SANDERS & ASSOCIATES GEOSTRUCTURAL ENGINEERING
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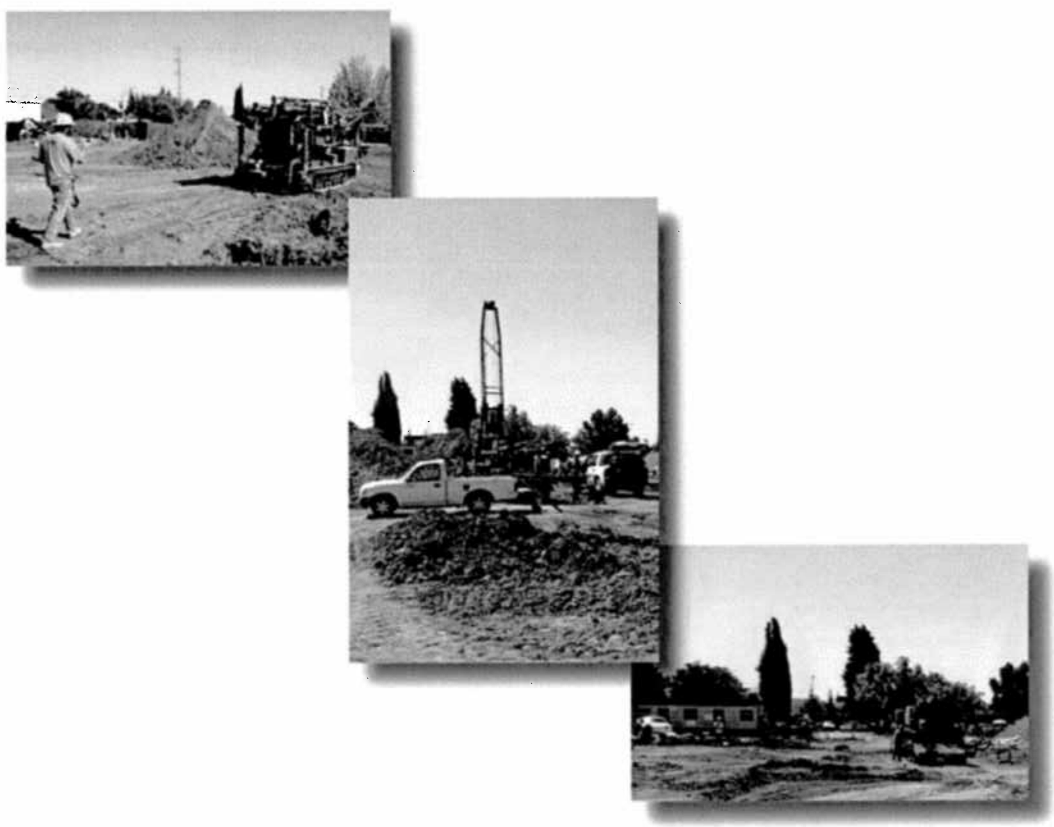
INTEGRATING EARTH & STRUCTURE

Final Geotechnical Investigation Report

Consolidated Outpatient Surgical Specialty Clinic
Veteran Affairs Medical Center
Mather, California

SAGE Project No. 10-024.00



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July 11, 2012
SAGE Project No. 10-024.00

Mr. Robert McCormick
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1325 J Street, Suite 1300
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Re: FINAL GEOTECHNICAL INVESTIGATION REPORT
Consolidated Outpatient Surgical Specialty Clinic
Veteran Affairs Medical Center
Mather, CA

Dear Mr. McCormick:

Sanders & Associates Geostructural Engineering, Inc. (SAGE) is pleased to submit this final report presenting the results of our geotechnical investigation for the proposed Consolidated Outpatient Surgical Specialty (COSS) Clinic to be constructed at the Veterans Affairs (VA) Medical Center in Mather, California. We are submitting one (1) electronic copy and four (4) hard copies of this report for your use. Our services have been performed in general accordance with the scope of services provided to HDR, Inc. (HDR) in our revised proposal dated June 12, 2012.

The project consists of an approximately 17,400 square-foot (SF), two-story building and associated site improvements which will be built in the location of a previous paved parking area. The steel frame building will have a concrete slab-on-grade floor with structural loads supported on reinforced concrete footings. Existing geotechnical information from nearby facilities was used for the preliminary basis of design for the COSS by others; however, differing soil conditions were encountered during the demolition of the parking area and initial rough grading of the building site. Our investigation was conducted to evaluate surface and subsurface conditions specific to the building site; assess the potential for adverse geologic conditions which may impact the feasibility and/or constructability of the proposed project; to obtain information to develop final geotechnical design criteria for design of the proposed building; and to provide recommendations for construction of the proposed building and associated appurtenances.

The report submitted herewith contains detailed recommendations regarding foundation design, slab-on-grade design, and site grading that should be reviewed in their entirety. These recommendations are based on limited subsurface exploration and laboratory testing as discussed herein. Consequently, variations between expected and actual soil conditions may be found during construction. A Geotechnical Engineer of Record should be retained to observe earthwork and foundation installation to assist in identifying such variations. These observations will allow them to evaluate whether the recommendations remain valid for the actual geotechnical conditions encountered during construction. SAGE can provide these services upon request.

Please call us should you have questions.

Sincerely yours,
Sanders & Associates Geostructural Engineering, Inc.

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FINAL GEOTECHNICAL INVESTIGATION REPORT
CONSOLIDATED OUTPATIENT SURGICAL SPECIALTY CLINIC
VETERAN AFFAIRS MEDICAL CENTER
MATHER, CALIFORNIA

1.0 INTRODUCTION

This report presents the results of our geotechnical investigation for the proposed Consolidated Outpatient Surgical Specialty (COSS) Clinic to be constructed at the Veterans Affairs (VA) Medical Center in Mather, California. The site is located southeast of Interstate 50 within the former Mather Air Force Base (AFB), in the central portion of Sacramento County, California (see Figure 1). The project is located northeast of the intersection of Denker Street and Hospital Way, adjacent to the southwest face of the Sacramento VA Medical Center. The site is relatively flat with site grades ranging from approximately Elevation 90.0 to 92.0 feet.[1]

The project consists of an approximately 17,400 square foot (SF), two-story building and associated site improvements which will be built in the location of a previous paved parking area (see Figure 2). The steel frame building will have a concrete slab-on-grade floor with structural loads supported on reinforced concrete footings. According to the project Structural Engineer, ZFA Structural Engineers, the maximum column load is 300 kips. Wall loads are expected to be on the order of 4.5 to 8.0 kips/lineal foot. Grading for the proposed development is expected to consist of cuts and fills of less than two (2) vertical feet to achieve the design grades.

2.0 SCOPE OF SERVICES

We performed this investigation in general accordance with the scope of services presented in our June 12, 2012 proposal to HDR. Our scope of services consisted of a review of existing geotechnical and geologic data for the site and vicinity, coordinating our exploration, performing a subsurface exploration program including four (4) soil borings, and laboratory analysis of select soil samples to evaluate site-specific subsurface conditions for the building site. Our field exploration is discussed further in Appendix A. Based on the results of our field investigation, we performed geotechnical engineering analyses to develop conclusions and recommendations regarding:

- subsurface conditions;
- regional seismicity and seismic hazards;
- bearing capacity, expected settlement, and friction factor for shallow foundations;
- concrete slabs-on-grade;
- fill quality and compaction; and
- utility installation.

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[1] Site grades estimated from Design Drawing C-101, dated April 15, 2011, prepared by HDR, Inc.

It is our understanding that Geologic Hazards and Site Specific Ground Response Reports were not required for this project, and therefore were not included in our scope of services.

3.0 GEOLOGIC SETTING

The site is located in the Great Valley geomorphic province of California, which is an alluvial plain approximately 50 miles wide and 400 miles long in the central part of California. The Great Valley is a structural depression that has been filled with a thick sequence of Mesozoic and Tertiary marine sediments covered by Quaternary alluvial sediments. Subsequent deformation has folded these older sediments into a northwest-trending asymmetrical syncline with its axis off center toward the Coast Ranges.

The Great Valley province is characterized by meandering fluvial systems, particularly along the Sacramento River, which drains the northern part of the Great Valley. Coarse-grained (sand and gravel) alluvial fan deposits are typically found along the perimeter of the valley as stream terrace deposits, as well as near the meandering Sacramento River; fine-grained (silt and clay) alluvial deposits are typically found towards the center of valley and on the floodplains of the tributary rivers and streams.

Regional geologic maps (Helley and Harwood, 1985, and Wagner et. al., 1981) depict alluvial fan deposits of the Riverbank Formation at the project site. The upper member of the Riverbank Formation is generally composed of weathered dark-brown to red alluvium composed of gravel, sand, and silt with minor clay (Helley and Harwood, 1985). Based on the above description we believe the deposits of sandy clay and clayey gravel encountered during this investigation are part of the upper member of the Riverbank Formation.

4.0 SITE AND SUBSURFACE CONDITIONS

Prior to construction of the adjacent hospital and parking area, the site was developed with single family base housing (Wallace-Kuhl Associates, 1999). Our review of historic aerial photographs (1957, 1964) indicates that two homes and a roadway previously existed within the building footprint. No evidence of these structures was observed by SAGE during this investigation. Prior to our investigation, the previous site improvements included a paved parking area with associated curbs, sidewalks, landscaping, and asphalt pavements.

We explored the subsurface soil conditions at the site by drilling four (4) test borings on June 18, 2012. All the borings were drilled within the building footprint and extended to depths ranging between 9.0 and 15.7 feet below ground surface. The subsurface soil conditions encountered in the borings generally consist of 7 to 10 feet of dry to damp, red brown, very stiff to hard, sandy clay and dense to very dense clayey sand. However, medium stiff to stiff, grey brown, organic laden sandy clay (topsoil) was encountered in Borings B1 and B2, on the southwest and northwest corners of the building, respectively. These soils extend to depths of 3 to 4 feet, have high moisture content, and appear to be soft and compressible. The surficial soils are underlain in the building location by deposits of tan, hard to very hard sandy clay and very dense clayey gravel of the Riverbank Formation.

Groundwater was not encountered during the drilling of the borings to a maximum explored depth of 15.7 feet. According to groundwater data from nearby wells compiled by the California Department of Water Resources (DWR) (http://www.water.ca.gov/waterdatalibrary/), the actual groundwater table is located at approximate Elevation +10 feet (Above Mean Sea Level), a depth of approximately 80 feet beneath the ground surface at the site.

The results of Atterberg limit tests performed on near surface (0 to 2.5 feet deep) soil samples are presented in Table 1. These tests indicate the on-site clays have a low plasticity which generally indicates a low expansion potential (Holtz and Gibbs, 1956).[2] This is consistent with our observations on the site in which we did not observe any expansive soil related distress.

Direct shear testing was performed on one near surface sample obtained from boring B2 to evaluate the strength of the underlying soil for foundation design. A bulk sample from boring B2 was also tested for water soluble sulfate corrosion potential. Additional tests such as fine and coarse sieve analyses were performed on select samples for the purpose of classification. The results of selected laboratory tests are presented on the boring logs and all of the test results are attached in Appendix B.

TABLE 1

Summary of Atterberg Limit Tests

Boring	Depth (feet)	Soil Type	LL	PI	Expansion Potential
B1	1.0 to 2.5	SANDY CLAY	21	12	Low
B2	1.0 to 2.5	SANDY CLAY	23	12	Low
B3	1.0 to 2.5	SANDY CLAY	19	9	Low

5.0 SEISMICITY

5.1 Regional Seismicity

Seismicity is defined as the geographical and historical distribution of earthquakes, or more simply, earthquake activity. The potential for ground shaking at the site is related to earthquake activity that might occur along nearby or distant faults. Based on historical earthquake activity and fault hazard mapping, the general site region is considered to have a relatively low to moderate potential for seismic activity.

Based on our review of available published geologic maps, U.S. Geological Survey (USGS) Quaternary Fault and Fold Database, and State of California Alquist-Priolo Earthquake Fault maps,

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2 Expansive soils change volume (i.e., shrink or swell) due to changes in moisture content.

there are no active[3] surface fault traces mapped in the site vicinity (Wagner et. al., 1981; Hart and Bryant, 2007; USGS, 2012). Although there are no Quaternary (movement within the last 1,600,000 years) faults mapped in the immediate site vicinity, there are several Quaternary faults mapped in the project region.

The major active fault systems that might affect the site region are the San Andreas fault system located in the Coast Range, the Great Valley thrust fault system along the western margin of the Central Valley, and the Eastern California Shear Zone along the eastern side of the Sierra Nevada. Relative to the project site, the nearest known potentially active[4] fault is the Foothills fault system located approximately 22 miles northeast of the site (Wagner et. al., 1981; USGS, 2012; Jennings et al., 2010).

On August 1, 1975, a magnitude 5.7 earthquake and associated surface ruptures occurred near Oroville (Sherburne and Hauge, 1975), focusing attention on the Foothills fault system as a potential area of active faulting. However, the general absence of Quaternary age deposits in the Sierra Nevada foothills has made it difficult to assess the recency of fault activity along the fault system. Where investigated, fault displacement rates appear to be low during the past 100,000 years (Schwartz et al., 1996).

The maximum moment magnitude[5] earthquake estimated for the Foothills fault system is M_w 6.5, with a recurrence interval of about 12,500 years (CDMG, 1996). The Foothills fault system is not currently zoned as active under the State of California Alquist-Priolo Earthquake Fault Zoning Act (Hart and Bryant, 2007), except for the Cleveland Hill fault which experienced ground rupture during the 1975 Oroville earthquake (Bryant and Hart, 2007; CDMG, 1977). The Cleveland Hill fault is located approximately 60 miles north of the site.

5.2 Seismic Hazards

An earthquake on a segment of one of the regional faults could result in low to moderate ground shaking at the site. We evaluated the anticipated level of shaking to determine if seismic hazards, such as liquefaction or ground fault rupture, could impact the project site. Our evaluation of the potential seismic hazards at the site is presented in the following subsections.

5.2.1 Ground Shaking

We expect the site will experience low to moderate ground shaking. The intensity of ground shaking at the site depends on many factors, including the size of the fault generating an earthquake event, the distance from the fault rupture to the project site, and the duration of strong ground shaking.

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[3] Active faults are defined as those exhibiting either surface ruptures, topographic features created by faulting, surface displacements of Holocene (younger than about 11,000 years old) deposits, tectonic creep along fault lines, and/or close proximity to linear concentrations or trends of earthquake epicenters.

[4] Potentially active faults displace geologic deposits of Pleistocene age (about 2 million to 11,000 years old).

[5] Moment magnitude (M_w) is directly related to average slip and rupture fault area, while the Richter magnitude scale reflects the amplitude of a particular type of seismic wave.

Based on review of the USGS Probabilistic Hazards Curves (2002) and design parameters for use with the 2010 California Building Code (CBC), the estimated peak ground acceleration (PGA) at the site is about 0.18 g for Site Class D (deep soil deposits), which corresponds to a low to moderate level of shaking. Design parameters for use with the 2010 CBC are presented later in this report.

5.2.2 Soil Liquefaction and Associated Hazards

Our investigation was limited to evaluating the subsurface conditions and engineering characteristics of the soils within the upper 15 feet of the site. Detailed investigations to evaluate liquefaction potential typically require investigating the subsurface soils to depths of 40 to 50 feet below the ground surface. However, because the groundwater is estimated to be about 80 feet below existing site grades, exploration to deeper depths was not required.

Soil liquefaction is the sudden and rapid reduction in the shear strength of a soil due to an increase in excess pore pressure caused by cyclic loading under undrained loading conditions, most commonly, strong ground shaking. In the case of complete soil liquefaction, physical properties of the soil become similar to a heavy fluid rather than a soil, and a nearly complete loss of shear strength can occur. Soils most prone to liquefaction are clean, fine-grained, uniformly graded sands. However, sand with varying amounts of silt and clay, non-plastic silts, some fine gravel, and sensitive clays may also liquefy and/or lose strength during strong cyclic loading. Phenomena associated with liquefaction include sand boils, flow failure, lateral spreading, differential settlement, loss of bearing strength, and ground fissures.

Because liquefaction occurs due to the buildup of pore-water pressure within the soil skeleton, potentially liquefiable soils are generally below the groundwater table. Groundwater was not encountered during this investigation to the maximum depth explored of 15.7 feet, and is stimated to be approximately 80 feet below the ground surface.

The site is underlain at a relatively shallow depth by Riverbank Formation materials which consist of a very dense/very hard mixture of sand, gravel, silt and clay. The soils above the Riverbank Formation consist of firm to hard sandy clay. Based on the depth of groundwater and the density of the subsurface soils, it is our opinion the potential for liquefaction at the site is nil.

5.2.3 Seismically Induced Densification

Seismically induced densification of non-saturated sand (sand above the groundwater table) due to earthquake vibrations may also cause settlement. However, the soil deposits encountered at the site have either sufficient density and/or cohesion such that the risk of seismically induced densification is negligible.

5.2.4 Fault Rupture

Historically, ground surface displacements closely follow the trace of geologically young faults. No known active or potentially active faults appear to exist on the site. We therefore conclude the risk of fault offset at the site from a known active fault is low.

6.0 DISCUSSION AND RECOMMENDATIONS

From a geotechnical standpoint, the proposed construction is feasible as planned provided the recommendations presented in the remainder of this report are incorporated into the proposed construction.

As previously discussed, the site was most recently occupied by a parking lot with planters and prior to the parking lot, military housing and associated improvements. The site is an active construction site so the previous parking lot, planters, and vegetation have generally been removed, leaving bare earth. The extent of the prior military structure removal is unknown; however, we did not observe any visible signs of the structures or their associated foundations. The foundations from the prior structures were likely shallow and may have even been supported at-grade. We did observe an area adjacent to an existing medical building to the east where roots from former trees and bushes are exposed at the ground surface, and will require removal.

Based on the grey brown, compressible sandy clay observed in borings B1 and B2, and the former improvements that existed on the site, we recommend that mitigative grading be performed to ensure the new structure is supported on a relatively uniform building pad. Mitigative grading recommendations are provided in Section 6.1.2 below.

The near surface soils at the site comprise clays that are characterized as having a low plasticity index. Generally accepted correlations of plasticity to expansion potential indicate these soils have a non-expansive to very low expansion potential. Based on the information obtained during our investigation, and provided the building pad is graded as recommended herein, it is our opinion that the proposed COSS Clinic building may be satisfactorily supported on continuous and isolated spread footings in conjunction with a slab-on-grade. Specific design recommendations are provided in Section 6.2 below.

6.1 Site Grading

6.1.1 General Requirements

Grading at the site is generally expected to consist of minor cuts and fills of approximately two vertical feet or less to achieve the design grades. We expect grading can be accomplished with conventional construction equipment.

The materials removed from site excavations, including utility trench excavations, are expected to consist predominantly of low plasticity red brown clayey soils or blended materials comprising the grey brown and red brown clays. To facilitate compaction, on-site soil will require moisture conditioning prior to its reuse as general, on-site engineered fill. On-site material used as engineered fill should be free of organics, trash, and other debris and should not contain oversize particles larger than three inches in greatest dimension. On-site soils including the near surface clayey soils may be used as engineered fill beneath the building and foundations provided they are prepared as recommended in Section 6.1.3. If imported fill is required, it should be similar to the native onsite soils and have a maximum Liquid Limit of 35, a maximum Plasticity Index of 12, and contain no materials larger than 3 inches.

All fill material, including on-site fill, should be submitted to the geotechnical engineer of record for approval at least 72 hours before it is to be used on site. Where imported fill is required, the fill supplier should provide analytical test results or other suitable environmental documentation at least three days before use at the site indicating the proposed fill material is free of hazardous materials, such as heavy metals or petroleum hydrocarbons.

6.1.2 Overexcavation, Fill Placement, and Compaction – Building Pad

Based on the soil conditions observed and the former improvements that existed on the site, we recommend the building pad be overexcavated to a minimum depth of 12 inches below the existing grade. The limits of the overexcavation should extend laterally at least 5 feet beyond the building footprint. The bottom of the overexcavation should be scarified to a depth of at least 8 inches, moisture conditioned to an above optimum moisture content[6], and compacted to at least 90 percent relative compaction[7]. Engineered fill may then be placed in maximum 8-inch-thick loose lifts, moisture conditioned, and compacted as noted above. The soil should not be allowed to dry out between the placement of lifts. The contractor should be prepared to keep all soil surfaces moist until they have been covered by improvements. If the soil is allowed to dry out, it should be scarified eight inches, moisture conditioned, and recompacted.

Additional overexcavation and recompaction is recommended where localized areas of unsuitable soils were encountered in the building pad. These areas are where grey brown, organic-laden, medium stiff to stiff sandy clay is present on the northwest and southwest portions of the building pad and where root systems from former trees/bushes were observed on the eastern side of the building footprint. Soft and compressible (grey brown) materials must be removed from the building pad to expose the underlying competent red brown sands and clays, and the base of the deepened overexcavation should be scarified to a depth of at least 8 inches. If any other areas of loose/soft soil are observed, they must also be removed. The base of the overexcavation, as well as the recompacted fill, should be placed and compacted to at least 95 percent relative compaction in these localized areas. Prior to compaction, the soils should be moisture conditioned to above optimum moisture content; drying of the on-site, overexcavated material may be required.

The Geotechnical Engineer of Record should be on-site to observe all areas to receive fill at the time of grading to check conformance with the recommendations presented in this report.

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[6] Moisture content refers to the amount of water within the soil expressed as a percentage of the dry weight of soil as determined by the ASTM D2216 laboratory procedure.

[7] Relative compaction refers to the in-place dry density of soil, determined in accordance with ASTM D6938, expressed as a percentage of the maximum dry density of the same material, as determined by the ASTM D1557 laboratory compaction procedure.

6.1.3 Fill Placement and Compaction – Other Areas

At areas outside the building pad where improvements are proposed, such as at walkways or patios, the existing ground surface should be scarified, moisture conditioned, and compacted to at least 90 percent as noted in Section 6.1.2. If fill is required, it should be similarly processed.

Where soft native soil is encountered in non-building pad areas to be graded, it should be overexcavated to expose firm soil, up to a maximum depth of three feet. If firm soil is not encountered, select fill or aggregate base backfill over a geotextile fabric (Mirafi 500X or equivalent) may be required to bridge over the soft soil. The overexcavation should be backfilled with engineered fill placed to a minimum relative compaction of 90 percent.

6.1.4 Utility Trenches

Backfill for utility trenches is also considered fill, and it should be compacted to at least 90% relative compaction. Jetting of trench backfill is not allowed. Special care should be taken when backfilling utility trenches in pavement areas. Poor compaction may cause excessive settlements, resulting in damage to the pavement section.

To provide uniform support, pipes or conduits should be bedded on a minimum of four inches of sand or fine gravel. After the pipes and conduits are tested, inspected (if required) and approved, they should be covered to a depth of six inches with sand or fine gravel, which should be mechanically tamped. Backfill should be placed in lifts of eight inches or less, moisture-conditioned to achieve near-optimum moisture content, and compacted to at least 90 percent relative compaction. Clean sand (defined as sand material with less than 10 percent passing the No. 200 sieve) should be moisture conditioned to near optimum moisture content and compacted to at least 95 percent relative compaction.

In order to reduce the potential for utility trenches to act as a groundwater conduit, a sand-cement slurry or concrete plug should be constructed where utilities pass beneath the building perimeter or other improvements. The trench plugs minimize the migration of water from adjacent landscaping from passing beneath the improvement through the bedding sand or gravel.

6.1.5 Surface Drainage

Drainage control design should include provisions for positive surface gradients so that surface runoff is not permitted to pond, particularly adjacent to building foundations, roadways, pavements, or slabs. Surface runoff should be directed away from foundations and collected in lined ditches or drainage swales. The water collected should be directed to a storm drain or paved roadway. Discharge from the roof gutter and downspout systems should be included in the collection system and not allowed to infiltrate the subsurface near the structures or in the vicinity of slopes. The finished pad grade around the building should be compacted and sloped away from the exterior foundations and as required in Section 1804.3 of the 2010 CBC.

6.2 Foundation Support

Provided the building pad is graded as recommended above, it is our opinion that the proposed building may be supported on a spread footing foundation system. To reduce the potential for moisture migration and fluctuations beneath the proposed structure, we recommend a continuous perimeter footing be constructed. We recommend the perimeter and interior footings be bottomed at least 18 inches below the lowest adjacent soil subgrade. This depth should be measured from finished grade at the exterior of the building or the bottom of the capillary moisture break, whichever is lower.

Footings for the proposed buildings should be at least 12 inches wide. We recommend the foundations be designed for an allowable dead plus live load bearing pressure of 3,000 pounds per square foot (psf). For total loads, including wind or seismic forces, the allowable bearing pressure may be increased by 1/3. These values include factors of safety of at least 2.0 and 1.5 for dead plus live and total loads, respectively. Under these bearing pressures, we expect total settlement to be on the order of ¾ inch or less. We estimate total differential settlement due to moisture variations and foundation settlement will be less than ¼ inch.

Lateral loads can be resisted by a combination of passive pressure acting on the vertical face of the footings and friction on the base of the footings. We recommend passive pressure on the face of the footing be computed using an equivalent fluid weight (triangular distribution) of 200 pounds per cubic foot (pcf). The upper foot of soil should be neglected unless the ground surface is covered with slabs or pavement. Frictional resistance should be computed using a value of 0.40. The values presented for passive and frictional resistance can be used in combination and include factors of safety of at least 1.5 to reduce the potential for lateral movement.

The footing excavations should be free of standing water, debris, and disturbed materials prior to placing concrete. The Geotechnical Engineer of Record should check the foundation excavations after cleaning but prior to placement of reinforcing steel to confirm the excavations are bottomed in suitable bearing material and have been cleaned properly. If loose soil is encountered at the bottom of a footing excavation, it should be removed and replaced as described above. The bottoms and sides of footings should be maintained in a moist condition until concrete is placed.

6.3 Moisture Vapor Retarder

To reduce water vapor transmission through the floor slab, we recommend a capillary moisture break and a water vapor retarder be installed beneath the floor. A capillary moisture break consists of at least four inches of clean, free-draining gravel or crushed rock. The vapor retarder should meet the requirements for Class A vapor retarders stated in ASTM E1745. We recommend using a polyolefin, Class A vapor retarding membrane such as Stegowrap 10 mil (or equivalent). All seams in the vapor retarder should be overlapped by at least six inches, taped, and sealed in accordance with ASTM E1643 and the manufacturer's specifications. All penetrations should be similarly sealed. The vapor retarder should be covered with two inches of sand to aid in curing the concrete and to protect the vapor retarder during slab construction. The particle size of the gravel/crushed rock and sand should meet the gradation requirements presented in Table 2.

TABLE 2

Gradation Requirements for Capillary Moisture Break

Sieve Size	Percentage Passing Sieve
Gravel or Crushed Rock	
1 inch	90 – 100
3/4 inch	30 – 100
1/2 inch	5 – 25
3/8 inch	0 – 6
No. 200	0 – 2
Sand	
No. 4	100
No. 200	0 – 5

The sand overlying the membrane should be moist at the time concrete is placed; however, there should be no free water present in the sand. Excess water trapped in the sand could eventually be transmitted as vapor through the slab. If rain is forecast prior to pouring the slab, the sand should be covered with plastic sheeting to avoid wetting. If the sand becomes wet, concrete should not be placed until the sand has been dried or replaced. In addition, the contractor should take care to prevent "sand waves" from forming during concrete placement operations so that a consistent slab thickness is maintained.

Concrete mixes with high water/cement (w/c) ratios result in excess water in the concrete, which increases the cure time and results in excessive vapor transmission through the slab. Therefore, concrete for the floor slab should have a low w/c ratio - less than 0.50. If approved by the project structural engineer, the sand can be eliminated and the concrete can be placed directly over the vapor retarder, provided the w/c ratio of the concrete does not exceed 0.45 and water is not added in the field. If necessary, workability should be increased by adding plasticizers. In addition, the slab should be properly cured. Before the floor covering is placed, the contractor should check that the concrete surface and moisture emission levels (if emission testing is required) meet the manufacturer's requirements.

6.4 Seismic Design

For design in accordance with the 2010 CBC, the site may be classified as Site Class D, and the following design parameters should be used:

- $S_S = 0.462$
- $F_a = 1.430$
- $S_1 = 0.214$
- $F_v = 1.972$
- $S_{MS} = 0.661$
- $S_{M1} = 0.422$
- $S_{DS} = 0.441$
- $S_{D1} = 0.281$

6.5 Temporary Excavations

Where excavation is performed for foundation construction or utility installation, we anticipate temporary slopes will be used. All temporary slopes should be excavated in accordance with the latest edition of the CAL-OSHA excavation and trench safety standards as a minimum (OSHA, 2012). Site soils should be preliminarily classified as Type A according to the CAL-OSHA classification system. The maximum allowable slope for Type A soil is 0.75H:1V. If granular soils or seepage is observed in the cut face, the soil should be classified as Type C and a maximum slope of 1.5H:1V should be used. Where vertical sidewalls are used at the base of excavations in cohesive soils, the maximum height of the vertical walls should be limited to four feet.

The contractor should be responsible for all temporary slopes and shoring systems used at the site, and should designate one of their on-site employees as a "competent person" who is responsible for trench and excavation safety. The competent person should be responsible for determination of the actual OSHA soil type and should direct the excavation crews to adjust slopes inclinations if appropriate.

6.6 Corrosivity

The corrosion potential of on-site soils to concrete was evaluated in the laboratory using a representative sample obtained from Boring B2. Laboratory testing was performed to assess the effects of sulfate content on concrete and the results are presented in Appendix B. Based on a review of the referenced California Building Code (CBC, 2010) and American Concrete Institute ACI 318-08 Table 4.2.1, the tested soil is considered to have an Exposure Class of S0. In accordance with ACI 318 Table 4.3.1, there is no restriction as to the type of cement used.

7.0 LIMITATIONS

This report has been prepared for the sole use of HDR and the Department of Veterans Affairs, and their agents specifically for the design of the proposed Consolidated Outpatient Surgical Specialty (COSS) Clinic. The opinions, conclusions and recommendations contained in this report are based upon the information obtained from our site reconnaissance and exploration, our engineering studies, experience, and engineering judgment, and have been formulated in accordance with generally accepted geotechnical engineering practices that exist at the time this report was prepared.

No other warranty, expressed or implied, is made or should be inferred. In addition, the recommendations presented in this report are based on the subsurface conditions encountered in widely spaced test borings. Actual conditions may vary. If subsurface conditions encountered in the field differ from those described in this report, we should be consulted to determine if changes to our conclusions or supplemental recommendations are required.

The opinions presented in this report are valid as of the date of this report for the property being evaluated. Changes in the condition of a property can occur with the passage of time, whether due to natural processes or the works of man. If site conditions vary from those described herein, we should be consulted to evaluate the impact of the changes, if any. In addition, changes in applicable standard of practice can occur, whether from legislation or the broadening of knowledge. Accordingly, the opinions presented in this report may be invalidated, wholly or partially, by changes outside of SAGE's control. In any case, this report should not be relied upon after a period of three years without prior review and approval by SAGE.

8.0 REFERENCES

2010 California Building Code, California Code of Regulations, Title 24, Volume 2 of Part 2

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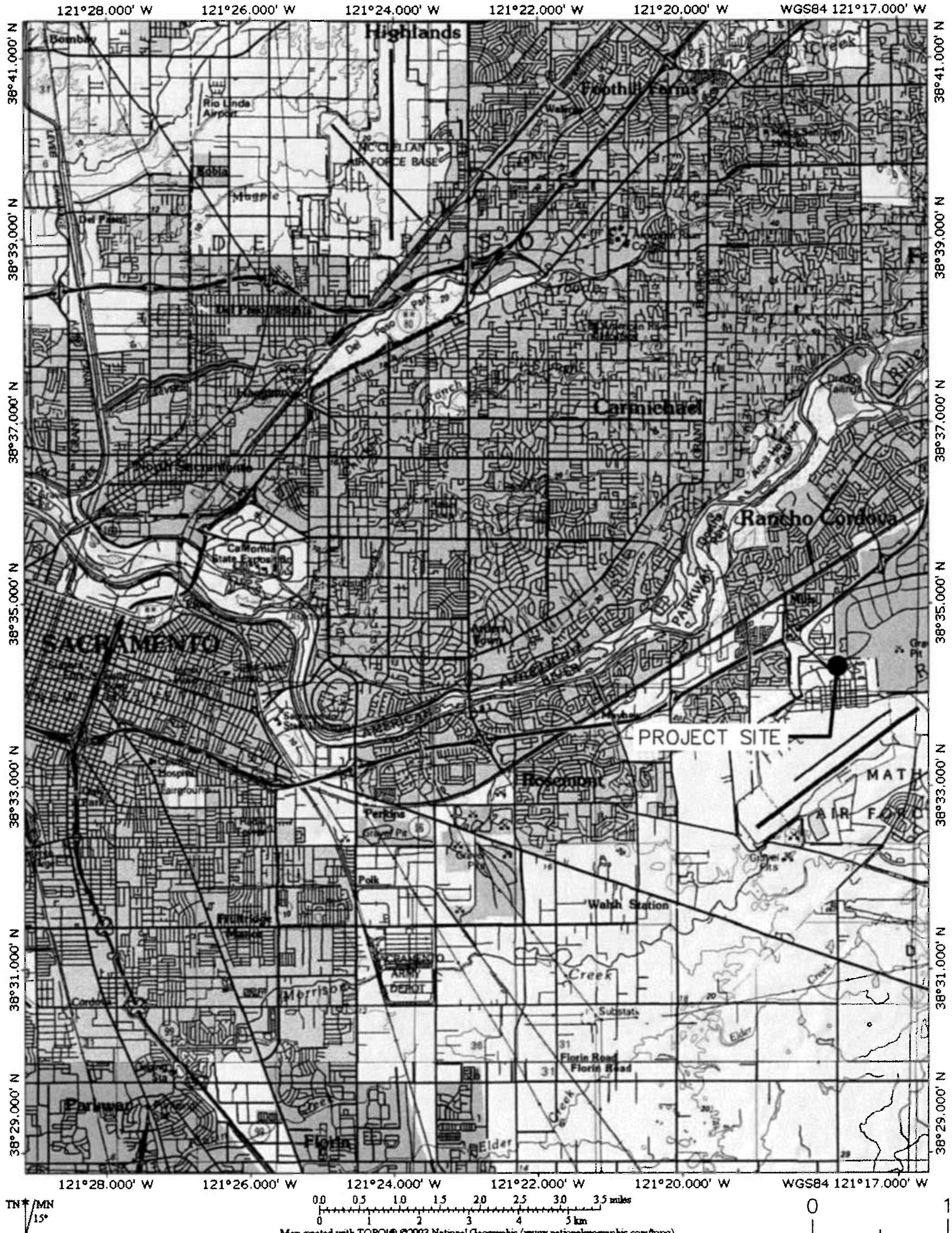
FIGURES

TOPO! map printed on 06/27/12 from "Untitled.tpo"



0 1"

REFERENCE SCALE

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SITE LOCATION MAP

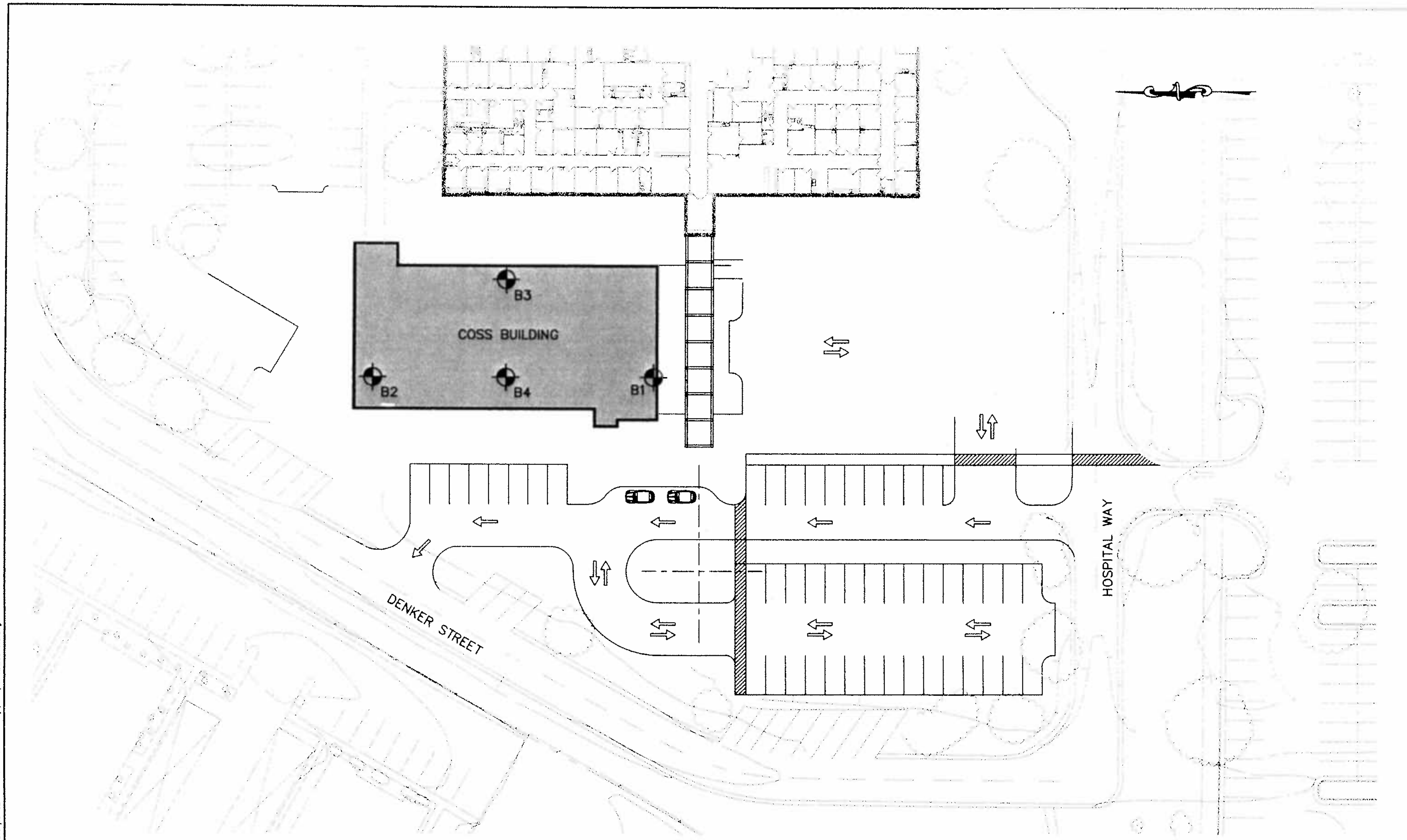
VA COSS CLINIC

MATHER | SACRAMENTO COUNTY | CALIFORNIA

FIGURE 1

1 of 2

DATE: 07/11/12 | FILE: 10-024.00-FIG | SCALE: AS NOTED | JOB NO. 10-024.00



0 1"
REFERENCE SCALE

BASE MAP REFERENCE:

SITE PLAN PHASE 1, VA MATHER COSS, BY HDR, DATED 12/16/10

LEGEND

APPROXIMATE BORING LOCATION BY SAGE, JUNE 18, 2012

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SITE PLAN VA COSS CLINIC			FIGURE 2
MATHER	SACRAMENTO COUNTY	CALIFORNIA	2 of 2
DATE: 07/11/12	FILE: 10-024.00-FIG	SCALE: 1" = 40'	JOB NO. 10-024.00

PLOT DATE: Wednesday, July 11, 2012 TIME: 4:02:41 PM BY: JERRY PASCOE
FILE: N:\1-PROJECTS\2010\10-024 VA-Mather COSS Facility\CAD\10-024.00-FIG.dwg

APPENDIX A
Test Borings and Field Exploration Program

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A.1 Field Exploration Program

Our field exploration program consisted of drilling four (4), 4-inch-diameter geotechnical test borings to obtain data necessary to evaluate subsurface conditions for the proposed project. The approximate boring locations, designated B1 through B4, are presented on Figure 2. The borings were drilled from the rough graded building pad elevation of approximately 92 feet above mean sea level.

Prior to drilling, the site limits were marked and Underground Services Alert was notified to locate and mark underground utilities in the project limits. In addition, Cruz Brothers Locators (sub-contractor to SAGE) performed a secondary underground utility location survey for non-public utilities prior to the drilling.

Borings B1 through B4 were drilled on June 18, 2012. All the borings were drilled by Taber Drilling using a track-mounted CME55 drill rig equipped with four-inch-diameter solid flight augers. The borings were drilled to depths ranging from 9.0 and 15.7 below the existing ground surface. During drilling, a Professional Geologist logged the materials encountered and obtained representative samples for visual classification and laboratory testing. The materials encountered were classified in general accordance with the Unified Soil Classification System (USCS) as summarized on Figure A-1. Logs for borings B1 through B4 are presented as Figures A-2 through A-5.

Representative soil samples for this investigation were recovered using the following sampler types:

- Modified California (MCA) split-barrel sampler with a 3.0-inch-outside diameter fitted with 2.43-inch-inside-diameter, six-inch-long brass liners; and
- Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch-outside diameter, without liners.

Due to the stiffness/density of the materials encountered, Shelby Tube thin-walled samplers were not used. Both split-barrel samplers were driven with a 140-pound, automatic safety hammer falling 30 inches. The blow counts required to drive the samplers over a standard 18-inch-drive were recorded in six-inch increments in the field. Where refusal was encountered (defined as greater than 50 blows over any six-inch increment) drive lengths less than 12 inches were also recorded. The final 12-inches of the drive (less in the case of refusal) were added to develop the reported blow count. The blow counts for the MCA sampler were corrected for the effects of sampler size and converted to SPT values using a conversion factor of 0.65. Finally all blow counts were corrected to SPT N_{60} values using a conversion factor of 1.45 (based on a measured auto-hammer efficiency of 87% provided by Taber). The final, corrected values for each drive are presented on the boring logs and represent N_{60} values.

Upon completion of drilling, the holes were backfilled with soil cuttings to the ground surface.

UNIFIED SOIL CLASSIFICATION SYSTEM

Major Divisions		Symbols	Typical Names
Coarse-Grained Soils (more than half of soil > No. 200 sieve size)	Gravels (More than half of coarse fraction > No. 4 sieve size)	GW	Well-graded gravels or gravel-sand mixtures, little or no fines
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines
		GM	Silty gravels, gravel-sand-silt mixtures
		GC	Clayey gravels, gravel-sand-clay mixtures
	Sands (More than half of coarse fraction > No. 4 sieve size)	SW	Well-graded sands or gravelly sands, little or no fines
		SP	Poorly-graded sands or gravelly sands, little or no fines
		SM	Silty sands, sand-silt mixtures
		SC	Clayey sands, sand-clay mixtures
Fine-Grained Soils (more than half of soil < No. 200 sieve size)	Silts and Clays LL = < 50	ML	Inorganic silts and clayey silts of low plasticity, sandy silts, gravelly silts
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, lean clays
		OL	Organic silts and organic silt-clays of low plasticity
	Silts and Clays LL = > 50	MH	Inorganic silts of high plasticity
		CH	Inorganic clays of high plasticity, fat clays
		OH	Organic silts and clays of high plasticity
Highly Organic Soils		PT	Peat and other highly organic soils

GRAIN SIZE CHART

Classification	Range of Grain Sizes: U.S. Standard Sieve Size	Range of Grain Sizes: Grain Size in Millimeters
Boulders	Above 12"	Above 305
Cobbles	12" to 3"	305 to 76.2
Gravel	3" to No.4	76.2 to 4.76
coarse	3" to 3/4"	76.2 to 19.1
fine	3/4" to No. 4	19.1 to 4.76
Sand	No. 4 to No. 200	4.76 to 0.074
coarse	No. 4 to No. 10	4.76 to 2.00
medium	No. 10 to No. 40	2.00 to 0.420
fine	No. 40 to No. 200	0.420 to 0.074
Silt and Clay	Below No. 200	Below 0.074

TYPES OF STRENGTH TESTS

PP	Pocket Penetrometer
TV	Field Torvane
LVS	Laboratory Vane Shear
UC	Unconfined Compression
TXUU	Triaxial, unconsolidated, undrained
DS	Direct Shear

▽ Unstablilized (initial) groundwater level

▼ Stabilized groundwater level

SAMPLER TYPE

C — Core barrel

O — Osterberg piston sampler using 3.0-inch outside diameter, thin-walled Shelby tube

PT — Pitcher tube sampler using 3.0-inch outside diameter, thin-walled Shelby tube

ST — Shelby tube (3.0-inch outside diameter, thin-walled tube) advanced with hydraulic pressure

CC — CME Continuous Sample Tube System sampler advanced with hollow stem auger

BULK — Disturbed grab sample

CA — California split-barrel sampler with 2.5-inch outside diameter and 1.93-inch inside diameter

MCA — Modified California split-barrel sampler with 3.0-inch outside diameter and 2.5-inch inside diameter

SPT — Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch outside diameter and a 1.5-inch inside diameter

Sampling attempted without recovery

NOTE:
Shaded portion of sampler symbol represents portion of sample recovered

Examples:

MCA SPT CC } Portion recovered

VA COSS CLINIC

MATHER SACRAMENTO COUNTY CALIFORNIA

SOIL CLASSIFICATION CHART

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Project No. 10-024.00

Date 07/11/12

Figure A-1

PROJECT:	VA COSS Clinic Mather, California	**LOG OF BORING B1**	Sheet 1 of 1
BORING LOCATION: See Figure 2		**DRILLING SUBCONTRACTOR:** Taber Drilling, Inc.	
DATE STARTED: 6/18/2012	**DATE FINISHED:** 6/18/2012	**DRILL RIG:** Track-mounted CME-55	
LOGGED BY: C. Smith		**DRILLING METHOD:** 4-inch solid-flight auger	
ELEVATION (FT): 92.0	**DATUM:** Design Drawing C-101		
GW DEPTH (FT): N/A	**GW DATE:** N/A	**HAMMER TYPE:** Automatic	
CASING NOTES: N/A		**HAMMER WT (LBS):** 140	**HAMMER DROP (IN):** 30
BACKFILL MATERIAL: Soil Cuttings		**SAMPLERS:** MCA, SPT	

DEPTH (FT)	ELEV. (FT)	SAMPLE TYPE	SPT N60 VALUE	LITHOLOGY	DESCRIPTION	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	FINES (%)	TYPE of TEST	UNCONFINED STRENGTH (tsf)	SHEAR STRENGTH (ksf)	PLASTICITY LL	PLASTICITY PI
1	91.0			CL	SANDY CLAY (CL) medium brown to dark red brown, grey brown, medium stiff to stiff, moist, contains rootlets, fine sand and pebbles; some organic odor. Sieve: See Appendix B								
2	90.0	SPT	10					62.0				21	12
					becomes very stiff; pebbles continued; maganese oxide in areas (dark grey).								
3	89.0	MCA	18		TOP SOIL (0 to 4.5 ft)								
4	88.0												
5	87.0			SC/CL	CLAYEY SAND TO SANDY CLAY (SC/CL) tan brown, very dense to very hard, dry to damp, contains gravel and abundant calcium carbonate coatings, gravel well rounded up to 1" in size.								
6	86.0	SPT	67										
7	85.0												
8	84.0	MCA	85										
9	83.0												
10	82.0			GC	CLAYEY GRAVEL WITH SAND (GC) tan brown, dense to very dense, dry, driller added water to promote easier drilling. Matrix ranging between clayey sand (SC) and sandy clay (CL) with variable gravel. Cementation increasing. Sieve: See Appendix B.			25.0					
11	81.0	SPT	58										
12	80.0												
13	79.0												
14	78.0												
15	77.0	MCA	47/5"		dark gray to gray matrix, refusal at 15.7'								
16	76.0												
17	75.0												
18	74.0												
19	73.0												

Boring terminated at a depth of 15.7 feet below existing ground surface.
Blow counts for the MCA sampler were converted to SPT values using a conversion factor of 0.65. Blow counts were then corrected to SPT N60 values using a conversion factor of 1.45 (auto-hammer efficiency of 87% provided by Taber).

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Project No: 10-024.00
Figure: A-2

PROJECT: VA COSS Clinic, Mather, California	LOG OF BORING B2 — Sheet 1 of 1
BORING LOCATION: See Figure 2	DRILLING SUBCONTRACTOR: Taber Drilling, Inc.
DATE STARTED: 6/18/2012 — DATE FINISHED: 6/18/2012	DRILL RIG: Track-mounted CME-55
LOGGED BY: C. Smith	DRILLING METHOD: 4-inch solid-flight auger
ELEVATION (FT): 92.0 — DATUM: Design Drawing C-101	
GW DEPTH (FT): N/A — GW DATE: N/A	HAMMER TYPE: Automatic
CASING NOTES: N/A	HAMMER WT (LBS): 140 — HAMMER DROP (IN): 30
BACKFILL MATERIAL: Soil Cuttings	SAMPLERS: MCA, SPT

DEPTH (FT)	ELEV. (FT)	SAMPLE TYPE	SPT N60 VALUE	LITHOLOGY	DESCRIPTION	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	FINES (%)	TYPE of TEST	UNCONFINED STRENGTH (tsf)	SHEAR STRENGTH (ksf)	PLASTICITY LL	PLASTICITY PI
0–4	92.0–88.0			CL	SANDY CLAY (CL) dark brown, olive grey brown, and red brown, stiff to very stiff (increasing with depth), moist, contains rootlets, fine sand, and some organic odor, contains some well rounded gravel up to 1.25". TOP SOIL								
2	90.0	SPT	12		Sieve: See Appendix B			66.0				23	12
3	89.0	MCA	14		Direct Shear: See Appendix B	18.5	109						
4–9.5	88.0–82.5			SC/CL	CLAYEY SAND TO SANDY CLAY (SC/CL) tan brown, very dense to very hard, dry to damp, contains gravel up to 1.5" in size. Matrix varies from clayey sand (SC) to sandy clay (CL). Some clay coatings and clayey zones on and between clasts. Gravel well rounded.								
6	86.0	SPT	81					63.0					
7	85.0				driller added water to boring								
8	84.0	MCA	57										
9.5–13	82.5–79.0			GC	CLAYEY GRAVEL WITH SAND (GC) tan brown, very dense, dry								
11	81.0	SPT	118										

Boring terminated at a depth of 13.0 feet below existing ground surface.
Blow counts for the MCA sampler were converted to SPT values using a conversion factor of 0.65. Blow counts were then corrected to SPT N60 values using a conversion factor of 1.45 (auto-hammer efficiency of 87% provided by Taber).

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Project No: 10-024.00

Figure: A-3

PROJECT: VA COSS Clinic, Mather, California	LOG OF BORING B3 — Sheet 1 of 1
BORING LOCATION: See Figure 2	**DRILLING SUBCONTRACTOR:** Taber Drilling, Inc.
DATE STARTED: 6/18/2012 **DATE FINISHED:** 6/18/2012	**DRILL RIG:** Track-mounted CME-55
LOGGED BY: C. Smith	**DRILLING METHOD:** 4-inch solid-flight auger
ELEVATION (FT): 92.0 **DATUM:** Design Drawing C-101	
GW DEPTH (FT): N/A **GW DATE:** N/A	**HAMMER TYPE:** Automatic
CASING NOTES: N/A	**HAMMER WT (LBS):** 140 **HAMMER DROP (IN):** 30
BACKFILL MATERIAL: Soil Cuttings	**SAMPLERS:** MCA, SPT

DEPTH (FT)	ELEV. (FT)	SAMPLE TYPE	SPT N60 VALUE	LITHOLOGY	DESCRIPTION	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	FINES (%)	TYPE of TEST	UNCONFINED STRENGTH (tsf)	SHEAR STRENGTH (ksf)	LL	PI
0–3	92.0–89.0			CL	SANDY CLAY (CL) red brown, hard, damp, minor fill at surface (~0.2'), contains glass fragments.								
1	91.0												
2	90.0	SPT	52		Sieve: See Appendix B			61.0				19	9
3	89.0	MCA	44	SC/CL	CLAYEY SAND TO SANDY CLAY (SC/CL) red brown, dense to very dense, hard to very hard, damp to dry, contains some interspersed pebbles/gravel								
4	88.0												
5	87.0				increasing coarsness with depth								
6	86.0	SPT	86										
7	85.0			GC	CLAYEY GRAVEL WITH SAND (GC) tan, very dense, dry, gravel to 1.5" in size, rounded, cemented. water added to boring to ease drilling								
8	84.0	MCA	47/5"										
9	83.0												
10	82.0	SPT	47/5"										
11	81.0				gravel wedged in shoe								
12	80.0												
13	79.0												
14	78.0												
15	77.0												
16	76.0												
17	75.0												
18	74.0												
19	73.0												

Boring terminated at a depth of 12.1 feet below existing ground surface.
Blow counts for the MCA sampler were converted to SPT values using a conversion factor of 0.65. Blow counts were then corrected to SPT N60 values using a conversion factor of 1.45 (auto-hammer efficiency of 87% provided by Taber).

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Figure: A-4

LOG OF BORING CORE LOGS.GPJ SAGE.GDT 7/2/12

PROJECT: VA COSS Clinic, Mather, California	LOG OF BORING B4 — Sheet 1 of 1
BORING LOCATION: See Figure 2	DRILLING SUBCONTRACTOR: Taber Drilling, Inc.
DATE STARTED: 6/18/2012 — DATE FINISHED: 6/18/2012	DRILL RIG: Track-mounted CME-55
LOGGED BY: C. Smith	DRILLING METHOD: 4-inch solid-flight auger
ELEVATION (FT): 92.0 — DATUM: Design Drawing C-101	
GW DEPTH (FT): N/A — GW DATE: N/A	HAMMER TYPE: Automatic
CASING NOTES: N/A	HAMMER WT (LBS): 140 — HAMMER DROP (IN): 30
BACKFILL MATERIAL: Soil Cuttings	SAMPLERS: MCA, SPT

DEPTH (FT)	ELEV. (FT)	SAMPLE TYPE	SAMPLE	SPT N60 VALUE	LITHOLOGY	DESCRIPTION	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	FINES (%)	TYPE of TEST	UNCONFINED STRENGTH (tsf)	SHEAR STRENGTH (ksf)	PLASTICITY LL	PLASTICITY PI
0–4.5					CL	SANDY CLAY (CL) red brown, hard, damp to moist, contains some gravel and pebbles. Some zones with fine sand interspersed. Abundant rootlets and olive brown areas in upper 4.5'.								
1	91.0													
2	90.0	SPT		30										
3	89.0	MCA		22										
4	88.0													
4.5–9.0					SC/CL	CLAYEY SAND TO SANDY CLAY (SC/CL) reddish tan brown, medium dense to very dense, very stiff to very hard, damp to dry, contains gravel up to 1.5" in size. Some clay pockets around and in between weathered clasts. Matrix contains areas of increased sand content.								
5	87.0													
6	86.0	SPT		26										
7	85.0													
8	84.0	MCA		64										
9	83.0													
10	82.0													
11	81.0													
12	80.0													
13	79.0													
14	78.0													
15	77.0													
16	76.0													
17	75.0													
18	74.0													
19	73.0													

Boring terminated at a depth of 9.0 feet below existing ground surface.
Blow counts for the MCA sampler were converted to SPT values using a conversion factor of 0.65. Blow counts were then corrected to SPT N60 values using a conversion factor of 1.45 (auto-hammer efficiency of 87% provided by Taber).

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Project No: 10-024.00
Figure: A-5

LOG OF BORING CORE LOGS.GPJ SAGE.GDT 7/2/12

APPENDIX B
Laboratory Test Results

B.1 Laboratory Testing

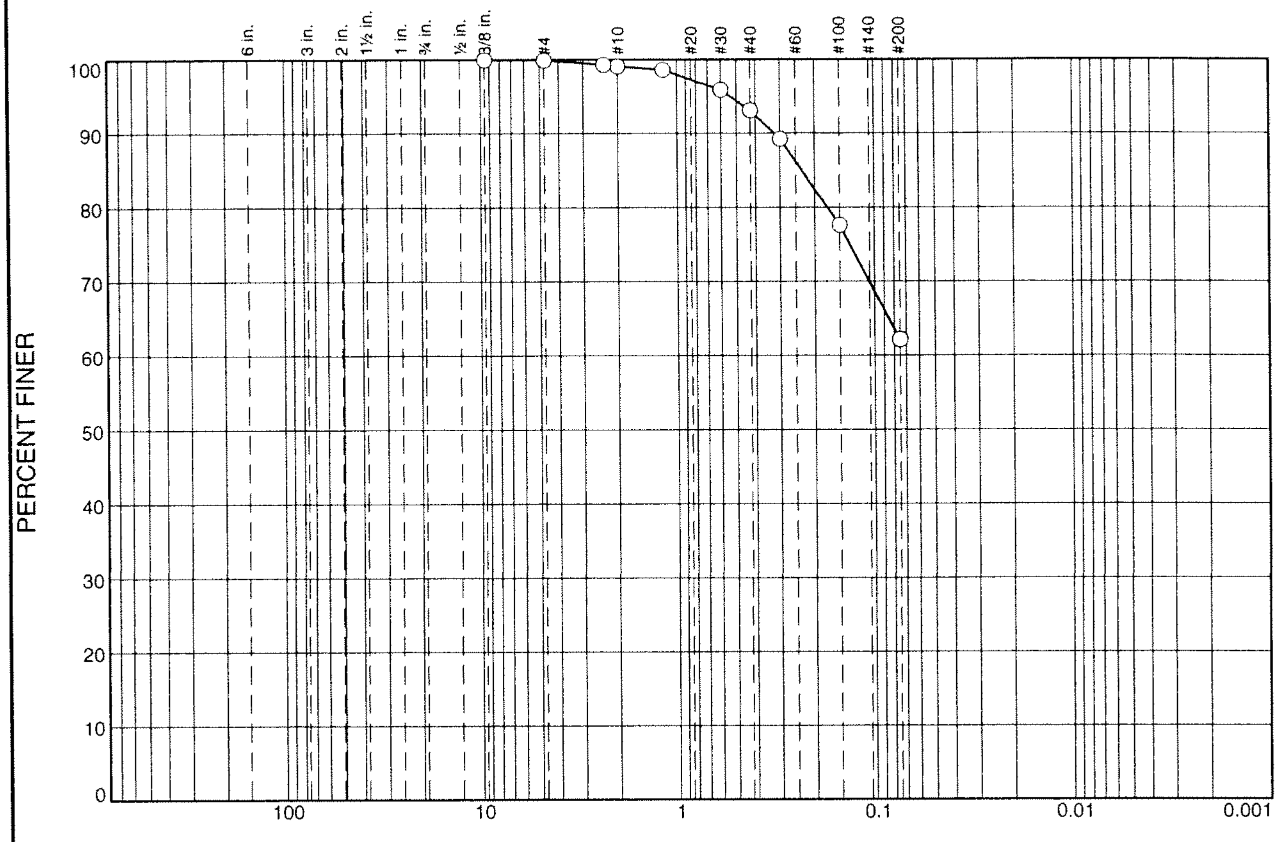
Representative soil samples obtained from the borings were reviewed in our office to confirm field classifications and selected samples were submitted for laboratory testing. Geotechnical testing was performed by Sierra Testing Laboratories of El Dorado Hills, California, which has been certified by the USACE Materials Testing Center for the testing performed. Laboratory testing was performed to determine the following properties:

- Atterberg Limits (Plasticity Index) per ASTM D4318;
- Particle-Size Distribution (Gradation) of Soils Using Sieve Analysis per ASTM D6913;
- Percent Passing the No. 200 sieve (Fines Content) per ASTM D1140;
- Particle-Size Analysis of Soils per ASTM D422; and
- Consolidated-Drained Direct Shear per ASTM D3080.

Additionally, a representative near surface sample (upper 4 feet) was tested for corrosion potential (water soluble sulfates) by Sunland Analytical of Rancho Cordova, California.

The laboratory reporting sheets for the laboratory testing follow.

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	0	0	1	6	31	62	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8 Inch	100		
#4	100		
#8	99		
#10	99		
#16	99		
#30	96		
#40	93		
#50	89		
#100	78		
#200	62		

* (no specification provided)

Soil Description

Atterberg Limits

PL= 9 LL= 21 PI= 12

Coefficients

D_{90}= 0.3210 D_{85}= 0.2330 D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CL AASHTO= A-6(4)

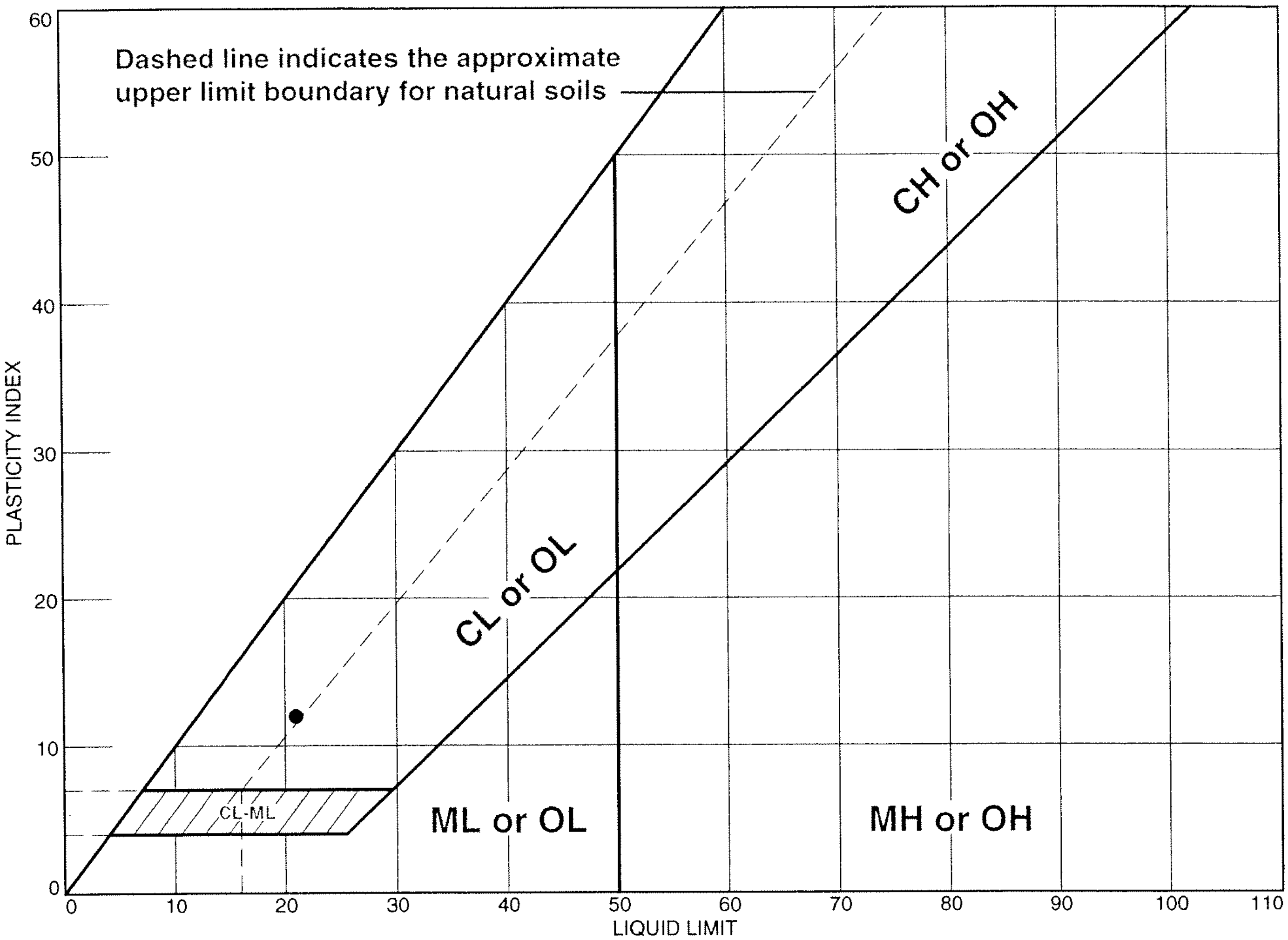
Remarks

Location: B1, STP-1
Sample Number: S37943 **Depth:** 1-2.5 **Date:** 6/20/12

SIERRA TESTING LABS, INC. El Dorado Hills, CA	**Client:** SAGE **Project:** Mather VA Medical Center COSS Building 10-024.00, Task Order 1 **Project No:** 12-081 **Figure**

Tested By: jm **Checked By:** mn

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●		21	9	12	93	62	CL

Project No. 12-081 **Client:** SAGE

Project: Mather VA Medical Center COSS Building
10-024.00, Task Order 1

● **Location:** B1, STP-1 **Depth:** 1-2.5 **Sample Number:** S37943

Remarks:

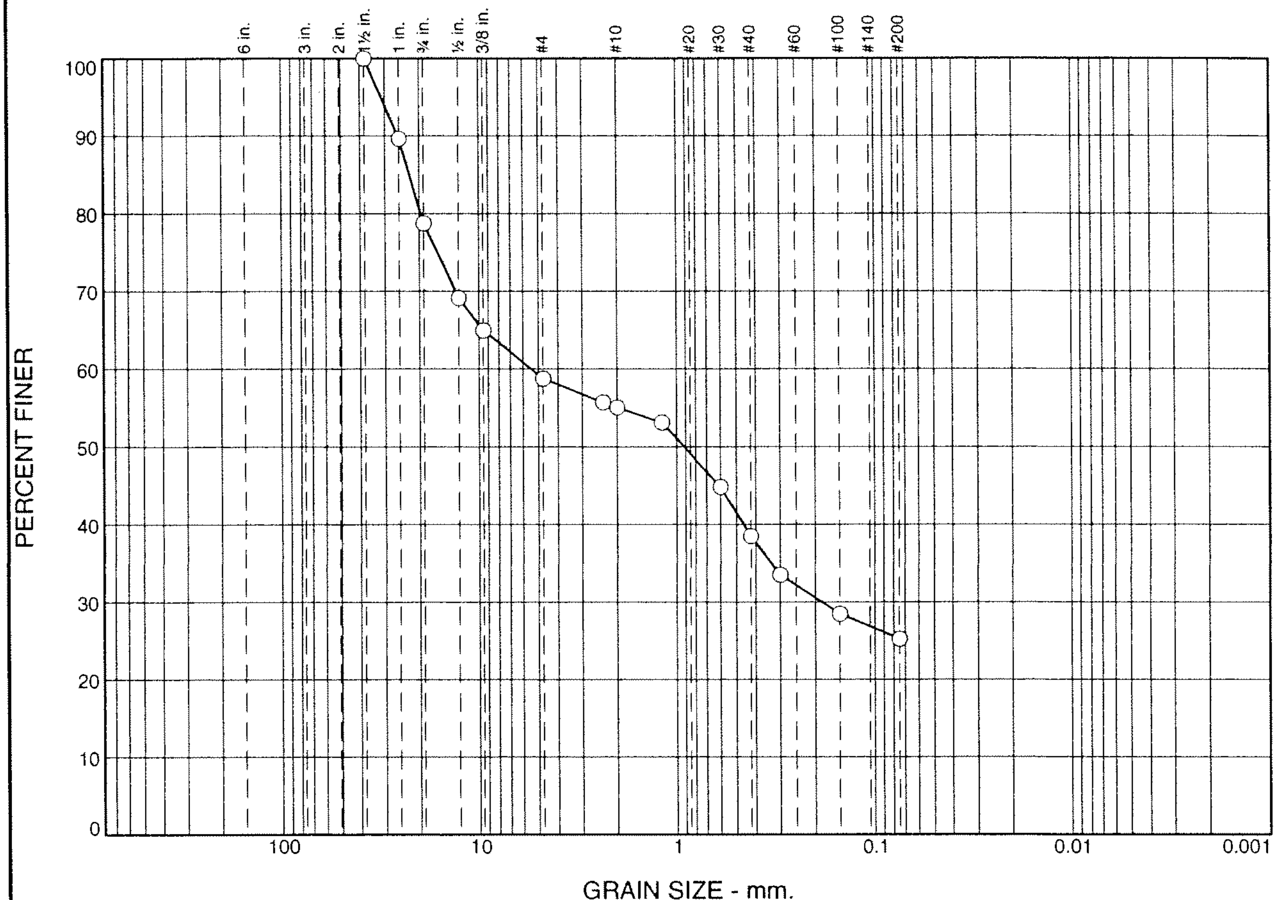
SIERRA TESTING LABS, INC.

El Dorado Hills, CA

Figure

Tested By: jm **Checked By:** mn

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	21	20	4	16	14	25	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 1/2 Inch	100		
1 Inch	90		
3/4 Inch	79		
1/2 Inch	69		
3/8 Inch	65		
#4	59		
#8	56		
#10	55		
#16	53		
#30	45		
#40	39		
#50	34		
#100	28		
#200	25		

* (no specification provided)

Soil Description

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 25.7721 D_{85}= 22.4777 D_{60}= 5.4555
D_{50}= 0.9148 D_{30}= 0.1853 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

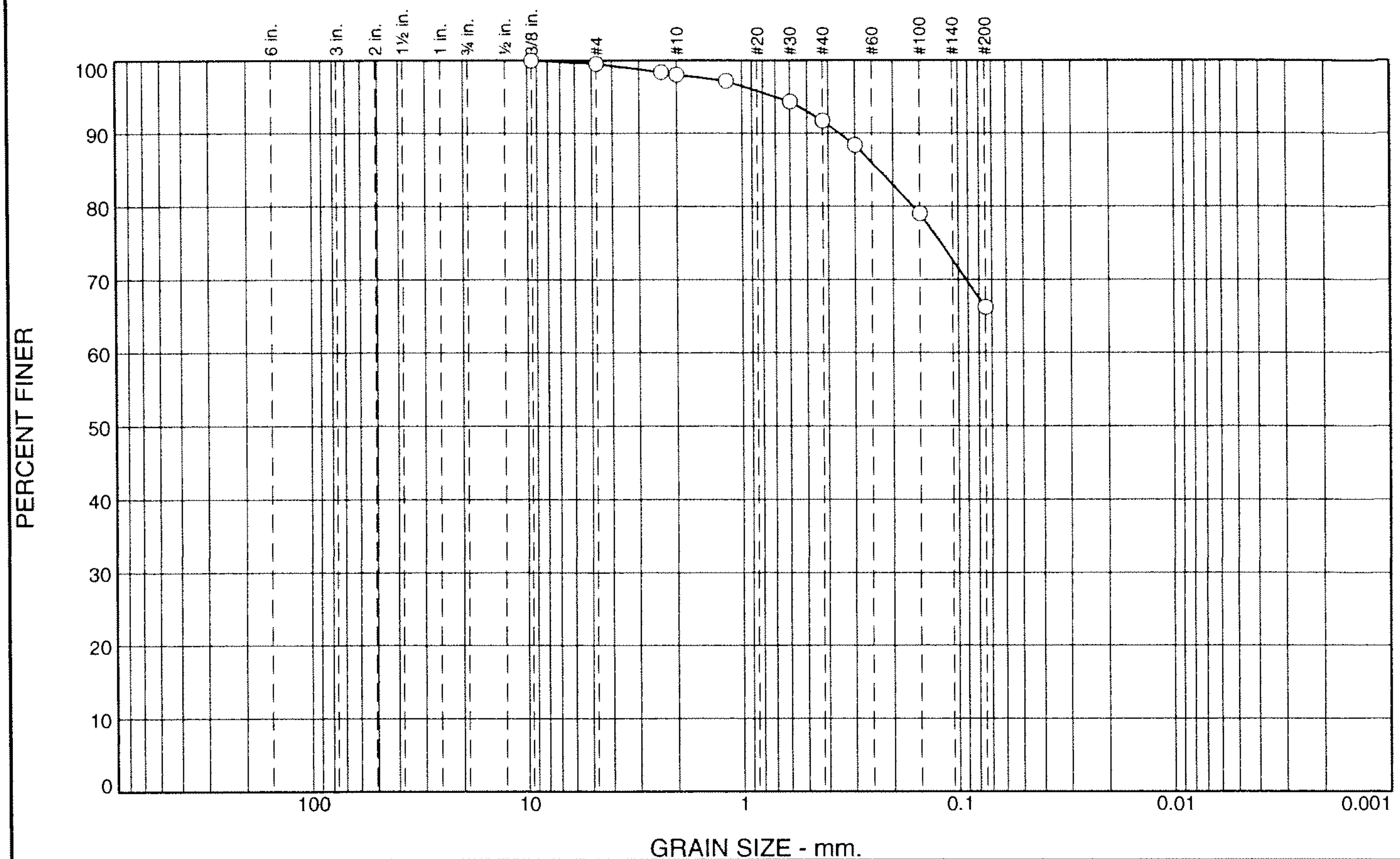
Remarks

Location: B1, STP-3
Sample Number: S37944 **Depth:** 10-11.5 **Date:**

SIERRA TESTING LABS, INC.
El Dorado Hills, CA

Client: SAGE
Project: Mather VA Medical Center COSS Building 10-024.00, Task Order 1
Project No: 12-081 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	0	0	2	6	26	66	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8 Inch	100		
#4	100		
#8	98		
#10	98		
#16	97		
#30	94		
#40	92		
#50	88		
#100	79		
#200	66		

* (no specification provided)

Material Description

Atterberg Limits

PL= 11 LL= 23 PI= 12

Coefficients

D_{90}= 0.3547 D_{85}= 0.2334 D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CL AASHTO= A-6(5)

Remarks

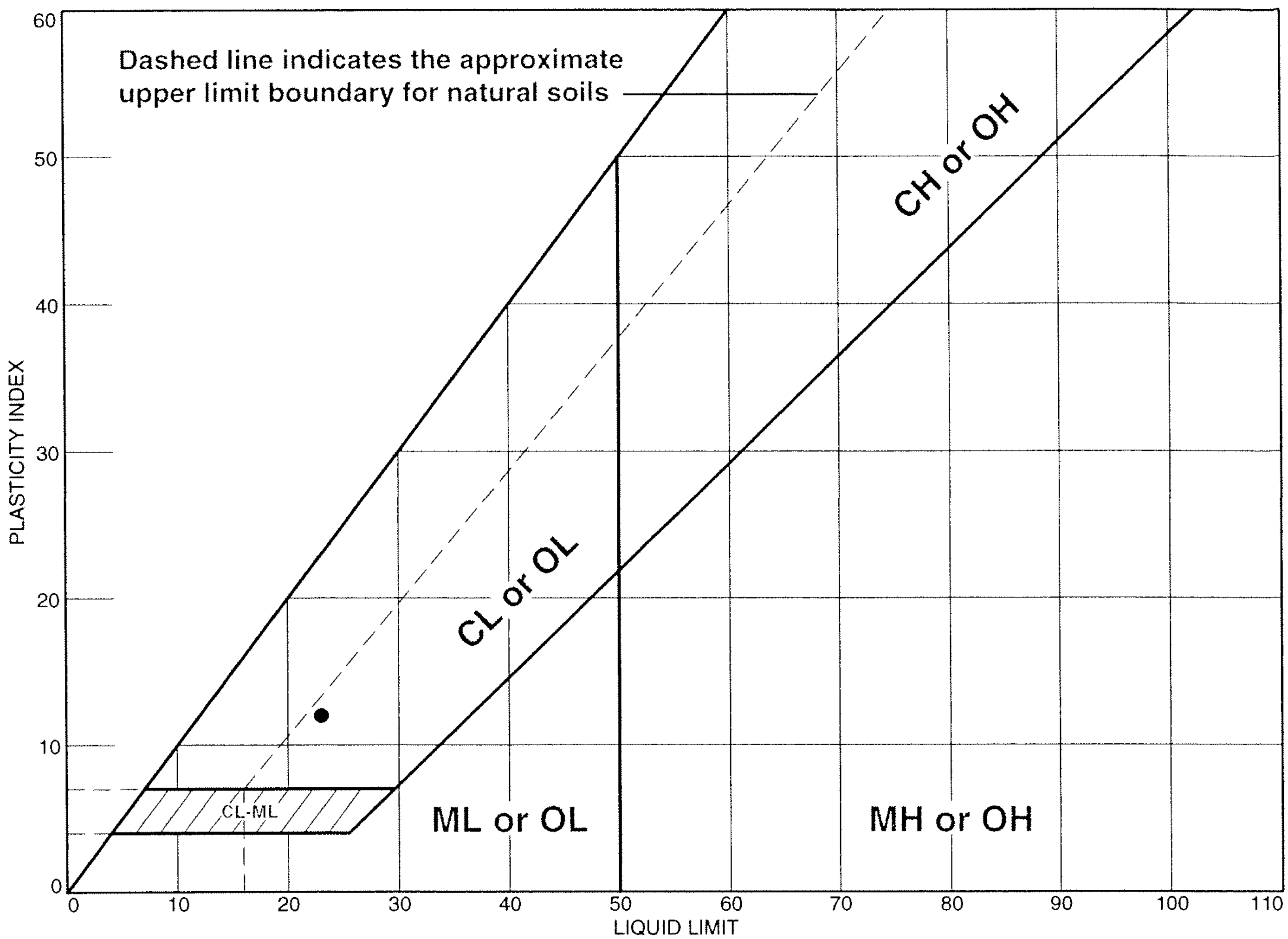
Location: B2, SPT-1
Sample Number: S37945 **Depth:** 1-2.5 **Date:** 6/20/12

SIERRA TESTING LABS, INC.
El Dorado Hills, CA

Client: SAGE
Project: Mather VA Medical Center COSS Building
10-024.00, Task Order 1
Project No: 12-081 **Figure**

Tested By: jm **Checked By:** mn

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●		23	11	12	92	66	CL

Project No. 12-081 **Client:** SAGE

Project: Mather VA Medical Center COSS Building
10-024.00, Task Order 1

● **Location:** B2, SPT-1 **Depth:** 1-2.5 **Sample Number:** S37945

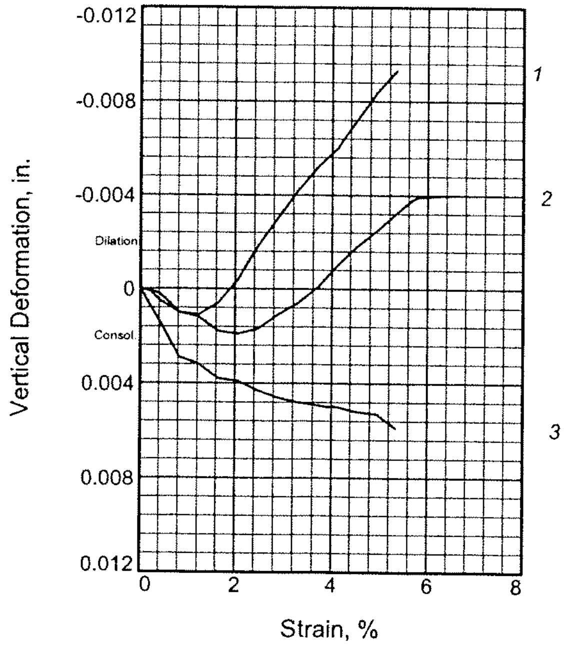
Remarks:

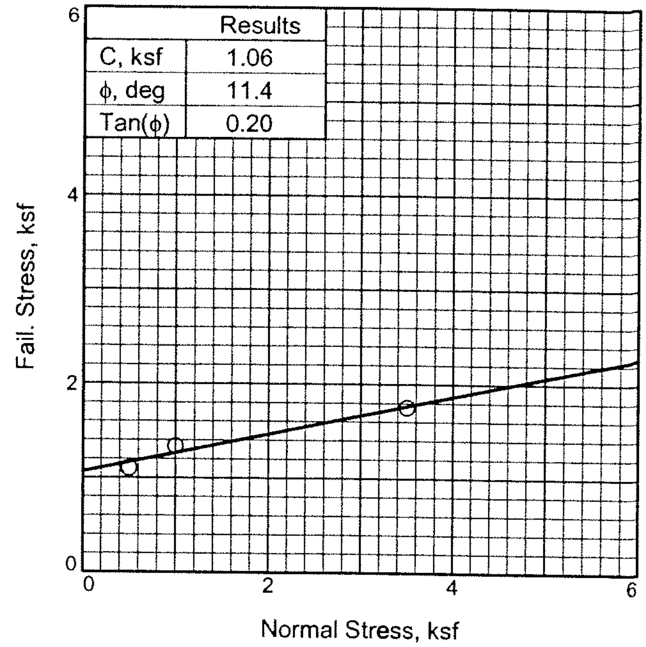
SIERRA TESTING LABS, INC.

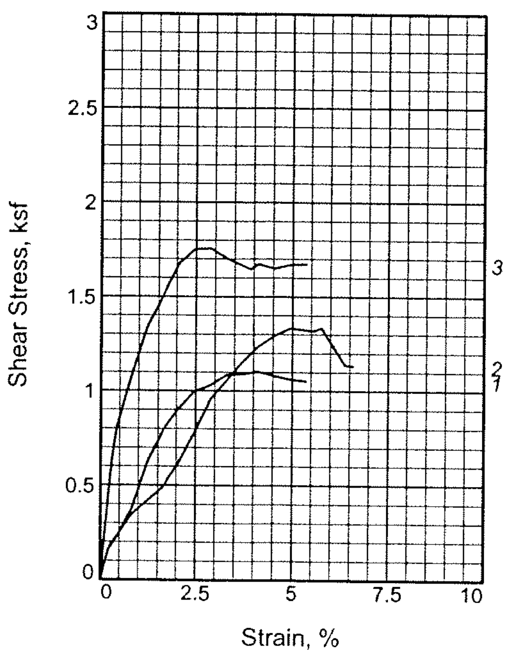
El Dorado Hills, CA

Figure

Tested By: jm **Checked By:** mn







	Sample No.	1	2	3
Initial	Water Content, %	20.9	18.0	16.6
	Dry Density, pcf	106.7	109.1	110.3
	Saturation, %	98.4	90.4	86.1
	Void Ratio	0.5679	0.5337	0.5166
	Diameter, in.	2.43	2.43	2.43
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	20.8	19.1	16.5
	Dry Density, pcf	107.3	110.7	116.0
	Saturation, %	99.7	100.0	99.8
	Void Ratio	0.5585	0.5107	0.4421
	Diameter, in.	2.43	2.43	2.43
	Height, in.	0.99	0.99	0.95
	Normal Stress, ksf	0.50	1.00	3.50
	Fail. Stress, ksf	1.10	1.34	1.75
	Strain, %	4.1	5.8	2.9
	Ult. Stress, ksf			
	Strain, %			
	Strain rate, in./min.	0.03	0.03	0.03

Sample Type: Undisturbed
Description:

Assumed Specific Gravity= 2.68
Remarks:

Figure ______

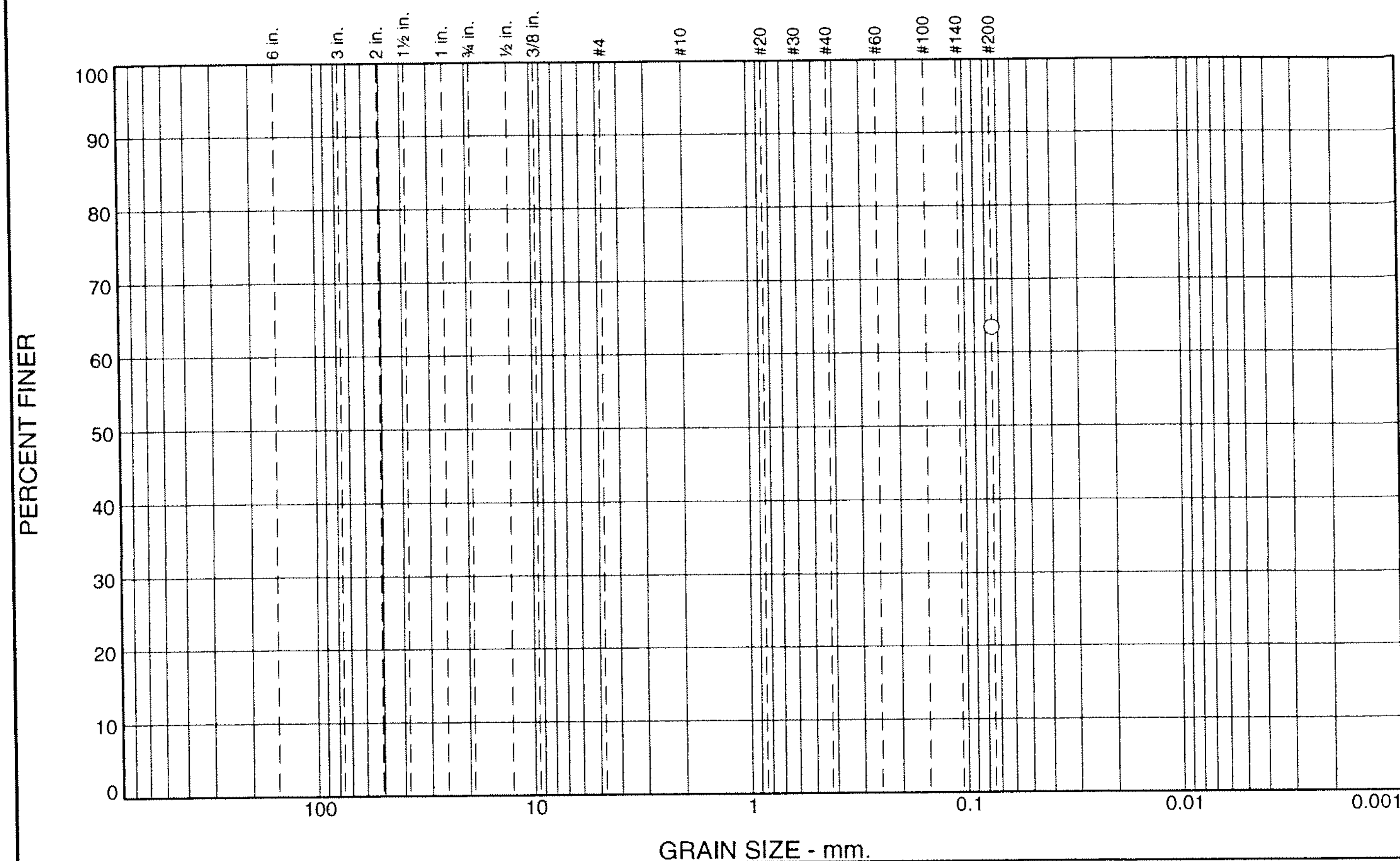
Client: SAGE

Project: Mather VA Medical Center COSS Building
10-024.00, Task Order 1
Location: B2, MC-1
Sample Number: S37946 **Depth:** 3.5-4.0
Proj. No.: 12-081 **Date Sampled:** 6-20-2012 (Teste

DIRECT SHEAR TEST REPORT
SIERRA TESTING LABS, INC.
El Dorado Hills, CA

Tested By: mpw **Checked By:** mn

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						63	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	63		

* (no specification provided)

Soil Description

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

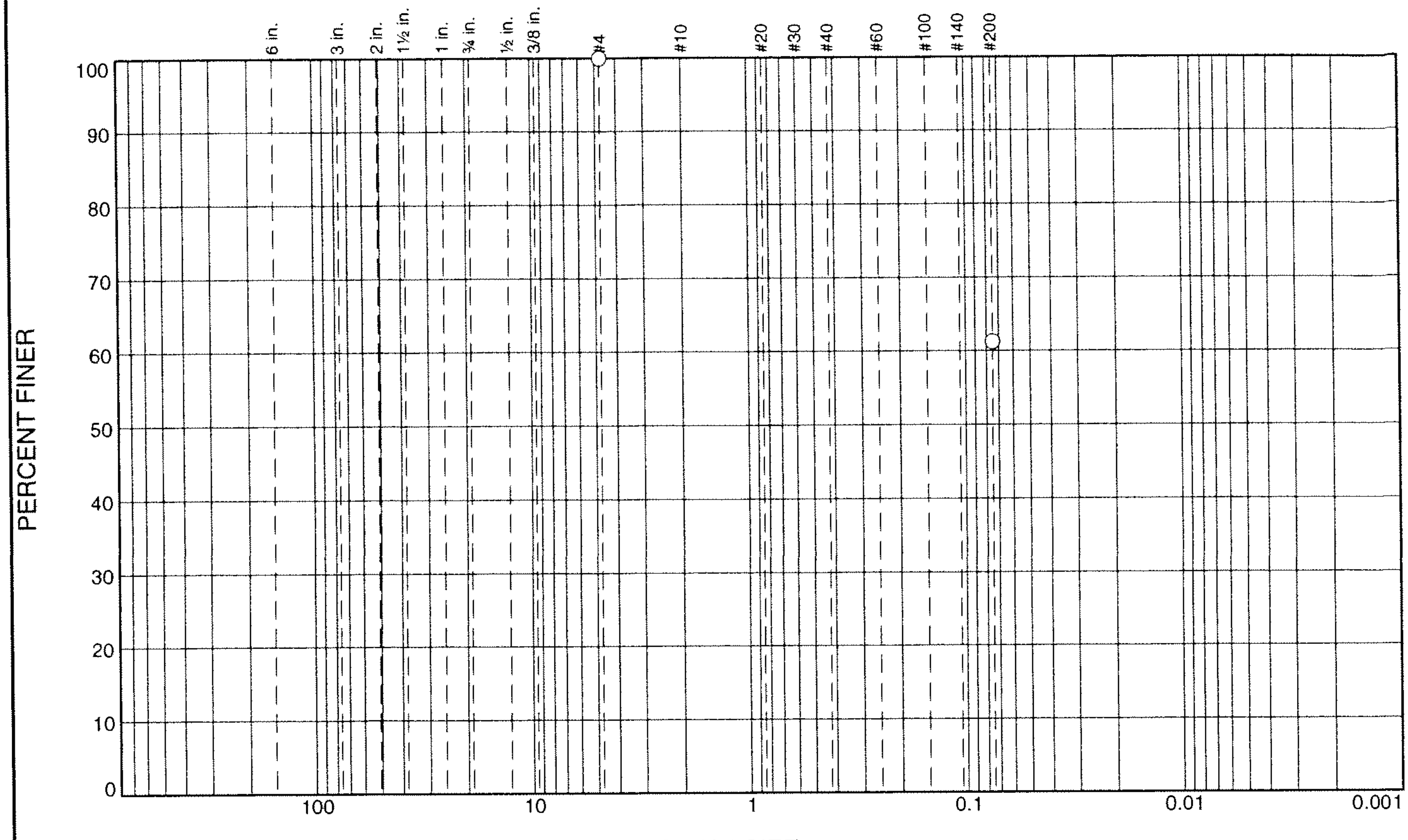
Location: B2, SPT-2
Sample Number: S37947 **Depth:** 5-6.5 **Date:** 6/20/12

SIERRA TESTING LABS, INC.
El Dorado Hills, CA

Client: SAGE
Project: Mather VA Medical Center COSS Building
10-024.00, Task Order 1
Project No: 12-081 **Figure**

Tested By: mpw **Checked By:** mn

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	0	0	8	15	16	61	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100		
#200	61		

* (no specification provided)

Material Description

sandy lean clay

Atterberg Limits

PL= 10 LL= 19 PI= 9

Coefficients

D_{90}= 1.6275 D_{85}= 0.9527 D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CL AASHTO= A-4(2)

Remarks

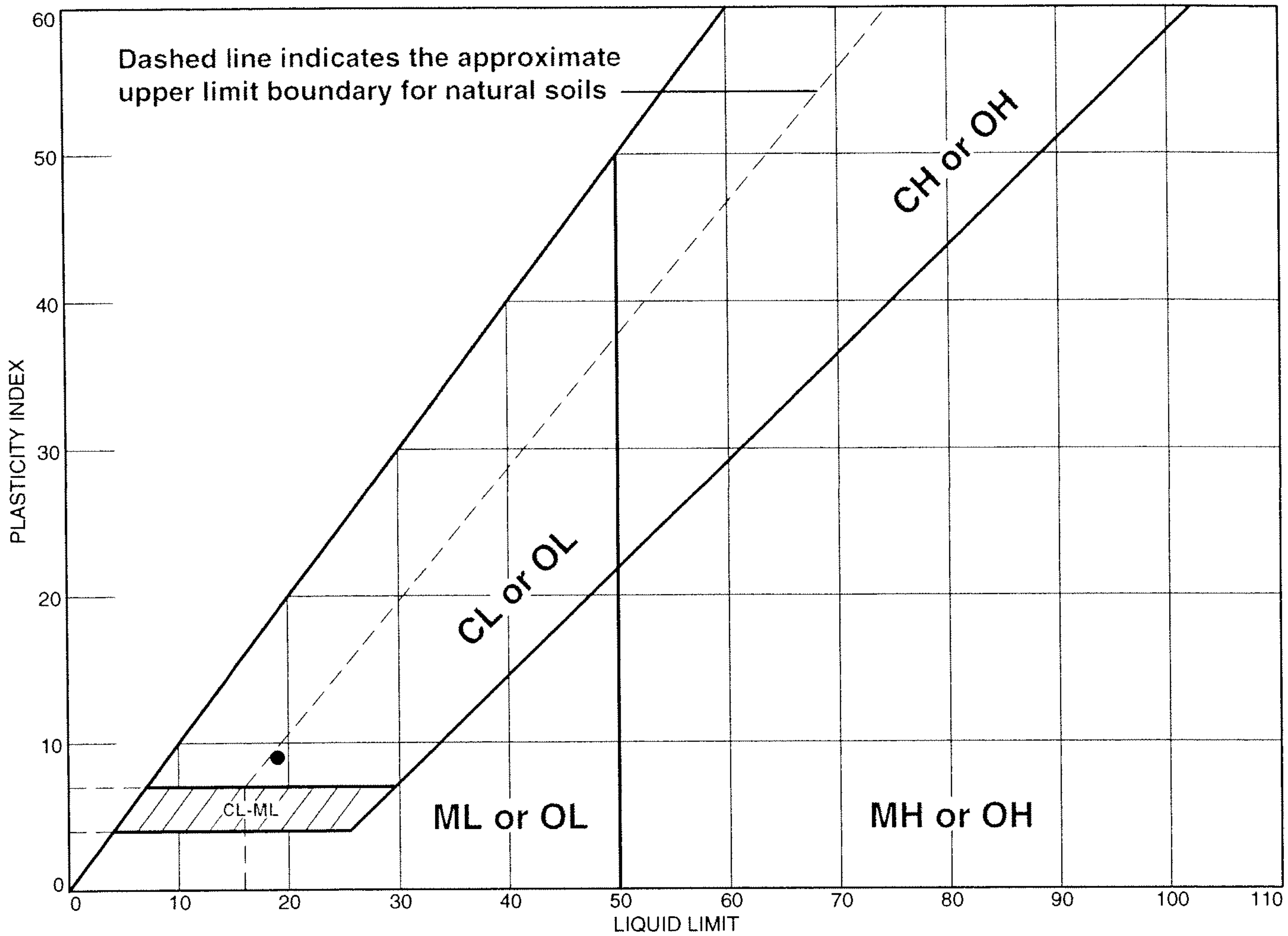
Location: B3, SPT-1
Sample Number: S37948 **Depth:** 1-2.5 **Date:** 6/20/12

SIERRA TESTING LABS, INC.
El Dorado Hills, CA

Client: SAGE
Project: Mather VA Medical Center COSS Building
10-024.00, Task Order 1
Project No: 12-081 **Figure**

Tested By: mpw **Checked By:** mn

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	sandy lean clay	19	10	9		61	CL

Project No. 12-081 **Client:** SAGE

Project: Mather VA Medical Center COSS Building

10-024.00, Task Order 1

● **Location:** B3, SPT-1 **Depth:** 1-2.5 **Sample Number:** S37948

Remarks:

SIERRA TESTING LABS, INC.

El Dorado Hills, CA

Figure

Tested By: jm **Checked By:** mn

Sunland Analytical

11353 Pyrites Way, Suite 4
Rancho Cordova, CA 95670
(916) 852-8557

Date Reported 06/27/2012
Date Submitted 06/21/2012

To: Jerry Pascoe
Sanders & Assoc. Geostructural Eng.
4180 Douglas Blvd. Ste #100
Granite Bay, Ca 95746

From: Gene Oliphant, Ph.D.
General Manager

The following is the report of analysis requested on SUN Order 62517.
Your purchase order number is .
Thank you for your business.

SUN #	Sample Describ	Sample #	Chloride as ppm Cl /Dry Wt.	Sulfate as ppm SO4 /Dry Wt.
128720	10-024.00/VA COSS	B2 @ 1-4'	No Test	25.2

Methods: Sulfate-Cal Trans #417, Chloride-Cal Trans #422