GEOTECHNICAL INVESTIGATION
5-STORY MIXED-USE BUILDING
2100 SAN PABLO AVENUE
BERKELEY, CALIFORNIA
September 9, 2015
2790-1, L-30339

Mr. Ali Kia Shabahangi
Spirit Living Group
126 Haight Street
San Francisco, CA 94102

RE: Geotechnical Investigation
5-Story Mixed-Use Building
2100 San Pablo Avenue
Berkeley, California

Dear Mr. Shabahangi:

At your request, we have performed a geotechnical engineering investigation for the proposed new mixed-use building to be constructed at 2100 and 2120 San Pablo Avenue, in Berkeley, California. The approximate site coordinates (WGS84) for this location are 37.868°N, 122.292°W. The project location is shown on the Vicinity Map, Figure 1.

1.00 PROPOSED CONSTRUCTION

We understand that the proposed project consists of a new five-story, mixed-use building with an underground parking garage. The street level story will most likely be concrete construction (to be used as retail space) and the upper four levels will be wood-framed construction (to be used as residential units). The below grade parking garage may extend up to about 14 feet below the street level to accommodate parking stackers.

Plans or details for the proposed new building were not provided to us at the time of our investigation. However, building loads are anticipated to be typical for this type of construction based on buildings in the area having similar characteristics.

2.00 PURPOSE

The purpose of our investigation was to evaluate the suitability of the site for the proposed mixed-use development from a geotechnical standpoint and to provide geotechnical design and construction criteria for the following aspects of the work:

- Site preparation and earthwork;
- Building foundation system;
- Building code seismic design parameters;
• Retaining walls; and

• Site drainage;

3.00 SCOPE

As outlined in our proposal dated July 28, 2015, the scope of our work to accomplish the stated purpose included:

• A reconnaissance of the site and portions of the immediate surrounding properties to evaluate general geotechnical and site conditions;

• A review of published geotechnical materials with data relevant to the site;

• A field subsurface exploration program consisting of drilling two exploratory borings at the site to evaluate the properties of the materials recovered;

• Laboratory index, classification, and strength tests on samples recovered;

• Geotechnical engineering analyses of the collected data; and

• Preparation of this geotechnical investigation report for the proposed development.

The scope of our services did not include an environmental assessment or investigation for the presence of hazardous or toxin materials in the soil, groundwater, or air on, below, or around the site. An evaluation of the potential presence of sulfates in the soil, or other corrosive, naturally-occurring elements was beyond our scope.

4.00 SITE INVESTIGATION

4.01 Existing Geotechnical Data Review

A variety of published materials were reviewed to evaluate geotechnical data relevant to the subject site. These sources included geotechnical literature, reports, and maps published by various public agencies. Maps which were reviewed included topographic, geologic, and preliminary photointerpretive landslide maps prepared by the United States Geological Survey, as well as geologic, landslide, and fault maps prepared by the California Geological Survey (formerly the California Division of Mines and Geology). A detailed citation of materials reviewed is presented in the References section at the end of this report.

4.02 Review of Previous Investigations

We reviewed subsurface information from previous investigations conducted by our firm and by other firms in the immediate vicinity of the subject site. Specifically, we reviewed information for investigations conducted at 1122 University Avenue, University Avenue between 9th and 10th Streets, and 1800 San Pablo Avenue. A summary of our review is presented in “Section 5.01, Data from Previous Investigations”. A detailed citation of these reports is also included in the References section at the end of this report.
4.03 **Surface Reconnaissance Visits**

Surface reconnaissance visits were performed at various times between July and August 2015. These visits were intended to make observations of surface conditions present, to note whether any obvious geotechnical concerns were exposed, and to mark the proposed boring locations.

4.04 **Subsurface Exploration**

Our subsurface exploration program was performed on August 10, 2015, to investigate and sample the subsurface materials at the site. Two borings were drilled at the site to a depth of about 31½ feet at the locations shown on the Site Plan, Figure 2.

The two borings were drilled within the existing property on the parking lot. The borings were drilled using truck-mounted drill rig equipment with solid stem flight augers. An Alan Kropp & Associates (AKA) engineer directed the drilling operations, logged the subsurface materials encountered, and obtained samples at frequent intervals. Soil samples of the materials encountered were obtained using a 2-inch outside diameter (O.D.) Standard Penetration Test (SPT) and a 3-inch O.D. California Modified sampler. The samplers were driven with a 140-pound hammer falling 30 inches. The hammer blows required to drive the sampler the final 12 inches of each 18-inch drive are presented on the attached boring logs. Following the drilling operations, the borings were grouted in accordance with the drilling permit requirements of the City of Berkeley, Toxics Management Division. The samples obtained were transported to our laboratory for subsequent visual observation and geotechnical testing.

Approximate measurements of the unconfined strength of selected soil samples were performed during the drilling operations using a pocket penetrometer testing device. These values are shown on the boring logs at their respective depths. Detailed boring logs are included in Appendix A.

4.05 **Laboratory Testing**

Our geotechnical laboratory testing program was directed toward a quantitative and qualitative evaluation of the physical and mechanical properties of the soils underlying the site. The following geotechnical laboratory tests were performed on selected soil samples:

- Water content per ASTM Test Designation D-2216;
- Dry Density per ASTM Test Designation D-2937;
- Atterberg Limits per ASTM Test Designation D-4318; and
- Percent passing No. 200 sieve per ASTM Test Designation D-1140.

The tests were conducted in general accordance with the current edition of the referenced standards at the time the tests were performed. The results of these tests are presented on the boring logs at the appropriate sample depths.

5.00 **SITE CONDITIONS**

The topographic map for this area (the Oakland West Quadrangle) prepared by the United States Geological Survey indicates the site is located at an elevation of approximately 55 feet on the gently sloping flatland between the Berkeley Hills and the San Francisco Bay.
A geologic map of the area by Radbruch (1957) indicates the site is underlain by the Temescal Formation. This formation is typically described as clayey gravel, sandy and silty clay, and sand-clay-silt mixtures of a pale to dark yellowish-orange color. Gravel in this unit consists of quartz, soft sandstone, shale, chert and other igneous rock fragments. The Temescal Formation covers most of the surface between the Berkeley Hills and the San Francisco Bay and is estimated to extend anywhere from 5 to 60 feet in thickness.

A more recent geologic map by Witter (2006) indicates the site is underlain by Holocene alluvial fan deposits. The accompanying text of the map indicates this unit typically includes sand, gravel, silt, and clay and is moderately to poorly sorted, and moderately to poorly bedded. These deposits are mapped on the west side of the East Bay hills from Oakland to Richmond and their liquefaction susceptibility is typically moderate where the ground water is within 15 feet of the ground surface. Deposits where the ground water is considerably lower may be less susceptible.

A creeks and watershed map of the area by Sowers (2009) indicates the nearest creek is Strawberry Creek which formerly flowed about 500 feet north from the site toward the bay and which currently flows within an underground culvert below city streets.

The California Geologic Survey has released a map covering this area which indicates areas (Seismic Hazard Zones) that may be prone to earthquake-induced ground failure (landsliding and/or liquefaction) during a major earthquake. The map (CGS, 2003) indicates areas where sufficient concern exists to merit a site-specific evaluation, not necessarily that the hazard is actually present. The site is located immediately adjacent to the boundary of a State of California designated Seismic Hazard Zone (SHZ) for potential liquefaction hazards. The City of Berkeley website indicates the site is NOT within the liquefaction hazard zone but lots immediately south and west of the site are within the hazard zone which requires additional investigation. If the site were determined to be within this zone, significant addition investigative and analysis efforts would be required to meet state standards.

The site is approximately 2.1 miles west of the nearest active trace of the Hayward fault (California Division of Mines and Geology, 1982; Lienkaemper, 1992). The site is also located about 16.4 miles northeast and 15.8 miles southwest of the active San Andreas and Concord faults, respectively (USGS, 2006). The site is not located within any Alquist-Priolo Earthquake Fault Zone designated by the State of California.

5.01 Data from Previous Investigations

Materials reported from previous investigations by AKA and other firms at nearby sites are summarized in the following section.

5.01.1 1122 University Avenue

In 2004, Lawrence B. Karp Consulting Engineer (LBK) conducted a geotechnical investigation for the University Condominium at 1122 University Avenue. The site is located less than 400 feet northwest from the subject site. University Condominium has a full size underground basement and is supported on a mat foundation. Three exploratory borings were drilled to depths between 14½ and 23 feet below ground surface. The materials encountered generally consisted of soft to very stiff clays and loose to medium dense clayey sands with varying amounts of silts and trace gravel. Groundwater was observed at about 6 feet below the ground surface.
5.01.2  University Avenue between 9th and 10th Street

In 2005, AKA conducted a preliminary geotechnical investigation for a proposed five-story building to be constructed at University Avenue between 9th and 10th Streets. The site is located about 600 feet northeast from the subject site. Two exploratory borings were drilled to depths between 41 and 51 feet. In general, the materials encountered consisted of fill and native soils. The fill materials consisted of firm to soft clay and very loose clayey sands with some silt. The native soils were generally described as firm to hard lean clays and fat clays, loose to medium dense clayey sands and some layers of clayey gravels. Groundwater levels were measured at about 8 feet below the ground surface.

5.01.3  1800 San Pablo Avenue

In 2003, AKA conducted a geotechnical investigation for Delaware Court project, constructed at 1800 San Pablo Avenue. This site is about 1300 feet north of the subject site. Three exploratory borings were drilled to depths between 21 and 31 feet below ground surface. The materials encountered generally consisted of up to 3 feet of fill soils, classified as stiff, lean clay with sand and silt, underlain by native materials. The native soils consisted of firm to hard lean, silty clays with varying amounts of sands and fine gravel and interbedded layers of dense to medium dense clayey gravel. Groundwater was observed at depths between 11 and 13 feet during drilling and at 8½ feet below ground surface several hours after drilling.

5.02  Surface

The site is roughly rectangular in shape with maximum plan dimensions of approximately 125 feet by 200 feet. The site is located at the southwest corner of the intersection of San Pablo Avenue and Addison Street. The property is bordered on the west by two-story houses and on the south by a single-story building.

The parcel at 2100 San Pablo is currently occupied by a single story building, which is located on the northeast corner of the lot and has plan dimensions of about 40 feet by 40 feet. The parcel at 2120 San Pablo consists of a narrow strip of land along the southern side and it contains a single story building with approximate plan dimensions of 40 feet by 25 feet. The rest of the site is currently a parking lot and an unpaved driveway along the western side of the property. The parking lot area and the rear driveway are separated by a 2- to 3-foot-high retaining wall.

A topographic survey was not available at the time of our investigation. However, approximate surface elevation data obtained from Google Earth Pro suggests the terrain varies in elevation from about 55 feet at the northeast corner to 51 feet at the southwest.

During our reconnaissance, no areas of major instability were observed in the vicinity of the proposed new building.

5.03  Subsurface

The materials encountered in our exploratory borings generally consisted of clays with sand, sandy clays and clayey sands, with some layers containing significant amounts of fine, angular gravel. The top 4½ to 5½ feet of the surficial materials appear to have characteristics of artificial fill with a stiff to very stiff consistency. The fill material is underlain by native firm to very stiff lean and fat clayey soils with
varying amounts of silt, sand and fine gravel that extend to the maximum depth explored of about 31½ feet. Some interbedded clayey, sand layers of loose consistency were also encountered. These native soils are relatively consistent with materials observed at nearby locations during previous investigations. Atterberg Limits tests conducted on samples at about 1½ and 15½ feet below existing grade resulted in Plasticity Indices (PI) of 31 and 48, respectively; these values are indicative of soils having a high to critical potential for expansive behavior.

Detailed descriptions of the materials encountered in the borings are found on the boring logs presented in Appendix A. Soil classifications were adapted from ASTM D2488 and Caltrans Soil and Rock Classification Manual, which are based on the Unified Soil Classification System (USCS). A Key to Exploratory Boring Logs, Figure A-1, is also included in Appendix A. The attached logs and related information depict subsurface conditions only at the specific locations shown on the Site Plan and on the particular date designated on the logs. These logs may have been modified from the original logs recorded during drilling as a result of further study on the collected samples, laboratory tests, or other efforts. Also, the passage of time may result in changes in the subsurface conditions due to environmental changes. The locations of the borings were approximately determined by measuring off of existing structures at the site and the ground surface elevations at the boring locations were approximately determined by interpolation of topographic data. The location and elevation should be considered accurate only to the degree implied by the method used.

Groundwater was encountered at a depth of 30½ feet in Boring 1 one hour after drilling. Groundwater in Boring 2 was encountered at a depth of 25 feet and it was at a depth of about 17 feet, 30 minutes after drilling.

It should be noted that groundwater measurements in the borings might have been made prior to allowing a sufficient period of time for the equilibrium groundwater conditions to become established. In addition, fluctuations in the groundwater level may occur due to variations in rainfall, temperature, and other factors not evident at the time the measurements were made.

6.00 EVALUATIONS AND CONCLUSIONS

6.01 General Site Suitability

Based on our investigation, it is our opinion the site is suitable for the construction of the proposed mixed-use development from a geotechnical standpoint. However, all of the conclusions and recommendations presented in this report should be incorporated in the design and construction of the project to minimize possible geotechnical problems.

The primary considerations for geotechnical design at the site are:

- The shrink/swell behavior of the surficial soils;
- The presence of existing fill soils at the site;
- Groundwater considerations;
- Excavations and temporary shoring;
- Earthquake hazards; and
Foundation selection.

Each of these conditions is discussed individually below.

6.02 Expansive Surficial Soils

The surficial clay soils are highly expansive, and are prone to significant volume changes (shrinkage and swelling) with seasonal fluctuations in soil moisture. Such shrink/swell behavior can damage shallow foundation elements or other elements located directly on them such as sidewalks and driveways. To help minimize tilting and cracking of these we recommend they be underlain by a layer of imported, non-expansive material and compacted in accordance with the compaction recommendations provided in Section 7.01.4.

6.03 Existing Fill

Based on the results of our subsurface exploration, it appears that the site is underlain by about 5 feet of fill soils. The proposed excavations for the parking garage are anticipated to be in the order of 14 feet and will extend through the fill layer. However, if any other critical elements are to be supported within the fill layer, we recommend these old fill soils be over-excavated and replaced with engineered fill placed in accordance with the recommendations provided in this report.

6.04 Groundwater Considerations

Groundwater was encountered in Boring 2 at about 25 feet below the existing grade during drilling, and it was measured at about 17 feet before grouting. Data from the nearby sites indicate groundwater levels were encountered at approximate depths between 6 and 13 feet. We believe the variations in the groundwater level observed during our investigation versus those reported at the nearby sites can be attributed to the proximity of those sites to the Strawberry Creek. A map of the area with approximate depths of historically highest ground water levels (CGS, 2003) indicates the depth to ground water at the site may be as shallow as 5 feet.

Considering that the subsurface exploration was completed in mid-August of one of the driest years in the history of California, the observed groundwater may be at the lower end of any anticipated range of seasonal fluctuations. Based upon the information obtained in our subsurface exploration, we judge that a design groundwater level at 10 feet below the existing grade may be used for design purposes.

We understand that excavations for the parking garage will extend to a depth of approximately 14 feet below grade. The contractor should be made aware of the potential for high groundwater and a temporary de-watering method during construction may still become necessary. Temporary construction dewatering methods may include sumps and pumps placed in a low spot within the excavations. Several sumps and pumps may be required depending on the magnitude of water encountered. The design and implementation of temporary construction de-watering is considered the responsibility of the contractor.

Caution should be exercised to prevent softening of the subgrade soils exposed within the excavations. Equipment operated upon saturated subgrade soils tends to cause rutting and weakening, which will required over-excavation of the weakened subgrade. Standing water within the excavation can also cause weakening of the subgrade soils. A temporary mud slab or gravel pad may needed at the base of the garage and/or parking lifts excavations to provide a clean, dry working area.
Given the potential for an increase in the groundwater level over the life of the building, we recommend the garage be designed to resist lateral and uplift hydrostatic pressures and appropriate waterproofing of the walls and floor be installed. A waterproofing expert should be consulted to provide recommendations.

6.05 Excavations and Temporary Shoring

Excavations of up to 14 feet below existing grade are anticipated as part of this project. Temporary shoring and/or underpinning should be implemented by the contractor to protect adjacent improvements during site excavations. Existing improvement that may be affected by the proposed development include, but are not limited to, adjacent structures, sidewalks, curbs, pavements, and underground utilities. The contractor is responsible for installation and performance of all shoring and underpinning measures.

6.06 Building Foundation

A concrete mat bearing on native materials or engineered fill is suitable for support of the proposed structure. Where foundation elevations are near or below the design groundwater level and drainage provisions do not provide for lowering of groundwater within the structure footprint, foundation and floor will be subjected to hydrostatic uplift forces. Hydrostatic uplift can be resisted by a combination of the weight of the structure itself and structural hold-downs.

6.07 Earthquake Hazards

As noted earlier, the subject site is located in the highly seismic San Francisco Bay Area, and there is a strong probability that a moderate to severe earthquake will occur during the life of the structure. The site is not mapped in the immediate proximity of any active or inactive faults; therefore, the likelihood of fault rupture directly below the proposed home is very remote.

During strong earthquakes, various forms of ground failure can occur, such as liquefaction and/or landsliding. Due to the relatively level topography on the site and in the site vicinity, earthquake-induced landsliding is not considered a site hazard. Liquefaction generally occurs in relatively loose granular (and sometime silty) soils that are below the groundwater table. The site is located immediately adjacent to the boundary of a State of California designated Seismic Hazard Zone (SHZ) for potential liquefaction hazards. However, we performed a preliminary screening evaluation of liquefaction potential for the granular soils encountered below the groundwater table and our results suggest that soil liquefaction could occur within the clayey sand layers.

The most likely consequence of soil liquefaction occurring at the site would be several inches settlement of the ground below the mat foundation. Our recommendation to support the proposed building on a reinforced mat foundation was done taking into consideration the results of our preliminary screening evaluation of liquefaction potential.

The proposed development will very likely experience strong ground shaking during a major earthquake in the life of the structure. The California Building Code has adopted provisions for incorporation of strong ground shaking into the design of all structures. Our recommendations for geotechnical parameters to be used in the structural design for the project are presented in “Section 7.06, California Building Code Seismic Design Parameters”.
7.00 RECOMMENDATIONS

It is the responsibility of you or your representative to confirm that the recommendations presented in this report are called to the attention of the contractor, subcontractors, and any governmental body which may have jurisdiction and that these recommendations are carried out in the field.

7.01 Site Preparation and Earthwork

7.01.1 Clearing and Site Preparation

The area of the proposed development should initially be cleared of selected surface and subsurface obstructions including existing foundations. Underground utilities that interfere with the proposed construction should be re-routed or abandoned. Holes resulting from the removal of underground obstructions extending below the proposed finished grade should be cleared and backfilled with suitable materials compacted in accordance with our recommendations presented in “Section 7.015, Compaction”.

7.01.2 Excavations and Shoring

As previously mentioned, it appears that excavations at the site will extend to a depth of approximately 14 feet below existing grade. Due to the depth of the anticipated excavations and the potential for presence of relatively shallow groundwater at the site it is anticipated that temporary shoring will be required for this project.

Temporary shoring should be used as required to prevent the movement of materials exposed in the face of the excavations. We recommend the excavations and subsequent parking structure construction be continuous in order to minimize the length of time the excavations is exposed. However, since we have no control over the methods and timing used by the contractor, the stability of any excavation is solely the responsibility of the contractor. It is however, recommended that our firm review the shoring plans in order to evaluate the potential interaction between the temporary shoring and the permanent structure.

7.01.3 Subgrade Preparation

Following site preparation and completion of proposed excavations, a representative of our firm should observe the base of the excavations to determine if problematic areas exist. The exposed soils in those areas to receive structural fill, slabs-on-grade, or mats should be firm, unyielding, and compacted to the requirements for structural fill. Soft or yielding subgrade soils should be excavated to expose firm, non-yielding materials. Proof-rolling may be helpful in identifying soft or yielding subgrade areas. The subgrade soils should be scarified to a depth of 6 inches. The scarified soils should then be moisture conditioned to at least 3 percent above optimum water content and compacted to the specified relative compaction.

It is possible that exposed subgrade soils may be excessively wet or dry depending on the moisture content at the time of construction. If the subgrade soils are too wet, they may be dried by aeration, mixing with drier materials, or lime/cement treatment.

7.01.4 Material for Fill

On-site soils below the cleared site having an organic content of less than 3 percent by volume are suitable for use as fill except in areas where specific fill materials are recommended. Fill placement at the
site should not contain rocks or lumps greater than 6 inches in greatest dimension with not more than 15 percent larger than 2.5 inches. In addition, imported select fill used at the site should be a non-expansive material with a plasticity index of 12 or less.

7.01.5 Compaction

Clayey soils should be moisture conditioned to at least 3 percent over optimum water content and compacted to at least 90 percent relative compaction by mechanical means only as determined by ASTM Test Designation D1557 (latest revision). Sandy soils should be moisture conditioned to near optimum water content and compacted to at least 95 percent relative compaction. The upper 6 inches of subgrade soils and base rock materials should be compacted to at least 95 percent relative compaction. Fill should be placed on a firm, unyielding surface in lifts not exceeding 8 inches in uncompacted thickness.

7.01.6 Trench Backfill

Pipeline trenches should be backfilled with fill placed in lifts not exceeding 8 inches in uncompacted thickness. Native backfill materials should be compacted to at least 90 percent relative compaction and granular import material should be compacted to at least 95 percent relative compaction. These compaction recommendations assume a reasonable “cushion” layer around the pipe.

If imported granular soil is used, sufficient water should be added during the trench backfilling operations to prevent the soil from “bulking” during compaction.

7.02 Mat Slab Foundation

We recommend that the new building be supported on a reinforced concrete mat slab foundation system. The mat slab should be a minimum of 18 inches thick and the base of the mat slab should extend at least 12 inches below the adjacent ground surface. The mat can be designed assuming an allowable bearing pressure of 1,000 pounds per square foot for dead plus live loads, with a one-third increase for all loads including wind or seismic. This allowable bearing pressure is a new value; therefore, the weight of the mat can be neglected for design purposes. The mat should be integrally connected to all portion of the structure so the entire foundation system moves as a unit. The mat should be reinforced with top and bottom steel in both directions to allow the foundation to span local irregularities that may result from potential differential settlement. As a minimum, we recommend that the mat be reinforced with sufficient top and bottom steel to span as a simple beam an unsupported distance of at least 10 feet. The mat can be designed using a modulus of subgrade reaction of 100 kips per cubic foot. The mat foundation should also be designed to resist uplift pressures resulting from hydrostatic loading due to groundwater. The design groundwater can be considered at 10 feet below existing ground surface.

Lateral loads on the structure may be resisted by passive pressures acting against the sides of the mat. We recommend an allowable passive pressure equal to an equivalent fluid weighing 300 psf per foot of depth (factor of safety ≈ 2). Alternatively, an allowable friction coefficient of 0.20 (factor of safety ≈ 2) can be used between the bottom of the mat and the subgrade soils. If the perimeter of the mat is poured neat against the soils, the passive pressure and friction coefficient may be used in combination.

It is recommended that a waterproofing expert be consulted in regards to waterproofing details for this project. We also recommend that the specifications for the mat require moisture emission tests to be performed on the mat prior to the installation of the flooring. No flooring should be installed until safe moisture emission levels are recorded for the type of flooring to be used.
7.03 Driveway Ramp & Slabs on Grade

We recommend that portions of the driveway ramp underlain by highly expansive surficial soils be supported on 12 inches non-expansive compacted fill. A representative of our firm should observe subgrade conditions to assist with identifying areas requiring over-excavation. Prior to final construction of the driveway ramp, the subgrade surface should be proof-rolled to provide a smooth, firm surface for driveway support.

Exterior slabs-on-grade should be supported on a minimum of 12 inches of imported, compacted, non-expansive fill. The surficial soils beneath the non-expansive fill should scarified to a depth of at least 6 inches, moisture conditioned, and compacted in accordance with compaction recommendations presented in Section 7.01.5.

7.04 Retaining Walls

Retaining walls should be designed to resist both lateral earth pressures and any additional lateral loads caused by surcharge loads on the adjoining ground surface. Undrained retaining walls should be designed to resist lateral earth pressures, hydrostatic loads, surcharge loads, and seismic loading. We recommend walls be designed using an equivalent fluid pressure (not including surcharge loading) of 60 pounds per cubic foot (pcf) above the groundwater level and 95 pcf below the groundwater level. For retaining wall design, we recommend assuming a design groundwater depth of 10 feet below the currently existing grade.

Basement walls 6 feet or greater in height should also be designed for a temporary seismic load. The temporary seismic load can be modeled as a uniform lateral pressure applied over the height of the wall of 10H psf, where H is the height in feet. The provisional recommendations for seismic earth pressures on building basement wall, as provided by Lew, et al. (2010), were considered in development of the seismic load criteria given.

The values given above assume level backfill behind the wall with no surcharge loads. For additional surcharge loads, such as heavy slab loads, concentrated loads, or vehicular loading, design pressures should be increased by an additional uniform pressure equivalent to one-half of the maximum anticipated surcharge load applied to the surface behind the wall.

Structural backfill placed behind the retaining wall should be compacted in accordance with the requirements provided in Section 7.01.5. Retaining walls should be supported on mat slab foundations designed in accordance with “Section 7.02, Mat Slab Foundation”.

7.05 Surface Drainage

We recommend that collected surface water be transmitted through gutters and downspout to closed pipes that discharge to an appropriate discharge facility. Flexible pipe (flexline), 2,000-pound crush pipe, leachfield, and ASTM F810 pipe are NOT recommended for use in these drainage systems because of the likelihood of damage to the pipe during installation due the weak strength of these pipes. In addition, these drainpipes are sometime difficult to clean with mechanical equipment without damaging the pipe. We recommend the use of Schedule 40 PCV, SDR 35 PVC or ABS, Contech A-2000 PCV drainpipe, or approved equivalent for the drain system.
Positive surface gradient of at least 2 percent should be provided adjacent to the structure to direct water away from the foundations and slabs toward a suitable discharge facility. Ponding of surface water should not be allowed adjacent to the structure or on pavements. The project civil engineer should develop the provisions necessary to conform to current city/county regulations. Such measures may include retention basins, grassy swales, or other provisions, which may allow some water to eventually flow onto the street and into nearby inlet leading to the storm drain systems. We should note that suitable discharge facilities do not include so called “dry wells” and these should be avoided.

7.06 California Building Code Seismic Design Parameters

Based on our review of the site location, assumed soil conditions, and the 2013 California Building Code (CBC), we recommend the following parameters be used for seismic design of the building:

- Site Class = D
- Mapped Spectral Acceleration for Short Period (S_s, Site Class B) = 2.034g
- Mapped Spectral Acceleration for 1-Second Period (S_1, Site Class B) = 0.831g
- Maximum Considered Earthquake (MCE) Spectral Response Acceleration for Short Period (S_{MS}, Site Class D) = 2.034g
- MCE Spectral Response Acceleration for 1-Second Period (S_{M1}, Site Class D) = 1.246g
- Design Spectral Response Acceleration for Short Period (S_{DS}, Site Class D) = 1.356g
- Design Spectral Response Acceleration for 1-Second Period (S_{D1}, Site Class D) = 0.831g

7.07 Supplemental Recommendations

As previously mentioned, at the time of our investigation project plans or detail had not been developed. Once the project layout and dimensions are established supplemental recommendations may be necessary. If necessary, please contact us when near final plans have been completed.

7.08 Plan Review

We recommend our firm be provided the opportunity for a general review of the geotechnical aspects of the final plans and specifications for this project in order that the geotechnical recommendations may be properly interpreted and implemented. If our firm is not accorded the privilege of making the recommended review, we can assume no responsibility for misinterpretation of our recommendations.

7.09 Construction Observation

The analyses and recommendations submitted in this report are based in part upon the data obtained from the two soil borings. The nature and extent of variations across the site may not become evident until construction. If variations then become apparent, it will be necessary to re-examine the recommendations of this report.

We recommend our firm be retained to provide geotechnical engineering services during the earthwork, foundation construction, and drainage phases of the work. This is to observe compliance with the design concepts, specifications, and recommendations and to allow design changes in the event that subsurface conditions differ from those anticipated prior to the start of construction.
It should be noted that earthwork and foundation observations by our firm, as the project geotechnical engineer of record, are required by most cities and counties. Drainage observations by our firm are not typically required, but in our experience, we have often discovered adverse drainage installations that otherwise would have created problems following construction, and this is why we recommend our services be utilized. Nonetheless, it is usually the owner's prerogative whether they wish to engage our services or simply rely on the quality of their contractor's work regarding drainage improvements.

In order to effectively accomplish our observations during the project construction, we recommend that a pre-construction meeting be held to develop a mechanism for proper communications throughout the project. We also request that the client or the client's representative (the contractor) contact our firm at least two working days prior to the commencement of any of the items listed above. If our representative makes a site visit in response to a request from the client or the client's representative and it turns out that the visit was not necessary, our charges for the visit will still be forwarded to the client.

7.10 **Wet Weather Construction**

Although it is possible for construction to proceed during or immediately following the wet winter months, a number of geotechnical problems may occur which may increase costs and cause project delays. The water content of on-site soils may increase during the winter and rise significantly above optimum moisture content for compaction of subgrade or backfill materials. If this occurs, the contractor may be unable to achieve the recommended levels of compaction without using special measures and would likely have to:

- Wait until the materials are dry enough to become workable;
- Dispose of the wet soils and import dry soils; or
- Use lime or cement on the native materials to absorb water and achieve workability.

If utility trenches or excavations are open during winter rains, then caving of the trenches or excavations may occur. Also, if the trenches fill with water during construction, or if saturated materials are encountered at the anticipated bottom of the excavations, excavations may need to be extended to greater depths to reach adequate support capacity than would be necessary if dry weather construction took place.

We should also note that it has been our experience that increased clean-up costs will occur, and greater safety hazards will exist, if the work proceeds during the wet winter months. Furthermore, engineering costs to observe construction are increased because of project delays, modifications, and rework.

If you have any questions concerning this letter, please call us.
Very truly yours,

Alan Kropp, G.E.
Principal Engineer

AL/AK/jc

Copies: Addressee (3+PDF) - aksdevelopmentgroup@gmail.com

2790-1 2100 San Pablo GI Report
REFERENCES

Published Data

California Division of Mines and Geology, 1982, Special Studies Zone Map, Oakland West Quadrangle.


Unpublished Reports


Alan Kropp & Associates, 2005, titled “Preliminary Geotechnical Investigation, Townhouse/Condominium and Commercial Center, University Avenue between 9th and 10th Streets, Berkeley, California,” prepared for Mr. Lyman Jee, dated May 24, 2005, Job Number 2332-2A.

FIGURES
**LEGEND**

- **B-1** - Approximate location of exploratory boring
- **Approximate limits of proposed project**

**SITE PLAN**

**2100 SAN PABLO AVENUE**
Berkeley, California

<table>
<thead>
<tr>
<th>PROJECT NO.</th>
<th>DATE</th>
<th>FIGURE</th>
</tr>
</thead>
<tbody>
<tr>
<td>2790-1</td>
<td>September 2015</td>
<td>2</td>
</tr>
</tbody>
</table>
APPENDIX A

Boring Logs
## Soil Classification Chart

### Primary Divisions

<table>
<thead>
<tr>
<th>Gravels</th>
<th>Clean Gravels</th>
<th>Cu ≥ 4 and 1 ≤ Cc ≤ 3.4</th>
<th>GW</th>
<th>Well-graded gravel</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Gravels with Finer More than 12% Fines</td>
<td>Cu &lt; 4 and 1 &gt; Cc &gt; 3</td>
<td>GP</td>
<td>Poorly-graded gravel</td>
</tr>
<tr>
<td>Sands</td>
<td>Clean Sands</td>
<td>Cu ≥ 6 and 1 ≤ Cc ≤ 3</td>
<td>SW</td>
<td>Well-graded sand</td>
</tr>
<tr>
<td></td>
<td>Sands with Finer More than 12% Fines</td>
<td>Cu &lt; 6 and 1 &gt; Cc &gt; 3</td>
<td>SP</td>
<td>Poorly-graded sand</td>
</tr>
</tbody>
</table>

### Secondary Divisions

- **Gravels**
  - More than 50% of Coarse Fraction Retained on No. 4 Sieve
  - Finer than 5% Finers
- **Gravels with Finer More than 12% Finers**
- **Sands**
  - Less than 5% Finers
  - Finer than 12% Finers
- **Sands with Finer More than 12% Finers**
- **Silt and Clays**
  - Liquid Limit Less than 50%
  - Liquid Limit 50% or More
- **Highly Organic Soils**
  - Primarily Organic Matter, Dark in Color, and Organic Odor

### Criteria

- **Cu ≥ 4 and 1 ≤ Cc ≤ 3.4**
- **Cu < 4 and 1 > Cc > 3**
- **Cu ≥ 6 and 1 ≤ Cc ≤ 3**
- **Cu < 6 and 1 > Cc > 3**

### Grain Sizes

<table>
<thead>
<tr>
<th>U.S. Standard Series Sieve</th>
<th>Clear Square Sieve Openings</th>
</tr>
</thead>
<tbody>
<tr>
<td>200</td>
<td>40</td>
</tr>
<tr>
<td>Silts and Clays</td>
<td>Fine</td>
</tr>
</tbody>
</table>

### Abbreviations

- **INDEX TESTS**
  - LL: Liquid Limit (%)
  - PI: Plasticity Index (%)
  - -200: Passing No. 200 Sieve (%)

- **STRENGTH TESTS**
  - PP: Field Pocket Penetrometer test of unconfined compressive strength (tsf)
  - TV: Field Torvane test of shear strength (psf)
  - UC: Laboratory unconfined compressive strength (psf) (ASTM D 2166/2166M-13)
  - TXUU: Laboratory unconsolidated, undrained triaxial test of undrained shear strength (psf) (ASTM D 2850-03a)

- **MISCELLANEOUS**
  - ATOD: At time of drilling
  - psf/tsf: pounds per square foot / tons per square foot
  - psi: pounds per square inch (indicates relative force required to advance Shelby tube sampler)

### Symbols

- Standard Penetration Test Split Spoon (2-inch O.D.)
- Modified California Sampler (3-inch O.D.)
- Thin-walled Sampler Tube (either Pitcher or Shelby) (3-inch O.D.)
- Rock Core
- Bag Sample
- Groundwater Level during drilling
- Groundwater Level after drilling

---

**KEY TO EXPLORATORY BORING LOGS**

- **ALAN KROPP & ASSOCIATES**
  - Geotechnical Consultants
  - 2100 San Pablo Avenue
  - Berkeley, California

- **PROJECT NO.** 2790-1
- **DATE** September 2015
- **FIGURE** A-1
<table>
<thead>
<tr>
<th>BORING NO.</th>
<th>PROJECT NO.</th>
<th>SURFACE ELEVATION</th>
<th>DEPTH TO GROUNDWATER</th>
<th>DRILL RIG</th>
<th>BORING DIAMETER</th>
<th>DATE DRILLED</th>
<th>LOGGED BY</th>
</tr>
</thead>
<tbody>
<tr>
<td>2790-1</td>
<td>2100 SAN PABLO AVENUE</td>
<td>55 (see notes)</td>
<td>30.5 feet (see notes)</td>
<td>B-24, Solid Flight Auger</td>
<td>4 in. inches</td>
<td>8/10/15</td>
<td>AL</td>
</tr>
</tbody>
</table>

**DESCRIPTION AND REMARKS**

<table>
<thead>
<tr>
<th>SAMPLE</th>
<th>COLOR</th>
<th>CONSISTENCY</th>
<th>SOIL TYPE</th>
<th>DEPTH (ft)</th>
<th>MOISTURE CONTENT (%)</th>
<th>DRY DENSITY (pcf)</th>
<th>OTHER TESTS</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>AC/ Base rock - small amount of light brown sandy silt</td>
<td>Dark Gray &amp; Black</td>
<td>CL</td>
<td>1</td>
<td>15</td>
<td>86</td>
<td>PP = 4.5 tsf LL = 19 PI = 31 -200 = 62%</td>
</tr>
<tr>
<td>2</td>
<td>FILL</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>CLAY, Lean, Sandy - with silt, fine-grained sand, some fine, angular gravel (FILL)</td>
<td>FILL</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>CLAY, Lean - with fine to medium-grained sand, some silt, some thin lense of clayey sand, trace fine gravel, moist</td>
<td>Brown to reddish brown</td>
<td>CL</td>
<td>12</td>
<td>12</td>
<td>88</td>
<td>PP = 4.5 tsf</td>
</tr>
<tr>
<td>5</td>
<td>CLAY, Lean - with sand to sandy, medium plasticity, with fine to coarse-grained sand, trace fine sub-angular gravel, trace silt</td>
<td>Brown to reddish &amp; yellowish brown</td>
<td>CL</td>
<td>21</td>
<td>21</td>
<td>92</td>
<td>PP = 2.0 tsf</td>
</tr>
<tr>
<td>6</td>
<td>CLAY, Fat - with rounded, coarse-grained sand, some silt, moist</td>
<td>reddish brown, trace Bluish Gray, trace Black</td>
<td>CH</td>
<td>13</td>
<td>26</td>
<td>92</td>
<td>PP = 3.0 tsf LL = 69 PI = 48 -200 = 82%</td>
</tr>
<tr>
<td>7</td>
<td>CLAY, Fat - with sand, trace fine gravel, moist</td>
<td>Brown to reddish brown</td>
<td>CH</td>
<td>18</td>
<td>18</td>
<td>105</td>
<td>PP = 3.0-3.25 tsf</td>
</tr>
<tr>
<td>8</td>
<td>SAND, Clayey - fine to medium-grained, trace coarse sand, trace silt, moist, some wet pockets</td>
<td>Brown to yellowish brown</td>
<td>SC</td>
<td>21</td>
<td>21</td>
<td>99</td>
<td>-200 = 46%</td>
</tr>
<tr>
<td>9</td>
<td>CLAY, Lean - with silt to silty, trace fine sand, wet</td>
<td>reddish brown &amp; gray</td>
<td>CL</td>
<td>35</td>
<td>35</td>
<td>85</td>
<td></td>
</tr>
</tbody>
</table>

Bottom of boring at 31.5 feet.

**NOTES:**

(Continued on Next Page)
1. Groundwater was encountered at approximately 31 feet at the time of drilling and was at a depth of about 30.5 feet 1 hour after drilling. (See report for discussion.)

2. Stratification lines represent the approximate boundaries between material types and the transitions may be gradual.

3. Penetration resistance values (blow counts) enclosed in brackets ([ ]) were recorded with a 3.0-inch O.D. Modified California sampler; these are not standard penetration resistance values.

4. Elevations were determined from Google Earth Pro data, dated May 11, 2015.

5. Approximate unconfined compressive strength values were recorded in the field using a pocket penetrometer. These values are shown on the logs and are preceded by the symbol "PP".
## EXPLORATORY BORING LOG

**DRILL RIG:** B-24, Solid Flight Auger  
**SURFACE ELEVATION:** 52 (see notes)  
**LOGGED BY:** AL  
**DEPTH TO GROUNDWATER:** 17 feet (see notes)  
**BORING DIAMETER:** 4 in. inches  
**DATE DRILLED:** 8/10/15  

### DESCRIPTION AND REMARKS

<table>
<thead>
<tr>
<th>BORING NO.</th>
<th>PROJECT NO.</th>
<th>SHEET</th>
<th>BORING NO.</th>
</tr>
</thead>
<tbody>
<tr>
<td>2790-1</td>
<td>2100 SAN PABLO AVENUE</td>
<td>September 2015</td>
<td>2</td>
</tr>
</tbody>
</table>

**NOTES:**

*(Continued on Next Page)*
1. Groundwater was encountered at approximately 25 feet at the time of drilling and was at a depth of about 17 feet .5 hours after drilling. (See report for discussion.)

2. Stratification lines represent the approximate boundaries between material types and the transitions may be gradual.

3. Penetration resistance values (blow counts) enclosed in brackets ([]) were recorded with a 3.0-inch O.D. Modified California sampler; these are not standard penetration resistance values.

4. Elevations were determined from Google Earth Pro data, dated May 11, 2015.

5. Approximate unconfined compressive strength values were recorded in the field using a pocket penetrometer. These values are shown on the logs and are preceded by the symbol "PP".

**EXPLORATORY BORING LOG**

<table>
<thead>
<tr>
<th>BORING NO.</th>
<th>DATE</th>
<th>SHEET</th>
<th>BORING NO.</th>
</tr>
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<tbody>
<tr>
<td>2790-1</td>
<td>September 2015</td>
<td>2 of 2</td>
<td>2</td>
</tr>
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</table>