REVISED GEOTECHNICAL INVESTIGATION REPORT
OAKLEY RECREATION IMPROVEMENTS
OAKLEY, CALIFORNIA

BSK PROJECT NO.: G16-221-11L

PREPARED FOR:

GATES+ASSOCIATES
2671 CROW CANYON ROAD
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March 1, 2017

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March 1, 2017 **BSK JOB No. G16-221-11L**

Gates+Associates
2671 Crow Canyon Road
San Ramon, California 94583

ATTENTION: Ms. Kimmy Chen, (kimmy@dgates.com)

SUBJECT: **Revised Geotechnical Investigation Report**
Oakley Recreation Center
Oakley, California

Dear Ms. Chen:

BSK Associates (BSK) is pleased to submit our revised geotechnical investigation report for the above-referenced project. This revised report was originally issued on December 19th, 2016 for Phase 1 of the project and has been revised to incorporate our February 2, 2017 supplemental geotechnical investigation for the planned Phase 1 and 2 buildings and associated improvements. The enclosed report describes the geotechnical investigations performed and presents our geotechnical recommendations for foundations, earthwork, a retaining wall, and pavements for the project.

In summary, it is our opinion that the site is suitable for the proposed project, and the related improvements are feasible geotechnically, provided the recommendations presented in this report are incorporated in the design and construction of the project. The primary geotechnical concerns at this site are:

1. The potential for strong ground shaking to affect the site during a future significant seismic event (typical of the entire San Francisco Bay Area);
2. The potential for near-surface loose, unsaturated sand layers to undergo dynamic compaction (also known as seismic settlement) during a design-level earthquake; and
3. The potential for loose to medium dense, saturated (i.e., submerged) sand layers to experience liquefaction-induced settlement during a design-level earthquake.

Ground shaking can be addressed by incorporating the seismic design parameters presented herein and other seismically related aspects of the 2016 California Building Code into the design of new structures. If the structural engineer determines that the planned buildings cannot accommodate the estimated magnitude of dynamic compaction and liquefaction-induced settlements during a future design-level earthquake using conventional spread footings, the new buildings could instead be supported on reinforced mat foundations, post-tensioned (PT) slabs, or footing and grade beam “waffle” type foundations.

Other geotechnical concerns that will affect the project are the presence of poorly graded sand near the site surface and the presence of stockpiles of undocumented soil at the site. The former concern will likely require casing or use of the slurry displacement method during installation of CIDH piers and shoring/sloping/benching of excavations due to the potential for the site soils to cave. With respect to the soil stockpiles, they will need to be evaluated to check if they may be re-used on-site as general fill or need to be off-hauled from the site. These concerns and other geotechnically relevant aspects of the project are discussed further in detail in the “Conclusions and Discussions” and “Recommendations” sections of this report.

The conclusions and recommendations presented in the enclosed report are based on limited subsurface investigation and laboratory testing programs. Consequently, variations between anticipated and actual subsurface soil conditions may be found in localized areas during construction. If significant variation in the subsurface conditions is encountered during construction, BSK should review the recommendations presented herein and provide supplemental recommendations, if necessary.

Additionally, design plans should be reviewed by our office prior to their issuance for conformance with the general intent of our recommendations presented in the enclosed report.

We appreciate the opportunity of providing our services to you on this project and trust this report meets your needs at this time. If you have any questions concerning the information presented, please contact us at (925) 315-3151.

Sincerely,

BSK Associates, Inc.

Margaret McNally

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Plates and Appendices

1. INTRODUCTION

This report presents the results of BSK Associates' (BSK) geotechnical investigation for the planned Oakley Recreation Center improvements in Oakley, California. A Vicinity Map showing the location of the project site is presented on Plate 1. This report contains a description of our site investigation methods and findings, including field and laboratory data. It also provides earthwork and foundation recommendations and other construction considerations. This revised report was originally issued on December 19, 2016 for Phase 1 of the project and has been revised to incorporate our February 2, 2017 supplemental geotechnical investigation for the planned Phase 1 and 2 buildings and associated improvements.

1.1 Project Description

The improvements for the planned Oakley Recreation Center are being performed in two phases (refer to the preliminary layout plans presented in Appendix A). As shown on the Site Plan, Plate 1, we understand that an approximately 10,000 square foot, single story building will be constructed during Phase 1 of the project and that construction of that building will require relocation of an existing approximately 3,000 square foot, single story building. In addition, an approximately 10,000 square foot, single story building and a 10- to 12-foot tall trellis will be constructed immediately to the east of the new building for Phase 1. Also as part of Phase 2, an amphitheater will be constructed between the two new buildings, which will require construction of an approximately 5-foot tall retaining wall in this area. We anticipate that the new buildings will be lightly to moderately loaded, will have slab-on-grade floors, and will have no basements.

Other improvements to be constructed as part of Phase 1 will include an athletic field with natural turf within the approximately eastern half of the site, site grading, new asphalt concrete paved parking and driveways, concrete flatwork, bleachers and a baseball backstop for the athletic field, fences, underground utilities, landscaping, bioswales/infiltration basins, and a new traffic signal at O'Hara Avenue near the entrance to the site. Portions of the proposed location of the Phase 1 building are currently covered by pavement. Therefore, the project will require demolition of existing improvements prior to grading.

Although grading plans for the project are not currently available, we anticipate on the order of 5+ feet of cut and fill to reach design finish grades, grade the site to drain, and remove the existing soil stockpiled in the unimproved areas of the site. We expect excavations for new utility lines will be up to 5 feet deep.

If the actual project description differs significantly from that anticipated above, especially the amount of grading, we should be notified so that we may review our recommendations presented herein for applicability.

1.2 Purpose and Scope of Services

The purpose of this investigation was to explore and evaluate the subsurface conditions at the site in order to provide geotechnical input for the design and construction of the planned improvements and the associated earthwork for the project. The scope of services consisted of two field investigations, laboratory testing, engineering analysis, and preparation of this revised report as well as our original report dated December 19, 2016. Our scope of services did not include the evaluation of contaminants in the soil, water, or air.

2. SITE INVESTIGATION

2.1 Field Investigation

Two field investigations occurred for this project; the first was performed on November 17, 2016 and the second (supplemental investigation) on February 2, 2017. The first investigation was tailored to only address Phase 1 of the project, excluding the new building. The second investigation included three supplemental borings to evaluate the planned buildings for Phases 1 and 2 and related improvements.

Soil classifications made in the field from auger cuttings and samples were re-evaluated in the laboratory after further examination and testing. The soils were classified in the field in general accordance with the Unified Soil Classification System (Visual/Manual Procedure - ASTM D2488). Where laboratory tests were performed, the designations reflect the laboratory test results in general accordance with ASTM D2487 as presented on Plate B-1. A Soil Description Key is presented on Plate B-2 and a key to the symbols used in the boring logs is presented on the Log Key, Plate B-3. Sample classifications, blow counts recorded during sampling on February 2, 2017, and other related information were recorded on the soil boring logs. Logs of borings B-1 through B-6 are presented in Appendix B. A discussion of the subsurface conditions encountered at the site is presented in the "Subsurface Conditions" section of this report

The locations of the borings were estimated by a BSK staff engineer based on rough measurements from existing features at the site. Elevations shown on the boring logs were estimated using the elevation information available on Google Earth Pro. As such, the elevations and locations of the borings should be considered approximate to the degree implied by the methods used.

2.1.1 First Field Investigation

Our first field investigation was performed on November 17, 2016 and consisted of drilling three (3) soil borings (labeled B-1 through B-3) at the approximate locations shown on the Site Plan, Plate 2. The borings extended to depths of about 6 to 8½ feet below the ground surface (BGS) using a 4-inch diameter hand auger to drill into the underlying soils. The boreholes were advanced and logged by a BSK staff engineer, who selected the locations, depths, and sampling intervals of the boreholes. After completion, the boreholes were filled with soil cuttings.

Relatively undisturbed samples of the subsurface soils were taken by our staff engineer during drilling using a hand-driven sampler equipped with 2-inch diameter stainless steel liners. Disturbed bulk samples were also collected from the auger cuttings at each boring. After the sampler was withdrawn from the boreholes, the sample liners were removed, sealed to reduce moisture loss, and labeled. After completion of our field investigation, the samples were brought to BSK's Livermore laboratory for storage and testing.

2.1.2 Second Field Investigation

Prior to the start of our second field investigation, we contacted Underground Service Alert (USA) to provide utility clearance and we retained the services of Geotech Utility Locating of Moraga, California to help locate detectable underground utility lines near our boring locations. In addition, we obtained a drilling permit from the Contra Costa Environmental Health Division (County).

Exploration GeoServices of San Jose, California was subcontracted to provide drilling services during our second field investigation using a truck-mounted drill rig equipped with an automatic hammer and 8-inch hollow-stem augers. This work was performed on February 2, 2017 and consisted of drilling three (3) borings (labeled B-4 through B-6) at the approximate locations shown on the Site Plan, Plate 2. A BSK staff engineer selected the locations, depths, sampling intervals, and observed the drilling operation. Two borings were drilled to a depth of approximately 20 feet BGS and one boring was drilled to a depth of approximately 50 feet BGS. Immediately after free groundwater was observed at the deeper boring, the augers for this boring were immediately flooded with water to stabilize the hydraulic pressure at the bottom of the boreholes and reduce the risk of heaving sand adversely affecting the blow counts.

Upon completion of our field investigation, the borings were backfilled with cement grout per County requirements. In addition, the upper approximately 6 inches of the borings located in paved areas of the site where patched with Quikrete. Excess cuttings generated during drilling were disposed of in soil stockpiles near the site.

Relatively undisturbed samples of the subsurface soils were obtained using a split spoon sampler with a 2.5-inch inside diameter (I.D.) and a 3-inch outside diameter (O.D.) fitted with stainless steel liners. The samplers were driven 18 inches using a 140-pound, automatic trip hammer falling 30 inches, and blow counts for successive 6-inch penetration intervals were recorded and reported on the final boring logs. After the sampler was withdrawn from the borehole, the samples were removed, sealed to reduce moisture loss, labeled, and returned to our laboratory. Prior to sealing the samples, strength characteristics of the cohesive soil samples recovered were evaluated using a hand-held pocket penetrometer. The results of these tests are shown adjacent to the samples on the boring logs. After completion of our field investigation, the samples were brought to BSK's Livermore laboratory for storage and testing.

2.2 Laboratory Testing

Laboratory tests were performed on selected soil samples to evaluate their physical characteristics and engineering properties. The laboratory testing program included dry density and moisture content, sieve analysis (percent passing the No. 200 sieve), and Resistance (R)-Value testing. Most of the laboratory test results are presented on the individual boring logs in Appendix B. The R-Value test result is presented graphically in Appendix C.

Analytical testing was performed as part of our investigation on a sample of the subsurface soils obtained from boring B-1 at depths of approximately 1 to 2 feet BGS to assist in evaluating the corrosion potential of the near-surface soils. The results of the corrosion testing performed by CERCO Analytical of Concord, California using ASTM methods, are presented in Appendix C.

3. SITE CONDITIONS

3.1 Site Description

The project site is located at 1250 O'Hara Avenue in Oakley, California. As shown on Plate 2, the approximately 6-acre site is bounded to the north by the O'Hara Park Middle School, to the east and south by the Delta de Anza Regional Trail and the Contra Costa Canal levee, and on the west by O'Hara Avenue. The western half of the site contains a paved parking lot and five temporary 1-story buildings housing a recreation facility. The eastern half of the site is currently undeveloped and is covered by sparse vegetation and stockpiles of undocumented soil. Historical aerial photographs indicate the site used to contain an orchard that was removed sometime between 1979 and 1993 and that the existing buildings were constructed between 2004 and 2005. Although the site appears to be relatively level, ground surface elevations vary from about 40 to 50 feet according to Google Earth Pro. The Contra Costa Canal levee is about 10 feet higher in elevation than the project site.

3.2 Geologic Setting

According to geology mapping by Dibblee and Minch (2006)[1], the site is underlain by Holocene age (less than 11,000 years old) wind-blown sand dunes and by levee fill along the southern margins of the site, where the Contra Costa Canal is located.

3.3 Subsurface Conditions

According to the borings drilled for our investigations, the paved areas of the site are covered by approximately 3 inches of asphalt concrete (AC) over the native sandy soils. The soils in the upper approximately 12 to 15 feet of the site generally consist of loose to medium dense poorly graded to silty sand containing varying amounts of gravel. This material has a low expansion potential. Below this depth, our borings encountered interbedded layers of firm to hard clay and medium dense poorly graded to silty sand to the maximum depth of our exploration (about 50 feet BGS). Boring B-3 encountered refusal at a depth of 6 feet BGS, possibly attributed to a large gravel particle. The soils encountered in our borings are consistent with the mapped geology for the site, which is discussed in the "Geologic Setting" section above.

Free groundwater was encountered at a depth of approximately 28½ feet BGS in boring labeled B-5. This groundwater depth is consistent with groundwater level data[2] for wells located within about 1 mile of the site, which indicate groundwater to be deeper than 20 feet BGS. It should be noted that groundwater levels can fluctuate several feet depending on factors such as seasonal rainfall, groundwater withdrawal, and construction activities on this or adjacent properties.

[1] Dibblee, T.W., and Minch, J.A. (2006), Geologic Map of the Antioch South & Brentwood Quadrangles, Contra Costa County, California: Dibblee Geological Foundation, Dibblee Foundation Map DF-193, scale 1:24,000.
[2] Data obtained from http://geotracker.waterboard.ca.gov and http://www.water.ca.gov/waterdatalibrary/.

The above is a general description of soil and groundwater conditions encountered at the site. For a more detailed description of the soils encountered, refer to the boring logs in Appendix B.

It should be noted that soil and subsurface conditions can deviate from those conditions encountered at the boring locations. If significant variation in the subsurface conditions is encountered during construction, it may be necessary for BSK to review the recommendations presented herein and recommend adjustments as necessary.

4. DISCUSSIONS AND CONCLUSIONS

Based on the results of our investigation, it is our opinion that the proposed project and related improvements are feasible geotechnically and that the site may be developed as presently planned. This conclusion is based on the assumption that the recommendations presented in this report will be incorporated into the design and construction of this project. The primary geotechnical concerns at this site are:

1. The potential for strong ground shaking to affect the site during a future significant seismic event (typical of the entire San Francisco Bay Area);

2. The potential for near-surface, loose, unsaturated sand layers to undergo dynamic compaction (also known as seismic settlement) during a design-level earthquake; and

3. The potential for loose to medium dense, saturated (i.e., submerged) sand layers to experience liquefaction-induced settlement during a design-level earthquake.

Additional discussions of the conclusions drawn from our investigation, including general recommendations, are presented below. Specific recommendations regarding geotechnical design and construction aspects for the project are presented in the "Recommendations" section of this report.

4.1 Geologic and Seismic Hazards

4.1.1 Faulting and Seismicity

The San Francisco Bay Area is seismically dominated by the active San Andreas Fault system. This fault system movement is distributed across a complex system of generally strike-slip, right-lateral parallel and sub-parallel faults including, among others, the San Andreas, San Gregorio, Hayward and Calaveras faults.

The site is not located within an Alquist-Priolo Earthquake Fault Zone, and no mapped active fault traces are known to transverse the site. Therefore, the likelihood of surface fault rupture to occur across the site is considered very low.

According to the USGS 2008 Seismic Hazard Maps[3], the site is located approximately 12 kilometers (km) from the Great Valley 5 fault, 16 km from the Greenville fault, 32 km from the Calaveras fault, 46 km from the Hayward fault, and 76 km from the San Andreas fault, all of which are considered to be active. Because the site is located in a seismically active area of California, we expect the site to be subjected to substantial ground shaking due to a major seismic event on the active faults in the Bay Area and

[3] https://geohazards.usgs.gov/cfusion/hazfaults_2008_search/query_main.cfm

surrounding regions during the design life of the project. According to a recent study[4], there is a 63 percent probability that one or more magnitude M6.7 or greater earthquakes will occur in the San Francisco Bay Area between 2007 and 2036.

As has been demonstrated recently by the 1989 (M6.9) Loma Prieta, the 1994 (M6.7) Northridge, and the 1995 (M6.9) Kobe earthquakes, earthquakes of this magnitude range can cause severe ground shaking and significant damage to modern urban environments. Therefore, the design of the new buildings and pertinent structures should incorporate the seismic design parameters presented in the "2016 CBC Seismic Design Parameters" section of this report.

4.1.2 Expansive Soils

The surficial soils encountered in our borings consisted of poorly graded to silty sand. These soils are considered to have a low expansion potential. Therefore, our recommendations do not include mitigation measures for addressing soil expansion, such as the use of "non-expansive" fill underneath the interior building slab or exterior concrete flatwork.

4.1.3 Liquefaction

Soil liquefaction is a condition where saturated, granular soils undergo a substantial loss of strength and deformation due to pore pressure increase resulting from cyclic stress application induced by earthquakes. In the process, the soil acquires mobility sufficient to permit both horizontal and vertical movements if the soil mass is not confined. Soils most susceptible to liquefaction are saturated, loose, clean, uniformly graded, and fine-grained sand deposits. If liquefaction occurs, foundations resting on or within the liquefiable layer may undergo settlements and/or a loss of bearing capacity.

Based on our findings, the site has a moderate to high susceptibility to experience liquefaction-induced settlements during a design-level earthquake. We performed liquefaction analyses on our deepest boring (B-5), which extended to a depth of about 50 feet BGS and total liquefaction-induced settlements are estimated to be up to about 3½ inches within the liquefiable layers. The potentially liquefiable layers at boring B-5 are about 5 feet thick and are located at depths of approximately 28½ and 36½ feet BGS. **Considering the inherent variability of the subsurface materials, and depth and thickness of the liquefiable layers, we judge that up to about two-thirds of the estimated settlement (or about 2½ inches) could potentially occur at the ground surface.** This settlement would be in addition to the static and dynamic compaction settlements discussed in this report.

Our liquefaction analyses were based on the methods by Youd et al (2001)[5], Seed et al. (2003)[6], and Idriss and Boulanger (2004)[7] using the following input parameters:

[4] Field, E.H., Miler, K.R., and the 2007 Working Group on California Earthquake Probabilities (2008), Forecasting California's Earthquakes – What Can We Expect in the Next 30 Years?: U.S. Geological Survey, Fact Sheet 2008-3027, 4 p. (http//:pubs.usgs.gov/fs/2008/3027/).

- A design groundwater depth of 28½ feet.
- A PGA_M of 0.5g (refer to the "2016 CBC Seismic Design Parameters" section later in this memorandum).
- An earthquake magnitude of M6.6 was used based on the USGS Interactive Deaggregation website (USGS, 2008)[8].

Liquefaction-induced differential settlement is expected to be up to about two-thirds of the total value discussed above and to occur over a horizontal distance of approximately 30 feet.

Based on Youd and Garris (1995)[9], we consider the potential for liquefaction-induced ground surface disruption (i.e., sand boils and ground fissures) to occur at the site to be low due to the relatively large thickness of the non-liquefiable layers overlying the liquefiable layers.

4.1.4 Lateral Spread

Lateral spread is a potential hazard commonly associated with liquefaction where extensional ground cracking and settlement occur as a response to lateral migration of subsurface liquefiable material. These phenomena typically occur adjacent to free faces such as slopes, creek channels, and levees. The eastern and southern perimeter of the site is bordered by the Contra Costa Canal levee. According to elevation profiles available in Google Earth, the canal channel is at or above the site-surface elevations. Because the canal channel bottom is shallower than the potentially liquefiable layers identified in boring B-5, we conclude that the potential for lateral spread to affect the project site is low.

4.1.5 Dynamic Compaction/Seismic Settlement

Another type of seismically induced ground failure, which can occur as a result of seismic shaking, is dynamic compaction, or seismic settlement. Such phenomena typically occur in unsaturated, loose

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[5] Youd, T.L., Idriss, I.M. Andrus, R.D. Arango, I., Castro, G., Christian, J.T., Dobry, R., Liam Finn, W.D.L., Harder, L.F., Jr., Hynes, M.E., Ishihara, K., Koester, J.P., Liao, S.S.C., Marcuson, W.F., III, Martin, G.R., Mitchell, J.K., Moriwaki, Y., Power, M.S., Robertson, P.K., Seed, R.B., Stokoe, K.H., II (2001), Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils, ASCE, Journal of Geotechnical and Geoenvironmental Engineering, V. 127, No. 10, p 817-833.

[6] Seed, R.B., K, O., Cetin, R.E.S., Moss, A., Kammerer, J., Wu, J.M., Pestana, M.F., Riemer, R.B., Sancio, J.D., Bray, R.E., Kayen, R.E., Faris, A. (2003), "Recent Advances in Soil Liquefaction Engineering: a unified and consistent framework," Keynote Address, 26th Annual Geotechnical Spring Seminar, Los Angeles Section of the GeoInstitute, American Society of Civil Engineers, H.M.S. Queen Mary, Long Beach, California, USA.

[7] Idriss, I.M. and Boulanger, R.W. (2004), "Semi-empirical procedures for evaluating liquefaction potential during earthquakes," in Proceedings, 11th International Conference on Soil Dynamics and Earthquake Geotechnical Engineering, and 3rd International Conference on Earthquake Geotechnical Engineering, D. Doolin et al., eds., Stallion Press, Vol. 1, pp. 32-56.

[8] USGS (2008), 2008 Interactive Deaggregations, http://geohazards.usgs.gov/deaggint/2008/

[9] Youd, T. L. and Garris, C. T. (1995), Liquefaction-Induced Ground-Surface Disruption, Journal of Geotechnical Engineering, ASCE, Vol. 121, No. 11, November, pp. 805-809.

granular material or uncompacted fill soils. Based on our analysis[10], the site could experience up to about ½-inch of dynamic compaction settlement of dry sand during a design-level earthquake. This settlement would be in addition to the static and liquefaction-induced settlements discussed in this report. Differential dynamic compaction settlement of dry sand is expected to be up to about half of the total value discussed above and to occur over a horizontal distance of approximately 30 feet.

4.2 Building Foundations

4.2.1 Foundation Type

The primary geotechnical consideration for foundation support for the new buildings and the relocated building is the potential for the sand layers underlying the site to experience dynamic compaction and liquefaction-induced settlements during a design-level earthquake. The buildings may be supported on continuous and isolated spread footings if the structural engineer determines that the planned buildings can accommodate the estimated dynamic compaction and liquefaction-induced settlement magnitudes presented in the "Geologic and Seismic Hazards" section of this report. Otherwise, the buildings should be supported on reinforced mat foundations, post-tensioned (PT) slabs, or footing and grade beam "waffle" type foundations. In our opinion, deep foundations extending through the potentially liquefiable layers or ground improvement would be cost prohibitive for this project.

4.2.2 Foundation Settlements

Provided the recommendations presented in this report are properly followed, we estimate **total static settlement** for the buildings will be less than 1-inch and should occur during construction as the building loads are applied. Differential settlement is expected to be about half of the total static settlement over a horizontal distance of 30 feet (typical column spacing). **Note that static settlement is in addition to the estimated dynamic compaction and liquefaction-induced settlements discussed in the "Geologic and Seismic Hazards" section above.**

4.3 Backstop, Light Pole, Score Board, Traffic Signal, and Trellis Foundation

We expect that lateral loading will govern the design of the baseball backstop, light poles, score boards, traffic signals, and the trellis. Therefore, these structures may be supported on cast-in-drilled-hole (CIDH) piers provided that the recommendations presented in the "Recommendations" section of this report are followed. Static settlement of CIDH piers is expected to be negligible. However, the CIDH piers could experience upwards of 3 inches of total dynamic compaction and liquefaction-induced settlement during a design-level earthquake.

Due to the presence of poorly graded sand underlying the project site, there will be a potential for caving to occur during drilling of the CIDH piers. Therefore, we recommend the piers be either

[10] Tokimatsu, K. and Seed, H. B. (1987), Evaluation of Settlements in Sands Due to Earthquake Shaking, Journal of Geotechnical Engineering, ASCE, Vol. 113, No. 8, August, pp. 861-878.

temporarily/permanently cased or installed using the slurry displacement method. If temporary casing is used, it should consist of smooth-walled steel casing. Corrugated metal pipe (CMP) should not be permitted as temporary casing because it results in excessive voids and/or disturbance of the surrounding soil during removal of the casing.

We expect conventional drilling equipment can be used for the installation of the CIDH piers. However, hard drilling could be encountered during construction if gravel is present. For instance, boring B-3 encountered auger refusal at a depth of about 6 feet BGS during our investigation due to an obstruction (possibly a large gravel particle).

A representative of BSK should be present on a full-time basis during installation of the piers to confirm that subsurface conditions are similar to those encountered in our borings and to check if the contractor is properly casing or using slurry to drill the pier holes.

4.4 Bleacher and Retaining Wall Foundation

The planned bleachers for the athletic field and the 5-foot tall retaining wall that will separate the Phase 1 and Phase 2 buildings may be supported on spread footings, but would be subject to the settlement estimates discussed in the “Foundation Settlements” section above. If desired, these structures can instead be supported on CIDH piers. As noted in the preceding section of this report, static settlement of CIDH piers is expected to be negligible. However, the CIDH piers could experience upwards of 3 inches of total dynamic compaction and liquefaction-induced settlement during a design-level earthquake.

4.5 Slabs-on-Grade

4.5.1 Interior Slabs

In order to provide enhanced subgrade support, we recommend underlying the interior slab-on-grade floors for the new buildings with 6 inches of ¾-inch compacted crushed rock. If this layer is desired to also serve as a capillary break, it should contain less than 5 percent by weight of material passing the No. 4 sieve. The subgrade soils for interior slabs should be properly moisture conditioned prior to placement of concrete.

4.5.2 Exterior Concrete Flatwork

Exterior concrete flatwork may be supported directly on moisture conditioned and compacted native sand subgrade soils or engineered fill. The subgrade soils for exterior flatwork should be properly moisture conditioned prior to placement of concrete. If concrete placement does not take place immediately after the subgrade is prepared, the subgrade soils should be periodically moisturized until concrete is placed.

4.6 Excavations

We anticipate that excavations at the site can be made with standard earthwork equipment, such as excavators, dozers, backhoes, and trenchers. Because the site is underlain by poorly graded sand, shoring or sloping of cut faces and trench walls will likely be necessary to protect personnel and to provide temporary stability. OSHA guidelines should be followed for excavations performed at the site.

4.7 Stockpiles

Stockpiles of undocumented soil are currently located along the eastern half of the site. We understand the soil contained in these stockpiles was brought to the site some time ago from other locations in the City. The stockpiles have vegetation growing on them. If desired, after the vegetation is stripped from these stockpiles during construction, the soils in the stockpiles should be evaluated by BSK to check whether they are suitable for re-use as general fill or backfill at the site (i.e., free of vegetation, organics, debris, deleterious matter, and oversize material; refer to the "Earthwork" section of this report for additional information). If the soils in the stockpiles are not deemed suitable for re-use on-site, they would need to be off-hauled from the site unless otherwise indicated by BSK and the City.

4.8 Staging/Work Area

Existing AC pavement within the project limits may be left in place to provide for a stable staging/work area and for providing drainage relief. In the past, we have found that the subgrade soils beneath paved areas tend to be on the wet side and may require additional time to dry. Therefore, if the existing pavement is left as a staging area, then time should be factored into the construction schedule to allow for drying of the subgrade soils once the AC is removed.

5. RECOMMENDATIONS

Presented below are recommendations for foundations, seismic considerations, exterior flatwork, earthwork, construction considerations, site drainage, storm water infiltration, and pavements for this project.

5.1 Foundations

Depending on how capable the new buildings will be to accommodate the estimated dynamic compaction and liquefaction-induced settlements previously discussed, the planned buildings may be supported on continuous and isolated spread footings, mat foundations, post-tensioned (PT) slabs, or a footing and grade beam "waffle" type foundation. The allowable bearing values provided below assume that the building foundations uniformly bear on properly compacted, firm, and stable subgrade.

The near surface soils should provide adequate bearing support for the proposed buildings provided the foundation excavations bottoms do not expose unsuitable, soft/loose materials. Otherwise, the bottom of the foundations should be overexcavated to a depth deemed suitable by a BSK representative and then be backfilled with properly moisture conditioned and compacted engineered fill, sand-cement slurry (2-sack mix), or lean concrete. A BSK representative should be present during the overexcavation. Unit prices for such overexcavation and backfilling should be obtained during contractor bidding for this project.

5.1.1 Spread Footings

Our recommended allowable soil bearing pressure, depth of embedment, and width of spread footings are presented below. **The bottom of all footing excavations should be properly compacted as recommended in Exhibit 1 in Appendix D of this report.**

Spread Footing Recommendations			
Footing Type	**Allowable Bearing Pressure (psf)***	**Minimum Embedment (in)****	**Minimum Width (in)**
Exterior Continuous Footings	2,500	18	12
Isolated Interior Footings	2,500	18	18x18

* Pounds per square foot, dead plus live load. Includes factor of safety (FS) of at least 2. The allowable bearing pressure may be increased by 1/3 for short-term seismic and wind loads.

** Below lowest adjacent grade defined as bottom of slab on the interior and finish grade at the exterior.

Where footings are located adjacent to below-grade structures (including existing footings) or near major underground utilities, the footings should extend below a 1H:1V (horizontal to vertical) plane projected upward from the structure footing or bottom of the underground utility to avoid surcharging the below grade structure and underground utility with building loads.

Concrete for footings should be placed neat against properly compacted soil or engineered backfill. It is important that footing excavations not be allowed to dry before placing concrete. Even then, because the subsurface soils at the site consist predominantly of sand, it is possible the excavation sidewalls could slough. Therefore, it may be necessary to form the footings before placing concrete. The footing excavations should be monitored by a BSK representative for compliance with appropriate moisture control and to confirm the adequacy of the bearing materials.

5.1.2 Mat Foundation

If a mat foundation is chosen, the mat should have a minimum depth at the edges of 12 inches. It is anticipated that the mat foundation will impose a modest bearing pressure (less than 500 psf). If isolated areas of imposed stress concentrations occur, the mats may be designed for an allowable bearing pressure of 1,500 psf within these isolated areas. The allowable bearing pressure value may be increased by 1/3 for short term seismic and wind loads. The bearing capacity value includes a factor of safety of at least 2. We recommend that immediately after the mat foundation excavation is completed, the bottom of the excavation be covered by either a 2- to 3-inch rat slab or a 4- to 6-inch layer of Caltrans Class 2 aggregate base. This underlayment material would serve as a leveling course and would reduce the risk for the exposed soils at the bottom of the mat excavation to dry out prior to concrete placement. **The bottom of all mat foundation excavations should be properly compacted as recommended in Exhibit 1 in Appendix D of this report.**

5.1.3 Post-Tensioned Slab

If the planned buildings cannot tolerate the amount of dynamic compaction and liquefaction-induced settlement discussed in the "Geologic and Seismic Hazards" section of this report, post-tensioned (PT) slabs may be used to support the buildings. PT slabs use high-strength tensioned steel strands to compress the slabs, keeping the majority of the concrete in compression thus allowing for longer spans. Although liquefaction-induced settlement is not directly mentioned as a design condition in the Post-Tensioning Institute's "Design of Post-Tensioned Slab-on-Ground" manual[11], such settlement may be modeled based on Section 6.13.3 of the manual, which discusses slabs constructed on compressible soils with high expected differential settlements. According to Section 6.13.3 of the manual and Appendix A.8 of the 2nd edition of the Design and Construction of Post-Tensioned Slabs-on-Ground[12], the soil parameters required for design of a PT slab are: 1) soil type, 2) expected soil settlement, 3) allowable soil bearing pressure, and 4) expected differential settlement. The recommended values for these parameters for this project are listed below:

[11] Design of Post-Tensioned Slabs-on-Ground, 2004, Third Edition, Post-Tensioning Institute.

[12] Design and Construction of Post-Tensioned Slabs-on-Ground, 1996, Second Edition, Post-Tensioning Institute.

1. Soil Type[13]: Poorly Graded Sand with Silt (SP-SM)

2. Expected Soil Settlement[13]: δ = 3 inches (this is the total expected dynamic compaction and liquefaction-induced settlement at the ground surface)

3. Allowable soil bearing pressure[13]: q_{allow} = 1,500 psf

4. Expected differential settlement[13]: y_m = 2 inches (this is two-thirds of the total expected dynamic compaction and liquefaction-induced settlement)

The bottom of all PT slab excavations should be properly compacted as recommended in Exhibit 1 in Appendix D of this report.

5.1.4 CIDH Piers

CIDH piers should derive their load capacities through skin friction on the side of the piers. For resistance to uplift loads, the weight of the piers and the skin friction between the piers and the surrounding soils may be used. An allowable skin friction value of 18L psf may be used to resist downward dead plus live loads. L is defined as the effective embedment depth of the pier in units of feet and the maximum skin friction developed is limited to an L of 30 feet. A one-third increase is permitted for wind and/or seismic loading. The dead plus live load friction resistance includes a safety factor of about 2 and the total design downward frictional resistance (including wind and seismic) includes a safety factor of about 1½. Uplift loads for short-term conditions should not exceed 2/3 of the allowable downward skin friction. Piers should be a minimum of 18 inches in diameter and be embedded a minimum of 5 feet below the ground surface. Piers should be spaced a minimum of 3 diameters apart (center to center) or the allowable skin friction would need to be reduced.

We expect conventional drilling equipment can be used for the installation of piers. However, hard drilling, especially in gravelly zones, could be encountered during construction. Unit prices for slower than anticipated drilling should be obtained during bidding. We recommend that steel reinforcement and concrete be placed within about 4 to 6 hours upon completion of each pier hole. As a minimum, the holes should be poured the same day they are drilled. The steel reinforcement should be centered in the pier hole. Due to caving concerns of the subsurface soils, the pier holes should be either temporarily/permanently cased during installation or should be drilled using the slurry displacement method. If the piers are installed using slurry, then the concrete should be placed using tremie methods and the end of the tremie pipe must remain below the surface of the in-place concrete at all times.

If temporary casing is used during construction in lieu of the slurry displacement method, it should consist of smooth-walled steel casing. Corrugated metal pipe (CMP) should <u>not</u> be permitted as temporary casing because it results in excessive voids and/or disturbance of the surrounding soil during removal of the casing. Also, the bottom of the pier holes should be cleaned and/or tamped such that no

[13] Refer to Sections A.8.1.C and A.8.2.A of Appendix A.8 the Design and Construction of Post-Tensioned Slabs-on-Ground, 1996, Second Edition, Post-Tensioning Institute.

more than 2 inches of loose soil remains in the hole prior to the placement of concrete. Concrete used for pier construction should be discharged vertically into the pier holes to reduce aggregate segregation. Under no circumstances should concrete be allowed to free-fall against either the steel reinforcement or the sides of the casing (if used) during construction.

In order to develop the design skin friction value provided above, concrete used for pier construction should have a slump of 6 to 8 inches. A concrete mix with a low water/cement ratio should be used in the construction of the piers to reduce shrinkage of the concrete. To increase the fluidity of the mix for improved consolidation and bond with the reinforcing steel, increased slump may be desirable. If this is the case, the slump should be increased via use of a plasticizer, rather than by adding water to the mix, because a low water to cement ratio is desired for shrinkage control.

If more than 6 inches of standing water is present at the bottom of the pier holes during concrete placement, either the water needs to be pumped out or the concrete needs to be placed into the hole using tremie methods.

BSK should review the foundation plans prior to the issuance of these documents for bidding to confirm that the recommendations presented in this report have been properly incorporated into these documents. In addition, a BSK representative should be present on a full-time basis during installation of the piers to confirm that subsurface conditions are similar to those encountered in our borings and to check if the contractor is properly casing or using slurry to drill the pier holes.

5.1.5 Lateral Resistance

Lateral loads may be resisted by a combination of friction between the foundation bottoms and the supporting subgrade, and by passive resistance acting against the vertical faces of the foundations, including grade beams. An allowable friction coefficient of 0.25 between the foundation and supporting subgrade soils may be used. For passive resistance, an allowable equivalent fluid pressure of 250 pounds per cubic foot (pcf) may be used. The friction coefficient and passive resistance may be used concurrently, and the passive resistance can be increased by one-third for wind and/or seismic loading. The friction and passive values include factors of safety of about 1½. We based these lateral load resistance values on the assumption that the concrete for foundations is either placed directly against engineered fill or that the voids created from the use of forms are backfilled with properly compacted onsite soil per the recommendations in the "Earthwork" section of this report or other approved material, such as sand-cement slurry.

Resistance to lateral loads for CIDH piers can be provided by passive resistance against the piers using an allowable equivalent fluid pressure of250 pcf up to a maximum of 2,000 psf acting against the piers. The passive resistance may be applied to a width of twice the diameter of the piers. Piers should be spaced at least 6 diameters apart (center to center) in the direction of loading or lateral resistance capacity reductions may be necessary. The passive pressure value for CIDH piers includes a factor of safety of about 1½.

Passive resistance and skin friction in the upper foot of soil cover below finished grades should be neglected unless the ground surface is confined by concrete flatwork, pavements, or other such positive protection.

5.2 Modulus of Subgrade Reaction

A modulus of subgrade reaction, K_{V1}, of 120 pounds per square inch per inch (pci) of deflection (based on a one square foot bearing plate) is considered applicable for footings and slabs. This modulus is typically reduced for mat slab sizes larger than 1 square foot. For various slab sizes, the subgrade modulus for sandy soils may be calculated using the following formula:

$$K_S = (K_{V1}) \times \left(\frac{\mathrm{B}+1}{2B}\right)^2$$

Where:

- K_{V1} is the modulus of subgrade reaction for a 1 square foot plate (in units of pci);
- B is the width of the foundation/slab (in units of feet);
- K_S is the adjusted modulus of subgrade reaction based on the actual dimensions of the foundation/slab (in units of pci).

If a computer program is used to design the project foundations and it requires the input of a modulus of subgrade reaction for the site, the designer should check whether the program requires input of the unadjusted or adjusted modulus of subgrade reaction.

5.3 2016 CBC Seismic Design Parameters

The seismicity of the region surrounding the site is discussed in the "Faulting and Seismicity" section of this report. From that discussion, it is important to note that the site is in a region of high seismic activity and will likely be subjected to major shaking during the life of the project. As a result, structures to be constructed on the site should be designed in accordance with applicable seismic provisions of the 2016 California Building Code (CBC).

Based on the results of our analyses, the site subsurface soils are susceptible to liquefaction during a design-level earthquake. Therefore, according to Table 20.3-1 of ASCE 7-10, the site should be classified as Site Class F, which requires site-specific response analysis. However, Sections 11.4.7 and 20.3.1 of ASCE 7-10 state that for a short period (less than ½ second) structure on liquefiable soils, these factors may be based on the assessment of the site class assuming no liquefaction.

Provided the planned buildings and pertinent structures have fundamental periods of less than about ½ second, we recommend using Site Class D (stiff soil profile) for design of these structures and use of

the 2016 CBC mapped seismic design criteria would be considered appropriate for this site. If this is the case, the seismic parameters presented in the table below should be considered applicable for the design of the new building. Otherwise, we should be consulted to evaluate whether a site-specific response analysis is required for the project.

2016 CBC Seismic Design Parameters*			
Seismic Design Parameter	**Value**		**Reference**
Site Class	D		Table 20.3-1, ASCE 7-10
MCE_R Mapped Spectral Acceleration (g)	S_S = 1.444	S_1 = 0.494	USGS Mapped Values based on Figures 1613.3.1(1) and 1613.3.1(2), 2016 CBC
Site Coefficients	F_a = 1.000	F_v = 1.506	Tables 1613.3.3(1) and 1613.3.3(2), 2016 CBC
MCE_R Mapped Spectral Acceleration Adjusted for Site Class Effects (g)	S_{MS} = 1.444	S_{M1} = 0.744	Section 1613.3.3, 2016 CBC
Design Spectral Acceleration (g)	S_{DS} = 0.962	S_{D1} = 0.496	Section 1613.3.4, 2016 CBC
Seismic Design Category	D		Section 1613.3.5, 2016 CBC
MCE_G peak ground acceleration adjusted for Site Class effects (g)	PGA_M = 0.5		Section 11.8.3, ASCE 7-10
Definitions: MCE_R = Risk-Targeted Maximum Considered Earthquake MCE_G = Maximum Considered Earthquake Geometric Mean **** These seismic design parameters are based on the assumption that the new buildings and pertinent structures will have fundamental periods of less than about ½ second. If that is not the case, BSK should evaluate whether a site-specific response analysis is required.***			

As shown above, the short period design spectral response acceleration parameter, S_{DS}, is greater than 0.5 and the long period design spectral response acceleration parameter, S_{D1}, is greater than 0.2. These values characterize the site as Seismic Design Category D as specified in Section 1613.3.5 of the 2016 CBC. In accordance with Section 1613.3.5 of the 2016 CBC, each structure shall be assigned to the more severe seismic design category in accordance with Table 1613.3.5(1) or 1613.3.5(2), irrespective of the fundamental period of vibration of the structure.

5.4 Slabs-on-Grade

Slabs-on-grade for this project will consist of interior concrete floor slabs and exterior concrete flatwork. As previously discussed, the near-surface soils at the site have a low expansion potential. Therefore, we consider the potential for these slabs to be subjected to shrink/swell cycles with fluctuations in moisture content to be low.

5.4.1 Interior Concrete Floor Slabs

We recommend underlying interior floor slabs with 6 inches of ¾-inch compacted crushed rock to provide enhanced subgrade support. If this layer is desired to also serve as a capillary break, it should contain less than 5 percent by weight of material passing the No. 4 sieve. It is important that the crushed rock material be placed as soon as possible after moisture conditioning and compaction of the subgrade materials to reduce drying of the pad subgrade. A representative of BSK should be present to assess the subgrade condition and observe/test the preparation of the subgrade prior to slab construction.

Slab thickness and reinforcing should be designed by a Structural Engineer. As a minimum, we suggest the concrete floor slabs be at least 5 inches thick and properly reinforced. Special care should be taken to ensure that reinforcement is placed at the slab mid-height. The floor slabs should be separated from footings, structural walls, and utilities, and provisions should be made to allow for differential movements at these interfaces. If this is not possible from a structural or architectural design standpoint, it is recommended that the slab connection to footings be reinforced such that there will be resistance to potential differential movement.

5.4.2 Floor Slab Moisture

Subsurface moisture and moisture vapor naturally migrate upward through the soil and, where the soil is covered by a building or pavement, this subsurface moisture will collect. To reduce the impact of the subsurface moisture and potential impact of future introduced moisture (such as landscape irrigation or precipitation), the current industry standard is to place a vapor retarder on the compacted crushed rock layer underlying the slab. This membrane typically consists of visqueen or polyvinyl plastic sheeting at least 15 mils in thickness. It should be noted that although vapor barrier systems are currently the industry standard, this system may not be completely effective in preventing floor slab moisture problems. These systems typically will not necessarily assure that floor slab moisture transmission rates will meet floor-covering manufacturer standards and that indoor humidity levels be appropriate to inhibit mold growth. The design and construction of such systems are totally dependent on the proposed use and design of the proposed building and all elements of building design and function should be considered in the slab-on-grade floor design. Building design and construction have a greater role in perceived moisture problems since sealed buildings/rooms or inadequate ventilation may produce excessive moisture in a building and affect indoor air quality.

Various factors such as surface grades, adjacent planters, the quality of slab concrete and the permeability of the on-site soils affect slab moisture and can control future performance. In many cases, floor moisture problems are the result of either improper curing of floors slabs or improper application of flooring adhesives. We recommend contacting a flooring consultant experienced in the area of concrete slab-on-grade floors for specific recommendations regarding your proposed flooring applications.

Special precautions must be taken during the placement and curing of all concrete slabs. Excessive slump (high water-cement ratio) of the concrete and/or improper curing procedures used during either hot or cold weather conditions could lead to excessive shrinkage, cracking, or curling of the slabs. High water-cement ratio and/or improper curing also greatly increase the water vapor permeability of concrete. We recommend that all concrete placement and curing operations be performed in accordance with the American Concrete Institute (ACI) manual.

It is emphasized that we are not floor moisture proofing experts. We make no guarantee nor provide any assurance that use of capillary break/vapor retarder system will reduce concrete slab-on-grade floor moisture penetration to any specific rate or level, particularly those required by floor covering manufacturers. The builder and designers should consider all available measures for floor slab moisture protection.

Exterior grading will have an impact on potential moisture beneath the floor slab. Recommendations for exterior draining are provided in the "Site Drainage and Storm Water Infiltration" section of this report.

5.4.3 Exterior Concrete Flatwork

Exterior concrete flatwork may be supported directly on properly compacted and moisture conditioned native sand subgrade soils. The subgrade to receive exterior concrete flatwork should be moisture conditioned and compacted according to the recommendations in Exhibit 1 in Appendix D. Where concrete flatwork is to be exposed to vehicle traffic, it should be underlain by a minimum of 6 inches of Caltrans Class 2 aggregate base, as specified in the 2015 Caltrans Standard Specifications, compacted as recommended in Exhibit 1 in Appendix D.

Flatwork should have control joints (i.e., weakened plane joints) spaced no more than 8 feet on centers. Prior to construction of the flatwork, a BSK representative should check subgrade for proper moisture conditioning to near optimum moisture content. If the moisture is found to be below these levels required in Exhibit 1, the flatwork areas will need to be soaked until the proper moisture content is reached. Expansion joint material should be used between flatwork and building and wherever deemed appropriate.

5.5 Retaining Walls

A retaining wall about 5 feet high is planned between the Phase 1 and Phase 2 buildings. This retaining wall may be supported on a continuous footing per the recommendations contained in the "Foundations" section of this report and should be designed to resist static lateral earth pressures due to the adjacent soil loads and any surcharge loads that could be present. Flexible walls should be designed for an active equivalent fluid lateral earth pressure of 35 pcf, while rigid walls should be designed for an at-rest pressure of 50 pcf. These pressures apply to backfill with a gradient of 6H:1V or less. These pressures do not include hydrostatic pressures that might be caused by groundwater or water trapped behind the walls.

For surcharge loads imposed on the walls, a rectangular distribution with a uniform pressure equal to one-third of the surcharge pressure should be used for unrestrained walls (i.e., active earth pressure condition). A uniform pressure equal to one-half of the surcharge pressure should be used for restrained walls (i.e., at-rest earth pressure condition). To reduce the potential for adversely surcharging the walls, construction equipment should stay a minimum lateral distance of one wall height behind the walls. Where this is not feasible, lightweight equipment should be used within this zone.

According to Section 1803.5.12 of the 2016 CBC, dynamic seismic lateral earth pressures need to be included in the design of foundation walls and retaining walls supporting more than 6 feet of backfill height. We recommend using seismic pressures of 11H and 30H psf (where H is the height of the wall in feet) for flexible and restrained walls, respectively. A uniform rectangular pressure distribution with the resultant force acting at the mid-height of the wall may be used for this seismic increment.

Portions of retaining walls higher than 2 feet should be well-drained to reduce hydrostatic pressure. A typical drainage system consists of a 1- to 2-foot wide zone of Caltrans Class 2 Permeable material immediately adjacent to the wall with a perforated pipe at the base of the wall discharging to a storm drain or other appropriate discharge facility. As an alternative, a prefabricated drainage board may be used in lieu of the Class 2 Permeable material.

5.6 Demolition

5.6.1 Existing Utilities

Active or inactive utilities within the construction area should be protected, relocated, or abandoned. Pipelines that are 2 inches in diameter or less may be left in place provided they are cut off and capped at the ends. Pipelines larger than 2 inches in diameter should be removed or filled with a 1-sack sand-cement slurry mix. Active utilities to be reused should be carefully located and protected during demolition and during construction.

5.6.2 Excavation and Backfill of Existing Foundations and Below-Grade Structures

If applicable, all existing foundations and below-grade structures to be abandoned should be demolished and removed. The resulting excavations should then be properly backfilled with compacted engineered fill per the requirements of the "Earthwork" section of this report. A BSK representative should observe and test the compaction of for earthwork activities during construction.

5.6.3 Reuse of On-site Concrete and Asphalt Concrete

Existing concrete flatwork and asphalt concrete (AC) may be pulverized and mixed with the underlying gravel layer (i.e., aggregate base), if present, for use as general engineered fill if it meets the gradation requirements discussed in the "Re-Use of On-site Soils and Imported Fill Material" section of this report.

Consideration should also be given to processing the existing concrete, AC, and underlying aggregate base (if present) for re-use as Caltrans Class 2 aggregate base for future paved areas and exterior flatwork provided it meets the gradation, R-Value, durability index, and sand equivalent requirements of Section 26 of the 2015 Caltrans Standard Specifications unless otherwise indicated by BSK during construction.

5.7 Earthwork

Earthwork at the site will generally consist of mass grading, subgrade preparation, and placement of aggregate base for exterior flatwork and pavements; placement of crushed rock for interior building slabs; excavation, removal, and backfill of existing underground utility lines; and excavation and backfill of new underground utility lines; foundation excavations; and retaining wall backfill. Although grading plans for the project are not currently available, we anticipate on the order of 5+ feet of cut and fill to reach design finish grades, grade the site to drain, and remove the existing soil stockpiled in the unimproved areas of the site. We expect excavations for new utility lines will be up to 5 feet deep.

BSK should review the grading plans for conformance to our design recommendations prior to construction bidding. In addition, it is important that a representative of BSK observe and evaluate the competency of existing soils or new fill underlying structures, concrete flatwork, and pavements. In general, soft/loose or unsuitable materials encountered should be over excavated, removed, and replaced with compacted engineered fill material under the observation of a BSK representative.

Site preparation and grading for this project should be performed in accordance with the site-specific recommendations provided below. A summary of compaction requirements for this project is presented in Exhibit 1 in Appendix D. Additional earthwork recommendations are presented in related sections of this report.

5.7.1 Site Preparation and Grading

Prior to the start of grading and subgrade preparation operations, the site should first be cleared and stripped to remove all surface vegetation, organic laden topsoil and debris generated during the demolition of existing pavements, concrete flatwork, and landscaping located within the site. Stripping should extend laterally a minimum of 3 feet beyond the limits of exterior flatwork and pavement and a minimum of 5 feet beyond the limits of new buildings, where feasible. Stripped topsoil from landscaped areas may be stockpiled for later use in landscaping areas; however, this material should not be reused for engineered fill.

Any buried tree stumps, roots, or major root systems thicker than approximately 1-inch in diameter, septic tanks and leach field lines uncovered during site stripping and/or grading activities should be removed. Unit prices for removal of such material should be obtained during bidding.

Following stripping and removal of deleterious materials, exposed subgrade areas and areas to receive fill should be scarified to a minimum depth of 12 inches, moisture conditioned, and recompacted as indicated in Exhibit 1. Scarification and recompaction should extend laterally a minimum of 5 feet beyond the limits of structures (defined as the outside perimeter of building walls or footing outer limits, whichever results in the greatest building envelope) and 3 feet beyond the edge of exterior flatwork and pavements, where feasible. All fills should be compacted in lifts of 8-inch maximum uncompacted thickness. A summary of compaction requirements for the project is presented in Exhibit 1. Laboratory maximum dry density and optimum moisture content relationships should be evaluated based on ASTM Test Designation D1557 (latest edition).

Proper moisture conditioning is important. After subgrade soils are properly moisture conditioned, their moisture content should be maintained until they are covered by improvements. This may require periodic moisturizing of the subgrade soils if they are allowed to dry. Where aggregate base is used, it should be placed immediately over the prepared subgrade to avoid drying of the subgrade.

All site preparation and fill placement should be observed by a BSK representative. It is important that, during the stripping and scarification process, our representative be present to observe whether any undesirable material is encountered in the construction area and whether exposed soils are similar to those encountered during our field investigation.

5.7.2 Re-Use of On-site Soil and Imported Fill Material

On-site soils are considered suitable for re-use as general engineered fill and backfill provided vegetation, organic materials, and deleterious matter are removed. A BSK representative should be present on-site during grading to visually confirm the suitability of the on-site soil to be used as fill and backfill, especially the existing stockpiles. Particles larger than 3 inches within the on-site soils (if encountered) should either be removed and disposed offsite or broken down to 3 inches or less prior to using the soil as engineered fill. Nesting (i.e., concentration) of larger particles should be avoided to reduce the potential that this could create voids and allow future settlement in the overlaying fill/backfill.

Maximum particle size for fill material should be limited to 3 inches, with at least 90 percent by weight passing the 1-inch sieve. Proper granular bedding and shading should be used beneath and around new utilities. Where imported fill is required, it should be granular in nature, adhere to the above gradation recommendations, and conform to the following minimum criteria:

Imported Fill Criteria	
Plasticity Index	15 or less
Liquid Limit	Less than 30%
% Passing #200 Sieve	8 % – 40%
R-Value*	50 or greater

* R-Value requirement applies to import fill to be placed within the upper 2 feet below finished pavement subgrade and within 3 feet laterally of the pavement limits.

Open graded materials such as crushed rock and pea gravel are not recommended for use as backfill for excavations because these materials can result in migration of finer particles above and surrounding the backfill into voids in these materials, which could result in settlement of the ground surface above and surrounding the backfill.

Imported fill material should not be any more corrosive than the on-site soils and should not be classified as being more corrosive than "moderately corrosive." Prior to transporting proposed imported materials to the site, the contractor should make representative samples of the material available to BSK at least 10 working days in advance to allow the engineer enough time to confirm the material meets the above requirements. Additional time may be required if corrosion test results for the proposed imported materials are not readily available at the time the material is submitted for our review. All on-site or imported fill material should be compacted to the recommendations provided for engineered fill in Exhibit 1.

5.7.3 Excavation and Backfill

We anticipate that excavations at the site can be made with standard earthwork equipment, such as excavators, dozers, backhoes, and trenchers. Because the site is underlain by poorly graded sand, shoring or sloping of cut faces and trench walls will likely be necessary to protect personnel and to provide temporary stability.

All excavations made at the site should be evaluated to monitor stability prior to personnel entering them. All trenches and excavations should conform to the current OSHA requirements for work safety. It is the contractor's responsibility to follow OSHA temporary excavation guidelines and grade the slopes with adequate layback or provide adequate shoring and underpinning of existing structures and improvements, as needed. Slope layback and/or shoring measures should be adjusted as necessary in the field to suit the actual conditions encountered in order to protect personnel and equipment within excavations. Construction equipment and soil stockpiles should be set back a minimum horizontal distance of H away from the edge of excavations, where H is equal to the depth of the excavation. This setback distance also applies to shored excavations unless the shoring design takes into account any surcharge loads associated with the construction equipment and stockpiles.

Care should be taken during construction to reduce the impact of trenching on adjacent structures and pavements. Excavations should be located so that no structures, foundations, and slabs, existing or new, are located above a plane projected 1H:1V (horizontal to vertical) upward from any point in an excavation, regardless of whether it is shored or unshored.

During our field investigations, free groundwater was observed at a depth of approximately 28½ feet BGS. However, the actual depth at which groundwater may be encountered in trenches and excavations may vary. As a minimum, provisions should be made to ensure that conventional sump pumps used in typical trenching and excavation projects are available during construction in case substantial runoff water accumulates within the excavations as a result of wet weather conditions.

Appropriate provisions should be made to prevent surface water from ponding adjacent to the top of trenches and excavations and flowing over the sides of the excavations, otherwise the excavation side walls and/or slopes could be compromised. Backfill for trenches and other excavations beneath pavements and concrete flatwork should be compacted as noted in Exhibit 1. Special care should be taken in the control of utility trench backfilling under structures, flatwork/slab areas, and other improvements. Poor compaction may cause excessive settlements resulting in damage to overlying improvements.

5.8 Site Drainage

Proper site drainage is important for the long-term performance of the new buildings, pavements, and concrete flatwork. The site should generally be graded so as to carry surface water away from foundations at a minimum of 2 percent in paved areas and 5 percent in landscaped area to a minimum of 10 feet laterally from structures as required by the 2016 CBC. In addition, all roof gutters should be connected directly into a storm drainage system, or drain onto impervious surface (not splash blocks) that drain away from the structure, provided that a safety hazard is not created.

5.9 Storm Runoff Mitigation

Storm runoff regulations require pretreatment of runoff and infiltration of storm water to the extent feasible. Typically, this results in the use of bioretention areas, vegetated swales, infiltration trenches, or permeable pavement near or within parking lots and at the location of roof run-off collection. These features are well suited for coarse-grained soils such as the sand soils encountered at the site, because these soils typically have moderate to high permeability rates and typically do not require significant time for infiltration to occur. Nevertheless, bioretention areas, vegetated swales, and infiltration areas should be located in landscaped areas and well away from pavements, buildings, slopes, and levees to reduce adverse impacts to these improvements. If it is not possible to locate these infiltration systems away from such improvements, alternatives that isolate the infiltrated water from planned improvements, such as flow-through planters, should be considered.

Based on our experience, we expect the near-surface poorly graded sand soils underlying the site to have moderate permeability. Therefore, we classify the site's surficial soils as predominantly hydrologic soil group B per Chapter 7 of Part 630 Hydrology National Engineering Handbook (United States Department of Agriculture, 2007). A hydraulic conductivity of 1.6 inches/hour may be used in the design of bioswales and infiltration basins for the project. This value only applies to the native surficial poorly graded sand encountered in our borings.

Note that the infiltration rates may be reduced over time as silting of bioswales and infiltration basins occurs. Furthermore, if the bottom of such facilities is compacted by heavy equipment, infiltration rates are expected to be significantly reduced. Infiltration of water through soil is highly dependent on such factors as grain size distribution of the soil particles, particle shape, clay content, and density. Small changes in soil conditions, including density, can cause large differences in infiltration rates. **Therefore,**

we recommend that construction equipment not be allowed to drive over or compact the bottom of bioswales and infiltration basins for the project unless otherwise indicated by BSK. If fill material needs to be placed in bioswales and infiltration basins, the fill material should consist of select, free-draining sand meeting the requirements set forth by the civil engineer. BSK should review the criteria for such fill prior to the start of construction.

It should be noted that during periods of prolonged precipitation, the underlying soils tend to become saturated to greater and greater depths/extents. Therefore, infiltration rates tend to decrease with prolonged rainfall.

5.10 Pavements

5.10.1 Asphalt Concrete Pavements

We have made our asphalt concrete pavement designs assuming the pavement subgrade soil will be similar to the near surface soils described in the boring logs. The near surface soils at the site appear to have a low expansion potential and are therefore expected to have a high Resistance (R) Value. We ran R-Value testing on a sample collected from the upper 5 feet at boring B-1, which resulted in an R-Value of 83. Due to the potential variability of the sand and silt content contained in the surficial soils at the site, we recommend using an R-Value of 50 for design purposes.

Pavement designs for various Traffic Indices (TIs) based on an R-Value of 50 are presented below. Each TI represents a different level of use. The owner or designer should determine which level of use best reflects the project and select appropriate pavement section(s) accordingly. The recommended pavement sections presented in the table below were developed using the Caltrans Flexible Pavement Design Method.

Note that any imported soil to be used as engineered fill within the upper 2 feet below finished pavement subgrade and within 3 feet laterally of the pavement limits must have an R-Value of 50 or greater.

Pavement Design Recommendations (R-Value = 50)		
Traffic Index	AC[1] (inches)	Class 2 AB[2] (inches)
5.0	2.5	4.0
5.5	3.0	4.0
6.0	3.0	4.0
6.5	3.5	4.0

1. Type A Asphalt Concrete
2. Caltrans Class 2 Aggregate Base (Minimum R-Value = 78)

We recommend that the subgrade soil over which the pavement sections are to be placed be moisture conditioned and compacted according to the recommendations in Exhibit 1. Subgrade preparation should extend a minimum of 3 feet laterally beyond the back of curb or edge of pavement, where feasible.

Paved areas should be sloped and drainage gradients maintained to carry all surface water to appropriate collection points. Surface water should not be allowed to pond anywhere on the site during or after construction. Additionally, we recommend that the pavement section be positively isolated from intrusion of irrigation or surface water from any adjacent landscaped or vacant areas. Concrete curbs should extend a minimum of 2 inches below the aggregate base section and into the subgrade to provide a barrier against lateral migration of landscape water into the pavement section. Weep holes spaced at 4 feet on centers should also be provided. In lieu of the weep holes, a more effective system is to install a subdrain behind the curbs.

In addition, we recommend that all pavements conform to the following criteria:

- All trench backfills, including utility and sprinkler lines, should be properly placed and adequately compacted to provide a stable subgrade, in accordance with the compaction recommendations in Exhibit 1;

- An adequate drainage system should be provided to prevent surface water or subsurface seepage from saturating the subgrade soil;

- The asphalt concrete and aggregate base materials should conform to the 2015 Caltrans Standard Specifications; and

- Placement and compaction of pavements should be performed and tested in accordance to appropriate Caltrans test procedures.

5.10.2 Portland Cement Concrete Pavements

If used, Portland Cement Concrete (PCC) pavement for vehicle traffic should have a minimum thickness of 6 inches supported over 6 inches of Caltrans Class 2 aggregate base. The aggregate base and subgrade for PCC pavements should be moisture conditioned and compacted per Exhibit 1 in Appendix D. Construction joints should be located no more than 12 feet apart in both directions. Concrete compressive strength should be tested in lieu of third point loading for rupture strength. A minimum 28-day compressive strength of 3,000 pounds per cubic foot (psi) should be specified for the concrete mix design. The PCC pavement should be continuously reinforced using No. 4 bars (or larger) spaced no more than 18 inches on center in both directions. Steel reinforcement should be located near the mid thickness of the concrete slab. Final design of the PCC pavement is the responsibility of the civil or structural engineer for the project.

5.11 Corrosion

A sample was collected during our field investigation at depths of approximately 1 to 2 feet BGS in boring B-1 and was submitted for corrosion testing. The sample was tested by CERCO Analytical, a State-certified laboratory in Concord, California, for redox potential, pH, resistivity, chloride content, and sulfate content in accordance with ASTM test methods. The test results are presented at the end of Appendix C. Also included is the evaluation by CERCO Analytical of the corrosion test results. Because we are not corrosion specialists, we recommend that a corrosion specialist be consulted for advice on proper corrosion protection for underground piping which will be in contact with the soils and other design details.

Based upon the resistivity measurements, the sample tested is classified as "moderately corrosive" by CERCO Analytical. They recommend that all buried iron, steel, cast iron, ductile iron, galvanized steel, and dielectric coated steel or iron be properly protected against corrosion depending upon the critical nature of the structure. They also recommend all buried metallic pressure piping, such as ductile iron firewater pipelines, should be protected against corrosion.

The above are general discussions. A more detailed investigation may include more or fewer concerns, and should be directed by a corrosion expert. BSK does not practice corrosion engineering. Consideration should also be given to soils in contact with concrete that will be imported to the site during construction, such as topsoil and landscaping materials. For instance, any imported soil materials should not be any more corrosive than the on-site soils and should not be classified as being more corrosive than "moderately corrosive." Also, on-site cutting and filling may result in soils contacting concrete that were not anticipated at the time of this investigation.

5.12 Plan/Specification Review and Construction Observation

We recommend that BSK be retained by the Client to review the geotechnical aspects of the 90 percent complete grading (i.e., civil) and structural plans and specifications before they go out to bid. It has been our experience that this review provides an opportunity to detect misinterpretation or misunderstandings of our recommendations prior to the start of construction.

Variations in soil types and conditions are possible and may be encountered during construction. To permit correlation between the soil data obtained during this investigation and the actual soil conditions encountered during construction, we recommend that BSK be retained to provide observation and testing services during site earthwork and foundation construction. This will allow us the opportunity to compare actual conditions exposed during construction with those encountered in our investigation and to provide supplemental recommendations if warranted by the exposed conditions. Earthwork should be performed in accordance with the recommendations presented in this report, or as recommended by BSK during construction. BSK should be notified at least two weeks prior to the start of construction and prior to when observation and testing services are needed.

6. ADDITIONAL SERVICES AND LIMITATIONS

6.1 Additional Services

The review of plans and specifications, and field observation and testing during construction by BSK are an integral part of the conclusions and recommendations made in this report. If BSK is not retained for these services, the client will be assuming BSK's responsibility for any potential claims that may arise during or after construction due to the misinterpretation of the recommendations presented herein. The recommended tests, observations, and consultation by BSK during construction include, but are not limited to:

- review of plans and specifications;
- observations of site grading, including stripping and engineered fill construction;
- observation of foundation construction; and
- in-place density testing of fills, backfills, and finished subgrades.

6.2 Limitations

The recommendations contained in this report are based on our field observations and subsurface exploration, limited laboratory tests, and our present knowledge of the proposed construction. It is possible that soil conditions could vary between or beyond the points explored. If soil conditions are encountered during construction that differ from those described herein, we should be notified immediately in order that a review may be made and any supplemental recommendations provided. If the scope of the proposed construction, including the proposed loads or structural locations, changes from that described in this report, our recommendations should also be reviewed.

We prepared this report in substantial accordance with the generally accepted geotechnical engineering practice as it exists in the site area at the time of our study. No warranty, either express or implied, is made. The recommendations provided in this report are based on the assumption that an adequate program of tests and observations will be conducted by BSK during the construction phase in order to evaluate compliance with our recommendations. Other standards or documents referenced in any given standard cited in this report, or otherwise relied upon by the author of this report, are only mentioned in the given standard; they are not incorporated into it or "included by reference", as that latter term is used relative to contracts or other matters of law.

This report may be used only by the Client and only for the purposes stated within a reasonable time from its issuance, but in no event later than two (2) years from the date of the report, or if conditions at the site have changed. If this report is used beyond this period, BSK should be contacted to evaluate whether site conditions have changed since the report was issued.

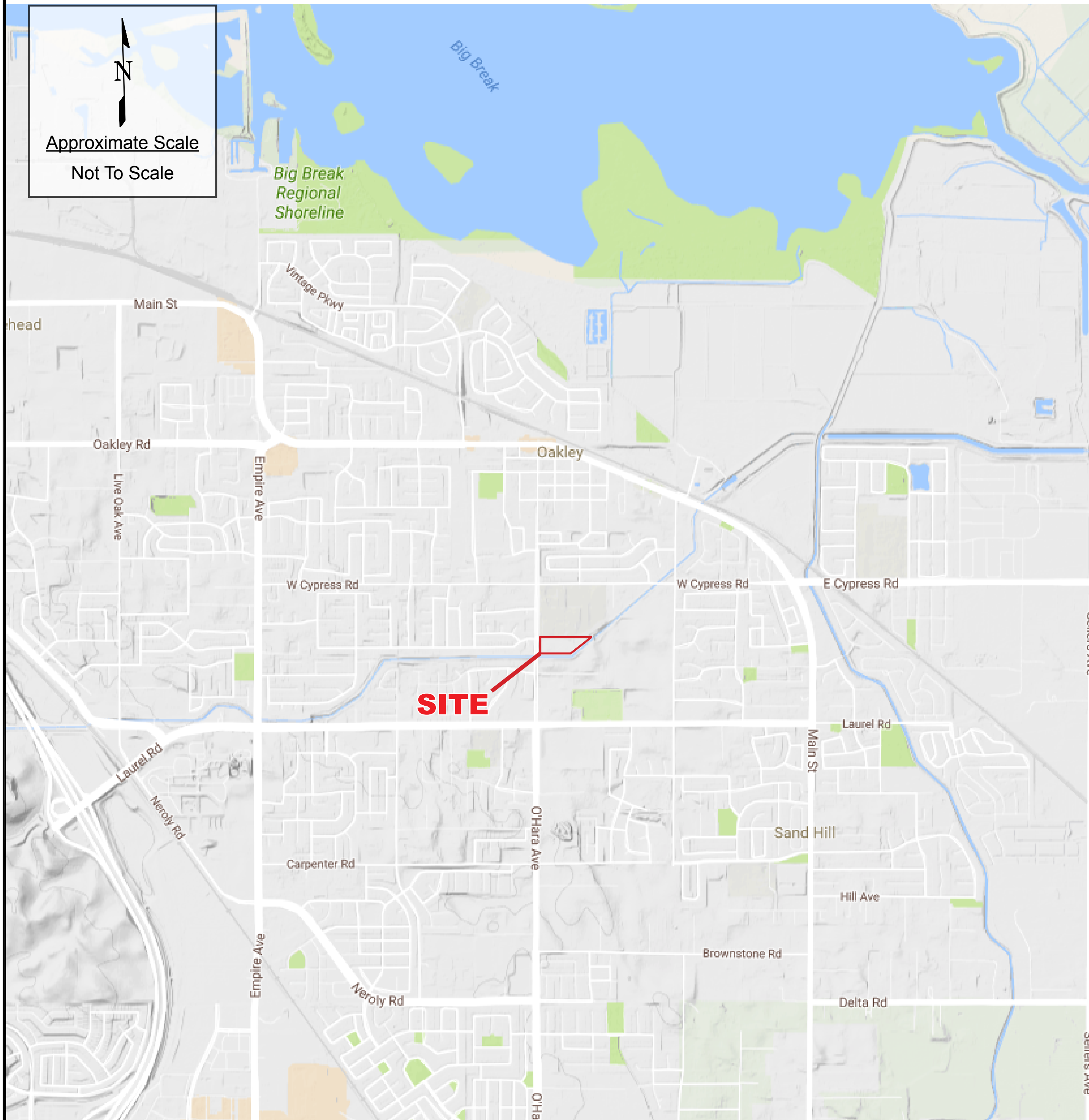
Also, land or facility use, on and off-site conditions, regulations, or other factors may change over time, and additional work may be required with the passage of time. Based on the intended use of the report, BSK may recommend that additional work be performed and that an updated report be issued.

The scope of services for this subsurface investigation and geotechnical report did not include environmental assessments or evaluations regarding the presence or absence of wetlands or hazardous substances in the soil, surface water, or groundwater at this site.

BSK conducted subsurface exploration and provided recommendations for this project. We understand that BSK will be given the opportunity to perform a formal geotechnical review of the final project plans and specifications. In the event BSK is not retained to review the final project plans and specifications to evaluate if our recommendations have been properly interpreted, we will assume no responsibility for misinterpretation of our recommendations.

We recommend that all earthwork during construction be monitored by a representative of BSK, including site preparation, foundation excavation, placement of engineered fill, and trench backfill. The purpose of these services would be to provide BSK the opportunity to observe the actual soil conditions encountered during construction, evaluate the applicability of the recommendations presented in this report to the soil conditions encountered, and recommend appropriate changes in design or construction procedures if conditions differ from those described herein.

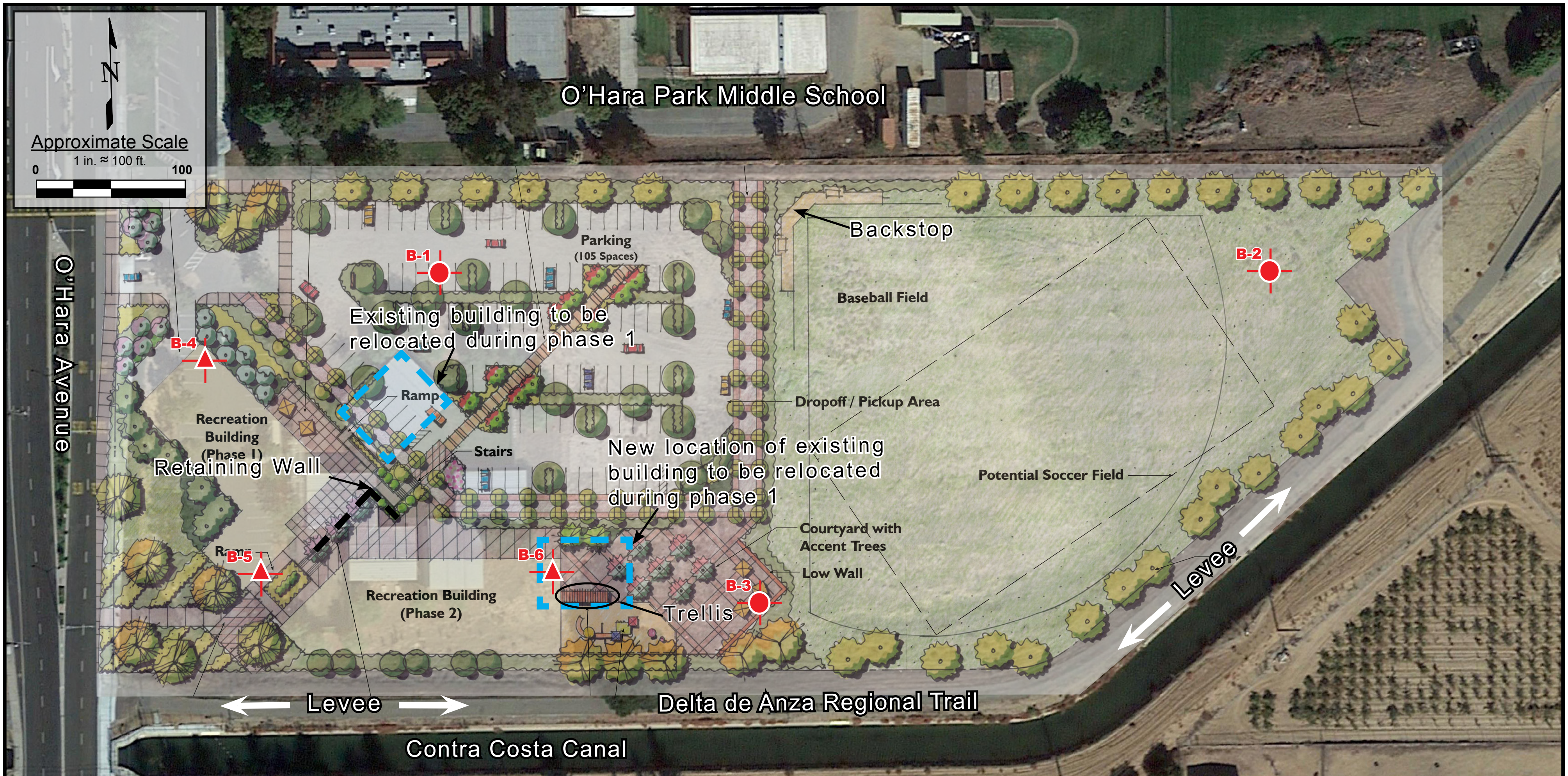
PLATES



Reference: http://maps.google.com, 2016

BSK ASSOCIATES

PROJECT NO. G16-221-11L	**VICINITY MAP**	PLATE
DRAWN: 12/14/16		
DRAWN BY: M. McNally		1
CHECKED BY: C. Melo	Oakley Recreation Center 1250 O'Hara Avenue Oakley, CA	
FILE NAME: SitePlan.indd		



Legend

B-1 Approximate Boring Location (BSK, 2016)

B-4 Approximate Boring Locations (BSK, 2017)

References:
1. http://earth.google.com, 2016
2. All markings are approximate
3. Master Plan Illustrative for Oakley Recreation Center, Oakley California, dated December 2016 by Gates + Associates
4. Phase 1 Illustrative for Oakley Recreation Center, Oakley, California, dated December 2016 by Gates + Associates

BSK ASSOCIATES

PROJECT NO.	G16-221-11L
DRAWN:	2/24/17
DRAWN BY:	M. McNally
CHECKED BY:	C. Melo
FILE NAME:	SitePlan.indd

SITE PLAN

Oakley Recreation Center
1250 O'Hara Avenue
Oakley, California

PLATE **2**

APPENDIX A

PRELIMINARY LAYOUT PLANS FOR PHASES 1 AND 2

GATES +ASSOCIATES

OAKLEY RECREATION CENTER OAKLEY, CALIFORNIA

PHASE I ILLUSTRATIVE
DECEMBER 2016

GATES +ASSOCIATES

OAKLEY RECREATION CENTER OAKLEY, CALIFORNIA

MASTER PLAN ILLUSTRATIVE
DECEMBER 2016

APPENDIX B

BORING LOGS

TABULATED BORING LOGS

Boring No.	Approximate Depth Below Ground Surface (feet)	Description	Remarks
B-1	0 to ¾	**Poorly Graded Sand with Silt and Gravel (SM)** – brown, dry to slightly moist, fine grained sand, up to 1¼-inch subangular gravel, appears to be well compacted, approx. 9 inches thick (FILL)	- Performed on 11/17/16 - Elevation = 40 feet* - Boring terminated at approx. 8½ feet. No free groundwater observed. - Boring backfilled with soil cuttings - At 1 to 2 feet, corrosivity testing (see Appendix C) - At 2 feet, Passing No. 200 Sieve = 10% - At 1 to 5 feet, R-Value = 83 (see Appendix C) - At 5 feet, DD = 111 pcf and MC = 4%
	¾ to 8½	**Poorly Graded Sand with Silt and Gravel (SP-SM)** – brown, slightly moist, appears loose to medium dense, fine grained sand, up to ¼-inch subangular gravel No gravel encountered below 5 feet	
B-2	0 to 8½	**Poorly Graded Sand (SP)** – yellowish brown, slightly moist, appears loose to medium dense, fine grained sand	- Performed on 11/17/16 - Elevation = 45 feet* - Boring terminated at approx. 8½ feet. No free groundwater observed. - Boring backfilled with soil cuttings - At 2 feet, DD = 108 pcf and MC = 3% - At 8 feet, Passing No. 200 Sieve = 3%
B-3	0 to 6	**Poorly Graded Sand with Gravel (SP)** – brown, slightly moist, appears loose to medium dense, fine grained sand, up to 1¼-inch subangular gravel Refusal at 6 feet due obstruction, possibly large gravel particle	- Performed on 11/17/16 - Elevation = 43 feet* - Boring terminated at approx. 6 feet. No free groundwater observed. - Boring backfilled with soil cuttings - At 5 feet, DD = 107 pcf and MC = 4%

Notes/Abbreviations:
DD = in-situ dry unit weight
MC = in-situ moisture content
* Estimated ground surface elevation based on Google Earth.

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D 2487/2488)

MAJOR DIVISIONS				*GRAPHIC LOG*	*TYPICAL DESCRIPTIONS*
COARSE GRAINED SOILS (More than half of material is larger than the #200 sieve)	GRAVELS (More than half of coarse fraction is larger than the #4 sieve)	CLEAN GRAVELS WITH <5% FINES	$Cu \geq 4$ and $1 \leq Cc \leq 3$	GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
			$Cu < 4$ and/or $1 > Cc > 3$	GP	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
		GRAVELS WITH 5 to 12% FINES	$Cu \geq 4$ and $1 \leq Cc \leq 3$	GW-GM	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE FINES
				GW-GC	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE CLAY FINES
			$Cu < 4$ and/or $1 > Cc > 3$	GP-GM	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE FINES
				GP-GC	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE CLAY FINES
		GRAVELS WITH >12% FINES		GM	SILTY GRAVELS, GRAVEL-SILT-SAND MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
				GC-GM	CLAYEY GRAVELS, GRAVEL-SAND-CLAY-SILT MIXTURES
	SANDS (More than half of coarse fraction is smaller than the #4 sieve)	CLEAN SANDS WITH <5% FINES	$Cu \geq 6$ and $1 \leq Cc \leq 3$	SW	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
			$Cu < 6$ and/or $1 > Cc > 3$	SP	POORLY-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
		SANDS WITH 5 to 12% FINES	$Cu \geq 6$ and $1 \leq Cc \leq 3$	SW-SM	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE FINES
				SW-SC	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE CLAY FINES
			$Cu < 6$ and/or $1 > Cc > 3$	SP-SM	POORLY-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE FINES
				SP-SC	POORLY-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE CLAY FINES
		SANDS WITH >12% FINES		SM	SILTY SANDS, SAND-GRAVEL-SILT MIXTURES
				SC	CLAYEY SANDS, SAND-GRAVEL-CLAY MIXTURES
				SC-SM	CLAYEY SANDS, SAND-SILT-CLAY MIXTURES
FINE GRAINED SOILS (More than half of material is smaller than the #200 sieve)	SILTS AND CLAYS (Liquid limit less than 50)			ML	INORGANIC SILTS AND VERY FINE SANDS, SILTY OR CLAYEY FINE SANDS, SILTS WITH SLIGHT PLASTICITY,
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				CL-ML	INORGANIC CLAYS-SILTS OF LOW PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS & ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS (Liquid limit greater than 50)			MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILT
				CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
				OH	ORGANIC CLAYS & ORGANIC SILTS OF MEDIUM-TO-HIGH PLASTICITY

BSK ASSOCIATES

PROJECT NO.	**G16-221-11L**
DRAWN:	2/28/17
DRAWN BY:	D. Tower
CHECKED BY:	C. Melo
FILE NAME:	Legend.indd

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D 2487/2488)

Oakley Recreation Center
1250 O'Hara Avenue
Oakley, CA

PLATE

B-1

SOIL DESCRIPTION KEY

MOISTURE CONTENT

DESCRIPTION	ABBR	FIELD TEST
Dry	D	Absence of moisture, dusty, dry to the touch
Moist	M	Damp but no visible water
Wet	W	Visible free water, usually soil is below water table

CEMENTATION

DESCRIPTION	FIELD TEST
Weakly	Crumbles or breaks with handling or slight finger pressure
Moderately	Crumbles or breaks with considerable finger pressure
Strongly	Will not crumble or break with finger pressure

PLASTICITY

DESCRIPTION	ABBR	FIELD TEST
Non-plastic	NP	A 1/8-in. (3 mm) thread cannot be rolled at any water content.
Low (L)	LP	The thread can barely be rolled and the lump or thread cannot be formed when drier than the plastic limit.
Medium (M)	MP	The thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be rerolled after reaching the plastic limit. The lump or thread crumbles when drier than the plastic limit
High (H)	HP	It takes considerable time rolling and kneeding to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump or thread can be formed without crumbling when drier than the plastic limit

STRUCTURE

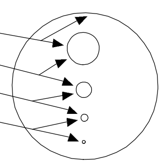
DESCRIPTION	CRITERIA
Stratified	Alternating layers of varying material or color with layers at least 1/4 in. thick, note thickness
Laminated	Alternating layers of varying material or color with the layer less than 1/4 in. thick, note thickness
Fissured	Breaks along definite planes of fracture with little resistance to fracturing
Slickensided	Fracture planes appear polished or glossy, sometimes striated
Blocky	Cohesive soil that can be broken down into small angular lumps which resist further breakdown
Lensed	Inclusion of small pockets of different soils, such as small lenses of sand scattered through a mass of clay; note thickness
Homogeneous	Same color and appearance throughout

CONSISTENCY - FINE-GRAINED SOIL

CONSISTENCY	ABBR	FIELD TEST
Very Soft	VS	Thumb will penetrate soil more than 1 in. (25 mm)
Soft	S	Thumb will penetrate soil about 1 in. (25 mm)
Firm	F	Thumb will indent soil about 1/4 in. (6 mm)
Hard	H	Thumb wil not indent soil but readily indented with thumbnail
Very Hard	VH	Thumbnail will not indent soil

GRAIN SIZE

DESCRIPTION		SIEVE SIZE	GRAIN SIZE	APPROXIMATE SIZE
Boulders		>12"	>12"	Larger than basketball-sized
Cobbles		3 - 12'	3 - 12"	Fist-sized to basketball-sized
Gravel	coarse	3/4 -3"	3/4 -3"	Thumb-sized to fist-sized
	fine	#4 - 3/4"	0.19 - 0.75"	Pea-sized to thumb-sized
Sand	coarse	#10 - #4	0.079 - 0.19"	Rock salt-sized to pea-sized
	medium	#40 - #10	0.017 - 0.079"	Sugar-sized to rock salt-sized
	fine	#200 - #10	0.0029 - 0.017"	Flour-sized to sugar-sized
Fines		Passing #200	<0.0029	Flour-sized and smaller

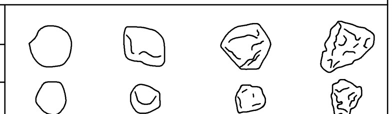


REACTION WITH HCL

DESCRIPTION	FIELD TEST
None	No visible reaction
Weak	Some reaction, with bubbles forming slowly
Strong	Violent reaction, with bubbles forming immediately

ANGULARITY

DESCRIPTION	ABBR	CRITERIA
Angular	A	Particles have sharp edges and relatively plane sides with unpolished surfaces
Subangular	SA	Particles are similar to angular description but have rounded edges
Subrounded	SR	Particles have nearly plane sides but have well-rounded corners and edges
Rounded	R	Particles have smoothly curved sides and no edges



APPARENT / RELATIVE DENSITY - COARSE-GRAINED SOIL

APPARENT DENSITY	ABBR	SPT (# blows/ft)	MODIFIED CA SAMPLER (# blows/ft)	CALIFORNIA SAMPLER (# blows/ft)	RELATIVE DENSITY (%)	FIELD TEST
Very Loose	VL	<4	<4	<5	0 - 15	Easily penetrated with 1/2-inch reinforcing rod by hand
Loose	L	4 - 10	5 - 12	5 - 15	15 - 35	Difficult to penetrate with 1/2-inch reinforcing rod pushed by hand
Medium Dense	MD	10 - 30	12- 35	15 - 40	35 - 65	Easily penetrated a foot with 1/2-inch reinforcing rod driven with 5-lb. hammer
Dense	D	30 - 50	35 - 60	40 - 70	65 - 85	Difficult to penetrate a foot with 1/2-inch reinforcing rod driven with 5-lb. hammer
Very Dense	VD	>50	>60	>70	85 - 100	Penetrated only a few inches with 1/2-inch reinforcing rod driven with 5-lb. hammer

LOG SYMBOLS

	BULK / BAG SAMPLE	-4	PERCENT FINER THAN THE NO. 4 SIEVE (ASTM Test Method C 136)
	SPLIT BARREL SAMPLER (2-1/2 inch outside diameter)	-200	PERCENT FINER THAN THE NO. 200 SIEVE (ASTM Test Method C 117)
	SPLIT BARREL SAMPLER (3 inch outside diameter)	LL	LIQUID LIMIT (ASTM Test Method D 4318)
	STANDARD PENETRATION SPLIT SPOON SAMPLER (2 inch outside diameter)	PI	PLASTICITY INDEX (ASTM Test Method D 4318)
	CONTINUOUS CORE	TXUU	UNCONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION (EM 1110-1-1906)/ASTM Test Method D 2850
	SHELBY TUBE	EI	EXPANSION INDEX (UBC STANDARD 18-2)
	ROCK CORE	COL	COLLAPSE POTENTIAL
	GROUNDWATER LEVEL (encountered at time of drilling)	UC	UNCONFINED COMPRESSION (ASTM Test Method D 2166)
	GROUNDWATER LEVEL (measured after drilling)		
	SEEPAGE	MC	MOISTURE CONTENT (ASTM Test Method D 2216)

GENERAL NOTES

Boring log data represents a data snapshot.

This data represents subsurface characteristics only to the extent encountered at the location of the boring.

The data inherently cannot accurately predict the entire subsurface conditions to be encountered at the project site relative to construction or other subsurface activities.

Lines between soil layers and/or rock units are approximate and may be gradual transitions.

The information provided should be used only for the purposes intended as described in the accompanying documents.

In general, Unified Soil Classification System designations presented on the logs were evaluated by visual methods.

Where laboratory tests were performed, the designations reflect the laboratory test results.

BSK ASSOCIATES

PROJECT NO. G16-221-11L	**LOG KEY**	**PLATE**
DRAWN: 2/28/17		
DRAWN BY: D. Tower		**B-3**
CHECKED BY: C. Melo	**Oakley Recreation Center**	
FILE NAME: Legend.indd	**1250 O'Hara Avenue**	
	Oakley, CA	

BSK ASSOCIATES

BSK Associates
324 Earhart Way
Livermore, CA 94551
Telephone: (925)-315-3151
Fax: (925)-315-3152

LOG OF BORING NO. B-4

Project Name: **Oakley Recreation Center**
Project Number: **G16-221-11L**
Project Location: **1250 O'Hara Avenue Oakley, CA**
Logged by: **M. McNally**
Checked by: **D. Tower**

Surface El.: **41 ft**
Location: **1250 O'Hara Ave**

Depth, feet	MATERIAL DESCRIPTION	Sample Number	Penetration Blows / 6 inches	Pocket Penetrometer, TSF	% Passing No. 200 Sieve	In-Situ Dry Weight (pcf)	In-Situ Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
0	**ASPHALT:** approximately 3 inches of asphalt concrete (AC)									
	SILTY SAND (SM): yellowish brown, slightly moist, medium dense, fine grained sand									
		1A 1B 1C	7 9 11			111	4			
5										
	loose	2A 2B 2C	4 4 5		12					
		3A 3B 3C	3 5 7							
10	brown, increased silt content with depth									
	SILTY LEAN CLAY (CL): brown, slightly moist, hard, low plasticity, increase silt content with depth									
15		4A 4B 4C	12 17 29	>4.5						
	low to medium plasticity	5A 5B 5C	7 13 17	3.5						
20	Boring terminated at approximately 20 feet. No free groundwater observed. Boring backfilled with cement grout and patched the approximately upper 6 inches with Quikrete.									
25										

Completion Depth: 20.0
Date Started: 2/2/17
Date Completed: 2/2/17
California Sampler: 2.5-inch inner diameter
SPT Sampler:

Drilling Equipment: Exploration GeoServices Mobile Blue B-53
Drilling Method: Hollow Stem
Drive Weight: 140 lbs
Hole Diameter: 8-inches
Drop: 30-inches
Remarks:

BSK Associates
324 Earhart Way
Livermore, CA 94551
Telephone: (925)-315-3151
Fax: (925)-315-3152

LOG OF BORING NO. B-5

Project Name: **Oakley Recreation Center**
Project Number: **G16-221-11L**
Project Location: **1250 O'Hara Avenue Oakley, CA**
Logged by: **M. McNally**
Checked by: **D. Tower**

Surface El.: **43 ft**
Location: **1250 O'Hara Ave**

Depth, feet	MATERIAL DESCRIPTION	Sample Number	Penetration Blows / 6 inches	Pocket Penetrometer, TSF	% Passing No. 200 Sieve	In-Situ Dry Weight (pcf)	In-Situ Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
	ASPHALT: approximately 3 inches of asphalt concrete (AC)									
	POORLY GRADED SAND (SP): reddish brown, moist, medium dense, fine grained sand, trace silt									
		1A 1B 1C	9 11 11							
5		2A 2B 2C	4 5 5							
	loose									
	slightly moist, loose to medium dense	3A 3B 3C	4 6 9		4					
10										
	SILTY LEAN CLAY (CL): olive yellow, moist, hard, low to medium plasticity, manganese oxide staining									
		4A 4B 4C	6 12 17	>4.5						
15	silt seam									
	medium plasticity	5A 5B 5C	7 12 15	>4.5						
20										
	SILTY SAND (SM): yellowish brown, moist, medium dense, fine grained sand									
		6A 6B 6C	12 13 18							
25										
	POORLY GRADED SAND WITH SILT (SP-SM): brown, wet, medium dense, fine grained sand									

Completion Depth: 50.0
Date Started: 2/2/17
Date Completed: 2/2/17
California Sampler: 2.5-inch inner diameter
SPT Sampler:

Drilling Equipment: Exploration GeoServices Mobile Blue B-53
Drilling Method: Hollow Stem
Drive Weight: 140 lbs
Hole Diameter: 8-inches
Drop: 30-inches
Remarks:

BSK Associates
324 Earhart Way
Livermore, CA 94551
Telephone: (925)-315-3151
Fax: (925)-315-3152

LOG OF BORING NO. B-5

Project Name: **Oakley Recreation Center**
Project Number: **G16-221-11L**
Project Location: **1250 O'Hara Avenue Oakley, CA**
Logged by: **M. McNally**
Checked by: **D. Tower**

Surface El.: **43 ft**
Location: **1250 O'Hara Ave**

Depth, feet	MATERIAL DESCRIPTION	Sample Number	Penetration Blows / 6 inches	Pocket Penetrometer, TSF	% Passing No. 200 Sieve	In-Situ Dry Weight (pcf)	In-Situ Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
	POORLY GRADED SAND WITH SILT (SP-SM): brown, wet, medium dense, fine grained sand *(continued)*									
	▽ 10:55 am, 2/2/17	7A 7B 7C	6 9 10		5	101	28			
30										
	SANDY SILT (ML): brown, wet, firm, low plasticity, manganese oxide staining, fine grained sand	8A 8B 8C	12 16 16							
35	sandy lean clay seam									
	POORLY GRADED SAND WITH SILT (SP-SM): brown, wet, medium dense, fine grained sand	9A 9B 9C	6 11 15		5	104	27			
40										
		10A 10B	12 17							
45	**LEAN CLAY (CL):** olive yellow, wet, hard, medium platicity, fine grained sand	10C	20	3.5						
	SILTY SAND (SM): brown, wet, medium dense, fine grained sand	11A	12							
	SILTY LEAN CLAY (CL): olive yellow, moist, firm to hard, medium plasticity	11B 11C	16 17	2.5-3.5						
50	Boring terminated at approximtaely 50 feet. Free groundwater observed at approximately 28.5 feet; boring immediately flooded with water. Boring backfilled with cement grout and patched the upper approximately 6 inches with Quikrete.									

Completion Depth: 50.0
Date Started: 2/2/17
Date Completed: 2/2/17
California Sampler: 2.5-inch inner diameter
SPT Sampler:

Drilling Equipment: Exploration GeoServices Mobile Blue B-53
Drilling Method: Hollow Stem
Drive Weight: 140 lbs
Hole Diameter: 8-inches
Drop: 30-inches
Remarks:

LOG OF BORING NO. B-6

BSK Associates
324 Earhart Way
Livermore, CA 94551
Telephone: (925)-315-3151
Fax: (925)-315-3152

Project Name:	**Oakley Recreation Center**
Project Number:	**G16-221-11L**
Project Location:	**1250 O'Hara Avenue Oakley, CA**
Logged by:	**M. McNally**
Checked by:	**D. Tower**

Surface El.: **43 ft**

Location: **1250 O'Hara Ave**

Depth, feet	Graphic Log	MATERIAL DESCRIPTION	Samples	Sample Number	Penetration Blows / 6 inches	Pocket Penetrometer, TSF	% Passing No. 200 Sieve	In-Situ Dry Weight (pcf)	In-Situ Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
		POORLY GRADED SAND WITH SILT (SP-SM): yellowish brown, slightly moist, fine grained sand										
		loose		1A 1B 1C	4 4 6		7					
5		fine to medium grained sand, decreased silt content with depth		2A 2B 2C	3 5 7							
10				3A 3B 3C	4 5 7			106	8			
		increased silt content		4A	5							
		SILTY LEAN CLAY (CL): yellowish brown, slightly moist, hard, low plasticity, manganese oxide staining, slightly porous		4B 4C	12 17	>4.5						
15												
		fine grained sand present		5A 5B 5C	4 8 22	>4.5 >4.5						
20		Boring terminated at approximately 20 feet. No free groundwater observed. Boring backfilled with cement grout.										
25												

Completion Depth:	20.0	**Drilling Equipment:**	Exploration GeoServices Mobile Blue B-53
Date Started:	2/2/17	**Drilling Method:**	Hollow Stem
Date Completed:	2/2/17	**Drive Weight:**	140 lbs
California Sampler:	2.5-inch inner diameter	**Hole Diameter:**	8-inches
SPT Sampler:		**Drop:**	30-inches
		Remarks:	

APPENDIX C

LABORATORY TEST RESULTS

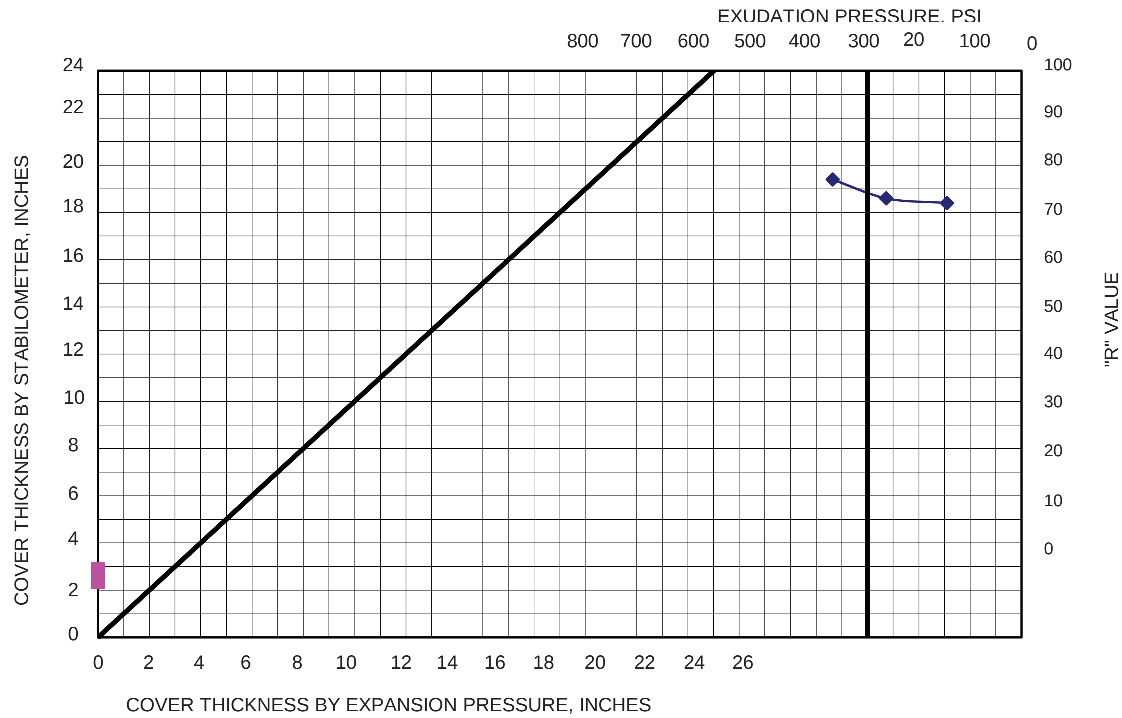
R-Value Test

324 Earhart Way
Livermore, CA 94551
Ph: (925) 315-3151
Fax: (925) 315-3152

Caltrans Test Method 301

Project Name: Oakley Recreation Center
Project Number: G16-221-11L
Sample Source: 0
Lab Tracking ID: 16-883
Sample Location: B1@1-5'

Sample Date: 11/17/2016
Sample By: J. Wu (Fremont)
Test Date: 11/21/2016
Report Date: 11/22/2016
Tested By: RC



Sample Description: Brown, Poorly Graded Sand with Silt and Gravel (SP-SM)

SPECIMEN	A	B	C
EXUDATION PRESSURE, LOAD (lb)	4620	3314	1824
EXUDATION PRESSURE, PSI	368	264	145
EXPANSION, * 0.0001 IN	0.0001	0.0003	0.0002
EXPANSION PRESSURE, PSF	0	0	0
STABILOMETER PH AT 2000 LBS	30	33	35
DISPLACEMENT	3.19	3.47	3.5
RESISTANCE VALUE "R"	77	73	72
"R" VALUE CORRECTED FOR HEIGHT	77	73	72
% MOISTURE AT TEST	11.0	12.3	12.8
DRY DENSITY AT TEST, PCF	118.3	117.0	116.4
"R" VALUE AT 300 PSI EXUDATION PRESSURE	**83**		
"R" VALUE BY EXPANSION PRESSURE TI = 4.0, GF=1.50	**N/A**		

BSK ASSOCIATES

PROJECT NO. G16-221-11L
DRAWN: 12/19/16
DRAWN BY: M. McNally
CHECKED BY: C. Melo
FILE NAME: R-Value.indd

RESISTANCE (R)-VALUE

Phase 1 of Oakley Recreation Center
1250 O'Hara Avenue
Oakley, California

PLATE
C-1

California State Certified Laboratory No. 2153

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 462 2771 Fax. 925 462 2775
www.cercoanalytical.com

2 December, 2016

Job No. 1611185
Cust. No. 12667

Ms. Maggie McNally
BSK Associates Engineers & Laboratories
324 Earhart Way
Livermore, CA 94551

Subject: Project No.: G16-221-11L
Project Name: Oakley Rec.
Corrosivity Analysis – ASTM Test Methods

Dear Ms. McNally:

Pursuant to your request, CERCO Analytical has analyzed the soil sample submitted on November 22, 2016. Based on the analytical results, this brief corrosivity evaluation is enclosed for your consideration.

Based upon the 100% saturation resistivity measurement, this sample is classified as "moderately corrosive". All buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron should be properly protected against corrosion depending upon the critical nature of the structure. All buried metallic pressure piping such as ductile iron firewater pipelines should be protected against corrosion.

The chloride ion concentration reflects none detected with a reporting limit of 15 mg/kg.

The sulfate ion concentration is 65 mg/kg and is determined to be insufficient to damage reinforced concrete structures and cement mortar-coated steel at this location.

The pH of the soil is 7.84, which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

The redox potential is 470-mV, which is indicative of aerobic soil conditions.

This corrosivity evaluation is based on general corrosion engineering standards and is non-specific in nature. For specific long-term corrosion control design recommendations or consultation, please call *JDH Corrosion Consultants, Inc. at (925) 927-6630.*

We appreciate the opportunity of working with you on this project. If you have any questions, or if you require further information, please do not hesitate to contact us.

Very truly yours,
CERCO ANALYTICAL, INC.

Cheryl McNeil for

J. Darby Howard, Jr., P.E.
President

JDH/jdl
Enclosure

California State Certified Laboratory No. 2153

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 462 2771 Fax. 925 462 2775
www.cercoanalytical.com

Client:	BSK Associates Engineers & Laboratories
Client's Project No.:	G16-221-11L
Client's Project Name:	Oakley Rec.
Date Sampled:	17-Nov-16
Date Received:	22-Nov-16
Matrix:	Soil
Authorization:	Signed Chain of Custody

Date of Report: 2-Dec-2016

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Conductivity (umhos/cm)*	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
1611185-001	B1 @ 1' - 2'	470	7.84	-	4,200	-	N.D.	65

Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Reporting Limit:	-	-	10	-	50	15	15
	30-Nov-2016	30-Nov-2016	-	2-Dec-2016	-	30-Nov-2016	30-Nov-2016

* Results Reported on "As Received" Basis

N.D. - None Detected

Cheryl McMillen
Laboratory Director

Quality Control Summary - All laboratory quality control parameters were found to be within established limits

APPENDIX D

EXHIBIT 1 – SUMMARY OF COMPACTION RECOMMENDATIONS

EXHIBIT 1

SUMMARY OF COMPACTION RECOMMENDATIONS

Area	Compaction Recommendations (See Notes 1, 2, 3, 4, 7)
Subgrade Preparation and Placement of General Engineered Fill[5], Including Imported Fill	Compact upper 12 inches of subgrade and entire fill to a minimum of 90 percent compaction at near optimum moisture content.
Foundation Excavation Bottoms	Compact to a minimum of 90 percent compaction at near optimum moisture content.
Trenches[6]	Compact trench backfill to a minimum of 90 percent compaction at near optimum moisture content for granular soils. Proper granular bedding and shading should be used beneath and around new utilities. Where trenches will be under flatwork or paving, the upper 12 inches of the trench backfill should be compacted as recommended below.
Exterior Flatwork	Compact upper 12 inches of subgrade to a minimum of 90 percent compaction at near optimum moisture content. Where exterior flatwork is exposed to vehicular traffic, compact aggregate base and upper 12 inches of subgrade to the pavement requirements below.
Pavements	Compact upper 12 inches of subgrade and aggregate base to a minimum of 95 percent compaction near optimum moisture content.

Notes:

(1) Depths are below finished subgrade elevation.

(2) All compaction requirements refer to relative compaction as a percentage of the laboratory standard described by ASTM D 1557.

(3) Fill material should be compacted in lifts not exceeding 8 inches in loose thickness.

(4) All subgrades should be firm and stable.

(5) Including building pads and backfill.

(6) In landscaping areas only, the percent compaction in trenches may be reduced to 85 percent.

(7) Where fills are greater than 7 feet in depth below finish grade, the zone below a depth of 7 feet should be compacted to a minimum of 95 percent compaction.