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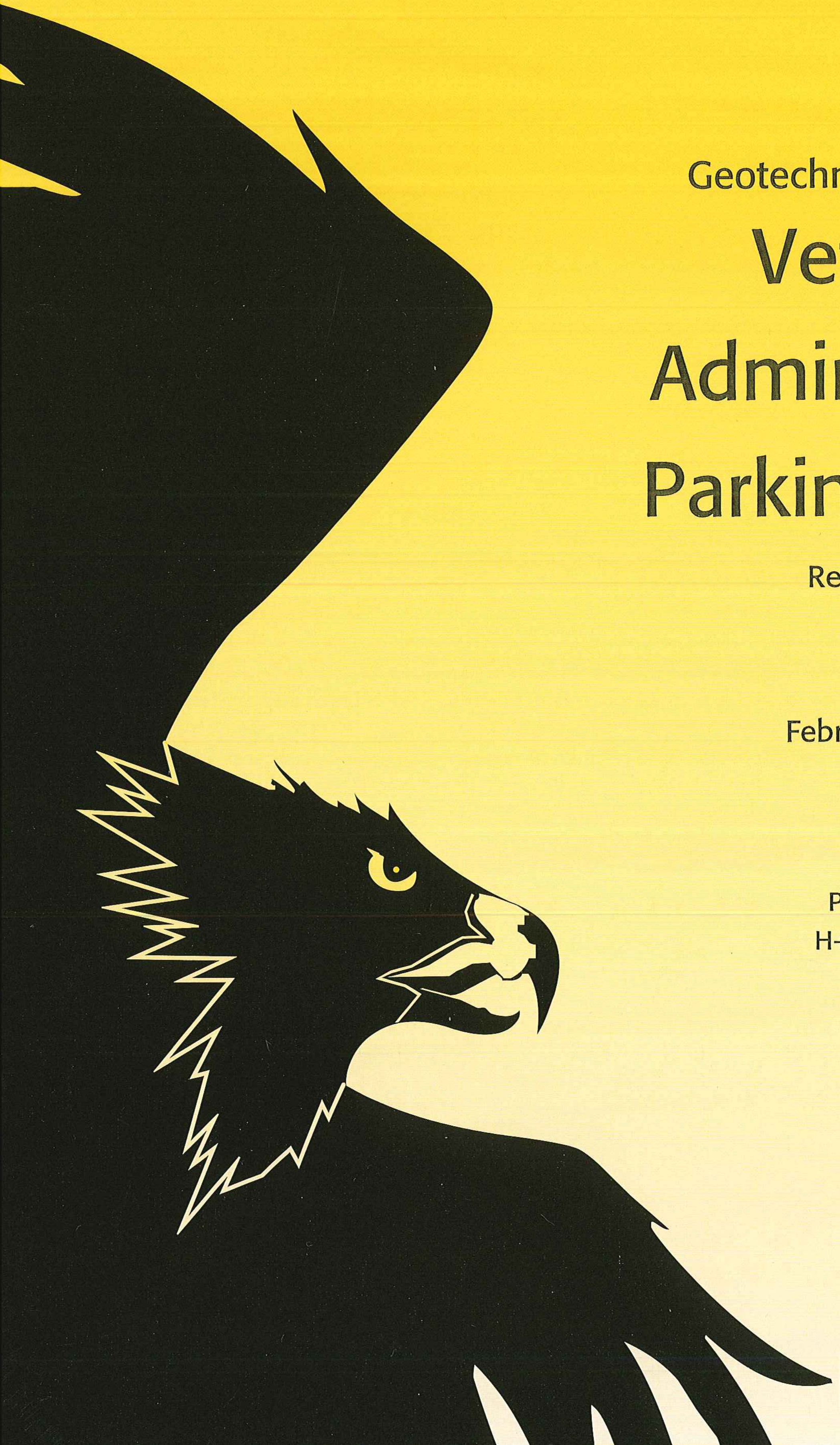

Geotechnical Investigation

Veterans Administration Parking Garage

Reno, Nevada

February 2, 2012

Prepared for
H+K Architects

Black Eagle Consulting, Inc.
Geotechnical & Construction Services

Mr. Max Hershenow, AIA
H+K Architects
5485 Reno Corporate Drive, Suite 100
Reno, Nevada 89511

February 2, 2012
Project No.: 0833-04-1

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RE: Geotechnical Investigation
Veterans Administration Parking Garage
Reno, Nevada

Dear Mr. Hershenow:

Black Eagle Consulting, Inc. is pleased to present the results of our geotechnical investigation for the above-referenced project. Our investigation consisted of research, field exploration, laboratory testing, and engineering analysis to allow formulation of geotechnical conclusions and recommendations for design and construction of this facility.

The proposed Veterans Administration (VA) Parking Garage project will involve the design and construction of a multi-story parking garage with 266 parking stalls. The structure will most likely have cast-in-place reinforced Portland cement concrete (PCC) columns and beams with post-tensioned PCC floor slabs above ground level. The ground floor will use a conventional cast-in-place reinforced PCC slab-on-grade. Foundation support will be provided by shallow reinforced PCC foundations. Any necessary utilities will be extended to the site from existing utilities in the area, and some existing utilities present within the parking garage footprint will need to be located. Associated asphalt concrete driveway areas, as well as PCC curb, gutter and sidewalk, will be constructed as a part of this project.

The site is suitable to host the proposed facility, subject to the recommendations contained in the attached report. The site is generally underlain by a surface layer of clay-rich soils and at a relatively shallow depth by coarse granular soils. Structural improvements will need to be separated from the clay-rich soils, as discussed in the **Site Preparation** section. Trenching will be extremely difficult due to the presence of abundant cobbles and large boulders as discussed in the **Trenching and Excavation** section. The existing asphalt concrete pavement present across the site is suitable for re-use as structural fill and/or recycled asphalt concrete base, as discussed in the **Mass Grading** section. Native coarse granular soils are suitable for re-use as structural fill after screening, but native surficial clay-rich soils should only be placed as fill in nonstructural areas, as described in the **Mass Grading** section.

We appreciate having the opportunity to work with you on this project. If you have any questions regarding the content of the attached report, please do not hesitate to contact us.

Sincerely,

Black Eagle Consulting, Inc.

PROFESSIONAL ENGINEER - STATE OF NEVADA
PATRICK A.
PILLING
Exp. 12-31-13
CIVIL
No. 9153

2-2-12

Patrick Pilling, Ph.D., P.E., D.GE.
President

Copies to: Addressee (4 copies and PDF via email)

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Introduction

Presented herein are the results of the Black Eagle Consulting, Inc. (BEC) geotechnical investigation, laboratory testing, and associated geotechnical design recommendations for the proposed Veterans Administration (VA) Parking Garage project to be located in Reno, Nevada. These recommendations are based on surface and subsurface conditions encountered in our explorations, and on details of the proposed project as described in this report. The objectives of this study were to:

1. **Determine general soil and ground water conditions pertaining to design and construction of the proposed VA parking garage.**

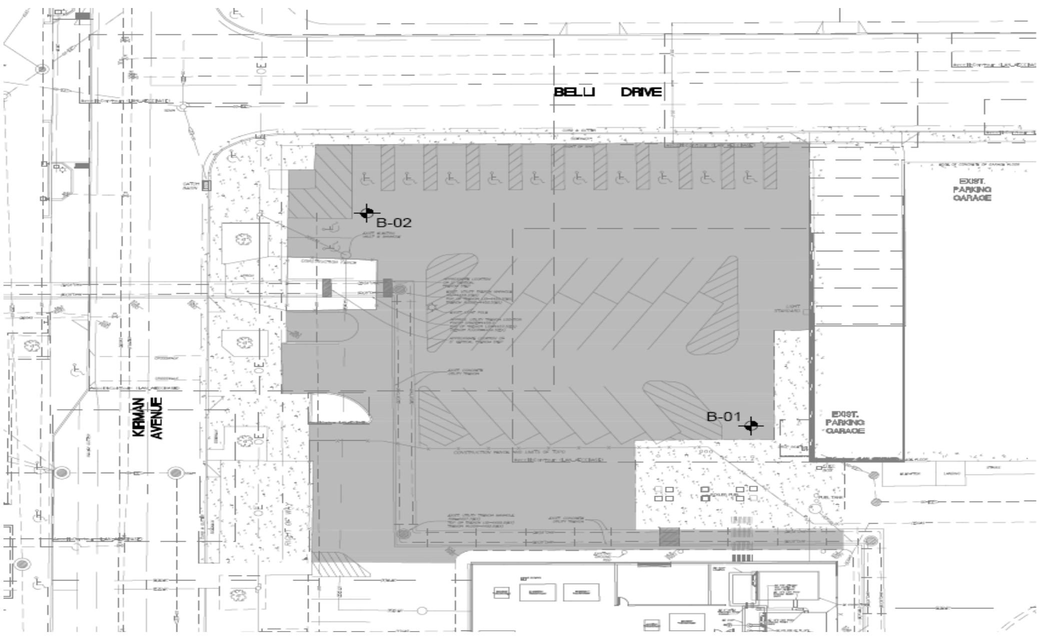
2. **Provide recommendations for design and construction of the project, as related to these geotechnical conditions.**

The area covered by this report is shown on Plate 1 (Plot Plan). Our investigation included field exploration, laboratory testing, and engineering analysis to determine the physical and mechanical properties of the various on-site materials. Results of our field exploration and testing programs are included in this report and form the basis for all conclusions and recommendations.

The services described above were conducted in accordance with the BEC Professional Geotechnical Agreement dated November 4, 2011, which was signed by Max Hershenow of H+K Architects.

Project Description

The proposed parking garage site consists of a square parcel of approximately 0.3 acres located in Reno, Nevada. The site is entirely contained in Section 13, Township 19, Range 19 East, Mount Diablo Meridian. The parcel is bordered to the north by Belli Drive; to the east by an existing, multi-story parking garage; to the south by existing hospital facilities; and to the west by Kirman Avenue. The site is presently an asphalt concrete paved parking lot. Access to the site is obtained from Kirman Avenue or Belli Drive.



Project Location

Structure Information

The proposed VA Parking Garage project will involve the design and construction of a multi-story parking garage with 266 parking stalls. The structure will most likely have cast-in-place reinforced Portland cement concrete (PCC) columns and beams with post-tensioned PCC floor slabs above ground level. The ground floor will use a conventional cast-in-place reinforced PCC slab-on-grade. Foundation support will be provided by shallow reinforced PCC foundations. Any necessary utilities will be extended to the site from existing utilities in the area, and some existing utilities present within the parking garage footprint will need to be located. Associated asphalt concrete driveway areas, as well as PCC curb, gutter and sidewalk, will be constructed as a part of this project.

Grading Concepts

The finish floor elevation of the ground level of the parking garage will be at or near grade elevation of the existing parking lot. As a result, the footings will bear approximately 2 feet below finished grade, while the ground floor PCC slab will bear at or slightly below existing ground elevations. Surface improvements around the exterior of the parking garage will match existing grades in the parking lot. No significant cuts or fills are anticipated as a part of this project.

Existing Information

Black Eagle Consulting, Inc. previously performed a geotechnical investigation for the adjacent parking garage, as documented in their report titled *Geotechnical Investigation, Veterans Hospital Parking Garage, Reno, Nevada*, dated September 2004 (BEC, 2004). The information contained in that report was used to supplement the work performed as a part of the current investigation.

Site Conditions

Existing Structures

The site is currently an active asphalt concrete parking lot that serves the VA Hospital to the west. A storm drain line with associated drainage inlets is present in an east/west direction in the southern portion of the proposed parking garage footprint. At least one underground storage tank (UST) is present on the east side of the parcel. Several hospital support facility buildings are present immediately to the south of the proposed parking garage, and active light poles are present across the existing parking lot. An existing multi-story parking garage is present to the east of the site. Portland cement concrete curb, gutter, and sidewalk borders the northern and western margins of the site.

Proposed VA Parking Garage Site

Topography

The site is relatively flat, with a minor slope of approximately 2 percent to the southeast. Existing ground elevations vary from a high of approximately 4,559 feet in the northwest portion of the site to a low of approximately 4,556 feet in the southeast site area. Drainage is accomplished by sheet flow to the southeast and into the storm drain facilities present at the site.

Vegetation

Since the site is currently a paved parking area, it is void of vegetation.

Exploration

Drilling

Exploration Drilling

The VA Parking Garage site was explored on January 21, 2012 by drilling two test borings. The borings were advanced using 6-inch-outside-diameter (O.D.), 3-¼-inch-inside-diameter (I.D.), hollow-stem augers and a truck-mounted CME 55 soils sampling drill rig. The maximum depth of exploration was 15.9 feet below the existing ground surface. The locations of the test borings are shown on Plate 1.

The native soils were sampled in-place every 2 to 5 feet by use of a standard, 2-inch O.D., split-spoon sampler driven by a 140-pound drive hammer with a 30-inch stroke operated with a rope and cathead. The number of blows to drive the sampler the final 12 inches of an 18-inch penetration (Standard Penetration Test [SPT] - American Society for Testing and Materials [ASTM] D 1586) into undisturbed soil is an indication of the density and consistency of the material.

Due to the relatively small diameter of the samplers, the maximum particle size that could be obtained was approximately 1 inch. The final logs may not, therefore, adequately represent the actual quantity or presence of cobbles or boulders.

Material Classification

A geologist examined and identified all soils in the field in accordance with ASTM D 2488. During drilling, representative bulk samples were placed in sealed plastic bags and returned to our Reno, Nevada laboratory for testing. Additional soil classification was subsequently performed in accordance with ASTM 2487 (Unified Soil Classification System [USCS]) upon completion of laboratory testing as described in the **Laboratory Testing** section. Logs of the test borings are presented as Plate 2 (Boring Logs), and a USCS chart has been included as Plate 3 (Graphic Soils Classification Chart).

Laboratory Testing


Grain Size Analysis

All soils testing performed in the BEC soils laboratory is conducted in accordance with the standards and methodologies described in Volume 4.08 of the ASTM Standards.

Index Tests

Samples of each significant soil type were analyzed to determine their in situ moisture content (ASTM D 2216), grain size distribution (ASTM D 422), and plasticity index (ASTM D 4318). The results of these tests are shown on Plate 4 (Index Test Results). Test results were used to classify the soils according to ASTM D 2487 and to verify field logs, which were then updated as appropriate. Classification in this manner provides an indication of the soil's mechanical properties and can be correlated with standard penetration testing and published charts (Bowles, 1996; Naval Facilities Engineering Command [NAVFAC], 1986a and b) to evaluate bearing capacity, lateral earth pressures, and settlement potential.

Direct Shear Test

A direct shear test (ASTM D 3080) was performed on a representative sample of silty gravel with sand. The test was run on remolded, inundated samples under various normal loads in order to develop a Mohr's strength envelope. For remolded samples, the sample was screened to remove particles larger than the number 4 sieve prior to testing. Results of these tests are shown on Plate 5 (Direct Shear Test Results) and were used in calculation of bearing capacities, friction factors, and lateral earth pressures.

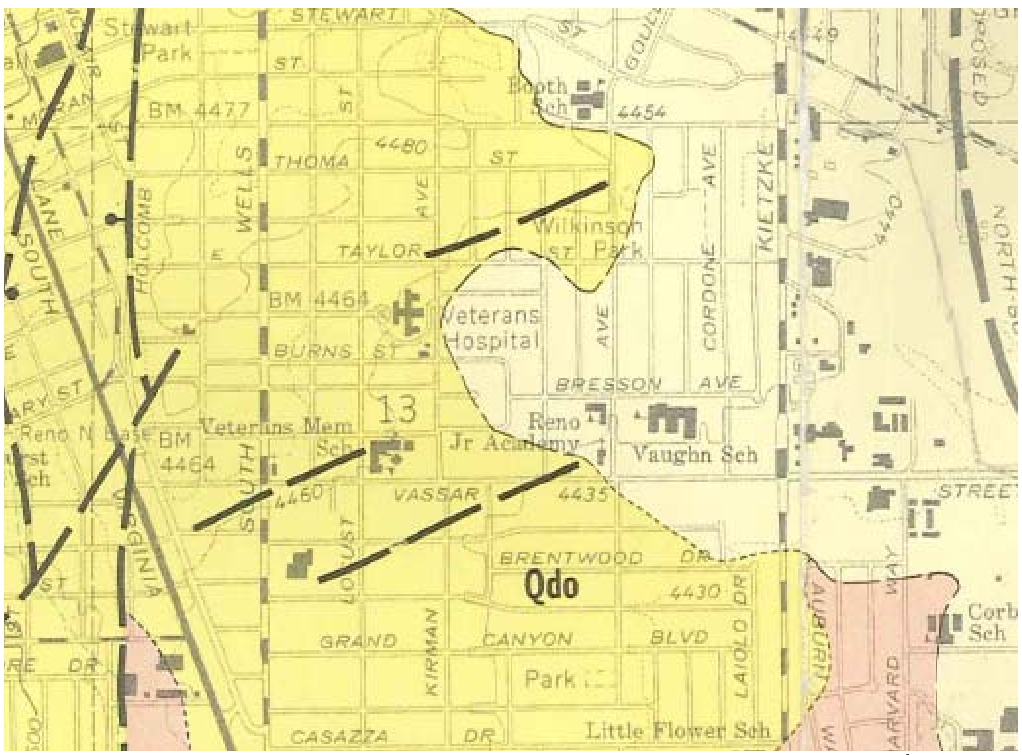

Direct Shear Test

Chemical Tests

Chemical testing was performed on representative samples of site foundation soils to evaluate the site materials' potential to corrode steel and PCC in contact with the ground. The samples were tested for pH, resistivity, redox potential, soluble sulfates and sulfides. The results of the chemical tests are shown on Plate 6 (Chemical Test Results). Chemical testing was performed by Sierra Environmental Monitoring of Reno, Nevada.

Geologic and General Soil Conditions

The site lies in an area mapped by the Nevada Bureau of Mines and Geology (NBMG) (Bonham and Bingler, 1973) as consisting of Quaternary Age Tahoe Outwash (Qto). The NBMG describes this material as consisting of *Boulder to cobble gravel, sandy gravel, and gravely sand. Contains giant boulders. Rock clasts are rounded to subrounded and, in decreasing order of abundance, are granitic, volcanic, and metamorphic.*



Geologic Map

Materials encountered during site exploration generally consist of an asphalt concrete pavement section (including an aggregate base layer) overlying clay-rich soils that extend approximately 4-½ to 7 feet below existing grade. Coarse granular soil (Tahoe Outwash) was encountered below the surface clay-rich layer to the depths explored (15.9 feet). The asphalt concrete pavement section consists of approximately 4 inches of asphalt concrete overlying a variable section of aggregate base between 2 and 6 inches thick. The underlying clayey sand/sandy lean clay/fat clay soils were generally described as moist, dense or firm to hard, and as containing between 20 and 70 percent high plasticity fines. The coarse granular soils beneath the surficial clay-rich soils were generally described as moist, medium dense to very dense, and as containing approximately 20 percent non-plastic to low plasticity fines. The drill response indicated the presence of abundant cobbles and boulders in the coarse granular soils. Due to the very dense nature of these materials at depth and the presence of abundant cobbles and boulders, practical drilling refusal occurred in both borings.

Ground water was not encountered during exploration and is expected to lie at a depth that would affect construction.

Geologic Hazards

Seismicity

Geologic Hazard Map

Much of the Western United States is a region of moderate to intense seismicity related to movement of crustal masses (plate tectonics). By far, the most active regions, outside of Alaska, are in the vicinity of the San Andreas Fault system of western California. Other seismically active areas include the Wasatch Front in Salt Lake City, Utah, which forms the eastern boundary of the Basin and Range physiographic province, and the eastern front of the Sierra Nevada Mountains, which is the western margin of the province. The Reno-Sparks area lies along the eastern base of the Sierra Nevada, within the western extreme of the Basin and Range.

The Truckee Meadows lies within an area with a high potential for strong earthquake shaking. Seismicity within the Reno-Sparks area is considered about average for the western Basin and Range Province (Ryall and Douglas, 1976). It is generally accepted that a maximum credible earthquake in this area would be in the range of magnitude 7 to 7.5 along the frontal fault system of the Eastern Sierra Nevada. The most active segment of this fault system in the Reno area is located at the base of the mountains near Thomas Creek, Whites Creek, and Mt. Rose Highway, some 4-½ miles south of the project.

Recurrence intervals for Nevada earthquakes along faults that have been studied are estimated to be in the range of 6,000 to 18,000 years in western Nevada (Bell, 1984). The very active eastern boundary faults of the Sierra Nevada Mountains may have a shorter recurrence interval of 1,000 to 2,000 years.

Faults

The published geologic hazards map (Bingler, 1974) shows several Post-Illinoian (Quaternary) faults within one-half mile of the site, but no faults as passing through the property. The Nevada Earthquake Safety Council (NESC, 1998) has developed and adopted the criteria for evaluation of Quaternary age earthquake faults. *Holocene*

Active Faults are defined as those with evidence of movement within the past 10,000 years (Holocene time). Those faults with evidence of displacement during the last 130,000 years are termed *Late Quaternary Active Faults*. A *Quaternary Active Fault* is one that has moved within the last 1.6 million years. An *Inactive Fault* is a fault *without recognized activity within Quaternary time* (last 1.6 million years). Holocene Active Faults normally require that occupied structures be set back a minimum of 50 feet (100-foot-wide zone) from the ground surface fault trace. An *Occupied Structure* is defined in the *International Building Code* as *a building, which is expected to have a human occupancy rate of more than 2,000 hours per year.*

The setback from Quaternary Active Faults is left to the judgment of the geologist/engineer; however, no *Critical Facility* is permitted to be placed over the trace of a Late Quaternary Active Fault. A *Critical Facility* is defined as *a building or structure that is considered critical to the function of the community or the project under consideration. Examples include, but are not limited to, hospitals, fire stations, emergency management operations centers and schools.*

Based on the geologic map, the faults in the vicinity of the project are considered Late Quaternary Active or Quaternary Active faults. Based on this mapped age and since no faults are mapped as passing through the subject site or were observed during site exploration, no additional fault hazard investigation or mitigation, or building offset, is considered necessary.

Ground Motion and Liquefaction

Mapping by the United States Geological Survey (USGS, 2011) indicates that there is a 2 percent probability that a *bedrock* ground acceleration of 0.66 will be exceeded in any 50-year interval. Only localized amplification of ground motion would be expected during an earthquake.

Because the site area is underlain by dense granular soils with a relatively deep ground water, liquefaction potential is minimal.

Flood Plains

The Federal Emergency Management Agency (FEMA) has identified the site as lying in unshaded Zone X, or outside the limits of a 500-year flood plain (FEMA, 2009).

Other Geologic Hazards

A moderate potential for dust generation is present if grading is performed in dry weather. Expansive clay-rich surficial soils are present across the site. No other geologic hazards were identified.

Discussion and Recommendations

General Information

The existing pavement is generally underlain by a surface layer of clay-rich soils and at a relatively shallow depth by coarse granular soils. Structural improvements will need to be separated from the clay-rich soils, as discussed in the Site Preparation section. Trenching will be difficult due to the presence of abundant cobbles and large boulders, as discussed under Trenching and Excavation. The existing asphalt concrete pavement present across the site is suitable for re-use as structural fill and/or recycled asphalt concrete base, as discussed in the Mass Grading section. Native coarse granular soils are suitable for re-use as structural fill after screening, but native surficial clay-rich soils should only be placed as fill in nonstructural areas, as described in Mass Grading.

The recommendations provided herein, and particularly under Geotechnical Design Recommendations, Construction and Civil Engineering Design Recommendations, and Quality Control, are intended to minimize risks of structural distress related to consolidation or expansion of native soils and/or structural fills. These recommendations, along with proper design and construction of the structure and associated improvements, work together as a system to improve overall performance. If any aspect of this system is ignored or poorly implemented, the performance of the project will suffer. Sufficient quality control should be performed to verify that the recommendations presented in this report are followed.

Structural areas referred to in this report include all areas of buildings, concrete slabs, asphalt pavements, as well as pads for any minor structures. All compaction requirements presented in this report are relative to ASTM D 1557. For the purposes of this project:

- Fine-grained soils are defined as those with more than 40 percent by weight passing the number 200 sieve, and a plastic index lower than 15.
- Clay soils are defined as those with more than 30 percent passing the number 200 sieve, and a plastic index greater than 15.
- Granular soils are those not defined by the above criteria.

Any evaluation of the site for the presence of surface or subsurface hazardous substances is beyond the scope of this investigation. When suspected hazardous substances are encountered during routine geotechnical investigations, they are noted in the exploration logs and immediately reported to the client. No such substances were revealed during our exploration. At least two, but possibly more, UST's were observed in the southeast corner of the site and will need to be removed/relocated as a part of this project. Any associated evaluation or analysis of these tanks was beyond the scope of our work.

Geotechnical Design Recommendations

Seismic Design Parameters

Seismic design of the VA Parking Structure is based on the recommendations contained in the publication titled *Seismic Design Requirements, H-18-8* (Department of Veterans Affairs [DVA], 2011), which follows recommendations for site evaluation contained in the 2009 *International Building Code* (International Code Council [ICC], 2009). The *International Building Code* requires a detailed soils evaluation to a depth of 100 feet to develop appropriate soils criteria. The geophysical analyses performed at the adjacent VA parking garage project (BEC, 2004) revealed an average shear-wave velocity of 1,476 feet per second for upper 100 feet of foundation materials, which corresponds to a Site Class C soil profile and is considered appropriate for this site. Based on this information, the recommended seismic design criteria are as follow:

TABLE 1 - SEISMIC DESIGN CRITERIA USING *H-18-8* (DVA, 2011) AND THE 2009 *INTERNATIONAL BUILDING CODE* (USGS, 2011)	
Approximate Latitude	39.516
Approximate Longitude	-119.798
Spectral Response at Short Periods, S_s, percent of gravity	150.3
Spectral Response at 1-Second Period, S_1, percent of gravity	60.3
Site Class	C
Site Coefficient F_a, decimal	1.00
Site Coefficient F_v, decimal	1.30
Site Adjusted Spectral Response at Short Periods, S_{MS}, percent of gravity	150.3
Site Adjusted Spectral Response at Long Periods, S_{M1}, percent of gravity	78.4
Design Spectral Response Acceleration at Short Periods, S_{DS}, percent of gravity	100.2
Design Spectral Response Acceleration at Long Periods, S_{D1}, percent of gravity	52.2

Foundation Design Parameters

The near-surface clay-rich soils are poor foundation materials such that footings should not bear directly in these materials. The most economical method of foundation support lies in spread footings bearing on structural fill or extended down to bear on properly prepared native granular soils.

Individual column footings and continuous wall footings underlain by a minimum of 2 feet of structural fill or properly prepared native granular soils can be designed for a net maximum allowable bearing pressure 4,000 pounds per square foot (psf), and should have minimum footings widths of 24 inches. The net allowable bearing pressure is the pressure at the base of the footing in excess of the adjacent overburden pressure. This allowable bearing value should be used for dead plus ordinary live loads. Ordinary live loads are that portion of the design live load which will be present during the majority of the life of the structure. Design live loads are loads which are produced by the use of the structure, such as by moveable objects, including people or automobiles, as well as snow loads. This bearing value may be increased by one-third for total loads. Total loads are defined as the maximum load imposed by the required combinations of dead load, design live loads, snow loads, and wind or seismic loads.

With this allowable bearing pressure, total foundation movements of approximately ¾ of an inch should be anticipated. Differential movement between footings with similar loads, dimensions, and base elevations should not exceed ½ inch. The majority of the anticipated movement will occur during the construction period as loads are applied.

Lateral loads, such as wind or seismic, may be resisted by passive soil pressure and friction on the bottom of the footing. The recommended coefficient of base friction is 0.48 and has been reduced by a factor of 1.5 on the ultimate soil strength. Design values for active and passive equivalent fluid pressures are 35 and 440 pounds per square foot per foot of depth, respectively. These design values are based on spread footings bearing on structural fill or properly prepared native granular soils, and backfilled with structural fill. All exterior footings should be placed a minimum two feet below adjacent finish grade for frost protection.

Metal Pipe Design Parameters

Laboratory testing was performed to evaluate the corrosion potential of the surficial site soils with respect to metal pipe in contact with the ground. The results of the laboratory testing indicate that the surficial clay-rich soils exhibit *moderate corrosion potential* (American Water Works Association, 1999). As a result, metal pipe in contact with this material will require corrosion protection. The deeper granular soils are not corrosive to buried metal pipe.

Portland Cement Concrete Mix Design Parameters

Soluble sulfate content has been determined for representative samples of the surficial clay-rich soils, and the results of the testing indicate that concrete in contact with the surficial clay-rich soils should experience only moderate degradation due to reaction with soil sulfate. Clay-rich soils will be over-excavated or otherwise separated from all structural areas; therefore, Type II cement can be used for all concrete work. Concrete mix designs for this project should exhibit a 28-day unconfined compressive strength of 4,000 pounds per square inch (psi) and a maximum water to cement ratio of 0.5.

Asphalt Concrete Pavement Design

Paved areas subject to truck traffic should consist of 4 inches of Type 3 asphalt concrete underlain by 6 inches of Type 2, Class B, aggregate base. Paved areas restricted to automobile traffic and parking can consist of 3 inches of Type 3 asphalt concrete underlain by 6 inches of Type 2, Class B, aggregate base. All base material beneath asphalt pavements should be densified to at least 95 percent relative compaction.

Construction and Civil Engineering Design Recommendations

Site Preparation

All existing asphalt concrete pavement should be either demolished and removed from the site; pulverized to be re-used as recycled asphalt concrete base material; or broken up to be re-used as structural fill in accordance with the applicable material specifications contained in the *Standard Specifications for Public Works Construction (SSPWC, 2007).*

Underground storage tanks will require removal from the site. Any associated excavation shall be backfilled with structural fill placed in accordance with the recommendations provided in the Mass Grading section.

Moisture sensitive soils, such as those present near the surface at this site, that are overlain by asphalt concrete pavement typically exist at moisture contents that are above optimum moisture levels for the material. Such materials require drying or sometimes stabilization, depending on the level of moisture present and the construction schedule. Our test results indicate that these soils are a few percent or more above optimum moisture content. Normally, soil moisture, particularly beneath pavements, increases some during the winter and spring months. If necessary, recommendations for stabilization are provided below.

Clay-rich soils were found to exist from immediately below the asphalt concrete to depths of 7 feet below the ground surface. These materials were classified as moist, dense or firm to hard, and as containing between 31 and to 61 percent high plasticity fines. Laboratory testing performed on these materials indicates they exhibit plasticity indices in the range of 14 to 44, indicative of moderately to very highly expansive soils (Nelson and Miller, 1992).

All clay-rich soils should be separated from overlying structural improvements by a minimum of 2 feet of structural fill beneath footings, concrete slabs, and asphalt pavements excluding any aggregate base sections. As an alternate to over-excavation beneath footings, the base of the footing could be extended to bear on the native granular soils present at variable depth beneath the clay-rich soils.

It must be emphasized that unless clay soils are completely removed from structural areas some differential movement should be anticipated. During our exploration, these clays were found to range from 2.5 feet to as much as 4 feet in thickness.

Any over-excavation should be backfilled with structural fill to footing grade, or subgrade for pavements and slabs. The width of over-excavation should extend laterally from the edge of footings, concrete slabs or asphalt pavements at least one-half the depth of the over-excavation. Based on proposed and existing grades, considerable over-excavation should be anticipated.

Clay-rich soils to be left in place and covered with fill should be moisture-conditioned to 2 to 4 percent over optimum for a minimum depth of 12 inches. This moisture level will significantly decrease the magnitude of shrink-swell movements in the upper foot of the soil. The high moisture content must be maintained by periodic surface wetting, or other methods, until the surface is covered by at least one lift of fill. If allowed to dry out, subsequent expansion of clay-rich soils beneath foundations and floor slabs could significantly exceed the design criteria set forth previously.

Where boulders are present and encroach upon design elevations, thereby requiring removal, the resulting void should be backfilled with structural fill.

If loose, soft, wet, or disturbed soils are encountered at the foundation subgrade, these soils should be removed to expose undisturbed native soils, and the resulting over-excavation backfilled with compacted structural fill. The base of all excavations should be dry and free of loose soils at the time of concrete placement.

All areas to receive structural fill or structural loading should be densified to, at least, 90 percent relative compaction. Where less than 70 percent passes the 3/4-inch sieve, soils are too coarse for standard density testing techniques. In this case, as will likely occur here, a proof rolling of a minimum five single passes with a minimum 10-ton roller in mass grading, or five complete passes with hand compactors in footing trenches is recommended. This alternate has proved to provide adequate project performance, as long as all other geotechnical recommendations are closely followed. In all cases, the final surface should be smooth, firm, and exhibit no signs of deflection.

Our test results indicate that the clay-rich soils are slightly to well above optimum moisture and may be impossible to compact in some areas. Wet weather construction would further increase moisture levels. In summer months, moisture conditioning may be possible by scarifying the top 12 inches of subgrade and allowing it to air dry to near-optimum moisture, prior to compaction. Where this procedure is ineffective or where construction schedules preclude delays, mechanical stabilization will be necessary. Mechanical stabilization may be achieved by over-excavation and/or placement of an initial 12- to 18-inch-thick lift of 12-inch-minus, 3-inch-plus, well graded, angular rock fill. The more angular and well graded the rock is, the more effective it will be. This fill should be densified with large equipment, such as a self-propelled sheeps-foot or a large loader, until no further deflection is noted. Additional lifts of rock may be necessary to achieve adequate stability. The use of a geotextile will prevent mud from pumping up between the rocks, thereby increasing rock-to-rock contact and decreasing the required thickness of stabilizing fill. The geotextile should meet or exceed the minimum properties presented in Table 2 (Minimum Average Roll Strength Properties for Geotextile).

TABLE 2 - MINIMUM AVERAGE ROLL STRENGTH PROPERTIES FOR GEOTEXTILE	
Trapezoid Strength (ASTM D 4533)	80 x 80 lbs.
Puncture Strength (ASTM D 4833)	500 lbs.
Grab Tensile Strength/Elongation (ASTM D 4632)	200 x 200 @ 50 %

As an alternate to rock fill, a geotextile/gravel system may be used for stabilization. Aggregate base, Class C or D drain rock, or pit-run gravels should be placed above the geotextile. Regardless of which alternate is selected, a test section is recommended to determine the required thickness of stabilization.

Trenching and Excavation

Trenching will be difficult due to the presence of abundant cobbles and large boulders present at depth. Neat-line footing excavations will be impossible in the coarser granular site soil present at depth. Trench walls may tend to ravel.

Temporary trenches with near-vertical sidewalls should be stable to a depth of approximately 5 feet. Temporary trenches are defined as those that will be open for less than 24 hours. Excavations to greater depths will require shoring or laying back of sidewalls to maintain adequate stability. Regulations contained in Part 1926, Subpart P, of Title 29 of the Code of Federal Regulations (CFR) (January 1, 2010) require that temporary sidewall slopes be no greater than those presented in Table 3 (Maximum Allowable Temporary Slopes).

TABLE 3 - MAXIMUM ALLOWABLE TEMPORARY SLOPES	
Soil or Rock Type	**Maximum Allowable Slopes[1] for Deep Excavations less than 20 Feet Deep[2]**
Stable Rock	Vertical (90 degrees)
Type A[3]	3H:4V (53 degrees)
Type B	1H:1V (45 degrees)
Type C	3H:2V (34 degrees)
Notes:	
1. Numbers shown in parentheses next to maximum allowable slopes are angles expressed in degrees from the horizontal. Angles have been rounded off.	
2. Sloping or benching for excavations greater than 20 feet deep shall be designed by a registered professional engineer.	
3. A short-term (open 24 hours or less) maximum allowable slope of 1H:2V (63 degrees) is allowed in excavation in Type A soils that are 12 feet or less in depth. Short-term maximum allowable slopes for excavations greater than 12 feet in depth shall be 3H:4V (53 degrees).	

The State of Nevada, Department of Industrial Relations, Division of Occupational Safety and Health Administration (OSHA), has adopted and strictly enforces these regulations, including the classification system and the maximum slopes. In general, Type A soils are cohesive, non-fissured soils, with an unconfined compressive strength of 1.5 tons per square foot (tsf) or greater. Type B are cohesive soils with an unconfined compressive strength between 0.5 and 1.5 tsf. Type C soils have an unconfined compressive strength below 0.5 tsf. Numerous additional factors and exclusions are included in the formal definitions. The client, owner, design engineer, and contractor shall refer to Appendix A and B of Subpart P of the previously referenced Federal Register for complete definitions and requirements on sloping and benching of trench sidewalls. Appendices C through F of Subpart P apply to requirements and methodologies for shoring.

On the basis of our exploration, the surface clay-rich soils are Type B, while the underlying granular soils are Type C. Any area in question should be considered Type C, unless specifically examined by the geotechnical engineer during construction. All trenching should be performed and stabilized in accordance with local, state, federal and OSHA standards.

Mass Grading

Existing asphalt concrete to be removed as a part of this project is suitable for re-use as structural fill as long as the material is broken down to 4-inch-minus material

(*SSPWC* Section 200.01.09) and placed in accordance with the requirements of Section 304.04.03 of the *SSPWC* (2007). In addition, this material may be pulverized to satisfy specifications for on-site recycled asphalt concrete base (*SSPWC* Section 200.01.04). Existing aggregate base can also be used, provided it can be salvaged without contamination form the underlying clay-rich soil. The variable thickness of the base will make salvage operations challenging. Pulverizing to a depth of 6 inches should result in minimal soil contamination of an asphalt concrete/aggregate base blend. Native clay soils should be placed as fill only in nonstructural areas. Native granular soils will be suitable for structural fill provided particles larger than 4 inches are removed. Since the excavated material at this site will generally be unsuitable clay-rich soil, import of structural fill is anticipated. Imported structural fill shall exhibit a resistivity in excess of 3,000 ohm-cm and satisfy the additional specifications of Table 4 (Guideline Specification for Imported Structural Fill).

TABLE 4 - GUIDELINE SPECIFICATION FOR IMPORTED STRUCTURAL FILL		
Sieve Size	**Percent by Weight Passing**	
4 Inch	100	
3/4 Inch	70 – 100	
No. 40	15 – 70	
No. 200	5 – 30	
Percent Passing No. 200 Sieve	**Maximum Liquid Limit**	**Maximum Plastic Index**
5 – 10	50	20
11 – 20	40	15
21 – 30	35	10

These recommendations are intended as guidelines to specify a readily available, prequalified material. Adjustments to the recommended limits can be provided to allow the use of other granular, non-expansive material. Any such adjustments must be made and approved by the geological engineer, in writing, prior to importing fill to the site.

All structural fill should be placed in maximum 8-inch-thick (loose) lifts, each densified to, at least, 95 percent relative compaction. Nonstructural fill should be densified to, at least, 85 percent relative compaction to minimize consolidation and erosion.

The native granular soils and recycled asphalt concrete at this site will have greater than 30 percent retained on the 3/4-inch sieve, such that standard density testing is not valid. If these materials are incorporated into mass grading at the site, they will be treated as rock fills with a maximum allowable lift thickness and particle size of 12 inches. A proof rolling program of at least five single passes of a large, vibratory sheepsfoot compactor is recommended during mass grading. Acceptance of this rock fill is based upon observation of particle size, lift thickness, moisture content, and applied compactive effort. Compaction must continue to the satisfaction of the geotechnical engineer. In all cases, the finished surface should be firm and show no signs of deflection. Grading should not be performed with or on frozen soils.

Utility Trench Backfill

The maximum particle size in trench backfill should be 4 inches. Bedding and initial backfill 12 inches over the pipe will require import and should conform to Class A specifications (*SSPWC*, 2007) or the requirements of the utility agency having jurisdiction. Bedding and initial backfill should be densified to at least 90 percent relative compaction. Native granular soil will provide adequate final backfill as long as oversized particles are excluded, and should be placed in maximum 8-inch-thick loose lifts that are compacted to a minimum of 90 percent relative compaction in all structural areas. When drain rock is used as trench backfill, it shall be considered a rock backfill (greater than 30 percent retained on the 3/4-inch sieve) and should be placed in maximum 12-inch-thick loose lifts, with each lift densified by at least five complete passes with approved compaction equipment and until no deflection is observed. A separator geotextile such as Mirafi® 140N or equivalent should be placed between the drain rock and any native soil backfill.

Excavations below the ground water table are not anticipated.

Subsidence and Shrinkage

Granular alluvial soils excavated and recompacted in structural fills will experience quantity shrinkage. The exact amount of shrinkage will be dependent on the amount of oversize material (cobbles and boulders) present in the subgrade that will be removed during screening operations. Since the amount of cobble and boulders present cannot be accurately estimated from exploration borings, an exact shrinkage value cannot be estimated; however, shrinkage will most likely vary between 20 and 50 percent locally.

Slope Stability and Erosion Control

There are no major cut or fill slopes planned for this project. Dust potential at this site will be moderate during dry periods. The contractor shall prevent dust from being generated during construction in compliance with all applicable city, county, state, and federal regulations. The contractor shall submit an acceptable dust control plan to the Washoe County District Health Department prior to starting site preparation or earthwork. Project specifications should include an indemnification by the contractor of the owner and engineer for any dust generation during the construction period. The owner will be responsible for mitigation of dust after accepting the project.

Temporary (during construction) and permanent (after construction) erosion control will be required for all disturbed areas. In order to minimize erosion and downstream impacts to sedimentation from this site, best management practices with respect to storm water discharge should be implemented at this site.

Site Drainage

Adequate surface drainage should be provided so moisture is directed away from the structure. A system of roof drains is recommended to collect roof drainage and direct it away from the foundations unless pavement extends to the walls. Surface drainage and roof drains should not be directed towards existing buildings and especially those with basements. Foundation backfill should be thoroughly compacted to decrease permeability and reduce the potential for irrigation and storm water to migrate below the ground level PCC slab. The ponding of water on finish grade or at the edge of pavements should be prevented by proper grading.

If planters are to be located adjacent to foundation areas, they should be lined and sloped to drain away from foundation to improve foundation performance. Raised planters bearing directly on pavement would be preferred. Planters are defined as localized landscaped and irrigated areas lying within 10 feet of the building perimeter and confined by decorative structures such as rock, wood, or brick.

Portland Cement Concrete Flatwork

All PCC slabs should be directly underlain by Type 2, Class B, aggregate base. Recycled asphalt concrete base that satisfies *SSPWC* Section 200.01.04 may be acceptable. The thickness of base material shall be 6 inches beneath curb and gutters, 4 inches beneath sidewalks and 4 inches beneath floor slabs and private flatwork. Aggregate base courses should be densified to at least 95 percent relative compaction.

The Reno area is a region with exceptionally low relative humidity. As a consequence, concrete flatwork is prone to excessive shrinking and curling. Concrete mix proportions and construction techniques, including the addition of water and improper curing, can adversely affect the finished quality of concrete and result in cracking, curling, and the spalling of slabs. We recommend that all placement and curing be performed in accordance with procedures outlined by the American Concrete Institute (ACI, 2008) and this report. Special considerations should be given to concrete placed and cured during hot or cold weather temperatures, low humidity conditions, and windy conditions such as are common in the Truckee Meadows area. Proper control joints and reinforcement should be provided to minimize any damage resulting from shrinkage.

Concrete shall not be placed on frozen soils.

Asphalt Concrete

All asphalt pavement shall be directly underlain by 6 inches of Type 2, Class B, aggregate base or recycled asphalt concrete base (*SSPWC* Section 200.01.04) if allowed. All aggregate base beneath asphalt pavements should be densified to, at least, 95 percent relative compaction.

Asphalt concrete pavements have been designed for a standard 20-year life expectancy. Due to the local climate and available construction aggregates, an entire 20 years of performance life is not achieved without substantial maintenance. Between 15 and 20 years after initial construction (average 17 years), major rehabilitation (structural overlay or reconstruction) is generally required. To maximize performance life, periodic maintenance is required. Such maintenance includes regular crack sealing, seal coats, and patching as necessary. Failure to provide the required maintenance will significantly reduce pavement design life and performance.

Anticipated Construction Problems

The soils beneath the existing pavement surface may tend to rut under construction traffic and make it difficult for construction equipment to travel and operate. Wet weather construction would further deteriorate these moisture-sensitive, clay-rich soils. Stabilization would be required under extreme circumstances if the construction schedule does not allow for time to air-dry over-optimum moisture content soils.

Trenching will be difficult due to the presence of abundant cobbles and large boulders at depth. Neat-line excavations will be impossible in these materials and trench walls may ravel.

Quality Control

All plans and specifications should be reviewed for conformance with this geotechnical report and approved by the geotechnical engineer prior to submitting them to the building department for review.

The recommendations presented in this report are based on the assumption that sufficient field testing and construction review will be provided during all phases of construction. We should review the final plans and specifications to check for conformance with the intent of our recommendations. Prior to construction, a pre-job conference should be scheduled to include, but not be limited to, the owner, architect, civil engineer, the general contractor, earthwork and materials subcontractors, building official, and geotechnical engineer. The conference will allow parties to review the project plans, specifications, and recommendations presented in this report and discuss applicable material quality and mix design requirements. All quality control reports should be submitted to and reviewed by the geotechnical engineer.

During construction, we should have the opportunity to provide sufficient on-site observation of preparation and grading, over-excavation, fill placement, foundation installation, and paving. These observations would allow us to verify that the geotechnical conditions are as anticipated and that the contractor's work is in conformance with the approved plans and specifications.

Standard Limitations Clause

This report has been prepared in accordance with generally accepted geotechnical practices. The analyses and recommendations submitted are based on field exploration performed at the locations shown on Plate 1 of this report. This report does not reflect soils variations that may become evident during the construction period, at which time re-evaluation of the recommendations may be necessary. We recommend our firm be retained to perform construction observation in all phases of the project related to geotechnical factors to ensure compliance with our recommendations. The owner shall be responsible for distributing this geotechnical investigation to all designers and contractors whose work is related to geotechnical factors.

Equilibrium water level readings were made on the date shown on Plate 2 of this report. Fluctuations in the water table may occur due to rainfall, temperature, seasonal runoff or adjacent irrigation practices. Construction planning should be based on assumptions of possible variations in the water table.

This report has been produced to provide information allowing the architect or engineer to design the project. The owner is responsible for distributing this report to all designers and contractors whose work is affected by geotechnical aspects. In the event there are changes in the design, location, or ownership of the project from the time this report is issued, recommendations should be reviewed and possibly modified by the geotechnical engineer. If the geotechnical engineer is not granted the opportunity to make this recommended review, he or she can assume no responsibility for misinterpretation or misapplication of his or her recommendations or their validity in the event changes have been made in the original design concept without his or her prior review. The geotechnical engineer makes no other warranties, either expressed or implied, as to the professional advice provided under the terms of this agreement and included in this report.

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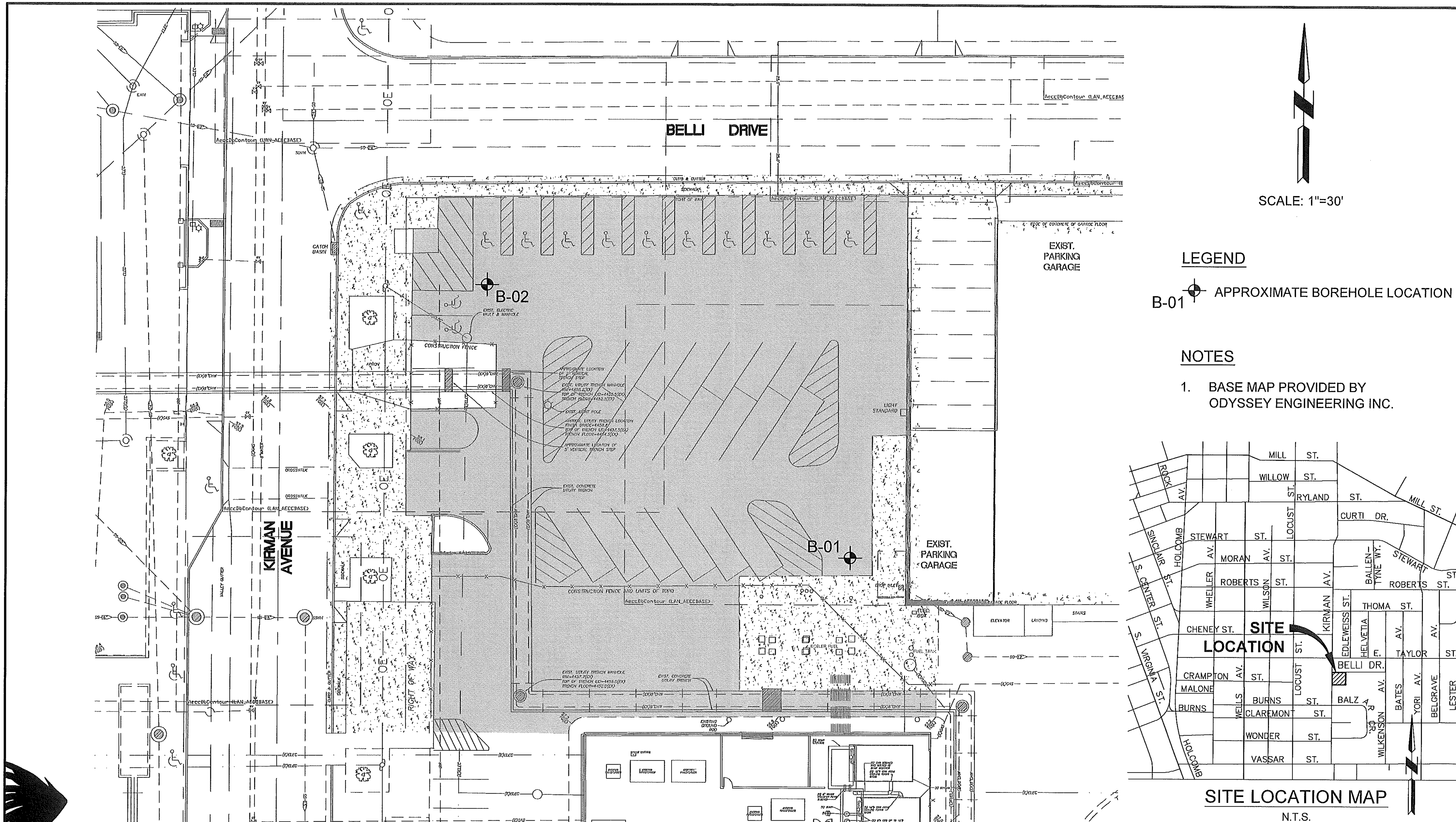
PLATES



Black Eagle Consulting, Inc.
Geotechnical & Construction Services
1345 Capital Boulevard, Suite A
Reno, Nevada 89502-7140
Telephone: 775/359-6600
Facsimile: 775/359-7766

H + K ARCHITECTS

PLOT PLAN

VETERANS ADMINISTRATION PARKING GARAGE

RENO, NEVADA

Project No.
0833-04-1

Plate 1

BORING LOG

BORING NO.: B-01 | DATE: 1/21/2012

TYPE OF RIG: CME 55 | DEPTH TO GROUND WATER (ft): NE

LOGGED BY: GCS | GROUND ELEVATION (ft): 4465

SAMPLE NO.	SAMPLE TYPE	BLOWS/12 inches	MOISTURE (%)	PLASTICITY INDEX	DEPTH (ft)	USCS SYMBOL	DESCRIPTION
					0–0.33		**Asphalt Concrete** 4 inches
					0.33–0.67		**Aggregate Base** 4 Inches
1	SPT	33	14.4	22	0.67–4.5	SC	**Clayey Sand with Gravel** Brown, moist, dense, with 31% high plasticity fines, 50% fine to coarse sand, and 19% fine gravel. Unit contains abundant cobbles and boulders as indicated by drill response.
2	SPT	38	20.1	14	4.5–7	CL	**Sandy Lean Clay** Brown, moist, hard, with 61% medium plasticity fines, 38% fine to coarse sand, and 1% fine gravel.
3	SPT	50/4"			7–12	SM	**Silty Sand with Gravel** Light brown, moist, very dense with an estimated 20% low plasticity fines, 60% fine to coarse sand, and 20% fine to coarse gravel.
~~4~~	~~SPT~~	~~50/1"~~					
5	SPT	50/5"			12–15	GM	**Silty Gravel** Light brown, moist, very dense, with an estimated 20% low plasticity fines, 30% fine to coarse sand, and 50% fine to coarse gravel.
6	SPT	50/5"			15–15.9	SM	**Silty Sand with Gravel** Light brown, moist, very dense, with an estimated 20% non-plastic fines, 60% fine to coarse sand, and 20% fine to coarse gravel.

Refusal at 15.9 feet due to boulders

LITHOLOGY

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1345 Capital Blvd., Suite A
Reno, Nevada 89502-7140
(775) 359-6600

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Reno, Nevada

PROJECT NO.: 0833-04-1

PLATE: 2

SHEET 1 OF 1

BORING LOG

BORING NO.: B-02 | DATE: 1/21/2012

TYPE OF RIG: CME 55 | DEPTH TO GROUND WATER (ft): NE

LOGGED BY: GCS | GROUND ELEVATION (ft): 4465

SAMPLE NO.	SAMPLE TYPE	BLOWS/12 inches	MOISTURE (%)	PLASTICITY INDEX	DEPTH (ft)	USCS SYMBOL	LITHOLOGY	DESCRIPTION
					0			**Asphalt Concrete** 4 inches
								Aggregate Base 6 Inches
						CH		**Sandy Fat Clay** Light brown, moist, firm, with 52% high plasticity fines, 45% fine to coarse sand, and 3% fine to coarse gravel.
1	SPT	7	29.9	44	2–4			
2	SPT	19	9.8	NP	4.5	GM		**Silty Gravel with Sand** Light brown, moist, medium dense to very dense, with 18% non-plastic fines, 31% fine to coarse sand, and 51% fine to coarse gravel. Unit contains abundant cobbles and boulders as indicated by drill response.
3	SPT	50/3"			7			
4	SPT	50/0"			9.5			Refusal at 9.5 feet due to boulders

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(775) 359-6600

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PROJECT NO.: 0833-04-1

PLATE: 2

SHEET 1 OF 1

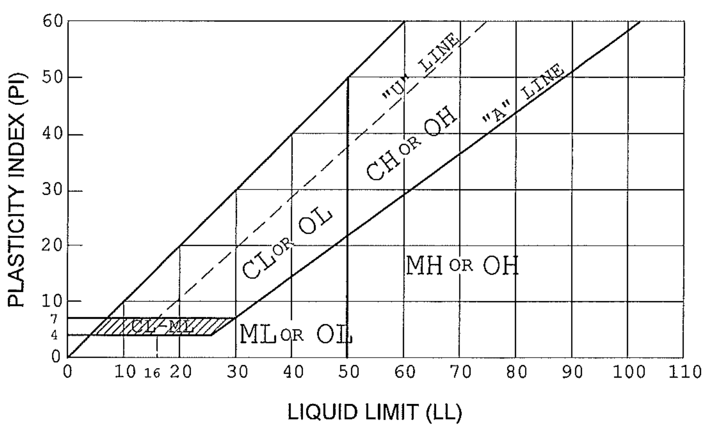
BORING_LOG 0833041.GPJ BLKEAGLE.GDT 2/1/2012

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS
FILL MATERIAL				--	FILL MATERIAL, NON-NATIVE

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS.

PLASTICITY CHART



FOR CLASSIFICATION OF FINE-GRAINED SOILS AND FINE-GRAINED FRACTION OF COARSE-GRAINED SOILS

EXPLORATION SAMPLE TERMINOLOGY

Sample Type	Sample Symbol	Sample Code
Auger Cuttings		Auger
Bulk (Grab) Sample		Grab
Modified California Sampler		MC
Shelby Tube		SH or ST
Standard Penetration Test		SPT
Split Spoon		SS
No Sample		

GRAIN SIZE TERMINOLOGY

Component of Sample	Size Range
Boulders	Over 12 in. (300mm)
Cobbles	12 in. to 3 in. (300mm to 75mm)
Gravel	3 in. to #4 sieve (75mm to 2mm)
Sand	# 4 to #200 sieve (2mm to 0.074mm)
Silt or Clay	Passing #200 sieve (0.074mm)

RELATIVE DENSITY OF GRANULAR SOILS

N - Blows/ft	Relative Density
0 - 4	Very Loose
5 - 10	Loose
11 - 30	Medium Dense
31 - 50	Dense
greater than 50	Very Dense

CONSISTENCY OF COHESIVE SOILS

Unconfined Compressive Strength, psf	N - Blows/ft	Consistency
less than 500	0 - 1	Very Soft
500 - 1,000	2 - 4	Soft
1,000 - 2,000	5 - 8	Firm
2,000 - 4,000	9 - 15	Stiff
4,000 - 8,000	16 - 30	Very Stiff
8,000 - 16,000	31 - 60	Hard
greater than 16,000	greater than 60	Very Hard

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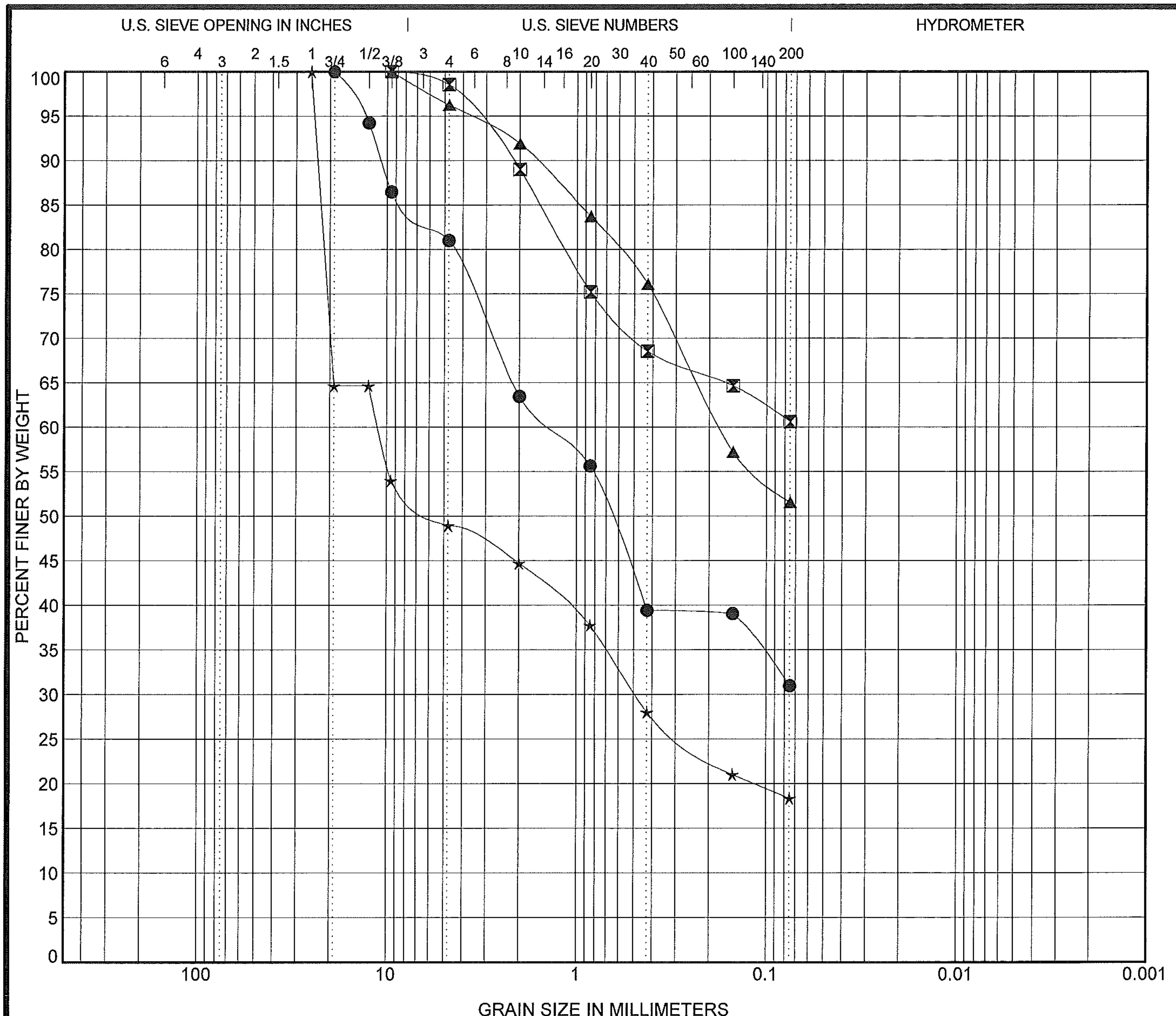
1345 Capital Blvd., Suite A
Reno, Nevada 89502-7140
Telephone: (775) 359-6600
Fax: (775) 359-7766

USCS Soil Classification Chart

Project: Veterans Administration Parking Garage

Location: Reno, Nevada

Project Number: 0833-04-1 Plate: 3



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Specimen Identification			USCS Classification	LL	PL	PI	Cc	Cu
●	**B-01**	**2.0'**	**CLAYEY SAND with GRAVEL (SC)**	**40**	**18**	**22**		
⊠	**B-01**	**4.5'**	**SANDY LEAN CLAY (CL)**	**38**	**24**	**14**		
▲	**B-02**	**2.0'**	**SANDY FAT CLAY (CH)**	**63**	**19**	**44**		
★	**B-02**	**4.5'**	**SILTY GRAVEL with SAND (GM)**	**NP**	**NP**	**NP**		

Specimen Identification			D100	D60	D30	D10	MC %	%Gravel	%Sand	%Silt	%Clay
●	**B-01**	**2.0'**	**19**	**1.369**			**14.4**	**19.0**	**50.0**	**31.0**	
⊠	**B-01**	**4.5'**	**9.5**				**20.1**	**1.5**	**37.9**	**60.6**	
▲	**B-02**	**2.0'**	**9.5**	**0.175**			**29.9**	**3.7**	**44.7**	**51.6**	
★	**B-02**	**4.5'**	**25**	**11.09**	**0.49**		**9.8**	**51.1**	**30.6**	**18.4**	

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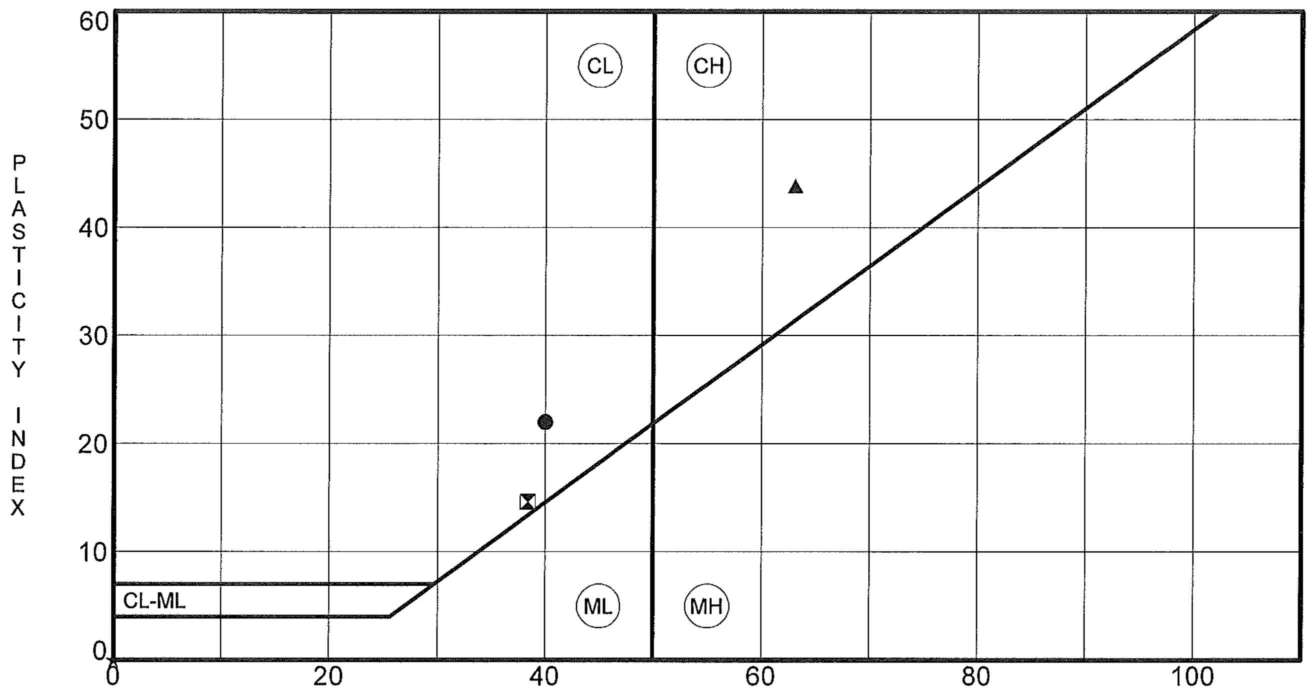
Telephone: (775) 359-6600
Fax: (775) 359-7766

GRAIN SIZE DISTRIBUTION

Project: Veterans Administration Parking Garage

Location: Reno, Nevada

Project Number: 0833-04-1 Plate: 4a



Specimen Depth in Feet.

	Specimen Identification			LL	PL	PI	Fines	USCS Classification
●	B-01	1	2.0'	40	18	22	31	**CLAYEY SAND with GRAVEL (SC)**
⊠	B-01	2	4.5'	38	24	14	61	**SANDY LEAN CLAY (CL)**
▲	B-02	1	2.0'	63	19	44	52	**SANDY FAT CLAY (CH)**
★	B-02	2	4.5'	NP	NP	NP	18	**SILTY GRAVEL with SAND (GM)**

US ATTERBERG LIMITS 0833041.GPJ US LAB.GDT 2/2/2012

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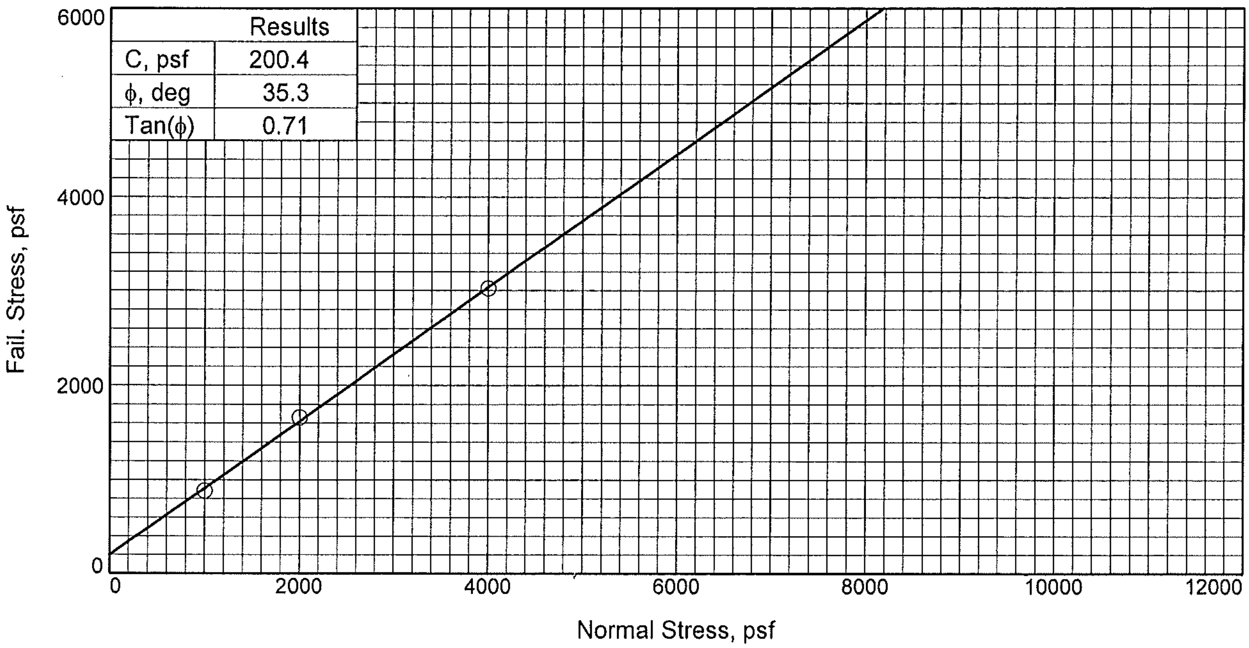
1345 Capital Blvd., Suite A
Reno, Nevada 89502-7140
Telephone: (775) 359-6600
Fax: (775) 359-7766

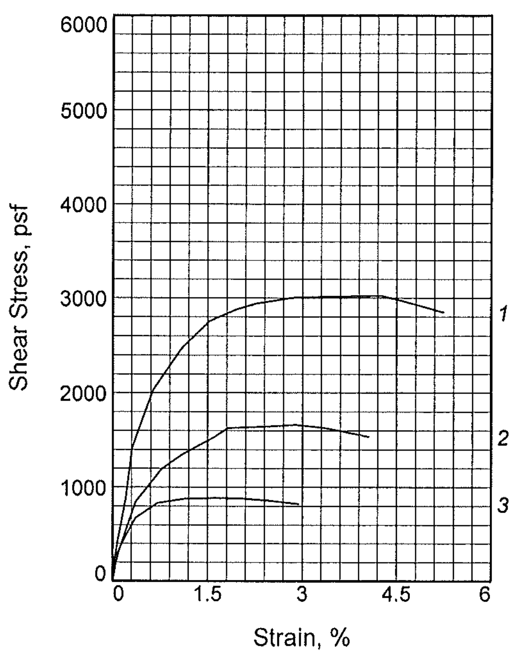
ATTERBERG LIMITS RESULTS

Project: Veterans Administration Parking Garage

Location: Reno, Nevada

Project Number: 0833-04-1 Plate: 4b





	Sample No.	1	2	3
Initial	Water Content, %	10.0	10.0	10.0
	Dry Density, pcf	109.2	108.8	109.3
	Saturation, %	49.7	49.1	49.8
	Void Ratio	0.5439	0.5497	0.5418
	Diameter, in.	2.420	2.420	2.420
	Height, in.	1.000	1.010	1.000
At Test	Water Content, %	16.7	17.9	18.7
	Dry Density, pcf	114.9	113.1	111.6
	Saturation, %	96.9	98.5	99.0
	Void Ratio	0.4667	0.4897	0.5099
	Diameter, in.	2.420	2.420	2.420
	Height, in.	0.950	0.971	0.979
	Normal Stress, psf	4000.0	2000.0	1000.0
	Fail. Stress, psf	3024.3	1659.3	882.9
	Strain, %	4.3	2.9	1.6
	Ult. Stress, psf			
	Strain, %			
	Strain rate, in./min.	0.020	0.020	0.020

Sample Type: Remolded
Description: Silty Gravel with Sand

LL= No Value **PI=** Non Plastic
Assumed Specific Gravity= 2.7
Remarks: Laboratory Number 2573

Plate 5

Client: H & K Architects

Project: Veterans Administration Parking Garage

Source of Sample: B 02 **Depth:** 4.5'
Sample Number: 2
Proj. No.: 0833-04-1 **Date Sampled:** 01/21/12

DIRECT SHEAR TEST REPORT
BLACK EAGLE CONSULTING, INC.
Reno, Nevada

Tested By: G. Bomberger

Laboratory Report
Report ID: 118461

Sierra Environmental Monitoring, Inc.

Black Eagle Consulting, Inc.
Attn: Pat Pilling
1345 Capital Blvd., Suite A
Reno, NV 89502-7140

Date: 1/27/2012
Client: BEC-100
Taken by: Gretchen Schma
PO #:

Analysis Report

Laboratory Sample ID	Customer Sample ID	Date Sampled	Time Sampled	Date Received
S201201-0897	0833-04-1 B-01 Sample 2	1/21/2012	10:00 AM	1/23/2012

Parameter	Method	Result	Units	Reporting Limit	Analyst	Date Analyzed	Data Flag
pH - Saturated Paste	SW-846 9045A	7.54	pH Units		Seher	1/25/2012	
pH - Temperature	SW-846 9045A	23.0	°C		Seher	1/25/2012	
Redox Potential	SM 2580 B	339	MV		Faulstich	1/27/2012	
Resistivity	EPA 120.1	1100	ohm cm		Faulstich	1/27/2012	
Sulfate - Ion Chromatography	EPA 300.0	190	mg/Kg	10	Faulstich	1/25/2012	Jl
Sulfide	EPA 376.1	NEGATIVE	Pos/Neg	1	Faulstich	1/24/2012	

Data Flag Legend:

Jl - The batch MS and/or MSD were outside acceptance limits. The batch LCS was acceptable.

John Kobza, Ph.D
Laboratory Director

1135 Financial Blvd.
Reno, Nv 89502-2348
Phone (775) 857-2400
Fax (775) 857-2404
sem@sem-analytical.com

John C. Seher
Special Consultant
Quality Assurance Manager