GEOTECHNICAL INVESTIGATION
EAST/WEST WING RENOVATION
COURTYARD PROJECT
240 CHURCH STREET
SALINAS, CALIFORNIA

PROJECT # 20163829.001A

FEBRUARY 25, 2016

February 25, 2016
Project No: 20163829.001A

MONTEREY COUNTY
RESOURCE MANAGEMENT AGENCY
168 W. Alisal Street, 2nd Floor
Salinas, California 93901

Attention: Ms. Judy Jeska

SUBJECT: Geotechnical Investigation for the Planned East/West Wing Renovation Courtyard Project at 240 Church Street, Salinas, California

Dear Ms. Jeska:

This report presents the results of our geotechnical investigation for the planned East/West Wing Renovation Courtyard Project at 240 Church Street, Salinas, California. Our work consisted of a review of existing Kleinfelder geotechnical and geologic information from the site vicinity, field investigation, laboratory testing, engineering analysis and preparation of this report.

Based on the results of our investigation and from a geotechnical standpoint, it is our opinion the site may be developed as planned provided the recommendations presented in this report are incorporated into the design and construction of the project.

The primary consideration for the courtyard is the presence of near surface highly to very highly expansive clay soils. Expansive soils have the potential to crack and damage ground coverings and shallow foundations as a result of the shrink and swell of the soils from drying and wetting due to weather or irrigation. As part of our field investigation, we performed a ground penetrating radar screening of the courtyard area to identify buried utilities and debris from the demolition of the previous structure at the site. We also identified demolition debris in our geotechnical borings. This layer of debris will need to be removed from the subgrade soils as part of the improvements. An additional concern at the site is the presence of loose weak near-surface soils and deeper liquefiable soils. The presence of loose weak near-surface soils and deeper liquefiable soils means that in the event of a nearby major earthquake, the soils underlying the site have the potential to dynamically settle resulting in differential movement and damage to the new courtyard features and structures. Liquefaction is typically mitigated through foundation design utilizing deep foundations that derive their structural support on the competent soils below the liquefiable soil layer. Deep foundations for the courtyard improvements are likely

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impractical based on the limited site access and expense. Other methods for supporting light-weight structures on liquefiable soils have been used successfully locally. These considerations are discussed in the report. Please refer to the report for detailed recommendations.

As noted in our report, Kleinfelder should be commissioned to review project plans and specifications prior to the start of construction, and to observe and test during earthwork and foundation construction. This will allow us to compare conditions exposed during construction with those encountered during our investigation and to present supplemental recommendations if warranted by different site conditions.

We appreciate the opportunity to provide services to the Monterey County Resource Management Agency. If you have any questions, please contact us at (831) 755-7900.

Respectfully submitted,

KLEINFELDER, INC.

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1. INTRODUCTION

This report presents the results of our geotechnical investigation for the planned East/West Wing Renovation Courtyard Project at 240 Church Street in Salinas, California. The approximate location of the site is shown on the Site Vicinity Map, Figure 1. The layout of the existing courtyard, with our boring locations is shown on the Site Plan, Figure 2, and the results of our utility locating program are shown on the Utility Plan, Figure 3.

The conclusions and recommendations presented in this report are based on the subsurface soil conditions encountered at the locations drilled during our site exploration and the provisions and requirements outlined in the Limitations section of this report. The findings, conclusions and recommendations presented herein should not be extrapolated to other areas or used for other projects without our review.

1.1. PROJECT DESCRIPTION

The project consists of the complete renovation of the central courtyard at the East/West Wing Building. The existing central courtyard is located in the area of a previous wood frame building that was removed after the construction and occupation of the surrounding existing East/West Wing Building. Over time, apparent soil settlement has occurred in the central courtyard resulting in an uneven surface of the brick pavers (not original) and changes in grades that are not code compliant. An existing reflective pond with an historic Jo Mora sculpture is located in the center of the existing courtyard and will remain with some modifications to the pond. Grades and surfaces will be renovated to make them code compliant and landscaping will be improved. Other associated improvements are expected to include new underground utilities.

Structural loading information was not available to us when we prepared this report. We assume that structural loads and foundation layouts will be similar to other similarly sized structures. We expect that the grading will be limited to site leveling and shallow foundation excavations, and possibly utility line excavations up to about 4 feet deep. No other significant grading or basements are planned.

If the actual project differs from that described above, Kleinfelder should be contacted to review our conclusions and recommendations and present any necessary modifications to address the different project development plans.

1.2. PURPOSE AND SCOPE OF SERVICES

The purpose of our services was to perform utility locating and mapping, and to perform a geotechnical engineering investigation to evaluate the subsurface conditions at the proposed project site and to provide seismic design parameters and geotechnical recommendations regarding debris removal, expansive soils, settlement, site grading, drainage, foundation design and construction, concrete slabs-on-grade, and utility trenches. As instructed, our field exploration activities were performed so as not to interfere with or encroach on the existing reflective pond and historic statue.

Our scope of services is presented in our proposal letter dated December 4, 2015 (Kleinfelder Document No. SAL15P31258). Environmental services such as chemical analysis of the soils and groundwater were not included in our scope of services.

2. SITE INVESTIGATION

2.1. SITE DESCRIPTION

The project site is located in the open courtyard in the middle of the East/West Wing Building at 240 Church Street, in Salinas, California. Church Street is located to the east of the building, West Alisal Street is located to the south of the building, and other government buildings are located to the north and west of the building. The building is surrounded by sidewalks and the adjacent paved streets to the east and south. The location of the project site is shown on the Site Vicinity Map, Figure 1.

The courtyard consists of 4 roughly rectangular grass landscaped areas that are surrounded by hedges and contain trees. These landscaped areas are located in the northeast, northwest, southeast and southwest portions of the courtyard. In the center of the courtyard is the existing rectangular reflective pond with the historic Jo Mora sculpture in the geometric center of the courtyard. The pond is also surrounded by hedges. Benches facing north are located with their backs against the hedge on the northern side of the pond. The remaining portions of the courtyard consist of walkways surfaced with brick pavers in a herringbone pattern and concrete surfaced areas. Access to the courtyard is by way of porticos on the north and south sides of the courtyard and located between the wings of the East/West Wing Building. Stairways just inside the porticos lead down to the slightly lower elevation courtyard area. Access to the East and West wings is by way of doors located on the eastern and western sides of the courtyard. The approximate location of the proposed construction is shown on the Site Plan, Figure 2.

The project site is located on fairly level ground, except for a slight elevation change at the steps.

2.2. UTILITY LOCATING

Prior to our subsurface exploration, Underground Service Alert (USA) was contacted to provide utility clearance. In addition, a private utility locator was retained to evaluate the site for buried utilities and other obstructions within the courtyard area. The utility locator marked the utilities in the field and Kleinfelder documented the utility locations and prepared a scale drawing showing the locations of the detected buried utilities and other obstructions within the courtyard area. The private utility locator also cleared the areas of our proposed exploratory borings for underground utilities. A drawing showing the location of the buried utilities and obstructions that were detected is included as Figure 3, Utility Plan, herein. Our scope of work excluded having the utility lines surveyed by a land surveyor.

Ground penetrating radar was used to attempt to locate buried debris at the courtyard site. We had limited success locating debris, partly due to the wet subgrade conditions which impeded the radar. It is possible that some buried utilities are located under the courtyard that were not detected by our instruments. All excavations should proceed with caution.

A discussion of the buried debris encountered in our subsurface exploration is detailed in the section on Subsurface Conditions, herein.

2.3. SUBSURFACE INVESTIGATION

The geotechnical field investigation consisted of a site reconnaissance and drilling of four (4) borings on February 1, and 2, 2016. The borings were drilled with a portable hydraulic drill equipped with 4-inch diameter solid flight augers.

The borings were all drilled to a depth of about 31½ feet below existing ground surface. The approximate locations of the borings are shown on the Site Plan (Figure 2). These approximate exploration point locations were estimated by our personnel in the field based on pacing and measuring from the limits of existing site features.

The soils encountered in our borings were visually classified in the field in general accordance with the Unified Soil Classification System (ASTM D 2488) by our engineering staff. The results of our laboratory tests were used to refine the field classifications based on ASTM D 2487. Keys for classification of the soils are presented on Figures A-1 through A-2, and the logs of Borings B-1 through B-4 are presented on Figures A-3 through A-6, in Appendix A.

Representative samples of subsurface soils were obtained from the borings as drilling progressed. The samples were primarily obtained by driving a 1-3/8-inch inside diameter standard penetration sampler or a 2.5-inch inside diameter California tube sampler up to a depth of 18 inches into the underlying soil using a 140-pound hammer falling 30 inches using a rope and cathead system. The number of blows required to drive the sampler was recorded for each 6-inch penetration interval, and is shown as blows per foot on the boring logs at the approximate sample depth. The blowcounts noted on our boring logs are field measured blowcounts and have not been corrected for energy efficiency or other parameters. We also collected a bulk samples of the near surface soil. Samples collected from the borings were returned to our laboratory for further evaluation and testing. The borings were backfilled with spoils and capped with concrete in paved areas.

2.4. LABORATORY TESTING

Laboratory testing was performed on selected soil samples collected from the borings. Tests performed included natural moisture content, in-place density, Atterberg Limits, percent passing #200 sieve, unconfined compression strength, consolidation, and R-Value. Most of the laboratory test results are presented on the boring logs. Graphic presentations of the results of the Atterberg Limits, unconfined compression strength, consolidation, and R-Value tests are included in Appendix B. In addition, a selected sample was sent to CERCO Analytical, Inc., for preliminary corrosivity screening. A letter containing the preliminary corrosivity screening test results and evaluation will be forwarded to you once testing is completed.

3. SUBSURFACE CONDTIONS

3.1. SOIL CONDITIONS

The near-surface soils encountered at the site generally consisted of fat clay to depths of 7 to 8 feet below ground surface levels (bgs). This fat clay was of very high plasticity, olive-brown or dark brown in color, firm to stiff, and moist at the time of the investigation.

In the landscape areas, at Borings B-1 and B-2, we encountered approximately 2 to 4 feet thickness of fill over the fat clay layer. The fill consisted of silty clay, fat clay and demolition debris. Demolition debris, primarily consisted of dark red brick, similar to the darker colored brick located around the courtyard perimeter. The dark red brick was encountered in a thin layer between about 1½ and 2½ feet below existing ground surface in Boring B-2. We had to relocate Boring B-2 about 2 feet to the east of the originally planned location to drill through the brick debris layer. A gravel layer was encountered in Boring B-1 between 3 and 4 feet below the existing ground surface. The gravel from this layer appeared to be demolition debris. In Borings B-3 and B-4 at the brick walkways, we encountered the near-surface fat clay layer directly below the bottom of the brick surface.

Below the near-surface fat clay layer, the soil generally consisted of sands and silts to the limits of the investigation at approximately 31½ feet bgs. The sands and silts were generally yellow-brown or olive-brown in color and moist above groundwater elevation and wet below at the time of the investigation. The sand fraction was generally fine to medium grained. These soils varied from loose to medium dense and soft to very stiff.

3.2. GROUNDWATER

Groundwater was initially encountered at a depth of about 23 feet below existing ground surface at the time of drilling in all of our borings (B-1 through B-4). Groundwater was also measured at completion of drilling in Boring B-1. The groundwater at the end of drilling in Boring B-1 remained at 23 feet below existing ground surface. Note that fluctuations in the groundwater level may occur due to variations in rainfall, temperature, and possibly as the result of other factors that were not evident at the time of our investigation. If significant variations in the groundwater level are encountered during construction, it may be necessary for Kleinfelder to review the recommendations presented herein and recommend adjustments as necessary.

The above is a general description of the subsurface conditions encountered at the site. A detailed description of the subsurface conditions encountered can be found on the logs of the borings on Figures A-3 through A-6 in Appendix A.

4. DISCUSSION AND CONCLUSIONS

The main geotechnical concern for the site is expansive clay soils. Based on the laboratory testing results, the near surface clay soils can be classified as having high or very high plasticity. Over-excavation of these soils and replacement with non-expansive material is recommended to mitigate potential shrink-swell movements. It appears that the apparent ground settlement or movement that has occurred at the courtyard site consists primarily of the seasonal shrink/swell movement of the near-surface clay soils, and elastic compression of the clay soils in the foundation zones. The demolition debris encountered had much better in-situ bearing strength than the surrounding clay soil. It is possible that the debris were used to bridge over the weaker clays as part of the courtyard construction. Other geotechnical concerns for site development include liquefaction settlement. These concerns are discussed in more detail below.

Our opinions, conclusions, and recommendations are based on our field and office studies, the subsurface soil information collected from our field exploration, the results of our laboratory testing, and our understanding of the proposed development.

4.1. COMPRESSIBLE CLAY SOILS

Some of the clayey soils encountered in our borings have relatively low blowcounts and shear strength, suggesting that these soils are compressible under typical structural loads. Some of this elastic compression may have taken place after the existing improvements were constructed and could be responsible for some of the apparent settlement at the site. Note that this settlement under existing structures should now be complete; however, additional settlement could take place under new structures, or if the loading of the existing structures changes. Significant elastic compression settlement for the upgraded structures is not anticipated as long as the light foundation loading recommended in this report is used.

4.2. EXPANSIVE SOIL

The results of our laboratory testing indicate areas of near-surface clay soil with expansion characteristics ranging from high to very high. Expansive clay soils are characterized by their ability to undergo significant volume change (shrink or swell) due to variations in moisture content. Changes in soil moisture content can result from rainfall, landscape irrigation, perched groundwater, drought or other factors. Changes in soil moisture may result in unacceptable settlement or heave of structures, concrete slabs supported on-grade or walkways/pavers supported on these materials. We recommend that all walkways and settlement sensitive areas be underlain by a layer of "non-expansive" fill.

4.3. LIQUEFACTION POTENTIAL AND DYNAMIC COMPACTION

Soil liquefaction is a phenomenon in which saturated, cohesionless soils lose their strength due to the build-up of excess pore water pressure during cyclic loading such as that induced by earthquakes. The primary factors affecting the liquefaction potential of a soil deposit include: 1) intensity and duration of earthquake shaking; 2) soil type and relative density; 3) overburden pressure; and 4) depth to groundwater. Soils most susceptible to liquefaction are saturated clean, loose, fine-grained sands, and low plasticity silts and clays.

To assess the potential for liquefaction of subsurface soils at the site, we used the liquefaction analysis procedures outlined in Youd et.al. (2001), Seed et.al (2003), and Idriss and Boulanger (2004 and 2008). For estimating the resulting ground settlements, we used the methods proposed by Tokimatsu and Seed (1987), Cetin et.al (2009), and Idriss and Boulanger (2008), respectively.

As recommended in Section 1803.5.12 of 2013 California Building Code (CBC), the peak ground acceleration (PGA) used in the liquefaction analysis was estimated in accordance with Section 11.8.3 of ASCE 7-10. A PGAM of 0.599g with an earthquake magnitude of 8.02 was used as the design-level seismic event for our liquefaction analyses, which is defined as an earthquake event with 2 percent probability of being exceeded in 50 years (return period of about 2,475 years) according to the 2013 CBC and ASCE/SEI 7-10.

We evaluated the liquefaction potential at the site using the Standard Penetration Test (SPT) data. The evaluation of liquefaction in response to an earthquake is based on a comparison of a soil's resistance to liquefaction and the cyclic load or demand placed on the soil by the earthquake. A factor of safety (FOS) against liquefaction is commonly defined as the ratio of the cyclic shear stress required to cause liquefaction (cyclic resistance ratio, or CRR) to the equivalent cyclic shear stress induced by the earthquake (cyclic stress ratio, or CSR). A layer may not experience liquefaction triggering but it may experience settlements due to dissipation of excess pore pressure. We have used FOS values of 1.0 and 1.3 for liquefaction triggering and settlement calculations, respectively.

Based on the SPT data and our engineering analyses, and using a groundwater level of 23 feet bgs, it is our opinion that loose to medium dense sands encountered may be subject to liquefaction in the event of a major earthquake occurring on a nearby fault. Based on our analyses, we estimate that seismically-induced settlement due to strong ground shaking during a design-level seismic event could be on the order of 2 to 3 inches. The amount of differential settlement will depend on the uniformity of the subsurface profile. For uniform subsurface conditions, differential settlement on the order of 50 percent of the total seismic settlement could be expected. For highly heterogeneous sites, differential settlements on the order of 75 to 100 percent of the total seismic settlement could be expected. We judge that differential settlement at this site may be as much as 2 inches over a horizontal distance of 50 feet.

The risk of lateral spreading at this site is considered low because the site is not located close to a free face, such as a creek channel.

Dynamic compaction is the densification of granular soils as the result of earthquake shaking. This generally occurs in loose to medium dense sand above groundwater. The potential impact of dynamic compaction is settlement of the ground surface. Based on the results of the field investigation, we have calculated dynamic settlements in the silts and sands above the groundwater table. In the event of an earthquake inducing dynamic compaction, the total settlements due to dynamic compaction are expected to be up to 1 inch, with differential settlement up to ½ inch over a horizontal distance of 50 feet.

5. RECOMMENDATIONS

We recommend that Kleinfelder be retained to provide observation and testing services during site earthwork and foundation construction. This will allow us the opportunity to compare conditions exposed during construction with those inferred in our investigation and, if necessary, to expedite supplemental recommendations if warranted by the exposed subsurface conditions. We also recommend that Kleinfelder be retained to review your final foundation and grading plans and specifications. It has been our experience that this review provides an opportunity to detect misinterpretation or misunderstandings prior to the start of construction.

In order to minimize settlement and return the subgrade to a suitable condition for the courtyard renovation we recommend removing and replacing the top 18 inches of subgrade soils with non-expansive fill and minimizing the foundation loads in the settlement sensitive areas. This should mitigate against differential movements of the walkways and concrete slabs-on-grade and static settlement of the larger improvements, such as the pond. To reduce the effects of dynamic settlement from liquefaction and dynamic compaction on the larger improvements, such as the pond, we recommend using a relatively rigid grid foundation. This will minimize the differential movement of the structure and reduce the potential for cracking and damage to the structure, although some releveling could be required in a major earthquake. The intent is to prevent structural damage with the understanding that some releveling may need to be performed if a major earthquake occurs near the site. If no movement of the structure is to be allowed during a major earthquake, a deep foundation system, such as driven piles would be required.

5.1. EARTHWORK

As discussed, we recommend that the existing highly expansive soils be over-excavated and replaced with non-expansive fill in any new structures, walkways or settlement sensitive areas. This will reduce the potential for cracking due to shrink/swell movements in the underlying expansive soils. Earthwork at the site is expected to be limited to that required for grading and the installation of underground utilities.

5.1.1 Site Clearing and Stripping

Site preparation will include demolition and removal of the existing walkway and landscape materials. We understand that the County has not yet decided as to whether the reflective pond and sculpture will be moved during construction or remain in place. Debris including, but not limited to, concrete slabs-on-grade, foundations, bricks, pavements, building materials, and buried utilities (to be removed) generated by current or previous demolition should be removed from the site. Depressions, voids, holes and loose soils generated by demolition that extend below proposed finished grades should be cleaned and backfilled with engineered fill compacted to the requirements given under the Section on "Fill Placement and Compaction." All site clearing and backfill work should be performed under the observation of a representative from Kleinfelder.

Based on our review of our ground penetrating radar survey, there appears to be buried utility ducts throughout the courtyard area. If the ducts are demolished and relocated, the debris should be removed from the site and the excavation backfilled with engineered fill. Utilities to remain active under the courtyard area and/or fill areas should be evaluated by the project civil engineer to determine if they are suitable for the new soil loads, and the construction and structural loads that will be generated by the new courtyard development. If they are not suitable, they should be replaced as determined by the project civil engineer.

5.1.2 Excavations

We anticipate that excavations can be performed with conventional excavation equipment. After demolition and removal, the loose subgrade soils in the courtyard walkway and settlement sensitive areas should be excavated to a minimum depth of 18 inches below finished subgrade elevation or to the depth disturbed during demolition/clearing, whichever is deeper. The exposed subgrade should be scarified, moisture conditioned, and recompacted and then backfilled to finished subgrade with "non-expansive" engineered fill, as discussed in the sections on Subgrade Preparation, and Fill Placement and Compaction. The excavation should extend at least 2 feet horizontally beyond the perimeter of the new walkways or settlement sensitive areas. If it is determined that the reflection pond and sculpture will remain in place during construction, the excavation may be terminated at the edge of the structures to remain.

In this case, shoring or buttressing will likely be required to ensure the stability of the reflection pond and sculpture.

The excavated soils may be stockpiled on-site for use as general fill provided they are not located within 18 inches below the bottoms of any footings, walkways or settlement sensitive areas. Due to the highly expansive nature of the clay soils encountered they should not be re-used as general fill within the upper 18 inches of the subgrade on the site.

5.1.3 Subgrade Preparation

After demolition and removal of existing walkways, pavements and landscape materials, areas to receive engineered fill, proposed structures, proposed walkways and/or settlement sensitive areas, should be over-excavated a minimum of 18 inches below the bottom of footings or walkways, scarified, moisture conditioned, and compacted to the requirements given under Fill Placement and Compaction, and Exhibit 1 in Appendix D. Non-expansive fill should then be placed as outlined below, up to finished subgrade levels. Subgrade preparation in the walkway and settlement sensitive areas should extend laterally at least 2 feet beyond the perimeter of the new structures and adjoining perimeter walkways, unless obstructed by adjacent sidewalks walkways or other structures to remain (i.e. the reflection pond and sculpture, and existing building). Where obstructions to remain are encountered, the subgrade preparation may be terminated at the edge of the adjacent obstruction. After the soil subgrades have been properly prepared, the areas may be raised to design grades by placement of engineered fill. Moisture conditioning of subgrade soils will consist of adding water if the soils are too dry and allowing the soils to dry, if the soils are too wet.

Unstable or wet subgrade soil encountered during construction should be stabilized prior to placement of new fill and further construction. The method of stabilization should be evaluated by a representative of Kleinfelder at the time of construction depending on the exposed conditions. Typical remedial measures include scarifying and air-drying wet soils during dry weather; mixing wet soils with drier materials, removing and replacing wet soils with an approved fill material, stabilizing with a geotextile fabric or geogrid, or treating wet soils with an approved lime product. Prior to bidding, we suggest that the site be made available to potential bidders to explore the moisture conditions of the near-surface soils. Earthwork contractors should include unit prices for wet soil mitigation in their bids.

5.1.4 "Non-expansive" Fill

To reduce the effects of seasonal volume changes of the highly expansive on-site soils, we recommend that walkways and settlement sensitive areas be constructed on a layer of compacted "non-expansive" engineered fill at least 18 inches thick. Angular gravel (capillary break material), or aggregate base material used for slab support is considered "non-expansive" material and may be counted towards the recommended thickness of the "non-expansive" fill layer. The "non-expansive" fill should extend laterally outward from the perimeter of the walkway and settlement sensitive areas and adjoining perimeter walkways a minimum of two feet on every side, unless obstructed by existing improvements to remain. In that case the "non-expansive" fill may be terminated at the edge of the existing improvements to remain. The "non-expansive" fill should meet the requirements given for import fill in the Material for Fill section and should be placed and compacted in accordance with the recommendations given in the Fill Placement and Compaction section.

5.1.5 Material for Fill

In general, on-site soils with an organic content of less than 3 percent by weight and free of any deleterious materials or hazardous substances may be used as general fill provided they are not located within 18 inches below the bottoms any new footings, walkways or settlement sensitive areas. This is to reduce the potential of shrink/swell of the on-site clay soils from impacting the new development. Imported fill material should be predominately granular, should contain no rocks or lumps larger than 3 inches in the greatest dimension, should not contain more than 15 percent of the material larger than 1-1/2 inches, should have between 8 and 40 percent passing the U.S. Standard No. 200 mesh sieve, and should have a Plasticity Index of 15 or less. At least 5 working days before delivery to the site, a representative sample of the proposed import fill should be delivered to our laboratory for evaluation and testing. All import fills should be approved by the project Geotechnical Engineer before delivery to the site.

5.1.6 Fill Placement and Compaction

Fill materials should be placed in lifts each not exceeding 8 inches in uncompacted thickness and should be compacted by mechanical means only. Due to equipment limitations, thinner lifts may be necessary to achieve the recommended level of compaction. Relative compaction or compaction is defined as the in-place dry density of the compacted soil divided by the laboratory compacted maximum dry density as determined by ASTM Test D 1557, latest edition, expressed as a percentage. Specifications for compaction are provided in Exhibit 1 of Appendix D.

Moisture conditioning includes adding water to the soil if its moisture content is too low, or scarifying or disking and drying of the soil if its moisture content exceeds the recommended value above. Careful control of soil moisture is recommended during all site earthwork. If additional water is required during the grading operation, care should be taken to avoid over-watering of the soil.

5.1.7 Utility Trench Excavation and Backfill

We anticipate that excavations for foundations and utility trenches can be readily made with either a conventional backhoe or excavator. All excavations should be constructed in accordance with OSHA and Cal-OSHA Safety Standards. Safety in and around foundation and utility trenches is the responsibility of the contractors.

Shallow excavations for foundations and utilities up to 4 feet deep into engineered fill with some fines and/or native clay soils should be able to stand near vertical with bracing provided proper moisture content in the soils is maintained. Deeper excavations and/or excavations into any loose soils encountered or into granular fill should be properly shored, or the sidewalls laid back at a safe inclination, to protect personnel and to increase stability. In addition, excavations should be located so that no structures or surface surcharge loading are located above a plane projecting 1.5:1 (Horizontal: Vertical) upward from any point in an excavation, regardless of whether it is shored or unshored.

Pipes and conduits should be bedded and shaded in accordance with the requirements of the utility company. Where no specific requirements exist, we recommended the use of relatively clean sand as bedding and backfill to 12 inches above the top of pipe or conduit. This sand should be consolidated by wetting together with mechanical compaction prior to further backfill placement. Imported soil or imported sand may be used for backfilling utility trenches above the initial sand backfill. Onsite clayey soils may be used for general backfill provided they are not located within 18 inches below the bottoms of new structures. Trench backfill material should be properly moisture conditioned (or allowed to dry) before placement in the trenches. Trench backfill should be placed as recommended in Exhibit 1 of Appendix D. Trenches should be capped with at least 12 inches of compacted soil similar to that of the adjoining subgrade. Compaction should be performed by mechanical means only. Water jetting and flooding to attain compaction above the sand shading should not be permitted.

5.1.8 Surface Drainage

Final site grading should provide surface drainage away from structures, slabs-on-grade and edge of walkways to reduce the percolation of water into the underlying soils. Ponding of surface water should not be allowed adjacent to the structures and on walkways. Grades should be sloped away from the structures a minimum of 5 percent for a horizontal distance of at least ten feet in landscape areas and 2 percent for a horizontal distance of at least ten feet in sidewalk and walkway areas. Rainwater collected on the roof of the existing building should be transported through gutters, downspouts and closed pipes which discharge on the walkway or lead directly to the site storm sewer system. If discharging onto the walkway, safety of pedestrian traffic should be considered.

5.1.9 Seepage Control

Where utility lines extend through or beneath buildings or other structures, permeable backfill should be terminated at least one foot from the structure. Concrete should be used around the pipe to act as a seepage cutoff. Beneath foundations, the pipe should be “sleeved” through concrete cut-offs, and the annular space around the pipe should be filled with a waterproof caulk. This will help reduce the amount of water seeping through the previous trench backfill and collecting under the structure.

Where exterior slabs or walkways abut against landscaped areas, some method should be used to protect the baserock layer and subgrade soils against saturation from water from the landscaped areas. Methods of reducing seepage under exterior slabs and walkways may include vertical curbs extending below the base rock/subgrade interface, or use of commercially available impervious root guards or subdrains behind the slabs and walkways in landscape areas. Also, care should be taken to prevent over-watering of landscaped areas adjacent to slabs and walkways.

Vertical cut-offs, such as a thickened edge or equivalent, extending at least 2 inches below the native subgrade/base rock interface will help reduce the amount of lateral seepage under slabs and walkways from adjacent landscaped areas. Concrete cut-offs must be carefully constructed such that they extend below the baserock section and are poured neat against undisturbed native soil or compacted clayey fill. The cut-offs should be continuous and any utility trenches (irrigation lines, electrical conduit, etc.) that extend through, or under the slabs and walkways, should be sealed with compacted clayey soil or poured in-place concrete.

5.1.10 Wet Weather Construction

If site grading and construction is to be performed during or shortly after the winter rainy months, the owner and contractors should be fully aware of the potential impact of wet weather. Rainstorms can cause delay to construction and damage to previously completed work, such as saturating a compacted pad or subgrade, or flooding an excavation. Runoff can also cause erosion.

Earthwork during rainy months will require extra effort and caution by the contractors. The soils may be too wet to compact which will require processing to dry the soil. The grading contractor should be responsible to protect his work to avoid damage by rainstorms, including smooth rolling to seal off a pad or subgrade surface to facilitate drainage and to reduce rain damage, and covering the trenches with plastic sheeting. Ponded water should be pumped out immediately. Construction in wet weather, if a possibility for project construction, should be addressed in the project construction bid documents and/or specifications. We recommend the grading contractor submit a wet weather construction plan outlining procedures they will employ to protect their work and to reduce damage to their work by rainstorms.

5.1.11 Construction Observation

Variations in soil types and conditions are possible and may be encountered during construction. In order to permit correlation between the soil data and the actual soil conditions encountered during construction, and to check for conformance with our recommendations as originally contemplated, we should be retained to perform continuous observation and testing, as needed, during earthwork, excavation, and all foundation phases of construction. All earthwork should be performed in accordance with the recommendations presented in this report, or as recommended by Kleinfelder during construction.

5.2. FOUNDATIONS AND SETTLEMENT

5.2.1 Foundation Recommendations

The results of the geotechnical exploration program indicate highly expansive clay soils in the upper 7 to 8 feet bgs. These soils not only have the potential for swelling but also for elastic compression under the proposed structures. The degree of settlement will depend on the details of the new development including size and loading conditions on the new structures.

Loading information for the proposed development was not available to us at the time this report was prepared, however, we anticipate the loads to be relatively light. If new lightly loaded structures are proposed, we recommend they be supported on a grid foundation. All continuous footings should be tied together, and isolated footings should be tied to the other foundations with continuous footings or grade beams. If new structures are proposed adjacent to the existing building, we recommend that the bottom of the new footings be at the same elevation as the adjacent bottom of the below grade crawlways of the existing building to avoid overloading the existing building walls with the structural loads from the new structure or possibly undermining the existing building. Precautions should be taken to avoid undermining the existing building during construction.

For any new structures, a foundation system consisting of interconnected grid footings should be used to evenly distribute the structure's loads to the foundation soils. Additionally, interconnected grid footings will add strength to the foundations allowing the structure to more easily tolerate total and differential settlements under the foundations.

For the foundation systems outlined above, the highly expansive site soils below the proposed development should be prepared as indicated above in the Earthwork section.

For an interconnected grid foundation system, grid spacing should be no greater than 10 feet. All footings should be founded at least 24 inches below the interior pad grade or exterior finished grade, whichever provides a deeper embedment into the foundation soils prepared in accordance with the Earthwork section above. Footings/grade beams should have a minimum width of 12 inches. Foundations should be designed to distribute the structure's loads more uniformly onto the supporting soil and to span over areas where temporary loss of support or settlement occurs.

For design of an interconnected grid foundation system, the following parameters may be assumed:

- Allowable bearing pressure = 1,500 pounds per square foot, dead plus live loads, with a one-third increase when including transient loads such as seismic or wind;
- Vertical modulus of subgrade reaction, Kv1 = 65 pounds per square inch;
- Edge cantilever length = 5 feet; and
- Interior unsupported length = 10 feet.

Use of these design parameters will increase the stiffness of the section, resulting in reduced differential movement as a result of localized soil settlement.

To maintain the desired support, foundations located adjacent to utility trenches should be deepened 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 utility trench.

It is critical that footing excavations not be allowed to dry before placement of concrete. If visible cracks appear in the foundation excavations, the excavations should be thoroughly moisture conditioned beginning at least two days prior to placement of concrete to close all cracks. It is also important that the base of the footing excavations not be allowed to become excessively wet, resulting in soft soils. Water should not be allowed to pond in the bottom of the excavations. Areas which become water damaged should be over-excavated to a firm base. The footing excavations should be monitored by a representative of Kleinfelder for compliance with appropriate moisture control and to confirm the adequacy of the bearing materials. We recommend that Kleinfelder be retained to observe the footing excavations prior to placing reinforcing steel to check that footings are founded in the anticipated bearing soil.

Lateral loads may be resisted by a combination of friction between the foundation bottom and the supporting subgrade and passive resistance acting against the vertical faces of the foundations. A coefficient of friction of 0.25 may be used between the supporting subgrade and the bottom of the foundation. For calculating passive resistance, an ultimate equivalent fluid weight of 350 pounds per cubic foot, to a maximum pressure of 1500 pounds per square foot, may be assumed acting against the embedded face of the foundation. The passive pressure can be assumed to act starting at a depth of 1 foot below grade in unpaved areas. It should be noted that the lateral load resistance values discussed above are only applicable where the concrete for the foundation is either placed directly against undisturbed engineered fill soils or the voids created from the use of forming are backfilled with properly compacted soil.

5.2.2 Anticipated Settlement

Settlement, for similar loads, will be less for smaller structures. As outlined above, to reduce the degree of differential settlement, it is recommended that a foundation system consisting of continuous interconnected grid footings be used to more evenly distribute the structural loads to the foundation soils. Additionally, interconnected grid footings will add strength to the foundations allowing the structure to more easily tolerate total and differential settlements under the foundations/slabs-on-grade. Based on the soils encountered and a maximum allowable load of 1,500 psf, static settlements are estimated to be up to 1 inch total with differential settlement of ½ inch over 50 feet, assuming the foundations are designed and constructed in accordance with our recommendations given above. Settlement due to liquefaction and dynamic compaction may occur in addition to the static settlements outline herein.

Seismically-induced settlement (i.e. liquefaction and dynamic compaction settlement) that may occur as a result of soil liquefaction would be in addition to these estimated settlements. Liquefaction settlement is anticipated to be on the order of 2 to 3 inches total settlement, with a differential settlement of as much as 2 inches over a horizontal distance of 50 feet. In addition to liquefaction settlement, the total settlements due to dynamic compaction are expected to be up to 1 inch, with differential settlement up to ½ inch over a horizontal distance of 50 feet.

5.3. CONCRETE SLABS-ON-GRADE AND WALKWAYS

We understand this project will include the reconstruction of the courtyard area including new concrete slabs-on-grade, walkways and landscape areas. In addition, new lightly loaded structures may also be proposed. We understand that the County has not yet decided as to whether the reflective pond and sculpture will be moved during construction or remain in place. All concrete slabs-on-grade and new walkways, should be constructed on a layer of “non-expansive” fill as recommended under the Earthwork section herein. Concrete slabs-on-grade supported directly on engineered fill, installed as recommended herein, may be designed using a modulus of subgrade reaction (K_{V1}) of 140 pounds per cubic inch.

Once the slab subgrade soil has been moisture conditioned and compacted, the soil should not be allowed to dry prior to concrete placement. If the subgrade soil is too dry, the moisture content of the soil should be restored to the recommended value prior to placement of concrete.

Proper moisture conditioning and compaction of subgrade soils is important. Even with proper site preparation, we anticipate that over time there will be some effects of soil moisture change on concrete flatwork. Exterior flatwork will be subjected to edge effects due to the drying out or wetting of subgrade soils where adjacent to landscaped or vacant areas. To help reduce edge effects, lateral cutoffs such as an inverted curb are strongly recommended. Control joints should be used to reduce the potential for unsightly panel cracks as a result of soil displacement. Steel reinforcement will aid in keeping the control joints and other cracks tightly closed.

Exterior concrete slabs-on-grade should be cast free from adjacent footings or other non-heaving edge restraints. This may be accomplished by using a strip of 1/2-inch asphalt-impregnated felt divider material between the slab edges and the adjacent structure. Frequent construction or control joints should be provided in all concrete slabs where cracking is objectionable. Continuous reinforcing or dowels at the construction and control joints will also aid in reducing moisture related uneven slab movements. It is generally not cost effective to design exterior concrete slabs-on-grade to resist liquefaction settlement for seismic case conditions. The exterior concrete slabs-on-grade recommendations presented herein are made with the understanding that in the event of liquefaction settlement, exterior concrete slabs on grade may need to be re-leveled, or demolished and replaced.

5.4. CBC SEISMIC DESIGN PARAMETERS

Based on the Alquist-Priolo Earthquake Fault Zone mapping data provided by CGS (http://www.conservation.ca.gov/cgs/rghm/ap/Pages/Index.aspx), the site is not located within an Alquist-Priolo Earthquake Fault Zone. However, the site is in a region of high seismic activity and will likely be subjected to major shaking during the life of the project. As a result, structures to be constructed on the site should be designed in accordance with applicable seismic provisions of the building codes.

We assume that seismic design will be based on the 2013 California Building Code (CBC), which is based on the 2012 International Building Code (IBC) and the American Society of Civil Engineers (ASCE) Standard 7-10. Based on information obtained during our field exploration, published geologic literature and maps, and our interpretation of CBC criteria, it is our opinion that this site can be classified as Site Class D according to Table 20.3-1 of ASCE 7-10. This classification applies to a stiff soil condition in the upper 100 feet, with Standard Penetration Test (SPT) blow counts generally between 15 and 50 blows per foot.

Section 20.3.1 states that a site underlain by soil susceptible to liquefaction should be classified as site Class F. However, because the proposed structures for this project are assumed to have a fundamental period of equal to or less than 0.5 seconds, the exception provided in Section 20.3.1 applies and the site can be classified according to Table 20.3-1 of ASCE 7-10 (Site Class D as indicated above). Detailed seismic analysis was not included in our scope of work. If the periods for the proposed structures exceed 0.5 seconds, Kleinfelder should be engaged to provide this additional analysis.

To provide the ground motion parameters associated with the 2013 CBC, an online tool (http://geohazards.usgs.gov/designmaps/us/application.php) was used, which was developed by the USGS based on the Seismic Design Maps in the 2012 IBC. Estimated values of PGA are based on mapped values of Maximum Considered Earthquake Geometric Mean (MCEG) Peak Ground Accelerations (Figure 22-7, ASCE 7-10). The resulting 2013 CBC seismic design factors (for a risk factor of I, II, or III) are summarized in the following table.

Table 1
2013 CBC Seismic Design Parameters

Design Parameter	Recommended Value
Site Class	D
S_s (g)	1.658
S_1 (g)	0.585
F_a	1.0
F_v	1.5
S_{MS} (g)	1.658
S_{M1} (g)	0.877
S_{DS} (g)	1.105
S_{D1} (g)	0.585
PGA_M (g)	0.599

6. LIMITATIONS

This work was performed in a manner consistent with that level of care and skill ordinarily exercised by other members of Kleinfelder's profession practicing in the same locality, under similar conditions and at the date the services are provided. Our conclusions, opinions and recommendations are based on a limited number of observations and data. It is possible that conditions could vary between or beyond the data evaluated. Kleinfelder makes no other representation, guarantee or warranty, express or implied, regarding the services, communication (oral or written), report, opinion, or instrument of service provided.

This report may be used only by the Client and the registered design professional in responsible charge and only for the purposes stated for this specific engagement within a reasonable time from its issuance, but in no event later than two (2) years from the date of the report.

The work performed was based on project information provided by Client. If Client does not retain Kleinfelder to review any plans and specifications, including any revisions or modifications to the plans and specifications, Kleinfelder assumes no responsibility for the suitability of our recommendations. In addition, if there are any changes in the field to the plans and specifications, Client must obtain written approval from Kleinfelder's engineer that such changes do not affect our recommendations. Failure to do so will vitiate Kleinfelder's recommendations.

The scope of services was limited to four borings, utility locating, laboratory testing of selected soil samples, engineering analysis, and preparation of this report. It should be recognized that definition and evaluation of subsurface conditions are difficult. Judgments leading to conclusions and recommendations are generally made with incomplete knowledge of the subsurface conditions present due to the limitations of data from field studies. The conclusions of this assessment are based on four borings to a maximum depth of about 31½ feet below the existing ground surface, groundwater level measurements, laboratory testing, and engineering analyses.

Kleinfelder offers various levels of investigative and engineering services to suit the varying needs of different clients. Although risk can never be eliminated, more detailed and extensive studies yield more information, which may help understand and manage the level of risk. Since detailed study and analysis involves greater expense, our clients participate in determining levels of service, which provide information for their purposes at acceptable levels of risk. The client and key members of the design team should discuss the issues covered in this report with Kleinfelder, so that the issues are understood and applied in a manner consistent with the owner's budget, tolerance of risk and expectations for future performance and maintenance.

Recommendations contained in this report are based on our field observations and subsurface explorations, limited laboratory tests, and our present knowledge of the proposed construction. It is possible that soil, rock or groundwater conditions could vary between or beyond the points explored. If soil, rock or groundwater conditions are encountered during construction that differ from those described herein, the client is responsible for ensuring that Kleinfelder is notified immediately so that we may reevaluate the recommendations of this report. If the scope of the proposed construction, including the estimated structural loads, and the design depths or locations of the foundations, changes from that described in this report, the conclusions and recommendations contained in this report are not considered valid unless the changes are reviewed, and the conclusions of this report are modified or approved in writing, by Kleinfelder.

As the geotechnical engineering firm that performed the geotechnical evaluation for this project, Kleinfelder should be retained to confirm that the recommendations of this report are properly incorporated in the design of this project, and properly implemented during construction. This may avoid misinterpretation of the information by other parties and will allow us to review and modify our recommendations if variations in the soil conditions are encountered.

As a minimum Kleinfelder should be retained to provide the following continuing services for the project:

- Review the pre-final project plans and specifications, including any revisions or modifications;

- Observe and evaluate the site earthwork operations to confirm subgrade soils are suitable for construction of slabs-on-grade, other walkways/pavers, and placement of engineered fill;
- Confirm engineered fill and utility trench backfill are placed and compacted per the project specifications; and
- Observe new pond foundation excavations to confirm subsurface conditions are as anticipated and to verify adequate structural support.

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

Kleinfelder cannot be responsible for interpretation by others of this report or the conditions encountered in the field. Kleinfelder must be retained so that all geotechnical aspects of construction will be monitored on a full-time basis by a representative from Kleinfelder, including site preparation, preparation of foundations, and placement of engineered fill and trench backfill. These services provide Kleinfelder the opportunity to observe the actual soil, rock and groundwater conditions encountered during construction and to evaluate the applicability of the recommendations presented in this report to the site conditions. If Kleinfelder is not retained to provide these services, we will cease to be the engineer of record for this project and will assume no responsibility for any potential claim during or after construction on this project. If changed site conditions affect the recommendations presented herein, Kleinfelder must also be retained to perform a supplemental evaluation and to issue a revision to our original report.

This report, and any future addenda or reports regarding this site, may be made available to bidders to supply them with only the data contained in the report regarding subsurface conditions and laboratory test results at the point and time noted. Bidders may not rely on interpretations, opinion, recommendations, or conclusions contained in the report. Because of the limited nature of any subsurface study, the contractor may encounter conditions during construction which differ from those presented in this report. In such event, the contractor should promptly notify the owner so that Kleinfelder's geotechnical engineer can be contacted to confirm those conditions. We recommend the contractor describe the nature and extent of the differing conditions in writing and that the construction contract include provisions for dealing with differing conditions. Contingency funds should be reserved for potential problems during earthwork and foundation construction. Furthermore, the contractor should be prepared to handle contamination conditions if encountered at this site, which may affect the excavation, removal, or disposal of soil; dewatering of excavations; and health and safety of workers.

FIGURES

0 2000 4000

SCALE: 1" = 2000' SCALE IN FEET

REFERENCE: Google Earth Pro.,
Imagery date 4-13-2015

SITE

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PROJECT NO.	20163829
DRAWN BY:	JDS
CHECKED BY:	RH
DATE:	02-22-2016
REVISED:	

SITE VICINITY MAP

EAST/WEST WING
RENOVATION COURTYARD
240 CHURCH STREET
SALINAS, CALIFORNIA

FIGURE
1

0 60 120

SCALE: 1" = 60' SCALE IN FEET

REFERENCE: Google Earth Pro., Imagery date 4-13-2015

LEGEND

COURTYARD PERIMETER

B-1 SOIL BORING (By Kleinfelder, 2/2016)

The information included on this graphic representation has been compiled from a variety of sources and is subject to change without notice. Kleinfelder makes no representations or warranties, express or implied, as to accuracy, completeness, timeliness, or rights to the use of such information. This document is not intended for use as a land survey product nor is it designed or intended as a construction design document. The use or misuse of the information contained on this graphic representation is at the sole risk of the party using or misusing the information.

NOTE: Locations are approximate.

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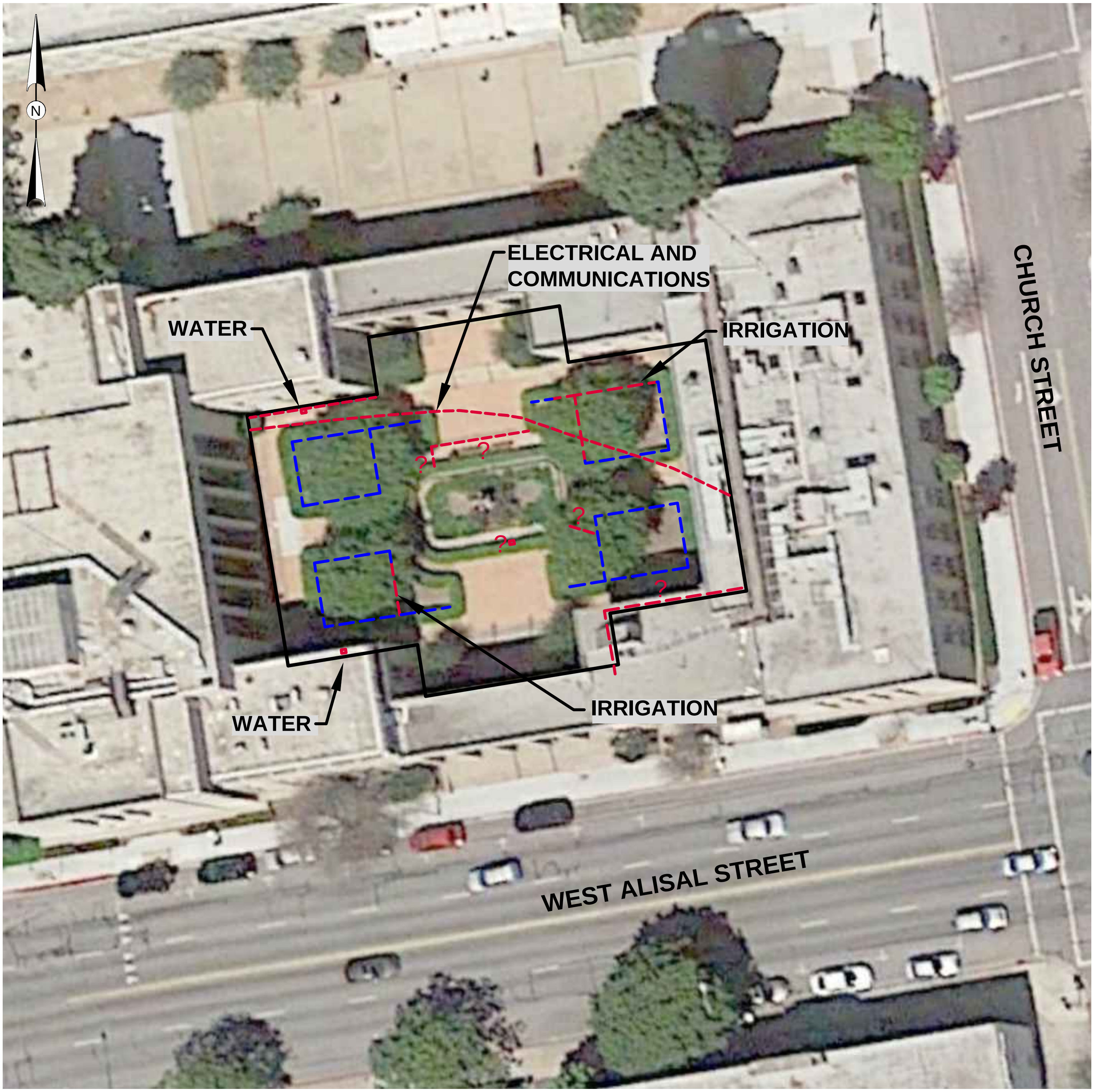
PROJECT NO. 20163829	SITE PLAN	FIGURE
DRAWN BY: JDS		
CHECKED BY: RH	EAST/WEST WING RENOVATION COURTYARD 240 CHURCH STREET SALINAS, CALIFORNIA	2
DATE: 02-22-2016		
REVISED:		

PLOTTED: 2/22/2016 10:42 AM BY: jeff sala

CAD FILE: L:\2016\Projects\20163829.001A - East-West Wing Renovation Courtyard\SITE.dwg

PLOTTED: 2/25/2016 1:13 PM BY: jeff sala



0 40 80

SCALE: 1" = 40' SCALE IN FEET

REFERENCE: Google Earth Pro., Imagery date 4-13-2015

LEGEND

- COURTYARD PERIMETER
- CONJECTURED IRRIGATION LINES
- DETECTED UTILITY LINES

NOTE: Locations are approximate.

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PROJECT NO.	20163829
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CHECKED BY:	RH
DATE:	02-25-2016
REVISED:	

UTILITY PLAN

EAST/WEST WING
RENOVATION COURTYARD
240 CHURCH STREET
SALINAS, CALIFORNIA

FIGURE

3

CAD FILE: L:\2016\Projects\20163829.001A - East-West Wing Renovation Courtyard\SITE.dwg

APPENDIX A

SAMPLE/SAMPLER TYPE GRAPHICS

CALIFORNIA SAMPLER
(3 in. (76.2 mm.) outer diameter)

STANDARD PENETRATION SPLIT SPOON SAMPLER
(2 in. (50.8 mm.) outer diameter and 1-3/8 in. (34.9 mm.) inner diameter)

GROUND WATER GRAPHICS

- WATER LEVEL (level where first observed)
- WATER LEVEL (level after exploration completion)
- WATER LEVEL (additional levels after exploration)
- OBSERVED SEEPAGE

NOTES

- The report and graphics key are an integral part of these logs. All data and interpretations in this log are subject to the explanations and limitations stated in the report.
- Lines separating strata on the logs represent approximate boundaries only. Actual transitions may be gradual or differ from those shown.
- No warranty is provided as to the continuity of soil or rock conditions between individual sample locations.
- Logs represent general soil or rock conditions observed at the point of exploration on the date indicated.
- In general, Unified Soil Classification System designations presented on the logs were based on visual classification in the field and were modified where appropriate based on gradation and index property testing.
- Fine grained soils that plot within the hatched area on the Plasticity Chart, and coarse grained soils with between 5% and 12% passing the No. 200 sieve require dual USCS symbols, ie., GW-GM, GP-GM, GW-GC, GP-GC, GC-GM, SW-SM, SP-SM, SW-SC, SP-SC, SC-SM.
- If sampler is not able to be driven at least 6 inches then 50/X indicates number of blows required to drive the identified sampler X inches with a 140 pound hammer falling 30 inches.

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D 2487)

Major Division	Subdivision	Fines	Criteria	Symbol	Description
COARSE GRAINED SOILS (More than half of material is larger than the #200 sieve)	GRAVELS (More than half of coarse fraction is larger than the #4 sieve)	CLEAN GRAVEL WITH <5% FINES	Cu ≥4 and 1≤ Cc≤3	GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
			Cu <4 and/ or 1>Cc >3	GP	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
		GRAVELS WITH 5% TO 12% FINES	Cu ≥4 and 1≤ Cc≤3	GW-GM	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE FINES
				GW-GC	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE CLAY FINES
			Cu <4 and/ or 1>Cc >3	GP-GM	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE FINES
				GP-GC	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE CLAY FINES
		GRAVELS WITH > 12% FINES		GM	SILTY GRAVELS, GRAVEL-SILT-SAND MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
				GC-GM	CLAYEY GRAVELS, GRAVEL-SAND-CLAY-SILT MIXTURES
	SANDS (More than half of coarse fraction is smaller than the #4 sieve)	CLEAN SANDS WITH <5% FINES	Cu ≥6 and 1≤ Cc≤3	SW	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
			Cu <6 and/ or 1>Cc >3	SP	POORLY GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
		SANDS WITH 5% TO 12% FINES	Cu ≥6 and 1≤ Cc≤3	SW-SM	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE FINES
				SW-SC	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE CLAY FINES
			Cu <6 and/ or 1>Cc >3	SP-SM	POORLY GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE FINES
				SP-SC	POORLY GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE CLAY FINES
		SANDS WITH > 12% FINES		SM	SILTY SANDS, SAND-GRAVEL-SILT MIXTURES
				SC	CLAYEY SANDS, SAND-GRAVEL-CLAY MIXTURES
				SC-SM	CLAYEY SANDS, SAND-SILT-CLAY MIXTURES

Major Division	Group	Symbol	Description
FINE GRAINED SOILS (More than half of material is smaller than the #200 sieve)	SILTS AND CLAYS (Liquid Limit less than 50)	ML	INORGANIC SILTS AND VERY FINE SANDS, SILTY OR CLAYEY FINE SANDS, SILTS WITH SLIGHT PLASTICITY
		CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
		CL-ML	INORGANIC CLAYS-SILTS OF LOW PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
		OL	ORGANIC SILTS & ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS (Liquid Limit greater than 50)	MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILT
		CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
		OH	ORGANIC CLAYS & ORGANIC SILTS OF MEDIUM-TO-HIGH PLASTICITY

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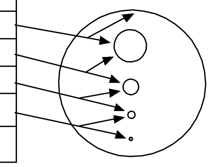
PROJECT NO.:	20163829	GRAPHICS KEY	FIGURE
DRAWN BY:	JDS		A-1
CHECKED BY:	RH	EAST/WEST WING RENOVATION COURTYARD 240 CHURCH STREET SALINAS, CALIFORNIA	
DATE:	2/17/2016		
REVISED:	-		

GRAIN SIZE

DESCRIPTION		SIEVE SIZE	GRAIN SIZE	APPROXIMATE SIZE
Boulders		>12 in. (304.8 mm.)	>12 in. (304.8 mm.)	Larger than basketball-sized
Cobbles		3 - 12 in. (76.2 - 304.8 mm.)	3 - 12 in. (76.2 - 304.8 mm.)	Fist-sized to basketball-sized
Gravel	coarse	3/4 -3 in. (19 - 76.2 mm.)	3/4 -3 in. (19 - 76.2 mm.)	Thumb-sized to fist-sized
	fine	#4 - 3/4 in. (#4 - 19 mm.)	0.19 - 0.75 in. (4.8 - 19 mm.)	Pea-sized to thumb-sized
Sand	coarse	#10 - #4	0.079 - 0.19 in. (2 - 4.9 mm.)	Rock salt-sized to pea-sized
	medium	#40 - #10	0.017 - 0.079 in. (0.43 - 2 mm.)	Sugar-sized to rock salt-sized
	fine	#200 - #40	0.0029 - 0.017 in. (0.07 - 0.43 mm.)	Flour-sized to sugar-sized
Fines		Passing #200	<0.0029 in. (<0.07 mm.)	Flour-sized and smaller

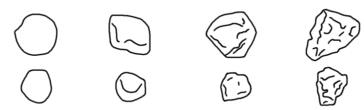


Munsell Color

NAME	ABBR
Red	R
Yellow Red	YR
Yellow	Y
Green Yellow	GY
Green	G
Blue Green	BG
Blue	B
Purple Blue	PB
Purple	P
Red Purple	RP
Black	N

ANGULARITY

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



Particles Present

Amount	Percentage
trace	<5
few	5-10
little	15-25
some	30-45
and	50
mostly	50-100

PLASTICITY

DESCRIPTION	LL	FIELD TEST
Non-plastic	NP	A 1/8-in. (3 mm.) thread cannot be rolled at any water content.
Low (L)	< 30	The thread can barely be rolled and the lump or thread cannot be formed when drier than the plastic limit.
Medium (M)	30 - 50	The thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be rerolled after reaching the plastic limit. The lump or thread crumbles when drier than the plastic limit
High (H)	> 50	It takes considerable time rolling and kneading to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump or thread can be formed without crumbling when drier than the plastic limit

MOISTURE CONTENT

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

REACTION WITH HYDROCHLORIC ACID

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

APPARENT / RELATIVE DENSITY - COARSE-GRAINED SOIL

APPARENT DENSITY	SPT-N_{60} (# blows/ft)	MODIFIED CA SAMPLER (# blows/ft)	CALIFORNIA SAMPLER (# blows/ft)	RELATIVE DENSITY (%)
Very Loose	<4	<4	<5	0 - 15
Loose	4 - 10	5 - 12	5 - 15	15 - 35
Medium Dense	10 - 30	12 - 35	15 - 40	35 - 65
Dense	30 - 50	35 - 60	40 - 70	65 - 85
Very Dense	>50	>60	>70	85 - 100

NOTE: AFTER TERZAGHI AND PECK, 1948

CONSISTENCY - FINE-GRAINED SOIL

CONSISTENCY	UNCONFINED COMPRESSIVE STRENGTH (q_u)(psf)	CRITERIA
Very Soft	< 1000	Thumb will penetrate soil more than 1 in. (25 mm.)
Soft	1000 - 2000	Thumb will penetrate soil about 1 in. (25 mm.)
Firm	2000 - 4000	Thumb will indent soil about 1/4-in. (6 mm.)
Hard	4000 - 8000	Thumb will not indent soil but readily indented with thumbnail
Very Hard	> 8000	Thumbnail will not indent soil

STRUCTURE

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

CEMENTATION

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

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PROJECT NO.: 20163829	SOIL DESCRIPTION KEY	FIGURE
DRAWN BY: JDS		
CHECKED BY: RH	EAST/WEST WING RENOVATION COURTYARD 240 CHURCH STREET SALINAS, CALIFORNIA	A-2
DATE: 2/17/2016		
REVISED: -		

PLOTTED: 02/22/2016 02:17 PM BY: JSala

BORING LOG B-1

Date Begin - End:	2/01/2016	**Drilling Co.-Lic.#:**	Cenozoic - #682910		
Logged By:	R. Hasseler	**Drill Crew:**	Mike & Dan		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	Portable Drill	**Hammer Type - Drop:**	140 lb. Rope & Cathead - 3
Plunge:	-90 degrees	**Drilling Method:**	Solid Flight Auger		
Weather:	Clear/Cold	**Exploration Diameter:**	4 in. O.D.		

FIELD EXPLORATION / LABORATORY RESULTS

Surface Condition: Grass/Landscape

Depth (feet)	Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
0	**Silty CLAY (CL-ML)**: medium plasticity, topsoil, dark brown, wet, soft												
1		1		BC=4 3 6	12"								
2	**Fat CLAY (CH)**: very high plasticity, olive brown, moist, soft gravels from 3' to 4' mixed with silty clay						33.0	82.6					TXUU Su = 074 ksf
5		2		BC=3 8 11	12"								
8	**Poorly graded SAND with Silt (SP-SM)**: fine-grained sand, non-plastic, yellow brown, moist, medium dense												
10		3		BC=6 8 8 PP=.5	18"								
15	**Sandy SILT (ML)**: fine-grained sand, low plasticity, olive brown, moist, soft	4		BC=4 5 8	18"								
20		5		BC=5 9 12	18"								
21	**Poorly graded SAND with Silt (SP-SM)**: fine-grained sand, non-plastic, olive brown, moist, medium dense wet												
25		6		BC=2 4 11	18"	SM				13			
29	**Poorly graded SAND (SP)**: fine-grained sand, non-plastic, gray brown, wet, loose, trace medium grained sand												
30		7		BC=4 4 5	18"								
31	**Well-graded SAND (SW)**: fine to coarse-grained sand, non-plastic, gray brown, wet, loose												
35													

The boring was terminated at approximately 31.5 ft. below ground surface. The boring was backfilled with auger cuttings on February 01, 2016.

GROUNDWATER LEVEL INFORMATION:
- ▽ Groundwater was observed at approximately 23 ft. below ground surface during drilling.
- ▼ Groundwater was observed at approximately 23 ft. below ground surface at the end of drilling.

GENERAL NOTES:

KLEINFELDER Bright People. Right Solutions.	PROJECT NO.: 20163829	BORING LOG B-1	FIGURE
	DRAWN BY: JDS		
	CHECKED BY: RH	EAST/WEST WING RENOVATION COURTYARD 240 CHURCH STREET SALINAS, CALIFORNIA	A-3
	DATE: 2/17/2016		
	REVISED: -		PAGE: 1 of 1

gINT FILE: L:\2016\projects\20163829.001a - East-West Wing Renovation Courtyard\20163829 Blogs.gpj
gINT TEMPLATE: PROJECTWISE: KLF_STANDARD_GINT_LIBRARY_2016.GLB [KLF_BORING/TEST PIT SOIL LOG]

Date Begin - End: 2/01/2016
Logged By: R. Hasseler
Hor.-Vert. Datum: Not Available
Plunge: -90 degrees
Weather: Clear/Cold

Drilling Co.-Lic.#: Cenozoic - #682910
Drill Crew: Mike & Dan
Drilling Equipment: Portable Drill
Drilling Method: Solid Flight Auger
Exploration Diameter: 4 in. O.D.

Hammer Type - Drop: 140 lb. Rope & Cathead - 30 in.

BORING LOG B-2

Surface Condition: Grass/Landscape

Depth (feet)	Lithologic Description	Sample Number	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
	Approximately 1-inch of topsoil											R-Value = 3 psi
	Silty CLAY (CL-ML): medium plasticity, dark brown, moist to wet, soft	1	BC=7 PP=1.75	6"								
	Fat CLAY (CH): very high plasticity, olive brown, moist, soft											
	brick debris in clay matrix from 1..5 feet to 2.5 feet											
5	**Fat CLAY (CH)**: very high plasticity, dark brown, moist, soft	2	BC=5 6 8 PP=.5	7"		33.3	83.8			55	27	
	Poorly graded SAND with Silt (SP): fine-grained sand, non-plastic, yellowish brown, moist, loose to medium dense											
10		3	BC=5 5 5	18"								
	denser											
	less dense											
15	**Sandy SILT (ML)**: fine-grained sand, low plasticity, olive brown, moist, soft	4	BC=3 8 10	18"								
20	**Poorly graded SAND with Silt (SP-SM)**: fine-grained sand, non-plastic, olive brown, moist, medium dense	5	BC=6 8 9	18"								
	wet											
25		6	BC=3 5 7	18"								
30		7	BC=6 7 7	18"								
	Well-graded SAND (SW): fine to coarse-grained sand, non-plastic, olive brown, wet, medium dense											
	Lean CLAY (CL): low plasticity, olive brown, wet, firm											
35												

The boring was terminated at approximately 31.5 ft. below ground surface. The boring was backfilled with auger cuttings on February 01, 2016.

GROUNDWATER LEVEL INFORMATION:
Groundwater was observed at approximately 23 ft. below ground surface during drilling.
GENERAL NOTES:

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PROJECT NO.:	20163829
DRAWN BY:	JDS
CHECKED BY:	RH
DATE:	2/17/2016
REVISED:	-

BORING LOG B-2

EAST/WEST WING
RENOVATION COURTYARD
240 CHURCH STREET
SALINAS, CALIFORNIA

FIGURE
A-4

Date Begin - End:	2/02/2016	**Drilling Co.-Lic.#:**	Cenozoic - #682910		**BORING LOG B-3**
Logged By:	R. Hasseler	**Drill Crew:**	Mike & Dan		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	Portable Drill	**Hammer Type - Drop:**	140 lb. Rope & Cathead - 3
Plunge:	-90 degrees	**Drilling Method:**	Solid Flight Auger		
Weather:	Overcast/Rain	**Exploration Diameter:**	4 in. O.D.		

Depth (feet)	Graphical Log	FIELD EXPLORATION: Surface Condition: Brick Herringbone — Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	LABORATORY RESULTS: USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
		Approximately 4-inches of Brick												
		Fat CLAY (CH): very high plasticity, olive brown, moist, firm	1		BC=4 5 5	10"		25.6						
5			2		BC=5 7 9	18"								
		Poorly graded SAND with Silt (SP-SM): fine-grained sand, non-plastic, yellowish brown, moist, medium dense												
10			3		BC=5 5 6	15"								
		Sandy SILT (ML): fine-grained sand, low plasticity, olive brown, moist, soft												
15		**Lean CLAY (CL)**: low plasticity, olive brown, moist, soft	4		BC=4 5 8	18"								
		Sandy SILT (ML): fine-grained sand, low plasticity, olive brown, moist, soft												
		Lean CLAY (CL): low plasticity, olive brown, moist, soft												
20		**Poorly graded SAND with Silt (SP-SM)**: fine-grained sand, non-plastic, olive brown, moist, medium dense	5		BC=6 8 8	18"								
⊻		wet												
25		**Silty SAND (SM)**: fine-grained sand, non-plastic, olive brown, wet, medium dense	6		BC=5 7 10	18"					10			
		denser at 27'												
30			7		BC=4 3 3	18"								
		Lean CLAY (CL): low plasticity, olive brown, wet, soft												
35														

The boring was terminated at approximately 31.5 ft. below ground surface. The boring was backfilled with auger cuttings and patched with concrete on February 02, 2016.

GROUNDWATER LEVEL INFORMATION:
⊻ Groundwater was observed at approximately 23 ft. below ground surface during drilling.
GENERAL NOTES:

KLEINFELDER Bright People. Right Solutions.	PROJECT NO.: 20163829	BORING LOG B-3	FIGURE
	DRAWN BY: JDS		A-5
	CHECKED BY: RH	EAST/WEST WING RENOVATION COURTYARD 240 CHURCH STREET SALINAS, CALIFORNIA	
	DATE: 2/17/2016		
	REVISED: -		PAGE: 1 of 1

Date Begin - End:	2/02/2016	**Drilling Co.-Lic.#:**	Cenozoic - #682910		**BORING LOG B-4**
Logged By:	R. Hasseler	**Drill Crew:**	Mike & Dan		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	Portable Drill	**Hammer Type - Drop:**	140 lb. Rope & Cathead - 3
Plunge:	-90 degrees	**Drilling Method:**	Solid Flight Auger		
Weather:	Overcast/Rain	**Exploration Diameter:**	4 in. O.D.		

FIELD EXPLORATION / LABORATORY RESULTS

Surface Condition: Brick Herringbone

Depth (feet)	Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
	Approximately 4-inches of Brick												
	Fat CLAY (CH): very high plasticity, olive brown, moist, firm	1		BC=4 5 7	18"		31.1				66	39	
5	dark brown	2		BC=3 3 5	18"								
	Poorly graded SAND with Silt (SP-SM): fine-grained sand, non-plastic, yellowish brown, moist												
10	**Lean CLAY (CL)**: medium plasticity, olive brown, moist, soft	3		BC=2 2 2	18"								
	Poorly graded SAND with Silt (SP-SM): yellowish brown, moist												
15	**Sandy SILT (ML)**: fine-grained sand, low plasticity, olive brown, moist, firm	4		BC=4 6 10	18"								
20	medium plasticity, dark brown, moist soft to firm	5		BC=4 8 9	18"								
▽	**Poorly graded SAND with Silt (SP-SM)**: fine-grained sand, non-plastic, olive brown, wet, medium dense												
25		6		BC=7 7 12	18"								
30	**Lean CLAY (CL)**: low plasticity, olive brown, moist to wet, soft	7		BC=6 6 7									

The boring was terminated at approximately 31.5 ft. below ground surface. The boring was backfilled with auger cuttings and patched with concrete on February 02, 2016.

35

GROUNDWATER LEVEL INFORMATION:
▽ Groundwater was observed at approximately 23 ft. below ground surface during drilling.
GENERAL NOTES:

KLEINFELDER
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PROJECT NO.: 20163829	BORING LOG B-4	FIGURE
DRAWN BY: JDS		
CHECKED BY: RH	EAST/WEST WING RENOVATION COURTYARD 240 CHURCH STREET SALINAS, CALIFORNIA	A-6
DATE: 2/17/2016		
REVISED: -		PAGE: 1 of 1

APPENDIX B

Exploration ID	Depth (ft.)	Sample No.	Sample Description	Water Content (%)	Dry Unit Wt. (pcf)	Sieve Analysis (%)			Atterberg Limits			Additional Tests
						Passing 3/4"	Passing #4	Passing #200	Liquid Limit	Plastic Limit	Plasticity Index	
B-1	2.0		OLIVE BROWN FAT CLAY (CH)	33.0	82.6							TXUU Su = 074 ksf
B-1	25.0	6	OLIVE BROWN POORLY GRADED SAND WITH SILT (SP-SM)					13				
B-2	0.0 - 2.0		OLIVE BROWN FAT CLAY (CH)									R-Value = 3 psi
B-2	6.0		DARK BROWN FAT CLAY (CH)	33.3	83.8				55	28	27	
B-3	1.0	1	OLIVE BROWN FAT CLAY (CH)	25.6								
B-3	25.0	6	OLIVE BRWON POORLY GRADED SAND WITH SILT (SP-SM)					10				
B-4	1.0	1	OLIVE BROWN FAT CLAY (CH)	31.1					66	27	39	

Refer to the Geotechnical Evaluation Report or the supplemental plates for the method used for the testing performed above.
NP = NonPlastic
NA = Not Available

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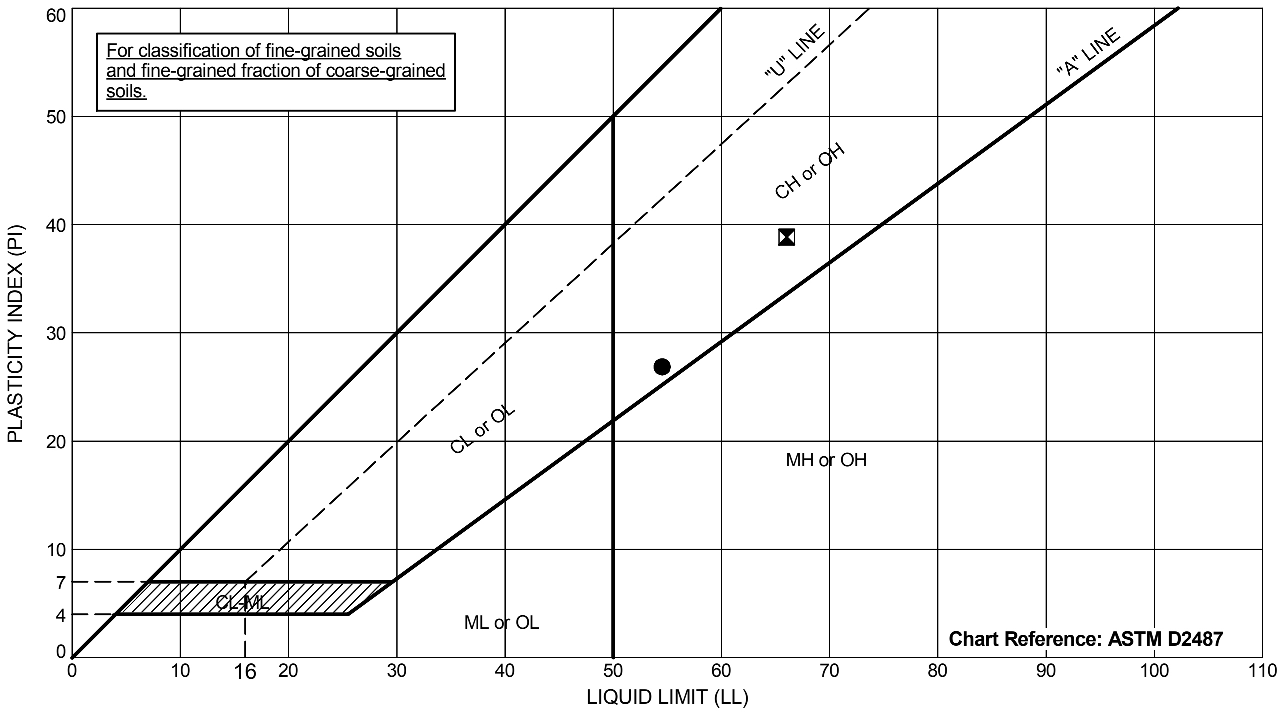
PROJECT NO.:	20163829
DRAWN BY:	JDS
CHECKED BY:	RH
DATE:	2/17/2016
REVISED:	-

LABORATORY TEST
RESULT SUMMARY

EAST/WEST WING
RENOVATION COURTYARD
240 CHURCH STREET
SALINAS, CALIFORNIA

FIGURE
B-1



	Exploration ID	Depth (ft.)	Sample Number	Sample Description	Passing #200	LL	PL	PI
●	B-2	6	NA	DARK BROWN FAT CLAY (CH)	NM	55	28	27
⊠	B-4	1	1	OLIVE BROWN FAT CLAY (CH)	NM	66	27	39

Testing perfomed in general accordance with ASTM D4318.
NP = Nonplastic
NA = Not Available
NM = Not Measured

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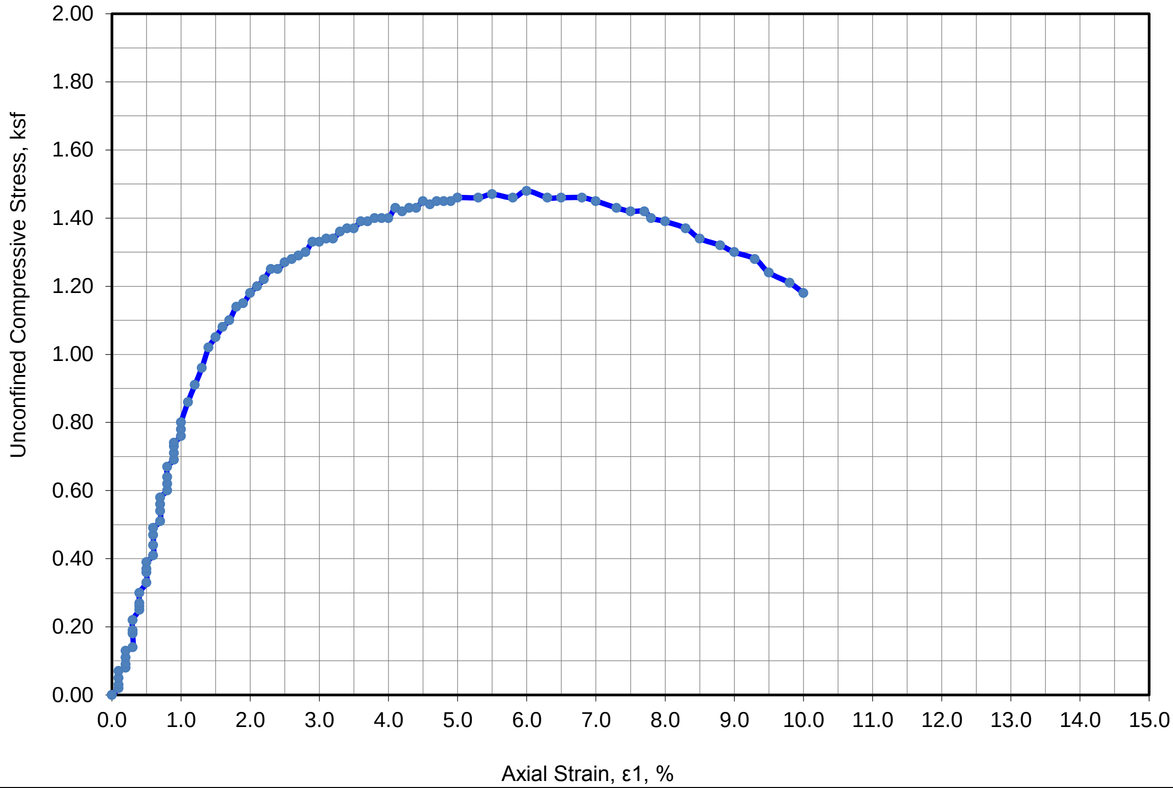
PROJECT NO.:	20163829
DRAWN BY:	JDS
CHECKED BY:	RH
DATE:	2/17/2016
REVISED:	-

ATTERBERG LIMITS

EAST/WEST WING
RENOVATION COURTYARD
240 CHURCH STREET
SALINAS, CALIFORNIA

B-2



Specimen Failure Picture	Specimen No.			1
	Initial	Diameter, in	D_O	2.38
		Height, in	H_O	5.63
		Height to Diameter Ratio		2.36
		Water Content, %	ω_O	33.0
		Dry Density, lbs/ft^3	γd_o	82.6
		Saturation, %	S_O	87.3
		Void Ratio	e_O	1.002
	Time to Failure, min.		t_f	6.0
	Unconfined Compressive Strength, ksf		q_u	1.48
	Shear Strength, ksf		s_u	0.74
	Strain at Failure, %		ε_f	6.0
	Average Rate of Strain to Failure, %/min		ε	1.0

Description of Specimen: Olive Fat Clay (CH)

Amount of Material Finer than the No. 200, %: nm

LL: nm	PL: nm	PI: nm	G_S: 2.65 Assumed	Specimen Type: Intact	Test Method: ASTM D2166

Boring:	B-1	Remarks: nm= not measured, na = not applicable
Sample:	1-1-1	
Depth, ft:	2.0	
Test Date:	2/10/16	

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2601 Barrington Court, Hayward, California 94545

Project No.:	20163829	UNCONFINED COMPRESSION TEST (UC)	FIGURE
Date:	2/12/2016		
Entry By:	CP	EAST/WEST WING RENOVATION COURTYARD 240 CHURCH STREET SALINAS CALIFORNIA	B-3
Checked By:	CP		
File Name:	HL8602		

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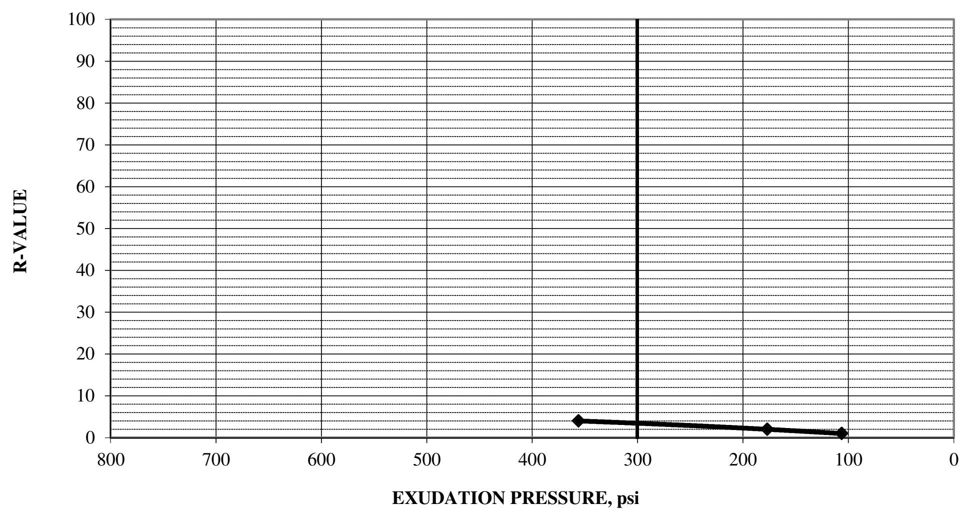
Laboratory Test Report

Project Name: COUNTY OF MONTEREY
Project No.: 20163829
Lab No.: HL8602
Sample Date: February 1, 2016
Sample No.: East-West Wing Courtyard
Sample Location: B-2 @ 0-2.0'
Material Description: Sandy Lean Clay
Report Date: February 12, 2016

Resistance R-Value and Expansion Pressure of Compacted Soils (ASTM D2844, CTM 301)



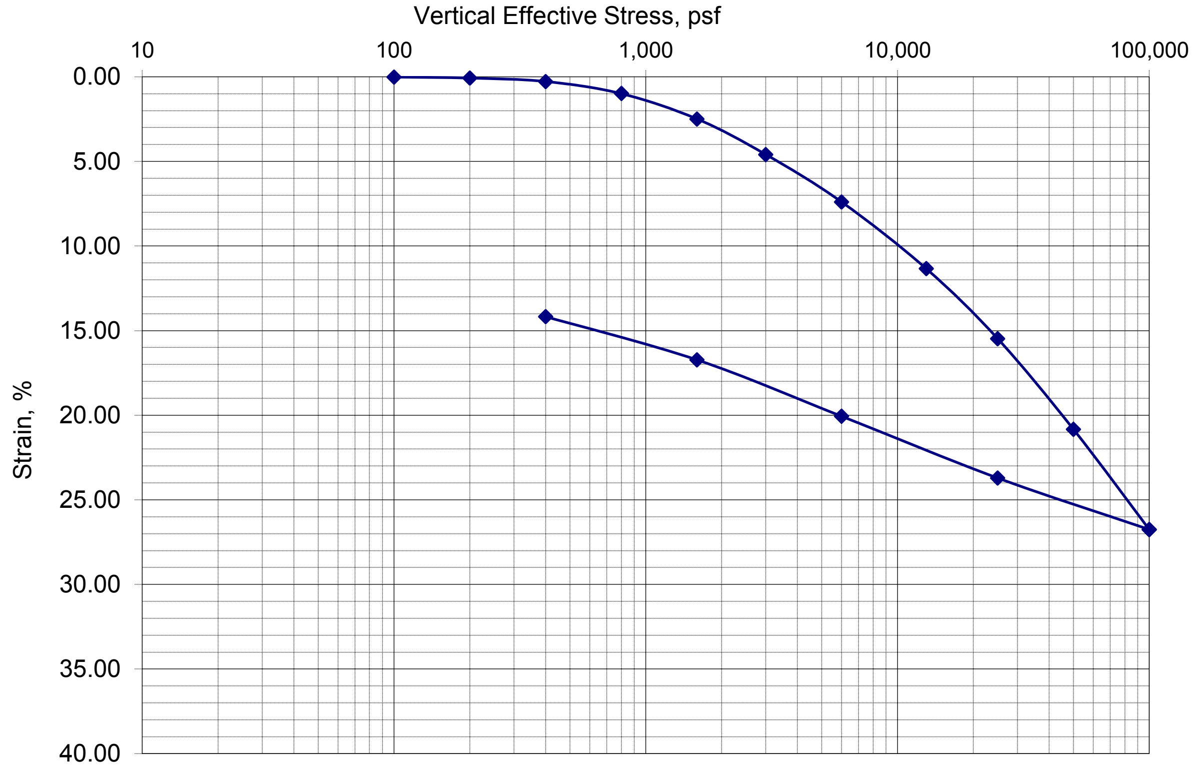
Briquette No.	A	B	C
Moisture at Test, %	33.7	31.6	28.5
Dry Unit Weight at Test, pcf	82.2	84.8	88.8
Expansion Pressure, psf	35	65	108
Exudation Pressure, psi	107	177	356
Resistance Value	1	2	4
R - Value at 300 psi Exudation Pressure:			**3**

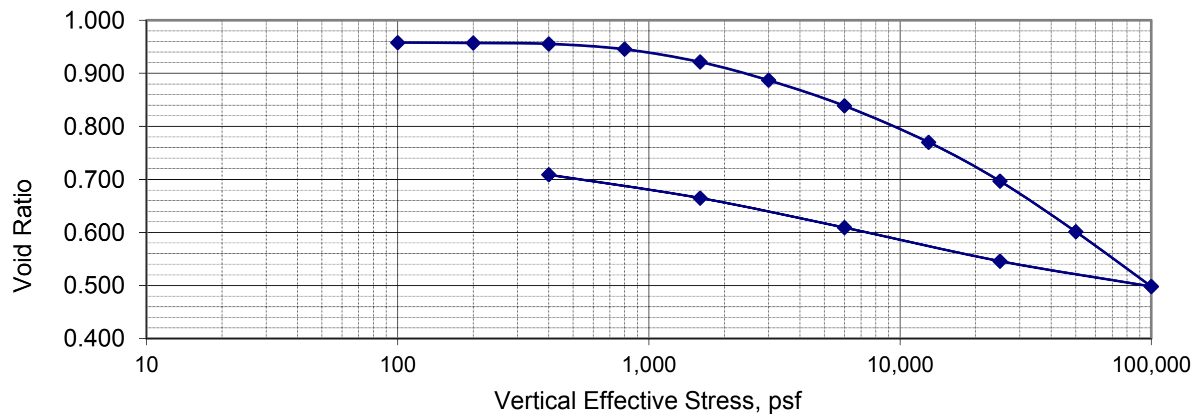
Reviewed By on 2/12/2016:

for **Aaron Kidd**
Laboratory Manager

Limitations: Pursuant to applicable building codes, the results presented in this report are for the exclusive use of the client and the registered design professional in responsible charge. The results apply only to the samples tested. If changes to the specifications were made and not communicated to Kleinfelder, Kleinfelder assumes no responsibility for pass/fail statements (meets/did not meet), if provided.

KLEINFELDER Bright People. Right Solutions.	PROJECT NO.: 20163829 DRAWN BY: JDS CHECKED BY: RH DATE: 2/22/2016 REVISED:	COMPACTION CURVE EAST/WEST WING RENOVATION COURTYARD 240 CHURCH STREET SALINAS, CALIFORNIA	PLATE B-4





Test Method: ASTM D2435			Sample Type: Intact				
Gs: 2.65 Measured	LL: nm		PI: nm	Amount of Material Finer than the No. 200, %: nm			
	Height, in.	Diameter, in.	Water Content, %	Wet Density, lb/f^3	Dry Density, lb/f^3	Saturation, %	Void Ratio
Initial	0.795	1.973	33.9	113.1	84.5	94.0	0.958
Final	0.694	1.973	28.8	125.4	97.3	108.0	0.709

Boring:	B-1	Remarks:
Sample:	1-2-1	
Depth, ft:	6	
Test Date:	2/10/16	

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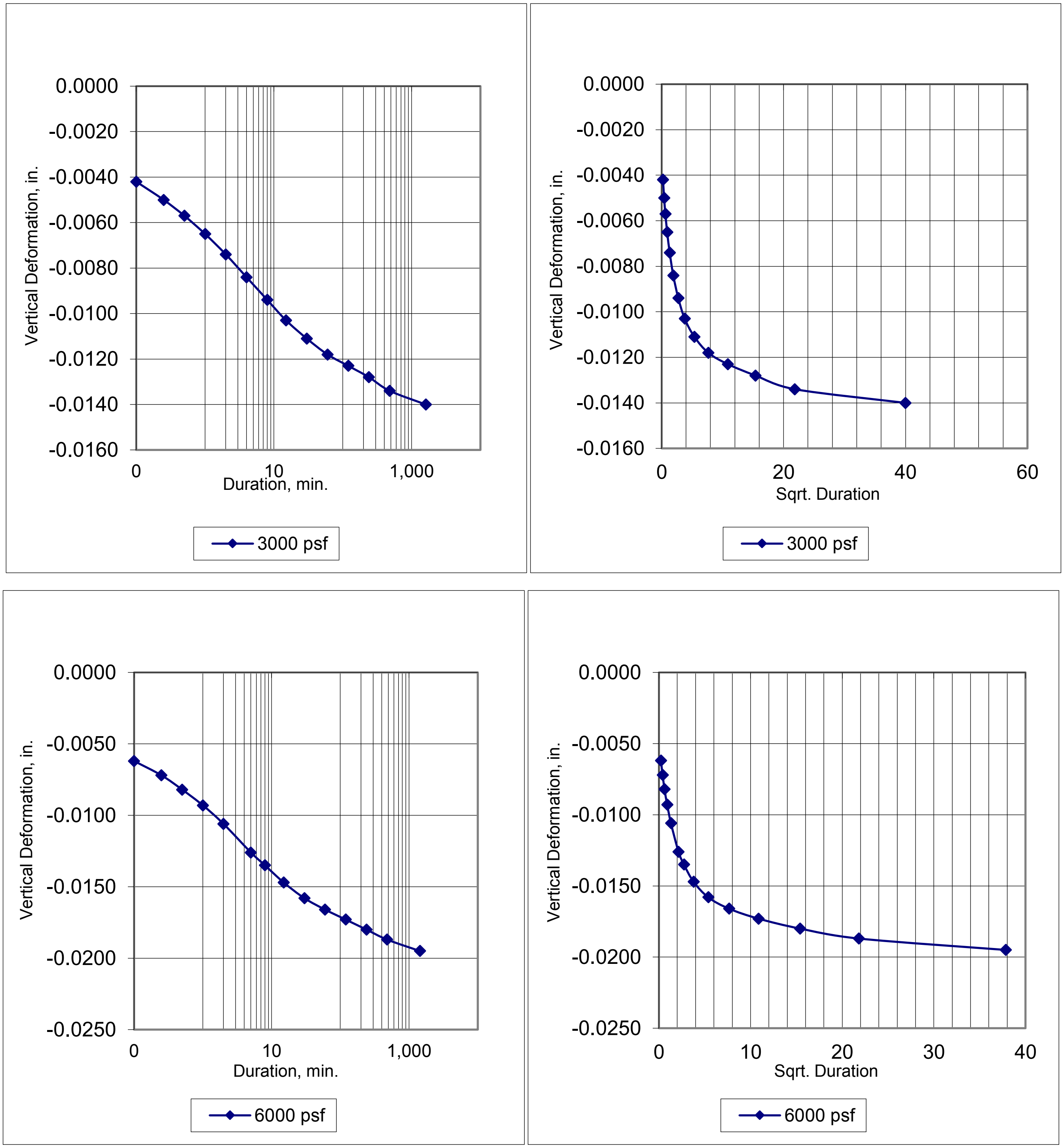
2601 Barrington Ct, Hayward, CA, 94545

Project Number:	20163829	ONE DIMENSIONAL CONSOLIDATION TEST	FIGURE 1 of 3
Date:	2/25/2016		
Entry By:	CP	EAST/WEST WING RENOVATION COURTYARD 240 CHURCH STREET SALINAS, CALIFORNIA	B-5a
Checked By:	CP		
File Name:	HL8602		



Boring:	B-1	Remarks:
Sample:	1-2-1	
Depth, ft:	6	
Test Date:	2/10/16	

KLEINFELDER
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2601 Barrington Ct, Hayward, CA, 94545

Project Number: 20163829	ONE DIMENSIONAL CONSOLIDATION TEST	FIGURE	2 of 3
Date: 2/25/2016	EAST/WEST WING RENOVATION COURTYARD 240 CHURCH STREET SALINAS, CALIFORNIA	B-5b	
Entry By: CP			
Checked By: CP			
File Name: HL8602			

APPENDIX C

Important Information about This

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 constructor — 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 this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you* — should apply this 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.

Geotechnical Engineers Base Each Report on a Unique Set of Project-Specific Factors

Geotechnical engineers consider many 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;
- the elevation, configuration, location, orientation, or weight of the proposed structure;
- the 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 geotechnical engineer performed the study. *Do not rely on a geotechnical-engineering report whose adequacy may have been affected by*: the passage of time; man-made events, such as construction on or adjacent to the site; or natural events, such as floods, droughts, earthquakes, or groundwater fluctuations. *Contact the geotechnical engineer before applying this 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 geotechnical-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 confirmation-dependent recommendations included in your report. *Confirmation-dependent 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 confirmation-dependent recommendations if that engineer does not perform the geotechnical-construction observation required to confirm the recommendations' applicability.*

A Geotechnical-Engineering Report Is Subject to Misinterpretation

Other design-team members' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront 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. Constructors can also misinterpret a geotechnical-engineering report. Confront that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing geotechnical 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 Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make constructors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give constructors the complete geotechnical-engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise constructors 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 constructors have sufficient time* to perform additional study. Only then might you be in a position to give constructors 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 constructors fail to 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.

Environmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform an *environmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical-engineering report does not usually relate any environmental 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 environmental 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, many 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 GBC-Member Geotechnical Engineer for Additional Assistance

Membership in the Geotechnical Business Council of the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project. Confer with you GBC-Member geotechnical engineer for more information.

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APPENDIX D

Exhibit 1
Summary of Compaction Recommendations

Area	Compaction Recommendation [1,2,3,4]
General Engineered Fill[5]	Compact on-site clayey fill materials to a minimum of 90 percent compaction and at least 2 percent over the optimum moisture content. On-site clayey materials should not be used in the upper 18 inches below concrete slabs-on-grade, or pavers. Compact import fill materials to a minimum of 90 percent compaction at over the optimum moisture content
Trenches [6]	Compact on-site clayey backfill materials to a minimum of 90 percent compaction and at least 2 percent over the optimum moisture content. Compact sand and import backfill materials to a minimum of 90 percent compaction at over the optimum moisture content
Exterior Flatwork [7]	Compact on-site clayey fill materials to a minimum of 90 percent compaction and at least 2 percent over the optimum moisture content. On-site clayey materials should not be used in the upper 18 inches below concrete slabs-on-grade, or pavers. Compact import fill materials to a minimum of 90 percent compaction at over the optimum moisture content

Notes:
1. All compaction requirements refer to relative compaction as a percentage of the laboratory standard described by ASTM D 1557.
2. All lifts to be compacted shall be a maximum of 8 inches loose thickness, unless otherwise recommended.
3. All compacted surfaces should be firm, stable, and unyielding under compaction equipment.
4. Where fills are deeper than 7 feet, the portion below 7 feet should be compacted to a minimum of 95 percent.
5. Includes building pads.
6. In landscaping areas, this percent compaction in trenches may be reduced to 85 percent.
7. Depths are below finished subgrade elevation.