GEOTECHNICAL INVESTIGATION
STEAD MAIN REPLACEMENT PHASE 2
RAILROAD CROSSING
WASHOE COUNTY, NEVADA

PREPARED FOR:

TRUCKEE MEADOWS WATER AUTHORITY

DECEMBER 2016
FILE: 1921

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

December 19, 2016
File: 1921

Mr. Kelly McGlynn, PE
Truckee Meadows Water Authority
1355 Capital Boulevard
Reno, Nevada 89502

RE: Geotechnical Investigation Proposal
Stead Water Main Replacement Phase 2 Railroad Crossing
Reno, Washoe County, Nevada

Dear Mr. McGlynn:

Construction Materials Engineers Inc. (CME) is pleased to submit the results of our geotechnical investigation report for the Stead Water Main Replacement Phase 2 Railroad Crossing, located in 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 project.

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

Sincerely,

CONSTRUCTION MATERIALS ENGINEERS, INC.

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GEOTECHNICAL INVESTIGATION
STEAD MAIN REPLACEMENT PHASE 2
UPRR WATERLINE CROSSING
WASHOE COUNTY, NEVADA

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 Union Pacific Rail Road Waterline Crossing, in Stead, Washoe County, Nevada. The geotechnical recommendations contained herein are based on surface and subsurface conditions encountered during our field exploration, and on details of the proposed project as described in this report. The objectives of this study were to:

1. Investigate general soil and ground water conditions pertaining to design and construction of the proposed project; and

2. Provide geotechnical recommendations for design and construction of the project, based on the results of our field and laboratory studies.

Our geotechnical study included subsurface field exploration, laboratory testing, and engineering analysis to provide recommendations for project design.

The area covered by this report is shown on Plate A-1 (Field Exploration Location Map) in Appendix A. Results of our field exploration and testing programs form the basis for all conclusions and recommendations.

2.0 SITE DESCRIPTION

The project site is located on the south side of North Virginia Street, approximately 400 feet northwest of the Stead Boulevard intersection. The project boundaries are located entirely in Section 7, T20N, R19E (M.D.M). The general project vicinity map is included as Figure 1 (General Project Vicinity Map).

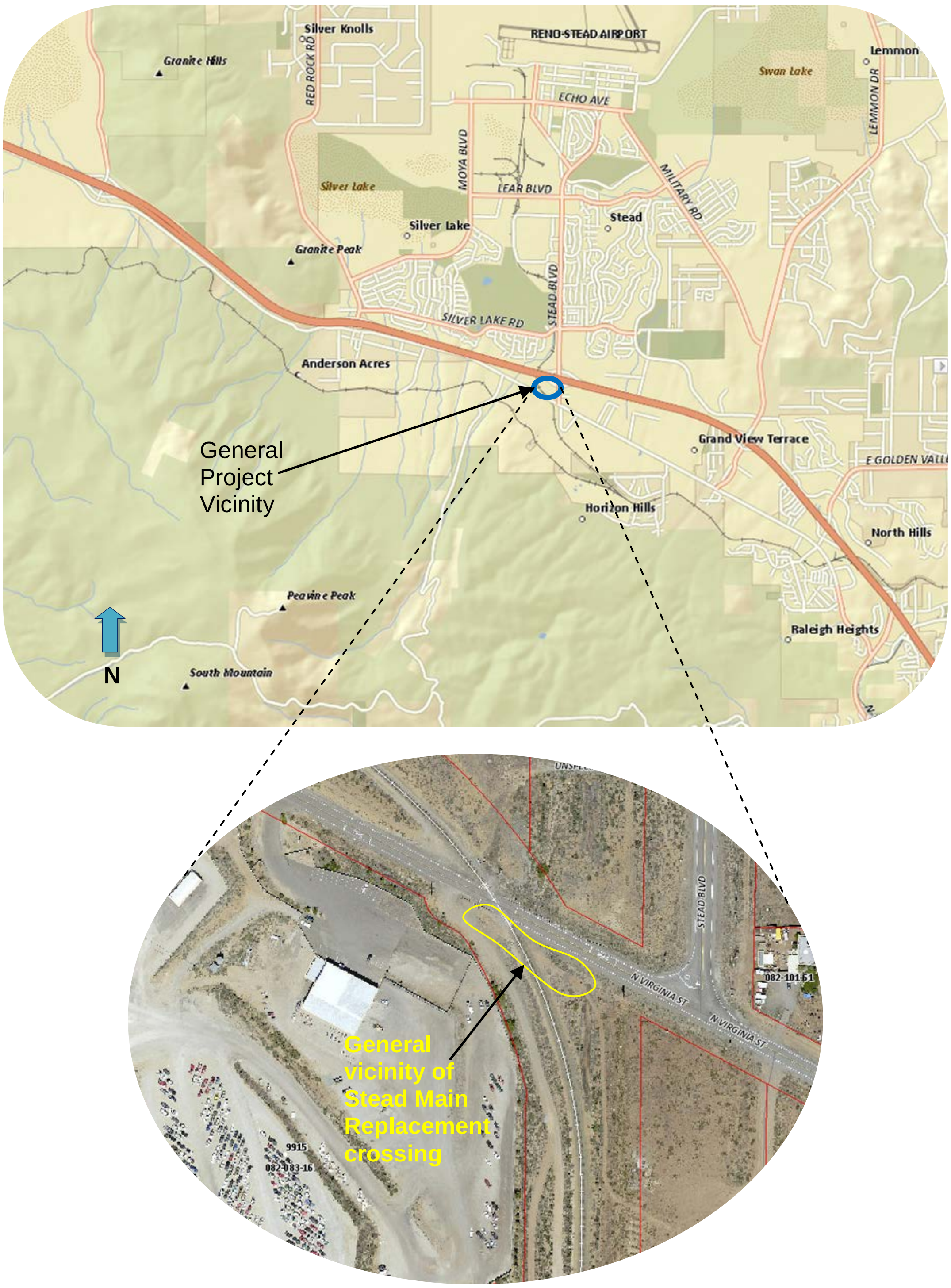


Figure 1: General Project Vicinity Map (Not to Scale)

(Reference: Washoe County Technology Services-Regional Services Division (GIS) www.washoecounty.us/gis , December 2016).

The Stead Main Phase 2 replacement includes approximately 11,881 linear feet of waterline. Our scope of work focused on the alignment segment that crosses beneath the existing Union Pacific Rail Road-

Stead Spur railroad tracks. The approximate limits of the crossing are shown on Figure 1 (General Project Vicinity Map). The Stead Spur rail road generally trends in a north to south direction within the vicinity of the project, and crosses North Virginia Street about 400 feet northwest of Stead Boulevard Intersection.

The project site is accessed via existing dirt roads located northwest and southeast of the existing rail road crossing on North Virginia Street. The site is drained via sheet flow. The north side of the alignment has little to no vegetation, as the dirt road serves as a main access for an adjacent property. Vegetation on the south side of the alignment consists of sparse to moderately dense brush.

Site topography is gently rolling with slope gradients on the order of 2 to 3 percent.

3.0 PROJECT DESCRIPTION

It is understood that the existing Stead water main will be replaced with a 20-inch diameter ductile iron pipe. At about Station 87+40, the water main will be routed below the existing Stead Spur rail road tracks. The anticipated construction method is Jack and Bore. A 30-inch diameter steel casing will be placed below the tracks, prior to the placement of the proposed waterline.

The top of the 30-inch diameter steel casing will be located approximately 10 feet below the rail road track grade. A sender and receiver pit will be excavated on either side of the rail road tracks with a radial distance varying from about 30 to 70 feet. The Sending Pit will have an anticipated dimension of 20 feet by 40 feet with a depth of about 15 feet. The Receiving Pit will have an approximate dimension of 20 feet by 30 feet with a depth of about 14 feet

Figure 2 (Stead Main Replacement Sending and Receiving Pit Locations), shows the approximate location of the sending and receiving pits main alignment.

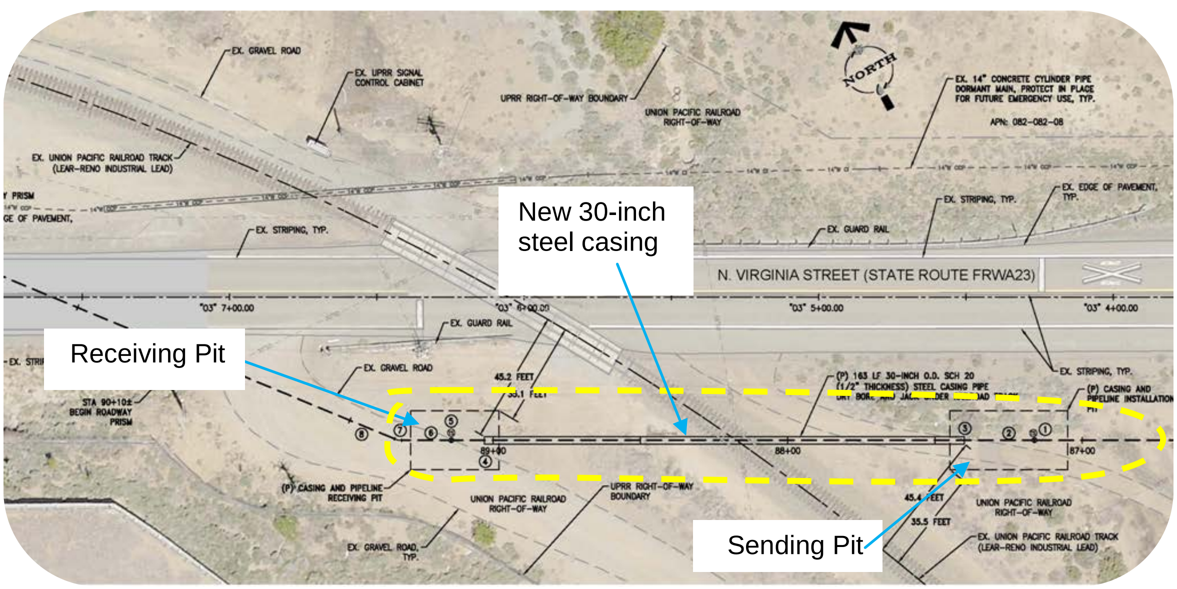


Figure 2: Stead Main Replacement Sending and Receiving Pit Locations
(Reference: Base map taken from Stead Main Replacement Phase 2, N. Virginia Street (State Route FRWA23), Plan and Profile Sta 86+50 to 91+60, Sheet P16, dated November 2016 by TMWA.)

4.0 FIELD EXPLORATION

Subsurface exploration was completed on November 22, 2016 and included drilling two (2) test borings, one at the sending pit and one at the receiving pit locations. The borings were drilled using a truck-mounted CME 85 drill rig with automatic hammer. The drilling method included hollow stem continuous-flight augers with 6-inch outside diameter (O.D.), 3¼-inch inside diameter (I.D.).

Photograph 1: Looking north at Boring B-1, located in the general vicinity of the Receiving Pit.

The maximum depth of exploration was 21 ½ feet below the existing ground surface. The approximate locations of the test borings are shown on Plate A-1 (Field Exploration Location Map).

Soils were sampled in-place every 2½ feet using a standard 2-inch O.D.[1] or 3-inch O.D. split-spoon sampler driven by a 140-pound automatic hammer with a 30-inch drop.

The 3-inch O.D. split-spoon sampler was used where tube or ring samples were required. Sampling methods used were similar to the 2-inch O.D. (SPT) but also include the use of either 2½-inch diameter, 6-inch-long sampling tubes or 2½-inch diameter, 1-inch-long sampling rings placed inside the split-spoon sampler. Due to 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[2]. Boring logs indicate the type of sampler used for each sample.

[1] The number of blows to drive the 2-inch O.D. sampler the final 12 inches of an 18-inch penetration into undisturbed soil is an indication of the density and consistency of the material (Standard Penetration Test (SPT) - ASTM D 1586).

[2] Several methods are available to convert non-standard blow counts to SPT blow counts. In general equations correct for overburden, hammer efficiency, sampler size, etc. Corrections vary and are dependent on the calculation method used.

Stratification lines on the logs represent the approximate boundary between soil types and the transition should be considered gradual. The borings were backfilled with cement grout to within 2 feet of the existing ground surface. The remaining void was backfilled with the cuttings and graded to the extent possible using the equipment at hand.

5.0 LABORATORY TESTING

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

Significant soil types were selected and analyzed to determine index properties and engineering properties. The following laboratory tests were completed as part of this investigation[3]:

- Insitu moisture content (ASTM D 2216) (Appendix A);
- Plasticity Index and Liquid Limit (ASTM D 4318) (Appendix A);
- Grain size distribution (ASTM D 422) (Appendix B);
- Moisture density relationship (ASTM D 1557) (Appendix B);
- Soil unit weight (ASTM D 7263);
- Direct Shear (ASTM D 3080) on remolded sample(Appendix B); and
- Corrosion testing (soluble sulfates, resistivity, pH, chlorides, and redox potential) was completed by an outside laboratory (Appendix B).

[3] Unconfined compressive strength testing was attempted; however, once soil samples were extruded, due to the gravel content of the existing soils, a representative soil specimen could not be prepared.

6.0 GEOLOGIC AND GENERAL SOIL PROFILE DESCRIPTIONS

Based on a review of the Verdi Quadrangle Geologic Map (J.W. Bell and L. J. Garside, 1987), the project site is underlain by older alluvial fan deposits, described as muddy sandy cobble to boulder gravel, and subangular metamorphic clasts.

The NRCS Web Soil Survey maps the site as being underlain by Cassiro gravelly sandy loam.

The geologic conditions encountered will be discussed separately for the north receiving pit and south sending pit in Sections 6.1 and 6.2, respectively.

6.1 Receiving Pit (North Side of Stead Spur Tracks)

The soils profile encountered within Boring B-1, located at the north receiving pit, consisted predominately of granular materials ranging from silty, clayey sand with gravel **(SC-SM)** to clayey sand with gravel **(SC)**.

The boring was completed in an existing improved dirt road. The uppermost soils horizon consisted of a poorly graded gravel with sand fill **(GP)** underlain by silty, clayey sand with gravel **(SC-SM)** to a depth of 5 feet. Below a depth of 5 feet, granular soils encountered ranged from clayey sand **(SC)** to clayey sand with gravel **(SC)** to the depth of exploration. A sandy lean clay (CL) interbed was encountered from a depth of 10 to 15 feet below grade. Based on SPT blow counts, granular soils were encountered in a dense to very dense relative density below a depth of 5 feet. Fine grained soils were encountered with a hard consistency. The boring log should be reviewed for more detailed descriptions.

6.2 Sending Pit (South Side of Stead Spur Tracks)

Soils encountered within Boring B-2, located at the south sending pit, also consisted predominately of granular materials ranging from poorly graded gravel and cobbles **(GP)** to clayey sand with gravel **(SC).**

The upper soils horizon ranged from silty sand with gravel **(SM)** to poorly graded gavel with sand **(GP)** and contained cobble sized particles to a depth of 5 feet. Below this depth, soils consisted of interbedded clayey sand **(SC)** and clayey sand with gravel **(SC)**. Based on SPT blow counts, granular soils were encountered in a medium dense to dense relative density below a depth of 5 feet. Unlike the north side of the Stead Spur Tracks, a fine grained soil interbed was not encountered.

Photograph 2: Boring B-2 at a depth of about 7 ½ feet. Note cobbles present on the ground surface.

6.3 Soil Moisture and Groundwater Conditions

In general, soils were encountered in a moist condition. Groundwater was not encountered during the subsurface exploration. Although groundwater was not encountered, it should be noted that fluctuations in subsurface moisture conditions may occur during spring runoff or times of increased precipitation.

7.0 SEISMIC AND GEOLOGIC HAZARDS

7.1 Seismicity

The Western United States is a region of moderate to intense seismicity related movement of the crustal masses (plate tectonics). The most active regions outside of Alaska are along the San Andres Fault zone of western California and the Wasatch Front in Salt Lake City.

Figure 3: Overview Map Showing the Great Basin (N.T.S)
(Image obtained from https://upload.wikimedia.org/wikipedia/commons/5/56/Greatbasinmap.png)

The Wasatch Front in Salt Lake City, Utah, forms the eastern boundary of the Basin and Range physiographic province, and the eastern form of the Sierra Nevada Mountains, which is the western margin of the province. The subject site is located in a seismically active zone within the western extreme of the Basin and Range.

7.2 Faults

To determine the location of mapped earthquake faulting trending through or near the project site, a review of the following published information was completed:

1) USGS Website: *Earthquake Hazards Program Quaternary Faults in Google Earth* (excerpt included as Figure 4);

2) The referenced geologic map (J.W. Bell and L. J. Garside, 1987) (excerpt included as Figure 5);

3) The USGS Interactive Fault Map (http://earthquake.usgs.gov/hazards/qfaults/map/); and

4) Quaternary Faults in Nevada, (dePolo, 2008).

The project site is located within the lower slope along the on the eastern slope face of Peavine Mountain in an area with prominent range-front faults. Three predominate fault traces are located northwest of sending/receiving pits. The nearest fault trace is located between 200 and 300 feet northwest of the receiving pit. These faults are associated with the North Peavine Mountain Fault zone, and are mapped as Latest Quaternary aged (<15,000 years).

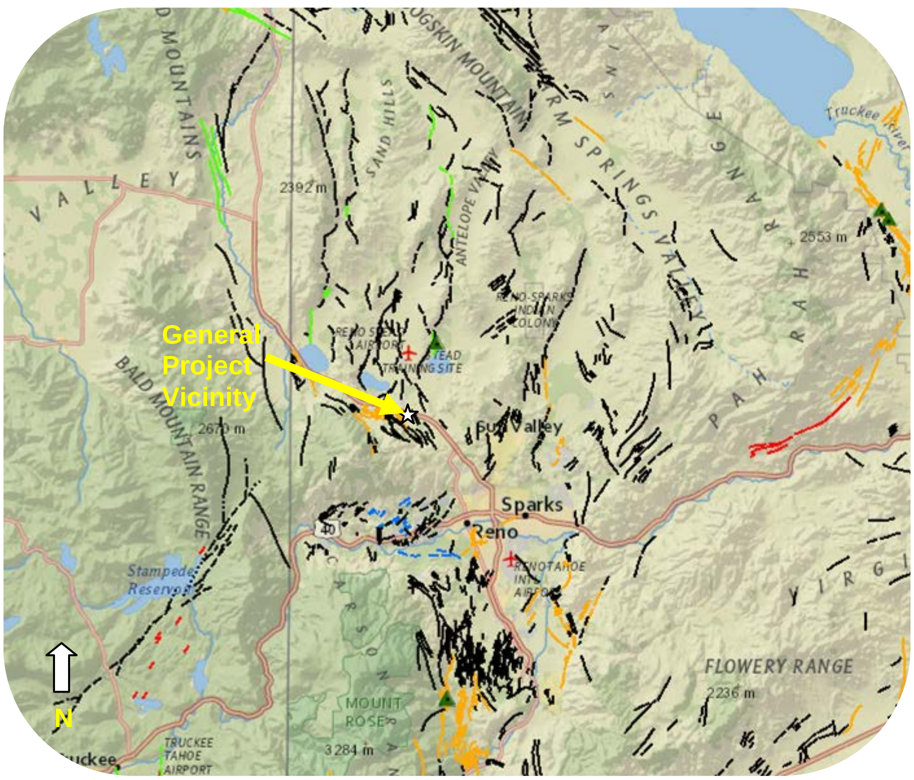


Figure 4: USGS Interactive Fault Map Excerpt (N.T.S)

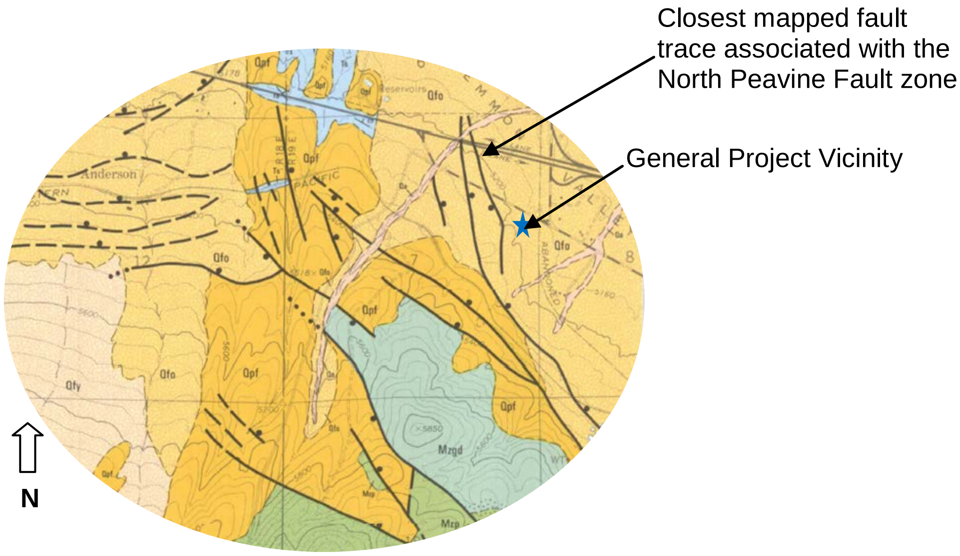


Figure 5: Geologic Map Excerpt Showing Mapped Fault Traces (N.T.S)

Quaternary earthquake fault evaluation criterion has been formulated by a professional committee for the State of Nevada Seismic Safety Council, 2006, 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 to be a fault that does not comply with these age groups. The faults located near the project site are classified as Late Quaternary Active Faults.

Mapped faults in the vicinity of the proposed waterline crossing are located between 200 and 400 feet from the project site. Therefore, based on the anticipated location of the proposed waterline crossing, no fault set backs are required.

7.3 Liquefaction and Lateral Displacement

Liquefaction is 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.

Soils encountered during the subsurface exploration consisted predominately of dense to very dense clayey sands. Groundwater was not encountered during the subsurface exploration, and will likely lie at depths greater than 30 feet. Based on soil conditions encountered, and our experience in the area, it is our professional opinion that the potential for liquefaction at the site is low.

8.0 DISCUSSION AND RECOMMENDATIONS

8.1 General Discussion

The following recommendations shall apply for this project:

- All compaction requirements presented in this report are relative to ASTM D1557[4]; and
- Unless otherwise stated in this report, all related construction should be in general accordance with Standard Specification for Public Works Construction (2016).

The recommendations provided herein, and particularly under **Site Preparation, Grading and Sender and Receiving Pit Backfill,** 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 structure and associated improvements, work together as a system to improve overall performance. If any aspect of this system is ignored or poorly implemented, the performance of the project will suffer. Sufficient construction observation and testing should be performed to document that the recommendations presented in this report are followed.

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.

8.2 Construction and Grading Recommendations

8.2.1 Site Clearing and Preparation

8.2.1.1 Site Clearing

Minor site clearing is anticipated in the vicinity of the proposed receiving pit , as this site is mostly devoid of vegetation. Moderate site clearing will be required for the sending pit located on the south end of the pipe jack alignment. Surface vegetation and topsoil, where encountered, should be stripped and grubbed and stockpiled separately from native soils anticipated to be used as backfill for proposed pit excavations. The stockpiled strippings and topsoil can be used as surface treatment in ***non-structural*** areas or hauled-off and disposed of in an approved location. Stripped and grubbed material ***should not*** be incorporated into structural fill.

Stripping and grubbing depths will depend on the depth of organic material (e.g. plant root region), especially in areas of concentrated tree roots. In general, stripping depths are estimated to be approximately 3 inches with localized deeper zones where low established brush is encountered.

8.2.1.2 Site Preparation

Prior to placement of structural fill, soils shall be scarified 8-inches, moisture conditioned and densified to at least 90 percent relative compaction.

[4] 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 D1557 laboratory test procedure. Optimum moisture content is the corresponding moisture content of the same soil at it maximum dry density.

8.2.2 Grading and Sender and Receiving Pit Backfill

Site grading is anticipated to be minimal. It is recommended, following pipeline installation, the sender and receiving pits be backfilled with densified structural fill. Structural backfill material type will require approval from the Union Pacific Railroad prior to placement at the site. It should be noted that *imported* structural fill may be required and will be determined prior to construction.

It is our opinion that native granular soils with a maximum particle size of 6 inches that is free of debris, vegetation, and organics can be used as structural fill (subject to UPPR approval). Additionally, fine grained soils, as encountered in Boring B-1 could also be used as structural fill, if placed at least 4 feet below finished grade (subject to UPPR approval). This material will be more difficult to moisture condition and densify, the native or import granular material would be preferable over fine grained soils.

Structural fill should be uniformly moisture conditioned within three percent of optimum moisture content, placed in layers of 8 inches or less in loose thickness, and densified to at least 90 percent relative compaction[5]. Moisture contents greater than 3 percent of optimum moisture are acceptable if the soil lift is stable and required relative compaction can be attained in the soil lift and succeeding lifts.

No fill material should be placed, spread or rolled while it is frozen, thawing, or during unfavorable weather conditions.

8.2.3 Trenching and Confined Excavations

All excavations regardless of depth should be evaluated to check the stability prior to occupation by construction personnel. Shoring or sloping of trench walls may be required to protect construction personnel and provide temporary stability.

Where temporary confined excavations are unstable, trench boxes may be used to provide safe ingress and egress for construction personnel.

Excavations should comply with current OSHA safety requirements (Federal Register 29 CFR, Part 1926)[6]. Soils or bedrock are classified as Type A, B or C, which requires different temporary excavation, cut slope gradients (Table 1-Maximum Allowable Temporary Slopes).

[5] A 95 percent relative compaction may be required for sender/receiver pit backfill by the UPPR, and will be determined prior to construction.

[6] Deeper excavations where layered geotechnical strata is encountered, extending into the weathered granodiorite bedrock can be evaluated by the project geotechnical engineer to determine the maximum allowable slope configuration for the layered system.

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

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 (63º) 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 (53º).

Soil types generally are cohesive, but zones of cohesionless soils, anticipated in the upper portion of the soil profile should be anticipated. Consequently, soils encountered within unshored trench sidewalls should be evaluated during construction to determine slope gradients. However, the predominant clayey sand **(SC)** soil type encountered appears to comply with either OSHA Type A or B soils. Intermittent layers of cohesionless granular soils may be encountered are anticipated to comply with OSHA Type C excavation guidelines.

Trench excavations should be protected from surface water/runoff. Temporary drainage swales should be excavated to divert surface flows into a collection area away from the open excavation. If warranted, dewatering of pipe trench excavations can be accomplished by use of a temporary dewatering system.

If subsurface water conditions differ from those encountered during our subsurface exploration, the engineer should be notified immediately to determine if alternative dewatering recommendations are warranted.

8.2.4 Trenchless Construction

The water line will be placed by trenchless construction methods below the existing Stead Spur rail road. The proposed construction will entail the installation of a 30-inch diameter steel sleeve prior to installing the ductile iron pipe.

Trenchless construction methods include pipe jacking, horizontal boring, or directional drilling. Based on our understanding of proposed construction methods, pipe jacking will be the proposed method of installation and will be the primary focus of the trenchless construction recommendations.

8.2.4.1 Pipe Jacking Methods for Small Diameter Steel Casings

This construction method consists of pushing a casing through the ground from a jacking pit. A laser guided boring machine is typically placed in front of the casing and the assembly is pushed by a jack plate. Spoil material is transported from the head of the casing to the drive shaft, where it is removed and disposed. After each pipe segment has been installed, the rams of the jacks are retracted so that another pipe segment can be placed in position for the jacking cycle to begin again. This process is repeated until the trenchless pipeline length has been completed. Because the line and grade can be controlled by excavating in front of the pipeline and laser guided boring machines are used, good control of the direction and grade is possible.

Several pipe jacking methods can be considered for the installation of small diameter steel casing pipe including press-in, auguring, pipe rotation, and slurry. Each of these methods has advantages and disadvantages depending on the contractor's preference and soil conditions.

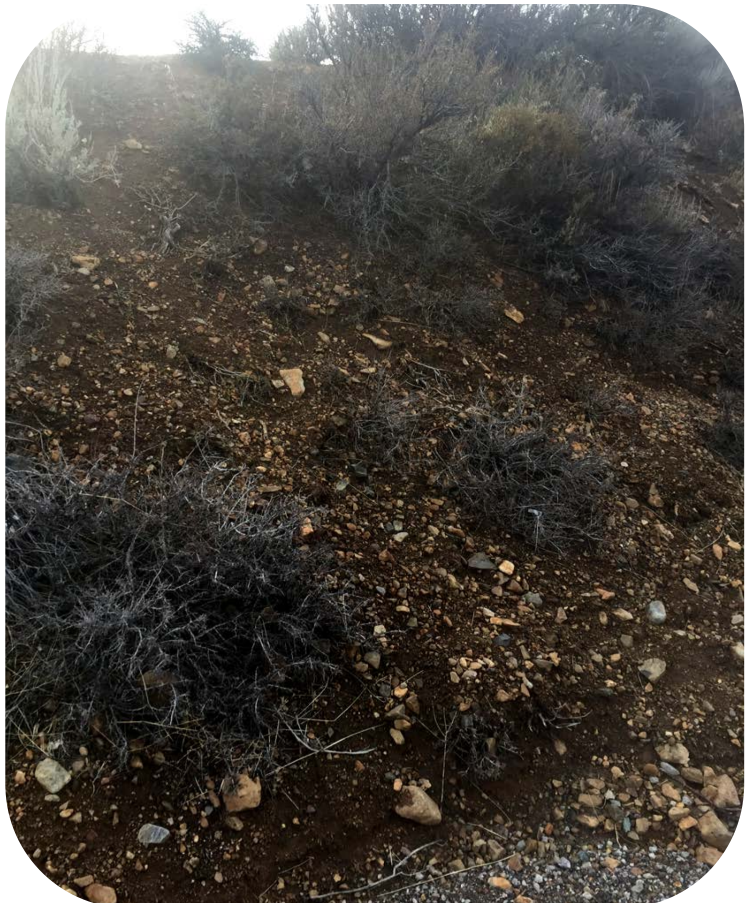
8.2.4.2 Pipe Jacking Method Constraints

As a general rule, cohesive soils free of cobbles and boulders are the most favorable soil types for pipe jacking. In areas where cohesionless granular soils are present or groundwater; pipe jacking methods become more limited as the risk of collapse and instability of the trenchless excavation increases.

It is possible to use pipe jacking in unstable soil conditions by incorporating construction procedures such as effective dewatering techniques; slurry; or other method to assist in stabilizing soils susceptible to caving (i.e. cohesionless sands).

In general soils encountered appear to be granular clayey sand **(SC)**; however, interbedded zones of cohesionless sands may be encountered at intermittent locations.

Additionally, cobbles were encountered at Boring B-2 to a depth of 5 feet. Below this depth clayey sand with gravel sized particles were encountered. In general, it is anticipated that gravels should not create impedance during pipe jacking. However, based on observation of adjacent cut slopes near the project site, cobbles may also be present within the soil stratum. It is recommended that potential contractors observe existing cut slope profiles and determine if observed cobble sized particles will be problematic to pipe jacking, as the percentage of cobble sized particles are difficult to ascertain in exploratory borings.



Photograph 3: Taken of existing cut slope adjacent to Stead Spur, located on the northeast side of N. Virginia approximately 120 feet northeast of the proposed receiving pit.

8.2.4.3 Sending and Receiving Pit Shoring

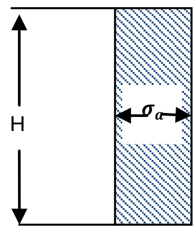
The temporary shoring system shall be designed in accordance with the October 2004 publication entitled, *Guidelines for Temporary Shoring*, by Union Pacific Rail Road Office AVP Engineering-Design and Burlington Northern Railway Santa Fe Assistant Director Structure Design.

The 2004 Guideline provides multiple temporary shoring types including shoring box, anchored sheet pile, anchored soldier beam with lagging, cantilevered sheet pile, cantilever solder beam with lagging, braced excavations, and cofferdam.

A braced temporary shoring system will likely be used for sidewall support for the sending and receiving pits. The 2004 Guidelines provide recommendations to determine railroad live load and lateral forces including active earth pressure and active earth pressure due to surcharge load. The supplied equations do not consider cohesion present in the soil profile.

The shape of active soil pressure loading imposed by the clayey sand soils, can be considered rectangular. The apparent earth pressure distribution can be delineated by the following equation (Terzaghi and Peck, 1969):

$$\sigma_a \left(\frac{psf}{f}\right) = 0.65 * Ka * \gamma * H$$



Pressure Distribution Diagram

Where:

H is the height of the excavation (ft);
γ is the unit weight of soil (pcf);
Ka is the active earth pressure;

It is recommended that an active pressure (σ_a) of 21H (psf/ft) be used for project design.

The pressure distribution does not include surcharge loading occurring at the top of the excavation. It is recommended that equipment, supply loading, or excavated spoils have a minimum horizontal distance away from the top of the trench that is equal to the vertical depth of the trench (1H:1V). However, if it is critical that surcharge loading is closer to the top of the trench, this loading should be evaluated to determine increased sidewall pressures for shoring design.

Because of the distance between the railroad tracks and the proposed locations of the sender and receiver pits it does not appear that loading from railroad cars will exert a surcharge on the pit walls.

The following table (Table 2-General Soil Properties for Shoring Design), provides general soil properties that can be used for project design.

<table>
<tr><th colspan="9">TABLE 2 – General Soil Properties for Shoring Design[1]</th></tr>
<tr><th>Pit</th><th>Predominant Soil Type</th><th>In-Place Dry Unit Weight[2]</th><th>In- Place Moist Unit Weight</th><th>Cohesion[3] (psf)</th><th>Internal Soil Friction Angle[3] φ</th><th>Maximum Dry Density (pcf)</th><th>Optimum Moisture Content</th><th>Recommended Young's Modulus (E)[5]</th></tr>
<tr><td>Receiving Pit (North)</td><td>Clayey Sand (SC))</td><td rowspan="2">122.5</td><td rowspan="2">132.4</td><td rowspan="2">280</td><td rowspan="2">34</td><td>131.0</td><td>9.5%</td><td>1,900 ksf</td></tr>
<tr><td>Sending Pit (South)</td><td>Clayey Sand (SC)</td><td>ND[4]</td><td>ND[4]</td><td>1,400 ksf</td></tr>
</table>

Notes:

1) Soil properties presented in Table 2 may not be representative of all soils present at the site, additional testing may be required during construction if soil conditions vary from those encountered during the subsurface exploration.

2) Unit Weight determined from laboratory testing of a soil sample collected from Boring B-1 at a depth of 7 ½ feet. In-place unit weight may fluctuate depending on the gravel content throughout the soil profile.

3) Cohesion and internal friction angle determined from remolded direct shear completed on a bulk sample of cuttings obtained from a depth of 5 to 10 feet in Boring B-1.

4) ND: Not Determined, a moisture density curve was not completed for these soils.

5) Young's Modulus (E) was determined using empirical equations (Bowels,1996) based on the SPT blow counts corrected for hammer efficiency. Values presented in the table were averaged over the top 15 feet of the soils profile in Borings B-1 and B-2.

8.2.4.4 Backstop Design

The rigid plate used during jack and bore installation, generally referred to as the "backstop", is placed at the back of the sending pit. The backstop is used to distribute the reaction force of jack as it pushes the pipe, along the sidewall of the sending pit. Based on the anticipated depth of the proposed sending pit (between 12 and 14 feet), a passive pressure of 350 psf/foot of depth is recommended for project design.

8.3 Corrosion Test Results

A soil sample from Boring B-1 taken at a depth of 5 to 6 ½ feet and Boring B-2 taken at a depth of 12 ½ to 14 feet, was submitted to Silver State Analytical Laboratories for soil chemistry testing including soluble sulfate testing, pH, chloride, redox potential, and resistivity testing. These tests were completed to determine the potential corrosiveness of the soils to ferrous metal. A brief summary of the results is presented below.

- **Soluble sulfates (ASTM 1580C):** Soluble sulfate test results detected a level of less than 0.02 ppm (parts per million) for each submitted sample, indicating that site soils have a negligible sulfate exposure.

- **pH (SW-846 9045D):** The pH test results ranged from 7.81 to 8.0 indicating the site soils are moderately alkaline and have a moderate to high potential for corrosion with ferrous metal in direct contact with the soil (Baboian, 2005).

- **Chloride (SW-846 9056A):** The chloride content was 23 mg/kg for Boring B-1 sample and less than 10 mg/kg for the Boring B-2 sample. The presence of chloride ions causes the resistivity to be lower.

- **Redox potential (SM 2580 B):** Redox potential measures the value of soil oxidation reduction and is an indicator of soil corrosivity. The value of this soil redox potential depends on the amount of dissolved oxygen present in the pore water of the tested sample. The redox potential was measured at 462 mV to 482 mV indicating the site soils are strongly aerated. In general redox potential greater than 400 mV is considered noncorrosive.

- **Resistivity (ASTM G57):** Resistivity test results ranging from 2,363 to 4,095 (ohms x centimeter) were detected. Resistivity results indicate that the site soils have a high corrosion potential to ferrous metal in direct contact with the soil (Baboian, 2005).

Typically, a 10-point soil evaluation system is used to determine if soils are corrosive to ductile iron piping (Ductile Iron Pipe Research Association, 2005). Soils with 10-points or more generally indicate the site soils are corrosive to iron pipe and corrosion protection is warranted.

The laboratory test having the greatest influence for corrosion potential is resistivity, which provides a measurement of the soil's conductivity potential. The corrosion potential increases as the soil resistivity decreases. Total points for the representative soil samples are presented in Table 3 (Chemical Test Results and Corrosion Potential Total Points).

TABLE 3 – Chemical Test Results and Corrosion Potential Total Points						
Boring Sample Number	**Laboratory Tests**					
	Resistivity (ohm x cm)	**Redox potential (mV)**	**Sulfide**	**pH**	**Field moisture conditions**	**Corrosion Potential Total points**
B-1 (5 to 6 ½ feet)	2,363	462	Not Tested	7.81	moist	2[a]
B-5 (12 ½ to 14 feet)	4,095	482	Not Tested	8.08	moist	1[a]

Notes:

a. Sulfide detection was not included in the corrosion test panel. Based on the overall test results it is likely the total points will fall below the 10-point soil test evaluation for iron pipe based on the samples tested.

Based on the anticipated total points, site soils have mild corrosion potential to iron pipe. However; the soil resistivity is low. A corrosion specialist should be consulted to determine if corrosion protection is warranted.

9.0 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.

10.0 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-1 (Field Exploration Location Map) 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, no responsibility can be assumed for misinterpretation or misapplication of recommendations (contained herein) or their validity in the event changes have been made in the original design concept without the geotechnical engineers 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. CME (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.

REFERENCES

American Society for Testing and Materials (ASTM), 1993, *Soil and Rock; Dimension Stone; Geosynthetics,* Volume 4.08.

Craig M. dePolo*, Quaternary Faults in Nevada, Nevada*, *Map 167,* Nevada Bureau of Mines and Geology (NBMG), *2008.*

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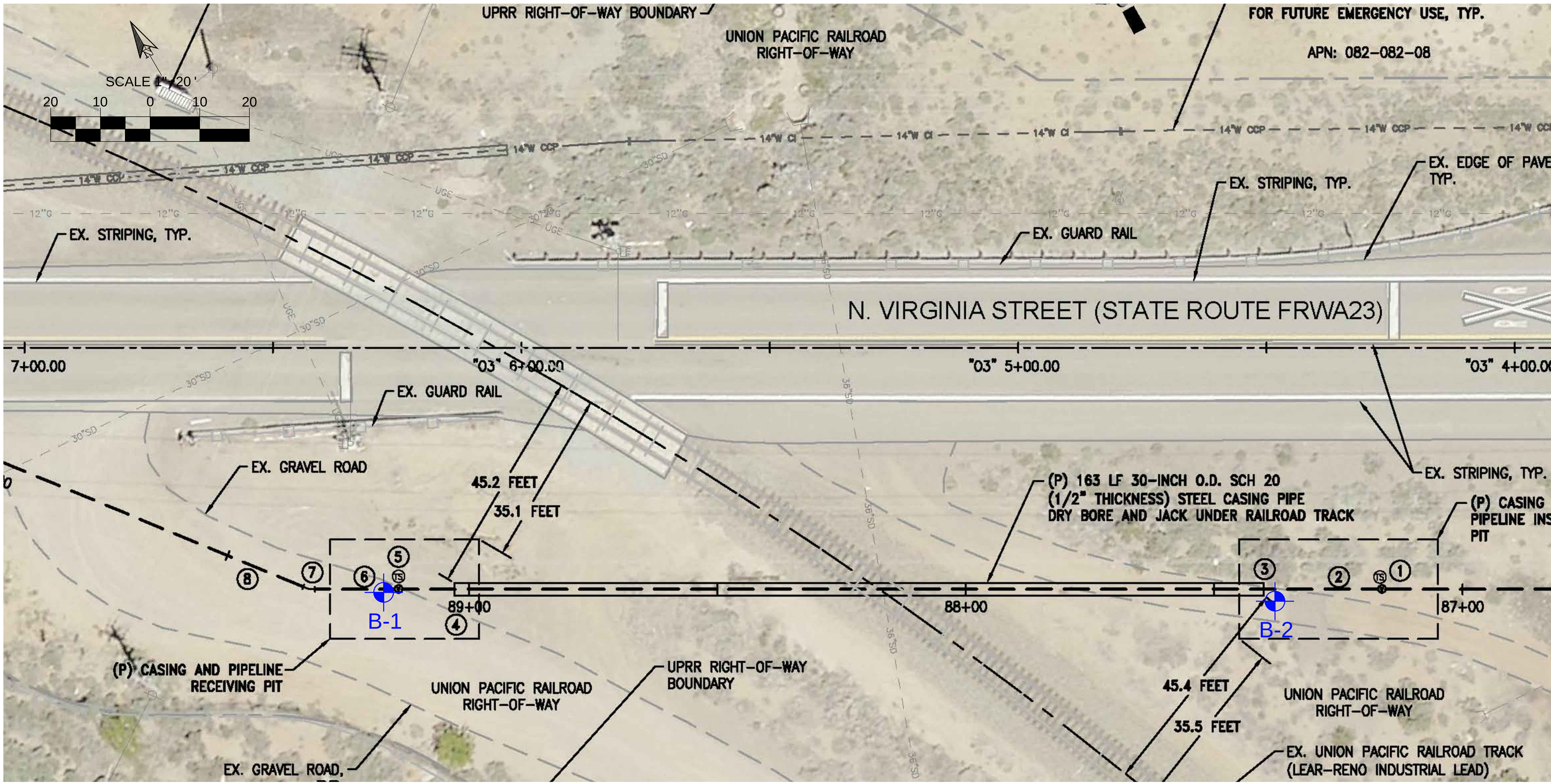
APPENDIX A



REFERENCE: BASE MAP "STEAD MAIN REPLACEMENT PHASE 2, N. VIRGINIA STREET (STATE ROUTE FRWA23), PLAN AND PROFILE, STA 86+65 TO 91+6, SHEET P16" BY TMWA, DATED NOVEMBER 2016

V:\Active\1921\Autocad\site plan.dwg

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6980 Sierra Center Parkway, Suite 90
Reno, NV 89511

TMWA
STEAD MAIN REPLACEMENT PHASE 2
FIELD EXPLORATION LOCATION MAP
UPPR WATERLINE CROSSING

PROJECT NO.: 1921 DATE: 12/13/2016

LEGEND

B-1 APPROXIMATE BORING LOCATION

PLATE

A-1

LOG OF TEST BORING NO. B-1

PROJECT STEAD RAILROAD CROSSING **RIG & BORING TYPE** CME 85
LOCATION AT THE PROPOSED NORTH PIPE JACK PIT (SEE PLATE A-1)
CLIENT: TMWA **DATE** 11/22/2016
PROJECT NO. 1921 **LOGGED BY:** SAM **SURFACE ELEVATION** ≅5,171' (SHEET P16 TMWA)
BLOW COUNTS: Corrected Not Corrected NC **HAMMER TYP.:** AUTOMATIC

Depth in Feet	Unified Soil Classification	Graphic Log	Sample	Sample Type	Sample No.	Blow Counts (SPTs)	Consistency/ Density	Moisture	Visual Description	%-200	Liquid Limit	Plasticity Index	Specific Gravity	Pocket Pen. (tsf)	Dry Density (pcf)	Moisture Content %	Laboratory Tests
0	GP								0-0.25': POORLY GRADED GRAVEL WITH SAND FILL, mostly fine subrounded gravel, little fine to medium sand, brown								
	SC-SM								0.25'-5': SILTY CLAYEY SAND WITH GRAVEL, mostly fine to medium sand, little subangular gravel, yellowish red-brown								
2.5				S	1A	19	MED. DENSE	MOIST									
5	SC			S	1B	89/11	DENSE	MOIST	5'-6': CLAYEY SAND, some fine to medium sand, yellow brown Note: Decomposed andesite visible in sample								
	SC						VERY DENSE	MOIST	6'-7½': CLAYEY SAND WITH GRAVEL, mostly fine to medium sand, little subangular gravel, yellow-brown								
7.5	SC-SM			U	1C	89	VERY DENSE	MOIST	7½'-10': SILTY, CLAYEY SAND WITH GRAVEL, mostly fine to medium sand, little subangular gravel, low plasticity, yellow brown						122.5	8.1	MD
10	CL			S	1D	100/11	HARD	MOIST	10'-15': SANDY LEAN CLAY, some fine to medium sand, little weak subangular gravel, yellowish red-brown Note: Laboratory testing completed on bulk sample of cuttings obtained from a depth of 10 to 15 feet. Uncorrected Max Dry Density - 129.0 pcf @ 10% Rock Corrected Max Dry Density - 131.0pcf @ 9.5%	53.1	25	9				9.4	A, G, MD
12.5				S	1E	77											
15	SC			S	1F	31	DENSE	MOIST	15'-17½': CLAYEY SAND, mostly fine to coarse sand, few subangular gravel, low plasticity, reddish brown								

GROUNDWATER

DEPTH	HOUR	DATE
N.E.		

SAMPLE TYPE

A - Drill Cuttings B - Bulk Sample
R - 3" O.D. 2.42" I.D. Ring Sample
S - 2" O.D. 1.38" I.D. Sampler
U - 3" O.D. 2.42" I.D. Tube Sample
T - 3" O.D. Thin-Walled Shelby Tube

LABORATORY TESTS

A - Atterberg Limits
G - Grain Size
C - Consolidation
MD - Moisture/Density
DS - Direct Shear
TX - Triaxial

PLATE NO.: A-2a

CME CONSTRUCTION MATERIALS ENGINEERS, INC.

LOG OF TEST BORING NO. B-1

PROJECT STEAD RAILROAD CROSSING **RIG & BORING TYPE** CME 85
LOCATION AT THE PROPOSED NORTH PIPE JACK PIT (SEE PLATE A-1)
CLIENT: TMWA **DATE** 11/22/2016
PROJECT NO. 1921 **LOGGED BY:** SAM **SURFACE ELEVATION** ≅5,171' (SHEET P16 TMWA)
BLOW COUNTS: Corrected / Not Corrected: NC **HAMMER TYP.:** AUTOMATIC

Depth in Feet	Unified Soil Classification	Graphic Log	Sample	Sample Type	Sample No.	Blow Counts (SPTs)	Consistency/ Density	Moisture	Visual Description	%-200	Liquid Limit	Plasticity Index	Specific Gravity	Pocket Pen. (tsf)	Dry Density (pcf)	Moisture Content %	Laboratory Tests
17.5	GC			S	1G	52	DENSE	MOIST	17½'-20': CLAYEY SAND WITH GRAVEL, some subangular gravel, little fine to coarse sand, greyish red-brown								
20	SC			S	1H	38	DENSE	MOIST	20'-21½': CLAYEY SAND, mostly fine to medium sand, few subangular gravel, low plasticity, reddish brown								
22.5									TERMINATED AT 21½ FEET, NO FREE WATER ENCOUNTERED NOTE: BULK SAMPLES OF AUGER CUTTINGS COLLECTED AT DEPTHS OF: 0'-5' 5'-10' 10'-15'								
25																	
27.5																	
30																	
32.5																	

GROUNDWATER

DEPTH	HOUR	DATE
N.E.		

SAMPLE TYPE

A - Drill Cuttings B - Bulk Sample
R - 3" O.D. 2.42" I.D. Ring Sample
S - 2" O.D. 1.38" I.D. Sampler
U - 3" O.D. 2.42" I.D. Tube Sample
T - 3" O.D. Thin-Walled Shelby Tube

LABORATORY TESTS

A - Atterberg Limits
G - Grain Size
C - Consolidation
MD - Moisture/Density
DS - Direct Shear
TX - Triaxial

PLATE NO.: A-2a

CME CONSTRUCTION MATERIALS ENGINEERS, INC.

LOG OF TEST BORING NO. B-2

PROJECT STEAD RAILROAD CROSSING **RIG & BORING TYPE** CME 85
LOCATION AT THE PROPOSED SOUTH PIPE JACK PIT (SEE PLATE A-1)
CLIENT: TMWA **DATE** 11/22/2016
PROJECT NO. 1921 **LOGGED BY:** SAM **SURFACE ELEVATION** ≅5,167' (SHEET P16 TMWA)
BLOW COUNTS: Corrected Not Corrected NC **HAMMER TYP.:** AUTOMATIC

Depth in Feet	Unified Soil Classification	Sample Type	Sample No.	Blow Counts (SPTs)	Consistency/ Density	Moisture	Visual Description	%-200	Liquid Limit	Plasticity Index	Specific Gravity	Pocket Pen. (tsf)	Dry Density (pcf)	Moisture Content %	Laboratory Tests
0	SM					MOIST	0'-2': SILTY SAND WITH GRAVEL AND COBBLES, mostly fine to medium sand, little subangular to subrounded gravel and cobbles, dark brown								
2.5	GP	S	2A	80	VERY DENSE	MOIST	2'-5': POORLY GRADED GRAVEL AND COBBLES WITH SAND, some fine to coarse gravel, little fine to coarse sand, possible cobble size particles, grey brown								
5	SC-SM	S	2B	29	MED. DENSE to	MOIST	5'-12½': SILTY, CLAYEY SAND WITH GRAVEL, mostly fine to medium sand, little weak subangular gravel, low plasticity, yellowish red-brown								
7.5		S	2C	37	DENSE			39.0	25	5				10.4	A, G
10		U	2D	47			NOTE: No sample recovery at 10 feet, catcher added to sampler								
12.5	SC	S	2E	27	MED. DENSE	MOIST	12½'-15': CLAYEY SAND, mostly fine to coarse sand, few subangular gravels, visible decomposed andesite, yellowish-reddish brown								
15	SC	S	2F	37	DENSE	MOIST	15'-21½': CLAYEY SAND WITH GRAVEL, mostly fine to coarse sand, little angular gravel, yellow-grey brown								

GROUNDWATER

DEPTH	HOUR	DATE
N.E.		

SAMPLE TYPE

A - Drill Cuttings B - Bulk Sample
R - 3" O.D. 2.42" I.D. Ring Sample
S - 2" O.D. 1.38" I.D. Sampler
U - 3" O.D. 2.42" I.D. Tube Sample
T - 3" O.D. Thin-Walled Shelby Tube

LABORATORY TESTS

A - Atterberg Limits
G - Grain Size
C - Consolidation
MD - Moisture/Density
DS - Direct Shear
TX - Triaxial

PLATE NO.: A-2b

CME CONSTRUCTION MATERIALS ENGINEERS, INC.

LOG OF TEST BORING NO. B-2

PROJECT STEAD RAILROAD CROSSING **RIG & BORING TYPE** CME 85
LOCATION AT THE PROPOSED SOUTH PIPE JACK PIT (SEE PLATE A-1)
CLIENT: TMWA **DATE** 11/22/2016
PROJECT NO. 1921 **LOGGED BY:** SAM **SURFACE ELEVATION** ≅5,167' (SHEET P16 TMWA)
BLOW COUNTS: Corrected Not Corrected NC **HAMMER TYP.:** AUTOMATIC

Depth in Feet	Unified Soil Classification	Graphic Log	Sample	Sample Type	Sample No.	Blow Counts (SPTs)	Consistency/ Density	Moisture	Visual Description	%-200	Liquid Limit	Plasticity Index	Specific Gravity	Pocket Pen. (tsf)	Dry Density (pcf)	Moisture Content %	Laboratory Tests
17.5																	
				S	2G	25				39.9	26	10				12.7	A, G
20																	
				S	2H	45											
									TERMINATED AT 21½ FEET, NO FREE WATER ENCOUNTERED. NOTE: BULK SAMPLE OF AUGER CUTTINGS COLLECTED AT A DEPTH OF 5'-10'								
22.5																	
25																	
27.5																	
30																	
32.5																	

GROUNDWATER

DEPTH	HOUR	DATE
N.E.		

SAMPLE TYPE

A - Drill Cuttings B - Bulk Sample
R - 3" O.D. 2.42" I.D. Ring Sample
S - 2" O.D. 1.38" I.D. Sampler
U - 3" O.D. 2.42" I.D. Tube Sample
T - 3" O.D. Thin-Walled Shelby Tube

LABORATORY TESTS

A - Atterberg Limits
G - Grain Size
C - Consolidation
MD - Moisture/Density
DS - Direct Shear
TX - Triaxial

PLATE NO.: A-2b

CME CONSTRUCTION MATERIALS ENGINEERS, INC.

UNIFIED SOIL CLASSIFICATION CHART

COARSE-GRAINED SOILS
(more than 50% of material is larger than No. 200 sieve size.)

GRAVELS More than 50% of coarse fraction larger than No. 4 sieve size	Clean Gravels (Less than 5% fines)	
	GW	Well-graded gravels, gravel-sand mixtures, little or no fines
	GP	Poorly-graded gravels, gravel-sand mixtures, little or no fines
	Gravels with fines (More than 12% fines)	
	GM	Silty gravels, gravel-sand-silt mixtures
	GC	Clayey gravels, gravel-sand-clay mixtures
SANDS 50% or more of coarse fraction smaller than No. 4 sieve size	Clean Sands (Less than 5% fines)	
	SW	Well-graded sands, gravelly sands, little or no fines
	SP	Poorly graded sands, gravelly sands, little or no fines
	Sands with fines (More than 12% fines)	
	SM	Silty sands, sand-silt mixtures
	SC	Clayey sands, sand-clay mixtures

FINE-GRAINED SOILS
(50% or more of material is smaller than No. 200 sieve size.)

SILTS AND CLAYS Liquid limit less than 50%	ML	Inorganic silts and very fine sands, rock flour, silty of clayey fine sands or clayey silts with slight plasticity
	CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
	OL	Organic silts and organic silty clays of low plasticity
SILTS AND CLAYS Liquid limit 50% or greater	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, organic silts
HIGHLY ORGANIC SOILS	PT	Peat and other highly organic soils

ESTIMATED PERCENTAGES OF GRAVEL, SAND, AND FINES BASED ON VISUAL DESCRIPTION

TRACE	<5%
FEW	5%-15%
LITTLE	15%-30%
SOME	30%-50%
MOSTLY	>50%

CORRELATION OF PENETRATION RESISTANCE WITH RELATIVE DENSITY

SAND AND GRAVEL		SILT AND CLAY	
NO. OF BLOWS	RELATIVE DENSITY	NO. OF BLOWS	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
		OVER 31	HARD

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TMWA
STEAD MAIN REPLACEMENT PHASE 2
SOIL CLASSIFICATION CHART
UPPR WATERLINE CROSSING

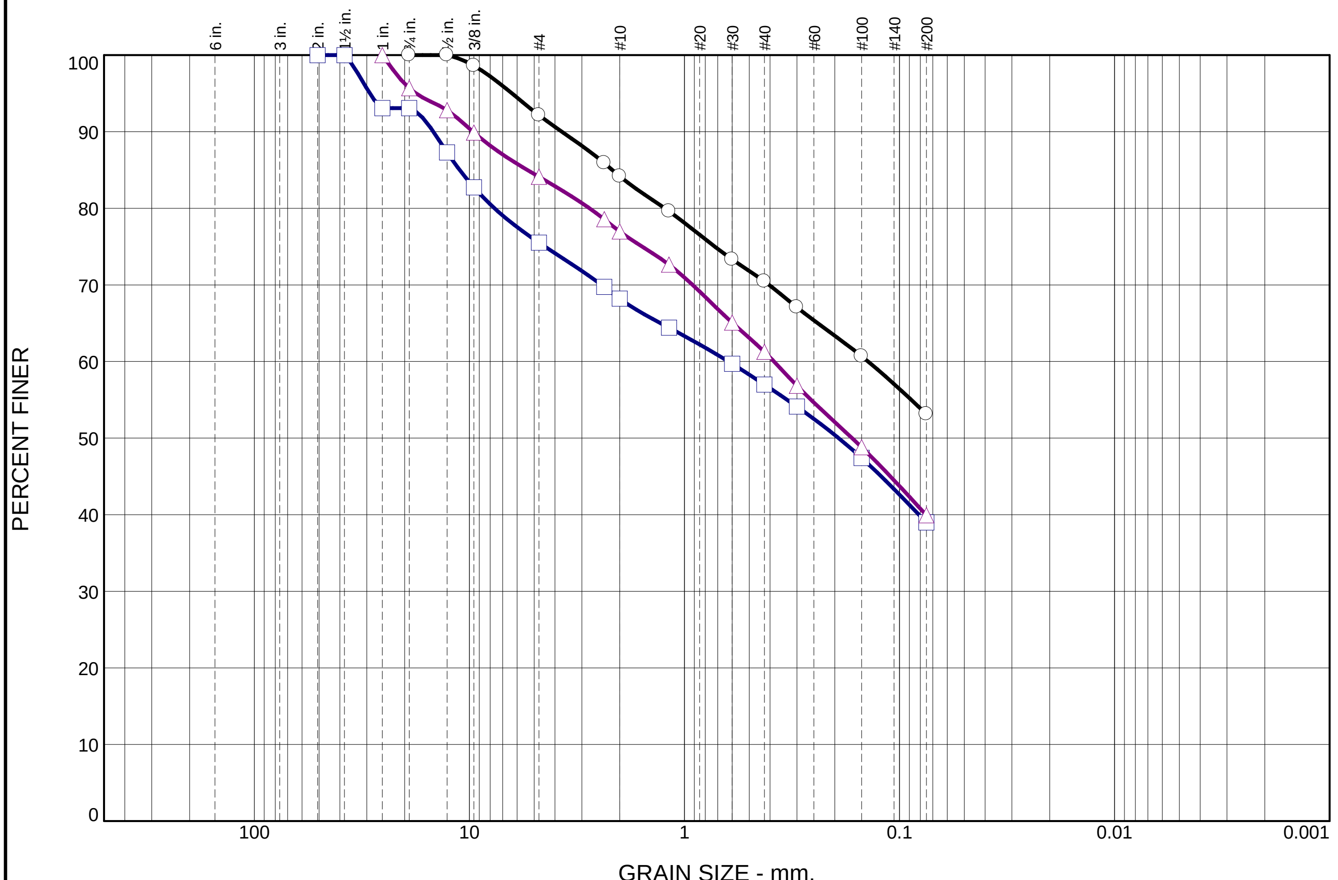
PROJECT NO.:1921 DATE: 12/13/2016

PLATE
A-3

CONSTRUCTION MATERIALS ENGINEERS, INC.

APPENDIX B

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	0.0	7.8	8.0	13.7	17.4	53.1	
□	0.0	6.9	17.6	7.3	11.2	18.0	39.0	
△	0.0	4.3	11.6	7.1	15.7	21.4	39.9	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	25	16	2.1637	0.1399						
□	25	20	11.0966	0.6248	0.1920					
△	26	16	5.3713	0.3837	0.1662					

MATERIAL DESCRIPTION	TEST DATE	USCS	NM
○ sandy lean clay	11/30/2016	CL	9.4
□ silty, clayey sand with gravel	11/30/2016	SC-SM	10.4
△ clayey sand with gravel	11/30/2016	SC	12.7

Project No. 1921 **Client:** TMWA

Project: STEAD RAILRAOD CROSSING

○ **Location:** B-1 , 10'-15' **Sample Number:** 30971

□ **Location:** B-2, 2C, 7.5'-9.0' **Sample Number:** 30971

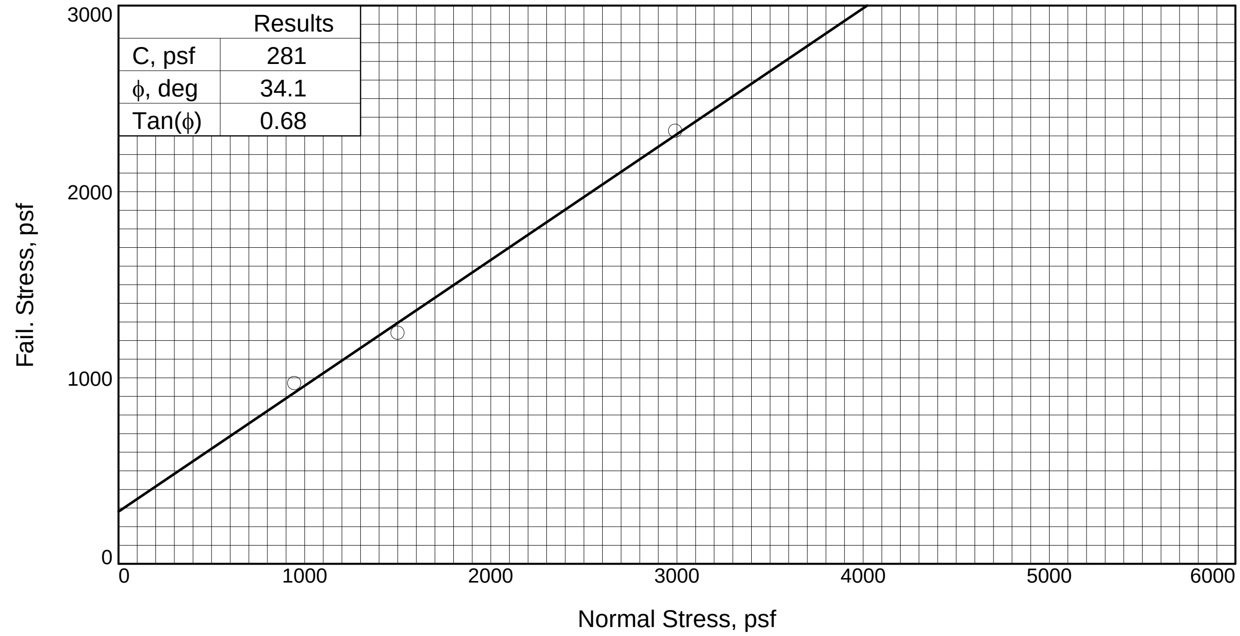
△ **Location:** B-2, 2G, 12.5'-14.0' **Sample Number:** 30971

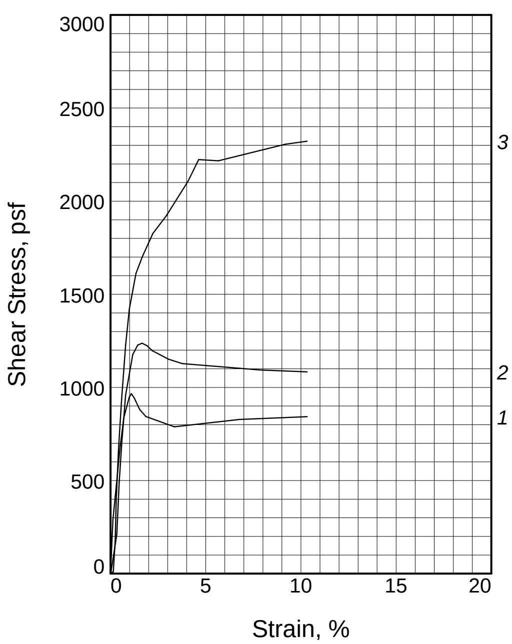
CME CONSTRUCTION MATERIALS ENGINEERS, INC.

Remarks:

Figure B-1

Tested By: ○ A. HAMPEL □ A. HAMPEL △ A. Hampel **Checked By:** S. HEIN





Sample No.		1	2	3
Initial	Water Content, %	8.1	8.1	8.1
	Dry Density, pcf	120.1	122.1	121.8
	Saturation, %	60.0	63.9	63.2
	Void Ratio	0.3511	0.3298	0.3331
	Diameter, in.	2.42	2.42	2.42
	Height, in.	1.03	1.01	1.01
At Test	Water Content, %	13.3	12.1	11.9
	Dry Density, pcf	120.7	123.3	123.8
	Saturation, %	99.8	99.5	99.6
	Void Ratio	0.3452	0.3166	0.3115
	Diameter, in.	2.42	2.42	2.42
	Height, in.	1.03	1.00	0.99
Normal Stress, psf		949	1504	2995
Fail. Stress, psf		966	1237	2322
Strain, %		1.1	1.7	10.3
Ult. Stress, psf				
Strain, %				
Strain rate, in./min.		0.00	0.00	0.00

Sample Type: REMOLDED
Description:

Assumed Specific Gravity= 2.60
Remarks:

Figure B-2

Client: TMWA

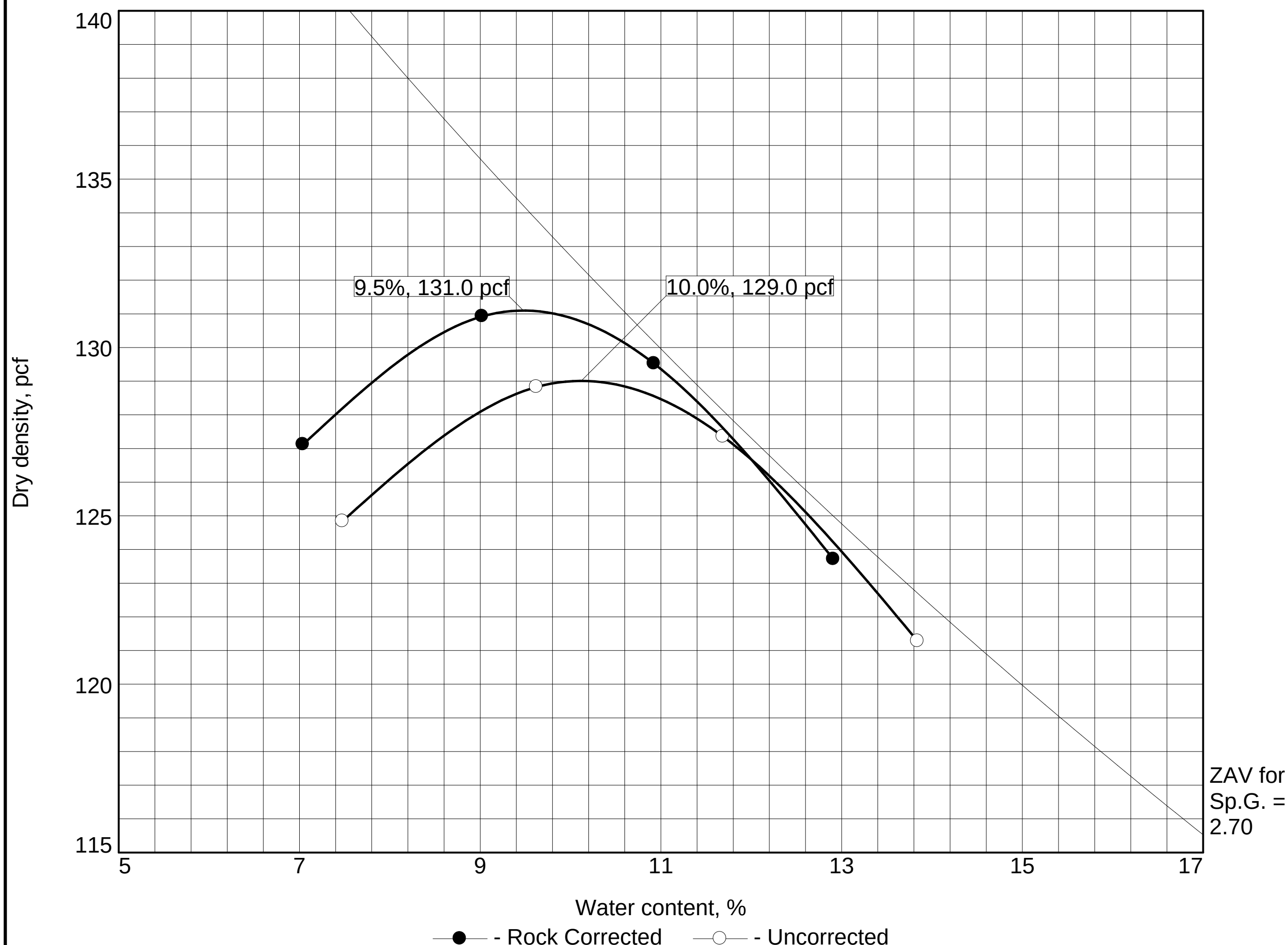
Project: STEAD RAILRAOD CROSSING

Location: B-2, 5'-10',
Sample Number: 30971
Proj. No.: 1921 **Date Sampled:** 11/30/2016

CME CONSTRUCTION MATERIALS ENGINEERS, INC.

Tested By: S. HEIN **Checked By:** S. MONTALVO

COMPACTION TEST REPORT



Test specification: ASTM D 1557-07 Method A Modified
ASTM D 4718-87 Oversize Corr. Applied to Each Test Point

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > #4	% < No.200
	USCS	AASHTO						
	CL	A-4(2)	9.4		25	9	7.8	53.1

ROCK CORRECTED TEST RESULTS	UNCORRECTED	MATERIAL DESCRIPTION
Maximum dry density = 131.0 pcf	129.0 pcf	sandy lean clay
Optimum moisture = 9.5 %	10.0 %	

Project No. 1921 **Client:** TMWA
Project: STEAD RAILRAOD CROSSING
Date: 12/2/2016
○ **Location:** B-1 , 10'-15' **Sample Number:** 30971

CME CONSTRUCTION MATERIALS ENGINEERS, INC.

Remarks:

Figure B-3

Tested By: G. MORALES **Checked By:** S. HEIN

SilverState Analytical Laboratories

Sierra Environmental Monitoring

EnviroTech.

Laboratory Report

Report ID: 151786

CME-Construction Materials Engineers, Inc		**Date:**	12/9/2016
Attn: Stella Montalvo		**Client:**	CON-160418
69800 Sierra Center Parkway, Suite 90		**Taken by:**	S. Montalvo
Reno, Nevada 89511		**PO #:**	

Analysis Report

Laboratory Accreditation Number: NV-00015

Laboratory Sample ID	Customer Sample ID	Date Sampled	Time Sampled	Date Received
S201612-0082	Boring B-1, sample # 1B, Depth 5-6.5'	11/22/2016		12/2/2016

Parameter	Method	Result	Units	Reporting Limit	Analyst	Date Analyzed	Data Flag
Chloride - Ion Chromatography	SW-846 9056A	23	mg/Kg	10	Faulstich	12/8/2016	
pH - Saturated Paste	SW-846 9045D	7.81	pH Units		Bergstrom	12/8/2016	
pH - Temperature	SW-846 9045D	21.2	°C		Bergstrom	12/8/2016	
Redox Potential	SM 2580 B	462	MV		Bergstrom	12/8/2016	
Resistivity ASTM	ASTM G57	2363	ohm cm		Bergstrom	12/8/2016	
Sulfate ASTM 1580C	ASTM 1580C	<0.02	%	0.02	Bergstrom	12/8/2016	

Laboratory Accreditation Number: NV-00015

Laboratory Sample ID	Customer Sample ID	Date Sampled	Time Sampled	Date Received
S201612-0083	Boring B-2, sample # 2E, Depth 12.5-14'	11/22/2016		12/2/2016

Parameter	Method	Result	Units	Reporting Limit	Analyst	Date Analyzed	Data Flag
Chloride - Ion Chromatography	SW-846 9056A	<10	mg/Kg	10	Faulstich	12/8/2016	
pH - Saturated Paste	SW-846 9045D	8.08	pH Units		Bergstrom	12/8/2016	
pH - Temperature	SW-846 9045D	21.7	°C		Bergstrom	12/8/2016	
Redox Potential	SM 2580 B	482	MV		Bergstrom	12/8/2016	
Resistivity ASTM	ASTM G57	4095	ohm cm		Bergstrom	12/8/2016	
Sulfate ASTM 1580C	ASTM 1580C	<0.02	%	0.02	Bergstrom	12/8/2016	

Data Flag Legend:

V:\Active\1921\Autocad\soil classification chart and rock description.dwg

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Reno, NV 89511

TMWA
STEAD MAIN REPLACEMENT PHASE 2
CORROSION TEST RESULTS
UPPR WATERLINE CROSSING

PROJECT NO.:1921 DATE: 12/13/2016

PLATE

B-4