GEOTECHNICAL INVESTIGATION

MULLER LANE (SR 757) BRIDGE B-474 REPLACEMENT
DOUGLAS COUNTY, NEVADA

E.A. 73800
NOVEMBER 2017

NEVADA DOT
GEOTECHNICAL ENGINEERING
MATERIALS DIVISION

DEPARTMENT OF TRANSPORTATION

MATERIALS DIVISION

GEOTECHNICAL SECTION

GEOTECHNICAL REPORT
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E.A. 73800

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1.0 Introduction

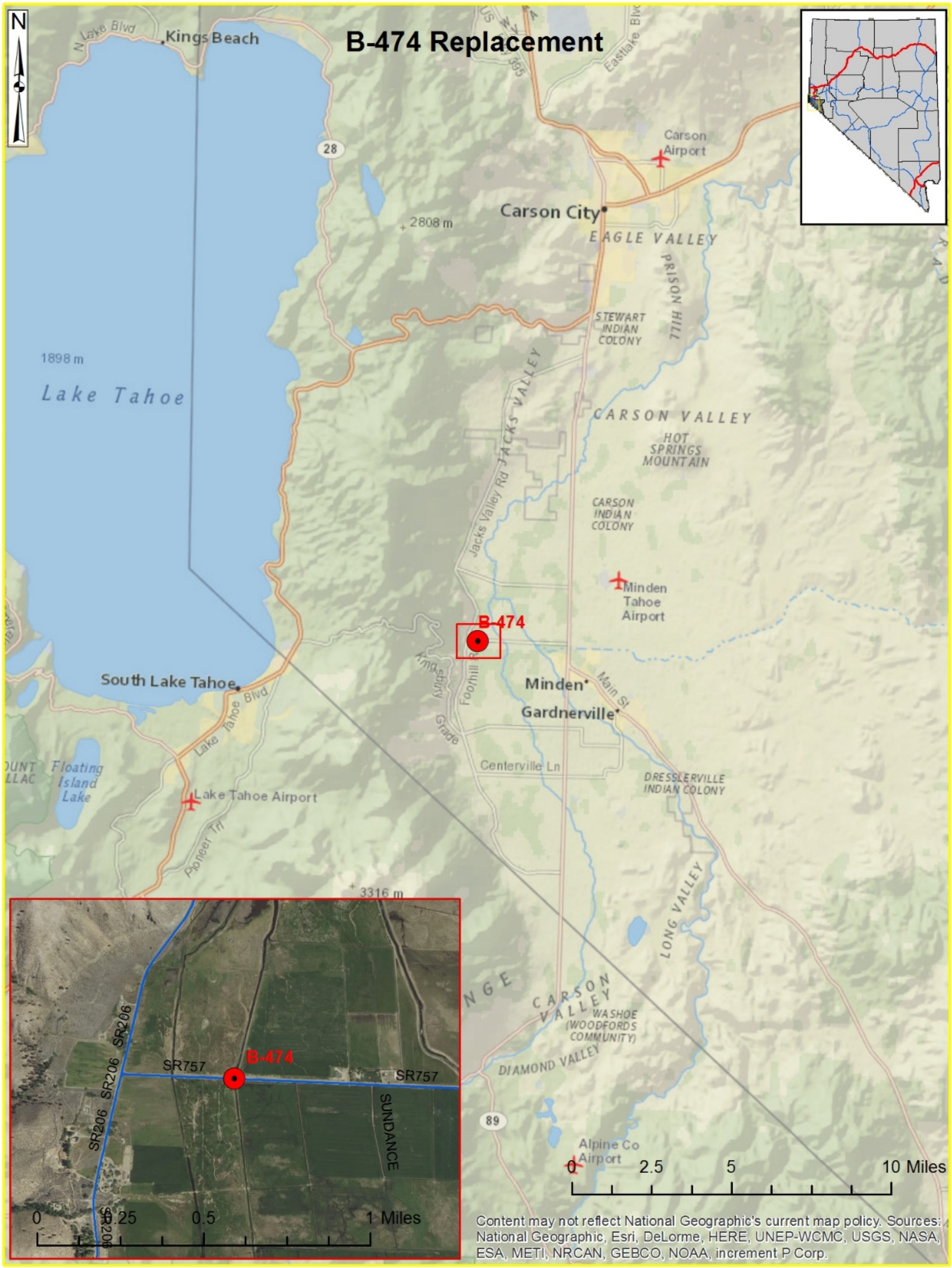
1.1 Project Location and Purpose

This report has been prepared for the Nevada Department of Transportation proposed replacement of a substandard bridge, Structure B-474, located over a waterway on SR 757 near Foothill Road in the vicinity of Genoa, Nevada. State Route 757 runs approximately east-west at this location, and the irrigation ditch, an offshoot of the west fork of the Carson River runs approximately north-south. The proposed plan calls for construction of a replacement structure, maintaining the surface profile of SR 757 unchanged and eliminating the pier in the waterway.

1.2 Project Description

The project bridge site is located in western Douglas County, in the Carson River basin in *Section 28 of Township 15 North, Range 20 East, M.D.M.*, about 3 miles west of US 395 and one-quarter mile east of SR 206 Foothill Road near Minden, Nevada as shown on the next page.

Structure B-474 was built in 1947, replacing an earlier bridge, and crosses a constructed irrigation waterway diverted from the western channel of the Carson River. The structure has two-spans, utilizing square concrete columns, infilled to make curtain walls. The abutments had been surfaced with grouted riprap. Plans show two rows of five untreated timber piles with estimated lengths of 30 feet each supporting the concrete pile cap and abutment on each end of the bridge. A single row of 11 timber piles is shown supporting the pier. Structure B-474 received an overall sufficency rating of 58.00 during its inspection in 2017. The substandard score is attributed to the overall scour potential at the pier and the exposed piles at both the pier and west abutment. These factors, as well as the obsolete superstructure safety features and overall condition of the structure, make this a candidate for replacement.



Project Location Map

Plans indicate the proposed new structure will be a single span concrete bridge structure over the waterway. This will eliminate the pier and related scour issues. Safety features are also addressed.

The replacement bridge will continue to convey one lane of traffic in each direction over the waterway. The structure is proposed to be 36.67 feet in width and 75.00 feet in length with a single row of six, 18-inch diameter driven closed-end steel pipe piles supporting each abutment. The completed structure will have the same footprint and road surface elevation as the existing structure. Traffic will use local roads for detours during construction.

2.0 Scope of Work and Limitations

2.1 Scope of Work

The purpose of this geotechnical investigation is to provide information regarding the subsurface soil and groundwater conditions at the proposed bridge replacement project site, provide recommended geotechnical design values and identify potential risk factors for construction. The completed scope of work for this report consists of a review of published maps and reports, geotechnical investigation and analysis, and determination of recommendations for design and construction. The investigation included gathering data from past field explorations and reports, in addition to information obtained from recent field reconnaissance, subsurface explorations consisting of two borings, soil sampling, and analysis of field and laboratory testing data. This report includes boring logs and summaries of test results from the field investigations and the laboratory-testing regimen. These may be found in the appendices.

2.2 Limitations

This report follows the guidelines of generally accepted geotechnical practice. The Geotechnical Report is based on field observations of the project Geotechnical Engineer, a summary of the subsurface exploration, and the results of laboratory testing of collected soil samples. The report is based on our interpretations of the findings in the two exploratory borings and the geophysical investigation. Therefore, this report may not quantify the exact natural variation of in-situ soils or depth to water. Depth to water can vary based on overall weather patterns, seasonal variation, and local agricultural practice making it difficult to predict at any given time. Any additional analysis or interpretations of the boring logs and other test data, provided by third parties, are not the responsibility of the Department (NDOT). If conditions are encountered during construction, which differ from those found in this report, or if the scope of construction is significantly changed, the Geotechnical Section should be notified to provide additional recommendations.

3.0 Geologic Conditions and Seismicity

3.1 Local Site Geology

The project site is located in the Carson Valley, which is bounded by the Carson Range on the west and by the Pine Nut Mountain Range on the east. According to available references (Geologic Map of Lyon, Douglas, and Ormsby Counties, Nevada; Nevada Bureau of Mines, 1969, Bulletin 75, Plate 1), the project site is located on Quaternary aged river flood plain deposits composed of fine sand, silt and clay.

Glaciers deposited substantial amounts of sediments in the Carson Valley during the late Pleistocene and early Holocene Epochs, which is during the later Quaternary Period, 30,000 to 10,000 years before the present. This deposition occurred as glaciers receded, freeing up the ground rock that the ice had scraped out of the mountains and transporting that debris to the valley with the assistance of the melted ice and snow. The project site is located on these sediments, which are called meander belt deposits, as the local river and stream channels are flowing slowly over near level terrain.

The Carson River has extensively meandered across the Carson Valley and has an extensive flood plain. The river is in extreme old age; therefore, the valley is predominately flat with a gently sloping alluvial fan composed of coarse to fine grained sediments. These descriptions are similar to the conditions encountered during the site visit and subsequent exploration.

3.2 Geologic Setting: Faulting and Seismicity

A review of Major Quaternary and Suspected Quaternary Faults in Nevada indicates there are significant faults in the general area which experienced fault displacement within the last few hundred years.

The largest local fault, the Genoa fault, is a Class A major fault with a slip rate of between 1.0 and 5.0 mm/yr and is located about 1.25 miles northwest of the project site at the base of the Sierra Nevada. This fault was a major contributor to the high local seismic risk. Most of the displacements on faults within the area occurred during the Tertiary geologic period (66 million to 2.58 million years ago), which immediately preceded the Quaternary geologic period. However, earthquake activity continues into the present day. These faults roughly parallel the Sierra Nevada mountain range and do not extend into the alluvium of the valley floor. Occasional fault scarps are present in the alluvial fans near the mountain front, but no mapped faults cross or trend towards the project site.

3.3 Site Class Determination and Seismic Parameters

The seismic provisions of the AASHTO LRFD specifications Article 3.10 are applied to bridge design in Nevada. Earthquake force effects were determined in accordance with AASHTO LRFD article 3.10. Seismic coefficients from the AASHTO LRFD Specifications used for design must meet or exceed the minimum seismic coefficients shown in Figure 12.3-H of the NDOT Structures Manual unless otherwise approved by the Chief Structures Engineer. The coefficients for Douglas County are shown below along with other seismic design parameters. However, these minimum seismic coefficients are not the ultimate design values. They are superseded by the USGS site specific analysis discussed later in this Section.

Nevada General Seismic Design Parameters

Douglas County: Based on NDOT Bridge Structures Division policy:

- Peak Ground Acceleration Coefficient (PGA) = 0.50g
- Short-Period Spectral Acceleration Coefficient (S_s) = 1.25
- Long-Period Spectral Acceleration Coefficient (S_1) = 0.50

AASHTO LRFD Table 3.10.33.1-1, Site Class Definitions: the site is classified as Site Class D.

Response Modification Factor = R = varies, see AASHTO Table 3.10.7.1-1

Vertical Acceleration Coefficient = 0 [AASHTO Appendix A11]

Poisson's ratio for granular backfill material = μ = 0.30

Young Modulus for granular backfill material (E_s; AASHTO Table C10.4.6.3-1):

- E_s= 0.139$N1_{60}$ (ksi) ≈ 4.448 ksi ; for $N1_{60}$ = 32 (estimated)]

Shear Modulus (G) for granular backfill material = E_s / 2(1+μ) ≈ 1.7 ksi

Minimum Seismic Coefficients

AASHTO 3.10.1 recommends selecting your Peak Ground Acceleration (PGA) based on the Horizontal Peak Ground acceleration coefficient with seven percent probability of exceedance in 75 years (Approximately 1000-year return period). The PGA, short, and long period response spectral accelerations S_s and S_1 for the site were obtained using the United States Geological Survey (USGS) Design Maps Tool. For the project site, AASHTO recommends a PGA of 0.46g, from figure 3.10.2.1-1. These seismic design parameters are based on Site Class D and adjustments should be made for other site classes, as needed, as shown in AASHTO 3.4.2.3.

The Site Class for the project location is Site Class D, in accordance with Table 3.10.3.1-1 of AASHTO Guide Specifications for LRFD Bridge Design, based on the average shear wave velocity of the upper 100 ft. (V_{s100}). The average shear wave velocity was obtained utilizing the AASHTO Site Classification System as well as Refraction MicroTremor (ReMi™) geophysical testing method as discussed further below in Field Investigation. The recommended peak ground acceleration using site coordinates and USGS hazard data exceeds both the AASHTO Design Map and the NDOT Structures Manual. The final recommended design response spectrum is shown below:

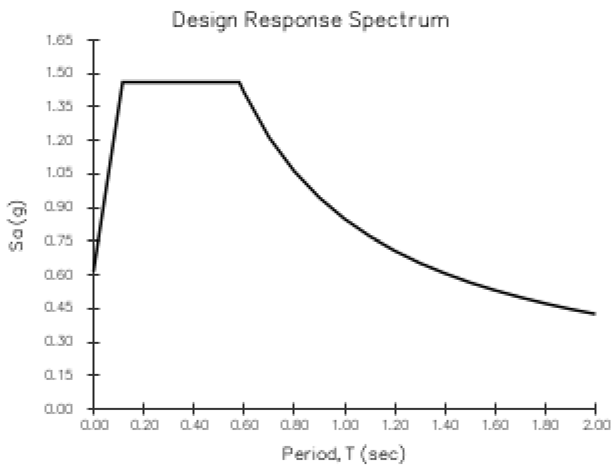
USGS–Provided Output

Building Code Reference Document 2009 AASHTO Guide Specifications for LRFD Seismic Bridge Design (which utilizes USGS hazard data available in 2002)

Site Coordinates 38.971°N, 119.8319°W

Site Soil Classification Site Class D – "Stiff Soil"

PGA=	0.615 g	**A_s =**	0.615 g
S_S =	1.461 g	**S_{DS} =**	1.461 g
S_1 =	0.566 g	**S_{D1} =**	0.849 g



B-474 Muller Lane Design Response Spectrum

4.0 Field Investigation

4.1 Exploratory Borings

The Nevada Department of Transportation (NDOT) Geotechnical Section conducted a subsurface investigation at the proposed project site in April of 2014. Additional exploration was conducted in December of 2017. Subsurface soil conditions were explored by drilling two boreholes (MU-1 and MU-2) to a maximum depth of 54.5 feet below ground surface, corresponding to an elevation of 4621.5 feet, during the April 2014 exploration. Borehole MU-1 was re-drilled to a maximum depth of 86.5 feet below ground surface, corresponding to an elevation of 4589.5, during the December 2017 exploration. Approximate boring locations are shown on the Boring Location Map in Appendix A. One boring was drilled at each end of the bridge. The Boring Logs are included in Appendix B. Surface elevations of the borehole locations were obtained by surveying from known reference elevation points.

Borehole MU-1 was drilled by a Mobile B-57 drill rig using a rotary mud drilling method with bentonite slurry. The first 5 feet of borehole MU-2 was drilled by a Mobile B-57 drill rig using a rotary mud drilling method with bentonite slurry. Below 5 feet, borehole MU-2 was drilled by a Mobile B-57 drill rig using a hollow stem auger without slurry. The boring method was changed due to a complete loss of drilling fluid. The fluid migrated through the cobbles and gravels, deformed the pavement with fluid bubbles under the surface, and flowed to the surface through pavement cracks. Boring MU-2 then encountered practical refusal due to heaving sands, so the boring was terminated at 22 feet below ground surface. Both boreholes were capped and left open after completion of drilling for two days in March 2014 so ground water elevation levels could normalize. At that time, the water levels were measured and both boreholes were backfilled and grouted per Nevada regulations. (No groundwater measurement was taken in December 2017.) Both boreholes showed the same elevation of groundwater, 4668.5 feet.

Pavement distress and drilling fluid upwelling through pavement at Borehole MU-1

The on-site soil conditions were not suitable for using samplers other than a Standard Penetration Test (SPT) sampler, or a driven California Modified Sampler (CMS) due to gravel inclusions; therefore, all recovered samples were disturbed. Soil samples and standard penetration resistance values (N-Values), uncorrected for overburden pressure were obtained utilizing the SPT procedure as set forth in ASTM Standard Test T 206. In addition, N-Values were obtained for the CMS samples, but no STP N-values were calculated. The conversion factor is provided on the Boring Log Key in Appendix B.

The soils were characterized on-site using field classifications in accordance with Visual-Manual procedure (ASTM D 2488) and were recorded at the time of drilling. These logs were then updated as appropriate using laboratory test results and the Unified Soil Classification System (USCS) using ASTM D2487.

4.2 Laboratory Analysis

Laboratory analyses were performed on samples collected from the two boreholes. The testing program consisted of sieve analyses, Atterberg limits, consolidation, moisture content and chemical analyses. Further information is presented in the test results summaries in Appendix C.

4.3 Geophysical Site Investigation

4.3.1 Seismic Data Collection

For this survey, geophones were spaced 20 ft. apart for the line, which ran next to the pavement north of the west bound lane. Background (ambient) noise was used to generate seismic waves during the ReMi™ survey. The drill rig was in operation at the time, delivering consistent sound generation. This local seismic process can aid interpretation of subsurface shear wave velocity at shallow depths. Occasionally, walking and other light disturbances can be used to increase the amplitude of noise energy over a variety of frequencies when working in quiet environments. Noise recordings for ReMi™ analysis were 30 second recording periods with a 2 ms sampling interval. Each individual record is stored in SEG-Y format. In general, 10 individual noise recordings are made for each line. About 30 individual noise recordings were made for this line. Individual records are not stacked or modified until final processing.

4.3.2 ReMi™ Seismic Data Analysis

The analysis and interpretation of the seismic data collected for this project was performed by Optim of Reno, NV. The noise data collected for ReMi™ was analyzed using the proprietary software SeisOpt ReMi™, which was developed by Optim of Reno, NV. The field exploration, noise data acquisition, location survey, and preliminary data verification was performed by geotechnical staff at NDOT on December 4, 2017 and is shown in Appendix D.

4.4 Discussion

4.4.1 Subsurface Conditions

Drilling was conducted in the west bound travel lane about ten feet from each end of the existing bridge. The field investigation and results of the soil sample testing identified the soils under the existing roadbed subgrade to be primarily dense to very dense silty and gravelly sands, with clay and cobbles in near surface locations. Samples were taken through pavement, so surface asphaltic concrete was underlain by roadbed subgrade, gravel and gravelly sand, which are fill materials. These fill materials were present from ground surface to a depth of between 6 feet and 8.5 feet below ground surface. Roadbed subgrade remediation was apparent in an underlayer of cobbles and gravel, contaminated to a greater or lesser extent with underlying clay and silt. The clay layer is moderately plastic, with or without cobbles and gravels and lies immediately under the roadbed subgrade rock remediation layer. The clay layer was medium stiff. Clay soils were primarily a near surface feature, with a 4-foot to 6-foot layer of moderately plastic clay at the surface of native ground, under or including the cobble and gravel layer. Native ground below the clay was predominately silty sand, in the east abutment grading to sand, and in the west abutment including cobbles and gravels to about 12 feet below ground surface before grading to silty sand. Based on blow count resistance from driven samplers, most granular soils were classified as medium dense to dense. Plasticity Index (PI) values ranged from non-plastic to 26. Liquid limits values ranged between 17 and 50. These values indicate soil layers generally able to be categorized as saturated silty sand within the elevations estimated to be affected by bridge foundation construction.

4.4.2 General Site Conditions

Based on the results of our geotechnical investigations, the project site is suitable for the proposed bridge reconstruction. No geotechnical or geologic hazards were observed that would make the development of the proposed bridge replacement unsuitable.

4.4.3 Slope Stability

Stability of slopes for this project is limited to embankments, if constructed. All permanent slopes should be constructed to lie at a 2:1 (H:V) slope or flatter. A 2:1 slope or flatter is also recommended in front of the abutments.

4.4.4 Corrosion

Chemical testing indicated corrosion is not of concern for this site. Test results for resistivity, pH, sulfates and chlorides all fell outside the ranges indicative of potential pile deterioration.

4.4.5 Excavation

All excavation quantities need to be determined based on the NDOT 2014 Standard Specifications for Road and Bridge Construction and the NDOT 2017 Standard Plans for Road and Bridge Construction. The Contractor shall be responsible for all necessary shoring. The Contractor is responsible for following OSHA regulations. Variable site conditions include the possibility of encountering high groundwater levels, artesian hydraulic conditions, caving soils, saturated clays, large cobbles, and heaving sands.

5.0 Foundation Design Recommendations

5.1 General

Shallow foundations are not recommended for this site due to the existence of a high water table and surface clay layer. Drilled shafts are not recommended for this site due to caving soils in subsurface water. This site had been proposed for Geosynthetic Reinforced Soil Integrated Bridge System (GRS-IBS) but other non-geotechnical factors have precluded this design.

We recommend a driven pipe pile foundation for support of the abutments using a closed end with a conical or hemispherical tip. The pipe piles should be 18-inches in diameter, have a nominal wall thickness of at least 0.5 inches and be constructed of Grade 3 steel with a minimum yield stress of f_y >=45 ksi. The NDOT Bridge Division provided an ultimate design load of 1167 kips per abutment with a total of 6 piles per each of the two abutments. Piles were designed using a combination of skin friction and end bearing. Design tip elevation for 18-inch steel pipe piles was estimated to be 4623.5 feet for both abutments. The estimated pile lengths were determined based on axial capacity, lateral pile stability, drivability and predicted scour elevations. The pile length was determined to be 33.5 feet bearing in native soil below scour depth elevation. Dynamic Load tests are required to verify the design capacity of the piles. Pile steel thickness of ½-inch was determined to be adequate for withstanding the estimated driving stresses. Pre-auguring will only be permitted as noted in the plans and specifications.

Total settlements of less than ¼-inch are expected for 18-inch driven steel pipe piles with nominal capacities of 430 kips per pile, most of which will occur as loads are applied during construction. Differential settlements of ¼-inch or less are expected between each support location.

The proposed pile foundation arrangement as provided by the Structural Engineers for B-474 is as follows:

B-474 Bridge Foundation Piles One Pile Row per Abutment	
Abutment 1	6 per row
Abutment 2	6 per row

Based on the provided information from NDOT Hydraulic Section, the contraction scour elevation for the 100-year event (Design Flood) is 4657 feet. NDOT Hydraulic Section is proposing to install riprap to mitigate scour. However, a riprap revetment can only eliminate the abutment scour. The pile foundations were designed using the contraction scour elevations provided.

5.2 Foundation Loads

The Structural Engineer provided the following foundation design loads for each abutment:

B-474 Bridge Foundation Piles	
Service Load	766 kips
Ultimate Load	1167 kips

The geotechnical axial compression resistance of a single driven pile at Strength I and the settlement (vertical deformation) at Service I, includes the effect of scour at the design flood (100-year flood).

5.3 Driven Pipe Pile Design Recommendations

The soil profile below the pile caps mainly consists of cohesionless silty sand with occasional gravel and cobbles. The soil below the 100-year scour depth is dense. The pile capacity is a combination of side resistance and end-bearing. The side resistance and the end-bearing capacities were estimated using the Nordlund/Thurman Method (AASHTO Table 10.5.5.2.3-1). A resistance factor of 0.65 was used in the expectation that dynamic testing would be performed. A resistance factor of 0.35 would be required using the Nordlund/Thurman Method without dynamic testing.

We recommend that the following summary table be included in the bridge construction plans:

BRIDGE B-474			
LOCATION	**MIN. TIP ELEVATION (FEET)**	**DESIGN TIP ELEVATION (FEET)**	**REQUIRED PILE DRIVING RESISTANCE (KIPS)**
ABUTMENT #1	**4633.5**	**4623.5**	**430**
ABUTMENT # 2	**4633.5**	**4623.5**	**430**

We anticipate that piles lengths of 45.5 feet will be acceptable for the project.

5.4 Lateral Earth Pressure on Abutment Walls and Wing Walls

Seat Type Abutment: We understand that the bridge abutments will be pre-cast pile caps with abutment walls supported on the pile caps. The abutment walls will be seat-type, which can deflect at the top and cause lateral active earth pressure to develop. The following soil parameters are recommended for the structural design of the abutment walls:

LATERAL EARTH PRESSURE DESIGN PARAMETERS SEAT-TYPE ABUTMENTS Deflection at the top of the wall is more than 0.5% of the wall height. Design allows no build-up of hydrostatic pressure. Do not use heavy static or dynamic compaction equipment within a distance of one-half the wall height behind the wall.	
Static Active Earth Pressure Coefficient = K_A = 0.256	Coulomb's equation for δ/ϕ_f = 0.5
Static Active Earth Pressure = $K_A \gamma H$	
Static Active Earth Force by the Driving Wedge = ½ $K_A \gamma H^2$	located at 1/3 from the bottom of the wall footing
Seismic Active Earth Pressure Coefficient = K_{AE} = 0.330	Mononobe and Okabe
$K_{h0} = F_{pga}$ PGA = A_s = (1.5) (.615) = 0.923	Zero wall displacement
$K_h = 0.5\ K_{h0}$ where wall is capable of displacement of 1.0 to 2.0 in. = 0.462	
Seismic Active Earth Pressure = $K_{AE} \gamma H$	
Seismic Active Earth Force by the Driving Wedge = ½ $K_{AE} \gamma H^2$	
Abutment Backfill Internal Friction Angle (ϕ_f)	34 degrees
Backfill Unit Weight (γ_f)	0.125 kcf
Interface Friction Coefficient (tan δ) for formed concrete against soil Interface Friction Coefficient (tan δ) for mass concrete on soil	tan δ = 0.35 tan δ = 0.50

5.5 Earthquake-induced Soil Liquefaction

Earthquake induced soil liquefaction is evaluated under Extreme Event I limit state (AASHTO 10.7.4). Initial liquefaction screening criteria to determine whether or not a liquefaction analysis is needed for this bridge were done according to AASHTO 10.5.4.2 Since $(N_1)_{60}$ of the soil layers is greater than 25 blows/ft and the normalized shear wave velocity, V_{S1}, is greater than 600 feet/second for soils below the design scour depth, the potential for soil liquefaction occurrence at this site is minimal.

5.6 Earthquake-induced Downdrag

Earthquake induced downdrag was applied to the piles in combination with other applied loads under Extreme Event I limit state (AASHTO 10.7.4, 3.11.8). Since all piles resistance are based on combination of skin friction and end bearing and the pile tips will bear on dense soils and the equivalent footing will be located within the dense granular soil, the possibility of downdrag forces on piles are negligible.

5.7 Recommended Construction Observations, Testing and Implementation

Two Pile Driving Analyzer (PDA) tests are required, one for each abutment. The resistance factors used for pile design were predicated on the use of dynamic testing.

6.0 Dynamic Analysis

6.1 Pile Drivability

Pile drivability is truly a construction limit state, but it is treated as a strength limit state. Driving resistance of the driven piles (the ability of the piles to withstand stresses induced during installation) was evaluated by the wave equation method, using computer program GRLWEAP 2010. In addition, the wave equation analyses determine the driving stresses and blow counts based upon hammer size. Thus, the wall thickness and required hammer size were determined to reach a desired capacity.

Pile driving stress (σ_{dr}) anywhere in the pile determined from the analysis shall be as:

$$\sigma dr \leq 0.9\ \varphi_{da} fy$$

φda: AASHTO Table 10.5.5.2.3-1

We recommend pile driving points (shoes) be used on all the piles to minimize the pile damage during the driving.

A trial hammer Delmag D46-02 was used in GRLWEAP 2010 to check the drivability of the piles at this bridge. The software output, included in Appendix E, shows that the piles are drivable and the compression stresses in the piles are within the limit.

7.0 Construction Issues

Construction issues may include complications due to a clay layer four to six feet in thickness, lying from between 4.5 feet to 6 feet below the pavement surface and extending to 8.5 feet to 12 feet below the pavement surface. Cobbles lie directly above this clay in a layer one to two feet thick. Hydraulic conductivity is high within the cobble layer and the gravel and gravelly sand fill above the cobbles, directly below the pavement. The drilling fluid was completely lost to these layers during site exploration drilling at boring MU-2. The water table will likely be high. It was 7.5 feet below the roadway surface during exploration and water has been known to overtop the structure during flood events. Heaving sands were encountered, indicating quick conditions, which should be anticipated from a depth of about 12 feet below the pavement surface to the bottom of the drilling exploration.

8.0 References

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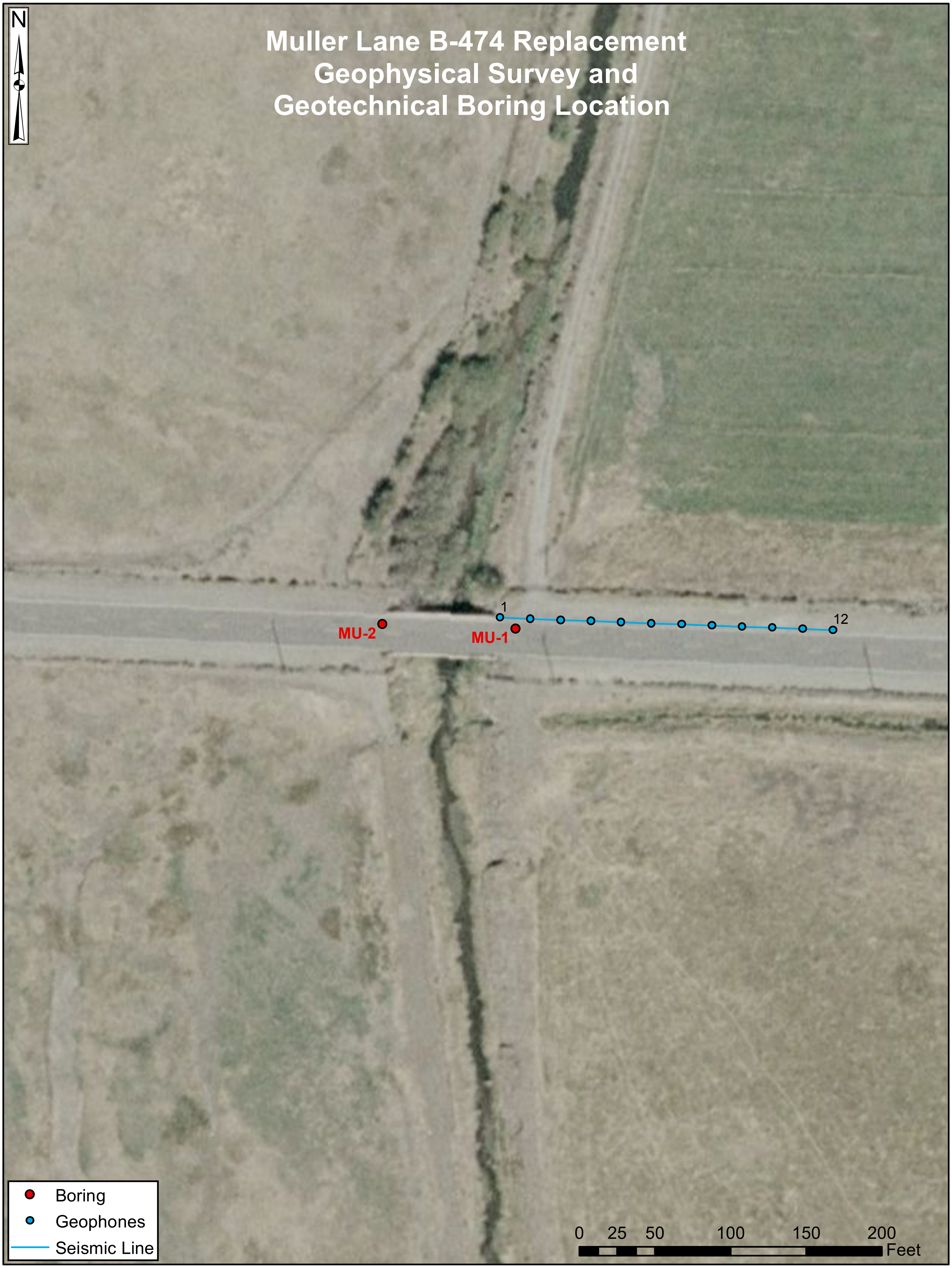
APPENDIX A

Borehole Location Map


N
Muller Lane B-474 Replacement
Geophysical Survey and
Geotechnical Boring Location
1
12
MU-2
MU-1
Boring
Geophones
Seismic Line
0
25
50
100
150
200
Feet

APPENDIX B

Key to Boring Logs
Boring Logs

KEY TO BORING LOGS

PARTICLE SIZE LIMITS							
CLAY	SILT	SAND			GRAVEL		COBBLES	BOULDERS
		FINE	MEDIUM	COARSE	FINE	COARSE		

.002 mm #200 #40 #10 #4 ¾ inch 3 inch 12 inch

USCS GROUP	TYPICAL SOIL DESCRIPTION
GW	Well graded gravels, gravel-sand mixtures, little or no fines
GP	Poorly graded gravels, gravel-sand mixtures, little or no fines
GC	Clayey gravels, poorly graded gravel-sand-clay mixtures
SW	Well graded sands, gravelly sands, little or no fines
SP	Poorly graded sands, gravelly sands, little or no fines
SM	Silty sands, poorly graded sand-silt mixtures
SC	Clayey sands, poorly graded sand-clay mixtures
ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity
CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
OL	Organic silts and organic silt-clays of low plasticity
MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
CH	Inorganic clays of high plasticity, fat clays
OH	Organic clays of medium to high plasticity
PT	Peat and other highly organic soils

MOISTURE CONDITION CRITERIA

Description	Criteria
Dry	Absence of moisture, dusty, dry to touch.
Moist	Damp, no visible free water.
Wet	Visible free water, usually below groundwater table.

∇ ▼ Groundwater Elevation Symbols

SOIL CEMENTATION CRITERIA

Description	Criteria
Weak	Crumbles or breaks with handling or little finger pressure.
Moderate	Crumbles or breaks with considerable finger pressure.
Strong	Won't break or crumble w/finger pressure

STANDARD PENETRATION CLASSIFICATION*			
GRANULAR SOIL		CLAYEY SOIL	
BLOWS/FT	DENSITY	BLOWS/FT	CONSISTENCY
0 - 4	VERY LOOSE	0 - 1	VERY SOFT
5 – 10	LOOSE	2 - 4	SOFT
11 - 30	MEDIUM DENSE	5 - 8	MEDIUM STIFF
31 - 50	DENSE	9 - 15	STIFF
OVER 50	VERY DENSE	16 - 30	VERY STIFF
		31 - 60	HARD
		OVER 60	VERY HARD

*Standard Penetration Test (N) 140 lb hammer 30 inch free fall on 2 inch O.D. x 1.4 inch I.D. sampler.

Field Blow counts on California Modified Sampler (N_{CMS}) for ($6<N_{CMS}<50$) can be converted to $N_{SPT\ field}$ by:

($N_{CMS\ field}$)(0.62) = $N_{SPT\ field}$

Blow counts from Automatic SPT Hammers can be converted to Standard SPT N_{60} by:

Rig #1627: ($N_{SPT\ field}$) (1.2) =N_{60}

Rig #1082: ($N_{SPT\ field}$) (1.45) =N_{60}

TEST ABBREVIATIONS

CD	CONSOLIDATED DRAINED	O	ORGANIC CONTENT
CH	CHEMICAL (CORROSIVENESS)	OC	CONSOLIDATION
CM	COMPACTION	PI	PLASTICITY INDEX
CU	CONSOLIDATED UNDRAINED	RQD	ROCK QUALITY DESIGNATION
D	DISPERSIVE SOILS	RV	R-VALUE
DS	DIRECT SHEAR	S	SIEVE ANALYSIS
E	EXPANSIVE SOIL	SL	SHRINKAGE LIMIT
G	SPECIFIC GRAVITY	U	UNCONFINED COMPRESSION
H	HYDROMETER	UU	UNCONSOLIDATED UNDRAINED
HC	HYDRO-COLLAPSE	UW	UNIT WEIGHT
K	PERMEABILITY	W	MOISTURE CONTENT

SOIL COLOR DESIGNATIONS ARE FROM THE MUNSELL SOIL/ROCK COLOR CHARTS.

EXAMPLE: (7.5 YR 5/3) BROWN

SAMPLER NOTATION

CMS	CALIF. MODIFIED SAMPLER[1]
CPT	CONE PENETRATION TEST
CS	CONTINUOUS SAMPLER[2]
PB	PITCHER BARREL
RC	ROCK CORE[3]
SH	SHELBY TUBE[4]
SPT	STANDARD PENETRATION TEST
TP	TEST PIT

1- I.D.= 2.421 inch
2- I.D.=3.228 inch with tube; 3.50 inch w/o tube
3- NXB I.D.= 1.875 inch
4- I.D.= 2.875 inch

Revised August 2010

EXPLORATION LOG

NEVADA DEPARTMENT OF TRANSPORTATION
GEOTECHNICAL ENGINEERING

START DATE 4/7/14
END DATE 12/5/17
JOB DESCRIPTION Muller Lane Bridge Replacement
LOCATION B-474
BORING MU-1
E.A. # 73800
GROUND ELEV. 4676.00 (ft)
HAMMER DROP SYSTEM Automatic

GROUNDWATER LEVEL		
DATE	DEPTH ft	ELEV. ft
4/10/14	7.50	4668.5

STATION 51+27
OFFSET 5' Left
ENGINEER CALLAGHAN
EQUIPMENT MOBILE B-57
OPERATOR ALTAMIRANO/MARSHAL
DRILLING METHOD Wet w/ Bentonite Slurry
BACKFILLED YES DATE 4/10/2014

ELEV. (ft)	DEPTH (ft)	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT 6 inch Increments	BLOW COUNT Last 1 foot	Percent Recov'd	LAB TESTS
4671.0	5						
	8.00–9.50	A	SPT	5, 11, 3	14	27	W, PI
4666.0	10						
	10.00–11.50	B	MC	4, 4, 2	6	0	
	13.00–14.50	C	MC	3, 4, 4	8	44	W, UW, S, PI
4661.0	15						
	16.00–17.50	D	MC	5, 8, 9	17	67	W, UW, S, PI
	19.00–20.50	E	MC	11, 14, 16	30	61	S, PI
4656.0	20						
	23.00–24.50	F	SPT	10, 10, 13	23	67	W, S, PI
4651.0	25						
	28.00–29.50	G	SPT	13, 15, 13	28	67	W, S, PI

Depth (ft)	USCS Group	MATERIAL DESCRIPTION	REMARKS
0.42		**Asphaltic Concrete**	
0.42–3.50	GW	**Gravel** ; clean, moist, black, semi-rounded fill	fill materials for roadbed construction
3.50–4.50		**Cobbles** ; clean, moist, black fill	
4.50–8.50	CL	**Clay** ; moderate plasticity, moist, black with cobbles and gravel	cobbles appear to be roadbed construction platform
8.50–15.00	ML	**Silt** ; moist, black to dark gray with cobbles and gravel	cobble stuck in sampler tip
15.00–18.00	SM	**Silty Sand** ; moist, black to dark gray with cobbles	
18.00–25.00	SP SM	**Silty Sand** ; moist, black to dark gray with cobbles and gravel	decomposed granite to bottom of hole
25.00–30.00	SW SM	**Silty Sand** ; moist, black to light gray	some heaving 28-30 feet

NEVADA DEPARTMENT OF TRANSPORTATION
GEOTECHNICAL ENGINEERING

EXPLORATION LOG

START DATE 4/7/14
END DATE 12/5/17
JOB DESCRIPTION Muller Lane Bridge Replacement
LOCATION B-474
BORING MU-1
E.A. # 73800
GROUND ELEV. 4676.00 (ft)
HAMMER DROP SYSTEM Automatic

GROUNDWATER LEVEL		
DATE	DEPTH ft	ELEV. ft
4/10/14	7.50	4668.5

STATION 51+27
OFFSET 5' Left
ENGINEER CALLAGHAN
EQUIPMENT MOBILE B-57
OPERATOR ALTAMIRANO/MARSHAL
DRILLING METHOD Wet w/ Bentonite Slurry
BACKFILLED YES DATE 4/10/2014

ELEV. (ft)	DEPTH (ft)	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT 6 inch Increments	BLOW COUNT Last 1 foot	Percent Recov'd	LAB TESTS	USCS Group	MATERIAL DESCRIPTION	REMARKS
								SP SM	**Sand** ; moist, medium to light gray with silt	
	33.00 – 34.50	H	SPT	10 13 14	27	67	W, S, PI			
4641.0	35									
	38.00 – 39.50	I	SPT	12 20 19	39	61	W, S, PI			
4636.0	40									
	42.00							SM	**Silty Sand** ; moist, dark to light gray	
	43.00 – 44.50	J	SPT	12 16 25	41	72	W, S, PI			
4631.0	45									
	45.00							SP SM	**Sand** ; moist, black to light gray with silt	
	48.00 – 49.50	K	SPT	15 26 20	46	77	W, S, PI			
4626.0	50									
	50.00								**Silty Sand** ; moist, black to light gray	
	53.00 – 54.50	L	SPT	11 18 19	37	83	W, S, PI			
4621.0	55									end drilling 54.5 feet April 7, 2014
	55.00 – 56.50	M	SPT	14 15 20	35	87	W, S, PI			begin drilling December 4, 2017
	60.00									

EXPLORATION LOG

NEVADA DEPARTMENT OF TRANSPORTATION — GEOTECHNICAL ENGINEERING

START DATE 4/7/14
END DATE 12/5/17
JOB DESCRIPTION Muller Lane Bridge Replacement
LOCATION B-474
BORING MU-1
E.A. # 73800
GROUND ELEV. 4676.00 (ft)
HAMMER DROP SYSTEM Automatic

GROUNDWATER LEVEL		
DATE	DEPTH ft	ELEV. ft
4/10/14	7.50	4668.5

STATION 51+27
OFFSET 5' Left
ENGINEER CALLAGHAN
EQUIPMENT MOBILE B-57
OPERATOR ALTAMIRANO/MARSHAL
DRILLING METHOD Wet w/ Bentonite Slurry
BACKFILLED YES DATE 4/10/2014

ELEV. (ft)	DEPTH (ft)	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT 6 inch Increments	BLOW COUNT Last 1 foot	Percent Recov'd	LAB TESTS	USCS Group	MATERIAL DESCRIPTION	REMARKS
	61.50	N	SPT	16 20 18	38	87	W, S, PI	SM		begin drilling December 5, 2017
4611.0	65 / 65.00–66.50	O	SPT	15 25 26	51	84	W, S, PI	SM		
4606.0	70 / 70.00–71.50	P	SPT	13 16 31	47	94	W, S, PI	SM		
4601.0	75 / 75.00–76.50	Q	SPT	29 41 40	81	78	W, S, PI	SM		
4596.0	80 / 80.00–81.50	R	SPT	28 23 20	43	78	W, S, PI	SM		
	83.00							SP SM	**Silty Sand** ; moist, black to dark gray with gravel	
4591.0	85 / 85.00–86.50	S	SPT	30 58 51	109	72	W, S, PI	SP SM		
	86.50								**Bottom of Hole**	

EXPLORATION LOG

NEVADA DEPARTMENT OF TRANSPORTATION
GEOTECHNICAL ENGINEERING

START DATE 4/8/14
END DATE 4/8/14
JOB DESCRIPTION Muller Lane Bridge Replacement
LOCATION B-474
BORING MU-2
E.A. # 73800
GROUND ELEV. 4676.00 (ft)
HAMMER DROP SYSTEM Automatic

GROUNDWATER LEVEL		
DATE	DEPTH ft	ELEV. ft
4/10/14	7.50	4668.5

STATION 50+37
OFFSET 5' Left
ENGINEER CALLAGHAN
EQUIPMENT MOBILE B-57
OPERATOR ALTAMIRANO
DRILLING METHOD 6" H.S. AUGER
BACKFILLED YES DATE 4/10/2014

ELEV. (ft)	DEPTH (ft)	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT 6 inch Increments	BLOW COUNT Last 1 foot	Percent Recov'd	LAB TESTS
	3.00						
	4.50	A	SPT	2 10 15	25	22	W, S
4671.0	5						
	5.00						
	6.50	B	SPT	14 16 17	33	33	W, S
	8.00						
	9.50	C	MC	1 2 3	5	94	W, UW, S, PI
4666.0	10						
	11.00						
	12.50	D	MC	2 3 3	6	100	W, UW, S, PI
	14.00						
4661.0	15						
	15.50	E	MC	2 4 6	10	83	W, UW, S, PI
	17.00						
	18.50	F	MC	2 3 9	12	100	W, UW, S, PI
4656.0	20						
4651.0	25						

Depth	USCS Group	MATERIAL DESCRIPTION
0.40		**Asphaltic Pavement**
	SP	**Gravelly Sand** ; moist, gray fill
4.00		**Cobbles** with clean gravel, moist, black fill
6.00	CL	**Clay** ; moderately plastic, moist, dark gray
12.00	SM	**Silty Sand** ; well graded, wet, dark gray
16.00	SP	**SAND** ; poorly graded, wet, gray
22.00		**Bottom of Hole**

REMARKS

- begin wet drilling with bentonite slurry
- fill materials for roadbed construction
- cobble stuck in sampler tip
- cobbles appear to be roadbed construction platform
- complete loss of circulation, drilling fluid bubbling up through pavement cracks
- switch to auger drilling
- sampler bouncing during first set of blows
- decomposed granite to bottom of hole
- sampler overdrives 3 in.
- heaving 1 ft. unable to control with full head of water
- heaving 2 ft.

APPENDIX C

Test Result Summary Sheets
Soil Particle Size Distribution Reports (Gradation Curves)
Consolidation Test Reports
Chemical Analysis

SUMMARY OF RESULTS
N.D.O.T. GEOTECHNICAL SECTION

EA/Cont # 73800 **Job Description** Muller Lane B-474 Abut. 2

Boring No. MU - 1 **Elevation (ft)** **Station** 51+27, 5' Lt. **Date** 4/8/2014

SAMPLE NO.	SAMPLE DEPTH (ft)	SAMP-LER TYPE	N BLOWS per ft.	SOIL GROUP	W%	DRY UW pcf	% PASS #200	LL %	PL %	PI %	STRENGTH TEST					COMMENTS
											TEST TYPE	Φ deg.	C psi	Φ deg.	C psi	
												Peak		Residual		
A	8.0	SPT	14		22.1			44	20	24						
B			6													No Sample Recovered
C	14.0	CMS	8	ML	37.9	82.6	83.6	34	26	8						H, OC
D	17.0	CMS	17	SM	44.2	95.7	14.4	23	NP	NP						CU
E	20.0	CMS	30	SP-SM			6.3	17	NP	NP						
F	23.0	SPT	23	SP-SM	22.0		7.6	17	NP	NP						
G	28.0	SPT	28	SW-SM	19.7		8.0	18	NP	NP						
H	33.0	SPT	27	SP-SM	20.9		8.8	20	NP	NP						
I	38.0	SPT	39	SP-SM	19.8		6.9	19	NP	NP						
J	43.0	SPT	41	SM	17.9		17.6	25	NP	NP						
K	48.0	SPT	46	SP-SM	15.8		10.2	21	NP	NP						
L	53.0	SPT	37	SM	18.9		12.8	23	NP	NP						

CMS = California Modified Sampler 2.42" ID
SPT = Standard Penetration 1.38" ID
CS = Continuous Sample 3.23" ID
RC = Rock Core
PB = Pitcher Barrel
CSS = Calif. Split Spoon 2.42" ID
CPT = Cone Penetration Test
TP = Test Pit
P = Pushed, not driven
R = Refusal
Sh = Shelby Tube 2.87" ID

U = Unconfined Compressive
UU = Unconsolidated Undrained
CD = Consolidated Drained
CU = Consolidated Undrained
DS = Direct Shear
Φ = Friction
C = Cohesion
N = No. of blows per ft., sampler
N = Field SPT N = $(N_{css})(0.62)$

H = Hydrometer
S = Sieve
G = Specific Gravity
PI = Plasticity Index
LL = Liquid Limit
PL = Plastic Limit
NP = Non-Plastic
OC = Consolidation
Ch = Chemical
RV = R - Value
MD = Moisture Density

CM = Compaction
E = Swell/Pressure on Expansive Soils
SL = Shrinkage Limit
UW= Unit Weight
W = Moisture Content
K = Permeability
O = Organic Content
D = Dispersive
RQD = Rock Quality Designation
X = X-Ray Defraction
HCpot = Hydro-Collapse Potential

*** = Average of subsamples**

SUMMARY OF RESULTS
N.D.O.T. GEOTECHNICAL SECTION

EA/Cont # 73800 **Job Description** B-474 Replacement Muller Lane

Boring No. MU-1 **Elevation (ft)** 4676 **Station** **Date** 12/5/2017

SAMPLE NO.	SAMPLE DEPTH (ft)	SAMP-LER TYPE	N BLOWS per ft.	SOIL GROUP	W%	DRY UW pcf	% PASS #200	LL %	PL %	PI %	STRENGTH TEST: TEST TYPE	Peak: Φ deg.	Peak: C psi	Residual: Φ deg.	Residual: C psi	COMMENTS
M	55.0 - 56.5	SPT	35	SM	28.5		26.3	26	NP	NP						
N	60.0 - 61.5	SPT	38	SM	26.5		20.2	25	NP	NP						
O	65.0 - 66.5	SPT	51	SM	19.5		15.7	21	NP	NP						
P	70.0 - 71.5	SPT	47	SM	26.5		21.1	24	NP	NP						
Q	75.0 - 76.5	SPT	81	SM	14.0		14.6	20	NP	NP						
R	80.0 - 81.5	SPT	43	SM	22.8		19.9	25	NP	NP						
S	85.0 - 86.5	SPT	109	SP-SM	14.0		9.8	19	NP	NP						

CMS = California Modified Sampler 2.42" ID	U = Unconfined Compressive	H = Hydrometer	CM = Compaction
SPT = Standard Penetration 1.38" ID	UU = Unconsolidated Undrained	S = Sieve	E = Swell/Pressure on Expansive Soils
CS = Continuous Sample 3.23" ID	CD = Consolidated Drained	G = Specific Gravity	SL = Shrinkage Limit
RC = Rock Core	CU = Consolidated Undrained	PI = Plasticity Index	UW= Unit Weight
PB = Pitcher Barrel	DS = Direct Shear	LL = Liquid Limit	W = Moisture Content
CSS = Calif. Split Spoon 2.42" ID	Φ = Friction	PL = Plastic Limit	K = Permeability
CPT = Cone Penetration Test	C = Cohesion	NP = Non-Plastic	O = Organic Content
TP = Test Pit	N = No. of blows per ft., sampler	OC = Consolidation	D = Dispersive
P = Pushed, not driven		Ch = Chemical	RQD = Rock Quality Designation
R = Refusal	N = Field SPT N = $(N_{css})(0.62)$	RV = R - Value	X = X-Ray Defraction
Sh = Shelby Tube 2.87" ID		MD = Moisture Density	HCpot = Hydro-Collapse Potential

*** = Average of subsamples**

SUMMARY OF RESULTS
N.D.O.T. GEOTECHNICAL SECTION

EA/Cont # 73800 **Job Description** Muller Lane B-474 Abut. 1

Boring No. MU - 2 **Elevation (ft)** **Station** 50+34, 5' Lt. **Date** 4/8/2014

SAMPLE NO.	SAMPLE DEPTH (ft)	SAMP-LER TYPE	N BLOWS per ft.	SOIL GROUP	W%	DRY UW pcf	% PASS #200	LL %	PL %	PI %	STRENGTH TEST: TEST TYPE	Peak: Φ deg.	Peak: C psi	Residual: Φ deg.	Residual: C psi	COMMENTS
A	3.0	SPT	25		8.1		9.3									
B	5.0	SPT	33		2.6		7.5									
C1	8.5	CMS	5	CL	39.0	79.9	85.9	47	22	25						H
C2	9.0	CMS		CH	42.9	77.2	87.5	50	24	26						OC
D1	11.5	CMS	6	CL	35.2	85.8	66.5	45	20	25						H, CU
D2	12.0	CMS		CL	35.8	84.5	61.9	39	21	18						OC
E	15.0	CMS	10	SM	30.5	89.6	29.5	26	24	2						
F1	17.0	CMS	12													Ch
F2	18.0	CMS		SP	22.8	94.3	4.8	21	NP	NP	DS	39	0.5	33	0.3	

CMS = California Modified Sampler 2.42" ID
SPT = Standard Penetration 1.38" ID
CS = Continuous Sample 3.23" ID
RC = Rock Core
PB = Pitcher Barrel
CSS = Calif. Split Spoon 2.42" ID
CPT = Cone Penetration Test
TP = Test Pit
P = Pushed, not driven
R = Refusal
Sh = Shelby Tube 2.87" ID

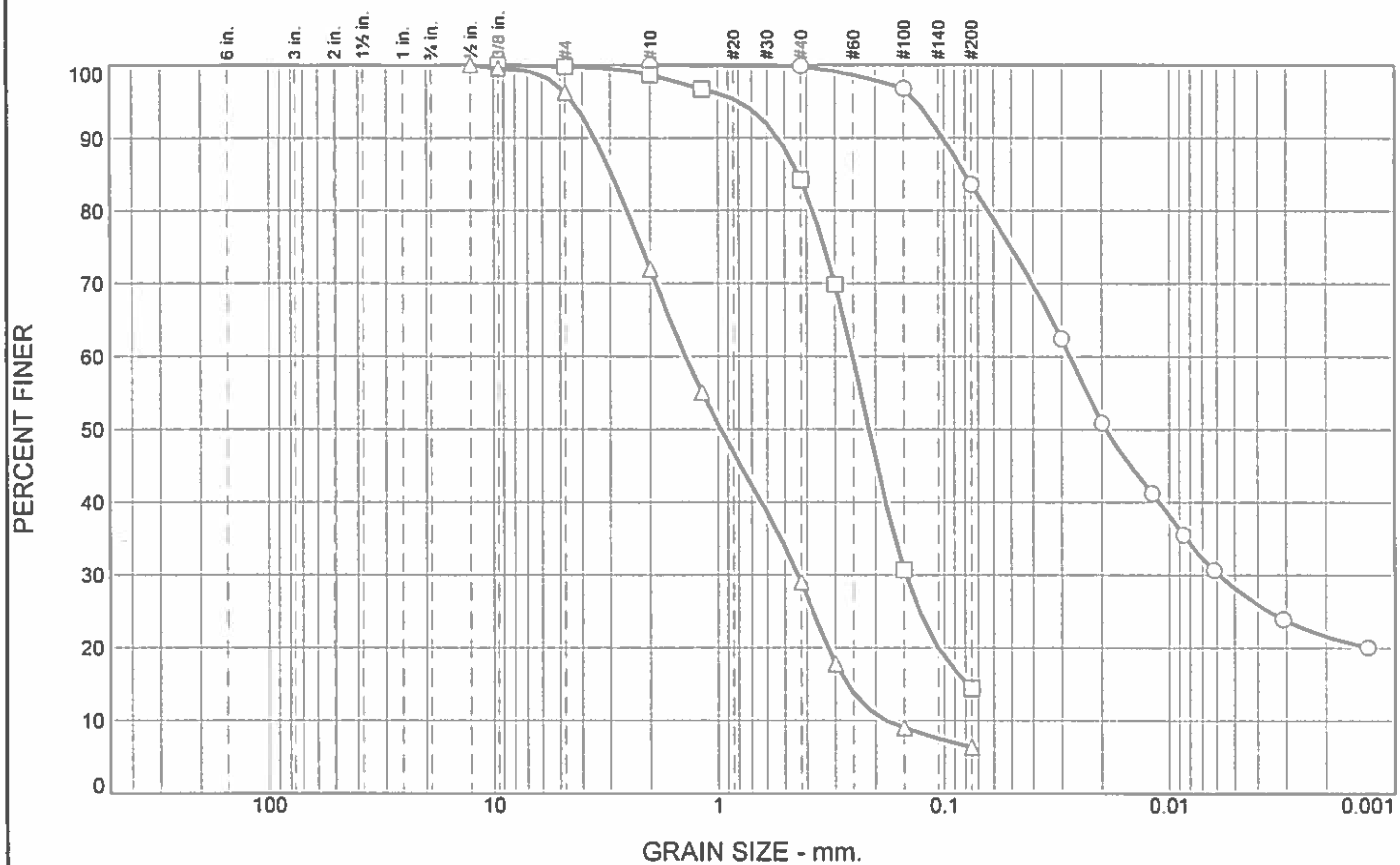
U = Unconfined Compressive
UU = Unconsolidated Undrained
CD = Consolidated Drained
CU = Consolidated Undrained
DS = Direct Shear
Φ = Friction
C = Cohesion
N = No. of blows per ft., sampler
N = Field SPT N = $(N_{css})(0.62)$

H = Hydrometer
S = Sieve
G = Specific Gravity
PI = Plasticity Index
LL = Liquid Limit
PL = Plastic Limit
NP = Non-Plastic
OC = Consolidation
Ch = Chemical
RV = R - Value
MD = Moisture Density

CM = Compaction
E = Swell/Pressure on Expansive Soils
SL = Shrinkage Limit
UW= Unit Weight
W = Moisture Content
K = Permeability
O = Organic Content
D = Dispersive
RQD = Rock Quality Designation
X = X-Ray Defraction
HCpot = Hydro-Collapse Potential

*** = Average of subsamples**

Particle Size Distribution Report



	+3"	% GRAVEL	% SAND	% SILT	% CLAY	USCS	AASHTO	PL	LL
○	0.0	0.0	16.4	62.0	21.6	ML	A-4(7)	26	34
□	0.0	0.2	85.4	14.4		SM	A-2-4(0)	NP	23
△	0.0	3.8	89.9	6.3		SP-SM	A-1-b	NP	17

SIEVE inches size	PERCENT FINER ○	□	△
1/2"			100.0
3/8"		100.0	99.5
	GRAIN SIZE		
D_{60}	0.0276	0.2519	1.3948
D_{30}	0.0059	0.1476	0.4389
D_{10}			0.1801
	COEFFICIENTS		
C_c			0.77
C_u			7.75

SIEVE number size	PERCENT FINER ○	□	△
#4		99.8	96.2
#10	100.0	98.6	72.0
#16		96.7	55.0
#40	99.8	84.2	29.0
#50		69.8	17.8
#100	96.7	30.7	9.0
#200	83.6	14.4	6.3

<u>Material Description</u>

○ silt with sand

□ silty sand

△ poorly graded sand with silt

<u>REMARKS:</u>

○

□

△

○ Source of Sample: MU-1 Depth: 14.0' Sample Number: C

□ Source of Sample: MU-1 Depth: 17.0' Sample Number: D

△ Source of Sample: MU-1 Depth: 20.0' Sample Number: E

NEVADA DEPARTMENT OF TRANSPORTATION

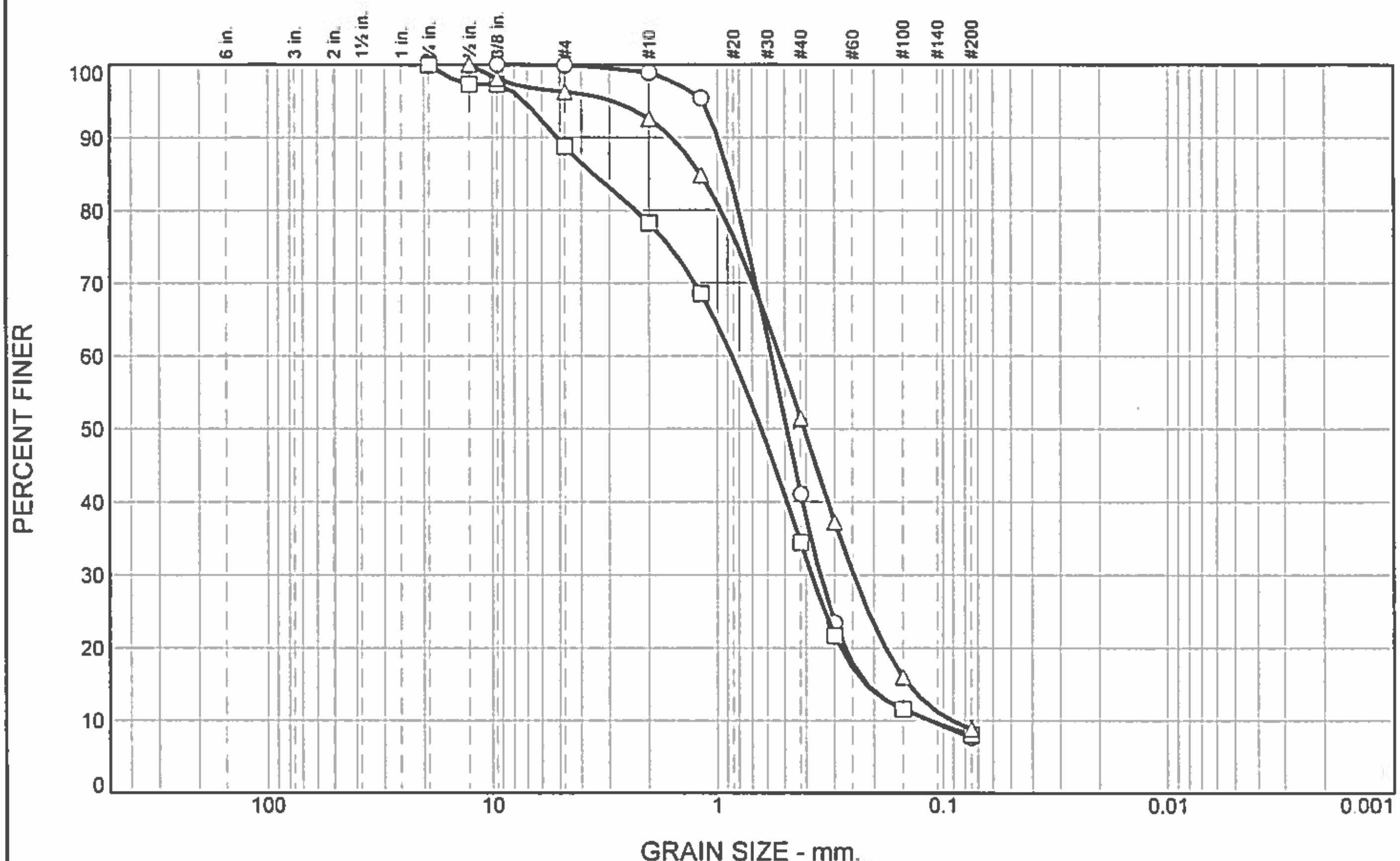
Client: C. Callaghan

Project: Muller Lane B-474

Project No.: FL-4-14

Figure

Particle Size Distribution Report



	+3"	% GRAVEL	% SAND	% SILT	% CLAY	USCS	AASHTO	PL	LL
○	0.0	0.1	92.3	7.6		SP-SM	A-1-b	NP	17
□	0.0	11.3	80.7	8.0		SW-SM	A-1-b	NP	18
△	0.0	3.8	87.4	8.8		SP-SM	A-3	NP	20

SIEVE inches size	PERCENT FINER ○	□	△
3/4"		100.0	
1/2"		97.3	100.0
3/8"	100.0	97.3	98.1
	GRAIN SIZE		
D_{60}	0.5756	0.8654	0.5282
D_{30}	0.3478	0.3798	0.2470
D_{10}	0.1151	0.1145	0.0917
	COEFFICIENTS		
C_c	1.83	1.46	1.26
C_u	5.00	7.56	5.76

SIEVE number size	PERCENT FINER ○	□	△
#4	99.9	88.7	96.2
#10	98.9	78.3	92.6
#16	95.4	68.5	84.8
#40	41.1	34.5	51.4
#50	23.5	21.7	37.3
#100	11.6	11.5	15.9
#200	7.6	8.0	8.8

Material Description

○ poorly graded sand with silt

□ well-graded sand with silt

△ poorly graded sand with silt

REMARKS:

○

□

△

○ Source of Sample: MU-1 Depth: 23.0' Sample Number: F

□ Source of Sample: MU-1 Depth: 28.0' Sample Number: G

△ Source of Sample: MU-1 Depth: 33.0' Sample Number: H

NEVADA DEPARTMENT OF TRANSPORTATION

Client: C. Callaghan

Project: Muller Lane B-474

Project No.: FL-4-14 Figure

Particle Size Distribution Report

	+3"	% GRAVEL	% SAND	% SILT	% CLAY	USCS	AASHTO	PL	LL
○	0.0	3.1	90.0	6.9		SP-SM	A-1-b	NP	19
□	0.0	9.1	73.3	17.6		SM	A-1-b	NP	25

SIEVE inches size	PERCENT FINER ○	□	
1/2"		100.0	
3/8"	100.0	99.4	
	GRAIN SIZE		
D_{60}	0.8228	1.1149	
D_{30}	0.4830	0.2230	
D_{10}	0.1518		
	COEFFICIENTS		
C_c	1.87		
C_u	5.42		

SIEVE number size	PERCENT FINER ○	□	
#4	96.9	90.9	
#10	87.7	70.2	
#16	77.5	60.9	
#40	24.6	42.1	
#50	16.2	35.3	
#100	9.9	24.2	
#200	6.9	17.6	

Material Description
○ poorly graded sand with silt

□ silty sand

REMARKS:
○

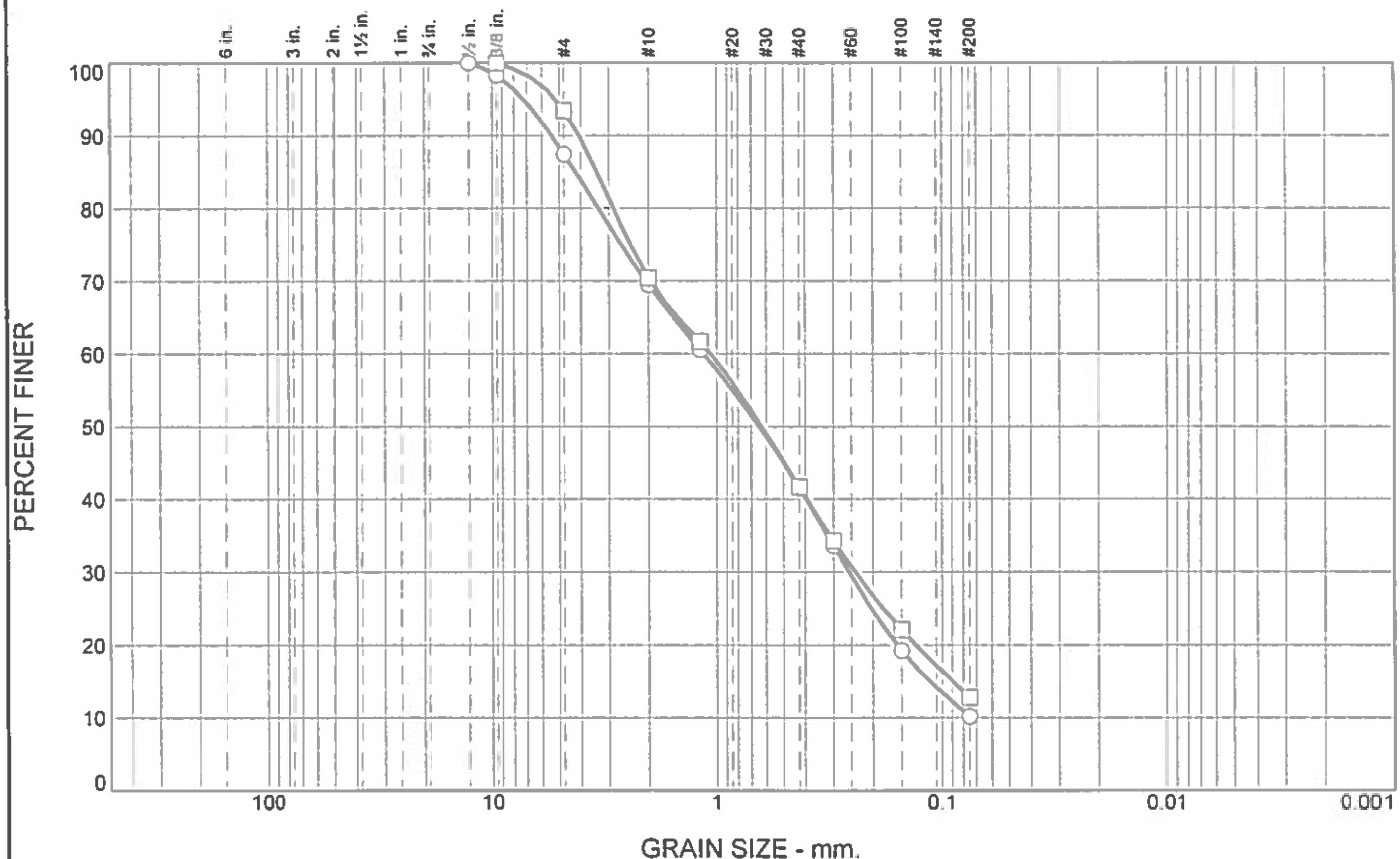
□

○ Source of Sample: MU-1 Depth: 38.0' Sample Number: I
□ Source of Sample: MU-1 Depth: 43.0' Sample Number: J

NEVADA DEPARTMENT OF TRANSPORTATION

Client: C. Callaghan
Project: Muller Lane B-474
Project No.: FL-4-14 Figure

Particle Size Distribution Report



	+3"	% GRAVEL	% SAND	% SILT	% CLAY	USCS	AASHTO	PL	LL
○	0.0	12.6	77.2	10.2		SP-SM	A-1-b	NP	21
□	0.0	6.6	80.6	12.8		SM	A-1-b	NP	23

SIEVE inches size	PERCENT FINER ○	□	
1/2"	100.0		
3/8"	98.3	100.0	
	GRAIN SIZE		
D_{60}	1.1424	1.0653	
D_{30}	0.2558	0.2401	
D_{10}			
	COEFFICIENTS		
C_c			
C_u			

SIEVE number size	PERCENT FINER ○	□	
#4	87.4	93.4	
#10	69.5	70.4	
#16	60.5	61.6	
#40	41.5	41.7	
#50	33.6	34.3	
#100	19.2	22.1	
#200	10.2	12.8	

Material Description

○ poorly graded sand with silt

□ silty sand

REMARKS:

○

□

○ Source of Sample: MU-1 Depth: 48.0' Sample Number: K

□ Source of Sample: MU-1 Depth: 53.0' Sample Number: L

NEVADA DEPARTMENT OF TRANSPORTATION

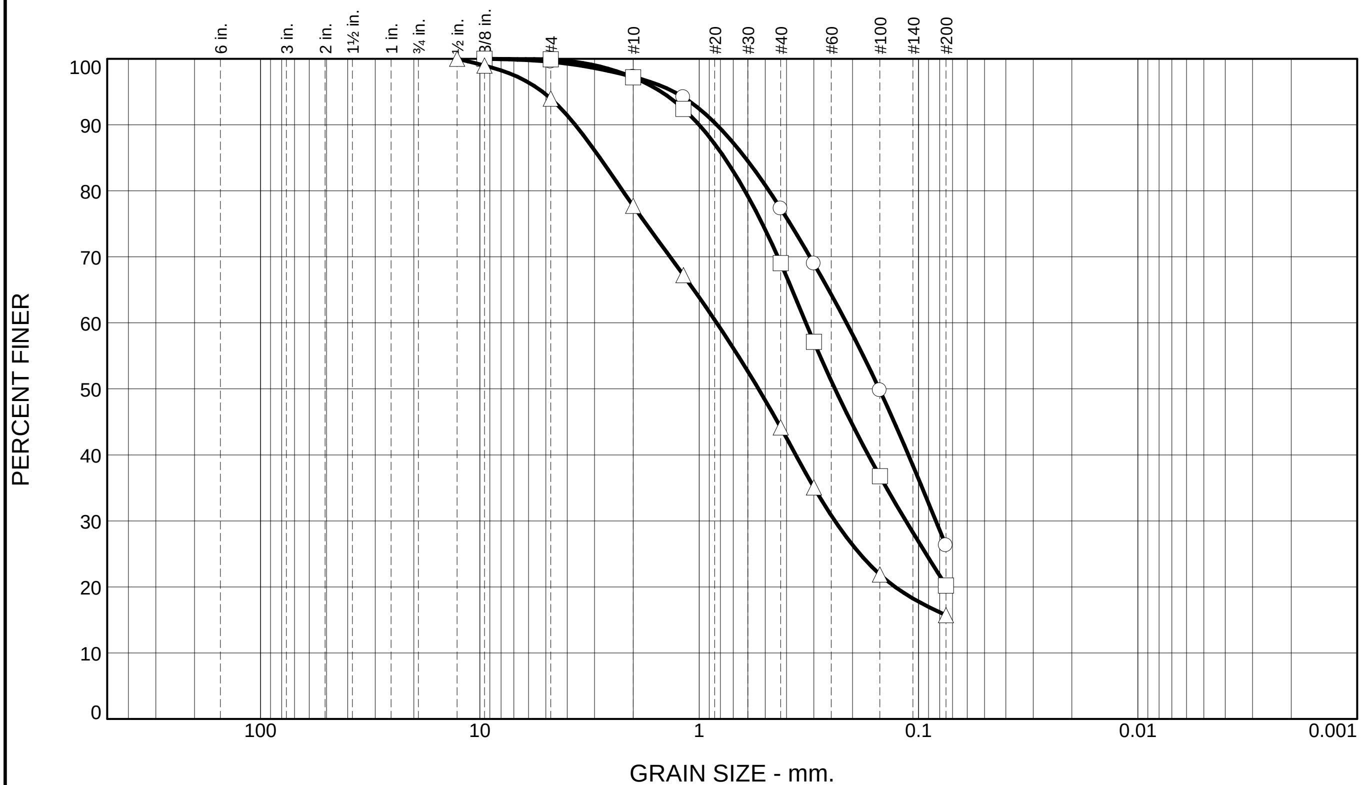
Client: C. Callaghan

Project: Muller Lane B-474

Project No.: FL-4-14

Figure

Particle Size Distribution Report



	+3"	% GRAVEL	% SAND	% SILT	% CLAY	USCS	AASHTO	PL	LL
○	0.0	0.5	73.2	26.3		SM	A-2-4(0)	NP	26
□	0.0	0.1	79.7	20.2		SM	A-2-4(0)	NP	25
△	0.0	6.0	78.3	15.7		SM	A-1-b	NP	21

SIEVE inches size	PERCENT FINER ○	□	△
1/2"			100.0
3/8"	100.0	100.0	99.0
	GRAIN SIZE		
D_{60}	0.2130	0.3263	0.8324
D_{30}	0.0833	0.1142	0.2406
D_{10}			
	COEFFICIENTS		
C_c			
C_u			

SIEVE number size	PERCENT FINER ○	□	△
#4	99.5	99.9	94.0
#10	97.3	97.2	77.7
#16	94.2	92.4	67.2
#40	77.3	69.1	44.2
#50	69.0	57.1	35.1
#100	49.8	36.8	21.9
#200	26.3	20.2	15.7

Material Description
○ silty sand
□ silty sand
△ silty sand

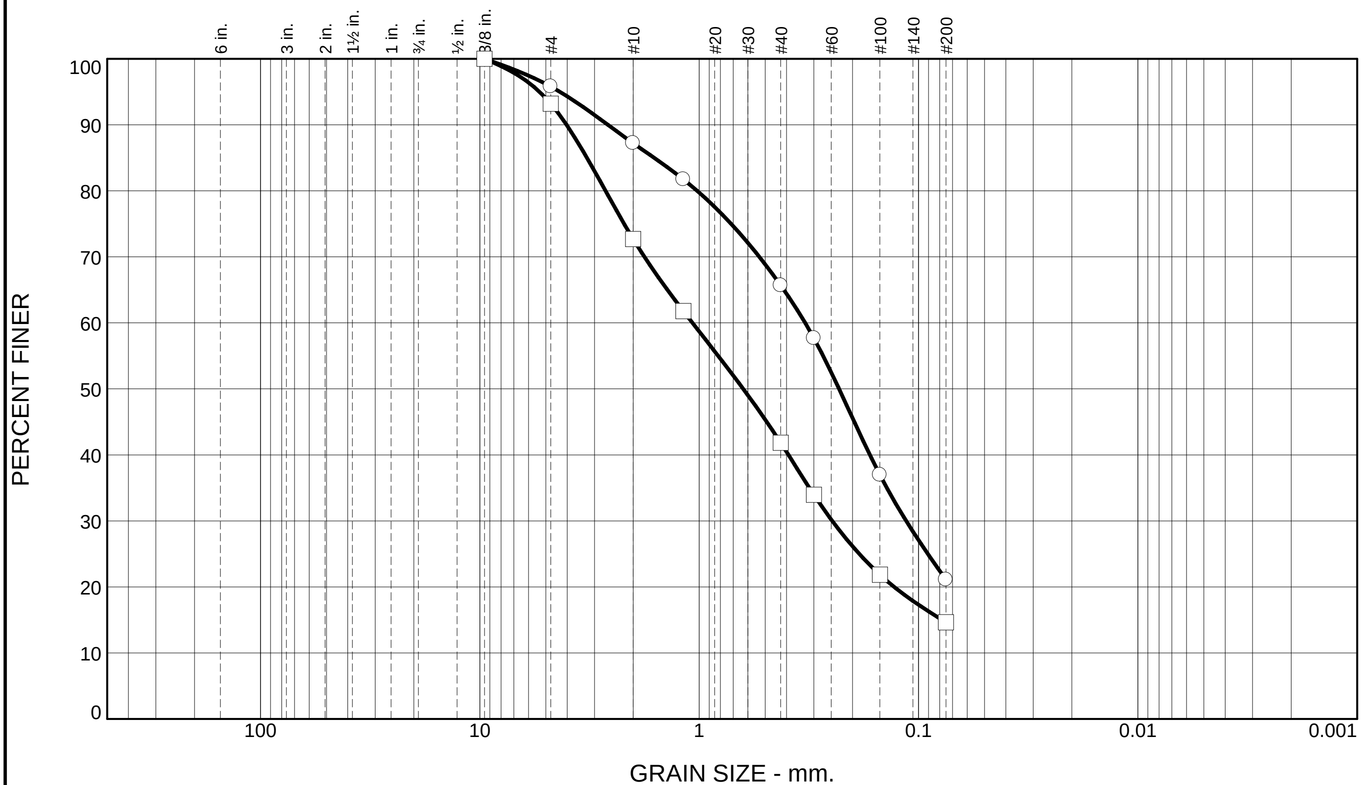
REMARKS:
○
□
△

○ Source of Sample: MU-1 Depth: 55.0' - 56.5 Sample Number: M
□ Source of Sample: MU-1 Depth: 60.0' - 61.5' Sample Number: N
△ Source of Sample: MU-1 Depth: 65.0' - 66.5' Sample Number: O

NEVADA DEPARTMENT OF TRANSPORTATION

Client: C. Callaghan
Project: B-474 Replacement Muller Lane
Project No.: EA 73800 Figure

Particle Size Distribution Report



	+3"	% GRAVEL	% SAND	% SILT	% CLAY	USCS	AASHTO	PL	LL
○	0.0	4.2	74.7	21.1		SM	A-2-4(0)	NP	24
□	0.0	6.8	78.6	14.6		SM	A-1-b	NP	20

SIEVE inches size	PERCENT FINER ○	□	
3/8"	100.0	100.0	
	GRAIN SIZE		
D_{60}	0.3288	1.0727	
D_{30}	0.1138	0.2474	
D_{10}			
	COEFFICIENTS		
C_c			
C_u			

SIEVE number size	PERCENT FINER ○	□	
#4	95.8	93.2	
#10	87.2	72.7	
#16	81.7	61.8	
#40	65.7	41.8	
#50	57.7	34.0	
#100	37.0	21.9	
#200	21.1	14.6	

Material Description
○ silty sand

□ silty sand

REMARKS:
○

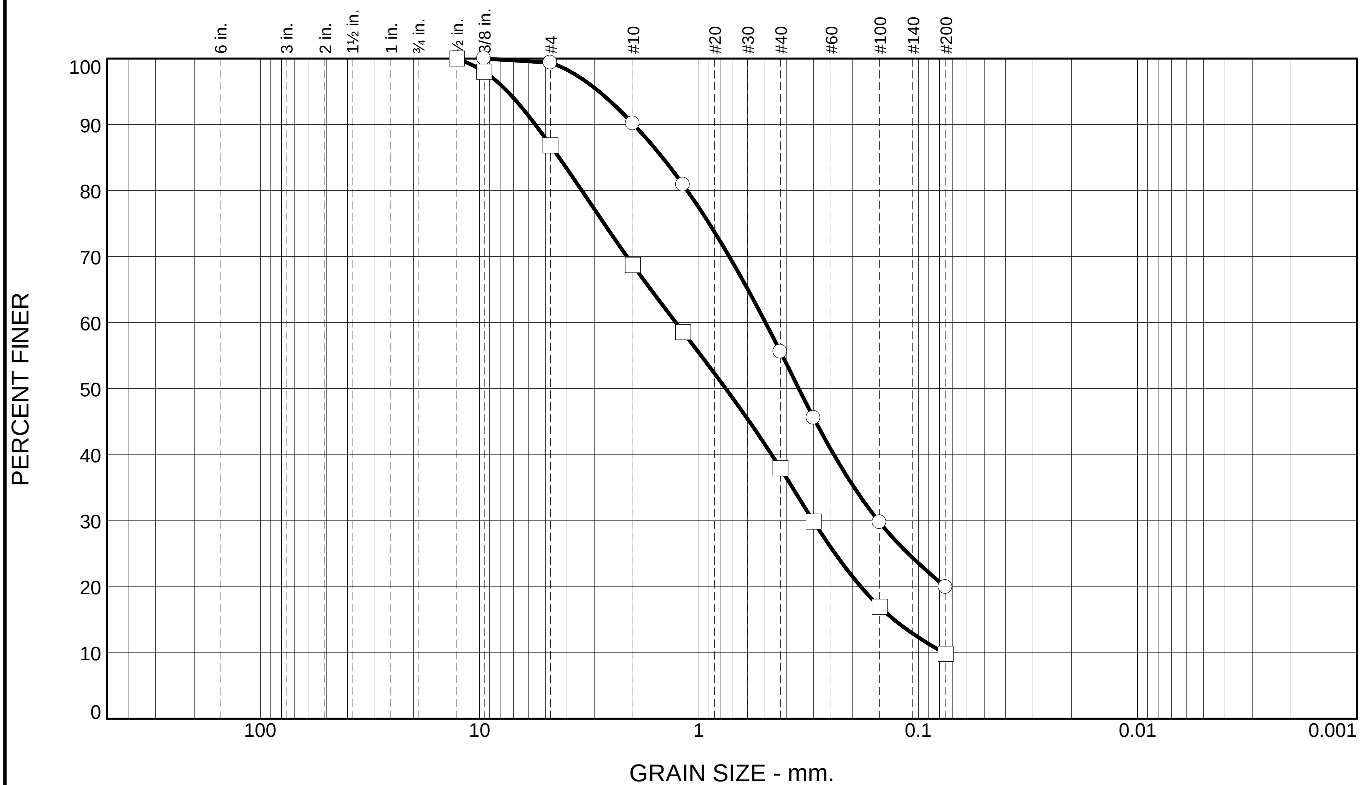
□

○ Source of Sample: MU-1 Depth: 70.0' - 71.5' Sample Number: P
□ Source of Sample: MU-1 Depth: 75.0' - 76.5' Sample Number: Q

NEVADA DEPARTMENT OF TRANSPORTATION

Client: C. Callaghan
Project: B-474 Replacement Muller Lane
Project No.: EA 73800 Figure

Particle Size Distribution Report



	+3"	% GRAVEL	% SAND	% SILT	% CLAY	USCS	AASHTO	PL	LL
○	0.0	0.6	79.5	19.9		SM	A-2-4(0)	NP	25
□	0.0	13.2	77.0	9.8		SP-SM	A-1-b	NP	19

SIEVE inches size	PERCENT FINER ○	PERCENT FINER □	
1/2"		100.0	
3/8"	100.0	98.0	
	GRAIN SIZE		
D_{60}	0.4969	1.2735	
D_{30}	0.1522	0.3018	
D_{10}		0.0766	
	COEFFICIENTS		
C_c		0.93	
C_u		16.62	

SIEVE number size	PERCENT FINER ○	PERCENT FINER □	
#4	99.4	86.8	
#10	90.1	68.7	
#16	80.9	58.6	
#40	55.6	37.9	
#50	45.6	29.9	
#100	29.7	16.9	
#200	19.9	9.8	

Material Description

○ silty sand

□ poorly graded sand with silt

REMARKS:

○

□

○ Source of Sample: MU-1 Depth: 80.0' - 81.5' Sample Number: R

□ Source of Sample: MU-1 Depth: 85.0' - 86.5' Sample Number: S

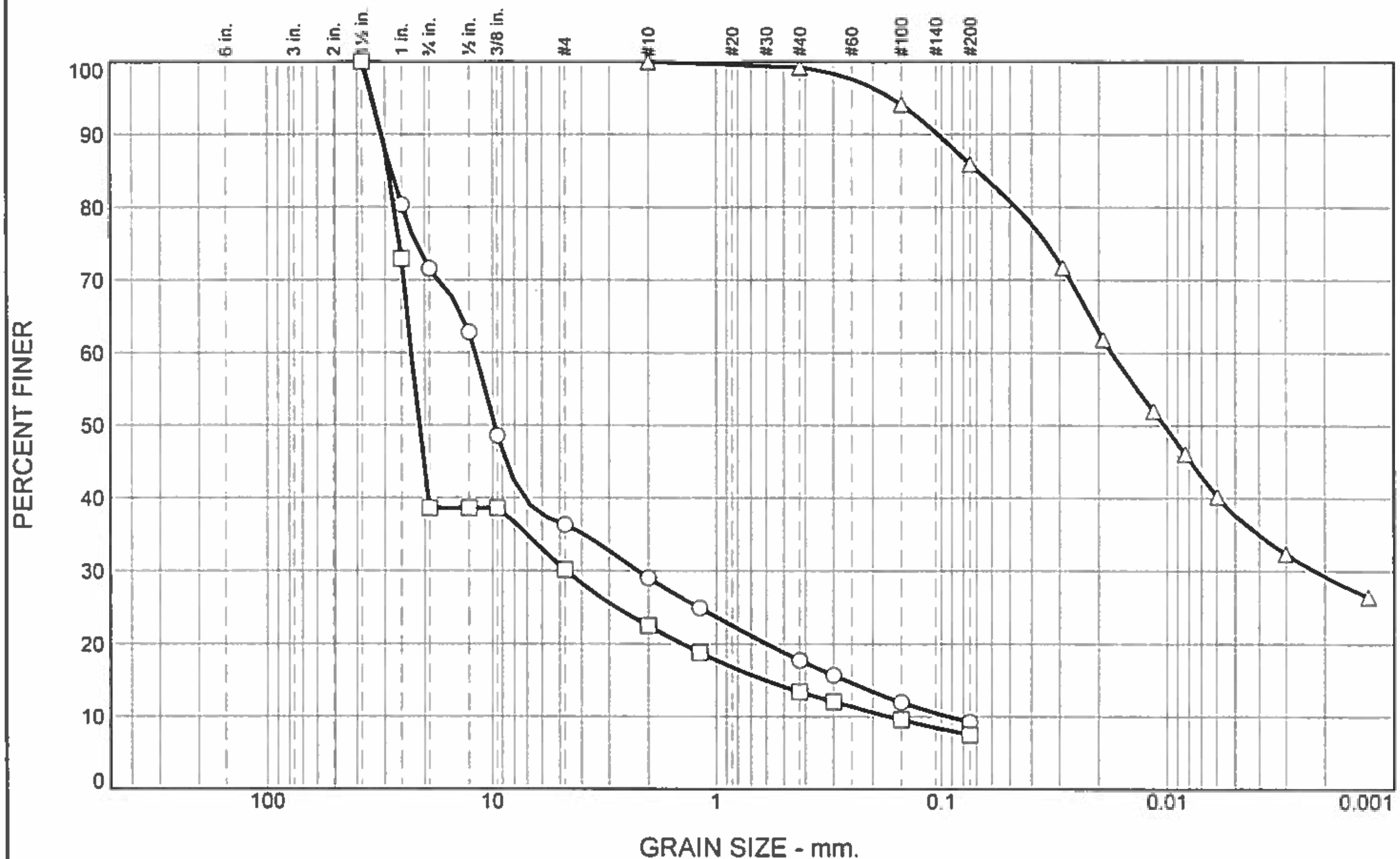
NEVADA DEPARTMENT OF TRANSPORTATION

Client: C. Callaghan

Project: B-474 Replacement Muller Lane

Project No.: EA 73800 Figure

Particle Size Distribution Report



	+3"	% GRAVEL	% SAND	% SILT	% CLAY	USCS	AASHTO	PL	LL
○	0.0	63.7	27.0	9.3					
□	0.0	69.9	22.6	7.5					
△	0.0	0.0	14.1	56.6	29.3	CL	A-7-6(23)	22	47

SIEVE inches size	PERCENT FINER ○	□	△
1-1/2"	100.0	100.0	
1"	80.4	72.9	
3/4"	71.6	38.6	
1/2"	62.8	38.6	
3/8"	48.5	38.6	
	GRAIN SIZE		
D_{60}	11.9137	22.9560	0.0176
D_{30}	2.2235	4.6946	0.0022
D_{10}	0.0925	0.1694	
	COEFFICIENTS		
C_c	4.49	5.67	
C_u	128.84	135.49	

SIEVE number size	PERCENT FINER ○	□	△
#4	36.3	30.1	
#10	29.1	22.5	100.0
#16	24.9	18.8	
#40	17.7	13.4	99.2
#50	15.7	12.1	
#100	12.0	9.6	94.1
#200	9.3	7.5	85.9

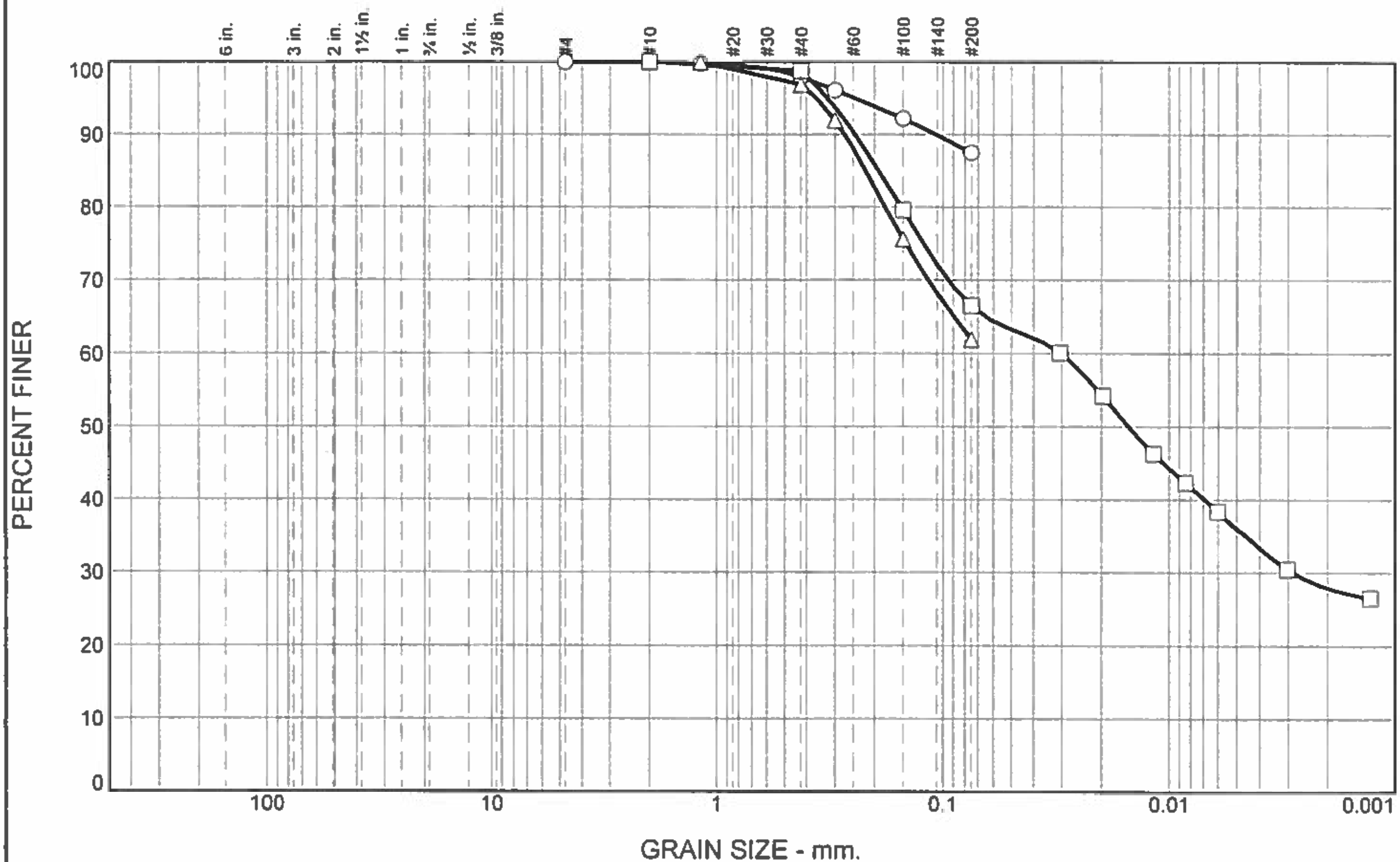
Material Description
○
□
△ lean clay

REMARKS:
○
□
△

○ Source of Sample: MU-2 Depth: 3.0' Sample Number: A
□ Source of Sample: MU-2 Depth: 5.0' Sample Number: B
△ Source of Sample: MU-2 Depth: 8.5' Sample Number: C1

NEVADA DEPARTMENT OF TRANSPORTATION	Client: C. Callaghan Project: Muller Lane B-474 Project No.: FL-4-14 Figure

Particle Size Distribution Report



	+3"	% GRAVEL	% SAND	% SILT	% CLAY	USCS	AASHTO	PL	LL
○	0.0	0.0	12.5	87.5		CH	A-7-6(25)	24	50
□	0.0	0.0	33.5	38.6	27.9	CL	A-7-6(15)	20	45
△	0.0	0.0	38.1	61.9		CL	A-6(9)	21	39

SIEVE inches size	PERCENT FINER ○	□	△
	GRAIN SIZE		
D_{60}		0.0303	
D_{30}		0.0029	
D_{10}			
	COEFFICIENTS		
C_c			
C_u			

SIEVE number size	PERCENT FINER ○	□	△
#4	100.0		
#10	100.0	100.0	
#16	99.8		100.0
#40	97.9	98.8	96.9
#50	96.1		91.9
#100	92.2	79.6	75.6
#200	87.5	66.5	61.9

Material Description
○ fat clay
□ sandy lean clay
△ sandy lean clay

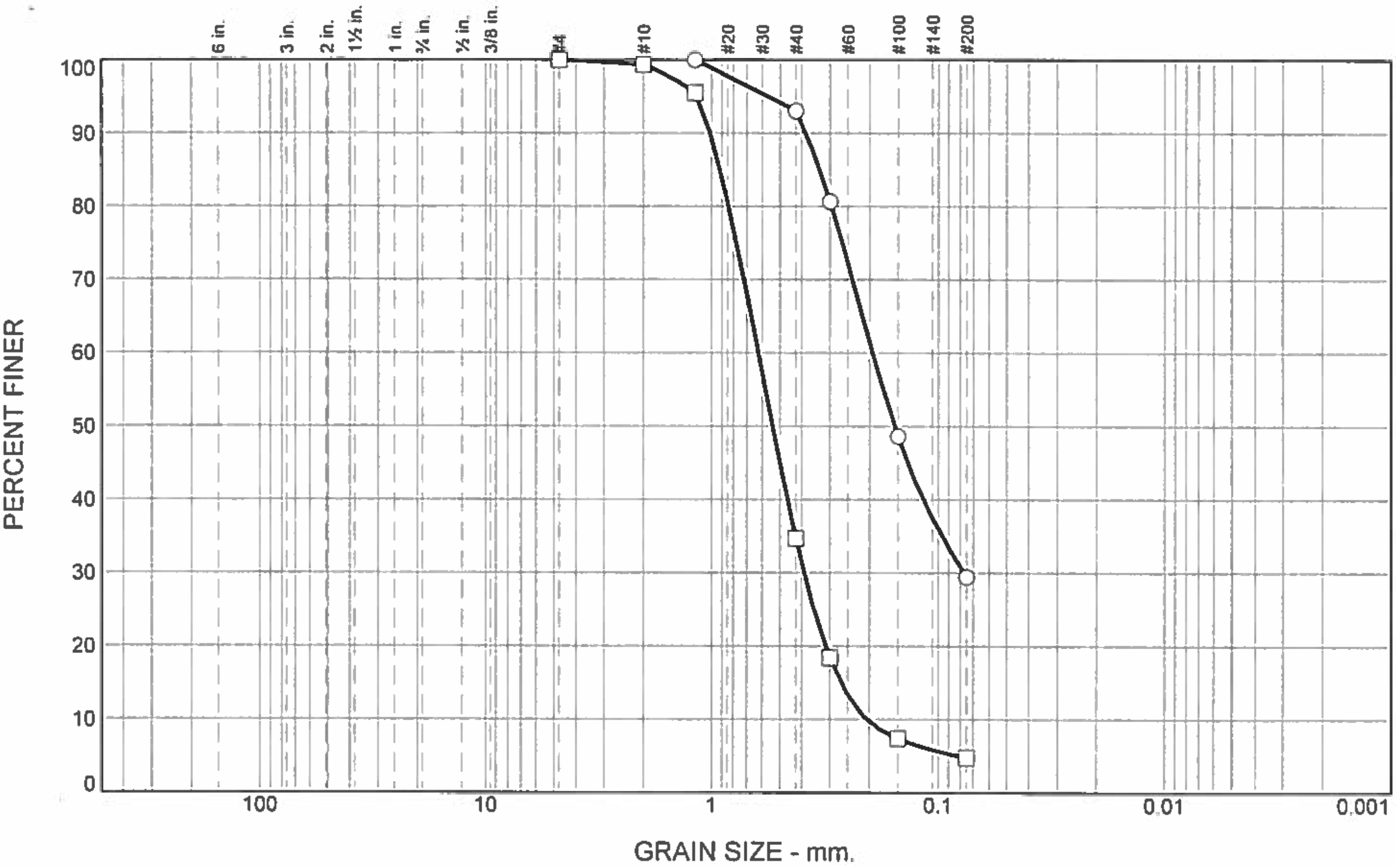
REMARKS:
○
□
△

○ Source of Sample: MU-2 Depth: 9.0' Sample Number: C2
□ Source of Sample: MU-2 Depth: 11.5' Sample Number: D1
△ Source of Sample: MU-2 Depth: 12.0' Sample Number: D2

NEVADA DEPARTMENT OF TRANSPORTATION

Client: C. Callaghan
Project: Muller Lane B-474
Project No.: FL-4-14
Figure

Particle Size Distribution Report



	+3"	% GRAVEL	% SAND	% SILT	% CLAY	USCS	AASHTO	PL	LL
○	0.0	0.0	70.5	29.5		SM	A-2-4(0)	24	26
□	0.0	0.0	95.2	4.8		SP	A-1-b	NP	21

SIEVE inches size	PERCENT FINER ○	□	

	GRAIN SIZE ○	□	
D_{60}	0.1942	0.6218	
D_{30}	0.0769	0.3907	
D_{10}		0.2051	

	COEFFICIENTS ○	□	
C_c		1.20	
C_u		3.03	

SIEVE number size	PERCENT FINER ○	□	
#4		100.0	
#10		99.3	
#16	100.0	95.5	
#40	93.0	34.7	
#50	80.7	18.4	
#100	48.6	7.4	
#200	29.5	4.8	

Material Description

○ silty sand

□ poorly graded sand

REMARKS:

○

□

○ Source of Sample: MU-2 Depth: 15.0' Sample Number: E

□ Source of Sample: MU-2 Depth: 18.0' Sample Number: F2

NEVADA DEPARTMENT OF TRANSPORTATION

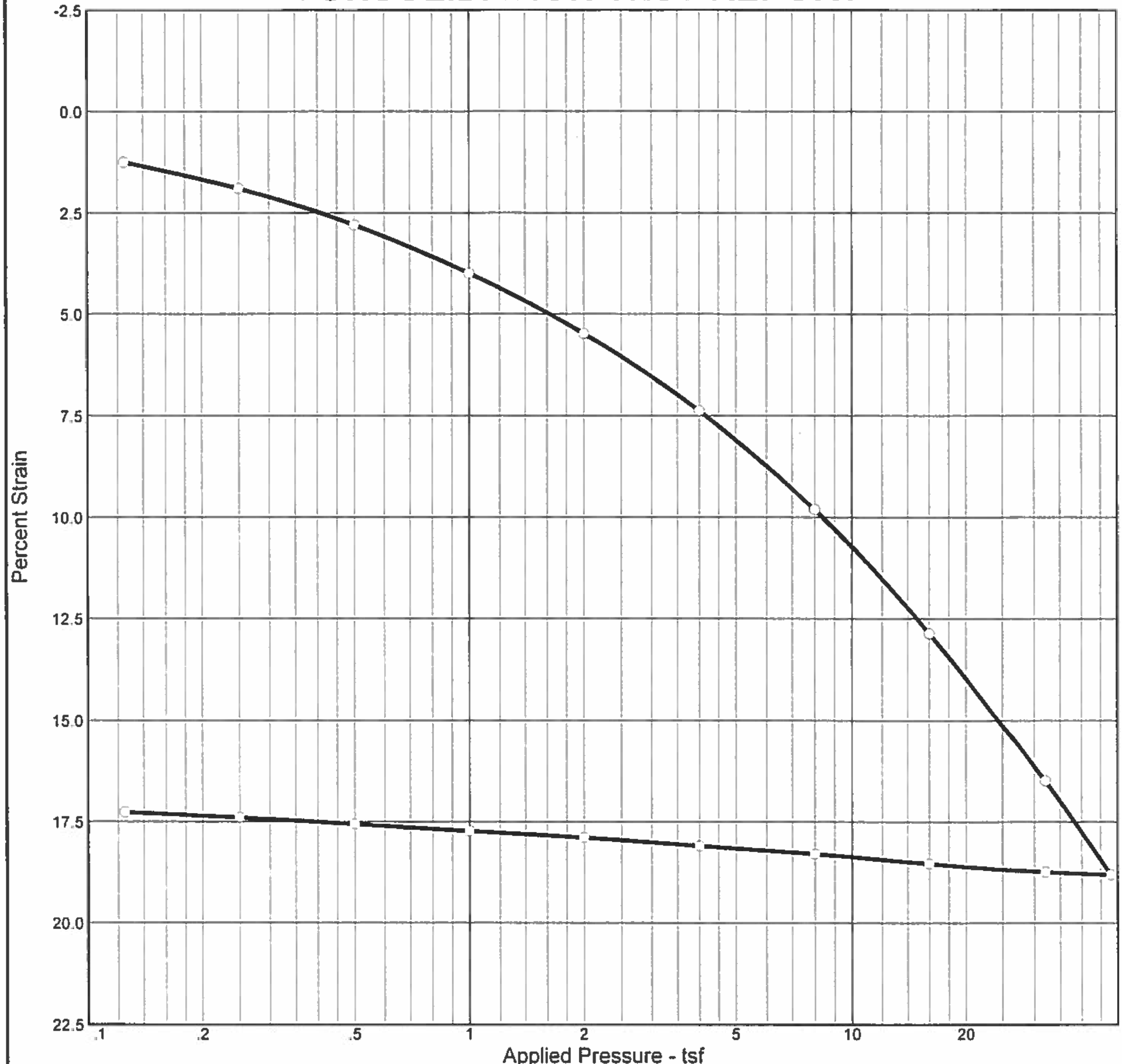
Client: C. Callaghan

Project: Muller Lane B-474

Project No.: FL-4-14

Figure

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (tsf)	P_c (tsf)	C_c	C_r	Initial Void Ratio
Saturation	Moisture									
101.5 %	35.1 %	87.8	34	8	2.741	0.70	11.63	0.26	0.01	0.948

MATERIAL DESCRIPTION	USCS	AASHTO
silt with sand	ML	A-4(7)

Project No. FL-4-14 **Client:** C. Callaghan

Project: Muller Lane B-474

Source: MU-1 **Sample No.:** C a **Elev./Depth:** 14.0'

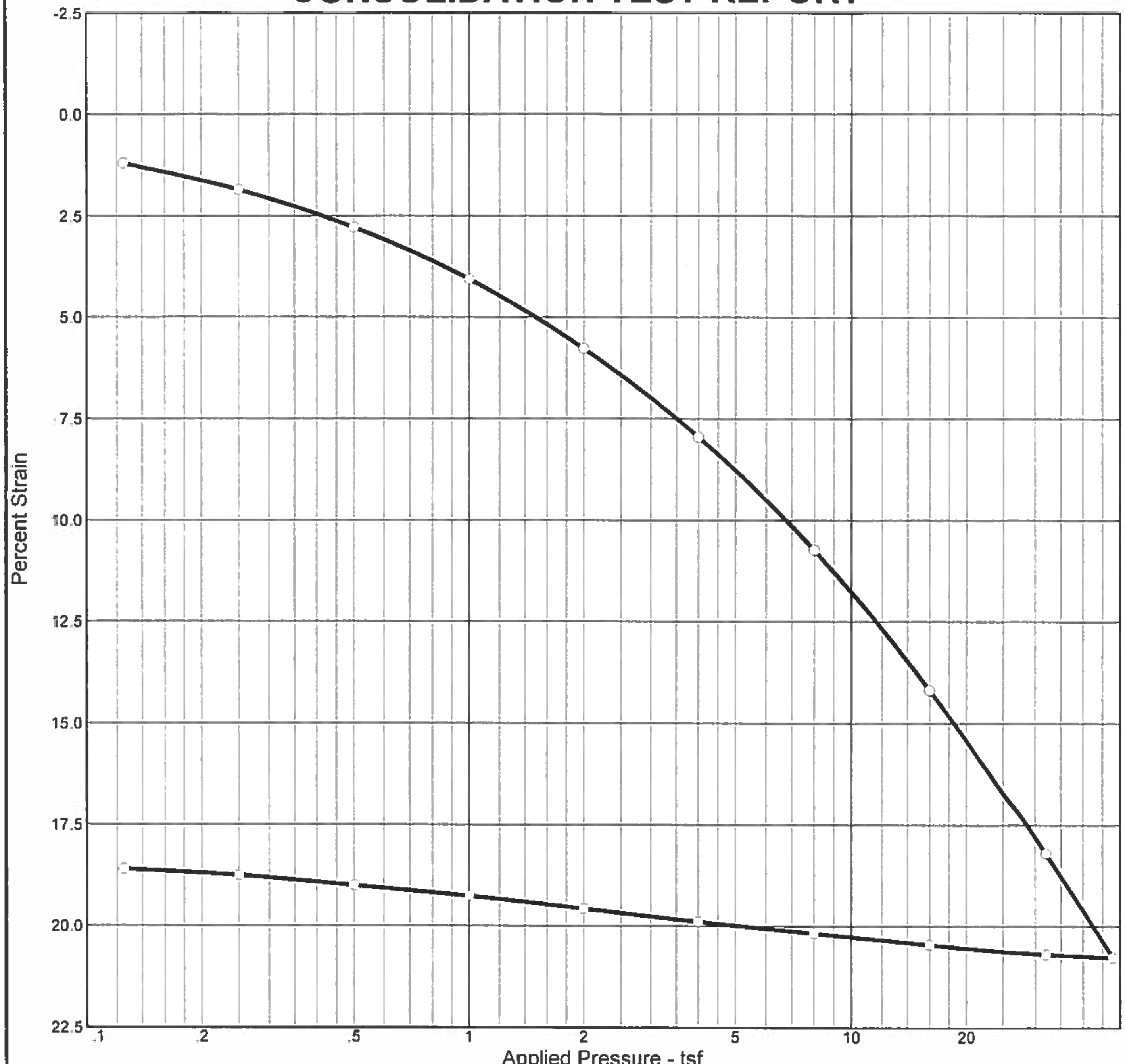
CONSOLIDATION TEST REPORT

NEVADA DEPARTMENT OF TRANSPORTATION

Remarks:

Figure

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (tsf)	P_c (tsf)	C_c	C_r	Initial Void Ratio
Saturation	Moisture									
101.9 %	37.0 %	85.7	34	8	2.741	0.70	7.82	0.29	0.02	0.996

MATERIAL DESCRIPTION	USCS	AASHTO
silt with sand	ML	A-4(7)

Project No. FL-4-14 **Client:** C. Callaghan

Project: Muller Lane B-474

Source: MU-1 **Sample No.:** C b **Elev./Depth:** 14.0'

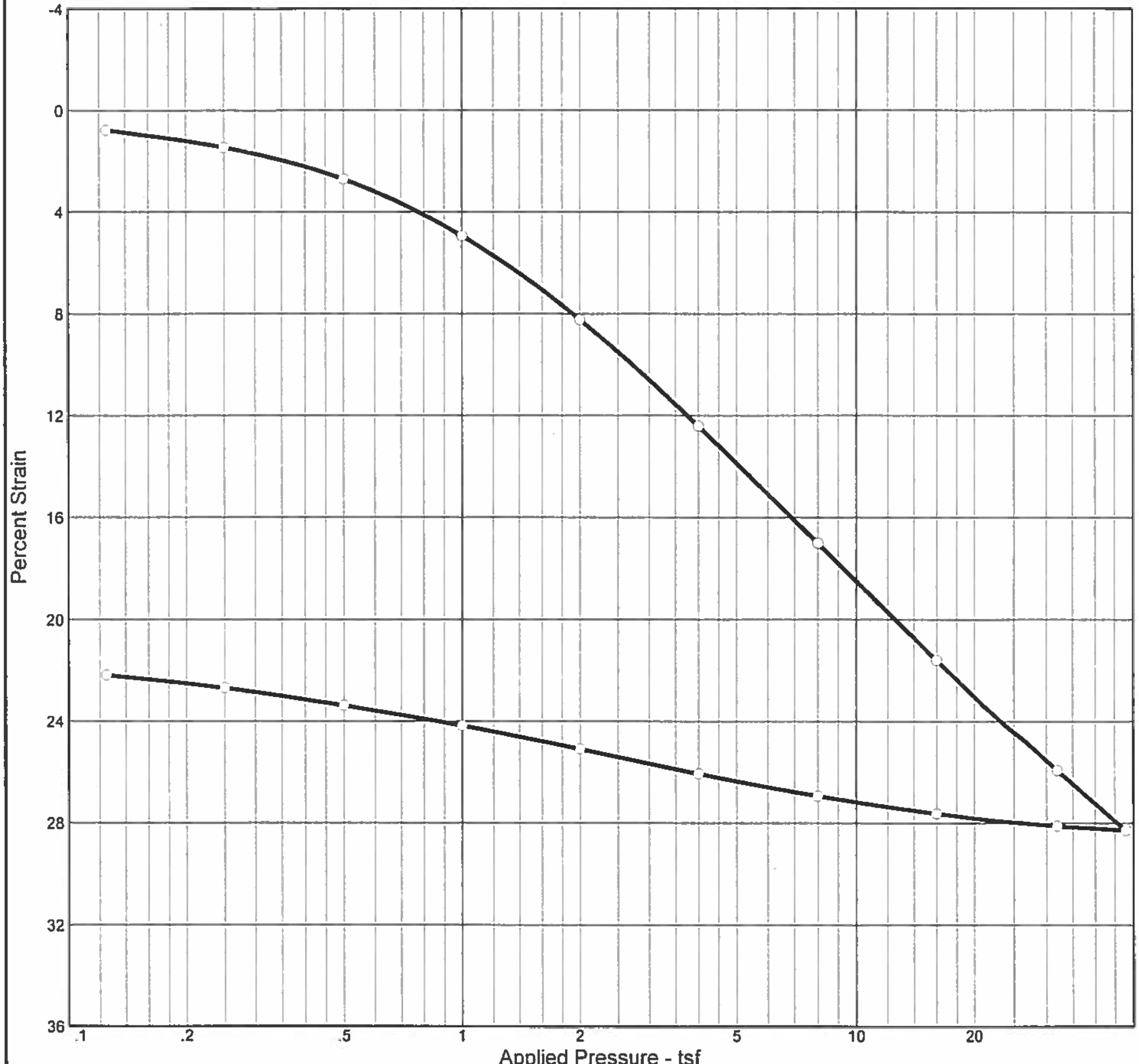
CONSOLIDATION TEST REPORT

NEVADA DEPARTMENT OF TRANSPORTATION

Remarks:

Figure

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (tsf)	P_c (tsf)	C_c	C_r	Initial Void Ratio
Saturation	Moisture									
97.3 %	41.7 %	78.7	50	26	2.746	0.50	0.61	0.29	0.05	1.177

MATERIAL DESCRIPTION	USCS	AASHTO
fat clay	CH	A-7-6(25)

Project No. FL-4-14 **Client:** C. Callaghan

Project: Muller Lane B-474

Source: MU-2 **Sample No.:** C2 **Elev./Depth:** 9.0'

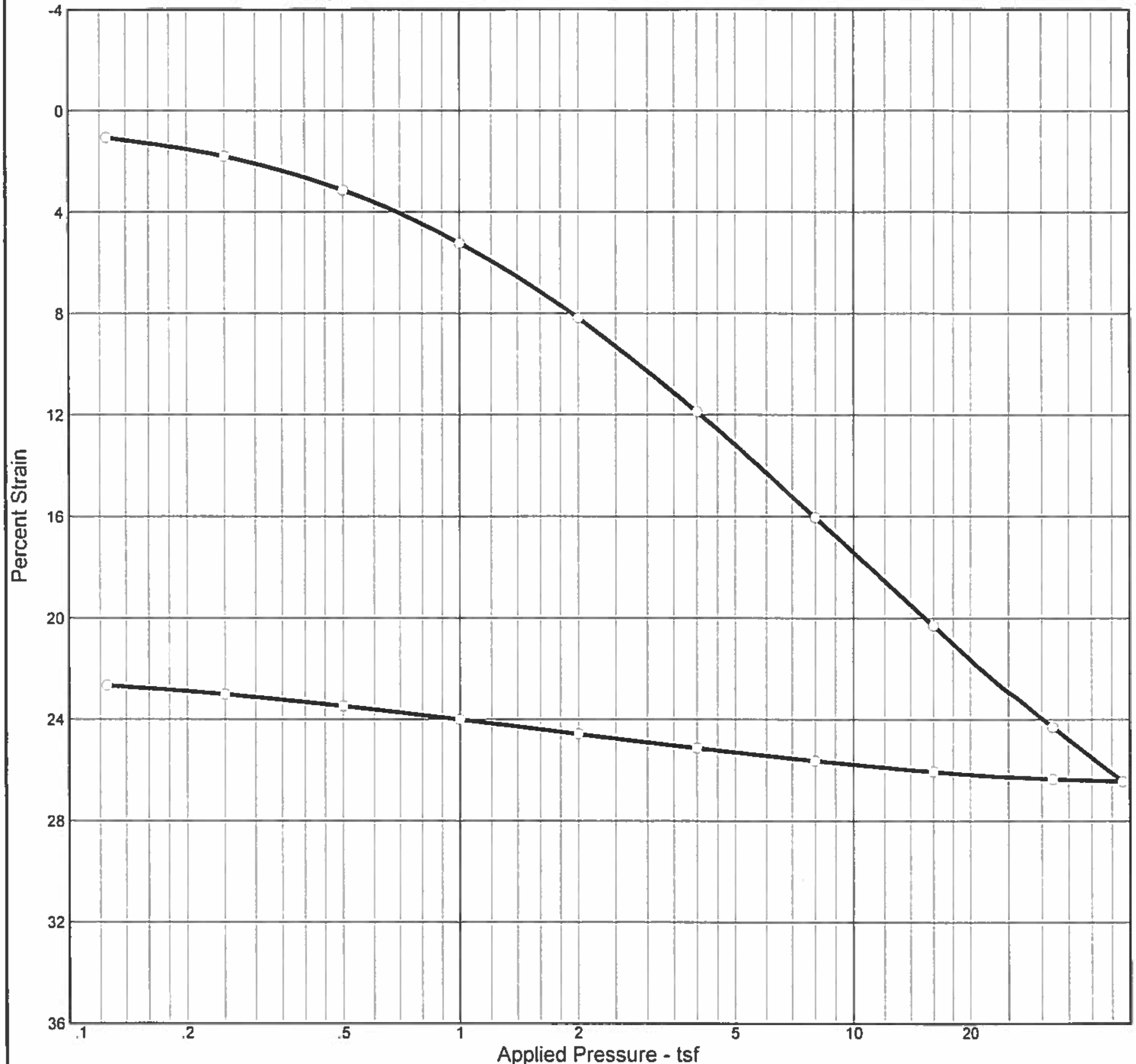
Remarks:

CONSOLIDATION TEST REPORT

NEVADA DEPARTMENT OF TRANSPORTATION

Figure

CONSOLIDATION TEST REPORT



Natural Saturation	Natural Moisture	Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (tsf)	P_c (tsf)	C_c	C_r	Initial Void Ratio
98.6 %	35.1 %	86.8	39	18	2.748	0.60	0.79	0.24	0.03	0.977

MATERIAL DESCRIPTION	USCS	AASHTO
sandy lean clay	CL	A-6(9)

Project No. FL-4-14 **Client:** C. Callaghan

Project: Muller Lane B-474

Source: MU-2 **Sample No.:** D2 a **Elev./Depth:** 12.0'

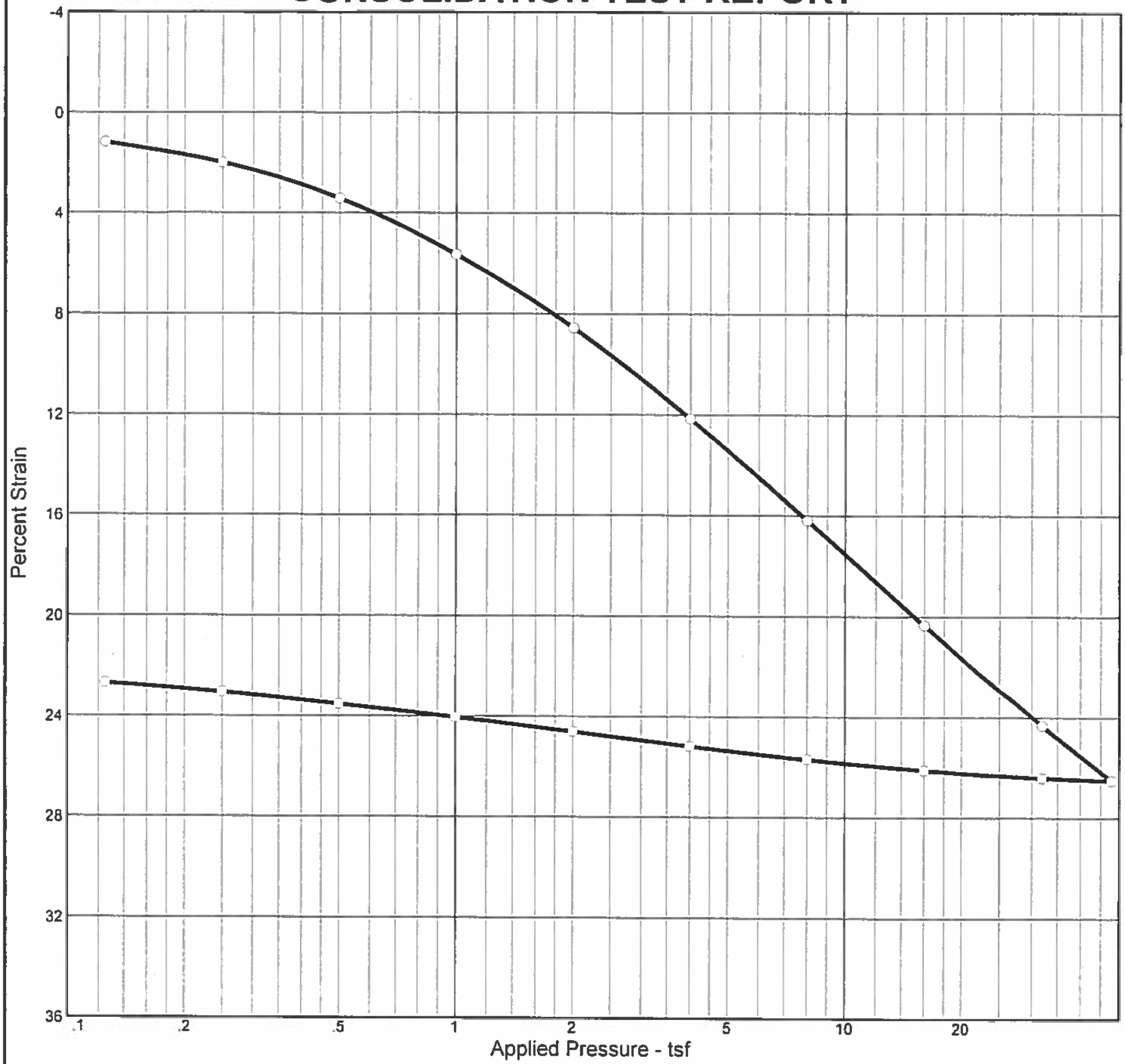
CONSOLIDATION TEST REPORT

NEVADA DEPARTMENT OF TRANSPORTATION

Remarks:

Figure

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (tsf)	P_c (tsf)	C_c	C_r	Initial Void Ratio
Saturation	Moisture									
100.7 %	35.5 %	87.1	39	18	2.748	0.60	0.74	0.24	0.03	0.969

MATERIAL DESCRIPTION	USCS	AASHTO
sandy lean clay	CL	A-6(9)

Project No. FL-4-14 **Client:** C. Callaghan

Project: Muller Lane B-474

Source: MU-2 **Sample No.:** D2 b **Elev./Depth:** 12.0'

Remarks:

CONSOLIDATION TEST REPORT

NEVADA DEPARTMENT OF TRANSPORTATION

Figure

NEVADA DEPARTMENT OF TRANSPORTATION
GEOTECHNICAL SECTION

CHEMICAL ANALYSIS

E.A. No.: 73800

Project: Muller Lane B-474

Sample ID	Date Tested	Chlorides ppm AASHTO T 291 A	Sulfates ppm AASHTO T 290 B	pH AASHTO T 289	Resistivity ohm - cm AASHTO T 288
MU - 2 F1	4/22/14	10	22	5.6	5150*

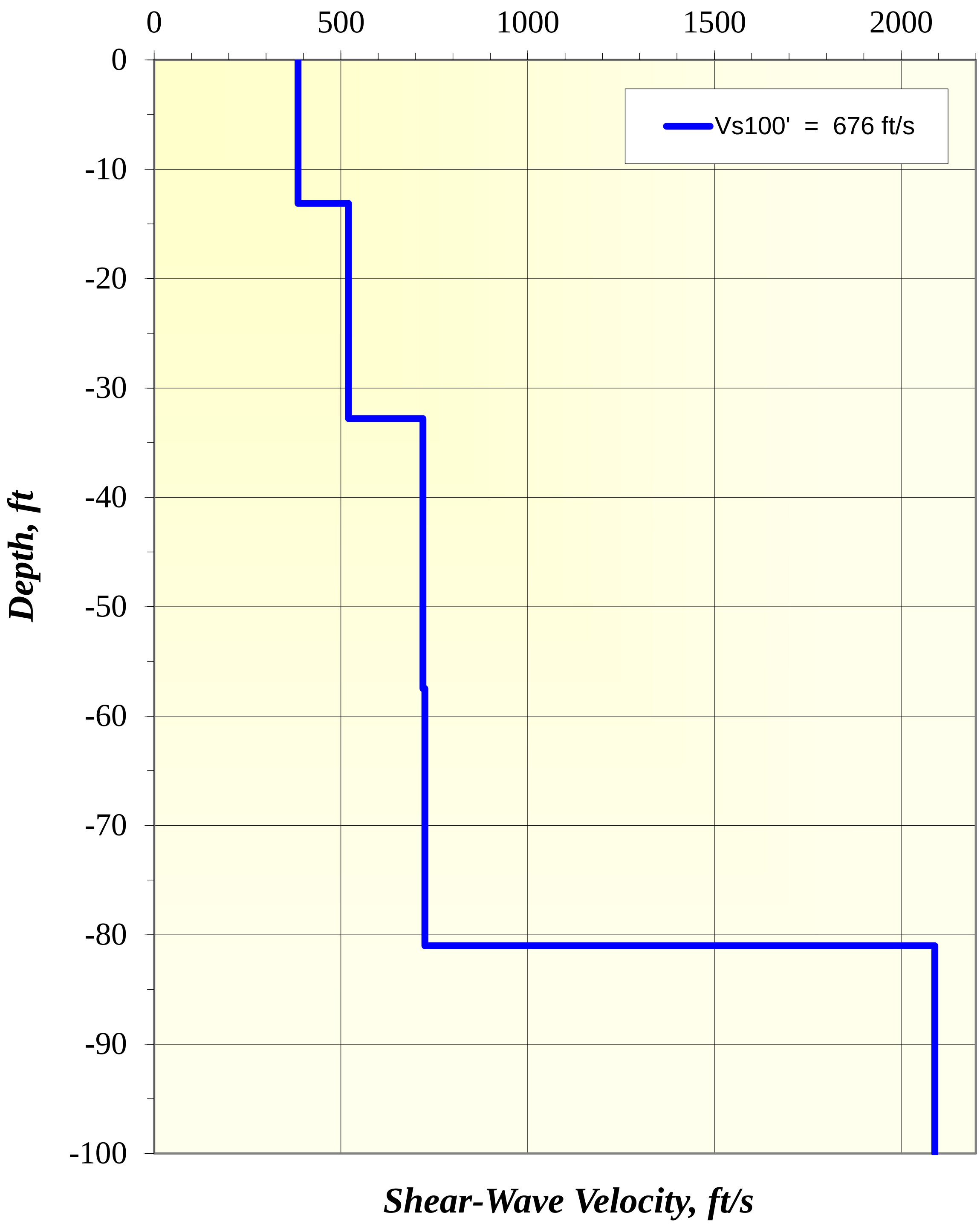
* Deviated from AASHTO T 288 by using a small 4 pin soil box.

<u>APPENDIX D</u>

ReMi Results

Shear Wave Velocity Profile

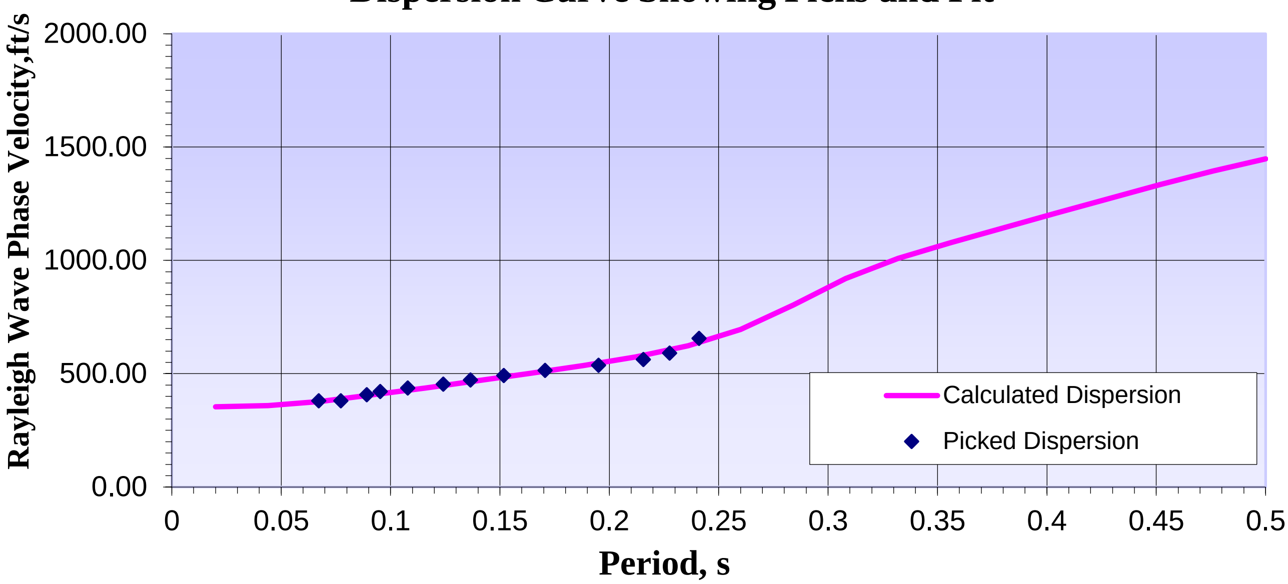
Muller B-474



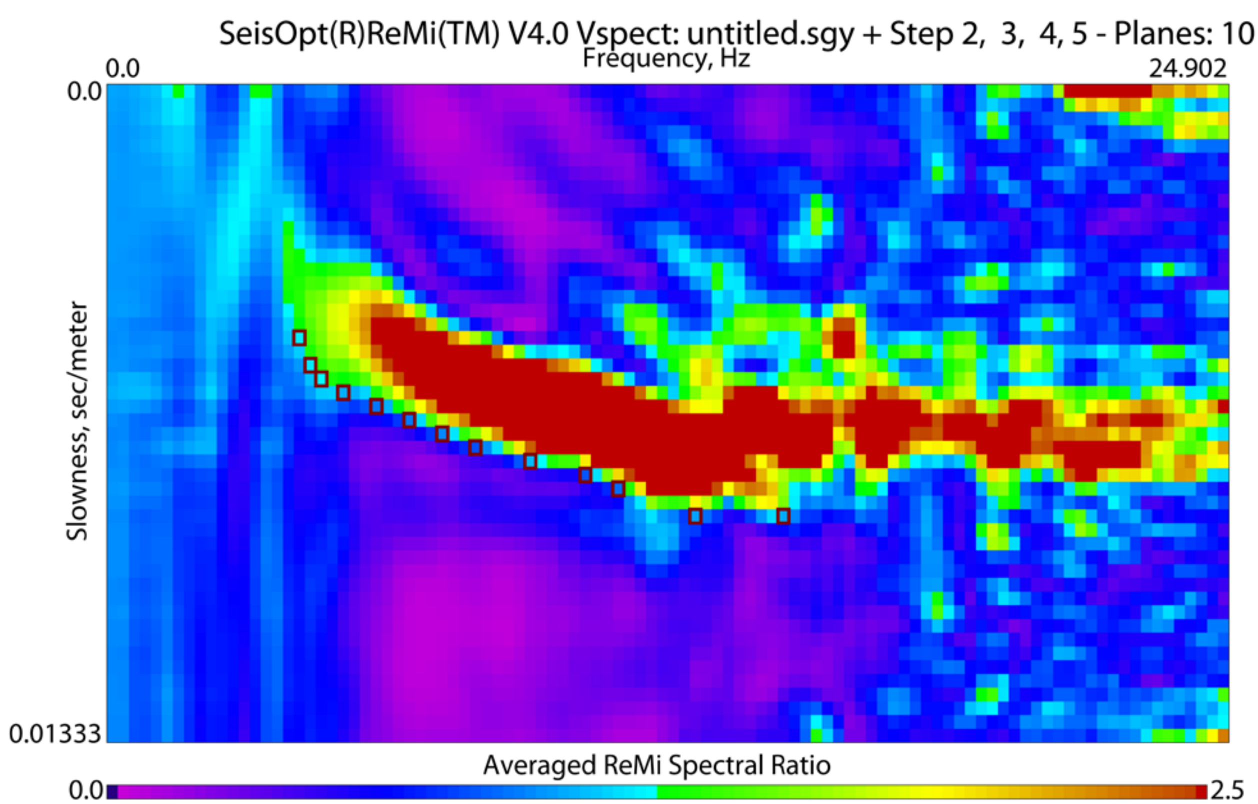
Dispersion Curve and Slowness Spectrum

Muller B-474

Dispersion Curve Showing Picks and Fit



p-f Image with Dispersion Modeling Picks

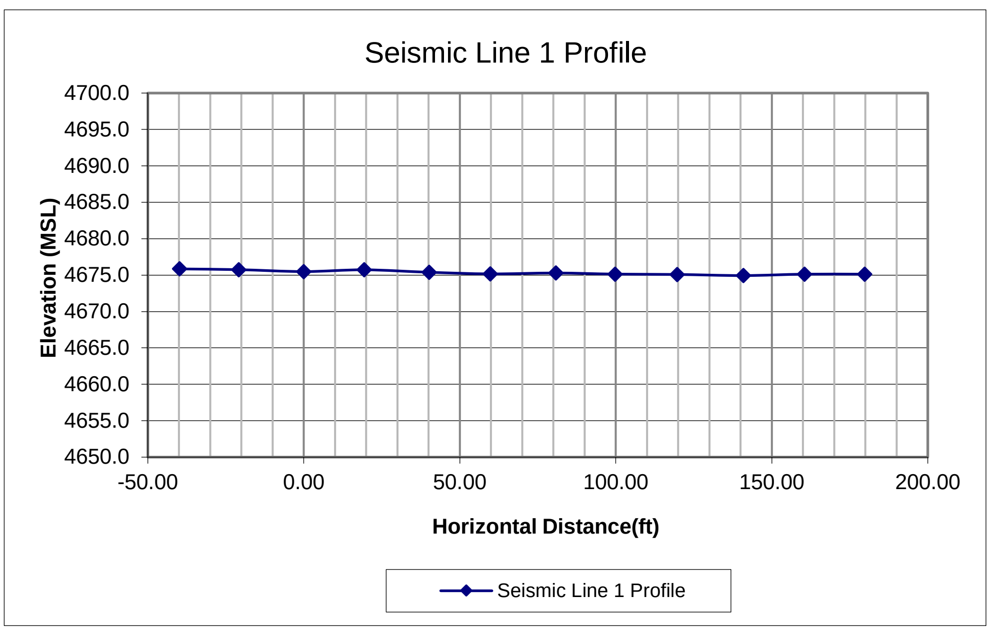


Geologic Investigation at Bridge B-474, SR757
Nevada Department of Transportation Geotechnical Engineering (028)
Seismic Refraction Survey
Participants: C. Callaghan, S. Jensen, K. Conrad, D. Geiger
Dates: 12/4/2017

Seismic Line 1 - Northeast

Total Station Location:	Line 1, Geophone 3
Reference Elevation Location:	USC&GS BM "T 323 1951"
Reference Elevation [ft]:	4676.3
Rod & Prism Height [ft]:	5.95
Total Station [TS] Height of Instrument [ft]:	4.04
Elevation Reading [ft]:	2.72
Benchmark Elevation [ft]:	4675.5
Groundwater Elevation [ft]:	4668.5

Geophone (#)	Vertical (ft) above/below TS	Horizontal(ft) dis. from TS	ΔVertical (ft)	ΔHoriz. (ft)	Elevation (MSL) (ft)
12	1.55	179.77	1.55	19.32	4675.1
11	1.55	160.45	1.55	19.51	4675.1
10	1.36	140.94	1.36	21.27	4674.9
9	1.51	119.67	1.51	219.39	4675.1
8	1.56	99.72	1.56	180.51	4675.1
7	1.71	80.79	1.71	-18.93	4675.3
6	1.59	59.80	1.59	-20.99	4675.2
5	1.81	40.14	1.81	-19.66	4675.4
4	2.15	19.38	2.15	-20.76	4675.7
3		**0.00**	**0.00**	**0.00**	**4675.5**
2	2.17	-20.88	2.17	-20.88	4675.8
1	2.27	-39.82	2.27	-18.94	4675.9



Geologic Investigation at Bridge B-474, SR757
Nevada Department of Transportation Geotechnical Engineering (028)
Seismic Refraction Survey
Participants: C. Callaghan, S. Jensen, K. Conrad, D. Geiger
Dates: 12/4/2017

	vert.	elevation
Borehole MU-1:	2.31	4675.89
Borehole MU-2:		
Creek (downstream bridge):	-4.88	4668.70
Borehole MU-1 Water Level:		4668.50

Seismic Refraction Details

Record numbers and plate locations for seismic refraction survey at each line.

Special Notes:

ReMi Details

Record numbers for ReMi survey.

Special Notes: drill rig running near GPs 1 & 2; Occasional traffic on road south of line during survey; stream @ west end of line, water level @ 4668.7' elevation; Geophone #7 appears to have malfunctioning-replaced geophone, reset and buried geophone with no effect-possible cable issue?

Seismic Line 1	
Plate Location	Record #'s
10' past Geophone #12	
Geophone #9-10	
Geophone #6-7	
Geophone #3-4	
10' before Geophone #1	
Seismic Line 2	
10' past Geophone #12	
Geophone #9-10	
Geophone #6-7	
Geophone #3-4	
10' before Geophone #1	
Seismic Line 3	
10' past Geophone #12	
Geophone #9-10	
Geophone #6-7	
Geophone #3-4	
10' before Geophone #1	
Seismic Line 4	
Plate Location	Record #'s
10' past Geophone #12	
Geophone #9-10	
Geophone #6-7	
Geophone #3-4	
10' before Geophone #1	
Seismic Line 5	
Plate Location	Record #'s
10' past Geophone #12	
Geophone #9-10	
Geophone #6-7	
Geophone #3-4	
10' before Geophone #1	
Seismic Line 6	
Plate Location	Record #'s
10' past Geophone #12	
Geophone #9-10	
Geophone #6-7	
Geophone #3-4	
10' before Geophone #1	

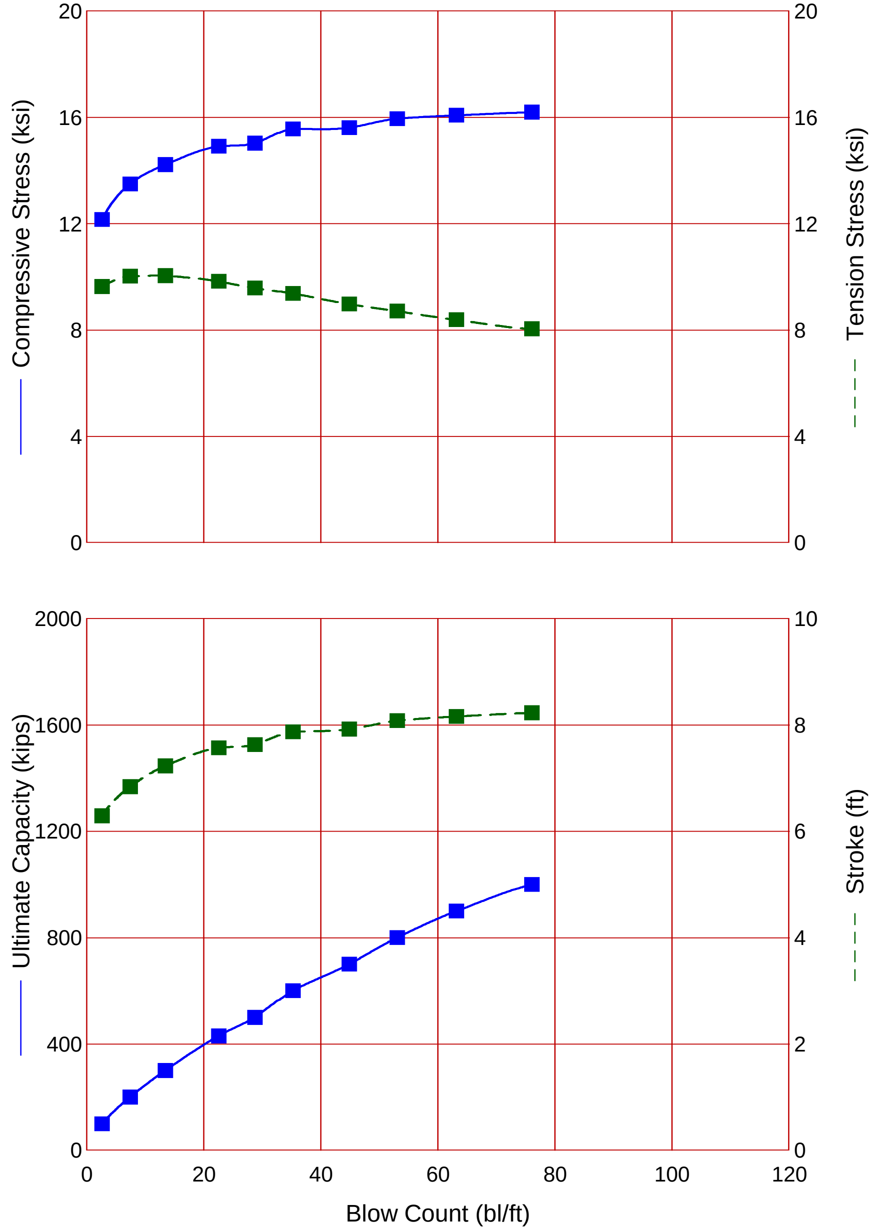
Seismic Line 1	
ReMi	Record #'s
#1	1 to 31
Seismic Line 2	
ReMi	
Seismic Line 3	
ReMi	

APPENDIX E

GRLWEAP Results

NDOT Geotechnical
Muller final

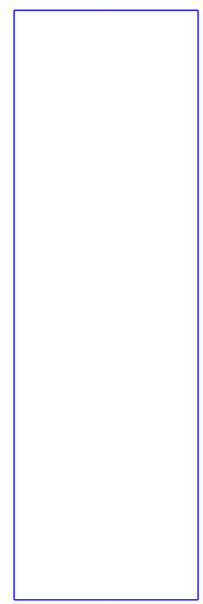
12-Dec-2017
GRLWEAP Version 2010



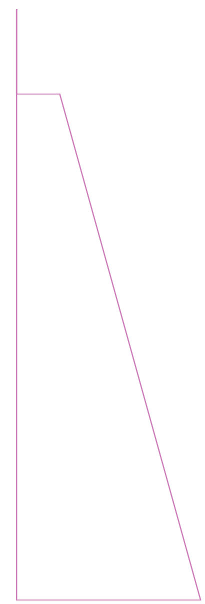
DELMAG D 46-02

Ram Weight	10.14	kips
Efficiency	0.800	
Pressure	1100 (100%)	psi
Helmet Weight	2.50	kips
Hammer Cushion	109975	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.100	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	45.50	ft
Pile Penetration	39.00	ft
Pile Top Area	254.50	in2

Pile Model



Skin Friction
Distribution



Res. Shaft = 58 %
(Proportional)

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count bl/ft	Stroke ft	Energy kips-ft
100.0	12.15	9.63	2.7	6.29	42.48
200.0	13.49	10.02	7.5	6.84	36.88
300.0	14.22	10.04	13.5	7.23	34.19
430.0	14.91	9.83	22.6	7.57	32.96
500.0	15.02	9.58	28.8	7.63	31.63
600.0	15.55	9.38	35.3	7.87	31.90
700.0	15.61	8.98	44.9	7.92	31.89
800.0	15.95	8.71	53.1	8.08	32.10
900.0	16.07	8.38	63.2	8.16	32.27
1000.0	16.19	8.04	76.1	8.23	32.56