SALEM Engineering Group, Inc.

GEOTECHNICAL ENGINEERING INVESTIGATION
PROPOSED YOSEMITE CASCADES DEVELOPMENT
NWC HENNESS RIDGE ROAD AND HENNESS RIDGE DRIVE
YOSEMITE WEST, CALIFORNIA

PREPARED FOR:

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Job No. 1-209-0461
September 29, 2009

SALEM Engineering Group, Inc.

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Mr. Jeff Hornacek
7509 Yosemite Park Way
Yosemite National Park, CA 95389

Re: Geotechnical Engineering Investigation
Proposed Yosemite Cascades Development
NWC Henness Ridge Road and Henness Ridge Drive
Yosemite West, California

Dear Mr. Hornacek:

At your request and authorization, SALEM Engineering Group, Inc. (SALEM) has prepared this Geotechnical Engineering Investigation for the site of the proposed Yosemite Cascades development to be located in Yosemite West, California.

We appreciate the opportunity to assist you with this project. If you have any questions, or if we may be of further assistance, please do not hesitate to contact our office at (559) 271-9700.

Respectfully submitted,

SALEM Engineering Group, Inc.

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Exp. Dec. 31, 2010
GEOTECHNICAL
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TABLE OF CONTENTS

1.0 INTRODUCTION 1

2.0 PROJECT DESCRIPTION 1

3.0 SITE LOCATION AND DESCRIPTION 2

4.0 GEOLOGIC/SEISMIC CONDITIONS 2

5.0 PURPOSE AND SCOPE 3

6.0 FIELD EXPLORATION 3

7.0 LABORATORY TESTING 3

8.0 SOIL AND GROUNDWATER CONDITIONS 4

8.0 SOIL AND GROUNDWATER CONDITIONS 4

9.0 CONCLUSIONS AND RECOMMENDATIONS 5

9.1 Site and Subsurface Conditions 7
9.2 Soil Liquefaction and Seismic Settlement 7
9.3 Site Preparation and Grading 8
9.4 Filling and Compaction 9
9.5 Surface Drainage Control 10
9.6 Excavation Stability 11
9.7 Foundations - Conventional 11
9.8 Foundations - Caissons 13
9.9 Concrete Slabs-on-Grade 14
9.10 Lateral Earth Pressures and Frictional Resistance 15
9.11 Retaining Walls 16
9.12 Soil Resistivity and Soil-Corrosivity Protection 16
9.13 Utility Pipe Bedding and Backfillings 17
9.14 Pavement Design 17
9.15 Seismic Coefficients 18
9.16 Slope Stability 19
9.17 Percolation Testing 20

10.0 PLAN REVIEW, CONSTRUCTION OBSERVATIONS AND TESTING 21

11.0 CHANGED CONDITIONS 22

SITE LOCATION AND SITE PLAN ..... Following Text

LOGS OF PITS / LABORATORY TESTING ..... Appendix A

GENERAL EARTHWORK/PAVEMENT SPECIFICATIONS ..... Appendix B

GEOTECHNICAL ENGINEERING INVESTIGATION
PROPOSED YOSEMITE CASCADES DEVELOPMENT
NWC HENNESS RIDGE ROAD AND HENNESS RIDGE DRIVE
YOSEMITE WEST, CALIFORNIA

1.0 INTRODUCTION

This report presents the results of our Geotechnical Engineering Investigation for the site of the proposed Yosemite Cascades development to be located at the Yosemite West subdivision adjacent to Yosemite National Park in central California (see Figure 1).

The investigation included a field exploration program of logging a total of 12 backhoe exploration (test) pits, the collection of intact and bulk soil samples, and a variety of laboratory tests to supplement the field data. Discussions regarding site conditions are presented herein, together with conclusions and recommendations pertaining to road cut stability, site preparation, Engineered Fill, utility trench backfill, drainage and landscaping, foundations, concrete floor slabs and exterior flatwork, retaining walls, soil liquefaction, seismic-induced settlement, soil cement reactivity, and pavement design. Three percolation tests were also conducted in three of the above test pits. The location of the test pits are shown on Figure 2, Site Plan.

The results of the field exploration and laboratory and field test data are included in Appendix "A." Earthwork / Pavement Specifications are presented in Appendix "B." If conflicts in the text of the report occur with the specifications in the appendices, the recommendations in the text of the report have precedence.

2.0 PROJECT DESCRIPTION

SALEM's understanding of the project is based upon discussions with you, as well as a review of the Conceptual Geotechnical Engineering Evaluation prepared by Kleinfelder, Inc[1]. The proposed Cascades development will consist of hillside construction of 108 structures including: wood-framed guest houses (83), offices, small retail store, meeting hall, fire station, sewage treatment building, and other small support buildings. The buildings are anticipated to involve both caisson/raised floor construction over the hillside and slab-on-grade floors in cut. It is anticipated that wall and column loads will not exceed 2 to 3 kips/foot and 15 to 25 kips, respectively. The development will utilize on-site open space for infiltration of treated effluent. On-site parking and landscaping are also planned to be associated with the development.

[1] Kleinfelder, Inc., 2006, Conceptual Geotechnical Engineering Evaluation, Proposed Yosemite Cascades, Yosemite West, California; private consultants report dated Jan. 20, 2006; 12 pages.

A site grading plan was not available at the time of preparation of this report. As the existing project area is located on sloping ground, we anticipate that cuts and fills during earthwork will be moderate. In the event that changes occur in the nature or design of the project, the conclusions and recommendations contained in this report will not be considered valid unless the changes are reviewed and the conclusions of our report are modified.

3.0 SITE LOCATION AND DESCRIPTION

The site is triangular in shape and is located north of the intersection of Henness Ridge Road and Henness Ridge Drive, at the Yosemite West subdivision in central California (see Figure 2, Site Plan). The site encompasses an area of approximately 15 acres along the western boundary of Yosemite National Park in Mariposa County. Yosemite West, located approximately 60 miles north of Fresno, is a private subdivision of approximately 300 lots on 120 acres (with underground utilities and paved roads) that includes residences, vacation homes and apartments, and condominiums. Mountain residences are within 1,000 feet south and west of the proposed development.

The proposed development is located on the northwest facing slope of Henness Ridge, at an elevation of nearly 6,000 feet. A small creek runs generally north-south through the site with slopes on either side facing northwest or northeast. At the time of our field exploration (September 10, 2009), the site was vacant forested land. The site topography is fairly steep, with total relief of approximately 250 feet across the site. Existing slope gradients range from about 0.8:1 (H:V) to 8:1, with typical slopes in the range of 2:1 to 3:1.

4.0 GEOLOGIC/SEISMIC CONDITIONS

The project area is located on the north face of Henness Ridge, approximately 2 miles south of the Merced River canyon within the central portion of the Sierra Nevada Geomorphic Province of California. The Sierra Nevada, a fault block dipping gently southwestward, are comprised of igneous and metamorphic rocks of pre-Tertiary age that form the basement complex beneath the San Joaquin Valley. The extensive uplift of the block on its eastern border created much steeper relief on the eastern side of the range compared to the western side. Consequently, drainages on the eastern flank tend to be steeper and narrower than those on the western flank. Altitudes along the west slope of the Sierra Nevada vary from the few hundreds of feet msl in the foothill areas of the Sacramento and San Joaquin Valleys to 14,496 feet msl at Mount Whitney. The mountain range varies in width from about 40 to 80 miles. According to available geologic maps, the subject site is underlain at shallow depth by Cretaceous granitic bedrock (Finegold Intrusive Complex).

There are no active fault traces in the project vicinity. Accordingly, the project area is not within an Earthquake Fault Zone (Special Studies Zone) and will not require a special site investigation by an Engineering Geologist. Yosemite West residents could feel the affects of a large seismic event on one of the nearby active or potentially active fault zones. Yosemite West has experienced ground shaking from earthquakes in the historical past. According to the USGS, ground-shaking of VI and IV intensity (Modified Mercalli Scale) was felt in Yosemite West from the 1857 Owens Valley Earthquake and 1906 Great San Francisco Earthquake, respectively. These are the largest known earthquake events affecting the Yosemite West area.

Secondary hazards from earthquakes include rupture, seiche, landslides, liquefaction and subsidence. Since there are no known faults within the immediate area, ground rupture from surface faulting should not be a potential problem. Liquefaction potential (sudden loss of shear strength in a saturated cohesionless soil) should be low due to bedrock underlying the site. Seiche and deep subsidence are not hazards in the area either. Based on the presence of fairly unweathered and structureless bedrock at a shallow depth, and the lack of historical landslides in the area, issues related to potential landsliding are considered low.

5.0 PURPOSE AND SCOPE

The purpose of this investigation is to evaluate the subsurface conditions encountered during field exploration and to provide geotechnical engineering recommendations for site preparation, earthwork procedures, road construction, foundation and slab system design parameters, and percolation capacities of shallow deposits. The scope of our investigation included a program of field exploration, laboratory testing, percolation testing, engineering analysis and preparation of this report.

6.0 FIELD EXPLORATION

Our field exploration consisted of site surface reconnaissance and subsurface exploration. Exploratory tests boring (H-1 through H-13) were excavated by the client on March 31, 2009 within the proposed development areas at the approximate locations shown on Figure 2, Site Plan. The test pits were advanced with a tracked backhoe with a 24-inch bucket. The test pits were extended to depths ranging from 10 feet to backhoe refusal at depths of only 2 feet below the existing grade.

The materials encountered in the test pits were visually classified in the field, and logs were recorded by a Field Engineer under the oversight of a Professional Engineer at that time. Visual classification of the materials encountered in the test pits was generally made in accordance with the Unified Soil Classification System (ASTM D2487). A soil classification chart and key to sampling is presented on the Unified Soil Classification Chart, in Appendix "A." The logs of the test pits tests are presented in Appendix "A."

Relatively undisturbed subsurface soil samples were obtained by driving a hand drive sampler into the pit sidewalls. Penetration resistance blow counts necessary to drive the sampler to a maximum depth of 6 inches were recorded. The number of blows required to drive the last 6 inches is recorded as Penetration Resistance (blows/foot) on the logs of pits.

Soil samples were obtained from the test pits at the depths shown on the logs of pits. The samples were covered and capped at both ends to preserve the samples at their natural moisture content. At the completion of excavation and sampling, the test pits were backfilled with excavation cuttings.

7.0 LABORATORY TESTING

Laboratory tests were performed on selected soil samples to evaluate their physical characteristics and engineering properties. The laboratory testing program was formulated with emphasis on the evaluation of natural moisture, density, shear strength, consolidation potential, expansion potential, gradation and moisture-density relationships of the materials encountered.

In addition, soil resistivity and chemical tests were performed to evaluate the corrosivity of the soils to buried concrete and metal. Details of the laboratory test program and the results of laboratory test are summarized in Appendix "A." This information, along with the field observations, was used to prepare the final boring logs in Appendix "A."

8.0 SOIL AND GROUNDWATER CONDITIONS

Based on our findings, the subsurface conditions encountered appear typical of those found in the geologic region of the site. The surface of the subject property includes smaller areas of resistant granitic bedrock surface outcrops to larger areas of shallow to very deeply weathered bedrock. In general, the surface soil predominately consisted of 6 to 24 inches of soft silty sand residual soil with abundant root growth. These soils are disturbed, have low strength characteristics, and are highly compressible when saturated. Weathered bedrock below the residual soil layer is moderately strong and moderately to highly collapsible when saturated.

Within some test pits, the residual soil layer contained abundant cobble to boulder sized granitic core stones developed by weathering of bedrock along fractures. At some locations where large granitic matrix blocks were present in residual soil above generally massive and minimally fractured weathered bedrock, the blocks are thought to represent colluvial deposits generated by down slope movement from areas of surface bedrock outcrop. Below the surface residual soils, silty sand graded rapidly to gradually downward into progressively more weathered granitic bedrock at depths of approximately 2 feet to over 10 feet below the surface. The more weathered silty sand soils are moderately strong and are slightly compressible when saturated. These soils and decomposed bedrock extended to the termination depth of our pits.

The soils were classified in the field during the pit logging operations. The stratification lines were approximated by the field engineer on the basis of observations made at the time of logging. The actual boundaries between different soil types may be gradual and soil conditions may vary. For a more detailed description of the materials encountered, the Boring Logs (Figures A-1 through A-12, in Appendix "A") should be consulted.

The Pit Logs include the soil type, color, moisture content, dry density, and the applicable Unified Soil Classification System symbol. The locations of the test pits were determined by measuring from features shown on the Site Plan, provided to us. Hence, accuracy can be implied only to the degree that this method warrants.

Test pit locations were checked for the presence of groundwater following the excavation operations. Free groundwater was not encountered. We understand that a 400-feet deep water well drilled by the client on the subject site exhibits a water table at an approximate depth of 200 feet. It should be recognized that water table elevations may fluctuate with time, being dependent upon seasonal precipitation, irrigation, land use, and climatic conditions as well as other factors. Therefore, water level observations at the time of the field investigation may vary from those encountered during the construction phase of the project. The evaluation of such factors is beyond the scope of this report.

9.0 CONCLUSIONS AND RECOMMENDATIONS

Based upon the data collected during this investigation, and from a geotechnical engineering standpoint, it is our opinion that the site is suitable for the proposed construction. The proposed buildings may be supported on caissons with or without grade beams or shallow reinforced concrete foundations provided that the recommendations presented herein are incorporated in the design and construction of the project.

The site is covered by dense forest growth and the surface soils have a loose consistency. These soils are disturbed, have low strength characteristics, and are highly compressible when saturated. Accordingly, it is recommended that these surface soils be recompacted. This compaction effort should stabilize the surface soils and locate any unsuitable or pliant areas not found during our field investigation.

The upper soils within the project site are moisture-sensitive and are moderately to highly compressible under saturated conditions. Accordingly, mitigation measures are recommended to reduce potential excessive soil settlement. For a minimum bearing capacity of 1,500 psf, the exposed subgrade in the building areas should be scarified to a depth of 12 inches, worked until uniform and free from large clods, moisture-conditioned as necessary, and recompacted to a minimum of 90 percent of maximum density based on ASTM D1557 Test Method. Higher bearing pressures may be obtained with additional overexcavation below the base of the footings and the slab system, moisture-conditioning as necessary, and recompaction — please refer to Section 9.7 for details. The over-excavation should extend a minimum of 5 feet beyond footing lines.. Recommendations pertaining to the removal and recompaction of these moisture-sensitive soils are presented herein.

The site is located on sloping ground. It is recommended that cut and fill slopes (if proposed) be constructed to 2:1 (horizontal to vertical) or less. In lieu of those slopes, a retaining wall may be used. Cut and fill slopes for the retaining wall should not exceed 2:1 (horizontal to vertical). Cut and fill slopes may be revised as recommended by the Soils Engineer upon his review of a more definite site plan.

Cut and fill within the proposed building area is anticipated at the site. In order to minimize post-construction differential settlement, all structures that are in a cut/fill transition zones should be cut a minimum of 2 feet below foundation depth. Additional cut is required for cut/fill transition zones greater than 4 feet. All structures that are in cut/fill transition zones greater than 4 feet should be cut ½ the thickness of the fill placed on the "fill" portion (10 feet maximum). This excavation should extend a minimum of 5 feet beyond structural elements or to a minimum distance equal to the depth of over-excavation, whichever is greater.

Site grading near the crowns of reconstructed slopes should be accomplished such that excessive sheet run-off is prevented. The completed slopes should be seeded or otherwise vegetated to protect from future erosion. Well-vegetated slopes at the recommended configuration should be reasonably protected from typical erosional effects. However, vegetated slopes may not be protected from unusual flow conditions, such as flood events or over-topping of the development's storm drainage system.

If erosion control from unusual flow conditions is desired, more substantial erosion protection measures, such as grouted cobble slope facing or manufactured slope protection products should be considered. Undercutting the toe of the slope should not be permitted. Undercutting the toe of the slope will affect the stability of the slope. However, the instability can be repaired prior to affecting the overall stability of the slope. It is recommended that site be graded so that water is directed away from the slope. The grade change will actually reduce the amount of surface water impacting the slope; therefore, no adverse affects are anticipated.

Storm water should be collected and directed toward streets. The water should be disposed in a storm drain system. The storm drain system will minimize seepage of storm water and will reduce natural subsurface drainage of water. Therefore, it is not anticipated that there will be an increase of subsurface drainage along the slope. The Soils Engineer, for conformance of the soils investigation, should perform coordination review and approval of the site grading and drainage plan prepared by the Project Engineer.

The shrinkage on recompacted soil and fill placement is estimated at 15 to 20 percent. Subsidence within building areas will be less than 0.01 feet, due to the previous Engineered Fill placement.

Sandy soil conditions were encountered at the site. These cohesionless soils have a tendency to cave in trench wall excavations. Shoring or sloping back trench sidewalls may be required within these sandy soils.

Liquefaction potential was evaluated at the site. Based on our findings, it is our opinion that the potential for liquefaction at the site is low. Therefore, no mitigation measures would be warranted.

Moderately to highly weathered granitic bedrock underlying a surficial residual soil was encountered at the site. The weathered bedrock is an excellent foundation bearing material due to its high strength and low porosity. However, these characteristics also retard the free percolation of surface water from residual soils into the deeper materials, frequently resulting in a temporary perched water table condition at or near the ground surface. This perched water condition has a substantial effect on the strength characteristics of the surface soils. As a mitigation measure, very positive drainage should be established around the proposed structures.

After completion of the recommended site preparation, the site should be suitable for shallow footing support. Structure isolated column or continuous footings may be designed utilizing an allowable bearing pressure of 1,500 to 3,000 psf for dead-plus-live loads. Please refer to Section 9.7 for alternative bearing capacities and associated overexcavation requirements. Based on a minimum embedment into weathered bedrock of 5 feet below the base of the residual soil layer, caissons should accommodate an end bearing of 4,000 psf.

Bearing wall or isolated column footings considered for the structure should be continuous with a minimum width of 12 inches and should be extended to a minimum depth of 12 inches below the lowest adjacent grade. Detailed geotechnical engineering recommendations are presented in the remaining portions of the text. The recommendations are based on the properties of the materials identified during our investigation.

9.1 Groundwater Influence on Structures/Construction

Based on our findings and historical records, it is not anticipated that groundwater will rise within the zone of structural influence or affect the construction of foundations and pavements for the project. However, if earthwork is performed during or soon after periods of precipitation, the subgrade soils may become saturated, "pump," or not respond to densification techniques.

Typical remedial measures include: discing and aerating the soil during dry weather; mixing the soil with dryer materials; removing and replacing the soil with an approved fill material; or mixing the soil with an approved lime or cement product. Our firm should be consulted prior to implementing remedial measures to observe the unstable subgrade conditions and provide appropriate recommendations.

9.2 Soil Liquefaction and Seismic Settlement

Soil liquefaction is a state of soil particles suspension caused by a complete loss of strength when the effective stress drops to zero. Liquefaction normally occurs under saturated conditions in soils such as sand in which the strength is purely frictional. However, liquefaction has occurred in soils other than clean sand. Liquefaction usually occurs under vibratory conditions such as those induced by seismic events. To evaluate the liquefaction potential of the site, the following items were evaluated:

1) Soil type
2) Groundwater depth
3) Relative density
4) Initial confining pressure
5) Intensity and duration of groundshaking

The soils encountered within the depth of 10 feet on the project site, predominately consisted of silty sand and decomposed granite. Low to very low cohesion strength is associated with the sandy soil. Groundwater was not encountered during our subsurface exploration to a maximum depth explored of 10 feet. A seismic hazard, which could cause damage to the proposed development during seismic shaking, is the post-liquefaction settlement of the liquefied sands. As such, the site was evaluated for liquefaction potential. Based on the shallow bedrock nature of the soil profile and the low seismicity of the region, it is our opinion that the site soils have a low potential for liquefaction under seismic conditions; therefore, no mitigation measures are warranted.

One of the most common phenomena during seismic shaking accompanying any earthquake is the induced settlement of loose unconsolidated soils. Based on site subsurface conditions and the moderate seismicity of the region, any loose fill materials at the site could be vulnerable to this potential hazard. However, this hazard can be mitigated by following the design and construction recommendations of our Geotechnical Engineering Investigation (over-excavation and rework of the loose soils and/or fill).

Based on the high penetration resistance measured, the native deposits underlying the surface materials do not appear to be subject to significant seismic settlement.

9.3 Site Preparation and Grading

The upper 6 to 24 inches of the soils containing vegetation, roots and other objectionable organic matter encountered at the time of grading should be stripped and processed to remove the organic material (greater than 3 percent by weight and roots greater than ½ inch in diameter) from the building and pavement areas and at least 5 feet outside the building perimeter. Deeper stripping may be required in localized areas. These materials will not be suitable for use as Engineered Fill. However, stripped topsoil may be stockpiled and reused in landscape or non-structural areas.

Of primary importance in the development of this site is the removal/recompaction of moisture-sensitive and potentially compressible soils from the areas of the proposed structures. <u>To minimize post-construction soil movement and to achieve a minimum allowable bearing capacity of 1,500 psf, it is recommended that the exposed subgrade in the building areas be scarified to a depth of 12 inches, worked until uniform and free from large clods, moisture-conditioned as necessary, and recompacted to a minimum of 90 percent of maximum density based on ASTM D1557 Test Method. For higher bearing pressures, 1 to 3 feet of soil may be excavated below the base of the footings and the slab system, moisture-conditioned as necessary, and recompacted to a minimum of 90 percent of maximum density based on ASTM D1557 Test Method (see Section 9.7 for details).</u> Native sand, silty sand, or silty sand/sand soils are suitable for reuse as Engineered Fill. Over-excavation should extend to a minimum of 5 feet beyond the structural elements.

The exposed subgrade in the exterior flatwork and pavement areas should be scarified to a depth of 12 inches, worked until uniform and free from large clods, moisture-conditioned as necessary, and recompacted to a minimum of 90 percent of maximum density based on ASTM D1557 Test Method.

In order to minimize post-construction differential settlement, all structures that are in a cut/fill transition zones should be cut a minimum of 2 feet below foundation depth. Additional cut is required for cut/fill transition zones greater than 4 feet. All structures that are in cut/fill transition zones greater than 4 feet should be cut ½ the thickness of the fill placed on the “fill” portion (10 feet maximum). This excavation should extend a minimum of 5 feet beyond structural elements, or to a minimum distance equal to the depth of over-excavation, whichever is greater.

Where fills greater than 8 feet are to be constructed on original ground that slopes at inclinations steeper than 6:1 (horizontal to vertical), benches should be cut into the existing slope as the filling operations proceed. Each bench should consist of a level terrace a minimum of 6 feet wide, with the rise to the next bench held to 3 feet or less. Where fills of comparable height will be constructed on ground that slopes at an inclination steeper than 4:1 (horizontal to vertical), a keyway should be provided in addition to the benches. Each keyway should consists of a level trench at least 8 feet wide and at least 2 feet deep, with side slopes not exceeding 1:1 (horizontal to vertical), cut into the existing slope.

The site is located on sloping ground. It is recommended that the proposed cut and fill slopes be constructed to 2:1 (horizontal to vertical). In lieu of those slopes, a retaining wall may be used. Cut and fill slopes for the building pad should not exceed 2:1 (horizontal to vertical). Cut and fill slopes may be revised as recommended by the Soils Engineer upon his review of a more definite site plan.

Sandy soil conditions were also encountered at the site. These cohesionless soils have a tendency to cave in trench wall excavations. Shoring or sloping back trench sidewalls may be required within these sandy soils. The shrinkage on recompacted soil and fill placement is estimated at 15 to 20 percent.

The upper soils, during wet winter months, become very moist due to the absorption characteristics of the soil. Earthwork operations performed during winter months may encounter very moist unstable soils, which may require removal of soil to a stable building foundation. Project site winterization consisting of placement of aggregate base and protecting exposed soils during construction should be performed.

Excavations, depressions, or soft and pliant areas extending below planned finished subgrade levels should be cleaned to firm, undisturbed soil and backfilled with Engineered Fill. Any buried structures encountered during construction should be properly removed and backfilled. In general, any septic tanks, debris pits, cesspools, or similar structures should be entirely removed. Concrete footings should be removed to an equivalent depth of at least 3 feet below proposed footing elevations or as recommended by the Geotechnical Engineer. Any other buried structures should be removed in accordance with the recommendations of the Geotechnical Engineer. Resulting excavations should be properly backfilled.

A representative of our firm should be present during all site clearing and grading operations to test and observe earthwork construction. This testing and observation is an integral part of our service as acceptance of earthwork construction is dependent upon compaction of the material and the stability of the material. The Geotechnical Engineer may reject any material that does not meet compaction and stability requirements. Further recommendations of this report are predicated upon the assumption that earthwork construction will conform to recommendations set forth in this section and in Section 9.4.

9.4 Filling and Compaction

The organic-free, on-site soils are predominantly silty sand and decomposed granite. The sandy soils will be suitable for reuse as engineered fill, provided they are cleansed of excessive organics, debris, and oversize material.

The preferred materials specified for engineered fill are suitable for most applications with the exception of exposure to erosion. Project site winterization and protection of exposed soils during the construction phase should be the sole responsibility of the Contractor, since he has complete control of the project site.

Imported non-expansive non-corrosive fill should consist of a well-graded, slightly cohesive silty fine sand or sandy silt, with relatively impervious characteristics when compacted. This material should be approved by the Engineer prior to use and should typically possess the following characteristics:

Maximum Percent Passing No. 200 Sieve	50
Maximum Particle Size	3"
Maximum Plasticity Index	10
Maximum UBC Standard 29-2 Expansion Index	15

Fill soils should be placed in lifts approximately 6 inches thick, moisture-conditioned as necessary and compacted to achieve at least 90 percent of the maximum dry density as determined by ASTM D1557. Additional lifts should not be placed if the previous lift did not meet the required dry density or if soil conditions are not stable.

9.5 Surface Drainage Control

Within building and paved areas, cut and fill slopes should not exceed 2:1 (horizontal to vertical). The ground surface should slope away from building pad and pavement areas toward appropriate drop inlets or other surface drainage devices. It is recommended that adjacent exterior grades be sloped a minimum of 2 percent for a minimum distance of 5 feet away from structures. Subgrade soils in pavement areas should be sloped a minimum of 1 percent and drainage gradients maintained to carry all surface water to collection facilities and off-site. These grades should be maintained for the life of the project.

Slots or weep holes should be placed in drop inlets or other surface drainage devices in pavement areas to allow free drainage of adjoining base course materials. Cutoff walls should be installed at pavement edges adjacent to vehicular traffic areas; these walls should extend to a minimum depth of 6 inches below pavement subgrades, to limit the amount of seepage water that can infiltrate the pavements. Where cutoff walls are undesirable, subgrade drains can be constructed to transport excess water away from planters to drainage interceptors. If cutoff walls can be successfully used at the site, construction of subgrade drains is considered unnecessary. The ground surface should slope away from building pad and pavement areas toward appropriate drop inlets or other surface drainage devices. It is recommended that adjacent exterior grades be sloped a minimum of 2 percent for a minimum distance of 5 feet away from structures.

Subgrade soils in pavement areas should be sloped a minimum of 1 percent and drainage gradients maintained to carry all surface water to collection facilities and off site. These grades should be maintained for the life of the project. Roof drains should be installed with appropriate downspout extensions out-falling on splash blocks so as to direct water a minimum of 5 feet away from the structures or be connected to the storm drain system for the development.

The site should be graded to prevent water/run-off flow over the face of cut and fill slopes. To accomplish this, we recommend the use of asphalt berms, brow ditches, or other measures to intercept and slowly redirect flow. All disturbed areas should be planted with erosion-resistant vegetation suited to the area. As an alternative, jute netting or geotextile erosion control mats may be considered for control of erosion. Slopes should be inspected periodically for erosion and repaired immediately if detected.

9.6 Excavation Stability

Temporary excavations planned for the construction of the proposed building and other associated underground structures may be excavated, according to the accepted engineering practice following Occupational Safety and Health Administration (OSHA) standards by a contractor experienced in such work. Open, unbraced excavations in undisturbed soils should be made according to the table below.

Recommended Excavation Slopes	
Depth of Excavation (ft)	**Slope (Horizontal:Vertical)**
0-5	1:1
5-10	1½:1

If, due to space limitation, excavations near existing structures are performed in a vertical position, braced shorings or shields may be used for supporting vertical excavations. Therefore, in order to comply with the local and state safety regulations, a properly designed and installed shoring system would be required to accomplish planned excavations and installation.

A Specialty Shoring Contractor should be responsible for the design and installation of such a shoring system during construction. Braced shorings should be designed for a maximum pressure distribution of 25H, *(where H is the depth of the excavation in feet).* The foregoing does not include excess hydrostatic pressure or surcharge loading. Fifty percent of any surcharge load, such as construction equipment weight, should be added to the lateral load given herein. Equipment traffic should concurrently be limited to an area at least 3 feet from the shoring face or edge of the slope.

The excavation and shoring recommendations provided herein are based on soil characteristics derived from the test pits within the area. Variations in soil conditions will likely be encountered during the excavations. SALEM Engineering Group, Inc. should be afforded the opportunity to provide field review to evaluate the actual conditions and account for field condition variations not otherwise anticipated in the preparation of this recommendation. Slope height, slope inclination, or excavation depth should in no case exceed those specified in local, state, or federal safety regulation, (e.g. OSHA) standards for excavations, 29 CFR part 1926, or Assessor's regulations.

9.7 Foundations - Conventional

Footing concrete should be placed into a neat excavation. The bottom of footing excavations should be maintained free of loose and disturbed soil. Footings constructed as recommended herein may be designed for the maximum bearing capacity shown below. These values are for dead and sustained live loads and may be increased by one-third (1/3) to include wind and seismic effects.

	Allowable Loading*
Dead Load Only	1,150 psf
Dead-Plus-Live Load	1,500 psf
Total Load, Including Wind or Seismic Loads	2,000 psf

** Footings are supported by in-situ, prepared native soils*

Load	Allowable Loading**
Dead Load Only	1,500 psf
Dead-Plus-Live Load	2,000 psf
Total Load, Including Wind or Seismic Loads	2,650 psf

*** Footings are supported by 1 feet of Engineered Fill, Compacted 90 % Relative Compaction*

Load	Allowable Loading***
Dead Load Only	2,000 psf
Dead-Plus-Live Load	2,500 psf
Total Load, Including Wind or Seismic Loads	3,300 psf

**** Footings are supported by 2 feet of Engineered Fill, Compacted 90 % Relative Compaction*

Load	Allowable Loading****
Dead Load Only	2,250 psf
Dead-Plus-Live Load	3,000 psf
Total Load, Including Wind or Seismic Loads	4,000 psf

***** Footings are supported by 3 feet of Engineered Fill, Compacted 90 % Relative Compaction*

Bearing wall footings considered for the structure should be continuous with a minimum width of 12 inches and extend to a minimum depth of 12 inches below the lowest adjacent grade. Isolated column footings should have a minimum width of 18 inches and extend a minimum depth of 18 inches below the lowest adjacent grade. Lowest adjacent grade is defined herein as sub-slab soil grade or exterior grade, whichever is lower. Footings constructed less than 5 feet from the face of the slope should be extended an additional foot for every foot closer to the face of the slope. Minimum embedment depths may be decreased due to the presence of minimally weathered, resistant (non-rippable) bedrock.

For design purposes, total settlement on the order of ½ to ¾ inch may be assumed for shallow foundations. Differential settlement, along a 30-foot exterior wall footing or between adjoining column footings, should be ¼ to ½ inch, producing an angular distortion of 0.002. Most of the settlement is expected to occur during construction as the loads are applied. However, additional post-construction settlement may occur if the foundation soils are flooded or saturated. The footing excavations should not be allowed to dry out any time prior to pouring concrete

Resistance to lateral footing displacement can be computed using an allowable friction factor of 0.45 acting between the base of foundations and the supporting subgrade. Lateral resistance for footings can alternatively be developed using an ultimate equivalent fluid passive pressure of 325 pounds per cubic foot acting against the appropriate vertical footing faces. The frictional and passive resistance of the soil may be combined without reduction in determining the total lateral resistance. A one-third increase in the value above may be used for short duration, wind, or seismic loads.

9.8 Foundations - Caissons

Friction Caissons

Structures associated with the proposed improvements planned to be constructed using deep foundations can be supported on caissons using an allowable sidewall friction of 300 psf. This value is for dead-plus-live loads. Uplift loads can be resisted by caissons using an allowable sidewall friction of 170 psf of the surface area and the weight of the pier. This value may be increased one-third for short duration loads, such as wind or seismic. Caissons should have a minimum embedment depth of 10 feet, or to competent (non-rippable) bedrock.

End-Bearing Caissons

Caissons may be designed using an end bearing capacity of 4,000 psf. Caissons constructed as recommended herein may be designed for the maximum end bearing capacity shown below. These values are for dead and sustained live loads and may be increased by one-third (1/3) to include wind and seismic effects.

Load	Allowable Loading
Dead Load Only	3,000 psf
Dead-Plus-Live Load	4,000 psf
Total Load, Including Wind or Seismic Loads	5,300 psf

The caisson diameter should be at least 36 inches. Casing and shaft diameter should match. Undersized casing should not be used and the casings should have adequate strength to reduce earth pressure. The casings should be progressively pushed before drilling. Precautions should accordingly be taken to minimize caving. The bottom of the caisson should be cleaned. Drilling speed and timing of concrete placement should be coordinated. Concrete pumps with adequate hose length to allow gradual impact-free filling of pier cavities is recommended. Concrete in the sandy area should not be allowed to fall freely more than 3 feet and should be prevented from striking the walls of the drilled hole, thus, creating soil sloughing and caving. Concrete with a slump on the order of 5 to 6 inches should be used.

Caisson Lateral Loads

Lateral loads for caissons may be designed utilizing the Isolated Pole Formula and Specifications shown on Table 1804.2, Sections 1804.3.1 and 1808.2.2 of the 2006 International Building Code (IBC).

The drilled caissons may be designed for a lateral capacity of 255 pounds per square foot per foot of depth below the lowest adjacent grade (assuming a 3:1 (H:V) slope) to a maximum of 4,000 psf.

These values may be increased by one-third when using the alternative load combinations in Section 1605.3.2 of the 2006 IBC that include wind or earthquake loads. These values should not be doubled since the values given herein are higher than the tabular values shown on the Table 1804.2. The lateral loading criteria is based on the assumption that the load application is applied at the ground level, flexible cap connections applied and a minimum embedment depth of 10 feet.

The total settlement of the caissons is not expected to exceed 1 inch. Differential settlement should be less than ½ inch. Most of the settlement is expected to occur during construction as the loads are applied.

Sandy soil and shallow water conditions were encountered at the site. These cohesionless soils have a tendency to cave during pier drilling. Casing of the drilled piers may be required.

9.9 Concrete Slabs-on-Grade

We recommend that non-structural slabs-on-grade be a minimum of 4 inches thick. In areas where it is desired to reduce floor dampness where moisture-sensitive coverings are anticipated, slab-on-grade construction should have a suitable waterproof vapor retarder (a minimum of 10 mil vapor retarder sheeting) incorporated into the floor slab design. The water vapor retarder should be installed in accordance with ASTM Specification E 1643-94. According to ASTM Guidelines, interior slabs-on-grade should have at least 2 inches of clean free-draining concrete sand placed below the floor slab.

The sand should conform to ASTM C33 requirements for fine aggregate. An impervious retarder (vapor barrier) should be placed under the 2 inches of sand. Because of the importance of the membrane, joints and perforations should be properly sealed. This system of 2 inches of sand and a vapor barrier should be underlain by an additional 3 inches of compacted, clean, open-graded coarse rock of ¾-inch maximum size to prevent capillary moisture rise and to facilitate concrete placement and curing. The subgrade should be kept in a moist condition until time of slab placement.

Moisture within the structure may be derived from water vapors, which were transformed from the moisture within the soils. This moisture vapor can travel through the vapor membrane and penetrate the slab-on-grade. This moisture vapor penetration can affect floor coverings and produce mold and mildew in the structure. To minimize moisture vapor intrusion, it is recommended that a vapor retarder be installed in accordance with ASTM guidelines.

It is recommended that the utility trenches within the structure be compacted, as specified in our report, to minimize the transmission of moisture through the utility trench backfill. Special attention to the immediate drainage and irrigation around the building is recommended. Positive drainage should be established away from the structure and should be maintained throughout the life of the structure. Ponding of water should not be allowed adjacent to the structure. Over-irrigation within landscaped areas adjacent to the structure should not be performed. In addition, ventilation of the structure is recommended to reduce the accumulation of interior moisture.

Slabs subject to structural loading may be designed utilizing a modulus of subgrade reaction K of 250 pounds per square inch per inch. The K value was approximated based on inter-relationship of soil classification and bearing values (Portland Cement Association, Rocky Mountain Northwest). In order to regulate cracking of the slabs, we recommend that full depth construction joints or control joints be provided at a maximum spacing of 15 feet in each direction for 5-inch thick slabs and 12 feet for 4-inch thick slabs. Control joints should have a minimum of one-quarter of the slab thickness.

The exterior floors should be poured separately in order to act independently of the walls and foundation system. Exterior finish grades should be sloped a minimum of 1 to 1½ percent away from all interior slab areas to preclude ponding of water adjacent to the structures. All fills required to bring the building pads to grade should be Engineered Fills.

9.10 Lateral Earth Pressures and Frictional Resistance

Active, at-rest and passive unit lateral earth pressures against footings and walls are presented below. These values assume a minimum shear angle for backfill material of 36 degrees.

Lateral Pressure Conditions	Ultimate Equivalent Fluid Pressure, pcf			
	Horiz	3:1 Slope	2:1 Slope	1:1 Slope
Active Pressure, Drained	22	25	31	42
At-Rest Pressure, Drained	35	40	48	65
Passive Pressure	325	255	190	70

For passive resistance associated with caisson design, the upper 2 feet of the soil profile (creep zone) has a value of 0 pcf; from 2 to 5 feet bgs a value of 250 pcf; and from 5 feet bgs and deeper a value of 325 pcf.

Active pressure applies to walls, which are free to rotate. At-rest pressure applies to walls, which are restrained against rotation. The preceding lateral earth pressures assume sufficient drainage behind retaining walls to prevent the build-up of hydrostatic pressure. The top one-foot of adjacent subgrade should be deleted from the passive pressure computation. An allowable coefficient of friction of 0.45 may be used between soil subgrade and footings or slabs.

The foregoing values of lateral earth pressures and frictional coefficients represent ultimate soil values and a safety factor consistent with the design conditions should be included in their usage. For stability against lateral sliding, which is resisted solely by the passive pressure, we recommend a minimum safety factor of 1.5. For stability against lateral sliding, which is resisted by the combined passive and frictional resistance, a minimum safety factor of 2.0 is recommended. For lateral stability against seismic loading conditions, we recommend a minimum safety factor of 1.1.

9.11 Retaining Walls

Retaining and/or below grade walls (if required) should be drained with either perforated pipe encased in free-draining gravel or a prefabricated drainage system. The gravel zone should have a minimum width of 12 inches wide and should extend upward to within 12 inches of the top of the wall. The upper 12 inches of backfill should consist of native soils, concrete, asphaltic-concrete or other suitable backfill to minimize surface drainage into the wall drain system. The aggregate should conform to Class II permeable materials graded in accordance with Section 68-1.025 of the CalTrans Standard Specifications (January 1988). Prefabricated drainage systems, such as Miradrain®, Enkadrain®, or an equivalent substitute, are acceptable alternatives in lieu of gravel provided they are installed in accordance with the manufacturer's recommendations.

If a prefabricated drainage system is proposed, our firm should review the system for final acceptance prior to installation. Drainage pipes should be placed with perforations down and should discharge in a non-erosive manner away from foundations and other improvements. The top of the perforated pipe should be placed at or below the bottom of the adjacent floor slab or pavements. The pipe should be placed in the center line of the drainage blanket and should have a minimum diameter of 4 inches. Slots should be no wider than 1/8-inch in diameter, while perforations should be no more than ¼-inch in diameter.

If retaining walls are less than 6 feet in height, the perforated pipe may be omitted in lieu of weep holes on 4 feet maximum spacing. The weep holes should consist of 4-inch diameter holes (concrete walls) or unmortared head joints (masonry walls) and placed no higher than 18 inches above the lowest adjacent grade. Two 8-inch square overlapping patches of geotextile fabric (conforming to Section 88-1.03 of the CalTrans Standard Specifications for "edge drains") should be affixed to the rear wall opening of each weep hole to retard soil piping.

During grading and backfilling operations adjacent to any walls, heavy equipment should not be allowed to operate within a lateral distance of 5 feet from the wall, or within a lateral distance equal to the wall height, whichever is greater, to avoid developing excessive lateral pressures. Within this zone, only hand operated equipment ("whackers," vibratory plates, or pneumatic compactors) should be used to compact the backfill soils.

9.12 Soil Resistivity and Soil Corrosivity Protection

Excessive sulfate in either the soil or native water may result in an adverse reaction between the cement in concrete (or stucco) and the soil. A commonly accepted correlation between electrical resistivity and corrosivity to ferrous metals is presented below.

Below 1,000 ohm-cm	--	Severely Corrosive
1,000 to 2,000 ohm-cm	--	Highly Corrosive
2,000 to 10,000 ohm-cm	--	Moderately Corrosive
Over 10,000 ohm-cm	--	Mildly Corrosive

One soil sample obtained from the project site underwent chemical and soil resistivity testing for the evaluation of the potential for concrete deterioration or steel corrosion due to attack by soil-borne soluble salts. The following table presents the results of our testing.

Sample No.	Soluble SO_4 (mg/kg)	Soluble Cl (mg/kg)	pH	Resistivity (ohm-cm)
HA-5 @ 5’	69	15	6.6	206,435

Based on the above values, negligible corrosion hazard for concrete exists. Normally-formulated concrete has been shown to adequately resist the above soil sulfate concentration. Based on the soil resistivity and chloride content, soils may be very slightly corrosive to buried steel or ductile iron piping and conduit. At a minimum, the manufacturer’s recommendations for corrosion protection for this material should be followed. A concrete cover of 3 inches using Type I or II cement is considered adequate to provide protection for reinforcing steel.

9.13 Utility Pipe Bedding and Backfilling

Utility trenches should be excavated according to accepted engineering practice following OSHA (Occupational Safety and Health Administration) standards by a contractor experienced in such work. The responsibility for the safety of open trenches should be borne by the contractor. Traffic and vibration adjacent to trench walls should be minimized; cyclic wetting and drying of excavation side slopes should be avoided. Depending upon the location and depth of some utility trenches, groundwater flow into open excavations could be experienced; especially during or following periods of precipitation.

Sandy soil conditions were encountered at the site. These cohesionless soils have a tendency to cave in trench wall excavations. Shoring or sloping back trench sidewalls may be required within these sandy soils.

Utility trench backfill placed in or adjacent to buildings and exterior slabs should be compacted to at least 90 percent of maximum density based on ASTM D1557 Test Method. The upper 2 feet of utility trench backfill placed in pavement areas should be compacted to at least 90 percent of maximum density based on ASTM D1557 Test Method. Pipe bedding should be in accordance with pipe manufacturer recommendations.

The contractor is responsible for removing all water-sensitive soils from the trench regardless of the backfill location and compaction requirements. The contractor should use appropriate equipment and methods to avoid damage to the utilities and/or structures during fill placement and compaction.

9.14 Pavement Design

The following table shows the recommended pavement sections for various traffic indices. These recommended values assume an R-value of 50, which was based on the engineering properties of similar soil types. Following site grading, test(s) for R-value should be conducted to determine final design parameters.

ASPHALTIC CONCRETE

Traffic Index	Asphaltic Concrete	Class II Aggregate Base*	Compacted Subgrade**
4.5	2.5"	4.0"	12.0"
5.0	2.5"	4.0"	12.0"
5.5	3.0"	4.0"	12.0"
6.0	3.0"	4.0"	12.0"
6.5	3.5"	4.0"	12.0"
7.0	4.0"	4.5"	12.0"

*** 95% compaction based on ASTM D1557 Test Method***

*** 90% compaction based on ASTM D1557 Test Method*

9.15 Seismic Coefficients

For seismic design of the structures, and in accordance with the seismic provisions of the 2006 IBC and 2007 CBC, our recommended parameters are shown below. These parameters are based on Probabilistic Ground Motion of 2% Probability of Exceedance in 50 years. The Site Classification was determined based on the results of field exploration as documented in this geotechnical report.

Seismic Item	Symbol	Value	2006 IBC Reference
Site Coordinates (Datum = NAD 83)		37.6499 Lat -119.7154 Lon	
Site Class		B	Table 1615.5.2
Soil Profile Name		Rock	Table 1615.5.2
Mapped Spectral Acceleration (Short period - 0.2 sec)	S_S	0.71 g	Figure 1613.5.1*
Mapped Spectral Acceleration (1.0 sec. period)	S_1	0.24 g	Figure 1613.5.1*
Site Class Modified Site Coefficient	F_a	1.0	Table 1613.5.3(1)
Site Class Modified Site Coefficient	F_v	1.0	Table 1613.5.3(2)
MCE Spectral Response Acceleration (Short period - 0.2 sec) $S_{MS} = F_a S_S$	S_{MS}	0.71 g	Equation 16-37
MCE Spectral Response Acceleration (1.0 sec. period) $S_{M1} = F_v S_1$	S_{M1}	0.24 g	Equation 16-38
Design Spectral Response Acceleration $S_{DS} = b\ S_{MS}$ (short period - 0.2 sec)	S_{DS}	0.47 g	Equation 16-39
Design Spectral Response Acceleration $S_{D1} = b\ S_{M1}$ (1.0 sec. period)	S_{D1}	0.16 g	Equation 16-40

* Also used USGS National Seismic Hazard Mapping Program Java applet tool to determine site-specific accelerations (available at http://earthquake.usgs.gov/research/hazmaps/design/).

9.16 Slope Stability

Inspection of the subject building site and vicinity revealed it to be composed granitic bedrock. The site surface is composed of approximately two to four feet of colluvium (residual soil) from weathering of the subjacent bedrock. Below the colluvium, the bedrock is massive to slightly-fractured, forming a massive to blocky texture — bedrock structure in this zone is generally lacking.

Man-made and natural vertical features were inspected for evidence of slope creep (e.g.; leaning fence posts or leaning or "pistol butt" trees). Evidence of slope instability in the form of creep was not generally observed.

Based on our research and investigation, the proposed footing embedment should provide adequate slope stability protection provided the proposed structure footings are constructed as recommended in this report.

Following are the data (generated during SALEM's subject investigations at the site) and engineering assumptions used to calculate slope stability for typical road cuts for the proposed development. These values are for geologic material below a depth of approximately 2 to 4 feet bgs, below the soil mantle and within the upper portion of weathered bedrock.

Slope Height	20 ft
Slope Angle	45° (1:1 H:V)
Soil Shear Angle	35°
Cohesion	350 psf (weathered granitic bedrock)
Soil Density	95 pcf (saturated)

SALEM used a minimum static factor of safety of 1.5 to determine whether possible building pad or road cuts would be stable relative to deep-seated failure. The factor of safety of the proposed slopes was calculated using the above data and is based upon a circular failure surface and charts in Hoek and Bray (1977)[2]. The chart used assumed a fully saturated slope. Based on the above input parameters, a minimum factor of safety of 1.7 was calculated; therefore, the proposed construction slope below the residual soil layer appears to be stable.

Cuts constructed in to more competent bedrock than that analyzed above (using higher cohesion and shear angle values) should be stable at slopes to at least a 0.75:1 (H:V).

The recommendations provided herein are based on soil characteristics derived from the test borings within the subject site area. Variations in soil conditions may be encountered during the excavations. SALEM Engineering Group, Inc. should be afforded the opportunity to provide field review to evaluate the actual conditions and account for field condition variations not otherwise anticipated in the preparation of this recommendation. Slope height, slope inclination, or excavation depth should in no case exceed those specified in local, state, or federal safety regulation, (e.g. OSHA) standards for excavations, 29 CFR part 1926, or Assessor's regulations.

[2] Hoek, E. and Bray, J. (1974): Rock Slope Engineering. 3rd ed. 1981. Publisher: Institute of Mining and Metallurgy, Spon Press.

9.17 Percolation Testing

Three percolation tests were conducted in general accordance with the criteria set forth in the "Manual of Septic Tank Practice," published by the Department of Health, Education, and Welfare.

Test pits H-1, H-3, and H-12 were the locations of percolation tests P-1, P-2, and P-3, respectively. Logs of the pits are presented in Appendix A. After excavating the test pits to approximately 8 to 10 feet deep using a backhoe, an approximately 1 square foot by 1 foot deep hole was hand dug in the excavation bottom. The bottom of the hole was covered with 2 inches of gravel and a standpipe was placed into the gravel to use a depth reference point. Each hole was pre-soaked for at least 24 hours. Following the pre-soaking period, the holes were filled water to approximately 12 inches above the bottom of the hole, and the water level was allowed to fall with percolation. Water level readings were recorded at each hole at the frequency noted on the attached percolation test worksheets (Appendix A) to determine the rate of percolation. After falling from 2 to 6 inches, the test holes were re-filled, and the process repeated. At the completion of percolation testing, the test pits were backfilled with excavated material.

Results of the falling head tests are presented in the attachments to this report. The data, which are presented in tabular format, indicate poor percolation rates within the soil strata at the site. The test results are summarized in the table below.

SUMMARY OF PERCOLATION TEST RESULTS

Test No.	Depth (feet)	Percolation (min/inch)	Absorption Rate (gal/ft2/day)	Soil Type
P-1	10	20	1.1	Loose to Med. Dense Silty Sand (SM)
P-2	8	12	1.4	Med. Dense Silty Sand (SM)
P-3	10	4	2.5	Dense Silty Sand (SM)

Based upon the data collected during this investigation, and from a geotechnical engineering standpoint, it is our opinion that the site is **feasible and suitable for the proposed on-site septic system**. The proposed buildings may utilize on-site shallow leach fields to be designed by the project civil engineer provided that the recommendations presented herein are incorporated in the design and construction of the project.

The subsurface soils appear to be suitable to support an on-site sewage disposal system. This design percolation rate should be verified within the proposed sewage disposal field of each building, prior to the installation of each septic system. Based on our observed percolation rates, we recommend a septic system utilizing a shallow leach field with depths of 4 to 5 feet bgs. A minimum factor of safety should be utilized. It is further recommended that a representative of our firm be present during the installation of the system to confirm soil conditions below the bottom of the system.

A separation of 150 feet shall be incorporated between water wells and the disposal field area. A 100 percent expansion disposal field should be set aside in case the primary system cannot absorb all the sewage in the future. The wastewater stub-out from the structure should be as shallow as practical. Code requires the septic tank be at least 5 feet from the structure. If the required grade cannot be maintained, the installation contractor shall install an effluent sump pump. All specifications, dimensions, and clearances not specifically shall conform with the Uniform Plumbing Code unless superseded by Mariposa County Standards. Storm and irrigation water should be directed away from the disposal field area.

If the field conditions deviate from our test results, the system's performance could be influenced. The system's performance may also be influenced by personal hygiene, meal preparation, etc. The system is not designed to accommodate high water demand items, such as hot tubs or swimming pools. Positive grade should be established around the disposal field area. Mounding of storm water within the disposal field area may damage the disposal field and make the septic system non-operative. The system is not designed to accommodate storm water runoff. The life span of the design system may be substantially reduced if subjected to excessive sewage flows. It is warranted that additional soil absorption area will be necessary if the variables are significantly different from those assumed by the design engineer.

It is recommended that the Design Engineer be present during the installation of the septic system. The inspection will verify that the septic system is installed in accordance with design criteria. Our office should be contacted at least 2 days prior to the construction of the trenches/pits. Supplemental recommendations may be made at the time of the inspection to ensure the designed system will adequately reflect the actual soils encountered. The owner should be aware that he will be responsible for payment of the inspection fees during the installation of the sewage disposal field.

During our investigation, groundwater was not encountered. Leach fields, if utilized, should be placed a minimum of 100 feet from the any existing well. Leach fields are not designed to accommodate storm water. Leach fields should be designed so surface water is directed away from the disposal area. The leach field should be graded so water does not pond on top of the disposal field. In addition, the Mariposa County Development Standards indicate no sewage or sewage effluent may be discharged within 100 feet horizontally of any water source or the high water mark of a river, stream, canal, lake, or other surface body of water. Sewage disposal systems shall be located as far as practical from a non-classified stream or its established easement and in no case closer that 25 feet thereto unless certified by a qualified engineer that it is safe to do so without creating a nuisance or endangering the water shed. The design of the proposed septic system is beyond the scope of our services. Detailed design of the septic system may be provided upon request.

10.0 PLAN REVIEW, CONSTRUCTION OBSERVATIONS AND TESTING

We recommend that a review of plans and specifications with regard to foundations, and earthwork be completed by SALEM Engineering Group, Inc. (SALEM) prior to construction bidding. SALEM should be present at the site during site preparation to observe site clearing, preparation of exposed surfaces after clearing, and placement, treatment and compaction of fill material.

SALEM's observations should be supplemented with periodic compaction tests to establish substantial conformance with these recommendations. Moisture content of the building pad (footings and slab subgrade) should be tested immediately prior to concrete placement.

SALEM should observe foundation excavations prior to placement of reinforcing steel or concrete to assess whether the actual bearing conditions are compatible with the conditions anticipated during the preparation of this report. SALEM should also observe placement of foundation and slab concrete.

11.0 CHANGED CONDITIONS

The analyses and recommendations submitted in this report are based upon the data obtained from the test pits excavated at the approximate locations shown on the Site Plan, Figure 2.

The report does not reflect variations which may occur between pits. The nature and extent of such variations may not become evident until construction is initiated. If variations then appear, a re-evaluation of the recommendations of this report will be necessary after performing on-site observations during the excavation period and noting the characteristics of such variations.

The findings and recommendations presented in this report are valid as of the present and for the proposed construction. If site conditions change due to natural processes or human intervention on the property or adjacent to the site, or changes occur in the nature or design of the project, or if there is a substantial time lapse between the submission of this report and the start of the work at the site, the conclusions and recommendations contained in our report will not be considered valid unless the changes are reviewed by SALEM and the conclusions of our report are modified or verified in writing.

The validity of the recommendations contained in this report is also dependent upon an adequate testing and observations program during the construction phase. Our firm assumes no responsibility for construction compliance with the design concepts or recommendations unless we have been retained to perform the on-site testing and review during construction.

SALEM has prepared this report for the exclusive use of the owner and project design consultants. The report has been prepared in accordance with generally accepted geotechnical engineering practices in the area. No other warranties, either expressed or implied, are made as to the professional advice provided under the terms of our agreement and included in this report.

If you have any questions, or if we may be of further assistance, please do not hesitate to contact our office at (559) 271-9700.

Respectfully submitted,

SALEM Engineering Group, Inc.

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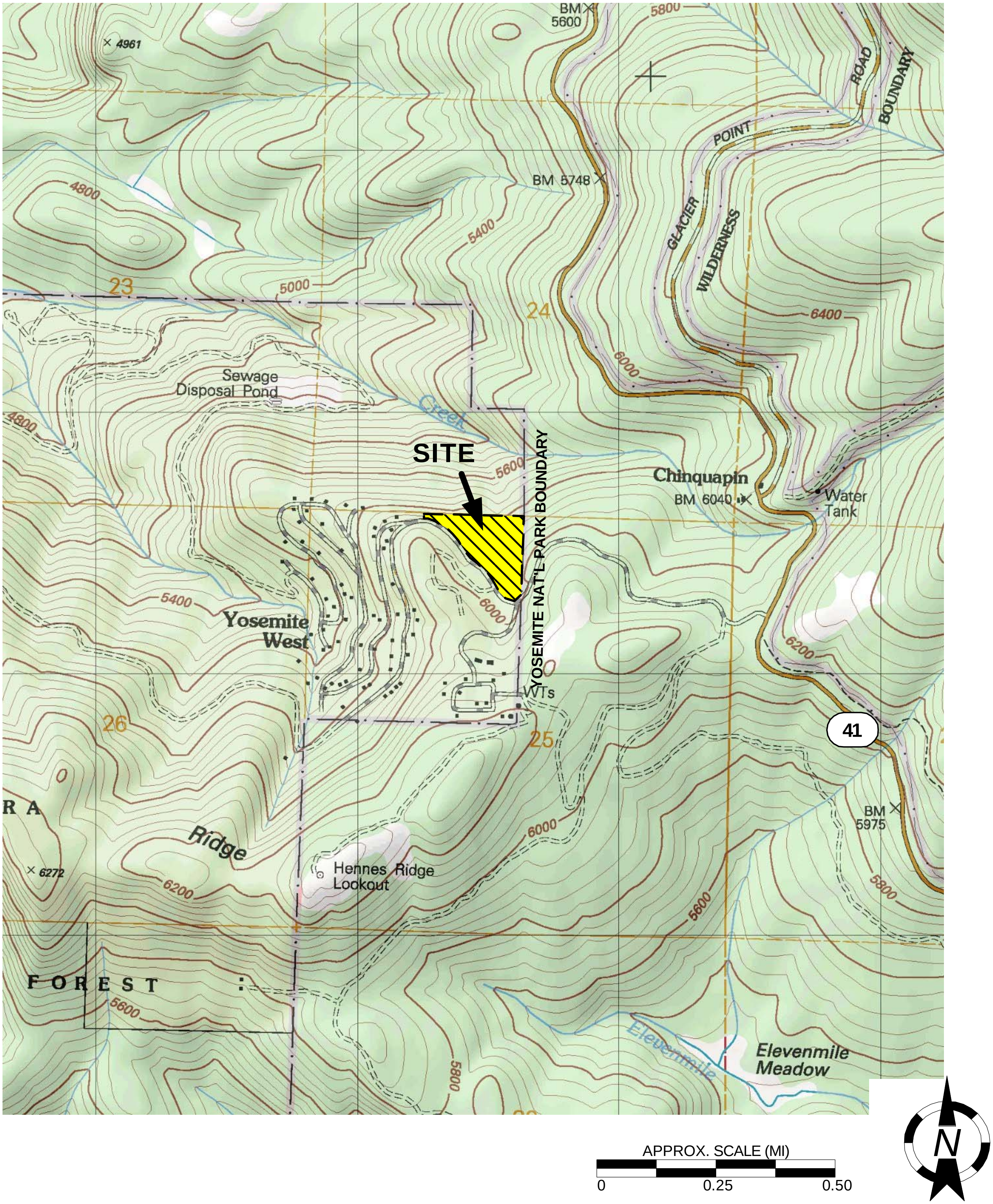
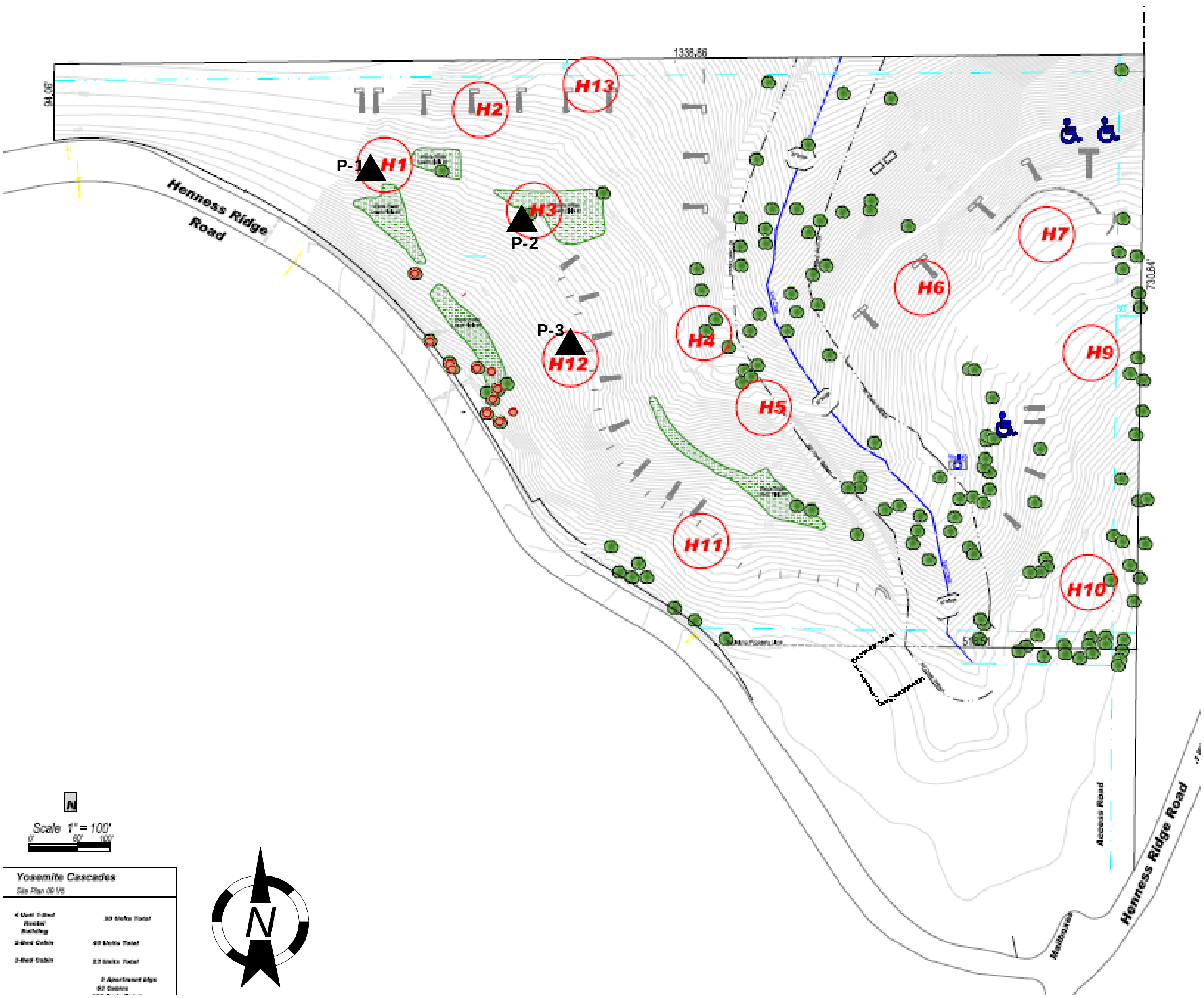


SITE LOCATION MAP GEOTECHNICAL ENGINEERING INVESTIGATION PROPOSED YOSEMITE CASCADES DEVELOPMENT YOSEMITE WEST, CALIFORNIA	SCALE: AS SHOWN	DATE: 9/29/09	SALEM Engineering Group, Inc.
	DRAWN BY: BEM	APPROVED BY: BEM	
	PROJECT NO. 1-209-0461	FIGURE NO. **1**	



SITE PLAN

GEOTECHNICAL ENGINEERING INVESTIGATION
PROPOSED YOSEMITE CASCADES DEVELOPMENT
YOSEMITE WEST, CALIFORNIA

SCALE: As Shown	DATE: 9/29/09
DRAWN BY: BEM	APPROVED BY: BEM
PROJECT NO. 1-209-0461	FIGURE NO. 2

LEGEND:

H12 Test Pit Location

P-1 Perc Test Location

(All Locations Approximate)

SALEM
Engineering Group, Inc.

APPENDIX "A"

APPENDIX A

FIELD AND LABORATORY INVESTIGATIONS

1.0 FIELD INVESTIGATION: The field investigation consisted of a surface reconnaissance and a subsurface exploratory program. Exploratory pits were advanced at the site. The boring locations are shown on the attached site plan.

The soils encountered were logged in the field during the exploration and with supplementary laboratory test data are described in accordance with the Unified Soil Classification System.

Penetration and/or Resistance tests were performed at selected depths. These tests represent the resistance to driving a 2-and/or3-inch outside diameter core barrel, respectively, 18 inches into the soil. The N-Value obtained from the Standard Penetration Test (SPT) and/or driving the Modified California Sampler (MCS) was recorded based on the number of blows required to penetrate the last 12 inches. The driving energy was provided by a hammer weighing 140 pounds, falling 30 inches. Relatively undisturbed soil samples were obtained while performing this test. Bag samples of the disturbed soil were obtained from the auger cuttings. All samples were returned to our Fresno laboratory for evaluation.

2.0 LABORATORY INVESTIGATION: The laboratory investigation was programmed to determine the physical and mechanical properties of the foundation soil underlying the site. Test results were used as criteria for determining the engineering suitability of the surface and subsurface materials encountered.

In situ moisture content, dry density, consolidation, direct shear, and sieve analysis tests were determined for the undisturbed samples representative of the subsurface material. These tests, supplemented by visual observation, comprised the basis for our evaluation of the site material.

The logs of the exploratory pits and laboratory determinations are presented in this Appendix.

Unified Soil Classification System

Major Divisions			Letter	Symbol	Description
Coarse-grained Soils More than ½ retained on the No. 200 Sieve	**Gravels** More than ½ coarse fraction retained on the No. 4 sieve	Clean Gravels	GW		Well-graded gravels and gravel-sand mixtures, little or no fines.
			GP		Poorly-graded gravels and gravel-sand mixtures, little or no fines.
		Gravels With Fines	GM		Silty gravels, gravel-sand-silt mixtures.
			GC		Clayey gravels, gravel-sand-clay mixtures.
	Sands More than ½ passing through the No. 200 sieve	Clean Sands	SW		Well-graded sands and gravelly sands, little or no fines.
			SP		Poorly-graded sands and gravelly sands, little or no fines.
		Sands With Fines	SM		Silty sands, sand-silt mixtures
			SC		Clayey sands, sandy-clay mixtures.
Coarse-grained Soils More than ½ passing through the No. 200 Sieve	**Silts and Clays** Liquid Limit less than 50%		ML		Inorganic silts, very fine sands, rock flour, silty or clayey fine sands.
			CL		Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
			OL		Organic clays of medium to high plasticity.
	Silts and Clays Liquid Limit greater than 50%		MH		Inorganic silts, micaceous or diatomaceous fines sands or silts, elastic silts.
			CH		Inorganic clays of high plasticity, fat clays.
			OH		Organic clays of medium to high plasticity.
Highly Organic Soils			PT		Peat, muck, and other highly organic soils.

Consistency Classification

Granular Soils			***Cohesive Soils***		
Description - Blows Per Foot (Corrected)			Description - Blows Per Foot (Corrected)		
	MCS	SPT		MCS	SPT
Very loose	<5	<4	Very soft	<3	<2
Loose	5 – 15	4 – 10	Soft	3 – 5	2 – 4
Medium dense	16 – 40	11 – 30	Firm	6 – 10	5 – 8
Dense	41 – 65	31 – 50	Stiff	11 – 20	9 – 15
Very dense	>65	>50	Very Stiff	21 – 40	16 – 30
			Hard	>40	>30

MCS = Modified California Sampler SPT = Standard Penetration Test Sampler

Project: Proposed Yosemite Cascades

Client: Mr. Jeff Hornacek

Location: Yosemite National Park, CA

Depth to Water> **Initial:** None **At Completion:** None

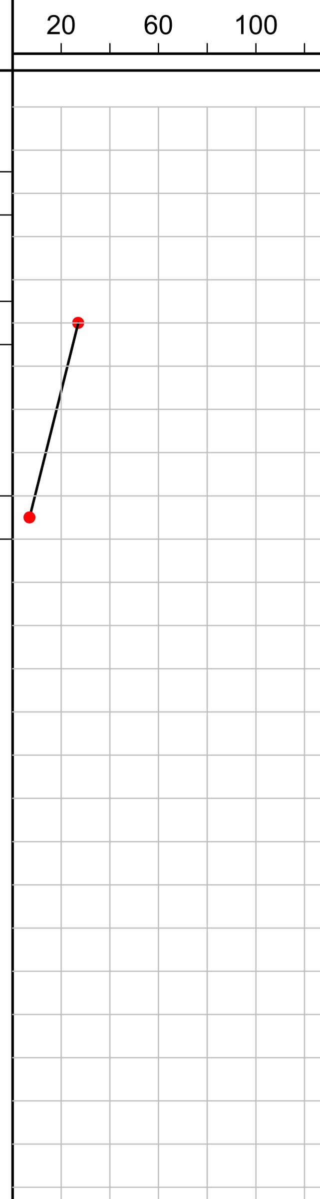
Test Pit No. H-1(P-1)

Project No: 1-209-0461

Figure No.: A-1

Logged By: A.R.

Depth (ft)	Description	Dry Density (pcf)	Moisture Content (%)	Sample Type	Penetration	Blow Count
0	Ground Surface					
0–5	**Silty Sand (SM) [Residual Soil and Colluvium]** Medium dense; dry; light red-tan; medium to fine-grained; abundant roots to 1" diameter; up to 25% granitic blocks to 8" diameter.	N/A	1.6	Bulk		
5–10	**Silty Sand (SM) [Extremely to Highly Weathered Granitic Bedrock]** Medium dense; damp; light tan; coarse to fine-grained; roots in upper portion; massive structure. Grades loose; moist to wet; finer-grained.	82.7	5.5	DS		27
		81.7	15.6	DS		7
10	Bottom of Pit					



Water Level

Excavation Method: Backhoe

Equipment: Drive Sampler

Operator: A.R.

SALEM
Engineering Group, Inc.

Excavation Date: 9.10.09

Pit Size: 3'W x 10'L x 10'D

Sheet: 1 of 1

Project: Proposed Yosemite Cascades

Client: Mr. Jeff Hornacek

Location: Yosemite National Park, CA

Depth to Water>

Test Pit No. H-2

Initial: None

Project No: 1-209-0461

Figure No.: A-2

Logged By: A.R.

At Completion: None

Depth (ft)	Symbol	Description (SUBSURFACE PROFILE)	Dry Density (pcf)	Moisture Content (%)	Sample Type	Penetration	Blow Count	Penetration Test (20, 60, 100)	Water Level
		Ground Surface							
0		**Silty Sand (SM) [Residual Soil]** Medium dense; dry; light brown; medium to fine-grained; abundant roots to 1" diameter; up to 30% granitic matrix blocks.							
5		**Silty Sand (SM) [Highly to Moderately Weathered Granitic Bedrock]** Medium dense to very dense; dry; light brown to tan; coarse to fine-grained; blocky structure - matrix blocks to 12" diameter.	N/A	1.8	Bulk				
10		Backhoe Refusal							
15									
20									
25									

Excavation Method: Backhoe

Equipment: Drive Sampler

Operator: A.R.

SALEM
Engineering Group, Inc.

Excavation Date: 9.10.09

Pit Size: 3'W x 10'L x 4'D

Sheet: 1 of 1

Project: Proposed Yosemite Cascades

Client: Mr. Jeff Hornacek

Location: Yosemite National Park, CA

Depth to Water> **Initial:** None **At Completion:** None

Test Pit No. H-3 (P-2)

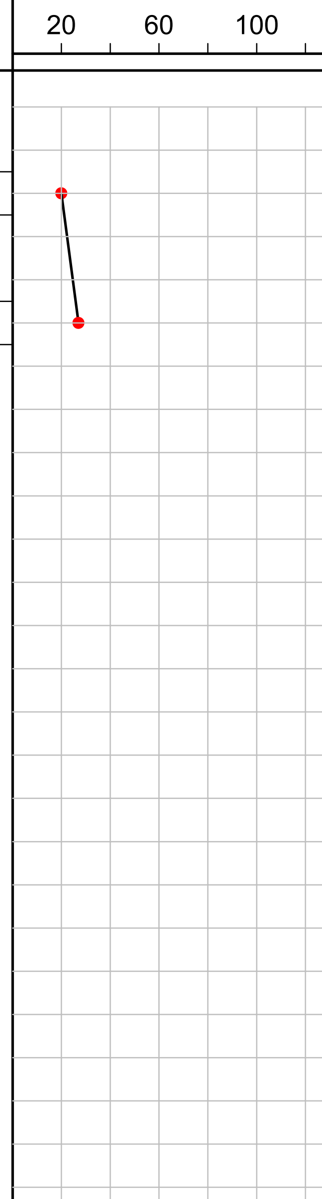
Project No: 1-209-0461

Figure No.: A-3

Logged By: A.R.

Depth (ft)	Description	Dry Density (pcf)	Moisture Content (%)	Sample Type	Blow Count
0	Ground Surface				
0–3	**Silty Sand (SM) [Residual Soil]** Medium dense; damp; light red-tan; medium to fine-grained; abundant roots to ¾" diameter.	85.8	6.2	DS	20
3–8	**Silty Sand (SM) [Extremely to Highly Weathered Granitic Bedrock]** Medium dense; moist; light tan; coarse to fine-grained; massive structure.	89.0	9.7	DS	27
8	Bottom of Pit				

Penetration Test



Excavation Method: Backhoe

Equipment: Drive Sampler

Operator: A.R.

SALEM
Engineering Group, Inc.

Excavation Date: 9.10.09

Pit Size: 3'W x 10'L x 8'D

Sheet: 1 of 1

Test Pit No. H-4

Project: Proposed Yosemite Cascades
Client: Mr. Jeff Hornacek
Location: Yosemite National Park, CA
Depth to Water>
Initial: None

Project No: 1-209-0461
Figure No.: A-4
Logged By: A.R.
At Completion: None

SUBSURFACE PROFILE			SAMPLE						
Depth (ft)	Symbol	Description	Dry Density (pcf)	Moisture Content (%)	Sample Type	Penetration	Blow Count	Penetration Test (20, 60, 100)	Water Level
		Ground Surface							
0		**Silty Sand (SM) [Residual Soil]** Medium dense; dry; light red-tan; medium to fine-grained; abundant roots to 1½" diameter; massive to blocky structure.							
5		**Silty Sand (SM) [Highly to Moderately Weathered Granitic Bedrock]** Dense to very dense; damp; light tan; coarse to fine-grained; blocky structure.							
		Backhoe Refusal							
10									
15									
20									
25									

Excavation Method: Backhoe
Equipment: Drive Sampler
Operator: A.R.

SALEM
Engineering Group, Inc.

Excavation Date: 9.10.09
Pit Size: 3'W x 10'L x 6'D
Sheet: 1 of 1

Project: Proposed Yosemite Cascades

Client: Mr. Jeff Hornacek

Location: Yosemite National Park, CA

Depth to Water>

Test Pit No. H-5

Initial: None

Project No: 1-209-0461

Figure No.: A-5

Logged By: A.R.

At Completion: None

SUBSURFACE PROFILE			SAMPLE						
Depth (ft)	Symbol	Description	Dry Density (pcf)	Moisture Content (%)	Sample Type	Penetration	Blow Count	Penetration Test (20 60 100)	Water Level
		Ground Surface							
0		**Silty Sand (SM) [Residual Soil]** Medium dense; dry; tan; medium to fine-grained; abundant roots in upper 2½ feet; blocky structure with blocks to 4" diameter.							
5		**Silty Sand (SM) [Highly to Moderately Weathered Granitic Bedrock]** Dense to very dense; dry; tan to light gray; coarse to fine-grained; massive to tabular structure.	N/A	1.1	Bulk				
		Backhoe Refusal							
10									
15									
20									
25									

Excavation Method: Backhoe

Equipment: Drive Sampler

Operator: A.R.

SALEM
Engineering Group, Inc.

Excavation Date: 9.10.09

Pit Size: 3'W x 10'L x 6'D

Sheet: 1 of 1

Project: Proposed Yosemite Cascades

Client: Mr. Jeff Hornacek

Location: Yosemite National Park, CA

Depth to Water> **Initial:** None **At Completion:** None

Test Pit No. H-6

Project No: 1-209-0461

Figure No.: A-6

Logged By: A.R.

SUBSURFACE PROFILE			SAMPLE						
Depth (ft)	Symbol	Description	Dry Density (pcf)	Moisture Content (%)	Sample Type	Penetration	Blow Count	Penetration Test (20, 60, 100)	Water Level
		Ground Surface							
0		**Silty Sand (SM) [Residual Soil]** Loose to medium dense; damp; light red-brown; medium to fine-grained; abundant roots to ¾" diameter; blocky structure. Grades rapidly to moderately weathered granitic bedrock.	79.1	4.9	DS		10		
5		Backhoe Refusal							
10									
15									
20									
25									

Excavation Method: Backhoe

Equipment: Drive Sampler

Operator: A.R.

SALEM
Engineering Group, Inc.

Excavation Date: 9.10.09

Pit Size: 3'W x 10'L x 2'D

Sheet: 1 of 1

Project: Proposed Yosemite Cascades

Client: Mr. Jeff Hornacek

Location: Yosemite National Park, CA

Depth to Water>

Test Pit No. H-7

Initial: None

Project No: 1-209-0461

Figure No.: A-7

Logged By: A.R.

At Completion: None

SUBSURFACE PROFILE			SAMPLE						
Depth (ft)	Symbol	Description	Dry Density (pcf)	Moisture Content (%)	Sample Type	Penetration	Blow Count	Penetration Test (20, 60, 100)	Water Level
		Ground Surface							
0		**Silty Sand (SM) [Residual Soil]** Loose to medium dense; damp; light red-brown; medium to fine-grained; abundant roots to ¾" diameter; blocky structure. Grades rapidly to moderately weathered granitic bedrock.	N/A	1.4	Bulk				
5		Backhoe Refusal							
10									
15									
20									
25									

Excavation Method: Backhoe

Equipment: Drive Sampler

Operator: A.R.

SALEM
Engineering Group, Inc.

Excavation Date: 9.10.09

Pit Size: 3'W x 10'L x 2'D

Sheet: 1 of 1

Project: Proposed Yosemite Cascades
Client: Mr. Jeff Hornacek
Location: Yosemite National Park, CA
Depth to Water>

Test Pit No. H-9

Project No: 1-209-0461
Figure No.: A-9
Logged By: A.R.

Initial: None
At Completion: None

SUBSURFACE PROFILE			SAMPLE					Penetration Test	Water Level
Depth (ft)	Symbol	Description	Dry Density (pcf)	Moisture Content (%)	Sample Type	Penetration	Blow Count	20 60 100	
		Ground Surface							
0		**Silty Sand (SM) [Residual Soil]** Medium dense; damp; med. brown; medium to fine-grained; abundant roots to 1" diameter in upper 1½ feet; blocky structure.	83.0	5.6	DS		15		
5		**Silty Sand (SM) [Extremely to Highly Weathered Granitic Bedrock]** Medium dense to dense; moist; light red-tan; coarse to fine-grained; massive to slightly blocky structure. Grades to massive structure.	N/A	10.8	Bulk		30		
10		Bottom of Pit							
15									
20									
25									

Excavation Method: Backhoe
Equipment: Drive Sampler
Operator: A.R.

SALEM
Engineering Group, Inc.

Excavation Date: 9.10.09
Pit Size: 3'W x 10'L x 10'D
Sheet: 1 of 1

Project: Proposed Yosemite Cascades

Client: Mr. Jeff Hornacek

Location: Yosemite National Park, CA

Depth to Water> **Initial:** None **At Completion:** None

Test Pit No. H-10

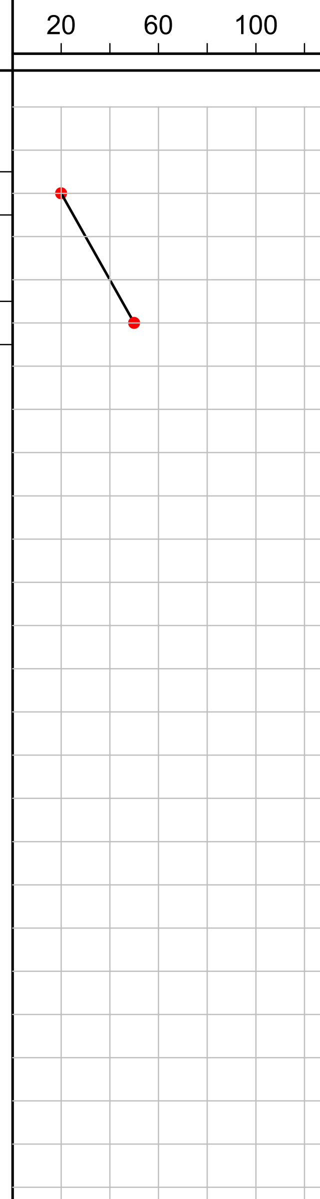
Project No: 1-209-0461

Figure No.: A-10

Logged By: A.R.

Depth (ft)	Description	Dry Density (pcf)	Moisture Content (%)	Sample Type	Blow Count
0	Ground Surface				
0–4	**Silty Sand (SM) [Residual Soil]** Medium dense; damp; light red-brown; medium to fine-grained; abundant roots to 1" diameter in upper 3 feet; massive structure.	83.1	4.8	DS	20
4–10	**Silty Sand (SM) [Extremely to Highly Weathered Granitic Bedrock]** Dense to very dense; damp; tan; coarse to fine-grained; massive structure. Grades moist; red-tan.	N/A	N/A		50
10	Bottom of Pit				

Penetration Test



Water Level

Excavation Method: Backhoe

Equipment: Drive Sampler

Operator: A.R.

SALEM
Engineering Group, Inc.

Excavation Date: 9.10.09

Pit Size: 3'W x 10'L x 10'D

Sheet: 1 of 1

Project: Proposed Yosemite Cascades

Client: Mr. Jeff Hornacek

Location: Yosemite National Park, CA

Depth to Water>

Test Pit No. H-11

Initial: None

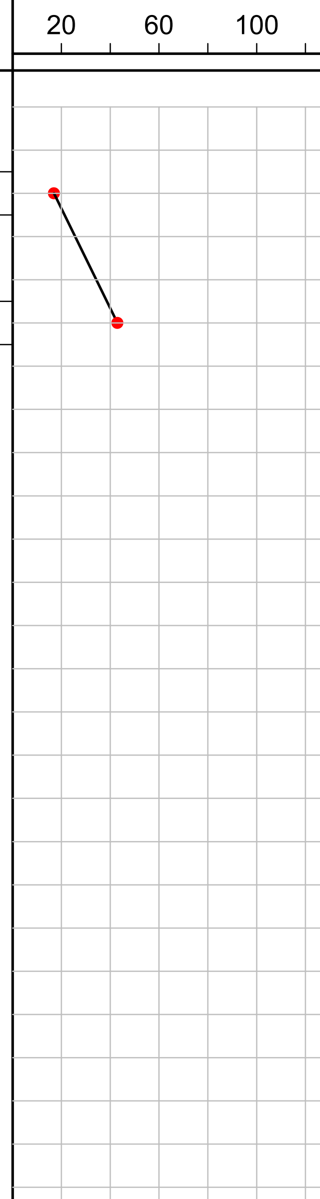
Project No: 1-209-0461

Figure No.: A-11

Logged By: A.R.

At Completion: None

SUBSURFACE PROFILE			SAMPLE						
Depth (ft)	**Symbol**	**Description**	**Dry Density (pcf)**	**Moisture Content (%)**	**Sample Type**	**Penetration**	**Blow Count**	**Penetration Test** (20, 60, 100)	**Water Level**
		Ground Surface							
0		**Silty Sand (SM) [Residual Soil]** Medium dense; damp; light red-brown; medium to fine-grained; abundant roots to 1" diameter 3 feet deep; slightly blocky structure.	N/A	5.2			17		
5		**Silty Sand (SM) [Highly to Moderately Weathered Granitic Bedrock]** Dense to very dense; damp; tan; medium to fine-grained; slightly blocky to massive structure.	N/A	4.3			43		
10		Backhoe Refusal							
15									
20									
25									



Excavation Method: Backhoe

Equipment: Drive Sampler

Operator: A.R.

SALEM
Engineering Group, Inc.

Excavation Date: 9.10.09

Pit Size: 3'W x 10'L x 6'D

Sheet: 1 of 1

Project: Proposed Yosemite Cascades

Client: Mr. Jeff Hornacek

Location: Yosemite National Park, CA

Depth to Water> **Initial:** None

Test Pit No. H-12 (P-3)

Project No: 1-209-0461

Figure No.: A-12

Logged By: A.R.

At Completion: None

SUBSURFACE PROFILE			SAMPLE						
Depth (ft)	Symbol	Description	Dry Density (pcf)	Moisture Content (%)	Sample Type	Penetration	Blow Count	Penetration Test (20, 60, 100)	Water Level
		Ground Surface							
0		**Silty Sand (SM) [Residual Soil and Colluvium]** Medium dense; dry; light yellow-tan; medium to fine-grained; slightly blocky structure; layered granitic blocks to 1½ feet diameter from a depth of 2-3 feet.							
5		**Silty Sand (SM) [Extremely to Highly Weathered Granitic Bedrock]** Dense to very dense; dry; tan; coarse to fine-grained; massive to slightly blocky structure. Grades damp; light red-tan.	86.4	3.5	Bulk				
10		Bottom of Pit							
15									
20									
25									

Excavation Method: Backhoe

Equipment: Drive Sampler

Operator: A.R.

SALEM
Engineering Group, Inc.

Excavation Date: 9.10.09

Pit Size: 3'W x 10'L x 10'D

Sheet: 1 of 1

Project: Proposed Yosemite Cascades

Client: Mr. Jeff Hornacek

Location: Yosemite National Park, CA

Depth to Water> **Initial:** None **At Completion:** None

Test Pit No. H-13

Project No: 1-209-0461

Figure No.: A-13

Logged By: A.R.

SUBSURFACE PROFILE			SAMPLE						
Depth (ft)	Symbol	Description	Dry Density (pcf)	Moisture Content (%)	Sample Type	Penetration	Blow Count	Penetration Test (20, 60, 100)	Water Level
		Ground Surface							
0		**Silty Sand (SM) [Residual Soil]** Medium dense; dry; light red-brown; medium to fine-grained; abundant roots to 1" diameter; blocky structure with up to 25% matrix blocks.							
5		**Silty Sand (SM) [Highly to Moderately Weathered Granitic Bedrock]** Dense to very dense; dry; light brown; coarse to fine-grained; blocky tabular structure.	N/A	1.6	Bulk				
		Backhoe Refusal							
10									
15									
20									
25									

Excavation Method: Backhoe

Equipment: Drive Sampler

Operator: A.R.

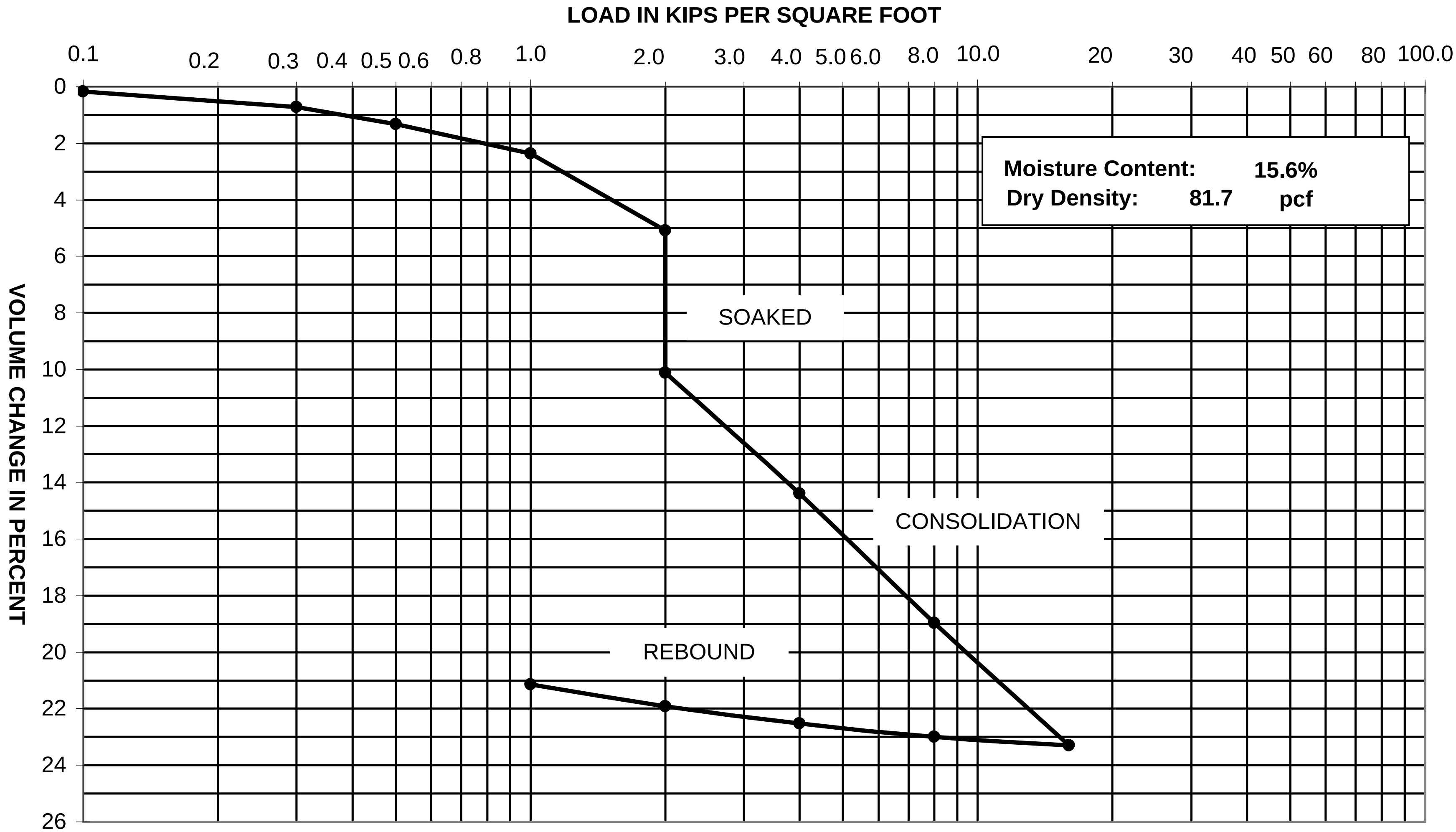
SALEM
Engineering Group, Inc.

Excavation Date: 9.10.09

Pit Size: 3'W x 10'L x 5'D

Sheet: 1 of 1

CONSOLIDATION - PRESSURE TEST DATA
ASTM D 2435



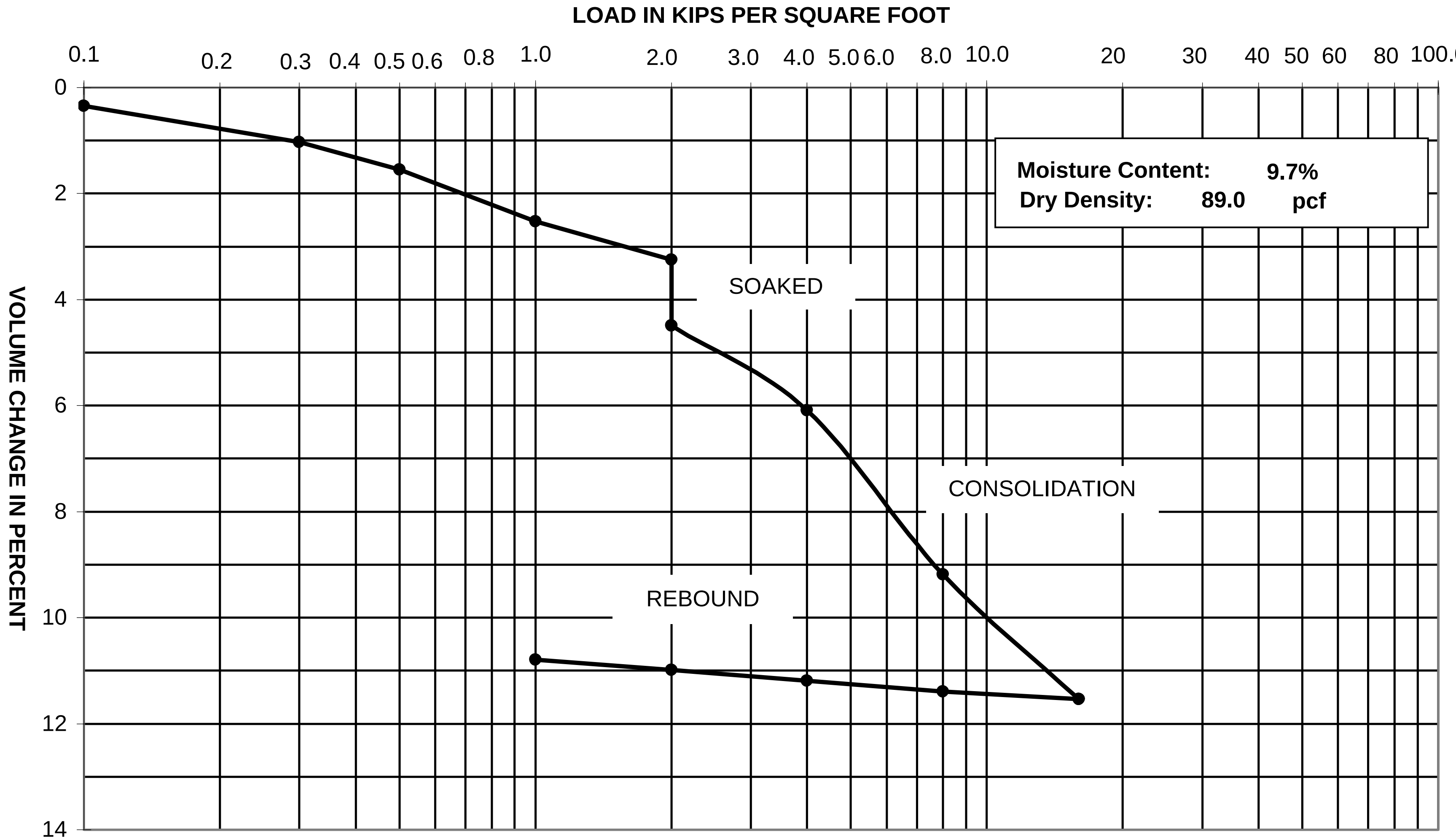
Project Name: Yosemite Cascades

Job Number: 1-209-0461

Boring: HA-1 @ 10'

SALEM Engineering Group, Inc.

CONSOLIDATION - PRESSURE TEST DATA
ASTM D 2435



Project Name: Yosemite Cascades

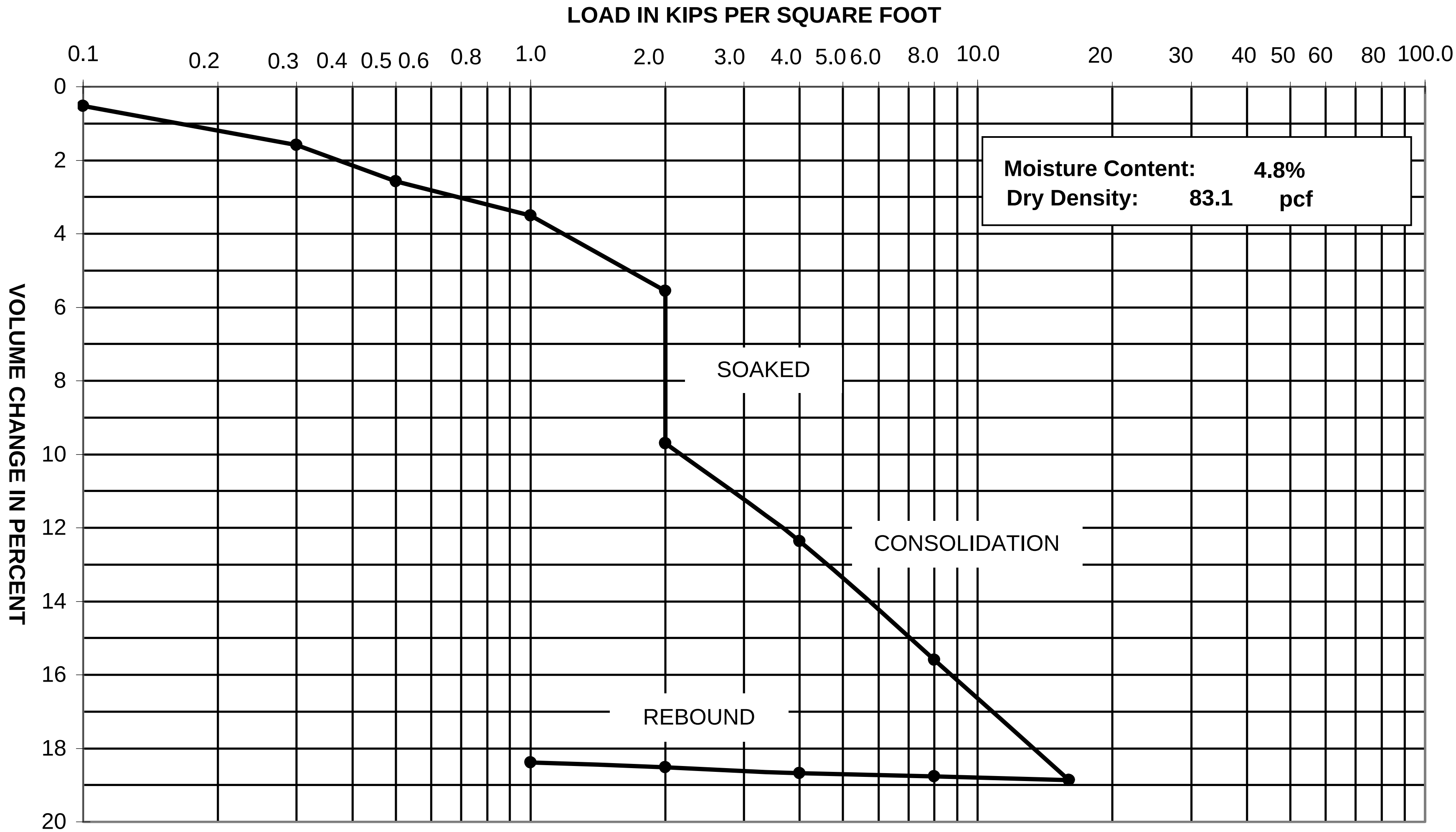
Job Number: 1-209-0461

Boring: HA-3 @ 5'

SALEM Engineering Group, Inc.

CONSOLIDATION - PRESSURE TEST DATA

ASTM D 2435



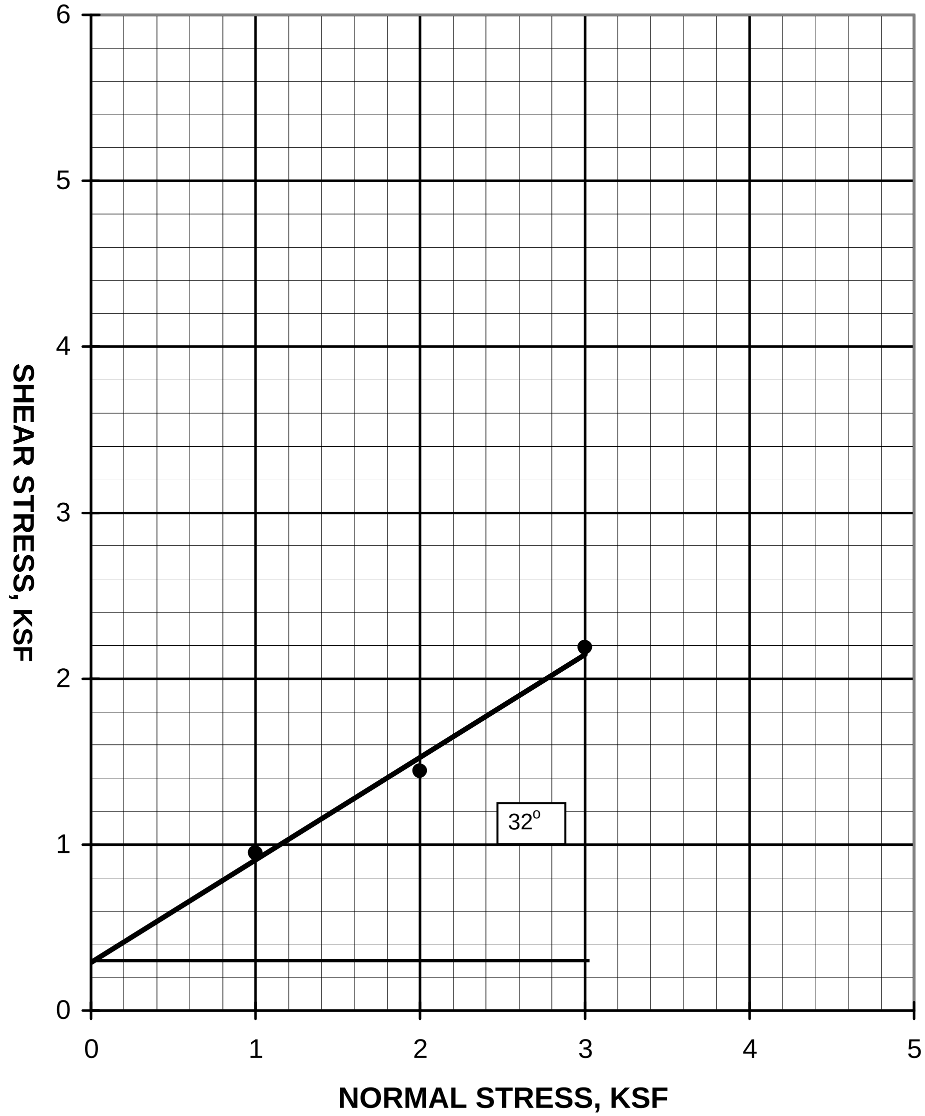
Project Name: Yosemite Cascades

Job Number: 1-209-0461

Boring: HA-10 @ 2'

SALEM Engineering Group, Inc.

SHEAR STRENGTH DIAGRAM
(DIRECT SHEAR)
ASTM 3080



Project Name: Yosemite Cascades

Job Number: 1-209-0461

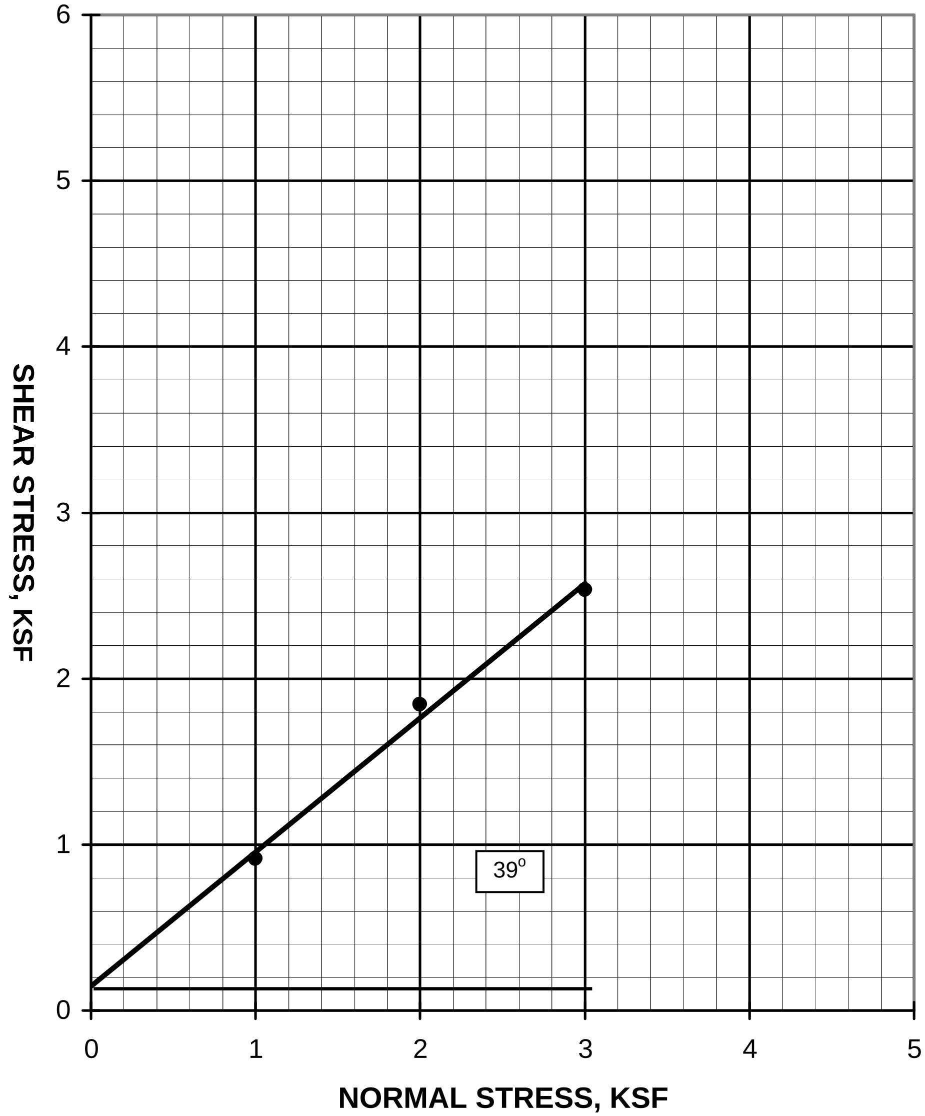
Boring: HA-1 @ 5'
Soil Type: Silty Sand (SM)

Friction Angle:	**32**	**degrees**
Cohesion:	**285**	**psf**

Moisture Content	**5.5%**	
Dry Density	**82.7**	**pcf**

SALEM Engineering Group, Inc.

SHEAR STRENGTH DIAGRAM
(DIRECT SHEAR)
ASTM 3080



Project Name: Yosemite Cascades

Job Number: 1-209-0461

Boring: HA-3 @ 2'
Soil Type: Silty Sand (SM)

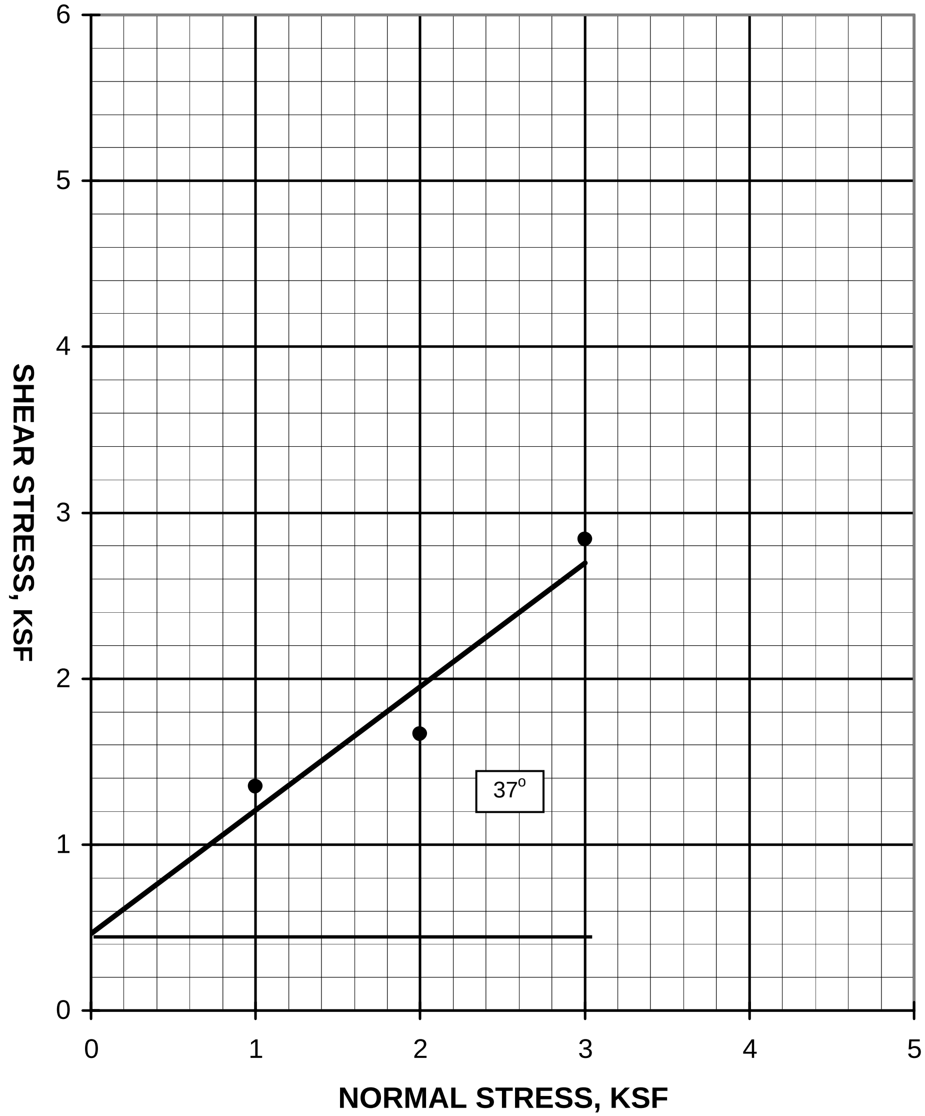
Friction Angle: 39 degrees
Cohesion: 145 psf

Moisture Content 6.2%

Dry Density 85.8 pcf

SALEM Engineering Group, Inc.

SHEAR STRENGTH DIAGRAM
(DIRECT SHEAR)
ASTM 3080



Project Name: Yosemite Cascades

Job Number: 1-209-0461

Boring: HA-12 @ 5'
Soil Type: Silty Sand (SM)

Friction Angle: 37 degrees
Cohesion: 460 psf

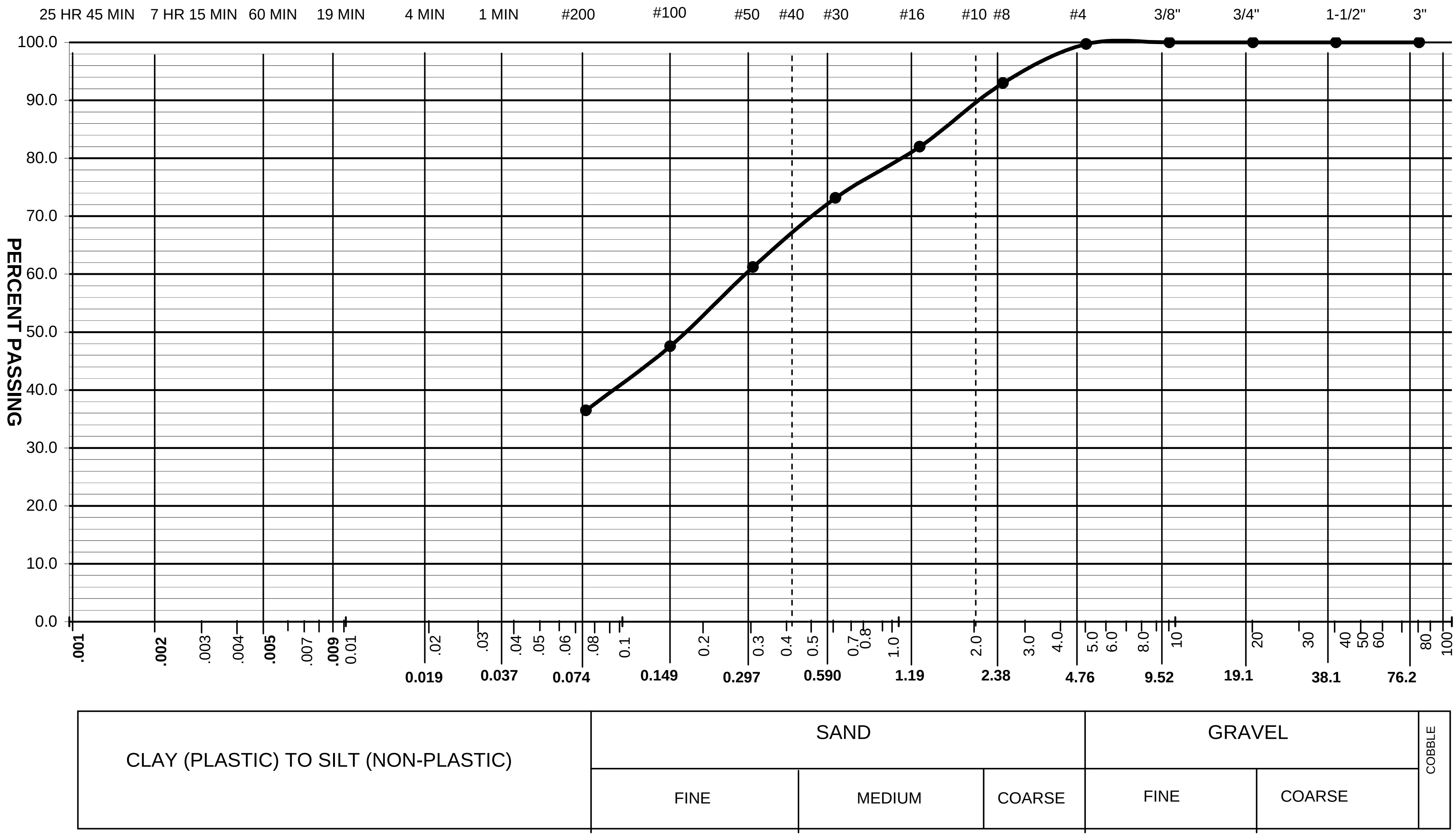
Moisture Content 3.5%

Dry Density 86.4 pcf

SALEM Engineering Group, Inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM D 421



Project Name: Yosemite Cascades

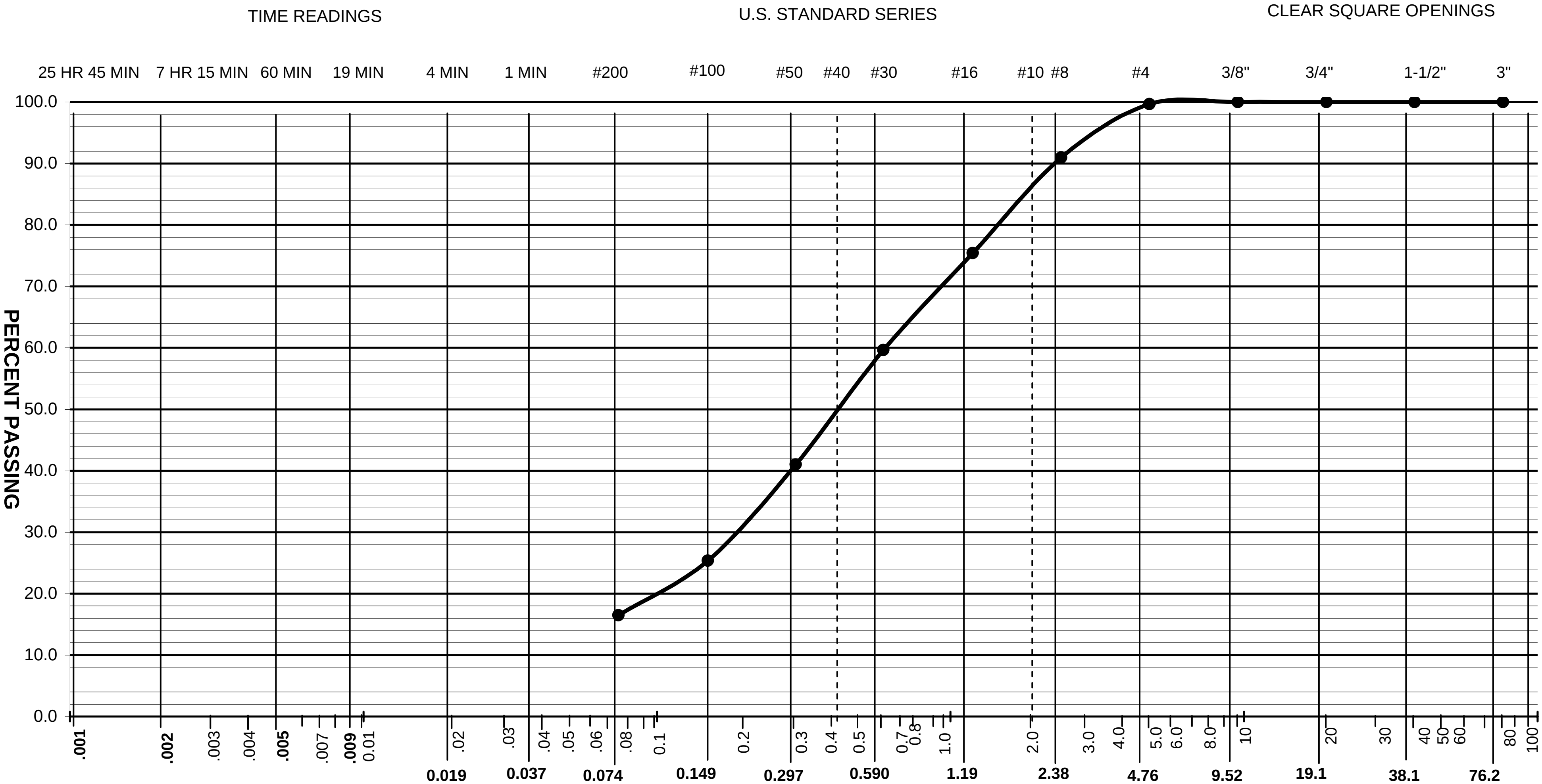
Job Number: 1-209-0461

Boring: HA-1 @ 10'

SALEM Engineering Group, Inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM D 421



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<tr><td>FINE</td><td>MEDIUM</td><td>COARSE</td><td>FINE</td><td>COARSE</td></tr>
</table>

Project Name: Yosemite Cascades

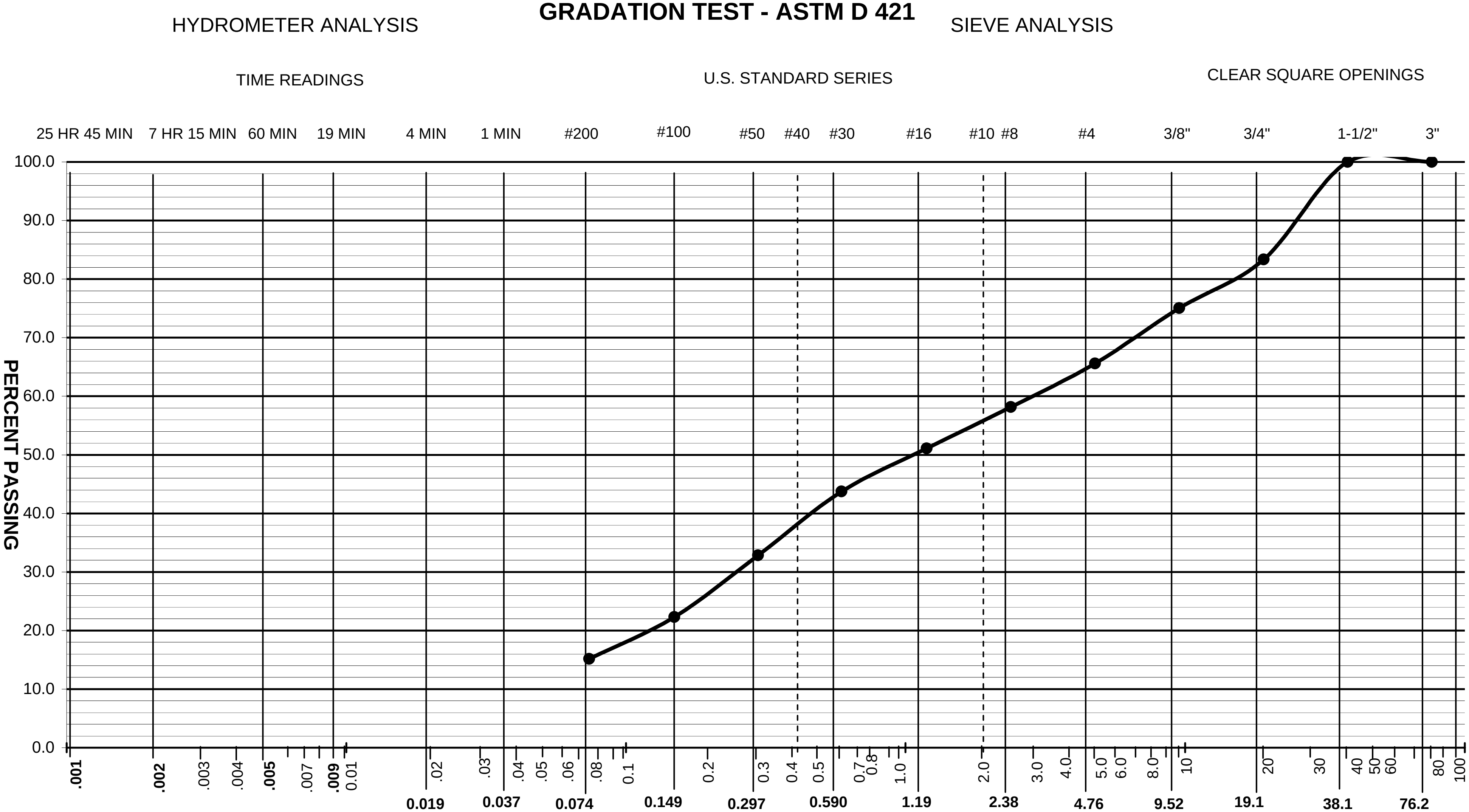
Job Number: 1-209-0461

Boring: HA-3 @ 5'

SALEM Engineering Group, Inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM D 421



CLAY (PLASTIC) TO SILT (NON-PLASTIC)	SAND			GRAVEL		COBBLE
	FINE	MEDIUM	COARSE	FINE	COARSE	

Project Name: Yosemite Cascades

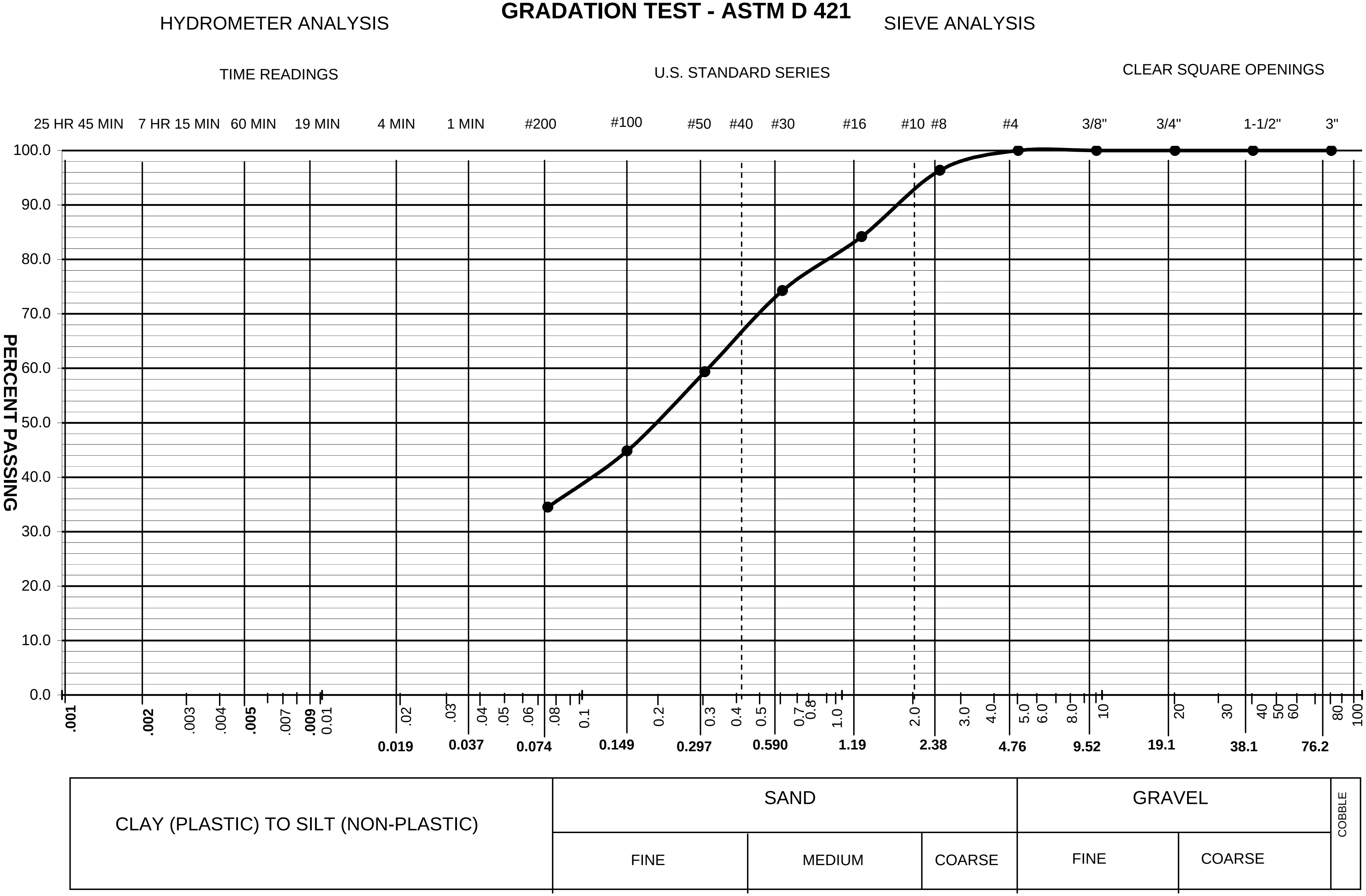
Job Number: 1-209-0461

Boring: HA-7 @ 2'

SALEM Engineering Group, Inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM D 421



Project Name: Yosemite Cascades

Job Number: 1-209-0461

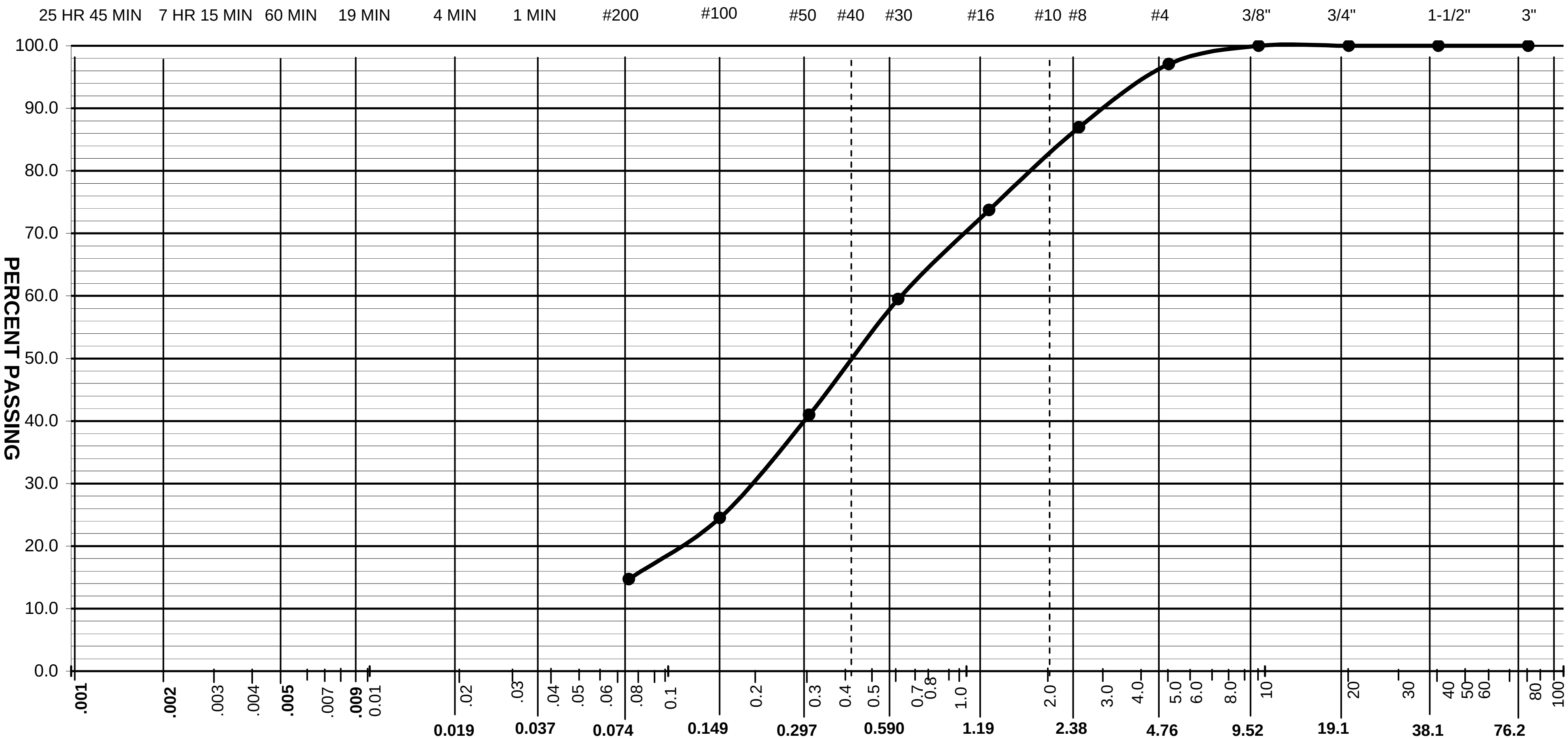
Boring: HA-9 @ 5'

SALEM Engineering Group, Inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM
GRADATION TEST - ASTM D 421

HYDROMETER ANALYSIS | SIEVE ANALYSIS

TIME READINGS | U.S. STANDARD SERIES | CLEAR SQUARE OPENINGS



CLAY (PLASTIC) TO SILT (NON-PLASTIC)	SAND			GRAVEL		COBBLE
	FINE	MEDIUM	COARSE	FINE	COARSE	

Project Name: Yosemite Cascades

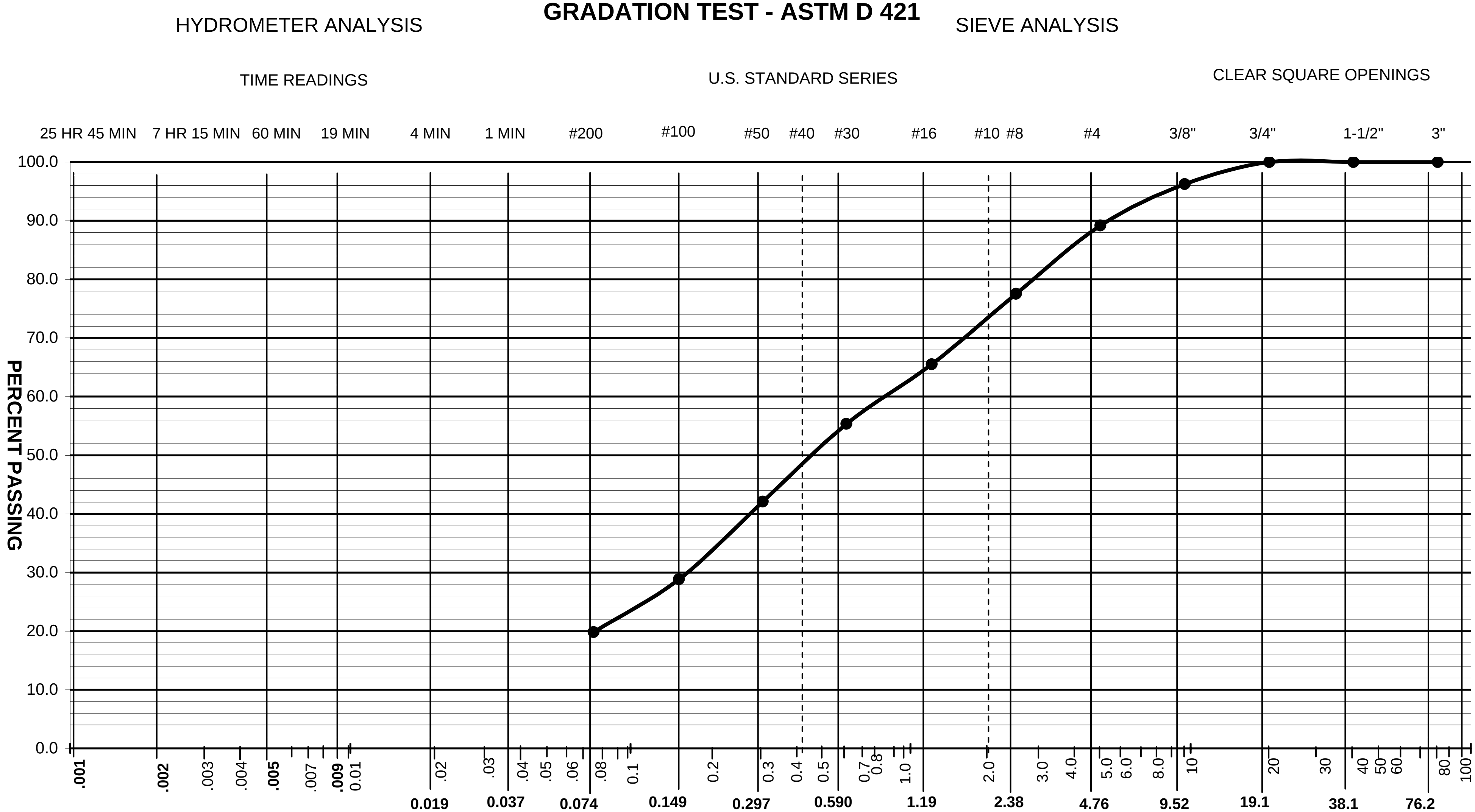
Job Number: 1-209-0461

Boring: HA-10 @ 5'

SALEM Engineering Group, Inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM D 421



CLAY (PLASTIC) TO SILT (NON-PLASTIC)	SAND			GRAVEL		COBBLE
	FINE	MEDIUM	COARSE	FINE	COARSE	

Project Name: Yosemite Cascades

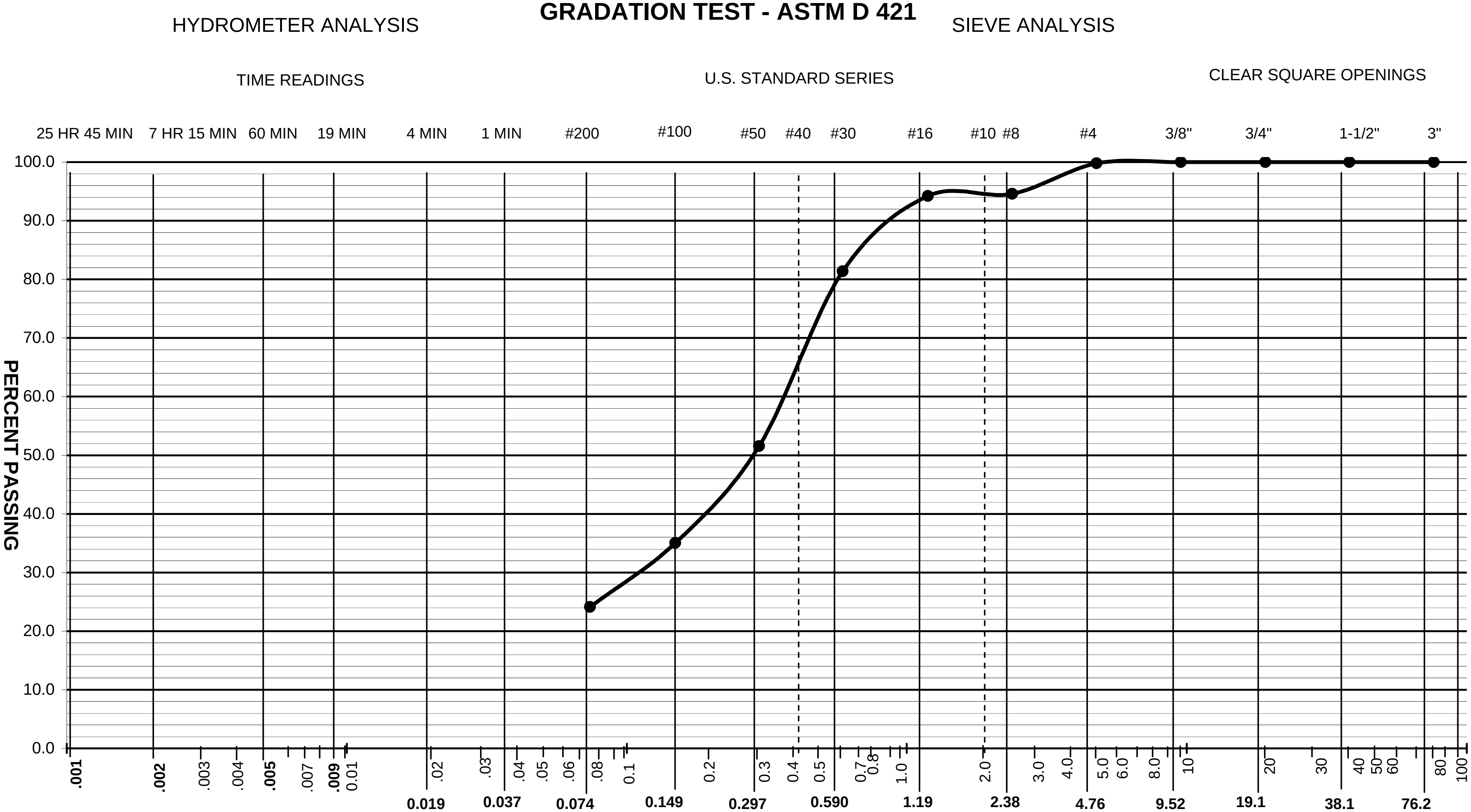
Job Number: 1-209-0461

Boring: HA-12 @ 5'

SALEM Engineering Group, Inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM D 421



CLAY (PLASTIC) TO SILT (NON-PLASTIC)	SAND			GRAVEL		COBBLE
	FINE	MEDIUM	COARSE	FINE	COARSE	

Project Name: Yosemite Cascades

Job Number: 1-209-0461

Boring: HA-3 @ 0-7'

SALEM Engineering Group, Inc.

CHEMICAL ANALYSIS

SO_4 - Modified Caltrans 417 & Cl - Modified Caltrans 417/422

Project Name: Yosemite Cascades
Job Number: 1-209-0461
Date: 9/14/2009
Soil Classification: Silty Sand (SM)

Sample Number	Sample Location	Soluble Sulfate SO_4-S		Soluble Chloride Cl		pH
1a.	HA-5 @ 5'	72	mg/Kg	5	mg/Kg	6.6
1b.	HA-5 @ 5'	65	mg/Kg	6	mg/Kg	6.6
1c.	HA-5 @ 5'	70	mg/Kg	5	mg/Kg	6.6
Average		**69**	**mg/Kg**	**5**	**mg/Kg**	**6.6**

SOIL RESTIVITY

Cal 643

Project Name: Yosemite Cascades
Job Number: 1-209-0461
Sample Location: HA-5 @ 5'

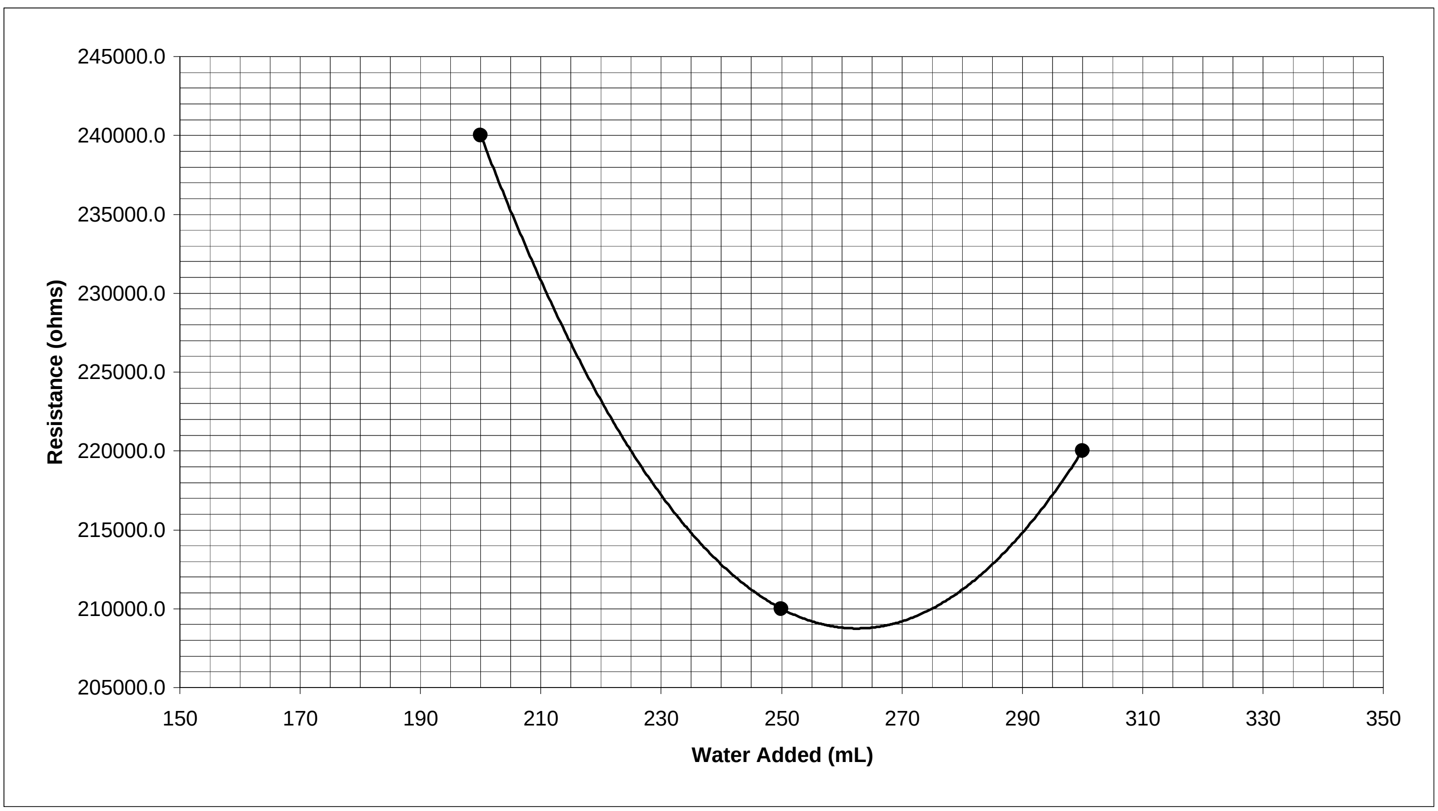
Date Tested: 9/14/2009
Tested by: SM
Sample Description: Silty Sand (SM)

Soil pH: 6.6   Sulfate Content: 69 mg/Kg   Chloride Content: 5 mg/Kg

Initial Sample Weight: 700 gms
Test Box Constant: 1.0 cm

Test Data:

Trial #	Water Added (mL)	Meter Dial Reading	Multiplier Setting	Resistance (ohms)	Resistivity (ohm cm)
1	200	2.4	100,000	240000.0	237336.0
2	250	2.1	100,000	210000.0	207669.0
3	300	2.2	100,000	220000.0	217558.0



Minimum Resistivity:	**206435** ohm cm

SALEM Engineering Group, Inc.

Percolation Test

Project: Proposed Yosemite Cascades **Job No.:** 1-208-0461

Test Hole No.: **P-3** **Date Excavated:** 9/10/2009

Depth of Test Hole (ft): 10.5 **Soil Classification:** SM

Dim. of Test Hole: 12" x 12" x 12"

Tested by: AR **Date:** 9/11/2009 **Presoaking Date:** 9/10/2009

Time Start	Time Finish	Total Elapsed Time (hrs:min)	Initial Water Level* (feet)	Final Water Level* (feet)	Δ Water Level (in.)	Δ Min	Percolation Rate (min/inch)	
FILL								
9:10	9:45	0:35	9.40	10.30	10.80	35	3	
FILL								
9:45	10:00	0:15	9.85	10.48	7.56	15	2	
FILL								
10:00	10:15	0:15	9.88	10.48	7.20	15	2	
FILL								
11:05	11:40	0:35	9.78	10.48	8.40	35	4	

APPENDIX "B"

APPENDIX B

EARTHWORK /PAVEMENT SPECIFICATIONS

When the text of the report conflicts with the general specifications in this appendix, the recommendations in the report have precedence.

1.0 **SCOPE OF WORK:** These specifications and applicable plans pertain to and include all earthwork associated with the site rough grading, including, but not limited to, the furnishing of all labor, tools and equipment necessary for site clearing and grubbing, stripping, preparation of foundation materials for receiving fill, excavation, processing, placement and compaction of fill and backfill materials to the lines and grades shown on the project grading plans and disposal of excess materials.

2.0 **PERFORMANCE:** The Contractor shall be responsible for the satisfactory completion of all earthworks in accordance with the project plans and specifications. This work shall be inspected and tested by a representative of SALEM Engineering Group, Incorporated, hereinafter referred to as the Soils Engineer and/or Testing Agency. Attainment of design grades, when achieved, shall be certified by the project Civil Engineer. Both the Soils Engineer and the Civil Engineer are the Owner's representatives. If the Contractor should fail to meet the technical or design requirements embodied in this document and on the applicable plans, he shall make the necessary adjustments until all work is deemed satisfactory as determined by both the Soils Engineer and the Civil Engineer. No deviation from these specifications shall be made except upon written approval of the Soils Engineer, Civil Engineer, or project Architect.

No earthwork shall be performed without the physical presence or approval of the Soils Engineer. The Contractor shall notify the Soils Engineer at least 2 working days prior to the commencement of any aspect of the site earthwork.

The Contractor agrees that he shall assume sole and complete responsibility for job site conditions during the course of construction of this project, including safety of all persons and property; that this requirement shall apply continuously and not be limited to normal working hours; and that the Contractor shall defend, indemnify and hold the Owner and the Engineers harmless from any and all liability, real or alleged, in connection with the performance of work on this project, except for liability arising from the sole negligence of the Owner or the Engineers.

3.0 **TECHNICAL REQUIREMENTS**: All compacted materials shall be densified to no less that 95 percent of relative compaction based on ASTM D1557 Test Method-78, UBC or CAL-216, as specified in the technical portion of the Soil Engineer's report. The location and frequency of field density tests shall be as determined by the Soils Engineer. The results of these tests and compliance with these specifications shall be the basis upon which satisfactory completion of work will be judged by the Soils Engineer.

4.0 **SOILS AND FOUNDATION CONDITIONS**: The Contractor is presumed to have visited the site and to have familiarized himself with existing site conditions and the contents of the data presented in the Geotechnical Engineering Report.

The Contractor shall make his own interpretation of the data contained in the Geotechnical Engineering Report and the Contractor shall not be relieved of liability under the Contractor for any loss sustained as a result of any variance between conditions indicated by or deduced from said report and the actual conditions encountered during the progress of the work.

5.0 DUST CONTROL: The work includes dust control as required for the alleviation or prevention of any dust nuisance on or about the site or the borrow area, or off-site if caused by the Contractor's operation either during the performance of the earthwork or resulting from the conditions in which the Contractor leaves the site. The Contractor shall assume all liability, including court costs of codefendants, for all claims related to dust or wind-blown materials attributable to his work.

Site preparation shall consist of site clearing and grubbing and preparation of foundation materials for receiving fill.

6.0 CLEARING AND GRUBBING: The Contractor shall accept the site in this present condition and shall demolish and/or remove from the area of designated project earthwork all structures, both surface and subsurface, trees, brush, roots, debris, organic matter and all other matter determined by the Soils Engineer to be deleterious. Such materials shall become the property of the Contractor and shall be removed from the site.

Tree root systems in proposed building areas should be removed to a minimum depth of 3 feet and to such an extent which would permit removal of all roots greater than 1 inch in diameter. Tree roots removed in parking areas may be limited to the upper 1½ feet of the ground surface. Backfill or tree root excavation should not be permitted until all exposed surfaces have been inspected and the Soils Engineer is present for the proper control of backfill placement and compaction. Burning in areas which are to receive fill materials shall not be permitted.

7.0 SUBGRADE PREPARATION: Surfaces to receive Engineered Fill, building or slab loads, shall be prepared as outlined above, scarified to a minimum of 6 inches, moisture-conditioned as necessary, and recompacted to 95 percent relative compaction.

Loose soil areas and/or areas of disturbed soil shall be moisture-conditioned as necessary and recompacted to 95 percent relative compaction. All ruts, hummocks, or other uneven surface features shall be removed by surface grading prior to placement of any fill materials. All areas which are to receive fill materials shall be approved by the Soils Engineer prior to the placement of any of the fill material.

8.0 EXCAVATION: All excavation shall be accomplished to the tolerance normally defined by the Civil Engineer as shown on the project grading plans. All over-excavation below the grades specified shall be backfilled at the Contractor's expense and shall be compacted in accordance with the applicable technical requirements.

9.0 FILL AND BACKFILL MATERIAL: No material shall be moved or compacted without the presence of the Soils Engineer. Material from the required site excavation may be utilized for construction site fills, provided prior approval is given by the Soils Engineer. All materials utilized for constructing site fills shall be free from vegetation or other deleterious matter as determined by the Soils Engineer.

10.0 PLACEMENT, SPREADING AND COMPACTION: The placement and spreading of approved fill materials and the processing and compaction of approved fill and native materials shall be the responsibility of the Contractor. However, compaction of fill materials by flooding, ponding, or jetting shall not be permitted unless specifically approved by local code, as well as the Soils Engineer. Both cut and fill shall be surface-compacted to the satisfaction of the Soils Engineer prior to final acceptance.

11.0 SEASONAL LIMITS: No fill material shall be placed, spread, or rolled while it is frozen or thawing, or during unfavorable wet weather conditions. When the work is interrupted by heavy rains, fill operations shall not be resumed until the Soils Engineer indicates that the moisture content and density of previously placed fill is as specified.

12.0 DEFINITIONS - The term "pavement" shall include asphaltic concrete surfacing, untreated aggregate base, and aggregate subbase. The term "subgrade" is that portion of the area on which surfacing, base, or subbase is to be placed.

The term “Standard Specifications”: hereinafter referred to is the January 1991 Standard Specifications of the State of California, Department of Transportation, and the "Materials Manual" is the Materials Manual of Testing and Control Procedures, State of California, Department of Public Works, Division of Highways. The term "relative compaction" refers to the field density expressed as a percentage of the maximum laboratory density as defined in the applicable tests outlined in the Materials Manual.

13.0 SCOPE OF WORK - This portion of the work shall include all labor, materials, tools, and equipment necessary for, and reasonably incidental to the completion of the pavement shown on the plans and as herein specified, except work specifically notes as "Work Not Included."

14.0 PREPARATION OF THE SUBGRADE - The Contractor shall prepare the surface of the various subgrades receiving subsequent pavement courses to the lines, grades, and dimensions given on the plans. The upper 12 inches of the soil subgrade beneath the pavement section shall be compacted to a minimum relative compaction of 95 percent. The finished subgrades shall be tested and approved by the Soils Engineer prior to the placement of additional pavement courses.

15.0 UNTREATED AGGREGATE BASE - The aggregate base material shall be spread and compacted on the prepared subgrade in conformity with the lines, grades, and dimensions shown on the plans. The aggregate base material shall conform to the requirements of Section 26 of the Standard Specifications for Class II material, 1½ inches maximum size. The aggregate base material shall be compacted to a minimum relative compaction of 95 percent. The aggregate base material shall be spread and compacted in accordance with Section 26 of the Standard Specifications. The aggregate base material shall be spread in layers not exceeding 6 inches and each layer of aggregate material course shall be tested and approved by the Soils Engineer prior to the placement of successive layers.

16.0 AGGREGATE SUBBASE - The aggregate subbase shall be spread and compacted on the prepared subgrade in conformity with the lines, grades, and dimensions shown on the plans. The aggregate subbase material shall conform to the requirements of Section 25 of the Standard Specifications for Class II material. The aggregate subbase material shall be compacted to a minimum relative compaction of 95 percent, and it shall be spread and compacted in accordance with Section 25 of the Standard Specifications. Each layer of aggregate subbase shall be tested and approved by the Soils Engineer prior to the placement of successive layers.

17.0 ASPHALTIC CONCRETE SURFACING - Asphaltic concrete surfacing shall consist of a mixture of mineral aggregate and paving grade asphalt, mixed at a central mixing plant and spread and compacted on a prepared base in conformity with the lines, grades, and dimensions shown on the plans. The viscosity grade of the asphalt shall be AR-4000. The mineral aggregate shall be Type B, ½ inch maximum size, medium grading, and shall conform to the requirements set forth in Section 39 of the Standard Specifications. The drying, proportioning, and mixing of the materials shall conform to Section 39.

The prime coat, spreading and compacting equipment, and spreading and compacting the mixture shall conform to the applicable chapters of Section 39, with the exception that no surface course shall be placed when the atmospheric temperature is below 50 degrees F. The surfacing shall be rolled with a combination steel-wheel and pneumatic rollers, as described in Section 39-6. The surface course shall be placed with an approved self-propelled mechanical spreading and finishing machine.

18.0 **FOG SEAL COAT** - The fog seal (mixing type asphaltic emulsion) shall conform to and be applied in accordance with the requirements of Section 37.