Appendix G

Geotechnical Engineering Investigation

DRAFT

Geotechnical Engineering Report

Proposed 12-Story Office Building
12th and O Street
Sacramento, California

December 23, 2016

Terracon Project No. NB16P135

Prepared for:
HGA Architects and Engineers
Sacramento, California

Prepared by:
Terracon Consultants, Inc.
Sacramento, California

December 23, 2016

HGA Architects and Engineers
1200 R Street, Suite 100
Sacramento, Ca 95811

Attn: Ms. Beth Young, AIA
E: BYoung@hga.com

Re: DRAFT - Geotechnical Engineering Report
Proposed 12-Story Office Building
12th and O Street
Sacramento, California
Terracon Project No. NB16P135

Dear Ms. Young,

Terracon Consultants, Inc. (Terracon) has completed the geotechnical engineering services for the above referenced project. These services were performed in general accordance with the proposal (PNB165146) dated September 28, 2016.

This geotechnical engineering report presents the results of the subsurface exploration and provides geotechnical engineering recommendations concerning earthwork and the design and construction of foundations and floor slabs for the proposed project.

We appreciate the opportunity to be of service to you on this project. If you have any questions concerning this report, or if we may be of further service, please contact us.

Sincerely,
Terracon Consultants, Inc.

F. Fred Buhamdan, P.E.
Geotechnical Department Manager

Rob Holmer, G.E.
Principal Engineer

REH:FFB:NNN:DAB

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EXECUTIVE SUMMARY

A geotechnical exploration has been performed for the proposed 12-Story office building project to be located at 12th and O Street, Sacramento, California. Terracon's geotechnical scope of work included the advancement of four (4) test borings to approximate depths ranging between 18½ and 101½ feet below the ground surface (bgs), three (3) CPT soundings advanced to approximately 17½ feet bgs, and a multichannel analysis of surface wave (MASW) study. Our investigation also included laboratory testing on representative samples of the subsurface materials, engineering analyses, and development of engineering recommendations for design and construction of foundations and floor slabs.

The site appears suitable for the proposed construction based upon geotechnical conditions encountered in the test borings and provided our recommendations contained in this report are properly implemented in the design and construction. Based on the information obtained from our engineering analyses of the field and laboratory data, the following geotechnical considerations were identified:

- The existing 4-story building will be demolished to accommodate the proposed construction. It is our understanding that existing building is supported on deep foundations. For existing piles that do not overlap with new pile foundations and/or caps, the existing pile foundations shall be excavated down to the bottom of the basement excavation, , and then cut off and abandoned in place. Terracon should be notified if new foundations will overlap with existing piles, Mitigation measures including complete removal of existing piles may be necessary to avoid differential movements and provide homogenous subsurface conditions beneath pile caps.

- Based on results of the borings, fill materials were encountered in all borings to approximate depth of 6½ to 7½ feet bgs. The subsurface conditions on the project site can be generalized as medium stiff to stiff silt with variable sand to a depth of approximately 31 to 33 feet underlain by medium dense to dense gravel and sand soils to a depth of 43 to 45 feet. Gravel and sand soils were underlain by very stiff to dense sand and silt soils to a depth of 68 to 70 feet. Sand and silt soils were underlain by very stiff to hard fat clay with variable sand to a depth of 85 to 89 feet, which in turn was underlain by hard to very hard silt to sandy silt to the maximum depth of exploration of 101½ feet bgs.

- Groundwater was observed in our borings at the time of field exploration at a depth ranging between 17½ feet and 18 feet below existing ground surface. Groundwater levels across the site are anticipated to fluctuate relative to the water levels of the nearby Sacramento River. This project is planned to include a one-level below grade parking garage with an anticipated excavation depth of 15 feet bgs. We anticipate that seasonal water levels and/or temporary periodic events will cause rise in the groundwater to levels above the garage depth. Cut off walls and/or dewatering systems may be required. Additional information is required to determine the impact that groundwater will have on construction of the proposed structure. We recommend the installation of monitoring wells equipped with data recorders to aid in measuring the seasonal or otherwise temporary fluctuations of groundwater at this site.

- Fill materials consisting of poorly graded sand to silt with variable sand were encountered across the site to a depth of 6½ to 7½ feet. It is our assumption that the upper fill materials encountered in the borings were placed during historical grading operations and construction of the existing buildings.

Terracon does not have any documentation to show if the grading operations were monitored, and if the backfill activities were controlled or inspected. Therefore, the encountered fill materials are considered undocumented and should not be used to support the proposed structure. We anticipate these fill materials will be removed during excavation of the parking garage, and therefore, will not impact the proposed development.

- The subgrade soils in the upper 27 feet are highly compressible, have low bearing capacity, and are susceptible to settlement under expected structural loading. For this reason, we recommend that the building foundation penetrate through the weak upper silts and derive their support from the dense to hard soils encountered below a depth of 35 feet. A considerable increase in density and shear strength, and therefore a considerable increase in the capacity of the foundation, was encountered at a depth of 90 feet bgs which will provide adequate support for the proposed 12-story structure.

- We investigated the use of several different foundation systems to support the building including a mat slab, drilled shafts, driven pre-cast piles, and auger cast piles. Due to the shallow groundwater, the weak and compressible soils in the upper 27 feet, and the heavy loads anticipated from the building, we believe an auger cast or auger displacement pile foundation is most appropriate to support the building. Pre-cast driven piles are also a suitable option, however, we anticipate they may not be selected due to the excessive vibration and noise associated with their installation. Our office is available to discuss additional foundation solutions if proposed by a project bidder.

- Lightly loaded detached tertiary structures (such as landscaping walls or equipment sheds) may be supported on shallow spread footings that bear on a minimum 24 inches of engineered fill comprised of recompacted onsite fill/native materials.

- Lateral loads may be supported by a combination of passive resistance of the basement excavation, frictional resistance of the basement foundation walls, lateral loading on the piles, and uplift of the piles. There is an existing building adjacent to this site that is set back approximately 10 feet from the new proposed building. The structural engineer should consider whether it is appropriate to rely on passive resistance adjacent to this building.

- The on-site native soils and existing fill materials are considered suitable for use as engineered fill on the project.

- The subgrade soils immediately beneath the existing structure and at the depth of the garage excavation are very moist and soft and will become unstable and pump under the weight of heavy equipment. Stabilization of the subgrade soils, using gap graded crushed stone, reinforcing geogrid, and/or chemical or cement treatment should be anticipated.

- Earthwork on the project should be observed and evaluated by Terracon. The evaluation of earthwork should include observation and testing of engineered fill, subgrade preparation, slab bearing soils, and other geotechnical conditions exposed during construction.

This geotechnical executive summary should be used in conjunction with the entire report for design and/or construction purposes. It should be recognized that specific details were not included or fully

Commented [HF1]: These are ground improvement methods, not foundation systems

Commented [HF2]: yoi dont discuss this subgrade preparation anywhere else in the report developed in this section, and the report must be read in its entirety for a comprehensive understanding of the items contained herein. The section titled General Comments should be read for an understanding of the report limitations.

GEOTECHNICAL ENGINEERING REPORT
PROPOSED 12-STORY OFFICE BUILDING
12TH AND O STREET
SACRAMENTO, CALIFORNIA

Terracon Project No. NB16P135
December 23, 2016

1.0 INTRODUCTION

This report presents the results of our geotechnical engineering services performed for the proposed 12-Story Office Building project located at 12th and O Street in Sacramento, California. The Site Location Plan (Exhibit A-1) is included in Appendix A of this report. The purpose of these services is to provide information and geotechnical engineering recommendations relative to:

- subsurface soil conditions
- earthwork
- seismic considerations
- floor slab design and construction
- groundwater conditions
- foundation design and construction
- pavement design and construction
- below grade design and construction

Our geotechnical engineering scope of work included the advancement of four (4) test borings to approximate depths ranging between 18 and 101½ feet bgs. Additionally, three (3) CPT soundings were advanced to approximately 17½ feet bgs, and a multichannel analysis of surface waves (MASW) study was performed adjacent to the existing building.

Logs of the borings along with a Boring Location Diagram (Exhibit A-2) are included in Appendix A of this report. The results of the laboratory testing performed on soil samples obtained from the site during the field exploration are included in Appendix B of this report. Descriptions of the field exploration, laboratory testing, and analysis are included in their respective appendices.

2.0 PROJECT INFORMATION

2.1 Project Description

ITEM	DESCRIPTION
Site layout	Refer to the Boring Location Diagram, Exhibit A-2.
Structures	The site is currently considered for a 12-story office building with one level of below grade parking. The new building will have a maximum height of 150 feet and a footprint of approximately 30,000 square-feet. We understand the project structural engineer is currently considering a reinforced concrete frames for the proposed structure.

ITEM	DESCRIPTION
Below Grade Structures	The building is planned to include one level of below grade parking with an anticipated excavation depth of 15 feet. Due to shallow groundwater levels at the site, temporary shoring of the basement excavation with slurry cut-off walls and/or a permanent dewatering system are anticipated.
Finished floor elevation	Within one foot of existing grade.
Maximum loads (assumed)	Column load – 800 to 1,200 kips. Continuous Wall Load – 5 to 10 klf. Maximum Uniform Floor Slab Load – 150 psf.
Grading	Grading will include excavation of the existing building pad to accommodate the proposed below grade parking garage.

2.2 Site Location and Description

Item	Description
Location	The site is located at the northeast corner of 12th and O Street in Sacramento, California.
Existing Improvements	The site is currently improved with an existing 4-story, concrete office building that was originally constructed in the 1950's. Original construction drawings indicate the building is founded on driven concrete piles. The existing building will be demolished to accommodate the proposed new building. There is a multi-story bridge that spans over the alley and connects the1215 O Street building with the building located at 1220 N Street to the adjacent north of the site. We understand this bridge will remain, as it carries utilities between the buildings and provides requisite fire egress.
Surrounding developments	The site is located in a highly developed downtown Sacramento urban area, and is bordered to the north and east by multi-story office buildings, to the west by 12th street and to the south by O street followed by a parking lot.
Current ground cover	The majority of the site is covered by the existing building, surrounded by concrete flatwork, pavements, and landscaping areas.
Existing topography	The site is relatively flat.

3.0 SUBSURFACE CONDITIONS

A description of our field exploration is presented in Appendix A. Laboratory tests were conducted on selected soil samples obtained during our exploration. A description of the laboratory testing is presented in Appendix B.

3.1 Site Geology

The site is situated within the Great Velley Geomorphic Provence of Northern California. Geologic structures within this Provence are dominated by alluvial floodplain processes. The Great Valley Geomorphic Provence extends south to the Transverse Range, and is bounded by the Sierra Nevada Mountains to the east, and the Coast Range to the west, and the Klamath Mountains to the north.[1, 2] Surficial geologic units mapped at the site consists of Alluvial deposits of recent Quaternary age[3].

3.2 Typical Subsurface Profile

Specific conditions encountered at the boring locations are indicated on the individual boring logs. Stratification boundaries on the boring logs represent the approximate location of changes in soil types; in-situ, the transition between materials may be gradual. Details for the borings can be found on the boring logs included in Appendix A. Based on the results of the borings, the subsurface conditions on the project site can be generalized as follows:

Description	Approximate Depth to Bottom of Stratum	Material Encountered	Consistency/Density
Surface Materials	9 to 10 inches 3 feet 6½ to 7½ feet	Concrete Floor Slab (Interior) Fill: Poorly Graded Sand with Silt Fill: Silt with Variable Sand	-- Loose to Medium Dense Medium Stiff to Stiff
Stratum 1	31 to 33 feet	Silt with Variable Sand	Medium Stiff to Stiff
Stratum 2	43 to 45 feet	Sand with Variable amounts of Silt and Gravel	Medium Dense to Very Dense
Stratum 3	50 feet	Silt with Variable Sand	Very Stiff
Stratum 4	68 to 70 feet	Silty Sand	Medium Dense to Dense
Stratum 5	85 to 89 feet	Fat Clay with Variable Sand	Very Stiff to Hard
Stratum 3	101½ (maximum depth of exploration)	Silt with Variable Sand	Hard to Very Hard

1 Harden, D. R., "*California Geology, Second Edition*," Pearson Prentice Hall, 2004.

2 Norris, R. M. and Webb, R. W., *"Geology of California, Second Edition,"* John Wiley & Sons, Inc., 1990.

3 *Helly, E.J., "Preliminary Geologic Map of Cenozoic Deposits of the Davis, Knights Landing, Lincoln, and Fair Oaks Quadrangles, California", by the U.S. Geological Survey, Report OF-79-583, Dated 1979, Scale 1:62,500.*

Laboratory tests were conducted on selected soil samples and the test results are presented in Appendix B and on the boring logs. Atterberg limit test results indicate that on-site silt materials within the upper 68 to 70 feet exhibit non-plastic behavior. Fat clay encountered from below a depth of 68 to 70 feet exhibited highly plastic behavior. Consolidation test results indicate that silt soils encountered in the upper 27 feet exhibit normally consolidated behavior when saturated under expected structural loads.

3.3 Groundwater

Groundwater was observed in our borings at the time of field exploration at a depth ranging between 17½ and 18 feet below existing ground surface. These observations represent groundwater conditions at the time of the field exploration and may not be indicative of other times, or at other locations. Groundwater conditions can change with varying seasonal and weather conditions, and other factors.

Groundwater levels across the site are anticipated to fluctuate relative to the water levels of the Sacramento River and could potentially rise to basement level depth. Additional information may be required to determine the impact that groundwater will have on construction of the proposed structure. We recommend the installation of monitoring wells equipped with data recorders to aid in measuring the seasonal or otherwise temporary fluctuations groundwater at this site.

In silt soils with lower permeability, the accurate determination of groundwater level may not be possible without long term observation. Long term observation after drilling could not be performed as borings were backfilled immediately upon completion due to safety concerns and county guidelines. Groundwater levels can best be determined by implementation of a groundwater monitoring plan. Such a plan would include installation of groundwater monitoring wells, and periodic measurement of groundwater levels over a sufficient period of time.

3.4 Cone Penetration Testing

Cone penetration test (CPT) soundings were performed at the site by California Push Technologies, Inc. on November 3rd, 2016. CPT soundings were advanced at three (3) locations within the existing buildings to approximate depths of 17 to 17½ feet bgs, where penetrometer refusal was encountered on cemented soils. Subsequent drilling inside the boring encountered this cemented section at similar depths. The CPT report is included as Exhibit C-4 of this report.

3.5 Multi-Channel Analysis of Surface Waves

A multi-channel analysis of surface waves (MASW) survey was conducted at the site by NorCal Geophysics Inc. on November 23rd, 2016. The MASW survey was conducted on the west side of the existing 4-story building and was oriented parallel to 12th street with the center point of the survey approximately 25-feet south of Boring No. B-1. The MASW method is used to determine the seismic velocity of S-waves in subsurface materials. The MASW report is included as Exhibit C-5 of this report.

4.0 SITE SPECIFIC GROUND MOTION STUDY

4.1 Probabilistic Seismic Hazard Analysis

A probabilistic seismic hazard analysis (PSHA) considers all potential earthquake sources that may contribute to strong ground shaking at a specific site. Magnitude, distance, and probability of occurrence are all factored into the computation. We used data from the U.S. Geological Survey (USGS) Earthquake Hazards Program (http://geohazards.usgs.gov/deaggint/2008/) to estimate the ground motions. The ground motions were geometric mean (geomean) spectral acceleration values (5 percent critical damping) with a 2 percent probability of exceedance in 50 years, corresponding to a return period of 2,475 years. Based on the results of the shear wave velocity measurements at the site, we selected values for material with a shear wave velocity of 1,750 feet/second (i.e., Site Class C). The geomean, uniform hazard values were converted to maximum rotated component (MRC), uniform risk, maximum considered earthquake, (i.e., ASCE 7-10 MCE_R) values by multiplying the spectral accelerations by the factors listed in the following table.

Period (seconds)	MCE Spectral Acceleration (g)	MRC Factor	Risk Coefficient (C_R)	MCE_R Spectral Acceleration (g)
0.0	0.258	1.1	1.109	0.314
0.1	0.513	1.1	1.109	0.625
0.2	0.622	1.1	1.109	0.759
0.3	0.562	1.1	1.111	0.687
0.5	0.426	1.1	1.115	0.522
1.0	0.257	1.3	1.125	0.376
2.0	0.140	1.3	1.125	0.205
3.0	0.087	1.3	1.125	0.128
4.0	0.060	1.3	1.125	0.087

Deaggregation of the hazard generally identified gridded background seismicity as the primary contributor to strong ground shaking at relatively short periods. At longer periods, the San Andreas Fault system and the Cascadia Subduction Zone at approximate distances of 125 and 300 km, respectively, also contribute to the hazard. Exhibit D-3 shows the interactive deaggregations available from the USGS for 0.0 (peak ground acceleration, PGA) and 1.0 second periods.

4.2 Selection and Scaling of Earthquake Records

The MCE_R spectral acceleration values (response spectrum for Site Class C) were used as a target spectrum for selecting and scaling time histories of acceleration. The time histories served

as input motions to our site response analysis. The recorded motions were obtained from the Pacific Earthquake Engineering Research (PEER) Center and are listed in the table below.
Given the deaggragation results, we selected the PEER recordings from a database of events with a moment magnitude range of 5.5 to 8.0 and a source-to-site distance of 15 to 100 km. Recording stations located over materials with a shear wave velocity (V_s) of 1,475 to 2,000 feet/second in the upper 100 feet were also part of the selection process. The selected PEER recordings provided the minimum mean squared error after scaling relative to the target spectrum.

Earthquake	Station	V_s (ft/sec)	Component	Moment Magnitude	Distance (km)	Scale Factor
1980 Irpinia, Italy	Brienza	1,840	000	6.2	42	8.3
1994 Northridge, California	LA – Temple & Hope	1,485	090	6.7	29	2.1
1989 Loma Prieta, California	Berkeley – Strawberry Canyon	1,680	045	6.9	78	4.1
1978 Santa Barbara, California	Cachuma Dam Toe	1,530	250	5.9	24	5.2
1984 Lazio-Abruzzo, Italy	Roccamonfina	1,560	000	5.8	45	8.2

The recorded time histories were amplitude scaled using the scale factors listed in the above table. Exhibit D-4 shows the MCE_R target spectrum and the computed response spectra from the scaled time histories. The period range of significance for selection and scaling ranged from 0.26 to 2.0 seconds assuming a fundamental period of vibration for the building of 1.3 seconds.

4.3 Site Response Analysis

We evaluated the one-dimensional, equivalent linear response of the site soils using the computer program SHAKE2000 (Ordonez, 2004). We developed the soil profile for the site from the shear wave velocity profile described in Section 4.2. The scaled motions were input at a depth of 43.4 feet. In addition to the shear wave velocity values, the soil model for analysis included material density and plasticity values based on samples collected from the borings and tested in our laboratory. The variation of shear modulus and damping ratio with cyclic shear strain used in our analysis was based on relationships published by Darendeli (2001).

Exhibit D-5 presents the results of the site response analysis. In addition to the five response spectra computed at the ground surface, the exhibit shows the average spectrum from the input motions and the average spectrum from the output motions. Comparing the average input and output spectra indicates that the soil profile will primarily amplify ground motions in the period range of 0.2 to 1.0 seconds.

4.4 Conclusions to Site Specific Ground Motion Study

We followed ASCE 7-10 Chapter 21 procedures to develop the site-specific, surface MCE_R ground motion response spectrum. That is, ratios of 5 percent damped response spectra of surface ground motions to input motions were calculated at select periods and the ratio at each period was multiplied by the appropriate MCE_R spectral value. The table below lists the calculated amplification ratios and the resulting surface MCE_R spectral acceleration values. Note that the surface MCE_R spectral acceleration values in the table are all probabilistic ground motions, because the probabilistic values are lower than the deterministic values given the relatively long distance to characteristic earthquake sources with large magnitude events.

Period (seconds)	Calculated Ratio	Site Class C MCE_R Spectral Acceleration (g)	Surface MCE_R Spectral Acceleration (g)
0.0	1.343	0.314	0.422
0.1	1.046	0.625	0.654
0.2	1.312	0.759	0.996
0.3	1.757	0.687	1.206
0.5	1.922	0.522	1.004
1.0	1.204	0.376	0.452
2.0	1.050	0.205	0.215
3.0	1.035	0.128	0.132
4.0	1.026	0.087	0.090

ASCE 7-10 Section 21.3 states that the design response spectrum is determined by reducing the site-specific MCE_R spectrum by one-third. However, the lower bound for design ground motions is 80 percent of the response spectrum determined in accordance with ASCE 7-10 Section 11.4.5. Exhibit D-6 compares the ASCE 7-10 Section 11.4.5 design response spectrum, the 80 percent spectrum, and the site-specific spectrum.

ASCE 7-10 Section 21.4 states that the parameter S_{DS} from the site-specific study is equal to the spectral acceleration obtained at a period of 0.2 seconds, except that it shall not be taken as less than 90 percent of the peak spectral acceleration at any period larger than 0.2 seconds. It also states that the parameter S_{D1} from the site-specific study shall be taken as the greater of the spectral acceleration at a period of 1.0 second or two times the spectral acceleration at a period of 2.0 seconds. Given these requirements, the site-specific value of S_{DS} was increased to 0.724g. The following table lists the design acceleration parameters from the various procedures. Note that the 2013 CBC and ASCE 7-10 Section 11.4.5 spectra are the same.

Procedure	S_{DS} (g)	S_{D1} (g)
2013 CBC Site Class D	0.568	0.355
80 percent 2013 CBC	0.454	0.284

Procedure	S_{DS} (g)	S_{D1} (g)
Site-Specific	0.724	0.302

We recommend that the site-specific ground motion study be updated as the project develops. Potential updates include:

- Performing a site-specific PSHA rather than the region-specific USGS Earthquake Hazards Program PSHA.
- Extending the soil profile to the Site Class B/C boundary (i.e., V_s = 2,500 feet/second).
- Selecting different or additional input motions to the site response analysis based on refinement of the fundamental period of vibration for the building.
- Accounting for embedment of the building.
- Performing nonlinear site response analysis.
- Completing additional parametric studies to evaluate uncertainty with the site-specific ground motion values.
- Providing representative accelerograms for a time-history or performance-based approach to estimating building response.

4.5 Liquefaction Potential

Liquefaction is a mode of ground failure that results from the generation of high pore water pressures during earthquake ground shaking, causing loss of shear strength. Liquefaction is typically a hazard where loose sandy soils exist below groundwater. The California Geologic Survey (CGS) has designated certain areas within Southern California as potential liquefaction hazard zones. These are areas considered at a risk of liquefaction-related ground failure during a seismic event, based upon mapped surficial deposits and the presence of a relatively shallow water table. The project site is not mapped within a liquefaction hazard zone designated by the CGS.

The subsurface materials generally consisted of medium stiff to stiff silt with variable sand to a depth of approximately 31 to 33 feet underlain by medium dense to dense gravel and sand soils to a depth of 43 to 45 feet. Gravel and sand soils were underlain by very stiff to dense sand and silt soils to a depth of 68 to 70 feet. Sand and silt soils were underlain by very stiff to hard fat clay with variable sand to a depth of 85 to 89 feet, which in turn was underlain by hard to very hard silt to sandy silt to the maximum depth of exploration of 101½ feet bgs. Groundwater was encountered at an approximate depth ranging between 17½ and 18 feet bgs during the field exploration.

A liquefaction analysis for the site was performed in general accordance with the DMG Special Publication 117. The liquefaction study utilized the software “LiquefyPro” by CivilTech Software. This analysis was based on the soil data from the soil borings. A Peak Ground Acceleration (PGA) of 0.309g and the mean magnitude of 6.32 for the project site were used. Calculations utilized both the encountered groundwater depth and the shallowest depth from the nearby monitoring well. Settlement analysis used the Ishihara / Yoshimine method. Fines were corrected for liquefaction using the modified Stark/Olson method.

A Liquefaction potential analysis were calculated from a depth of 0 to 50 feet below the ground surface. Liquefaction potential analysis is attached in Appendix D of this report.

Based on the calculation results, liquefaction potential onsite appears to be low. Total seismically induced settlement was found to be less than ½ inch, while seismically induced differential settlement was found to be less than ¼ inch

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5.0 RECOMMENDATIONS FOR DESIGN AND CONSTRUCTION

5.1 Geotechnical Considerations

The site appears suitable for the proposed construction based upon geotechnical conditions encountered in the test borings and provided the geotechnical engineering recommendations included in this report are implemented in the design and construction of the project.

The subgrade soils in the upper 27 feet are highly compressible, have low bearing capacity, and are susceptible to settlement under expected structural loading. For this reason, we recommend that the building foundation penetrate through the weak upper silts and derive their support from the dense to hard soils encountered below a depth of 35 feet. A considerable increase in density and shear strength, and therefore a considerable increase in the capacity of the foundation, was encountered at a depth of 90 feet bgs which will provide adequate support for the proposed 12-story structure.

Commented [HF3]: Again, I though the soft compressible stuff is at 27

Geotechnical engineering recommendations for foundation systems and other earth connected phases of the project are outlined within this report. The recommendations contained in this report are based upon the results of field and laboratory testing (which are presented in Appendices A and B), engineering analyses, and our current understanding of the proposed project.

5.2 Earthwork

The following presents recommendations for site preparation, excavation, subgrade preparation and placement of engineered fills on the project. The recommendations presented for the design and construction of earth supported elements including, foundations and slabs are contingent upon following the recommendations outlined in this section. All grading for each structure should incorporate the limits of the proposed structure plus a minimum lateral distance of 5 feet.

Earthwork on the project should be observed and evaluated by Terracon. The evaluation of earthwork should include observation and testing of engineered fill, subgrade preparation, foundation bearing soils, and other geotechnical conditions exposed during the construction of the project.

5.2.1 Site Preparation

Strip and remove demolition debris, pavements, vegetation, concrete flatwork and other deleterious materials from the proposed building and pavement areas. Exposed surfaces should be free of mounds and depressions which could prevent uniform compaction.

Demolition of the existing building should include complete removal of all foundation elements and remaining underground utilities within the excavation depth. It is our understanding that existing building is supported on deep foundations. For existing piles that do not overlap with new pile foundations and/or caps, the existing pile foundations shall be excavated down to the bottom of the basement excavation, and then cut off and abandoned in place. Terracon should be notified if new foundations will overlap with existing piles, Mitigation measures including complete removal of existing piles may be necessary to avoid differential movements and provide homogenous subsurface conditions beneath pile caps.

Our explorations indicate the presence of approximately 6½ to 7½ feet of existing fill material at the site. The fill materials were encountered in all borings at the time of our investigation. Terracon has not been provided with any documentation to indicate if the fill placement or grading operations were inspected and if fill compaction was tested during construction. We anticipate these fill materials will be removed during excavation of the parking garage, and therefore, will not impact the proposed development.

Site preparation should include removal of any loose backfill found adjacent to existing foundations. All materials derived from the demolition of existing structures should be removed from the site and not be allowed for use as on-site fill.

During our investigation, groundwater was encountered between 17½ and 18 feet bgs. Depending upon depth of excavation and seasonal conditions, groundwater may be encountered in excavations on the site. Pumping from sumps may be utilized to control water within excavations. Well points may be required for significant groundwater flow, or where excavations penetrate groundwater to a significant depth. The excavation sidewalls will not stand vertically during construction and shoring will be required.

If encountered, abandoned underground utilities and facilities should be removed and the excavation thoroughly cleaned prior to backfill placement and/or construction.

5.2.2 Subgrade Preparation

Onsite soils beneath the proposed basement foundation should be scarified, moisture conditioned, and compacted to a minimum depth of 10 inches. The subgrade soils immediately beneath the existing structure and at the depth of the garage excavation are very moist and soft and will become unstable and pump under the weight of heavy equipment. Stabilization of the subgrade soils, using gap graded crushed stone, reinforcing geogrid, and/or chemical or cement treatment should be anticipated.

Shallow spread footings for tertiary structures detached from the building foundations should bear on a minimum 24 inches of engineered fill comprised of recompacted onsite fill/native materials. Exposed areas which will receive fill, once properly cleared, should be scarified to a minimum depth of 10 inches, moisture conditioned, and compacted per the compaction requirements in Section 4.2.4.

Commented [HF4]: are we over-excavating under the basement bottom slab? pile caps? shallow foundations supporting minor structure?

Commented [NNM5]: Scarify/recompact beneath basement bottom and any detached shallow footings.

Subgrade beneath exterior slabs and pavements should be scarified, moisture conditioned, and compacted to a minimum depth of 10 inches. The moisture content and compaction of subgrade soils should be maintained until pavement construction.

5.2.3 Fill Materials and Placement

All fill materials should be inorganic soils free of vegetation, debris, and fragments larger than three inches in size. Pea gravel or other similar non-cementitious, poorly-graded materials should not be used as fill or backfill without the prior approval of the geotechnical engineer.

The on-site native soils and existing fill materials are considered suitable for use as engineered fill on the project. Approved on-site and imported materials may be used as fill for the following:

- general site grading
- foundation areas
- interior floor slab areas
- foundation wall backfill
- pavement areas
- exterior slab areas

Imported soils (if required) for use as fill material within proposed building and structure areas should conform to low volume change materials as indicated in the following recommendations:

Gradation	**Percent Finer by Weight (ASTM C 136)**
3"	100
No. 4 Sieve	50-100
No. 200 Sieve	15-40

- Liquid Limit .... 30 (max)
- Plasticity Index .... 15 (max)
- Maximum expansive index* .... 20 (max)

*ASTM D 4829

Engineered fill should be placed and compacted in horizontal lifts, using equipment and procedures that will produce recommended moisture contents and densities throughout the lift. Fill lifts should not exceed 8 inches loose thickness.

5.2.4 Compaction Requirements

Recommended compaction and moisture content criteria for engineered fill materials are as follows:

Material Type and Location	Per the Modified Proctor Test (ASTM D 1557)		
	Minimum Compaction Requirement	Range of Moisture Contents for Compaction Above Optimum	
		Minimum	Maximum
On-site or approved imported fill materials:			
Beneath shallow foundations:	90%	0%	+4%
Beneath slabs:	90%	0%	+4%
Utility trenches (pavement and structural areas):	95%	0%	+4%
Miscellaneous backfill:	90%	0%	+4%
Beneath concrete pavements:	95%	0%	+4%
General site grading:	90%	0%	+4%
Exterior slabs:	90%	0%	+4%
Exposed ground receiving fill:	90%	0%	+4%
Utility trenches (landscape areas):	90%	0%	+4%
Aggregate base (beneath pavements):	95%	0%	+4%

5.2.5 Grading and Drainage

Positive drainage should be provided during construction and maintained throughout the life of the development. Infiltration of water into utility trenches or foundation excavations should be prevented during construction. In areas where sidewalks or paving do not immediately adjoin a structure, we recommend that protective slopes be provided with a minimum grade of approximately five percent for at least 10 feet from perimeter walls. Backfill against footings, exterior walls, and in utility and sprinkler line trenches should be well compacted and free of all construction debris to reduce the possibility of moisture infiltration.

Roof drainage should discharge into splash blocks or extensions when the ground surface beneath such features is not protected by exterior slabs or paving. Sprinkler systems and landscaped irrigation should not be installed within 5 feet of foundation walls.

5.2.6 Exterior Slab Design and Construction

Exterior slabs-on-grade, exterior architectural features, and utilities founded on, or in backfill may experience some movement due to the volume change of the backfill. To reduce the potential for damage caused by movement, we recommend:

- minimizing moisture increases in the backfill;
- controlling moisture-density during placement of backfill;
- using designs which allow vertical movement between the exterior features and adjoining structural elements;
- placing effective control joints on relatively close centers.

5.2.7 Utility Trenches

It is anticipated that the on-site soils will provide suitable support for underground utilities and piping that may be installed. Any soft and/or unsuitable material encountered at the bottom of excavations should be removed and be replaced with an adequate bedding material. A non-expansive granular material with a sand equivalent greater than 30 should be used for bedding and shading of utilities, unless otherwise allowed by the utility manufacturer.

On-site materials are considered suitable for backfill of utility and pipe trenches from one foot above the top of the pipe to the final ground surface, provided the material is free of organic matter and deleterious substances. Trench backfill should be mechanically placed and compacted as discussed earlier in this report. Compaction of initial lifts should be accomplished with hand-operated tampers or other lightweight compactors. Where trenches are placed beneath slabs or footings, the backfill should satisfy the gradation and expansion index requirements of engineered fill discussed in this report. Flooding or jetting for placement and compaction of backfill is not recommended.

5.2.8 Construction Considerations

It is anticipated that excavations for the proposed construction can be accomplished with conventional earthmoving equipment. Depending upon depth of excavation and seasonal conditions, groundwater or perched groundwater may be encountered in excavations on site. Pumping from sumps may be utilized to control water within excavations. Well points may be required for significant groundwater flow, or where excavations penetrate groundwater to a significant depth.

The subgrade soils immediately beneath the existing structure and at the depth of the garage excavation are very moist and soft and will become unstable and pump under the weight of heavy equipment. Stabilization of the subgrade soils, using gap graded crushed stone, reinforcing geogrid, and/or chemical or cement treatment should be anticipated.

Soils from the excavation should not be stockpiled higher than six 6 feet or within ten 10 feet of the edge of an open trench. Construction of open cuts adjacent to existing structures, including underground pipes, is not recommended within a 1H:1V plane extending beyond and down from the perimeter of the structure. Cuts that are proposed within five 5 feet of light standards, other utilities, underground structures, and pavement should be provided with temporary shoring.

Upon completion of filling and grading, care should be taken to maintain the subgrade moisture content prior to construction of floor slabs and concrete pavements. Construction traffic over the completed subgrade should be avoided to the extent practical. The site should also be graded to prevent ponding of surface water on the prepared subgrades or in excavations. If the subgrade should become desiccated, saturated, or disturbed, the affected material should be removed or these materials should be scarified, moisture conditioned, and recompacted prior to floor slab and pavement construction.

The geotechnical engineer should be retained during the construction phase of the project to observe earthwork and to perform necessary tests and observations during subgrade preparation, proof-rolling, placement and compaction of controlled compacted fills, backfilling of excavations to the completed subgrade.

We recommend that the earthwork portion of this project be completed during extended periods of dry weather if possible. If earthwork is completed during the wet season (typically November through April) it may be necessary to take extra precautionary measures to protect subgrade soils. Wet season earthwork operations may require additional mitigative measures beyond that which would be expected during the drier summer and fall months. This could include diversion of surface runoff around exposed soils and draining of ponded water on the site. Once subgrades are established, it may be necessary to protect the exposed subgrade soils from construction traffic.

The individual contractor(s) is responsible for designing and constructing stable, temporary excavations as required to maintain stability of both the excavation sides and bottom. Excavations should be sloped or shored in the interest of safety following local, and federal regulations, including current OSHA excavation and trench safety standards. The soils at this site classify as OSHA Type 'C', requiring open excavations to be laid back at 1½ to 1 (H:V).

5.3 Foundations

The subgrade soils in the upper 27 feet are highly compressible, have low bearing capacity, and are susceptible to settlement under expected structural loading. For this reason, we recommend that the building foundation penetrate through the weak upper silts and derive their support from the dense to hard soils encountered below a depth of 35 feet. Our capacity calculation considered a negative skin friction in the upper 27 feet to account for down drag forces.

A considerable increase in density and shear strength, and therefore a considerable increase in the capacity of the foundation, was encountered at a depth of 90 feet bgs which will provide adequate support for the proposed 12-story structure.

We investigated the use of several different foundation systems to support the building including a mat slab, drilled shafts, driven pre-cast piles, and auger cast piles. Due to the shallow groundwater, the weak and compressible soils in the upper 27 feet, and the heavy loads anticipated from the building, we believe an auger cast or auger displacement pile foundation is most appropriate to support the building. Pre-cast driven piles are also a suitable option, however, we anticipate they may not be selected due to the excessive vibration and noise associated with their installation. Our office is available to discuss additional foundation solutions if proposed by a project bidder.

5.3.1 Pre-Cast Driven Piles

FOUNDATION OPTION 1 DESCRIPTION	RECOMMENDATION – DRIVEN PILES
Foundation Type	Driven pre-cast concrete piles
Bearing Material	Medium dense sands for 80' deep shafts Very dense to hard sands for 90' deep shafts
Minimum Shaft Embedment	80 feet (as measured from existing ground surface)
Minimum Shaft Dimensions	24 inches diameter
Total Estimated Settlement	1-inch
Estimated Differential Settlement	½ -inch over 40 feet
Allowable Compression Axial Capacity[1,2]	■ 305 kips for piles extending 80 feet ■ 819 kips for piles extending 90 feet
Allowable Uplift Axial Capacity[1]	■ 325 kips for piles extending 80 feet ■ 374 kips for piles extending 90 feet
Additional Allowable Skin Friction for shafts Deeper than 90 feet	1,000 psf

1. Axial loads were calculated using a factor of safety of 2.5
2. Considered a negative friction for the upper 27 feet to account for downdrag forces

Commented [HF6]: The one we ran was 24" diameter round piles

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The design capacity of a single-driven pile is a function of several factors including:

- Size and type of pile;
- Type and capacity of pile installation equipment;
- Engineering properties of subsurface soils.

Recommended soil parameters for lateral load analysis of driven shaft foundations have been developed for use in the LPILE computer program. Based upon the review of the logs of borings for the project and the Standard Penetration Test (SPT) results, engineering properties have been estimated for the soil conditions as shown in the following table.

Lateral Compression Load Analysis Estimated Engineering Properties of Soils						
Top Depth Bottom Depth	Unit Weight (pcf)	L-Pile Soil Type	Internal Friction ϕ/ Cohesion	Passive Pressure K_P (psf/ft)**	Modulus of Horizontal Subgrade Reaction Ks* (pci)	ε_{50}
0 / 15	Basement Level					
15 / 17	118	SAND	30°	350	20	--
17 / 32	120	SAND	31°	375	40	--
32 / 68	125	SAND	36°	480	60	--
68 / 89	125	STIFF CLAY W WATER	3,500 psf	375	1,400	0.005
89 / 100	125	SAND	38°	520	125	--

* Static Loading
** Passive pressure should be capped at a depth equal to 15 times the pile width/diameter

Lateral load design parameters are valid within the elastic range of the soil. The passive pressure and coefficient of subgrade reaction are ultimate values; therefore, appropriate factors of safety should be applied in the shaft design or deflection limits should be applied to the design.

We recommend that all driven pile installations be observed on a full-time basis by a representative of Terracon in order to confirm that soils encountered are consistent with the recommended design parameters.

5.3.2 Auger Cast or Auger Displacement Design Recommendations

DESCRIPTION	VALUE
Foundation Type	Auger Cast or Auger Displacement Pile
Structures	12-Story Office Building
Estimated Settlement	1 inch or less
Minimum Diameter	24 inches
Minimum Embedment Depth Below Finished Grade	35 feet
Axial Allowable Capacity of Shafts	■ 226 kips for piles extending 80 feet ■ 477 kips for piles extending 90 feet

DESCRIPTION	VALUE
Allowable Uplift Capacity	■ 227 kips for piles extending 80 feet ■ 272 kips for piles extending 90 feet
Additional Allowable Skin Friction for shafts Deeper than 90 feet	1,000 psf

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The allowable axial shaft capacities were determined using both end bearing and side friction components of resistance. Allowable axial and uplift capacity charts are provided in Appendix E of this report. The allowable uplift capacities shown are based only on the side friction of the shaft; however, the weight of the foundation should be added to these values to obtain the actual allowable uplift capacities for drilled shafts. The allowable axial and uplift capacities are based on a minimum factor of safety of 2.5.
Driven Piles, auger cast, or auger displacement piles should be considered to work in group action if the horizontal spacing is less than three shaft diameters. A minimum practical horizontal spacing between shafts of at least 2.5 diameters should be maintained, and adjacent shafts in groups should bear at the same elevation. The axial and lateral capacity of individual shafts might have to be reduced when considering the effects of group action. Capacity reduction is a function of shaft diameter and spacing. If the spacing between the proposed drilled shafts will be designed closer than three shaft diameters, the group efficiency factor for axial loading ranges between 0.65 and 1.0 for 2.5d to 3d spacing respectively.

Calculations for allowable axial and uplift capacities, were performed using CivilTech AllPile V7.15b software, and are attached to this report as Exhibit C-6. The allowable axial capacities consider ultimate down-drag forces in the upper 33 feet of the pile group. The allowable uplift capacities are based on the side friction of the piles and weight of the piles. The allowable axial and uplift capacities are based on a minimum factor of safety of 2.5 for skin friction and end bearing

5.3.3 Pile Design and Construction Considerations

All piles should be reinforced full-depth for the applied axial, lateral and uplift stresses imposed. Special sequencing of pile construction should be specified when the center to center spacing between adjacent shafts is less than three diameters. A minimum of 24 hours should be allowed between placement of concrete and initiation of drilling in shafts less than three diameters (center to center spacing) apart from each other.

If casing is used for foundation construction, it should be withdrawn in a slow continuous manner maintaining a sufficient head of concrete to prevent caving or the creation of voids in the drilled shaft concrete.

We recommend that all pile installations be observed on a full-time basis by an experienced geotechnical engineer in order to evaluate that the soils encountered are consistent with the recommended design parameters.

The Contractor should check for gas and/or oxygen deficiency prior to any workers entering the excavation for observation and manual cleanup. All necessary monitoring and safety precautions as required by OSHA, State or local codes should be strictly enforced.

5.3.4 Spread Footings

Tertiary structures, such as equipment sheds or low height landscaping walls detached from the building foundations may be supported on shallow spread footings with the following design parameters:

DESIGN ITEM	DESIGN RECOMENDATION
Foundation Type	Conventional Shallow Spread Footings
Bearing Materials	A minimum 24 inches of engineered fill comprised of recompacted onsite fill/native materials.
Allowable Bearing Pressures	2,000 psf for foundations bearing on 24 inches of engineered fill material
Minimum Plan Dimensions	12 inches
Minimum Embedment Depth Below Finished Grade	Approximately 18 inches
Total Estimated Settlement	1-inch
Estimated Differential Settlement	½ inch in 40 feet.

Finished grade is defined as the lowest adjacent grade within five feet of the foundation footings. The allowable foundation bearing pressures apply to dead loads plus design live load conditions. The design bearing pressure may be increased by one-third when considering total loads that include wind or seismic conditions. The weight of the foundation concrete below grade may be neglected in dead load computations.

Foundations should be reinforced as necessary to reduce the potential for distress caused by differential foundation movement. The use of control joints at openings or other discontinuities in masonry walls is recommended.

Foundation excavations should be observed by the geotechnical engineer. If the soil conditions encountered differ significantly from those presented in this report, supplemental recommendations will be required.

5.4 Floor Slabs

DESIGN ITEM	DESIGN RECOMMENDATION
Interior floor system	Slab-on-grade concrete.
Floor slab support	Minimum 10 inches of scarified, moisture conditioned, and recompacted native soils
Modulus of subgrade reaction	100 pounds per square inch per inch (psi/in) (The modulus was obtained based on engineered fill beneath floor slabs, and estimates obtained from NAVFAC 7.1 design charts). This value is for a small loaded area (1 Sq. ft or less) such as for forklift wheel loads or point loads and should be adjusted for larger loaded areas.

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In areas of exposed concrete, control joints should be saw cut into the slab after concrete placement in accordance with ACI Design Manual, Section 302.1R-37 8.3.12 (tooled control joints are not recommended). Additionally, dowels should be placed at the location of proposed construction joints. To control the width of cracking (should it occur) continuous slab reinforcement should be considered in exposed concrete slabs.

The use of a vapor retarder or barrier should be considered beneath concrete slabs on grade that will be covered with moisture sensitive or impervious coverings, or when the slab will support equipment sensitive to moisture to prevent moisture migration. When conditions warrant the use of a vapor retarder, the slab designer and slab contractor should refer to ACI 302 and ACI 360 for procedures and cautions regarding the use and placement of a vapor retarder/barrier.

5.5 Lateral Earth Pressures

Lateral loads may be supported by a combination of passive resistance of the basement excavation, frictional resistance of the basement foundation walls, lateral loading on the piles, and uplift of the piles. There is an existing building adjacent to this site that is set back approximately 10 feet from the new proposed building. The structural engineer should consider whether it is appropriate to rely on passive resistance adjacent to this building.

The lateral earth pressure recommendations herein are applicable to the design of rigid basement retaining walls or gravity type concrete walls for the design of lateral loading against foundation walls supporting the various structures on the site.

Commented [HF11]: rigid basement walls are not subject to rotation!!

These recommendations are not applicable to the design of geogrid-reinforced-backfill walls. Recommendations covering these types of wall systems are beyond the scope of services for this project; however, we are available to develop recommendations for the design of such wall systems upon request.

For engineered fill comprised of onsite soils or fill materials above any free water surface, recommended equivalent fluid pressures for unrestrained foundation elements are:

ITEM	VALUE[1,2]
Active Case	40 psf/ft
Passive Case	360 psf/ft
At-Rest Case	60 psf/ft
Surcharge Loads	0.33*(Surcharge)
Coefficient of Friction	0.30

[1]Note: The values are based on the on-site fill materials used as backfill.

[2] Earth pressures should be increased by 35 percent for a slope of 2H:1V behind wall. Earth pressures increase exponentially for back slopes steeper than 2H:1V.

Dynamic loading from seismic ground motions may be accommodated by applying 16 pcf EFP using the Mono Okabe formula and more recent studies by NCEER[4]. This loading may be added to the active and at rest lateral earth pressures. For clarification, this additional loading can be applied using a regular pressure distribution triangle, similar to the earth pressure above.

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

The subgrade soils at this site will not stand vertical in the basement excavation. Shoring will be required. The design of retaining structures and shoring systems should consider surcharge loads imposed by the foundations of the existing building located east of the project site. In addition, the design should take into consideration new footing loads and anticipated vehicular loads in the vicinity of the proposed basement walls. In general, surcharge loads should be considered where they are located within a horizontal distance behind the wall equal to the height of the wall.

Surcharge loads acting at the top of the wall should be applied to the wall over the backfill as a uniform pressure over the entire wall height, and should be added to the static earth pressures. Surcharge stresses due to point loads, line loads, and those of limited extent, such as compaction equipment, should be evaluated using elastic theory.

Adequate drainage should be provided behind the retaining walls to collect water from irrigation, landscaping, surface runoff, or other sources, to achieve a free-draining backfill condition. The wall back drain should consist of Class 2 permeable materials[5] that are placed behind the entire wall height to within 18 inches of ground surface at the top of the wall. As a minimum, the width of Class 2 permeable materials behind the wall should be two feet. Drainage fabric may be used

[4] Mikola, G.M., Candia, G., Sitar, N., Seismic Earth Pressures on Retaining Structures and Basement Walls, 10th US NCEER Frontiers of Earthquake Engineering Conference in Anchorage, Alaska, 2014.
[5] In accordance with the requirements and specifications of the State of California Department of Transportation.

in lieu of gravel. Water collected by the back drain should be directed to an appropriate outlet, such as perforated pipes, for disposal.

Commented [HF13]: where is the seismic lateral parameters?

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For the design of braced shoring, we recommend such shoring be designed using a rectangular-shaped distribution of lateral earth pressure of 26H (in psf) (H is the total height of the braced excavation).

Fill against foundation and retaining walls should be compacted to densities specified in Earthwork. Compaction of each lift adjacent to walls should be accomplished with hand-operated tampers or other lightweight compactors. Over-compaction may cause excessive lateral earth pressures which could result in wall movement.

The design of the shored excavation should be performed by an engineer knowledgeable and experienced with the on-site soil conditions. The contractor should be aware that slope height, slope inclination or excavation depths should in no case exceed those specified in local, state or federal safety regulations, e.g. OSHA Health and Safety Standards for Excavation, 29 CFR Part 1926, or successor regulations. Such regulations are strictly enforced and, if not followed, the owner or the contractor could be liable for substantial penalties

5.6 Pavements

5.6.1 Design Recommendations

Traffic patterns and anticipated loading conditions were not available at the time that this report was prepared. However, we anticipate that on-site traffic loads for the below grade parking garage will consist of small vehicle parking and drives.

Pavement performance is affected by its surroundings. In addition to providing preventive maintenance, the civil engineer should consider the following recommendations in the design and layout of pavements:

- Final grade adjacent to above grade parking and drives should slope down from pavement edges at a minimum 2%;
- The subgrade and the pavement surface should also have a minimum 2% slope to promote proper surface drainage;
- Install pavement drainage surrounding areas anticipated for frequent wetting (e.g., garden centers, wash racks);
- Install joint sealant and seal cracks immediately;
- Seal all landscaped areas in, or adjacent to pavements to reduce moisture migration to subgrade soils;
- Place compacted, low permeability backfill against the exterior side of curb and gutter; and,
- Place curb and/or gutter directly on low permeability subgrade soils rather than on unbound granular base course materials.

5.6.2 Minimum Pavement Thickness

Assuming the pavement subgrades will be prepared as recommended within this report, the following pavement sections should be considered minimums for this project for the traffic indices assumed in the table below. As more specific traffic information becomes available, we should be contacted to reevaluate the pavement calculations.

	Recommended Pavement Section Thickness (inches)*	
	Standard Duty Areas Traffic Index (TI) = 4.5	**Off-Site Alley or other Heavy Duty areas Traffic Index (TI) = 7.0**
Section I Portland Cement Concrete (600 psi Flexural Strength)	5.0" Concrete over 4" Class II Aggregate Base over 10" of scarified, moisture conditioned, and compacted soils	6.0" Concrete over 4" Class II Aggregate Base over 10" of scarified, moisture conditioned, and compacted soils
Section II Asphaltic Concrete	2.5" Asphaltic Concrete over 6" Class II Aggregate Base over 10" of scarified, moisture conditioned, and compacted soils	4" Asphaltic Concrete over 12" Class II Aggregate Base over 10" of scarified, moisture conditioned, and compacted soils

* All materials should meet the CALTRANS Standard Specifications for Highway Construction.

These pavement sections are considered minimal sections based upon the expected traffic and the existing subgrade conditions. However, they are expected to function with periodic maintenance and overlays if good drainage is provided and maintained.

All concrete for rigid pavements should have a minimum flexural strength of 600 psi, and be placed with a maximum slump of four inches. Proper joint spacing will also be required to prevent excessive slab curling and shrinkage cracking. All joints should be sealed to prevent entry of foreign material and dowelled where necessary for load transfer.

5.6.3 Pavement Maintenance

The pavement sections provided in this report represent minimum recommended thicknesses and, as such, periodic maintenance should be anticipated. Therefore preventive maintenance should be planned and provided through an on-going pavement management program. Preventive maintenance activities are intended to slow the rate of pavement deterioration, and to preserve the pavement investment. Preventive maintenance consists of both localized maintenance (e.g., crack and joint sealing and patching) and global maintenance (e.g., surface sealing). Preventive maintenance is usually the first priority when implementing a planned pavement maintenance program and provides the highest return on investment for pavements. Prior to implementing any maintenance, additional engineering observation is recommended to determine the type and extent of preventive maintenance. Even with periodic maintenance, some movements and related cracking may still occur and repairs may be required.

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Commented [NNM16]: This is fairly conservative. We can go to 6" if you agree

6.0 GENERAL COMMENTS

Terracon should be retained to review the final design plans and specifications so comments can be made regarding interpretation and implementation of our geotechnical recommendations in the design and specifications. Terracon also should be retained to provide observation and testing services during grading, excavation, foundation construction and other earth-related construction phases of the project.

The analysis and recommendations presented in this report are based upon the data obtained from the borings performed at the indicated locations and from other information discussed in this report. This report does not reflect variations that may occur between borings, across the site, or due to the modifying effects of construction or weather. The nature and extent of such variations may not become evident until during or after construction. If variations appear, we should be immediately notified so that further evaluation and supplemental recommendations can be provided.

The scope of services for this project does not include either specifically or by implication any environmental or biological (e.g., mold, fungi, bacteria) assessment of the site or identification or prevention of pollutants, hazardous materials or conditions. If the owner is concerned about the potential for such contamination or pollution, other studies should be undertaken.

This report has been prepared for the exclusive use of our client for specific application to the project discussed and has been prepared in accordance with generally accepted geotechnical engineering practices. No warranties, either express or implied, are intended or made. Site safety, excavation support, and dewatering requirements are the responsibility of others. In the event that changes in the nature, design, or location of the project as outlined in this report are planned, the conclusions and recommendations contained in this report shall not be considered valid unless Terracon reviews the changes and either verifies or modifies the conclusions of this report in writing.

APPENDIX A

FIELD EXPLORATION

Field Exploration Description

A total of four (4) test borings were advanced to approximate depths ranging between 18 and 101½ feet bgs. Additionally, three (3) CPT soundings were advanced to approximately 17½ feet bgs, and a multichannel analysis of surface waves (MASW) study was performed adjacent to the existing building.

The borings were located in the field by using the proposed site plan, aerial photographs of the site, and measuring distances from existing site features. The accuracy of boring locations should only be assumed to the level implied by the method used.

Continuous lithologic logs of the borings were recorded by the field engineer during the drilling operations. At selected intervals, samples of the subsurface materials were taken by driving split-spoon or ring-barrel samplers. Bulk samples of subsurface materials were also obtained. Groundwater conditions were evaluated in the borings at the time of site exploration.

Penetration resistance measurements were obtained by driving the split-spoon and ring-barrel samplers into the subsurface materials with a 140-pound automatic hammer falling 30 inches. The penetration resistance value is a useful index in estimating the consistency or relative density of materials encountered.

An automatic hammer was used to advance the split-barrel sampler in the borings performed on this site. A significantly greater efficiency is achieved with the automatic hammer compared to the conventional safety hammer operated with a cathead and rope. This higher efficiency has an appreciable effect on the SPT-N value. The effect of the automatic hammer's efficiency has been considered in the interpretation and analysis of the subsurface information for this report.

The samples were tagged for identification, sealed to reduce moisture loss, and taken to our laboratory for further examination, testing, and classification. Information provided on the boring logs attached to this report includes soil descriptions, consistency evaluations, boring depths, sampling intervals, and groundwater conditions. The borings were backfilled with auger cuttings and capped with asphalt patches prior to the drill crew leaving the site.

APPENDIX B
LABORATORY TESTING

Laboratory Testing

Samples retrieved during the field exploration were taken to the laboratory for further observation by the project geotechnical engineer and were classified in accordance with the Unified Soil Classification System (USCS) described in Appendix C. At that time, the field descriptions were confirmed or modified as necessary and an applicable laboratory testing program was formulated to determine engineering properties of the subsurface materials.

Laboratory tests were conducted on selected soil samples and the test results are presented in this appendix. The laboratory test results were used for the geotechnical engineering analyses, and the development of foundation and earthwork recommendations. Laboratory tests were performed in general accordance with the applicable ASTM, local or other accepted standards.

Selected soil samples obtained from the site were tested for the following engineering properties:

- In-situ Dry Density
- Soluble Chlorides
- pH
- Atterberg Limits
- Sieve Analysis
- In-situ Water Content
- Soluble Sulfates
- Minimum Resistivity
- Consolidation/Swell Potential
- Direct Shear

APPENDIX C

SUPPORTING DOCUMENTS

APPENDIX D

SEISMIC ANALYSIS

APPENDIX E

PILE CAPACITY ANALYSIS