Preliminary Geotechnical Report & Fault Study

WSUP23-0025
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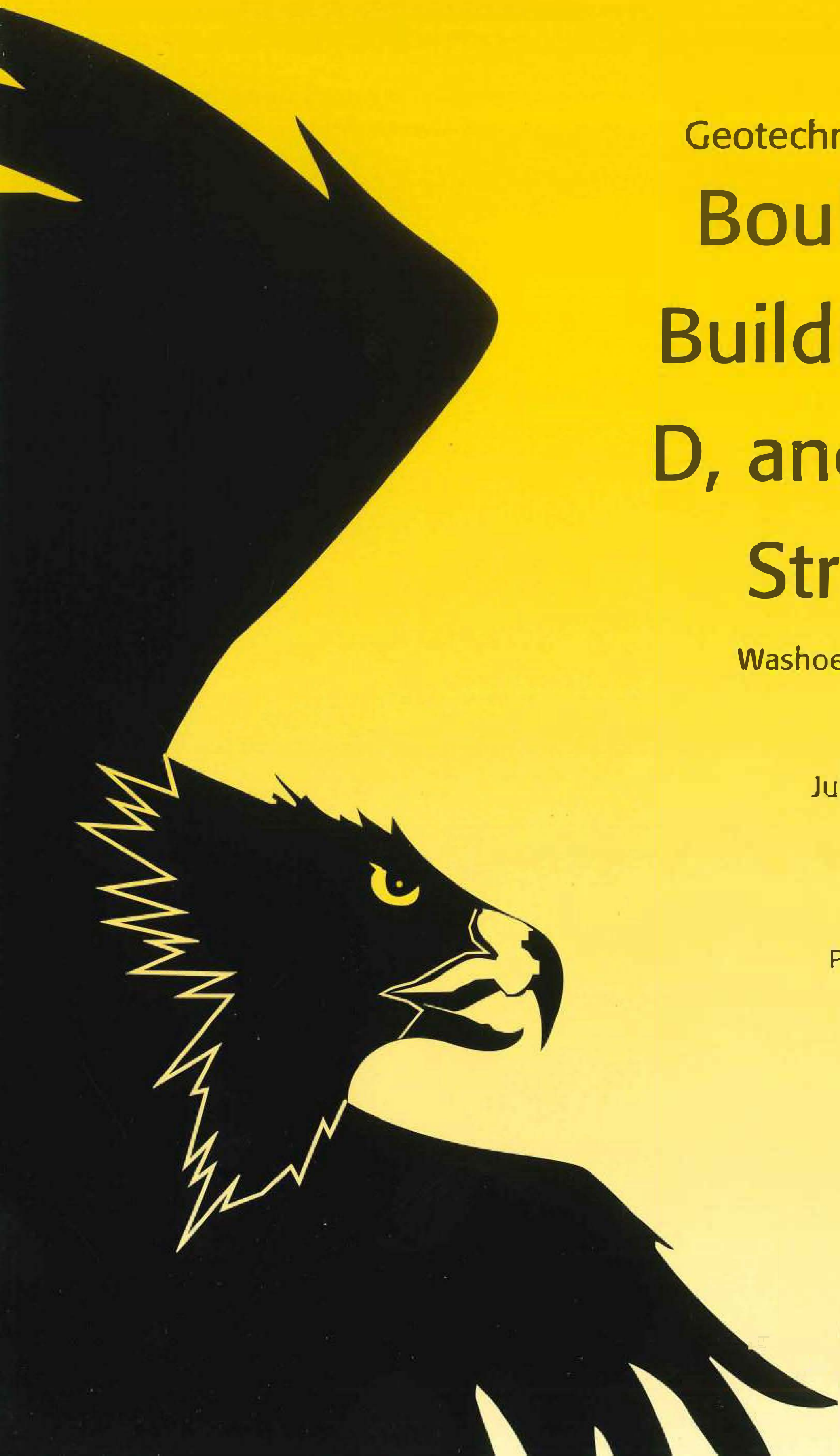
Black Eagle Consulting, Inc.



Geotechnical Investigation

Boulder Bay Buildings B, C, D, and Parking Structure

Washoe County, Nevada

July 13, 2018

Prepared for
CFA, Inc.

Black Eagle Consulting, Inc.
Geotechnical & Construction Services

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Mr. Mike Wilhelm, P.E., W.R.S.
CFA, Inc.
1150 Corporate Boulevard
Reno, Nevada 89502

July 13, 2018
Project No.: 0091-52-1

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RE: Geotechnical Investigation
Boulder Bay Buildings B, C, D, and Parking Structure
Crystal Bay, Washoe County, Nevada

Dear Mr. Wilhelm:

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 project.

The overall Boulder Bay project involves the complete redevelopment of the current Tahoe Biltmore property located in Crystal Bay, Washoe County, Nevada. The first phase of the project includes Building A, which is currently under construction. The second phase consists of the northbound extension of Stateline Road to connect with Lakeview Avenue and ultimately with Wassou Road to form a perimeter roadway around the Boulder Bay project. Subsequent future phases of the project will involve the construction of 7 additional buildings and a parking structure. This report is relevant to Buildings B, C, D, and the proposed parking structure.

The site exhibits a thin silty sand soil cover underlain by generally weathered granitic bedrock; these on-site materials will provide excellent support for the proposed improvements in cuts and also as compacted structural fill. The most significant constraint to construction of the project includes moderate to steeply sloping topography and below-grade building levels that will necessitate significant cuts and fills and tall site and building retaining walls.

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
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Introduction

Presented herein are the results of Black Eagle Consulting, Inc.'s (BEC's) geotechnical investigation, laboratory testing, and associated geotechnical design recommendations for Buildings B, C, D, and the proposed parking structure at the Boulder Bay project in the Crystal Bay community area of Washoe County, Nevada, directly east of the State of California border. 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, bedrock, and groundwater conditions pertaining to design and construction of the proposed project.
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 proposal dated January 3, 2018, and the associated CFA, Inc. Professional Services Agreement dated March 15, 2018, which was signed by Mr. Bob LaRiviere of CFA, Inc.

Project Description

The overall Boulder Bay project will involve the complete redevelopment of the current Tahoe Biltmore property as well as the realignment of Wassou Road, connecting the north end of Stateline Road to Lakeview Avenue, and connecting Lakeview Avenue to Wassou Road. The overall project will be a mixed-use development with 8 separate buildings that will host a hotel, condominiums, a health and wellness center, meeting and banquet space, a restaurant, retail shops, a fitness center, a small casino, a swimming pool, and a spa. Buildings are proposed to include 2 to 8 stories, with some including 1 or more below-grade levels. A 3-story, above-grade parking structure is proposed south and east of Building D. The overall Boulder Bay project area is contained in Sections 19 and 30, Township 16 North, Range 18 East, Mount Diablo Meridian.

The first phase of the project is in progress and includes Building A, which is currently under construction. Building A will host 18 luxury condominiums. The second phase consists of the northbound extension of Stateline Road to connect with Lakeview Avenue and ultimately with Wassou Road to form a perimeter roadway around the Boulder Bay project. Black Eagle Consulting, Inc. prepared a geotechnical investigation report for the second phase titled *Preliminary Geotechnical Investigation, Boulder Bay Stateline Road – Lakeview Avenue – Wassou Road Interconnect, Washoe County, Nevada*, dated June 14, 2018 (BEC, 2018). This second phase will involve various realignment, reconstruction, and extension of the existing streets as well as abandonment of portions of Reservoir Road and Wassou Road. This report and the recommendations contained here are relevant to the design and construction of Buildings B, C, D, and the proposed 3-story parking structure.

Detailed plans regarding the type of construction were unavailable at the time of this report; however, we anticipate the structures will utilize some combination of Portland Cement Concrete (PCC) columns and walls and steel-framed construction. We assume the buildings will be supported on PCC spread and continuous footing systems with PCC slab-on-grade ground floors and PCC post-tensioned or conventionally reinforced floor decks.

We understand current plans are considering 2 different potential options for Building B. The first option would be a 6-story building with an approximate lower level finished floor of 6,465 feet above mean sea level (msl). The second option would be an 8-story building with an approximate lower level finished floor elevation of 6,445 feet above msl. Material cuts of a few feet up to greater than 20 feet will be needed to establish a finished floor elevation of 6,465 feet above msl and about 20 to greater than 40 feet if the 8-story option is selected.

Buildings C and D are proposed as 6-story structures with approximate lower level finished floor elevations of 6,422 feet above msl. This design grade will require material cuts on the order of 15 to 30 feet.

We anticipate the 3-story parking structure will be a PCC structure with conventionally reinforced PCC columns and walls and post-tensioned PCC beams and parking decks. The parking structure lower level floor will match the elevation of Buildings C and D. This design grade will require material cuts on the order of 20 to 30 feet. A pool deck area is planned for the roof of the parking structure.

Site Conditions

The site of proposed Building B consists of a previously graded area currently being utilized for staging of construction trailers, equipment and materials needed for construction of Boulder Bay Building A, as well as a portion of the existing Wassou Road alignment. The site of proposed Building C consists of an area of the existing asphalt concrete parking lot, a portion of the existing Reservoir Road alignment, and a previously graded area associated with the current construction of Boulder Bay Building A. The site of proposed Building D as well as the proposed parking structure consists of an asphalt concrete parking area with a small retaining wall. The parking area slopes at approximately 8 to 10 percent to the south.

The overall site topography in the area of proposed Buildings B, C, D, and the parking structure generally slopes in a south to southeasterly direction at gradients ranging from approximately 5 to 15 percent.

Exploration

The overall Boulder Bay site was explored by advancing borings and hand auger holes and performing shear wave velocity surveys of the subsurface materials.

Drilling

The Boulder Bay site was explored during mid-April 2018 by drilling 12 test borings. The exploration associated with Buildings B, C, D, and the parking structure included 8 of the 12 borings (borings B-05 through B-12). The borings were advanced using solid-stem auger (SSA) and HQ coring techniques with a track-mounted CME 550 drill rig and a truck-mounted Diedrich D-120 drill rig. The SSA borings were advanced using 4-inch- and 6-inch-outside-diameter (O.D.) augers. The HQ core barrels are 96 millimeter (mm) O.D. and 63.5 mm inside diameter. Where refusal occurred using SSA drilling techniques, borings were advanced using HQ coring techniques to obtain continuous sampling of the bedrock/soil matrix. The maximum depth of drilling exploration was approximately 41 feet below the existing ground surface. The locations of the test borings are shown on Plate 1. All borings drilled for the project throughout the project site, including along the roadway alignments, are included for reference.

During SSA drilling, the native soils were sampled in-place every 2.5 to 5 feet by use of a standard, 2-inch-O.D., split-spoon sampler driven by a 140-pound automatic drive hammer with a 30-inch stroke. 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.

A 3-½-inch-O.D., split-spoon sampler (ASTM D 3550), also known as a Modified California (MC) sampler, was used to sample soils containing gravel or where approximate in-place densities of subsurface materials were required. Sampling methods used were similar to the SPT but also included the use of 2-½-inch-diameter, 6-inch-long, brass sampling tubes placed inside the split-spoon sampler. Because of the larger diameter of the sampler, blow counts are typically higher than those obtained with the SPT and should not be directly equated to SPT blow counts. The logs indicate the type of sampler used for each sample.

Bedrock was continuously cored at 1 boring location, boring B-09, starting at a depth of 20.5 feet through the maximum depth of exploration, approximately 41 feet. Rock cores were extracted from the HQ core barrels and placed in core boxes. Rock cores were sampled in accordance with ASTM D2113-08 to identify various indicators regarding the geological, physical, and engineering nature of the bedrock.

Shear Wave Velocity Survey

Four refraction micro-tremor surveys were performed to evaluate the average shear wave velocity within the upper 100 feet of subsurface materials. Shear wave velocity is used to determine the seismic soil profile classification per the *International Building Code* ([*IBC*] International Code Council [ICC], 2012). Shear wave velocity is also used to estimate the rippability of site bedrock using seismic velocity charts developed by Caterpillar, Inc. (2012). The compressional or seismic wave velocities were estimated by multiplying shear wave velocities by a factor of 2.5. The methodology of shear wave velocity analysis is included in Appendix A (Shear Wave Velocity Modeling Results). The approximate locations of the geophysical survey lines are shown on Plate 1. Results below 75 feet depth are generally not very meaningful or reliable, but shear wave velocities are expected to increase with depth relative to the values measured at shallower depths.

Material Classification

A geologist examined and identified all materials in the field in accordance with ASTM D 2488. During SSA drilling, representative samples were placed in sealed plastic bags and returned to our Reno, Nevada laboratory for testing. Recovered rock cores were handled in accordance with ASTM D 5079 and placed in cardboard core boxes and returned to our Reno, Nevada laboratory for testing and further analysis. During HQ coring, the sampled core in each core run was logged, describing weathering, fracturing, strength, and quality of the rock as measured by Rock Quality Designation. Rock Quality Designation is a scale describing the proportion of intact, durable rock within the formation. The scale, from 0 to 100 percent, is broken into the categories of Very Poor (0 to 25 percent), Poor (25 to 50 percent), Fair (50 to 75 percent), Good (75 to 90 percent), and Excellent (90 to 100 percent) Rock Quality.

Logs on the test borings and hand auger holes are presented as Plate 2 (Exploration Logs), and a Unified Soil Classification System (USCS) chart has been included as Plate 3 (USCS Soil Classification Chart).

Laboratory Testing

All soils testing performed in the BEC soils laboratory is conducted in general 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 published charts (Bowles, 1996; Naval Facilities Engineering Command [NAVFAC], 1986a and b). The index test results on both soils and bedrock sampled as soils were used to evaluate bearing capacity, lateral earth pressures, settlement potential, and their suitability for use as fills.

Direct Shear Test

A direct shear test (ASTM D 3080) was performed on a representative sample of material. The test was run on a remolded, inundated sample under various normal loads in order to develop a Mohr's strength envelope. For the remolded sample, the sample was screened to remove particles larger than the number 4 sieve prior to testing. Results of the test 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

R-Value Test

A resistance value (R-value) test (ASTM D 2844) was performed on a representative sample of soil/bedrock materials to be used in roadways. Resistance value testing is a measure of subgrade strength and expansion potential and is used in design of flexible pavements. Results of the R-value test are shown on Plate 6 (R-Value Test Results).

Laboratory Moisture-Density Relation Test

A moisture-density relation test (ASTM D 1557) was performed on a representative sample of the native soils. The maximum density shown by this test is compared with field densities to determine the percent of relative compaction. The moisture density curve is included as Plate 7 (Compaction Test Report).

Unconfined Compressive Strength Test

An intact rock core was tested to determine its unconfined compressive strength. The core was trimmed to exhibit a height to diameter ratio of approximately 2:1. The unconfined compressive strength can be used to evaluate bearing capacity of intact in-place rock.

Unconfined compressive strength testing was performed in general accordance with ASTM D 2166 and D 7012. Test results are shown on Plate 8 (Rock Core Analyses).

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 in Appendix B (Chemical Test Results). Chemical testing was performed by Silver State Analytical Laboratories of Reno, Nevada.

Geologic and General Soil Conditions

The site is located within the Lake Tahoe basin of the Sierra Nevada. Lake Tahoe formed within a fault bounded basin adjacent to the eastern front of the Sierra Nevada. Overall, the area consists of young, unconsolidated glacial, lacustrine and fluvial sediments overlying shallow granitic bedrock of the Sierra batholith. The Nevada Bureau of Mines and Geology (NBMG) has mapped the site as Cretaceous age *Hornblende granodiorite* described as *light to medium gray, medium grained, hypidiomorphic. Massive-structureless to weakly foliated on mafic minerals. Sparse mafic inclusions occurs on peninsula of Stateline Point* (Grose, 1985).

The site soils and bedrock are generally consistent with the NBMG geologic map. Our exploration encountered a surficial layer of alluvial silty sand soils up to 3 feet thick. Isolated areas along the existing roads contain fill soils derived from the alluvium; fill soils were encountered up to 4 feet thick in boring B-04. Granitic bedrock underlies the alluvial and fill soils, becoming less weathered and harder with depth. The granitic rock is variably weathered, with moderate to severe weathering through depths of 7 to 20 feet beneath the existing grade. The deeper granitic rock is slightly to moderately weathered and is generally weak to moderately strong to the maximum depth of exploration, 41 feet beneath the ground surface. The deeper granitic bedrock includes hard "corestones" of intact hard bedrock (see photo of cores).

Granitic Bedrock Cores
Boring B-09, 20.5 to 40.8 Feet

The surficial fill and alluvial soils were difficult to distinguish in our borings and are described here together. The surficial silty sand materials are described as brown, moist, medium dense, and as containing 15 to 26 percent non-plastic fines and up to 20 percent subangular to subrounded gravel.

The underlying weathered granitic rock has been weathered to soil materials but still retains its original rock textures. These materials were easily drilled using SSA drilling techniques and will excavate like soil materials. The weathered zone may contain durable cobble/boulder-sized corestones, but they are expected to be in relatively low quantities. These rock materials were sampled as silty sand in SPT/MC samples and auger cuttings and are described as tan to brown to light gray, moist, medium dense to dense, and as containing about 15 to 26 percent non-plastic fines, 74 to 85 percent fine to coarse sand, and trace amounts of fine gravel.

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Deeper decomposed granite materials are present starting at depths of 7 to 15 feet and have been variably weathered to weak to moderately strong rock. These materials will include durable cobbles and boulders and larger areas of intact hard rock. These rock materials were sampled as silty sand in SPT/MC samples and auger cuttings and are described as light gray, moist, medium dense to dense, and as containing about 15 to 25 percent non-plastic to low plasticity fines, 75 to 85 percent fine to coarse sand, and trace amounts of fine gravel. Generally, this unit sampled with refusal SPT blow counts and required rock coring in 1 location.

Groundwater was not encountered during exploration, and the static groundwater table is expected to lie at a depth well below that which would affect construction. However, seasonal snowmelt runoff will produce perched water conditions. This is particularly true with respect to shallow groundwater seepage that may occur as a result of sloping topography combined with fracture systems and a stratigraphy that consists of surficial soils overlying relatively impermeable bedrock.

Geologic Hazards

Seismicity

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 seismically 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 Lake Tahoe area lies within the eastern extent of the Sierra Nevada, within the western extreme of the Basin and Range. It must be recognized that there are probably few regions in the United States not underlain at some depth by older bedrock faults. Even areas within the interior of North America have a history of strong seismic activity.

Lake Tahoe lies within a region with a high potential for strong earthquake shaking. Seismicity within the north Lake Tahoe 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 north Lake Tahoe area is located at the base of the mountains near Washoe Lake, some 8 miles east of the project.

Faults

An earthquake hazards map is not available for the project area. The NBMG *My Hazards* web mapping tool (NBMG, 2018) and the geologic map (Grose, 1985) show the North Tahoe fault located approximately ½ mile east of the site and oriented in a north-south direction. The Nevada Earthquake Safety Council (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 considered a building, as defined by the *IBC, which is expected to have a human occupancy rate of more than 2,000 hours per year* (ICC, 2012).

The North Tahoe fault mapped in the general area of the project site is considered a *Late Quaternary Active Fault*. Because no fault is mapped as passing through or adjacent to the project, no additional fault investigation or setbacks are necessary for this project.

Ground Motion and Liquefaction

Mapping by the United States Geological Survey (USGS, 2018) indicates that there is a 2 percent probability that a *bedrock* ground acceleration of 0.66 g 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 a thin cover of soils and bedrock at shallow depths, liquefaction is not possible.

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. No other geologic hazards were identified.

Discussion and Recommendations

General Information

The overall Boulder Bay project involves the complete redevelopment of the current Tahoe Biltmore property with a mixed-use development consisting of 8 separate buildings and a 3-story parking structure. As part of the project, several of the surrounding roadways will be reconfigured and reconstructed. The first phase of the project is in progress and includes the design and construction of Building A, which is currently under construction. The second phase consists of the necessary roadway realignment and reconstruction. Subsequent phases following the roadway work will consist of constructing the remaining buildings. This report pertains to the design and construction of Buildings B, C, D, and the proposed 3-story parking structure.

The recommendations provided herein, and particularly under **Site Preparation, Mass Grading, Foundation, Retaining Walls,** and **Quality Control**, are intended to minimize risks of structural distress related to consolidation 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 is 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 and asphalt pavements, as well as pads for any minor structures. The term engineer, as presented below, pertains to the civil or geological engineer that has prepared the geotechnical engineering report for the project or who serves as a qualified geotechnical professional on behalf of the owner.

All compaction requirements presented in this report are relative to ASTM D 1557.

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.

Site Preparation

All vegetation shall be stripped and grubbed from structural areas and removed from the site. A stripping depth of 0.5 feet is anticipated in portions of the site. Large trees and associated roots greater than ½ inch in diameter shall be removed, where necessary, to a minimum depth of 12 inches below finished grade. Large roots (greater than 6 inches in diameter) shall be removed to the maximum depth possible. Vegetation and topsoil should be hauled

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off site or stockpiled for use in landscaping areas. Resulting excavations shall be backfilled with structural fill compacted to 90 percent relative compaction.

The project will include demolition of various existing site improvements. Where needed, the existing pavement shall be removed either by pulverizing or simply by heavy equipment. Pulverized or recycled asphalt pavement may be reused as structural fill or aggregate base. Remnants from demolition activities should be removed from the site, including all existing foundation elements and slabs. Demolition of existing improvements should include rerouting, removal, or in-place abandonment of underground utilities. Utilities should be adequately capped or rerouted at the project perimeter in accordance with the requirements of the governing agencies. Abandoned underground utility pipes should be removed from the site or, if the pipes are left in place, they should be filled with flowable fill such as grout or controlled low-strength material. The contractor should take adequate precautions when grading the site to reduce the potential for damage to existing utilities that are to remain in service.

All soils areas to receive structural fill or structural loading shall be densified to at least 90 percent relative compaction. Bedrock shall be cleaned as much as practical to remove loose materials. Asphalt concrete pavement areas that will receive structural fill may be removed by heavy equipment or broken up utilizing a large sheeps-foot roller.

Trenching, Excavation and Utility Backfill

Excavation Characteristics

The site is overlain by a relatively thin layer of native overburden soils with areas of granular fill derived from native materials. Granitic bedrock underlies the entire area at shallow depths and was encountered in our borings at depths ranging from 2 to 5.5 feet below the existing ground surface. The overburden soils and any fill materials will be excavatable using conventional earthmoving equipment. The granitic bedrock exhibits varying degrees of weathering and is generally moderately weathered to decomposed in the upper 20 feet and fresh to slightly weathered below 20 feet. However, it should be understood that bedrock can always contain isolated, very hard corestones at any depth. The excavation rate will be slow within the granitic bedrock, and the use of aggressive excavation techniques such as single-shank rippers, hydraulic hammers, or other rock breaking equipment may be needed to achieve proposed site grades. In general, the deeper the excavations advance into bedrock, the more difficult excavation will become.

Table 1 (Shear Wave Velocity Results) identifies the calculated seismic velocities based on the measured average shear wave velocity survey conducted throughout the overall Boulder Bay site. The seismic velocity values can be correlated to published rippability charts (Caterpillar, 2012); rippability charts for CAT© D8 and D9 bulldozers are included in Appendix C (Rippability Charts). The published rippability charts do not take into account efficiency or resulting particle size of ripped bedrock material. Based on our site exploration and the shear wave velocity results, the site bedrock will be rippable using a CAT© D8 ripping dozer with a single shank to depths of up to 20 feet. Use of larger equipment, such as a CAT© D9 or larger, will result in more reliable ripping production and will be needed when excavations extend deeper than 20 feet. Again, harder bedrock will require more aggressive techniques.

TABLE 1 – SHEAR WAVE VELOCITY RESULTS		
Line Number/ Location	**Shear Wave Velocity (fps[1])**	**P-Wave Velocity (fps)**
1	2,045	5,113
2	1,590	3,975
3	1,540	3,850
4	1,390	3,475

[1] Average shear wave velocity within 100 feet depth. FPS = feet per second. Refer to Appendix A for detailed shear wave velocity analysis results.

Temporary trenches with near-vertical sidewalls should be stable to a depth of approximately 4 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 (2010) require that temporary sidewall slopes be no greater than those presented in Table 2 (Maximum Allowable Temporary Slopes).

TABLE 2 - 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 overburden soils and fill materials are considered Type C. The granitic bedrock is generally Type A with areas of stable rock at depth. Any soil areas in question shall be considered Type C, and any bedrock areas in question shall be considered Type B, unless specifically examined by the engineer during construction. All trenching shall be performed and stabilized in accordance with local, state, and OSHA standards.

Utility Trench Backfill

The maximum particle size in trench backfill shall be 4 inches. Bedding and initial backfill 12 inches over the pipe will require import and shall conform to the requirements of the utility having jurisdiction. Bedding and initial backfill shall be densified to at least 90 percent relative compaction. Native granular soil and excavated bedrock will provide adequate final backfill as long as oversized particles are excluded, and it shall be placed in maximum 8-inch-thick loose lifts that are compacted to a minimum of 90 percent relative compaction in all structural areas.

Construction Dewatering

Groundwater was not encountered in our borings, and the static groundwater table is expected to be at a depth well below that which would affect construction. However, if construction occurs during the spring snowmelt season, perched seepage water flowing along the soil and bedrock interface and possible fracture systems may be encountered during excavation, such that construction dewatering may be necessary. If significant seepage water is encountered during earthwork, we should be contacted to provide site-specific recommendations based on the observed conditions.

Mass Grading

Vertical relief across the site is high, and the buildings are anticipated to have below-grade levels. We expect deep cuts in excess of 30 feet and potentially up to 40 feet within the underlying granitic bedrock will be needed to achieve design grades. The proposed finished floor elevations and approximate existing elevations are listed in Table 3 (Proposed Finished Floor Elevations and Expected Excavation Depths).

TABLE 3 – PROPOSED FINISHED FLOOR ELEVATIONS AND EXPECTED EXCAVATION DEPTHS			
Building Name/Option	**Lowest Finished Floor Elevation***	**Approximate Ground Surface Elevations***	**Expected Excavation Depths (ft)**
B – Option 1	6,465	6,468 – 6,485	3 – 25
B – Option 2	6,445	6,468 – 6,485	25 – 45
C	6,422	6,445 – 6,453	28 – 36
D	6,422	6,440 – 6,452	23 – 35
Parking Structure	6,422	6,440 – 6,450	23 - 33
* Elevations in feet above msl.			

Native granular soils and excavated bedrock will be suitable for structural fill provided particles larger than 6 inches are removed. If imported structural fill is required on this project, we recommend it satisfy the specifications presented in 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 – 20	
Percent Passing No. 200 Sieve	**Maximum Liquid Limit**	**Maximum Plastic Index**
5 – 10	50	20
11 – 20	40	15

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 engineer, in writing, prior to importing fill to the site.

All fill placed on hillsides steeper than 5H:1V (horizontal to vertical) shall be keyed into existing materials in equipment-wide benches. The maximum vertical separation between benches shall be 6 feet.

Whenever possible, structure foundations shall not be placed partially on bedrock and partially on structural fill. Where structure foundations will be placed partially on bedrock and partially on structural fill due to cut and fill operations, differential settlement of the structural fill may be on the order of 1 percent of the maximum fill height, which would result in differential settlement of structure foundations. Such differential settlement should be minimized. Measures to minimize such differential settlement may include providing a gradual transition from the bedrock to structural fill and/or over-excavating a portion of the bedrock and backfilling with structural fill.

Excavated fresh to slightly weathered granitic bedrock materials may not break down into soil-sized particles under the mechanics of ripping, loading, transportation, placement and/or compaction. Such materials may have greater than 30 percent retained on the ¾-inch sieve, such that standard density testing is not valid. These materials will be treated as rock fills with a maximum lift thickness and maximum particle size of 12 inches. A proof rolling program of at least 5 single passes of a minimum CAT® 815 roller in mass grading, or at least 5 complete passes with hand compactors in footing trenches, is recommended.

Any structural fill within building areas shall be placed in maximum 8-inch-thick loose lifts, each densified to at least 95 percent relative compaction. All other structural fill shall be densified to a minimum 90 percent relative compaction.

Grading shall not be performed with or on frozen soils.

Seismic Design Parameters

The 2012 *IBC* (ICC, 2012), adopted by Washoe County, requires a detailed soils evaluation to a depth of 100 feet to develop appropriate soils criteria. Site-specific geophysical analyses were performed and indicate that the subsurface materials exhibit shear wave velocities in the range of 1,390 to 2,045 feet per second. The results of the geophysical analyses performed at the site indicate that Site Class C is appropriate. The recommended seismic design criteria are presented in Table 5 (Seismic Design Criteria Using 2012 *International Building Code*).

TABLE 5 - SEISMIC DESIGN CRITERIA USING 2012 *INTERNATIONAL BUILDING CODE* (USGS, 2018)	
Approximate Latitude	39.229
Approximate Longitude	120.005
Spectral Response at Short Periods, S_s, percent of gravity	166.1
Spectral Response at 1-Second Period, S_1, percent of gravity	57.0
Site Class	C
Risk Category	II
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	166.1
Site Adjusted Spectral Response at Long Periods, S_{M1}, percent of gravity	74.1
Design Spectral Response at Short Periods, S_{DS}, percent of gravity	110.7
Design Spectral Response at Long Periods, S_{D1}, percent of gravity	49.4
Seismic Design Category	D

Foundation

The most economical method of foundation support lies in spread footings bearing on structural fill or granitic bedrock. Individual column footings and continuous wall footings underlain by compacted native soils or structural fill can be designed for a net maximum allowable bearing pressure of 3,500 pounds per square foot (psf). Based on the proposed finished floor elevations, the majority of foundations are anticipated be at elevations in excess of 20 feet below the existing ground surface. Individual column footings and continuous wall footings underlain by competent granitic bedrock can be designed for a net maximum allowable bearing pressure ranging from 5,000 psf for foundations less than 20 feet below the existing ground surface to 8,000 psf for foundations constructed at depths greater than 20 feet below the existing ground surface.

Column and wall footings should have minimum footing widths of 30 and 18 inches, respectively. 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 that will be present during the majority of the life of the structure. Design live loads are loads that are produced by the use and occupancy of the building, such as by moveable objects, including people or equipment, as well as snow loads. These bearing values 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 these allowable bearing pressures, total foundation movements of approximately ¾ inch should be anticipated for foundations supported on native soils or structural fill. Foundations bearing on granitic bedrock will experience negligible settlement. Differential movement between footings with similar loads, dimensions, and base elevations should not exceed two-thirds of the values provided above for total movements. 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 0.5 for soils and structural fill and 0.6 for granitic bedrock. These values have been reduced by a factor of 1.5 on the ultimate soil strength. Design values for active and passive equivalent fluid pressures are 34 and 400 psf per foot of depth, respectively, for spread footings bearing on compacted native soils or structural fill and backfilled with structural fill. Design values for active and passive equivalent fluid pressures bearing on bedrock and placed against undisturbed granitic bedrock are 25 and 600 psf per foot of depth, respectively. All exterior footings should be placed a minimum 2 feet below adjacent finished grade for frost protection.

If loose, soft, wet, or disturbed soils are encountered at the foundation subgrade, these soils should be removed to expose undisturbed granular soils or granitic bedrock 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.

Retaining Walls

Based on the existing topography and the proposed lower level finished floor elevations of the buildings, we anticipate the project will incorporate multiple retaining walls which are likely to include both site retaining walls and below-grade basement retaining walls for portions of each of the buildings. We assume site retaining walls may include some combination of shallowly founded, flexible-type retaining walls, such as mechanically stabilized earth walls or gravity block walls, or rigid cast-in-place PCC walls. Below-grade basement walls are likely to be rigid cast-in-place PCC walls, potentially in combination with a reinforced excavation utilizing soil nails or rock bolts and shotcrete. Specialized contractors are readily available for design/build of any needed specialized walls. Black Eagle Consulting, Inc. can coordinate with these contractors as well as provide special inspection as desired.

Retaining Wall Design Parameters

Table 6 (Lateral Earth Pressure Values [Equivalent Fluid Density]) provides design parameters for fully drained retaining walls with vertical back faces, horizontal backfill, and no surcharge loads next to the top of the wall. Recommendations for retaining wall drainage are provided in the Retaining Wall Drainage Design section. Surcharge loads, including construction and traffic loads, should be added to the following values. While the recommendations here may be suitable for other conditions, we should be consulted for retaining walls with unusual conditions such as sloping backfill (steeper than provided in Table 6), sloping retaining walls, or the presence of hydrostatic pressure. We should also be consulted where retaining walls exceed 20 feet in height. It is noted that the Table 6 parameters assume temporary excavations into soil and bedrock at typical slopes and backfilling with retaining wall backfill. These values are conservative for retaining walls in the area where cut is to be made into competent bedrock at a steeper ratio or where permanent shoring systems are to be used. Depending on the final design and retaining wall configurations, BEC can provide reduced, appropriate earth pressure values when requested as a separate scope of work.

TABLE 6 - LATERAL EARTH PRESSURE VALUES (EQUIVALENT FLUID DENSITY), pcf[1]

Retained Slope	Static		Dynamic	
	Active*	Passive**	Active*	Passive**
Level	31	150	50	230
3H:1V	39	NA[2]	77	NA[2]

[1]Pounds per Cubic Foot
[2]No sloping ground considered on passive side. Use values for level ground.
*For walls that are free to yield at least 0.2 percent of the wall height.
**The values presented have been reduced from the ultimate passive resistance values by 67 and 50 percent to limit deflection under static and dynamic conditions, respectively.

Restrained walls should be designed to resist an at-rest equivalent fluid density (static value) of 55 pounds per cubic foot.

The allowable bearing pressure values for retaining wall foundations are provided above in the Foundation section. Lateral loads will be resisted by friction along the base of retaining wall footings and by passive resistance against buried foundation walls. Foundation wall footings cast directly on compacted native soils or structural fill can be designed using a coefficient of base friction of 0.5. Retaining wall footings cast directly on competent bedrock can be designed using a coefficient of base friction of 0.6. These values have been reduced by a factor of 1.5 on the ultimate soil strength.

Retaining Wall Drainage Design

For cast-in-place PCC and gravity walls, subsurface foundation drainage must be installed along the retaining wall foundations. The wall foundation drainage system for these walls may be accomplished by placing a non-woven geotextile/gravel system with a network of perforated drain pipes below and along the outside base of the footings. The geotextile shall meet or exceed the minimum properties presented in Table 7 (Minimum Required Properties for Drainage Geotextile).

TABLE 7 - MINIMUM REQUIRED PROPERTIES FOR DRAINAGE GEOTEXTILE	
Grab Tensile (ASTM D 4632)	90 lbs.
Puncture Strength (ASTM D 4833)	50 lbs.
Burst Strength (ASTM D 3786)	150 psi.

A trench shall be excavated to a depth of at least 6 inches below the base and directly adjacent to the outside of the footings. A perforated, 4-inch-diameter drain pipe shall be placed in the bottom of the trench and graded to drain downslope. A minimum of 12 inches of Class C drain rock (*Standard Specifications for Public Works Construction* [*SSPWC*], 2012) shall be placed above the drain pipe and around the footing, then covered by the geotextile.

All retaining walls should have an appropriate drainage system to reduce accumulation of water and development of pore water pressure unless the walls are designed to resist hydrostatic pressure. Retaining wall drainage for site retaining walls can be accomplished by installing granular backfill and a weep hole drain system at the bottom of the wall (or a prefabricated drain system discussed below, if preferred). The drain rock section shall be a minimum of 18 inches wide and extend to within 12 inches of finished grade. A drainage geotextile (Table 7) shall be placed between the drain rock backfill and the native soils to prevent migration of fines into the drain rock. The drainage geotextile may be eliminated where retaining walls are constructed against bedrock and the backfill is to include entirely drain rock.

Retaining wall drainage for below-grade building walls shall include a drain section discussed above or the installation of a prefabricated drain system that is hydraulically connected to the foundation drain system. A prefabricated drain system consists of a three-dimensional mesh or waffle structure with a geotextile on one side, such as Mirafi® *Miradrain G100N*, that is fastened to the back side of the wall with the geotextile side facing the backfill. The prefabricated drain mat connects at the bottom of the wall either to a drain pipe or empties into drain rock backfill wrapped in a geotextile at the base of the wall that then drains downslope of the structure to a storm drain or to one or more sump locations from which collected water can be pumped into a storm drain.

A concrete interceptor swale or properly designed rock-lined swale shall be included at the backfill surface to direct runoff away from the wall.

Snow storage locations on the project site should be restricted to paved areas where positive surface drainage is maintained. Snow should not be stored above the retained zone of the retaining walls.

Retaining Wall Backfill

Native soils and excavated bedrock can be used as wall backfill provided particles larger than 4 inches are removed. Backfill behind retaining walls shall be compacted to 90 percent of the material's maximum dry density in accordance with ASTM D 1557, but it shall not be densified to more than approximately 92 percent relative

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density to minimize pressure against the walls. Care must be exercised when compacting backfill against retaining walls and foundations. To reduce temporary construction loads on the walls, heavy equipment shall not be used for placing and compacting fill within a region as determined by a 0.5H:1V line drawn upward from the bottom of the wall, or within 3 feet of the wall, whichever is greater. We recommend that hand-operated compaction equipment be used to compact soils adjacent to retaining walls.

Where structural improvements (e.g., sidewalks, drives, etc.) are to be located above retaining wall backfill, it is critically important that compaction of these materials be diligently tested and inspected to minimize any undesirable differential movement.

Waterproofing Walls

Cast-in-place PCC walls should be waterproofed in accordance with the recommendations of the project structural engineer. To reduce the potential for water- and sulfate/salt-related damage or efflorescence to the retaining walls, particular care should be taken in selection of the appropriate type of waterproofing material to be utilized and in the application of this material. Any cold joints, such as between footings and walls, should be waterproofed with an appropriate, highly durable sealant. Basement seepage is extremely difficult and costly to repair; therefore, the wall drainage and waterproofing systems for basement retaining walls of the buildings must be well-designed and properly installed.

Retaining Wall Backfill Settlement

We anticipate retaining walls up to 20 feet tall will be constructed. In general, the compacted backfill could undergo internal consolidation of about one half a percent of the fill depth. This internal consolidation of compacted backfill could be significant for wall backfill in excess of 10 feet. The settlement associated with internal consolidation of compacted backfill 20 feet thick could be on the order of 1 to 1.5 inches. This level of settlement may adversely impact any structural improvements founded on the wall backfill (e.g., pavements and flatwork). With the use of granular structural fill, we anticipate the majority of internal consolidation of backfill soils will be complete about 30 days after fill placement. We recommend improvements such as exterior flatwork constructed over backfill zones be minimized as much as possible or alternatively constructed to span across the backfill zone. At an absolute minimum, all structural improvements that are to be founded on backfill of 10 feet or more shall be delayed a minimum of 30 days after completion of backfill placement. The project schedule shall incorporate this required time delay.

Subsidence and Shrinkage

Subsidence of native soils or granitic bedrock exposed in cut should be negligible. On-site soils excavated and recompacted in structural fills should experience quantity shrinkage of approximately 10 percent, including removal of oversized particles. In other words, 1 cubic yard of excavated granular alluvium will generate about 0.9 cubic yards of structural fill at 95 percent relative compaction. The quantity of shrinkage/swell of granitic bedrock materials is difficult to predict and will vary depending on the degree of weathering and the presence of hard, oversized rocks within the generally weathered bedrock. Considering a low percentage of oversized particles, we expect the quantity of shrinkage/swell of the on-site, generally weathered granitic bedrock will vary between 5 percent shrinkage to 5 percent swell.

Slope Stability and Erosion Control

Stability of cut and filled surfaces involves 2 separate aspects. The first concerns true slope stability related to mass wasting, landslides, or the en masse downward movement of soil or rock. Stability of cut and fill slopes is dependent upon shear strength, unit weight, moisture content, and slope angle. The *IBC* (ICC, 2012), adopted by Washoe County, allows cut and fill slopes up to 2H:1V in the type of soils present at this site. The exploration and testing program conducted during this investigation confirms 2H:1V slopes will be stable at the site. Steeper slopes will be allowed in competent granitic bedrock but should be evaluated on a case-by-case basis. Once final design details become available, BEC can perform location-specific slope stability analyses to evaluate any steeper slopes when requested.

The second aspect of stability involves erosion potential and is dependent on numerous factors involving grain size distribution, cohesion, moisture content, slope angle, and the velocity of water or wind on the ground surface. We recommend erosion control of cut and fill for soil slopes that are 5H:1V or steeper. Soil slopes between 3H:1V and 5H:1V can be stabilized by hydroseeding. Soil slopes steeper than 3H:1V require mechanical stabilization with such alternatives as rock rip-rap or erosion control matting. The shallow, weathered granitic bedrock at the site may also need to be considered soil-like material depending on the severity of weathering and the potential for erosion. Erosion protection is not necessary for cut slopes made into competent granitic bedrock; however, such rock is generally only present at depth within the site.

Dust potential at this site will be moderate during dry periods. Temporary (during construction) and permanent (after construction) erosion control will be required for all disturbed areas. 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.

In order to minimize erosion and downstream impacts to sedimentation from this site, best management practices with respect to stormwater discharge shall be implemented.

Concrete Slabs

All concrete slabs shall be directly underlain by at least 4 inches of imported Type 2, Class B aggregate base (*SSPWC*, 2012). Aggregate base courses shall be densified to at least 95 percent relative compaction.

Final design of the building floor slab (both thickness and reinforcement) shall be performed by the project structural engineer. Coefficient of subgrade reaction (K-value) values of 200 and 350 pounds per cubic inch are appropriate for use in design of concrete slabs founded on compacted soil/structural fill and granitic bedrock, respectively. Any interior concrete slab-on-grade floors shall be a minimum of 4 inches thick. Floor slab reinforcement, as a minimum, shall consist of No. 3 reinforcing steel placed on 24-inch centers in each direction, or flat sheets of 6x6, W4.0xW4.0 welded wire mesh (WWM). Rolls of WWM are not recommended for use because vertically centered placement of rolled WWM within a floor slab is difficult to achieve. All reinforcing steel and WWM shall be centered in the floor slab through the use of concrete dobies or an approved equivalent

Valley gutters shall include at least 6 inches of fibermesh concrete (4,000 pounds per square inch [psi]). These exterior rigid pavements have been designed using the American Association of State Highway and Transportation Officials (1993) method for concrete with a 28-day flexural strength of 570 psi (approximately 4,000 psi compressive strength).

The Crystal Bay area is a region with 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 (2008) and this report. Special considerations shall 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 Crystal Bay area.

Proper control joints and reinforcement shall be provided to minimize any damage resulting from shrinkage, as discussed below. In particular, crack-control joints shall be installed on maximum 10-foot centers and shall be installed to a minimum depth of 25 percent of the slab thickness. Saw-cuts, zip strips, and/or trowel joints are acceptable; however, saw-cut joints must be installed as soon as initial set allows and prior to the development of internal stresses that will result in a random crack pattern. If trowel joints are used, they will need to be grouted over prior to installation of floor coverings.

Concrete shall not be placed on frozen in-place soils.

Any interior concrete slab-on-grade floors will require a moisture barrier system. Installation shall conform to the specifications provided for a Class B vapor restraint (ASTM E 1745-97). The vapor barrier shall consist of placing a

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10-mil-thick Stego® Wrap Vapor Barrier or an approved equal directly on a properly prepared subgrade surface. A 4-inch-thick layer of aggregate base shall be placed over the vapor barrier and compacted with a vibratory plate.

The base layer that overlies the moisture barrier membrane shall remain compacted and a uniform thickness maintained during the concrete pour, as its intended purpose is to facilitate even curing of the concrete and minimize curling of the slab. Extra attention shall be given during construction to ensure that rebar reinforcement and equipment do not damage the integrity of the vapor barrier. Care must be taken so that concrete discharge does not scour the base material from the vapor barrier. This can be accomplished by maintaining the discharge hose in the concrete and allowing the concrete to flow out over the base layer.

Site Drainage

The collection and diversion of surface and subsurface water away from buildings, paved areas, and retaining walls is vital to satisfactory performance of this project. The subsurface and surface drainage systems should be carefully designed to facilitate removal of water from structures and paved areas. Allowing surface water to pond on or adjacent to pavements will cause premature pavement deterioration. Permitting increases in moisture to the building supporting soils may result in a decrease in bearing capacity and an increase in settlement and/or differential movement. Surface drainage should be intercepted by drainage ditches and curbs and gutters and directed toward a suitable outlet. As previously discussed, seasonal snowmelt runoff will produce perched water conditions through the sloping topography along the soil and bedrock interface and may be compounded in cut slopes. Additionally, the construction process itself may compound seepage in areas of cut and could necessitate implementation of adequate drainage controls to prevent the saturation of subgrade and foundation bearing soils. Additional drainage measures will be necessary for retaining structures, as discussed in the **Retaining Wall Drainage Design** section of this report.

Asphalt Concrete

Asphalt Concrete Pavement Design

Specific traffic loadings for the project were not available for our analysis; however, we assume the project pavements constructed as part of the building phase will experience relatively light traffic. Paved areas subject to truck traffic shall consist of 4 inches of asphalt concrete underlain by 6 inches of Type 2, Class B aggregate base (*SSPWC*, 2012). Paved areas restricted to automobile parking can consist of 3 inches of asphalt concrete underlain by 6 inches of aggregate base. All aggregate base beneath asphalt pavements shall be densified to at least 95 percent relative compaction.

Pavement Maintenance

Asphalt concrete pavements have been designed for a standard 20-year life expectancy as detailed above. Due to the local climate and available construction aggregates, a 20-year performance life requires diligent maintenance.

Between 15 and 20 years after initial construction (average 17 years), major rehabilitation (structural overlay or reconstruction) is often necessary if maintenance has been lax. To achieve maximum performance life, maintenance must include regular crack sealing, seal coats, and patching as needed. Crack filling is commonly necessary every year or at least every other year. Seal coats, typically with a Type II slurry seal, are generally needed every 3 to 6 years depending on surface wear. Failure to provide thorough maintenance will significantly reduce pavement design life and performance.

Corrosion Potential

Metal Pipe Design Parameters

Laboratory testing was performed to evaluate the corrosion potential of the soils with respect to metal pipe in contact with the ground. The results of the laboratory testing indicate that the site soils are not corrosive to buried metal (American Water Works Association, 1999). As a result, metal pipe in contact with the ground will not require corrosion protection.

Portland Cement Concrete Mix Design Parameters

Soluble sulfate content has been determined for representative samples of the site foundation soils. The sulfate was extracted from the soil at a 10:1 water to soil ratio in order to assure that all soluble sodium sulfate was dissolved. The results are reported in milligrams of sulfate per kilogram of soil and can be directly converted to percent by dividing by 10,000. The percent sulfate in the soil is used to determine the sulfate exposure Class (S) from the information presented in Table 8 (Sulfate Exposure Class).

TABLE 8 - SULFATE EXPOSURE CLASS*

			Water-Soluble Sulfate (SO_4) in Soil, Percent by Weight
S Sulfate	Not Applicable	S0	$SO_4 < 0.10$
	Moderate	S1	$0.10 \leq SO_4 < 0.20$
	Severe	S2	$0.20 \leq SO_4 \leq 2.00$
	Very Severe	S3	$SO_4 > 2.00$

*From Table 4.2.1 Exposure Categories and Classes. ACI 318, *Buildings Code and Comments.*

The results of the testing (Appendix B) indicate that concrete in contact with the site foundation soils should be designed for Class S0 Sulfate exposure. Therefore, Type II cement can be used for all concrete work.

Anticipated Construction Problems

Excavations into slopes during the spring snowmelt season may encounter significant perched groundwater resulting in seepage that may affect construction of the project and require dewatering. Difficult excavation is likely within granitic bedrock, particularly within excavations deeper than 20 feet; these conditions may necessitate the utilization of aggressive excavation and trenching techniques.

Quality Control

All plans and specifications should be reviewed for conformance with this geotechnical report and approved by the 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, general contractor, earthwork and materials subcontractors, building official, and 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 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. 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.

Static groundwater was not encountered in our exploration. However, seasonal snowmelt runoff will produce perched water conditions, as discussed within this report. Construction planning should be based on the assumption of the possibility of encountering perched water.

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 engineer. If the 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 engineer makes no other warranties, either express or implied, as to the professional advice provided under the terms of this agreement and included in this report.

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PLATES

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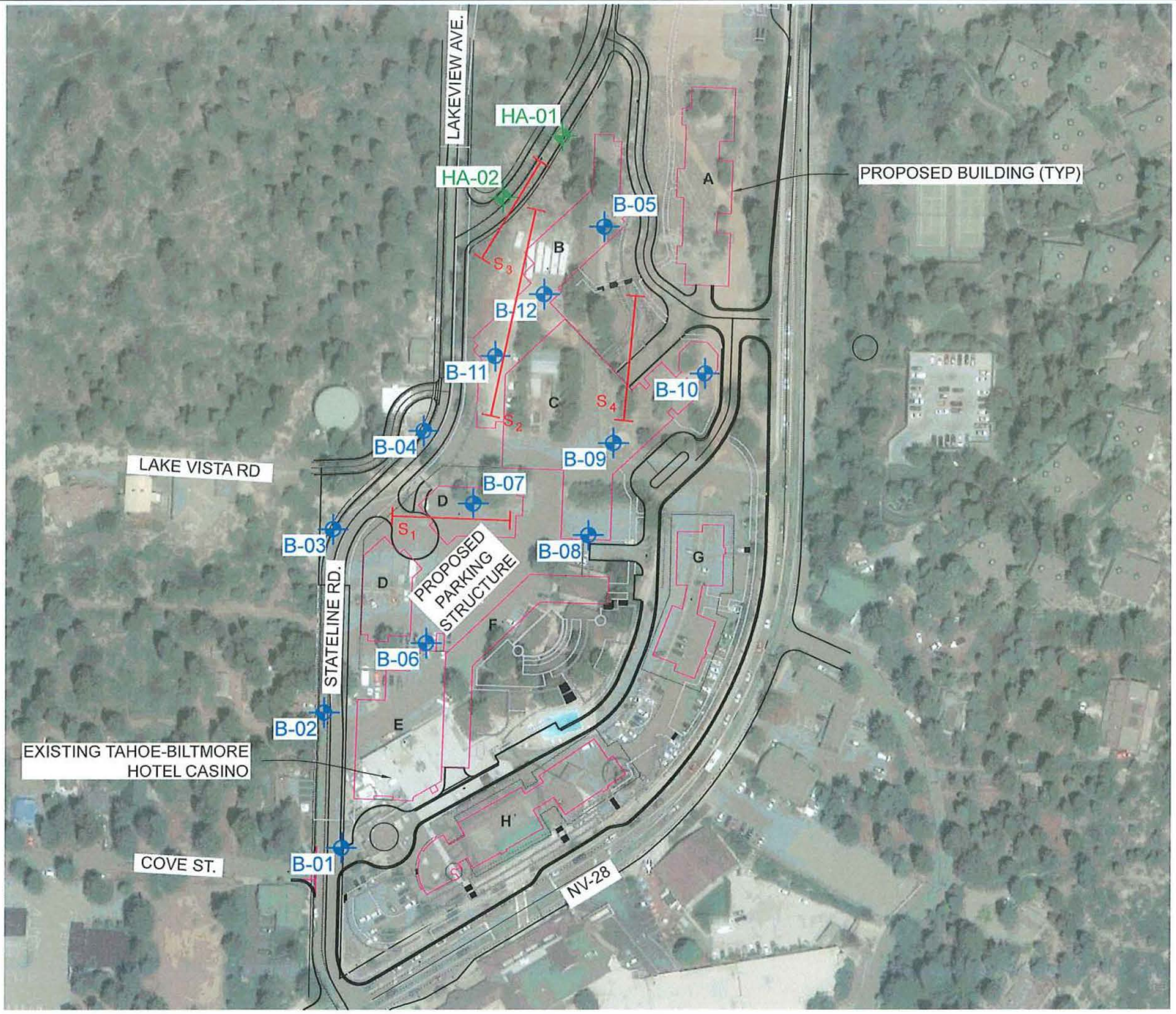

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OVERALL SCALE: 1"=150'

LEGEND

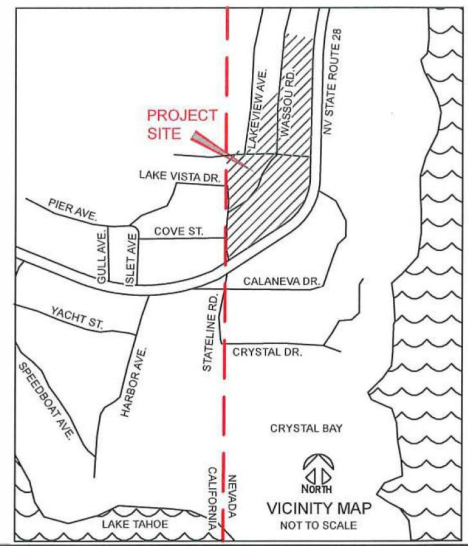
B-01 APPROXIMATE BORING LOCATION

HA-01 APPROXIMATE HAND AUGER LOCATION

S_1 APROXIMATE SEAR WAVE VELOCITY SURVEY LINE

NOTES

1. BASE MAP PROVIDED BY LUMOS & ASSOCIATES.



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CFA, INC.

PLOT PLAN

BOULDER BAY - BUILDINGS B,C,D AND PARKING STRUCTURE

WASHOE COUNTY, NEVADA

Project No.
0091-52-1

Plate 1

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BORING LOG

BORING NO.:	B-01	DATE:	4/11/2018
TYPE OF RIG:	CME 55 Track	DEPTH TO GROUND WATER (ft):	NE
LOGGED BY:	JP	GROUND ELEVATION (ft):	NA

SAMPLE NO.	SAMPLE TYPE	BLOWS/12 inches	MOISTURE (%)	PLASTICITY INDEX	DEPTH (ft)	USCS SYMBOL	LITHOLOGY	DESCRIPTION
					0			**Asphalt Concrete** An approximate 3.75-inch-thick layer of asphalt concrete pavement. No aggregate base.
A	SPT	12				SM		**Silty Sand** Brown, moist, medium dense, with 18% non-plastic fines and 82% fine to medium sand.
B	SPT	17	11.9	NP	2–4	SM		Includes friable granitic clasts.
C	SPT	76			6	SM		**Weathered Granite** Granitic bedrock weathered to decomposed granite. Rock fabric intact and recognizable. Breaks down to **Silty Sand** during auger drilling and in SPT samples. Light gray, moist, very dense, with an estimated 15% non-plastic fines and 85% fine to coarse sand.

Depth scale (ft): 2, 4, 6, 8, 10, 12, 14, 16, 18, 20

6-inch diameter solid stem auger (SSA).
N 4346335 E 758474 UTM NAD83

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CFA, Inc.
Boulder Bay
Washoe County, Nevada

PROJECT NO.: 0091-52-1

PLATE: 2

BORING_LOG 0091521.GPJ BLKEAGLE.GDT 6/20/2018

BORING LOG

BORING NO.: B-02 DATE: 4/11/2018

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

LOGGED BY: JP GROUND ELEVATION (ft): NA

SAMPLE NO.	SAMPLE TYPE	BLOWS/12 inches	MOISTURE (%)	PLASTICITY INDEX	DEPTH (ft)	USCS SYMBOL	DESCRIPTION
							Asphalt Concrete An approximate 3.75-inch-thick layer of asphalt concrete pavement.
						GW	**Aggregate Base** An approximate 4-inch-thick layer of aggregate base.
A	SPT	20				SM	**Silty Sand with Gravel** Brown, moist, medium dense to very dense, with an estimated 15% non-plastic fines, 55-65% fine to coarse sand, and 20-30% subrounded gravel up to 1.25 inches in diameter. Includes friable granitic clasts.
B	SPT	55/6"			2		
C	SPT	76			4		
					6	SM	**Weathered Granite** Granitic bedrock weathered to decomposed granite. Rock fabric intact and recognizable. Breaks down to **Silty Sand with Gravel** during auger drilling and in SPT samples. Light gray, moist, very dense, with an estimated 15% non-plastic fines, 55% fine to coarse sand, and 30% angular gravel up to 1.25 inches in diameter. Increasing strength with depth.
					8		
D	SPT	30/1"			10		Auger refusal in hard rock materials.
					12		
					14		
					16		
					18		
					20		

6-inch diameter SSA to 5 feet, 4-inch diameter SSA to 10 feet.
N 4346410 E 758485 UTM NAD83

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PLATE: 2

SHEET 1 OF 1

BORING_LOG 0091521.GPJ BLKEAGLE.GDT 6/20/2018

BORING LOG

BORING NO.: B-03 | DATE: 4/11/2018

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

LOGGED BY: JP | GROUND ELEVATION (ft): NA

SAMPLE NO.	SAMPLE TYPE	BLOWS/12 inches	MOISTURE (%)	PLASTICITY INDEX	DEPTH (ft)	USCS SYMBOL	LITHOLOGY	DESCRIPTION
								Asphalt Concrete An approximate 3.75-inch-thick layer of asphalt concrete pavement. No aggregate base.
	GRAB		11.1	NP	2	SM		**Silty Sand** Brown, moist, medium dense, with 26% non-plastic fines, 67% fine to coarse sand, and 7% subrounded gravel up to 1.25 inches in diameter.
A	SPT	34						Includes friable granitic clasts.
					4			**Weathered Granite** Granitic bedrock weathered to decomposed granite, friable. Rock fabric intact and recognizable. Breaks down to **Silty Sand** during auger drilling and in SPT samples. Light gray, moist, dense to very dense, with an estimated 20% non-plastic fines and 80% fine to coarse sand.
B	SPT	37			6	SM		
C	SPT	52			8			
D	MC	50/3.5"			10			**Granite** Granitic bedrock, slight to moderate weathering, friable to weak. Breaks down to **Silty Sand** during auger drilling and in SPT samples. Light gray, moist, very dense, with an estimated 20% non-plastic fines and 80% fine to coarse sand.
					12			
					14			
E	SPT	50/3"			16			
					18			
F	MC	65/1"			20	SM		

6-inch diameter SSA to 5 feet, 4-inch diameter SSA to 30 feet.
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PROJECT NO.: 0091-52-1

PLATE: 2

SHEET 1 OF 2

BORING LOG

BORING NO.: B-03 | DATE: 4/11/2018

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

LOGGED BY: JP | GROUND ELEVATION (ft): NA

SAMPLE NO.	SAMPLE TYPE	BLOWS/12 inches	MOISTURE (%)	PLASTICITY INDEX	DEPTH (ft)	USCS SYMBOL	LITHOLOGY	DESCRIPTION
					24			
G	SPT	30/0"						
					26			
						SM		Very hard drilling at 27 feet beneath the ground surface (bgs).
					28			
H	SPT	50/1"			30			
					32			
					34			
					36			
					38			
					40			
					42			

6-inch diameter SSA to 5 feet, 4-inch diameter SSA to 30 feet.
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PLATE: 2

BORING_LOG 0091521.GPJ BLKEAGLE.GDT 6/20/2018

BORING LOG

BORING NO.:	B-04	DATE:	4/12/2018
TYPE OF RIG:	Diedrich D-120	DEPTH TO GROUND WATER (ft):	NE
LOGGED BY:	JP	GROUND ELEVATION (ft):	NA

SAMPLE NO.	SAMPLE TYPE	BLOWS/12 inches	MOISTURE (%)	PLASTICITY INDEX	DEPTH (ft)	USCS SYMBOL	LITHOLOGY	DESCRIPTION
					0 – 0.2			**Asphalt Pavement** An approximate 2.25-inch-thick layer of asphalt concrete pavement.
					2	SM		**Silty Sand (Fill)** Brown, very moist, loose, with an estimated 20% non-plastic fines, 75% fine to coarse sand, and 5% subrounded gravel up to 3/4 inch in diameter.
A	SPT	6						Includes friable granitic clasts.
					4			**Weathered Granite** Granitic bedrock weathered to decomposed granite, friable. Rock fabric intact and recognizable. Breaks down to **Silty Sand** during auger drilling and in SPT samples. Brown to light gray, moist, medium dense, with 26% non-plastic fines and 74% fine to coarse sand.
B	MC	35	10.4	NP	6	SM		Includes friable granitic clasts.
C	SPT	64			8			**Granite** Granitic bedrock, slight to moderate weathering, friable to moderately strong. Breaks down to **Silty Sand** during auger drilling and in SPT samples. Light gray, moist, very dense, with an estimated 15% non-plastic fines and 85% fine to coarse sand.
D	MC	65/4"			10			
					12			
					14	SM		
E	SPT	50/3.5"			16			
					18			
F	SPT	50/5.5"			20			

6-inch diameter SSA to 5 feet, 4-inch diameter SSA to 20 feet.
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PLATE: 2

BORING LOG

BORING NO.: B-05

DATE: 4/12/2018

TYPE OF RIG: Diedrich D-120

DEPTH TO GROUND WATER (ft): NE

LOGGED BY: JP

GROUND ELEVATION (ft): NA

SAMPLE NO.	SAMPLE TYPE	BLOWS/12 inches	MOISTURE (%)	PLASTICITY INDEX
A	MC	25		
B	SPT	11		
C	MC	30	4.3	NP
D	SPT	15		
E	MC	53		
F	SPT	50/5.5"		

DEPTH (ft)	USCS SYMBOL	DESCRIPTION
0–2	SM	**Silty Sand (Fill)** Brown, moist, medium dense, with an estimated 20% non-plastic fines, 70% fine to coarse sand, and 10% subrounded gravel up to 1 inch in diameter.
2–5	SM	**Silty Sand** Brown, moist, medium dense, with an estimated 30% non-plastic to low plasticity fines, 65% fine to medium sand, and 5% subangular to subrounded gravel up to 1 inch in diameter.
5–17	SM	**Weathered Granite** Granitic bedrock weathered to decomposed granite, friable. Rock fabric intact and recognizable. Breaks down to **Silty Sand** during auger drilling and in SPT/MC samples. Light gray, moist, medium dense to dense, with 22% non-plastic fines and 78% fine to coarse sand. Contact indicated by drilling response.
17–21.5	SM	**Granite** Granitic bedrock, slight to moderate weathering, friable to moderately strong. Breaks down to **Silty Sand** during auger drilling and in SPT samples. Light gray, moist, very dense, with an estimated 20% non-plastic fines and 80% fine to coarse sand.

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BORING_LOG 0091521.GPJ BLKEAGLE.GDT 6/20/2018

BORING LOG

BORING NO.: B-05

DATE: 4/12/2018

TYPE OF RIG: Diedrich D-120

DEPTH TO GROUND WATER (ft): NE

LOGGED BY: JP

GROUND ELEVATION (ft): NA

SAMPLE NO.	SAMPLE TYPE	BLOWS/12 inches	MOISTURE (%)	PLASTICITY INDEX	DEPTH (ft)	USCS SYMBOL	LITHOLOGY	DESCRIPTION
					24	SM		
G	SPT	50/3.5"						
					26			
					28			
					30			
H	SPT	30/0"						
					32			
					34			
					36			
					38			
					40			
					42			

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PLATE: 2

BORING LOG

BORING NO.: B-06

DATE: 4/17/2018

TYPE OF RIG: Diedrich D-120

DEPTH TO GROUND WATER (ft): NE

LOGGED BY: JP

GROUND ELEVATION (ft): NA

SAMPLE NO.	SAMPLE TYPE	BLOWS/12 inches	MOISTURE (%)	PLASTICITY INDEX	DEPTH (ft)	USCS SYMBOL	LITHOLOGY	DESCRIPTION
					0	GW		**Asphalt Concrete** An approximate 2.75-inch-thick layer of asphalt concrete pavement.
								Aggregate Base An approximate 2.5-inch-thick layer of aggregate base.
A	SPT	19	7.3	4	2–4	SM		**Silty Sand with Gravel** Brown, moist, medium dense to very dense, with an 22% low plasticity fines, 53% fine to coarse sand, and 25% subrounded gravel up to 1.25 inches in diameter.
B	SPT	50/5"			5.5			
C	SPT	43			6–12	SM		**Weathered Granite** Granitic bedrock weathered to decomposed granite, friable. Rock fabric intact and recognizable. Breaks down to **Silty Sand** during auger drilling and in SPT samples. Brown to light gray, moist, dense to very dense, with an estimated 30% non-plastic to low plasticity fines and 70% fine to coarse sand. Very hard drilling 6-7 feet bgs.
D	SPT	48						
E	MC	50/4"			12–22	SM		**Granite** Granitic bedrock, slight to moderate weathering, friable to weak. Breaks down to **Silty Sand** during auger drilling and in SPT samples. Light gray, moist, very dense, with an estimated 20% non-plastic fines and 80% fine to coarse sand.
F	SPT	50/4"						

Refusal at 5 and 9.7 feet 4/11/18. Drilled to 30 feet on 4/17/18.
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BORING LOG

BORING NO.: B-06 | DATE: 4/17/2018

TYPE OF RIG: Diedrich D-120 | DEPTH TO GROUND WATER (ft): NE

LOGGED BY: JP | GROUND ELEVATION (ft): NA

SAMPLE NO.	SAMPLE TYPE	BLOWS/12 inches	MOISTURE (%)	PLASTICITY INDEX	DEPTH (ft)	USCS SYMBOL	LITHOLOGY	DESCRIPTION
					24			
G	SPT	50/5.5"						
					26			
						SM		
					28			
					30			
H	SPT	30/0"						
								Auger refusal at 5 and 9.7 feet bgs, third attempt advanced to target depth with augers.
					32			
					34			
					36			
					38			
					40			
					42			

Refusal at 5 and 9.7 feet 4/11/18. Drilled to 30 feet on 4/17/18.
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BORING_LOG 0091521.GPJ BLKEAGLE.GDT 6/20/2018

BORING LOG

BORING NO.: B-07
DATE: 4/12/2018
TYPE OF RIG: Diedrich D-120
DEPTH TO GROUND WATER (ft): NE
LOGGED BY: JP
GROUND ELEVATION (ft): NA

SAMPLE NO.	SAMPLE TYPE	BLOWS/12 inches	MOISTURE (%)	PLASTICITY INDEX	DEPTH (ft)	USCS SYMBOL	LITHOLOGY	DESCRIPTION
								Asphalt Concrete An approximate 3.75-inch-thick layer of asphalt concrete pavement. No aggregate base.
					2	SM		**Silty Sand** Brown, moist, medium dense, with an estimated 20% non-plastic fines, 70% fine to coarse sand, and 10% subrounded gravel up to 1.25 inches in diameter.
A	SPT	99/11.5"						Includes friable granitic clasts.
					4			**Granite** Granitic bedrock, slight to moderate weathering, friable to weak. Breaks down to **Silty Sand** during auger drilling and in SPT samples. Light gray, moist, very dense, with an estimated 20% non-plastic fines and 80% fine to coarse sand.
B	MC	65/5.5"						
					6			Very difficult drilling 6-13 feet bgs. Possible intact rock/corestone.
C	SPT	30/0"						No recovery at 7.5, 10, 30, and 35 feet bgs.
					8			
D	SPT	30/0"			10			
					12			
					14			
E	SPT	50/3"						
					16			
					18			
						SM		
					20			
F	SPT	50/5"						Slight orange weathering stain.

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BORING_LOG 0091521.GPJ BLKEAGLE.GDT 6/20/2018

BORING LOG

BORING NO.: B-07

DATE: 4/12/2018

TYPE OF RIG: Diedrich D-120

DEPTH TO GROUND WATER (ft): NE

LOGGED BY: JP

GROUND ELEVATION (ft): NA

SAMPLE NO.	SAMPLE TYPE	BLOWS/12 inches	MOISTURE (%)	PLASTICITY INDEX	DEPTH (ft)	USCS SYMBOL	LITHOLOGY	DESCRIPTION
					24			
G	SPT	50/2"						
					26			
					28			
					30	SM		
H	SPT	30/0"						
					32			
					34			
I	SPT	30/0"						
					36			
					38			
					40			
					42			

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EXHIBIT R

BORING_LOG 0091521.GPJ BLKEAGLE.GDT 6/20/2018

BORING LOG

BORING NO.: B-08

DATE: 4/12/2018

TYPE OF RIG: Diedrich D-120

DEPTH TO GROUND WATER (ft): NE

LOGGED BY: JP

GROUND ELEVATION (ft): NA

SAMPLE NO.	SAMPLE TYPE	BLOWS/12 inches	MOISTURE (%)	PLASTICITY INDEX	DEPTH (ft)	USCS SYMBOL	LITHOLOGY	DESCRIPTION
					0–2	SM		**Asphalt Concrete** An approximate 3-inch-thick layer of asphalt concrete pavement. No aggregate base. **Silty Sand** Brown, moist, medium dense, with an estimated 20% non-plastic fines, 70% fine to coarse sand, and 10% subrounded gravel up to 1.25 inches in diameter. Includes friable granitic clasts.
A	SPT	17						
B	MC	57	7.0	2		SM		**Weathered Granite** Granitic bedrock weathered to decomposed granite, friable. Rock fabric intact and recognizable. Breaks down to **Silty Sand** during auger drilling and in SPT samples. Brown to light gray, moist, medium dense to dense, with 32% low plasticity fines and 68% fine to coarse sand.
C	SPT	30						
D	MC	45						
E	SPT	91				SM		**Granite** Granitic bedrock, slight to moderate weathering, friable to weak. Breaks down to **Silty Sand** during auger drilling and in SPT samples. Light gray, moist, very dense, with an estimated 20% non-plastic fines and 80% fine to coarse sand.
F	SPT	50/5"						

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BORING_LOG 0091521.GPJ BLKEAGLE.GDT 6/20/2018

BORING LOG

BORING NO.: B-08 | DATE: 4/12/2018

TYPE OF RIG: Diedrich D-120 | DEPTH TO GROUND WATER (ft): NE

LOGGED BY: JP | GROUND ELEVATION (ft): NA

SAMPLE NO.	SAMPLE TYPE	BLOWS/12 inches	MOISTURE (%)	PLASTICITY INDEX	DEPTH (ft)	USCS SYMBOL	LITHOLOGY	DESCRIPTION
						SM		
					24			
G	SPT	50/4"						
					26			
					28			
H	SPT	50/4.5"			30			
					32			
					34			
					36			
					38			
					40			
					42			

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PLATE: 2

BORING LOG

BORING NO.: B-09 | DATE: 4/16/2018

TYPE OF RIG: Diedrich D-120 | DEPTH TO GROUND WATER (ft): NE

LOGGED BY: JP | GROUND ELEVATION (ft): NA

SAMPLE NO.	SAMPLE TYPE	BLOWS/12 inches	MOISTURE (%)	PLASTICITY INDEX	DEPTH (ft)	USCS SYMBOL	LITHOLOGY	DESCRIPTION
					0–2.7	SM		**Gravel Surfacing** **Silty Sand** Brown, moist, medium dense, with an estimated 20% non-plastic fines, 70% fine to coarse sand, and 10% subrounded gravel up to 1.25 inches in diameter.
A	SPT	16			2.7–10	SM		Includes friable granitic clasts. **Weathered Granite** Granitic bedrock weathered to decomposed granite, friable. Rock fabric intact and recognizable. Breaks down to **Silty Sand** during auger drilling and in SPT/MC samples. Brown to light gray, moist, medium dense to dense, with an estimated 20% low plasticity fines and 80% fine to coarse sand.
B	SPT	36						
C	MC	38						
D	SPT	50/5"			10–20.5	SM		**Granite** Granitic bedrock, slight to moderate weathering, friable to weak. Breaks down to **Silty Sand** during auger drilling and in SPT samples. Light gray, moist, very dense, with an estimated 20% non-plastic fines and 80% fine to coarse sand.
E	MC	76						
								Auger refusal at 20 feet bgs. HQ coring from 20-40.8 feet bgs.
F	SPT	30/0"			20.5–			**Granite** Light gray, fresh to moderate weathering, very strong to extremely strong, moderate to wide fracture spacing. Minimum fracture spacing 12 inches.

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BORING_LOG 0091521.GPJ BLKEAGLE.GDT 6/20/2018

BORING LOG

BORING NO.: B-09 | DATE: 4/16/2018

TYPE OF RIG: Diedrich D-120 | DEPTH TO GROUND WATER (ft): NE

LOGGED BY: JP | GROUND ELEVATION (ft): NA

SAMPLE NO.	SAMPLE TYPE	BLOWS/12 inches	MOISTURE (%)	PLASTICITY INDEX	DEPTH (ft)	USCS SYMBOL	LITHOLOGY	DESCRIPTION
G	CORE				24			Sample G: Rock Quality Designation (RQD) = 77, Good Quality. Coring rate = 1.6 minutes per foot (min/ft).
H	CORE				26			Sample H: RQD = 67, Fair Quality. Coring rate = 1.8 min/ft.
					28			**Granite** Light gray, fresh to moderate weathering, weak to extremely strong, moderate to wide fracture spacing. Minimum fracture spacing 12 inches.
I	CORE				30, 32, 34			Sample I: No Recovery, RQD = 0, Very Poor Quality. Coring rate 1.2 min/ft.
J	CORE				36, 38			**Granite** Light gray, fresh to moderate weathering, moderately strong to extremely strong, moderate to wide fracture spacing. Minimum fracture spacing 12 inches. Sample J: RQD = 32, Poor Rock Quality. Coring rate 2.4 min/ft.
K	CORE				40			Sample K: RQD = 67, Fair Rock Quality. Coring rate = 3.5 min/ft.
					42			

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BORING_LOG 0091521.GPJ BLKEAGLE.GDT 6/20/2018

BORING LOG

BORING NO.: B-10 DATE: 4/17/2018

TYPE OF RIG: Diedrich D-120 DEPTH TO GROUND WATER (ft): NE

LOGGED BY: JP GROUND ELEVATION (ft): NA

SAMPLE NO.	SAMPLE TYPE	BLOWS/12 inches	MOISTURE (%)	PLASTICITY INDEX	DEPTH (ft)	USCS SYMBOL	LITHOLOGY	DESCRIPTION
					0–4.5	SM		**Silty Sand** Brown, moist, medium dense, with an estimated 30% non-plastic fines, 65% fine to coarse sand, and 5% subrounded gravel up to 1 inch in diameter.
A	SPT	14						
B	SPT	12						
C	MC	32	7.6	NP				
D	SPT	37			4.5–20	SM		**Weathered Granite** Granitic bedrock weathered to decomposed granite, friable. Rock fabric intact and recognizable. Breaks down to **Silty Sand** during auger drilling and in SPT/MC samples. Light gray, moist, medium dense to dense, with 21% non-plastic fines and 79% fine to coarse sand.
E	SPT	50/5"			20–	SM		**Granite** Granitic bedrock, slight to moderate weathering, friable to moderately strong. Breaks down to **Silty Sand** during auger drilling and in SPT samples. Light gray, moist, very dense, with an

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BORING_LOG 0091521.GPJ BLKEAGLE.GDT 6/20/2018

BORING LOG

BORING NO.: B-10

DATE: 4/17/2018

TYPE OF RIG: Diedrich D-120

DEPTH TO GROUND WATER (ft): NE

LOGGED BY: JP

GROUND ELEVATION (ft): NA

SAMPLE NO.	SAMPLE TYPE	BLOWS/12 inches	MOISTURE (%)	PLASTICITY INDEX	DEPTH (ft)	USCS SYMBOL	LITHOLOGY	DESCRIPTION
						SM		estimated 15% non-plastic fines and 85% fine to coarse sand.
					24			
F	SPT	50/3"						
					26			
					28			
					30			
					32			
					34			
					36			
					38			
					40			
					42			

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BORING_LOG 0091521.GPJ BLKEAGLE.GDT 6/20/2018

BORING LOG

BORING NO.:	B-11	DATE:	4/12/2018
TYPE OF RIG:	Diedrich D-120	DEPTH TO GROUND WATER (ft):	NE
LOGGED BY:	JP	GROUND ELEVATION (ft):	NA

SAMPLE NO.	SAMPLE TYPE	BLOWS/12 inches	MOISTURE (%)	PLASTICITY INDEX
A	SPT	44	6.0	NP
B	SPT	68		
C	MC	50/5"		
D	SPT	50/3.5"		
E	SPT	50/2.5"		
F	SPT	50/5"		

USCS SYMBOL	LITHOLOGY	DESCRIPTION
SM		**Silty Sand** Brown, moist, medium dense, with an estimated 30% non-plastic fines, 65% fine to coarse sand, and 5% subrounded gravel up to 1 inch in diameter.
SM		**Weathered Granite** Granitic bedrock weathered to decomposed granite, friable. Rock fabric intact and recognizable. Breaks down to **Silty Sand** during auger drilling and in SPT/MC samples. Light gray, moist, dense to dense, with 21% non-plastic fines and 79% fine to coarse sand.
CL		**Weathered Granite** Granitic bedrock completely weathered to soil. Breaks down to **Sandy Lean Clay** during auger drilling and in MC samples. Light gray, moist, very hard, with an estimated 60% low to medium plasticity fines and 40% fine sand.
SM		**Granite** Granitic bedrock, slight to moderate weathering, friable to moderately strong. Breaks down to **Silty Sand** during auger drilling and in SPT samples. Light gray, moist, very dense, with an estimated 20% non-plastic fines and 80% fine to coarse sand.

DEPTH (ft): 2, 4, 6, 8, 10, 12, 14, 16, 18, 20

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PLATE: 2

WSUP23-0025
EXHIBIT R

BORING LOG

BORING NO.: B-11

DATE: 4/12/2018

TYPE OF RIG: Diedrich D-120

DEPTH TO GROUND WATER (ft): NE

LOGGED BY: JP

GROUND ELEVATION (ft): NA

SAMPLE NO.	SAMPLE TYPE	BLOWS/12 inches	MOISTURE (%)	PLASTICITY INDEX	DEPTH (ft)	USCS SYMBOL	LITHOLOGY	DESCRIPTION
					24	SM		
G	SPT	50/3.5"						
					26			
					28			
					30			
					32			
					34			
					36			
					38			
					40			
					42			

N 4346544 E 758529 UTM NAD83

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CFA, Inc.
Boulder Bay
Washoe County, Nevada

PROJECT NO.: 0091-52-1

PLATE: 2

WSUP23-0025
EXHIBIT R

BORING_LOG 0091521.GPJ BLKEAGLE.GDT 6/20/2018

BORING LOG

BORING NO.:	B-12	DATE:	4/16/2018
TYPE OF RIG:	Diedrich D-120	DEPTH TO GROUND WATER (ft):	NE
LOGGED BY:	JP	GROUND ELEVATION (ft):	NA

SAMPLE NO.	SAMPLE TYPE	BLOWS/12 inches	MOISTURE (%)	PLASTICITY INDEX	DEPTH (ft)	USCS SYMBOL	LITHOLOGY	DESCRIPTION
					0–5	SM		**Silty Sand with Gravel (Fill)** Brown, moist, medium dense to very dense, with an estimated 25% low plasticity fines, 50% fine to coarse sand, and 25% subrounded gravel up to 1.25 inches in diameter.
A	SPT	50/5.5"						Gravel stuck in SPT sampler.
B	SPT	27			5–10	SM		**Weathered Granite** Granitic bedrock weathered to decomposed granite, friable. Rock fabric intact and recognizable. Breaks down to **Silty Sand** during auger drilling and in SPTsamples. Light gray, moist, medium dense, with an estimated 25% low plasticity fines, 70% fine to coarse sand, and 5% angular gravel up to 0.5 inch in diameter.
C	SPT	15						
D	MC	36			10–15	SM		**Weathered Granite** Granitic bedrock weathered to decomposed granite, friable. Rock fabric intact and recognizable. Breaks down to **Silty Sand** during auger drilling and in MC samples. Light gray, moist, medium dense, with an estimated 20% non-plastic fines and 80% fine to coarse sand.
E	SPT	69			15–21.5	SM		**Granite** Granitic bedrock, slight to moderate weathering, friable to moderately strong. Breaks down to **Silty Sand** during auger drilling and in SPT samples. Light gray, moist, very dense, with an estimated 20% non-plastic fines and 80% fine to coarse sand.
F	SPT	64						

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Washoe County, Nevada

PROJECT NO.: 0091-52-1

PLATE: 2

BORING_LOG 0091521.GPJ BLKEAGLE.GDT 6/20/2018

BORING LOG

BORING NO.: B-12 | DATE: 4/16/2018

TYPE OF RIG: Diedrich D-120 | DEPTH TO GROUND WATER (ft): NE

LOGGED BY: JP | GROUND ELEVATION (ft): NA

SAMPLE NO.	SAMPLE TYPE	BLOWS/12 inches	MOISTURE (%)	PLASTICITY INDEX	DEPTH (ft)	USCS SYMBOL	LITHOLOGY	DESCRIPTION
						SM		
					24			
G	SPT	50/5.5"						
					26			
					28			
H	SPT	50/4"			30			
					32			
					34			
					36			
					38			
					40			
					42			

N 4346572 E 758553 UTM NAD83

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Washoe County, Nevada

PROJECT NO.: 0091-52-1

PLATE: 2

TEST HOLE LOG

BORING NO.: HA-01

DATE: 4/24/2018

TYPE OF RIG: Hand Auger

DEPTH TO GROUND WATER (ft): NE

LOGGED BY: JP

GROUND ELEVATION (ft): NA

SAMPLE NO.	SAMPLE TYPE	BLOWS/12 inches	MOISTURE (%)	PLASTICITY INDEX	DEPTH (ft)	USCS SYMBOL	LITHOLOGY	DESCRIPTION
A	GRAB				0–5	SM		**Silty Sand with Gravel** Brown, moist, medium dense, with an estimated 20% non-plastic fines, 65% fine to coarse sand, and 15% subangular gravel up to 3 inches in diameter.
					5			Hand auger refusal at 5 feet bgs.

Depth scale: 2, 4, 6, 8

BORING_LOG 0091521.GPJ BLKEAGLE.GDT 6/20/2018

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PROJECT NO.: 0091-52-1

PLATE: 2

TEST HOLE LOG

BORING NO.: HA-02

DATE: 4/24/2018

TYPE OF RIG: Hand Auger

DEPTH TO GROUND WATER (ft): NE

LOGGED BY: JP

GROUND ELEVATION (ft): NA

SAMPLE NO.	SAMPLE TYPE	BLOWS/12 inches	MOISTURE (%)	PLASTICITY INDEX	DEPTH (ft)	USCS SYMBOL	LITHOLOGY	DESCRIPTION
					0–3	SM		**Silty Sand with Gravel (Fill)** Brown, moist, medium dense, with an estimated 20% non-plastic fines, 65% fine to coarse sand, and 15% subangular gravel up to 3 inches in diameter. Includes some cobbles and construction debris including PCC and AC chunks.
					3			Hand auger refusal at 3 feet bgs, due to obstructions. Attempted in multiple locations.

Depth scale: 2, 4, 6, 8

BORING_LOG 0091521.GPJ BLKEAGLE.GDT 6/20/2018

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Boulder Bay
Washoe County, Nevada

PROJECT NO.: 0091-52-1

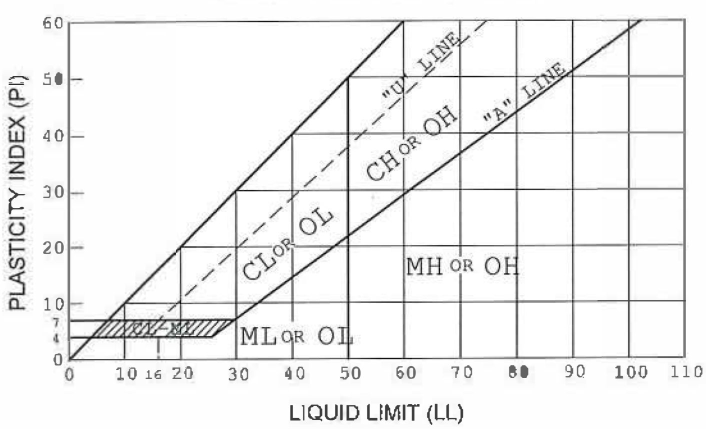
PLATE: 2

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 4.75mm)
Sand	# 4 to #200 sieve (4.75mm 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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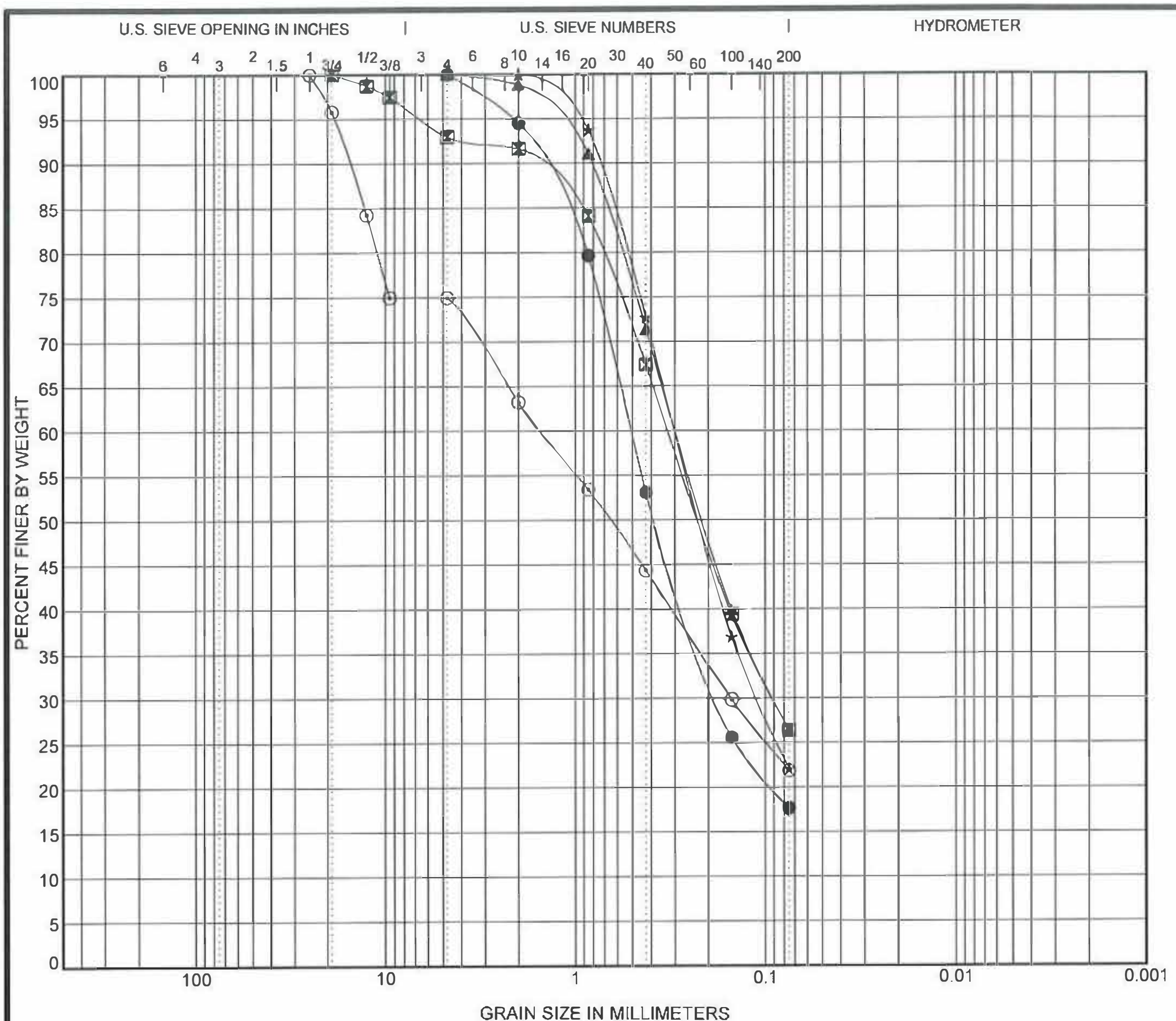
USCS Soil Classification Chart

Project: Boulder Bay
Location: Washoe County, Nevada
Project Number: 0091-52-1 Plate: 2



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Specimen Identification			USCS Classification	LL	PL	PI	Cc	Cu
●	B-01	2.5'	SILTY SAND (SM)	NP	NP	NP		
⊠	B-03	1.0'	SILTY SAND (SM)	NP	NP	NP		
▲	B-04	5.0'	SILTY SAND (SM)	NP	NP	NP		
★	B-05	7.5'	SILTY SAND (SM)	NP	NP	NP		
⊙	B-06	2.5'	SILTY, CLAYEY SAND with GRAVEL (SC-SM)	23	19	4		

Specimen Identification			D100	D60	D30	D10	MC %	%Gravel	%Sand	%Silt	%Clay
●	B-01	2.5'	4.75	0.509	0.177		11.9	0.0	82.3	17.7	
⊠	B-03	1.0'	19	0.322	0.091		11.1	7.0	66.6	26.3	
▲	B-04	5.0'	4.75	0.291	0.09		10.4	0.0	73.6	26.4	
★	B-05	7.5'	2	0.294	0.109		4.3	0.0	77.9	22.1	
⊙	B-06	2.5'	25	1.506	0.151		7.3	25.0	53.1	21.9	

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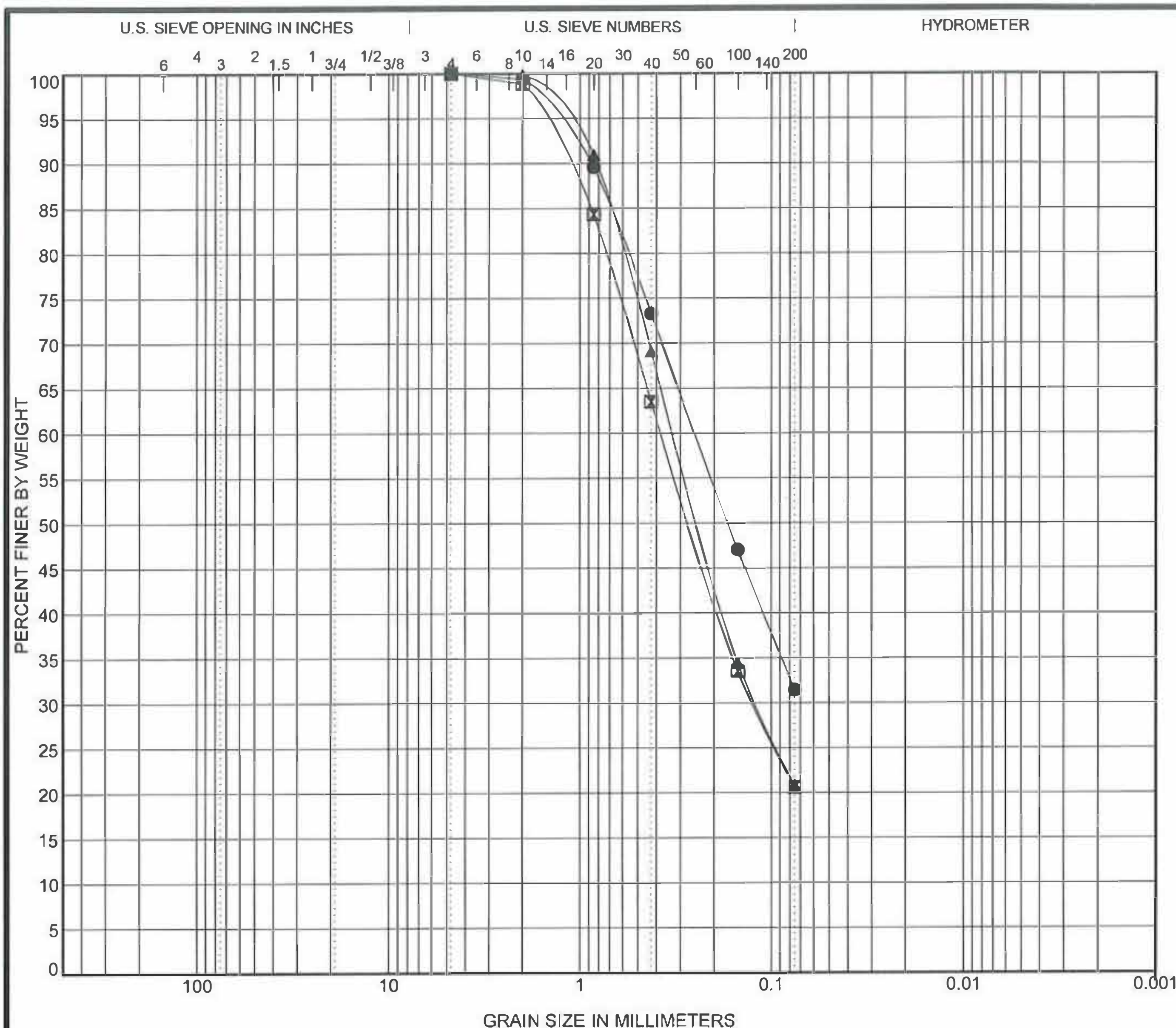
GRAIN SIZE DISTRIBUTION

Project: Boulder Bay
Location: Washoe County, Nevada
Project Number: 0091-52-1 Plate: 4a

US GRAIN SIZE2 0091521.GPJ US LAB.GDT 6/21/2018



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen Identification		USCS Classification	LL	PL	PI	Cc	Cu
●	B-08	5.0'	**SILTY SAND (SM)**	27	25	2		
⊠	B-10	10.0'	**SILTY SAND (SM)**	NP	NP	NP		
▲	B-11	2.5'	**SILTY SAND (SM)**	NP	NP	NP		

	Specimen Identification		D100	D60	D30	D10	MC %	%Gravel	%Sand	%Silt	%Clay
●	B-08	5.0'	4.75	0.251			7.0	0.0	68.5	31.5	
⊠	B-10	10.0'	4.75	0.376	0.124		7.6	0.0	79.4	20.6	
▲	B-11	2.5'	4.75	0.323	0.12		6.0	0.0	79.1	20.9	

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GRAIN SIZE DISTRIBUTION

Project: Boulder Bay
Location: Washoe County, Nevada
Project Number: 0091-52-1 Plate: 4b

US_GRAIN_SIZE2 0091521.GPJ US_LAB.GDT 6/21/2018

WSUP23-0025
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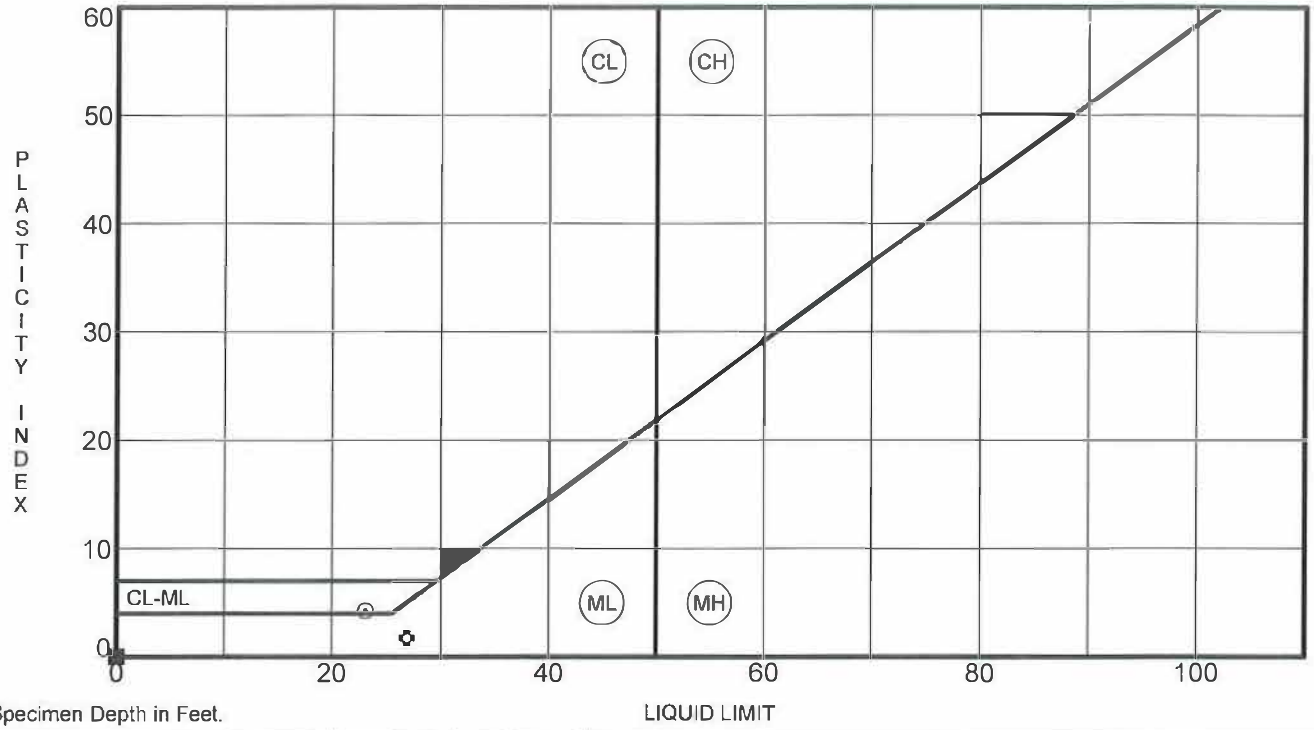

Specimen Depth in Feet.

	Specimen Identification			LL	PL	PI	Fines	USCS Classification
●	B-01	B	2.5'	NP	NP	NP	18	SILTY SAND (SM)
⊠	B-03		1.0'	NP	NP	NP	26	SILTY SAND (SM)
▲	B-04	B	5.0'	NP	NP	NP	26	SILTY SAND (SM)
★	B-05	C	7.5'	NP	NP	NP	22	SILTY SAND (SM)
⊙	B-06	A	2.5'	23	19	4	22	SILTY, CLAYEY SAND with GRAVEL (SC-SM)
✪	B-08	B	5.0'	27	25	2	31	SILTY SAND (SM)
○	B-10	C	10.0'	NP	NP	NP	21	SILTY SAND (SM)
△	B-11	A	2.5'	NP	NP	NP	21	SILTY SAND (SM)

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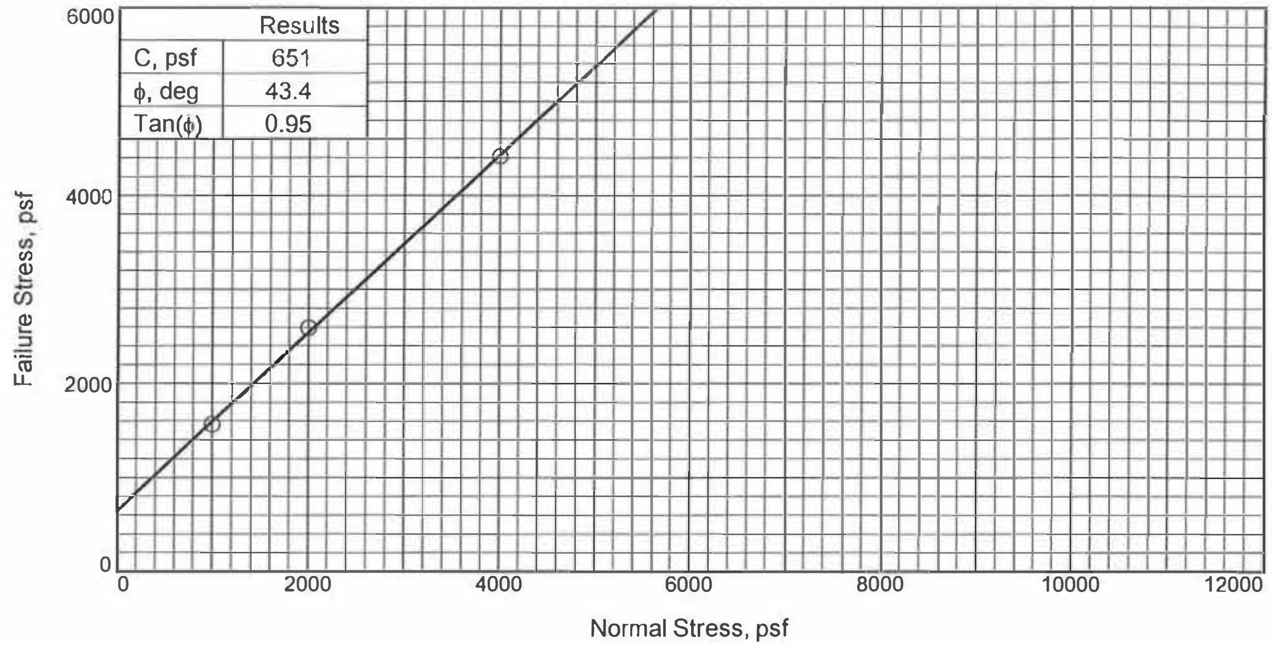
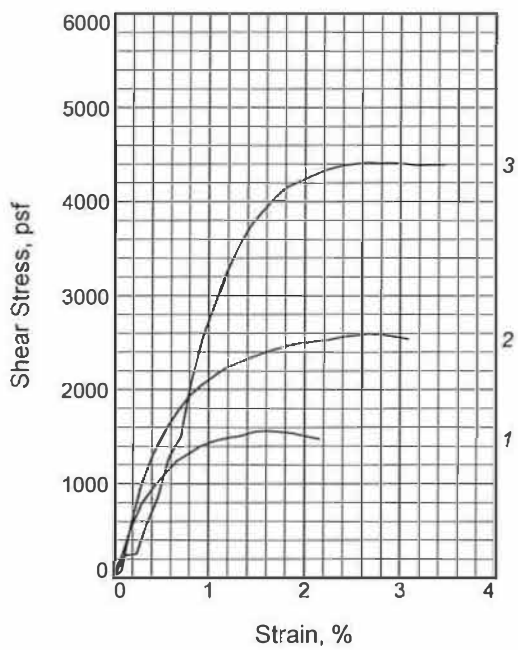
ATTERBERG LIMITS RESULTS

Project: Boulder Bay

Location: Washoe County, Nevada

Project Number: 0091-52-1 Plate: 4c





Sample No.		1	2	3
Initial	Water Content, %	13.4	13.4	13.4
	Dry Density, pcf	113.9	113.9	113.9
	Saturation, %	78.4	78.4	78.4
	Void Ratio	0.4528	0.4528	0.4528
	Diameter, in.	2.420	2.420	2.420
	Height, in.	1.000	1.000	1.000
At Test	Water Content, %	16.0	20.5	20.3
	Dry Density, pcf	115.3	114.9	116.3
	Saturation, %	97.8	123.3	127.3
	Void Ratio	0.4346	0.4397	0.4229
	Diameter, in.	2.420	2.420	2.420
	Height, in.	0.988	[illegible].991	0.979
Normal Stress, psf		1000	2000	4000
Failure Stress, psf		1563	2593	4416
Strain, %		1.6	2.7	2.6
Ult. Stress, psf				
Strain, %				
Strain rate, in./min.		0.002	0.002	0.002

Sample Type: Remold
Description:

LL= NV **PI=** NP
Assumed Specific Gravity= 2.65
Remarks: Laboratory Log 6430

Figure 5a

Client: CFA, Inc.

Project: Boulder Bay

Source of Sample: B-04 **Depth:** 5'
Sample Number: B
Proj. No.: 0091-52-1 **Date Sampled:** 04/11/18

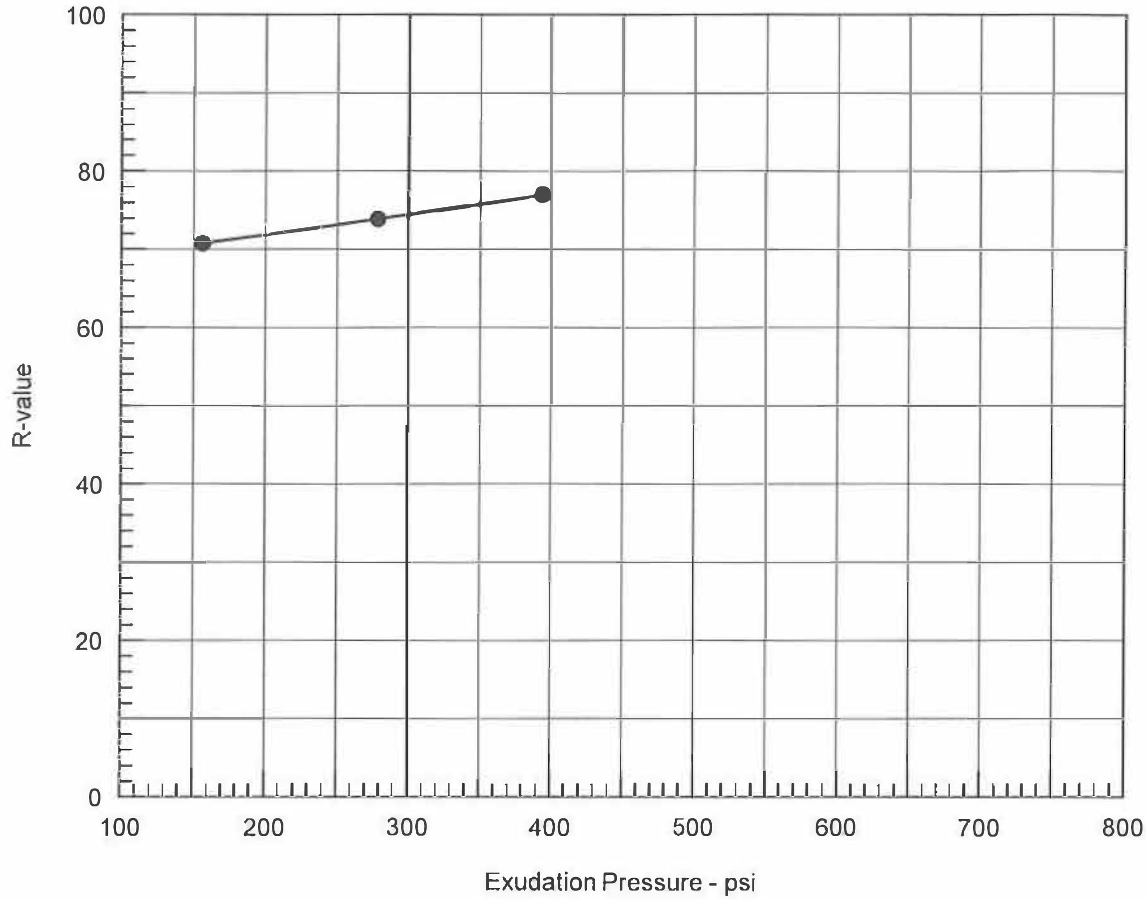
DIRECT SHEAR TEST REPORT

BLACK EAGLE CONSULTING, INC.

Tested By: GLO **Checked By:** SRS

WSUP23-0025
EXHIBIT R

R-VALUE TEST REPORT



Resistance R-Value and Expansion Pressure - ASTM D 2844

No.	Compact. Pressure psi	Density pcf	Moist. %	Expansion Pressure psi	Horizontal Press. psi @ 160 psi	Sample Height in.	Exud. Pressure psi	R Value	R Value Corr.
1	350	122.7	11.1	0.18	30	2.47	278	74	74
2	350	122.4	10.6	0.18	26	2.52	394	77	77
3	350	122.9	11.7	0.12	35	2.46	156	71	71

Test Results	Material Description
R-value at 300 psi exudation pressure = 74	

Project No.: 0091-52-1
Project: Boulder Bay
Source of Sample: B-03 **Depth:** 1'-5'
Sample Number: Bulk
Date: 5/18/2018

Tested by: GLO
Checked by: SRS
Remarks:
Laboratory Log 6430

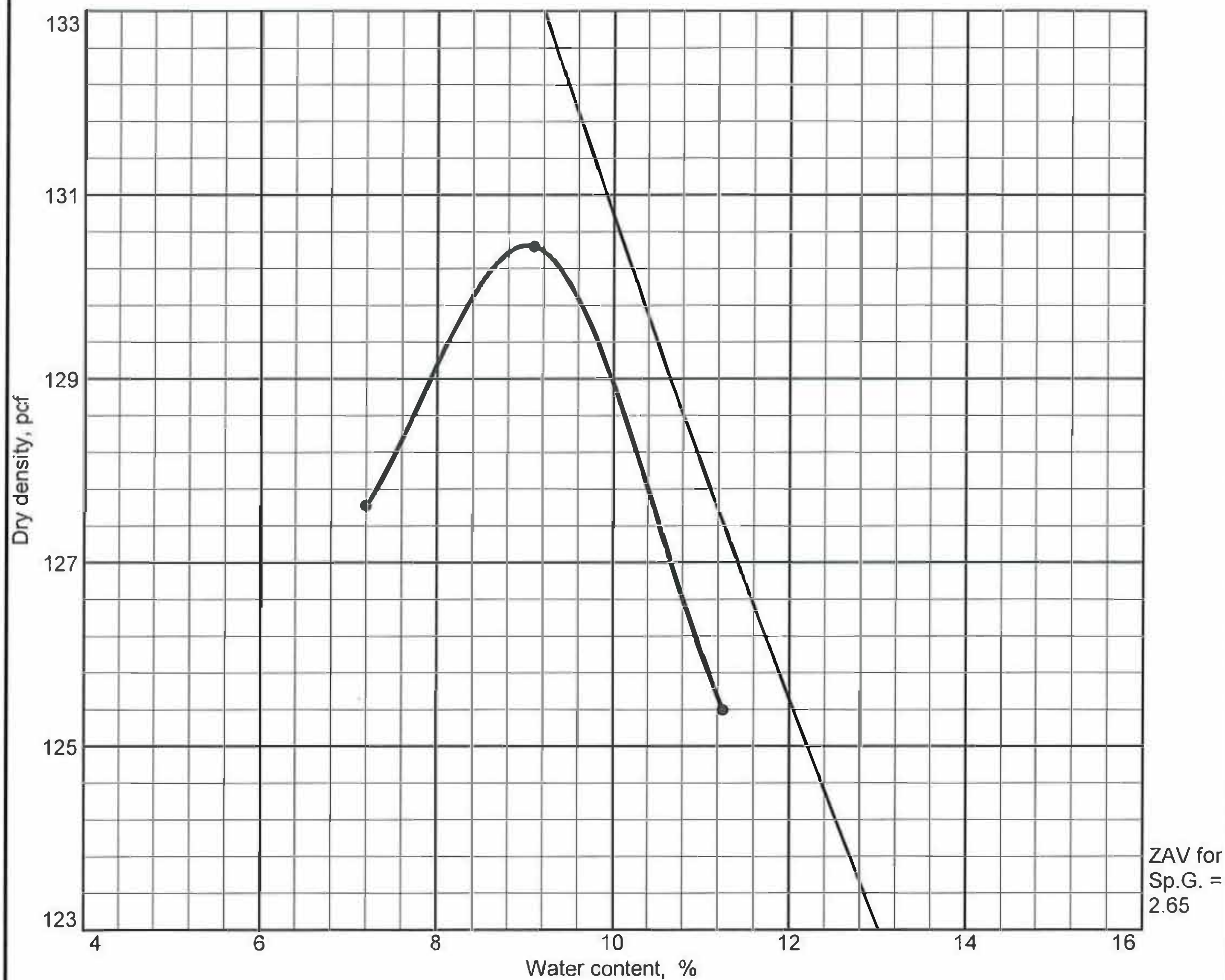
R-VALUE TEST REPORT

BLACK EAGLE CONSULTING, INC.

Figure 6a

WSUP23-0025
EXHIBIT R

COMPACTION TEST REPORT



Test specification: ASTM D 1557-78 Method B Modified

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > No.4	% < No.200
	USCS	AASHTO						
	SM						75.0	22

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 130.5 pcf Optimum moisture = 9.0 %	Silty Sand

Project No. 0091-52-1 **Client:** CFA, Inc.
Project: Boulder Bay

● **Source:** B-3, 1'-5' & B-12, 5'-10'

Remarks:
Laboratory Log 6488

BLACK EAGLE CONSULTING, INC.
Reno, Nevada

Figure 7

WSUP23-0025
EXHIBIT R

ROCK CORE ANALYSES

PROJECT: Boulder Bay

PROJECT NO.: 0091-52-1

CLIENT: CFA, Inc.

CORE NO.	B-09						
LOCATION	22.0' – 22.5'						
DATE CORED	04/13/18						
DATE TESTED	05/29/18						
UNCAPPED LENGTH - INCHES	5.40						
CAPPED LENGTH - INCHES	5.62						
DIAMETER - INCHES	2.38						
AREA – IN^2	4.45						
L/D RATIO	2.36						
L/D CORRECTION	0.0						
TOTAL LOAD - LBS	86,580						
UNIT LOAD - PSI	19,460						
CORRECTED UNIT LOAD - PSI	**19,460**						
AVERAGE UNIT LOAD - PSI	**19,460**						

REMARKS:

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Respectfully Submitted By:

Scott R. Shipley
Laboratory Manager
Date: May 30, 2018

APPENDIX A

SHEAR WAVE VELOCITY MODELING RESULTS

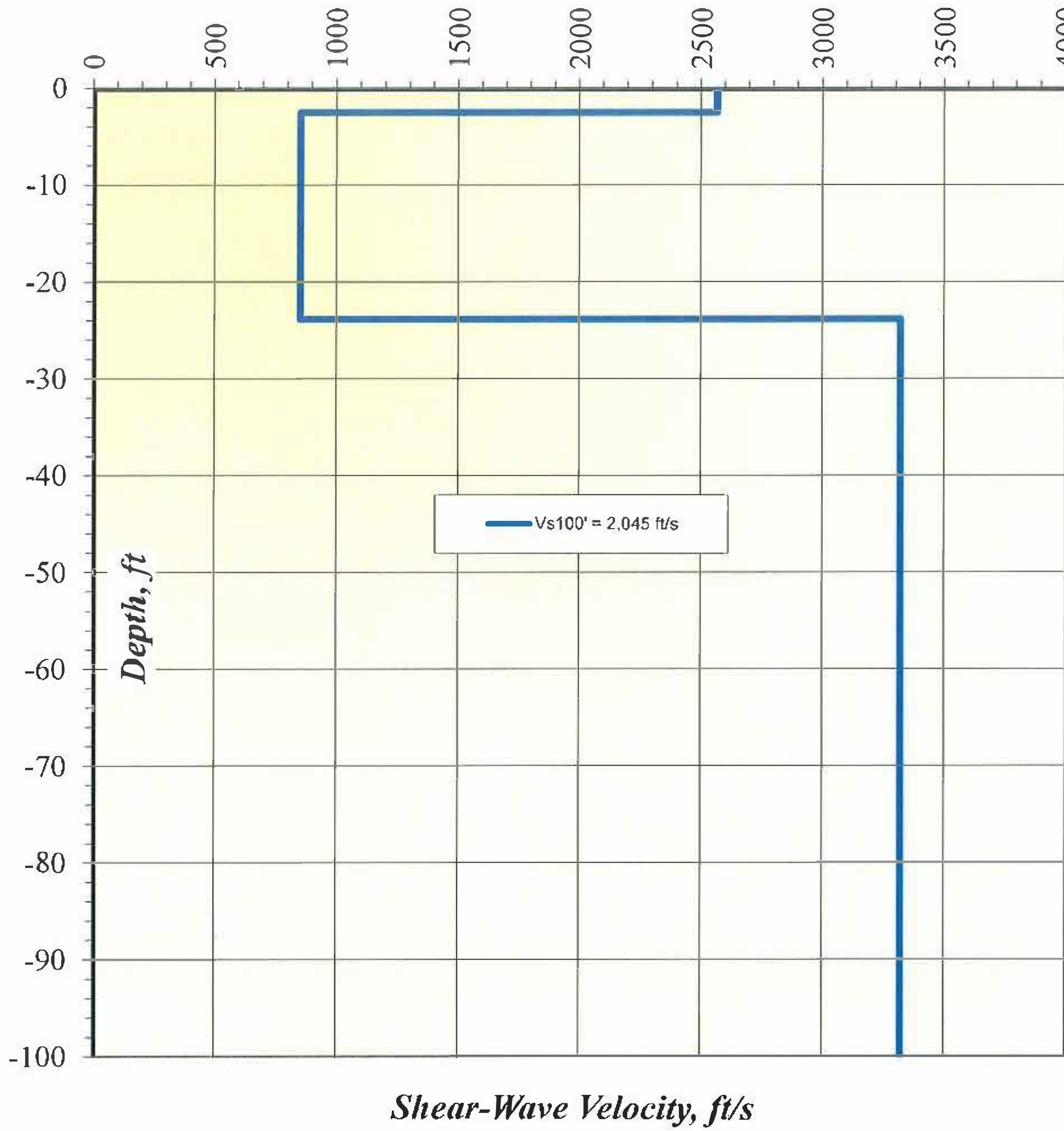
MICROTREMOR SHEAR-WAVE ANALYSIS

Shear-wave velocities for subsurface strata were collected using a multiple channel digital acquisition data logger and geophone system. A DAQLink II™ 24-bit, 2-channel analog to digital data logger, coupled with 12, 4.5-Hz geophones on 3-meter spacings, was used to record background micro tremor refraction data. SeisOpt ReMi® software was then used to model the digital refraction data using a wave field transformation data processing technique and an interactive Rayleigh-wave dispersion model. Model output after data processing is presented as a spectral solution of wave frequency vs. slowness, the modeled Rayleigh-wave phase-velocity dispersion curve, and a graphical representation of shear-wave velocity vs. depth at the modeled location.

The Raleigh-wave dispersion curve and slowness-frequency wave dispersion are contained in Appendix A. For standard 8-meter (25-foot) geophone spacing, estimation of Rayleigh-wave phase-velocity dispersion curves by slowness-frequency wave field transformation has been shown to be an effective method for estimation of 30-meter (100-foot) average shear-wave velocities and one-dimensional shear-wave profile within 20 percent accuracy to 100 meters depth[1]. The shear-wave velocity versus depth model is also contained in this Appendix A.

[1] Louie, John N., April 2001, "Faster, Better: Shear-Wave Velocity to 100 Meters Depth for Refraction Microtremor Arrays." *Bulletin of the Seismological Society of America,* v. 91, n. 2, p. 347-396.

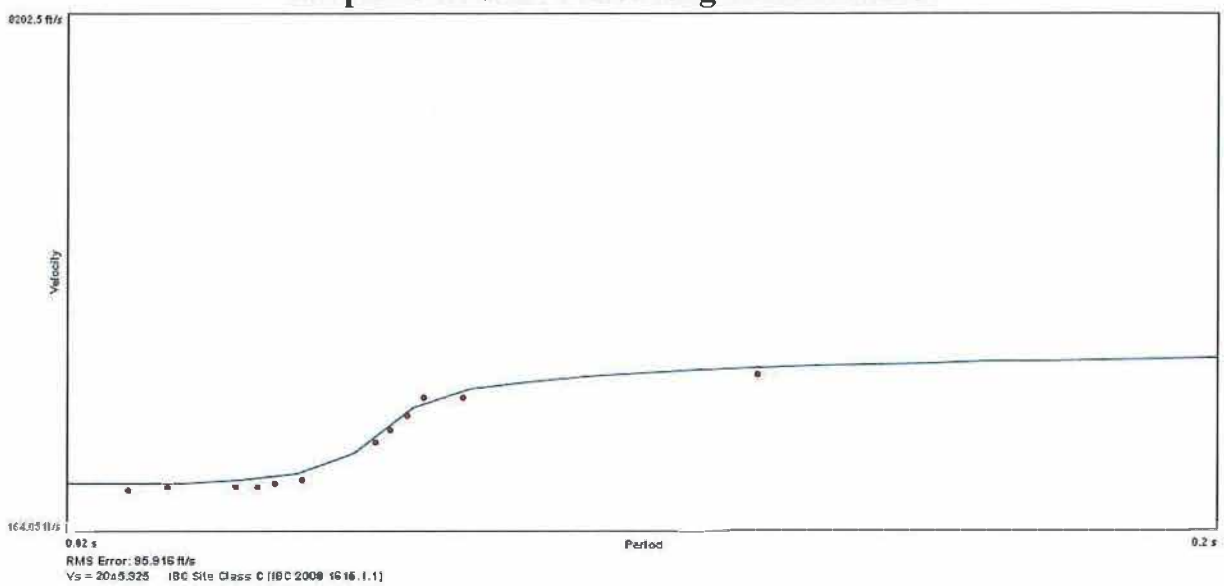
Shear Wave Velocity Modeling Results
Boulder Bay - Line S_1



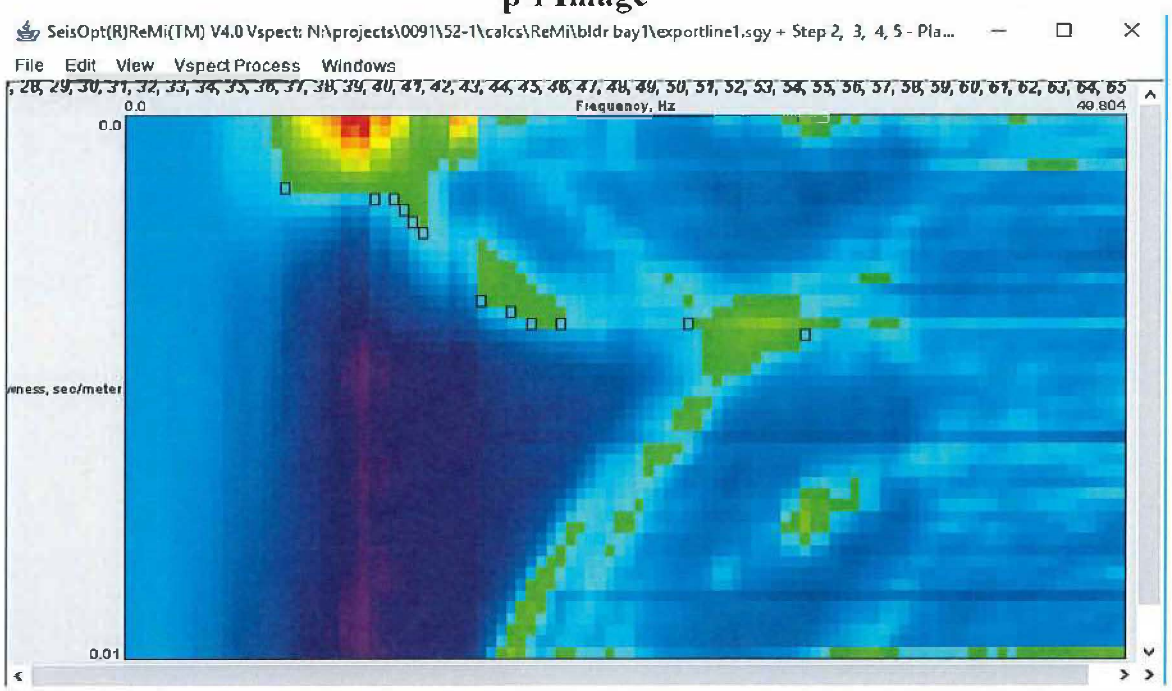
WSUP23-0025
EXHIBIT R

Boulder Bay - Line S_1

Dispersion Curve Showing Picks and Fit

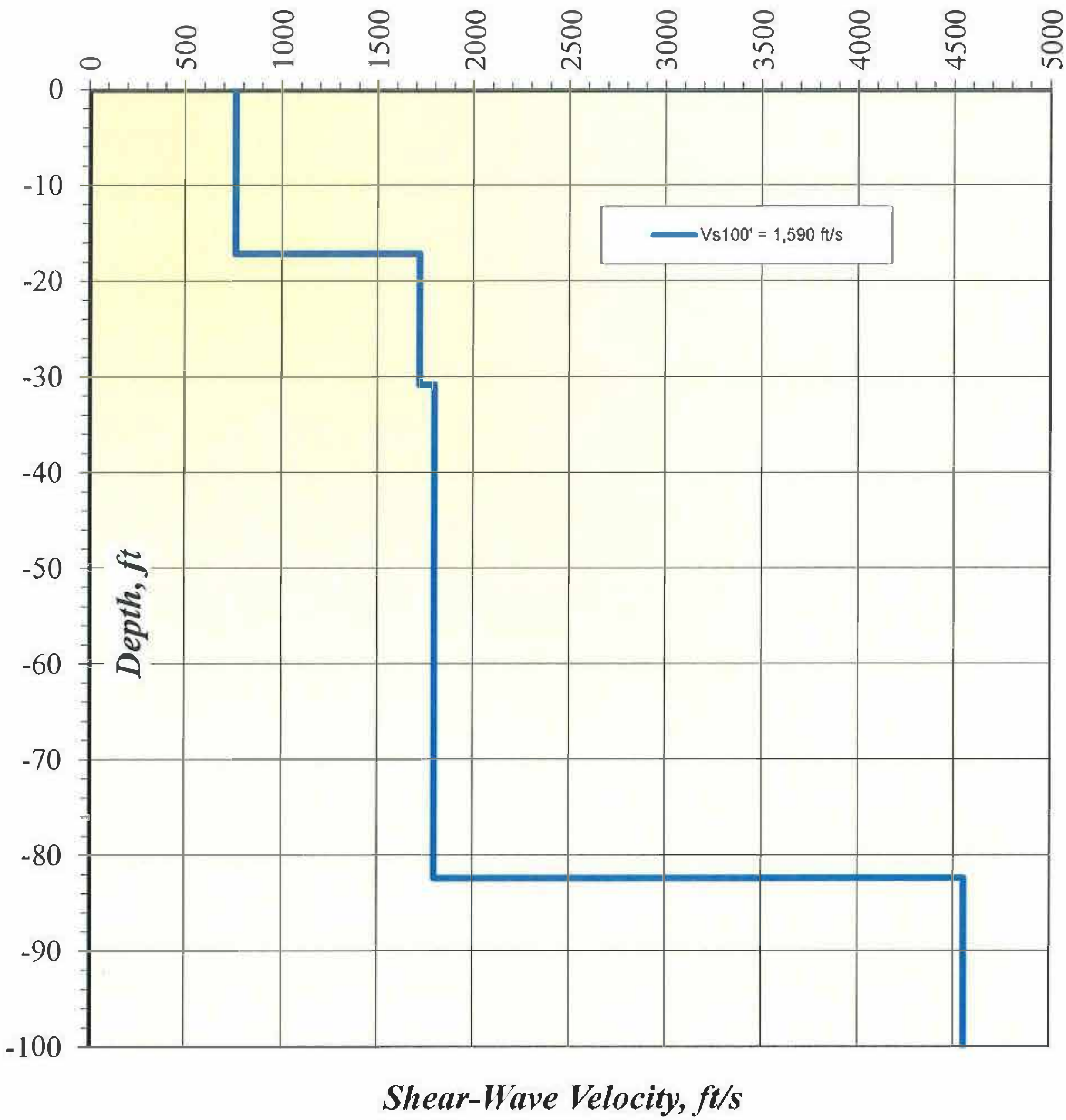


p-f Image



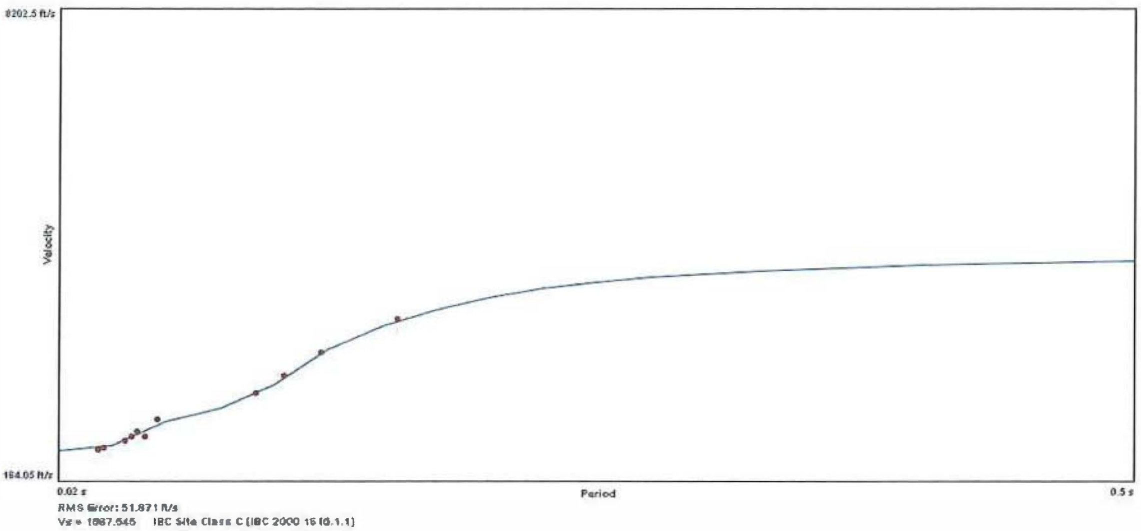
WSUP23-0025
EXHIBIT R

Shear Wave Velocity Modeling Results
Boulder Bay - Line S_2

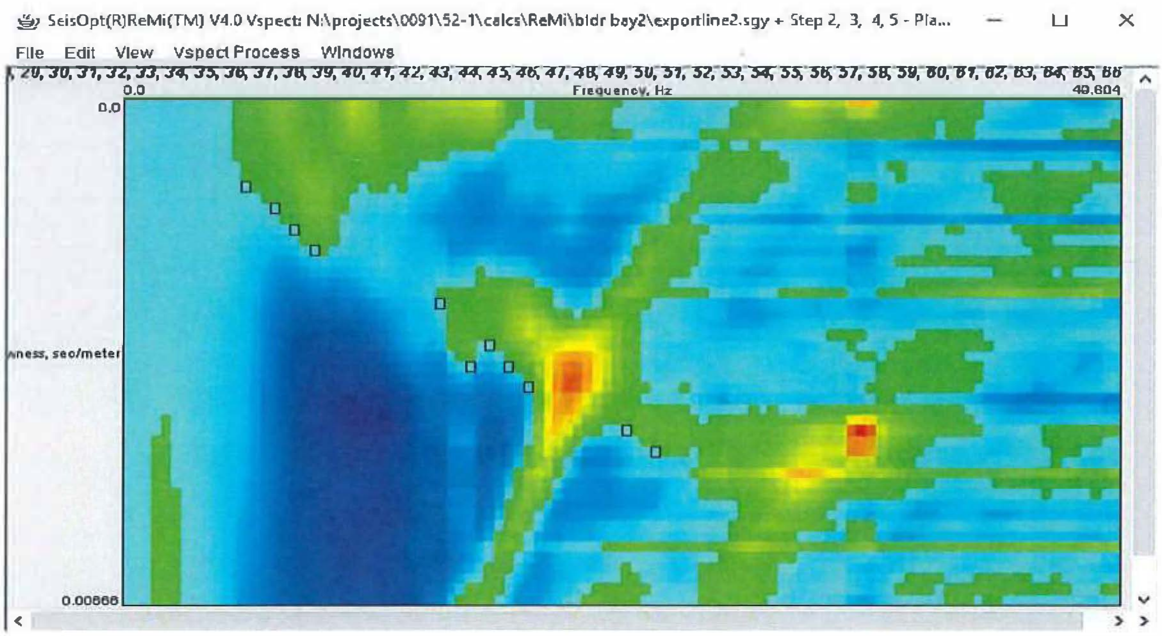


Boulder Bay - Line S_2

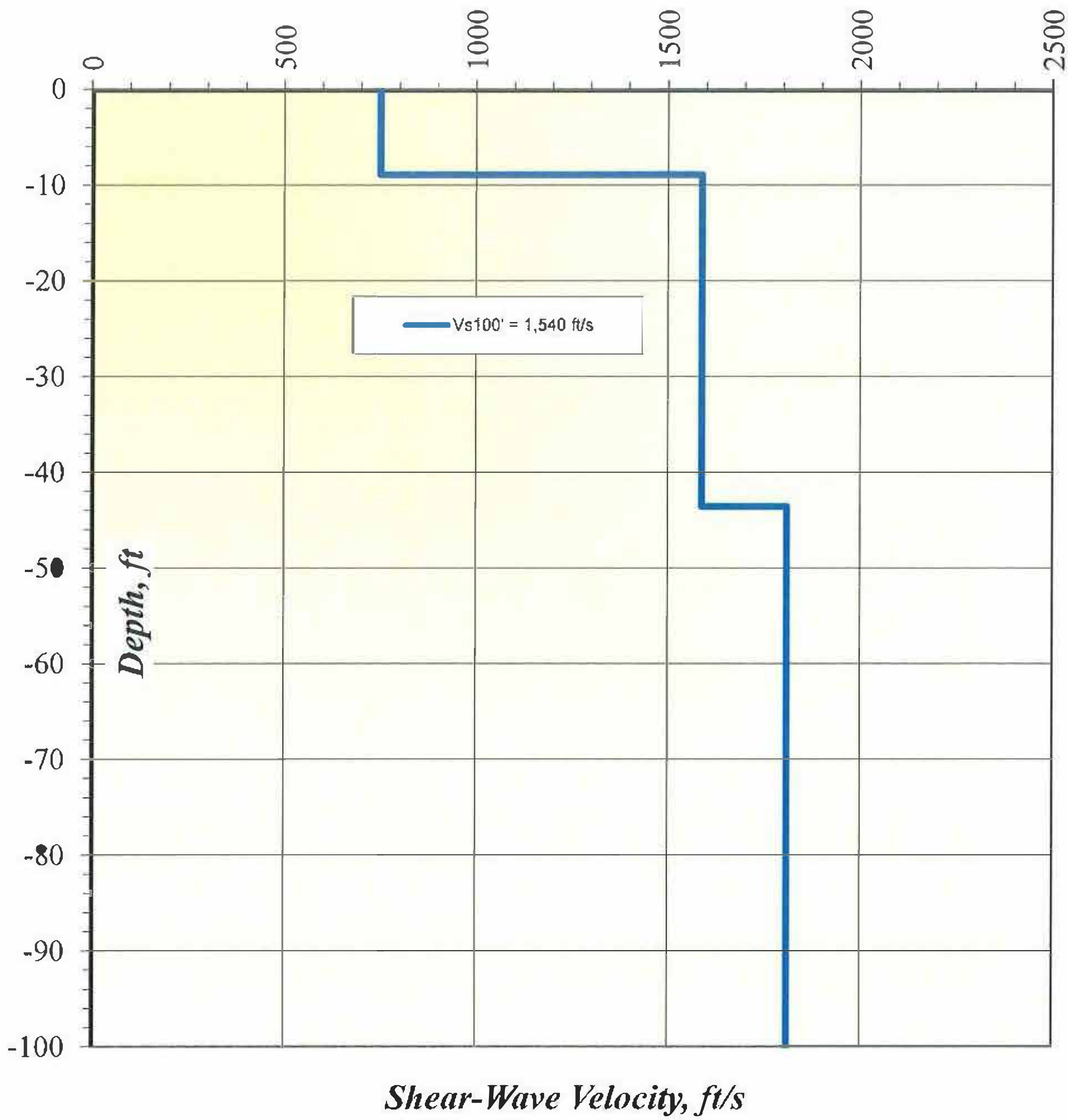
Dispersion Curve Showing Picks and Fit



p-f Image

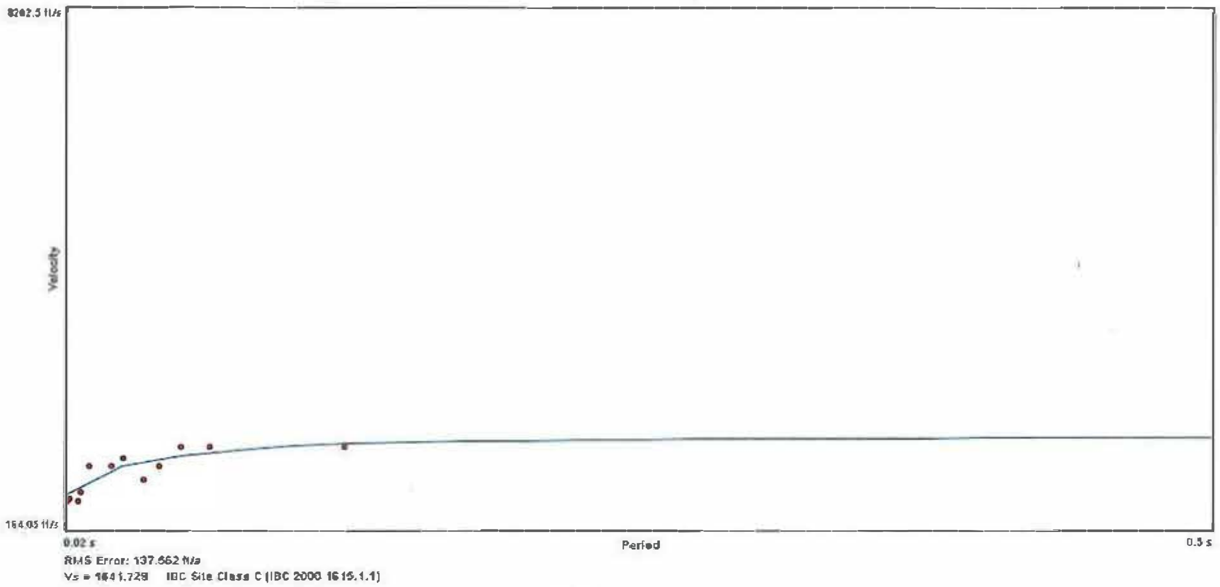


Shear Wave Velocity Modeling Results
Boulder Bay - Line S_3

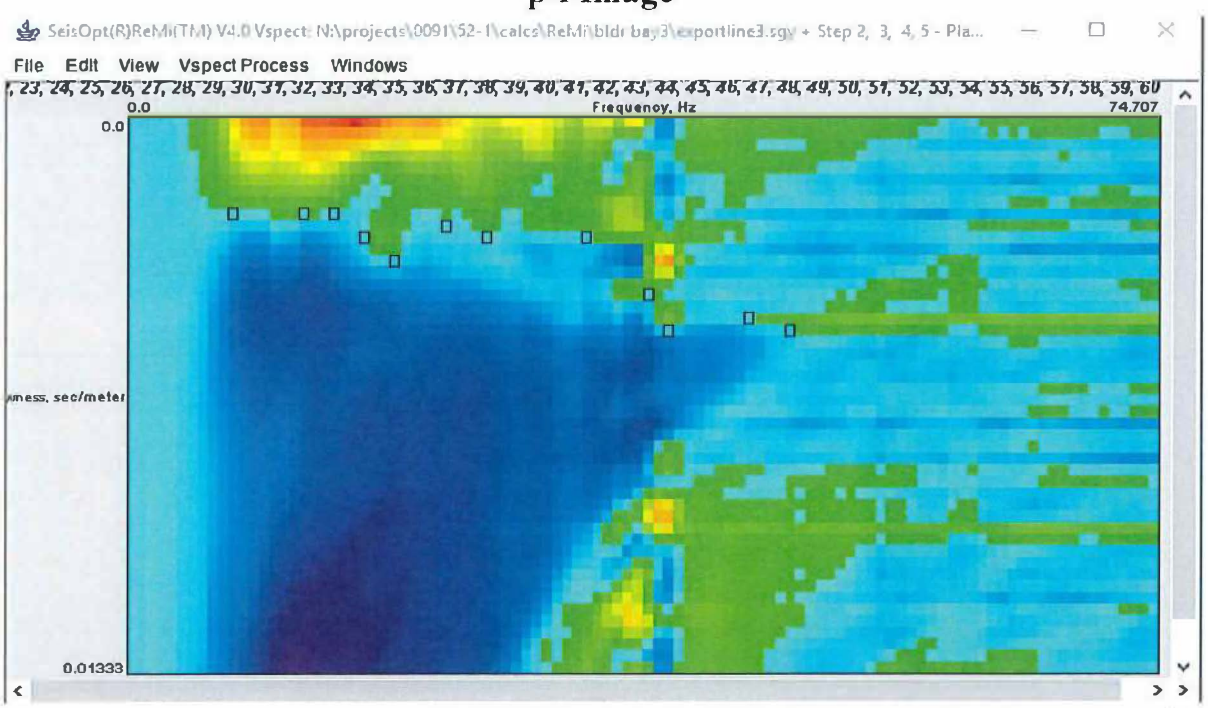


Boulder Bay - Line S_3

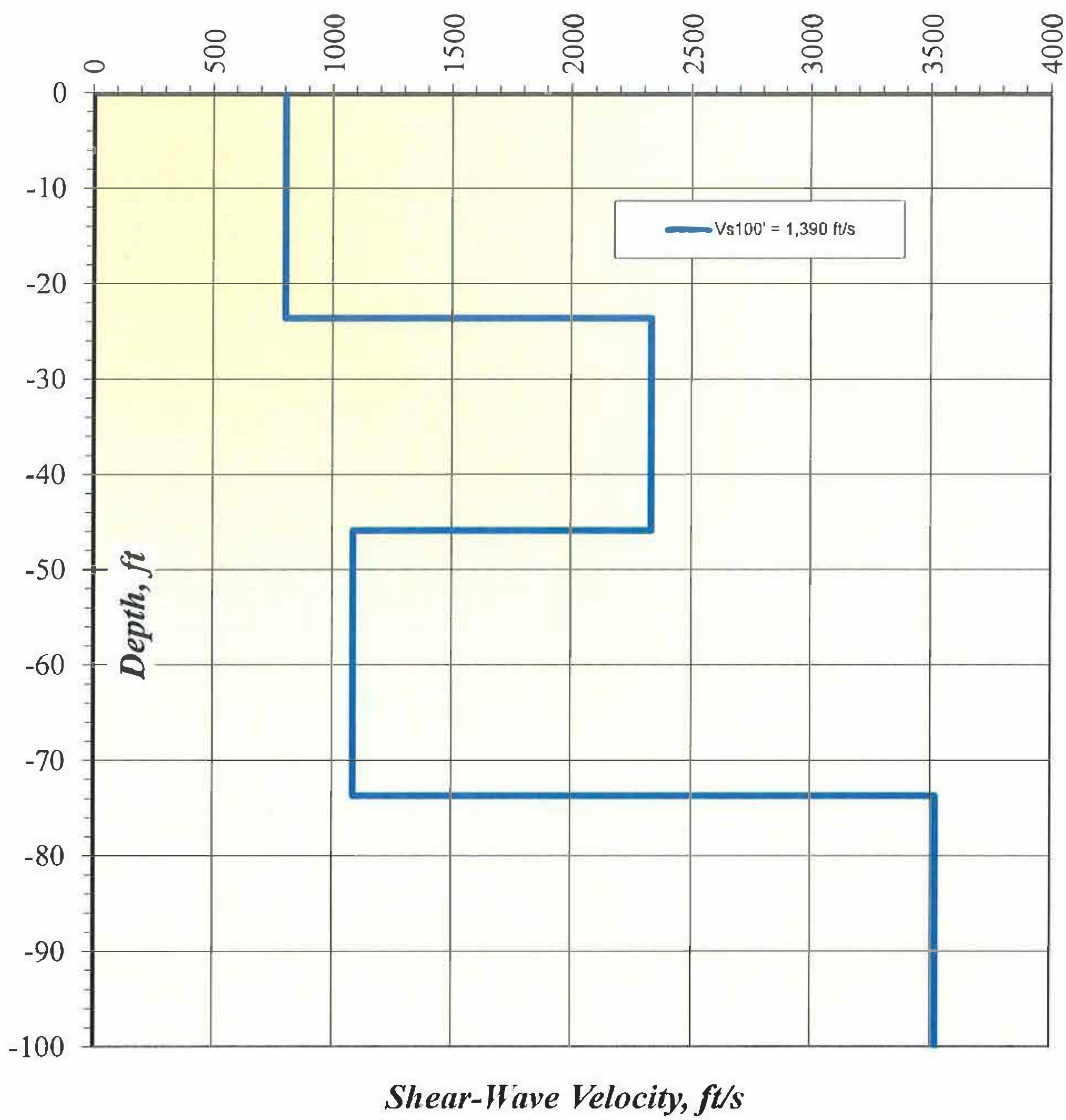
Dispersion Curve Showing Picks and Fit



p-f Image

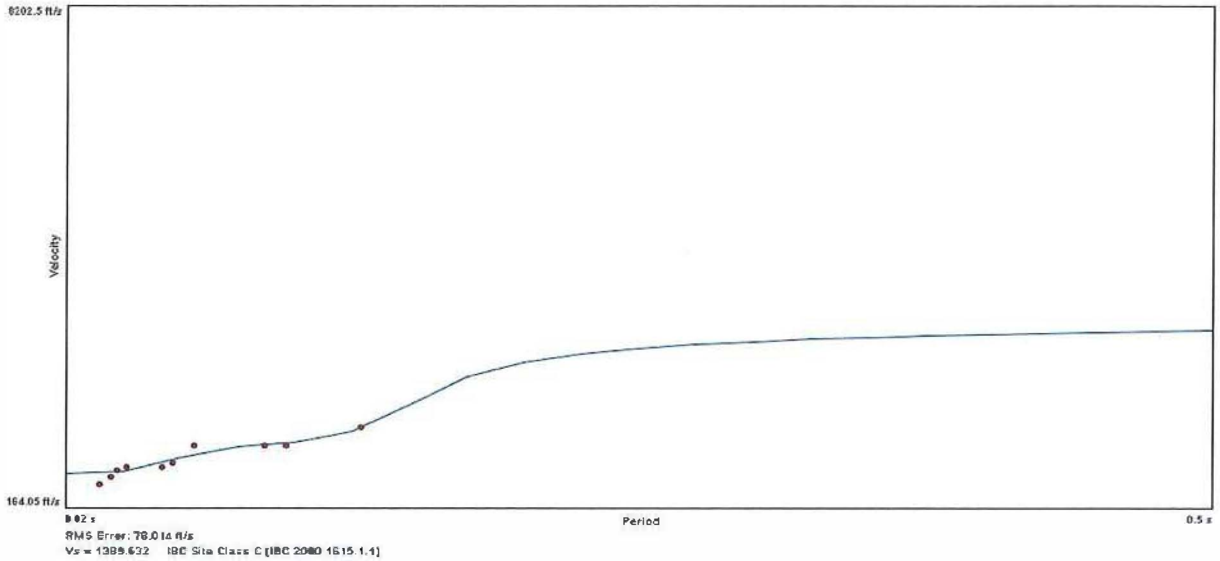


Shear Wave Velocity Modeling Results
Boulder Bay - Line S_4

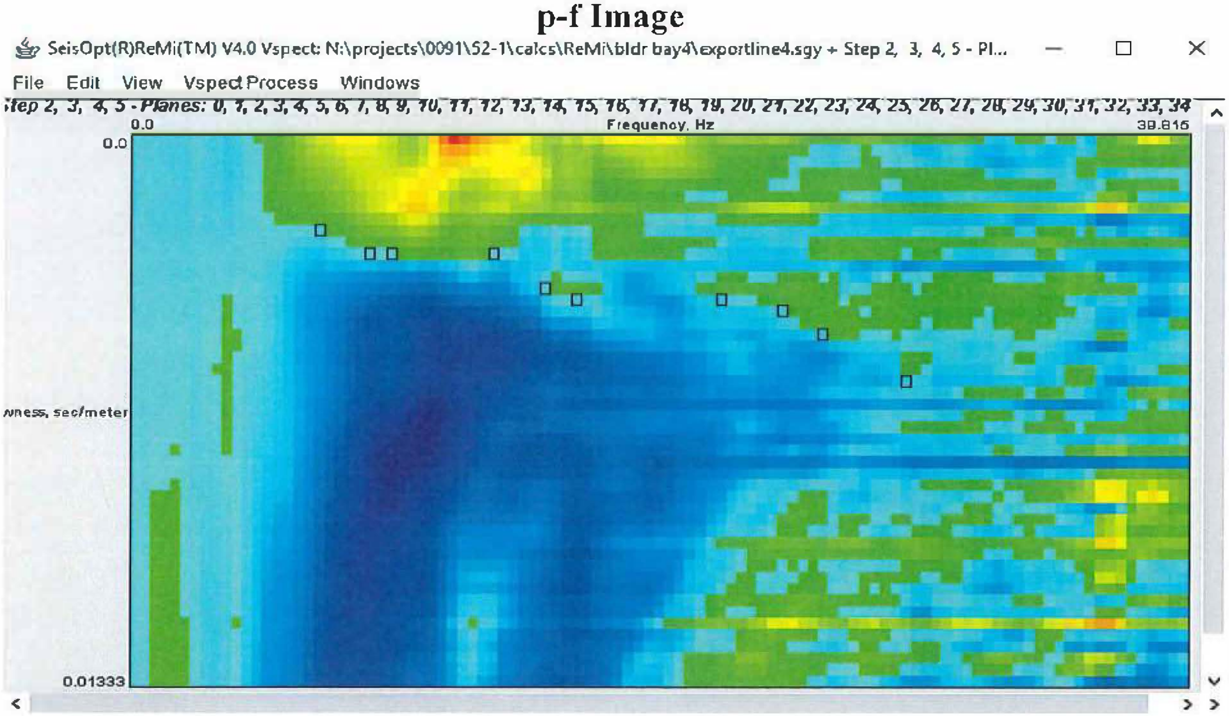


Boulder Bay - Line S_4

Dispersion Curve Showing Picks and Fit



p-f Image



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APPENDIX B

CHEMICAL TEST RESULTS

SilverState Analytical Laboratories
Sierra Environmental Monitoring

Silver State Labs-Reno
1135 Financial Blvd
Reno, NV 89502
(775) 857-2400 FAX: (888) 398-7002
www.ssalabs.com

Analytical Report

Workorder#: **18050305**
Date Reported: **5/16/2018**

Client: Black Eagle Consulting, Inc
Sampled By: J. Payne
Project Name: 0091-52-1
PO #: 0091-52-1

Laboratory Accreditation Number: NV015/CA2990

Laboratory ID	Client Sample ID	Date/Time Sampled	Date Received
18050305-01	0091-52-1 B-03 1-5'	04/11/2018 15:30	5/7/2018

Parameter	Method	Result	Units	PQL	Analyst	Date/Time Analyzed	Data Flag
Oxidation-Reduction Potential	SM 2580B	135	mV		LRB	05/09/2018 9:06	
pH	SW-846 9045D	6.24	pH Units		LRB	05/11/2018 9:55	
pH Temperature	SW-846 9045D	23.0	°C		LRB	05/11/2018 9:55	
Resistivity	EPA 120.1	30000	Ohms-cm		LRB	05/09/2018 9:49	
Sulfate	EPA 300.0	8	mg/Kg	2	JF	05/08/2018 15:29	
Sulfide	AWWA C105	Negative	POS/NEG		LRB	05/14/2018 15:30	

Laboratory Accreditation Number: NV015/CA2990

Laboratory ID	Client Sample ID	Date/Time Sampled	Date Received
18050305-02	0091-52-1 B-11A 2-5'	04/12/2018 9:00	5/7/2018

Parameter	Method	Result	Units	PQL	Analyst	Date/Time Analyzed	Data Flag
Oxidation-Reduction Potential	SM 2580B	119	mV		LRB	05/09/2018 9:06	
pH	SW-846 9045D	6.55	pH Units		LRB	05/11/2018 9:55	
pH Temperature	SW-846 9045D	23.0	°C		LRB	05/11/2018 9:55	
Resistivity	EPA 120.1	24000	Ohms-cm		LRB	05/09/2018 9:49	
Sulfate	EPA 300.0	<2	mg/Kg	2	JF	05/08/2018 15:55	
Sulfide	AWWA C105	Negative	POS/NEG		LRB	05/14/2018 15:30	

APPENDIX C

RIPPABILITY CHARTS

Rippers

D8R Ripper Performance

- Multi or Single Shank No. 8 Series D Ripper
- Estimated by Seismic Wave Velocities

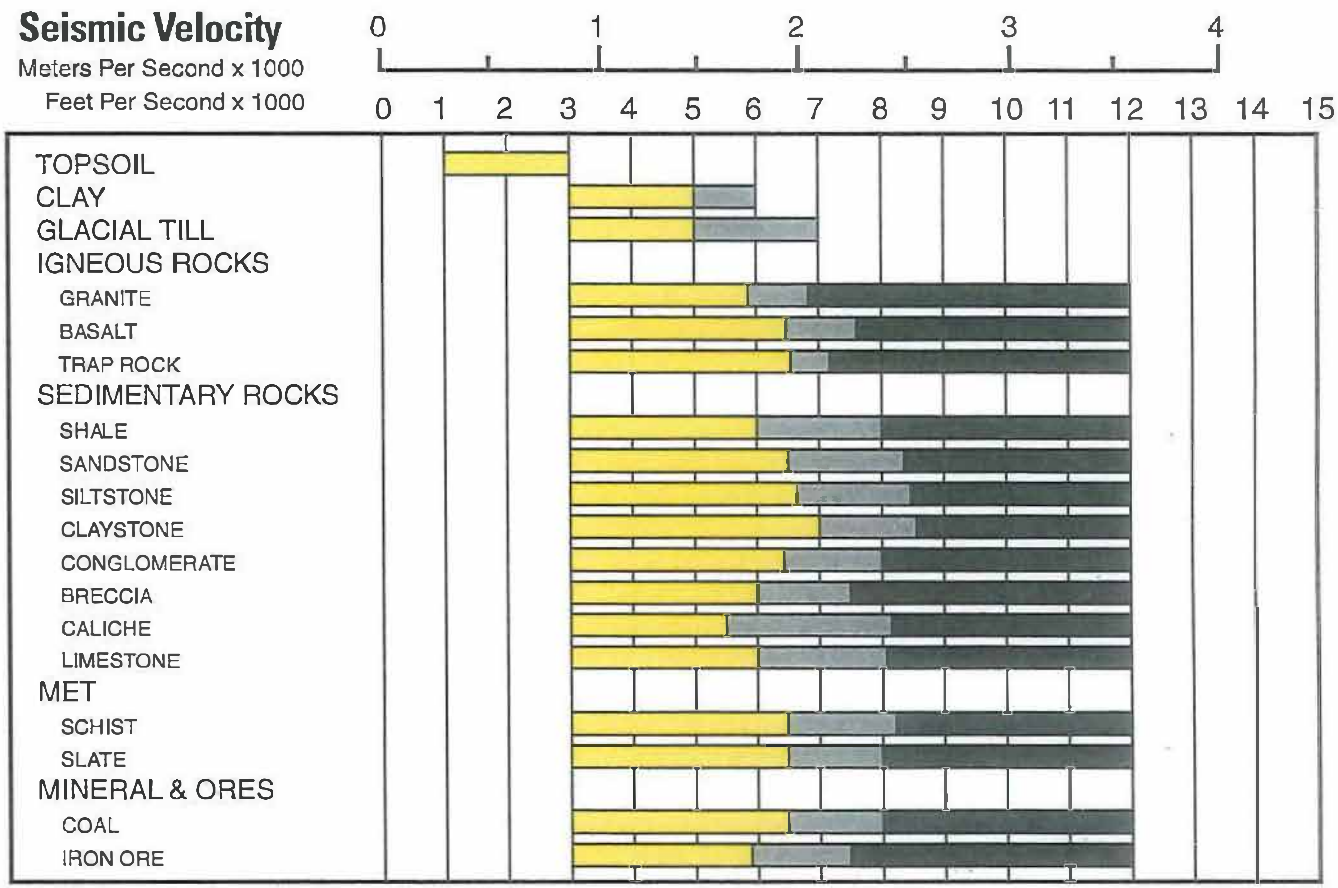

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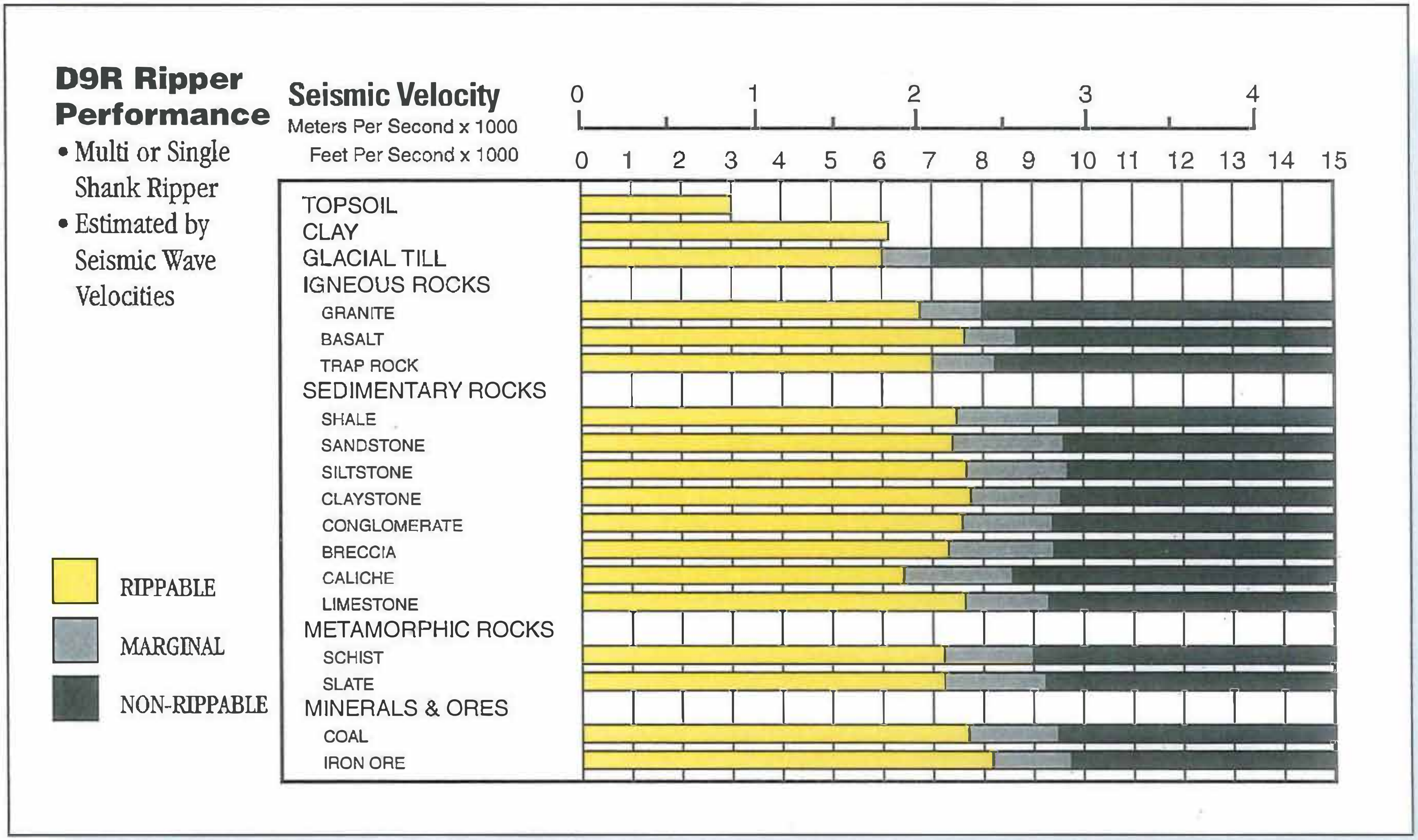
EXHIBIT R

Rippers



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