GEOTECHNICAL INVESTIGATION

NORTH VALLEY ALIGNMENT

LEMMON VALLEY, WASHOE COUNTY, NEVADA

PREPARED FOR:

TRUCKEE MEADOWS WATER AUTHORITY

NOVEMBER 2014
FILE: 1660

CME CONSTRUCTION MATERIALS ENGINEERS, INC.
6980 Sierra Center Parkway, Suite 90
Reno, NV 89511

November 6, 2014
Project No:1660

Juan Esparza, PE
TRUCKEE MEADOWS WATER AUTHORITY
1355 Capital Boulevard
Reno, NV 89502

RE: Geotechnical Investigation
North Valley Alignment
Lemmon Valley, Washoe County, Nevada

Dear Mr. Esparza:

Construction Materials Engineers Inc. (CME) is pleased to submit the results of our geotechnical investigation report for the North Valley Alignment in Lemmon Valley, Washoe County, Nevada.

The following report includes the results of our field and laboratory investigations and presents our recommendations for the design and construction of the North Valley Alignment. We wish to thank you for the opportunity to provide our services and look forward to working with you on future endeavors.

Please feel free to call us should you have any questions or require additional information.

Sincerely,

CONSTRUCTION MATERIALS ENGINEERS, INC.

Randal A. Reynolds

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1.0 INTRODUCTION

Presented herein are the results of Construction Materials Engineers, Inc. (CME) geotechnical exploration, laboratory testing, and associated geotechnical design recommendations for the proposed North Valley Alignment in North Reno and Lemmon Valley, Washoe County, Nevada. These recommendations are based on 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:

- Determine general soil and ground water conditions pertaining to design and construction of the proposed improvements.

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

The proposed water line alignment is contained in Sections 22, 26, 27, 34 Township 21N, Range 19E, and Sections 3, 4, 9, 16 Township 20N, Range 19E, MDBM. The area covered by this report is shown on Plate A-1 (Vicinity Plan) in Appendix A.

Our study included field exploration, laboratory testing and engineering analyses to identify 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.

2.0 PROJECT DESCRIPTION

2.1 WATERLINE LOCATION

The proposed water line alignment begins at the intersection of Waterash Street and Lemmon Drive and extends south along Lemmon Drive terminating at North Virginia Street. The water line crosses several major intersections including Military Road, Sky Vista Parkway, and US 395 near the Lemmon Drive underpass. The water line has a total length of about 5.1 miles

2.2 WATERLINE DESCRIPTION

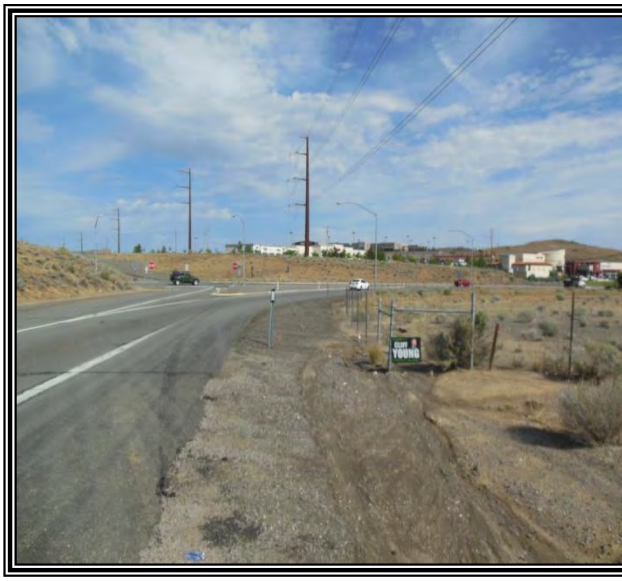
Additional information and assumptions for the water line are as follows:

- The pipe will likely be ductile iron and have a diameter of 24 inches;

- The water line will have both self-restraining joints and thrust blocks;

Looking south along west roadway shoulder for Lemmon Drive near Chickadee Drive

- The maximum depth of the water line trench is anticipated to be 10 feet;
- The majority of the water line alignment in the central to northern segments will be located in the unpaved shoulder area of Lemmon Drive. Toward the southern end of the line, south of military Road, the majority of the water line will be located below paved areas;
- Where the water line crosses existing roadways, it is assumed that the replacement pavement patch will match the existing structural section; and
- The water line in NDOT right-a-way at US 395 will require trenchless construction. The preferable alignment is located southeast of the Lemmon Drive Underpass. The trenchless construction will entail placement of a minimum 36 inch diameter steel casing.



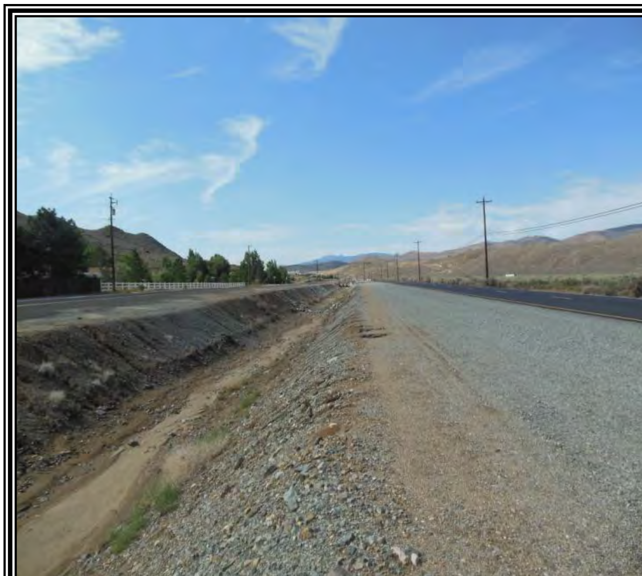
Looking northwest from the US 395 off- ramp toward Lemmon Drive.

3.0 SITE CONDITIONS

The proposed water line alignment is located in the eastern and southern portions of Lemmon Valley. Lemmon Drive is the primary roadway access through this area. Residential subdivisions are located in the eastern portions of Lemon Valley, while commercial and manufacturing businesses are located in the southern portions of Lemmon Valley.

The northern to central segments of the water line alignment are located in Washoe County jurisdiction, while the southern segments of the water line alignment are located in City of Reno jurisdiction. From North Hills Boulevard to Surge Street, Lemmon Drive represents the boundary line between the City of Reno and Washoe County.

It is understood that the majority of the water line alignment will be located in the shoulder area of Lemmon Drive between Waterash Drive and Military Road. Lemmon Drive is a two-lane roadway between Waterash Drive to about 1200 feet south of Patrician Drive. The roadway then transitions into a divided arterial roadway with two travel lanes in both the northbound and southbound directions. This divided roadway configuration extends to Military Road. The center divider area has a distance that ranges from 50 to 100 feet.

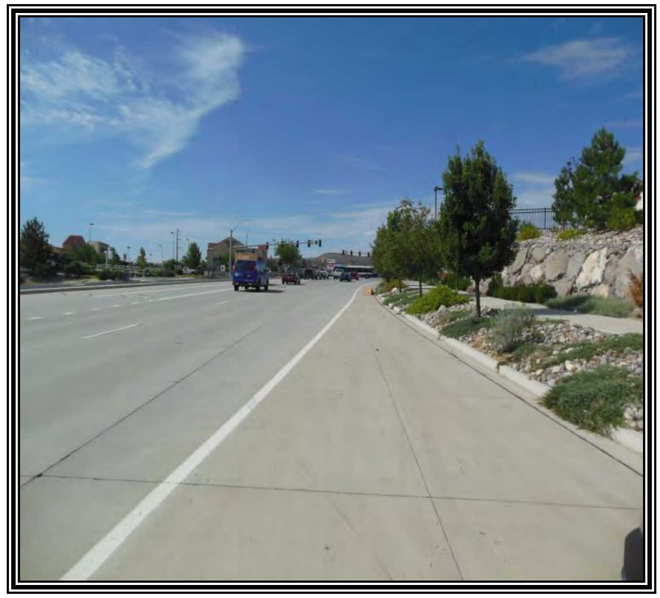


Looking south toward Military Road showing divided roadway area.

A large drainage ditch is located in the majority of this divider area. Cross street connector roads are located between the northbound and southbound travel lanes.

Beginning at the intersection with Military Road, Lemmon Drive transitions from a divided roadway to an undivided four lane roadway (2 lanes in each travel direction). This roadway configuration extends to the west side of the Lemmon Drive underpass**.** The roadway then transitions back to a two-lane roadway that extends to the end of the water line alignment at North Virginia Street.

Except for short segments of the roadway, the majority of Lemmon Drive has undeveloped shoulders. Developed shoulders consisting of curb and gutter and sidewalk are located at the intersection with Military Road; approximately between North Hills Boulevard to US 395; and in the western segment of Lemmon Drive between US 395 to North Virginia Street.



Looking southwest toward intersection with Sky Vista Parkway

The roadway is paved accept for isolated areas located at the intersections with Military Road, Sky Vista Parkway, and US 395 where the roadway surface is Portland Cement Concrete (PCC).

In general, the roadway gradient is relatively flat between Waterash Drive to Patrician Drive. Starting at about Patrician Drive the roadway begins a gentle ascent at a gradient of less than 1 percent to Military Road. The total elevation rise in this segment of roadway is about 70 feet. Starting at about Military Road, the roadway ascends with a variable gradient that ranges from about 1 to 6 percent to North Virginia Street. The total elevation difference between Military Road to North Virginia Street is about 210 feet.

Looking west along south side of Lemmon Drive near intersection with North Virginia Street

4.0 EXPLORATION

4.1 DRILLING

The project site was explored in July and August 2013 by drilling a series of 27 test borings. The borings were drilled using a truck-mounted CME 55 soil sampling drill rig and hollow-stem flight augers. The maximum depth of exploration was 10 feet below the existing ground surface (bgs).

The native soils were sampled in-place every 2 to 5 feet using a standard 2-inch OD split-spoon sampler driven by a standard 140-pound drive hammer with a 30-inch stroke. The number of blows to drive the sampler the final 12 inches of an 18-inch penetration into undisturbed soil (Standard Penetration Test (SPT) - ASTM D 1586) is an indication of the soil's relative density (granular soils) or consistency (fine-grained soils).

A 3-inch O.D. split-spoon sampler was also used to sample soils containing gravel or where approximate in-place densities of subsurface materials were required. Sampling methods used were similar to the SPT but also include the use of 2½-inch diameter, 6-inch-long brass sampling tubes placed inside the split-spoon sampler. Because of the larger diameter of the sampler, blow counts are typically higher than those obtained with the SPT and should not be directly equated to SPT blow counts. The logs indicate the type of sampler used for each sample.

Pocket Penetrometer testing was completed on fine-grained soil samples. This testing provides a measure of the soil's in-situ shear strength in tons/ft^2. Cohesive strength is approximately 50 percent of the in-situ measured shear strength.

Because of the small diameter of the sampler, the boring logs may not adequately represent the actual quantity or presence of gravels or cobbles.

4.2 EXPLORATORY TEST PITS

Field exploration consisted of excavating 3 test pits with a rubber tired backhoe. One test pit was excavated near the intersection with North Virginia Street; another test pit was excavated in the roadway shoulder area along the US 395 on-ramp from Lemmon Drive; and the remaining test pit was excavated near Boring B-12. Boring B-12 met refusal at a shallow depth and the test pit was excavated to determine soil conditions below the boring depth. The maximum depth of exploration was 10 feet below the existing ground surface (bgs).

Our geotechnical personnel logged material encountered during exploration. Representative soil samples were returned to our laboratory for testing.

Looking Northwest US 395 on-ramp near Lemmon Drive showing existing cut slope

4.3 EXPLORATION LOCATIONS AND GROUND ELEVATIONS

Exploration locations were determined by referencing to existing improvements and are presented on the Field Exploration Location Maps: Plate A-2 in Appendix A. Ground surface elevations were determined by linear interpolation between ground contour line elevations presented on an existing topographic map (Washoe County GIS) and should be considered approximate.

4.4 MATERIAL CLASSIFICATION

CME personnel examined and classified all soils in the field in general accordance with ASTM D 2488. During drilling, representative bulk samples were placed in sealed plastic bags and returned to our Reno, Nevada laboratory for testing. After the completion of laboratory testing as described in the **Laboratory Testing** section, the boring logs were checked and corrected in accordance with ASTM 2487 (Unified Soil Classification System). Logs of the borings and test pits are presented as Plate A-3 and a USCS chart has been included as Plate A-4 in Appendix A.

4.5 PAVEMENT CORING

Pavement coring was completed at four locations: Nectar Street, Patrician Drive, Surge Street, and Hydraulic Street. The existing structural section encountered consisted of asphaltic pavement overlying a decomposed granite sand that classified as a poorly graded sand with silt **(SP-SM)**. The asphalt and base course thicknesses are presented in Table 1.

Table 1 – Existing Structural Section Thicknesses

Street	Base Course (inches)	Asphalt (inches)
Nectar Street	NE	8½ (4 separate pavement layers)
Patrician Drive	NE	5½ (3 separate pavement layers)
Surge Street	NE	7 (4 separate pavement layers)
Hydraulic Street	NE	7½ (2 separate pavement layers)

Note:
1) NE: Not encountered.

5.0 RESEARCH

Six different geotechnical investigations completed within the vicinity of the proposed water line route were researched. Geotechnical profiles encountered and laboratory test results from these geotechnical investigations were used to supplement the field exploration and laboratory testing results from this investigation. Exploration locations are presented in Appendix A, while boring logs and laboratory test results are presented in Appendix C. A listing of these investigations and locations to the proposed water line alignment are presented below:

1) **Reno-Stead Waterline - Phase 1**

   This geotechnical investigation was for a 5.75 mile long 24-inch ductile iron water line that began at the intersection of Waterash Street and Lemmon Drive and extended in a westerly direction along Lemmon Drive through the northern portions of the Reno-Stead Airport. The geotechnical soil boring information and laboratory test results from the beginning portion of this water line were incorporated into this report. Soil borings were drilled to depths of 10 feet below the existing ground surface.

2) **Lemmon Valley Estates No. 1**

   This soils investigation was for a residential subdivision located between Patrician Drive to about 400 feet north of Surge Street along the east side of Lemmon Drive. A total of 9 soil borings with a spacing of 500 feet were drilled adjacent to Lemmon Drive. Soil borings were drilled to depths of 10 feet below the existing ground surface.

3) **Lemmon Valley Estates No. 4**

   This soils investigation was for a residential subdivision located between Patrician Drive to Palace Drive along the east side of Lemmon Drive. A total of 3 soil borings with a spacing of 500 feet were drilled adjacent to Lemmon Drive. Soil borings were drilled to depths of 10 feet below the existing ground surface.

4) **Lemmon Drive and Military Road Intersection Improvements**

   This geotechnical investigation was for the reconstruction of the Lemmon Drive and Military Road intersection. Soil borings and test pits were located on both sides of Lemmon Drive north of Military Road. The maximum depth of the explorations were to 7 feet below the existing ground surface.

5) **Lemmon Drive/Sky Vista/Buck Drive Intersection Improvements**

   This geotechnical investigation was for the reconstruction of the Lemmon Drive/Sky Vista/Buck Drive Intersection. Soil borings were located on both sides of Lemmon Drive near the intersection with Sky Vista Parkway and Buck Drive. Soil borings were drilled to depths of 5 feet below the existing ground surface.

6) **Lemmon Drive Extension**

   This geotechnical investigation was for the extension of Lemmon Drive from US 395 to North Virginia Street. Four test pits were located in the existing roadway alignment (new proposed Lemmon Drive extension alignment at the time of the investigation) and four test pits and two borings were located in the former roadway alignment located immediately south of the present alignment. Exploration depths for the soil borings and test pits ranged from 5 to 14 feet.

6.0 LABORATORY TESTING

All soils testing performed in CME's soils laboratory was conducted in accordance with the standards and methodologies described in Volume 4.08 of the ASTM Standards.

6.1 INDEX TESTING

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 presented on Plate B-1 and also on the boring and test pit logs.

Results of these tests were used to classify the soils according to ASTM D 2487 and to check the field logs, which were then updated as appropriate.

6.2 SOIL CORROSION TESTING

Western Environmental testing Laboratory (WETLAB) completed soil corrosion testing including soluble sulfates, resistivity, pH, redox potential, and sulfides on selected samples of native soils. Test results are reported on Plate B-2 – Corrosive Soil Analysis in Appendix B of this report and are discussed in Section 12.9.

7.0 REGIONAL PHYSIOGRAPHIC SETTING

Lemmon Valley is located along the western margin of the Basin and Range physiographic province and the eastern border of the Sierra Nevada physiographic province. One of a series of north-trending basins in northwestern Nevada, Lemmon Valley is bordered by Granite Hills to the west, Wedekind Hills to the east, to the north by Freds Mountain and Hungry Mountain, and to the south by Peavine Mountain. Lemmon Valley is a structural depression that is filled with unconsolidated and partially consolidated subaerial and lacustrine deposits.

Ranges bordering Lemmon Valley are deeply dissected, complex, fault-block mountains composed of igneous, metamorphic and sedimentary rocks. These ranges have been broken into troughs and ridges by normal faults. The mountains surrounding Lemmon Valley consist largely of granodiorite and other granitic rocks, with limited intruded tertiary volcanic rocks. Peavine Mountain differs from the other ranges in that it contains a higher percentage of extrusive rocks and metamorphic rocks. Topographic relief of the mountains resulted principally from uplift and warping associated with movement along normal faults, however, internal structure, volcanism, sedimentation, and erosion have also been major factors in the formation of the present land forms.

8.0 GEOLOGIC AND GENERAL SOIL CONDITIONS

8.1 NORTHERN TO CENTRAL PORTIONS OF THE WATERLINE ALIGNMENT GEOLOGY

The majority of Lemmon Valley consisted of a shallow desert lake during the last glacial period. As climatic conditions changed, a smaller inflow into the closed basin resulted and the lake began to dry up transcending into an intermittent or playa lake. As the lake receded, alluvial fan deposits were laid down rapidly around the edges of the surrounding hills often covering the earlier lacustrine sediments.

8.2 SOUTHERN PORTIONS OF THE WATERLINE ALIGNMENT GEOLOGY

The geologic conditions in this section of the water line alignment is predominantly comprised by coarse-grained alluvial fan deposits derived from Peavine Mountain. Toward the central portions of the alignment near Military Road, alluvial fan deposits derived from adjacent granodioritic bedrock outcrops were encountered. Both volcanic andesitic and granodioritic bedrock outcrops are located near this section of the water line alignment.

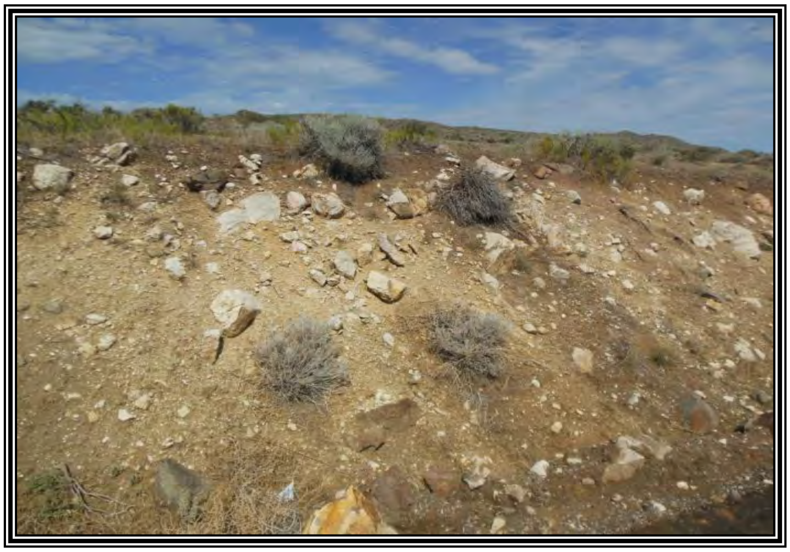
8.3 GENERAL SOIL AND BEDROCK CONDITIONS

Published geologic maps for the area (Cordy, 1985; and H.F Bonham Jr. and E.C. Bingler, 1973) indicate that the water line route traverses several different geologic deposits, as follows:

- The northern segment of the water line route is located within fine-grained lacustrine deposits that blanket the low-lying portions of Lemmon Valley;
- In the central portions of the water line alignment, the geology consists of a transition zone between fine-grained lacustrine and finer grained side stream and wash deposits. The side stream and wash deposits represent reworked alluvial fan deposits;
- Near the intersection with Military Road, the geologic unit is classified as a granular alluvium soil deposit derived from weathering of adjacent granodioritic bedrock;
- South of Military Road to Sky Vista Parkway, the water line alignment is located in a transition zone between alluvial fan deposits and tertiary andesitic bedrock. Localized volcanic bedrock outcrops are exposed in the cut slopes along the west side of Lemmon Drive; and
- The southern portion of the line between Sky Vista Parkway and North Virginia Street is located in coarse grained alluvial fan deposits of Peavine Mountain.

Generalized descriptions of the soil profile encountered in our field explorations are presented in Table 2 on the following page. Refer to logs in Appendix A and C for detailed descriptions. These descriptions are presented as seven (7) different geologic sections, which were encountered along the water line route. Refer to Plate A-5 in Appendix A for site plans showing the geologic sections and corresponding field exploration locations.

Table 2 – Generalized Soil Profile Descriptions		
Geologic Sections	**Location**	**Soil Profile Description**
1	**B-1 to B-11 (Waterash to Deodar)**	The geotechnical profile encountered has two predominant soil layers. The uppermost soil layer consists of a fill material classified as either a poorly graded gravel with silt and sand **(GP-GM)** or a clayey sand with gravel **(SC).** The lowermost soil layer consists of a fat clay **(CH)** or fat clay with sand **(CH).**
2	**B-12 to B-13 (Deodar to Patrician)**	Borings B-12 and B-13 met refusal at shallow depths of 1½ and 5 feet below the existing ground surface (bgs). A subsequent test pit was completed near B-12 and was excavated to a depth of 10 feet bgs. The uppermost soil horizon encountered consists of granular fill soils classified as silty sand **(SM),** clayey sand **(SC),** poorly graded sand with gravel **(SP-SM).** In Boring B-13, it appears that a geotextile was encountered at 2 feet below the existing ground surface. Below the uppermost fill zone, native soils are anticipated to consist of sandy fat clays **(CH)**.
3	**B-14 to B-20 (Patrician to Bernoulli Street)**	The geotechnical profile encountered has three predominant soil layers. The uppermost soil layer consists of a fill soil predominantly classified as silty sand with gravel **(SM)** with a thickness ranging from 1½ to 4 feet. Below this layer is a granular alluvium predominantly classified as silty sand **(SM)** with a thickness ranging from 3 to 4 feet. The lowermost soil layer generally consists of a clayey sand **(SC)** or sandy lean clay with sand **(CL).** Localized zones of poorly gravel with silt and sand **(GP-GM)** and poorly graded sand with silt **(SP-SM)** were also encountered.
4	**B-21 to B-22 (Bernoulli Street to ~1000' south of Military Road)**	The geotechnical profile encountered has three predominant soil layers. The uppermost soil layer consists of a fill material classified as silty sand **(SM)** or silty sand with gravel **(SM)** encountered to 1½ to 2 feet below grade. Below this layer is a silty with gravel **(SM)** encountered with a thickness of about 2 to 3 feet. The lowermost horizon encountered to the depth explored generally consists of clayey gravel with sand **(GC).**
5	**B-23 to B-25 (~1000' feet south of Military to Sky Vista)**	This section of the alignment is located in Tertiary volcanic bedrock and alluvial fan deposits. A residual soil layer consisting of clay soils is generally encountered above the volcanic bedrock. The generalized geotechnical profile encountered in the roadway consists of a structural section including a granular subbase material to depths of 2 to 3 feet below grade. Below the subbase to the depth explored, clay soils were encountered consisting either of lean clay with sand **(CL)** or fat clay **(CH).** Clay soils were interbedded with silty sand **(SM).**
6	**B-26 to B-27 (Sky Vista to US 395)**	This section of the alignment is located in finer grained alluvium. Two predominate soil horizons were encountered to the depth explored and were classified as either clayey sand **(SC)** or silty, clayey sand **(SC-SM).** The relative density of this soil horizons ranged from medium dense to dense.
7	**TP-1 to TP-2 (US 395 to North Virginia Street)**	This section of the alignment is located in a coarser grained alluvial fan deposit. The predominant soil horizons encountered to the depth explored were clayey sand with gravel **(SC)** or silty gravel **(GM)**. Some cobbles to about 12 inches in diameter were also encountered.



Andesitic bedrock cut slope located near road shoulder south of Military Drive

8.4 SOIL MOISTURE AND GROUNDWATER CONDITIONS

Groundwater was not encountered to the exploration depths completed with this investigation. Soils were generally encountered in a moist condition. Groundwater depth may fluctuate due to changes in precipitation, seasonal variations, or other conditions not noted at the time of our investigation.

9.0 SEISMIC HAZARDS

9.1 SEISMICITY

Much of the Western United States is a region of moderate to intense seismicity related to movement of the crustal masses (plate tectonics). By far, the most active regions outside of Alaska are along the San Andreas Fault zone 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 project site lies near the eastern base of the Sierra Nevada, within the western extreme of the Basin and Range.

It is generally accepted that the maximum credible earthquake in this area would be in the range of magnitude 7 to 7.5 originating from the frontal fault system of the Eastern Sierra Nevada. The most active segment of this fault system that is closest to the Reno-Stead area is located at the base of the eastern flank of the Carson Range near Thomas Creek, Whites Creek and Mt. Rose Highway, some 18 miles south of the project site.

9.2 FAULTS

Based on a review of the Earthquake Hazards Map (Bingler, 1974 and Cordy, 1985) and USGS Quaternary Faults on Google Earth Map, a mapped fault is trending through the proposed water line alignment near the intersection with North Virginia Street. This fault trends in a northeasterly direction, and is part of a series of en echelon faulting that consist of several short, discontinuous faults located near the base of Peavine Mountain.

Several other faults are located near the water line alignment. The closest mapped fault is located less ¼ miles west of the proposed water line alignment near Military Road. Other mapped faults are located about 1 mile west of the proposed water line alignment, near the north end of the project. This fault is known as the Airport Fault, which is mapped as concealed (younger sediments have concealed fault scarps). This fault trends in a near north to south direction and is shown to extend continuously along the east side of the Reno-Stead Airport, continuing along the base of the eastern flank of Hungry Mountain.

Quaternary earthquake fault evaluation criterion has been formulated by a professional committee for the State of Nevada Seismic Safety Council. These guidelines are consistent with the State of California Alquist-Priolo Act of 1972, which defines Holocene Active Faults as those with evidence of displacement within the past 10,000 years (Holocene time). Those faults with evidence of displacement during Pleistocene time (10,000 to 1,600,000 years before present) are classified as either late Quaternary Active Fault (10,000 to 130,000 years) or Quaternary Active Fault (> 130,000 years). Both of the latter fault designations are considered to have a decreased potential for activity compared to the Holocene Active Fault. An inactive fault is considered is a fault that does not comply with these age groups.

Based on the referenced earthquake hazard maps, the fault that is crossing the water line alignment is considered a Quaternary Active Fault. The other faults located near the project site are considered either Quaternary Active or inactive faults.

9.3 LIQUEFACTION

Liquefaction is a nearly a complete loss of soil shear strength that can occur during an earthquake, as cyclic shear stresses generate excessive pore water pressure between the soil grains. The higher the ground acceleration caused by a seismic event or the longer the duration of shaking, the more likely liquefaction will occur.

The soil types most susceptible to liquefaction are loose to medium dense cohesionless sands, soft to stiff non-plastic to low plastic silts, or any combination of silt-sand mixtures lying below the groundwater table. Liquefaction is generally limited to depths of 50 feet or less below the existing ground surface.

The referenced Earthquake Hazards Maps indicate that the northern to central portions of the water line alignment (north of Military Road) may be subjected to soil liquefaction during an earthquake event. The primary consequence of soil liquefaction is the potential for ground surface disruption consisting of either vertical settlements or lateral movements. Lateral movements are caused by lateral spread as non-liquefiable soil layers located above liquefiable soil layers are moved by gravitational forces during the earthquake event generally toward a slope face. Additional exploration consisting of deep borings to 50 feet below the existing surface, laboratory testing, and analysis would be required to determine potential settlements and lateral movements.

10.0 SEISMIC DESIGN PARAMETERS

Seismic design parameters are based on site-specific estimates of spectral response ground acceleration as designated in the International Building Code (IBC, 2012). The benefit of this approach is that a response spectrum can be developed from this data and, based on the period of the structure, a spectral acceleration for that structure can be determined. These values are based on two criteria: site classification and site location (latitude and longitude). Site classification is based on the substrata soil profile type, as presented in Table 3 (Site Classification Definitions).

Table 3 – Site Classification Definitions	
Site Classification	**Soil Profile Type Description**
A	Hard Rock
B	Rock
C	Very Dense Soil and Soft Rock
D	Stiff Soil Profile
E	Soft Soil Profile
F	Soil Type Requiring Site-Specific Evaluation

The soil/bedrock profile classification is based on two criteria: density (primarily for soils based on SPT blow count data or shear wave velocity) or hardness (based on shear wave velocity primarily for bedrock sites). These two criteria have to be determined to a depth of 100 feet bgs. Based on the soil profile encountered and known geologic conditions, a Site Classification of D (stiff soil profile) is recommended for all areas of the waterline alignment except for the northern portion where a softer soil profile with potential liquefiable soils are anticipated. The soil profile may classify as a site classification of E or even F in this portion of the alignment. If required, additional exploration will be required in this area to determine the design site classification.

Spectral response acceleration values (S_s & S_1) are based on structures underlain by bedrock with a site classification of B. Acceleration values may amplify or attenuate depending on the subsurface geologic conditions. Therefore, the IBC provides correction factors (F_a & F_v) to modify the acceleration values if the site is located overlying subsurface geologic conditions with a site classification other than B.

Spectral response acceleration values were determined from the USGS website: *U.S. Seismic Design Maps* Table 4 provides a summary of seismic design parameters, based on 2010 ASCE 7, as referenced by IBC, including correction factors F_a & F_v. A printout of the design information including spectral response acceleration values is provided in Appendix D.

Table 4 – Seismic Design Parameters[1]			
Design Parameters	**Design Values**		
	North Lemmon Valley[2]	**Military Road[3]**	**US 395[4]**
Spectral Response at Short Period (S_s), percent of gravity	1.577	1.581	1.567
Spectral Response at 1-Second Period (S_1), percent of gravity	0.513	0.521	0.520
Site Classification	D,E,F	D	D
Site Coefficient F_a, decimal	1.0	1.0	1.0
Site Coefficient Fv, decimal	1.5	1.5	1.5

Notes:
1) Based on 2012 IBC
2) lat.=39.6573^0 long= 119.8282^0
3) lat.=39.6227^0 long= 119.8482^0
4) lat.=39.6106^0 long= 119.8521^0

11.0 DISCUSSION AND RECOMMENDATIONS

The proposed water line alignment is located in the eastern and southern portions of Lemmon Valley and traverses a variety of alluvium soil types with different depositional environments. The bulk of the soils encountered originated from lacustrine and floodplain deposits, the reworking of older fan deposits, or coarse grained alluvial fan deposits. A localized outcrop of volcanic bedrock was encountered between Military Road and Sky Vista Parkway.

The soil profile characteristics encountered along the water line route are variable ranging from fine grained lacustrine deposits to coarse grained alluvial fan deposits. In non-structural areas granular soils encountered can generally be used as trench backfill. However, fine-grained lacustrine deposits will be difficult to densify and will preclude their use as trench backfill.

The majority of the water line will be constructed by open trench methods. However, a portion of the water line, located beneath US 395, will be placed by trenchless construction methods. Trenchless construction methodologies and potential construction difficulties will be discussed in this report.

It is anticipated that water line trenches can be excavated with standard construction equipment consisting of a track-mounted excavator. A bedrock outcrop area located between Military Road and Sky Vista Parkway may cause excavation difficulties even with a large trackhoe and specialized hydraulic equipment, such as a chipping hammer may be required.

The water line alignment crosses a mapped fault near the intersection of North Virginia Street and Lemmon Drive. This fault is classified as a Quaternary Active Fault, which has a low probability of future movement when compared with faults classified as Holocene or Late Quaternary. Consequently, recommendations for setbacks or specialized pipeline design through the fault trace are not required.

Clay soils were encountered throughout the water line alignment. Clay soils exhibiting high plasticity characteristics were encountered in Geologic Sections 1, 2 and 5. Clay soil can shrink or swell in response to moisture changes. Moisture changes within these soils can occur as a result of seasonal variations in precipitation, poor site drainage, landscape irrigation, leaking underground pipes, capillary action, or from other sources. Based on studies and experience, clay soil volume changes can cause differential movements within structural elements constructed within their sphere of influence. Therefore, structural elements should not bear directly on clay soil.

For purposes of this project, the following definitions shall apply:

- Fine-grained soil is defined as soil with more than 40 percent by weight passing the number 200 sieve and a plasticity index lower than 15.
- Clay soil is defined as a soil with more than 20 percent of the soil particles by weight passing the number 200 sieve and a plasticity index equal or greater than 15.
- Granular soil is defined as soil not meeting the above criteria with a particle sizing of less than 4-inches.

The recommendations provided herein, and particularly under **Site Preparation, Grading and Filling,** and **Construction Observation and Testing** are intended to reduce risks of structural distress related to consolidation or expansion of native soils and/or structural fills. These recommendations, along with proper design and construction of the planned structures 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 construction observation and testing should be performed to document that the recommendations presented in this report are followed.

Structural areas referred to in this report include all areas of 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*. Unless otherwise stated in this report, all related construction should be in accordance with the Standard Specifications for Public Works Construction, dated 2012.

Any evaluation of the site for the presence of surface or subsurface hazardous substances is beyond the scope of this study. When suspected hazardous substances are encountered during routine geotechnical investigations, they are noted in the exploration logs and reported to the client. No such substances were identified during our exploration.

The test pits were excavated by backhoe at the approximate locations shown on the site plan. All test pits were backfilled upon completion of the field portion of our study. The backfill was compacted to the extent possible with the equipment on hand. However, the backfill was not compacted to the requirements presented herein under **Grading and Filling**. If structures, concrete flatwork, pavement, utilities or other improvements are to be located in the vicinity of any of the test pits, the backfill should be removed and recompacted in accordance with the requirements contained in the soils report. Failure to properly compact backfill could result in excessive settlement of improvements located over test pits.

11.1 PIPELINE RESTRAINT DESIGN

Design recommendations will be given for two types of pipeline restraint: thrust blocks or restrained joints. Each of these design types will be discussed separately in the following sections.

11.1.1 Thrust Blocks

Design allowable bearing pressures for thrust blocks are based on the passive pressure lateral loading at the design depths for the thrust blocks. Passive pressures are based on the soil's shear strength (internal friction angle and cohesion) and unit weight. Because of the different soil types encountered including unit weights and shear strengths, thrust block allowable bearing pressures will vary somewhat along the water line route. Table 5 summarizes recommended design allowable bearing pressures for the different Geologic Sections encountered.

* Relative compaction refers to the ratio (percentage of the in-place density of a soil divided by the same soil's maximum dry density as determined by the ASTM D 1557 laboratory test procedure. Optimum moisture content is the corresponding moisture content of the same soil at its maximum dry density.

Table 5 – Allowable Bearing Pressures for Thrust Blocks	
Location[1]	**Allowable Bearing Pressure [2,3] (psf)**
Geologic Sections 1 & 2	1,500
Geologic Sections 3 & 4	2,000
Geologic Section 5	2,000
Geologic Section 6 &7	2,000

Note:

1) Refer to Table 2 (Generalized Soil Profile Descriptions) for Geologic Section Locations

2) The passive pressure value (allowable bearing pressure) is based on the following equation:

$$P_p = \gamma H_c N_\Phi + 2C_s\sqrt{N_\Phi} \text{ (DIPRA, 2006)}$$

γ = unit weight (pcf)

N_Φ = coefficient of passive pressure (Kp)

Cs = soil cohesion (#/ft^2)

H_c = mean depth from surface to centerline of pipe.

3) **Based on a depth to centerline of pipe of 5 feet. Other depths can be given, as requested.**

11.1.2 Restrained Joints

Pipeline/bedding material interface friction is used in the calculation of pipeline restraint. The total unit frictional resistance (Fs) is based on two components: unit normal force (earth pressure, pipe load, and water weight) and the pipeline/bedding material interface friction. The Fs value applies to ductile iron piping and is derived from recommendations by the Ductile Iron Pipe Research Association (DIPRA, 2006).

The pipeline/bedding interface friction depends on bedding material type, pipeline laying condition, and pipeline coating. A pipeline/bedding interface friction (tan δ) value of 0.55 is recommended for design. The following assumptions were used to determine this friction value:

- Asphalt coated piping;
- Granular bedding consisting of clean sand or gravel;
- Pipe laying condition Type 5 (DIPRA, 2006), which assumes that the bedding material is densified to at least 90 percent relative compaction and fully encapsulates the pipe.

If the pipeline is encased in polyethylene, a reduction factor of 0.7 is recommended by DIPRA. Therefore, the pipeline/bedding interface value is reduced to tan δ = 0.36. To determine earth pressure, a soil unit weight of 125 pcf is recommended.

12.0 CONSTRUCTION RECOMMENDATIONS

12.1 SITE PREPARATION

In non-structural areas all vegetation, topsoil, and other organics should be stripped and grubbed and removed from the site.

The asphaltic concrete in trenched areas shall be removed and isolated from existing pavements with full-depth saw cuts. In rigid pavement areas, the entire concrete panel may have to be removed depending on the location of the trench. Existing base aggregate could be re-used as backfill material, if carefully separated from underlying native soils.

All areas outside the trench area to receive structural fill or structural loading should be densified to at least 90 percent relative compaction in accordance with ASTM D 1557 for a minimum depth of 8 inches. This densification requirement on soils that are firm and unyielding, as determined by a representative of the geotechnical engineer, may be waived providing that all loosened material is removed to undisturbed ground.

It is recommended that soils have moisture contents of plus or minus 3 percent of optimum moisture (ASTM D1557) prior to densification. Higher moisture contents will be acceptable if the soil horizon is stable and density can be achieved in subsequent structural fill lifts. Scarification and moisture conditioning may be required to achieve the required soil moisture content recommendations. It is recommended that prior to densification, the moisture content of the soils be determined, to evaluate the need for moisture conditioning. After the densification process, a firm, stable surface should be produced. Unstable soils due to excessive moisture content may be encountered and should be scarified and allowed to dry or removed and replaced with structural fill. If these stabilization methods are not appropriate due to either time constraints or depth of unstable material, the unstable soil areas could be bridged using construction methodology discussed in Section 12.3. The appropriate construction method to treat unstable soil areas will be determined during construction.

12. 2 TRENCHING

12.2.1 Trench Excavation

It is anticipated that excavations can be made with a large track-mounted excavator. In bedrock areas, if encountered, excavation difficulties are possible. However, based on the fracturing patterns observed in the existing bedrock outcrop, it is anticipated that the bedrock can the excavated with a trackhoe; however, specialized construction equipment such as a chipping hammer may be required. Bedrock excavation may cause an enlargement of the trench width due to the removal of larger rock particles.

12.2.2 Trench Sidewall Stability

Regulations amended in Part 1926, Volume 54, Number 209 of the Federal Register (Table B-1, October 31, 1989) require that the temporary sidewall slopes be no greater than those presented in Table 6.

Table 6 - Maximum Allowable Temporary Slopes		
Soil or Rock Type	**Maximum Allowable Slopes[1] For Excavations Less Than 20 Feet Deep[2]**	
Stable Rock	Vertical	(90 degrees)
Type A[3]	3H:4V	(53 degrees)
Type B	1H:1V	(45 degrees)
Type C	3H:2V	(34 degrees)

NOTES:
1. Numbers shown in parentheses next to maximum allowable slopes are angles expressed in degrees from the horizontal. Angles have been rounded off.
2. Sloping or benching for excavations greater than 20 feet deep shall be designed by a registered professional engineer.
3. A short-term (open 24 hours or less) maximum allowable slope of 1H:2V (63 degrees) is allowed in excavations in Type A soil that are 12 feet or less in depth. Short-term maximum allowable slopes for excavations greater than 12 feet in depth shall be 3H:4V (53 degrees).

In general, Type A soils are cohesive, non-fissured soils, with an unconfined compressive strength of 1.5 tons per square foot (tsf) or greater. Type B are cohesive soils with an unconfined compressive strength between 0.5 and 1.5 tsf, while those designated as Type C have an unconfined compressive strength below 0.5 tsf. Numerous additional factors and exclusions are included in the formal definitions. Complete definitions and requirements on sloping and benching of trench sidewalls can be found in Appendix A and B of Subpart P of the previously referenced Federal Register. Appendices C through F of Subpart P apply to requirements and methodologies for shoring.

On the basis of our exploration, the soil type classification to assess the allowable temporary slope gradient is variable along the water line route and will classify as Type A, B, of C. A general guideline of OSHA soil types for each geologic section is presented in Table 7 on the following page.

Table 7 – Generalized Soil Type Description [1]		
Geologic Sections	**Location**	**Soil Type Description**
1	**B-1 to B-11 (Waterash to Deodar)**	The geotechnical profile encountered consisted of an uppermost granular fill stratum overlying a fat clay **(CH)** stratum. It is our opinion that the uppermost granular stratum classifies as a Type C soil. Based on field strength tests, the fat clay **(CH)** horizon classifies as either a Type A or B.
2	**B-11 to B-13 (Deodar to Patrician)**	The geotechnical profile is similar to Section 1, except that a clayey sand **(SC)** horizon was encountered below 6 feet in depth. It is our opinion that the clayey sand stratum classifies as a Type B soil.
3 & 4	**B-13 to B-22 (Patrician to~1000' south of Military Road)**	Predominantly granular soils classified as silty sand **(SM)** or clayey sand **(SC)** with variable percentages of gravel were encountered in Geologic Section 3. It is our opinion that this geotechnical profile classifies as either a Type B or C.
5	**B-23 to B-25 (~1000' feet south of Military to Sky Vista)**	The soil profile encountered is complexly interbedded with fine-grained and granular soil horizons. It is our opinion that this geotechnical profile classifies as either a Type B or C.
6	**B-26 to B-27 (Sky Vista to US 395)**	The soil profile is generally granular consisting of clayey sands **(SC)** exhibiting low to medium plasticity. Based on the relative density of these soils and field strength tests, it is our opinion that this geotechnical profile will classify as either a Type A or B soil.
7	**TP-1 to TP-2 (US 395 to North Virginia Street)**	The soil profile is generally similar as Section 7, but does contain granular cohesionless soil horizons. It is our opinion that this geotechnical profile will classify as either a Type B or C soil.

Notes:
1) All soil types presented in this table shall be verified during construction.
2) Refer to Table 2 (Generalized Soil Profile Descriptions) for Geologic Section Locations

Except for Type C soil areas, it is our opinion that the lower 3 ½ to 4 feet of the trench, as verified during construction, can excavated with a near vertical sidewall. Type C soils generally consist of uncemented, non-cohesive granular soils, which will be encountered throughout the pipeline route. Therefore, identification of the different soil types will be critical during construction.

All trenching should be performed and stabilized in accordance with local, state, and OSHA standards. In any case bank stability will remain the responsibility of the contractor, who is present at the site, able to observe changes in ground conditions, and has control over personnel and equipment.

12.3 BOTTOM OF TRENCH PREPARATION

Bottom of trench preparation in areas with firm, unyielding soils, as determined during construction, shall consist of removing all loose soil particles from the bottom of the trench.

Soils encountered in the northern to central segments of the water line route (Geologic Sections 1 to 3) may become unstable at the bottom of the trench or yield such that densification of the bedding sand layer could be difficult. Unstable soils encountered should be removed and replaced with a geotextile/gravel system as described in Section 12.3.1.

12.3.1 Stabilization Construction Methods

Over-excavation and replacement with a geotextile/gravel system may be used for lower trench stabilization. This system has two separate components: stabilizing fill geotextile and stabilizing fill.

- Stabilizing fill geotextile shall conform to the requirements provided in Section 12.3.2.1 and shall be placed between the stabilizing fill and native soils to provide separation and reinforcement;

- Stabilizing fill shall conform to the requirements provided in Section 12.3.2.2. The thickness of the fill depends on the level of instability; however, a minimum thickness of 12 inches is anticipated.

12.3.2 Materials

12.3.2.1 Stabilizing Fill Geotextile

The stabilizing fill geotextile should be woven and meet or exceed the following minimum properties presented in Table 8.

Table 8 - Stabilizing Fill Geotextile

Mechanical Properties	Minimum Average Roll Value (MARV)	
	MD (#/ft)	CD (#/ft)
Tensile Strength at ultimate (ASTM D 4595)	4600	4800
Tensile Strength at 5% strain (ASTM D 4595)	1400	1400
Apparent Opening Size (AOS)	***0.43 mm maximum***	

Products such as a Mirafi HP565, Terra Tex HPG-70 or approved equal can be utilized for this project.

12.3.2.2 Stabilizing Fill

Stabilizing fill shall consist of an angular, clean drain rock, meeting the requirements of Table 9. Class "C" backfill, meeting the requirements of Section 200.03.05 of the referenced SSPWC, can be used as stabilizing fill.

Table 9 – Stabilizing Fill Gradation Specifications

Sieve Size	Percent by Dry Weight Passing
1 inch	100
¾ inch	90 – 100
$^3/_8$ inch	10 – 55
No. 4	0 – 10

12.3.3 Placement Recommendations

The geotextile should be laid in accordance with the manufacturer's recommendation with a minimum joint overlap of 18 inches. Unless different recommendations are given by the manufacturer, the following minimum placement recommendations shall be followed:

- Prior to placement of the geotextile, the underlying soil surface should be smooth without sharp particles or abrupt edges;
- Construction equipment is prohibited from traveling directly over the geotextile;
- Stabilizing fill shall be placed from outside the excavation;
- It is recommended that the initial lift of stabilizing fill have a minimum loose lift thickness of 12 inches;
- Stabilizing fill should be densified with at least 5 passes with a vibratory plate whacker or equivalent equipment;
- The stabilizing fill shall be fully encapsulated by the stabilizing fill geotextile with an overlap of at least 12-inches.

12.4 TRENCH BEDDING AND BACKFILL

Bedding and backfill shall comply with requirements provided in Section 5 and Appendix 10L-6 of the Truckee Meadows Water Authority (TMWA) Engineering and Construction Standards, 2011. This section provides a summary of these requirements based on the geotechnical profile encountered with this investigation.

Based on the geologic profile encountered, bedding including the initial backfill 12 inches over the pipe will require import. Pipe bedding including the initial 12 inches of backfill over the pipe shall comply with the specifications given for a Class A backfill material (SSPWC, 2012).

Native granular soils with a particle sizing of less than 4-inches and free of debris, organics, or other deleterious material meeting the requirements of a Class E Backfill material (SSPWC, 2012) can be used as backfill material in non-structural areas. Aggregate base material meeting the requirements of a Type 2, Class B aggregate base (SSPWC, 2012) shall be used as backfill material in structural areas. Table 10 provides a general guideline of available native soils that can be used as backfill in non-structural areas based on the geologic sections described in Table 7. All backfill soils shall be tested for conformance with project specifications prior to use as a trench backfill soil.

Table 10 – Anticipated Available Native Backfill Materials [1]

Geologic Sections[2]	Location	Soil Type Description
1 & 2	**B-1 to B-13 (Waterash to Patrician)**	The geotechnical profile encountered consisted of an uppermost granular fill stratum overlying a fat clay **(CH)** stratum. It is our opinion that the uppermost granular fill stratum can be used for a backfill soil. The native soils consisting of the fat clay stratum cannot be used as backfill.
3 & 4	**B-13 to B-22 (Patrician to ~1000' south of Military Road)**	The uppermost soil horizons encountered are generally granular exhibiting low plasticity soil characteristics. It is our opinion that these uppermost granular soils could be used as trench backfill. Below depths of 4 to 5 feet, the plasticity of the soil profile increases such that these soils should not be used for backfill. Cobble sized particles may also be encountered in the granular soil deposits, especially toward Military Road, and may have to be segregated from trench backfill soils.
5	**B-23 to B-25 (~1000' feet south of Military to Sky Vista)**	The soil profile encountered is complexly interbedded with fine-grained and granular soil horizons. Except for existing base aggregates and granular subbase soils, native soils are generally not acceptable for use as trench backfill.
6	**B-26 to B-27 (Sky Vista to US 395).**	The soil profile is generally granular and anticipated to be acceptable for use as trench backfill. However, localized areas of clayey sands **(SC)** exhibiting medium plasticity characteristics may not be acceptable as trench backfill.
7	**TP-1 to TP-2 (US 395 to North Virginia Street)**	This soil profile is similar as Section 6, however, cobble sized particles may be encountered and may have to be segregated form trench backfill soils.

Notes:

1) For use in non-structural areas. All soil types presented in this table shall be verified during construction.

2) Refer to Table 2 (Generalized Soil Profile Descriptions) for geologic section locations.

12.4.1 Densification and Maximum Lift Thickness Requirements

Bedding and the initial 12-inches of backfill shall be placed in maximum 8-inch thick (loose) lifts and densified to a minimum of 95 percent relative compaction. Backfill shall be placed in maximum 12-inch (loose) lifts and densified to 85 percent in non-structural areas and 90 percent in structural areas. The upper 12 inches of backfill in structural areas shall be densified to a minimum of 95 relative compaction.

It is recommended that soils have moisture contents of at least plus or minus 3 percent of optimum moisture (ASTM D1557). Higher moisture contents are acceptable if the soil lift is stable and required relative compaction can be attained in the soil lift and succeeding soil lifts.

Backfill shall not consist of frozen soils or be placed on frozen soils.

12.5 TRENCHLESS CONSTRUCTION

Trenchless construction is anticipated for the water line alignment located beneath US 395. The current water line alignment is through the approach fill embankment on the south side of the Lemmon Drive overpass. Several trenchless construction methods are available consisting of pipe jacking, horizontal boring, or directional drilling. Each of these methods will be discussed in this section.

12.5.1 Horizontal Boring

This method involves excavating a sender and receiver pit. The drilling equipment is placed in the sender pit and a small pilot hole (2 to 4 inches in diameter) is drilled through the pit sidewall in the direction of the receiver pit. The pilot hole is subsequently reamed out with larger diameter drilling equipment to the required diameter, which is typically 25 percent greater than the pipeline diameter. After reaming, the pipe is pulled through the hole with the pulling head and reamer. The pipeline is attached to a swivel, so the rotation of the reamer is not transmitted to the pipe.

This method can achieve relatively good accuracy for line and grade over short distances. Drilling mud can be used to reduce support soil caving potential.

12.5.2 Directional Drilling

Directional drilling is similar to horizontal drilling except the drilling is started at the ground surface and sender or receiver pits are not required. The start of the drilling is at a location away from the design line and grade. Typically, the pilot hole is launched from the surface at an angle between 8 to 20 degrees from horizontal and transitions to horizontal as the required depth is reached. The reamed hole consists of a gradual curvature path to allow the pipeline to stay within the allowable joint deflection and curve radius.

12.5.3 Pipe Jacking and Micro Tunneling

This construction method consists of pushing a pipeline section through the ground from a jacking pit. The material is excavated from inside the pipe and the pipeline is advanced through the ground by the jacking operation. This process is repeated until the trenchless pipeline length has been completed. True pipe jacking without micro tunneling requires construction personnel to manually excavate the tunnel from inside the pipe. This method is generally applicable for larger diameter pipe (42 inches or greater).

Because the line and grade can be controlled by excavating in front of the pipeline and lasar guided boring machines are used, good control of the direction and grade is possible.

12.5.4 Trenchless Construction Summary

It is understood that the method of trenchless construction will be determined by the contractor. A disadvantage with horizontal boring is boulders in the soil profile could deflect the drilling head. Large cobbles or boulders were not encountered in our field explorations near the trenchless construction location. However, small boulders and large cobbles were observed in the cut slope located immediately east of TPWL-2 alongside the southbound travel lanes for US 395. It is recommended that the bidding contractor complete his own site assessment and determine if these particles will be problematic to the drilling operation.

12.5.5 Sending and Receiving Pit Shoring

A braced temporary shoring system will likely be used for sidewall support. To determine the active soil pressure loading, it is recommended to use a rectangular apparent earth pressure distribution delineated by 25H (psf), where H is the height of the excavation. This pressure distribution does not include surcharge loading occurring at the top of the excavation. Surcharge pressures shall be evaluated on a case by case basis.

12.6 STRUCTURAL FILL

Structural fill, where required., shall comply with the requirements for a Class E backfill material (SSPWC, 2012). Structural fill shall be placed in maximum 12-inch (loose) lifts and densified to at least 90 percent relative compaction.

It is recommended that soils have moisture contents of at least plus or minus 2 percent of optimum moisture (ASTM D1557). Higher moisture contents are acceptable if the soil lift is stable and required relative compaction can be attained in the soil lift and succeeding soil lifts.

12.7 THRUST BLOCK FOUNDATION PREPARATION

All thrust block foundations should be constructed against firm, unyielding soils. All loose soils generated from the excavation process should be removed from the foundation area. If soft or unstable native soils are encountered they should be removed and replaced with structural fill.

If clay soils are encountered with a plasticity index of 20 or greater within 2 feet of foundation grade, they should be removed to at least 2 feet from foundation grade and replaced with structural fill. These soils will be encountered within Geologic Sections 1, 2 & 6. To achieve this requirement, the trench will have to be widened at the thrust block location and additional bedding material shall be placed. Besides providing a separation fill between the clay soils and the foundation, the structural fill will provide a firm surface to bear the thrust block, as native soils, especially in Geologic Sections 1 & 2 are anticipated to be very moist and relatively soft.

12.8 CONCRETE SLABS

All concrete slabs should be directly underlain by at least 6 inches of Type 2, Class B aggregate base. Aggregate base courses should be densified to at least 95 percent relative compaction. If clay soils are encountered within 2 feet of subgrade elevation, they should be removed to at least 2 feet below subgrade elevation and replaced with structural fill.

Type II cement should be used for all concrete work. The Reno-Stead area is a region with exceptionally low relative humidity. As a consequence, concrete flatwork is prone to excessive shrinking and curling. Concrete mix proportions and construction techniques, including the addition of water and improper curing, can adversely affect the finished quality of the concrete and result in cracking, curling and spalling of slabs. We recommend that all placement and curing be performed in accordance with procedures outlined by the American Concrete Institute. Special considerations should be given to concrete placed and cured during hot or cold weather conditions. Proper control joints and reinforcing should be provided to minimize any damage resulting from shrinkage.

12.9 SOIL CORROSION TEST RESULTS

Soil corrosion tests include pH, redox potential, sulfides, soluble sulfates, and resistivity. Except for soluble sulfates, these test results are used to determine if ductile iron piping needs to be encased with polyethylene. A listing of all test results by sample location including anticipated field moisture conditions is presented in Table 11. A brief summary of the soil corrosion tests is presented below:

- **Soluble sulfates:** Soluble sulfate test results range from ND to 200 ppm. These results indicate a negligible sulfate exposure to concrete.

- **pH:** The pH test results ranged from 7.1 to 9.1, which indicates a slightly alkaline soil condition.

- **Resistivity:** Resistivity test results ranged from 320 to 2,000 ohms x cm. In general, soils with a resistivity below 3,000 ohms x cm are corrosive to metal pipes. The resistivity results indicate that soil corrosion potential is quite variable along the water line route and ranges from highly to moderately corrosive to the pipeline. Typically, fine-grained soils with high in-situ moisture contents have lower resistivity results.

- **Redox potential:** The redox potential indicates the degree of aeration in the soil. The redox potential ranged from 310 to 400 mv, which indicates an aerobic soil condition and is generally non-corrosive to metal pipes.

- **Sulfides:** Sulfides were not detected in the soils. The presence of sulfides indicates that sulfate-reducing bacteria may be present, which can be corrosive to metal pipes.

Table 11 - Soil Corrosion Test Results

Exploration location[1]	Laboratory Tests					
	Resistivity (ohmxcm)	Redox potential (mv)	Sulfide	pH	Sulfates (ppm)	Field moisture conditions
B-4 (4B)	640	320	Negative	8.6	ND	moist
B-9 (9B & 9C combined)	490	330	Negative	9.1	ND	moist
B-15 (15B&15C combined)	1400	320	Negative	8.4	23	moist
B-20 (20B & 20C combined)	2000	310	Negative	7.1	ND	moist
B-26 (26A & 26B combined)	320	NC	NC	7.1	200	moist
TPWL-1 (1B 1-4')	900	400	Negative	6.7	ND	moist

Notes:
1) Refer to site plan and logs for soil sample location.
2) NC: not completed
3) ND: non-detected

12.10 CONCRETE

A concrete mix with a maximum water/cementitious ratio of 0.5 should be utilized for all concrete work in contact with native soils, including foundations. Concrete exposed to freezing and thawing in a moist condition or to deicing chemicals should consist of a mix with a maximum of 0.45 water/cementitious ratio and have a compressive strength of 4,500 psi in 28 days.

12.11 STRUCTURAL SECTION

12.11.1 Structural Section Design

The minimum structural section for trench patches in Washoe County is 4-inches of asphaltic concrete overlying 6-inches of Type 2, Class B aggregate base (Standard Details for Public Works Construction, dwg no. W-22). Also, in accordance with the requirements of the trench patch specification, the asphaltic concrete thickness shall match the thickness of the existing contiguous pavement, if greater than 4-inches. Existing pavement thickness is greater than 4 inches and ranged from 5½ to 8½ inches.

Cross streets to Lemmon Drive are either classified as local or collector streets. The minimum structural street for local and collector streets are 3 or 4 inches of AC, respectively, overlying 6 inches of base aggregate.

Based on the many different layers of asphalt observed in the pavement cores and structural distress, the existing pavement has been in service for many years. Consequently, the structural strength of the existing pavement is not equivalent to the structural strength of a new pavement. AASHTO pavement design uses strength coefficients for both the base aggregate and pavement. Typically, a coefficient of 0.39 is used for each inch of new pavement and 0.12 is used for each inch of Type 2, Class B aggregate base material. The total structural number for 4-inches of AC over 6-inches of base is 2.28. By using a conservative structural coefficient for the existing pavement (70 percent of a new pavement layer), the existing structural section has a total structural number that varies from 1.5 (5½") to 2.32 (8½"). This structural number is equal to or less than the structural number for the recommended minimum trench patch structural section. Accordingly, it is recommended that the structural section for the trench patch is 4 inches of asphaltic concrete overlying 6-inches of aggregate base in existing cross street areas.

It is understood that the water line alignment will be routed through existing rigid pavement areas at the intersections of Military Road and Sky Vista Parkway. The referenced geotechnical investigations for these intersections provided recommended rigid pavement thicknesses. At the intersections of Lemmon Drive with Military Road and Sky Vista Parkway, the recommended structural section was 10½ inches of PCC overlying 8 inches of aggregate base. These thicknesses shall be verified during construction, but provides a starting point to determine quantities.

12.11.2 Structural Section Construction

.

The following presents construction recommendations for the structural section.

1) Subgrade soil should be prepared in accordance with the recommendations of Section 12.1- Site Preparation and Section 12.3.1. Base Material should be densified to at least 95 percent relative compaction.

2) A Type 3 plantmix aggregate in accordance with Section 200.02 of the referenced standard specifications for public works improvement should be utilized for the pavement. All pavement construction shall conform to the referenced standard specifications.

3) The contractor should submit a pavement mix design to the owner at least 10 working days prior to construction for approval. It is recommended that when pavement is placed adjacent to concrete flatwork, the finish compacted grade of the pavement be at least ¼ to ½ of an inch higher than the edge of adjacent concrete surface. This is to allow adequate compaction of the pavement without damaging the concrete.

12.12 ANTICIPATED CONSTRUCTION PROBLEMS

Some difficulty will be encountered in trenching due to the presence of moderately cemented zones and volcanic bedrock. Yielding/unstable soils may be encountered in the northern to central segments of the water line route and stabilization may be required.

13.0 CONSTRUCTION OBSERVATION AND TESTING SERVICES

The recommendations presented in this report are based on the assumption that the owner/project manager provides sufficient field testing and construction review during all phases of construction. Prior to construction, the owner/project manager should schedule a pre-job conference to include, but not be limited to: owner/project manager, project engineer, general contractor, earthwork and materials subcontractors, and geotechnical engineer. It is the owner's/project manager's responsibility to set-up this meeting and contact all responsible parties. 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 the owner/project manager for review and distributed to the appropriate parties.

14.0 STANDARD LIMITATION CLAUSE

This report has been prepared in accordance with generally accepted local geotechnical practices. The analyses and recommendations submitted are based upon field exploration performed at the locations shown on Plate A-2 – Field Exploration Location Maps 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. Sufficient construction observation should be completed in all phases of the project related to geotechnical factors to document compliance with our recommendations.

This report has been prepared to provide information allowing the engineer to design the project. The owner/project manager is responsible for distribution of this report to all designers and contractors whose work is affected by geotechnical recommendations. In the event of changes in the design, location, or ownership of the project after presentation of this report, our recommendations should be reviewed and possibly modified by the geotechnical engineer. If the geotechnical engineer is not accorded the privilege of making this recommended review, he can assume no responsibility for misinterpretation or misapplication of his recommendations or their validity in the event changes have been made in the original design concept without his prior review. The 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.

This report was prepared by CME for the account of the Truckee Meadows Water Authority. The material in it reflects our best judgment in light of the information available to us at the time of preparation. Any use which a third party makes of this report, or any reliance on or decisions to be made based upon it, are the responsibility of such third parties. Construction Materials Engineers Inc. accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

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