9 November 2020

Mr. Michael Horvath
Hines
2800 Post Oak Boulevard, 49th Floor
Houston, Texas 77056

SUBJECT: Preliminary Geotechnical Evaluation
1125 Arguello Street
Redwood City, California
Langan Project No. 770671801

Dear Mr. Horvath:

This letter report presents our preliminary geotechnical evaluation for the proposed development at 1125 Arguello Street in Redwood City, California. We are currently performing a geotechnical investigation for the project and a report will be issued in the next month. We are using the data collected from the geotechnical investigation to prepare our preliminary conclusions and recommendations.

The project site is bound by Whipple Avenue to the west, Arguello Street to the north, a commercial building with paved parking lot to the east and Caltrain rails to the south. The site is currently occupied by several commercial buildings (1111 and 1125 Arguello Street), paved asphalt parking lot and three historic home parcels at 1203, 1219 and 1227 Arguello Street.

We understand plans for the site will include the demolition of the existing commercial building structures at 1111 and 1125 Arguello Street and the home at 1203 Arguello Street. Plans are to construct three new structures:

- a four-story office building above three-levels of basement parking at the western and central portion of the site; current plans show the eastern column line of the office building structure will be constructed at-grade

- a four-story residential building at-grade located at the eastern portion of the site

- a one-story childcare addition structure at the northwest corner of the site, adjacent to historic homes at 1219 and 1227 Arguello Street.

The purpose of our study is to provide preliminary conclusions and recommendations for the proposed office building development regarding the following:

- subsurface conditions including groundwater levels

- site seismicity and seismic hazards, if any
- appropriate foundation type(s) for new improvements
- excavations and temporary shoring
- lateral earth pressure for design of permanent basement walls.

1.0 SITE AND SUBSURFACE CONDITIONS

We began our site and subsurface evaluation by reviewing available subsurface data in our database, including the following previous investigation performed at the project site:

- Report titled Geotechnical Exploration, 1125 Arguello Street, Redwood City, California, by ENGEO Incorporated, dated 23 January 2019.

From 3 through 11 September and 1 October 2020, Langan drilled five borings and performed five cone penetration tests (CPTs) to depths of approximately 61½ feet to 100 feet below ground surface (bgs) at the site. In addition, four borings were converted to piezometers. The boring and CPT logs and results of the laboratory test program will be presented in our design geotechnical investigation report.

The site is currently occupied by several commercial buildings (1111 and 1125 Arguello Street) and three historic home parcels at 1203, 1219 and 1227 Arguello Street. The commercial office buildings are one- to two-story buildings constructed at-grade. The historic homes are one-story buildings constructed at-grade. Foundation plans for these buildings are unavailable at this time.

Based on our review of subsurface information from the current and previous geotechnical investigations performed at the site and site vicinity, the site is generally underlain by alluvial soil, which primarily consists of moderately compressible, medium stiff to hard clay with interbedded layers of medium dense to very dense sand and gravel layers. The near-surface clay has moderate to high expansion potential[1], with plasticity indices (PI) of 24 and 28 where tested.

Based on the California Geological Survey (CGS) Seismic Hazard Zone Report for the Palo Alto 7.5-Minute Quadrangle[2], the historic high groundwater level is between approximately 0 to 10 feet bgs. Groundwater was measured during the current and previous geotechnical investigations at the site at depths of approximately 8 to 15 feet bgs, corresponding to approximately Elevations 2 to 10 feet[3]. In some instances, these depths were recorded during and immediately after exploration and may not represent stabilized levels. In addition, we have installed four piezometers to monitor the groundwater levels. Recent measurements of these

[1] Highly expansive soil undergoes large volume changes with changes in moisture content.
[2] Department of Conservation (2006). "Seismic Hazard Zone Report for the Palo Alto 7.5-Minute Quadrangle, San Mateo and Santa Clara Counties, California.
[3] All elevations are based on a topographic survey provided by BKF Engineers dated 28 July 2020.

piezometers show groundwater levels at depths of 8 to 11 feet bgs, corresponding to approximately Elevations 7 to 9 feet.

Therefore, we preliminarily recommend a design groundwater level at approximately Elevation 10 feet.

2.0 DISCUSSIONS AND PRELIMINARY RECOMMENDATIONS

The primary geotechnical issues that should be addressed during design development are adequate foundation support, settlement behavior and shoring design. Our discussions and preliminary conclusions and recommendations regarding foundation alternatives and other geotechnical aspects of the project are presented in the remainder of this letter report. The conclusions and recommendations presented herein are preliminary and will be finalized in our design geotechnical investigation report.

2.1 Seismic Hazards

The site is in a seismically active area and will be subject to very strong shaking during a major earthquake. Strong ground shaking during an earthquake can result in ground failure such as that associated with soil liquefaction[4], lateral spreading[5], and seismic densification[6]. Each of these conditions has been preliminarily evaluated based on our review of the available subsurface data.

2.1.1 Liquefaction

The site is within a zone designated with the potential for liquefaction, as identified in a map prepared by the California Geologic Survey (formerly known as the California Department of Conservation, Division of Mines and Geology) titled *State of California Seismic Hazard Zones, Palo Alto Quadrangle, Official Map,* dated 18 October 2006.

When saturated soil with little to no cohesion liquefies during a major earthquake, it experiences a temporary loss of shear strength as a result of a transient rise in excess pore water pressure generated by strong ground motion. Flow failure, lateral spreading, differential settlement, loss of bearing, ground fissures, and sand boils are evidence of excess pore pressure generation and liquefaction.

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[4] Liquefaction is a transformation of soil from a solid to a liquefied state during which saturated soil temporally loses strength resulting from the buildup of excess pore water pressure, especially during earthquake-induced cyclic loading. Soil susceptible to liquefaction includes loose to medium dense sand and gravel, low-plasticity silt, and some low-plasticity clay deposits.

[5] Lateral spreading is a phenomenon in which surficial soil displaces along a shear zone that has formed within an underlying liquefied layer. Upon reaching mobilization, the surficial blocks are transported downslope or in the direction of a free face by earthquake and gravitational forces.

[6] Seismic densification is a phenomenon in which non-saturated, cohesionless soil is compacted by earthquake vibrations, causing ground surface settlement.

We used the results of the rotary wash borings and CPTs recently performed by Langan to evaluate the liquefaction potential. Thin and discontinuous layers of saturated medium dense silty sand, clayey sand and gravel were encountered below the groundwater table. Based on the results of our analyses, we conclude that these layers could potentially liquefy during a major earthquake and may experience liquefaction-induced settlement. In the areas surrounding the project site (no basement excavation), we conclude up to one inch of seismically induced-settlement could occur.

We conclude several layers generally above approximately 30 feet bgs are potentially liquefiable during a major earthquake. The excavation for the basement of the proposed development will remove most of these layers. For a three-level basement (assume excavation of approximately 30 feet), we conclude that less than ½ inch seismically induced-settlement could occur beneath the basement.

2.1.2 Seismic Densification

Seismic densification of non-saturated, cohesionless soil following a major earthquake was analyzed using the procedure outlined by Tokimatsu and Seed (1987) and the Pradel (1998) method. The CPTs and borings indicate layers of loose to medium dense sand with varying amounts of clay and silt were encountered above the groundwater level. Using the Pradel (1998) method for evaluating seismically-induced settlement in dry sand, we estimate seismic densification settlements in these layers of up to approximately ¼ inch during a major earthquake. The excavation for the basement levels would remove this layers; however, these settlements could occur outside the building footprint.

2.1.3 Lateral Spreading

Lateral spreading is a phenomenon in which a surficial soil displaces along a shear zone that has formed within an underlying liquefied layer. The surficial blocks are transported downslope or in the direction of a free face, such as a channel, by earthquake and gravitational forces. Lateral spreading is generally the most pervasive and damaging type of liquefaction-induced ground failure generated by earthquakes. The project site is relatively flat, the potentially liquefiable soil layers are not continuous, and the nearest free face is over ½-mile from the site; therefore, we judge that lateral spreading at the site is low.

2.2 Expansive Soil

The primary geotechnical concern at the project site is the presence of moderately to highly expansive surface soil.

Expansive surface soil is subject to high volume changes during seasonal fluctuations in moisture content. These volume changes can cause cracking of foundations and floor slabs. Therefore, foundations and slabs should be designed and constructed to resist the effects of the expansive soil. These effects can be mitigated by moisture conditioning the expansive soil and providing select, non-expansive fill below interior and exterior slabs and supporting foundations below the zone of severe moisture change.

For at-grade structures, interior slabs-on-grade should be underlain by at least 12 inches of select, nonexpansive fill. Previous experience with similar soil types indicates exterior concrete slabs-on-grade should perform satisfactorily if they are supported on a layer of select fill at least eight inches thick.

2.3 Settlement and Foundations

We preliminarily conclude the portion of the new office building with the three-level basement can be supported on a mat foundation. However, if shallow foundations are used for the eastern column line of the office building, we judge the static and earthquake-induced settlement and resulting differential settlement would be excessive for the long-term structural performance of the structure. Therefore, we conclude deep foundations should be considered for where the office building is at-grade.

Building loads are currently unavailable for the at-grade residential building or the childcare addition structure. However, based on the proposed height of the residential building and its vicinity to the office building's basement, we judge if the building is supported on a shallow foundation, the static and earthquake-induced settlement and resulting differential settlement would be excessive for the long-term structural performance of the structure. Therefore, we preliminarily conclude the residential structure should be supported on deep foundations.

For the childcare addition structure, based on the proposed height of the structure, we preliminarily conclude the structure can be supported on shallow spread footings.

The potential foundation types for the proposed structures, including shallow and deep foundations, are discussed in the following subsections.

2.3.1 Mat Foundation

We understand a four-story office building will be constructed above three-levels of basement parking with the eastern column line of the office building will be constructed at-grade. Based on preliminary structural loads provided by Magnusson Klemencic Associates (MKA), the project structural engineer for the office structure, dead plus live column loads for the office structure are approximately 740 kips; currently, plans show the finished floor elevation of the basement will extend approximately 28 feet bgs.

Where three levels of basements planned for the office building, the soil exposed at the bottom of the excavation will consist of stiff clay or dense sand. The clay layers should be capable of supporting moderate foundation loads without large settlement because of the pressure relief which will result from the proposed basement excavation. Furthermore, basement levels will place the foundation below the groundwater table and it will need to be designed for hydrostatic uplift. Therefore considering the anticipated settlements and depth to groundwater, we conclude a mat foundation is feasible for support of the buildings where there are three basement levels.

The mat foundation should be designed to tolerate the predicted settlements. Initially, as the proposed excavations are made, we expect the removal of soil will create pressure relief and the base of the excavation will rebound (rise), especially near the center of the excavation. After the new foundation is constructed and new building loads are applied, the pressure will increase and the clay layer will partially recompress. Preliminary estimates of settlements associated with this recompression could range between ½ and 1 inch. We estimate post-construction differential settlement between columns may be on the order of ½ inch. This estimate does not include the rigidity of the mat which will tend to reduce differential settlement. With these settlement, our preliminary recommendation is for the mat to span an unsupported area of five feet in diameter in any location within the interior.

For design of the mat foundations (assuming three basement level) using a subgrade modulus method, we recommend using a subgrade modulus of 20 kcf. The modulus value is representative of the settlement under the anticipated building loads. Once the structural engineer estimates the distribution of bearing stress on the bottom of the mat, we should review the distribution and revise the modulus of subgrade, if appropriate. Depending upon the stiffness of the mat and its ability to distribute stresses uniformly to the soil, there could be stress concentrations beneath columns. The recommended subgrade modulus is approximate for a maximum dead plus live load bearing pressure of 2,000 pounds per square foot (psf). Preliminarily we recommend the allowable bearing pressure for total loads should not exceed 5,000 psf.

Lateral forces on mats can be resisted by a combination of friction along the base of the mat, and passive resistance against the vertical faces of the foundation and, where applicable, the basement walls perpendicular to the direction of earthquake shaking. The allowable friction factor will depend on if waterproofing is used at the base of the mat. For bentonite-based waterproofing membranes, such as Paraseal and Voltex, a friction factor of 0.15 should be used. Friction factors for other types of waterproofing membranes can be provided upon request. Where waterproofing is not used then a friction factor of 0.30 may be used. If passive pressure on the walls is relied upon for lateral resistance, the walls should be designed to resist the passive pressure. To calculate the passive resistance against the vertical faces of the mat and basement walls, we preliminarily recommend a uniform pressure of 2,000 psf. The upper foot should be ignored unless confined by a slab. The values for the friction coefficient and passive pressures include a factor of safety of 1.5.

2.3.2 Deep Foundations

As previously discussed, the eastern portion of the office building structure will be constructed at-grade and the residential building will be constructed at-grade. For at-grade structures, we conclude deep foundations should be considered.

We considered several deep foundations for this project, including driven PCPS concrete piles or steel H-piles, drilled displacement (DD) piles and augered cast-in-place (ACIP) piles. Driven piles are installed using a heavy diesel or hydraulic hammer to advance the piles into the ground, which causes noise and vibrations. If there are noise and vibration constraints in the area, pile driving may not acceptable. Drilled piles, such as ACIP piles or DD piles, can be used because they are low-vibration, low-noise, deep foundation options. These pile types are designed and installed by specialty contractors. The specialty contractors can consider ACIP piles (non-displacement) or DD piles (partial-displacement or full-displacement) based on their review of subsurface conditions presented and their proprietary equipment. If drilled piles are used, they will need to be load tested to confirm the design values.

Non-displacement drilled piles are installed by drilling to the required depth with a hollow-stem auger. The soil cuttings are carried to the ground surface on the flights of the auger and removed at the ground surface by earthmoving equipment. Partial- and full-displacement drilled pile systems partially- or fully- displace the soil within the pile cross section into the surrounding soil and have the advantage of generating significantly less soil cuttings during foundation installation. Typically, partial- and full- displacement drilled piles achieve greater load carrying capacity per length of pile. However, partial- and full-displacement drilled piles can encounter installation difficulties when being installed within deep, dense sand strata.

For all drilled pile types (non-, partial-, and full-displacement), cement grout or concrete is injected through the hollow-stem auger or drill string when the auger or drill string reaches the required depth. Grout or concrete is injected continuously as the auger or drill string, still rotating in a forward direction, is slowly withdrawn, replacing the removed soil. While the grout is still fluid, a steel reinforcing cage is inserted into the shaft. Drilled piles can range in diameter; however, 16- and 18-diameter drilled piles are typical.

2.3.2.1 Axial Capacity

In the San Francisco Bay Area, ACIP and DD piles are typically designed and installed under a design-build contract by specialty foundation contractors; therefore, we are not providing specific recommendations for their design. However, for preliminary design and cost estimating purposes, we are providing an estimate of vertical capacities of the piles based on our experience, the results of load tests, and engineering analysis. The actual capacities and lengths should be determined by the design-build foundation contractor. We should review the pile design prior to installation.

Based on our experience in the site vicinity, we preliminarily recommend an allowable dead plus live load frictional capacity of 800 psf (includes a Factor of Safety (FS) of 2.0) for ACIP and DD piles, with a one-third increase for total loads including wind or seismic. Higher capacities may be achieved for piles tipped in dense to very dense sands and gravels. As outlined in Section 1810.3.3.1.5 of the 2019 California Building Code, the allowable uplift capacities should be determined using a FS of 3.0 for permanent loading and 2.0 for temporary (wind or seismic) loading. If tension load testing in accordance with ASTM D3689 is performed, then a FS of 2.0 may be used for uplift permanent loading and a FS of 1.5 may be used for temporary uplift loading.

Properly constructed ACIP and DD piles should have a total settlement less than one inch, with less than ½ inch of differential settlements between columns, under static conditions. Most of these static settlements are expected to occur during construction.

2.3.2.2 Lateral Capacity

Lateral load resistance can be mobilized by the individual piles in combination with other foundation elements embedded below the ground surface. Lateral resistance of piles will depend on the stiffness of the pile, the strength of the surrounding soil, the allowable deflection of the pile top, and the bending moment induced in the pile.

TABLE 1
Preliminary Lateral Pile Capacities

Pile Type	Connection Type	Axial Load (kips)	Lateral Load (kips)	Maximum Moment (kip-in)	Depth To Maximum Moment (feet)[1]	Pile Head Deflection (inches)
16-inch Diameter Augered Cast-in-Piles	Fixed	220 (Compression)	31	1,000	0	0.2
	Free	220 (Compression)	26.5	860	5.2	0.5

Note:
1. Deflection limited by moment capacity of pile of about 1,000 kip-inches for the 16-inch ACIP.

Once the final pile type and loads have been determined, the appropriate deflection and moment profiles for a single pile should be developed.

The lateral capacities in Table 1 are for single piles only. To account for group effects, the lateral load capacity of a single pile should be multiplied by the appropriate reduction factors shown in Table 2. However, the maximum moment for a single pile with an unfactored load should be used to check the design of individual piles in a group. The reduction factors are based on a minimum center-to-center spacing of three pile widths. Where piles are spaced at least six pile diameters in all directions, no group reduction factors need to be applied. Reduction for other pile group spacing can be provided once the number and arrangement of piles are known.

TABLE 2

Lateral Group Reduction Factors

Number of Piles within Pile Cap	Lateral Group Reduction Factor
2	0.9
3 to 5	0.8
$\geq$6	0.7

Additional lateral load resistance can be developed by passive resistance acting against the faces of the pile caps and grade beams. Passive resistance may be computed using an uniform pressure of 2,000 psf. The upper foot should be ignored unless it is confined by a slab. Frictional resistance should be computed using a base friction coefficient of 0.30. These values have a factor of safety of about 1.5 and may be used in combination without reduction.

2.3.3 Spread Footings

The childcare addition will be a one-story at-grade structure. We conclude the childcare addition structure can be supported on shallow, spread footings bearing on firm, native soil or engineered fill. The bottom of the footings should be embedded at least 18 inches below the lowest adjacent soil subgrade and should be at least 18 inches wide for continuous footings and 24 inches for isolated spread footings. Footings adjacent to utility trenches (or other footings) should bear below an imaginary 1.5:1 (horizontal to vertical) plane projected upward from the bottom edge of the utility trench (or adjacent footings).

For the recommended minimum embedment, the footings bearing on firm native soil or engineered fill may be designed for an allowable bearing pressure of 3,000 psf for dead plus live loads, with a one-third increase for total loads, including wind and/or seismic loads. Footings designed in accordance with these recommendations should not settle more than one inch; differential settlement between adjacent footings, typically 20 to 30 feet apart, should not exceed ½ inch. In addition to the anticipated static settlement, up to one inch of liquefaction induced settlement is anticipated for foundations at grade.

2.4 Excavation and Temporary Shoring

Excavations at the site will be made to construct the basement and to install underground utilities. We anticipate the proposed excavation can be made using conventional earth moving equipment. Removal of existing on-site improvements, including the foundations of the existing building, may require equipment capable of breaking concrete. The excavation contractor should note that previous foundations, building debris, and other obstructions may be encountered during shoring installation and excavation. These obstructions may have to be partially removed before the shoring can be installed.

Excavations deeper than five feet that will be entered by workers should be shored or sloped for safety in accordance with the Occupational Safety and Health Administration (OSHA) standards (29 CFR Part 1926). Inclinations of temporary slopes should not exceed those specified in local, state or federal safety regulations. Temporary slopes should be less than 12 feet high and no steeper than 1½:1 (horizontal to vertical); deeper cuts or cuts where there is insufficient space for a sloped excavation should be shored.

Temporary slopes should not be open for an extended period of time. If temporary slopes are open for extended periods of time, exposure to weathering and rain could result in sloughing and erosion. All vehicles and other surcharge loads should be kept at least 10 feet away from the top of temporary slopes. The slopes should be protected from excessive saturation during construction.

Groundwater has been encountered at the site at depths between 8 and 15 feet bgs. We conclude the high groundwater level at the site should be assumed to be 8 feet bgs, corresponding to Elevation 10 feet. Because the excavation will extend below a depth of 8 feet, dewatering will likely be required.

We judge that a solider pile and lagging system along with tiebacks or internal bracing is the most economical system for shoring the basement excavation. A soldier pile and lagging system usually consists of steel beams and concrete placed in predrilled holes extending below the bottom of the excavation. Wood lagging is then placed between the soldier beams as the excavation proceeds.

We are currently evaluating the lateral earth pressures, including the loads from the Caltrain train and rails, and design parameters for the tied-back or braced shoring system. We can provide the lateral pressures recommended for designing temporary shoring system in our design-level geotechnical investigation report.

2.5 Basement Walls

We recommend all basement walls be designed to resist lateral pressures imposed by the adjacent soil and vehicles. Because the site is in a seismically active area, the design should also be checked for seismic conditions. Under seismic loading conditions, there will be a seismic pressure increment that should be added to active earth pressures (Sitar et al., 2012). We used the procedures outlined in Sitar et al. (2012) and the peak ground acceleration based on the Design Earthquake ground motion level to compute the seismic pressure increment. Basement walls should be designed for the more critical loading condition of static or seismic conditions using the equivalent fluid weights and pressures presented in Table 3.

TABLE 3
Basement Wall Design Earth Pressures
(Drained Conditions)

Condition	Static Conditions		Seismic Conditions[2]
	Unrestrained Walls (Active)	Restrained Walls (At-rest)	Total Pressure – Active Plus Seismic Pressure Increment
Above Groundwater[1]	40 pcf	60 pcf	70 pcf
Below Groundwater[1]	80 pcf	90 pcf	100 pcf

Notes:
1. Recommended design groundwater elevation is Elevation 10 feet.
2. The more critical condition of either at-rest pressure for static conditions or active pressure plus a seismic pressure increment for seismic conditions should be checked.
3. pcf = pounds per cubic foot

Where traffic will pass within 10 feet of basement walls, temporary traffic loads should be considered in the design of the walls. Traffic loads may be modeled by a uniform pressure of 100 psf applied in the upper 10 feet of the walls. Additionally, the loads for Caltrain train and rails should be considered in the design of the walls. We are currently evaluating the associated Caltrain loads and will provide the loads in our design geotechnical report.

If the basement walls are designed to resist lateral forces such as wind or earthquake loading they should be checked using passive pressures. To calculate the passive resistance against the below-grade walls, we recommend a uniform pressure of 2,000 psf. This value includes a factor of safety of about 1.5. The structural engineer should check the structural capacity of the walls and the amount of movement necessary to develop the passive pressure. We can provide passive mobilization curves, if needed to estimate the amount of wall movement for a given passive pressure.

The lateral earth pressures provided in Table 3 assume the walls are properly backdrained above the water table to prevent the buildup of hydrostatic pressure. One acceptable method for backdraining the wall is to place a prefabricated drainage panel against the back of the wall. The drainage panel should extend down to a four-inch-diameter perforated PVC collector pipe to the design groundwater elevation of Elevation 10 feet. The pipe should be connected to a suitable discharge point.

If the walls are not drained, we recommend the full height of the basement wall be designed for an equivalent fluid weight of 90 pounds per cubic foot (pcf) to account for hydrostatic pressure.

To protect against moisture migration, below-grade walls should be waterproofed and water stops placed at all construction joints. The waterproofing should be placed directly against the backside of the walls.

2.6 Tiedown Anchors

If the weight of the office building is not sufficient to resist the hydrostatic uplift loads or the mat cannot resist the uplift pressure between columns, tiedown anchors should be installed. Tiedowns typically consist of relatively small-diameter, drilled, grout-filled shafts with steel bars or tendons embedded in the grout. Tiedowns develop their uplift resistance from friction between the perimeter of the shaft and the surrounding soil.

Tiedowns should be spaced at least four shaft diameters apart or a minimum center-to-center spacing of four feet, whichever is greater. Because specialty contractors who install tiedowns use different installation procedures, the uplift capacity of the tiedowns will vary with the procedure. For planning purposes, however, we recommend using an allowable friction of 1,000 psf for post-grouted tiedowns installed in the native stiff clays; this value includes a factor of safety of 2.0 for permanent uplift loads (i.e. hydrostatic uplift). Higher values can be obtained depending upon the installation techniques employed by the contractor and the results of pullout tests.

Special attention should be given to waterproofing the connections between the tiedowns and the mat. Because the tiedowns will be permanent, we recommend they be double corrosion protected.

The tiedowns will be installed below the water table; therefore, the contractor should use an auger-cast system or be prepared to case the holes to prevent caving. High strength bars or strands may be used as tensile reinforcement in the anchors. For stressing, the steel bars and strands should have at least 10 and 15 feet of free length, respectively. After testing, tiedowns should be locked-off at 10 percent of the design load or higher, if required by the structural engineer to limit deformation of the tiedown under the hydrostatic loading.

The bond length should be at least 15 feet. The design capacity of the tiedowns should be confirmed by a performance- and proof-test program conducted under our observation. We recommend the first two production tiedowns and two percent of the remaining tiedowns be performance tested to 2.0 times the design load. The remainder should be proof tested to 1.5 times the design load. Replacement tiedowns should be provided, as directed by the structural engineer, for tiedowns that fail the test. All tiedowns should be locked off. The lock-off load and allowable amount of deformation after the tiedown is locked off should be determined by the structural engineer.

In addition, piles such as ACIP or DD can also be used as tiedown elements. Preliminary allowable friction capacity for uplift and preliminary recommendation for load tests for ACIP or DD piles can be found in Section 2.3.2.1.

3.0 DESIGN GEOTECHNICAL REPORT

The preliminary conclusions and recommendations presented in this letter result are based on subsurface information at and in the vicinity of the site. They should not be used to develop final design drawings. We are currently preparing a design-level geotechnical investigation that should be used to develop final design drawings.

Sincerely yours,
Langan Engineering and Environmental Services, Inc.

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