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August 9, 2017

ACKNOWLEDGEMENT STATEMENT

3884 - 3990 Depot Road, Hayward

Site Cleanup Program (SCP) Case RO0002499

In accordance with the following Alameda County Department of Environmental Health (ACDEH):

All work plans, technical reports, or technical documents submitted to ACDEH must be accompanied by a cover letter from the responsible party that states, at a minimum, the following:

I have read and acknowledge the content, recommendations and/or conclusions contained in the attached document or report submitted on my behalf to ACDEH's FTP server and the SWRCB's GeoTracker website.

Sincerely,

KEVIN OLIVERO, CEO

Bay Area Concrete LLC (WMBE)

Stevens Ferrone & Bailey
Engineering Company, Inc

Geotechnical Engineering
Engineering Geology
Storm Water Management
Construction Observation & Testing Services

May 16, 2014

Bay Area Concrete Recycling
24701 Clawiter Road
Hayward, CA 94545

Re: Geotechnical Investigation
Concrete Recycling Facility at 3898 Depot Road, Hayward, CA
SFB Project No.: 635-1

As requested, Stevens, Ferrone & Bailey Engineering Company, Inc. has performed a geotechnical investigation for the proposed concrete recycling facility at 3898 Depot Road in Hayward, California. The accompanying report presents the results of our field investigation, laboratory tests, and engineering analysis. The geotechnical conditions are discussed, and recommendations for the geotechnical engineering aspects of the project are presented. Conclusions and recommendations contained herein are based upon applicable standards of our profession at the time this report has been prepared. Should you have any questions or require additional information, please do not hesitate to contact me.

Sincerely,

Stevens, Ferrone & Bailey
Engineering Company, Inc.

Ken Ferrone
President

TC/KCF:lc\encl.
Copies: Addressee (1 by email)
Mr. Ken Alcock (Milani & Associates, 1 by email)

Geotechnical Engineering
Engineering Geology
Storm Water Management
Construction Observation & Testing Services

May 16, 2014

GEOTECHNICAL INVESTIGATION
CONCRETE RECYCLING FACILITY
3898 DEPOT ROAD
HAYWARD, CALIFORNIA
SFB PROJECT NO. 635-1

Prepared For:

Bay Area Concrete Recycling
24701 Clawiter Rd
Hayward, CA94545

Prepared By:

Stevens, Ferrone & Bailey Engineering Company, Inc.

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1.0 INTRODUCTION

This report presents the results of our geotechnical investigation for the proposed concrete recycling facility at 3898 Depot Road in Hayward, California as shown on the Site Plan, Figure 1. The purpose of our investigation was to evaluate the geotechnical conditions at the site and provide recommendations regarding the geotechnical engineering aspects of the project.

Based on the information indicated on the Site Plan, as well as information provided by Mr. Ken Alcock of Milani & Associates, it is our understanding that the project will consist of developing approximately 2.7 acres for a concrete recycling facility. The facility will include a raw concrete lay down area, crusher with conveyor belts, crushed rock area, truck unloading and loading access ways and scales, a maintenance and fueling area, C3 treatment area, receiving station, and parking. A storm drain system will also be installed. Nominal grading is anticipated.

The conclusions and recommendations provided in this report are based upon the information presented above; Stevens, Ferrone & Bailey Engineering Company, Inc. (SFB) should be consulted if any changes to the project occur to assess if the changes affect the validity of this report.

2.0 SCOPE OF WORK

This investigation included the following scope of work:

- Reviewing published and unpublished geotechnical and geological literature relevant to the site;
- Performing reconnaissance of the site and surrounding area;
- Performing a subsurface exploration program, including drilling four exploratory borings to a maximum depth of about 31-1/2 feet;
- Performing laboratory testing of samples retrieved from the borings;
- Performing engineering analysis of the field and laboratory data; and
- Preparing this report.

The data obtained and the analyses performed were for the purpose of providing geotechnical design and construction criteria for site earthwork, installation of underground utilities, drainage, foundations for structures, and pavements. Chemical concentration assessments of onsite materials or groundwater (including mold) was beyond our scope of work. Evaluating the potential for flooding was also beyond our scope of work.

3.0 SITE INVESTIGATION

Reconnaissance of the site and surrounding area was performed on May 5, 2014. Subsurface exploration was performed using a truck-mounted drill rig equipped with 4-inch diameter, continuous flight, solid stem augers. Four exploratory borings were drilled on May 5, 2014 to a maximum depth of about 31-1/2 feet. The approximate locations of the borings are shown on the Site Plan, Figure 1. Logs of SFB's borings and details regarding SFB's field investigation are included in Appendix A. The results of SFB's laboratory tests are discussed in Appendix B. It should be noted that changes in the surface and subsurface conditions can occur over time as a result of either natural processes or human activity and may affect the validity of the conclusions and recommendations in this report.

3.1 Surface

At the time of our investigation and as shown on Figure 1, the site was bounded by Depot Road on the north, auto dismantler yards on the east and northwest, a stormwater detention pond of the Russell City Energy Center power plant on the south, and a drainage channel on the west. The site was irregular in shape, generally level, and had a plan area of about 2.7 acres with maximum dimensions of about 530 feet by 30 feet. Most of the site was covered with gravel and used for truck parking. A small asphalt concrete paved parking lot was also located to the north end of the site. Large diameter trees were located to the south of the parking lot.

Based on our review of available historical aerial photographs of the site and vicinity, it is our understanding that the site was previously occupied by an auto dismantler yard that was probably removed in 2013.

3.2 Subsurface

The near-surface soil materials encountered at the site generally consisted of gravels and clayey or sandy fills that extended to depths of about 3 to 4 feet deep. These fills were heterogeneous, and potentially weak and compressible if they were not placed and compacted in accordance with acceptable engineering standards. Boring SFB-4 also encountered approximately 2 inches thick of asphalt concrete at surface. Below the surface fills, stiff, over-consolidated, native clays, locally known as Bay Mud, were encountered that extended to depths of about 8 feet. Underlying the Bay Mud layer, stiff to very stiff alluvial clays were encountered that extended to the maximum depth explored of about 31-1/2 feet.

The surface gravel fills are generally non-expansive. The underlying more clayey surface fills and soils have a high to very high plasticity and high to critical expansion potential. Detailed descriptions of the materials encountered in our exploratory borings are presented on the boring logs in Appendix A. Our attached boring logs and related information depict location specific subsurface conditions encountered during our field investigation. The approximate locations of our borings were determined using pacing or landmark references and should be considered accurate only to the degree implied by the method used.

3.3 Groundwater

Groundwater was encountered in our borings at depths of about 7 to 9 feet during exploration. SFB's borings were backfilled with lean cement grout in accordance with Alameda County Public Work Agency requirements prior to leaving the site. Historically, ground water in the vicinity of the site has been measured at a depth less than 5 feet[1]. It should be noted that our borings might not have been left open for a sufficient period of time to establish equilibrium ground water conditions. In addition, fluctuations in the ground water level could occur due to change in seasons, variations in rainfall, and other factors.

3.4 Geology and Seismicity

According to Helley and Graymer (1997), the site (below surficial fills) is underlain by Holocene basin deposits that are described as very fine silty clay to clay deposits occupying flat-floored basins at the distal edge of alluvial fans adjacent to the Bay Mud [2].

The project site is located in the San Francisco Bay Area that is considered one of the most seismically active regions in the United States. Significant earthquakes have occurred in the San Francisco Bay Area and are believed to be associated with crustal movements along a system of sub-parallel fault zones that generally trend in a northwesterly direction. The approximate direction and distance from the site to nearby active faults are summarized in the table below[3].

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[1]State of California, 2003, *Seismic Hazard Zone Report of the San Leandro 7.5-Minute Quadrangle, Alameda County, California*, CGS Seismic Hazard Zone Report 078.

[2]Helley & Graymer, 1997, *Quaternary Geology of Alameda County, and Parts of Contra Costa, Santa Clara, San Mateo, San Francisco, Stanislaus, and San Joaquin Counties, California: A Digital Database,* USGS Open File Report 97-97.

[3]Information based on Jennings and Bryant, 2010, *Fault Activity Map of California*, CGS Geological Data Map No.6.

Fault Name	**Approximate Distance to Fault (Miles)**	**Direction to Fault**
Hayward	3.6	Northeast
Calaveras	11.8	Northeast
Pleasanton	13.6	Northeast
San Andreas	14.9	Southwest
Serra	15.6	West
Crosley	17.0	Southeast
Hayward – SE Extension	17.0	Southeast
Monte Vista	18.6	South
Concord	20.0	Northeast
Seal Cove	22.2	Southwest
Marsh Creek	22.4	Northeast
Clayton	22.8	Northeast
Greenville	23.3	East
Carnegie	29.4	East
San Gregorio	30.0	Southwest

According to the Alquist-Priolo Earthquake Fault Zones Map of the San Leandro Quadrangle, the site is not located in an earthquake fault zone as designated by the State of California[4].

Earthquake intensities will vary throughout the San Francisco Bay Area, depending upon numerous factors including the magnitude of earthquake, the distance of the site from the causative fault, and the type of materials underlying the site. The U.S. Geological Survey (2008) indicated that there is a 63 percent chance of at least one magnitude 6.7 or greater earthquake

[4]Hart and Bryant, *Fault-Rupture Hazard Zones in California*, CDMG Special Publication 42, Interim Revision 2007.

striking the San Francisco Bay region between 2008 and 2037[5]. Therefore, the site will probably be subjected to at least one moderate to severe earthquake that will cause strong ground shaking.

According to the Probabilistic Seismic Hazard Analysis (NSHMP PSHA) interactive deaggregation model developed by U.S. Geological Survey (2008), the site has a 10% probability of exceeding a peak ground acceleration of about 0.5g in 50 years (design basis ground motion based on stiff soil site condition; mean return time of 475 years). The actual ground surface acceleration might vary depending upon the local seismic characteristics of the underlying bedrock and the overlying unconsolidated soils.

3.5 Liquefaction & Lateral Spreading

Soil liquefaction is a phenomenon primarily associated with saturated, cohesionless, soil layers located close to the ground surface. These soils lose strength during cyclic loading, such as imposed by earthquakes. During the loss of strength, the soil acquires mobility sufficient to permit both horizontal and vertical movements. Soils that are most susceptible to liquefaction are clean, loose, uniformly graded, saturated, fine-grained sands that lie close to the ground surface. According to ABAG and the U.S. Geological Survey, the site is located in an area that has been characterized as having moderate liquefaction susceptibility[6,7]. According to the Seismic Hazard Zones Map of the San Leandro Quadrangle, the site is located in a seismic hazard zone due to liquefaction as designated by the State of California[8]. The site, however, is located in an area having an absence of liquefaction-related features observed following historical earthquakes according to CGS Seismic Hazard Zone Report 078.

Based on our review of available literature and the results of exploratory borings at the site, it is our opinion that the potential for ground surface damage at the proposed site development resulting from liquefaction is low.

As part of our analyses, we evaluated the potential for lateral spreading impacting the site development. Lateral spreading occurs when soils liquefy during an earthquake event and the liquefied soils with the overlying soils move laterally to unconfined spaces (for example, the drainage channel banks), which causes significant horizontal ground displacements. It is our

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[5]Field, Edward H., Milner, Kevin 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.

[6]Witter, Knudsen, Sowers, Wentworth, Koehler, and Randolph, 2006, *Maps of Quaternary Deposits and Liquefaction Susceptibility in the Central San Francisco Bay Region, California*", USGS Open File Report 2006-1037.

[7]Knudsen, Sowers, Witter, Wentworth, and Helly, 2000, "*Preliminary Maps of Quaternary Deposits and Liquefaction Susceptibility, Nine-County San Francisco Bay Region, California*", USGS Open File Report 00-444.

[8]State of California, Seismic Hazard Zones, San Jose East Quadrangle, Official Map, Released: January 17, 2001.

opinion that the potential for lateral spreading adjacent to the drainage channel adversely impacting the site development is low due to the low liquefaction potential at the site.

4.0 CONCLUSIONS AND RECOMMENDATIONS

It is our opinion that the site is suitable for the proposed project from a geotechnical engineering standpoint. The conclusions and recommendations presented in this report should be incorporated in the design and construction of the project to reduce soil or foundation related issues. The following are the primary geotechnical considerations for development of the site.

EXISTING FILL MATERIALS: As described previously, fills blanket the entire site and extend to depths of about 3 to 4 feet. These fills are heterogeneous, potentially weak, and compressible. If grading records, including compaction test results, are not available to review, then there is a potential that the fills were not place in accordance with current geotechnical engineering standards. In order to reduce the potential for damaging differential settlement of overlying improvements (such as permanent foundations, paved driveways/pavements, and exterior flatwork), we recommend over-excavation and re-compaction be performed to provide an at least 3 foot thick engineered fill layer below proposed foundations, paved driveways/pavements, and exterior flatwork. It is our opinion that over-excavation is not necessary at other areas (such as open gravel lots, material stockpile areas, and C3 treatment areas) where periodic maintenance and re-leveling can be performed and there is less concern regarding ground settlement.

The over-excavation process can consist of removing the upper 2 feet of fills, scarifying and re-compacting the bottom 12 inches, and placing well-blended, compacted engineered fill over the properly prepared subgrade. The over-excavation and re-compaction should also extend at least 5 feet beyond building footprints and at least 3 feet beyond paved driveways/pavements and exterior flatwork wherever possible. Where the over-excavation limits abut adjacent property, SFB should be consulted to determine the actual vertical and lateral extent of over-excavation so that adjacent property is not adversely impacted. Over-excavations should be performed so that no more than 5 feet of differential fill thickness exists below proposed foundations. The removed fill materials can be used as new fill provided they are placed and compacted in accordance with the recommendations presented in this report. The extent of the removal and re-compaction will vary across the site and should be determined in the field by SFB at the time of the earthwork operations.

MATERIAL STOCKPILE STABILITY: In order to maintain the general site stability against slope stability and ground failures, we recommend material stockpiles at the site be setback at least 10 feet from the site boundary and the height of the stockpiles not exceed 25 feet in height. The onsite soils below stockpiles may experience consolidation under the stockpile loads. We estimate up to about 4 inches of consolidation settlement may occur under 25 feet of material stockpile loads at the site. The actual magnitude of settlement may be more or less than what we estimated. SFB should be consulted if higher stockpiles are desired at the site.

SHALLOW GROUNDWATER: Groundwater was encountered in SFB's borings at depths of about 7 to 9 feet below the existing ground surface. Dewatering of excavations in the shallow groundwater areas will be needed where excavations extend below the groundwater level, such as during underground utility installations. Installing shoring and/or temporary dewatering wells may also be necessary to aid in the stabilization of underground trench walls.

ADDITIONAL RECOMMENDATIONS: Detailed drainage, earthwork, foundation, and pavement recommendations for use in design and construction of the project are presented below. We recommend SFB review the design and specifications to verify that the recommendations presented in this report have been properly interpreted and implemented in the design, plans, and specifications. We also recommend SFB be retained to provide consulting services and to perform construction observation and testing services during the construction phase of the project to observe and test the implementation of our recommendations, and to provide supplemental or revised recommendations in the event conditions different than those described in this report are encountered. We assume no responsibility for misinterpretation of our recommendations if we do not review the plans and specifications and are not retained during construction.

4.1 Earthwork

4.1.1 Clearing and Site Preparation

The site should be cleared of all obstructions including any utilities and pipelines and their associated backfill, the existing parking lot, designated trees and their associated entire root systems, and debris. Holes resulting from the removal of underground obstructions extending below the proposed finish grade should be cleared and backfilled with fill materials as specified in **Section 4.1.4, *Fill Material***, and compacted to the requirements in **Section 4.1.5, *Compaction***. Tree roots may extend to depths of about 3 to 4 feet. Wells and septic systems, if they exist onsite, should be abandoned in accordance with Alameda County standards.

From a geotechnical standpoint, any existing trench backfill materials, pavements, or concrete that are removed can be used as new fill onsite provided debris is removed and it is broken up to meet the size requirement for fill material in **Section 4.1.4, *Fill Material***. Consideration should be given to placing these materials below pavements, directly under building footprints, or in deeper excavations. We recommend backfilling operations for any excavations be performed under the observation and testing of SFB.

4.1.2 Existing Fill Re-Compaction

As described previously, fills blanket the entire site and extend to depths of about 3 to 4 feet below the existing ground surface. In order to reduce the potential for damaging differential settlement of overlying improvements (such as permanent foundations, paved driveways/pavements, and exterior flatwork), we recommend over-excavation and re-compaction be performed at the site to provide at least 3 feet of engineered fill below proposed permanent foundations, paved driveways/pavements, and exterior flatwork. It is our opinion that over-excavation is not necessary at other areas (such as open gravel lots, material stockpile areas, and C3 treatment area,) where periodic maintenance and re-leveling can be performed and there is less concern regarding ground settlement. The over-excavation process can consist of removing the upper 2 feet of fills, scarifying and re-compacting the bottom 12 inches, and placing well-blended, compacted engineered fill over the properly prepared subgrade. The over-excavation and re-compaction should also extend at least 5 feet beyond foundation footprints and at least 3 feet beyond driveways/pavements and exterior flatwork wherever possible. Where the over-excavation limits abut adjacent property, SFB should be consulted to determine the actual vertical and lateral extent of over-excavation so that adjacent property is not adversely impacted. Over-excavations should be performed so that no more than 5 feet of differential fill thickness exists below the proposed building foundations. The extent of the removal and re-compaction will vary across the site and should be determined in the field by SFB at the time of the earthwork operations.

Removed existing fill materials may be used as new fill onsite provided they satisfy the recommendations provided in **Section 4.1.4, *Fill Material***. Compaction should be performed in accordance with the recommendations in **Section 4.1.5, *Compaction***.

4.1.3 Subgrade Preparation

If foundation pads or pavement subgrade are allowed to remain exposed to sun, wind or rain for an extended period of time, or are disturbed by animals, equipment, or vehicles, the exposed pads or pavement subgrade may need to be reconditioned (moisture conditioned and/or scarified and re-compacted) prior to foundation or pavement construction. SFB should be consulted on the need for subgrade reconditioning when the subgrade is left exposed for extended periods of time.

The soil exposed in areas to receive improvements (such as building foundations, paved driveways/pavements, and exterior flatwork) should be scarified to a depth of about 12 inches, moisture conditioned to approximately 3 percent over optimum water content, and compacted to the requirements for structural fill.

4.1.4 Fill Material

From a geotechnical and mechanical standpoint, onsite fills and soils having an organic content of less than 3 percent by volume can be used as fill. Fill should not contain rocks or lumps larger than 6 inches in greatest dimension with not more than 15 percent larger than 2.5 inches. Larger sized rock may be used as fill onsite provided it is closely monitored, placed properly to achieve compaction, and are located at depths below anticipated, future excavations; SFB should be consulted regarding the use of larger rock pieces in fill materials. If required, imported fill should have a plasticity index of 20 or less and have a significant amount of cohesive fines.

In addition to the mechanical properties specifications, all imported fill material should have a resistivity (100% saturated) no less than the resistivity for the onsite soils, a pH of between approximately 6.0 and 8.5, a total water soluble chloride concentration less than 300 ppm, and a total water soluble sulfate concentration less than 500 ppm. We recommend import samples be submitted for corrosion and geotechnical testing at least two weeks prior to being brought onsite.

4.1.5 Compaction

We recommend structural fill be compacted to at least 90 percent relative compaction as determined by ASTM D1557 (latest edition). We recommend the new fill be moisture conditioned approximately 3 percent over optimum water content. The upper 6 inches of subgrade soils beneath pavements should be compacted to at least 95 percent relative compaction. Fill material should be spread and compacted in lifts not exceeding approximately 8 to 12 inches in uncompacted thickness.

4.1.6 Utility Trench Backfill

Pipeline trenches should be backfilled with fill placed in lifts of approximately 8 inches in uncompacted thickness. Thicker lifts can be used provided the method of compaction is approved by SFB and the required minimum degree of compaction is achieved. Backfill should be placed by mechanical means only. Jetting is not permitted.

Onsite trench backfill should be compacted to at least 90 percent relative compaction. Imported sand trench backfill should be compacted to at least 95 percent relative compaction and sufficient water is added during backfilling operations to prevent the soil from "bulking" during compaction. The upper 3 feet of trench backfill in foundation, slab, and pavement areas should be entirely compacted to at least 95 percent relative compaction. To reduce piping and settlement of overlying improvements, we recommend rock bedding and rock backfill (if used) be completely surrounded by a filter fabric such as Mirafi 140N (or equivalent); alternatively, filter fabric would not be necessary if Caltrans Class 2 permeable material is used in lieu of rock bedding and rock backfill.

Sand or gravel backfilled trench laterals that extend toward driveways, exterior slabs-on-grade, or under foundations, and are located below irrigated landscaped areas such as lawns or planting strips, should be plugged with onsite clays, low strength concrete, or sand/cement slurry. The plug for the trench lateral should be located below the edge of pavement or slabs, and under the perimeter of the foundation. The plug should be at least 24 inches thick, extend the entire width of the trench, and extend from the bottom of the trench to the top of the sand or gravel backfill.

4.1.7 Exterior Flatwork

We recommend that exterior slabs (including patios and sidewalks) be placed directly on the properly compacted fills. We do not recommend using aggregate base, gravel, or crushed rock below these improvements. If imported granular materials are placed below these elements, subsurface water can seep through the granular materials and cause the underlying soils to saturate or pipe. Prior to placing concrete, subgrade soils should be moisture conditioned to increase their moisture content to approximately 3 percent above laboratory optimum moisture (ASTM D-1557).

The more expansive clayey soils at the site could be subjected to volume changes during fluctuations in moisture content. As a result of these volume changes, some vertical movement of exterior slabs (such as driveways, sidewalks, patios, exterior flatwork, etc.) should be anticipated. This movement could result in damage to the exterior slabs and might require periodic maintenance or replacement. Adequate clearance should be provided between the exterior slabs and building elements that overhang these slabs, such as window sills or doors that open outward.

Consideration should be given to reinforcing exterior slabs with steel bars in lieu of wire mesh. To reduce potential crack formation, the installation of #4 bars spaced at approximately 18 inches on center in both directions should be considered. Score joints and expansion joints should be used to control cracking and allow for expansion and contraction of the concrete slabs. We recommend appropriate flexible, relatively impermeable fillers be used at all cold/expansion joints. The installation of dowels at all expansion and cold joints will reduce differential slab movements; if used, the dowels should be at least 30 inches long and should be spaced at a maximum lateral spacing of 18 inches. Although exterior slabs that are adequately reinforced will still crack, trip hazards requiring replacement of the slabs will be reduced if the slabs are properly reinforced.

4.1.8 Construction During Wet Weather Conditions

If construction proceeds during or shortly after wet weather conditions, the moisture content of the onsite soils could be significantly above optimum. Consequently, subgrade preparation, placement and/or reworking of onsite soil or fills as structural fill might not be possible. Alternative wet weather construction recommendations can be provided by our representative in the field at the time of construction, if appropriate. All the drainage measures recommended in this report should be implemented and maintained during and after construction, especially during wet weather conditions.

4.1.9 Surface Drainage, Irrigation, and Landscaping

Ponding of surface water must not be allowed on pavements, adjacent to foundations, at the top or bottom of slopes, and at the top or adjacent to retaining walls. Ponding of water should also not be allowed on the ground surface adjacent to or near exterior slabs, including driveways, walkways, and patios. Surface water should not be allowed to flow over the top of slopes, down slope faces, or over retaining walls.

If expansion and softening of subsurface soils is a concern, then we recommend bio-swales, porous pavement, and water detention basins be lined with a relatively impermeable membrane in order to reduce the potential for damage to the improvements. The relatively impermeable membrane should consist of STEGO Wrap 15-mil or equivalent and should direct collected water into subdrain pipes. The membrane should be lapped and sealed in accordance with the manufacture's specifications, including taping joints where pipes penetrate the membrane.

We recommend positive surface gradients of at least 2 percent be provided adjacent to foundations to direct surface water away from the foundations and toward suitable discharge facilities. We recommend the surface drainage be designed in accordance with the latest edition of the California Building Code.

In order to reduce differential movements, landscaping should be placed uniformly adjacent to the foundation and exterior slabs. We recommend trees be no closer to the structure or exterior slabs than half the mature height of the tree; in no case should tree roots be allowed to extend near or below the foundations or exterior slabs.

Landscaping drainage inlets and/or drainage swales must provided and maintained around the structures at all times that adequately collect irrigation and storm water and direct the water onto pavement or into storm water collection systems. Drainage inlets should be provided within enclosed planter areas and the collected water should be discharged onto pavement, into drainage swales, or into an enclosed storm drain system. The drainage inlets and associated swales should be designed and constructed so that the moisture content of the soils surrounding the foundations do not become elevated and no ponding of water occurs. The inlets should be kept free of debris and be lower in elevation than the adjacent ground surface.

We recommend regular maintenance of the drainage systems be performed, including maintenance prior to rainstorms. The inspection should include checking drainage patterns to make sure they are performing properly, making sure drainage systems and inlets are functional and not clogged, and checking that erosion control measures are adequate for anticipated storm events. Immediate repairs should be performed if any of these measures appears to be inadequate.

Irrigation should be performed in a uniform, systematic manner as equally as possible on all sides of the foundations and exterior slabs to maintain moist soil conditions. Over-watering must be avoided. To reduce moisture changes in the natural soils and fills in landscaped areas, we recommend that drought resistant plants and "drip" irrigation systems be used. Low flow watering systems should also be used. All irrigation systems should be inspected for leakage regularly.

4.1.10 Future Maintenance

In order to reduce water created issues, we recommend regular maintenance of the site be performed, including maintenance prior to rainstorms. Maintenance should include the re-compaction of loosened soils, collapsing and infilling holes with compacted soils or low strength sand/cement grout, removal and control of digging animals, modifying storm water drainage patterns to allow for sheet flow into drainage inlets or ditches rather than concentrated flow or ponding, removal of debris within drainage ditches and inlets, and immediately repairing any erosion or soil flow. The inspection should include checking drainage patterns, making sure drainage systems are functional and not clogged, and erosion control measures are adequate for anticipated storm events. Immediate repair should be performed if any of these measures appears to be inadequate. Temporary and permanent erosion and sediment control measures should be installed over any exposed soils immediately after repairs are made.

Differential movement of exterior slabs can occur over time as a result of numerous factors. We recommend the development owner perform inspections and maintenance of the slabs, including infilling significant cracks, providing fillers at slab offsets, and replacing slabs if severely damaged.

4.1.11 Additional Recommendations

We recommend the drainage, irrigation, landscaping, and maintenance recommendations provided in this report be forwarded to your designers and contractors, and we recommend they be included in disclosures to owners and future owners.

4.2 Foundation Support

4.2.1 Conventional Spread Footings

Where foundations are necessary to support structures, the structures can be supported on spreading footing foundations bearing in the properly prepared compacted structural fill. Recommendations for foundation pad preparation were described previously in **Section 4.1.2, *Existing Fill Re-Compaction*** and **Section 4.1.3, *Subgrade Preparation.*** Footings should be at least 12 inches wide and should be founded at least 24 inches below lowest adjacent finished grade. A continuous footing should be provided around the perimeter of proposed buildings. Continuous footings should be designed with steel reinforcing, both top and bottom, to provide structural continuity and permit spanning of local irregularities.

The footings should be designed using an allowable bearing pressure of 2,000 pounds per square foot due to dead loads, 3,000 pounds per square foot due to dead plus live loads, and 4,000 pounds per square foot for all loads, including wind or seismic. These allowable bearing pressures are net values; therefore, the weight of the footing can be neglected for design purposes.

Lateral load resistance can be developed by friction between the footing foundation bottom and the supporting subgrade. A friction coefficient of 0.35 is considered applicable. As an alternative, a passive resistance equal to an equivalent fluid weighing 350 pcf acting against the vertical face of the foundations can be used; however the upper 12 inches should be ignored in the passive resistance design. If foundations are poured neat against the subgrade, the friction and passive resistance can be used in combination.

At least 10 feet of soil cover must be provided between the face of the footings and the face of slopes, as measured horizontally. The portion of the footing located closer than 10 feet from the face of slopes should be ignored in both the vertical and lateral load design.

Where foundations are located adjacent to utility trenches, the foundation bearing surface should bear below an imaginary 1 horizontal to 1 vertical plane extending upward from the bottom edge of the adjacent utility trench. Alternatively, the foundation reinforcing could be increased to span the area defined above assuming no soil support is provided.

Wetting prior to construction of the foundations should close any visible cracks in the bottoms of the footing excavations. We recommend that we observe the footing excavations prior to placing reinforcing steel or concrete to check that footings are founded on appropriate material.

4.2.2 Interior Slabs-on-Grade

We recommend that interior slabs-on-grade (used in conjunction with footing foundations) be at least 5 inches thick and be supported on properly prepared compacted fills. The actual thickness of the slabs should be based upon the actual use and loading of the slabs. Vehicular garage slabs should also be underlain by at least 6 inches of Caltrans Class 2 aggregate base. All slabs should be reinforced with at least #4 bars on 18-inch centers, both ways; however, the actual reinforcing should be provided with the anticipated use and loading of the slab. In order to control concrete shrinkage cracking, the slabs should have deep score joints that are spaced at approximately 10-feet on center in both directions.

A vapor retarder must be placed between the subgrade and the bottom of new interior slabs-on-grade. We recommend the vapor retarder consist of a single layer of Stego Wrap Vapor Barrier 15 mil or equivalent provided the equivalent satisfies the following criteria: a permeance as tested before and after mandatory conditioning of less than 0.01 Perms and strength of Class A as determined by ASTM E 1745 (latest edition), and a thickness of at least 15 mils. Installation of the vapor retarder should conform to the latest edition of ASTM E 1643 (latest edition) and the manufacturers requirements, including all joints should be lapped at least 6 inches and sealed with Stego Tape or equal in accordance with the manufacturer's specifications. Protrusions where pipes or conduit penetrate the membranes should be sealed with either one or a combination of Stego Tape, Stego Mastic, Stego Pipe Boots, or a product of equal quality as determined by the manufacturer's instructions and ASTM E 1643. Care must be taken to protect the membrane from tears and punctures during construction.

We do not recommend placing sand or gravel over the membrane located below new interior slabs-on-grade. In addition, we recommend that 4 inches of ½ to ¾ inch drain rock be placed below the vapor retarder where interior slabs-on-grade are used, except where the slabs are underlain by the 6 inches of baserock. Prior to placement of the vapor retarder, the subgrade surfaces should be proof-rolled to provide a smooth, unyielding surface for slab support. The edges of the vapor retarder membrane should be draped over the interior side of the footing excavations and at least 12 inches below the pad grade prior to pouring the concrete. We recommend that the interior slabs-on-grade (other than the garage or vehicular slabs) be poured monolithically with the footings.

The edges of vehicular garage slabs should be structurally separated from surrounding foundations; a relatively impermeable and flexible filler such as Greenstreak Swellstop (3/8" x 3/4" size) or equivalent should be used in the joint between the garage or vehicular slabs and the footing foundation. If a garage door is used, both the driveway and garage or vehicular slabs should be connected to the perimeter footing below the garage door opening with dowels to reduce the potential for differential movements.

Concrete slabs retain moisture and often take many months to dry; construction water added during the concrete pour further increases the curing time. If the slabs are not allowed to completely cure prior to constructing the super-structure, the concrete slabs will expel water vapor and the vapor will be trapped under impermeable flooring. The concrete mix design for the slabs should have a maximum water/cement ratio of 0.45; the actual water/cement ratio may need to be reduced if the concentration of soluble sulfates or chlorides in the supporting subgrade is detrimental to the concrete. We recommend you consult with your concrete slab designers and concrete contractors regarding methods to reduce the potential for differential concrete curing.

4.2.3 Seismic Design Criteria

The following parameters were calculated using the U.S. Geological Survey Ground Motion Parameters computer program (Version 5.1.0) and U.S. Seismic Design Map program (Version 3.1.0)[9], and were based on the site being located at approximate latitude 37.637°N and longitude 122.136°W. For seismic design using the 2013 California Building Code (CBC), we recommend the following seismic design values be used.

2013 CBC SEISMIC PARAMETERS		
Seismic Parameter	Design Value	CBC Reference
Site Class	D	Section 1613.3.2
S_S	1.61	Figure 1613.3.1(1)
S_1	0.63	Figure 1613.3.1(2)
F_a	1.0	Table 1613.3.3(1)
F_v	1.5	Table 1613.3.3(2)

4.3 Pavements

If differing conditions will exist than those described below, SFB should be consulted to provide supplemental recommendations.

4.3.1 Asphalt Concrete

Due to the present of the abundant gravelly fills at site surface, we recommend that an R-value of 25 be used in preliminary asphalt concrete pavement design. We recommend additional R-value tests be performed once the pavement subgrade is established to confirm the R-value used in the design.

[9]USGS Website, *http://earthquake.usgs.gov/hazards/designmaps/usdesign.php*, Version 3.1.0, last updated 7/11/13.

We developed the following alternative preliminary pavement sections using Topic 608 of the State of California Department of Transportation Highway Design Manual, the recommended R-value, and typical traffic indices similar facilities. The pavement thicknesses shown below are SFB's recommended minimum values; governing agencies may require pavement thicknesses greater than those shown. Preliminary pavement sections should be revised, if necessary, when actual traffic indices are known and pavement subgrade elevations are determined.

PRELIMINARY PAVEMENT DESIGN ALTERNATIVES SUBGRADE R-VALUE = 25				
Location	Pavement Components			Total Thickness (inches)
	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)	Aggregate Subbase (inches)	
T.I. = 4.5 (auto & light truck parking)	2.5	6.0	-	8.5
	2.5	4.0	3.0	9.5
T.I. = 5.0 (auto & light truck access way)	3.0	8.0	-	11.0
	3.0	5.0	3.0	11.0
T.I. = 10.0 (heavy truck access way; up to about 100 trucks per day)	6.0	17.0	-	23.0
	6.0	10.0	8.0	24.0
T.I. = 12.0 (heavy truck access way; up to about 300 trucks per day)	7.5	20.0	-	27.5
	7.5	12.0	10.0	29.5

Pavement baserock and asphalt concrete should be compacted to at least 95 percent relative compaction. The asphalt concrete compacted unit weight should be determined using Caltrans Test Method 308-A or ASTM Test Method D1188. Asphalt concrete should also satisfy the S-value requirements by Caltrans. We recommend regular maintenance of the asphalt concrete be performed at approximately five year intervals. Maintenance may include sand slurry sealing, crack filling, and chip seals as necessary. If regular maintenance is not performed, the asphalt concrete layer could experience premature degradation requiring more extensive repairs.

4.3.2 Concrete Pavement

Using the design program (StreetPave) developed American Concrete Pavement Association (ACPA), the recommended subgrade R-value of 25, an assumed Traffic Index of 12.0 (up to about 300 trucks per day), and a design life of 20 years, we recommend the concrete pavement consist of at least 8 inches of concrete having a modulus of rupture of at least 600 psi overlying 12 inches of Caltrans Class 2 aggregate base. We recommend all joints (both transverse and longitudinal) not exceed 12-feet on center in both directions. We also recommend that dowels be used at transverse joints with #8 bars. The dowels should be at least 18 inches long and should be spaced at approximately 12 inches on center. Pavement subgrade should be prepared in accordance with our recommendations provided in our report in **Section 4.1.3, *Subgrade Preparation***. In addition, we recommend a geotextile such as Mirafi 500X or equivalent be placed between the aggregate base and the soil subgrade to reduce the effects of subgrade deterioration on the pavement section.

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5.0 CONDITIONS AND LIMITATIONS

SFB is not responsible for the validity or accuracy of information, analyses, test results, or designs provided to SFB by others or prepared by others. The analysis, designs, opinions, and recommendations submitted in this report are based in part upon the data obtained from our field work and upon information provided by others. Site exploration and testing characterizes subsurface conditions only at the locations where the explorations or tests are performed; actual subsurface conditions between explorations or tests may be different than those described in this report. Variations of subsurface conditions from those analyzed or characterized in this report are not uncommon and may become evident during construction. In addition, changes in the condition of the site can occur over time as a result of either natural processes (such as earthquakes, flooding, or changes in groundwater levels) or human activity (such as construction adjacent to the site, dumping of fill, or excavating). If changes to the site's surface or subsurface conditions occur since the performance of the field work described in this report, or if differing subsurface conditions are encountered, we should be contacted immediately to evaluate the differing conditions to assess if the opinions, conclusions, and recommendations provided in this report are still applicable or should be amended.

We recommend SFB be retained to provide geotechnical services during design, reviews, earthwork operations, paving operations, and foundation installation to confirm and observe compliance with the design concepts, specifications and recommendations presented in this report. Our presence will also allow us to modify design if unanticipated subsurface conditions are encountered or if changes to the scope of the project, as defined in this report, are made.

This report is a design document that has been prepared in accordance with generally accepted geological and geotechnical engineering practices for the exclusive use of Bay Area Concrete Recycling and their consultants for specific application to the proposed concrete recycling facility at 3898 Depot Road, California, and is intended to represent our design recommendations to Bay Area Concrete Recycling for specific application to the proposed concrete recycling facility project. The conclusions and recommendations contained in this report are solely professional opinions. It is the responsibility of Bay Area Concrete Recycling to transmit the information and recommendations of this report to those designing and constructing the project. We will not be responsible for the misinterpretation of the information provided in this report. We recommend SFB be retained to review geological and geotechnical aspects of the construction calculations, specifications, and plans; we should also be retained to participate in prebid and preconstruction conferences to clarify the opinions, conclusions, and recommendations contained in this report.

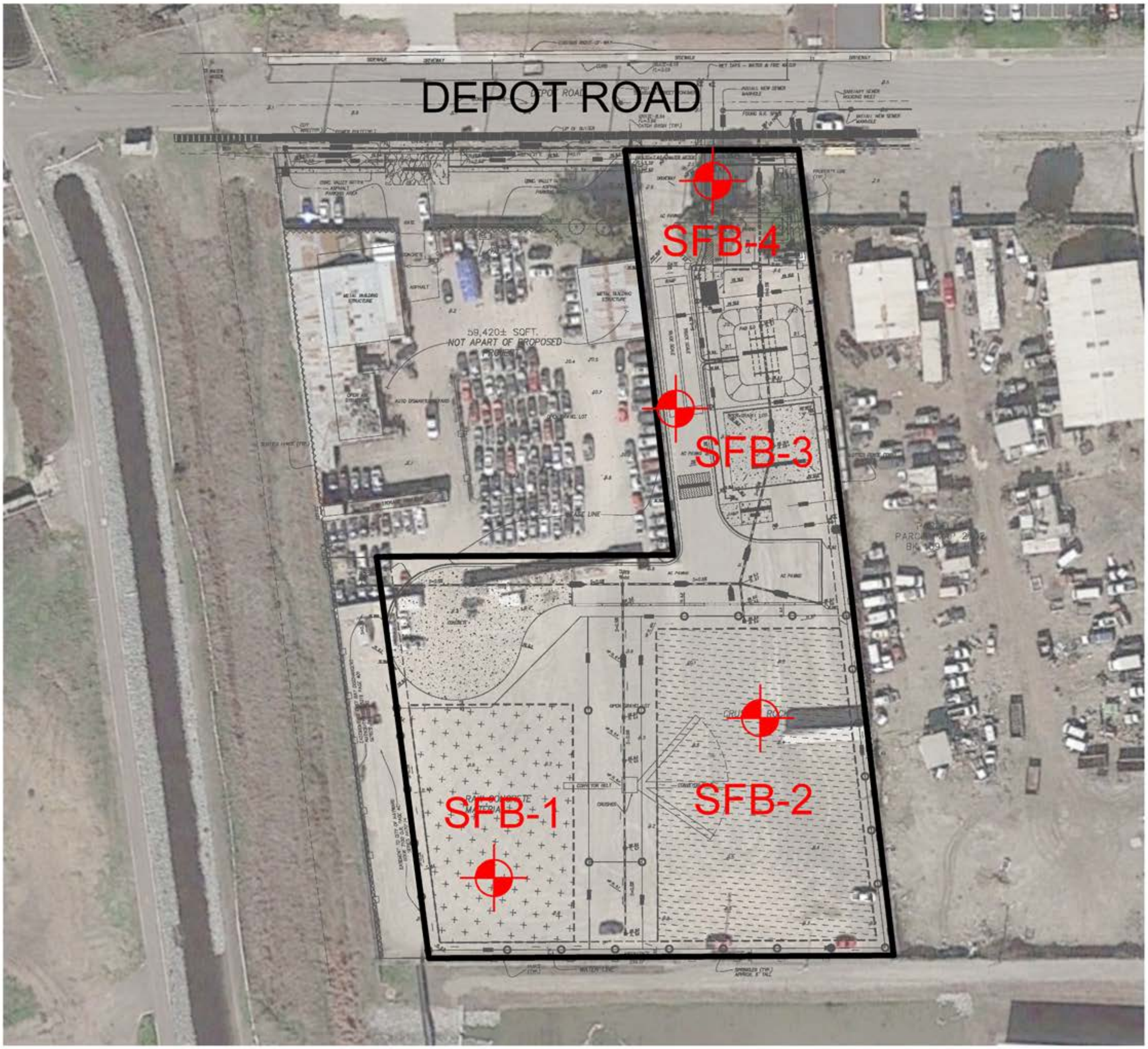
It should be understood that advancements in the practice of geotechnical engineering and engineering geology, or discovery of differing surface or subsurface conditions, may affect the validity of this report and are not uncommon. SFB strives to perform its services in a proper and professional manner with reasonable care and competence but we are not infallible. Geological engineering and geotechnical engineering are disciplines that are far less exact than other engineering disciplines; therefore we should be consulted if it is not completely understood what the limitations to using this report are.

In the event that there are any changes in the nature, design or location of the project, as described in this report, or if any future additions are planned, the conclusions and recommendations contained in this report shall not be considered valid unless we are contacted in writing, the project changes are reviewed by us, and the conclusions and recommendations presented in this report are modified or verified in writing. The opinions, conclusions, and recommendations contained in this report are based upon the description of the project as presented in the introduction section of this report.

This report does not necessarily represent all of the information that has been communicated by us to Bay Area Concrete Recycling and their consultants during the course of this engagement and our rendering of professional services to Bay Area Concrete Recycling. Reliance on this report by parties other than those described above must be at their own risk unless we are first consulted as to the parties' intended use of this report and only after we obtain the written consent of Bay Area Concrete Recycling to divulge information that may have been communicated to Bay Area Concrete Recycling. We cannot accept consequences for use of segregated portions of this report.

Please refer to Appendix C for additional guidelines regarding use of this report.

FIGURE



KEY

SFB-4 ⌖ APPROXIMATE LOCATION OF SFB EXPLORATORY BORING (5/5/14)

APPROXIMATE PROJECT LIMIT

N

APPROXIMATE SCALE: 1" = 100'

0 100' 200'

NOTE: Base map was created by overlying the project exhibit plan Sheets 2 & 3 prepared by Milani & Associates and dated February 2014 on Google Earth Image dated 2/23/14.

DATE	Stevens Ferrone & Bailey Engineering Company, Inc	1600 Willow Pass Court Concord, CA 94520 Tel 925.688.1001 Fax 925.688.1005 www.SFandB.com	SITE PLAN	FIGURE
May 2014			3898 DEPOT ROAD Hayward, California	1
PROJECT NO.				
635-1				

APPENDIX A
Field Investigation

APPENDIX A
Field Investigation

Our field investigation for the proposed concrete recycling facility at 3898 Depot Road in Hayward, California, consisted of surface reconnaissance and a subsurface exploration program. Geotechnical reconnaissance of the site and surrounding area was performed on May 5, 2014. Subsurface exploration was performed using a truck-mounted drill rig equipped with 4-inch diameter, continuous flight, solid stem augers. Four exploratory borings were drilled on May 5, 2014 to a maximum depth of about 31-1/2 feet. Our representative continuously logged the soils encountered in the borings in the field. The soils are described in general accordance with the Unified Soil Classification System (ASTM D2487). The logs of the borings and CPT's as well as a key for the classification of the soil (Figure A-1) are included as part of this appendix.

Representative samples were obtained from our exploratory borings at selected depths appropriate to the investigation. Relatively undisturbed samples were obtained using a 3-inch O.D. split barrel sampler with liners, and disturbed samples were obtained using the 2-inch O.D. split spoon sampler. All samples were transmitted to our offices for evaluation and appropriate testing. Both sampler types are indicated in the "Sampler" column of the boring logs as designated in Figure A-1. The elevations discussed in this report and shown on the boring logs in this appendix were obtained from the base map shown on Figure 1; datum unknown.

Resistance blow counts were obtained in our borings with the samplers by dropping a 140-pound safety hammer through a 30-inch free fall. The sampler was driven 18 inches and the number of blows were recorded for each 6 inches of penetration. The blows per foot recorded on the boring logs represent the accumulated number of converted blows that were required to drive the last 12 inches, or the number of inches indicated where hard resistance was encountered. The blow counts recorded on the boring logs have been converted to equivalent SPT field blowcounts, but have not been corrected for overburden, silt content, or other factors.

The attached boring logs and related information show our interpretation of the subsurface conditions at the dates and locations indicated, and it is not warranted that they are representative of subsurface conditions at other locations and times.

UNIFIED SOIL CLASSIFICATION SYSTEM

Major Divisions		grf	ltr	Description
Coarse Grained Soils	Gravel Gravelly Soils		GW	Well-graded gravels or gravel sand mixtures, little or no fines
			GP	Poorly-graded gravels or gravel sand mixture, little or no fines
			GM	Silty gravels, gravel-sand-silt mixtures
			GC	Clayey gravels, gravel-sand-clay mixtures
	Sand And Sandy Soils		SW	Well-graded sands or gravelly sands, little or no fines
			SP	Poorly-graded sands or gravelly sands, little or no fines
			SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, and-clay mixtures

Major Divisions		grf	ltr	Description
Soils	Silts And Clays LL < 50		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
			OL	Organic silts and organic silt-clays of low plasticity
	Silts And Clays LL > 50		MH	Inorganic silts, micaceous or diatomaceous fine or silty soils, elastic silts
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic clays of medium to high plasticity
Highly Organic Soils			PT	Peat and other highly organic soils

GRAIN SIZES

U.S. STANDARD SERIES SIEVE — CLEAR SQUARE SIEVE OPENINGS

200 — 40 — 10 — 4 — 3/4" — 3" — 12"

Silts and Clays	Sand			Gravel		Cobbles	Boulders
	Fine	Medium	Coarse	Fine	Coarse		

RELATIVE DENSITY

Sands and Gravels	Blows/Foot*
Very Loose	0 - 4
Loose	4 - 10
Medium Dense	10 - 30
Dense	30 - 50
Very Dense	Over 50

CONSISTENCY

Silts and Clays	Blows/Foot*	Strength (tsf)**
Very Soft	0 - 2	0 - 1/4
Soft	2 - 4	1/4 - 1/2
Firm	4 - 8	1/2 - 1
Stiff	8 - 16	1 - 2
Very Stiff	16 - 32	2 - 4
Hard	Over 32	Over 4

*Number of Blows for a 140-pound hammer falling 30 inches, driving a 2-inch O.D. (1-3/8" I.D.) split spoon sampler.
**Unconfined compressive strength.

SYMBOLS & NOTES

- Standard Penetration sampler (2" OD Split Barrel)
- Modified California sampler (3" OD Split Barrel)
- California Sampler (2.5" OD Split Barrel)
- Shelby Tube
- Pitcher Barrel
- HQ Core
- Ground Water level initially encountered
- Ground Water level at end of drilling

PI = Plasticity Index
LL = Liquid Limit
R = R-Value

Increasing Visual Moisture Content

Saturated
Wet
Moist
Damp
Dry

Constituent Percentage

trace	<5%
some	5-15%
with	16-30%
-y	31-49%

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KEY TO EXPLORATORY BORING LOGS

3898 DEPOT ROAD
Hayward, CA

PROJECT NO.	DATE	FIGURE NO.
635-1	May 2014	A-1

DRILL RIG Mobile B-24 CFA	SURFACE ELEVATION 9.0 feet	LOGGED BY TC
DEPTH TO GROUND WATER 7 feet	BORING DIAMETER 4-inch	DATE DRILLED 05/05/14

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	ELEVATION	SAMPLER	SPT N-VALUE	WATER CONTENT (%)	DRY DENSITY (PCF)	UNC. COMP. (KSF)	OTHER TESTS
FILL: GRAVEL (GM), gray, fine to coarse, angular to subrounded, sandy(fine- to coarse-grained), with silt, dry.	dense		0			16				
FILL: SILT (ML), light gray, sandy(fine-grained), clayey, moist to damp.	stiff					10				
CLAY (CH), dark gray to black, silty, some organics, dry to damp.	stiff		5	5		12	29	93	3.4	At 6': Liquid Limit = 63 Plasticity Index = 47 Medium Sand = 1% Fine Sand = 3% Silt = 30% Clay = 66%
CLAY (CL/CH), mottled brown gray, silty, dry to damp.	very stiff		10	0		16	25	103	4.7	
CLAY (CL), yellowish brown, silty, with to sandy(fine- to coarse-grained), moist.	very stiff		15	-5		17				
Some sand(fine-grained), damp.			20	-10		19	23	106	4.5	
No recovery. Trace gravel(fine, angular).			25	-15		19				
Damp to moist.	stiff		30	-20		9				
Bottom of Boring = 31.5 feet Notes: Stratification is approximate, variations must be expected. Blowcounts converted to SPT N-values. See Report for additional details.				-25						

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EXPLORATORY BORING LOG

3898 DEPOT ROAD
Hayward, CA

PROJECT NO.	DATE	BORING NO.
635-1	**May 2014**	**SFB-1**

DRILL RIG Mobile B-24 CFA	SURFACE ELEVATION 9.5 feet	LOGGED BY TC
DEPTH TO GROUND WATER 7 feet	BORING DIAMETER 4-inch	DATE DRILLED 05/05/14

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	ELEVATION	SAMPLER	SPT N-VALUE	WATER CONTENT (%)	DRY DENSITY (PCF)	UNC. COMP. (KSF)	OTHER TESTS
FILL: GRAVEL (GM), mottled red gray, fine to coarse, subangular to subrounded, sandy(fine- to coarse-grained), with silt, dry. Wet at 2'.	dense / medium dense		0			11				
				5		9				
CLAY (CH), dark gray to black, silty, trace organics, dry to damp.	stiff		5			13	30	93	3.3	
CLAY (CL/CH), mottled brown gray, silty, damp.	very stiff		10	0		16				
Bottom of Boring = 11.5 feet Notes: Stratification is approximate, variations must be expected. Blowcounts converted to SPT N-values. See Report for additional details.			15	-5						
			20	-10						
			25	-15						
			30	-20						
				-25						

EXPLORATORY BORING LOG 635-1.GPJ STEVENS FERRONE BAILEY.GDT 5/16/14

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Concord, CA 94523
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EXPLORATORY BORING LOG

3898 DEPOT ROAD
Hayward, CA

PROJECT NO.	DATE	BORING NO.
635-1	**May 2014**	**SFB-2**

DRILL RIG Mobile B-24 CFA	SURFACE ELEVATION 10.0 feet	LOGGED BY TC
DEPTH TO GROUND WATER 9 feet	BORING DIAMETER 4-inch	DATE DRILLED 05/05/14

DESCRIPTION AND REMARKS	CONSIST	DEPTH (FEET)	ELEVATION	SAMPLER	SPT N-VALUE	WATER CONTENT (%)	DRY DENSITY (PCF)	UNC. COMP. (KSF)	OTHER TESTS
FILL: GRAVEL (GM), gray, fine to coarse, subangular to subrounded, sandy(fine- to coarse-grained), with silt, dry.	medium dense	0	10						
FILL: SAND (SM), brown, fine- to coarse-grained, gravelly(fine, subangular to subrounded), dry.	loose				8				
FILL: CLAY (CH), mottled brown dark gray, silty, some sand clasts(fine- to coarse-grained), dry to damp.	stiff				9				
CLAY (CH), dark gray to black, silty, dry to damp.	stiff	5	5		14				
CLAY (CL/CH), mottled brown gray, silty, damp.	stiff	10	0		11				
CLAY (CL), mottled gray yellowish brown, silty, with sand(fine- to coarse-grained), damp to moist.	very stiff	15	-5		20				
Bottom of Boring = 16.5 feet Notes: Stratification is approximate, variations must be expected. Blowcounts converted to SPT N-values. See Report for additional details.		20	-10						
		25	-15						
		30	-20						

EXPLORATORY BORING LOG 635-1.GPJ STEVENS FERRONE BAILEY.GDT 5/16/14

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EXPLORATORY BORING LOG

3898 DEPOT ROAD
Hayward, CA

PROJECT NO.	DATE	BORING NO.
635-1	**May 2014**	**SFB-3**

DRILL RIG Mobile B-24 CFA	SURFACE ELEVATION 7.5 feet	LOGGED BY TC
DEPTH TO GROUND WATER Not Encountered	BORING DIAMETER 4-inch	DATE DRILLED 05/05/14

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	ELEVATION	SAMPLER	SPT N-VALUE	WATER CONTENT (%)	DRY DENSITY (PCF)	UNC. COMP. (KSF)	OTHER TESTS
Asphalt concrete (AC) 2" thick.			0							
FILL: CLAY (CL), yellowish brown, gravelly(fine to coarse, angular to subrounded), with sand(fine- to coarse-grained), with silt, dry to damp.	stiff			5		8				
CLAY (CH), dark gray to black, silty, trace organics, damp.	stiff					13				
CLAY (CL), grayish brown, silty, with sand clasts(fine- to coarse-grained), dry to damp.	very stiff		5			19				
Bottom of Boring = 6.5 feet Notes: Stratification is approximate, variations must be expected. Blowcounts converted to SPT N-values. See Report for additional details.				0						
			10	-5						
			15	-10						
			20	-15						
			25	-20						
			30	-25						

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EXPLORATORY BORING LOG

3898 DEPOT ROAD
Hayward, CA

PROJECT NO.	DATE	BORING NO.
635-1	**May 2014**	**SFB-4**

APPENDIX B

Laboratory Investigation

APPENDIX B
Laboratory Investigation

Our laboratory testing program for the proposed concrete recycling facility at 3898 Depot Road in Hayward, California was directed toward a quantitative and qualitative evaluation of the physical and mechanical properties of the soils underlying the site.

The natural water content was determined on four samples of the subsurface soils. The water contents are recorded on the boring logs at the appropriate sample depths.

Dry density determination was performed on four samples of the subsurface soils to evaluate their physical properties. The results of the tests are shown on the boring logs at the appropriate sample depths.

Atterberg Limit determinations were performed on one sample of the subsurface soils to determine the range of water content over which these materials exhibit plasticity. These values are used to classify the soil in accordance with the Unified Soil Classification System and to indicate the soil's compressibility and expansion potentials. The results of the tests are presented on the boring log at the appropriate sample depth.

Gradation and hydrometer tests were performed on one sample of the subsurface soils. These tests were performed to assist in the classification of the soils and to determine their grain size distribution. The results of the tests are presented on the boring log at the appropriate sample depth.

Unconfined compression test was performed on four relatively undisturbed samples of the subsurface soils to evaluate the undrained shear strengths of these materials. Failure was taken as the peak normal stress. The results of the tests are presented on the boring logs at the appropriate sample depths.

APPENDIX C

ASFE Guidelines

Important Information about Your

Geotechnical Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you* — should apply the report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations" many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, a number of mold prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; ***none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.***

Rely on Your ASFE-Member Geotechnical Engineer for Additional Assistance

Membership in ASFE/The Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with your ASFE-member geotechnical engineer for more information.

ASFE THE GEOPROFESSIONAL BUSINESS ASSOCIATION

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