ADDENDUM NO. 1

BID-CONTRACT DOCUMENTS
FOR
Bid No. 2029
Orange Coast College Swap Meet Restroom Building

COAST COMMUNITY COLLEGE DISTRICT
COSTA MESA, CALIFORNIA
COUNTY OF ORANGE

COAST COMMUNITY COLLEGE DISTRICT
1370 Adams Avenue
Costa Mesa, CA  92626
The following changes, additions, deletions, clarifications, or corrections shall become part of the Bid-Contract Documents for Coast Community College District Bid No. 2029, Orange Coast College Swap Meet Restroom Building. All other terms, specifications, and conditions remain the same.

Modifications are identified by “clouds” and the following: Deletions strikethrough. Insertions/Substitutions italic-underlined.

Item 1: **Add** the attached responses to questions through May 16, 2014.

**END OF ADDENDUM**

Prepared By:

[Signature]

John Eriksen
Director of Purchasing
1. Is detail 6/S.1 showing single angle connection typical for all channel connections? If so, this would replace Detail 3/S.1 Beam 1 Side Connection. Please confirm. No, follow details referenced on the drawings (for example: 3/S.1 is reference in section detail 26/S1).

2. Are 1 ½” pipe braces on column center line? If so, what is the location of channel and bracing plate per 14/S.1? Correct, pipes/gusset plates are aligned with column web/centerlines (architect to provide any remaining requested dimensions). See 02/a3.2 for location of gusset plate from face of column.

3. It was understood from the bidder’s job walk that any irrigation and/or landscaping work would be completed by the owner. Please confirm. Yes, OCC will handle all the landscaping and irrigation for this project. The contractor will be responsible to make sure all measures possible are used to protect existing trees adjacent to District Transportation. All areas that are trenched are to be returned to grade and raked cleaned of debris.

4. The following Specification Sections included in the Project Manual are apparently not applicable to this project. Please confirm.
   - Section 02441 Irrigation System Not Applicable
   - Section 02480 Landscape Planting Not Applicable
   - Section 09110 Non-load bearing wall framing system Not Applicable
   - Section 09200 Lath and Plaster Not Applicable
   - Section 09250 Gypsum Wallboard Required for ceiling construction

5. Note 7 on Sheet E2.1 advises to stub down a 1 ‘-Yz” conduit to the Janitor Room space and connect to 1 "Yz" conduit routed from the Horticulture Warehouse Building. Site Plan 1/E2.1 shows 2" conduit being routed from an existing communication pull box. Please clarify if the 2" is to be 1 ‘-Yz” or the 1 ‘-W’ is to be 2”? Provide 1 ½” conduit from roof mounted 12”x12”x4” pull box. Terminate conduit at +12” AFF in Janitor #103.

6. Note 8 on Sheet E2.1 refers to 1 ‘-Yz” telephone stub up location. Electrical site plan 1/E2.1 shows a 2” conduit for the telephone/fiber cable. Please clarify. Provide 2” conduit from (E) communication pull box to Janitor #103.

7. Water Heater (WH-1) is scheduled on Sheet P-0.1asa208 volt single phase unit. Panel R is shown in Sheet E0.1 as a 120/208 volt three (3) phase panel. Please confirm that specified WH-1 is convertible to a three (3) phase. Provide 80A/2 pole circuit breaker in Panel R. Provide 1”C, 2#4, 1#8G feeder between panel and water heater.

8. It appears that Note 6 on Partial Site Plan 02/A2.1 is for a light pole. No Note 6 is included in Site keynotes. Please confirm. Remove callout.
9. Wall Section 04/A3.2 shows masonry continuous from footing to roof steel at trough drain. Detail 24/A3.2 shows a 6” curb at trough drain. Detail 25/S-I shows a 3’ -1/2” minimum pier off the top of footing then masonry. Please clarify. Wall section is generic, follow architectural and structural details.

10. Foundation Note 1 refers to Geotechnical Report by Willdan Geotechnical dated May 20, 2013. It would be helpful if a copy was distributed to all bidders. Geotechnical report is attached.

11. Geotechnical Note 1 on Sheet CO.I states that soils are potentially high expansive and over-excavation to a depth of 5’ may be required. Please provide soils report or guidance to the depth of over-excavation. See attached Geotechnical report. The note referencing the soils report was put on the plan last year as directed by OCC. The geotechnical engineer should have a technician at the site to review the over excavation to meet their requirements and to verify the depth of excavation. The technician/soils engineer may have additional requirements based on field observations to mitigate against expansive soils.

12. Paragraph 2.06 of the Summary of Work, 01010-5 states that the geotechnical investigation proposal and a copy of the report are available for inspection at the office of the contractor. Who is the contractor that has this report so that copy may be obtained? See questions #10 and #11.

13. The pad dimension is shown on Detail “A” on Sheet C0.1 as 62.32. The components that are added to the pad to reach finish floor elevation are the following:
   - Sand Base .33 (4”)
   - Concrete SOG .42 (5”)
   - Epoxy Floor Finish .02 (114”)
   - Total: .77 (9”+)
   - Sheet CO.I Pad to FF .67 (1”+ delta)

   Please advise if sand base should be reduced to 3”.
   It appears that the slab section thickness has been increased as pointed out in this RFI. The pad elevation should be adjusted based on the latest structural slab section thickness so that the FF elevation as shown on the plan is held. Therefore the plan elevation should be set to elevation 61.55 as I have marked on the attached grading plan.

14. The cover between the bottom of the sewer pipe to the top of the electrical utility conduit is approximately 1’ -5”. Is this adequate cover for the utility conduit? Please advise. The 1’ -5” clearance between the sewer and electrical conduit is acceptable.

15. Please advise if an inspection field office is to be provided as noted in Article 41 of the General Conditions dated July 2013. No.
16. Please advise if temporary fence needs to be provided at the curb of area that is being trenched for 670 LF of 2'-1/2” conduit for power and 440 LF for 2” conduit for telephone/fiber cable.  Yes.

17. The CI Fixture is not listed on the lighting fixture schedule. Please advise if it is the same as the C fixture that is listed.  Yes.
June 25, 2013

Mr. Jerry Marchbank  
Coast Community College District  
1378 Adams Avenue  
Costa Mesa, Ca 92626

Subject: Geotechnical Investigation  
Proposed Restroom Building at Orange Coast Community College  
S Street, 1000 feet South of Adams Avenue, Costa Mesa, California  
Willdan Geotechnical Project No. 101655-5000

Dear Mr. Marchbank,

This report presents results of our geotechnical investigation for the proposed restroom building located at S Street, 1000 feet south of Adams Avenue in Orange Coast Community College, Costa Mesa, California.

Based on the results of our investigation, the proposed development is feasible from a geotechnical standpoint. This report contains our conclusions and recommendations for the design and construction of the proposed restroom building in the above subject sites.

We appreciate the opportunity to assist you and look forward to future projects. If you have any questions, please contact us.

Respectfully submitted,

WILLDAN GEOTECHNICAL

Ross Khiabani, PE, GE  
Director of Geotechnical Services  
C 37156, GE 2202

Distribution:  1. Addressee (3 wet stamped sets and pdf copy via e-mail)  
2. Mr. Chris Polaski of Bundy-Finkel Architects (pdf copy via e-mail)
REPORT

GEOTECHNICAL INVESTIGATION
PROPOSED RESTROOM BUILDING AT S STREET, 1000 FEET SOUTH OF ADAMS AVENUE, ORANGE COAST COMMUNITY COLLEGE
COSTA MESA, CALIFORNIA

Prepared for

Coast Community College District
1378 Adams Avenue
Costa Mesa, Ca 92626

Prepared by

Willdan Geotechnical
1515 South Sunkist Street, Suite E
Anaheim, California 92806
Willdan Geotechnical Project No. 101655-5000

June 25, 2013
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APPENDICES

Appendix A. Boring Logs
Appendix B. Laboratory Test Results
Appendix C. Typical Retaining Wall Backfill
1.0 INTRODUCTION

This report presents the results of our geotechnical engineering investigation performed for the proposed restroom building located at S Street, 1000 feet south of Adams Avenue in Orange Coast Community College, Costa Mesa, California. Site location map is provided in Figure 1, Site Location Map. The approximate locations of the soil borings advanced for this investigation are also shown on Figure 2, Boring Location Plan.

This report includes geotechnical findings, conclusions, and recommendations with respect to site preparation, earthwork procedures, site seismic hazards, and foundation design parameters at the above subject project. A boring log key and the boring logs are presented in Appendix A. The geotechnical laboratory test results are in Appendix B. Retaining wall backfill and typical subdrain details for conditions of native soil, imported sand, or crushed rock are provided in Appendix C.
2.0 SCOPE OF SERVICES

This investigation was conducted to explore and evaluate the site soil engineering conditions to depths that may significantly influence the proposed construction. Our scope of services included the following:

- A site reconnaissance by a member of our engineering staff to evaluate the surface conditions at the project site.
- Review of selected published geologic maps, reports and literature pertinent to the site and surrounding areas.
- A field investigation consisting of drilling two (2) borings to the maximum depth of 51.5 feet bgs in order to evaluate subsurface conditions at the subject project sites.
- Performance of laboratory tests on representative soil samples obtained from the borings to evaluate the physical and engineering properties of the subsurface soils and corrosivity of the soil as it pertains to soil/cement reactivity and buried metals.
- Engineering evaluation of the data obtained from the field investigation and laboratory testing program.
- Preparation of this report summarizing our investigation and findings, results of geotechnical laboratory testing, and our conclusions and recommendations for the geotechnical aspects of project design and construction.

Environmental assessment services, such as chemical analysis of soil and groundwater for hazardous substances, were not included in our scope of services. Geotechnical services related to soil corrosivity have been limited to screening of limited representative soil sample collected from within the near surface zone. Thorough corrosivity evaluation with respect to the potential for on-site soils to affect cement (concrete) or buried metal pipe was not included in our scope of services for this project.
3.0 SITE DESCRIPTION AND PROPOSED DEVELOPMENT

The project site is located at Orange Coast Community College in the City of Costa Mesa, California. The latitude and longitude at the approximate center of the site are 33.6721° N and 117.9139° W, respectively. At the time of our investigation, the site was covered by vegetation and concrete pavement.

As we understand from the plans and information provided to us, the project consists of construction of a new restroom building. The proposed building will be supported by conventional shallow footings. Site development will include site grading, building pad preparation, and underground utility installation. We were not provided with finish grade information, but based on the area topography site grading is expected to consist of shallow cuts and fills not exceeding two to three feet in height. In the event the structural or grading details are inconsistent with the final design, we should be notified so that the potential impacts of the changed conditions can be reviewed and revised recommendations can be prepared if necessary.
4.0 GEOLOGY

4.1 GEOLOGICAL SETTING

The subject site is located in an area of widely diverse terrain at the southern margin of the Los Angeles Basin and is within the area covered by the USGS Newport Beach 7.5-minute Quadrangle. This area is situated on a broad mesa that extents southwestward to join the San Joaquin Hills. Commonly known as Newport Mesa, this upland has deeply dissected by stream erosion, resulting in moderate to steep bluffs along the upper Newport Bay estuary. The oldest geologic units mapped in the area are the tertiary marine rocks, marine siltstone and siliceous shale of the Monterey Formation, sandstone of the Niguel Formation, and siltstone of Capistrano Formation. The surface distribution of Quaternary to Holocene sediments also suggests that Santa Ana River has deposited sediments across the subject project site.

4.2 REGIONAL AND LOCAL FAULTS

Maps of designated Earthquake Fault Zones have been published by the California Geological Survey in accordance with the Alquist-Priolo Special Studies Zones Act of 1972, which regulates development near active faults. Based on our review of these maps, the site does not lie within an Alquist-Priolo (AP) Earthquake Fault Zone. In general terms, fault related ground surface displacement; either sudden as in an earthquake or gradual as in fault creep is caused when strain energy in rocks is released by movement along a plane of weakness. Surface rupture usually occurs along the traces of known active or potentially active faults, although many historic events have occurred on faults not previously known to be active.

Strong to severe ground shaking will be experienced in the project area if a large magnitude earthquake occurs on one of the nearby active or potentially active faults and should be anticipated within the life expectancy of the structure.

The site is located in southern California which is considered by most geologists as seismically active. Seismic risk can be considered high as compared to other areas of California because of the proximity to local active faulting. These faults are classified by USGS (USGS, 2009) and lists known active and potentially faults. Several faults are in the general area of the project site and are summarized below:

1. The Elsinore Fault zone (Whittier section) is considered active or potentially active and is located approximately 18 miles north-northeast of the project site and is capable of producing an earthquake of magnitude 6.9.
2. The Newport Inglewood Fault zone (S. Los Angeles Basin section-southern) is considered active or potentially active and is located approximately 3 miles south-southwest of the project site and is capable of producing an earthquake of magnitude 7.2.

3. The Palos Verdes Fault is considered active or potentially active and is located approximately 15 miles southwest of the project site and is capable of producing an earthquake of magnitude 7.2.

4. The San Andreas (San Bernardino) Fault is considered active or potentially active and is located approximately 49 miles north of the project site and is capable of producing an earthquake of magnitude 7.9.

5. The San Joaquin Fault is considered active or potentially active and is located approximately 2 miles north northeast of the project site and is capable of producing an earthquake of magnitude 7.

6. The Sierra Madre Fault zone (Sierra Madre D) is considered active or potentially active and is located approximately 32 miles north of the project site and is capable of producing an earthquake of magnitude 7.2.

The closest known active fault is the San Joaquin Fault, located approximately 2 miles north-northeast of the site.
5.0 GEOTECHNICAL INVESTIGATIONS

5.1 FIELD INVESTIGATION

Field exploration for this investigation consisted of drilling and sampling two (2) borings to the maximum depth of 51.5 feet below ground surface. Approximate locations of borings are shown on Figure 2, Boring Location Plan. These locations were selected by our personnel in the field by measuring from the limits of existing site features. Prior to field exploration, a site visit was performed to mark the boring locations and evaluate access conditions for drilling equipment. Underground Service Alert (USA) of Southern California was then notified for clearance of underground utilities in the vicinity of the subsurface exploration locations (USA Ticket Numbers of A31210201).

Soil borings for the current investigations were advanced using a truck-mounted B-61 rig equipped with 8-inch diameter hollow-stem augers. Relatively undisturbed and bulk soil samples were collected from each soil boring during drilling. Bulk samples were collected from auger cuttings obtained from within the near-surface soils. At select intervals throughout the boring depths, relatively undisturbed soil samples were collected by driving a three-inch outside diameter Modified California Sampler lined with brass rings. The samplers were driven into the underlying soil to a depth of 18 inches, or the interval noted on the boring logs, with a 140-pound hammer falling 30 inches. The number of blows required to drive the sampler was recorded for each 6-inch penetration interval and is shown on the boring logs. Soil samples were retained for possible laboratory testing. The number of blows required to drive the sampler the last 12 inches was used to estimate the in-situ relative density of granular soils and to a lesser degree of accuracy, the consistency of cohesive soils.

Visual classification of the soils encountered in our exploratory borings was made in general accordance with the Unified Soil Classification System (ASTM D2487). A key for the classification of the soils (USCS classifications) along with the logs of our borings is included as Appendix A.

Upon completion of drilling, the borings were backfilled with soil cuttings and tamped. Pavement surfaces were repaired using quick cement. Soil samples collected from the field were delivered to Willdan’s laboratory for testing.
5.2 LABORATORY TESTING

Laboratory tests were performed on selected soil samples to evaluate their physical characteristics and engineering properties. Laboratory testing included determination of in-situ moisture and density, Atterberg limit, expansion index, and shear strength characteristics for soil samples collected from various depths. Corrosion potential of soils collected from within the shallow subsurface was also determined. Laboratory tests were conducted in general accordance with American Society for Testing of Materials (ASTM) Standards or California Test Methods. The in-situ dry density and moisture content test results are shown on the boring logs. The remaining laboratory test results are presented in Appendix B, Laboratory Test Results.

5.3 SUBSURFACE CONDITIONS

Based on our field observations of material type and laboratory test data, the upper soil layer, extending to approximately 38 feet deep, consists of stiff to very stiff sandy clay/ fat clay with interbedded thin layer of silt. This layer is underlain by dense silty sand to the maximum depth explored at 51.5 feet bgs.

Based on our findings, the subsurface conditions encountered appear typical of those found in the geologic region of the site. The above is a general description of soil conditions encountered at the site in the borings drilled for this investigation. For a more detailed description of the soil conditions encountered, please refer to the boring logs in Appendix A.

5.4 SOIL EXPANSION

The expansion index (EI) test performed on a sample obtained from the borings indicates that the near surface sandy clay soils have an expansion index of 68. Therefore, according to ASTM D4829, the shallow on-site soils are determined to have medium expansion potential.

5.5 GROUNDWATER

The subject area is in the east part of Newport Quadrangle where historically highest groundwater was identified as 30 feet (CGS, 1997). The borings conducted for the current investigation were monitored for visible signs of free groundwater during and immediately after completion of the borehole. Groundwater was not encountered during our drilling operation in May 6, 2013.

Depth to groundwater can be expected to fluctuate both seasonally and from year to year. Fluctuations in the groundwater level may occur due to variations in precipitation, flow in nearby creeks, irrigation practices at the site and in the surrounding areas, climatic conditions, pumping from wells, and possibly as the result of other factors that were not evident at the time of our
investigation. Because of the type of the proposed developments and expected depth of grading and/or excavation, it is unlikely that groundwater would be encountered during the course of construction for the proposed developments.
6.0 SEISMIC HAZARDS

6.1 SITE CHARACTERIZATION – SITE CLASS

The subsurface soil profile at the site can be classified from a seismic standpoint based on the conditions encountered in our exploratory borings, and anticipated within the upper 100 feet of the site based on geologic mapping, as being a stiff soil with undrained shear strength of at least 1,000 pounds per square foot (psf) and SPT N values of 15 to 50 blows per foot. Based on the soils encountered within the upper 50 feet of the subject site and with consideration of the geologic units mapped in the area, it is our opinion that the site soil profile corresponds to Site Class D as per Table 1613.5.2 of the California Building Code (CBC, 2010).

6.2 GROUND SHAKING

Although ground rupture is not considered to be a major concern at the subject site, the site will likely be subject to moderate to severe seismic shaking during its lifetime. Some degree of structural damage due to stronger seismic shaking should be expected at the site, but the risk can be reduced through adherence to seismic design codes. Using the seismic hazard assessment program developed by the United States Geologic Survey (USGS, 2009), the mean value of the Peak Ground Acceleration (PGA) under the Maximum Considered Earthquake (MCE) for the site was estimated at 0.65g. The design ground motion, taken as two-thirds of the MCE ground motion, is thus 0.43g.

6.3 SOIL LIQUEFACTION

Soil liquefaction is a state of temporary soil particle suspension caused by loss of strength due to pore pressure increase resulting from cyclic stress application induced by earthquakes, and the resultant drop in effective stress and soil shear strength. Liquefaction normally occurs in saturated granular soils, such as sands, in which the strength is purely frictional. Soils most susceptible to liquefaction are saturated, loose, uniformly graded, fine-grained sand deposits. However, liquefaction has occurred in soils other than clean sands. Silty sands and sandy silts have also been reported to be susceptible to liquefaction or partial liquefaction. The occurrence of liquefaction is generally limited to soils located within about 50 feet of the ground surface. Primary factors affecting the potential for a soil to undergo liquefaction include:

1) Depth to groundwater;
2) Soil type;
3) Relative density of the soil and initial confining (overburden) pressure;
4) Intensity and duration of ground shaking.
Potential problems associated with soil liquefaction include ground surface settlement, loss of foundation bearing support strength, and lateral spreading. Ground surface settlement due to densification of the liquefied soils can be approximated using procedures developed by Tokimatsu and Seed (1987) and Ishihara and Yoshimine (1992). While confinement of saturated sand layers is required for liquefaction to occur, a phenomenon referred to as sand boils is the primary cause for catastrophic failure of building foundations. Sand boils occur when the sudden compression of groundwater in a layer of saturated, clean, loose sand builds up sufficient pressure to rupture up through the upper soil mantle to the ground surface. When this occurs, displacement of the liquefied sand results in the sudden loss of support of structures supported by shallow foundations.

The project site has not been mapped as being within a zone susceptible to liquefaction as designated by the State of California. Site-specific liquefaction potential was not evaluated because the data obtained from the present geotechnical investigation indicate that the liquefaction potential in the subject project site is unlikely. This is due to the fact that the predominant clay/silt materials encountered in the subject project site are not likely to undergo liquefaction.

6.4 SEISMICALLY INDUCED SETTLEMENT OF UNSATURATED SANDS

In addition to the settlement of sand deposits that undergo liquefaction, strong seismic shaking can also cause settlement or compaction of sands above the groundwater as well. Seismic-induced settlement of sands above the groundwater can potentially result in settlement of the ground surface. The estimated settlement under the design seismic scenario for stiff to hard clay/silt encountered in the shallow subsurface is estimated to be less than one-third of an inch (≈ 8 mm) everywhere within the project area.

6.5 LATERAL SPREADING

Lateral spreading happens when surficial soil moves in a direction parallel to the ground surface due to liquefaction of underlying subsurface soils layers. Lateral spreading usually occurs where the ground surface has a slope less than 6 percent and may result in damage to structures or other improvements due to differential lateral movements (Naeim, 1989). Considering the site condition which is not susceptible to liquefaction, lateral spreading is not likely to occur at the project site during seismic events.
6.6 GROUND LURCHING

Ground lurching is movement of the ground surface during seismic event, resulting in cracks and ridges developing perpendicular to the slope face. Areas underlain by thick alluvium with loose granular soils or clay soils with high moisture are susceptible to ground lurching. Ground lurching often causes damage to lightly loaded structures such as pavements, walkways, pipelines, and other near-surface improvements located within the failure zone. The shallow subsurface in the project area consists predominately of stiff to hard clay/silt. These areas are not susceptible to ground lurching during the design earthquake event.
7.0 CONCLUSIONS AND RECOMMENDATIONS

7.1 GENERAL

Based on our geotechnical investigation, the proposed developments are feasible from a geotechnical viewpoint, provided the recommendations contained in this report are implemented in the design and construction of the project.

7.2 EARTHWORK

Earthwork should be performed in accordance with the latest edition of the Standard Specifications for Public Works Construction (Greenbook, 2012) and grading requirements of the City of Costa Mesa. Excavations and cuts should be inspected during grading.

7.2.1 Site Preparation

Any soils disturbed during site clearing operations in the construction areas should be removed down to undisturbed soils. The exposed subgrade soils can then be prepared to receive any required engineered fills for the grading of the site. Based on our field investigations, over-excavation and backfilling of up to 2 feet below existing ground surface may be required in building and pavement areas. If unsuitable soils are encountered during excavation, additional excavation will be required to remove the unsuitable materials to expose a firm and unyielding surface.

Prior to construction, vegetation, trash, and debris should be cleared and disposed of offsite. During grading, the contractor should take all necessary measures to protect existing utilities within the grading limits. All abandoned utilities encountered should be drained for all content, if any, and properly capped.

Prior to placing fill, the subgrade should be scarified to a depth of 8 inches, moisture-conditioned to approximately 3 to 5 percent above optimum and compacted to at least 90 percent relative compaction. The finish subgrade should be maintained moist at all time prior to placing fill or other improvements.

Unless stated otherwise, all fill materials should be placed in loose lifts of 8 inches or less, moisture-conditioned within 3 percent above optimum and compacted to at least 90 percent relative compaction of the maximum density as determined by the ASTM D1557 test procedure. Compaction should be verified by observation, probing, and testing by a geotechnical
consultant’s representative.

Moisture conditioning is intended to adjust the soil moisture content through either the addition of water where soil moisture is below the recommended level or by allowing evaporation to occur where elevated soil moisture contents are present. Moisture conditioning operations should include sufficient mixing of the materials to produce a relatively uniform soil mixture and moisture condition. Moisture conditioning should be performed prior to the application of compaction effort.

Once the subgrade and fill soil have been moisture conditioned and compacted, the soil should not be allowed to dry out prior to additional fill placement or concrete placement at finished grade. If it is dried out prior to compaction of the fill or prior to foundation and slab-on-grade construction, reprocessing of the soil is required in order to reestablish the recommended soil moisture content. Even with proper site preparation, there will be some effects of soil moisture change on concrete flatwork.

When the work is interrupted by heavy rains, fill operations shall not be resumed until the Geotechnical Engineer indicates that the moisture content, density and stability of previously placed fill are as specified. All soft or wet subgrade soil encountered during construction should be stabilized prior to the placement of new fill and further construction. If earthwork is performed during or soon after periods of precipitation or in late winter to early spring, the subgrade soils may be near their saturation level. Wet to saturated soils may become unstable or “pump” under dynamic loading such as equipment movement during grading and may not respond to densification techniques. Typical remedial measures include discing and aerating the soil during dry weather, mixing the soil with dryer materials, removing and replacing the soil with an approved fill material, or mixing the soil with an approved lime or cement product. Our firm should be consulted prior to implementing remedial measures to observe the unstable subgrade conditions and provide appropriate recommendations.

7.2.2 Fill Materials

The on-site near surface clayey materials have been determined to be medium expansive and therefore are not suitable for being used for backfilling purposes. Imported granular soils may be used in the required compacted fills and backfills. Imported materials should contain sufficient fines (binder material) so as to be relatively impermeable and result in a stable subgrade when compacted. The imported materials should also be non-expansive, with an EI less than 30 and free of organic materials, debris, and cobbles larger than 3 inches, with no more than 25 percent of materials being larger than 2 inches in size and no more than 25 percent passing #200 sieve. Within the upper 2 feet of fills and utility trench backfills, the materials should be free of
particles greater than 2 inches in size. A bulk sample of potential import material, weighing at least 30 pounds, should be submitted to the Geotechnical Consultant at least 48 hours before fill operations. All proposed import materials should be approved by the Geotechnical Consultant prior to being placed at the site.

The on-site soils may be used for backfilling purposes provided the soils expansivity is considered and incorporated in design of the slab-on-grade and foundations. We recommend performing Expansion Index (EI) test on a sample obtained from the soils intended to use for backfill to determine the EI of the potential backfill material prior to final design. As an alternative it is recommended to pre-saturate the subgrade soils to 3 to 5 percent above optimum, immediately prior to placement of the concrete slab, to a minimum depth of 12 inches.

7.2.3 Utility Trench Bedding and Backfill

Bedding materials consisting of sand, gravel, or crushed aggregate should be used to backfill around utility pipes to approximately 1 foot above the top of a pipe. Onsite soils which have a Sand Equivalent (SE) of 30 or greater can also be used as bedding material. Prior to placing the pipes, the pipe trench subgrade should be observed by a representative of the project geotechnical engineer. If the exposed subgrade is loose or unstable, the unsuitable subgrade soil must be excavated and replaced with bedding material. Bedding must be placed uniformly on each side of the pipe and mechanically compacted. Flooding or jetting to densify the bedding materials is not allowed due to the clayey nature of onsite soils. The fill should be placed in loose lifts not to exceed 8 inches, moisture-conditioned to 3 to 5 percent above optimum, and mechanically compacted to at least 90 percent relative compaction in accordance with ASTM D1557. Thinner lifts may be necessary to achieve the recommended level of compaction of the backfill due to equipment limitations.

Trenches in pavement areas should be capped with at least 12 inches of compacted, on-site soil similar to that of the adjoining subgrade. The upper 12 inches of trench backfill in areas to be paved should be compacted to at least 95 percent relative compaction. Special care should be taken in the control of utility trench backfilling in the pavement areas. Poor compaction may cause excessive settlement resulting in damage to the pavement structural section.

Where trenches exceed ten feet in depth from design finished grade, the percent relative compaction on cohesive soils may need to be increased to reduce the potential for trench backfill settlement. Should these conditions exist, compaction requirements should be reviewed by the Geotechnical Engineer as a part of the plan review process.
7.2.4 Temporary Excavation

Temporary excavations must be properly sloped or shored. Based on the earth materials encountered in our borings, excavation of 5 feet or less in depth may be performed with vertical sidewalls. Deeper excavation up to a depth of 15 feet can be accomplished in accordance with the Occupational Safety and Health Administration (OSHA) requirements for Type B soils.

The contractor is responsible for maintaining the stability of the cuts and personnel safety in the field during construction. All excavations shall be performed in accordance with applicable requirements established by the State, County, or local government. The regulatory requirement may supersede the recommendations presented in this section. The Geotechnical Engineer of Record’s representative should be present during all excavations.

7.3 SEISMIC DESIGN PARAMETERS

The site class per Table 1613.5.2, of the 2010 CBC is based upon the site soil conditions. It is our opinion that Site Class D is most consistent with the subject site soil conditions. For design of the structures based on the seismic provisions of the 2010 CBC, we recommend the parameters in the following Table 1:

<table>
<thead>
<tr>
<th>Seismic Item</th>
<th>Value</th>
<th>CBC Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Site Class</td>
<td>D</td>
<td>Table 1613.5.2</td>
</tr>
<tr>
<td>Site Coefficient $F_a$</td>
<td>1.0</td>
<td>Table 1613.5.3 (1)</td>
</tr>
<tr>
<td>$S_s$</td>
<td>1.666</td>
<td>Figure 1613.5 (3)</td>
</tr>
<tr>
<td>$S_{MS}$</td>
<td>1.666</td>
<td>Section 1613.5.3</td>
</tr>
<tr>
<td>$S_{DS}$</td>
<td>1.111</td>
<td>Section 1613.5.4</td>
</tr>
<tr>
<td>Site Coefficient $F_v$</td>
<td>1.5</td>
<td>Table 1613.5.3 (2)</td>
</tr>
<tr>
<td>$S_1$</td>
<td>0.597</td>
<td>Figure 1613.5 (4)</td>
</tr>
<tr>
<td>$S_{MI}$</td>
<td>0.895</td>
<td>Section 1613.5.3</td>
</tr>
<tr>
<td>$S_{D1}$</td>
<td>0.597</td>
<td>Section 1613.5.4</td>
</tr>
</tbody>
</table>

Site Coordinates: Latitude: $33.6721\degree$ N Longitude: $117.9139\degree$ W
7.4 FOUNDATION DESIGN CRITERIA

7.4.1 General

It is our opinion that the proposed building may be supported on conventional spread and/or strip footings. The entire footprint area of the proposed structure should be supported on at least 2 feet of the engineered fill that extends to a minimum depth of 3.5 feet below the lowest adjacent finished grade, whichever provides the deeper fill.

Over-excavation shall laterally extend at least 5 feet from outer faces of the perimeter footings in all directions, where possible. The over-excavated area shall be backfilled to the designated grade. The backfill materials shall conform to the requirements provided to “Fill Materials” section of this report. The backfill materials shall be placed and compacted as recommended in “Site Preparation” section of this report.

After removal of compressible soils, as required and prior to placement of backfill, the bottom of removal shall be observed and confirmed to be competent by the Geotechnical Engineer of Record. Following the over-excavation, we recommend that the areas to receive engineered fill be scarified to a minimum depth of 8 inches, moisture-conditioned to approximately 3 to 5 percent above optimum and compacted to at least 90% of the maximum dry density obtained per ASTM D1557.

By the time of preparation of this report, we have not been provided with the order of the anticipated structural loads applicable on the foundations for the proposed structure. The above recommendations are based on the assumption that the imposed column loads will be less than 30 kilo pounds (kips), and imposed continuous footing loads will be less than 2 kips per foot (kpf) for structures and retaining walls.

Bearing Capacity

Spread and strip footings should be at least 24 and 18 inches wide, respectively. The footings may be designed to impose a maximum allowable pressure of 2,500 pounds per square foot (psf) due to dead plus live loads. This value can be increased by 120 psf for every foot increase in width and by 250 psf for every foot increase in depth and to a maximum value of 4,500 psf. The bearing capacity may be increased by one-third for transient loads such as seismic or wind. The footings should be embedded at least 18 inches below the lowest adjacent grade.

In order to maintain adequate support for the foundations located adjacent to utility trenches, including existing utility trenches or other footings, should be deepened as necessary so that their bearing surfaces are below an imaginary plane having an inclination of 1.5 horizontal to 1.0.
vertical, extending upward from the bottom edge of the adjacent trench or footing.

**Settlements**

Based on the results of our investigation, total settlements due to building loads are expected to be less than one (1) inch, and maximum differential settlements are expected to be of the order of ½ inch over a 50-foot span.

### 7.5 CONCRETE SLAB-ON-GRADE

Concrete slab-on-grade should be constructed on compacted fill prepared per recommendations provided in above section. The slab-on-grade should be at least 5 inches thick and reinforced with No. 3 rebar at 18 inches on center. Concrete slab-on-grade may be designed using a maximum bearing pressure of 1000 psf. Concrete slabs should be underlain by a minimum 2 inches and maximum 4 inches of sand or granular material having a minimum sand equivalency of 30.

Frequent construction or control joints should be provided in all concrete slabs where cracking is objectionable. This can be done by installing contraction joint material as the concrete is placed or by saw cutting the fresh concrete. Contraction or weakened plane joints should extend slightly deeper than one-quarter the slab thickness to be effective. Control joints should be spaced a maximum of 30 times the slab thickness to reduce the potential for unsightly panel cracks as a result of soil displacement and concrete shrinkage. This would result in contraction joints at 10-foot centers for a four-inch thick slab. In the event that control or contraction joints are to be constructed by saw cutting of the slabs, saw cuts should be made by off-cut sawing within 4 to 12 hours after the initial hardening (not curing) of the concrete, as required by atmospheric conditions. The contractor should be responsible for monitoring of the concrete during initial set or hardening and to determine the optimal timing for cutting of the slabs.

Exterior concrete slab-on-grade may be subjected to periods of drying, and consequently, to edge effects due to the fluctuation in the moisture content of the subgrade soils along the outer edges of the slab. Deepened edge sections (also referred to as down turned curbs) will aid in reducing the potential for the shrinkage and swelling of the underling soils. By deepening the edge section to a minimum of 12 inches below the subgrade soils, there is less potential for soil moisture change below at least the perimeter of the slabs.

The above recommendations, including deepened edge sections and steel reinforcement are intended to help reduce the potential for distress in concrete slab, but may not eliminate such distress completely.
7.6 LATERAL EARTH PRESSURES AND FRICTION COEFFICIENTS

Anticipated lateral soil pressures and frictional coefficients for the design of the foundations and retaining structures at the site are listed in the following Table 2.

Active pressure should be used in computations for a retaining wall which is free to rotate at the top. At-rest pressures should be utilized if the wall is restrained from moving at the top, or in the case of below-grade walls of structures such as the planned inspection pits, or any utility and/or cable vault walls.

**TABLE 2. SUMMARY OF LATERAL LOAD/RESISTANCE FACTORS**

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Active Pressure</td>
<td>50 pcf</td>
</tr>
<tr>
<td>Passive Pressure</td>
<td>250 pcf</td>
</tr>
<tr>
<td>Friction Factor</td>
<td>0.35</td>
</tr>
<tr>
<td>Adhesion</td>
<td>850 psf</td>
</tr>
<tr>
<td>(foundations resting on compacted granular fill)</td>
<td></td>
</tr>
<tr>
<td>(foundations resting on native stiff, cohesive soils)</td>
<td></td>
</tr>
</tbody>
</table>

_Notes: A total unit weight of 127 pounds per cubic foot (pcf) was used in calculating the earth pressures._

The distribution of active and passive pressures on a cantilever wall is equal to that pressure developed by an equivalent fluid with a density as presented in the above table. The at-rest pressure distribution on a restrained wall is a trapezoidal distribution with zero value at the top and base of the wall, and maximum pressure of 35H psf at 0.2H and 0.8H distance from the base of the wall, where H is the retained height in feet.

In addition to the static lateral earth pressures, all cantilever retaining walls with retained height greater than 12 feet should be designed to support an additional seismic earth pressure. The recommended seismic earth pressure acting on retaining walls may be considered to be in the shape of an inverted triangle with the resultant acting at two-thirds the height of the wall above the base of the wall. The inverted triangular pressure distribution may be evaluated using a pressure imposed by an equivalent fluid having a density of 15 pcf.

The top one foot of the subgrade should be deleted in passive pressure computations for building foundations and buried structures. For computations of total lateral resistance of a buried structure, the frictional resistance against sliding at the base of the footing can be added to the passive resistance of the vertical face to compute the total lateral resistance.
A drainage system should be provided behind the walls to reduce the potential for development of hydrostatic pressure. Retaining wall backfill and typical subdrain details for conditions of native soil, imported sand or crushed rock are provided in Appendix C.

For stability against lateral sliding which is resisted solely by the passive pressure, we recommend a minimum safety factor of 1.15. For stability against lateral sliding which is resisted by the combined passive and frictional/adhesive resistance, a minimum safety factor of 2.0 is recommended. For lateral stability against seismic loading conditions, we recommend a minimum safety factor of 1.1.

Retaining Wall Backfill

All the backfill should be placed in layers which, when loose, should not exceed 8 inches per layer, and compacted to a minimum relative compaction of 90% of maximum density per ASTM D1557. Subdrain systems shall be installed to prevent hydrostatic pressure build-up acting as an additional lateral load. The Geotechnical Consultant may recommend additional subdrains and/or changes in subdrain extent, location, grade, or material depending on conditions encountered during grading. Retaining wall backfill and typical subdrain details for conditions of native soil, imported sand, or crushed rock are provided in Appendix C.

7.7 SURFACE DRAINAGE

Inadequate control of run-off water and/or heavy irrigation after construction of the proposed developments may lead to adverse conditions. Maintaining adequate surface drainage, proper disposal of run-off water, and control of irrigation will help reduce the potential for future moisture related problems and differential movements from soil heave/settlement.

Surface drainage should be carefully taken into consideration during grading, landscaping and building construction. Positive surface drainage should be provided to direct surface water away from wall and toward a suitable drainage device.

7.8 SOIL CORROSIVITY AND SULFATE ATTACK POTENTIAL

A representative bulk sample obtained from the borings drilled within the subject project site was tested for pH, minimum resistivity, soluble chloride content and soluble sulfate content. The test results indicate that the onsite soils have moderate sulfate concentration. As such, sulfate resistant cement is required for concrete in contact with onsite soils. Type II Portland cement may be used. The concentration of soluble chloride indicates a mildly corrosive environment for reinforced concrete. Assuming a 50-year life-span, we recommend a minimum concrete cover of 2 inches over reinforcement for structural components in direct contact with the native subgrade.
soils. The measured resistivity and pH indicate that some areas of the site are corrosive to buried ferrous metals. We recommend a minimum pipe thickness of 16 gage for buried metal pipes for protection against perforation over a 50 year life.

7.9 REVIEW OF CONSTRUCTION PLANS

Recommendations contained in this report are based on preliminary plans. The geotechnical consultant should review the final construction plans and specifications in order to confirm that the general intent of the recommendations contained in this report have been implemented into the final construction documents. Recommendations contained in this report may require modification or additional recommendations may be necessary based on the final design.

7.10 GEOTECHNICAL OBSERVATION AND TESTING

It is recommended that inspection and testing be performed by the geotechnical consultant during the following stages of construction:

- Grading operations, including over-excavation and placement of compacted fill;
- Observation of foundation excavation;
- Retaining wall footing excavation and subdrain installations;
- Excavations and backfilling for retaining walls and utility trenches; and
- When any unusual subsurface conditions are encountered.
8.0 CLOSURE

This report is intended for the use by Orange Coast Community College District and its consultants for design and construction associated with proposed restroom building at Orange Coast Community College in Costa Mesa, California, at the locations indicated on Figure 1, Boring Location Plan.

The findings and recommendations contained in this report are based on the results of the field investigation, laboratory tests, and engineering analyses, combined with an extrapolation of subsurface conditions between and beyond the boring locations.

Services performed by Willdan Geotechnical have been conducted in accordance with generally accepted professional geotechnical engineering principles and practices at this time. No other representation, express or implied, and no warranty or guarantee is included or intended.
9.0 REFERENCES


CBC. California Building Code.2010


State of California, Division of Mines and Geology (CDMG). Seismic Hazard Zones, Newport Beach 7.5 Minute Quadrangle, Orange County, California.1997.

State of California, Division of Mines and Geology (CDMG), Seismic Hazard Zones, Newport 7.5 Minute Quadrangle, Orange County, California. Seismic Hazard Zone Report 03. 1997.


FIGURE 1: SITE LOCATION MAP

PROPOSED RESTROOM BUILDING
CITY OF COSTA MESA, CALIFORNIA

Drawn By: SM Date: 8-May-13
Approved By: RK Project No. 101655-5000
FIGURE 2: BORING LOCATION PLAN

PROPOSED RESTROOM BUILDING
CITY OF COSTA MESA, CALIFORNIA

Drawn By: SM Date: 8-May-13
Approved By: RK Project No.101655-5000
APPENDIX A. BORING LOGS
<table>
<thead>
<tr>
<th>MAJOR DIVISIONS</th>
<th>SYMBOLS</th>
<th>TYPICAL NAMES</th>
</tr>
</thead>
<tbody>
<tr>
<td>GRAVELS</td>
<td>GW</td>
<td>Well graded gravels, gravel-sand mixtures</td>
</tr>
<tr>
<td></td>
<td>GP</td>
<td>Poorly graded gravels, gravel-sand mixtures</td>
</tr>
<tr>
<td></td>
<td>GM</td>
<td>Silty gravels, poorly graded gravel-sand-silt mixtures</td>
</tr>
<tr>
<td></td>
<td>GC</td>
<td>Clayey gravels, poorly graded gravel-sand-clay mixtures</td>
</tr>
<tr>
<td>SANDS</td>
<td>SW</td>
<td>Well graded sands, gravelly sands</td>
</tr>
<tr>
<td></td>
<td>SP</td>
<td>Poorly graded sands, gravelly sands</td>
</tr>
<tr>
<td></td>
<td>SM</td>
<td>Silty sands, poorly graded sand-silt mixtures</td>
</tr>
<tr>
<td></td>
<td>SC</td>
<td>Clayey sands, poorly graded sand-clay mixtures</td>
</tr>
<tr>
<td>SILTS AND CLAYS</td>
<td>ML</td>
<td>Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity</td>
</tr>
<tr>
<td></td>
<td>CL</td>
<td>Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays</td>
</tr>
<tr>
<td></td>
<td>OL</td>
<td>Organic clays and organic silty clays of low plasticity</td>
</tr>
<tr>
<td>FINE GRAINED SOILS</td>
<td>MH</td>
<td>Inorganic silts, micaceous or diatomaceous fine, sandy or silty soils, elastic silts</td>
</tr>
<tr>
<td></td>
<td>CH</td>
<td>Inorganic clays of high plasticity, fat clays</td>
</tr>
<tr>
<td></td>
<td>OH</td>
<td>Organic clays of medium to high plasticity, organic silts</td>
</tr>
<tr>
<td>HIGHLY ORGANIC SOILS</td>
<td>Pt</td>
<td>Peat and other highly organic soils</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>SAND AND GRAVEL BLOWS/FOOT*</th>
<th>SILTS &amp; CLAYS STRENGTH (ksf) BLOWS/FOOT*</th>
</tr>
</thead>
<tbody>
<tr>
<td>VERY LOOSE</td>
<td>VERY SOFT</td>
</tr>
<tr>
<td>LOOSE</td>
<td>SOFT</td>
</tr>
<tr>
<td>MEDIUM DENSE</td>
<td>FIRM</td>
</tr>
<tr>
<td>DENSE</td>
<td>STIFF</td>
</tr>
<tr>
<td>VERY DENSE</td>
<td>VERY STIFF</td>
</tr>
<tr>
<td></td>
<td>HARD</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
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</tbody>
</table>

<table>
<thead>
<tr>
<th>RELATIVE DENSITY</th>
<th>CONSISTENCY</th>
</tr>
</thead>
</table>

* Applicable only for Standard Penetration Tests (ASTM D-1586)

**TEST TYPE**
- Results shown in Appendix B
- Chemical Analysis
- Sieve Analysis
- Unconfined Compression
- Hydrometer Analysis
- Expansion Index
- Compaction
- % Passing #200 Sieve
- Pocket Penetrometer
- Direct Shear
- Direct Shear (Remolded)
- Atterberg Limits
- Consolidation
- R-Value

**OTHER**
- CA
- SA
- UC
- HA
- EI
- W
- PP
- DS
- DS<sub>p</sub>
- AL
- CN
- R

---

**EXPLORATION LOG KEY**

- **Figure No.**
- **Project No.**
- 101655-5000
- A-1

---

**STANDARD PENETRATION TEST**
- Split Barrel sampler in accordance with ASTM D 1586-84

**DRIVE SAMPLE**
- 2.42" inside diameter, 140# weight, 30" drop (unless otherwise specified on boring log)

**NO SAMPLE RECOVERY**

**BULK SAMPLE**
- Loose cuttings from exploration

**WATER TABLE**
Borehole Location: **See Figure 2**

Borehole Coordinates: **N** **W**

Drilling Equipment: **B-61**

Drilling Method: **Auger Boring**

Driller: **Whitcomb Drilling Inc.**

Logged By: **SM**

Checked By: **RK**

Hammer Information:

<table>
<thead>
<tr>
<th>Elevation (ft)</th>
<th>Depth (ft)</th>
<th>Lithology</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td></td>
<td></td>
<td><strong>5&quot; Concrete, 8&quot; Aggregate Base</strong></td>
</tr>
<tr>
<td>5</td>
<td></td>
<td></td>
<td><strong>Sandy CLAY (CL), very stiff, brown, moist</strong> PP&gt;4.5</td>
</tr>
<tr>
<td>10</td>
<td></td>
<td></td>
<td><strong>Sandy SILT with Gravel (ML), hard, brown, moist</strong> PP&gt;4.5</td>
</tr>
<tr>
<td>15</td>
<td></td>
<td></td>
<td><strong>Sandy CLAY (CL), very stiff, brown, moist</strong> PP&gt;4.5</td>
</tr>
<tr>
<td>20</td>
<td></td>
<td></td>
<td><strong>Stiff, brown, moist</strong></td>
</tr>
</tbody>
</table>

Remarks:

- **Sampling**: B-1 R-1, B-1 R-2, B-1 R-3, B-1 R-4, B-1 R-5, B-1 R-6, B-1 R-7
- **Blows/6"**: 8/11/13, 11/18/21, 12/14/20, 14/50/(5"), 9/13/18, 14/18/21, 8/12/12
- **Moisture Content (%)**: 12.3, 17.8, 13.9, 16.4, 19.2, 24.6, 24.5
- **Dry Density (pcf)**: 121, 108, 107, 104, 98, 105, 101
- **Additional Tests**: AL, CA, SA, DS

Project Number: **101655-5000**

**OCCC Restroom Building**

**FIGURE A-2**
**LOG OF BORING  B-1**

<table>
<thead>
<tr>
<th>Elevation (ft)</th>
<th>Depth (ft)</th>
<th>Lithology</th>
<th>Description</th>
<th>Remarks</th>
<th>Sampler</th>
<th>Number</th>
<th>Blows/6&quot;</th>
<th>Moisture Content (%)</th>
<th>Dry Density (pcf)</th>
<th>Additional Tests</th>
</tr>
</thead>
<tbody>
<tr>
<td>35</td>
<td></td>
<td>Sandy CLAY (CL), very stiff, brown, moist</td>
<td>PP&gt;4.5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>40</td>
<td></td>
<td>Silty SAND (SM), dense, light gray, moist (fine-grained)</td>
<td></td>
<td></td>
<td>R-8</td>
<td>15/18/32</td>
<td>2.1</td>
<td>97</td>
<td>SA</td>
<td></td>
</tr>
<tr>
<td>45</td>
<td></td>
<td></td>
<td>grades to very dense</td>
<td></td>
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<td></td>
</tr>
<tr>
<td>50</td>
<td></td>
<td></td>
<td>Total Depth 51.5 ft</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<td></td>
</tr>
<tr>
<td>55</td>
<td></td>
<td></td>
<td>GW Not Encountered.</td>
<td></td>
<td>R-9</td>
<td>18/24/44</td>
<td>2.8</td>
<td>92</td>
<td></td>
<td></td>
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<tr>
<td>60</td>
<td></td>
<td></td>
<td>Backfilled with Excavated Spoils and patched with Quick Cement.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Borehole Location:** See Figure 2  
**Date Started:** 05/06/13  
**Date Finished:** 05/06/13  
**Drilling Equipment:** B-61  
**Drilling Method:** Auger Boring  
**Borehole Diameter:** 8"  
**Driller:** Whitcomb Drilling Inc.  
**Logged By:** SM  
**Checked By:** RK

---

**OCCC Restroom Building**

**Project Number:** 101655-5000

**FIGURE A-2**
### LOG OF BORING  B-2

**Borehole Location:** See Figure 2  
**Borehole Coordinates:** N W  
**Drilling Equipment:** B-61  
**Drilling Method:** Auger Boring  
**Driller:** Whitcomb Drilling Inc.  
**Logged By:** SM  
**Checked By:** RK  
**Date Started:** 05/06/13  
**Date Finished:** 05/06/13  
**Total Depth:** 31.5 ft  
**Depth to Groundwater:** GW Not Encountered.

**Hammer Information:**

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Remarks</th>
<th>Sample Number</th>
<th>Blows/6&quot;</th>
<th>Moisture Content (%)</th>
<th>Dry Density (pcf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>5&quot; Concrete, 8&quot; Aggregate Base</td>
<td>B-1 R-1</td>
<td>6/10/14</td>
<td>16</td>
<td>112</td>
</tr>
<tr>
<td>5</td>
<td>Fat CLAY with Local Cementations (CH), very stiff, brown, moist PP&gt; 4.5</td>
<td>R-2</td>
<td>4/8/14</td>
<td>17.7</td>
<td>112</td>
</tr>
<tr>
<td>9.5</td>
<td>Sandy SILT with Gravel (ML), very stiff, brown, moist PP&gt;4.25</td>
<td>R-3</td>
<td>9/12/16</td>
<td>25</td>
<td>103</td>
</tr>
<tr>
<td>15</td>
<td>Sandy CLAY (CL), stiff, brown, moist PP=3.25</td>
<td>R-4</td>
<td>8/14/19</td>
<td>19.3</td>
<td>103</td>
</tr>
<tr>
<td>20</td>
<td>Stiff, brown, moist PP=3</td>
<td>R-5</td>
<td>7/9/13</td>
<td>28.9</td>
<td>96</td>
</tr>
<tr>
<td>25</td>
<td>PP=3.5</td>
<td>R-6</td>
<td>7/7/14</td>
<td>29.4</td>
<td>94</td>
</tr>
<tr>
<td>30</td>
<td>grades to very stiff PP=4.5</td>
<td>R-7</td>
<td>6/11/17</td>
<td>29.9</td>
<td>93</td>
</tr>
</tbody>
</table>

**Total Depth** 31.5 ft  
**GW Not Encountered.**  
**Backfilled with Excavated Spoils and patched with Quick Cement.**
APPENDIX B. LABORATORY TEST RESULTS
## TABLE B-1. SUMMARY OF LABORATORY TEST RESULTS
OCCE RESTROOM BUILDING, COSTA MESA, CALIFORNIA

<table>
<thead>
<tr>
<th>Boring No.</th>
<th>Depth (ft)</th>
<th>USCS Soil Description</th>
<th>Liquid Limit</th>
<th>Plasticity Index</th>
<th>Gradation (ASTM D422)</th>
<th>Expansion Index (ASTM D4829)</th>
<th>Direct Shear (ASTM D3080 / CT-236)</th>
<th>Corrosivity (CT-412, 417, 543)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Atterberg Limits (ASTM D4318)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B-1</td>
<td>1.5 to 5</td>
<td>Sandy CLAY (CL)</td>
<td>3:26:71</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B-1</td>
<td>5.0</td>
<td>Sandy CLAY (CL)</td>
<td>44</td>
<td>29</td>
<td></td>
<td></td>
<td>1460 19 700 19</td>
<td></td>
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<tr>
<td>B-1</td>
<td>40.0</td>
<td>Silty SAND (SM)</td>
<td>0:86:14</td>
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<td></td>
<td></td>
<td>68</td>
<td></td>
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<tr>
<td>B-2</td>
<td>1.5 to 5</td>
<td>Sandy CLAY (CL)</td>
<td>68</td>
<td></td>
<td></td>
<td></td>
<td>210 32 0 32</td>
<td></td>
</tr>
<tr>
<td>B-2</td>
<td>10.0</td>
<td>Fat CLAY (CH)</td>
<td>53</td>
<td>34</td>
<td></td>
<td></td>
<td>210 32 0 32</td>
<td></td>
</tr>
</tbody>
</table>

Project No. 101655-5000
**PROJECT NAME:** OCCC RESTROOM  
**PROJECT NO.:** 101655-5000  
**BORING NO.:** B-1  
**SAMPLE NO./DEPTH:** R-2 /5'-6.5'  
**TESTED BY:** RMC  
**DATE:** 9-May-13

**SAMPLE DESCRIPTIONS/CLASSIFICATION:** BROWN LEAN CLAY (CL)

---

### Plastic Limit

<table>
<thead>
<tr>
<th>DETERMINATION NO.</th>
<th>1</th>
<th>2</th>
</tr>
</thead>
<tbody>
<tr>
<td>DISH NO.</td>
<td>22</td>
<td></td>
</tr>
<tr>
<td>MASS, DISH + WET SOIL (g)</td>
<td>33.08</td>
<td></td>
</tr>
<tr>
<td>MASS, DISH + DRY SOIL (g)</td>
<td>31.86</td>
<td></td>
</tr>
<tr>
<td>MASS OF WATER (g)</td>
<td>1.22</td>
<td></td>
</tr>
<tr>
<td>MASS OF DISH (g)</td>
<td>23.62</td>
<td></td>
</tr>
<tr>
<td>MASS OF DRY SOIL (g)</td>
<td>8.24</td>
<td></td>
</tr>
<tr>
<td>MOISTURE CONTENT (%)</td>
<td>14.8</td>
<td></td>
</tr>
</tbody>
</table>

### Liquid Limit

<table>
<thead>
<tr>
<th>DETERMINATION NO.</th>
<th>1</th>
<th>2</th>
<th>3</th>
</tr>
</thead>
<tbody>
<tr>
<td>DISH NUMBER</td>
<td>13</td>
<td>10</td>
<td>20</td>
</tr>
<tr>
<td>MASS, DISH + WET SOIL (g)</td>
<td>37.70</td>
<td>40.24</td>
<td>38.40</td>
</tr>
<tr>
<td>MASS, DISH + DRY SOIL (g)</td>
<td>33.81</td>
<td>36.07</td>
<td>34.23</td>
</tr>
<tr>
<td>MASS OF WATER (g)</td>
<td>3.89</td>
<td>4.17</td>
<td>4.17</td>
</tr>
<tr>
<td>MASS OF DISH (g)</td>
<td>24.79</td>
<td>26.94</td>
<td>25.57</td>
</tr>
<tr>
<td>MOISTURE CONTENT (%)</td>
<td>43.1</td>
<td>45.7</td>
<td>48.2</td>
</tr>
<tr>
<td>NUMBER OF BLOWS</td>
<td>28</td>
<td>20</td>
<td>15</td>
</tr>
</tbody>
</table>

---

**FLOW CURVE**

- **Moisture Content (%)** vs. **Number of Blows**

---

**Plasticity Chart**

- **Equation of "A" line**  
  Horizontal @ PL = 4 to LL = 25.5  
  then PL = 0.73(LL - 20)

- **Equation of "U" line**  
  Vertical @ LL = 16 to PL = 7  
  then PL = 0.60(LL - 8)

---

**Result Summary**

- **Natural Moisture Content, (%)** -
- **Liquid Limit (LL)** 44
- **Plastic Limit (PL)** 15
- **Plasticity Index (PI)** 29
- **Symbol from Plasticity Chart** CL

**Remarks:**

---

**Atterberg Limits**  
( ASTM D4318 )

---

**Willdan Geotechnical**
PROJECT NAME: OCCC RESTROOM
PROJECT NO: 101655-5000
BORING NO: B-2
SAMPLE NO./DEPTH: R-3 / 10'-11.5'
TESTED BY: RMC
DATE: 9-May-13
SAMPLE DESCRIPTIONS/CLASSIFICATION: LT. BROWN FAT CLAY (CH)

<table>
<thead>
<tr>
<th>DETERMINATION NO.</th>
<th>1</th>
<th>2</th>
</tr>
</thead>
<tbody>
<tr>
<td>DISH NO.</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td>MASS, DISH + WET SOIL (g)</td>
<td>47.49</td>
<td></td>
</tr>
<tr>
<td>MASS, DISH + DRY SOIL (g)</td>
<td>44.78</td>
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</tr>
<tr>
<td>MASS OF WATER (g)</td>
<td>2.71</td>
<td></td>
</tr>
<tr>
<td>MASS OF DISH (g)</td>
<td>30.38</td>
<td></td>
</tr>
<tr>
<td>MASS OF DRY SOIL (g)</td>
<td>14.4</td>
<td></td>
</tr>
<tr>
<td>MOISTURE CONTENT (%)</td>
<td>18.8</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>DETERMINATION NO.</th>
<th>1</th>
<th>2</th>
<th>3</th>
</tr>
</thead>
<tbody>
<tr>
<td>DISH NUMBER</td>
<td>18</td>
<td>17</td>
<td>14</td>
</tr>
<tr>
<td>MASS, DISH + WET SOIL (g)</td>
<td>34.89</td>
<td>33.93</td>
<td>39.83</td>
</tr>
<tr>
<td>MASS, DISH + DRY SOIL (g)</td>
<td>31.44</td>
<td>30.19</td>
<td>34.96</td>
</tr>
<tr>
<td>MASS OF WATER (g)</td>
<td>3.45</td>
<td>3.74</td>
<td>4.87</td>
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<tr>
<td>MASS OF DISH (g)</td>
<td>24.80</td>
<td>23.30</td>
<td>26.45</td>
</tr>
<tr>
<td>MOISTURE CONTENT (%)</td>
<td>52.0</td>
<td>54.3</td>
<td>57.2</td>
</tr>
<tr>
<td>NUMBER OF BLOWS</td>
<td>30</td>
<td>21</td>
<td>15</td>
</tr>
</tbody>
</table>

FLOW CURVE

NUMBER OF BLOWS vs MOISTURE CONTENT (%)

PLASTICITY CHART

NATURAL MOISTURE CONTENT, (%)
LIQUID LIMIT (LL)
PLASTIC LIMIT (PL)
PLASTICITY INDEX (PI)

RESULT SUMMARY

ATTERBERG LIMITS

( ASTM D4318 )
## Grain Size Analysis

### Sample No. B-1

<table>
<thead>
<tr>
<th>Depth</th>
<th>Symbol</th>
<th>Classification</th>
<th>Nat. W%</th>
<th>LL</th>
<th>PL</th>
<th>PI</th>
</tr>
</thead>
<tbody>
<tr>
<td>40'-41.5'</td>
<td>SM</td>
<td>LT. GRAY SILTY FINE SAND</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

**OCCC Restroom**

Project No. 101655-5000  
5/9/2013
### Ultimate Shear Test

- **Shear Type:**
  - Saturated
  - Undisturbed
- **Peak:**

### Test Results

<table>
<thead>
<tr>
<th>Boring No.</th>
<th>Strength Intercept (C)</th>
<th>Friction Angle (Ø)</th>
<th>Normal Stress (ksf)</th>
<th>Peak Stress (ksf)</th>
<th>Ultimate Stress (ksf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-1</td>
<td>1.16 (ksf)</td>
<td>22 Degree</td>
<td>55.73 (kPa)</td>
<td>47.50 (kPa)</td>
<td>60.14 (kPa)</td>
</tr>
<tr>
<td>R-2</td>
<td>1.00 (ksf)</td>
<td>15 Degree</td>
<td>191.52 (kPa)</td>
<td>98.93 (kPa)</td>
<td>133.87 (kPa)</td>
</tr>
</tbody>
</table>

### Description
- **Sample No.:** 55.73 (kPa)
- **Depth (ft/m):** 5.0
- **Description:** Brown Sandy Clay (CL)

### Other Details
- **Remark:** B-1

### Graphs
- **Graph 1:** Shear Stress vs Normal Stress
- **Graph 2:** Shear Stress vs Horizontal Deformation

---

**OCCC RESTROOM**

**Proj. No. 101655-5000**

**Date:** 05/08/2013

---

**WILLDAN Geotechnical**

**extending your reach**

---
Ultimate : ○  Shear Type : Saturated Undisturbed  Peak : ●

**Direct Shear Test**

OCCC RESTROOM

Proj. No. 101655-5000  Date: 05/08/2013
APPENDIX C. TYPICAL RETAINING WALL BACKFILL
NATIVE SOIL BACKFILL

Sloped or level ground surface

Compacted on-site soil

Recommended backcut*

Waterproofing compound

Install subdrain system

Minimum 12-inch-wide column of 3/4" - 1 1/2" open graded gravel wrapped in filter fabric.

Filter fabric (should consist of Mirafi 140N or equivalent)

4 inch perforated pipe. Perforated pipe should consist of 4" diameter ABS SDR-35 or PVC Schedule 40 or approved equivalent with the perforations laid down. Pipe should be laid on at least 2 inches of open-graded gravel.

* Vertical height (h) and slope angle of backcut per soils report. Based on geologic conditions, configuration of backcut may require revisions (i.e., reduced vertical height, revised slope angle, etc.)
IMPOR TED GRAV EL OR CRU SHED ROCK BACK FILL

- Sloped or level ground surface
- On-site native soil cap (1/2" thick)
- Non-expansive imported gravel or crushed rock
- Install filter fabric (Mirafi 140N or equal) to prevent migration of fines into backfill.
- Waterproofing compound

- 4" inch perforated pipe. Perforated pipe should consist of 4" diameter ABS SDR35 or PVC Schedule 40 or approved equivalent with the perforations laid down. If pea gravel used, pipe should be encased in 1 cubic foot per foot min. of 3/4" - 1 1/2" open-graded gravel wrapped in filter fabric (Mirafi 140N or equal). Pipe should be laid at least 2 inches of gravel.

* At base of wall, the non-expansive backfill materials should extend to a min. distance of 2' or to a horizontal distance equal to the heel width of the footing, whichever is greater.
IMPORTED SAND BACKFILL

- Sloped or level ground surface
- On-site native soil cap (12" thick)
- Non-expansive imported sand, SE > 30.
- Waterproofing compound
- Install subdrain system
- 1 cubic foot per foot min. of 3/4" - 1 1/2" open graded gravel wrapped in filter fabric.
- Filter fabric (should consist of Mirafi 140N or equivalent)
- 4 inch perforated pipe. Perforated pipe should consist of 4" diameter ABS SDR-35 or PVC Schedule 40 or approved equivalent with the perforations laid down. Pipe should be laid on at least 2 inches of open-graded gravel.

* At base of wall, the non-expansive backfill materials should extend to a min. distance of 2' or to a horizontal distance equal to the heel width of the footing, whichever is greater.