PARTNER

GEOTECHNICAL REPORT

New 6-Story Hotel and Restaurant
1580 Brookhollow Drive
Santa Ana, California 92705

July 18, 2018
Partner Project Number: 18-217764.1

Prepared for:

Moda Hotels, LLC
17510 Pioneer Boulevard, Suite 221
Artesia, California 90701

Engineers who understand your business

July 18, 2018

M. Paresh Bhakta
Moda Hotels, LLC
17510 Pioneer Boulevard, Suite 221
Artesia, California 90701

Subject: **Geotechnical Report**
New 6-Story Hotel and Restaurant
1580 Brookhollow Drive
Santa Ana, California 92705
Partner Project No. 18-217764.1

Dear **M.** Bhakta:

Partner Assessment Corporation (Partner) presents the following general opinion regarding the geotechnical conditions at the subject site, based on the information contained within this 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 more difficult in comparison with other similar sites*; given challenges associated with liquefiable site soils.***

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.

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

Sincerely,

REGISTERED PROFESSIONAL ENGINEER
MATTHEW MARCUS
C-81591
CIVIL
STATE OF CALIFORNIA

Matthew Marcus, PE
Technical Director – Geotechnical Engineering

Francisca Chan, EIT
Project Engineer

** "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

Geologic Zones and Site Hazards:

According to the report*: The subject property is located within the Coastal Plain, which is situated in the northwest corner of the Peninsular Range Geomorphic Province of California. The Coastal Plain is bounded by the Santa Monica Mountains on the north, and the La Merced Hills, Puente Hills, Peralta/Santiago Hills and San Joaquin Hills to the northeast, east and south. According to the Geologic Map of California, Santa Ana Sheet published in 1965, the property is underlain by undifferentiated Quaternary-age Recent Alluvial deposits. In our review, the site was located in a liquefaction seismic hazard zone by the California Department of Conservation. The site is susceptible to 6-inches of liquefaction induced settlement due to high historic groundwater table and sandy material underlying the property. Ground improvement aggregate piers would be required for the use of spread foundations. Otherwise, deep foundations are recommended on the site.

Excavation Conditions:

According to the report*: The subject property is currently vacant land with previously existing commercial building. We anticipate site excavations can be made using conventional construction equipment in good working condition. Remnants of previous construction, including concrete and steel if present may be difficult to remove. Likewise, loose fill soils and native sand soils may be prone to caving during excavation, and clayey soils may be difficult to traverse during wet weather. Groundwater was encountered at 21 feet below ground surface during drilling; however, groundwater levels can fluctuate over time and deep foundation are likely to encounter wet soils and groundwater.

Foundation/Slab Support:

According to the report*: Given the shallow groundwater, soft native soils at depth, and seismic setting, we recommend spread foundations supported on aggregate piers. The spacing and column diameter should be designed by a specialty contractor who has successfully performed similar projects in the City of Santa Ana. Alternatively, deep foundations such as drilled shafts can be used. Caving of holes should be anticipated in sandy material, casing and/or other stabilization methods should be used. Moderately expansive clays were encountered across the site and is not suitable below building slabs, 2 feet of non-expansive engineering fill should be used below slabs.

Soil Reuse:

According to the report*: Site soils that are moderately or highly expansive will not be usable as engineered fill in the building areas. It is recommended to use non-expansive structural fill that is free of deleterious materials, and is properly moisture conditioned and compacted to 95% of the modified proctor (ASTM D 1557) is recommended.

Pavement Design: According to the report*:

Roadway Type	*Subgrade Preparation*	*Pavement Section*
Parking Area Light Duty (TI=4)	Compacted Subgrade	4-in asphalt & 8-in aggregate base
Parking Area Heavy Duty (TI=7)	Compacted Subgrade	4-in asphalt & 10-in aggregate base

This summary in no way replaces or overrides the detailed sections of the 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 are included in Appendix A of the report, and laboratory testing is included in Appendix B of the report. 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:	Multi-story hotel and possible future restaurant
Building footprint/height	6 stories above grade, roughly 13,500 sf footprint
Land Acreage (Ac):	Approx. 2.5 Ac
Number of Buildings:	2
Expected Cuts and Fills	Less than 5 feet
Type of Construction:	Concrete slab on grade, lightweight framing
Foundations Type	Spread Foundations / Deep Foundations
Anticipated Loads	4,000 -6,000 psf
Traffic Loading	Parking lot/ dumpster pad
Site Information Sources:	Preliminary Site Plan

2.3 References

The following references were used to generate this report:

California Dept. of Transportation, ARS Online, accessed 7/10/2018

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

California Geological Survey, Geologic map of the San Bernardino and Santa Ana 30′ X 60′ Quadrangles, 1: 100,000 scale, Morton and Miller, 2006.

Federal Emergency Management Agency, FEMA Flood Map Service Center, accessed 7/12/2018

Google Earth Pro (Online), accessed 7/12/2018

Historic Aerials by NETR Online, accessed 7/12/2018

United States Geological Survey, Lower 48 States 2014 Seismic Hazard Map, accessed online 7/12/2018

United States Geological Survey Topographic Map 2015, 7.5 minute series, *Tustin, CA*, accessed via internet, accessed 7/12/2018

United States Geologic Survey, Earthquake Hazards Program (Online), accessed 7/12/18

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. Any parties relying on this report do so having accepted Partner's standard Terms and Conditions, a copy of which can be found at http: / www.partneresi.com/terms-and-conditions.php

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 a 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 a currently vacant lot in Santa Ana, California. The immediately surrounding properties consist of commercial buildings on both sides and a busy freeway entrance on the south. 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**
1896-1946	Topographic Maps	Undeveloped/ Agricultural
1946-1979	Aerial Photograph	Agricultural use
1979 - 2004	Aerial Photographs, Building Records, City Directories, DTSC Records, Interviews	Building on the east side; rest is vacant lot
2004-Present	Aerial Photographs, Building Records, City Directories, Interviews, Onsite Observations	Vacant Lot

3.2 Geologic Setting

The subject property is located within the Coastal Plain, which is situated in the northwest corner of the Peninsular Range Geomorphic Province of California. The Peninsular Range consists of northwest-treading mountain ranges and associated valleys. The Coastal Plain is bounded by the Santa Monica Mountains on the north, and the La Merced Hills, Puente Hills, Peralta/Santiago Hills and San Joaquin Hills to the northeast, east and south. According to the Geologic Map of California, Santa Ana Sheet published in 1965, the property is underlain by undifferentiated Quaternary-age Recent Alluvial deposits. In our review, the site was located in a liquefaction seismic hazard zone by the California Department of Conservation. The site is susceptible to 6-inches of liquefaction induced settlement due to high historic groundwater table and sandy material underlying the property. Ground improvement aggregate piers would be required for the use of spread foundations. Otherwise, deep foundation is recommended on the site.

Geologic Data		
Parameter	**Value**	**Source**
Geomorphic Zone	Peninsular Ranges	CGS
Ground Elevation	58 feet above MSL	USGS
Flood Elevation	Zone X (Minimal Flood Hazard)	FEMA
Seismic Hazard Zone	Moderate	USGS
Geologic Hazards	Moderately Expansive Soils	CGS
Surface Cover	Disturbed Alluvium	Google Earth
Site Modifications	Previous agricultural field	Google Earth
Surficial Geology	Alluvium	USGS

Geologic Data		
Parameter	**Value**	**Source**
Depth to Bedrock	Unknown	Not Applicable
Groundwater Depth	21 feet	Boring Log

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, and liquefaction. According to California Department of Transportation's ARS Online Database, the three faults most relevant to the site are the San Joaquin Hills – 1.9 miles from site, MMax 7.0, the Newport Inglewood fault zone (Los Angeles Basin) – 6.2 miles from site, MMax 7.2 and Compton -10.4 miles from site, MMax 6.9. The site is susceptible to seismic induced liquefaction.

The seismic design parameters based on the USGS Design Maps Detailed Report for ASCE 7-10 Standard Method are presented below.

Seismic Item	Value	Seismic Item	Value
Site Classification	D	Seismic Design Category	D
Fa	1.000	Fv	1.5
Ss	1.509g	S_1	0.558
S_{MS}	1.509g	S_{M1}	0.837g
S_{DS}	1.006g	S_{D1}	0.558g
PGA Max (ASCE '10)	0.565g	67% PGA (ASCE '10)	0.379g

4. GEOTECHNICAL EXPLORATION & LABORATORY RESULTS

Our evaluation of soils on the site included field exploration and laboratory testing. The field exploration and laboratory testing programs are briefly described below. Data reports from the field exploration and laboratory testing are provided in Appendix A and Appendix B, respectively.

4.1 Soil Borings

The soil boring program was conducted on June 25, 2018. Six (6) borings were advanced by the use of a truck-mounted drill using hollow flight auger drilling techniques. The borings were made to depths of 20 to 50 feet in the buildings footprint (B1, B3-B5), and 5 feet in the parking areas (B2 and B6). In addition, three (3) infiltration tests to 5 feet below ground surface in landscaping areas. The approximate locations of the exploratory borings and infiltration tests are shown on Figure 2.

Logs of subsurface conditions encountered in the borings were prepared in the field by a representative of Partner Engineering. Soil samples consisting of relatively undisturbed brass ring samples and Standard Penetration Tests (SPT) samples were collected at approximately 2.5 and 5-foot depth intervals and were returned to the laboratory for testing. The SPTs were performed in accordance with ASTM D 1586. Typed boring logs were prepared from the field logs and are presented in Appendix A. A summary table description is provided below:

Surficial Geology–		
Strata	**Depth to Bottom of Layer (bgs*)**	**Description**
Fill Material	3-5 feet	Silty Sand Fill
Native Stratum 1	15 feet	Clayey Alluvium
Native Stratum 2	35 feet	Sandy Alluvium
Native Stratum 3	45 feet	Clayey Alluvium
Native Stratum 4	50 feet	Sandy Alluvium
Groundwater	21 feet	In boring
Bedrock	NA	Not observed

bgs – below ground surface

4.2 Groundwater/Soil Moisture:

Groundwater was encountered on the site during drilling at 21 feet below ground surface. However, 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 are presented in Appendix B. Site soils contained large amounts of clay, which might be expansive, and sandy liquefiable soils.

4.4 Infiltration Test Results:

Three (3) infiltration tests were attempted, only one was fully completed. Locations are shown on Figure 2. The tests were performed at a depth of 5 feet, due to clayey/silty layers less than 1 inch of infiltration occurred during a span of multiple hours in P1 and P3 and tests were abandoned. Results indicate the site is adverse to surficial storm water infiltration. Data is shown in Appendix B, and is summarized below:

Parameter	***P2***
Location	North Side
Elevation of Tested Area	5 feet
Pre-soak Depth	3 feet
Test Start Depth	17 in
Water Drop During Test	0.61 in
Un-factored Infiltration Rate	1.2 in/hr
Reduction Factor	5
Site Siltation Factor	2
Site Variability Factor	1
Reduced Infiltration Rate	**.05 in/hr**

5. GEOTECHNICAL RECOMMENDATIONS & PARAMETERS

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 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 subject property is located within the Coastal Plain, which is situated in the northwest corner of the Peninsular Range Geomorphic Province of California. Based on our liquefaction evaluation using Novoliq software and the Boulanger/Idriss 2014 analysis, liquefaction induced settlement and lateral spreading is anticipated to be a factor on the site. The site is susceptible to 6-inches of liquefaction induced settlement due to high historic groundwater table and sandy material underlying the property. Ground improvement aggregate piers would be required for the use of spread foundations. Otherwise, deep foundations are recommended on the site.

Excavation Considerations

- The site is currently vacant dirt and grass covered land. We anticipate site excavations can be made using conventional construction equipment in good working condition. Loose fill soils and native sand soils may be prone to caving during excavation. Groundwater was encountered at 21feet below ground surface during drilling. However, groundwater levels can fluctuate over time. Excavations should be sloped and/or shored to protect worker safety and adjacent properties, per OSHA and local guidelines.
- The previous construction and/or undocumented site fills could contain deleterious, expansive and/or soft fills, buried utilities, construction debris, etc. that may be difficult to remove and other materials that may call for special handling and haul off. Following removal of deleterious soils, the base materials should be evaluate by an engineer to evaluate that a stable subgrade for replacement fill has been achieved.
- The groundwater table was encountered at depths at 21 feet below the ground surface. Excavations into the groundwater table will call for excavation stabilization techniques for deep foundations or aggregate piers and described below.

Drilled Shaft Foundation Option

- The building structure could be supported on a system of drilled shafts, pile caps, and grade beams that would support the building walls and roof. The drilled shafts should extend at least 40 feet below the ground surface into relatively dense, non-liquefiable soils. 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 and California State DOT guidelines. 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 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, 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 California DOT specifications.
- A slab on grade could also be used with this system, however, it would require preparation of the soil subgrade as described in the next section.

<u>Spread Foundation on Stone Column Option</u>

- We recommended improving the site by the use of rammed aggregate piers, also known as stone columns. Following complete site demolition and haul-off, the stone columns should be advanced to 40 feet throughout the site. The columns should be installed on a grid that would be designed by the specialty contractor in order to reduce the sum of the static and liquefaction induced settlement on the site to less than 1 inch, with less than 0.75 inches of total differential settlement. The spacing and column diameter should be designed by a specialty contractor who has successfully performed similar projects in the City of Santa Ana.
- Given the liquefiable section from 20 to 35 feet, friction resistance in that zone should not be included on the contractor's design. If the contractor would like to take responsibility for frictional resistance in that liquefaction zone then contractor should perform a test area, and then our office would perform additional soil borings or CPT soundings to determine if the column installation was successful. Following a successful installation of stone columns the spread foundations should be proportioned to bear on top of the stone columns in such a way as to provide the proper support against settlement. The slab on grade would also be used in this situation as described below.

<u>On-Grade Construction Considerations</u>

- All grass, roots and other plant materials should be removed from structural areas of the site. Given the presence of moderately expansive clay soils across the site, slab-on-grade areas should be supported on a 2-foot thick layer of non-expansive engineered fill overlying competent native soil.
- In new fill areas more than 2 feet below planned slabs or in new 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. 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. The improvements should extend a distance of 2 feet beyond the planned area of new construction at finished grade (for fill sites, 2 feet inside the top of slope).

Soil Reuse Considerations

- Some of the excavated site soils should be suitable for reuse as engineered fill, provided that they are clean of debris and deleterious materials, properly moisture conditioned, and compacted. Previous development debris may contain potentially hazardous materials will require testing prior to haul off and/or reuse on the site as would any soil materials with high organic content. Engineered fill should be compacted to 95% of maximum dry density generally near to optimum moisture content, according to the modified proctor, ASTM D1557. Moderately expansive clay soil was encountered across the site, and it should not be used within the upper 2 feet of concrete slabs on grade.

Concrete Considerations

- Concrete should be corrosion resistant, using Type II/V Portland Cement, and fly ash mixtures of 25 percent cement replacement. We recommend a water/cement ratio of 0.45 or less. Site soil may be corrosive to un-protected metallic elements such as pipes, poles, etc. Concrete exposed to freezing weather in cold climates should be air-entrained.

Site Storm Water Considerations

- The site surficial soils are generally clayey and are adverse to storm water infiltration. 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.

5.2 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).

Subgrade Preparation Parameters – (hyperlink to Construction Considerations)

Subgrade Preparation				
Structure	**Bearing Capacity**	**Embedment Depth**	**Bearing Surface** [a]	**Settlement** [d]
Grade Slabs	k=150 pci [b]	NA	Proofrolled and compacted subgrade	<1 inch
Drilled Shafts	See Chart	~40 feet or more	Dense native soil	<1 inch
Spread Foundations	4,000-6,000 [c] psf	24 inches	Aggregate Piles to 40-ft (As per Design Contractor)	<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)*

[c] *Can be increased by 1/3 for temporary loading such as seismic and wind*

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

Paving Structural Sections – (hyperlink to Construction Considerations)

Pavement Sections		
Roadway Type	**Subgrade Preparation [a]**	**Pavement Section [b]**
Parking Area Light Duty (TI=4)	Proofrolled Subgrade	4-in asphalt & 8-in aggregate base
Parking Area Heavy Duty (TI=7)	Proofrolled Subgrade	4-in asphalt & 10-in aggregate base
Parking Area Heavy Duty (TI=7)	Proofrolled Subgrade	7-in concrete & 4-in aggregate base

[a] *Repairs in proofrolled areas should be structural fill per the recommendation of the Earthwork (hyperlink to Construction Considerations) 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.*

[b] *1 inch of pavement may be reduced if 6-in of lime-treated soil is used with a 500 psi 28-day compressive strength*

Laterally Loaded Structures Parameters– (hyperlink to Construction Considerations)

Lateral Earth Pressures –				
Soil Type	**Coefficient of Friction (µ)**	**Static Fluid Pressure (pcf)**	**Active Fluid Pressure (pcf)**	**Passive Fluid Pressure (pcf)**
Fill Soil (Upper 5 feet)	0.3	75	55	200
Native/ Clay Soil (5 - 15 ft)	0.3	60	40	250
Native /Sandy Soil (15 – 35 ft)	0.35	60+62.4 [a]	40+62.4*	370
Native /Clay Soil (35 – 45 ft)	0.3	60+62.4 [a]	40+62.4*	250
Native /Sandy Soil (45 – 50 ft)	0.35	60+62.4 [a]	40+62.4*	370

[a] *Assumed GW table at 21 ft bgs, for underground structures where water is only on one side, the hydrostatic pressure of 62.4 psf should be added*

Liquefaction Analysis:

The obtained data from geologic research, soil borings, and laboratory testing was entered into the Novoliq software program for liquefaction analysis. The SPT blow counts using the modified ring sampler were reduced in half, and the appropriate correction factors were used in the analysis. The historic high groundwater was published on the California Department of Conservation website as 10 feet bgs. The correction factors we applied to account for the hammer type, sampler type, borehole diameter, and rod length.

The analysis relied on the NCEER Workshop (1997) and the Boulanger Idriss (2014) for layer factors of safety, and Ishihara and Yoshimi (1992) for settlement calculations. The anticipated liquefaction settlement and differential settlement are shown below. In general, spread and deep foundations can tolerate a maximum of 0.75 inches differential settlement. The detailed analysis is presented in Appendix D.

Liquefaction and Static Settlement Estimates for Foundation Options				
Seismic Item	**Total Seismic Settlement (in)**	**Seismic Differential Settlement (in)**	**Total Static +Seismic Settlement (in)**	**Total Differential Settlement (in)**
Isolated on Stone Columns (40 ft)	0.5	0.25	1.5	0.75
Drilled Shafts (minimum of 40 ft bgs)	0.5	0.25	1.0	0.75

Differential settlement is assumed to be half of total settlement

Final depth of shafts/stone columns should be designed by specialty contractor

FIGURES

- Site Location Map
- Site Exploration Map
- Deep Foundation Capacity (2)

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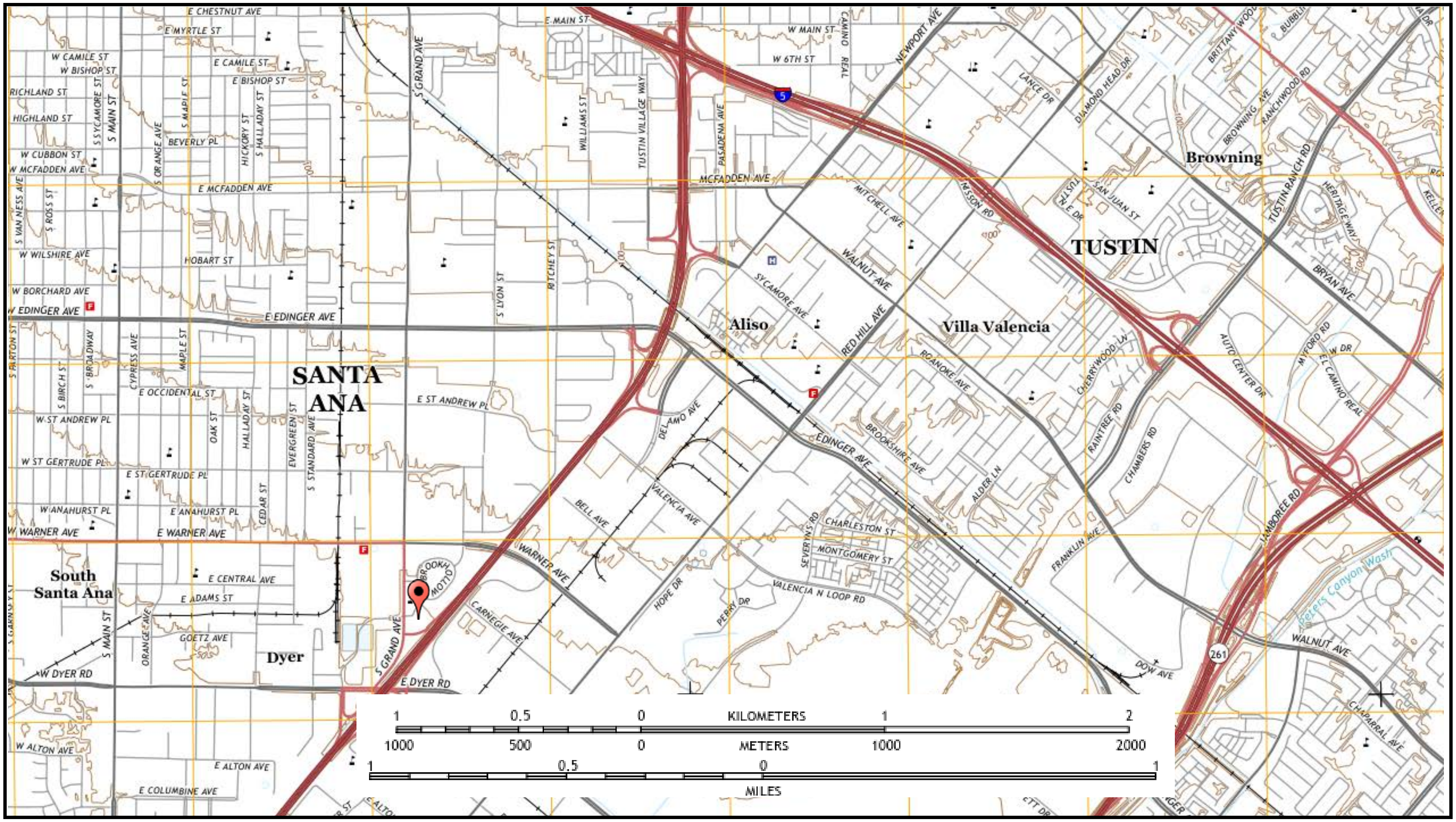
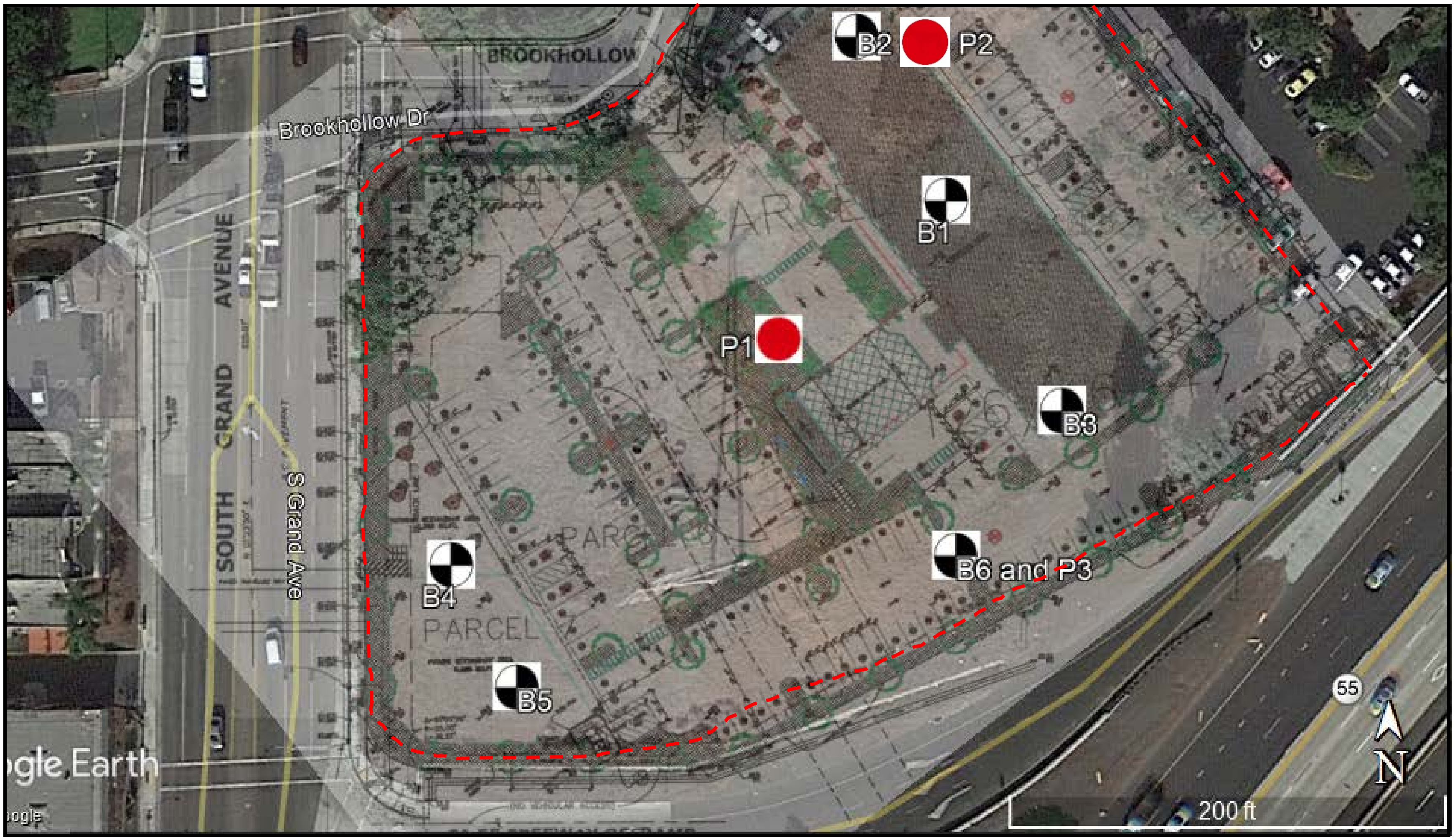


Source: USGS Topographic Tustin Quadrangle Map 2015.

N

Key:

Approximate Site Location



Source: Google Earth, 2018; Santa Ana Site Plan, 2018.

Key:

Approximate Boring Location

Approximate Infiltration Location

Approximate Project Limits

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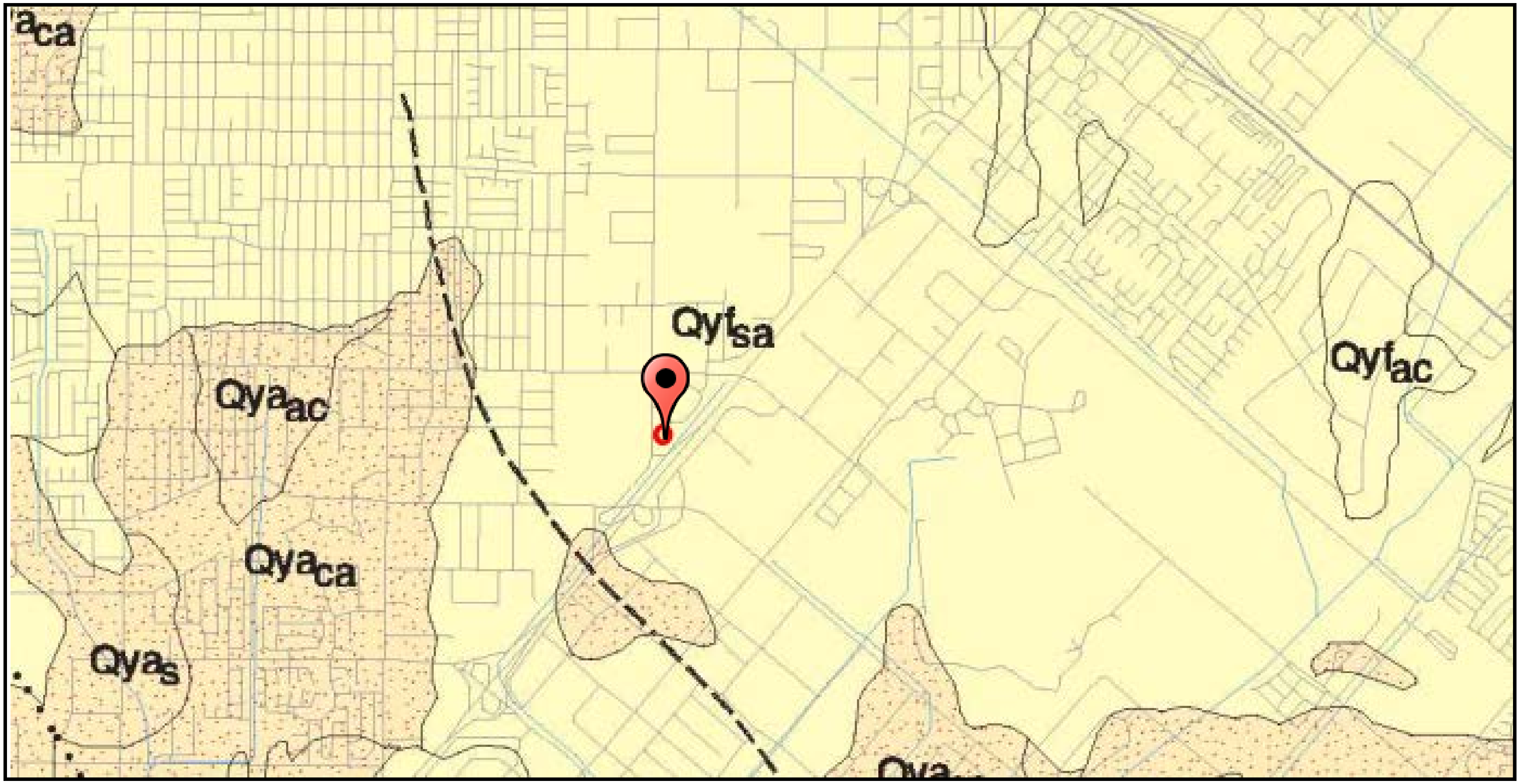

Source: CGS, Geologic Map of the San Bernardino and Santa Ana 30' X 60' Quadrangles, 1: 100,000 scale. Compiled by Morton and Miller, 2006.

N

Key:

Approximate Site Location

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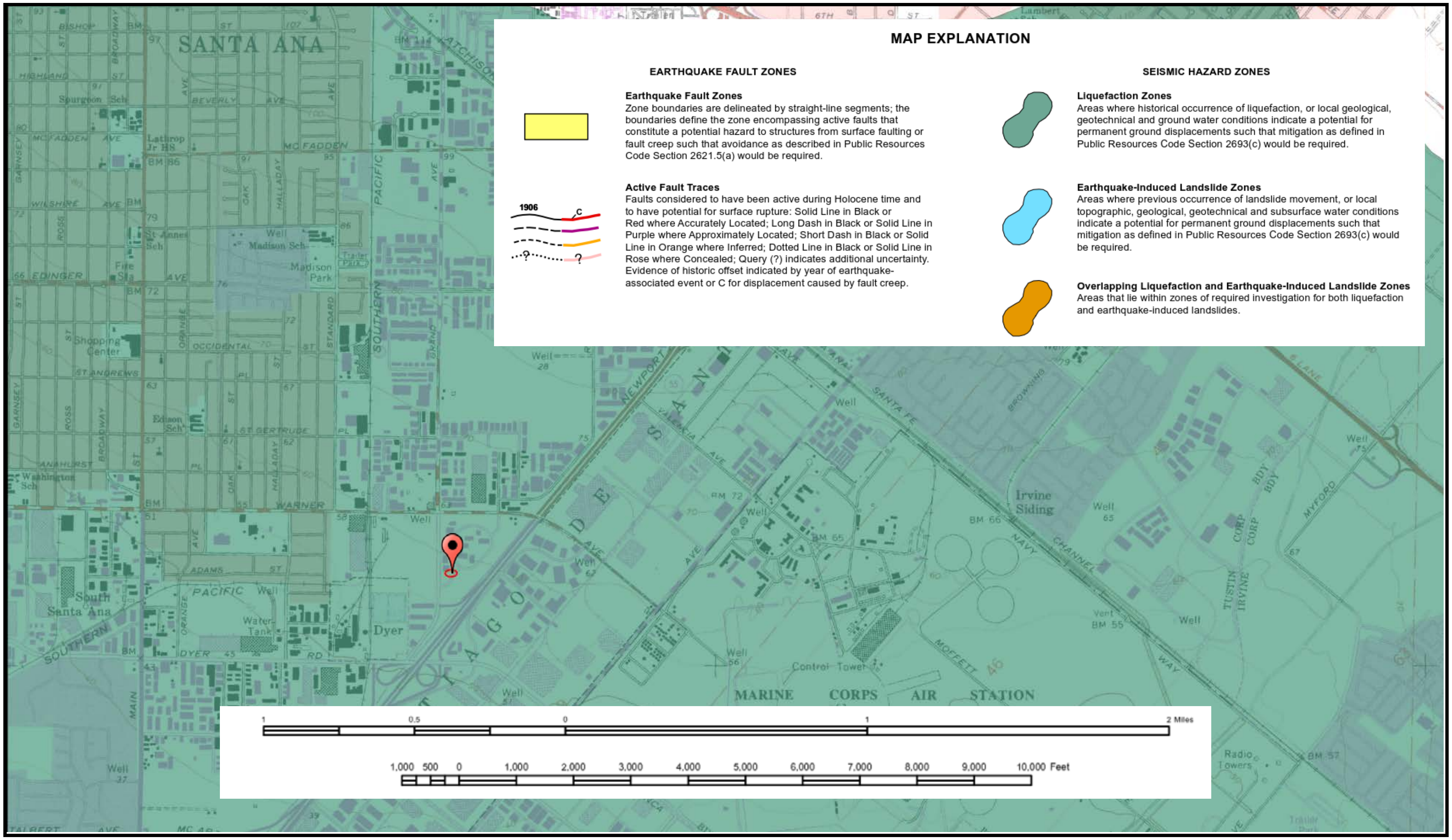

Source: CGS, Earthquake Zones of Required Investigation, Tustin Quadrangle, 2001.

N

Key:

Approximate Site Location

APPENDIX A

Boring Logs

INFILTRATION TEST DATA

Location P2

Percolation Test - P2 @ 5 feet bgs						
Pre Soak Time: 8: 20 to 10:31						
Percolation Reading #	**Start Time/ End Time**	**Time Δ time (min)**	**WL BTP (in)**	**WL Δ (in)**	**Percolation Rate for Reading (in/hr)**	**Calculations**
1	11:45 12:25	40	2.75 3.6	0.85	1.7	**Total Reduction Factor (Rf) =** **5**
2	12:28 12:59	31	3.1 3.8	0.7	1.4	*d* 1 = 17
3	1:02 1:30	31	3.2 3.75	0.55	1.1	Δd = 0.25 DIA = 2
4	1:31 2:00	31	3.15 3.75	0.6	1.2	**(Rf) =** 5.00
5	2:11 2:43	32	3.2 3.81	0.61	1.22	**Infiltration Rate =** Pre-adjusted Percolation Rate / Reduction Factor
6	2:51 3:04	30	3.2 3.53	0.59	1.18	Pre adjucted Percolation Rate* = 0.24 Reduction Factor = 5.00
7						
						Infiltration Rate (in/hr) = 0.05

Notes:	
btp - below top of pipe	*d* 1 = Depth to Initial Water Depth (in.)
WL - water level	Δd = Water Level Drop of
min - minutes	DIA - Diameter of the
ft - feet	

***Pre adjusted percolation rate is the average drop of the stabilized rate over the last 3 consecutive readings**

Boring Number:	B1 (cont.)	Boring Log Page 1 of 1	
Location:	middle	Date Started:	6/25/2018
Site Address:	1580 Brookhollow Dr. Santa Ana, CA 92705	Date Completed:	6/25/2018
	Santa Ana, CA 92705	Depth to Groundwater:	20'
Project Number:	18-217764.1	Field Technician:	YK/FC
Drill Rig Type:	CME-95	**Partner Engineering and Science**	
Sampling Equipment:	SPT & Rings	2154 Torrance Blvd., Suite 200	
Borehole Diameter:	8"	Torrance, CA 90501	

Depth	Sample	N-Value	USCS	Description
30	S	4	SC	Brown, saturated, loose, clayey SAND (Fines: 27.7%)
31				
32				layer change at around 31.5'
33				
34				
35	S	10	CL	firm, saturated, firm, CLAY
36				
37				
38				
39				
40	S	18		
41				
42				
43				
44				
45	S	24	SC	stiff, clayey SAND (Fines: 41.9%)
46				
47				
48				
49				
50	S	8	SP	brown, saturated, loose, poorly graded SAND with gravel
51				Boring terminated at 51.5'
52				Backfilled with spoils upon completion
53				Groundwater encountered at 20'
54				
55				
56				
57				
58				
59				

APPENDIX B

Lab Data

HAMILTON
& Associates

1641 Border Avenue • Torrance, CA 90501 T 310.618.2190 888.618.2190 F 310.618.2191 W hamilton-associates.net

July 16, 2018
H&A Project No. 18-2472
Partner Project No. 18-217764.1

Partner Engineering and Science, Inc.
4518 N.12 Street Suite 201
Phoenix, AZ 85016

Attention: Mr. Matthew Marcus, Technical Director- Geotechnical Engineering

Subject: Laboratory Testing of Soil Samples:
1580 Brookhollow Drive, Santa Ana, CA

Dear Mr. Marcus:

We have completed the laboratory tests on the samples provided for the subject project. Enclosed is a summary of laboratory test results.

We thank you for the opportunity to provide laboratory testing services. If there are any questions, please do not hesitate to contact the undersigned.

Respectfully submitted,
HAMILTON & ASSOCIATES, INC.

Rosa E. Murrieta
Laboratory Supervisor | Staff Geologist

David T. Hamilton, PE, GE
President

Distribution: (1) Matthew Marcus, mmarcus@partneresi.com
(2) Brett Bova, bbova@partneresi.com

MOISTURE CONTENT AND DENSITY TESTS

Relatively undisturbed soil retained within the rings of the Modified California barrel sampler were tested in the laboratory to determine in-place dry density and moisture content. Test results are presented in Table 1.

NO. 200 SIEVE (WASH)

No. 200 Sieves (Wash) were performed on selected samples to determine the fines content. The results of these tests are shown on Table 1.

CONSOLIDATION TESTS

Consolidation (ASTM D 2435) tests were performed on selected relatively undisturbed samples or remolded samples to determine the settlement characteristics of various soil samples, respectively. The results of this test are shown graphically on the appended 'C' Plates.

ATTERBERG LIMITS

Atterberg Limits (ASTM D 4318) tests were performed on selected samples to determine the liquid limit, plastic limit, and the plasticity index of soils. Boring 1 at 15 feet, and Boring 3 at 10 feet has granular sand, therefore non-plastic limits and Atterberg limits cannot be determined. The results of this test are shown graphically on the appended 'E' Plates.

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& Associates

TABLE 1: LABORATORY RESULTS

JOB TITLE: Partner (Santa Ana)
H&A PROJECT NO. 18-2472
SCHEDULED BY: FC
DATE SAMPLES DROPPED OFF: 6/26/2018
DATE ASSIGNED: **6/29/2018**
SHEET: 1 of 2

Test Pit/ Boring	Depth (ft)	Sampler /No. Rings	Field Dry Desnity (pcf)	Field Moisture (%)	Atterberg	Consolidation	Corrosivity Suite	Direct Shear	Expansion Index	Fines Fraction (Minus No. 200)	Hydraulic Conductivity	Maximum Density/ Optimum Moisture	No. 200 Wash / Grain Size Analysis (Particle Size)	Particle Size w/ Hydrometer	R-Value	Reshear (4/7 passes)	Specific Gravity	Sulfate	Triaxial (UU)	Triaxial (CU)	Unconfined Compression	Other	Other	Remarks
B-6	2to5	Bulk																						
B-1	5	R	103.5	19.9	X	X				78.2														
	10	SPT																						
	15	R	112.9	11.9	X	X				42.2														
	20	SPT								15.4														
	25	SPT																						
	30	SPT								27.7														
	35	SPT																						
	40	SPT																						
	45	SPT								41.9														
	50	SPT																						
B-2	5	R	106.2	18.6																				
	10	SPT																						
	15	SPT																						
	20	SPT																						
B-3	5	R	101.3	19.4																				
	10	SPT			X					58.7														
	15	SPT																						

SPECIAL INSTRUCTIONS: See next page

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TABLE 1: LABORATORY RESULTS

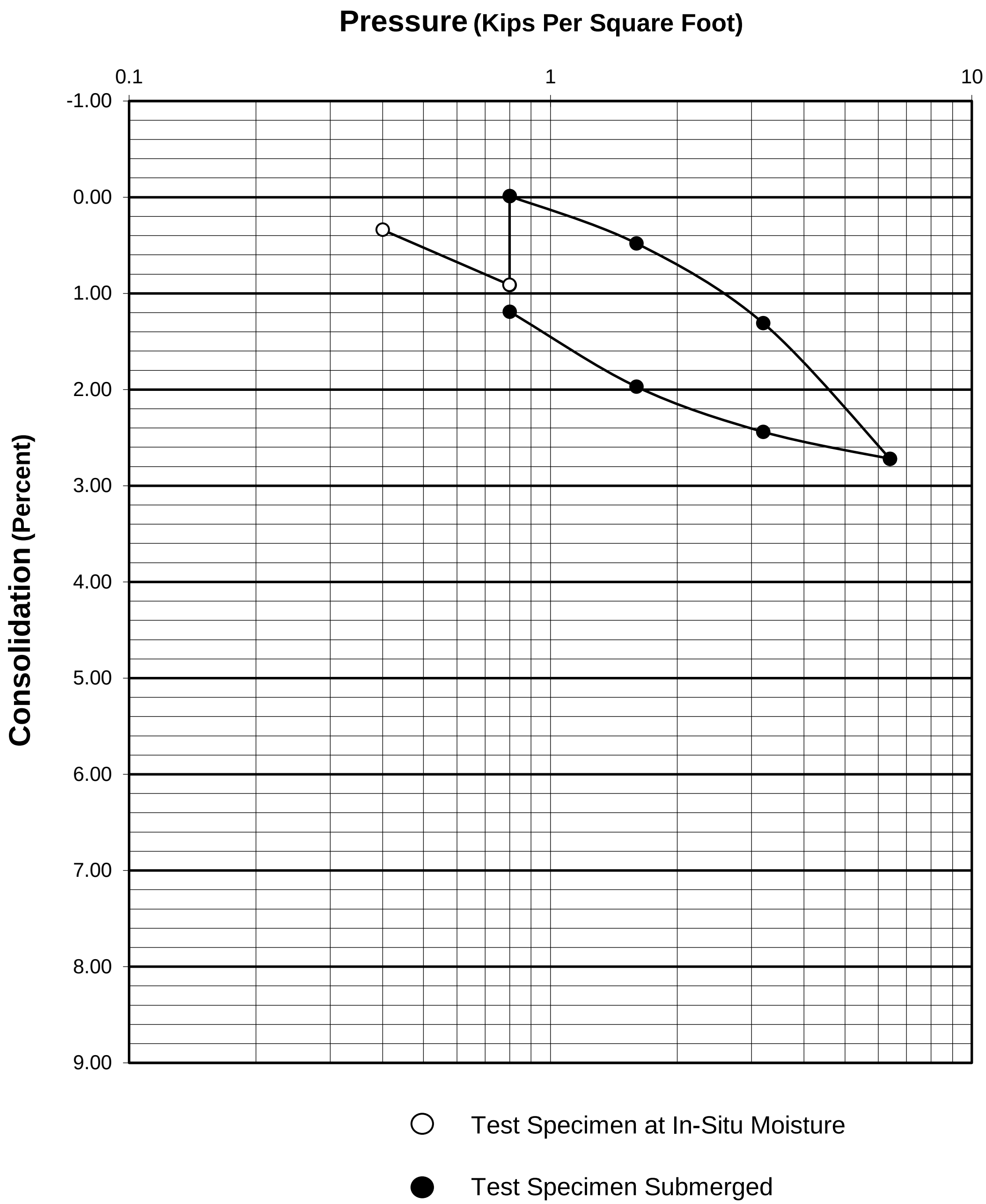
JOB TITLE: Partner (Santa Ana)
H&A PROJECT NO. 18-2472
SCHEDULED BY: FC
DATE SAMPLES DROPPED OFF: 6/26/2018
DATE ASSIGNED: **6/29/2018**
SHEET: 2 of 2

Test Pit/ Boring	Depth (ft)	Sampler /No. Rings	Field Dry Desnity (pcf)	Field Moisture (%)	Atterberg	Consolidation	Corrosivity Suite	Direct Shear	Expansion Index	Fines Fraction (Minus No. 200)	Hydraulic Conductivity	Maximum Density/ Optimum Moisture	No. 200 Wash / Grain Size Analysis (Particle Size)	Particle Size w/ Hydrometer	R-Value	Reshear (4/7 passes)	Specific Gravity	Sulfate	Triaxial (UU)	Triaxial (CU)	Unconfined Compression	Other	Other	Remarks
B-3	20	SPT																						
B-4	5	R	102.2	21.9	X	X				84.5														
	10	SPT																						
	15	SPT																						
	20	SPT																						
B-5	5	R	103.8	17.3																				
	10	SPT																						
	15	SPT								30.2														
	20	SPT																						
B-6	5	SPT								63.4														

SPECIAL INSTRUCTIONS:

CONSOLIDATION TEST RESULTS

B-1 at 5 feet

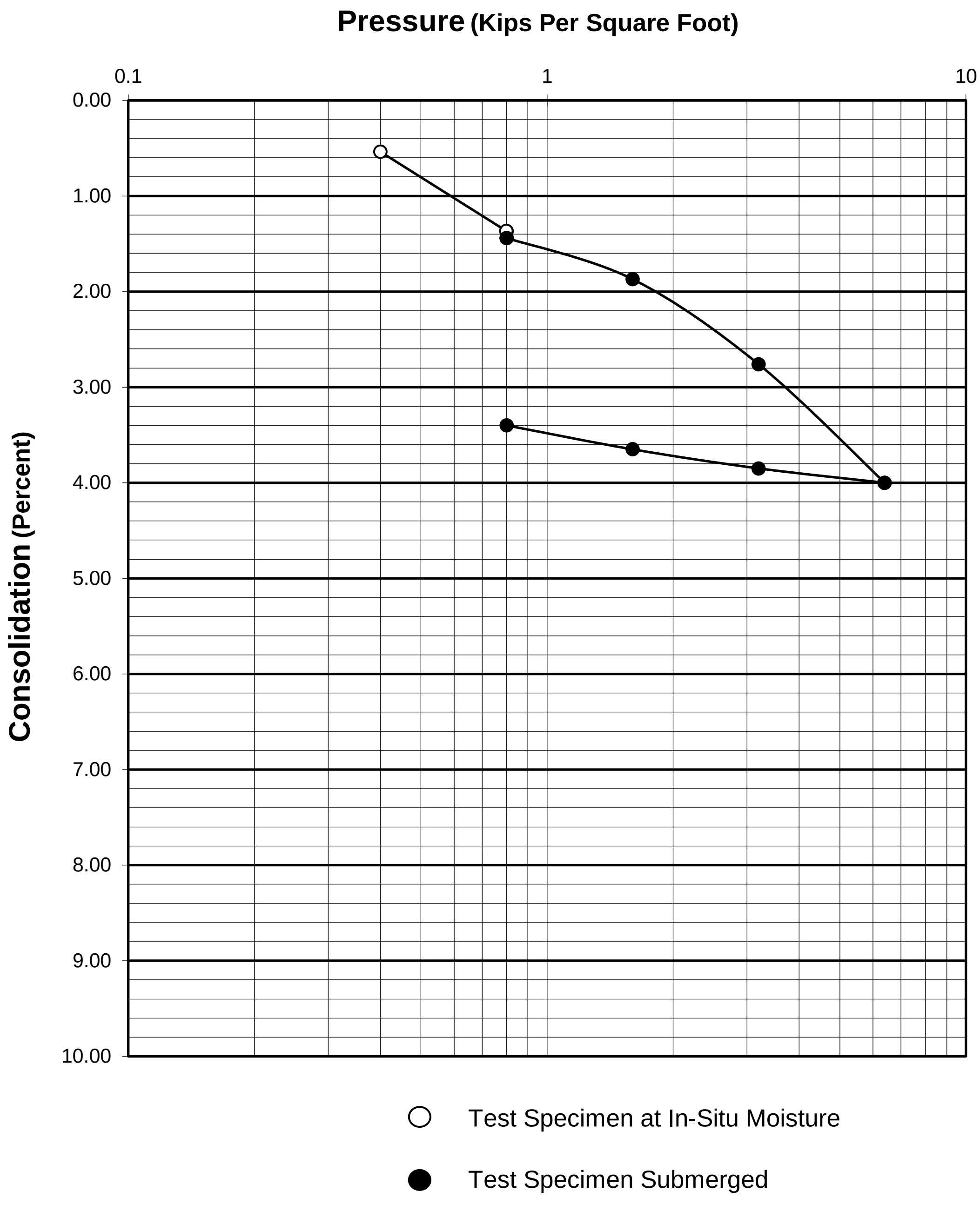


Geotechnical Engineering Investigation 1580 Brookhollow Drive Santa Ana, CA	Project No.	18-2472
	Plate	C-1

HAMILTON & ASSOCIATES, INC.

CONSOLIDATION TEST RESULTS

B-1 at 15 feet

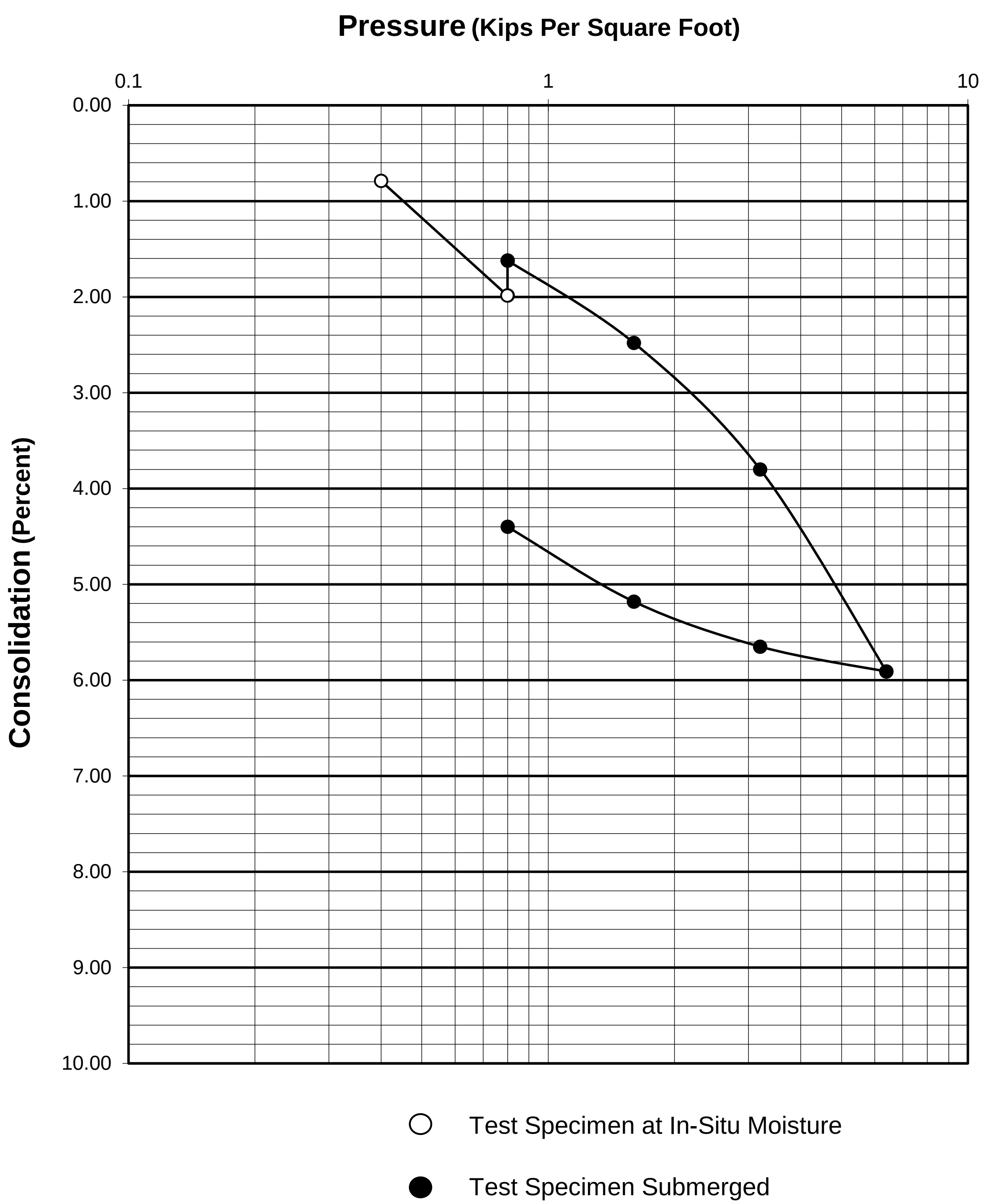


Geotechnical Engineering Investigation 1580 Brookhollow Drive Santa Ana, CA	Project No. 18-2472
	Plate C-2

HAMILTON & ASSOCIATES, INC.

CONSOLIDATION TEST RESULTS

B-4 at 5 feet



Geotechnical Engineering Investigation 1580 Brookhollow Drive Santa Ana, CA	Project No.	18-2472
	Plate	C-3

HAMILTON & ASSOCIATES, INC.

HAMILTON & Associates

ATTERBERG LIMITS
ASTM D4318

Project Name: Partner (Santa Ana)
Project No. : 18-2472
Boring No. : B-1
Sample No. : N/A

Tested By: RM
Checked By:
Depth (ft.): 5
Date: 7/9/2018

Sample Description: Brown lean clay

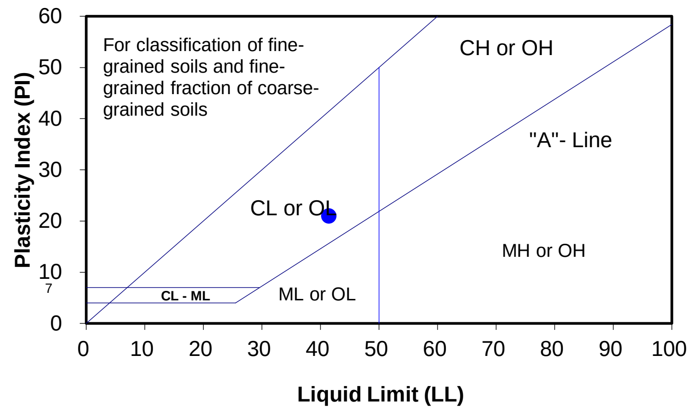
	PLASTIC LIMIT		LIQUID LIMIT			
	1	2	1	2	3	4
Number of Blows [N]:			35	25	20	
Tare No.:	KB-15	KB-16	J-2	P-1	P-9	
Wt. of Tare (gm):	3.19	3.20	16.33	15.23	15.96	
Wet Wt. of Soil + Tare (gm):	8.40	8.60	50.63	48.23	47.33	
Dry Wt. of Soil + Tare (gm):	7.50	7.70	40.90	38.50	38.00	
Moisture Content (%) [Wn]:	20.88	20.00	39.60	41.81	42.33	

Liquid Limit	**41**
Plastic Limit	**20**
Plasticity Index	**21**
USCS Classification	**CL**

PI at "A" - Line = 0.73(LL-20) = 15.63773

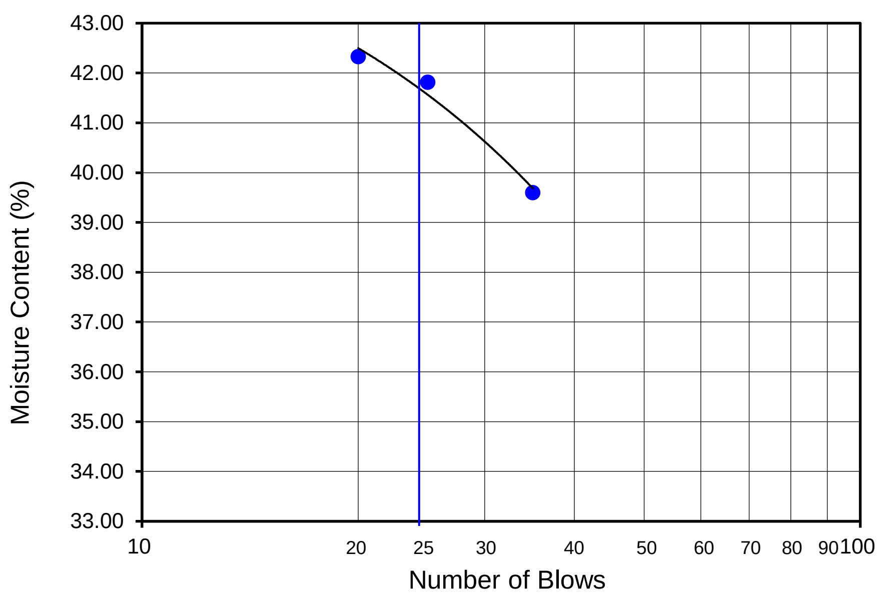
One - Point Liquid Limit Calculation

LL =Wn(N/25)$^{0.121}$



PROCEDURES USED

	Wet Preparation Multipoint - Wet
X	Dry Preparation Multipoint - Dry
X	Procedure A Multipoint Test
	Procedure B One-point Test



ATTERBERG LIMITS
ASTM D4318

HAMILTON & Associates

Project Name: Partner (Santa Ana)
Project No. : 18-2472
Boring No. : B-4
Sample No. : N/A

Tested By: RM
Checked By:
Depth (ft.): 5
Date: 7/9/2018

Visual Sample Description: Brown lean clay

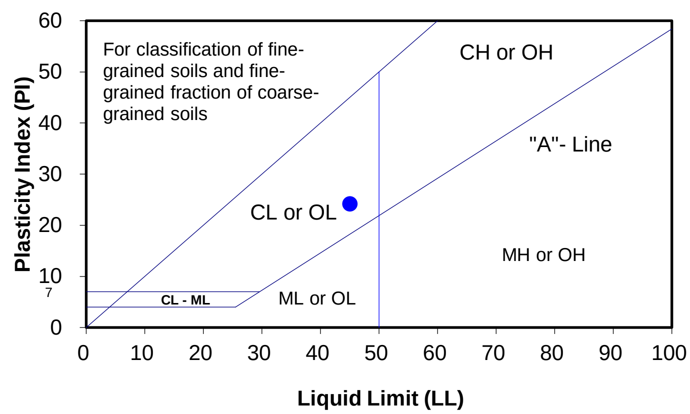
	PLASTIC LIMIT		LIQUID LIMIT			
	1	2	1	2	3	4
Number of Blows [N]:			30	20	16	
Tare No.:	KB-25	AM-5	C-13	Z-92	KB-62	
Wt. of Tare (gm):	3.22	3.20	3.81	3.24	3.22	
Wet Wt. of Soil + Tare (gm):	8.79	8.60	34.59	34.07	33.41	
Dry Wt. of Soil + Tare (gm):	7.80	7.70	25.00	24.40	23.80	
Moisture Content (%) [Wn]:	21.62	20.00	45.26	45.70	46.70	

Liquid Limit	**45**
Plastic Limit	**21**
Plasticity Index	**24**
USCS Classification	**CL**

PI at "A" - Line = 0.73(LL-20) = 18.24753

One - Point Liquid Limit Calculation

LL =Wn(N/25)$^{0.121}$



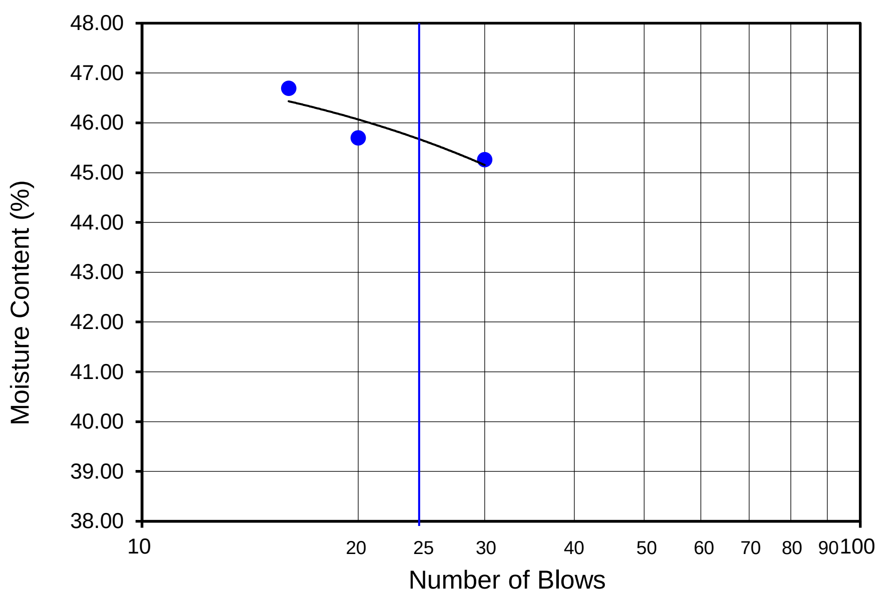
PROCEDURES USED

[ ] Wet Preparation
Multipoint - Wet

[X] Dry Preparation
Multipoint - Dry

[X] Procedure A
Multipoint Test

[ ] Procedure B
One-point Test



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 poritons 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, flyash, 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 tremmie 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 tremmie 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 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

Liquefaction Analysis

PARTNER

Liquefaction Analysis Results

Partner Engineering and Science

Job Title : Hotel
Job No. : 18-217764.1
Client : Partner
Address : Santa Ana, CA
Calculated By : FC/ MM Reviewed By : MM

Borehole : B1
Coordinates:
X = 0
Y = 0
Z = 0

Input Assumption	Setting
Field Test Type :	Standard Penetration Test (SPT)
Apply All Corrections to SPT?	False
Groundwater Level (ft) =	10
Earthquake Magnitude M =	7.2
Magnitude Scaling Factor (MSF) :	1.11 (Idriss, 1997 -NCEER)
Fines Content Correction :	(according to user settings)
Depth Reduction Factor (Rd) :	Idriss 1999, Golesorkhi 1989
Relative Density (Dr) Estimation :	Idriss & Boulanger, 2003
Site Topography :	Gently Sloped : 0.5 %
Ground Improvement Feature :	None
Peak Ground Acceleration PGA (g) =	0.379

CRR Formula	Selected?
NCEER Workshop (1997)	True
Boulanger & Idriss (2014)	True
Vancouver Task Force (2007)	False
Cetin et al. (2004)	False
Chinese Code	False
Seed et al. (1983)	False
Japanese Highway Bridge Code	False
Tokimatsu & Yoshimi (1983)	False
Shibata (1981)	False
Kokusho et al. (1983)	False

Depth (ft)	SPT Blow Counts (N)
5	35
10	19
15	14
20	6
25	7
30	6
35	14
40	24
45	32
50	11

Layer Thickness (ft)	Soil Type	Unit Weight (lb/ft3)	Fines Content (%)	D50 (mm)	Check Liquefaction/ S	Su (ksf)
20	Clay	105	70	0.002	True	500
20	Sand	124	15	2	True	100
10	Clay	105	70	0.002	True	500
5	Sand	124	15	2	True	100

Liquefaction Analysis Results

Partner Engineering and Science

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Client : Partner
Address : Santa Ana, CA
Calculated By : FC/ MM   Reviewed By : MM

Borehole : B1
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Y = 0
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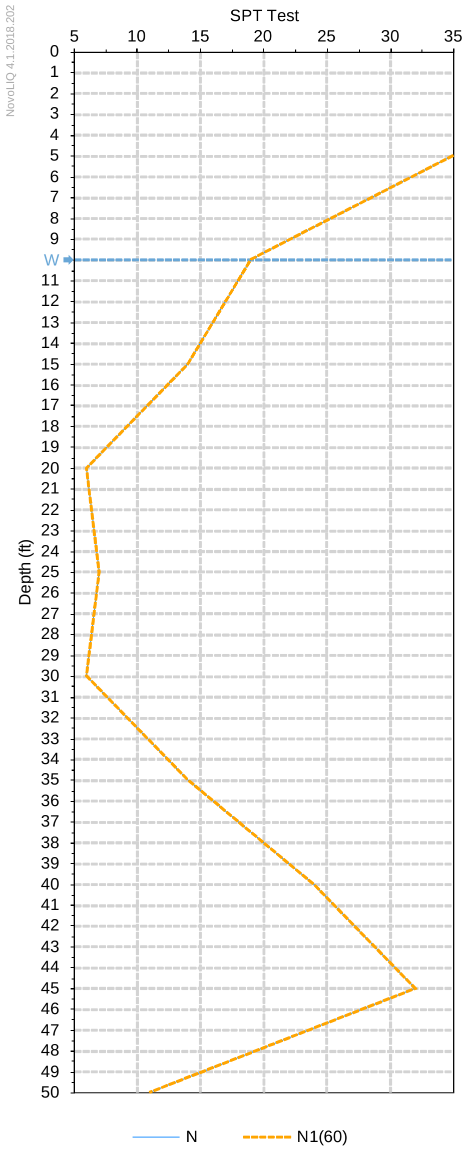

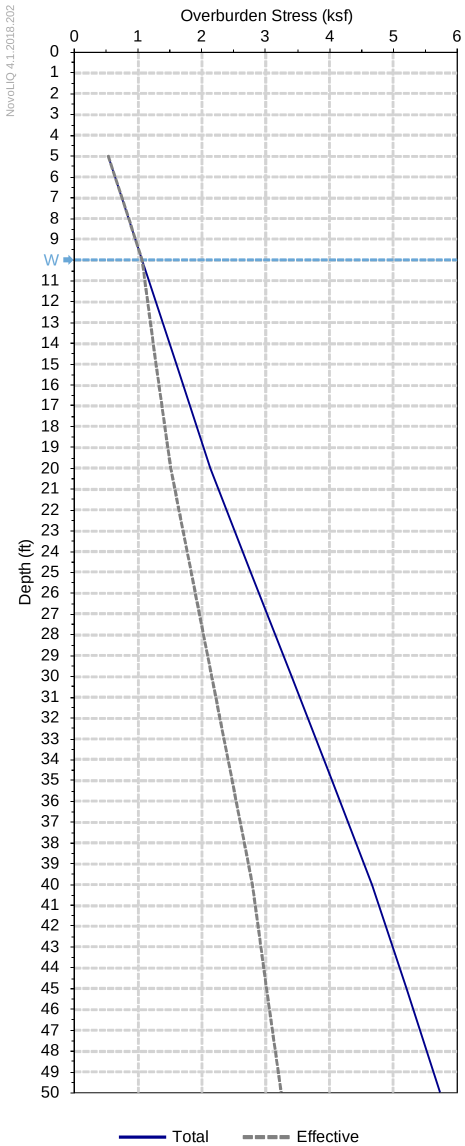

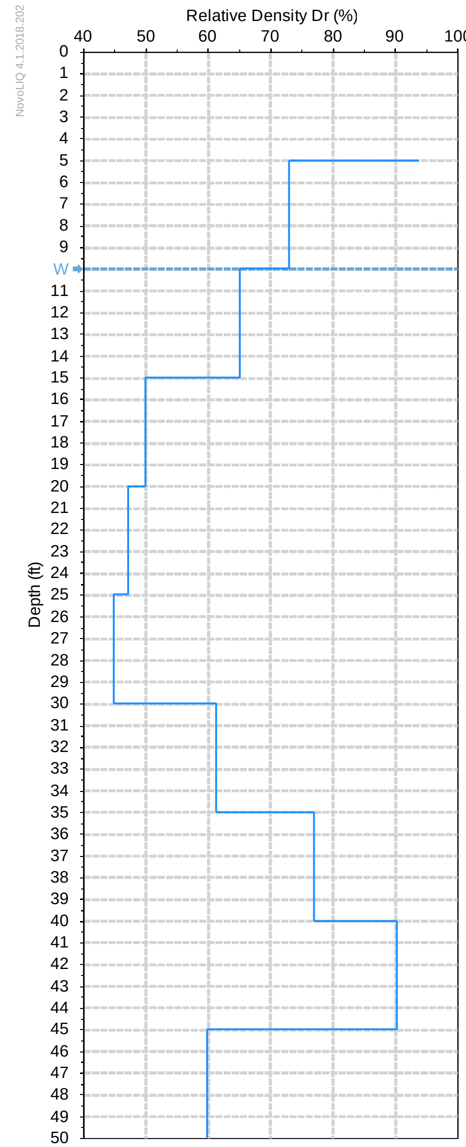

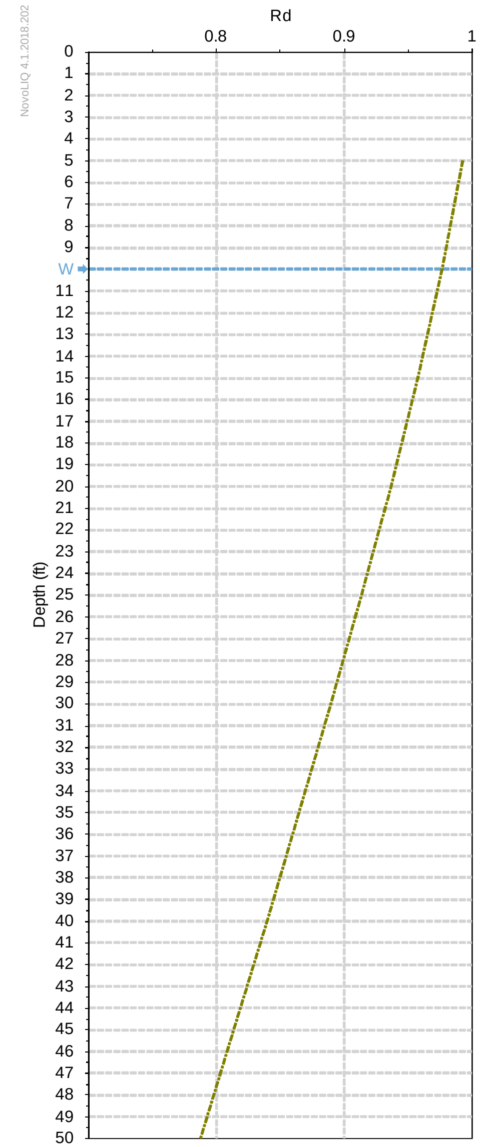

PARTNER

Liquefaction Analysis Results

Partner Engineering and Science

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Borehole : B1
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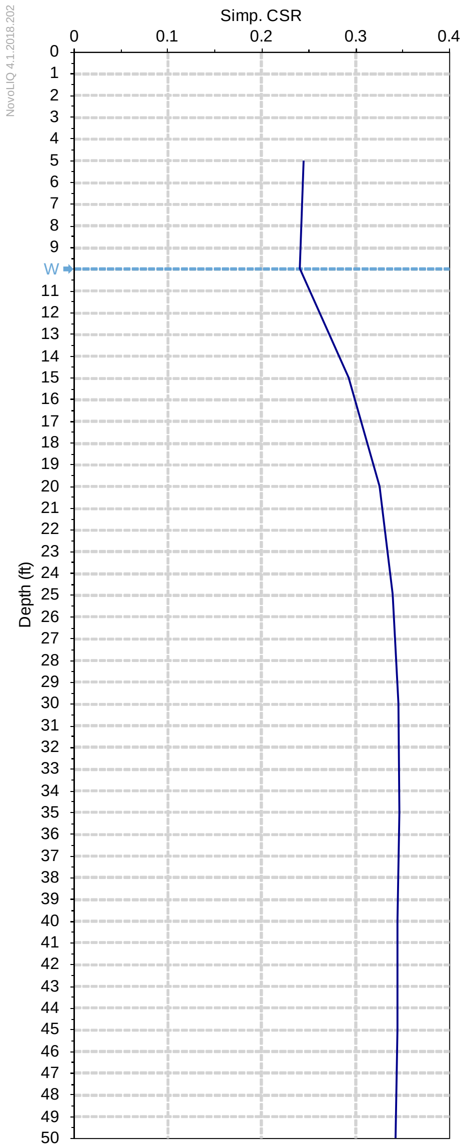
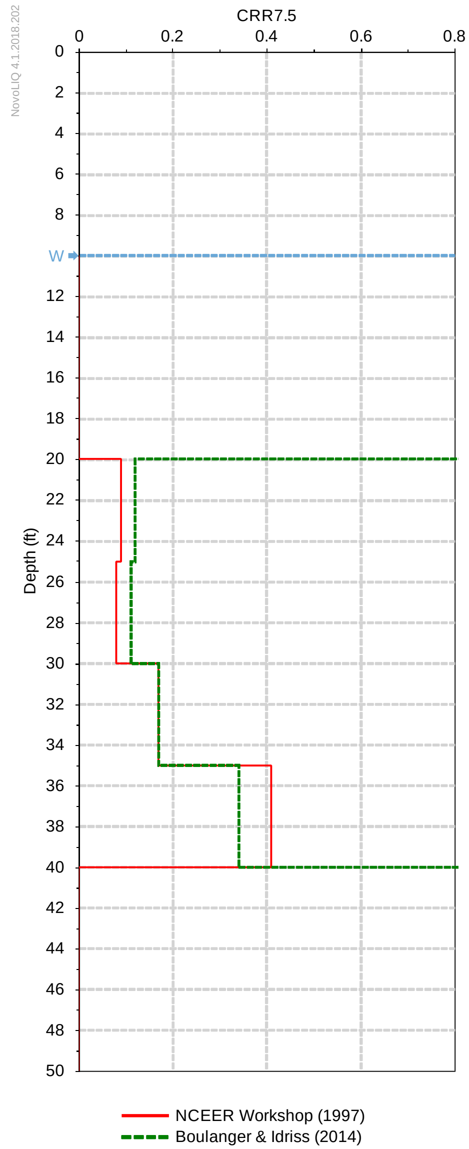
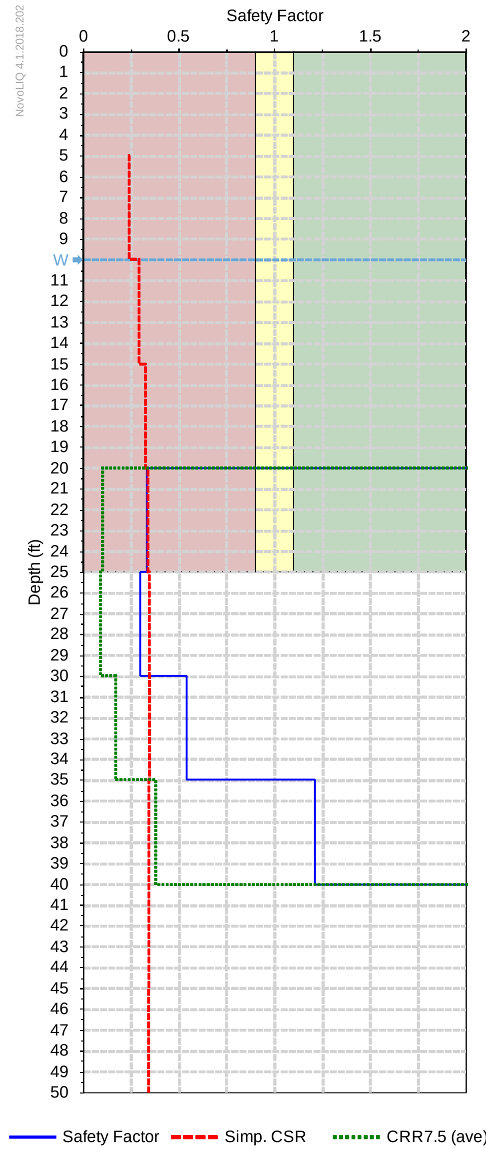
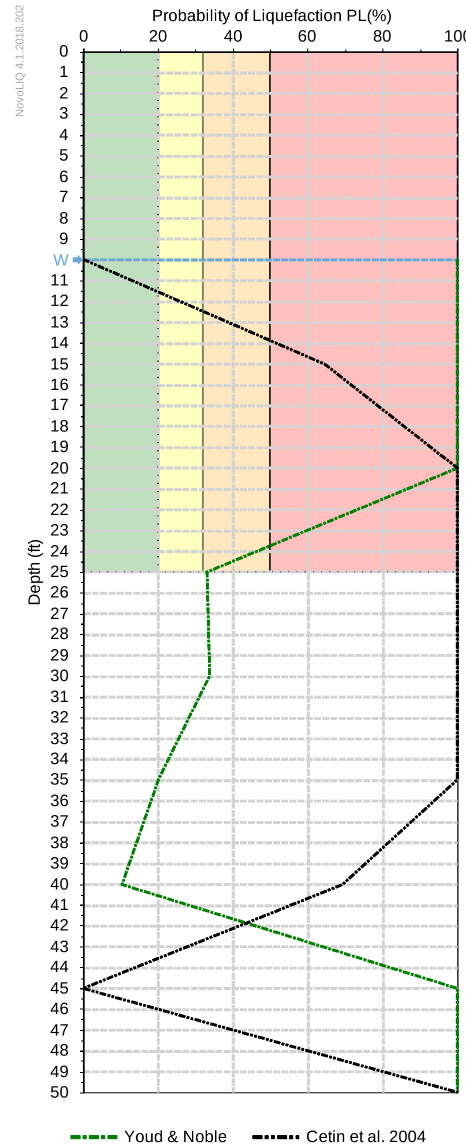
Y = 0
Z = 0









Liquefaction Analysis Results

Partner Engineering and Science

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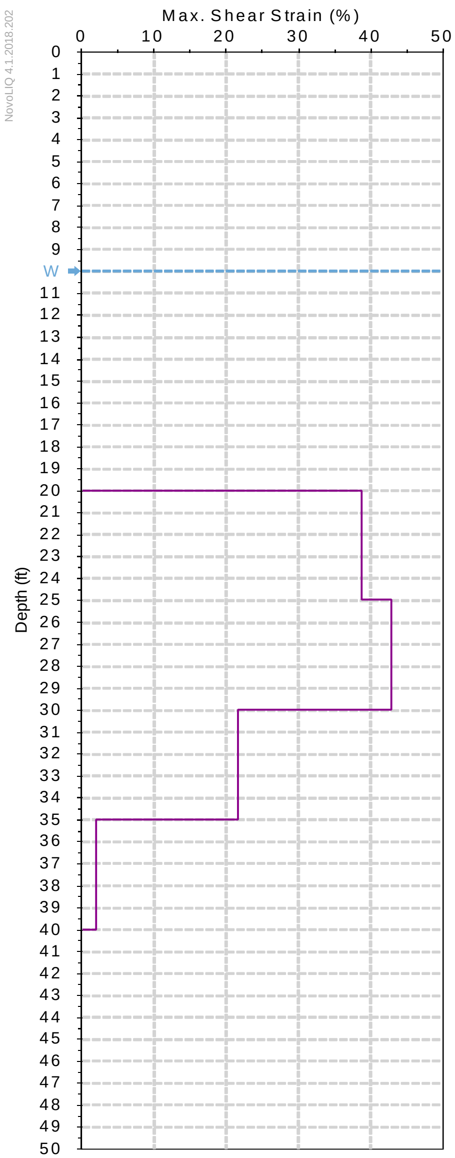
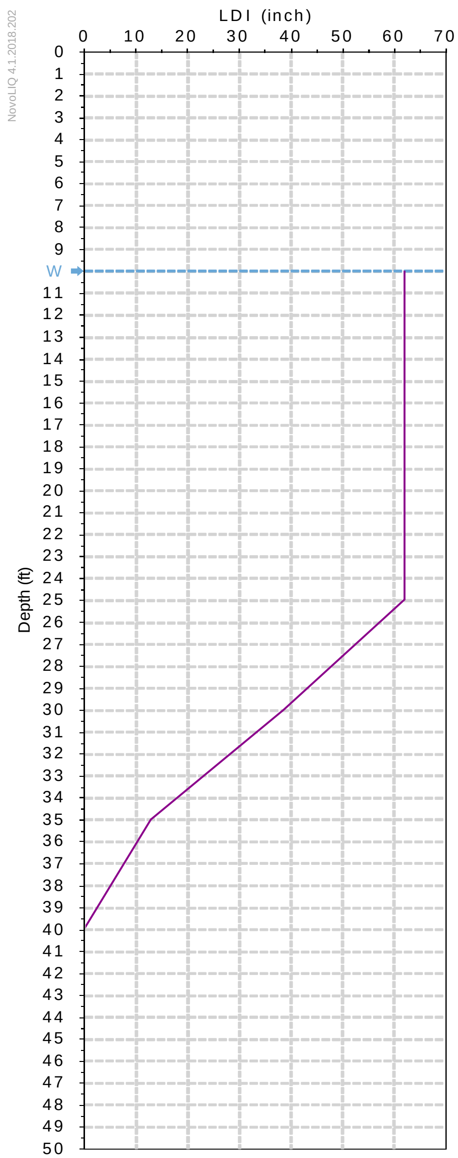
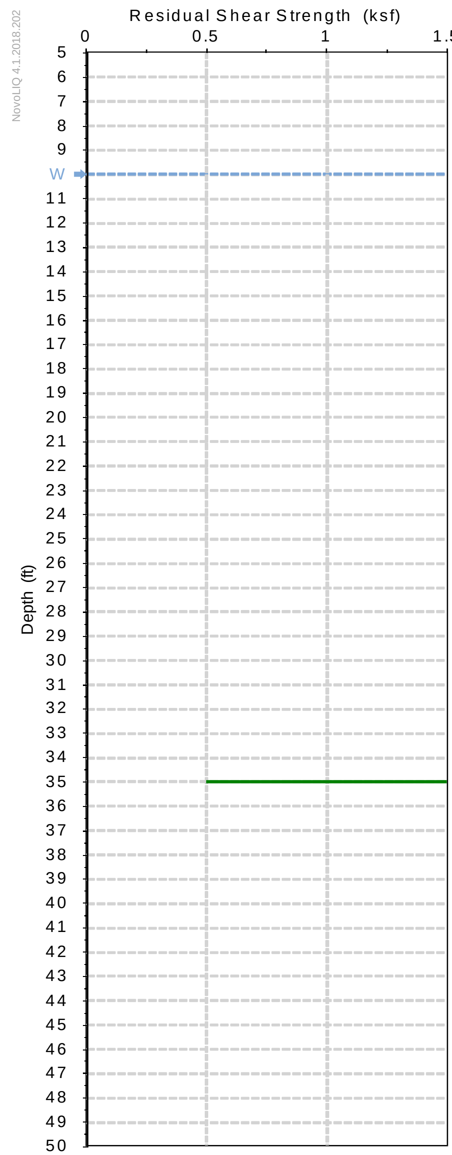
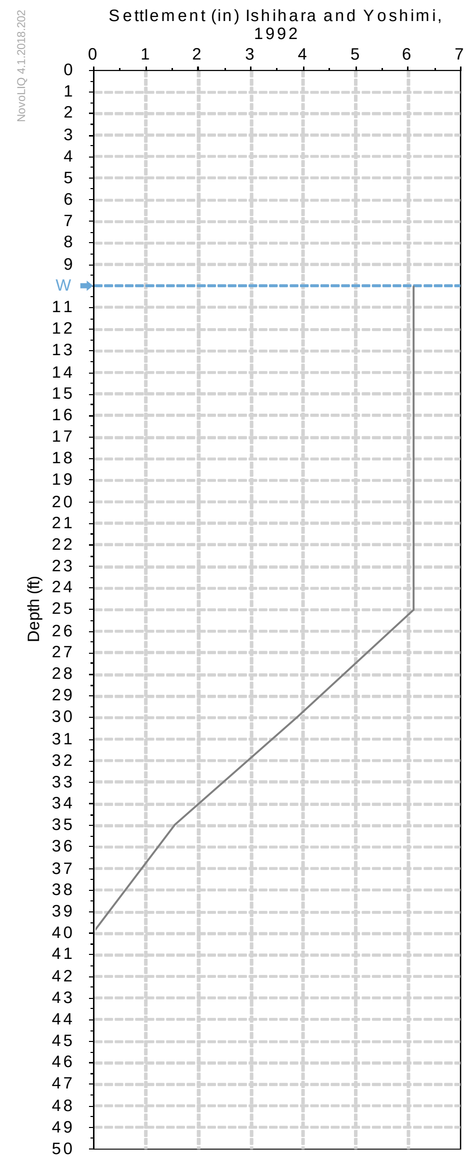








Layer Thickness (ft)	Soil Type	Unit Weight (lb/ft3)	Fines Content	D50 (mm)	Check Liquefaction/	Su (ksf)
20	Clay	105	70	0.002	☑	500
20	Sand	124	15	2	☑	100
10	Clay	105	70	0.002	☑	500
5	Sand	124	15	2	☑	100
					☐	

Depth (ft)	SPT Blow Counts (N)
5	35
10	19
15	14
20	6
25	7
30	6
35	14
40	24
45	32
50	11

Depth (ft)	Rd	Rd_I&B	Overburden Stress (ksf)		Fines Content (%)	SPT Test				Relative Density Dr (%)	Simp. CSR	CSR_I&B	CRR7.5		CRR7.5 (ave)
			Total	Effective		N	Co	Cn	N1(60)				NCEER Worksho	Boulange r & Idriss	
5	0.993	0.993	0.54	0.54	70	35	1	1	35	93.8	0.245	0.245	-	-	-
10	0.977	0.977	1.07	1.07	70	19	1	1	19	73	0.241	0.241	0	373.42	373.42
15	0.958	0.958	1.61	1.29	70	14	1	1	14	65.1	0.293	0.293	0	308.97	308.97
20	0.937	0.937	2.14	1.52	70	6	1	1	6	50	0.326	0.326	0	52.7	52.7
25	0.914	0.914	2.77	1.84	15	7	1	1	7	47.2	0.34	0.34	0.09	0.12	0.1
30	0.89	0.89	3.41	2.16	15	6	1	1	6	44.9	0.346	0.346	0.08	0.11	0.09
35	0.865	0.865	4.04	2.48	15	14	1	1	14	61.3	0.347	0.347	0.17	0.17	0.17
40	0.84	0.84	4.67	2.8	15	24	1	1	24	77	0.345	0.345	0.41	0.34	0.38
45	0.814	0.814	5.21	3.02	70	32	1	1	32	90.3	0.345	0.345	0	132.32	132.32
50	0.788	0.788	5.74	3.25	70	11	1	1	11	59.9	0.343	0.343	0	123.21	123.21

Depth (ft)	Safety Factor		Safety Factor	Probability of Liquefaction PL(%)	
	NCEER Worksho	Boulang er &		Youd & Noble	Cetin et al. 2004
5	-	-	**-**	-	-
10	0	1572.04	**1572.04**	100	0
15	0	1068.61	**1068.61**	100	64.6
20	0	163.88	**163.88**	100	100
25	0.29	0.38	**0.33**	33.1	100
30	0.24	0.35	**0.3**	33.8	100
35	0.54	0.55	**0.54**	20.1	100
40	1.31	1.11	**1.21**	10.4	69.3
45	0	388.17	**388.17**	100	0
50	0	363.44	**363.44**	100	100

Depth (ft)	Zt (in)	Zb (in)	dZ (in)	Lateral Spreading Indexes (in)			Settlement (in) Ishihara and			Residual Strength Sr (ksf)	
				Max. Shear Strain (%)	delta LDI	LDI	Vol. Strain (%)	delta S	S	Lower limit	Upper limit
5			-	-	-	-	-	-	-	-	-
10	120	150	30	0	0	62.03	0	0	6.11	-	-
15	150	210	60	0	0	62.03	0	0	6.11	-	-
20	240	240	0	0	0	62.03	0	0	6.11	-	-
25	270	330	60	38.8	23.3	62.03	3.7	2.21	6.11	0.3	0.3
30	330	390	60	42.9	25.72	38.72	3.9	2.35	3.9	0.3	0.3
35	390	450	60	21.7	13	13	2.6	1.56	1.56	0.5	1.5
40	480	480	0	2.1	0	0	0.5	0	0	-	-
45	510	570	60	0	0	0	0	0	0	-	-
50	570	600	30	0	0	0	0	0	0	-	-

Type	Method	Movement
Lateral Spreading	Zhang, Robertson and Brachman, 2004	43
	Faris, 2006	32
	Youd et al., 2002	11
	Barlett and Youd, 1992	12
	Hamada et al., 1986	61
	Youd and Perkins, 1987	LSI ~52 see details for
Vertical Settlement	Ishihara and Yoshimi, 1992	6

USGS Design Maps Detailed Report

ASCE 7-10 Standard (33.71195°N, 117.84967°W)

Site Class D – "Stiff Soil", Risk Category I/II/III

Section 11.4.1 — Mapped Acceleration Parameters

Note: Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain S_S) and 1.3 (to obtain S_1). Maps in the 2010 ASCE-7 Standard are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 11.4.3.

From Figure 22-1 [1]	S_S = 1.509 g
From Figure 22-2 [2]	S_1 = 0.558 g

Section 11.4.2 — Site Class

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class D, based on the site soil properties in accordance with Chapter 20.

Table 20.3–1 Site Classification

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard Rock	>5,000 ft/s	N/A	N/A
B. Rock	2,500 to 5,000 ft/s	N/A	N/A
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff Soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the characteristics: • Plasticity index *PI* > 20, • Moisture content *w* ≥ 40%, and • Undrained shear strength $\bar{s}_u$ < 500 psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

Section 11.4.3 — Site Coefficients and Risk–Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameters

Table 11.4–1: Site Coefficient F_a

Site Class	Mapped MCE_R Spectral Response Acceleration Parameter at Short Period				
	$S_S \le 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_S

For Site Class = D and S_S = 1.509 g, F_a = 1.000

Table 11.4–2: Site Coefficient F_v

Site Class	Mapped MCE_R Spectral Response Acceleration Parameter at 1–s Period				
	$S_1 \le 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \ge 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_1

For Site Class = D and S_1 = 0.558 g, F_v = 1.500

Equation (11.4–1): $S_{MS} = F_aS_S = 1.000 \times 1.509 = 1.509$ g

---

Equation (11.4–2): $S_{M1} = F_vS_1 = 1.500 \times 0.558 = 0.837$ g

---

Section 11.4.4 — Design Spectral Acceleration Parameters

Equation (11.4–3): $S_{DS} = ⅔\ S_{MS} = ⅔ \times 1.509 = 1.006$ g

---

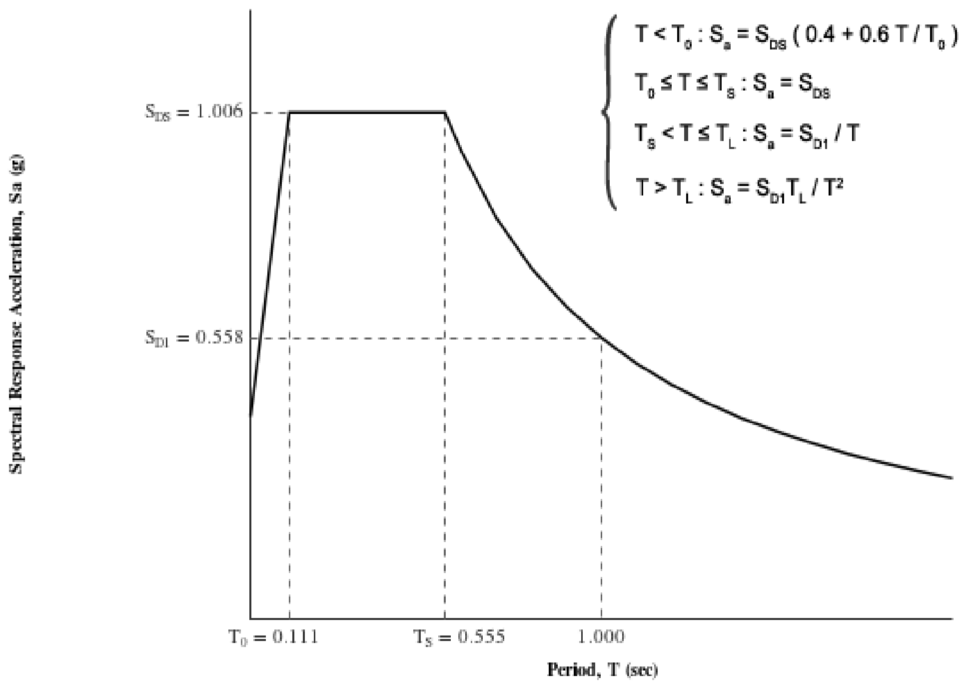
Equation (11.4–4): $S_{D1} = ⅔\ S_{M1} = ⅔ \times 0.837 = 0.558$ g

---

Section 11.4.5 — Design Response Spectrum

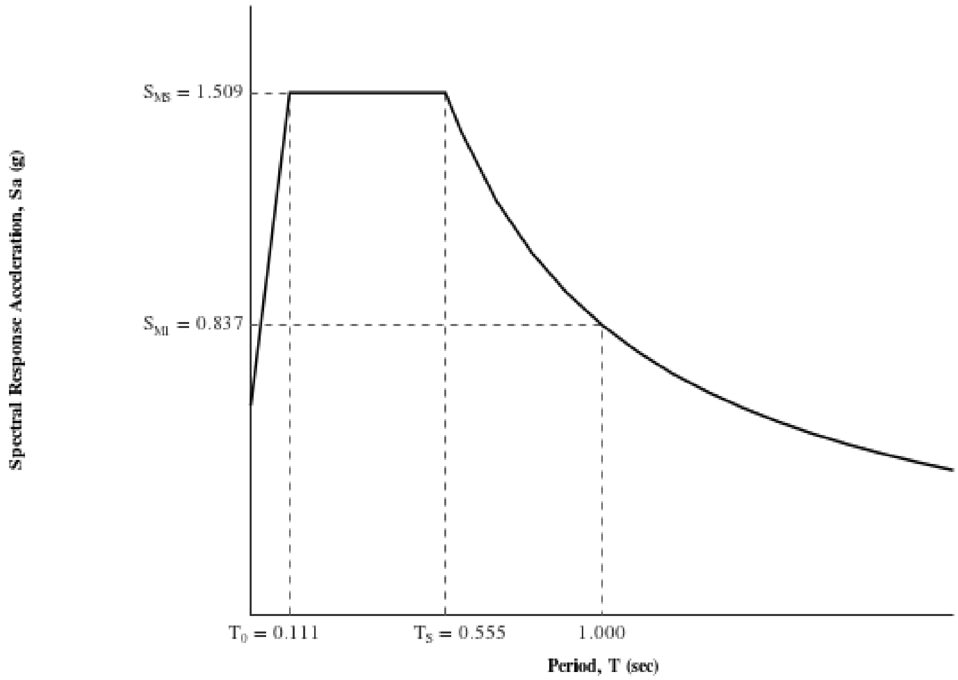
From Figure 22-12 [3] T_L = 8 seconds

Figure 11.4–1: Design Response Spectrum



Section 11.4.6 — Risk-Targeted Maximum Considered Earthquake (MCE_R) Response Spectrum

The MCE_R Response Spectrum is determined by multiplying the design response spectrum above by 1.5.



Section 11.8.3 — Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F

From Figure 22-7 [4] PGA = 0.565

Equation (11.8–1): $PGA_M = F_{PGA}PGA = 1.000 \times 0.565 = 0.565$ g

Table 11.8–1: Site Coefficient F_{PGA}

Site Class	Mapped MCE Geometric Mean Peak Ground Acceleration, PGA				
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA ≥ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of PGA

For Site Class = D and PGA = 0.565 g, F_{PGA} = 1.000

Section 21.2.1.1 — Method 1 (from Chapter 21 – Site-Specific Ground Motion Procedures for Seismic Design)

From Figure 22-17 [5] C_{RS} = 1.000

From Figure 22-18 [6] C_{R1} = 1.034

Section 11.6 — Seismic Design Category

Table 11.6-1 Seismic Design Category Based on Short Period Response Acceleration Parameter

VALUE OF S_{DS}	RISK CATEGORY		
	I or II	III	IV
S_{DS} < 0.167g	A	A	A
0.167g ≤ S_{DS} < 0.33g	B	B	C
0.33g ≤ S_{DS} < 0.50g	C	C	D
0.50g ≤ S_{DS}	D	D	D

For Risk Category = I and S_{DS} = 1.006 g, Seismic Design Category = D

Table 11.6-2 Seismic Design Category Based on 1-S Period Response Acceleration Parameter

VALUE OF S_{D1}	RISK CATEGORY		
	I or II	III	IV
S_{D1} < 0.067g	A	A	A
0.067g ≤ S_{D1} < 0.133g	B	B	C
0.133g ≤ S_{D1} < 0.20g	C	C	D
0.20g ≤ S_{D1}	D	D	D

For Risk Category = I and S_{D1} = 0.558 g, Seismic Design Category = D

Note: When S_1 is greater than or equal to 0.75g, the Seismic Design Category is **E** for buildings in Risk Categories I, II, and III, and **F** for those in Risk Category IV, irrespective of the above.

Seismic Design Category ≡ "the more severe design category in accordance with Table 11.6-1 or 11.6-2" = D

Note: See Section 11.6 for alternative approaches to calculating Seismic Design Category.

References

1. *Figure 22-1*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-1.pdf
2. *Figure 22-2*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-2.pdf
3. *Figure 22-12*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-12.pdf
4. *Figure 22-7*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-7.pdf
5. *Figure 22-17*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-17.pdf
6. *Figure 22-18*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-18.pdf