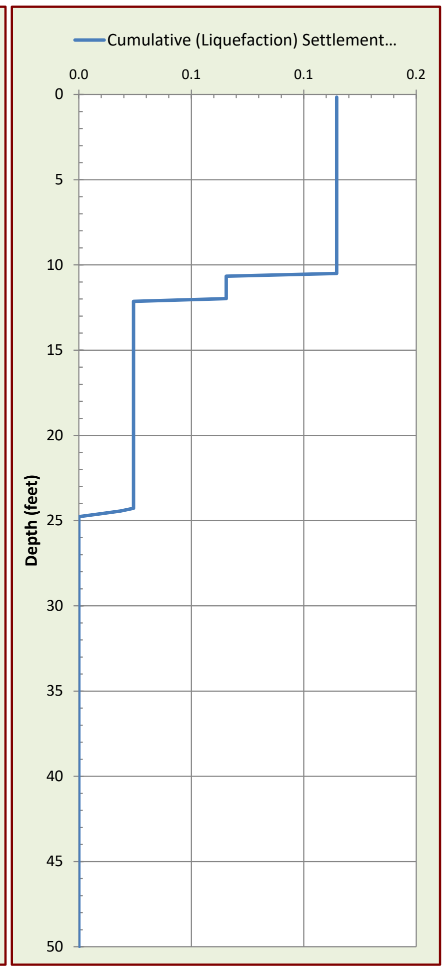
0.0 to **0.0** feet

[1]Not Valid for L/H Values < 4 and > 40.
[2]LDI Values Only Summed to 2H Below Grade.









APPENDIX A: FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program using truck-mounted, hollow-stem auger drilling equipment and 25-ton truck-mounted Cone Penetration Test equipment. Two 8-inch-diameter exploratory borings were drilled on Jul 13, 2022 to depths of 40 to 80 feet. Three CPT soundings were also performed in accordance with ASTM D 5778-95 (revised, 2002) on July 8, 2022, to depths ranging from 80 to 125 feet. The approximate locations of exploratory borings and CPTs are shown on the Site Plan, Figure 2. The soils encountered were continuously logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D2488). Boring logs, as well as a key to the classification of the soil and bedrock, are included as part of this appendix.

Boring and CPT locations were approximated using existing site boundaries, a hand held GPS unit, and other site features as references. Boring and CPT elevations were not determined. The locations of the borings and CPTs should be considered accurate only to the degree implied by the method used.

Representative soil samples were obtained from the borings at selected depths. All samples were returned to our laboratory for evaluation and appropriate testing. The standard penetration resistance blow counts were obtained by dropping a 140-pound hammer through a 30-inch free fall. The 2-inch O.D. split-spoon sampler was driven 18 inches and the number of blows was recorded for each 6 inches of penetration (ASTM D1586). 2.5-inch I.D. samples were obtained using a Modified California Sampler driven into the soil with the 140-pound hammer previously described. Relatively undisturbed samples were also obtained with 2.875-inch I.D. Shelby Tube sampler which were hydraulically pushed. Unless otherwise indicated, the blows per foot recorded on the boring log represent the accumulated number of blows required to drive the last 12 inches. The various samplers are denoted at the appropriate depth on the boring logs.

The CPT involved advancing an instrumented cone-tipped probe into the ground while simultaneously recording the resistance at the cone tip (q_c) and along the friction sleeve (f_s) at approximately 5-centimeter intervals. Based on the tip resistance and tip to sleeve ratio (R_f), the CPT classified the soil behavior type and estimated engineering properties of the soil, such as equivalent Standard Penetration Test (SPT) blow count, internal friction angle within sand layers, and undrained shear strength in silts and clays. A pressure transducer behind the tip of the CPT cone measured pore water pressure (u_2). Graphical logs of the CPT data is included as part of this appendix.

Field tests included an evaluation of the unconfined compressive strength of the soil samples using a pocket penetrometer device. The results of these tests are presented on the individual boring logs at the appropriate sample depths.

Attached boring and CPT logs and related information depict subsurface conditions at the locations indicated and on the date designated on the logs. Subsurface conditions at other locations may differ from conditions occurring at these boring and CPT locations. The passage of time may result in altered subsurface conditions due to environmental changes. In addition, any stratification lines on the logs represent the approximate boundary between soil types and the transition may be gradual.

UNIFIED SOIL CLASSIFICATION (ASTM D-2487-98)

MATERIAL TYPES	CRITERIA FOR ASSIGNING SOIL GROUP NAMES			GROUP SYMBOL	SOIL GROUP NAMES & LEGEND
COARSE-GRAINED SOILS >50% RETAINED ON NO. 200 SIEVE	GRAVELS >50% OF COARSE FRACTION RETAINED ON NO 4. SIEVE	CLEAN GRAVELS <5% FINES	Cu>4 AND 1<Cc<3	GW	WELL-GRADED GRAVEL
			Cu>4 AND 1>Cc>3	GP	POORLY-GRADED GRAVEL
		GRAVELS WITH FINES >12% FINES	FINES CLASSIFY AS ML OR CL	GM	SILTY GRAVEL
			FINES CLASSIFY AS CL OR CH	GC	CLAYEY GRAVEL
	SANDS >50% OF COARSE FRACTION PASSES ON NO 4. SIEVE	CLEAN SANDS <5% FINES	Cu>6 AND 1<Cc<3	SW	WELL-GRADED SAND
			Cu>6 AND 1>Cc>3	SP	POORLY-GRADED SAND
		SANDS AND FINES >12% FINES	FINES CLASSIFY AS ML OR CL	SM	SILTY SAND
			FINES CLASSIFY AS CL OR CH	SC	CLAYEY SAND
FINE-GRAINED SOILS >50% PASSES NO. 200 SIEVE	SILTS AND CLAYS LIQUID LIMIT<50	INORGANIC	PI>7 AND PLOTS>"A" LINE	CL	LEAN CLAY
			PI>4 AND PLOTS<"A" LINE	ML	SILT
		ORGANIC	LL (oven dried)/LL (not dried)<0.75	OL	ORGANIC CLAY OR SILT
	SILTS AND CLAYS LIQUID LIMIT>50	INORGANIC	PI PLOTS >"A" LINE	CH	FAT CLAY
			PI PLOTS <"A" LINE	MH	ELASTIC SILT
		ORGANIC	LL (oven dried)/LL (not dried)<0.75	OH	ORGANIC CLAY OR SILT
HIGHLY ORGANIC SOILS	PRIMARILY ORGANIC MATTER, DARK IN COLOR, AND ORGANIC ODOR			PT	PEAT

OTHER MATERIAL SYMBOLS

- Poorly-Graded Sand with Clay
- Clayey Sand
- Sandy Silt
- Artificial/Undocumented Fill
- Poorly-Graded Gravelly Sand
- Topsoil
- Well-Graded Gravel with Clay
- Well-Graded Gravel with Silt
- Sand
- Silt
- Well Graded Gravelly Sand
- Gravelly Silt
- Asphalt
- Boulders and Cobble

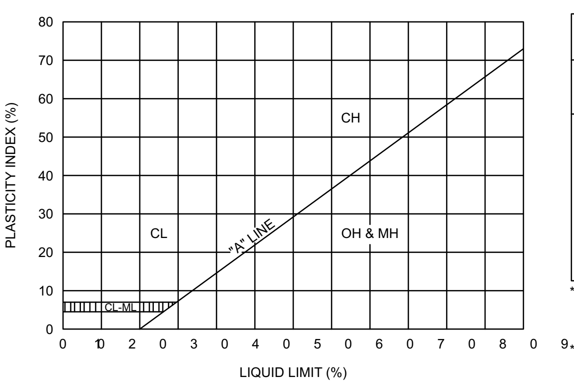
SAMPLER TYPES

- SPT
- Modified California (2.5" I.D.)
- Rock Core
- Shelby Tube
- No Recovery
- Grab Sample

ADDITIONAL TESTS

CA	-	CHEMICAL ANALYSIS (CORROSIVITY)
CD	-	CONSOLIDATED DRAINED TRIAXIAL
CN	-	CONSOLIDATION
CU	-	CONSOLIDATED UNDRAINED TRIAXIAL
DS	-	DIRECT SHEAR
PP	-	POCKET PENETROMETER (TSF)
(3.0)	-	(WITH SHEAR STRENGTH IN KSF)
RV	-	R-VALUE
SA	-	SIEVE ANALYSIS: % PASSING #200 SIEVE
▼	-	WATER LEVEL
PI	-	PLASTICITY INDEX
SW		SWELL TEST
TC	-	CYCLIC TRIAXIAL
TV	-	TORVANE SHEAR
UC	-	UNCONFINED COMPRESSION
(1.5)	-	(WITH SHEAR STRENGTH IN KSF)
UU	-	UNCONSOLIDATED UNDRAINED TRIAXIAL

PLASTICITY CHART



PENETRATION RESISTANCE (RECORDED AS BLOWS / FOOT)				
SAND & GRAVEL		SILT & CLAY		
RELATIVE DENSITY	BLOWS/FOOT*	CONSISTENCY	BLOWS/FOOT*	STRENGTH** (KSF)
VERY LOOSE	0 - 4	VERY SOFT	0 - 2	0 - 0.25
LOOSE	4 - 10	SOFT	2 - 4	0.25 - 0.5
MEDIUM DENSE	10 - 30	MEDIUM STIFF	4 - 8	0.5 - 1.0
DENSE	30 - 50	STIFF	8 - 15	1.0 - 2.0
VERY DENSE	OVER 50	VERY STIFF	15 - 30	2.0 - 4.0
		HARD	OVER 30	OVER 4.0

* NUMBER OF BLOWS OF 140 LB HAMMER FALLING 30 INCHES TO DRIVE A 2 INCH O.D. (1-3/8 INCH I.D.) SPLIT-BARREL SAMPLER THE LAST 12 INCHES OF AN 18-INCH DRIVE (ASTM-1586 STANDARD PENETRATION TEST).

** UNDRAINED SHEAR STRENGTH IN KIPS/SQ. FT. AS DETERMINED BY LABORATORY TESTING OR APPROXIMATED BY THE STANDARD PENETRATION TEST, POCKET PENETROMETER, TORVANE, OR VISUAL OBSERVATION.

CORNERSTONE EARTH GROUP

LEGEND TO SOIL DESCRIPTIONS

Figure Number A-1

BORING NUMBER EB-1

CORNERSTONE EARTH GROUP

PROJECT NAME Eden Landing Road
PROJECT NUMBER 234-49-1
PROJECT LOCATION Hayward, CA

DATE STARTED 7/13/22 **DATE COMPLETED** 7/13/22
GROUND ELEVATION **BORING DEPTH** 40 ft.
DRILLING CONTRACTOR Exploration Geoservices Inc.
LATITUDE 37.626849° **LONGITUDE** -122.119179°
DRILLING METHOD Mobile B-61, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY EA
▽ **AT TIME OF DRILLING** 13 ft.
NOTES
▼ **AT END OF DRILLING** 9 ft.

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

UNDRAINED SHEAR STRENGTH, ksf: ○ HAND PENETROMETER; △ TORVANE; ● UNCONFINED COMPRESSION; ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL (scale 1.0, 2.0, 3.0, 4.0)

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
	0	4 inches asphalt concrete over 4 inches aggregate base							
		Lean Clay (CL) very stiff, moist, dark brown, trace fine sand, moderate plasticity Liquid Limit = 39, Plastic Limit = 18	9	MC-1B	94	23	21		○
			19	MC-2B	104	20			○
	5	**Lean Clay with Sand (CL)** very stiff, moist, brown with gray mottles, fine sand, moderate plasticity	19	MC-3B	109	18			○
		Silty Sand (SM) loose, moist, brown, fine sand							
	▼	**Lean Clay with Sand (CL)** medium stiff, moist, gray, fine sand, low plasticity	7	MC-4B	97	25			○
	10				84	29			○
				ST-5					
	▽	**Clayey Sand (SC)** loose, wet, brown, fine sand, some fine subrounded gravel	8	MC					○
	15	**Lean Clay with Sand (CL)** medium stiff, moist, brown, fine sand, low plasticity							
		Lean Clay (CL) very stiff, moist, brown, some fine sand, moderate plasticity	16	MC-7B	108	20			○
	20								
		becomes stiff	14	MC					○
	25								

Continued Next Page

BORING NUMBER EB-1

CORNERSTONE EARTH GROUP

PROJECT NAME Eden Landing Road

PROJECT NUMBER 234-49-1

PROJECT LOCATION Hayward, CA

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

UNDRAINED SHEAR STRENGTH, ksf: ○ HAND PENETROMETER; △ TORVANE; ● UNCONFINED COMPRESSION; ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL (scale 1.0, 2.0, 3.0, 4.0)

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
	25	**Lean Clay (CL)** very stiff, moist, brown, some fine sand, moderate plasticity	13	MC-9B	113	17			○ (hand penetrometer)
	29.5	**Sandy Lean Clay (CL)** very stiff, moist, brown, fine sand, some fine subrounded gravel, low plasticity		ST					
	32	**CLayey Sand (SC)** medium dense, moist, brown, fine to medium sand, fine subrounded gravel							
	33	**Well Graded Sand with Silt and Gravel (SW-SM)** loose, wet, brown, fine to coarse sand, fine to coarse subangular to subrounded gravel	16	MC					
			9	SPT					
	38	**Lean Clay (CL)** stiff, moist, brown, some fine to medium sand, moderate plasticity	15	SPT-13		23			○ (hand penetrometer)
	40	Bottom of Boring at 40.0 feet.							

BORING NUMBER EB-2

CORNERSTONE EARTH GROUP

PROJECT NAME Eden Landing Road
PROJECT NUMBER 234-49-1
PROJECT LOCATION Hayward, CA

DATE STARTED 7/13/22 **DATE COMPLETED** 7/13/22
GROUND ELEVATION **BORING DEPTH** 80 ft.
DRILLING CONTRACTOR Exploration Geoservices Inc.
LATITUDE 37.626576° **LONGITUDE** -122.119919°
DRILLING METHOD Mobile B-61, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY EA
∇ **AT TIME OF DRILLING** 10.8 ft.
NOTES
▼ **AT END OF DRILLING** 8 ft.

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UNDRAINED SHEAR STRENGTH, ksf: ○ HAND PENETROMETER; △ TORVANE; ● UNCONFINED COMPRESSION; ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL (scale 1.0, 2.0, 3.0, 4.0)

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION
	0	4 inches asphalt concrete over 4 inches aggregate base
		Sandy Lean Clay (CL) [Fill] very stiff, moist, brown, fine to coarse sand, moderate plasticity
		Lean Clay (CL) very stiff, moist, dark brown, trace fine sand, moderate plasticity. Liquid Limit = 47, Plastic Limit = 16
	5	**Sandy Lean Clay (CL)** very stiff, moist, brown, fine sand, low plasticity
		Lean Clay with Sand (CL) medium stiff, moist, brown with gray mottles, fine sand, low plasticity
	10	
		Sandy Lean Clay (CL) medium stiff, moist, brown, fine sand, low plasticity
	15	**Lean Clay (CL)** very stiff, moist, brown with dark brown mottles, fine sand, moderate plasticity
	20	
	25	*Continued Next Page*

N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
20	MC-1B	93	21	31	
20	MC-2C	102	20		
13	MC-3C	105	15		
5	SPT				
9	MC-5C	95	27		
8	MC-6C	94	28		80
6	MC-7C	95	27		
8	SPT				
20	MC-9B	112	17		
24	MC-10B	111	19		

BORING NUMBER EB-2

CORNERSTONE EARTH GROUP

PROJECT NAME Eden Landing Road

PROJECT NUMBER 234-49-1

PROJECT LOCATION Hayward, CA

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

UNDRAINED SHEAR STRENGTH, ksf: ○ HAND PENETROMETER; △ TORVANE; ● UNCONFINED COMPRESSION; ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL (scale 1.0, 2.0, 3.0, 4.0)

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
	25		**Lean Clay with Sand (CL)** stiff, moist, brown, fine sand, moderate plasticity		ST-11	107	20		
				11	MC				
	30								
			Lean Clay (CL) very stiff, moist, brown with gray mottles, trace fine sand, moderate plasticity						
				16	MC-13B	110	19		
	35								
			Lean Clay with Sand (CL) stiff, moist, brown with gray mottles, fine sand, low plasticity						
				15	MC-14B				
	40								
			Lean Clay (CL) very stiff, moist, brown with gray mottles, trace fine sand, moderate plasticity						
				14	MC				
	45								
			Lean Clay with Sand (CL) stiff, moist, brown, fine sand, low plasticity						
				17	MC-16B	96	28		
	50								
					ST				

Continued Next Page

CORNERSTONE EARTH GROUP2 - CORNERSTONE 0812.GDT - 7/28/22 14:07 - P:\DRAFTING\GINT FILES\234-49-1 EDEN LANDING RD.GPJ

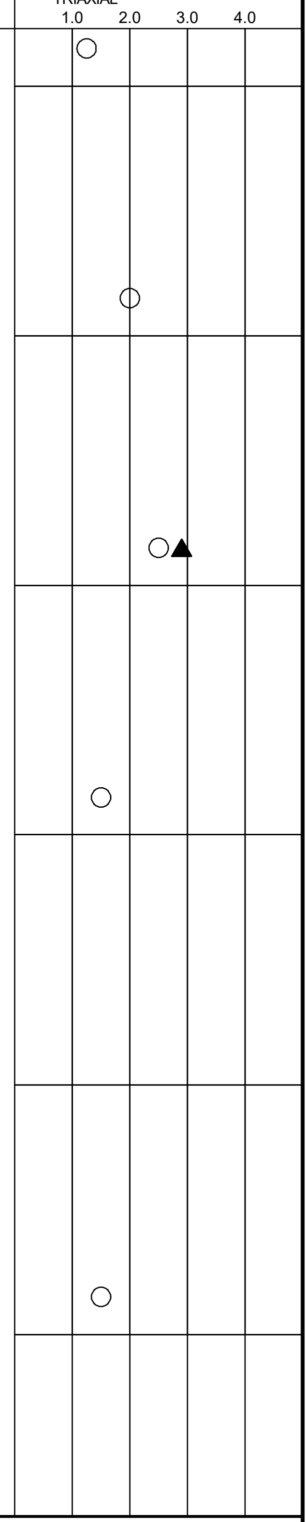
CORNERSTONE EARTH GROUP

PROJECT NAME Eden Landing Road
PROJECT NUMBER 234-49-1
PROJECT LOCATION Hayward, CA

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
			Lean Clay with Sand (CL) stiff, moist, brown, fine sand, low plasticity	18	MC				
	55								
				20	MC-19B	101	23		
	60								
			becomes very stiff	45	MC-20B	116	17		
	65								
			becomes stiff	15	SPT-21		19		
	70								
			Poorly Graded Sand (SP) medium dense to loose, wet, brown, fine sand, trace fines	18	MC-22C	104	19		
	75								
				19	SPT-23B		27		
	80		**Lean Clay (CL)** stiff, moist, brown, trace fine sand, moderate plasticity						
			Bottom of Boring at 80.0 feet.						

UNDRAINED SHEAR STRENGTH, ksf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL

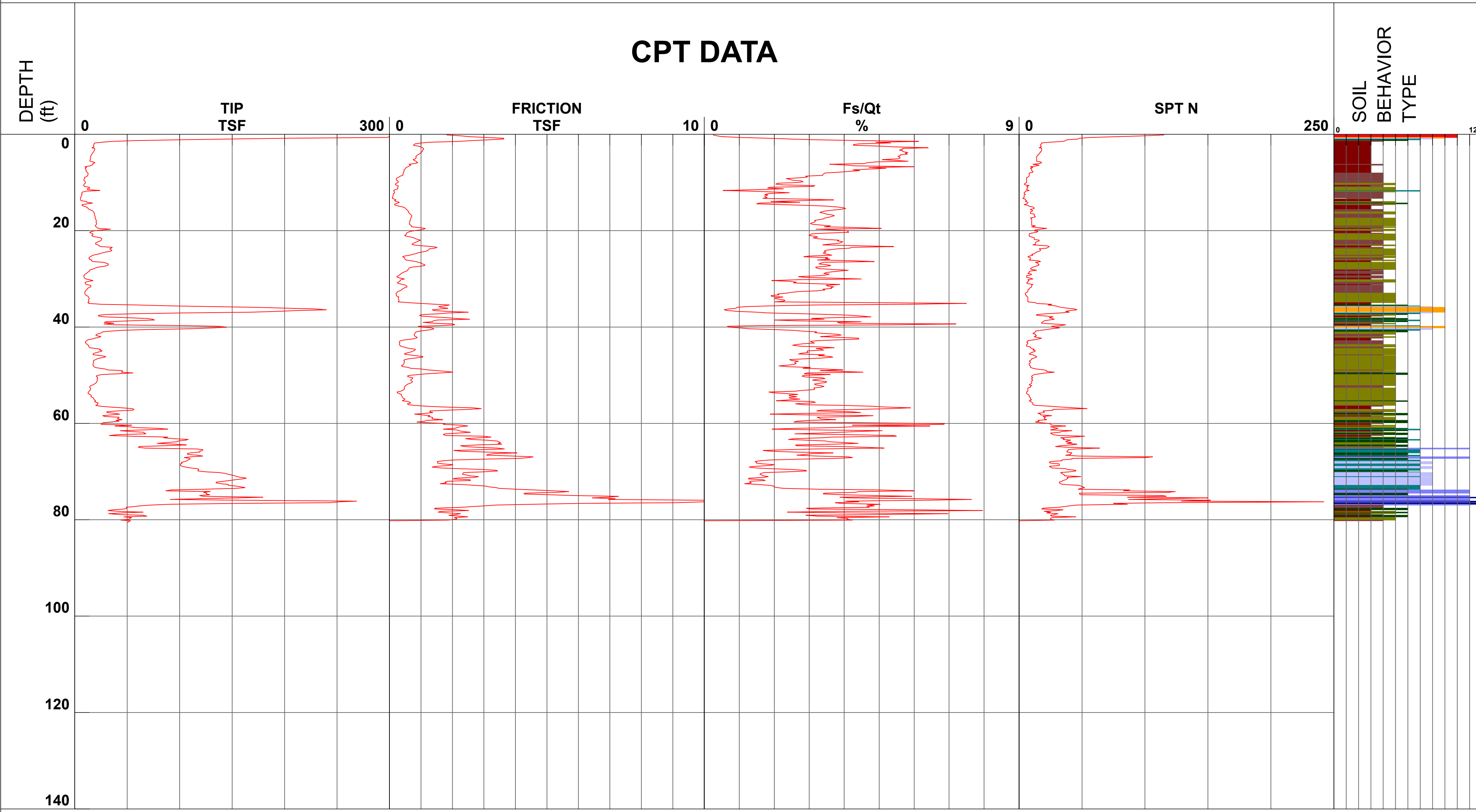


Cornerstone Earth Group

Project	Eden Landing Road GI	Operator	AJ-IM	Filename	SDF(952).cpt
Job Number	234-49-1	Cone Number	DDG1587	GPS	
Hole Number	CPT-01	Date and Time	7/8/2022 8:59:52 AM	Maximum Depth	80.54 ft
EST GW Depth During Test		7.00 ft			

Net Area Ratio .8

CPT DATA



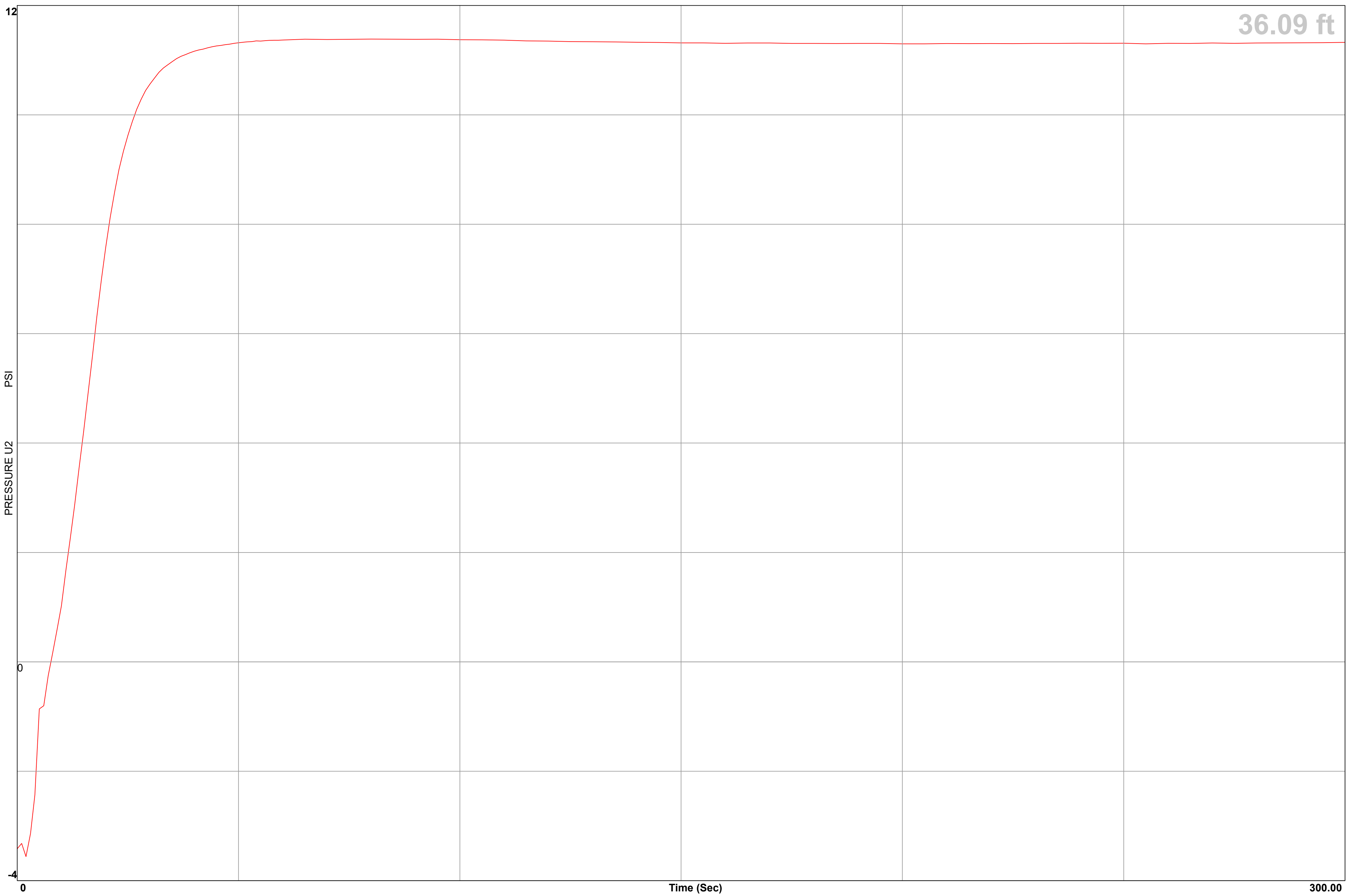
1 - sensitive fine grained	4 - silty clay to clay	7 - silty sand to sandy silt	10 - gravelly sand to sand
2 - organic material	5 - clayey silt to silty clay	8 - sand to silty sand	11 - very stiff fine grained (*)
3 - clay	6 - sandy silt to clayey silt	9 - sand	12 - sand to clayey sand (*)

Cone Size 15cm squared

S*Soil behavior type and SPT based on data from UBC-1983

Cornerstone Earth Group

Location	Eden Landing Road GI	Operator	AJ-IM		
Job Number	234-49-1	Cone Number	DDG1587	GPS	
Hole Number	CPT-01	Date and Time	7/8/2022 8:59:52 AM		
Equilized Pressure	11.2	EST GW Depth During Test	10.1		

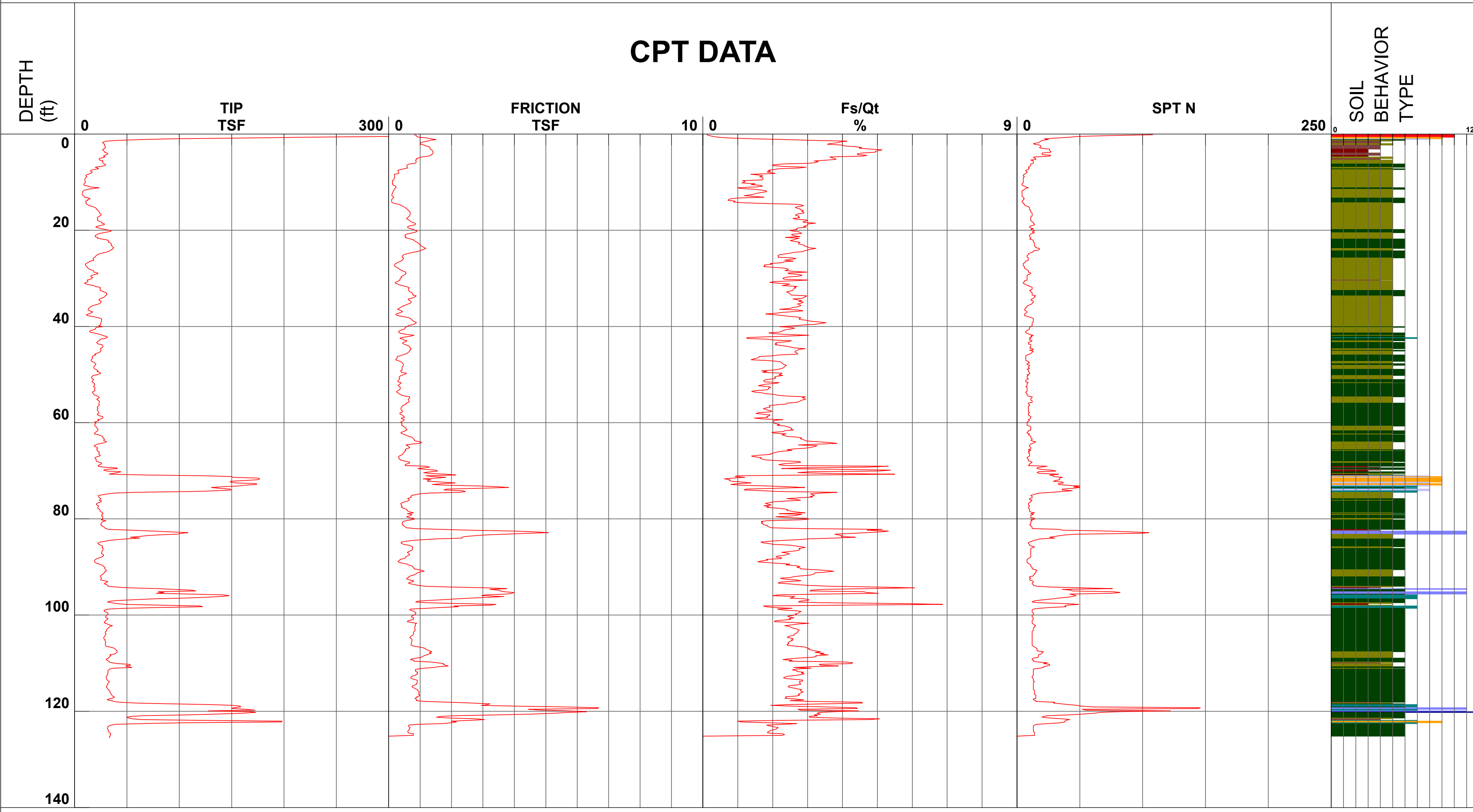


Cornerstone Earth Group

Project	Eden Landing Road GI	Operator	AJ-IM	Filename	SDF(951).cpt
Job Number	234-49-1	Cone Number	DDG1587	GPS	
Hole Number	CPT-02	Date and Time	7/8/2022 7:27:13 AM	Maximum Depth	125.49 ft
EST GW Depth During Test			9.00 ft		

Net Area Ratio .8

CPT DATA



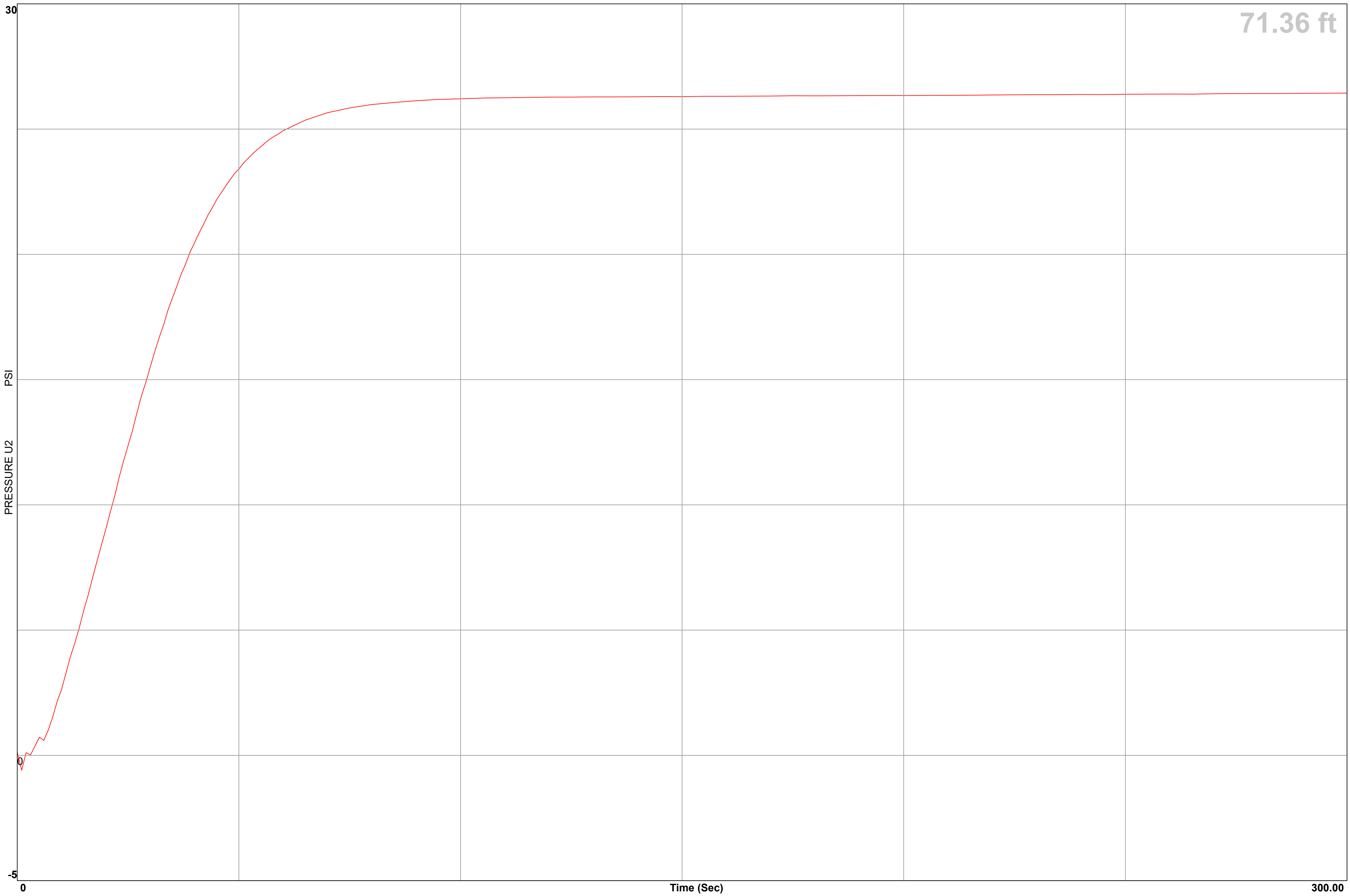
1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

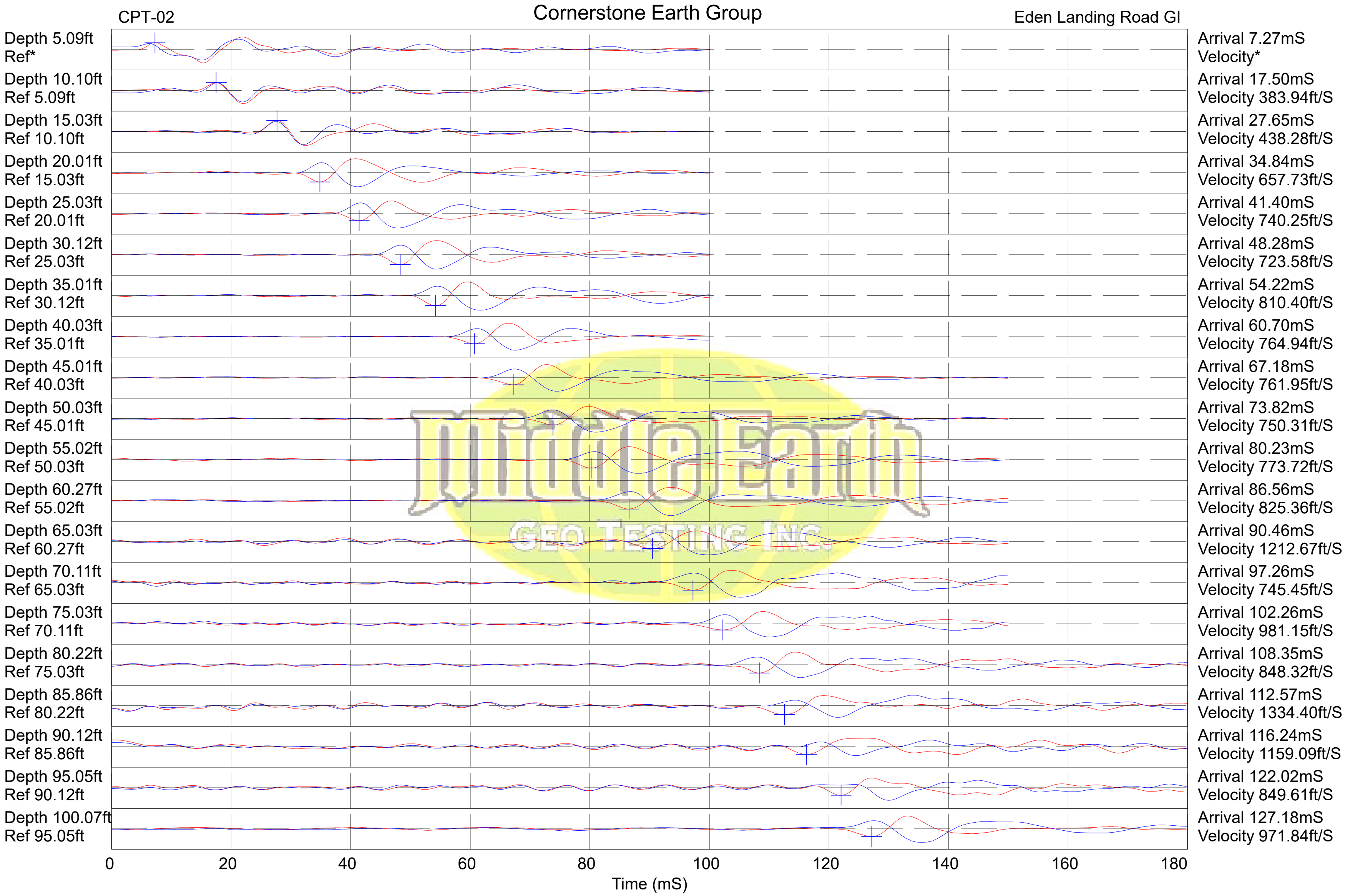
Cone Size 15cm squared

S*Soil behavior type and SPT based on data from UBC-1983

Cornerstone Earth Group

Location	Eden Landing Road GI	Operator	AJ-IM		
Job Number	234-49-1	Cone Number	DDG1587	GPS	
Hole Number	CPT-02	Date and Time	7/8/2022 7:27:13 AM		
Equilized Pressure	26.3	EST GW Depth During Test	10.4		

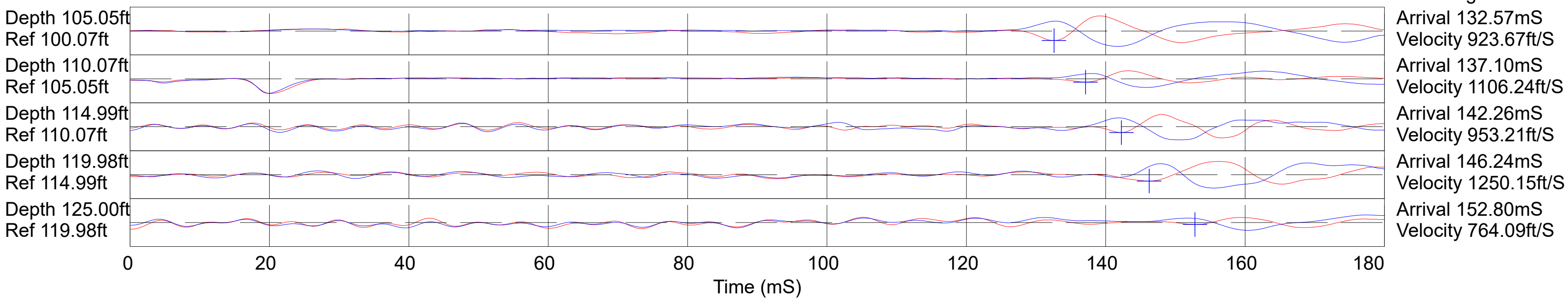




Hammer to Rod String Distance (ft): 5.83
* = Not Determined

COMMENT:

CPT-02 SEISMIC TEST Eden Landing Road GI



Hammer to Rod String Distance (ft): 5.83
* = Not Determined

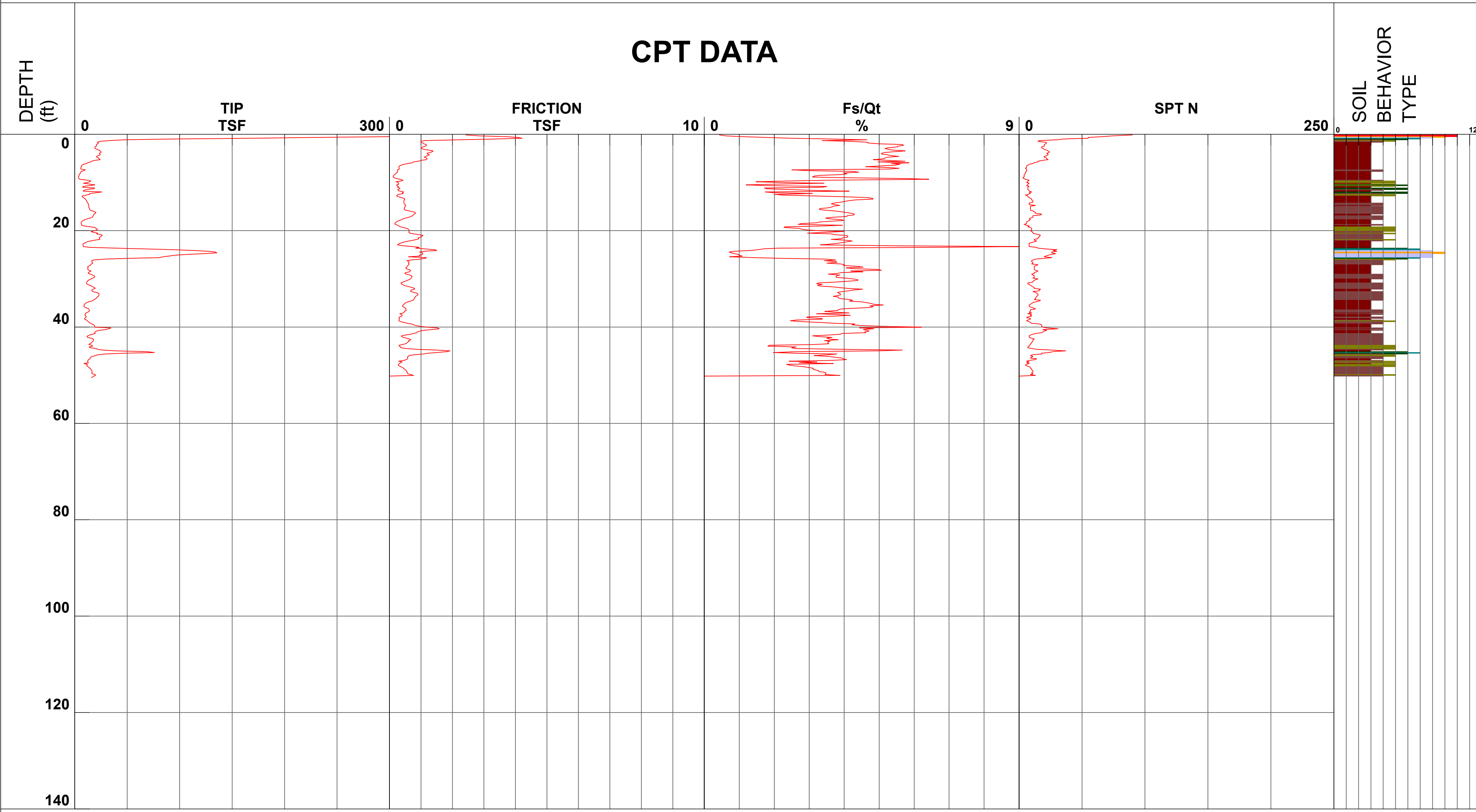
COMMENT:

Cornerstone Earth Group

Project	Eden Landing Road GI	Operator	AJ-IM	Filename	SDF(953).cpt
Job Number	234-49-1	Cone Number	DDG1587	GPS	
Hole Number	CPT-03	Date and Time	7/8/2022 9:57:02 AM	Maximum Depth	50.52 ft
EST GW Depth During Test		7.00 ft			

Net Area Ratio .8

CPT DATA



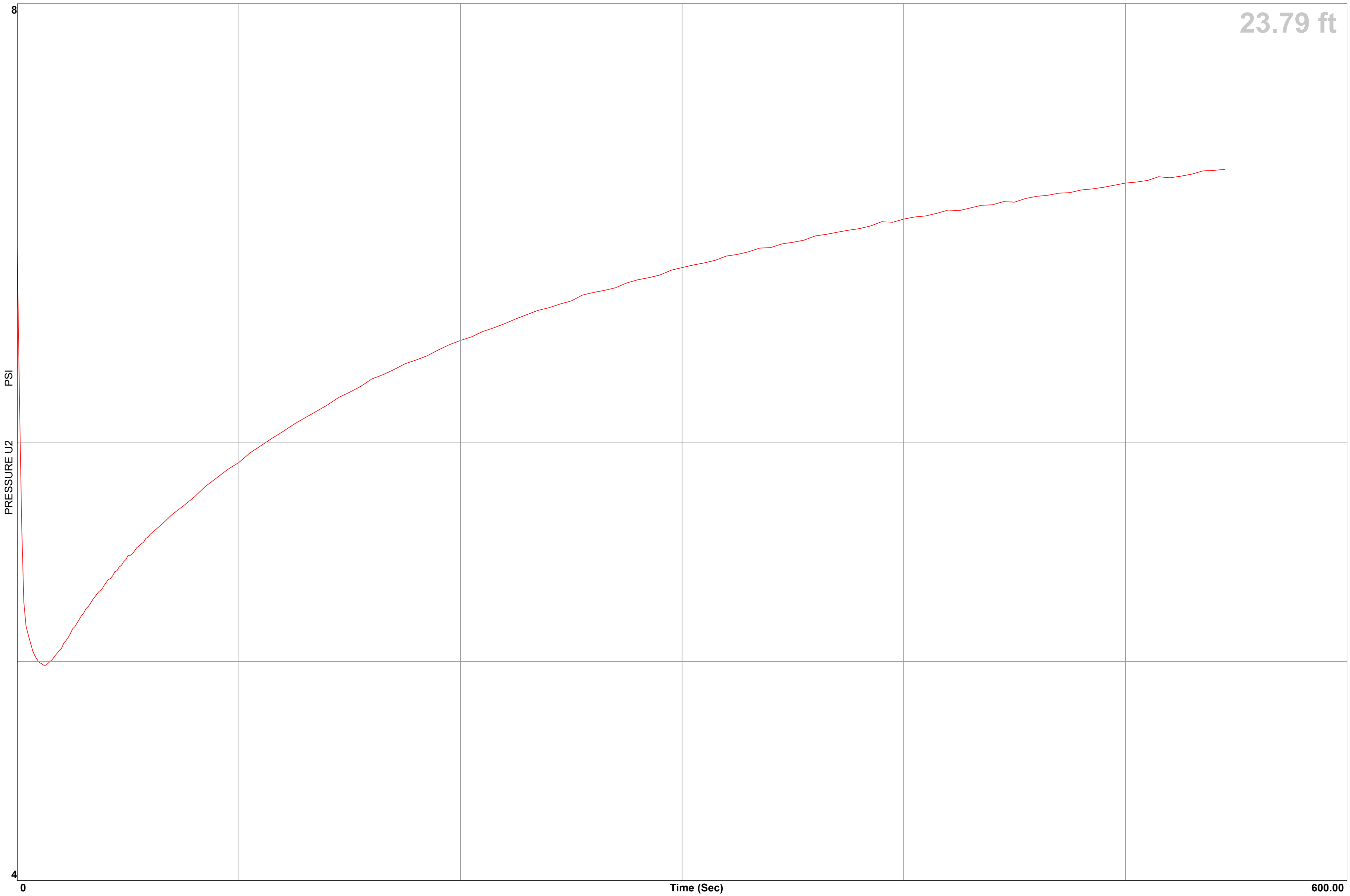
1 - sensitive fine grained	4 - silty clay to clay	7 - silty sand to sandy silt	10 - gravelly sand to sand
2 - organic material	5 - clayey silt to silty clay	8 - sand to silty sand	11 - very stiff fine grained (*)
3 - clay	6 - sandy silt to clayey silt	9 - sand	12 - sand to clayey sand (*)

Cone Size 15cm squared

S*Soil behavior type and SPT based on data from UBC-1983

Cornerstone Earth Group

Location	Eden Landing Road GI	Operator	AJ-IM		
Job Number	234-49-1	Cone Number	DDG1587	GPS	
Hole Number	CPT-03	Date and Time	7/8/2022 9:57:02 AM		
Equilized Pressure	7.2	EST GW Depth During Test	7.0		



APPENDIX B: LABORATORY TEST PROGRAM

The laboratory testing program was performed to evaluate the physical and mechanical properties of the soils retrieved from the site to aid in verifying soil classification.

Moisture Content: The natural water content was determined (ASTM D2216) on 23 samples of the materials recovered from the borings. These water contents are recorded on the boring logs at the appropriate sample depths.

Dry Densities: In place dry density determinations (ASTM D2937) were performed on 21 samples to measure the unit weight of the subsurface soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

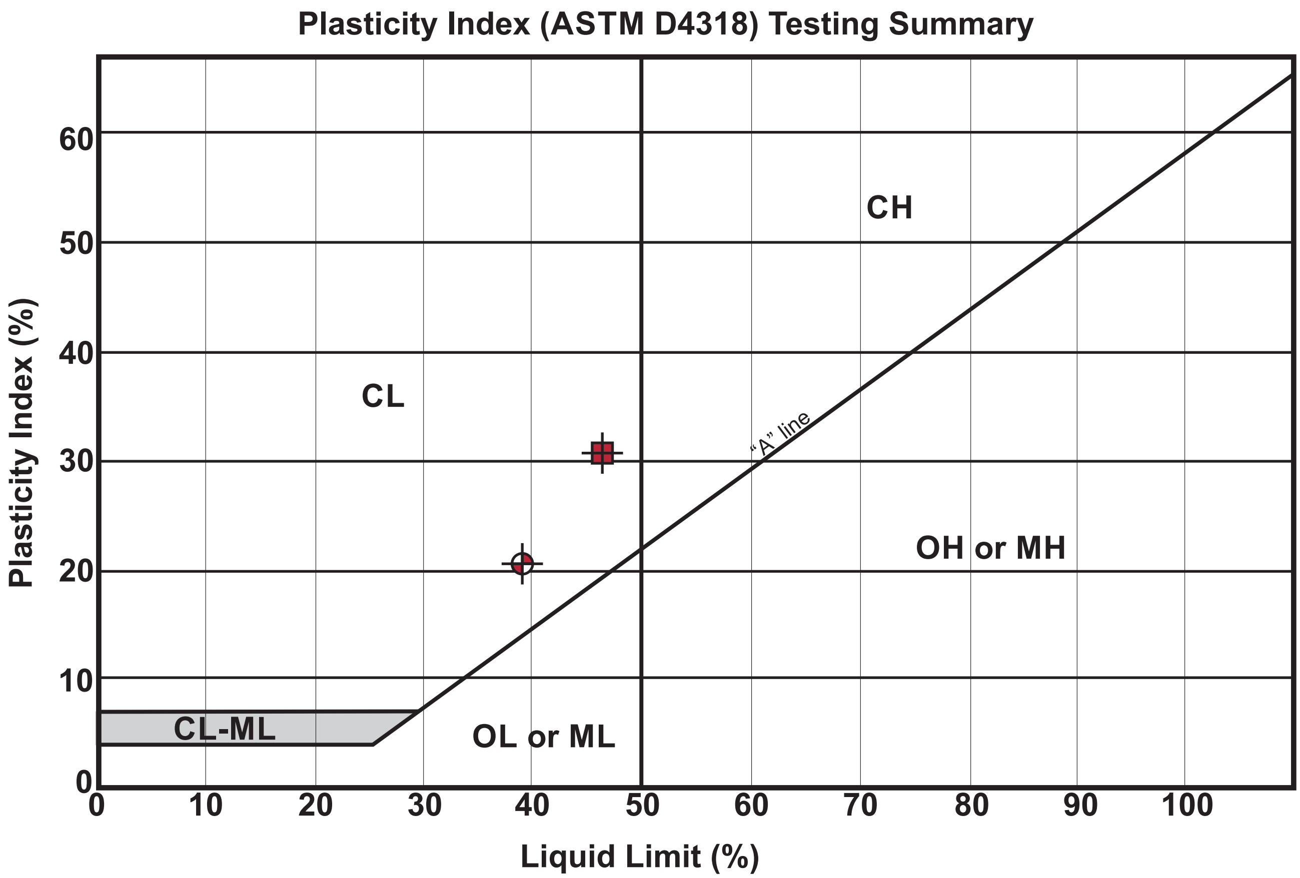
Washed Sieve Analyses: The percent soil fraction passing the No. 200 sieve (ASTM D1140) was determined on one sample of the subsurface soils to aid in the classification of these soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

Plasticity Index: Two Plasticity Index determinations (ASTM D4318) were performed on samples of the subsurface soils to measure the range of water contents over which this material exhibits plasticity. The Plasticity Index was used to classify the soil in accordance with the Unified Soil Classification System and to evaluate the soil expansion potential. Results of these tests are shown on the boring logs at the appropriate sample depths.

Undrained-Unconsolidated Triaxial Shear Strength: The undrained shear strength was determined on two relatively undisturbed sample(s) by unconsolidated-undrained triaxial shear strength testing (ASTM D2850). The results of this test are included as part of this appendix.

Consolidation: One consolidation test (ASTM D2435) was performed on a relatively undisturbed sample of the subsurface clayey soils to assist in evaluating the compressibility property of this soil. Results of the consolidation test are presented graphically in this appendix.

Corrosion: Corrosion test suites were performed on two samples from depths of 3 to 5½ feet. Corrosion tests included: 100% saturated resistivity (ASTM G57), chloride (Caltrans 422), soluble sulfate (Caltrans 417), pH (ASTM G51), Redox (SM 2580B), and sulfide.



Symbol	Boring No.	Depth (ft)	Natural Water Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index	Passing No. 200 (%)	Group Name (USCS - ASTM D2487)
⊕	EB-1	2.0	23	39	18	21	—	Lean Clay (CL)
■	EB-2	2.0	21	47	16	31	—	Lean Clay (CL)

CORNERSTONE EARTH GROUP

Plasticity Index Testing Summary

Eden Landing Business Park
Hayward, CA

Project Number 234-49-1

Figure Number Figure B1

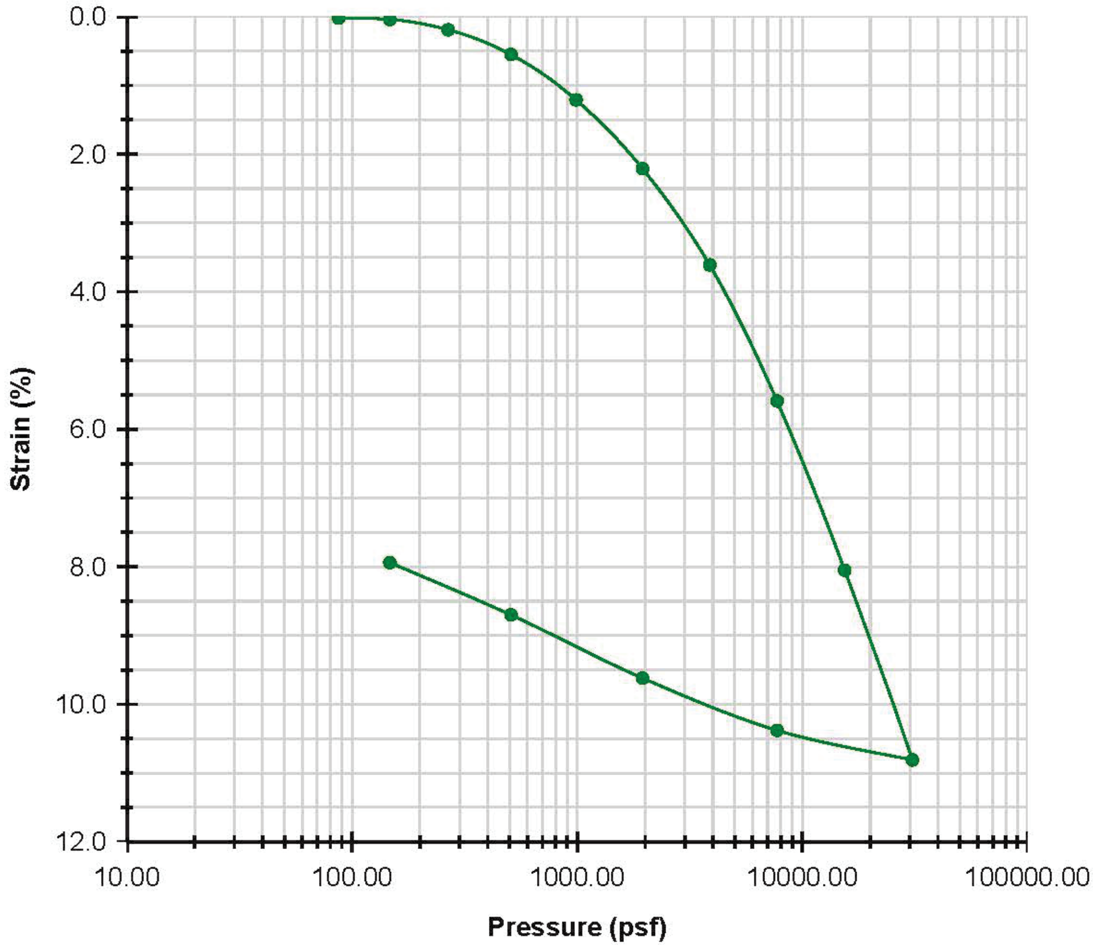
Date July 2022

Drawn By FLL

Consolidation Test ASTM D2435

Boring: EB-2 **Sample:** 11 **Depth:** 27.5’

Desription: Lean Clay with Sand (CL)



	BEFORE	AFTER
Moisture (%)	19.5	16.7
Dry Density (pcf)	106.9	116.8
Saturation (%)	90.0	100.0
Void Ratio	0.59	0.45

(A) Stress Strain Curve

CORNERSTONE EARTH GROUP

Strain-Log Curve - EB-2 @ 27.5’

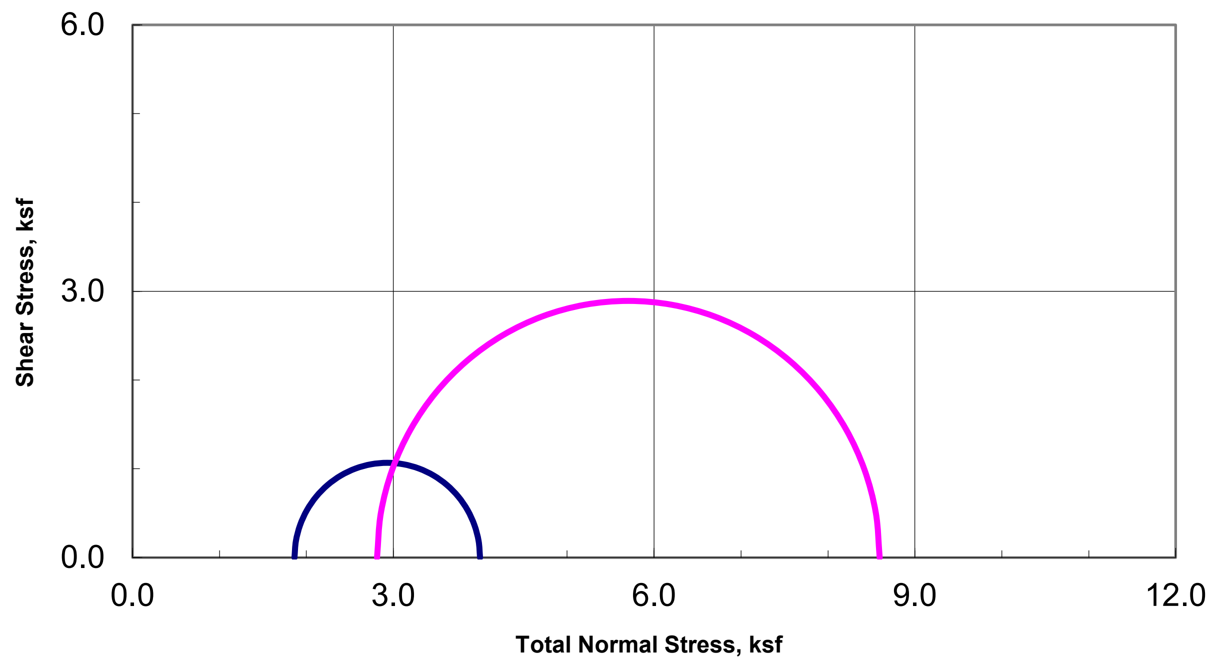
Eden Landing Business Park
Hayward, CA

Project Number	234-49-1
Figure Number	Figure B2
Date	July 2022
Drawn By	FLL

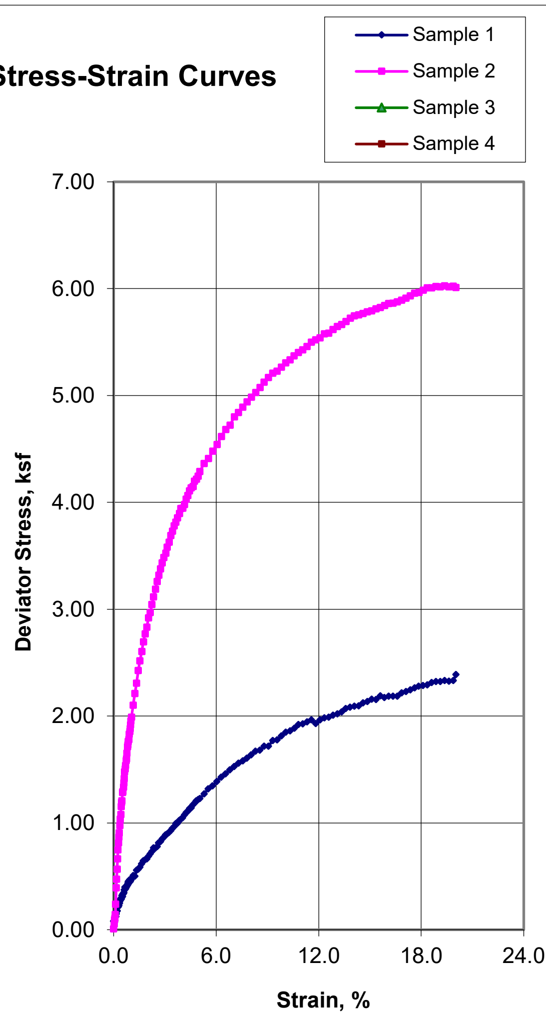
Cooper Testing Labs, Inc.
937 Commercial Street
Palo Alto, CA 94303

COOPER TESTING LABORATORY

Unconsolidated-Undrained Triaxial Test
ASTM D2850



Stress-Strain Curves



Sample Data				
	1	2	3	4
Moisture %	22.8	16.6		
Dry Den,pcf	103.8	116.0		
Void Ratio	0.624	0.453		
Saturation %	98.7	98.9		
Height in	5.00	5.10		
Diameter in	2.41	2.41		
Cell psi	12.9	19.5		
Strain %	15.00	15.00		
Deviator, ksf	2.135	5.782		
Rate %/min	1.00	1.00		
in/min	0.050	0.051		
Job No.:	**640-1511**			
Client:	**Cornerstone Earth Group**			
Project:	**234-49-1**			
Boring:	EB-2	EB-2		
Sample:	14B	20B		
Depth ft:	39.5	64.5		

Visual Soil Description	
Sample #	
1	Yellowish Brown Sandy CLAY
2	Yellowish Brown CLAY w/ Sand
3	
4	

Remarks:

Note: Strengths are picked at the peak deviator stress or 15% strain which ever occurs first per ASTM D2850.

Corrosivity Tests Summary

CORNERSTONE EARTH GROUP

Job Number	234-49-1	Date Tested	7/27/2022
Job Name	Eden Landing Road	Tested By	BBA, FL
Location	Hayward, CA		

Sample I.D.			Soil Visual Description	Moisture Content %	pH	Temp. at Testing C°	Resistivity (Ohm-cm) Corrected to 15.5 C°		Chloride Dry Wt. mg/kg	Sulfate Dry Wt. mg/kg
Boring	Sample No.	Depth, ft.					As Received	Saturated		
				ASTM D2216	ASTM G51		G57	ASTM G57	ASTM D4327	ASTM D4327
EB-1	3A	5.5	Brown Lean Clay with Sand (CL)	17.2	6.9	22.6	-	1,096	12	58
EB-2	2A	3.0	Dark Brown Lean Clay (CL)	19.1	6.5	21.9	-	607	91	75

APPENDIX C: THERMAL RESISTIVITY EVALUATION

Geotherm USA

21239 FM529 Rd., Bldg. F
Cypress, TX 77433
Tel: 281-985-9344
Fax: 832-427-1752
info@geothermusa.com
http://www.geothermusa.com

July 22, 2022

Cornerstone Earth Group
1259 Oakmead Pkwy
Sunnyvale, California 94085
Attn: Maura Ruffatto

Re: Thermal Analysis of Native Soil Samples (Project No.234-49-1)
Eden Landing Due Diligence – Hayward, CA

The following is the report of thermal dryout characterization tests conducted on the four (4) tube samples of native soil from the referenced project sent to our laboratory.

Thermal Resistivity Tests: The tube samples were tested 'as received'. The tests were conducted in accordance with the IEEE standard 442-2017. The results are tabulated below and the thermal dryout curves are presented in **Figures 1 to 4.**

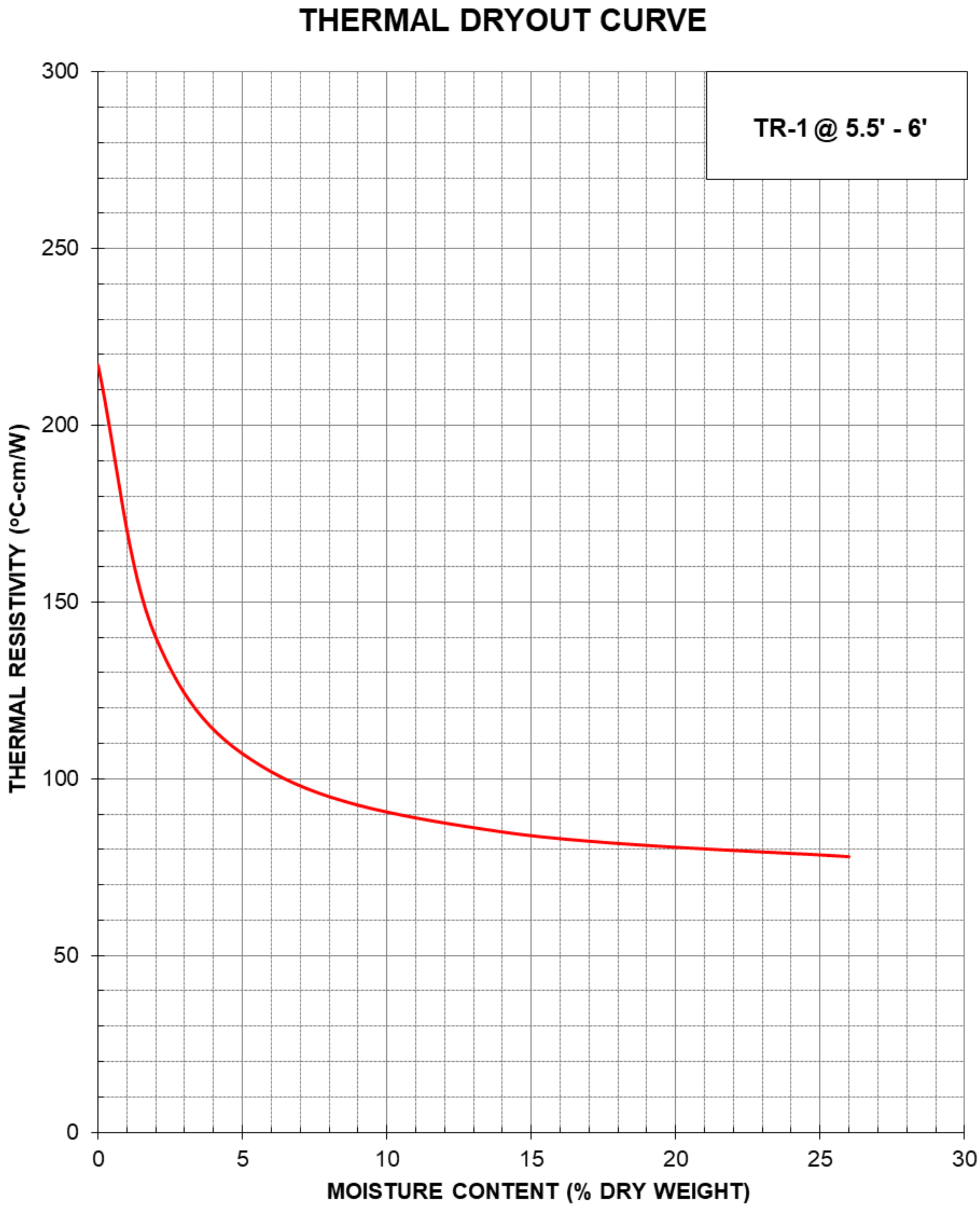
Sample ID, Description, Thermal Resistivity, Moisture Content and Density

Sample ID	Depth (ft)	Description (Cornerstone)	Thermal Resistivity (°C-cm/W)		Moisture Content (%)	Dry Density (lb/ft^3)
			Wet	Dry		
TR-1	5.5 - 6	Sandy lean clay (CL)	78	217	26	92
TR-2	5.5 - 6	Sandy lean clay (CL)	79	170	15	106
TR-3	5.5 - 6	Sandy lean clay (CL)	63	173	3	118
TR-4	5.5 - 6	Sandy lean clay (CL)	77	190	8	99

Please contact us if you have any questions or if we can be of further assistance.

Geotherm USA

Nimesh Patel



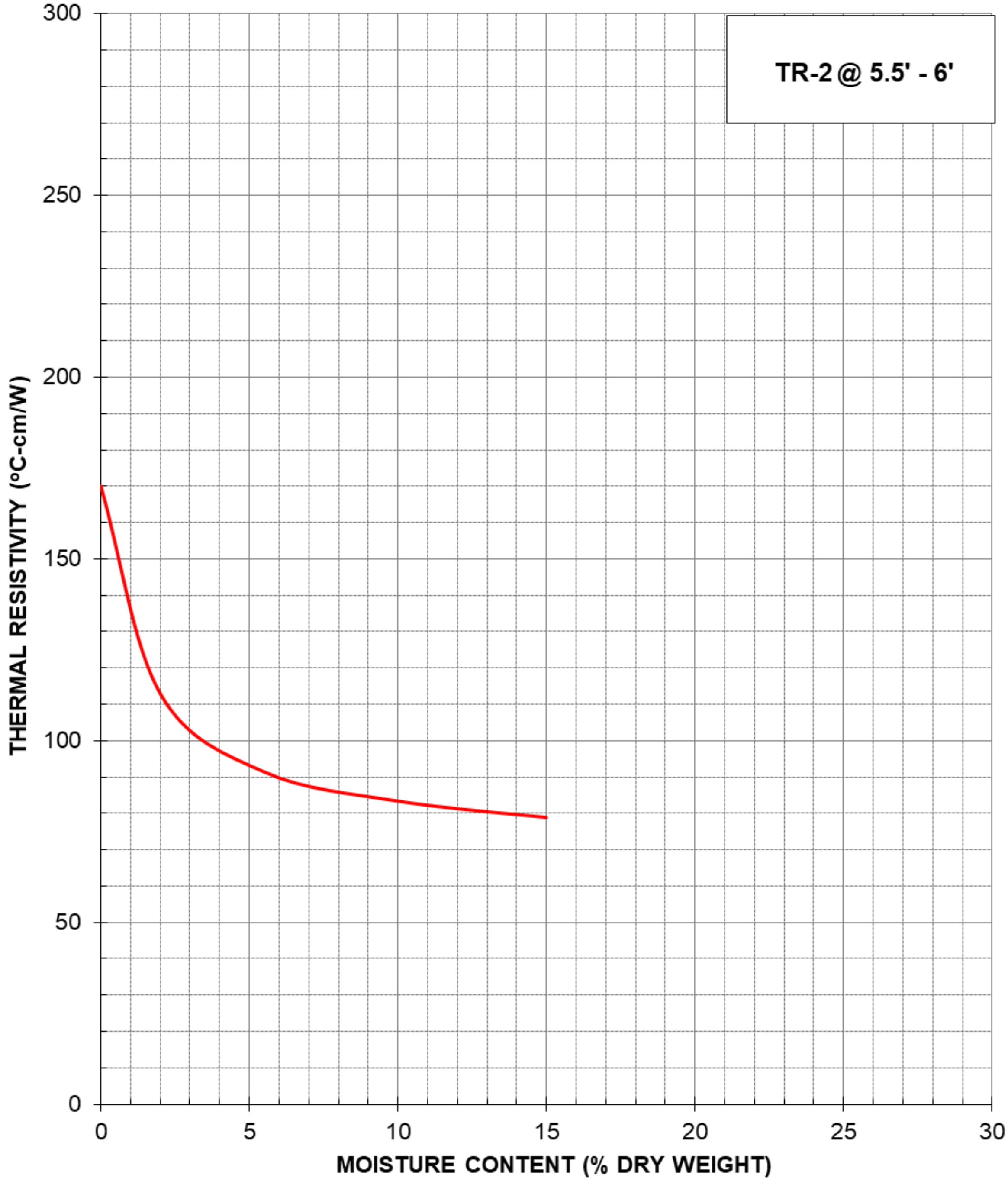
Cornerstone Earth Group (Project No.234-49-1)

Eden Landing Due Diligence – Hayward, CA

Thermal Analysis of Native Soil Samples

July 2022 **Figure 1**

THERMAL DRYOUT CURVE

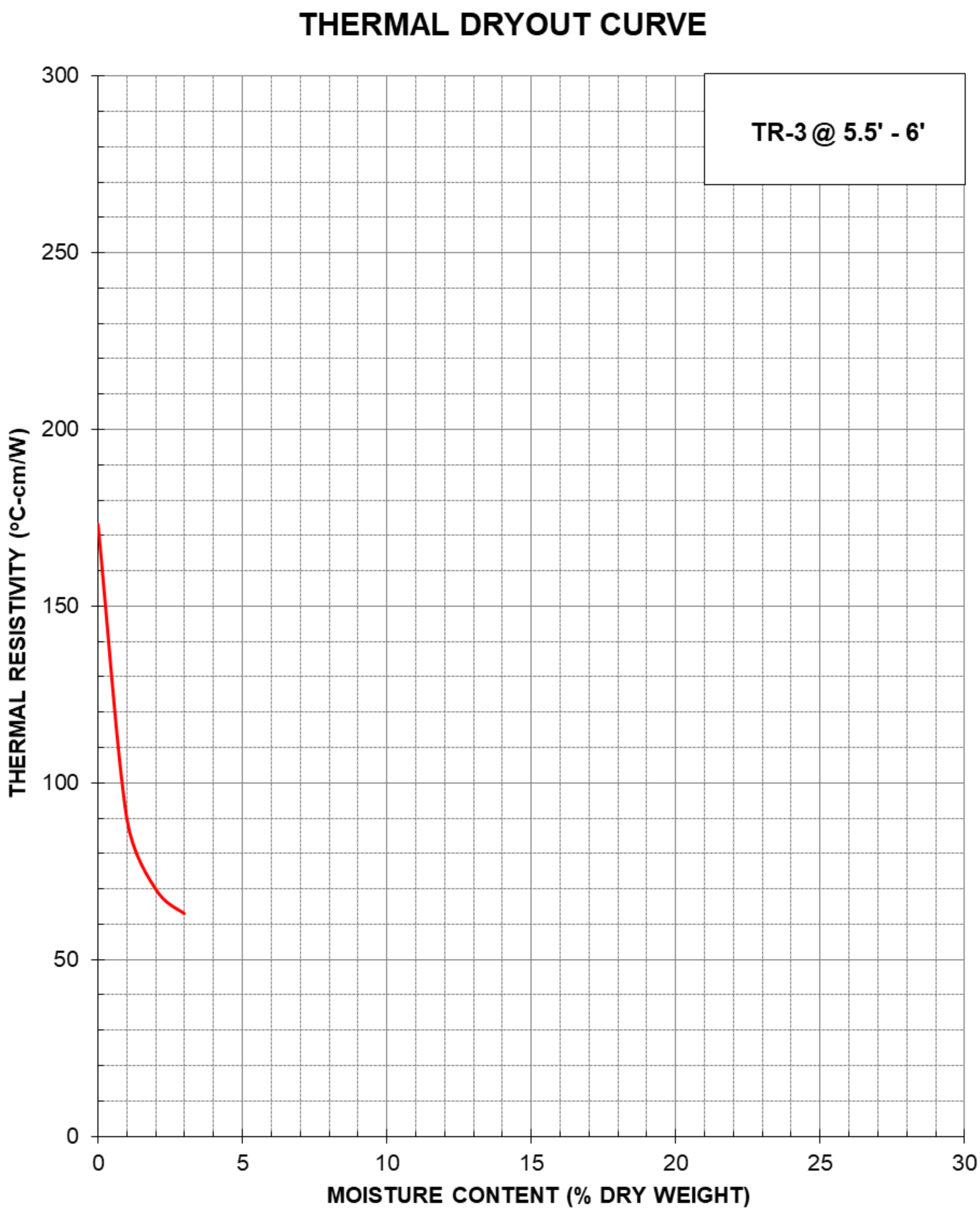


Cornerstone Earth Group (Project No.234-49-1)

Eden Landing Due Diligence – Hayward, CA

Thermal Analysis of Native Soil Samples

July 2022 **Figure 2**

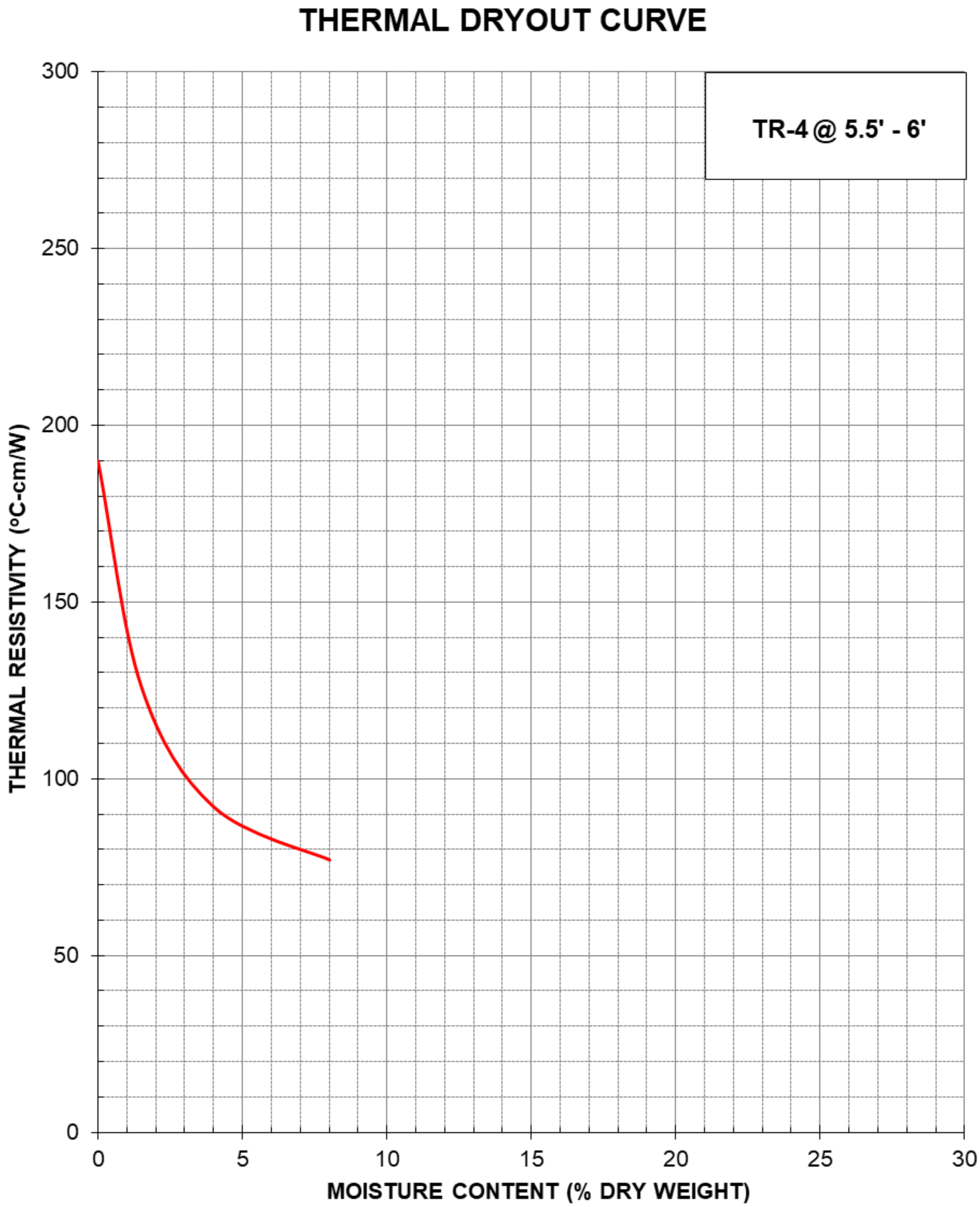


Cornerstone Earth Group (Project No.234-49-1)

Eden Landing Due Diligence – Hayward, CA

Thermal Analysis of Native Soil Samples

July 2022 **Figure 3**



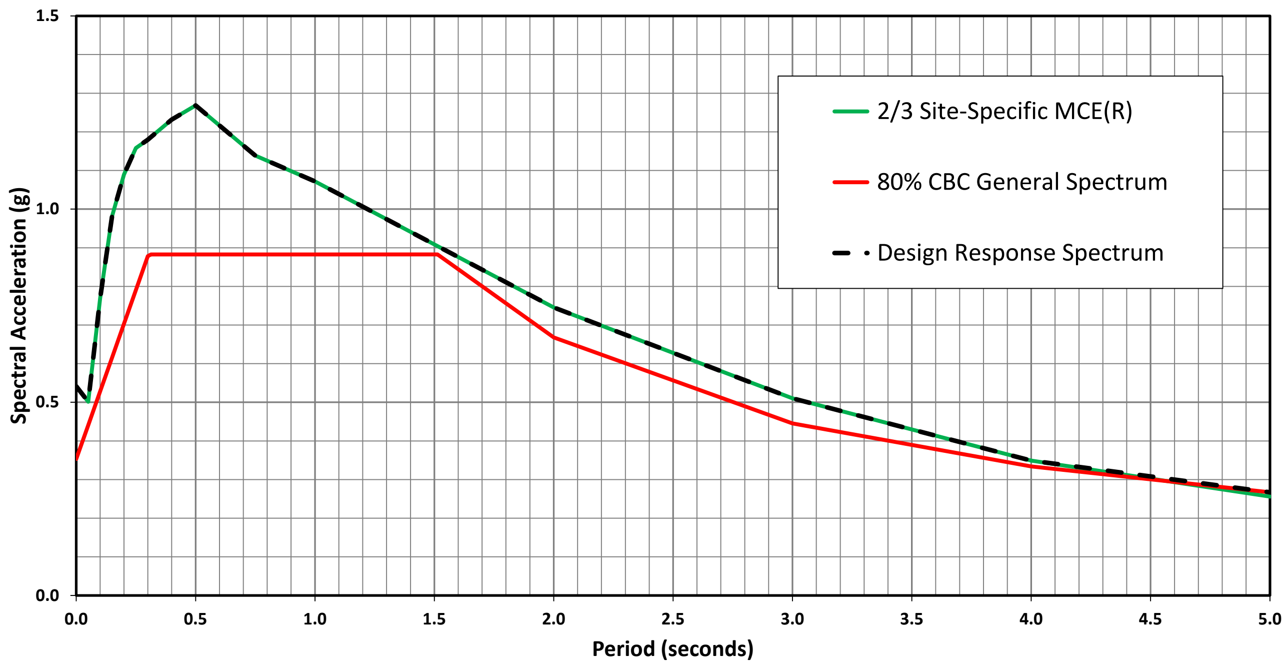
Cornerstone Earth Group (Project No.234-49-1)

Eden Landing Due Diligence – Hayward, CA

Thermal Analysis of Native Soil Samples

July 2022 **Figure 4**

APPENDIX D: GROUND MOTION HAZARD ANALYSIS



The Site-Specific Design Response Spectrum per Section 21.2, 21.3 and 21.4 of ASCE 7-16 is defined as the greater of the following at all periods:

- 2/3 of the Site-Specific MCE_R, or
- 80% of the CBC General Spectrum.

Design Response Spectra	
Period (Seconds)	Spectral Acceleration (g)
0.00	0.541
0.05	0.501
0.10	0.769
0.15	0.982
0.20	1.089
0.25	1.158
0.30	1.180
0.31	1.186
0.40	1.232
0.50	1.268
0.75	1.139
1.00	1.072
1.51	0.904
2.00	0.746
3.00	0.510
4.00	0.349
5.00	0.267

Site Design	Design Values
Site Class (Per Chapter 20 ASCE 7-16)	E
Shear Wave Velocity, V_{S30} (m/sec)	153
Site Latitude (degrees)	37.626665
Site Longitude (degrees)	-122.119579
Risk Category	II
Building Period (sec)	Unknown
Importance Factor, I_e	1
[1]Site Specific PGA_M (g)	0.74

[1] Lower of Deterministic and Probabilistic, but not less than 80% of mapped value of FM x PGA, determined in accordance with Section 21.5 of ASCE 7-16.

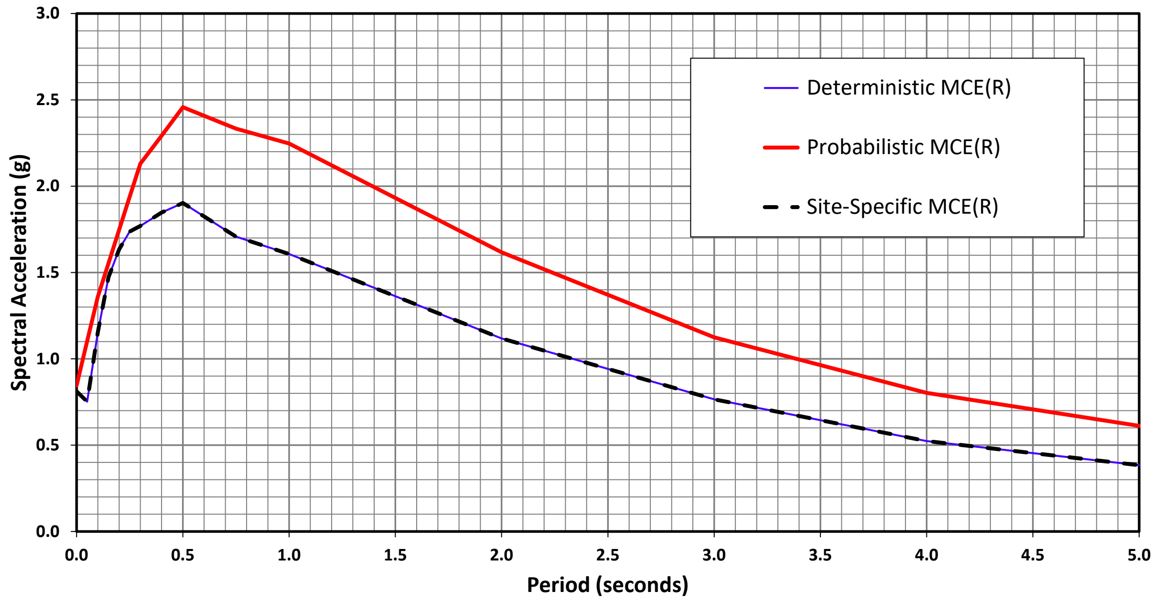
Design Acceleration Parameters[1]	
S_{DS}	1.142
S_{D1}	1.531
S_{MS}	1.712
S_{M1}	2.296

References:

ASCE/SEI 7-16: Minimum Design Loads and Associated Criteria for Buildings and Other Strutures with Supplement No. 1.
2019 California Building Code, Title 24, Part 2, Volume 2

CORNERSTONE EARTH GROUP

DESIGN RESPONSE SPECTRA	FIGURE 6	
Eden Landing Buisness Center 3475 Investment Boulevard Hayward, CA	PROJECT NO. 234-49-1	
	July 27, 2022	MFR



The Site-Specific Maximum Considered Earthquake (MCE_R) is defined as the lesser of the following at all periods:

- Deterministic MCE_R – maximum 84th percentile deterministic, or
- Probabilistic MCE_R – defined as the 2,475–year ground motion.

Site-Specific MCE_R	
Period (Seconds)	Spectral Acceleration (g)
0.00	0.812
0.05	0.752
0.10	1.153
0.15	1.472
0.20	1.634
0.25	1.737
0.30	1.769
0.31	1.780
0.40	1.848
0.50	1.903
0.75	1.708
1.00	1.607
1.51	1.357
2.00	1.118
3.00	0.765
4.00	0.524
5.00	0.384

References:

ASCE/SEI 7-16: Minimum Design Loads and Associated Criteria for Buildings and Other Strutures with Supplement No. 1.
2019 California Building Code, Title 24, Part 2, Volume 2