

January 27, 2015

Mr. Brent Marino
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707 17th Street, Suite 2400
Denver, CO 80202

Re: Geotechnical Investigation
Maintenance Building, Fort Rosecrans National Cemetery, San Diego, California
SFB Project No.: 361-20

Mr. Marino:

As requested, Stevens, Ferrone & Bailey Engineering Company, Inc. has performed a geotechnical investigation for the maintenance building project at Fort Rosecrans National Cemetery in San Diego, 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)

January 27, 2015

**GEOTECHNICAL INVESTIGATION
MAINTENANCE BUILDING
FORT ROSECRANS NATIONAL CEMETERY
SAN DIEGO, CALIFORNIA
*SFB PROJECT NO. 361-20***

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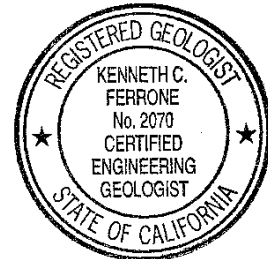


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1.0 INTRODUCTION

This report presents the results of our geotechnical investigation for the proposed maintenance building at the Fort Rosecrans National Cemetery in San Diego, 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. Brent Marino of Jacobs, it is our understanding that the project will consist of constructing a new maintenance building of about 900 square feet and a new covered vehicle storage structure of about 450 square feet at the existing maintenance yard site. A new retaining wall is also proposed to replace the existing screen wall/retaining wall located along the western site boundary. Both asphalt concrete paved and gravel covered areas are also proposed. Designated existing structures and facilities will be removed prior to new construction. 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 available published and unpublished geotechnical and geological literature relevant to the site and vicinity;
- Reviewing a provided preliminary geotechnical investigation report prepared by American Geotechnical and dated May 19, 1999 for cemetery columbarium sites;
- Reviewing a provided geotechnical evaluation report prepared by MACTEC Engineering and Consulting, Inc. and dated July 16, 2010 for the recently built columbarium walls located to the immediate north of the site;
- Performing reconnaissance of the site and surrounding area;
- Performing a subsurface exploration program which included drilling 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, building foundations, retaining walls, and pavements. Toxicity potential assessment of onsite materials or groundwater (including mold) was beyond our scope of work.

3.1 SITE INVESTIGATION

Reconnaissance of the site and surrounding area was performed on January 2, 2015. Subsurface exploration was performed using a truck-mounted drill rig equipped with 4-inch diameter, continuous flight, solid stem augers and 2-inch diameter hand augers. Six exploratory borings were drilled on January 2, 2015 to a maximum depth of about 31-1/2 feet. Previously, six exploratory borings and four test pits were performed by MACTEC in June 2010 to a maximum depth explored of about 20 feet at the columbarium wall site located immediate north of the maintenance yard site. The approximate locations of SFB's borings and adjacent previous borings and pit by others 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. Logs of the previous borings by others for the adjacent site are provided in Appendix C for reference. 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.

3.2 Surface

At the time of our investigation and as shown on Figure 1, the existing maintenance yard site was bounded by Eastern Drive on the west, a recently developed columbarium walls site on the north, and open slope areas on the east and south. The eastern and southern slopes generally sloped downward toward the southeast with an average inclination of about 1.5:1 (horizontal to vertical).

The site was irregular in shape and had a plan area of about half of an acre with maximum dimensions of about 280 feet by 120 feet. The site was approximately level and occupied by several existing maintenance structures (including sheds, containers, and temporary buildings) and associated facilities. Deteriorated asphalt concrete pavement was also observed within the yard. An existing concrete screen wall retaining about 5 to 8 vertical feet of soil was located along the western site boundary. To create the level pad for an existing above-ground fuel storage/pump station, we observed what appeared to be a cut about 3 feet deep had been made recently at the toe of the screen wall near the northwestern corner of the site. An existing columbarium wall was closely located above and along the screen wall/retaining wall (at distances varying from about 2 to 10 feet from the screen wall/retaining wall). According to the previous as-built columbarium construction structural details (Package #3A Drawing S-1.3A) prepared by Burkett & Wong Engineers and dated August 19, 2002, the existing columbarium is possibly supported by footings founded at a depth of about 2 feet below grade. The construction details of the screen wall/retaining wall are unknown.

3.3 Subsurface

The results of our exploratory borings generally indicate the site (below pavement sections where existing) is blanketed by undocumented fills that extend to depths varying from about 18 feet at the east end to about 4-1/2 feet at the west end. These fills consist of very loose to medium dense sands with occasional thin lenses of gravels and clays. The fills encountered in our borings were heterogeneous and potentially weak and compressible if they were not placed and compacted in accordance with acceptable engineering standards. Below the surficial fill layers, very stiff to hard native clays and very dense native sands and gravels were encountered that extended to the maximum depth explored of about 31-1/2 feet. The subsurface materials encountered by previous borings and pits by others at the columbarium wall site located to the north are generally similar to those encountered by our borings. According to the results of our laboratory testing, the near-surface sandy fills have a low plasticity and low expansion potential. Cross-Section A-A' (attached as Figure 2) shows our interpretation of the subsurface conditions encountered by our borings.

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.

3.4 Groundwater

No groundwater was encountered in our borings at the site to the maximum depth explored of about 31-1/2 feet. SFB's borings were backfilled prior to leaving the site. 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.

3.5 Geology and Seismicity

According to Kennedy and Tan (2005)¹, the site (below pavement and surficial fills) is underlain by middle to early Pleistocene very old paralic deposits that have been described as poorly sorted, moderately permeable, reddish brown, interfingered strandline, beach, estuarine, and colluvial deposits composed of siltstone, sandstone, and conglomerate. These paralic deposits

¹Kennedy and Tan, 2005, *Geologic Map of the San Diego 30'X60' Quadrangle, California*, CGS Regional Geologic Map series, Map No.3.

were previously termed Linda Vista Formation by Kennedy and Peterson (1975)² in the site area and are underlain by Upper Cretaceous Cabrillo Formation that has been described as mostly massive medium-grained sandstone.

The project site is located in the San Diego area that is considered one of the most seismically active regions in the United States. The approximate direction and distance from the site to nearby active faults are summarized in the table below³.

Fault Name	Approximate Distance to Fault (Miles)	Direction to Fault
Spanish Bight	2.7	East
Coronado	4.0	East
Silver Strand	4.9	East
Coronado bank	7.0	West
Rose Canyon	7.4	North
San Diego Trough	18.7	Southwest

The site is also located in close proximity of the potentially active, late Quaternary Point Loma fault that shows evidence of displacement during the last 700,000 years. According to the Alquist-Priolo Earthquake Fault Zones Map of the Point Loma Quadrangle, the site is not located in an earthquake fault zone as designated by the State of California⁴.

Earthquake intensities will vary throughout the San Diego, 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. 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 0.25 g in 50 years (design basis ground motion based on stiff soil site condition; mean return time of 475

²Kennedy and Peterson, 1975, *Point Loma Quadrangle, Geology of the San Diego metropolitan area, California: California Division of Mines and Geology Bulletin 200*.

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

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

years). The actual ground surface acceleration might vary depending upon the local seismic characteristics of the underlying bedrock and the overlying unconsolidated soils.

3.6 Slope Stability

According to the 1995 California Geological Survey Landslide Hazard Identification Map No. 33 (Landslide Hazard in the Southern Part of the San Diego Metropolitan Area), the site is not mapped as having previously identified landslides. However, Map No. 33 shows the site to be located within Landslide Hazard Area 4-1 that is characterized as being generally located outside the boundaries of definite mapped landslides but contains observably unstable slopes underlain by both weak materials and adverse geologic structure. Map No. 33 shows that the existing massive Ft. Rosecrans Landslide is located to the northeast of the site. Information contained in the 1973 IGC Field Trip Guide: *Engineering Geology in San Diego, California* by Hart supports the information shown in Map. No. 33.

Based on the results of our reconnaissance, field exploration, and review of documents, we did not observe obvious evidence of active, deep seated slope movement onsite or in the close proximity of the site. Our scope of work, however, did not include performing a comprehensive slope stability evaluation of the site and surrounding area under both static and earthquake conditions. The stability of the site and surrounding area is affected by numerous factors including the underlying soil and rock characteristics, rainfall, irrigation, earthquake shaking, slope erosion, and changes to the topography. Therefore, the stability of the site and surrounding area can change over time. In addition, it is not uncommon for relatively shallow slope movements to occur within the soils and fills blanketing the site and the vicinity. These movements may include downslope creep, erosion, and slumping.

3.7 Liquefaction

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. As of the date of this report, the liquefaction potential of the site has not been evaluated by the State of California⁵.

Based on our review of available literature and the results of exploratory borings, and field and laboratory testing by SFB, it is our opinion that the potential for ground surface damage at the

⁵Seismic Hazards Mapping Act, 1990.

site resulting from liquefaction is low due to the lack of saturated liquefiable soils at the site and the presence of Upper Cretaceous Cabrillo Formation at relative shallow depths.

4.1 CONCLUSIONS AND RECOMMENDATIONS

It is our opinion that the site is suitable for the proposed maintenance building 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.

UNDOCUMENTED FILLS: As described previously and based on the results of our borings, undocumented fills blanket the entire site and extend to depths varying from about 18 feet at the east end to about 4-1/2 feet at the west end. These fills are heterogeneous and potentially weak and compressible if they were not placed and compacted in accordance with acceptable engineering standards. In order to reduce the potential for damaging differential settlement of overlying new building foundations and retaining walls, we recommend that these weak undocumented fills be completely removed and re-compacted at the new building and retaining wall locations. The over-excavation should extend to depths where competent soil or rock is encountered. The over-excavation and re-compaction should also extend to a lateral distance beyond building and retaining wall footprints that at least equals the actual over-excavation depths (i.e., if the over-excavation depth is 10 feet then the over-excavation lateral limits should extend at least 10 feet beyond the footprint of the structure). In addition, we also recommend 3-foot deep over-excavation and re-compaction (that extend at least 3 feet beyond pavement and exterior flatwork) be performed for new pavement and exterior flatwork areas. Where the over-excavation limits abut adjacent developments, SFB should be consulted to determine the actual vertical and lateral extent of over-excavation so that adjacent developments are not adversely impacted. The existing columbarium wall located along the western site boundary may require underpinning during the site construction. Over-excavations should be performed so that no more than 5 feet of differential fill thickness exists below the proposed structure foundations. The removed fill, soil, and rock 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.

SETBACK FROM EASTERN SLOPE: In order to reduce the potential for damage to improvements caused by surficial slope movements, setbacks should be used. We recommend improvements be setback at least 30 feet from the crest of the eastern boundary slope. It should be noted this setback distance is estimated based on our review of available literature, topographic maps of the site and vicinity, the results of our exploratory borings, and field and laboratory testing. Improvements located between the setback line and the eastern slope may

have higher potential to experience movement as a result of surficial slope features including erosion and localized slumping. As described previously, however, it was beyond our scope of work to perform detailed slope stability analyses of the site and surrounding area; results of slope stability analyses may indicate that greater setbacks be used and/or ground strengthening be performed.

CORROSION POTENTIAL: Two onsite soil samples were tested for pH (ASTM D4972), chlorides (ASTM D4327), sulfates (ASTM D4327), sulfides (ASTM D4658M), resistivity at 100% saturation (ASTM G57), and Redox potential (ASTM D1498) for use in evaluating the potential for corrosion on concrete and buried metal such as utilities and reinforcing steel. The results of these tests are included in Appendix B. We recommend these test results be forwarded to your underground contractors, pipeline designers, and foundation designers and contractors.

SEEPAGE, SURFACE, AND SUBSURFACE WATER: Water seepage will occur during and after periods of rainfall and as a result of irrigation. To reduce the potential for seepage within the site, consideration should be given to installing subdrains at the base of the over-excavations and/or wherever surface and seepage water is directed toward the site. After construction is complete, seepage may occur as the seepage patterns below the ground surface resulting from irrigation and storm water flow develop over time which would require the installation of additional subdrains. Surface water should not be allowed to flow over the top of slopes and retaining walls. We recommend that the actual location and extent of subdrains should be assessed by SFB during the development of the grading and improvement plans.

EROSION AND SLOPE MAINTENANCE: Drainage and erosion control measures should be maintained during and after construction. Short-term and long-term erosion control are critical for the stability of any hillsides. The erosion control measures will require inspection, modification, and re-mediation during the rainy seasons. Slope maintenance may include re-establishing drainage patterns, controlling water infiltration, and repairing shallow slope movements.

ADDITIONAL RECOMMENDATIONS: Detailed drainage, earthwork, foundation, retaining wall, 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.

4.2 Earthwork

4.2.1 Clearing and Site Preparation

Where improvements are proposed, we recommend the proposed area be cleared of all obstructions including any existing structures and their entire foundation systems, any existing utilities and pipelines and their associated backfill, septic systems (if they exist), 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. Septic systems should be abandoned in accordance with appropriate standards.

From a geotechnical standpoint, any existing trench backfill materials, pavements, or concrete slabs 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**. We recommend backfilling operations for any excavations be performed under the observation and testing of SFB.

4.2.2 Existing Fill Re-compaction

As described previously and based on the results of our borings, undocumented fills blanket the entire site and extend to depths varying from about 18 feet at the east end to about 4-1/2 feet at the west end. These fills are heterogeneous and potentially weak and compressible if they were not placed and compacted in accordance with acceptable engineering standards. In order to reduce the potential for damaging differential settlement of overlying new building foundations and retaining walls, we recommend that these weak undocumented fills be completely removed and re-compacted at the new building and retaining wall locations. The over-excavation should extend to depths where competent soil or rock is encountered. The over-excavation and re-compaction should also extend to a lateral distance beyond building and retaining wall footprints that at least equals the actual over-excavation depths (i.e., if the over-excavation depth is 10 feet then the over-excavation lateral limits should extend at least 10 feet beyond the footprint of the structure). In addition, we also recommend 3-foot deep over-excavation and re-compaction (that extend at least 3 feet beyond pavement and exterior flatwork) be performed for new pavement and exterior flatwork areas. Where the over-excavation limits abut adjacent developments, SFB should be consulted to determine the actual vertical and lateral extent of over-excavation so that adjacent developments are not adversely impacted. The existing columbarium wall located

along the western site boundary may require underpinning during the site construction. Over-excavations should be performed so that no more than 5 feet of differential fill thickness exists below the proposed structure foundations. The removed fill, soil, and rock 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.

Removed fill, soil, and rock materials may be used as new fill onsite provided it satisfies 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***.

4.2.3 Subgrade Preparation

If building and retaining wall pads, or exterior slab 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 subgrade may need to be reconditioned (moisture conditioned and/or scarified and recompacted) 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 structural fill, building and retaining foundations, pavement, and exterior flatwork) should be scarified to a depth of about 12 inches, moisture conditioned to approximately 2 to 3 percent over optimum water content, and compacted to the requirements for structural fill.

4.2.4 Fill Material

From a geotechnical and mechanical standpoint, onsite 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. If required, imported fill should have a plasticity index of 12 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.

4.2.5 Compaction

We recommend structural fill be compacted to at least 95 percent relative compaction as determined by ASTM D1557 (latest edition). We recommend the new fill be moisture conditioned to about 2 to 3 percent over optimum water content. The upper 12 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.

4.2.6 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 95 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. 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 pavements, exterior slabs-on-grade, or under the building 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.

4.2.7 Exterior Flatwork

We recommend that new exterior slabs be placed directly on the properly compacted subgrade. Prior to placing concrete, subgrade soils should be moisture conditioned to increase their moisture content to about 2 to 3 percent above laboratory optimum moisture (ASTM D1557).

Consideration should be given to reinforcing new 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.

4.2.8 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.

4.2.9 Surface Drainage, Irrigation, and Landscaping

Ponding of surface water must not be allowed 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 pavement and exterior slabs. Surface water should not be allowed to flow over the top of slopes, down slope faces, or over retaining walls.

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 building code.

In order to reduce differential foundation 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 should be 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 low flow watering systems be used. All irrigation systems should be inspected for leakage regularly.

4.2.10 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 recompaction 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 owners perform inspections and maintenance of the slabs, including infilling significant cracks, providing fillers at slab offsets, and replacing slabs if severely damaged.

4.2.11 Additional Recommendations

We recommend the drainage, irrigation, landscaping, and maintenance recommendations provided in this report be forwarded to your designers and contractors.

4.3 Foundation Support

4.3.1 Conventional Spread Footings

The proposed new maintenance building and covered storage structure can be supported on spreading footing foundations bearing in the onsite compacted fills. Footings should be at least 12 inches wide and should be founded at least 18 inches below lowest adjacent finished grade. A continuous footing should be provided around the perimeter of the proposed buildings. Continuous footings should be designed with steel reinforcing, both top and bottom, to provide structural continuity and permit spanning of local irregularities.

New and existing footings should be evaluated 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.

4.3.2 Interior Slabs-on-Grade

We recommend that new interior slabs-on-grade (used in conjunction with new 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 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 $\frac{1}{2}$ to $\frac{3}{4}$ 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 slabs should be structurally separated from surrounding foundations; a relatively impermeable and flexible filler such as Greenstreak Swellstop ($\frac{3}{8}$ " x $\frac{3}{4}$ " 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/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. The results of sulfate and chloride testing of onsite soil samples are included in Appendix B for reference. We recommend you consult with your concrete slab designers and concrete contractors regarding methods to reduce the potential for differential concrete curing.

4.3.3 Retaining Walls

Where walls retain soil, they must be designed to resist both lateral earth pressures and any additional lateral loads caused by surcharging such as building/structure and roadway loads.

The recommendations provided below are for retaining walls that are located at least $1.5H$ feet away from a building/structure, where H is the height of the retaining portion of the walls. Where concrete or masonry walls are used to retain soil, we recommend unrestrained walls (walls free to deflect and disconnected from other structures) be designed to resist an equivalent fluid pressure of 40 pounds per cubic foot. This assumes a level backfill. Restrained walls (walls restrained from deflection) should be designed to resist an equivalent fluid pressure of 40 pounds per cubic foot plus a uniform pressure of $8H$ pounds per square foot, where H is the height of the wall in feet. Walls with inclined backfill should be designed for an additional equivalent fluid pressure of 1 pound per cubic foot for every 2 degrees of slope inclination. Walls subjected to surcharge loads should be designed for an additional uniform lateral pressure equal to one-third and one-half the anticipated surcharge load for unrestrained and restrained walls, respectively. These lateral pressures depend upon the moisture content of the retained soils to be constant over time; if the moisture content of the retained soils will fluctuate or increase compared to the moisture content at time of construction, then SFB should be consulted and provide written modifications to this design criteria.

For retaining walls located within $1.5H$ feet (where H is the height of the retaining portion of the walls) from a building/structure or walls that are designed to resist seismic lateral forces from the retained soils, we recommend the walls also be designed to resist a triangular pressure distribution equal to an equivalent fluid pressure of 25 pounds per cubic foot based on the ground acceleration from a design basis earthquake. This seismic pressure is in addition to the pressures noted above. Due to the transient nature of the seismic loading, a factor of safety of at least 1.1 can be used in the design of the walls when they resist seismic lateral loads. Some movement of

the walls may occur during moderate to strong earthquake shaking and may result in distress as is typical for all structures subjected to earthquake shaking.

The recommended lateral pressures assume walls are fully-back drained to prevent the build-up of hydrostatic pressures. This can be accomplished by using $\frac{1}{2}$ to $\frac{3}{4}$ inch crushed, uniformly graded gravel entirely wrapped in filter fabric such as Mirafi 140N or equal (an overlap of at least 12 inches should be provided at all fabric joints). The gravel and fabric should be at least 8 inches wide and extend from the base of the wall to within 12 inches of the finished grade at the top (Caltrans Class 2 permeable material (Section 68) may be used in lieu of gravel and filter fabric). A 4-inch diameter, perforated pipe should be installed at the base and centered within the gravel. The perforated pipe should be connected to a solid collector pipe that transmits the water directly to a storm drain, drainage inlet, or onto pavement. If weep holes are used in the wall, the perforated pipe within the gravel is not necessary provided the weep holes are kept free of animals and debris, are located no higher than approximately 6 inches from the lowest adjacent grade, and are able to function properly. As an alternative to using gravel, drainage panels (such as AWD SITEDRAIN Sheet 94 for walls or equal) may be used behind the walls in conjunction with perforated pipe (connected to solid collector pipe), weep holes, or strip drains (such as SITEDRAIN Strip 6000 or equal). If used, the drainage panels can be spaced on-center at approximately 2 times the panel width.

If heavy compaction equipment is used behind the walls, the walls should be appropriately designed to withstand loads exerted by the heavy equipment and/or temporarily braced. Fill placed behind walls should conform to the recommendations provided in **Section 4.1.4, *Fill Material***, and **Section 4.1.5, *Compaction***.

The retaining walls can be supported by conventional spread footings as described in our report **Section 4.2.1, *Conventional Spread Footings***, bearing in the onsite compacted fills. Alternatively, retaining walls can be supported on drilled, cast-in-place, straight shaft friction piers that develop their load carrying capacity in the materials underlying the site. The piers should have a minimum diameter of 12 inches and a center-to-center spacing of at least three times the shaft diameter. We recommend that piers be at least 6 feet long. The pier reinforcing should be based on structural requirements but in no case should less than two #4 bars for the entire length of the pier be used.

The actual design depth of the piers should be determined using an allowable skin friction of 350 pounds per square foot (psf) for dead plus live loads, with a one-third increase for all loads including wind or seismic. Seventy percent of the skin friction value can be used to resist uplift. Lateral load resistance can be developed in passive resistance for pier foundations. A passive resistance equal to an equivalent fluid weighing 350 pounds per cubic foot acting against twice the projected diameter of pier shafts can be used. The upper 1 foot of pier embedment should be

neglected in the vertical and passive resistance design as measured from finished grade. The portion of the pier shaft located within 10 feet (as measured laterally) of the nearest slope face should also be ignored in the design.

We recommend the pier foundations be located outside of (or beyond) a 1:1 (horizontal to vertical) plane projected upward from the base of any wall or utility trench, or the portion of a pier located within this zone should be ignored in the design of the pier.

The bottoms of the pier excavations should be relatively dry and free of all loose cuttings or slough prior to placing reinforcing steel and concrete. Any accumulated water in pier excavations should be removed prior to placing concrete. We recommend that the excavation of all piers be performed under the direct observation of SFB to confirm that the pier foundations are founded in suitable materials and constructed in accordance with the recommendations presented herein. Preliminarily, we recommend concrete pours of pier excavations be performed within 24 hours of excavation and prior to any rainstorms. Where caving or high ground water conditions exist, additional measures such as using casing, tremie methods, and pouring concrete immediately after excavating may be necessary. SFB should be consulted on the need for additional measures for pier construction as needed during construction.

4.3.4 Seismic Design Criteria

For seismic resistance design in accordance with 2012 International Building Code (IBC), we recommend the following seismic design values be used. The following parameters are calculated using the U.S. Seismic Design Map program (Version 3.1.0)⁶, and the 2012 IBC data set, and are based on the site being located at approximate latitude 32.689°N and longitude 117.244°W.

2012 IBC SEISMIC PARAMETERS		
Seismic Parameter	Design Value	Reference
Site Class	D	Section 1613.3.2
S _s	1.12	Figure 1613.3.1(1)
S ₁	0.42	Figure 1613.3.1(2)
F _a	1.0	Table 1613.3.3(1)
F _v	1.5	Table 1613.3.3(2)

4.4 Pavements

Our pavement recommendations provided below are based on the assumption that traffic will consist mainly of relatively light vehicles and equipment such as “bobtail” dump trucks, rubber tired backhoes, pickup trucks, flatbed trucks, and other similar maintenance vehicles. We also

⁶USGS Website, <http://earthquake.usgs.gov/hazards/designmaps/usdesign.php>, Version 3.1.0, last updated 7/11/13.

assumed that heavy vehicles such as fully loaded ten wheel dump trucks or other similar heavy construction equipment will not use the pavement. We should be consulted if the pavement use will differ than what we have assumed.

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.

4.4.1 Asphalt Concrete Pavement

Based on the results of laboratory testing of onsite subgrade materials, we recommend that an R-value of 15 be used in 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.

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 an assumed Traffic Index (T.I.) of 6.0 for typical cemetery operations as described in previous section. The project's Civil Engineer or appropriate agency should determine actual traffic indices. The pavement thicknesses shown below are SFB's recommended minimum values.

PRELIMINARY PAVEMENT DESIGN ALTERNATIVES SUBGRADE R-VALUE = 15			
Location	Pavement Components		Total Thickness (inches)
	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)	
T.I. = 6.0	3.0	12.0	12.0

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.

4.4.2 Concrete Pavement

We recommend the concrete pavement consist of at least 6 inches of concrete having a modulus of rupture of at least 600 psi overlying 6 inches of Caltrans Class 2 aggregate base. We

recommend all joints (both transverse and longitudinal) not exceed 10-feet on center in both directions. Consideration should be given to dowel the joints with #8 bars. The dowels should be at least 18 inches long and should be spaced at approximately 12 inches on center. Pavement baserock should be compacted to at least 95 percent relative compaction per ASTM 1557.

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 ground water 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 Jacobs Engineering for specific application to the proposed maintenance building at the Fort Rosecrans National Cemetery in San Diego, California, and is intended to represent our design recommendations to Jacobs Engineering for specific application to the maintenance building project. The conclusions and recommendations contained in this report are solely professional opinions. It is the responsibility of Jacobs Engineering 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 Jacobs Engineering and their consultants during the course of this engagement and our rendering of professional services to Jacobs Engineering. 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 Jacobs Engineering to divulge information that may have been communicated to Jacobs Engineering. We cannot accept consequences for use of segregated portions of this report.

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

FIGURES

SFB-6 - - APPROXIMATE LOCATION OF SFB
 EXPLORATORY BORING (11216)
B5-\$- APPROXIMATE LOCATION OF PREVIOUS
 EXPLORATORY BORING BY 011-IERS (818110)
TP1 - - -1 APPROXIMATE LOCATION OF PREVIOUS TEST
 PIT BY OTHERS (June 2010)
A // APPROXIMATE LOCATION OF CROSS SECTION
■ | (SEE FIGURE 2 FOR THE SECTION)

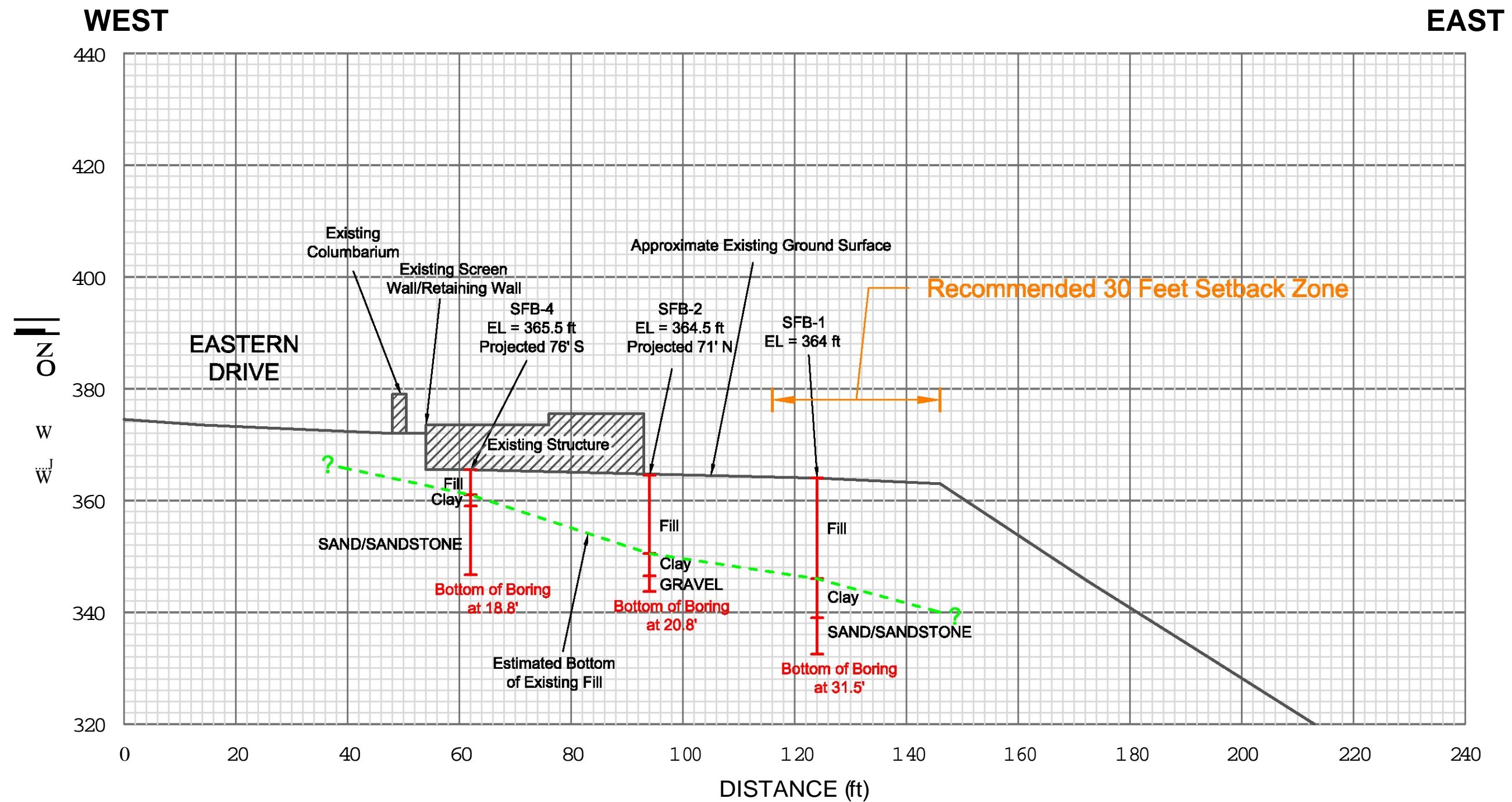
APPROXIMATE PROJECT LIMIT

APPROXIMATE SCALE: 1" = 40'
 0 40 80'

7/18/10 report on Google Earth Image dated 11/13/13.

Jen : 2015....

0 ' ,



CROSS-SECTION A - A'

NOTE:

1. See Figure 1 for location of section.
2. Improvements, elevations, and locations of explorations are approximate.
3. Refer to exploration Jogs for more details. Boring Jogs projected onto cross-section.
4. Elevations based on the site plan of MACTEC Engineering 7116110 report.
5. See report for additional conditions and limitations.

HORIZONTAL SCALE: 1" = 40'

DATE
1-January 2015

1-PR-30-J3

evens
errone &
Cousins

1600 Willow Pass Court
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CROSS-SECTION A - A'

MAINTENANCE BUILDING
Fort Rosecrans National Cemetery, San Diego, California

FIGURE

2

APPENDIX A
Field Investigation

APPENDIX A

Field Investigation

Our field investigation for the proposed maintenance building project at Fort Rosecrans National Cemetery in San Diego, California, consisted of surface reconnaissance and a subsurface exploration program. Geotechnical reconnaissance of the site and surrounding area was performed on January 2, 2015. Subsurface exploration was performed using a truck-mounted drill rig equipped with 4-inch diameter, continuous flight, solid stem augers and using a 2-inch diameter hand auger. Six exploratory borings were drilled on January 2, 2015 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 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 boring 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 pit logs in this appendix were obtained from the base map shown on Figure 1; datum unknown.

Resistance blow counts were obtained in our boring 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	Major Divisions		grf	ltr	Description
Coarse Grained Soils	Gravel		GW	Well-graded gravels or gravel sand mixtures, little or no fines	Soils	Sils And Clays LL < 50		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity
			GP	Poorly-graded gravels or gravel sand mixture, little or no fines				CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
	Gravelly Soils		GM	Silty gravels, gravel-sand-silt mixtures				OL	Organic silts and organic silt-clays of low plasticity
			GC	Clayey gravels, gravel-sand-clay mixtures				Sils And Clays LL > 50	
	Sand And Sandy Soils		SW	Well-graded sands or gravelly sands, little or no fines			CH		
			SP	Poorly-graded sands or gravelly sands, little or no fines			OH		Organic clays of medium to high plasticity
			SM	Silty sands, sand-silt mixtures		Highly Organic Soils		PT	Peat and other highly organic soils
			SC	Clayey sands, and-clay mixtures					

GRAIN SIZES

U.S. STANDARD SERIES SIEVE

CLEAR SQUARE SIEVE OPENINGS

	200	40	10	4	3/4"	3"	12"	
Sils 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

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

Increasing Visual Moisture Content

▲ Saturated
Wet
Moist
Damp
Dry

Constituent Percentage

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

KEY TO EXPLORATORY BORING LOGS

FORT ROSECRANS NATIONAL CEMETERY
San Diego, CA

PROJECT NO.	DATE	FIGURE NO.
361-20	January 2015	A-1

**Stevens,
Ferrone &
Bailey**
Engineering Company, Inc.

1600 Willow Pass Court
Concord, CA 94523
Tel: 925-688-1001

DRILL RIG	Mobile B-24 CFA	SURFACE ELEVATION	364 feet	LOGGED BY	RAC
DEPTH TO GROUND WATER	Not Encountered	BORING DIAMETER	4-inch	DATE DRILLED	01/02/15

DESCRIPTION AND CLASSIFICATION			DEPTH (FEET) ELEVATION	SAMPLER	SPT N-VALUE	WATER CONTENT (%)	DRY DENSITY (PCF)	UNC. COMP. (KSF)	OTHER TESTS
DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE							
ASPHALT CONCRETE (AC), 3.5" thick.			0						
AGGREGATE BASE (AB), 6" thick.									
FILL: SAND (SM), brown, fine- to medium-grained, some coarse-grained, silty, trace gravel(fine, angular to subrounded), dry to damp. Change color to orangish brown.	loose				8	10	112		
			360		7				
Change color to brown.	medium dense		5		10	8	105		
			355						
Abundant small asphalt fragments.	very dense		10		43/5"				
			350						
FILL: CLAY (CH), mottled black brown, silty, some sand(fine- to coarse-grained), trace gravel(fine, angular to subrounded), dry to damp. Change color to brown.	medium dense		15		15				
			345						
CLAY (CL), mottled orangish gray brown, sandy(fine- to medium-grained), damp.	hard		20		46	14	116	2.6	
			340						
SAND (SM), yellowish brown, fine- to medium-grained, with to silty, lightly cemented, dry. (Cabrillo Formation Sandstone).	very dense		25		37/6"				
			335						
			30		67				
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.			330						

EXPLORATORY BORING LOG 361-20.GPJ STEVENS FERRONE BAILEY GDT 1/29/15

**Stevens,
Ferrone &
Bailey**
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1600 Willow Pass Court
Concord, CA 94523
Tel: 925-688-1001

EXPLORATORY BORING LOG

FORT ROSECRANS NATIONAL CEMETERY
San Diego, CA

PROJECT NO.

DATE



















BORING NO.

361-20

January 2015

SFB-1

DRILL RIG	Mobile B-24 CFA	SURFACE ELEVATION	364.5 feet	LOGGED BY	RAC
DEPTH TO GROUND WATER	Not Encountered	BORING DIAMETER	4-inch	DATE DRILLED	01/02/15

DESCRIPTION AND CLASSIFICATION			DEPTH (FEET) ELEVATION	SAMPLER	SPT N-VALUE	WATER CONTENT (%)	DRY DENSITY (PCF)	UNC. COMP. (KSF)	OTHER TESTS
DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE							
ASPHALT CONCRETE (AC), 2" thick.			0						At 3': Fine Gravel = 3% Coarse Sand = 2% Medium Sand = 18% Fine Sand = 44% Silt and Clay = 33%
AGGREGATE BASE (AB), 5" thick.									
FILL: SAND (SM), mottled yellowish brown, fine- to medium-grained, some coarse-grained, with clay and silt, trace gravel(fine, angular), dry to damp.	loose				8	11	110		
With chunks of clay at 3'.	very loose				3				
	loose		360						
			5		6	10	100		
									
Change color to grayish brown.			10		7				
									
									
CLAY (CL/CH), brown, silty, some sand(fine- to coarse-grained), trace gravel(fine, rounded), dry to damp.	very stiff		15		24	21	105	2.6	
GRAVEL (GM), mottled orangish gray brown, fine to coarse, angular to subrounded, sandy(fine- to coarse-grained), with silt and clay, dry.	very dense		20		50/4"				
Bottom of Boring = 20.8 feet Notes: Stratification is approximate, variations must be expected. Blowcounts converted to SPT N-values. See Report for additional details.			25						
			30						
			330						

EXPLORATORY BORING LOG 361-20.GPJ STEVENS FERRONE BAILEY.GDT 1/29/15

**Stevens,
Ferrone &
Bailey**
Engineering Company, Inc.

1600 Willow Pass Court
Concord, CA 94523
Tel: 925-688-1001

EXPLORATORY BORING LOG

FORT ROSECRANS NATIONAL CEMETERY
San Diego, CA

PROJECT NO.

DATE






















BORING NO.

361-20

January 2015

SFB-2

DRILL RIG	Mobile B-24 CFA	SURFACE ELEVATION	365 feet	LOGGED BY	RAC
DEPTH TO GROUND WATER	Not Encountered	BORING DIAMETER	4-inch	DATE DRILLED	01/02/15

DESCRIPTION AND CLASSIFICATION			DEPTH (FEET) ELEVATION	SAMPLER	SPT N-VALUE	WATER CONTENT (%)	DRY DENSITY (PCF)	UNC. COMP. (KSF)	OTHER TESTS
DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE							
FILL: SAND (SM), brown, fine- to medium-grained, some coarse-grained, silty, trace gravel(fine, rounded to subrounded), trace clay, dry to damp. Change color to orangish brown. Change color to brown.	medium dense loose		0 365 1 2 3 4 5 360 6 7 8 9 10 355 11 12 13 14	      	23 7 5 5	9	110		
FILL: GRAVEL (GP), mottled gray black, fine to coarse, angular to subrounded, sandy(fine- to coarse-grained), trace silt, trace rootlets, dry.	dense		15 350 16 17 18 19	 	30				
CLAY (CL), grayish brown, silty, with sand(fine- to medium-grained), trace gravel(fine, subangular), dry to damp.	very stiff		20 345 21 22 23 24	   	30/3"				
Bottom of Boring = 20.8 feet Notes: Stratification is approximate, variations must be expected. Blowcounts converted to SPT N-values. See Report for additional details.			25 340 26 27 28 29 30 335 31 32 33 34 35	    					

EXPLORATORY BORING LOG 361-20.GPJ STEVENS FERRONE BAILEY GDT 1/29/15

**Stevens,
Ferrone &
Bailey**
Engineering Company, Inc.

1600 Willow Pass Court
Concord, CA 94523
Tel: 925-688-1001

EXPLORATORY BORING LOG

FORT ROSECRANS NATIONAL CEMETERY
San Diego, CA

PROJECT NO.

DATE

BORING NO.

361-20

January 2015

SFB-3

DRILL RIG	Mobile B-24 CFA	SURFACE ELEVATION	365.5 feet	LOGGED BY	RAC
DEPTH TO GROUND WATER	Not Encountered	BORING DIAMETER	4-inch	DATE DRILLED	01/02/15

DESCRIPTION AND CLASSIFICATION			DEPTH (FEET) ELEVATION	SAMPLER	SPT N-VALUE	WATER CONTENT (%)	DRY DENSITY (PCF)	UNC. COMP. (KSF)	OTHER TESTS
DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE							
ASPHALT CONCRETE (AC), 3.5" thick.			0						
AGGREGATE BASE (AB), 5" thick.			365						
FILL: SAND (SC), orangish brown, fine- to medium-grained, some coarse-grained, clayey, trace gravel(fine, subangular to subrounded), dry to damp.	medium dense				11	9	106		
FILL: SAND (SM), mottled gray brown, fine- to coarse-grained, silty, some gravel(fine to coarse, angular), some pieces of brick and charcoal, dry to damp.	loose				7				
CLAY (CL), brown, silty, trace gravel(fine, angular), trace sand(coarse-grained), damp.	very stiff		5		30/4"	18	109		
SAND (SM), orangish brown, fine- to medium-grained, some coarse-grained, with to gravelly(fine to coarse, subangular to subrounded), with silt, dry. (Cabrillo Formation Sandstone).	very dense				50/3"				
Difficult drilling below 8'.			10						
Some gravel(fine to coarse, subangular to subrounded).					36/6"	7	101		
			15						
Trace gravel(fine to coarse, subrounded).					39/4"				
			20						
Bottom of Boring = 18.8 feet									
Notes: Stratification is approximate, variations must be expected. Blowcounts converted to SPT N-values. See Report for additional details.									
			25						
			30						
			335						

EXPLORATORY BORING LOG 361-20.GPJ STEVENS FERRONE BAILEY.GDT 1/29/15



1600 Willow Pass Court
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EXPLORATORY BORING LOG

FORT ROSECRANS NATIONAL CEMETERY
San Diego, CA

PROJECT NO.

DATE


BORING NO.

361-20


January 2015

SFB-4


DRILL RIG	Hand Auger	SURFACE ELEVATION	363.5 feet	LOGGED BY	RAC
DEPTH TO GROUND WATER	Not Encountered	BORING DIAMETER	2-inch	DATE DRILLED	01/02/15

DESCRIPTION AND CLASSIFICATION									OTHER TESTS
DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE							
FILL: SAND (SM), brown, fine- to medium-grained, trace gravel(fine, angular to subrounded), dry. Change color to orangish brown.	loose		0						
Bottom of Boring = 4.5 feet Notes: Stratification is approximate, variations must be expected. Blowcounts converted to SPT N-values. See Report for additional details.			360						
			5						
			355						
			10						
			350						
			15						
			345						
			20						
			340						
			25						
			335						
			30						
			330						


EXPLORATORY BORING LOG 361-20.GPJ STEVENS FERRONE BAILEY GDT 1/29/15

 <div> 1600 Willow Pass Court Concord, CA 94523 Tel: 925-688-1001 </div>		EXPLORATORY BORING LOG		
		FORT ROSECRANS NATIONAL CEMETERY San Diego, CA		
		PROJECT NO.	DATE	BORING NO.
		361-20	January 2015	SFB-5

DRILL RIG	Hand Auger	SURFACE ELEVATION	370 feet	LOGGED BY	RAC
DEPTH TO GROUND WATER	Not Encountered	BORING DIAMETER	2-inch	DATE DRILLED	01/02/15

DESCRIPTION AND CLASSIFICATION										OTHER TESTS
DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE								
Wood Mulch 6" thick.			0	370						
FILL: SAND (SM), brown, fine- to medium-grained, trace clay, trace gravel(fine, subangular to subrounded), dry.	loose									
Bottom of Boring = 3.5 feet Notes: Stratification is approximate, variations must be expected. Blowcounts converted to SPT N-values. See Report for additional details.			5	365						
			10	360						
			15	355						
			20	350						
			25	345						
			30	340						

EXPLORATORY BORING LOG 361-20.GPJ STEVENS FERRONE BAILEY GDT 1/29/15

 <div> 1600 Willow Pass Court Concord, CA 94523 Tel: 925-688-1001 </div>		EXPLORATORY BORING LOG		
		FORT ROSECRANS NATIONAL CEMETERY San Diego, CA		
		PROJECT NO.	DATE	BORING NO.
		361-20	January 2015	SFB-6

APPENDIX B

Laboratory Investigation

APPENDIX B

Laboratory Investigation

Our laboratory testing program for the proposed maintenance building project at Fort Rosecrans National Cemetery in San Diego, 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 ten 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 ten 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.

Gradation 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 two 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.

Two onsite soil samples were tested for pH (ASTM D4972), chlorides (ASTM D4327), sulfates (ASTM D4327), sulfides (ASTM D4658M), resistivity at 100% saturation (ASTM G57), and Redox potential (ASTM D1498) for use in evaluating the potential for corrosion on concrete and buried metal such as utilities and reinforcing steel. The results of these tests are included in Appendix B. We recommend these test results be forwarded to your underground contractors, pipeline designers, and foundation designers and contractors.

1100 Willow Pass Court, Suite A

Concord, CA 94520-1006

925 462 2771 Fax 925 462 2775

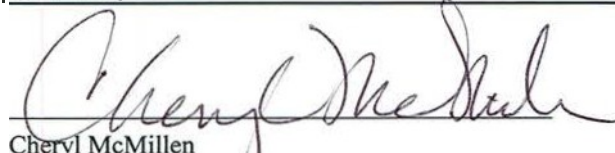
www.cercoanalytical.com

Client: Stevens, Ferrone & Bailey
 Client's Project No.: 361-20
 Client's Project Name: Fort Rosecrans National Cemetery, San Diego
 Date Sampled: 2-Jan-15
 Date Received: 6-Jan-15
 Matrix: Soil
 Authorization: Signed Chain of Custody

Date of Report: 21-Jan-2015

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)*
150I019-001	SFB-2 @ 5'	400	7.15	-	3,600	N.D.	N.D.	N.D.
150I019-002	SFB-4 @ 1'	380	7.50	-	1,700	N.D.	N.D.	19

Method:	ASTM 01498	ASTM 04972	ASTM 01125M	ASTM G57	ASTM 04658M	ASTM 04327	ASTM 04327
Detection Limit:			10		50	3	3
Date Analyzed:	9-Jan-2015	9-Jan-2015		8-Jan-2015	8-Jan-2015	20-Jan-2015	20-Jan-2015



Cheryl McMillen
Laboratory Director


* Results Reported on "As Received" Basis
 ND. - None Detected

APPENDIX C

Logs of Previous Borings by Others

SUBSURFACE EXPLORATION LEGEND

UNIFIED SOIL CLASSIFICATION CHART

SOIL DESCRIPTION		GROUP SYMBOL	TYPICAL NAMES
I. COARSE GRAINED, more than half of material is larger than No. 200 sieve size.			
GRAVELS More than half of coarse fraction is larger than No. 4 sieve size, but smaller than 3".	CLEAN GRAVELS	GW GP	Well graded gravels, gravel-sand mixtures, little or no fines. Poorly graded gravels, gravel-sand mixtures, little or no fines.
	GRAVELS WITH FINES (Appreciable amount of fines)	GM GC	Silty gravels, poorly graded gravel-sand mixtures. Clayey gravels, poorly graded gravel-sand, clay mixtures
SANDS More than half of coarse fraction is smaller than No. 4 sieve size.	CLEAN SANDS	SW SP	Well graded sand, gravelly sands, little or no fines Poorly graded sands, gravelly sands, little or no fines
	SANDS WITH FINES (Appreciable amount of fines)	SM SC	Silty sands, poorly graded sand and silty mixtures. Clayey sands, poorly graded sand and clay mixtures.
II. FINE GRAINED, more than half of material is smaller than No. 200 sieve size.			
	SILTS AND CLAYS Liquid Limit less than 50	ML CL OL	Inorganic silts and very fine sands, rock flour, sandy silt or clayey-silt-sand mixtures with slight plasticity Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays Organic silts and organic silty clays or low plasticity
	SILTS AND CLAYS Liquid Limit less than 50	MH CH OH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts. Inorganic clays of high plasticity, fat clays Organic clays of medium to high plasticity
III. HIGHLY ORGANIC SOILS			
		PT	Peat and other highly organic soils.
 - Water level at time of excavation or as indicated			
SAMPLES		TESTS	
TW	- Driven thin walled ring or tube sample	pH	- pH & Resistivity
SPT	- Standard Penetration Test Sample	PI	- Plasticity Index
CK	- Chunk sample	RC	- Relative Compaction
BK	- Bulk sample	RV	- R-Value
NR	- No recovery	SA	- Sieve Analysis
	TESTS	SC	- Sand Cone
AL	- Atterberg Limits	SF/CL	- Sulfate & Chloride CON
-	Consolidation	SP	- Standard Penetration Sample
DS	- Direct Shear	SPT	- Standard Penetration Sample
EI	- Expansion Index	ST	- Shelby Tube
MAX	- Maximum Density	TX	- Triaxial Compression
MD	- In-Situ Moisture & Density	UC	- Unconfined Compression
MS	- Maximum Size of Particle		
FORT ROSECRANS CEMETERY – EAST COLUMBARIUM SITE			
BY:		MF/PC	DATE: 6/25/2010
PROJECT NO:		4307-10-0009	PLATE: A1



LOG OF BORING 4

Drilling Contractor:	Tri-County Drilling	Date Drilled:	6/18/2010
Drill Rig:	CME 75	Logged by:	Steve Wagoner
Driller:	Bill Orton	Surface Elevation (ft. msl):	367
Hammer Type/Weight/Drop:	Automatic/140 lbs/30"	Depth to Water (ft):	N/A

DEPTH (ft)	ELEVATION (ft. msl)	USCS	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES		PENETRATION (blows-ft. of drive)	MOISTURE (%)	D Y UNIT WT. (pcf)	LABORATORY
				D IVE	SAMPLE BULK				
2	365	SM	UNDOCUMENTED FILL- Dark brown, moist, loose, fine SILTY SAND; some wood debris*			8			
4				TW					
6			@5': Becomes brown , very loose, no wood debris observed	SPT		3			
8	360								
10		SM	LINDAVISTA FORMATION- Reddish-brown, moist, medium dense, fine SILTY SANDSTONE; lightly cemented						
12	355		Bottom @ 11.5'	TW		37			
14			No groundwater encountered						
16			Backfilled with cuttings						
18	350		*Note: Non-detrimental fill debris consists mainly of asphalt fragments (typically 2"-6") with occasional brick and/or concrete fragments unless otherwise noted.						
20									



FORT ROSECRANS CEMETERY EAST COLUMBARIUM SITE

BY: SW/MF/PC	DATE: 6/25/2010
PROJECT NO: 4307-10-0009	PLATE: A5

LOG OF BORING 5

Drilling Contractor:	Tri-County Drilling	Date Drilled:	6/18/2010
Drill Rig:	CME 75	Logged by:	Steve Wagoner
Driller:	Bill Orton	Surface Elevation (ft, msl):	365
Hammer Type/Weight/Drop:	Automatic/140 lbs/30"	Depth to Water (ft):	N/A

DEPTH (ft)	ELEVATION (ft. msl)	USCS	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES		PENETRATION (blows/ ft. of drive)	MOISTURE (%)	D Y UNIT WT. (pcf)	LABORATORY TESTS
				D IVE SAMPLE	BULK				
2		SM	UNDOCUMENTED FILL- No sampling in upper 10', see log of adjacent Test Pit 1*						
4			*Note: Non-detrimental fill debris consists mainly of asphalt fragments (typically 2"-6") with occasional brick and/or concrete fragments unless otherwise noted.						
6	360								
8									
10	355	SM	UNDOCUMENTED FILL- Brown, moist, loose, fine SILTY SAND with GRAVEL and non-detrimental fill debris	SPT		6			
12			@12': Becomes reddish-brown						
14		SM	LINDAVISTA FORMATION- Reddish-brown to brown, moist, loose to medium dense, fine SILTY SANDSTONE; lightly cemented to uncemented						
16	350			TW		12			
18		SM	CABRILLO FORMATION- Brown to gray, moist, dense, fine SILTY SANDSTONE						
20			Bottom @ 20' No groundwater encountered Backfilled with cuttings	SPT		50			



FORT ROSECRANS CEMETERY EAST COLUMBARIUM SITE

BY: SW/MF/PC	DATE: 6/25/2010
PROJECT NO: 4307-10-0009	PLATE: A6

APPENDIX D
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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