ADDENDUM No. 2

DATE: September 7, 2021

TO: Prospective Proposers

FROM: Monterey Regional Airport (MRY)

SUBJECT: **Monterey Regional Airport**
Request for Qualifications/Proposals
Aircraft Rescue and Fire Fighting Facility

Statement of Qualification for the above referenced project must be received at the Monterey Regional Airport Planning Department, 200 Fred Kane Drive, Suite 200 Monterey, California 93940, by **3:00 PM on September 14th, 2021**. **It is the Proposer's responsibility to ensure that said SOQ is received by the Monterey Regional Airport Planning Department prior to the aforementioned designated date and time.**

The following additions and/or deletions are hereby made to and shall be considered as part of the RFQ/P:

I. NOTICE TO DESIGN-BUILD ENTITY:

A. Proposers' attention is directed to first paragraph of subsection "1.2 Contract Requirements" of the "Notice to Design-Build Entity" on page 6 to 49 of the RFQ/P. **Delete** the sentence that reads: "The Contractor shall be required to possess a California Contractor's License, Classification A.", and **replace** with "The Contractor shall be required to possess a California Contractor's License, Classification A or Classification B.".

II. THE FOLLOWING ARE RESPONSES TO WRITTEN QUESTIONS RECEIVED:

1. ***Question/Comment:*** I'm trying to get my hands on the actual RFQ/P documents for the subject project, but I can not get ahold of anyone at ARC. Is there an alternate source for these documents?

   Response: *The RFQ/P is only available through ARC Document Solutions, 2 Harris Ct., Suite 5, Monterey, California, (831) 646-1170.*

2. ***Question/Comment:*** Can you please specify the documentation that responders should submit for Section 9.2.3 Minimum Qualifications (i.e. bonding letter, insurance letter, etc.)?

   Response: *The SOQ minimum qualification documents should include: 1) evidence the Proposer can meet the bonding requirements as identified in the RFQ/P, 2) evidence the Proposer can meet the insurance requirements as identified in the RFQ/P. Evidence can be: 1) in the form of a letter from a surety possessing a valid certificate of authority issued by the California Department of Insurance that the Proposer has the ability to provide the bonding required; and 2) in the form of a letter from an insurance company rated "A" or better that the Proposer has the ability to provide the insurance required, or if the Proposer is self-insured provide evidence the Proposer has a certificate of authority to self-insure from the California Department of Industrial Relations.*

3. ***Question/Comment:*** Does the Airport have flexibility to increase the stated budget or reduce the project scope?

   Response: *It is the intent of MRY for the project to adhere to the stated conceptual budget presented in the RFQ/P. MRY is soliciting this project through the Design-Build to take full advantage of this procurement method to meet MRY's needs and the stated project objectives – reference Section 2.2 of the RFQ/P. Proposers are encouraged to fully understand these objectives.*

4. **_Question/Comment:_** Notice to Design Build Entity states that the Contractor must carry a Class A license. It seems that a Class B license is more appropriate for this vertical construction project. Please confirm that Contractors holding a Class B License can participate in this RFQ/P process.

   Response: *Please see Section I of this addendum for revisions.*

5. **_Question/Comment:_** The RFQ/P references an Exhibit C - Minimum Qualifications document. Please provide this document ASAP.

   Response: *Please see response to Section II Question 2 above.*

6. **_Question/Comment:_** Is there a preference/ requirement for drive thougth vs back in apparatus bays?

   Response: *MRY does not have a preference or requirement for drive through verses reverse-in for the new facility.*

7. **_Question/Comment:_** Please provide more specific on the types of vehicles to be housed in the new bays.

   Response: *Please refer to the Attachment No. 3 to this addendum for a list of the existing ARFF equipment at MRY. Please note: MRY equipment staged at the ARFF also includes a command vehicle, and it is MRY's intention to purchase a new ARFF vehicle in compliance with Index B from 14 CFR § 139.317 - Aircraft rescue and firefighting: Equipment and agents.*

8. **_Question/Comment:_** Section 9.2.5(c) indicates the proposer shall provide a listing of all subcontractors to who the Proposer intends to, or anticipates, subcontracting obligations under this solicitation. Additionally this section states this requirement applies to a construction subcontractor of any tier. Is this a requirement for the RFQ response or is this requirement intended for the RFP phase? Please provide clarification on the specific information needed and during which phase proposers are required to provide this information.

   Response: *This section speaks to both the RFQ and RFP steps of the solicitation. For clarification on the SOQ, MRY is encouraging prospective Proposers to include with their SOQ a listing of the design consultants/professionals (Architect/MEP/Civil) that are intended to be employed on the Design-Build Entities Team during the design phase of the project, and those Subcontractors employed under the contractor's pre-construction phase services only.*

9. **_Question/Comment:_** In section 10.3 of the sample contract there are references to the Surety being licensed to do business in Ohio. We assume that the Surety should be licensed to do business in California, not Ohio. Please confirm requirements.

   Response: *Acknowledged, this is a typo – it should read California and not Ohio.*

10. **_Question/Comment:_** Reference page 1 of 49 "Requests for qualifications / proposals" schedule for submitting SOQ of September 9, 2021.

    - Being that the pre-submittal conference was on the 2nd, the questions are due the following day on the Friday 9/3, and the responsive questions are not scheduled to be back in our hands on Tuesday the 7th, having time to understand and acknowledge your answers to these questions in our RFQ by the 9th is not reasonable or responsible.
    - Your RFQ for "Step One" is as would be expected quite detailed and involved and you would expect the response from the potential contractors to be equally detailed and involved doing that in a professional respectful manner in two days is near impossible

***Response:** In response MRY issued Addendum No. 1 on September 3rd which revised the SOQ submittal date to 3:00 PM on September 14th, 2021.*

11. ***Question/Comment:*** Reference page 6 section 1.2 “Contractor Requirements” first paragraph of the Notice to Design - Build Entity dated August 2021”.
    - Noted paragraph requires the contractor to possess a Class A license.
    - Would it be acceptable for the contractor to possess a class b license versus the noted class A license?

    ***Response:** Refer to Section I of this addendum for modifications.*

12. ***Question/Comment:*** Reference page 25 section 4.7 “Project Budget and Funding Limitations” first paragraph notes anticipate budget for the art facility is approximately 6.6 million.
    - Notes anticipate budget for the art facility is approximately 6.6 million.
    - We understand that this is just an approximate budget, but if it was generated back sometime earlier this year it may be low by more than 30% due to the huge cost increases of raw materials and the enormous delays and obtaining those materials. We would like to suggest the Monterey Regional Airport Planning Department look into their budget now so that when your DB contractors gets into step #2 you won't be shocked at the budgets and schedules you receive.

    ***Response:** Noted – thank you for your comment.*

13. ***Question/Comment:*** Reference page 27 section 5.5 “Elevation Process” Paragraph labeled “Step One”.
    - Step one appears to be our response to the SOQ and will be evaluated as outlined in table #3 on pages 28 - 31.
    - Our question is just confirming that our SOQ package due on the 9thj suit does not need to be included any of the information in an outline paragraph 5.5.2 label design criteria package evaluation and in table 4 on pages 32 and 33.

    ***Response:** Section 5.5.2, Section 5.5.3 and Table 4 are applicable to Step 2 of the solicitation.*

14. ***Question/Comment:*** It appears we do not have the opportunity for another RFI period so we would like to ask the following questions in anticipation for the following RFQ/P Steps: Please provide all civil and site utility related plans for the ongoing work at the project site to coordinate utility locations and tie ins. (work currently being performed by Teichert).

    ***Response:** MRY has made available plans from the North General Aviation Apron project as an appendix to the original solicitation document. No further plans are available at this time.*

15. ***Question/Comment:*** Please provide all PG&E Drawings.

    ***Response:** No PG&E drawings are available.*

16. ***Question/Comment:*** Please provide hazmat reports for the existing fire facility.

    ***Response:** No hazmat reports are available at this time.*

17. ***Question/Comment:*** Please provide any soils reports related to the new project site.

    ***Response:** Prior geotechnical reports are included in Attachment No. 2 of this addendum.*

IV. THE FOLLOWING ARE FOLLOW-UP RESPONSES TO QUESTIONS RECEIVED AT THE PRE_BID CONFERENCE:

1. ***Question/Comment:*** Will there be information provided after Step 1?

   Response: *For Step 2 of this RFQ/P, MRY is not intending to provide additional project information over the information already made available to prospective Proposers. The Sample contract revision is an example of the anticipated additional information to be provided.*

2. ***Question/Comment:*** What is the timeline for the selection and project?

   Response: *Proposers should refer to the information on procurement and project schedule provided in the RFQ/P (reference Sections 1.12 and 1.13).*

3. ***Question/Comment:*** Are there geotechnical reports available?

   Response: *The preliminary geotechnical report and the geotechnical report carried out for the North Apron project are included for Proposers' information as part of the Attachment No. 2 to this addendum.*

4. ***Question/Comment:*** Solicitation says Class A license, is Class B acceptable?

   Response: *Refer to Section I of this addendum.*

5. ***Question/Comment:*** Has PG&E Planning meeting been initiated?

   Response: *Coordination with PG&E has been ongoing as part of the North GA Apron project currently under construction. The transformer and switchgear for the ARFF are being installed as part of the current apron project. A conduit stub from the switchgear to the ARFF building pad is also installed for power hook-up.*

6. ***Question/Comment:*** Has the facility been designated as a Critical Operations Facility?

   Response: *The proposed facility will not be a Critical Operations Facility.*

7. ***Question/Comment:*** Are telecom and natural gas stubs included?

   Response: *There is no natural gas for the proposed ARFF. Proposers should refer to the North General Apron Plan set provided as part of the RFQ/P. The utility stubs provided includes: 2-inch communication, 8-inch sewer line, and 6-inch water main as the main to service both fire and domestic.*

8. ***Question/Comment:*** Do the wet utilities have capacity for the proposal ARFF facility?

   Response: *The wet utility stubs provided were sized for the proposed facility.*

The above additions, changes, and clarifications are hereby made to and shall be considered as part of the RFQ/P.

Pre-Submittal Conference Sign-In Sheet: Please see ATTACHMENT No. 1 for a copy of the sign-in sheet from the Pre-Submittal conference held on September 2, 2021.

NOTICE: The Proposer shall SIGN and attach this addendum to his/her SOQ. No SOQ will be considered unless this page of Addendum No. 2 is signed and attached thereto.

____

PROPOSER'S SIGNATURE DATE

Attachment No. 1

Addendum No. 2 – Pre-Submittal Sign-In Sheet

MONTEREY REGIONAL AIRPORT

REQUEST FOR QUALIFICATIONS/PROPOSALS
Design-Build Team
MONTEREY REGIONAL AIRPORT
Aircraft Rescue and Fire Fighting Facility
Pre-Submittal Conference September 2, 2021 Sign-In Sheet

	Name	Organization/Firm	Phone	E-mail
1.	Howard Paige	Avora Engineers	408-656-5026	hpaige@avoraengineers.co
2.	Kathryn Avila	Avila Construction	831-540-7858	Kathryn@avilaconst.com
3.	Mike Avila	" "	831-594-1288	mike@avilaconst.com
4.	Ernie Mill	MILL CONSTRUCTION	831-809-1450	EMILL@MILLCONSTRUCTION.COM
5.	Penn Shortes	GRANITE Construction	831-763-5505	Penn.shortes@GCinc.com
6.	Lance McIntosh	C&S Companies	602-769-0563	Lmcintosh@cscos.com
7.	Ryan Aeschliman	Swinerton	408 605 6415	raeschlima@swinerton.com
8.	ISAAC KONTOROVSKY	BKF ENGINEERS	650.690.2955	ik@BKF.COM
9.	John Mares	Corgan	310.873.3603	john.mares@corgan.com
10.	Karen Mar	MarJang Architecture	415-516-3454	karen@marjang.com
11.	BOB HAMILTON	KIMLEY-HORN	669-800-1577	BOB.HAMILTON@KIMLEY-HORN.com
12.	Raymond Duenas	List Engineering	562-533-6384	raymond@listengineering.com
13.	ERIC BROWARD	BROWARD BUILDERS/LEA ARCH	530-666-5635	ESTIMATING@BROWARDBUILDERS.COM
14.	Cheryl Johnson	Swinerton	4157931559	cjohnson@swinerton.com
15.	Christopher Stark	JM Electric	(831) 422-7819	cps@jmelectric.com
16.	Jason Hargreaves	Tartaglia Engineering	805 466-5660	Jason@tartaglia-engineering.com

REQUEST FOR QUALIFICATIONS/PROPOSALS

Design-Build Team

MONTEREY REGIONAL AIRPORT

Aircraft Rescue and Fire Fighting Facility

Pre-Submittal Conference September 2, 2021 Sign-In Sheet

	Name	Organization/Firm	Phone	E-mail
17.	GAUDENZ PANHOLZER	MONTEREY FIRE	831-646-3900	panholzer@monterey.org
18.	Dan McAweeney	DMC COMMERCIAL INC	831-241-8223	dan@dmcmp.com
19.	Candice Wong	Ten Over Studio, Inc	805.541.1010	candicew@tenoverstudio.com
20.	Chris Morello	MRY		
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Attachment No. 2

Addendum No. 2 – Geotechnical Reports

GEOTECHNICAL ENGINEERING REPORT
MONTEREY REGIONAL AIRPORT
NORTHSIDE GENERAL AVIATION
APRON CONSTRUCTION
MONTEREY, CALIFORNIA

June 16, 2020

Prepared for

Mr. John Smith, PE
Tartaglia Engineering
Project No. 20-13

Prepared by

Earth Systems Pacific
4378 Old Santa Fe Road
San Luis Obispo, CA 93401

Earth Systems
4378 Old Santa Fe Road | San Luis Obispo, CA 93401 | Ph: 805.544.3276 | www.earthsystems.com

June 16, 2020

FILE NO.: 303018-003

Mr. John Smith, PE
Tartaglia Engineering
PO Box 1930
Atascadero, CA 93423

PROJECT: MONTEREY REGIONAL AIRPORT
NORTHSIDE GENERAL AVIATION APRON CONSTRUCTION
200 FRED KANE DRIVE
MONTEREY, CALIFORNIA
TARTAGLIA ENGINEERING PROJECT NO. 20-13

SUBJECT: Geotechnical Engineering Report

CONTRACT REF: Proposal to Provide a Geotechnical Engineering Investigation and Recommendations – Monterey Regional Airport – North Side Apron Project, Monterey, California, by Earth Systems Pacific, Doc. No. 2002-039.PRP.REV, revised February 28, 2020

Dear Mr. Smith:

As per your authorization of the referenced proposal, this geotechnical engineering report has been prepared for use in the design of the Northside General Aviation Apron Construction project at Monterey Regional Airport in Monterey, California. Conclusions regarding CBR testing, subsurface water and soil moisture contents, soil erodibility, and seismic ground motions, as well as recommendations for site preparation, grading, utility trenches, retaining walls, drainage around improvements, and observation and testing, are provided. Two paper copies and a digital copy of this report are furnished for your use.

We appreciate the opportunity to have provided geotechnical services for this project and look forward to working with you again in the future. If there are any questions concerning this report, please do not hesitate to contact the undersigned.

Sincerely,

Earth Systems Pacific

Ryne Mettler
Staff Engineer

Fred J. Potthast, GE
Principal Engineer

FREDERICK J. POTTHAST
No. GE 2368
Exp. 03/31/22

Doc. No.: 2006-003.SER/cr

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APPENDICES

1.0 INTRODUCTION AND SITE SETTING

This geotechnical engineering report has been completed for the client's use in the development of plans and specifications for the Northside General Aviation Apron Construction project at Monterey Regional Airport in Monterey, California. The project location is indicated on Figure 1 – Site Vicinity Map in Appendix A. The project will consist of the construction of an apron on the north side of Runway 10L-28R, to allow the relocation of tenants from the southeast side of the airport, prior to the planned Terminal Relocation Project. The project will include clearing, grubbing and removal of surface vegetation, excavation and embankment with cuts and fills up to 7 feet from the existing topography, apron and taxilane pavement consisting of hot mix asphalt (HMA) or Portland cement concrete (PCC) over aggregate base (AB), underground utilities, stormwater detention facilities, apron lighting, and one taxilane connector to Runway 10L-28R. It is our understanding that the fill material will come from the designated borrow area directly west of the proposed apron. Retaining walls a maximum of 6 feet tall may also be constructed. Future improvements (hangars, a fuel farm, and an ARFF facility) are not in the scope of this project. No structures, light poles, drainage basins or other improvements are planned.

The project area slopes to the north-northwest at less than 5 percent. In the southeast corner of the site is the existing apron and connector taxiway to the Runway 10L-28R parallel taxiway; the apron and taxiway are paved with asphalt concrete (AC). Six T-hangars, two metal buildings and an above-ground fuel tank are on the north side of the existing apron. A smaller T-hangar and a vehicle parking lot with an access driveway to Airport Road are on the north side of the two metal buildings. A thin layer of gravel and/or surface treatment in poor condition approximately 75 to 90 feet wide extends beyond the west side of the existing apron, and behind the T-hangars on the north side of the existing apron. The remainder of the project area is covered by light to moderate grasses and weeds, and scattered low brush.

2.0 SCOPE OF SERVICES

The scope of work for this geotechnical engineering report included a general site reconnaissance, subsurface exploration, infiltration testing, laboratory testing of soil samples, review of a preliminary geotechnical investigation report provided by Cornerstone Earth Group (CEG 2017) for various improvements at the airport, engineering evaluation of the data collected, and the preparation of this report. The investigation and subsequent recommendations were based on information and base maps provided by the client.

The report and recommendations are intended to be in general accordance with AC 150/5320-6F (FAA 2016), the client's requested work scope, and common geotechnical engineering practice in this area under similar conditions at this time. The tests were performed in general conformance with the standards noted, as modified by common geotechnical practice in this area under similar conditions at this time.

It is our intent that this report be used exclusively by the client to form the geotechnical basis of the design of the project described herein. Application beyond this intent is strictly at the user's risk. As there may be geotechnical issues yet to be resolved, the geotechnical engineer should be retained to provide consultation as the project progresses, to assist in verifying that pertinent geotechnical issues have been addressed and to aid in conformance with the intent of this report. In the event this report is used to develop project plans, it may also be advantageous to retain the geotechnical engineer to review the project plans as they near completion to further aid in conformance of the plans with the intent of this report.

This report does not address issues in the domain of the contractor such as, but not limited to, site safety, excavatability, shoring, temporary slope angles, construction methods, etc. Analysis of site geology and of the soil for corrosive potential, radioisotopes, asbestos (either naturally occurring or in man-made products), lead or mold potential, hydrocarbons, or other chemical properties are beyond the scope of this investigation. Ancillary features beyond the pavement areas covered by this report are also not within our scope and are not addressed.

In the event that there are any changes in the nature of the work scope, or if any assumptions used in the preparation of this report prove to be incorrect, the conclusions and recommendations contained in this report shall not be considered valid unless the changes are reviewed and the conclusions of this report modified or verified in writing.

3.0 FIELD INVESTIGATION

On March 23, 2020, seven borings were drilled in the project area with a Mobile Drill rig, Model B-53, equipped with 6-inch outside diameter hollow stem auger and an automatic hammer for sampling. Four of the borings were drilled to a maximum of 10 feet below the existing ground surface for geotechnical logging and sampling. The remaining three borings were used for LID infiltration testing, with depths ranging from 2 to 8 feet bgs. The approximate locations of the borings are shown on Figure 2 - Exploration Location Map in Appendix A.

As the geotechnical log boring in the apron area were drilled, soil samples were obtained using a 3-inch outside diameter ring-lined barrel sampler (ASTM D 3550-17 with shoe similar to D 2937-17) at the estimated subgrade elevation (approximately 1 foot bgs). Standard penetration tests

(SPT) using a 2-inch outside diameter split-spoon sampler were also performed in the borings (ASTM D 1586-11) from 5 to 6.5 feet bgs and from 8.5 to 10.0 feet bgs in these boring. Bulk samples were secured from the auger cuttings.

The soils were initially classified and logged in general accordance with the Unified Soils Classification System (ASTM D 2488-17). Final classifications of the soils in accordance with the Unified Soils Classification System (ASTM D 2487-17) were made following completion of laboratory testing. Copies of the boring logs and a boring log legend can also be found in Appendix A. In reviewing the boring logs and legend, the reader should recognize that the legend is intended as a guideline only, and there are a number of conditions that may influence the soil characteristics as observed during drilling. These include, but are not limited to, cementation, variations in soil moisture, presence of groundwater, and other factors. Consequently, the logger must exercise judgment in interpreting soil characteristics, possibly resulting in soils descriptions that vary somewhat from the legend. Following completion of drilling, the borings were backfilled with cement-treated auger spoils and gravel.

4.0 LABORATORY INVESTIGATION

In situ moisture content and unit dry weight (ASTM D 2937-17, as modified for ring liners) were determined for the ring samples. Moisture content only (ASTM D 2216-10) were determined on four of the SPT samples. Two bulk samples were tested for the following: maximum density and optimum moisture (ASTM D 1557-12, modified), particle size distribution (ASTM D 422-63/07; D 1140-17), plasticity index (ASTM D 4318-17), and CBR (ASTM D 1883-16, for a range of moisture contents, with ASTM D 1557-12 as the reference standard for maximum density). One bulk sample was tested for angle of shearing resistance and cohesion by direct shear (ASTM D3080-11, compacted to 90 percent of maximum dry density). Please refer to Appendix B for the laboratory test results.

5.0 GENERAL SUBSURFACE PROFILE

In all the borings, Dune Sand comprised of poorly graded sand, poorly graded sand with silt, and silty sand was found for the full depths of the borings. The soils were described during drilling as being loose in the upper 6 inches, and medium dense to dense with depth. In situ moisture contents varied from slightly moist to very moist. Subsurface water was not encountered in any of the borings, to the maximum depth explored of 10 feet bgs.

Please refer to the logs in Appendix A for a more complete description of the subsurface conditions found in the borings.

6.0 LID INFILTRATION TESTING

The three borings drilled for infiltration testing extended to depths of 2 to 8 feet bgs. After drilling was completed, a 2-inch diameter perforated pipe was installed in each of the test borings and the annular spaces around the pipes were filled with gravel. Infiltration testing was performed in general accordance with the methods developed by this firm in cooperation with the Central Coast Low Impact Development Initiative (ESP 2013).

Initially, testing consisted of introducing water into each of the test borings to just below existing grade. This water level was then maintained at constant head for 30 minutes. After the 30-minute period, the water was shut off and the amount of water introduced into each of the test borings was recorded. Readings of the change in water level were then recorded at various time intervals over a period of approximately 4 hours. Following testing, the pipes were removed and the test borings were backfilled with on-site soil. The LID infiltration test results are included in Appendix C.

The test results only indicate the infiltration rate at the specific locations tested and under specific conditions. Sound engineering judgment should be exercised in extrapolating the test results for other conditions or locations. Technical design references vary in methods they present for using these types of test results. However, most references include reduction, safety, and/or correction factors for several parameters including, but not limited to, size of the LID system relative to the test volume, number of tests conducted, variability in the soil profile, anticipated silt loading, anticipated biological buildup, anticipated long-term maintenance, and other factors. Typically, in aggregate these factors range from about 2.5 to 50 depending upon the method used. The final determination of the means by which these data are used is left to the design engineer.

7.0 CONCLUSIONS

CBR Test Results

Per FAA AC 150/5320-6F, the degree of relative compaction required at subgrade for new pavement areas (and therefore the CBR value that can be used in the construction design) is based on the cohesive/non-cohesive classification of the subgrade soils. The poorly graded sands with silt and clay found in all of the borings are considered to be non-cohesive (plasticity index less than 3), therefore subgrade compaction in pavement areas is anticipated to be a minimum of 100 percent of maximum dry density.

The laboratory CBR test results indicate variations in the strengths of the soils tested based on their moisture contents. The recommended CBR values provided in this section are based on the assumption that the subgrade soils will be recompacted within a moisture conditioned range extending from 2 percent below optimum moisture content to 2 percent above optimum moisture content. If the subgrade soils are not maintained within this range, a reduction in the CBR value of the soils compacted in place will occur. Assuming the CBR values provided in this report will be utilized for design, the project plans should fully indicate the relatively narrow moisture content range as a specification requirement, to allow the contractor to plan earthwork operations accordingly. Provisions should also be taken (e.g., proper surface drainage and flowlines away from edges of pavement, regular maintenance of the pavement surface to fill any cracks that develop, etc.) to ensure that the moisture contents of the subgrade soils remain within the design range for the design life of the pavement sections.

The poorly graded sand with silt found in Borings 1 and 2 at the anticipated subgrade elevation yielded a minimum CBR value of approximately 36, at 100 percent relative compaction and with moisture contents ranging from 2 percent above to 2 percent below optimum moisture content.

The silty sand found in Borings 3 and 4 at the anticipated subgrade elevation yielded a minimum CBR value of approximately 9, at 100 percent relative compaction and with moisture contents ranging from 2 percent above to 2 percent below optimum moisture content.

Per AC 150/5320-6F (FAA 2016), Chapter 2.5.3, for flexible pavements, the elastic modulus E can be estimated from CBR test results using the following correlation: E (psi) = 1500 x CBR.

Swelling Soils

AC 150/5320-6F (FAA 2016) Chapter 3.10.1 describes the effects that swelling soils have on airport pavements, and recommends various treatments (removal and replacement, stabilization, modified compaction efforts and adequate drainage) to reduce the potential for damage to pavements due to swelling soils. Chapter 3.10.2 (FAA 2016) indicates swelling soils "usually have liquid limits above 40 and plasticity indexes above 25." The poorly graded sand with silt and silty sand found at the anticipated subgrade elevation in all of the borings was non-plastic, therefore no special consideration for swelling soils is considered to be necessary.

Earthwork Shrinkage

Soil volume loss, or "shrinkage", during earthwork can be attributed to three categories; soil loss due to stripping or demolition of existing improvements, subsidence of the underlying soils due to compaction, and shrinkage of fill soil as it is placed and compacted. These factors are partly due to the soil characteristics, but largely due to depths of cuts and fills, stripping techniques, type and weight of earthwork equipment, traffic pattern of earthwork equipment, and soil moisture at the time of grading.

In paved areas that are to be reconstructed, if any, removal of distinct AC and AB layers can result in less loss than from removal of vegetation in unpaved areas. The amount of soil loss that will occur is largely dependent upon how careful the contractor is in stripping and demolition/removal operations.

Subsidence of the site due to compaction of the soils below a fill area also occurs. Subsidence due to compaction is likely to be in the range of 0.1 to 0.2 feet. The main zone of subsidence is typically the upper two to three feet. Deeper subsidence is not expected as earthwork operations for pavement reconstruction are expected to be limited to the upper 2 to 3 feet below subgrade in the project area.

To estimate shrinkage of the subgrade, *in situ* soil density data from ring samples taken in the borings at approximate subgrade elevation were analyzed. Appendix D contains a summary of the existing relative compaction at each depth where a ring sample was secured, as well as calculated shrinkage assuming final relative compaction values ranging from 90 to 100 percent.

As loss, subsidence, and shrinkage are only partly due to the soil characteristics, and are largely influenced by the earthwork equipment, earthwork methods, and soil moisture, these factors cannot be precisely estimated.

Subsurface Water and Soil Moisture Contents

Subsurface water was not encountered in any of the borings to the maximum depth drilled of 10 feet below the existing pavement surface. However, if soil moisture contents are well above optimum during earthwork operations, the soils could become unstable under equipment traffic. Unstable conditions hinder compaction efforts and are not acceptable to support fill or pavement section placement. All grading areas should be firm and unyielding following compaction operations and prior to placement of fill, aggregate base or pavement.

Depending on the time of year that construction operations take place, the most effective methods to deal with unstable conditions due to high soil moisture could be scarification and aeration, or the use of geotextile stabilization fabrics. Scarification and aeration may only be possible if the weather conditions are clear and if the project schedule permits.

If the project schedule will not allow drying of the soil naturally, stabilization fabric could be typically utilized. Additional excavation below subgrade or below the grading plane in the erosion control area may also be needed before the stabilization fabric is placed; the depth of overexcavation should be determined by the geotechnical engineer based on conditions exposed at the time of construction. After all excavations are complete, and prior to placement of the geotextiles, the exposed surfaces are typically back-dragged to a smooth condition to the degree practicable with light earthwork equipment. Geotextile stabilization fabric (Mirafi RS380i, RS580i or similar material) is typically placed in the excavated area and extended up the sidewalls of the excavation to within 2 inches of the bottom of the AC layer in pavement areas, and at least 3 feet in general fill areas. Stabilization fabrics are rolled out along the long dimension of the reconstruction area (not perpendicular to it), and are stretched, overlapped and held in place according to the manufacturer's recommendations. In pavement areas, recycled subbase and/or imported aggregate base, per the overall pavement section design, is placed over the fabric in thin, moisture-conditioned lifts and compacted. Recycled subbase and/or aggregate base is placed by end-dumping on the fabric and spreading ahead of equipment; equipment traffic is typically not allowed to travel directly over the fabric. Initial lifts of subbase/base are spread and compacted by rubber-tired equipment; subsequent lifts are compacted using sheepsfoot and/or steel-drum equipment. Compaction equipment is usually operated in static mode only until base grade is reached, to reduce the potential for any free water in the underlying soils to be drawn through the fabric and into the subbase or aggregate base. In general fill areas, available on-site or imported materials can be used as fill over the stabilization fabric, provided the material is within the recommended moisture content range.

If it appears that stable conditions will not be created at base grade in paving areas after the use of geotextiles, a layer of geogrid (Tensar TriAx TX-7 or similar material) can be placed according to the manufacturer's recommendations as additional reinforcement at the approximate mid-depth of the subbase/aggregate base layer. Often sufficient material may not be in place over the geotextile stabilization fabric at mid-depth of the design subbase/aggregate base layer to fully mobilize its strength characteristics and to determine if geogrid will be needed, therefore it may be necessary to construct a full-scale test strip of the pavement section, with and without geogrid reinforcement. This test strip will give an indication as to whether or not geogrids will be required in any reconstruction areas. Geogrids or additional layers of stabilization fabric may also be used if unstable conditions persist in general fill areas.

Soil Erodibility

The site soils are considered to be highly erodible. It is essential that all surface drainage be controlled and directed to appropriate discharge points, and that surface soils, particularly those disturbed during construction, are stabilized by vegetation or means during and following construction.

Ground Motion Analyses

In accordance with the California Building Code (CBC) (CBSC 2019) and ASCE 7-16 (2017, 2018), an assessment was made to determine the need for employing "Site-Specific Ground Motion Procedures" to calculate the ground motion parameters for the project. The S_1 ground motion value obtained from the Structural Engineers Association of California website (SEAOC 2020) using the ASCE 7-16 (2017), for Site Class "D – Stiff Soil," was 0.492g, greater than 0.2; therefore, per Section 11.4.8 of ASCE 7-16 (2017), the project requires site-specific ground motion analyses unless certain exceptions that relate to the building's period are met. Because we have no information on the fundamental period of any structures that may be constructed on this project, we performed a site-specific ground motion analysis to develop the seismic design parameters in accordance with ASCE 7-16 (2017) Sections 11.4.8 and 21.2.

A risk-targeted maximum considered earthquake (MCE_R) modeling procedure was performed in accordance with ASCE 7-16 (2017), including a Probabilistic Seismic Hazard Analysis (PSHA) and a Deterministic Seismic Hazard Analysis (DSHA). These analyses are based on knowledge of the regional tectonic setting, geology, and seismicity. A PSHA using the United States Geologic Survey (USGS 2020) Unified Hazard Tool was used to access the Third California Earthquake Rupture Forecast (UCERF3) database (USGS 2913) and NGA-West2 ground motion prediction equations (PEER 2015) as described in ASCE 7-16 Section 21.2.1.1 (Method 1) to estimate the peak ground motion corresponding to the uniform hazards earthquake and MCE_R which has a 2% probability of being exceeded in 50 years.

For analysis purposes we assumed the Monterey Bay-Tularcitos fault, located 1.2 miles from the site, as the causative fault. The Monterey Bay-Tularcitos fault is a right-lateral fault and we specified an 90-degree dip and oblique slip conditions for the attenuation relationships. A magnitude of 7.3 was used as a basis to model fault rupture. The primary seismic risks are from earthquakes generated by the local Monterey Bay-Tularcitos, Rinconada, and San Gregorio faults and the more distant San Andreas fault. Although listed faults are thought to potentially generate the most severe seismic shaking any local fault could produce seismic shaking at the site.

The 2019 CBC seismic design criteria are based on a Design Earthquake that produces ground motion $^2/_3$ of the lesser of an earthquake with 2 percent probability of occurrence in 50 years, or maximum 84th percentile of the mean deterministic MCE. In this case the probabilistic spectra governed, scaled in accordance with ASCE 7-16 (2017) procedures.

Seismic Design Category

Section 1613.2.5 of the 2019 CBC states that *structures classified as Risk Category I, II or III that are located where the mapped spectral response acceleration parameter at 1-second period, S_1, is greater than or equal to 0.75 shall be assigned to Seismic Design Category E, others shall be assigned to Design Category D.* The mapped S_1 for the site is 0.492, less than 0.75; therefore, the site should be assigned to the seismic design category D.

Seismic Design Parameters

This site may be subject to strong ground shaking due to potential fault movements along regional faults including the Monterey Bay-Tularcitos fault, whose proximity and magnitude potential was considered during our site-specific analysis. Engineered design and earthquake-resistant construction increase safety and allow development of seismically active areas. The minimum seismic design should comply with the 2019 CBC and ASCE 7-16 (2017). The resulting seismic coefficients considering Site Class D are given in the table below.

TABLE 1: SEISMIC PARAMETERS (2019 CBC/ASCE 7-16)

Seismic Design Category	D
Site Class	D
Mapped and Code Based Ground Motion	
Short Period Spectral Response, S_S	1.332 g
1 second mapped Spectral Response, S_1	0.492 g
Design Earthquake Ground Motion	
Short Period Spectral Response, S_{DS}	1.047 g
1 second Spectral Response, S_{D1}	0.963 g
Peak Ground Acceleration (PGA_M)	0.659 g
MCE Spectral Response Acceleration	
Short Period Spectral Response, S_{MS}	1.571 g
1 Second Period Spectral Response, S_{M1}	1.445 g
Site Amplification Factors	
Short Period Site Coefficient, F_a	1.00
1 Second Period Site Coefficient, F_v	2.50
Vertical Site Coefficient, C_v	1.37
Risk Coefficient (Short Period), C_{RS}	0.930
Risk Coefficient (1 Second Period), C_{R1}	0.923

The estimated peak horizontal site acceleration based upon a probabilistic analysis (2 percent probability of occurrence in 50 years) is approximately 1.75 g for this site. Acceleration values provided are estimates only. Actual spectral acceleration values may be more or less than those provided and could exceed 1g assuming a maximum considered earthquake event occurs on a nearby fault. Vertical accelerations are typically ⅓ to ⅔ of the horizontal accelerations but can equal or exceed the horizontal accelerations depending upon the fault type, local site effects and amplification.

8.0 PRELIMINARY GEOTECHNICAL RECOMMENDATIONS

The following preliminary recommendations are for the project as described in the "Introduction" section of this report. If locations, elevations, etc., change, the recommendations contained herein may require modification.

Unless otherwise noted, the following definitions are used in the recommendations presented below. Where terms are not defined, definitions commonly used in the construction industry are intended.

- **Apron Area**: The apron area is defined as the area within the limits of the proposed apron pavement.
- **Retaining Wall Area:** The area within and extending a minimum of 3 feet beyond the foundation footprint of any retaining walls constructed as part of the project.
- **Grading Areas:** All areas beyond the apron and retaining wall areas that will be graded.
- **Scarified:** Plowed or ripped in two orthogonal directions to a depth of not less than 8 inches.
- **Moisture Conditioned:** Soil moisture content adjusted to the range of optimum plus or minus 2 percent prior to application of compactive effort.
- **Compacted/Recompacted:** Unless otherwise recommended, soils placed in level lifts not exceeding 8 inches in loose thickness and compacted to the minimum percentages of maximum dry density designated by the engineer. The minimum relative compaction should be 90 percent of maximum dry density. Relative compaction should be based on maximum dry density by ASTM D 1557-12 and field density by ASTM D 6938-17a, or other methods acceptable to the engineer, the geotechnical engineer and the jurisdiction.

Site Preparation

1. All areas to be graded should be prepared for construction by removing all vegetation, debris, existing pavement and improvements that are not planned to remain, and other

potentially deleterious material. Any existing utility lines that will not remain in service should be either removed or abandoned. The appropriate method of utility abandonment will depend upon the type and depth of the utility. Recommendations for abandonment during construction can be made as necessary.

2. Voids created by the removal of materials or utilities described above should be called to the attention of the geotechnical engineer. No fill should be placed unless the underlying soil has been observed by the geotechnical engineer.

Grading

1. Following site preparation and excavations to grade or prior to placement of fill in the apron areas, the prepared soil surfaces should be scarified, moisture conditioned and recompacted.

2. Following site preparation, retaining wall areas should be excavated to bottom of footing elevation or to 1 foot below existing grade, whichever is deeper. The resulting soil surfaces should be scarified, moisture conditioned and recompacted.

3. Following site preparation and excavations to grade or prior to placement of fill in the balance of the grading areas, the prepared soil surfaces should be scarified, moisture conditioned and recompacted.

4. During earthwork operations, if the soils are overly moist so that they become unstable, or if the minimum recommended compaction cannot be readily achieved due to excess soil moisture, then alternate methods (scarification/aeration, or geotextiles, or gravel punching) may be needed to achieve stability, as noted in the Conclusions Section of this report. Specific recommendations for stabilization should be provided by the geotechnical engineer based on conditions encountered at the time of construction.

5. The engineer should determine the depth of overexcavation required, the overall section thicknesses (HMA and AB), the degrees of compaction and the maximum density standard for the pavement areas, and the requirements for shoulder backing adjacent to pavement sections.

6. Fill material should be placed in level lifts not exceeding 8 inches in loose thickness. On-site soils or acceptable imported soils may be used for fill once they are cleaned of all

debris and deleterious materials. Imported soils to be used as fill should be equal to or better than the site soils with respect to strength characteristics. All proposed imported materials should be reviewed by the soils engineer before being transported to the site.

7. All earthwork areas should be firm and unyielding following compaction operations, prior to placement of fill, and at finish grade.

8. The recommendations of this section are minimums only and may be superseded by the requirements of the engineer or the governing jurisdiction.

Utility Trenches

1. A select, noncorrosive, granular, easily compacted material should be used as bedding and shading immediately around utilities. The site soil may be used for trench backfill above the select material.

2. The project engineer should designate the relative compaction percentage and the maximum dry density standard for trench backfill in all grading areas. Trench backfill in unimproved areas where settlement of the backfill would not be detrimental should be compacted to a minimum of 90 percent of maximum dry density.

3. Trench backfill should be placed in level lifts not exceeding 6 inches in loose thickness and compacted as noted above. Trench backfill should be moisture conditioned to at least optimum moisture content prior to application of compactive effort.

4. The recommendations of this section are minimums only and may be superseded by the requirements of the client, the governing jurisdiction, utility companies or pipe manufacturers.

Retaining Walls

1. Retaining walls footing excavations should be cut neatly into firm soil recompacted per the “Grading” Section of this report, and should extend to a minimum of 18 inches below lowest adjacent grade within 7 feet laterally of the base of the footing. All footing excavations should be horizontal, but they may be stepped to accommodate changes in topography.

2. Retaining wall footings should be reinforced in accordance with the requirements of the architect/engineer; minimum retaining wall footing reinforcement should consist of one No. 4 rebar top and bottom.

3. To reduce the potential for random cracking, joints should be provided in the wall and the footing whenever the footing is stepped to accommodate changes in topography.

4. Retaining wall design may be based on the following parameters:

TABLE 3: RETAINING WALL DESIGN PARAMETERS

Parameter	Backfill Type	Value
Active Equivalent Fluid Pressure	Site Soils or Imported Sand/Gravel	35 pcf
At-Rest Equivalent Fluid Pressure	Site Soils or Imported Sand/Gravel	50 pcf
Passive Equivalent Fluid Pressure	Onsite or Imported Recompacted Soil	300 pcf
Maximum Toe Pressure	Onsite or Imported Recompacted Soil	3,000 psf
Coefficient of Sliding Friction	Onsite or Imported Recompacted Soil	0.40

5. The active and at-rest pressures are applicable to a horizontal retained surface behind the wall. Walls having a retained surface that slopes upward from the wall should be designed for an additional equivalent fluid pressure of 1 pcf for the active case and 1.5 pcf for the at-rest case, for every degree of slope inclination.

6. Retaining walls should be designed to support the adjacent loading from improvements and vehicles, if any will be present. Generally, one-half of the resulting vertical pressures from these items will result in lateral pressures that should be applied to the full height of the wall. If desired, an analysis for lateral wall pressures resulting from these surcharge loads may be performed on a case-by-case basis.

7. The upper foot of backfill behind all retaining walls should consist of native soil, except in areas where flatwork or pavement will abut the top of the wall. In such cases, the backfill soil should extend to the sand, aggregate base, or other material below the improved surface, as appropriate. If gravel backfill is utilized, the gravel should be encased in a permeable synthetic filter fabric conforming to standard specification section 96-1.02B – Class C (Caltrans 2015).

8. Under the CBC (CBSC 2019), the Risk-Targeted Maximum Considered Earthquake (MCE_R) must be used for determining seismic pressures on walls. Further, Section 1807.2.2 of the 2019 CBC requires that dynamic seismic lateral earth pressures be provided by the geotechnical engineer for walls retaining more than 6 feet of backfill. As retaining walls for this project are not anticipated to exceed 6 feet in height, a seismic increment of earth pressure would not be needed.

9. Long-term settlement of properly compacted native soil retaining wall backfill or imported sand/gravel backfill should be assumed to be about 0.5 and 0.25 percent of the depth of the backfill, respectively. Improvements that are constructed near the tops of retaining walls should be designed to accommodate long-term settlement.

10. All retaining walls should be designed as drained walls. The wall drainage design may consist of a conventional system that includes perforated pipe encased in a free-draining gravel blanket or of a manufactured synthetic drainage system that consists of drainage panels that direct water flow into an integrated subdrain.

11. For the conventional system, the pipe should be placed with perforations facing downward, and should discharge in a non-erosive manner away from foundations and other improvements. The gravel blanket should have a width of approximately 1 foot and should extend upward to approximately 1 foot below the top of the wall backfill. The upper foot should be backfilled with native soil. In such cases, the gravel should extend to the sand, aggregate base, or other material below the improved surface, as appropriate. To reduce infiltration of the soil into the gravel, a permeable synthetic filter fabric conforming to Standard Specifications Section 96-1.02B – Class C (Caltrans 2015), should be placed between the two.

12. Manufactured synthetic drains, such as Miradrain or Enkadrain are acceptable alternatives to the use of gravel, provided they are installed in accordance with the recommendations of the manufacturer. The integrated subdrain should discharge in a non-erosive manner away from foundations and other improvements. The drainage panels should extend upward to approximately 1 foot below the top of the wall backfill.

13. Where weep hole drainage can be properly discharged, the perforated pipe or integrated subdrain may be omitted in lieu of weep holes on maximum 4-foot centers. A filter fabric as described above should be placed between the weep holes and the drain gravel when applied to a conventional drainage system.

14. If drainage cannot be provided for retaining walls, this firm should be contacted to provide an additional lateral hydrostatic pressure increment to be used in the wall design.

15. Walls facing areas where moisture transmission through the wall would be undesirable should be *thoroughly* waterproofed in accordance with the specifications of the engineer.

16. The engineer should bear in mind that retaining walls by their nature are flexible structures, and that surface treatments on walls often crack. Where walls are to be plastered or otherwise have a finish applied, the flexibility should be considered in determining the suitability of the surfacing material, spacing of horizontal and vertical control joints, etc. The flexibility should also be considered where a retaining wall will abut or be connected to a rigid structure, and where the geometry of the wall is such that its flexibility will vary along its length.

Drainage Around Improvements

1. Unpaved ground surfaces should be graded during construction, and finish graded to direct surface runoff toward drainage facilities in accordance with the requirements of the FAA. Swales with improved surfaces, area drains, etc. should be utilized where practical to divert drainage away from paved areas.

2. The site soils are highly erodible. Stabilization of soils, particularly those disturbed by construction, vegetation, or other means during and following construction, is recommended to reduce erosion damage. Care should be taken to establish and maintain vegetation.

3. Maintenance of drainage improvements is critical to the long-term life of the project. Site improvements should be inspected and maintained on a regular basis.

4. To reduce the potential for disruption of drainage patterns and undermining of pavement or foundations, all rodent activity should be aggressively controlled.

Observation and Testing

1. It must be recognized that the recommendations contained in this report are based on a limited number of borings and rely on continuity of the subsurface conditions encountered. Therefore, the geotechnical engineer should be retained to provide

consultation during the design phase, to review plans as they near completion, to interpret this report during construction, and to provide construction monitoring in the form of testing and observation.

2. At a minimum, the following should be provided by the geotechnical engineer during construction:

   - Professional observation during grading
   - Oversight of special inspection during grading

3. Special inspection of grading should be provided as per the requirements of the FAA or Section 1705.6 and Table 1705.6 of the CBC; the soils special inspector should be under the direction of the geotechnical engineer. Subject to approval by the building official or other jurisdiction, special inspection requirements should be addressed by the geotechnical engineer during the preconstruction meeting (see below) prior to the start of grading operations.

   At a minimum, the following items should be inspected and/or tested by the special inspector:

   - Stripping and clearing of vegetation and existing pavement (if any) where planned for removal
   - Excavations in grading and retaining wall areas, and corrective operations (scarification/aeration or placement of geotextile stabilization fabric) in any unstable areas
   - Fill, and imported aggregate base quality, placement, moisture conditioning, and compaction
   - Utility trench backfill quality, placement, moisture conditioning, and compaction

4. A program of quality control should be developed prior to beginning grading. The contractor or project manager should determine any additional inspection items required by the architect/engineer or the governing jurisdiction.

5. Locations and frequency of compaction tests should be as per the recommendation of the geotechnical engineer at the time of construction. The recommended test location

and frequency may be subject to modification by the geotechnical engineer, based upon soil and moisture conditions encountered, size and type of equipment used by the contractor, the general trend of the results of compaction tests, or other factors.

6. A preconstruction conference among the owner, the geotechnical engineer, the governing agency, the special inspector, the project inspector, the architect/engineer, and contractors is recommended to discuss planned construction procedures and quality control requirements.

7. The geotechnical engineer should be notified at least 48 hours prior to beginning construction operations. If Earth Systems Pacific is not retained to provide construction observation and testing services, it shall not be responsible for the interpretation of the information by others or any consequences arising therefrom.

9.0 CLOSURE

Our intent was to perform the investigation in a manner consistent with the level of care and skill ordinarily exercised by members of the profession currently practicing in the locality of this project and under similar conditions. No representation, warranty, or guarantee is either expressed or implied. This report is intended for the exclusive use by the client as discussed in the “Scope of Services” section. Application beyond the stated intent is strictly at the user's risk.

This report is valid for conditions as they exist at this time for the type of project described herein. The conclusions and recommendations contained in this report could be rendered invalid, either in whole or in part, due to changes in building codes, FAA regulations, standards of geotechnical or construction practice, changes in physical conditions, or the broadening of knowledge.

If changes with respect to development type or location become necessary, if items not addressed in this report are incorporated into plans, or if any of the assumptions used in the preparation of this report are not correct, this firm shall be notified for modifications to this report. Any items not specifically addressed in this report should comply with the FAA, the CBC and/or the requirements of the governing jurisdiction.

The preliminary recommendations of this report are based upon the geotechnical conditions encountered at the site and may be augmented by additional requirements of the engineer, or by additional recommendations provided by this firm based on conditions exposed at the time of construction.

This document, the data, conclusions, and recommendations contained herein are the property of Earth Systems Pacific. This report shall be used in its entirety, with no individual sections reproduced or used out of context. Copies may be made only by Earth Systems Pacific, the client, and the client's authorized agents for use exclusively on the subject project. Any other use is subject to federal copyright laws and the written approval of Earth Systems Pacific.

Thank you for this opportunity to have been of service. If you have any questions, please feel free to contact this office at your convenience.

End of Text.

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APPENDIX A

N

BASE MAP PROVIDED BY: Google Earth (2020)

NOT TO SCALE

Earth Systems Pacific
4378 Old Santa Fe Road, San Luis Obispo, CA 93401
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(805) 544-3276 • Fax (805) 544-1786

SITE VICINITY MAP

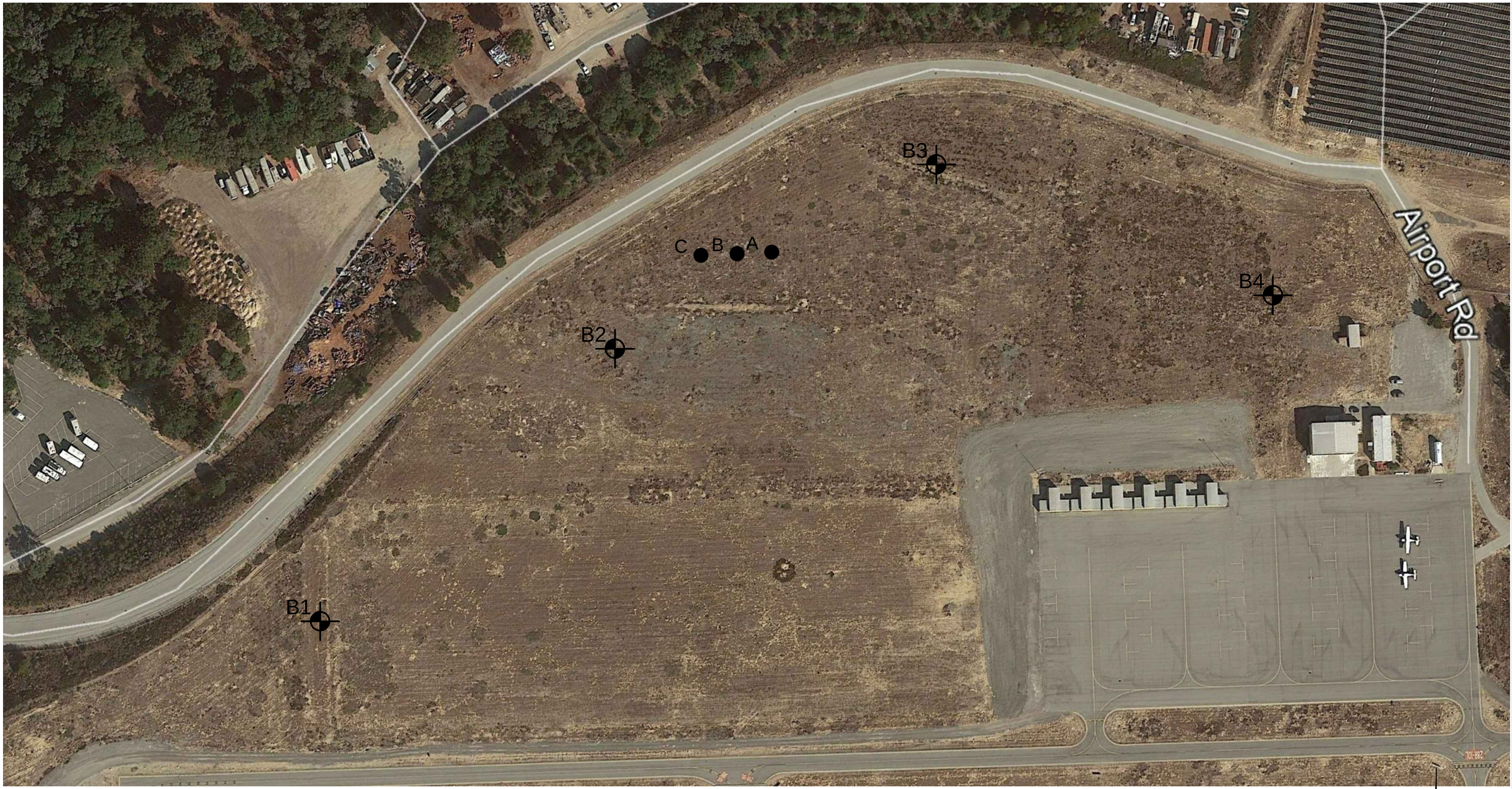
Monterey Regional Airport
Northside General Aviation Apron Construction
200 Fred Kane Dr
Monterey, California

Date
June 2020

Project No.
303018-003

Figure 1

303018–003 Monterey Regional Airport



LEGEND

B1 ⌖ Boring Location (Approx.)

A ● Infiltration Test Location (Approx.)

BASE MAP PROVIDED BY: Google Earth (2020)

NOT TO SCALE

Earth Systems Pacific

4378 Old Santa Fe Road, San Luis Obispo, CA 93401
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(805) 544-3276 • Fax (805) 544-1786

EXPLORATION LOCATION MAP

Monterey Regional Airport
Northside General Aviation Apron Construction
200 Fred Kane Dr
Monterey, California

Date
June 2020

Project No.
303018-003

Figure 2

303018-003 Monterey Regional Airport

Earth Systems Pacific

BORING LOG LEGEND

SAMPLE / SUBSURFACE WATER SYMBOLS	GRAPH. SYMBOL
CALIFORNIA MODIFIED	▬
STANDARD PENETRATION TEST (SPT)	●
SHELBY TUBE	▭
BULK	○
SUBSURFACE WATER DURING DRILLING	▼
SUBSURFACE WATER AFTER DRILLING	▽

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D 2487)

MAJOR DIVISIONS		GROUP SYMBOL	TYPICAL DESCRIPTIONS	GRAPH. SYMBOL
COARSE GRAINED SOILS	MORE THAN HALF OF MATERIAL IS LARGER THAN #200 SIEVE SIZE	GW	WELL GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	○
		GP	POORLY GRADED GRAVELS, OR GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	⬡
		GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES, NON-PLASTIC FINES	▦
		GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES, PLASTIC FINES	⬡
		SW	WELL GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES	▒
		SP	POORLY GRADED SANDS OR GRAVELLY SANDS, LITTLE OR NO FINES	░
		SM	SILTY SANDS, SAND-SILT MIXTURES, NON-PLASTIC FINES	▥
		SC	CLAYEY SANDS, SAND-CLAY MIXTURES, PLASTIC FINES	▨
FINE GRAINED SOILS	HALF OR MORE OF MATERIAL IS SMALLER THAN #200 SIEVE SIZE	ML	INORGANIC SILTS AND VERY FINE SANDS, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY	▥
		CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS	▧
		OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY	┄
		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS	▥
		CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS	▧
		OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS	┅
		PT	PEAT AND OTHER HIGHLY ORGANIC SOILS	▽

OBSERVED MOISTURE CONDITION

DRY	SLIGHTLY MOIST	MOIST	VERY MOIST	WET (SATURATED)

CONSISTENCY

COARSE GRAINED SOILS

BLOWS/FOOT		DESCRIPTIVE TERM
SPT	CA SAMPLER	
0-10	0-16	LOOSE
11-30	17-50	MEDIUM DENSE
31-50	51-83	DENSE
OVER 50	OVER 83	VERY DENSE

FINE GRAINED SOILS

BLOWS/FOOT		DESCRIPTIVE TERM
SPT	CA SAMPLER	
0-2	0-3	VERY SOFT
3-4	4-7	SOFT
5-8	8-13	MEDIUM STIFF
9-15	14-25	STIFF
16-30	26-50	VERY STIFF
OVER 30	OVER 50	HARD

GRAIN SIZES

	U.S. STANDARD SERIES SIEVE				CLEAR SQUARE SIEVE OPENING		
	# 200	# 40	# 10	# 4	3/4"	3"	12"
SILT & CLAY	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

TYPICAL BEDROCK HARDNESS

MAJOR DIVISIONS	TYPICAL DESCRIPTIONS
EXTREMELY HARD	CORE, FRAGMENT, OR EXPOSURE CANNOT BE SCRATCHED WITH KNIFE OR SHARP PICK; CAN ONLY BE CHIPPED WITH REPEATED HEAVY HAMMER BLOWS
VERY HARD	CANNOT BE SCRATCHED WITH KNIFE OR SHARP PICK; CORE OR FRAGMENT BREAKS WITH REPEATED HEAVY HAMMER BLOWS
HARD	CAN BE SCRATCHED WITH KNIFE OR SHARP PICK WITH DIFFICULTY (HEAVY PRESSURE); HEAVY HAMMER BLOW REQUIRED TO BREAK SPECIMEN
MODERATELY HARD	CAN BE GROOVED 1/16 INCH DEEP BY KNIFE OR SHARP PICK WITH MODERATE OR HEAVY PRESSURE; CORE OR FRAGMENT BREAKS WITH LIGHT HAMMER BLOW OR HEAVY MANUAL PRESSURE
SOFT	CAN BE GROOVED OR GOUGED EASILY BY KNIFE OR SHARP PICK WITH LIGHT PRESSURE, CAN BE SCRATCHED WITH FINGERNAIL; BREAKS WITH LIGHT TO MODERATE MANUAL PRESSURE
VERY SOFT	CAN BE READILY INDENTED, GROOVED OR GOUGED WITH FINGERNAIL, OR CARVED WITH KNIFE; BREAKS WITH LIGHT MANUAL PRESSURE

TYPICAL BEDROCK WEATHERING

MAJOR DIVISIONS	TYPICAL DESCRIPTIONS
UNWEATHERED	NO DISCOLORATION, NOT OXIDIZED
SLIGHTLY WEATHERED	DISCOLORATION OR OXIDATION IS LIMITED TO SURFACE OF, OR SHORT DISTANCE FROM, FRACTURES: SOME FELDSPAR CRYSTALS ARE DULL
MODERATELY WEATHERED	DISCOLORATION OR OXIDATION EXTENDS FROM FRACTURES, USUALLY THROUGHOUT; Fe-Mg MINERALS ARE "RUSTY", FELDSPAR CRYSTALS ARE "CLOUDY"
HIGHLY WEATHERED	DISCOLORATION OR OXIDATION THROUGHOUT; FELDSPAR AND Fe-Mg MINERALS ARE ALTERED TO CLAY TO SOME EXTENT, OR CHEMICAL ALTERATION PRODUCES IN SITU DISAGGREGATION
DECOMPOSED	DISCOLORATION OR OXIDATION THROUGHOUT, BUT RESISTANT MINERALS SUCH AS QUARTZ MAY BE UNALTERED; FELDSPAR AND Fe-Mg MINERALS ARE COMPLETELY ALTERED TO CLAY

drafting/masters/Boring Log Legend 062719.dwg

Earth Systems Pacific

Boring No. 1

LOGGED BY: S. Hemmer
DRILL RIG: Mobile B-53 with Automatic Hammer
AUGER TYPE: 6" Hollow Stem

PAGE 1 OF 1
JOB NO.: 303018-003
DATE: 3/23/2020

Monterey Regional Airport
Northside General Aviation Apron Construction
200 Fred Kane Dr
Monterey, California

DEPTH (feet)	USCS CLASS	SYMBOL	SOIL DESCRIPTION	SAMPLE DATA INTERVAL (feet)	SAMPLE TYPE	DRY DENSITY (pcf)	MOISTURE (%)	BLOWS PER 6 IN.
0	SP		POORLY GRADED SAND: brown, loose, very moist (Dune Sand)					
	SP-SM		POORLY GRADED SAND WITH SILT: light yellow brown, dense, moist	2.0 - 3.5	▬	105.8	6.9	9 20 33
	SP		POORLY GRADED SAND: light yellow brown, medium dense, very moist	5.0 - 6.5	●	- -	23.2	10 16 22
				8.5 - 10.0	●	- -	- -	11 16 22
10			End of Boring @ 10.0' No subsurface groundwater encountered					

LEGEND: ▬ Ring Sample ○ Grab Sample ▭ Shelby Tube Sample ● SPT

NOTE: This log of subsurface conditions is a simplification of actual conditions encountered. It applies at the location and time of drilling. Subsurface conditions may differ at other locations and times.

Earth Systems Pacific

Boring No. 2

PAGE 1 OF 1

LOGGED BY: S. Hemmer
DRILL RIG: Mobile B-53 with Automatic Hammer
AUGER TYPE: 6" Hollow Stem

JOB NO.: 303018-003
DATE: 3/23/2020

Monterey Regional Airport
Northside General Aviation Apron Construction
200 Fred Kane Dr
Monterey, California

DEPTH (feet)	USCS CLASS	SYMBOL	SOIL DESCRIPTION	INTERVAL (feet)	SAMPLE TYPE	DRY DENSITY (pcf)	MOISTURE (%)	BLOWS PER 6 IN.
0	SP		POORLY GRADED SAND: brown, loose, very moist (Dune Sand)					
2	SP-SM		POORLY GRADED SAND WITH SILT: light yellow brown, dense, moist	2.0 - 3.5	▬	110.0	9.0	10 24 39
3				0.5 - 5.0	○			
5	SP		POORLY GRADED SAND: light yellow brown, medium dense, moist,	5.0 - 6.5	●	- -	- -	9 12 16
9			dense	8.5 - 10.0	●	- -	9.3	10 16 18
10			End of Boring @ 10.0' No subsurface water encountered					

LEGEND: ▬ Ring Sample ○ Grab Sample ▭ Shelby Tube Sample ● SPT

NOTE: This log of subsurface conditions is a simplification of actual conditions encountered. It applies at the location and time of drilling. Subsurface conditions may differ at other locations and times.

Boring No. 3

LOGGED BY: S. Hemmer
DRILL RIG: Mobile B-53 with Automatic Hammer
AUGER TYPE: 6" Hollow Stem

PAGE 1 OF 1
JOB NO.: 303018-003
DATE: 3/23/2020

Monterey Regional Airport
Northside General Aviation Apron Construction
200 Fred Kane Dr
Monterey, California

DEPTH (feet)	USCS CLASS	SYMBOL	SOIL DESCRIPTION	INTERVAL (feet)	SAMPLE TYPE	DRY DENSITY (pcf)	MOISTURE (%)	BLOWS PER 6 IN.
0	SP		POORLY GRADED SAND: brown, loose, very moist (Dune Sand)					
1	SM		SILTY SAND: light yellow brown, medium dense, moist	2.0 - 3.5	▬	100.6	14.3	10 20 29
2								
3								
4								
5				5.0 - 6.5	●	- -	13.3	10 14 16
6								
7								
8	SP		POORLY GRADED SAND: light brown, dense, moist	8.5 - 10	●	- -	- -	11 16 18
9								
10			End of Boring @ 10.0' No subsurface water encountered					
11								
12								
13								
14								
15								
16								
17								
18								
19								
20								
21								
22								
23								
24								
25								
26								

LEGEND: ▬ Ring Sample ○ Grab Sample ▭ Shelby Tube Sample ● SPT

NOTE: This log of subsurface conditions is a simplification of actual conditions encountered. It applies at the location and time of drilling. Subsurface conditions may differ at other locations and times.

Earth Systems Pacific

Boring No. 4

PAGE 1 OF 1

LOGGED BY: S. Hemmer
DRILL RIG: Mobile B-53 with Automatic Hammer
AUGER TYPE: 6" Hollow Stem

JOB NO.: 303018-003
DATE: 3/23/2020

Monterey Regional Airport
Northside General Aviation Apron Construction
200 Fred Kane Dr
Monterey, California

DEPTH (feet)	USCS CLASS	SYMBOL	SOIL DESCRIPTION	SAMPLE DATA				
				INTERVAL (feet)	SAMPLE TYPE	DRY DENSITY (pcf)	MOISTURE (%)	BLOWS PER 6 IN.
0	SP		POORLY GRADED SAND: brown, loose, very moist (Dune Sand)					
	SM		SILTY SAND: light yellow brown, medium dense, moist	2.0 - 3.5	▬	108.2	12.1	5 7 10
				0.5 - 5.0	○			
				5.0 - 6.5	●	- -	- -	3 4 4
	SP		POORLY GRADED SAND: light brown, dense, moist	8.5 - 10	●	- -	5.6	1 2 3
10			End of Boring @ 10.0' No subsurface water encountered					

LEGEND: ▬ Ring Sample ○ Grab Sample ▭ Shelby Tube Sample ● SPT

NOTE: This log of subsurface conditions is a simplification of actual conditions encountered. It applies at the location and time of drilling. Subsurface conditions may differ at other locations and times.

APPENDIX B

Laboratory Test Results

Monterey Regional Airport
Northside General Aviation Apron Construction

303018-003

BULK DENSITY TEST RESULTS

ASTM D 2937-17 (modified for ring liners)

April 28, 2020

BORING NO.	DEPTH feet	MOISTURE CONTENT, %	WET DENSITY, pcf	DRY DENSITY, pcf
1	3.0 - 3.5	6.9	113.1	105.8
1	5.0 - 5.5	23.2	---	---
2	3.0 - 3.5	9.0	119.9	110.0
2	8.5 - 9.0	9.3	---	---
3	3.0 - 3.5	14.3	115.0	100.6
3	5.0 - 5.5	13.3	---	---
4	3.0 - 3.5	12.1	121.3	108.2
4	8.5 - 9.0	5.6	---	---

Monterey Regional Airport
Northside General Aviation Apron Construction

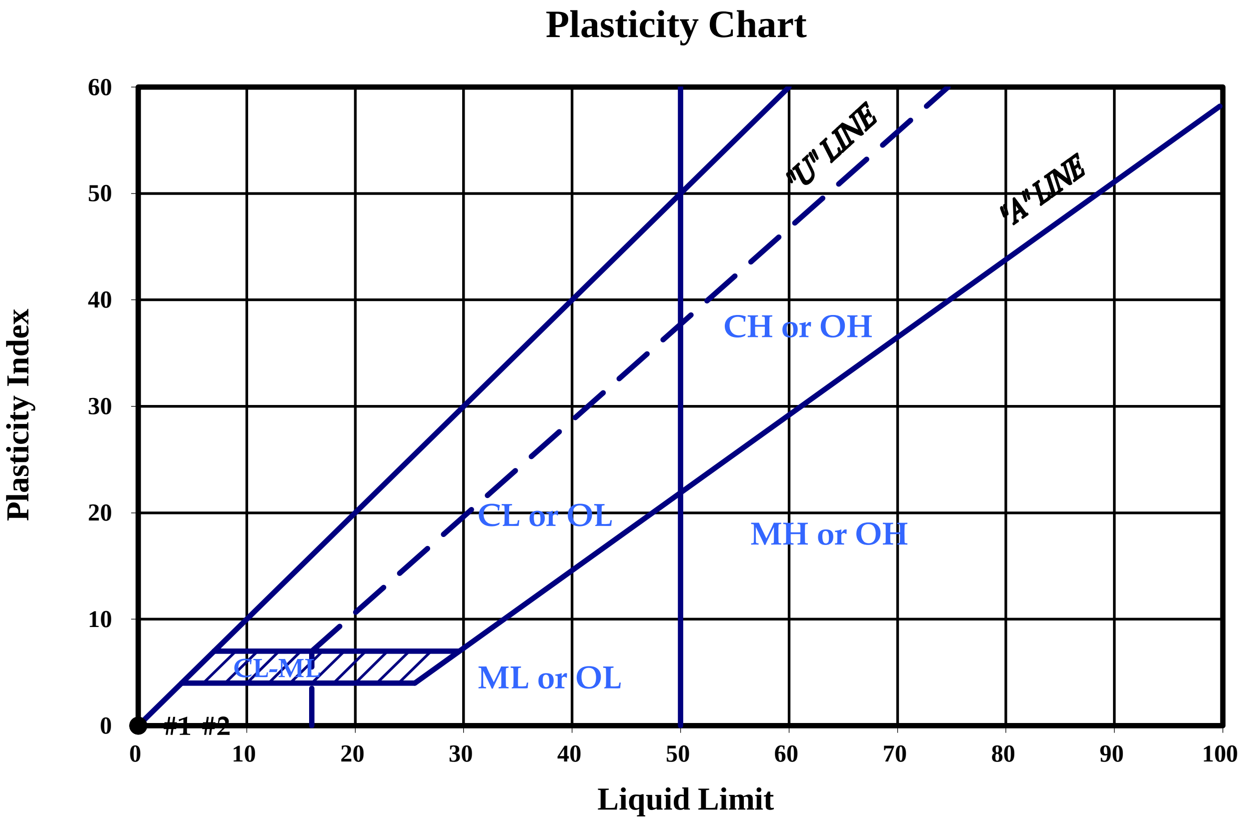
303018-003

PLASTICITY INDEX

ASTM D 4318-17

April 28, 2020

Test No.:	1	2	3	4	5
Boring No.:	2	4			
Sample Depth:	0.5 - 5.0'	0.5 - 5.0'			
Liquid Limit:	NL	NL			
Plastic Limit:	NP	NP			
Plasticity Index:	Nonplastic	Nonplastic			



Monterey Regional Airport
Northside General Aviation Apron Construction

303018-003

PARTICLE SIZE ANALYSIS

ASTM D 422-63/07

Boring #2 @ 0.5 - 5.0'

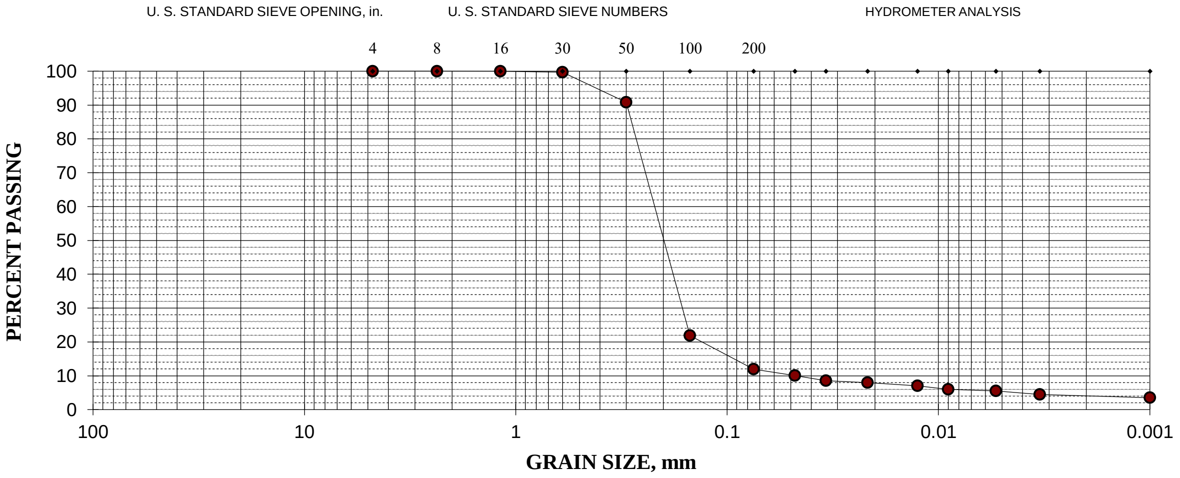
April 28, 2020

Poorly Graded Sand with Silt (SP-SM)

Specific Gravity = 2.65 (assumed)
Gravel = 0%; Sand = 88%; Silt = 6%; Clay = 6%
Cu = 4.6; Cc = 2.5

Sieve size	% Retained	% Passing
#4 (4.75-mm)	0	100
#8 (2.36-mm)	0	100
#16 (1.18-mm)	0	100
#30 (600-µm)	0	100
#50 (300-µm)	9	91
#100 (150-µm)	78	22
#200 (75-µm)	88	12

Hydrometer Analysis		
48-µm		10
34-µm		9
22-µm		8
13-µm		7
9-µm		6
5.3-µm		6
3.3-µm		5
Colloids		4



MOISTURE-DENSITY COMPACTION TEST

ASTM D 1557-12 (Modified)

PROCEDURE USED: A

April 28, 2020

PREPARATION METHOD: Moist

Boring #2 @ 0.5 - 5.0'

RAMMER TYPE: Mechanical

Light Yellow Brown Poorly Graded Sand with Silt (SP-SM)

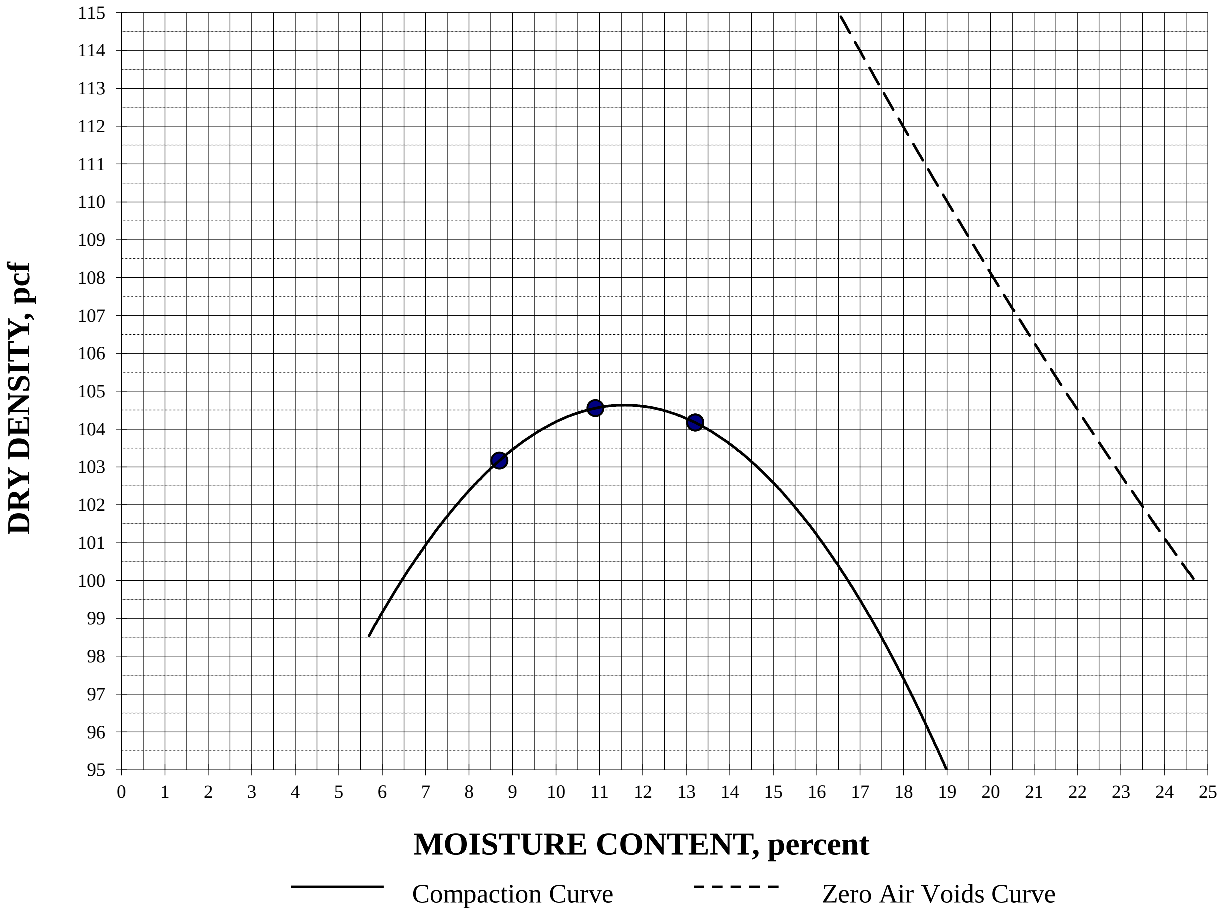
SPECIFIC GRAVITY: 2.65 (assumed)

SIEVE DATA:

Sieve Size	% Retained (Cumulative)
3/4"	0
3/8"	0
#4	0

MAXIMUM DRY DENSITY: 104.6 pcf

OPTIMUM MOISTURE: 11.6%



DIRECT SHEAR

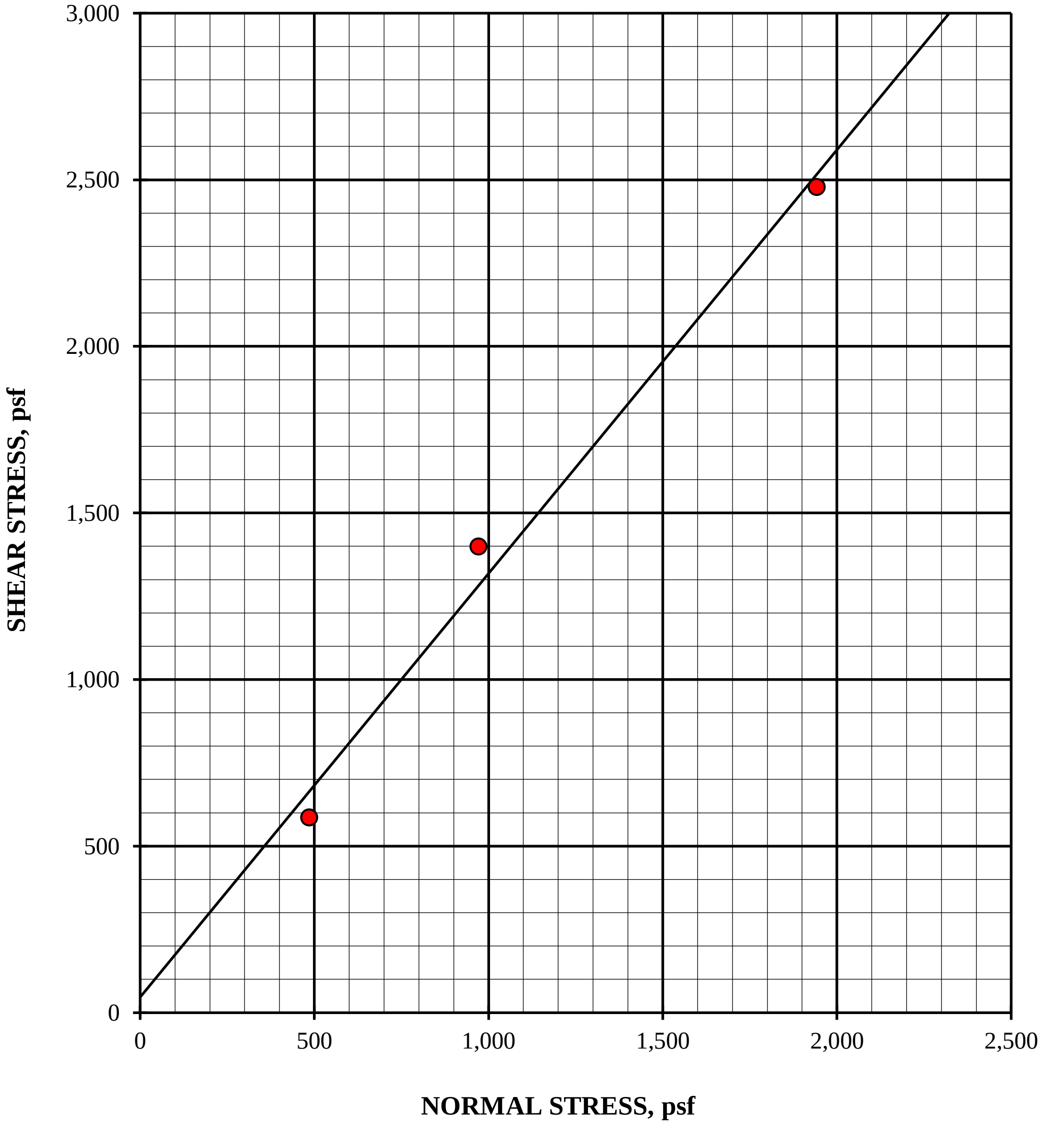
ASTM D 3080/D3080M-11 (modified for consolidated, undrained conditions)

April 28, 2020

Boring #2 @ 0.5 - 5.0'
Poorly Graded Sand with Silt (SP-SM)
Compacted to 90% RC, saturated

INITIAL DRY DENSITY: 94.2 pcf
INITIAL MOISTURE CONTENT: 11.6 %
PEAK SHEAR ANGLE (Ø): 52°
COHESION (C): 47 psf

SHEAR vs. NORMAL STRESS

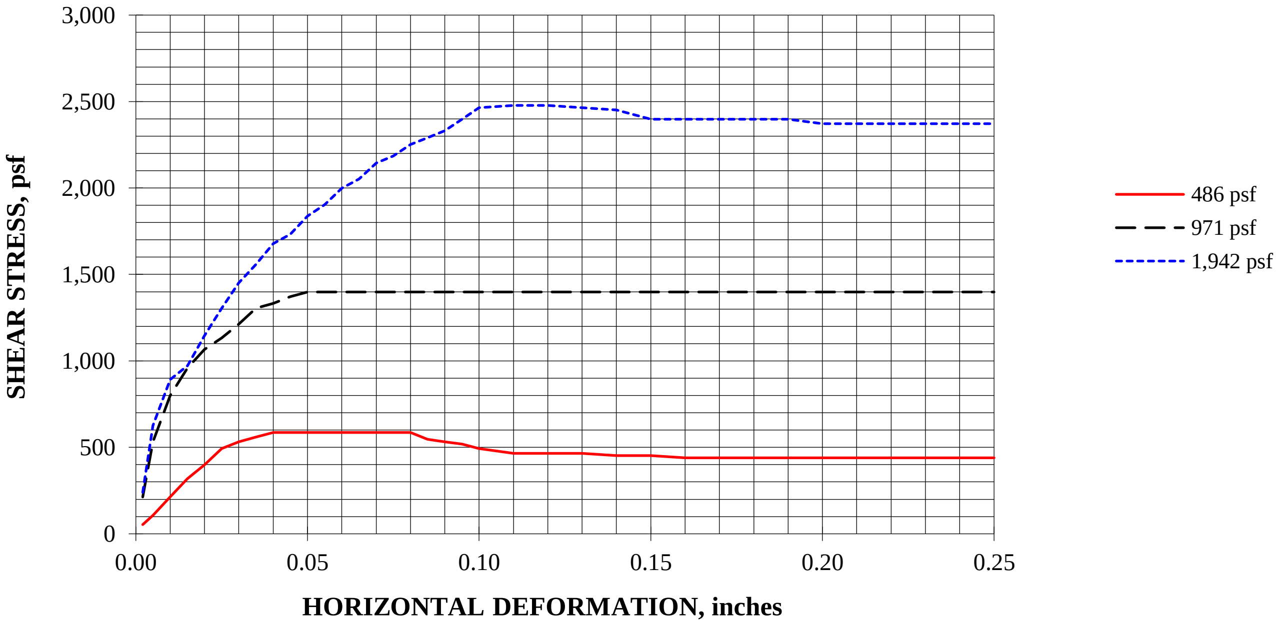


DIRECT SHEAR continued

ASTM D 3080/D3080M-11 (modified for consolidated, undrained conditions)

Boring #2 @ 0.5 - 5.0'
April 28, 2020
Poorly Graded Sand with Silt (SP-SM)
Compacted to 90% RC, saturated
SPECIFIC GRAVITY: 2.65 (assumed)

SAMPLE NO.:	1	2	3	AVERAGE
INITIAL				
WATER CONTENT, %	11.6	11.6	11.6	11.6
DRY DENSITY, pcf	94.2	94.2	94.2	94.2
SATURATION, %	40.7	40.7	40.7	40.7
VOID RATIO	0.756	0.756	0.756	0.756
DIAMETER, inches	2.410	2.410	2.410	
HEIGHT, inches	1.00	1.00	1.00	
AT TEST				
WATER CONTENT, %	21.1	21.1	21.1	
DRY DENSITY, pcf	95.5	96.7	97.8	
SATURATION, %	76.3	78.9	80.7	
VOID RATIO	0.731	0.710	0.691	
HEIGHT, inches	0.99	0.97	0.96	



Monterey Regional Airport 303018-003
Northside General Aviation Apron Construction

CALIFORNIA BEARING RATIO

ASTM D 1883-16 (For a Range of Moisture Contents)

Boring #2 @ 0.5 - 5.0'
Poorly Graded Sand with Silt (SP-SM)

April 28, 2020

10 BLOWS PER LIFT

	-3 Percent	Optimum Moisture	+ 3 percent
Dry density, pcf, before soak	98.2	102.1	104.7
Moisture content, %, before soak	8.6	11.6	14.6
Moisture content, %, after soak, avg.	20.6	18.0	16.4
Moisture content, %, after soak, top 1"	19.9	17.9	16.0
Expansion, %, 96 hour soak	0.0	0.0	0.0
Bearing Ratio, 0.100" penetration	17.8	30.9	31.6

25 BLOWS PER LIFT

	-3 Percent	Optimum Moisture	+ 3 percent
Dry density, pcf, before soak	103.5	104.4	103.2
Moisture content, %, before soak	8.6	11.6	14.6
Moisture content, %, after soak, avg.	18.3	17.1	15.9
Moisture content, %, after soak, top 1"	17.7	15.3	16.1
Expansion, %, 96 hour soak	0.0	0.0	0.0
Bearing Ratio, 0.100" penetration	36.5	50.7	28.7

75 BLOWS PER LIFT

	-3 Percent	Optimum Moisture	+ 3 percent
Dry density, pcf, before soak	105.0	109.4	106.0
Moisture content, %, before soak	8.6	11.6	14.6
Moisture content, %, after soak, avg.	18.0	15.1	16.0
Moisture content, %, after soak, top 1"	17.1	13.1	14.9
Expansion, %, 96 hour soak	0.0	0.0	0.0
Bearing Ratio, 0.100" penetration	31.5	85.4	40.3

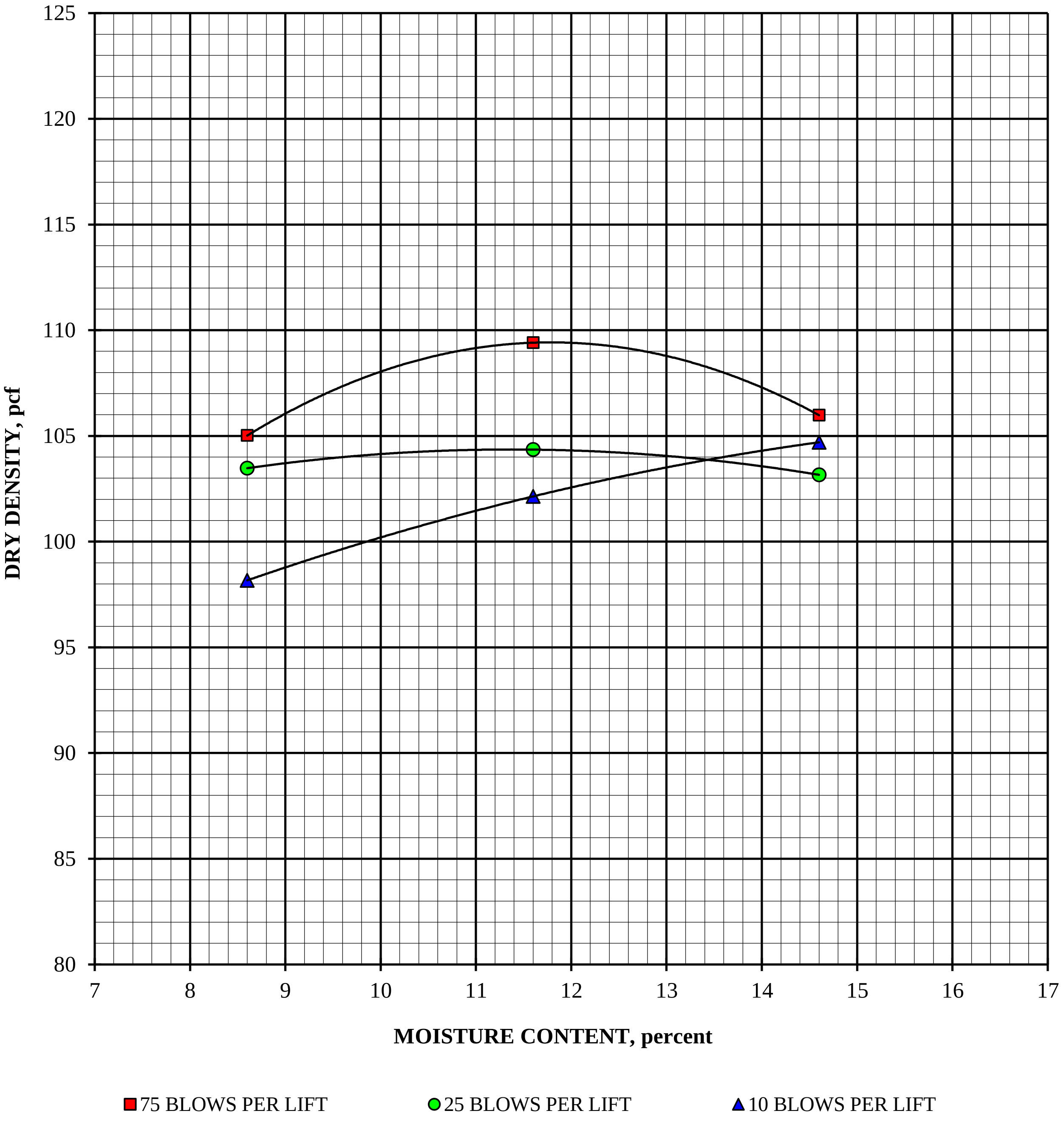
CALIFORNIA BEARING RATIO

ASTM D 1883-16 (For a Range of Moisture Contents)

Boring #2 @ 0.5 - 5.0'
Poorly Graded Sand with Silt (SP-SM)

April 28, 2020

DRY DENSITY vs. MOISTURE CONTENT



CALIFORNIA BEARING RATIO

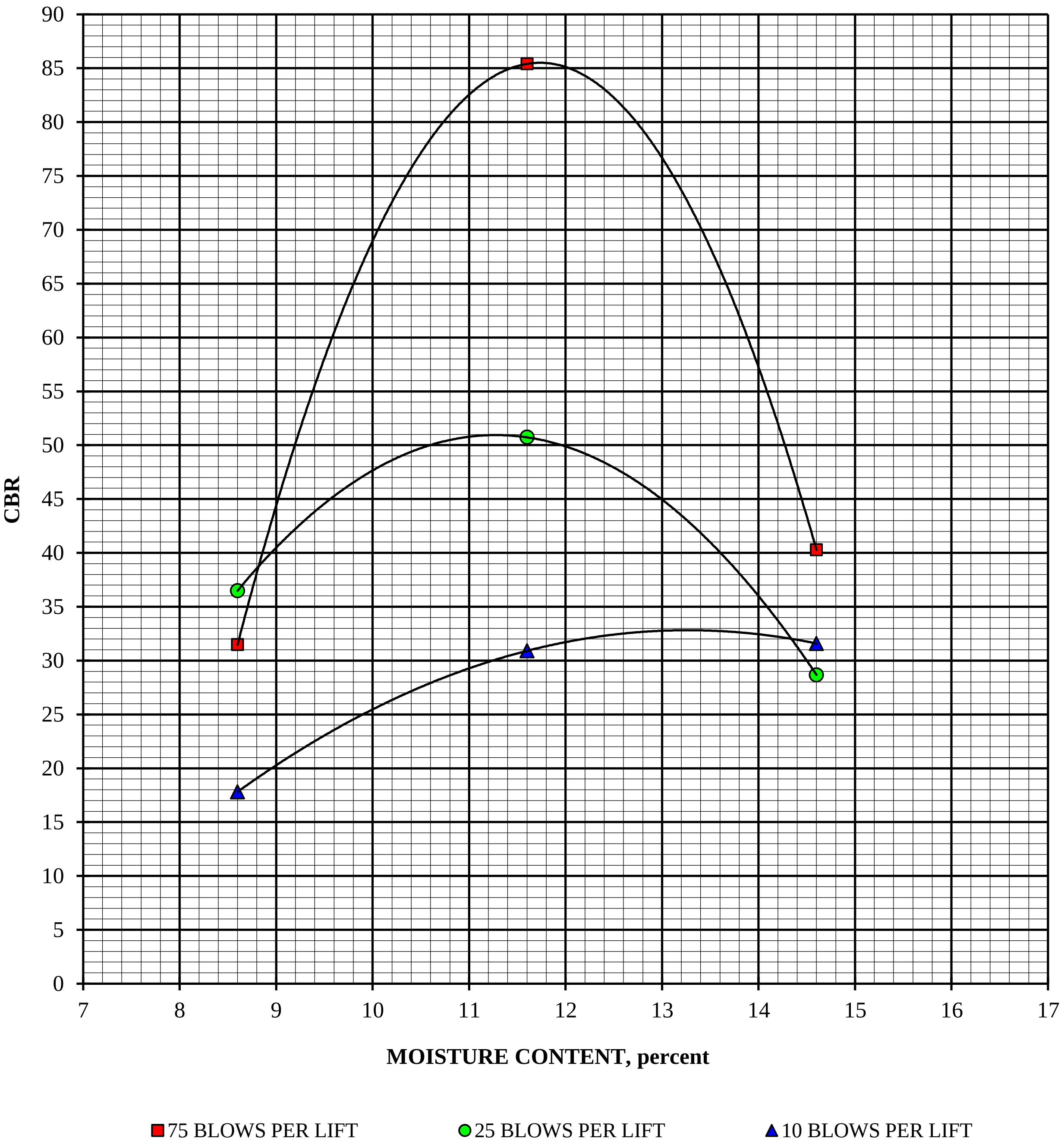
ASTM D 1883-16 (For a Range of Moisture Contents)

Boring #2 @ 0.5 - 5.0'

April 28, 2020

Poorly Graded Sand with Silt (SP-SM)

CBR vs. MOISTURE CONTENT



Monterey Regional Airport 303018-003
Northside General Aviation Apron Construction

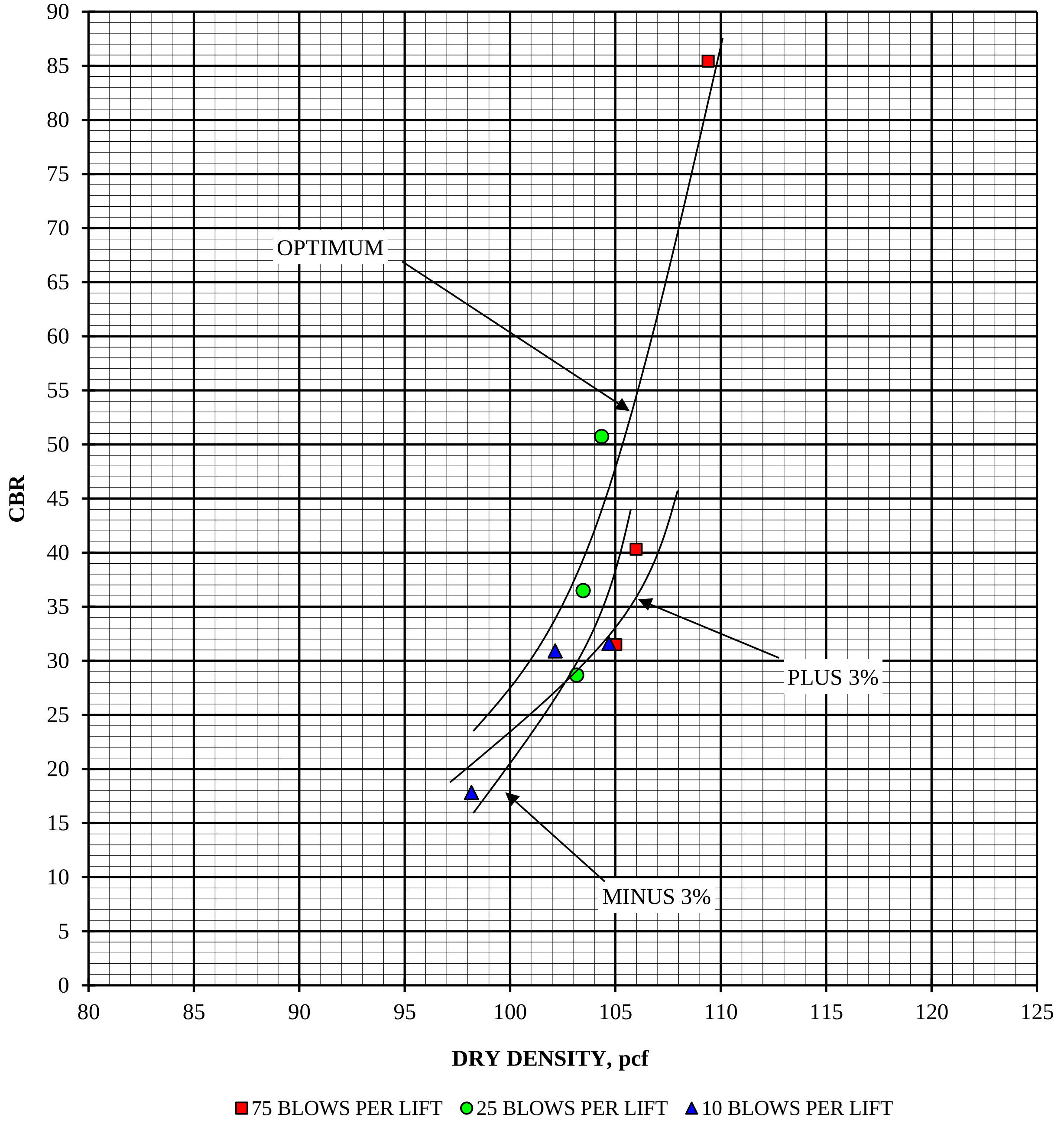
CALIFORNIA BEARING RATIO

ASTM D 1883-16 (For a Range of Moisture Contents)

Boring #2 @ 0.5 - 5.0' April 28, 2020
Poorly Graded Sand with Silt (SP-SM)

DRY DENSITY vs. CBR

Arranged According to Moisture Content



Monterey Regional Airport 303018-003
Northside General Aviation Apron Construction

PARTICLE SIZE ANALYSIS

ASTM D 422-63/07

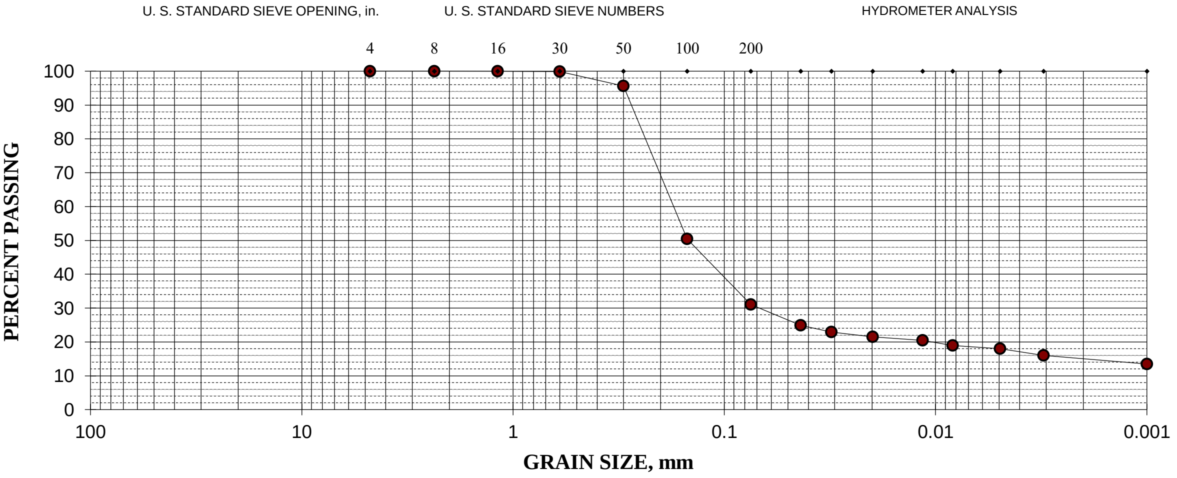
Boring #4 @ 0.5 - 5.0' April 28, 2020

Silty Sand (SM) Specific Gravity = 2.65 (assumed)

Gravel = 0%; Sand = 69%; Silt = 13%; Clay = 18%

Sieve size	% Retained	% Passing
#4 (4.75-mm)	0	100
#8 (2.36-mm)	0	100
#16 (1.18-mm)	0	100
#30 (600-µm)	0	100
#50 (300-µm)	4	96
#100 (150-µm)	50	50
#200 (75-µm)	69	31

Hydrometer Analysis	
43-µm	25
31-µm	23
20-µm	21
12-µm	20
8-µm	19
4.9-µm	18
3.1-µm	16
Colloids	13



Monterey Regional Airport
Northside General Aviation Apron Construction

303018-003

MOISTURE-DENSITY COMPACTION TEST

ASTM D 1557-12 (Modified)

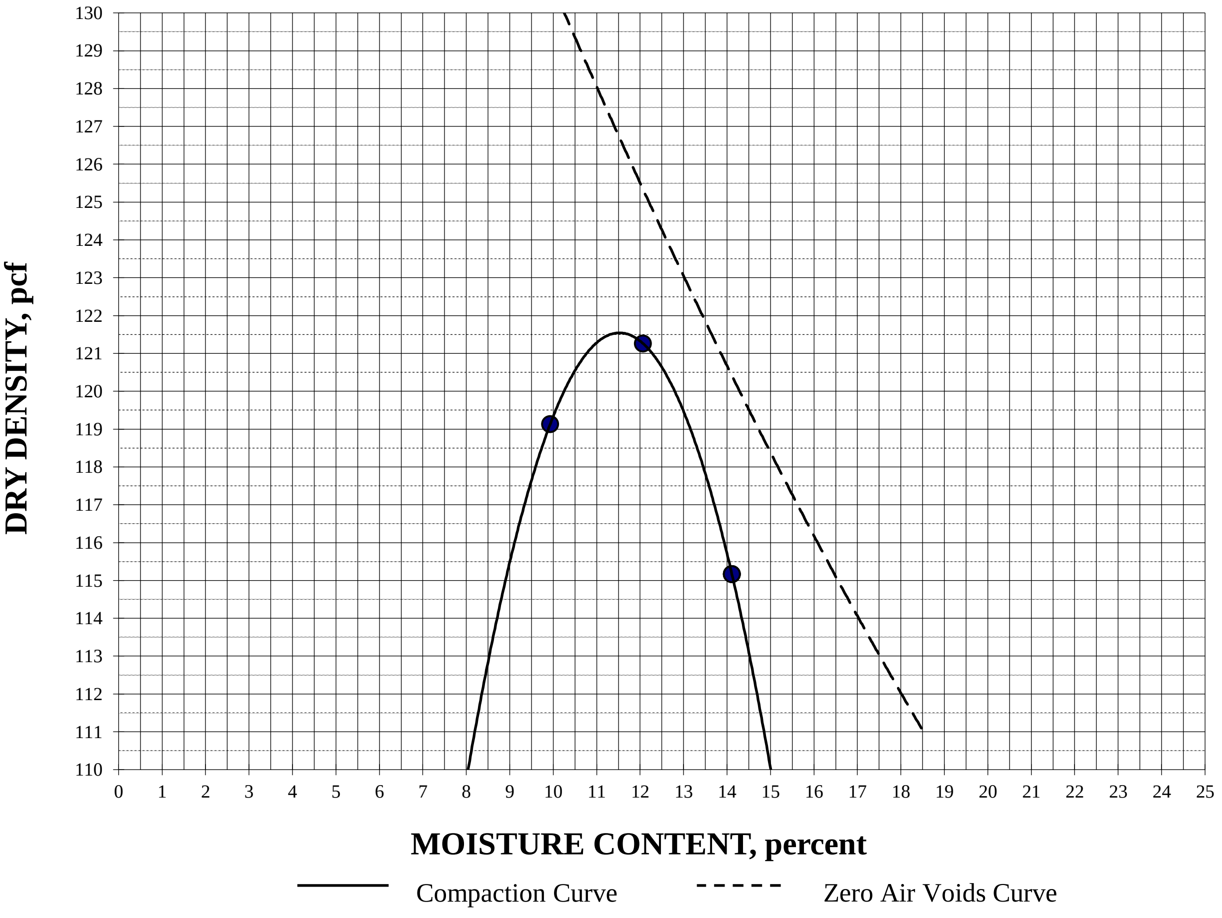
PROCEDURE USED: A
PREPARATION METHOD: Moist
RAMMER TYPE: Mechanical
SPECIFIC GRAVITY: 2.65 (assumed)

April 28, 2020
Boring #4 @ 0.5 - 5.0'
Light Yellow Brown Silty Sand (SM)

SIEVE DATA:

Sieve Size	% Retained (Cumulative)
3/4"	0
3/8"	0
#4	0

MAXIMUM DRY DENSITY: 121.5 pcf
OPTIMUM MOISTURE: 11.5%



Monterey Regional Airport
Northside General Aviation Apron Construction

303018-003

CALIFORNIA BEARING RATIO

ASTM D 1883-16 (For a Range of Moisture Contents)

Boring #4 @ 0.5 - 5.0'
Silty Sand (SM)

April 28, 2020

10 BLOWS PER LIFT

	-3 Percent	Optimum Moisture	+ 3 percent
Dry density, pcf, before soak	109.7	114.9	113.6
Moisture content, %, before soak	8.5	11.5	14.5
Moisture content, %, after soak, avg.	15.4	12.8	14.8
Moisture content, %, after soak, top 1"	16.5	13.7	14.8
Expansion, %, 96 hour soak	0.1	0.0	0.0
Bearing Ratio, 0.100" penetration	13.2	19.6	3.0

25 BLOWS PER LIFT

	-3 Percent	Optimum Moisture	+ 3 percent
Dry density, pcf, before soak	116.2	119.3	114.1
Moisture content, %, before soak	8.5	11.5	14.5
Moisture content, %, after soak, avg.	13.7	12.8	14.6
Moisture content, %, after soak, top 1"	13.7	12.5	13.9
Expansion, %, 96 hour soak	0.0	0.0	0.0
Bearing Ratio, 0.100" penetration	17.4	24.0	3.9

75 BLOWS PER LIFT

	-3 Percent	Optimum Moisture	+ 3 percent
Dry density, pcf, before soak	122.0	121.0	115.0
Moisture content, %, before soak	8.5	11.5	14.5
Moisture content, %, after soak, avg.	11.6	13.1	15.3
Moisture content, %, after soak, top 1"	11.9	12.0	14.0
Expansion, %, 96 hour soak	0.0	0.0	0.0
Bearing Ratio, 0.100" penetration	22.2	32.1	3.0

Monterey Regional Airport
Northside General Aviation Apron Construction

303018-003

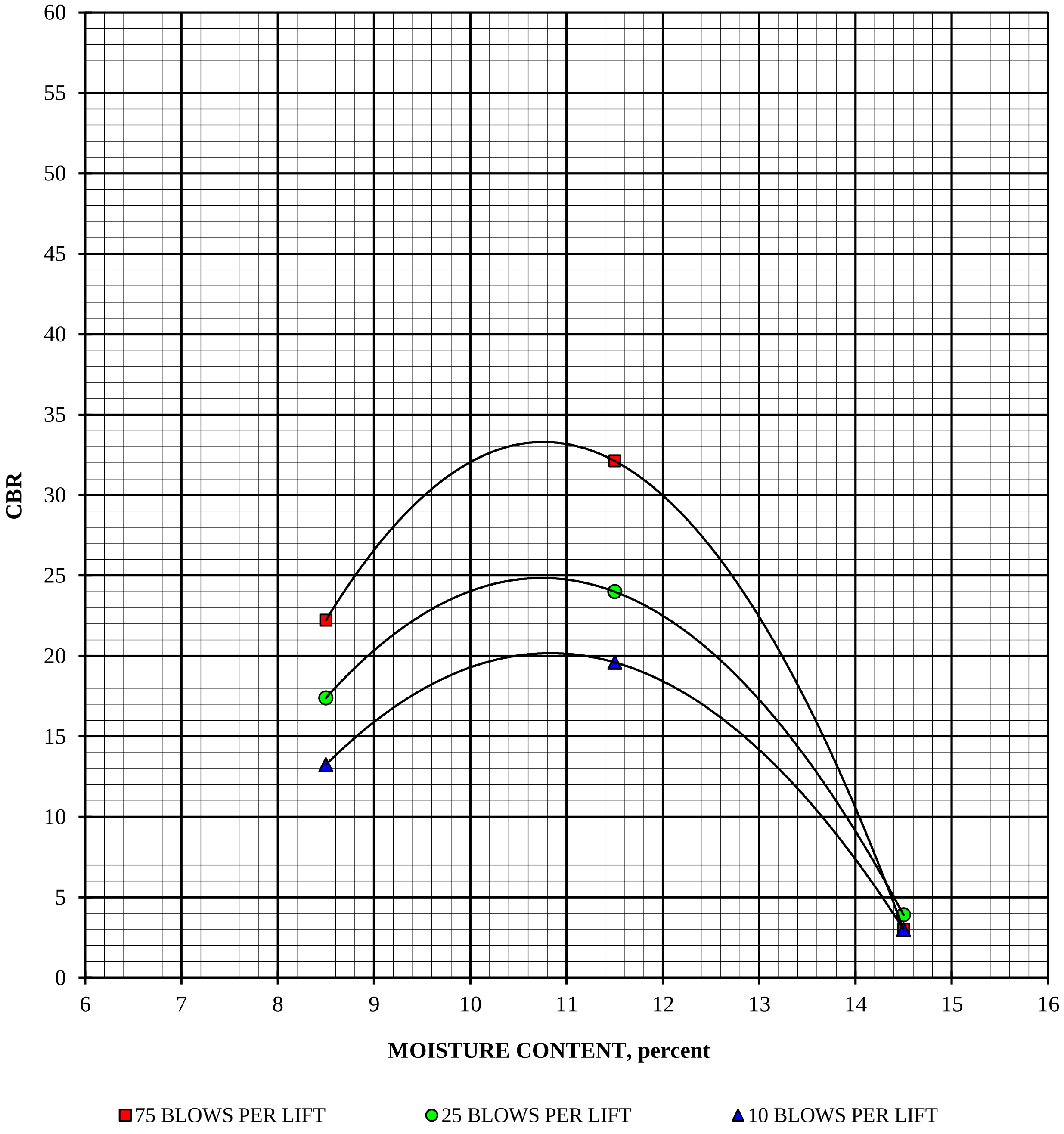
CALIFORNIA BEARING RATIO

ASTM D 1883-16 (For a Range of Moisture Contents)

Boring #4 @ 0.5 - 5.0'
Silty Sand (SM)

April 28, 2020

CBR vs. MOISTURE CONTENT



Monterey Regional Airport
Northside General Aviation Apron Construction

303018-003

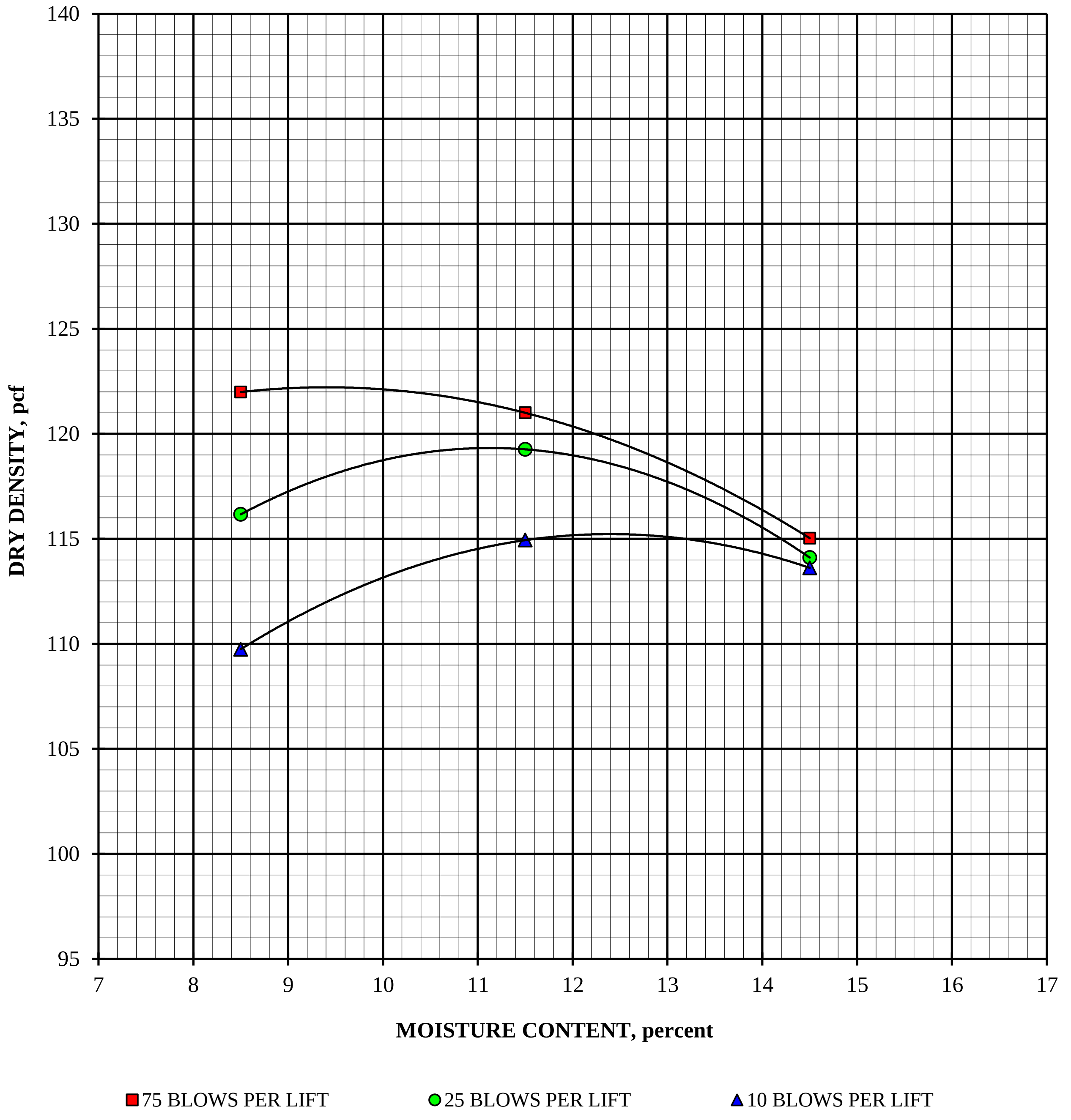
CALIFORNIA BEARING RATIO

ASTM D 1883-16 (For a Range of Moisture Contents)

Boring #4 @ 0.5 - 5.0'
Silty Sand (SM)

April 28, 2020

DRY DENSITY vs. MOISTURE CONTENT



Monterey Regional Airport
Northside General Aviation Apron Construction

303018-003

CALIFORNIA BEARING RATIO

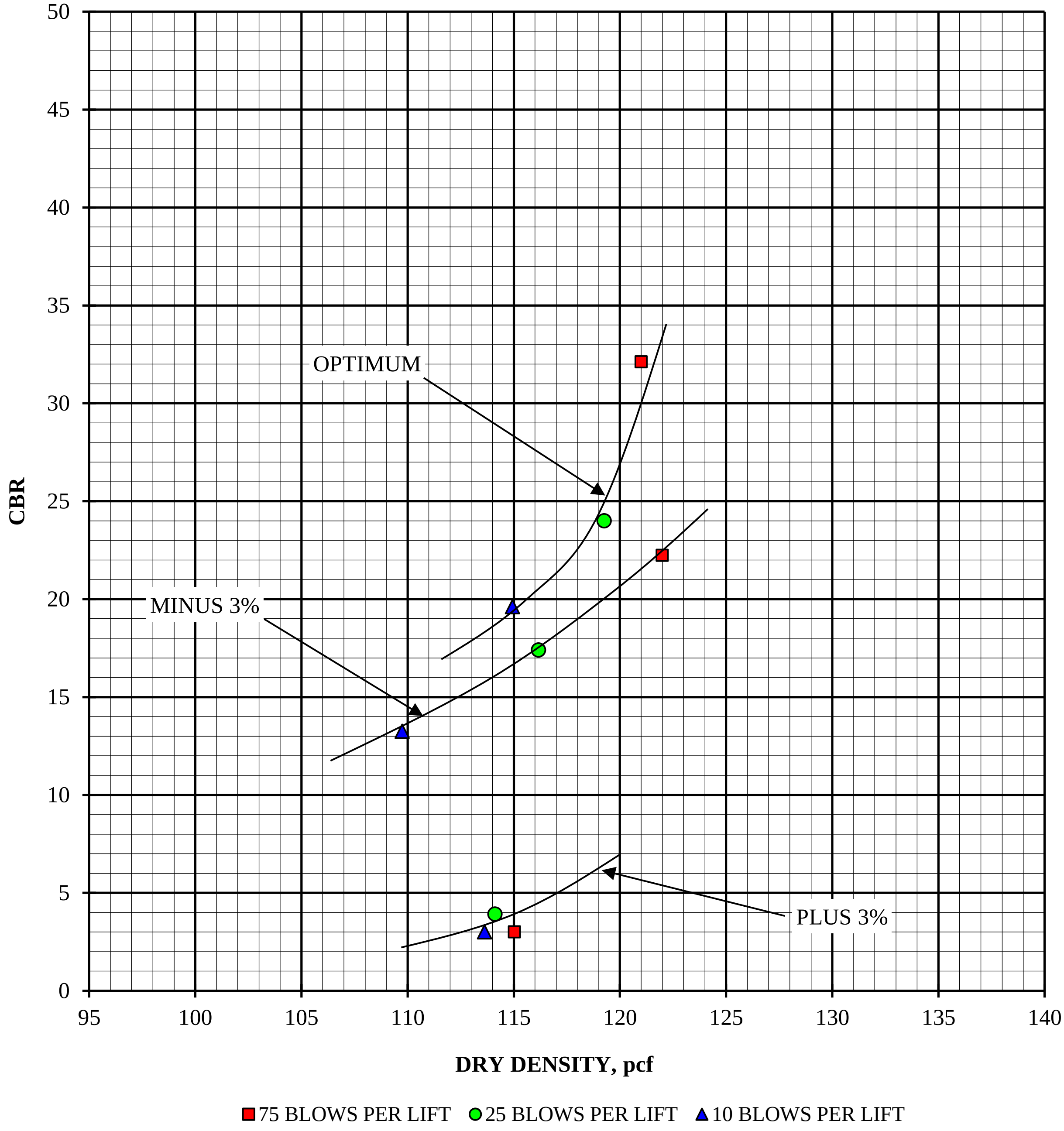
ASTM D 1883-16 (For a Range of Moisture Contents)

Boring #4 @ 0.5 - 5.0'
Silty Sand (SM)

April 28, 2020

DRY DENSITY vs. CBR

Arranged According to Moisture Content



APPENDIX C

LID Infiltration Test Results

Project: Monterey Regional Airport
Northside General Aviation Apron Construction

303018-003
Page 1 of 2

INFILTRATION TEST RESULTS

INFILTRATION TEST: **A**

DATE DRILLED: **3/23/2020**

TEST HOLE DIAMETER: **6 inches**

DATE TESTED: **3/23/2020**

TEST HOLE DEPTH: **2 feet**

TECHNICIAN: **SH**

TEST DURATION: **4 hours**

CONSTANT HEAD DATA

Time of Constant Head: 30 minutes

Volume Added During Constant Head: 1.5 gallons

FALLING HEAD DATA

INTERVAL (Minutes)	READING (Inches)	INCREMENTAL FALL (Inches)	INFILTRATION RATE (Minutes / Inch)	INFILTRATION RATE (Inches / Hour)
0	3.00	---	---	---
10	6.75	3.75	2.67	22.50
10	12.00	5.25	1.90	31.50
10	16.00	4.00	2.50	24.00
10	18.25	2.25	4.44	13.50
10	20.00	1.75	5.71	10.50
10	22.00	2.00	5.00	12.00
10	23.50	1.50	6.67	9.00
10	25.00	1.50	6.67	9.00
0	3.00	---	---	---
10	6.25	3.25	3.08	19.50
10	11.25	5.00	2.00	30.00
10	14.25	3.00	3.33	18.00
10	17.25	3.00	3.33	18.00
10	19.50	2.25	4.44	13.50
10	21.25	1.75	5.71	10.50
10	23.00	1.75	5.71	10.50
10	24.25	1.25	8.00	7.50
10	25.50	1.25	8.00	7.50
0	2.00	---	---	---

INFILTRATION TEST RESULTS

INFILTRATION TEST: **A**

DATE DRILLED: **3/23/2020** **TEST HOLE DIAMETER:** **6 inches**

DATE TESTED: **3/23/2020** **TEST HOLE DEPTH:** **2 feet**

TECHNICIAN: **SH** **TEST DURATION:** **4 hours**

CONSTANT HEAD DATA

Time of Constant Head: 30 minutes

Volume Added During Constant Head: 1.5 gallons

FALLING HEAD DATA

INTERVAL (Minutes)	READING (Inches)	INCREMENTAL FALL (Inches)	INFILTRATION RATE (Minutes / Inch)	INFILTRATION RATE (Inches / Hour)
10	4.75	2.75	3.64	16.50
10	9.25	4.50	2.22	27.00
10	13.25	4.00	2.50	24.00
10	16.50	3.25	3.08	19.50
10	19.00	2.50	4.00	15.00
10	20.75	1.75	5.71	10.50
10	21.50	0.75	13.33	4.50
10	22.25	0.75	13.33	4.50

Project: Monterey Regional Airport 303018-003
Northside General Aviation Apron Construction

INFILTRATION TEST RESULTS

INFILTRATION TEST: **B**

DATE DRILLED: **3/23/2020**

TEST HOLE DIAMETER: **6 inches**

DATE TESTED: **3/23/2020**

TEST HOLE DEPTH: **5 feet**

TECHNICIAN: **SH**

TEST DURATION: **2 hours**

CONSTANT HEAD DATA

Time of Constant Head: 30 minutes

Volume Added During Constant Head: 6 gallons

FALLING HEAD DATA

INTERVAL (Minutes)	READING (Inches)	INCREMENTAL FALL (Inches)	INFILTRATION RATE (Minutes / Inch)	INFILTRATION RATE (Inches / Hour)
0	4.00	---	---	---
10	31.00	27.00	0.37	162.00
10	45.00	14.00	0.71	84.00
10	52.50	7.50	1.33	45.00
0	4.00	---	---	---
10	25.50	21.50	0.47	129.00
10	39.75	14.25	0.70	85.50
10	47.00	7.25	1.38	43.50
10	52.00	5.00	2.00	30.00
0	4.00	---	---	---
10	23.00	19.00	0.53	114.00
10	37.00	14.00	0.71	84.00
10	45.00	8.00	1.25	48.00
10	49.50	4.50	2.22	27.00
10	53.00	3.50	2.86	21.00
10	57.00	4.00	2.50	24.00

Project: Monterey Regional Airport
Northside General Aviation Apron Construction

303018-003
Page 1 of 2

INFILTRATION TEST RESULTS

INFILTRATION TEST: **C**

DATE DRILLED: **3/23/2020**

TEST HOLE DIAMETER: **6 inches**

DATE TESTED: **3/23/2020**

TEST HOLE DEPTH: **8 feet**

TECHNICIAN: **SH**

TEST DURATION: **4 hours**

CONSTANT HEAD DATA

Time of Constant Head: 30 minutes

Volume Added During Constant Head: 10 gallons

FALLING HEAD DATA

INTERVAL (Minutes)	READING (Inches)	INCREMENTAL FALL (Inches)	INFILTRATION RATE (Minutes / Inch)	INFILTRATION RATE (Inches / Hour)
0	2.00	---	---	---
10	41.25	39.25	0.25	235.50
10	59.00	17.75	0.56	106.50
10	68.50	9.50	1.05	57.00
10	74.00	5.50	1.82	33.00
10	79.50	5.50	1.82	33.00
10	89.00	9.50	1.05	57.00
10	92.00	3.00	3.33	18.00
0	2.00	---	---	---
10	38.00	36.00	0.28	216.00
10	53.00	15.00	0.67	90.00
10	61.00	8.00	1.25	48.00
10	67.50	6.50	1.54	39.00
10	72.25	4.75	2.11	28.50
10	76.00	3.75	2.67	22.50
10	81.00	5.00	2.00	30.00
10	84.00	3.00	3.33	18.00
10	87.00	3.00	3.33	18.00
10	89.50	2.50	4.00	15.00
10	92.00	2.50	4.00	15.00
0	2.00	---	---	---

INFILTRATION TEST RESULTS

INFILTRATION TEST: **C**

DATE DRILLED: **3/23/2020**

TEST HOLE DIAMETER: **6 inches**

DATE TESTED: **3/23/2020**

TEST HOLE DEPTH: **8 feet**

TECHNICIAN: **SH**

TEST DURATION: **4 hours**

CONSTANT HEAD DATA

Time of Constant Head: 30 minutes

Volume Added During Constant Head: 10 gallons

FALLING HEAD DATA

INTERVAL (Minutes)	READING (Inches)	INCREMENTAL FALL (Inches)	INFILTRATION RATE (Minutes / Inch)	INFILTRATION RATE (Inches / Hour)
10	29.25	27.25	0.37	163.50
10	48.50	19.25	0.52	115.50
10	58.00	9.50	1.05	57.00
10	64.50	6.50	1.54	39.00
10	68.75	4.25	2.35	25.50
10	73.00	4.25	2.35	25.50
10	77.00	4.00	2.50	24.00
10	80.25	3.25	3.08	19.50
10	86.50	6.25	1.60	37.50
10	89.50	3.00	3.33	18.00
10	92.00	2.50	4.00	15.00

APPENDIX D

Estimate of Earthwork Shrinkage

Monterey Regional Airport **ESP File No. 303018-003**

Northside General Aviation Apron Construction

Estimates of Soil Shrinkage Using In-Place Density Values from Borings and Assumed Final Relative Compaction Values. All Calculations Based on Uniform Density, Moisture Content and Compaction Effort Negative Values Indicate Expansion (Bulking).

Boring No.	Depth	Material Description	USCS Classification	Maximum Density, pcf	Optimum Moisture, %
1	0.5 - 5.0 ft	Poorly Graded Sand with Silt	SP-SM	104.6	11.6
2	0.5 - 5.0 ft	Poorly Graded Sand with Silt	SP-SM	104.6	11.6
3	0.5 - 5.0 ft	Silty Sand	SM	121.5	11.5
4	0.5 - 5.0 ft	Silty Sand	SM	121.5	11.5

Boring	Depth, Ft. Below Ext. Grade	Moisture in Place, %	Dry Density in Place, pcf	Maximum Dens., pcf	Existing Rel.Comp. %	Shrinkage, % at 90.0 % Rel. Comp.	Shrinkage, % at 91.0 % Rel. Comp.	Shrinkage, % at 92.0 % Rel. Comp.	Shrinkage, % at 93.0 % Rel. Comp.	Shrinkage, % at 94.0 % Rel. Comp.	Shrinkage, % at 95.0 % Rel. Comp.	Shrinkage, % at 96.0 % Rel. Comp.	Shrinkage, % at 97.0 % Rel. Comp.	Shrinkage, % at 98.0 % Rel. Comp.	Shrinkage, % at 99.0 % Rel. Comp.	Shrinkage, % at 100.0 % Rel. Comp.
1	3.0 - 3.5	6.9	105.8	104.6	101.1	-11.0	-10.0	-9.0	-8.1	-7.1	-6.1	-5.1	-4.1	-3.1	-2.1	-1.1
2	3.0 - 3.5	9.0	110.0	104.6	105.2	-14.4	-13.5	-12.5	-11.6	-10.6	-9.7	-8.7	-7.8	-6.8	-5.9	-4.9
3	3.0 - 3.5	14.3	100.6	121.5	82.8	8.7	9.9	11.1	12.3	13.5	14.7	15.9	17.2	18.4	19.6	20.8
4	3.0 - 3.5	12.1	108.2	121.5	89.1	1.1	2.2	3.3	4.4	5.6	6.7	7.8	8.9	10.0	11.2	12.3
Average Shrinkage, percent, all locations :						**-3.9**	**-2.9**	**-1.8**	**-0.7**	**0.4**	**1.4**	**2.5**	**3.6**	**4.6**	**5.7**	**6.8**
						At 90.0 % Rel. Comp.	**At 91.0 % Rel. Comp.**	**At 92.0 % Rel. Comp.**	**At 93.0 % Rel. Comp.**	**At 94.0 % Rel. Comp.**	**At 95.0 % Rel. Comp.**	**At 96.0 % Rel. Comp.**	**At 97.0 % Rel. Comp.**	**At 98.0 % Rel. Comp.**	**At 99.0 % Rel. Comp.**	**At 100.0 % Rel. Comp.**

GEOTECHNICAL ENGINEERING REPORT
MONTEREY REGIONAL AIRPORT
NORTHEAST VEHICLE SERVICE
ROAD IMPROVEMENTS
200 FRED KANE DRIVE
MONTEREY, CALIFORNIA

June 17, 2020

Prepared for

Mr. John Smith, PE
Tartaglia Engineering
Project No. 20-13

Prepared by

Earth Systems Pacific
4378 Old Santa Fe Road
San Luis Obispo, CA 93401

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June 17, 2020

FILE NO.: 303018-004

Mr. John Smith, PE
Tartaglia Engineering
PO Box 1930
Atascadero, CA 93423

PROJECT: MONTEREY REGIONAL AIRPORT
NORTHEAST VEHICLE SERVICE ROAD IMPROVEMENTS
200 FRED KANE DRIVE
MONTEREY, CALIFORNIA
TARTAGLIA ENGINEERING PROJECT NO. 20-13

SUBJECT: Geotechnical Engineering Report

CONTRACT REF: Proposal to Provide a Geotechnical Engineering Investigation and Recommendations – Monterey Regional Airport – Northeast Service Road Improvements, Monterey, California, by Earth Systems Pacific, Doc. No. 2002-040.PRP.REV, revised February 28, 2020

Dear Mr. Smith:

As per your authorization of the referenced proposal, this geotechnical engineering report has been prepared for use in the design of the Northeast Vehicle Service Road Improvement project at Monterey Regional Airport in Monterey, California. Conclusions regarding the subsurface conditions encountered, as well as recommendations for site preparation, grading, utility trenches, pavement sections, drainage around improvements, and observation and testing are provided. Two paper copies and a digital copy of this report are furnished for your use.

We appreciate the opportunity to have provided geotechnical services for this project and look forward to working with you again in the future. If there are any questions concerning this report, please do not hesitate to contact the undersigned.

Sincerely,

Earth Systems Pacific

Ryne Mettler
Staff Engineer

Fred J. Potthast, GE
Principal Engineer

REGISTERED PROFESSIONAL ENGINEER
FREDERICK J. POTTHAST
No. GE 2369
Exp. 03/31/22
GEOTECHNICAL
STATE OF CALIFORNIA

Doc. No.: 2006-004.SER/cr

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APPENDICES

1.0 INTRODUCTION AND SITE SETTING

This geotechnical engineering report has been completed for the client's use in the development of plans and specifications for the Northeast Vehicle Service Road Improvements project at the Monterey Regional Airport in Monterey, California. The project will consist of the construction of a service road approximately 1,400 feet long to connect two existing access roads (Airport Road and Airport Circle) on the northeast side of the airport. The road will be 24 feet wide, with three-foot wide shoulders, and will include drainage improvements and pavement markings. The project will include clearing, grubbing and removal of surface vegetation, roadway pavement composed of hot mix asphalt (HMA) or Portland cement concrete (PCC) over aggregate base (AB), underground utilities, and stormwater detention facilities. Cuts and fills a maximum of 2 feet from the existing topography are anticipated. No retaining walls, light poles, structures or other improvements are planned.

The proposed road alignment is currently an unimproved dirt track extending along the north side of the airport perimeter security fence. Moderate to heavy brush and scattered scrub oak trees line the edges of the existing unimproved road. The locations and dispositions of utility lines in the area are unknown, however two underground utility vaults were noted adjacent to the existing unimproved road on the west end of the project area.

2.0 SCOPE OF SERVICES

The scope of work for this geotechnical engineering report included a general site reconnaissance, subsurface exploration, laboratory testing of soil samples, engineering evaluation of the data collected, and the preparation of this report. The investigation and subsequent recommendations were based on information and base maps provided by the client.

This report and recommendations are intended to comply with the applicable considerations of Sections 1803.1 through 1803.6, J104.3, and J104.4 of the California Building Code (CBSC 2019), the client's requested work scope, and common geotechnical engineering practice in this area under similar conditions at this time. The tests were performed in general conformance with the standards noted, as modified by common geotechnical practice in this area under similar conditions at this time.

It is our intent that this report be used exclusively by the client to form the geotechnical basis of the design of the project described herein. Application beyond this intent is strictly at the user's risk. As there may be geotechnical issues yet to be resolved, the geotechnical engineer should be retained to provide consultation as the project progresses, to assist in verifying that pertinent geotechnical issues have been addressed and to aid in conformance with the intent of this report. In the event this report is used to develop project plans, it may also be advantageous to retain the geotechnical engineer to review the project plans as they near completion to further aid in conformance of the plans with the intent of this report.

This report does not address issues in the domain of the contractor such as, but not limited to, site safety, excavatability, shoring, temporary slope angles, construction methods, etc. Analysis of site geology and of the soil for corrosive potential, radioisotopes, asbestos (either naturally occurring or in man-made products), lead or mold potential, hydrocarbons, or other chemical properties are beyond the scope of this investigation. Ancillary features beyond the pavement areas covered by this report are also not within our scope and are not addressed.

In the event that there are any changes in the nature of the work scope, or if any assumptions used in the preparation of this report prove to be incorrect, the conclusions and recommendations contained in this report shall not be considered valid unless the changes are reviewed and the conclusions of this report modified or verified in writing.

3.0 FIELD INVESTIGATION

On March 23, 2020, four borings were drilled along the proposed road alignment to a maximum depth of 10 feet below the existing ground surfaces (bgs) with a Mobile Drill rig, Model B-53, equipped with 6-inch outside diameter hollow stem auger and an automatic hammer for sampling. The approximate locations of the borings are shown on Figure 1 - Exploration Location Map in Appendix A.

As the borings were drilled, soil samples were obtained using a 3-inch outside diameter ring-lined barrel sampler (ASTM D 3550-17 with shoe similar to D 2937-17) at the estimated subgrade elevation (approximately 2 feet bgs). Standard penetration tests (SPT) using a 2-inch outside diameter split-spoon sampler were also performed in the borings (ASTM D 1586-11) from 5 to 6.5 feet bgs and from 8.5 to 10.0 feet bgs in these boring. Bulk samples were secured from the auger cuttings.

The soils were initially classified and logged in general accordance with the Unified Soils Classification System (ASTM D 2488-17). Final classifications of the soils in accordance with the Unified Soils Classification System (ASTM D 2487-17) were made following completion of laboratory testing. Copies of the boring logs and a boring log legend can also be found in

Appendix A. In reviewing the boring logs and legend, the reader should recognize that the legend is intended as a guideline only, and there are a number of conditions that may influence the soil characteristics as observed during drilling. These include, but are not limited to, cementation, variations in soil moisture, presence of groundwater, and other factors. Consequently, the logger must exercise judgment in interpreting soil characteristics, possibly resulting in soils descriptions that vary somewhat from the legend. Following completion of drilling, the borings were backfilled with cement-treated auger spoils and gravel.

4.0 LABORATORY INVESTIGATION

In situ moisture content and unit dry weight (ASTM D 2937-17, as modified for ring liners) were determined for the ring samples. Two bulk samples were tested for the following: maximum density and optimum moisture (ASTM D 1557-12, modified), particle size distribution (ASTM D 422-63/07; D 1140-17), plasticity index (ASTM D 4318-17), and R-value (ASTM D 2844/2844M-13). Please refer to Appendix B for the laboratory test results.

5.0 GENERAL SUBSURFACE PROFILE

In all four borings, Dune Sand comprised of silty sand, poorly graded sand, and poorly graded sand with silt was encountered for the full depths of the borings. The soils were described during drilling as being medium dense to dense, although loose conditions were found in Boring 3 from the surface to 4 feet, and in Boring 4 from 4.5 to 8 feet. In situ moisture contents varied from slightly moist to moist. Subsurface water was not encountered in any of the borings to the maximum depth explored of 10 feet bgs.

Please refer to the logs in Appendix A for a more complete description of the subsurface conditions found in the borings.

6.0 CONCLUSIONS

In our opinion, the existing unimproved dirt track alignment is suitable, from a geotechnical engineering standpoint, for the proposed improvement project, provided the recommendations contained in this report are incorporated into the design and construction. From a geotechnical engineering standpoint, the primary concerns are the loose conditions found in the upper 4 feet of Boring 3, and the highly erodible nature of the site soils. Although subsurface water was not found in the exploratory borings, there is a potential for subsurface water depending on weather conditions immediately preceding and during construction that could cause instability in the soil. The site soils were found to have a high resistance, or R-value, to the types of loads imposed by vehicle traffic, therefore relatively thin pavement sections are recommended.

The loose conditions found in the upper 4 feet of Boring 3 could be an anomaly local to that boring, or they could extend to a significant distance beyond the boring. A relatively minor program of earthwork is recommended to prepare subgrade for the new pavement improvements in the majority of the project area. Where the loose conditions are present, it is recommended that the earthwork extend an additional foot below subgrade elevation.

The site soils are considered to be highly erodible. It is essential that all surface drainage be controlled and directed to appropriate discharge points, and that surface soils, particularly those disturbed during construction, are stabilized by vegetation or means during and following construction.

Subsurface water was not encountered in any of the borings to the maximum depth drilled of 10 feet below ground surface. However, if soil moisture contents are well above optimum during earthwork operations, the soils could become unstable under equipment traffic. Unstable conditions hinder compaction efforts and are not acceptable to support fill or pavement section placement. General recommendations for stabilization of unstable conditions are provided. Specific recommendations for stabilization should be provided by the geotechnical engineer based on conditions encountered at the time of construction.

7.0 PRELIMINARY GEOTECHNICAL RECOMMENDATIONS

The following preliminary recommendations are for the project as described in the "Introduction" section of this report. If locations, elevations, etc., change, the recommendations contained herein may require modification.

Unless otherwise noted, the following definitions are used in the recommendations presented below. Where terms are not defined, definitions commonly used in the construction industry are intended.

- **Pavement Areas:** The area within and extending a minimum of 1 foot beyond the limits of any areas to receive HMA or PCC pavement or other pavement-related improvements.
- **Grading Area:** The entire area to be graded, including the pavement areas and any areas where surface improvements will be constructed or fill will be placed.
- **Subgrade:** The elevation of the surface upon which aggregate base will be placed for pavement or pavement-related improvements.
- **Existing Grade:** Elevations of the site that existed as of the date of this report.

- **Scarified:** Plowed or ripped in two orthogonal directions to a depth of not less than 8 inches.
- **Moisture Conditioned:** Soil moisture content adjusted to optimum or just above prior to application of compactive effort.
- **Compacted/Recompacted:** Unless otherwise recommended, soils placed in level lifts not exceeding 8 inches in loose thickness and compacted to the minimum percentages of maximum dry density designated by the engineer. The minimum relative compaction should be 90 percent of maximum dry density. Relative compaction should be based on maximum dry density by ASTM D 1557-12 and field density by ASTM D 6938-15, or other methods acceptable to the engineer, the geotechnical engineer and the jurisdiction.

Site Preparation

1. All areas to be graded should be prepared for construction by removing all vegetation, debris, existing pavement and improvements that are not planned to remain, and other potentially deleterious material. Any existing utility lines that will not remain in service should be either removed or abandoned. The appropriate method of utility abandonment will depend upon the type and depth of the utility. Recommendations for abandonment during construction can be made as necessary.

2. Voids created by the removal of materials or utilities described above should be called to the attention of the geotechnical engineer. No fill should be placed unless the underlying soil has been observed by the geotechnical engineer.

Grading

1. Following site preparation, soil in the majority of the pavement areas should be excavated to subgrade elevation. The exposed surfaces should then be scarified, moisture conditioned and recompacted to a minimum of 95 percent of maximum dry density. If fill is required to bring existing grades up to subgrade elevation, the existing grades should be scarified, moisture conditioned, and compacted. Fill should than be placed and compacted in thin, moisture conditioned lifts, with the top 12 inches below subgrade elevation compacted to a minimum of 95 percent of maximum dry density.

2. Following site preparation in the vicinity of Boring 3, where loose conditions were encountered to approximately 4 feet bgs, the existing soils should be excavated to 1 foot below subgrade elevation, or 1 foot below existing grade, whichever is deeper. The

resulting surfaces should be scarified, moisture conditioned and recompacted. Fill should than be placed in compacted in thin, moisture conditioned lifts, with the top 12 inches below subgrade elevation compacted to a minimum of 95 percent of maximum dry density. A representative of this firm should observe the conditions exposed at time of construction and determine the extent of this additional excavation and recompaction operation that is needed. For estimating and bidding purposes, the designer may wish to indicate a minimum extent of this overexcavation (i.e., 50 to 100 feet on each side of Boring 3) on the project plans.

3. In any remaining areas to be graded, the prepared soil surfaces should be scarified, moisture conditioned, and recompacted prior to the placement of any fill.

4. Voids created by dislodging rocks and/or debris during scarification should be backfilled and replaced with appropriate fill material, and the dislodged materials should be removed from the work area.

5. During earthwork operations, if the soils are overly moist so that they become unstable, or if the minimum recommended compaction cannot be readily achieved due to excess soil moisture, then alternate methods (scarification/aeration, or geotextiles, or gravel punching) may be needed to achieve stability.

6. Depending on the time of year that construction operations take place, the most effective methods to deal with unstable conditions due to high soil moisture could be scarification and aeration, or the use of geotextile stabilization fabrics. Scarification and aeration may only be possible if the weather conditions are clear and if the project schedule permits.

7. If the project schedule will not allow drying of the soil naturally, stabilization fabric could be typically utilized. Additional excavation below subgrade or below the grading plane in the erosion control area may also be needed before the stabilization fabric is placed; the depth of overexcavation should be determined by the geotechnical engineer based on conditions exposed at the time of construction. After all excavations are complete, and prior to placement of the geotextiles, the exposed surfaces are typically back-dragged to a smooth condition to the degree practicable with light earthwork equipment. Geotextile stabilization fabric (Mirafi RS380i, RS580i or similar material) is typically placed in the excavated area and extended up the sidewalls of the excavation to within 2 inches of the bottom of the AC layer in pavement areas. Stabilization fabrics are rolled out along the

long dimension of the reconstruction area (not perpendicular to it), and are stretched, overlapped and held in place according to the manufacturer's recommendations. In pavement areas, recycled subbase and/or imported aggregate base, per the overall pavement section design, is placed over the fabric in thin, moisture-conditioned lifts and compacted. Available dry site or imported material, or recycled subbase and/or aggregate base is placed by end-dumping on the fabric and spreading ahead of equipment; equipment traffic is typically not allowed to travel directly over the fabric. Initial lifts of subbase/base are spread and compacted by rubber-tired equipment; subsequent lifts are compacted using sheepsfoot and/or steel-drum equipment. Compaction equipment is usually operated in static mode only until base grade is reached, to reduce the potential for any free water in the underlying soils to be drawn through the fabric and into the subbase or aggregate base. In general fill areas, available on-site or imported materials can be used as fill over the stabilization fabric, provided the material is within the recommended moisture content range.

8. If it appears that stable conditions will not be created at base grade in pavement areas after the use of geotextiles, a layer of geogrid (Tensar TriAx TX-7 or similar material) can be placed according to the manufacturer's recommendations as additional reinforcement at the approximate mid-depth of the subbase/aggregate base layer. Often sufficient material may not be in place over the geotextile stabilization fabric at mid-depth of the design subbase/aggregate base layer to fully mobilize its strength characteristics and to determine if geogrid will be needed, therefore it may be necessary to construct a full-scale test strip of the pavement section, with and without geogrid reinforcement. The test strip will give an indication as to whether or not geogrids will be required in any reconstruction areas.

9. All fill material should be placed in level lifts not exceeding 8 inches in loose thickness. On-site soils or acceptable imported soils may be used for fill once they are cleaned of all debris and deleterious materials. Imported soils to be used as fill should be equal to or better than the site soils with respect to strength characteristics. All proposed imported materials should be reviewed by the soils engineer before being transported to the site.

10. All materials used as fill should be cleaned of any rocks larger than 6 inches in maximum dimension. When fill material includes rocks, the rocks should be placed in a sufficient soil matrix to ensure that voids caused by nesting of the rocks will not occur and that the fill can be properly compacted.

11. All earthwork areas should be firm and unyielding following compaction operations, prior to placement of fill, and at subgrade, aggregate base grade, and finish grade.

12. Permanent cut and fill slopes should not exceed 2:1 (horizontal to vertical) gradient, unless otherwise recommended by the geotechnical engineer.

13. The recommendations of this section are minimums only and may be superseded by the requirements of the engineer or the governing jurisdiction.

Utility Trenches

1. A select, noncorrosive, granular, easily compacted material should be used as bedding and shading immediately around utilities. The site soil may be used for trench backfill above the select material.

2. In pavement areas, the upper 1 foot of trench backfill below subgrade and all aggregate base should be compacted to a minimum of 95 percent of maximum dry density. At depths greater than 1 foot below subgrade elevation, the trench backfill should be compacted to a minimum of 90 percent of maximum dry density. The project engineer should designate the relative compaction percentage and the maximum dry density standard for trench backfill in all other grading areas. Trench backfill in unimproved areas where settlement of the backfill would not be detrimental should be compacted to a minimum of 90 percent of maximum dry density.

3. Trench backfill should be placed in level lifts not exceeding 6 inches in loose thickness and compacted as noted above. Trench backfill should be moisture conditioned to at least optimum moisture content prior to application of compactive effort.

4. The recommendations of this section are minimums only and may be superseded by the requirements of the client, the governing jurisdiction, utility companies or pipe manufacturers.

Pavement Sections

HMA Pavement

1. The following HMA pavement sections are based upon the tested R-values of 77 and 78, and assumed Traffic Indices (TIs) of 5.0 through 7.0. Determination of the appropriate TI the project is left to the design engineer. The HMA sections were calculated in accordance

with the method presented in the "Highway Design Manual" (Caltrans 2018a). The calculated HMA and Class 2 aggregate base (AB) thicknesses are for compacted material. Normal Caltrans construction tolerances should apply. HMA should conform to the requirements of Section 39 of the Standard Specifications (Caltrans 2018b).

HMA Pavement Sections

Traffic Index	HMA (in)	Class 2 AB* (in)
5.0	2.75	4.0
5.5	3.00	4.0
6.0	3.25	4.0
6.5	3.75	4.0
7.0	4.00	4.5

PCC Pavement

1. If unreinforced Portland cement concrete pavement is planned, the following minimum section is recommended:
    - 8 inches plain PCC (4,000 psi minimum)
    - Joint spacing at a maximum of 12 feet on-center each way
    - No. 4 smooth joint dowels at 12-inch centers
    - 6 inches Class 2 AB and subgrade compacted to a minimum of 95 percent of maximum dry density
2. If reinforced concrete pavement is planned, the following minimum section may be used:
    - 6 inches PCC (4,000 psi minimum)
    - Joint spacing at a maximum of 12 feet on-center each way
    - No. 4 rebar at 18-inch centers each way
    - No. 4 smooth joint dowels at 18-inch centers
    - 6 inches Class 2 AB and subgrade compacted to a minimum of 95 percent of maximum dry density
3. Alternately, the pavement may be designed by the architect/engineer for the appropriate loads. Provided that a minimum of 6 inches of AB compacted to a minimum of 95 percent

of maximum dry density is provided, the design may be based on a subgrade modulus of 250 pci (psi/in). For this condition, specification of concrete properties and reinforcing steel is left to the architect/engineer.

General

1. Aggregate base (AB) should conform to the requirements of Section 26 of the Standard Specifications (Caltrans 2018b).

2. HMA and PCC pavement should be constrained by curbs, gutters, flatwork, walls, etc.; free edges to the pavement should be avoided.

3. HMA and PCC pavement should be set back a minimum of 5 feet from any descending slope. Alternately, deepened curbs may be used to constrain the pavement. Where curbs will be deepened in lieu of the recommended setback, the individual situation should be reviewed and specific recommendations prepared by the geotechnical engineer.

4. Subgrade and AB should be firm and unyielding when proof-rolled with heavy, rubber-tired grading equipment prior to continuing construction.

5. Finished pavement surfaces should be sloped to freely drain toward appropriate drainage facilities. Water should not be allowed to stand or pond on or adjacent to pavement, as it could cause premature pavement deterioration or improvement damage.

6. To reduce migration of surface drainage into the subgrade, maintenance of pavement areas is critical. Any cracks that develop in the pavement should be promptly sealed.

7. The local jurisdiction may have additional requirements for pavement or pavers that could take precedence over the above recommendations.

Drainage Around Improvements

1. Unpaved ground surfaces should be graded during construction, and finish graded to direct surface runoff toward drainage facilities in accordance with the requirements of the governing jurisdiction. Swales with improved surfaces, area drains, etc. should be utilized where practical to divert drainage away from paved areas.

2. The site soils are highly erodible. Stabilization of soils, particularly those disturbed by construction, vegetation, or other means during and following construction, is

recommended to reduce erosion damage. Care should be taken to establish and maintain vegetation.

3. Maintenance of drainage improvements is critical to the long-term life of the project. Site improvements should be inspected and maintained on a regular basis.

4. To reduce the potential for disruption of drainage patterns and undermining of pavement or foundations, all rodent activity should be aggressively controlled.

Observation and Testing

1. It must be recognized that the recommendations contained in this report are based on a limited number of borings and rely on continuity of the subsurface conditions encountered. Therefore, the geotechnical engineer should be retained to provide consultation during the design phase, to review plans as they near completion, to interpret this report during construction, and to provide construction monitoring in the form of testing and observation.

2. At a minimum, the following should be provided by the geotechnical engineer during construction:

    - Professional observation during grading
    - Oversight of special inspection during grading

3. Special inspection of grading should be provided as per the requirements of Section 1705.6 and Table 1705.6 of the CBC; the soils special inspector should be under the direction of the geotechnical engineer. Subject to approval by the building official or other jurisdiction, special inspection requirements should be addressed by the geotechnical engineer during the preconstruction meeting (see below) prior to the start of grading operations.

    At a minimum, the following items should be inspected and/or tested by the special inspector:

    - Stripping and clearing of vegetation and deleterious materials (if any) where planned for removal
    - Excavations in grading areas, and corrective operations (scarification/aeration or placement of geotextile stabilization fabric) in any unstable areas

- Fill and imported aggregate base quality, placement, moisture conditioning, and compaction
- Utility trench backfill quality, placement, moisture conditioning, and compaction

4. A program of quality control should be developed prior to beginning grading. The contractor or project manager should determine any additional inspection items required by the architect/engineer or the governing jurisdiction.

5. Locations and frequency of compaction tests should be as per the recommendation of the geotechnical engineer at the time of construction. The recommended test location and frequency may be subject to modification by the geotechnical engineer, based upon soil and moisture conditions encountered, size and type of equipment used by the contractor, the general trend of the results of compaction tests, or other factors.

6. A preconstruction conference among the owner, the geotechnical engineer, the governing agency, the special inspector, the project inspector, the architect/engineer, and contractors is recommended to discuss planned construction procedures and quality control requirements.

7. The geotechnical engineer should be notified at least 48 hours prior to beginning construction operations. If Earth Systems Pacific is not retained to provide construction observation and testing services, it shall not be responsible for the interpretation of the information by others or any consequences arising therefrom.

8.0 CLOSURE

Our intent was to perform the investigation in a manner consistent with the level of care and skill ordinarily exercised by members of the profession currently practicing in the locality of this project and under similar conditions. No representation, warranty, or guarantee is either expressed or implied. This report is intended for the exclusive use by the client as discussed in the “Scope of Services” section. Application beyond the stated intent is strictly at the user's risk.

This report is valid for conditions as they exist at this time for the type of project described herein. The conclusions and recommendations contained in this report could be rendered invalid, either in whole or in part, due to changes in building codes, FAA regulations, standards of geotechnical or construction practice, changes in physical conditions, or the broadening of knowledge.

If changes with respect to development type or location become necessary, if items not addressed in this report are incorporated into plans, or if any of the assumptions used in the preparation of this report are not correct, this firm shall be notified for modifications to this report. Any items not specifically addressed in this report should comply with the FAA, the CBC and/or the requirements of the governing jurisdiction.

The preliminary recommendations of this report are based upon the geotechnical conditions encountered at the site and may be augmented by additional requirements of the engineer, or by additional recommendations provided by this firm based on conditions exposed at the time of construction.

This document, the data, conclusions, and recommendations contained herein are the property of Earth Systems Pacific. This report shall be used in its entirety, with no individual sections reproduced or used out of context. Copies may be made only by Earth Systems Pacific, the client, and the client’s authorized agents for use exclusively on the subject project. Any other use is subject to federal copyright laws and the written approval of Earth Systems Pacific.

Thank you for this opportunity to have been of service. If you have any questions, please feel free to contact this office at your convenience.

End of Text.

TECHNICAL REFERENCES

Caltrans (California Department of Transportation). 2018a. Highway Design Manual.

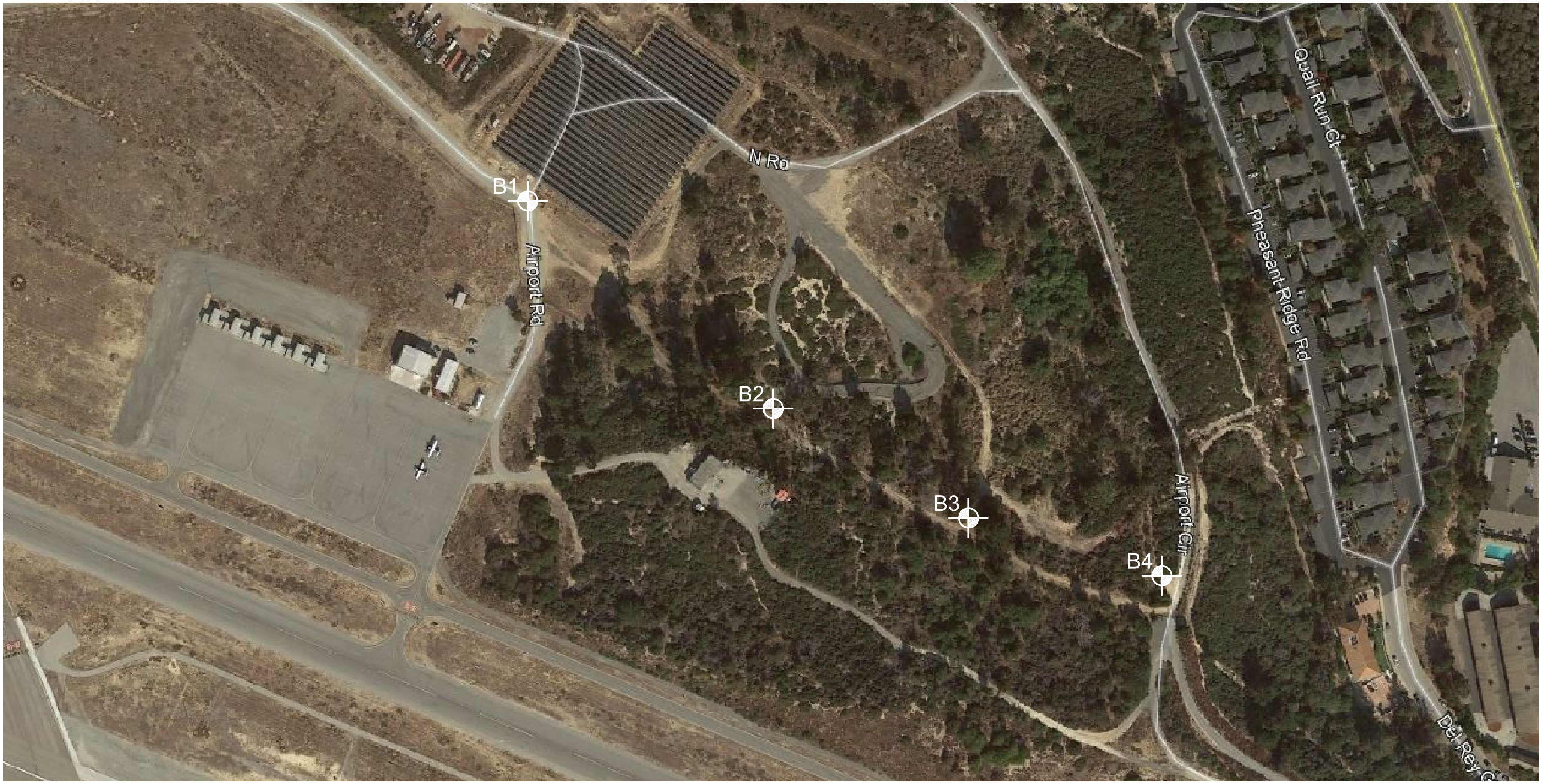
Caltrans (California Department of Transportation). 2018b. Standard Specifications.

CBSC (California Building Standards Commission). 2019. California Building Code (CBC).

Google Earth. 2020. Google Earth [website], retrieved from: http://www.google.com/earth/index.html

APPENDIX A

Figure 1 - Exploration Location Map
Boring Log Legend
Boring Logs



LEGEND

B1 ⊕ Boring Location (Approx.)

BASE MAP PROVIDED BY: Google Earth (2020)

NOT TO SCALE

Earth Systems Pacific

4378 Old Santa Fe Road, San Luis Obispo, CA 93401
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(805) 544-3276 • Fax (805) 544-1786

EXPLORATION LOCATION MAP

Monterey Regional Airport
Northeast Vehicle Service Road Improvements
200 Fred Kane Drive
Monterey, California

Date
June 2020

Project No.
303018-004

Figure 1

Monterey Regional Airport – Northeast Vehicle Service Road Improvements052920map

Earth Systems Pacific

BORING LOG LEGEND

SAMPLE / SUBSURFACE WATER SYMBOLS	GRAPH. SYMBOL
CALIFORNIA MODIFIED	▬
STANDARD PENETRATION TEST (SPT)	●
SHELBY TUBE	▭
BULK	○
SUBSURFACE WATER DURING DRILLING	▼
SUBSURFACE WATER AFTER DRILLING	▽

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D 2487)

MAJOR DIVISIONS	GROUP SYMBOL	TYPICAL DESCRIPTIONS	GRAPH. SYMBOL
COARSE GRAINED SOILS MORE THAN HALF OF MATERIAL IS LARGER THAN #200 SIEVE SIZE	GW	WELL GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	▨
	GP	POORLY GRADED GRAVELS, OR GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	▨
	GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES, NON-PLASTIC FINES	▨
	GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES, PLASTIC FINES	▨
	SW	WELL GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES	▨
	SP	POORLY GRADED SANDS OR GRAVELLY SANDS, LITTLE OR NO FINES	▨
	SM	SILTY SANDS, SAND-SILT MIXTURES, NON-PLASTIC FINES	▨
	SC	CLAYEY SANDS, SAND-CLAY MIXTURES, PLASTIC FINES	▨
FINE GRAINED SOILS HALF OR MORE OF MATERIAL IS SMALLER THAN #200 SIEVE SIZE	ML	INORGANIC SILTS AND VERY FINE SANDS, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY	▨
	CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS	▨
	OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY	▨
	MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS	▨
	CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS	▨
	OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS	▨
	PT	PEAT AND OTHER HIGHLY ORGANIC SOILS	▽ ▽ ▽

OBSERVED MOISTURE CONDITION

DRY	SLIGHTLY MOIST	MOIST	VERY MOIST	WET (SATURATED)

CONSISTENCY

COARSE GRAINED SOILS

BLOWS/FOOT		DESCRIPTIVE TERM
SPT	CA SAMPLER	
0-10	0-16	LOOSE
11-30	17-50	MEDIUM DENSE
31-50	51-83	DENSE
OVER 50	OVER 83	VERY DENSE

FINE GRAINED SOILS

BLOWS/FOOT		DESCRIPTIVE TERM
SPT	CA SAMPLER	
0-2	0-3	VERY SOFT
3-4	4-7	SOFT
5-8	8-13	MEDIUM STIFF
9-15	14-25	STIFF
16-30	26-50	VERY STIFF
OVER 30	OVER 50	HARD

GRAIN SIZES

U.S. STANDARD SERIES SIEVE				CLEAR SQUARE SIEVE OPENING		
# 200	# 40	# 10	# 4	3/4"	3"	12"

SILT & CLAY	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

TYPICAL BEDROCK HARDNESS

MAJOR DIVISIONS	TYPICAL DESCRIPTIONS
EXTREMELY HARD	CORE, FRAGMENT, OR EXPOSURE CANNOT BE SCRATCHED WITH KNIFE OR SHARP PICK; CAN ONLY BE CHIPPED WITH REPEATED HEAVY HAMMER BLOWS
VERY HARD	CANNOT BE SCRATCHED WITH KNIFE OR SHARP PICK; CORE OR FRAGMENT BREAKS WITH REPEATED HEAVY HAMMER BLOWS
HARD	CAN BE SCRATCHED WITH KNIFE OR SHARP PICK WITH DIFFICULTY (HEAVY PRESSURE); HEAVY HAMMER BLOW REQUIRED TO BREAK SPECIMEN
MODERATELY HARD	CAN BE GROOVED 1/16 INCH DEEP BY KNIFE OR SHARP PICK WITH MODERATE OR HEAVY PRESSURE; CORE OR FRAGMENT BREAKS WITH LIGHT HAMMER BLOW OR HEAVY MANUAL PRESSURE
SOFT	CAN BE GROOVED OR GOUGED EASILY BY KNIFE OR SHARP PICK WITH LIGHT PRESSURE, CAN BE SCRATCHED WITH FINGERNAIL; BREAKS WITH LIGHT TO MODERATE MANUAL PRESSURE
VERY SOFT	CAN BE READILY INDENTED, GROOVED OR GOUGED WITH FINGERNAIL, OR CARVED WITH KNIFE; BREAKS WITH LIGHT MANUAL PRESSURE

TYPICAL BEDROCK WEATHERING

MAJOR DIVISIONS	TYPICAL DESCRIPTIONS
UNWEATHERED	NO DISCOLORATION, NOT OXIDIZED
SLIGHTLY WEATHERED	DISCOLORATION OR OXIDATION IS LIMITED TO SURFACE OF, OR SHORT DISTANCE FROM, FRACTURES: SOME FELDSPAR CRYSTALS ARE DULL
MODERATELY WEATHERED	DISCOLORATION OR OXIDATION EXTENDS FROM FRACTURES, USUALLY THROUGHOUT; Fe-Mg MINERALS ARE "RUSTY", FELDSPAR CRYSTALS ARE "CLOUDY"
HIGHLY WEATHERED	DISCOLORATION OR OXIDATION THROUGHOUT; FELDSPAR AND Fe-Mg MINERALS ARE ALTERED TO CLAY TO SOME EXTENT, OR CHEMICAL ALTERATION PRODUCES IN SITU DISAGGREGATION
DECOMPOSED	DISCOLORATION OR OXIDATION THROUGHOUT, BUT RESISTANT MINERALS SUCH AS QUARTZ MAY BE UNALTERED; FELDSPAR AND Fe-Mg MINERALS ARE COMPLETELY ALTERED TO CLAY

drafting/masters/Boring Log Legend 062719.dwg

Earth Systems Pacific

Boring No. B1

LOGGED BY: S. Hemmer
DRILL RIG: Mobile B-53 with Automatic Hammer
AUGER TYPE: 6" Hollow Stem

PAGE 1 OF 1
JOB NO.: 303018-004
DATE: 3/23/2020

Monterey Regional Airport
Northeast Vehicle Service Road Improvements
200 Fred Kane Drive
Monterey, California

DEPTH (feet)	USCS CLASS	SYMBOL	SOIL DESCRIPTION	SAMPLE DATA: INTERVAL (feet)	SAMPLE TYPE	DRY DENSITY (pcf)	MOISTURE (%)	BLOWS PER 6 IN.
0	SM		SILTY SAND: yellow brown, dense, very moist (Dune Sand)					
2				2.0 - 3.5	▬	102.4	11.1	13 27 33
3	SP		POORLY GRADED SAND: light yellow brown, dense, moist					
5				5.0 - 6.5	●	- -	- -	5 6 7
8.5				8.5 - 10.0	●	- -	- -	4 7 8
10			End of Boring @ 10.0' No subsurface water encountered					

LEGEND: ▬ Ring Sample ○ Grab Sample ▭ Shelby Tube Sample ● SPT

NOTE: This log of subsurface conditions is a simplification of actual conditions encountered. It applies at the location and time of drilling. Subsurface conditions may differ at other locations and times.

Earth Systems Pacific

Boring No. B2

LOGGED BY: S. Hemmer
DRILL RIG: Mobile B-53 with Automatic Hammer
AUGER TYPE: 6" Hollow Stem

PAGE 1 OF 1
JOB NO.: 303018-004
DATE: 3/23/2020

Monterey Regional Airport
Northeast Vehicle Service Road Improvements
200 Fred Kane Drive
Monterey, California

				SAMPLE DATA				
DEPTH (feet)	USCS CLASS	SYMBOL	SOIL DESCRIPTION	INTERVAL (feet)	SAMPLE TYPE	DRY DENSITY (pcf)	MOISTURE (%)	BLOWS PER 6 IN.
0	SP-SM		POORLY GRADED SAND WITH SILT: light brown, medium dense, moist (Dune Sand)					
2				2.0 - 3.5	▬	97.1	8.4	7 11 20
3				0.0 - 5.0	○			
5			brown	5.0 - 6.5	●	- -	- -	4 8 11
6			light brown					
8.5				8.5 - 10.0	●	- -	7.1	5 7 9
10			End of Boring @ 10.0' No subsurface water encountered					

LEGEND: ▬ Ring Sample ○ Grab Sample ▭ Shelby Tube Sample ● SPT

NOTE: This log of subsurface conditions is a simplification of actual conditions encountered. It applies at the location and time of drilling. Subsurface conditions may differ at other locations and times.

Boring No. B3

LOGGED BY: S. Hemmer
DRILL RIG: Mobile B-53 with Automatic Hammer
AUGER TYPE: 6" Hollow Stem

PAGE 1 OF 1
JOB NO.: 303018-004
DATE: 3/23/2020

DEPTH (feet)	USCS CLASS	SYMBOL	**Monterey Regional Airport** **Northeast Vehicle Service Road Improvements** **200 Fred Kane Drive** **Monterey, California** SOIL DESCRIPTION	SAMPLE DATA				
				INTERVAL (feet)	SAMPLE TYPE	DRY DENSITY (pcf)	MOISTURE (%)	BLOWS PER 6 IN.
0	SP		POORLY GRADED SAND: light brown, loose, slightly moist (Dune Sand)					
1								
2				2.0 - 3.5	▬	95.8	4.5	3 5 8
3								
4			Medium dense					
5				5.0 - 6.5	●	- -	4.1	3 6 9
6								
7								
8				8.5 - 10.0	●	- -	- -	5 6 8
9								
10			End of Boring @ 10.0' No subsurface water encountered					
11								
12								
13								
14								
15								
16								
17								
18								
19								
20								
21								
22								
23								
24								
25								
26								

LEGEND: ▬ Ring Sample ○ Grab Sample ▭ Shelby Tube Sample ● SPT

NOTE: This log of subsurface conditions is a simplification of actual conditions encountered. It applies at the location and time of drilling. Subsurface conditions may differ at other locations and times.

Earth Systems Pacific

Boring No. B4

LOGGED BY: S. Hemmer
DRILL RIG: Mobile B-53 with Automatic Hammer
AUGER TYPE: 6" Hollow Stem

PAGE 1 OF 1
JOB NO.: 303018-004
DATE: 3/23/2020

DEPTH (feet)	USCS CLASS	SYMBOL	**Monterey Regional Airport** **Northeast Vehicle Service Road Improvements** **200 Fred Kane Drive** **Monterey, California** SOIL DESCRIPTION	SAMPLE DATA				
				INTERVAL (feet)	SAMPLE TYPE	DRY DENSITY (pcf)	MOISTURE (%)	BLOWS PER 6 IN.
0	SM		SILTY SAND: light brown, medium dense, moist (Dune Sand)					
1								
2				2.0 - 3.5	▬	93.6	9.9	5 8 12
3				0.0 - 5.0	○			
4								
5			loose	5.0 - 6.5	●	- -	- -	3 4 4
6								
7								
8			medium dense	8.5 - 10.0	●	- -	- -	2 5 8
9								
10			End of Boring @ 10.0' No subsurface water encountered					
11								
12								
13								
14								
15								
16								
17								
18								
19								
20								
21								
22								
23								
24								
25								
26								

LEGEND: ▬ Ring Sample ○ Grab Sample ▭ Shelby Tube Sample ● SPT

NOTE: This log of subsurface conditions is a simplification of actual conditions encountered. It applies at the location and time of drilling. Subsurface conditions may differ at other locations and times.

APPENDIX B

Laboratory Test Results

Monterey Regional Airport
Northeast Vehicle Service Road Improvements

303018-004

BULK DENSITY TEST RESULTS

ASTM D 2937-17 (modified for ring liners)

April 24, 2020

BORING NO.	DEPTH feet	MOISTURE CONTENT, %	WET DENSITY, pcf	DRY DENSITY, pcf
1	3.0 - 3.5	11.1	113.7	102.4
2	3.0 - 3.5	8.4	105.3	97.1
2	9.5 - 10.0	7.1	---	---
3	3.0 - 3.5	4.5	100.1	95.8
3	6.0 - 6.5	4.1	---	---
4	3.0 - 3.5	9.9	102.9	93.6

Monterey Regional Airport
Northeast Vehicle Service Road Improvements

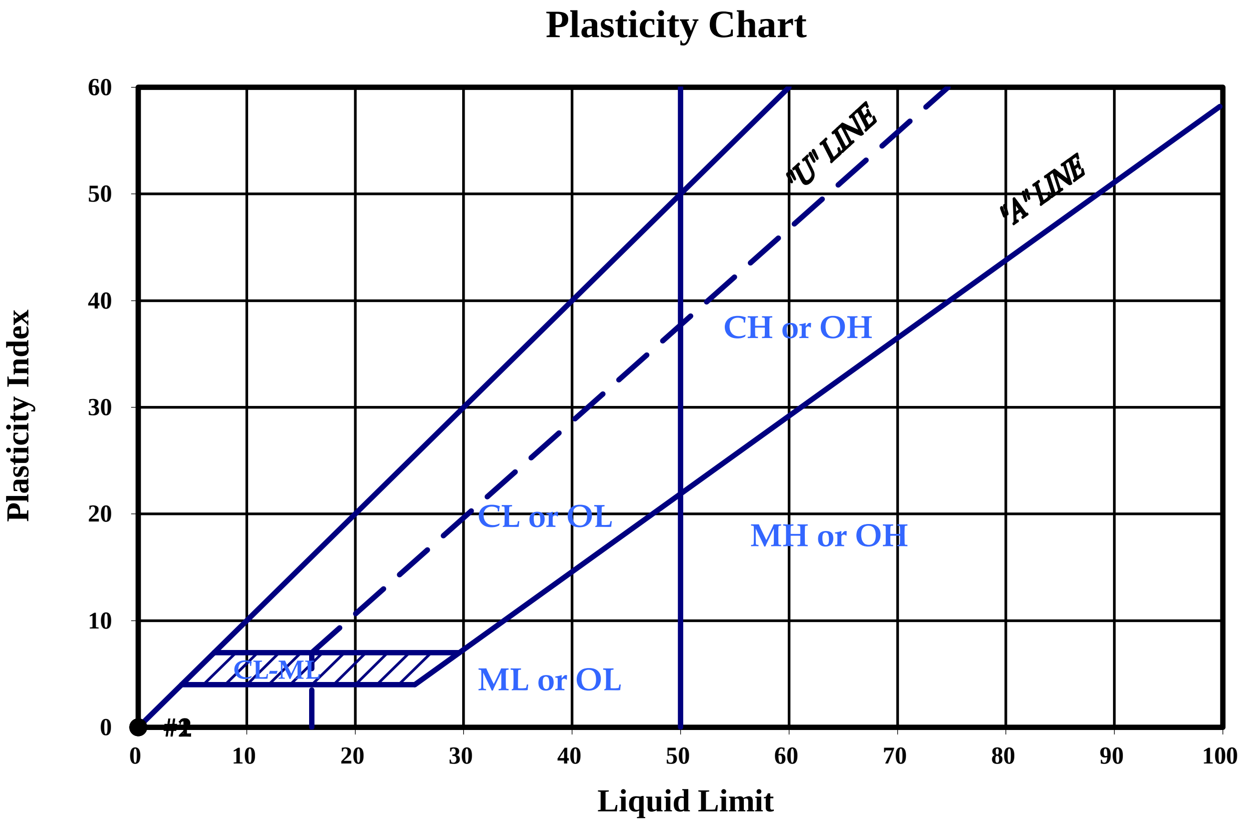
303018-004

PLASTICITY INDEX

ASTM D 4318-17

April 24, 2020

Test No.:	1	2	3	4	5
Boring No.:	2	4			
Sample Depth:	0.0 - 5.0'	0.0 - 5.0'			
Liquid Limit:	NL	NL			
Plastic Limit:	NP	NP			
Plasticity Index:	NP	NP			



Monterey Regional Airport 303018-004
Northeast Vehicle Service Road Improvements

PARTICLE SIZE ANALYSIS

ASTM D 422-63/07

Boring #2 @ 0.0 - 5.0' April 24, 2020

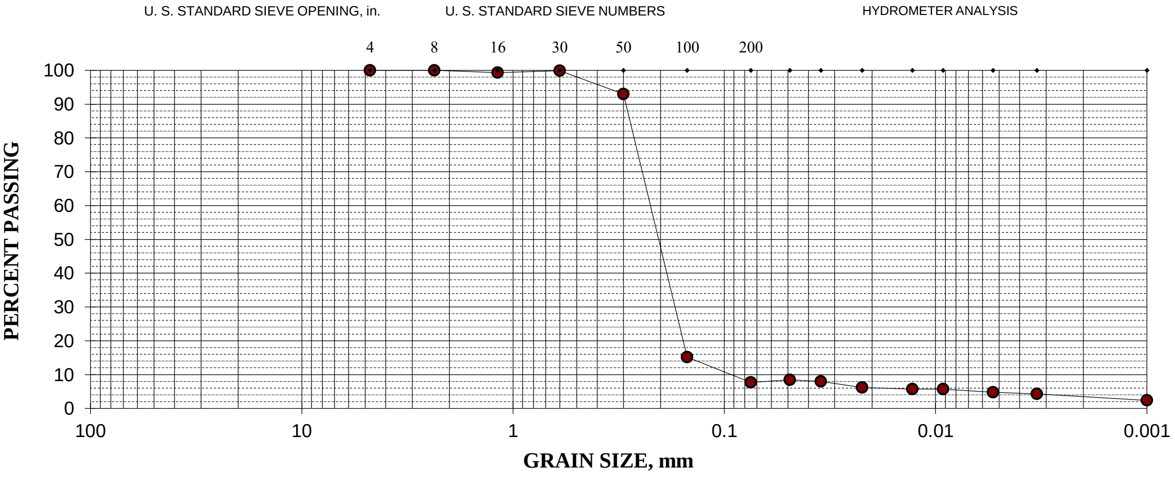
Poorly Graded Sand with Silty Clay (SP-SC) Specific Gravity = 2.65 (assumed)

Gravel = 0%; Sand = 92%; Silt = 3%; Clay = 5%

PI = Nonplastic Cu = 2.4; Cc = 1.4

Sieve size	% Retained	% Passing
#4 (4.75-mm)	0	100
#8 (2.36-mm)	0	100
#16 (1.18-mm)	1	99
#30 (600-µm)	0	100
#50 (300-µm)	7	93
#100 (150-µm)	85	15
#200 (75-µm)	92	8

Hydrometer Analysis	
49-µm	8
35-µm	8
22-µm	6
13-µm	6
9-µm	6
5.3-µm	5
3.3-µm	4
Colloids	2



Monterey Regional Airport
Northeast Vehicle Service Road Improvements

303018-004

MOISTURE-DENSITY COMPACTION TEST

ASTM D 1557-12 (Modified)

PROCEDURE USED: A

April 24, 2020

PREPARATION METHOD: Moist

Boring #2 @ 0.0 - 5.0'

RAMMER TYPE: Mechanical

Light Brown Poorly Graded Sand with Silty Clay (SP-SC)

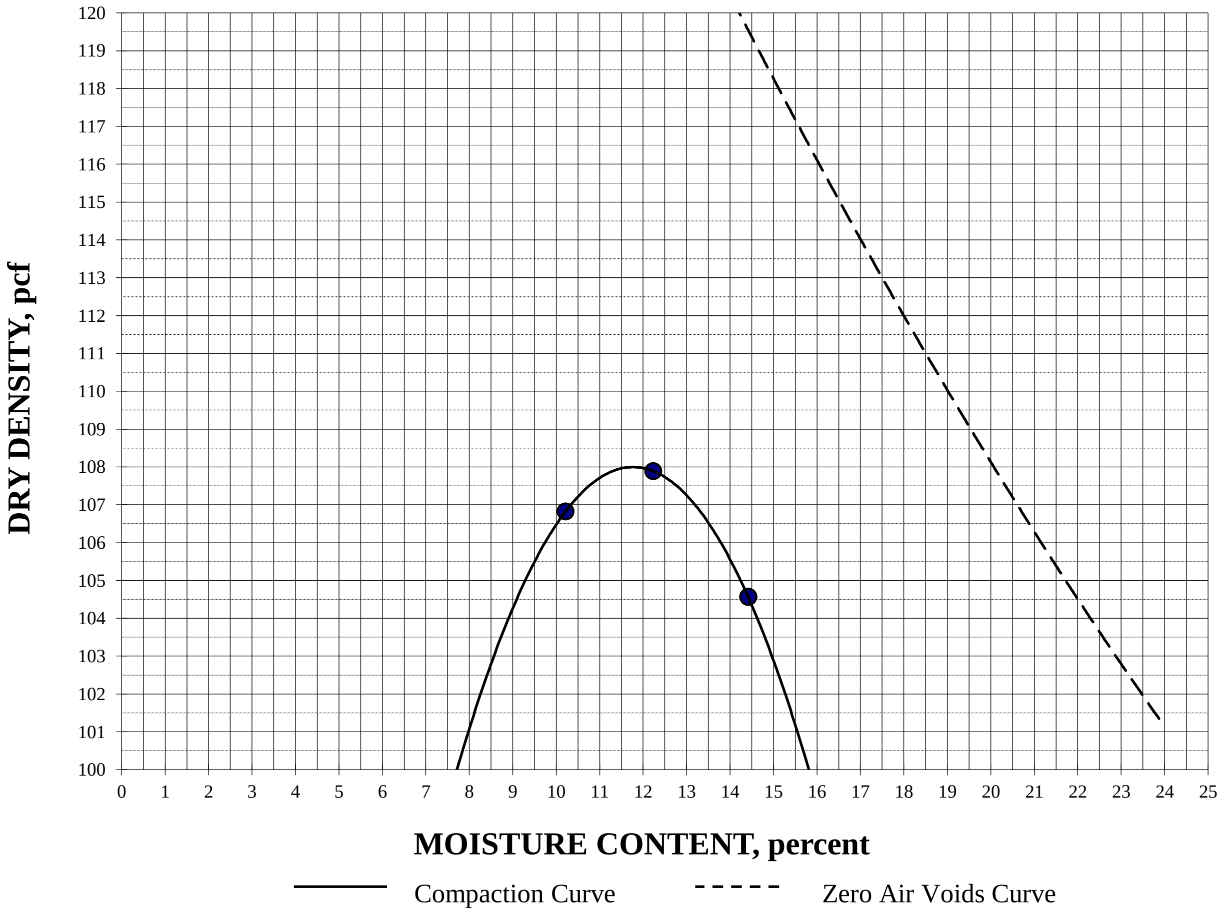
SPECIFIC GRAVITY: 2.65 (assumed)

SIEVE DATA:

Sieve Size	% Retained (Cumulative)
3/4"	0
3/8"	0
#4	0

MAXIMUM DRY DENSITY: 108.0 pcf

OPTIMUM MOISTURE: 11.8%



Monterey Regional Airport
Northeast Vehicle Service Road Improvements

303018-004

RESISTANCE '*R*' VALUE AND EXPANSION PRESSURE

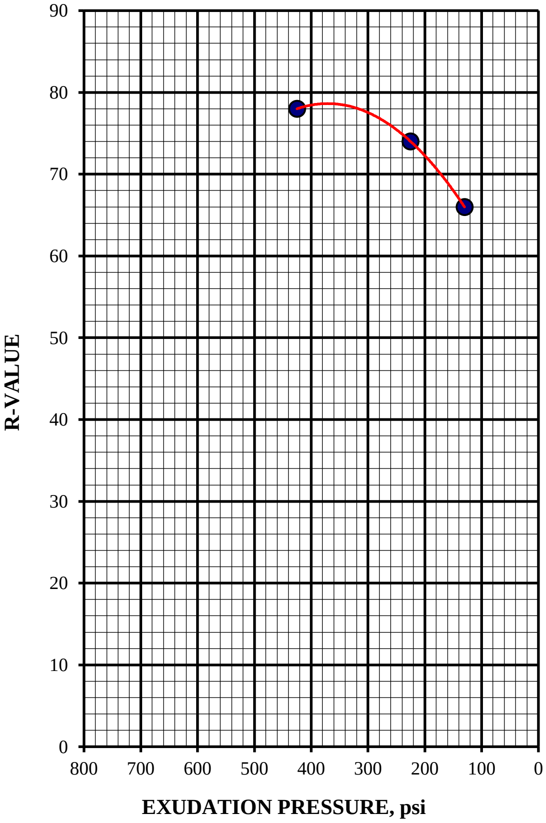
ASTM D 2844/D2844M-13

April 24, 2020

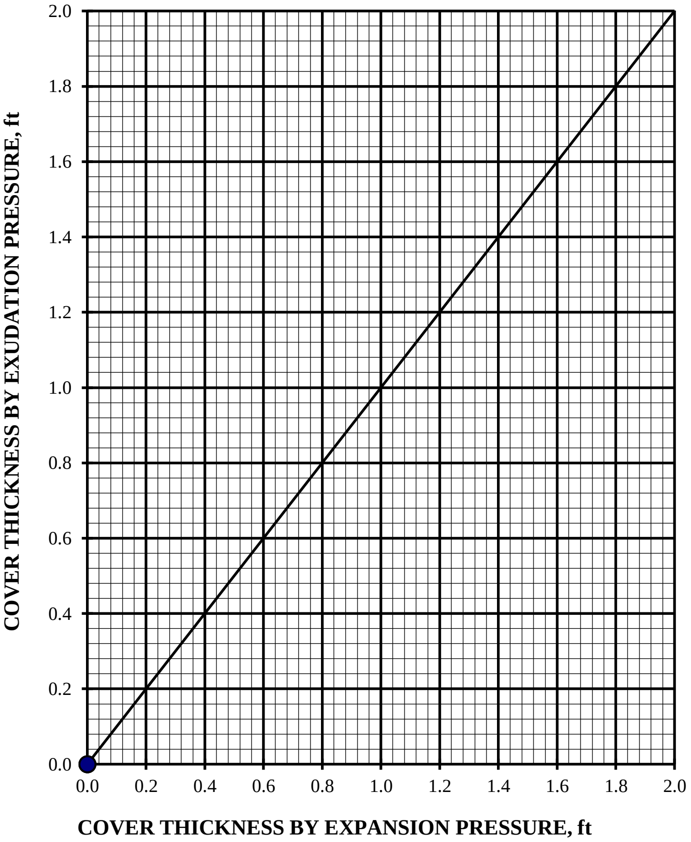
Boring #2 @ 0.0 - 5.0'
Light Brown Poorly Graded Sand with Clay and Gravel (SP-SC)

Dry Density @ 300 psi Exudation Pressure: 115.2-pcf
%Moisture @ 300 psi Exudation Pressure: 13.8%
R-Value - Exudation Pressure: 78
R-Value - Expansion Pressure: N/A
R-Value @ Equilibrium: 78

EXUDATION PRESSURE CHART



EXPANSION PRESSURE CHART



Monterey Regional Airport
Northeast Vehicle Service Road Improvements

303018-004

PARTICLE SIZE ANALYSIS

ASTM D 422-63/07

Boring #4 @ 0.0 - 5.0'

April 24, 2020

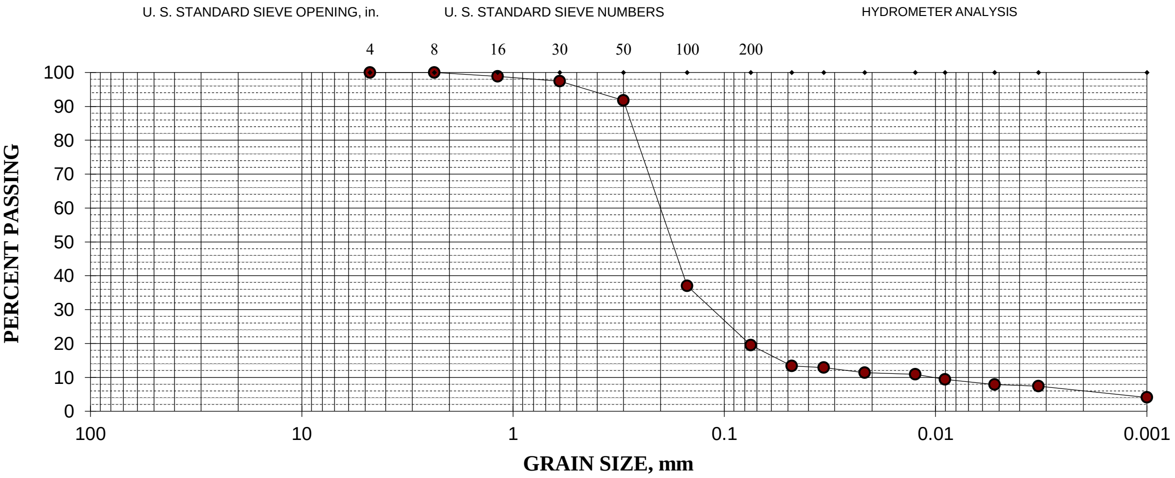
Silty, Clayey Sand (SC-SM)

Specific Gravity = 2.65 (assumed)

Gravel = 0%; Sand = 80%; Silt = 12%; Clay = 8%

PI = Nonplastic

Sieve size	**% Retained**	**% Passing**
#4 (4.75-mm)	0	100
#8 (2.36-mm)	0	100
#16 (1.18-mm)	1	99
#30 (600-µm)	3	97
#50 (300-µm)	8	92
#100 (150-µm)	63	37
#200 (75-µm)	80	20
Hydrometer Analysis		
48-µm		13
34-µm		13
22-µm		11
12-µm		11
9-µm		9
5.3-µm		8
3.3-µm		7
Colloids		4



Monterey Regional Airport
Northeast Vehicle Service Road Improvements

303018-004

MOISTURE-DENSITY COMPACTION TEST

ASTM D 1557-12 (Modified)

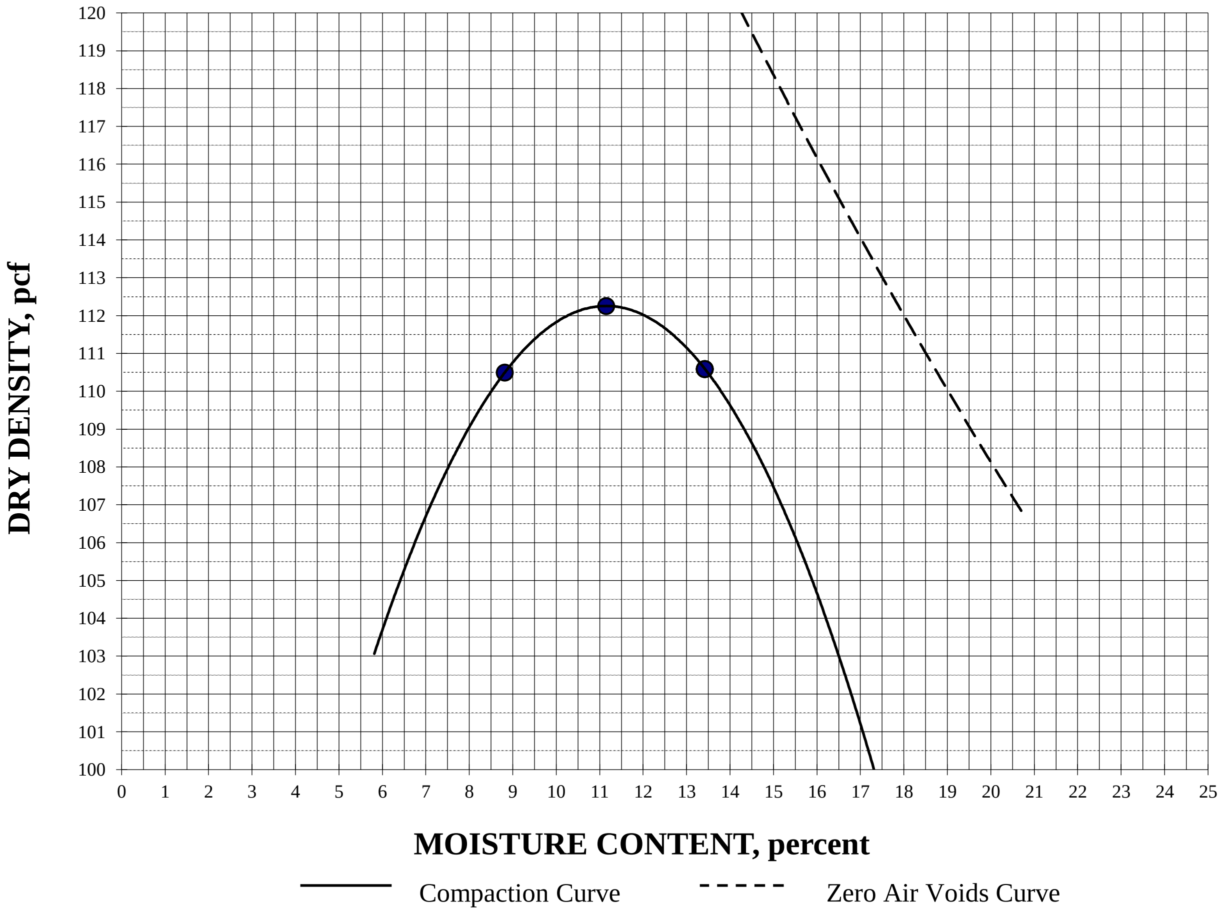
PROCEDURE USED: A
PREPARATION METHOD: Moist
RAMMER TYPE: Mechanical
SPECIFIC GRAVITY: 2.65 (assumed)

April 24, 2020
Boring #4 @ 0.0 - 5.0'
Light Brown Silty, Clayey Sand (SC)

SIEVE DATA:

Sieve Size	% Retained (Cumulative)
3/4"	0
3/8"	0
#4	0

MAXIMUM DRY DENSITY: 112.3 pcf
OPTIMUM MOISTURE: 11.1%



Monterey Regional Airport
Northeast Vehicle Service Road Improvements

303018-004

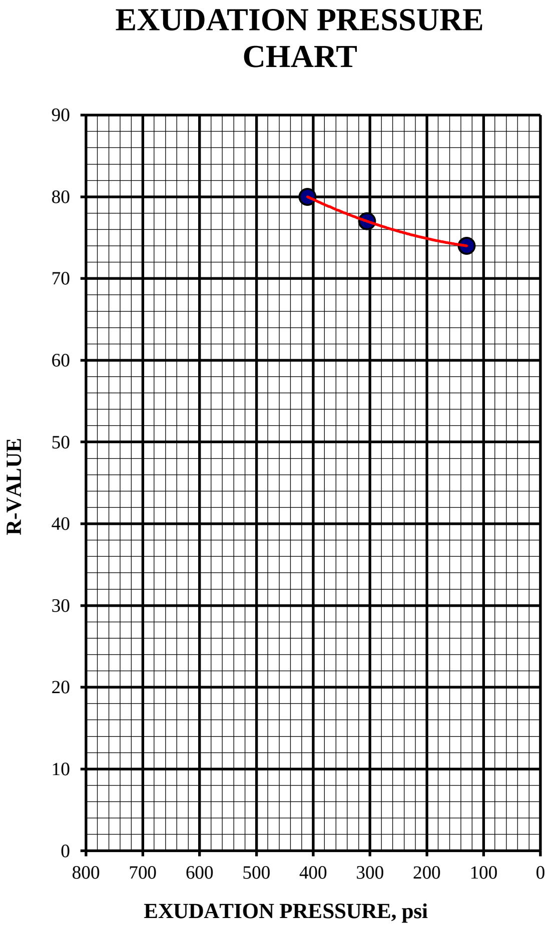
RESISTANCE '*R* ' VALUE AND EXPANSION PRESSURE

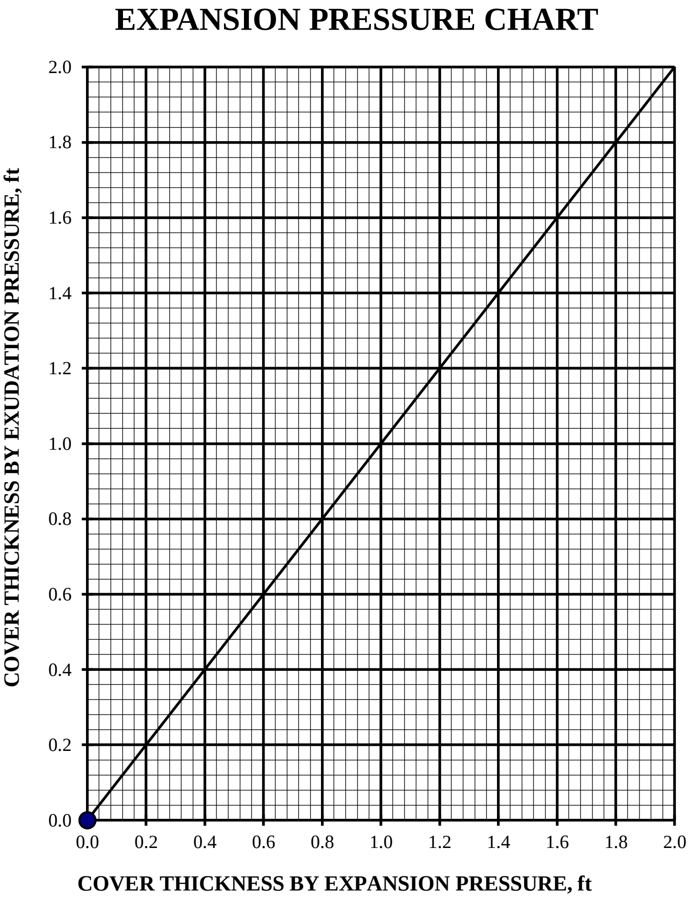
ASTM D 2844/D2844M-13

April 24, 2020

Boring #4 @ 0.0 - 5.0'
Light Brown Silty, Clayey Sand (SC)

Dry Density @ 300 psi Exudation Pressure: 110.5-pcf
%Moisture @ 300 psi Exudation Pressure: 15.6%
R-Value - Exudation Pressure: 77
R-Value - Expansion Pressure: N/A
R-Value @ Equilibrium: 77





CORNERSTONE
EARTH GROUP

TYPE OF SERVICES Preliminary Geotechnical Investigation

PROJECT NAME Monterey Peninsula Airport Terminal Building, Parking Structure, Apron Area, and North Side Improvement Areas

LOCATION Salinas Highway (Highway 68) and
Olmstead Road
Monterey, California

CLIENT Kimley-Horn and Associates, Inc.

PROJECT NUMBER 234-2-7

DATE March 29, 2017

CORNERSTONE
EARTH GROUP

Type of Services	**Preliminary Geotechnical Investigation**
Project Name	**Monterey Peninsula Airport Terminal Building, Parking Structure, Apron Area, and North Side Improvement Areas**
Location	**Salinas Highway (Highway 68) and Olmstead Road** **Monterey, California**
Client	**Kimley-Horn and Associates, Inc.**
Client Address	**765 The City Drive, Suite 200** **Orange, California**
Project Number	**234-2-7**
Date	**March 29, 2017**

Prepared by

Matthew J. Schaffer, P.E.
Project Engineer
Geotechnical Project Manager

MATTHEW J. SCHAFFER
No. 81968
3/31/20
CIVIL

Scott E. Fitinghoff, P.E., G.E.
Principal Engineer
Quality Assurance Reviewer

SCOTT E. FITINGHOFF
NO. 2379
6/30/20
GEOTECHNICAL

TABLE OF CONTENTS

Type of Services	**Preliminary Geotechnical Investigation**
Project Name	**Monterey Peninsula Airport Terminal Building, Parking Structure, Apron Area, and North Side Improvement Areas**
Location	**Salinas Highway (Highway 68) and Olmstead Road Monterey, California**

SECTION 1: INTRODUCTION

This preliminary geotechnical report was prepared for the sole use of Kimley-Horn and Associates, Inc. for the Monterey Peninsula Airport Terminal Building, Parking Structure, Apron Area, and North Side Improvements Areas project in Monterey, California. The location of the site is shown on the Vicinity Map, Figure 1. For our use we were provided the following documents:

- An autocad file showing existing elevations (datum unknown) for the airport,
- A google earth image and a topo map both showing preliminary fill placement and grading limits on the north side of airport and a cross section of the preliminary fill placement and grading.

1.1 PROJECT DESCRIPTION AND REQUIREMENTS

The project site is located north of Salinas Highway (Highway 68) at the Monterey Peninsula Airport in Monterey, California. The project is to include relocating the Monterey Peninsula Airport terminal building and apron area southeast of their present location and reconfiguring Taxiway A as part of the relocation project. The new terminal apron and terminal building will overlay the existing parking, hanger, and ramp areas on the south side of the airport. Due to the existing elevation differences between the parking, hanger, and ramp areas and the existing elevation of the taxiway and runways, we understand 10- to 12-foot cuts with retaining walls will be required to retain existing soils and transition the existing grade down to the existing taxiway and runway levels. The new terminal building will most likely be two levels to accommodate the elevation difference. To the west of the proposed apron area, ancillary 3-level at-grade parking structure and likely smaller structures are planned. Additionally, an access road will be constructed around the perimeter of the apron.

On the north side of the airport, a general aviation apron will be constructed adjacent to the existing apron and Navy Flying Club facilities. Hangers will also be relocated to this new apron area. Construction in this area will include placing 5 to 10 feet of fill to raise the site grades. Additionally, approximately 100,000 cubic yards of excess material generated from the planned cuts from the south side terminal and apron relocation will be placed on the north side of the airport.

Structural loads for the proposed terminal building are not know at this time, however are expected to be typical of this type of building.

1.2 SCOPE OF SERVICES

Our scope of services was presented in our proposal dated November 25, 2016 and consisted of field and laboratory programs to evaluate physical and engineering properties of the subsurface soils, engineering analysis to prepare preliminary recommendations for earthwork, pavements, building foundations, retaining walls, temporary cut slopes and shoring, and fill placement and slope configurations for fill placement on the north side of airport, and preparation of this report. Brief descriptions of our exploration and laboratory programs are presented below.

1.3 EXPLORATION PROGRAM

Field exploration consisted of 19 borings drilled on December 11 to 14, 2016, with truck-mounted, hollow-stem auger drilling equipment. The borings were drilled to depths ranging from 10½ to 40 feet. The borings were backfilled with cement grout in accordance with local requirements; exploration permits were obtained as required by local jurisdictions.

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

1.4 LABORATORY TESTING PROGRAM

In addition to visual classification of samples, the laboratory program focused on obtaining data for preliminary pavement design, retaining wall design, foundation design, and seismic ground deformation estimates. Testing included moisture contents (ASTM D2216), dry densities (ASTM D2937), grain size analyses (sieve and hydrometer – ASTM D422), washed sieve analyses (ASTM D1140), Plasticity Index (ASTM D4318), Compaction Tests (ASTM D1557), and California Bearing Ratios (ASTM D1883). Details regarding our laboratory program are included in Appendix B.

1.5 PRELIMINARY CORROSION EVALUATION

Two samples from our borings were tested for saturated resistivity, pH, soluble sulfates, and chlorides. The laboratory data is presented in Appendix C of this report. The development of corrosion protection recommendations and/or designs should be performed by a corrosion engineer.

1.6 ENVIRONMENTAL SERVICES

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

SECTION 2: REGIONAL SETTING

2.1 GEOLOGICAL SETTING

The site is located within the coast range geomorphic province of central California. Throughout the Cenozoic Era, central California has been affected by tectonic forces associated with lateral or transform plate motion between the North American and Pacific crustal plates, producing a complex system of northwest-trending faults - the San Andreas Fault system (Page, 1998). Uplift, erosion and subsequent re-deposition of sedimentary rocks within this province have been driven primarily by the northwest directed, strike-slip movement of the tectonic plates and the associated northeast oriented compressional stress. The northwest-trending coastal mountain ranges are the result of an orogeny (formation of mountains by the process of tectonic uplift) believed to have been occurring since the Pleistocene epoch (approximately 2-3 million years before present).

The portion of the Santa Lucia Mountains where the site exists is within the Salina Block, which is bound by the San Andreas fault on the east, and by the San Gregorio - Palo Colorado fault to the west. The Salina block is composed of an elongate prism of granites and metamorphic rock types. The Salina basement complex is overlain primarily by marine sedimentary rocks of tertiary age and terrestrial rocks of Pliocene to Pleistocene age. The geologic formations in the vicinity of the site consist of granitic basement rocks overlain by a sequence of marine and terrestrial sedimentary rocks that have been deformed into a series of northwest trending folds by regional tectonic forces. A Regional Geologic Map based on Dibblee (2007) is presented as Figure 3.

2.2 REGIONAL SEISMICITY

The San Francisco Bay area region is one of the most seismically active areas in the Country. While seismologists cannot predict earthquake events, geologists from the U.S. Geological Survey have recently updated earlier estimates from their 2014 Uniform California Earthquake Rupture Forecast (Version 3) publication. The estimated probability of one or more magnitude 6.7 earthquakes (the size of the destructive 1994 Northridge earthquake) expected to occur somewhere in the San Francisco Bay Area has been revised (increased) to 72 percent for the period 2014 to 2043 (Aagaard et al., 2016). The faults in the region with the highest estimated probability of generating damaging earthquakes between 2014 and 2043 are the Hayward (33%), Rodgers Creek (33%), Calaveras (26%), and San Andreas Faults (22%). In this 30-year period, the probability of an earthquake of magnitude 6.7 or larger occurring is 22 percent along the San Andreas Fault and 33 percent for the Hayward or Rodgers Creek Faults. As seen with damage in San Francisco and Oakland due to the 1989 Loma Prieta earthquake that was centered about 50 miles south, 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)**
Monterey Bay-Tularcitos (Navy)	1.1	1.8
Rinconada	7.7	12.4
San Gregorio	9.3	15.0

The San Andreas fault is located approximately 40 kilometers to the northeast. The Chupines fault, which is not a State-considered fault, is located about 2.1 miles to the southeast. Fault distances for the Navy and Chupines faults were taken from Dibblee (2007). A regional fault map is presented as Figure 4, illustrating the relative distances of the site to significant fault zones.

SECTION 3: SITE CONDITIONS

3.1 SITE GEOLOGY

Published maps covering the regional geology in the general vicinity of the site include those by Clark et al. (1974, 1997), Greene (1977), and Dibblee (2007). These regional maps are based upon aerial photo interpretation, reconnaissance style mapping and field checking at sparsely distributed locations in the area and do not include site-specific data. Figure 3, Regional Geologic Map is a partial reproduction of the geologic map of Dibblee (2007), the most detailed mapping that has been done in the vicinity.

All the maps reviewed indicate that the site is underlain by Quaternary deposits which in turn overlie bedrock of the Miocene Monterey Formation, which is folded into a series of northwest trending anticinal and synclinal structures. Dibblee (2007) indicates the Airport property is underlain by three Quaternary age formations; “Dissected older alluvium” (“Qoa”) across much of the Airport property, Aromas sand (“Qar”) at the south property line, and “Older stabilized dune and drift sand” (“Qos”) capping the very top of the terrace in the eastern portion of the property.

3.2 SURFACE DESCRIPTION

3.2.1 North Side Improvements

This area of the project is located on the north side of the airport runway and taxiways. The proposed location of the general aviation apron and relocated hangers is adjacent to the Navy Flying Club facilities to the east, a taxiway to the south, and Airport Road beyond to the north. This area is a generally flat graded area sloping down to the west with approximately Elevation 216 feet at the east end and approximately Elevation 202 feet at the west end. Beyond the north side of the generally flat graded area, grades slope gradually down to Airport Road with a steeper approximately 3:1 (H:V) 6-foot drop down to Airport Road at the northwest corner of the graded area. This flat graded area proposed for the general aviation apron is generally covered in weeds and small shrub like plants. Borings EB-4 and EB-5 were performed in this area.

The proposed location for the approximately 100,000 cubic yards of material generated from the planned cuts on the south side of the airport is located to the south of Airport Circle and the residential buildings beyond, to the east of Airport Circle, and to the north and east of N Road and Airport Road beyond to the south. This area slopes up from the residential buildings and Airport Circle to the top of a previously placed berm that generally parallels Airport Circle. The top of berm is approximately 10 to 15 feet higher than Airport Circle and the residential buildings beyond and has gradients up to about 3½:1 to 4:1 (H:V). Grades slope down to the south approximately 7 to 10 feet from the top of berm in most locations with a similar gradient as the north side before sloping downward more gradually to the south and southwest to a flatter graded area that appears to be used as a staging area to the north of N Road. To the south of N Road grades slope up about 20 to 25 feet to Airport Road with gradients up to about 3.5:1 to 3:1 (H:V). Areas other than the staging area and roadways are generally covered with mature trees and shrubs. The top of berm is generally an unimproved pathway with evidence vehicular travel. Borings EB-1 and EB-2 were performed on the unimproved pathway along the berm and Boring EB-3 was performed within the staging area.

3.2.2 New Terminal Building, Parking Structure, and Apron Area

This area of the project is located on the south side of the main airport runway and is bounded by Olmsted Road, Salinas Highway (68), and other office/commercial buildings to the south and southeast. The existing Taxiway A and existing terminal apron are located along the northern extent of this project area running generally west to east. The taxiway and apron slope gradually up to the east with approximate Elevation 178 feet at our most westerly Boring EB-6 and approximate Elevation 213 feet at our most easterly Boring EB-9. Borings EB-6, EB-8, and EB-9 were performed on the existing Taxiway A and Boring EB-7 was performed on the existing terminal apron. The taxiway consisted of asphalt concrete and the apron consisted of Portland cement concrete.

To the south of the existing Taxiway A and terminal apron, the existing terminal building, long term parking lot, and an airport hanger area are located west to east, respectively. The existing terminal building is generally at the elevation of the terminal apron while the long term parking lot and airport hanger area have elevations range up to about 10 feet above the taxiway and apron level along the northern extent of parking lot and hanger area. The parking lot and hanger area gradually slope up to the east and southeast, however grades are more gradual than Taxiway A to the north resulting in the most easterly portion of the hanger area only roughly about 2 feet above Taxiway A grades at the adjacent location. Between the long term parking area and hanger area a pavement ramp inclines up about 8 to 10 feet connecting Taxiway A below with the hanger area and associated areas above. Another ramp is located at the eastern extend of the hanger area, but as mentioned above, only ramps up about 2 feet as Taxiway A is close to the elevation of the hanger area at this extent. Borings EB-12, EB-14, and EB-15 were performed within the long term parking lot and Borings EB-10, EB-11, EB-13, EB-16, EB-17, and EB-19 were performed within the hanger area. The parking lot and hanger area consist of asphalt concrete. An airport fire station is located between the long term parking lot and terminal apron area with the upper level connecting to the parking lot and the lower level at the apron elevation. Landscaping consisting of grass and small shrubs is located in the sloped area between the parking lot and hanger area and the taxiway and apron below. Beyond the anticipated project boundary along the eastern and southeast border of the hanger area, grades slope up to the south and east to office/commercial building developments beyond.

Olmsted Way runs along the south end of the long term parking lot and connects Olmsted Road with the hanger area. An employee parking lot and staging area is located to the south of Olmsted Way and to the west of the southern portion of the hanger area. Grades in this area are generally level. Boring EB-18 was performed within the staging area, which may be the site for a new at-grade parking structure in the future. The parking lot consists of asphalt concrete and the staging area is covered in gravel/aggregate base material.

3.3 SUBSURFACE CONDITIONS

Nineteen borings were drilled as part of our geotechnical investigation for the new terminal building, apron area improvements, and improvements on the north side of the airport. Borings EB-1 to EB-5 were performed on the north side of the airport, Borings EB-6, EB-8, and EB-9 were performed on the existing taxiway, Boring EB-7 was performed on the existing terminal apron, and Borings EB-10 to EB-19 were performed in the existing long-term parking, employee parking, and hanger areas at the south side of airport. Borings EB-6 to EB-17 were located within areas of existing pavements and were cored prior to the borings being performed. A summary of the pavement core thicknesses and underlying aggregate base (AB) thicknesses at these boring locations are shown in Table 2.

Table 2: Summary of Pavement Core Thicknesses

Boring Number	Total Pavement Thickness(1)	AB Thickness(1)
EB-6	9¾" AC(2)	5" AB
EB-7	14½" PCC	4" AB
EB-8	12⅛" AC(2)	5" AB
EB-9	11¼" AC(2)	8" AB
EB-10	1½" AC	5" AB
EB-11	1½" AC	4" AB
EB-12	4¾" AC(2)	3½" AB
EB-13	2¾" AC	4" AB
EB-14	4½" AC(2)	No AB
EB-15	2½" AC	No AB
EB-16	2" AC	4" AB
EB-17	2¾" AC	5" AB

Note: [1] AC = Asphalt Concrete or Hot Mixed Asphalt (HMA), [1]AB = Aggregate Base, [1] (All pavement section thicknesses were measured by hand in the field and may vary).

[2] Pavement thickness included an overlay separated by pavement fabric. Overlay thickness was approximately 2 inches for EB-6, EB-8, and EB-12. Overlay thickness was about 2⅜ inches for EB-9 and 2¼ inches for EB-14.

3.3.1 Proposed Stockpile Soil Area – North Side of Airport

Borings EB-1 to EB-3 were performed in the area proposed for the approximately 100,000 cubic yards of cut material from the south side improvements. Borings EB-1 and EB-2 were performed along the top of the existing berm in the area. Below the ground surface, these borings encountered 11 to 12 feet of undocumented fill consisting of medium dense to very dense silty, clayey, and poorly graded sand with silt. Beneath the fill, native loose to dense silty, clayey, and poorly graded sands with variable amounts of silt were encountered to the maximum depths explored of 35 feet. The upper portion of these native soils was generally found to be loose to medium dense. Boring EB-3 was performed within the graded staging area. Beneath the ground surface, native loose poorly graded sand with silt was encountered to a depth of about 5 feet followed by medium dense poorly graded sand with silt and very dense silty sand to the maximum depth explored of about 14 feet.

3.3.2 General Aviation Apron Relocated Hangers Area – North Side of Airport

Boring EB-4 and EB-5 were performed in the area proposed for the general aviation apron and relocated hangers adjacent to the Navy Flying Club facilities. Beneath the ground surface, these borings encountered native medium dense to dense poorly graded sand with variable amounts of silt to the maximum depth explored of 15 feet.

3.3.3 Existing Taxiway A and Terminal Apron Area

Borings EB-6 to EB-9 were performed in the existing Taxiway A and terminal apron area. Beneath the pavement section (AC and AB) for Boring EB-6, undocumented fill consisting of dense silty, clayey sand and medium dense poorly graded sand with silt and gravel were encountered to a depth of about 5 feet below the surface. Beneath the fill in Boring EB-6 and the pavement section (AC or PCC and AB) in Borings EB-7 to EB-9, medium dense to very dense silty sand and poorly graded sand was generally encountered to a depth of 8 to 11½ feet. Medium dense to dense clayey sand was encountered in Boring EB-7 and EB-9 starting at a depth of 8 and 9 feet, respectively. The terminal depth of these boring was 10½ to 11½ feet.

3.3.4 New Terminal Building and Terminal Apron Area

Borings EB-10 to EB-18 were performed in areas anticipated to be cut up to about 10 to 12 feet to bring grades down to the new terminal building lower level and terminal apron levels, corresponding to the level of the existing Taxiway A and terminal apron. Beneath the pavement sections (AC and AB), undocumented fill consisting of very dense clayey sand with gravel and loose to very dense poorly graded sand with silt was encountered in Borings EB-10 to EB-12 to depths of about 2½ to 3¼ feet beneath the surface. Beneath the undocumented fill in Borings EB-10 to EB-12, pavement sections in Borings EB-13 to EB-17, and a surficial aggregate base section in EB-18, generally sands with variable amounts of silt and clay were encountered to the maximum depths explored of 40 feet below the surface. These sands were generally loose to medium dense to a depth of about 7½ to 12 feet in Borings EB-10, EB-12 to EB-15, and EB-18. The sands were generally dense to very dense in the other borings and beneath the loose to medium dense sands in Borings EB-10, EB-12 to EB-15, and EB-18. A minor layer of very stiff sandy silty clay about 1 to 2½ feet thick was encountered at a depth of about 15 and 17½ feet in Boring EB-10 and EB-12, respectively. A layer of very dense to hard lean clay and sandy lean clay about 3 to 6 feet thick was encountered at a depth of about 17½ feet in Boring EB-14 and EB-15. A very stiff fat clay layer was encountered at a depth of 37 feet down to the terminal depth of 40 feet in Boring EB-18.

Boring EB-19 was performed in the location of an anticipated ramp down to the new terminal building lower level and apron level. Beneath the ground surface medium dense poorly graded sand with silt was encountered to about 2½ feet followed by generally dense to very dense sands with variable amounts of silt and clay to the terminal depth of 20 feet. A thin ½-foot thick layer of very stiff lean clay was encountered at 5½ feet beneath the surface.

Detailed boring logs, as well as a key to the classification of the soil encountered in our investigation can be found in Appendix A of this report.

3.3.5 Plasticity/Expansion Potential

We performed seven Plasticity Index (PI) determination tests on representative subsurface samples at depths ranging from 1½ to 19½ feet below grade. The test results were used to evaluate plasticity and expansion potential of the proposed new terminal apron and taxiway bearing soils and to evaluate the plasticity and expansion potential of soils beneath the proposed terminal building. The PI test results ranged from non-plastic to 21, indicating none to moderate expansion potential. Tests from the soils beneath the new terminal apron and taxiway ranged from non-plastic to 14, indicating none to low expansion potential. A Plasticity Index Testing Summary is presented as Figure B1 in Appendix B.

3.4 GROUND WATER

Ground water was encountered in Borings EB-14, EB-17, and EB-18, at depths of 32, 13, and 26½ feet, respectively, below current site grades. Ground water was not encountered in our other borings. All measurements were taken at the time of drilling and may not represent the stabilized levels that can be higher than the initial levels encountered.

Ground water levels are not currently mapped at the site by the State of California. We reviewed the GeoTracker website regarding ground water depths in the site area. Based on our GeoTracker website search, there is no available data within the site area.

Fluctuations in ground water levels occur due to many factors including the following: seasonal water fluctuations, underground drainage patterns causing perched water conditions above steady ground water levels, regionally the average ground water level can fluctuate over long time periods, and other factors.

On a preliminary basis, ground water appears to be below the depth of the new terminal building and apron area (on the order of 3 feet or greater). Depth to ground water should be further evaluated during a design-level investigation to establish a design ground water depth and evaluate any effects, if any, ground water may have on pavements and the terminal building.

3.5 CORROSION SCREENING

We tested two samples for resistivity, pH, soluble sulfates, and chlorides collected from Borings EB-13 and EB-14 at depths of approximately 11½ and 10 feet, respectively. These depths are anticipated to be at the approximate depth of the future terminal apron and lower level of the terminal building. The laboratory test results are summarized in Table 3A and provided in Appendix C. The development of corrosion protection recommendations and/or designs should be performed by a corrosion engineer.

Table 3A: Summary of Corrosion Test Results

Boring/Sample	Depth (feet)	Soil pH[1]	Resistivity[2] (ohm-cm)	Chloride[3,5] (mg/kg)	Sulfate[4,5] (mg/kg)
EB-13/6	11½	6.5	26,138	<2	2
EB-14/5	10	7.0	21,525	<2	25

Notes: [1] ASTM G51
[2] ASTM G57 - 100% saturation
[3] ASTM D4327/Cal 422 Modified
[4] ASTM D4327/Cal 417 Modified
[5] 1 mg/kg = 0.0001 % by dry weight

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

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

In accordance with the 2016 CBC Section 1904.1, alternative cementitious materials shall be determined in accordance with ACI 318-14 Table 19.3.1.1, Table R19.3.1, and Table 19.3.2.1. Based on the laboratory sulfate test results, no cement type restriction is required, although, in our opinion, it is generally a good idea to include some sulfate resistance and to maintain a relatively low water-cement ratio. We have summarized applicable sulfate design values and parameters from ACI 318-14, Chapter 19 below in Table 3B.

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

Table 3B: ACI Sulfate Soil Corrosion Design Values and Parameters

Category	Water-Soluble Sulfate (SO4) in Soil (% by weight)	Sulfate (S) Class	Cementitious Materials (2)
S, Sulfate	< 0.10	S0	no type restriction

Notes: (1) above values and parameters are from on ACI 318-14, Table 19.3.1.1, Table R19.3.1, and Table 19.3.2.1
(2) cementitious materials are in accordance with ASTM C150, ASTM C595, and ASTM C1157

SECTION 4: GEOLOGIC HAZARDS

4.1 FAULT RUPTURE

As discussed above several significant faults are located within 25 kilometers of the site. The site is not located within a State-designated Alquist Priolo Earthquake Fault Zone. As shown in Figure 4, 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 a value equal to F_{PGA} x PGA, as allowed in the 2016 edition of the California Building Code. For our analysis we used a PGA of 0.576g.

4.3 LIQUEFACTION POTENTIAL

The site is not located within an area covered by the current State mapping for Liquefaction Hazard Zones. However, the airport is map as being in areas of low susceptibility and variable (may range from high to low) susceptibility (Rosenberg, L.I.; 2001). We screened the site for liquefaction during our site exploration by retrieving samples from the site, performing visual classification on sampled materials, and performing various tests to further classify the soil properties.

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

Borings EB-12, EB-14, EB-15, and EB-18 extended to a maximum depth of 40 feet below existing site grades and were performed in the anticipated location of the future terminal and parking structure buildings. As previously mentioned, we understand the parking structure will be at-grade while site grades will be lowered about 10 to 12 feet within the terminal building area. Additionally, as discussed, ground water appears to be below the depth of the new terminal building. Below a depth of about 10 feet, the above mentioned borings generally encountered very stiff to hard cohesive clays and medium dense to very dense granular soils. In our opinion, granular soils with SPT “N” values of 30 or greater are too dense to liquefy and have been screened out of our analysis. Based on the above, our preliminary analysis and screening within the anticipated terminal building location indicates the soils could theoretically experience liquefaction triggering that could result in soil softening and post-liquefaction total settlement of up to about ¼ inch based on the Yoshimine et.al (2006) method. The potential for liquefaction settlement in the location of the terminal building should be further evaluated during a design-level geotechnical investigation once the building footprint has been established.

Borings EB-4 and EB-5 were performed in the location of the proposed general aviation apron and relocated hangers and extended to a maximum depth of 15 feet below existing site grades. Ground water was not encountered in these borings. Boring EB-3 was performed to the north of this area and positioned at a ground surface elevation about 23 and 36 feet below Boring EB-5 and EB-4, respectively. Ground water was not encountered to the maximum depth explored in Boring EB-3 of about 14 feet which corresponds to a depth of about 50 feet below Boring EB-4 and 37 feet below Boring EB-5. Therefore, since ground water is anticipated to be at greater depths, in our opinion, the potential for liquefaction settlement in the location of the relocated hangers is 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.

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

4.5 SEISMIC SETTLEMENT/UNSATURATED SAND SHAKING

Loose to medium dense unsaturated sandy soils can settle during strong seismic shaking. We evaluated the potential for seismic compaction of the unsaturated sandy soils in the anticipated location of the terminal building, relocated hangers, and proposed engineered fill slope for the placement of the approximately 100,000 cubic yards of material generated from the planned cuts for the terminal building and apron relocation based on the work by Pradell (1998).

As mentioned, we understand site grades will be lowered about 10 to 12 feet within the terminal building area. Our analyses indicate that the unsaturated sandy soils beneath a depth of about 10 feet could experience up to about ¼ inch of movement after strong seismic shaking. However, if the bottom level of the terminal building is not lowered about 10 to 12 feet as currently anticipated and is constructed near existing grades, our analyses indicate an additional ¼ inch of movement could occur. In the area of the relocated hangers, our analyses indicate that the unsaturated sandy soils could experience up to about ¼ inch of movement after strong seismic shaking. In the location of the proposed engineered fill slope for placement of the cut materials, our analyses indicated that the existing soils could experience 1 to 1½ inches of movement after strong seismic shaking. Once the footprint locations of the terminal building and relocated hangers has been established, the potential for unsaturated sand shaking should be further evaluated during a design-level geotechnical investigation.

SECTION 5: SLOPE STABILITY

5.1 PRELIMINARY SLOPE STABILITY EVALUATION

As discussed, the approximately 100,000 cubic yards of material anticipated to be generated from the planned cuts for the terminal building and apron relocation are to be placed along the north side of the airport. Based on the preliminary grading and fill placement maps provided and our discussions with you, we prepared Geologic Cross Section A-A’, Figure 5. Cross Section A-A’ depicts the existing topography and the proposed 3:1 (horizontal:vertical) engineered fill slope within the area of proposed fill placement. This cross section was evaluated for static and seismic stability.

Existing grades beneath the proposed engineered fill slope range up to about 3.5:1 (horizontal:vertical). Additionally, as mentioned in the subsurface section, the upper portion of the native soils beneath the existing berm fills were found to be generally loose to medium dense. As such, we recommend a keyway be cut below the loose native soils at the toe of proposed engineered fill slope and the existing berm be benched into as engineered fill is placed up to final proposed grades. A keyway and benching has been modeled in Cross Section A-A'. Detailed discussions of our evaluation are presented in the following paragraphs.

The analyses presented in this report should be considered preliminary because the project grading plans are conceptual at this stage. The purpose of the analysis is to demonstrate that the overall grading concept is feasible. Additional slope stability analyses should be performed as part of the design-level geologic and geotechnical investigation for the project.

5.1.1 Method of Analysis

The stability of a slope is influenced by many factors including but not limited to the geologic structure and composition, inclination, and height of a slope, ground water, climatic factors such as rainfall, and irrigation. In geotechnical engineering, “stability” is expressed as a ratio of resisting moments and forces divided by driving moments and forces termed the factor of safety (FS). Factors of safety can be calculated for static and seismic conditions. In performing the slope stability analysis, we followed the guidelines set forth by California Geologic Survey (CGS) in Special Publication 117A (2008).

The stability of the geologic cross section was evaluated using the computer program GSTABL7, and circular modes of failure. Input parameters for the analyses include slope geometry, soil layers or zones, total and saturated unit weights and strength parameters, ground water conditions, and assumed areal surface loading.

In evaluating the stability of slopes under seismic conditions, GSTABL7 uses a "pseudo-static" method of analysis. The pseudo-static method models the effects of transient or pulsating earthquake loading on a potential slide mass by using an “equivalent” static horizontal acceleration acting on the mass of the potential landslide in a limit-equilibrium analysis.

The ground motion parameter used in a pseudo-static analysis is referred to as the seismic coefficient “k”. CGS (2008) has published recommendations for the selection of the “k” value in a publication titled, “Guidelines for Evaluation and Mitigation of Seismic Hazards in California, SP 117A.” The site is located near the active Monterey Bay-Tularcitos, Rinconada and San Gregorio Faults and moderately high ground shaking can be expected during a seismic event near the site. In accordance with the CGS Guidelines, we have performed our pseudo-static analysis using a “k” value of 0.28, which considered an earthquake magnitude of 7.4 for the Monterey Bay-Tularcitos fault. This “k” value also considered a threshold of displacement on the order of 6 inches (15 cm) or less.

Based on our understanding of current standards of practice, the minimum allowable factor of safety with respect to slope stability generally ranges from 1.5 to 2 for static conditions and 1 to 1.2 for seismic conditions. A pseudo-static factor of safety of 1.0 typically implies "movement" of the slope mass and does not necessarily result in complete slope failure. In our opinion, acceptable factors of safety for static and seismic (pseudo-static) conditions may be considered to be 1.5 and 1.0, respectively. For our analyses, we judged that if seismic stability factors of safety met or exceeded 1.0, that yielding or movement will likely be limited to less than a few to several inches. This is consistent with the guidelines set forth in SP 117A (CGS, 2008) and appears reasonable for the intended use and importance of the proposed improvements.

5.1.2 Selection of Preliminary Soil Properties

The engineered fill will be soil generated from keyway and benching excavation and the planned cuts for the terminal building and apron relocation. The strength parameters for the engineered fill was based on laboratory testing of similar materials from our previous “Monterey Peninsula Airport Runway Safety Area Study” performed in 2011 and our engineering judgement.

The strength parameters for the in-place undocumented fills and native sand deposits were determined based on our boring data including “N” values, densities, and moistures, and our engineering judgement. We estimate the in-place sand deposits to have an internal angle of friction (ϕ) of 30 to 36 degrees, and cohesion (c) generally ranging from 0 to 50 pounds per square foot. A summary of the soil parameters used in our analyses are presented in the table below.

Table 4: Summary of Preliminary Soil Strength Properties

Material Description	Total Unit Weight (pcf)	Saturated Unit Weight (pcf)	Cohesion (psf)	Friction Angle (degrees)
Engineered Fill	110	115	0	36
Sand (SM) – loose	81	86	0	30
Sand (SP) – m. dense	100	105	0	33
Undocumented Fill Sand (SC) – m. dense	110	115	50	33
Sand (SP) - dense	100	105	0	36
Undocumented Fill Sand (SM) – v. dense	124	130	0	36

As the slopes are unsaturated, with the ground water table anticipated to be at greater depths not influencing the stability of the proposed fill slope, the effective strength parameters used for the static analyses were also used for the seismic analyses, following discussions in SP 117A (CGS, 2008).

5.1.3 Anticipated Surficial Loads

At this time, we are not aware of any proposed surficial loads on the slope and fill placement area. As such, no surficial loading have been included in our analyses.

5.1.4 Results of Analysis

The output (calculations) from our stability analyses for both static and seismic (pseudo-static) conditions are presented in Appendix D. Our preliminary analyses for the cross section indicate acceptable factors of safety with respect to slope movement under static and seismic loading conditions with a static factor of safety of 3.3 and a seismic factor of safety of 1.4. The final slope fill slope geometry should be evaluated during a design-level geotechnical investigation.

SECTION 6: CONCLUSIONS

6.1 SUMMARY

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

- Fill slope stability
- Potential for static and seismic settlements
- Cohesionless soils
- Ground water
- Differential movement at on-grade to on-structure transitions
- Highly erodible soils
- Terminal apron subgrade soil conditions
- Retaining wall construction difficulties with cohesionless sands

6.1.1 Fill Slope Stability

As discussed, we performed a preliminary slope stability analyses for the proposed placement of fill from the approximately 100,000 cubic yards of material to be cut for the new terminal building and apron. Our analyses indicate acceptable factors of safety with respect to slope movement under static and seismic loading conditions provided the proposed engineered fill slope is maintained at a 3:1 (horizontal:vertical) and that a keyway at the bottom of slope is constructed and the existing berm slope is benched into as discussed in the “Earthwork” section below.

6.1.2 Potential for Static and Seismic Settlements

In addition to static settlement due to building loads, our preliminary analyses indicate areas of the proposed terminal building may experience seismically-induced total and differential settlements from dry sand settlement and from liquefaction settlement and the relocated hanger buildings may experience seismically-induced total and differential settlements from dry sand settlement. Provided the lower level of the terminal building is about 10 to 12 feet below existing site grades, we anticipate total combined seismic settlements from dry sand shaking and liquefaction up to about ½ inch and total static settlement from building loads of about ⅔ inch. Combined differential static and seismic settlements are anticipated on the order of ½ to ⅔ inch between independent foundation elements. As previously discussed, if the lower level of the terminal building is not 10 to 12 feet below existing grades, we anticipate an additional ¼ inch of settlement from dry sand settlement could occur.

For the relocated hanger buildings, we anticipate total seismic settlement from dry sand shaking of up to ¼ inch and total static settlement from building loads of about ½ inch. Combined differential static and seismic settlements are anticipated on the order of ½ inch between independent foundation elements. Seismic and static settlement estimates should be further evaluated during a design-level investigation once building layouts and configurations have been determined.

6.1.3 Cohesionless Soils

The site is generally underlain by cohesionless, sandy soils. Due to the cohesionless sandy soils, excavation sidewalls for foundations, utility trenches, etc. may cave in or accumulate a significant amount of slough. Contractors should plan on forming footings, preparing slab-on-grade subgrade just prior to concrete placement, and other similar construction issues as relates to utility excavations, etc.

6.1.4 Ground Water

As discussed, ground water was encountered in Borings EB-14, EB-17, and EB-18 at depths of 32, 13, and 26 ½ feet, respectively, below existing site grades. Borings EB-14 and EB-18 were performed in the anticipated location of the proposed terminal building, which has an anticipated lower level about 10 to 12 feet below existing site grades. At these depths, ground water is not anticipated to affect foundations for the terminal building, however the depth to ground water should be further evaluated during a design-level investigation to determine any affects ground water may have on the building.

Boring EB-17 was performed toward the eastern side of the proposed terminal apron. Grades are only anticipated to be lowered about 2 to 5 feet in this area, however if subgrade compaction and grading require grades to be lowered closer to the ground water, grading could be impacted. Impacts typically consist of potentially wet and unstable subgrade and difficulty achieving compaction. As discussed ground water levels can fluctuate due to many factors. Once final grading has been established, depth to ground water should be further evaluated during a design-level investigation to determine the potential for encountering ground water during grading and subgrade preparation.

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

Some improvements may transition from on-grade support to overlying the basement (on-structure) for the new terminal building. Where improvements transition from on-grade to the basement, these transition areas typically experience increased differential movement due to a variety of causes, including difficulty in achieving compaction of retaining wall backfill closest to the wall. We recommend consideration be given to including subslabs beneath flatwork or pavers that can cantilever at least 3 feet beyond the wall. If surface improvements are included that are highly sensitive to differential movement, additional measures may be necessary. We also recommend that retaining wall backfill be compacted to 95 percent where surface improvements are planned.

6.1.6 Highly Erodible Soils

The sands encountered in our exploration and generally present at the site consist of fine to medium sands with fine contents generally less than 30 percent. These types of soils are highly subject to erosion from wind and water. We recommend that final slopes in sand be 3:1 (H:V) or flatter to limit erosion. All exposed surfaces should be vegetated or otherwise protected from erosion. Additional recommendations are provided in the “Earthwork” section below.

6.1.7 Terminal Apron Subgrade Soil Conditions

The soils anticipated to be exposed at the terminal apron subgrade level appear to be variable. These soils also have the potential to become unstable during grading and other construction activities. If the upper soils of the terminal apron subgrade are cement treated, a more consistent subgrade surface would be created and the potential for instability will be greatly reduced. Additionally, cement treatment would increase the design CBR for the subgrade. Additional recommendations are provided in the “Earthwork” section below.

6.1.8 Retaining Wall Construction Difficulties with Cohesionless Sands

The sands generally consist of fine to medium sands with silty and clayey fines generally less than 30 percent. These sands will likely not stand vertical when excavated. The contractor who will construct the retaining walls to transitions grades down to the terminal apron and lower level of the terminal building will need to address this issue. Temporary vertical elements for face stability will likely be required. Recommendations addressing this concern are presented in the “Earthwork” and “Retaining Walls” section of this report.

6.2 DESIGN-LEVEL GEOTECHNICAL INVESTIGATION

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

SECTION 7: PRELIMINARY EARTHWORK RECOMMENDATIONS

In general, earthwork for the proposed improvements should be performed in general accordance with guidelines provided in AC 150/5370-10G titled “Standards for Specifying Construction of Airports”, Part II Earthwork (FAA, 2014) and the errata sheet for AC 150/5370-10G (FAA, 2016). The earthwork guidelines provided by the FAA are generic guidelines that should be “tailored” by the design engineer to meet the specific conditions of each project. Our preliminary recommendations for the geotechnical aspects of the project earthwork are presented below and should be incorporated into the project earthwork specifications.

7.1 SITE DEMOLITION, CLEARING AND PREPARATION

7.1.1 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. We anticipate that most of the required stripping will consist of tree and shrub removal, as addressed below. Organic laden stripping should be disposed off-site or stockpiled for re-use in landscape areas or otherwise designated in the project specifications.

7.1.2 Tree and Shrub Removal

Trees and shrubs designated for removal should have the rootballs and any roots greater than 1½-inch diameter removed completely. The depth of the excavations is anticipated to be on the order of 24 to 48 inches. Grade depressions resulting from rootball removal should be cleaned of loose material and backfilled in accordance with the recommendations in the “Compaction” section of this report.

7.1.3 Demolition of Existing Slabs, Foundations and Pavements

All slabs, foundations, and pavements should be completely removed from within planned improvement areas. Any material that cannot be reused as engineered fill should be disposed of as designated in the project specifications or as directed by the Engineer. A discussion of recycling existing improvements is provided later in this report.

7.1.4 Abandonment of Existing Utilities

All utilities not to remain as shown on the project plans and specifications should be completely removed from within planned improvement areas. All associated trench back fills should be removed and replaced as engineered fill with the trench side slopes flattened to at least 1:1 (horizontal to vertical). The contractor should assume that all utilities will be removed from within improvement areas unless provided written confirmation from both the owner's representative and the engineer.

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

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

7.2 REMOVAL OF EXISTING FILLS

As discussed, due to elevation differences, we understand 10-foot cuts are planned in the location of the new terminal building. Therefore, undocumented fills encountered are anticipated to be removed. However, if fills are encountered beneath the new terminal building or beneath the relocated hangers at the general aviation apron on the north side of the airport, the fills should be completely removed from within the building areas and to a lateral distance of at least 5 feet beyond the building footprints or to a lateral distance equal to fill depth below the perimeter footing, whichever is greater. Provided the fills meet the “Material for Fill” requirements below, the fills may be reused when backfilling the excavations. If materials are encountered that do not meet the requirements, such as debris, wood, trash, those materials should screened out of the remaining material and be removed from the site. Backfill of excavations should be placed in lifts and compacted in accordance with the “Compaction” section below.

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

7.3 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. On a preliminary basis, the upper 10 feet may be classified as OSHA Soil Type C materials. A competent person should determine the actual soil classification during construction and be responsible for implementing and maintaining safe excavation slopes and/or shoring at the site during construction. Preliminary recommended soil parameters for temporary shoring are provided in the “Temporary Shoring” section below.

7.4 BELOW-GRADE EXCAVATIONS

As discussed, we understand 10-foot cuts will be required to transition existing grades down to the existing taxiway and runway levels and to the lower level of the terminal building. Below-grade excavations may be constructed with temporary slopes in accordance with the "Temporary Cut and Fill Slopes" section above if space allows. Alternatively, temporary shoring may support the planned cuts up to 15 feet. We have provided preliminary geotechnical parameters for shoring design in the section below. The choice of shoring method should be left to the contractor's judgment based on experience, economic considerations and adjacent improvements such as utilities, pavements, and foundation loads. Temporary shoring should support adjacent improvements without distress and should be the contractor's responsibility. A pre-condition survey including photographs and installation of monitoring points for existing site improvements should be included in the contractor's scope. We should be provided the opportunity to review the geotechnical parameters of the shoring design prior to implementation; the project structural engineer should be consulted regarding support of adjacent structures.

7.4.1 Temporary Shoring

Based on the site conditions encountered during our investigation, on a preliminary basis, the cuts may be supported by soldier beams and tie-backs, braced excavations, soil nailing, or potentially other methods. If soil nailing is desired, the contractor should likely plan on limited sections where excavations may be left open, potentially constructing the nails through temporary sloped cuts, and other similar measures for sandy soil conditions.

Where shoring will extend more than about 10 feet, restrained shoring will most likely be required to limit detrimental lateral deflections and settlement behind the shoring. In addition to soil earth pressures, the shoring system will need to support adjacent loads such as construction vehicles and incidental loading, existing structure foundation loads, and street loading. We recommend that heavy construction loads (cranes, etc.) and material stockpiles be kept at least 15 feet behind the shoring. Where this loading cannot be set back, the shoring will need to be designed to support the loading. The shoring designer should provide for timely and uniform mobilization of soil pressures that will not result in excessive lateral deflections. Preliminary minimum suggested geotechnical parameters for shoring design are provided in the table below.

Table 5: Preliminary Temporary Shoring Design Parameters

Design Parameter	Design Value
Minimum Lateral Wall Surcharge (upper 5 feet)	120 psf
Cantilever Wall – Triangular Earth Pressure	35 pcf
Restrained Wall – Uniform Earth Pressure	25H(1)(2)
Passive Pressure – Starting below the bottom of the adjacent excavation(3)	400 pcf up to 3,000 psf maximum uniform pressure

(1) H equals the height of the excavation; passive pressures are assumed to act over twice the soldier pile diameter

(2) The cantilever and restrained pressures are for drained designs. If undrained shoring is designed, an additional 40 pcf should be added for hydrostatic pressures.

(3) Bottom of adjacent excavation is bottom of mass excavation or bottom of footing excavation, whichever is deeper directly adjacent to the shoring element.

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

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

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

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

Based on our borings and depth to ground water encountered, ground water is anticipated to be below the bottom of anticipated 10-foot cuts and appears as though will not affect shoring design. However, depth to ground water and potential effects on shoring should be further evaluated during a design-level investigation.

The above considerations are for the use of the design team during conceptual planning and preliminary design. Additional subsurface exploration and engineering analysis should be performed during the design-level geotechnical investigation to develop final shoring design parameters, as needed. A California-licensed civil or structural engineer must design and be in responsible charge of the temporary shoring design.

7.5 SUBGRADE PREPARATION

After site clearing and demolition is complete, and prior to backfilling any excavations that extend below the excavation plane for the subgrade resulting from fill removal or demolition, the excavation subgrade and subgrade within areas to receive additional site fills should be scarified to a depth of 6 inches, moisture conditioned, and compacted in accordance with the "Compaction" section below. Further recommendations for preparation of subgrade in pavement areas is discussed in Sections 7.6 and 7.7.

Sandy subgrades that are allowed to dry out after compaction will be subject to disturbance by both foot and vehicle traffic. In pavement areas, we recommend that aggregate base sections be placed immediately after the subgrade is prepared to reduce rework. In building areas, we recommend that subgrade compaction and proof rolling be performed within 24 hours of capillary break layer or slab-on-grade construction to reduce rework.

7.6 SUBGRADE STABILIZATION FOR TERMINAL APRON AND TAXIWAY AREAS USING CEMENT TREATMENT

Native soil and undocumented fill materials, especially soils with high fines contents such as clays, silts, clayey sands and silty sands, can become unstable due to high moisture content, whether from high in-situ moisture contents, winter rains, or overwatering during construction. As the moisture content increases over the laboratory optimum, more likely the materials will be subject to softening and yielding (pumping) from construction loading or become unworkable during placement and compaction. Due to the variable soil conditions potential at the terminal apron subgrade, potential for unstable conditions to arise during over-excavation and subgrade grading, and to increase the design subgrade CBR, we recommend the upper 18 inches of the terminal apron and taxiway subgrade be cement treated. This will also enable a firm and stable working surface for construction activities.

7.7 SUBGRADE STABILIZATION MEASURES FOR THE GENERAL AVIATION AREAS USING CEMENT TREATMENT

As discussed above, native soil and undocumented fill materials, especially soils with high fines contents such as clays, silts, clayey sands and silty sands, can become unstable due to high moisture content, whether from high in-situ moisture contents, winter rains, or overwatering during construction. As the moisture content increases over the laboratory optimum, more likely the materials will be subject to softening and yielding (pumping) from construction loading or become unworkable during placement and compaction. Due to the variable soil conditions potential at the general aviation pavement subgrade areas, potential for unstable conditions to arise during over-excavation and subgrade grading, and to increase the design subgrade CBR, we recommend the upper 12 inches of the general aviation pavement subgrade be cement treated. This will also enable a firm and stable working surface for construction activities.

7.8 SUBGRADE STABILIZATION MEASURES FOR GENERAL NON-TERMINAL APRON AREAS

As discussed in the section above, native soil and undocumented fill materials, especially soils with high fines contents such as clays, silts, clayey sands and silty sands, can become unstable due to high moisture content, whether from high in-situ moisture contents, winter rains, or overwatering during construction. As the moisture content increases over the laboratory optimum, more likely the materials will be subject to softening and yielding (pumping) from construction loading or become unworkable during placement and compaction.

There are several methods to address potentially unstable soil conditions and facilitate fill placement and trench backfill. Some of these 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.

7.8.1 Scarification and Drying

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

7.8.2 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.

7.8.3 Chemical Treatment

Where the unstable area exceeds about 5,000 to 10,000 square feet and/or site winterization is desired, chemical treatment with 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.

7.9 MATERIAL FOR FILL

7.9.1 Re-Use of On-site Soils

On-site soils with an organic content less than 3 percent by weight may be reused as general fill. General fill should not have lumps, clods or cobble pieces larger than 6 inches in diameter; 85 percent of the fill should be smaller than 2½ inches in diameter. Minor amounts of oversized 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. Also note that during summer months and dry months, these soils will need a large amount of water and mixing to raise the moisture content to a compactable range.

7.9.2 Re-Use of On-Site Site Improvements

We anticipate that asphalt concrete (AC) grindings and aggregate base (AB) will be generated during site demolition. If the AC grindings are mixed with the underlying AB to meet project specifications, they may be reused within the new pavement structural sections. AC/AB grindings may not be reused beneath the terminal building areas. Laboratory testing will be required to confirm the grindings meet project specifications.

If the site area allows for on-site pulverization of PCC and provided the PCC is pulverized to meet the “Material for Fill” requirements discussed within this report, it may be used as select fill within the terminal building areas, excluding the capillary break layer, and pavement structural sections baring the pulverized PCC meets project specifications. Laboratory testing will be required to confirm the grindings meet project specifications. PCC grindings also make good winter construction access roads, similar to a cement-treated base (CTB) section.

7.9.3 Potential Import Sources

Imported material should be inorganic with a Plasticity Index (PI) of 15 or less, and not contain recycled asphalt concrete where it will be used within the terminal building or other habitable 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 (P-208 Aggregate Base Course, etc.) listing current laboratory testing data (not older than 1 month from the import date) may be provided for our review without providing a sample. If current data is not available, specification testing will need to be completed prior to approval.

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

7.10 COMPACTION REQUIREMENTS

All fills, and subgrade areas where fill, slabs-on-grade, and pavements are planned, should be placed in loose lifts 8 inches thick or less and compacted in accordance with ASTM D1557 (latest version) requirements as shown in the table below. In general, clayey soils should be compacted with sheepsfoot equipment and sandy/gravelly soils with vibratory equipment; open-graded materials such as Class 2 Permeable Material should be placed in lifts no thicker than 18 inches and 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 the Engineer or a Cornerstone representative) should evaluate the in-situ moisture conditions, as the use of vibratory equipment on soils with high moistures can cause unstable conditions. General recommendations for soil stabilization are provided in the "Subgrade Stabilization Measures" section of this report.

Table 6: Compaction Requirements – Mass Grading and Building Areas

Description	Material Description	Minimum Relative[1] Compaction (percent)	Moisture[2] Content (percent)
General Fill (within upper 5 feet)	On-Site Soils	90	>1
General Fill (below upper 5 feet)	On-Site Soils	95	>1
Retaining Wall Backfill	Without Surface Improvements	90	>1
Retaining Wall Backfill	With Surface Improvements	95	0 to 2
Trench Backfill (upper 6 inches of subgrade)	On-Site Soils	95	0 to 2
Trench Backfill (below upper 6 inches of subgrade)	On-Site Soils	90	>1
Drainage Rock	Caltrans Class 2 Permeable Material	Consolidate In-Place	NA
Pavement Subgrade (upper 6 inches of subgrade)	On-Site Soils	95	0 to 2
Pavement Aggregate Base	Caltrans Class II AB[3]	95	Optimum
Asphalt Concrete	Asphalt Concrete	95 (Marshall)	NA

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

Table 7: Compaction Requirements – Terminal Apron, Taxiway, and General Aviation

Description	Material Description	Minimum Relative[1] Compaction (percent)	Moisture[2] Content (percent)
General Aviation AC Pavement Cement Treated Subgrade[3] (0 to 12 inches)	On-Site Soils	93	0 to 4
Terminal Apron and Taxiway PCC/AC Pavement Cement Treated Subgrade[3] (0 to 18 inches)	On-Site Soils	93	0 to 4
Pavement Aggregate Base	P-209 Crushed AB	100	Optimum
Asphalt Concrete (Base Course)	P-401 AC	As recommended by FAA	NA
Asphalt Concrete (Surface Course)	P-401 AC	As recommended by FAA	NA
Asphalt Concrete	P-403 AC	As recommended by FAA	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 – Cement treatment of the upper 12 to 18 inches of pavement subgrade will be required. Cement treat 12 inches in general aviation area and 18 inches in terminal apron and taxiway areas.

7.9.1 Construction Moisture Conditioning

Based on our previous experience at the Monterey Regional Airport, to achieve compaction of the sandy soils, we anticipate significant amounts of water will be required to moisture condition the soils. The contractor should anticipate needing to add significant moisture and thoroughly mixing the sandy soils to achieve the proper compaction.

7.10 TRENCH BACKFILL

All utility lines should be bedded and shaded to at least 6 inches over the top of the lines with well-graded sand and gravel materials conforming to the pipe manufacturer’s requirements. Open-graded materials should not be used. 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 psi.

As discussed above, the native soils generally consist of sands with little to some silt and clay fines. Trench excavations will likely not stand vertical and will require temporary slopes or shields, if vertical excavations are needed.

7.11 PERMANENT CUT AND FILL SLOPES

All permanent cut slopes in native soil and fill slopes constructed of native sandy soils should have a maximum inclination of 3:1 (horizontal:vertical). Fill slopes should be overbuilt and trimmed back, exposing engineered fill when complete. Refer to the “Erosion Control” section of this report for a discussion regarding protection of slope surfaces.

7.11.1 Keyways and Benches

Fill placed on existing ground inclined at 6:1 or greater should be benched into the existing slope and a keyway constructed at the toe of the fill. Benches should be angled into the slope (minimum 2 percent inclination), be spaced vertically at no greater than 10 feet between benches, be at least 5 feet wide, and extend at least 5 feet laterally into competent materials, whichever is greater. Depending on the soils encountered, the benches may need to be widened beyond the minimum width to extend into competent soils. The keyway should also be angled slightly into the slope (minimum 2 percent inclination), extend at least 5 to 10 feet below grade and at least 2 feet into competent material, whichever is greater, and be at least 25 feet wide. The side slopes of all keyways and benches should be no steeper than 1:1 and may need to be flattened in localized areas if loose unstable sands are encountered.

As previously discussed in the slope stability section of this report, a keyway and benching should be constructed for the proposed fill slope from placement of the cut material for relocation of the terminal building and apron. Typical keyway and benching is depicted in Figure 5 for Cross Section A-A’.

The above recommendations are preliminary and additional analysis and recommendations should be performed and provided during a design-level investigation.

7.12 SITE DRAINAGE

Surface runoff should not be allowed to flow over the top of or pond at the top or toe of engineered slopes or retaining walls. Ponding should also not be allowed on or adjacent to building foundations, slabs-on-grade, pavements, or concrete flatwork. Surface drainage should be directed towards suitable drainage facilities such as lined v-ditches or drain inlets. Hardscape surfaces should slope at least 1 percent towards suitable discharge facilities; landscape areas should slope at least 2 percent towards suitable discharge facilities unless specified otherwise by the project engineer. 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. Lined v-ditches should be included at the top of slopes. All v-ditches and drain inlets should be sized to accommodate the design storm events for the upslope tributary area. Concrete-lined v-ditches should be reinforced as required and have adequate control and construction joints, and should be constructed neat in excavations; backfill around formed ditches should not be allowed.

7.13 PERMANENT EROSION CONTROL MEASURES

At a minimum all slopes should be vegetated by hydroseeding or other landscape ground cover. The establishment of vegetation will help reduce runoff velocities, allow some infiltration and transpiration, trap sediment within runoff, and protect the soil from raindrop impact. In addition, more aggressive erosion control measures will likely be needed to protect slopes for one or more winter seasons while vegetation is establishing. This may consist of straw matting, or erosion control blankets used in combination with planting.

Both construction and post-construction Storm Water Pollution Prevention Plans (SWPPPs) should be prepared for the project-specific requirements.

7.14 LANDSCAPE CONSIDERATIONS

Since the site soils are generally erosive, we recommend that the landscaping plan address permanent erosion control measures. This can typically be achieved by:

- Using drip irrigation in lieu of spray,
- Regulating the amount of water distributed to planter areas by using irrigation timers, and
- Selecting landscaping that requires little or no watering.

We recommend that the landscape architect consider these items when developing landscaping plans.

SECTION 8: PRELIMINARY FOUNDATION RECOMMENDATIONS

8.1 SUMMARY OF RECOMMENDATIONS

On a preliminary basis, in our opinion, the proposed terminal building and relocated hangers may be supported on shallow foundations.

8.2 SEISMIC DESIGN CRITERIA

The 2016 California Building Code (CBC) 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 borings and review of local geology, the site is underlain by quaternary deposits with typical SPT "N" values between 15 and 50 blows per foot. Therefore, we have classified the site as Soil Classification C. The mapped spectral acceleration parameters S_S and S_1 were calculated using the USGS computer program *U.S. Seismic Design Maps*, located at http://earthquake.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 8: CBC Site Categorization and Site Coefficients

Classification/Coefficient	Design Value
Site Class	D
Site Latitude	36.58551°
Site Longitude	-121.8477°
0.2-second Period Mapped Spectral Acceleration[1], S_S	1.488g
1-second Period Mapped Spectral Acceleration[1], S_1	0.543g
Short-Period Site Coefficient – Fa	1.0
Long-Period Site Coefficient – Fv	1.5
0.2-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects - S_{MS}	1.488g
1-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects – S_{M1}	0.814g
0.2-second Period, Design Earthquake Spectral Response Acceleration – S_{DS}	0.992g
1-second Period, Design Earthquake Spectral Response Acceleration – S_{D1}	0.543g
Mapped MCE Geometric Mean Peak Ground Acceleration – PGA_M	0.576g
Site Coefficient Based on PGA and Site Class - F_{PGA}	1.0

[1]For Site Class B, 5 percent damped.

8.3 SHALLOW FOUNDATIONS

8.3.1 Spread Footings

On a preliminary basis, the terminal building and relocated hangers may be supported on spread footings, which should bear on natural, undisturbed soil or engineered fill, be at least 24 inches wide, and extend at least 18 to 24 inches below the lowest adjacent grade. Lowest adjacent grade is defined as the deeper of the following: 1) bottom of the adjacent interior slab-on-grade, or 2) finished exterior grade, excluding landscaping topsoil.

On a preliminary basis, terminal building and parking structure footings constructed to the above dimensions are capable of supporting maximum allowable bearing pressures of 2,500 psf for dead loads, 3,750 psf for combined dead plus live loads, and 5,000 psf for all loads including wind and seismic. On a preliminary basis, the relocated hanger building footings constructed to the above dimensions are capable of supporting maximum allowable bearing pressures of 2,000 psf for dead loads, 3,000 psf for combined dead plus live loads, and 4,000 psf for all loads including wind and seismic. The above pressures are based on factors of safety of 3.0, 2.0, and 1.5 applied to the ultimate bearing pressure for dead, dead plus live, and all loads, respectively. These pressures are net values; the weight of the footing may be neglected for the portion of the footing extending below grade (typically, the full footing depth). Top and bottom mats of reinforcing steel should be included in continuous footings to help span irregularities and differential settlement.

8.3.2 Footing Settlement

Structural loads were not provided to us at the time this report was prepared; therefore, we assumed the typical loading in the following table.

Table 9: Assumed Structural Loading

Foundation Area	Range of Assumed Loads
Terminal Building Interior Isolated Column Footing	200 to 300 kips
Terminal Building Exterior Isolated Column Footing	100 to 150 kips
Parking Structure Interior Isolated Column Footing	300 to 400 kips
Parking Structure Exterior Isolated Column Footing	150 to 200 kips
Relocated Hanger Buildings Isolated Column Footing	100 to 150 kips

On a preliminary basis and based on the above loading and the allowable bearing pressures presented above, we estimate that the total static footing settlement will be on the order of ⅔-inch for the terminal building, with about ⅓-inch of post-construction differential settlement between adjacent foundation elements. As mentioned, we understand site grades will be cut about 10 to 12 feet to the lower level of the terminal building. Based on this understanding, we estimate differential seismic movement from liquefaction and dry sand shaking below the bottom of proposed building will be on the order of ¼- to ⅓-inch between independent foundation elements, resulting in a total estimated differential footing movement of up to about ½- to ⅔-inch between foundation elements.

On a preliminary basis and based on the above loading and the allowable bearing pressures presented above, we estimate that the total static footing settlement will be on the order of 1-inch for the parking structure, with about ½-inch of post-construction differential settlement between adjacent foundation elements. As mentioned, we understand the parking structure will be at-grade. Based on this understanding, we estimate differential seismic movement from liquefaction and dry sand shaking below the bottom of parking structure will be on the order of ½-inch between independent foundation elements, resulting in a total estimated differential footing movement of up to about 1-inch between foundation elements.

On a preliminary basis and based on the above loading and the allowable bearing pressures presented above, we estimate that the total static footing settlement will be on the order of ½-inch for the relocated hangers, with about ¼-inch of post-construction differential settlement between adjacent foundation elements. In addition, we estimate differential seismic movement from dry sand shaking will be on the order of ¼-inch between independent foundation elements, resulting in a total estimated differential footing movement of up to about ½-inch between foundation elements.

As our footing loads were assumed and the depth of footings below existing grades for the terminal building are not know at this time, we recommend footing settlements be further evaluated for all buildings during a design level investigation.

8.3.3 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. On a preliminary basis, an ultimate frictional resistance of 0.4 applied to the footing 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 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.

8.3.4 Spread Footing Construction Considerations

Cohesionless sands will likely be encountered within footing excavations. Excavation walls may not stand vertical and may need to be sloped to a minimum 1:1 inclination where footings are located within sands or Stay-Form or similar be placed within the footing excavations as they are excavated during construction of the foundation elements. Granular material encountered in the footing bottoms will likely be disturbed to a depth of 6 to 8 inches following excavation and will need to be consolidated with vibratory equipment prior to steel placement. Care should be taken to not disturb the compacted granular material during steel placement. Footing excavations should also be kept moist by regular sprinkling with water to prevent desiccation and potential raveling of the granular materials. As an alternative, a rat slab can be placed over the granular material to protect the granular material prior to steel placement.

SECTION 9: PRELIMINARY PAVEMENT RECOMMENDATIONS

9.1 CONCRETE TERMINAL APRON AND GENERAL AVIATION APRON

The pavement design analysis for the new terminal apron and general aviation apron will be performed with the computer program FAARFIELD by Kimley Horn and Associates, Inc. using geotechnical design parameters recommended by Cornerstone Earth Group. Based on the results of our CBR tests and engineering judgement, on a preliminary basis we recommend a design CBR of 10 be used. Based on this information and discussions with Kimley Horn and Associates, Inc., preliminary pavement designs are as follows:

- Terminal Apron (PCC) – 13 inches PCC over 5 inches P-401/P403 over 6 inches P-209 over 18 inches cement treated subgrade (modulus = 30,000)
- Terminal Apron (Bitumnous Concrete (AC)) – 4 inches P-401 over 5 inches P-401/P403 over 6 inches P-209 (6 inch Minimum) over 18 inches cement treated subgrade (modulus = 30,000)
- General Aviation Apron – 4 inches P-403 over 5 inches P-209 over 12 inches cement treated subgrade (modulus = 30,000).

SECTION 10: PRELIMINARY RETAINING WALL RECOMMENDATIONS

As previously discussed, we understand approximately 10-foot cuts with retaining walls are planned to retain existing soils and transition existing grades down to the existing taxiway and runway levels for the proposed terminal apron and terminal building. The following sections provide preliminary retaining wall recommendations for the anticipated 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 10: Recommended Lateral Earth Pressures

Wall Condition	Lateral Earth Pressure*	Additional Surcharge Loads
Unrestrained – Cantilever Wall	35 pcf	⅓ of vertical loads at top of wall
Restrained – Braced Wall	35 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

10.2.1 Restrained Basement Walls

Retaining walls for the lower level of the new terminal building will likely be required. As mentioned, 10-foot cuts are anticipated and as such the lower level new terminal building walls may be 10 to 12 feet in height. The 2016 California Building Code (CBC) states that lateral pressures from earthquakes should be considered in the design of basements and retaining walls. We developed preliminary seismic earth pressures for the proposed terminal building basement (lower level) wall using interim recommendations generally based on refinement of the Mononobe-Okabe method (Lew et al., SEAOC 2010). Because the walls may be 12 feet in height, and peak ground accelerations are greater than 0.40g, we checked the result of the seismic increment when added to the recommended active earth pressure against the recommended fixed wall earth pressures. Basement walls are not free to deflect, and should therefore be designed for static conditions as a restrained wall, which is also a CBC requirement. Based on our preliminary analysis and current recommendations for seismic earth pressures, it appears that active earth pressures plus a seismic increment exceed the restrained (i.e. at-rest), static wall earth pressures. Therefore, we recommend checking the walls for the seismic condition in accordance with the interim recommendations of the above referenced paper and the 2013 CBC.

The CBC prescribes basic load combinations for structures, components and foundations with the intention that their design strength equals or exceeds the effects of the factored loads. With respect to the load from lateral earth pressure and ground water pressure, the CBC prescribes the basic combinations shown in CBC equations 16-2 and 16-7 below.

$1.2(D + F) + 1.6(L + H) + 0.5(L_r \text{ or } S \text{ or } R)$ [Eq. 16-2]

In Eq. 16-2: H - should represent the total static lateral earth pressure, which for the basement wall will be restrained (use 45 pcf + 8H psf)

$0.9(D + F) + 1.0E + 1.6H$ [Eq. 16-7]

In Eq. 16-7: H - Should represent the static “active” earth pressure component under seismic loading conditions (use 45 pcf).

E - Should represent the seismic increment component in Eq. 16-7. On a preliminary basis, a seismic increment component equal to a triangular load with a resultant force of $4.5H^2$, which should be applied one third of the height up from the base of the wall (and which can also be expressed as an equivalent fluid pressure equal to 9 pcf) can be utilized for restrained basement walls. During a design level investigation, the seismic increment component should be further evaluated.

The interim recommendations in the SEAOC paper more appropriately split out "active" earth pressure (and not the restrained ["at-rest"] pressure) from our report and provide the total seismic increment so that different load factors can be applied in accordance with different risk levels.

10.2.2 Unrestrained Site Walls

Unrestrained site retaining walls up to approximately 10 feet in height may be constructed. The 2016 CBC states that lateral pressures from earthquakes should be considered in the design of basements and retaining walls. Because the walls are greater than about 6 feet, and peak ground accelerations greater than 0.40g are expected, we recommend checking the walls for the seismic condition in accordance with the interim recommendations from the SEAOC (2010) paper and the 2016 CBC.

The CBC prescribes basic load combinations for structures, components and foundations with the intention that their design strength equals or exceeds the effects of the factored loads. With respect to the load from lateral earth pressure and ground water pressure, the CBC prescribes the basic combinations shown in CBC equations 16-2 and 16-7 below.
$1.2(D + F) + 1.6(L + H) + 0.5(L_r \text{ or } S \text{ or } R)$ [Eq. 16-2]

In Eq. 16-2: H - should represent the total static lateral earth pressure, which for the site walls will be unrestrained (use 45 pcf)

$0.9(D + F) + 1.0E + 1.6H$ [Eq. 16-7]

In Eq. 16-7: H - Should represent the static “active” earth pressure component under seismic loading conditions (use 45 pcf).

E - Should represent the seismic increment component in Eq. 16-7. On a preliminary basis, a seismic increment component equal to a triangular load with a resultant force of $12.5H^2$, which should be applied one third of the height up from the base of the wall (and which can also be expressed as an equivalent fluid pressure equal to 25 pcf) can be utilized for unrestrained site walls. During a design level investigation, the seismic increment component should be further evaluated.

The interim recommendations in the SEAOC paper more appropriately split out "active" earth pressure from the seismic earth pressure increment so that different load factors can be applied in accordance with different risk levels.

10.3 WALL DRAINAGE

10.3.1 At-Grade Site Walls

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

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

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

10.3.2 Below-Grade Walls

Miradrain, AmerDrain or other equivalent drainage matting should be used for wall drainage where below-grade walls are temporarily shored and the shoring will be flush with the back of the permanent walls. The drainage panel should be connected at the base of the wall by a horizontal drainage strip and closed or through-wall system such as the TotalDrain system from AmerDrain.

Sections of horizontal drainage strips should be connected with either the manufacturer's connector pieces or by pulling back the filter fabric, overlapping the panel dimples, and replacing the filter fabric over the connection. At corners, a corner guard, corner connection insert, or a section of crushed rock covered with filter fabric must be used to maintain the drainage path.

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

10.4 BACKFILL

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

Consideration should be given to the transitions from on-grade to on-structure. Subslabs or other methods for reducing differential movement of flatwork or pavements across this transition should be included in the project design.

10.5 FOUNDATIONS

On a preliminary basis, site retaining walls and basement retaining walls for the lower level of the terminal building may be supported on a continuous spread footings designed in accordance with the preliminary recommendations presented in the “Preliminary Foundations” section of this report.

SECTION 11: LIMITATIONS

This report, an instrument of professional service, has been prepared for the sole use of Kimley-Horn and Associates, Inc. specifically to support the design of the Monterey Peninsula Airport Terminal Building, Apron Area, and North Side Improvements Areas project in Monterey, 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.

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

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

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

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

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

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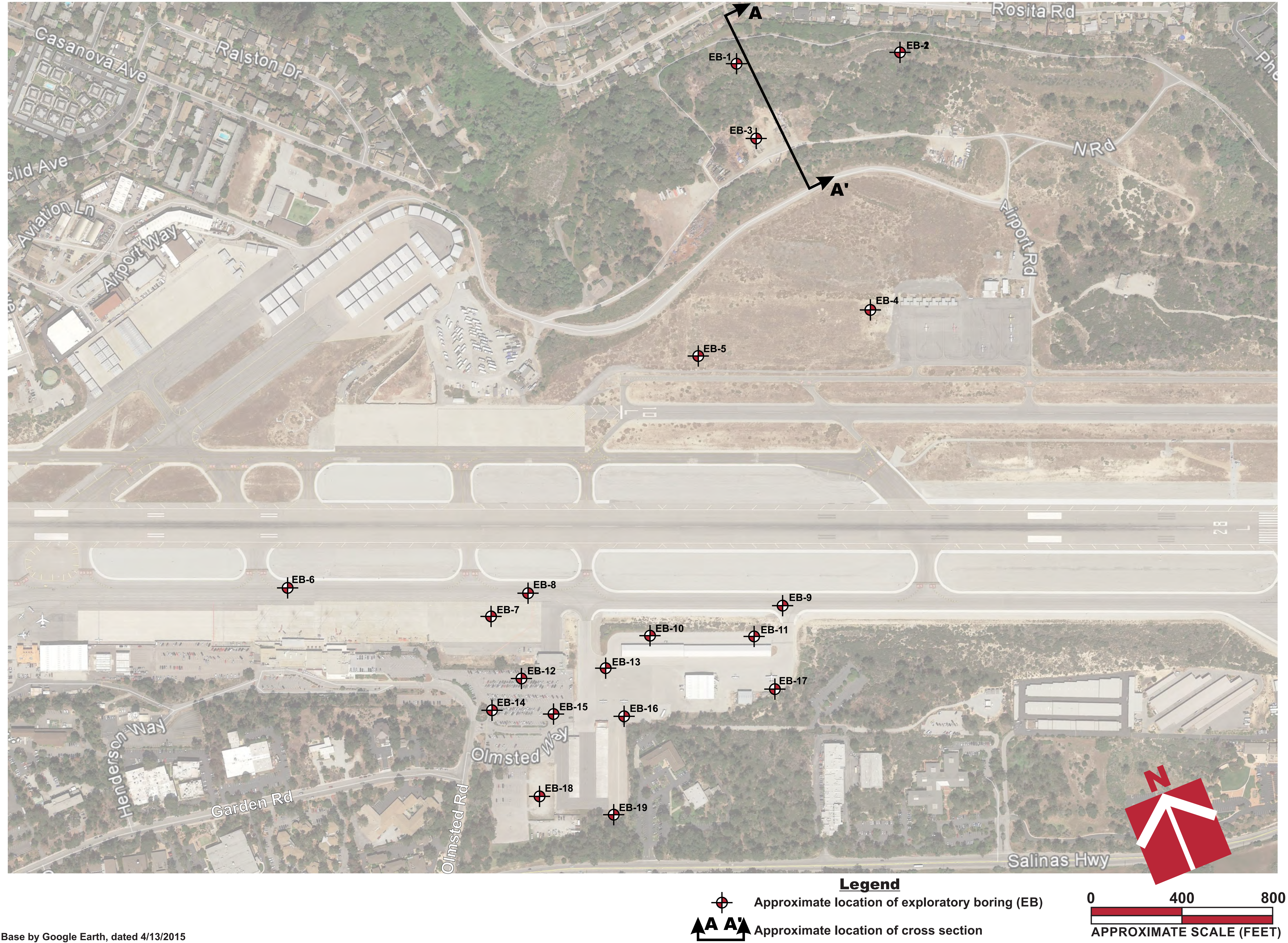
Vicinity Map

Monterey Peninsula Airport Terminal Building, Apron Area, and North Side Improvements Areas Monterey, CA

Project Number	234-2-7
Figure Number	Figure 1
Date	January 2017
Drawn By	RRN



Legend

⌖ Approximate location of exploratory boring (EB)

A A' Approximate location of cross section

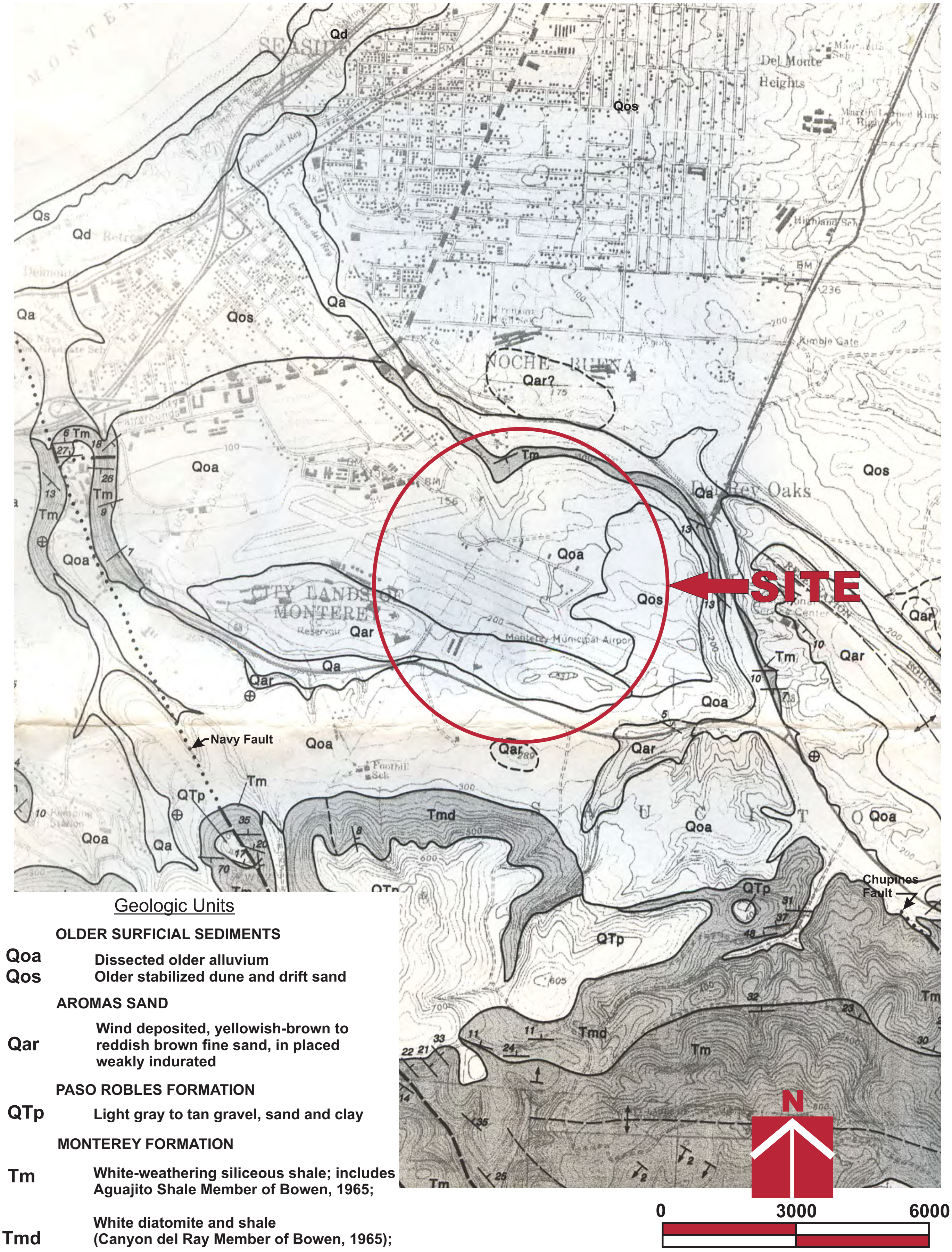
Base by Google Earth, dated 4/13/2015

Project Number	234-2-7
Figure Number	Figure 2
Date	January 2017
Drawn By	RRN

Site Plan

Monterey Peninsula Airport Terminal Building, Apron Area, and North Side Improvements Areas Monterey, CA

CORNERSTONE EARTH GROUP



Geologic Units

OLDER SURFICIAL SEDIMENTS

Qoa	Dissected older alluvium
Qos	Older stabilized dune and drift sand

AROMAS SAND

Qar	Wind deposited, yellowish-brown to reddish brown fine sand, in placed weakly indurated

PASO ROBLES FORMATION

QTp	Light gray to tan gravel, sand and clay

MONTEREY FORMATION

Tm	White-weathering siliceous shale; includes Aguajito Shale Member of Bowen, 1965;
Tmd	White diatomite and shale (Canyon del Ray Member of Bowen, 1965);

Base by Dibblee (2007)

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Regional Geologic Map

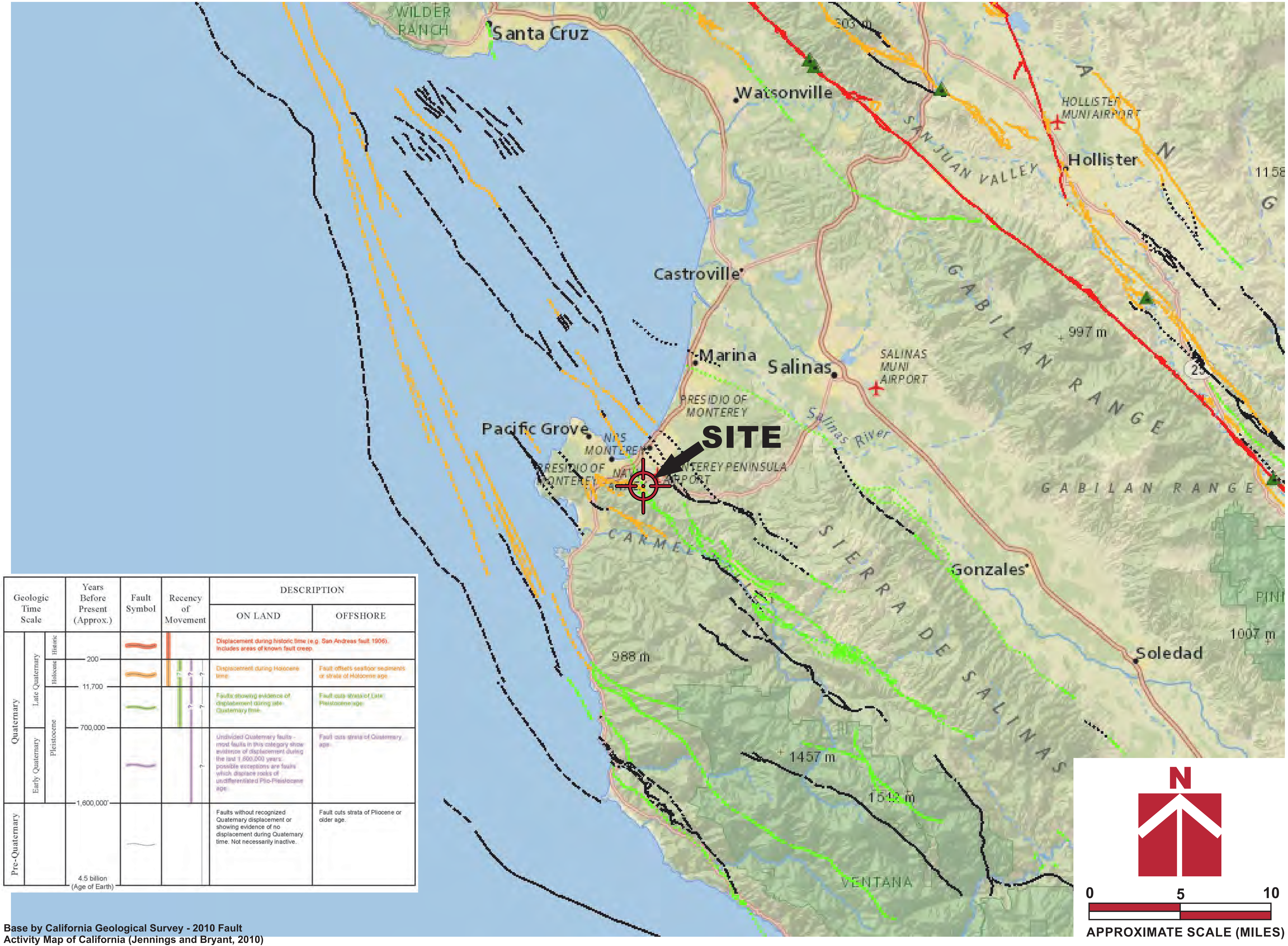
Monterey Peninsula Airport Terminal Building, Apron Area, and North Side Improvements Areas Monterey, CA

Project Number 234-2-7

Figure Number Figure 3

Date January 2017

Drawn By RRN



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

Project Number
234-2-7

Figure Number
Figure 4

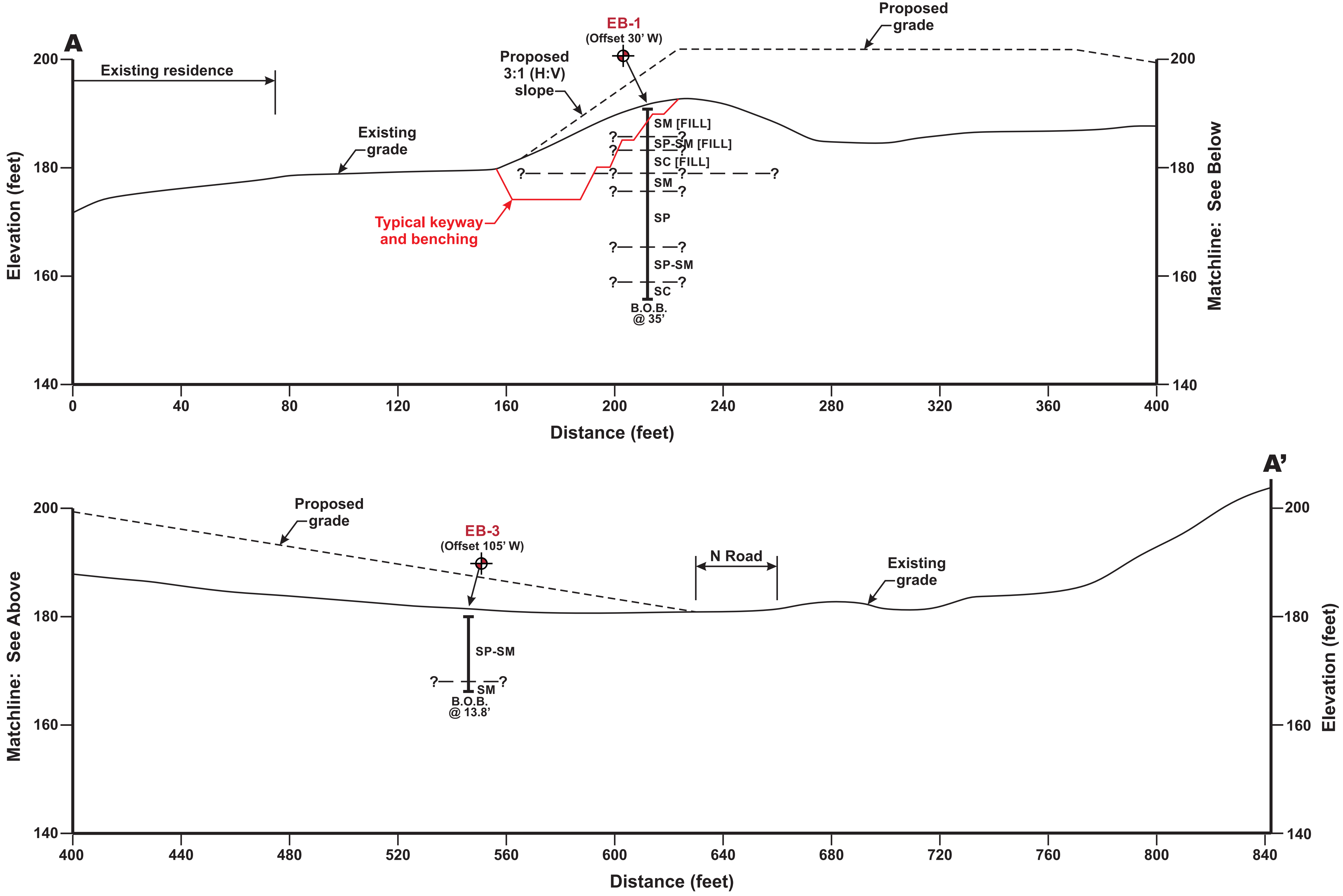
Date
January 2017

Drawn By
RRN

Regional Fault Map

Monterey Peninsula Airport
Terminal Building, Apron Area, and North Side Improvements Areas
Monterey, CA

CORNERSTONE EARTH GROUP



Section A-A’

(View Looking East)
1”=40’ Horizontal
1"=20' Vertical

Symbols

SC	Clayey Sand
SM	Silty Sand
SP-SM	Poorly Graded Sand with Silt
⌖	Approximate location of exploratory boring (EB)

Notes:
1) Surficial fills associated with existing pavements, landscaping or utilities are not shown.
2) The subsurface profile is conceptual and is based on limited subsurface data obtained from widely spaced borings. Actual subsurface conditions may vary significantly between borings.
3) See Figure 2 for location of cross section.

Geologic Cross Section A-A’

Monterey Peninsula Airport
Terminal Building, Apron Area,
and North Side Improvements Areas
Monterey, CA

Project Number 234-2-7
Figure Number Figure 5
Date January 2017
Drawn By RRN

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APPENDIX A: FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program using concrete core drilling equipment and truck-mounted, hollow-stem auger drilling equipment. Nineteen 8-inch-diameter exploratory borings were drilled on December 11 to December 14, 2016 to depths of 10½ to 40 feet. Twelve pavement cores were drilled on December 11 and 12, 2016 at Boring EB-6 to EB-17 locations. Approximate exploratory boring locations are shown on the Site Plan, Figure 2. The soils encountered were continuously logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D2488). Boring logs, as well as a key to the classification of the soil, are included as part of this appendix.

Boring locations were approximated using existing site boundaries, existing buildings, and other site features as references. Boring elevations were interpolated using the autocad file provided showing existing contours (datum unknown). The locations of the borings 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 appropriate depth on the boring logs.

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

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

UNIFIED SOIL CLASSIFICATION (ASTM D-2487-98)

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

OTHER MATERIAL SYMBOLS

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

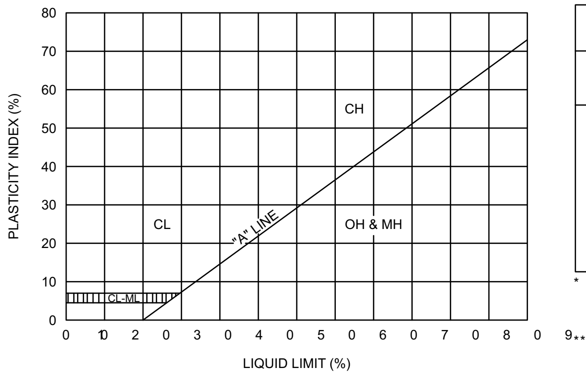
SAMPLER TYPES

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

ADDITIONAL TESTS

CA	CHEMICAL ANALYSIS (CORROSIVITY)	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

PAGE 1 OF 2

CORNERSTONE EARTH GROUP

PROJECT NAME MRY Terminal, Apron and North Side Improvements
PROJECT NUMBER 234-2-7
PROJECT LOCATION Monterey, CA
DATE STARTED 12/14/16 **DATE COMPLETED** 12/14/16
GROUND ELEVATION 191 FT +/- **BORING DEPTH** 35 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 36.591911° **LONGITUDE** -121.840960°
DRILLING METHOD Mobile B-56, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY DL
▽ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

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

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
191.0	0	**Silty Sand (SM) [Fill]** very dense, moist, brown, fine sand, trace fine gravel D1557 Max. Dry Density: 114.8pcf @ 11.5%						
			50 6"	MC-1B	117	6		
			50 5.5"	MC-2B	115	8		
186.0	5	**Poorly Graded Sand with Silt (SP-SM) [Fill]** very dense, moist, light brown, fine sand	50 6"	MC-3	103	9		
183.5		**Clayey Sand (SC) [Fill]** medium dense, moist, brown and gray mottled, fine to medium sand						
			21	SPT				
	10							
179.0		**Silty Sand (SM)** loose, moist, dark brown, fine sand, trace organics	13	MC-5A	73	11		
			17	MC-6B				
176.0	15	**Poorly Graded Sand (SP)** medium dense, moist, light brown, fine sand	14	SPT				
			15	SPT				
			35	MC-9B	98	3		
	20		27	MC				
			25	MC-11B	96	3		
			32	MC-12B	98	5		
	25		21	SPT				
165.5								

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CORNERSTONE
EARTH GROUP

BORING NUMBER EB-1

PAGE 2 OF 2

PROJECT NAME MRY Terminal, Apron and North Side Improvements
PROJECT NUMBER 234-2-7
PROJECT LOCATION Monterey, CA

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UNDRAINED SHEAR STRENGTH, ksf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL
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
165.5			**Poorly Graded Sand with Silt (SP-SM)** dense, moist, brown, fine sand	33	SPT					
				31	SPT-15		8			
	30									
159.0			**Clayey Sand (SC)** dense, moist, light reddish brown with gray mottles, fine to medium sand							
				31	SPT-16		13			
156.0	35		Bottom of Boring at 35.0 feet.							
	40									
	45									
	50									

CORNERSTONE EARTH GROUP

BORING NUMBER EB-2

PAGE 1 OF 2

PROJECT NAME MRY Terminal, Apron and North Side Improvements
PROJECT NUMBER 234-2-7
PROJECT LOCATION Monterey, CA
DATE STARTED 12/14/16 **DATE COMPLETED** 12/14/16
GROUND ELEVATION 208 FT +/- **BORING DEPTH** 30 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 36.59130° **LONGITUDE** -121.83865°
DRILLING METHOD Mobile B-56, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY DL
∇ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

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UNDRAINED SHEAR STRENGTH, ksf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL
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
208.0	0		**Poorly Graded Sand with Silt (SP-SM) [Fill]** dense, moist, brown and light brown mottled, fine sand D1557 Max. Dry Density: 114.8pcf @ 11.5%							
				60	MC-1B	112	5			
				62	MC-2B	112	4			
	5		becomes medium dense	24	MC					
				30	MC-4B	99	3			
				26	MC-5B	95	5			
	10			13	SPT					
197.0			**Poorly Graded Sand (SP)** medium dense to dense, moist, light brown, fine sand	20	SPT					
				31	SPT-8		3			
				33	SPT					
	15									
				20	SPT-10		6			
	20									
187.5			**Poorly Graded Sand with Silt (SP-SM)** dense, moist, light brown, fine sand	32	SPT					
				46	SPT-12		3			
	25									
182.0										

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CORNERSTONE
EARTH GROUP

BORING NUMBER EB-2

PAGE 2 OF 2

PROJECT NAME MRY Terminal, Apron and North Side Improvements

PROJECT NUMBER 234-2-7

PROJECT LOCATION Monterey, CA

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

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 1.0 2.0 3.0 4.0
182.0			**Poorly Graded Sand with Silt (SP-SM)** dense, moist, light brown, fine sand							
180.5			**Silty Sand (SM)** dense, moist, brown, fine sand							
				35	☒ SPT-13		8			
178.0	30		Bottom of Boring at 30.0 feet.							
	35									
	40									
	45									
	50									
	55									

CORNERSTONE
EARTH GROUP

BORING NUMBER EB-3

PAGE 1 OF 1

PROJECT NAME MRY Terminal, Apron and North Side Improvements

PROJECT NUMBER 234-2-7

PROJECT LOCATION Monterey, CA

DATE STARTED 12/14/16 **DATE COMPLETED** 12/14/16

GROUND ELEVATION 180 FT +/- **BORING DEPTH** 13.75 ft.

DRILLING CONTRACTOR Exploration Geoservices, Inc.

LATITUDE 36.591035° **LONGITUDE** -121.841119°

DRILLING METHOD Mobile B-56, 8 inch Hollow-Stem Auger

GROUND WATER LEVELS:

LOGGED BY DL

∇ **AT TIME OF DRILLING** Not Encountered

NOTES

▼ **AT END OF DRILLING** Not Encountered

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ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
180.0	0	**Poorly Graded Sand with Silt (SP-SM)** loose, moist, reddish brown to brown, fine sand	11	MC-1B	99	5			
			13	MC-2B	99	5			
			16	MC					
175.0	5	**Poorly Graded Sand with Silt (SP-SM)** medium dense, moist, light brown, fine sand	26	MC-4B	103	8			
			20	MC-5B	101	8			
	10								
168.0		**Silty Sand (SM)** very dense, moist, brown, fine sand							
166.3		Bottom of Boring at 13.8 feet.	50 4.5"	SPT					
	15								
	20								
	25								

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

1.0 2.0 3.0 4.0

BORING NUMBER EB-4

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CORNERSTONE EARTH GROUP

PROJECT NAME MRY Terminal, Apron and North Side Improvements

PROJECT NUMBER 234-2-7

PROJECT LOCATION Monterey, CA

DATE STARTED 12/12/16 **DATE COMPLETED** 12/12/16

GROUND ELEVATION 216 FT +/- **BORING DEPTH** 15 ft.

DRILLING CONTRACTOR Exploration Geoservices, Inc.

LATITUDE 36.58854° **LONGITUDE** -121.84049°

DRILLING METHOD Mobile B-56, 8 inch Hollow-Stem Auger

GROUND WATER LEVELS:

LOGGED BY DL

∇ **AT TIME OF DRILLING** Not Encountered

NOTES

▼ **AT END OF DRILLING** Not Encountered

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UNDRAINED SHEAR STRENGTH, ksf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL
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
216.0	0	**Poorly Graded Sand with Silt (SP-SM)** dense to medium dense, moist, light reddish brown to brown, fine sand 90% Sand, 5% Silt, 5% Clay D1557 Max. Dry Density: 107.5pcf @ 13.0% NP = Non-plastic	51	MC-1B	103	7	NP	10
			25	MC-2B	101	7		
			20	MC-3B	98	7		
211.0	5	**Poorly Graded Sand (SP)** medium dense, moist, light brown, fine sand	24	MC-4B	81	5		
			27	MC-5B	104	8		
	10							
			39	MC-6B	91	5		
201.0	15	Bottom of Boring at 15.0 feet.						
	20							
	25							

BORING NUMBER EB-5

PAGE 1 OF 1

CORNERSTONE EARTH GROUP

PROJECT NAME MRY Terminal, Apron and North Side Improvements
PROJECT NUMBER 234-2-7
PROJECT LOCATION Monterey, CA
DATE STARTED 12/12/16 **DATE COMPLETED** 12/12/16
GROUND ELEVATION 203 FT +/- **BORING DEPTH** 15 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 36.58888° **LONGITUDE** -121.84323°
DRILLING METHOD Mobile B-56, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY DL
▽ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

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UNDRAINED SHEAR STRENGTH, ksf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL
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
203.0	0	**Poorly Graded Sand with Silt (SP-SM)** dense to very dense, moist, brown, fine sand D1557 Max. Dry Density: 107.5pcf @ 13.0%						
			57	MC-1C	97	3		
			50 6"	MC-2B	102	5		
			59	SPT				
	5		64	MC-4B	103	5		
		becomes medium dense	22	SPT				
	10							
			48	MC-6B	101	7		
188.0	15	Bottom of Boring at 15.0 feet.						
	20							
	25							

BORING NUMBER EB-6

PAGE 1 OF 1

CORNERSTONE EARTH GROUP

PROJECT NAME MRY Terminal, Apron and North Side Improvements
PROJECT NUMBER 234-2-7
PROJECT LOCATION Monterey, CA
DATE STARTED 12/12/16 **DATE COMPLETED** 12/12/16
GROUND ELEVATION 177 FT +/- **BORING DEPTH** 11.5 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 36.588179° **LONGITUDE** -121.850246°
DRILLING METHOD Mobile B-56, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY DL
▽ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

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UNDRAINED SHEAR STRENGTH, ksf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL
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
177.0	0	9¾ inches asphalt concrete over 5 inches aggregate base							
176.2									
175.6		**Silty, Clayey Sand (SC-SM) [Fill]** dense, moist, dark brown to gray, fine to coarse sand Liquid Limit = 28, Plastic Limit = 21							
174.5			70	MC-1B	112	15	7		
		Poorly Graded Sand with Silt and Gravel (SP-SM) [Fill] medium dense, moist, brown, fine sand, fine to coarse subangular gravel	33	MC-2B	112	6			
171.8	5		19	3B MC 3C	107 97	8 10			
		Silty Sand (SM) medium dense, moist, reddish brown to brown, fine sand							
			28	MC-4B	94	5			
	10		16	SPT					
165.5		Bottom of Boring at 11.5 feet.							
	15								
	20								
	25								

CORNERSTONE EARTH GROUP

BORING NUMBER EB-7

PAGE 1 OF 1

PROJECT NAME MRY Terminal, Apron and North Side Improvements
PROJECT NUMBER 234-2-7
PROJECT LOCATION Monterey, CA
DATE STARTED 12/12/16 **DATE COMPLETED** 12/12/16
GROUND ELEVATION 194 FT +/- **BORING DEPTH** 11.5 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 36.586933° **LONGITUDE** -121.847572°
DRILLING METHOD Mobile B-56, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY DL
▽ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

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UNDRAINED SHEAR STRENGTH, ksf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL
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
194.0	0	14½ inches Portland cement concrete over 4 inches aggregate base						
192.6								
192.3		**Silty Sand (SM)** medium dense, moist, brown and light brown mottled, fine sand	19	SPT-1		15		15
		D1557 Max. Dry Density: 125.2pcf @ 10.0% see CBR Results	26	SPT-2	99	17		15
	5	becomes dense NP = Non-plastic 71% Sand, 11% Silt, 18% Clay	36	SPT-3		11	NP	29
			64	MC-4A	117	11		
186.0		**Clayey Sand (SC)** medium dense, moist, gray and reddish brown mottled, fine sand	51	MC-5A	119	11		
	10		18	SPT				
182.5		Bottom of Boring at 11.5 feet.						
	15							
	20							
	25							

BORING NUMBER EB-8

PAGE 1 OF 1

CORNERSTONE
EARTH GROUP

PROJECT NAME MRY Terminal, Apron and North Side Improvements
PROJECT NUMBER 234-2-7
PROJECT LOCATION Monterey, CA
DATE STARTED 12/12/16 **DATE COMPLETED** 12/12/16
GROUND ELEVATION 196 FT +/- **BORING DEPTH** 11.5 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 36.58701° **LONGITUDE** -121.84690°
DRILLING METHOD Mobile B-56, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY DL
▽ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

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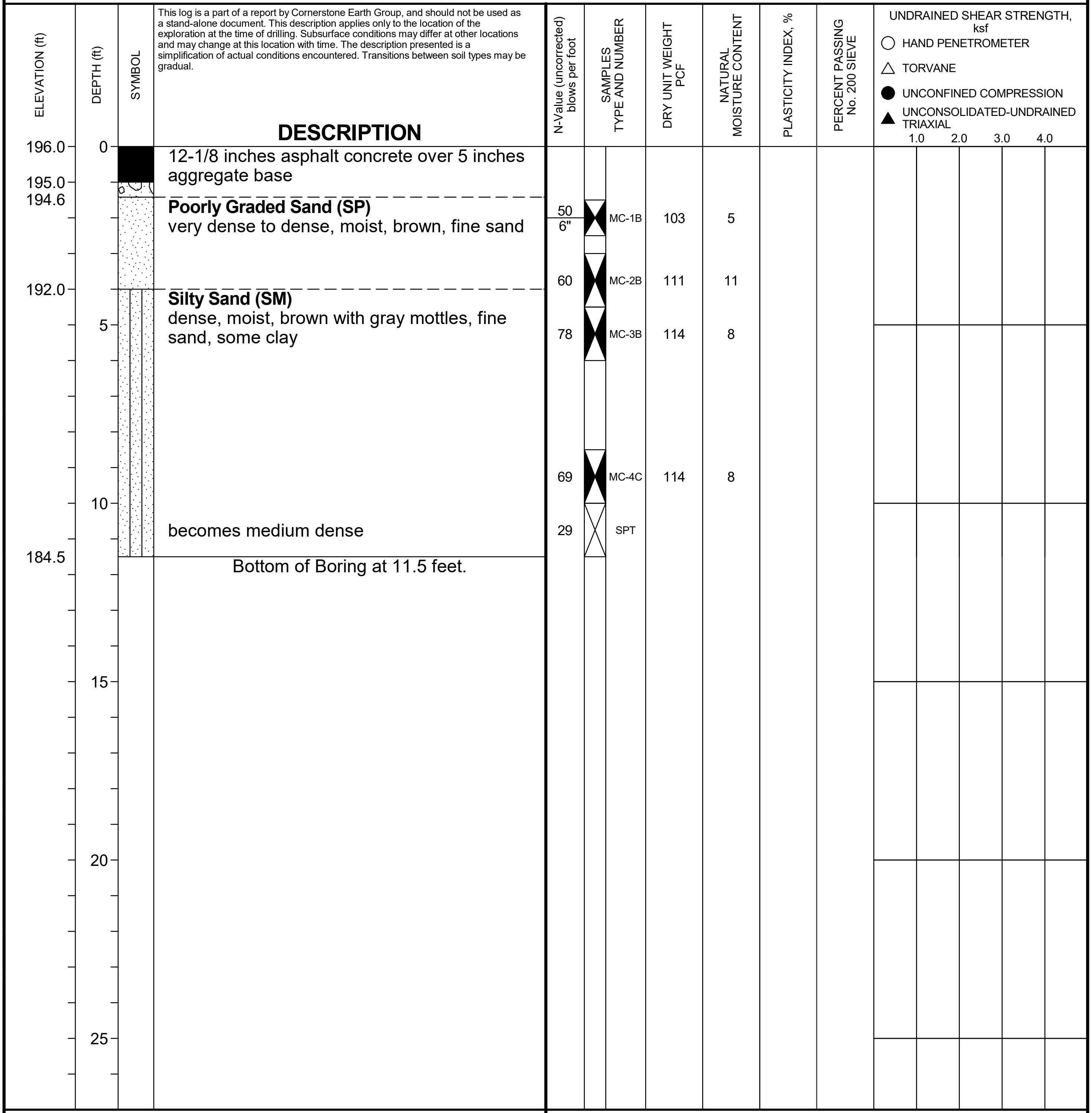

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
196.0	0	12-1/8 inches asphalt concrete over 5 inches aggregate base						
195.0								
194.6		**Poorly Graded Sand (SP)** very dense to dense, moist, brown, fine sand	50 6"	MC-1B	103	5		
			60	MC-2B	111	11		
192.0		**Silty Sand (SM)** dense, moist, brown with gray mottles, fine sand, some clay						
	5		78	MC-3B	114	8		
			69	MC-4C	114	8		
	10	becomes medium dense	29	SPT				
184.5		Bottom of Boring at 11.5 feet.						

BORING NUMBER EB-9

CORNERSTONE EARTH GROUP

PROJECT NAME MRY Terminal, Apron and North Side Improvements
PROJECT NUMBER 234-2-7
PROJECT LOCATION Monterey, CA
DATE STARTED 12/12/16 **DATE COMPLETED** 12/12/16
GROUND ELEVATION 213 FT +/- **BORING DEPTH** 10.5 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 36.585683° **LONGITUDE** -121.843469°
DRILLING METHOD Mobile B-56, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY DL
▽ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

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UNDRAINED SHEAR STRENGTH, ksf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL
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
213.0	0	11¼ inches asphalt concrete over 8 inches aggregate base						
212.1								
211.3		**Poorly Graded Sand with Silt (SP-SM)** dense, moist, light brown, fine sand	58	MC-1B	101	4		
210.5		**Silty Sand (SM)** very dense, moist, gray with brown, fine sand NP = Non-plastic 69% Sand, 14% Silt, 17% Clay D1557 Max. Dry Density: 124.7pcf @ 9.7% see CBR Results	50 5"	MC-2B	118	11	NP	31
	5		50 5"	MC-3B	102	8		
204.0		**Clayey Sand (SC)** dense, moist, gray with brown mottles, fine sand	50 5.5"	MC-4	108	9		
	10		62	SPT				
202.5		Bottom of Boring at 10.5 feet.						
	15							
	20							
	25							

CORNERSTONE
EARTH GROUP

BORING NUMBER EB-10

PAGE 1 OF 1

PROJECT NAME MRY Terminal, Apron and North Side Improvements
PROJECT NUMBER 234-2-7
PROJECT LOCATION Monterey, CA

DATE STARTED 12/11/16 **DATE COMPLETED** 12/11/16
GROUND ELEVATION 214 FT +/- **BORING DEPTH** 20 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 36.58596° **LONGITUDE** -121.84554°
DRILLING METHOD Mobile B-56, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY DL
∇ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

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ELEVATION (ft)	DEPTH (ft)	DESCRIPTION
214.0 213.9	0	1½ inch asphalt concrete over 5 inches aggregate base
213.5		**Clayey Sand with Gravel (SC) [Fill]** very dense, moist, brown, fine to coarse sand, fine to coarse gravel
210.8		**Poorly Graded Sand with Silt (SP-SM)** dense to medium dense, moist, light brown, fine sand
	5	
	10	
202.5		**Silty Sand (SM)** dense, moist, brown with gray mottles, fine sand
201.0		**Silty, Clayey Sand (SC-SM)** very dense, moist, brown with gray mottles, fine sand
	15	
198.8		**Sandy Silty Clay (CL-ML)** very stiff, moist, gray, fine sand, low plasticity
198.0		**Clayey Sand (SC)** dense, moist, gray, fine sand
194.0	20	Bottom of Boring at 20.0 feet.
	25	

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
50 5"	MC-1B	129	12			
60	SPT					
11	SPT-3		4			
16	SPT					
11	SPT-5		5			
31	SPT-6		10			
82	SPT-7		13			
77	SPT-8B	104	16			○ 3.0
44	SPT-9B	114	16			

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

BORING NUMBER EB-11

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CORNERSTONE
EARTH GROUP

PROJECT NAME MRY Terminal, Apron and North Side Improvements
PROJECT NUMBER 234-2-7
PROJECT LOCATION Monterey, CA
DATE STARTED 12/12/16 **DATE COMPLETED** 12/12/16
GROUND ELEVATION 216 FT +/- **BORING DEPTH** 25 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 36.58549° **LONGITUDE** -121.84407°
DRILLING METHOD Mobile B-56, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY DL
▽ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

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UNDRAINED SHEAR STRENGTH, ksf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL
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
216.0 215.9	0		1½ inch asphalt concrete over 4 inches aggregate base							
215.6			**Poorly Graded Sand with Silt (SP-SM) [Fill]** very dense, moist, light gray, fine sand	50/4"	MC-1B	107	7			
213.5			**Clayey Sand (SC)** very dense, moist, gray with brown mottles, fine sand, some silty sand lenses	50/6"	SPT					
	5			50/5"	MC-3B	110	8			
				80	SPT					
	10			50/5"	MC-5	89	10			
				63	SPT-6		12		25	
				50/3"	MC-7B	109	12			
202.0			**Silty Sand (SM)** dense, moist, brown, fine sand							
	15			40	SPT-8		11			
				44	SPT-9		12			
	20									
			becomes very dense	60	SPT-10		11			
191.0	25		Bottom of Boring at 25.0 feet.							

BORING NUMBER EB-12

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CORNERSTONE EARTH GROUP

PROJECT NAME MRY Terminal, Apron and North Side Improvements
PROJECT NUMBER 234-2-7
PROJECT LOCATION Monterey, CA
DATE STARTED 12/13/16 **DATE COMPLETED** 12/13/16
GROUND ELEVATION 208 FT +/- **BORING DEPTH** 20 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 36.586089° **LONGITUDE** -121.847517°
DRILLING METHOD Mobile B-56, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY DL
▽ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

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UNDRAINED SHEAR STRENGTH, ksf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL
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
208.0 207.6 207.3	0	4¾ inches asphalt concrete over 3½ inches aggregate base							
		Poorly Graded Sand with Silt (SP) [Fill] loose, moist, brown, fine sand	17	MC-1C	78	8			
205.5		**Poorly Graded Sand with Silt (SP-SM)** loose to medium dense, moist, brown, fine sand	12	MC-2B	96	4			
	5		20	MC-3B	96	4			
200.0		**Silty Sand (SM)** medium dense to very dense, moist, reddish brown with gray mottles, fine sand	28	MC-4B	110	12			
	10		50 6"	MC-5B	116	11			
		67% Sand, 21% Silt, 12% Clay	82	SPT-6		9		33	
			50 6"	MC-7B	117	12			
	15		60	SPT					
190.5		**Sandy Silty Clay (CL-ML)** very stiff, moist, gray with brown mottles, fine sand, low plasticity Liquid Limit = 27, Plastic Limit = 20	62	MC-9B	106	20	7		○
188.0	20	Bottom of Boring at 20.0 feet.							
	25								

BORING NUMBER EB-13

PAGE 1 OF 1

CORNERSTONE EARTH GROUP

PROJECT NAME MRY Terminal, Apron and North Side Improvements

PROJECT NUMBER 234-2-7

PROJECT LOCATION Monterey, CA

DATE STARTED 12/12/16 **DATE COMPLETED** 12/12/16

GROUND ELEVATION 212 FT +/- **BORING DEPTH** 20 ft.

DRILLING CONTRACTOR Exploration Geoservices, Inc.

LATITUDE 36.585812° **LONGITUDE** -121.846289°

DRILLING METHOD Mobile B-56, 8 inch Hollow-Stem Auger

GROUND WATER LEVELS:

LOGGED BY DL

▽ **AT TIME OF DRILLING** Not Encountered

NOTES

▼ **AT END OF DRILLING** Not Encountered

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

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
212.0	0	2¾ inches asphalt concrete over 4 inches aggregate base							
211.8									
211.4		**Silty Sand (SM)** medium dense, moist, brown, fine sand							
			41	MC-1C	104	11			
			20	SPT					
	5								
			26	MC-3B	107	9			
204.5		**Poorly Graded Sand with Silt (SP-SM)** very dense, moist, brown, fine sand							
			50	SPT-4		3			
202.0	10	**Clayey Sand (SC)** very dense, moist, gray with brown mottles, fine sand	50 5.5"	MC-5	100	21			
200.3		**Silty Sand (SM)** very dense, moist, gray with light brown mottles, fine sand D1557 Max. Dry Density: 124.0pcf @ 10.2%							
			64	SPT-6		12			
			50 6"	MC-7B	117	7			
198.0		**Clayey Sand (SC)** dense, moist, gray with brown mottles, fine sand							
	15		43	SPT-8		13			
			62	MC-9C	106	17			
192.0	20	Bottom of Boring at 20.0 feet.							
	25								

CORNERSTONE
EARTH GROUP

BORING NUMBER EB-14

PAGE 1 OF 2

PROJECT NAME MRY Terminal, Apron and North Side Improvements
PROJECT NUMBER 234-2-7
PROJECT LOCATION Monterey, CA
DATE STARTED 12/13/16 **DATE COMPLETED** 12/13/16
GROUND ELEVATION 209 FT +/- **BORING DEPTH** 40 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 36.58589° **LONGITUDE** -121.84813°
DRILLING METHOD Mobile B-56, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY DL
▽ **AT TIME OF DRILLING** 33.5 ft.
NOTES
▼ **AT END OF DRILLING** 32 ft.

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UNDRAINED SHEAR STRENGTH, ksf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL
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
209.0	0		4½ inches asphalt concrete							
208.6			**Poorly Graded Sand with Silt (SP-SM)** medium dense, moist, brown to light brown, fine sand	41	MC-1B	100	5			
206.0			**Silty Sand (SM)** loose, moist, reddish brown, fine sand	15	MC					
204.0	5		**Poorly Graded Sand with Silt (SP-SM)** medium dense, moist, brown to light brown, fine sand	26	MC-3B	95	3			
				26	MC-4B	92	5			
	10			18	SPT-5	7				
197.0			**Silty Sand (SM)** very dense, moist, brown, fine sand, slightly cemented	50 5"	MC-6B	113	8			
	15			50 5"	SPT-7		7			
191.5			**Sandy Lean Clay (CL)** hard, moist, gray, fine sand, low plasticity Liquid Limit = 34, Plastic Limit = 13	42	SPT-8		17	21	59	>4.5 ○
188.5	20		**Clayey Sand (SC)** dense, moist, reddish brown with gray mottles, fine sand, some fine subangular gravel	31	SPT-9		16			
	25			57	MC-10B	109	20			
183.0										

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CORNERSTONE
EARTH GROUP

BORING NUMBER EB-14

PROJECT NAME MRY Terminal, Apron and North Side Improvements
PROJECT NUMBER 234-2-7
PROJECT LOCATION Monterey, CA

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UNDRAINED SHEAR STRENGTH, ksf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL
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
183.0								
182.5		**Silty Sand (SM)** dense, moist to wet, gray brown, fine sand						
			34	SPT-11		25		
	30							
		becomes medium dense	48	MC-12B	103	25		
	35							
		becomes dense	40	SPT				
			70	MC-14B	104	22		
169.0	40	Bottom of Boring at 40.0 feet.						
	45							
	50							
	55							

CORNERSTONE
EARTH GROUP

BORING NUMBER EB-15

PAGE 1 OF 2

PROJECT NAME MRY Terminal, Apron and North Side Improvements
PROJECT NUMBER 234-2-7
PROJECT LOCATION Monterey, CA

DATE STARTED 12/13/16 **DATE COMPLETED** 12/13/16
DRILLING CONTRACTOR Exploration Geoservices, Inc.
DRILLING METHOD Mobile B-56, 8 inch Hollow-Stem Auger
LOGGED BY DL
NOTES

GROUND ELEVATION 213 FT +/- **BORING DEPTH** 40 ft.
LATITUDE 36.58556° **LONGITUDE** -121.84731°
GROUND WATER LEVELS:
▽ **AT TIME OF DRILLING** Not Encountered
▼ **AT END OF DRILLING** Not Encountered

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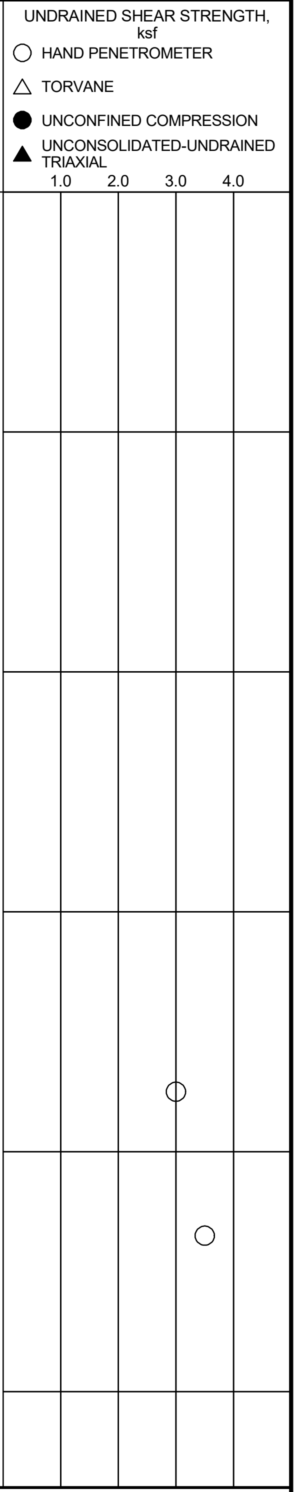
ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
213.0 212.8	0	2½ inches asphalt concrete						
		Poorly Graded Sand with Silt (SP-SM) medium dense, moist, brown to light brown, fine sand	21	MC-1B	95	5		
			19	MC				
	5		17	MC-3B	92	6		
205.0		**Clayey Sand (SC)** very dense, moist, gray with brown mottles, fine sand	50 6"	MC-4B	115	10		
	10							
201.0		**Silty Sand (SM)** very dense, moist, brown to light gray, fine sand	35	SPT				
	15		84	MC-6B	114	11		
195.5		**Sandy Lean Clay (CL)** very stiff, moist, brown with gray mottles, fine sand, low plasticity	28	SPT-7		15		
	20							
192.5		**Lean Clay (CL)** very stiff, moist, gray with brown mottles, some fine sand, moderate plasticity	35	MC-8B	90	31		
189.5	25	**Silty Sand (SM)** dense, moist, gray with brown mottles, fine sand, some clayey sand lenses	60	MC-9B	99	21		
187.0								



Continued Next Page

BORING NUMBER EB-15

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PROJECT NAME MRY Terminal, Apron and North Side Improvements

PROJECT NUMBER 234-2-7

PROJECT LOCATION Monterey, CA

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ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf ○ HAND PENETROMETER △ TORVANE ● UNCONFINED COMPRESSION ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL 1.0 2.0 3.0 4.0
187.0			**Silty Sand (SM)** dense, moist, gray with brown mottles, fine sand, some clayey sand lenses							
				66	MC-10B	107	18			
	30									
181.5			**Silty Sand (SM)** dense, wet, gray brown, fine sand							
				52	MC-11B	97	25			
	35									
				59	MC-12B	96	27			
173.0	40		Bottom of Boring at 40.0 feet.							
	45									
	50									
	55									

CORNERSTONE
EARTH GROUP

BORING NUMBER EB-16

PAGE 1 OF 1

PROJECT NAME MRY Terminal, Apron and North Side Improvements
PROJECT NUMBER 234-2-7
PROJECT LOCATION Monterey, CA
DATE STARTED 12/13/16 **DATE COMPLETED** 12/13/16
GROUND ELEVATION 217 FT +/- **BORING DEPTH** 20 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 36.58522° **LONGITUDE** -121.84633°
DRILLING METHOD Mobile B-56, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY DL
▽ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

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UNDRAINED SHEAR STRENGTH, ksf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL
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
217.0 216.8	0	2 inches asphalt concrete over 4 inches aggregate base						
216.5		**Silty Sand (SM)** very dense, moist, brown, fine sand	50 4"	MC-1B	115	12		
			50 6"	SPT-2		9		
	5	color becomes gray with brown mottles	50 6"	SPT				
			50 3.5"	SPT-4		9		
		slightly cemented						
	10		50 6"	MC-5	102	12		
		dense	55	SPT-6		11		
		D1557 Max. Dry Density: 124.0pcf @ 10.2%	50 5"	MC-7B	111	11		
202.5	15	**Clayey Sand (SC)** very dense, moist, brown and gray mottled, fine sand Liquid Limit = 27, Plastic Limit = 13	74	SPT-8		17	14	38
		becomes medium dense	28	SPT-9		16		
197.0	20	Bottom of Boring at 20.0 feet.						
	25							

BORING NUMBER EB-17

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CORNERSTONE EARTH GROUP

PROJECT NAME MRY Terminal, Apron and North Side Improvements

PROJECT NUMBER 234-2-7

PROJECT LOCATION Monterey, CA

DATE STARTED 12/14/16 **DATE COMPLETED** 12/14/16

GROUND ELEVATION 219 FT +/- **BORING DEPTH** 20 ft.

DRILLING CONTRACTOR Exploration Geoservices, Inc.

LATITUDE 36.58477° **LONGITUDE** -121.84406°

DRILLING METHOD Mobile B-56, 8 inch Hollow-Stem Auger

GROUND WATER LEVELS:

LOGGED BY DL

▽ **AT TIME OF DRILLING** 15 ft.

NOTES

▼ **AT END OF DRILLING** 13 ft.

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UNDRAINED SHEAR STRENGTH, ksf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL
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
219.0 218.8	0	2¾ inches asphalt concrete over 5 inches aggregate base						
218.3		**Silty Sand (SM)** dense, moist, reddish brown to brown, fine sand						
			60	MC-1B	107	11		
			50 6"	MC-2B	108	11		
	5		50	MC				
211.0		**Silty, Clayey Sand (SC-SM)** medium dense, moist, brown with gray mottles, fine sand						
			31	MC-4B	103	20		
	10		44	MC-5B	103	18		26
		becomes dense	32	SPT-6		20		
206.0 ▼		**Clayey Sand (SC)** dense, moist, reddish brown with gray mottles, fine to coarse sand, some cemented nodules	50 6"	MC-7B	98	24		
▽	15		44	SPT-8		24		
202.0		**Silty Sand (SM)** very dense, moist to wet, gray brown, fine sand						
			65	SPT-9		19		
199.0	20	Bottom of Boring at 20.0 feet.						
	25							

BORING NUMBER EB-18

CORNERSTONE EARTH GROUP

PROJECT NAME MRY Terminal, Apron and North Side Improvements
PROJECT NUMBER 234-2-7
PROJECT LOCATION Monterey, CA
DATE STARTED 12/14/16 **DATE COMPLETED** 12/14/16
GROUND ELEVATION 212 FT +/- **BORING DEPTH** 40 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 36.58469° **LONGITUDE** -121.84794°
DRILLING METHOD Mobile B-56, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY DL
∇ **AT TIME OF DRILLING** 28 ft.
NOTES
▼ **AT END OF DRILLING** 26.5 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.

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf ○ HAND PENETROMETER △ TORVANE ● UNCONFINED COMPRESSION ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL 1.0 2.0 3.0 4.0
212.0	0		6 inches aggregate base							
211.5			**Poorly Graded Sand with Silt (SP-SM)** medium dense, moist, brown, fine sand							
				31	MC-1B	94	5			
				23	MC					
	5									
				50	MC-3B	111	12			
204.5			**Clayey Sand (SC)** dense, moist, gray with brown mottles, fine sand							
				79	MC-4B	116	12			
	10									
201.0			**Silty Sand (SM)** very dense, moist, brown with gray mottles, fine sand, some clay							
				50 6"	MC-5B	108	15			
	15									
				89	SPT-6		16			
				36	SPT-7		18		26	
			becomes dense							
192.0	20		**Silty Sand (SM)** dense to medium dense, moist to wet, gray brown, fine sand							
				61	MC-8B	110	18			
				50	MC-9B	108	18			
	25									
186.0										

Continued Next Page

BORING NUMBER EB-18

PAGE 2 OF 2

CORNERSTONE EARTH GROUP

PROJECT NAME MRY Terminal, Apron and North Side Improvements
PROJECT NUMBER 234-2-7
PROJECT LOCATION Monterey, CA

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

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

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
186.0	▼ ▽	**Silty Sand (SM)** dense to medium dense, moist to wet, gray brown, fine sand							
		some thin clay seams	38	MC-10B	97	22			
	30		58	SPT					
			50	SPT-12		26			
	35								
175.0		**Fat Clay (CH)** very stiff, moist, gray, some fine sand, high plasticity							
			51	MC-13C	81	40			○
172.0	40	Bottom of Boring at 40.0 feet.							
	45								
	50								
	55								

BORING NUMBER EB-19

PAGE 1 OF 1

CORNERSTONE EARTH GROUP

PROJECT NAME MRY Terminal, Apron and North Side Improvements
PROJECT NUMBER 234-2-7
PROJECT LOCATION Monterey, CA
DATE STARTED 12/14/16 **DATE COMPLETED** 12/14/16
GROUND ELEVATION 216 FT +/- **BORING DEPTH** 20 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 36.58418° **LONGITUDE** -121.84704°
DRILLING METHOD Mobile B-56, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY DL
▽ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

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

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf ○ HAND PENETROMETER △ TORVANE ● UNCONFINED COMPRESSION ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL 1.0 2.0 3.0 4.0
216.0	0	**Poorly Graded Sand with Silt (SP-SM)** medium dense, moist, gray, fine to medium sand							
			21	MC-1B	105	16			
213.5		**Silty Sand (SM)** dense, moist, light gray to gray, fine sand							
			79	MC-2B	120	10			
	5								
210.5		**Sandy Lean Clay (CL)** very stiff, moist, gray, fine sand, low plasticity	54	MC-3A	116	10			○ 2.0
210.0		**Poorly Graded Sand with Silt (SP-SM)** very dense, moist, gray, fine to medium sand	50 5.5"	MC-4	104	13			
208.0		**Clayey Sand (SC)** dense, moist, brown with light gray mottles, fine sand	40	SPT-5		21			
	10								
205.0		**Poorly Graded Sand with Silt (SP-SM)** very dense, moist, light gray, fine sand							
			50	SPT-6		16			
	15								
		becomes dense	35	SPT-7		20			
196.0	20	Bottom of Boring at 20.0 feet.							
	25								

APPENDIX B: LABORATORY TEST PROGRAM

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

Moisture Content: The natural water content was determined (ASTM D2216) on 135 samples of the materials recovered from the borings. These water contents are recorded on the boring logs at the appropriate sample depths.

Dry Densities: In place dry density determinations (ASTM D2937) were performed on 97 samples to measure the unit weight of the subsurface soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

Washed Sieve Analyses: The percent soil fraction passing the No. 200 sieve (ASTM D1140) was determined on 11 samples of the subsurface soils to aid in the classification of these soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

Grain Size Analyses: The particle size distribution (ASTM D422) was determined on four samples of the near-surface soils to aid in the classification of these soils and determining the percent silt and clay. Results of these tests are shown in the table below.

Table B-1: Summary of Grain Size Analyses

Sieve Size	EB-4 1.5’	EB-7 2 – 7’	EB-9 1.5 – 6.5’	EB-12 11.5’
	% Passing	% Passing	% Passing	% Passing
¾-inch	100	100	100	100
No. 4	100	100	100	100
No. 10	100	99.7	99.6	100
No. 16	100	99.5	99.4	99.9
No. 30	99.8	98.5	98.3	99.4
No. 50	86.7	88.3	90.0	95.4
No. 100	18.6	42.0	44.0	54.1
No. 200	10.1	29.4	30.8	32.7
Silt	5%	11%	14%	21%
Clay	5%	18%	17%	12%

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

Compaction: Five compaction tests (ASTM D 1557-00 Method B) were performed on sand samples to assist in evaluating the maximum dry unit weight and optimum moisture content of the soils. Results of the compaction tests are included as part of this appendix.

California Bearing Ratio: California Bearing Ratio Tests (CBR) (ASTM D1883) were performed on two samples of the subsurface soils ranging from 1½ to 7 feet below grade to evaluate the mechanical strength of this soil. Results of the CBR tests are shown below and graphically in this appendix.

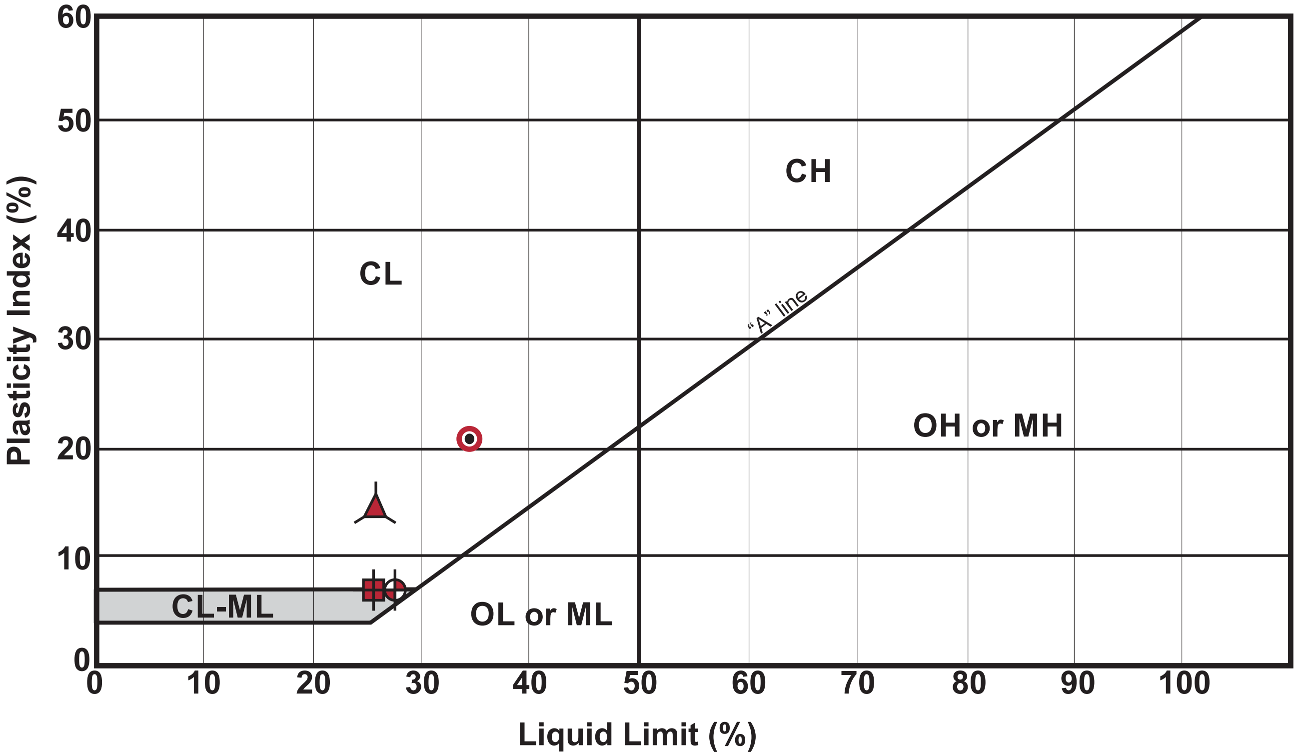
Table B-2: Results of California Bearing Ratio Test Boring EB-7

Sample	**Depth of Soil Sample**	**Classification**	**CBR at Required In-Situ Field Density (%)**			
EB-7	2 – 7 feet	Sand	Rel. Comp. (%)	91	95	98.3
			@ 0.1 in.	18.2	29.1	41.5
			@ 0.2 in.	20.9	32.8	54.7

Table B-3: Results of California Bearing Ratio Test Boring EB-9

Sample	**Depth of Soil Sample**	**Classification**	**CBR at Required In-Situ Field Density (%)**			
EB-7	1½ – 6½ feet	Sand	Rel. Comp. (%)	90.7	95	99.6
			@ 0.1 in.	16.6	32.4	46.4
			@ 0.2 in.	17.9	38.1	60.6

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-4	1.5	7	determined non-plastic			10	Poorly Graded Sand with Silt (SP-SM)
⊕	EB-6	1.5	15	28	21	7	—	Silty, Clayey Sand (SC-SM) [Fill]
	EB-7	5.0	11	determined non-plastic			29	Silty Sand (SM) (ML fines)
	EB-9	3.5	11	determined non-plastic			31	Silty Sand (SM) (ML fines)
⊞	EB-12	19.5	20	27	20	7	—	Sandy Silty Clay (CL-ML)
◉	EB-14	18.5	17	34	13	21	59	Sandy Lean Clay (CL)
▲	EB-16	15.0	17	27	13	14	38	Clayey Sand (SC) (CL fines)

Samples prepared in accordance with ASTM D421

CORNERSTONE
EARTH GROUP

Plasticity Index Testing Summary

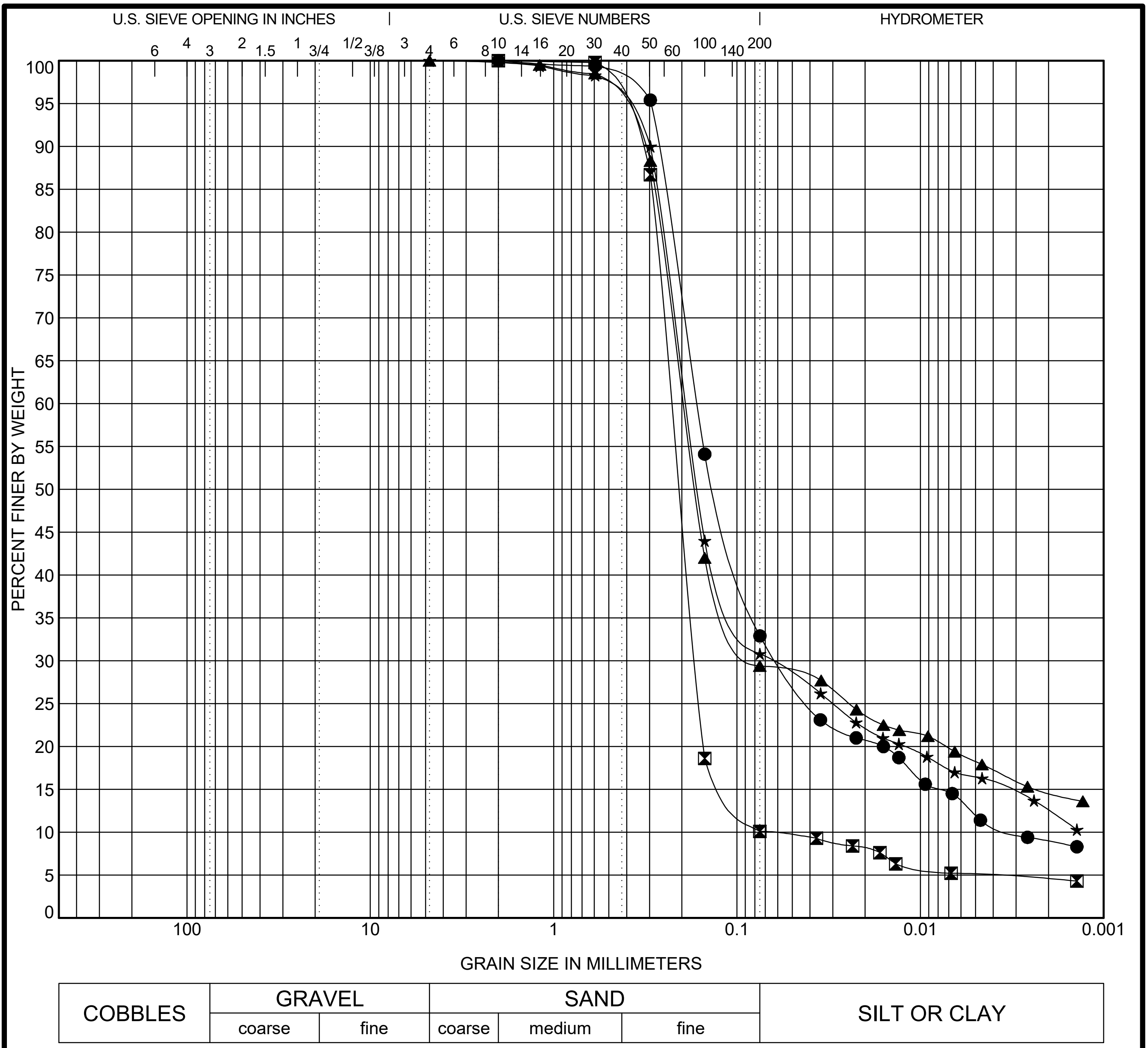
Monterey Peninsula Airport
Terminal Building, Apron Area,
and North Side Improvements Areas
Monterey, CA

Project Number 234-2-7

Figure Number Figure B1

Date January 2017

Drawn By FLL



Specimen Identification			Classification	LL	PL	PI	Cc	Cu
●	**EB-12**	**11.5**	**Silty Sand (SM)**				**6.99**	**53.3**
☒	**EB-4**	**0.5**	**Poorly Graded Sand with Silt (SP-SM)**				**1.81**	**3.3**
▲	**EB-7**	**5.0**	**Silty Sand (SM)**					
★	**EB-9**	**3.0**	**Silty Sand (SM)**					

Specimen Identification			D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**EB-12**	**11.5**	**2**	**0.165**	**0.06**	**0.003**	**0.0**	**67.1**	**32.9**	
☒	**EB-4**	**0.5**	**2**	**0.227**	**0.168**	**0.069**	**0.0**	**89.9**	**10.1**	
▲	**EB-7**	**5.0**	**4.75**	**0.196**	**0.078**		**0.0**	**70.6**	**29.4**	
★	**EB-9**	**3.0**	**4.75**	**0.19**	**0.066**		**0.0**	**69.2**	**30.8**	

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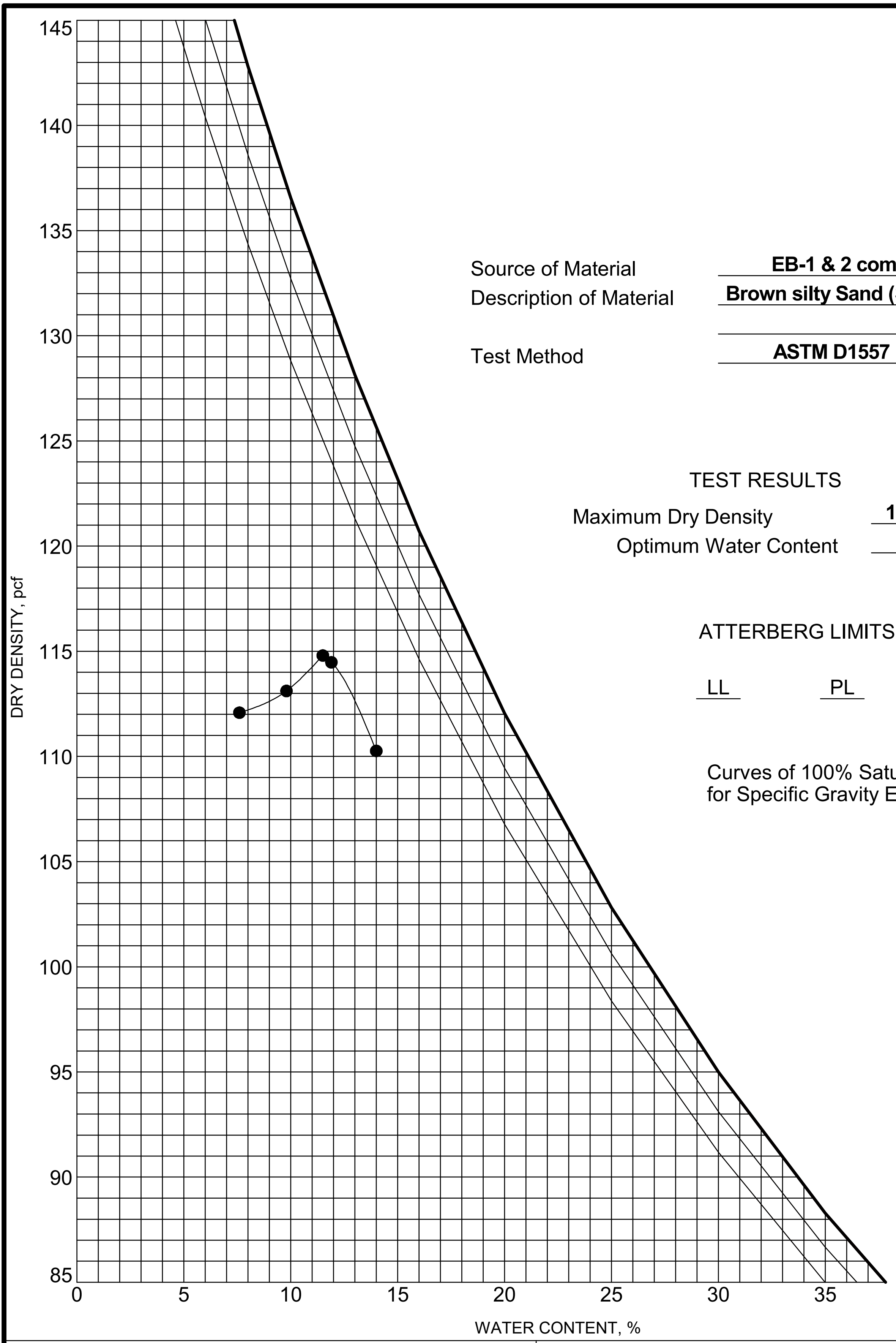
GRAIN SIZE DISTRIBUTION

Project: MRY Terminal, Apron Area and N. Side Improv.
Location: Monterey, CA
Number: 234-2-7

US GRAIN SIZE - CORNERSTONE 0812.GDT - 1/5/17 07:22 - P:\DRAFTING\GINT FILES\234-2-7 MRY TERMINAL.GPJ



Source of Material **EB-1 & 2 composite 2.5**

Description of Material **Brown silty Sand (SM)**

Test Method **ASTM D1557 Method B**

TEST RESULTS

Maximum Dry Density **114.8 PCF**

Optimum Water Content **11.5 %**

ATTERBERG LIMITS

LL	PL	PI

Curves of 100% Saturation
for Specific Gravity Equal to:

2.80

2.70

2.60

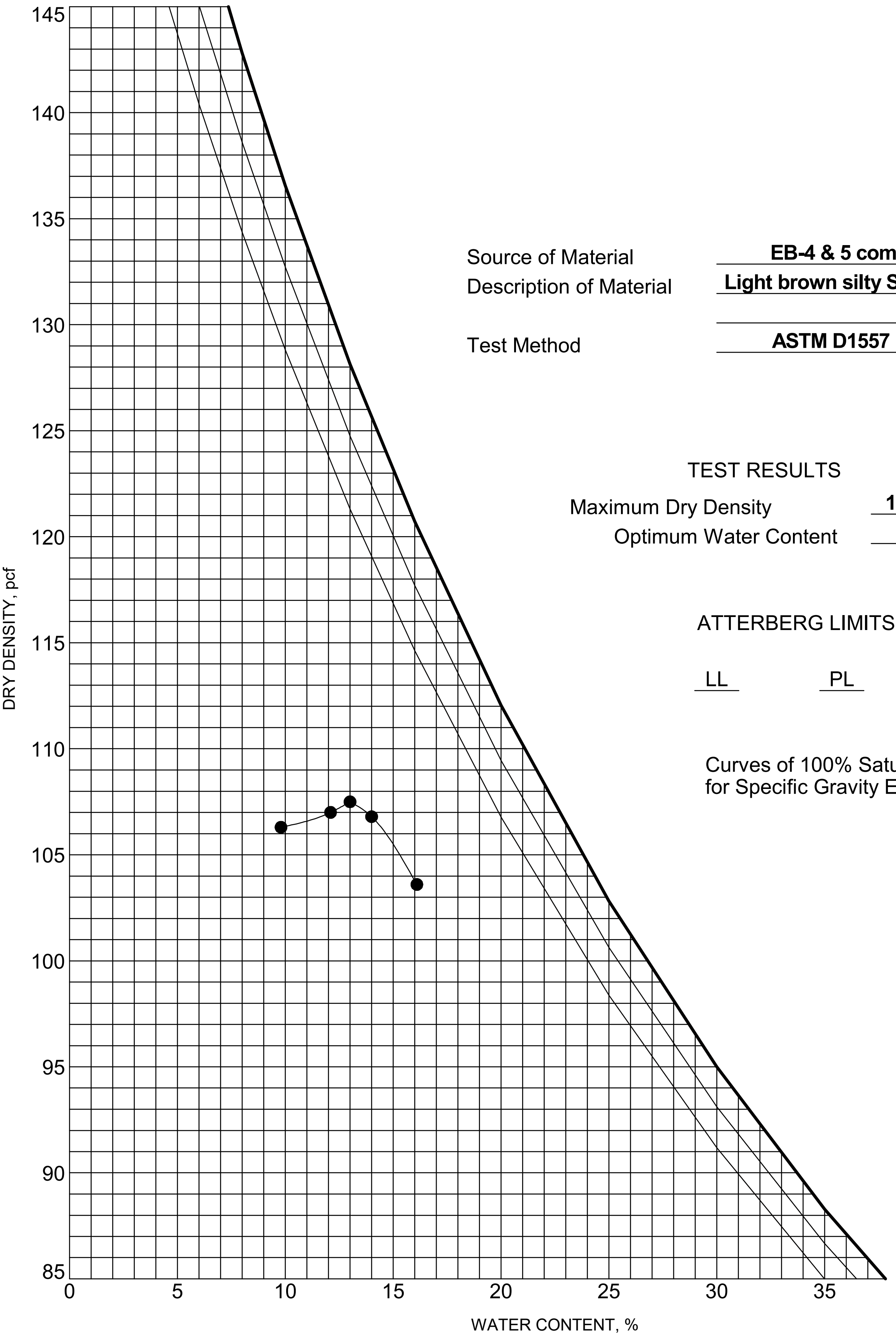
CORNERSTONE
EARTH GROUP

MOISTURE-DENSITY RELATIONSHIP

Project: MRY Terminal, Apron Area and N. Side Improv.

Location: Monterey, CA

Number: 234-2-7



Source of Material	**EB-4 & 5 composite 2.5**
Description of Material	**Light brown silty Sand (SM)**
Test Method	**ASTM D1557 Method B**

TEST RESULTS

Maximum Dry Density **107.5 PCF**
Optimum Water Content **13.0 %**

ATTERBERG LIMITS

LL	PL	PI

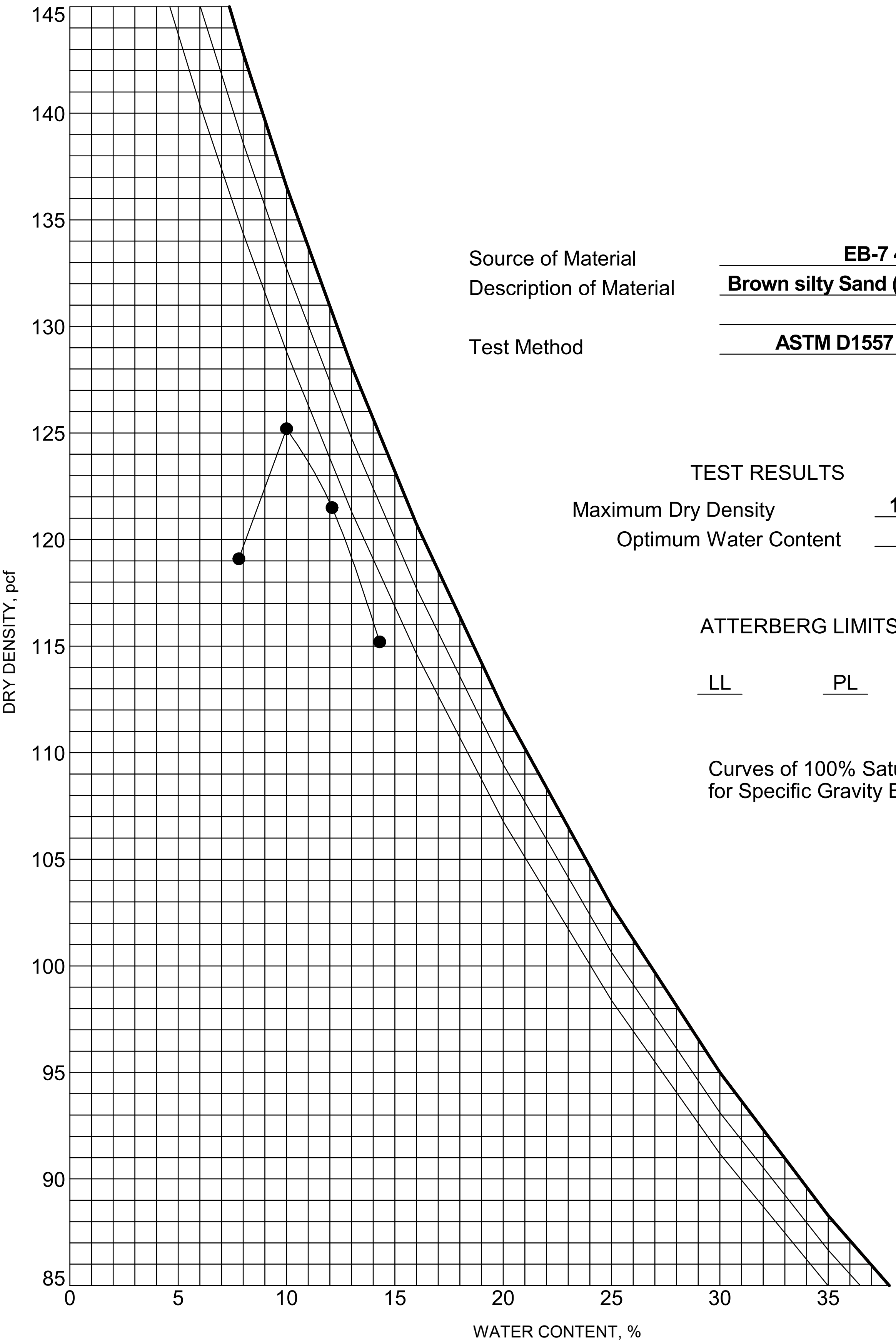
Curves of 100% Saturation
for Specific Gravity Equal to:
2.80
2.70
2.60

CORNERSTONE
EARTH GROUP

MOISTURE-DENSITY RELATIONSHIP

Project: MRY Terminal, Apron Area and N. Side Improv.
Location: Monterey, CA
Number: 234-2-7

US COMPACTION - CORNERSTONE 0812.GDT - 1/5/17 07:08 - P:\DRAFTING\GINT FILES\234-2-7 MRY TERMINAL.GPJ



Source of Material **EB-7 4.5**

Description of Material **Brown silty Sand (SM)**

Test Method **ASTM D1557 Method B**

TEST RESULTS

Maximum Dry Density **125.2 PCF**

Optimum Water Content **10.0 %**

ATTERBERG LIMITS

LL	PL	PI

Curves of 100% Saturation
for Specific Gravity Equal to:

2.80

2.70

2.60

MOISTURE-DENSITY RELATIONSHIP

Project: MRY Terminal, Apron Area and N. Side Improv.

Location: Monterey, CA

Number: 234-2-7

CORNERSTONE
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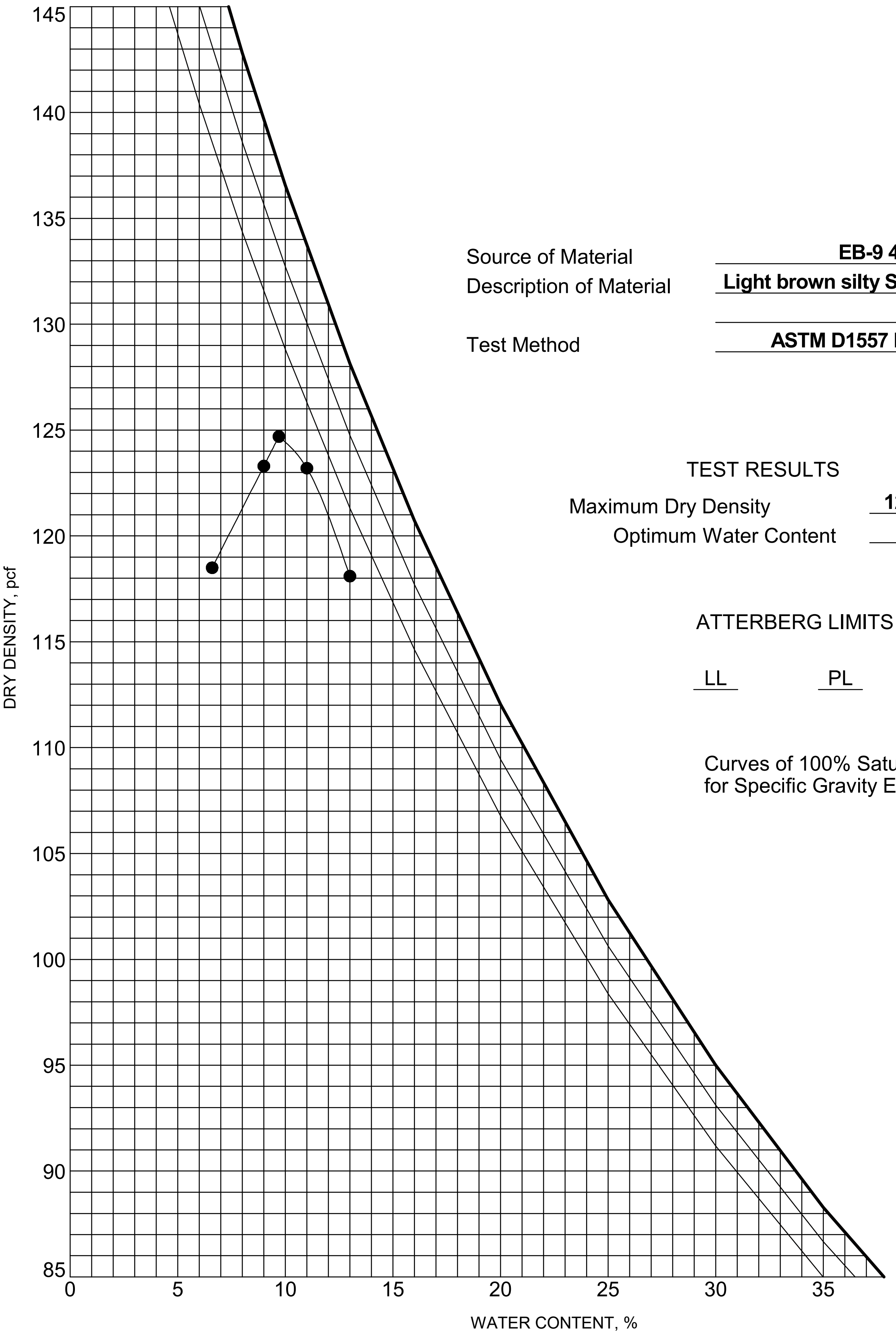

Source of Material **EB-9 4.0**

Description of Material **Light brown silty Sand (SM)**

Test Method **ASTM D1557 Method B**

TEST RESULTS

Maximum Dry Density **124.7** **PCF**

Optimum Water Content **9.7** **%**

ATTERBERG LIMITS

LL	PL	PI

Curves of 100% Saturation
for Specific Gravity Equal to:

2.80

2.70

2.60

MOISTURE-DENSITY RELATIONSHIP

Project: MRY Terminal, Apron Area and N. Side Improv.

Location: Monterey, CA

Number: 234-2-7

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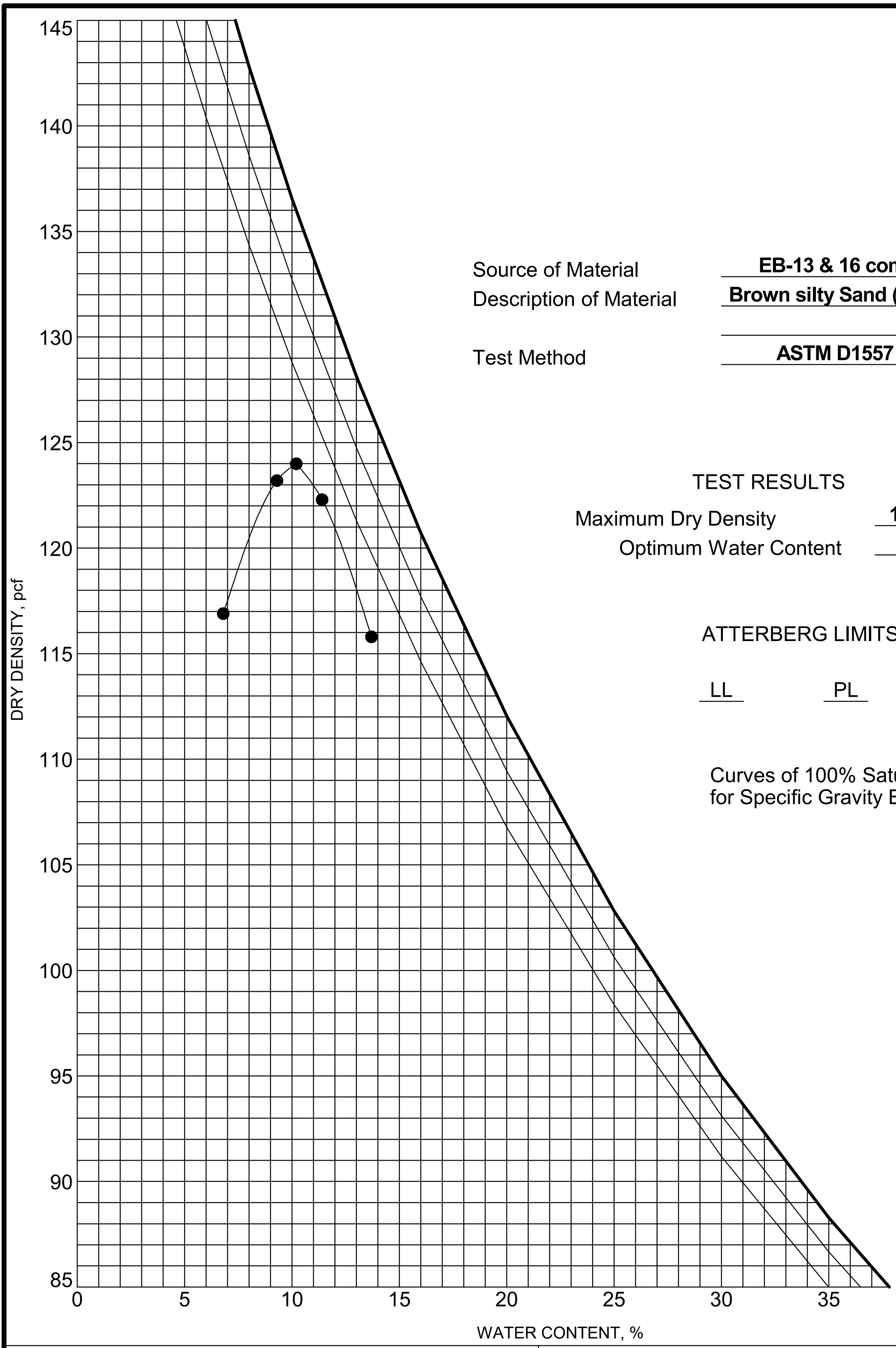


Source of Material **EB-13 & 16 composite 13.0**

Description of Material **Brown silty Sand (SM)**

Test Method **ASTM D1557 Method B**

TEST RESULTS

Maximum Dry Density **124.0** **PCF**

Optimum Water Content **10.2** **%**

ATTERBERG LIMITS

LL	PL	PI

Curves of 100% Saturation
for Specific Gravity Equal to:

2.80

2.70

2.60

MOISTURE-DENSITY RELATIONSHIP

CORNERSTONE
EARTH GROUP

Project: MRY Terminal, Apron Area and N. Side Improv.

Location: Monterey, CA

Number: 234-2-7

US_COMPACTION - CORNERSTONE 0812.GDT - 1/5/17 07:10 - P:\DRAFTING\GINT FILES\234-2-7 MRY TERMINAL.GPJ

COOPER TESTING LABORATORY

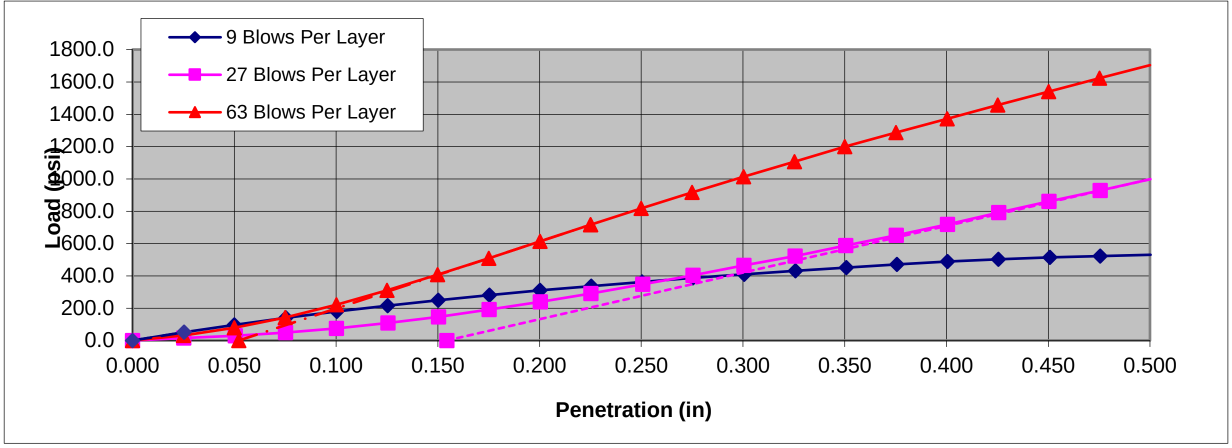
California Bearing Ratio ASTM D 1883

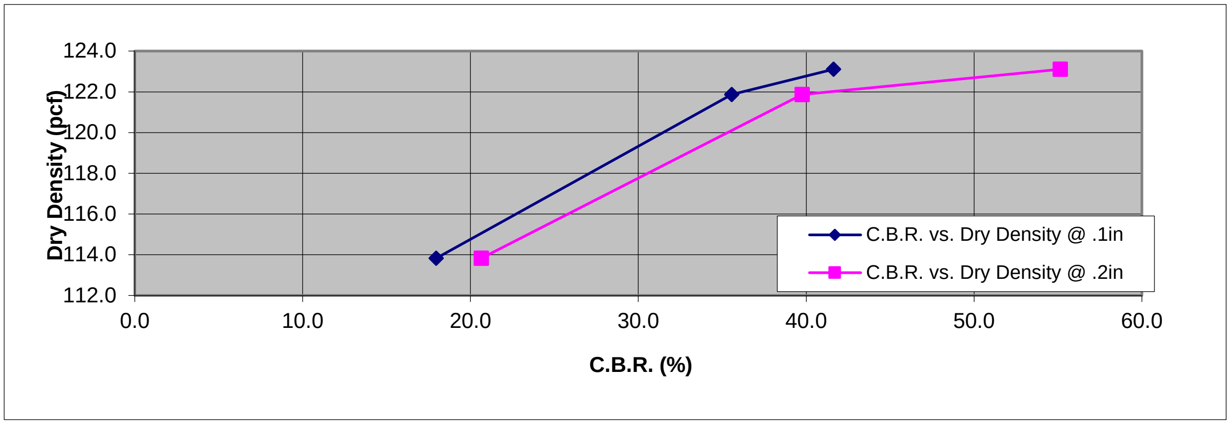
CTL Job No.: 640-1065
Boring: EB-7
Date: 12/20/2016
Client: Cornerstone Earth Group
Sample: Bulk
Tested : PJ
Project Name: MRY New Terminal and Apron Imp
Depth (ft.): 2'-7'
Checked: DC
Project No: 234-2-7
Visual Description: Brown Silty SAND (SM)

Maximum Dry Density (pcf):	125.2	Optimum Moisture Content (%):	10.0	Rel. Comp. For CBR Evaluation (% of max):	98.3
Maximum Density & Optimum Moisture Obtained By:	ASTM D1557	Sample Condition:	Soaked	Surcharge Weight (kg):	4.54
Percent Retained on 3/4in. Sieve:	0.0	Remarks:	The CBR at 0.2 inches was greater than the CBR at 0.1 inches. The confirmatory test was not run. Due to the material characteristics, the 9 blows per layer point exceeded the minimum target relative compaction of 90%, the CBR was reported at 91.0% relative compaction. Did not exceed the maximum target relative compaction of 100% at 63 blows per layer, CBR reported at 98.3% relative compaction.		

Sample Information

Number of Blows per Layer	Dry Density (pcf)		Moisture Content (%)		Expansion (%)	C.B.R. (%)	
	Before Soaking	After Soaking	Before Soaking	After Soaking		@ 0.1in.	@ 0.2in.
9	113.8	113.8	10.9	13.8	0.1	18.0	20.6
27	121.9	121.9	11.1	12.4	0.0	35.6	39.8
63	123.1	122.9	10.7	11.3	0.2	41.6	55.1





C.B.R. at Required In Field Density (%)			
%RC	91	95	98.3
@ 0.1in	18.2	29.1	41.5
@ 0.2in	20.9	32.8	54.7

COOPER TESTING LABORATORY

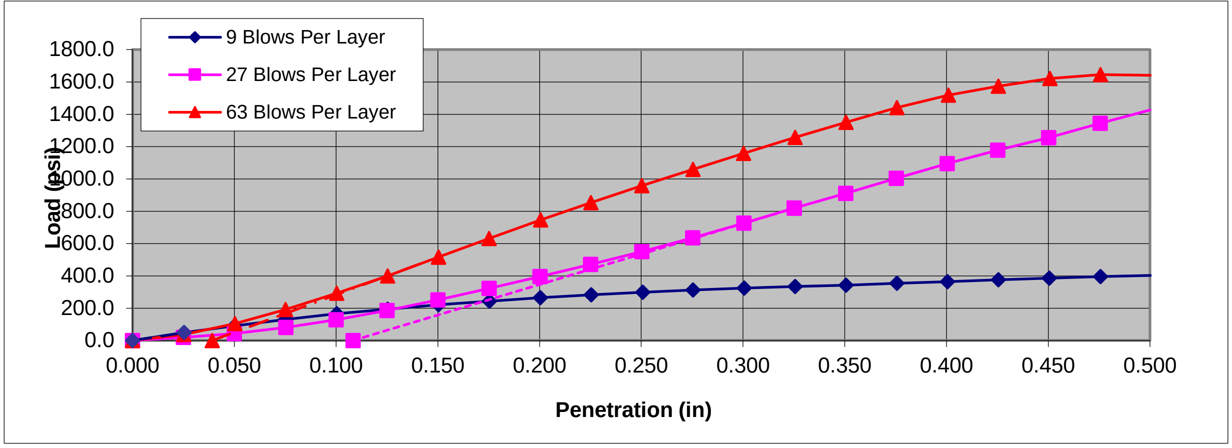
California Bearing Ratio ASTM D 1883

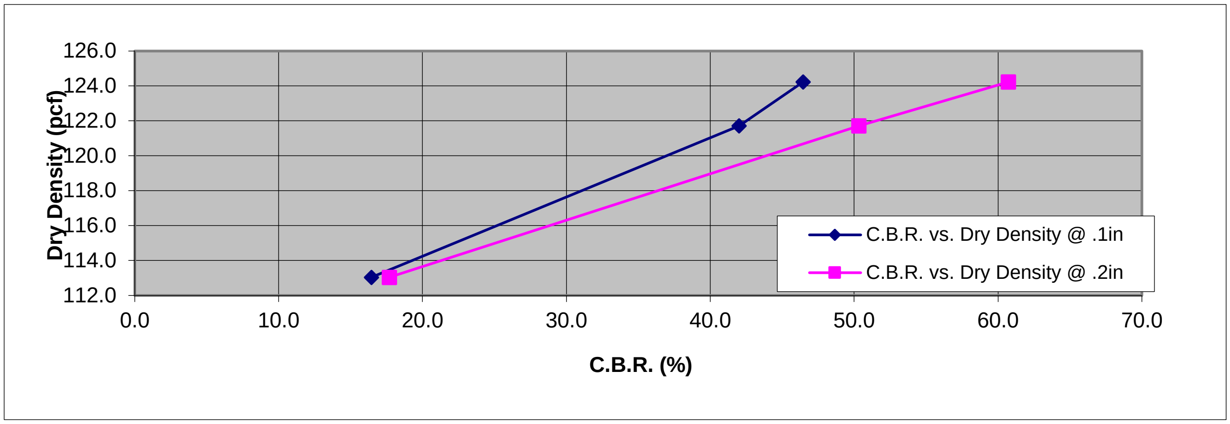
CTL Job No.: 640-1065
Boring: EB-9
Date: 12/20/2016
Client: Cornerstone Earth Group
Sample: Bulk
Tested : PJ
Project Name: MRY New Terminal and Apron Imp
Depth (ft.): 1.5'-6.5'
Checked: DC
Project No: 234-2-7
Visual Description: Light Brown Silty SAND (SM)

Maximum Dry Density (pcf):	124.7	Optimum Moisture Content (%):	9.7	Rel. Comp. For CBR Evaluation (% of max):	99.6
Maximum Density & Optimum Moisture Obtained By:	ASTM D1557	Sample Condition:	Soaked	Surcharge Weight (kg):	4.54
Percent Retained on 3/4in. Sieve:	0.0	Remarks:	The CBR at 0.2 inches was greater than the CBR at 0.1 inches. The confirmatory test was not run. Due to the material characteristics, the 9 blows per layer point exceeded the minimum target relative compaction of 90%, the CBR was reported at 90.7% relative compaction. Did not exceed the maximum target relative compaction of 100% at 63 blows per layer, CBR reported at 99.7% relative compaction.		

Sample Information

Number of Blows per Layer	Dry Density (pcf)		Moisture Content (%)		Expansion (%)	C.B.R. (%)	
	Before Soaking	After Soaking	Before Soaking	After Soaking		@ 0.1in.	@ 0.2in.
9	113.0	113.0	10.5	14.5	0.0	16.4	17.7
27	121.7	121.7	10.5	11.9	0.0	42.0	50.3
63	124.2	124.2	10.5	11.3	0.0	46.4	60.7





C.B.R. at Required In Field Density (%)			
%RC	90.7	95	99.6
@ 0.1in	16.6	32.4	46.4
@ 0.2in	17.9	38.1	60.6

APPENDIX C: SITE CORROSION TEST DATA

Corrosivity Tests Summary

CTL # 640-1068 **Date:** 12/23/2016 **Tested By:** PJ **Checked:** PJ

Client: Cornerstone Earth Group **Project:** MRY Terminal Apron and Bldg **Proj. No:** 234-2-7

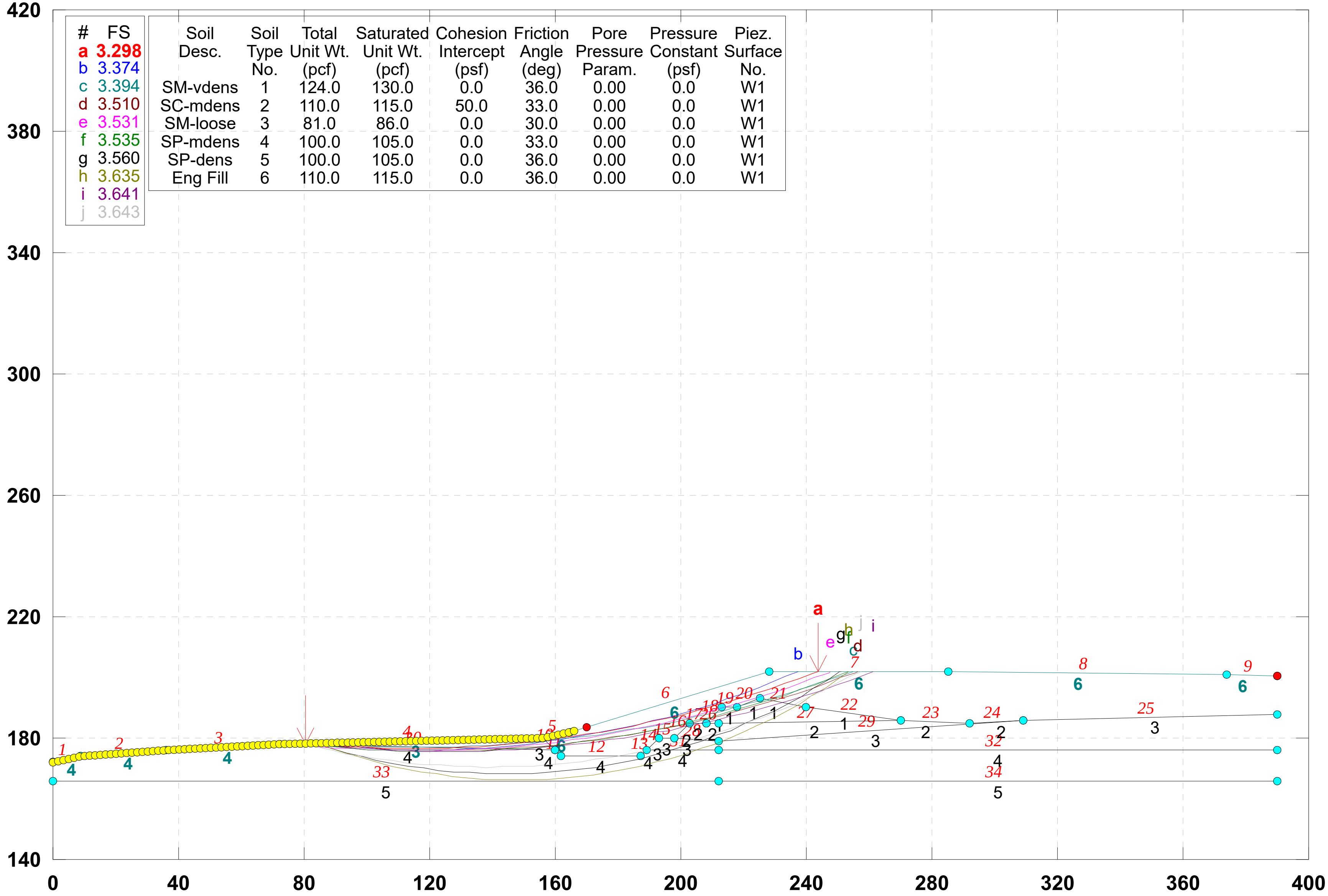
Remarks:

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-13	6	11.5	-	-	26,138	<2	2	0.0002	6.5	-	-	-	12.0	Olive Brown SAND w/ Silt
EB-14	5	10.0	-	-	21,525	<2	25	0.0025	7.0	-	-	-	6.9	Oale Brown SAND w/ Silt

APPENDIX D: STABILITY ANALYSIS OUTPUT

Monterey Airport Cross-Section A-A' Static

p:\projects\200\234 kimley horn and associates\234-2-7 mry terminal apron and building gi\slope stability\north airport fill slope\final prelim report files\monterey airport cross-section a-a' static with 3 to 1 proposed fill slope.pl2 Run By: Matthew Schaffer, Cornerstone Earth 1/24/2017



#	FS
a	3.298
b	3.374
c	3.394
d	3.510
e	3.531
f	3.535
g	3.560
h	3.635
i	3.641
j	3.643

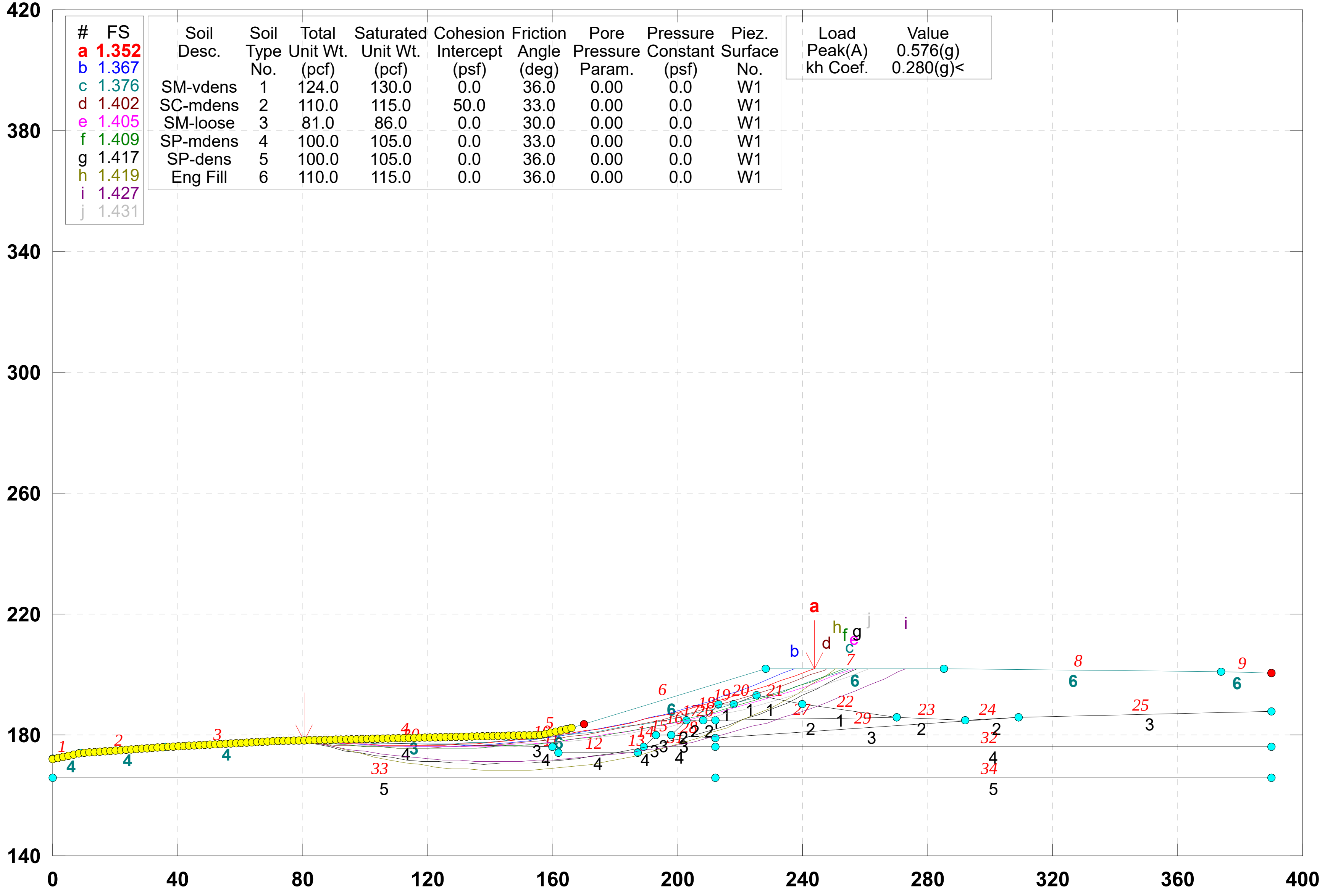
Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
SM-vdens	1	124.0	130.0	0.0	36.0	0.00	0.0	W1
SC-mdens	2	110.0	115.0	50.0	33.0	0.00	0.0	W1
SM-loose	3	81.0	86.0	0.0	30.0	0.00	0.0	W1
SP-mdens	4	100.0	105.0	0.0	33.0	0.00	0.0	W1
SP-dens	5	100.0	105.0	0.0	36.0	0.00	0.0	W1
Eng Fill	6	110.0	115.0	0.0	36.0	0.00	0.0	W1

GSTABL7 v.2 FSmin=3.298

Safety Factors Are Calculated By The Modified Bishop Method

Monterey Airport Cross-Section A-A' Seismic

p:\projects\200\234 kimley horn and associates\234-2-7 mry terminal apron and building gi\slope stability\north airport fill slope\final prelim report files\monterey airport cross-section a-a' seismic with 3 to 1 proposed fill slope.pl2   Run By: Matthew Schaffer, Cornerstone Earth   1/24/2017



#	FS
a	1.352
b	1.367
c	1.376
d	1.402
e	1.405
f	1.409
g	1.417
h	1.419
i	1.427
j	1.431

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
SM-vdens	1	124.0	130.0	0.0	36.0	0.00	0.0	W1
SC-mdens	2	110.0	115.0	50.0	33.0	0.00	0.0	W1
SM-loose	3	81.0	86.0	0.0	30.0	0.00	0.0	W1
SP-mdens	4	100.0	105.0	0.0	33.0	0.00	0.0	W1
SP-dens	5	100.0	105.0	0.0	36.0	0.00	0.0	W1
Eng Fill	6	110.0	115.0	0.0	36.0	0.00	0.0	W1

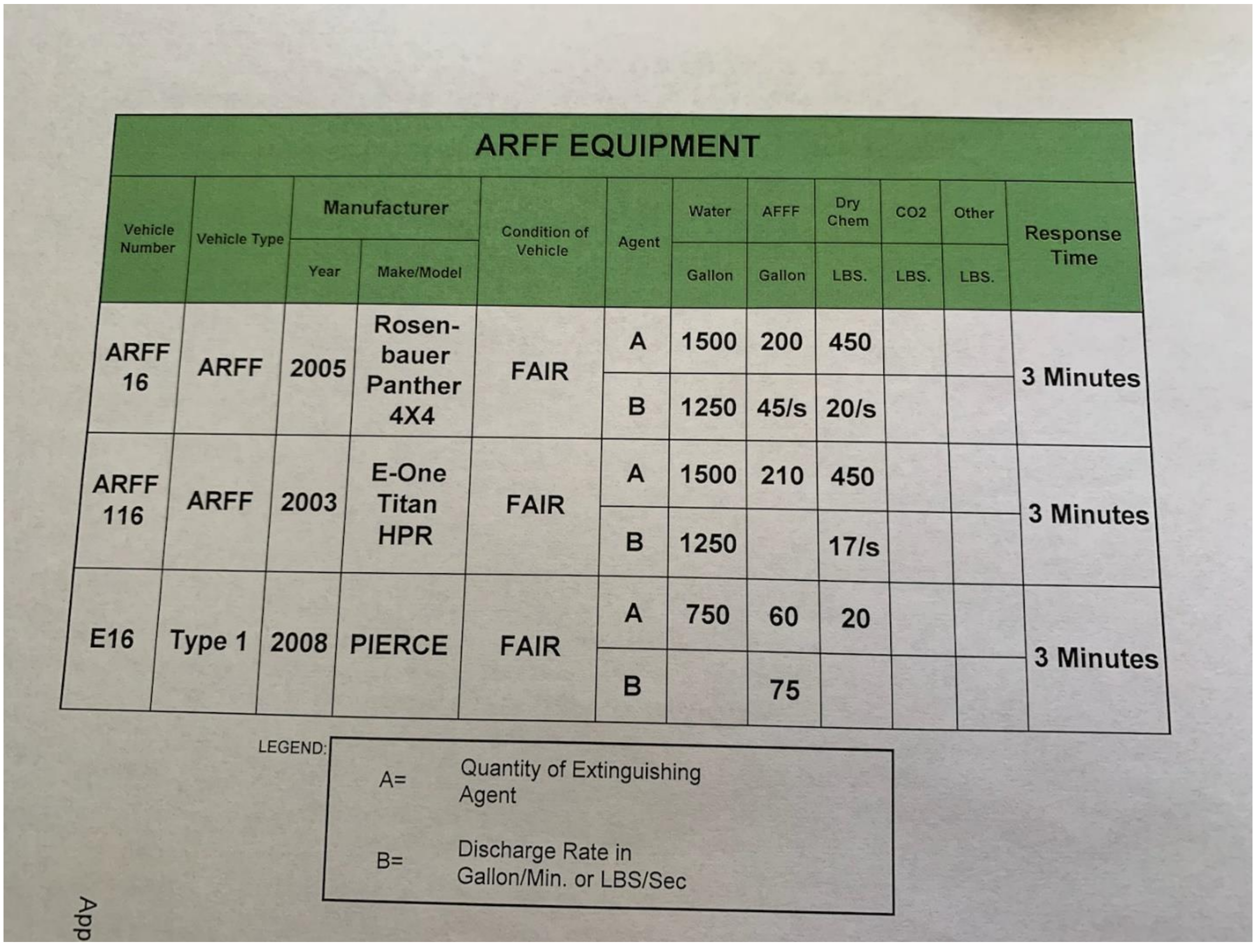
Load	Value
Peak(A)	0.576(g)
kh Coef.	0.280(g)<

GSTABL7 v.2 FSmin=1.352

Safety Factors Are Calculated By The Modified Bishop Method

Attachment No. 3

Addendum No. 2 – Existing ARFF Equipment at MRY



ARFF EQUIPMENT

Vehicle Number	Vehicle Type	Manufacturer		Condition of Vehicle	Agent	Water	AFFF	Dry Chem	CO2	Other	Response Time
		Year	Make/Model			Gallon	Gallon	LBS.	LBS.	LBS.	
ARFF 16	ARFF	2005	Rosenbauer Panther 4X4	FAIR	A	1500	200	450			3 Minutes
					B	1250	45/s	20/s			
ARFF 116	ARFF	2003	E-One Titan HPR	FAIR	A	1500	210	450			3 Minutes
					B	1250		17/s			
E16	Type 1	2008	PIERCE	FAIR	A	750	60	20			3 Minutes
					B		75				

LEGEND:

A= Quantity of Extinguishing Agent

B= Discharge Rate in Gallon/Min. or LBS/Sec

App