PRELIMINARY STORMWATER CONTROL PLAN

For:
908 Ocean Street
Santa Cruz, CA 95060

February 28, 2023

Prepared for:

908 Ocean Street
Santa Cruz, CA 95060

Prepared by:

Scott Schork
R.C.E. No. 47813
BKF ENGINEERS
1730 N. First Street, Suite 600
San Jose, CA 95112
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Table of Contents

I. Project Data

Project Name	908 Ocean Street
Project Location	908 Ocean Street, Santa Cruz, CA 95060
Project Phase No.	N/A
Project Type and Description	Demolition of existing various use buildings and constructing 3 mix-used buildings.

II. Project Site Assessment Summary

II.A. Project Location and Description

Project Name:	908 Ocean Street, Santa Cruz, CA. 95060 APN: 008-331-03 and 04, 05, 06, 07, 08, 12, 13, 14, 15, 25, 26, 27, 28, 29, 30, 31, 32, 35, 41
Facility Activities:	Demolish Existing Residential and Commercial Buildings Demolish Curb, Gutter, Sidewalk, AC Pavement Abandon On-Site/Off-Site Utilities Construct 3 mix-used buildings consisting of 350 units.

II.B. Geology and Soil Types

Geotechnical investigation performed by Cornerstone Earth Group, dated November 22, 2019 is provided for soil and geological characteristic for the project site. See Appendix A, under section 3.2 for existing subsurface conditions.

II.C. Hydrologic Considerations

The project site currently drains via sheet flow and curb and gutter to catch basins located northeast corner of Hubbard Street and May Avenue intersection and northeast corner of May Avenue and Water Street intersection. The stormwater that is collected at these two catch basins will then be conveyed through city’s main storm drain system to be discharged to concrete lined Branciforte Creek.
The design groundwater depth is determined to be at 5’ below the existing ground. See Appendix C for “Supplemental Geotechnical Recommendations”, prepared by Cornerstone Earth Group, dated April 26, 2021.

III. Stormwater Performance Criteria and Drainage Management

III.A. Development Area and BMP Requirement Tier

Total Project Site Area	175,048 SF
Pre-Project Impervious Area	144,526 SF
Post-Project Impervious Area	152,527 SF
Replaced Impervious Area	144,526 SF
New Impervious Area	8,001 SF
Total New and Replaced Impervious Area	152,527 SF
Net Impervious Area	152,527 SF

III.B. Drainage Management Areas

DRAINAGE AREA	TOTAL AREA (SF)	IMPERVIOUS SURFACE (SF)	PERVIOUS SURFACE (SF)	STORM WATER CONTROL MEASURE	SCM AREA REQUIRED (4% OF NEW IMPERVIOUS SURFACE)	SCM AREA PROVIDED	SCM
DMA-1	5198	3881	1317	BIORETENTION BASIN	155	1317	1
DMA-2	1973	1292	681	BIORETENTION BASIN	52	681	2
DMA-3	2940	2470	470	BIORETENTION BASIN	99	470	3
DMA-4	3753	3282	471	BIORETENTION BASIN	131	471	4
DMA-5	2163	1695	468	BIORETENTION BASIN	68	468	5
DMA-6	330	231	99	BIORETENTION BASIN	9	99	6
DMA-7	6993	6459	534	BIORETENTION BASIN	258	382	7
DMA-8	8565	7225	1340	BIORETENTION BASIN	289	893	8
DMA-9	7444	6891	553	BIORETENTION BASIN	276	303	9
DMA-10	3353	2893	460	BIORETENTION BASIN	116	460	10
DMA-11	1297	1106	191	BIORETENTION BASIN	44	191	11
DMA-12	2203	1858	345	BIORETENTION BASIN	74	345	12
DMA-13	2205	1860	345	BIORETENTION BASIN	74	345	13
DMA-14	1534	1344	190	BIORETENTION BASIN	54	190	14
DMA-15	2654	2434	220	BIORETENTION BASIN	97	220	15
DMA-16	5339	4698	641	BIORETENTION BASIN	188	219	16
DMA-17	20754	19302	1452	BIORETENTION BASIN	772	1217	17
DMA-18	2310	2063	247	BIORETENTION BASIN	83	247	18
DMA-19	39914	34299	5615	BIORETENTION BASIN	1372	4617	19
DMA-20	3641	3264	377	BIORETENTION BASIN	131	133	20
DMA-21	3359	2989	370	BIORETENTION BASIN	120	121	21
DMA-22	41226	36028	5198	BIORETENTION BASIN	1441	1510	22
DMA-23	3425	2975	450	BIORETENTION BASIN	119	120	23
DMA-24	2475	1988	487	BIORETENTION BASIN	80	487	24
TOTAL	175048	152527	22521		TOTAL	15506	

IV. Site Design and SCMs

IV.A. Runoff Retention Infeasibility

Due to high groundwater (5’ below existing grade), the project is not able to provide 3’ clearance from the bottom of the retention facility to the ground water elevation. Therefore, it is infeasible for the project to implement the Tier 3 runoff retention requirement. Instead, the project proposes to dedicate 10% of effective impervious area as retention-based treatment area. See TM8.1 of the plan for demonstration of providing retention-based treatment.

IV.B. Summary of Site Design and Runoff Reduction Measure

The project proposes three (3) mixed use buildings. Since the project exceeds 22,500 threshold of replaced and new impervious surface, it is considered to fulfill Tier 4 of post-construction BMP requirements. However, this project is exempted from the Tier 4 requirement because the ultimate stormwater discharge is routed to “highly altered channel” (concrete lined Branciforte Creek), as it is described in the Santa Cruz’s Chapter 6B of the Best Management Practices Manual for the City’s Storm Water Management Program. Therefore, the project is only proposing Tier 1 through Tier 3.

For the Stormwater Control Measure, the project proposes bioretention basins to treat the runoff quality.

IV.C. SCM Sizing Calculation

- DMA 1
  - Tier 2 Surface Area Required = 3,881 SF X 0.04 (4%) = 155 SF

  - Provided Area = 1,317 SF
- DMA 2
  - Tier 2 Surface Area Required = 1,292 SF X 0.04 (4%) = 52 SF
  - Provided Area = 681 SF
- DMA 3
  - Tier 2 Surface Area Required = 2,470 SF X 0.04 (4%) = 99 SF
  - Provided Area = 470 SF
- DMA 4
  - Tier 2 Surface Area Required = 3,382 SF X 0.04 (4%) = 131 SF
  - Provided Area = 471 SF
- DMA 5
  - Tier 2 Surface Area Required = 1,695 SF X 0.04 (4%) = 68 SF
  - Provided Area = 468 SF
- DMA 6
  - Tier 2 Surface Area Required = 231 SF X 0.04 (4%) = 9 SF
  - Provided Area = 99 SF
- DMA 7
  - Tier 2 Surface Area Required = 6,459 SF X 0.04 (4%) = 258 SF
  - Provided Area = 382 SF
- DMA 8
  - Tier 2 Surface Area Required = 7,225 SF X 0.04 (4%) = 289 SF
  - Provided Area = 893 SF
- DMA 9
  - Tier 2 Surface Area Required = 6,891 SF X 0.04 (4%) = 276 SF
  - Provided Area = 303 SF
- DMA 10
  - Tier 2 Surface Area Required = 2,893 SF X 0.04 (4%) = 116 SF
  - Provided Area = 460 SF
- DMA 11
  - Tier 2 Surface Area Required = 1,106 SF X 0.04 (4%) = 44 SF
  - Provided Area = 191 SF
- DMA 12
  - Tier 2 Surface Area Required = 1,858 SF X 0.04 (4%) = 74 SF
  - Provided Area = 345 SF
- DMA 13
  - Tier 2 Surface Area Required = 1,860 SF X 0.04 (4%) = 74 SF
  - Provided Area = 345 SF
- DMA 14
  - Tier 2 Surface Area Required = 1,344 SF X 0.04 (4%) = 54 SF
  - Provided Area = 190 SF
- DMA 15
  - Tier 2 Surface Area Required = 2,434 SF X 0.04 (4%) = 97 SF
  - Provided Area = 220 SF
- DMA 16
  - Tier 2 Surface Area Required = 4,698 SF X 0.04 (4%) = 188 SF
  - Provided Area = 219 SF
- DMA 17
  - Tier 2 Surface Area Required = 19,302 SF X 0.04 (4%) = 772 SF
  - Provided Area = 1,217 SF
- DMA 18

  - Tier 2 Surface Area Required = 2,063 SF X 0.04 (4%) = 83 SF
  - Provided Area = 247 SF
- DMA 19
  - Tier 2 Surface Area Required = 34,299 SF X 0.04 (4%) = 1,372 SF
  - Provided Area = 4,617 SF
- DMA 20
  - Tier 2 Surface Area Required = 3,264 SF X 0.04 (4%) = 131 SF
  - Provided Area = 133 SF
- DMA 21
  - Tier 2 Surface Area Required = 2,989 SF X 0.04 (4%) = 120 SF
  - Provided Area = 121 SF
- DMA 22
  - Tier 2 Surface Area Required = 36,028 SF X 0.04 (4%) = 1,441 SF
  - Provided Area = 1,510 SF
- DMA 23
  - Tier 2 Surface Area Required = 2,975 SF X 0.04 (4%) = 119 SF
  - Provided Area = 120 SF
- DMA 24
  - Tier 2 Surface Area Required = 1,988 SF X 0.04 (4%) = 80 SF
  - Provided Area = 487 SF

V. Storm Water Facility Maintenance

V.A. Ownership and Responsibility for Maintenance in Perpetuity

The Principal maintenance objective is to prevent sediment buildup and clogging, which reduces pollutant removal efficiency and may lead to bioretention area failure. Routine maintenance activities, and the frequency at which they will be conducted, are shown below.

A. Maintenance Objectives

A comprehensive monitoring and maintenance program is an essential element of a long-term stormwater management plan. The proposed stormwater system for the subject project will operate in an automatic and reliable manner. However, as with all physical infrastructure, the stormwater system will need adequate routine maintenance to function as designed.

- To monitor all BMPs to assess whether they continue to function as appropriate mitigation for the effects of urban non-point source pollution on receiving waters in a manner consistent with the highest regard for public safety;
- To set forth the expected routine maintenance functions and associated schedules that allow the BMPs to function as designed;
- To anticipate non-routine maintenance needs that may arise and suggest appropriate responses to these needs;
- The operations and maintenance plan will be a “living document” that can be modified in the future to save costs (without compromising the goals of the program) and to adjust to changes at the site or in regulatory guidance.

B. Scheduling of Monitoring and Maintenance

Routine maintenance for the BMPs should be carried out on a schedule similar to the rest of the stormwater system. This will typically require a thorough inspection and maintenance visit in late summer or early fall prior to the rainy season. Observations and recommendations for corrective measure (if necessary) will be recorded and kept by Ocean Place. Remedial maintenance will be performed immediately or scheduled to take place within a reasonable time frame. Records will be available to the City of Santa Cruz for review upon request.

The following general monitoring and maintenance guidelines shall be performed:

- A thorough inspection and maintenance of all the BMP's mentioned above shall be conducted in late summer or early fall prior to the rainy season (October 1st).
- All BMP's mentioned above shall be monitored following major storm events (greater that 1-inch of rain).
- Any debris and/or sediment encountered anywhere on the project site shall be removed as necessary.
- Remedial maintenance shall be performed immediately as conditions allow.
- See Appendix D for a Sample BMP Inspection/Maintenance Form and for Bio-Retention Area Maintenance Plan and Operation and Maintenance Inspection Report.
- If mosquito larvae are present and persistent, contact the County for information and advice. Mosquito larvicide should be applied only when absolutely necessary and then only by a licensed individual or contractor.
- Representatives of the City, the local vector control district and the Regional Water Quality Control Board may enter the common areas for purposes of verifying proper operation and maintenance of the BMP's outlined in the approved plan.
- It is the responsibility of Ocean Place to ensure that all monitoring and maintenance of treatment control measures is performed on time and as scheduled.

A summary of the inspection and maintenance schedule for source control and treatment control BMP's is shown in Table 1.

Table 1: Inspection and Maintenance Schedule Summary

Areas*	Inspection	Schedule
Landscaping (Includes Bio-retention areas)	Inspect for erosion, damage to vegetation, channelization of flow and sediment accumulation	Twice a year: before and after the rainy season (before October 1st and after April 1st)
	Mow grass to maintain an acceptable height. Irrigate areas during dry seasons. Aerate soil by cultivating and adding mulch.	As needed (frequent seasonally)
Storm Drainage Collection System	Inspect area drains, catch basins, drop inlets, and manholes	Twice a year: before and after the rainy season (before October 1st and after April 1st)
	Clean area drains, catch basins, drop inlets, and manholes	Twice a year: before and after the rainy season (before October 1st and after April 1st). After every major storm event

Stormwater Treatment/ Retention/ Detention System (Bio-retention areas)	Inspect overflow drains	Twice a year: before and after the rainy season (before October 1st and after April 1st). After every major storm event
	Repair any damaged areas within the bio-retention areas. Remove sediment from the bio-retention areas if vegetation growth is inhibited or if the sediment is blocking the even spreading of water.	Twice a year: before and after the rainy season (before October 1st and after April 1st) Ensure paving area is clean of debris, dewaters between storms and is clean of sediment (monthly).

V.B. Summary of Maintenance Requirements for Stormwater Facility

The maintenance for all source and treatment control BMP's is as described below. See Table 1 for a summary of the inspection and maintenance schedule. Records of observations and recommendations shall be kept by the Ocean Place and made available to the City of Santa Cruz upon request.

1. Landscape Maintenance

The following landscape maintenance shall be performed on all landscape areas including all bio-retention areas:

- *Landscape areas (including bio-retention areas) within the project site shall be covered with plants or some type of ground cover to minimize erosion. No areas are to be left as bare dirt that could erode.*
- *Pesticides and fertilizers shall be stored as hazardous materials and in appropriate packaging. Over spraying onto paved areas shall be avoided when applying fertilizers and pesticides. Pesticides and fertilizers will be prohibited from being stored outside.*
- *Landscape areas (including bio-retention areas) shall be inspected for debris and obstructions to drainage flow. All debris and obstructions to drainage flow shall be removed.*

2. Storm Drainage Collection System Maintenance

The storm drainage collection system consists of overflow drains, area drains, catch basins, drop inlets, distribution piping, and manholes. The following maintenance shall be performed on all storm drainage collection systems:

- *Inlet and Catch Basin Cleaning.* Inspect all overflow drains, area drains, catch basins drop inlets and manholes twice a year for debris and sediment before and after the rainy season (before October 1st and after April 1st). During inspection, all debris and sediment shall be removed.
- *Regular Street Sweeping.* Regular street sweeping can have a significant impact on the control of such constituents of concern as trash and debris, particulates, and heavy metals. All streets should be swept on a regular basis to control the

build-up of sediment and trash with particular attention to the early fall period prior to the onset of the winter rainy season. Street Sweeping schedules will follow City of Santa Cruz standards, but should not be less than monthly.

3. Stormwater Treatment/Retention/Detention System Maintenance

The stormwater treatment system consists of bio-retention areas. To ensure that the stormwater treatment system is properly functional and operational, the following routine maintenance, but not limited to, shall be performed:

- *Overflow drains within the bio-retention areas shall be inspected twice a year before and after the rainy season for debris and sediment (before October 1st and after April 1st). Any debris or accumulations of sediment encountered shall be removed.*
- *After every major* storm event (greater that 1-inch of rain) all overflow drains, storm drain clean out boxes and manholes shall be inspected to remove any obstructions to the flow.
- *If eroded areas are observed in the bio-retention areas, repair the area by placing a seeded blanket on eroded area as soon as scour is observed.*
- *Herbicides, pesticides or non-organic fertilizers should not be used in the bio-retention areas. Instead, use integrated pest management techniques and hand weed these areas.*
- *When water stands in the bioretention basins between storms and does not drain within 48-72 hours after rainfall, the 24" thick treatment soil section (infiltration rate of 5 to 10 inches per hour) and planting shall be replaced per the development Improvement Plans.*
- *In addition to above, the City of Santa Cruz shall follow the Bio-Retention Area Maintenance Plan and Operation and Maintenance Inspection Report in Appendix D.*

APPENDIX A

Storm Water & LID Checklist

APPENDIX A
STORM WATER AND LOW-IMPACT DEVELOPMENT BMP REQUIREMENT WORKSHEET

How to Use This Worksheet

The City's Storm Water BMP requirements are based on project type, proposed impervious area, and location within the watershed. This worksheet was developed to help permit applicants determine and meet storm water BMP requirements applicable to a proposed development or redevelopment

1 - Download this fillable form online at **www.cityofsantacruz.com/LID**
2 - Fill out the Worksheet to determine what stormwater BMP requirements apply to a proposed project.
3 - Attach Worksheet and additional documentation required as listed in the City Storm Water Best Management Practices for Private and Public Development Projects to plans for review by the Department of Public Works
4 - Please contact the Public Works Environmental Project Analyst at 420-5160 if you have any questions on completing the worksheet.

Project Address: 908 Ocean St, Santa Cruz **Bldg Permit #:** TBD

A - Project Type

Check project type that applies:

☐ Single Family Home ☒ Multi-family, Commercial, Industrial, Public facilities

Check development type that applies:

☐ New Development ☒ Redevelopment / Remodel

B - Proposed Development Area and Impervious Area:

Pre-project impervious surface area: 144,526 sq ft
Post-project impervious surface area: 152,527 sq ft
Amount of impervious surface area that will be **replaced**: 144,526 sq ft
Amount of new impervious surface area that will be **created**: 8,001 sq ft
Reduced Impervious Area Credit: 0 sq ft

New and Replaced Impervious Area = **0** **sq ft**

Net Impervious Area = **0** **sq ft**

(Net Impervious Area = Impervious Area created + Impervious Area replaced - Reduced Impervious Area Credit)

C - Post-Construction BMP Tier requirement:

Check Project Type and Impervious Area (from calculations above) that applies.

BMP requirements are cumulative (e.g. a project subject to BMP Tier 3 is also subject to Tiers 1 and 2), permit review fees are not cumulative.

Projects requiring a Stormwater Control Plan will need to involve a civil engineer.

SINGLE-FAMILY HOMES	BMP TIER	Permit Review Fee	Stormwater Control Plan required?
☐ Single-family Home with Net Impervious Area **< 15,000 sf**, **please consult Chapter 6A, BMPs for Single-Family Homes on Small Lots**	**N/A**	$0	No
☐ Net Impervious Area **≥ 15,000 sf**; New and replaced impervious area **< 22,500 sf**	**3**	$330	Yes
☐ New and replaced impervious area ≥ **22,500 sf**	**4**	$550	Yes

MULTI-FAMILY, COMMERCIAL, INDUSTRIAL, PUBLIC FACILITIES	BMP TIER	Permit Review Fee	Stormwater Control Plan Required?
☐ New and Replaced Impervious Area **≥ 2,500 sf**; Net Impervious Area **< 5,000 sf**	**1**	$0	No
☐ Net Impervious Area **≥ 5,000 sf**; New and Replaced Impervious Area **< 15,000 sf**	**2**	$330	Yes
☐ New and Replaced Impervious Area **≥ 15,000 sf** but **< 22,500 sf**	**3**	$550	Yes
☒ New and replaced impervious area ≥ **22,500 sf**	**4**	$550	Yes

If the proposed project is only subject to BMP Tiers 1 or 2, skip to Step F.

D - Watershed Management Zones - For projects subject to Tiers 3 Post-Construction BMP requirements only.
Watershed Management Zones are viewable online on the City of Santa Cruz GIS website at: http://gis.cityofsantacruz.com/gis/index.html

Watershed Management Zones and associated Tier 3 (Runoff Retention) Post-Construction BMP requirements
If Tier 3 BMP requirements are applicable to the project, check the watershed management zone area where the project is located.

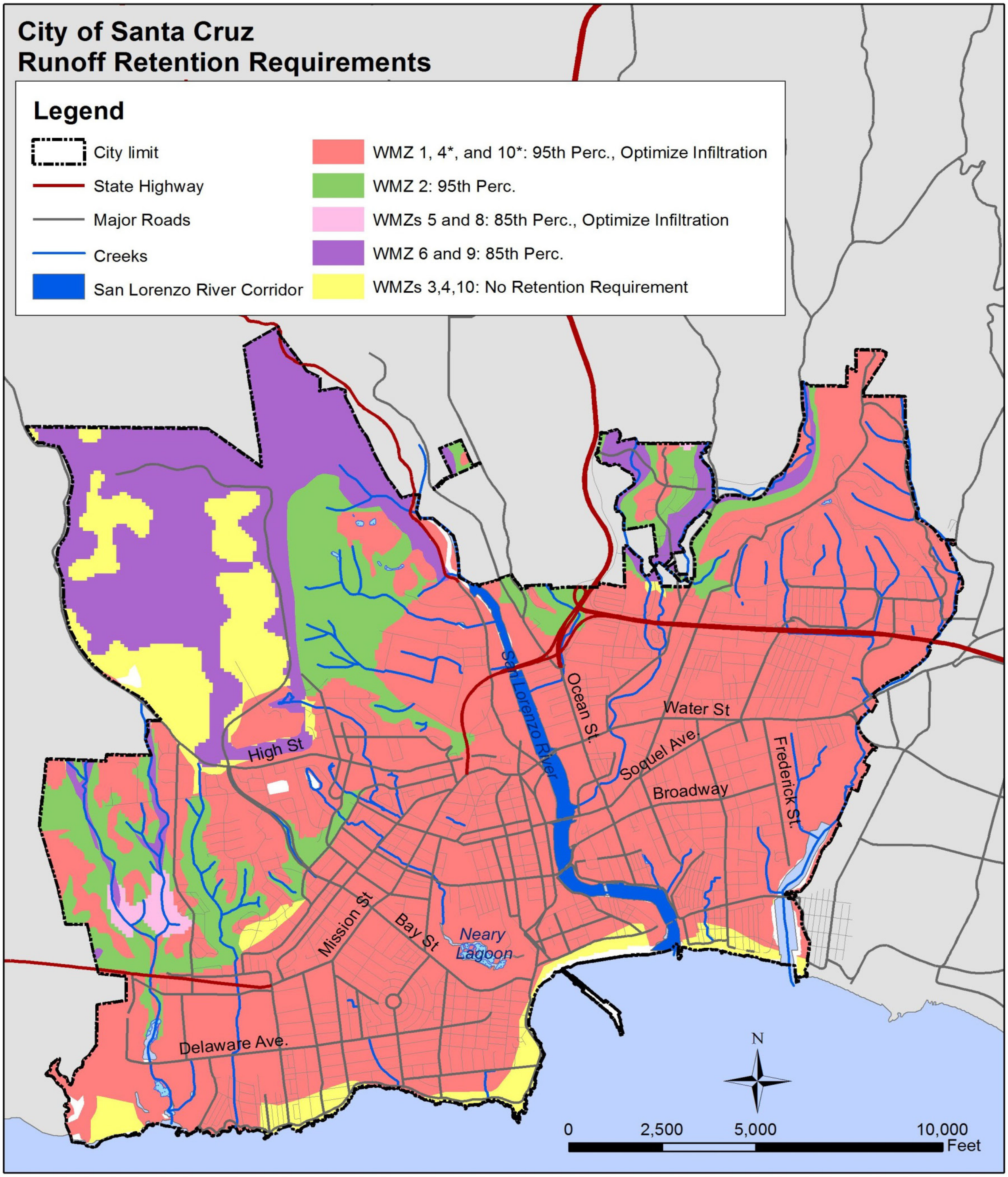
☒ WMZ 1, and portions of 4, and 10 overlying groundwater basin

☐ WMZ 2

☐ WMZ 5 and 8

☐ WMZ 6 and 9

☐ WMZ 3, 4 and 10



E - Special Circumstances - For projects subject to Tiers 3 and 4 Post-Construction BMP requirements only.
Check if special circumstance applies to the project

☒ Highly Altered Channel and Intermediate Flow Control Facility

☐ Urban Sustainability Area

F - Additional Stormwater BMP Requirements for Multi-family, Commercial and Industrial projects
Check if additional BMP requirements apply to the project

a) State Construction Activities Storm Water General Permit

☒ Construction activity resulting in land disturbance of one acre or more, or part of a larger common plan of development

b) Additional Source Control BMP requirements for specific facilities

☐ Commercial or industrial facility
☐ Material Storage Areas
☐ Vehicle fueling, maintenance and wash areas
☐ Equipment and accessory wash areas
☒ Interior and parking garage floor drains
☒ Parking areas
☒ Pools, spas and other water features
☒ Trash Storage Areas
☒ Restaurants and food processing or manufacturing facilities
☐ Miscellaneous drain or wash water

G - Complete if your project is <u>only</u> subject to Tier 1 Requirements - Site planning and LID design measures.
LID design measures shall be clearly marked on site plans

Check applicable boxes and provide short description of measure and location

☐ Conserve natural areas, riparian areas and wetlands

Description: ______________________________

☐ Concentrate improvements on the least-sensitive portions of the site and minimize grading

Description: ______________________________

☐ Direct roof runoff into cisterns or rain barrels

Description: ______________________________

☐ Direct roof downspouts to landscaped areas or rain gardens

Description: ______________________________

☐ Use pervious pavement (pervious concrete or asphalt, turf block, crushed aggregate, etc.)

Description: ______________________________

☐ Disperse runoff from paved areas to adjacent pervious areas

Description: ______________________________

Appendix B

Plans

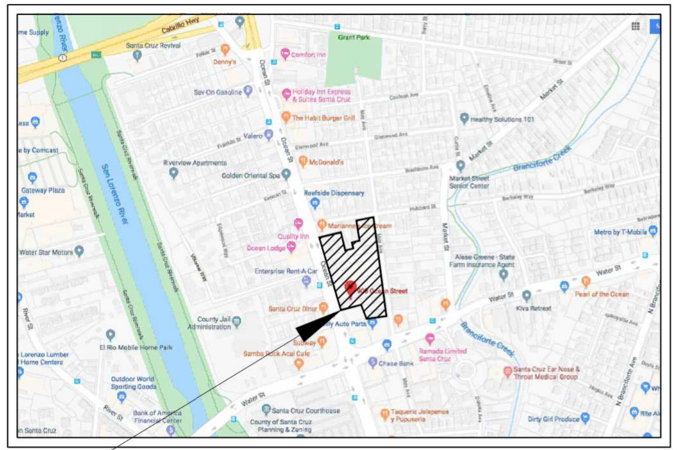
VESTING TENTATIVE PARCEL MAP
FOR CONDOMINIUM PURPOSES
908 OCEAN STREET MIXED USE
SANTA CRUZ, SANTA CRUZ COUNTY, CALIFORNIA

GENERAL NOTES:

1. APN:
   008–331–03, 008–331–04, 008–331–05, 008–331–06, 008–331–07, 008–331–08, 008–331–12, 008–331–13, 008–331–14, 008–331–15, 008–331–25, 008–331–26, 008–331–27, 008–331–28, 008–331–29, 008–331–30, 008–331–31, 008–331–32, 008–331–35 & 008–331–41
2. EXISTING ZONING: CC (COMMUNITY COMMERCIAL)
3. PROPOSED ZONING: CC (COMMUNITY COMMERCIAL)
4. EXISTING GENERAL LAND USE: AUTO–RELATED SERVICES, MULTI–FAMILY HOUSING, SINGLE FAMILY HOUSING, RETAIL AND PERSONAL SERVICES, OFFICES, VACANT BUILDING, MIXED–USE
5. PROPOSED GENERAL LAND USE: MIXED USE MEDIUM DENSITY (MXMD)
6. NUMBER OF UNITS: 354 UNITS
7. SITE AREA: 3.84± ACRES
8. UTILITIES:
   WATER CITY OF SANTA CRUZ
   SANITARY SEWER CITY OF SANTA CRUZ
   STORM DRAIN CITY OF SANTA CRUZ
   ELECTRIC/GAS PG&E
   COMMUNICATION AT&T
   CABLE COMCAST
9. FIRE PROTECTION: SANTA CRUZ FIRE DEPARTMENT
10. UNDERLYING AERIAL MAPPING BY AERIAL 360, INC. DATE OF PHOTOGRAPHY IS AUGUST 3, 2018. CONTOUR INTERVAL IS ONE FOOT.
11. BENCHMARK: SANTA CRUZ CITY BENCHMARK No. C4–08A, A LEAD PLUG, NAIL AND TAG SET IN THE TOP OF THE CURB AT THE SOUTHEAST CORNER OF HUBBARD STREET AND OCEAN STREET. ELEVATION = 26.28 FEET (NGVD 29).
12. BASIS OF BEARINGS: N13°04'34"E ALONG TIE LINE FROM 1/2" IRON PIPE (NO TAG) FOUND ALONG THE EASTERLY SIDELINE OF OCEAN STREET, AT THE SOUTHWEST CORNER OF LOT 3 (TRUNCATED PER STREET WIDENING) AND THE CALCULATED POSITION OF THE NORTHEAST CORNER OF LOT 24 (PER TIES SHOWN ON CORNER RECORD No. 47) AS SAID LOTS ARE SHOWN ON THE RECORD OF SURVEY RECORDED IN VOLUME 45 OF MAPS, AT PAGE 30, SANTA CRUZ COUNTY RECORDS (SAID LOTS ARE ORIGINALLY SHOWN ON THE MAP ENTITLED, "MAP OF BUILDING LOTS ON OCEAN STREET" RECORDED IN VOLUME 10 OF MAPS, AT PAGE 1, SANTA CRUZ COUNTY RECORDS), AND AS SHOWN HEREON.
13. FEMA FLOOD ZONE: THIS PROPERTY IS LOCATED WITHIN ZONE X: AREA OF MINIMAL FLOOD HAZARD. FLOOD INSURANCE RATE MAP 06087C0332E, DATED MAY 16, 2012.

PURPOSE:

EXISTING LOTS 5–14 AND 24–37 TO BE COMBINED INTO ONE LOT (LOT 1) FOR CONDOMINIUM PURPOSES. THIS IS A MAP OF A CONDOMINIUM PROJECT AS DEFINED IN CALIFORNIA CIVIL CODE SECTION 6542 OR SUCCESSOR STATUTES AND AUTHORIZES THE ESTABLISHMENT OF A MAXIMUM OF THREE CONDOMINIUMS.



PROJECT LOCATION

VICINITY MAP

NTS

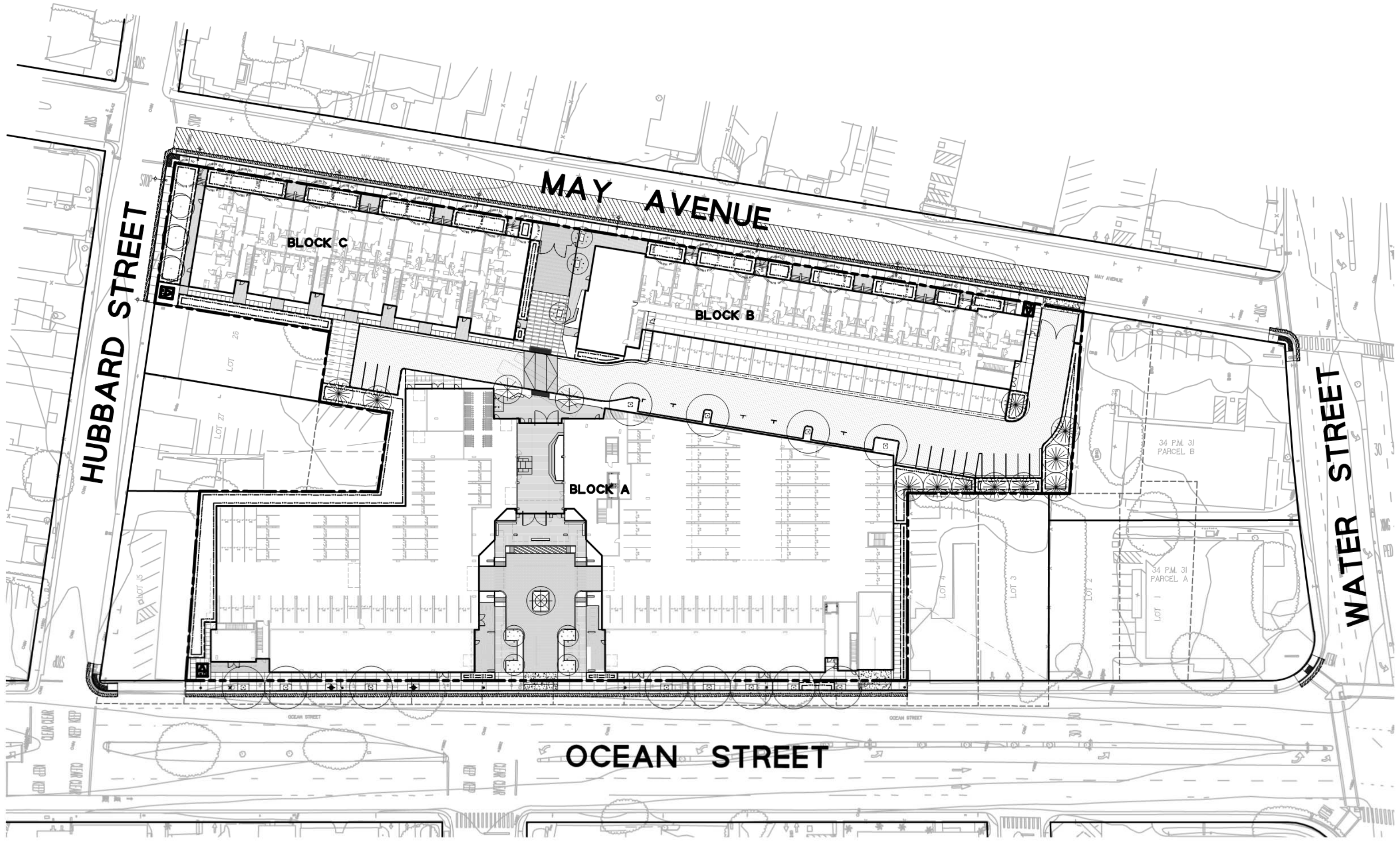
CONSULTANTS:

ARCHITECT:
BDE ARCHITECTURE
934 HOWARD STREET
SAN FRANCISCO, CA 94103
TEL: (415) 967–6815
ATTN: NATHAN SIMPSON

CIVIL ENGINEER:
BKF ENGINEERS
1730 NORTH FIRST STREET, SUITE 600
SAN JOSE, CA 95112
TEL: (408) 467–9100
ATTN: JEREMY MARELLO

LANDSCAPE ARCHITECT:
CREO LANDSCAPE ARCHITECTURE
535 MISSION STREET, 14TH FLOOR
SAN FRANCISCO, CA 94105
TEL: (415) 688–2506
ATTN: SCOTT MULHOLLAND

SHEET INDEX



SITE MAP

1"=60'

ENGINEER'S STATEMENT

THIS VESTING TENTATIVE MAP HAS BEEN PREPARED BY ME OR UNDER MY DIRECTION IN ACCORDANCE WITH STANDARD ENGINEERING PRACTICE.

SCOTT R. SCHORK, P.E.
BKF ENGINEERS
R.C.E. NO. 47813

DATE

S. R. SCHORK
No. 47813
Exp. 12–31–23
CIVIL

TITLE SHEET **TM1.O**

908 OCEAN ST

SANTA CRUZ, CALIFORNIA

FEBRUARY 28, 2023

LEGEND

PROPERTY LINE	▬ ▬ · ▬
EXISTING LOT LINE	— — —
ADJACENT PROPERTY LINE	———
STREET CENTER LINE	— - — - —
EXISTING FENCE	—X—X—
EXISTING EASEMENT	- - - -
EXISTING ELECTRICAL LINE	—E—
EXISTING GAS LINE	—G—
EXISTING STORM DRAIN LINE	—SD—
EXISTING SANITARY SEWER LINE	—SS—
EXISTING WATER LINE	—W—
EXISTING OVERHEAD WIRE	—OH—
EXISTING SANITARY SEWER MANHOLE	○MH
EXISTING STORM DRAIN CATCH BASIN	CB▨
EXISTING FIRE HYDRANT	HYD -○-
EXISTING GUY ANCHOR	○—
EXISTING JOINT POLE	-◆-
EXISTING SIGN	-○-
SPOT ELEVATION	+ 46.21
CONTOUR LINE	—XX.XX—
EXISTING SURVEY MONUMENT	●

EXISTING CONDITIONS TM2.0

BDE ARCHITECTURE

HIGHSTREET RESIDENTIAL

creo landscape architecture

BKF

908 OCEAN ST

SANTA CRUZ, CALIFORNIA

FEBRUARY 28, 2023

SUMMARY

OVERALL PROJECT AREA (INCLUDING APN 008-331-28) = 3,844 AC.

LEGEND

ADJACENT PROPERTY LINE	━━ ━ ━ ━━
ADJACENT PROPERTY LINE	—— – – ——
STREET CENTER LINE	— - — - —
EXISTING FENCE	— X — X —
SAWCUT LINE	- - - - - - -
EXISTING EASEMENT	—— ——
EXISTING ELECTRICAL LINE	— E —
EXISTING GAS LINE	— G —
EXISTING STORM DRAIN LINE	— SD —
EXISTING SANITARY SEWER LINE	— SS —
EXISTING WATER LINE	— W —
EXISTING OVERHEAD WIRE	— OH —
EXISTING SANITARY SEWER MANHOLE	○MH
EXISTING STORM DRAIN CATCH BASIN	CB▤
EXISTING FIRE HYDRANT	HYD
EXISTING GUY ANCHOR	←
EXISTING JOINT POLE	●
EXISTING SIGN	○
SPOT ELEVATION	+46.21
CONTOUR LINE	— XX.XX —
REMOVE EXISTING UTILITY LINE	×—×—×—×—×
ABANDON EXISTING UTILITY LINE PER SANTA CRUZ WATER DEPARTMENT SPECIFICATION CHAPTER 8.	—//—//—//—
REMOVE ASPHALT AND BASEROCK	▓
REMOVE EXISTING BUILDING, FOUNDATION, AND BASEROCK	▨
REMOVE CONCRETE PAVEMENT AND BASEROCK (INCLUDES CURB, GUTTER, WALKWAYS, FOUNDATION AND ETC.)	▭
CLEAR AND GRUB EXISTING LANDSCAPE AREA	░

DEMOLITION PLAN TM3.0

BDE ARCHITECTURE
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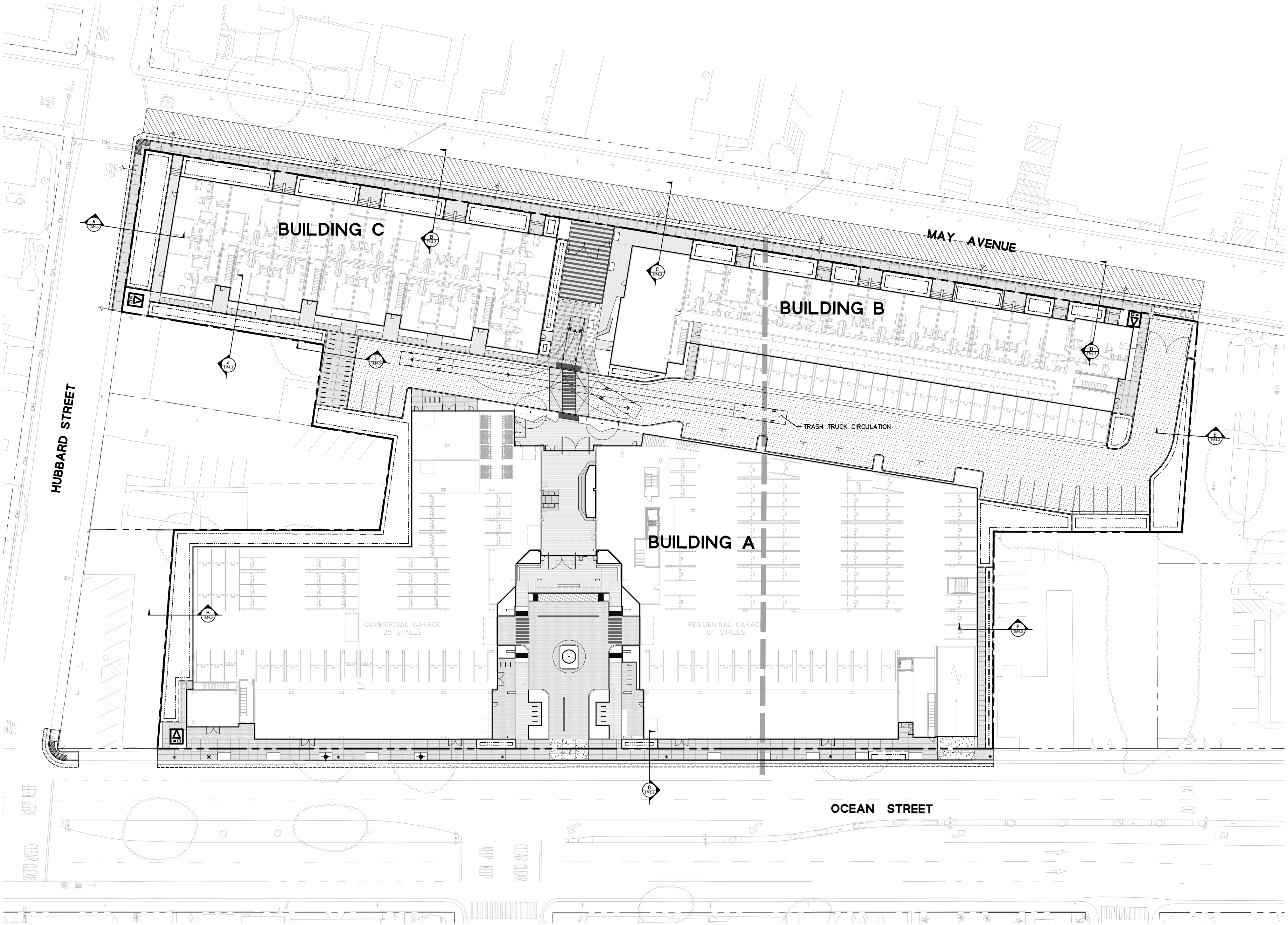
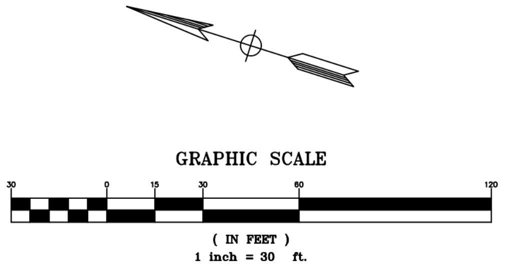
FEBRUARY 28, 2023



LEGEND

EXISTING PROPERTY LINE	▬ ▬ ▬
PROPOSED PROPERTY LINE	— - —
ADJACENT PROPERTY LINE	— - —
STREET CENTER LINE	— - —
SAWCUT LINE	- - - -
BIORETENTION AREA	▭
VERTICAL CURB & GUTTER	═
VALLEY GUTTER	═
NEW CONCRETE SIDEWALK	▒
NEW VEHICULAR CONCRETE	▒
NEW AC	▒
AC GRIND & OVERLAY	▨
MATCHLINE	▬ ▬ ▬



OVERALL SITE PLAN TM4.0

BDE ARCHITECTURE

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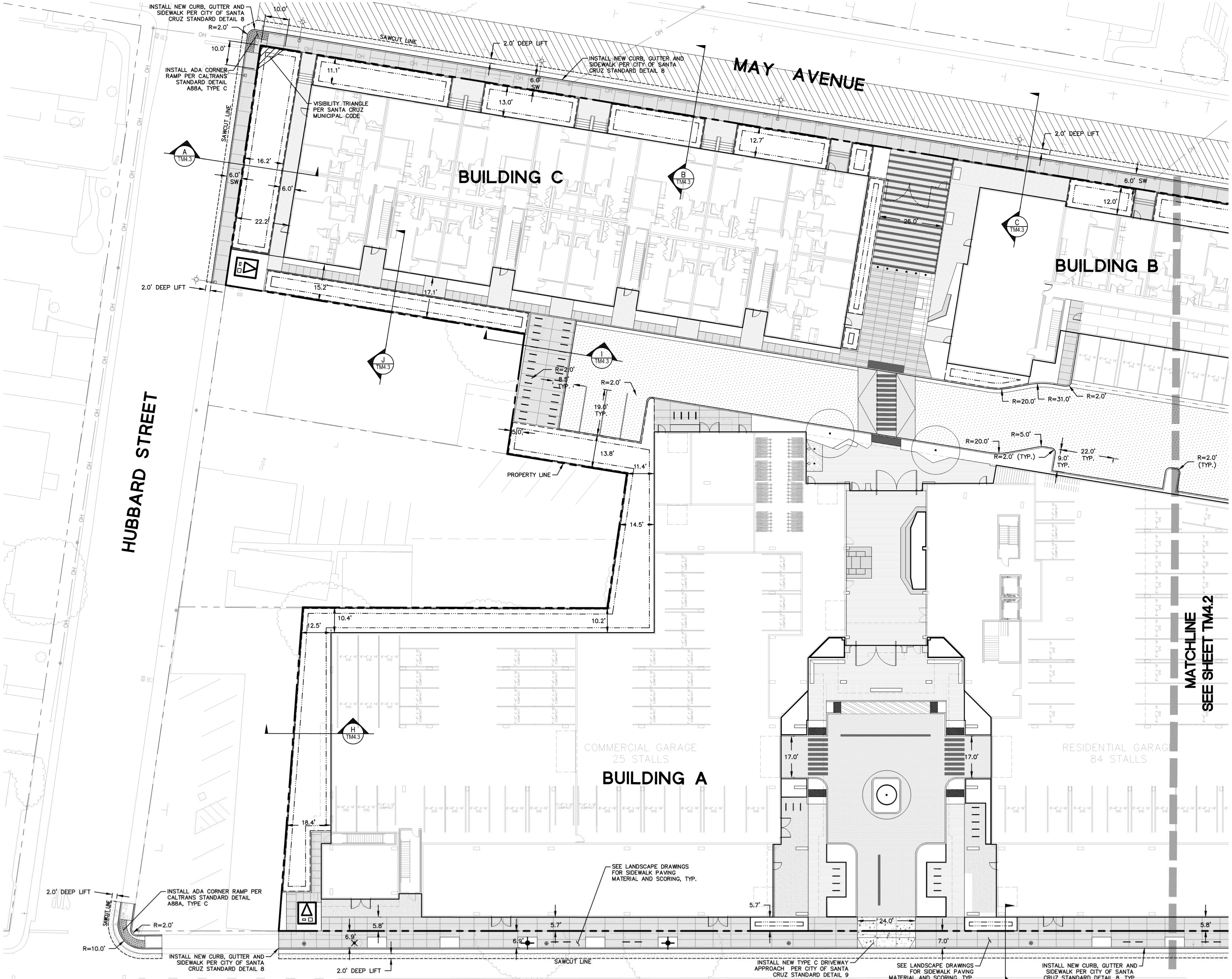

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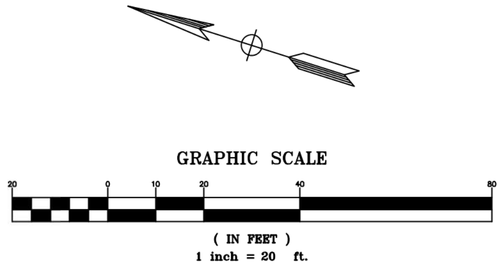
PROPOSED PROPERTY LINE	— - - — - -
ADJACENT PROPERTY LINE	— - —
STREET CENTER LINE	— - — -
SAWCUT LINE	- - - - - -
BIORETENTION AREA	⬚
VERTICAL CURB & GUTTER	═
VALLEY GUTTER	═
NEW CONCRETE SIDEWALK	▒
NEW VEHICULAR CONCRETE	░
NEW AC	▓
AC GRIND & OVERLAY	▨
MATCHLINE	▬ ▬ ▬



SITE PLAN

TM4.1

BDE ARCHITECTURE

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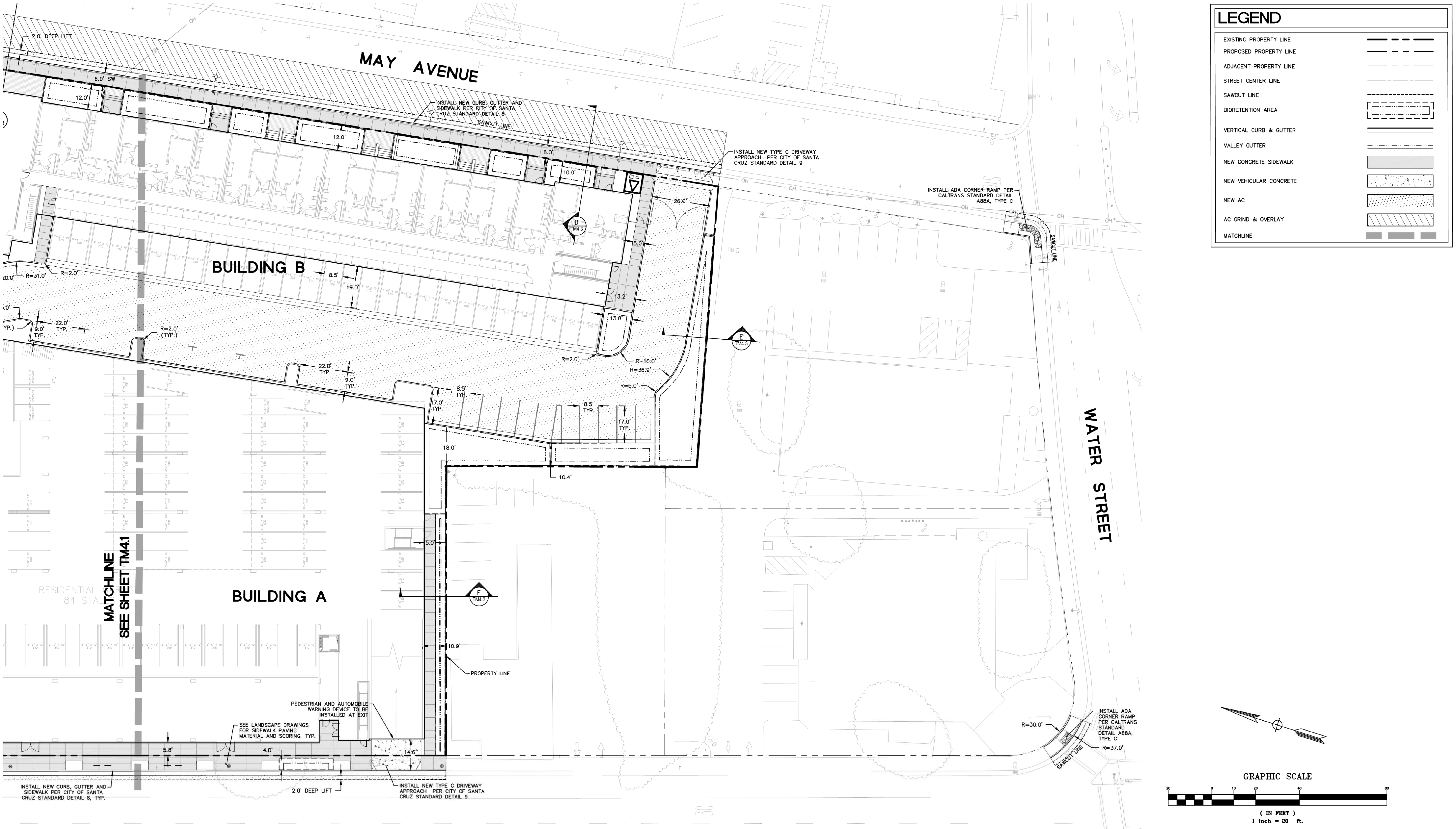

SITE PLAN

TM4.2

BDE ARCHITECTURE

HIGHSTREET RESIDENTIAL

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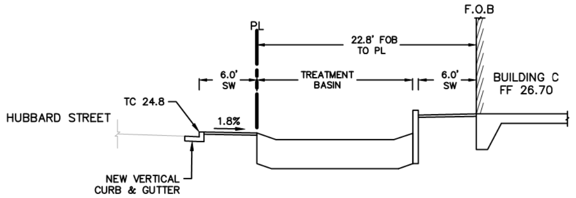
FEBRUARY 28, 2023



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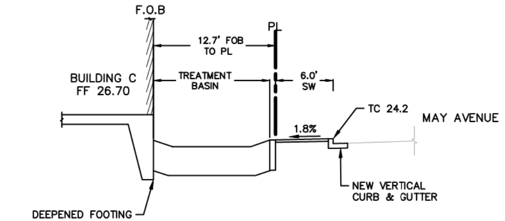

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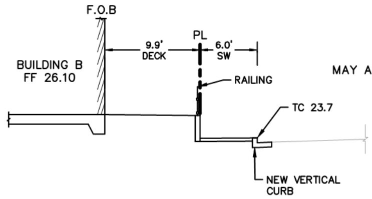

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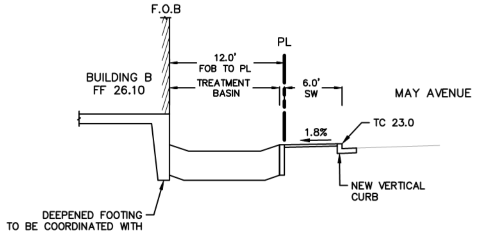

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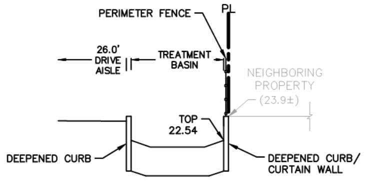

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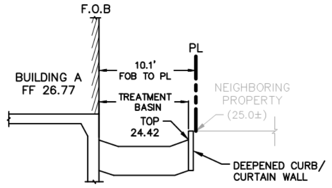

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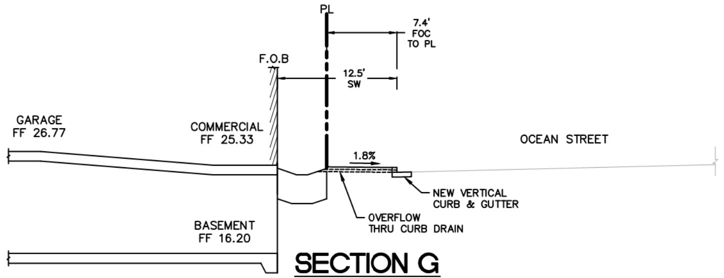

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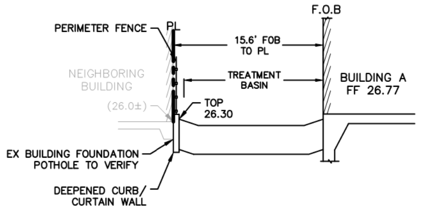

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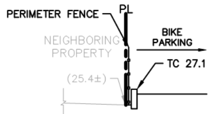

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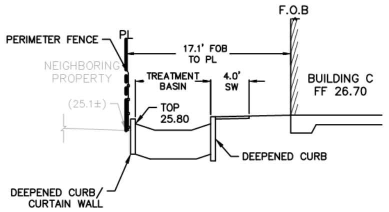
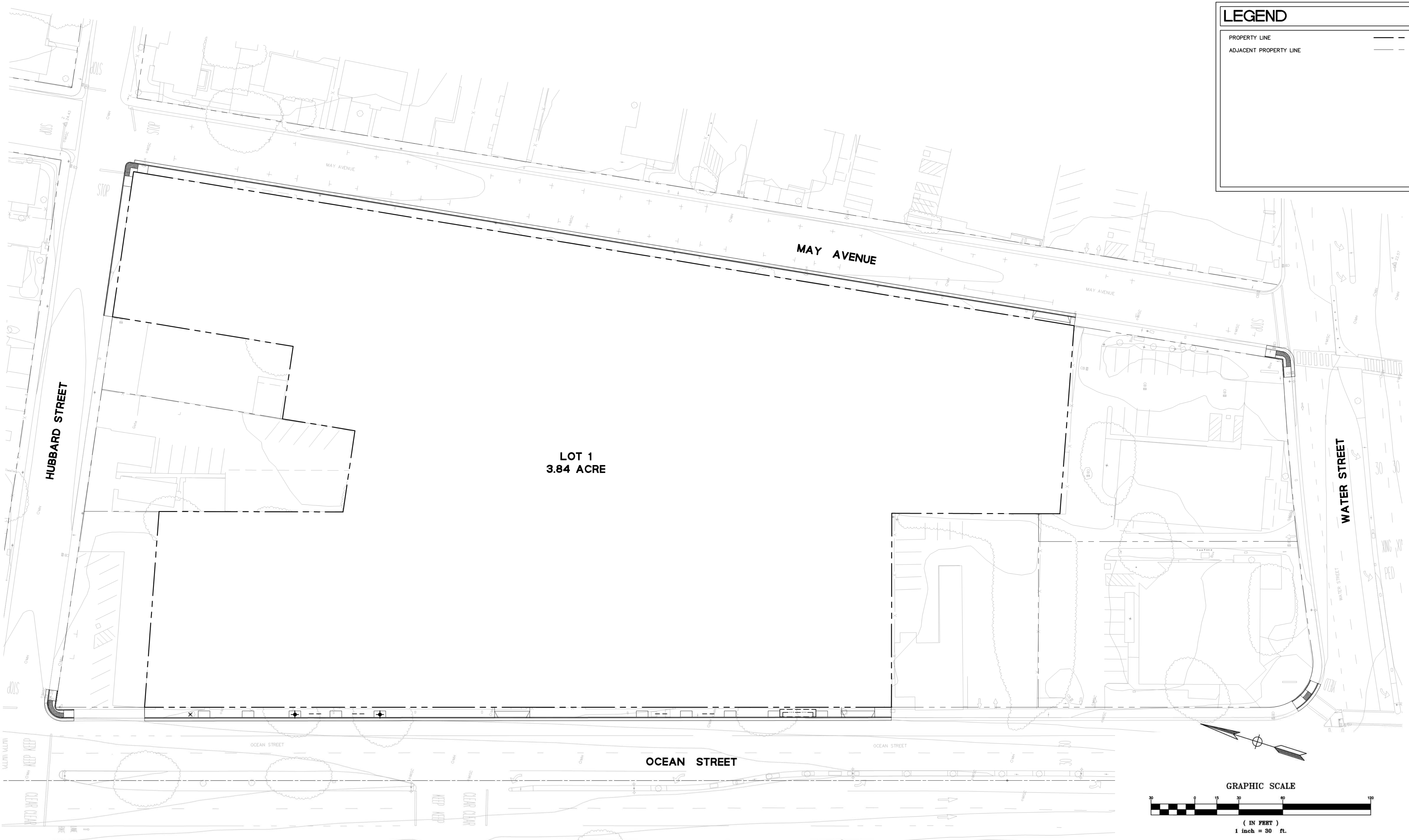


SECTION J

1"=10'

CROSS SECTIONS



LEGEND

PROPERTY LINE	— — - —
ADJACENT PROPERTY LINE	—— - - ——

VESTING TENTATIVE PARCEL MAP

TM5.0

BDE ARCHITECTURE

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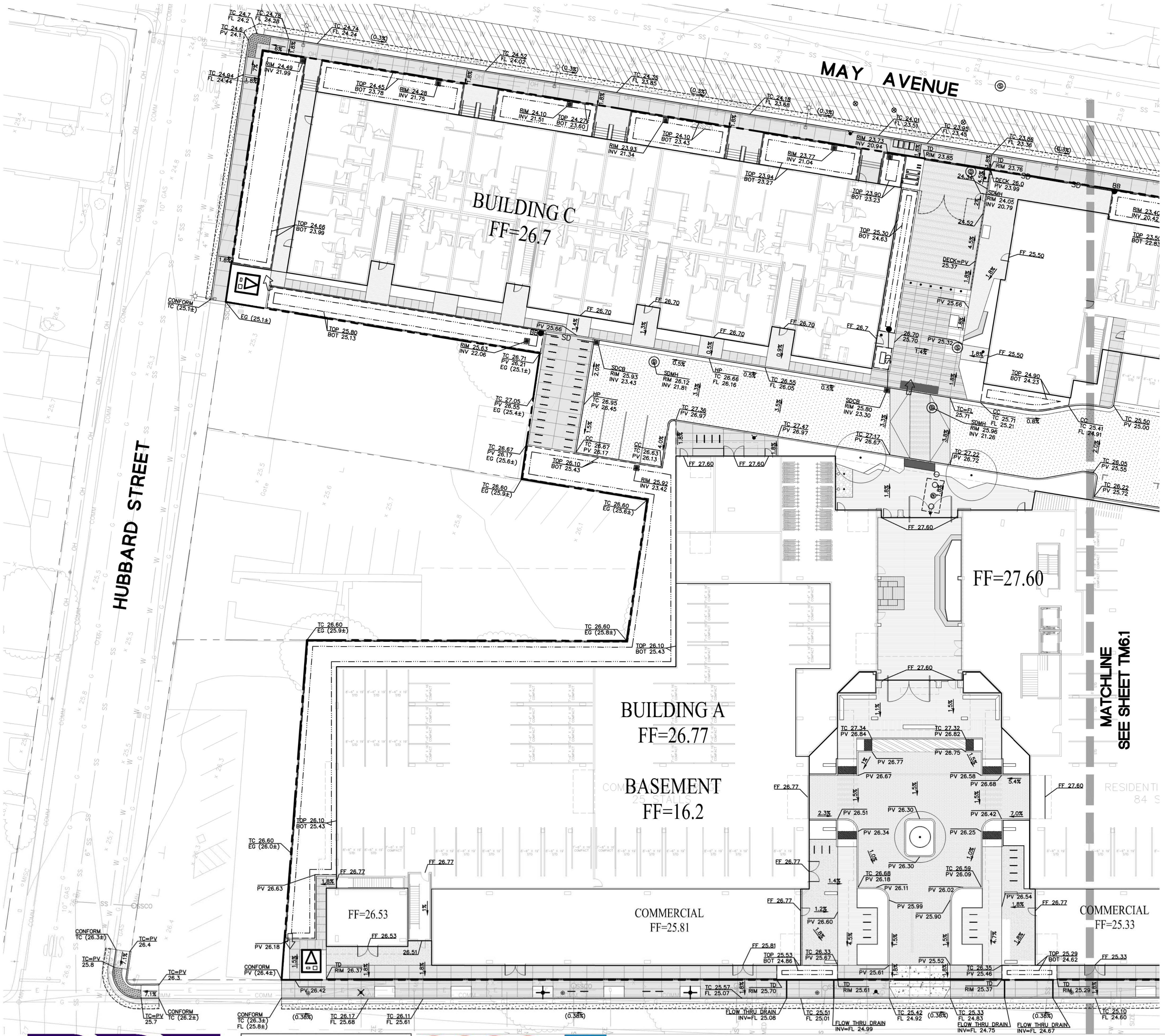
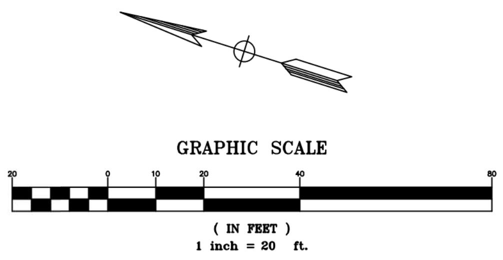


LEGEND

PROPERTY LINE	— - —
ADJACENT PROPERTY LINE	— — —
STREET CENTER LINE	— - —
SAWCUT LINE	- - -
BIORETENTION AREA	▭
VERTICAL CURB & GUTTER	═
VALLEY GUTTER	═
NEW CONCRETE SIDEWALK	▨
AC GRIND & OVERLAY	▨
NEW VEHICULAR CONCRETE	▨
NEW AC	▨
STORM DRAIN MANHOLE	◎
SANITARY SEWER MANHOLE	◎
SANITARY SEWER CLEANOUT	•
TRANASFORMER	XFMR
OVERLAND RELEASE ARROW	⇨



GRADING PLAN TM6.0

ARCHITECTURE

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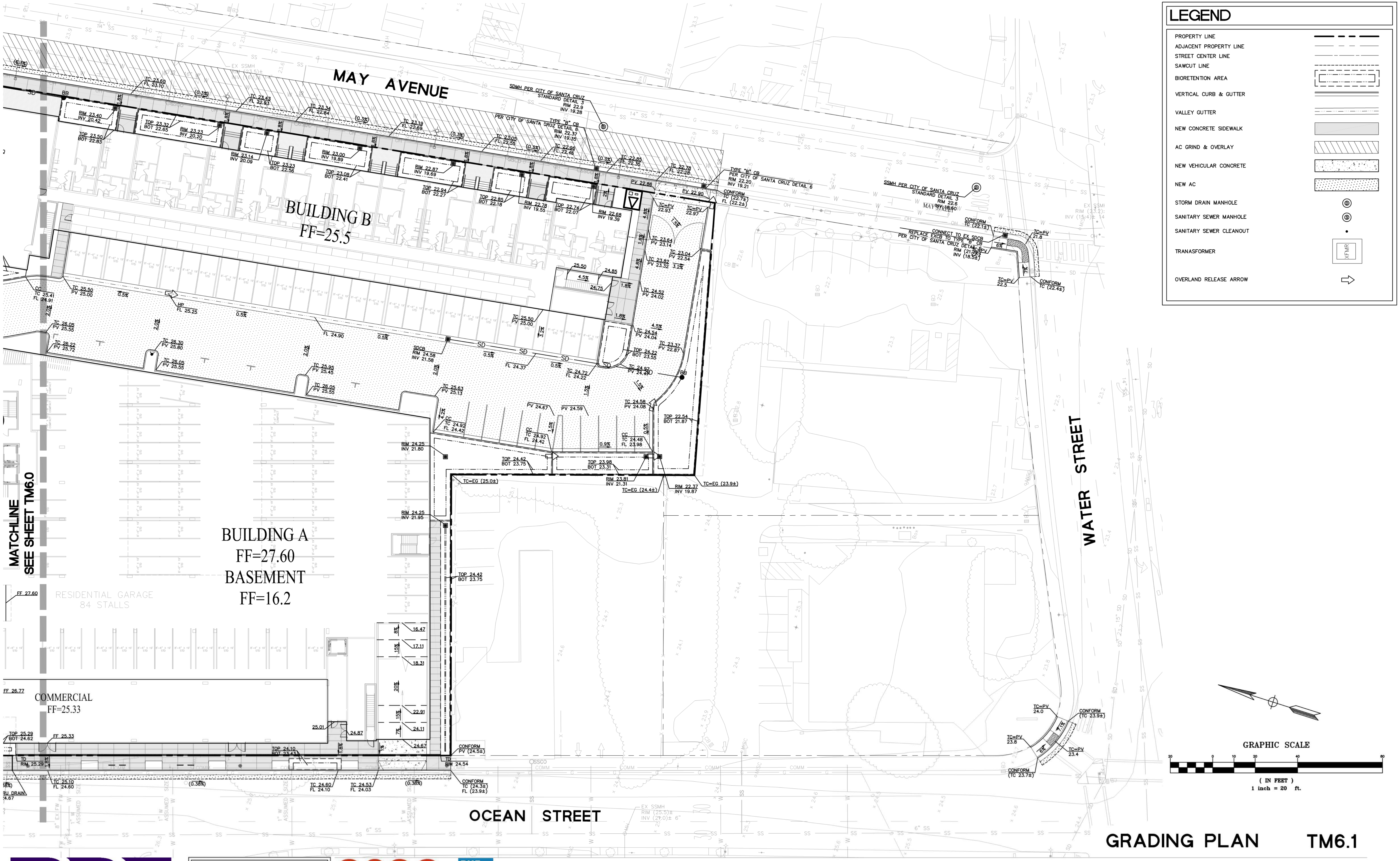

GRADING PLAN TM6.1

3DE ARCHITECTURE

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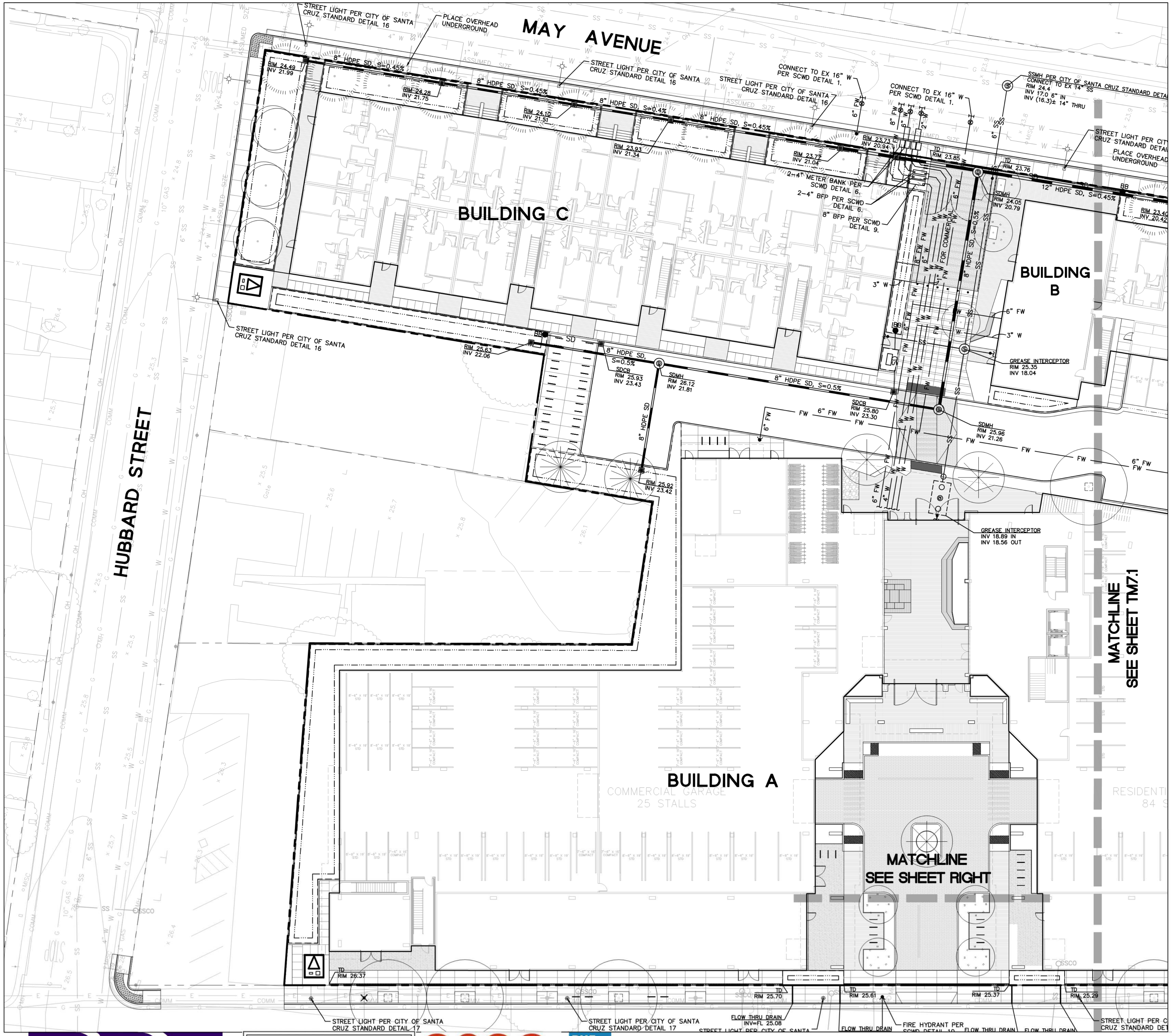
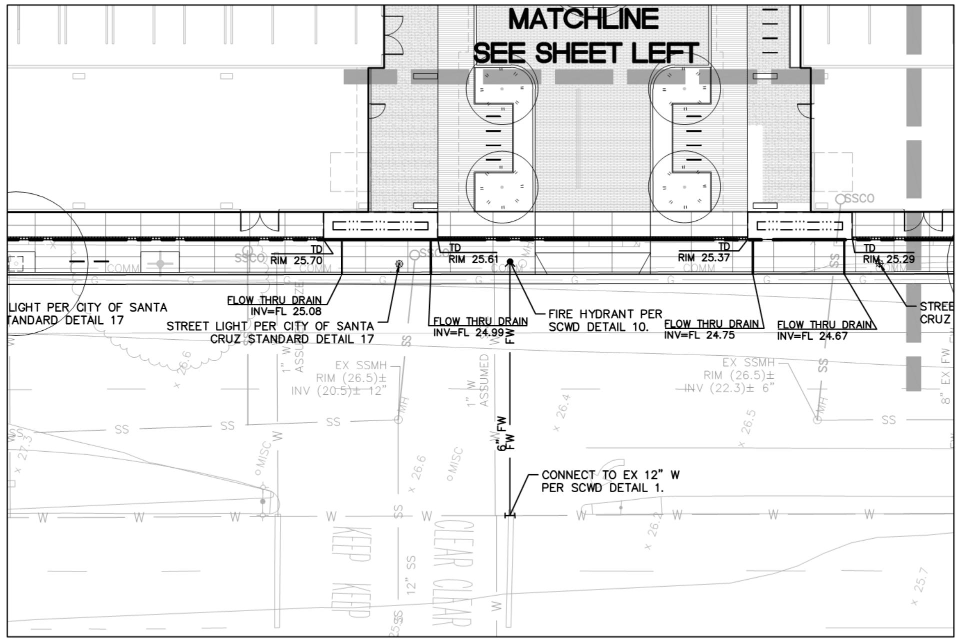
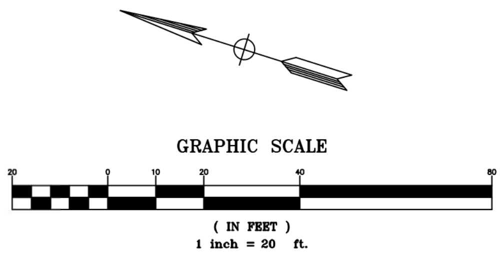


LEGEND

PROPERTY LINE	━━ ━ ━ ━━
ADJACENT PROPERTY LINE	—— – – ——
BIORETENTION AREA	▭
STORM DRAIN MANHOLE	Ⓓ
SANITARY SEWER MANHOLE	Ⓢ
CATCH BASIN	▦
RIM ELEVATION	RIM XX.XX
INVERT ELEVATION	INV XX.XX
WATER VALVE	⊗
FIRE HYDRANT	●
WATER METER	WM
BACKFLOW PREVENTION	▭
SANITARY SEWER CLEANOUT	•
TRANASFORMER	XFMR
STORM DRAIN LINE (UNTREATED)	—— SD ——
STORM DRAIN LINE (TREATED)	━━━━
SANITARY SEWER LINE	——SS——
FIREWATER LINE	—— FW ——
DOMESTIC WATER LINE	—— W ——
TRENCH DRAIN	▬▬▬





UTILITY PLAN TM7.0

BDE ARCHITECTURE

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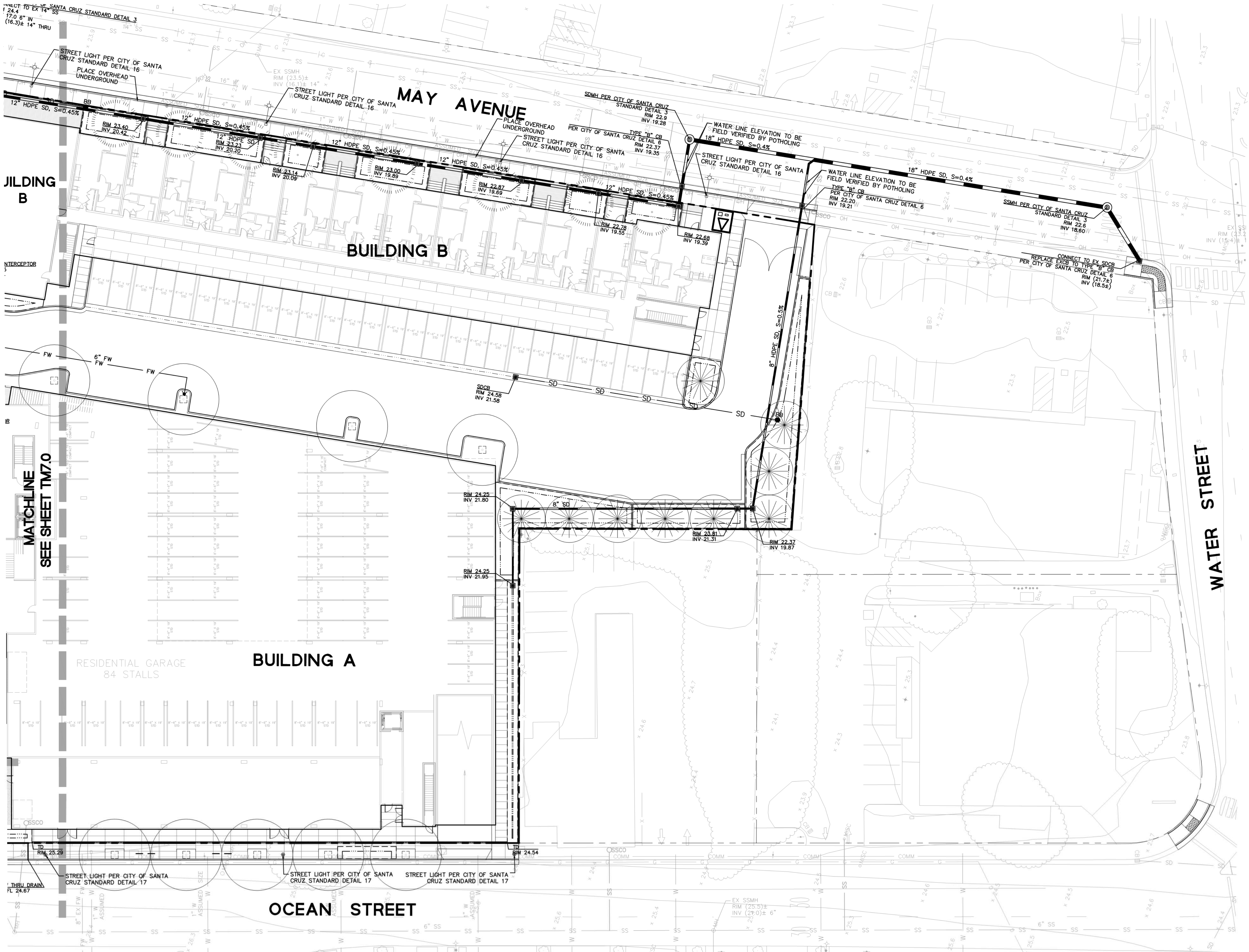
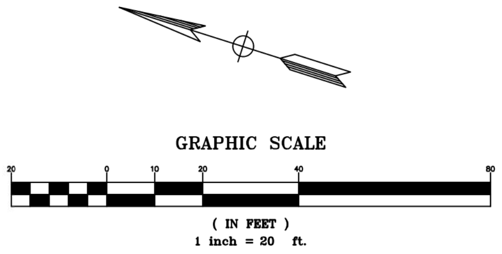
FEBRUARY 28, 2023



LEGEND

PROPERTY LINE	— — —
ADJACENT PROPERTY LINE	— — —
BIORETENTION AREA	▭
STORM DRAIN MANHOLE	Ⓓ
SANITARY SEWER MANHOLE	Ⓢ
CATCH BASIN	▦
RIM ELEVATION	RIM XX.XX
INVERT ELEVATION	INV XX.XX
WATER VALVE	⊗
FIRE HYDRANT	●
WATER METER	WM
BACKFLOW PREVENTION	▭
SANITARY SEWER CLEANOUT	•
TRANASFORMER	XFMR
STORM DRAIN LINE (UNTREATED)	SD
STORM DRAIN LINE (TREATED)	━━━
SANITARY SEWER LINE	SS
FIREWATER LINE	FW
DOMESTIC WATER LINE	W
TRENCH DRAIN	▬▬▬



UTILITY PLAN TM7.1

ARCHITECTURE

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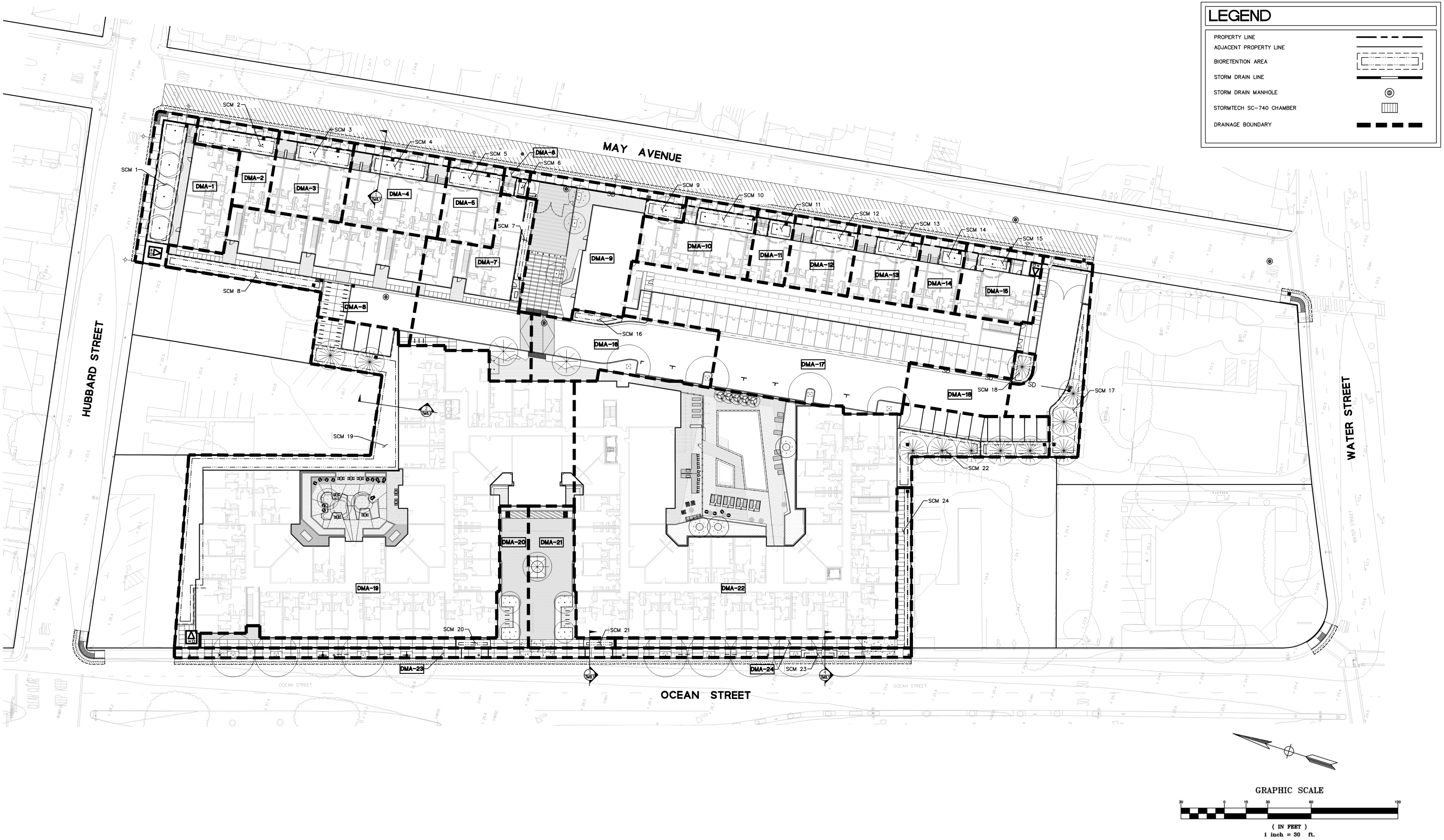
FEBRUARY 28, 2023



STORMWATER CONTROL PLAN TM8.0

BDE ARCHITECTURE

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FEBRUARY 28, 2023

REQUIREMENT CRITERIA

TIER 1. RUNOFF REDUCTION
– SITE IMPERVIOUS SURFACE IS OPTIMIZED.

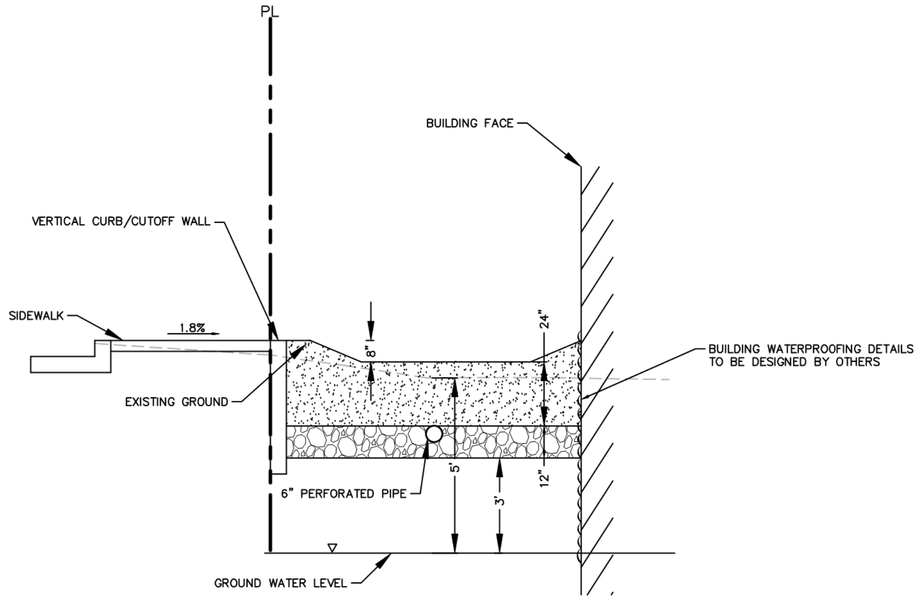
TIER 2. WATER QUALITY TREATMENT
– BIORETENTION AREA IS PROVIDED (MINIMUM 4% OF NEW IMPERVIOUS SURFACE)

TIER 3. RETENTION REQUIREMENT
– SEE CALCULATION BELOW.

TIER 4. PEAK FLOW MANAGEMENT
– EXEMPT. CURRENT SITE IS DISCHARGING TO THE CONCRETE–LINED CHANNEL.

TIER 2 WATER QUALTITY TREATMENT SUMMARY

DRAINAGE AREA	TOTAL AREA (SF)	IMPERVIOUS SURFACE (SF)	PERVIOUS SURFACE (SF)	STORM WATER CONTROL MEASURE	SCM AREA REQUIRED (4% OF NEW IMPERVIOUS SURFACE)	SCM AREA PROVIDED	SCM
DMA-1	5198	3881	1317	BIORETENTION BASIN	155	1317	1
DMA-2	1973	1292	681	BIORETENTION BASIN	52	681	2
DMA-3	2940	2470	470	BIORETENTION BASIN	99	470	3
DMA-4	3753	3282	471	BIORETENTION BASIN	131	471	4
DMA-5	2163	1695	468	BIORETENTION BASIN	68	468	5
DMA-6	330	231	99	BIORETENTION BASIN	9	99	6
DMA-7	6993	6459	534	BIORETENTION BASIN	258	382	7
DMA-8	8565	7225	1340	BIORETENTION BASIN	289	893	8
DMA-9	7444	6891	553	BIORETENTION BASIN	276	303	9
DMA-10	3353	2893	460	BIORETENTION BASIN	116	460	10
DMA-11	1297	1106	191	BIORETENTION BASIN	44	191	11
DMA-12	2203	1858	345	BIORETENTION BASIN	74	345	12
DMA-13	2205	1860	345	BIORETENTION BASIN	74	345	13
DMA-14	1534	1344	190	BIORETENTION BASIN	54	190	14
DMA-15	2654	2434	220	BIORETENTION BASIN	97	220	15
DMA-16	5339	4698	641	BIORETENTION BASIN	188	219	16
DMA-17	20754	19302	1452	BIORETENTION BASIN	772	1217	17
DMA-18	2310	2063	247	BIORETENTION BASIN	83	247	18
DMA-19	39914	34299	5615	BIORETENTION BASIN	1372	4617	19
DMA-20	3641	3264	377	BIORETENTION BASIN	131	133	20
DMA-21	3359	2989	370	BIORETENTION BASIN	120	121	21
DMA-22	41226	36028	5198	BIORETENTION BASIN	1441	1510	22
DMA-23	3425	2975	450	BIORETENTION BASIN	119	120	23
DMA-24	2475	1988	487	BIORETENTION BASIN	80	487	24
TOTAL	175048	152527	22521		TOTAL	15506	



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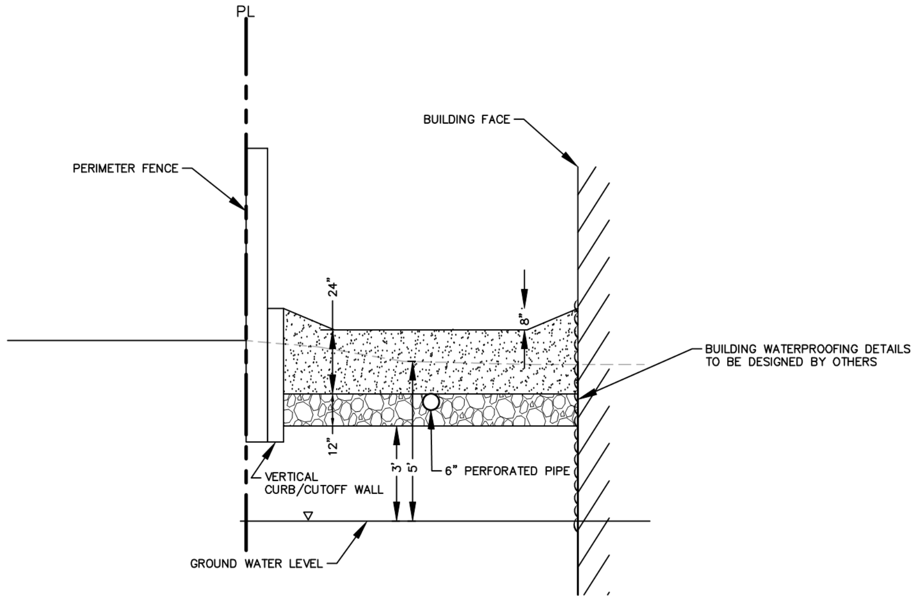

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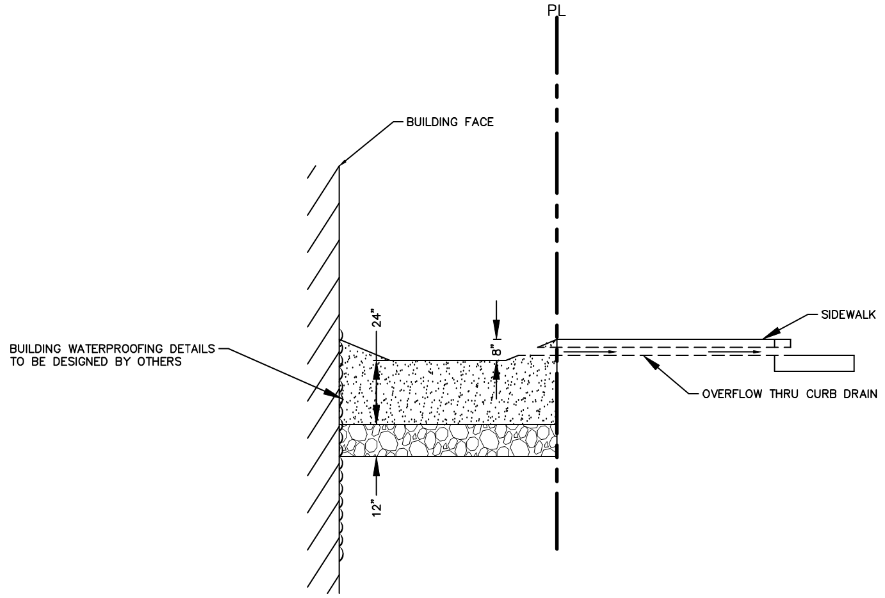

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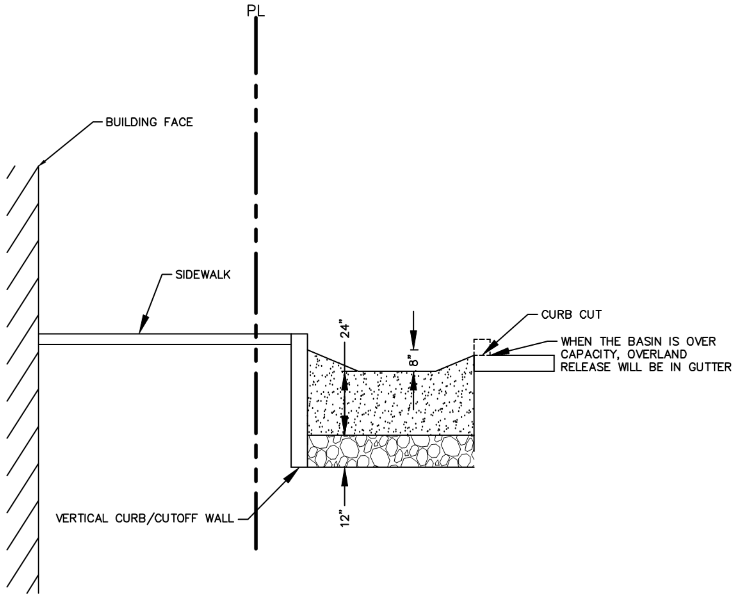


SECTION 2

TIER 3 10% PROJECT RETENTION BASED TREATMENT SUMMARY

TOTAL PROJECT AREA: 175,048 SF
PROPOSED PROJECT PERVIOUS AREA: 22,521 SF
PROPOSED PROJECT IMPERVIOUS AREA: 152,527 SF

PROJECT EFFECTIVE IMPERVIOUS AREA = 152,527 SF + 22,521 SF X 0.1 (LANDSCAPE CORRECTION FACTOR) = 154,779 SF
RETENTION BASED TREATMENT REQUIRED = 10% OF PROJECT EFFECTIVE IMPERVIOUS AREA = 15,478 SF
SITE RETENTION BASIN PROVIDED = 15,506 SF

STORMWATER CONTROL PLAN TM8.1

Appendix C

Geotechnical Report

CORNERSTONE
EARTH GROUP

TYPE OF SERVICES	Geotechnical Investigation
PROJECT NAME	Ocean Place
LOCATION	Ocean, Hubbard, and May Street Santa Cruz, California
CLIENT	Salvatore Caruso Design Corporation
PROJECT NUMBER	908-3-1
DATE	November 22, 2019

CORNERSTONE
EARTH GROUP

Type of Services Geotechnical Investigation
Project Name Ocean Place
Location Ocean, Hubbard, and May Street
Santa Cruz, California
Client Salvatore Caruso Design Corporation
Client Address 980 El Camino Real, Suite 200
Santa Clara, California
Project Number 908-3-1
Date November 22, 2019

Maura Ruffatto

Prepared by **Maura F. Ruffatto, P.E.**
Project Engineer
Geotechnical Project Manager

REGISTERED PROFESSIONAL ENGINEER
MAURA F. RUFFATTO
No. 84145
Exp. 09-30-21
CIVIL
STATE OF CALIFORNIA

Danh T. Tran, P.E.
Senior Principal Engineer
Quality Assurance Reviewer

TABLE OF CONTENTS

Type of Services **Geotechnical Investigation**
Project Name **Ocean Place**
Location **Ocean, Hubbard, and May Street**
Santa Cruz, California

SECTION 1: INTRODUCTION

This geotechnical report was prepared for the sole use of Salvatore Caruso Design Corporation for the Ocean Place project in Santa Cruz, 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 set of architectural plans prepared by Salvatore Caruso Design Corporation dated August 6, 2018
- A Topographic Survey prepared by BKF October 9, 2018

1.1 PROJECT DESCRIPTION

The project will consist of redeveloping the 20-parcel, 3¼-acre site for a new multi-family residential mixed-use project. The project will consist of a four-level podium over a one-level, below-grade parking structure. We assume the parking levels will be of concrete-frame construction and the residential and mixed-use levels will be of wood-frame construction. The first floor will also include open spaces, private unit yards, and retail spaces. The retail spaces will front along Ocean Street. Ramps for the garage access will be located on Ocean Street and May Street. Appurtenant utilities, landscaping, and other improvements necessary for site development are also planned.

Structural loads are not available at this time; however, structural loads are expected to be typical for similar structures. Cuts on the order of 10 to 15 feet are expected for the below-grade level.

1.2 SCOPE OF SERVICES

Our scope of services was presented in our proposal dated September 12, 2019 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 five borings drilled on October 10, 14, and 15, 2019 with truck-mounted hollow-stem auger and track-mounted, limited-access hollow-stem auger drilling equipment and six Cone Penetration Tests (CPTs) advanced on October 4, 2019. The borings were drilled to depths of about 30 to 61½ feet; the CPTs were advanced to depths of about 50 to 85 feet, where drilling refusal was encountered. Seismic shear wave velocity measurements were collected from CPT-2 and CPT-3. Our exploratory borings EB-1, EB-2, EB-3, and EB-5 were advanced adjacent to CPT-1, CPT-2, CPT-6, and CPT-5, respectively 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, washed sieve analyses, Plasticity Index tests, and consolidation tests. Details regarding our laboratory program are included in Appendix B.

1.5 ENVIRONMENTAL SERVICES

We understand that environmental services for the project are being provided by Weber-Hayes. If environmental concerns are present, Weber-Hayes should review our geotechnical recommendations for compatibility with the environmental concerns.

SECTION 2: REGIONAL SETTING

2.1 REGIONAL SEISMICITY

The San Francisco Bay area region is one of the most seismically active areas in the Country. While seismologists cannot predict earthquake events, geologists from the U.S. Geological Survey have recently updated earlier estimates from their 2015 Uniform California Earthquake Rupture Forecast (Version 3) 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%), Rodgers Creek (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 or Rodgers Creek Faults.

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)
Monterey Bay-Tularcitos	7.3	11.8
San Gregorio	7.5	12.1
Zayante-Vergeles	10.3	16.6
San Andreas (1906)	10.8	17.4
Sargent	12.1	19.5

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 project site is located between Ocean Street and May Avenue in Santa Cruz, California. The site is currently occupied by one- to two-level at-grade residential and commercial buildings and at-grade parking lots. The site is bounded by Ocean Street to the west, commercial development and Hubbard Street to the north, May Avenue to the east, and residential and commercial development to the south. The site is relatively level with elevations of approximately 23 to 26 feet above sea level based on the topographic survey provided (NGVD 29).

Surface pavements at Borings EB-3 and EB-5 generally consisted of approximately 1½ to 2 inches of asphalt concrete over 4 inches of aggregate base and surface pavements at Boring EB-1 consisted of approximately 5½ inches of Portland cement concrete. Based on visual observations, the existing pavements are in poor shape with significant alligator cracking.

3.2 SUBSURFACE CONDITIONS

Below the surface pavements or existing ground surface, our explorations EB-2 through EB-5 encountered approximately 2 to 3½ feet of undocumented fill consisting of stiff to very stiff lean clays with varying amounts of sand and medium dense sands with varying amounts of clays. Beneath the undocumented fill or surface pavements, our borings generally encountered medium stiff to stiff lean clay with varying amounts of sand to depths of about 15 to 19½ feet below existing ground surface underlain by loose clayey and/or silty sand to depths ranging from about 22 to 31½ feet, or terminal boring depth in EB-1. Below the silty sand, our borings EB-2 and EB-3 generally encountered medium stiff to stiff lean clay to depths of about 30 to 36 feet, or terminal boring depths. Below the silty sands, Boring EB-4 generally encountered soft to stiff lean clay to about 39½ feet, underlain by soft to stiff sandy silt to 52½, and medium dense to very dense silty sand to the maximum depth explored of 61½ feet. Below the silty sand, Boring EB-5 encountered interbedded layers of medium dense silty sand, medium stiff lean clay, and medium stiff sandy silt to a depth of about 37½ feet, underlain by very soft to stiff lean clay to the terminal boring depth of 47½ feet.

Our CPTs generally encountered interbedded layers of medium stiff to hard clays with varying amounts of sands and silts and loose to very dense sands to the maximum depths explored of about 85 feet, where practical refusal was encountered.

3.2.1 Plasticity/Expansion Potential

We performed two Plasticity Index (PI) tests on representative samples. The test results were used to evaluate expansion potential of surficial soils and the plasticity of fines in potentially liquefiable layers. The surficial PI test resulted in a PI of 23, indicating moderate expansion potential to wetting and drying cycles. The result of the PI test in a potentially liquefiable layer indicated a PI of 12.

3.2.2 In-Situ Moisture Contents

Laboratory testing indicated that the in-situ moisture contents within the upper 15 feet range from about 8 to 20 percent over the estimated laboratory optimum moisture.

3.3 GROUNDWATER

Groundwater was encountered in our exploratory borings EB-2 through EB-5 at depths ranging from about 9 to 11 feet below current grades. Groundwater was estimated at depths of about 5 to 14 feet below current grades based on pore pressure dissipation tests performed in CPT-1 through CPT-6. 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.

Groundwater data available on Geotracker in the project vicinity is recorded at depths of approximately 3½ to 5 feet below existing grades. For our analysis, we assumed a design high groundwater level to be at 5 feet below current grades.

Fluctuations in groundwater levels occur due to many factors including seasonal fluctuation, underground drainage patterns, regional fluctuations, and other factors.

SECTION 4: GEOLOGIC HAZARDS

4.1 FAULT 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 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 21, Section 21.2 of ASCE 7-16 and Supplement No. 1. For our liquefaction analysis we used a PGA_M of 0.62g which was determined in accordance with Section 21.5 of ASCE 7-16.

4.3 LIQUEFACTION POTENTIAL

The site is not currently mapped by California Geologic Survey but is mapped as an area of high to very high liquefaction potential by the County of Santa Cruz (Santa Cruz County, 2015). 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 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 and CPT-6) are presented on Figures 4A and 4F of this report. Calculations for these CPTs are attached as Appendix D.

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 about 1 to 7½ inches 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 ⅔-inch to 5 inches between independent foundation elements.

4.3.4 Ground Rupture Potential

The methods used to estimate liquefaction settlements assume that there is a sufficient cap of non-liquefiable material to prevent ground rupture or sand boils. For ground rupture 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. We evaluated the potential for surface rupture based on the work of Youd and Garris (1995). Based on our analyses, the potential for liquefaction-induced ground rupture at the site is considered high for the northern half of the site due to the loose to medium dense sands encountered in CPT-1, CPT-2, and CPT-3. If ground rupture occurred, the estimated ground movement could be significantly increased over the estimates recommended above. Please refer to the "Conclusions" section of this report for mitigation recommendations.

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.

The closest free face to the site is the San Lorenzo River located to the east of the site, approximately 1,000 feet from the western property line. The river channel is approximately 20 feet deep with an approximately 12 feet high terrace separating the river from the property site. The potential for lateral spreading at the site is high based on the Lateral Displacement Index (LDI) estimates. We analyzed the site for lateral spreading using analytical methods outlined in the 2008 monograph, *Soil Liquefaction During Earthquakes* (Idriss and Boulanger, 2008) and *CPT and SPT Based Liquefaction Triggering Procedures* (Boulanger and Idriss, 2014) by calculating Lateral Displacement Index (LDI) values at each CPT location. The LDI is calculated by integrating maximum shear strains versus depth, representing a measure of the potential maximum displacement (Zhang et al., 2004).

Our analysis indicates a potential for lateral displacement at the site with LDI values ranging from 0.67 to 1.46 calculated for CPT-1 through CPT-6 in the area of the proposed structure, and potential lateral displacement ranging from 0.1 to 2.9 feet. Mitigation options for lateral spreading are presented in subsequent sections of this report.

Provided mitigation measures are taken to improve the loose sandy soils encountered across the site between depths of approximately 5 to 35 feet below existing site grades, the potential for lateral spreading to affect the new construction is low.

4.5 SEISMIC SETTLEMENT/UNSATURATED SAND SHAKING

Loose unsaturated sandy soils can settle during strong seismic shaking. We evaluated the potential for seismic compaction of the loose to medium dense sands above the design groundwater depth based on the work by Robertson and Shao (2010). Our analyses indicate that the unsaturated sands in CPT-3 could experience up to 8 inches of movement after strong seismic shaking. However, we anticipate the loose to medium dense sands will be removed for the below-grade basement excavation. However, any portions of the structure that are at-grade, or at-grade improvements, should be designed for this additional seismic settlement, sands should be over-excavated and re-compacted, or ground improvement should be designed to mitigate additional seismic settlement for at-grade areas.

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 around the northern tip of the Monterey Bay. The site is approximately 1¼ miles inland from the Pacific Ocean shoreline and is approximately 23 to 26 feet above mean sea level (NGVD 29). The site is also mapped by California Geologic Survey as being outside a tsunami inundation zone (CGS, 2009). 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, 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.

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. Descriptions of each concern with brief outlines of our recommendations follow the listed concerns.

- Shallow groundwater, excavation, and construction below groundwater
- Hydrostatic uplift pressures and waterproofing
- Presence of undocumented fill
- Significant static and seismic settlements
- Potential for lateral spreading
- Differential movement at on-grade to on-structure transitions

5.1.1 Shallow Groundwater, Excavation, and Construction Below Groundwater

Shallow groundwater was measured in our exploratory borings at depths ranging from approximately 9 to 11 feet below the existing ground surface and inferred from pore pressure dissipation tests in our CPTs at depths ranging from about 5 to 14 feet. In addition, historic high groundwater in the area is estimated to be on the order of 5 feet below existing grades. Our experience with similar sites in the vicinity indicates that shallow groundwater could significantly impact grading and underground construction. The one-level below-grade basement should be designed to withstand hydrostatic pressure. In our experience, supporting the below-grade structure on a mat foundation overlying ground improvement designed to resist uplift hydrostatic pressures, static and seismic settlement, and ground rupture appears to be feasible for the subsurface conditions encountered at the site. Further discussion of these issues are presented in the “Foundations” section of this report.

Dewatering and shoring of the basement excavation will be required at the site during construction and should be anticipated. Carefully planned and implemented temporary dewatering should be anticipated for the construction of this project. Typically, permanent dewatering of the below-grade basement is not desired due to potential construction complications such as a settlement of adjacent structures and long-term maintenance and operations costs of the site.

As the planned basement excavation will extend below the current groundwater level, we anticipate the need for stabilization of the excavation bottom where construction activities are planned. Further details are provided in the “Earthwork” section of this report.

Based on the site conditions encountered during our investigation, the cuts may be supported by shoring with tie-backs, braced excavations, or using a soil mixed cutoff wall. Because of the groundwater table depth, shoring combined with temporary dewatering will be needed to control the water inflow for the shoring system. Some shoring methods such as the use of wooden lagging may be problematic for installation because of the water seepage and potential flowing sands and may not be feasible below the water table. Where excavations will extend more than about 10 feet, restrained shoring will most likely be required to limit detrimental lateral deflections and settlement behind the shoring. In addition to soil earth pressures, the shoring system will need to support adjacent loads such as construction vehicles and incidental loading, existing structure foundation loads, and street loading. Underpinning of the adjacent structures may be needed depending on the proximity of the excavation to the property line.

We recommend that the contractor implement a monitoring program to monitor the effects of the construction on nearby improvements, including the monitoring of cracking and vertical movement of adjacent structures, nearby streets, sidewalks, parking and other improvements. In critical areas, we recommend that inclinometers or other instrumentation be installed as part of the shoring system to closely monitor lateral movement. A discussion of the general shoring issues are provided in the “Earthwork” section of this report.

5.1.2 Hydrostatic Uplift Pressures and Waterproofing

As previously discussed, it is our opinion that groundwater will be encountered during construction at depths ranging from approximately 5 to 10 feet below current grades. However, for design purposes, including hydrostatic uplift and waterproofing, we recommend a design groundwater depth of 5 feet. Where portions of the mat foundation and related basement structures extend below the design groundwater level, including bottoms of mat foundations, they should be waterproofed and designed to resist potential hydrostatic uplift pressures. Further recommendations are provided in the "Hydrostatic Uplift and Waterproofing" section below.

5.1.3 Presence of Undocumented Fill

As discussed in the "Subsurface" section, several of our borings encountered approximately two to three feet of undocumented fill. Based on the plans provided and the anticipated depth of the proposed basement, the undocumented fills are expected to be removed from within the building footprint. However, if there are portions of the building without a basement or near grade, undocumented fill should be completely removed from within the building footprint.

Undocumented fill outside basement excavation areas, which is left in place, may pose a risk to at-grade footings, slabs-on-grade, and/or exterior surface improvements, such as sidewalks and at-grade pavements. Fills beneath the building footprint not removed during excavation of the basement should be completely removed and replaced as engineered fill. Fills extending into planned pavement and flatwork areas may be left in place provided they are determined to be a low risk for future differential settlement and that the upper 12 inches of fill below pavement subgrade is reworked and compacted. Detailed recommendations are included in the "Earthwork" and "Foundation" sections of our report.

5.1.4 Significant Static and Seismic Settlements

As discussed, our liquefaction analysis indicates there is a potential for liquefaction of localized sand layers during a significant seismic event. Our analysis indicates liquefaction-induced settlement on the order of 1 to 7½ inches, resulting in differential settlements ranging from ⅔-inch to 5 inches between independent foundation elements. In addition, up to 8 inches of dry sand settlement should be expected near CPT-3 for portions of the structure that are at-grade or at-grade improvements. We anticipate the dry sand settlement will be mitigated during the basement excavation. In addition, the site has a moderate to high potential for ground rupture to occur in the northern half of the site that could result in ground deformation and settlements in addition to the estimated liquefaction settlements.

Additionally, we also evaluated immediate and consolidation settlement due to static building loads. We used estimated average mat contact pressures of 650 to 750 pounds per square feet (psf) for the analysis. For a rigid mat foundation, total static settlement was estimated to be on the order of 2 to 3 inches. In addition, depending on final excavation bottom and mat contact pressures, based on the soil conditions encountered, the soil bearing capacities may not to be sufficient for a rigid mat bearing on existing native soils (unimproved).

To mitigate the low soil bearing capacities and the combined potential total and differential settlements mentioned above, we recommend the proposed structure be supported on a mat foundation over ground improvement. Detailed recommendations are provided in the "Foundations" section of this report.

5.1.5 Potential for Lateral Spreading

As previously discussed, there is a potential for lateral displacement towards the adjacent San Lorenzo River. Potential for lateral spreading appears high for the proposed building, particularly on the northern half of the site. Typical techniques to mitigate the potential for lateral spreading include ground improvement to construct a shear key or the installation of shear (pin) piers to effectively create a shear key. Additional recommendations are provided in subsequent sections of this report.

5.1.6 Differential Movement At On-grade to On-Structure Transitions

Some of the at-grade improvements will transition from on-grade support to overlying the basements. Where the depth of soil cover overlying the basement roof in the plaza area is thin or where basement walls extend to within inches of finished grade, these transition areas typically experience increased differential movement due to a variety of causes, including difficulty in achieving compaction of retaining wall backfill closest to the wall. We recommend consideration be given to where engineered fill is placed behind retaining walls extending to near finished grade, and that subslabs be included beneath flatwork or pavers that can cantilever at least 3 feet beyond the wall. If surface improvements are included that are highly sensitive to differential movement, additional measures may be necessary. We also recommend that retaining wall backfill be compacted to 95 percent where surface improvements are planned (see "Retaining Wall" section).

5.2 PLANS AND SPECIFICATIONS REVIEW

We recommend that we be retained 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.

5.3 CONSTRUCTION OBSERVATION AND TESTING

As site conditions may vary significantly between the small-diameter borings performed during this investigation, we also recommend that a Cornerstone representative be present to provide geotechnical observation and testing during earthwork and foundation construction. This will allow us to form an opinion and prepare a letter at the end of construction regarding contractor compliance with project plans and specifications, and with the recommendations in our report. We will also be allowed to evaluate any conditions differing from those encountered during our investigation, and provide supplemental recommendations as necessary. For these reasons, the recommendations in this report are contingent of Cornerstone providing observation and testing during construction. Contractors should provide at least a 48-hour notice when scheduling our field personnel.

SECTION 6: EARTHWORK

6.1 SITE DEMOLITION

All existing improvements not to be reused for the current development, including all foundations, flatwork, pavements, utilities, and other improvements should be demolished and removed from the site. Recommendations in this section apply to the removal of these improvements, which are currently present on the site, prior to the start of mass grading or the construction of new improvements for the project.

Cornerstone should be notified prior to the start of demolition, and should be present on at least a part-time basis during all backfill and mass grading as a result of demolition. Occasionally, other types of buried structures (wells, cisterns, debris pits, etc.) can be found on sites with prior development. If encountered, Cornerstone should be contacted to address these types of structures on a case-by-case basis.

6.1.1 Demolition of Existing Slabs, Foundations and Pavements

We anticipate all slabs, foundations, and pavements will be removed from within the building areas during the excavation of the below-grade basement. If any portion of the building is at-grade, the slabs, foundations, and pavements should be completely removed from within the building areas.

Special care should be taken during the demolition and removal of existing floor slabs, foundations, utilities and pavements to minimize disturbance of the subgrade. Excessive disturbance of the subgrade, which includes either native or previously placed engineered fill, resulting from demolition activities can have serious detrimental effects on planned foundation and paving elements.

Existing foundations are typically mat-slabs, shallow footings, or piers/piles. If slab or shallow footings are encountered, they should be completely removed. If drilled piers are encountered, they should be cut off at an elevation at least 60-inches below the final subgrade elevation. The remainder of the drilled pier could remain in place.

6.1.2 Abandonment of Existing Utilities

All utilities should be completely removed from within planned building areas. We anticipate all utilities will be removed from within the building areas during the excavation of the below-grade basement. If any portions of the buildings are at-grade, the utilities should be completely removed from within the building areas.

Utilities extending beyond the building area may be abandoned in place provided the ends are plugged with concrete, they do not conflict with planned improvements, and that the trench fills do not pose significant risk to the planned surface improvements.

The risk for owners associated with abandoning utilities in place include the potential for future differential settlement of existing trench fills, and/or partial collapse and potential ground loss into utility lines that are not completely filled with grout.

6.2 SITE CLEARING AND PREPARATION

6.2.1 Site Stripping

We anticipate that any surface vegetation and subsurface improvements will be removed during the excavation of the below-grade basement. If there are areas of the site that are at-grade, these areas should be stripped of surface vegetation, and surface and subsurface improvements are to be removed within the proposed development area. A detailed discussion of removal of existing fills is provided later in this report. Demolition of existing improvements is discussed in the prior paragraphs.

6.2.2 Tree and Shrub Removal

Any trees and shrubs designated for removal should have the root balls and any roots greater than ½-inch diameter removed completely. Mature trees are estimated to have root balls extending to depths of 2 to 4 feet, depending on the tree size. Significant root zones are anticipated to extend to the diameter of the tree canopy. Grade depressions resulting from root ball removal should be cleaned of loose material and backfilled in accordance with the recommendations in the “Compaction” section of this report.

6.3 REMOVAL OF EXISTING FILLS

As discussed above, we encountered approximately 1¾ to 3½ feet of undocumented fill consisting of stiff sandy lean clay and medium dense clayey sand in our borings EB-2, EB-3, EB-4 and EB-5. We anticipate all fills will be completely removed from within the building areas during the excavation of the below-grade basement. If any portion of the building is at-grade, or deeper fills are encountered in the basement excavation, the fills should be completely removed from within building areas and to a lateral distance of at least 5 feet beyond the building footprint or to a lateral distance equal to fill depth below the perimeter footing, whichever is greater. Provided the fills meet the “Material for Fill” requirements below, the fills may be reused when backfilling the excavations. Based on review of the samples collected from our borings, it appears that the fill may be reused. If materials are encountered that do not meet the requirements, such as debris, wood, trash, those materials should be screened out of the remaining material and be removed from the site. Backfill of excavations should be placed in lifts and compacted in accordance with the “Compaction” section below.

Fills extending into planned pavement and flatwork areas may be left in place provided they are determined to be a low risk for future differential settlement and that the upper 12 to 18 inches of fill below pavement subgrade is re-worked and compacted as discussed in the “Compaction” section below.

6.4 TEMPORARY CUT AND FILL SLOPES

The contractor is responsible for maintaining all temporary slopes and providing temporary shoring where required. Temporary shoring, bracing, and cuts/fills should be performed in accordance with the strictest government safety standards. On a preliminary basis, the upper 20 feet at the site may be classified as OSHA Soil Type C materials. A Cornerstone representative should be retained to confirm the preliminary site classification. Recommended soil parameters for temporary shoring are provided in the "Temporary Shoring" section of this report.

Excavations performed during site demolition and fill removal should be sloped at 3:1 (horizontal:vertical) within the upper 5 feet below building subgrade. Excavations extending more than 5 feet below building subgrade and excavations into pavement and flatwork areas should be sloped or shored in accordance with OSHA soil classification.

6.5 BELOW-GRADE EXCAVATIONS

Temporary shoring may support the planned cuts up to 15 feet. We have provided geotechnical parameters for shoring design in the section below. The choice of shoring method should be left to the contractor's judgement based on experience, economic considerations and adjacent improvements such as utilities, pavements and foundation loads. Temporary shoring should support adjacent improvements without distress and should be the contractor's responsibility. A pre-condition survey including photographs and installation of monitoring points for existing site improvements should be included in the contractor's scope. We should be provided the opportunity to review the geotechnical parameters of the shoring design prior to implementation; the project structural engineer should be consulted regarding support of adjacent structures.

6.5.1 Temporary Shoring

Based on the site conditions encountered during our investigation, the cuts may be supported by soldier beams and tie-backs, braced excavations, soil nailing, or potentially other methods. Where shoring will extend more than about 10 feet, restrained shoring will most likely be required to limit detrimental lateral deflections and settlement behind the shoring. In addition to soil earth pressures, the shoring system will need to support adjacent loads such as construction vehicles and incidental loading, existing structure foundation loads, and street loading. We recommend that heavy construction loads (cranes, etc.) and material stockpiles be kept at least 15 feet behind the shoring. Where this loading cannot be set back, the shoring will need to be designed to support the loading. The shoring designer should provide for timely and uniform mobilization of soil pressures that will not result in excessive lateral deflections. Minimum suggested geotechnical parameters for shoring design are provided in the table below. Our recommended shoring design parameters are based on encountering primarily medium stiff to stiff clays in the upper 15 to 20 feet underlain by interbedded layers of loose sands and soft to stiff clays and silts, and a design groundwater depth of 5 feet below current grades.

Table 2: Suggested Temporary Shoring Design Parameters

Design Parameter	Design Value
Minimum Lateral Wall Surcharge (upper 5 feet)	120 psf
Cantilever Wall – Triangular Earth Pressure	40 pcf(2)
Restrained Wall – Trapezoidal Earth Pressure	Increase from 0 to 25H(1)(3) psf
Passive Pressure – Starting at 2 feet below the bottom of the adjacent excavation(2)(3)	300 pcf up to 1,000 psf maximum uniform pressure

(1) H equals the height of the excavation; passive pressures are assumed to act over 2.5 times the soldier pile diameter.
(2) The cantilever and restrained pressures are for drained designs with dewatering. If undrained shoring is designed, an additional 40 pcf should be added for hydrostatic pressures below the water table.
(3) Bottom of adjacent excavation is bottom of mass excavation or bottom of mat foundation excavation, whichever is deeper directly adjacent to the shoring element.

The restrained earth estimated for the “soft to medium” clay case shown on Figure 23C of the *FHWA Circular No. 4 – Ground Anchors and Anchored Systems.*

If shotcrete lagging is used for the shoring facing, the permanent retaining wall drainage materials, as discussed in the “Wall Drainage” section of this report, will need to be installed during temporary shoring construction. At a minimum, 2-foot-wide vertical panels should be placed between soil nails or tiebacks that are spaced at 6-foot centers. For 8-foot centers, 4-foot-wide vertical panels should be provided. A horizontal strip drain connecting the vertical panels should be provided, or pass-through connections should be included for each vertical panel.

We performed our borings with hollow-stem auger drilling equipment and as such were not able to evaluate the potential for caving soils, which can create difficult conditions during soldier beam, tie-back, or soil nail installation; caving soils can also be problematic during excavation and lagging placement. The contractor is responsible for evaluating excavation difficulties prior to construction. Where relatively clean sands (especially encountered below groundwater) or difficult drilling or cobble conditions were encountered during our exploration, pilot holes performed by the contractor may be desired to further evaluate these conditions prior to the finalization of the shoring budget.

In addition to anticipated deflection of the shoring system, other factors such as voids created by soil sloughing, and erosion of granular layers due to perched water conditions can create adverse ground subsidence and deflections. The contractor should attempt to cut the excavation as close to neat lines as possible; where voids are created they should be backfilled as soon as possible with sand, gravel, or grout.

As previously mentioned, we recommend that a monitoring program be developed and implemented to evaluate the effects of the shoring on adjacent improvements. All sensitive improvements should be located and monitored for horizontal and vertical deflections and distress cracking based on a pre-construction survey. The monitoring frequency should be established and agree to by the project team prior to start of shoring construction.

The above recommendations are for the use of the design team; the contractor in conjunction with input from the shoring designer should perform additional subsurface exploration they deem necessary to design the chosen shoring system. A California-licensed civil or structural engineer must design and be in responsible charge of the temporary shoring design. The contractor is responsible for means and methods of construction, as well as site safety.

6.5.2 Underpinning

Where foundations for adjacent buildings are above an imaginary 1:1 line drawn up from the bottom of the proposed basement excavation, they should be underpinned, or the shoring should be designed to provide vertical and lateral support for adjacent structures. If underpinning is required, we judge slant piers or helical anchors will be acceptable methods to underpin adjacent structures; if significant debris is encountered during underpinning installation, hand-dug underpinning piers may be required. The following preliminary parameters are presented for consideration by the shoring/underpinning designers. Cornerstone can provide supplemental recommendations if needed once the designer has been engaged.

The vertical capacity of the piers may be designed based on an allowable skin friction of 500 psf for combined dead plus live loads based on a factor of safety of 2.0; dead loads should not exceed two-thirds of the allowable capacities. If underpinning is desired, we will work with the design team to determine minimum pier depth and diameters. Adjacent pier/pile centers should be spaced at least three diameters apart, otherwise, a reduction for group effects may be required. The allowable skin friction may be increased by one-third for wind and seismic loads. Where underpinning piers are less than 3 pier diameters from the excavation, only half of the allowable skin friction should be used for vertical capacity.

An at-rest lateral earth pressure increment of 45 + 8H pcf may be included in the design of the underpinning piers or piles.

To reduce movement and provide adequate foundation support during installation of the underpinning piers, adjacent piers should not be drilled or excavated concurrently. If slant piles are used, they should be designed by the underpinning contractor, and we should review the geotechnical aspects of the underpinning design.

Pier excavations below the design groundwater table of 5 feet may need to be cased, as the saturated sands may cave into the excavations. Groundwater will most-likely be present at the bottom of each excavation; therefore, all groundwater should be pumped out of each hole prior to placing concrete, or the concrete may be placed by tremie pipe. The tops of the piers should be dry packed and jacks used to engage the pier vertical support beneath the building foundations. We recommend that the excavation of all piers be performed under our direct observation to establish that the piers are founded in suitable materials and constructed in accordance with the recommendations presented in this report.

6.5.2 Construction Dewatering

Groundwater levels are expected to be about 5 to 10 feet above the planned excavation bottom; therefore temporary dewatering will be necessary during construction. Design, selection of the equipment and dewatering method, and construction of temporary dewatering should be the responsibility of the contractor. Modifications to the dewatering system are often required in layered alluvial soils and should be anticipated by the contractor. The dewatering plan, including planned dewatering well filter pack materials, should be forwarded to our office for review prior to implementation.

The dewatering design should maintain groundwater at least 5 feet below the bottom of the mass excavation, and at least 2 feet below localized excavations such as deepened footings, elevator shafts, and utilities. If the dewatering system was to shut down for an extended period of time, destabilization and/or heave of the excavation bottom requiring over-excavation and stabilization, flooding and softening, and/or shoring failures could occur; therefore, we recommend that a backup power source be considered.

Temporary draw down of the groundwater table can cause the subsidence outside the excavation area, causing settlement of adjacent improvements. As a draw down of 10 to 15 feet is planned, we evaluated the potential deflection of adjacent improvements. We estimate that there could be up to 1 to 1⅓ inches of settlement. If this settlement is deemed excessive, we recommend alternative shoring methods such as tied back slurry walls or soil mixed curtain walls be considered.

Depending on the groundwater quality and previous environmental impacts to the site and surrounding area, settlement and storage tanks, particulate filtration, and environmental testing may be required prior to discharge, either into storm or sanitary, or trucked to an off-site facility.

6.6 SUBGRADE PREPARATION

After site clearing and demolition is complete, and prior to backfilling any excavations resulting from fill removal or demolition, the excavation subgrade and subgrade within areas to receive additional site fills, slabs-on-grade and/or pavements should be scarified to a depth of 6 inches, moisture conditioned, and compacted in accordance with the “Compaction” section below.

The subgrade for any mat foundation extending to or below groundwater (i.e. the basement level) should generally be cut to the desired grades, including the thickness for any subgrade stabilization, as discussed below.

6.7 SUBGRADE STABILIZATION MEASURES

Soil subgrade and fill materials, especially soils with high fines contents such as clays and silty soils, can become unstable due to high moisture content, whether from high in-situ moisture contents or from winter rains. As the moisture content increases over the laboratory optimum, it becomes more likely the materials will be subject to softening and yielding (pumping) from construction loading or become unworkable during placement and compaction.

As discussed in the "Subsurface" section in this report, the in-situ moisture contents are about 8 to 20 percent over the estimated laboratory optimum in the upper 15 feet of the soil profile. The contractor should anticipate drying the soils prior to reusing them as fill. In addition, repetitive rubber-tire loading will likely de-stabilize the soils.

Even presuming that temporary dewatering will be included for the below-grade parking garage excavations, the soils above the depressed water table will be nearly saturated and will be wet and difficult to work with.

There are several methods to address potential unstable soil conditions and facilitate fill placement and trench backfill. Some of the methods are briefly discussed below. Implementation of the appropriate stabilization measures should be evaluated on a case-by-case basis according to the project construction goals and the particular site conditions.

6.7.1 Below-Grade Excavation Stabilization

The proposed building excavation will extend into saturated clay and sand with varying strength. Due to the high moisture content of this material, it may become unstable under the weight of track-mounted or rubber-tired construction equipment. To provide a firm base for construction of the foundation, it may be necessary to remove an additional 12 to 18 inches of native soil below the foundation level and replace it with a bridging layer, such as crushed rock over a layer of stabilization fabric (Mirafi R5580i, or equivalent). The crushed rock should be consolidated in place with light vibratory equipment. Rubber-tire equipment should not be allowed to operate on the exposed subgrade; the crushed rock should be stockpiled and pushed out over the stabilization fabric. Otherwise, a layer of lean cement-sand slurry layer ("rat slab") may be considered, or a combination of the two. Temporary dewatering to a depth of at least 5 feet below the bottom of the building excavation is recommended during construction.

6.7.2 Scarification and Drying

The subgrade may be scarified to a depth of 6 to 12 inches and allowed to dry to near optimum conditions, if sufficient dry weather is anticipated to allow sufficient drying. More than one round of scarification may be needed to break up the soil clods.

6.7.3 Removal and Replacement

As an alternative to scarification, the contractor may choose to over-excavate the unstable soils and replace them with dry on-site or import materials. A Cornerstone representative should be present to provide recommendations regarding the appropriate depth of over-excavation, whether a geosynthethic (stabilization fabric or geogrid) is recommended, and what materials are recommended for backfill.

6.7.4 Chemical Treatment

Where the unstable area exceeds about 5,000 to 10,000 square feet and/or site winterization is desired, chemical treatment with quicklime (CaO), kiln-dust, or cement may be more costeffective than removal and replacement. Recommended chemical treatment depths will typically range from 12 to 18 inches depending on the magnitude of the instability.

6.8 MATERIAL FOR FILL

6.8.1 Re-Use of On-site Soils

On-site soils with an organic content less than 3 percent by weight may be reused as general fill. General fill should not have lumps, clods or cobble pieces larger than 6 inches in diameter; 85 percent of the fill should be smaller than 2½ inches in diameter. Minor amounts of oversize material (smaller than 12 inches in diameter) may be allowed provided the oversized pieces are not allowed to nest together and the compaction method will allow for loosely placed lifts not exceeding 12 inches.

6.8.2 Potential Import Sources

Imported and non-expansive material should be inorganic with a Plasticity Index (PI) of 15 or less, and not contain recycled asphalt concrete where it will be used within the habitable building areas. To prevent significant caving during trenching or foundation construction, imported material should have sufficient fines. Samples of potential import sources should be delivered to our office at least 10 days prior to the desired import start date. Information regarding the import source should be provided, such as any site geotechnical reports. If the material will be derived from an excavation rather than a stockpile, potholes will likely be required to collect samples from throughout the depth of the planned cut that will be imported. At a minimum, laboratory testing will include PI tests. Material data sheets for select fill materials (Class 2 aggregate base, ¾-inch crushed rock, quarry fines, etc.) listing current laboratory testing data (not older than 6 months from the import date) may be provided for our review without providing a sample. If current data is not available, specification testing will need to be completed prior to approval.

Environmental and soil corrosion characterization should also be considered by the project team prior to acceptance. Suitable environmental laboratory data to the planned import quantity should be provided to the project environmental consultant; additional laboratory testing may be required based on the project environmental consultant’s review. The potential import source should also not be more corrosive than the on-site soils, based on pH, saturated resistivity, and soluble sulfate and chloride testing.

6.9 COMPACTION REQUIREMENTS

All fills, and subgrade areas where fill, slabs-on-grade, and pavements are planned, should be placed in loose lifts 8 inches thick or less and compacted in accordance with ASTM D1557 (latest version) requirements as shown in the table below. In general, clayey soils should be compacted with sheepsfoot equipment and sandy/gravelly soils with vibratory equipment; open-graded materials such as crushed rock should be placed in lifts no thicker than 18 inches consolidated in place with vibratory equipment. Each lift of fill and all subgrade should be firm and unyielding under construction equipment loading in addition to meeting the compaction requirements to be approved. The contractor (with input from a Cornerstone representative) should evaluate the in-situ moisture conditions, as the use of vibratory equipment on soils with high moistures can cause unstable conditions. General recommendations for soil stabilization are provided in the "Subgrade Stabilization Measures" section of this report. Where the soil's PI is 20 or greater, the expansive soil criteria should be used.

Table 3: Compaction Requirements

Description	Material Description	Minimum Relative[1] Compaction (percent)	Moisture[2] Content (percent)
General Fill (within upper 5 feet)	On-Site Expansive Soils	87 – 92	>3
	Low Expansion Soils	90	>1
General Fill (below a depth of 5 feet)	On-Site Expansive Soils	95	>3
	Low Expansion Soils	95	>1
Basement Wall Backfill	Without Surface Improvements	90	>1
Basement Wall Backfill	With Surface Improvements	95[4]	>1
Trench Backfill	On-Site Expansive Soils	87 – 92	>3
Trench Backfill	Low Expansion Soils	90	>1
Trench Backfill (upper 6 inches of subgrade)	On-Site Low Expansion Soils	95	>1
Crushed Rock Fill	¾-inch Clean Crushed Rock	Consolidate In-Place	NA
Non-Expansive Fill	Imported Non-Expansive Fill	90	Optimum
Flatwork Subgrade	On-Site Expansive Soils	87 - 92	>3
Flatwork Subgrade	Low Expansion Soils	90	>1
Flatwork Aggregate Base	Class 2 Aggregate Base[3]	90	Optimum
Pavement Subgrade	On-Site Expansive Soils	87 - 92	>3
Pavement Subgrade	Low Expansion Soils	95	>1
Pavement Aggregate Base	Class 2 Aggregate Base[3]	95	Optimum
Asphalt Concrete	Asphalt Concrete	95 (Marshall)	NA

1 – Relative compaction based on maximum density determined by ASTM D1557 (latest version)
2 – Moisture content based on optimum moisture content determined by ASTM D1557 (latest version)
3 – Class 2 aggregate base shall conform to Caltrans Standard Specifications, latest edition, except that the relative compaction should be determined by ASTM D1557 (latest version)
4 – Using light-weight compaction or walls should be braced

6.10 TRENCH BACKFILL

Utility lines constructed within public right-of-way should be trenched, bedded and shaded, and backfilled in accordance with the local or governing jurisdictional requirements. Utility lines in private improvement areas should be constructed in accordance with the following requirements unless superseded by other governing requirements.

All utility lines should be bedded and shaded to at least 6 inches over the top of the lines with crushed rock (⅜-inch-diameter or greater) or well-graded sand and gravel materials conforming to the pipe manufacturer's requirements. Open-graded shading materials should be consolidated in place with vibratory equipment and well-graded materials should be compacted to at least 90 percent relative compaction with vibratory equipment prior to placing subsequent backfill materials.

General backfill over shading materials may consist of on-site native materials provided they meet the requirements in the "Material for Fill" section, and are moisture conditioned and compacted in accordance with the requirements in the "Compaction" section.

6.11 SITE DRAINAGE

Ponding should not be allowed adjacent to 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 towards suitable discharge facilities. Roof runoff should be directed away from building areas in closed conduits, to approved infiltration facilities, or on to hardscaped surfaces that drain to suitable facilities. Retention, detention or infiltration facilities should be spaced at least 10 feet from buildings, and preferably at least 5 feet from slabs-on-grade or pavements. However, if retention, detention or infiltration facilities are located within these zones, we recommend that these treatment facilities meet the requirements in the Storm Water Treatment Design Considerations section of this report.

6.12 LOW-IMPACT DEVELOPMENT (LID) IMPROVEMENTS

The Municipal Regional Permit (MRP) requires regulated projects to treat 100 percent of the amount of runoff identified in Provision C.3.d from a regulated project's drainage area with low impact development (LID) treatment measures onsite or at a joint stormwater treatment facility. LID treatment measures are defined as rainwater harvesting and use, infiltration, evapotranspiration, or biotreatment. A biotreatment system may only be used if it is infeasible to implement harvesting and use, infiltration, or evapotranspiration at a project site.

Technical infeasibility of infiltration may result from site conditions that restrict the operability of infiltration measures and devices. Various factors affecting the feasibility of infiltration treatment may create an environmental risk, structural stability risk, or physically restrict infiltration. The presence of any of these limiting factors may render infiltration technically infeasible for a proposed project. To aid in determining if infiltration may be feasible at the site, we provide the following site information regarding factors that may aid in determining the feasibility of infiltration facilities at the site.

- The near-surface soils at the site are clayey, and categorized as Hydrologic Soil Group D, and is expected to have infiltration rates of less than 0.2 inches per hour. In our opinion, these clayey soils will significantly limit the infiltration of stormwater.

- Historic high groundwater is anticipated at a depth of 5 feet below site grades, and therefore is expected to be within 10 feet below the base of the infiltration measure.

- In our opinion, infiltration locations within 10 feet of the buildings would create a geotechnical hazard.

- The site has a known geotechnical hazard consisting of soils subject to liquefaction; therefore, stormwater infiltration facilities may not be feasible.

6.12.1 Storm Water Treatment Design Considerations

If storm water treatment improvements, such as shallow bio-retention swales, basins or pervious pavements, are required as part of the site improvements to satisfy Storm Water Quality (C.3) requirements, we recommend the following items be considered for design and construction.

6.12.1.1 General Bioswale Design Guidelines

- If possible, avoid placing bioswales or basins within 10 feet of the building perimeter or within 5 feet of exterior flatwork or pavements. If bioswales must be constructed within these setbacks, the side(s) and bottom of the trench excavation should be lined with 10-mil visqueen to reduce water infiltration into the surrounding expansive clay.

- Bioswales constructed within 3 feet of proposed buildings may be within the foundation zone of influence for perimeter wall loads. Therefore, where bioswales will parallel foundations and will extend below the “foundation plane of influence,” an imaginary 1:1 plane projected down from the bottom edge of the foundation, the foundation will need to be deepened so that the bottom edge of the bioswale filter material is above the foundation plane of influence.

- The bottom of bioswale or detention areas should include a perforated drain placed at a low point, such as a shallow trench or sloped bottom, to reduce water infiltration into the surrounding soils near structural improvements, and to address the low infiltration capacity of the on-site clay soils.

6.12.1.2 Bioswale Infiltration Material

- Gradation specifications for bioswale filter material, if required, should be specified on the grading and improvement plans.

- Compaction requirements for bioswale filter material in non-landscaped areas or in pervious pavement areas, if any, should be indicated on the plans and specifications to satisfy the anticipated use of the infiltration area.

- If required, infiltration (percolation) testing should be performed on representative samples of potential bioswale materials prior to construction to check for general conformance with the specified infiltration rates.

- It should be noted that multiple laboratory tests may be required to evaluate the properties of the bioswale materials, including percolation, landscape suitability and possibly environmental analytical testing depending on the source of the material. We recommend that the landscape architect provide input on the required landscape suitability tests if bioswales are to be planted.

- If bioswales are to be vegetated, the landscape architect should select planting materials that do not reduce or inhibit the water infiltration rate, such as covering the bioswale with grass sod containing a clayey soil base.

- If required by governing agencies, field infiltration testing should be specified on the grading and improvement plans. The appropriate infiltration test method, duration and frequency of testing should be specified in accordance with local requirements.

- Due to the relatively loose consistency and/or high organic content of many bioswale filter materials, long-term settlement of the bioswale medium should be anticipated. To reduce initial volume loss, bioswale filter material should be wetted in 12 inch lifts during placement to pre-consolidate the material. Mechanical compaction should not be allowed, unless specified on the grading and improvement plans, since this could significantly decrease the infiltration rate of the bioswale materials.

- It should be noted that the volume of bioswale filter material may decrease over time depending on the organic content of the material. Additional filter material may need to be added to bioswales after the initial exposure to winter rains and periodically over the life of the bioswale areas, as needed.

6.12.1.3 Bioswale Construction Adjacent to Pavements

If bio-infiltration swales or basins are considered adjacent to proposed parking lots or exterior flatwork, we recommend that mitigative measures be considered in the design and construction of these facilities to reduce potential impacts to flatwork or pavements. Exterior flatwork, concrete curbs, and pavements located directly adjacent to bio-swales may be susceptible to settlement or lateral movement, depending on the configuration of the bioswale and the setback between the improvements and edge of the swale. To reduce the potential for distress to these improvements due to vertical or lateral movement, the following options should be considered by the project civil engineer:

- Improvements should be setback from the vertical edge of a bioswale such that there is at least 1 foot of horizontal distance between the edge of improvements and the top edge of the bioswale excavation for every 1 foot of vertical bioswale depth, or

- Concrete curbs for pavements, or lateral restraint for exterior flatwork, located directly adjacent to a vertical bioswale cut should be designed to resist lateral earth pressures in accordance with the recommendations in the “Retaining Walls” section of this report, or concrete curbs or edge restraint should be adequately keyed into the native soil or engineered to reduce the potential for rotation or lateral movement of the curbs.

SECTION 7: 2019 CBC 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 SITE LOCATION AND PROVIDED DATA FOR 2019 CBC SEISMIC DESIGN

The project is located at longitude 36.981291° and latitude -122.021198°, which is based on Google Earth (WGS84) coordinates at the center of the site at Ocean, Hubbard, and May Streets, Santa Cruz, California. We have assumed that a Seismic Importance Factor (I_e) of 1.00 has been assigned to the structure(s) 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. We understand the Risk Category may change. Once the above information has been confirmed by the project structural engineer we may need to revise our recommendations below.

7.2 SITE CLASSIFICATION – CHAPTER 20 OF ASCE 7-16

Code-based site classification and ground motion attenuation relationships are based on the time-weighted average shear wave velocity of the top approximately 100 feet (30 meters) of the soil profile (V_{S30}).

As discussed in Section 3 of our report, our borings encountered loose to very dense silty and clayey sands, soft to hard lean clays, and soft to stiff silt deposits to a depth of 85 feet, the maximum depth explored (practical refusal encountered). Shear wave velocity (V_S) measurements were performed while advancing CPT-2 and CPT-3, resulting in a time-averaged shear wave velocity for the top 30 meters (V_{S30}) of 196 meters per second. Provided ground improvement is performed to mitigate liquefaction potential per our recommendations, in accordance with Table 20.3-1 of ASCE 7-16, we recommend the site be classified as Soil Classification D, which is described as a “stiff 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 outlined below. Our site-specific ground motion hazard analysis considered a V_{S30} of 196 m/s (643 ft/s).

We note that if ground improvement is not implemented the site will fall under the criteria of Site Class E or F and the seismic design parameters presented in Sections 7.4 through 7.6 of this report will no longer be valid. If the site cannot be classified as Soil Classification D as discussed above, our analysis will have to be revised including a site-specific response analysis following Section 21.1 of ASCE 7-16.

7.3 CODE-BASED SEISMIC DESIGN PARAMETERS

Code-based spectral acceleration parameters were determined based on mapped acceleration response parameters adjusted for the specific site conditions. Mapped Risk-Adjusted Maximum

Considered Earthquake (MCE_R) spectral acceleration parameters (S_S and S_1) were determined using the ATC Hazards by Location website (https://hazards.atcouncil.org).

The mapped acceleration parameters were adjusted for local site conditions based on the average soil conditions for the upper 100 feet (30 meters) of the soil profile. Code-based MCE_R spectral response acceleration parameters adjusted for site effects (S_{MS} and S_{M1}) and design spectral response acceleration parameters (S_{DS} and S_{D1}) are presented in Table 4.

In accordance with CBC Section 1613.2.5, Risk Category I, II, or III structures with mapped spectral response acceleration parameter at the 1-second period (S_1) equal to or greater than 0.75, are assigned Seismic Design Category E. In accordance with Section 11.4.8 of ASCE 7-16, structures on Site Class D sites with mapped 1-second period spectral acceleration (S_1) values greater than or equal to 0.2 require a site-specific ground motion hazard analysis be performed in accordance with Section 21.2 of ASCE 7-16. **Design site-specific seismic parameters are presented in Table 7, Section 7.5. The values in Table 4 should not be used for design**. Values summarized in Table 4 are only used to determine Seismic Design Category and comparison with minimum code requirements in our site-specific ground motion hazard analysis (Section 7.4 to follow).

Table 4: 2019 CBC Site Categorization and Site Coefficients

Classification/Coefficient	Design Value
Site Class	D
Site Latitude	36.981291°
Site Longitude	-122.021198°
Risk Category	II
Seismic Design Category	To be determined by S.E.
Short Period Mapped Spectral Acceleration – S_S	1.666g
1-second Period Mapped Spectral Acceleration – S_1	0.639g
Short-Period Site Coefficient – F_a	1.0
Long-Period Site Coefficient – F_v	2.5
Short Period MCE Spectral Response Acceleration Adjusted for Site Effects – S_{MS}	1.666g
1-second Period MCE Spectral Response Acceleration Adjusted for Site Effects – S_{M1}	1.598g
Short Period, Design Earthquake Spectral Response Acceleration – S_{DS}	1.111g
1-second Period, Design Earthquake Spectral Response Acceleration – S_{D1}	1.065g
Long-Period Transition – T_L	12 seconds
Site Coefficient – F_{PGA}	1.1
Site Modified Peak Ground Acceleration – PGA_M	0.769g

Note: S.E. = Structural Engineer

7.4 SITE-SPECIFIC GROUND MOTION HAZARD ANALYSIS

In accordance with Section 11.4.8 of ASCE 7-16, we performed a ground motion hazards analysis (GMHA) in accordance with Chapter 21, Section 21.2 of ASCE 7. Following the methodology outlined in Section 21.2, 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.

We performed a site-specific GMHA in accordance with ASCE 7-16 Chapter 21.2 and 2019 CBC Section 1803.6. Our analyses were performed using the USGS interface Unified Hazard Tool (UHT) based on the UCERF 3 Data Set, Business Seismic Safety Council (BSSC) Scenario Catalog 2014 event set (BSSC 2014), and the 2014 National Seismic Hazard Maps – Source Parameters (NSHMP deterministic event set). Additionally, we utilized the USGS program Response Spectra Plotter with combined models (Combined: WUS 2014 (4.1)).

Our analysis utilized the mean ground motions predicted by four of the Next Generation Attenuation West 2 (NGA-West 2) relationships: Boore-Atkinson (2013), Campbell-Bozorgnia (2013), Chiou-Youngs (2013), and Abrahamson-Silva (2013). Rotation factors (scale factors) were determined as specified in ASCE 7-16 Chapter 21, Section 21.2, to calculate the maximum rotated component of ground motions (ASCE, 2016).

7.4.1 Probabilistic MCE_R

We performed a probabilistic seismic hazard analysis (PSHA) in accordance with ASCE 7-16 Section 21.2.1. The probabilistic MCE acceleration response spectrum is defined as the 5 percent damped acceleration response spectrum having a 2 percent probability of exceedance in a 50-year period (2,475-year return period). The probabilistic MCE spectrum was multiplied by Risk Coefficients (C_R) to determine the probabilistic MCE_R. We used Risk Coefficients (C_{RS} and C_{R1}) of 0.933 and 0.912, respectively, based on ASCE 7-16 Section 21.2.1.1 - Method 1 and the ATC website. Risk coefficients for the various periods are presented in Table 5, Column 3.

The resulting probabilistic MCE_R is presented on Figure 5 (red line). Spectral ordinates are tabulated in Table 5, Column 6.

7.4.2 Deterministic MCE_R

We performed deterministic seismic hazard analyses in accordance with ASCE 7-16 Section 21.2.2 and ASCE 7-16 Supplement No. 1. The deterministic MCE_R acceleration response spectrum is calculated as the largest 84th percentile ground motion in the direction of maximum horizontal response for each period for characteristic earthquakes on all known active faults within the region. As shown in Table 1, the site is located within approximately 25 kilometers of four major fault sources. The largest deterministic ground motion for all periods resulted from a M_w 7.44 earthquake on the San Gregoria (North) Fault, located at a distance of approximately 17.6 km from the site. The top two significant seismic sources determined for the site based on deaggragation of the UCERF 3 data set using the Unified Hazard Tool were the San Andreas

(Santa Cruz Mountains) Fault with a magnitude of 7.15 and a distance of approximately 17.0 km from the site and the San Gregorio (North) Fault with a magnitude of 7.44 and a distance of approximately 17.6 km from the site.

In accordance with Supplement No.1 of ASCE 7-16, when the largest spectral response acceleration of the resulting deterministic ground motion response spectrum is less than $1.5F_a$ then the largest 84th percentile rotated response spectrum (Table 5, Column 4) shall be scaled by a single factor such that the maximum response spectral acceleration equals $1.5F_a$. For Site Classes A, B, C and D, F_a is determined using Table 11.4.1 with the value of S_s taken as 1.5; for Site Class E, F_a shall be taken as 1.0. When the largest spectral response acceleration of the probabilistic ground motion response of 21.2.1 is less than $1.2F_a$, the deterministic ground motion response spectrum does not need to be calculated.

As the largest probabilistic spectral response acceleration was determined to be 2.098 which is greater than $1.2F_a$, where F_a is taken as 1.000 from Table 11.4-1 in ASCE 7-16 Supplement No.1, the 84th percentile rotated response spectrum was calculated as part of the deterministic analyses. The maximum spectral acceleration from the 84th percentile rotated response spectrum was then compared to $1.5F_a$ to determine if a scale factor needed to be applied. The deterministic MCE spectrum are tabulated in Table 5, Column 5. The deterministic MCE_R is presented graphically on Figure 5 (blue line).

7.4.3 Site-Specific MCE_R

The site-specific MCE_R is defined by ASCE 7-16 Section 21.2.3 as the lesser of the deterministic and probabilistic MCE_R's at each period. The site-specific MCE_R spectrum was calculated by taking the lesser of the deterministic MCE_R and the probabilistic MCE_R. Spectral ordinates for the site-specific MCE_R are tabulated in Table 5, Column 7 and shown graphically on Figure 5 (dashed black line).

Table 5: Development of Site-Specific MCE_R Spectrum

Period (seconds)	CBC General Spectrum (g)	Risk Coefficient	Det. 84th Percentile Rotated	Deterministic MCE_R (g)	Probabilistic MCE_R (g)	Site-Specific MCE_R (g)
0.000	0.444	0.933	0.549	0.610	0.796	0.610
0.050	0.618	0.933	0.567	0.629	1.083	0.629
0.100	0.792	0.933	0.851	0.945	1.370	0.945
0.150	0.965	0.933	1.080	1.199	1.592	1.199
0.192	1.111	0.933	1.180	1.310	1.778	1.310
0.200	1.111	0.933	1.199	1.331	1.813	1.331
0.250	1.111	0.932	1.282	1.423	1.949	1.423

*Table 5 Continued on next page

Table 5: Development of Site-Specific MCE_R Spectrum (continued)

Period (seconds)	CBC General Spectrum (g)	Risk Coefficient	Det. 84th Percentile Rotated	Deterministic MCE_R (g)	Probabilistic MCE_R (g)	Site-Specific MCE_R (g)
0.300	1.111	0.930	1.324	1.470	2.085	1.470
0.400	1.111	0.928	1.351	1.500	2.092	1.500
0.500	1.111	0.925	1.338	1.485	2.098	1.485
0.750	1.111	0.919	1.123	1.246	1.801	1.246
0.959	1.111	0.913	0.994	1.104	1.612	1.104
1.000	1.065	0.912	0.969	1.076	1.574	1.076
2.000	0.533	0.912	0.545	0.605	0.928	0.605
3.000	0.355	0.912	0.370	0.411	0.657	0.411
4.000	0.266	0.912	0.268	0.298	0.504	0.298
5.000	0.213	0.912	0.206	0.229	0.407	0.229

7.4.4 Design Response Spectrum

The site-specific Design Response Spectrum (DRS) is defined in ASCE 7-16 Section 21.3 as two-thirds of the site-specific MCE_R, but not less than 80% of the general design response spectrum. Spectral accelerations corresponding to two-thirds of the MCE_R are tabulated in Table 6, Column 2. Ordinates corresponding to 80% of the general Site Class D response spectrum are tabulated below in Table 6, Column 3. Ordinates of the site-specific DRS are tabulated in Table 6, Column 4. Development of the site-specific DRS is presented graphically on Figure 6 (dashed black line).

Table 6: Development of Site-Specific Design Response Spectrum

Period (seconds)	2/3 Site-Specific MCE_R (g)	80% CBC General Spectrum (g)	Design Response Spectrum (g)
0.000	0.406	0.355	0.406
0.050	0.420	0.494	0.494
0.100	0.630	0.633	0.633
0.150	0.799	0.772	0.799
0.192	0.873	0.889	0.889
0.200	0.887	0.889	0.889
0.250	0.949	0.889	0.949
0.300	0.980	0.889	0.980
0.400	1.000	0.889	1.000

*Table 6 Continued on next page

Table 6: Development of Site-Specific Design Response Spectrum (continued)

Period (seconds)	2/3 Site-Specific MCE_R (g)	80% CBC General Spectrum (g)	Design Response Spectrum (g)
0.500	0.990	0.889	0.990
0.750	0.831	0.889	0.889
0.959	0.736	0.888	0.888
1.000	0.717	0.852	0.852
2.000	0.403	0.426	0.426
3.000	0.274	0.284	0.284
4.000	0.199	0.213	0.213
5.000	0.153	0.170	0.170

7.5 DESIGN ACCELERATION PARAMETERS

Site-specific design acceleration parameters (S_{DS} and S_{D1}) were determined in accordance with Section 21.4 of ASCE 7-16. S_{DS} is defined as the design spectral acceleration at 90% of the maximum spectral acceleration, S_a, obtained from the site-specific spectrum, at any period within the range from 0.2 to 5 seconds, inclusive. S_{D1} is defined as the maximum value of the product, TS_a, for periods from 1 to 2 seconds for sites with $v_{s,30} > 1{,}200$ ft/s ($v_{s,30} > 365.76$ m/s) and for periods from 1 to 5 seconds for sites with $v_{s,30} \leq 1{,}200$ ft/s ($v_{s,30} \leq 365.76$ m/s).

Site-specific MCE_R spectral response acceleration parameters (S_{MS} and S_{M1}) are calculated as 1.5 times the S_{DS} and S_{D1} values, respectively, but not less than 80% of the code-based values presented in Table 4. Site-specific design acceleration parameters are summarized in Table 7.

When using the Equivalent Lateral Force Procedure, ASCE 7-16 Section 21.4 allows using the spectral acceleration at any period (T) in lieu of S_{D1}/T in Eq. 12.8-3 and $S_{D1}T_L/T^2$ in Eq. 12.8-4. The site-specific spectral acceleration at any period may be calculated by interpolation of the spectral ordinates in Table 6, Column 4.

Table 7: Site-Specific Design Acceleration Parameters

Parameter	Value
S_{DS}	0.900
S_{D1}	0.852
S_{MS}	1.350
S_{M1}	1.278

7.6 SITE-SPECIFIC MCE_G PEAK GROUND ACCELERATION

We calculated the Site-Specific MCE_G Peak Ground Acceleration (PGA_M) in accordance with ASCE 7-16 Section 21.5. The Site-Specific PGA_M is calculated as the lesser of probabilistic and deterministic geometric mean PGA. The 2% in 50-year probabilistic geometric mean PGA is 0.78g. The deterministic PGA is considered the greater of the largest 84th percentile deterministic geometric mean PGA (0.50g) or one-half of the tabulated F_{PGA} value from ASCE 7-16 Table 11.8.1 with the value of PGA taken as 0.5g. For Site Class D, F_{PGA} is 1.100 and one-half of the F_{PGA} is 0.55g; therefore, the deterministic PGA is 0.55g. Additionally, the Site-Specific PGA_M may not be less than 80% of the mapped PGA_M determined from ASCE 7-16 Equation 11.8-1. The mapped PGA_M for the site is 0.77g; 80% of PGA_M is 0.62g. Therefore, the Site-Specific PGA_M for the site is 0.62g.

We note that if ground improvement is not implemented that the seismic design parameters presented in Sections 7.4 through 7.6 of this report would have to be revised including a site specific response analysis following Section 21.1 of ASCE 7-16. We can provide a fee proposal for this work, if desired.

SECTION 8: FOUNDATIONS

8.1 SUMMARY OF RECOMMENDATIONS

In our opinion, the proposed structures may be supported on a mat foundation over ground improvement provided the recommendations in the “Earthwork” section and the sections below are followed.

8.2 REINFORCED CONCRETE MAT FOUNDATION OVER GROUND IMPROVEMENT

As previously discussed, due to the estimated low bearing capacities of the soils below the bottom of basement, high static and seismic differential settlements, and potential for ground rupture, the structure may be supported on a mat foundation overlying ground improvement as recommended below. We recommend design consideration is given for shallow groundwater including waterproofing and dewatering techniques.

Based on our experience, we estimate a mat foundation underlain by ground improvement may be designed for maximum average allowable bearing pressures of 2,500 to 3,000 pounds per square foot (psf) with localized allowable bearing pressures of 4,000 to 6,000 pounds per square foot (psf) for dead plus live loads. Top and bottom mats of reinforcing steel should be included as required to help span irregularities and differential settlement.

The above bearing pressure estimates should be evaluated further once a design-build ground improvement contractor has been chosen. Recommendations for ground improvement are provided in the following sections.

8.2.1 Settlement

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 1½ inches or less, with no more than 1-inch for either the static or seismic component of the total settlement. This total settlement is preliminary and this criteria should be confirmed collaboratively with the structural engineer and owner.

8.2.2 Mat Modulus of Soil Subgrade Reaction

We recommend using a variable modulus of subgrade reaction to provide a more accurate soil response and prediction of shears and moments in the mat. This will require at least one iteration between our soil model and the structural SAFE (or similar) analysis for the mat. As the mat foundation will be underlain by ground improvement, the modulus of subgrade reaction will be affected by the ground improvement method and the bearing pressures across the mat. Once ground improvement design has been confirmed and initial bearing pressures determined, please forward the contact pressures plan for the mat (to scale and in color).

8.2.3 Lateral Loading

Lateral loads may be resisted by friction between the bottom of mat foundation and the supporting subgrade, and also by passive pressures generated against deepened mat edges. An ultimate frictional resistance of 0.45 applied to the mat dead load, and an ultimate passive pressure based on an equivalent fluid pressure of 450 pcf may be used in design. The structural engineer should apply an appropriate factor of safety (such as 1.5) to the ultimate values above.

8.2.4 Hydrostatic Uplift and Waterproofing

Where portions of the structures extend below the design groundwater level, including bottoms of mat foundations, they should be designed to resist potential hydrostatic uplift pressures. Retaining walls extending below design groundwater should be waterproofed and designed to resist hydrostatic pressure for the full wall height. Where portions of the walls extend above the design groundwater level, a drainage system may be added as discussed in the “Retaining Wall” section.

In addition, the portions of the structures extending below design groundwater should be waterproofed to limit moisture infiltration, including mat foundation areas, all construction joints, and any retaining walls. We recommend that a waterproof specialist design the waterproofing system.

8.2.5 Mat Foundation Construction Considerations

Prior to placement of any waterproofing and mat construction, the subgrade should be proof-rolled and visually observed by a Cornerstone representative to confirm stable subgrade conditions. As the planned basement excavation will extend below the current groundwater level, we recommend that the contractor plan for stabilization of the excavation bottom to provide a working platform upon which to construct the foundation. This may include excavating an additional 12 to 18 inches below subgrade, placing a layer of stabilization fabric (Mirifi R5880i or approved equivalent) at the bottom, and backfilling with clean, crushed rock. The crushed rock should be consolidated in place with vibratory equipment. Rubber tired and heavy track equipment should not be allowed to operate on the exposed subgrade; the crushed rock should be stockpiled and pushed out over the stabilization fabric. Because of the water table, we anticipate that chemically treating the bottom with lime treatment may not be feasible due to the concern of additional water inflow during the time frame needed for the mixing, curing and compaction. The pad moisture should also be checked at least 24 hours prior to vapor barrier or mat reinforcement placement to confirm that the soil has a moisture content of at least 1 percent over optimum in the upper 12 inches.

8.3 GROUND IMPROVEMENT

8.3.1 Ground Improvement Requirements

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 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 the potentially liquefiable sands by laterally displacing and/or densifying the existing in-place soils. The degree to which the density is increased will depend on the improvement method and spacing. Ground improvement should 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 the soil as well as the vibrations from consolidating and expanding the gravel column laterally. One of the disadvantages of these densification pile types are the noise and vibration (and sometimes dust) produced during construction. The vibrations 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 and 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, 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 sampling and loading methods.

Based on the chosen ground improvement technique, the upper 1 to 2 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.

8.3.2 Ground Improvement Design Guidelines

The ground improvement columns will extend from building subgrade to near the bottom of the potentially liquefiable layers as necessary to meet the design criteria, estimated to be as deep as 30 to 35 feet below existing grades. The ground improvement design should reduce the total (static plus seismic) settlement to 1½ inches or less, with no more than 1-inch of static nor 1-inch of seismic settlement allowed as a component of the total settlement. This total settlement is preliminary and this criteria should be confirmed collaboratively with the structural engineer and owner.

We anticipate a ground improvement element spacing of about 4 to 6 feet on center beneath foundations to meet the performance criteria given above. Due to the variability and uncertainty of ground conditions, we recommend that ground improvement element spacing not exceed 6 feet in foundation areas. We anticipate a tighter spacing will likely be required for the CLSM column methodology, as vibratory consolidation of sandy soils is typically more effective laterally at densification than non-vibratory displacement column construction.

Research indicates that pore pressure migration can affect even improved areas, and it is common to continue densification improvement to a distance outside of the building area. For that reason, ground improvement should be designed to provide adequate confinement around all foundations at the perimeter of the structure (at least one row of columns beyond the foundation limits) in addition to the foundation elements.

We recommend that the ground improvement design include, but not be limited to: 1) drawings showing the ground improvement layout, spacing and diameter, 2) the foundation layout plan, 3) proposed ground improvement length, 4) top and bottom elevations. We should be retained to review the ground improvement contractor's plan and settlement estimates prior to construction, and to review and confirm that the contractor's ground improvement design will satisfactorily meet the design criteria based on the performance testing. Following the completion of the Ground Improvement Performance Testing indicated below, a final ground improvement design report and calculation package, including support for the ground improvement design and indicating that the design criteria will be met, should be submitted to the design team for review and approval.

Ground improvement would generally be constructed as follows: 1) clear the site of existing demolition debris, 2) mass grading to the building pad subgrade elevation, 3) install performance test arrays to confirm the design spacing achieves the densification requirements, verified by CPT testing and additional liquefaction analyses, 4) install the ground improvement on the approved layout, and 5) re-compact top of building pad, as required, prior to construction of remainder of pad and the foundations.

8.3.3 Ground Improvement Performance Testing

On a preliminary basis, foundation areas must meet the above total settlement criteria, which will include all settlement estimated from static loads and seismic shaking. Analysis of settlement for static loading should include compression within the treatment area due to structural loads, and long-term consolidation estimated for below the zone of treatment. Analysis of settlement for seismic loading should include settlement due to liquefaction strain, as well as any dry sand settlement. Ground improvement must also provide adequate support for the design bearing capacity.

Performance testing typically consists of a pre-construction test section to confirm design spacing with post-installation CPT testing to confirm that suitable ground improvement has occurred to meet the design criteria. If the design criteria have not been met, then additional testing may be required. Verification testing involves carrying out pre- and post-array penetration testing of the soil equidistant between treatment points for the analysis of liquefaction, and comparison with measurements before treatment. We recommend that liquefaction analysis methods used include the methods proposed by Idriss and Boulanger (2014). Because of detrimental effects of pore pressure on the results of testing, we recommend that testing of ground improvement test arrays occur no sooner than two weeks after their installation. This should be incorporated into project planning, as well as the possibility that additional arrays and testing may be required if proposed spacing is inadequate.

Verification testing also includes the performance of a modulus test at each array location. To validate the parameters selected for a specific project, a modulus load test is performed on a test pier typically constructed in locations chosen in coordination with the geotechnical engineer.

Modulus tests are conducted to a pressure equal to at least 150% of the maximum design top of pier stress to assure a reasonable level of safety which supports long term settlement control and demonstrates that the ground improvement element has adequate strength. Performing modulus testing beyond the limit state top of pier stress meets the intent of the building code with respect to shallow foundation support. Modulus testing should be performed in general accordance with ASTM D1143.

For the proposed residential mixed-use building at the site, we recommend that at least two test arrays including liquefaction and modulus testing be performed.

We should observe and monitor installation of the test arrays and production ground improvement on a full-time basis and review the post-test array settlement analyses provided by the contractor.

SECTION 9: PEDESTRIAN PAVEMENTS

9.1 EXTERIOR FLATWORK

9.1.1 Pedestrian Concrete Flatwork

Exterior concrete flatwork subject to pedestrian and/or occasional light pick up loading should be at least 4 inches thick and supported on at least 6 inches of non-expansive fill overlying subgrade prepared in accordance with the “Earthwork” recommendations of this report. The upper 4 inches of NEF should also meet Class 2 aggregate base requirements. 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. To help reduce the potential for uncontrolled shrinkage cracking, 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. Flatwork should be isolated from adjacent foundations or retaining walls except where limited sections of structural slabs are included to help span irregularities in retaining wall backfill at the transitions between at-grade and on-structure flatwork.

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 judgment considering the variable surface conditions.

Table 8: Asphalt Concrete Pavement Recommendations, Design R-value = 5

Design Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base* (inches)	Total Pavement Section Thickness (inches)
4.0	2.5	7.5	10.0
4.5	2.5	9.5	12.0
5.0	3.0	10.0	13.0
5.5	3.0	12.0	15.0
6.0	3.5	12.5	16.0
6.5	4.0	14.0	18.0

*Caltrans Class 2 aggregate base; minimum R-value of 78

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 exterior Portland Cement Concrete (PCC) pavement recommendations tabulated below are based on methods presented in the Portland Cement Association (PCA) design manual (PCA, 1984). Recommendations for garage slabs-on-grade were provided in the “Concrete Slabs and Pedestrian Pavements” section above. We have provided a few pavement alternatives as an anticipated Average Daily Truck Traffic (ADTT) was not provided. An allowable ADTT should be chosen that is greater than what is expected for the development.

Table 9: PCC Pavement Recommendations, Design R-value = 5

Allowable ADTT	Minimum PCC Thickness (inches)
13	5.5
130	6.0

The PCC thicknesses above are based on a concrete compressive strength of at least 3,500 psi, supporting the PCC on at least 6 inches of Class 2 aggregate base compacted as recommended in the "Earthwork" section, 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 7 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 10: 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

Basement walls should be designed as restrained walls. 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 Basement Walls

The 2019 California Building Code (CBC) states that lateral pressures from earthquakes should be considered in the design of basements and retaining walls. We developed seismic earth pressures for the proposed basement using interim recommendations generally based on refinement of the Mononobe-Okabe method (Lew et al., SEAOC 2010). Because the walls are greater than 12 feet in height, and peak ground accelerations are greater than 0.40g, we checked the result of the total seismic increment when added to the recommended active earth pressure against the recommended fixed (restrained) wall earth pressures. Because the wall is restrained, or will act as a restrained wall, and will be designed for 45 pcf (equivalent fluid pressure) plus a uniform earth pressure of 8H psf, based on current recommendations for seismic earth pressures, it appears that active earth pressures plus a seismic increment do not exceed the fixed wall earth pressures. Therefore, an additional seismic increment above the design earth pressures is not required as long as the walls are designed for the restrained wall earth pressures recommended above in accordance with the CBC.

11.2.2 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.

11.3.2 Below-Grade Walls

Miradrain, AmerDrain or other equivalent drainage matting should be used for wall drainage where below-grade walls are temporarily shored and the shoring will be flush with the back of the permanent walls. The drainage panel should be connected at the base of the wall by a horizontal drainage strip and 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. In addition, where drainage panels will connect from a horizontal application for plaza areas to vertical basement wall drainage panels, the drainage path must be maintained. We are not aware of manufactured corner protection suitable for this situation; therefore, we recommend that a section of crushed rock be placed at the transitions. The crushed rock should be at least 3 inches thick, extend at least 12 inches horizontally over the top of the basement roof and 12 inches down from the top of the basement wall, and have a layer of filter fabric covering the crushed rock

Drainage panels should terminate 18 to 24 inches from final exterior grade unless capped by hardscape. The drainage panel filter fabric should be extended over the top of and behind the panel to protect it from intrusion of the adjacent soil. If the shoring system will be offset behind the back of permanent wall, the drainage systems discussed in the "At-Grade Site Walls" section may also be used.

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.

As discussed previously, consideration should be given to the transitions from on-grade to on-structure. Providing subslabs or other methods for reducing differential movement of flatwork or pavements across this transition should be included in the project design.

11.5 AT-GRADE SITE RETAINING WALL FOUNDATIONS

Minor at-grade site retaining walls (less than 6 feet in height) may be supported on a continuous spread footing. Spread footings should bear on natural, undisturbed soil or entirely on engineered fill, and extend at least 18 inches below the lowest adjacent grade. Footings constructed to the above dimensions and in accordance with the "Earthwork" recommendations of this report are capable of supporting a maximum allowable bearing pressure of 2,000 psf for combined dead plus live loads. This pressure is based on a factor of safety of 2.0 applied to the ultimate bearing pressure for dead plus live. This pressure is a net value; 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 Salvatore Caruso Design Corporation specifically to support the design of the Ocean Place project in Santa Cruz, 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.

Salvatore Caruso Design Corporation may have provided Cornerstone with plans, reports and other documents prepared by others. Salvatore Caruso Design Corporation 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.

SECTION 13: REFERENCES

Aagaard, B.T., Blair, J.L., Boatwright, J., Garcia, S.H., Harris, R.A., Michael, A.J., Schwartz, D.P., and DiLeo, J.S., 2016, Earthquake outlook for the San Francisco Bay region 2014–2043 (ver. 1.1, August 2016): U.S. Geological Survey Fact Sheet 2016–3020, 6 p., http://dx.doi.org/10.3133/fs20163020.

"ATC Hazards". *Hazards.Atcouncil.Org*, 2019, https://hazards.atcouncil.org/#/.

Boulanger, R.W. and Idriss, I.M., 2004, Evaluating the Potential for Liquefaction or Cyclic Failure of Silts and Clays, Department of Civil & Environmental Engineering, College of Engineering, University of California at Davis.

Boulanger, R.W. and Idriss, I.M., 2014, CPT and SPT Based Liquefaction Triggering Procedures, Department of Civil & Environmental Engineering, College of Engineering, University of California at Davis, Report No. UCD/GCM-14/01, April 2014

California Building Code, 2019, Structural Engineering Design Provisions, Vol. 2.

California Department of Conservation Division of Mines and Geology, 1998, Maps of Known Active Fault Near-Source Zones in California and Adjacent Portions of Nevada, International Conference of Building Officials, February, 1998.

California Division of Mines and Geology (2008), "Guidelines for Evaluating and Mitigating Seismic Hazards in California, Special Publication 117A, September.
Federal Emergency Management Administration (FEMA), 2012, FIRM City of Santa Cruz, California, Community Panel #0603550332E.

Idriss, I.M., and Boulanger, R.W., 2008, Soil Liquefaction During Earthquakes, Earthquake Engineering Research Institute, Oakland, CA, 237 p.

Ishihara, K., 1985, Stability of Natural Deposits During Earthquakes: Proceedings Eleventh International Conference on Soil Mechanics and Foundation Engineering, San Francisco.

Ishihara, K. and Yoshimine, M., 1992, Evaluation of Settlements in Sand Deposits Following Liquefaction During Earthquakes, Soils and Foundations, 32 (1): 173-188.

Lew, M. et al, 2010, Seismic Earth Pressures on Deep Building Basements, Proceedings, SEAOC Convention, Indian Wells, CA.

Portland Cement Association, 1984, Thickness Design for Concrete Highway and Street Pavements: report.

Robertson, P.K., Shao, Lisheng, 2010, Estimation of Seismic Compression in Dry Soils Using the CPT, 5th International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, Paper No. 4.05a, May 24-29, 2010.

Schwartz, D.P. 1994, New Knowledge of Northern California Earthquake Potential: in Proceedings of Seminar on New Developments in Earthquake Ground Motion Estimation and Implications for Engineering Design Practice, Applied Technology Council 35-1.

Seed, H.B. and I.M. Idriss, 1971, A Simplified Procedure for Evaluation soil Liquefaction Potential: JSMFC, ASCE, Vol. 97, No. SM 9, pp. 1249 – 1274.
Seed, H.B. and I.M. Idriss, 1982, Ground Motions and Soil Liquefaction During Earthquakes: Earthquake Engineering Research Institute.

Seed, Raymond B., Cetin, K.O., Moss, R.E.S., Kammerer, Ann Marie, Wu, J., Pestana, J.M., Riemer, M.F., Sancio, R.B., Bray, Jonathan D., Kayen, Robert E., and Faris, A., 2003, Recent Advances in Soil Liquefaction Engineering: A Unified and Consistent Framework., University of California, Earthquake Engineering Research Center Report 2003-06.

Southern California Earthquake Center (SCEC), 1999, Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction Hazards in California, March.

State of California Department of Transportation, 2015, Highway Design Manual, Fifth Edition, December 31, 2015.

Tokimatsu, K., and Seed, H. Bolton, 1987, Evaluation of Settlements in Sands due to Earthquake Shaking, ASCE Journal of Geotechnical Engineering, Vol. 113, August 1987, pp. 861-878.

Townley, S.D. and M.W. Allen, 1939, Descriptive Catalog of Earthquakes of the Pacific Coast of the United States, 1769 to 1928: Bulletin of the Seismological Society of America, Vol. 29, No. 1, pp. 1247-1255.

Working Group on California Earthquake Probabilities, 2008, Uniform Earthquake Rupture Forecast, Version 2 (UCERF 2), U.S. Geological Survey Open File Report 2007-1437 (CGS Special Report 203; SCEC Contribution #1138).

Working Group on California Earthquake Probabilities, 2015, The Third Uniform California Earthquake Rupture Forecast, Version 3 (UCERF), U.S. Geological Survey Open File Report 2013-1165 (CGS Special Report 228). KMZ files available at: www.scec.org/ucerf/images/ucerf3_timedep_30yr_probs.kmz.

Yoshimine, M., Nishizaki, H., Amano, KI, and Hosono, Y., 2006, Flow Deformation of Liquefied Sand Under Constant Shear Load and Its Application to Analysis of Flow Slide in Infinite Slope, Soil Dynamics and Earthquake Eng. 26, 253-264.

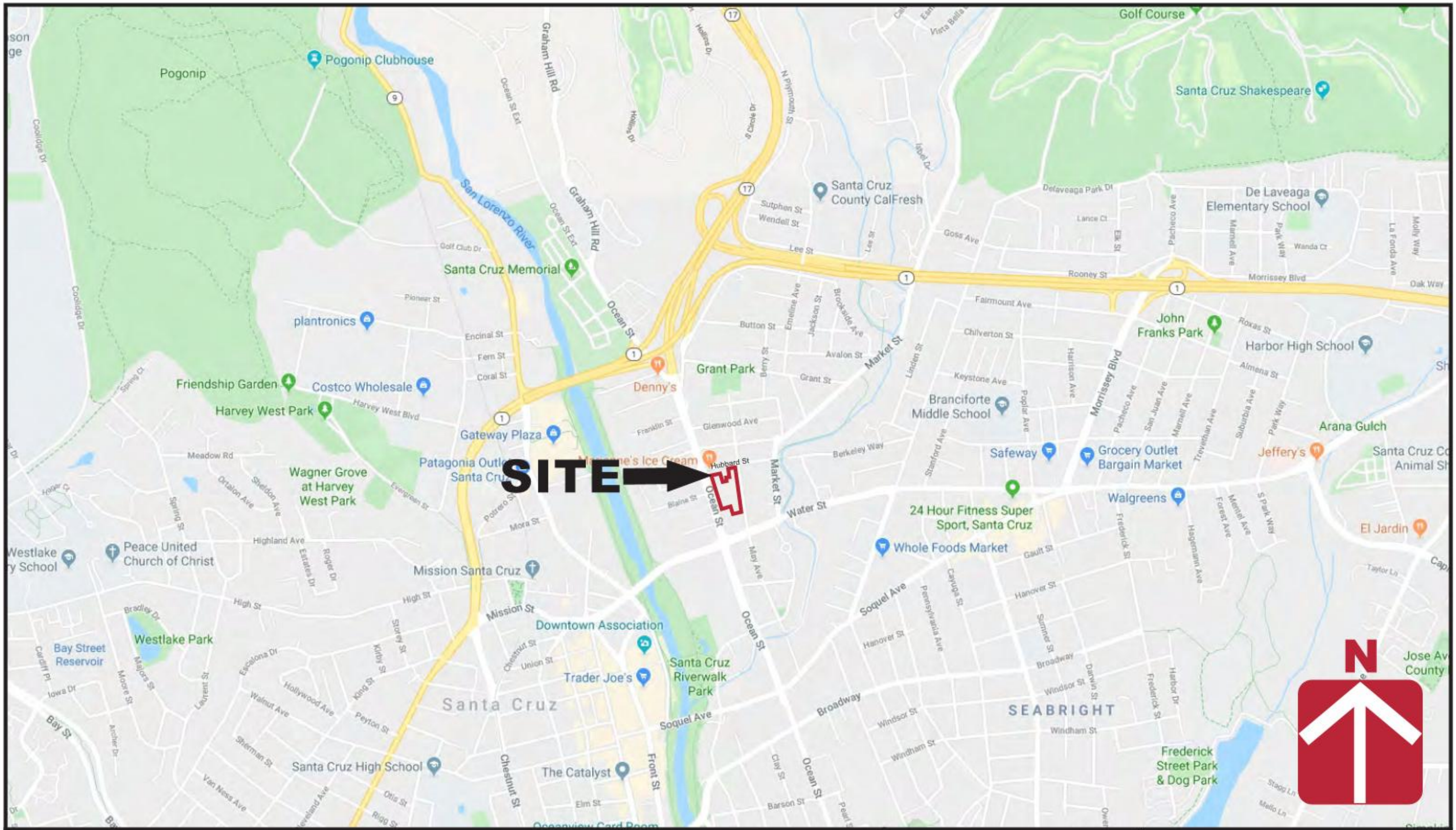
Youd, T.L. and C.T. Garris, 1995, Liquefaction-Induced Ground-Surface Disruption: Journal of Geotechnical Engineering, Vol. 121, No. 11, pp. 805 - 809.

Youd, T.L. and Idriss, I.M., et al, 1997, Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils: National Center for Earthquake Engineering Research, Technical Report NCEER - 97-0022, January 5, 6, 1996.

Youd et al., 2001, “Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils,” ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vo. 127, No. 10, October, 2001.

Youd, T. Leslie, Hansen, Corbett M., and Bartlett, Steven F., 2002, Revised Multilinear Regression Equations for Prediction of Lateral Spread Displacement: ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vol. 128, December 2002, p 1007-1017.

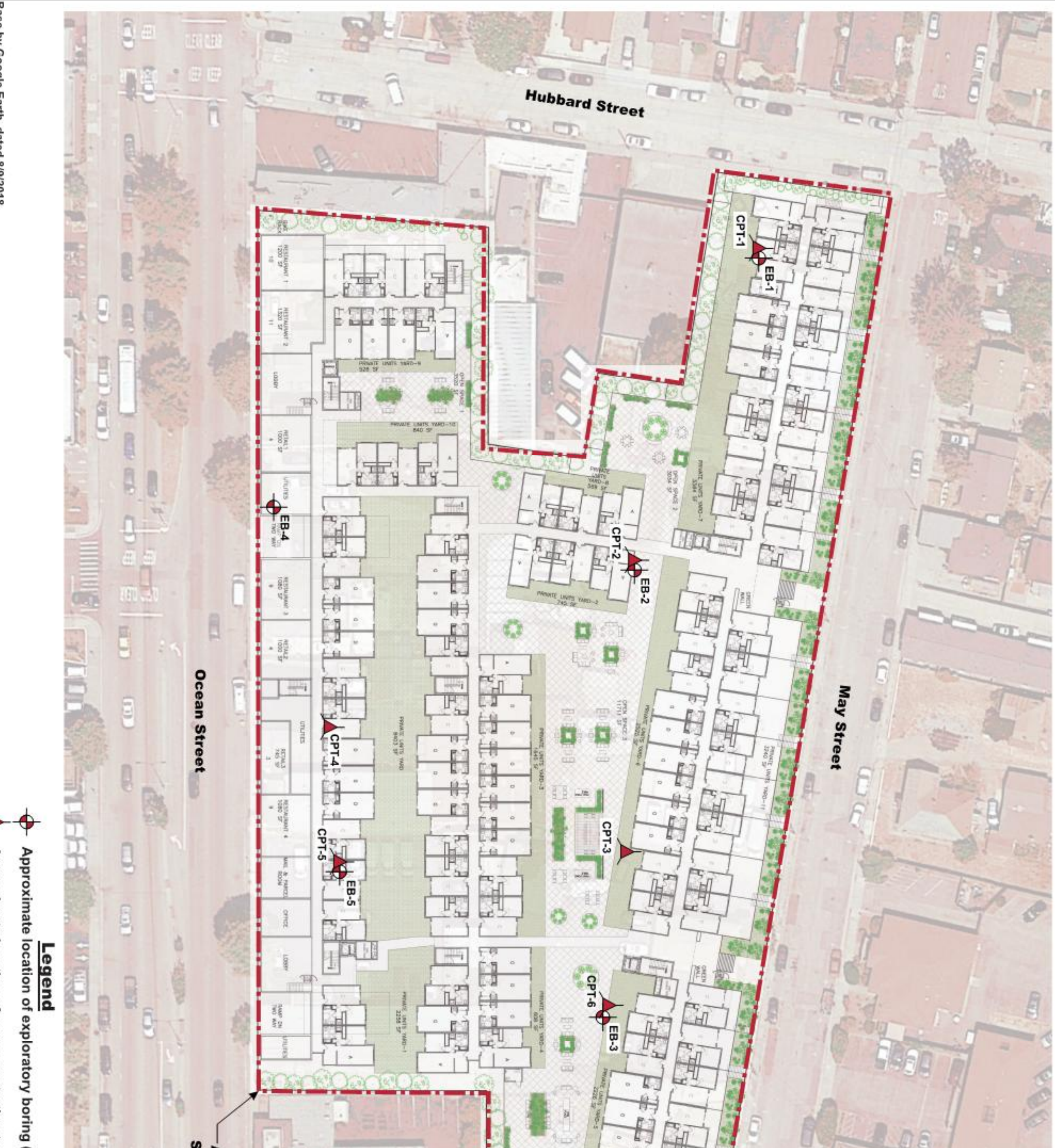
Youd, T.L. and Hoose, S.N., 1978, Historic Ground Failures in Northern California Triggered by Earthquakes, United States Geologic Survey Professional Paper 993.



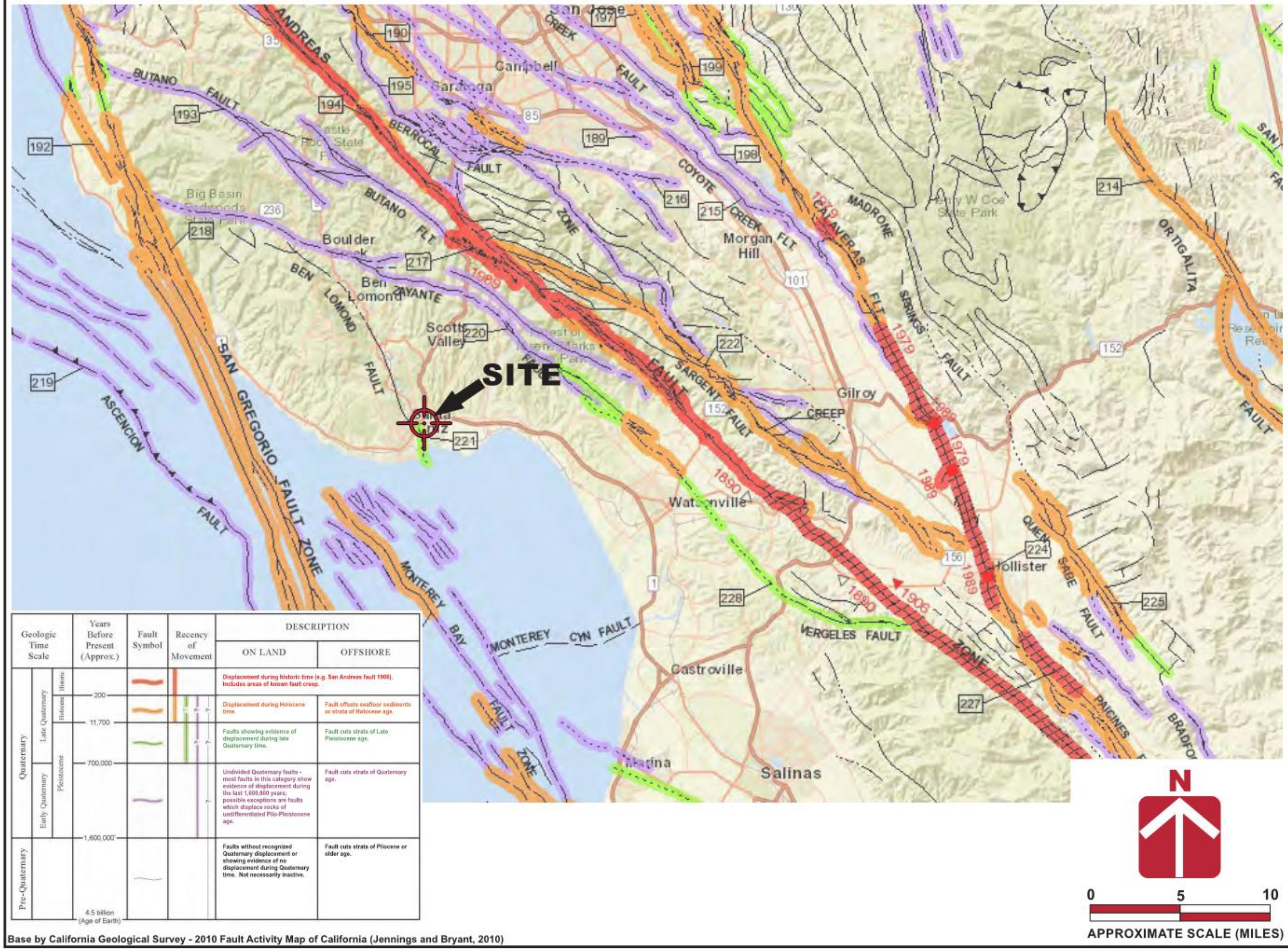
CORNERSTONE EARTH GROUP

Vicinity Map	Project Number 908-3-1
Ocean Place **Santa Cruz, CA**	Figure Number Figure 1
	Date October 2019 | Drawn By RRN

Base by Google Earth, dated 8/9/2018
Overlay by SCDC Architecture, First Floor Plan - A1.2, dated 10/3/2018



CORNERSTONE EARTH GROUP	Site Plan	Project Number 908-3-1
	Ocean Place Santa Cruz, CA	Figure Number Figure 2
		Date October 2019 Drawn By RRN



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

CORNERSTONE EARTH GROUP

Regional Fault Map

Ocean Place
Santa Cruz, CA

Project Number 908-3-1

Figure Number Figure 3

Date October 2019

Drawn By RRN

CORNERSTONE EARTH GROUP

FIGURE 4A

CPT NO. 1

PROJECT/CPT DATA

Project Title **Ocean Place**
Project No. **908-3-1**
Project Manager **MFR**

SEISMIC PARAMETERS

Controlling Fault **San Andreas**
Earthquake Magnitude (Mw) **7.9**
PGA (Amax) **0.62** (g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet) **5.2**
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**
7.18 **(Inches)**

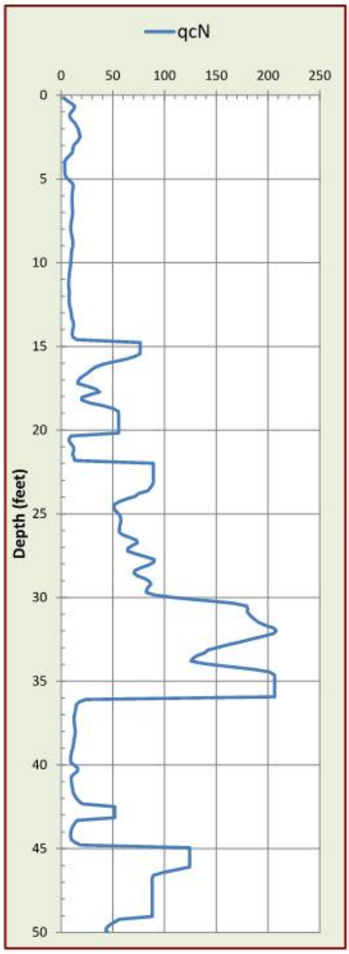
TOTAL SEISMIC SETTLEMENT 7.2 **INCHES**

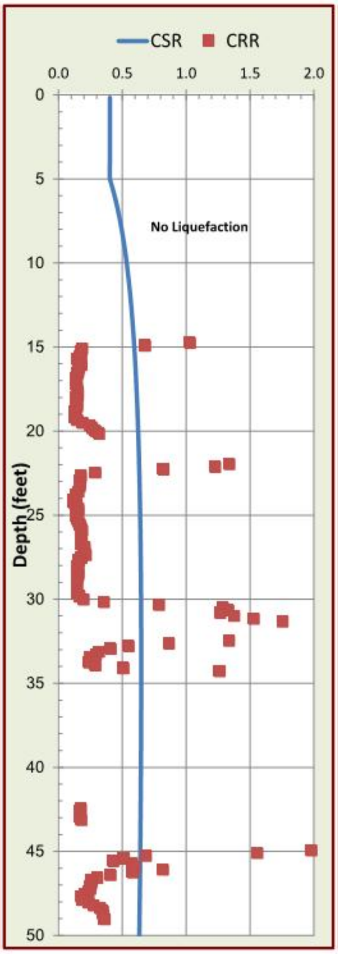
POTENTIAL LATERAL DISPLACEMENT

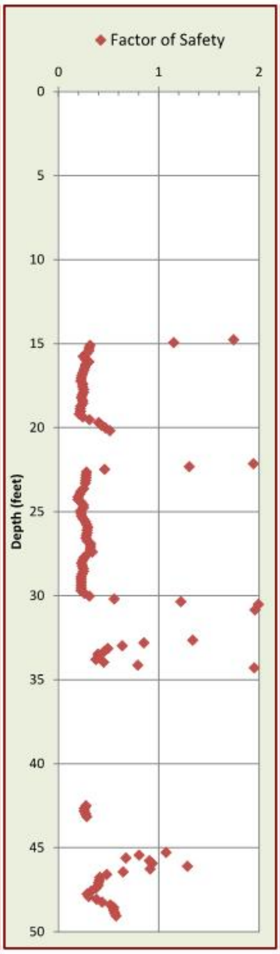
LDI[2] 5.12 **L/H** 65.0
LDI[1]**Corrected for Distance** 1.09 (4 < L/H < 40)

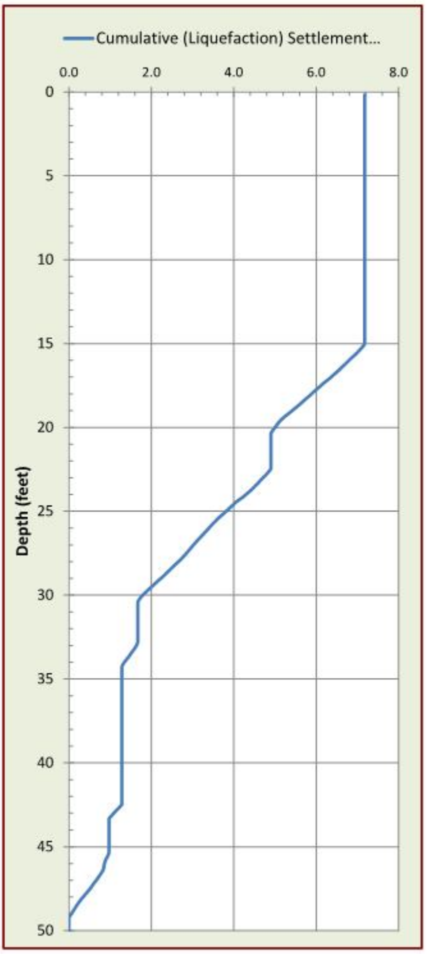
EXPECTED RANGE OF DISPLACEMENT
0.5 **to** 2.2 **feet**

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









CORNERSTONE EARTH GROUP

FIGURE 4B

CPT NO. 2

PROJECT/CPT DATA

Project Title **Ocean Place**

Project No. **908-3-1**

Project Manager **MFR**

SEISMIC PARAMETERS

Controlling Fault **San Andreas**

Earthquake Magnitude (Mw) **7.9**

PGA (Amax) **0.62** (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**

7.39 **(Inches)**

TOTAL SEISMIC SETTLEMENT 7.4 **INCHES**

POTENTIAL LATERAL DISPLACEMENT

LDI[2] 4.94 **L/H** 65.0

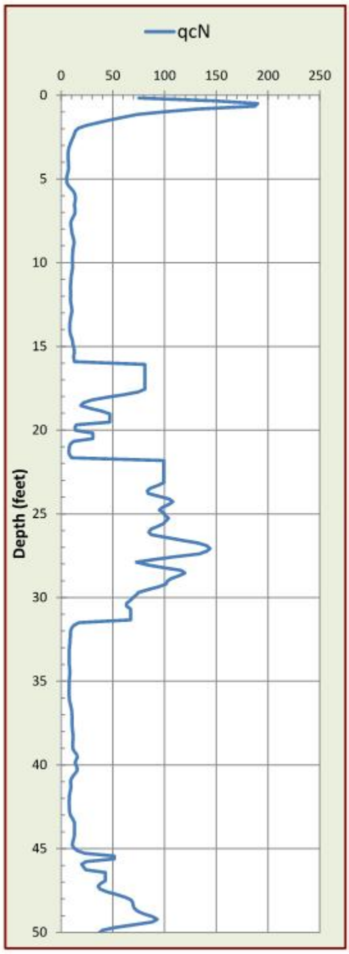
LDI[1] **Corrected for Distance** 1.05 (4 < L/H < 40)

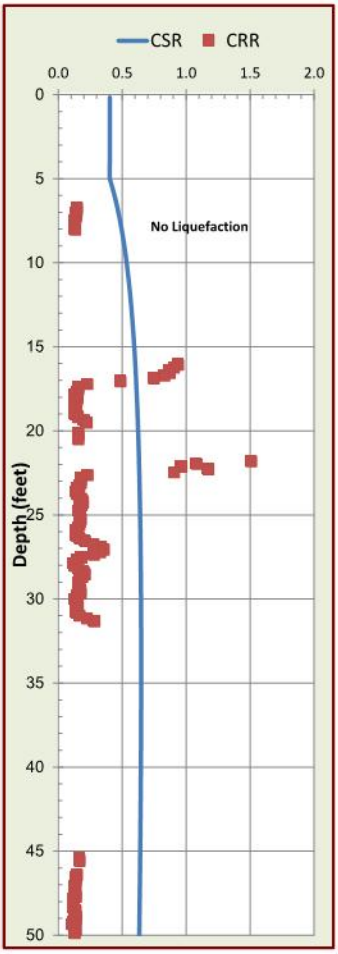
EXPECTED RANGE OF DISPLACEMENT

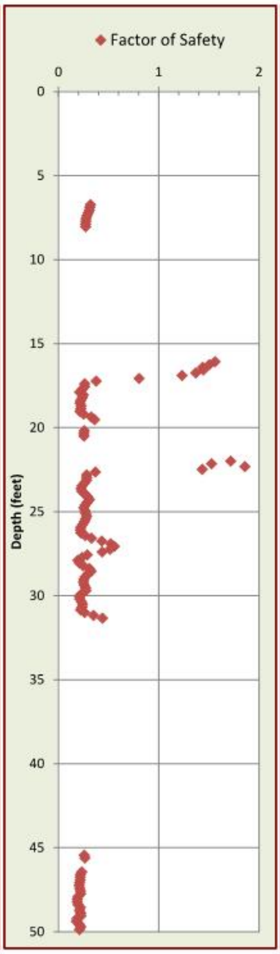
0.5 to **2.1** **feet**

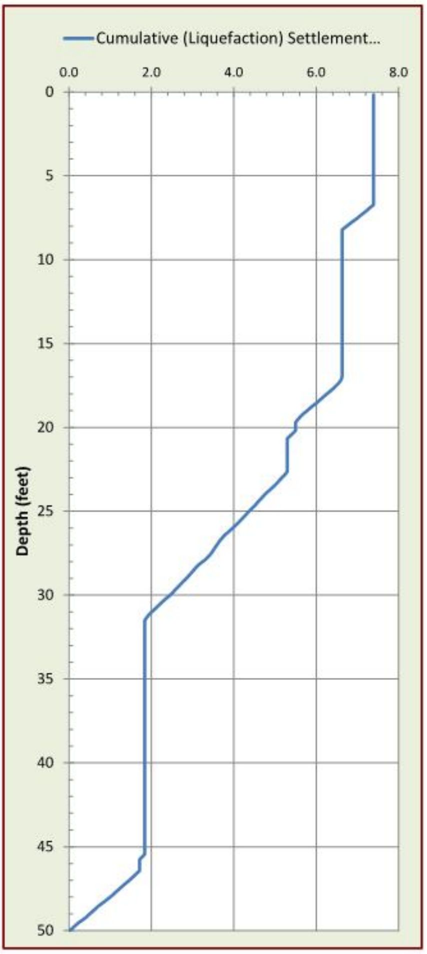
[1] Not Valid for L/H Values < 4 and > 40.

[2] LDI Values Only Summed to 2H Below Grade.









CORNERSTONE EARTH GROUP

FIGURE **4C**

CPT NO. **3**

PROJECT/CPT DATA

Project Title	**Ocean Place**
Project No.	**908-3-1**
Project Manager	**MFR**

SEISMIC PARAMETERS

Controlling Fault	**San Andreas**	
Earthquake Magnitude (Mw)	**7.9**	
PGA (Amax)	**0.62**	(g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet)	**10.5**
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**

7.94 **(Inches)**

LIQUEFACTION SETTLEMENT FROM 50 **FEET**

7.15 **(Inches)**

TOTAL SEISMIC SETTLEMENT 15.1 **INCHES**

POTENTIAL LATERAL DISPLACEMENT

LDI[2] 6.29 **L/H** 65.0

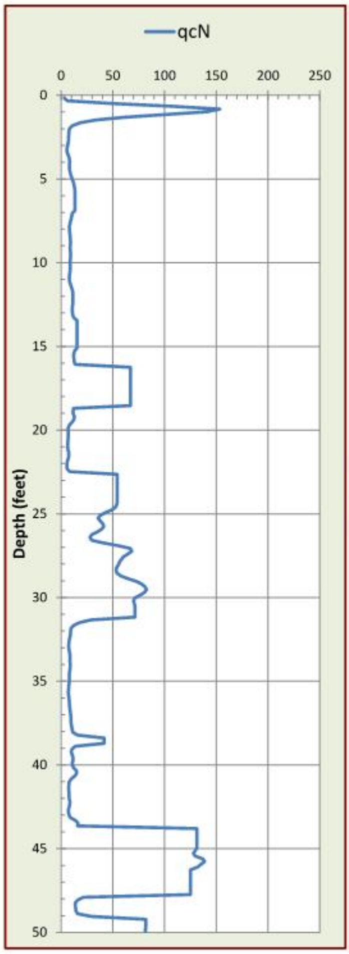
LDI[1]**Corrected for Distance** 1.34 (4 < L/H < 40)

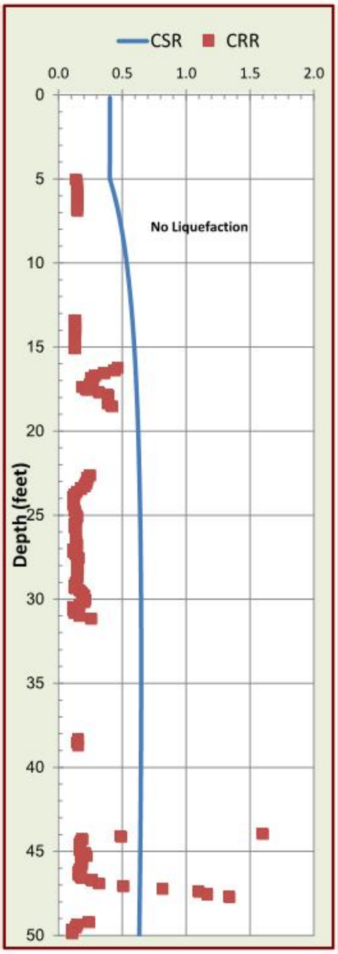
EXPECTED RANGE OF DISPLACEMENT

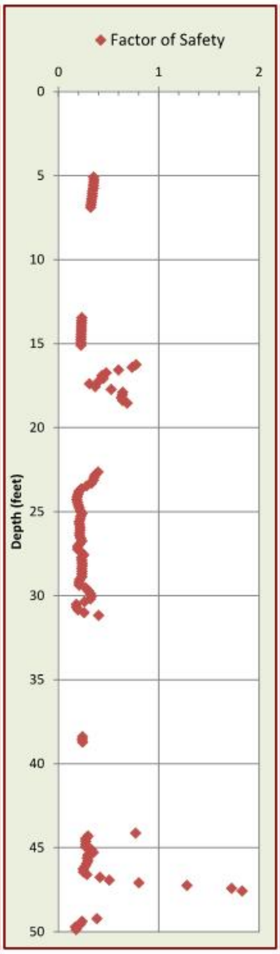
0.7 **to** **2.7** **feet**

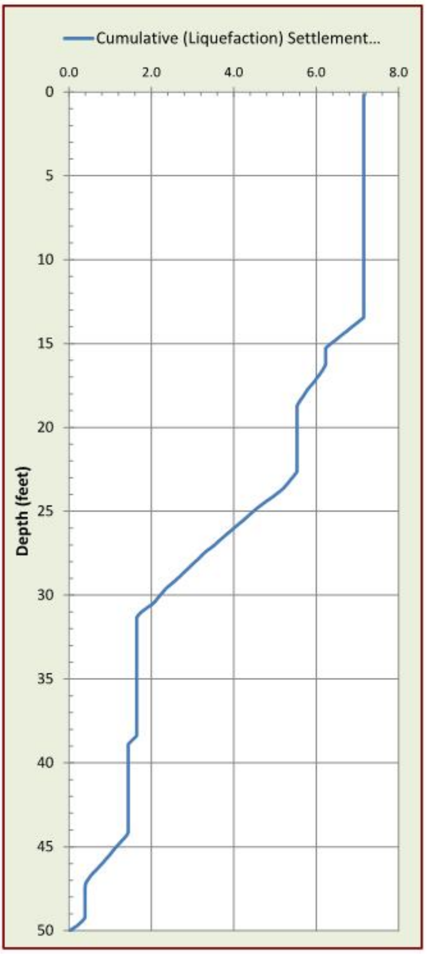
[1] Not Valid for L/H Values < 4 and > 40.

[2] LDI Values Only Summed to 2H Below Grade.









CORNERSTONE EARTH GROUP

FIGURE 4D

CPT NO. 4

PROJECT/CPT DATA

Project Title	**Ocean Place**
Project No.	**908-3-1**
Project Manager	**MFR**

SEISMIC PARAMETERS

Controlling Fault	**San Andreas**	
Earthquake Magnitude (Mw)	**7.9**	
PGA (Amax)	**0.62**	(g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet)	**14**
Design Water Depth (feet)	**5**
Ave. Unit Weight Above GW (pcf)	**121**
Ave. Unit Weight Below GW (pcf)	**121**

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 5 **FEET**

0.10 **(Inches)**

LIQUEFACTION SETTLEMENT FROM 50 **FEET**

1.02 **(Inches)**

TOTAL SEISMIC SETTLEMENT 1.1 **INCHES**

POTENTIAL LATERAL DISPLACEMENT

LDI[2] 1.15 **L/H** 55.0

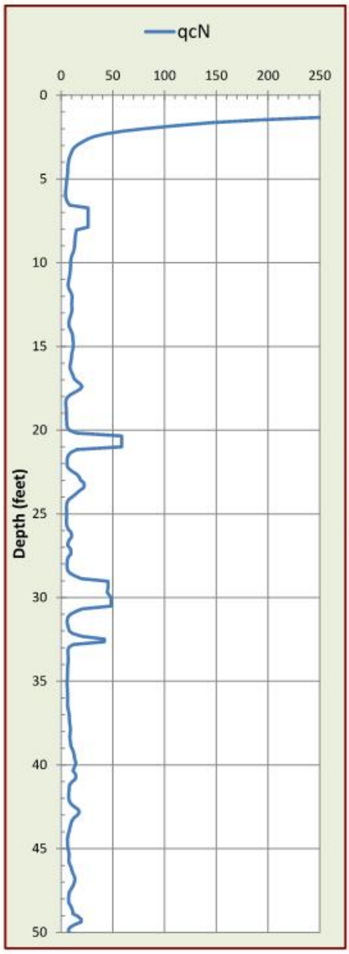
LDI[1]**Corrected for Distance** 0.28 (4 < L/H < 40)

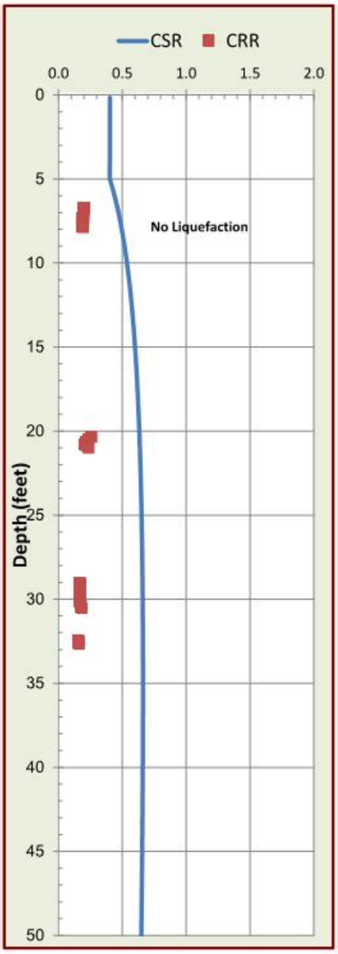
EXPECTED RANGE OF DISPLACEMENT

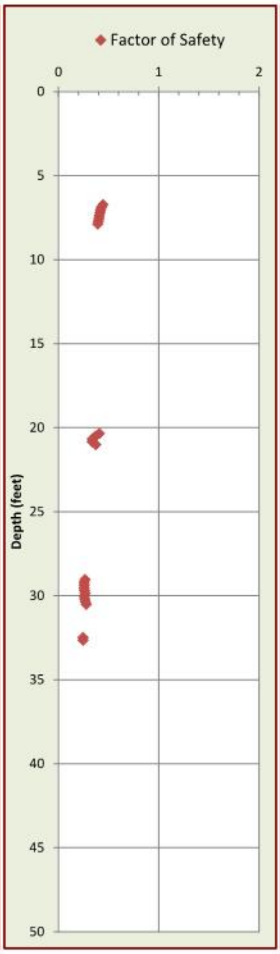
0.1 **to** **0.6** **feet**

[1] Not Valid for L/H Values < 4 and > 40.

[2] LDI Values Only Summed to 2H Below Grade.







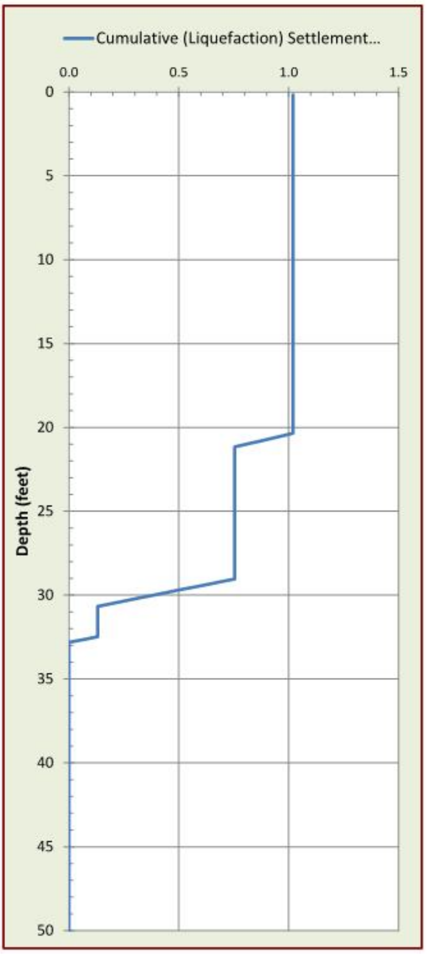


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FIGURE 4E

CPT NO. 5

PROJECT/CPT DATA

Project Title	**Ocean Place**
Project No.	**908-3-1**
Project Manager	**MFR**

SEISMIC PARAMETERS

Controlling Fault	**San Andreas**	
Earthquake Magnitude (Mw)	**7.9**	
PGA (Amax)	**0.62**	(g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet)	**6.8**
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**

0.02 **(Inches)**

LIQUEFACTION SETTLEMENT FROM 50 **FEET**

2.93 **(Inches)**

TOTAL SEISMIC SETTLEMENT 3.0 **INCHES**

POTENTIAL LATERAL DISPLACEMENT

LDI^2 2.06 **L/H** 55.0

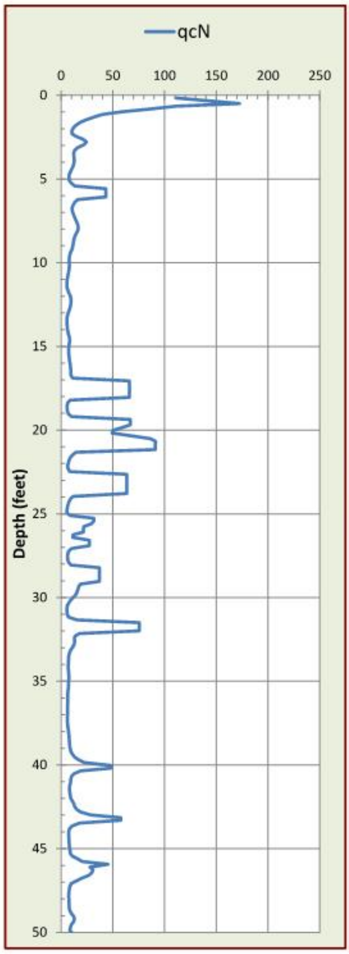
LDI^1 **Corrected for Distance** 0.50 (4 < L/H < 40)

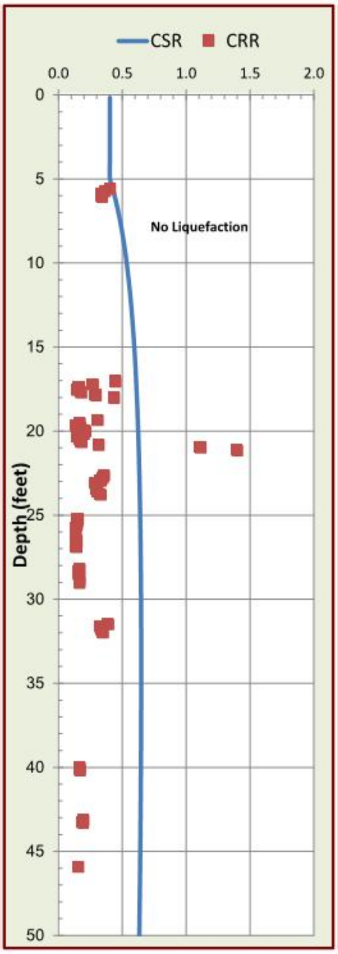
EXPECTED RANGE OF DISPLACEMENT

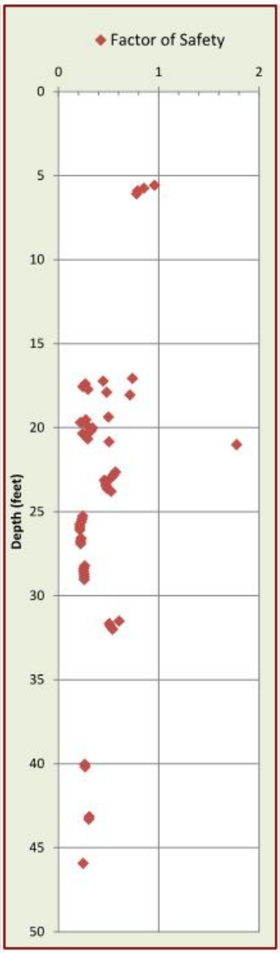
0.3 **to** **1.0** **feet**

[1] Not Valid for L/H Values < 4 and > 40.

[2] LDI Values Only Summed to 2H Below Grade.







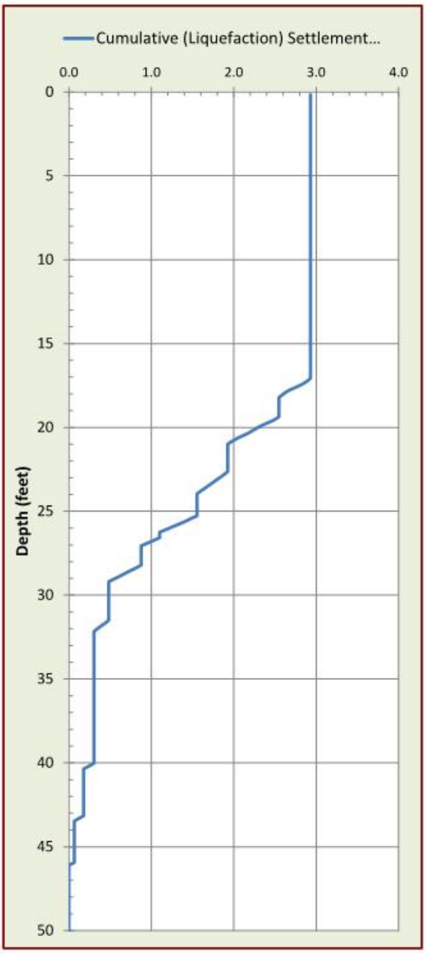


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FIGURE 4F

CPT NO. 6

PROJECT/CPT DATA

Project Title **Ocean Place**

Project No. **908-3-1**

Project Manager **MFR**

SEISMIC PARAMETERS

Controlling Fault **San Andreas**

Earthquake Magnitude (Mw) **7.9**

PGA (Amax) **0.62** (g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet) **6.9**

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**

0.13 **(Inches)**

LIQUEFACTION SETTLEMENT FROM 50 **FEET**

3.28 **(Inches)**

TOTAL SEISMIC SETTLEMENT 3.4 **INCHES**

POTENTIAL LATERAL DISPLACEMENT

LDI[2] 2.32 **L/H** 65.0

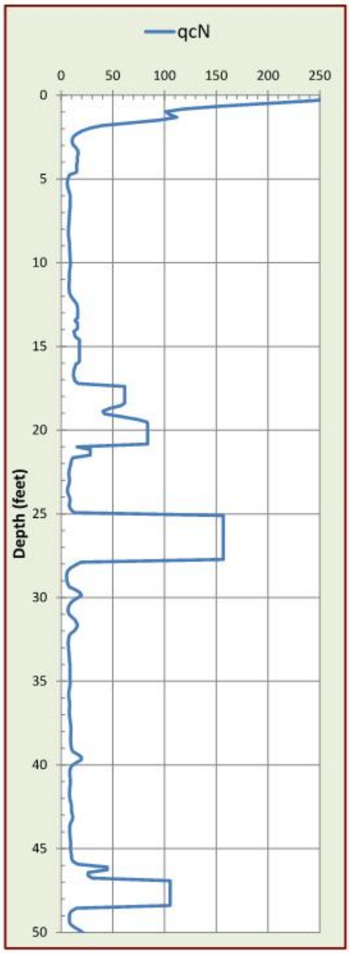
LDI[1]**Corrected for Distance** 0.49 (4 < L/H < 40)

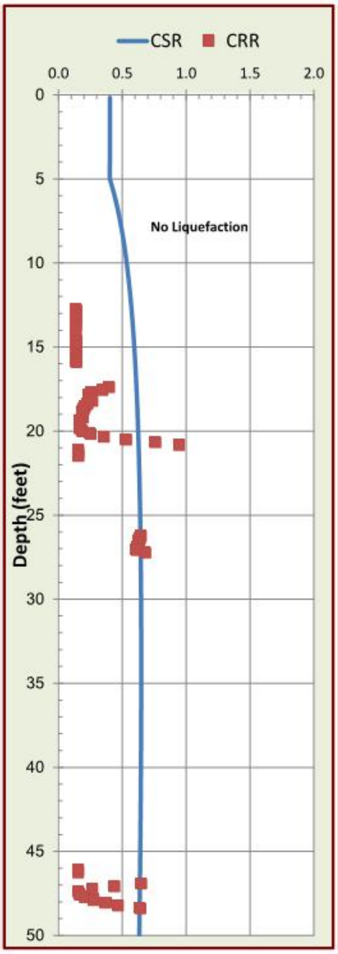
EXPECTED RANGE OF DISPLACEMENT

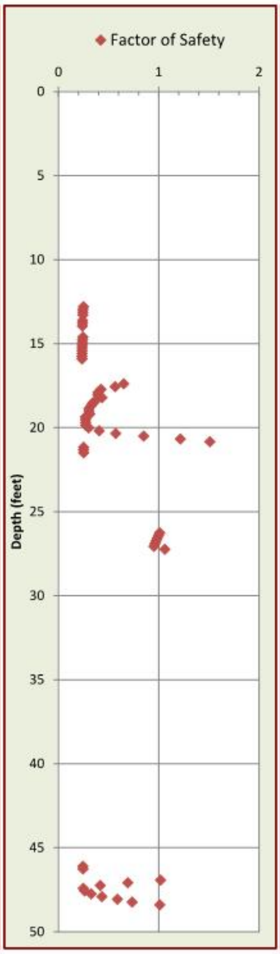
0.2 **to** **1.0** **feet**

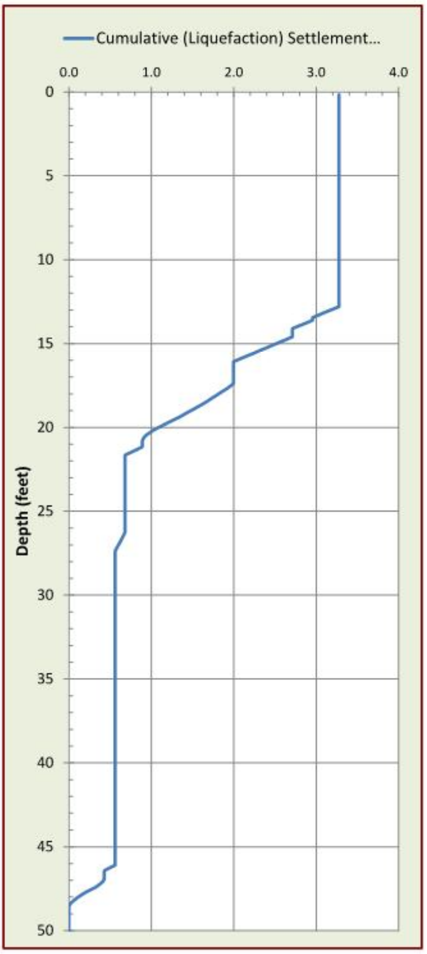
[1] Not Valid for L/H Values < 4 and > 40.

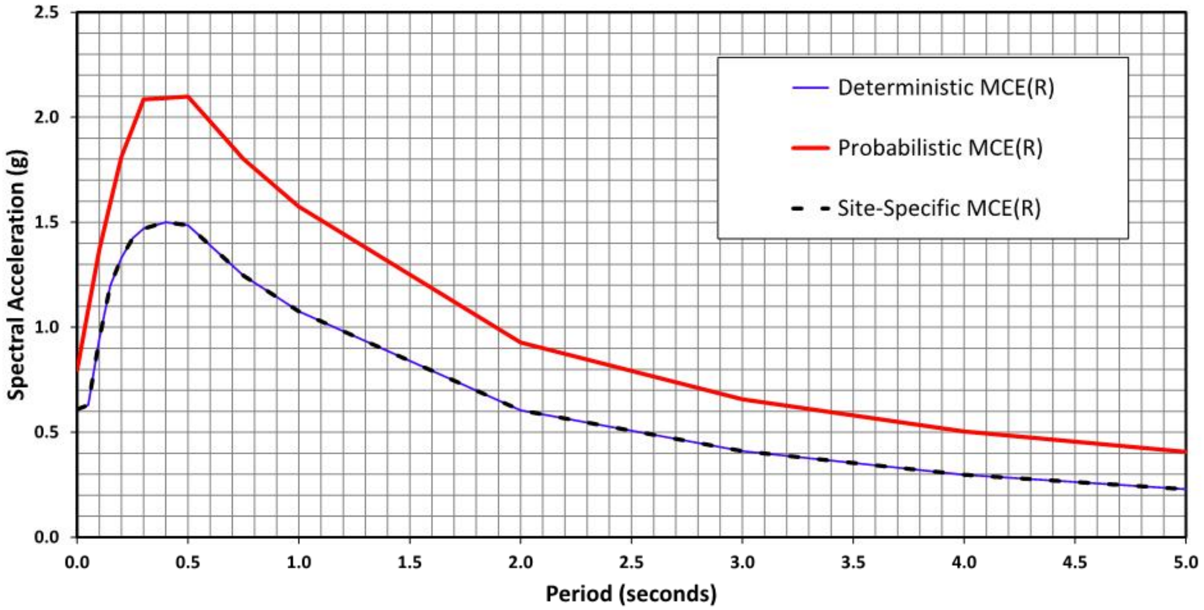
[2] LDI Values Only Summed to 2H Below Grade.











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	0.610
0.05	0.629
0.10	0.945
0.15	1.199
0.19	1.310
0.20	1.331
0.25	1.423
0.30	1.470
0.40	1.500
0.50	1.485
0.75	1.246
0.96	1.104
1.00	1.076
2.00	0.605
3.00	0.411
4.00	0.298
5.00	0.229

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

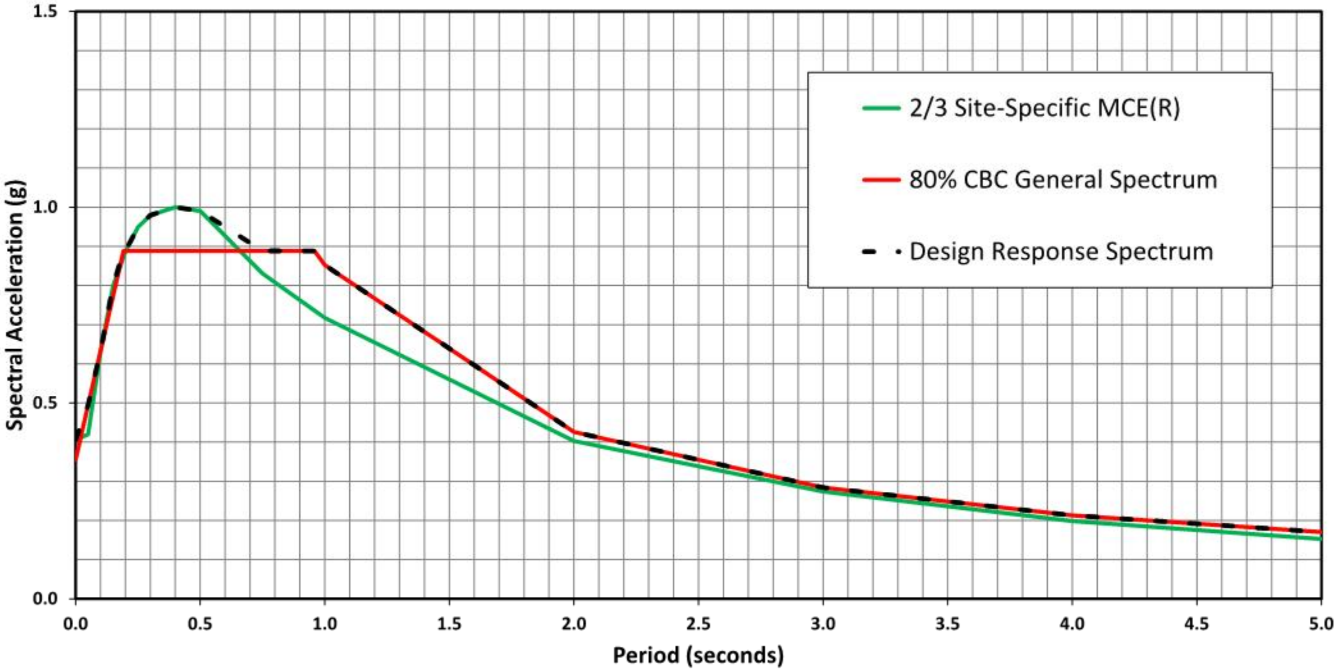
MCE_R RESPONSE SPECTRA

FIGURE 5

Ocean Place
Ocean, Hubbard, and May Street
Santa Cruz, California

PROJECT NO. 908-3-1

November 7, 2019 | RSM



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	0.406
0.05	0.494
0.10	0.633
0.15	0.799
0.19	0.889
0.20	0.889
0.25	0.949
0.30	0.980
0.40	1.000
0.50	0.990
0.75	0.889
0.96	0.888
1.00	0.852
2.00	0.426
3.00	0.284
4.00	0.213
5.00	0.170

Site Design	Design Values
Site Class (Per Chapter 20 ASCE 7-16)	D
Shear Wave Velocity, V_{S30} (m/sec)	196
Site Latitude (degrees)	36.981291
Site Longitude (degrees)	-122.021198
Risk Category	II
Building Period (sec)	Unknown
Importance Factor, I_e	1
[1]Site Specific PGA_M (g)	0.62

[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}	0.900
S_{D1}	0.852
S_{MS}	1.350
S_{M1}	1.278

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

Ocean Place
Ocean, Hubbard, and May Street
Santa Cruz, California

PROJECT NO. 908-3-1

November 7, 2019 | RSM

APPENDIX A: FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program using truck-mounted hollow-stem auger and track-mounted, hollow-stem, limited-access auger drilling equipment and 20-ton truck-mounted Cone Penetration Test equipment. Three 8-inch-diameter exploratory borings and two 6½-inch diameter exploratory borings were drilled on October 10, 14, and 15, 2019 to depths of 30 to 61½ feet. Six CPT soundings were also performed in accordance with ASTM D 5778-95 (revised, 2002) on October 4, 2019, to depths ranging from 50 to 85 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 and other site features as references. Boring and CPT elevations were based on interpolation of plan contours estimated from the provided topographic survey (based on NGVD 29). The locations and elevations 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	⁘	Sand
▨	Clayey Sand	▥	Silt
▥	Sandy Silt	⁘	Well Graded Gravelly Sand
▩	Artificial/Undocumented Fill	▥	Gravelly Silt
⁘	Poorly-Graded Gravelly Sand	■	Asphalt
⁘	Topsoil	●	Boulders and Cobble
●	Well-Graded Gravel with Clay		
●	Well-Graded Gravel with Silt		

SAMPLER TYPES

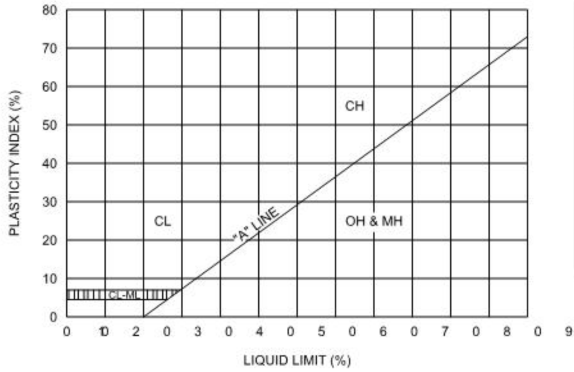
☒	SPT	■	Shelby Tube
⧗	Modified California (2.5" I.D.)	○	No Recovery
▮	Rock Core	✋	Grab Sample

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

CORNERSTONE
EARTH GROUP

BORING NUMBER EB-1

PAGE 1 OF 2

PROJECT NAME Ocean Place
PROJECT NUMBER 908-3-1
PROJECT LOCATION 908 Ocean Street, Santa Cruz, CA
DATE STARTED 10/10/19 **DATE COMPLETED** 10/10/19
GROUND ELEVATION 25 FT +/- **BORING DEPTH** 30 ft.
DRILLING CONTRACTOR Cuesta Geo
LATITUDE 36.982051° **LONGITUDE** -122.021032°
DRILLING METHOD MPP LAD Track Rig, 6½ inch Hollow-Stem Auger
GROUNDWATER LEVELS:
LOGGED BY JLC
▽ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

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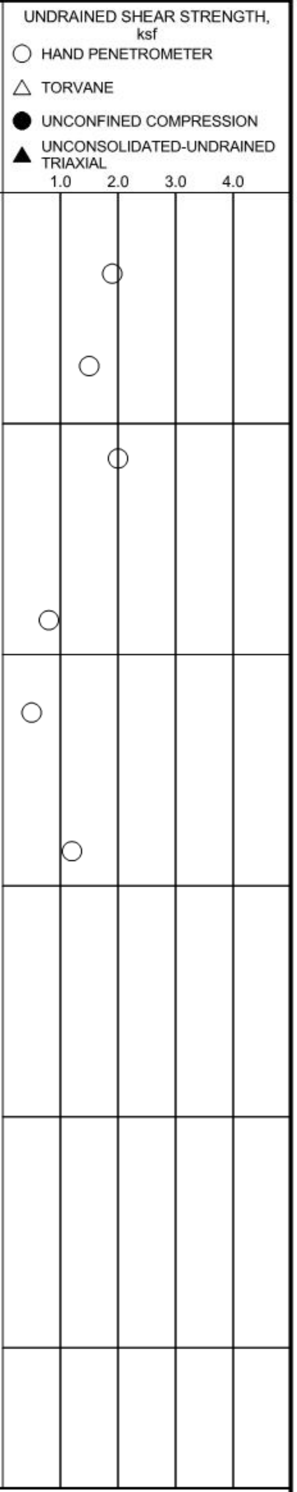
ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
25.0	0	5½ inches Portland cement concrete						
24.6		**Lean Clay (CL)** stiff, moist, dark brown, some fine sand, moderate plasticity Liquid Limit = 49, Plastic Limit = 26	11	MC-1B	77	35	23	
			10	MC-2B	81	33		
	5		11	MC-3B	87	30		
		becomes medium stiff	7	MC-4B	92	30		
	10	no recovery in shelby tube		ST				
12.0		**Lean Clay with Sand (CL)** stiff, moist, dark brown, fine sand, moderate plasticity	9	MC				
10.0	15	**Silty Sand (SM)** loose, moist, grayish brown, fine sand	8	MC-7B	96	24		
	20		5	MC				
	25	some medium to coarse sand	8	SPT-9		26		
-2.0								



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BORING NUMBER EB-1

PROJECT NAME Ocean Place
PROJECT NUMBER 908-3-1
PROJECT LOCATION 908 Ocean Street, Santa Cruz, CA

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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	UNDRAINED SHEAR STRENGTH, ksf ○ HAND PENETROMETER △ TORVANE ● UNCONFINED COMPRESSION ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL 1.0 2.0 3.0 4.0
-2.0			**Silty Sand (SM)** loose, moist, grayish brown, fine sand							
			trace fine subrounded gravel, abundant organics (wood)	8	☒ SPT-10		41			
-5.0	30		Bottom of Boring at 30.0 feet.							
	35									
	40									
	45									
	50									
	55									

CORNERSTONE
EARTH GROUP

BORING NUMBER EB-2

PAGE 1 OF 2

PROJECT NAME Ocean Place
PROJECT NUMBER 908-3-1
PROJECT LOCATION 908 Ocean Street, Santa Cruz, CA
DATE STARTED 10/10/19 **DATE COMPLETED** 10/10/19
GROUND ELEVATION 25 FT +/- **BORING DEPTH** 36 ft.
DRILLING CONTRACTOR Cuesta Geo
LATITUDE 36.981516° **LONGITUDE** -122.021094°
DRILLING METHOD MPP LAD Track Rig, 6½ inch Hollow-Stem Auger
GROUNDWATER LEVELS:
LOGGED BY JLC
▽ **AT TIME OF DRILLING** 9 ft.
NOTES
▼ **AT END OF DRILLING** 9 ft.

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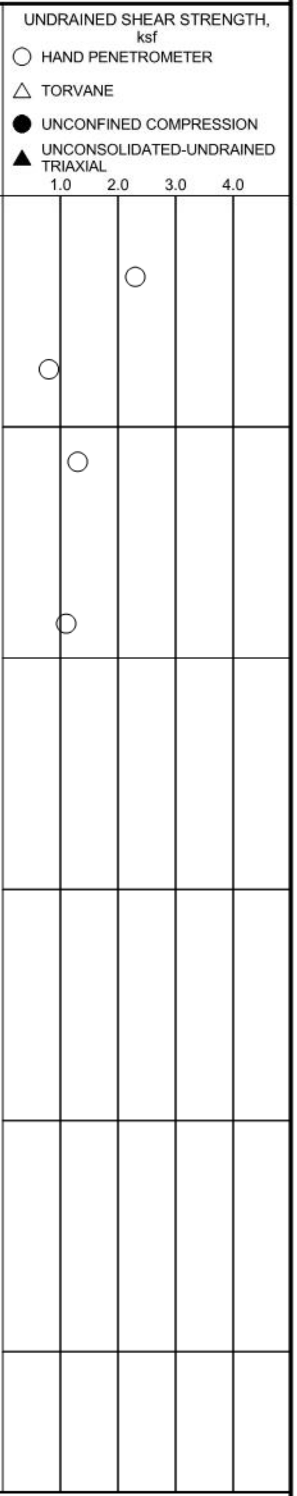
ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
25.0	0	**Lean Clay with Sand (CL) [Fill]** very stiff, moist, dark brown with brown mottles, fine to medium sand, moderate plasticity	8	MC-1B	89	25		
22.5		**Lean Clay (CL)** medium stiff, moist, dark brown, some fine sand, moderate plasticity	6	MC				
20.0	5	**Lean Clay with Sand (CL)** stiff, moist, brown, fine to medium sand, low to moderate plasticity	7	MC-3B	96	25		
18.0		**Clayey Sand (SC)** loose, moist, brown, fine sand	7	MC				
17.0		**Lean Clay (CL)** stiff, moist, dark brown, trace fine sand, moderate plasticity	8	MC-5B	90	30		
	10	no recovery in shelby tube		ST				
	15							
9.5		**Clayey Sand (SC)** loose, moist, grayish brown, fine sand	7	MC-7C	93	28		36
8.0		**Silty Sand (SM)** loose, moist, grayish brown, fine sand	3	MC-8B	93	24		
	20							
3.0		**Clayey Sand (SC)** loose, wet, dark brown, fine sand	2	SPT-9B		27		
0.5	25	**Silty Sand (SM)** loose, wet, dark brown, fine sand						
-2.0								



Continued Next Page

BORING NUMBER EB-2

PAGE 2 OF 2

CORNERSTONE EARTH GROUP

PROJECT NAME Ocean Place

PROJECT NUMBER 908-3-1

PROJECT LOCATION 908 Ocean Street, Santa Cruz, CA

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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)	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	UNDRAINED SHEAR STRENGTH, ksf
-2.0			**Silty Sand (SM)** loose, wet, dark brown, fine sand							
				10	SPT-10		27			
	30									
-6.5			**Lean Clay (CL)** medium stiff, moist, gray with brown mottles, moderate plasticity							
					ST-11	86	34			○
	35									
-11.0			Bottom of Boring at 36.0 feet.							
	40									
	45									
	50									
	55									

CORNERSTONE
EARTH GROUP

BORING NUMBER EB-3

PAGE 1 OF 2

PROJECT NAME Ocean Place
PROJECT NUMBER 908-3-1
PROJECT LOCATION 908 Ocean Street, Santa Cruz, CA
DATE STARTED 10/14/19 **DATE COMPLETED** 10/14/19
GROUND ELEVATION 25 FT +/- **BORING DEPTH** 30 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 36.980821° **LONGITUDE** -122.020876°
DRILLING METHOD Mobile B-56, 8 inch Hollow-Stem Auger
GROUNDWATER LEVELS:
LOGGED BY JLC
▽ **AT TIME OF DRILLING** 11 ft.
NOTES
▼ **AT END OF DRILLING** 10 ft.

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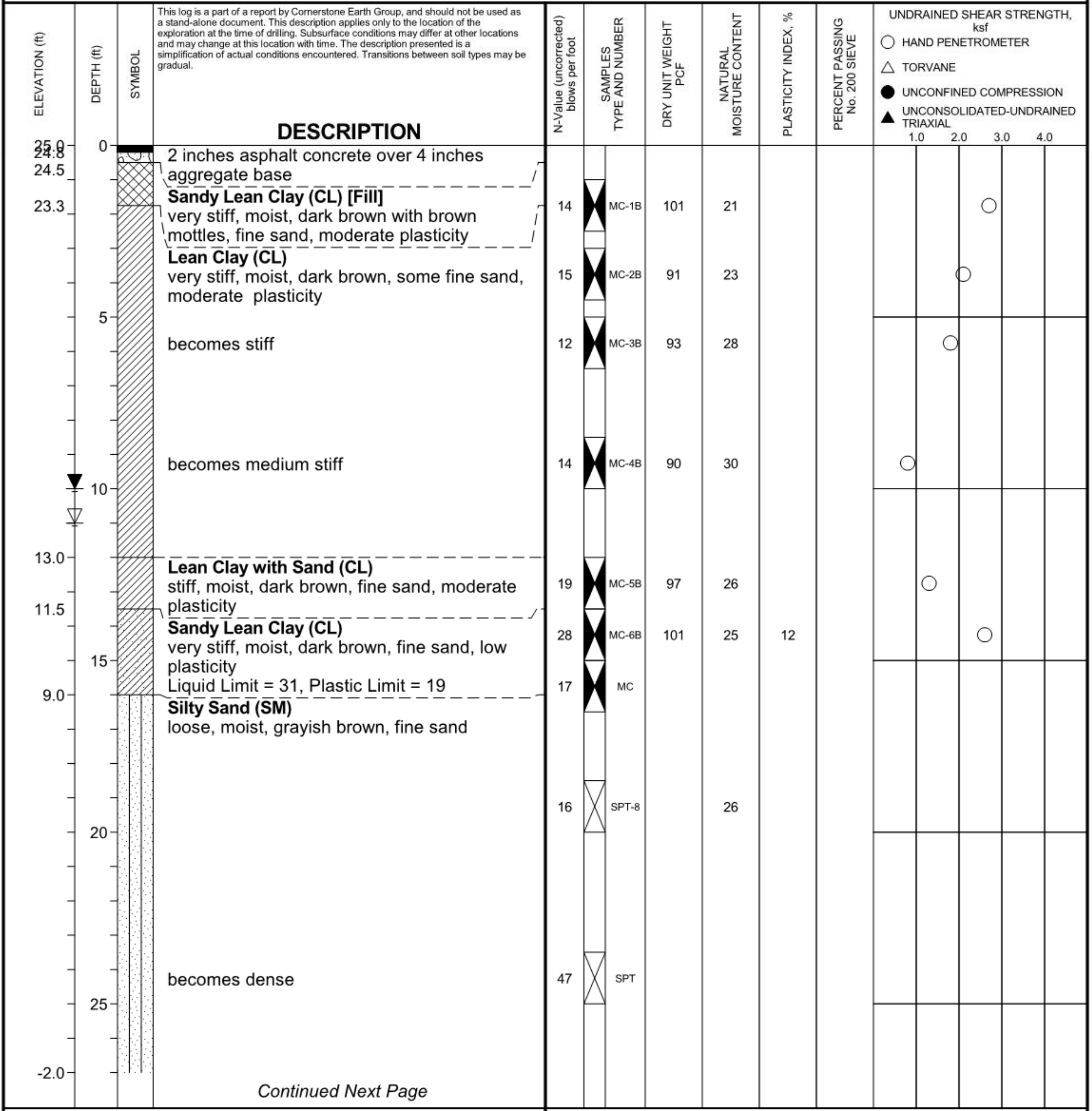

ELEVATION (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
25.0 24.8	2 inches asphalt concrete over 4 inches aggregate base						
24.5	**Sandy Lean Clay (CL) [Fill]** very stiff, moist, dark brown with brown mottles, fine sand, moderate plasticity	14	MC-1B	101	21		
23.3	**Lean Clay (CL)** very stiff, moist, dark brown, some fine sand, moderate plasticity	15	MC-2B	91	23		
	becomes stiff	12	MC-3B	93	28		
	becomes medium stiff	14	MC-4B	90	30		
13.0	**Lean Clay with Sand (CL)** stiff, moist, dark brown, fine sand, moderate plasticity	19	MC-5B	97	26		
11.5	**Sandy Lean Clay (CL)** very stiff, moist, dark brown, fine sand, low plasticity Liquid Limit = 31, Plastic Limit = 19	28	MC-6B	101	25	12	
		17	MC				
9.0	**Silty Sand (SM)** loose, moist, grayish brown, fine sand	16	SPT-8		26		
	becomes dense	47	SPT				
-2.0							

Continued Next Page

CORNERSTONE EARTH GROUP

BORING NUMBER EB-3

PAGE 2 OF 2

PROJECT NAME Ocean Place

PROJECT NUMBER 908-3-1

PROJECT LOCATION 908 Ocean Street, Santa Cruz, CA

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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	UNDRAINED SHEAR STRENGTH, ksf ○ HAND PENETROMETER △ TORVANE ● UNCONFINED COMPRESSION ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL 1.0 2.0 3.0 4.0
-2.0										
-3.0		▨	**Sandy Lean Clay (CL)** medium stiff, moist, grayish brown, fine sand, low to moderate plasticity	21	☒ SPT-10A		28			○
-5.0	30		Bottom of Boring at 30.0 feet.							
	35									
	40									
	45									
	50									
	55									

CORNERSTONE
EARTH GROUP

BORING NUMBER EB-4

PAGE 1 OF 2

PROJECT NAME Ocean Place
PROJECT NUMBER 908-3-1
PROJECT LOCATION 908 Ocean Street, Santa Cruz, CA
DATE STARTED 10/14/19 **DATE COMPLETED** 10/14/19
GROUND ELEVATION 25 FT +/- **BORING DEPTH** 61.5 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 36.981432° **LONGITUDE** -122.021789°
DRILLING METHOD Mobile B-56, 8 inch Hollow-Stem Auger
GROUNDWATER LEVELS:
LOGGED BY JLC
∇ **AT TIME OF DRILLING** 11 ft.
NOTES
▼ **AT END OF DRILLING** 10 ft.

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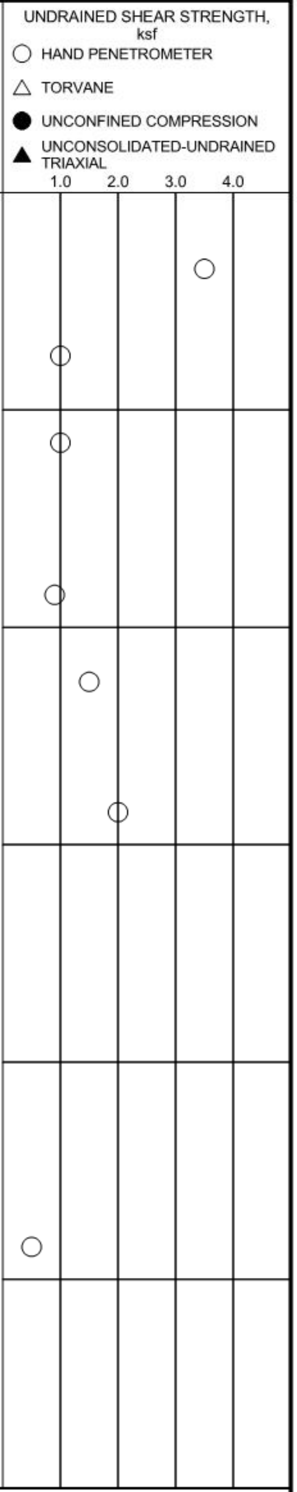
ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
25.0	0	**Clayey Sand (SC) [Fill]** medium dense, moist, brown, fine to medium sand	21	MC-1B	93	20		
22.8		**Lean Clay (CL)** very stiff, moist, dark brown, some fine sand, moderate plasticity						
22.0		**Sandy Lean Clay (CL)** very stiff, moist, brown, fine sand, moderate plasticity, trace organics	13	MC-2B	89	20		
	5		25	MC-3B	96	26		
19.0		**Lean Clay (CL)** stiff, moist, dark brown, some fine sand, moderate plasticity						
		becomes medium stiff	17	MC-4B	91	28		
	10	becomes stiff		ST				
		becomes very stiff, color changes to gray with brown mottles	12	MC				
	15							
6.0		**Silty Sand (SM)** medium dense, moist, grayish brown, fine sand	25	MC-7B		26		
	20							
1.5		**Lean Clay with Sand (CL)** medium stiff, moist, dark gray with brown mottles, fine sand, moderate plasticity	4	SPT				
-0.5	25	**Silty Sand (SM)** medium dense, wet, grayish brown, fine sand	13	SPT-9A		34		
-3.5 -3.7								



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BORING NUMBER EB-4

PROJECT NAME Ocean Place

PROJECT NUMBER 908-3-1

PROJECT LOCATION 908 Ocean Street, Santa Cruz, CA

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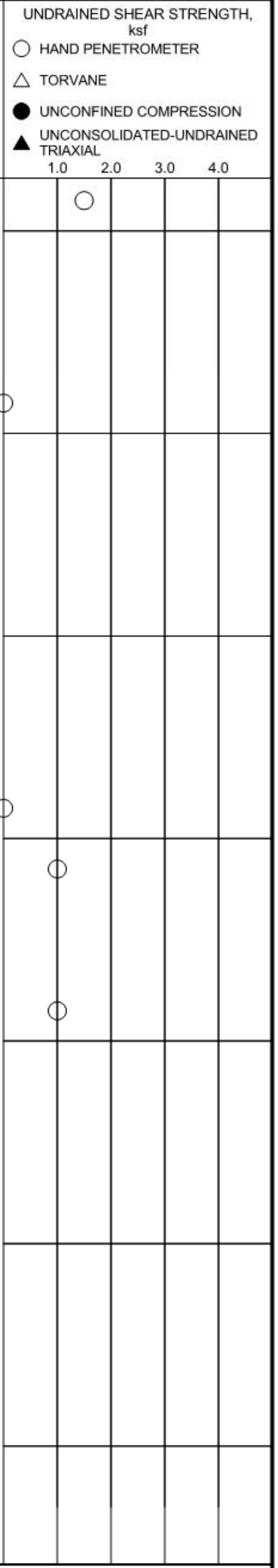
ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
-3.7	30	**Sandy Lean Clay (CL)** stiff, moist, dark gray with brown mottles, fine sand, moderate plasticity	10	SPT				
-7.0	35	**Lean Clay with Sand (CL)** soft, moist, gray with brown mottles, fine sand, moderate plasticity	12	MC-11B	86	32		
			15	MC				
-14.5	40	**Sandy Silt (ML)** soft, moist, gray with brown mottles, fine sand, low plasticity		ST				
-19.0		**Lean Clay (CL)** soft, moist, dark brown, moderate plasticity	21	MC-14C	78	42		
-19.5	45	**Sandy Silt (ML)** stiff, moist, gray with brown mottles, fine sand, low plasticity	13	SPT				
	50		16	MC				
-27.5		**Silty Sand (SM)** medium dense, moist, gray and brown, fine to medium sand	25	MC-17B	89	28		
	55		24	SPT				
			35	MC-19B	86	31		
	60	becomes very dense	54	SPT				
-36.5		Bottom of Boring at 61.5 feet.						



CORNERSTONE EARTH GROUP

BORING NUMBER EB-5

PAGE 1 OF 2

PROJECT NAME Ocean Place
PROJECT NUMBER 908-3-1
PROJECT LOCATION 908 Ocean Street, Santa Cruz, CA
DATE STARTED 10/15/19 **DATE COMPLETED** 10/15/19
GROUND ELEVATION 23.5 FT +/- **BORING DEPTH** 47.5 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 36.980909° **LONGITUDE** -122.021463°
DRILLING METHOD Mobile B-56, 8 inch Hollow-Stem Auger
GROUNDWATER LEVELS:
LOGGED BY SCO
▽ **AT TIME OF DRILLING** 11 ft.
NOTES
▼ **AT END OF DRILLING** 11 ft.

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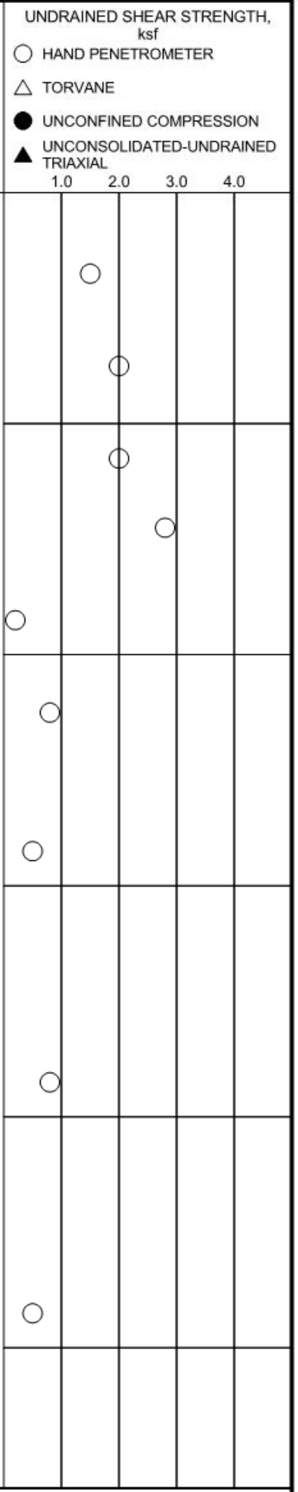
ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
23.5 23.4	0	1½ inches asphalt concrete over 4 inches aggregate base						
23.1		**Sandy Lean Clay (CL) [Fill]** stiff, moist, gray and dark brown mottled, fine to coarse sand, low plasticity, wood present	15	MC-1B	91	18		
20.5		**Poorly Graded Sand with Clay (SP-SC) [Fill]** medium dense, moist, gray, fine to medium sand	18	MC				
20.0	5	**Lean Clay (CL)** stiff, moist, dark brown, some fine sand, moderate plasticity	21	MC-3C	84	28		
17.7		**Silty Sand (SM)** medium dense, moist, gray, fine sand	32	MC				
17.0		**Lean Clay (CL)** very stiff, moist, dark brown to brown, some fine sand, moderate plasticity	16	MC-5B	88	32		
	10	becomes soft to medium stiff		ST-6	91	29		
10.5	15	**Lean Clay with Sand (CL)** medium stiff, moist, gray with brown mottles to gray, fine sand, moderate plasticity	13	MC-7B	83	36		
		becomes medium stiff	25	MC				
4.0	20	**Silty Sand (SM)** medium dense, moist, grayish brown, fine sand	19	SPT-10B		28		
1.5	25	**Sandy Lean Clay (CL)** medium stiff, moist, gray, fine sand, low plasticity	9	MC				
-3.5								



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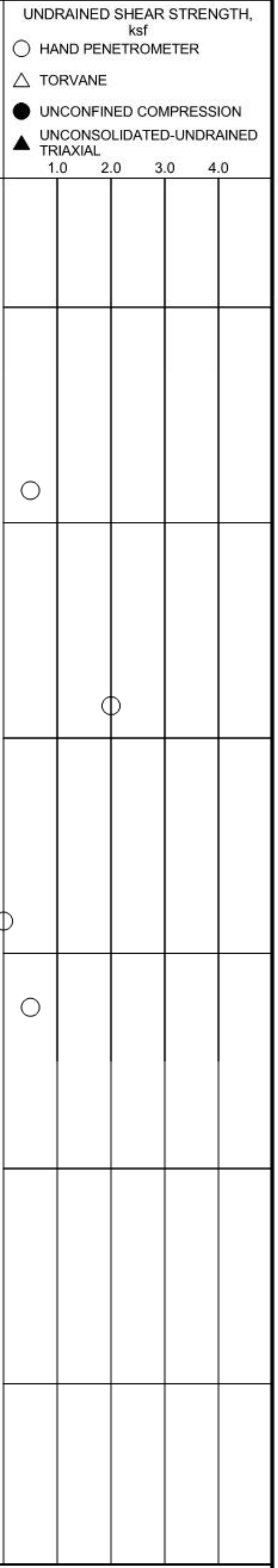
BORING NUMBER EB-5

PROJECT NAME Ocean Place
PROJECT NUMBER 908-3-1
PROJECT LOCATION 908 Ocean Street, Santa Cruz, 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
-3.5									
-4.0			**Sandy Silt (ML)** medium stiff, moist, gray, fine sand, low plasticity	13	MC-12B	88	31		
-6.5	30		**Lean Clay with Sand (CL)** medium stiff, moist, gray, fine to medium sand, low plasticity						
-9.0			**Sandy Silt (ML)** medium stiff, moist, gray, fine sand, low plasticity	16	MC				
	35								
-14.0			**Lean Clay with Sand (CL)** stiff, moist, gray, fine to medium sand, low plasticity	20	MC-14B	98	27		
	40								
			becomes very soft, wet	27	MC-15B		29		
	45								
			becomes medium stiff		ST				
-24.0			Bottom of Boring at 47.5 feet.						
	50								
	55								

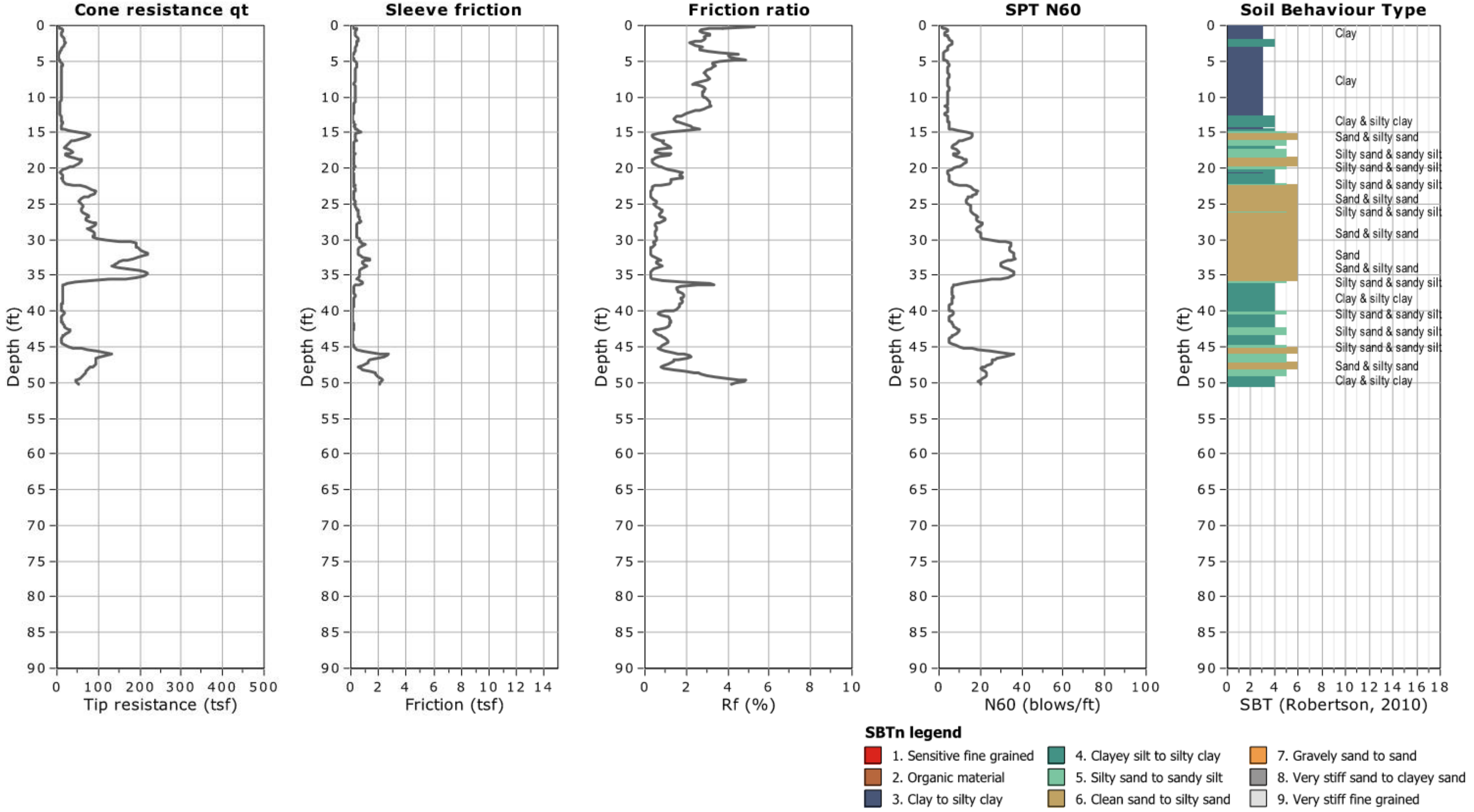


GREGG DRILLING, INC.
www.greggdrilling.com

CPT: CPT-1

CLIENT: CORNERSTONE EARTH GROUP
SITE: OCEAN PLACE - SANTA CRUZ, CA

FIELD REP: STEPHEN
Total depth: 50.20 ft, Date: 10/4/2019



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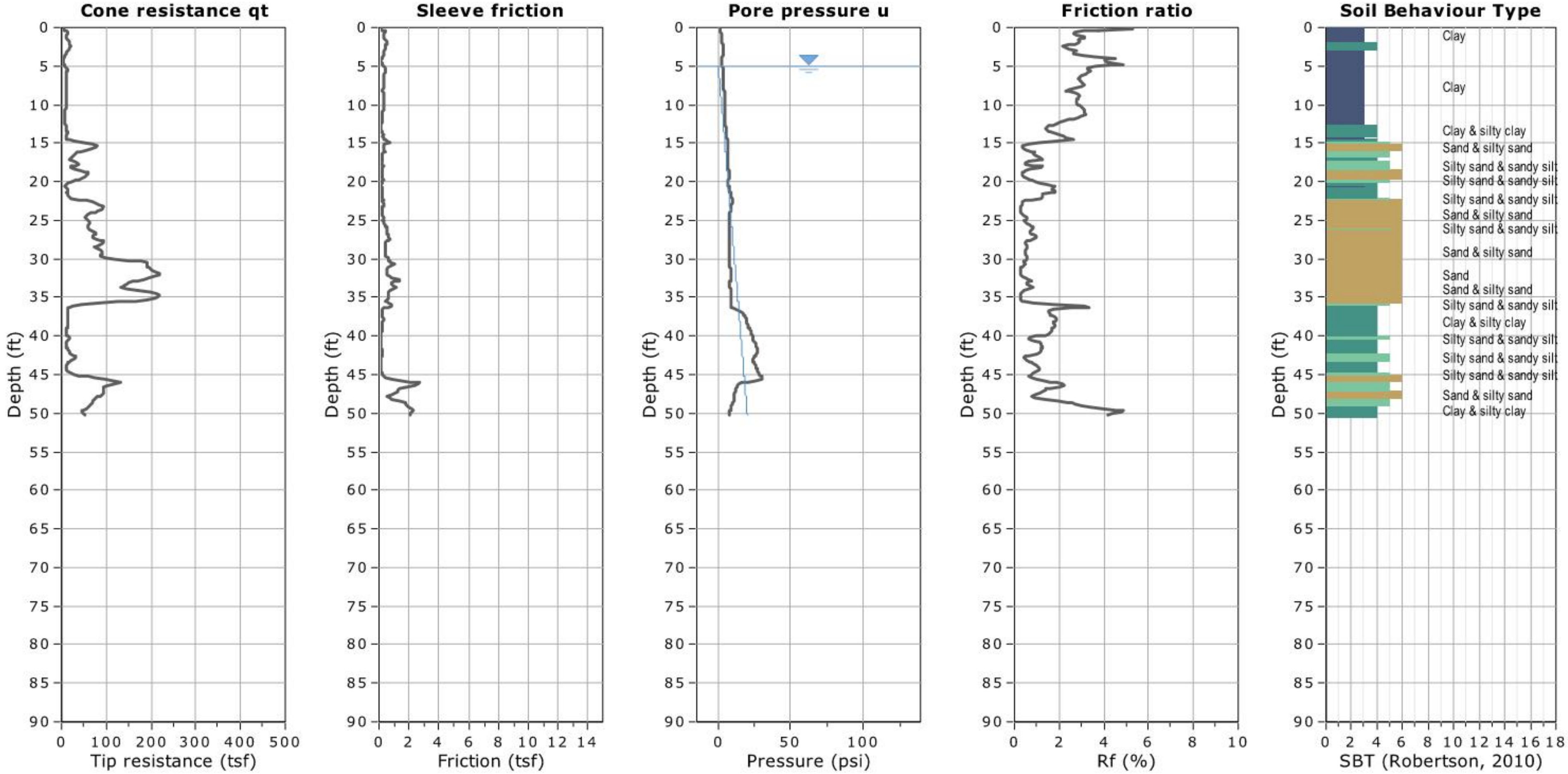
GREGG DRILLING, INC.
www.greggdrilling.com

CPT: CPT-1

CLIENT: CORNERSTONE EARTH GROUP
SITE: OCEAN PLACE - SANTA CRUZ, CA

Field Rep: STEPHEN
Total depth: 50.20 ft, Date: 10/4/2019



SBTn legend

- 🟥 1. Sensitive fine grained
- 🟫 2. Organic material
- 🟦 3. Clay to silty clay
- 🟩 4. Clayey silt to silty clay
- 🟩 5. Silty sand to sandy silt
- 🟨 6. Clean sand to silty sand
- 🟧 7. Gravely sand to sand
- ⬛ 8. Very stiff sand to clayey sand
- ⬜ 9. Very stiff fine grained

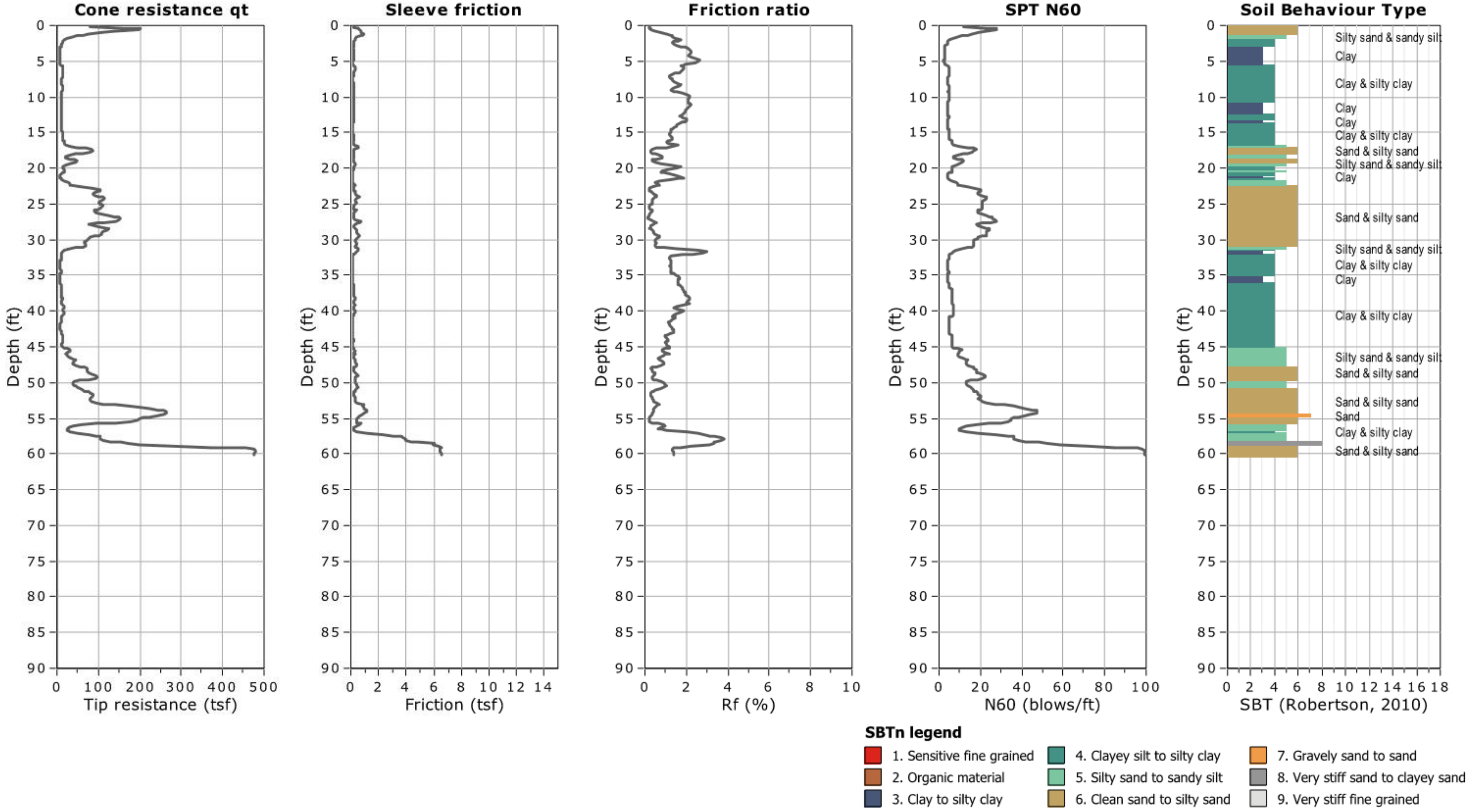
WATER TABLE FOR ESTIMATING PURPOSES ONLY

GREGG DRILLING, INC.
www.greggdrilling.com

CPT: SCPT-2

CLIENT: CORNERSTONE EARTH GROUP
SITE: OCEAN PLACE - SANTA CRUZ, CA

FIELD REP: STEPHEN
Total depth: 60.04 ft, Date: 10/4/2019

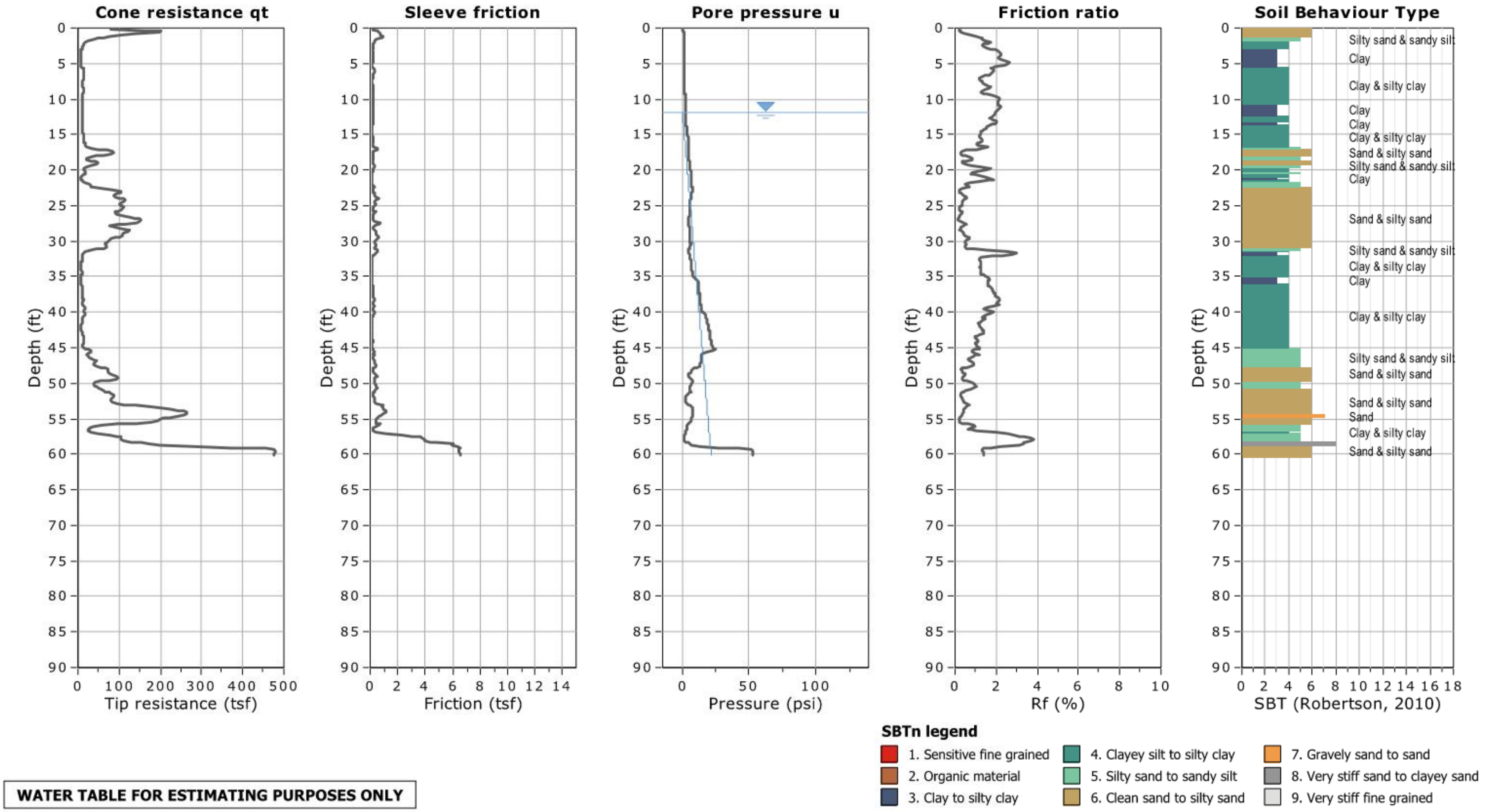

Cone resistance qt · **Sleeve friction** · **Friction ratio** · **SPT N60** · **Soil Behaviour Type**

GREGG DRILLING, INC.
www.greggdrilling.com

CPT: SCPT-2

CLIENT: CORNERSTONE EARTH GROUP
SITE: OCEAN PLACE - SANTA CRUZ, CA

Field Rep: STEPHEN
Total depth: 60.04 ft, Date: 10/4/2019



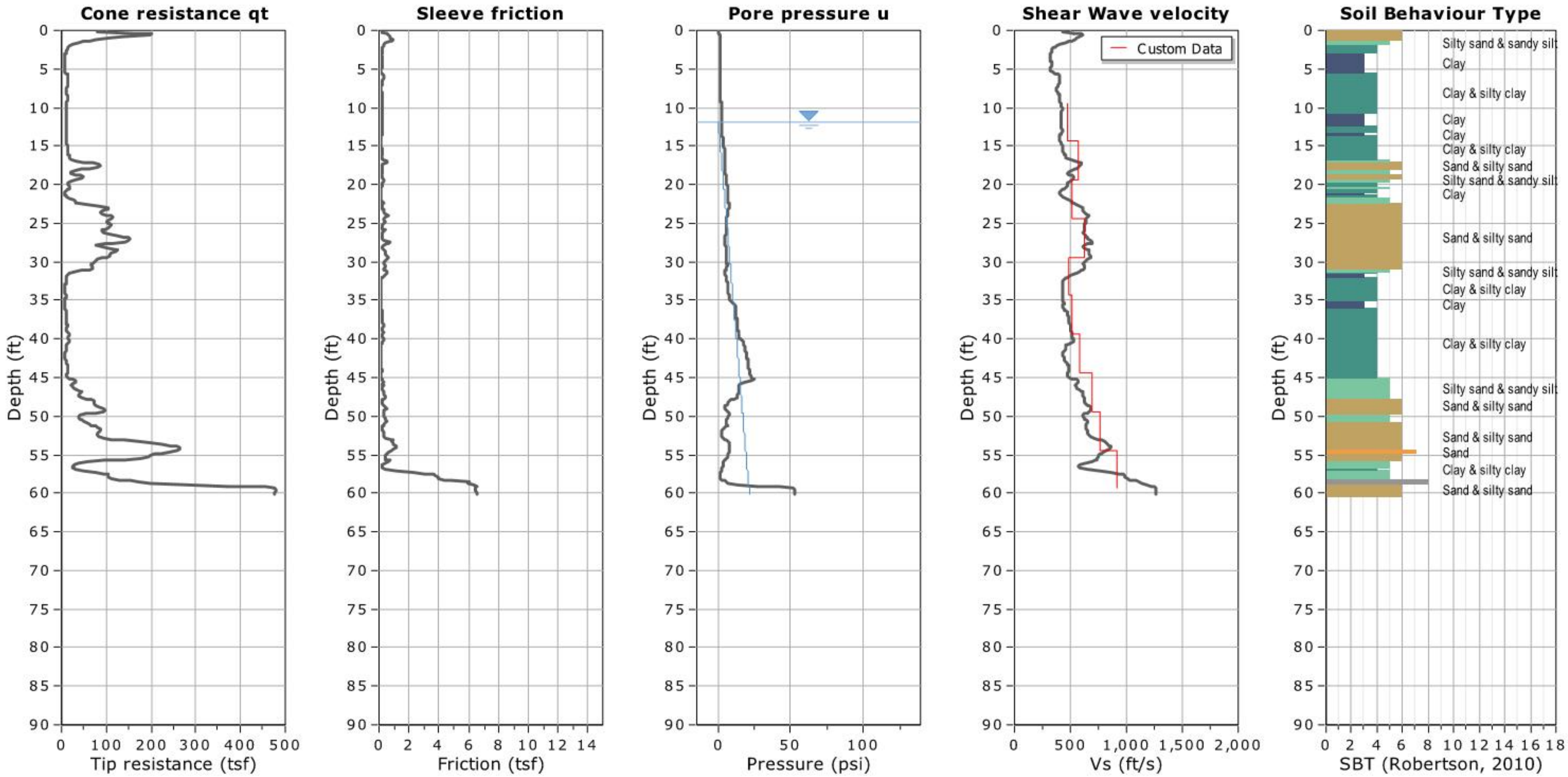
WATER TABLE FOR ESTIMATING PURPOSES ONLY

GREGG DRILLING, INC.
www.greggdrilling.com

CPT: SCPT-2

CLIENT: CORNERSTONE EARTH GROUP
SITE: OCEAN PLACE - SANTA CRUZ, CA

Field Rep: STEPHEN
Total depth: 60.04 ft, Date: 10/4/2019



SBTn legend

- 🟥 1. Sensitive fine grained
- 🟫 2. Organic material
- 🟦 3. Clay to silty clay
- 🟩 4. Clayey silt to silty clay
- 🟩 5. Silty sand to sandy silt
- 🟨 6. Clean sand to silty sand
- 🟧 7. Gravely sand to sand
- ⬛ 8. Very stiff sand to clayey sand
- ⬜ 9. Very stiff fine grained

WATER TABLE FOR ESTIMATING PURPOSES ONLY

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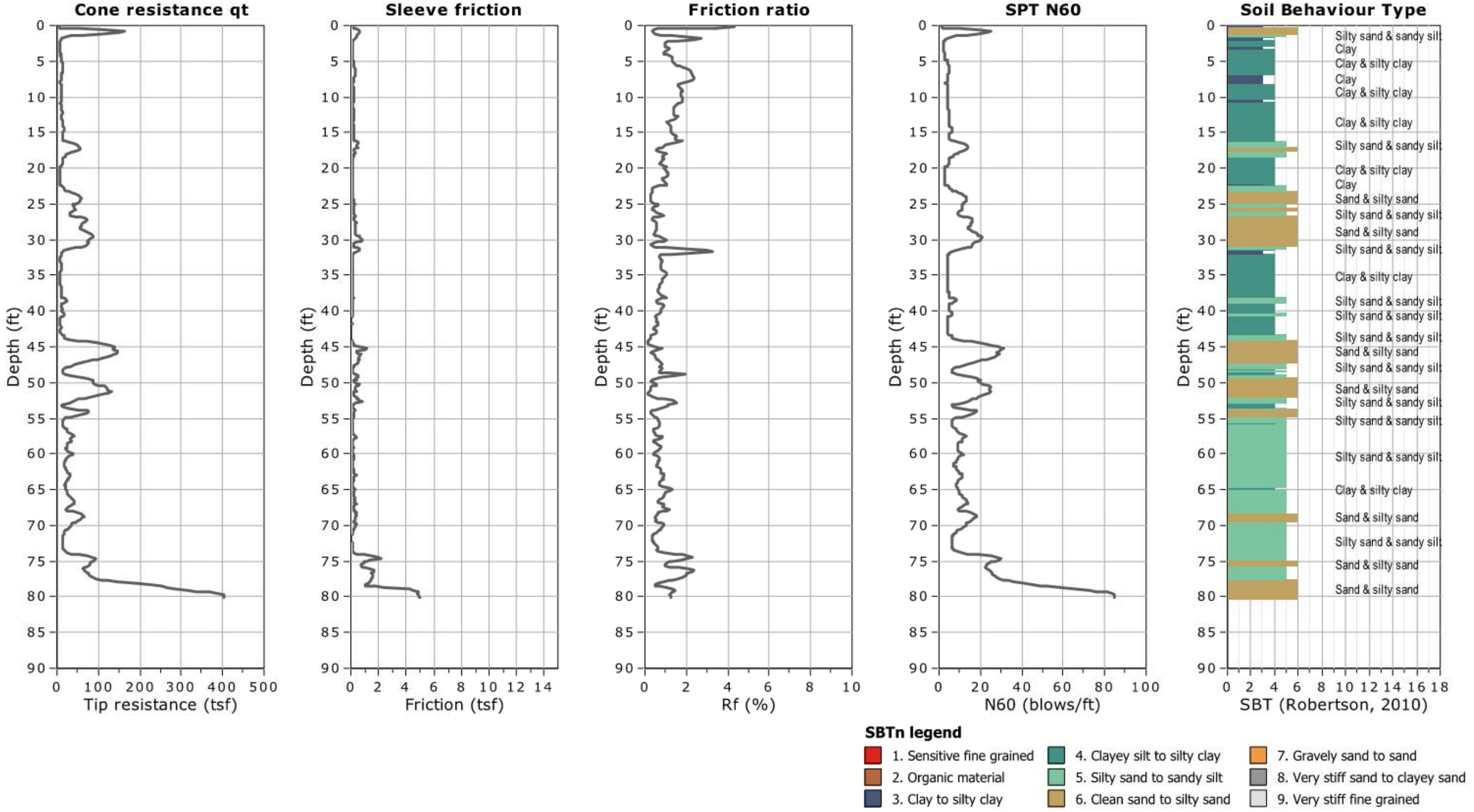
CPT: CPT-3

CLIENT: CORNERSTONE EARTH GROUP
SITE: OCEAN PLACE - SANTA CRUZ, CA

FIELD REP: STEPHEN
Total depth: 80.05 ft, Date: 10/4/2019

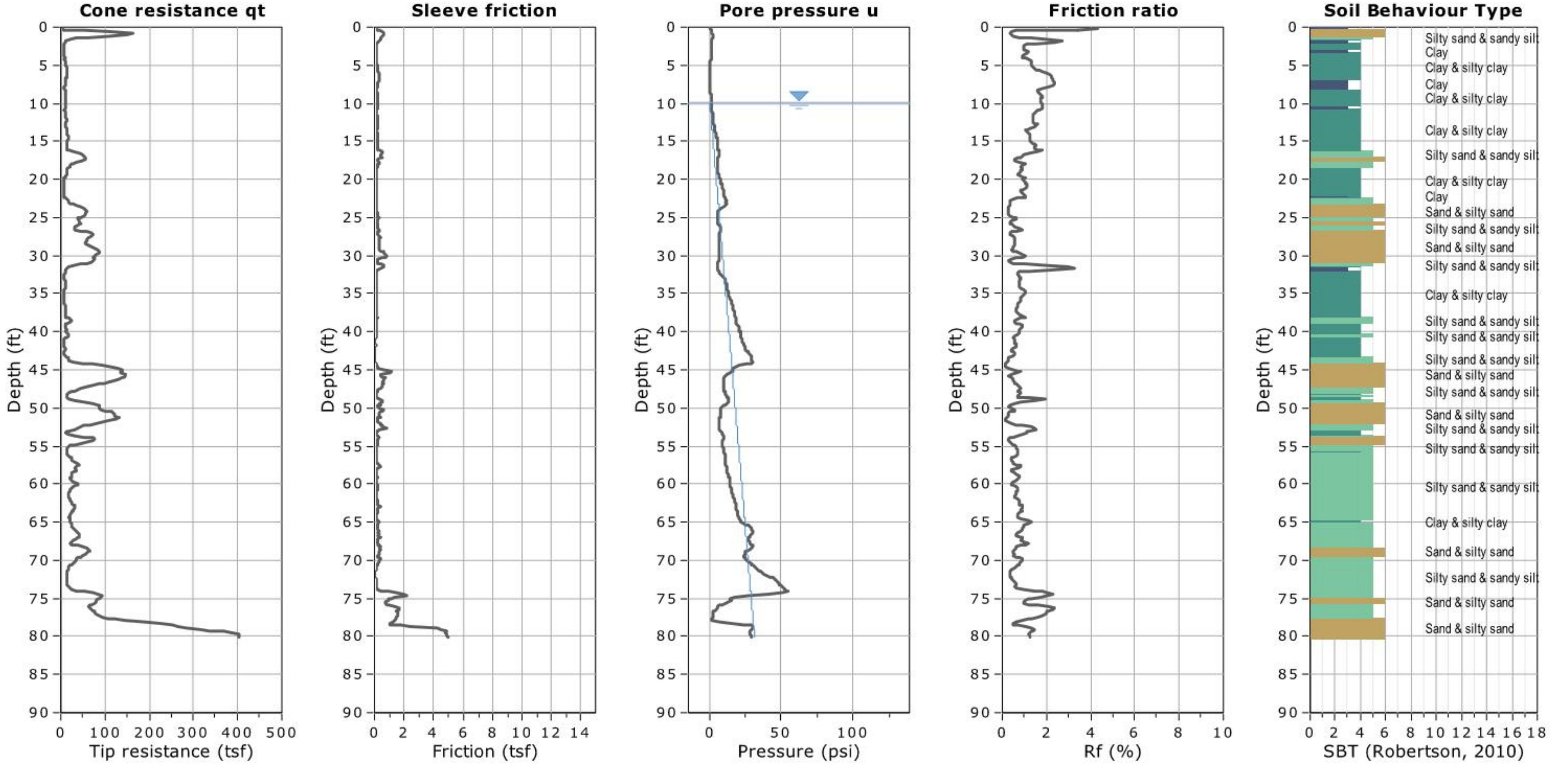


GREGG DRILLING, INC.
www.greggdrilling.com

CPT: CPT-3

CLIENT: CORNERSTONE EARTH GROUP
SITE: OCEAN PLACE - SANTA CRUZ, CA

Field Rep: STEPHEN
Total depth: 80.05 ft, Date: 10/4/2019



SBTn legend

- 🟥 1. Sensitive fine grained
- 🟫 2. Organic material
- 🟦 3. Clay to silty clay
- 🟩 4. Clayey silt to silty clay
- 🟩 5. Silty sand to sandy silt
- 🟨 6. Clean sand to silty sand
- 🟧 7. Gravely sand to sand
- ⬛ 8. Very stiff sand to clayey sand
- ⬜ 9. Very stiff fine grained

WATER TABLE FOR ESTIMATING PURPOSES ONLY

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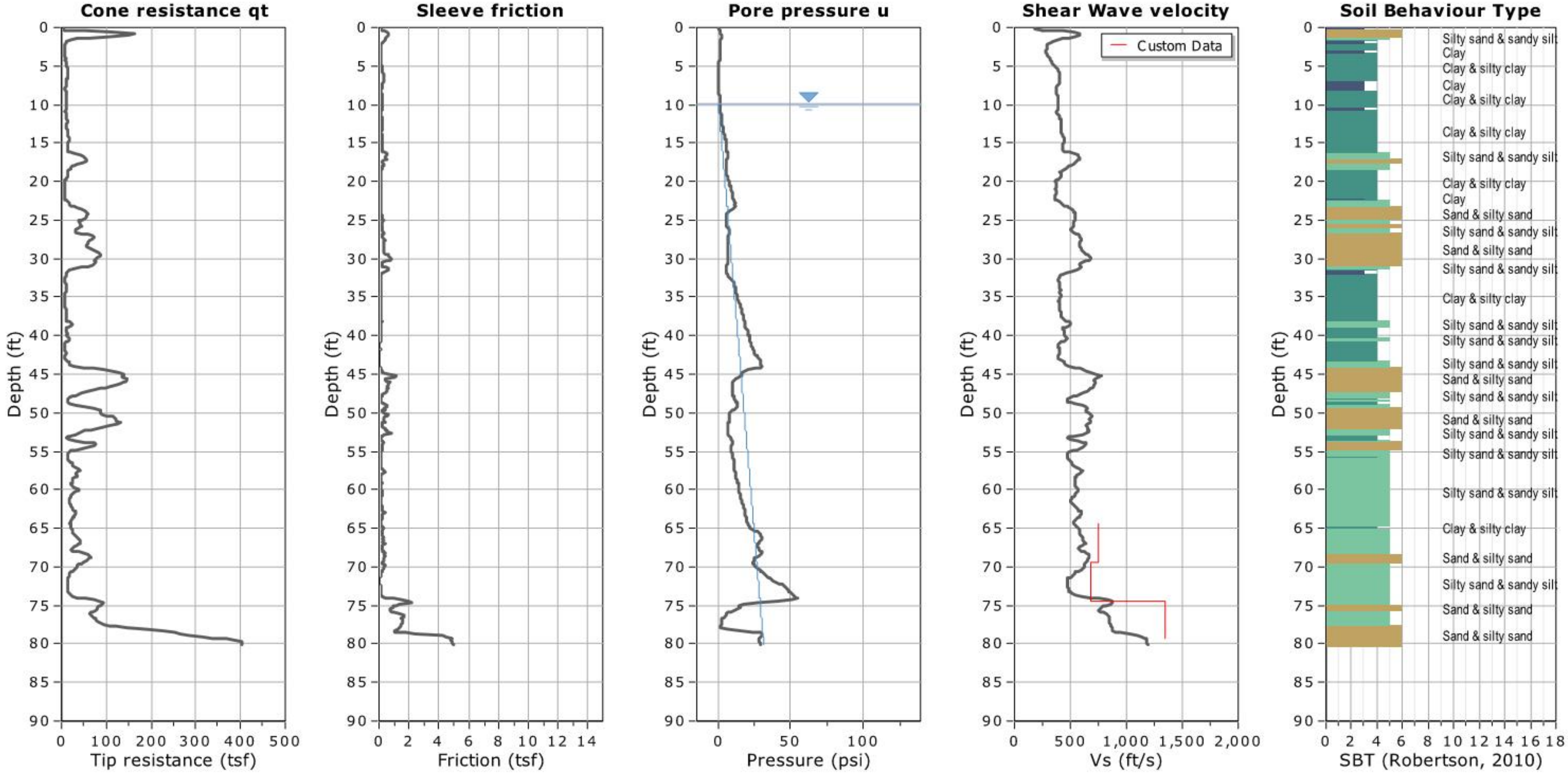
GREGG DRILLING, INC.
www.greggdrilling.com

CPT: CPT-3

CLIENT: CORNERSTONE EARTH GROUP
SITE: OCEAN PLACE - SANTA CRUZ, CA

Field Rep: STEPHEN
Total depth: 80.05 ft, Date: 10/4/2019



SBTn legend

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- 🟨 6. Clean sand to silty sand
- 🟧 7. Gravely sand to sand
- ⬛ 8. Very stiff sand to clayey sand
- ⬜ 9. Very stiff fine grained

WATER TABLE FOR ESTIMATING PURPOSES ONLY

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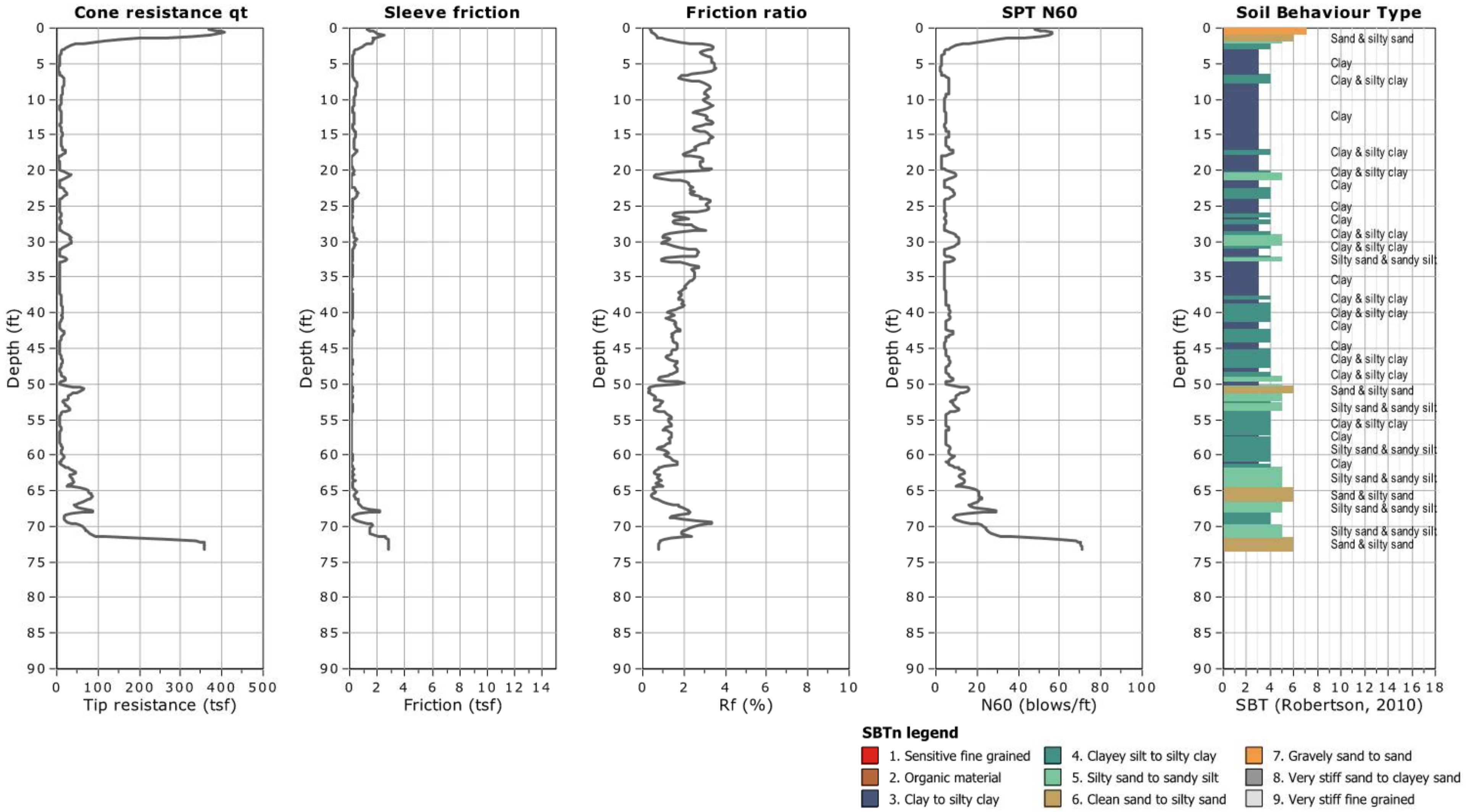
Project file: C:\Users\Frank Stolfi\OneDrive - Gregg Drilling\SH-2019\190621SH\REPORT\190621.cpt

GREGG DRILLING, INC.
www.greggdrilling.com

CPT: CPT-4

CLIENT: CORNERSTONE EARTH GROUP
SITE: OCEAN PLACE - SANTA CRUZ, CA

FIELD REP: STEPHEN
Total depth: 73.16 ft, Date: 10/4/2019

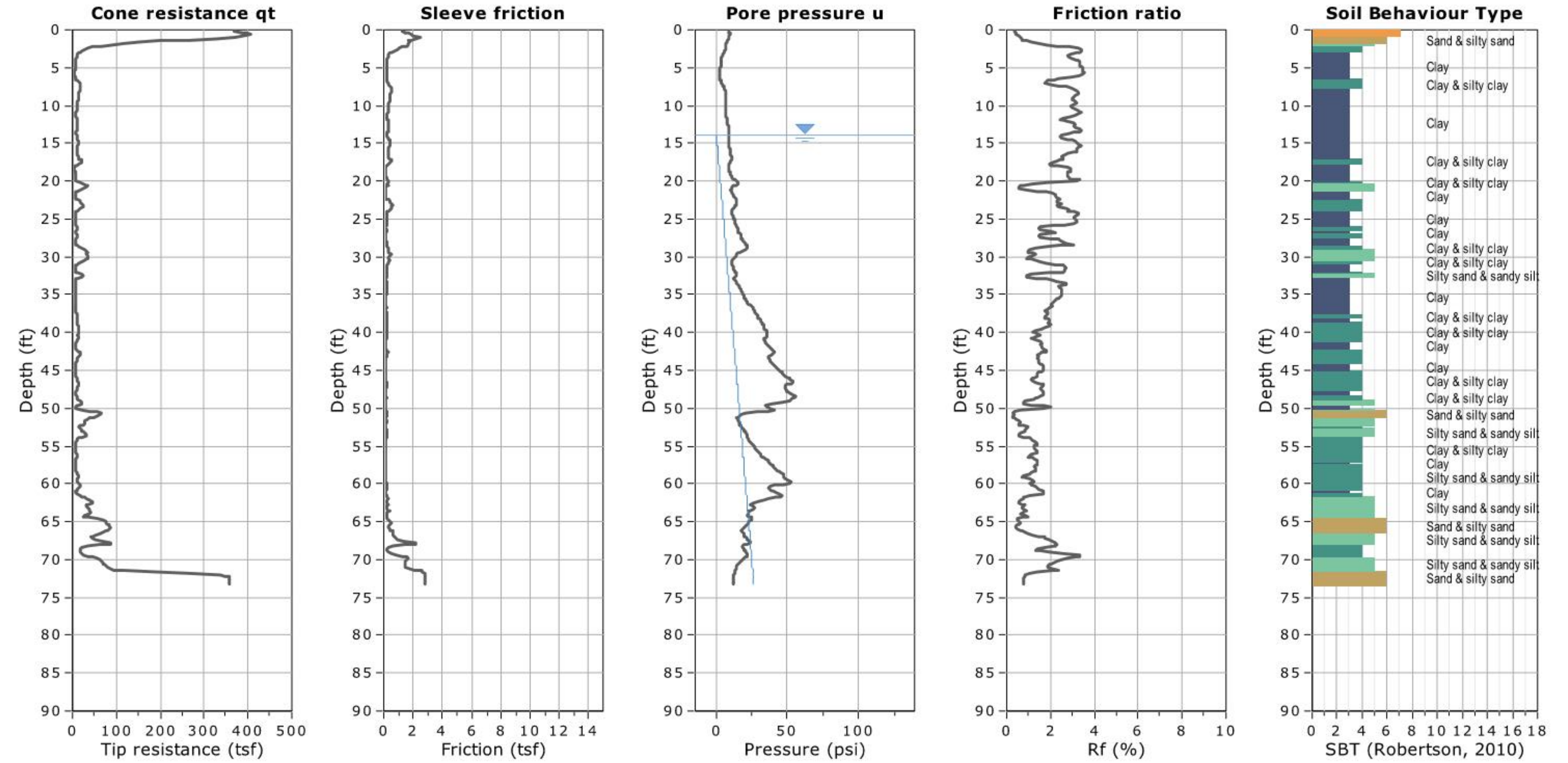


GREGG DRILLING, INC.
www.greggdrilling.com

CPT: CPT-4

CLIENT: CORNERSTONE EARTH GROUP
SITE: OCEAN PLACE - SANTA CRUZ, CA

Field Rep: STEPHEN
Total depth: 73.16 ft, Date: 10/4/2019



SBTn legend

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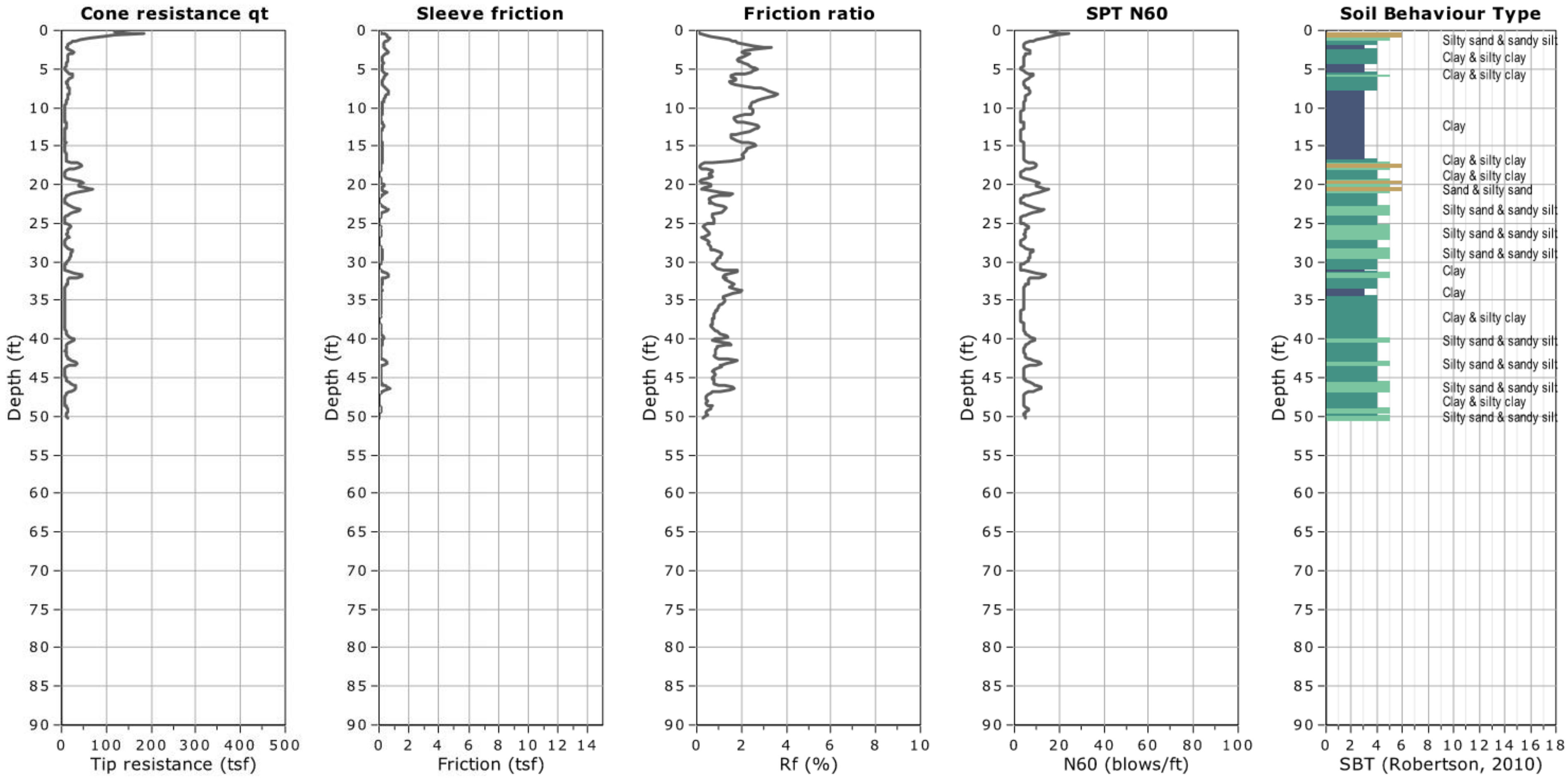
- 🟩 5. Silty sand to sandy silt
- 🟫 6. Clean sand to silty sand
- 🟧 7. Gravely sand to sand
- ⬛ 8. Very stiff sand to clayey sand
- ⬜ 9. Very stiff fine grained

WATER TABLE FOR ESTIMATING PURPOSES ONLY

CPT: CPT-5

CLIENT: CORNERSTONE EARTH GROUP
SITE: OCEAN PLACE - SANTA CRUZ, CA

FIELD REP: STEPHEN
Total depth: 50.20 ft, Date: 10/4/2019



SBTn legend

- 🟥 1. Sensitive fine grained
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- 🟦 3. Clay to silty clay
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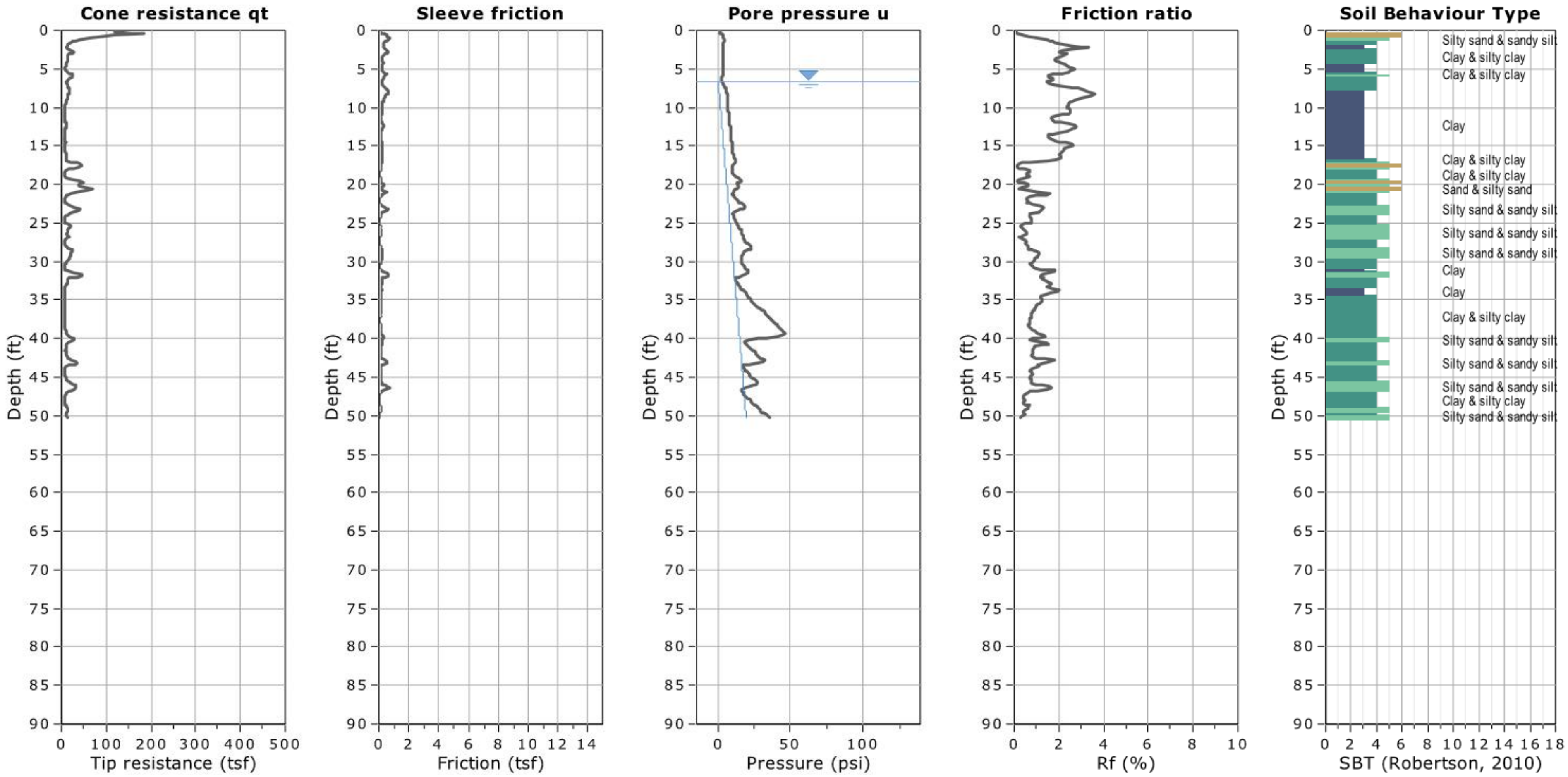
- 🟨 6. Clean sand to silty sand
- 🟧 7. Gravely sand to sand
- ⬛ 8. Very stiff sand to clayey sand
- ⬜ 9. Very stiff fine grained

GREGG DRILLING, INC.
www.greggdrilling.com

CPT: CPT-5

CLIENT: CORNERSTONE EARTH GROUP
SITE: OCEAN PLACE - SANTA CRUZ, CA

Field Rep: STEPHEN
Total depth: 50.20 ft, Date: 10/4/2019



SBTn legend

- 🟥 1. Sensitive fine grained
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- 🟩 4. Clayey silt to silty clay
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- 🟨 6. Clean sand to silty sand
- 🟧 7. Gravely sand to sand
- ⬜ 8. Very stiff sand to clayey sand
- ⬜ 9. Very stiff fine grained

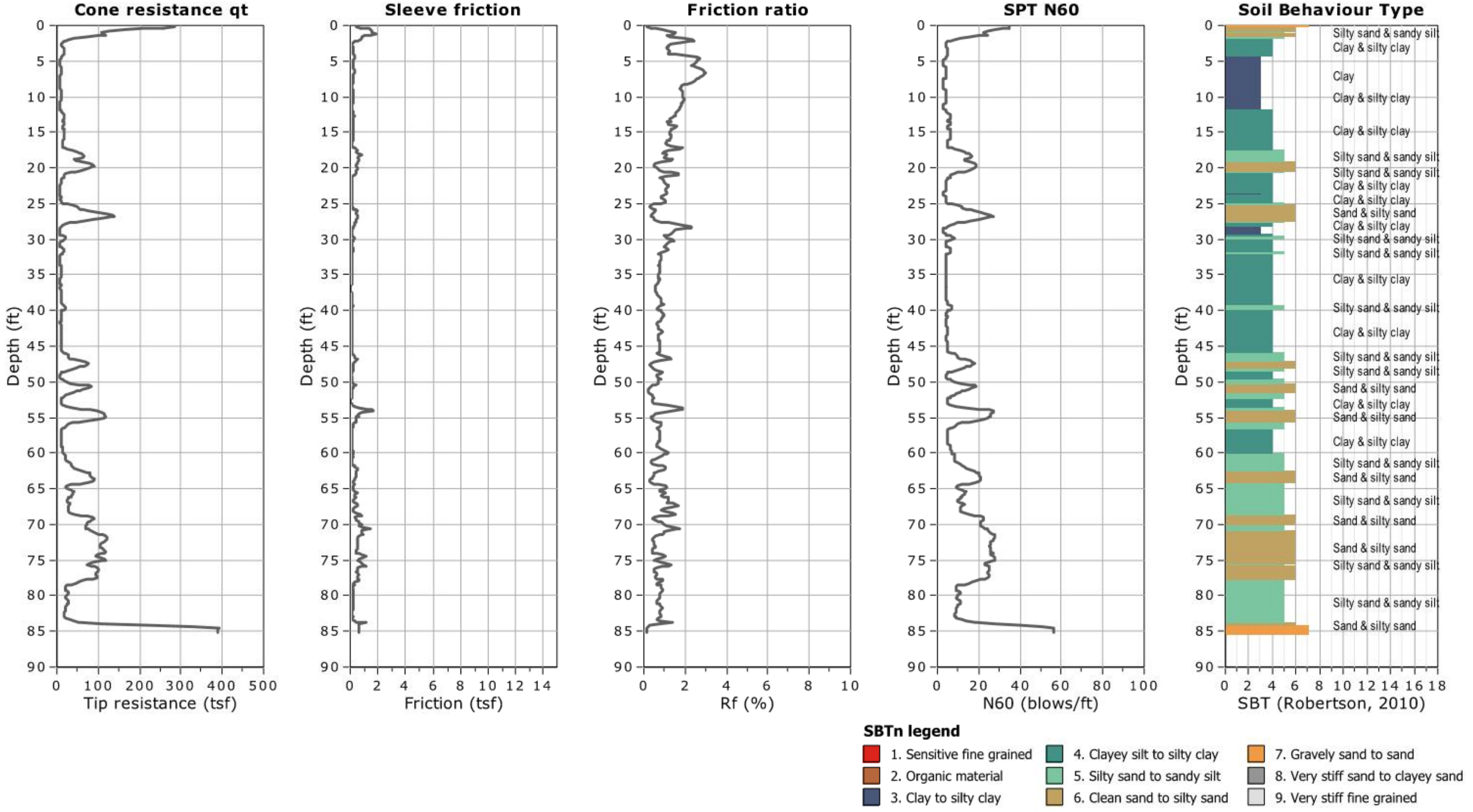
WATER TABLE FOR ESTIMATING PURPOSES ONLY

GREGG DRILLING, INC.
www.greggdrilling.com

CPT: CPT-6

CLIENT: CORNERSTONE EARTH GROUP
SITE: OCEAN PLACE - SANTA CRUZ, CA

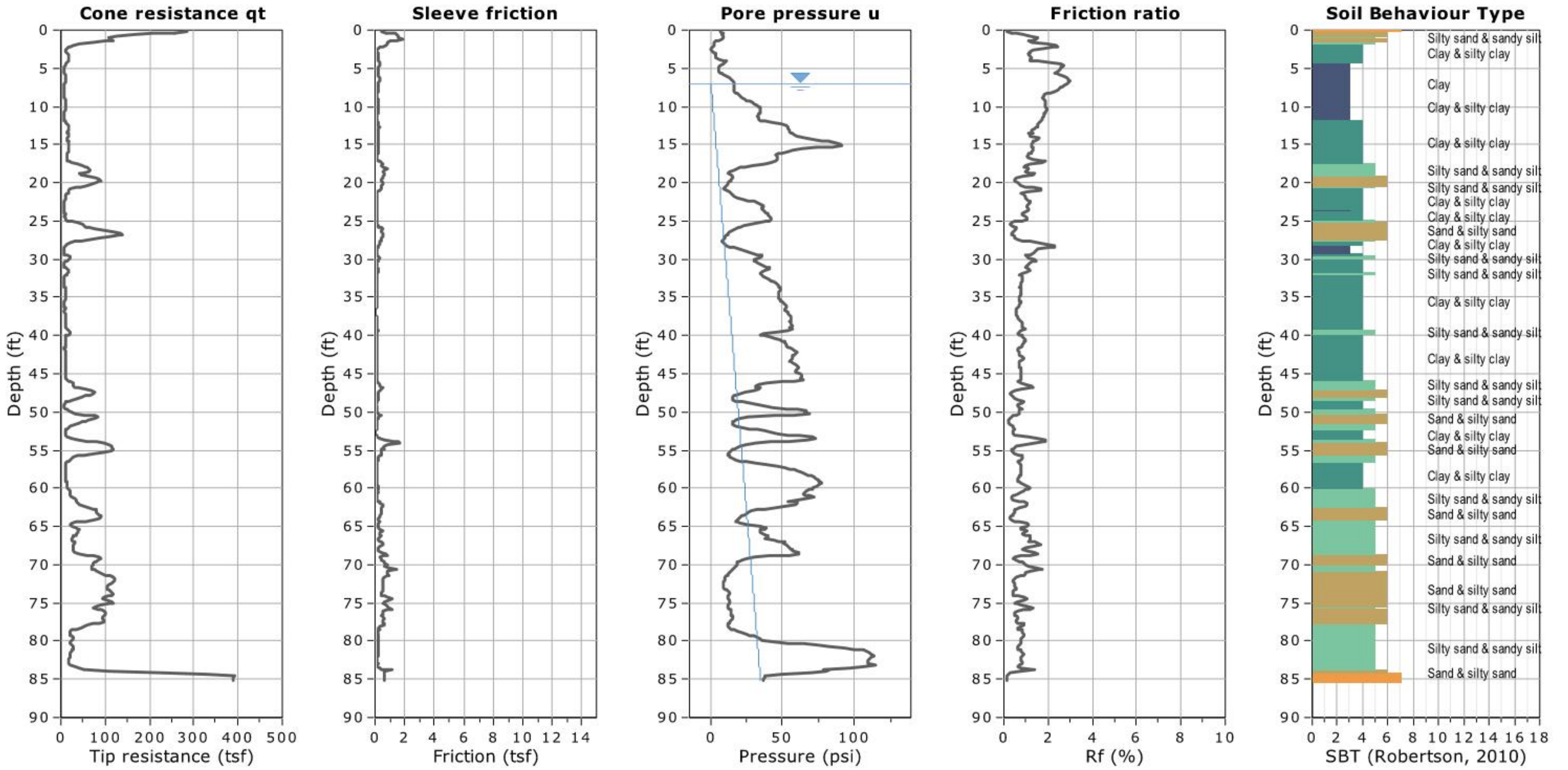
FIELD REP: STEPHEN
Total depth: 85.14 ft, Date: 10/4/2019



CPT: CPT-6

CLIENT: CORNERSTONE EARTH GROUP
SITE: OCEAN PLACE - SANTA CRUZ, CA

Field Rep: STEPHEN
Total depth: 85.14 ft, Date: 10/4/2019



SBTn legend

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- 🟩 4. Clayey silt to silty clay
- 🟩 5. Silty sand to sandy silt
- 🟨 6. Clean sand to silty sand
- 🟧 7. Gravely sand to sand
- ⬛ 8. Very stiff sand to clayey sand
- ⬜ 9. Very stiff fine grained

WATER TABLE FOR ESTIMATING PURPOSES ONLY

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 43 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 32 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. The result of this test is shown on the boring logs at the appropriate sample depth.

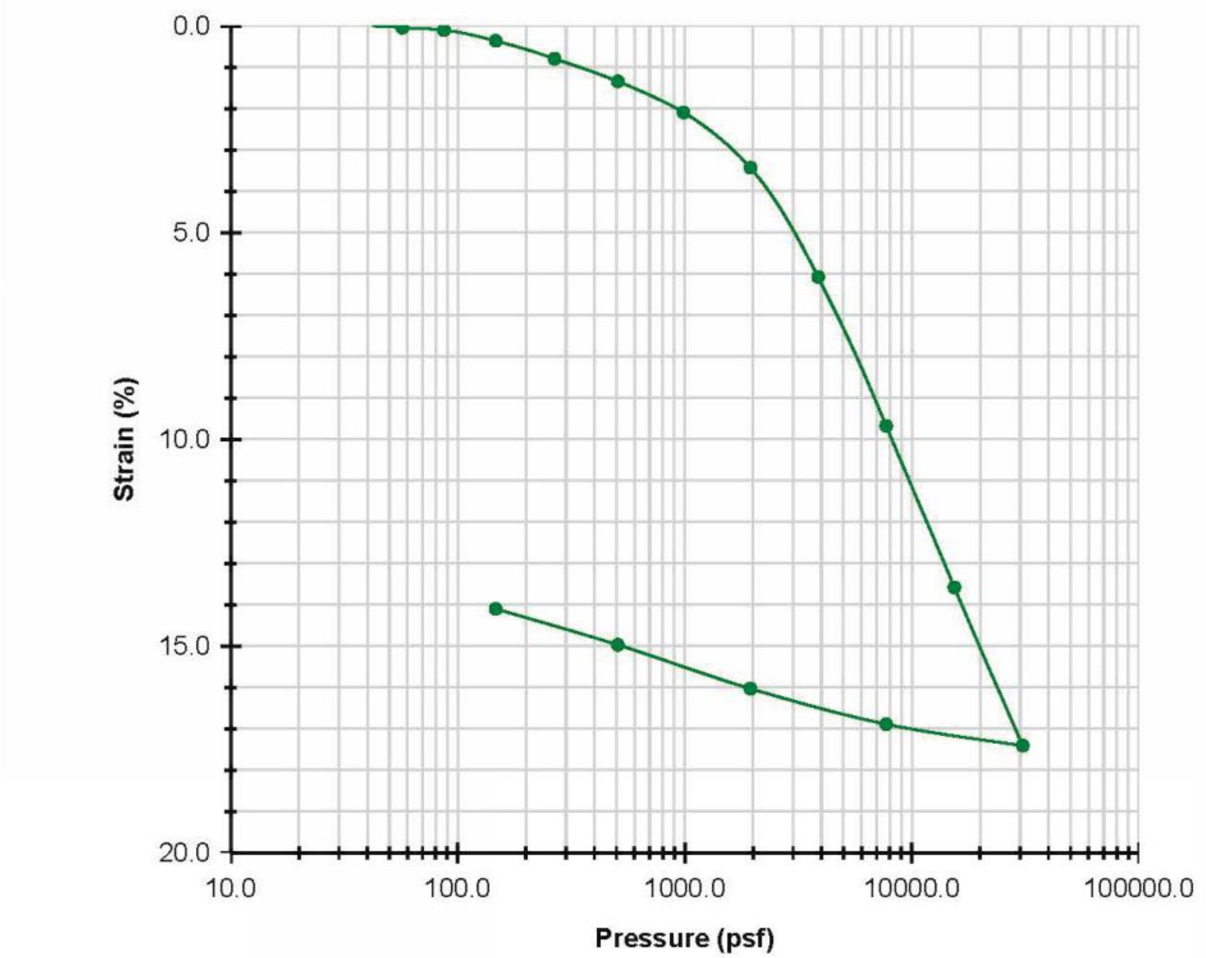
Plasticity Index: Two Plasticity Index determinations (ASTM D4318) were performed on samples of the subsurface soils to measure the range of water contents over which this material exhibits plasticity. The Plasticity Index was used to classify the soil in accordance with the Unified Soil Classification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

Consolidation: Two consolidation tests (ASTM D2435) were performed on relatively undisturbed samples of the subsurface clayey soils to assist in evaluating the compressibility property of this soil. Results of the consolidation tests are presented graphically in this appendix.

Consolidation Test ASTM D2435

Boring: EB-2 **Sample:** 11 **Depth:** 35.7’

Description: Lean Clay (CL)



	BEFORE	AFTER
Moisture (%)	34.4	25.3
Dry Density (pcf)	85.5	100.5
Saturation (%)	94.9	100.0
Void Ratio	0.99	0.69

—●— (A) Stress Strain Curve

CORNERSTONE EARTH GROUP

Strain-Log Curve - EB-2 @ 35.7’

Ocean Place
Santa Cruz, CA

Project Number 908-3-1

Figure Number Figure B2

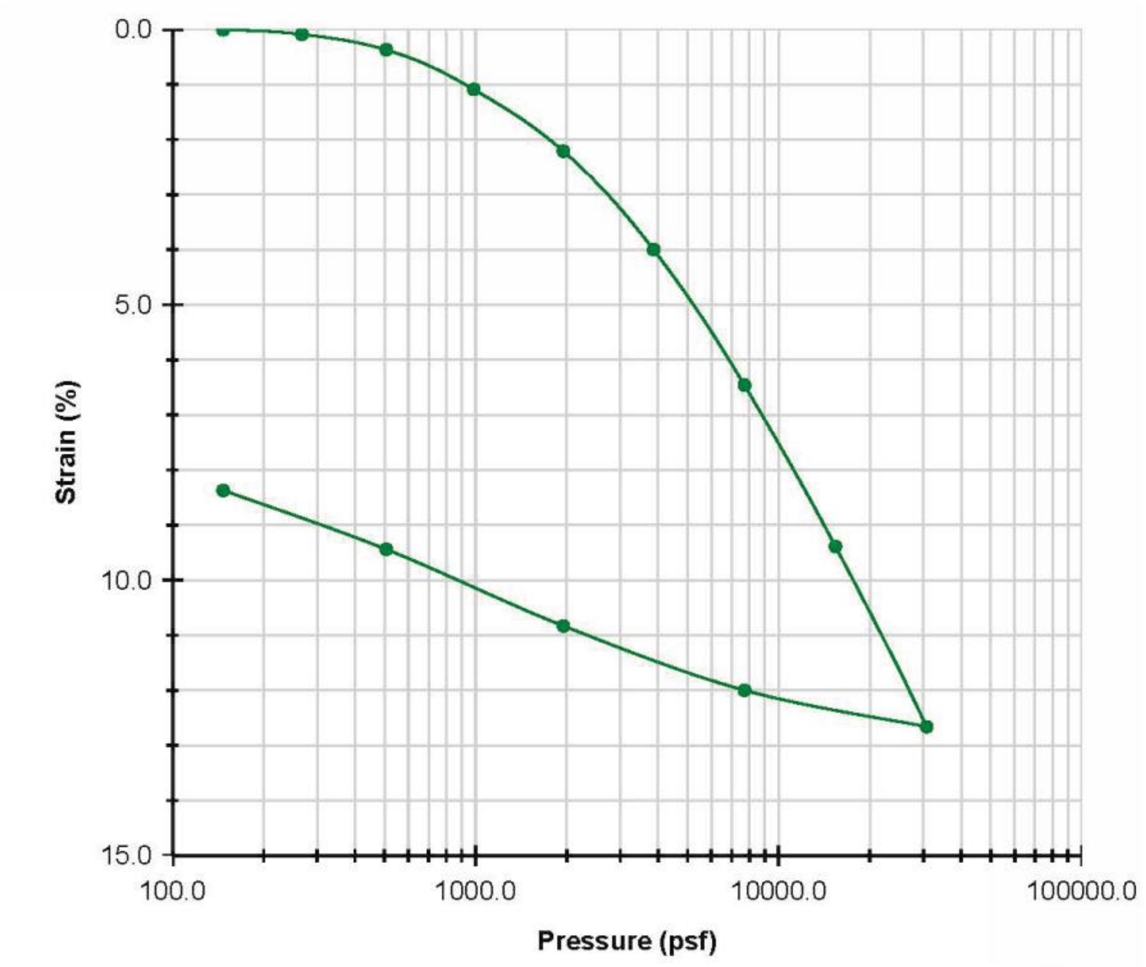
Date October 2019

Drawn By FLL

Consolidation Test ASTM D2435

Boring: EB-5 **Sample:** 6 **Depth:** 12.0'

Description: Lean Clay (CL)



	BEFORE	AFTER
Moisture (%)	29.3	24.3
Dry Density (pcf)	91.3	102.2
Saturation (%)	92.8	100.0
Void Ratio	0.86	0.66

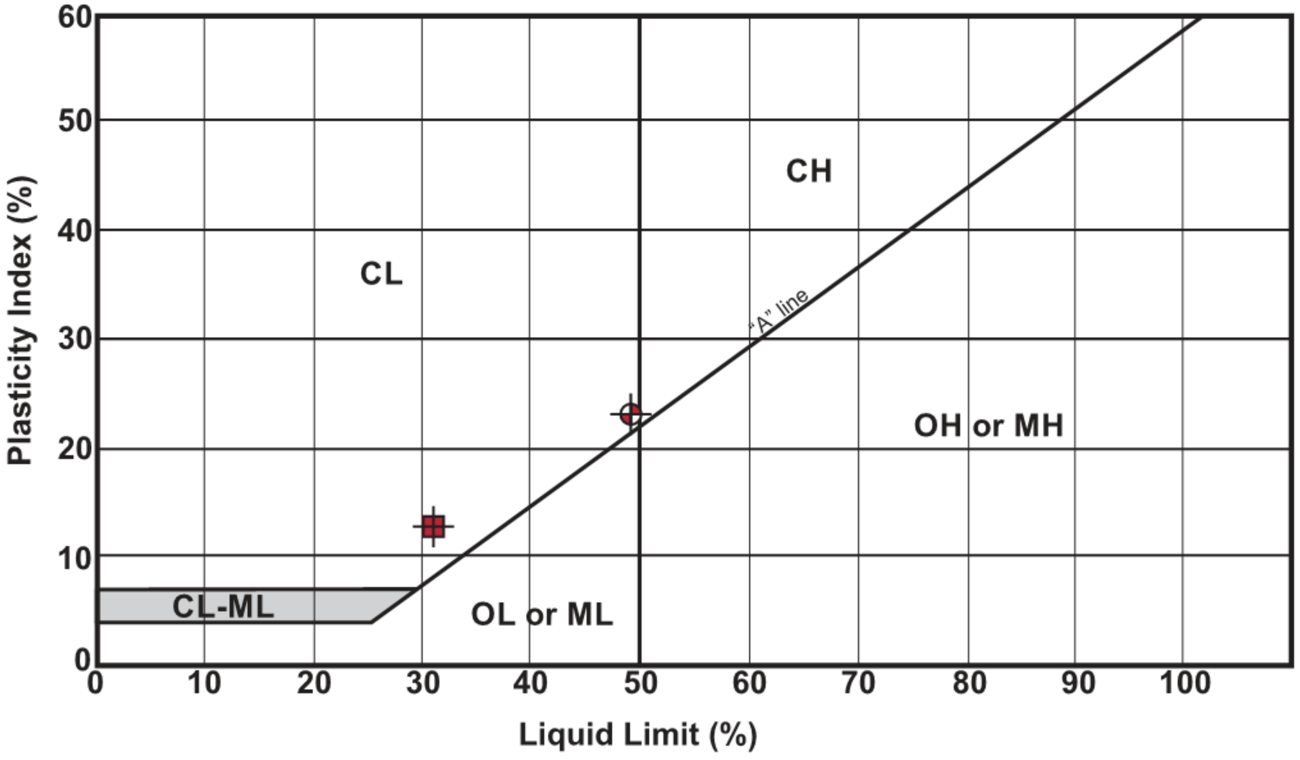
CORNERSTONE
EARTH GROUP

Strain-Log Curve - EB-5 @ 12.0'

Ocean Place
Santa Cruz, CA

Project Number	908-3-1
Figure Number	Figure B3
Date	October 2019
Drawn By	FLL

Plasticity Index (ASTM D4318) Testing Summary



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	35	49	26	23	—	Lean Clay (CL)
⊞	EB-3	14.5	25	31	19	12	—	Sandy Lean Clay (CL)

CORNERSTONE EARTH GROUP

Plasticity Index Testing Summary

Ocean Place
Santa Cruz, CA

Project Number 908-3-1

Figure Number Figure B1

Date October 2019

Drawn By FLL

Appendix D

Supplemental Geotechnical Recommendations

CORNERSTONE
EARTH GROUP

Date: April 26, 2021
Project No.: 908-3-1

Prepared For: Mr. Sean Quin
SALVATORE CARUSO DESIGN CORPORATION
980 El Camino Real, Suite 200
Santa Clara, California 95050

Re: Supplemental Geotechnical Recommendations
Ocean Place
Ocean Street
Santa Cruz, California

Dear Mr. Quin:

As requested, this letter provided additional comments and clarifications regarding the depth to groundwater for the project referenced above. We have completed a geotechnical investigation for the project with recommendations provided in our November 22, 2019 report. That report should be referred to for additional recommendations not provided in this letter.

GROUNDWATER

Groundwater was encountered in our exploratory borings EB-2 through EB-5 at depths ranging from about 9 to 11 feet below current grades (conventional drilled boring). These depths were measured and estimated. Groundwater was estimated at depths of about 5 to 14 feet below current grades based on pore pressure dissipation tests performed in CPT-1 through CPT-6. These depths were strictly estimated based on CPT data and correlations. These depths were not actually measured in the field. Depending on the soil conditions encountered, the time required to reach stabilized water levels can take from several hours to several days. The above measurements (from drilled borings) were taken at the time of drilling and may not represent the stabilized levels that can be higher than the initial levels encountered.

In addition, we also reviewed available groundwater data on Geotracker for the project vicinity. Based on our review, recorded depths to groundwater ranged from approximately 3½ to 5 feet below existing grades. These data were for project several hundred feet to greater than about 1 mile away from the project site.

For our analysis, we used a design high groundwater level to be at 5 feet below current grades. This design groundwater depth was determined based on our experience, engineering judgement, above available measurements, and available historical high groundwater data.

Fluctuations in groundwater levels occur due to many factors including seasonal fluctuation, underground drainage patterns, regional fluctuations, and other factors. Please be advised the actual depth to groundwater can be different than our estimates noted above and from the actual measured groundwater depth at the time of construction. Therefore, variation to the above estimates and recommendations should be expected and planned for.

CLOSURE

Should you have any questions, or if we may be of further service, please contact us and we will be glad to discuss them with you.

Sincerely,

Cornerstone Earth Group, Inc.

Danh T. Tran, P.E.
Senior Principal Engineer

Appendix E

Post Construction BMP Maintenance and/or Source Control Activities Table

Bioretention Area Maintenance Plan for Ocean Place

Bioretention areas function as soil and plant-based filtration devices that remove pollutants through a variety of physical, biological, and chemical treatment processes. These facilities normally consist of a grass buffer strip, sand bed, ponding area, organic layer or mulch layer, planting soil, and plants.

Project Address and Cross Streets:__________

908 Ocean Street, Santa Cruz, CA 95060

Assessor's Parcel No.:

The property contains 17 bioretention area(s) located as described below and as shown in the Storm Water Management Plan, see Appendix A.

I. Routine Maintenance Activities

The principal maintenance objective is to prevent sediment buildup and clogging, which reduces pollutant removal efficiency and may lead to bioretention area failure. Routine maintenance activities, and the frequency at which they will be conducted, are shown in Table 1.

Table 1-Routine Maintenance Activities for Bioretention Areas

No.	Maintenance Task	Frequency of Task
1	Remove obstructions, debris and trash from bioretention area and dispose of properly.	Monthly, or as needed after storm events
2	Inspect bioretention area to ensure that it drains between storms and within five days after rainfall.	Monthly, or as needed after storm events
3	Inspect inlets for channels, soil exposure or other evidence of erosion. Clear obstructions and remove sediment.	Monthly, or as needed after storm events
4	Remove and replace all dead and diseased vegetation.	Twice a year

Table 1-Routine Maintenance Activities for Bioretention Areas		
5	Maintain vegetation and the irrigation system. Prune and weed to keep bioretention area neat and orderly in appearance.	Before wet season begins, or as needed
6	Check that mulch is at appropriate depth (3 inches per soil specifications) and replenish as necessary before wet season begins.	Monthly
7	Inspect bioretention area using the attached inspection checklist.	Monthly, or after large storm events, and after removal of accumulated debris or material

II. Prohibitions

The use of pesticides and quick release fertilizers shall be minimized, and the principles of integrated pest management (IPM) followed:

1. Employ non-chemical controls (biological, physical and cultural controls) before using chemicals to treat a pest problem.
2. Prune plants properly and at the appropriate time of year.
3. Provide adequate irrigation for landscape plants. Do not over water.
4. Limit fertilizer use unless soil testing indicates a deficiency. Slow-release or organic fertilizer is preferable. Check with municipality for specific requirements.
5. Pest control should avoid harming non-target organisms, or negatively affecting air and water quality and public health. Apply chemical controls only when monitoring indicates that preventative and non-chemical methods are not keeping pests below acceptable levels. When pesticides are required, apply the least toxic and the least persistent pesticide that will provide adequate pest control. Do not apply pesticides on a prescheduled basis.
6. Sweep up spilled fertilizer and pesticides. Do not wash away or bury such spills.
7. Do not over apply pesticide. Spray only where the infestation exists. Follow the manufacturer’s instructions for mixing and applying materials.
8. Only licensed, trained pesticide applicators shall apply pesticides.
9. Apply pesticides at the appropriate time to maximize their effectiveness and minimize the likelihood of discharging pesticides into runoff. With the exception of pre-emergent pesticides, avoid application if rain is expected.
10. Unwanted/unused pesticides shall be disposed as hazardous waste.

Standing water shall not remain in the treatment measures for more than five days, to prevent mosquito generation. Should any mosquito issues arise, contact the Santa Cruz County Mosquito Abatement & Vector Control Division, as needed for assistance. Mosquito larvicides shall be applied only when absolutely necessary, as indicated by the Santa Cruz County Mosquito Abatement & Vector Control Division, and then only by a licensed professional or contractor. Contact information is provided below.

III. Mosquito Abatement Contact Information

Santa Cruz County Mosquito Abatement & Vector Control Division
870 17th Ave.
Santa Cruz, CA 95062
PH:(831) 454-2590

IV. Inspections

The attached Bioretention Area Inspection and Maintenance Checklist shall be used to conduct inspections monthly (or as needed), identify needed maintenance, and record maintenance that is conducted.

Bioretention Area
Inspection and Maintenance Checklist

Property Address: 908 Ocean Street Santa Cruz, CA 95060
Property Owner: City of Santa Cruz

Treatment Measure No.: ________ Date of Inspection: _______

Type of Inspection:
☐ Monthly ☐ Pre-Wet Season ☐ After heavy runoff ☐ End of Wet Season
☐ Other:_________________
Inspector(s): ________________________________

Defect	**Conditions When Maintenance Is Needed**	**Maintenance Needed? (Y/N)**	**Comments** (Describe maintenance completed and if needed maintenance was not conducted, note when it will be done)	**Results Expected When Maintenance Is Performed**
1. Standing Water	When water stands in the bioretention area between storms and does not drain within five days after rainfall.			There should be no areas of standing water once inflow has ceased. Any of the following may apply: sediment or trash blockages removed, improved grade from head to foot of bioretention area, or added underdrains.
2. Trash and Debris Accumulation	Trash and debris accumulated in the bioretention area.			Trash and debris removed from bioretention area and disposed of properly.
3. Sediment	Evidence of sedimentation in bioretention area.			Material removed so that there is no clogging or blockage. Material is disposed of properly.
4. Erosion	Channels have formed around inlets, there are areas of bare soil, and/or other evidence of erosion.			Obstructions and sediment removed so that water flows freely and disperses over a wide area. Obstructions and sediment are disposed of properly.
5. Vegetation	Vegetation is dead, diseased and/or overgrown.			Vegetation is healthy and attractive in appearance.
6. Mulch	Mulch is missing or patchy in appearance. Areas of bare earth are exposed, or mulch layer is less than 3 inches in depth.			All bare earth is covered, except mulch is kept 6 inches away from trunks of trees and shrubs. Mulch is even in appearance, at a depth of 3 inches.
7. Miscellaneous	Any condition not covered above that needs attention in order for the bioretention area to function as designed.			Meet the design specifications.

Stormwater Treatment Measure Operation and Maintenance Inspection Report for the Classics at Lawrence Station Project

This report and attached Inspection and Maintenance Checklists document the inspection and maintenance conducted for the identified stormwater treatment measure(s) subject to the Maintenance Agreement between the City and the property owner during the annual reporting period indicated below.

I. Property Information:

Property Address or APN: 908 Ocean Street, Santa Cruz, CA 95060______

Property Owner: City of Santa Cruz________________________________

II. Contact Information:

Name of person to contact regarding this report:______________________

Phone number of contact person:________________ Email: _______________________

Address to which correspondence regarding this report should be directed:

____

____

III. Reporting Period:

This report, with the attached completed inspection checklists, documents the inspections and maintenance of the identified treatment measures during the time period from ____________ to __________.

IV. Stormwater Treatment Measure Information:

The following stormwater treatment measures (identified treatment measures) are located on the property identified above and are subject to the Maintenance Agreement:

Identifying Number of Treatment Measure	Type of Treatment Measure	Location of Treatment Measure on the Property

V. Summary of Inspections and Maintenance:

Summarize the following information using the attached Inspection and Maintenance Checklists:

Identifying Number of Treatment Measure	Date of Inspection	Operation and Maintenance Activities Performed and Date(s) Conducted	Additional Comments

VI. Sediment Removal:

Total amount of accumulated sediment removed from the stormwater treatment measure(s) during the reporting period: _________ cubic yards.

How was sediment disposed?

☐ landfill

☐ other location on-site as described in and allowed by the maintenance plan

☐ other, explain____

VII. Inspector Information:

The inspections documented in the attached Inspection and Maintenance Checklists were conducted by the following inspector(s):

Inspector Name and Title	Inspector’s Employer and Address

VIII. Certification:

I hereby certify, under penalty of perjury, that the information presented in this report and attachments is true and complete:

____ __________________
Signature of Property Owner or Other Responsible Party Date

____
Type or Print Name

____
Company Name

____
Address

Phone number:__________________ Email:______________________________

SAMPLE BMP INSPECTION & MAINTENANCE FORM

Date:______________________________

Responsible Inspector:____

LANDSCAPE MAINTENANCE

Location	Date	Observations Maintenance or Repair Needed? Debris? Erosion Problems?	Action Taken	Date Completed

STORM DRAINAGE COLLECTION SYSTEM MAINTENANCE

Location	Date	Observations Debris or Sediment? Silt Accumulation?	Action Taken	Date Completed

1. STORMWATER TREATMENT SYSTEM MAINTENANCE

Location	Date	Observations Flow Obstructions? Overflow Drain Obstructions? Debris or Sediment? Erosion Problems?	Action Taken	Date Completed

SAMPLE FORM ONLY
INSPECTOR/OWNER TO EXPAND AND MODIFY AS NECESSARY

Employee Training Program Table

Table A-4: Employee Training Program

Name of Responsible Part responsible for training: _________________________
Provide the following information:
Address__________________
Phone________________ Fax________________ E-mail:

Description of Items for Training (e.g. maintenance, inspection, pesticide use, others as applicable to site)	Training Schedule	Employees To Be Trained (Job Category or Title)
Maintenance	Yearly	
Inspection	Yearly	