Appendix D

Geotechnical Reports

REVISED GEOTECHNICAL REPORT

Student Housing Building – 2128 Oxford Street
2128 Oxford Street
Berkeley, California 94704

June 16, 2022
Partner Project Number: 20-297761.3

Prepared for:

Core Campus Manager, LLC
1643 N. Milwaukee Avenue, 5th Floor
Chicago, Illinois 60647

Engineers who understand your business

Mark Goehausen
Core Campus Manager, LLC
1643 N. Milwaukee Avenue, 5th Floor
Chicago, Illinois 60647

Subject: **Revised Geotechnical Report**
Student Housing Building
2128 Oxford Street
Berkely, California 94704
Partner Project No. 20-297761.3

Dear Mark Goehausen:

Partner Assessment Corporation (Partner) presents the following general opinion regarding the geotechnical conditions at the subject site, based on the information contained within this revised geotechnical report and our general experience with construction practices and geotechnical conditions on other sites. This statement does not constitute an engineering recommendation.

- *The geotechnical conditions on the site related to the planned construction are expected to be similar to more difficult in comparison with other similar sites*; given challenges associated with relatively shallow historic high groundwater, and possible deep excavations which will require shoring systems and possible dewatering.*

The descriptions and findings of our geotechnical report are presented for your use in this electronic format, for your use as shown in the hyperlinked outline below. To return to this page after clicking a hyperlink, hold "alt" and press the "left arrow key" on your keyboard.

1.0 Geotechnical Executive Summary
2.0 Report Overview and Limitations
3.0 Geologic Conditions and Hazards
4.0 Geotechnical Exploration and Laboratory Results
5.0 Geotechnical Recommendations

Figures & Appendices

We appreciate the opportunity to be of service during this phase of the work.

Sincerely,

PROFESSIONAL ENGINEER MATTHEW MARCUS GE 3156 EXP. 09/30/2023 GEOTECHNICAL STATE OF CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER ANDREW J. ATRY C 84728 CIVIL STATE OF CALIFORNIA

Matthew Marcus, PE, PG
Principal Geotechnical Engineer

Andrew J. Atry, PE
Project Engineer

Yuri Kawashima, GIT
Project Geologist

* "similar sites" refers to sites with similar planned and current use, where we have recently performed similar work, and is a general statement not based on statistical analysis.

1. GEOTECHNICAL EXECUTIVE SUMMARY

The executive summary is meant to consolidate information provided in more detail in the body of this report. This summary in no way replaces or overrides the detailed sections of the report.

Geologic Zones and Site Hazards

The site is located in the City of Berkeley within the Coast Ranges geomorphic province of California. Surficial geology at the site is mapped as older Quaternary alluvium and marine deposits. The site grades are relatively flat, gently sloping down towards the west. The site is currently occupied by a mixed-use residential and commercial property consisting of two abutting two-story buildings with associated asphalt parking lots in the rear. The site may be impacted by existing buried foundations, utility lines, undocumented fills as well as other remnants of previous construction. This portion of the state is prone to strong ground shaking and the site is mapped less than 1 mile from the Hayward Fault Zone. No other hazards were known or suspected on the site.

Excavation Conditions

We anticipate excavations on the site to depths of up to 14 feet for building foundations and/or slabs on grade, and 5 feet for utility lines. The currently planned basement will require support of excavation shoring to establish the anticipated finished floor level located approximately 10 feet below the ground surface. Such a system could consist of soldier piles with lagging and require heavy construction equipment as described in Section 5. As previously mentioned, undocumented fills and remnants of previous construction are present on the site and could cave or be difficult to remove and require additional planning and equipment. Groundwater was measured at 28 and 30 feet in borings B1 and B2, respectively, at the time of our investigation. However, groundwater levels fluctuate over time and may be different at the time of construction and during the project life. We estimate historic high groundwater levels of up to 10 feet below the existing ground surface.

Foundation/Slab Support

We anticipate that the new building and floor slabs will be supported on deep drilled foundations such as drilled shafts or auger-cast-in-place (ACIP) piles. Based on our geotechnical investigation, review of Harza's soil borings, and knowledge of local geologic conditions at the site and in the area, we anticipate deep foundation elements will need to extend at least 10 feet into the competent bedrock, which is likely to be encountered at approximately 70 feet below site grades. the contractor should be prepared for drilling below the groundwater table using the slurry method or other "wet construction" means, with temporary casings. If auxiliary structures, such as site walls require foundations, shallow spread foundations can be used as described in Section 5.2. The base of excavation for new shallow foundations and slabs on grade should be evaluated by the engineer, with additional removal of soft or deleterious material if needed and should then be compacted in-place prior to the placement of new fills or foundations. Areas for new slabs on grade should be evaluated by proofrolling with soft, unstable areas removed and replaced with compacted fill.

Soil Reuse

We presume that this will be primarily an export site from a grading perspective. However, based on our borings, site soils will generally be unsuitable for reuse as engineered fill/backfill in structural areas, given the presence clayey soils throughout the site. Therefore, the import of suitable structural fill material should be anticipated. Existing structural materials such as concrete, asphalt, crushed aggregate, or others could potentially be re-used as site fills if processed to meet fill requirements on the site. We recommend engineered granular fill for the site be moisture conditioned and compacted to 95% of the Proctor determined maximum dry density, in accordance with Appendix C of this report.

2. REPORT OVERVIEW & LIMITATIONS

2.1 Report Overview

To develop this report, Partner accessed existing information and obtained site specific data from our exploration program. Partner also used standard industry practices and our experience on previous projects to perform engineering analysis and provide recommendations for construction along with construction considerations to guide the methods of site development. The opinions on the cover letter of this report do not constitute engineering recommendations, and are only general, based on our recent anecdotal experiences and not statistical analysis. Section 1.0, Executive Geotechnical Summary, compiles data from each of the report sections, while each of sections in the report presents a detailed description of our work. The detailed descriptions in Section 5.0 and Appendix C constitute our engineering recommendations for the project, and they supersede the Executive Geotechnical Summary.

The report overview, including a description of the planned construction and a list of references, as well as an explanation of the report limitations is provided in Section 2.0. The findings of Partner's geologic review are included in Section 3.0 Geologic Conditions and Hazards. The descriptions of our methods of exploration and testing, as well as our findings are included in Section 4.0 Geotechnical Exploration and Laboratory Results. In addition, logs of our exploration excavations and laboratory test data along with the previous boring logs and test data by Harza Engineering Company are included in Appendix A of the report, results of our settlement anylsis is included in Appendix B of the report, and results of the geophysical Evaluation is included in Appendix D. Site Location and Site Plan maps are included as Figures in the report.

2.2 Assumed Construction

Partner's understanding of the planned construction was based on information provided by the project team. The proposed site plan is included as Figure 2 to this report. Partner's assumptions regarding the new construction are presented in the below table.

Property Data	
Property Use:	Student Housing Building
Building footprint/height	Approximately 35,000 sf, twenty-five-stories with one subteranian level
Land Acreage (Ac):	Approximately 0.82 acres
Expected Cuts and Fills	Deep excavations of 12 feet or more to establish foundation subgrade elevation for proposed basement
Type of Construction:	High-strength steel and concrete
Foundations Type	Assumed deep foundations
Anticipated Loads	Assumed 2,500 ksf
Traffic Loading	Primarily frequent vehicular traffic with occasional heavy truck traffic
Site Information Sources:	Google Earth Pro and Site Plan, HUB, Berkeley, California, prepared by Kimley Horn dated March 25, 2021

2.3 References

The following references were used to generate this report:

California Dept. of Transportation, ARS Online, accessed 04/28/2021

California Geological Survey, Note 36, *California Geomorphic Provinces*, 2002.

California State Water Resources Control Board, GeoTracker tool

Federal Emergency Management Agency, FEMA Flood Map Service Center, accessed 04/28/2021

Google Earth Pro (Online), accessed 04/28/2021

Rockridge Geotechnical, *Geotechnical Consultation Proposed Extended Stay Hotel, 2136 Center Street, Berkeley, California.* Report dated February 5, 2015.

Historic Aerials by NETR Online, accessed 04/28/2021

OSHPD Seismic Design Maps, accessed online 04/28/2021

Partner Engineering and Science, Inc., Phase 1 Environmental Assessment Report – *2128 Oxford Street, Berkeley*, *California*, Report dated 04/21/2021

Temblor Online, accessed 04/28/2021

United States Department of Agriculture, Web Soil Survey, accessed online 04/28/2021

United States Geological Survey, California Interactive Geologic Map accessed 04/28/2021

United States Geological Survey, Lower 48 States 2014 Seismic Hazard Map, accessed online 04/28/2021

United States Geologic Survey, Earthquake Hazards Program (Online), accessed 04/28/2021

2.4 Limitations

The conclusions, recommendations, and opinions in this report are based upon soil samples and data obtained in widely spaced locations that were accessible at the time of exploration and collected based on project information available at that time. Our findings are subject to field confirmation that the samples we obtained were representative of site conditions. If conditions on the site are different than what was encountered in our borings, the report recommendations should be reviewed by our office, and new recommendations should be provided based on the new information and possible additional exploration if needed. It should be noted that geotechnical subsurface evaluations are not capable of predicting all subsurface conditions, and that our evaluation was performed to industry standards at the time of the study, no other warranty or guarantee is made.

Likewise, our document review and geologic research study made a good-faith effort to review readily available documents that we could access and were aware of at the time, as listed in this letter. We are not able to guarantee that we have discovered, observed, and reviewed all relevant site documents and conditions. If new documents or studies are available following the completion of the report, the recommendations herein should be reviewed by our office, and new recommendations should be provided based on the new information and possible additional exploration if needed.

This report is intended for the use of the client in its entirety for the proposed project as described in the text. Information from this report is not to be used for other projects or for other sites. All of the report must be reviewed and applied to the project or else the report recommendations may no longer apply. If pertinent changes are made in the project plans or conditions are encountered during construction that appear to be different than indicated by this report, please contact this office for review. Significant variations may necessitate a re-evaluation of the recommendations presented in this report. The findings in this report are valid for one year from the date of the report. This report has been completed under specific Terms and Conditions relating to scope, relying parties, limitations of liability, indemnification, dispute resolution, and other factors relevant to any reliance on this report.

If parties other than Partner are engaged to provide construction geotechnical services, they must be notified that they will be required to assume complete responsibility for the geotechnical phase of the project by concurring with the findings and recommendations in this report or providing alternate recommendations.

3. GEOLOGIC CONDITIONS & HAZARDS

This section presents the results of a geologic review performed by Partner, for the proposed new construction on site. The general location of the project is shown on Figure 1.

3.1 Site Location and Project Information

The planned construction will be situated on an occupied parcel within a residential/commercial area of Berkeley, California. The subject property is currently occupied by mixed-commercial and residential buildings. The project site is bordered by Oxford Lane to the south followed by commercial buildings; Oxford Street to the east followed by the University of California, Berkeley (UC Berkeley) campus; Center Street to the north followed by Berkeley Art Museum; and a residential/commercial building to the west. Figure 2 presents the project site and the locations of our site exploration. Based on our review of available documents, the site has had the following previous uses:

Historical Use Information		
Period/Date	**Source**	**Description/Use**
1890 –1904	Sanborn Maps, Topographic Maps	School, commercial and residential uses
1911 –1993	Sanborn Maps, Aerial Photographs, Topographic Maps, City Directories	Mixed-commercial, and residential, and parking lot uses
1995 – 1996	Municipal Records	Construction of the current 2128 building
1996 – Present	Municipal Records, City Directories, Topographic Maps, Aerial Photographs, Sanborn Maps, Interviews	Mixed-commercial and residential, and parking lot uses

3.2 Geologic Setting

The site is located in the City of Berkeley within the Coast Ranges geomorphic province of California. Surficial geology at the site is mapped as older Quaternary alluvium and marine deposits. The site grades are relatively flat, gently sloping down towards the west. The site is currently occupied by a mixed-use residential and commercial property consisting of two abutting two-story buildings with associated asphalt parking lots in the rear. The site may be impacted by existing buried foundations, utility lines, undocumented fills as well as other remnants of previous construction. This portion of the state is prone to strong ground shaking. The site is partially mapped within a liquefaction hazard zone per the CGS Earthquake Zones of Required Investigation Map, and the site is mapped less than 1 mile from the Hayward Fault Zone. No other hazards were known or suspected on the site.

Based on information obtained from the USDA Natural Resources Conservation Service Web Soil Survey online database, the subject property is mapped as 146- Urban land. Urban land complex soils are those soils in which the soil's original structure and content have been so altered by human activities it has lost its original characteristics and is therefore unidentifiable. Urban soils consist of nearly level to moderately steep areas where the soils have been altered or obscured by urban development and structures. Included in the mapping unit are many small areas where the original soil material has been disturbed by construction and areas where fill materials have been added. As such the soil properties and characteristics vary. A general summary of the geologic data compiled for this project is provided in the below table.

Geologic Data		
Parameter	**Value**	**Source**
Geomorphic Zone	Coast Ranges	CGS
Ground Elevation	Approx. 201 feet above MSL	USGS
Flood Elevation	Flood Hazard Zone X	FEMA
Seismic Hazard Zone	High	USGS
Geologic Hazards	Ground shaking, liquefaction	CGS
Surface Cover	Asphalt cover	Onsite Observations
Surficial Geology	Older Quaternary Alluvium and Marine Deposits	USGS
Depth to Bedrock	Unknown	-
Groundwater Depth	15 feet	Partner Phase 1
Historical Groundwater Depth	Approximately 10 feet bgs	CGS GeoTracker

3.3 Geologic Hazards

California is tectonically active and contains numerous large, active faults. As a result, geologic hazards with the greatest potential to affect California include earthquakes and related hazards such as tsunamis, landslides, liquefaction, and ground shaking. According to the California Geological Survey (CGS) Fault Activity Map tool, the three faults most relevant to the site are the Hayward Fault Zone (0.73 miles from the site), Mount Diablo Thrust Fault (12.40 miles from the site), and the Green Valley Connected Fault (14.42 miles from the site). The site is not mapped within a zone of seismically included hazard for landslide or tsunami. The site is partially mapped within a liquefaction hazard zone per the CGS Earthquake Zones of Required Investigation Map.

3.4 Seismic Design Parameters

The site latitude and longitude are 37.870252 degrees N and -122.266585 degrees W respectively.

Based on the recent edition of the American Society of Civil engineers (ASCE), document 7-16, a site-specific ground motion hazard analysis (GMHA) is required for sites with:

- Structures on Site Class E with Ss greater than or equal to 1.0
- Structures on Site Class D and E sites with S_1 greater than or equal to 0.2.

Because the site does not meet either of the criteria, a GMHA is not required. Based on the Refraction Micrometer (ReMi) survey performed by Atlas on November 3, 2021, the site has a time-averaged shear-wave velocity to 30 m depth (Vs30) is determined to be 1,242 feet per second. Therefore, the site can be classified as Site Class C, the results of the Geophysical study performed by Atlas wil be presented in Appendix D of our report. Using information obtained from the SEAOC (Structural Engineers Association of California) /OSHPD (Office of Statewide Health Planning and Development) Seismic Design Maps for ASCE 7-16, for a Site Class of C and risk category of IV, the following values were obtained as shown on the below table.

The seismic design parameters based on the USGS Design Maps Detailed Report for ASCE 7-16 Standard Method are presented below. State, County, City, and other jurisdictions in seismically active areas update seismic standards on a regular basis. The design team should carefully evaluate all of the building requirements for the project.

Seismic Item	Value	Seismic Item	Value
Site Classification	C	Seismic Design Category	E
F_a	1.2	F_v	1.4
S_s	2.191g	S_1	0.845g
S_{MS}	2.629g	S_{M1}	1.184g
S_{DS}	1.753g	S_{D1}	1.753g
PGA_M	1.105g	Design PGA (2/3 PGA_M)	0.737g

4. GEOTECHNICAL EXPLORATION & LABORATORY RESULTS

Our preliminary evaluation of soils on the site included review of Harza's field exploration and laboratory testing. The field exploration and laboratory testing programs are briefly described below. Harza's boring logs and laboratory testing results are provided in Appendix A.

4.1 Soil Borings

Partner mobilized to the site a total of two times to complete the exploration program. On March 31, 2021, one (1) percolation test was advanced to 5 feet. We re-mobilized to the site on December 3, 2021 and advanced two (2) borings to depths of up to 80.5 feet. Logs of subsurface conditions encountered in the borings are presented in Appendix A. A summary table description from our investigation is provided below.

We also analyzed data from soil borings conducted by Harza on October 16, 2000. Three (3) borings designated EB-1, EB-2 and EB-3, were advanced by the use of a truck-mounted Mobile B-61 drill rig using hollow-stem auger drilling techniques. The borings were advanced to depths of 30 feet. The approximate locations of the exploratory borings are shown on Figure 2.

Surficial Geology		
Strata	**Depth to Bottom of Layer (bgs*)**	**Description**
Surface Cover	Approximately 5.5 - 6 inches	Asphalt Pavement / Crushed Rock Base Course
Native Stratum 1	75 feet	Interbedded Sandy CLAY and clayey SAND soils
Bedrock	Unknown	Serpentinite and graywacke sandstone
Groundwater	Approx. 28-30 feet	Partner Borings

4.2 Groundwater

Groundwater was encountered on the site in Partner borings at approximately 28 to 30 feet at the time of drilling. Harza's Borings EB-2 and EB-3 showed groundwater at depths of approximately 18 and 17 feet, respectively. Groundwater levels fluctuate over time and may be different at the time of construction and during the project life.

4.3 Laboratory Evaluation

Selected samples collected during drilling activities were tested in the laboratory to assist in evaluating engineering properties of subsurface materials at the site. The results of laboratory analyses conducted as part of our geotechnical investigation and a the limited laboratory testing results conducted by Harza are presented in Appendix A.

4.4 Infiltration Test Results

One infiltration test designated P-1 was performed at the location shown on Figure 2. The test was performed at a depth of 5 feet. The test was performed using the borehole percolation test method. The measured infiltration rate reported below is the unfactored rate. The rate was calculated using the Porchet method. The civil engineer should apply the proper reduction factors or factors of safety based on the type of system used. Data is summarized below:

Parameter	P-1
Location	See Figure 2
Elevation of Tested Area	195 ft
Pre-soak Depth (from top of pipe)	1.0 ft
Test Start Depth (from top of pipe)	41 in.
Average Water Drop During Final Three Readings	14.6 in.
Unfactored Infiltration Rate	**4.4 in./hr**

Given the presence of a planned basement on the site and predominantly clay soils encountered, on-site infiltration is not recommended.

5. PRELIMINARAY GEOTECHNICAL RECOMMENDATIONS

The following discussion of findings for the site is based on the assumed construction, geologic review, results of the field exploration, and laboratory testing programs. The recommendations of this report are contingent upon adherence to Appendix C of this report, General Geotechnical Design and Construction Considerations. For additional details on the below recommendations, please see Appendix C.

5.1 Preliminary Geotechnical Recommendations

The proposed construction is generally feasible from a geotechnical perspective provided the recommendations and assumptions of this report are followed.

Geologic/General Site Considerations

- The site is located in the City of Berkeley within the Coast Ranges geomorphic province of California. Surficial geology at the site is mapped as older Quaternary alluvium and marine deposits. The site grades are relatively flat, gently sloping down towards the west. The site is currently occupied by a mixed-use residential and commercial property consisting of two abutting two-story buildings with associated asphalt parking lots in the rear. The site may be impacted by existing buried foundations, utility lines, undocumented fills as well as other remnants of previous construction. This portion of the state is prone to strong ground shaking and the site is mapped less than 1 mile from the Hayward Fault Zone. No other hazards were known or suspected on the site.
- Given the presence of the site in a seismically active area, ground shaking during earthquakes should be anticipate during the project life. State, County, City, and other jurisdictions in seismically active areas update seismic standards on a regular basis. The design team should carefully evaluate all of the building requirements for the project. With the basement excavation, planning around seasonal weather should be considered, as rain fall can complicated open excavation construction.

Excavation Considerations

- We anticipate excavations on the site to depths of up to 12 feet for building slabs on grade, and 5 feet for utility lines. The currently planned basement will require support of excavation shoring to establish the anticipated finished floor level located approximately 10 feet below the ground surface. As previously mentioned, undocumented fills and remnants of previous construction are present on the site and could cave or be difficult to remove and require additional planning and equipment.
- Given the depth of the anticipated planned excavation and the presence of nearby structures, a specially designed shored excavation will be needed to establish foundation subgrade levels. Such a system could consist of a drilled soldier pile wall with lagging and soil anchors, but other systems may also be acceptable. The design of this system should be performed by the contractor performing the work, and should consider the impacts of installing anchors, deflection of the soil behind the walls, and dealing with groundwater and surface water that may enter the excavation during inclement weather. All of these factors could result in damage to surrounding properties. The design can use soil data from section 5.2 of this report. The groundwater levels used in the design can be adjusted based on the monitoring data obtained as well as engineering judgement,

however, we do not anticipate stabilized groundwater levels shallower than 10 feet on the site. Appendix C of this report contains a section regarding additional Excavation and Dewatering considerations for the site. Nearby properties should be protected during demolition and excavation, and pre- and post-construction surveys of the nearby properties are recommended, as is site monitoring during construction.

- Groundwater was measured at 28 and 30 feet in borings B1 and B2, respectively, at the time of our investigation. However, historic high groundwater levels may be as high as 10 feet. The contractor should be prepared to manage groundwater during excavation, which will require special planning and equipment, that could include the use of sumps, pumps, trench drains, or other measures. One option would be the installation of a number of groundwater monitoring wells which the contractor could monitor prior to construction to evaluate if groundwater is likely to be encountered during temporary excavations. This should be done at multiple locations on the site, and based on the contractor's judgement, temporary excavations could be designed based on real time monitoring data. It should be noted that groundwater levels can change quickly.
- At the time of our study, multiple existing structures were located on the site and are slated to be demolished. Given the proximity to neighboring properties and roadways, sloping, shoring, and/or supported excavations will be called for on the site during building removal. All site excavation should proceed per OSHA and local guidelines. The presence of existing utilities should be thoroughly and carefully checked prior to digging. As previously mentioned the basement excavation should be planned during seasonally dry periods. If rainfall occurs, the excavation should be properly protected by drainage berms, tarps, mud mats, or other methods determined by the contractor. Exposed clay soils at the base of the excavation would tend to retain water, and require removal or extensive drying after pumping of water.
- Appendix C further discusses excavation recommendations in the following sections, which can be accessed by clicking hyperlinks: Earthwork, Underground Pipeline, Excavation De-Watering.

Deep Foundation Considerations

- The new building foundations and floor slabs should be supported on deep drilled foundations such as drilled shafts or auger-cast-in-place (ACIP) piles. Based on our geotechnical investigation, review of Harza's soil borings, and knowledge of local geologic conditions at the site and in the area, we anticipate deep foundation elements will need to extend at least 10 feet into the competent bedrock, which is likely to be encountered at approximately 70 feet below site grades.
- Since the excavations will likely be below the groundwater table, the contractor should be prepared to drill the shafts with the slurry method of construction and with drill casings, per FHWA guidelines and the current California Building Code (CBC). This would also require tremie piping of concrete to displace the slurry and groundwater. A contractor who is familiar with the installation of drilled shafts in this area should be consulted for the selection of the appropriate slurry system and casing types and depths.
- The installation of drilled shafts or ACIP piles should be continuously monitored in the field by a representative of the geotechnical engineer to verify that the foundations are installed into the proper bearing stratum (moderately strong claystone), that they are the specified diameter and

depth, that the drilling is plum, that the proper reinforcement is placed and that the correct concrete mix and placement techniques are used. In addition, the shafts placed in the first 3 days should contain access tubes for cross-hole sonic logging and gamma-gamma quality testing. Provided that no failures are detected in the shafts placed in the first 3 days, the access tubes can be reduced to 25% of the shafts placed. This should be done per per FHWA guidelines and the current California Building Code (CBC).

- Given the needed capacities, each column would need to be supported on a pile group. Pile group effects will need to be considered as discussed in the following sections. For the basement and at-grade floors, a slab supported on 24 inches of imported granular fill (as described above) or on grade beams would be required.

Shallow Foundations

- If auxiliary structures, such as site walls require foundations, shallow spread foundations can be used as described in Section 5.2. The base of excavation for new foundations should be evaluated by the engineer, with additional removal of soft or deleterious material if needed and should then be compacted in-place prior to the placement of new fills or foundations. Areas for new slabs on grade should be evaluated by proofrolling with soft, unstable areas removed and replaced with compacted fill. Slabs and auxiliary foundations should be supported on 12 inches of reworked granular soil (PI < 15 or fines % < 35), which may call for removal and replacement of existing site soil.
- Section 5.2 of this report provides a table outlining the embedment depth, bearing capacity, settlement and other parameters for foundation design and construction.

On-Grade Construction Considerations

- In new structural areas of the site, all remnants of previous construction, vegetation and/or deleterious materials should be completely removed to exposed clean subgrade soil. In new fill, structural, and pavement areas, cleaned subgrade should be proofrolled and evaluated by the engineer with a loaded water truck (4,000 gallon) or equivalent rubber-tired equipment. In locations where proofrolling is not feasible, probing, dynamic cone penetration testing or other methods may be employed. Soft or unstable areas should be repaired per the direction of the engineer. Once approved, the subgrade soil should be scarified to a depth of 12 inches, moisture conditioned, and compacted as engineered fill. Improvements in these areas should extend laterally beyond the new structure limits 2 feet or a distance equal to or greater than the layer thickness, whichever is greater. This zone should extend vertically from the bearing grade elevation to the base of the fill. The thicknesses of the layer, settlement estimates, and modulus values are provided on the design tables in the next section.
- Based on the Harza borings, we anticipate that some over-excavation may result from proofrolling operations. In areas where deep instability is encountered, we recommend test pits be excavated and an engineer be called to perform an evaluation of the issue and to propose a resolution. Such resolutions may include but are not limited to the use of geotextiles, chemical treatments (soil cement, hydrated lime, etc.) thickened slabs or pavements sections, lime-treated aggregate base,

or others. Pavement sections provided in Section 5.2 are based on approved, compacted in-place soils being used in the subgrade. If subgrade conditions in the upper 3 feet of pavement areas vary or are improved, the pavement sections may be modified.

- Appendix C provides additional recommendations for foundations in the following sections: Cast-in-place Concrete, Foundations, Earthwork, Paving, Subgrade Preparation which can be accessed by clicking the hyperlinks.

Soil Reuse Considerations

- We presume that this will be primarily an export site from a grading perspective. However, based on our borings, site soils will generally be unsuitable for reuse as engineered fill/backfill in structural areas, given the presence clayey soils throughout the site. Therefore, the import of suitable structural fill material should be anticipated. Existing structural materials such as concrete, asphalt, crushed aggregate, or others could potentially be re-used as site fills if processed to meet fill requirements on the site. We recommend engineered granular fill for the site be moisture conditioned and compacted to 95% of the Proctor determined maximum dry density, in accordance with Appendix C of this report.
- Appendix C provides additional recommendations for foundations in the following sections: EARTHWORK, SUBGRADE PREPARATION which can be accessed by clicking the hyperlinks.

Geotechnical Concrete and Steel Construction Considerations

- Soil/rock may be corrosive to concrete. We recommend using corrosion resistant concrete (*e.g.* Type II/V Portland Cement, a fly ash mixture of 25 percent cement replacement, and a water/cement ratio of 0.45 or less) as directed by the producer, engineer or other qualified party based on their knowledge of the materials and site conditions. Concrete exposed to freezing weather should be air-entrained. Mix designs should be well-established and reviewed by the project engineers prior to placement, to verify the design is appropriate to meet the project needs and parameters provided in this report. Quality control testing should be performed to verify appropriate mixes are used and are properly handled and placed. Please refer to Appendix C, Cast In-Place Concrete for more details.
- Soil/rock may be corrosive to un-protected metallic elements such as pipes, poles, rebar, etc. We recommend the use of coatings and/or cathodic protection for metals in contact with the ground, as directed by the product manufacturer, engineer or other qualified party based on their knowledge of the materials to be used and site soil conditions.

Site Storm Water Considerations

- Testing indicated near surface soils are conducive to storm water infiltration. However, due to the presence of the clayey material below the lowest finished floor elevation on site storm water infiltration is not advised. Additonal testing should be conducted if infiltration is desired below the lowest finished floor elevation. Surface drainage and landscaping design should be carefully planned to protect the new structures from erosion/undermining, and to maintain the site earthwork and structure subgrades in a relatively consistent moisture condition. Water should not flow towards or pond near to new structures, and high water-demand plants should not be planned near to structures. Appendix C provides additional recommendations for foundations in the following sections: SITE GRADING AND DRAINAGE, WATER PROOFING which can be accessed by clicking the hyperlinks.

- We recommend consulting with the landscape designer and civil engineer regarding management of site storm water and irrigation water, as changes in moisture content below the site after construction will lead to soil movement and potential distress to the building.

5.2 Preliminary Geotechnical Parameters

Based on the findings of our field and laboratory testing, we recommend that design and construction proceed per industry accepted practices and procedures, as described in Appendix C, General Geotechnical Design and Construction Considerations (Considerations).

Preliminary Prepared Subgrade Parameters – (hyperlink to Construction Considerations)

Prepared Subgrade Parameters				
Structure	**Design Values**	**Cover Depth**	**Bearing Surface** [a]	**Static Settlement** [d]
Auxiliary Spread Foundations	$q_{all} =$ 1.5 ksf[c] μ = 0.35	18 inches	Compacted granular structural fill (<35% fines and PI <10) that extends to native soils (approximately 5 feet below existing grades)	<1 inch

[a] Repairs in bearing *surface areas should be structural fill per the recommendation of the* Earthwork *section of Appendix C that is moisture conditioned to within 3 percent below to optimum moisture content and compacted to 95 percent or more of the soil maximum dry density per ASTM D1557. Expansive material should not be located within the upper* 3 feet of the soil subgrade.

[b] *Subgrade modulus value "k", assuming the grade slab is supported by aggregate layer roughly equal to slab thickness (minimum 4 inches), as required for capillary break*

[c] *Can be increased by 1/3 for temporary loading such as seismic and wind, allowable parameters, estimated FS of 2.5*

[d] *Differential settlement is expected to be half to ¾ of total settlement*

Laterally Loaded Structures Preliminary Parameters– (hyperlink to Construction Considerations)

Lateral Earth Pressures[a]				
Soil Type	**Coefficient of Friction (μ)**	**Static Fluid Pressure (pcf)**	**Active Fluid Pressure (pcf)**	**Passive Fluid Pressure (pcf)**
Clayey Soils (above Groundwater Table)	0.40	60	40	300
Clayey Soils (below Groundwater Table)	0.45	60+62.4[b]	40+62.4[b]	375
Bedrock	0.45	50	35	425

[a] *These values are unfactored, "raw" numbers and appropriate safety factors should be applied by the wall designer. Assumed GW table at rock surface, for underground structures where water is only on one side, the hydrostatic pressure of 62.4 psf should be added*

[b] *This applies to cases where free standing water is located on only one side of the wall.*

Drilled Pile Parameters – (hyperlink to Construction Considerations)

Axial capacities of pile foundations were estimated based on soil properties observed in the borings. The tabular results below are non-factored values. The capacities should be calculated by the structural engineer and they should select the appropriate safety factors for their design.

From (ft)	To (ft)	Unit Weight (pcf)	Cohesion (psf)	Friction Angle (deg)	Ultimate Uplift Skin Friction (ksf)	Ultimate Comp. Skin Friction (ksf)	Ultimate End Bearing Capacity (ksf)	Passive Equivalent Fluid Pressure (pcf)
1	6	100	1000	32	0.2	0.4	13.5	375
6	18	110	1000	32	0.5	1.0	13.5	375
18	24	105	1000	32	1.5	3.5	13.5	375
24	26	100	1000	32	6.0	12.0	13.5	375
26	38	110	1000	32	1.5	3.0	13.5	375
38	65	105	1000	32	1.0	2.5	34.0	375
65	100	120	1400	34	6.5	13.0	42.0	425

L-Pile parameters – in accordance with Appendix C:

Lateral structural loads can be resisted by the structural strength on the pile in bending and the passive resistance of the soil adjacent to the pile cap. Conditions of lateral loading can be evaluated using the computer software, LPILE, developed by Ensoft, Inc. of Austin, Texas. For the lateral load analysis, we have assumed the water table occurs at depths greater than 21.5 feet below the ground surface. Recommended input parameters for the various soil units for the LPILE analysis are tabulated below and are appropriate for both static and seismic conditions.

Soil Properties for LPILE Analysis							
Soil Unit	Depth Below Existing Grades (ft)	L-Pile Soil Type	Cohesion (psf)	E50	K (pci)	γ', (pcf)	ϕ'
Medium Stiff Clay	0 to 6	Stiff Clay without Free Water	1000	0.01	100	100	32
Stiff Clay	6 to 18	Stiff Clay without Free Water	1000	0.007	500	110	32
Hard Clay	18 to24	Stiff Clay with Free Water	1000	0.004	2000	105	32
Medium Stiff Clay	24 to 26	Stiff Clay with Free Water	1000	0.01	100	100	32
Stiff Clay	26 to 38	Stiff Clay with Free Water	1000	0.007	500	110	32

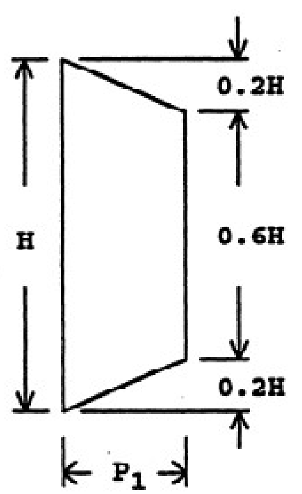
Hard Clay	38 to 65	Stiff Clay with Free Water	1000	0.004	2000	105	32
Bedrock	65 to 85	Weak Rock (Reese)	1400	0.001	2000	120	34

It should be noted that LPILE provides isolated, single pile lateral capacities. Depending on the direction of the loading and the layout of the piles, group effects may need to be considered. Group effects can be modeled in LPILE by applying an appropriate p-modifier in non-liquefiable soils. The p-modifier is a function of the center-to-center spacing and is tabulated below.

P-Modifiers for Group Effects	
Center-to-Center Pile Spacing	**P-modifiers for Rows 1, 2, and 3+**
3D	0.7, 0.6, 0.5
4D	0.8, 0.6, 0.5
5D	0.9, 0.85, 0.7
6D	1.0, 1.0, 0.9
7D	1.0, 1.0, 1.0

Lateral Loading Considerations

The below diagram (from Caltrans Trenching and Shoring Manual) depicts the stress distributions around a shored excavation where struts are used. Depending on the types of walls and soil types encountered, different distributions may be needed. The conditions of this diagram should be carefully considered prior to use, and values given are unfactored. We recommend that a specialty contractor with in-house engineering capability perform the design of temporary shoring.



$P_1 = P_A = 0.8KwH$

For shoring or permanent retaining walls surcharges from traffic and adjacent buildings should be considered as shown in the below equations. The distribution of soil pressures on retaining structures will depend on the type of systems used, and whether they are braced or anchored. The shoring and retaining wall designer should be familiar with the appropriate distribution diagrams to be used and use care in the selection of the appropriate model. The walls should be designed to dissipate nuisance water to the sump system, through an interconnected series of drains. In general, this will not result in lowering of the groundwater table.

Building Foundation Surcharge Loading Equation

Resultant Lateral force:

$$\boldsymbol{R = \frac{0.3P \quad ^2}{x^2 + h^2}}$$

Location lateral resultant:

$$\boldsymbol{d = x\left[\left(\frac{x^2}{h^2} + 1\right)\left(ta \quad ^{-1}\frac{h}{x}\right) - \left(\frac{x}{h}\right)\right]}$$

Where:

R	= Resultant lateral force measured in pounds per foot of wall width.
P	= Resultant surcharge loads of continuous or isolated footings measured in pounds per foot of length to the wall.
x	= Distance of resultant load from back face of wall measured in feet.
h	= depth below point of application of surcharge loading to top of wall footing measured ion feet.
d	= Depth of lateral resultant below point of application of surcharge loading measured in feet.
$\tan^{-1}\frac{h}{x}$	= The angle in radians whose tangent is equal to $\frac{h}{x}$.

Loads applied within a horizontal distance equal to the wall stem height, measured from the back face of the wall, shall be considered as surcharge.

For isolated footings having a width parallel to the wall less than 3 feet, “R” may be reduced to one-sixth the calculated value.

Vertical pressure due to surcharge applied to the top of the wall footing may be considered to spread uniformly within the limits of the stem and planes making an angle of 45 degrees with the vertical

Traffic Surcharge Loading Equation

$$q = k \times \gamma_s \times H_{eq}$$

Where:

- q = Lateral surcharge pressure measured in pounds per square foot in a rectangular distribution.
- k = Active or at-rest earth pressure coefficient as presented in section 5.2 of this report.
- γ_s = Total unit weight of soil measured in pounds per cubic foot
- H_{eq} = Equivalent height of soil from the below table.

Equivalent Height of Soil for Vehicular Loading on Retaining Wall and Shoring Parallel to Traffic*		
Excavation/Wall Height (ft)	**Distance from the edge of Excavation (ft)**	
	0.0 ft	**≥1.0 ft**
5.0	5.0	2.0
10.0	3.5	2.0
≥20.0	2.0	2.0

**From Table 3.11.6.4-2 of the AASHTO LRFD Bridge Design Specifications*

Seismic Surcharge Equations

Combined effect of static and seismic lateral forces:

$$P_{AE} = F_1 + F_2$$

$$F_1 = \frac{1}{2} \times A \times H^2$$ *Resultant acting at a distance of $\frac{H}{3}$ from base of the wall*

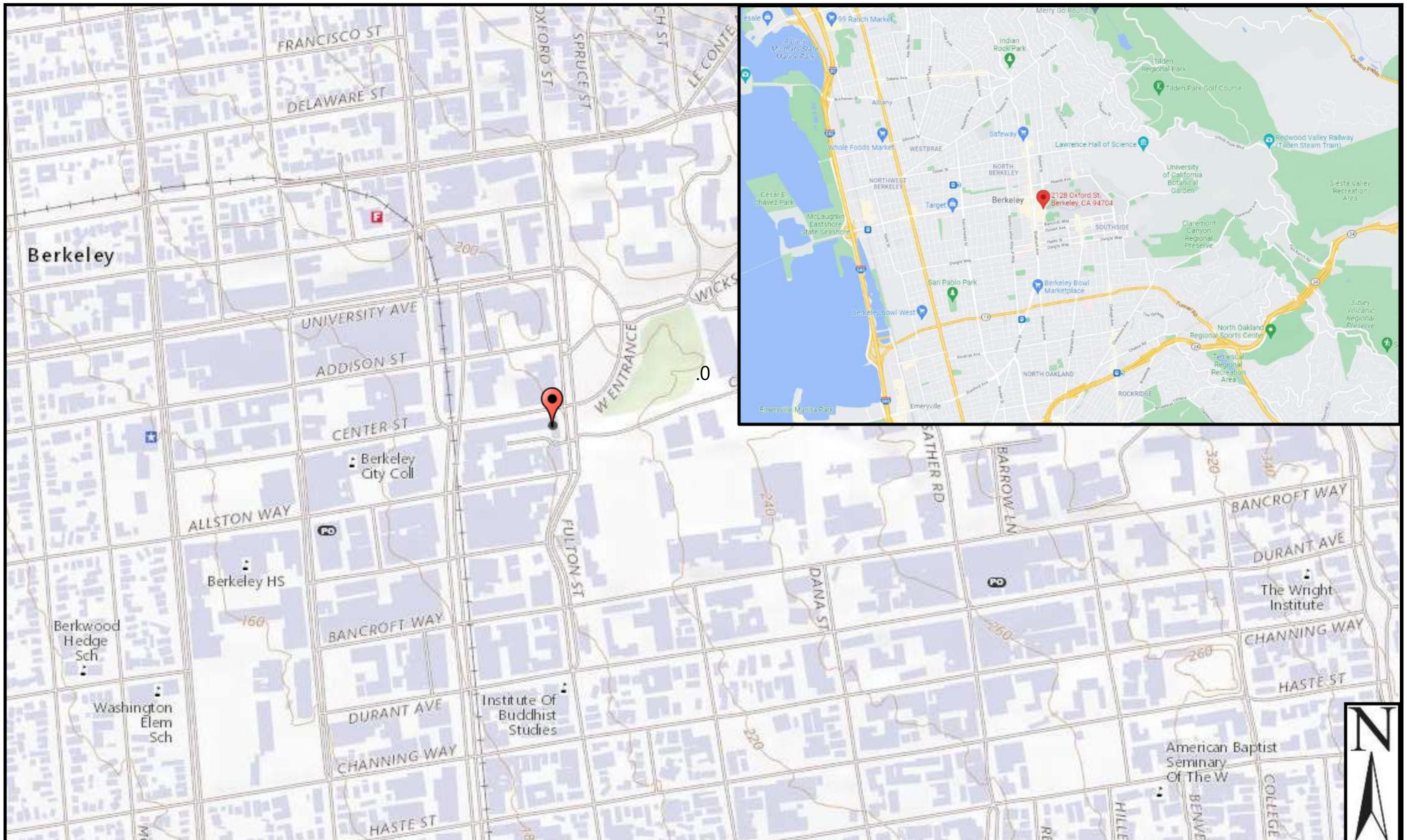
$$F_2 = \frac{3}{8} \times K_h \times \gamma \times H^2$$ *Resultant acting at a distance of $(0.6 \times H)$ from base of the wall*

Where:

- F_1 = Static force, measured in pounds per linear foot, based on active pressure.
- F_2 = Seismic Lateral Force, measured in pounds per linear foot, based on seismic pressure
- γ = 120 pounds per square foot
- K_h = $S_{DS}/2.5$
- A = Active Pressure, measured in pounds per cubic foot.
- H = Height of retained soil, measured in feet.

FIGURES

- Site Vicinity Plan
- Site Exploration Map
- Scaled Boring Location Plan
- Geologic Map
- Geologic Hazard Map

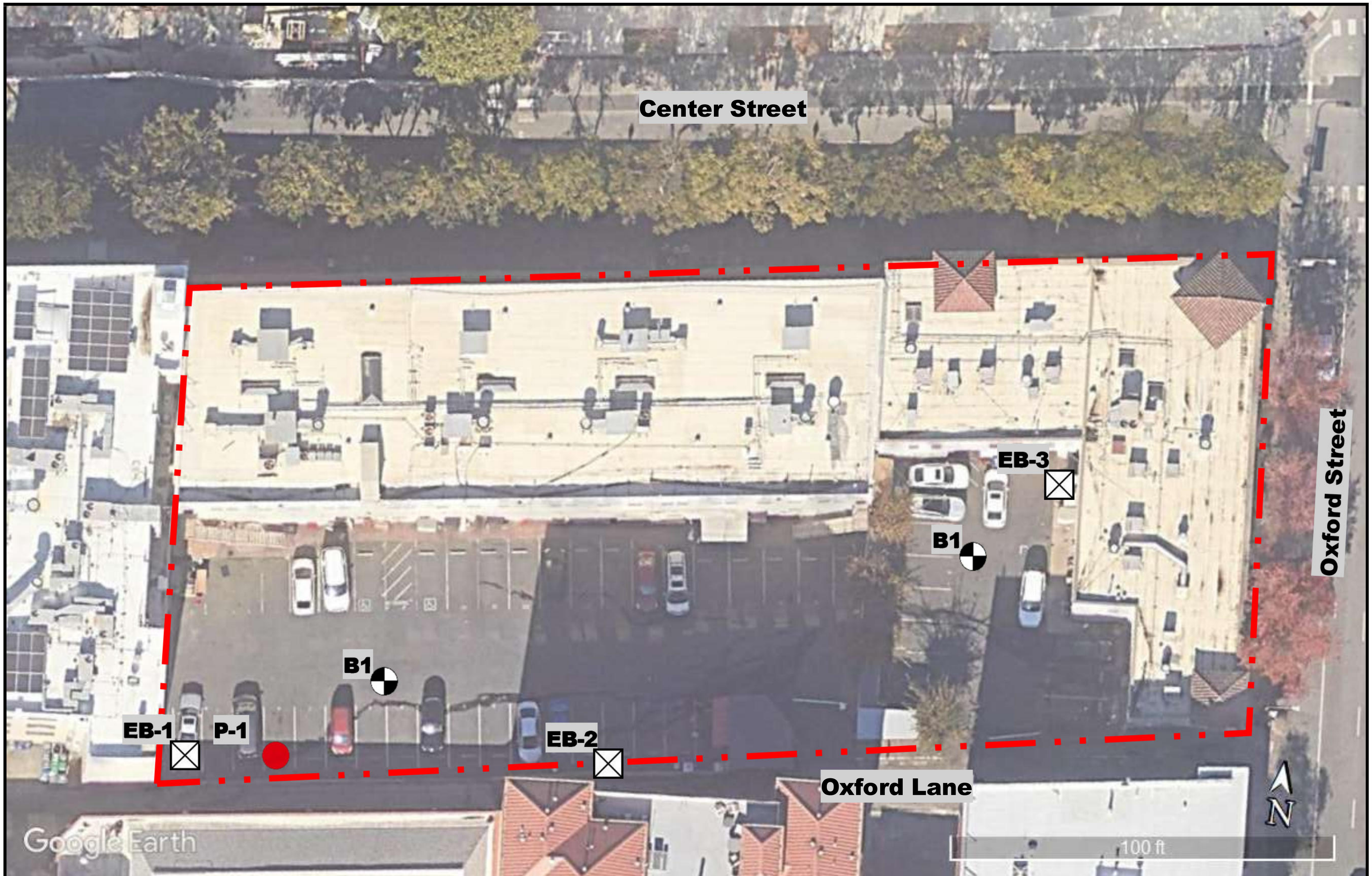


Source: U.S. Geological Survey, USGS US Topo 7.5-minute map for Berkeley, CA 2018: USGS - National Geospatial Technical Operations Center (NGTOC)

FIGURE 1 – SITE VICINITY PLAN

KEY

Approximate Site Location



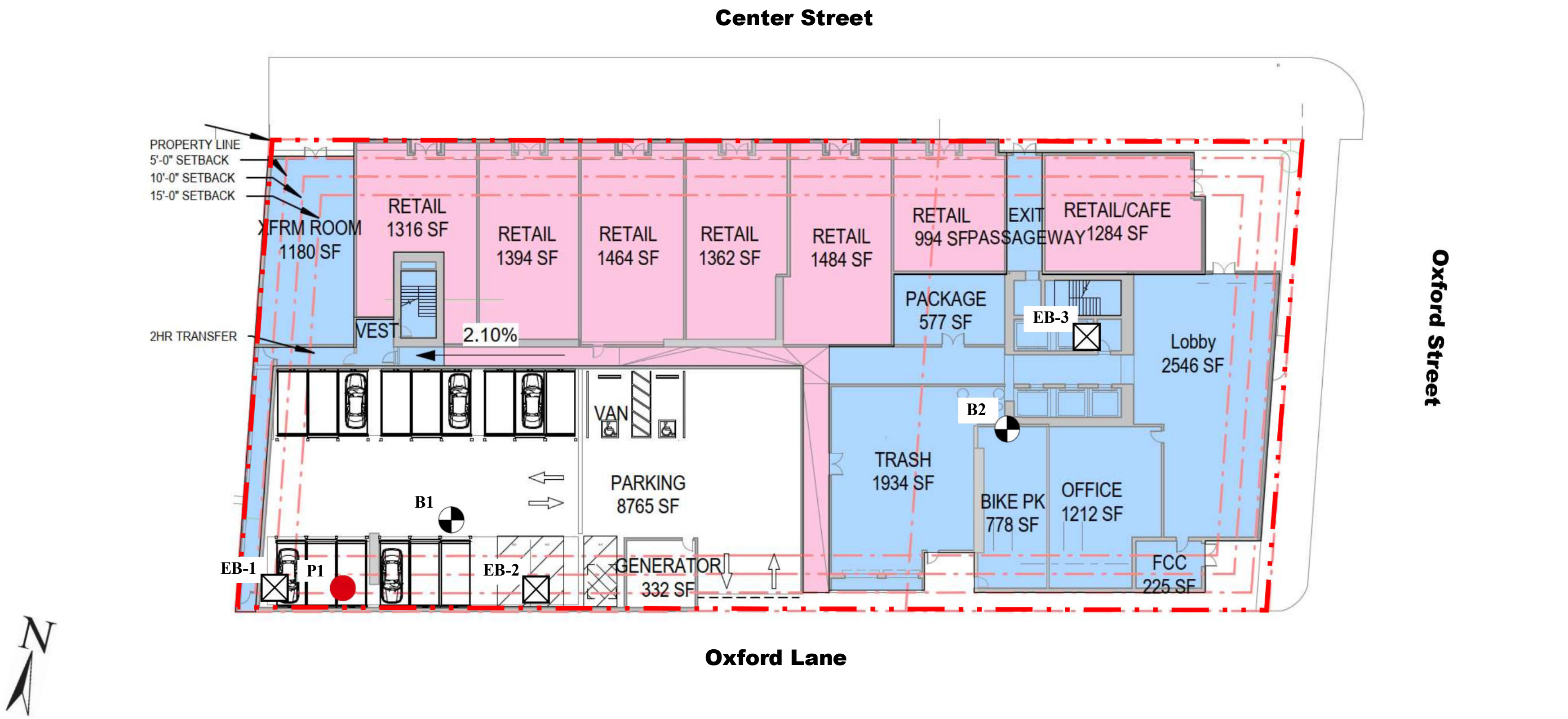
Source: Google Earth Pro

FIGURE 2 – SITE EXPLORATION MAP

KEY

- Approximate Location of Current Borings (Partner, 2021)
- Approximate Location of Previous Borings (Harza, 2000)
- Approximate Location of Previous Infiltration Test (Partner, 2021)
- Approximate Site Limits

c



Source: Kimley Horn, oLiv, Ground Level - Base Detail, Proposed Project Plans, Pre-application SB330 Submittal, Sheet 2-A2, dated 06/02/2022

FIGURE 3 – SCALED BORING LOCATION PLAN

KEY

Approximate Location of Current Borings (Partner, 2021)

Approximate Location of Previous Borings (Harza, 2000)

Scale
1 inch = 30 feet

Approximate Location of Previous Infiltration Test (Partner, 2021)

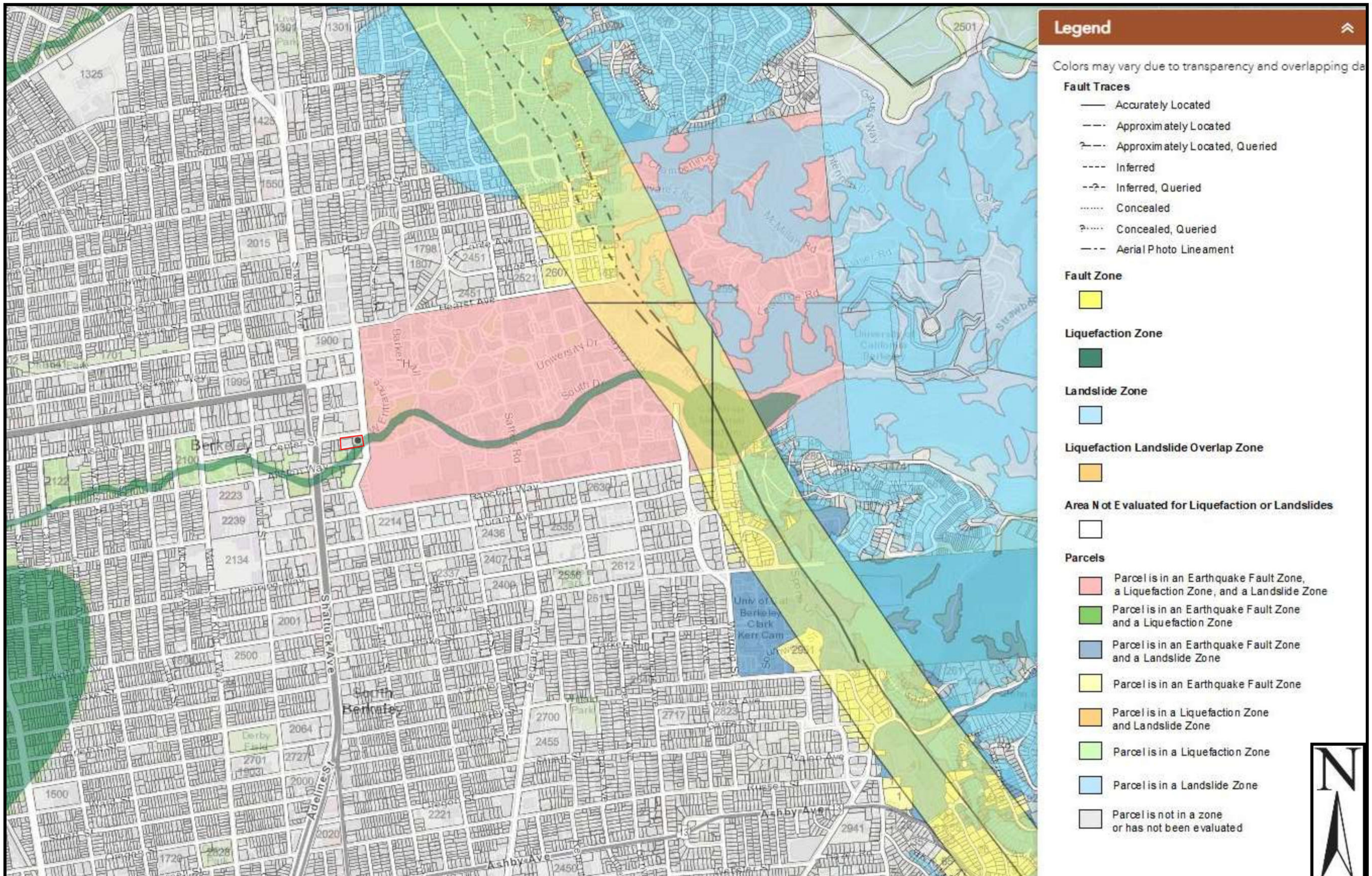
Approximate Site Limits

0 30' 60' 90' 120' 240' 360'

Source: Radbruch, D.H., ed., 1957, Areal and engineering geology of the Oakland West quadrangle, California: U.S. Geological Survey, Miscellaneous Geologic Investigations Map I-239, scale 1:24,000

FIGURE 4 – GEOLOGIC MAP

KEY Approximate Site Location



Source: California Geological Survey, 1998, Earthquake Zones of Required Investigation, Oakland West Quadrangle, scale 1:24,000

FIGURE 5 – GEOLOGIC HAZARD MAP

KEY Approximate Site Limits

APPENDIX A

Current Boring Logs

Prior Boring Logs and Laboratory work by Harza Consulting Engineers and Scientists

PARTNER

Current Boring Logs

BORING LOG KEY - EXPLANATION OF TERMS

SURFACE COVER: General discription with thickness to the inch, ex. Topsoil, Concrete, Asphalt, etc,

FILL: General description with thickness to the 0.5 feet. Ex. Roots, Debris, Processed Materials (Pea Gravel, etc.)

NATIVE GEOLOGIC MATERIAL: Deposit type, 1.Color, 2.moisture, 3.density, 4.SOIL TYPE, other notes - Thickness to 0.5 feet

1. Color - Generalized
Light Brown (usually indicates dry soil, rock, caliche)
Brown (usually indicates moist soil)
Dark Brown (moist to wet soil, organics, clays)
Reddish (or other bright colors) Brown (moist, indicates some soil development/or residual soil)
Greyish Brown (Marine, sub groundwater - not the same as light brown above)
Mottled (brown and gray, indicates groundwater fluctuations)

2. Moisture
dry - only use for wind-blown silts in the desert
damp - soil with little moisture content
moist - near optimum, has some cohesion and stickyness
wet - beyond the plastic limit for clayey soils, and feels wet to the touch for non clays
saturated - Soil below the groundwater table, sampler is wet on outside

3A. Relative Density for Granular Soils

Relative Density	Ring	SPT
very loose	0-7	0-4
loose	7-14	4-10
medium dense	14-28	10-30
dense	28-100	30-50
very dense	100+	Over 50

3B. Consistency of Fine-Grained Cohesive Soils

Consistnecy	SPT	Undrained Shear Strength, tsf
very soft	0-2	less than 0.125
soft	2-4	0.125 - 0.25
medium stiff	4-8	0.25 - 0.50
stiff	8-15	0.50 - 1.0
very stiff	15-30	1.0 - 2.0
hard	Over 30	Over 2.0

4. Classification
Determine percent Gravel (Material larger than the No. 4 Sieve)
Determine percent fines (Material passing the No. 200 Sieve)
Determine percent sand (Passing the No. 4 and retained on the No. 200 Sieve)
Determine if clayey (make soil moist, if it easily roll into a snake it is clayey)

Coarse Grained Soils (Less than 50% Passing the No. 200 Sieve)				
GP	SP	Mostly sand and gravel, with less than 5 % fines	sandy GRAVEL	SAND
GP-GM	SP-SM	Mostly sand and gravel 5-12% fines, non-clayey	sandy GRAVEL with silt	SAND with Silt
GP-GC	SP-SC	Mostly sand and gravel 5-12% fines, clayey	sandy GRAVEL with clay	SAND with clay
GC	SC	Mostly sand and gravel >12% fines clayey	clayey GRAVEL	clayey SAND
GM	SM	Mostly sand and gravel >12% fines non-clayey	silty GRAVEL	silty SAND

Fine Grained Soils (50% or more passes the No. 200 Sieve)		
ML	Soft, non clayey	SILT with sand
MH	Very rare, holds a lot of water, and is pliable with very low strength	high plasticity SILT
CL	If sandy can be hard when dry, will be stiff/plastic when wet	CLAY with sand/silt
CH	Hard and resiliant when dry, very strong/sticky when wet (may have sand in it)	FAT CLAY

H = Liquid limit over 50%, L - LL under 50%
C = Clay
M = Silt

Samplers
S = Standard split spoon (SPT)
R = Modified ring
Bulk = Excavation spoils
ST = Shelby tube
C = Rock core

Boring Number:	B1	Boring Log Page 1 of 4	
Location:	See Figure 2	Date Started:	12/3/2021
Site Address:	2128 Oxford Street Berkeley, California 94704	Date Completed:	12/3/2021
		Depth to Groundwater:	28 feet
Project Number:	20-297761.3	Field Technician:	M. Hachey
Drill Rig Type:	CME-55	Partner Engineering and Science	
Sampling Equipment:	Cal Mod / Split Spoon Sampler	1017 22nd Avenue, Suite 107	
Borehole Diameter:	8 inch	Oakland, CA 94606	

Depth, FT	Sample	N-Value	USCS	Description
0				**Surface Cover:** 1.5 inches of Asphalt over 4 inches of Base
0.5				
1				Little recovery due to debris
1.5				
2	R	7	SC	**FILL:** Dark gray-brown, moist, loose, Clayey SAND, with trace silt, trace construction
2.5				debris (i.e.: brick fragments)
3				
3.5				
4	S	7		--- Decreased construction material (i.e.: brick fragments) below 4 feet
4.5				(PI: 22, LL: 40, Fines: 44%)
5				
5.5				
6	R	13	CL	**NATIVE:** Dark gray-brown, moist, stiff, Sandy CLAY, trace silt
6.5				(Dry Density: 97.5 pcf, Moisture Content: 23.3%)
7				
7.5				
8	S	6		---Gray to grayish-brown, moist to very moist, CLAY with sand, trace gravel, some
8.5				manganese oxide staining
9				
9.5				
10				
10.5				
11				
11.5				
12				
12.5				
13				
13.5				
14				
14.5				
15	R	24		--- Becomes gray to tannish-gray, moist, very stiff, sandy CLAY
15.5				(Dry Density: 111.8 pcf, Moisture Content: 18.3%)
16				
16.5				
17				
17.5				
18				
18.5				
19				
19.5				
20	S	17		--- Becoming light orange-brown, decreased sand
20.5				(PI: 23, LL: 40, Fines: 74%)
21				
21.5				(continues on next page)

Boring Number:	B1	Boring Log Page 2 of 4	
Location:	See Figure 2	Date Started:	12/3/2021
Site Address:	2128 Oxford Street Berkeley, California 94704	Date Completed:	12/3/2021
		Depth to Groundwater:	28 feet
Project Number:	20-297761.3	Field Technician:	M. Hachey
Drill Rig Type:	CME-55	Partner Engineering and Science	
Sampling Equipment:	Cal Mod / Split Spoon Sampler	1017 22nd Avenue, Suite 107	
Borehole Diameter:	8 inch	Oakland, CA 94606	

Depth, FT	Sample	N-Value	USCS	Description
20	S	17	CL	Light orange-brown, moist, very stiff, sandy CLAY, with trace gravel, some manganese
20.5				oxide stains
21				(PI: 23, LL: 40, Fines: 74%)
21.5				
22				
22.5				
23				
23.5				
24				
24.5				
25	S	9		--- Stiff, increased sand
25.5				
26				
26.5				
27				
27.5			∇	**Groundwater encountered**
28				
28.5				
29				
29.5				
30	S	19	SC	Grayish brown, saturated, medium dense, clayey SAND, with abundant iron staining
30.5				
31				
31.5				
32				
32.5				
33				
33.5				
34				
34.5				
35	S	10	CL	Gray, moist, stiff, CLAY, with trace sand, with some iron staining and some manganese
35.5				oxide staining
36				
36.5				
37				
37.5				
38				
38.5				
39				
39.5				
40	S	23	CL	--- Very stiff, some iron oxide and manganese oxide staining
40.5				(PI: 23, LL: 38, Fines: 69%)
41				
41.5				(continues on next page)

Boring Number:	B1	Boring Log Page 2 of 4	
Location:	See Figure 2	Date Started:	12/3/2021
Site Address:	2128 Oxford Street	Date Completed:	12/3/2021
	Berkeley, California 94704	Depth to Groundwater:	28 feet
Project Number:	20-297761.3	Field Technician:	M. Hachey
Drill Rig Type:	CME-55	Partner Engineering and Science	
Sampling Equipment:	Cal Mod / Split Spoon Sampler	1017 22nd Avenue, Suite 107	
Borehole Diameter:	8 inch	Oakland, CA 94606	

Depth, FT	Sample	N-Value	USCS	Description
40	S	23	CL	Gray, moist, very stiff, CLAY, trace coarse sand, some iron staining and some
40.5				manganese oxide staining
41				(PI: 23, LL: 38, Fines: 69%)
41.5				
42				
42.5				
43				
43.5				
44				
44.5				
45	S	26	SC	Gray, moist, medium dense, clayey SAND, trace gravel
45.5				(PI: 23, LL: 40, Fines: 40%)
46				
46.5				
47				
47.5				
48				
48.5				
49				
49.5				
50	S	16	CL	Gray, moist, very stiff, sandy CLAY, decreased sand and gravel
50.5				
51				
51.5				
52				
52.5				
53				
53.5				
54				
54.5				
55	S	41		--- Becoming hard, increased sand and gravel
55.5				
56				
56.5				
57				
57.5				
58				
58.5				
59				
59.5				
60	S	24	SC	Gray, moist, medium dense, clayey SAND, with some gravel, highly weathered bedrock
60.5				
61				
61.5				(continues on next page)

Boring Number:	B1	Boring Log Page 2 of 4	
Location:	See Figure 2	Date Started:	12/3/2021
Site Address:	2128 Oxford Street	Date Completed:	12/3/2021
	Berkeley, California 94704	Depth to Groundwater:	28 feet
Project Number:	20-297761.3	Field Technician:	M. Hachey
Drill Rig Type:	CME-55	Partner Engineering and Science	
Sampling Equipment:	Cal Mod / Split Spoon Sampler	1017 22nd Avenue, Suite 107	
Borehole Diameter:	8 inch	Oakland, CA 94606	

Depth, FT	Sample	N-Value	USCS	Description
60	S	24	SC	Gray, moist, medium dense, clayey SAND, some gravel, highly weathered bedrock
60.5				
61				
61.5				
62				
62.5				
63				
63.5				
64				
64.5				
65	S	19	CL	Gray, moist, very stiff, sandy CLAY
65.5				
66				
66.5				
67				
67.5				
68				
68.5				
69				
69.5				
70	R	16-50/5.5"		Purple-ish brown, moist, hard, sandy CLAY, with abundant gravels
70.5				(Dry Density: 124.8 pcf, Moisture Content: 15.5%)
71				
71.5				
72				
72.5				
73				
73.5				
74				
74.5				
75	S	50/2 in.		**BEDROCK:** Olive green and brown, moist, hard, serpentenite, with claystone
75.5				Boring terminated at 75.5 feet
76				Groundwater encountered at 28 feet
76.5				Boring grouted and patched upon completion
77				
77.5				
78				
78.5				
79				
79.5				
80				
80.5				
81				
81.5				

Boring Number:	B2			Boring Log Page 1 of 4
Location:	See Figure 2		Date Started:	12/3/2021
Site Address:	2128 Oxford Street		Date Completed:	12/3/2021
	Berkeley, California 94704		Depth to Groundwater:	30 feet
Project Number:	20-297761.3		Field Technician:	M. Hachey
Drill Rig Type:	CME-55		Partner Engineering and Science	
Sampling Equipment:	Cal Mod / Split Spoon Sampler		1017 22nd Avenue, Suite 107	
Borehole Diameter:	8 inch		Oakland, CA 94606	

Depth, FT	Sample	N-Value	USCS	Description
0				**SURFACE COVER:** 1.5 inches of Asphalt over 4 inches of Base
0.5				
1				
1.5				
2				
2.5				
3	R	18	CL	**FILL:** Dark brown, dry, stiff, silty CLAY, trace sand and trace fine debris
3.5				
4				(Dry Density: 98.0, Moisture Content: 18.5%)
4.5				
5				
5.5				
6	S	19	CL	**NATIVE:** Grayish brown, moist, very stiff, CLAY
6.5				
7				
7.5				
8				
8.5				
9	R	26		--- Becomes sandy CLAY
9.5				(Dry Density: 117.4 pcf, Moisture Content: 15.8%)
10				
10.5				
11				
11.5				
12				
12.5				
13				
13.5				
14				
14.5				
15	S	7		--- Becoming medium stiff
15.5				
16				
16.5				
17				
17.5				
18				
18.5				
19				
19.5				
20	S	41		--- Very moist, CLAY with sand
20.5				
21				
21.5				(continues on next page)

Boring Number:	B2	Boring Log Page 2 of 4	
Location:	See Figure 2	Date Started:	12/3/2021
Site Address:	2128 Oxford Street	Date Completed:	12/3/2021
	Berkeley, California 94704	Depth to Groundwater:	30 feet
Project Number:	20-297761.3	Field Technician:	M. Hachey
Drill Rig Type:	CME-55	Partner Engineering and Science	
Sampling Equipment:	Cal Mod / Split Spoon Sampler	1017 22nd Avenue, Suite 107	
Borehole Diameter:	8 inch	Oakland, CA 94606	

Depth, FT	Sample	N-Value	USCS	Description
20	S	41	CL	Grayish brown, very moist, medium stiff, CLAY with sand
20.5				
21				
21.5				
22				
22.5				
23				
23.5				
24				
24.5				
25				
25.5				
26				
26.5				
27				
27.5				
28				
28.5				
29				
29.5			▽	**Groundwater encountered**
30	S	14		Light orange-brown, saturated, stiff, with trace manganese oxide staining
30.5				
31				
31.5				
32				
32.5				
33				
33.5				
34				
34.5				
35				
35.5				
36				
36.5				
37				
37.5				
38				
38.5				
39				
39.5				
40	S	40		Hard, sandy CLAY, with abundant gravel, some manganese oxide staining
40.5				
41				
41.5				(continues on next page)

Boring Number:	B2		Boring Log Page 2 of 4
Location:	See Figure 2	Date Started:	12/3/2021
Site Address:	2128 Oxford Street	Date Completed:	12/3/2021
	Berkeley, California 94704	Depth to Groundwater:	30 feet
Project Number:	20-297761.3	Field Technician:	M. Hachey
Drill Rig Type:	CME-55	Partner Engineering and Science	
Sampling Equipment:	Cal Mod / Split Spoon Sampler	1017 22nd Avenue, Suite 107	
Borehole Diameter:	8 inch	Oakland, CA 94606	

Depth, FT	Sample	N-Value	USCS	Description
40	S	40	CL	Orange-brown, moist, hard, sandy CLAY, abundant gravel, some manganese oxide
40.5				staining
41				
41.5				
42				
42.5				
43				
43.5				
44				
44.5				
45				
45.5				
46				
46.5				
47				
47.5				
48				
48.5				
49				
49.5				
50	S	26		--- Becoming wet, very stiff, increased sand, decreased gravel
50.5				
51				
51.5				
52				
52.5				
53				
53.5				
54				
54.5				
55				
55.5				
56				
56.5				
57				
57.5				
58				
58.5				
59				
59.5				
60	S	27		--- Increased gravel
60.5				
61				
61.5				(continues on next page)

Boring Number:	B2	Boring Log Page 2 of 4	
Location:	See Figure 2	Date Started:	12/3/2021
Site Address:	2128 Oxford Street	Date Completed:	12/3/2021
	Berkeley, California 94704	Depth to Groundwater:	30 feet
Project Number:	20-297761.3	Field Technician:	M. Hachey
Drill Rig Type:	CME-55	Partner Engineering and Science	
Sampling Equipment:	Cal Mod / Split Spoon Sampler	1017 22nd Avenue, Suite 107	
Borehole Diameter:	8 inch	Oakland, CA 94606	

Depth, FT	Sample	N-Value	USCS	Description
60	S	27	CL	Orange-brown, wet, very stiff, sandy CLAY, abundant gravel, some manganese oxide
60.5				staining
61				
61.5				
62				
62.5				
63				
63.5				
64				
64.5				
65	S	17		--- Becoming light gray, moist, abundant iron staining
65.5				
66				
66.5				
67				
67.5				
68				
68.5				
69				
69.5				
70	S	50/6"		--- Becoming hard, with abundant fractured bedrock shards of graywacke sandstone
70.5				
71				
71.5				
72				
72.5				
73				
73.5				
74				
74.5				
75	S	50/2"		**BEDROCK:** Dark gray, moist, hard, graywacke sandstone, highly weathered
75.5				
76				
76.5				
77				
77.5				
78				
78.5				
79				
79.5				
80	S	50/6"		
80.5				Boring terminated at 80.5 feet
81				Groundwater encountered at 30 feet
81.5				Boring grouted and patched upon completion

Prior Boring Logs and Laboratory work by Harza Consulting Engineers and Scientists

PARTNER

DRILL RIG	Mobile B-61, HSA	SURFACE ELEVATION	—	LOGGED BY	JND
DEPTH TO GROUND WATER	Not Encountered	BORING DIAMETER	8-inch	DATE DRILLED	10/16/00

DESCRIPTION AND CLASSIFICATION: DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT(%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
PAVEMENT: 3 inches AC over 8 inches AB									
FILL: SAND (SM/SC), brown, fine- to coarse-grained, some silt and clay, wet	Medium Dense				24	22	105		
CLAY (CL), black, silty, some sand (fine- to medium-grained), damp to moist	Firm				4				
			5		6	27	95		
SAND (SC), brown, fine- to coarse-grained, with gravel (fine to coarse, angular to rounded), some clay, moist to wet	Dense		10		66				
GRAVEL (GC), brown, fine to coarse, with sand (fine- to coarse-grained), trace clay, damp	Very Dense				50/5"				
			15						
CLAY (CL), brown, with silt, trace sand (medium- to coarse-grained), trace gravel (fine, subangular to subrounded), damp	Hard				49				
			20						
(grades moist, no gravel)					43				
			25						
	Stiff				19				
			30						

Bottom of Boring = 30 Feet

Notes:

1. The stratification lines represent the approximate boundaries between soil types and the transition may be gradual.
2. For an explanation of penetration resistance values, see the first page of Appendix A.
3. Samplers were driven with an automatic wire trip hammer falling 30 inches.
4. The boring was backfilled with neat cement after completion.

HARZA Engineering Company

EXPLORATORY BORING LOG		
EASTMAN BUILDING Berkeley, California		
PROJECT NO. 18292-CA	DATE November 2000	BORING NO. EB-1

File Name: G:\ENGINEER\GINTW\PROJECTS\18292CA.GPJ Report Template: H Output Date: 11/2/00

DRILL RIG	Mobile B-61, HSA	SURFACE ELEVATION	—	LOGGED BY	JND
DEPTH TO GROUND WATER	18 feet	BORING DIAMETER	8-inch	DATE DRILLED	10/16/00

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT(%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
PAVEMENT: 3½ inches AC over 9 inches AB									
FILL: CLAY (CL), dark brown, silty, some sand (fine-grained), moist	Very Stiff				28	19	104		LL = 37, PI = 20, Passing No. 200 Sieve = 63%
(brick fragments, trace fine and subangular gravel)	Stiff				11				
CLAY (CL), dark brown, silty, some sand (fine-grained), trace gravel (fine, subangular), moist	Very Stiff		5		28	21	103		LL = 37, PI = 20, Passing No. 200 Sieve = 67%
					30				
SAND (SC), brown, fine- to coarse-grained, some clay, some silt, some gravel (fine, subangular), moist	Medium Dense		10						
CLAY (CL), brown, with silt, with sand (fine-grained), moist	Stiff				63				
GRAVEL (GC), brown, fine to coarse, subangular to subrounded, some clay, some sand (fine- to coarse-grained), moist	Very Dense		15						
CLAY (CL), brown, with silt, trace gravel (fine, subangular to subrounded), moist	Hard				49				
			20						
(grades silty, trace fine- to coarse-grained sand, damp)					67	32	91	9.3	
			25						
CLAY (CL/CH), brown, silty, trace sand (fine- to coarse-grained), damp	Hard				33				
			30						

Bottom of Boring = 30 Feet

Notes:

1. The stratification lines represent the approximate boundaries between soil types and the transition may be gradual.
2. For an explanation of penetration resistance values, see the first page of Appendix A.
3. Samplers were driven with an automatic wire trip hammer falling 30 inches.
4. The boring was backfilled with neat cement after completion.

HARZA Engineering Company

EXPLORATORY BORING LOG

EASTMAN BUILDING
Berkeley, California

PROJECT NO.	DATE	BORING NO.
18292-CA	November 2000	EB-2

DRILL RIG	Mobile B-61, HSA	SURFACE ELEVATION	—	LOGGED BY	JND
DEPTH TO GROUND WATER	17 feet	BORING DIAMETER	8-inch	DATE DRILLED	10/16/00

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT(%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
PAVEMENT: 3½ inches AC over 14½ inches AB									
FILL: CLAY (CL), dark brown, silty, trace sand (fine- to coarse-grained), damp, trace gravel (fine, subangular to subrounded), trace brick fragmants, damp	Stiff				22	21	98		LL = 38, PI = 22, Passing No. 200 Sieve = 74%
	Hard				49				
GRAVEL (GP/GC), brown, fine to coarse, subangular to subrounded, with sand (fine- to coarse-graiend), trace clay and silt, damp	Dense		5		51	8	101		
SAND (SP/SC), brown, fine- to coarse-grained, some gravel (fine, subangular to subrounded), trace clay and silt, damp	Dense				40				
			10						
CLAY (CL), brown, silty, some sand (fine-grained), wet	Very Stiff				21				
			15						
GRAVEL (GC), brown, fine to coarse, subangular to subrounded, some sand (fine- to coarse-grained), some clay, saturated	Very Dense				50/5"				
			20						
CLAY (CL), brown, silty, trace sand (fine- to coarse-grained), damp to moist	Hard				68				
			25						
					40				
			30						

Bottom of Boring = 30 Feet

Notes:

1. The stratification lines represent the approximate boundaries between soil types and the transition may be gradual.
2. For an explanation of penetration resistance values, see the first page of Appendix A.
3. Samplers were driven with an automatic wire trip hammer falling 30 inches.
4. The boring was backfilled with neat cement after completion.

File Name: G:\ENGINEERGINTWPROJECTS\18292CA.GPJ Report Template: H Output Date: 11/2/00

HARZA
Engineering Company

EXPLORATORY BORING LOG

EASTMAN BUILDING
Berkeley, California

PROJECT NO.	DATE	BORING NO.
18292-CA	November 2000	EB-3

UNIFIED SOIL CLASSIFICATION SYSTEM

Major Divisions		grf	ltr	Description
Coarse Grained Soils	Gravel And Gravelly Soils		GW	Well-graded gravels or gravel sand mixtures, little or no fines
			GP	Poorly-graded gravels or gravel sand mixture, little or no fines
			GM	Silty gravels, gravel-sand-silt mixtures
			GC	Clayey gravels, gravel-sand-clay mixtures
	Sand And Sandy Soils		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, and-clay mixtures

Major Divisions		grf	ltr	Description
Fine Grained Soils	Silts And 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 silt-clays of low plasticity
	Silts And Clays LL > 50		MH	Inorganic silts, micaceous or diatomaceous fine or silty soils, elastic silts
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic clays of medium to high plasticity
Highly Organic Soils			PT	Peat and other highly organic soils

GRAIN SIZES

U.S. STANDARD SERIES SIEVE: 200, 40, 10, 4 — CLEAR SQUARE SIEVE OPENINGS: 3/4", 3", 12"

Silts and Clays	Sand			Gravel		Cobbles	Boulders
	Fine	Medium	Coarse	Fine	Coarse		

RELATIVE DENSITY

Sands and Gravels	Blows/Foot*
Very Loose	0 - 4
Loose	4 - 10
Medium Dense	10 - 30
Dense	30 - 50
Very Dense	Over 50

CONSISTENCY

Silts and Clays	Blows/Foot*	Strength (tsf)**
Very Soft	0 - 2	0 - 1/4
Soft	2 - 4	1/4 - 1/2
Firm	4 - 8	1/2 - 1
Stiff	8 - 16	1 - 2
Very Stiff	16 - 32	2 - 4
Hard	Over 32	Over 4

*Number of Blows for a 140-pound hammer falling 30 inches, driving a 2-inch O.D. (1-3/8" I.D.) split spoon sampler.

**Unconfined compressive strength.

SYMBOLS

- Standard Penetration sample
- Modified California sample
- Shelby Tube sample
- ▽ Ground Water level during drilling
- ▼ Stabilized Ground Water level

Increasing Visual Moisture Content

Dry
Damp
Moist
Wet
Saturated

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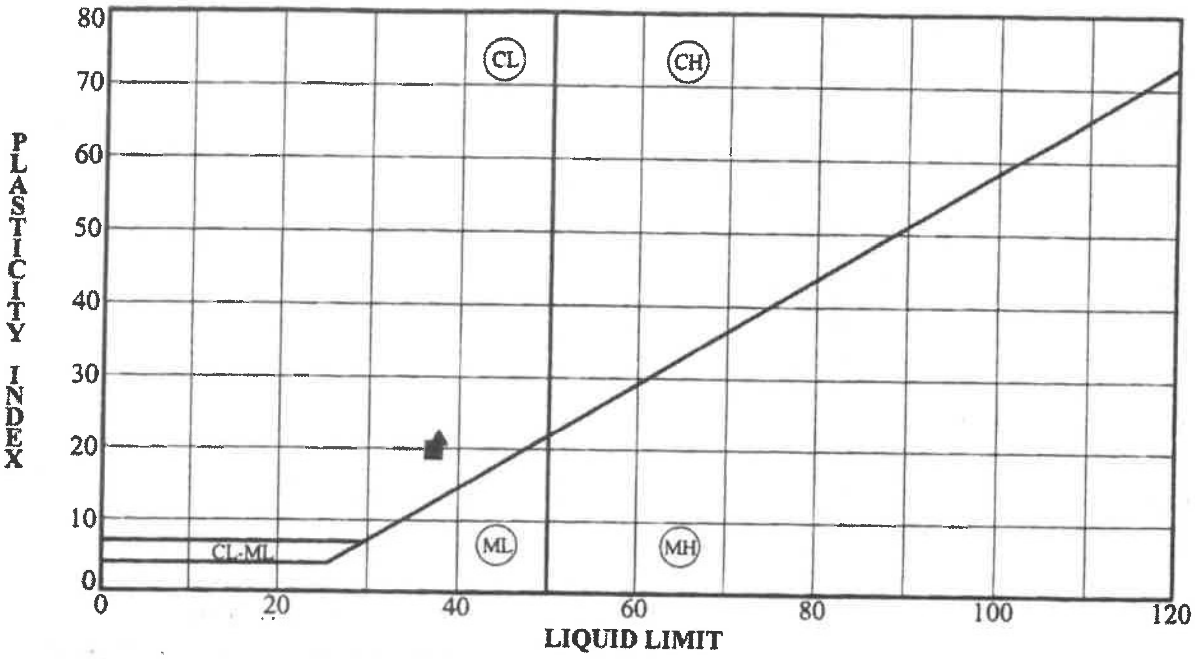
Engineering Company

KEY TO EXPLORATORY BORING LOGS		
EASTMAN BUILDING Berkeley, California		
PROJECT NO.	DATE	FIGURE NO.
18292-CA	November 2000	A-1



Key Symbol	Boring No.	Depth (Feet)	Liquid Limit	Plasticity Index	Liquidity Index	Water Content (%)	% Passing #200 Sieve	USCS
●	EB-2	0.5	37	20	---	---	63	CL
⊠	EB-2	2.0	37	20	0.054	19	67	CL
▲	EB-3	2.0	38	22	0.205	21	74	CL

HARZA Engineering Company

PREP'D BY
APP'D BY
DATE 11/2/00
DWG FILE 18292-CA.GPJ

PLASTICITY CHART AND DATA

EASTMAN BUILDING
Berkeley, California

FIGURE B-1

PROJECT No. 18292-CA

File Name: H:\ENGINEER\GINTW\PROJECTS\18292-CA.GPJ Report Template: ATT B Output Date: 11/2/00

APPENDIX B

Laboratory Test Results

PARTNER

MOISTURE CONTENT & UNIT WEIGHT TEST RESULTS

Sample Identification	Depth, ft.	Wet Unit Weight, lb/ft.^{3}	Dry Unit Weight, lb/ft.^{3}	Moisture Content, %
1-3 @ 7'	7'	120.2	97.5	23.3
1-5 @ 15'	15'	132.3	111.8	18.3
2-1 @ 4'	4'	116.1	98.0	18.5

Test Method: ASTM D2216, ASTM D2937

PROJECT NUMBER:	21-290	December 10, 2021

GS GULF SHORE
EXPLORATION AND TESTING

3362 Fitzgerald Road
Rancho Cordova, CA 95742
Phone: (916) 939-4117
FAX: (916) 635-4315

Core Campus Berkeley Development

MOISTURE CONTENT & UNIT WEIGHT TEST RESULTS

Sample Identification	Depth, ft.	Wet Unit Weight, lb/ft.^{3}	Dry Unit Weight, lb/ft.^{3}	Moisture Content, %
1-16 @ 70.5'	70.5'	144.1	124.8	15.5
2-3 @ 10'	10'	136.0	117.4	15.8

Test Method: ASTM D2216, ASTM D2937

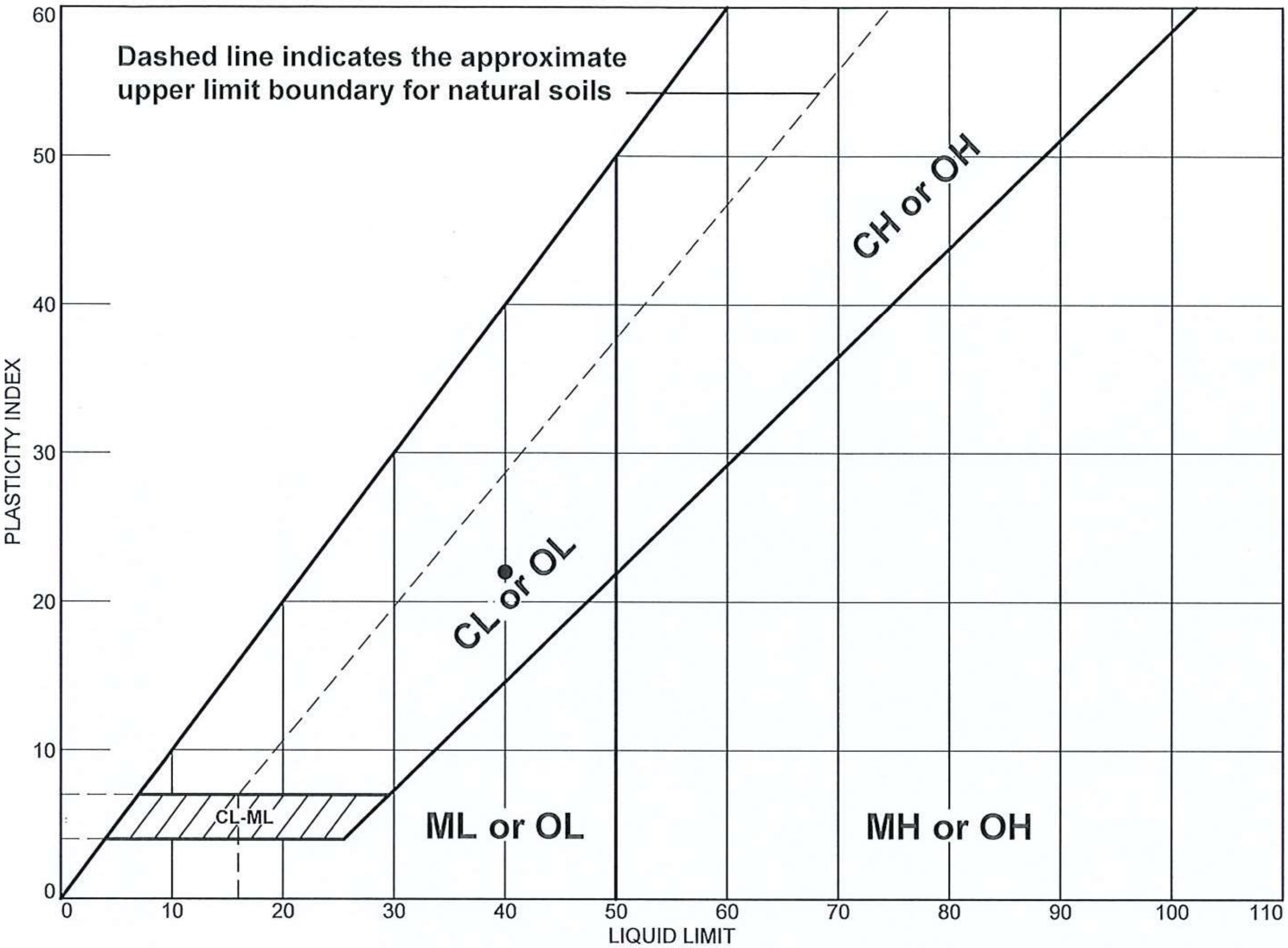
PROJECT NUMBER: 21-290 | December 14, 2021

GS GULF SHORE EXPLORATION AND TESTING

3362 Fitzgerald Road
Rancho Cordova, CA 95742
Phone: (916) 939-4117
FAX: (916) 635-4315

Core Campus Berkeley Development

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●		40	18	22		44	

Project No. 21-290 **Client:** Partner Engineering and Science

Project: Core Campus Berkeley Development

● **Location:** 1-2 @ 4' **Depth:** 4' **Sample Number:** 73013

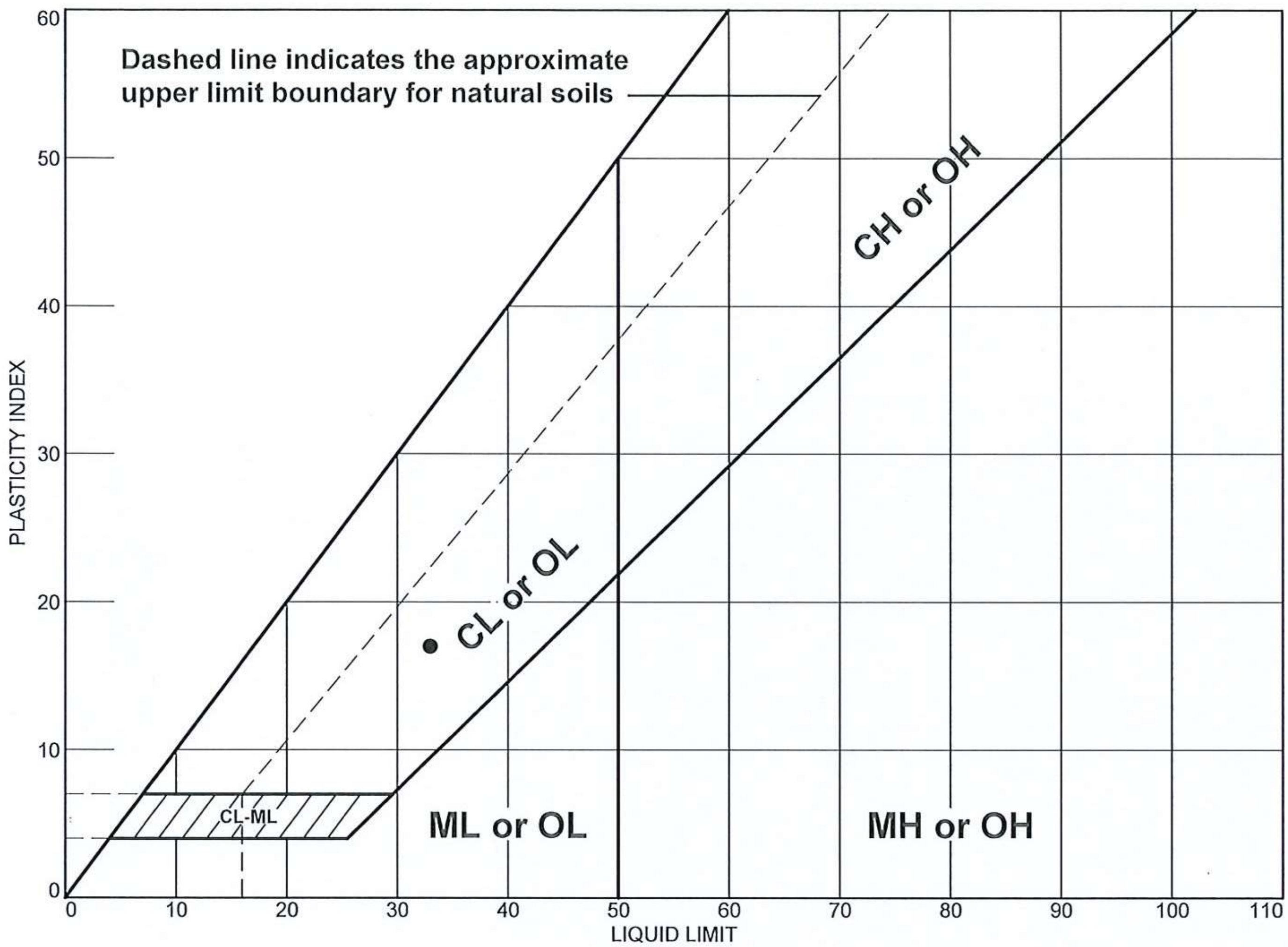
Remarks:

GS GULF SHORE EXPLORATION AND TESTING

Figure

Tested By: FI **Checked By:** JML

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●		33	16	17		74	

Project No. 21-290 **Client:** Partner Engineering and Science

Project: Core Campus Berkeley Development

● **Location:** 1-6 @ 20' **Depth:** 20' **Sample Number:** 73016

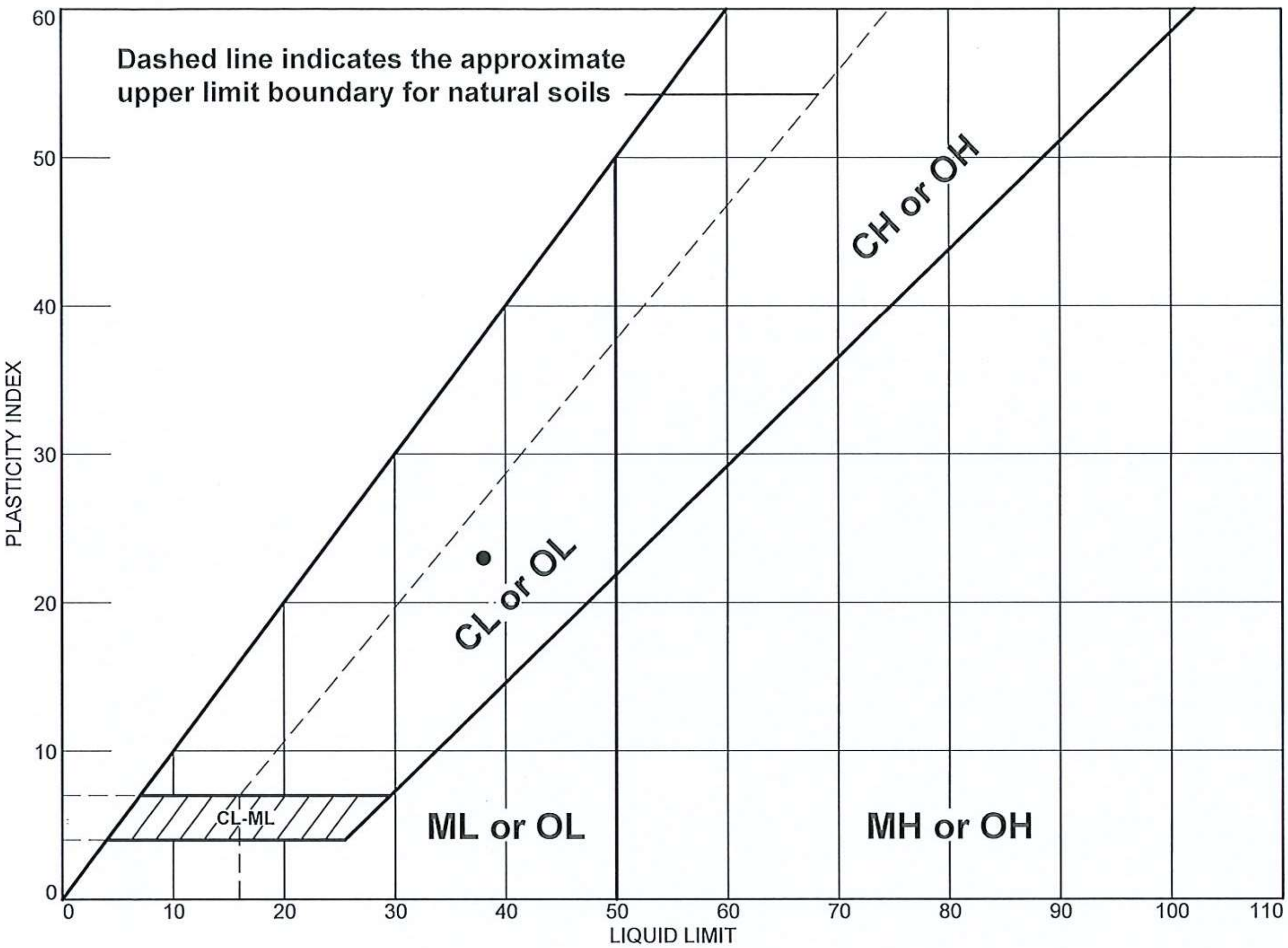
Remarks:

GS GULF SHORE EXPLORATION AND TESTING

Figure

Tested By: MM **Checked By:** JML

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●		38	15	23		69	

Project No. 21-290 **Client:** Partner Engineering and Science

Project: Core Campus Berkeley Development

● **Location:** 1-10 @ 40' **Depth:** 40' **Sample Number:** 73017

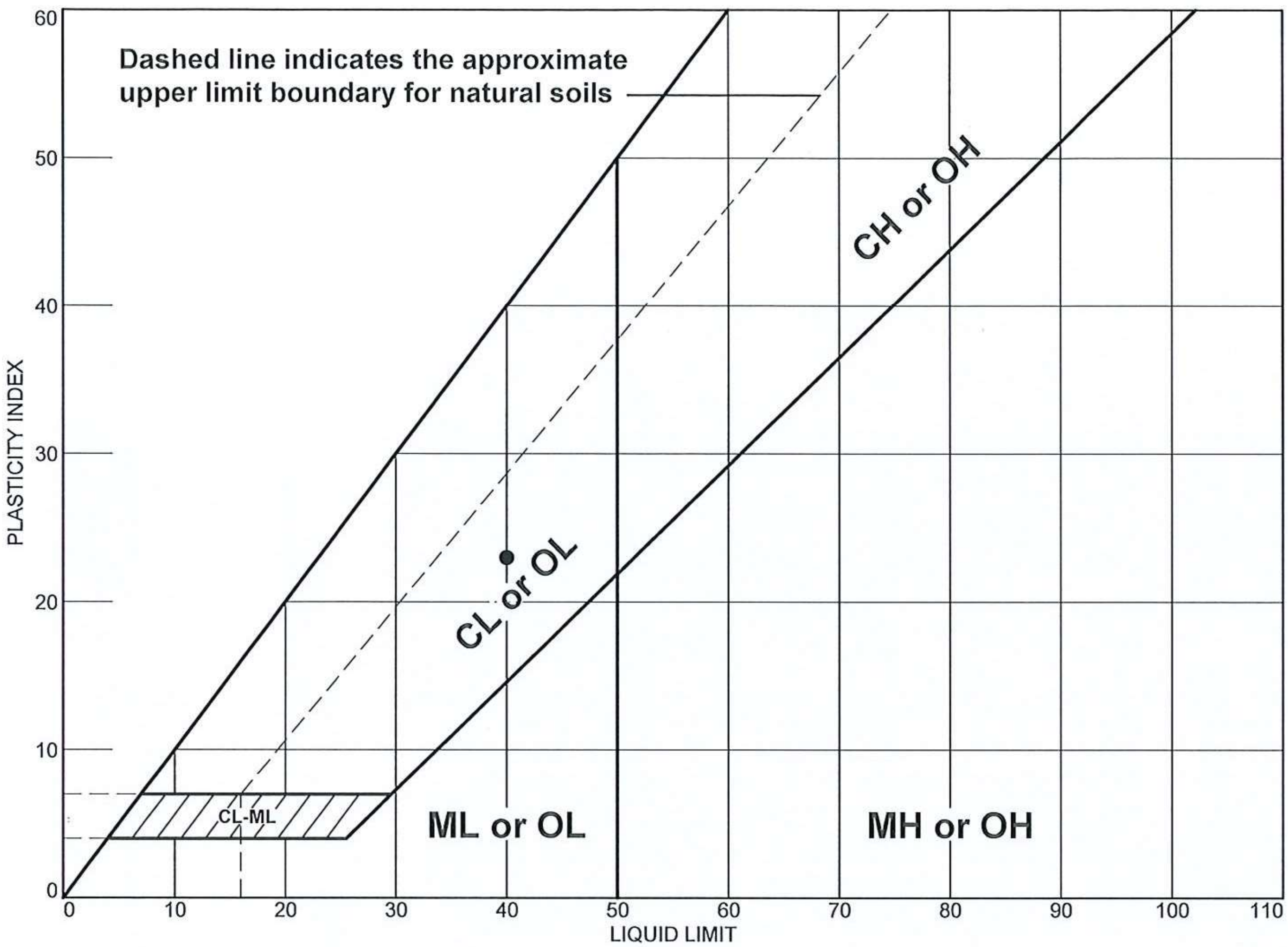
GS GULF SHORE EXPLORATION AND TESTING

Remarks:

Figure

Tested By: FI **Checked By:** JML

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●		40	17	23		40	

Project No. 21-290 **Client:** Partner Engineering and Science

Project: Core Campus Berkeley Development

● **Location:** 1-11 @ 45' **Depth:** 45' **Sample Number:** 73018

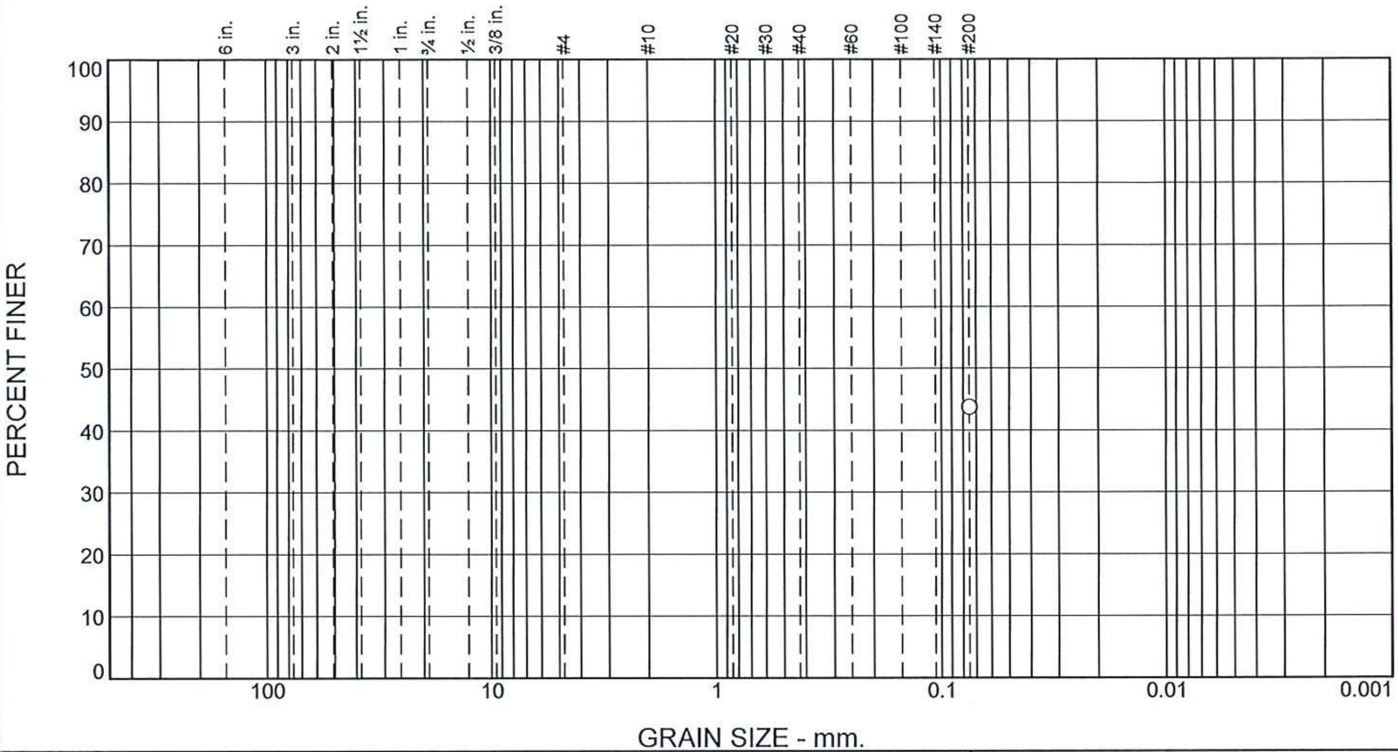
Remarks:

GS GULF SHORE EXPLORATION AND TESTING

Figure

Tested By: SH **Checked By:** JML

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines
	Coarse	Fine	Coarse	Medium	Fine	
						44

Test Results (ASTM D6913 & ASTM D1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#200	44		

* (no specification provided)

Material Description

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

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

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

Date Received: 12/7/21 **Date Tested:** 12/14/21

Tested By: MJW

Checked By: JML

Title: PM

Location: 1-2 @ 4'
Sample Number: 73013 **Depth:** 4'

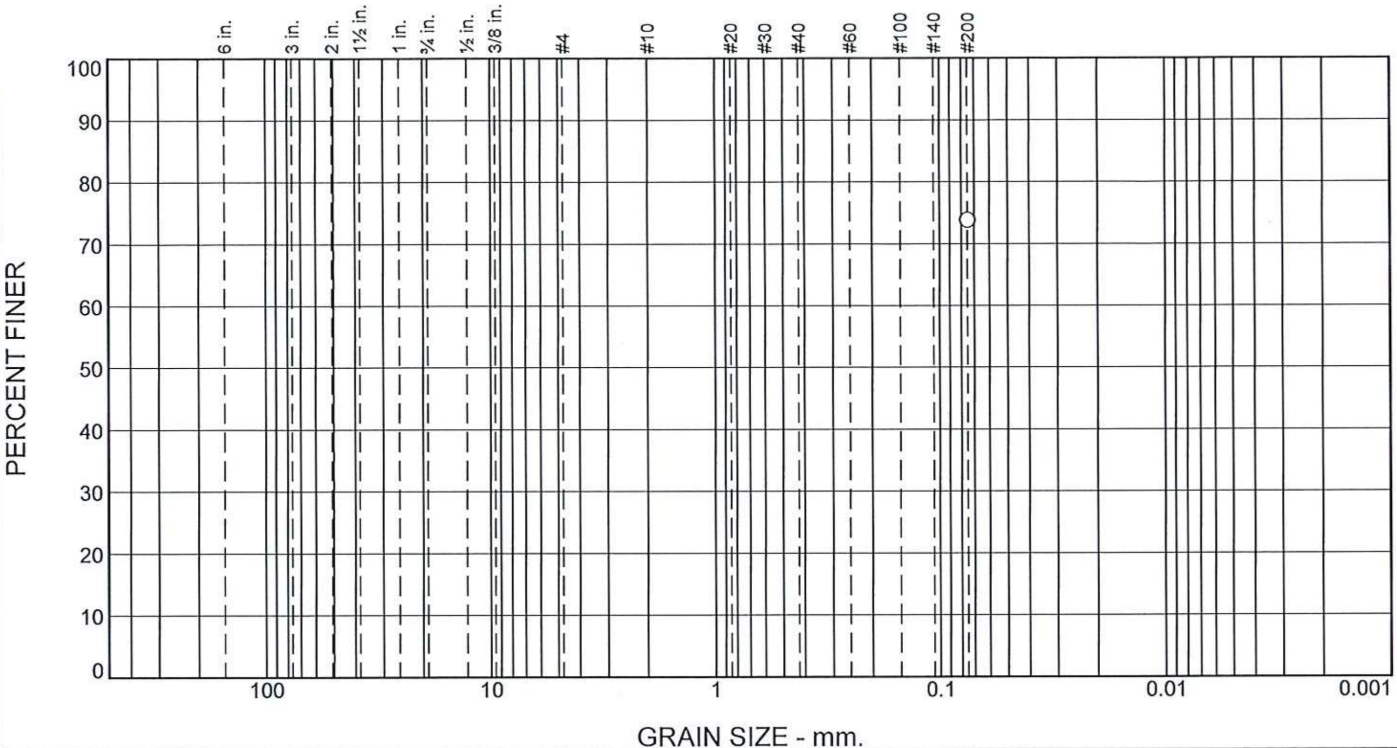
Date Sampled: 12/3/21

GS GULF SHORE EXPLORATION AND TESTING

Client: Partner Engineering and Science
Project: Core Campus Berkeley Development

Project No: 21-290 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines
	Coarse	Fine	Coarse	Medium	Fine	
						74

Test Results (ASTM D6913 & ASTM D1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#200	74		

* (no specification provided)

Material Description

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

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

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

Date Received: 12/7/21 **Date Tested:** 12/14/21

Tested By: MJW

Checked By: JML

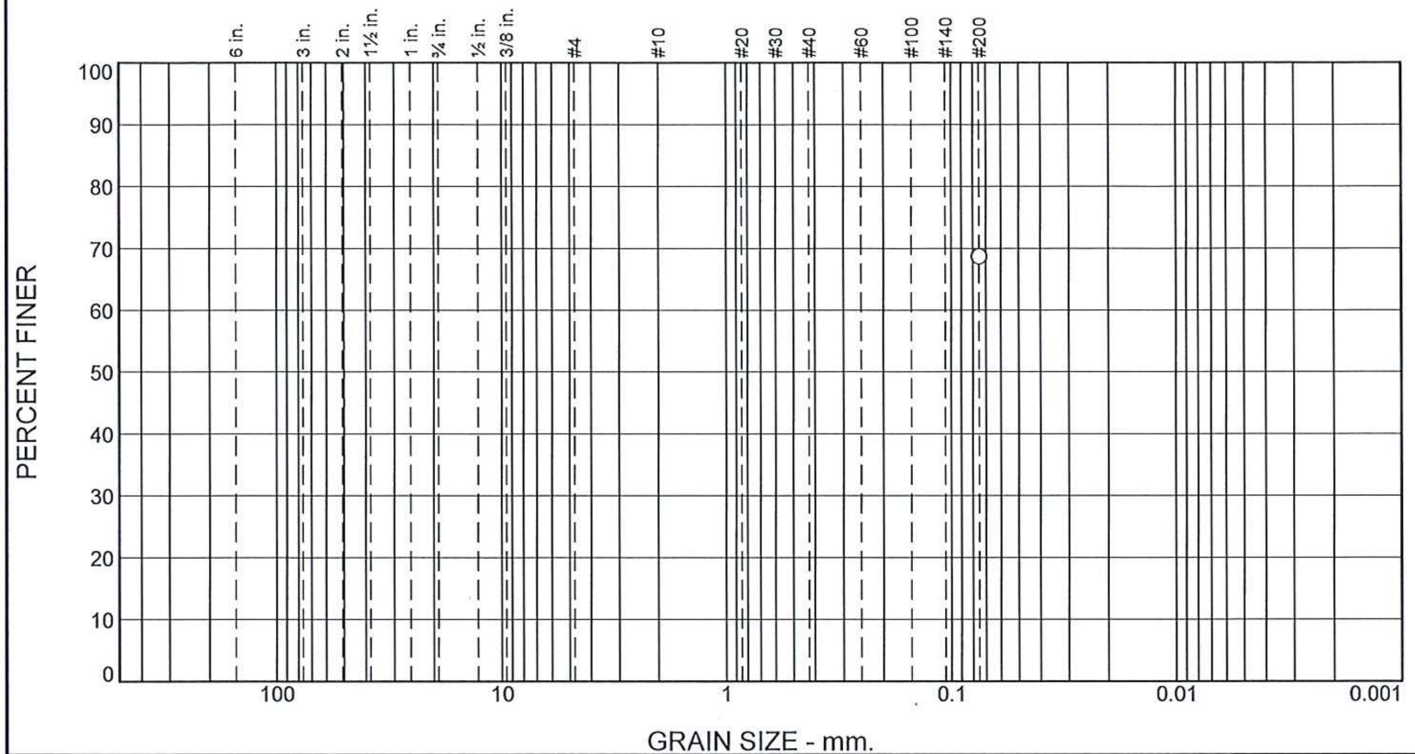
Title: PM

Location: 1-6 @ 20'
Sample Number: 73016 **Depth:** 20'
Date Sampled: 12/3/21

GS GULF SHORE EXPLORATION AND TESTING

Client: Partner Engineering and Science
Project: Core Campus Berkeley Development
Project No: 21-290 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines
	Coarse	Fine	Coarse	Medium	Fine	
						69

Test Results (ASTM D6913 & ASTM D1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#200	69		

* (no specification provided)

Material Description

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

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

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

Date Received: 12/7/21 **Date Tested:** 12/14/21

Tested By: MJW

Checked By: JML

Title: PM

Location: 1-10 @ 40'
Sample Number: 73017 **Depth:** 40'

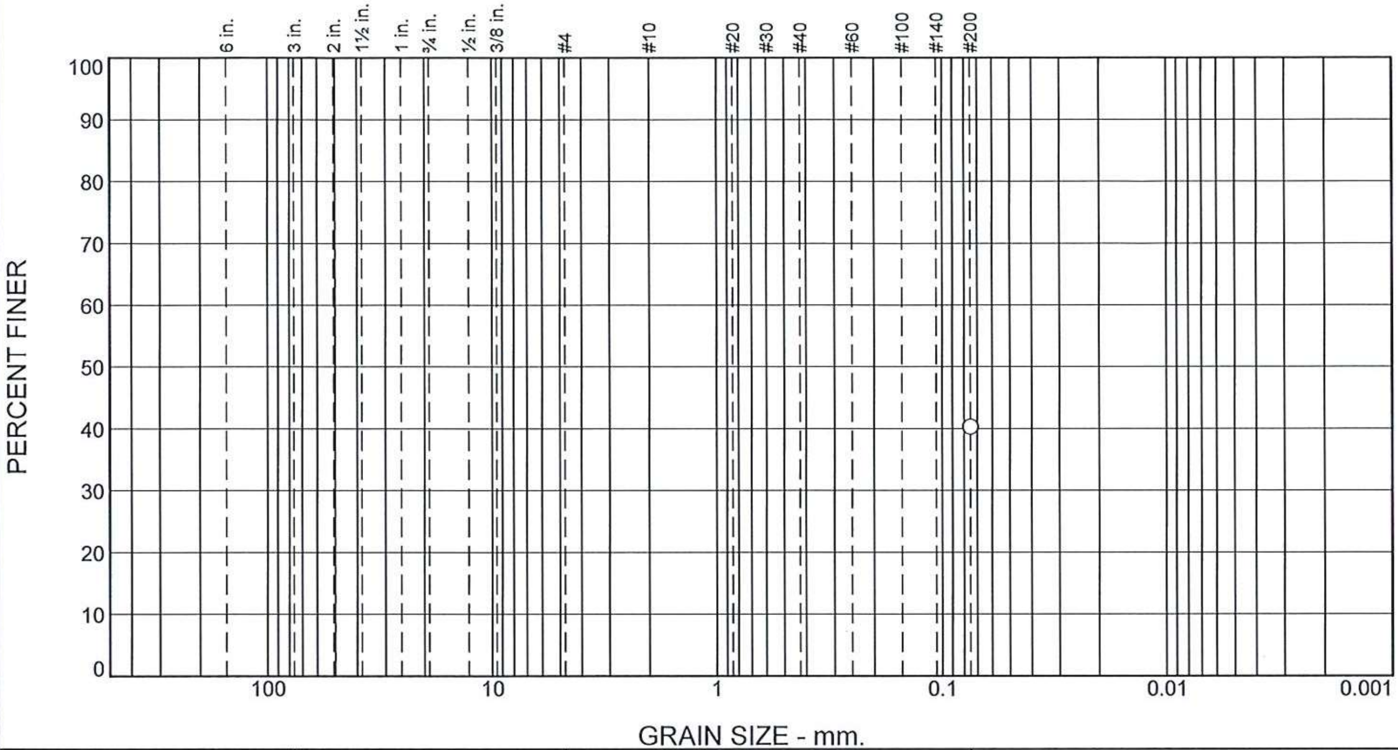
Date Sampled: 12/3/21

GS GULF SHORE EXPLORATION AND TESTING

Client: Partner Engineering and Science
Project: Core Campus Berkeley Development

Project No: 21-290 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines
	Coarse	Fine	Coarse	Medium	Fine	
						40

Test Results (ASTM D6913 & ASTM D1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#200	40		

* (no specification provided)

Material Description

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

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

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

Date Received: 12/7/21 **Date Tested:** 12/14/21

Tested By: MJW

Checked By: JML

Title: PM

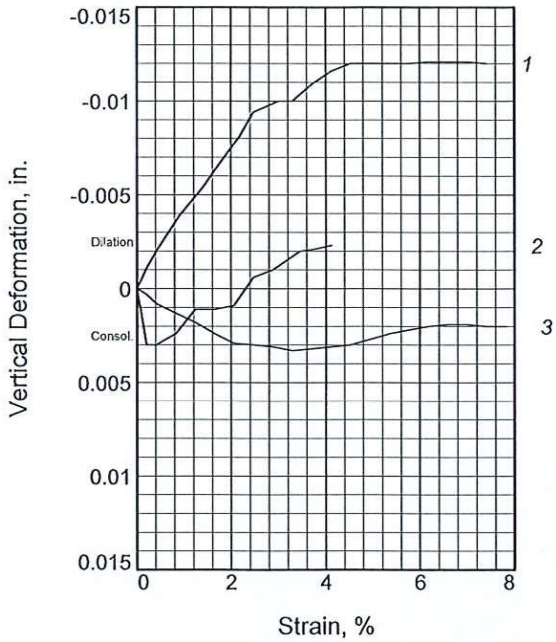
Location: 1-11 @ 45'
Sample Number: 73018 **Depth:** 45'

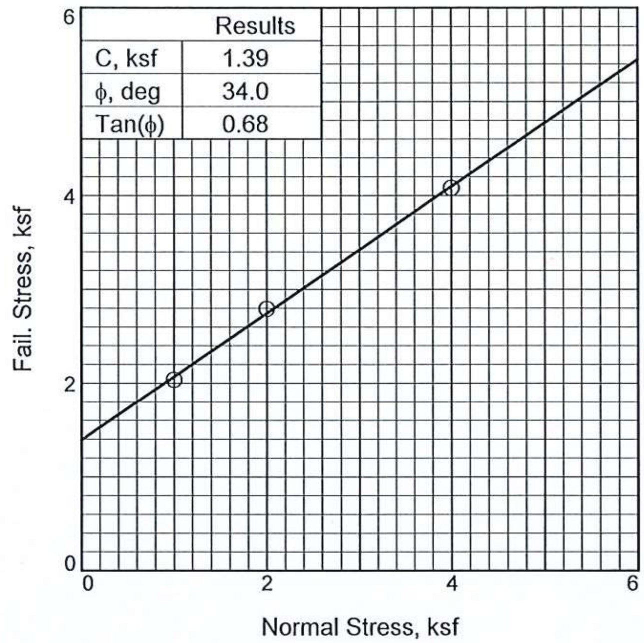
Date Sampled: 12/3/21

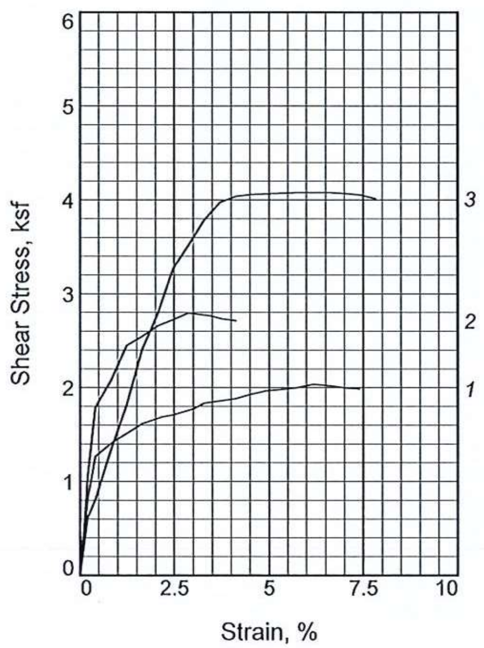
GULF SHORE EXPLORATION AND TESTING

Client: Partner Engineering and Science
Project: Core Campus Berkeley Development

Project No: 21-290 **Figure**







	Sample No.	1	2	3
Initial	Water Content, %	14.0	16.0	15.5
	Dry Density, pcf	121.2	117.5	117.9
	Saturation, %	96.6	99.2	97.6
	Void Ratio	0.3902	0.4343	0.4299
	Diameter, in.	2.43	2.43	2.43
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	13.5	15.0	13.9
	Dry Density, pcf	123.6	119.8	122.6
	Saturation, %	99.9	99.9	99.9
	Void Ratio	0.3642	0.4066	0.3747
	Diameter, in.	2.43	2.43	2.43
	Height, in.	0.98	0.98	0.96
	Normal Stress, ksf	1.00	2.00	4.00
	Fail. Stress, ksf	2.04	2.79	4.08
	Strain, %	6.2	2.9	5.8
	Ult. Stress, ksf			
	Strain, %			
	Strain rate, in./min.	0.008	0.008	0.008

Sample Type: Tube

Description:

Specific Gravity= 2.70

Remarks:

Figure ________

Client: Partner Engineering and Science

Project: Core Campus Berkeley Development

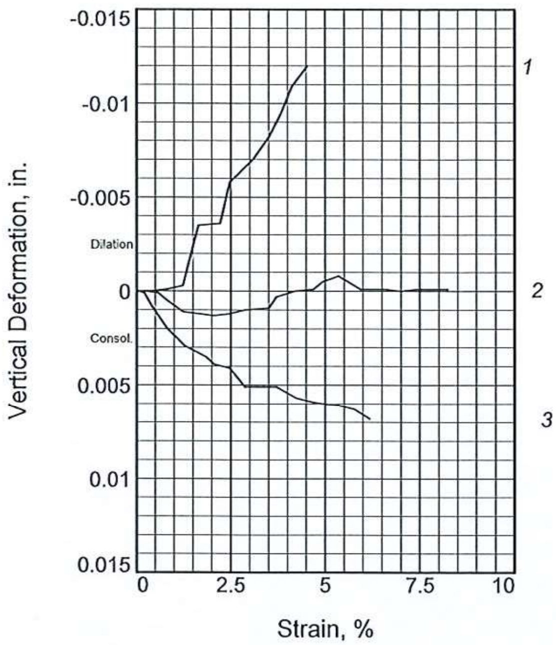
Location: 1-16 @ 70.5'

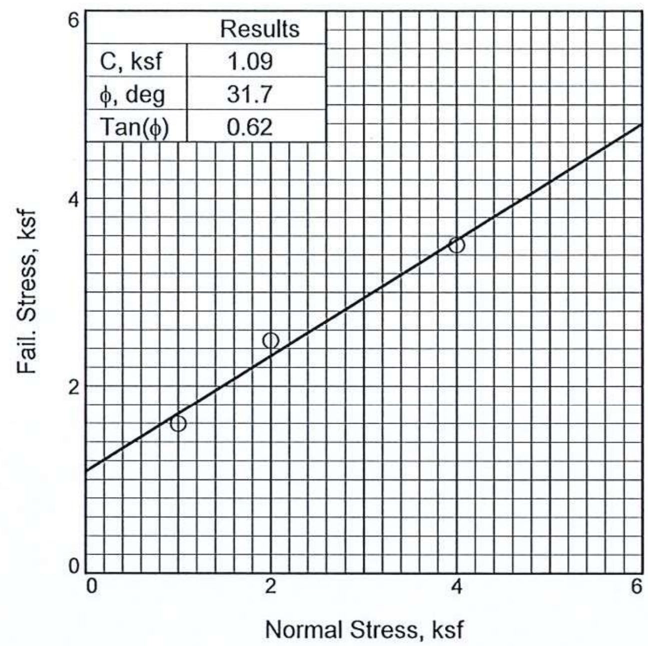
Sample Number: 73019 **Depth:** 70.5'

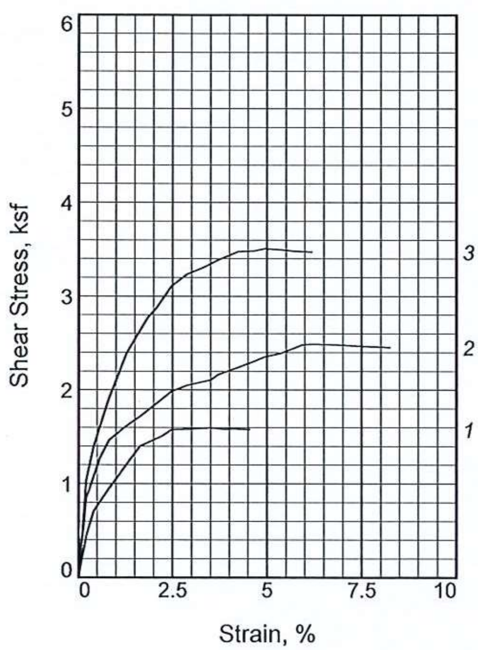
Proj. No.: 21-290 **Date Sampled:**

GS GULF SHORE EXPLORATION AND TESTING

Tested By: MPW **Checked By:** JML







	Sample No.	1	2	3
Initial	Water Content, %	15.8	15.9	15.9
	Dry Density, pcf	111.9	108.6	108.6
	Saturation, %	84.1	77.7	77.7
	Void Ratio	0.5066	0.5522	0.5522
	Diameter, in.	2.43	2.43	2.43
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	17.3	19.9	18.5
	Dry Density, pcf	115.0	109.7	112.4
	Saturation, %	100.0	99.9	100.0
	Void Ratio	0.4659	0.5365	0.4995
	Diameter, in.	2.43	2.43	2.43
	Height, in.	0.97	0.99	0.97
	Normal Stress, ksf	1.00	2.00	4.00
	Fail. Stress, ksf	1.60	2.49	3.51
	Strain, %	3.5	6.2	4.9
	Ult. Stress, ksf			
	Strain, %			
	Strain rate, in./min.	0.005	0.005	0.005

Sample Type: Tube
Description:

Specific Gravity= 2.70
Remarks:

Figure ________

Client: Partner Engineering and Science

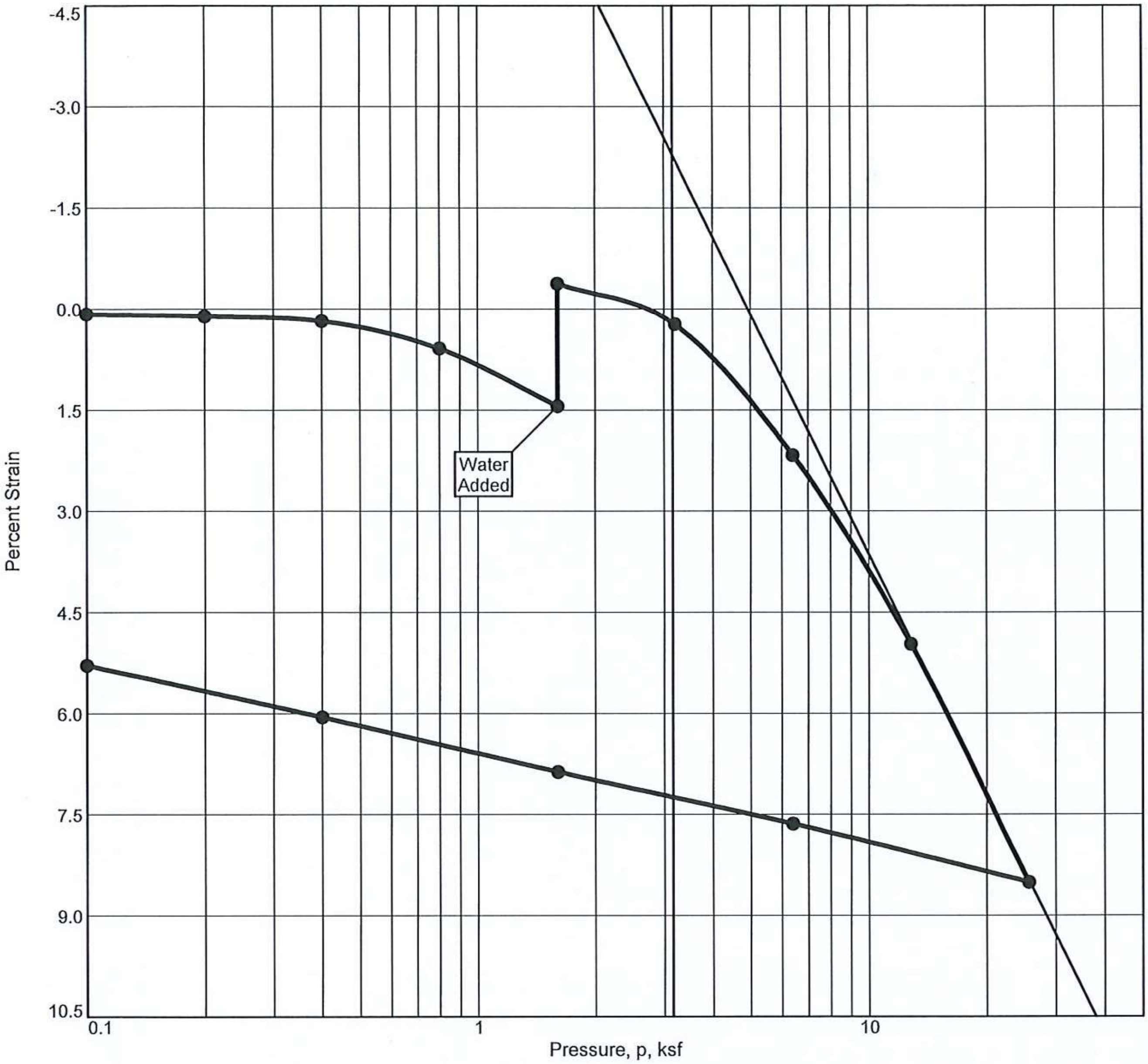
Project: Core Campus Berkeley Development

Location: 2-3 @ 10'
Sample Number: 73021 **Depth:** 10'
Proj. No.: 21-290 **Date Sampled:**

GS GULF SHORE
EXPLORATION AND TESTING

Tested By: MPW **Checked By:** JML

CONSOLIDATION TEST REPORT



SUMMARY OF TEST RESULTS

	DRY DENSITY (pcf)	MOISTURE CONTENT, (%)	SATURATION (%)	VOID RATIO	SPECIFIC GRAVITY	OVERBURDEN (ksf)	P_C (ksf)	C_C	SWELL PRESS. (ksf)
INITIAL	98.9	23.3	89.5	0.704	2.70	0	5.6	0.20	5.1
FINAL		23.2	100.0	0.614					

Location: 1-3 @ 7' **Depth:** 7' **Sample Number:** 73014
Material Description: **USCS:** **AASHTO:**
Remarks:

GS GULF SHORE EXPLORATION AND TESTING

Client: Partner Engineering and Science
Project: Core Campus Berkeley Development
Project No.: 21-290 **Figure**

Tested By: MPW **Checked By:** JML

SWELL/CONSOLIDATION TEST DATA

1/4/2022

Client: Partner Engineering and Science
Project: Core Campus Berkeley Development
Project Number: 21-290
Location: 1-3 @ 7'
Depth: 7'
Sample Number: 73014
Tested by: MPW
Checked by: JML

Test Specimen Data

NATURAL MOISTURE		VOID RATIO		AFTER TEST	
Wet w+t =	506.60 g.	Spec. Gr.	= 2.70	Wet w+t =	506.40 g.
Dry w+t =	478.50 g.	Est. Ht. Solids	= 0.587 in.	Dry w+t =	478.50 g.
Tare Wt. =	358.10 g.	Init. V.R.	= 0.704	Tare Wt. =	358.10 g.
Moisture =	23.3 %	Init. Sat.	= 89.5 %	Moisture =	23.2 %
UNIT WEIGHT		**TEST START**		Dry Wt. =	120.40 g.
Height =	1.000 in.	Height	= 1.000 in.		
Diameter =	2.430 in.	Diameter	= 2.430 in.		
Weight =	148.50 g.				
Dry Dens. =	98.9 pcf				

End-Of-Load Summary

Pressure (ksf)	Final Dial (in.)	Deformation (in.)	C_v (ft.^{2}/day)	C_α	Void Ratio	% Strain
start	0.40000	0.00000			0.704	
0.10	0.39920	0.00080			0.703	0.1 Comprs.
0.20	0.39889	0.00111			0.702	0.1 Comprs.
0.40	0.39817	0.00183			0.701	0.2 Comprs.
0.80	0.39410	0.00590			0.694	0.6 Comprs.
1.60	0.38557	0.01443			0.680	1.4 Comprs.
water	0.40374	-0.00374			0.711	0.4 Swell
3.20	0.39771	0.00229			0.700	0.2 Comprs.
6.40	0.37830	0.02170			0.667	2.2 Comprs.
12.80	0.35029	0.04971		0.005	0.620	5.0 Comprs.
25.60	0.31495	0.08505			0.559	8.5 Comprs.
6.40	0.32360	0.07640			0.574	7.6 Comprs.
1.60	0.33132	0.06868			0.587	6.9 Comprs.
0.40	0.33944	0.06056			0.601	6.1 Comprs.
0.10	0.34712	0.05288			0.614	5.3 Comprs.

Compression index (C_c), ksf = 0.20 **Preconsolidation pressure (P_p), ksf** = 5.6 **Void ratio at P_p (e_m)** = 0.675
Overburden (σ_{vo}), ksf = 0 **Swell index (C_s)** = 0.02
Swell (ε_s), % = 1.8 **Swell pressure, ksf** = 5.1

Pressure: 0.10 ksf — TEST READINGS — Load No. 1

No.	Elapsed Time	Dial Reading
1	0	0.40000
2	(final)	0.39920

Void Ratio = 0.703 **Compression** = 0.1%

Pressure: 0.20 ksf	TEST READINGS	Load No. 2

No.	Elapsed Time	Dial Reading
1	0	0.39920
2	(final)	0.39889

Void Ratio = 0.702 **Compression =** 0.1%

Pressure: 0.40 ksf	TEST READINGS	Load No. 3

No.	Elapsed Time	Dial Reading
1	0	0.39889
2	(final)	0.39817

Void Ratio = 0.701 **Compression =** 0.2%

Pressure: 0.80 ksf	TEST READINGS	Load No. 4

No.	Elapsed Time	Dial Reading
1	0	0.39817
2	(final)	0.39410

Void Ratio = 0.694 **Compression =** 0.6%

Pressure: 1.60 ksf	TEST READINGS	Load No. 5

No.	Elapsed Time	Dial Reading
1	0	0.39410
2	(final)	0.38557

Void Ratio = 0.680 **Compression =** 1.4%

Pressure: 1.60 ksf	TEST READINGS	Load No. 6

No.	Elapsed Time	Dial Reading
1	0	0.38557
2	(final)	0.40374

Void Ratio = 0.711 **Swell =** 0.4%

Pressure: 3.20 ksf	TEST READINGS	Load No. 7

No.	Elapsed Time	Dial Reading
1	0	0.40374
2	(final)	0.39771

Void Ratio = 0.700 **Compression =** 0.2%

Pressure: 6.40 ksf	TEST READINGS	Load No. 8

No.	Elapsed Time	Dial Reading
1	0	0.39771
2	(final)	0.37830

Void Ratio = 0.667 **Compression =** 2.2%

Pressure: 12.80 ksf		TEST READINGS			Load No. 9
No.	Elapsed Time	Dial Reading	No.	Elapsed Time	Dial Reading
1	0	0.37830	11	60	0.35549
2	.1	0.37402	12	120	0.35360
3	.25	0.37113	13	240	0.35272
4	.5	0.36980	14	1440	0.35029
5	1	0.36771			
6	2	0.36605			
7	4	0.36402			
8	8	0.36221			
9	15	0.36020			
10	30	0.35778			

Void Ratio = 0.620 **Compression** = 5.0%

Pressure: 25.60 ksf — TEST READINGS — Load No. 10

No.	Elapsed Time	Dial Reading
1	0	0.35029
2	(final)	0.31495

Void Ratio = 0.559 **Compression** = 8.5%

Pressure: 6.40 ksf — TEST READINGS — Load No. 11

No.	Elapsed Time	Dial Reading
1	0	0.31495
2	(final)	0.32360

Void Ratio = 0.574 **Compression** = 7.6%

Pressure: 1.60 ksf — TEST READINGS — Load No. 12

No.	Elapsed Time	Dial Reading
1	0	0.32360
2	(final)	0.33132

Void Ratio = 0.587 **Compression** = 6.9%

Pressure: 0.40 ksf — TEST READINGS — Load No. 13

No.	Elapsed Time	Dial Reading
1	0	0.33132
2	(final)	0.33944

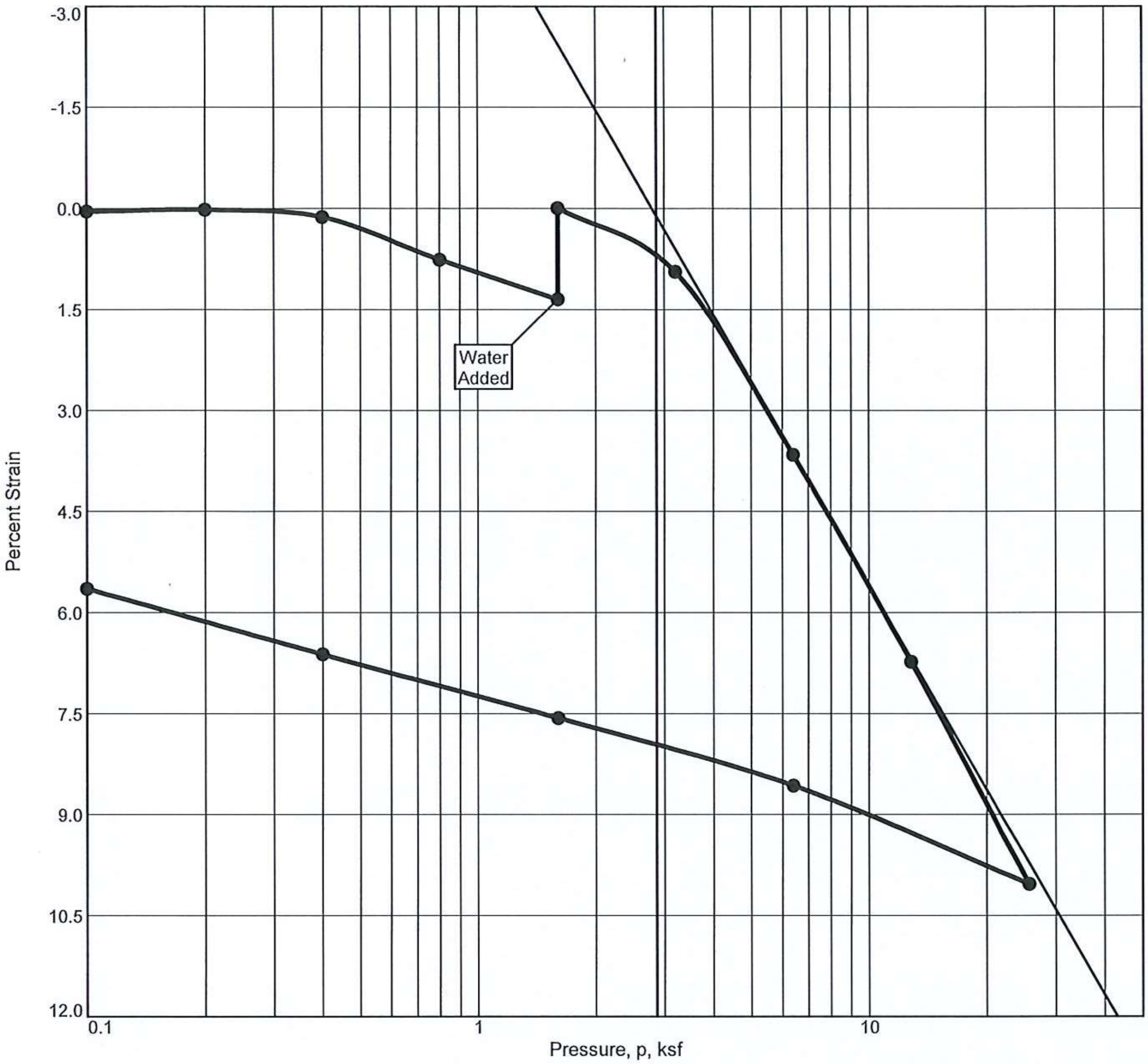
Void Ratio = 0.601 **Compression** = 6.1%

Pressure: 0.10 ksf — TEST READINGS — Load No. 14

No.	Elapsed Time	Dial Reading
1	0	0.33944
2	(final)	0.34712

Void Ratio = 0.614 **Compression** = 5.3%

CONSOLIDATION TEST REPORT



SUMMARY OF TEST RESULTS

	DRY DENSITY (pcf)	MOISTURE CONTENT, (%)	SATURATION (%)	VOID RATIO	SPECIFIC GRAVITY	OVERBURDEN (ksf)	P_C (ksf)	C_C	SWELL PRESS. (ksf)
INITIAL	101.1	18.5	75.0	0.667	2.70	0	3.4	0.17	3.7
FINAL		21.3	100.0	0.573					

Location: 2-1 @ 4' **Depth:** 4' **Sample Number:** 73020
Material Description: **USCS:** **AASHTO:**
Remarks:

GULF SHORE EXPLORATION AND TESTING

Client: Partner Engineering and Science
Project: Core Campus Berkeley Development
Project No.: 21-290 **Figure**

Tested By: MPW **Checked By:** JML

SWELL/CONSOLIDATION TEST DATA

1/4/2022

Client: Partner Engineering and Science
Project: Core Campus Berkeley Development
Project Number: 21-290
Location: 2-1 @ 4'
Depth: 4' **Sample Number:** 73020
Tested by: MPW **Checked by:** JML

Test Specimen Data

NATURAL MOISTURE		VOID RATIO		AFTER TEST	
Wet w+t =	455.20 g.	Spec. Gr.	= 2.70	Wet w+t =	458.60 g.
Dry w+t =	432.40 g.	Est. Ht. Solids	= 0.600 in.	Dry w+t =	432.40 g.
Tare Wt. =	309.30 g.	Init. V.R.	= 0.667	Tare Wt. =	309.30 g.
Moisture =	18.5 %	Init. Sat.	= 75.0 %	Moisture =	21.3 %
UNIT WEIGHT		**TEST START**		Dry Wt. =	123.10 g.
Height =	1.000 in.	Height	= 1.000 in.		
Diameter =	2.430 in.	Diameter	= 2.430 in.		
Weight =	145.90 g.				
Dry Dens. =	101.1 pcf				

End-Of-Load Summary

Pressure (ksf)	Final Dial (in.)	Deformation (in.)	C_v (ft.^{2}/day)	C_α	Void Ratio	% Strain
start	0.40000	0.00000			0.667	
0.10	0.39953	0.00047			0.666	0.0 Comprs.
0.20	0.39980	0.00020			0.667	0.0 Comprs.
0.40	0.39872	0.00128			0.665	0.1 Comprs.
0.80	0.39241	0.00759			0.654	0.8 Comprs.
1.60	0.38651	0.01349			0.644	1.3 Comprs.
water	0.40005	-0.00005			0.667	0.0 Swell
3.20	0.39061	0.00939			0.651	0.9 Comprs.
6.40	0.36340	0.03660			0.606	3.7 Comprs.
12.80	0.33268	0.06732		0.002	0.555	6.7 Comprs.
25.60	0.29969	0.10031			0.500	10.0 Comprs.
6.40	0.31429	0.08571			0.524	8.6 Comprs.
1.60	0.32433	0.07567			0.541	7.6 Comprs.
0.40	0.33378	0.06622			0.557	6.6 Comprs.
0.10	0.34351	0.05649			0.573	5.6 Comprs.

Compression index (C_c), ksf = 0.17 **Preconsolidation pressure (P_p), ksf** = 3.4 **Void ratio at P_p (e_m)** = 0.648
Overburden (σ_{VO}), ksf = 0 **Swell index (C_s)** = 0.03
Swell (ε_s), % = 1.4 **Swell pressure, ksf** = 3.7

Pressure: 0.10 ksf — TEST READINGS — Load No. 1

No.	Elapsed Time	Dial Reading
1	0	0.40000
2	(final)	0.39953

Void Ratio = 0.666 **Compression** = 0.0%

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Pressure: 0.20 ksf	TEST READINGS	Load No. 2

No.	Elapsed Time	Dial Reading
1	0	0.39953
2	(final)	0.39980

Void Ratio = 0.667 **Compression** = 0.0%

Pressure: 0.40 ksf	TEST READINGS	Load No. 3

No.	Elapsed Time	Dial Reading
1	0	0.39980
2	(final)	0.39872

Void Ratio = 0.665 **Compression** = 0.1%

Pressure: 0.80 ksf	TEST READINGS	Load No. 4

No.	Elapsed Time	Dial Reading
1	0	0.39872
2	(final)	0.39241

Void Ratio = 0.654 **Compression** = 0.8%

Pressure: 1.60 ksf	TEST READINGS	Load No. 5

No.	Elapsed Time	Dial Reading
1	0	0.39241
2	(final)	0.38651

Void Ratio = 0.644 **Compression** = 1.3%

Pressure: 1.60 ksf	TEST READINGS	Load No. 6

No.	Elapsed Time	Dial Reading
1	0	0.38651
2	(final)	0.40005

Void Ratio = 0.667 **Swell** = 0.0%

Pressure: 3.20 ksf	TEST READINGS	Load No. 7

No.	Elapsed Time	Dial Reading
1	0	0.40005
2	(final)	0.39061

Void Ratio = 0.651 **Compression** = 0.9%

Pressure: 6.40 ksf	TEST READINGS	Load No. 8

No.	Elapsed Time	Dial Reading
1	0	0.39061
2	(final)	0.36340

Void Ratio = 0.606 **Compression** = 3.7%

Pressure: 12.80 ksf — **TEST READINGS** — **Load No. 9**

No.	Elapsed Time	Dial Reading	No.	Elapsed Time	Dial Reading
1	0	0.36340	11	60	0.33610
2	.1	0.35970	12	120	0.33500
3	.25	0.35820	13	240	0.33340
4	.5	0.35695	14	1440	0.33268
5	1	0.35524			
6	2	0.35408			
7	4	0.35241			
8	8	0.34770			
9	15	0.34406			
10	30	0.33850			

Void Ratio = 0.555 **Compression =** 6.7%

Pressure: 25.60 ksf — **TEST READINGS** — **Load No. 10**

No.	Elapsed Time	Dial Reading
1	0	0.33268
2	(final)	0.29969

Void Ratio = 0.500 **Compression =** 10.0%

Pressure: 6.40 ksf — **TEST READINGS** — **Load No. 11**

No.	Elapsed Time	Dial Reading
1	0	0.29969
2	(final)	0.31429

Void Ratio = 0.524 **Compression =** 8.6%

Pressure: 1.60 ksf — **TEST READINGS** — **Load No. 12**

No.	Elapsed Time	Dial Reading
1	0	0.31429
2	(final)	0.32433

Void Ratio = 0.541 **Compression =** 7.6%

Pressure: 0.40 ksf — **TEST READINGS** — **Load No. 13**

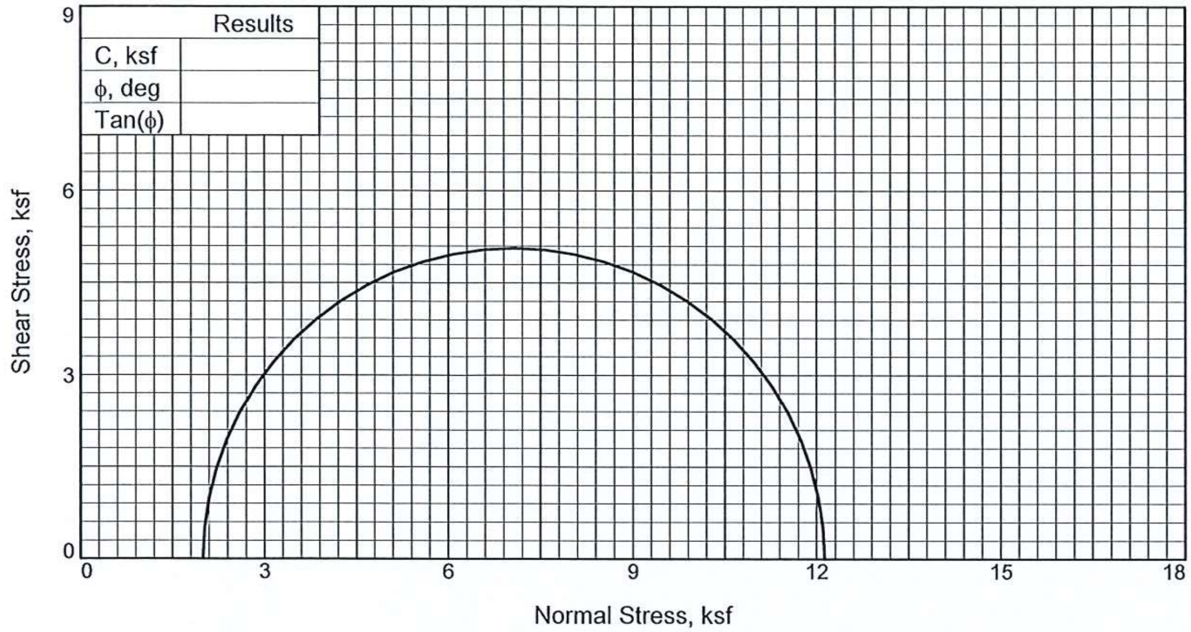
No.	Elapsed Time	Dial Reading
1	0	0.32433
2	(final)	0.33378

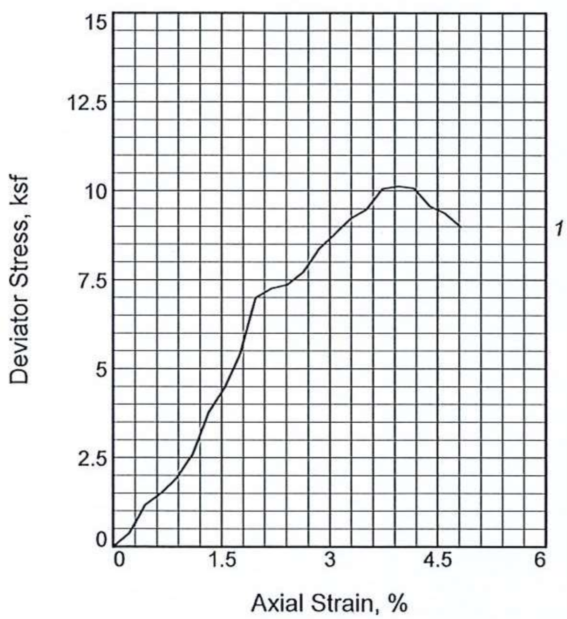
Void Ratio = 0.557 **Compression =** 6.6%

Pressure: 0.10 ksf — **TEST READINGS** — **Load No. 14**

No.	Elapsed Time	Dial Reading
1	0	0.33378
2	(final)	0.34351

Void Ratio = 0.573 **Compression =** 5.6%





Sample No.		1
Initial	Water Content, %	16.3
	Dry Density, pcf	114.6
	Saturation, %	93.6
	Void Ratio	0.4712
	Diameter, in.	2.41
	Height, in.	4.56
At Test	Water Content, %	16.1
	Dry Density, pcf	114.6
	Saturation, %	92.3
	Void Ratio	0.4712
	Diameter, in.	2.41
	Height, in.	4.56
Strain rate, in./min.		0.005
Back Pressure, psi		0.00
Cell Pressure, psi		13.89
Fail. Stress, ksf		10.1
Ult. Stress, ksf		
σ_1 Failure, ksf		12.1
σ_3 Failure, ksf		2.0

Type of Test:
Unconsolidated Undrained

Sample Type: Tube

Description:

Specific Gravity= 2.70

Remarks: Small gravel & voids throughout lg rock (full diameter of tube) at one end

Figure ________

Client: Partner Engineering and Science

Project: Core Campus Berkeley Development

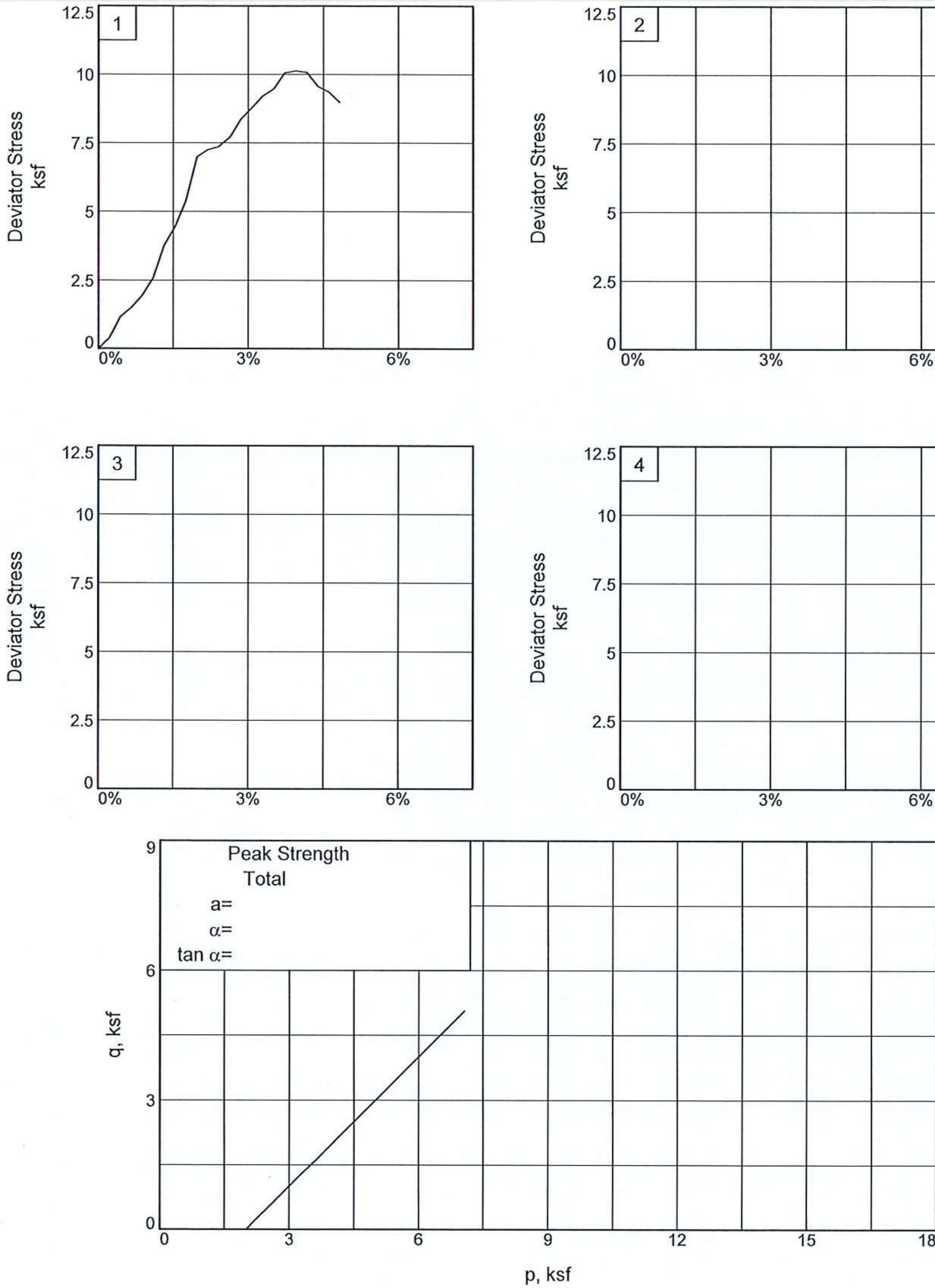
Location: 2-11 @ 76'

Sample Number: 73022 **Depth:** 76'

Proj. No.: 21-290 **Date Sampled:**

GS GULF SHORE EXPLORATION AND TESTING

Tested By: MPW **Checked By:** JML



Client: Partner Engineering and Science
Project: Core Campus Berkeley Development
Location: 2-11 @ 76' **Depth:** 76' **Sample Number:** 73022
Project No.: 21-290 **Figure** _______

Gulf Shore Construction Services, LLC

Tested By: MPW **Checked By:** JML

APPENDIX C

General Geotechnical Design and Construction Considerations

Subgrade Preparation

Earthwork – Structural Fill/Excavations

Underground Pipeline Installation – Structural Backfill

Cast-in-Place Concrete

Foundations

Laterally Loaded Structures

Excavations and Dewatering

Waterproofing and Drainage

Chemical Treatment of Soils

Paving

Site Grading and Drainage

PARTNER

SUBGRADE PREPARATION

1. In general, construction should proceed per the project specifications and contract documents, as well as governing jurisdictional guidelines for the project site, including but not limited to the applicable State Department of Transportation, City and/or County, Army Corps of Engineers, Federal Aviation, Occupational Safety and Health Administration (OSHA), and any other governing standard details and specifications. In areas where multiple standards are applicable the more stringent should be considered. Work should be performed by qualified, licensed contractors with experience in the specific type of work in the area of the site.
2. Subgrade preparation in this section is considered to apply to the initial modifications to existing site conditions to prepare for new planned construction.
3. Prior to the start of subgrade preparation, a detailed conflict study including as-builts, utility locating, and potholing should be conducted. Existing features that are to be demolished should also be identified and the geotechnical study should be referenced to determine the need for subgrade preparation, such as over-excavation, scarification and compaction, moisture conditioning, and/or other activities below planned new structural fills, slabs on grade, pavements, foundations, and other structures.
4. The site conflicts, planned demolitions, and subgrade preparation requirements should be discussed in a pre-construction meeting with the pertinent parties, including the geotechnical engineer, inspector, contractors, testing laboratory, surveyor, and others.
5. In the event of preparations that will require work near to existing structures to remain in-place, protection of the existing structures should be considered. This also includes a geotechnical review of excavations near to existing structures and utilities and other concerns discussed in General Geotechnical Design and Construction Considerations, EARTHWORK and UNDERGROUND PIPELINE INSTALLATION.
6. Features to be demolished should be completely removed and disposed of per jurisdictional requirements and/or other conditions set forth as a part of the project. Resulting excavations or voids should be backfilled per the recommendations in the General Geotechnical Design and Construction Considerations, EARTHWORK section.
7. Vegetation, roots, soils containing organic materials, debris and/or other deleterious materials on the site should be removed from structural areas and should be disposed of as above. Replacement of such materials should be in accordance with the recommendations in the General Geotechnical Design and Construction Considerations, EARTHWORK section
8. Subgrade preparation required by the geotechnical report may also call for as over-excavation, scarification and compaction, moisture conditioning, and/or other activities below planned structural fills, slabs on grade, pavements, foundations, and other structures. These requirements should be provided within the geotechnical report. The execution of this work should be observed by the geotechnical engineering representative or inspector for the site. Testing of the subgrade preparation should be performed per the recommendations in the General Geotechnical Design and Construction Considerations, EARTHWORK section.

9. Subgrade Preparation cannot be completed on frozen ground or on ground that is not at a proper moisture condition. Wet subgrades may be dried under favorable weather if they are disked and/or actively worked during hot, dry, weather, when exposed to wind and sunlight. Frozen ground or wet material can be removed and replaced with suitable material. Dry material can be pre-soaked or can have water added and worked in with appropriate equipment. The soil conditions should be monitored by the geotechnical engineer prior to compaction. Following this type of work, approved subgrades should be protected by direction of surface water, covering, or other methods, otherwise, re-work may be needed.

EARTHWORK – STRUCTURAL FILL

1. In general, construction should proceed per the governing jurisdictional guidelines for the project site, including but not limited to the applicable State Department of Transportation, City and/or County, Army Corps of Engineers, Federal Aviation, Occupational Safety and Health Administration (OSHA), and any other governing standard details and specifications. In areas where multiple standards are applicable the more stringent should be considered. Work should be performed by qualified, licensed contractors with experience in the specific type of work in the area of the site.
2. Earthwork in this section is considered to apply to the re-shaping and grading of soil, rock, and aggregate materials for the purpose of supporting man-made structures. Where earthwork is needed to raise the elevation of the site for the purpose of supporting structures or forming slopes, this is referred to as the placement of structural fill. Where lowering of site elevations is needed prior to the installation of new structures, this is referred to as earthwork excavations.
3. Prior to the start of earthwork operations, the geotechnical study should be referenced to determine the need for subgrade preparation, such as over-excavation or scarification and compaction of unsuitable soils below planned structural fills, slabs on grade, pavements, foundations, and other structures. These required preparations should be discussed in a pre-construction meeting with the pertinent parties, including the geotechnical engineer, inspector, contractors, testing laboratory, surveyor, and others. The preparations should be observed by the inspector or geotechnical engineer representative, and following such subgrade preparation, the geotechnical engineer should observe the prepared subgrade to approve it for the placement of earthwork fills or new structures.
4. Structural fill materials should be relatively free of organic materials, man-made debris, environmentally hazardous materials, and brittle, non-durable aggregate, frozen soil, soil clods or rocks and/or any other materials that can break down and degrade over time.
5. In deeper structural fill zones, expansive soils (greater than 1.5 percent swell at 100 pounds per square foot surcharge) and rock fills (fills containing particles larger than 4 inches and/or containing more than 35 percent gravel larger than ¾-inch diameter or more than 50 percent gravel) may be used with the approval and guidance of the geotechnical report or geotechnical engineer. This may require the placement of geotextiles or other added costs and/or conditions. These conditions may also apply to corrosive soils (less than 2,000 ohm-cm resistivity, more than 50 ppm chloride content, more than 0.1 percent sulfates)
6. For structural fill zones that are closer in depth below planed structures, low expansive materials, and materials with smaller particle size are generally recommended, as directed by the geotechnical report (see criteria above in 5). This may also apply to corrosive soils.
7. For structural fill materials, in general the compaction equipment should be appropriate for the thickness of the loose lift being placed, and the thickness of the loose lift being placed should be at least two times the maximum particle size incorporated in the fill.
8. Fill lift thickness (including bedding) should generally be proportioned to achieve 95 percent or more of a standard proctor (ASTM D689) maximum dry density (MDD) or 90 percent or more of a

modified proctor (ASTM D1557) MDD, depending on the state practices. For subgrades below roadways, the general requirement for soil compaction is usually increased to 100 percent or more of the standard proctor MDD and 95 percent or more of the modified proctor MDD.

9. Soil compaction should be performed at a moisture content generally near optimum moisture content determined by either standard or modified proctor, and ideally within 3 percent below to 1 percent over the optimum for a standard proctor, and from 2 percent below to 2 percent above optimum for a modified proctor.
10. In some instances, fill areas are difficult to access. In such cases a low-strength soil-cement slurry can be used in the place of compacted fill soil. In general, such fills should be rated to have a 28-day strength of 75 to 125 psi, which in some areas is referred to as a "1-sack" slurry. It should be noted that these materials are wet during placement and require a period of 2 days (24 hours) to cure before additional fill can be placed above them. Testing of this material can be done using concrete cylinder compression strength testing equipment, but care is needed in removing the test specimens from the molds. Field testing using the ball method and spread, or flow testing is also acceptable.
11. For fills to be placed on slopes, benching of fill lifts is recommended, which may require cutting into existing slopes to create a bench perpendicular to the slope where soil can be placed in a relatively horizontal orientation. For the construction of slopes, the slopes should be over-built and cut back to grade, as the material in the outer portion of the slope may not be well compacted.
12. For subgrade below roadways, runways, railways or other areas to receive dynamic loading, a proofroll of the finished, compacted subgrade should be performed by the geotechnical engineer or inspector prior to the placement of structural aggregate, asphalt or concrete. Proofrolling consists of observing the performance of the subgrade under heavy-loaded equipment, such as full, 4,000 Gallon water truck, loaded tandem-axel dump truck or similar. Areas that exhibit instability during proofroll should be marked for additional work prior to approval of the subgrade for the next stage of construction.
13. Quality control testing should be provided on earthwork. Proctor testing should be performed on each soil type, and one-point field proctors should be used to verify the soil types during compaction testing. If compaction testing is performed with a nuclear density gauge, it should be periodically correlated with a sand cone test for each soil type. Density testing should be performed per project specifications and or jurisdictional requirements, but not less than once per 12 inches elevation of any fill area, with additional tests per 12-inch fill area for each additional 7,500 square-foot section or portion thereof.
14. For earthwork excavations, OSHA guidelines should be referenced for sloping and shoring. Excavations over a depth of 20 feet require a shoring design. In the event excavations are planned near to existing structures, the geotechnical engineer should be consulted to evaluate whether such excavation will call for shoring or underpinning the adjacent structure. Pre-construction and post-construction condition surveys and vibration monitoring might also be helpful to evaluate any potential damage to surrounding structures.

15. Excavations into rock, partially weathered rock, cemented soils, boulders and cobbles, and other hard soil or "hard-pan" materials, may result in slower excavation rates, larger equipment with specialized digging tools, and even blasting. It is also not unusual in these situations for screening and or crushing of rock to be called for. Blasting, hard excavating, and material processing equipment have special safety concerns and are more costly than the use of soil excavation equipment. Additionally, this type of excavation, especially blasting, is known to cause vibrations that should be monitored at nearby structures. As above, a pre-blast and post-blast conditions assessment might also be warranted.

UNDERGROUND PIPELINE – STRUCTURAL BACKFILL

1. In general, construction should proceed per the governing jurisdictional guidelines for the project site, including but not limited to the applicable State Department of Transportation, the State Department of Environmental Quality, the US Environmental Protection Agency, City and/or County Public Works, Occupational Safety and Health Administration (OSHA), Private Utility Companies, and any other governing standard details and specifications. In areas where multiple standards are applicable the more stringent should be considered, and in some cases, work may take place to multiple different standards. Work should be performed by qualified, licensed contractors with experience in the specific type of work in the area of the site.
2. Underground pipeline in this section is considered to apply to the installation of underground conduits for water, storm water, irrigation water, sewage, electricity, telecommunications, gas, etc. Structural backfill refers to the activity of restoring the grade or establishing a new grade in the area where excavations were needed for the underground pipeline installation.
3. Prior to the start of underground pipeline installation, a detailed conflict study including as-builts, utility locating, and potholing should be conducted. The geotechnical study should be referenced to determine subsurface conditions such as caving soils, unsuitable soils, shallow groundwater, shallow rock and others. In addition, the utility company responsible for the line also will have requirements for pipe bedding and support as well as other special requirements. Also, if the underground pipeline traverses other properties, rights-of-way, and/or easements etc. (for roads, waterways, dams, railways, other utility corridors, etc.) those owners may have additional requirements for construction.
4. The required preparations above should be discussed in a pre-construction meeting with the pertinent parties, including the geotechnical engineer, inspector, contractors, testing laboratory, surveyor, and other stake holders.
5. For pipeline excavations, OSHA guidelines should be referenced for sloping and shoring. Excavations over a depth of 20 feet require a shoring design. In the event excavations are planned near to existing structures or pipelines, the geotechnical engineer should be consulted to evaluate whether such excavation will call for shoring or supporting the adjacent structure or pipeline. A pre-construction and post-construction condition survey and vibration monitoring might also be helpful to evaluate any potential damage to surrounding structures.
6. Excavations into rock, partially weathered rock, cemented soils, boulders and cobbles, and other hard soil or "hard-pan" materials, may result in slower excavation rates, larger equipment with specialized digging tools, and even blasting. It is also not unusual in these situations for screening and or crushing of rock to be called for. Blasting, hard excavating and material processing equipment have special safety concerns and are more costly than the use soil excavation equipment. Additionally, this type of excavation, especially blasting, is known to cause vibrations that should be monitored at nearby structures. As above, a pre-blast and post-blast conditions assessment might also be warranted.

7. Bedding material requirements vary between utility companies and might depend of the type of pipe material and availability of different types of aggregates in different locations. In general, bedding refers to the material that supports the bottom of the pipe and extends to 1 foot above the top of the pipe. In general, the use of aggregate base for larger diameter pipes (6-inch diameter or more) is recommended lacking a jurisdictionally specified bedding material. Gas lines and smaller diameter lines are often backfilled with fine aggregate meeting the ASTM requirements for concrete sand. In all cases bedding with less than 2,000 ohm-cm resistivity, more than 50 ppm chloride content or more than 0.1 percent sulfates should not be used.
8. Structural backfill materials above the bedding should be relatively free of organic materials, man-made debris, environmentally hazardous materials, frozen material, and brittle, non-durable aggregate, soil clods or rocks and/or any other materials that can break down and degrade over time.
9. In general, the backfill soil requirements will depend on the future use of the land above the buried line, but in most cases, excessive settlement of the pipe trench is not considered advisable or acceptable. As such, the structural backfill compaction equipment should be appropriate for the thickness of the loose lift being placed. The thickness of the loose lift being placed should be at least two times the maximum particle size incorporated in the fill. Care should be taken not to damage the pipe during compaction or compaction testing.
10. Fill lift thickness (including bedding) should generally be proportioned to achieve 95 percent or more of a standard proctor (ASTM D689) maximum dry density (MDD) or 90 percent or more of a modified proctor (ASTM D1557) MDD, depending on the state practices (in general the modified proctor is required in California and for projects in the jurisdiction of the Army Corps of Engineers). For backfills within the upper portions of roadway subgrades, the general requirement for soil compaction is usually increased to 100 percent or more of the standard proctor MDD and 95 percent or more of the modified proctor MDD.
11. Soil compaction should be performed at a moisture content generally near optimum moisture content determined by either standard or modified proctor, and ideally within 3 percent below to 1 percent over the optimum for a standard proctor, and from 2 percent below to 2 percent above optimum for a modified proctor.
12. In some instances, fill areas are difficult to access. In such cases a low-strength soil-cement slurry can be used in the place of compacted fill soil. In general, such fills should be rated to have a 28-day strength of 75 to 125 psi, which in some areas is referred to as a "1-sack" slurry. It should be noted that these materials are wet and require a period of 2 days (24 hours) to cure before additional fill can be placed above it. Testing of this material can be done using concrete cylinder compression strength testing equipment, but care is needed in removing the test specimens from the molds. Field testing using the ball method and spread, or flow testing is also acceptable.
13. Quality control testing should be provided on structural backfill to assist the contractor in meeting project specifications. Proctor testing should be performed on each soil type, and one-point field proctors should be used to verify the soil types during compaction testing. If compaction testing is

performed with a nuclear density gauge, it should be periodically correlated with a sand cone test for each soil type.

14. Density testing should be performed on structural backfill per project specifications and or jurisdictional requirements, but not less than once per 12 inches elevation in each area, and additional tests for each additional 500 linear-foot section or portion thereof.

CAST-IN-PLACE CONCRETE

SLABS-ON-GRADE/STRUCTURES/PAVEMENTS

1. In general, construction should proceed per the governing jurisdictional guidelines for the project site, including but not limited to the applicable American Concrete Institute (ACI), International Code Council (ICC), State Department of Transportation, City and/or County, Army Corps of Engineers, Federal Aviation, Occupational Safety and Health Administration (OSHA), and any other governing standard details and specifications. In areas where multiple standards are applicable the more stringent should be considered. Work should be performed by qualified, licensed contractors with experience in the specific type of work in the area of the site.
2. Cast-in-place concrete (concrete) in this section is considered to apply to the installation of cast-in-place concrete slabs on grade, including reinforced and non-reinforced slabs, structures, and pavements.
3. In areas where concrete is bearing on prepared subgrade or structural fill soils, testing and approval of this work should be completed prior to the beginning of concrete construction.
4. In locations where a concrete is approved to bear on in-place (native) soil or in locations where approved documented fills have been exposed to weather conditions after approval, a concrete subgrade evaluation should be performed prior to the placement of reinforcing steel and or concrete. This can consist of probing with a "t"-handled rod, borings, penetrometer testing, dynamic cone penetration testing and/or other methods requested by the geotechnical engineer and/or inspector. Where unsuitable, wet, or frozen bearing material is encountered, the geotechnical engineer should be consulted for additional recommendations.
5. Slabs on grade should be placed on a 4-inch thick or more capillary barrier consisting of non-corrosive (more than 2,000 ohm-cm resistivity, less than 50 ppm chloride content and less than 0.1 percent sulfates) aggregate base or open-graded aggregate material. This material should be compacted or consolidated per the recommendations of the structural engineer or otherwise would be covered by the General Considerations for EARTHWORK.
6. Depending on the site conditions and climate, vapor barriers may be required below in-door grade-slabs to receive flooring. This reduces the opportunity for moisture vapor to accumulate in the slab, which could degrade flooring adhesive and result in mold or other problems. Vapor barriers should be specified by the structural engineer and/or architect. The installation of the barrier should be inspected to evaluate the correct product and thickness is used, and that it has not been damaged or degraded.
7. At times when rainfall is predicted during construction, a mud-mat or a thin concrete layer can be placed on prepared and approved subgrades prior to the placement of reinforcing steel or tendons. This serves the purpose of protecting the subgrades from damage once the reinforcement placement has begun.
8. Prior to the placement of concrete, exposed subgrade or base material and forms should be wetted, and form release compounds should be applied. Reinforcement support stands or ties should be

checked. Concrete bases or subgrades should not be so wet that they are softened or have standing water.

9. For a cast-in-place concrete, the form dimensions, reinforcement placement and cover, concrete mix design, and other code requirements should be carefully checked by an inspector before and during placement. The reinforcement should be specified by the structural engineering drawings and calculations.
10. For post-tension concrete, an additional check of the tendons is needed, and a tensioning inspection form should be prepared prior to placement of concrete.
11. For Portland cement pavements, forms an additional check of reinforcing dowels should performed per the design drawings.
12. During placement, concrete should be tested, and should meet the ACI and jurisdictional requirements and mix design targets for slump, air entrainment, unit weight, compressive strength, flexural strength (pavements), and any other specified properties. In general concrete should be placed within 90 minutes of batching at a temperature of less than 90 degrees Fahrenheit. Adding of water to the truck on the jobsite is generally not encouraged.
13. Concrete mix designs should be created by the accredited and jurisdictionally approved supplier to meet the requirements of the structural engineer. In general, a water/cement ratio of 0.45 or less is advisable, and aggregates, cement, fly ash, and other constituents should be tested to meet ASTM C-33 standards, including Alkali Silica Reaction (ASR). To further mitigate the possibility of concrete degradation from corrosion and ASR, Type II or V Portland Cement should be used, and fly ash replacement of 25 percent is also recommended. Air entrained concrete should be used in areas where concrete will be exposed to frozen ground or ambient temperatures below freezing.
14. Control joints are recommended to improve the aesthetics of the finished concrete by allowing for cracking within partially cut or grooved joints. The control joints are generally made to depths of about 1/4 of the slab thickness and are generally completed within the first day of construction. The spacing should be laid out by the structural engineer and is often in a square pattern. Joint spacing is generally 5 to 15 feet on-center but this can vary and should be decided by the structural engineer. For pavements, construction joints are generally considered to function as control joints. Post-tensioned slabs generally do not have control joints.
15. Some slabs are expected to meet flatness and levelness requirements. In those cases, testing for flatness and levelness should be completed as soon as possible, usually the same day as concrete placement, and before cutting of control joints if possible. Roadway smoothness can also be measured and is usually specified by the jurisdictional owner if is required.
16. Prior to tensioning of post-tension structures, placement of soil backfills or continuation of building on newly placed concrete, a strength requirement is generally required, which should be specified by the structural engineer. The strength progress can be evaluated by the use of concrete compressive strength cylinders or maturity monitoring in some jurisdictions. Advancing with backfill, additional concrete work or post-tensioning without reaching strength benchmarks could result in damage and failure of the concrete, which could result in danger and harm to nearby people and property.

17. In general, concrete should not be exposed to freezing temperatures in the first 7 days after placement, which may require insulation or heating. Additionally, in hot or dry, windy weather, misting, covering with wet burlap or the use of curing compounds may be called for to reduce shrinkage cracking and curling during the first 7 days.

FOUNDATIONS

1. In general, construction should proceed per the governing jurisdictional guidelines for the project site, including but not limited to the applicable American Concrete Institute (ACI), International Code Council (ICC), State Department of Transportation, City and/or County, Army Corps of Engineers, Federal Aviation, Occupational Safety and Health Administration (OSHA), and any other governing standard details and specifications. In areas where multiple standards are applicable the more stringent should be considered. Work should be performed by qualified, licensed contractors with experience in the specific type of work in the area of the site.
2. Foundations in this section are considered to apply to the construction of structural supports which directly transfer loads from man-made structures into the earth. In general, these include shallow foundations and deep foundations. Shallow foundations are generally constructed for the purpose of distributing the structural loads horizontally over a larger area of earth. Some types of shallow foundations (or footings) are spread footings, continuous footings, mat foundations, and reinforced slabs-on-grade. Deep foundations are generally designed for the purpose of distributing the structural loads vertically deeper into the soil by the use of end bearing and side friction. Some types of deep foundations are driven piles, auger-cast piles, drilled shafts, caissons, helical piers, and micro-piles.
3. For shallow foundations, the minimum bearing depth considered should be greater than the maximum design frost depth for the location of construction. This can be found on frost depth maps (ICC), but the standard of practice in the city and/or county should also be consulted. In general, the bearing depth should never be less than 18 inches below planned finished grades.
4. Shallow continuous foundations should be sized with a minimum width of 18 inches and isolated spread footings should be a minimum of 24 inches in each direction. Foundation sizing, spacing, and reinforcing steel design should be performed by a qualified structural engineer.
5. The geotechnical engineer will provide an estimated bearing capacity and settlement values for the project based on soil conditions and estimated loads provided by the structural engineer. It is assumed that appropriate safety factors will be applied by the structural engineer.
6. In areas where shallow foundations are bearing on prepared subgrade or structural fill soils, testing and approval of this work should be completed prior to the beginning of foundation construction.
7. In locations where the shallow foundations are approved to bear on in-place (native) soil or in locations where approved documented fills have been exposed to weather conditions after approval, a foundation subgrade evaluation should be performed prior to the placement of reinforcing steel. This can consist of probing with a "t"-handled rod, borings, penetrometer testing, dynamic cone penetration testing and/or other methods requested by the geotechnical engineer and/or inspector. Where unsuitable foundation bearing material is encountered, the geotechnical engineer should be consulted for additional recommendations.
8. For shallow foundations to bear on rock, partially weathered rock, hard cemented soils, and/or boulders, the entire foundation system should bear directly on such material. In this case, the rock surface should be prepared so that it is clean, competent, and formed into a roughly horizontal,

stepped base. If that is not possible, then the entire structure should be underlain by a zone of structural fill. This may require the over-excavation in areas of rock removal and/or hard dig. In general, this zone can vary in thickness, but it should be a minimum of 1 foot thick. The geotechnical engineer should be consulted in this instance.

9. At times when rainfall is predicted during construction, a mud-mat or a thin concrete layer can be placed on prepared and approved subgrades prior to the placement of reinforcing steel. This serves the purpose of protecting the subgrades from damage once the reinforcing steel placement has begun.
10. For cast-in-place concrete foundations, the excavations dimensions, reinforcing steel placement and cover, structural fill compaction, concrete mix design, and other code requirements should be carefully checked by an inspector before and during placement.

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11. For deep foundations, the geotechnical engineer will generally provide design charts that provide foundations axial capacity and uplift resistance at various depths given certain-sized foundations. These charts may be based on blow count data from drilling and or laboratory testing. In general safety factors are included in these design charts by the geotechnical engineer.
12. In addition, the geotechnical engineer may provide other soil parameters for use in the lateral resistance analysis. These parameters are usually raw data, and safety factors should be provided by the shaft designer. Sometimes, direct shear and or tri-axial testing is performed for this analysis.
13. In general, the spacing of deep foundations is expected to be 6 shaft diameters or more. If that spacing is reduced, a group reduction factor should be applied by the structural engineer to the foundation capacities per FHWA guidelines. The spacing should not be less than 2.5 shaft diameters.
14. For deep foundations, a representative of the geotechnical engineer should be on-site to observe the excavations (if any) to evaluate that the soil conditions are consistent with the findings of the geotechnical report. Soil/rock stratigraphy will vary at times, and this may result in a change in the planned construction. This may require the use of fall protection equipment to perform observations close to an open excavation.
15. For driven foundations, a representative of the geotechnical engineer should be on-site to observe the driving process and to evaluate that the resistance of driving is consistent with the design assumptions. Soil/rock stratigraphy will vary at times and may this may result in a change in the planned construction.
16. For deep foundations, the size, depth, and ground conditions should be verified during construction by the geotechnical engineer and/or inspector responsible. Open excavations should be clean, with any areas of caving and groundwater seepage noted. In areas below the groundwater table, or areas where slurry is used to keep the trench open, non-destructive testing techniques should be used as outlined below.
17. Steel members including structural steel piles, reinforcing steel, bolts, threaded steel rods, etc. should be evaluated for design and code compliance prior to pick-up and placement in the foundation. This includes verification of size, weight, layout, cleanliness, lap-splices, etc. In addition, if non-destructive testing such as crosshole sonic logging or gamma-gamma logging is required,

access tubes should be attached to the steel reinforcement prior to placement, and should be relatively straight, capped at the bottom, and generally kept in-round. These tubes must be filled with water prior to the placement of concrete.

18. In cases where steel welding is required, this should be observed by a certified welding inspector.
19. In many cases, a crane will be used to lower steel members into the deep foundations. Crane picks should be carefully planned, including the ground conditions at placement of outriggers, wind conditions, and other factors. These are not generally provided in the geotechnical report but can usually be provided upon request.
20. Cast-in-place concrete, grout or other cementations materials should be pumped or distributed to the bottom of the excavation using a tremie pipe or hollow stem auger pipe. Depending on the construction type, different mix slumps will be used. This should be carefully checked in the field during placement, and consolidation of the material should be considered. Use of a vibrator may be called for.
21. For work in a wet excavation (slurry), the concrete placed at the bottom of the excavation will displace the slurry as it comes up. The upper layer of concrete that has interacted with the slurry should be removed and not be a part of the final product.
22. Bolts or other connections to be set in the top after the placement is complete should be done immediately after final concrete placement, and prior to the on-set of curing.
23. For shafts requiring crosshole sonic logging or gamma-gamma testing, this should be performed within the first week after placement, but not before a 2-day curing period. The testing company and equipment manufacturer should provide more details on the requirements of the testing.
24. Load testing of deep foundations is recommended, and it is often a project requirement. In some cases, if test piles are constructed and tested, it can result in a significant reduction of the amount of needed foundations. The load testing frame and equipment should be sized appropriately for the test to be performed and should be observed by the geotechnical engineer or inspector as it is performed. The results are provided to the structural engineer for approval.

LATERALLY LOADED STRUCTURES - RETAINING WALLS/SLOPES/DEEP FOUNDATIONS/MISCELLANEOUS

1. In general, construction should proceed per the governing jurisdictional guidelines for the project site, including but not limited to the applicable American Concrete Institute (ACI), International Code Council (ICC), State Department of Transportation, City and/or County, Army Corps of Engineers, Federal Aviation, Occupational Safety and Health Administration (OSHA), and any other governing standard details and specifications. In areas where multiple standards are applicable the more stringent should be considered. Work should be performed by qualified, licensed contractors with experience in the specific type of work in the area of the site.
2. Laterally loaded structures for this section are generally meant to describe structures that are subjected to loading roughly horizontal to the ground surface. Such structures include retaining walls, slopes, deep foundations, tall buildings, box culverts, and other buried or partially buried structures.
3. The recommendations put forth in General Geotechnical Design and Construction Considerations for FOUNDATIONS, CAST-IN-PLACE CONCRETE, EARTHWORK, and SUBGRADE PREPARATION should be reviewed, as they are not all repeated in this section, but many of them will apply to the work. Those recommendations are incorporated by reference herein.
4. Laterally loaded structures are generally affected by overburden pressure, water pressure, surcharges, and other static loads, as well as traffic, seismic, wind, and other dynamic loads. The structural engineer must account for these loads. In addition, eccentric loading of the foundation should be evaluated and accounted for by the structural engineer. The structural engineer is also responsible for applying the appropriate factors of safety to the raw data provided by the geotechnical engineer.
5. The geotechnical report should provide data regarding soil lateral earth pressures, seismic design parameters, and groundwater levels. In the report the pressures are usually reported as raw data in the form of equivalent fluid pressures for three cases. 1. Static is for soil pressure against a structure that is fixed at top and bottom, like a basement wall or box culvert. 2. Active is for soil pressure against a wall that is free to move at the top, like a retaining wall. 3. Passive is for soil that is resisting the movement of the structure, usually at the toe of the wall where the foundation and embedded section are located. The structural engineer is responsible for deciding on safety factors for design parameters and groundwater elevations based on the raw data in the geotechnical report.
6. Generally speaking, direct shear or tri-axial shear testing should be performed for this evaluation in cases of soil slopes or unrestrained soil retaining walls over 6 feet in height or in lower walls in some cases based on the engineer's judgment. For deep foundations and completely buried structures, this testing will be required per the discretion of the structural engineer.
7. For non-confined retaining walls (walls that are not attached at the top) and slopes, a geotechnical engineer should perform overall stability analysis for sliding, overturning, and global stability. For walls that are structurally restrained at the top, the geotechnical engineer does not generally perform this analysis. Internal wall stability should be designed by the structural engineer.

8. Cut slopes into rock should be evaluated by an engineering geologist, and rock coring to identify the orientation of fracture plans, faults, bedding planes, and other features should be performed. An analysis of this data will be provided by the engineering geologist to identify modes of failure including sliding, wedge, and overturning, and to provide design and construction recommendations.
9. For laterally loaded deep foundations that support towers, bridges or other structures with high lateral loads, geotechnical reports generally provide parameters for design analysis which is performed by the structural engineer. The structural engineer is responsible for applying appropriate safety factors to the raw data from the geotechnical engineer.
10. Construction recommendations for deep foundations can be found in the General Geotechnical Design and Construction Considerations-FOUNDATIONS section.
11. Construction of retaining walls often requires temporary slope excavations and shoring, including soil nails, soldier piles and lagging or laid-back slopes. This should be done per OSHA requirements and may require specialty design and contracting.
12. In general, surface water should not be directed over a slope or retaining wall but should be captured in a drainage feature trending parallel to the slope, with an erosion protected outlet to the base of the wall or slope.
13. Waterproofing for retaining walls is generally required on the backfilled side, and they should be backfilled with an 18-inch zone of open graded aggregate wrapped in filter fabric or a synthetic draining product, which outlets to weep holes or a drain at the base of the wall. The purpose of this zone, which is immediately behind the wall is to relieve water pressures from building behind the wall.
14. Backfill compaction around retaining walls and slopes requires special care. Lighter equipment should be considered, and consideration to curing of cementitious materials used during construction will be called for. Additionally, if mechanically stabilized earth walls are being constructed, or if tie-backs are being utilized, additional care will be necessary to avoid damaging or displacing the materials. Use of heavy or large equipment, and/or beginning of backfill prior to concrete strength verification can create dangers to construction and human safety. Please refer to the General Geotechnical Design and Construction Considerations-CAST-IN-PLACE CONCRETE section. These concerns will also apply to the curing of cell grouting within reinforced masonry walls.
15. Usually safety features such as handrails are designed to be installed at the top of retaining walls and slopes. Prior to their installation, workers in those areas will need to be equipped with appropriate fall protection equipment.

EXCAVATION AND DEWATERING

1. In general, construction should proceed per the governing jurisdictional guidelines for the project site, including but not limited to the applicable American Concrete Institute (ACI), International Code Council (ICC), State Department of Transportation, City and/or County, Army Corps of Engineers, Federal Aviation, Occupational Safety and Health Administration (OSHA), and any other governing standard details and specifications. In areas where multiple standards are applicable the more stringent should be considered. Work should be performed by qualified, licensed contractors with experience in the specific type of work in the area of the site.
2. Excavation and Dewatering for this section are generally meant to describe structures that are intended to create stable, excavations for the construction of infrastructure near to existing development and below the groundwater table.
3. The recommendations put forth in General Geotechnical Design and Construction Considerations for LATERALLY LOADED STRUCTURES, FOUNDATIONS, CAST-IN-PLACE CONCRETE, EARTHWORK, and SUBGRADE PREPARATION should be reviewed, as they are not all repeated in this section, but many of them will apply to the work. Those recommendations are incorporated by reference herein.
4. The site excavations will generally be affected by overburden pressure, water pressure, surcharges, and other static loads, as well as traffic, seismic, wind, and other dynamic loads. The structural engineer must account for these loads as described in Section 5.2 of this report. In addition, eccentric loading of the foundation should be evaluated and accounted for by the structural engineer. The structural engineer is also responsible for applying the appropriate factors of safety to the raw data provided by the geotechnical engineer.
5. The geotechnical report should provide data regarding soil lateral earth pressures, seismic design parameters, and groundwater levels. In the report the pressures are usually reported as raw data in the form of equivalent fluid pressures for three cases. 1. Static is for soil pressure against a structure that is fixed at top and bottom, like a basement wall or box culvert. 2. Active is for soil pressure against a wall that is free to move at the top, like a retaining wall. 3. Passive is for soil that is resisting the movement of the structure, usually at the toe of the wall where the foundation and embedded section are located. The structural engineer is responsible for deciding on safety factors for design parameters and groundwater elevations based on the raw data in the geotechnical report.
6. The parameters provided above are based on laboratory testing and engineering judgement. Since numerous soil layers with different properties will be encountered in a large excavation, assumptions and judgement are used to generate the equivalent fluid pressures to be used in design. Factors of safety are not included in those numbers and should be evaluated prior to design.
7. Groundwater, if encountered will dramatically change the stability of the excavation. In addition, pumping of groundwater from the bottom of the excavation can be difficult and costly, and it can result in potential damage to nearby structures if groundwater drawdown occurs. As such, we recommend that groundwater monitoring be performed across the site during design and prior to construction to assist in the excavation design and planning.

8. Groundwater pumping tests should be performed if groundwater pumping will be needed during construction. The pumping tests can be used to estimate drawdown at nearby properties, and also will be needed to determine the hydraulic conductivity of the soil for the design of the dewatering system.
9. For excavation stabilization in granular and dense soil, the use of soldier piles and lagging is recommended. The soldier pile spacing, and size should be determined by the structural engineer based on the lateral loads provided in the report. In general, the spacing should be more than two pile diameters, and less than 8 feet. Soldier piles should be advanced 5 feet or more below the base of the excavation. Passive pressures from Section 5.2 can be used in the design of soldier piles for the portions of the piles below the excavation.
10. If the piles are drilled, they should be grouted in-place. If below the groundwater table, the grouting should be accomplished by tremie pipe, and the concrete should be a mix intended for placement below the groundwater table. For work in a wet excavation, the concrete placed at the bottom of the excavation will displace the water as it comes up. The upper layer of concrete that has interacted with the water should be removed and not be a part of the final product. Lagging should be specially designed timber or other lagging. The temporary excavation will need to account for seepage pressures at the toe of the wall as well as hydrostatic forces behind the wall.
11. Depending on the loading, tie back anchors and/or soil nails may be needed. These should be installed beyond the failure envelope of the wall. This would be a plane that is rotated upward 55 degrees from horizontal. The strength of the anchors behind this plane should be considered, and bond strength inside the plane should be ignored. If friction anchors are used, they should extend 10 feet or more beyond the failure envelope. Evaluation of the anchor length and encroachment onto other properties, and possible conflicts with underground utilities should be carefully considered. Anchors are typically installed 25 to 40 degrees below horizontal. The capacity of the anchors should be checked on 10% of locations by loading to 200% of the design strength. All should be loaded to 120% of design strength, and should be locked off at 80%
12. The shoring and tie backs should be designed to allow less than ½ inch of deflection at the top of the excavation wall, where the wall is within an imaginary 1:1 line extending downward from the base of surrounding structures. This can be expanded to 1 inch of deflection if there is no nearby structure inside that plane. An analysis of nearby structures to locate their depth and horizontal position should be conducted prior to shored excavation design.
13. Assuming that the excavations will encroach below the groundwater table, allowances for drainage behind and through the lagging should be made. The drainage can be accomplished by using an open-graded gravel material that is wrapped in geotextile fabric. The lagging should allow for the collected water to pass through the wall at select locations into drainage trenches below the excavation base. These trenches should be considered as sump areas where groundwater can be pumped out of the excavation.
14. The pumped groundwater needs to be handled properly per jurisdictional guidelines.

15. In general, surface water should not be directed over a slope or retaining wall but should be captured in a drainage feature trending parallel to the slope, with an erosion protected outlet to the base of the wall or slope.
16. Safety features such as handrails or barriers are to be designed to be installed at the top of retaining walls and slopes. Prior to their installation, workers in those areas will need to be equipped with appropriate fall protection equipment.

Waterproofing and Back Drainage

1. In general, construction should proceed per the governing jurisdictional guidelines for the project site, including but not limited to the applicable American Concrete Institute (ACI), International Code Council (ICC), State Department of Transportation, City and/or County, Army Corps of Engineers, Federal Aviation, Occupational Safety and Health Administration (OSHA), and any other governing standard details and specifications. In areas where multiple standards are applicable the more stringent should be considered. Work should be performed by qualified, licensed contractors with experience in the specific type of work in the area of the site.
2. Waterproofing and Back drainage structures for this section are generally meant to describe permanent subgrade structures that are planned to be below the historic high groundwater elevation of 20 feet below existing grades.
3. The recommendations put forth in General Geotechnical Design and Construction Considerations for FOUNDATIONS, CAST-IN-PLACE CONCRETE, EARTHWORK, and SUBGRADE PREPARATION should be reviewed, as they are not all repeated in this section, but many of them will apply to the work. Those recommendations are incorporated by reference herein.
4. In general, surface water should not be directed over a slope or retaining wall but should be captured in a drainage feature trending parallel to the slope, with an erosion protected outlet to the base of the wall or slope.
5. Waterproofing for retaining walls is generally required on the backfilled side, and they should be backfilled with an 18-inch zone of open graded aggregate wrapped in filter fabric or a synthetic draining product, which outlets to weep holes or a drain at the base of the wall. The purpose of this zone, which is immediately behind the wall is to relieve water pressures from building behind the wall.
6. For the basement walls on this site, sump pumps will be needed to reduce the build-up of water in the basement. The design should be for a historic high groundwater level of 20 feet bgs. The pumping system should be designed to keep the slab and walls relatively dry so that mold, efflorescence, and other detrimental effects to the concrete structure will not result.
7. Backfill compaction around retaining walls and slopes requires special care. Lighter equipment should be considered, and consideration to curing of cementitious materials used during construction will be called for. Additionally, if mechanically stabilized earth walls are being constructed, or if tie-backs are being utilized, additional care will be necessary to avoid damaging or displacing the materials. Use of heavy or large equipment, and/or beginning of backfill prior to concrete strength verification can create dangers to construction and human safety. Please refer to the General Geotechnical Design and Construction Considerations-CAST-IN-PLACE CONCRETE section. These concerns will also apply to the curing of cell grouting within reinforced masonry walls.

CHEMICAL TREATMENT OF SOIL

1. In general, construction should proceed per the governing jurisdictional guidelines for the project site, including but not limited to the applicable American Concrete Institute (ACI), International Code Council (ICC), State Department of Transportation, State Department of Environmental Quality, the US Environmental Protection Agency, City and/or County, Army Corps of Engineers, Federal Aviation, Occupational Safety and Health Administration (OSHA), and any other governing standard details and specifications. In areas where multiple standards are applicable the more stringent should be considered. Work should be performed by qualified, licensed contractors with experience in the specific type of work in the area of the site.
2. Chemical treatment of soil for this section is generally meant to describe the process of improving soil properties for a specific purpose, using cement or chemical lime.
3. A mix design should be performed by the geotechnical engineer to help it meet the specific strength, plasticity index, durability, and/or other desired properties. The mix design should be performed using the proposed chemical lime or cement proposed for use by the contractor, along with samples of the site soil that are taken from the material to be used in the process.
4. For the mix design the geotechnical engineer should perform proctor testing to determine optimum moisture content of the soil, and then mix samples of the soil at 3 percent above optimum moisture content with varying concentrations of lime or cement. The samples will be prepared and cured per ASTM standards, and then after 7-days for curing, they will be tested for compression strength. Durability testing goes on for 28 days.
5. Following this testing, the geotechnical engineer will provide a recommended mix ratio of cement or chemical lime in the geotechnical report for use by the contractor. The geotechnical engineer will generally specify a design ratio of 2 percent more than the minimum to account for some error during construction.
6. Prior to treatment, the in-place soil moisture should be measured so that the correct amount of water can be used during construction. Work should not be performed on frozen ground.
7. During construction, special considerations for construction of treated soils should be followed. The application process should be conducted to prevent the loss of the treatment material to wind which might transport the materials off site, and workers should be provided with personal protective equipment for dust generated in the process.
8. The treatment should be applied evenly over the surface, and this can be monitored by use of a pan placed on the subgrade. This can also be tested by preparing test specimens from the in-place mixture for laboratory testing.
9. Often, after or during the chemical application, additional water may be needed to activate the chemical reaction. In general, it should be maintained at about 3 percent or more above optimum moisture. Following this, mixing of the applied material is generally performed using specialized equipment.

10. The total amount of chemical provided can be verified by collecting batch tickets from the delivery trucks, and the depth of the treatment can be verified by digging of test pits, and the use of reagents that react with lime and or cement.
11. For the use of lime treatment, compaction should be performed after a specified amount of time has passed following mixing and re-grading. For concrete, compaction should be performed immediately after mixing and re-grading. In both cases, some swelling of the surface should be expected. Final grading should be performed the following day of the initial work for lime treatment, and within 2 to 4 hours for soil cement.
12. Quality control testing of compacted treated subgrades should be performed per the recommendations of the geotechnical report, and generally in accordance with General Geotechnical Design and Construction Considerations - EARTHWORK

PAVING

1. In general, construction should proceed per the governing jurisdictional guidelines for the project site, including but not limited to the applicable American Concrete Institute (ACI), International Code Council (ICC), State Department of Transportation, City and/or County, Army Corps of Engineers, Federal Aviation, Occupational Safety and Health Administration (OSHA), and any other governing standard details and specifications. In areas where multiple standards are applicable the more stringent should be considered. Work should be performed by qualified, licensed contractors with experience in the specific type of work in the area of the site.
2. Paving for this section is generally meant to describe the placement of surface treatments on travel-ways to be used by rubber-tired vehicles, such as roadways, runways, parking lots, etc.
3. The geotechnical engineer is generally responsible for providing structural analysis to recommend the thickness of pavement sections, which can include asphalt, concrete pavements, aggregate base, cement or lime treated aggregate base, and cement or lime treated subgrades.
4. The civil engineer is generally responsible for determining which surface finishes and mixes are appropriate, and often the owner, general contractor and/or other party will decide on lift thickness, the use of tack coats and surface treatments, etc.
5. The geotechnical engineer will generally be provided with the planned traffic loading, as well as reliability, design life, and serviceability factors by the jurisdiction, traffic engineer, designer, and/or owner. The geotechnical study will provide data regarding soil resiliency and strength. A pavement modeling software is generally used to perform the analysis for design, however, jurisdictional minimum sections also must be considered, as well as construction considerations and other factors.
6. The geotechnical report will generally provide pavement section thicknesses if requested.
7. For construction of overlays, where new pavement is being placed on old pavement, an evaluation of the existing pavement is needed, which should include coring the pavement, evaluation of the overall condition and thickness of the pavement, and evaluation of the pavement base and subgrade materials.
8. In general, the existing pavement is milled and treated with a tack coat prior to the placement of new pavement for the purpose of creating a stronger bond between the old and new material. This is also a way of removing aged asphalt and helping to maintain finished grades closer to existing conditions grading and drainage considerations.
9. If milling is performed, a minimum of 2 inches of existing asphalt should be left in-place to reduce the likelihood of equipment breaking through the asphalt layer and destroying its integrity. After milling and before the placement of tack coat, the surface should be evaluated for cracking or degradation. Cracked or degraded asphalt should be removed, spanned with geosynthetic reinforcement, or be otherwise repaired per the direction of the civil and or geotechnical engineer prior to continuing construction. Proofrolling may be requested.

10. For pavements to be placed on subgrade or base materials, the subgrade and base materials should be prepared per the General Geotechnical Design and Construction Considerations – EARTHWORK section.
11. Following the proofrolling as described in the General Geotechnical Design and Construction Considerations – EARTHWORK section, the application of subgrade treatment, base material, and paving materials can proceed per the recommendations in the geotechnical report and/or project plans. The placement of pavement materials or structural fills cannot take place on frozen ground.
12. The placement of aggregate base material should conform to the jurisdictional guidelines. In general, the materials should be provided by an accredited supplier, and the material should meet the standards of ASTM C-33. Material that has been stockpiled and exposed to weather including wind and rain should be retested for compliance since fines could be lost. Frozen material cannot be used.
13. The placement of asphalt material should conform to the jurisdictional guidelines. In general, the materials should be provided by an accredited supplier, and the material should meet the standards of ASTM C-33. The material can be placed in a screed by end-dumping, or it can be placed directly on the paving surface. The temperature of the mix at placement should generally be on the order of 300 degrees Fahrenheit at time of placement and screeding.
14. Compaction of the screeded asphalt should begin as soon as practical after placement, and initial rolling should be performed before the asphalt has cooled significantly. Compaction equipment should have vibratory capabilities and should be of appropriate size and weight given the thickness of the lift being placed and the sloping of the ground surface.
15. In cold and/or windy weather, the cooling of the screeded asphalt is a quality issue, so preparations should be made to perform screeding immediately after placement, and compaction immediately after screeding.
16. Quality control testing of the asphalt should be performed during placement to verify compaction and mix design properties are being met and that delivery temperatures are correct. Results of testing data from asphalt laboratory testing should be provided within 24 hours of the paving.

SITE GRADING AND DRAINAGE

1. In general, construction should proceed per the governing jurisdictional guidelines for the project site, including but not limited to the applicable American Concrete Institute (ACI), International Code Council (ICC), State Department of Transportation, State Department of Environmental Quality, the US Environmental Protection Agency, City and/or County, Army Corps of Engineers, Federal Aviation, Occupational Safety and Health Administration (OSHA), and any other governing standard details and specifications. In areas where multiple standards are applicable the more stringent should be considered. Work should be performed by qualified, licensed contractors with experience in the specific type of work in the area of the site.
2. Site grading and drainage for this section is generally meant to describe the effect of new construction on surface hydrology, which impacts the flow of rainfall or other water running across, onto or off-of, a newly constructed or modified development.
3. This section does not apply to the construction of site grading and drainage features. Recommendations for the construction of such features are covered in General Geotechnical Design and Construction Considerations for Earthwork – Structural Fills section and Underground Pipeline Installation – Backfill section.
4. In general, surface water flows should be directed towards storm drains, natural channels, retention or detention basins, swales, and/or other features specifically designed to capture, store, and or transmit them to specific off-site outfalls.
5. The surface water flow design is generally performed by a site civil engineer, and it can be impacted by hydrology, roof lines, and other site structures that do not allow for water to infiltrate into the soil, and that modify the topography of the site.
6. Soil permeability, density, and strength properties are relevant to the design of storm drain systems, including dry wells, retention basins, swales, and others. These properties are usually only provided in a geotechnical report if specifically requested, and recommendations will be provided in the geotechnical report in those cases.
7. Structures or site features that are not a part of the surface water drainage system should not be exposed to surface water flows, standing water or water infiltration. In general, roof drains and scuppers, exterior slabs, pavements, landscaping, etc. should be constructed to drain water away from structures and foundations. The purpose of this is to reduce the opportunity for water damage, erosion, and/or altering of structural soil properties by wetting. In general, a 5 percent or more slope away from foundations, structural fills, slopes, structures, etc. should be maintained.
8. Special considerations should be used for slopes and retaining walls, as described in the General Geotechnical Design and Construction Considerations - LATERALLY LOADED STRUCTURES section.
9. Additionally, landscaping features including irrigation emitters and plants that require large amounts of water should not be placed near to new structures, as they have the potential to alter soil moisture states. Changing of the moisture state of soil that provides structural support can lead to damage to the supported structures.

APPENDIX D

Geophysical Evaluation

PARTNER

REFRACTION MICROTREMOR/1-D MASW STUDY

2128 OXFORD STREET

Berkeley, California

PREPARED FOR:

Andrew Atry P.E.
Partner Engineering and Science Inc.
2154 Torrance Boulevard
Torrance, California 90501

PREPARED BY:

Atlas Technical Consultants LLC
6280 Riverdale Street
San Diego, California 92120

November 17, 2021

6280 Riverdale Street
San Diego, CA 92120
(877) 215-4321 | oneatlas.com

November 17, 2021

Atlas No. 121425SWGrev
Report No. 1

MR. ANDREW ATRY, P.E.
PARTNER ENGINEERING AND SCIENCE, INC.
2154 TORRANCE BOULEVARD
TORRANCE, CALIFORNIA 90501

Subject: Refraction Microtremor/1-D MASW Study
2128 Oxford Street
Berkeley, California

Dear Mr. Atry:

In accordance with your authorization, Atlas Technical Consultants has performed a geophysical evaluation pertaining to the 2128 Oxford Street Refraction Microtremor/1-D MASW Study in Berkeley, California. The purpose of our study was to develop one-dimensional (1-D) shear-wave velocity profiles to be used for design and construction at the project site. Our services were conducted on November 3rd, 2021. This report presents the survey methodology, equipment used, analysis, and findings from our study.

If you have any questions, please call us at (858) 527-0849.

Respectfully submitted,
Atlas Technical Consultants LLC

REGISTERED PROFESSIONAL GEOPHYSICIST
PATRICK F. LEHRMANN
No. 1043
Exp. 1/31/2022
STATE OF CALIFORNIA

Paul W. Gresoro
Staff Geophysicist

Patrick F. Lehrmann, P.G., P.Gp.
Principal Geologist/Geophysicist

PN:PWG:PFL:ds

Distribution: aatry@partneresi.com

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1. INTRODUCTION

In accordance with your authorization, Atlas Technical Consultants has performed a geophysical evaluation pertaining to the 2128 Oxford Street Refraction Microtremor/1-D MASW Study in Berkeley, California (Figure 1). The purpose of our study was to develop one-dimensional (1-D) shear-wave velocity profiles to be used for design and construction at the project site. Our services were conducted on November 3rd, 2021. This report presents the survey methodology, equipment used, analysis, and findings from our study.

2. SCOPE OF SERVICES

Our scope of services included:

- Performance of one Refraction Microtremor (ReMi) profile and one 1-D Multichannel Analysis of Surface Waves (MASW) profile at the study site.
- Compilation and analysis of the data collected.
- Preparation of this data report presenting our results and conclusions.

3. SITE AND PROJECT DESCRIPTION

The project site is located at 2128 Oxford Street in Berkeley, California (Figure 1). The seismic traverses were oriented approximately east-west and were conducted in the alleyway area south of the building. Figures 2 and 3 depict the general site conditions in the area of the seismic traverses. Based on our discussions with you, it is our understanding that new construction activities are proposed at the site and the results of our study may be used for the design and construction parameters pertaining to the project.

4. STUDY METHODOLOGY

4.1 Refraction Microtremor (ReMi)

The ReMi technique uses recorded surface waves (specifically Rayleigh waves) that are contained in background noise to develop a shear-wave velocity profile of the study area down to a depth, in this case, of approximately 100 feet. The depth of exploration is dependent on the length of the line and the frequency content of the background noise. The results of the ReMi method are displayed as a one-dimensional profile which represents the average condition across the length of the line. The ReMi method does not require an increase of material velocity with depth; therefore, low velocity zones (velocity inversions) are detectable with ReMi.

Our ReMi evaluation included the use of a 24-channel Geometrics Geode seismograph and 24, 4.5-Hz vertical component geophones. Geophones were spaced 10 feet apart for a total line length of 230 feet. Figures 1 and 2 illustrate the approximate sounding location, while Figure 3 depicts the general site conditions in the study area. Fifteen records, each 32 seconds long, were recorded and then downloaded to a portable field computer. The data was later processed using Surface Plus 9.1 - Advanced Surface Wave Processing Software (Geogiga Technology Corp., 2020), which uses the refraction microtremor method (Louie, 2001), and other surface wave analysis methods. The program generates phase-velocity dispersion curves for each record and provides an interactive dispersion modeling tool where the users determine the best fitting model. The result is a one-dimensional shear-wave velocity model of the site with roughly 85 to 95 percent accuracy (Louis 2001).

4.2 Multichannel Analysis of Surface Waves (MASW)

The active source 1-D MASW method is based on the collection of seismic surface waves (specifically Rayleigh waves) which develop a 1-D shear-wave velocity profile of the study area. The results of the 1-D MASW method are displayed as a one-dimensional sounding which represents the average condition across the length of the line. The surface waves were generated by a hammer and plate (shot) and were recorded using a 24-channel Geometrics Geode seismograph and 24, 4.5-Hz vertical component geophones. Geophones were spaced 10 feet apart along with the profile for a total line length of 230 feet with shot points conducted at 10, 20, 40, 40, 60, 80, and 100 feet off the end of the line. Three records, one second long, were recorded at each shot location. These shot locations of surface wave data were evaluated for near and far field effects. The optimum offset-shot data was then combined with 15 records of recoded passive data and evaluated for our study. Figures 1 and 2 illustrate the approximate sounding location, while Figure 3 depicts the general site conditions in the study area. The recorded data were processed using SurfSeis® (Kansas Geological Survey, 2012), a Multichannel Analysis of Surface Waves (MASW) software program. One-dimensional (1-D) shear-wave velocity profiles were generated for each shot location, which corresponds to the middle of the geophone array (midpoint solution).

5. RESULTS AND CONCLUSIONS

As previously indicated, one ReMi traverse and one MASW traverse were conducted as part of our study. Table 1 and Figure 4 presents the ReMi model generated from our analysis. Table 2 and Figure 5 presents the results for the MASW evaluations. Based on the results, it appears the project site is generally underlain by low-velocity materials in the near-surface with a velocity increase at roughly 50 feet. The MASW results show more variation in the shallow velocities. The discrepancy between the two types of models is most likely due to the MASW method, which utilizes an active source. The active source typically provides higher resolution in the shallow layers but does not always provide data to the depths achieved with ReMi. Based on our analysis of the collected data for RL-1 and ML-1, the average characteristic site shear-wave velocities down to a depth of 100 feet and 91 feet are 1,242 and 1,109 feet per second (ft/s), respectively (IBC, 2018). The values indicated by the ReMi data correspond to a IBC seismic Site Class of **C**. It should be noted the ReMi and MASW results represent the average condition across the length of the line. Additionally, the velocities indicate that the site classification is very close to the boundary between C and D.

Table 1 – ReMi (RL-1) Results

Line No.	Depth (feet)	Shear Wave Velocity (feet/second)
RL-1 (N-S)	0 – 6	516
	6 – 11	740
	11 – 17	939
	17 – 24	842
	24 – 32	1111
	32 – 41	1109
	41 – 52	1178
	52 – 68	1537
	68 – 83	2605
	83 – 100	2626

Table 2 –MASW (ML-1) Results

Line No.	Depth (feet)	Shear Wave Velocity (feet/second)
ML-1 (N-S)	0 – 3	517
	3 – 6	515
	6 – 11	727
	11 – 16	780
	16 – 23	938
	23 – 32	948
	32 – 43	980
	43 – 56	1498
	56 – 73	1468
	73 – 91	2079

6. LIMITATIONS

The field evaluation and geophysical analyses presented in this report have been conducted in general accordance with current practice and the standard of care exercised by consultants performing similar tasks in the project area. No warranty, express or implied, is made regarding the conclusions, recommendations, and opinions presented in this report. There is no evaluation detailed enough to reveal every subsurface condition. Variations may exist and conditions not observed or described in this report may be present. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface surveying will be performed upon request.

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. Atlas should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document. This report is intended exclusively for use by the client. Any use or reuse of the findings, conclusions, and/or recommendations of this report by parties other than the client is undertaken at said parties' sole risk.

7. SELECTED REFERENCES

Kansas Geological Survey, 2010, SurfSeis© 5 MASW (Multichannel Analysis of Surface Waves): Version 5.3.0.8.

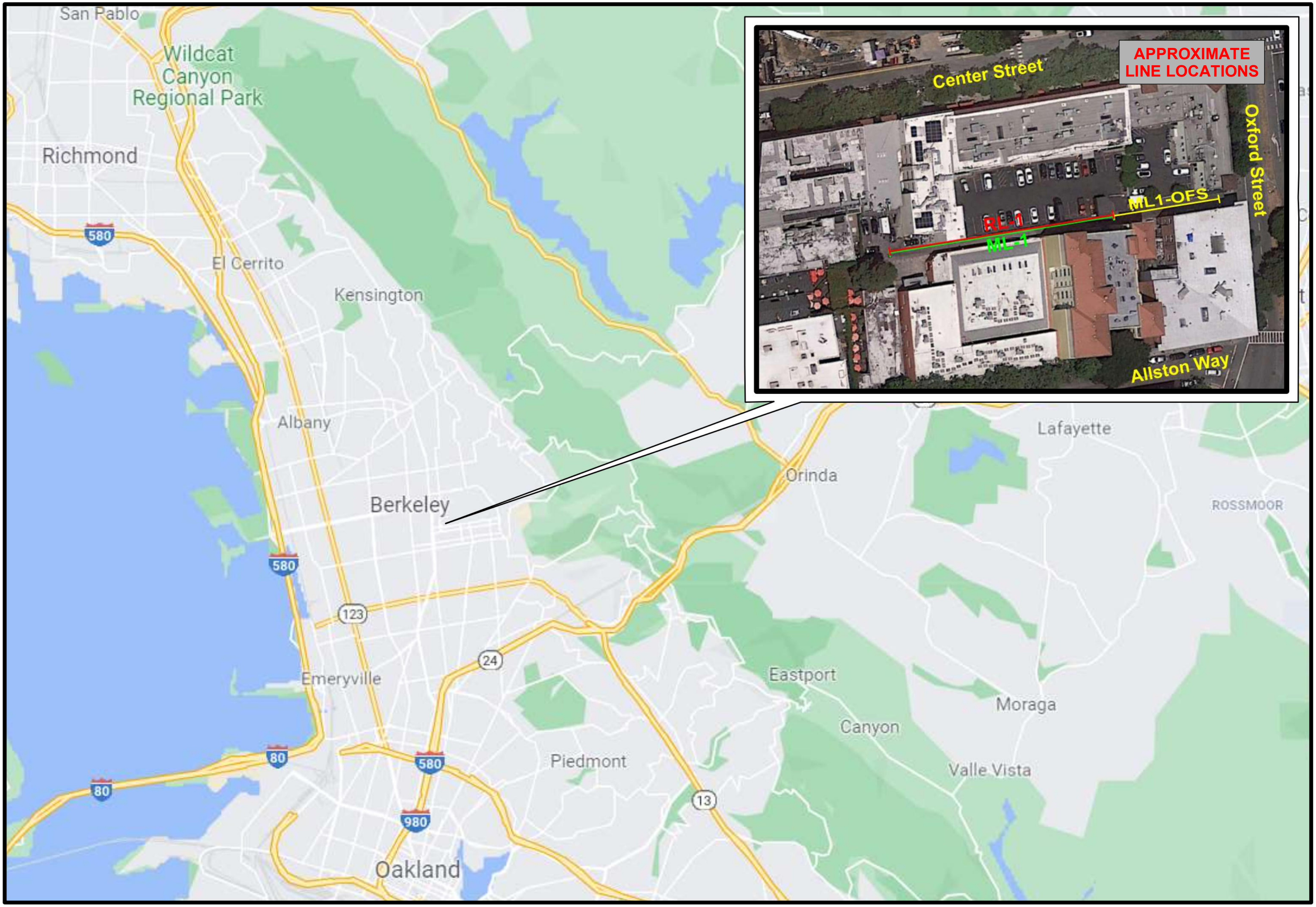
Mooney, H.M., 1976, Handbook of Engineering Geophysics, dated February.

Saito, M., 1979, Computations of Reflectivity and Surface Wave Dispersion Curves for Layered Media; I, Sound wave and SH wave: Butsuri-Tanko, v. 32, no. 5, p. 15-26.

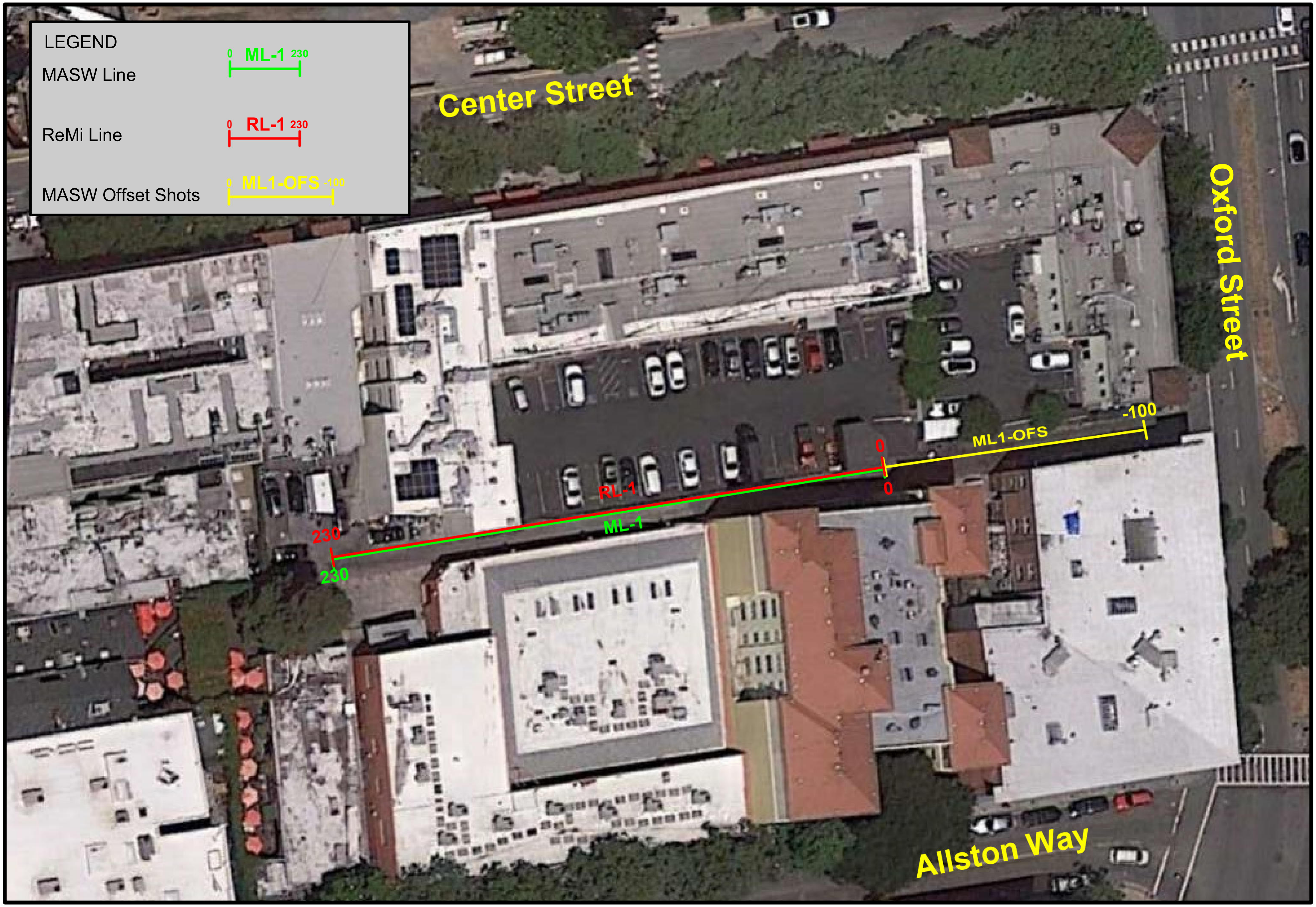
Saito, M., 1988, Compound Matrix Method for the Calculation of Spheroidal Oscillation of the Earth: Seismol. Res. Lett., v. 59, p. 29.

Telford, W.M., Geldart, L.P., Sheriff, R.E., and Keys, D.A., 1976, Applied Geophysics, Cambridge University Pres.

Xia, J., Miller, R. D., and Park, C. B., 1999, Estimation of Near-Surface Shear-wave Velocity by Inversion of Rayleigh Wave: Geophysics, v. 64, p. 691-7.



SITE LOCATION MAP	N	2128 Oxford Street Berkeley, California		ATLAS Figure 1
		Project No.: 121425SWG	Date: 11/21	



SEISMIC LINE LOCATION MAP

N

2128 Oxford Street
Berkeley, California

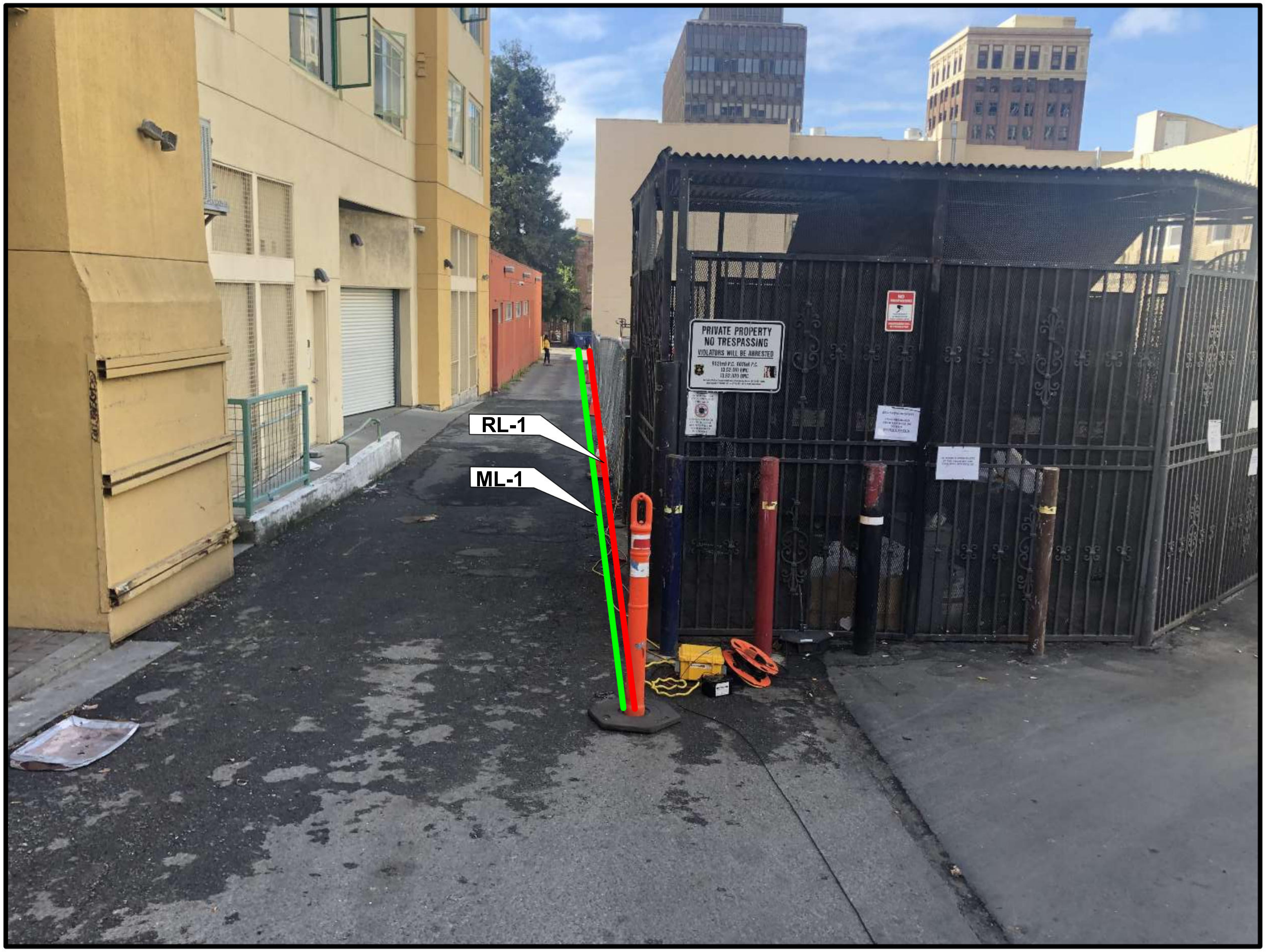
Project No.: 121425SWG | Date: 11/21

ATLAS

Figure 2

0 50 100

approximate scale in feet



SITE PHOTOGRAPH

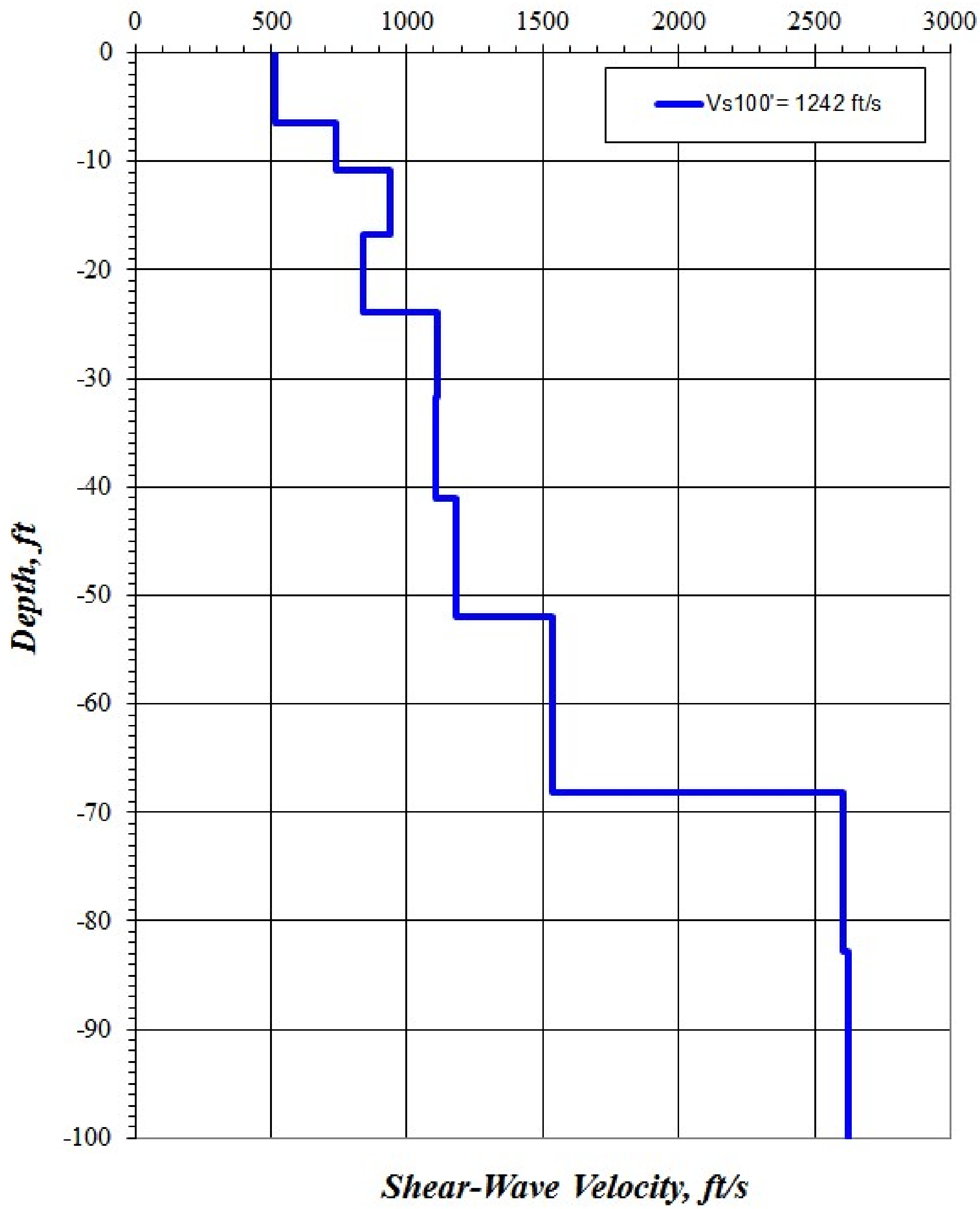
2128 Oxford Street
Berkeley, California

Project No.: 121425SWG

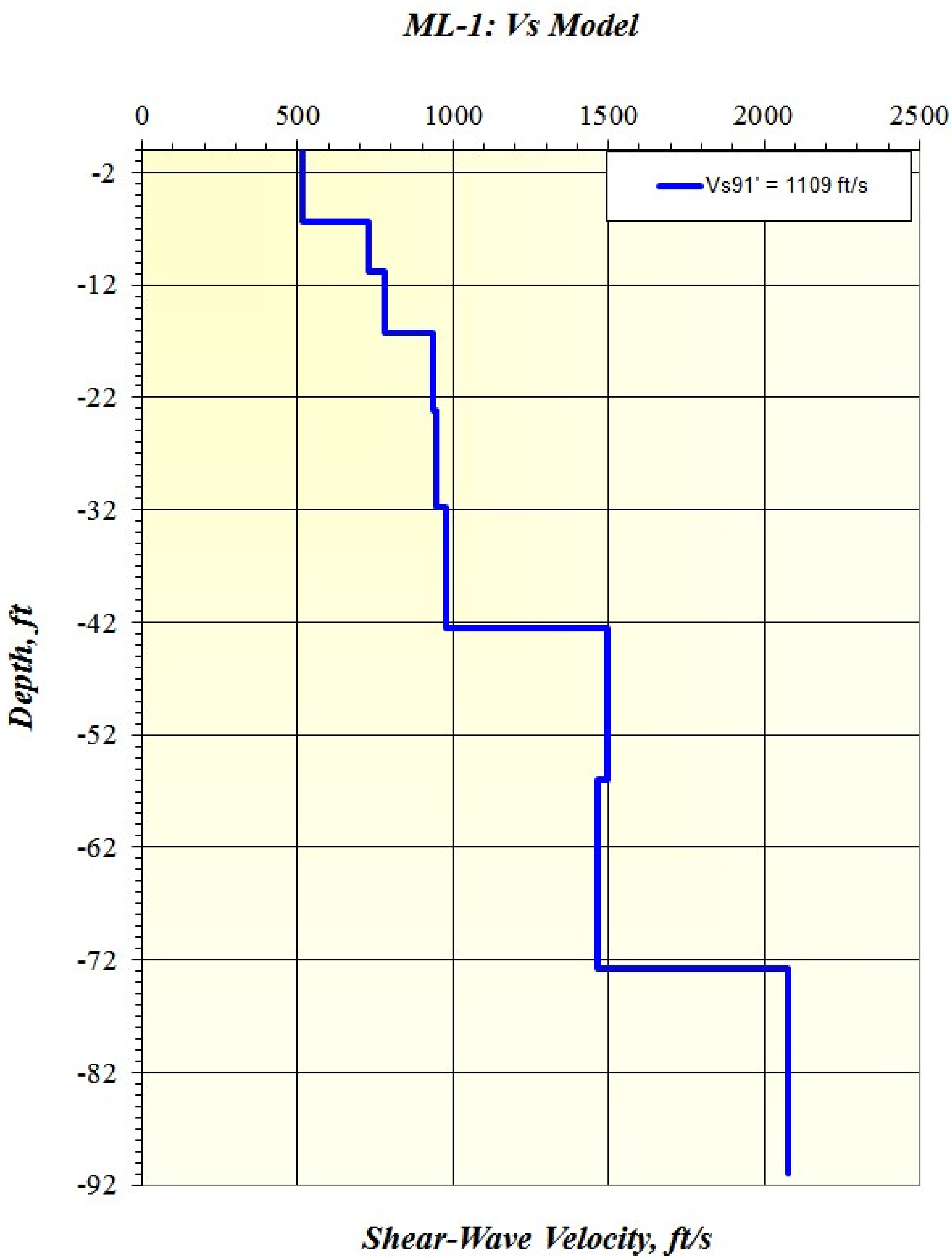
Date: 11/21

Figure 3

RL-1: Vs Model



ReMi RESULTS **RL-1**	2831 Oxford Street Berkeley, California Project No.: 121425SWG — Date: 11/21	ATLAS Figure 4



MASW RESULTS
ML-1

2831 Oxford Street
Berkeley, California

Project No.: 121425SWG

Date: 11/21

ATLAS

Figure 5

July 20, 2023
Z6101B

TO: Sharon Gong
Senior Planner
CITY OF BERKELEY
1947 Center Street, 2nd floor
Berkeley, California 94704

SUBJECT: **Supplemental Geotechnical Peer Review – Liquefaction Zone**
RE: Core Campus Manager LLC; 25-Story Student Housing Building
ZP2022-0135
2128 to 2136 Oxford Street and 2132 to 2154 Center Street, Berkeley

At your request, we have completed a geotechnical peer review of the proposed use permit application at the subject property using:

- Revised Geotechnical Report prepared by Partner, Inc., dated June 16, 2022; and

- California Geologic Survey, Guidelines for Evaluating and Mitigating Seismic Hazards in California – Special Publication 117A re-adopted September 11, 2008.

In addition, we have reviewed pertinent technical maps and reports from our office files.

DISCUSSION

Based on the referenced final report provided for our peer review, we understand the applicant proposes to demolish existing structures and improvements to construct a new 25-story multi-use building with basement parking. The report indicates that basement excavations are anticipated to extend approximately 14 feet below the ground surface. Portions of the proposed project are located within a liquefaction hazard zone as mapped by the California Geological Survey. According to the State's Seismic Hazards Mapping Act, a qualifying project in this zone must be supported by a site-specific geotechnical investigation (report) addressing the mapped hazard. In our previous geotechnical peer review letter dated March 4, 2022 we recommended supplemental evaluations to the geotechnical investigation that included liquefaction hazard analysis, geotechnical analysis, and evaluations of site hazards as well as foundation recommendations and pertinent design criteria for the project.

The purpose of this supplemental geotechnical peer review is to determine whether the referenced June 2022 report is consistent with State criteria for project approval with respect to liquefaction hazards. When site seismic hazards are confirmed to exist, the State requires that a minimum level of mitigation for a project be performed to reduce the risk of ground failure during an earthquake to a level that does not cause the collapse of buildings for human occupancy. Our geotechnical peer review does not include evaluation of detailed construction plans and is not intended to address all geotechnical aspects of proposed project design. We refer to our prior geotechnical peer review for a description of the site conditions.

CONCLUSIONS AND RECOMMENDATIONS

The Project Geotechnical Consultant (Partner) has completed an investigation that included one percolation test and two borings to a maximum advanced depth of 80.5 feet below the ground surface. The applicant's Consultant also reviewed the results of a prior site investigation by HARZA and geophysical testing by Atlas, as well as pertinent technical hazard maps and reports. Partner finds that groundwater levels are unlikely to be shallower than 10 feet below grade, and that site seismic design is best represented by a Site Class C. We understand the Project Geotechnical Consultant encountered serpentinite bedrock at approximately 75 feet below the ground surface in their two recent borings. Above site bedrock, the applicant's Consultant reported encountering up to 5 feet of fill overlying native clays. The Project Geotechnical Consultant did not discuss or provide their findings and conclusions regarding the risk of liquefaction to the project. The Project Geotechnical Consultant also notes that proposed construction excavations could damage neighboring improvements and provides general recommendations for potential groundwater monitoring and vibration monitoring.

The proposed site development is constrained by seismic ground shaking, expansive soils, as well as mapped liquefaction hazards, relatively deep bedrock, and relatively shallow groundwater conditions. We find the referenced June 2022 report provided for our peer review does not meet the State's criteria for evaluating the potential seismic hazards at the site. We also recommend the applicant's Consultant provide additional clarifications. Consequently, in order to meet the State's criteria for evaluating and mitigating the seismic hazards at the site, the Project Geotechnical Consultant should address the following Item 1:

1. **<u>Supplemental Geotechnical Evaluations and Recommendations</u>** – The Project Geotechnical Consultant should address the following:

   a. <u>Liquefaction Hazard</u> – The Project Geotechnical Consultant should evaluate and analyze, the potential hazard related to liquefaction and seismic densification at the subject property. The Geotechnical Consultant should provide estimates of seismically induced settlements, if applicable, and recommendations for suitable mitigation measures, if necessary.

   b. <u>Seismic Site Class Designation</u> – The Project Geotechnical Consultant indicates that the Site Class C was selected based on a ReMi survey of 1,242 ft/sec. Based on Table 20.2.1 in ASCE 7-22, a shear wave velocity of 1,242 ft/sec would be classified as a Site Class CD. The Geotechnical consultant should re-visit the selected site classification.

   c. <u>Anticipated Foundation Settlements</u> – We recommend the Project Geotechnical Consultant clarify anticipated static differential settlements, specifically the potential differential settlement at the transition between basement and at-grade portions of the structure.

   d. <u>Foundation and Wall Design Clarifications and Considerations</u> – The Project Geotechnical Consultant should provide recommended factors of safety for lateral resistance. The Consultant should also clarify the recommended active and at-rest coefficients (k/k_0). In addition, the Consultant should clarify if the proposed basement should be designed for active or at-rest pressures, and clarify the seismic loading acting on the basement wall (e.g., triangular distribution, etc. per "Seismic Earth Pressures on Deep Building Basements", Lew et al., 2010). The Consultant should also provide recommendations for vehicle surcharge loading and indicate which walls should be designed to resist this additional loading.

e. Construction Monitoring Clarifications and Thresholds – The Project Geotechnical Consultant should discuss and consider the benefits of additional monitoring program(s), including but not limited to installation of inclinometers and survey monuments (measured during construction), to evaluate potential impacts to adjacent structures. The Consultant should also provide recommended thresholds for groundwater drawdown, construction ground accelerations, as well as surface and/or subsurface displacements/stresses that would result in a stop work order and additional shoring or other mitigation requirements.

The Project Geotechnical Consultant should compile the results of their supplemental evaluations into a letter-report with appropriate data, results, and recommendations as applicable, to be submitted to the City for supplemental peer review by the City Geotechnical Consultant prior to approval of subject use permit applications. The Geotechnical Consultant should provide specific responses to these items, as opposed to boiler plate text.

LIMITATIONS

This supplemental geotechnical peer review has been performed to provide technical advice to assist the City with its discretionary permit decisions. Our services have been limited to review of the documents previously identified. Our opinions and conclusions are made in accordance with generally accepted principles and practices of the geotechnical profession. This warranty is in lieu of all other warranties, either expressed or implied.

Respectfully submitted,

COTTON, SHIRES AND ASSOCIATES, INC.
CITY GEOTECHNICAL CONSULTANT

M. Joseph Durdella
Supervising Engineering Geologist
CEG 2531

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PARTNER

August 10, 2023

Mark Goehausen
Core Campus Manager, LLC
1643 N. Milwaukee Avenue, 5th Floor
Chicago, Illinois 60647

Subject: **Response to City Comments**
Geotechnical Report
Student Housing Building
2128 Oxford Street
Berkely, California 94704
Partner Project No. 20-297761.3

Dear Mark Goehausen:

Partner Assessment Corporation (Partner) performed a geotechnical investigation for the proposed Student Housing Building to be located at 2128 Oxford Street in Berkley, California, and summarized the results in a report dated June 16, 2022. The City of Berkeley (The City) requested a review of our report by Cotton, Shires and Associates, INC. and provided comments in their letter dated July 18, 2023 (Project Number No ZB2022-0135).

This letter presents our responses to the Cotton, Shires and Associates, INC's review comments, as listed below:

> ***The City Comment a:*** *Liquefaction Hazard – The Project Consultant should evaluate and analyze, the potential hazard related to liquefaction and seismic densification at the subject property. The Geotechnical consultant should provide estimates of seismically induced settlements, if applicable, and recommendations for suitable mitigation measures, if necessary*
>
> **RESPONSE:** Based on our review of the Seismic Hazard Zone Report for the Oakland 7.5-Minute Quadrangle (CGS, 2003), the historical high groundwater level for the site is approximately 5 feet below the ground surface. Materials encountered below a depth of 5 feet in both our borings and the prior borings by Harza Engineering Company show clayey material with a liquid limit (LL) ranging from 37 to 40 and a plasticity index (PI) ranging from 20 to 23. California Geologic Survey, Guidelines for evaluating and Mitigating Seismic Hazards in California – Special Publication 117A states that soils are not susceptible to liquefaction if they have a PI>18 or have a PI>12 and a Moisture Content (MC%)>85% of the LL. The clayey soils encountered in our borings fail to meet these criteria there for we feel that the amount of potential liquefaction settlement at the site can be considered to be negligible.
>
> ***The City Comment b:*** *Seismic Site Class Designation – The project Geotechnical Consultant indicates that the Site Class C was selected based on a ReMi Survey of 1,242 ft/sec. Based on the Table 20.2.1 in ASCE 7-22, a shear wave velocity of 1,242 ft/sec would be classified as a Site Class CD. The Geotechnical consultant should re-visit the selected site class classification.*
>
> **RESPONSE:** It is our understanding that the current project will be constructed using the 2022 California Building Code (CBC). It is also our understanding that the 2022 CBC did adopt ASCE 7-22 and

therefore, still follows ASCE 7-16. Table 20.3-1 in ASCE 7-16 classifies a site with a shear wave velocity of 1,242 ft/sec as site class C.

The City Comment c: *Anticipated Foundation Settlements – We Recommend the Project Geotechnical consultant clarify anticipated static differential settlements, specifically the potential differential settlement at the transition between basement and at-grade portions of the structure.*

RESPONSE: It is our understanding that the current proposed project no longer consists of a partial basement, therefore there is no transition between basement and at-grade portions of the structure.

The City Comment d: *Foundation and Wall Design Clarifications and Considerations -The Project Geotechnical Consultant should provide recommended factors of safety for lateral resistance. The consultant should also clarify the recommended active and at-rest coefficients (k/k_0). In addition, the consultant should clarify if the proposed basement should be designed for active or at-rest pressures and clarify the seismic loading acting on the basement wall (e.g., triangular distribution, etc. per "seismic Earth Pressures on Deep Building Basements", Lew et al., 2010). The Consultant should also provide recommendations for the vehicle surcharge loading and indicate which walls should be designed to resist this additional loading.*

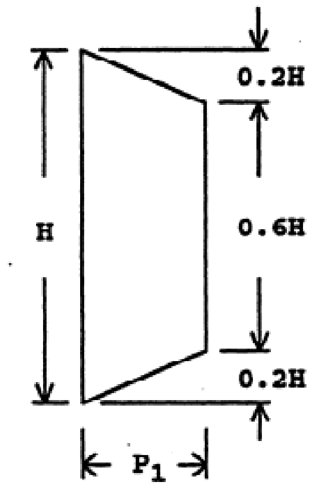
RESPONSE: Lateral earth pressures presented in our geotechnical report are unfactored, "raw" numbers and we defer to the wall designer on the appropriate safety factors that should be applied. Fluid pressures are the unit weight (120 pcf) times lateral earth coefficients, rounded conservatively. Equations used are:

- $K_o = (1 - \sin \phi')$
- $K_a = \tan^2(45^o - \phi/2)$
- $K_p = \tan^2(45^o + \phi/2)$

Recommendations regarding the active or at-rest pressures and the seismic loading acting on the basement wall was provided in our laterally loaded Structures Parameters in section 5.2 of our report and Appendix C of our report, this information is presented again here for your convenience.

In the report the pressures are usually reported as raw data in the form of equivalent fluid pressures for three cases. 1. Static is for soil pressure against a structure that is fixed at top and bottom, like a basement wall or box culvert. 2. Active is for soil pressure against a wall that is free to move at the top, like a retaining wall. 3. Passive is for soil that is resisting the movement of the structure, usually at the toe of the wall where the foundation and embedded section are located. The structural engineer is responsible for deciding on safety factors for design parameters and groundwater elevations based on the raw data in the geotechnical report.

The below diagram (from Caltrans Trenching and Shoring Manual) depicts the stress distributions around a shored excavation where struts are used. Depending on the types of walls and soil types encountered, different distributions may be needed. The conditions of this diagram should be carefully considered prior to use, and values given are unfactored. We recommend that a specialty contractor with in-house engineering capability perform the design of temporary shoring.



$P_1 = P_A = 0.8KwH$

For shoring or permanent retaining walls surcharges from traffic and adjacent buildings should be considered as shown in the below equations. The distribution of soil pressures on retaining structures will depend on the type of systems used, and whether they are braced or anchored. The shoring and retaining wall designer should be familiar with the appropriate distribution diagrams to be used and use care in the selection of the appropriate model. The walls should be designed to dissipate nuisance water to the sump system, through an interconnected series of drains. In general, this will not result in lowering of the groundwater table.

Building Foundation Surcharge Loading Equation

Resultant Lateral force:

$$R = \frac{0.3Ph^2}{x^2 + h^2}$$

Location lateral resultant:

$$d = x\left[\left(\frac{x^2}{h^2} + 1\right)\left(tan^{-1}\frac{h}{x}\right) - \left(\frac{x}{h}\right)\right]$$

Where:

R = Resultant lateral force measured in pounds per foot of wall width.

P = Resultant surcharge loads of continuous or isolated footings measured in pounds per foot of length to the wall.

x = Distance of resultant load from back face of wall measured in feet.

h = depth below point of application of surcharge loading to top of wall footing measured ion feet.

d = Depth of lateral resultant below point of application of surcharge loading measured in feet.

$\tan^{-1}\frac{h}{x}$ = The angle in radians whose tangent is equal to $\frac{h}{x}$.

Loads applied within a horizontal distance equal to the wall stem height, measured from the back face of the wall, shall be considered as surcharge.

For isolated footings having a width parallel to the wall less than 3 feet, "R" may be reduced to one-sixth the calculated value.

Vertical pressure due to surcharge applied to the top of the wall footing may be considered to spread uniformly within the limits of the stem and planes making an angle of 45 degrees with the vertical

Traffic Surcharge Loading Equation

$$q = k \times \gamma_s \times H_{eq}$$

Where:

q = Lateral surcharge pressure measured in pounds per square foot in a rectangular distribution.

k = Active or at-rest earth pressure coefficient as presented in section 5.2 of this report.

γ_s = Total unit weight of soil measured in pounds per cubic foot

H_{eq} = Equivalent height of soil from the below table.

Equivalent Height of Soil for Vehicular Loading on Retaining Wall and Shoring Parallel to Traffic*		
Excavation/Wall Height (ft)	**Distance from the edge of Excavation (ft)**	
	0.0 ft	**≥1.0 ft**
5.0	5.0	2.0
10.0	3.5	2.0
≥20.0	2.0	2.0

**From Table 3.11.6.4-2 of the AASHTO LRFD Bridge Design Specifications*

Seismic Surcharge Equations

Combined effect of static and seismic lateral forces:

$$P_{AE} = F_1 + F_2$$

$$F_1 = \frac{1}{2} \times A \times H^2$$

Resultant acting at a distance of $\frac{H}{3}$ from base of the wall

$$F_2 = \frac{3}{8} \times K_h \times \gamma \times H^2$$

Resultant acting at a distance of $(0.6 \times H)$ from base of the wall

Where:

F_1	= Static force, measured in pounds per linear foot, based on active pressure.
F_2	= Seismic Lateral Force, measured in pounds per linear foot, based on seismic pressure
γ	= 120 pounds per square foot
K_h	= ${S_{DS}}/{2.5}$
A	= Active Pressure, measured in pounds per cubic foot.
H	= Height of retained soil, measured in feet.

The City Comment e: *Construction Monitoring Clarifications and thresholds – The Project Geotechnical Consultant should discuss and consider the benefits of additional monitoring program(s), including but not limited to installation of inclinometers and survey monuments (measured during construction), to evaluate potential impacts to adjacent structures. The Consultant should also provide recommended thresholds for groundwater drawdown, construction ground accelerations, as well as surface and/or subsurface displacement stresses that would result in a stop work order and additional shoring or other mitigation requirements.*

RESPONSE: Geotechnical testing and observation during construction is considered to be a continuing part of the geotechnical consultation. To confirm that the recommendations presented in our geotechnical report and subsequent addendum remain applicable, our representative should be present at the site to provide appropriate observation and testing during the following primary activities:

- Solider pile and tieback installation
- Tieback anchor testing
- Lagging installation
- Installation of wall back-drainage provisions
- Foundation bottom observation and approval
- Placement and compaction of fill material
- Removal of shoring within the public right-of-way upon completion of the project
- De-tensioning of tieback anchors
- Installation of drywells

Some means of monitoring the performance of the shoring system is recommended. At a minimum shoring monitoring should consist of periodic surveying of the lateral and vertical locations of the tops of all the soldier piles. When design of the shoring system has been finalized, we can discuss this further with the design consultants and the contractor.

It is difficult to accurately predict the amount of deflection of a shoring system. It should be realized, however, that some deflection will occur. We estimate that this deflection could be on the order of 1 inch at the top of the shored embankment. If greater deflection occurs during construction, additional bracing may be necessary to minimize settlement of the utilities in the adjacent streets. If it is desired to reduce the deflection of the shoring, a greater active pressure could be used in the shoring design. Other monitoring programs that should be considered where further discussed in appendix C of our report and presented again here for your convenience.

Excavations over a depth of 20 feet require a shoring design. In the event excavations are planned near to existing structures, the geotechnical engineer should be consulted to evaluate whether such excavation will call for shoring or underpinning the adjacent structure. Pre-construction and post-construction condition surveys and vibration monitoring might also be helpful to evaluate any potential damage to surrounding structures.

Groundwater will dramatically change the stability of the excavation. In addition, pumping of groundwater from the bottom of the excavation can be difficult and costly, and it can result in potential damage to nearby structures if groundwater drawdown occurs. As such, we recommend that groundwater monitoring be performed across the site during design and prior to construction to assist in the excavation design and planning.

Groundwater pumping tests should be performed if groundwater pumping will be needed during construction. The pumping tests can be used to estimate drawdown at nearby properties, and also will be needed to determine the hydraulic conductivity of the soil for the design of the dewatering system.

We appreciate the opportunity to be of service during this phase of the work.

Sincerely,

Andrew J Atry, PE
Senior Engineer

REGISTERED PROFESSIONAL ENGINEER
ANDREW J. ATRY
C 84728
CIVIL
STATE OF CALIFORNIA

Attachments: City Comments

CITY COMMENTS

PARTNER

July 18, 2023
Z6101B

TO: Sharon Gong
Senior Planner
CITY OF BERKELEY
1947 Center Street, 2nd floor
Berkeley, California 94704

SUBJECT: **Supplemental Geotechnical Peer Review – Liquefaction Zone**
RE: Core Campus Manager LLC; 17-Story Student Housing Building
ZP2022-0135
2128 to 2136 Oxford Street and 2132 to 2154 Center Street, Berkeley

At your request, we have completed a geotechnical peer review of the proposed use permit application at the subject property using:

- Revised Geotechnical Report prepared by Partner, Inc., dated June 16, 2022; and

- California Geologic Survey, Guidelines for Evaluating and Mitigating Seismic Hazards in California – Special Publication 117A re-adopted September 11, 2008.

In addition, we have reviewed pertinent technical maps and reports from our office files.

DISCUSSION

Based on the referenced final report provided for our peer review, we understand the applicant proposes to demolish existing structures and improvements to construct a new 17-story multi-use building with basement parking. The report indicates that basement excavations are anticipated to extend approximately 14 feet below the ground surface. Portions of the proposed project are located within a liquefaction hazard zone as mapped by the California Geological Survey. According to the State's Seismic Hazards Mapping Act, a qualifying project in this zone must be supported by a site-specific geotechnical investigation (report) addressing the mapped hazard. In our previous geotechnical peer review letter dated March 4, 2022 we recommended supplemental evaluations to the geotechnical investigation that included liquefaction hazard analysis, geotechnical analysis, and evaluations of site hazards as well as foundation recommendations and pertinent design criteria for the project.

The purpose of this supplemental geotechnical peer review is to determine whether the referenced June 2022 report is consistent with State criteria for project approval with respect to liquefaction hazards. When site seismic hazards are confirmed to exist, the State requires that a minimum level of mitigation for a project be performed to reduce the risk of ground failure during an earthquake to a level that does not cause the collapse of buildings for human occupancy. Our geotechnical peer review does not include evaluation of detailed construction plans and is not intended to address all geotechnical aspects of proposed project design. We refer to our prior geotechnical peer review for a description of the site conditions.

CONCLUSIONS AND RECOMMENDATIONS

The Project Geotechnical Consultant (Partner) has completed an investigation that included one percolation test and two borings to a maximum advanced depth of 80.5 feet below the ground surface. The applicant's Consultant also reviewed the results of a prior site investigation by HARZA and geophysical testing by Atlas, as well as pertinent technical hazard maps and reports. Partner finds that groundwater levels are unlikely to be shallower than 10 feet below grade, and that site seismic design is best represented by a Site Class C. We understand the Project Geotechnical Consultant encountered serpentinite bedrock at approximately 75 feet below the ground surface in their two recent borings. Above site bedrock, the applicant's Consultant reported encountering up to 5 feet of fill overlying native clays. The Project Geotechnical Consultant did not discuss or provide their findings and conclusions regarding the risk of liquefaction to the project. The Project Geotechnical Consultant also notes that proposed construction excavations could damage neighboring improvements and provides general recommendations for potential groundwater monitoring and vibration monitoring.

The proposed site development is constrained by seismic ground shaking, expansive soils, as well as mapped liquefaction hazards, relatively deep bedrock, and relatively shallow groundwater conditions. We find the referenced June 2022 report provided for our peer review does not meet the State's criteria for evaluating the potential seismic hazards at the site. We also recommend the applicant's Consultant provide additional clarifications. Consequently, in order to meet the State's criteria for evaluating and mitigating the seismic hazards at the site, the Project Geotechnical Consultant should address the following Item 1:

1. **Supplemental Geotechnical Evaluations and Recommendations** – The Project Geotechnical Consultant should address the following:

   a. Liquefaction Hazard – The Project Geotechnical Consultant should evaluate and analyze, the potential hazard related to liquefaction and seismic densification at the subject property. The Geotechnical Consultant should provide estimates of seismically induced settlements, if applicable, and recommendations for suitable mitigation measures, if necessary.

   b. Seismic Site Class Designation – The Project Geotechnical Consultant indicates that the Site Class C was selected based on a ReMi survey of 1,242 ft/sec. Based on Table 20.2.1 in ASCE 7-22, a shear wave velocity of 1,242 ft/sec would be classified as a Site Class CD. The Geotechnical consultant should re-visit the selected site classification.

   c. Anticipated Foundation Settlements – We recommend the Project Geotechnical Consultant clarify anticipated static differential settlements, specifically the potential differential settlement at the transition between basement and at-grade portions of the structure.

   d. Foundation and Wall Design Clarifications and Considerations – The Project Geotechnical Consultant should provide recommended factors of safety for lateral resistance. The Consultant should also clarify the recommended active and at-rest coefficients (k/k_0). In addition, the Consultant should clarify if the proposed basement should be designed for active or at-rest pressures, and clarify the seismic loading acting on the basement wall (e.g., triangular distribution, etc. per "Seismic Earth Pressures on Deep Building Basements", Lew et al., 2010). The Consultant should also provide recommendations for vehicle surcharge loading and indicate which walls should be designed to resist this additional loading.

e. Construction Monitoring Clarifications and Thresholds – The Project Geotechnical Consultant should discuss and consider the benefits of additional monitoring program(s), including but not limited to installation of inclinometers and survey monuments (measured during construction), to evaluate potential impacts to adjacent structures. The Consultant should also provide recommended thresholds for groundwater drawdown, construction ground accelerations, as well as surface and/or subsurface displacements/stresses that would result in a stop work order and additional shoring or other mitigation requirements.

The Project Geotechnical Consultant should compile the results of their supplemental evaluations into a letter-report with appropriate data, results, and recommendations as applicable, to be submitted to the City for supplemental peer review by the City Geotechnical Consultant prior to approval of subject use permit applications. The Geotechnical Consultant should provide specific responses to these items, as opposed to boiler plate text.

LIMITATIONS

This supplemental geotechnical peer review has been performed to provide technical advice to assist the City with its discretionary permit decisions. Our services have been limited to review of the documents previously identified. Our opinions and conclusions are made in accordance with generally accepted principles and practices of the geotechnical profession. This warranty is in lieu of all other warranties, either expressed or implied.

Respectfully submitted,

COTTON, SHIRES AND ASSOCIATES, INC.
CITY GEOTECHNICAL CONSULTANT

M. Joseph Durdella
Supervising Engineering Geologist
CEG 2531

David T. Schrier
Principal Geotechnical Engineer
GE 2334

DTS:JD:st

September 13, 2023
Z6101C

TO: Sharon Gong
Senior Planner
CITY OF BERKELEY
1947 Center Street, 2nd floor
Berkeley, California 94704

SUBJECT: **Supplemental Geotechnical Peer Review – Liquefaction Zone**
RE: Core Campus Manager LLC; 25-Story Student Housing Building
ZP2022-0135
2128 to 2136 Oxford Street and 2132 to 2154 Center Street, Berkeley

At your request, we have completed a supplemental geotechnical peer review of the proposed use permit application at the subject property using:

- Response to City Comments for Geotechnical Report prepared by Partner, Inc., dated August 10, 2023;
- Revised Geotechnical Report prepared by Partner, Inc., dated June 16, 2022; and
- California Geologic Survey, Guidelines for Evaluating and Mitigating Seismic Hazards in California – Special Publication 117A re-adopted September 11, 2008.

In addition, we have reviewed pertinent technical maps and reports from our office files.

DISCUSSION

Based on the referenced letter-report response to City comments concerning the final geotechnical report provided for our peer review, we understand the applicant proposes to demolish existing structures and improvements to construct a new 25-story multi-use building with basement parking. The letter-report response indicates that a partial basement is no longer proposed in the anticipated excavations extending to approximately 14 feet below the ground surface. Portions of the proposed project are located within a liquefaction hazard zone as mapped by the California Geological Survey. According to the State's Seismic Hazards Mapping Act, a qualifying project in this zone must be supported by a site-specific geotechnical investigation (report) addressing the mapped hazard. In our previous geotechnical peer review letter dated July 18, 2023 we recommended supplemental evaluations to the geotechnical investigation that included liquefaction hazard analysis, geotechnical analysis, and evaluations of site hazards as well as foundation recommendations and pertinent design criteria for the project.

The purpose of this supplemental geotechnical peer review is to determine whether the referenced August 10, 2023 letter-report is consistent with State criteria for project approval with respect to liquefaction hazards. When site seismic hazards are confirmed to exist, the State requires that a minimum level of mitigation for a project be performed to reduce the risk of ground failure during an earthquake to a level that does not cause the collapse of buildings for human occupancy. Our geotechnical peer review does not include evaluation of detailed construction plans and is not intended to address all geotechnical aspects of proposed project design. We refer to our prior geotechnical peer review for a description of the site conditions.

CONCLUSIONS AND RECOMMENDATIONS

The Project Geotechnical Consultant (Partner) has completed an investigation that included one percolation test and two borings to a maximum advanced depth of 80.5 feet below the ground surface. The applicant's Consultant also reviewed the results of a prior site investigation by HARZA and geophysical testing by Atlas, as well as pertinent technical hazard maps and reports. Partner finds that groundwater levels are unlikely to be shallower than 10 feet below grade, and that site seismic design is best represented by a Site Class C. We understand the Project Geotechnical Consultant encountered serpentinite bedrock at approximately 75 feet below the ground surface in their two recent borings. Above site bedrock, the applicant's Consultant reported encountering up to 5 feet of fill overlying native clays. The Project Geotechnical Consultant also notes that proposed construction excavations could damage neighboring improvements and provides general recommendations for potential groundwater monitoring and vibration monitoring. The Project Geotechnical Consultant has satisfactorily addressed the a) risk of liquefaction to the project, b) the seismic site class designation, c) anticipated foundation settlements, d) foundation and wall design clarifications and considerations, e) and construction monitoring clarifications and thresholds in their August 10, 2023 response letter.

The proposed site development is constrained by seismic ground shaking, expansive soils, as well as mapped liquefaction hazards, relatively deep bedrock, and relatively shallow groundwater conditions. We find the referenced August 10, 2023 letter-report provided for our peer review does meet the State's criteria for evaluating the potential seismic hazards at the site. The Project Geotechnical Consultant has performed a site investigation and provided recommendations that are generally consistent with prevailing standards of practice for similar projects in the area:

We recommend geotechnical approval of the subject land use permit application with the following conditions attached:

1. **Geotechnical Plan Review** - The applicant's geotechnical consultant should review and approve all geotechnical aspects of the final project building and grading plans (i.e., site preparation and grading including removal and replacement/treatment of expansive soils, site surface and subsurface drainage improvements including site runoff discharge, and design parameters for foundations, temporary shoring excavation and installation, etc.) to ensure that their recommendations have been properly incorporated.

   The results of the plan review should be summarized by the Geotechnical Consultant in a letter and submitted to the City Engineer for review and approval prior to issuance of building permits.

2. **Geotechnical Plan Review** - Geotechnical Consultant should inspect, test (as needed), and approve all geotechnical aspects of the project construction. The inspections should include, but not necessarily be limited to: site preparation and grading, site surface and subsurface drainage improvements, and excavations for foundations and other improvements prior to the placement of steel and concrete.

   The results of these inspections and the as-built conditions of the project should be described by the geotechnical consultant in a letter and submitted to the City Engineer for review prior to final (granting of occupancy) project approval.

LIMITATIONS

This supplemental geotechnical peer review has been performed to provide technical advice to assist the City with its discretionary permit decisions. Our services have been limited to review of the documents previously identified. Our opinions and conclusions are made in accordance with generally accepted principles and practices of the geotechnical profession. This warranty is in lieu of all other warranties, either expressed or implied.

Respectfully submitted,

COTTON, SHIRES AND ASSOCIATES, INC.
CITY GEOTECHNICAL CONSULTANT

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