STORM WATER MANAGEMENT PLAN

SELF-STORAGE FACILITY

1HANSON COURT
Milpitas, California
June 01, 2015

PREPARED BY:

KIER & WRIGHT CIVIL ENGINEERS & SURVEYORS

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1.0 PROJECT DESCRIPTION

A. PROJECT LOCATION AND DESCRIPTION

The project site is located at 1 Hanson Court, Milpitas CA. The site consists of approximately 4.27 acres of land. This self-storage development consists of multiple buildings and onsite parking. The site will be treated by biotreatment pond.

The project currently drains to a municipal stormdrain system that outfalls to the Calera Creek, and ultimately to the South San Francisco Bay.

B. ASSESSOR'S PARCEL NUMBER

The project consists of one parcel: APN 022-31-020.

C. PROJECT ZONING AND USE

The project is zoned as a heavy industrial district (M2).

D. POLLUTANTS OF CONCERN

The anticipated pollutants of concern are listed as follows:

a. Petroleum Hydrocarbons
b. Oil & Grease
c. Sediments
d. Pesticides
e. Trash

2.0 HMP APPLICABILITY DETERMINATION

A review of the HMP Applicability Map for the City of Milpitas indicates that the project site's "subwatersheds and catchments are greater than or equal to 65% impervious" thus is not subject to HMP requirements, though HMP controls are recommended.

3.0 SITE CONSTRAINT AND BMP MEASURES

Based on the calculations shown in Appendix A for the existing and proposed impervious/pervious areas, the project site is replacing more than 50% of the pre-existing impervious surface. Therefore, the entire site will be subject to numerical sizing for storm water BMPs.

As part of the storm water treatment measures, the project was designed to utilize the same pre-development watershed areas for storm water runoff; this will prevent to need to upsize downstream storm systems.

The BMP summary table in Appendix F identifies the areas associated with each BMP measure based on their flow requirements and weighted runoff coefficients.

A. BIOTREATMENT POND

The project will be treated by biotreatment pond. The storm runoff from the site will be directed to the on-site storm drainage system before runoff is discharged to the pond. Refer to Appendix F for actual location of treatment ponds.

The treatment pond is a depressed landscaping area that allows the collection of stormwater runoff to percolate through a sandy loam soil into an under-drain, thereby promoting pollutant removal.

B. LABELING OF STORMWATER INLETS

Storm water inlets shall have metal badges installed with the logo “No Dumping -Flows to Bay”. This educational measure is intended to prevent unlawful dumping of waste materials such as motor oil or trash into the inlets.

C. INTEGRATED PEST MANAGEMENT

Alternative methods for pest reduction methods will be employed to limit the usage of pesticides. Method includes the incorporation of planting materials. Owner and maintenance staff shall review and adhere to the Landscape Maintenance Techniques for Pest Reduction in Appendix I.

4.0 POST CONSTRUCTION BMP MAINTENANCE AND SOURCE CONTROL

A. SPILL RESPONSE PROCEDURES

Due to the nature of the proposed uses at the site, spill responses are not anticipated.

B. PREVENTIVE MAINTENANCE OF STRUCTURAL BMPs

The property owner will enter into a perpetual maintenance contract for the maintenance of the biotreatment pond and flow-through planters.

Regular maintenance, sweeping, and trash pick-up from the parking and landscaping areas will be employed to decrease the incidence of solids and pollutants entering into the on-site storm drainage system. See Appendix I.

C. MATERIALS HANDLING AND STORAGE

No outside storage of materials is anticipated or allowed. Materials handling will only be allowed for normal business operations for office use. No car washing will be allowed within the project site. No vehicle storage will be anticipated on-site.

D. POST-CONSTRUCTION BMP MAINTENANCE AND/OR SOURCE CONTROL ACTIVITIES

Name of Party or Agency/Company responsible for BMP Maintenance: One Hanson LLC
If different from above, identify each of the parties responsible for Source Control Activities and attach to this report. (e.g., sweeping, litter pick up, landscape maintenance, if a part of the BMP)

Address: 1484 Prince Edward Way,
Sunnyvale, CA 94087
Phone: (408) 431-4694
E-mail: (Bertrand Irissou) Bertrand@irissou.com

Structural BMP / Source Control Measure Descriptions	Date When BMP Began Operation	Proposed Maintenance Schedule (daily, weekly, quarterly, etc.) and description of maintenance activities
Biotreatment Pond	At start of position	Once in the dry season: Inspect unit and remove sediments or replace cartridges as needed.
		Once in the rainy season: Inspect unit and remove sediments or replace cartridges as needed.
		Once after every major storm: Remove sediments and replace cartridges as needed.
Sweeping/Litter Removal	At start of position	Monthly: Inspect and sweep parking lots.

E. SELF-INSPECTION PROGRAM DESCRIPTION

Name of Party or Agency/Company responsible for Self-Inspections: One Hanson LLC
If different from above, identify the party responsible for Inspections and attach to this report.

Address: 1484 Prince Edward Way,
Sunnyvale, CA 94087
Phone: (408) 431-4694
E-mail: (Bertrand Irissou) Bertrand@irissou.com

Description of Items for Self Inspection (e.g. BMP, non-storm water discharges, BMP maintenance actions, soil erosion, and others as applicable to site)	**Self-Inspection Schedule**
Biotreatment Pond	
Remove obstructions, debris and trash from bioretention area and dispose of properly.	Monthly, or as needed after storm events.
Inspect bioretention area for ponded water. If ponded water does not drain within 2-3 days, till and replace the surface soil and replant.	Monthly, or as needed after storm events.
Inspect inlets for channels, soil exposure or other evidence of erosion. Clear obstructions and remove sediment.	Monthly, or as needed after storm events.
Remove and replace all dead and diseased vegetation.	Twice a year.
Maintain vegetation and irrigation system. Prune and weed to keep bioretention area neat and orderly in appearance. Remove and or replace any dead plants.	Twice a year.
Check that mulch is at appropriate depth (2 inches per soil specifications) and replenish as necessary before wet season begins.	Monthly

F. EMPLOYEE TRAINING PROGRAM

Name of Party or Agency/Company responsible for training: One Hanson LLC
If different from above, identify party responsible for training and attach to this report.

Address: 1484 Prince Edward Way,
Sunnyvale, CA 94087
Phone: (408) 431-4694
E-mail: (Bertrand Irissou) Bertrand@irissou.com

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)
Building maintenance staff will be trained to comply with the storm water inlet labels painted with the logo "No Dumping/Flows to Bay". This educational measure is intended to prevent unlawful dumping of waste materials, such as motor oil, into the storm drains.	At start of position	Building Maintenance Staff.
Maintenance staff will be trained to in the maintenance of the plants and use pesticides as a last resort. When pesticides must be used, maintenance staff will be trained to do so with the least impact.	At start of position	Building Maintenance Staff

G. RECORD KEEPING

The owner shall be responsible for record keeping of all inspection and maintenance reports.

The types of records kept shall be:

1. Biotreatment Pond:
    a. Inspection Report (Appendix I)

H. RESPONSIBLE PARTY

The party responsible for maintenance, inspections, and record keeping of the storm water measures contained within this report shall be the property owner-of-record:

Contact: Bertrand Irissou
One Hanson LLC
1484 Prince Edward Way,
Sunnyvale, CA 94087

(408) 431-4694
Bertrand@irissou.com

APPENDIX A

C.3 DATA FORM

City of Milpitas – Stormwater Requirements C.3 Data Form
Santa Clara Valley Urban Run-Off Pollution Prevention Program

Which Projects Must Comply with Stormwater Requirements?

All projects that create and/or replace **10,000 sq. ft.** or more of impervious surface on the project site must fill out this worksheet and submit it with the development project application.

All restaurants, auto service facilities, retail gasoline outlets, and uncovered parking lot projects (stand-alone or part of another development project, including the top uncovered portion of parking structures) that create and/or replace **5,000 sq. ft.** or more of impervious surface on the project site must also fill out this worksheet.

Interior remodeling projects, routine maintenance or repair projects such as re-roofing and re-paving, and single family homes that are not part of a larger plan of development are **NOT** required to complete this worksheet.

What is an Impervious Surface?

An impervious surface is a surface covering or pavement that prevents the land's natural ability to absorb and infiltrate rainfall/stormwater. Impervious surfaces include, but are not limited to rooftops, walkways, paved patios, driveways, parking lots, storage areas, impervious concrete and asphalt, and any other continuous watertight pavement or covering. Pervious pavement, underlain with pervious soil or pervious storage material (e.g., drain rock), that infiltrates rainfall at a rate equal to or greater than surrounding unpaved areas OR that stores and infiltrates the water quality design volume specified in Provision C.3.d of the Municipal Regional Stormwater Permit (MRP) is not considered an impervious surface.

For More Information

For more information regarding selection of Best Management Practices for stormwater pollution prevention or stormwater treatment in Santa Clara County: http://www.scvurppp-w2k.com/c3_handbook_2012.shtml

1. Project Information

Project Name: PROPOSED SELF STORAGE **APN #** 022-31-020

Project Address: 1 HANSON COURT MILPITAS, CA

Cross Streets: HANSON CT & N MILPITAS BLVD

Applicant/Developer Name: Nektarios Matheou, Kier & Wright

Project Phase(s): 1 **of** 1 **Engineer:** Kier & Wright

Project Type (Check all that apply): ☑ New Development ☐ Redevelopment

☐ Residential ☐ Commercial ☑ Industrial ☐ Mixed Use ☐ Public ☐ Institutional

☐ Restaurant ☐ Uncovered Parking ☐ Retail Gas Outlet ☐ Auto Service (SIC code)_______ (5013-5014, 5541, 7532-7534, 7536-7539)

☐ Other ____________

Project Description: Redevelop an existing 4.26 acre concrete yard into a self storage site

Project Watershed/Receiving Water (creek, river, or bay): Calera Creek

2. Project Size

a. Total Site Area: 4.5 acre	b. Total Site Area Disturbed: 4.27 acre (including clearing, grading, or excavating)			
	Existing Area (ft^2)	Proposed Area (ft^2)		Total Post-Project Area (ft^2)
		Replaced	New	
Impervious Area				
Roof	14,590	14,590	86,308	100,898
Parking	0	0	51,444	51,444
Sidewalks and Streets	171,286	1,468	0	1,468
c. Total Impervious Area	185,876	16,058	137,752	153,810
d. Total new and replaced impervious area		153,810		
Pervious Area				
Landscaping	0	0	32,066	32,066
Pervious Paving	0	0	0	0
Other (e.g. Green Roof)	0	0	0	0
e. Total Pervious Area	0	0	32,066	32,066
f. Percent Replacement of Impervious Area in Redevelopment Projects (Replaced Total Impervious Area ÷ Existing Total Impervious Area) x 100% = 9 %				

3. State Construction General Permit Applicability:

a. Is #2.b. equal to one acre or more?

☑ Yes, applicant must obtain coverage under the State Construction General Permit (i.e., file a Notice of Intent and prepare a Stormwater Pollution Prevention Plan) (see www.swrcb.ca.gov/water_issues/programs/stormwater/construction.shtml for details).

☐ No, applicant does not need coverage under the State Construction General Permit.

4. MRP Provision C.3 Applicability:

a. Is #2.d. equal to **10,000** sq. ft. or more, or **5,000** sq. ft. or more for restaurants, auto service facilities, retail gas outlets, and uncovered parking?

☑ Yes, C.3. source control, site design, and treatment requirements apply.

☐ No, C.3. source control and site design requirements may apply – check with local agency

b. Is #2.f. equal to 50% or more?

☐ Yes, C.3. requirements (site design, source control, as appropriate, and stormwater treatment) apply to entire site.

☑ No, C.3. requirements only apply to impervious area created and/or replaced.

5. Hydromodification Management (HM) Applicability:

a. Does project create and/or replace one acre or more of impervious surface AND is the total post-project impervious area greater than the pre-project (existing) impervious area?

☑ Yes (continue) ☐ No – exempt from HM, go to page 3

b. Is the project located in an area of HM applicability (green area) on the HM Applicability Map? (www.scvurppp-w2k.com/hmp_maps.htm)

☐ Yes, project must implement HM requirements

☑ No, project is exempt from HM requirements

6. Selection of Specific Stormwater Control Measures:

Site Design Measures

- ☐ Minimize land disturbed
- ☐ Minimize impervious surfaces
- ☐ Minimum-impact street or parking lot design
- ☐ Cluster structures/ pavement
- ☐ Disconnected downspouts
- ☐ Pervious pavement
- ☐ Green roof
- ☐ Microdetention in landscape
- ☑ Other self-treating area
- ☐ Self-retaining area
- ☐ Rainwater harvesting and use (e.g., rain barrel, cistern connected to roof drains) [1]
- ☐ Preserved open space: _______ ac. or sq. ft .(circle one)
- ☐ Protected riparian and wetland areas/buffers (Setback from top of bank: _______ft.)
- ☐ Other _______________

Source Control Measures

- ☐ Alternative building materials
- ☐ Wash area/racks, drain to sanitary sewer[2]
- ☑ Covered dumpster area, drain to sanitary sewer[2]
- ☐ Sanitary sewer connection or accessible cleanout for swimming pool/spa/fountain[2]
- ☐ Beneficial landscaping (minimize irrigation, runoff, pesticides and fertilizers; promotes treatment)
- ☐ Outdoor material storage protection
- ☐ Covers, drains for loading docks, maintenance bays, fueling areas
- ☐ Maintenance (pavement sweeping, catch basin cleaning, good housekeeping)
- ☑ Storm drain labeling
- ☐ Other _______________

Treatment Systems

- ☐ None (all impervious surface drains to self-retaining areas)

LID Treatment

- ☐ Rainwater harvest and use (e.g., cistern or rain barrel sized for C.3.d treatment)
- ☐ Infiltration basin
- ☐ Infiltration trench
- ☐ Exfiltration trench
- ☐ Underground detention and infiltration system (e.g. pervious pavement drain rock, large diameter conduit)

Biotreatment [3]

- ☑ Bioretention area
- ☐ Flow-through planter
- ☐ Tree box with bioretention soils
- ☐ Other _______________

Other Treatment Methods

- ☐ Proprietary tree box filter[4]
- ☐ Media filter (sand, compost, or proprietary media)[4]
- ☐ Vegetated filter strip[5]
- ☐ Dry detention basin[5]
- ☐ Other _______________

Flow Duration Controls for Hydromodification Management (HM)

- ☐ Detention basin
- ☐ Underground tank or vault
- ☑ Bioretention with outlet control
- ☐ Other _______________

[1] Optional site design measure; does not have to be sized to comply with Provision C.3.d treatment requirements.
[2] Subject to sanitary sewer authority requirements.
[3] Biotreatment measures are allowed only with completed feasibility analysis showing that infiltration and rainwater harvest and use are infeasible.
[4] These treatment measures are only allowed if the project qualifies as a "Special Project".
[5] These treatment measures are only allowed as part of a multi-step treatment process.

7. Treatment System Sizing for Projects with Treatment Requirements

Indicate the hydraulic sizing criteria used and provide the calculated design flow or volume:

Treatment System Component	Hydraulic Sizing Criteria Used[3]	Design Flow or Volume (cfs or cu.ft.)
Biotreatment Pond	3	7,809 cu. ft.

[3]Key: 1a: Volume – WEF Method
1b: Volume – CASQA BMP Handbook Method
2a: Flow – Factored Flood Flow Method
2b: Flow – CASQA BMP Handbook Method
2c: Flow – Uniform Intensity Method
3: Combination Flow and Volume Design Basis

8. Alternative Certification: Was the treatment system sizing and design reviewed by a qualified third-party professional that is not a member of the project team or agency staff?

☐ Yes ☑ No Name of Reviewer: ______________________

9. Operation & Maintenance Information

A. Property Owner's Name: One Hanson LLC

B. Responsible Party for Stormwater Treatment/Hydromodification Control O&M:

a. Name: Bertrand Irissou
b. Address: 1484 Prince Edward Way, Sunnyvale, CA 94087
c. Phone/E-mail: 408.431.4694

This section to be completed by City of Milpitas staff.

O&M Responsibility Mechanism

Indicate how responsibility for O&M is assured. Check all that apply:

☑ O&M Agreement

☐ Other mechanism that assigns responsibility (describe below):

____

____

Reviewed:

Planning Department

Planning Division: ________

Other (Specify): ________

Public Works Department

Land Development: ________

Other (Specify): ________

APPENDIX B

INFILTRATION/HARVESTING INFEASIBILITY WORKSHEET

Rainwater Harvesting and Use Feasibility Worksheet

Municipal Regional Stormwater Permit (MRP)

Stormwater Controls for Development Projects

Complete this worksheet for all **C.3 Regulated Projects*** for which the project density exceeds the **screening density*** provided by municipal staff. Use this worksheet to determine the feasibility of treating the **C.3.d amount of runoff*** with rainwater harvesting and use for indoor, non-potable water uses. Where it is infeasible to treat the C.3d amount of runoff with either harvesting and use or infiltration, stormwater may be treated with **biotreatment*** measures. See Glossary (Attachment 1) for definitions of terms marked with an asterisk (*).

Complete this worksheet for the entire project area. If the project includes one or more buildings that each individually has a roof area of 10,000 square feet or more, complete a separate copy of this form for each of these buildings.

1. Enter Project Data.

1.1	Project Name:	Self Storage	
1.2	Project Address:	1 Hanson Court, Milpitas, CA	
1.3	Applicant/Agent Name:	Nektarios Matheou	
1.4	Applicant/Agent Address:	3350 Scott Blvd Santa Clara, CA	

(For projects with a potential non-potable water use other than toilet flushing, skip to Question 5.1)

1.5	Project Type: Commercial — If residential or mixed use, enter # of dwelling units:		
1.6	Enter square footage of non-residential interior floor area.:	99,976	
1.7	**Potential rainwater capture area*:**	153,810	sq.ft.
1.8	If it is a **Special Project***, indicate the percentage of **LID treatment*** reduction: *(Item 1.8 applies only to entire project evaluations, not individual roof area evaluations.)*	-	percent
1.9	Total potential rainwater capture area that will require LID treatment: *(This is the total rain capture area remaining after any Special Project LID treatment reduction is applied.)*	137,752	sq.ft.

2. Calculate Area of Self-Treating Areas, Self-Retaining Areas, and Areas Contributing to Self-Retaining Areas.

(For areas within the Potential Rain Capture Area only)

2.1	Enter square footage of any **self-treating areas*** in the area that is being evaluated:		sq.ft.
2.2	Enter square footage of any **self-retaining areas*** in the area that is being evaluated:		sq.ft.
2.3	Enter the square footage of areas contributing runoff to **self-retaining area***:		sq.ft.
2.4	TOTAL of Items 2.1, 2.2, and 2.3:	-	sq.ft.

3. Subtract credit for self-treating/self-retaining areas from area requiring treatment.

3.1	Subtract the TOTAL in Item 2.4 from the potential rainwater capture area in Item 1.9:	153,810	sq.ft.
3.2	Convert the remaining area required for treatment in Item 3.1 from square feet to acres:	3.53	acres

4. Determine feasibility of use for toilet flushing based on demand

4.1	Project's dwelling units per acre of adjusted potential rain capture area (Divide the number in 1.5 by the number in 3.2)		dwelling units/acre
4.2	Non-residential interior floor area per acre of adjusted potential rain capture area (Divide the number in 1.6 by the number in 3.2)	28,314	Int. non-res. floor area/acre

*Note: formulas in Items 4.1 and 4.2 are set up, respectively, for a residential or a non-residential project. Do not use these pre-set formulas for mixed use projects. **For mixed use projects**, evaluate the residential toilet flushing demand based on the dwelling units per acre for the residential portion of the project (use a prorated acreage, based on the percentage of the project dedicated to residential use). Then evaluate the commercial toilet flushing demand per acre for the commercial portion of the project (use a prorated acreage, based on the percentage of the project dedicated to commercial use).*

4.3	Refer to the applicable countywide table in Attachment 2. Identify the number of dwelling units per impervious acre needed in your Rain Gauge Area to provide the toilet flushing demand required for rainwater harvest feasibility.		dwelling units/acre
4.4	Refer to the applicable countywide table in Attachment 2. Identify the square feet of non-residential interior floor area per impervious acre needed in your Rain Gauge Area to provide the toilet flushing demand required for rainwater harvest feasibility.	70,000	int. non-res. floor area/acre

Check "Yes" or "No" to indicate whether the following conditions apply. If "Yes" is checked for any question, then rainwater harvesting and use is infeasible. As soon as you answer "Yes", you can skip to Item 6.1. If "No" is checked for all items, then rainwater harvesting and use is feasible and you must harvest and use the C.3.d amount of stormwater, unless you infiltrate the C.3.d amount of stormwater.*

4.5	Is the project's number of dwelling units per acre of adjusted area requiring treatment (listed in Item 4.1) LESS than the number identified in Item 4.3?	☐ Yes	☐ No
4.6	Is the project's square footage of non-residential interior floor area per acre of adjusted area requiring treatment (listed in Item 4.2) LESS than the number identified in Item 4.4?	☒ Yes	☐ No

5. Determine feasibility of rainwater harvesting and use based on factors other than demand.

5.1	Does the requirement for rainwater harvesting and use at the project conflict with local, state, or federal ordinances or building codes?	☐ Yes	☒ No
5.2	Would the technical requirements cause the harvesting system to exceed 2% of the Total Project Cost, or has the applicant documented economic hardship in relation to maintenance costs? (If so, attach an explanation.)	☐ Yes	☒ No
5.3	Do constraints, such as a slope above 10% or lack of available space at the site, make it infeasible to locate on the site a cistern of adequate size to harvest and use the C.3.d amount of water? (If so, attach an explanation.)	☐ Yes	☒ No
5.4	Are there geotechnical/stability concerns related to the surface (roof or ground) where a cistern would be located that make the use of rainwater harvesting infeasible? (If so, attach an explanation.)	☐ Yes	☒ No
5.5	Does the location of utilities, a septic system and/or **heritage trees*** limit the placement of a cistern on the site to the extent that rainwater harvesting is infeasible? (If so, attach an explanation.)	☐ Yes	☒ No

Note 1: It is assumed that projects with significant amounts of landscaping will either treat runoff with landscape dispersal (self-treating and self-retaining areas) or will evaluate the feasibility of havesting and using rainwater for irrigation using the curves in Appendix F of the LID Feasibility Report.

6. Results of Feasibility Determination

		Infeasible	Feasible
6.1	Based on the results of the feasibility analysis in Item 4.4 and Section 5, rainwater harvesting/use is (check one):	☒	☐

➔ If "FEASIBLE" is indicated for Item 6.1 the amount of stormwater requiring treatment must be treated with harvesting/use, unless it is infiltrated into the soil.

*➔ If "INFEASIBLE" is checked for Item 6.1, then the applicant may use appropriately designed **bioretention** *,1 facilities for compliance with C.3 treatment requirements. If Ksat > 1.6 in./hr., and infiltration is unimpeded by subsurface conditions, then the bioretention facilities are predicted to infiltrate 80% or more average annual runoff. If Ksat < 1.6, maximize infiltration of stormwater by using bioretention if site conditions allow, and remaining runoff will be discharged to storm drains via facility underdrains. If site conditions preclude infiltration, a lined bioretention area or flow-through planter may be used.*

____
Applicant (Print)

______________________________ ______________________________
Applicant (Sign) Date

* See definitions in Glossary (Attachment 1)

Infiltration/Harvesting and Use Feasibility Screening Worksheet

*Apply these screening criteria for **C.3 Regulated Projects*** required to implement Provision C.3 stormwater treatment requirements. See the Glossary (Attachment 1) for definitions of terms marked with an asterisk (*). Contact municipal staff to determine whether the project meets **Special Project*** criteria. If the project meets Special Project criteria, it may receive LID treatment reduction credits.*

1. **Applicant Info**

Site Address: 1 Hanson Court Milpitas, CA , CA APN: 022-31-020

Applicant Name: Nektarios Matheou Phone No.: 408.727.6665

Mailing Address: 3350 Scott Blvd Santa Clara, CA

2. **Feasibility Screening for Infiltration**

Do site soils either (a) have a **saturated hydraulic conductivity*** (Ksat) that will NOT allow infiltration of 80% of the annual runoff (that is, the Ksat is LESS than 1.6 inches/hour), or, if the Ksat rate is not available, (b) consist of Type C or D soils?[1]

☒ Yes (continue) ☐ No – complete the Infiltration Feasibility Worksheet. If infiltration of the C.3.d amount of runoff is found to be feasible, there is no need to complete the rest of this screening worksheet.

3. **Recycled Water Use**

Check the box if the project is installing and using a recycled water plumbing system for non-potable water use.

☐ The project is installing a recycled water plumbing system, and installation of a second non-potable water system for harvested rainwater is impractical, and considered infeasible due to cost considerations. Skip to Section 6.

4. **Calculate the Potential Rainwater Capture Area* for Screening of Harvesting and Use**

Complete this section for the entire project area. If rainwater harvesting and use is infeasible for the entire site, and the project includes one or more buildings that each have an individual roof area of 10,000 sq. ft. or more, then complete Sections 4 and 5 of this form for each of these buildings.

4.1 Table 1 for (check one): ☒ The whole project ☐ Area of 1 building roof (10,000 sq.ft. min.)

Table 1: Calculation of the Potential Rainwater Capture Area*
The Potential Rainwater Capture Area may consist of either the entire project area or one building with a roof area of 10,000 sq. ft. or more.

	1	2	3	4
	Pre-Project Impervious surface[2] (sq.ft.), if applicable	Proposed Impervious Surface[2] (IS), in sq. ft.		Post-project landscaping (sq.ft.), if applicable
		Replaced[3] IS	Created[4] IS	
a. Enter the totals for the area to be evaluated:	185,876	16,058	137,752	32,066
b. Sum of replaced and created impervious surface:	N/A	153,810		N/A
c. Area of existing impervious surface that will NOT be replaced by the project.	0	N/A		N/A

[1] Base this response on the site-specific soil report, if available. If this is not available, consult soil hydraulic conductivity maps in Attachment 3.
[2], Enter the total of all impervious surfaces, including the building footprint, driveway(s), patio(s), impervious deck(s), unroofed porch(es), uncovered parking lot (including top deck of parking structure), impervious trails, miscellaneous paving or structures, and off-lot impervious surface (new, contiguous impervious surface created from road projects, including sidewalks and/or bike lanes built as part of new street). Impervious surfaces do NOT include vegetated roofs or pervious pavement that stores and infiltrates rainfall at a rate equal to immediately surrounding, unpaved landscaped areas, or that stores and infiltrates the **C.3.d amount of runoff***.
[3] "Replaced" means that the project will install impervious surface where existing impervious surface is removed.
[4] "Created" means the project will install new impervious surface where there is currently no impervious surface.
* For definitions, see Glossary (Attachment 1).

4.2 Answer this question ONLY if you are completing this section for the entire project area. If existing impervious surface will be replaced by the project, does the area to be replaced equal 50% or more of the existing area of impervious surface? *(Refer to Table 1, Row "a". Is the area in Column 2 > 50% of Column 1?)*

☐ Yes, C.3. stormwater treatment requirements apply to areas of impervious surface that will remain in place as well as the area created and/or replaced. This is known as the 50% rule.

☒ No, C.3. requirements apply only to the impervious area created and/or replaced.

4.3 Enter the square footage of the **Potential Rainwater Capture Area***. If you are evaluating only the roof area of a building, or you answered "no" to Question 4.2, this amount is from Row "b" in Table 1. If you answered "yes" to Question 4.2, this amount is the sum of Rows "b" and "c" in Table 1.:

153,810 square feet.

4.4 Convert the measurement of the **Potential Rainwater Capture Area*** from square feet to acres (divide the amount in Item 4.3 by 43,560):

3.53 acres.

5. Feasibility Screening for Rainwater Harvesting and Use

5.1 Use of harvested rainwater for landscape irrigation:

Is the onsite landscaping LESS than 2.5 times the size of the **Potential Rainwater Capture Area*** (Item 4.3)? (Note that the landscape area(s) would have to be contiguous and within the same Drainage Management Area to use harvested rainwater for irrigation via gravity flow.)

☒ Yes (continue) ☐ No – Direct runoff from impervious areas to **self-retaining areas*** OR refer to Table 11 and the curves in Appendix F of the LID Feasibility Report to evaluate feasibility of harvesting and using the C.3.d amount of runoff for irrigation.

5.2 Use of harvested rainwater for toilet flushing or non-potable industrial use:

a. Residential Projects: Proposed number of dwelling units:________________

Calculate the dwelling units per impervious acre by dividing the number of dwelling units by the acres of the **Potential Rainwater Capture Area*** in Item 4.4. Enter the result here:

____________)

Is the number of dwelling units per impervious acre LESS than 100 (assuming 2.7 occupants/unit)?

☐ Yes (continue) ☐ No – complete the Harvest/Use Feasibility Worksheet.

b. Commercial/Industrial Projects: Proposed interior floor area: 99,976 (sq. ft.)

Calculate the proposed interior floor area (sq.ft.) per acre of impervious surface by *dividing the interior floor area (sq.ft.) by the acres of the* ***Potential Rainwater Capture Area**** *in Item 4.4. Enter the result here:* 28,314

Is the square footage of the interior floor space per impervious acre LESS than 70,000 sq. ft.?

☒ Yes (continue) ☐ No – complete the Harvest/Use Feasibility Worksheet

c. School Projects: Proposed interior floor area:________________ (sq. ft.)

Calculate the proposed interior floor area per acre of impervious surface by *dividing the interior floor area (sq.ft.) by the acres of the* ***Potential Rainwater Capture Area**** *in Item 4.4 . Enter the result here:*

____________.

Is the square footage of the interior floor space per impervious acre LESS than 21,000 sq. ft.?

☐ Yes (continue) ☐ No – complete the Harvest/Use Feasibility Worksheet

* For definitions, see Glossary (Attachment 1).

d. Mixed Commercial and Residential Use Projects

- Evaluate the residential toilet flushing demand based on the dwelling units per impervious acre for the residential portion of the project, following the instructions in Item 5.2.a, except you will use a prorated acreage of impervious surface, based on the percentage of the project dedicated to residential use.
- Evaluate the commercial toilet flushing demand per impervious acre for the commercial portion of the project, following the instructions in Item 5.2.a, except you will use a prorated acreage of impervious surface, based on the percentage of the project dedicated to commercial use.

e. Industrial Projects: Estimated non-potable water demand (gal/day): ____________________________

Is the non-potable demand LESS than 2,400 gal/day per acre of the Potential Rainwater Capture Area?

☐ Yes (continue) ☐ No – refer to the curves in Appendix F of the LID Feasibility Report to evaluate feasibility of harvesting and using the C.3.d amount of runoff for industrial use.

6. Use of Biotreatment

If only the "Yes" boxes were checked for all questions in Sections 2 and 5, or the project will have a recycled water system for non-potable use (Section 3), then the applicant may use appropriately designed bioretention facilities for compliance with C.3 treatment requirements. The applicant is encouraged to maximize infiltration of stormwater if site conditions allow.

7. Results of Screening Analysis

Based on this screening analysis, the following steps will be taken for the project (check all that apply):

☒ Implement biotreatment measures (such as an appropriately designed bioretention area).

☐ Conduct further analysis of infiltration feasibility by completing the Infiltration Feasibility Worksheet.

☐ Conduct further analysis of rainwater harvesting and use (check one):

- ☐ Complete the Rainwater Harvesting and Use Feasibility Worksheet for:
  - ☐ The entire project
  - ☐ Individual building(s), if applicable, describe:___________________________________
- ☐ Evaluate the feasibility of harvesting and using the C.3.d amount of runoff for irrigation, based on Table 11 and the curves in Appendix F of the LID Feasibility Report
- ☐ Evaluate the feasibility of harvesting and using the C.3.d amount of runoff for non-potable industrial use, based on the curves in Appendix F of the LID Feasibility Report.

* For definitions, see Glossary (Attachment 1).

APPENDIX C

SPECIAL PROJECTS WORKSHEET
(NOT APPLICABLE)

APPENDIX D

SOIL PROPERTIES

(GEOTECHNICAL REPORT)

CORNERSTONE EARTH GROUP

Type of Services	Geotechnical Investigation
Project Name	Hanson Self Storage
Location	1 Hanson Court Milpitas, California
Client	Irissou Family Partners, LP
Project Number	726-1-3
Date	October 15, 2014

CORNERSTONE
EARTH GROUP

Type of Services	**Geotechnical Investigation**
Project Name	**Hanson Self Storage**
Location	**1 Hanson Court** **Milpitas, California**
Client	**Irissou Family Partners, LP**
Client Address	**1484 Prince Edward Way** **Sunnyvale, California 94085**
Project Number	**726-1-3**
Date	**October 15, 2014**

REGISTERED PROFESSIONAL ENGINEER
NICHOLAS S. DEVLIN
No. 75613
Exp. 6-30-2016
CIVIL
STATE OF CALIFORNIA

Prepared by **Nicholas S. Devlin, P.E.**
Project Engineer

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

TABLE OF CONTENTS

CORNERSTONE EARTH GROUP

Type of Services	**Geotechnical Investigation**
Project Name	**Hanson Self Storage**
Location	**1 Hanson Court Milpitas, California**

SECTION 1: INTRODUCTION

This geotechnical report was prepared for the sole use of Irissou Family Partners, LP for the Hanson Self Storage project located at 1 Hanson Court in Milpitas, 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 plan titled, "Preliminary Site Plan, Proposed Self Storage, 1 Hanson Court, Milpitas, CA", prepared by Cubix Construction Company, dated March 26, 2014.
- A plan titled, "Floor Plans, Proposed Self Storage, 1 Hanson Court, Milpitas, CA", prepared by Cubix Construction Company, dated March 26, 2014.

1.1 PROJECT DESCRIPTION

The project will consist of six storage buildings (Buildings A through F) and a Manager's Building located on the approximately 4.3-acre site (Figure 2). Buildings A, B and D through F are one story and Building C and the Manager's Building are two stories. We understand that Buildings A through F will be of concrete tilt-up construction and have a footprint of approximately 95,100 square feet; the Manager's Building will be of wood-frame construction with a footprint of approximately 1,100 square feet. We also understand that up to 3 feet of fill will be placed at the site as part of the proposed improvements. Structural loads were not provided at the time of this report and are anticipated to be typical for these type of structures.

1.2 SCOPE OF SERVICES

Our scope of services was presented in our proposal dated June 27, 2014 and our confirmation of requested services dated July 1, 2014. Our services consisted of field and laboratory programs to evaluate physical and engineering properties of the subsurface soil, 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 (5) borings drilled on July 30, 2014, with truck-mounted hollow-stem auger drilling equipment and nine (9) Cone Penetration Test (CPT) soundings advanced on July 29, 2014, and August 1, 2014 (Figure 2). Exploratory Borings EB-1 through EB-5 were drilled to depths between 20 and 40 feet, and the CPT soundings were advanced to depths of between 50 and 86½ feet. Seismic shear wave velocity measurements were collected from CPT-1. Exploratory Borings EB-1 through EB-5 were drilled adjacent to CPT-1 through CPT-5 for evaluation of physical samples to correlate with soil behavior.

The borings and CPT soundings were backfilled with cement grout in conformance with the Santa Clara Valley Water District (SCVWD) requirements; exploration permits were obtained as required by local jurisdictions. The approximate locations of our exploratory borings and CPT soundings are shown on the Site Plan, Figure 2. Details regarding our field program are presented on the boring and CPT logs 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. Laboratory testing included moisture content, dry density, Plasticity Index (PI), corrosion, and triaxial compressive strength. Details regarding our laboratory program are included in Appendix B.

1.5 CORROSION EVALUATION

Two (2) samples from our borings at depths between 1½ and 4½ feet were tested for saturated resistivity, pH, and soluble sulfates and chlorides. In general, the on-site soil can be characterized as severely to very severely corrosive to buried metal, and non-corrosive to buried concrete.

1.6 ENVIRONMENTAL SERVICES

Cornerstone Earth Group is also providing environmental services for the project. Findings and recommendations from the environmental investigations are provided in separate reports.

SECTION 2: REGIONAL SETTING

2.1 GEOLOGICAL SETTING

The site is located within the Santa Clara Valley, which is a broad alluvial plane between the Santa Cruz Mountains to the southwest and west, and the Diablo Range to the northeast. The San Andreas Fault system, including the Monte Vista-Shannon Fault, exists within the Santa Cruz Mountains and the Hayward and Calaveras Fault systems exist within the Diablo Range. Alluvial soil thickness in the vicinity of the site is more than 500 feet (Rogers & Williams, 1974).

2.2 REGIONAL SEISMICITY

The San Francisco Bay area is one of the most seismically active areas in the Country. While seismologists cannot predict earthquake events, the U.S. Geological Survey's Working Group on California Earthquake Probabilities 2007 estimates there is a 63 percent chance of at least one magnitude 6.7 or greater earthquake occurring in the Bay Area region between 2007 and 2036. As seen with damage in San Francisco and Oakland due to the 1989 Loma Prieta earthquake that was centered about 50 miles south of San Francisco, significant damage can occur at considerable distances. Higher levels of shaking and damage would be expected for earthquakes occurring at closer distances.

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

Table 1: Approximate Fault Distances

Fault Name	Distance	
	(miles)	(kilometers)
Hayward (Southeast Extension)	2.1	3.4
Hayward (Total Length)	4.5	7.2
Calaveras	7.5	12.0
Monte Vista-Shannon	13.4	21.4

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 SITE BACKGROUND

The site is occupied by four commercial/industrial buildings with Portland cement concrete (PCC) pavement throughout the surrounding portions of the site. According to the referenced site plan, Building A is located along the northern side of the site, Building B is located along the eastern side of the site, Building E is located along the western side of the site, Buildings C and D are located in the central portion of the site, and the Manager's Building is located in the northeastern portion of the site. The site is bounded by Hanson Court to the east, a commercial property to the south, a Santa Clara County Transit Authority easement, and a SCVWD flood control channel (Calera Creek) to the north. The site is relatively level with a slight downward slope toward the north. The elevation of the site is approximately 14 feet above Mean Sea Level (MSL).

3.2 SURFACE DESCRIPTION

At the time of our subsurface exploration, the ground surface at the site was covered with PCC pavement. The concrete ranged from between 5½ and 10 inches thick at the locations of our exploratory borings. Based on the thicknesses of PCC pavement encountered at the boring locations, we estimate an average thickness of approximately 9 inches. The pavement was observed to be supported on subgrade soil consisting of clay and clayey sand. Distress to the concrete consisting of significant cracking was observed at the time of our subsurface exploration.

3.3 SUBSURFACE CONDITIONS

Materials encountered during our subsurface exploration included a pavement section, undocumented fill, and alluvium. Generalized descriptions of the units encountered are provided below. Additional descriptions are provided on the boring and CPT logs included in Appendix A.

Pavement – The pavement section encountered during our subsurface exploration consisted of approximately 5½ to 10 inches of Portland cement concrete.

Fill – Fill was encountered in EB-1 through EB-4 from below the pavement to depths of approximately 3 feet below the existing grade. The fill encountered generally consisted of moist, loose, sand and clayey sand with fine to coarse-grained gravel, and moist, stiff to very stiff, lean sandy clay and very stiff clay. Fill was not encountered in the remaining borings performed for this investigation.

Alluvium – Alluvium was encountered within our subsurface explorations from below the fill to about 86½ feet, the maximum depth explored. The alluvium generally consisted of moist to saturated, medium stiff to stiff, clay and sandy and silty lean clay; and moist to saturated, loose to dense, sand and silty and clayey sand with fine to coarse-grained gravel.

3.3.1 Plasticity/Expansion Potential

We performed six (6) Plasticity Index (PI) tests on representative samples. Test results were used to evaluate expansion potential of surficial soil, and the plasticity of the fines in potentially liquefiable layers. The surficial test resulted in a PI of 16, indicating a low to moderate expansion potential to wetting and drying cycles.

3.3.2 In-Situ Moisture Contents

Laboratory testing indicated that the in-situ moisture contents of the clayey soil within the upper 10 feet are up to 16 percent over the estimated laboratory optimum moisture, and in-situ moisture contents of the sandy soil within the upper 10 feet range from about 15 to 17 percent over the estimated laboratory optimum moisture.

3.3.3 Sulfate Contents

Laboratory testing indicated that the soluble sulfate contents range from 37 to 71 parts per million, indicating negligible corrosion potential to buried concrete.

3.4 GROUND WATER

Ground water was encountered in our borings at depths ranging from about 8 to 14 feet below current grades. The Seismic Hazard Zone Report (CGS, Milpitas 7.5 Minute Quadrangle, 2004) indicates the historic high ground water in the area to be about 5 feet below the ground surface, which we used for our liquefaction analyses and recommend be used for planning purposes.

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

3.5 CORROSION SCREENING

We tested two samples collected at depths of 1½ and 4½ feet for resistivity, pH, soluble sulfates, and chlorides. The laboratory test results are summarized in Table 2.

Table 2: Summary of Corrosion Test Results

Sample/Test Location Number	Depth (feet)	Soil pH	Minimum Resistivity (1) (ohm-cm)	Chloride (mg/kg)	Sulfate (% dry wt)
EB-3	1½	8.1	975	4	0.0037
EB-5	4½	7.9	1,130	18	0.0071

Note: (1) Laboratory resistivity measured at 100% saturation

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

Based on the laboratory test results summarized in Table 2, the soil is considered severely to very severely corrosive to buried metallic improvements (Palmer, 1989). In accordance with the 2013 CBC, Chapter 19, Section 1904A.2:

> *Concrete mixtures shall conform to the most restrictive maximum water-cementitious materials ratios, maximum cementitious admixtures, minimum air-entrained and minimum specified concrete compressive strength requirements of ACI 318 based on the exposure classes assigned in Section 1904A.1.*

We recommend that a corrosion engineer be retained to confirm the information provided and for additional recommendations, as required.

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 or a Santa Clara County Fault Hazard 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) was estimated for analysis using $PGA_M = F_{PGA} \times PGA_G$ (Equation 11.8-1) as allowed in the 2013 California Building Code. For our liquefaction analysis, we used a PGA of 0.70g. This estimated ground shaking is for Site Class D, based on the assumption that the natural periods of the buildings will be less than 0.5 second. If the natural periods of the buildings are greater than 0.5 seconds, further geotechnical analyses will be required. Additional recommendations are provided in the "Foundations" section of this report.

4.3 LIQUEFACTION POTENTIAL

The site is within a State-designated Liquefaction Hazard Zone (CGS, Milpitas Quadrangle, 2004) as well as a Santa Clara County Liquefaction Hazard Zone (Santa Clara County, 2003). Our field and laboratory programs addressed this issue by sampling potentially liquefiable layers to depths of at least 50 feet, performing visual classification on sampled materials, evaluating CPT correlations, and performing various tests to further classify the 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. Soil most susceptible to liquefaction is loose, non-cohesive, and bedded with poor drainage, such as bedded sand and silt layers under a cohesive cap.

4.3.2 Analysis

As discussed in the "Subsurface" section above, several sand layers were encountered below the design ground water depth of 5 feet. Following the procedures in the 2008 monograph, *Soil Liquefaction During Earthquakes* (Idriss and Boulanger, 2008) 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. 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 CRR is estimated from the in-situ measurements from CPT soundings 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 unreliable in sand below ground water. The tip pressures are corrected for effective overburden stresses, taking into consideration both the ground water level at the time of exploration and the design ground water 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.

In estimating post-liquefaction settlement at the site, we have implemented a depth weighting factor proposed by Cetin (2009). Following evaluation of 49 high-quality, cyclically induced, ground settlement case histories from seven different earthquakes, Cetin proposed the use of a weighting factor based on the depth of layers. The weighting procedure was used to tune the surface observations at liquefaction sites to produce a better model fit with measured data. Aside from the better model fit it produced, the rationale behind the use of a depth weighting factor is based on the following: 1) upward seepage, triggering void ratio redistribution, and resulting in unfavorably higher void ratios for the shallower sublayers of soil layers; 2) reduced induced shear stresses and number of shear stress cycles transmitted to deeper soil layers due to initial liquefaction of surficial layers; and 3) possible arching effects due to nonliquefied soil layers. All these may significantly reduce the contribution of volumetric settlement of deeper soil layers to the overall ground surface settlement (Cetin, 2009).

The results of our CPT analyses (CPT-1 through CPT-9) are presented on Figures 4A through 4I of this report.

4.3.3 Summary

Our analyses indicate that several layers could potentially experience liquefaction triggering that could result in soil softening and post-liquefaction total settlement ranging from about ½-inch up to 3 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. In our opinion, differential settlement is anticipated to range from about ⅓-inch up to 2 inches over a horizontal distance of about 30 feet.

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. The non-liquefiable cap ranged between 8 to 21 feet thick. As previously discussed, 3 feet of engineered fill will be placed at the site. The engineered fill will increase the thickness of the non-liquefiable layer. The work of Youd and Garris (1995) indicates that a non-liquefiable cap from about 11 to 24 feet thick will be sufficient to prevent ground rupture. Based on the thickness of the existing non-liquefiable cap and the placement of 3 feet of engineered fill, the potential for ground rupture is expected to be low.

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.

Calera Creek runs along the northern site boundary approximately 40 feet north of Building A. The creek along the north side of the site consists of a concrete-lined box channel with a bottom between approximately 4 and 5 feet below the adjacent existing grade. We understand that the SCVWD plans to widen and extend the walls of the existing channel adjacent to the site. We recommend that the channel addition be designed to resist seismic earth pressures.

Based on the box design and relatively shallow depth of Calera Creek channel adjacent to the site, in our opinion, the potential for lateral spreading is low.

4.5 SEISMIC SETTLEMENT/UNSATURATED SAND SHAKING

Loose unsaturated sandy soil can settle during strong seismic shaking. We evaluated the potential for seismic compaction of the loose sandy soil encountered above the historic high ground water level based on the work by Pradell (1998). Our analyses indicate that dry sand settlement of the soil in the upper 5 feet will be less than ¼-inch after strong seismic shaking.

4.6 FLOODING

Based on the Federal Emergency Management Agency (FEMA) flood map public database, the site is located within Zone AH, defined as “Flood depths of 1 to 3 feet (usually areas of ponding); Base Flood Elevations determined” (FEMA, 2009). We recommend the project civil engineer be retained to confirm this information and verify the base flood elevation, if appropriate.

The Association of Bay Area Governments (ABAG) has compiled a database of Dam Failure Inundation Hazard Maps (ABAG, 1995). The generalized hazard maps were prepared by dam owners as required by the State Office of Emergency Services; they are intended for planning purposes only. Based on these maps, the site is not located within a dam failure inundation area.

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.

- Potential for liquefaction-induced settlement
- Presence of undocumented fill
- Shallow ground water
- Soil corrosion potential
- Re-Development Considerations

5.1.1 Potential for Liquefaction-Induced Settlement

As discussed, our liquefaction analysis indicates there is a potential for liquefaction of localized sand layers resulting in liquefaction-induced settlement on the order of ½-inch to 3 inches following a significant seismic event. Therefore, we recommend the proposed buildings be supported on ground improvement elements designed to mitigate the potential for liquefaction-induced settlement.

Assuming ground improvement is performed in the upper 20 to 25 feet and 3 feet of engineered fill is placed across the site, the potential for liquefied sand to vent to the ground surface through cracks in the surficial soil would be low. Our analysis indicates that the liquefaction-induced settlement after ground improvement is performed would be less than 1 inch, resulting in differential settlement ranging from about ¼-inch to ⅔-inch over a horizontal distance of about 30 feet. Foundations should be designed to tolerate the anticipated total and differential settlement that may occur following installation of approved ground improvement alternatives. In our opinion, it is feasible to support the proposed buildings on spread footings provided ground improvement methods (such as compacted rock columns) are performed; however, the foundations and structures will need to be designed to tolerate total and differential settlement due to combined static loads and liquefaction-induced settlement. Additional recommendations are presented in the “Foundations” section that follows.

5.1.2 Presence of Undocumented Fill

As previously discussed, approximately 3 feet of undocumented fill material consisting of very stiff sandy lean clay and loose clayey sand was encountered within EB-1 through EB-4.

Records of previous fill placement are not available at this time; therefore, whether the fill was compacted to current compaction standards is not known. Undocumented fill may be variable in thickness, density, and consistency across the site. Since the proposed structures can likely be supported on shallow foundations over ground improvement, we recommend the ground improvement program also be incorporated to mitigate the undocumented fill. Recommendations for remedial grading are presented in the “Earthwork” section of this report.

5.1.3 Shallow Ground Water

Historic high ground water is mapped at about 5 feet below the ground surface. Our investigation encountered groundwater between 8 and 14 feet below the existing ground surface; however, there may not have been a sufficient amount of time for the groundwater to stabilize during our investigation.

Our experience with similar conditions indicate that shallow ground water could significantly impact grading and underground construction. These impacts typically consist of potentially wet and unstable pavement subgrade, difficulty achieving compaction, and difficult underground utility installation. The generally clean loose sand with low cohesion that was encountered within our exploratory borings could potentially cave, especially below the water table, in open excavations.

5.1.4 Soil Corrosion Potential

As discussed, a preliminary soil corrosion screening was performed based on the results of analytical tests on samples of the near-surface soil. In general, the test results indicate that the low sulfate exposure results in no cement-type restrictions for buried concrete. Conversely, the corrosion potential for buried metallic structures, such as metal pipes, is considered to be severe to very severe. Based on the results of the preliminary soil corrosion screening, special requirements for corrosion control will likely be required to protect metal pipes and fittings. We recommend a corrosion engineering specialist be retained for corrosion protection recommendations.

5.1.5 Re-Development Considerations

The site is currently occupied by four commercial/industrial buildings and associated improvements including Portland cement concrete pavement. Potential issues that are often associated with redeveloping sites include demolition of existing improvements, abandonment of existing utilities, and undocumented fill. Recommendations addressing these issues are presented in the “Earthwork” section of this report.

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, CLEARING AND PREPARATION

6.1.1 Site Stripping

The site should be stripped of all surface vegetation, and surface and subsurface improvements within the proposed development area. Demolition of existing improvements is discussed in detail below. Surface vegetation and topsoil should be stripped to a sufficient depth to remove all material greater than 3 percent organic content by weight. Based on our site observations, surficial stripping should extend about 4 to 6 inches below existing grade in vegetated areas.

6.1.2 Tree and Shrub Removal

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.

6.1.3 Demolition of Existing Slabs, Foundations and Pavements

All slabs, foundations, and pavements should be completely removed from within planned building areas. Slabs, foundations, and pavements that extend into planned flatwork, pavement, or landscape areas may be left in place provided there is at least 3 feet of engineered fill overlying the remaining materials, they are shown not to conflict with new utilities, and that asphalt and concrete more than 10 feet square is broken up to provide subsurface drainage. A discussion of recycling existing improvements is provided later in this report.

6.1.4 Abandonment of Existing Utilities

All utilities should be completely removed from within planned building areas. For any utility line to be considered acceptable to remain within building areas, the utility line must be completely backfilled with grout or sand-cement slurry (sand slurry is not acceptable), the ends outside the building area capped with concrete, and the trench backfill either removed and replaced as engineered fill with the trench side slopes flattened to at least 1:1, or the trench backfill is not considered to be a risk to the structure. The evaluation of the risk posed by the particular utility line will impact whether the utility may be abandoned in place or completely removed. The contractor should assume that all utilities will be removed from within building areas unless written confirmation is provided from both the owner and the geotechnical engineer.

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

The risks associated with abandoning utilities in place include the potential for future differential settlement of existing trench fill, and/or partial collapse and potential ground loss into utility lines that are not completely filled with grout. In general, the risk is relatively low for single utility lines less than 4 inches in diameter, and increases with increasing pipe diameter.

6.2 REMOVAL OF EXISTING FILLS

Undocumented fill was encountered in EB-1 through EB-4 up to a depth of about 3 feet below the existing grade. Undocumented fill encountered during site grading 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 material meets the “Material for Fill” requirements below, the fill may be reused when backfilling the excavations. If materials are encountered that do not meet the requirements, including debris, wood, and trash, such materials should screened out of the remaining fill 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.

6.3 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.

Excavations performed during site demolition and fill removal should be slope at 3:1 (horizontal:vertical) within the upper 5 feet below building subgrade. Excavations extending more than 5 feet below building subgrade and excavations in pavement and flatwork areas should be slope at a 1:1 inclination unless the OSHA soil classification indicates that slope should not exceed 1.5:1.

6.4 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.

6.5 SUBGRADE STABILIZATION MEASURES

Subgrade soil and fill materials with high fines contents, especially clay and silt, can become unstable due to high moisture content, whether from high in-situ moisture content or from winter rains. As the moisture content increases over the laboratory optimum, fine-grained materials are more likely to 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 for the clayey and sandy soil are up to 16 percent and 15 to 17 percent over the estimated laboratory optimum in the upper 10 feet of the soil profile, respectively. The contractor should anticipate drying the soil prior to reusing as fill. In addition, repetitive rubber-tire loading will likely de-stabilize the soil.

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.

6.5.1 Scarification and Drying

The subgrade may be scarified to a depth of 8 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.

6.5.2 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.

6.5.3 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 cost-effective than removal and replacement. Recommended chemical treatment depths will typically range from 12 to 18 inches depending on the magnitude of the instability. Further recommendations can be provided at the time of construction.

6.6 MATERIAL FOR FILL

6.6.1 Re-Use of On-site Soil

On-site soil with an organic content less than 3 percent by weight may be reused as general fill. General fill materials should not contain 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.

6.6.2 Reuse of On-site Improvements

[If the site area allows for on-site pulverization of PCC and provided the PCC is pulverized to meet the "Material for Fill" requirements of this report, it may be used as select fill within the building areas, excluding the capillary break layer; as typically pulverized PCC comes close to or meets Class 2 AB specifications, the recycled PCC may likely be used within the pavement structural sections. PCC grindings also make good winter construction access roads, similar to a cement-treated base (CTB) section.

6.6.3 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 enclosed 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 II 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 soil, based on pH, saturated resistivity, and soluble sulfate and chloride testing.

6.7 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 soil should be compacted with sheepsfoot equipment and sandy/gravelly soil with vibratory equipment; open-graded materials such as crushed rock should be placed in lifts not 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 soil 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 Soil	90	>1
General Fill (below a depth of 5 feet)	On-Site Soil	95	>1
Basement Wall Backfill	Without Surface Improvements	90	>1
Basement Wall Backfill	With Surface Improvements	95[4]	>1
Trench Backfill	On-Site Soil	90	>1
Trench Backfill (upper 6 inches of subgrade)	On-Site Soil	95	>1
Crushed Rock Fill	¾-inch Clean Crushed Rock	Consolidate In-Place	NA
Non-Expansive Fill	Imported Non-Expansive Fill	90	Optimum

Table 3 (cont.): Compaction Requirements

Description	Material Description	Minimum Relative[1] Compaction (percent)	Moisture[2] Content (percent)
Flatwork Subgrade	On-Site Soil	90	>1
Flatwork Aggregate Base	Class 2 Aggregate Base[3]	90	Optimum
Pavement Subgrade	On-Site Soil	95	>1
Pavement Aggregate Base	Class 2 Aggregate Base[3]	95	Optimum
Asphalt Concrete	Asphalt Concrete	95	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

6.8 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.

Where utility lines will cross perpendicular to strip footings, the footing should be deepened to encase the utility line, providing sleeves or flexible cushions to protect the pipes from anticipated foundation settlement, or the utility lines should be backfilled to the bottom of footing with sand-cement slurry or lean concrete. Where utility lines will parallel footings and will extend below the "foundation plane of influence," an imaginary 1:1 plane projected down from the bottom edge of the footing, either the footing will need to be deepened so that the pipe is above the foundation plane of influence or the utility trench will need to be backfilled with sand-cement slurry or lean concrete within the influence zone. Sand-cement slurry used within foundation influence zones should have a minimum compressive strength of 75 pound per square inch (psi).

6.9 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 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.

6.10 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.

- In general, the near-surface soils at the site are generally 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.

- Locally, seasonal high ground water is mapped at a depth of approximately 8 feet and was measured in our borings as shallow as 15 feet, and therefore, seasonally, could be expected to be within 10 feet of the base of the infiltration measure.

- Infiltration measures, devices, or facilities may conflict with the location of existing or proposed underground utilities or easements. Infiltration measures, devices, or facilities should not be placed on top of or very near to underground utilities such that they discharge to the utility trench, restrict access, or cause stability concerns.

6.10.1 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.

6.10.1.1 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 soil near structural improvements, and to address the low infiltration capacity of the on-site clay soil.

6.10.1.2 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.

6.10.1.3 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 restaint should be adequately keyed into the native soil or engineered to reduce the potential for rotation or lateral movement of the curbs.

SECTION 7: FOUNDATIONS

7.1 SUMMARY OF RECOMMENDATIONS

As previously discussed, the potential for liquefaction to impact the proposed improvements is considered to be high. In our opinion, the new structures may be supported on spread footings provided ground improvement is performed to mitigate the effects of liquefaction-induced settlement. We recommend a design-build contractor perform the ground improvement. The ground improvement should be designed to meet the project requirements as recommended in this report. Recommendations for ground improvement are provided in the following sections.

7.2 SEISMIC DESIGN CRITERIA

We understand that the project structural design will be based on the 2013 California Building Code (CBC), which provides criteria for the seismic design of buildings in Chapter 16. The "Seismic Coefficients" used to design buildings are established based on a series of tables and figures addressing different site factors, including the soil profile in the upper 100 feet below grade and mapped spectral acceleration parameters based on distance to the controlling seismic source/fault system. Based on our shear wave velocity measurement in CPT-1 to about 80 feet and review of available data, we estimate an average shear wave velocity of 820 feet per second (or 250 meters per second). Therefore, we have classified the site as Soil Classification D. The mapped spectral acceleration parameters S_S and S_1 were calculated using the USGS computer program *Design Maps*, located at http://geohazards.usgs.gov/designmaps/us/application.php based on the site coordinates presented below and the site classification. The table below lists the various factors used to determine the seismic coefficients and other parameters.

Table 4: 2013 CBC Site Categorization and Site Coefficients

Classification/Coefficient	Design Value
Site Class	D
Site Latitude	37.44539°
Site Longitude	-121.90965°
0.2-second Period Mapped Spectral Acceleration[1], S_S	1.819g
1-second Period Mapped Spectral Acceleration[1], S_1	0.727g
Short-Period Site Coefficient – F_a	1.0
Long-Period Site Coefficient – F_v	1.5
0.2-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects - S_{MS}	1.819g
1-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects – S_{M1}	1.091g
0.2-second Period, Design Earthquake Spectral Response Acceleration – S_{DS}	1.213g
1-second Period, Design Earthquake Spectral Response Acceleration – S_{D1}	0.727g

Because the potential for liquefaction at the site is high, based on Section 1613.3.2 of the 2013 California Building Code (CBC), which refers to Table 20.3-1, Site Classification, of ASCE 7-10, the site should be classified as Site Class F. Site Coefficients F_a and F_v are determined using Tables 11.4-1 and 11.4-2 of ASCE 7-10 or Tables 1613.3.3(1) and 1613.3.3(2) of the 2013 CBC. Site Class F of those tables refers the determination of Site Coefficients F_a and F_v to Section 11.4.7 of ASCE 7-10. ASCE 7-10 indicates that sites classified as Site Class F shall have a site response analysis performed in accordance with Section 21.1 of ASCE 7-10, unless the proposed structure meets the following exception.

> **EXCEPTION:** For structures having fundamental periods of vibration equal to or less than 0.5s, site-response analysis is not required to determine spectral accelerations for liquefiable soils. Rather, a site class is permitted to be determined in accordance with Section 20.3 and the corresponding values of F_a and F_v determined from Tables 11.4-1 and 11.4-2.

For these reasons and if ground improvement is performed per our recommendations, in our opinion, the above Site Classification of D in Table 4 of this report, and the presented seismic coefficients, appear valid due to the above exception, as the structure likely has a fundamental period equal to or less than 0.5 seconds. The project structural engineer should verify this assumption. If the structure will have a fundamental period of greater than 0.5 seconds, and meets the requirements for a Site Class designation of F, the requirement for a site response analysis will be triggered, and additional geotechnical analysis will need to be approved.

7.3 SHALLOW FOUNDATIONS

7.3.1 Spread Footings

Provided ground improvement is performed in accordance with recommendations in this report, spread footings should bear on a uniform layer of engineered fill or densified native soil, and be at least 18 inches wide, and extend at least 12 inches below the lowest adjacent grade. Bottom of footing is based on lowest adjacent grade, defined as the deeper of the following: 1) bottom of the adjacent interior slab-on-grade, or 2) finished exterior grade, excluding landscaping topsoil.

The design allowable bearing pressures will be dependent on the final ground improvement details including the layout and spacing; however, substantial improvement in bearing capacity would be expected. For your preliminary design, we expect allowable bearing pressures on the order of 4,000 to 6,000 psf for combined dead plus live loads would be feasible following ground improvement. The above estimates should be evaluated further once a design-build ground improvement contractor is chosen.

Ground improvement and the replacement of disturbed near-surface soil as engineered fill would be designed to reduce total settlement due to static and seismic conditions to a tolerable level.

7.3.2 Footing Settlement

Structural loads for the proposed buildings were not available at the time of this report. The typical structural loads in Table 5 were used for our initial foundation analysis.

Table 5: Estimated Structural Loading

Foundation Area	Range of Loads
Interior Isolated Column Footing	50 to 75 kips
Exterior Isolated Column Footing	50 to 75 kips
Perimeter Strip Footing	1½ to 2 kips per lineal foot

Based on the above loading, ground improvement would be modified to reduce estimated total and differential settlement to a tolerable level. These criteria and recommendations for ground improvement are provided in Section 7.4.

7.3.3 Lateral Loading

Lateral loads may be resisted by friction between the bottom of footing and the supporting subgrade, and also by passive pressures generated against footing sidewalls. An ultimate frictional resistance of 0.40 applied to the footing dead load, and an ultimate passive pressure based on an equivalent fluid pressure of 400 pounds per cubic foot (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. Where footings are adjacent to landscape areas without hardscape, the upper 12 inches of soil should be neglected when determining passive pressure capacity. Where footings are adjacent to landscape areas without hardscape, the upper 12 inches of soil should be neglected when determining passive pressure capacity.

7.3.4 Spread Footing Construction Considerations

Where utility lines will cross perpendicular to strip footings, the footing should be deepened to encase the utility line, providing sleeves or flexible cushions to protect the pipes from anticipated foundation settlement, or the utility lines should be backfilled to the bottom of footing with sand-cement slurry or lean concrete. Where utility lines will parallel footings and will extend below the "foundation plane of influence," an imaginary 1:1 plane projected down from the bottom edge of the footing, either the footing will need to be deepened so that the pipe is above the foundation plane of influence or the utility trench will need to be backfilled with sand-cement slurry or lean concrete within the influence zone. Sand-cement slurry used within foundation influence zones should have a minimum compressive strength of 75 psi. An effort to avoid future utility locations with ground densification column locations should be made, particularly at the perimeter foundation locations, as the utility installation may cause significant disturbance to the ground densification columns and reduce footing support in that area.

Footing excavations should be filled as soon as possible or be kept moist until concrete placement by regular sprinkling to prevent desiccation. A Cornerstone representative should observe all footing excavations prior to placing reinforcing steel and concrete. If there is a significant schedule delay between our initial observation and concrete placement, we may need to re-observe the excavations.

7.4 GROUND IMPROVEMENT

As noted, liquefaction settlement varies significantly across the site from about ½-inch to about 3 inches. Ground improvement, such as impact piers, stone columns, soil/cement mixing, grouted displacement columns, compaction grouting, or other similar methods, may be used to improve the subsurface soil beneath the recommended shallow foundations systems such that the combined total static and seismic settlement is reduced to less than 2 inches in foundation areas with seismic settlement not exceeding 1½ inches. Differential settlement (static and seismic) between columns (assumed to be 30 feet) should be less than 1¼ inches, where differential settlement is estimated as one-half of total static settlement, and two-thirds of total seismic settlement. If these settlements are not considered acceptable to the structural engineers, the maximum allowable foundation settlement should be reduced to acceptable levels or the structural details modified to allow for the estimated settlements. Ground improvement should provide adequate confining improvement around all foundations and should include an increase in allowable bearing pressure as discussed above.

Depending on the final spacing of ground improvement areas, differential settlement within slabs-on-grade could also be adequately mitigated to allow the use of a conventional slab-on-grade floor. The total settlement (static and seismic), under slab areas, also should not exceed 2 inches, with no limit on contributing proportions due to the lighter loading in slab areas. The intent of the ground improvement design would be to increase the density of the potentially liquefiable sand by laterally displacing and/or densifying the existing in-place soil. The degree to which the density is increased will depend on the improvement method and spacing. In addition to increasing the density, the methods listed above, except for compaction grouting, could provide an additional increase in bearing capacity and soil stiffness at the individual improvement locations, which could be taken into consideration during evaluation of the post-construction consolidation settlement.

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, and 5) post-construction CPT tip resistance criteria to be achieved in the sand layers after installation and refusal criteria. We should be retained to review the ground improvement contractor's plan and settlement estimates prior to construction.

Ground improvement would generally be constructed as follows: 1) clear the site of existing fills (as necessary), 2) grade site to rough grades removing and replacing undocumented fill, 3) install the ground improvement on the approved layout, and 4) if necessary, excavate the upper one to two feet, or as necessary based on ground improvement method chosen, and replace as engineered fill to repair disturbance to the near-surface soils resulting from ground improvement installation.

7.4.1 Ground Improvement Performance Testing

Performance testing typically consists of a pre-construction test section with post-installation CPT testing to confirm that the necessary soil densification increases were achieved to meet the settlement criteria. Post-installation CPT testing is also required during production installation and is discussed below. We should observe and monitor installation of the ground improvement on a full-time basis and review the post-installation settlement analyses provided by the contractor.

The proposed design spacing of the ground improvement will be confirmed prior to construction by the installation of at least one test array section of four ground improvement columns with installation lengths and spacing as initially agreed to between the ground improvement contractor and Cornerstone Earth Group.

To confirm the reduction in seismic settlements, supplemental CPT soundings will be performed at the center of the four-column test array, and the data analyzed for liquefaction settlement. The CPT soundings should be performed at least one week after installation of the test array, and preferably 30 days if feasible, to allow pore pressure dissipation in the finer grained soil. If the total liquefaction settlement calculated from the CPT at the center of the test array to a depth of 50 feet is less than the tolerable settlements indicated above, as determined by the Geotechnical Engineer, and the contractor warranties the ground improvement design goals, the initial spacing will be considered acceptable. If the total liquefaction at the center of the test array exceeds requirements, the contractor should revise their proposed spacing and perform a subsequent test array to show adequate improvement to meet the project design goals. Revised methods that do not include a reduction in spacing will not be acceptable as a revision.

Subsequent to a successful test array installation, production ground improvement can be installed. We recommend that during production installation additional CPT soundings be performed to monitor the effectiveness of the ground improvement. At least six (6) CPTs are recommended at locations to be chosen by Cornerstone Earth Group after the ground improvement layout plan is prepared. These CPTs should extend to 50 feet and the data analyzed by our office for liquefaction settlement and evaluated against the criteria used for the test array. If the liquefaction analyses indicate that the area of the CPT does not meet the acceptance criteria, additional CPTs will be required to delineate the horizontal extent of the area that does not meet the project ground improvement goals. Working with the structural engineer, the team will evaluate whether differential settlement estimates are tolerable or whether additional ground improvement is required.

We should observe and monitor installation of the ground improvement on a full-time basis and review the post-installation settlement analyses provided by the contractor as well as perform our independent analysis of the data.

The ground improvement contractor shall make their own interpretation of strength parameters for the soil, obtained or derived from the soil boring logs, cone penetration tests, and any geotechnical laboratory testing data provided in the Geotechnical Report and these specifications for bearing capacity analyses. Static settlement shall be assessed using appropriate soil parameters for an elastic settlement analysis based on an area replacement ratio considering the stiffness of the native soils, and the densification columns. Liquefaction and seismic settlement estimates shall be performed using methodology presented in the project geotechnical report, which followed the procedures in the 2008 monograph, *Soil Liquefaction During Earthquakes* (Idriss and Boulanger, 2008). Liquefaction and settlement shall be evaluated for the upper 50 feet of the soil profile. Any additional subsurface information needed to design the ground improvement shall be the responsibility of the contractor.

7.5 REINFORCED CONCRETE MAT FOUNDATION

7.5.1 General

As an alternative to spread footings and ground improvement, the buildings may be supported on a mat foundation with the understanding that the potential for liquefaction related damage to the proposed improvements is possible. As discussed above, the total and differential settlement due to liquefaction at the site without ground improvement is estimated to range from ½-inch to 3 inches and ⅓-inch to 2 inches (over a distance of 30 feet), respectively. The project structural engineer should be consulted regarding the impacts of the anticipated liquefaction settlement on a mat foundation.

7.5.2 Allowable Mat Bearing Pressure

We have assumed areal loading for our analysis based on our understanding of the project and planned structures. Based on the structural loads presented in Table 5, we have estimated an average areal pressure of about 600 pounds per square foot (psf) for the structures. We recommend the allowable bearing pressure at heavier loaded portions of the mat slabs be limited to 2,000 psf for dead plus live loads. The maximum bearing pressure may be increased by one-third for all loads, including wind or seismic. These pressures are net values; the mat weight may be neglected for the portion of the mat extending below grade. Top and bottom reinforcing steel should be included as required to help span irregularities and differential settlement. It is essential that we observe the mat foundation pad prior to placement of reinforcing steel.

7.5.3 Mat Foundation Settlement

We estimate that total settlement due to static loading would be about 1 inch and total post-construction differential movement of up to ½-inch across the mat area (in the short direction assumed to be on the order of 30 feet). In addition, the mat should be designed to accommodate up to 1 inch of seismic differential movement between the center and the edge of the mat. Accounting for liquefaction-induced and static differential settlement, we recommend the mat be designed to tolerate a total differential movement of approximately 2 inches from the center to the edge of the mat. If foundations designed in accordance with the above recommendations are not capable of resisting such differential movement, additional reinforcement or increased mat thickness may be required.

In addition, gravity flow utilities should be designed to account for any future settlement to avoid grade reversal or leakage from joint separation.

We recommend we be retained to review the final loading, and verify the settlement estimates above.

7.5.4 Mat Foundation 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 0.40 applied to the mat dead load, and an ultimate passive pressure based on an equivalent fluid pressure of 400 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. Where the mat is located adjacent to landscape areas without hardscape, the upper foot should be neglected when determining passive pressure capacity.

7.5.5 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 analysis for the mat. A preliminary modulus of subgrade reaction for the initial analysis is provided below.

As discussed above, we estimated an average areal pressure of 600 psf within the structures. Based on this pressure, we calculated a preliminary modulus of soil subgrade reaction for the mat foundation. For preliminary SAFE runs, we recommend that an initial soil modulus of 10 pounds per cubic inch (pci) be used toward the center portion of the mat. As discussed above, this modulus of soil subgrade reaction is intended for use in the first iteration of the structural SAFE analysis for the mat design. We will provide a revised plan with contours of equal modulus of subgrade reaction values following our final analysis.

7.5.6 Mat Foundation Construction Considerations

Prior to placement of any vapor retarder and mat construction, the subgrade should be proof-rolled and visually observed by a Cornerstone representative to confirm stable subgrade conditions. 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 2 percent over optimum in the upper 12 inches.

If moisture-sensitive floor coverings are planned, the recommendations in the “Moisture Protection Considerations for Mat Foundations” section below may be incorporated in the project design if desired.

7.6 DEEP FOUNDATIONS

As an alternative to a shallow foundation with ground improvement or if the estimated settlement exceeds the structural requirements for a mat foundation, alternative foundations including auger-casted piers, driven concrete piles, and micro- or mini-piles may also be feasible from a geotechnical standpoint. If deep foundations are preferred, we can provide recommendations upon request.

SECTION 8: INTERIOR CONCRETE SLABS

8.1 INTERIOR SLABS-ON-GRADE WITH SPREAD FOOTINGS

As the Plasticity Index (PI) of a representative sample of the surficial soil is 16, the proposed slabs-on-grade may be supported directly on subgrade prepared in accordance with the recommendations in the “Earthwork” section of this report. If moisture-sensitive floor coverings are planned, the recommendations in the “Interior Slabs Moisture Protection Considerations” section below may be incorporated in the project design if desired. If significant time elapses between initial subgrade preparation and slab-on-grade construction, the subgrade should be proof-rolled to confirm subgrade stability, and if the soil has been allowed to dry out, the subgrade should be re-moisture conditioned to near optimum moisture content.

The structural engineer should determine the appropriate slab reinforcement for the loading requirements and considering the expansion potential of the underlying soils. 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.

8.2 INTERIOR SLABS MOISTURE PROTECTION CONSIDERATIONS

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

- Place a minimum 10-mil vapor retarder conforming to ASTM E 1745, Class C requirements or better directly below the concrete slab; the vapor retarder should extend to the slab edges and be sealed at all seams and penetrations in accordance with manufacturer’s recommendations and ASTM E 1643 requirements. A 4-inch-thick capillary break, consisting of ½- to ¾-inch crushed rock with less than 5 percent passing the No. 200 sieve, should be placed below the vapor retarder and consolidated in place with vibratory equipment.

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

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

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

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

8.2.1 Moisture Protection Considerations for Mat Foundations

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

- Place a minimum 10-mil-thick vapor retarder conforming to ASTM E 1745, Class C requirements or better directly below the concrete mat; the vapor retarder should extend to within 12 to 18 inches from the mat edges and be sealed at all seams and penetrations in accordance with manufacturer's recommendations and ASTM E 1643 requirements. For mats 12 inches thick or less, a 4-inch-thick capillary break, consisting of ½- to ¾-inch crushed rock with less than 5 percent passing the No. 200 sieve, should be placed below the vapor retarder and consolidated in place with vibratory equipment.

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

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

- Polishing the concrete surface with metal trowels should not be allowed versus light broom or limited trowel finishing.

- Where floor coverings are planned, all concrete surfaces should be moist cured (kept continuously wet) for at least 7 days by soaking burlap, cotton mats, or carpet, or frequent sprinkling. The moist cure method should be placed as soon after concrete finishing as possible, while resisting surface damage. Chemical curing may be an option depending on the floor covering type.

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

8.3 EXTERIOR FLATWORK

8.3.1 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 4 inches of Class 2 aggregate base overlying subgrade prepared in accordance with the "Earthwork" recommendations of this report. 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 9: VEHICULAR PAVEMENTS

9.1 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. Pavement design recommendations based on the R-value of the proposed engineered fill may be evaluated after the fill material is selected and tested.

Table 6: 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 be use the pavements.

SECTION 10: RETAINING WALLS

10.1 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 7: Recommended Lateral Earth Pressures

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

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

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

10.2 SEISMIC LATERAL EARTH PRESSURES

The 2013 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 4 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.

10.3 WALL DRAINAGE

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.

10.4 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. If surface improvements are not planned, backfill should be compacted to at least 90 percent. If heavy compaction equipment is used, the walls should be temporarily braced.

10.5 FOUNDATIONS

Retaining walls may be supported on a continuous spread footing designed in accordance with the recommendations presented in the "Foundations" section of this report.

SECTION 11: LIMITATIONS

This report, an instrument of professional service, has been prepared for the sole use of Irissou Family Partners, LLC specifically to support the design of the Hanson Self Storage facility project at 1 Hanson Court in Milpitas, 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 ground water 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.

Irissou Family Partners, LLC may have provided Cornerstone with plans, reports and other documents prepared by others. Irissou Family Partners, LLC 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 12: REFERENCES

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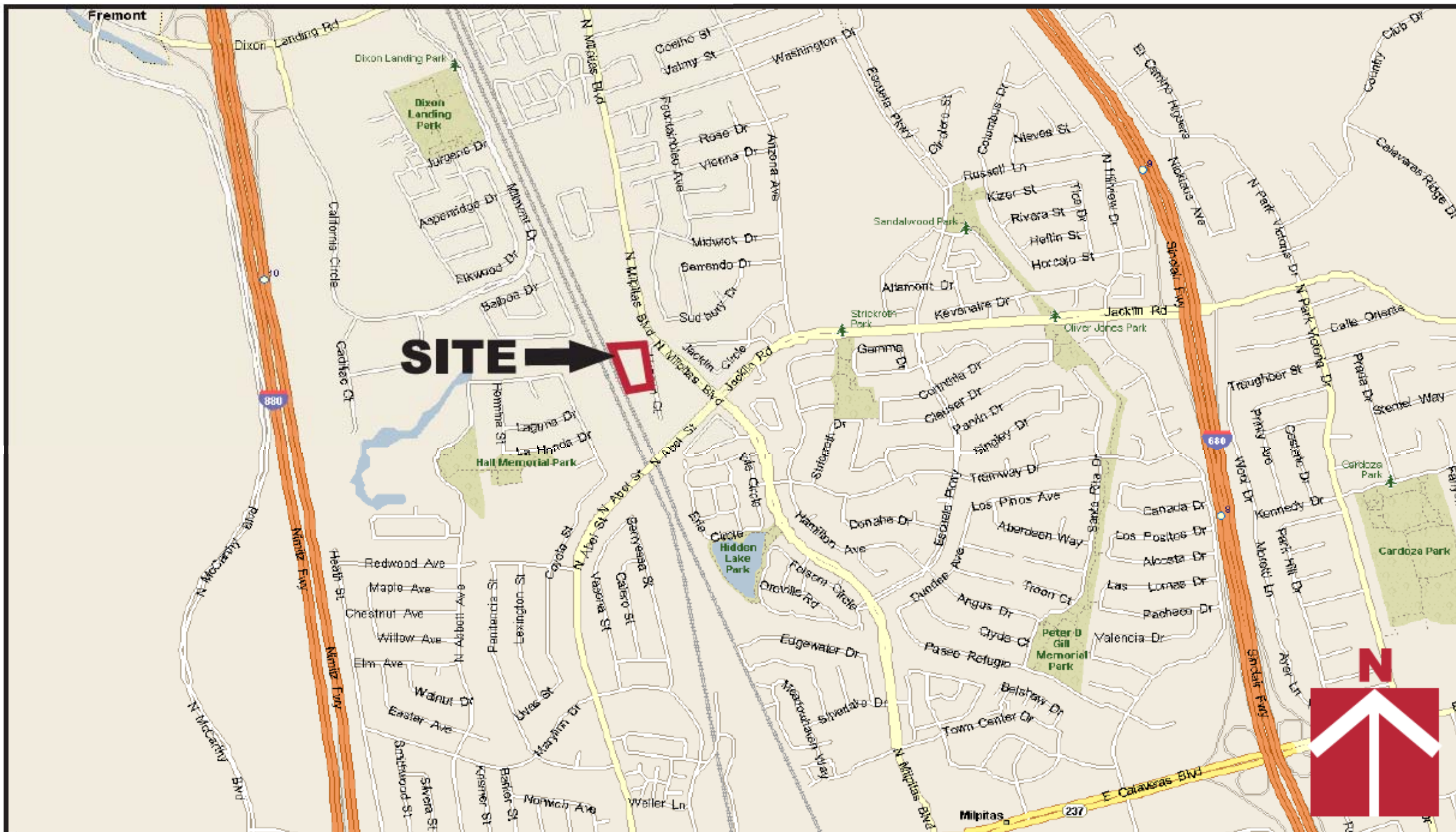

Vicinity Map	Project Number 726-1-3
Hanson Self Storage 1 Hanson Court Milpitas, CA	Figure Number Figure 1
	Date August 2014 / Drawn By RRN

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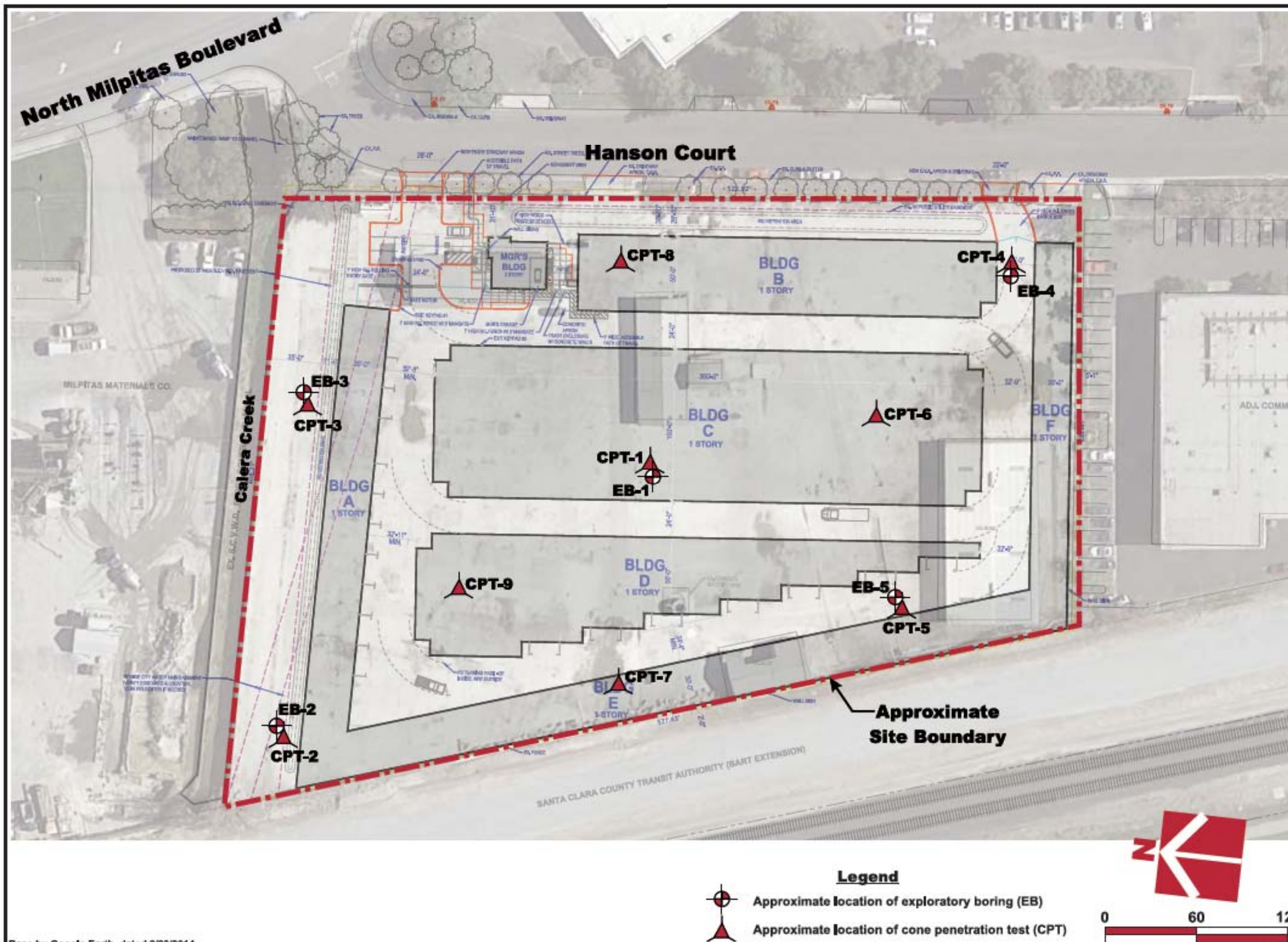

Base by Google Earth, dated 2/23/2014
Overlay by Cubix Construction Company, "Preliminary Site Plan - 1A," dated 3/26/2014

Project Number 726-1-3
Figure Number Figure 2
Date August 2014
Drawn By RRN

Site Plan

Hanson Self Storage
1 Hanson Court
Milpitas, CA

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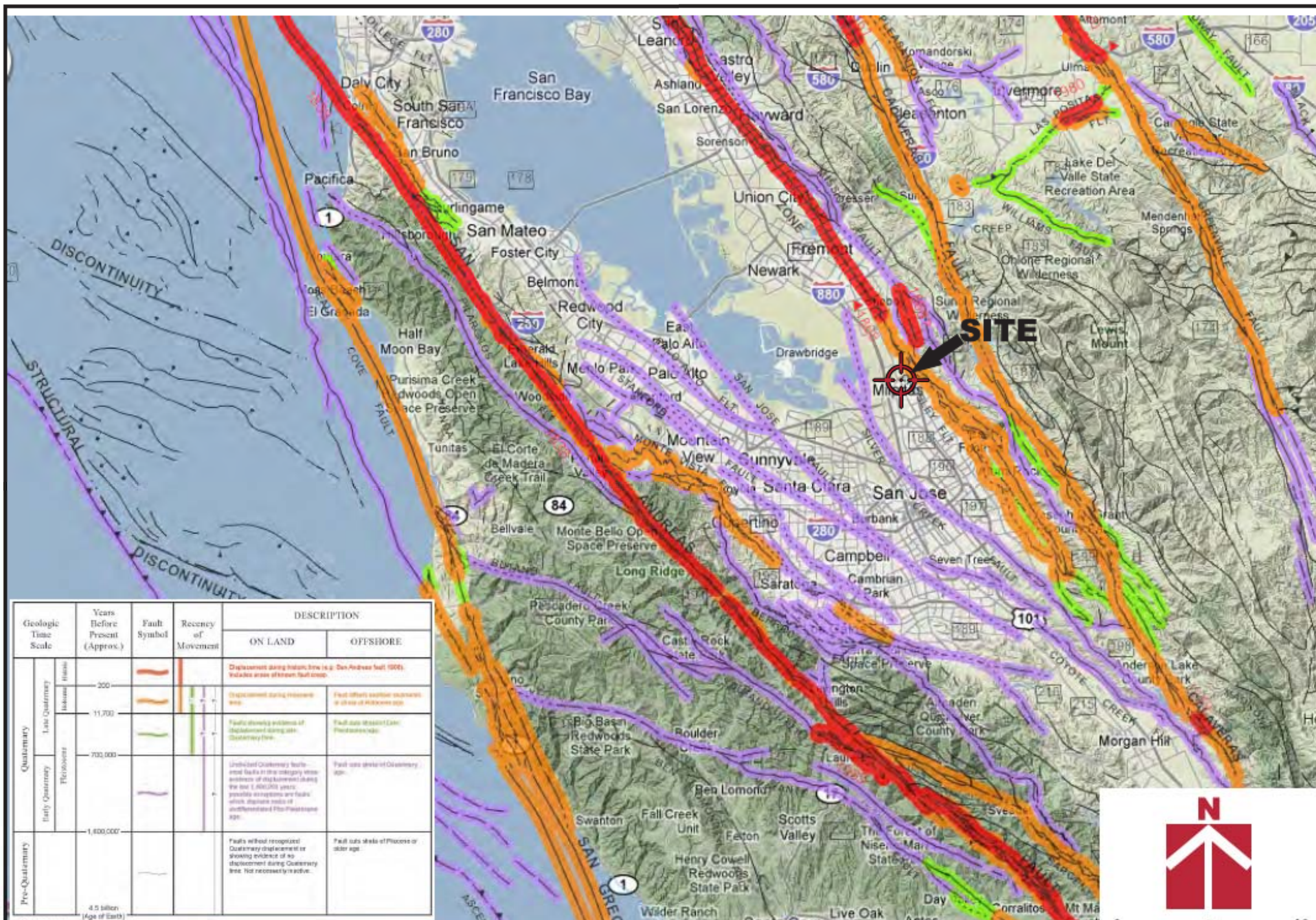

Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement	DESCRIPTION ON LAND	DESCRIPTION OFFSHORE
Quaternary	Late Quaternary	Historic	200			Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
		Holocene	11,700			Displacement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

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

Project Number: 726-1-3

Figure Number: Figure 3

Date: August 2014

Drawn By: RRN

Regional Fault Map

Hanson Self Storage
1 Hanson Court
Milpitas, CA

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FIGURE 4A

CPT NO. 1

PROJECT/CPT DATA

Project Title	**Hanson Self Storage**
Project No.	**726-1-3**
Project Manager	**NSD**

SEISMIC PARAMETERS

Controlling Fault	**Hayward**	
Earthquake Magnitude (Mw)	**7.1**	
PGA (Amax)	**0.7**	(g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet)	**14**
Design Water Depth (feet)	**5**
Ave. Unit Weight Above GW (pcf)	**125**
Ave. Unit Weight Below GW (pcf)	**120**

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 5 **FEET**

0.00 **(Inches)**

LIQUEFACTION SETTLEMENT FROM 50 **FEET**

2.42 **(Inches)**

TOTAL SEISMIC SETTLEMENT 2.4 **INCHES**

POTENTIAL LATERAL DISPLACEMENT

LDI[2] 0.68 **L/H** 25.0

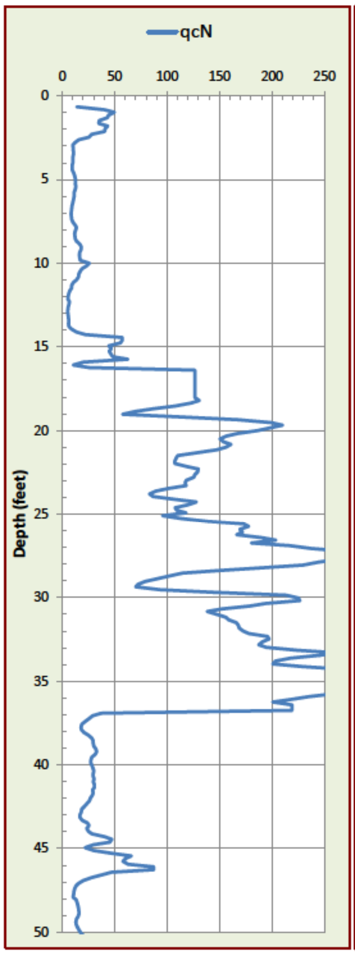
LDI[1] **Corrected for Distance** 0.31 (4 < L/H < 40)

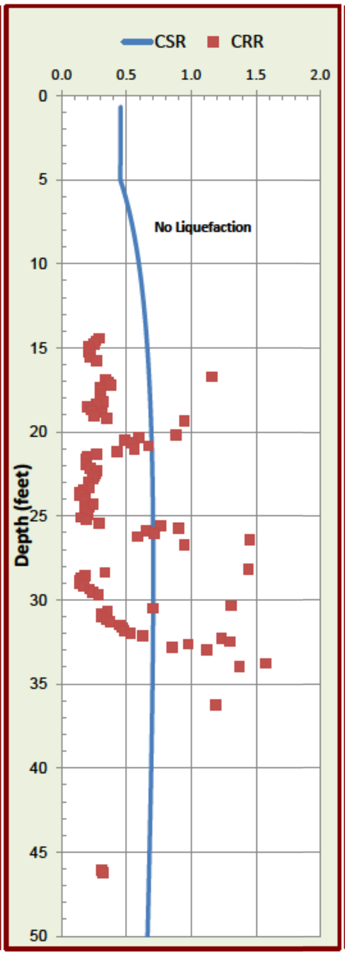
EXPECTED RANGE OF DISPLACEMENT

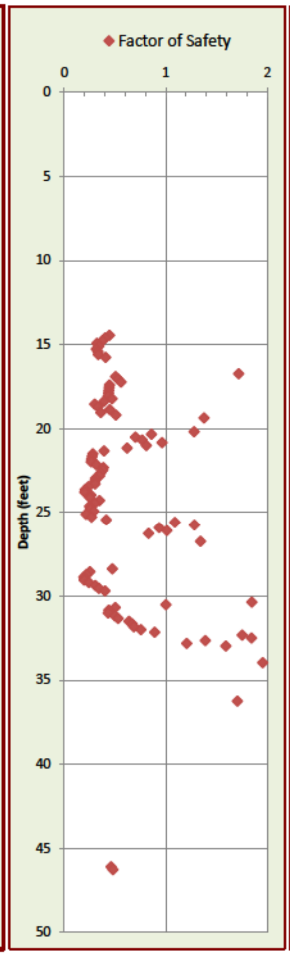
0.2 **to** **0.6** **feet**

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

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







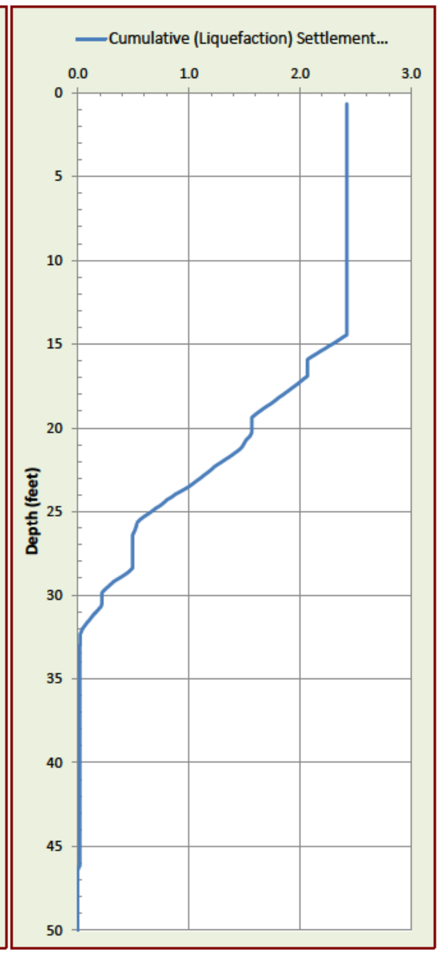


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FIGURE 4B

CPT NO. 2

PROJECT/CPT DATA

Project Title	Hanson Self Storage
Project No.	726-1-3
Project Manager	NSD

SEISMIC PARAMETERS

Controlling Fault	Hayward	
Earthquake Magnitude (Mw)	7.1	
PGA (Amax)	0.7	(g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet)	12.5
Design Water Depth (feet)	5
Ave. Unit Weight Above GW (pcf)	125
Ave. Unit Weight Below GW (pcf)	120

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 5 FEET

0.01 (Inches)

LIQUEFACTION SETTLEMENT FROM 50 FEET

1.89 (Inches)

TOTAL SEISMIC SETTLEMENT 1.9 INCHES

POTENTIAL LATERAL DISPLACEMENT

LDI[2] 0.25 L/H 3.0

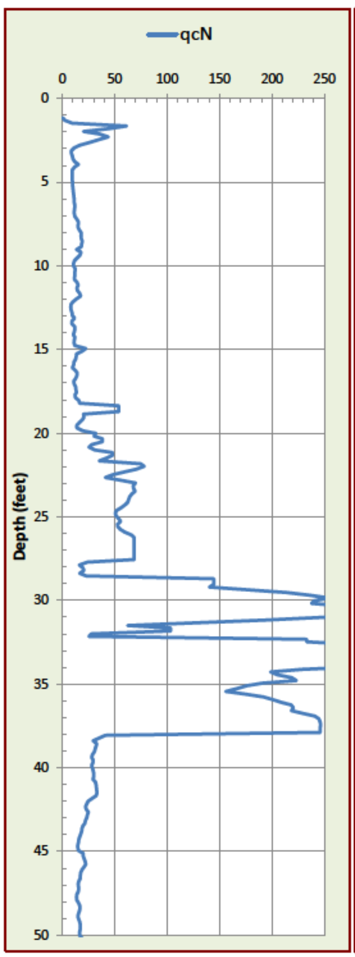
LDI[1] Corrected for Distance 0.62 (4 < L/H < 40)

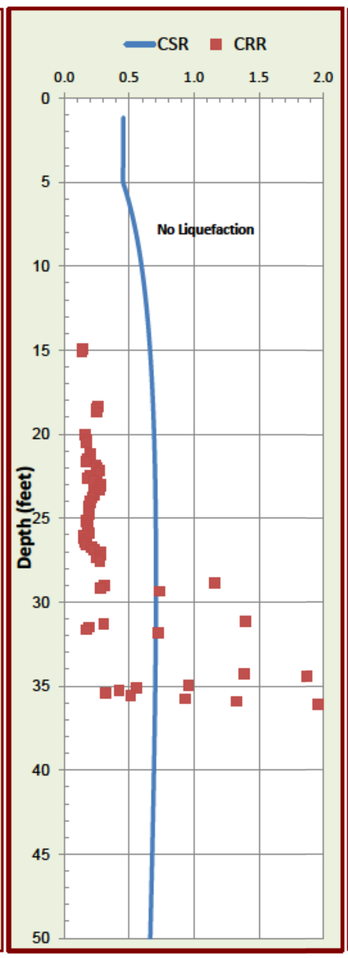
EXPECTED RANGE OF DISPLACEMENT

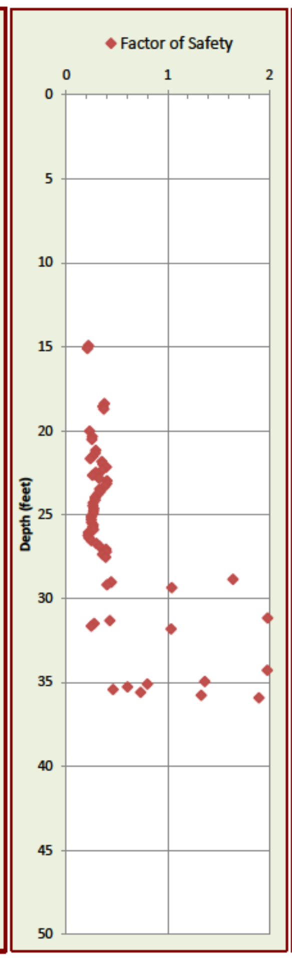
0.3 to 1.2 feet

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

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







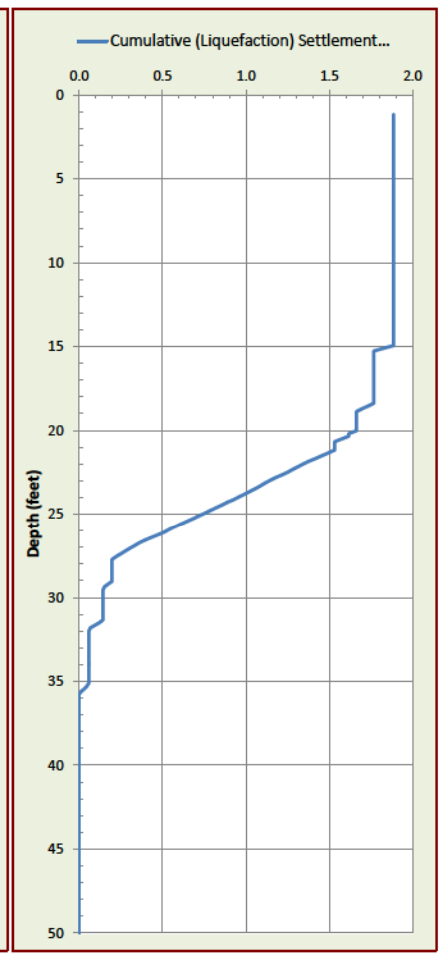


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FIGURE 4C

CPT NO. 3

PROJECT/CPT DATA

Project Title	Hanson Self Storage
Project No.	726-1-3
Project Manager	NSD

SEISMIC PARAMETERS

Controlling Fault	Hayward	
Earthquake Magnitude (Mw)	7.1	
PGA (Amax)	0.7	(g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet)	8
Design Water Depth (feet)	5
Ave. Unit Weight Above GW (pcf)	125
Ave. Unit Weight Below GW (pcf)	120

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 5 FEET

0.03 (Inches)

LIQUEFACTION SETTLEMENT FROM 50 FEET

1.53 (Inches)

TOTAL SEISMIC SETTLEMENT 1.6 INCHES

POTENTIAL LATERAL DISPLACEMENT

LDI[2] 0.65 L/H 3.0

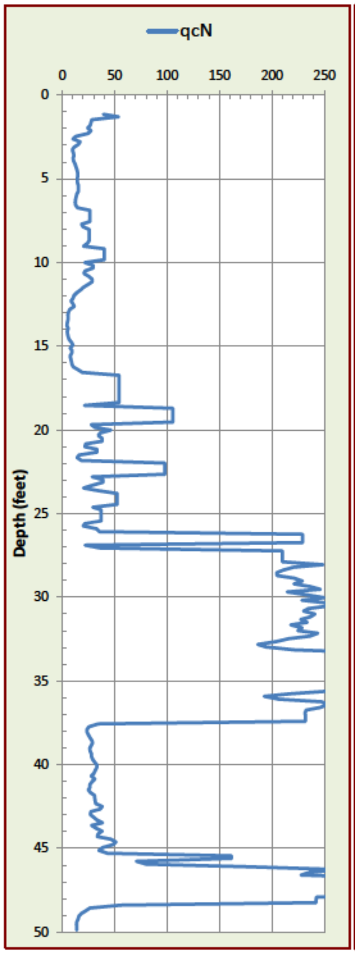
LDI[1] Corrected for Distance 1.63 (4 < L/H < 40)

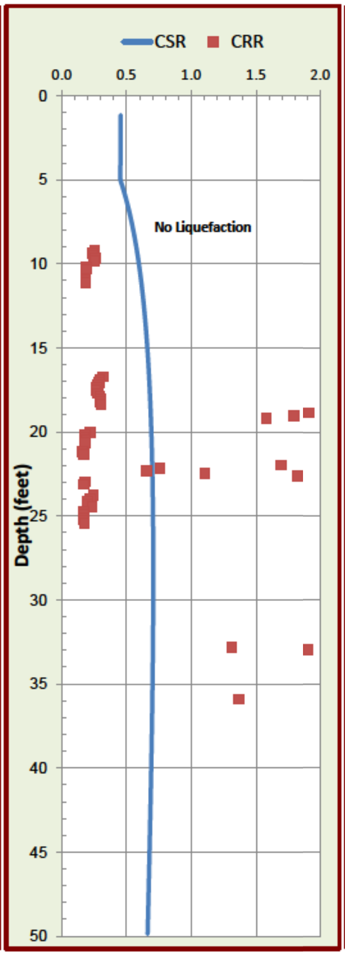
EXPECTED RANGE OF DISPLACEMENT

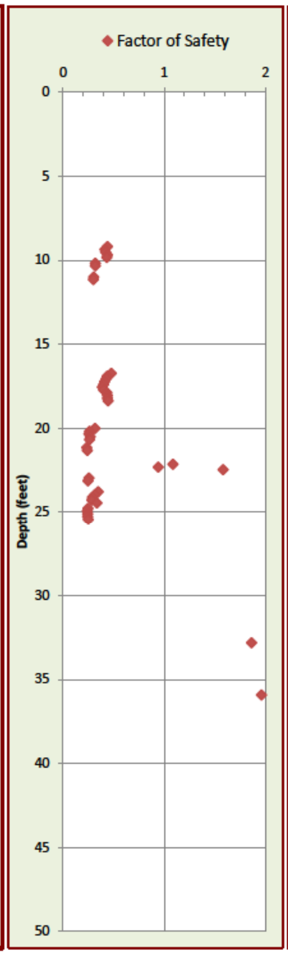
0.8 to 3.3 feet

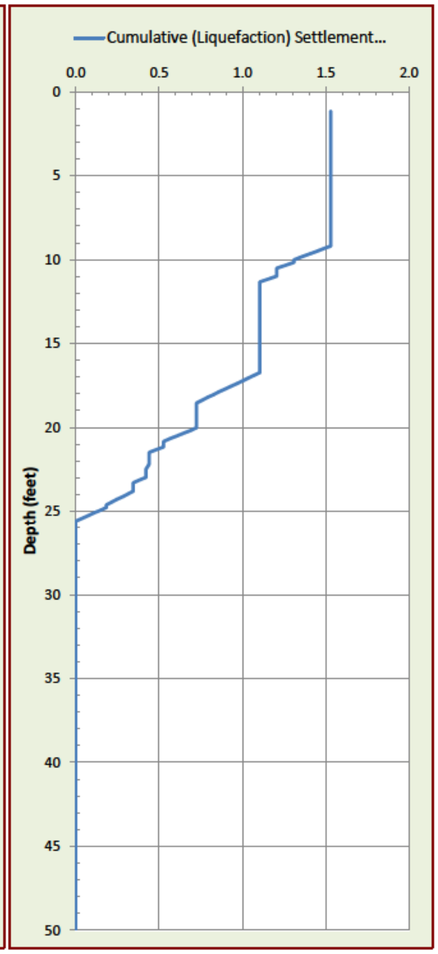
[1]Not Valid for L/H Values < 4 and > 40.

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









CORNERSTONE EARTH GROUP

FIGURE 4D

CPT NO. 4

PROJECT/CPT DATA

Project Title	Hanson Self Storage
Project No.	726-1-3
Project Manager	NSD

SEISMIC PARAMETERS

Controlling Fault	Hayward	
Earthquake Magnitude (Mw)	7.1	
PGA (Amax)	0.7	(g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet)	12
Design Water Depth (feet)	5
Ave. Unit Weight Above GW (pcf)	125
Ave. Unit Weight Below GW (pcf)	120

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 5 FEET

0.14 (Inches)

LIQUEFACTION SETTLEMENT FROM 50 FEET

1.50 (Inches)

TOTAL SEISMIC SETTLEMENT 1.6 INCHES

POTENTIAL LATERAL DISPLACEMENT

LDI[2] 0.58 L/H 48.5

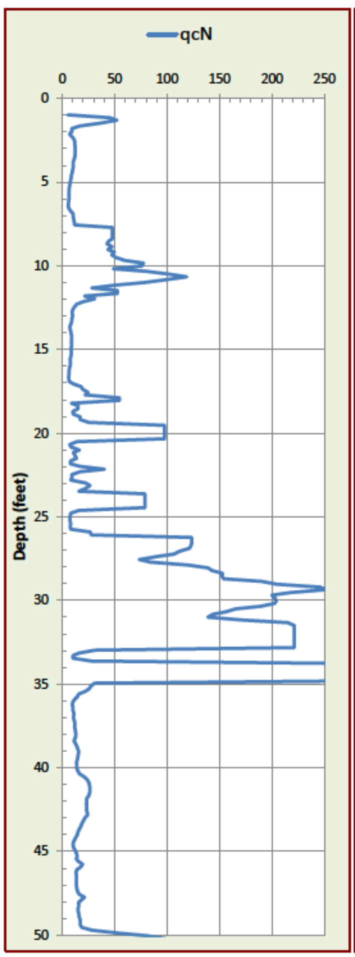
LDI[1] Corrected for Distance 0.16 (4 < L/H < 40)

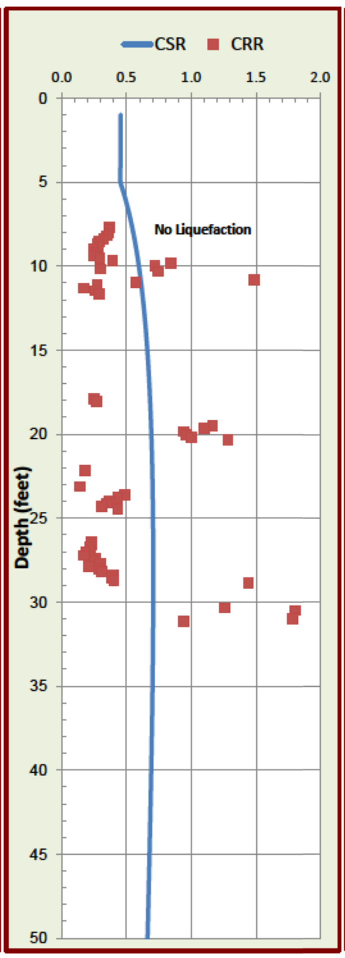
EXPECTED RANGE OF DISPLACEMENT

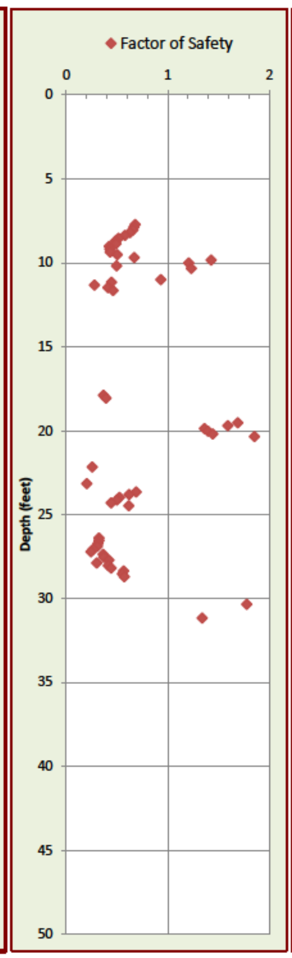
0.1 to 0.3 feet

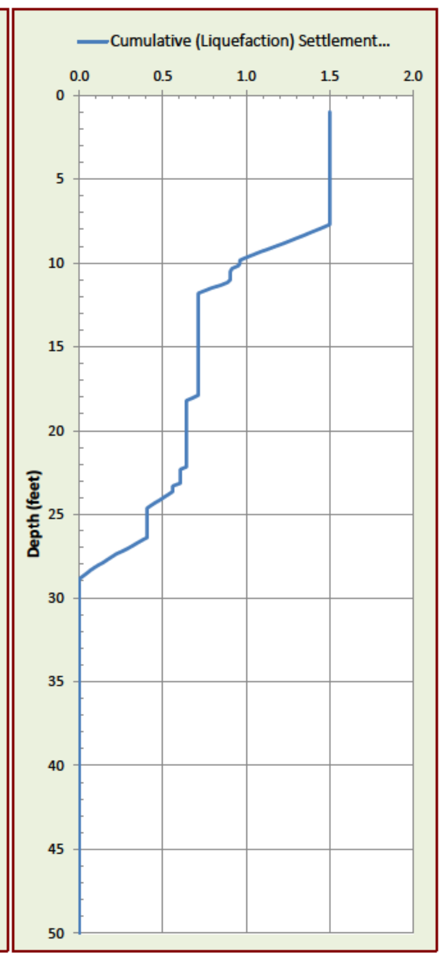
[1]Not Valid for L/H Values < 4 and > 40.

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









CORNERSTONE EARTH GROUP

FIGURE	4E
CPT NO.	5

PROJECT/CPT DATA

Project Title	Hanson Self Storage
Project No.	726-1-3
Project Manager	NSD

SEISMIC PARAMETERS

Controlling Fault	Hayward	
Earthquake Magnitude (Mw)	7.1	
PGA (Amax)	0.7	(g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet)	8
Design Water Depth (feet)	5
Ave. Unit Weight Above GW (pcf)	125
Ave. Unit Weight Below GW (pcf)	120

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 5 FEET

0.02 (Inches)

LIQUEFACTION SETTLEMENT FROM 50 FEET

2.42 (Inches)

TOTAL SEISMIC SETTLEMENT 2.4 INCHES

POTENTIAL LATERAL DISPLACEMENT

LDI[2] 1.34 L/H 43.5

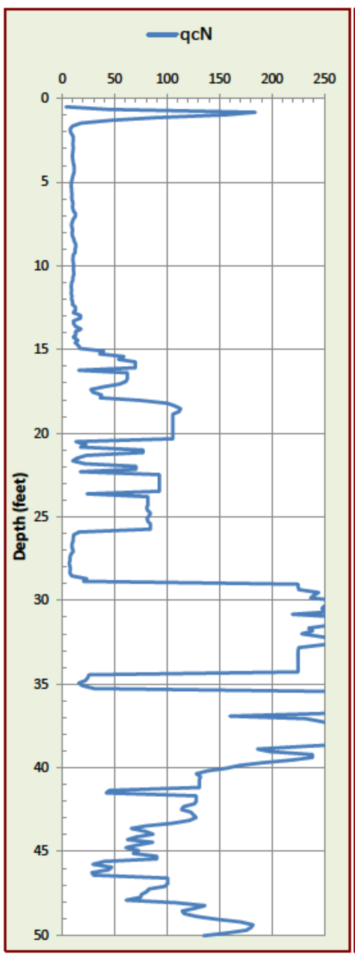
LDI[1] Corrected for Distance 0.39 (4 < L/H < 40)

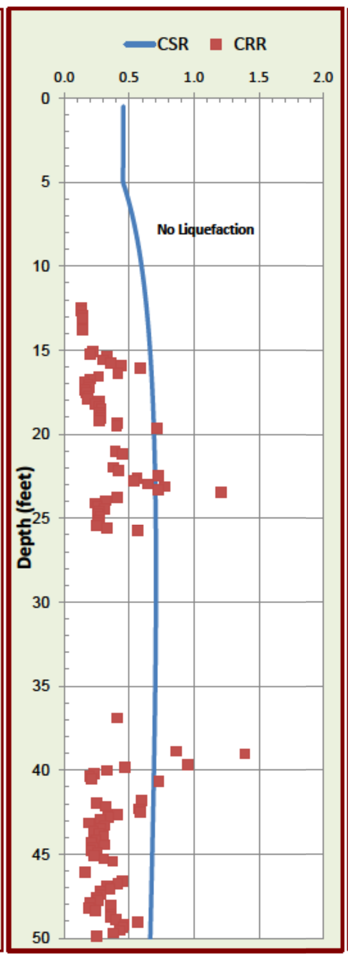
EXPECTED RANGE OF DISPLACEMENT

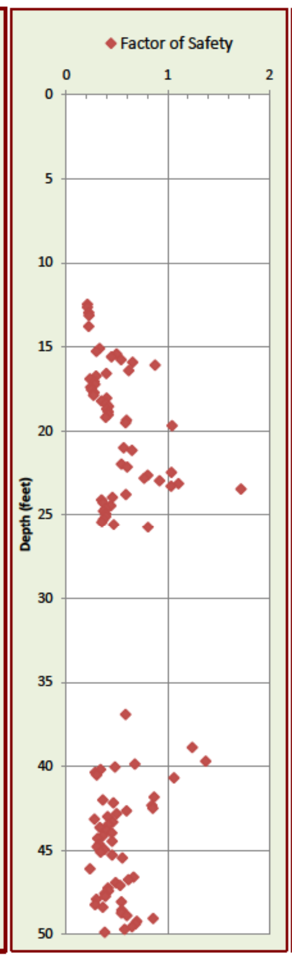
0.2 to 0.8 feet

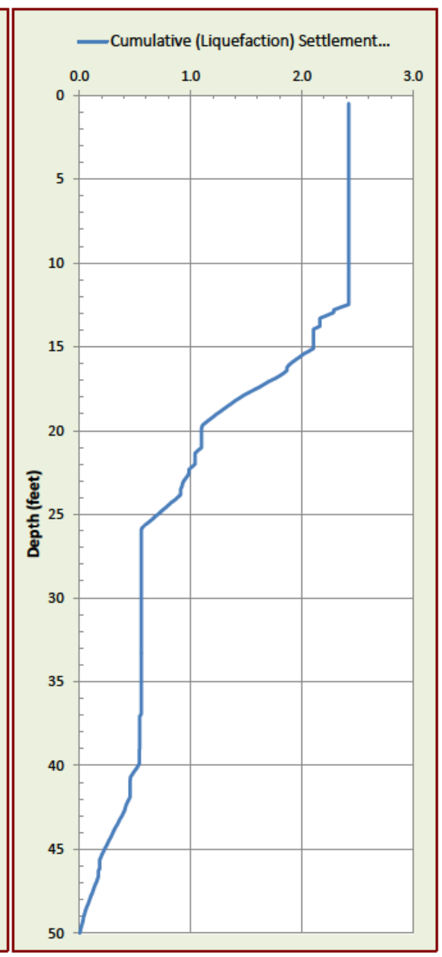
[1]Not Valid for L/H Values < 4 and > 40.

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









CORNERSTONE EARTH GROUP

FIGURE 4F

CPT NO. 6

PROJECT/CPT DATA

Project Title	Hanson Self Storage
Project No.	726-1-3
Project Manager	NSD

SEISMIC PARAMETERS

Controlling Fault	Hayward	
Earthquake Magnitude (Mw)	7.1	
PGA (Amax)	0.7	(g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet)	8
Design Water Depth (feet)	5
Ave. Unit Weight Above GW (pcf)	125
Ave. Unit Weight Below GW (pcf)	120

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 5 FEET

0.00 (Inches)

LIQUEFACTION SETTLEMENT FROM 50 FEET

1.30 (Inches)

TOTAL SEISMIC SETTLEMENT 1.3 INCHES

POTENTIAL LATERAL DISPLACEMENT

LDI[2] 0.21 L/H 43.5

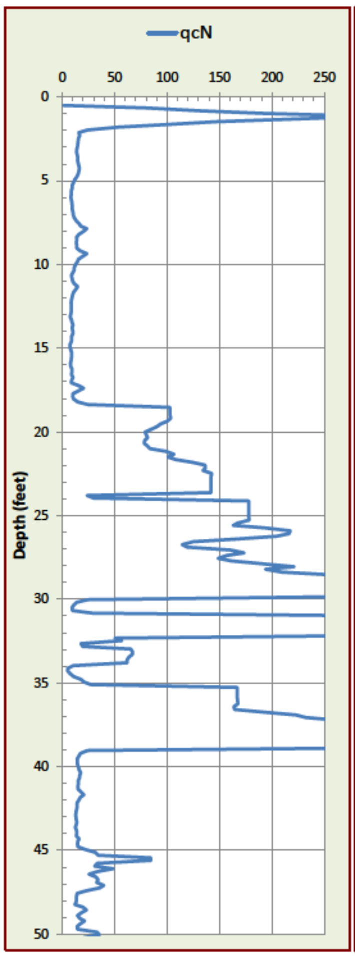
LDI[1] Corrected for Distance 0.06 (4 < L/H < 40)

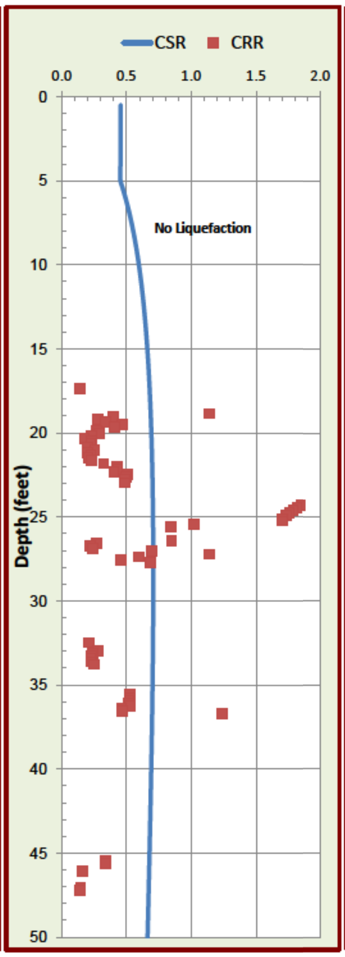
EXPECTED RANGE OF DISPLACEMENT

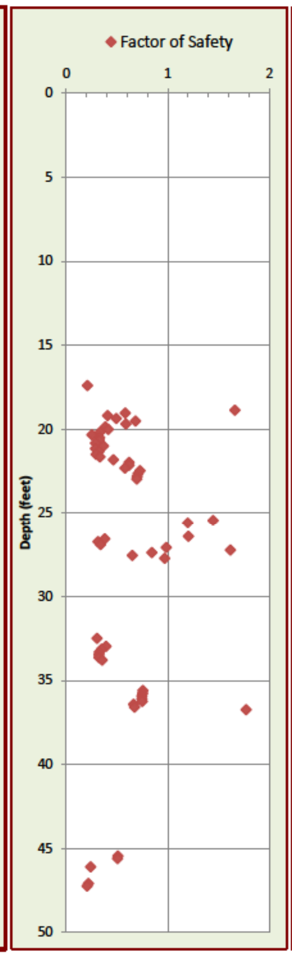
0.0 to 0.1 feet

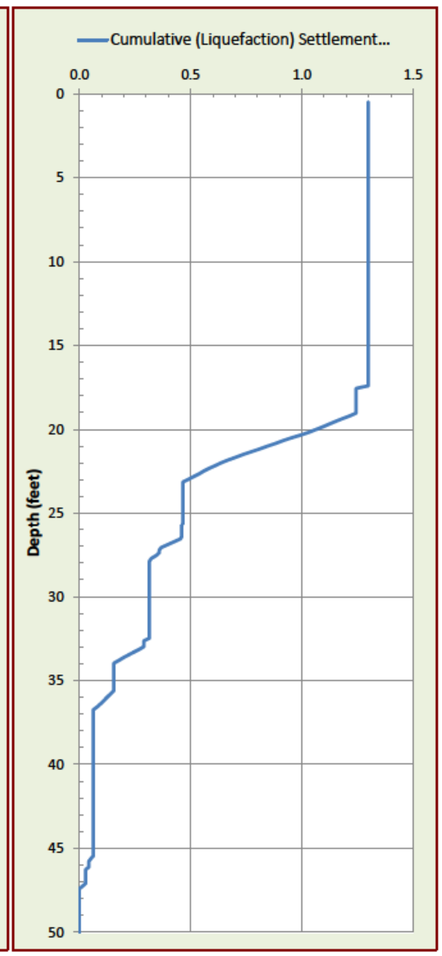
[1]Not Valid for L/H Values < 4 and > 40.

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









CORNERSTONE EARTH GROUP

FIGURE	4G
CPT NO.	7

PROJECT/CPT DATA

Project Title	Hanson Self Storage
Project No.	726-1-3
Project Manager	NSD

SEISMIC PARAMETERS

Controlling Fault	Hayward	
Earthquake Magnitude (Mw)	7.1	
PGA (Amax)	0.7	(g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet)	8
Design Water Depth (feet)	5
Ave. Unit Weight Above GW (pcf)	125
Ave. Unit Weight Below GW (pcf)	120

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 5 FEET

0.03 (Inches)

LIQUEFACTION SETTLEMENT FROM 50 FEET

2.54 (Inches)

TOTAL SEISMIC SETTLEMENT 2.6 INCHES

POTENTIAL LATERAL DISPLACEMENT

LDI[2] 0.90 L/H 24.0

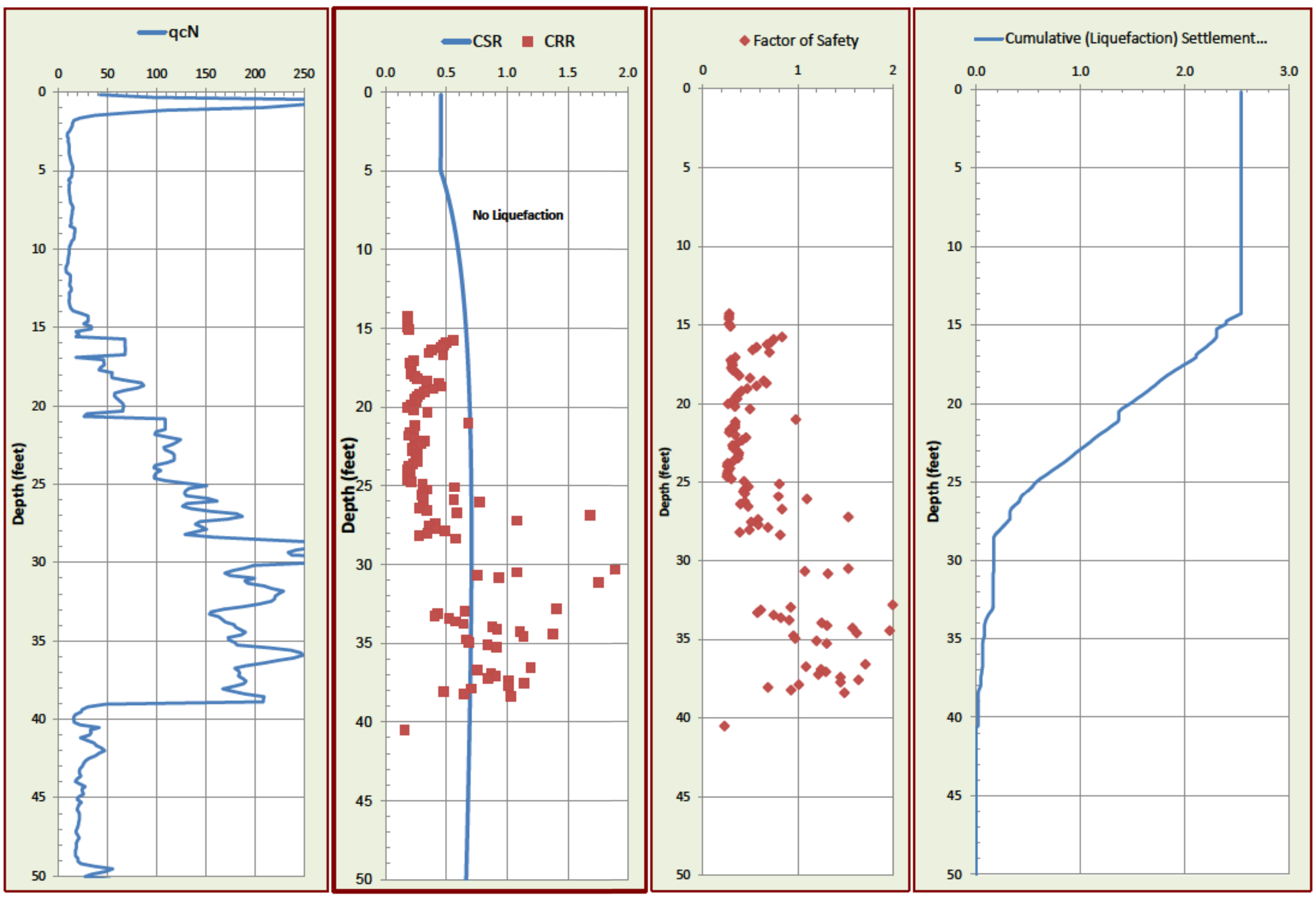
LDI[1] Corrected for Distance 0.43 (4 < L/H < 40)

EXPECTED RANGE OF DISPLACEMENT

0.2 to 0.9 feet

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

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



CORNERSTONE EARTH GROUP

FIGURE 4H

CPT NO. 8

PROJECT/CPT DATA

Project Title	**Hanson Self Storage**
Project No.	**726-1-3**
Project Manager	**NSD**

SEISMIC PARAMETERS

Controlling Fault	**Hayward**	
Earthquake Magnitude (Mw)	**7.1**	
PGA (Amax)	**0.7**	(g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet)	**8**
Design Water Depth (feet)	**5**
Ave. Unit Weight Above GW (pcf)	**125**
Ave. Unit Weight Below GW (pcf)	**120**

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 5 **FEET**

0.01 **(Inches)**

LIQUEFACTION SETTLEMENT FROM 50 **FEET**

2.09 **(Inches)**

TOTAL SEISMIC SETTLEMENT 2.1 **INCHES**

POTENTIAL LATERAL DISPLACEMENT

LDI[2] 0.99 **L/H** 23.0

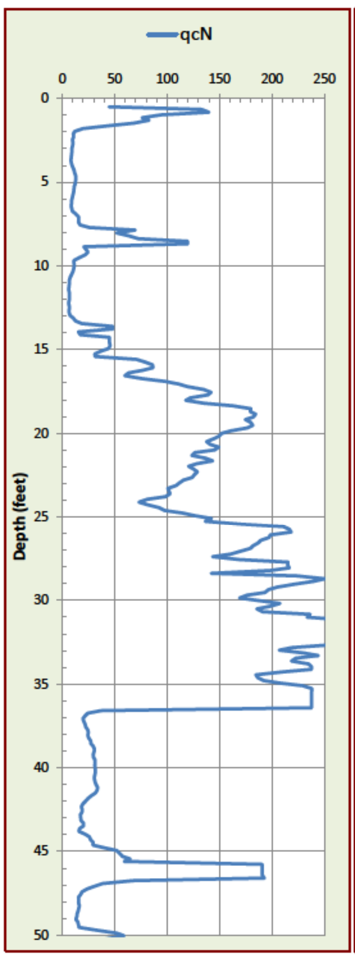
LDI[1] Corrected for Distance 0.48 (4 < L/H < 40)

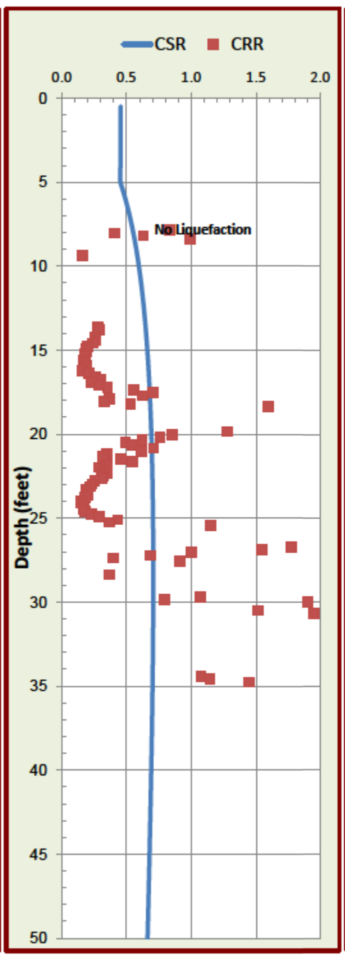
EXPECTED RANGE OF DISPLACEMENT

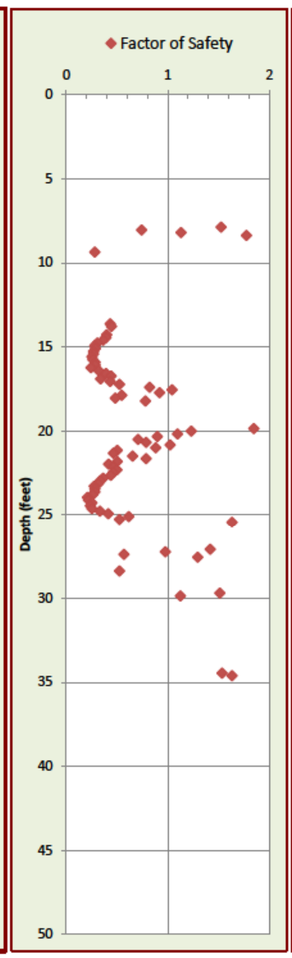
0.2 **to** **1.0** **feet**

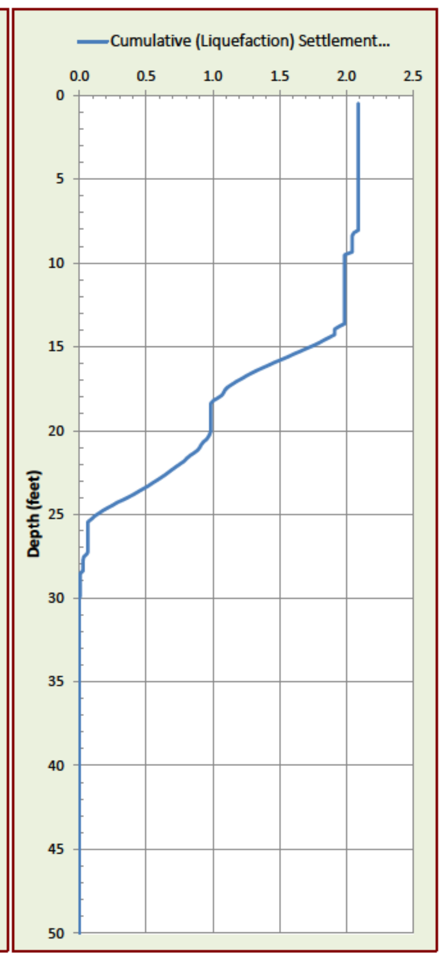
[1]Not Valid for L/H Values < 4 and > 40.

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









CORNERSTONE EARTH GROUP

FIGURE 4I

CPT NO. 9

PROJECT/CPT DATA

Project Title	Hanson Self Storage
Project No.	726-1-3
Project Manager	NSD

SEISMIC PARAMETERS

Controlling Fault	Hayward
Earthquake Magnitude (Mw)	7.1
PGA (Amax)	0.7 (g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet)	8
Design Water Depth (feet)	5
Ave. Unit Weight Above GW (pcf)	125
Ave. Unit Weight Below GW (pcf)	120

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 5 FEET

0.00 (Inches)

LIQUEFACTION SETTLEMENT FROM 50 FEET

0.49 (Inches)

TOTAL SEISMIC SETTLEMENT 0.5 INCHES

POTENTIAL LATERAL DISPLACEMENT

LDI[2] 0.00 L/H 12.7

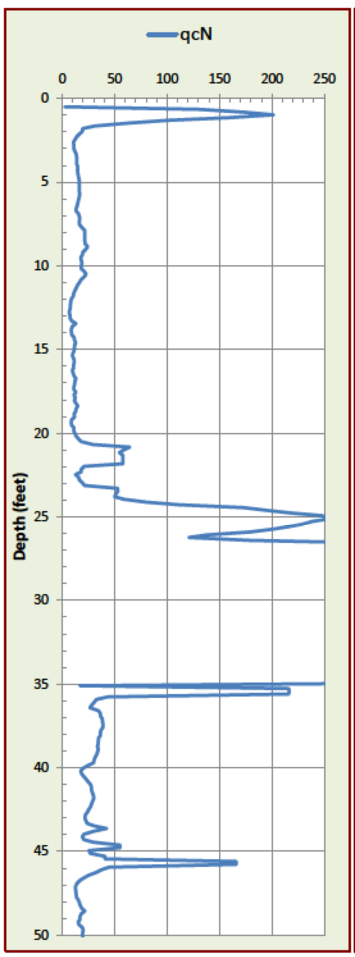
LDI[1] Corrected for Distance 0.00 (4 < L/H < 40)

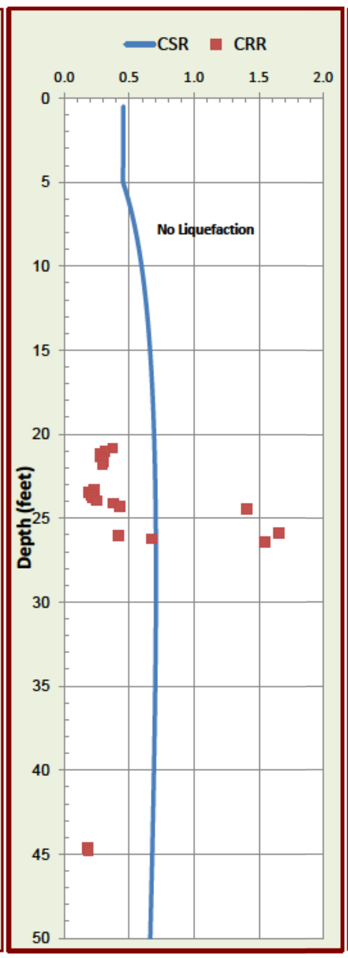
EXPECTED RANGE OF DISPLACEMENT

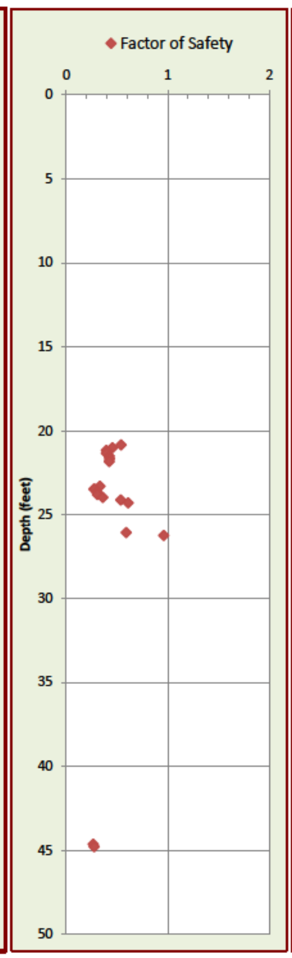
0.0 to 0.0 feet

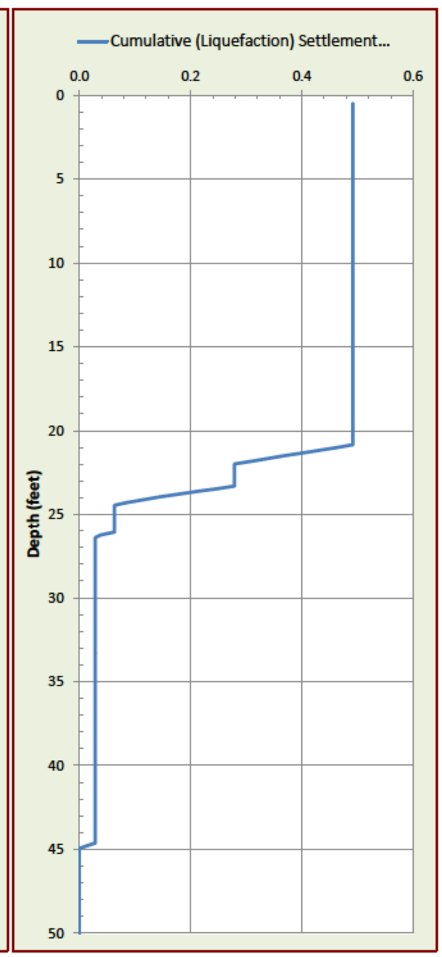
[1]Not Valid for L/H Values < 4 and > 40.

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









APPENDIX A: FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program using truck-mounted, hollow-stem auger drilling equipment and 20-ton truck-mounted CPT equipment. Five 8-inch-diameter exploratory borings were drilled on July 30, 2014, to depths of 20 to 40 feet. Nine CPT soundings were performed in accordance with ASTM D5778-95 (revised, 2002) on July 29, 2014, and August 1, 2014, to depths ranging from 50 to 86½ feet. The approximate locations of exploratory borings and CPT soundings are shown on the Site Plan, Figure 2. The soil encountered was continuously logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D2488). Boring logs and a key to the classification of the soil are included as part of this appendix.

Boring and CPT locations were approximated using existing site boundaries, a hand held GPS unit, and other site features as references. Boring and CPT elevations were not available at the time of this report. The locations of the borings and CPT soundings 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. 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 corresponding 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 silt and clay. 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. The results of these tests are presented on the individual boring logs at the corresponding sample depths.

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

UNIFIED SOIL CLASSIFICATION (ASTM D-2487-10)

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

OTHER MATERIAL SYMBOLS

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

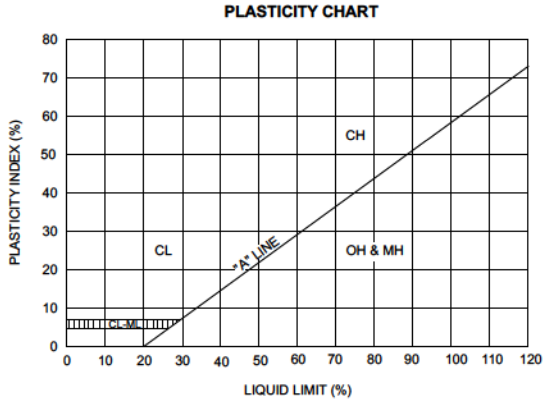
SAMPLER TYPES

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

ADDITIONAL TESTS

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

PLASTICITY CHART



PENETRATION RESISTANCE
(RECORDED AS BLOWS / FOOT)

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

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

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

CORNERSTONE EARTH GROUP

LEGEND TO SOIL DESCRIPTIONS

Figure Number A-1

BORING NUMBER EB-1

CORNERSTONE EARTH GROUP

PROJECT NAME Hanson Court Self Storage
PROJECT NUMBER 726-1-3
PROJECT LOCATION Milpitas, CA

DATE STARTED 7/30/14 **DATE COMPLETED** 7/30/14
GROUND ELEVATION **BORING DEPTH** 40 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE **LONGITUDE**
DRILLING METHOD Mobile B-40, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY RSM
▽ **AT TIME OF DRILLING** 14 ft.
NOTES
▼ **AT END OF DRILLING** 14 ft.

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

Column headings: 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

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

Depth scale (ft): 0, 5, 10, 15, 20, 25

DESCRIPTION

6½ inches Portland cement concrete

Clayey Sand with Gravel (SC) [Fill]
loose, moist, gray and brown, fine to coarse sand, fine to coarse angular gravel

Fat Clay (CH)
stiff to very stiff, moist, dark gray, trace fine sand, high plasticity

Lean Clay (CL)
stiff, moist, brown with gray mottles, some fine to coarse sand, moderate plasticity

Silty, Clayey Sand (SC-SM)
loose, wet, brown, fine to medium sand

Liquid Limit = 24, Plastic Limit = 18

Poorly Graded Sand with Silt (SP-SM)
medium dense, wet, gray, fine to coarse sand

dense

very dense

N-Value (blows per foot)	Sample	Dry Unit Weight (PCF)	Natural Moisture Content (%)	Plasticity Index (%)
6	MC-1B	91	29	
11	MC-2	78	32	
19	MC-3B	92	28	
9	MC-4B	102	22	
8	SPT-5		28	6
23	SPT			
47	SPT			
63	SPT			

Continued Next Page

CORNERSTONE EARTH GROUP

PROJECT NAME Hanson Court Self Storage

PROJECT NUMBER 726-1-3

PROJECT LOCATION Milpitas, CA

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
			Poorly Graded Sand with Silt (SP-SM) medium dense, wet, gray, fine to coarse sand							
			very dense	61	SPT					
	30									
				27	SPT					
	35									
				18	SPT					
			Lean Clay with Sand (CL) stiff, moist, gray to brown, fine to medium sand, low to moderate plasticity							
				13	MC					○
	40		Bottom of Boring at 40.0 feet.							
	45									
	50									
	55									

BORING NUMBER EB-2

PAGE 1 OF 1

CORNERSTONE EARTH GROUP

PROJECT NAME: Hanson Court Self Storage
PROJECT NUMBER: 726-1-3
PROJECT LOCATION: Milpitas, CA

DATE STARTED: 7/30/14 DATE COMPLETED: 7/30/14
GROUND ELEVATION: BORING DEPTH: 30 ft.
DRILLING CONTRACTOR: Exploration Geoservices, Inc.
LATITUDE: LONGITUDE:
DRILLING METHOD: Mobile B-40, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY: RSM
AT TIME OF DRILLING: 12.5 ft.
NOTES:
AT END OF DRILLING: 12.5 ft.

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

DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
0	8 inches Portland cement concrete							
	Sandy Lean Clay (CL) [Fill] very stiff, moist, gray and brown mottled, fine to coarse sand, moderate plasticity; Liquid Limit = 35 , Plastic Limit = 19	22	1A		16	16		>4.5
			MC 1B	108	19			
	Fat Clay (CH) stiff, moist, dark gray, trace fine sand, high plasticity	11	MC-2B	88	27			
5		19	MC-3B	87	32			
	Lean Clay with Sand (CL) very stiff, moist, brown and light brown mottled, fine to coarse sand, moderate plasticity	16	MC-4B	104	22			
10								
	Silty, Clayey Sand (SC-SM) loose, wet, brown, fine to medium sand	4	SPT-5		26			
15	**Sandy Silt (ML)** soft, wet, brown, fine sand, low plasticity	7	SPT					
		6	SPT					
20								
	Silty Sand (SM) medium dense, moist, gray, fine to medium sand	15	SPT					
25								
30	**Poorly Graded Sand with Silt (SP-SM)** dense, wet, gray, fine to coarse sand	37	SPT					

Bottom of Boring at 30.0 feet.

CORNERSTONE EARTH GROUP

BORING NUMBER EB-3

PROJECT NAME Hanson Court Self Storage
PROJECT NUMBER 726-1-3
PROJECT LOCATION Milpitas, CA

DATE STARTED 7/30/14 DATE COMPLETED 7/30/14
GROUND ELEVATION BORING DEPTH 20 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE LONGITUDE
DRILLING METHOD Mobile B-40, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY RSM
AT TIME OF DRILLING 8 ft.
NOTES
AT END OF DRILLING 8 ft.

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

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
	0	8½ inches Portland cement concrete							
		Lean Clay with Sand (CL) [Fill] very stiff, moist, gray to brown, fine to coarse sand, low plasticity	15	MC-1B	95	22			○
		Poorly Graded Sand (SP) [Fill] loose, moist, brown, fine to medium sand	12	MC-2B	90	27			○
	5	**Fat Clay (CH)** very stiff, moist, dark gray, trace fine sand, high plasticity	16	MC-3B	91	31			○
		Lean Clay with Sand (CL) hard to very stiff, moist, brown with light brown mottles, fine to coarse sand, moderate plasticity	16	MC-4B	113	17			>4.5 ○
			18	MC-5B	113	17			○
	10	**Poorly Graded Sand with Clay (SP-SC)** medium dense, wet, brown, fine to coarse sand	10	SPT					
		Silty, Clayey Sand (SC-SM) loose, wet, brown, fine to medium sand	4	SPT-7		27		38	
	15	**Silty Sand (SM)** loose, wet, light brown, fine to medium sand NP= Non Plastic	5	SPT-8		25	NP		
			9	SPT-9		25			
	20	Bottom of Boring at 20.0 feet.							
	25								

BORING NUMBER EB-4

CORNERSTONE EARTH GROUP

PROJECT NAME Hanson Court Self Storage
PROJECT NUMBER 726-1-3
PROJECT LOCATION Milpitas, CA

DATE STARTED 7/30/14 **DATE COMPLETED** 7/30/14
GROUND ELEVATION **BORING DEPTH** 25 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE **LONGITUDE**
DRILLING METHOD Mobile B-40, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY RSM
▽ **AT TIME OF DRILLING** 12 ft.
NOTES
▼ **AT END OF DRILLING** 12 ft.

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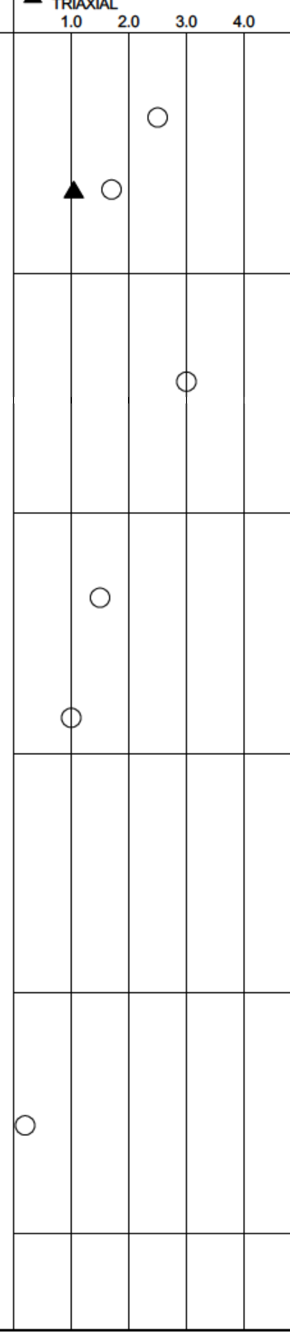
ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
	0	10 inches Portland cement concrete						
		Fat Clay (CH) [Fill]						
		Fat Clay (CH) very stiff to stiff, moist, dark gray, trace fine sand, high plasticity	19	MC-1B	85	30		
			14	MC-2B	88	34		
	5							
		Lean Clay with Sand (CL) very stiff, moist, brown and light brown mottled, fine to coarse sand, moderate plasticity; Liquid Limit = 41 , Plastic Limit = 14	25	MC-3B	117	15	27	
	10							
		becomes stiff	11	MC-4B	98	26		
			8	SPT-5		27		
	15							
		Silty, Clayey Sand (SC-SM) medium dense, wet, brown to gray, fine to medium sand; Liquid Limit = 27 , Plastic Limit = 21	11	SPT-6		30	6	
	20							
		Sandy Silty Clay (CL-ML) soft, wet, gray, fine sand, low plasticity	6	SPT				
		Silty, Clayey Sand (SC-SM) medium dense, wet, gray, fine sand	3	SPT				
	25	Bottom of Boring at 25.0 feet.						

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



BORING NUMBER EB-5

PAGE 1 OF 1

CORNERSTONE EARTH GROUP

PROJECT NAME Hanson Court Self Storage
PROJECT NUMBER 726-1-3
PROJECT LOCATION Milpitas, CA

DATE STARTED 7/30/14 DATE COMPLETED 7/30/14
GROUND ELEVATION BORING DEPTH 21.5 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE LONGITUDE
DRILLING METHOD Mobile B-40, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY RSM
AT TIME OF DRILLING 11 ft.
NOTES
AT END OF DRILLING 11 ft.

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

DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
0	5½ inches Portland cement concrete							
	Fat Clay (CH) very stiff to stiff, moist, dark gray, some fine sand, high plasticity	14	MC-1B	83	33			○
		9	MC-2B	89	33			○
5	**Fat Clay (CH)** stiff, moist, brown, some fine sand, high plasticity Liquid Limit = 65 , Plastic Limit = 15	10	MC-3B	95	28	50		○
		11	MC-4B	95	28			○
10								
11	**Silty, Clayey Sand (SC-SM)** loose to medium dense, wet, brown to gray, fine to medium sand	10	MC-5B	100	25			
		10	MC-6B	100	25			
15		11	SPT					
17	**Silty Sand (SM)** medium dense, wet, brown to gray, fine sand	31	SPT					
19	**Sandy Silty Clay (CL-ML)** soft, wet, gray, fine sand, low plasticity							
20		9	SPT					○
21.5	Bottom of Boring at 21.5 feet.							
25								

APPENDIX B: LABORATORY TEST PROGRAM

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

Moisture Content: The natural water content was determined (ASTM D2216) on 31 samples of the materials recovered from the borings. These water contents are recorded on the boring logs at the corresponding sample depths.

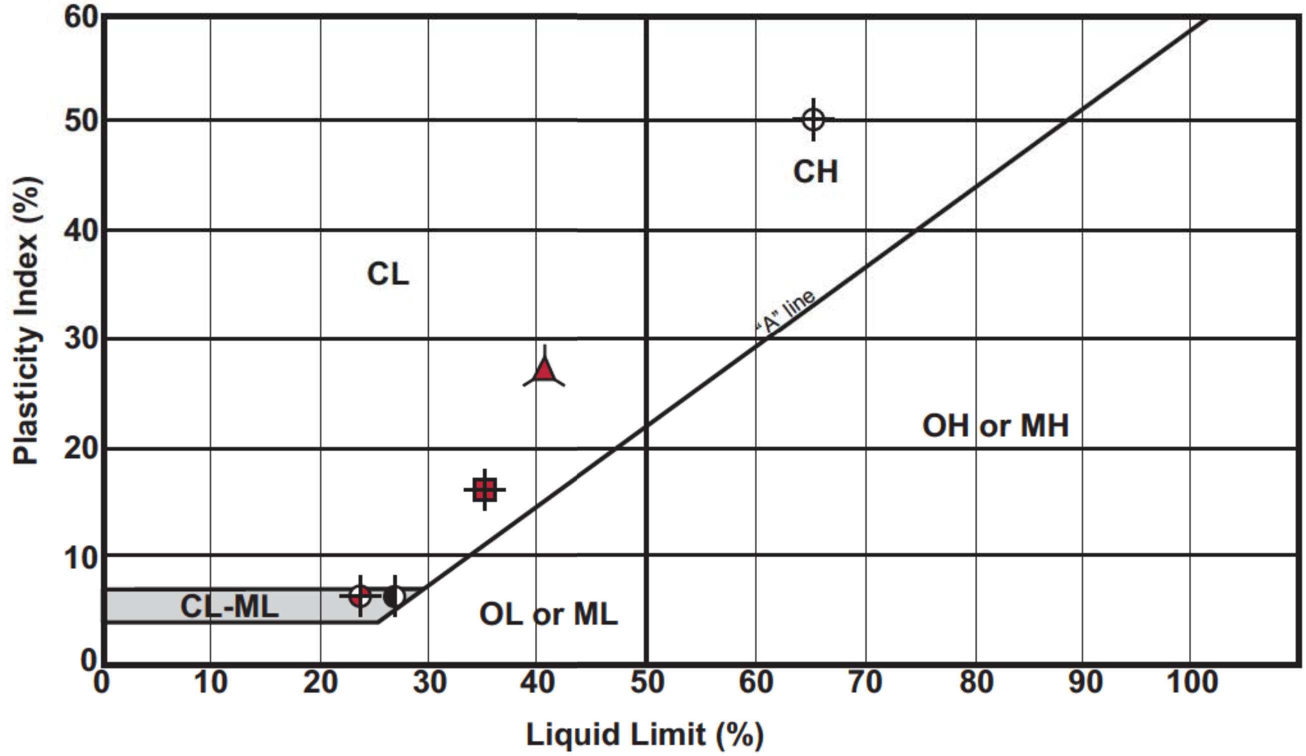
Dry Densities: In place dry density determinations (ASTM D2937) were performed on 21 samples to measure the unit weight of the subsurface soil. Results of these tests are shown on the boring logs at the corresponding sample depths.

Plasticity Index: Seven Plasticity Index tests (ASTM D4318) were performed on samples of the subsurface soil 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 corresponding sample depths.

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

Soluble Sulfate: Two soluble sulfate determinations (California Test Method No. 417-Modified) were performed on samples of the subsurface soil to measure the water soluble sulfate content. Results of these tests are attached is this appendix.

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	13.5	28	24	18	6	—	Silty, Clayey Sand (SC-SM)
	EB-2	1.5	16	35	19	16	---	Sandy Lean Clay (CL) [Fill]
	EB-3	17.0	25	determined non-plastic			—	Silty Sand (SM)
	EB-4	7.5	15	41	14	27	—	Lean Clay with Sand (CL)
	EB-4	19.0	30	27	21	6	—	Silty, Clayey Sand (SC-SM)
	EB-5	7.5	28	65	15	50	—	Fat Clay (CH)

Samples prepared in accordance with ASTM D421

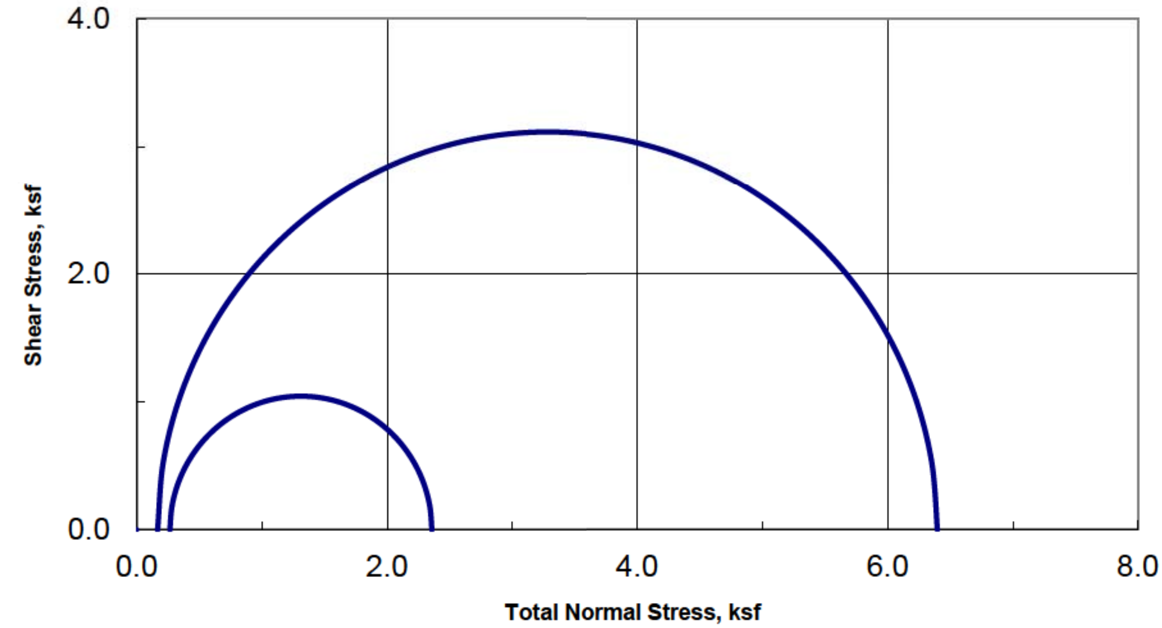
CORNERSTONE EARTH GROUP

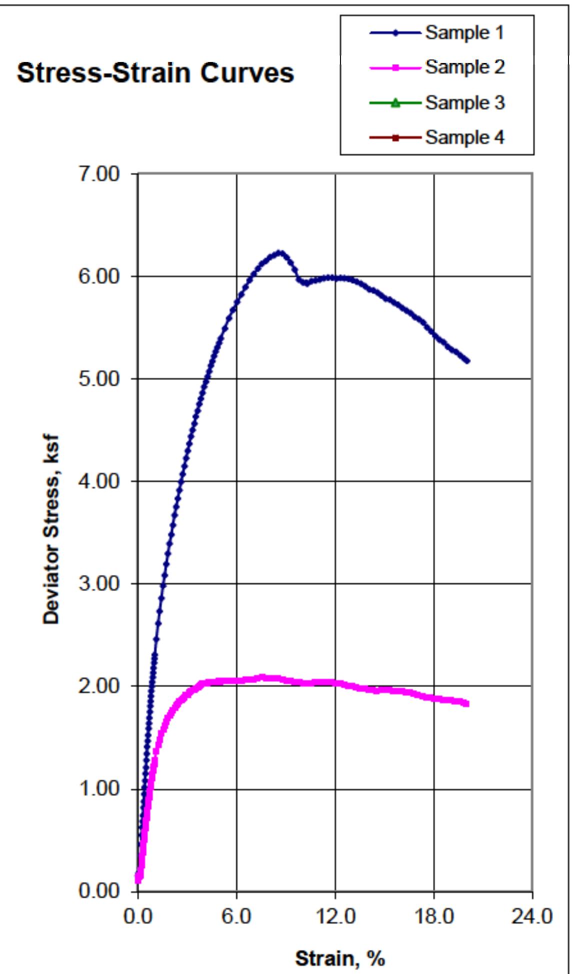
Plasticity Index Testing Summary	Project Number 726-1-3
Hanson Self Storage 1 Hanson Court Milpitas, CA	Figure Number Figure B1
	Date August 2014 — Drawn By FLL

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

Unconsolidated-Undrained Triaxial Test

ASTM D2850





Sample Data	1	2	3	4
Moisture %	19.1	34.2		
Dry Den,pcf	108.3	87.5		
Void Ratio	0.556	0.927		
Saturation %	92.8	99.7		
Height in	5.00	5.05		
Diameter in	2.41	2.40		
Cell psi	1.2	1.8		
Strain %	8.54	7.53		
Deviator, ksf	6.229	2.090		
Rate %/min	1.00	0.99		
in/min	0.050	0.050		
Job No.:	640-707			
Client:	Cornerstone Earth Group			
Project:	Hanson Ct Storage - 726-1-3			
Boring:	EB-2	EB-4		
Sample:	1B	2B		
Depth ft:	2.0	3.5		

Visual Soil Description	
Sample #	
1	Black Clayey SAND
2	Black CLAY
3	
4	

Remarks:

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

Corrosivity Tests Summary

CTL #: 640-707 **Date:** 8/5/2014 **Tested By:** PJ **Checked:** PJ
Client: Cornerstone Earth Group **Project:** Hanson Ct Storage **Proj. No:** 726-1-3
Remarks:

Sample Location or ID			Resistivity @ 15.5 °C (Ohm-cm)			Chloride	Sulfate		pH	ORP		Sulfide	Moisture	Soil Visual Description
			As Rec.	Min	Sat.	mg/kg	mg/kg	%		(Redox)		Qualitative	At Test	
						Dry Wt.	Dry Wt.	Dry Wt.		E_H (mv)	At Test	by Lead	%	
Boring	Sample, No.	Depth, ft.	ASTM G57	Cal 643	ASTM G57	ASTM D4327	ASTM D4327	ASTM D4327	ASTM G51	ASTM G200	Temp °C	Acetate Paper	ASTM D2216	
EB-3	3A	4.5	-	-	975	4	37	0.0037	8.1	-	-	-	28.9	Dark Gray CLAY w/ CaCO3
EB-5	1A	1.5	-	-	1,130	18	71	0.0071	7.9	-	-	-	16.4	Brown Clayey SAND w/ Gravel

APPENDIX E

MRP CALCULATIONS

SCVURPPP METHOD
SIZING FOR FLOW & VOLUME-BASED TREATMENT MEASURES
1 HANSON COURT Treatment Control Measure 01

STEP 1 **Contributing drainage area to the treatment measure:** **164,560**

STEP 2 Determine the equivalent impervious area draining to the treatment measure:

Impervious area draining to the treatment measure: **153,810** sq. ft.
Pervious area draining to the treatment measure: **10,750** sq. ft.

For grass, landscapeing or pervious paving, multiply the pervious area by a runoff coefficient of 0.10 to compute the equivalent impervious area.

Equivalent impervious area = pervious area x 0.10 = **1,075** sq. ft.

Total equivalent impervious area: **154,885** sq. ft.

STEP 3 Determine the required treatment volume using Adapted CASQA Stormwater BMP Handbook Approach

Volume Calculation:

Mean Annual Precipitation$_{(Site)}$	Map_{site}=	14.5	inches
Mean Annual Precipitation$_{(Gage)}$	Map_{gage}=	13.9	inches
Correction Factor		1.04	
Soil Type	Type D	Sandy Clay	
Average Slope of Site	s=	1.0%	
Unit Basin Storage	$UBS_{1\%}$=	0.58	inches
Unit Basin Storage	$UBS_{15\%}$=	0.60	inches
Adjusted Unit Basin Storage (UBS) Volume:	UBS_{site}=	**0.5800**	inches

Water Quality Design (WQD) Volume: **7,809** cu. ft.

STEP 4 Determine the design rainfall intensity (Section III.B, Step 7, or Section III.C, Step 3):

Design Rainfall Intensity: **0.2** in/hr

STEP 5 Assume that the rain event that generates the Unit Basin Storage Volume of runoff occurs at the design rainfall intensity for the entire length of the storm. Calculate the duration of the storm by dividing the adjusted Unit Basin Storage Volume by the design rainfall intensity. In other words, determine the amount of time required for the Unit Basin Storage Volume to

Duration = UBS Volume (inches) / Rainfall Intensity (inches/hour)
Duration = (Step 3) / (Step 4) = **2.90**

STEP 6 Make a preliminary estimate of the surface area of the bioretention facility by multiplying the area of impervious surface to be treated by a sizing factor of **0.03**.

Estimated Surface Area = **154,885** sq. ft. x **0.03** = **4,647** sq. ft.

Assume the modified surface area is 105%
of the preliminary estimate above, or **4,880** sq. ft.

STEP 7 Calculate the volume of runoff that filters through the biotreatment soil at a rate of 5 inches per hour (the design surface loadig rate for the bioretention facilities), for

Volume of Treated Runoff = Estimated Surface Area x 5 in/hr x (1ft/12in) x Duration

Volume of Treated Runoff = **5,896** cu. ft.

STEP 8 Calculate the portion of the water quality design (WQD) volume remaining after treatment is accomplished by filtering through the biotreatment soil. The result is the amount that Step 6.

Volume in ponding area = WQD Volume - Volume of Treated Runoff

Volume in ponding area = **1,913** cu. ft.

STEP 9 Calculate the depth of the volume in the ponding area by dividing this volume by the estimated surface area in Step 6.

Depth of ponding = Volume in Ponding Area / Estimated Survey Area

Depth of ponding = **0.39** ft
or **4.7** inches

Ponding shall be between 0.5' and 1.0'

APPENDIX F

BMP SIZING CALCULATIONS

STORAGE STORMWATER CALCULATIONS										
DMA	**TOTAL**	**PERV**	**IMPERV**	**BIO (REQ'D)**	**BIO (PROV'D)**	**4% RULE**	**EQ. IMPERV**	**TOTAL IMPR**	**COMBO CALC BIO (REQ'D)**	**Y/N**
1	164,560	10,750	153,810	6,152.4	4,880	NO	1075.0	154,885.0	4646.6	YES
TOTAL	164,560	10,750	153,810	6,152.4						

OVERALL TREATMENT AREA TOTALS

PERVIOUS AND IMPERVIOUS SURFACES COMPARISON TABLE

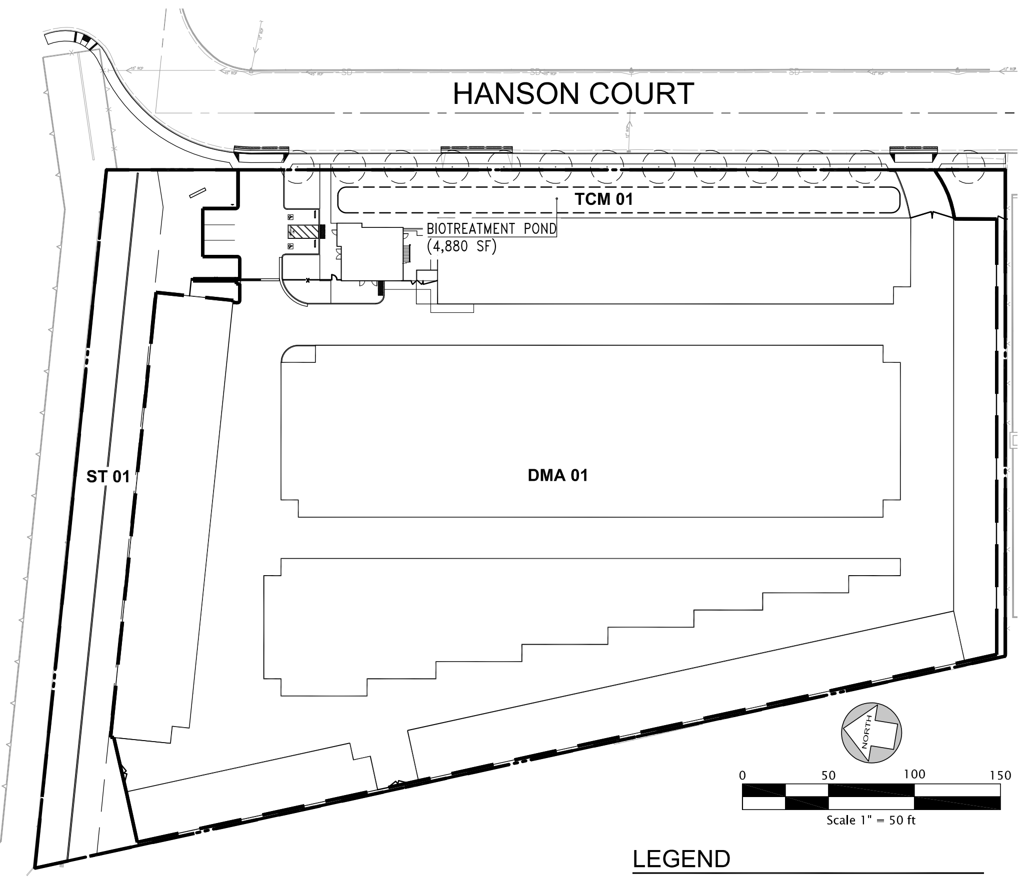
a. PROJECT PHASE NUMBER: ROOF AREA(S)	N/A	b. TOTAL SITE (ACRES):	4.27
c. TOTAL SITE EXISTING IMPERVIOUS SURFACES (S.F.)	0	d. TOTAL AREA OF SITE DISTURBED (ACRES):	4.27

e. IMPERVIOUS SURFACES	EXISTING CONDITION (S.F.)	REPLACED (S.F.)	NEW (S.F.)
ROOF AREA(S)	14,590	14,590	86,308
PARKING	0	0	51,444
S/W, PATIOS, PATHS ETC.	171,286	1,468	0
STREETS (PUBLIC)	0	0	0
STREETS/E.V.A (PRIVATE)	0	0	0
TOTAL IMPERVIOUS SURFACES	0	16,058	137,752
f. PERVIOUS SURFACES			
LANDSCAPING	0	0	32,066
PERVIOUS PAVING	0	0	0
OTHER PERVIOUS SURFACES	0	0	0
TOTAL PERVIOUS SURFACES	0	0	32,066

g) TOTAL PROPOSED REPLACED + NEW IMPERVIOUS SURFACES:	153,810
h) TOTAL PROPOSED REPLACED + NEW PERVIOUS SURFACES:	32,066
i) % OF REPLACEMENT OF IMPERVIOUS AREA IN REDEVELOPMENT PROJECTS:	0%

BIOTREATMENT MAINTENANCE

INSPECTION ACTIVITIES	SUGGESTED FREQUENCY
• INSPECT AFTER SEEDING AND AFTER FIRST MAJOR STORMS FOR ANY DAMAGES.	POST-CONTRUCTION
• INSPECT FOR SIGNS OF EROSION, DAMAGE TO VEGETATION, CHANNELIZATION OF FLOW, DEBRIS AND LITTER, AND AREAS OF SEDIMENT ACCUMULATION. PERFORM INSPECTIONS AT THE BEGINNING AND END OF THE WET SEASON. ADDITIONAL INSPECTIONS AFTER PERIODS OF HEAVY RUNOFF ARE DESIRABLE.	SEMI-ANNUAL
• INSPECT GRASS ALONG SIDE SLOPES FOR EROSION AND FORMATION OF RILLS OR GULLIES, AND SAND/SOIL BED FOR EROSION PROBLEMS.	ANNUAL
MAINTENANCE ACTIVITIES	**SUGGESTED FREQUENCY**
• MOW GRASS TO MAINTAIN A HEIGHT OF 3-4 INCHES, FOR SAFETY, AESTHETIC, OR OTHER PURPOSES. LITTER SHOULD ALWAYS BE REMOVED PRIOR TO MOWING. CLIPPINGS SHOULD BE COMPOSTED. • IRRIGATE DURING DRY SEASON (APRIL THROUGH OCTOBER) OR WHEN NECESSARY TO MAINTAIN THE VEGETATION. • PROVIDE WEED CONTROL, IF NECESSARY TO CONTROL INVASIVE SPECIES.	AS NEEDED (FREQUENT, SEASONALLY)
• REMOVE LITTER, BRANCHES, ROCKS BLOCKAGES AND OTHER DEBRIS AND DISPOSE OF PROPERLY. • REPAIR ANY DAMAGED AREAS IDENTIFIED DURING INSPECTIONS. EROSION RILLS OR GULLIES SHOULD BE CORRECTED AS NEEDED. BARE AREAS SHOULD BE REPLANTED AS NECESSARY.	SEMI-ANNUAL
• CORRECT EROSION PROBLEMS IN THE SAND/SOIL BED. • PLANT AN ALTERNATIVE GRASS SPECIES IF THE ORIGINAL GRASS COVER HAS NOT BEEN SUCCESSFULLY ESTABLISHED. RESEED AND APPLY MULCH TO DAMAGED AREAS.	ANNUAL (AS NEEDED)
• REMOVE ALL ACCUMULATED SEDIMENT THAT MAY OBSTRUCT THE PROPER OPERATION OF THE BIO TREATMENT POND. SEDIMENT SHOULD BE REMOVED WHEN IT BUILDS UP TO 3 IN. AT ANY SPOT, OR COVERS VEGETATION, OR ONCE IT HAS ACCUMULATED TO 10% OF THE ORIGINAL DESIGN VOLUME. REPLACE THE GRASS AREAS DAMAGED IN THE PROCESS. • ROTOTILL OR CULTIVATE THE SURFACE OF THE SAND/SOIL BED OF IF THE TREATMENT AREA DOES NOT DRAW DOWN WITHIN 48 HOURS.	AS NEEDED (INFREQUENT)



LEGEND

(dashed box)	BIOTREATMENT POND/PLANTER
(heavy line)	TREATMENT AREA LIMITS
TCM	TREATMENT CONTROL MEASURE
DMA	DRAINAGE MANAGMENT AREA
ST	SELF TREATING

STORMWATER CONTROL NOTES

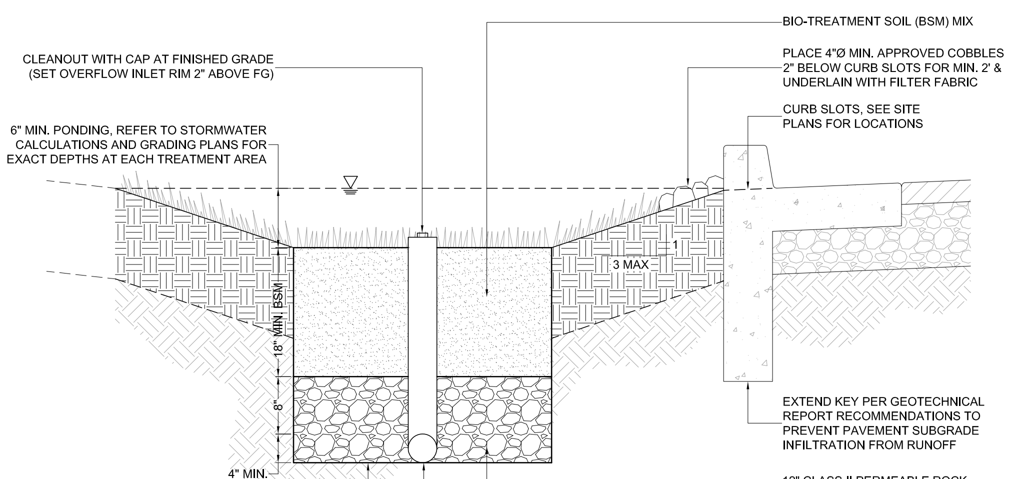
1. THE EXISTING SITE SOILS CONSIST OF CLAY (TYPE D) SOILS.
2. POTENTIAL POLLUTANTS INCLUDE MOTOR VEHICLE LUBRICANTS, COOLANTS, DISC BRAKE DUST, LITTER AND DEBRIS. POLLUTANT SOURCE AREAS INCLUDE THE ASPHALT CONCRETE PARKING LOT AND DRIVE AISLES, THE ROOF OF THE BUILDING, AND THE SITE STORM DRAIN INLETS. ALL INLETS WILL BE MARKED "NO DUMPING - DRAINS TO BAY". THE PARKING LOT SHALL BE SWEPT REGULARLY TO PREVENT THE ACCUMULATION OF LITTER AND DEBRIS.
3. BIOTREATMENT SIZING IS BASED ON THE COMBINATION FLOW/VOLUME BASED METHOD PER SCVURPPP HANDBOOK CHAPTER 5.

SOURCE CONTROL MEASURES IMPLEMENTED

SD-10: SITE DESIGN & LANDSCAPE PLANNING
- MAXIMIZED TREES AND PLANTING WITHIN HARDSCAPE AND LANDSCAPE AREAS.
- VEGETATED SLOPES FOR ALL LANDSCAPE SLOPES LESS THAN 1:5 SLOPE.

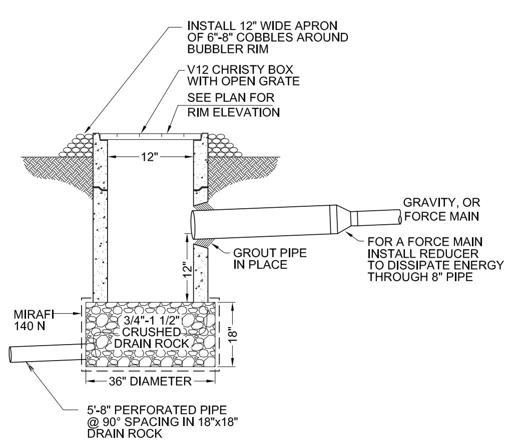
SD-11: EFFICIENT IRRIGATION
- RAIN-TRIGGERED SHUTOFF DEVICES TO PREVENT IRRIGATION AFTER PRECIPITATION.
- SYSTEM DESIGNED TO SITE-SPECIFIC WATER DEMANDS AND PLANTING REQUIREMENTS.

SD-13: STORM DRAIN SIGNAGE
- ALL CATCH BASINS TO BE STENCILED WITH PROHIBITIVE LANGUAGE PER CITY STANDARDS.



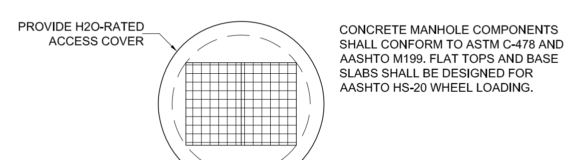
1. BIORETENTION AREAS SHALL BE CONSTRUCTED UNDER THE OBSERVATION OF THE SOILS ENGINEER.
2. SOIL AT BOTTOM OF RETENTION AREA SHALL HAVE A MINIMUM PERCOLATION RATE OF 5 INCHES/HOUR AND A MAXIMUM RATE OF 10 INCHES/HOUR.
3. IN-SITU TESTING SHALL BE PERFORMED BY THE SOILS ENGINEER BEFORE AND AFTER SOIL INSTALLATION TO VERIFY PERCOLATION RATE.

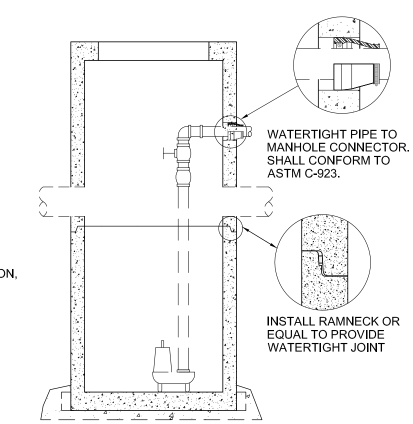
BIOTREATMENT POND DETAIL



SD BUBBLER DETAIL

STORM DRAIN PUMP NOTES





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- PROVIDE RAIL SYSTEMS FOR PUMP.
- PROVIDE 24"X36" ACCESS HATCH FOR PUMP.
- USE 3" SCHEDULE 40 PVC PIPING.

Revisions	Date

REGISTERED PROFESSIONAL ENGINEER NEKTARIOS MATHEOU 71236 CIVIL STATE OF CALIFORNIA

KIER & WRIGHT
CIVIL ENGINEERS & SURVEYORS, INC.
3350 Scott Boulevard, Building 22
Santa Clara, California 95054
(408) 727 6665
fax (408) 727 5641

SELF STORAGE
1 HANSON COURT
MILPITAS, CALIFORNIA

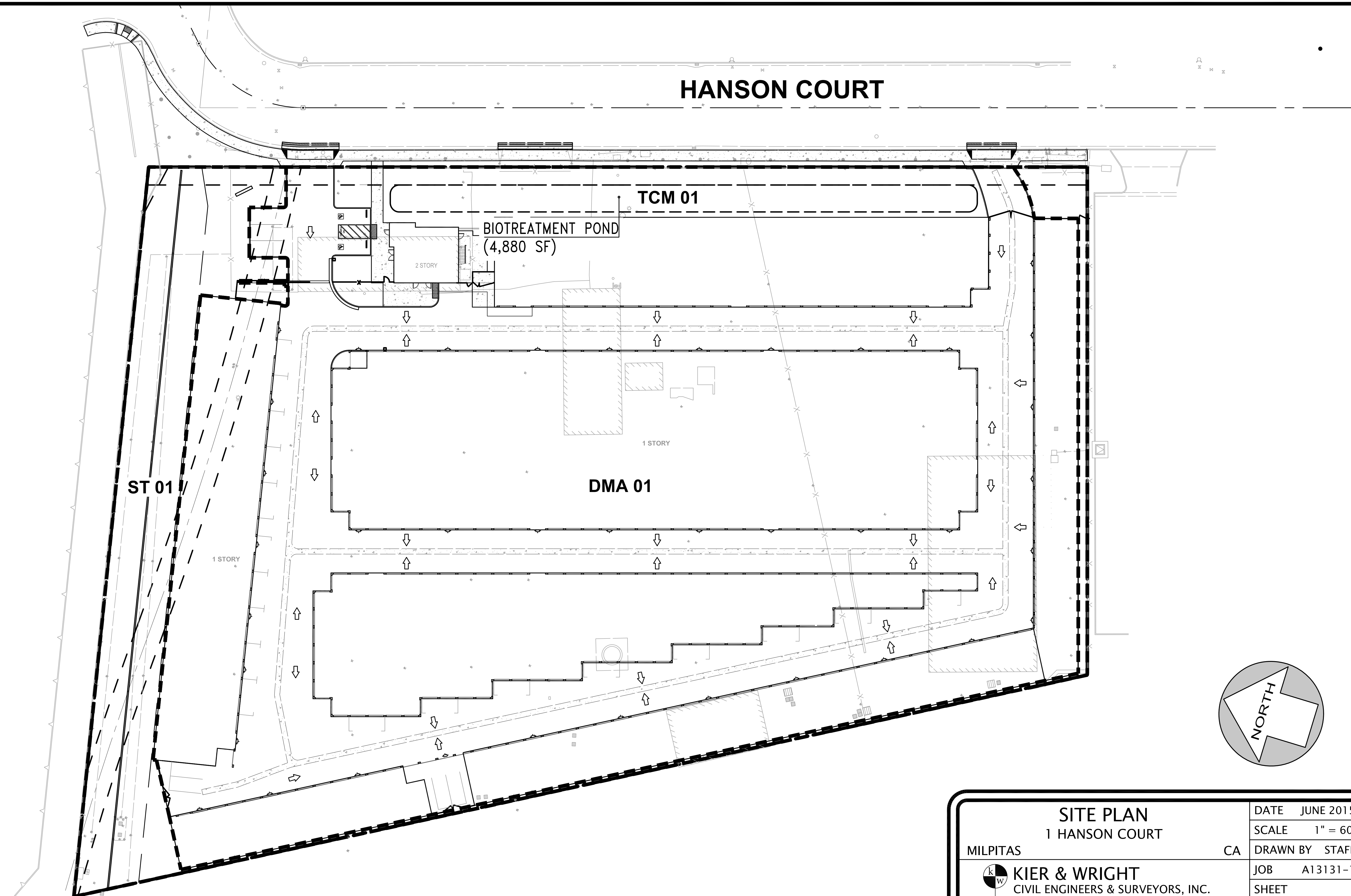
STORMATER CONTROL PLAN

Drawn By	STAFF
Date	06/01/15
Scale	AS SHOWN
File Name	
Planning File Numbers	
Sheet Number	C6.0

I:\PROJECTS\A13131\DWG\CIVIL\ C6.0 SWCP.dwg USER: jespartero DATE: 6/01/2015 - 2:36pm

APPENDIX G

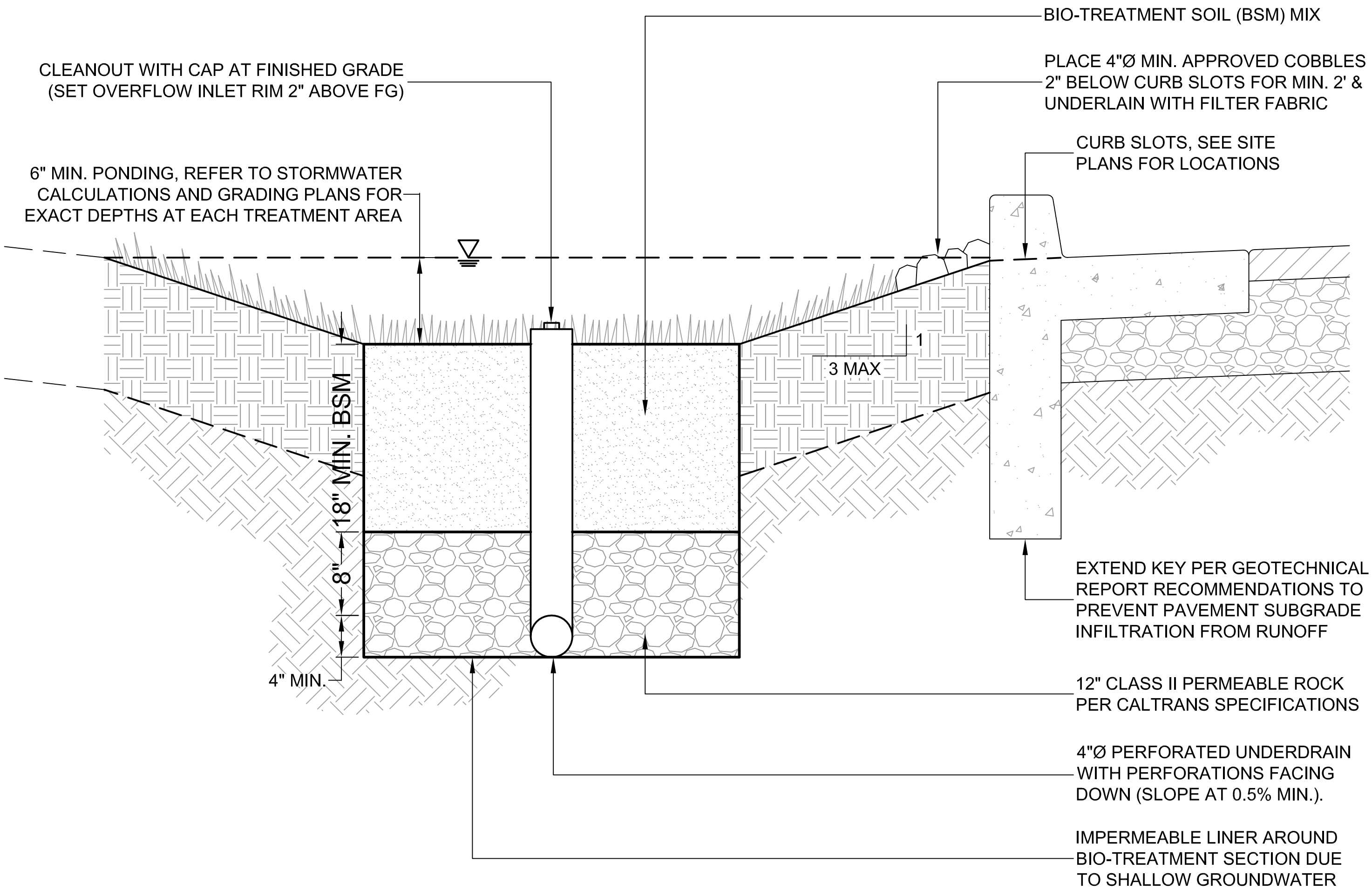
SITE PLAN



SITE PLAN 1 HANSON COURT MILPITAS CA	DATE JUNE 2015
	SCALE 1" = 60'
	DRAWN BY STAFF
KIER & WRIGHT CIVIL ENGINEERS & SURVEYORS, INC. 3350 Scott Boulevard, Building 22 (408) 727 6665 Santa Clara, California 95054 fax (408) 727 5641	JOB A13131-1
	SHEET 1 OF 1

APPENDIX H

TREATMENT MEASURE DETAILS



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BIOTREATMENT POND DETAIL

N.T.S

STORMWATER DETAIL

1 HANSON COURT

MILPITAS CA

DATE	JUNE 2015
SCALE	NTS
DR. BY	STAFF
JOB	A13131-1
SHEET NO.	1 OF 1

KIER & WRIGHT
CIVIL ENGINEERS & SURVEYORS, INC.
3350 Scott Boulevard, Building 22 (408) 727 6665
Santa Clara, California 95054 fax (408) 727 5641

APPENDIX I

OPERATIONS AND MAINTENANCE PLAN

Stormwater Treatment Measure Operation and Maintenance Inspection Report to the City of Milpitas, California

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: ____

Property Owner: ____

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:___________________________

Please submit the Operation and Maintenance Inspection reports, Maintenance Plan, and Inspection and Maintenance checklist for each BMP to the following address:

City of Milpitas
455 E. Calaveras Blvd.
Milpitas, CA 95035
Attn: Utility Engineer

Bioretention Area Maintenance Plan for
Self-Storage Facility,
1 Hanson Court, Milpitas, California, 95035
June, 2015

Project Address and Cross Streets____

Assessor's Parcel No.: ________________________________

Property Owner: __________________________ Phone No.:___________________

Designated Contact: _______________________ Phone No.:___________________

Mailing Address:____

The property contains one (1) bioretention area(s), located as described below and as shown in the attached site plan[1].

- **Bioretention Area No. 1** is located at [[north of Bldg B]].
- [[== Add descriptions of other bioretention areas, if applicable. ==]]

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 for ponded water. If ponded water does not drain within 2-3 days, till and replace the surface soil and replant.	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
5	Maintain vegetation and the irrigation system. Prune and weed to keep bioretention area neat and orderly in appearance. Remove and or replace any dead plants.	Twice a year
6	Check that mulch is at appropriate depth (2 inches per soil specifications) and replenish as necessary before wet season begins.	Monthly
7	Inspect the energy dissipation at the inlet to ensure it is functioning adequately, and that there is no scour of the surface mulch.	Annually, before the wet season begins
8	Inspect bioretention area using the attached inspection checklist.	Monthly, or after large storm events, and after removal of accumulated debris or material

[1] Attached site plan must match the site plan exhibit to Maintenance Agreement.

II. Use of Pesticides

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.

III. Vector Control

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 Clara Valley Vector Control District (District). Mosquito larvicides shall be applied only when absolutely necessary, as indicated by the District, and then only by a licensed professional or contractor. Contact information for the District is provided below.

Santa Clara Valley Vector Control District
1580 Berger Dr.
San José, California 95112
Phone: (408) 918-4770 / (800) 675-1155 - Fax: (408) 298-6356
www.sccgov.org/portal/site/vector

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: ____ Property Owner: ____________________________

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	Water stands in the bioretention area between storms and does not drain within 2-3 days after rainfall.			There should be no areas of standing water once storm event 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 2 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 2 inches.
7. Miscellaneous	Any condition not covered above that needs attention in order for the bioretention area to function as designed.			Meets the design specifications.

OVERALL TREATMENT AREA TOTALS

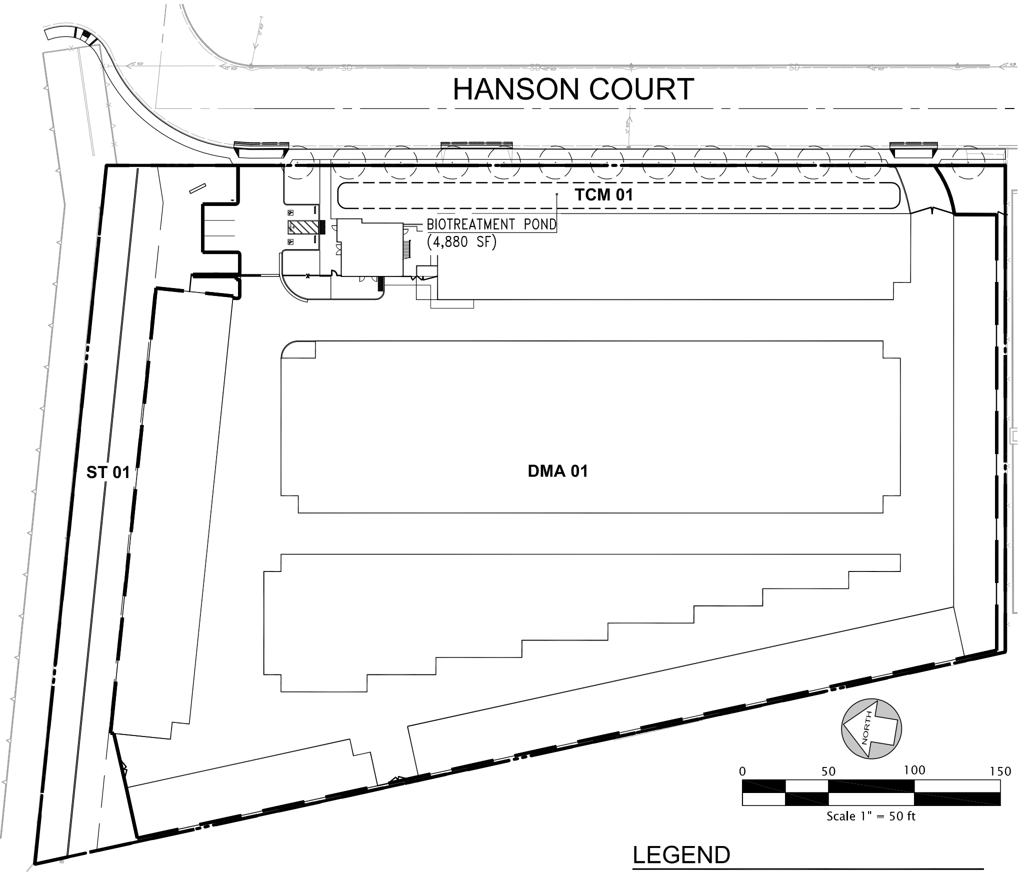
PERVIOUS AND IMPERVIOUS SURFACES COMPARISON TABLE			
a. PROJECT PHASE NUMBER: ROOF AREA(S)	N/A	b. TOTAL SITE (ACRES):	4.27
c. TOTAL SITE EXISTING IMPERVIOUS SURFACES (S.F.)	0	d. TOTAL AREA OF SITE DISTURBED (ACRES):	4.27

e. IMPERVIOUS SURFACES	EXISTING CONDITION (S.F.)	REPLACED (S.F.)	NEW (S.F.)
ROOF AREA(S)	14,590	14,590	86,308
PARKING	0	0	51,444
S/W, PATIOS, PATHS ETC.	171,286	1,468	0
STREETS (PUBLIC)	0	0	0
STREETS/E.V.A (PRIVATE)	0	0	0
TOTAL IMPERVIOUS SURFACES	0	16,058	137,752
f. PERVIOUS SURFACES			
LANDSCAPING	0	0	32,066
PERVIOUS PAVING	0	0	0
OTHER PERVIOUS SURFACES	0	0	0
TOTAL PERVIOUS SURFACES	0	0	32,066

g) TOTAL PROPOSED REPLACED + NEW IMPERVIOUS SURFACES:	153,810
h) TOTAL PROPOSED REPLACED + NEW PERVIOUS SURFACES:	32,066
i) % OF REPLACEMENT OF IMPERVIOUS AREA IN REDEVELOPMENT PROJECTS:	0%

BIOTREATMENT MAINTENANCE

INSPECTION ACTIVITIES	SUGGESTED FREQUENCY
• INSPECT AFTER SEEDING AND AFTER FIRST MAJOR STORMS FOR ANY DAMAGES.	POST-CONTRUCTION
• INSPECT FOR SIGNS OF EROSION, DAMAGE TO VEGETATION, CHANNELIZATION OF FLOW, DEBRIS AND LITTER, AND AREAS OF SEDIMENT ACCUMULATION. PERFORM INSPECTIONS AT THE BEGINNING AND END OF THE WET SEASON. ADDITIONAL INSPECTIONS AFTER PERIODS OF HEAVY RUNOFF ARE DESIRABLE.	SEMI-ANNUAL
• INSPECT GRASS ALONG SIDE SLOPES FOR EROSION AND FORMATION OF RILLS OR GULLIES, AND SAND/SOIL BED FOR EROSION PROBLEMS.	ANNUAL
MAINTENANCE ACTIVITIES	**SUGGESTED FREQUENCY**
• MOW GRASS TO MAINTAIN A HEIGHT OF 3-4 INCHES, FOR SAFETY, AESTHETIC, OR OTHER PURPOSES. LITTER SHOULD ALWAYS BE REMOVED PRIOR TO MOWING. CLIPPINGS SHOULD BE COMPOSTED. • IRRIGATE DURING DRY SEASON (APRIL THROUGH OCTOBER) OR WHEN NECESSARY TO MAINTAIN THE VEGETATION. • PROVIDE WEED CONTROL, IF NECESSARY TO CONTROL INVASIVE SPECIES.	AS NEEDED (FREQUENT, SEASONALLY)
• REMOVE LITTER, BRANCHES, ROCKS BLOCKAGES AND OTHER DEBRIS AND DISPOSE OF PROPERLY. • REPAIR ANY DAMAGED AREAS IDENTIFIED DURING INSPECTIONS. EROSION RILLS OR GULLIES SHOULD BE CORRECTED AS NEEDED. BARE AREAS SHOULD BE REPLANTED AS NECESSARY.	SEMI-ANNUAL
• CORRECT EROSION PROBLEMS IN THE SAND/SOIL BED. • PLANT AN ALTERNATIVE GRASS SPECIES IF THE ORIGINAL GRASS COVER HAS NOT BEEN SUCCESSFULLY ESTABLISHED. RESEED AND APPLY MULCH TO DAMAGED AREAS.	ANNUAL (AS NEEDED)
• REMOVE ALL ACCUMULATED SEDIMENT THAT MAY OBSTRUCT THE PROPER OPERATION OF THE BIO TREATMENT POND. SEDIMENT SHOULD BE REMOVED WHEN IT BUILDS UP TO 3 IN. AT ANY SPOT, OR COVERS VEGETATION, OR ONCE IT HAS ACCUMULATED TO 10% OF THE ORIGINAL DESIGN VOLUME. REPLACE THE GRASS AREAS DAMAGED IN THE PROCESS. • ROTOTILL OR CULTIVATE THE SURFACE OF THE SAND/SOIL BED OF IF THE TREATMENT AREA DOES NOT DRAW DOWN WITHIN 48 HOURS.	AS NEEDED (INFREQUENT)



LEGEND

(dashed box)	BIOTREATMENT POND/PLANTER
(heavy line)	TREATMENT AREA LIMITS
TCM	TREATMENT CONTROL MEASURE
DMA	DRAINAGE MANAGMENT AREA
ST	SELF TREATING

STORMWATER CONTROL NOTES

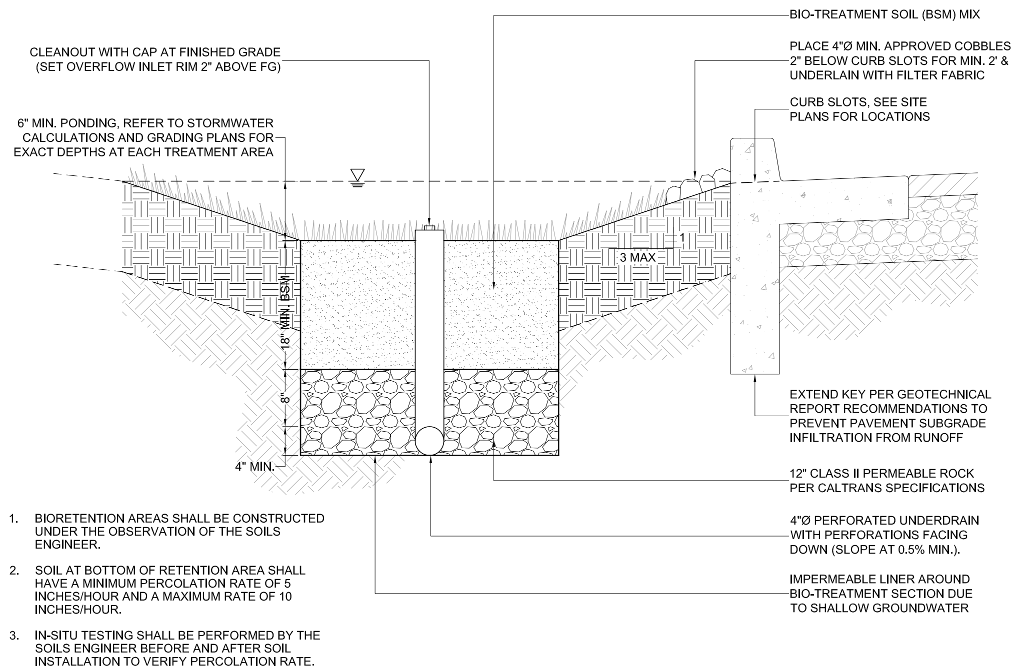
1. THE EXISTING SITE SOILS CONSIST OF CLAY (TYPE D) SOILS.
2. POTENTIAL POLLUTANTS INCLUDE MOTOR VEHICLE LUBRICANTS, COOLANTS, DISC BRAKE DUST, LITTER AND DEBRIS. POLLUTANT SOURCE AREAS INCLUDE THE ASPHALT CONCRETE PARKING LOT AND DRIVE AISLES, THE ROOF OF THE BUILDING, AND THE SITE STORM DRAIN INLETS. ALL INLETS WILL BE MARKED "NO DUMPING - DRAINS TO BAY". THE PARKING LOT SHALL BE SWEPT REGULARLY TO PREVENT THE ACCUMULATION OF LITTER AND DEBRIS.
3. BIOTREATMENT SIZING IS BASED ON THE COMBINATION FLOW/VOLUME BASED METHOD PER SCVURPPP HANDBOOK CHAPTER 5.

SOURCE CONTROL MEASURES IMPLEMENTED

SD-10: SITE DESIGN & LANDSCAPE PLANNING
- MAXIMIZED TREES AND PLANTING WITHIN HARDSCAPE AND LANDSCAPE AREAS.
- VEGETATED SLOPES FOR ALL LANDSCAPE SLOPES LESS THAN 1:5 SLOPE.

SD-11: EFFICIENT IRRIGATION
- RAIN-TRIGGERED SHUTOFF DEVICES TO PREVENT IRRIGATION AFTER PRECIPITATION.
- SYSTEM DESIGNED TO SITE-SPECIFIC WATER DEMANDS AND PLANTING REQUIREMENTS.

SD-13: STORM DRAIN SIGNAGE
- ALL CATCH BASINS TO BE STENCILED WITH PROHIBITIVE LANGUAGE PER CITY STANDARDS.



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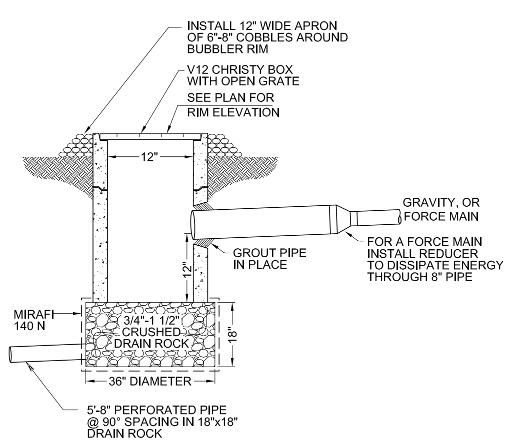

SD BUBBLER DETAIL

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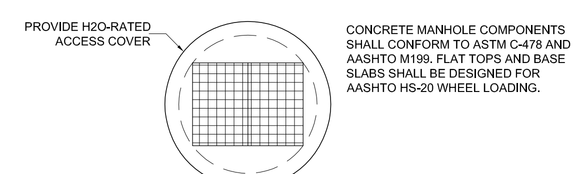
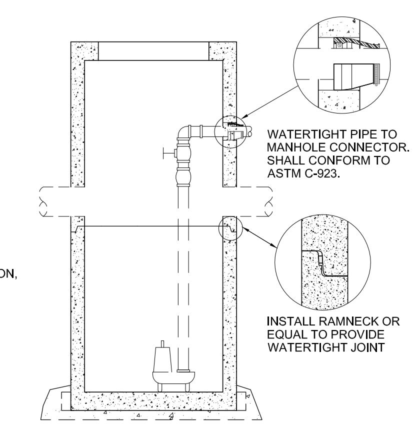
STORM DRAIN PUMP NOTES





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Revisions	Date

REGISTERED PROFESSIONAL ENGINEER
NEKTARIOS MATHEOU
71236
CIVIL
STATE OF CALIFORNIA

KIER & WRIGHT
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3350 Scott Boulevard, Building 22
Santa Clara, California 95054
(408) 727 6665
fax (408) 727 5641

SELF STORAGE
1 HANSON COURT
MILPITAS, CALIFORNIA

STORMATER CONTROL PLAN

Drawn By	STAFF
Date	06/01/15
Scale	AS SHOWN
File Name	
Planning File Numbers	
Sheet Number	C6.0

APPENDIX J

THIRD PARTY CERTIFICATION

APPENDIX J THIRD PARTY CERTIFICATION

A. THIRD PARTY ENGINEER'S CERTIFICATIONS

I hereby certify that the sizing, selection, and preliminary design of the treatment Best Management Practices and control measures in this Storm Water Management Plan meet the requirements of Sunnyvale Municipal Code, Chapter 12.60.150 (Numeric Sizing Criteria for Treatment Systems) and the NPDES Permit issued by the Regional Water Quality Control Board.

____

Caitlin J. Gilmore, P.E.
SCHAAF & WHEELER,
R.C.E. NO. 76810
EXPIRES 12-31-16

B. ENGINEER'S CERTIFICATIONS

I hereby certify that the sizing, selection, and preliminary design of the Best Management Practices and control measures in this Stormwater Management Plan meet the requirements of SMC12.60.150.

____

Netarios Matheou, P.E.
Kier & Wright Civil Engineers
R.C.E. NO. 71236
EXPIRES 06-30-17

C. OWNER'S CERTIFICATIONS

I hereby certify that the onsite, joint, or offsite stormwater treatment systems and HM controls installed to meet the requirements for regulated projects are properly operated and maintained for the life of the project pursuant to SMC Section 12.60.200 agreement to maintain best management practices.

____

Bertrand Irissou, Manager
One Hanson LLC
1484 Prince Edward Way,
Sunnyvale, CA 94087