APPENDIX F

Geotechnical Investigation for Two Proposed New Residential Structures

GEOFORENSICS INC. **Consulting Soil Engineering**

561-D Pilgrim Drive, Foster City, CA 94404 Phone: (650) 349-3369 Fax: (650) 571-1878

File: 216195
October 13, 2016

Mr. Michael Sneper
3333 South Bascom Avenue
Campbell, CA 95008

Subject: **Saratoga Creek Drive Property**
Saratoga Creek Drive (APNs 389-06-020 and 021)
Saratoga, California
GEOTECHNICAL INVESTIGATION FOR TWO PROPOSED NEW RESIDENTIAL STRUCTURES

Dear Mr. Sneper:

In accordance with your authorization, we have performed a subsurface investigation into the geotechnical conditions present at the location of the proposed improvements. This report summarizes the conditions we measured and observed, and presents our opinions and recommendations for the design and construction of the two proposed new residential structures.

Site Description

The subject site is two relatively level, irregularly-shaped parcels located off the south end of Saratoga Creek Drive (at the approximate location shown on Figure 1). Lot 1 is off the southwest end of Saratoga Creek Drive and Lot 2 is off the southeast end of Saratoga Creek Drive. The properties are bounded by commercial buildings to the north of both lots, and to the east of Lot 2. Saratoga Creek is west of Lot 1.

The sites are currently two empty lots. The grounds are vegetated with a variety of small to medium sized bushes and shrubs, small to large trees, and various native plants and grasses.

The ground surface in the site vicinity has an overall gentle slope down towards the north (as shown on Figure 2). At the site, the ground slopes gently down towards the north, and gently to steeply down towards the west along the creek bank. Surface gradients generally range from level to 20:1 (horizontal:vertical, H:V), with the western portion of Lot 1 along the creek with the 5 to 20 foot wide slope inclined between 5:1 to 1:1 slope down from the crest of the slope. Some areas have a near vertical face downslope of the 5:1 to 1:1 portions of the creek bank. It appears that little or no grading work has been previously performed on the site.

Proposed Construction

We understand that the current development for the site proposes the construction of two new residential buildings. The buildings are to include partial basement level garages. The buildings are to be of conventional, wood-framed construction with concrete walls for the basements. New foundation loads are expected to be typical for these types of structures (i.e. light).

Excavation work at the site is expected to include: crawlspace, foundation, and basement excavations up to about 10 feet deep. No significant fill placement is anticipated as part of this work. Cuts up to 10 feet tall for the basement are anticipated as part of the site development. Building and site retaining walls up to 10 feet tall will be required for the proposed construction. No pools are planned for the project.

INVESTIGATION

Scope and Purpose

The purpose of our investigation was to determine the nature of the subsurface soil conditions so that we could provide geotechnical recommendations for the construction of the proposed new buildings. In order to achieve this purpose, we have performed the following scope of work:

1 - visited the property to observe the geotechnical setting of the area to be developed;
2 - reviewed relevant published geotechnical maps;
3 - drilled five borings near the location of the proposed improvements;
4 - performed laboratory testing on collected soil samples;
5 - assessed the collected information and prepared this report.

The findings of these work items are discussed in the following sections of this report.

Site Observations

We visited the site on September 14, 2016 to observe the geotechnically relevant site conditions. During our visit, we noted the following conditions:

A - The base of Saratoga Creek is approximately 20 feet lower than the existing elevations of Lot 1.

B - We would characterize the drainage on the lots to be sheet flow to the north and west.

Geologic Map Review

We reviewed the *Geologic Map of the Cupertino and San Jose West Quadrangles, Santa Clara and Santa Cruz Counties, California: Dibblee Geological Foundation, Dibblee Foundation Map DF-351, scale 1:24,000*, by T.W. Dibblee and J.A. Minch (2007), and the *State of California Seismic Hazards Zone Map; Cupertino Quadrangle* (9/23/02). The relevant portions of the Dibblee and Minch map and State map have been reproduced in Figures 3 and 3a.

The Dibblee and Minch map indicates that the site is underlain by Surficial Sediments (map symbol "Qa.2"). Dibblee and Minch describe these materials as consisting of "alluvial gravels, sand, silt, and clay; represents younger alluvium in fan deposits."

Our subsurface exploration (see below) encountered clay and sand materials which we judged to be consistent with the mapping.

The Seismic Hazards Zone Map indicates the site is **outside** of the areas where: 1) there is a historic occurrence of liquefaction; 2) there have been previous occurrences of landslide movement; and 3) there are local topographic, local geological, geotechnical, and groundwater conditions would indicate a potential for permanent ground displacement such that mitigation, as defined in Public Resource Code Section 2693(c), would be required.

The active San Andreas Fault is mapped approximately 4.8 miles (7.7 km) southwest of the site, and the potentially active Monte Vista-Shannon Fault is mapped approximately 1.5 miles (2.4 km) to the southwest.

Subsurface Exploration

On September 14, 2016 we drilled five borings at the site at the locations shown on Figure 4. The borings were drilled using a Mobile B-24 truck-mounted drilling rig equipped with 4.0 inch diameter, helical flight augers. Logs of the soils encountered during drilling record our observations of the cuttings traveling up the augers and of relatively undisturbed samples collected from the base of the advancing holes. The final boring logs are based upon the field logs with occasional modifications made upon further laboratory examinations of the recovered samples and laboratory test results. The final logs are attached in Appendix A.

The relatively undisturbed samples were obtained by driving a 3.0 inch (outer diameter) Modified California Sampler and a Standard Penetration Sampler (as noted on logs) into the base of the advancing hole by repeated blows from a 140 pound hammer lifted 30 inches. On the logs, the number of blows required to drive the sampler the final 12 inches of the 18 inch drive, have been recorded as the Blow Counts. These blows have not been adjusted to reflect equivalent blows of any other type of sampler or hammer, or to account for the different samplers used.

Subsurface Conditions

Boring 1 first penetrated 5 feet of loose silty sand with varying amounts of roots, gravels and asphalt debris. This was underlain by medium dense to very dense silty gravelly sand down to the terminated boring depth of 27.5 feet. The boring encountered drilling refusal on cobbles at 26 feet, however, was sampled to 27.5 feet. The boring then encountered drilling refusal at 27.5 feet, where the boring was terminated at that depth.

Boring 2 first penetrated 3 feet of silty fine-gravelly sand with roots. This was underlain by medium dense silty gravelly sand down to the terminated boring depth of 18 feet. A thin layer of loose sand was encountered between 8.5 and 10.5 feet. The boring encountered drilling refusal on cobbles at 16.5 feet, however, was sampled to 18 feet. The boring again encountered drilling refusal at 16.5 feet. The boring was terminated at the sampled to depth of 18 feet.

Boring 3 penetrated 1.5 feet of silty gravelly sand. This was underlain by loose (@9 to 11 feet) to medium dense silty gravelly sand down to the terminated boring depth of 12.5 feet. The boring encountered drilling refusal on cobbles at 12.5 feet where the boring was terminated.

Boring 4 penetrated silty fine to coarse gravelly sand in a loose condition to a depth of 8 feet. Below 8 feet, the silty gravelly sand was medium dense condition down to the terminated boring depth of 17 feet. The boring encountered drilling refusal on cobbles at 17 feet where the boring was terminated.

Boring 5 penetrated 13.5 feet of loose silty sand with varying amounts of gravel. This was underlain by firm silty fine sandy clay to a depth of 18.5 feet. Below this was medium dense silty gravelly sand. The boring encountered drilling refusal on cobbles at 22 feet, however, was sampled to the terminated boring depth of 23.5 feet.

Please refer to Appendix A for a more detailed description of each boring.

No free groundwater was encountered during the drilling of the holes. However, during periods of heavy rain or late in the winter, groundwater seepage may exist at shallower depths.

Laboratory Testing

The relatively undisturbed samples collected during the drilling process were returned to the laboratory for testing of engineering properties. In the lab, selected soil samples were tested for moisture content, density, 200 sieve wash, and strength. The results of the laboratory tests are attached to this report in Appendix B.

A Sieve Analysis performed on samples of the near surface materials (Sample 1-2 @ 10 feet, Sample 1-3 @ 16 feet, Sample 1-4 @ 22 feet, and Sample 1-5 @ 27 feet) showed the tested materials are composed of 19.9 to 40.9 percent gravel, 46.5 to 71.4 percent sand, and 6.1 to 19.5 percent silt and clay. This would indicate that there is high gravel content, and a moderate fines content, which should minimize the potential for liquefaction.

Strength testing was conducted on several samples of the site materials with the following results:

Sample 1-1 (4'):	moderate unconfined compressive strength, 1025 psf
Sample 2-2 (9'):	moderate frictional strength, minimal cohesive strength (cohesion = 100 psf, friction angle = 18 degrees).
Sample 5-1 (5'):	high unconfined compressive strength, 4155 psf
Sample 5-2 (10'):	low to moderate unconfined compressive strength, 663 psf
Sample 5-3 (17'):	moderate frictional strength, but no cohesive strength (cohesion = 0 psf, friction angle = 33.4 degrees).

CONCLUSIONS AND RECOMMENDATIONS

General

Based upon our investigation, we believe that the proposed improvements can be safely constructed. Geotechnical development of the site is controlled by the presence of loose sandy soils to a depth of 13.5 feet and steep slopes along the western margins of Lot 1, however, is aided by the presence of non-expansive soils, and the proposed basement excavations.

The recommendations in this report should be incorporated into the design and construction of the proposed new residential buildings.

Seismicity

The greater San Francisco Bay Area is recognized by Geologists and Seismologists as one of the most active seismic regions in the United States. Several major fault zones pass through the Bay Area in a northwest direction which have produced approximately 12 earthquakes per century strong enough to cause structural damage. The faults causing such earthquakes are part of the San Andreas Fault System, a major rift in the earth's crust that extends for at least 700 miles along western California. The San Andreas Fault System includes the San Andreas, San Gregorio, Hayward, Calaveras Fault Zones, and other faults.

During 1990, the U.S. Geological Survey cited a 67 percent probability that an earthquake of Richter magnitude 7, similar to the 1989 Loma Prieta Earthquake, would occur on one of the active faults in the San Francisco Bay Region in the following 30 years. Recently, this probability was increased to 70 percent, as a result of studies in the vicinity of the Hayward Fault. A 23 percent probability is still attributed specifically to the potential for a magnitude 7 earthquake to occur along the San Andreas Fault by the year 2020.

Ground Rupture - The lack of mapped active fault traces through the site, suggests that the potential for primary rupture due to fault offset on the property is low.

Ground Shaking - The subject site is likely to be subject to very strong to violent ground shaking during its life span due to a major earthquake in one of the above-listed fault zones. Current (2013) building code design may be followed by the structural engineer to minimize damages due to seismic shaking, using the following input parameters from the USGS Java Ground Motion Parameter Calculator based upon ASCE 7-10 design parameters:

Site Class - D	$SM_S = 2.338$	$SM_1 = 1.238$	$SD_S = 1.559$	$SD_1 = 0.826$

Landsliding - The subject site and the surrounding area are generally level, but are gently to steeply sloping along the western margins of Lot 1. The proposed basements should preclude the potential impact of any creek bank sloughing, provided there is a minimum of 20 feet of horizontal distance between the creek channel (base of bank) and the proposed exterior face of the new basement. The potential for seismically induced landsliding on Lot 2 is nil.

Liquefaction - The State of California Seismic Hazards Zones map indicates that the site is not in an area potentially subject to liquefaction. Liquefaction most commonly occurs during earthquake shaking in loose fine sands and silty sands associated with a high ground water table. Although there are some loose sand deposits at the site, they are not saturated, and hence are unlikely to be subject to liquefaction. Therefore, we concur with the State that liquefaction is unlikely to affect the subject property.

Ground Subsidence - Ground subsidence may occur when poorly consolidated soils densify as a result of earthquake shaking. Although there are some near surface soils are sandy and loose, the basements will extend through these deposits. Any seismic settlements are projected to be less than 1 inch, which would not cause structural distress to a well designed building.

Lateral Spreading - Lateral spreading may occur when a weak layer of material, such as a sensitive silt or clay, loses its shear strength as a result of earthquake shaking. Overlying blocks of competent material may be translated laterally towards a free face. Such conditions were encountered along the western margins of the proposed Lot 1 building site, but the required separation between the creek channel and the proposed basement should preclude any impact of lateral spreading from reaching the proposed structure.

Site Preparation and Grading

All debris resulting from the demolition of existing improvements should be removed from the site and may not be used as fill. Any existing underground utility lines to be abandoned should be removed from within the proposed building envelope and their ends capped outside of the building envelope.

Any vegetation and organically contaminated soils should be cleared from the building area. All holes resulting from removal of tree stumps and roots, or other buried objects, should be overexcavated into firm materials and then backfilled and compacted with native materials.

It would be reasonable to use soils from the basement excavation to raise portions of the site grades to improve drainage of the site.

The placement of fills at the site is expected to include: utility trench backfill, retaining wall backfill, slab subgrade materials, and finished drainage and landscaping grading. These and all other fills should be placed in conformance with the following guidelines:

Fills may use organic-free soils available at the site or import materials. Import soils should be free of construction debris or other deleterious materials and be non-expansive. *A minimum of 3 days prior to the placement of any fill, our office should be supplied with a 30 pound sample (approximately a full 5 gallon bucket) of any soil or baserock to be used as fill (including native and import materials) for testing and approval.*

All areas to receive fills should be stripped of organics and loose or soft near-surface soils. Fills should be placed on level benches in lifts no greater than 6 inches thick (loose) and be compacted to at least 90 percent of their Maximum Dry Density (MDD), as determined by ASTM D-1557. In pavement (concrete or asphalt) areas to receive vehicular traffic, all baserock materials should be compacted to at least 95 percent of their MDD. Also, the upper 6 inches of soil subgrade beneath any pavements should be compacted to at least 95 percent of its MDD.

If fills in excess of 3 feet thick are to be placed, our office should be contacted for further recommendations.

Temporary, dry-weather, vertical excavations should remain stable for short periods of time to heights of 3 to 4 feet. Deeper cuts may experience raveling and sloughing. If this occurs, the cuts will need to be trimmed back per our recommendations made in the field. All excavations should be shored or sloped in accordance with OSHA standards.

Permanent cut and/or fill slopes should be no steeper than 2:1 (H:V). However, even at this gradient, minor sloughing of slopes may still occur in the future. Positive drainage improvements (e.g. drainage swales, catch basins, etc.) should be provided to prevent water from flowing over the tops of cut and/or fill slopes.

Basement Foundations

Due to the poor soil conditions in the upper 10 feet, we recommend that the entire building be supported by the basement area. If this is not possible, please contact our office for further recommendations for "at-grade" portions of the buildings.

Wall Forces – The basement retaining walls should be designed to resist an active pressure of 45 pcf Equivalent Fluid Weight (EFW), for retained slopes flatter than 4:1 (horizontal:vertical). If it is desired to create steeper retained slopes to reduce the heights of the walls, then the active pressure will need to be increased. An active pressure of 60 pcf EFW should be utilized for retained slopes with an inclination of 2:1 (H:V). Where retained slopes are greater than 4:1, though less than 2:1, the designer should linearly interpolate between 45 and 60 pcf EFW.

If the walls are considered to be restrained, they should be designed for an additional uniform pressure of 8H psf, where H is the height of the wall in feet. We leave it to the design professional's judgment in determining whether a wall is restrained or not.

It is our opinion that the above values are sufficiently conservative that they accommodate seismic design without further supplementation. However, if the structural engineer is determined to apply an additional seismic pressure, it should not exceed a uniform seismic force of 10H psf to the retaining wall in addition to the normal active pressures. The walls should also be designed to resist a point load applied at the midpoint of the wall, equal to ½ of the maximum applied surcharge (if any).

Wall Drainage - The above values have been provided assuming that a back-of-wall drain system will be installed to prevent build-up of hydrostatic pressures. This drainage system may consist of a prefabricated drainage panel (i.e. Miradrain) or a gravel and filter fabric type system. The walls should be waterproofed to prevent the transmission of efflorescence through the walls. The waterproofing should be specified by the designer, though we recommend the use of Bituthene, Miradri, or other similar waterproofing membrane.

Either drainage system should be installed with a minimum 3 inch diameter perforated pipe incorporated into the subslab granular section. Ideally the base of the pipe should be placed atop 1 to 2 inches of gravel, with its top even with the elevation of the basement subgrade (i.e. under the gravel. Perforations should be placed face-down (at 5 and 7 o'clock). Preferably, the exterior basement walls should be aligned with the exterior face of the slab to provide a planar surface for waterproofing installation across the cold joint.

If used, the gravel system should consist of a minimum 12 inch wide column of drain rock (¾ inch rock or ³/₈ inch pea gravel) extending the full width of the wall. The rock should continue to within 6 inches of finished grade. Prior to backfilling with the drain rock, a layer of filter fabric (Mirafi 140N or approved equivalent) should be placed against all soil surfaces to separate the rock and soil. The filter fabric should wrap over the top of the gravel and then a 6 inch thick cap of native soils should be placed at the top of the drain. If concrete flatwork is to directly overlay the back-of-wall drain, then the drain rock should continue to the base of the concrete. Additionally, where the drain will be located within crawlspace area, the gravel should continue to the crawlspace ground surface without the soil cap.

If prefabricated drainage panels are used, these panels should dead-end into the subslab gravel for collection under the slab. The tops of the panels should be sealed and secured in accordance with the manufacturer's specifications. The base of the drainage panels should extend down below the top of the filter fabric-wrapped drain rock.

Floor - The basement floor should consist of a mat slab. The entire slab should be underlain by at least 4 inches of clean, crushed drain rock. The drain rock should be covered by a moisture barrier which conforms to ASTM E1745-97 (e.g. Stego Wrap or an approved equivalent). The moisture barrier should wrap up the edges of the mat slab to be overlapped by the basement wall waterproofing. Perforated collector pipes should be embedded within the drain rock around the perimeter of the slab and at 20 foot spacing (one-way) under the slab to carry any water which gathers within the drain rock to the back-of-wall drain discharge location. The need for any sand over the top of the vapor barrier should be determined by the slab designer or architect.

Garage, Window Well and Access Well Drainage – The garage floor sump, and any window Well and Access Well Drainage may be tight lined to the same sump pump used for under-slab and wall drainage. This sump should be located in an easily accessible location (i.e. not exterior to the basement walls), and may discharge into the area drain system. A high water alarm should be provided in the sump. A backup generator may be prudent. No downspouts should discharge into the garage, or to any window well or stairwell/depressed patio.

Slabs-on-Grade

The at-grade building floors should not consist of concrete slabs-on-grade (although the basement floor may consist of a mat slab – see above). However, the driveway, any sidewalks or patios may consist of conventional concrete slabs-on-grade, though it should be expected that some post-construction shifting of such slabs may occur. We have provided guidelines to help reduce post-construction movements, however, it is nearly impossible to economically eliminate all shifting.

To help reduce cracking, we recommend slabs be a minimum of 4 inches thick and be nominally reinforced with #4 bars at 18 inches on center, each way. Slabs which are thinner or more lightly reinforced may experience undesirable cosmetic cracking. However, actual reinforcement and thickness should be determined by the structural engineer based upon anticipated usage and loading.

In large non-interior slabs (e.g. patios, garage, etc.), score joints should be placed at a maximum of 10 feet on center. In sidewalks, score joints should be placed at a maximum of 5 feet on center. All slabs should be separated from adjacent improvements (e.g. footings, porches, columns, etc.) with expansion joints. Interior floor slabs will experience shrinkage cracking. These cosmetic cracks may be sealed with epoxy or other measures specified by the architect.

All interior slabs (including garage slab) should be underlain by a minimum of 4 inches of clean ¾ inch crushed drain rock. The drain rock should be covered by a vapor barrier which conforms to ASTM E1745-97 (e.g. Stego Wrap or an approved equivalent). The architect or structural engineer should determine if sand is required over the vapor barrier.

Slabs which will be subject to light vehicular loads <u>and</u> through which moisture transmission is not a concern (e.g. driveway) should be underlain by at least 6 inches of compacted baserock, in lieu of any sand and gravel. Exterior landscaping flatwork (e.g. patios and sidewalks) may be placed directly on proof-rolled soil subgrade materials (e.g. no granular subgrade), however, they will be potentially subject to greater amounts of shifting and moisture transmission.

As stated previously, in pavement (concrete or asphalt) areas to receive vehicular traffic, all baserock materials should be compacted to at least 95 percent of their MDD. Also, the upper 6 inches of native soil subgrade beneath any pavements should be compacted to at least 95 percent of its MDD.

Drainage

Due to the flat nature of the site, it will be important to provide good drainage improvements at the property.

Surface Drainage - Adjacent to any buildings, the ground surface should slope at least 5 percent away from the foundations within 5 feet of the perimeter. Impervious surfaces should have a minimum gradient of 2 percent away from the foundation.

Surface water should be directed away from all buildings into drainage swales, or into a surface drainage system (i.e. catch basins and a solid drain line). "Trapped" planting areas should not be created next to any buildings without providing means for drainage (i.e. area drains).

All new roof eaves should be lined with gutters. The downspouts may be connected to solid drain lines, or may discharge onto paved surfaces which drain away from the structure. The downspouts may be connected to the same drain line as any catch basins, but must not connect to any perforated pipe drainage system

Drainage Discharge - The surface drain lines should discharge at least 15 feet away from the building, preferably at the street or creek. The discharge location(s) should be protected by energy dissipaters to reduce the potential for erosion. Care should be taken not to direct concentrated flows of water towards neighboring properties. This may require the use of multiple discharge points.

The under-slab and back-of-wall drain lines should discharge independently from the surface drainage system (except as noted above in the Basement section). A sump pump will be required for these subdrain systems. The surface and subsurface drain systems should not be connected to one another, except as discussed above for the basement light and stairwells.

Drainage Materials - Drain lines should consist of hard-walled pipes (e.g. SDR 35 or Schedule 40 PVC). In areas where vehicle loading is not a possibility, SDR 38 or HDPE pipes may be used. Corrugated, flexible pipes may not be used in any drain system installed at the property.

Surface drain lines (e.g. downspouts, area drains, etc.) should be laid with a minimum 2 percent gradient (¼ inch of fall per foot of pipe). Any subsurface drain systems (e.g. footing drains) should be laid with a minimum 1 percent gradient (1/8 inch of fall per foot of pipe).

Utility Lines

Unless they pass through the perimeter footing drain system, all utility trenches should be backfilled with compacted native clay-rich materials or a concrete plug within 5 feet of any buildings. This will help to prevent migration of surface water into trenches and then underneath the structures' perimeter. The rest of the trenches may be compacted with other native soils or clean imported fill. Only mechanical means of compaction of trench backfill will be allowed. Jetting of sands is not acceptable.

Trench backfill should be compacted to at least 90 percent of its MDD. However, under pavements, concrete flatwork, and footings the upper 12 inches of trench backfill must be compacted to at least 95 percent of its MDD.

Pavement

The new driveway may consist of concrete, interlocking pavers, or asphaltic concrete over Caltrans Class II aggregate base (baserock). The asphalt should have a minimum thickness of 3inches. The baserock should have a minimum thickness of 8 inches. All of the baserock and the upper 6 inches of soil subgrade should attain a minimum compaction of 95 percent of its MDD. Any fill below this layer should attain a minimum of 90 percent relative compaction.

Plan Review and Construction Observations

The use of the recommendations contained within this report is contingent upon our being contracted to review the plans, and to observe geotechnically relevant aspects of the construction.

We should be provided with a full set of plans to review at the same time the plans are submitted to the building/planning department for review. A minimum of one working week should be provided for review of the plans.

At a minimum, our observations should include: compaction testing of fills and subgrades; basement excavation; footing excavations; slab and driveway subgrade preparation; installation of any drainage system (e.g. back-of-wall, under-slab, and surface), and final grading. A minimum of 48 hours notice should be provided for all construction observations.

LIMITATIONS

This report has been prepared for the exclusive use of the addressee, and their architects and engineers for aiding in the design and construction of the proposed development. It is the addressee's responsibility to provide this report to the appropriate design professionals, building officials, and contractors to ensure correct implementation of the recommendations.

The opinions, comments and conclusions presented in this report were based upon information derived from our field investigation and laboratory testing. Conditions between or beyond our borings may vary from those encountered. Such variations may result in changes to our recommendations and possibly variations in project costs. Should any additional information become available, or should there be changes in the proposed scope of work as outlined above, then we should be supplied with that information so as to make any necessary changes to our opinions and recommendations. Such changes may require additional investigation or analyses, and hence additional costs may be incurred.

Our work has been conducted in general conformance with the standard of care in the field of geotechnical engineering currently in practice in the San Francisco Bay Area for projects of this nature and magnitude. We make no other warranty either expressed or implied. By utilizing the design recommendations within this report, the addressee acknowledges and accepts the risks and limitations of development at the site, as outlined within the report.

Respectfully Submitted;
GeoForensics, Inc.

Daniel F. Dyckman, PE, GE
Senior Geotechnical Engineer, GE 2145

Bernard A. Atendido
Field Engineer

cc: 5 to addressee