

GEOTECHNICAL INVESTIGATION
NEW CITY OF BERKELEY ANIMAL SHELTER
1 BOLIVAR DRIVE
BERKELEY, CALIFORNIA



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1258-8, L-28519

Mr. John Hykes
Design Community & Environment
1625 Shattuck Avenue, Suite 300
Berkeley, CA 94709

RE: Geotechnical Investigation
New City of Berkeley Animal Shelter
1 Bolivar Drive
Berkeley, California

Dear Mr. Hykes:

This report presents the results of our geotechnical investigation for the new City of Berkeley Animal Shelter to be located at 1 Bolivar Drive in Berkeley, California. During the course of our study, we obtained information regarding the proposed project from you, Primitivo Suarez-Wolfe, Assistant Architect for the City of Berkeley, and Stephen DeJesse, the project Structural Engineer.

1.00 PROJECT DESCRIPTION

As shown on the Vicinity Map, Figure 1, the project site is at the north end of Berkeley's Aquatic Park Lake directly east of Highway 80 near the intersection of Addison Street and Bolivar Drive. We understand the project will include demolishing the existing two-story, wood-framed structure on the site and constructing a new two-story, steel-framed building. The new building will occupy roughly the same footprint as the existing building. The structural engineer is estimating new column (static) loads of up to 150 kips. We understand site grades will be raised approximately 18 inches as part of the planned construction and the existing pavements and parking areas will be removed and replaced. It is our understanding that the existing building is currently supported on a shallow foundation. As shown on the Site Plan, Figure 2, an East Bay Municipal Utility District (EBMUD) easement runs along the southern edge of the existing building. The easement is 25 feet wide and contains a 66-inch-diameter, concrete sewer main.

2.00 PURPOSE AND SCOPE

The purpose of our investigation was to characterize the subsurface conditions at the site and provide geotechnical recommendations for the design of the new building. As outlined in our August 26, 2008 proposal, the scope of our Geotechnical Investigation included:

- Conducting a site reconnaissance;
- Reviewing geotechnical materials;
- Examining aerial photographs;

- Drilling geotechnical test borings;
- Performing geotechnical laboratory tests on samples obtained from the test borings;
- Engineering analyses, including a liquefaction evaluation; and
- Preparing this report presenting design-level recommendations for the project.

Please note that the scope of our services did not include an environmental assessment or investigation for the presence of hazardous, toxic, or corrosive materials on, below, or around the site.

3.00 METHODS OF INVESTIGATION

3.01 Review of Existing Data

A variety of published sources were reviewed to evaluate geotechnical data relevant to the subject site. These sources included geotechnical literature, reports, and maps published by various public agencies including, but not limited to, topographic, geologic, liquefaction, seismic hazard zones, and fault zone maps prepared by the United States Geological Survey and by the California Geological Survey (formerly the California Division of Mines and Geology). We also reviewed geotechnical data contained in a 2007 investigation by our firm for the proposed boardwalk and plaza improvements to be located at the north end of Aquatic Park Lake adjacent to the subject site. Information obtained from the map and literature review is discussed in Section 4.01, Data Review/Geologic Setting, and a complete list of the sources used in our investigation is presented at the end of this report.

3.02 Aerial Photo Review

Five sets of black and white aerial photographs were examined with a stereoscope to assist in our evaluation of the site conditions. The photographs were taken in 1939, 1946, 1957, 1969, and 1983. The photographs have scales ranging from 1:12,000 to 1:23,600. Further details regarding the identification of these photographs are presented in the attached list of references. A summary of our air photo observations can be found in Section 4.02, Aerial Photograph Information.

3.03 Site Reconnaissance Visits

Site reconnaissance visits were conducted at various times during the course of our field investigation in September 2008. These visits were intended to make observations of the surficial conditions present and to note whether any obvious geotechnical concerns were exposed. Observations made during our site reconnaissance visits are discussed in Section 4.03, Surface Conditions.

3.04 Subsurface Exploration

Our subsurface exploration program was performed on September 13, 2008, to investigate and sample the subsurface materials. Three borings were drilled at the site to depths that varied between approximately 22.5 feet and 50.5 feet at the locations shown on the Site Plan, Figure 2. The logs of the borings are attached in Appendix A.

Boring 1 was drilled with portable equipment utilizing a solid stem flight auger. Borings 2 and 3 were drilled with track-mounted drilling equipment. Boring 2 was drilled utilizing a hollow stem flight auger, while Boring 3 was drilled with a solid stem flight auger.

During drilling, our engineering staff logged the borings, directed the drilling, and visually classified the soil in general accordance with ASTM D2488 classifications which are based on the Unified Soil Classification System (USCS). The USCS is described on the Key to Exploratory Boring Logs, Figure A1. Soil samples were obtained using a 2-inch outside diameter (O.D.) Standard Penetration Test (SPT) sampler without liners and a 3-inch O.D. California Modified sampler with liners (unless otherwise noted on the logs). The samplers were driven with a rope and cathead system 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 boring logs. Following the field operations, the borings were grouted in accordance with the City of Berkeley permit regulations.

We note that the attached boring logs and related information depict subsurface conditions only at the approximate locations shown on the Site Plan on the particular date designated on the boring logs. The attached boring logs represent our current interpretation of the subsurface materials at the boring locations and the passage of time may result in changes in the subsurface conditions. The boring locations indicated on the attached materials were determined by measuring from existing curbs and building walls and should be considered approximate.

A summary of our findings from our subsurface exploration can be found in Section 4.04, Subsurface Conditions.

3.05 Laboratory Testing

Our geotechnical laboratory testing program was directed toward a quantitative and qualitative evaluation of the physical properties of the soils that underlie the site. The following geotechnical laboratory tests were performed:

- 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 minus #200 sieve per ASTM Test Designation D1140.

The results of the tests are presented on the boring logs at the corresponding sample depths.

4.00 SITE CONDITIONS

4.01 Data Review/Geologic Setting

The topographic map for this area (the Oakland West Quadrangle) prepared by the United States Geological Survey (USGS) shows the site is located on a relatively level plain adjacent to the coastline of the San Francisco Bay. The site is shown at an approximate elevation of +5.0 feet MLLW (mean lower low water). The surrounding areas are generally level with grades gradually rising toward the base of the Berkeley Hills, which is about 2.5 miles east of the site. The 1878 Historic Atlas of Alameda County (Thompson and West, 1878) shows the original (circa 1850) shoreline of the San Francisco Bay just to the west of 2nd Street about one-half block east of the site, with an inlet extending further to the east in the vicinity of the currently existing University Avenue.

The site is mapped as being underlain by artificial fill (Graymer, 2000) and artificial fill over San Francisco Bay Mud (Knudsen, et al., 2000). The artificial fill is described as man-made deposits of various materials and ages.

The site is approximately 4.0 kilometers (km) southwest of the Hayward fault, 23.7 km west of the Calaveras fault, 25.5 km northeast of the San Andreas fault, and 26.5 km southwest of the Concord-Green Valley fault (ICBO/California Department of Conservation Division of Mines and Geology, 1998). The site is not located within an Alquist-Priolo Earthquake Fault Zone designated by the State of California (CDMG, 1982).

Studies by the United States Geological Survey's Working Group on California Earthquake Probabilities have estimated a 62 percent probability of at least one magnitude 6.7 or greater earthquake occurring in the San Francisco Bay Region before the year 2031 (WGCEP, 2003). As part of their prediction, they estimated the probability to be 27 percent for a magnitude 6.7 or greater earthquake to occur on the Hayward/Rodgers Creek fault by the year 2031.

In 2003, the California Geological Survey released a Seismic Hazard Zone map for the Oakland West Quadrangle identifying areas that may be prone to earthquake-induced ground failure during a major earthquake. The map shows the site located within a zone that may be subject to earthquake-induced liquefaction (CGS, 2003).

Borings drilled for our 2007 geotechnical investigation for a proposed boardwalk and plaza area adjacent to the northern end of Aquatic Park Lake (immediately south of the subject site) encountered approximately 8 feet of fill material. The fill encountered was predominately clays and silts which varied in consistency from soft to stiff and which contained varying amounts of sands, gravels, wood pieces and other organics. Natural soils encountered below the fill generally consisted of firm to stiff clays with some inter-bedded sand and gravel lenses. Layers of relatively soft, low plasticity silts and clays that were generally encountered in the depth range of 5 to 9 feet in the borings were judged to be potentially liquefiable. Some potential for liquefaction in the deeper soil profile within the inter-bedded sand and gravel layers was also identified.

4.02 Aerial Photograph Information

The 1939 aerial photographs show a dike and road along the current alignment of Highway 80, creating Aquatic Park Lake from an area that was formerly part of San Francisco Bay. The subject site is a small lagoon adjacent to the northern tip of Aquatic Park Lake separated from the lake by a dike/road that wraps around the northern tip of the lake. In the 1946 photographs, the site remains as a small lagoon. By the time of the 1957 photographs, the lagoon that occupied the site had been filled, but the site remains undeveloped. In the 1969 photographs, the building and parking lot that currently occupy the site are in place. No significant changes in site conditions from the time of the 1969 photographs were noted in the 1983 photographs.

4.03 Surface Conditions

As shown on the Site Plan, Figure 2, the site is located just north of Aquatic Park Lake. The site is occupied by an existing two-story building with associated asphalt parking lot. The existing building shows no obvious indications of major distress; however, the surrounding pavement and parking lot areas are in fairly poor condition with multiple longitudinal cracks up to 1 inch wide and several areas of alligator cracking.

4.04 Subsurface Conditions

The borings drilled on the site encountered between about 8 and 9 feet of artificial fill overlying about 6 inches to 2.5 feet of native bay deposits which in turn overlie native alluvial deposits which extend to the bottom of the borings. Bedrock was not encountered in any of the borings. A brief description of the materials encountered is included in the following paragraphs:

Artificial Fill – The fill materials are poorly compacted and variable in consistency. Generally, the fill consisted of soft to stiff, lean to fat clays (CL and CH) with varying amounts of sand and gravel; medium dense, clayey gravel (GC); and medium dense, silty sand (SM). The near-surface fill was found to be moderately expansive with a Plasticity Index (PI) of 21.

Bay Deposits – The bay deposits found directly below the fill consisted of soft, silty fat clay (CH); loose, silty sand (SM); and very soft, sandy silt (ML).

Alluvial Deposits – The alluvial deposits generally consisted of firm to very stiff, lean to fat clays (CL and CH) and medium dense, silty and clayey sands (SM and SC).

Groundwater was encountered in the borings at the time of drilling at depths that ranged between approximately 10 and 13 feet. However, it is noted that the borings were grouted shortly after drilling in accordance with the City of Berkeley permit requirements. The borings were likely not left open for a sufficient period of time to establish equilibrium groundwater conditions. In addition, it is likely that fluctuations in the groundwater level will occur due to tidal fluctuations, seasonal variations in rainfall and other factors. Considering the site elevation and the close proximity of Aquatic Park Lake and the San Francisco Bay, we anticipate that the groundwater may at times be within 5 feet (or possibly even less) of the currently existing ground surface.

Detailed descriptions of the materials encountered in the borings are found on the boring logs presented in Appendix A. A Key to Exploratory Boring Logs, Figure A1, is also presented. 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 of 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.

5.00 EVALUATIONS AND CONCLUSIONS

5.01 General

The primary considerations for the geotechnical design at the site are:

- The presence of poorly compacted fill and soft, compressible bay deposits (up to 11 feet deep);
- Earthquake induced hazards including liquefaction and lateral spreading;
- Moderately expansive near-surface soils;
- Shallow groundwater; and
- Contaminated soil.

We have discussed these geotechnical issues with you, with Primitivo Suarez-Wolfe (Assistant Architect for the City of Berkeley), and with the project structural engineer (Stephen DeJesse). The two primary building support alternatives considered in order to address the geotechnical site conditions were: 1) support of the building on a driven pile foundation, and 2) support of the building on a mat slab foundation with ground improvement. Based on these discussions, it was decided to pursue the mat slab with ground improvement alternative.

We considered several different ground improvement approaches and decided that the use of Impact[®] Rammed Aggregate Piers (RAPs) was well suited for the site conditions. Impact[®] RAPs are vertical displacement elements that do not generate excessive (potentially contaminated) soil cuttings and are suitable for installation in shallow groundwater environments with a mixture of soil types. Ground improvement with Impact[®] RAPs to a depth of 25 feet below the bottom of the mat slab foundation is being recommended for this project. Ground improvement achieved with Impact[®] RAPs is anticipated to increase soil stiffness and also increase shear resistance in the reinforced zone, thereby reducing settlement and liquefaction/lateral spread potential in the reinforced soil zone. Additional information regarding the design and construction of Impact[®] RAPs is provided in Appendix B.

Our geotechnical recommendations for improving the existing ground with Impact[®] RAPs and supporting the building on a structural mat foundation are provided in Section 6.00, Recommendations. A more detailed discussion of the primary considerations for geotechnical design is provided below.

5.02 Poorly Compacted Fill and Compressible Bay Deposits

The near-surface soils at the site consisted of approximately 8 to 9 feet of fill with highly variable consistency underlain by soft and compressible bay deposits. Absent ground improvement, it is our opinion that there is the potential for several inches of differential settlement of the fill and bay deposits under the loads associated with the proposed building and raised site grade if the new building is supported on a shallow foundation system. Given the depth of the unsuitable material, the relatively high groundwater conditions anticipated at the site, and the potential for encountering contaminated soil and/or groundwater, removal and re-compaction of this unsuitable material is considered to be impractical. Ground improvement with Impact[®] RAPs is being recommended in order to address the settlement potential in the fill and compressible Bay Deposits that underlie the site.

5.03 Earthquake Hazards

5.03.1 Strong Ground Shaking and Seismic Performance

The site is likely to experience one or more episodes of strong ground shaking during the economic life of the project. The California Building Code (CBC) 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 seismic design of the structure are presented in Section 6.05, Building Code Seismic Design Parameters.

5.03.2 Fault Rupture

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 known active faults (CDMG, 1982 and Jennings, 1994); therefore, the likelihood of fault rupture is considered to be remote.

5.03.3 Liquefaction and Lateral Spreading

The site is mapped within a Seismic Hazard (Liquefaction) Zone by the State of California Geologic Survey (CGS). Liquefaction is a phenomenon whereby certain types of soils below groundwater lose strength in response to earthquake shaking. We utilized the recommendations presented in the NCEER/NSF summary report by Youd and Idriss (2001) to evaluate liquefaction susceptibility of coarse-grained soils. In these procedures, liquefaction susceptibility of coarse-grained soils is directly correlated to "corrected" Standard Penetration Test (SPT) blow counts.

Recent research has shown that fine-grained soils (silts and even clays) can also experience liquefaction-type behavior in response to earthquake shaking. We utilized the interim recommendations presented in the recent paper by Seed, et al. (2003) to evaluate liquefaction susceptibility of fine-grained soils. The interim recommendations correlate liquefaction susceptibility of fine-grained soils to their Plasticity Index (PI), Liquid Limit (LL), and natural water content (w_c).

We evaluated liquefaction potential using the data from the three site borings. The results of our analyses are presented on the attached Table 1. We note that for our analyses, the peak ground acceleration (PGA) (with 10 percent probability of being exceeded in 50 years) was obtained from the California Geological Survey (CGS) Probabilistic Seismic Hazards Mapping Ground Motion Page (<http://redirect.conservation.ca.gov/cgs/rghm/pshamap/pshamap.asp>). The moment magnitude (M_w) was obtained from published data on the Hayward fault. A moment magnitude of 7.1 and PGA of 0.6g were used in the analyses. For our analyses, we assumed a groundwater depth of 5 feet below the ground surface.

Boring 1 encountered one layer of soil that we consider potentially susceptible to liquefaction and which warranted further analysis as summarized below and in Table 1:

Clayey Sand (SC) – A medium dense predominantly coarse-grained layer encountered between depths of about 10.5 and 14 feet.

Boring 2 encountered four layers of soil that we consider potentially susceptible to liquefaction, which warranted further analysis as summarized below and in Table 1:

Silty Sand (SM) – A loose to medium dense, predominantly coarse-grained layer between depths of about 7.5 and 9.5 feet.

Silty Sand (SM) – A medium dense predominantly coarse-grained layer between depths of about 9.5 and 10.75 feet.

Clayey Sand (SC) – A medium dense predominately coarse-grained layer between approximately 10.75 and 15.5 feet.

Clayey Sand (SC) – A medium dense predominately coarse-grained layer between approximately 44.5 and 46.25 feet.

Boring 3 encountered two layers of soil that we consider potentially susceptible to liquefaction, which warranted further analysis as summarized below and in Table 1:

Sandy Silt (ML) – A very soft predominately fined-grained layer between approximately 8.5 and 11 feet.

Silty Sand (SM) – A medium dense predominately coarse-grained layer between approximately 16 and 17.25 feet.

As shown on Table 1, our analyses indicate that five of the seven layers considered potentially susceptible to liquefaction could liquefy during a major seismic event on a nearby fault. The silty sand layer between about 9.5 and 10.75 feet and the clayey sand layer between about 10.75 and 15.5 feet in Boring 2 had corrected SPT blow counts of 30 or more and were therefore judged to be too dense to liquefy.

5.03.4 Liquefaction Effects

Surface manifestations of liquefaction include settlement, bearing capacity failure, sand boils, and lateral spreading. Any of these effects, if severe enough, have the potential to cause damage to structures and other site improvements.

In Table 1, we calculate the amount of liquefaction-induced settlement that would be predicted to occur within each liquefied layer using standard analysis procedures (Tokimatsu and Seed, 1987). We evaluated the potential for lateral spreading, bearing capacity failure, and sand boils on a qualitative basis. The results of our evaluations are summarized below.

Settlement – As shown on Table 1, we calculate the potential for between approximately ½ to 1½ inches of dynamic, compressional settlement within the potentially liquefiable soil layers as encountered in the three site borings. It is noted that four of the five soil layers identified as susceptible to liquefaction during a major seismic event are located within 20 feet of the ground surface. The one layer identified as potentially liquefiable below a depth of 20 feet was a medium dense sand layer encountered in Boring 2 at a depth of approximately 45 feet. Borings 1 and 3 only extended to depths between 26.2 and 22.5 feet; therefore, for the purposes of our site analysis, we have assumed that the liquefaction potential in the deep soil profile beneath the site is characterized by the conditions as encountered in Boring 2. Ground improvement with Impact® RAPs extending to a depth of 25 feet below the bottom of the new building mat slab foundation is being recommended for this project. The ground improvement is anticipated to significantly reduce the liquefaction related settlement potential in the reinforced soil zone. While there would still be the possibility of liquefaction and related settlement in the deeper soil profile, the extent of liquefiable soil layers encountered below 25 feet is considered minimal. With ground improvement as recommended, we estimate total and differential liquefaction-induced settlement potential to be less than 1 inch.

Lateral Spreading – Lateral spreading can occur when there are laterally continuous liquefiable materials that extend to a free-face. The nearest free-face to the site is Berkeley Aquatic Park Lake which is located approximately 100 feet south of the site. Potentially liquefiable soils were encountered in all three of the site borings in the general depth range between about 7.5 and 14 feet. Potentially liquefiable soils were also encountered at a similar elevation in borings that were recently drilled by our firm for the proposed boardwalk and plaza improvements at the northern end of Aquatic Park Lake, which is immediately south of the subject site. In our opinion, absent ground improvement, there is a moderate to high potential for lateral spreading to significantly impact the proposed building site. As previously noted, ground improvement with Impact® RAPs that extend to a depth of 25 feet below the bottom of the new building mat slab foundation is recommended for this project. The ground improvement is anticipated to significantly reduce the liquefaction related lateral spread potential beneath the building footprint by increasing the shear resistance of the soil in the reinforced zone. In our opinion, the proposed ground improvement in combination with supporting the building on a stiff mat slab foundation will reduce the lateral spread hazard of the proposed building structure to an acceptable level. However, it should be recognized that there will still be the potential during a major seismic event for lateral spreading to effect exterior improvements, such as associated parking areas, roads and utilities.

Bearing Capacity Failure and Sand Boils – With the proposed mat slab construction and roughly 8 to 10 feet of non-liquefiable soils overlying the potentially liquefiable layers, it is likely that even absent ground improvement bearing capacity failure would not be a significant site issue. With the proposed ground improvement, it is our opinion that the potential for liquefaction induced bearing capacity failure associated with the proposed mat slab foundation system would be negligible. Sand boils are a potential in areas of un-

improved ground, potentially affecting exterior improvement such as pavements and utilities, but are not considered a significant design concern for the mat slab supported structure on improved ground.

5.04 Moderately Expansive Near-Surface Soils

Near-surface soils at the site were found to be moderately expansive. Expansive soils shrink and swell with changes in moisture and have the potential to damage improvements that are supported upon them. We recommend that planned near-surface improvements that are not part of the building mat slab foundation (e.g., exterior walkways, planters, etc.) be supported on a layer of non-expansive fill to mitigate the potential for future damage related to shrink/swell movements. It is our understanding that the project involves importation of fill to raise the site grades in the building area by approximately 18 inches. Use of non-expansive fill to raise site grades is recommended and it is anticipated that this fill layer may serve as most (or possibly all) of the required layer of select fill recommended below exterior near-surface improvements. Specific recommendations for non-expansive fill placement can be found in Section 6.01, Site Preparation and Earthwork.

5.05 Shallow Groundwater

We anticipate that the groundwater may at times be within 5 feet or possibly even less of the currently existing ground surface. We recommend carefully considering the elevations and locations of planned improvements in order to avoid excavations that extend below the groundwater.

5.06 Contaminated Soil

The scope of our services did not include an environmental assessment or investigation for the presence of hazardous, toxic, or corrosive materials on, below, or around the site. However, based on our conversations with the structural engineer and Primitivo Suarez-Wolfe, Assistant Architect for the City of Berkeley, we understand that soil and/or groundwater contamination is a potential concern at this site. During our subsurface investigation, we did observe hydrocarbon odors between depths of about 8 and 11 feet below the ground surface during sampling. Construction techniques that minimize the need for site excavations and/or dewatering are considered desirable. As noted previously, Impact[®] RAPs as proposed for ground improvement are vertical displacement elements that will not generate excessive (potentially contaminated) soil cuttings. The proposed mat slab construction is also anticipated to involve minimal site excavations. We also recommend grouting the gravel that is installed with the Impact[®] RAPs which will minimize the potential vertical migration of potential groundwater contaminants.

6.00 RECOMMENDATIONS

6.01 Site Preparation and Earthwork

6.01.1 Clearing and Site Preparation

Prior to the start of work, the contractor should locate and mark all active subsurface utilities in the general vicinity of the site. The contractor should protect all utilities that are to remain in and surrounding the site during onsite excavation and construction activities.

The site should then be cleared of the existing building and related foundations, pavements, sidewalks, and abandoned utilities. These materials should be removed from the site. Depressions resulting from the removal of underground obstructions that extend below the proposed finished grades should be cleared and the

depressions backfilled with suitable material compacted to the requirements given below in "Section 6.01.3, Subgrade Preparation and Fill Placement.

6.01.2 Material for Fill

All proposed fill materials should be approved by our firm prior to their use. The different types of fill material are described below:

General Fill – General fill is onsite material that has an organic content of less than 3 percent by volume and does not contain rocks or lumps larger than 6 inches in greatest dimension. General fill can be used anywhere except for areas that require non-expansive fill.

Non-Expansive Fill – Non-expansive fill should conform to the requirements for general fill, have a Plasticity Index (PI) no greater than 15, and have a liquid limit (LL) no greater than 40. A layer of non-expansive fill is required beneath exterior slabs on grade, and flatwork. See Section 6.06, Exterior Slabs on Grade and Flatwork, for more information.

Imported Fill – Import material should conform to the requirements for non-expansive fill and should be evaluated by our firm prior to its importation to the site.

Some of the materials cleared or excavated from the site may be suitable for re-use as general fill or non-expansive fill, from a geotechnical standpoint, if they can be processed to meet the above requirements. Material that cannot be mixed or processed to meet specification requirements should be disposed of offsite or stockpiled for other uses at the discretion of the owner. If the re-use of aggregate base or crushed asphalt is to be considered, it must first be approved by the owner in consultation with our firm.

6.01.3 Subgrade Preparation and Fill Placement

The subgrade surface in those areas to receive structural fill or mat slab construction 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. The exposed soils in those areas receiving structural fill should be scarified to a depth of 6 inches or the full depth of any existing shrinkage cracks. The scarified soils should then be moisture conditioned to slightly above optimum water content and compacted to at least 90 percent relative compaction based on the ASTM D-1557 test method (latest version).

All fill should be spread in lifts not exceeding 8 inches in uncompacted thickness, moisture conditioned, as necessary, and compacted to at least 90 percent relative compaction based on the ASTM D-1557 test method (latest version). In flexible pavement areas, the upper 6 inches of subgrade should be compacted to at least 95 percent relative compaction (per ASTM D-1557). AKA should observe proofrolling of pavement and slab-on-grade subgrades to confirm that they are firm and non-yielding. Aggregate base underneath flexible pavement should be compacted to at least 95 percent relative compaction.

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

6.01.4 Trench Backfill

Utility trenches should be backfilled with fill placed in lifts not exceeding 8 inches in uncompacted thickness. Trenches should be filled by placing a granular shading layer beneath and around the pipe, and then 6 to 12 inches of shading should be carefully placed and tamped above the pipe. The remaining portion of the trench should be backfilled with onsite or import soil. The backfill (above shading layers) should be placed and compacted to a minimum relative degree of compaction of 90 percent based on ASTM D1557. The compaction requirements given above should be considered minimum recommended requirements. If City and/or utility company specifications require more stringent backfill requirements, those specifications should be followed.

If imported granular soil is used, sufficient water should be added during the trench backfilling operations to prevent the soil from “bulking” during compaction. All compaction operations should be performed by mechanical means only. We recommend against jetting.

We recommend the contractor carefully evaluate the stability of all trenches and use temporary shoring, where appropriate. The design and installation of the temporary shoring should be wholly the responsibility of the contractor. In addition, all state and local regulations governing safety around such excavations should be carefully followed.

6.02 Ground Improvement with Impact® Rammed Aggregate Piers

Based on the site soil conditions and proposed building loads, we recommend improving the existing ground beneath the proposed building with Impact® Rammed Aggregate Piers (RAPs) and supporting the building on a structural mat foundation. RAP elements are designed and installed by a licensed and specialty ground improvement contractor. The Impact® RAP system is based on soil improvement that consists of ramming crushed rock columns into the existing soil. In addition, we recommend grouting the RAP columns during installation which will generate a higher capacity column and minimize contaminated groundwater migration. The Impact® RAP system uses vertical displacement RAP elements; therefore, no drilling or soil removal is required.

We have consulted with the local RAP installer, Farrell Design-Build Companies, Inc. (Farrell), regarding design capacities, and a preliminary feasibility letter produced by Farrell is included in Appendix C. Once the final building column loads and locations are established by the design team, Farrell can assist the design team by providing a complete design-build submittal with design recommendations, engineered plans and specifications. In addition, Alan Kropp & Associates will need to be retained to perform a geotechnical review of the RAP design.

The following minimum ground improvement design and performance requirements are recommended for this project:

- The minimum ground improvement area should include the entire building footprint plus 5 feet beyond the footprint, except where limited by existing adjacent improvements.
- Impact® RAP shaft lengths (RSL) should extend at least 25 feet below the bottom of the mat foundation.
- A minimum allowable RAP composite bearing capacity (DL+LL) of 1,500 pounds per square foot (psf) with an allowable 1/3 increase for seismic or wind loads should be developed across the

entire building footprint area. Based on our preliminary discussions with Farrell, it is anticipated that this can be accomplished with RAP element spacing of 6 to 7 feet on center.

- An allowable RAP composite bearing capacity (DL+LL) of 3,500 psf with an allowable 1/3 increase for seismic or wind loads should be developed in highly loaded areas such as below columns, as specified by the project structural engineer. Based on our preliminary discussions with Farrell, it is anticipated that this can be accomplished with RAP element spacing of 4 to 5 feet on center.
- Total and differential settlements under proposed building loads of less than 1 inch and ½ inch respectively, should be maintained with the improved ground.

We recommend the Impact® RAP system be monitored by our firm throughout the design and construction phases of the work. A quality assurance/quality control (QA/QC) program should be implemented to monitor drive depths, Impact® RAP shaft lengths (RSL), average lift thicknesses, installation procedures, aggregate quality, and densification of lifts.

Impact® RAP construction should include an indicator pier installation program which involves installing and load testing Impact® RAPs prior to the installation of the production piers. At least two indicator piers should be installed in two different locations within the building footprint. The location of the indicator piers should be selected by our firm and agreed upon by the RAP contractor. A full-scale load test should be performed on at least two indicator piers (one from each location). The allowable vertical capacity should be verified before any production piers are installed. Vertical capacity tests should be taken to 150 percent of the design load. The installation procedures used to install the indicator piers should be consistently used throughout the project. AKA should review and approve the contractor's indicator program and load testing procedures prior to implementation.

6.03 Surface Drainage

Positive surface drainage should be provided to direct surface water away from foundations of the proposed improvements. Ponding of surface water should not be allowed in any areas adjacent to the improvements. Rainwater should be collected in closed pipes that discharge away from the building or on non-permeable pavement or flatwork sloped away from the building.

6.04 Structural Mat Foundation

We recommend the new building be supported on a reinforced concrete mat foundation. The mat should be supported directly on the improved ground and Impact® RAPs. The bottom of the mat should be at least 18 inches below the lowest adjacent ground surface, and the mat should be at least 18 inches thick. The mat can be designed using an allowable bearing capacity of 1,500 psf for dead plus live loads and 2,000 psf for all loads including wind or seismic. In areas with high concentrated loads (i.e., beneath building columns), the Impact® RAPs should be spaced closer together to obtain a maximum allowable bearing capacity of 3,500 psf for dead plus live loads and 4,650 psf for all loads including wind or seismic. These allowable bearing pressures are net values; therefore, the weight of the mat can be neglected for design purposes. The mat should be integrally connected to all portions of the structure so that the entire foundation system acts as a unit. The mat should be reinforced with sufficient top and bottom steel (in both directions) to support an edge cantilever of 5 feet and a random interior clear span of 10 feet. For structural mat design purposes, a modulus of subgrade reaction of 200 kips per cubic foot can be used for the improved ground.

Lateral load resistance may be developed in friction between the foundation bottom and the supporting subgrade. An allowable friction coefficient of 0.35 (factor of safety ≈ 1.5) can be used between the bottom of the mat and the improved ground.

In areas where floor wetness would be undesirable, a vapor barrier (Stego 15 mil Class A, or equivalent) should be placed under the mat. Any tears in the retarder and all plumbing penetrations should be sealed with an approved mastic material. We also recommend that the specifications for the mat require moisture emission tests to be performed on the mat prior to the installation of any flooring. No flooring should be installed until safe moisture emission levels are recorded for the type of flooring to be used.

6.05 Building Code Seismic Design Parameters

Based on our review of the site location, geology, and the 2007 California Building Code (CBC), we recommend the following parameters be used for seismic design of the building:

- Site Class = D (after ground improvement)
- Mapped Spectral Acceleration for Short Period (S_S , Site Class B) = 1.630g
- Mapped Spectral Acceleration for 1-Second Period (S_1 , Site Class B) = 0.605g
- Maximum Considered Earthquake Spectral Response Acceleration for Short Period (S_{MS} , Site Class D) = 1.630g
- Maximum Considered Earthquake Spectral Response Acceleration for 1-Second Period (S_{M1} , Site Class D) = 0.908g
- Design Spectral Response Acceleration for Short Period (S_{DS} , Site Class D) = 1.086g
- Design Spectral Response Acceleration for 1-Second Period (S_{D1} , Site Class D) = 0.605g

6.06 Exterior Slabs-on-Grade and Flatwork

Exterior slabs-on-grade and flatwork can be supported directly on a 12-inch-thick layer of non-expansive fill placed in accordance with the recommendations outlined in Section 6.01, Site Preparation and Earthwork.

6.07 Flexible Pavement Design

Asphalt concrete pavements are planned for parking areas around the building. We developed the following recommended pavement sections using the Caltrans R-value design method for flexible pavements and assumed traffic indices.

Pavement Components

Traffic Index	Asphalt Concrete (inches)	Caltrans Class 2 Aggregate Base (inches)	Total Thickness (inches)
4.0	2.5	8.5	11.0
5.0	2.5	11.0	13.5
6.0	3.0	13.5	16.5

The above sections are based on an assumed R-value of 5. The R-value of the fill underlying pavement sections should be confirmed during construction. We note that R-values significantly higher than 5 could be used to substantiate a revised thinner and potentially more economical pavement section design.

The assumed traffic indices of 4.0 and 5.0 are considered to be reasonable values for automobile and light truck parking areas and access driveways, respectively. A traffic index of 6.0 is applicable for moderate truck access and parking areas. The traffic indices were not based on a traffic study; the project civil engineer should review the traffic indices to confirm that they are appropriate and select the areas where the sections will be used.

The subgrade beneath planned pavements should be compacted to at least 95 percent relative compaction per ASTM D-1557. Pavement subgrades should be proofrolled and confirmed to be uniformly firm and non-yielding prior to the placement of aggregate base. Aggregate base for use in pavements should conform to Caltrans Standard Specifications for Class 2 Aggregate Base. The aggregate base used in pavement sections should be compacted to at least 95 percent relative compaction as determined by ASTM D-1557.

6.08 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. This includes review of the RAP design and bid documents. If our firm is not accorded the privilege of making the recommended review, we can assume no responsibility for misinterpretation of our recommendations.

6.09 Construction-Phase Services

The analyses and recommendations submitted in this report are based in part upon the data obtained from the three 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.

AKA should review all submittals (including Impact[®] RAP design) from the contractors that are geotechnical in nature, before geotechnical materials are delivered or equipment is mobilized to the site.

We recommend our firm be retained to provide geotechnical engineering services during the construction of the proposed project. 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. During construction, AKA should observe the following:

- Impact[®] RAP installation and testing,
- Soil conditions exposed by site excavations,
- Subgrade preparation,
- Fill placement and compaction.

We 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 geotechnical related operation.

6.10 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 onsite 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, or dispose of the wet soils and import dry soils.

If excavations are left open during winter rains, caving of the walls may occur. Also, if excavations fill with water during construction or if saturated materials are encountered at the bottom of the excavation, the excavations may need to be dewatered and remedial work may be needed to provide an appropriate subgrade for foundations and/or non-expansive fill materials.

In general, we note that it has also been our experience that increased clean-up costs may be incurred, and greater safety hazards may exist, if the work proceeds during the wet winter months.

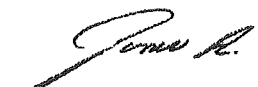
7.00 LIMITATIONS

This report has been prepared for the exclusive use of you and your consultants in accordance with generally accepted geotechnical engineering practices for specific application to the construction of the proposed new Berkeley Animal Shelter to be located at 1 Bolivar Drive in Berkeley, California. No other warranty, either expressed or implied, is made. In the event the nature, design, or location of the improvements differs significantly from what has been noted above, the conclusions and recommendations contained in this report should not be considered valid unless the changes are reviewed and the conclusions of this report are modified or verified in writing.

The findings of this report are valid as of the present date. However, the passing of time will likely change the conditions of the existing property due to natural processes or the works of man. In addition, due to legislation or the broadening of knowledge, changes in applicable or appropriate standards may occur. Accordingly, the findings of this report may be invalidated, wholly or partly, by changes beyond our control. Therefore, this report should not be relied upon after three years without being reviewed by this office.

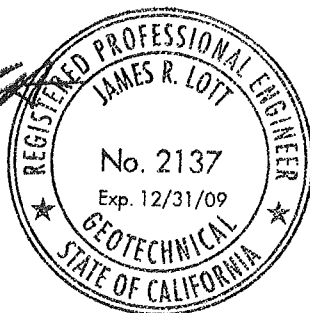
If you have any questions concerning this report, please call us.

Very truly yours,


James R. Lott, G.E.
Associate Engineer

DM/JRL/sa

Copies: Addressee (2)
Primitivo Suarez-Wolfe, City of Berkeley (2)
Stephen DeJesse, Ingraham-DeJesse Associates, Inc. (1)




Dona K. Mann, C.E.
Senior Engineer

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Published Materials

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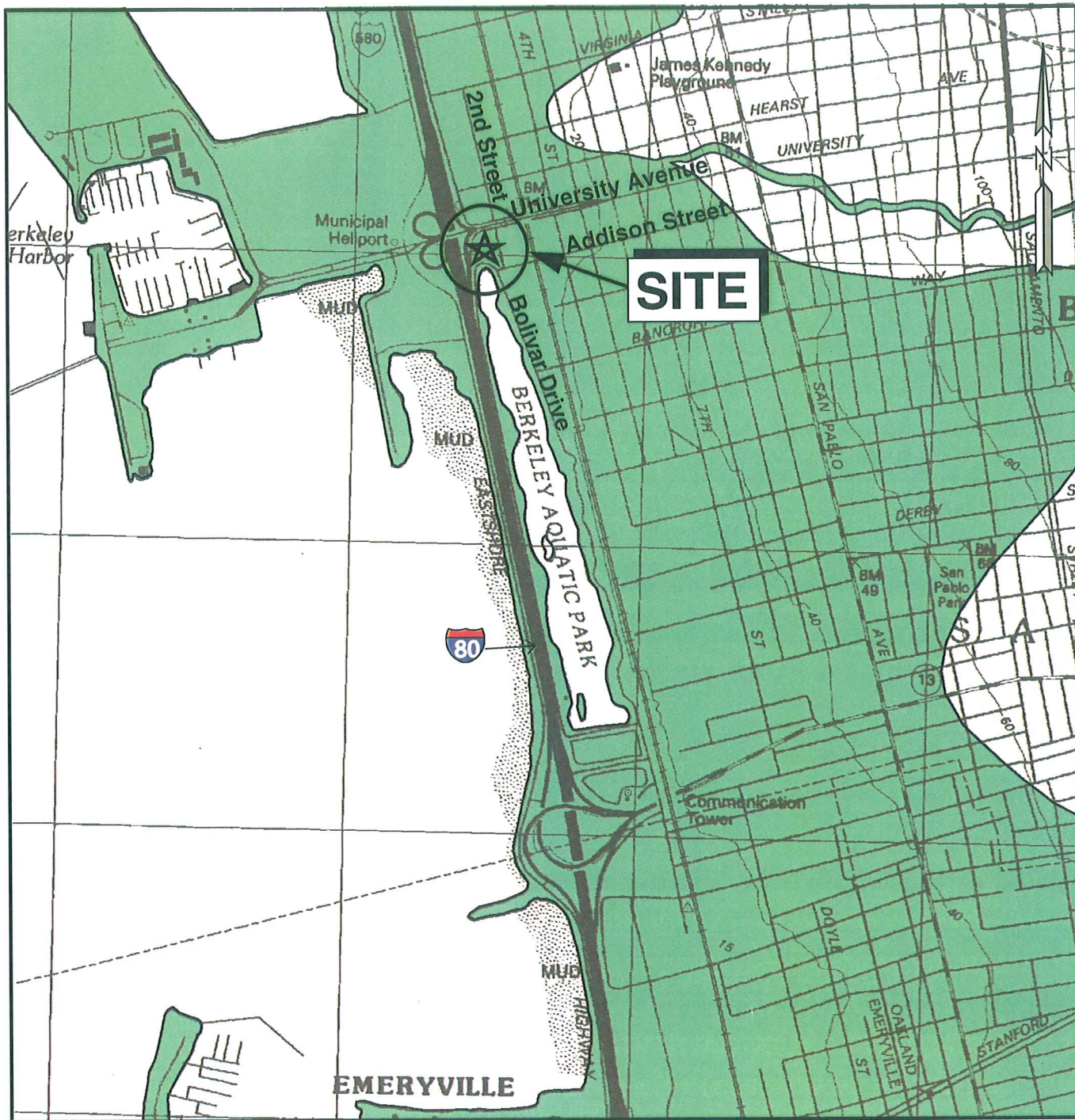
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Aerial Photographs

Type	Date	Scale	Photo No.	Source
Black and White, Stereo Pair	08/02/39	1:20,000	BUT-BUU-289-99, 100	United States Soil Conservation Service
Black and White, Stereo Pair	10/28/46	1:23,600	GS-CP-6-11,12	United States Geological Survey
Black and White, Stereo Pair	05/03/57	1:12,000	AV253-07-17,18	Pacific Aerial Surveys
Black and White, Stereo Pair	05/2/69	1:12,000	AV902-6-14,15	Pacific Aerial Surveys
Black and White, Stereo Pair	06/21/83	1:12,000	AV2300-7-13,14	Pacific Aerial Surveys



LEGEND:

 Potential Liquefaction Zone

Source: 1) USGS Oakland West Quadrangle, 1980
 2) CGS, "Seismic Hazard Zone Map" Oakland West Quadrangle, 2003



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VICINITY MAP

BERKELEY ANIMAL SHELTER
 1 Bolivar Drive; Berkeley, California

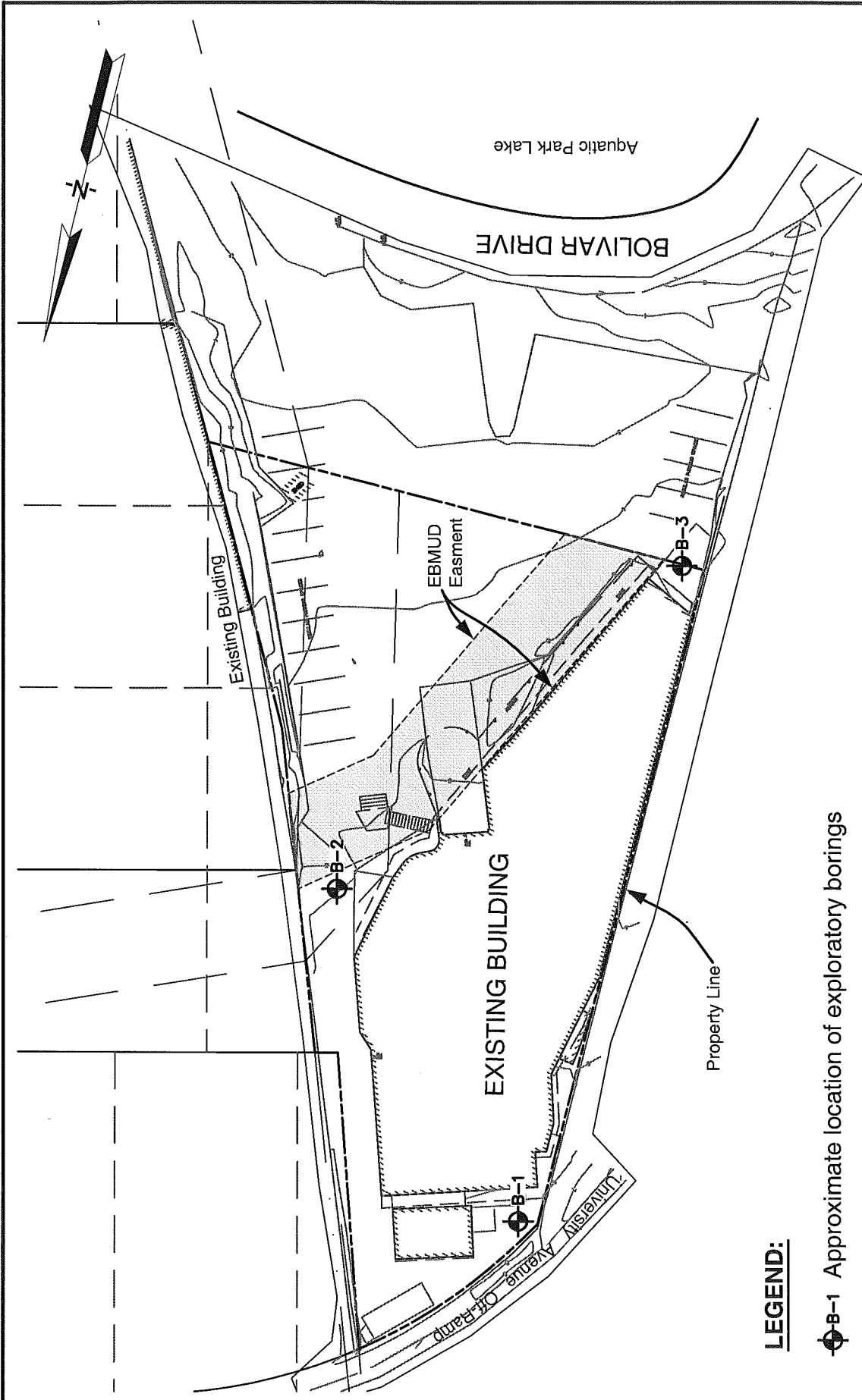
PROJECT NO.

1258-8

DATE

December 2008

FIGURE **1**



LEGEND:

⊙ B-1 Approximate location of exploratory borings

Base: "ALTA / ACSM LAND TITLE SURVEY, at 1 Bolivar Drive,"
drawn by Moran Engineering, Inc., dated August 2008.

SITE PLAN

BERKELEY ANIMAL SHELTER
1 Bolivar Drive; Berkeley, California

PROJECT NO.
1258-8

DATE
December 2008

FIGURE 2

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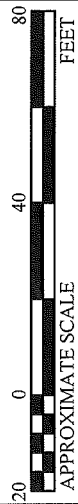


TABLE 1 - LIQUEFACTION ANALYSIS

Berkeley Animal Shelter
 Alan Kropp & Associates; Project # 1258-8
 10/8/08 By: DKM

Highlighted information must be entered depending upon site specific conditions.

C_R = Correction Factor for Rod Length; $10-13' = 0.75$; $13-20' = 0.85$; $20-33' = 0.95$; $33-99' = 1.0$
 C_S = Correction Factor for Sampling Method; SPT Sampler without liners = 1.2; Modified California Sampler = 0.63
 C_a = Correction Factor for Borehole Diameter; For Hollow Stem, use ID.; $2.5-4.5" = 1.0$; $5.9" = 1.05$; $7.9" = 1.15$
 C_E = Correction Factor for Hammer Energy Ratio; Automatic Trip Hammer = 1.2, Rope and Cathead = 1.0
 C_N = Correction Factor for Overburden Pressure

References:

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 Tokimatsu and Seed, 1987, Evaluation of settlements in sand due to earthquake shaking, Journal of Geotechnical Engineering, Vol. 113, No.8, (pg 405 in geotechnical engineering textbook)

Design M_w = 7.1
 Water table = 5.0 ft below ground surface

Boring 1																												
Youd, 2001												Tokimatsu and Seed (1987)																
Depth at Center of Layer (ft)	Unified Soil Class.	Layer Thickness (ft)	% Fines	N	C _R	C _S	C _B	C _E	N ₆₀	γ _m	σ _v	σ _v '	C _N	N _{1,60cs}	R _D	a _{max}	(τ _{max}) _r	CSR _{Mw}	MSF	CSR _{Mw=7.5}	CRR _{Mw=7.5}	FS	Liq	N _{1,60cs}	ε (%)	Settlement (in)		
12.25	SC	3.5	15	18	0.75	1.20	1.00	1.00	16	120	1470	1018	1.44	23	--	0.97	0.6	859	0.549	1.15	0.477	0.328	0.687	yes	27	1.1	0.46	
Total Settlement (in):																						0.46						
Boring 2																												
Youd, 2001												Tokimatsu and Seed (1987)																
Depth at Center of Layer (ft)	Unified Soil Class.	Layer Thickness (ft)	% Fines	N	C _R	C _S	C _B	C _E	N ₆₀	γ _m	σ _v	σ _v '	C _N	N _{1,60cs}	R _D	a _{max}	(τ _{max}) _r	CSR _{Mw}	MSF	CSR _{Mw=7.5}	CRR _{Mw=7.5}	FS	Liq	N _{1,60cs}	ε (%)	Settlement (in)		
8.50	SM	2	25	15	0.75	0.63	1.00	1.00	7	120	1020	802	1.62	11	--	0.98	0.6	601	0.488	1.15	0.424	0.176	0.415	yes	17	1.8	0.43	
10.13	SM	1.25	25	25	0.75	1.20	1.00	1.00	23	120	1216	895	1.54	34	42	0.98	0.6	714	0.518	1.15	0.451	--	--	no	42	--	--	
13.13	SC	4.75	37	15	0.85	1.20	1.00	1.00	15	120	1576	1068	1.41	21	--	30	0.97	0.6	919	0.559	1.15	0.486	--	no	30	--	--	
45.38	SC	1.75	25	32	1.00	0.63	1.00	1.00	20	120	5446	2928	0.85	17	--	0.80	0.6	2613	0.580	1.15	0.505	0.261	0.517	yes	23	1.4	0.28	
Total Settlement (in):																						0.7						
Boring 3																												
Youd, 2001												Tokimatsu and Seed (1987)																
Depth at Center of Layer (ft)	Unified Soil Class.	Layer Thickness (ft)	% Fines	N	C _R	C _S	C _B	C _E	N ₆₀	γ _m	σ _v	σ _v '	C _N	N _{1,60cs}	R _D	a _{max}	(τ _{max}) _r	CSR _{Mw}	MSF	CSR _{Mw=7.5}	CRR _{Mw=7.5}	FS	Liq	N _{1,60cs}	ε (%)	Settlement (in)		
9.75	ML	2.5	73	2	0.75	1.20	1.00	1.00	2	120	1170	874	1.56	2	--	7	0.98	0.6	688	0.512	1.15	0.445	0.091	0.204	yes	7	3.3	0.99
16.63	SM	1.25	27	16	0.85	0.63	1.00	1.00	9	120	1996	1270	1.29	11	--	17	--	0.96	0.6	1155	0.591	1.15	0.350	yes	17	1.8	0.27	
Total Settlement (in):																						1.3						

TABLE 1 - LIQUEFACTION ANALYSIS
 BERKELEY ANIMAL SHELTER
 1 Bolivar Drive, Berkeley, California
 Alan Kropp and Associates; Project #1258-8
 December 2008

APPENDIX A

Boring Logs

SOIL CLASSIFICATION CHART

PRIMARY DIVISIONS			SECONDARY DIVISIONS		
			CRITERIA *	GROUP SYMBOL	GROUP NAME
COARSE-GRAINED SOILS MORE THAN 50% RETAINED ON NO.200 SIEVE	GRAVELS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO.4 SIEVE	CLEAN GRAVELS LESS THAN 5% FINES	$C_u \geq 4$ AND $1 \leq C_c \leq 3^A$	GW	Well-graded gravel
			$C_u < 4$ AND/OR $1 > C_c > 3$	GP	Poorly-graded gravel
		GRAVELS WITH FINES - MORE THAN 12% FINES	FINES CLASSIFY AS ML OR MH	GM	Silty gravel
			FINES CLASSIFY AS CL OR CH	GC	Clayey gravel
	SANDS 50% OR MORE OF COARSE FRACTION PASSES NO. 4 SIEVE	CLEAN SANDS LESS THAN 5% FINES	$C_u \geq 6$ AND $1 \leq C_c \leq 3$	SW	Well-graded sand
			$C_u < 6$ AND/OR $1 > C_c > 3$	SP	Poorly-graded sand
		SANDS WITH FINES - MORE THAN 12% FINES	FINES CLASSIFY AS ML OR MH	SM	Silty sand
			FINES CLASSIFY AS CL OR CH	SC	Clayey sand
FINE-GRAINED SOILS 50% OR MORE PASSES THE NO.200 SIEVE	SILTS AND CLAYS LIQUID LIMIT LESS THAN 50%	INORGANIC	$PI > 7$ AND PLOTS ON OR ABOVE "A" LINE	CL	Lean clay
			$PI < 4$ OR PLOTS BELOW "A" LINE	ML	Silt
		ORGANIC	$\frac{\text{LIQUID LIMIT - OVEN DRIED}}{\text{LIQUID LIMIT - NOT DRIED}} < 0.75$	OL	Organic Clay & Organic Silt
	SILTS AND CLAYS LIQUID LIMIT 50% OR MORE	INORGANIC	PI PLOTS ON OR ABOVE "A" LINE	CH	Fat clay
			PI PLOTS BELOW "A" LINE	MH	Elastic silt
		ORGANIC	$\frac{\text{LIQUID LIMIT - OVEN DRIED}}{\text{LIQUID LIMIT - NOT DRIED}} < 0.75$	OH	Organic Clay & Organic Silt
	HIGHLY ORGANIC SOILS		PRIMARILY ORGANIC MATTER, DARK IN COLOR, AND ORGANIC ODOR	PT	Peat

REFERENCE: Unified Soil Classification System (ASTM D 2487-06)

* Criteria may be done on visual basis, not necessarily based on lab testing

$$A - C_u = D_{60}/D_{100} \quad \& \quad C_c = (D_{30})^2 / (D_{10} \times D_{60})$$

GRAIN SIZES

U. S. STANDARD SERIES SIEVE								CLEAR SQUARE SIEVE OPENINGS	
	200	40	10	4		3/4"	3"	12"	
SILTS AND CLAYS	SAND			GRAVEL		COBBLES	BOULDERS		
	FINE	MEDIUM	COARSE	FINE	COARSE				

ABBREVIATIONS

INDEX TESTS

- LL - Liquid Limit (%) (ASTM D 4318-05)
 PI - Plasticity Index (%) (ASTM D 4318-05)
 -200 - Passing No. 200 Sieve (%) (ASTM D 1140-00)

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 2166-06)
 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



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KEY TO EXPLORATORY BORING LOGS

BERKELEY ANIMAL SHELTER
1 Bolivar Drive, Berkeley, California

PROJECT NO.

1258-8

DATE

December 2008

FIGURE **A-1**

DRILL RIG: Portable, Solid Flight Auger	SURFACE ELEVATION: 5.9 feet (see notes)	LOGGED BY: AL
DEPTH TO GROUNDWATER: 12 feet (see notes)	BORING DIAMETER: 3.5 inches	DATE DRILLED: 9/13/08

DESCRIPTION AND REMARKS	COLOR	CONSISTENCY	SOIL TYPE	DEPTH (ft)	SAMPLER TYPE	SAMPLER BLOW COUNTS	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	OTHER TESTS
4" AC over 4" of Base Material									
GRAVEL, Clayey - with sand (FILL)	Reddish Brown to Greenish Gray	Medium Dense	GC	1	X	[14]	17.2	111	PP = 3.75 tsf LL = 32 PI = 21 -200 = 46%
				2					
CLAY, Lean to Fat, Sandy - with silt, gravels, with shells, pockets of fine sand (Possible Fill)	Bluish Gray	Firm	CL/CH	3	X	[9]			PP = 1.25 tsf PP = 0.5 tsf
				4					
CLAY, Fat - with fine sand, trace gravel, with shells (Possible Fill)	Bluish Gray & Reddish Brown	Soft to Firm	CH	5		4			
				6					
				7					
FILL ↑ CLAY, Fat - with silt, wet, hydrocarbon odor (Bay Deposits)	Black	Soft	CH	8					
				9					
				10					
SAND, Clayey - with fine gravel, coarse sand	Grayish Brown	Medium Dense	SC	11		18			-200 = 15%
				12					
				13					
CLAY, Lean - some silt, trace sand	Brown to Reddish Brown	Stiff	CL	14		11			
				15					
				16					
				17					
				18					
CLAY, Lean to Fat - with silt, some gravel, trace sand	Mottled Brown, Reddish Brown, Black	Stiff	CL/CH	19		[23]			PP = 1.25 tsf PP = 2.25 tsf
				20					
CLAY, Fat - with silt, trace sand	Light Brown	Stiff to Very Stiff	CH	21	X				
				22					
				23					
				24					

(Continued on Next Page)



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EXPLORATORY BORING LOG
BERKELEY ANIMAL SHELTER
1 Bolivar Drive; Berkeley, California

PROJECT NO.	DATE	SHEET	BORING NO.
1258-8	December 2008	1 of 2	1

DESCRIPTION AND REMARKS	COLOR	CONSISTENCY	SOIL TYPE	DEPTH (ft)	SAMPLER TYPE	SAMPLER BLOW COUNTS	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	OTHER TESTS
(Continued from Previous Page) CLAY, Fat - with silt, trace sand	Reddish Brown	Very Stiff	CH	26	X	[21]			PP = 2.5 tsf

Bottom of boring at 26.2 feet.

NOTES:

1. Groundwater was encountered at approximately 15 feet at the time of drilling and measured at 12 feet approximately 30 minutes after drilling was complete. The boring was backfilled immediately 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 Land Title Survey by Moran Engineering, Inc., dated August 2008.
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".



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EXPLORATORY BORING LOG

BERKELEY ANIMAL SHELTER
1 Bolivar Drive; Berkeley, California

PROJECT NO.	DATE	SHEET	BORING NO.
1258-8	December 2008	2 of 2	1

DRILL RIG: SIMCO4000, Hollow Stem Auger	SURFACE ELEVATION: 5.0 feet (see notes)	LOGGED BY: DM
DEPTH TO GROUNDWATER: 10 feet (see notes)	BORING DIAMETER: 6 inches	DATE DRILLED: 9/13/08

DESCRIPTION AND REMARKS	COLOR	CONSISTENCY	SOIL TYPE	DEPTH (ft)	SAMPLER TYPE	SAMPLER BLOW COUNTS	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	OTHER TESTS
4" AC over 6" of Gravelly Base Material									
CLAY, Lean, Sandy - with gravel (gravels up to 1.5" (FILL))	Reddish Brown & Gray	Firm	CL	1					
-hydrocarbon odor				2					
				3					
				4					
-pockets of CH below 4.5'	Bluish Gray & Reddish Brown	Firm to Stiff	CH	5	X				PP = 1.0 tsf
				6	X	[21]			
				7					
SAND, Silty - fine to coarse sand, with gravel, hydrocarbon odor (FILL)	Dark Gray	Medium Dense	SM	8	X				
				9	X				
SAND, Silty - some gravel, (Bay Deposits)	Black	Loose	SM	10		[15]	17.8		LL = 28 PI = 14 -200 = 25%
SAND, Silty - fine sand, some clay, trace coarse sand, with fine gravel	Bluish Gray	Medium Dense	SM	11		25			
SAND, Clayey - fine to medium sand, trace coarse sand	Reddish Brown	Medium Dense	SC	12					
				13					
				14					
				15			21		LL = 33 PI = 21 -200 = 37%
CLAY, Lean, Sandy - some fine gravel	Reddish Brown	Stiff	CL	16		15			
				17					
				18					
CLAY, Fat - with silt, decrease sand and gravel	Reddish Brown	Firm	CH	19					
				20	X				
				21	X	[10]			PP = 0.5 - 0.7 tsf
				22					
				23					
				24					

(Continued on Next Page)



**ALAN KROPP
& ASSOCIATES**
*Geotechnical
Consultants*

EXPLORATORY BORING LOG

BERKELEY ANIMAL SHELTER
1 Bolivar Drive; Berkeley, California

PROJECT NO.
1258-8

DATE
December 2008

SHEET
1 of 3

BORING NO.

2

DESCRIPTION AND REMARKS	COLOR	CONSISTENCY	SOIL TYPE	DEPTH (ft)	SAMPLER TYPE	SAMPLER BLOW COUNTS	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	OTHER TESTS
<i>(Continued from Previous Page)</i>									
CLAY, Fat - with silt, decrease sand and gravel	Reddish Brown	Firm to Stiff	CH	26	X	[14]			
				27					
				28					
				29					
CLAY, Lean, Sandy - with fine gravels (1/4" - 1/2")	Reddish Brown	Very Stiff	CL	30	X	[37]			
				31	X				
				32					
				33					
		Hard		34					
				35	X	[94/9"]			
				36	X				
				37					
				38					
				39					
-decrease sand and gravel, pockets of CH, increase plasticity	Light Brown	Very Stiff	CL/CH	40	X	[52]			
				41	X				
				42					
				43					
				44					
SAND, Clayey - medium to coarse sand with gravel	Reddish Brown	Medium Dense	SC	45	X	[32]			
				46	X				
CLAY, Lean - with silt, medium plasticity	Light Brown	Very Stiff	CL	47					
				48					
CLAY, Fat - with silt	Brown with Black Spots	Very Stiff	CH	49	X	[26]			
				50	X				

Bottom of boring at 50.5 feet.

NOTES:

- Groundwater was encountered at approximately 10 feet at the time of drilling and the boring was backfilled immediately after

(Continued on Next Page)

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Consultants*

EXPLORATORY BORING LOG

BERKELEY ANIMAL SHELTER
1 Bolivar Drive; Berkeley, California

PROJECT NO.

1258-8

DATE

December 2008

SHEET

2 of 3

BORING NO.

2

DESCRIPTION AND REMARKS	COLOR	CONSISTENCY	SOIL TYPE	DEPTH (ft)	SAMPLER TYPE	SAMPLER BLOW COUNTS	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	OTHER TESTS																		
<i>(Continued from Previous Page)</i>																											
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 Land Title Survey by Moran Engineering, Inc., dated August 2008. 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".																											
<table border="1"> <tr> <td colspan="2">  ALAN KROPP & ASSOCIATES <i>Geotechnical Consultants</i> </td> <td colspan="4"> EXPLORATORY BORING LOG BERKELEY ANIMAL SHELTER 1 Bolivar Drive; Berkeley, California </td> </tr> <tr> <td>PROJECT NO.</td> <td>DATE</td> <td>SHEET</td> <td colspan="3">BORING NO.</td> </tr> <tr> <td>1258-8</td> <td>December 2008</td> <td>3 of 3</td> <td colspan="3">2</td> </tr> </table>										 ALAN KROPP & ASSOCIATES <i>Geotechnical Consultants</i>		EXPLORATORY BORING LOG BERKELEY ANIMAL SHELTER 1 Bolivar Drive; Berkeley, California				PROJECT NO.	DATE	SHEET	BORING NO.			1258-8	December 2008	3 of 3	2		
 ALAN KROPP & ASSOCIATES <i>Geotechnical Consultants</i>		EXPLORATORY BORING LOG BERKELEY ANIMAL SHELTER 1 Bolivar Drive; Berkeley, California																									
PROJECT NO.	DATE	SHEET	BORING NO.																								
1258-8	December 2008	3 of 3	2																								

DRILL RIG: SIMCO4000, Solid Stem Auger	SURFACE ELEVATION: 4.9 feet (see notes)	LOGGED BY: DM
DEPTH TO GROUNDWATER: 13 feet (see notes)	BORING DIAMETER: 3.5 inches	DATE DRILLED: 9/13/08

DESCRIPTION AND REMARKS	COLOR	CONSISTENCY	SOIL TYPE	DEPTH (ft)	SAMPLER TYPE	SAMPLER BLOW COUNTS	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	OTHER TESTS	
3" CONCRETE										
CLAY, Lean, Gravelly - with sand (FILL)	Brown & Gray	Stiff	CL	1		[15]	38.1		- No liners in sampler	
				2						
CLAY, Lean to Fat - large gravel (2"), increase plasticity, wood pieces at 4' (FILL)	Dark Brown & Gray	Stiff	CL/CH	3		[19]				
				4						
				5						
CLAY, Fat - with shells, large angular gravel (2"), silty (FILL)	Dark Gray to Black	Firm	CH	6		[9]				
				7						
				8						
SILT, Lean, Sandy - with shells, hydrocarbon odor (Bay Deposits)	Black	Very Soft	ML	9		2				
				10						
CLAY, Lean to Fat - medium plasticity	Grayish Brown	Stiff	CL/CH	11		12				
				12						
				13						
				14						
				15						
SAND, Silty - fine to medium sand, trace rounded gravel (1/4" - 1/2")	Dark Gray	Medium Dense	SM	16		[16]				-200 = 27%
CLAY, Lean - with silt, trace coarse sand	Reddish Brown	Stiff	CL	17						
				18						
				19						
CLAY, Fat - with silt	Grayish Brown	Very Stiff to Hard	CH	20		[53]				
				21						
				22						

Bottom of boring at 22.5 feet.

NOTES:

(Continued on Next Page)



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EXPLORATORY BORING LOG

BERKELEY ANIMAL SHELTER
1 Bolivar Drive, Berkeley, California

PROJECT NO.

1258-8

DATE

December 2008

SHEET

1 of 2

BORING NO.

3

DESCRIPTION AND REMARKS	COLOR	CONSISTENCY	SOIL TYPE	DEPTH (ft)	SAMPLER TYPE	SAMPLER BLOW COUNTS	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	OTHER TESTS
<i>(Continued from Previous Page)</i>									
- Groundwater was encountered at approximately 13 feet at the time of drilling and the boring was backfilled immediately after drilling. (See report for discussion.) - Stratification lines represent the approximate boundaries between material types and the transitions may be gradual. - 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. - Elevations were determined from Land Title Survey by Moran Engineering, Inc., dated August 2008. - 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".									

 ALAN KROPP & ASSOCIATES <i>Geotechnical Consultants</i>	EXPLORATORY BORING LOG			
	BERKELEY ANIMAL SHELTER 1 Bolivar Drive; Berkeley, California			
	PROJECT NO. 1258-8	DATE December 2008	SHEET 2 of 2	BORING NO. 3

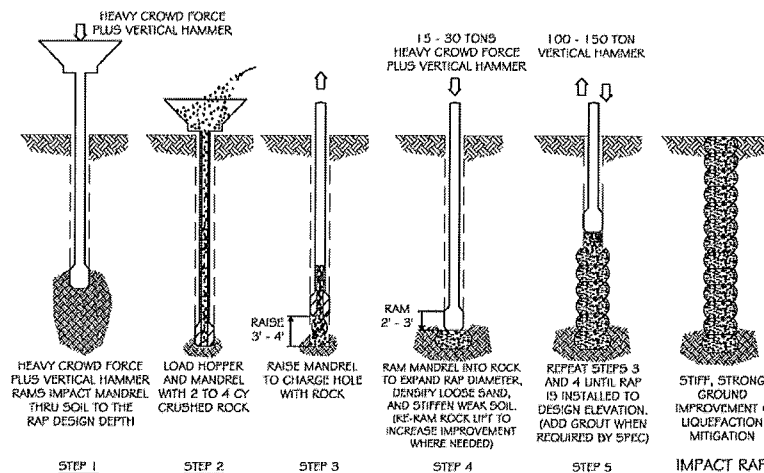
APPENDIX B

Typical Impact[®] RAP Description

TYPICAL IMPACT® RAP (Impact Pier) DESCRIPTION - DESIGN AND CONSTRUCTION

Impact Pier design is based on increased stiffness and shear reinforcement of loose and soft soil zones to depths of 45 feet plus. After the increased stiffness of the improved soil zone is estimated, a two-layer settlement analysis is performed to control total and differential settlements to project requirements. Settlements within the “upper zone” (zone of soil that is reinforced with Impact Piers) are computed using a weighted stiffness modulus method that accounts for the stiffness of the Impact Piers, the stiffness of the native soil, and the area coverage of the Impact Piers below supported mats and footings. Settlements within the “lower zone” (subsurface soils below the treated zone) are computed using conventional geotechnical settlement methods. The reduction of liquefaction using Impact Piers is estimated by shear reinforcement, densification of clean sands, and drainage where applicable. Once all of these estimates are made, a complete RAP design layout with calculations is prepared for the design team’s review and eventual construction document submittal to the approving jurisdiction.

Impact Piers are installed by first driving a specially designed 14 to 18 inch diameter rammer foot and hollow pipe mandrel into the ground using a 50 to 75 ton piling rig augmented by dynamic vertical impact energy with a 100 to 150 ton vibratory piling hammer. The Impact Piers typically extend between 20 and 40 feet below the ground surface, but may extend deeper depending on project requirements. After driving the Impact mandrel to the design depth, the hollow mandrel is loaded with crushed rock and serves as a conduit for the placement of crushed rock to the bottom of the pier. After rock is loaded, the mandrel is lifted approximately 3 to 4 feet loading the displaced shaft with rock and then the mandrel is driven and rammed back down into the rock lift 2 to 3 feet, forming a 1 foot thick compacted rock lift (see figure). Driving compaction is achieved through heavy equipment crowd force and dynamic impact energy from the vibratory hammer. The vibratory piling hammer densifies rock vertically and the beveled rammer foot forces rock laterally into the soil. Subsequent rock lifts are performed using the same approach of raising and driving the mandrel and rammer foot until the ground surface is met. Depending on the drive stroke length, RAP diameters range can from 20 to 30 inches, with typical 3/2 stroke RAPs averaging 24 inches in diameter.



The ramming of the rock causes pre-stressing and pre-straining of the native soil and improves the density of the sandy soils. The ramming both densifies the rock and surrounding soil and forces the rock laterally into the soil. This action increases the density and the lateral stress in surrounding soil; resulting in a higher stiffness of the composite RAP/soil mass. The result of Impact Pier installation is a significant strengthening and stiffening of subsurface soils that then support high bearing capacity mats and footings.

APPENDIX C

Impact® RAP Feasibility Letter

FEASIBILITY TRANSMITTAL

Page 1 of 2

DATE: December 11, 2008

To: Dona Mann **Fax:** (510) 841-8357
Alan Kropp & Associates **Ph** (510) 841-5095
2140 Shattuck Ave., #910
Berkeley, CA 94704

☐ Urgent ☒ As Requested ☐ For Your Information ☐ Please Comment & Reply ☐ Please Respond

From: Kyle Wallace **Pgs Sent:** 2
Job: Berkeley Animal Shelter **Job No.:** _____
RE: Feasibility of Impact® RAP Soil Reinforcement **Copy to follow in mail:** ☐ yes ☒ no

Dear Ms. Mann,

Thank you for considering Impact® Rammed Aggregate Pier soil reinforcement for use at the proposed Berkeley Animal Shelter in Berkeley, CA. Based on the soil data that you provided, Impact Rammed Aggregate Pier (RAP) soil reinforcement would provide reliable long-term foundation support for the planned construction. Keep in mind that we need to be involved early in the design process for the full benefits of the RAP design-build system to be realized.

BACKGROUND

We understand the proposed project consists of a new 3-story building with office space and an animal shelter facility. The building will be supported on a mat foundation. Based on the preliminary schematic plan provided the estimated square footage of the building is on the order of 8,500 square feet. Due to the preliminary nature of the project no foundation loads are available at this time.

We understand that Impact RAP soil reinforcement is being considered for this project due to the concerns of significant settlement, liquefaction, lateral spreading, and contaminated soil. Impact RAP soil reinforcement displaces and densifies the in situ soils therefore increasing the strength and stiffness of the weak soils.

ALLOWABLE RAP CAPACITIES

Bearing

For preliminary design purposes using the soil data you provided, we estimate an allowable vertical capacity for a grouted RAP to be about 50 kips for one 20-inch diameter RAP element. The RAP bearing elements would need to extend a minimum of 25 feet below the structural mat to penetrate through the fill/bay deposits/potential liquefiable materials to penetrate into the stiff to very stiff clay. The Impact RAP shaft will be grouted in the bay deposit zone for additional confinement strength.

A 1/3 increase to all allowable capacities is permitted for short term seismic and wind loads. The bearing capacity would be verified by full scale modulus load test.

Estimated Footing Settlement

Based on the following soil and RAP properties, the long-term total and differential settlements of the footings shown above would be less than 1-inch and 1/2-inch, respectively. Farrell can design RAP elements for lower settlement values based on specific performance requirements.

Feasibility Transmittal

DATE: December 11, 2008

Page 2 of 2

Geotechnical Soil Information

We need final boring logs, and any other geotechnical soil testing information currently available for the site. Typical information that allows Farrell to provide the most efficient design includes:

- standard penetration test (SPT) data to depths of 50 feet,
- moisture density data,
- shear strength data,
- consolidation data (where applicable).

We need a copy of the Geotechnical Report for final design. If cone penetration testing is conducted on the site, tabulated CPT data should be included in the geotechnical information provided.

CLOSING

Based on the soil conditions, RAP soil reinforcement is a good application for the support of the mat foundation for the Berkeley Animal Shelter project.

Please call us at (530) 621-4867 if you have any questions or need further information.

Best Regards,

FARRELL DESIGN-BUILD COMPANIES, INC.

Kyle R. Wallace, PE
Project Engineer