ADDENDUM NUMBER 02 TO THE RFP DOCUMENTS

Request for Proposals (RFP) DOCUMENT: 17/18-MB9
For Design-Build Services
Bill and Adele Jonas Center, Building 18 Alterations
College of Marin – Indian Valley Campus

Addendum Date: February 27, 2018

A. This addendum shall be considered part of the RFP documents for the above mentioned project as though it had been issued at the same time and shall be incorporated integrally therewith. Where provisions of the following supplementary data differ from those of the original bid documents, this Addendum shall govern and take precedence.

B. Proposers are hereby notified that they shall make any necessary adjustments in their estimates as a result of this Addendum. It will be construed that each bidder's proposal is submitted with full knowledge of all modifications and supplemental data specified herein.

The RFP documents are modified and clarified, as follows:

1. QUESTION: Is there a more current EIR report that addresses the Jonas Center?

RESPONSE: There is no current project level EIR, and one is being prepared. This is why the RFP states CEQA will be complete post award and the Board is not obligated to proceed with Phase 2 until after CEQA compliance.

2. QUESTION: What responsibilities will the Design team have with regards to contacting, meeting and negotiating with the Army Core of Engineers, CDFG, USFWS, NMRS, RWQCB?

RESPONSE: There should not be any, other than what is required under what has been described as the project benefiting from "grandfathering" of prior regulatory approvals.

3. QUESTION: Will the district be hiring biologists for the creek survey, bird and fish survey and wetland consultant as noted in EIR (ref. 2-6, 2-8, 2-10)?

RESPONSE: The District will need to comply with any conditions imposed by regulatory agencies for construction mitigation.

4. QUESTION: Is this a LEED silver project?

RESPONSE: No

1. 5. QUESTION: Should we be including the following in our proposal (ref brick meeting minutes with DSA dates 6/13/17):

   “common sense” evaluation of building 18 for potential weaknesses?

   RESPONSE: Yes, this evaluation should be included in the proposal. If the existing connections from the roof to the columns do not comply with DSA requirements for existing buildings, they need to be strengthened as a voluntary, partial seismic upgrade.

   6. QUESTION: Testing of existing piles. Tension or compression tests and rebar tests?

   RESPONSE: No additional rebar testing is required. Here it the excerpt from the structural memorandum (see full memorandum attached for your reference).

   If the existing piles are re-used, they must be load tested. The minutes from the DSA pre-application meeting specify the testing requirements for these piles. Also, the existing piles must be evaluated for the loads and displacements imposed by the new structure. For more information on the behavior of the existing piles, please refer to the draft geotechnical report by Geosphere Consultants, dated August 11, 2017.

   7. QUESTION: We know that the College of Marin is beginning to use a Planet Bids online procurement system. Will this procurement process utilize Planet Bids?

   RESPONSE: No.

   8. QUESTION: Is Webb Foodservice Design permitted to participate in the design-build selection process as part of one or more proposing design-build entities?

   RESPONSE: See Response question 2 of Addendum #1.

   9. QUESTION: Please confirm the pedestrian bridge over the creek that connects Parking lot 4 to the project site is excluded from the project scope.

   RESPONSE: Pedestrian Bridge is clearly noted Not in Contract (NIC).

   10. QUESTION: Regardless of the bridge being in/out of scope, will the path to & from Parking Lot 4 be part of the Projects Scope?

   RESPONSE: Project walkway/path scope is as defined by extent shown in the Architectural, Civil and Landscape criteria documents.

   11. QUESTION: Item 2.3 Experience indicates that “credit for experience…shall be based only on design-build experience and California school design and construction experience.” 9.7 Qualification Criteria, item 4 requests “a minimum of three K-14 building projects within the last five years, each in excess of $15,000,000, at least two must be approved and certified by the DSA, that involve construction and demolition of classrooms, labs, and learning environments.”

Can you please clarify if these three building projects must be California school design-build projects?

RESPONSE: response to question 13 in Addendum 1 is superseded. Qualification scoring will reflect on the submitted projects merits, experience and technical challenges compared to those expected with the Jonas Center.

12. QUESTION: Regarding the Bonding Requirements: What are the expected receivables from Marin College on bonding? Does Marin CCD have forms we need to submit?

RESPONSE: Performance and Payment Bonds must be on district forms which are attached.

13. QUESTION: Is it Marin CCD's intent to have a completed section 3. a. filling in the GMP dollar value?

RESPONSE: Yes see response to Question 4 in Addendum 1.

14. QUESTION: Is the Soils Report provided in the RFP from 2005 considered satisfactory to carry the design through approved DSA CD phase including through construction? If a revision or update is expected, on what date should it be expected to be received by the bidders?

RESPONSE: Updated Geo-hazard report attached.

15. QUESTION: Please confirm no stipend is being offered to the DBE on this RFP endeavor.

a. If a Stipend is being offered, please describe the details.

RESPONSE: No Stipend is offered.

16. QUESTION: Under Qualification Criteria, Item 9.7.4 states: "Identify and provide references for a minimum of three (3) K-14 building projects within the last (5) years, each in excess of $15,000,000, at least two must be approved and certified by DSA, that involve construction and demolition of classrooms, labs and learning environments**".** Will the following project experience/examples be considered?

a. Relevant projects for California State University or University of California?
b. Relevant projects completed within a time period greater than 5 years?
c. Projects of similar scope, but with budgets less than $15 million?

RESPONSE: Yes, response to question 13 in Addendum 1 is superseded. Qualification scoring will reflect on the submitted projects merits, experience and technical challenges compared to those expected with the Jonas Center.

17. QUESTION: Please confirm that the collective project experience of the Design-Build team will be considered (not just the architect, or not just the general contractor).

RESPONSE: Confirmed.

18. QUESTION: After reviewing the RFP for the proposal content and format, we are unclear about the requirements for the Technical Proposal. Per page 22, (9.13B) the required contents of the Technical Proposal are described under Tabs 1 through 10, with a maximum value of 250 points available. However, Section 9.7 (Qualification Criteria) describes specific criteria totaling 100 points that are not necessarily included in the requirements for the Technical Proposal. Our question is; is the District requesting a separate section of the proposal (in addition to the Technical Proposal) to describe the D-B qualifications, or are these requirements to be incorporated into the 10 Tabs of the Technical Proposal?

RESPONSE: Proposals received from firms that do not score the minimum qualification score of 75 points will be eliminated from consideration. See Response to Question 14 in Addendum 1.

19. QUESTION: Is the "Geotechnical Report prepared by Geosphere Consultants date August 11, 2017" as stated in Section 31 60 00 on Sheet S1.1 of the SD Drawings available?

RESPONSE: Yes.

20. QUESTION: Is a bid bond required with this RFP? Will the district be providing a template?

RESPONSE: Yes a proposal bond required per 2.6 of the RFP. Template will not be provided.

21 QUESTION: Article 11.2.2.- Minimum Limits of Insurance – Please confirm that the general liability and automobile liability coverage limits can be satisfied utilizing an umbrella or excess liability policy.

RESPONSE: Yes.

22. QUESTION: Article 11.2.3. – This sections references "additional insured endorsements attached" yet no forms attached.

RESPONSE: Reference included in error. All Certificates of Insurance (COI's) must name Marin Community College as additionally insured.

23. QUESTION: QUESTION: Can you provide a bid cost sheet containing blanks for all the $/% items you are requesting in Section 9.14?

RESPONSE: Design Build Proposal Form attached.

Acknowledge receipt of this addendum by signing and submitting along with your proposals which is due between 12:00pm and 2:00pm on March 20, 2018 to Fiscal Services Office, Building 8, 1800 Ignacio Boulevard, Novato, CA 94949

Name of Company: ____

Signature of authorized individual: __________________________

Name Printed: ____

Date: __________________________

End of Addendum #02

DOCUMENT 00 41 15

PROPOSAL FORM

To: Governing Board of Marin Community College District ("District" or "Owner")

From: ____

(Proper Name of Design Build Entity)

The undersigned declares that the Contract Documents including, without limitation, the Notice to Bidders and the Instructions to Proposers have been read and agrees and proposes to furnish all necessary labor, materials, and equipment to perform and furnish all work in accordance with the terms and conditions of the Contract Documents, including, without limitation, the Drawings and Specifications of RFP No. 17/18-MB-9. All section references below are to the RFP.

PROJECT: **Bill and Adele Jonas Center Building 18 Alterations Design-Build Services, Indian Valley Campus** and will accept in full payment for that Work the following proposed lump sum amount, all taxes included:

____________________________________	**dollars**	**$** ______________
Grand Total		

PHASE ONE

____________________________________	dollars	$	**%** _____
Design Preconstruction Phase *9.14C.a*			
PHASE TWO			
____________________________________	dollars	$	**%** _____
Estimated Direct Construction Cost *9.14C.b*			
____________________________________	dollars	$	**%** _____
General Conditions *9.14C.c*			
____________________________________	dollars	$	**%** _____
Overhead and Profit *9.14.C.d*			
____________________________________	dollars	$	**%** _____
Payment and Performance Bonds *9.14.C.e*			
____________________________________	dollars	$	**%** _____
Subcontractor Performance Bonds *9.14.C.f*			
____________________________________	dollars	$	
Design-Build entity Contingency *9.14.C.g.4*			**%** _____
Allowances *DBE choice to break out of direct cost of construction above 9.14.C.g.1*		$	**%**______

Descriptions of alternates are primarily scope definitions and do not necessarily detail the full range of materials and processes needed to complete the construction.

1. **Unit Prices.** The Proposer Base Bid includes the following unit prices, which the Proposer must provide and the District may, at it's discretion, utilize in valuing additive and/or deductive change orders: None

2. **Allowance.** The Owner may insert an owner allowance into the agreement for Design-Build Services in the amount of ten percent (10%) or less of the initial GMP.

   The above allowance shall only be allocated for unforeseen items relating to the Work. Contractor shall not bill for or be due any portion of this allowance unless the District has identified specific work, Contractor has submitted a price for that work or the District has proposed a price for that work, the District has accepted the cost for that work, and the District has prepared a change order incorporating that work. Contractor hereby authorizes the District to execute a unilateral deductive change order at or near the end of the Project for all or any portion of the allowance not allocated.

3. The undersigned has reviewed the Work outlined in the Contract Documents and fully understands the scope of Work required in this Proposal, understands the construction and project management function(s) is described in the Contract Documents, and the Proposer who is awarded a contract shall be in fact a prime contractor, not a subcontractor, to the District, and agrees that its Proposal, is accepted by the District, will be the basis for the Proposer to enter into a contract with the District in accordance with the intent of the Contract Documents.

4. The undersigned has notified the District in writing of any discrepancies or omissions or of any doubt, questions, or ambiguities about the meaning of any of the Contract Documents, and has contacted the Construction Manager before bid date to verify the issuance of any clarifying Addenda.

5. The undersigned agrees to commence work under this Contract on the date established in the Contract Documents and to complete all work within the time specified in the Contract Documents.

6. The liquidated damages clause of the General Conditions and Agreement is hereby acknowledged.

7. It is understood that the District reserves the right to reject this bid and that the bid shall remain open to acceptance and is irrevocable for a period of ninety (90) days.

8. The following documents are attached hereto:

   - District's forms for Performance & Payment Bonds

9. Receipt and acceptance of the following addenda is hereby acknowledged:

No.___, Dated ___________________	No.___, Dated ___________________
No.___, Dated ___________________	No.___, Dated ___________________
No.___, Dated ___________________	No.___, Dated ___________________

10. Proposer acknowledges that the license required for performance of the Work is a ___ ______ license.

11. The undersigned hereby certifies that Proposer is able to furnish labor that can work in harmony with all other elements of labor employed or to be employed on the Work.

12. Proposer specifically acknowledges and understands that if it is awarded the Contract, that it shall perform the Work of the Project while complying with all requirements of the Department of Industrial Relations [and with all requirements of the Project Labor Agreement].

13. Proposer specifically acknowledges and understands that if it is awarded the Contract, that it and its subcontractors shall participate in and comply with the owner-controlled or wrap-up insurance program (OCIP).

14. The Proposer represents that it is competent, knowledgeable, and has special skills with respect to the nature, extent, and inherent conditions of the Work to be performed. Proposer further acknowledges that there are certain peculiar and inherent conditions existent in the construction of the Work that may create, during the Work, unusual or peculiar unsafe conditions hazardous to persons and property.

15. Proposer expressly acknowledges that it is aware of such peculiar risks and that it has the skill and experience to foresee and to adopt protective measures to adequately and safely perform the Work with respect to such hazards.

16. Proposer expressly acknowledges that it is aware that if a false claim is knowingly submitted (as the terms "claim" and "knowingly" are defined in the California False Claims Act, Cal. Gov. Code, §12650 et seq.), the District will be entitled to civil remedies set forth in the California False Claim Act. It may also be considered fraud and the Contractor may be subject to criminal prosecution.

17. The undersigned Proposer certifies that it is, at the time of proposing, and shall be throughout the period of the contract, licensed by the State of California to do the type of work required under the terms of the Contract Documents and registered as a public works contractor with the Department of Industrial Relations. Proposer further certifies that it is regularly engaged in the general class and type of work called for in the Contract Documents.

Furthermore, Proposer hereby certifies to the District that all representations, certifications, and statements made by Design Build Enitiy, as set forth in this bid form, are true and correct and are made under penalty of perjury.

Dated this _______________ day of ____________________________________ 20 ___

Name of Proposer____

Type of Organization ____

Signed by ____

Title of Signer ____

Address of Proposer____

Taxpayer's Identification No. of Proposer______________________________________

Telephone Number ____

Fax Number ____

E-mail ______________________________ Web page __________________________

Contractor's License No(s): No.: ________ Class: ____________ Expiration Date: __________

No.: ________ Class: ____________ Expiration Date: __________

No.: ________ Class: ____________ Expiration Date: __________

Public Works Contractor Registration No.: ____________

If Proposer is a corporation, affix corporate seal.

Name of Corporation: ____

President: ____

Secretary: ____

Treasurer: ____

Manager: ____

END OF DOCUMENT

DOCUMENT 00 61 13.13

PERFORMANCE BOND
(100% of Contract Price)

(Note: Bidders must use this form, NOT a surety company form.)

KNOW ALL PERSONS BY THESE PRESENTS:

WHEREAS, the governing board ("Board") of the Marin Community College District, ("District") and ____ ("Principal") have entered into a contract for the furnishing of all materials and labor, services and transportation, necessary, convenient, and proper to perform the following project:

BILL AND ADELE JONAS CENTER

("Project" or "Contract") which Contract dated ____________________, 20___, and all of the Contract Documents attached to or forming a part of the Contract, are hereby referred to and made a part hereof; and

WHEREAS, said Principal is required under the terms of the Contract to furnish a bond for the faithful performance of the Contract.

NOW, THEREFORE, the Principal and ____ ____ ("Surety") are held and firmly bound unto the Board of the District in the penal sum of ____ Dollars ($________________), lawful money of the United States, for the payment of which sum well and truly to be made we bind ourselves, our heirs, executors, administrators, successors, and assigns jointly and severally, firmly by these presents, to:

- Perform all the work required to complete the Project; and
- Pay to the District all damages the District incurs as a result of the Principal's failure to perform all the Work required to complete the Project.

The condition of the obligation is such that, if the above bounden Principal, his or its heirs, executors, administrators, successors, or assigns, shall in all things stand to and abide by, and well and truly keep and perform the covenants, conditions, and agreements in the Contract and any alteration thereof made as therein provided, on his or its part to be kept and performed at the time and in the intent and meaning, including all contractual guarantees and warrantees of materials and workmanship, and shall indemnify and save harmless the District, its trustees, officers and agents, as therein stipulated, then this obligation shall become null and void, otherwise it shall be and remain in full force and virtue.

Surety expressly agrees that the District may reject any contractor or subcontractor proposed by Surety to fulfill its obligations in the event of default by the Principal. Surety shall not utilize Principal in completing the Work nor shall Surety accept a Bid from Principal for completion of the Work if the District declares the Principal to be in default and notifies Surety of the District's objection to Principal's further participation in the completion of the Work.

As a condition precedent to the satisfactory completion of the Contract, the above obligation shall hold good for a period equal to the warranty and/or guarantee period of the Contract, during which time Surety's obligation shall continue if Contractor shall fail to make full, complete, and satisfactory repair and replacements and totally protect the District from loss or damage resulting from or caused by defective materials or faulty workmanship. The obligations of Surety hereunder shall continue so long as any obligation of Contractor remains. Nothing herein shall limit the District's rights or the Contractor or Surety's obligations under the Contract, law or equity, including, but not limited to, California Code of Civil Procedure section 337.15.

The Surety, for value received, hereby stipulates and agrees that no change, extension of time, alteration, or addition to the terms of the contract or to the work to be performed thereunder or the specifications accompanying the same shall in any way affect its obligation on this bond, and it does hereby waive notice of any such change, extension of time, alteration, or addition to the terms of the Contract or to the work or to the specifications.

IN WITNESS WHEREOF, two (2) identical counterparts of this instrument, each of which shall for all purposes be deemed an original thereof, have been duly executed by the Principal and Surety above named, on the ______ day of _________________, 20___.

(Affix Corporate Seal)

____
Principal

____
By

____
Surety

____
By

____
Name of California Agent of Surety

____
Address of California Agent of Surety

____
Telephone No. of California Agent of Surety

Bidder must attach a Notarial Acknowledgment for all Surety's signatures and a Power of Attorney and Certificate of Authority for Surety. The California

Department of Insurance must authorize the Surety to be an admitted surety insurer.

END OF DOCUMENT

DOCUMENT 00 61 13.16

PAYMENT BOND
Contractor's Labor & Material Bond
(100% of Contract Price)

(Note: Bidders must use this form, NOT a surety company form.)

KNOW ALL PERSONS BY THESE PRESENTS:

WHEREAS, the governing board ("Board") of the Marin Community College District, (or "District") and ____, ("Principal") have entered into a contract for the furnishing of all materials and labor, services and transportation, necessary, convenient, and proper to perform the following project:

BILL AND ADELE JONAS CENTER

("Project" or "Contract") which Contract dated _____________________, 20___, and all of the Contract Documents attached to or forming a part of the Contract, are hereby referred to and made a part hereof; and

WHEREAS, pursuant to law and the Contract, the Principal is required, before entering upon the performance of the work, to file a good and sufficient bond with the body by which the Contract is awarded in an amount equal to one hundred percent (100%) of the Contract price, to secure the claims to which reference is made in sections 9000 through 9510 and 9550 through 9566 of the Civil Code, and division 2, part 7, of the Labor Code.

NOW, THEREFORE, the Principal and____________________________________ ("Surety") are held and firmly bound unto all laborers, material men, and other persons referred to in said statutes in the sum of ____ Dollars ($__________), lawful money of the United States, being a sum not less than the total amount payable by the terms of Contract, for the payment of which sum well and truly to be made, we bind ourselves, our heirs, executors, administrators, successors, or assigns, jointly and severally, by these presents.

The condition of this obligation is that if the Principal or any of his or its subcontractors, of the heirs, executors, administrators, successors, or assigns of any, all, or either of them shall fail to pay for any labor, materials, provisions, provender, or other supplies, used in, upon, for or about the performance of the work contracted to be done, or for any work or labor thereon of any kind, or for amounts required to be deducted, withheld, and paid over to the Employment Development Department from the wages of employees of the Principal or any of his or its subcontractors of any tier under Section 13020 of the Unemployment Insurance Code with respect to such work or labor, that the Surety will pay the same in an amount not exceeding the amount herein above set forth, and also in case suit is brought upon this bond, will pay a reasonable attorney's fee to be awarded and fixed by the Court, and to be taxed as costs and to be included in the judgment therein rendered.

It is hereby expressly stipulated and agreed that this bond shall inure to the benefit of any and all persons, companies, and corporations entitled to file claims under section 9100 of the Civil Code, so as to give a right of action to them or their assigns in any suit brought upon this bond.

Should the condition of this bond be fully performed, then this obligation shall become null and void; otherwise it shall be and remain in full force and affect.

And the Surety, for value received, hereby stipulates and agrees that no change, extension of time, alteration, or addition to the terms of Contract or the specifications accompanying the same shall in any manner affect its obligations on this bond, and it does hereby waive notice of any such change, extension, alteration, or addition.

IN WITNESS WHEREOF, two (2) identical counterparts of this instrument, each of which shall for all purposes be deemed an original thereof, have been duly executed by the Principal and Surety above named, on the ______ day of _________________, 20___.

(Affix Corporate Seal)

____
Principal

____
By

____
Surety

____
By

____
Name of California Agent of Surety

____
Address of California Agent of Surety

____
Telephone No. of California Agent of Surety

Bidder must attach a Notarial Acknowledgment for all Surety's signatures and a Power of Attorney and Certificate of Authority for Surety. The California Department of Insurance must authorize the Surety to be an admitted surety insurer.

END OF DOCUMENT

Mar
Structural Design

MEMORANDUM

To:	brick. 1266 66th Street, Suite 1 Emeryville, CA 94608	Date: Job No.:	December 14th, 2017 2017,016
Attn:	Mattison Ly	Total Pages:	1
Re:	Jonas Center and Building 18 Renovation		
By:	Ben Mohr	Filename:	2017-12-14 Evaluation of existing piles.odt
Cc:	David Mar		

Hi Mattison,

The Jonas Center will be constructed over the existing foundations of Building 19. The existing elements to remain include a slab on grade, grade beams, and drilled piles. The slab on grade and grade beams do not comply with DSA requirements, and must therefore be abandoned in place. We've spent some time investigating whether the existing piles can be re-used.

The original drawings specify that the spiral reinforcing in the piles can be either 60 ksi steel or 40 ksi steel, at the contractor's option. We concluded that, if the spirals have a yield strength of 60 ksi, the piles meet DSA requirements.

On June 12th, 2017, we attended a pre-application meeting with representatives from DSA to discuss approval requirements. During that meeting, Rich Denio (DSA) recommended that we test the spirals to determine their yield strength. On September 15th, 2017, Mar Structural design issued a memorandum providing the requirements for this testing.

On December 14th, 2017, CEL Consulting provided the test results for the spirals. The three samples each have a yield strength greater than 80 ksi. Therefore, we believe that these spirals comply with DSA requirements, and the existing piles can be re-used. However, this decision is subject to final approval by DSA.

If the existing piles are re-used, they must be load tested. The minutes from the DSA pre-application meeting specify the testing requirements for these piles. Also, the existing piles must be evaluated for the loads and displacements imposed by the new structure. For more information on the behavior of the existing piles, please refer to the draft geotechnical report by Geosphere Consultants, dated August 11, 2017.

Regardless of whether the existing piles can be re-used, some new piles will be required beneath the new lounge, as well as the new concrete walkway between the Jonas Center and Building 18. Additional piles may also be required beneath the Jonas Center, depending upon the final evaluation of the existing piles.

-Sincerely, Ben Mohr

GEOTECHNICAL ENGINEERING
AND GEOLOGIC HAZARDS STUDY

Jonas Center Project
College of Marin Indian Valley Campus
1800 Ignacio Boulevard
Novato, California 94949

Prepared for:

Marin Community College District
1800 Ignacio Boulevard
Novato, California 94949

Prepared by:

GEOSPHERE CONSULTANTS, INC.
2001 Crow Canyon Road, Suite 210
San Ramon, California 94583
Geosphere Project No. 91-03940-A

Geosphere Consultants, Inc.
AN ETS COMPANY
Geotechnical Engineering • Engineering Geology
Environmental Management • Water Resources

August 11, 2017

Marin Community College District
1800 Ignacio Boulevard
Novato, California 94949

Attention: Mr. Billy Pate, Project Manager – Measure B

Subject: ***Geotechnical Engineering and Geologic Hazards Study***
Jonas Center Project
College of Marin Indian Valley Campus
1800 Ignacio Boulevard, Novato, California 94949
Geosphere Project No. 91-03940-A

Dear Mr. Pate:

Geosphere Consultants, Inc. (Geosphere) has prepared the attached Geotechnical Engineering and Geologic Hazards Study for the Jonas Center Project, to be located at the College of Marin Indian Valley Campus at the general address of 1800 Ignacio Boulevard in Novato, California. It is our understanding that the proposed project will consist of the construction of a new banquet facility for the District's Indian Valley Campus, which will involve the reconstruction of the existing onsite Ohlone Cluster buildings to create the new center. We understand that the reconstruction will include the demolition of two existing buildings, construction of a new one-story building at the site of one of the demolished buildings, renovation of an existing building, and construction of a new paved parking lot.

Transmitted herewith are the results of our findings, conclusions, and recommendations for foundation, seismic design parameters, interior and exterior concrete slabs, site preparation, grading, foundation excavation, drainage, and utility trench backfilling. In general, the proposed improvements at the site are considered to be geotechnically as well as geologically feasible provided the recommendations of this report are implemented in the design and construction of the project.

Should you or members of the design team have questions or need additional information, please contact us at (925) 314-7180, or Mr. Dare by e-mail at cdare@geosphereinc.net. We greatly appreciate the opportunity to be of service to the Marin Community College District and to be involved in the design of this project.

Sincerely,
GEOSPHERE CONSULTANTS, INC.

PROFESSIONAL GEOLOGIST
JAMES M. JOYCE
No. 1517
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

James M. Joyce, PG, CEG
Certified Engineering Geologist

REGISTERED PROFESSIONAL ENGINEER
COREY T. DARE
No. 2013
Exp. 6-30-2018
GEOTECHNICAL
STATE OF CALIFORNIA

Corey T. Dare, PE, GE
Principal Geotechnical Engineer

Distribution: PDF to Addressee (415-612-7680); bpate@marin.edu

AL/JMJ/CTD:pmf

Geosphere Consultants, Inc.

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Geosphere Consultants, Inc.

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Geosphere Consultants, Inc.

GEOTECHNICAL ENGINEERING AND GEOLOGIC HAZARDS STUDY

Project: **Jonas Center Project**
College of Marin Indian Valley Campus
Novato, California

Client: **Marin Community College District**
Novato, California

1.0 INTRODUCTION

1.1 Purpose and Scope

The purposes of this study were to perform a geologic hazards study as required by the California Division of State Architect, as well as to evaluate the subsurface conditions at the site and prepare geotechnical recommendations for the new Jonas Center at the College of Marin Indian Valley Campus in Novato, California. This study provides recommendations for foundations, including seismic design parameters; interior and exterior concrete slabs, site preparation, grading, foundation excavation, drainage, utility trench backfilling, and pavements. This study was performed in accordance with the scope of work outlined in our proposal dated May 11, 2017.

The scope of this study included the review of available previous geotechnical and geologic literature for the site, the drilling of several subsurface borings within the project site, laboratory testing of selected samples retrieved from the borings, engineering analysis of the accumulated data, performing a geohazards study of the site in accordance with California Geological Survey (CGS) guidelines, development of geotechnical recommendations for design and construction of the project, and preparation of this report. The conclusions and recommendations presented in this report are based on the data acquired and analyzed during this study, and on prudent engineering judgment and experience. This study did not include an assessment of potentially toxic or hazardous materials that may be present on or beneath the site.

1.2 Site Description

The project site is located within the College of Marin's Indian Valley Campus with general address at 1800 Ignacio Boulevard in City of Novato, Marin County, California, as shown on *Figure 1, Site Vicinity Map*. The proposed Jonas Center Project will be located at the site of a group of three existing vacant buildings (Buildings 18, 19 and 20), referred to as the Ohlone Cluster, as shown on *Figure 2, Development Site Plan*. The Ohlone Cluster is situated adjacent to, and bounded by Ignacio Creek on the northern side of the cluster and by an unnamed tributary creek on the southeast and east side of the cluster. The southwestern side of the project area is cut into the base of an existing hill with an elevation difference on the order of five feet between the walkway on the southeast side of the complex and the pads for adjacent Buildings 18 and 20. The existing buildings consist of wood-frame, pier supported, one story buildings (Buildings 18 and 20) on the southwest portion of the cluster, and a two-story building on the northeast side of the cluster (Building 19).

The general topography of the site descends towards the creek channel, ranging from about El. 190 on the southwest side of Buildings 18 and 20 to about El. 180 at the top of the bank slopes for the adjacent creeks on the north and east side of Building 19, based on past site surveying performed for the campus by CSW Stuber-Stroeh Engineering Group (Fugro West, 2005). The bottom of the adjacent creeks are on the order of 12 to 15 feet below the level of Building 19. The coordinates of the project site used for seismic analysis were 38.0754° north latitude and 122.5792° west longitude.

1.3 Proposed Development

It is our understanding that the proposed development will consist of the select demolition and reconstruction of the group of three buildings (Buildings 18, 19, and 20) comprising the existing Ohlone Cluster group of buildings to create a new banquet facility for the campus. Specifically, we understand that existing Building 19 would be razed to the concrete floor slab, and a new one-story building constructed on the current building footprint using the existing slab and drilled pier foundations. New foundations would be added on an as-needed basis as judged by the project structural engineer. In addition, existing Building 18 would be remodeled, Building 20 would be demolished in order to construct a new paved parking lot, and additional new site infrastructure, including site utilities, flatwork, and landscape will likely be constructed or installed as part of the project.

2.0 PROCEDURES AND RESULTS

2.1 Literature Review

Available geologic and geotechnical literature pertaining to the site area was reviewed. These included various publications and maps issued by the United States Geological Survey (USGS), California Geological Survey (CGS), water agencies, and other government agencies, as listed in the References section. In addition, previous geotechnical engineering and geologic studies at this site performed by Cooper-Clark & Associates (CCA; 1967 and 1973), and Fugro West, Inc. (2005), were used as resources.

2.2 Field Exploration

A total of five borings were drilled at the site within the proposed building areas on May 25, 2017 at the approximate locations shown on *Figure 2 and Figure 3, Site Plan and Boring Location Map*. The borings were drilled to a maximum depth of approximately 25 feet below the existing ground surface in the vicinity of the proposed structure footprints using a track mounted, Mobile CME-45 drill rig equipped with a 4-inch diameter, solid flight auger.

A Geosphere staff engineer visually classified the materials encountered in the borings according to the Unified Soil Classification System as the borings were advanced. Relatively undisturbed soil samples were recovered at selected intervals using a three-inch outside diameter Modified California split spoon sampler containing six-inch long brass liners, and a two-inch outside diameter Standard Penetration Test (SPT) sampler. The samplers were driven by means of a 140-pound and 70-pound safety hammers with an approximate 30-inch fall. Resistance to penetration was recorded as the number of hammer blows required to drive the sampler the final foot of an 18-inch drive. All of the field blow counts recorded using Modified California (MC) split spoon sampler were converted in the final logs to equivalent SPT blow counts using appropriate modification factors suggested by Burmister (1948), i.e., a factor of 0.65 with inner diameter of 2.5 inches. Therefore, all blow counts shown on the final boring logs are either directly measured (SPT sampler) or equivalent SPT (MC sampler) blow counts.

The boring logs with descriptions of the various materials encountered in each boring, a key to the boring symbols, and select laboratory test results are included in Appendix A. Ground surface elevations indicated on the soil boring logs were estimates based on Google Earth Pro Software. In addition, Appendix A includes logs of borings previously performed by CCA at the Ohlone Cluster site as part of the engineering studies performed for initial development of the campus.

2.3 Laboratory Testing

Laboratory tests were performed on selected samples to determine some of the physical and engineering properties of the subsurface soils. The results of the laboratory testing are either presented on the boring logs, and/or are included in Appendix B. The following soil tests were performed for this study:

Dry Density and Moisture Content (ASTM D2216 and ASTM 2937) – In-situ dry density and/or moisture tests were conducted on 16 samples to measure the in-place dry density and moisture content of the subsurface materials. These properties provide information for evaluating the physical characteristics of the subsurface soils. Test results are shown on the boring logs.

Atterberg Limits (ASTM D4318 and CT204) - Atterberg Limits tests were performed on two samples of cohesive soils encountered at the site. Liquid Limit, Plastic Limit, and Plasticity Index are useful in the classification and characterization of the engineering properties of soil, and help to evaluate the expansive characteristics of the soil and determine the USCS soil classification. Test results are presented in Appendix B, and on the boring logs.

Particle Size Analysis (Wet and Dry Sieve) and Hydrometer (ASTM D422, D1140, and CT202) - Sieve analysis tests were conducted on two selected samples to determine the soil particle size distribution. This information is useful for the evaluation of liquefaction potential and characterizing the soil type according to USCS. Test results are presented in Appendix B.

R-Value Test (ASTM D2844 and CT301) – One R-value test was conducted on a bulk composite sample of near-surface clayey materials collected from cuttings generated from Boring B-1 between depths of one to five feet to provide data on prospective pavement subgrade materials for use in new pavement section design. Test results are presented in Section 6.9 and in Appendix B.

Soil Corrosivity, Redox (ASTM D1498), pH (ASTM D4972), Resistivity (ASTM G57), Chloride (ASTM D4327), and Sulfate (ASTM D4327) - Soil corrosivity testing was performed to determine the effects of constituents in the soil on buried steel and concrete. Water-soluble sulfate testing is required by the CBC and IBC. Soil corrosivity test results are summarized in Appendix B and are discussed in Section 4.3.

3.0 GEOLOGIC AND SEISMIC OVERVIEW

3.1 Regional Geologic Setting

The site is located in the central portion of the northern Coast Ranges geomorphic province of California. The Coast Ranges extend from the Transverse Ranges in Southern California to the Oregon border and are comprised of a northwest-trending series of mountain ranges and intervening valleys that reflect the northwest-trending faults and folds that characterize the transform boundary between the North American and Pacific plates. Translational motion along the plate boundary occurs across a distributed zone of right-lateral shear expressed as a nearly 50-mile-wide zone of northwest-trending, near-vertical active strike-slip faults. This motion occurs primarily along the active San Andreas, Hayward-Rodgers Creek, and Calaveras fault systems.

Bedrock in the Coast Ranges consists of a variably thick veneer of Cenozoic volcanic and sedimentary deposits overlying a Mesozoic basement of sedimentary, metamorphic, and basic igneous rocks of the Franciscan Assemblage and primarily marine sedimentary rocks of the Great Valley Sequence. The Coast Ranges are flanked on the east by sedimentary rocks of the Great Valley geomorphic province.

The Franciscan Assemblage is composed of weakly to strongly metamorphosed greywacke (sandstone), argillite, limestone, basalt, serpentinite, chert and other rocks. This rock was accreted onto the edge of the North American continent during the long period of active subduction of the Pacific Plate beneath the North American Plate. The formation is derived from Jurassic oceanic crust and pelagic deposits that are overlain by Late Jurassic to Late Cretaceous sedimentary deposits. Metamorphic grade in this rock is highly variable which reflects the complicated history of the Franciscan.

3.2 Local Geology

Locally, the project site is situated within the foothills of Marin County, between San Pablo Bay and the Pacific Ocean. The highland areas of the county are chiefly comprised of rocks of the Franciscan Assemblage, which underlie roughly half of southeastern Marin County. The geology of the western Novato region is shown on *Figure 4a, Site Vicinity Geologic Map*., based on Graymer et al. (2006). This map shows the site to be located within the Ignacio Creek Valley underlain by Holocene-age alluvium, with the surrounding hills underlain by Franciscan sandstone and shale, as well as Franciscan mélange complex rocks to the southwest of the site consisting of a mixture of small to large masses of various rock types, principally greywacke sandstone, greenstone (basalt), chert and serpentine in a matrix of sheared rock material.

Detailed geologic mapping of the campus was performed by Fugro West (2005), and is shown on *Figure 4b, Campus Geologic Map*. According to Fugro mapping, the hill slope facing the site from the southwest, as well as the other hill slopes on both sides of Ignacio Creek on campus are underlain by Franciscan bedrock indicated as unit KJfs on Figure 4b, generally consisting of competent blocks of greywacke sandstone interbedded with shale and siltstone. The bedrock mapped on the campus consists primarily of thin-bedded shales and siltstone with minor sandstone interbeds that strike west to northwest, dipping at angles ranging from 25° to 60° to the northeast. The rock is generally weak to friable with local moderately hard zones, and was observed by Fugro to be generally weak and friable to moderately to severely weathered.

According to the USDA – Natural Resources Conservation Service, online soil survey (Web Soil Survey, or WSS) for Marin County, the site surficial soils underlying the Jonas Center site were classified as "Urban Land – Xerorthents complex". As such, more detailed information regarding hydraulic conductivity and expansion potential for the soil classification were not provided. The upslope soils covering the adjacent hill slope and area to the southeast of the Jonas Center was classified as Tocaloma - Saurin association soils, characterized as gravelly to clay loam underlain by weathered bedrock (sandstone and shale) at shallow depth, and Hydrologic Soil Group B to C.

3.3 Geologic Evolution of the Northern Coast Ranges

The subject site is located within the tectonically active and geologically complex northern Coast Ranges, which have been shaped by continuous deformation resulting from tectonic plate convergence (subduction) beginning in the Jurassic period (about 145 million years ago). Eastward thrusting of the oceanic plate beneath the continental plate resulted in the accretion of materials onto the continental plate. These accreted materials now largely comprise the Coast Ranges. The dominant tectonic structures formed during this time include generally east-dipping thrust and reverse faults.

Beginning in the Cenozoic time period (about 25 to 30 million years ago), the tectonics along the California coast changed to a transpressional regime and right-lateral strike-slip displacements as well as thrusting were superimposed on the earlier structures resulting in the formation of northwest-trending, near-vertical faults comprising the San Andreas Fault System. The northern Coast Ranges were segmented into a series of tectonic blocks separated by major faults and fault zones including the San Andreas, Rodgers Creek, Maacama, and Hayward. The project site is situated between the active Rodgers Creek and San Andreas Faults, with the closest active fault with Holocene movement (i.e., last 11,000 years) located about 10 miles northeast of the site (Rodgers Creek Fault).

3.4 Regional Faulting and Tectonics

Regional transpression has caused uplift and folding of the bedrock units within the Coast Ranges. This structural deformation occurred during periods of tectonic activity that began in the Pliocene and continues today. The site is located in a seismically active region that has experienced periodic, large magnitude earthquakes during historic times. This seismic activity appears to be largely controlled by displacement between the Pacific and North American crustal plates, separated by the San Andreas Fault zone located on the order of 11 miles (18 km) southwest of the site. This plate displacement produced regional strain that is concentrated along major faults of the San Andreas Fault System including the San Andreas, Hayward, and Calaveras faults in the greater San Francisco Bay area.

The site is located in a seismically active region dominated by major faults of the San Andreas Fault System. Major active faults include the aforementioned San Andreas Fault; the Hayward-Rodgers Creek Fault, located about 10 miles (16 km) northeast of the site, the West Napa Fault zone, located approximately 17.5 miles (28 km) northeast of the site; the Concord-Green Valley Fault, located approximately 25 miles (40 km) northeast of the site; the Maacama Fault, located approximately 31 miles (50 km) north of the site, and the northernmost zoned portion of the Calaveras Fault, located on the order of 33 miles (53 km) southeast of the site.

According to available seismic data, the San Francisco Bay Area has been subject to as many as seven earthquakes of magnitude 6.5 or greater since 1800. The site location relative to active and potentially active faults in the San Francisco Bay Area is shown on *Figure 5, Regional Fault Map*. A discussion of these faults, ordered by increasing distance from the site, follows. Figure 5 also shows some faults (e.g., Burdell Mountain Fault, Tolay Fault) to the northeast of the project site. These faults show evidence of displacement sometime during the last 1.6 million years, but are not currently considered by CGS to be active, nor represent any fault rupture hazard to the project.

3.4.1 Hayward-Rodgers Creek Fault

The Hayward Fault trends northwesterly on the order of 88 km from the Milpitas area to San Pablo Bay. The Hayward Fault has been divided into two main segments, the Northern and Southern segments. The Rodgers Creek Fault, considered as a likely extension of the Hayward Fault, extends northward from beneath San Pablo Bay up to near Healdsburg, where it is aligned with the Healdsburg Fault zone. Recent studies in the Healdsburg area have shown that the Healdsburg Fault is Holocene-active, although it is not currently considered active by the State of California. The site is located approximately 10 miles (16 km) southwest of the Rodgers Creek Fault. The slip rate on the Rodgers Creek Fault is estimated to be about 9 mm/year and has been assigned a moment magnitude (M_{max}) of 7.0 (CGS, 2003). The Working Group on California Earthquake Probabilities (WG15) Uniform California Earthquake Rupture Forecast model UCERF3 has estimated that there is a 15 and 26 percent probability of at least one magnitude 6.7 or greater earthquake within the next 30 years along the Rodgers Creek Fault, and 18 and 26 percent probability of at least one magnitude 6.7 or greater earthquake within the next 30 years occurring along the northern and southern segments of the Hayward Fault, respectively.

3.4.2 San Andreas Fault

The northwest-trending San Andreas Fault runs along the western coast of California extending on the order of 625 miles from the north near Point Arena to the Salton Sea area in southern California (Jennings, 1994). The fault zone has been divided into 11 segments. The site is located about 11 miles (47 km) northeast of the North Coast South segment. The slip rate on the North Coast South segment of the San Andreas Fault is estimated to be about 24 mm/year and has been assigned a moment magnitude (M_{max}) of 7.4 (CGS, 2003). UCERF3 has estimated that there is a 13 and 9 percent probability of at least one magnitude 6.7 or greater earthquake within the next 30 years occurring along the North Coast South and Peninsula segments of the San Andreas Fault, respectively.

3.4.3 West Napa Fault Zone

The northwest-trending West Napa Fault zone extends from just south of American Canyon northwest to Yountville, with an older, probably inactive strand extending along the southwestern edge of the Napa Valley to the vicinity of St. Helena. The project site is located on the order of 17.5 miles (28 km) southwest of the southeastern most mapped trace of the West Napa Fault. The slip rate of the West Napa fault is estimated to be about 1 mm/year and has been assigned a moment magnitude (M_{max}) of 6.5 (CGS, 2002). UCERF3 has estimated that there is a 7 percent probability of at least one magnitude 6.7 or greater earthquake within the next 30 years occurring along the North Coast South and Peninsula segments of the San Andreas Fault, respectively.

The 2014 South Napa earthquake (magnitude 6.0) occurred on a previously unmapped trace of the West Napa Fault south and west of the City of Napa. The earthquake resulted in a ground rupture that extended south as far as the Napa Airport. Ground shaking in the Novato area was moderate. Right-lateral offsets of up to 1.3 feet were measured along the rupture zone.

3.4.4 Concord-Green Valley Fault

The north to northwest trending Green Valley Fault is thought to be an extension of the active Concord Fault, which extends from the approximate central Walnut Creek and Concord border, northward into the Green Valley

Fault. The Green Valley Fault extends northward from Suisun Bay up to just west of Lake Curry, northeast of Napa. The site is located on the order of 25 miles (40 km) southwest of the Green Valley Fault. The slip rate of the Green Valley Fault (south segment) is estimated to be about 5 mm/year and has been assigned a moment magnitude (M_{max}) of 6.2 (CGS, 2002). UCERF3 has estimated that there is a 7 and 3 percent probability of at least one magnitude 6.7 or greater earthquake within the next 30 years occurring along the Green Valley and Concord Faults, respectively.

3.4.5 Maacama Fault

The northwest-trending Maacama Fault has been mapped as extending from northeast of Santa Rosa to near Laytonville, a distance on the order of 110 miles. The southeastern end of this fault is located within 5 miles northeast of, and parallels the Rodgers Creek Fault, and may be related to or be an extension of the Hayward-Rodgers Creek fault system. The project site is located on the order of 31 miles (50 km) south of the southern end of the Maacama Fault. The slip rate of the southern segment of the fault is estimated to be about 9 mm/year and has been assigned a moment magnitude (M_{max}) of 6.9 (CGS, 2002). UCERF3 has estimated that there is a 23 percent probability of at least one magnitude 6.7 or greater earthquake within the next 30 years occurring along the southern segment of the Maacama Fault.

3.4.6 Calaveras Fault

The Calaveras Fault trends northwesterly about 123 km in length from near Hollister, extending to north of the Danville area. The Calaveras Fault has been divided into three segments, the Northern, Central, and Southern segments. The site is located on the order of 33 miles (53 km) northwest of the estimated northern end of northern segment of the Calaveras Fault. The slip rate on the north segment of the Calaveras Fault is estimated to be about 6 mm/year and has been assigned a moment magnitude (M_{max}) of 6.8 (CGS, 2003). UCERF3 has estimated that there is an 8 percent probability of at least one magnitude 6.7 or greater earthquake within the next 30 years occurring along the northern segment of the Calaveras Fault.

3.5 Historic Seismicity

As discussed above, the San Francisco Bay Area is subject to a high level of seismic activity. Within the period of 1800 to 2000 there were an estimated 20 earthquakes exceeding a Richter magnitude of 6.0 within an approximate 100 mile radius of the site, with seven exceeding 6.5, four exceeding 7.0 and one exceeding 7.5. There have been six major Bay Area earthquakes since 1800. Those were in 1836 and 1868 on the Hayward-

Rodgers Creek Fault, in 1861 on the Calaveras Fault, and in 1838, 1906, and 1989 on the San Andreas Fault.

Recent significant earthquakes have occurred on the West Napa Fault, including the M5.0 September 3, 2000 Yountville Earthquake and the August 24, 2014 M6.0 South Napa Earthquake. The South Napa Earthquake, with epicenter at Napa Valley Marina approximately 17.5 miles northeast of the project site, produced significant ground shaking in the North Bay area but no reported significant damage in the Novato area. In addition, the 1906 San Francisco Earthquake was reported to have produced very strong to severe ground shaking in the Marin and Sonoma County areas, causing extensive damage in downtown Santa Rosa, as well as occurrences of ground failures such as ground cracking in San Rafael, Petaluma and Santa Rosa; seismic settlement in Santa Rosa, and ground cracking and landslides in the Bolinas and Olema Valley areas (Youd and Hoose, 1978). No detailed accounts of ground failure in the Novato area were noted in the reviewed literature as a result of the 1906 earthquake other than lateral spreading that occurred in the vicinity of Black Point along the bay margin northeast of the site.

4.0 SUBSURFACE CONDITIONS

4.1 Subsurface Soil Conditions

During our subsurface exploration program, we investigated the subsurface soils and evaluated soil conditions to maximum depths in the borings ranging from 9 feet to 23 feet. From our collected data, we conclude that where and to the depths explored, the Jonas Center site is generally underlain by alluvial soils overlying sandstone bedrock at varying depths. Within our borings, a surficial layer of primarily clayey to silty sand was encountered, ranging in consistency from loose to dense, and extending to depths on the order of 3 to at least 15 feet. These surficial soils were underlain by layers of stiff to very stiff, lean sandy to silty clays and medium dense to dense clayey to silty sand overlying highly weathered sandstone bedrock of the Franciscan Assemblage which was encountered in southeastern side of the project site in Borings B-3 and B-4 at approximate depths of 21 and 7 feet, respectively. Previous Borings C-7 and C-6 (CCA, 1973) encountered sandstone at depths of 9.5 and 18.5 feet as well, indicating that the bedrock surface descends toward the northeast. The approximately uppermost 9.5 feet of soils consisting of medium dense to dense clayey sand observed in Boring B-2 appeared to be possible fill. Previous onsite borings by CCA also identified the surficial soils as a very stiff sandy clay.

Test results of near-surface soil samples recovered in the uppermost five feet of the native soil profile collected from Borings B-2 and B-5 indicated measured Liquid Limits of 22 and 23 and corresponding Plasticity Indices of 6 and 7. Based on these results, the near-surface soils are considered to have a low plasticity and a low expansion (shrink/swell) potential.

Our interpretations of the subsurface geologic and soil conditions are presented in Figures 6a and 6b. Additional details of materials encountered in the exploratory borings are included in the boring logs in Appendix A, and laboratory test summaries are presented in Appendix B.

4.2 Groundwater Conditions

Groundwater was encountered in Boring B-1 at a depth of 18 feet during drilling, but not in the one other boring (B-3) drilled to a deeper depth (i.e., 23 feet). Groundwater was not encountered in any of the five previous CCA borings which were drilled to a maximum depth of 25 feet. We note that the adjacent Ignacio Creek bed is on the order of 12 to 15 feet below the top of bank adjacent to Building 19. However, groundwater levels can vary in response to time of year, variations in seasonal rainfall, well pumping, irrigation, and alterations to site drainage. A detailed investigation of local groundwater conditions was not performed and is beyond the scope of this study.

4.3 Corrosion Testing

A sample collected from the upper one to four feet of the soil profile at Boring B-2 was tested to measure sulfate content, chloride content, redox potential, pH, resistivity, and presence of sulfides. Test results are included in Appendix B and are summarized on the following table.

Table 1: Summary of Corrosion Test Results

Soil Description	Sample Depth (feet)	Sulfate (mg/kg)	Chloride (mg/kg)	Redox (mV)	Resistivity (ohm-cm)	Sulfide	pH
Brown Clayey Sand w/ gravel	1-4	44	3	400	11,944	Negative	6.0

Water-soluble sulfate can affect the concrete mix design for concrete in contact with the ground, such as shallow foundations, piles, piers, and concrete slabs. Section 4.3.1 in American Concrete Institute (ACI) 318, as referenced by the CBC, provides the following evaluation criteria:

Table 2: Sulfate Evaluation Criteria

Sulfate Exposure	Water-Soluble Sulfate in Soil, Percentage by Weight or (mg/kg)	Sulfate in Water, ppm	Cement Type	Max. Water Cementitious Ratio by Weight	Min. Unconfined Compressive Strength, psi
Negligible	0.00-0.10 (0-1,000)	0-150	NA	NA	NA
Moderate	0.10-0.20 (1,000-2,000)	150-1,500	II, IP (MS), IS (MS)	0.50	4,000
Severe	0.20-2.00 (2,000-20,000)	1,500-10,000	V	0.45	4,500
Very Severe	Over 2.00 (20,000)	Over 10,000	V plus pozzolan	0.45	4,500

The water-soluble sulfate content was measured to be 44 mg/kg or 0.0044% by dry weight in the soil sample, suggesting the site soil should have negligible impact on buried concrete structures at the site. However, it should be pointed out that the water-soluble sulfate concentrations can vary due to the addition of fertilizer, irrigation, and other possible development activities.

Table 4.4.1 in ACI 318 suggests use of mitigation measures to protect reinforcing steel from corrosion where chloride ion contents are above 0.06 % by dry weight. The chloride content was measured to be 3 mg/kg or

0.0003% by dry weight in the soil sample. Therefore, the test result for chloride content does not suggest a corrosion hazard for mortar-coated steel and reinforced concrete structures due to high concentration of chloride.

In addition to sulfate and chloride contents described above, pH, oxidation reduction potential (Redox), and resistivity values were measured in the soil sample. For cast and ductile iron pipes, an evaluation was based on the 10-Point scaling method developed by the Cast Iron Pipe Research Association (CIPRA) and as detailed in Appendix A of the American Water Works Association (AWWA) publication C-105, and shown on Table 4.3.3.

Table 3: Soil Test Evaluation Criteria (AWWA C-105)

Soil Characteristics	Points		Soil Characteristics	Points
Resistivity, ohm-cm, based on single probe or water-saturated soil box.			**Redox Potential, mV**	
<700	10		>+100	0
700-1,000	8		+50 to +100	3.5
1,000-1,200	5		0 to 50	4
1,200-1,500	2		Negative	5
1,500-2,000	1		**Sulfides**	
>2,000	0		Positive	3.5
PH			Trace	2
0-2	5		Negative	0
2-4	3		**Moisture**	
4-6.5	0		Poor drainage, continuously wet	2
6.5-7.5	0		Fair drainage, generally moist	1
7.5-8.5	0		Good drainage, generally dry	0
>8.5	5			

Assuming fair site drainage, the tested soil sample had a total score of 1 point, indicating a non-corrosive rating. When total points on the AWWA corrosivity scale are at least 10, the soil is classified as corrosive to cast and ductile iron pipe, and use of cathodic corrosion protection is often recommended.

These results are preliminary, and provide information only on the specific soil sampled and tested. Other soil at the site may be more or less corrosive. Providing a complete assessment of the corrosion potential of the site soils are not within our scope of work. For specific long-term corrosion control design recommendations, we recommend that a California-registered professional corrosion engineer evaluate the corrosion potential of the soil environment on buried concrete structures, steel pipe coated with cement-mortar, and ferrous metals.

5.0 GEOLOGIC HAZARDS

5.1 Seismic Induced Hazards

Seismic hazards resulting from the effects of an earthquake generally include ground shaking, liquefaction, lateral spreading, dynamic settlement, fault ground rupture and fault creep, dam inundation, and tsunamis and seiches. The site is not necessarily impacted by all of these potential seismic hazards. These and other potential seismic and geologic hazards are discussed and evaluated in the following sections in relation to the planned construction.

5.1.1 Ground Shaking

The site will likely experience severe ground shaking from a major earthquake originating from the major active Bay Area faults, particularly the San Andreas Fault (approximately 11 miles from the site) or the Rodgers Creek Fault (approximately 10 miles from the site).

5.1.2 Liquefaction Induced Phenomena

Research and historical data indicate that soil liquefaction generally occurs in saturated, loose granular soil (primarily fine to medium-grained, clean sand deposits) during or after strong seismic ground shaking and is typified by a loss of shear strength in the affected soil layer, thereby causing the soil to flow as a liquid. However, because of the higher inter-granular pressure of the soil at greater depths, the potential for liquefaction is generally limited to the upper 40 feet of the soil. Potential hazards associated with soil liquefaction below or near a structure include loss of foundation support, lateral spreading, sand boils, and areal and differential settlement.

Lateral spreading is lateral ground movement, with some vertical component, as a result of liquefaction. The soil literally rides on top of the liquefied layer. Lateral spreading can occur on relatively flat sites with slopes less than two percent under certain circumstances. Lateral spreading can cause ground cracking and settlement. Due to type of subsurface condition and relatively level surface condition, the potential for lateral spreading is low.

The site is shown on a map on the Association of Bay Area Governments (ABAG) website as in an area of moderate susceptibility to liquefaction, as shown on *Figure 7, Liquefaction Susceptibility Map*. During our field investigation, we observed the site as being underlain predominantly by silty to clayey sands of relatively high fines content to sandy clays overlying sandstone bedrock at depths of 7 to over 20 feet below the project area. In addition, groundwater appears to be deeper than 15 feet at the site. No loose, predominantly granular materials were encountered within a potentially liquefiable zone. Therefore, we judge the potential for liquefaction and resulting settlements to be very low.

5.1.3 Dynamic Compaction (Settlement)

Dynamic compaction is a phenomenon where loose, relatively clean granular soil located above the water table densifies from vibratory loading, typically from seismic shaking or vibratory equipment. No loose granular soils of low fines content were encountered in our borings. Therefore, dynamic compaction settlement at this site should not be an issue of concern.

5.1.4 Fault Ground Rupture and Fault Creep

The State of California adopted the Alquist-Priolo Earthquake Fault Zone Act of 1972 (Chapter 7.5, Division 2, Sections 2621 – 2630, California Public Resources Code), which regulates development near active faults for the purpose of preventing surface fault rupture hazards to structures for human occupancy. In accordance with the Alquist-Priolo Act, the California Geological Survey established boundary zones or Earthquake Fault Zone surrounding faults or fault segments judged to be sufficiently active, well-defined, and mapped for some distance. Structures for human occupancy within designated Earthquake Fault Zone boundaries are not permitted unless surface fault rupture and fault creep hazards are adequately addressed in a site-specific evaluation of the development site.

The site is not currently within a designated Earthquake Fault Zone as defined by the State (Hart and Bryant, 1997). The closest Active Earthquake Fault Zone is associated with the Rodgers Creek Fault, located about 10 miles from the site, as shown on *Figure 8, Alquist-Priolo Earthquake Fault Map*. CCA (1973) during initial studies for campus development opined that an inactive fault may exist that passes through the campus along the alignment of Indian Valley, but subsequent research by Fugro West (2005) did not develop any evidence to support this hypothesis. Based on the results of previous geologic evaluations, and since the site is not within or near an Earthquake Fault Zone, the potential for fault ground rupture and fault creep hazards at the site are judged to be very low to nil.

5.2 Other Hazards

Potential geologic hazards other than those caused by a seismic event generally include ground failure and subsidence, landslides, expansive and collapsible soils, flooding, and soil erosion. These are discussed and evaluated in the following sections.

5.2.1 Ground Cracking and Subsidence

Withdrawal of groundwater and other fluids (i.e. petroleum and the extraction of natural gas) from beneath the surface has been linked to large-scale land subsidence and associated cracking on the ground surface. Other causes for ground cracking and subsidence include the oxidation and resultant compaction of peat beds, the decline of groundwater levels and consequent compaction of aquifers, hydro-compaction and subsequent settlement of alluvial deposits above the water table from irrigation, or a combination of any of these causes. Due to the absence of any of these factors, the potential for subsidence or related ground cracking is considered low.

5.2.2 Consolidation Settlement

Consolidation is the densification of soil into a more dense arrangement from additional loading, such as new fills or foundations. Consolidation of clayey soils is usually a long-term process, whereby the water is squeezed out of the soil matrix with time. Sandy soils consolidate relatively rapidly with an introduction of a load. Consolidation of soft and loose soil layers and lenses can cause settlement of the ground surface or buildings. Based on testing in the field, laboratory testing, and type of soils and depth of groundwater level, potential for consolidation settlement at this site of an extent to impact the proposed construction is judged to be low.

5.2.3 Expansive and Collapsible Soils

The result of the laboratory testing performed on representative samples of the near-surface soils indicated low plasticity soils. Hence, in our opinion, there is a low potential for expansion of the near-surface subgrade soils at this site.

The subsurface deposits encountered during the drilling program generally consisted of stiff to very stiff clay or medium dense to dense clayey sand. Collapsible soils are loose chemically bonded fine sandy and silty soils that have been laid down by the action of flowing water, usually in alluvial fan deposits. Terrace deposits and fluvial deposits can also contain collapsible soil deposits. The soil particles are usually bound together with a mineral precipitate. The loose structure is maintained in the soil until a load is imposed on the soil and water is introduced. The water breaks down the inter-particle bonds and the newly imposed loading densifies the soil. These types of soils are not present at this site. Therefore, the potential for collapsible soils underlying the site is considered to be low for this project site.

5.2.4 Landsliding

The site is shown on a map on the Association of Bay Area Governments (ABAG) website as in a lowland valley area (very few landslides) adjacent to an area of mostly landslide (the upslope area to the southwest of the site), as shown on *Figure 9, Existing Landslide Map*. As shown on Figures 4a and 4b, Fugro West (2005) in their baseline geologic hazard assessment for the campus did not identify landslides within or in close proximity to the subject project. In addition, our engineering geologist did not find evidence of landslides within the site or on the adjacent hillsides. We therefore judge that the risk of landslide within the proposed project are is low.

Fugro identified older landslide deposits in the creek drainages south of the main campus area that were derived from the underlying Franciscan mélange matrix containing such slide susceptible materials. Fugro West indicated a possibility for future smaller scale movement within the mélange matrix or existing old slide deposits, but in our opinion, any such movement would not impact the project site due to the distance of such deposits from the project location.

5.2.5 Inundation Due to Dam or Embankment Failure

The project site is not located within a previously identified potential dam failure inundation zone. Pacheco Pond, a small pond formed by a man-made embankment across Ignacio Creek is located within the Indian Valley Open Space about 0.5 miles upstream of the project site. Fugro West (2005) opined in their geologic assessment of the campus that any breach of this embankment would be contained within the banks of Ignacio Creek and not impact the campus. We note that if still a concern, a hydrologic assessment, if not previously performed, should be performed by a qualified civil engineer.

5.2.6 Flooding Hazard

The site is located in an area indicated as Zone X, or areas determined to be outside the 0.2% annual chance floodplain, as shown on *Figure 10, Flood Hazard Map*. Fugro West (2005) noted a previous study performed in 1967 during initial studies for the campus that indicated a computed maximum flood rate of Ignacio Creek of 500 cubic feet per second through the campus area, which was concluded at that time that the existing channel would carry that flow. Therefore, to our knowledge, no potential flooding hazard has previously been indicated at the site.

5.2.7 Soil Erosion

Present construction techniques and agency requirements have provisions to limit soil erosion and resultant siltation during construction. These measures will reduce the potential for soil erosion at the site during the various construction phases. Long-term erosion at the site will be reduced by landscaping and hardscape areas, such as parking lots and walkways, designed with appropriate surface drainage facilities.

Soil erosion of the creek banks adjoining the site due to very high creek flows is possible at this site, but evaluation of the hydrology of potential flows was not within the scope of this study, and although the creek banks are in relatively close proximity to the existing buildings (about 15 feet at isolated locations), the adjacent existing buildings are all supported by deep pier foundations extending to elevations near creek bottom elevations, so creek bank erosion should any such occur, is not expected to impact the performance of the existing buildings.

5.2.8 Naturally Occurring Asbestos (NOA)

The borings did not encounter any soils which are a concern for potential asbestos hazard. Additionally this site is not known to have past history or potential for NOA hazard.

5.2.9 Other Geologic Hazards

Due to the site location, subsurface soil conditions, groundwater levels and land use factors, in our opinion, the site is not subject to other potential geologic hazards such as tsunamis or seiches, loss of mineral resources, volcanism, cyclic softening of soils, or loss of unique geologic features.

6.0 CONCLUSIONS AND RECOMMENDATIONS

The following conclusions and recommendations are based upon the analysis of the information gathered during the course of this study and our understanding of the proposed improvements.

6.1 Conclusions

The site is considered geotechnically suitable for the proposed improvements provided the recommendations of this report are incorporated into the design and implemented during construction. The predominant geotechnical and geological issues that need to be addressed at this site are summarized below.

Seismic Ground Shaking – The site is located within a seismically active region. As a minimum, the building design should consider the effects of seismic activity in accordance with the latest edition of the California Building Code (CBC-2016).

Undocumented Fill – Undocumented fill was identified within the upper portion of one boring located closest to the top of the creek bank, east of Building 19. In general, undocumented fills should not be relied upon for structural support of significant buildings, but no proposed new foundation construction is located in this area other than possible new pier foundations which would derive supporting capacity below possible fill zones.

Winter Construction – If any grading occurs in the winter rainy season, appropriate erosion control measures will be required, and weatherproofing of exposed building pads, foundation excavations, and/or new pavement areas should be considered. Winter rains may also impact underground utilities.

Other potential geotechnical considerations, including those that should not significantly impact the project are explained below.

Groundwater – Groundwater was not encountered above a depth of 15 feet in any of our field explorations, so is not expected to be problematic during construction.

Expansive Soils – Expansive surficial soils were not encountered during the present and previous investigations at the site. Therefore, mitigative measures for expansive soils are not expected to be required at this site.

Utility Connections – As a general suggestion, where utility damage during a design seismic event may be an issue, the Structural Engineer may wish to consider using utility connections at building perimeters designed for up to one inch of potential movement in any direction where the utility enters the buildings. This flexibility would help accommodate potential differential movement during a seismic event.

6.2 Seismic Design Parameters

The proposed structures should be designed in accordance with local design practice to resist the lateral forces generated by ground shaking associated with a major earthquake occurring within the greater San Francisco Bay region. Based on the subsurface conditions encountered in our borings we judge Site Class "C", representative of the uppermost 100 feet of the subsurface profile to be appropriate for this site. For design of the proposed site structures in accordance with the seismic provisions of the CBC 2016 and American Society of Civil Engineers (ASCE) 7-10, the following seismic ground motion values should be used as a minimum for design.

Table 4: Seismic Coefficients Based on 2016 CBC (per ASCE 7-10)

Item	Value	2016 CBC Source[R1]	ASCE 7-10 Table/Figure[R2]
Site Class	C	Table 1613.3.2	Table 20.3-1
Mapped Spectral Response Accelerations			
Short Period, S_s	1.500 g		Figure 22-1
1-second Period, S_1	0.600 g		Figure 22-2
Site Coefficient, F_a	1.0	Table 1613.3.3(1)	Table 11.4-1
Site Coefficient, F_v	1.3	Table 1613.3.3(2)	Table 11.4-2
MCE (S_{MS})	1.500 g	Equation 16-37	Equation 11.4-1
MCE (S_{M1})	0.780 g	Equation 16-38	Equation 11.4-2
Design Spectral Response Acceleration			
Short Period, S_{DS}	1.000 g	Equation 16-39	Equation 11.4-3
1-second Period, S_{D1}	0.520 g	Equation 16-40	Equation 11.4-4
Peak Ground Acceleration, PGA_M	0.50 g		Equation 11.8-1

R1 California Building Standards Commission (CBSC), "California Building Code," 2016 Edition.
R2 U.S. Seismic "Design Maps" Web Application, https://geohazards.usgs.gov/secure/designmaps/us/application.php

ASCE 7-15 § 11.6-1 and 11.6-2 indicate that the Seismic Design Category for all Occupancy Categories is "D".

6.3 Site Grading and Site Preparation

6.3.1 General Grading, Demolition, Preparation, and Drainage

Site grading should be performed in accordance with these recommendations. A pre-construction conference should be held at the jobsite with representatives from the owner, general contractor, grading contractor, and Geosphere prior to starting the clearing and demolition operations at the site.

Site grading at this site is generally anticipated to consist of minor to moderate grading to construct the new parking lot as well as finish grading of new common and walkway areas and for other new site infrastructure. Import fill required as backfill for this project should be non-expansive, having a Plasticity Index of 12 or less, an R-Value greater than 40, and enough fines so the soil can bind together but not more than 20 percent. Imported soils should be free of organic materials and debris, and should not contain rocks or lumps greater than three inches in maximum size. The Geotechnical Engineer should approve imported fill prior to delivery onsite.

Existing excavated onsite soils generated following clearing and grubbing that are free of excess organic material (three percent or less by weight) or debris is suitable for reuse as structural fill at the site, as approved by the Geotechnical Engineer. Removed old pavement materials may be reused as structural fill provided the material is broken up to meet the size requirements for import fill, non-expansive, and is free of environmental contaminants.

Demolition of existing structures such as Building 20 and possibly some underground utilities at the project site will or may be required. Prior to commencement of grading activities, all existing pavements and hardscape to be removed, as well as foundation remnants, utilities, trees and roots, surface vegetation, organic-laden soils, building materials, existing loose soil, concrete, debris and other deleterious materials should be cleared. Debris resulting from site stripping operations should be removed from the site, unless otherwise permitted by the Geotechnical Engineer.

Excavations resulting from the removal of abandoned underground utilities, or deleterious materials should be cleaned down to firm soil, processed as necessary, and backfilled with engineered fill in accordance with the grading sections of this report. The Geotechnical Engineer's representative should verify the adequacy of site clearing operations during construction, prior to placement of engineered fill, and all engineered fill placed by the demolition contractor within future building areas should be observed and tested by the geotechnical engineer.

Existing underground utilities proposed to be abandoned, if present, should be properly grouted, closed, or removed as needed. The extent of removal/abandonment depends on the diameter of the pipe, depth of the pipe, and proximity to buildings and pavement.

Final grading should be designed to provide drainage away from structures, and from the top of slopes such as the Indian Creek bank unless such flows are dissipated or evaluated on a location specific basis. Exposed soil areas within 10 feet of proposed structures should slope at a minimum of five percent away from the building. Adjacent concrete hardscapes should slope a minimum two percent away from the building.

6.3.2 Project Compaction Recommendations

The following table provides the recommended compaction requirements for this project. Not all soils, aggregates and scenarios listed below may be applicable for this project. Specific grading recommendations are discussed individually within applicable sections of this report.

Table 5: Project Compaction Recommendations

Description	Min. Percent Relative Compaction (per ASTM D1557)	Percent Above/below Optimum Moisture Content
Fill Areas, Engineered Fill, Onsite Soil	90	+ 3
Fill Areas, Engineered Fill, Import Fill	90	+ 2
Building Pads, Onsite Soil – Scarified Subgrade or used as Fill	90	+ 3
Building Pads – Chemically Treated Soil	93	± 3
Building Pads – Baserock	90	+ 2
Concrete Flatwork, Subgrade Soil	90	+ 3
Concrete Flatwork, Baserock	90	± 3
Underground Utility Backfill – Building pad and flatwork areas	90	+ 3
Underground Utility Backfill - Below 3 feet in pavement areas	90	+ 3
Underground Utility Backfill - Upper 3 feet below pavements	95	+ 3
Underground Utility Backfill – Sand backfill	95	+ 3
AC Pavement – Onsite Subgrades (upper 8 inches)	95	+ 3
Pavement – Class 2 Aggregate Base Section	95	± 2

6.3.3 Structural Pads

New pad grading is not anticipated at Building 19 as the existing building slab and foundations are expected to be reused. However, if any other new structures requiring graded pads be added to the project, the following grading recommendations would apply.

New pad subgrade soil should be scarified to a depth of at least eight inches, moisture conditioned as needed, and compacted to the project compaction requirements listed on Table 5 as determined based on ASTM D1557 (Modified Proctor). If loose or soft soil is encountered, these soils should be removed to expose firm soil and backfilled with engineered fill. New fill should be moisture conditioned and thoroughly mixed during placement to provide uniformity in each layer. In order to achieve satisfactory compaction of the subgrade and fill materials, it may be necessary to adjust the water content at the time of construction. This may require that water be added to soils that are too dry, or that scarification and aeration be performed in any soils that are too wet. Engineered fill if required to reach pad subgrade elevation should be placed in maximum eight-inch thick, un-compacted lifts prior to processing and compacting.

The completed pad surface should be firm and unyielding and should be protected from damage caused by traffic or weather. Soil subgrades should be kept moist during construction.

6.3.4 Grading Flatwork and Pavement Areas

Areas to receive flatwork or pavements should be scarified to a depth of eight inches below existing grade or final subgrade, whichever is lower. Scarified areas should be moisture conditioned and compacted. Where required, engineered fill should be placed and compacted to reach design subgrade elevation. Once the compacted pavement subgrade has been reached, it is recommended that baserock in paved and on-grade concrete slab areas be placed as soon as practical after grading to protect the subgrade soil from drying. Alternatively, the subgrade should be kept moist by watering until baserock is placed.

Rubber-tired heavy equipment, such as a full water truck, should be used to proof load exposed subgrade areas where pumping is suspected. Proof loading will determine if the subgrade soil is capable of supporting construction equipment without excessive pumping or rutting.

6.3.5 Site Winterization and Unstable Subgrade Conditions

If grading occurs in the winter rainy season, unstable and unworkable subgrade conditions may be locally present and compaction of onsite soils may not be feasible. These conditions may be remedied using soil admixtures, such as lime-cement. A four percent mixture of lime-cement based on a dry soil unit weight of 100 pcf is recommended for planning purposes. Treatment may vary between 12 to 18 inches, depending on the anticipated construction equipment loads. More detailed and final recommendations can be provided during construction if needed. Stabilizing subgrade in small, isolated areas can be accomplished with the approval of the Geotechnical Engineer by over-excavating one foot, placing Tensar TriAx TX140 or equivalent geogrid on the soil, and then placing 12-inches of Class 2 baserock on the geogrid. The upper six inches of the baserock should be compacted to at least 90 percent (building pads) or 95 percent (pavement subgrades) relative compaction.

6.4 Utility Trench Construction

6.4.1 Trench Backfilling

Utility trenches may be backfilled with onsite selected soil above the utility bedding and shading materials. If rocks or concrete larger than four inches in maximum size are encountered, they should be removed from the fill material prior to placement in the utility trenches. Utility bedding and shading compaction requirements should be in conformance with the requirements of the local agencies having jurisdiction and as recommended by the pipe manufacturers. Jetting of trench backfill is not recommended. Compaction recommendations are presented in Table 5.

Pea gravel, rod mill, or other similar self-compacting material should not be utilized for trench backfill since this material will transmit the shallow groundwater to other locations within the site and potentially beneath the buildings. Additionally, fines may migrate into the voids in the pea gravel or rod mill, which could cause settlement of the ground surface above the trench.

If rain is expected and the trench will remain open, the bottom of the trench may be lined with one to two inches of gravel. This would provide a working surface in the trench bottom. The trench bottom may have to be sloped to a low point to pump the water out of the trench.

6.4.2 Utility Penetrations at Building Perimeter

Utility trenches should be sealed with concrete, clayey soil, sand-cement slurry, or controlled density fill (CDF) where the utility enters the building under the perimeter foundation. This would reduce the potential for migration of water beneath the building through the shading material in the utility trench.

As a general suggestion, flexible connections at building perimeters may be desired for critical utility lines going through perimeter foundations. This would provide flexibility during a seismic event. This could be provided by special flexible connections, pipe sleeving with appropriate waterproofing, or other methods.

6.4.3 Pipe Bedding and Shading

Pipe bedding material is placed in the utility trench bottom to provide a uniform surface, a cushion, and protection for the utility pipe. Shading material is placed around the utility pipe after installation and testing to protect the pipe. Bedding and shading material and placement are typically specified by the pipe manufacturer, agency, or project designer. Agency and pipe manufacturer recommendations may supersede our suggestions. These suggestions are intended as guidelines and our opinions based on our experience to provide the most cost-effective method for protecting the utility pipe and surrounding structures. Other geotechnical engineers, agency personnel, contractors, and civil engineers may have different opinions regarding this matter.

Bedding and Shading Material - The bedding and shading material should be the same material to simplify construction. The material should be clean, uniformly graded, fine to medium grained sand. It is suggested that bedding and shading material contain less than three percent fines with 100 percent passing the No. 8 sieve. Coarse sand, angular gravel or baserock should be avoided since this type of shading material may bridge when backfilling around the pipe, possibly creating voids, and may be too stiff as bedding material. Open graded gravel should be avoided for shading since this material contains voids, and the surrounding soil could wash into the voids, potentially causing future ground settlement. However, open graded gravel may be required for bedding material when water is entering the trench. This would provide a stable working surface and a drainage path to a sump pit in the trench for water in the trench. The maximum size for bedding material should be limited to about ¾ -inch.

Bedding Material Placement - The thickness of the bedding material should be minimized to reduce the amount of trench excavation, soil export, and imported bedding material. Two to three inches for pipes less than eight-inches in diameter and about four to six inches for larger pipes are suggested. Bedding for very large diameter pipes are typically controlled by the pipe manufacturer. Compaction is not required for thin layers of bedding material. The pipe needs to be able to set into the bedding, and walking on a thin layer of bedding material should sufficiently compact the sand. Rounded gravel may be unstable during construction, but once the pipe and shading material is in place, the rounded gravel will be confined and stable.

Shading Material Placement – Jetting is not recommended since the type of shading material is unknown when preparing the geotechnical report and agencies typically do not permit jetting. The shading material should be able to flow around and under the utility pipe during placement. Some compactive effort along the sides of the pipe should be made by the contractor to consolidate the shading material around the pipe. A minimum thickness of about six inches of shading material should be placed over the pipe to protect the pipe from compaction of the soil above the shading material. The contractor should provide some compactive effort to densify the shading material above the pipe. Relative compaction testing is not usually performed on the shading material. However, the contractor is ultimately responsible for the integrity of the utility pipe.

6.5 Temporary Excavation Slopes

Where temporary excavation slopes are required, the Contractor should incorporate all appropriate requirements of OSHA/ Cal OSHA into the design of any temporary construction slopes used during construction. Excavation safety regulations are provided in the OSHA Health and Safety Standards for Excavations, 29 CFR Part 1926, Subpart P, and apply to excavations greater than five feet in depth.

The Contractor, or his specialty subcontractor, should design temporary construction slopes to conform to the OSHA regulations and should determine actual temporary slope inclinations based on the subsurface conditions exposed at the time of construction. For pre-construction planning purposes, the subsurface materials in the areas of the site where excavation may take place may be assumed to consist of a stiff clay-sand mix categorized as OSHA Type B with temporary slope inclination of no steeper than 1:1 (horizontal to vertical). This maximum slope ratio is assumed to be uniform from top to toe of the slope. The type of slope material and actual temporary construction slopes should be confirmed or adjusted during construction by a person who is trained as a "competent person" as designated by OSHA and directly responsible to the grading contractor.

If temporary slopes are left open for extended periods of time, exposure to weather and rain could have detrimental effects such as sloughing and erosion on surficial soils exposed in the excavations. We recommend that all vehicles and other surcharge loads be kept at least 10 feet away from the top of temporary slopes, and that such temporary slopes are protected from excessive drying or saturation during construction. In addition, adequate provisions should be made to prevent water from ponding on top of the slope and from flowing over the slope face. Desiccation or excessive moisture in the excavation could reduce stability and require shoring or laying back side slopes.

6.6 Building Foundation Recommendations

6.6.1 Drilled Pier Foundations

We understand that the existing 30-inch diameter drilled pier foundations will be used to support the new replacement two-story structure on the footprint of Building 19, and it is possible that additional new drilled pier foundations may be required. For evaluation of existing piers and for design of new piers, the piers may be assumed to derive their supporting capacity by skin friction between the perimeter of the piers and the surrounding subsurface soils, with additional supporting capacity if needed to be provided by end bearing resistance of the piers in soil once skin friction resistance is fully mobilized.

Piers should have a minimum embedment length of 10 feet. Pier foundation axial capacities should be evaluated using an allowable skin friction value of 400 pounds per square foot (psf) up to an embedment depth of 10 feet, a 500 psf value between depths of 10 and 20 feet, and an 800 psf value below a depth of 20 feet. In addition, an additional allowable end bearing resistance of 9 kips per square foot may be used assuming the bottoms of the pier holes are/were cleaned of loose slough and disturbed material, and an additional settlement on the order of ½ to 1 inch occurs to mobilize the allowable end resistance. The aforementioned capacities may be increased by one-third for transient (wind or seismic) loads.

End bearing capacity should be ignored when computing allowable uplift capacities. In addition, the computed capacity from the uppermost one foot of embedment should be neglected. Piers should have a minimum center-to-center spacing of three times the shaft diameter. Pier reinforcing should be based on structural requirements.

Following drilling, the bottoms of the pier excavations should be relatively dry, and free of all loose cuttings or slough prior to placing reinforcing steel and concrete. Any accumulated water in pier excavations should be removed prior to placing concrete.

6.6.2 Shallow Spread Foundations

If desired, new lightly loaded structures or minor structures such as storage sheds, and other structural elements such as retaining walls can be supported on conventional spread footings bearing on well compacted firm subgrade soils. Footings should be founded a minimum of 24 inches below lowest adjacent finished grade. Continuous footings should have a minimum width of at least 24 inches, and isolated column footings should have a minimum width of 30 inches. Footings located adjacent to other footings or utility trenches should bear below an imaginary 1.5:1 (horizontal to vertical) plane projected upward from the bottom edge of the adjacent footings or utility trenches. Footing reinforcement should be determined by the project Structural Engineer.

For the design of footings bearing on approved competent native soils or engineered fill, we recommend the following allowable net bearing pressures, assuming design Factors-of-Safety of 3.0, 2.0 and 1.5 for dead loads, dead plus live loads, and total loads including transient loads, respectively, from the estimated ultimate bearing capacity.

Table 6: Allowable Bearing Pressures for Spread Footings for Minor Structures

Load Condition	Allowable Bearing Pressure (psf)
Dead Load	1,500
Dead plus Live Loads	2,250
Total Loads (including wind or seismic)	3,000

Pressures presented in Table 6 are net pressures, meaning that the weight of the foundations below grade have already been incorporated into the allowable pressures presented in the table.

Geosphere personnel should be retained to observe and confirm that footing excavations prior to formwork and reinforcing steel placement bear in soils suitable for the recommended maximum design bearing pressure. If unsuitable soil such as unanticipated fill is present, the excavation should be deepened until suitable supporting, undisturbed native material is encountered. The over-excavation should be backfilled using structural or lean concrete (or a sand-cement slurry mix acceptable to the Geotechnical Engineer) up to the bottom of the footing concrete.

Footing excavations should have firm bottoms and be free from excessive slough prior to concrete or reinforcing steel placement. Care should also be taken to prevent excessive wetting or drying of the bearing materials during construction. Extremely wet or dry or any loose or disturbed material in the bottom of the footing excavations should be removed prior to placing concrete. If construction occurs during the winter months, a thin layer of concrete (sometimes referred to as a rat slab) could be placed at the bottom of the footing excavations. This will protect the bearing soil and facilitate removal of water and slough if rainwater fills the excavations.

If site preparation and foundation observation services are conducted as outlined in this Geotechnical Study report, vertical settlement is not expected to exceed more than one inch for footings bearing within the materials described in the report and designed to the aforementioned allowable bearing pressures. Differential settlement between footings of similar loading is generally not expected to exceed ½ to ⅔ the total settlement within an assumed column bay distance of 30 feet.

6.6.3 Lateral Resistance

Lateral load resistance for pile-supported structures may be developed through pile deflection and soil interaction, and is influenced by such factors as pile and soil stiffness, embedment length, type of connection at the top of pile, pile yield moment capacity, and allowable lateral top deflection. Lateral pile capacity was evaluated for both existing and potential new pier foundations based on the p-y method using computer program LPILE Plus 5.0 (Ensoft, 2004). Pile response was evaluated for the existing 30-inch diameter, 13.5 and 16.5 foot-long drilled pier foundations for Building 19 as well as for new 18-inch diameter, 16.5-foot long piles under various axial and lateral loads prescribed by the project structural engineer, for both fixed head and free (pinned) head conditions. Plots of lateral deflection versus depth for existing 30-inch diameter piles under fixed head conditions are presented in Figures 11a and 11b, and for free head conditions in Figures 11c and 11d. Plots of shear force versus depth for existing 30-inch diameter piles under fixed head conditions are presented in Figures 12a and 12b, and for free head conditions in Figures 12c and 12d. Plots of bending moment versus depth for existing 30-inch diameter piles under fixed head conditions are presented in Figures 13a and 13b, and for pinned head conditions in Figures 13c and 13d. Plots of lateral deflection, shear force and bending moment versus depth for a new 18-inch diameter pile under both fixed and free head conditions with applied lateral loads of 7, 10 and 15 kips are presented in Figures 14a, 14b and 14c, respectively.

Shallow footing foundations can resist lateral loads with a combination of bottom friction and passive resistance. An allowable coefficient of friction of 0.35 between the base of the foundation elements and underlying material is recommended. In addition, an allowable passive resistance equal to an equivalent fluid weighing 300 pounds per cubic foot (pcf) acting against the foundation may be used for lateral load resistance against the sides of footings perpendicular to the direction of loading where the footing is poured neat against undisturbed material. For simple pier foundations such as those for overhead light structures, fence posts or signs, lateral resistance may be determined for onsite, unimproved soils using the aforementioned allowable passive resistance acting across 1½ times the pier diameter. The top foot of passive resistance at pier or footing foundations not adjacent to and confined by pavement, interior floor slab, or flatwork should be neglected. In order to fully mobilize this passive resistance, a lateral deflection on the order of one to two percent of the embedment of the footing or pier is required. If it is desired to limit the amount of lateral deflection to mobilize the passive resistance, a proportional safety factor should be applied. The friction between the bottom of a slab-on-grade floor and the underlying soil should not be utilized to resist lateral forces.

6.7 Interior Slabs-on-Grade

6.7.1 Concrete Floor Slabs

New non-structural concrete slab-on-grade floors, if any, should be a minimum of five-inches in thickness and should be reinforced as a minimum by No. 4 steel reinforcement placed at 18-inch centers each way. However, the actual thickness and reinforcing of the slab should be designed by the Structural Engineer.

Slab-on-grade concrete floors with moisture sensitive floor coverings may require protection from moisture transmission through the slab from the underlying subgrade soils. Geotechnical engineers are not experts in the protection of floor coverings from underslab moisture, and if of significant importance, an expert in concrete slab construction familiar with moisture transmission issues through concrete slabs should be consulted for specific slab moisture protection design. However, we provide the following general discussion on typical types of moisture protection used in local construction.

Primary protection from moisture transmission through floor concrete is typically provided by a moisture retarder consisting of a relatively impermeable vapor retarder placed between the subgrade soil and the bottom of the concrete slab. A capillary break consisting of at least four inches of free-draining gravel, such as ¾-inch, clean, crushed, uniformly graded gravel with less than three percent passing No. 200 sieve, or equivalent, has also been used by designers below the vapor retarder. The vapor retarder should be at least 10-mil thick and should conform to the requirements for ASTM E 1745 Class C Underslab Vapor Retarders (e.g., Griffolyn Type 65, Griffolyn Vapor Guard, Moistop Ultra C, or equivalent). If additional protection is desired by the owner, a higher quality vapor barrier conforming to the requirements of ASTM E 1745 Class A, with a water vapor transmission rate less than or equal to 0.006 gr/ft^2/hr (i.e., 0.012 perms) per ASTM E 96 (e.g., 15-mil thick "Stego Wrap Class A"), or to Class B (Griffolyn Type 85, Moistop Ultra B, or equivalent) may be used in place of a Class C retarder.

The vapor retarder or barrier should be placed directly under the slab. A sand layer is not required over the vapor retarder from a geotechnical standpoint. If sand on top of the vapor retarder is required by the design structural engineer, we suggest the thickness be minimized to less than one inch. If construction occurs in the winter months, water may pond within the sand layer since the vapor retarder may prevent the vertical percolation of rainwater.

ASTM E1643 should be utilized as a guideline for the installation of the vapor retarder. During construction, all penetrations (e.g., pipes and conduits,) overlap seams, and punctures should be completely sealed using awaterproof tape or mastic applied in accordance with the vapor retarder manufacturer's specifications.

The vapor retarder or barrier should extend to the perimeter cutoff beam or footing.

6.7.2 Exterior Concrete Flatwork (Non-Vehicular)

Exterior concrete flatwork intended for pedestrian traffic should be at least four-inches thick and supported on either compacted native subgrade or a baserock layer constructed in accordance with the applicable recommendations presented in Sections 6.3.2 and 6.3.4.

6.8 Retaining Walls

6.8.1 Lateral Earth Pressures

The following recommended lateral earth design pressures are based on the assumption that approved on-site soils will be used as wall backfill. For a level backfill condition, unrestrained walls (i.e., walls that are free to deflect or rotate) should be designed to resist an equivalent fluid pressure of 35 pounds per cubic foot. Restrained walls for a level backfill condition should be designed to resist an equivalent fluid pressure of 35 pounds per cubic foot, plus an additional uniform lateral pressure of 7H pounds per square foot, where H = height of backfill above the top of the wall footing, in feet. If required for seismic design, unrestrained walls and restrained walls with level backfill should be designed to resist an additional uniform load equal to 18H psf, where H equals the height of soil retained by the wall in feet. The seismic load should be added to the *unrestrained* condition in both cases.

Walls with inclined backfill should be designed for an additional equivalent fluid pressure of one pound per cubic foot for every two degrees of slope inclination from horizontal. Walls subjected to surcharge loads should be designed for an additional uniform lateral pressure equal to 0.33 times the anticipated surcharge load for unrestrained walls, and 0.50 times the anticipated surcharge load for restrained walls.

The lateral earth pressures herein do not include any factor-of-safety and are not applicable for submerged soils/hydrostatic loading. Additional recommendations may be necessary if submerged conditions are to be included in the design.

6.8.2 Wall Drainage

The aforementioned recommended lateral pressures assume that walls are fully back drained to prevent the build-up of hydrostatic pressures. Unless potential hydrostatic loading behind the wall is accounted for in wall design, a subsurface drain system should be constructed behind the walls. The drain system should consist of a minimum 12-inch width of free-draining granular soils containing less than five percent fines (by weight) passing a No. 200

sieve placed adjacent to the wall. The free-draining granular material should be graded to prevent the intrusion of fines (e.g., a Caltrans Class 2 permeable material) or encapsulated in a suitable filter fabric. A drainage system consisting of either weep holes or perforated drain lines (placed near the base of the wall) should be used to intercept and discharge water which would tend to saturate the backfill. Where used, drain lines should be embedded in a uniformly graded filter material and provided with adequate clean-outs for periodic maintenance if desired. An impervious soil should be used in the upper layer of backfill to reduce the potential for water infiltration. As an alternative, a prefabricated drainage structure such as a geocomposite drain (e.g., MiraDRAIN 6000) may be used as a substitute for the granular backfill adjacent to the wall. Wall drainage collector pipes should be sloped to discharge by gravity to an adjacent storm drain system or other appropriate facility.

6.8.3 Wall Backfill Construction

Below-grade wall structural backfill less than five feet deep should be compacted to at least 90 percent relative compaction using light compaction equipment. Structural backfill greater than a depth of five feet should be compacted to at least 95 percent relative compaction. If heavy compaction equipment is used, the walls should be appropriately designed to withstand loads exerted by the heavy equipment, and/or temporarily braced. Over compaction or surcharge from heavy equipment too close to the wall may cause excessive lateral earth pressures which could result in outward wall movement.

6.9 Pavement Design

Recommendations for the design of flexible asphalt concrete pavement sections were developed in accordance with the procedures outlined in the latest edition of the Caltrans Highway Design Manual. The Caltrans design method uses Traffic Indices (TI) to represent anticipated wheel loads and frequency of usage for a given design life. A design life of 20 years is typically used in California.

An R-value of 32 was obtained by a laboratory test on a representative sample of potential pavement subgrade material. An R-value of 30 was subsequently used for determining the design sections. A Traffic Index value of 4.5 was used for parking areas and drive aisles assuming passenger cars and occasional delivery truck type traffic. An optional, heavier pavement section using a TI value of 5.5 is also provided as an option for pavements intended to receive loads from occasional heavier truck traffic. The following are the resulting recommended structural asphalt concrete (AC)/ aggregate base (AB) pavement sections based on the provided TI values.

Table 7: Recommended Pavement Design Alternatives (20-Year Design Life)

Load Application (Traffic Index)	Asphalt Concrete (in.)	Class 2 AB (in.)	Total Section (in.)
Driveways and Parking Areas for Passenger Car and Light Truck Traffic (4.5)	2.5	6.0*	8.5
Moderate Heavy Truck Traffic (5.5)	3.0	7.0	10.0

* Minimum recommended aggregate base thickness

If the lighter pavements (TI = 4.5) are planned to be placed prior to, or during construction, the traffic indices and pavement sections may not be adequate for support of what is typically more frequent and heavier construction traffic. Therefore, if the pavement sections will be used for construction access, the heavier pavement section should be considered, or the asphalt concrete should be placed in phases (e.g., placement of final lift of AC after building construction is substantially completed) to minimize pavement damage caused by construction traffic.

Design based on the aforementioned traffic indices should provide the design pavement life with only a normal amount of pavement maintenance.

In areas where pavements will abut planted areas, the pavement aggregate base layer, pavement section subgrade soils and trench backfill should be protected against saturation. Planned concrete curbs should extend at least to the bottom of the aggregate base layer, forming a concrete barrier between the landscaped areas and the pavement section. In addition, water should never be allowed to pond behind the curb and gutter during or after the completion of construction.

AB for use in flexible pavements should conform to Caltrans Standard Specification Sections 26-1.02A and 26-1.02B (2010) for Class 2 AB. AB from recycled sources offered by AB suppliers as well as AC grindings mixed with existing baserock and meeting Class 2 specifications can be utilized in lieu of virgin Class 2 AB in pavement sections upon approval of the Geotechnical Engineer. AB used in pavement sections should be compacted to a minimum 95 percent relative compaction (ASTM D1557) and should be firm and unyielding at the time of asphalt concrete placement.

6.10 Stormwater Infiltration Design Considerations

If the requirements of Provision C.3 of the Municipal Regional Stormwater Permit (MRP) apply to the project, post-construction stormwater controls would be required as part of the project. Stormwater infiltration treatment systems utilizing measures such as biofiltration swales or planters, or pervious pavements or pavers should be designed considering the typical infiltration rates characteristic of the onsite surficial soils. The near-surface soils at the site were found to typically consist of clayey sand to sandy clay soils of low to medium plasticity, and would likely be categorized as either Hydrologic Soil Group “B” or “C” soils (USDA, 2007). Field percolation or infiltration tests were not within the scope of this study but can be performed if needed to further define the infiltration potential of the onsite soils. Where infiltration rates are judged to be too low to accommodate infiltration of collected stormwater to the underlying soils, use of a subdrainage layer consisting of an appropriate permeable material would be required.

In general, biofiltration swales or basins should not be placed directly adjacent to building perimeters in order to minimize impact on the long-term performance of shallow foundations. If such features must be constructed adjacent to foundations, the filter material should not be located within the footing zone of influence, considered to be the zone below an imaginary 1.5:1 (horizontal to vertical) plane projected downward from the bottom edge of the adjacent building footing. In addition, the bottom of the bioswale or biofiltration area should include a perforated subdrain pipe to carry collected infiltration water away from the foundations.

Biofiltration swales should preferably be placed a minimum of five feet away from pavements or exterior flatwork in order to reduce potential impacts on these features such as settlement or lateral movement. Where concrete curbs are located adjacent to bioswale or other filtration features, the loose biofiltration material should not be located within a zone below an imaginary 1:1 (horizontal to vertical) plane projected downward from the bottom edge of the adjacent curb.

6.11 Plan Review

We recommend that Geosphere be provided the opportunity to review the final project plans prior to construction. The purpose of this review is to assess the general compliance of the plans with the recommendations provided in this report and confirm the incorporation of these recommendations into the project plans and specifications.

6.12 Observation and Testing During Construction

We recommend that Geosphere be retained to provide observation and testing services during site preparation, mass grading, underground utility construction, foundation excavation, pavement construction, and to observe final site drainage. This is to observe compliance with the design concepts, specifications and recommendations, and to allow for possible changes in the event that subsurface conditions differ from those anticipated prior to the start of construction.

7.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

The recommendations of this report are based upon the soil and conditions encountered in the field explorations (i.e., borings). If variations or undesirable conditions are encountered during construction, Geosphere should be contacted so that supplemental recommendations may be provided.

This report is issued with the understanding that it is the responsibility of the owner or his representatives to see that the information and recommendations contained herein are called to the attention of the other members of the design team and incorporated into the plans and specifications, and that the necessary steps are taken to see that the recommendations are implemented during construction.

The findings and recommendations presented in this report are valid as of the present time for the development as currently proposed. However, changes in the conditions 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. Accordingly the findings and recommendations presented in this report may be invalidated, wholly or in part, by changes outside our control. Therefore, this report is subject to review by Geosphere after a period of three (3) years has elapsed from the date of issuance of this report. In addition, if the currently proposed design scheme as noted in this report is altered, Geosphere should be provided the opportunity to review the changed design and provide supplemental recommendations as needed.

Recommendations are presented in this report which specifically request that Geosphere be provided the opportunity to review the project plans prior to construction and that we be retained to provide observation and testing services during construction. The validity of the recommendations of this report assumes that Geosphere will be retained to provide these services.

This report was prepared upon your request for our services, and in accordance with currently accepted geotechnical engineering practice. No warranty based on the contents of this report is intended, and none shall be inferred from the statements or opinions expressed herein. The scope of our services for this report did not include an environmental assessment or investigation for the presence or absence of wetlands or hazardous or toxic materials in the soil, surface water, groundwater or air, on, below or around this site. Any statements within this report or on the attached figures, logs or records regarding odors noted or other items or conditions observed are for the information of our client only.

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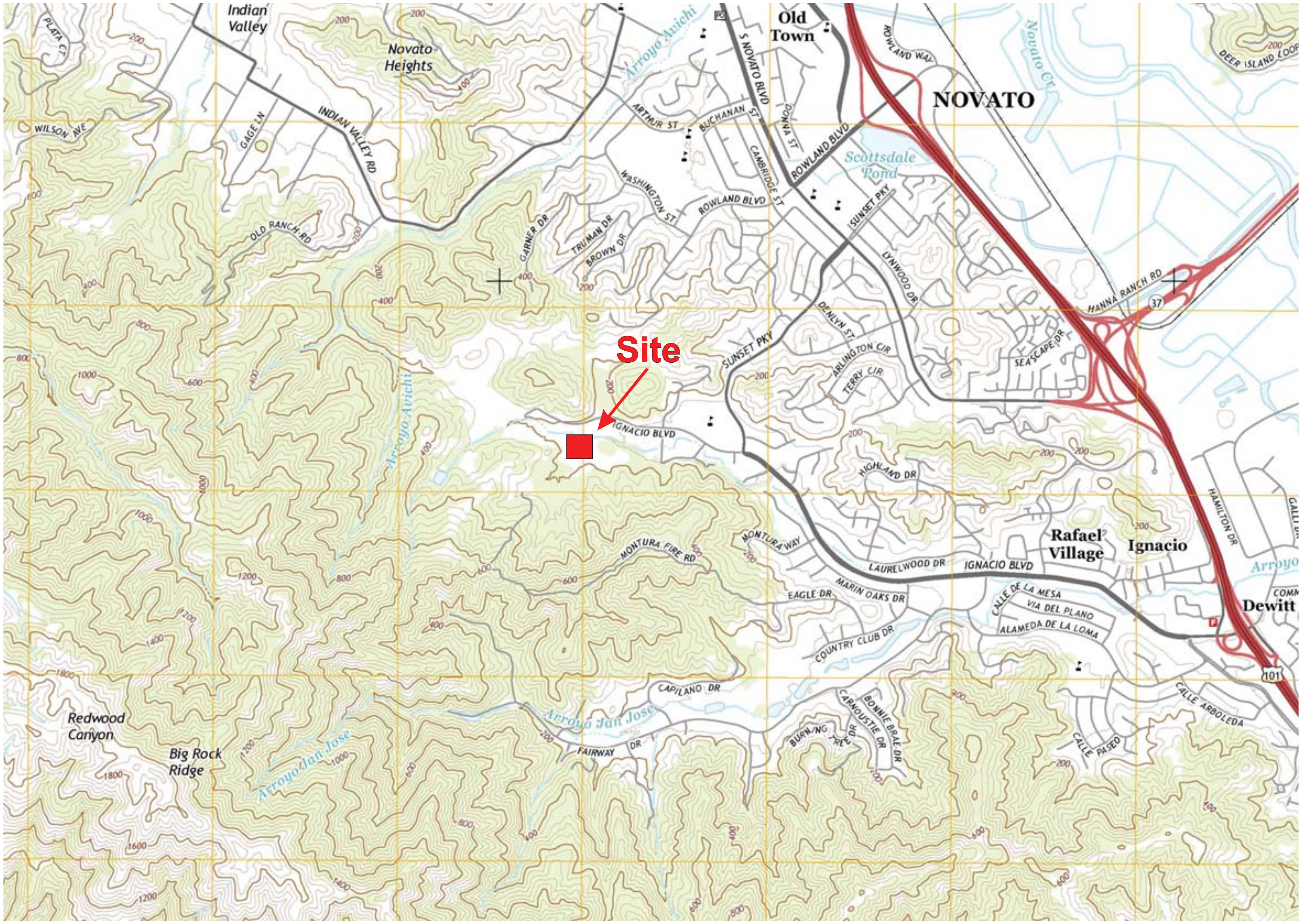
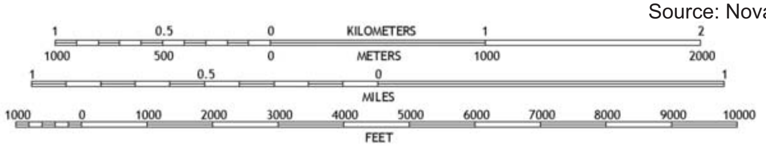
FIGURES



Source: Novato Quadrangle, US Topographic Map 7.5-Minute Series, United States Geological Survey (2015)



College of Marin Indian Valley Campus Jonas Center Project 1800 Ignacio Boulevard, Novato, California 94949	91-03940-A	August 2017
Geosphere Consultants, Inc AN ETS COMPANY Geotechnical Engineering · Engineering Geology Environmental Management · Water Resources	Site Vicinity Map	Figure 1

Base map: Site Plan, A1.1, College of Marin IVC Jonas Center & Bldg 18 Alterations, prepared by brick, dated July 10, 2017.

- Approximate Boring Location
- Geologic Cross Section

50ft

College of Marin Indian Valley Campus Jonas Center Project 1800 Ignacio Boulevard, Novato, California 94949	91-03940-A	August 2017
Geosphere Consultants, Inc	Developmental Site Plan	Figure 2

- Approximate Boring Location

Qha - Alluvium (Holocene)

- Geologic Cross Section

- Approximate Improvement

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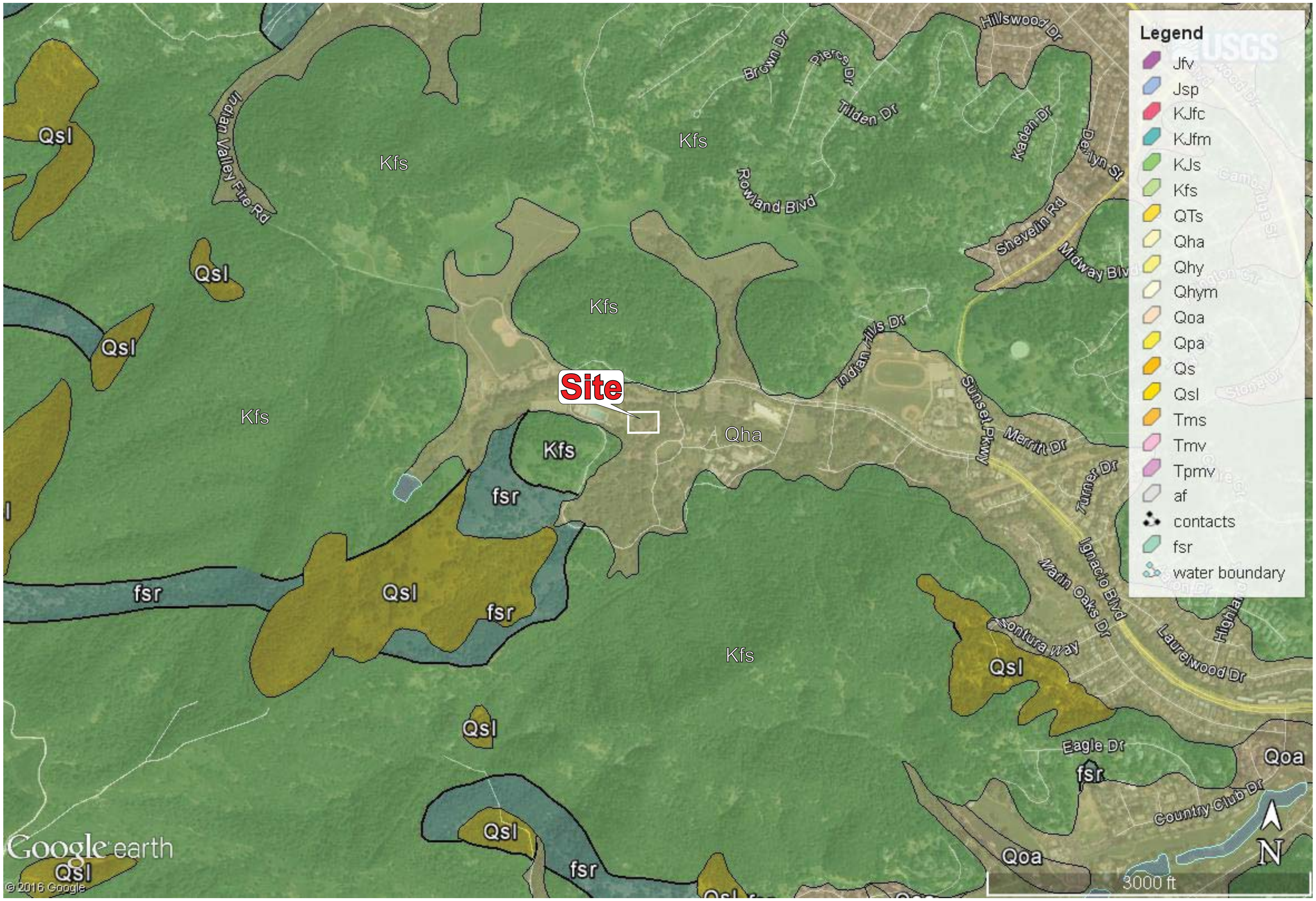
Geosphere Consultants, Inc	Site Plan & Boring Location Map	Figure 3



Source: USGS w/ California Geologic Survey, Scientific Investigations Map 2918

Qha - Alluvium (Holocene)
Kfs - Franciscan Complex sedimentary rocks (Cretaceous)
Qsl - Hillslope Deposits (Quaternary)
fsr - Franciscan Complex melange (Mesozoic)

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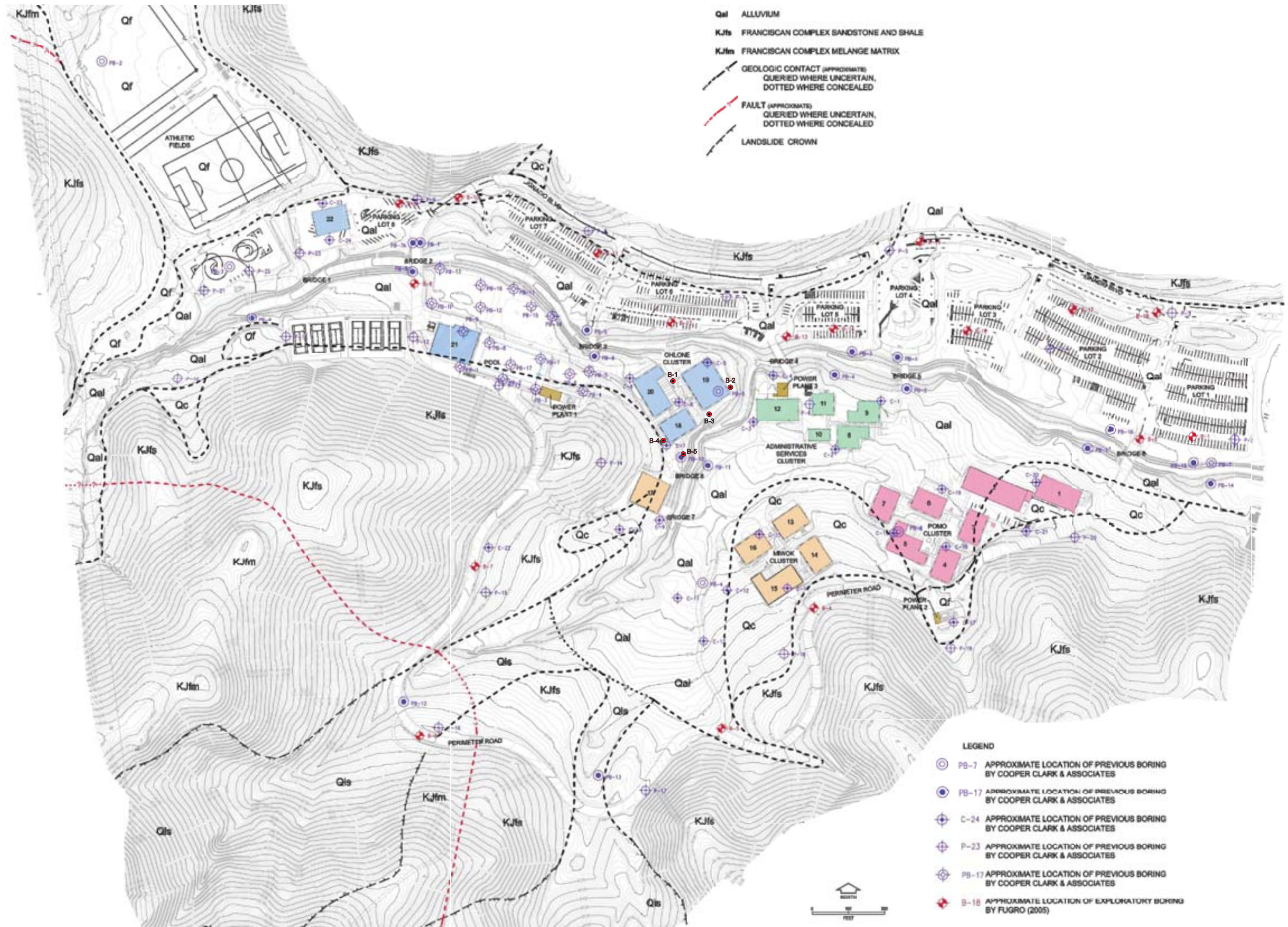
Geosphere Consultants, Inc	Site Vicinity Geologic Map	Figure 4a



Base map source: Plate 3 - Site Plan and Geologic Map, by Fugro West, Inc., 2005

- Approximate Geosphere Boring Location

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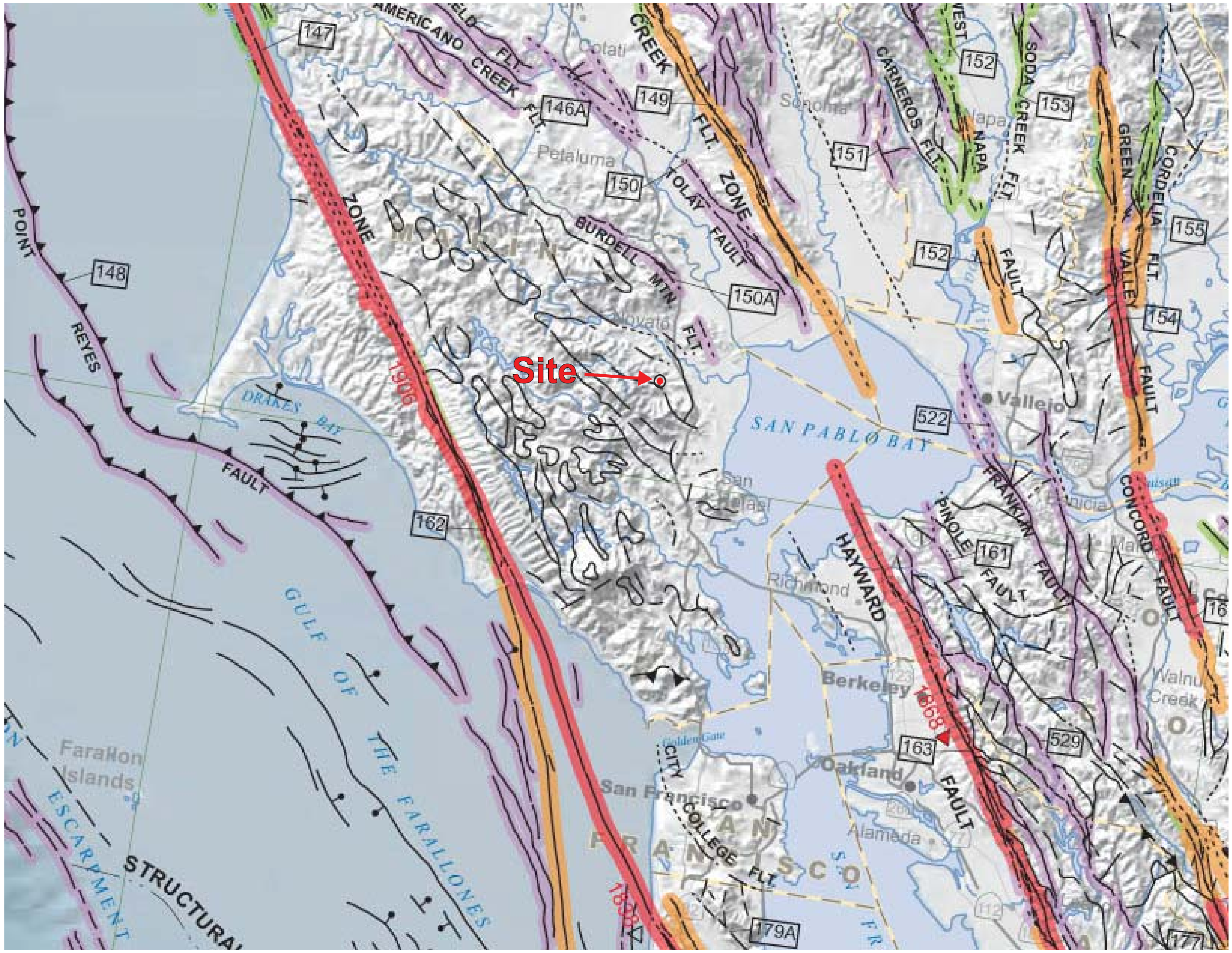
Geosphere Consultants, Inc.	Campus Geologic Map	Figure 4b

DESCRIPTION	
ON LAND	OFFSHORE
Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
Displacement during Holocene time.	Fault offsets seafloor sediments or strata of Holocene age.
Faults showing evidence of displacement during late Quaternary time.	Faults cuts strata of Late Pleistocene age.
Undivided Quaternary faults - most faults in this category show evidence of displacement during the last 1,600,000 years; possible exceptions are faults which displace rocks of undifferentiated Plio-Pleistocene age.	Fault cuts strata of Quaternary age.
Faults without recognized Quaternary displacement or showing evidence of no displacement during Quaternary time. Not necessarily inactive.	Fault cuts strata of Pliocene or older age.

Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement
Quaternary	Late Quaternary	Historic	200		
		Holocene	11,700		? ? ?
		Pleistocene	700,000		? ?
	Early Quaternary		1,600,000		?
Pre-Quaternary			4.5 billion (Age of Earth)		



Base Map Reference: California Geological Survey - 2010 Fault Activity Map of California

College of Marin Indian Valley Campus Jonas Center Project 1800 Ignacio Boulevard, Novato, California 94949	91-03940-A	August 2017
Geosphere Consultants, Inc.	Regional Fault Map	Figure 5

Geosphere Consultants, Inc.
AN ETS COMPANY
Geotechnical Engineering · Engineering Geology
Environmental Management · Water Resources
2001 Crow Canyon Rd, Ste 210
CA 94583
Telephone: 9253147180
Fax: 9258557140

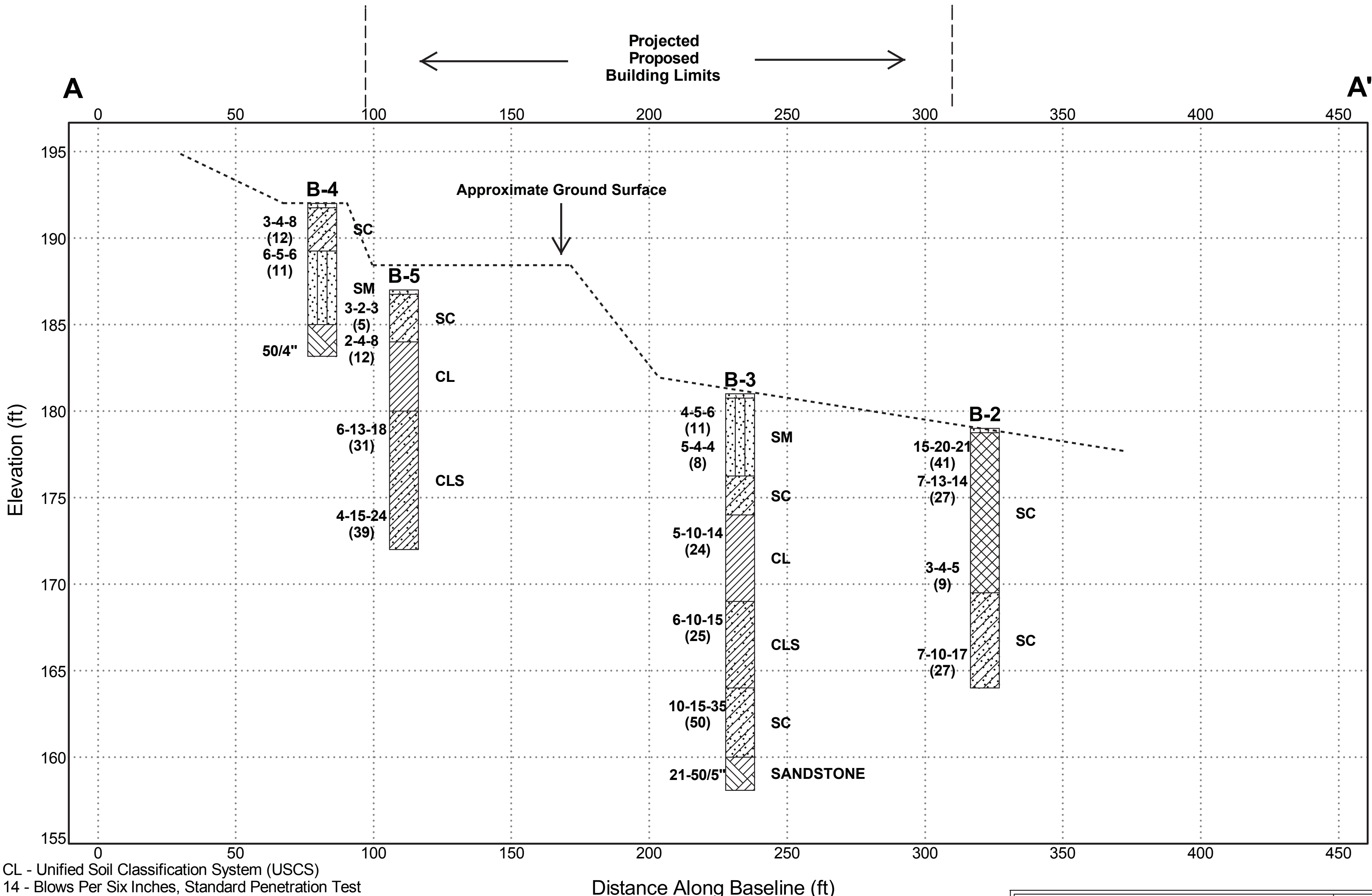
SUBSURFACE DIAGRAM

CLIENT Marin Community College District

PROJECT NUMBER 91-03940-A

PROJECT NAME Jonas Center Project

PROJECT LOCATION 1800 Ignacio Boulevard, Novato, CA 94949



CL - Unified Soil Classification System (USCS)
14 - Blows Per Six Inches, Standard Penetration Test
(21) - N Value

Schematic Geologic Cross-Section A-A' | Figure 6a

Geosphere Consultants, Inc.
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Environmental Management · Water Resources

2001 Crow Canyon Rd, Ste 210
CA 94583
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SUBSURFACE DIAGRAM

CLIENT Marin Community College District

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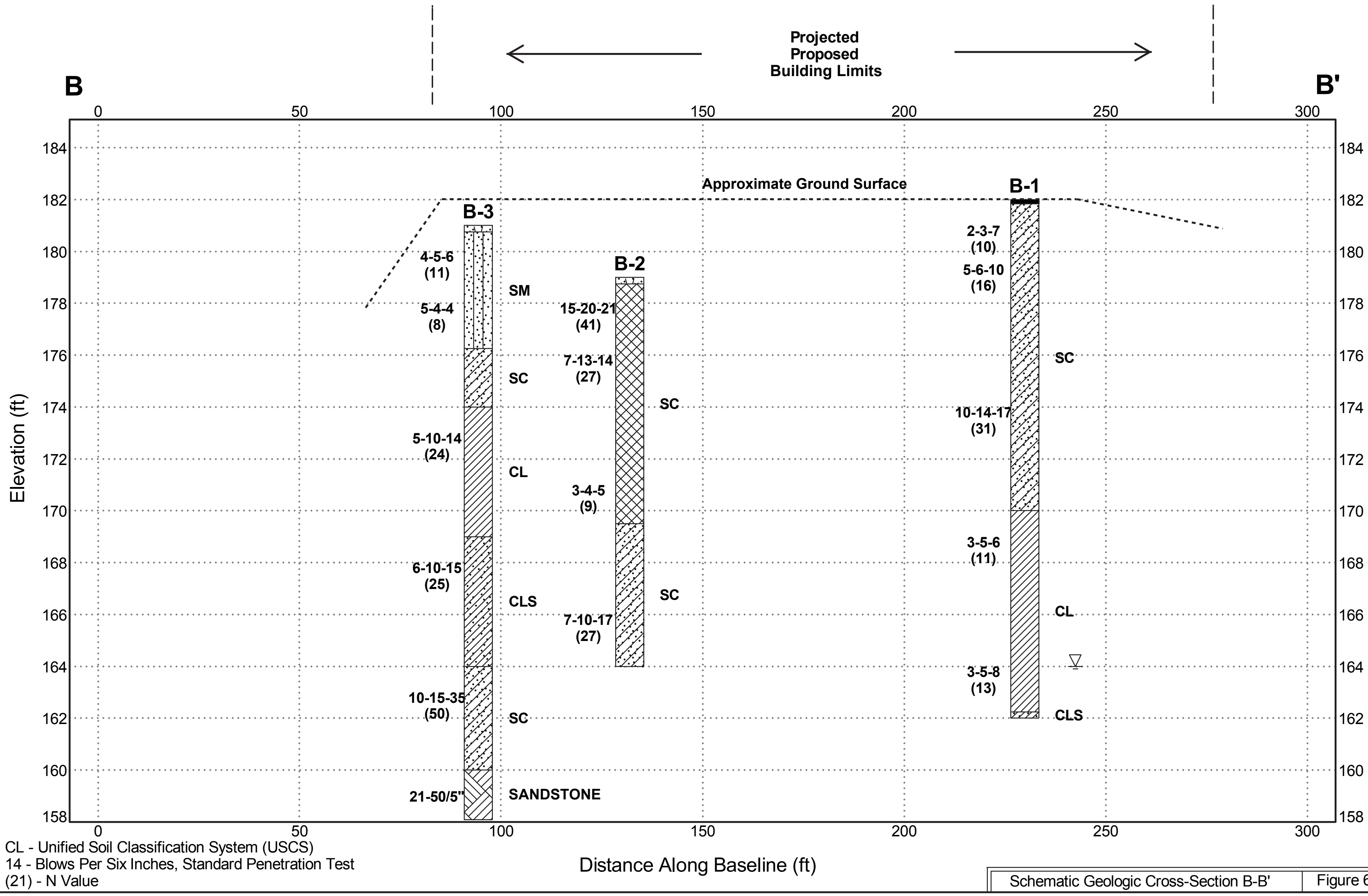


CL - Unified Soil Classification System (USCS)
14 - Blows Per Six Inches, Standard Penetration Test
(21) - N Value

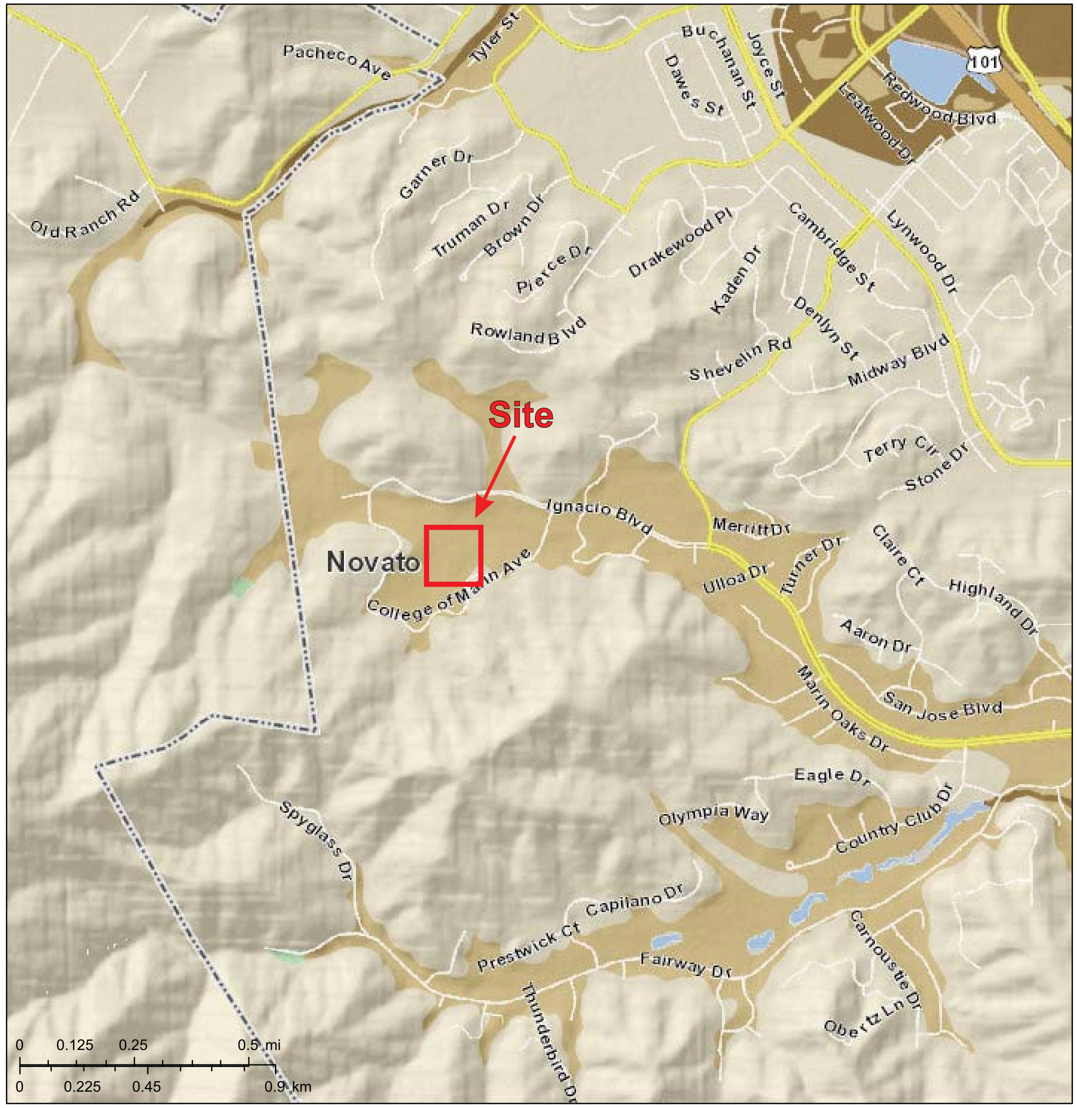
Schematic Geologic Cross-Section B-B' | Figure 6b

N

San Francisco Bay Area Hazards

Legend

- Very High Susceptibility
- High Susceptibility
- Moderate Susceptibility
- Low Susceptibility
- Very Low Susceptibility



This map is intended for planning only and is not intended to be site specific. Rather, it depicts the general risk within neighborhoods and the relative risk from community to community.

ASSOCIATION OF BAY AREA GOVERNMENTS RESILIENCE PROGRAM

May 11, 2017 ABAG GIS

Approximate School Site Location

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Geosphere Consultants, Inc. AN ATS COMPANY Geotechnical Engineering · Engineering Geology Environmental Management · Water Resources	Liquefaction Susceptibility Map	Figure 7

San Francisco Bay Area Hazards

Legend

Active Fault Zone

This map is intended for planning only and is not intended to be site specific. Rather, it depicts the general risk within neighborhoods and the relative risk from community to community.

ASSOCIATION OF BAY AREA GOVERNMENTS RESILIENCE PROGRAM

May 11, 2017 ABAG GIS

College of Marin Indian Valley Campus Jonas Center Project 1800 Ignacio Boulevard, Novato, California 94949	91-03940-A	August 2017
Geosphere Consultants, Inc	Alquist-Priolo Earthquake Fault Map	Figure 8

San Francisco Bay Area Hazards

Legend

- Mostly Landslide
- Many Landslides
- Few Landslides
- Very Few Landslides
- Surficial Deposits

This map is intended for planning only and is not intended to be site specific. Rather, it depicts the general risk within neighborhoods and the relative risk from community to community.

ASSOCIATION OF BAY AREA GOVERNMENTS RESILIENCE PROGRAM

May 11, 2017 ABAG GIS

College of Marin Indian Valley Campus Jonas Center Project 1800 Ignacio Boulevard, Novato, California 94949	91-03940-A	August 2017
Geosphere Consultants, Inc. AN ATS COMPANY Geotechnical Engineering · Engineering Geology Environmental Management · Water Resources	Existing Landslide Map	Figure 9

NFIP

NATIONAL FLOOD INSURANCE PROGRAM

PANEL 0279D

FIRM

FLOOD INSURANCE RATE MAP

MARIN COUNTY, CALIFORNIA
AND INCORPORATED AREAS

PANEL 279 OF 531

(SEE MAP INDEX FOR FIRM PANEL LAYOUT)

CONTAINS:

COMMUNITY	NUMBER	PANEL	SUFFIX
MARIN COUNTY	060173	0279	D
NOVATO, CITY OF	060178	0279	D

Notice to User: The **Map Number** shown below should be used when placing map orders; the **Community Number** shown above should be used on insurance applications for the subject community.

MAP NUMBER
06041C0279D

EFFECTIVE DATE
MAY 4, 2009

Federal Emergency Management Agency

FLOODWAY AREAS IN ZONE AE

The floodway is the channel of a stream plus any adjacent floodplain areas that must be kept free of encroachment so that the 1% annual chance flood can be carried without substantial increases in flood heights.

OTHER FLOOD AREAS

ZONE X — Areas of 0.2% annual chance flood; areas of 1% annual chance flood with average depths of less than 1 foot or with drainage areas less than 1 square mile; and areas protected by levees from 1% annual chance flood.

OTHER AREAS

ZONE X — Areas determined to be outside the 0.2% annual chance floodplain.

ZONE D — Areas in which flood hazards are undetermined, but possible.

MAP SCALE 1" = 500'

250 0 500 1000 FEET

150 0 150 300 METERS

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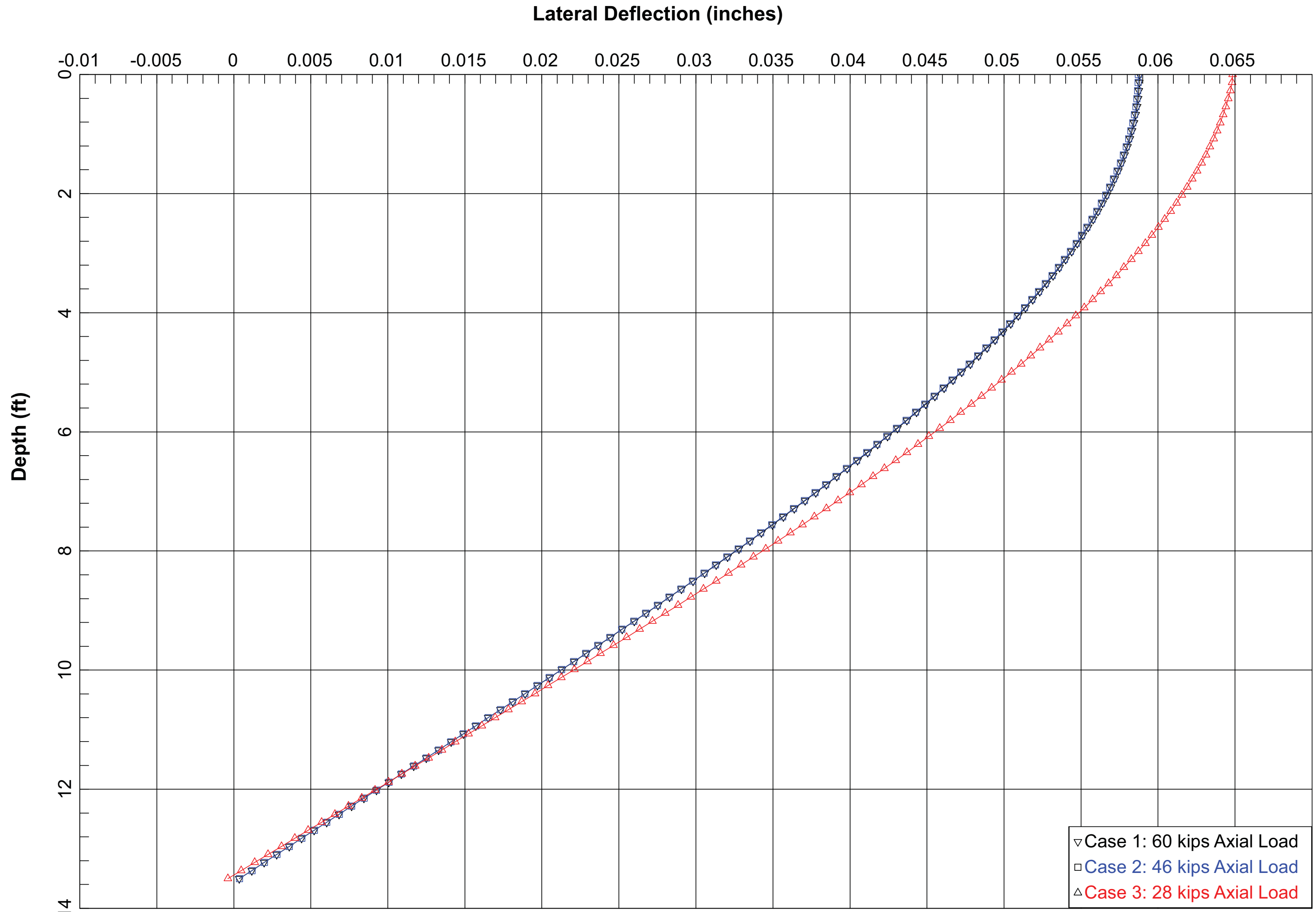
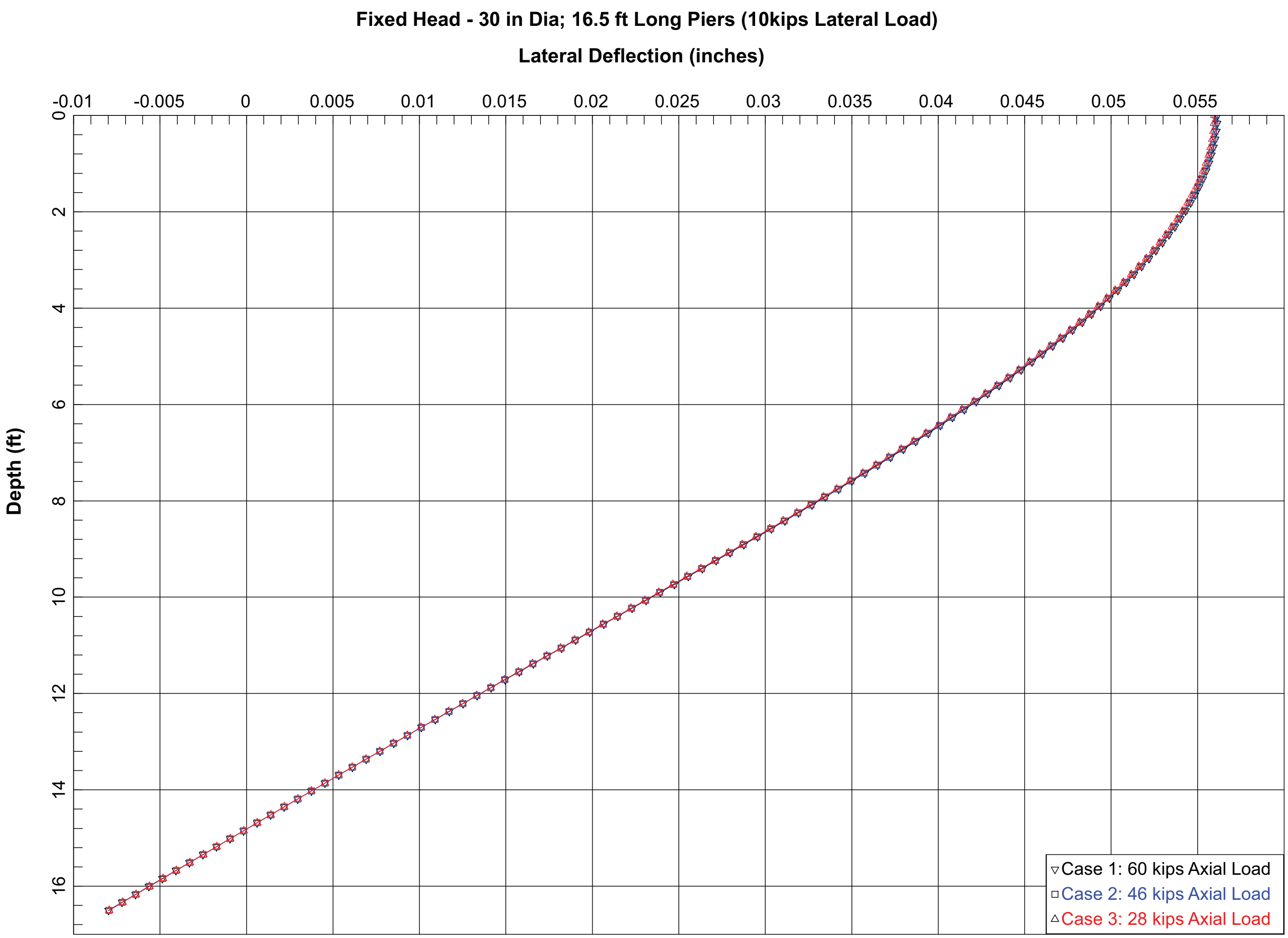
Geosphere Consultants, Inc. AN ETS COMPANY Geotechnical Engineering • Engineering Geology Environmental Management • Water Resources	Flood Hazard Map	Figure 10

Fixed Head - 30 in Dia; 13.5 ft Long Pier (10kips Lateral Load)



College of Marin Indian Valley Campus Jonas Center Project 1800 Ignacio Boulevard, Novato, California 94949	91-03940-A	August 2017
Geosphere Consultants, Inc AN ETS COMPANY	Lateral Deflection Fixed Head 13.5 ft	Figure 11a



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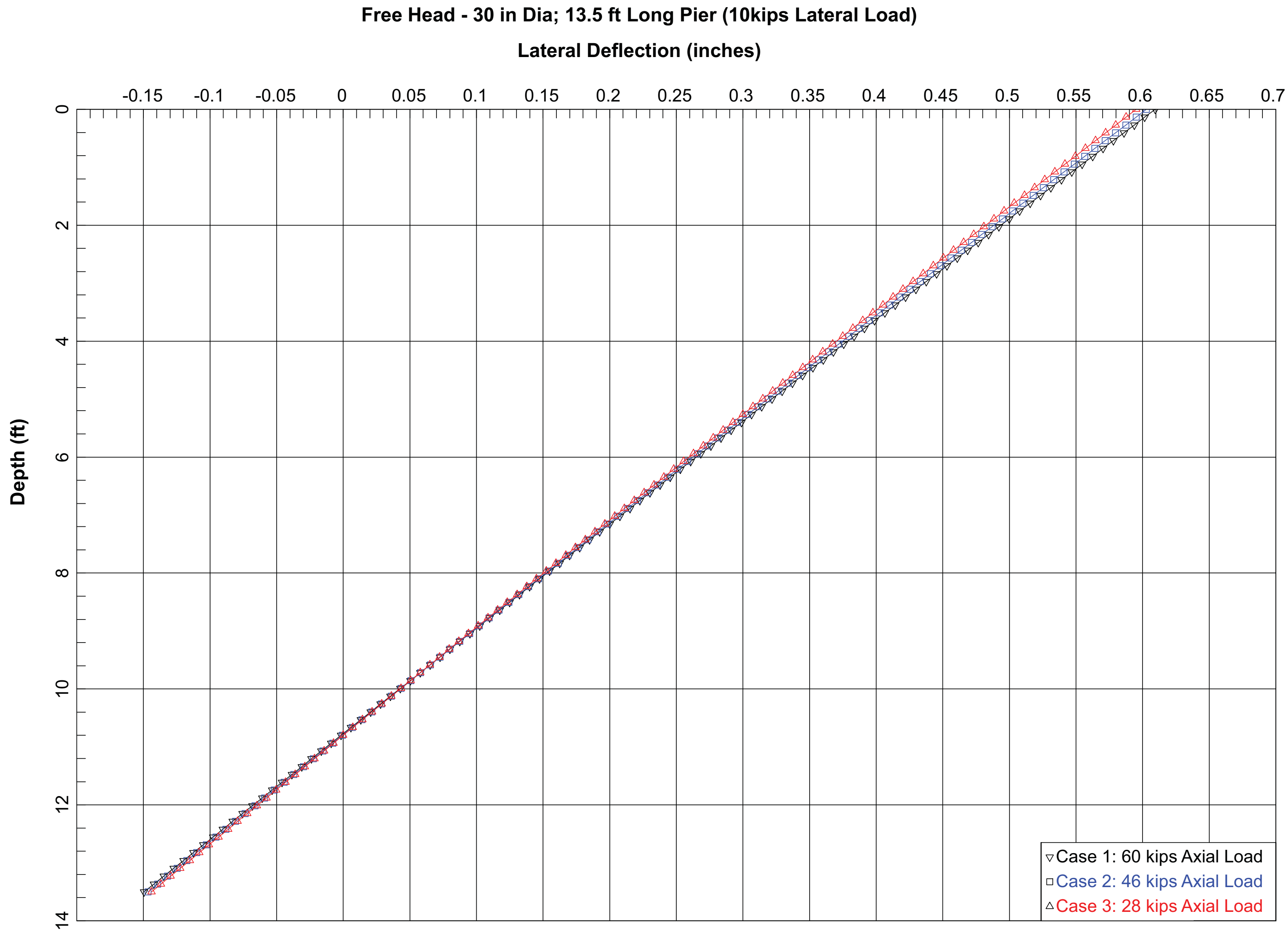
Geosphere Consultants, Inc AN ETS COMPANY Geotechnical Engineering · Engineering Geology Environmental Management · Water Resources	Lateral Deflection Fixed Head 16.5 ft	Figure 11b



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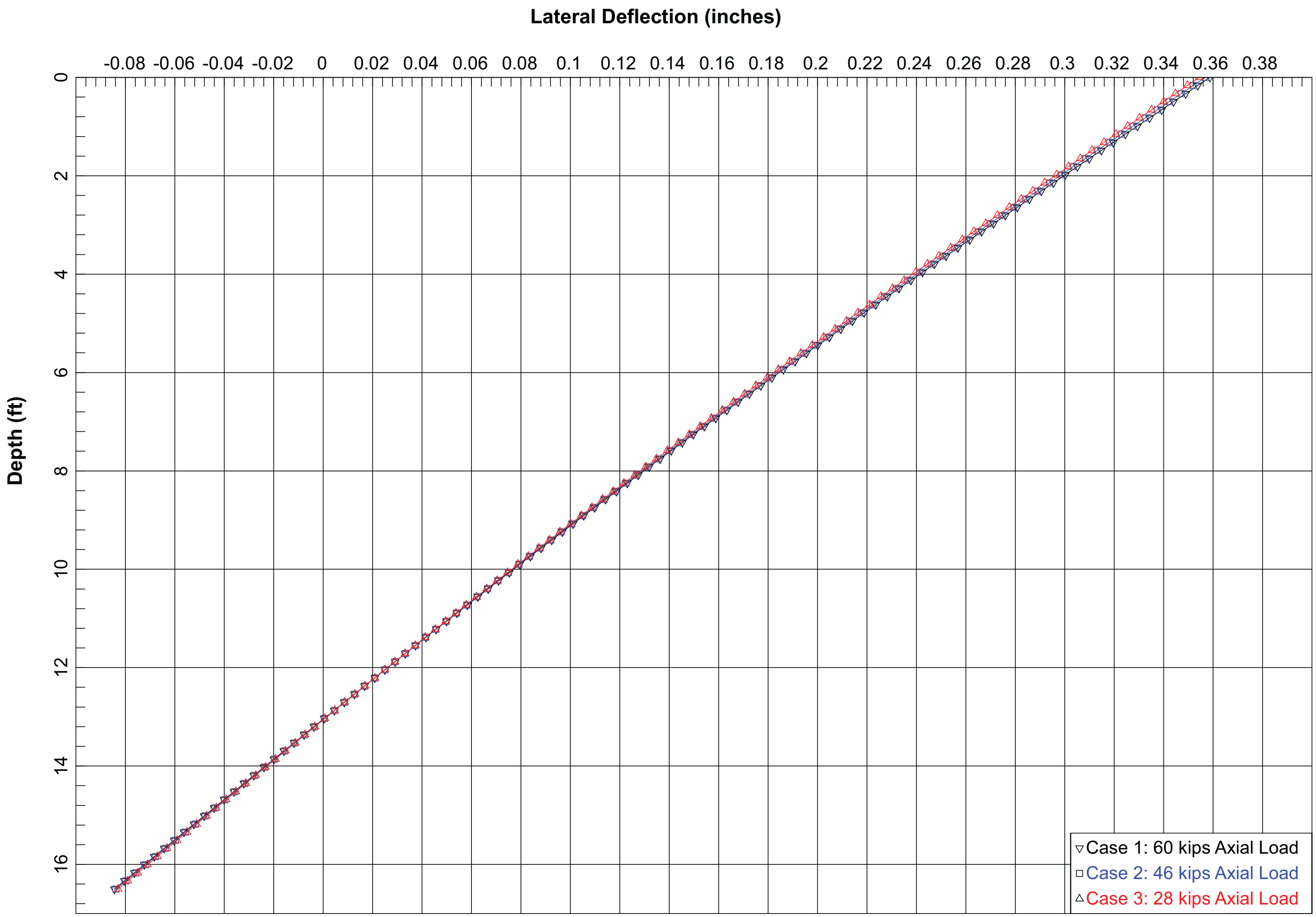
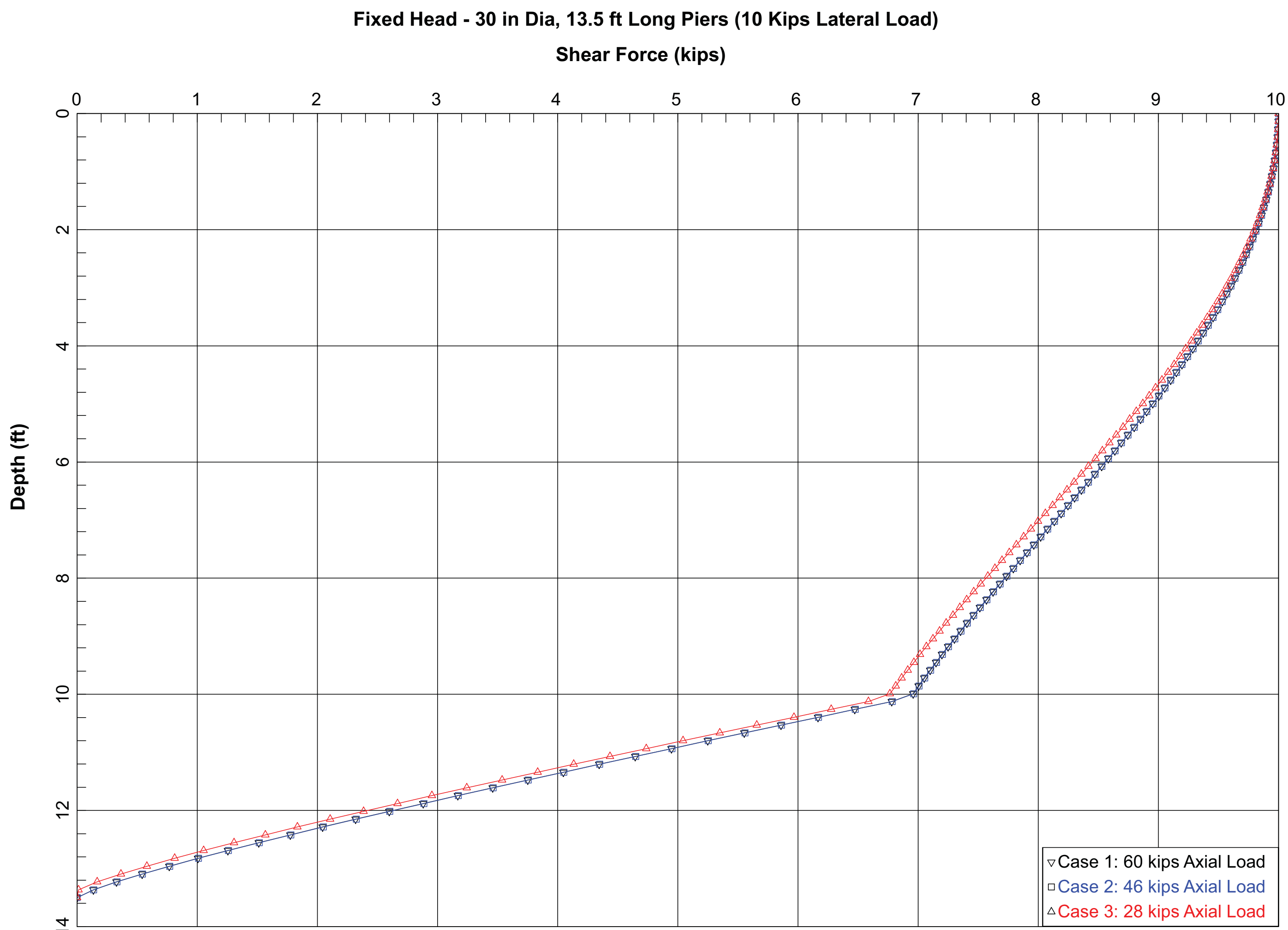
Geosphere Consultants, Inc AN ETS COMPANY Geotechnical Engineering · Engineering Geology Environmental Management · Water Resources	Lateral Deflection Free Head 13.5 ft	Figure 11c

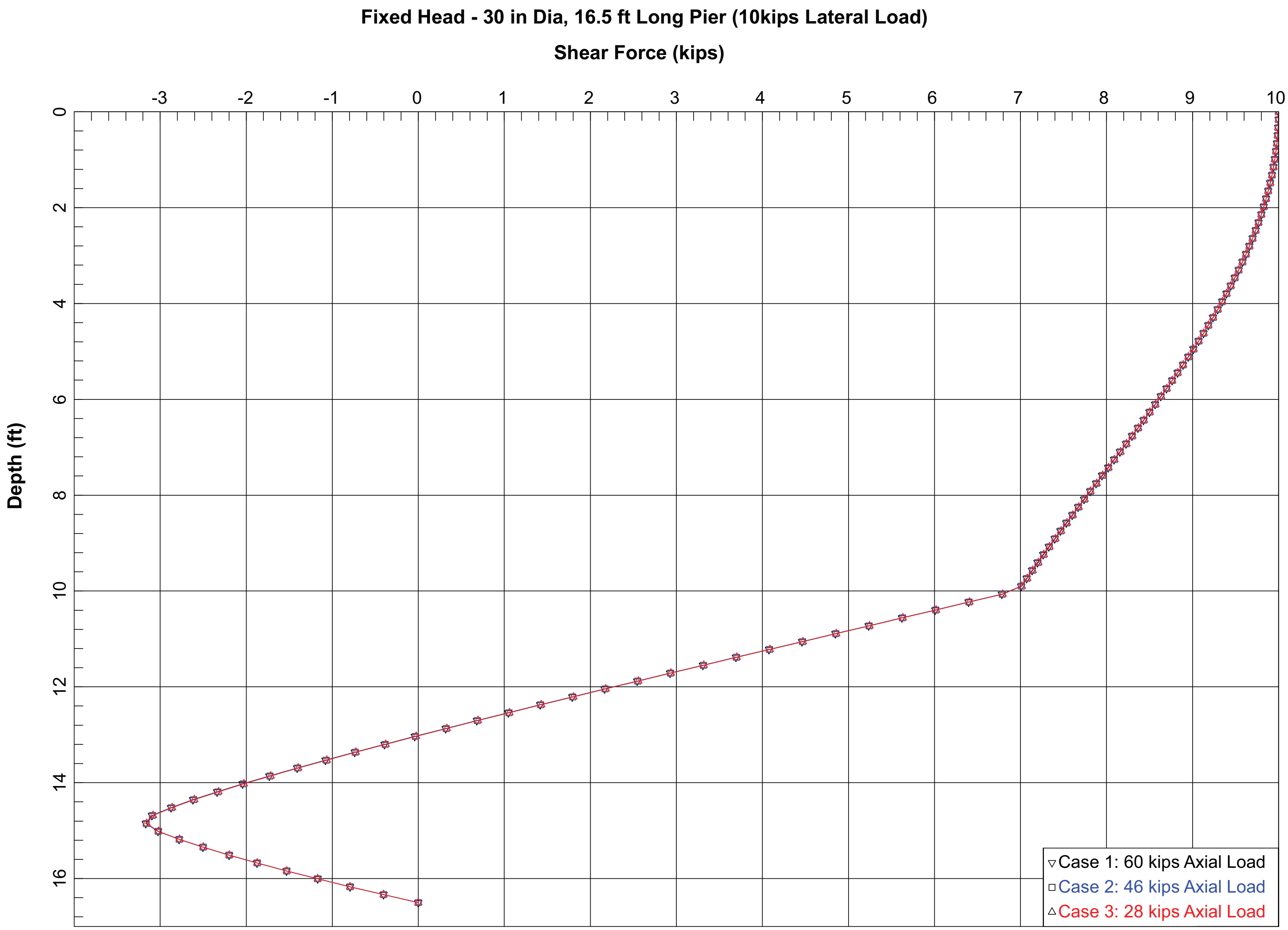
Free Head - 30 in Dia; 16.5 ft Long Pier (10kips Lateral Load)



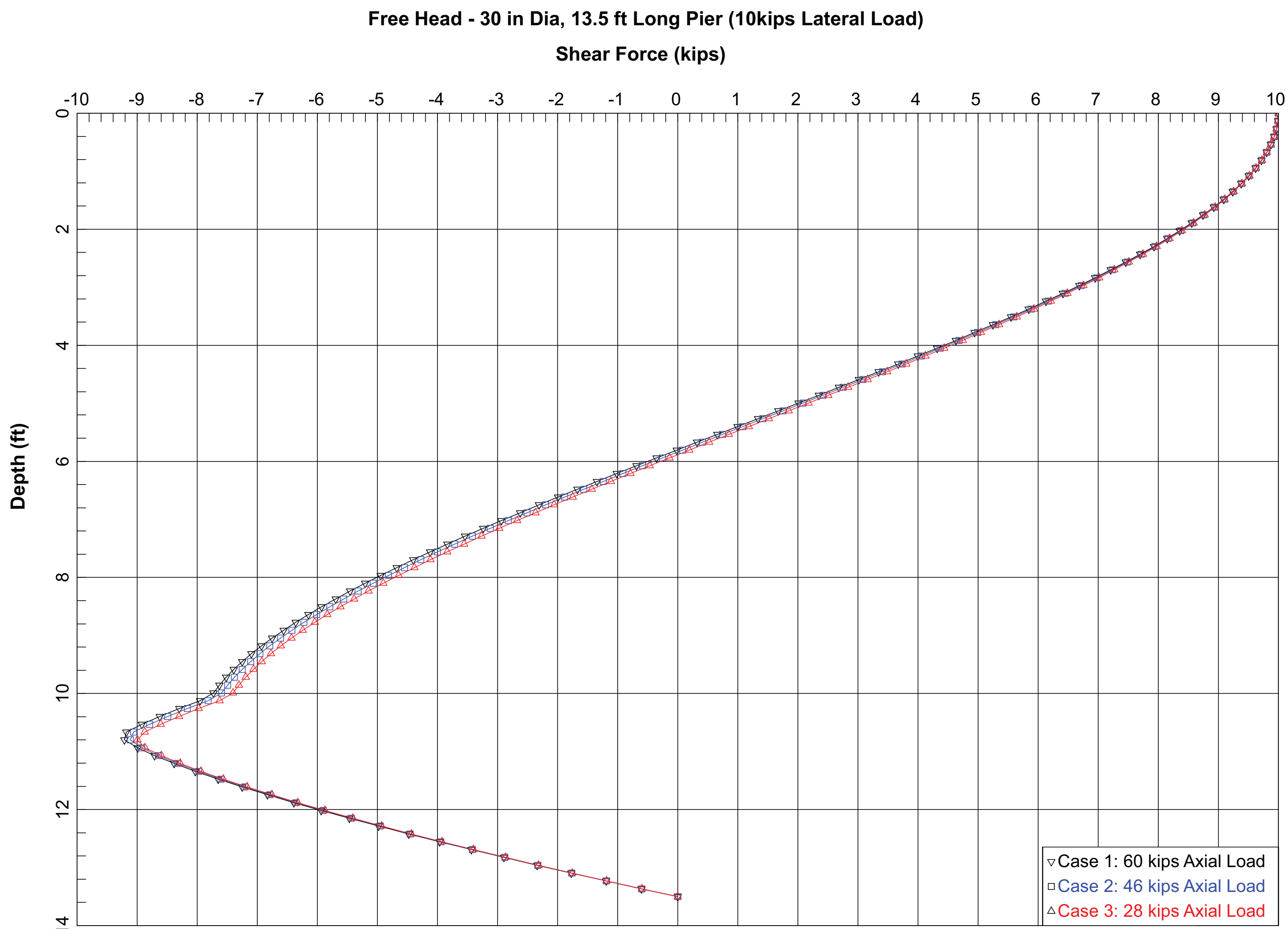
College of Marin Indian Valley Campus Jonas Center Project 1800 Ignacio Boulevard, Novato, California 94949	91-03940-A	August 2017
Geosphere Consultants, Inc	Lateral Deflection Free Head 16.5 ft	Figure 11d



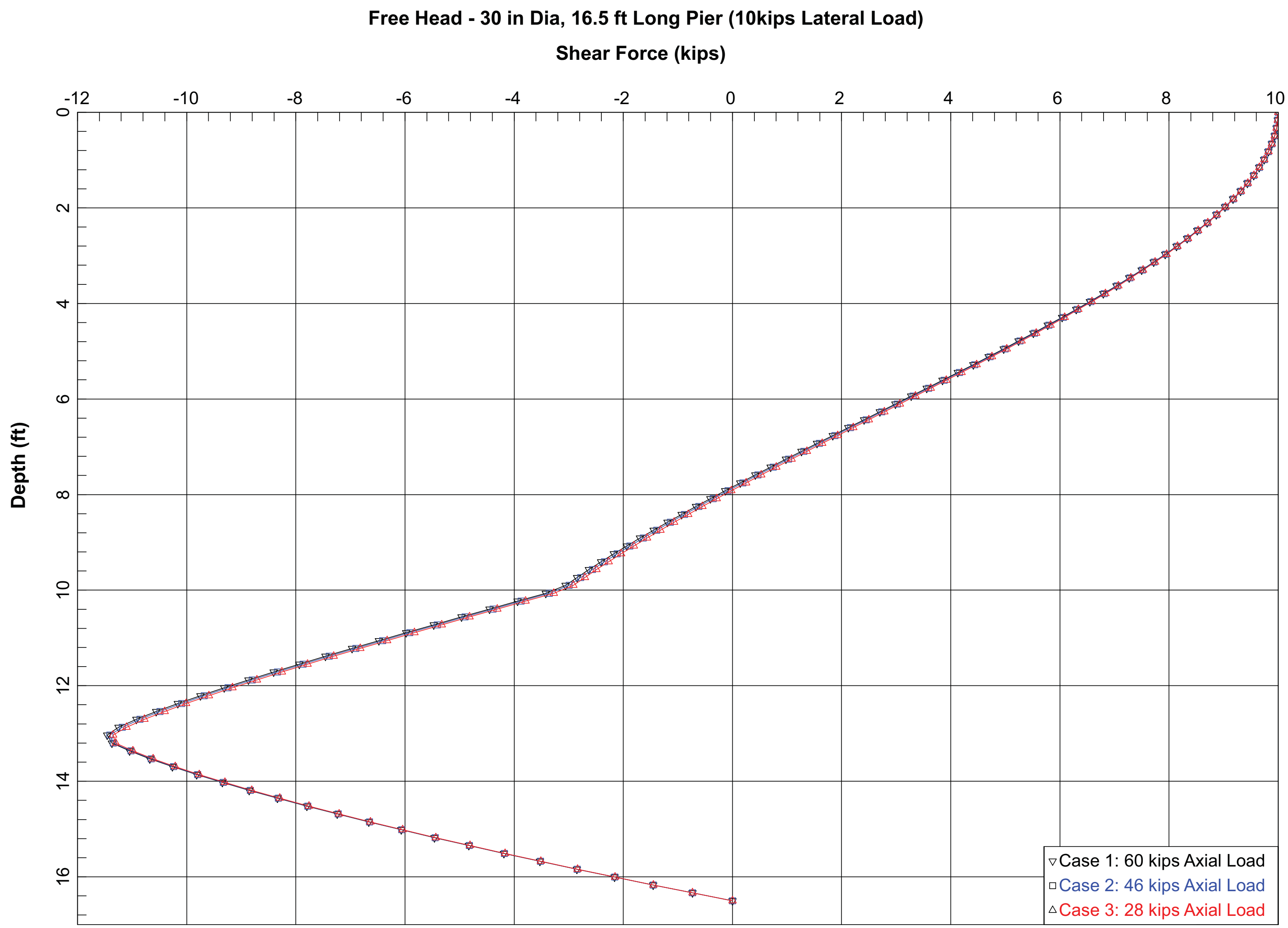
College of Marin Indian Valley Campus Jonas Center Project 1800 Ignacio Boulevard, Novato, California 94949	91-03940-A	August 2017
Geosphere Consultants, Inc	Shear Force Fixed Head 13.5 ft	Figure 12a



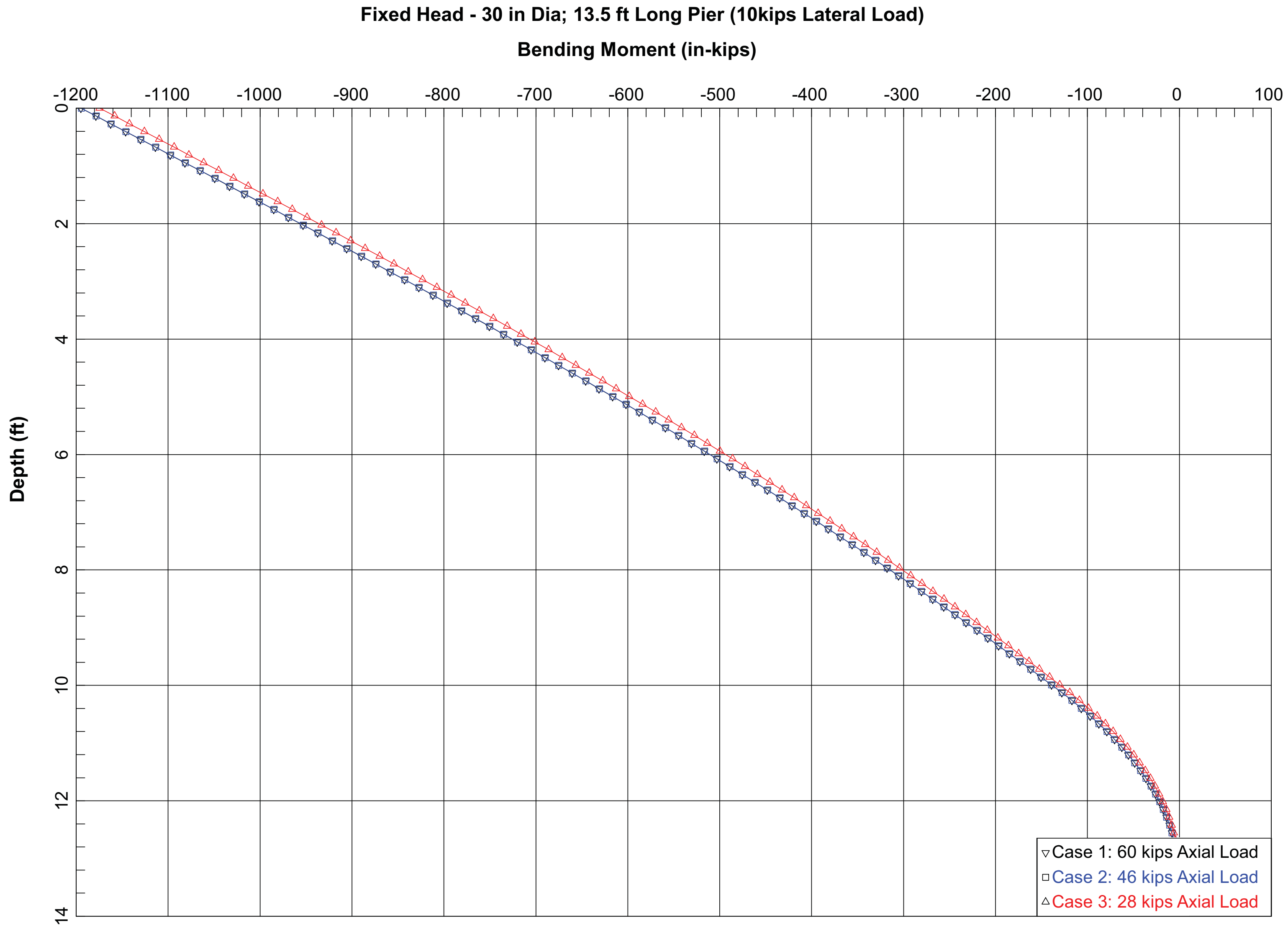
College of Marin Indian Valley Campus Jonas Center Project 1800 Ignacio Boulevard, Novato, California 94949	91-03940-A	August 2017
Geosphere Consultants, Inc AN ETS COMPANY	Shear Force Fixed Head 16.5 ft	Figure 12b



College of Marin Indian Valley Campus Jonas Center Project 1800 Ignacio Boulevard, Novato, California 94949	91-03940-A	August 2017
Geosphere Consultants, Inc	Shear Force Free Head 13.5 ft	Figure 12c



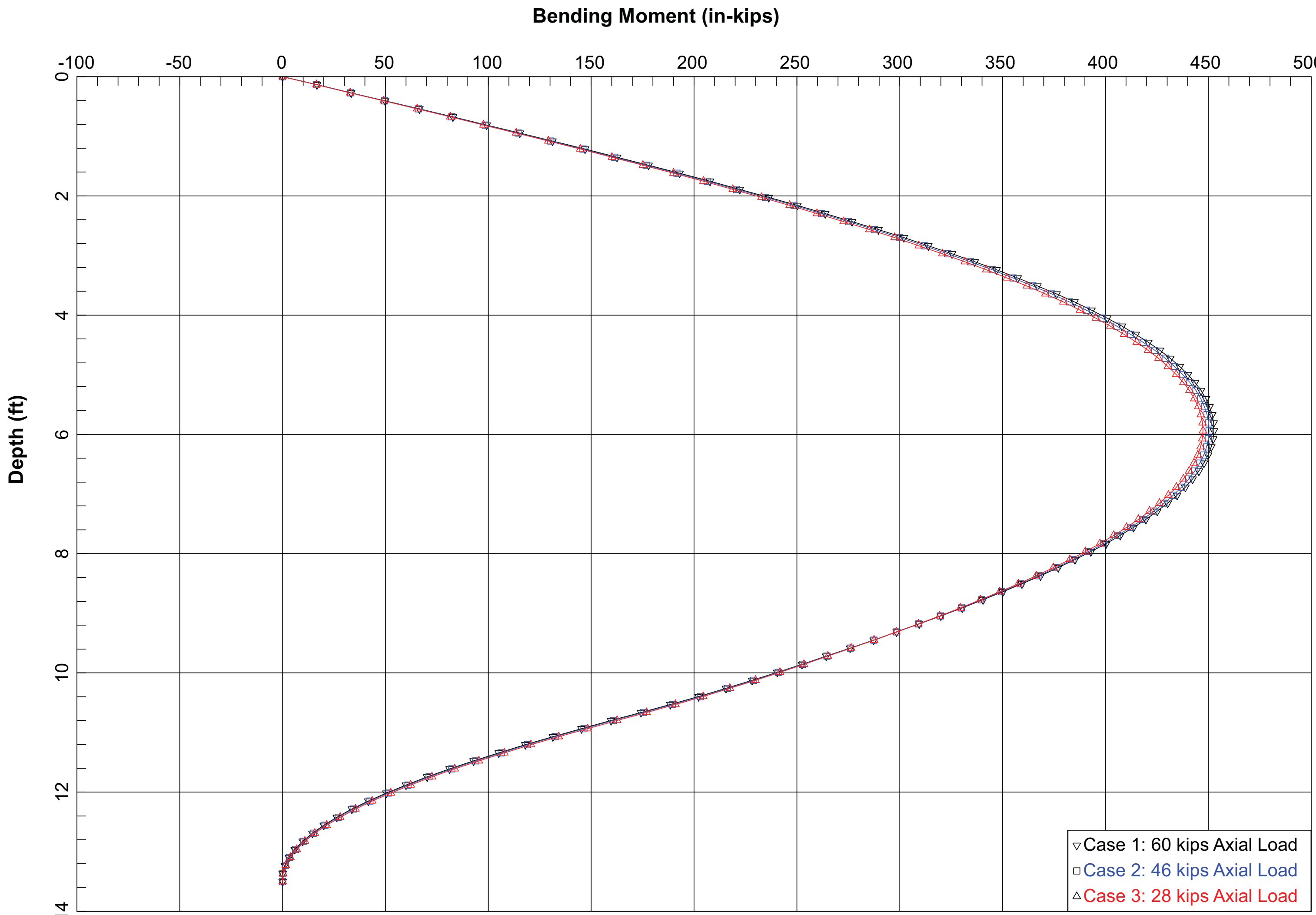
College of Marin Indian Valley Campus Jonas Center Project 1800 Ignacio Boulevard, Novato, California 94949	91-03940-A	August 2017
Geosphere Consultants, Inc	Shear Force Free Head 16.5 ft	Figure 12d



College of Marin Indian Valley Campus Jonas Center Project 1800 Ignacio Boulevard, Novato, California 94949	91-03940-A	August 2017
Geosphere Consultants, Inc	Bending Moment Fixed Head 13.5 ft	Figure 13a

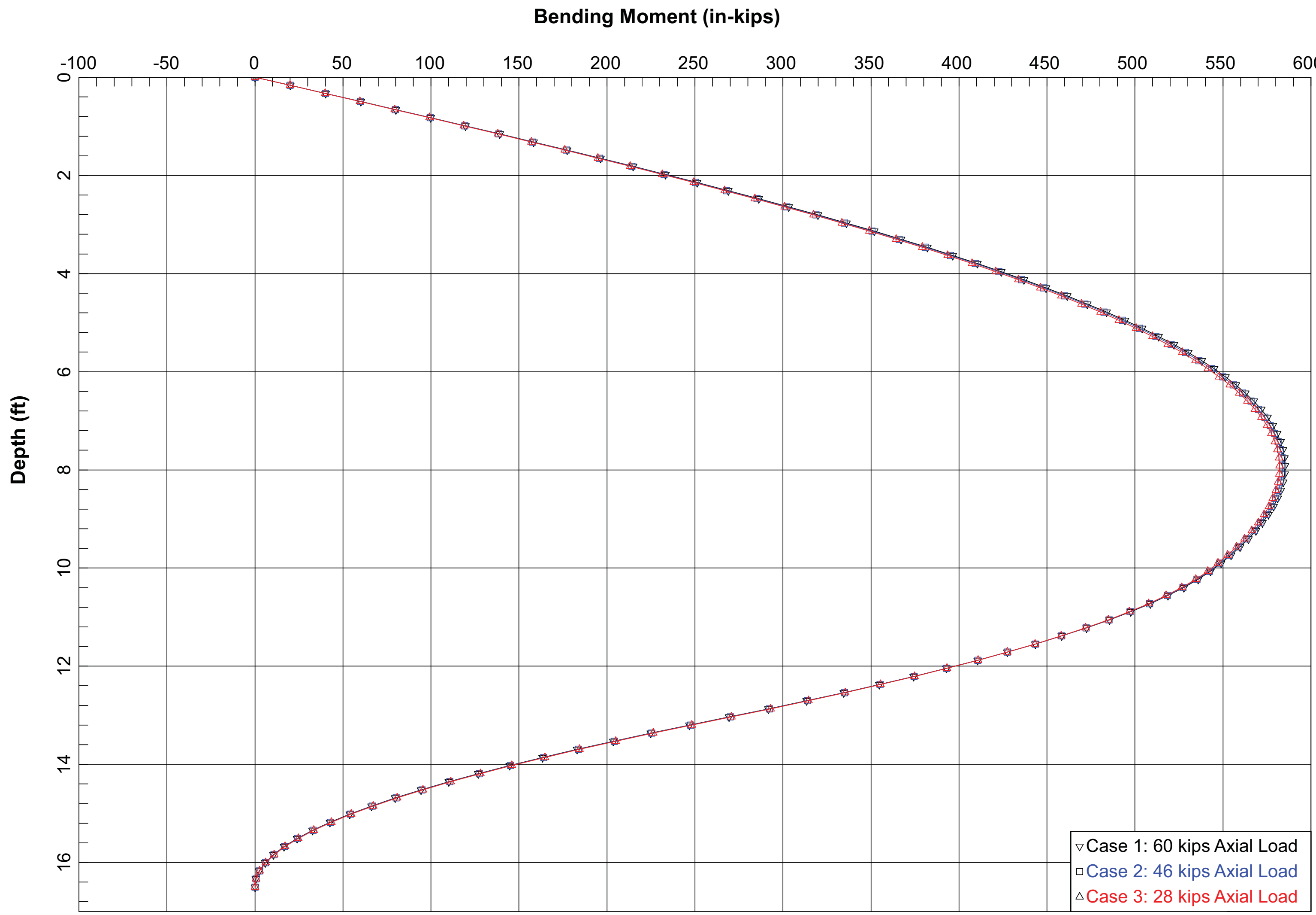
College of Marin Indian Valley Campus Jonas Center Project 1800 Ignacio Boulevard, Novato, California 94949	91-03940-A	August 2017
Geosphere Consultants, Inc	Bending Moment Fixed Head 16.5 ft	Figure 13b

Free Head - 30 in Dia; 13.5 ft Long Pier (10kips Lateral Load)

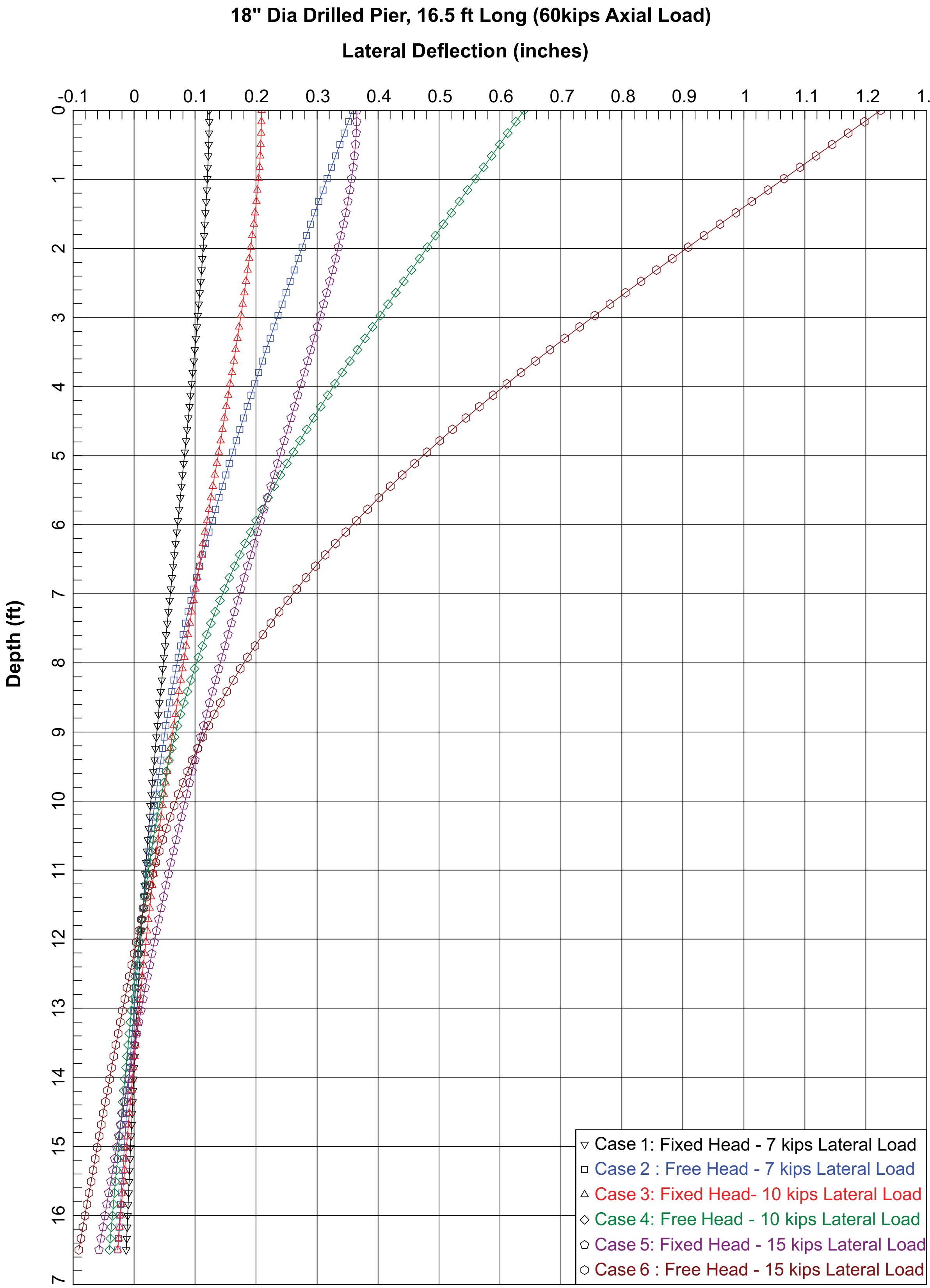


College of Marin Indian Valley Campus Jonas Center Project 1800 Ignacio Boulevard, Novato, California 94949	91-03940-A	August 2017
Geosphere Consultants, Inc AN ETS COMPANY	Bending Moment Free Head 13.5 ft	Figure 13c

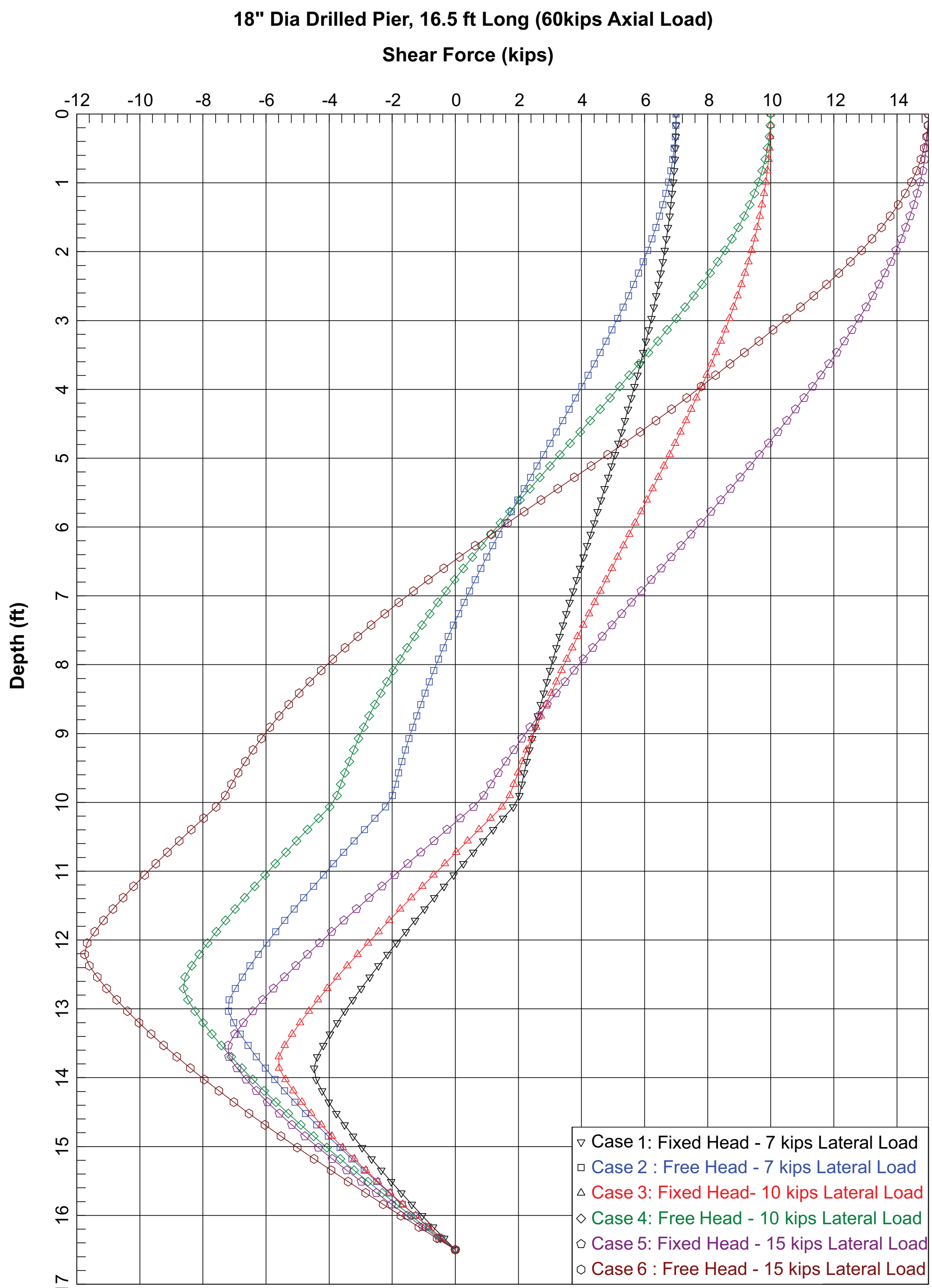
Free Head - 30 in Dia; 16.5 ft Long Pier (10kips Lateral Load)



College of Marin Indian Valley Campus Jonas Center Project 1800 Ignacio Boulevard, Novato, California 94949	91-03940-A	August 2017
Geosphere Consultants, Inc	Bending Moment Free Head 16.5 ft	Figure 13d

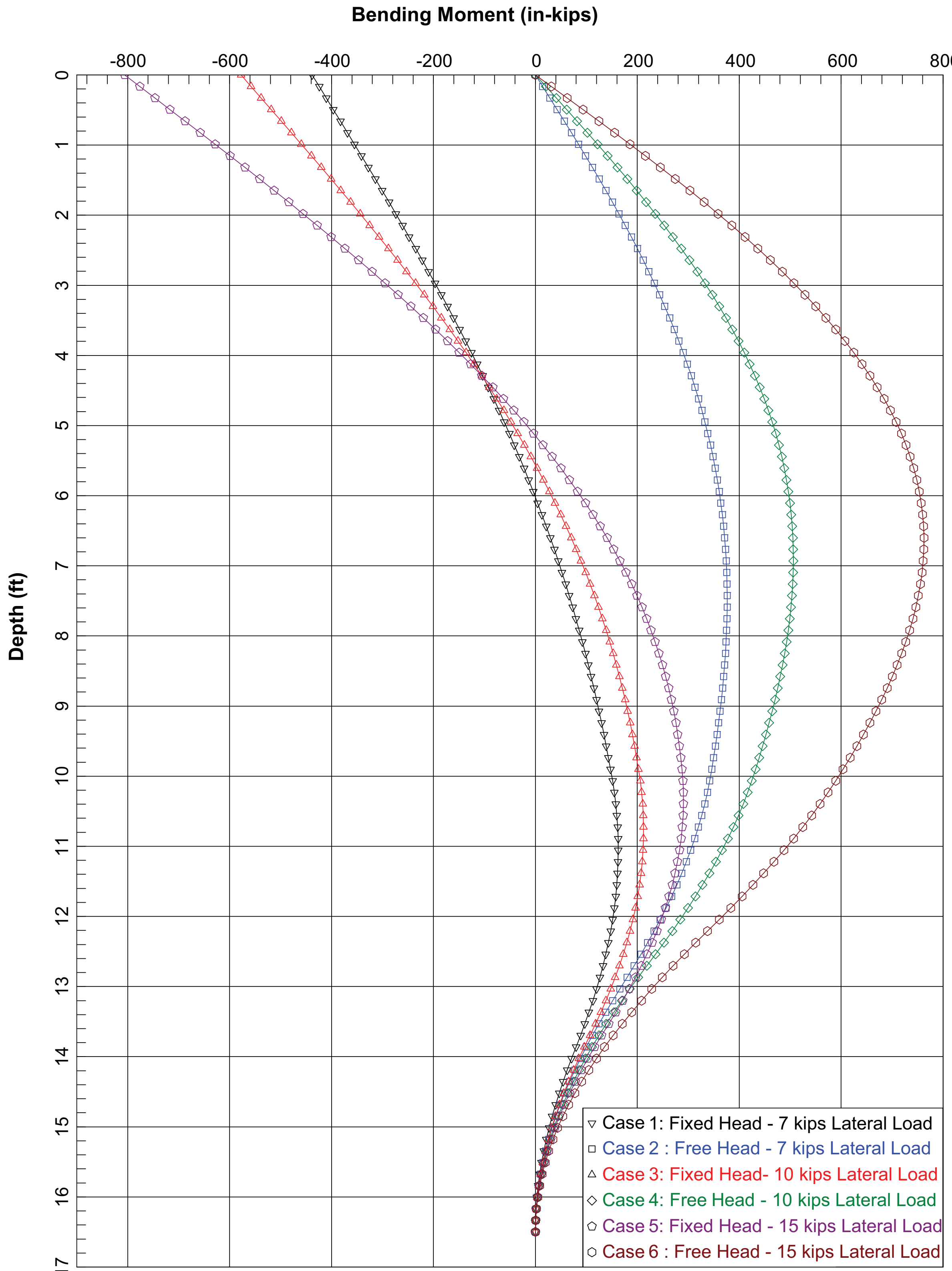


College of Marin Indian Valley Campus Jonas Center Project 1800 Ignacio Boulevard, Novato, California 94949	91-03940-A	August 2017
Geosphere Consultants, Inc. AN ETS COMPANY Geotechnical Engineering · Engineering Geology Environmental Management · Water Resources	Lateral Deflection 18in Dia	Figure 14a



College of Marin Indian Valley Campus Jonas Center Project 1800 Ignacio Boulevard, Novato, California 94949	91-03940-A	August 2017
Geosphere Consultants, Inc.	Shear Force 18in Dia	Figure 14b

18" Dia Drilled Pier, 16.5 ft Long (60kips Axial Load)



College of Marin Indian Valley Campus Jonas Center Project 1800 Ignacio Boulevard, Novato, California 94949	91-03940-A	August 2017
Geosphere Consultants, Inc. AN ETS COMPANY Geotechnical Engineering · Engineering Geology Environmental Management · Water Resources	Bending Moment 18in Dia	Figure 14c

APPENDIX A

FIELD EXPLORATION

Key to Boring Log Symbols
Boring Logs
Previous Boring Logs by Cooper Clark & Associates

UNIFIED SOIL CLASSIFICATION (ASTM D-2487)

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 Passes on No. 4 Sieve	Clean Gravels <5% Fines	Cu≥4 and 1≤Cc≤3	GW	Well-Graded Gravel	
			Cu<4 and/or [Cc<1 or Cc>3]	GP	Poorly-Graded Gravel	
		Gravels with Fines >12% Fines	Fines Classify as ML or MH	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/or [Cc<1 or Cc>3]	SP	Poorly-Graded Sand	
		Sands and Fines >12% Fines	Fines Classify as ML or MH	SM	Silty Sand	
			Fines Classify as CL or CH	SC	Clayey Sand	
Fine Grained Soils ≥50% Passes No. 200 Sieve	Silts and Clays Liquid Limits<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 Silt	
	Silts and Clays Liquid Limits≥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	
Highly Organic Soils		Primarily Organic Matter, Dark in Color and Organic Odor		PT	Peat	

PENETRATION RESISTANCE (RECORDED AS BLOWS/0.5 FEET)

SAND AND GRAVEL		SILT AND CLAY		
RELATIVE DENSITY	N-VALUE (BLOWS/FOOT)*	CONSISTENCY	N-VALUE (BLOWS/FOOT)*	COMPRESSIVE STRENGTH
Very Loose	0 - 3	Very Soft	0 - 1	0 - 0.25
Loose	4 - 10	Soft	2 - 4	0.25 - 0.50
Medium Dense	11 - 29	Medium Stiff	5 - 7	0.50 - 1.0
Dense	30 - 49	Stiff	8 - 14	1.0 - 2.0
Very Dense	50 +	Very Stiff	15 - 29	2.0 - 4.0
		Hard	30 +	Over 4.0

SOIL MOISTURE

DESCRIPTOR	DESCRIPTION
Dry	Dry of Standard Proctor Optimum
Damp	Sand Dry
Moist	Near Standard Proctor Optimum
Wet	Wet of Satandard Proctor Optimum
Saturated	Free Water in Sample

PARTICLES SIZES

COMPONENTS		SIZE OR SIEVE NUMBER
Boulders		Over 12 Inches
Cobbles		3 to 12 Inches
Gravels	-Coarse	3/4 to 3 Inches
	-Fine	Number 4 to 3/4 Inch
Sand	-Coarse	Number 10 to Number 4
	-Medium	Number 40 to Number 10
	-Fine	Number 200 to Number 40
Fines (Silt and Clay)		Below Number 200

Grab Bulk Sample

Standard Penetration Test

2.5 Inch Modified California

Shelby Tube

No Recovery

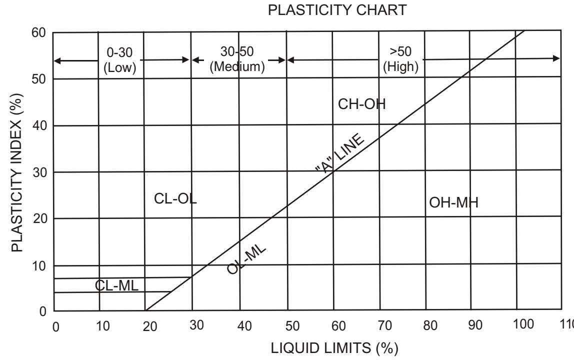
Initial Water Level Reading

Final Water Level Reading

Blow Count
The number of blows of the sampling hammer required to drive the sampler through each of three 6-inch increments. Less than three increments may be reported if more than 50 blows are counted for any increment. The notation 50/5" indicates 50 blows recorded for 5 inches of penetration.

N-Value
Number of blows 140 LB hammer falling 30 inches to drive a 2 inch outside diameter (1-3/8 inch I.D) split barrel sampler the last 12 inches of an 18 inch drive (ASTM-1586 Standard Penetration Test)

PLASTICITY CHART



CU -Consolidated Undrained triaxial test completed. Refer to laboratory results
DS - Results of Direct Shear test in terms of total cohesion (C, KSF) or effective cohesion and friction angles (C', KSF and degrees)
LL - Liquid Limit
PI - Plasticity Index
PP - Pocket Penetrometer test
TV - Torvane Shear Test results in terms of undrained shear strength (KSF)
UC - Unconfined Compression test results in terms of undrained shear strength (KSF)
#200 - Percent passing number 200 sieve
Cu - Coefficient of Uniformity
Cc - Coefficient of Concavity

General Notes

1. The boring locations were determined by pacing, sighting and/or measuring from site features. Locations are approximate. Elevations of borings (if included) were determined by interpolation between plan contours or from another source that will be identified in the report or on the project site plan. The location and elevation of borings should be considered accurate only to the degree implied by the method used.

2. The stratification lines represent the approximate boundary between soil types. The transition may be gradual.

3. Water level readings in the drill holes were recorded at time and under conditions stated on the boring logs. This data has been reviewed and interpretations have been made in the text of this report. However, it must be noted that fluctuations in the level of the groundwater may occur due to variations in rainfall, tides, temperature and other factors at the time measurements were made.

4. The boring logs and attached data should only be used in accordance with the report.

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KEY TO EXPLORATORY BORING LOGS

Geosphere Consultants, Inc.
AN ETS COMPANY
Geotechnical Engineering • Engineering Geology
Environmental Management • Water Resources
2001 Crow Canyon Rd, Ste 210
CA 94583
Telephone: 9253147180
Fax: 9258557140

BORING NUMBER B-1

CLIENT Marin Community College District
PROJECT NAME Jonas Center Project
PROJECT NUMBER 91-03940-A
PROJECT LOCATION 1800 Ignacio Boulevard, Novato, CA 94949
DATE STARTED 5/25/17 **COMPLETED** 5/25/17
GROUND ELEVATION 182 ft **HOLE SIZE** 4"
DRILLING CONTRACTOR Geo-Ex
GROUND WATER LEVELS:
DRILLING METHOD SFA CME-45
AT TIME OF DRILLING 18.00 ft / Elev 164.00 ft
LOGGED BY AL **CHECKED BY** CTD
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	SPT BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	FINES CONTENT (%)
0		**2" AC**										
		(SC) **CLAYEY SAND** : Med dense, dark brown, very moist.										
			MC 1-1		2-3-7 (10)	1.8 2.8	109	17				
		becomes med dense, brown, moist, mottled, pocket of sand.	SPT 1-2		5-6-10 (16)							
5												
10		becomes dense, reddish brown and yellow, coarse grained sand to rock fragments	MC 1-3		10-14-17 (31)							
		(CL) **SILTY CLAY** : Stiff, olive brown, moist.										
15			SPT 1-4		3-5-6 (11)	1.5						
	∇											
20			MC 1-5		3-5-8 (13)	1.8						
		(CLS) **SANDY CLAY** : Stiff, reddish brown, moist to very moist, with up to 1.5" rock fragments.										

Bottom of borehole at 20.0 feet.

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BORING NUMBER B-2

CLIENT Marin Community College District
PROJECT NAME Jonas Center Project
PROJECT NUMBER 91-03940-A
PROJECT LOCATION 1800 Ignacio Boulevard, Novato, CA 94949
DATE STARTED 5/25/17 **COMPLETED** 5/25/17
GROUND ELEVATION 179 ft **HOLE SIZE** 4"
DRILLING CONTRACTOR Geo-Ex
GROUND WATER LEVELS:
DRILLING METHOD SFA CME-45
AT TIME OF DRILLING ---
LOGGED BY AL **CHECKED BY** CTD
AT END OF DRILLING ---
NOTES
AFTER DRILLING --- No groundwater encountered.

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	SPT BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		**3" TOPSOIL**										
		(SC) **CLAYEY SAND** : Dense, dark brown, moist, pebbles. [FILL]										
			MC 2-1		15-20-21 (41)		130	11				
5		becomes med dense.	MC 2-2		7-13-14 (27)		114	15	22	16	6	
10		(SC) **CLAYEY SAND** : Loose, dark brown, moist, orange pebbles.	MC 2-3		3-4-5 (9)	0.75	109	12				49
15		fine content increased, becomes very stiff with up to 1.5" fragments and pockets of red.	MC 2-4		7-10-17 (27)	2.5						

Bottom of borehole at 15.0 feet.

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BORING NUMBER B-3

CLIENT Marin Community College District
PROJECT NAME Jonas Center Project
PROJECT NUMBER 91-03940-A
PROJECT LOCATION 1800 Ignacio Boulevard, Novato, CA 94949
DATE STARTED 5/25/17 **COMPLETED** 5/25/17
GROUND ELEVATION 181 ft **HOLE SIZE** 4"
DRILLING CONTRACTOR Geo-Ex
GROUND WATER LEVELS:
DRILLING METHOD SFA CME-45
AT TIME OF DRILLING ---
LOGGED BY AL **CHECKED BY** CTD
AT END OF DRILLING ---
NOTES
AFTER DRILLING --- No groundwater encountered.

DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	SPT BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0	**3" TOPSOIL**										
	(SM) **SILTY SAND** : Med dense, dark brown, dry-moist, trace clay, freq organics.										
		MC 3-1		4-5-6 (11)		88	10				
		MC 3-2		5-4-4 (8)		105	14				
5	(SC) **CLAYEY SAND** : Loose, dark brown, moist.										
	(CL) **LEAN CLAY** : Very stiff, reddish brown, grey clay pocket, med-high plasticity.										
10		MC 3-3		5-10-14 (24)	>4.5						
	(CLS) **SANDY CLAY** : Very stiff, reddish brown, grey & red pockets, rock fragments.										
15		SPT 3-4		6-10-15 (25)							
	(SC) **CLAYEY SAND** : Very dense, olive brown, pockets of grey clay.										
20		SPT 3-5		10-15-35 (50)							
	SANDSTONE : Tan brown to gray, highly weathered, mod hard, mod strong. auger refusal past 22'.	SPT 3-6		21-50/5"							

Bottom of borehole at 22.9 feet.

BORING NUMBER B-4

Geosphere Consultants, Inc.
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Geotechnical Engineering • Engineering Geology
Environmental Management • Water Resources

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CA 94583
Telephone: 9253147180
Fax: 9258557140

CLIENT Marin Community College District
PROJECT NAME Jonas Center Project
PROJECT NUMBER 91-03940-A
PROJECT LOCATION 1800 Ignacio Boulevard, Novato, CA 94949
DATE STARTED 5/25/17 **COMPLETED** 5/25/17
GROUND ELEVATION 192 ft **HOLE SIZE** 4"
DRILLING CONTRACTOR Geo-Ex
GROUND WATER LEVELS:
DRILLING METHOD SFA CME-45
AT TIME OF DRILLING ---
LOGGED BY AL **CHECKED BY** CTD
AT END OF DRILLING ---
NOTES
AFTER DRILLING --- No groundwater encountered.

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	SPT BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		**3" TOPSOIL**										
		(SC) **CLAYEY SAND** : Med dense, brown, moist, organics.	MC 4-1		3-4-8 (12)	2.8 >4.5	104	12				
		(SM) **SILTY SAND** : Med dense, reddish brown, trace clay.	SPT 4-2		6-5-6 (11)							48
5												
		SANDSTONE : Tan brown to gray, highly weathered, mod hard, mod strong.	SPT 4-3		50/4"							

Bottom of borehole at 8.8 feet.

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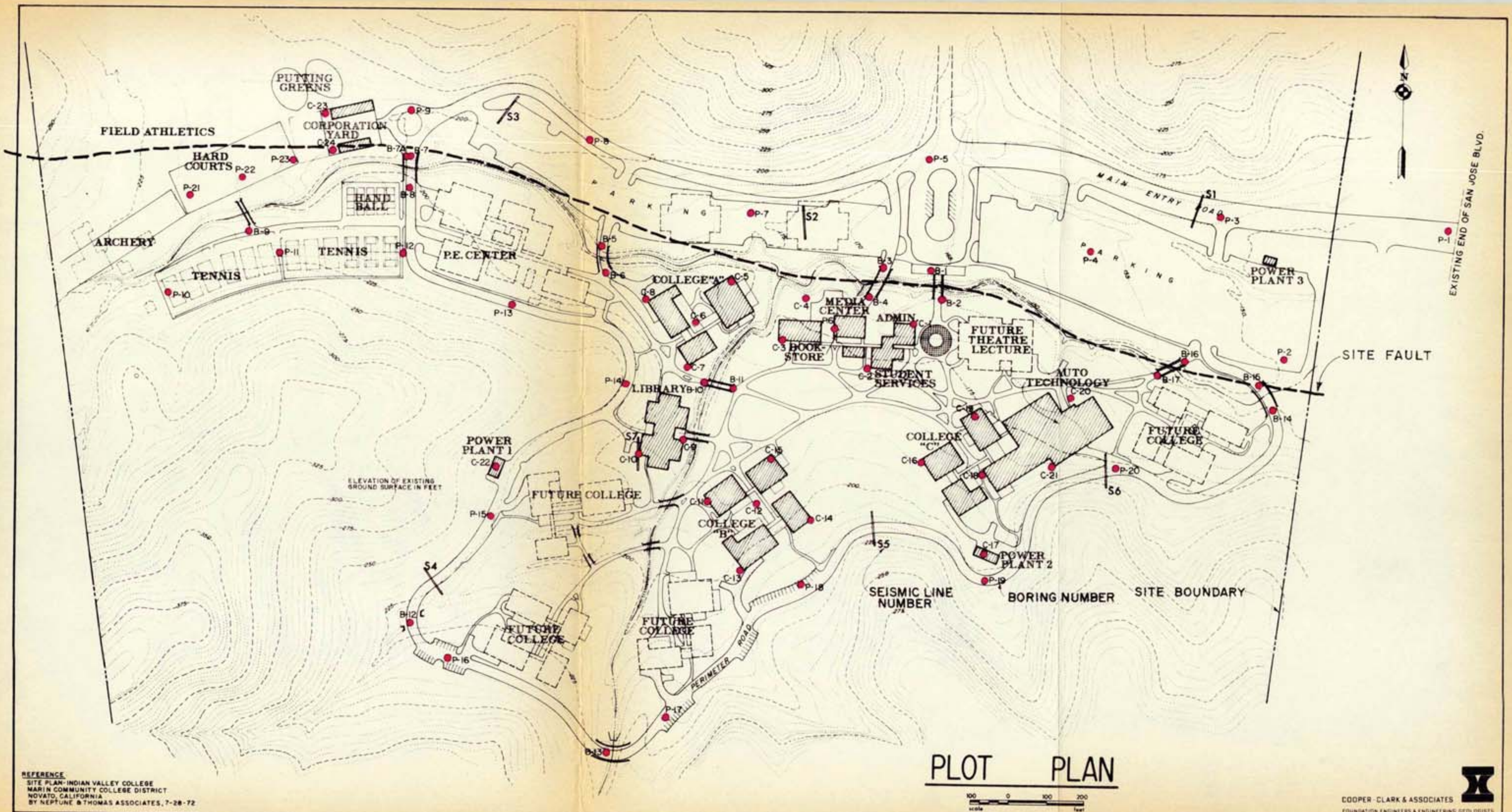
Telephone: 9253147180
Fax: 9258557140

BORING NUMBER B-5

CLIENT Marin Community College District
PROJECT NAME Jonas Center Project
PROJECT NUMBER 91-03940-A
PROJECT LOCATION 1800 Ignacio Boulevard, Novato, CA 94949
DATE STARTED 5/25/17 **COMPLETED** 5/25/17
GROUND ELEVATION 187 ft **HOLE SIZE** 4"
DRILLING CONTRACTOR Geo-Ex
GROUND WATER LEVELS:
DRILLING METHOD SFA CME-45
AT TIME OF DRILLING ---
LOGGED BY AL **CHECKED BY** CTD
AT END OF DRILLING ---
NOTES
AFTER DRILLING --- No groundwater encountered.

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	SPT BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		**3" TOPSOIL**										
		(SC) **CLAYEY SAND** : Loose, dark brown, moist, organics.										
			MC 5-1		3-2-3 (5)	2.0	88	14	23	16	7	
		(CL) **SILTY CLAY** : Stiff, dark brown, moist, sand pockets, mottled, organics.	SPT 5-2		2-4-8 (12)							
5												
		(CLS) **SANDY CLAY** : Hard, reddish brown, coarse sand to rock gragments, grey clay pockets.										
10			SPT 5-3		6-13-18 (31)							
15		no fragments.	SPT 5-4		4-15-24 (39)							

Bottom of borehole at 15.0 feet.



PUTTING GREENS
CORPORATION YARD
FIELD ATHLETICS
HARD COURTS
HAND BALL
ARCHERY
TENNIS
TENNIS
P.E. CENTER
PARKING
MAIN ENTRY ROAD
PARKING
POWER PLANT 3
EXISTING END OF SAN JOSE BLVD.
COLLEGE "A"
MEDIA CENTER
ADMIN.
BOOK-STORE
STUDENT SERVICES
FUTURE THEATRE LECTURE
AUTO TECHNOLOGY
SITE FAULT
LIBRARY
POWER PLANT 1
ELEVATION OF EXISTING GROUND SURFACE IN FEET
FUTURE COLLEGE
COLLEGE "B"
COLLEGE "C"
FUTURE COLLEGE
POWER PLANT 2
SEISMIC LINE NUMBER
BORING NUMBER
SITE BOUNDARY
FUTURE COLLEGE
FUTURE COLLEGE
PERIMETER ROAD
S1
S2
S3
S4
S5
S6
S7
PLOT PLAN
REFERENCE
SITE PLAN-INDIAN VALLEY COLLEGE
MARIN COMMUNITY COLLEGE DISTRICT
NOVATO, CALIFORNIA
BY NEPTUNE & THOMAS ASSOCIATES, 7-28-72
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FOUNDATION ENGINEERS & ENGINEERING GEOLOGISTS

PLATE 1

BORING C-5

ELEVATION 177 FEET

% FINES (-NO. 200)	LIQUID LIMIT	PLASTICITY INDEX	TYPE OF STRENGTH TEST	TEST SURCHARGE PRESSURE, LBS/SQ FT	TEST MOISTURE CONTENT, %	SHEAR STRENGTH, LBS/SQ FT	NATURAL MOISTURE CONTENT, %	DRY DENSITY, LBS/CU FT	BLOWS/FOOT
79	25	8	DSX DSX	100 2000	20.2 20.2	110 900	5.7	98	4
			DSX DSX	400 2000	16.2 16.2	740 1570	7.7	111	12
			DS DS	700 2000	Natural Natural	6160 8000	9.3	116	12
			DS DS	1300 3000	Natural Natural	4360 4540	16.5	110	12
			DS	2200	Natural	1850	18.5	110	4
			DS	3000	Natural	3560	17.9	107	6

DEPTH IN FEET

- 0 — Grayish-brown sandy clay (CL)(very stiff, dry)
- 10 — (Gray silty clay layer)
- Brown sandy clay (CL)(very stiff, moist)
- Brown clayey sand (SC)(dense, moist)
- 15 — Brown sandy clay (CL)(stiff, wet)
- (Grading occasional gravelly layers)
- 20 — (Grading mottled brown and gray, very stiff)
- 25 — NOTE: No free water encountered.

BORING C-6

ELEVATION 185 FEET

% FINES (-NO. 200)	LIQUID LIMIT	PLASTICITY INDEX	TYPE OF STRENGTH TEST	TEST SURCHARGE PRESSURE, LBS/SQ FT	TEST MOISTURE CONTENT, %	SHEAR STRENGTH, LBS/SQ FT	NATURAL MOISTURE CONTENT, %	DRY DENSITY, LBS/CU FT	BLOWS/FOOT
67	22	7	DSX DSX	200 2000	16.5 16.5	440 1500	7.1	108	10
			DSX DSX	450 2000	14.8 14.8	1830 3530	10.2	121	17
			DS	800	Natural	2630	14.6	120	12
			DS	1300	Natural	2890	13.3	120	14
			DS	2200	Natural	3250	15.7	115	9

DEPTH IN FEET

- 0 — Yellowish-brown sandy clay (CL-ML) (very stiff, dry)
- (Grading gravel)
- 5 — Yellowish-brown sandy clay with occasional gravelly zones (CL)(very stiff, moist)
- 10
- 15
- Yellowish-brown sandstone, weathered
- 20 — (Refusal to bucket rig)
- 25 — NOTE: No free water encountered.

BORING C-7

ELEVATION 190 FEET

% FINES (-NO. 200)	LIQUID LIMIT	PLASTICITY INDEX	TYPE OF STRENGTH TEST	TEST SURCHARGE PRESSURE, LBS/SQ FT	TEST MOISTURE CONTENT, %	SHEAR STRENGTH, LBS/SQ FT	NATURAL MOISTURE CONTENT, %	DRY DENSITY, LBS/CU FT
							6.8	91
							18.9	106
							11.3	102

DEPTH IN FEET

- 0 — Brown sandy clay (CL)(very stiff, dry)
- (Grading reddish-brown, some gravel)
- 5 — (Grading yellowish-brown)
- 10 — Yellowish-brown sandstone, weathered
- (Refusal to auger)
- 15 — NOTE: No free water encountered.

Revisions: By ______ Date ______ By ______ Date ______

By TT Date 9-15-72

Checked By ______

Job Number 730-A2 Name Marin Community College District Location Novato

BORING LOGS

COOPER · CLARK & ASSOCIATES

FOUNDATION ENGINEERS & ENGINEERING GEOLOGISTS

PLATE A-11

Revisions:
By ____ Date ____
By ____ Date ____

By TT Date 9-15-72
Checked By ____
Job Number 730-A2 Name Marin Community College District Location Novato

BORING C-8

ELEVATION 191 FEET

% Fines (-No. 200)	Liquid Limit	Plasticity Index	Type of Strength Test	Test Surcharge Pressure, lbs/sq ft	Test Moisture Content, %	Shear Strength, lbs/sq ft	Natural Moisture Content, %	Dry Density, lbs/cu ft	Blows/Foot
							11.7	116	14
							17.1	110	10
							15.0	108	11
							19.4	107	7
							14.7	119	14
									32/10"
							13.9	111	

Depth in feet: 0, 5, 10, 15, 20, 25

- Yellowish-brown clayey sand (SC)(dense, dry)
- Yellowish-brown sandy clay (CL)(very stiff, dry)
- (Grading occasional gravel, moist)
- (Grading grayish-brown)
- (Grading yellowish-brown, rock fragments)

NOTE: No free water encountered.

BORING C-9

ELEVATION 186 FEET

% Fines (-No. 200)	Liquid Limit	Plasticity Index	Type of Strength Test	Test Surcharge Pressure, lbs/sq ft	Test Moisture Content, %	Shear Strength, lbs/sq ft	Natural Moisture Content, %	Dry Density, lbs/cu ft
74	27	11						
			DS DS	450 2000	Natural Natural	2300 4200	7.3	98
			DS DS	950 3000	Natural Natural	3030 3580	9.9	104
							12.6	112
			DS	2000	Natural	3330	18.3	106
			DS	2600	Natural	2660	17.8	108
							14.9	115

Depth in feet: 0, 5, 10, 15, 20, 25

- Grayish-brown sandy clay with roots (CL) (very stiff, dry)
- (Grading yellowish-brown, without roots)
- (Grading less sand, wet)
- Yellowish-brown silty clay with gravel (CL) (stiff, wet)

NOTE: No free water encountered.

BORING C-10

ELEVATION 193 FEET

% Fines (-No. 200)	Liquid Limit	Plasticity Index	Type of Strength Test	Test Surcharge Pressure, lbs/sq ft	Test Moisture Content, %	Shear Strength, lbs/sq ft	Natural Moisture Content, %	Dry Density, lbs/cu ft
80	21	3						
							14.2	107
			DS	800	Natural	4040	12.6	106

Depth in feet: 0, 5, 10, 15, 20

- Yellowish-brown sandy clay (CL-ML) (very stiff, dry)
- (Grading moist)
- (Grading less sand)
- (Grading some gravel, hard)
- Reddish-brown chert, hard (Refusal to auger)

NOTE: No free water encountered.

BORING LOGS

COOPER · CLARK & ASSOCIATES
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T1-2... -68)

PLATE A-1M

By TT Date 9-15-72
Checked By ______ Date ______
Job Number 730-A2 Name Marin Community College District Location Novato

Revisions:
By ______ Date ______
By ______ Date ______

BORING P-9

ELEVATION 200 FEET

% Fines (-No. 200)	Liquid Limit	Plasticity Index	Type of Strength Test	Test Surcharge Pressure, Lbs/Sq Ft	Test Moisture Content, %	Shear Strength, Lbs/Sq Ft	Natural Moisture Content, %	Dry Density, Lbs/Cu Ft
			DSX DSX	400 2000	19.7 19.7	400 1220	7.2	102
			DSX DSX	750 2000	17.8 17.8	650 1300	6.8	107
			DS DS	1200 3000	Natural Natural	1260 2260	17.9	110
			DS DS	1450 4000	Natural Natural	1880 2810	17.4	111

Depth in feet: 0, 5, 10, 15

- 0: Brown clayey sand (SC)(dense, dry)
- 5: Brown sandy clay with gravel (CL)(very stiff, dry)
- (Grading stiff, moist)
- Water encountered 8/9/72
- (Grading mottled brown and gray)

BORING P-10

ELEVATION 228½ FEET

% Fines (-No. 200)	Liquid Limit	Plasticity Index	Type of Strength Test	Test Surcharge Pressure, Lbs/Sq Ft	Test Moisture Content, %	Shear Strength, Lbs/Sq Ft	Natural Moisture Content, %	Dry Density, Lbs/Cu Ft
							12.4	116

Depth in feet: 0, 5, 10

- 0: Yellowish-brown sandy clay (CL)(very stiff, dry)
- (Grading moist)
- (Grading gravelly)

BORING P-11

ELEVATION 218 FEET

% Fines (-No. 200)	Liquid Limit	Plasticity Index	Type of Strength Test	Test Surcharge Pressure, Lbs/Sq Ft	Test Moisture Content, %	Shear Strength, Lbs/Sq Ft	Natural Moisture Content, %	Dry Density, Lbs/Cu Ft
							14.6	111

Depth in feet: 0, 5, 10

- 0: Brown sandy clay (CL-ML) (very stiff, dry)
- Yellowish-brown sandy clay (CL)(very stiff, moist)
- (Refusal to auger on sandstone)

BORING P-12

ELEVATION 210 FEET

% Fines (-No. 200)	Liquid Limit	Plasticity Index	Type of Strength Test	Test Surcharge Pressure, Lbs/Sq Ft	Test Moisture Content, %	Shear Strength, Lbs/Sq Ft	Natural Moisture Content, %	Dry Density, Lbs/Cu Ft
	24	6					15.5	106

Depth in feet: 0, 5, 10

- 0: Brown sandy clay (CL–ML)(very stiff, dry)
- Yellowish-brown sandy clay with gravel (CL) (very stiff, moist)

BORING P-13

ELEVATION 208½ FEET

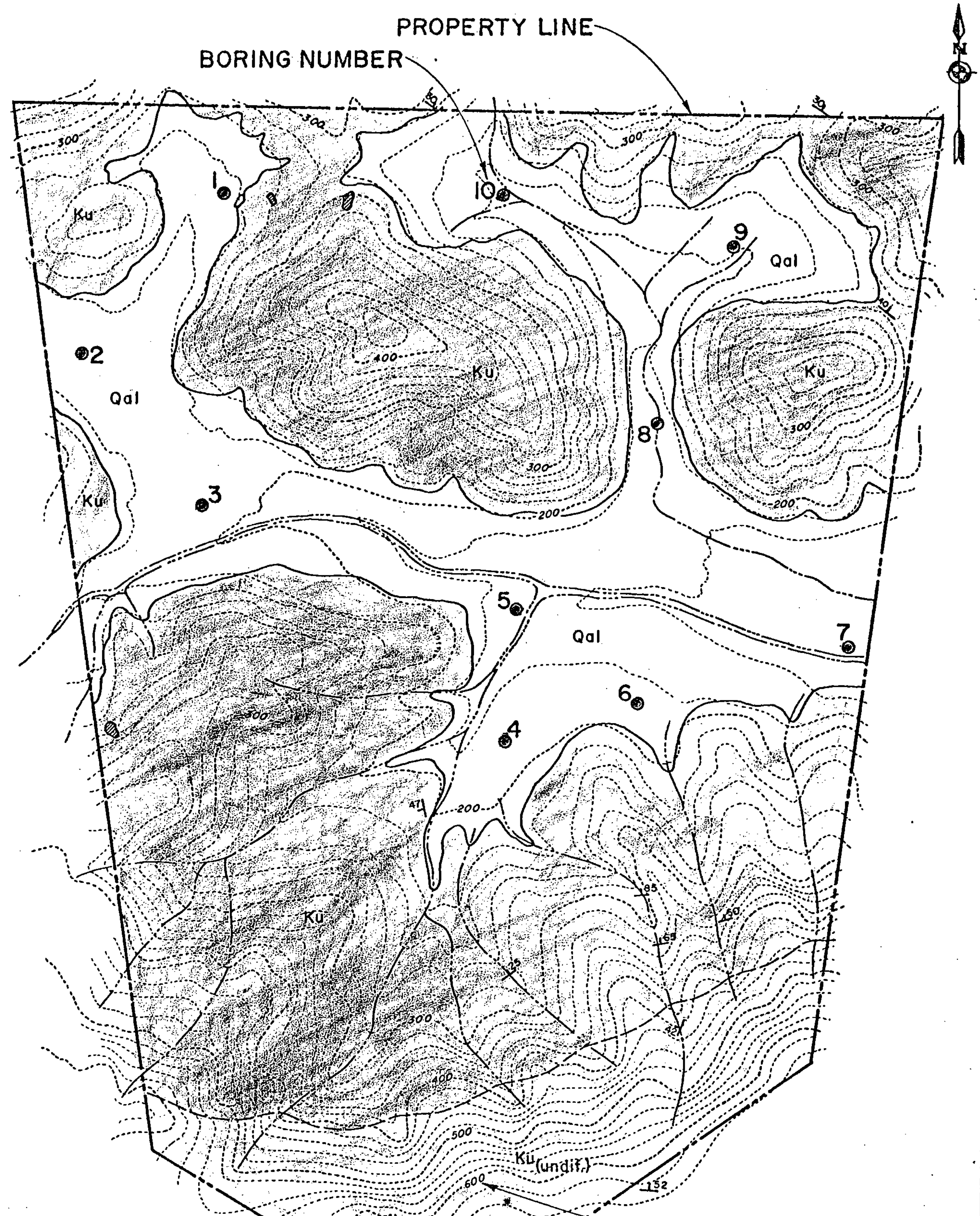
% Fines (-No. 200)	Liquid Limit	Plasticity Index	Type of Strength Test	Test Surcharge Pressure, Lbs/Sq Ft	Test Moisture Content, %	Shear Strength, Lbs/Sq Ft	Natural Moisture Content, %	Dry Density, Lbs/Cu Ft

Depth in feet: 0, 5, 10

- 0: Brown sandy clay (CL)(very stiff, dry)
- (Grading moist)

BORING LOGS

COOPER · CLARK & ASSOCIATES
FOUNDATION ENGINEERS & ENGINEERING GEOLOGISTS



LEGEND:

Qal	ALLUVIAL DEPOSITS
Ku(undif.)	MASSIVE ARKOSIC SANDSTONES WITH MINOR SHALE INTERBEDS
Ku	SHALES AND SILTSTONES WITH MINOR SANDSTONE INTERBEDS
30	STRIKE AND DIP OF BEDS
	CHERT
	SLIDES

PLOT PLAN

SCALE : 1" = 600'

REFERENCE:

BASED ON CITY OF NOVATO AND WATERSHED TOPOGRAPHIC MAPS PREPARED BY MURRAY & McCORMICK, INC., CONSULTING CIVIL ENGINEERS, NOVATO, CALIF.

COOPER - CLARK & ASSOCIATES

FOUNDATION ENGINEERS & ENGINEERING GEOLOGISTS

DRAWN BY — DATE 5-26-
CHECKED BY — DATE

JOB NUMBER — NAME

LOCATION Novato, California

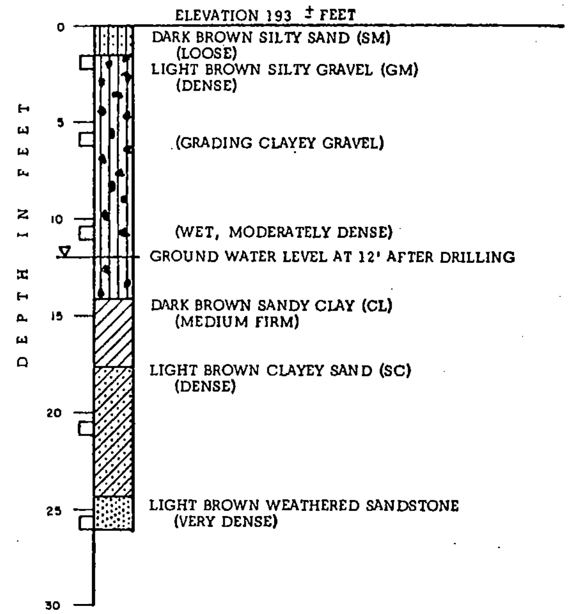
PLATE 2

LOCATION Novato Calif.

CHECKED BY T.C. DATE 5-29-67

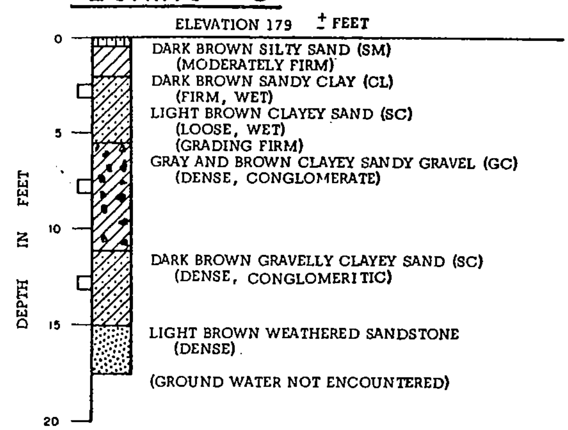
CLASSIFICATION DATA			STRENGTH DATA				MOISTURE-DENSITY DATA	
% FINES (-NO. 200)	LIQUID LIMIT	PLASTICITY INDEX	TYPE OF STRENGTH TEST	TEST SURCHARGE PRESSURE, LBS/SQ FT	TEST MOISTURE CONTENT, %	SHEAR STRENGTH, LBS/SQ FT	NATURAL MOISTURE CONTENT, %	DRY DENSITY, LBS/CU FT
							12.6	108
							10.2	121
							20.0	105
							12.7	121
							11.6	126

BORING 4



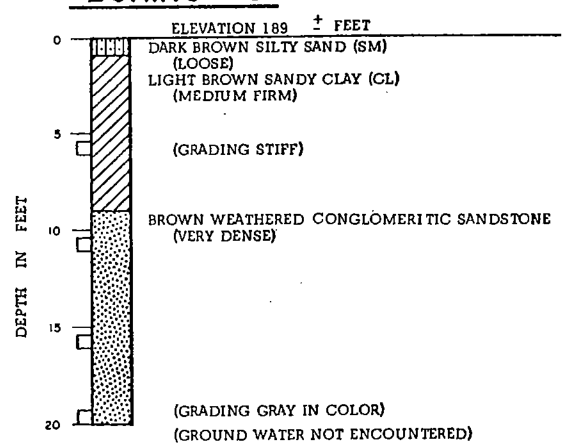
CLASSIFICATION DATA			STRENGTH DATA				MOISTURE-DENSITY DATA	
% FINES (-NO. 200)	LIQUID LIMIT	PLASTICITY INDEX	TYPE OF STRENGTH TEST	TEST SURCHARGE PRESSURE, LBS/SQ FT	TEST MOISTURE CONTENT, %	SHEAR STRENGTH, LBS/SQ FT	NATURAL MOISTURE CONTENT, %	DRY DENSITY, LBS/CU FT
			DS	400	NATURAL	550	16.3	112
			DS DS	500 1500	NATURAL NATURAL	1300 2200	14.3 14.7	115 114
							12.6	121

BORING 5



CLASSIFICATION DATA			STRENGTH DATA				MOISTURE-DENSITY DATA	
% FINES (-NO. 200)	LIQUID LIMIT	PLASTICITY INDEX	TYPE OF STRENGTH TEST	TEST SURCHARGE PRESSURE, LBS/SQ FT	TEST MOISTURE CONTENT, %	SHEAR STRENGTH, LBS/SQ FT	NATURAL MOISTURE CONTENT, %	DRY DENSITY, LBS/CU FT
			DS	700	NATURAL	1800	13.5	112
							10.6	121
							8.3	126
							7.8	124

BORING 6



BORING LOGS

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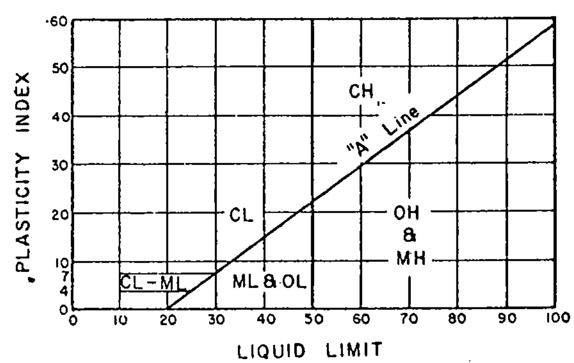
PLATE 3-B

MAJOR DIVISIONS		SYMBOLS	TYPICAL NAMES
COARSE GRAINED SOILS (More than 1/2 of soil > no. 200 sieve size)	GRAVELS (More than 1/2 of coarse fraction > no. 4 sieve size)	GW	Well graded gravels or 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
		GC	Clayey gravels, gravel-sand-clay mixtures
	SANDS (More than 1/2 of coarse fraction < no. 4 sieve size)	SW	Well graded sands or gravelly sands, little or no fines
		SP	Poorly graded sands or gravelly sands, little or no fines
		SM	Silty sands, sand-silt mixtures
		SC	Clayey sands, sand-clay mixtures
FINE GRAINED SOILS (More than 1/2 of soil < no. 200 sieve size)	SILTS & CLAYS LL < 50	ML	Inorganic silts and very fine sands, rock flour, 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
	SILTS & CLAYS LL > 50	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 silty clays, organic silts
HIGHLY ORGANIC SOILS		Pt	Peat and other highly organic soils

CLASSIFICATION CHART
(Unified Soil Classification System)

CLASSIFICATION	RANGE OF GRAIN SIZES	
	U.S. Standard Sieve Size	Grain Size in Millimeters
BOULDERS	Above 12"	Above 305
COBBLES	12" to 3"	305 to 76.2
GRAVEL	3" to No. 4	76.2 to 4.76
coarse	3" to 3/4"	76.2 to 19.1
fine	3/4" to No. 4	19.1 to 4.76
SAND	No. 4 to No. 200	4.76 to 0.074
coarse	No. 4 to No. 10	4.76 to 2.00
medium	No. 10 to No. 40	2.00 to 0.420
fine	No. 40 to No. 200	0.420 to 0.074
SILT & CLAY	Below No. 200	Below 0.074

GRAIN SIZE CHART



PLASTICITY CHART

METHOD OF SOIL CLASSIFICATION

COOPER-CLARK & ASSOCIATES
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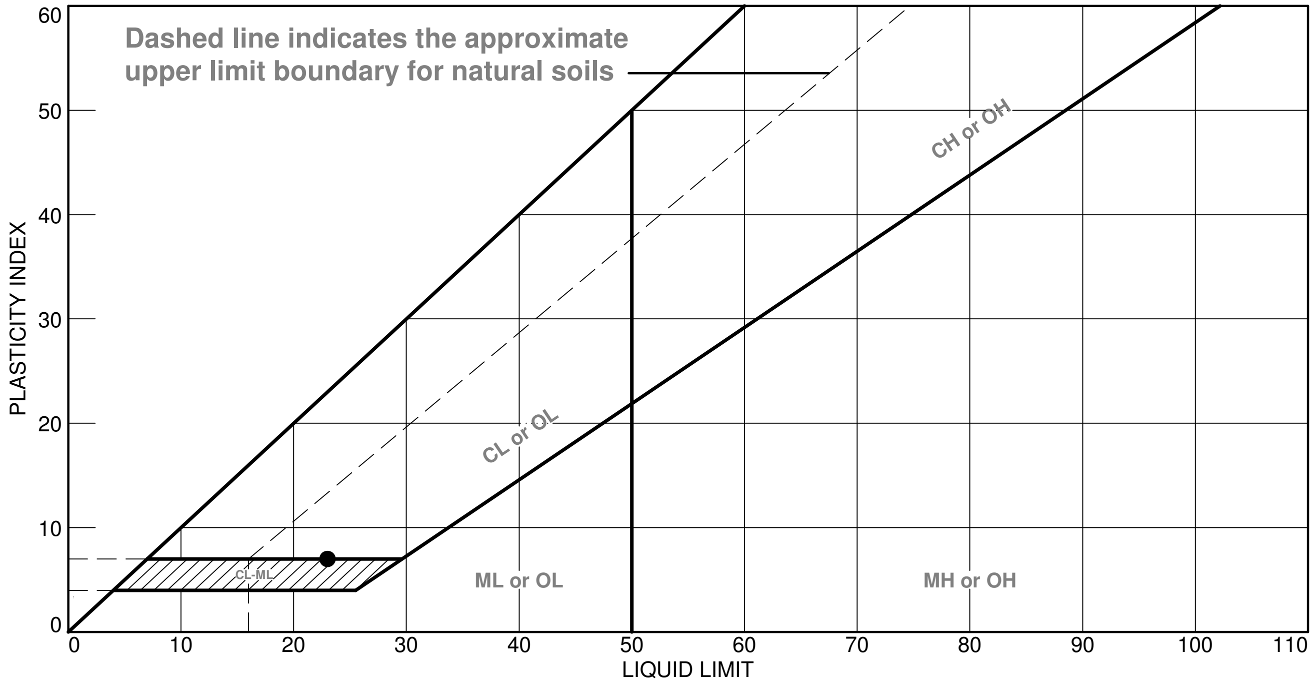
APPENDIX B

LABORATORY TEST RESULTS

Liquid and Plastic Limits Test Report (2)
Particle Size Distribution Report (2)
R-Value Test Report
Corrosivity Tests Summary
Previous Laboratory Test Results by Cooper-Clark & Associates

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Tan Clayey Sand Sample #1 Sampled on 6/7/17 by A. Lim	23	16	7			

Project No. 9103940A **Client:**

Project: MCCD - Jonas Center Project (GES & GHR)

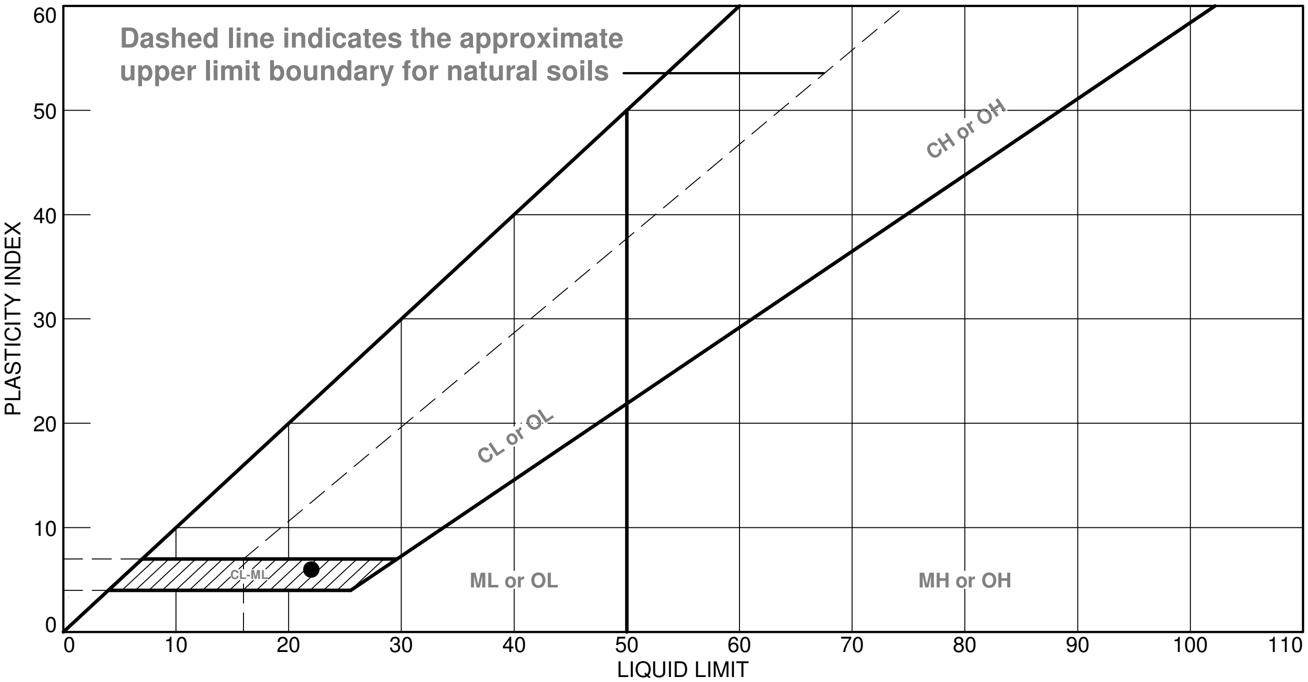
Location: B5-5-1@2.5
Sample Number: 10S170615-3

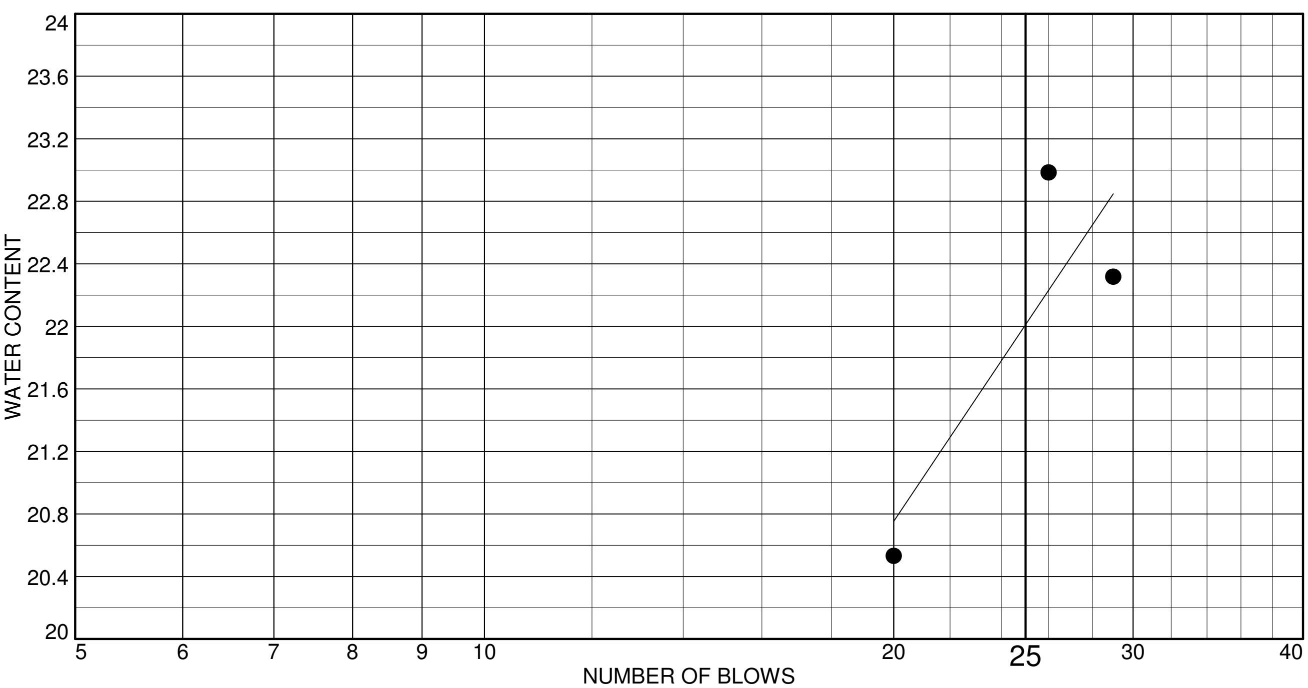
Remarks:

CONSOLIDATED ENGINEERING LABORATORIES

San Ramon, California

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Brown Clayey Sand Sampled on 6/7/17 by A. Lim	22	16	6			

Project No. 9103940A **Client:**

Project: MCCD - Jonas Center Project (GES & GHR)

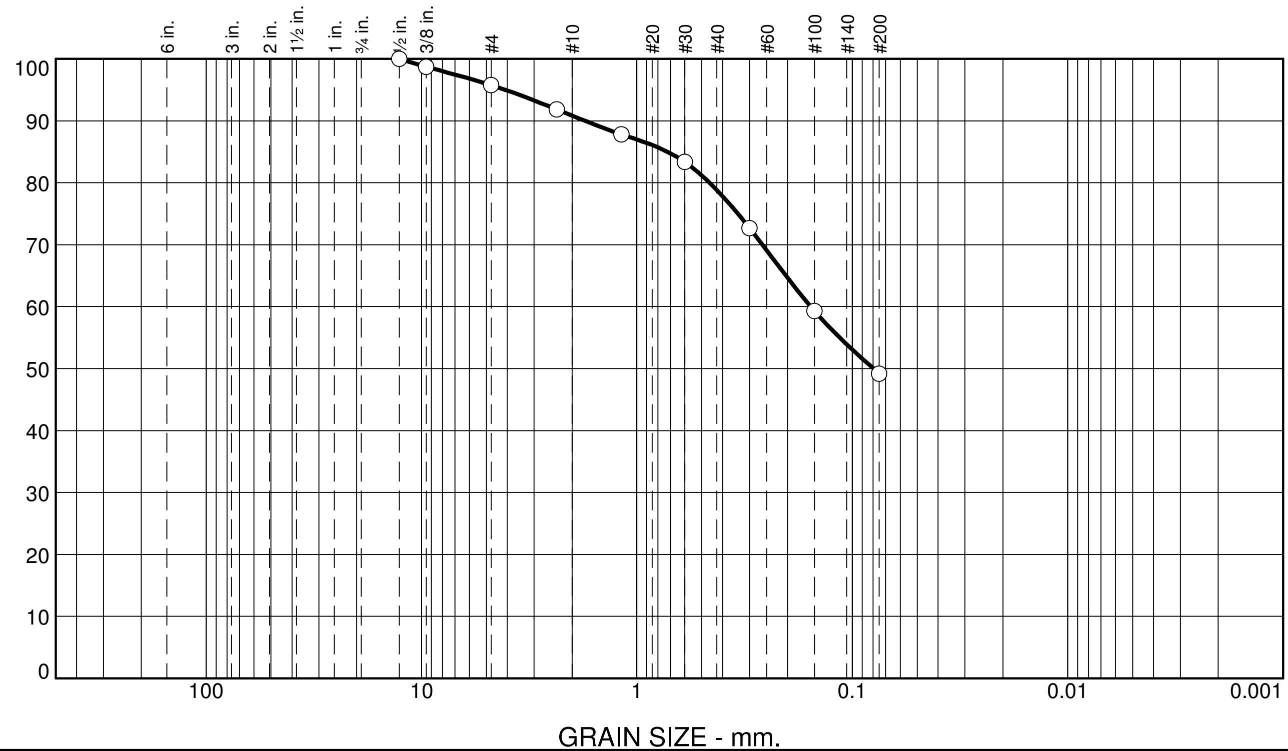
Location: B2-2-2@4.5
Sample Number: 10S170615-3

CONSOLIDATED ENGINEERING LABORATORIES

San Ramon, California

Remarks:

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	0	4	5	12	30	49	

TEST RESULTS

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1/2"	100		
3/8"	99		
#4	96		
#8	92		
#16	88		
#30	83		
#50	73		
#100	59		
#200	49		

* (no specification provided)

Material Description

Tan Clayey Sand Sample #1
Sampled on 6/7/17 by A. Lim

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= AASHTO (M 145)=

Coefficients

D_{90}= 1.7408 D_{85}= 0.7210 D_{60}= 0.1559
D_{50}= 0.0798 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

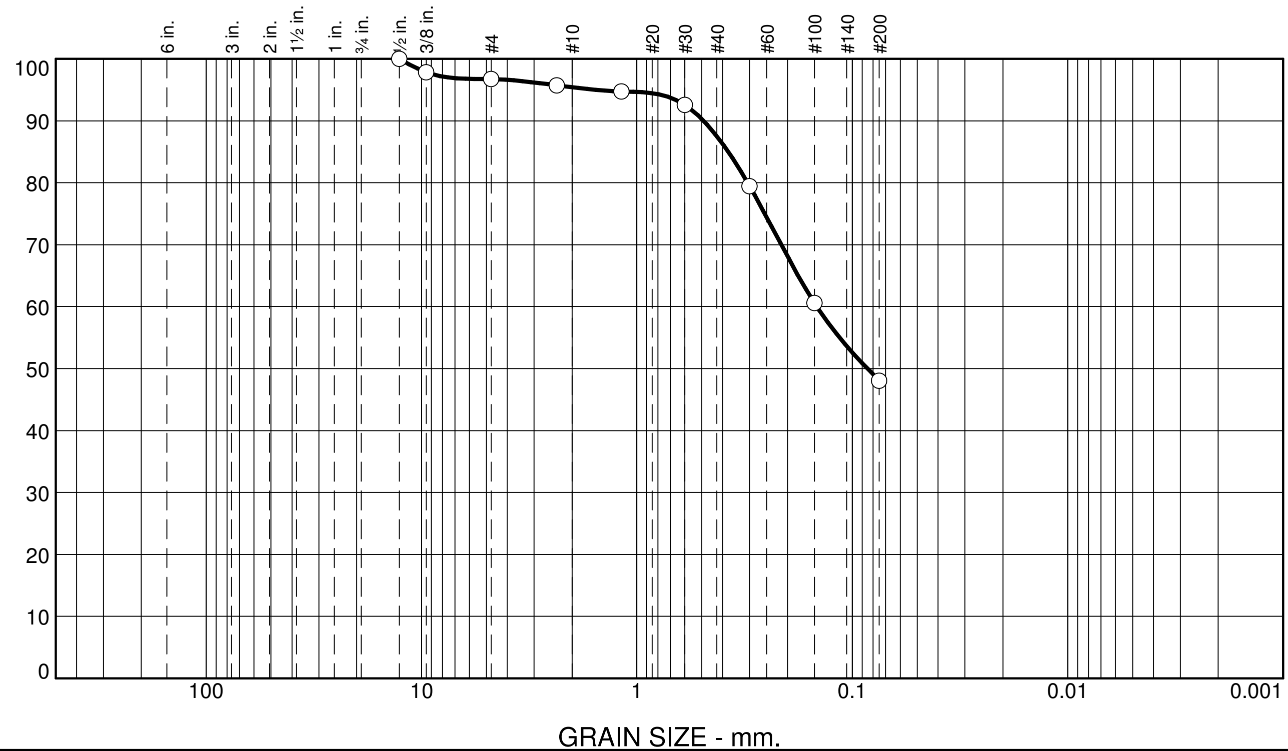
Date Received: **Date Tested:**
Tested By: JLA
Checked By: KC
Title:

Location: B2-2-3@9.5
Sample Number: 10S170615-3
Date Sampled: 6/7/17

CONSOLIDATED ENGINEERING LABORATORIES
San Ramon, California

Client:
Project: MCCD - Jonas Center Project (GES & GHR)
Project No: 9103940A

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	0	3	2	8	39	48	

TEST RESULTS			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1/2"	100		
3/8"	98		
#4	97		
#8	96		
#16	95		
#30	93		
#50	79		
#100	61		
#200	48		

* (no specification provided)

Material Description

Clayey Sand Sample #2
Sampled on 6/7/17 by A. Lim

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= AASHTO (M 145)=

Coefficients

D_{90}= 0.4910 D_{85}= 0.3772 D_{60}= 0.1461
D_{50}= 0.0851 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

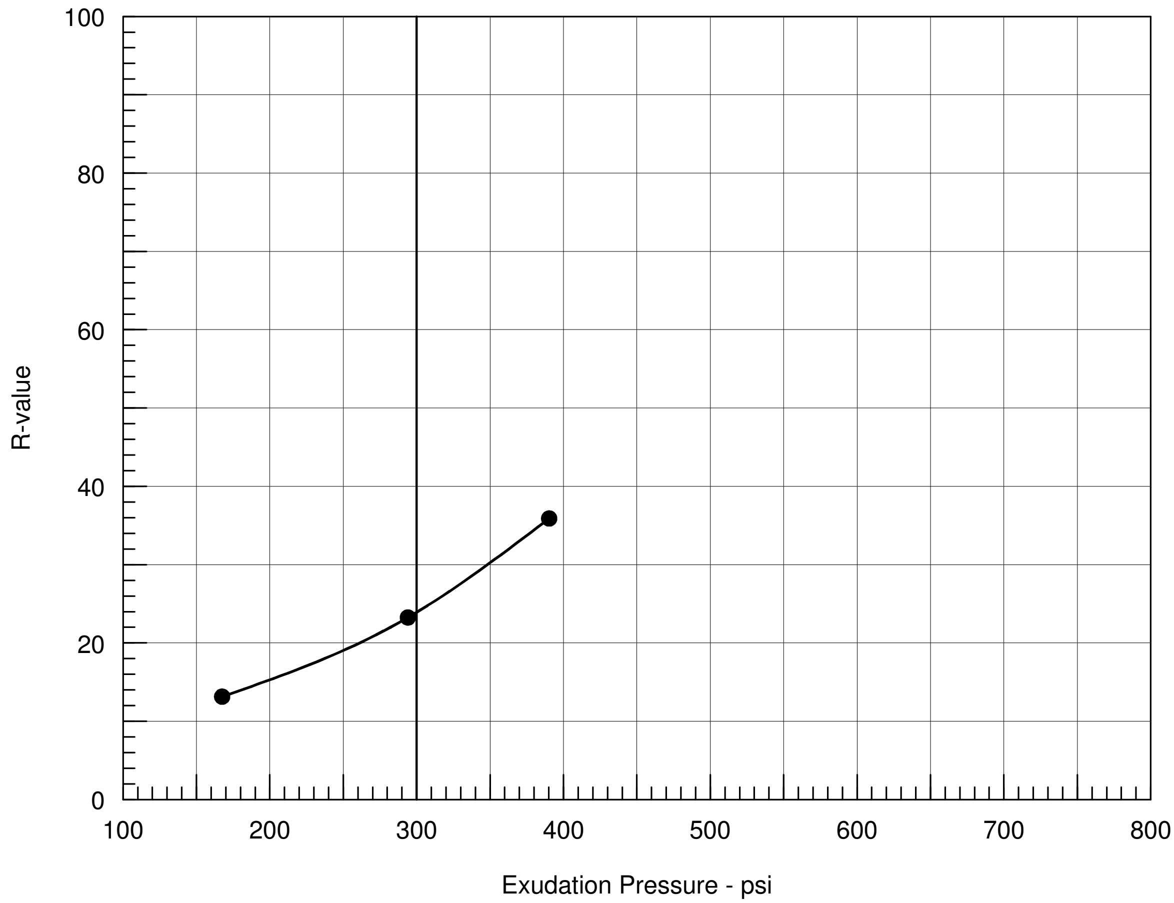
Date Received: **Date Tested:**
Tested By: JLA
Checked By: KC
Title:

Location: B4-4-2@3.5
Sample Number: 10S170615-3
Date Sampled: 6/7/17

CONSOLIDATED ENGINEERING LABORATORIES
San Ramon, California

Client:
Project: MCCD - Jonas Center Project (GES & GHR)
Project No: 9103940A

R-VALUE TEST REPORT



Resistance R-Value and Expansion Pressure - ASTM D 2844

No.	Compact. Pressure psi	Density pcf	Moist. %	Expansion Pressure psi	Horizontal Press. psi @ 160 psi	Sample Height in.	Exud. Pressure psi	R Value	R Value Corr.
1	100	151.7	-15.6	0.00	133	2.60	168	12	13
2	200	110.7	14.7	0.00	99	2.56	390	35	36
3	150	127.1	6.4	0.00	116	2.50	294	23	23

Test Results	Material Description
R-value at 300 psi exudation pressure = 24	Brown Sandy Clay Sampled on 5/24/17 by Others

Project No.: 9103940A

Project: Jonas Center Project (GES & GHR)

Location: N/A

Sample Number: 10S170602-3

Date: 6/5/2017

Tested by: JLA

Checked by: KC

Remarks:

R-VALUE TEST REPORT

CONSOLIDATED ENGINEERING LABORATORIES -- SAN RAMON, CA

10S170602-3

Corrosivity Tests Summary

CTL # 724-164 **Date:** 6/13/2017 **Tested By:** PJ **Checked:** PJ
Client: Geosphere Consultants **Project:** COM: Jonas Center **Proj. No:** 91-03940-A
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	
B2	-	1-4	-	-	11,944	3	44	0.0044	6.0	400	22	Negative	10.6	Brown Clayey SAND w/ Gravel

Job Number 730-A2 Client Marin Community College District
Location Novato
Prepared by TT Date 12/23/72
Checked by TT Date 12/26/72

SAMPLE DESCRIPTION	TEST SURCHARGE PSF	MOISTURE CONDITION	MOISTURE CONTENT %	DRY DENSITY PCF	% EXPANSION
Boring C1					
Depth 1½ ft.	100	Natural	11.5	101	0
Grayish brown		After Saturation	22.5	99	+2
Sandy clay		Air Dry	6.0	104	-3
(CL)		Oven Dry	0	107	-6
Boring C5					
Depth 1 ft	100	Natural	9.1	93	0
Grayish brown		After Saturation	24.9	93	0
Sandy clay		Air Dry	7.8	97	-5
(CL)		Oven Dry	0	98	-6
Boring C11					
Depth 1½ ft	100	Natural	9.1	95	0
Grayish brown		After Saturation	25.1	94	+1
Clayey sand		Air Dry	6.4	96	-1
(SC)		Oven Dry	0	98	-3
Boring C18					
Depth 1½ ft	100	Natural	7.1	94	0
Brown		After Saturation	24.2	96	-2
Sandy clay		Air Dry	4.8	98	-4
(CL-ML)		Oven Dry	0	101	-7

DESCRIPTION OF EXPANSION-CONTRACTION TEST PROCEDURE

An undisturbed sample of soil, at its natural moisture content, confined in the 1-inch-high, 2.375-inch-ID cylinder in which it was obtained in the field, is immersed in water while under a surcharge pressure. Measurements of expansion or contraction are taken until movement ceases. The surcharge is removed and the sample air dried, then oven dried. By measuring the dimensions of the sample under these various conditions, it is possible to determine the soil volume under the following conditions: 1) at field moisture content, 2) when completely saturated under the given surcharge, 3) when air dry, and 4) when oven dry. The dry density is computed from the dry weight of the specimen and its volume under the various moisture conditions. The percent expansion, relative to the natural field volume of the sample, is directly related to the various volumes and inversely related to the various dry densities of the sample.

EXPANSION-CONTRACTION TEST DATA

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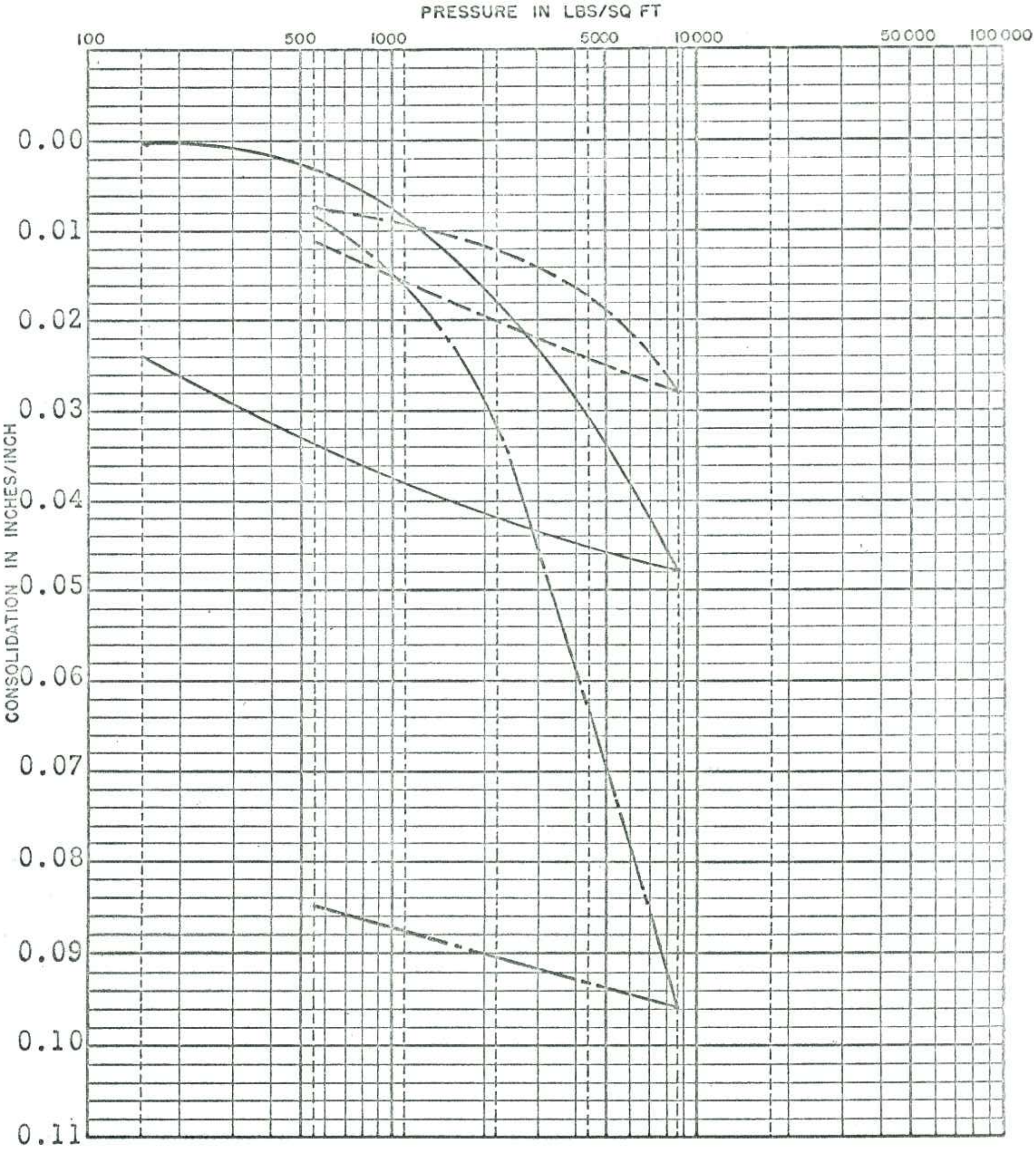
FOUNDATION ENGINEERS & ENGINEERING GEOLOGISTS

TL-18 (1-69)



KEY	BORING	DEPTH (FT)	SOIL DESCRIPTION	SOIL TYPE	NATURAL MOISTURE CONTENT %	NATURAL DRY DENSITY (P.C.F.)	SPECIFIC GRAVITY
———	C6	$1\frac{1}{2}$	Yellowish Brown Sandy Clay	CL / ML	8.6	114	
—·—	C15	$3\frac{1}{2}$	Brown Clayey Sand	SC	7.3	100	
— — —	C15	$10\frac{1}{2}$	Yellowish Brown Sandy Clay	CL	15.3	117	

CONSOLIDATION TEST DATA

COOPER · CLARK & ASSOCIATES
FOUNDATION ENGINEERS & ENGINEERING GEOLOGISTS

LOCATION: Novato

PLATE A-6D

Job Number 730-A2 Client Marin Community College District
Location Novato
Prepared by TT/pg Date 9/7/72
Checked by TT Date 9/8/72

SAMPLE NUMBER	SOIL CLASSIFICATION	TEST SPECIMEN	TEST MOISTURE CONTENT %	TEST DRY DENSITY PCF	EXPANSION PRESSURE PSF	EXUDATION PRESSURE PSI	TEST R-VALUE	R-VALUE (AT 300 PSI EXUDATION)
P9 Depth 1-2 Ft.	Brown Clayey Sand (SC)	A B C	14.9 15.8 16.7	115 112 108	43 22 0	337 287 188	18 13 10	14
P10 Depth 3-4 Ft.	Yellowish Brown Sandy Clay (CL)	A B C	21.8 22.8 23.9	107 103 99	87 65 43	342 291 202	11 8 6	9
P11 Depth $2\frac{1}{2}$-$3\frac{1}{2}$Ft.	Brown Sandy Clay (CL)	A B C	19.6 20.6 21.6	108 103 100	178 152 86	461 322 272	19 13 9	11
P12 Depth $1\frac{1}{2}$-$2\frac{1}{2}$Ft.	Brown Sandy Clay (CL-ML)	A B C	14.4 15.9 17.3	117 113 109	60 0 0	441 309 160	28 17 5	17
P14 Depth 8-9 Ft.	Yellowish Brown Highly Decomposed Sandstone	A B C	10.8 11.7 15.5	121 120 117	104 65 9	583 299 171	74 67 58	67
P15 Depth 2-3 Ft.	Brown Sandy Clay (CL-ML)	A B C	18.3 19.3 20.2	111 108 105	0 0 0	328 277 228	14 9 6	12

R-VALUE TEST DATA

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