Black Eagle Consulting, Inc.

Geotechnical Baseline Investigation

Virginia Street Bridge

Reno, Nevada

September 18, 2012

Prepared for
Jacobs Civil Inc.

Black Eagle Consulting, Inc.
Geotechnical & Construction Services

Mr. Bryan Gant
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985 Damonte Ranch Parkway, Suite 100
Reno, NV 89521

September 18, 2012
Project No.: 0500-03-1

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RE: Geotechnical Baseline Investigation
Virginia Street Bridge
Reno, Nevada

Dear Mr. Gant:

Black Eagle Consulting, Inc. is pleased to present the results of our geotechnical baseline investigation for the above-referenced project. Our investigation consisted of research, field exploration, and laboratory testing, to allow compilation of site-specific geotechnical data for this project.

The proposed Virginia Street Bridge project will ultimately involve the design and construction of a new bridge structure where Virginia Street crosses over the Truckee River in downtown Reno, Nevada. The current phase of work involves a bridge-type selection study and feasibility analysis of the project.

The site is suitable to host the proposed structure. The project area is generally underlain by coarse river deposits that include a substantial amount of large boulders to the depths explored. Trenching, excavation, drilling, and finish grading in such materials will be difficult. These materials will also tend to slough and cave when exposed in excavations for prolonged periods of time. Ground water is present at approximately 15 feet below existing grade but may fluctuate with the level of the Truckee River. Petroleum hydrocarbon-based contaminants were encountered at ground water elevation in borehole B-02 near the southern bridge abutment. Excavation and dewatering operations will, therefore, require on-site containment, and off-site treatment and disposal. All personnel involved in handling these materials will require State of Nevada, Department of Industrial Relations, Division of Occupational Safety and Health-40 certification.

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 me.

Sincerely,

Black Eagle Consulting, Inc.

PROFESSIONAL ENGINEER - STATE OF NEVADA
PATRICK A. PILLING
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CIVIL
No. 9153

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

9-18-12

Copies to: Addressee (2 copies and PDF via email)

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Introduction

Presented herein are the results of the Black Eagle Consulting, Inc. (BEC) geotechnical baseline investigation and laboratory testing, to aid in the bridge-type selection study and feasibility analysis of the planned Virginia Street Bridge (VSB) replacement in Reno, Nevada. The recommendations presented are based on surface and subsurface conditions encountered in our explorations, and on details of the proposed project as described in this report. The objectives of this study were to:

1. **Determine general soil and ground water conditions present at the proposed VSB site.**
2. **Provide preliminary geotechnical design recommendations to aid in conceptual design and bridge-type selection for the VSB.**
3. **Provide a brief discussion of potentially significant construction issues based on materials and conditions encountered during site exploration.**

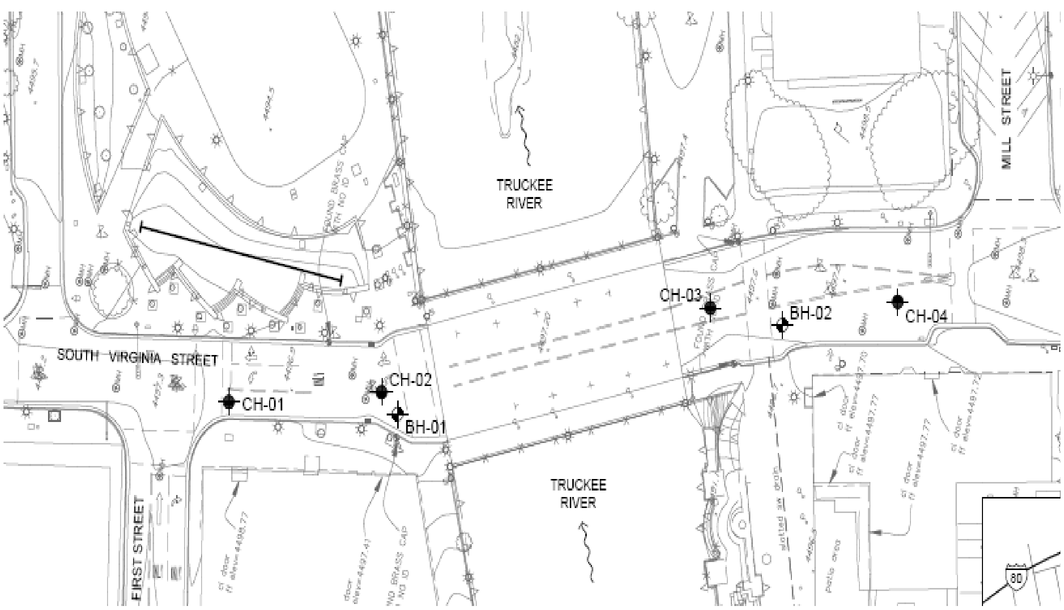
The area covered by this report is shown on Plate 1 (Plot Plan). Our investigation included field exploration, laboratory testing, and engineering analysis to determine the physical and mechanical properties of the various on-site materials. Results of our field exploration and testing programs are included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

The services described above were conducted in accordance with the Jacobs Subconsulting Agreement No. W4-X536-00-S11-0008, that was signed by Patrick Pilling of BEC on February 7, 2011.

Project Description

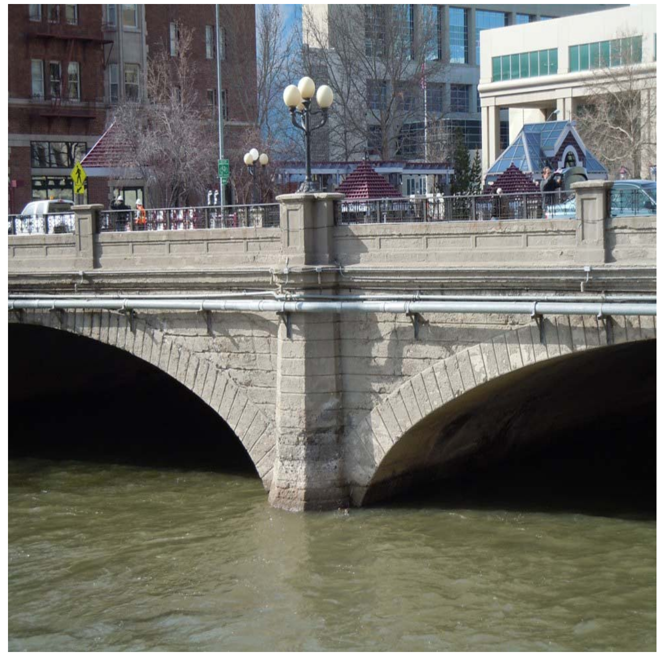
The proposed VSB site is located where Virginia Street crosses the Truckee River in downtown Reno, Nevada. The existing bridge is entirely contained in the southeast quarter of Section 11, Township 19 North, Range 19 East, M.D.M. The site is bordered to the north by 1st Street; to the south by Mill Street; and to the east and west by the Truckee River. The site presently hosts the existing VSB. Access to the site is obtained by Virginia Street.

The bridge is expected to consist of a single-span structure founded on either conventional shallow spread abutment footings or deep foundations at the abutments. Associated abutment retaining walls are anticipated. Finally, existing pavement approaches to the VSB will be reconstructed between 1st Street and Mill Street. Finish grade elevations must remain at or near existing grades in the area. Virginia Street is located within Nevada Department of Transportation (NDOT) right-of-way.



Existing VSB Layout

Site Conditions



Virginia Street Bridge

The VSB replacement site is located in the first block of South Virginia Street at the Truckee River in downtown Reno, Nevada. Virginia Street is a major traffic arterial that runs the length of Reno from north to south. The existing bridge was built in 1905 and is a reinforced Portland cement concrete, dual-arch bridge spanning the Truckee River. Originally known as Lakes Crossing Bridge, the bridge was added to the National Register of Historic Places by the United States Park Service in 1980.

The bridge deck lies at approximately 4,510 feet above mean sea level (msl) elevation and spans the west to east-flowing Truckee River which, in 2010, carried flows ranging from 247 cubic feet per second (cfs) to 2,270 cfs (United States Geological Survey [USGS], 2011). Numerous buried utilities are present along the river banks and crossing South Virginia Street near the bridge abutments. Some utilities cross the river attached to the exterior of the bridge. Electrical cable, which serves the bridge lighting system, is present beneath the sidewalks on both sides. A City of Reno ground water monitoring well is present just beyond the south end of the bridge, in the existing crosswalk.

Exploration

Prior to site exploration, BEC personnel met with city of Reno and Jacobs staff to coordinate exploration activities, locations and dates. Underground Service Alert (USA) was contacted to identify and mark existing utilities within the public right-of-way. Once identified, multiple exploration locations were marked in the field by BEC personnel. Subsequently, each exploration location was examined by a private utility locator, Comstock Inspection LLC, for possible utility conflicts.

During exploration, traffic control was provided by Nevada Barricade of Reno, Nevada, and on-site material containment as well as drilling fluid and cuttings removal was supplied by H2O Environmental of Reno, Nevada.

VSB North Abutment Exploration Drilling

Drilling

The VSB site was explored on March 16 and 17, 2011 by drilling 2 test borings performed by Cascade Drilling of Rancho Cordova, California. Prior to drilling, Diversified Concrete Cutting Inc. cored a 12-inch-diameter access through the existing pavement. The borings were then advanced using a combination of Overburden Drilling with Excentric Bit (ODEX) casing and reverse-circulation air with sonic casing advance technologies using a shop-built sonic drill rig. The drill casing was 7 inches in diameter and the button bit was 5-7/8 inches in diameter. The maximum depth of exploration was approximately 100 feet below the existing grade of Virginia Street; ground water levels were recorded when encountered. The locations of the test borings are shown on Plate 1.

The native soils were sampled in-place every 5 to 10 feet by use of a standard, 2-inch outside-diameter (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.

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

All materials encountered during drilling were placed within on-site 25 cubic yard roll-off storage units to ensure that any potentially contaminated soil or water were fully contained at site.

Upon completion, all borings were pressure grouted with cement and water grout up to the existing grade of Virginia Street in accordance with State of Nevada, Department of Water Resources (DWR) requirements. The core holes in Virginia Street were backfilled to existing street grade with quick-set cement and water grout.

Pavement Coring

Virginia Street was cored near the bridge on March 16 and 17, 2011. The purpose of coring was to determine the thickness of the existing pavement and underlying aggregate base sections, and to obtain representative samples of the aggregate base and underlying subgrade materials. Diversified Concrete Cutting Inc. cored the existing road surface with a 12-inch-diameter core barrel at 4 locations. Core holes were placed to provide an accurate representation of the thickness of asphalt and aggregate base along the length of South Virginia Street between 1st Street and Mill Street. The aggregate base section and the underlying subgrade soils were augered and excavated by hand for sampling. The depth of exploration ranged from 2 to 3 feet, depending on the number of cobbles encountered. The core holes were backfilled, compacted, and subsequently capped with 3 to 4 inches of quick-set concrete. All locations were identified in the field by approximate means and are shown on Plate 1.

Asphalt cores, aggregate base samples, and representative samples of the subgrade soils were collected from each of the exploration sites, placed in sealed plastic bags and returned to our Reno, Nevada office for potential testing.

Material Classification

A geotechnical engineering technician and/or geologist examined and identified all soils in the field in accordance with ASTM D 2488. During drilling/coring, representative bulk samples were placed in buckets and sealed plastic bags and returned to our Reno, Nevada, laboratory for testing. Additional soil classification was subsequently performed in accordance with ASTM 2487 (Unified Soil Classification

System [USCS]) upon completion of laboratory testing as described in the **Laboratory Testing** section. Logs of borings and pavement cores are presented as Plate 2 (Boring Logs). A USCS chart has been included as Plate 3 (Graphic Soils Classification Chart).

Geophysical Survey

Exploration of the site also included performing a Refraction Microtremor (ReMi) survey. This geophysical survey measures the average shear-wave velocity within the upper 100 feet of subsurface materials in order to accurately identify the appropriate soil profile for use in structure design. The work was performed by BEC personnel on March 23, 2011 at a location near the bridge, as shown on Plate 1. A summary report has been included in Appendix A (Geophysical Survey).

Laboratory Testing

Grain Size Analysis

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

Index Tests

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

Direct Shear Test

R-Value Tests

Resistance value (R-value) tests (ASTM D 2844) were performed on representative samples of subgrade soil present beneath the existing pavement structural section in Virginia Street. R-value testing is a measure of subgrade strength and expansion potential and is used in design of flexible pavements. Results of the R-value tests are shown on Plate 5 (R-Value Test Results).

Direct Shear Tests

Direct shear tests (ASTM D 3080) were performed on representative samples of subgrade material. The tests were run on remolded, inundated samples under various normal loads in order to develop a Mohr's strength envelope. Remolded samples were first screened to remove particles larger than the Number 4 sieve prior to testing. Results of these tests are shown on Plate 6 (Direct Shear Test Results) and were used in calculation of bearing capacities, friction factors, and lateral earth pressures.

Chemical Tests

Chemical testing was performed on representative samples of subsurface soils to evaluate the site materials' potential to corrode steel and Portland cement concrete in contact with the ground. The samples were tested for pH, resistivity, redox potential, soluble sulfates, sulfides, and chlorides. The results of the chemical tests are contained in Appendix B (Chemical Test Results). Chemical testing was performed by Sierra Environmental Monitoring Laboratory (SEM) of Reno, Nevada.

Environmental Tests

Because free-floating petroleum product was encountered on the ground water surface in borehole B-02, drill cuttings and water from both boreholes were combined and removed from the site by the certified hazardous materials transport company H2O Environmental. The solid and liquid drilling wastes were stored at the H2O Environmental storage facility pending results of chemical profiling to determine the acceptable disposal methods(s). Laboratory analysis included purgeable and extractable Total Petroleum Hydrocarbons (TPH), Environmental Protection Agency (EPA) method 5230/8260 for volatile organic compounds, and required RCRA metals. Results of drilling waste profiling are included in Appendix C (Environmental Test Results).

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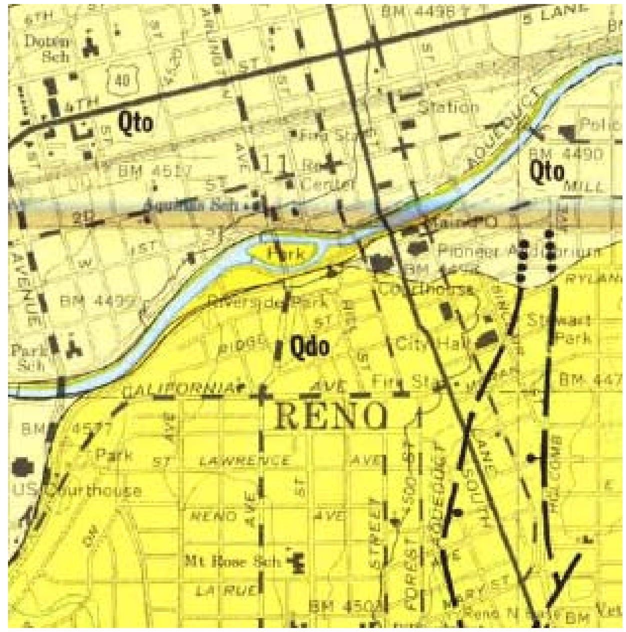
Geologic and General Soil Conditions



Geologic Map

The site lies on the broad flood plain of the Truckee River in an area mapped by the Nevada Bureau of Mines and Geology (Bonham and Bingler, 1973) as Quaternary Age Tahoe outwash. Sedimentation in the Truckee Meadows has been in progress at varying rates since the formation of the block-faulted basin. Most of the sediments, including the coarse grain, gravelly sands that underlie the majority of the Truckee Meadows, were deposited quite abruptly in the post-glacial period during torrential flooding. With the advent of a warmer, drier climate, the volume and size distribution of sediment transported was greatly reduced, and the sedimentation process became largely limited to the reworking of earlier deposits.

Site exploration at the bridge abutments revealed the subsurface is primarily composed of coarse granular soils to the depths explored (101.5 feet below existing grade). The materials were classified as slightly moist to wet, dense to very dense, and as containing 9 to 26 percent non-plastic to medium plasticity fines. The plasticity of the fines fraction generally increased with depth.

During exploration, the presence of abundant cobbles and boulders was indicated by drilling character. Drilling and sampling methods, however, did not allow for an estimation of the actual quantity and size of the cobbles and boulders present. Abundant cobbles and large boulders are known to be present in the downtown Reno area.

Cuttings collected from the 10-foot depth were wet and samples collected from 15 feet below surface contained free water indicating the ground water surface at the time of exploration was somewhere in the 10 to 15-foot range below surface. Water produced from borehole B-02 at the 15-foot depth contained an unidentified free-floating petroleum product.

Exploration in Virginia Street revealed the existing pavement structural section consists of asphalt concrete pavement underlain by aggregate base. The measured thickness of the existing asphalt concrete pavement and associated aggregate base is summarized in Table 1 (Existing Virginia Street Pavement Structural Sections).

TABLE 1 – EXISTING VIRGINIA STREET PAVEMENT STRUCTURAL SECTIONS

Core Hole (CH) No.	Location	Existing Asphalt Thickness (inches)	Existing Aggregate Base Thickness (inches)	Subgrade Description[1]
CH-01	Center of Southbound Lane Near Mill Street	11	13	GP-GM
CH-02	Center of Southbound Lane Near North Bridge Abutment	11	13	SP-SM
CH-03	Center of Southbound Lane Near South Bridge Abutment	12	12	GP-GC
CH-04	Center of Southbound Lane Near Mill Street	9	15	GP-GC

[1] USCS Descriptions – see Plate 3.

The underlying subgrade materials typically consist of poorly graded gravel with clay and sand. This material was classified as slightly moist, dense, and as containing 5 to 11 percent non-plastic to low plasticity fines.

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 active regions, outside of Alaska, are in the vicinity of the San Andreas Fault system of western California. Other seismically active areas include the Wasatch Front in Salt Lake City, Utah, which forms the eastern boundary of the Basin and Range physiographic province, and the eastern front of the Sierra Nevada Mountains, which is the western margin of the province. The Reno-Sparks area lies along the eastern base of the Sierra Nevada, within the western extreme of the Basin and Range.

The Truckee Meadows lies within an area with a high potential for strong earthquake shaking. Seismicity within the Reno-Sparks area is considered about average for the western Basin and Range Province (Ryall and Douglas, 1976). It is generally accepted that a maximum credible earthquake in this area would be in the range of magnitude 7 to 7.5 along the frontal fault system of the Eastern Sierra Nevada. The most active segment of this fault system in the Reno area is part of the Mount Rose Fault System located at the base of the mountains near Thomas Creek, Whites Creek, and Mount Rose Highway, some 10 miles south of the project.

Geologic Hazards Map

Faults

The nearest faults to the project site are part of the Mount Rose Fault System. The Mount Rose Fault System is a collection of north and northwest-striking subparallel normal faults which, in the Reno area, define the eastern slope of the Sierra Nevada Mountain Range (Sawyer, 1999). The published earthquake hazards map (Bingler, 1974) shows Late Pleistocene traces of the Mount Rose Fault Zone extending to within 1,500 feet southeast of the bridge, but no Holocene movement has been documented within 10 miles of the project site.

The Nevada Earthquake Safety Council (NESC, 1998) has developed and adopted the criteria for evaluation of Quaternary age earthquake faults. *Holocene Active Faults* are defined as those with evidence of movement within the past 10,000 years (Holocene time). Those faults with evidence of displacement during the last 130,000 years are termed *Late Quaternary Active Faults*. A *Quaternary Active Fault* is one that has moved within the last 1.6 million years. An *Inactive Fault* is a fault *without recognized activity within Quaternary time* (last 1.6 million years). Holocene Active Faults normally require that occupied structures be set back a minimum of 50 feet (100-foot-wide zone) from the ground surface fault trace. An *Occupied Structure* is considered …. *a building, as defined by the International Building Code, which is expected to have a human occupancy rate of more than 2,000 hours per year.*

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

Based on the geologic map, the faults in the vicinity of the project are Quaternary Active. Since no faults are mapped as crossing through the subject site or were identified during site exploration, no additional fault hazard investigation or structure off set from faulting is necessary.

Ground Motion and Liquefaction

Mapping by the USGS (2007) indicates that there is a 2 percent probability that a *bedrock* ground acceleration of 0.65g will be exceeded in any 50-year interval. Only localized amplification of ground motion would be expected during an earthquake.

Because the site area is underlain by dense granular soils, liquefaction potential is minimal.

Flood Plains

The Federal Emergency Management Agency (FEMA) has identified the site as lying in Zone AE with a 100-year base flood elevation of 4,495 feet (FEMA, 2009). The river has been at or above flood stage numerous times over its history. One of the reasons the bridge is being replaced is to reduce restriction during flood stage and to avoid the potential for the existing bridge to trap debris.

Other Geologic Hazards

A low potential for dust generation is present if grading is performed in dry weather. Free-floating petroleum product was documented at depth at the site. No other geologic hazards were identified.

Discussion and Recommendations

General Information

At this time, bridge-type selection and regulatory jurisdiction with respect to construction specifications has yet to be completed/determined such that final geotechnical design and construction recommendations are not possible. In order to aid in bridge-type selection and construction cost estimating, preliminary geotechnical design recommendations are presented to allow for conceptual structural design of the VSB and its associated approach improvements. In addition, potential construction issues and associated impacts to the project are briefly discussed.

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

Design of the VSB structure will follow Load Resistance Factor Design (LRFD) design methodologies (American Association of State Highway and Transportation Officials [AASHTO], 2007), as adopted by NDOT. All other improvements, including roadway improvements to Virginia Street and underground utilities, will follow City of Reno design standards.

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

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

Free-floating petroleum contamination is present at or near the ground water elevation at the site, in particular at the south abutment. When this material is encountered, either during dewatering or excavation activities, it will require on-site containment, and off-site treatment disposal. In addition, any personnel that will be handling such material will require State of Nevada, Department of Industrial Relations, Division of Occupational Safety and Health (OSHA)-40 certification.

Preliminary Geotechnical Design Recommendations

Seismic Design Parameters

The AASHTO design manual (AASHTO, 2007) shows the horizontal bedrock acceleration to be approximately 0.38g to 0.39g with a 10 percent probability of exceedance in 50 years. Per NDOT Materials Division policy, all bridges and other structures in this area should be designed for a horizontal bedrock acceleration of 0.40g. Therefore, an acceleration coefficient, A, of 0.40 is appropriate for use in design of the VSB. In addition, the VSB site is located in a Seismic Zone 4. Finally, a Site Coefficient, S, of 1.2, which corresponds to a Soil Profile Type II, is appropriate for use in design of the VSB. This value is supported by the results of the ReMi survey, which indicate the materials to a depth of 100 feet exhibit a weighted average shear-wave velocity of 1,532 feet per second.

Foundation Design Parameters

The materials present at both abutments of the existing VSB consist of dense to very dense granular river deposits that are considered excellent foundation materials. Depending on final loading conditions, conventional shallow foundations or deep foundations may be appropriate to support the proposed VSB. Therefore, preliminary geotechnical design parameters for both foundation systems are provided below. Because materials present contain a significant amount of large boulders, deep foundations would most likely require drilled shafts as opposed to driven piles. Drilled shafts should be sized taking into account the fact that there are abundant large boulders present in the subsurface materials.

Conventional Shallow Foundations

Design of conventional shallow foundations using LRFD criteria (AASHTO, 2007) considers service limit states, strength limit states, and extreme events limit states. Service limit state analysis considers settlement, horizontal movement and overall stability of the foundation, as well as scour at the design flood. Strength limit state analysis considers structural resistance, nominal bearing resistance, overturning or excessive loss of contact, sliding at the base of the footing, and loss of lateral and vertical support due to scour at the design flood event. Extreme event analysis considers scour, vessel and vehicle collision and seismic loading. Foundation design parameters for each state described above are developed using resistance factors, which are specified by AASTHO for different bearing material types. Since conventional shallow foundation design is often controlled by settlement, spread footings are typically proportioned at the service limit state and checked for adequate design at the strength and extreme limit states.

Settlement of footings on cohesionless soils can be estimated using conventional methods (AASHTO, 2007). The Schmertman Method (NAVFAC, 1986a) was used to estimate the bearing resistance values for various footing widths to limit the settlement 1 inch or less in the service limit state analysis. Differential movement between footings with similar loads, dimensions, and base elevations should not exceed two-thirds of the values provided for total movements. Much of the anticipated movement will occur during the construction period as loads are applied.

The factored bearing resistance of footings at the strength limit state is determined by applying a resistance factor to the nominal bearing resistance, which is determined using conventional methods (AASHTO, 2007). In our strength limit state analysis, the nominal bearing resistance values for various footing widths were estimated using the conventional bearing capacity equation, but utilizing reduced bearing capacity factors assuming the VSB footings will be founded on or near a sloping ground.

Preliminary shallow foundation design parameters for various footing widths are presented in Plate 7 (Shallow Foundation Design Parameters). Since only preliminary structural design will be performed during this phase of the project, extreme limit design parameters have not been developed. The overall stability of the VSB footings will also be analyzed once the footing size and orientations are finalized.

Deep Foundations

Geotechnical design criteria for deep foundations, in particular drilled shafts, incorporates the same service limit, strength limit and extreme events limits states as shallow foundations. Service limit state settlement and deflection criteria are determined in accordance with conventional methods (AASHTO, 2007). The axial settlement design criteria for drilled shaft foundations was determined using the standard load-deflection curves for cohesionless soils (O'Neil and Reese, 1999).

The design of drilled shaft foundations at the strength limit state also considers axial compression, uplift resistance, lateral resistance, down drag and punching for single shafts and groups. Drilled shaft resistance factors are used in conjunction with the nominal resistance estimated using conventional evaluation methods. The nominal axial compressive and uplift resistance of drilled shaft foundations was estimated using the procedures recommended by O'Neil and Reese (1999) for cohesionless gravels and sands.

Preliminary drilled shaft design parameters for various shaft diameters and depths are presented in Plate 8 (Drilled Shaft Design Parameters). Since only preliminary structural design will be performed during this phase of the project, service limit state lateral deflection and extreme limit design parameters have not been developed.

Retaining Wall Design Parameters

Design of conventional cantilever retaining walls using LRFD criteria (AASHTO, 2007) considers service limit states, strength limit states, and extreme events limit states. Service limit state analysis considers vertical and lateral displacement, as well as overall stability. Strength limit state analysis considers bearing resistance failure, lateral sliding, excessive loss of base contact, and structural failure. Extreme event analysis considers scour, vessel and vehicle collision and seismic loading. The structural engineer also evaluates the wall for overturning and scour conditions.

Bearing resistance and sliding at the strength limit state and vertical wall movement (settlement) of retaining wall footings are evaluated using the same methods for conventional shallow foundations. Therefore, shallow foundation design parameters provided in Plate 8 can also be used for preliminary sizing of retaining wall footings.

Lateral wall movement at the service limit state is evaluated using active, passive and at-rest lateral earth pressures developed using conventional methods (AASHTO, 2007), while overall stability is analyzed using limit-equilibrium methods of analysis.

Pseudo-static methods for seismic analysis can incorporate the Mononobe and Okabe (Federal Highway Administration [FHA], 1998) approach. Active and passive lateral earth pressure values for static loading conditions were estimated using the Coulomb equation and charts published by NAVFAC (1986a), respectively. Active lateral earth pressure values for seismic analysis were estimated using the Mononobe and Okabe equation and a horizontal acceleration coefficient equal to one-half the acceleration coefficient for the site.

Preliminary lateral earth pressure values for static and pseudo-static conditions are presented in Table 2 (Lateral Earth Pressure Values [Equivalent Fluid Density]). The passive lateral earth pressure values from Table 2 are factored and were developed by applying a reduction factor for the estimated nominal lateral earth pressure values (AASHTO, 2007). Appropriate load factors shall be applied to the active and at-rest lateral earth pressure values for various limit state design cases.

TABLE 2 - LATERAL EARTH PRESSURE VALUES (EQUIVALENT FLUID DENSITY), pcf

Retained Slope	Static		Dynamic	
	Active*	Passive**	Active*	Passive**
Level	23	650	38	650

*For walls that are free to yield at least 0.2 percent of the wall height.
**Factored passive earth pressure values developed by applying a reduction factor of 0.5 to the nominal passive lateral earth pressure values.

Restrained walls should be designed to resist an at-rest equivalent fluid density of 42 pounds per cubic foot (pcf).

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 native gravels, or on properly compacted structural fill, may be designed using a factored coefficient of base friction of 0.48. The factored coefficient of base friction was developed by applying a reduction factor of 0.8 to the nominal coefficient of base friction where wall footings will be underlain by native gravels or compacted structural fill.

Retaining Wall Drainage Design

Retaining wall drainage can be accomplished by installing granular backfill and a weep hole drain system at the bottom of the wall. The drain rock section should be a minimum of 18 inches wide and extend to within 12 inches of finish grade. A drainage filter geotextile such as Mirafi® *140NS* or approved equivalent should be placed between the drain rock backfill and the native soils to prevent migrations of fines into the drain rock. Such a drainage geotextile should satisfy the specifications provided in Table 3 (Drainage Geotextile Material Requirements)

TABLE 3 - DRAINAGE GEOTEXTILE MATERIAL REQUIREMENTS	
Grab Tensile (ASTM D 4632)	90 lbs.
Puncture Strength (ASTM D 4833)	50 lbs.
Burst Strength (ASTM D 3786)	150 psi.
or if native soils have sharp, angular rocks:	
Grab Tensile (ASTM D 4632)	130 lbs.
Puncture Strength (ASTM D 4833)	75 lbs.
Burst Strength (ASTM D 3786)	250 psi.

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 testing indicate that the site foundation soils are not corrosive to ductile-iron pipe in contact with the ground (American Water Works Association, 1999). As a result, corrosion protection of metal pipe in contact with the ground is not considered necessary.

Portland Cement Concrete Mix Design Parameters

Soluble sulfate content has been determined for representative samples of the site foundation soils, and the results of the testing indicate that concrete in contact with the site foundation soils should experience negligible to minimal degradation due to reaction with soil sulfate. Therefore, Type II cement can be used for all concrete work. Concrete placed for dedicated flatwork on this project should exhibit a minimum 28-day compressive strength of 4,000 pounds per square inch (psi) and exhibit a maximum water to cement ratio of 0.50.

Asphalt Concrete Flexible Pavement Design

R-value testing (ASTM 2844) was conducted on 3 samples of native subgrade soils collected from the core holes. R-value results are summarized below in Table 4 (Conversion of R-Value to Resilient Modulus). The conversion to resilient modulus was taken from Figure 6.2 of the NDOT Pavement Structural Section Design and Policy Manual for flexible pavement (NDOT, 1997). For design purposes, we have used a subgrade R-value of 44.

TABLE 4 - CONVERSION OF R-VALUE TO RESILIENT MODULUS

Core Hole (CH) No.	Depth (ft)	R-Value	Conversion Factor	Resilient Modulus (M_r) (psi)
CH-01	1.0	44	NDOT	10,760
CH-02	1.0	65	NDOT	21,490
CH-03	1.0	68	NDOT	23,720

Calculation of Equivalent 18-Kip Single-Axle Load

Traffic counts, including a breakdown of truck distribution, were not available for this segment of Virginia Street. Traffic projections were provided by Jacobs for years 2008 and 2030 (Appendix D [Traffic Data and Equivalent 18-kip Single-Axle Load [ESAL] Calculations]). The traffic projections show growth from 14,050 vehicles per day (vpd) in 2008 to 17,390 vpd in 2030. We applied that growth rate (1.02 percent) to the 2008 values to calculate a 2012 initial daily traffic of 14,633 vpd. We then used the typical truck percentages and factors from the NDOT 2009 Annual Traffic Report (NDOT, 2009) for a principal urban arterial, to calculate the truck distribution, based on a total of 14,633 vpd. The NDOT truck traffic distribution is summarized in Table 5 (Virginia Street-NDOT Typical Traffic for Major Arterial) and included in Appendix D. Trucks account for 6.71 percent of the total daily traffic, based on the NDOT data.

TABLE 5 - VIRGINIA STREET-NDOT TYPICAL TRUCK TRAFFIC FOR MAJOR ARTERIAL										
	Buses	**2-Axle Single Unit**	**3-Axle Single Unit**	**4-Axle Single Unit***	**4-Axle Single Trailer***	**5-Axle Single Trailer**	**6-Axle Single Trailer**	**5-Axle Multiple Trailer**	**6-Axle Multiple Trailer**	**7-Axle Multiple Trailer**
Vehicle Percent**	0.65	1.70	0.58	0.80	-	2.15	0.13	0.25	0.11	0.34
NDOT Truck Factor for Major Arterial**	1.012	0.269	0.906	1.088	1.088	1.223	1.313	2.253	0.952	1.920

*2009 NDOT data does not separate 4-axle trucks.
**Automobiles, motorcycles, and pick-up trucks account for 93.3 percent of the total traffic.

The design 20-year ESAL ($ESAL_{20}$) is shown below; the calculations are included in Appendix D.

TABLE 6 - CALCULATED DESIGN 20-YEAR ESAL	
Street	**$ESAL_{20}$**
Virginia Street (both directions)	7.3×10^6

Pavement Design

The AASHTO design method (AASHTO, 1993), which is used by NDOT, was employed in pavement design for a standard 20-year life. The following pavement design values were used in the procedure, in accordance with the NDOT Pavement Design Manual (NDOT, 1997).

TABLE 7 - NDOT FLEXIBLE PAVEMENT DESIGN VALUES	
Factor	**Value**
Reliability (R)	90%
Overall Standard Deviation (So)	0.45
Initial Serviceability	4.2
Terminal Serviceability	2.5
Pavement Coefficients:	
Plant Mix Surface	0.35
Cement-Treated Base	0.20
Aggregate Base	0.10

Two recommended flexible pavement alternates are summarized in Table 8 (Pavement Replacement Alternates for Virginia Street). Calculations are included in Appendix D.

TABLE 8 - PAVEMENT REPLACEMENT ALTERNATES FOR VIRGINIA STREET			
Alternate	**Asphalt Concrete**	**Type 2 Aggregate Base**	**Description**
1	11.5	0	Full-Depth Asphalt Concrete
2	8.0	13*	Asphalt Concrete over Untreated Aggregate Base
*Existing asphalt concrete and recycled base left in place or new imported base.			

The City of Reno minimum structural section for an arterial street is 6 inches of asphalt concrete over 12 inches of aggregate base. Using the structural coefficients of Table 7 yields a structural number of 3.30 for the minimum section. The existing structural section has a structural number ranging from 4.65 to 5.40 assuming all new material. The alternate sections both have a structural number of at least 4.03, greatly exceeding the City of Reno minimum. Generally, NDOT policy is to match the existing structural section, in this case 12 inches of asphalt concrete over 12 inches of aggregate base. Based on our calculations, the 12 inches of asphalt concrete alone would be sufficient to accommodate the projected 20-year traffic loading on the lowest strength subgrade soil that we tested. The existing section, if new, could accommodate an $ESAL_{20}$ of 60 million, or over 8 times the projected traffic. If 12 inches of asphalt concrete is simply removed and replaced, it will be underlain by 12 inches of existing base, essentially meeting NDOT standards of matching the existing section. The full-depth section also minimizes the depth of intrusion and potential to interfere with existing utilities. Alternate 2 could potentially consist of 8 inches of new asphalt concrete underlain by 4± inches of existing asphalt concrete and 12 inches of existing base, if only the surface 8 inches is milled out and replaced. Clearly, this would be more than adequate for the anticipated 20-year traffic.

Portland Cement Concrete Rigid Pavement

Depending on the length of replacement, Portland cement concrete (rigid) pavement may be practical for reconstruction of Virginia Street. Structural sections for rigid pavement we calculated with the same traffic data and growth rate, as used for flexible pavement. Both 20-year and standard 40-year design life sections are summarized below in Table 9 (Recommended Rigid Pavement Alternates for Virginia Street). Calculations are included in Appendix D.

TABLE 9 – RECOMMENDED RIGID PAVEMENT ALTERNATES FOR VIRGINIA STREET				
Design Life (years)	**Design ESAL**	**Design K-value (kci)**	**Portland Cement Concrete (inches)**	**Type 2 Base (inches)**
20	7.3×10^6	230	11.5	4*
40	1.46×10^7	230	12.5	4*
*Existing recycled base left in place or new imported base.				

Preliminary Construction Considerations

At the time of this report, regulatory oversight with respect to construction specifications had yet to be determined. As a result, detailed construction recommendations (i.e. compaction and material requirements, etc.) cannot be provided. The following discussion however, presents a general discussion of potentially significant construction issues that must be taken into consideration during the design process:

- Abundant cobbles and large to very large boulders are present in the subsurface of this site. Such materials will make drilled shaft installation, trenching and excavation, and finish grading extremely difficult. Significant screening of this material (and consequent quantity shrinkage) will also be required prior to any re-use of the existing materials as structural fill.

- The existing granular materials will tend to slough and cave when exposed in excavations, trenches, or foundation borings for prolonged periods of time. Sloughing could begin quickly as the exposed surface begins to dry out. Therefore, temporary construction slopes will need to be flatter than for cohesive soils in order to minimize this potential.

- This Truckee River bisects this project. The Truckee River is an environmentally sensitive river that cannot directly receive dewatering or runoff product. In addition, free-floating petroleum contamination is present at or near the ground water elevation. As a result, dewatering efforts, which will be necessary during construction, and excavations to such depth will require on-site containment, and off-site treatment disposal. In addition, any personnel that will be handling such material will require OSHA-40 certification. Finally, best management practices with respect to storm water runoff during construction will need to be adhered to in order to prevent runoff from directly entering the river system.

- Although not encountered during site exploration, clay soils are known to exist at very shallow depths in the area. If present at subgrade elevation, clay soils will need to be separated from overlying structural improvements. This typically requires over-excavation and removal of clay soils, with the resulting over-excavation backfilled with structural fill.

Quality Control

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

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

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

Standard Limitations Clause

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

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

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

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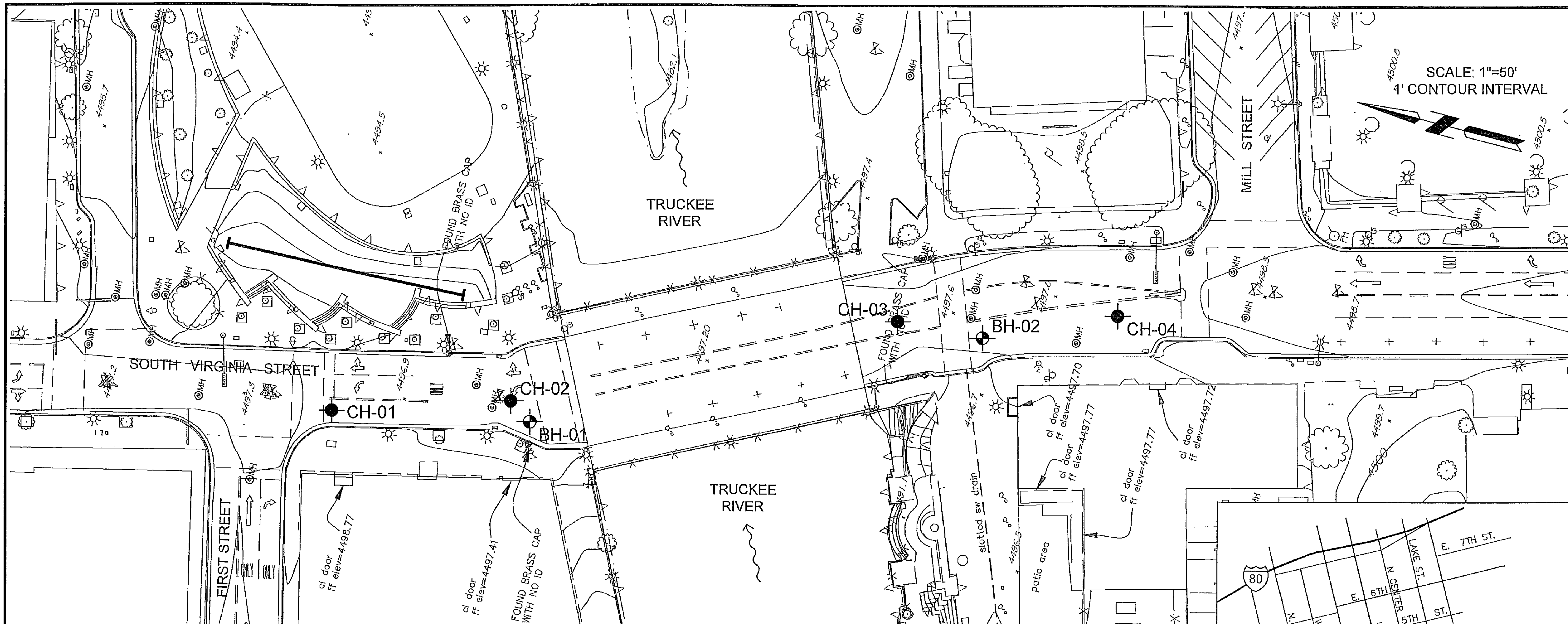
PLATES



LEGEND

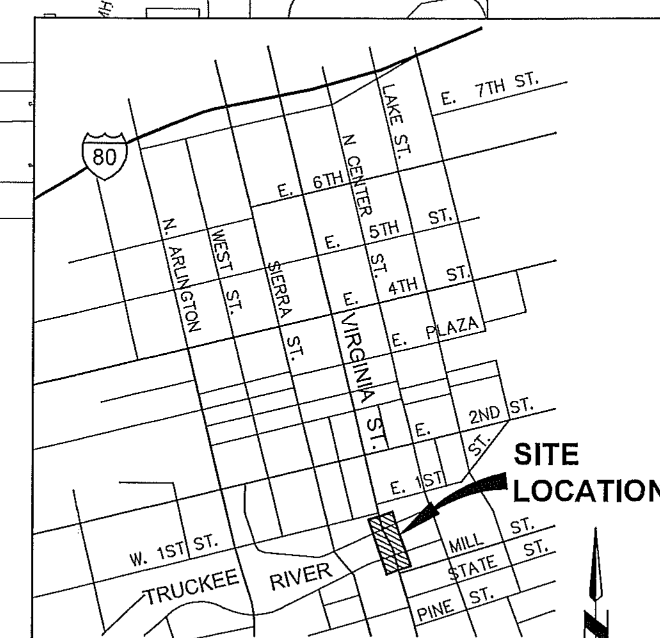
CH-01 — APPROXIMATE CORE HOLE LOCATION

B-01 — APPROXIMATE BORE HOLE LOCATION

ReMi SURVEY LOCATION

NOTES

1. BASE MAP PROVIDED BY JACOBS CIVIL INC.



SITE LOCATION MAP

N.T.S.

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1345 Capital Boulevard, Suite A
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JACOBS CIVIL INC.

PLOT PLAN

VIRIGINA STREET BRIDGE

RENO, NEVADA

Project No.
0500-03-1

Plate 1

BORING LOG

BORING NO.: B-01 | DATE: 3/16/2011

TYPE OF RIG: Shop-Built Sonic | DEPTH TO GROUND WATER (ft): 15

LOGGED BY: SMM | GROUND ELEVATION (ft): 4,483 ±

SAMPLE NO.	SAMPLE TYPE	BLOWS/12 inches	MOISTURE (%)	PLASTICITY INDEX	DEPTH (ft)	USCS SYMBOL	LITHOLOGY	DESCRIPTION
					0.0 - 1.2			0.0' - 1.2': **Asphalt Concrete Pavement**
					1.2 - 2.0	GM		1.2' - 2.0': **Silty Gravel with Sand (Aggregate Base)** Brown, tan, slightly moist dense to very dense with estimated 15% non plastic fines, 40% fine to coarse gravel and 45% angular to subangular fine to medium coarse gravel. Drill ODEX method, 1.2 - 50 feet.
A	SPT	19	9.5	NP	2.0 - 10.0	GP-GM		2.0' - 10.0': **Poorly Graded Gravel with Silt and Sand** Brown, tan, slightly moist, medium dense to very dense, with 11% non-plastic fines, 42% fine to coarse sand, and 47% fine to coarse angular to subrounded gravel. Cobbles and possible boulders indicated by drill character.
B	SPT	50 (5)			10.0 - 15.0	GM		10.0' - 15.0': **Silty Gravel with Sand** Brown, tan, moist, very dense, with an estimated 20% non-plastic fines, 25% fine to coarse sand, and 55% fine to coarse angular to subrounded gravel. Cobbles and possible boulders indicated by drill character.
C	SPT	50 (5.5)			15.0 - 40.0 (▼ 15)			15.0' - 40.0': **Silty Gravel with Sand** Brown, tan, wet, medium dense to very dense, with an estimated 20% non-plastic fines, 25% fine to coarse sand, and 55% fine to coarse angular to subrounded gravel. Cobbles and possible boulders indicated by drill character.
D	SPT	50 (5.5)			20			

North end of Virginia Street Bridge in southbound lane.

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Jacobs Civil Inc.

Virginia Street Bridge Replacement

Reno, Nevada

PROJECT NO.: 0500-03-1

PLATE: 2

BORING LOG

BORING NO.: B-01 | DATE: 3/16/2011

TYPE OF RIG: Shop-Built Sonic | DEPTH TO GROUND WATER (ft): 15

LOGGED BY: SMM | GROUND ELEVATION (ft): 4,483 ±

SAMPLE NO.	SAMPLE TYPE	BLOWS/12 inches	MOISTURE (%)	PLASTICITY INDEX	DEPTH (ft)	USCS SYMBOL	LITHOLOGY	DESCRIPTION
					24			
E	SPT	31			26			
					28	GM		
F	SPT	50 (5)			30			
					32			
					34			
					36			
					38			
G	SPT	74	14.1	6	40			40.0' - 81.5': **Silty, Clayey Sand with Gravel** Brown, gray, wet, very dense, with 26% low plasticity fines, 56% fine to coarse sand, and 18% fine to coarse subangular to subrounded gravel. Cobbles and possible boulders indicated by drill character.
					42			

North end of Virginia Street Bridge in southbound lane.

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PROJECT NO.: 0500-03-1

PLATE: 2

BORING_LOG 0500031.GPJ BLKEAGLE.GDT 4/8/2011

BORING LOG

BORING NO.: B-01

DATE: 3/16/2011

TYPE OF RIG: Shop-Built Sonic

DEPTH TO GROUND WATER (ft): 15

LOGGED BY: SMM

GROUND ELEVATION (ft): 4,483 ±

SAMPLE NO.	SAMPLE TYPE	BLOWS/12 inches	MOISTURE (%)	PLASTICITY INDEX	DEPTH (ft)	USCS SYMBOL	LITHOLOGY	DESCRIPTION
					46			
					48			
H	SPT	68			50			Drill reverse-circulation air with sonic casing advance, 50 - 80 feet.
					52			
					54			
					56			
					58			
I	SPT	48			60	SC-SM		
					62			
					64			

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PLATE: 2

BORING LOG

BORING NO.: B-01

DATE: 3/16/2011

TYPE OF RIG: Shop-Built Sonic

DEPTH TO GROUND WATER (ft): 15

LOGGED BY: SMM

GROUND ELEVATION (ft): 4,483 ±

SAMPLE NO.	SAMPLE TYPE	BLOWS/12 inches	MOISTURE (%)	PLASTICITY INDEX	DEPTH (ft)	USCS SYMBOL	LITHOLOGY	DESCRIPTION
					68			
					70			
					72			
					74			
					76			
					78			
J	SPT	47			80			
					82			
					84			
					86			

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PLATE: 2

BORING LOG

BORING NO.:	B-02	DATE:	3/17/2011
TYPE OF RIG:	Shop-Built Sonic	DEPTH TO GROUND WATER (ft):	15
LOGGED BY:	SMM	GROUND ELEVATION (ft):	4,483 ±

SAMPLE NO.	SAMPLE TYPE	BLOWS/12 inches	MOISTURE (%)	PLASTICITY INDEX	DEPTH (ft)	USCS SYMBOL	LITHOLOGY	DESCRIPTION
					0.0–1.2			0.0' - 1.2': **Asphalt Concrete Pavement**
					1.2–2.0	GM		1.2' - 2.0': **Silty Gravel with Sand (Aggregate Base)** Brown, tan, slightly moist, dense to very dense with estimated 15% non plastic fines, 40% fine to coarse gravel and 45% angular to subangular fine to medium coarse gravel.
					2.0–11.0	GW-GM		2.0' - 11.0': **Well Graded Gravel with Silt and Sand** Brown, tan, slightly moist, dense to very dense, with 9% non-plastic fines, 34% fine to coarse sand, and 57% fine to coarse angular to subrounded gravel. Cobbles and possible boulders indicated by drill character.
A	SPT	50			5.0–6.5			
								Borehole drilled reverse-circulation air with sonic casing advance from 2 - 100 feet.
B	SPT	69	4.7	NP	10.0–11.5			
					11.0–15.0	GM		11.0' - 15.0': **Silty Gravel with Sand** Brown, tan, moist, dense to very dense, with an estimated 20% non-plastic fines, 25% fine to coarse sand, and 55% fine to coarse angular to subrounded gravel. Cobbles and possible boulders indicated by drill character.
								Free petroleum product on water surface at 15 feet. Strong hydrocarbon odor.
					15 ▼			
C	SPT	48			15.0–16.5			
					15.0–24.0	GM		15.0' - 24.0': **Silty Gravel with Sand** Brown, tan, wet, dense to very dense, with an estimated 20% non-plastic fines, 25% fine to coarse sand, and 55% fine to coarse angular to subrounded gravel. Cobbles and possible boulders indicated by drill character.
	SPT	44			20.0–21.5			No sample recovery at 20 feet.

South end of Virginia Street Bridge in southbound lane.

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PROJECT NO.: 0500-03-1

PLATE: 2

BORING LOG

BORING NO.: B-02 DATE: 3/17/2011

TYPE OF RIG: Shop-Built Sonic DEPTH TO GROUND WATER (ft): 15

LOGGED BY: SMM GROUND ELEVATION (ft): 4,483 ±

SAMPLE NO.	SAMPLE TYPE	BLOWS/12 inches	MOISTURE (%)	PLASTICITY INDEX	DEPTH (ft)	USCS SYMBOL	LITHOLOGY	DESCRIPTION
					24			24.0' - 38.0': **Poorly Graded Gravel with Clay and Sand** Brown, gray, wet, very dense, with an estimated 10% medium plasticity fines, 30% fine to coarse sand, and 60% fine to coarse subangular to subrounded gravel. Cobbles and possible boulders indicated by drill character.
D	SPT	71			26			
					28			
E	SPT	50 (11.5)			30	GP-GC		
					32			
					34			
					36			
					38			38.0' - 58.0': **Well Graded Gravel with Silty Clay and Sand** Greenish gray, gray, wet, very dense, with 12% medium plasticity fines, 36% fine to coarse sand, and 52% fine to coarse subangular to subrounded gravel. Cobbles and possible boulders indicated by drill character.
					40			
F	SPT	96	13.9	5				
					42			

South end of Virginia Street Bridge in southbound lane.

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Reno, Nevada

PROJECT NO.: 0500-03-1

PLATE: 2

BORING LOG

BORING NO.: B-02 | DATE: 3/17/2011

TYPE OF RIG: Shop-Built Sonic | DEPTH TO GROUND WATER (ft): 15

LOGGED BY: SMM | GROUND ELEVATION (ft): 4,483 ±

SAMPLE NO.	SAMPLE TYPE	BLOWS/12 inches	MOISTURE (%)	PLASTICITY INDEX	DEPTH (ft)	USCS SYMBOL	LITHOLOGY	DESCRIPTION
					46			
					48	GW-GC		
G	SPT	50 (5)			50			
					52			
					54			
					56			
					58			58.0' - 98.0': **Silty Sand with Gravel** Greenish gray, gray, wet, very dense, with 12% non-plastic fines, 44% fine to coarse sand, and 44% fine to coarse subangular to subrounded gravel. Cobbles and possible boulders indicated by drill character.
					60			
H	SPT	77	16.1	NP				
					62			
					64			

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PROJECT NO.: 0500-03-1

PLATE: 2

BORING LOG

BORING NO.: B-02
DATE: 3/17/2011
TYPE OF RIG: Shop-Built Sonic
DEPTH TO GROUND WATER (ft): 15
LOGGED BY: SMM
GROUND ELEVATION (ft): 4,483 ±

SAMPLE NO.	SAMPLE TYPE	BLOWS/12 inches	MOISTURE (%)	PLASTICITY INDEX	DEPTH (ft)	USCS SYMBOL	LITHOLOGY	DESCRIPTION
					68			
					70			
I	SPT	50 (11.5)						
					72			
					74			
					76			
					78	SM		
					80			
J	SPT	58						
					82			
					84			
					86			

South end of Virginia Street Bridge in southbound lane.

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Reno, Nevada

PROJECT NO.: 0500-03-1

PLATE: 2

BORING LOG

BORING NO.: B-02 | DATE: 3/17/2011

TYPE OF RIG: Shop-Built Sonic | DEPTH TO GROUND WATER (ft): 15

LOGGED BY: SMM | GROUND ELEVATION (ft): 4,483 ±

SAMPLE NO.	SAMPLE TYPE	BLOWS/12 inches	MOISTURE (%)	PLASTICITY INDEX	DEPTH (ft)	USCS SYMBOL	LITHOLOGY	DESCRIPTION
					90			
					92			
					94			
					96			
					98			98.0' - 101.5': **Clayey Sand with Gravel** Brown, gray, wet, medium dense, with an estimated 15% medium plasticity fines, 50% fine to coarse sand, and 45% fine to coarse subangular to subrounded gravel. Cobbles and possible boulders indicated by drill character.
					100	SC		
K	SPT	22						
					102			
					104			
					106			
					108			

South end of Virginia Street Bridge in southbound lane.

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Reno, Nevada

PROJECT NO.: 0500-03-1

PLATE: 2

PAVEMENT CORE LOG CH-01

Date Excavated: 3/16/2011 Logged by: JW

Equipment: 2-inch Hand Auger Surface Elevation (ft) 4,483 ±

Depth to Ground Water: NE
Comments: Corner of First Street and Virginia Street in middle of southbound lane, north of bridge.

SAMPLE NUMBER	SAMPLE	BLOWS/FT	MOISTURE (%)	PI	DEPTH (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION
							0.0' - 0.9': **Asphalt Concrete Pavement**
A					1	GM	0.9' - 2.0': **Silty Gravel with Sand (Aggregate Base)** Brown, slightly moist, dense to very dense with 12% non-plastic fines, 37% fine to coarse gravel and 51% angular to subangular fine to medium coarse gravel.
B			7.5	NP			
C			5.6	NP	2	GP-GM	2.0' - 3.0': **Poorly Graded Gravel with Silt and Sand** Brown to dark brown, slightly moist, dense with 8% non-plastic fines, 31% fine to coarse sand and 61% subangular to rounded fine to coarse gravel with several cobbles.
					3		
					4		

PAVEMENT CORE LOG CH-02

Date Excavated: 3/17/2011 Logged by: JW

Equipment: 2-inch Hand Auger Surface Elevation (ft) 4,483 ±

Depth to Ground Water: NE
Comments: North side of bridge, approximately 50 ft north of CH-01, closer to bridge.

SAMPLE NUMBER	SAMPLE	BLOWS/FT	MOISTURE (%)	PI	DEPTH (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION
							0.0' - 0.9': **Asphalt Concrete Pavement**
A					1	GM	0.9' - 2.0': **Silty Gravel with Sand (Aggragate Base)** Brown, slightly moist, dense to very dense with estimated 10-15% non-plastic fines, 40-45% fine to coarse gravel and 40-45% angular to subangular fine to medium coarse gravel.
B			14.2	NP	2	SP-SM	2.0' - 3.0': **Poorly Graded Sand with Silt and Gravel** Brown to dark brown, slightly moist, dense with 7% non-plastic fines, 48% fine to coarse sand and 45% subangular to rounded fine to coarse gravel with several cobbles up to 6".
					3		
					4		

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Reno, Nevada 0500-03-1 Plate 2

PAVEMENT CORE LOG CH-03

Date Excavated: 3/17/2011 Logged by: JW

Equipment: 2-inch Hand Auger Surface Elevation (ft) 4,483 ±

Depth to Ground Water: NE
Comments: South side of bridge, approximately 20 ft north of sidewalk in median between lanes.

SAMPLE NUMBER	SAMPLE	BLOWS/FT	MOISTURE (%)	PI	DEPTH (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION
					0–1		0.0' - 1.0': **Asphalt Concrete Pavement**
A	Hand				1–2	GM	1.0' - 2.0': **Silty Gravel with Sand (Aggragate Base)** Brown, slightly moist, dense to very dense with estimated 10-15% non-plastic fines, 40-45% fine to coarse gravel and 40-45% angular to subangular fine to medium coarse gravel.
B	Hand				2–3	GP-GC	2.0' - 3.0': **Poorly Graded Gravel with Clay and Sand** Brown to dark brown, slightly moist, dense with an estimated 5-10% non-plastic to low plasticity fines, 50-55% fine to coarse sand and 35-40% subangular to rounded fine to coarse gravel with several cobbles up to 6".

PAVEMENT CORE LOG CH-04

Date Excavated: 3/16/2011 Logged by: JW

Equipment: 2-inch Hand Auger Surface Elevation (ft) 4,483 ±

Depth to Ground Water: NE
Comments: Near intersection of Mill Street and Virginia Street in southbound lane, south of bridge.

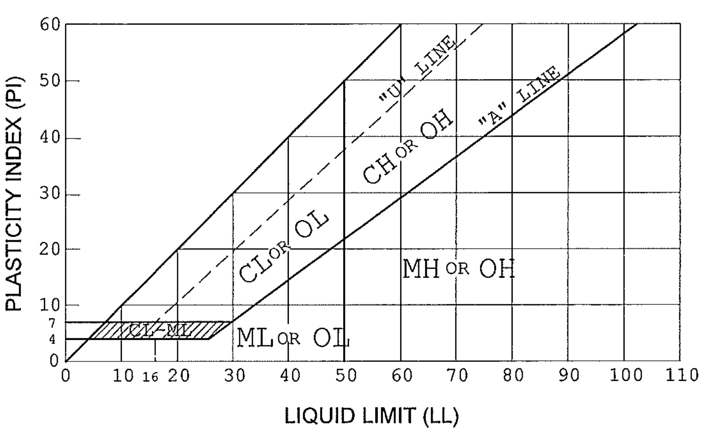
SAMPLE NUMBER	SAMPLE	BLOWS/FT	MOISTURE (%)	PI	DEPTH (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION
					0–0.8		0.0' - 0.8': **Asphalt Concrete Pavement**
A	Hand				0.8–2	GM	0.8' - 2.0': **Silty Gravel with Sand (Aggragate Base)** Brown, slightly moist, dense to very dense with estimated 10-15% non-plastic fines, 40-45% fine to coarse gravel and 40-45% angular to subangular fine to medium coarse gravel.
B	Hand						
C	Hand		6.5	8	2–3	GP-GC	2.0' - 3.0': **Poorly Graded Gravel with Clay and Sand** Brown to dark brown, slightly moist, dense with 11% low plasticity fines, 31% fine to coarse sand and 58% subangular to rounded fine to coarse gravel with several cobbles up to 6".

SOIL CLASSIFICATION CHART

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

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

PLASTICITY CHART



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

EXPLORATION SAMPLE TERMINOLOGY

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

GRAIN SIZE TERMINOLOGY

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

RELATIVE DENSITY OF GRANULAR SOILS

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

CONSISTENCY OF COHESIVE SOILS

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

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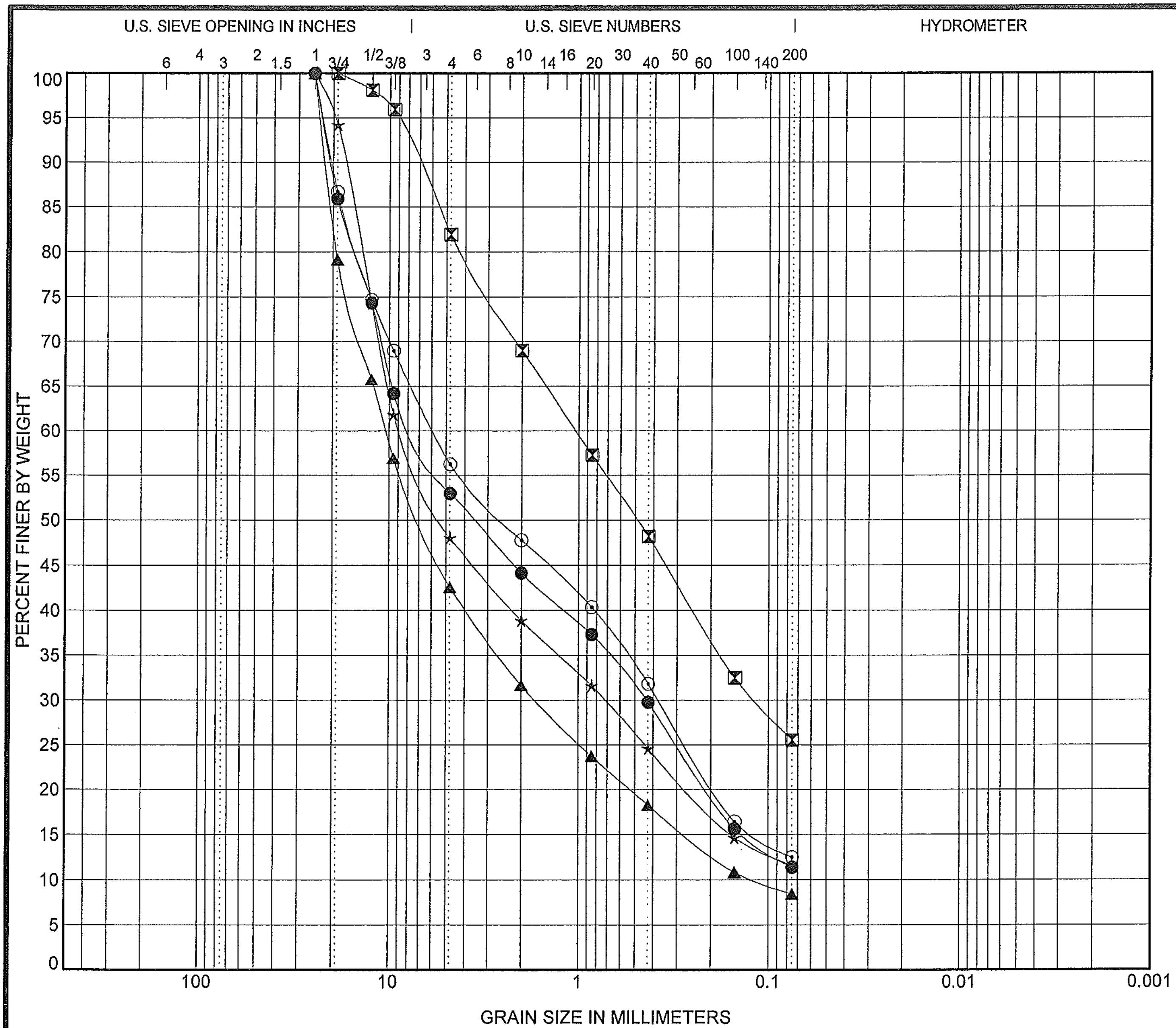
USCS Soil Classification Chart

Project: Virginia Street Bridge Replacement
Location: Reno, Nevada
Project Number: 0500-03-1 Plate: 3



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Specimen Identification		USCS Classification	LL	PL	PI	Cc	Cu
● B-01	5.0'	POORLY GRADED GRAVEL with SILT and SAND (GP-GM)	NP	NP	NP	0.43	122.21
⊠ B-01	45.0'	SILTY, CLAYEY SAND with GRAVEL (SC-SM)	22	16	6		
▲ B-02	10.0'	WELL-GRADED GRAVEL with SILT and SAND (GW-GM)	NP	NP	NP	2.26	88.34
★ B-02	40.0'	WELL-GRADED GRAVEL with SILTY CLAY and SAND (GW-GC)	24	19	5	1.17	167.05
⊙ B-02	60.0'	SILTY SAND with GRAVEL (SM)	NP	NP	NP	0.50	120.20

Specimen Identification		D100	D60	D30	D10	MC %	%Gravel	%Sand	%Silt	%Clay
● B-01	5.0'	25	7.331	0.434		9.5	47.0	41.6	11.4	
⊠ B-01	45.0'	19	1.036	0.117		14.1	18.1	56.4	25.6	
▲ B-02	10.0'	25	10.452	1.672	0.118	4.7	57.4	34.1	8.4	
★ B-02	40.0'	25	8.669	0.725		13.9	52.0	36.4	11.6	
⊙ B-02	60.0'	25	5.824	0.376		16.1	43.7	43.8	12.5	

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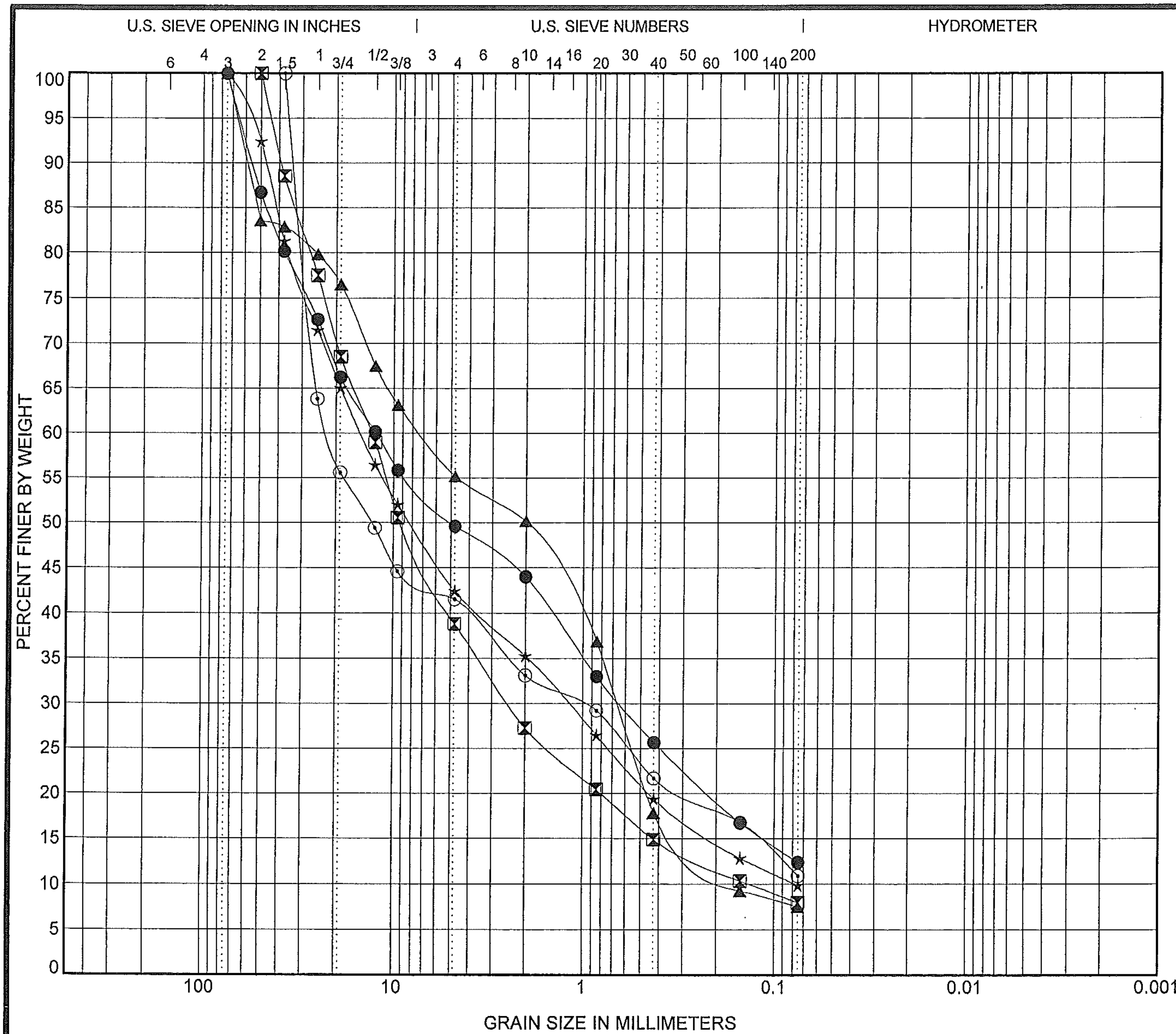
GRAIN SIZE DISTRIBUTION

Project: Virginia Street Bridge Replacement
Location: Reno, Nevada
Project Number: 0500-03-1 Plate: 4a



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Specimen Identification		USCS Classification	LL	PL	PI	Cc	Cu
● CH-01	1.0'	SILTY GRAVEL with SAND (GM)	NP	NP	NP	0.64	240.37
⊠ CH-01	2.5'	POORLY GRADED GRAVEL with SILT and SAND (GP-GM)	NP	NP	NP	3.38	96.26
▲ CH-02	1.0'	POORLY GRADED SAND with SILT and GRAVEL (SP-SM)	NP	NP	NP	0.37	43.96
★ CH-03/CH-04	1.0'	WELL-GRADED GRAVEL with SILT and SAND (GW-GM)	23	20	3	1.24	192.06
⊙ CH-04	2.0'	POORLY GRADED GRAVEL with CLAY and SAND (GP-GC)	27	19	8	0.68	325.47

Specimen Identification		D100	D60	D30	D10	MC %	%Gravel	%Sand	%Silt	%Clay
● CH-01	1.0'	75	12.372	0.639		7.5	50.4	37.2	12.4	
⊠ CH-01	2.5'	50	13.077	2.452	0.136	5.6	61.2	30.8	8.0	
▲ CH-02	1.0'	75	7.224	0.662	0.164	14.2	44.8	47.7	7.5	
★ CH-03/CH-04	1.0'	75	14.853	1.195	0.077	5.1	57.6	32.6	9.9	
⊙ CH-04	2.0'	37.5	21.999	1.007		6.5	58.5	30.7	10.9	

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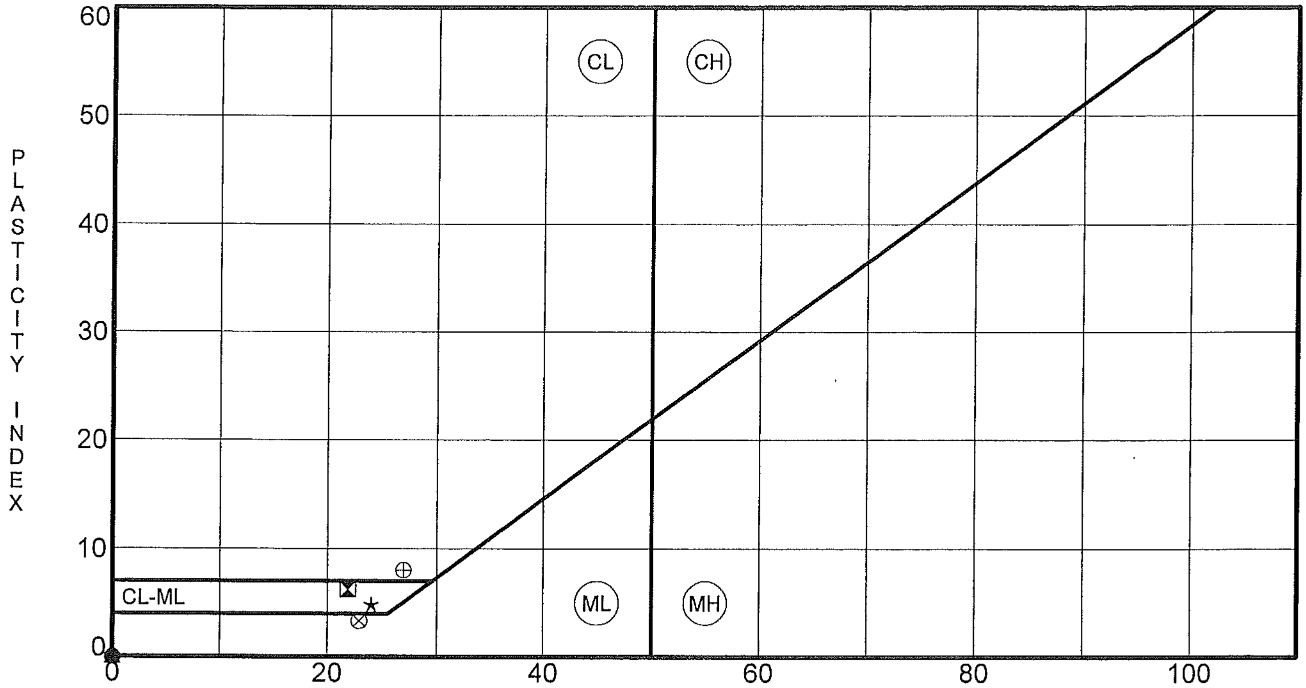
GRAIN SIZE DISTRIBUTION

Project: Virginia Street Bridge Replacement
Location: Reno, Nevada
Project Number: 0500-03-1 Plate: 4b

US_GRAIN_SIZE2 0500031.GPJ US_LAB.GDT 4/4/2011



Specimen Depth in Feet.

	Specimen Identification			LL	PL	PI	Fines	USCS Classification
●	B-01	A	5.0'	NP	NP	NP	11	POORLY GRADED GRAVEL with SILT and SAND (GP-GM)
⌛	B-01		45.0'	22	16	6	26	SILTY, CLAYEY SAND with GRAVEL (SC-SM)
▲	B-02	B	10.0'	NP	NP	NP	8	WELL-GRADED GRAVEL with SILT and SAND (GW-GM)
★	B-02	F	40.0'	24	19	5	12	WELL-GRADED GRAVEL with SILTY CLAY and SAND (GW-GC)
⊙	B-02	H	60.0'	NP	NP	NP	12	SILTY SAND with GRAVEL (SM)
✿	CH-01	A	1.0'	NP	NP	NP	12	SILTY GRAVEL with SAND (GM)
○	CH-01		2.5'	NP	NP	NP	8	POORLY GRADED GRAVEL with SILT and SAND (GP-GM)
△	CH-02	A	1.0'	NP	NP	NP	8	POORLY GRADED SAND with SILT and GRAVEL (SP-SM)
⊗	CH-03/CH-04		1.0'	23	20	3	10	WELL-GRADED GRAVEL with SILT and SAND (GW-GM)
⊕	CH-04	C	2.0'	27	19	8	11	POORLY GRADED GRAVEL with CLAY and SAND (GP-GC)

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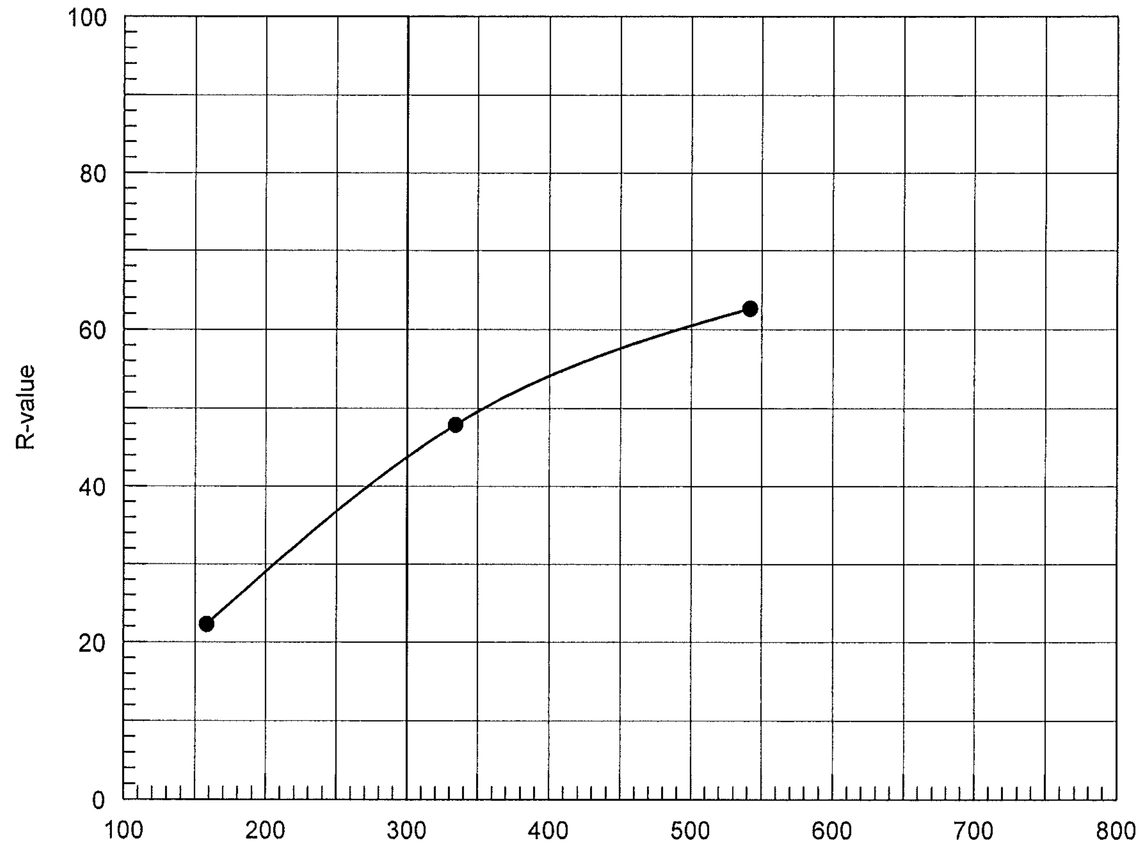
ATTERBERG LIMITS RESULTS

Project: Virginia Street Bridge Replacement
Location: Reno, Nevada
Project Number: 0500-03-1 Plate: 4c

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	250	132.1	7.5	0.06	71	2.68	334	43	48
2	200	130.0	8.5	0.00	110	2.61	158	21	22
3	300	134.7	6.5	0.09	49	2.64	542	59	63

Test Results	Material Description
R-value at 300 psi exudation pressure = 44	Silty Gravel with Sand

Project No.: 0500-03-1
Project: Virginia Street Bridge Replacement
Source of Sample: CH-01 **Depth:** 1.0'
Sample Number: Bulk
Date: 4/4/2011

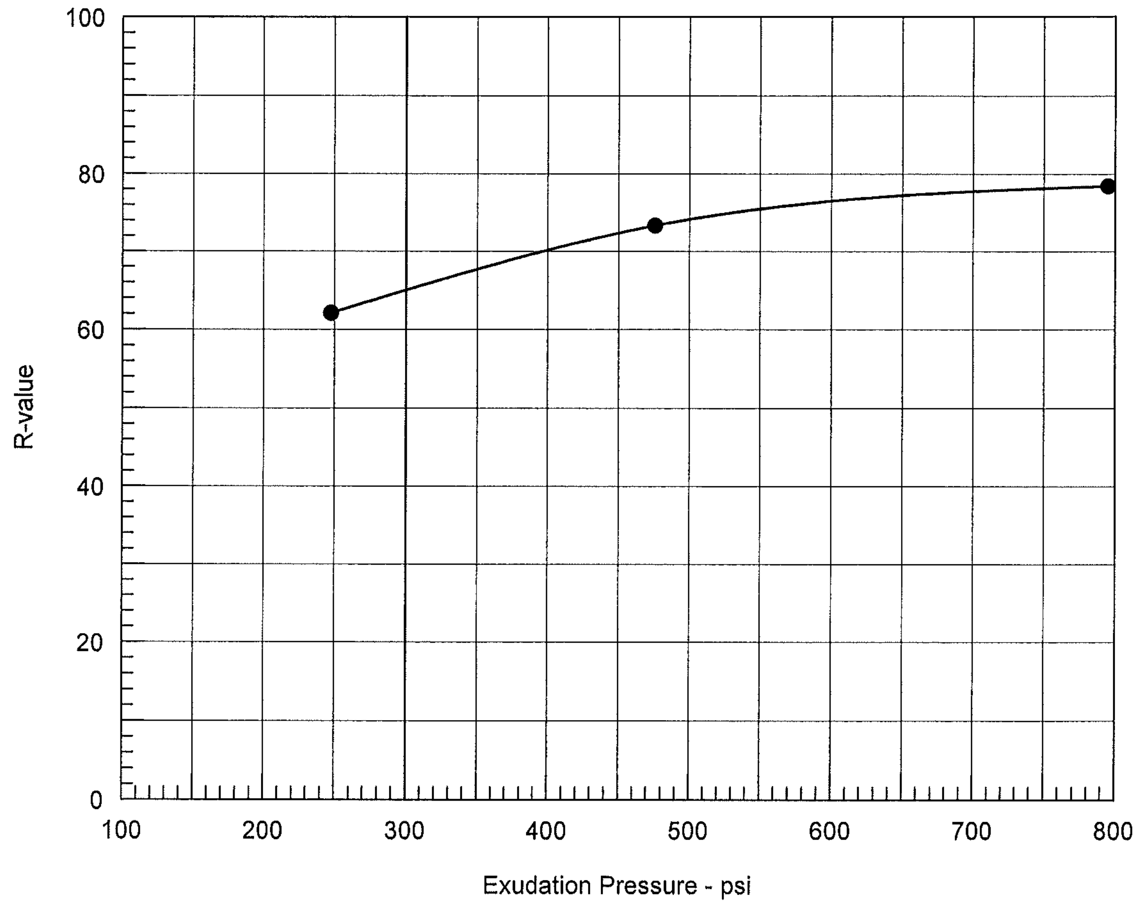
Tested by: G. Bomberger
Checked by: G. Bomberger
Remarks:
Laboratory Number 2135

R-VALUE TEST REPORT

BLACK EAGLE CONSULTING, INC.

Plate 5a

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	102.5	14.2	0.00	22	2.42	476	75	73
2	350	105.6	13.0	0.00	19	2.46	795	78	78
3	250	100.1	16.3	0.00	39	2.49	247	62	62

Test Results	Material Description
R-value at 300 psi exudation pressure = 65	Poorly Graded Sand with Silt and Gravel

Project No.: 0500-03-1
Project: Virginia Street Bridge Replacement
Source of Sample: CH-02 **Depth:** 1.0'
Sample Number: Bulk
Date: 4/4/2011

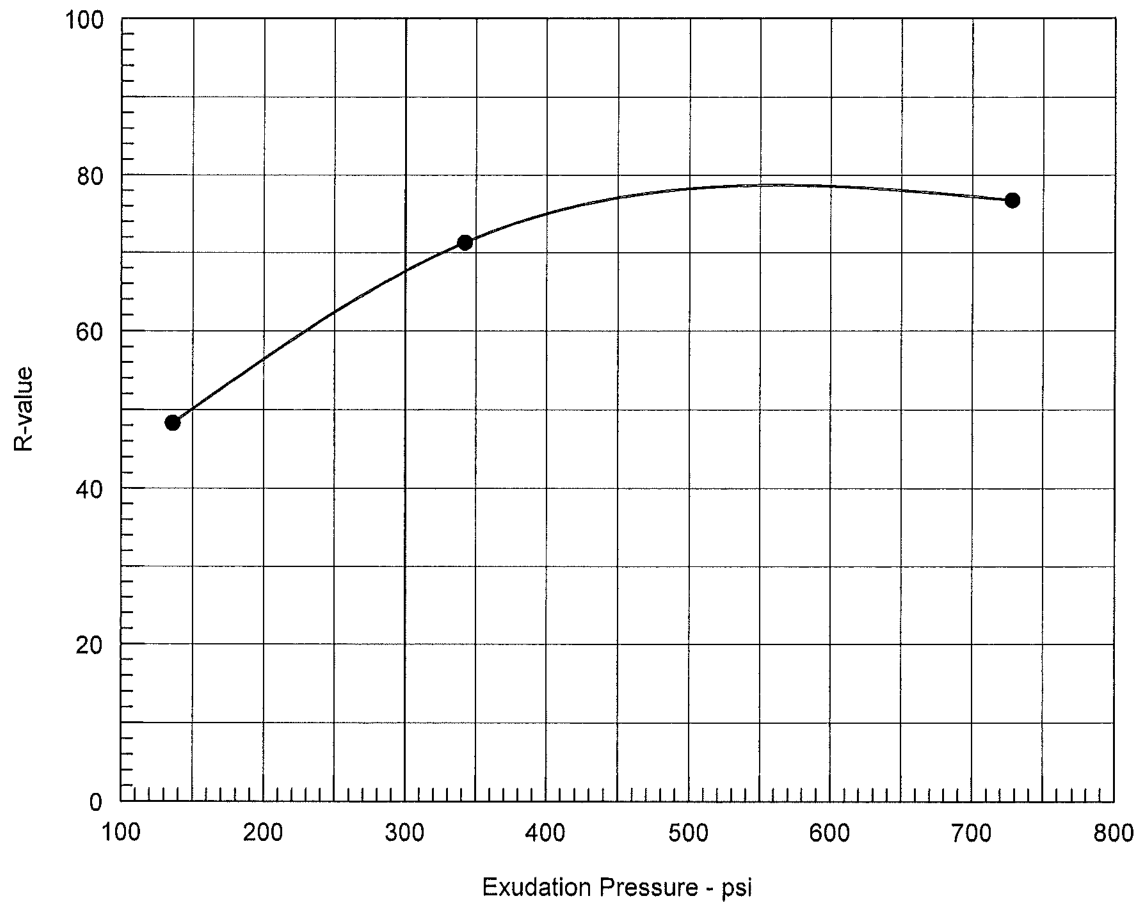
Tested by: G. Bomberger
Checked by: G. Bomberger
Remarks:
Laboratory Number 2135

R-VALUE TEST REPORT

BLACK EAGLE CONSULTING, INC.

Plate 5b

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	136.8	5.1	0.00	25	2.50	728	77	77
2	350	134.0	6.2	0.00	30	2.51	342	71	71
3	300	132.7	7.4	0.00	59	2.53	136	48	48

Test Results	Material Description
R-value at 300 psi exudation pressure = 68	Well - Graded Gravel with Silt and Sand

Project No.: 0500-03-1

Project: Virginia Street Bridge Replacement

Source of Sample: CH-03 / CH-04 **Depth:** 1.0' - 3.0'

Date: 4/4/2011

Tested by: G. Bomberger

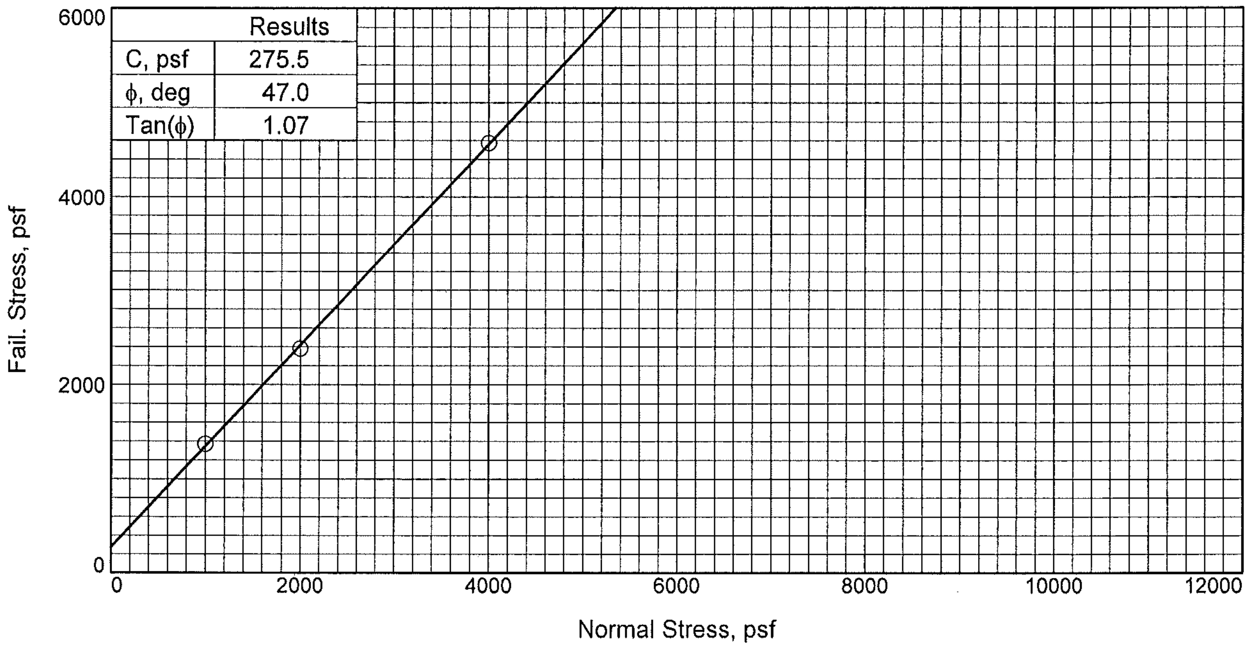
Checked by: G. Bomberger

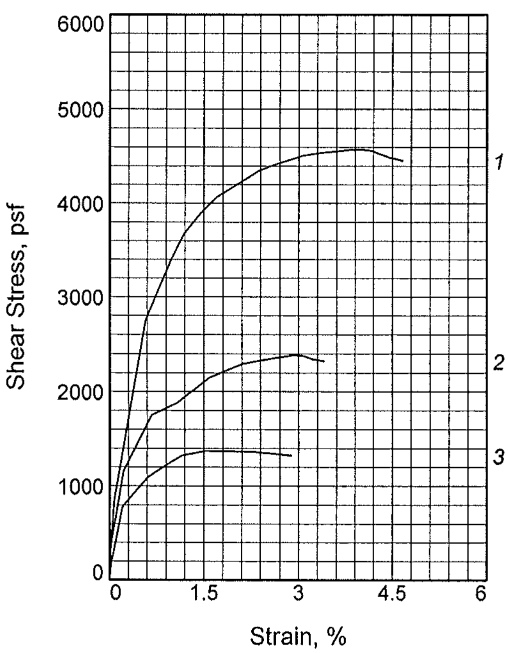
Remarks:
Laboratory Number 2135

R-VALUE TEST REPORT

BLACK EAGLE CONSULTING, INC.

Plate 5c





Sample No.		1	2	3
Initial	Water Content, %	9.9	9.9	9.9
	Dry Density, pcf	123.1	122.8	124.2
	Saturation, %	72.3	71.7	74.9
	Void Ratio	0.3696	0.3726	0.3567
	Diameter, in.	2.420	2.420	2.420
	Height, in.	1.000	1.055	1.030
At Test	Water Content, %	12.5	13.2	12.9
	Dry Density, pcf	125.5	123.8	124.7
	Saturation, %	98.1	98.8	99.0
	Void Ratio	0.3433	0.3617	0.3514
	Diameter, in.	2.420	2.420	2.420
	Height, in.	0.981	1.047	1.026
Normal Stress, psf		4000.0	2000.0	1000.0
Fail. Stress, psf		4574.0	2382.5	1371.2
Strain, %		4.0	2.9	1.5
Ult. Stress, psf				
Strain, %				
Strain rate, in./min.		0.020	0.020	0.020

Sample Type: Remolded
Description: Poorly Graded Gravel with Silt and Sand
LL= No Value **PI=** Non Plastic
Assumed Specific Gravity= 2.7
Remarks: Laboratory number 2135

Plate 6a

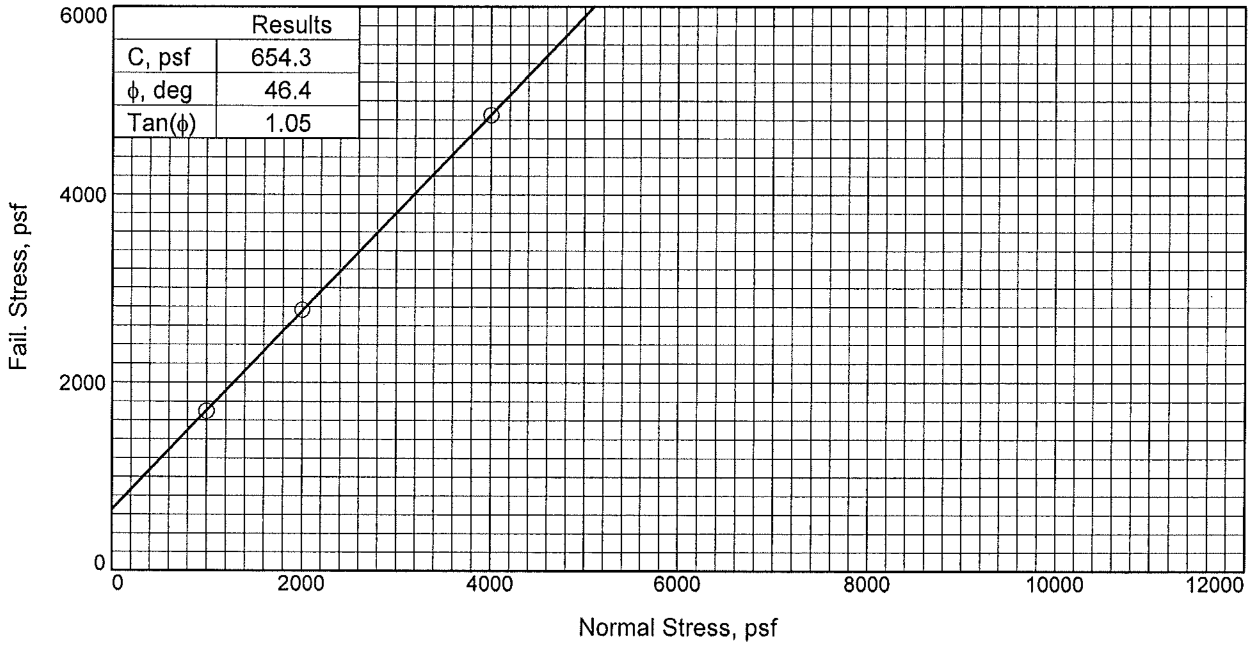
Client: Jacobs Civil, Inc

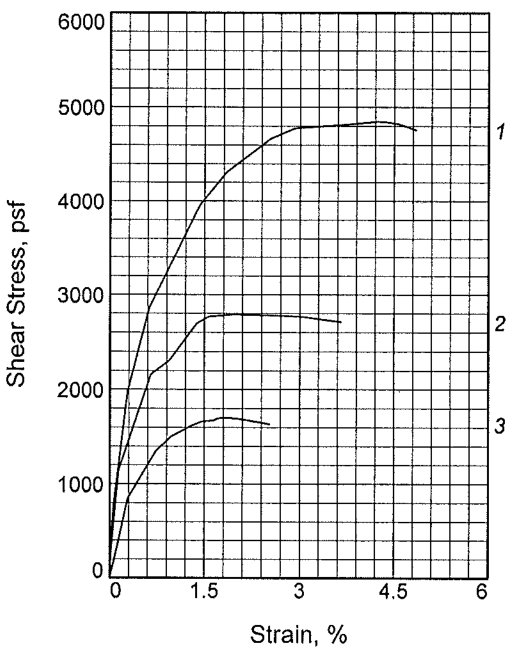
Project: Virginia Street Bridge Replacement

Source of Sample: CH-01 **Depth:** 2.5'
Sample Number: B & C
Proj. No.: 0500-03-1 **Date Sampled:** 03-16-11

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

Tested By: G. Bomberger





Sample No.		1	2	3
Initial	Water Content, %	10.5	10.5	10.5
	Dry Density, pcf	109.8	109.8	109.9
	Saturation, %	53.0	52.9	53.1
	Void Ratio	0.5347	0.5356	0.5336
	Diameter, in.	2.420	2.420	2.420
	Height, in.	1.180	1.040	1.040
At Test	Water Content, %	14.5	15.0	16.9
	Dry Density, pcf	120.2	119.5	115.3
	Saturation, %	97.1	98.8	99.1
	Void Ratio	0.4023	0.4101	0.4613
	Diameter, in.	2.420	2.420	2.420
	Height, in.	1.078	0.955	0.991
Normal Stress, psf		4000.0	2000.0	1000.0
Fail. Stress, psf		4849.5	2764.4	1696.8
Strain, %		4.2	3.1	1.8
Ult. Stress, psf				
Strain, %				
Strain rate, in./min.		0.020	0.020	0.020

Sample Type: Remolded
Description: Poorly Graded Gravel with Clay and Sand
LL= 27 **PL=** 19 **PI=** 8
Assumed Specific Gravity= 2.7
Remarks: Laboratory Number 2141

Plate 6b

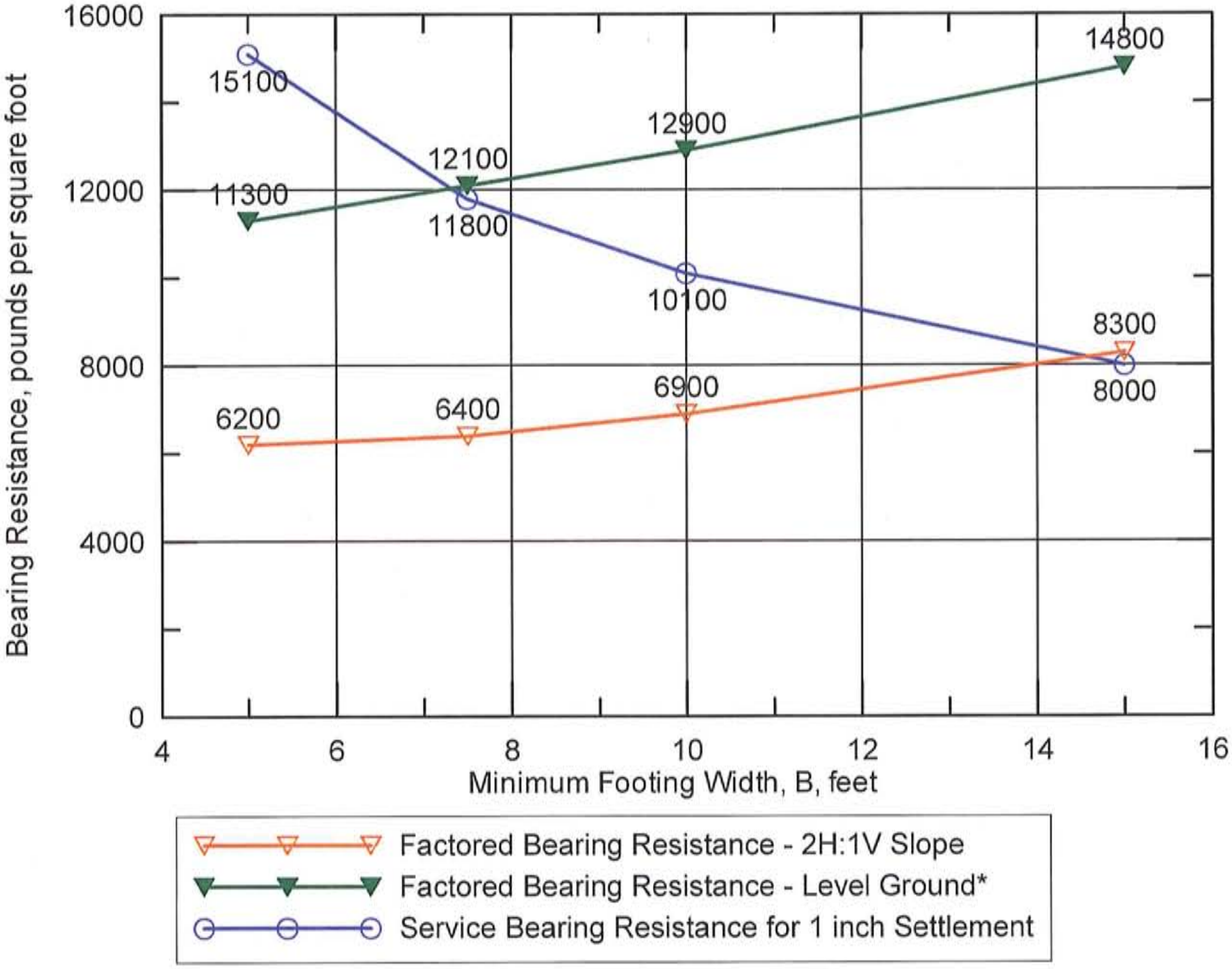
Client: Jacobs Civil, Inc

Project: Virginia Street Bridge Replacement

Source of Sample: CH-04 **Depth:** 2.0'
Sample Number: B
Proj. No.: 0500-03-1 **Date Sampled:** 03-16-11

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

Tested By: G. Bomberger **Checked By:** G. Bomberger



* Abutment edge is at least footing width away from a 2H:1V slope.

Nominal resistances were estimated using the procedures recommended by AASHTO (2007). Reduced bearing capacity factors were used in the nominal resitance calculations assuming abutments will be located on or near sloping ground.

Factored bearing resistances were calculated by applying a reduction factor of 0.45 to the nominal bearing resistance.

Nominal bearing resistance values and factored bearing resistance values shall be compared with factored load values for various strength limit cases recommended by AASHTO (2007).

Service bearing resistance values for 1 inch settlement were calculated using conventional Schmertmann Method of Settlement Analysis.

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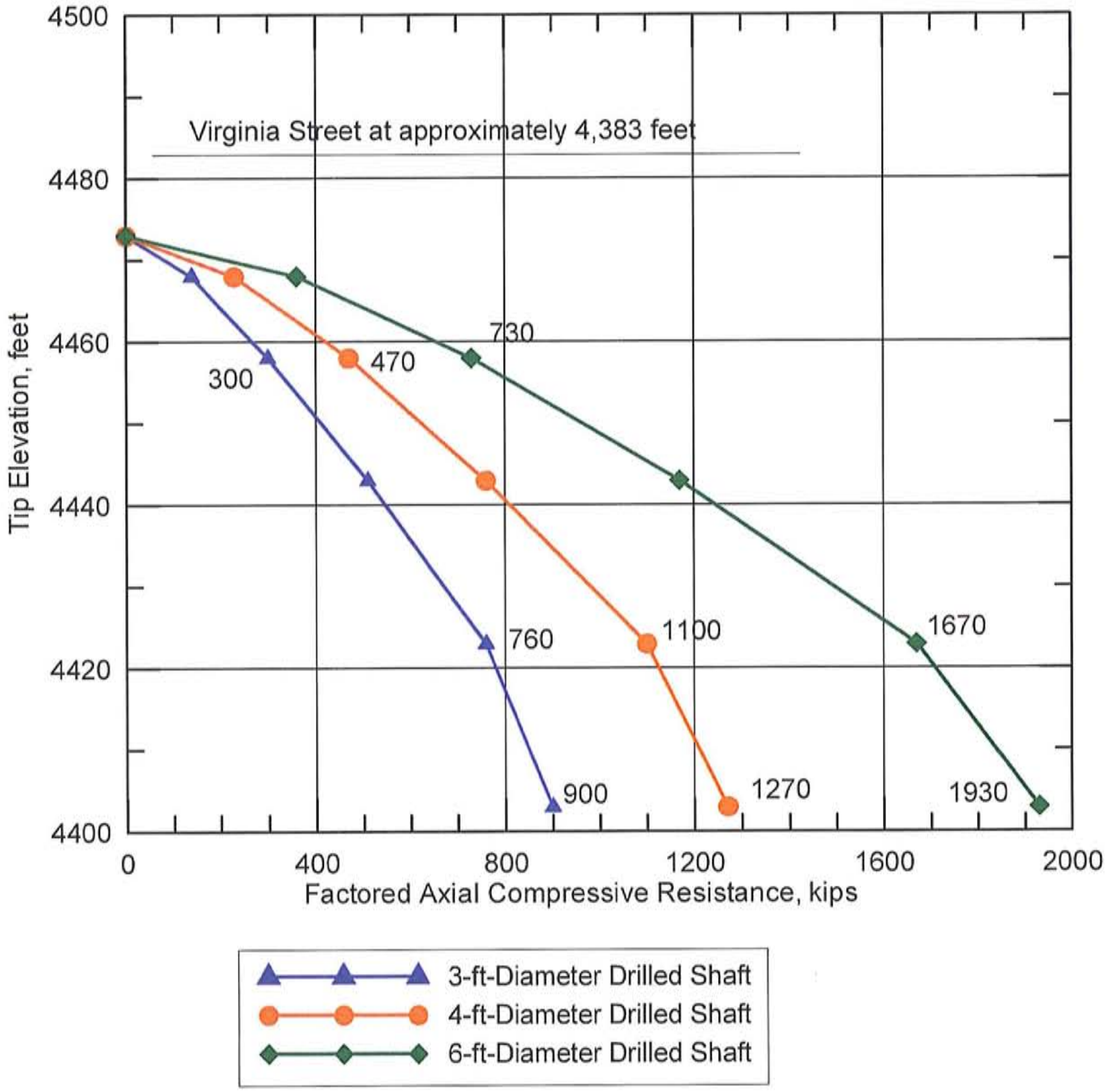
JACOBS CIVIL INC.
SHALLOW FOUNDATION DESIGN PARAMETERS
BEARING RESISTANCE VERSUS FOOTING WIDTH
VIRGINIA STREET BRIDGE ABUTMENTS
RENO, NEVADA

Project No:
0500-03-1

Plate 7



Nominal axial resiatances for drilled shaft foundations were estimated using the method rcommended by O'Neill and Reese (1999).

Factored axial resistances were calculated by applying appropriate resistance factors to the nominal axial resistance values (AASHTO, 2007).

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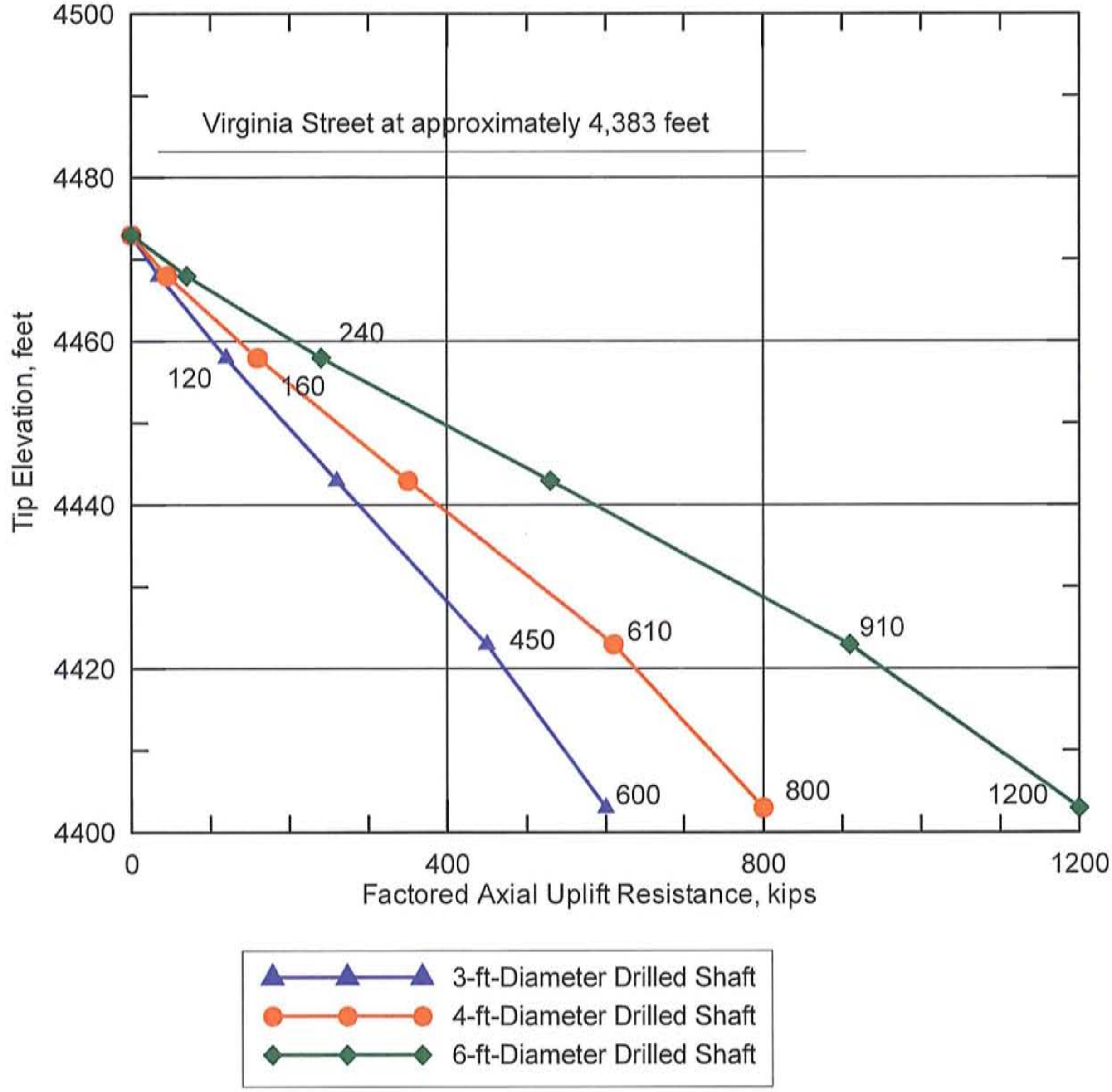
JACOBS CIVIL INC.
DRILLED SHAFT FOUNDATION DESIGN PARAMETERS
FACTORED AXIAL COMPRESSIVE RESISTANCE
VIRGINIA STREET BRIDGE ABUTMENTS
RENO, NEVADA

Project No:
0500-03-1

Plate 8a



Nominal axial resistances for drilled shaft foundations were estimated using the method rcommended by O'Neill and Reese (1999).

Factored axial resistances were calculated by applying appropriate resistance factors to the nominal axial resistance values (AASHTO, 2007).

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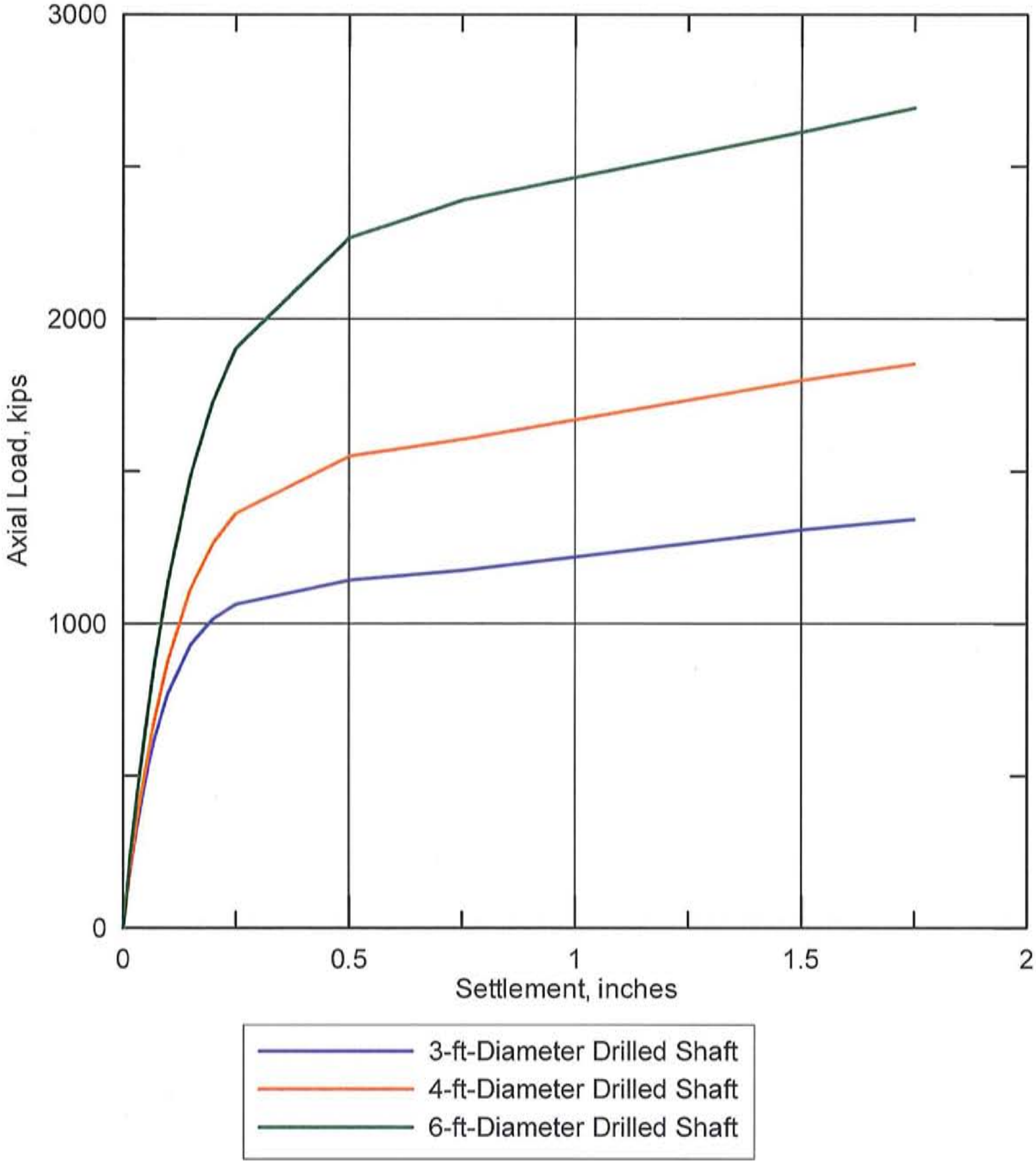
JACOBS CIVIL INC.
DRILLED SHAFT FOUNDATION DESIGN PARAMETERS
FACTORED AXIAL UPLIFT RESISTANCE
VIRGINIA STREET BRIDGE ABUTMENTS
RENO, NEVADA

Project No:
0500-03-1

Plate 8b



Predicted load-settlement response using normalized curves for cohesionless soils (O'Neil and Reese, 1999).

Factored axial loads for the Service Limit State shall be used to determine the anticipated maximum axial settlement.

Black Eagle Consulting, Inc.
Geotechnical and Construction Services
1345 Capital Boulevard Suite A
Reno, Nevada, 89502-7140
Telephone: (775) 359-6600
Fax: (775) 359-7766

JACOBS CIVIL INC.
DRILLED SHAFT FOUNDATION DESIGN PARAMETERS
AXIAL LOAD VERSES SETTLEMENT
VIRGINIA STREET BRIDGE ABUTMENTS
RENO, NEVADA

Project No:
0500-03-1

Plate 8c

APPENDIX A

GEOPHYSICAL SURVEY

APPENDIX A - GEOPHYSICAL SURVEY

MICROTREMOR SHEAR-WAVE ANALYSIS

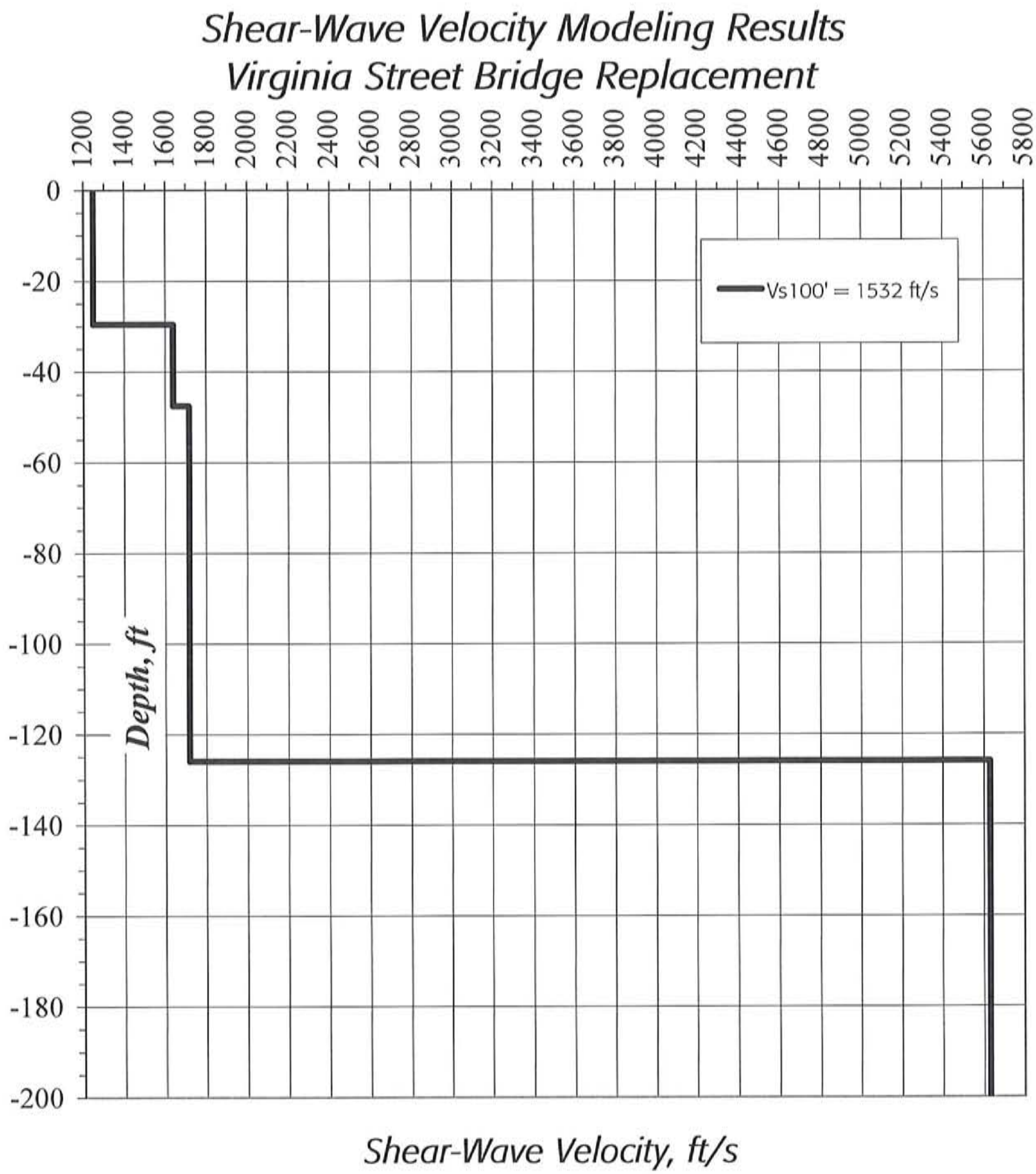
Shear-wave velocities for subsurface strata were collected using a multiple channel digital acquisition data logger and geophone system. A DAQLink II™ 24-bit, 2-channel analog to digital data logger, coupled with 12, 4.5-Hz geophones on 3-meter spacings, was used to record background micro tremor refraction 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 shown on the attached figure. For standard 8-meter 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 for the site is shown on Plate 4 in the main report. The inverse-weighted-average shear-wave velocity from 0 to 100 feet is also calculated, as shown on the plate.

---

[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
Virginia Street Bridge Replacement
1200
1400
1600
1800
2000
2200
2400
2600
2800
3000
3200
3400
3600
3800
4000
4200
4400
4600
4800
5000
5200
5400
5600
5800
0
-20
-40
-60
-80
-100
-120
-140
-160
-180
-200
Depth, ft
Vs100' = 1532 ft/s
Shear-Wave Velocity, ft/s

APPENDIX B

CHEMICAL TEST RESULTS

Laboratory Report
Report ID: 111930

Sierra Environmental Monitoring, Inc.

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

Date: 4/5/2011
Client: BEC-100
Taken by: S. Mulvaney
PO #:

Analysis Report

Laboratory Sample ID	Customer Sample ID	Date Sampled	Time Sampled	Date Received
S201103-1030	B-01ABC	3/16/2011	10:00 AM	3/22/2011

Parameter	Method	Result	Units	Reporting Limit	Analyst	Date Analyzed	Data Flag
Chloride - Ion Chromatography	EPA 300.0	78	mg/Kg	10	Faulstich	3/25/2011	
pH - Saturated Paste	SW-846 9045A	8.37	pH Units		Seher	3/24/2011	
pH - Temperature	SW-846 9045A	20.9	°C		Seher	3/24/2011	
Redox Potential	SM 2580 B	431	MV		Seher	3/28/2011	
Resistivity	EPA 120.1	2700	ohm cm		Pacheco	3/25/2011	
Sulfate - Ion Chromatography	EPA 300.0	180	mg/Kg	10	Faulstich	3/25/2011	
Sulfide	EPA 376.1	Negative	Pos/Neg	1	Seher	3/28/2011	

Laboratory Sample ID	Customer Sample ID	Date Sampled	Time Sampled	Date Received
S201103-1031	B-02BC	3/17/2011	9:00 AM	3/22/2011

Parameter	Method	Result	Units	Reporting Limit	Analyst	Date Analyzed	Data Flag
Chloride - Ion Chromatography	EPA 300.0	12	mg/Kg	10	Faulstich	3/25/2011	
pH - Saturated Paste	SW-846 9045A	8.79	pH Units		Seher	3/24/2011	
pH - Temperature	SW-846 9045A	20.7	°C		Seher	3/24/2011	
Redox Potential	SM 2580 B	434	MV		Seher	3/28/2011	
Resistivity	EPA 120.1	6000	ohm cm		Pacheco	3/25/2011	
Sulfate - Ion Chromatography	EPA 300.0	13	mg/Kg	10	Faulstich	3/25/2011	
Sulfide	EPA 376.1	Negative	Pos/Neg	1	Seher	3/28/2011	

Data Flag Legend:

John Kobza, Ph.D
Laboratory Director

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

John C. Seher
Special Consultant
Quality Assurance Manager

APPENDIX C

ENVIRONMENTAL TEST RESULTS

Alpha Analytical, Inc.

255 Glendale Ave. • Suite 21 • Sparks, Nevada 89431-5778
(775) 355-1044 • (775) 355-0406 FAX • 1-800-283-1183

ANALYTICAL REPORT

H2O Environmental
3510 Barron Way
Reno, NV 89511

Attn: Greg Scyphers
Phone: (775) 351-2237
Fax: (775) 351-2219
Date Received : 03/21/11

Job: Black Eagle

TCLP Metals by ICPMS
EPA Method SW6020 / SW6020A

	Parameter	Concentration	Reporting Limit	Date Extracted	Date Analyzed
Client ID: **BEVS-WD**					
Lab ID : H2O11032124-01A	Chromium (Cr)	ND	0.10 mg/L	03/23/11	03/23/11
Date Sampled 03/18/11 11:00	Arsenic (As)	ND	0.10 mg/L	03/23/11	03/23/11
	Selenium (Se)	ND	0.10 mg/L	03/23/11	03/23/11
	Silver (Ag)	ND	0.10 mg/L	03/23/11	03/23/11
	Cadmium (Cd)	ND	0.10 mg/L	03/23/11	03/23/11
	Barium (Ba)	1.2	1.0 mg/L	03/23/11	03/23/11
	Mercury (Hg)	ND	0.10 mg/L	03/23/11	03/23/11
	Lead (Pb)	ND	0.10 mg/L	03/23/11	03/23/11
Client ID: **BEVL-WD**					
Lab ID : H2O11032124-02A	Chromium (Cr)	ND	0.10 mg/L	03/23/11	03/23/11
Date Sampled 03/18/11 11:00	Arsenic (As)	ND	0.10 mg/L	03/23/11	03/23/11
	Selenium (Se)	ND	0.10 mg/L	03/23/11	03/23/11
	Silver (Ag)	ND	0.10 mg/L	03/23/11	03/23/11
	Cadmium (Cd)	ND	0.10 mg/L	03/23/11	03/23/11
	Barium (Ba)	ND	1.0 mg/L	03/23/11	03/23/11
	Mercury (Hg)	ND	0.10 mg/L	03/23/11	03/23/11
	Lead (Pb)	ND	0.10 mg/L	03/23/11	03/23/11

ND = Not Detected

Roger Scholl Randy Gardner Walter Hinchman

Roger L. Scholl, Ph.D., Laboratory Director • • Randy Gardner, Laboratory Manager • • Walter Hinchman, Quality Assurance Officer
Sacramento, CA • (916) 366-9089 / Las Vegas, NV • (702) 736-7522 / Carson, CA • (714) 386-2901 / info@alpha-analytical.com
Alpha certifies that the test results meet all requirements of NELAC unless footnoted otherwise.
Alpha Analytical, Inc. currently holds appropriate and available NDEP certifications for the data reported - certification #NV16.

3/28/11
Report Date

Alpha Analytical, Inc.

255 Glendale Ave. • Suite 21 • Sparks, Nevada 89431-5778
(775) 355-1044 • (775) 355-0406 FAX • 1-800-283-1183

ANALYTICAL REPORT

H2O Environmental
3510 Barron Way
Reno, NV 89511
Job: Black Eagle

Attn: Greg Scyphers
Phone: (775) 351-2237
Fax: (775) 351-2219

Alpha Analytical Number: H2O11032124-02A
Client I.D. Number: BEVL-WD

Sampled: 03/18/11 11:00
Received: 03/21/11
Extracted: 03/22/11
Analyzed: 03/22/11

TCLP Regulated VOCs
EPA Method SW1311 / 8260B

	Compound	Concentration	Reporting Limit
1	Vinyl chloride	ND	0.10 mg/L
2	1,1-Dichloroethene	ND	0.10 mg/L
3	2-Butanone (MEK)	ND	0.10 mg/L
4	Chloroform	ND	0.10 mg/L
5	1,2-Dichloroethane	ND	0.10 mg/L
6	Carbon tetrachloride	ND	0.10 mg/L
7	Benzene	ND	0.10 mg/L
8	Trichloroethene	ND	0.10 mg/L
9	Tetrachloroethene	ND	0.10 mg/L
10	Chlorobenzene	ND	0.10 mg/L
11	1,4-Dichlorobenzene	ND	0.10 mg/L

ND = Not Detected

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ANALYTICAL REPORT

H2O Environmental
3510 Barron Way
Reno, NV 89511

Attn: Greg Scyphers
Phone: (775) 351-2237
Fax: (775) 351-2219
Date Received : 03/21/11

Job: Black Eagle

Total Petroleum Hydrocarbons - Extractable (TPH-E) EPA Method SW8015B
Total Petroleum Hydrocarbons - Purgeable (TPH-P) EPA Method SW8015B

		Parameter	Concentration		Reporting Limit	Date Extracted	Date Analyzed
Client ID :	**BEVS-WD**						
Lab ID :	H2O11032124-01A	TPH-E (DRO)	570	L	100 mg/Kg	03/25/11	03/25/11
Date Sampled	03/18/11 11:00	TPH-E (ORO)	610		100 mg/Kg	03/25/11	03/25/11
		TPH-P (GRO)	ND		10 mg/Kg	03/23/11	03/23/11
Client ID :	**BEVL-WD**						
Lab ID :	H2O11032124-02A	TPH-E (DRO)	990		500 mg/L	03/22/11	03/23/11
Date Sampled	03/18/11 11:00	TPH-E (ORO)	830		500 mg/L	03/22/11	03/23/11
		TPH-P (GRO)	ND		0.50 mg/L	03/24/11	03/24/11

Diesel Range Organics (DRO) C13-C22
Gasoline Range Organics (GRO) C4-C13
L = DRO concentration may include contributions from heavier-end hydrocarbons that elute in the DRO range.
Oil Range Organics (ORO) C22-C40+

Sample results were calculated on a wet weight basis.
ND = Not Detected

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RG
3/28/11
Report Date

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ANALYTICAL REPORT

H2O Environmental
3510 Barron Way
Reno, NV 89511
Job: Black Eagle

Attn: Greg Scyphers
Phone: (775) 351-2237
Fax: (775) 351-2219

Alpha Analytical Number: H2O11032124-01A
Client I.D. Number: BEVS-WD

Sampled: 03/18/11 11:00
Received: 03/21/11
Extracted: 03/24/11
Analyzed: 03/24/11

TCLP Volatile Organics by GC/MS
EPA Method 624/SW8260B

	Compound	Concentration	Reporting Limit
1	Chloromethane	ND	20 µg/L
2	Vinyl chloride	ND	10 µg/L
3	Chloroethane	ND	10 µg/L
4	Bromomethane	ND	20 µg/L
5	Trichlorofluoromethane	ND	10 µg/L
6	1,1-Dichloroethene	ND	10 µg/L
7	Dichloromethane	ND	20 µg/L
8	trans-1,2-Dichloroethene	ND	10 µg/L
9	1,1-Dichloroethane	ND	10 µg/L
10	cis-1,2-Dichloroethene	ND	10 µg/L
11	Chloroform	ND	10 µg/L
12	1,2-Dichloroethane	ND	10 µg/L
13	1,1,1-Trichloroethane	ND	10 µg/L
14	Carbon tetrachloride	ND	10 µg/L
15	Benzene	ND	5.0 µg/L
16	1,2-Dichloropropane	ND	10 µg/L
17	Trichloroethene	ND	10 µg/L
18	Bromodichloromethane	ND	10 µg/L
19	cis-1,3-Dichloropropene	ND	10 µg/L
20	trans-1,3-Dichloropropene	ND	10 µg/L
21	1,1,2-Trichloroethane	ND	10 µg/L
22	Toluene	5.2	5.0 µg/L
23	Dibromochloromethane	ND	10 µg/L
24	Tetrachloroethene	ND	10 µg/L
25	Chlorobenzene	ND	10 µg/L
26	Ethylbenzene	ND	5.0 µg/L
27	m,p-Xylene	ND	5.0 µg/L
28	Bromoform	ND	10 µg/L
29	o-Xylene	ND	5.0 µg/L
30	1,1,2,2-Tetrachloroethane	ND	10 µg/L
31	1,3-Dichlorobenzene	ND	10 µg/L
32	1,4-Dichlorobenzene	ND	10 µg/L
33	1,2-Dichlorobenzene	ND	10 µg/L

ND = Not Detected

Roger Scholl   Randy Gardner   Walter Hinchman

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Alpha certifies that the test results meet all requirements of NELAC unless footnoted otherwise.
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3/28/11
Report Date

Alpha Analytical, Inc.

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(775) 355-1044 • (775) 355-0406 FAX • 1-800-283-1183

ANALYTICAL REPORT

H2O Environmental
3510 Barron Way
Reno, NV 89511

Attn: Greg Scyphers
Phone: (775) 351-2237
Fax: (775) 351-2219
Date Received : 03/24/11

Job: Black Eagle

Anions by IC
EPA Method 300.0

Parameter	Concentration	Reporting Limit	Date Extracted	Date Analyzed
Client ID: **BEV-WD** Lab ID : H2O11032521-01A Chloride Date Sampled 03/24/11 12:30	17	5.0 mg/L	03/25/11 11:34	03/25/11 13:42

Roger Scholl Randy Gardner Walter Hinchman

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Alpha Analytical, Inc. currently holds appropriate and available NDEP certifications for the data reported - certification #NV16.

✓
4/1/11
Report Date

Alpha Analytical, Inc.

255 Glendale Ave. • Suite 21 • Sparks, Nevada 89431-5778
(775) 355-1044 • (775) 355-0406 FAX • 1-800-283-1183

ANALYTICAL REPORT

H2O Environmental
3510 Barron Way
Reno, NV 89511

Attn: Greg Scyphers
Phone: (775) 351-2237
Fax: (775) 351-2219
Date Received : 03/24/11

Job: Black Eagle

Total Dissolved Solids (TDS)
SM2540C

	Parameter	Concentration	Reporting Limit	Date Extracted	Date Analyzed
Client ID: **BEV-WD** Lab ID : H2O11032521-01A Date Sampled 03/24/11 12:30	Solids, Total Dissolved (TDS)	210	10 mg/L	03/28/11	03/28/11

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4/1/11
Report Date

APPENDIX D

TRAFFIC COUNTS AND ESAL CALCULATIONS

URBAN
VEHICLE DISTRIBUTION and AVERAGE ESAL's
by ROADWAY FUNCTIONAL CLASSIFICATION

STATE: NEVADA   STATE FIPS CODE: 32   DATA YEAR: 2009   DATE: 25-May-10

FUNCTIONAL CLASSIFICATION	PERCENT OF TRAVEL												TRUCK PERCENT [T%]
	PASSENGER VEHICLES			LIGHT TRUCKS			HEAVY TRUCKS						
				SINGLE-UNIT TRUCKS			SINGLE-TRAILER TRUCKS			MULTI-TRAILER TRUCKS			
	MOTOR-CYCLES	AUTO-MOBILES	LIGHT TRUCKS [2 AXLE, 4 TIRE]	BUSSES	2 AXLE, 6 TIRE	3 AXLE OR MORE	4 AXLE OR LESS	5AXLE	6 AXLE OR MORE	5 AXLE OR LESS	6 AXLE	7 AXLE OR MORE	AVERAGE ESAL
URBAN													
INTERSTATE	0.09%	90.26%	1.54%	0.45%	1.14%	0.57%	0.30%	4.83%	0.14%	0.32%	0.11%	0.26%	8.11%
FLEXIBLE ESAL by VEHICLE GROUP	*	*	*	0.812	0.259	0.569	0.615	1.221	1.357	1.558	0.961	2.185	1.038
RIGID ESAL by VEHICLE GROUP	*	*	*	0.921	0.228	0.752	0.613	1.837	2.016	1.418	0.866	2.750	1.441
OTHER FREEWAY & EXPRESSWAY	0.09%	90.16%	5.73%	0.28%	1.35%	0.35%	0.26%	1.34%	0.06%	0.10%	0.04%	0.24%	4.02%
FLEXIBLE ESAL by VEHICLE GROUP	*	*	*	0.717	0.198	1.026	0.598	1.191	1.380	1.426	0.761	1.539	0.797
RIGID ESAL by VEHICLE GROUP	*	*	*	0.746	0.175	1.481	0.586	1.780	2.080	1.305	0.656	2.301	1.078
OTHER PRINCIPAL ARTERIALS	0.38%	88.82%	4.08%	0.65%	1.70%	0.58%	0.80%	2.15%	0.13%	0.25%	0.11%	0.34%	6.72%
FLEXIBLE ESAL by VEHICLE GROUP	*	*	*	1.012	0.269	0.906	1.088	1.223	1.313	2.253	0.952	1.920	0.989
MINOR ARTERIALS	0.18%	87.03%	7.83%	0.46%	2.25%	0.30%	0.54%	1.01%	0.05%	0.16%	0.05%	0.14%	4.96%
FLEXIBLE ESAL by VEHICLE GROUP	*	*	*	0.839	0.239	0.938	0.681	1.285	1.265	2.099	0.593	1.852	0.715
MINOR COLLECTORS	0.19%	95.20%	1.23%	0.40%	0.72%	0.35%	0.55%	0.31%	0.04%	0.23%	0.14%	0.82%	3.56%
FLEXIBLE ESAL by VEHICLE GROUP	*	*	*	*	*	*	*	*	*	*	*	*	*
LOCAL ROADS	0.19%	91.22%	4.08%	0.38%	2.93%	0.55%	0.04%	0.46%	0.03%	0.01%	0.01%	0.10%	4.51%
FLEXIBLE ESAL by VEHICLE GROUP	*	*	*	*	*	*	*	*	*	*	*	*	*

* Data not available for these Roadway Items

Virginia Street -- Urban Major Arterial

Description	Vehicle Class	Daily Volume	Adj. Total Volume	NDOT 2007 ESAL Factor	Daily ESAL
Motorcycles	1			-	-
Passenger Cars	2			-	-
Pickups / Vans	3			-	-
Buses	4	96	96	1.102	105.74
SU, 2 axle, 6 tire	5	251	251	0.269	67.65
SU, 3 axle	6	86	86	0.906	77.78
SU, 4 or more axle	7	118	118	1.088	128.57
ST, 4 or less axle	8	0	0	1.088	0.00
ST, 5 axle	9	318	318	1.223	389.10
ST, 6 or more axle	10	19	19	1.313	25.20
MT, 5 or less axle	11	37	37	2.253	84.19
MT, 6 axle	12	16	16	0.952	15.38
MT, 7 or more axle	13	51	51	1.920	96.96
	Total	993	993	0.678	990.57

check

100.00%

20 Year ESAL

7,304,916

Truck Volumes	# of days	% Truck Adj	NDOT Seasonal Factor	RTC Growth Rate
	1	1	1.0102	1

Truck Volumes
95
249
85
117
0
315
19
37
16
50

Virginia Street – Urban Major Arterial 40 year PCC

Description	Vehicle Class	Daily Volume	Adj. Total Volume	NDOT 2007 ESAL Factor	Daily ESAL
Motorcycles	1			-	-
Passenger Cars	2			-	-
Pickups / Vans	3			-	-
Buses	4	96	96	1.102	105.74
SU, 2 axle, 6 tire	5	251	251	0.269	67.65
SU, 3 axle	6	86	86	0.906	77.78
SU, 4 or more axle	7	118	118	1.088	128.57
ST, 4 or less axle	8	0	0	1.088	0.00
ST, 5 axle	9	318	318	1.223	389.10
ST, 6 or more axle	10	19	19	1.313	25.20
MT, 5 or less axle	11	37	37	2.253	84.19
MT, 6 axle	12	16	16	0.952	15.38
MT, 7 or more axle	13	51	51	1.920	96.96
	Total	993	993	0.678	990.57

Truck Volumes	# of days	% Truck Adj	NDOT Seasonal Factor	RTC Growth Rate
	1	1	1.0102	1
95				
249				
85				
117				
0				
315				
19				
37				
16				
50				

check
100.00%

20 Year
ESAL
14,609,832

BLACK EAGLE CONSULTING
Geotechnical and Construction Services
Designed By: DH
Checked By: mcd

Date: 5-13-11
Project No. 500-03-1 .

ROAD NAME: Virginia Street

TRAFFIC DATA CALCULATIONS

Traffic Data Provided by: Jacobs

1.) Calculate average annual growth rate between two different years of one way trafic projections:

$y_i := 2008$ $Y_i := 14050$ $y_o := 2030$ $Y_e := 17390$ (vehicles per day, two way)

$n := (y_o - y_i) - 1$ $n = 21$

For 21 years, Average Annual Growth Factor = $G_f := \left(\frac{Y_e}{Y_i}\right)^{\frac{1}{n}}$

$G_f = 1.0102$ = 1.02 % per year

CHECK

$Y_{2019} := (Y_i) \cdot G_f^{\,n}$

$Y_{2019} = 17390$ OK

2.) Using the average annual growth rate calculated above, back calculate an initial daily traffic to be assumed for the starting year, in this case the year 2012:

$Y_{2012} := Y_i \cdot G_f^{\,4}$

$Y_{2012} = 14633$ vehicles per day, two way

Use IDT of 14633 vpd for year 2012

BLACK EAGLE CONSULTING
Geotechnical and Construction Services
Designed By: DH
Checked By: mcd

Date: 5-13-11
Project No. 500-03-1

ROAD NAME: Virginia Street

STRUCTURAL SECTION DESIGN for FLEXIBLE PAVEMENT USING AASHTO/NDOT METHOD

References:

1.) AASHTO, 1993 : *Design manual for design of rigid and flexible pavements*

2.) Nevada Dept of Transporation, 2009: *Annual Traffic Report*.

3.) Traffic Projections provided by Jacobs

CALCULATION OF 20 YEAR DESIGN ESAL

$ESAL_{20} := 7304916$ (refer to Appendix A)

CALCULATION OF RESILILENT MODULUS, M_r

Design R-Value: $R_v := 44$ (NDOT Conversion to Resillient Modulus)

$logM := (.0143 \cdot R_v) + \log(17.43)$

$logM = 1.87$ $M_p := 10^{logM}$ $M_p = 74.216$ (in Mpa)

$M_r := M_p \cdot 145.03$ $M_r = 1.076 \times 10^4$ (in psi)

VARIABLES:

Reliability:

	Urban	Rural
Interstate:	85-95%	80-90%
U.S. Routes:	80-90%	75-85%
State Routes:	75-85%	70-85-%
Low Volume:	50-80%	50-80%

Select: $R := 90$

Standard Deviation: $S_o := .45$

Initial Serviceability Index: $P_o := 4.5$ for Profileograph < 5 in/mile

Terminal Serviceability Index:

ADT >750: 2.5
ADT= 750: 2.5
ADT<750: 2.0

Select: $P_t := 2.5$

Change in Serviceability: $\Delta PSI := P_o - P_t$ $\Delta PSI = 2$

SN to start iteration: $SN := 3$

$M_r = 1.076 \times 10^4$ $ESAL_{20} = 7.305 \times 10^6$

Interpolate Value for Z_R for the selected Reliability, R:

$$r := \begin{pmatrix} 50 \\ 60 \\ 70 \\ 80 \\ 90 \\ 95 \\ 99 \\ 99.9 \end{pmatrix} \qquad z := \begin{pmatrix} .000 \\ -.253 \\ -.524 \\ -.841 \\ -1.28 \\ -1.64 \\ -2.32 \\ -3.09 \end{pmatrix}$$

$Z_R := \text{linterp}(r, z, R)$ $\quad\quad Z_R = -1.28$

$$SN := \text{root}\left[Z_R \cdot S_o + 9.36 \cdot \log(SN + 1) - 0.20 + \frac{\log\left(\frac{\Delta PSI}{4.2 - 1.5}\right)}{0.40 + \frac{1094}{(SN + 1)^{5.19}}} + 2.32 \cdot \log(M_r) - 8.07 - \log(ESAL_{20}), SN\right]$$

$SN = 4.015$

PAVEMENT THICKNESS DESIGN

Layer Coefficients from Reference 2:

Material Type	Coefficient	
Plantmix Surface (AC):	0.35	$AC := 0.35$
Plantmix Base (PB):	0.32	$PB := 0.32$
Cement Treated Base (CTB):	0.23	$CTB := 0.23$
Type 2 Base (AB):	0.10	$AB := 0.10$
Borrow (SF):	0.07	$SF := 0.07$

Calculate required thickness of components where: SN = D x AC+ t x AB + x SF

D = thickness of Plantmix Surface, AC
t = thickness of Borrow (Subbase)

$t := 00$

D

AC thickness

$$D = \begin{pmatrix} 6 \\ 6.5 \\ 7 \\ 7.5 \\ 8 \\ 8.5 \\ 9 \\ 9.5 \\ 10 \\ 10.5 \\ 11 \\ 11.5 \\ 12 \\ 12.5 \\ 13 \end{pmatrix} \qquad \frac{SN - AC \cdot D - (t \cdot SF)}{AB} = \begin{pmatrix} 19.154 \\ 17.404 \\ 15.654 \\ 13.904 \\ 12.154 \\ 10.404 \\ 8.654 \\ 6.904 \\ 5.154 \\ 3.404 \\ 1.654 \\ -0.096 \\ -1.846 \\ -3.596 \\ -5.346 \end{pmatrix}$$

Aggregate Base thickness

DESIGN RECOMMENDATIONS

Note that the existing section consists of 12" AC over 12" AB in the thickest measured section. NDOT usually requires matching the existing section when it exceeds the design section.

The design section would be 11.5 inches of full depth AC or 8" AC on 13" AB. If we simply removed 8 inches of existing AC there would be approximately 4 inches of AC and 12 inches of AB left in-place. If we removed 12 inches there would still be 12 inches of base, essentially matching the existing section.

The City of Reno MINIMUM is 6" AC on 12" AB.

BLACK EAGLE CONSULTING
Geotechnical and Construction Services
Designed By: DH
Checked By: mcd

Date: 5-13-11
Project No. 500-03-1

ROAD NAME: Virginia Street (ESAL of Matching Section)

STRUCTURAL SECTION DESIGN for FLEXIBLE PAVEMENT USING AASHTO/NDOT METHOD

References:

1.) AASHTO, 1993 : *Design manual for design of rigid and flexible pavements*

2.) Nevada Dept of Transporation, 2009: *Annual Traffic Report*.

3.) Traffic Projections provided by N/A

CALCULATION OF 20 YEAR DESIGN ESAL

$ESAL_{20} := 60000000$ Find ESAL that can be accomodated by 12" AC over 12" AB

CALCULATION OF RESILILENT MODULUS, M_r

Design R-Value: $R_v := 44$ (NDOT Conversion to Resillient Modulus)

$logM := (.0143 \cdot R_v) + \log(17.43)$

$logM = 1.87$ $M_p := 10^{logM}$ $M_p = 74.216$ (in Mpa)

$M_r := M_p \cdot 145.03$ $M_r = 1.076 \times 10^4$ (in psi)

CALCULATION OF STRUCTURAL NUMBER, SN

VARIABLES:

Reliability:

	Urban	Rural
Interstate:	85-95%	80-90%
U.S. Routes:	80-90%	75-85%
State Routes:	75-85%	70-85-%
Low Volume:	50-80%	50-80%

Select: $R := 90$

Standard Deviation: $S_o := .45$

Initial Serviceability Index: $P_o := 4.5$ for Profileograph < 5 in/mile

Terminal Serviceability Index:

ADT >750: 2.5
ADT= 750: 2.5
ADT<750: 2.0

Select: $P_t := 2.5$

Change in Serviceability: $\Delta PSI := P_o - P_t$ $\Delta PSI = 2$

SN to start iteration: $SN := 3$

$M_r = 1.076 \times 10^4$ $ESAL_{20} = 6 \times 10^7$

Interpolate Value for Z_R for the selected Reliability, R:

$$r := \begin{pmatrix} 50 \\ 60 \\ 70 \\ 80 \\ 90 \\ 95 \\ 99 \\ 99.9 \end{pmatrix} \qquad z := \begin{pmatrix} .000 \\ -.253 \\ -.524 \\ -.841 \\ -1.28 \\ -1.64 \\ -2.32 \\ -3.09 \end{pmatrix}$$

$Z_R := \text{linterp}(r, z, R) \qquad Z_R = -1.28$

$$SN := \text{root}\left[Z_R \cdot S_o + 9.36 \cdot \log(SN + 1) - 0.20 + \frac{\log\left(\frac{\Delta PSI}{4.2 - 1.5}\right)}{0.40 + \frac{1094}{(SN + 1)^{5.19}}} + 2.32 \cdot \log(M_r) - 8.07 - \log(ESAL_{20}), SN \right]$$

$SN = 5.401$

PAVEMENT THICKNESS DESIGN

Layer Coefficients from Reference 2:

Material Type	Coefficient	
Plantmix Surface (AC):	0.35	$AC := 0.35$
Plantmix Base (PB):	0.32	$PB := 0.32$
Cement Treated Base (CTB):	0.23	$CTB := 0.23$
Type 2 Base (AB):	0.10	$AB := 0.10$
Borrow (SF):	0.07	$SF := 0.07$

Calculate required thickness of components where: SN = D x AC+ t x AB + x SF

D = thickness of Plantmix Surface, AC
t = thickness of Borrow (Subbase)

$t := 00$

$D := 11, 11.5 .. 13$

AC thickness $D = \begin{pmatrix} 11 \\ 11.5 \\ 12 \\ 12.5 \\ 13 \end{pmatrix}$ $\frac{SN - AC \cdot D - (t \cdot SF)}{AB} = \begin{pmatrix} 15.509 \\ 13.759 \\ 12.009 \\ 10.259 \\ 8.509 \end{pmatrix}$ Aggregate Base thickness

DESIGN RECOMMENDATIONS

Note that the existing section consists of 12" AC over 12" AB in the thickest measured section. NDOT usually requires matching the existing section when it exceeds the design section.

This section would accomodate an ESAL of 60 million which is over 8 times the design traffic.

BLACK EAGLE CONSULTING
Geotechnical and Construction Services
Sheet 1 of 3
Designed By: DH
Checked By: mcd

Date: 5-10-11
Project No. 500-03-1.

PROJECT NAME: Virginia Street 40 Year Design Life

STRUCTURAL SECTION DESIGN for RIGID PAVEMENT USING AASHTO/NDOT METHOD

References: 1.) AASHTO, 1993 : *Design manual for design of rigid and flexible pavements*

2.) Nevada Dept. of Transportation, 1997: *Pavement structural section design and policy manual*

CALCULATION OF SIMPLE EQUIVALENT SINGLE AXLE LOAD, ESAL

DATA:

Design Life in Years: $L := 40$

Assumed Average annual Growth:1.012%

$W_{18} := 14609832$

MODULUS OF SUBGRADE REACTION, k, in pci

Estimate of k-value from R-value: $k := 230$ psi

CONCRETE STRENGH PARAMETERS

Based on 28 day unconfined compresive strength of 4000 psi:

Modulus of Rupture (3 point flexural strength): $S_c := 570$ psi

Modulus of Elasticity: $E_c := 3.6 \cdot 10^6$ psi

SOLVE THE RIGID PAVEMENT DESIGN EQUATION FOR THICKNESS, D, in inches

$\log(W_{18}) = 7.165$

$$Z_R \cdot S_o + 7.35 \cdot \log(D+1) - .06 + \frac{\log\left(\frac{\Delta PSI}{4.5 - 1.5}\right)}{\frac{1.624 \cdot 10^7}{(D+1)^{8.46}}} + (4.22 - .032 \cdot P_t) \cdot \log\left[S_c \cdot C_d \cdot \frac{D^{.75} - 1.132}{215.63 \cdot J \cdot \left[D^{.75} - \frac{18.42}{\left(\frac{E_c}{k}\right)^{.25}}\right]}\right]$$

DEFINE VARIABLES:

Reliability: Use 90% for W_{18} less than 54,000,000 and 95% for > 54,000,000

$R := 90$

Standard Deviation: $S_o := .35$

Initial Design Serviceability Index: $P_o := 4.5$ always; entered in equations as 4.5)

Terminal Serviceability Index: Use 2.5 for Urban or 2.0 for Rural $P_t := 2.5$

Drainage Coefficient: Use 1.00 for Aggregate Base and CTB; 1.25 for bases with extensive drainage systems

$C_d := 1.00$

Load Transfer Coefficient: Use 3.9 for Aggergate Interlock and 2.8 for Dowelled Joints

$J := 3.9$

$\Delta PSI := P_o - P_t$ $\Delta PSI = 2$

Interpolate Value for Z_R for the selected Reliability, R:

$$r := \begin{pmatrix} 50 \\ 60 \\ 70 \\ 80 \\ 90 \\ 95 \\ 99 \\ 99.9 \end{pmatrix} \qquad z := \begin{pmatrix} .000 \\ -.253 \\ -.524 \\ -.841 \\ -1.28 \\ -1.64 \\ -2.32 \\ -3.09 \end{pmatrix}$$

$Z_R := \text{linterp}(r, z, R)$ $\qquad Z_R = -1.28$

$A := Z_R \cdot S_o - \log(W_{18}) - .06$ $\qquad A = -7.673$ $\qquad B := \log\left(\frac{\Delta PSI}{4.5 - 1.5}\right)$ $\qquad B = -0.176$

$C := 4.22 - 0.32 \cdot P_t$ $\qquad C = 3.42$ $\qquad E := S_c \cdot C_d$ $\qquad E = 570$

$F := 215.63 \cdot J$ $\qquad F = 840.957$ $\qquad G := \frac{18.42}{\left(\frac{E_c}{k}\right)^{.25}}$ $\qquad G = 1.647$

Estimate thickness for iteration:

$D := 4.0$ inches

$$D := \text{root}\left[A + 7.35 \cdot \log(D + 1) + \frac{B}{1 + \frac{1.624 \cdot 10^7}{(D + 1)^{8.46}}} + C \cdot \log\left[E \cdot \frac{D^{.75} - 1.132}{F \cdot \left(D^{.75} - G\right)}\right], D\right]$$

$D = 12.377$

Minimum PCC Thickness: $D = 12.377$ inches

BLACK EAGLE CONSULTING
Geotechnical and Construction Services
Sheet 1 of 3
Designed By: DH
Checked By: mcd

Date: 5-10-11
Project No. 500-03-1.

PROJECT NAME: Virginia Street -- 20 year Design Life

STRUCTURAL SECTION DESIGN for RIGID PAVEMENT USING AASHTO/NDOT METHOD

References: 1.) AASHTO, 1993 : *Design manual for design of rigid and flexible pavements*

2.) Nevada Dept. of Transportation, 1997: *Pavement structural section design and policy manual*

CALCULATION OF SIMPLE EQUIVALENT SINGLE AXLE LOAD, ESAL

DATA:

Design Life in Years: $L := 20$

Assumed Average annual Growth:1.012%

$W_{18} := 7304916$

MODULUS OF SUBGRADE REACTION, k, in pci

Estimate of k-value from R-value: $k := 230$ psi

CONCRETE STRENGH PARAMETERS

Based on 28 day unconfined compresive strength of 4000 psi:

Modulus of Rupture (3 point flexural strength): $S_c := 570$ psi

Modulus of Elasticity: $E_c := 3.6 \cdot 10^6$ psi

SOLVE THE RIGID PAVEMENT DESIGN EQUATION FOR THICKNESS, D, in inches

$\log(W_{18}) = 6.864$

$$Z_R \cdot S_o + 7.35 \cdot \log(D+1) - .06 + \frac{\log\left(\frac{\Delta PSI}{4.5 - 1.5}\right)}{\frac{1.624 \cdot 10^7}{(D+1)^{8.46}}} + (4.22 - .032 \cdot P_t) \cdot \log\left[S_c \cdot C_d \cdot \frac{D^{.75} - 1.132}{215.63 \cdot J \cdot \left[D^{.75} - \frac{18.42}{\left(\frac{E_c}{k}\right)^{.25}}\right]}\right]$$

DEFINE VARIABLES:

Reliability: Use 90% for W_{18} less than 54,000,000 and 95% for > 54,000,000

$R := 90$

Standard Deviation: $S_o := .35$

Initial Design Serviceability Index: $P_o := 4.5$ always; entered in equations as 4.5)

Terminal Serviceability Index: Use 2.5 for Urban or 2.0 for Rural $P_t := 2.5$

Drainage Coefficient: Use 1.00 for Aggregate Base and CTB; 1.25 for bases with extensive drainage systems

$C_d := 1.00$

Load Transfer Coefficient: Use 3.9 for Aggergate Interlock and 2.8 for Dowelled Joints

$J := 3.9$

$\Delta PSI := P_o - P_t$ $\Delta PSI = 2$

Interpolate Value for Z_R for the selected Reliability, R:

$$r := \begin{pmatrix} 50 \\ 60 \\ 70 \\ 80 \\ 90 \\ 95 \\ 99 \\ 99.9 \end{pmatrix} \qquad z := \begin{pmatrix} .000 \\ -.253 \\ -.524 \\ -.841 \\ -1.28 \\ -1.64 \\ -2.32 \\ -3.09 \end{pmatrix}$$

$$Z_R := \mathrm{linterp}(r, z, R) \qquad Z_R = -1.28$$

$$A := Z_R \cdot S_o - \log(W_{18}) - .06 \qquad A = -7.372 \qquad B := \log\left(\frac{\Delta PSI}{4.5 - 1.5}\right) \qquad B = -0.176$$

$$C := 4.22 - 0.32 \cdot P_t \qquad C = 3.42 \qquad E := S_c \cdot C_d \qquad E = 570$$

$$F := 215.63 \cdot J \qquad F = 840.957 \qquad G := \frac{18.42}{\left(\frac{E_c}{k}\right)^{.25}} \qquad G = 1.647$$

Estimate thickness for iteration:

$D := 4.0$ inches

$$D := \mathrm{root}\left[A + 7.35 \cdot \log(D + 1) + \frac{B}{1 + \frac{1.624 \cdot 10^{7}}{(D + 1)^{8.46}}} + C \cdot \log\left[E \cdot \frac{D^{.75} - 1.132}{F \cdot \left(D^{.75} \ldots\right)}\right], D\right] \qquad D = 11.107$$

Minimum PCC Thickness: $D = 11.107$ inches