GEOTECHNICAL INVESTIGATION
TMWA VERDI WATER MAIN EXTENSION
WASHOE COUNTY, NEVADA





CME CONSTRUCTION MATERIALS ENGINEERS, INC.




PREPARED FOR:

SHAW ENGINEERING

JULY 2018
FILE: 2008

CME CONSTRUCTION MATERIALS ENGINEERS, INC.
300 Sierra Manor Drive, Suite 1
Reno, NV 89511

July 17, 2018
File: 2008

Steve Brigman, PE
SHAW ENGINEERING
20 Vine Street
Reno, NV 89503

RE: Geotechnical Investigation- TMWA Verdi Watermain Extension
Verdi, Washoe County, Nevada

Dear Mr. Brigman:

Enclosed is our geotechnical investigation report for the proposed TMWA Verdi Watermain Extension, located in Verdi, Nevada.

The following geotechnical investigation report includes the results of our field and laboratory investigations and presents our recommendations for the design and construction of the project. We hope the attached report provides you with the information you require.

If you have any questions please contact the undersigned.

Sincerely,

CONSTRUCTION MATERIALS ENGINEERS, INC.

Randal A. Reynolds, PE
Senior Geotechnical Engineer
rreynolds@cmenv.com
Direct: 775-737-7576
Cell: 775-527-3264

Stella A. Montalvo, PE
Geotechnical Project Manager
RE Number 21801
Expiration Date: 12-31-19
smontalvo@cmenv.com
Direct: 775-737-7569

PROFESSIONAL ENGINEER - STATE OF NEVADA
STELLA A. MONTALVO
Exp. 12-31-2019
CIVIL
No. 021801

7-17-2018

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1.0 INTRODUCTION

The proposed Verdi Water Main Extension will include installation of approximately 4,800 linear feet of waterline adjacent to Highway 40, in Verdi, Washoe County, Nevada. The project site is completely contained in Section 8, Township 19N, Range 18E, MDM. The general project vicinity is included as Figure 1 (Vicinity Map).

Figure 1: Vicinity Map
(Reference: Base Map is from City of Reno GIS, accessed July 2017, http://cityofreno.maps.arcgis.com/apps/MapTools)

The recommendations provided in this report 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 investigate the general soil and groundwater conditions at the subject site and provide geotechnical recommendations for design and construction of the project.

The area covered by this report is included as Plate A-1 (Field Exploration Location Map) in Appendix A and on Figure 2 (Approximate Limits of Project). Our study included subsurface field exploration, geophysical field measurements, laboratory testing, and engineering analysis 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.

Figure 2: Approximate Limits of Project
(Reference: Base map obtained at http://cityofreno.maps.arcgis.com/apps/MapTools)

2.0 PROJECT DESCRIPTION AND SITE CONDITIONS

2.1 Project Description

The project includes construction of approximately 4,800 linear feet of 18-inch diameter ductile iron waterline. The depth of pipe installation ranges from 5 feet to 8 feet. The deeper installation depths consists of the pipe being routed through an existing sleeve below the Truckee River, which was installed during previous construction work outside the scope of this project.

The waterline alignment is presented on Figure 2. A description of the waterline alignment is presented in Section 2.1.1.

2.1.1. Waterline Alignment Description

- The waterline begins on the northside of Highway 40, approximately 800 feet west-southwest of Milepost 4.27.
- The waterline alignment crosses the highway at Milepost 4.27 (northwest corner of the Verdi Postal Service parcel) and continues westerly along the southern shoulder of the highway for approximately 1,500 feet. The waterline then angles in a southerly direction for 300 feet along the west bank of the Truckee River before crossing the river.

- At the east end of the river crossing, the waterline alignment angles in a northerly direction along the west edge of Riveredge Drive for approximately 300 feet to the Stoneridge Drive intersection, where it angles in a westerly direction along the south edge of Stoneridge Drive.
- At the Fallbrook Drive intersection, the waterline angles in a northerly direction and crosses Highway 40. The waterline continues in a westerly direction along the shoulder of the highway for a distance of about 1000 feet before terminating at the future entrance road into the proposed West Meadows Residential Development.

2.2 Site Conditions

A majority of the waterline alignment will be located adjacent to Highway 40 within the Nevada Department of Transportation (NDOT) Right-of-Way.

Photograph 1 (left) and 2 (right): Photograph 1 taken looking westerly at Milepost 4.27 south edge of Highway 40 (left), Photograph 2 taken looking east at the River Bell Mobile Home Park west entrance near Boring BH-2.

Roadway shoulders are mostly unpaved, a shallow swale is located on the north side of Highway 40, and gently slops to the south. Several utilities are located within the southern roadway shoulder including gas, fiber optic/communications, and overhead electricity. A subsurface gas main trends parallel to the pavement along the northside of the highway. Additional utilities may be encountered along the alignment, especially in the vicinity of the Truckee River Crossing.

Photograph 3: Looking easterly on the north side of Highway 40, near the West Meadows Development. Note deepened (±3 feet) swale and adjacent side slope. This area may be regraded once the residential development is complete, bringing the proposed finished grade within a few feet of the roadway elevation.

Vegetation along the alignment generally consist of sparse grass and shrubs. Trees are located on the west end of the proposed alignment north of Highway 40.

3.0 GEOTECHNICAL EXPLORATION INFORMATION

3.1 Field Exploration

The site was explored in June 2017 by drilling five test borings to depths of 10 feet below the existing ground surface (bgs). The test boring locations were restricted due to exploration permission limitations within the Nevada Department of Transportation (NDOT) right-of-way, nearby privately owned and managed communities, and the presence of existing subsurface infrastructure.

Borings were performed using two exploration methodologies: Air knife excavation and rotary-vibratory (sonic) drilling techniques.

Photograph 4: Air Knife Potholing at Boring BH-2, note rounded cobble cuttings removed by hand, sands and smaller gravels are removed by vacuumed and discharged into the vacuum truck storage tank.

The upper 4 to 5 feet of boreholes located adjacent to Highway 40 were excavated using air knife vacuum equipment to ensure no utility conflicts such as existing fiber optic lines were present below the proposed exploration location.

Following air knife excavations, sonic drilling was performed to the depth of exploration. Sonic drilling utilizes an override casing with interior core barrel drilling system that engages high frequency mechanical vibration to obtain continuous soil core samples. The override casing has an exterior diameter of about 6 inches and interior core barrel diameter of approximately 5 ½ inches. Bulk samples of the subsurface soils were collected from the interior casing and bagged in the field to reduce moisture loss.

Photograph 5: Sonic drilling at Boring BH-4 completed adjacent to a residential development currently under construction located on the north side of Highway 40. Note the large boulder spoils from the nearby grading activities.

Soils were sampled on a continuous basis to the depth of termination (as noted on the boring logs). The approximate boring locations are shown on Plate A-1 (Field Exploration Location Map). Upon completion of laboratory testing, additional soil classification and verification of the field classifications were subsequently performed in accordance with the Unified Soil Classification System (USCS), as presented in ASTM D 2487. A description of the USCS is presented on Plate A-3. Boring logs are included in Appendix A as Plate A-2 (Boring Log).

4.0 LABORATORY TESTING

All soil testing performed in CME's soils laboratory is conducted in accordance with the standards and methodologies described in Volume 4.08 (Soil and Rock) 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:

- *Insitu* moisture (ASTM D2216) (Appendix A);
- Grain size distribution (ASTM C136) (Appendix B);
- Plasticity index (ASTM D4318) (Appendix A); and
- Moisture Density Curve (ASTM D1557) (Appendix B)

In addition, our firm contracted with an outside laboratory to complete the following analytical testing for the corrosion potential of the site soils:

- Resistivity (ASTM G-57);
- Paste pH (SW-846 9045D);
- Soluble Sulfates (ASTM 1580C);
- Redox Potential (SM 2580B);
- Sulfides (AWWA C105); and
- Chlorides (EPA 300.0).

Laboratory test results are included on the boring logs (Plates A-2) and in Appendix B.

5.0 GEOLOGIC AND GENERAL SOIL PROFILE DESCRIPTIONS

Based on a review of the referenced geologic map (Figure 3-Geologic Map Excerpt). The project site is underlain by Holocene to late Pleistocene terrace and outwash deposits of the Truckee River. These deposits are described as a brownish gray to brown sandy cobble to boulder gravel and gravelly sand (Bell & Garside, 1987).

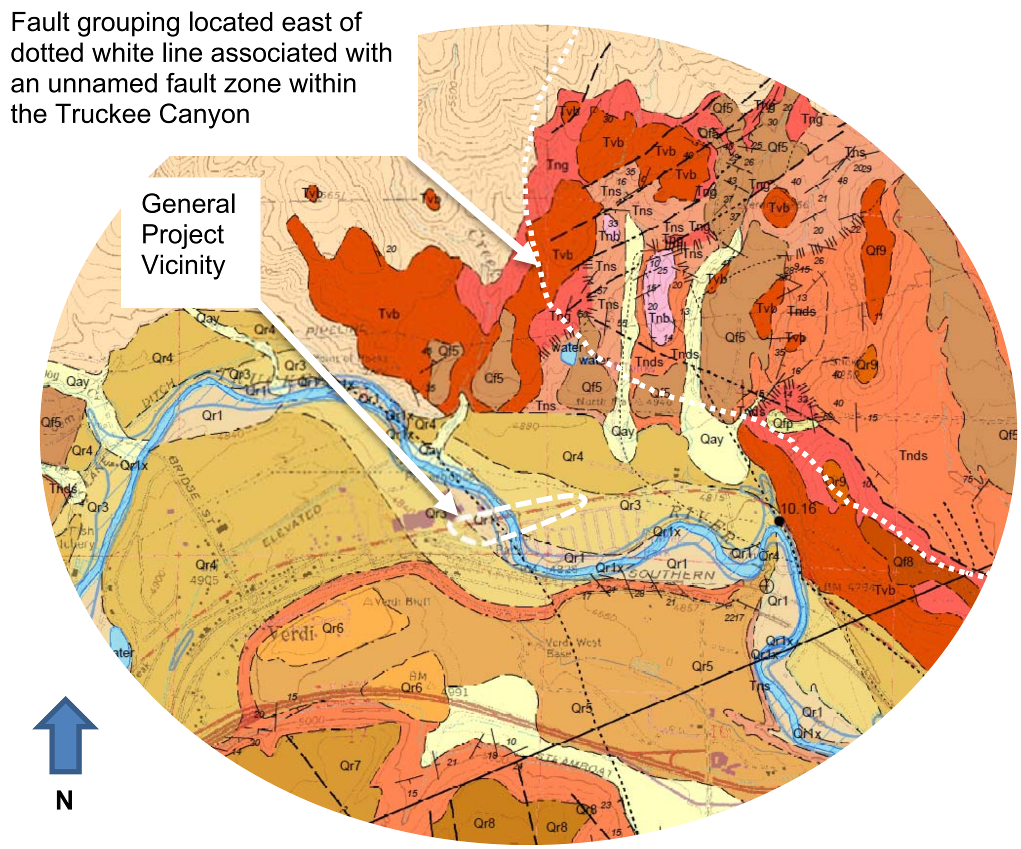

Figure 3: Geologic Map Excerpt (N.T.S)
Reference: Base Map *Preliminary Revised Geologic Maps of the Reno Urban Area, Nevada, Plate 1*, by Ramelli, et al., 2011

5.1 General Soil Profile and Consistency

Soils encountered were relatively consistent with the mapped geology. The predominate soil types encountered are granular materials consisting of clayey gravel with sand and cobbles (**GC**), poorly graded sand with silt, gravels and cobbles (**SP**), poorly graded gravel with clay and sand (**GP-GC**), and silty sand with gravel and cobbles (**SM**). Cobbles up to 12-inch nominal diameter were encountered. Based on the geologic profile and noted surface boulders, many of these boulders are large exceeding 4 feet in diameter, boulders are likely to be encountered during construction.

An atypical soil profile was encountered in Boring BH-4 and consisted on an upper most silty sand with gravel and cobbles (**SM**) layer underlain by a thick layer of sandy elastic silt (**MH**) encountered from a depth of 4 to 10 feet (i.e. depth of exploration) below the existing ground surface (bgs). This layer was not encountered within the other exploratory boings. Due to the limited number of subsurface test borings completed, the horizontal extent of these fine-grained soils is unknown.

5.2 Soil Moisture and Groundwater Conditions

Granular soils were in a slightly moist to moist condition with soil moisture contents ranging from about 3 to 5 percent.

A significantly higher moisture content of approximately 42 percent was measured in the sandy elastic silt layer encountered in BH-4. Comparing this moisture content to the moisture density curve relationship completed on these soils the measured moisture content is is about 6 percent over optimum moisture content, which indicates that these soils may be prone to instability during construction.

In areas where elastic silt or other clay/fine-grained soils are present, higher moisture contents should be anticipated, especially in areas where these soils are located below the pavement areas. The high moisture content in combination with the fine-grained nature of these soils will promote instabilities during construction. If encountered, these unstable areas can be mitigated by following the recommendations in Section 8.2.4.1.

The groundwater table was not encountered during the subsurface exploration. However, it should be noted that fluctuations in groundwater may occur due to increased irrigation, precipitation, during spring runoff, and in areas within close proximity to the Truckee River.

6.0 SEISMIC AND GEOLOGIC HAZARDS

6.1 Seismicity

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

Figure 4: 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 scarp of the Sierra Nevada Mountains, which is the western margin of the province. The subject site is located at the north terminus of the Carson Range, southeast of the Verdi Range, in a seismically active zone within the western extreme of the Basin and Range.

6.2 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*;
2) The USGS Interactive Fault Map, (http://earthquake.usgs.gov/hazards/qfaults/map/); and
3) Quaternary Faults in Nevada, (dePolo, 2008), excerpt included as Figure 5 (Excerpt from the NBMG Interactive Fault Map).

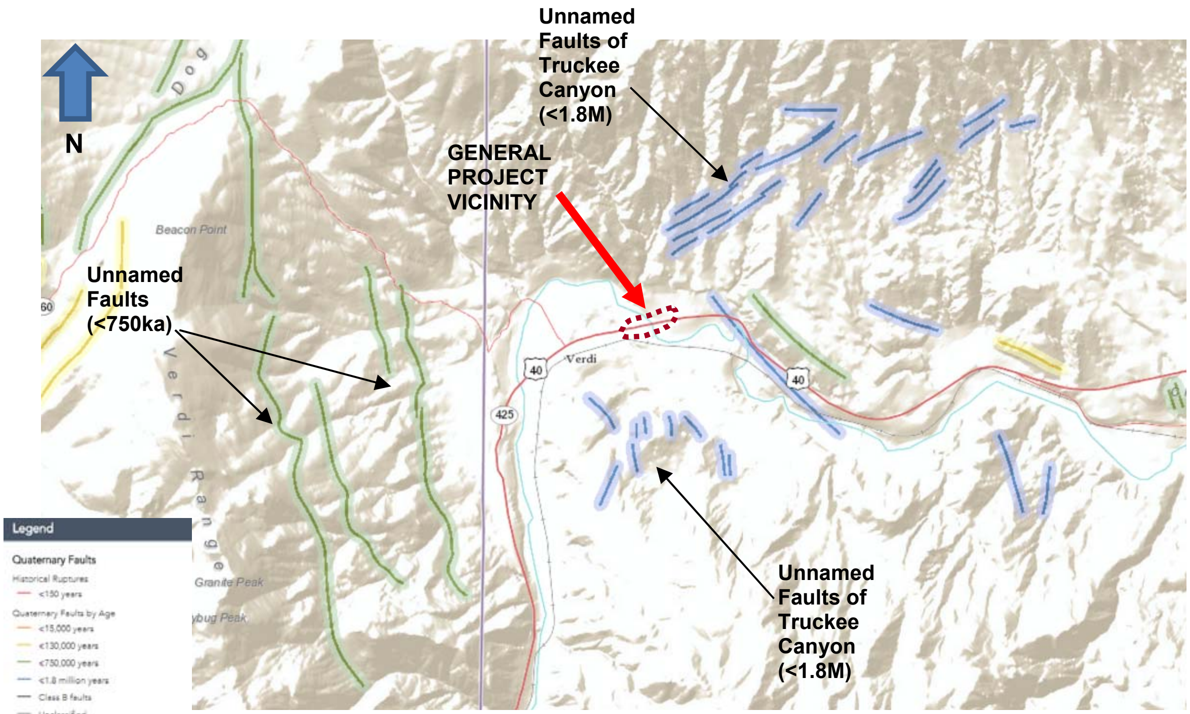

Figure 5: Excerpt from the NBMG Interactive Fault Map Application (N.T.S)
(Reference: Quaternary Faults in Nevada, NBMG, https://gisweb.unr.edu/QuaternaryFaults/[1])

No mapped faults are located traversing through the project site. The closest published faults to the site are located about ½ mile northeast of the site, as presented in Figure 5. These unnamed faults of the Truckee Canyon trend in a northeast to southwest direction and are mapped as Quaternary age (<1.8M[2]). An additional unnamed Quaternary aged (<1.8M) fault grouping of the Truckee River Canyon is located about ½ mile south of the project site. These faults trend predominately in a northwest to southeast direction.

[1] The referenced application features a 1:1,000,000-scale map of Quaternary faults in Nevada showing age of last rupture and movement type. Data was prepared in support from the U. S. Geological Survey and the Nevada Earthquake Safety Council, and was compiled by Craig M. dePolo in 2008.

[2] M=million years

Located approximately 1½ mile west of the project site is an unnamed Quaternary aged (<750Ka[3]) fault group. This unnamed fault group is located on the east side of the Verdi Range trending in an almost parallel direction to the Verdi Range (northeast to southwest).

Quaternary earthquake fault evaluation criteria have been formulated by a professional committee for the State of Nevada Seismic Safety Council. These guidelines define 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 (10ka to 1.6M years before present) are classified as either later Quaternary Active Fault (10,000 to 130,000 years) or Quaternary Active Fault (>130ka years). Both of the latter fault designations are considered to have a decreased potential for activity compared to the Holocene Active Fault. An Pre-Quaternary (>1.6M) fault is considered as a fault without recognized displacement or rupture within the Quaternary Active Fault groups. The age of the faults located closest to the subject property have been classified as either late Quaternary Active or Pre-Quaternary.

8.0 DISCUSSION AND RECOMMENDATIONS

8.1 General Discussion

Based on the results of our field and laboratory studies, the subsurface infrastructure may be constructed as currently proposed. In general, the proposed installation will be located between 5 and 8 feet below the existing grade across a majority of the proposed improvement area. Deeper installations may be required adjacent to the Truckee River Crossing and near the proposed West Meadows residential development on the east end of the alignment beginning at waterline Station 47+00.

It is our opinion that the predominate construction concerns include the following:

1) Large cobbles or boulders encountered in the pipeline trench. These cobbles and boulders were deposited over the period of several glaciations, glaciers extended eastward from Squaw Valley and Bear Creek blocking the movement of water through the course of the Truckee River causing significant rise in Lake Tahoe. As ice dams, broke mass-movement creating significant debris flows through the Truckee Canyon were released. Boulders on the order of 8 to 10 meters (26.2 to 32.8 feet) nominal diameter (Sylvester, 2012) have been encountered in the Verdi area near the Truckee Canyon. No large boulders were encountered within the explored locations; however, spoils piles from the nearby residential development construction indicate that larger boulders (on the order of 3 to 5 feet) are present within the area.

2) Potential for bottom of trench preparation instabilities, in areas where clay/elastic silt soils are encountered. These soils were encountered near Station 47+00 of the waterline. Stabilization of these soils should be anticipated to allow proper densification of bedding and backfill soils.

3) It is understood that restrained joints will be the primary method of pipe restraint for the project. Pipeline restraint recommendations are included as Section 8.2.5 (Pipeline Restraint). Thrust blocks may also be considered for project design; however, in areas where the pipeline is located within fine-grained or clayey soils, thrust blocks may not provide adequate restraint as these soils may be compressible with repetitive dynamic thrust loading. A dead man anchoring system may have to considered in these soil areas. Thrust blocks should be limited to areas where medium dense to dense granular soils are present. Additional recommendations for pipe restraint can be provided upon request.

[3] Ka=thousand years

8.1.1 General Information

It is recommended that the project design team and contractor review all of the recommendations contained in Section 8.0 prior to completing design and/or construction of the project. Failure to review this report in its entirety may result in poor performance of structural elements and construction deficiencies. The recommendations provided herein 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 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 adhered too[4].

The following is an abbreviated list of definitions and specifications that shall apply for this project:

- Fine-grained soil is defined as a soil with more than 40 percent by weight passing the number 200 sieve and a plasticity index less than 15.
- Clay soil is defined as a soil with more than 20 percent by weight passing the number 200 sieve and a plasticity index more than or equal to 15.
- Granular soil is defined as a soil not meeting the requirement for a fine-grained or clay soil and having a particle size of 4-inches or less.
- Structural areas referred to in this report include all areas that will be used for the support of concrete slabs, flat work, and asphalt pavements;
- All compaction requirements presented in this report are relative to ASTM D1557[5]; and
- Unless otherwise stated in this report, all related construction should be in general accordance with the Standard Specifications for Public Works Construction (SSPWC), dated 2016.
- Subgrade is defined as the soil located directly below the roadway structural section. Structural section construction and design recommendations are included in Section 8.3[6] (Structural Section);
- Existing utilities should be completely removed and/or abandoned in general accordance with Washoe County or local governing agency guidelines.
- Abandoned utilities located below structural elements (excluding the roadway structural section) shall be completely removed and disposed of in an approved location.

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[4] 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.

[5] 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.

[6] It should be noted that structural section construction methods within the NDOT right-of-way are subject to agency approval. Recommendations contained herein are applicable for residential street construction outside the limits of the State controlled right-of-way.

8.2 Subsurface Utility Construction and Design Recommendations

8.2.1 Trench Excavation

It is anticipated that trenches can be excavated with standard construction equipment consisting of a trackhoe or similar earthwork equipment; however, the following excavation difficulties are anticipated:

- Large boulders may require splitting to remove from confined excavations. Alternatively, boulder removal may be achieved by widening the trench.
- It is anticipated that confined excavations may make removal of large diameter boulders difficult.
- Boulders may have to be split using a pneumatic hammer or other rock splitting equipment.

Depending on the season of construction, seepage may be encountered. If significant seepage is encountered, dewatering may be required. Again this is based on the level of the river and weather conditions during construction. The method of dewatering will be determined during construction by the Contractor, based on the conditions encountered.

If dewatering is required, it is anticipated that a series of sump pumps located at the bottom of the excavation can likely be employed to remove groundwater. Discharge of groundwater shall comply with local, state, and federal guidelines. The contractor is ultimately responsible for dewatering and discharge.

8.2.2 Trench Sidewall Stability

All excavations regardless of depth should be evaluated for stability including scaling trench sidewalls to remove loose material prior to occupation by construction personnel. Shoring or sloping of trench walls will be required to protect construction personnel and provide temporary stability. In areas where temporary confined excavations may be unstable, trench boxes/trench shields may be used to provide safe ingress and egress for construction personnel.

Soils predominantly consisted of granular sands and gravels corresponding to OSHA safety requirements for *Type C* soils (Federal Register 29 CFR, Part 1926), which should be adjusted as needed for compliance during construction. Excavations within the vicinity of Boring BH-4 may consist of near surface Type C soils underlain by Type B material. The contractor will need to determine soil type during construction and implement the appropriate sloping/benching slope for trench excavations. Excavations should be carefully evaluated during construction to ensure the appropriate means of trench stabilization are enacted.

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

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

NOTES:
1. Angles have been rounded off.
2. Sloping or benching for excavations greater than 20 feet deep shall be designed by a Nevada Registered Professional Engineer.
3. For detailed soil descriptions visit the US Department of Labor Safety and Health Topics website at: https://www.osha.gov/SLTC/trenchingexcavation/construction.html

Although groundwater was not encountered, trench excavations should be protected from surface water/runoff. If warranted, dewatering of pipe trench excavations can be accomplished by use of a temporary dewatering system. If subsurface soil and/or water conditions differ from those encountered during our subsurface exploration, the soils engineer should be notified immediately to determine if alternative dewatering recommendations are warranted.

Sloughing or deformation of the trench side wall is a potential and bank stability will remain the responsibility of the contractor present at the site. Large cobbles may need to be scaled from the trench sidewall prior to trench occupancy.

8.2.3 Trench Bedding and Backfill

Any material used as pipe bedding or trench backfill should meet the minimum requirements of the SSPWC.

8.2.3.1 Pipe Zone Bedding

Pipe zone bedding is the trench backfill located immediately above and below the pipe. It is recommended that pipe zone bedding be placed in (loose) lifts not exceeding 8-inches thick. Pipe zone bedding shall be densified to a minimum of 90 percent relative compaction. Compaction equipment shall be carefully selected to avoid damage to the pipeline.

Pipe zone bedding shall conform to the requirements of either a Class A or C backfill (Table 200.03.02-1, SSPWC). Class A backfill can be used in trenches which are bottomed above the existing groundwater elevation. Class C backfill may be an alternative in areas with trench bottom instabilities such as soft or pumping soils.

8.2.3.2 Intermediate Trench Backfill

Intermediate trench backfill is located between the pipe zone bedding and subgrade elevation or ground surface. Intermediate trench backfill may consist of native ***granular*** soils, provided they are screened to removed oversized particles (4-inches or greater). Based on the material characteristics and moisture contents of the sandy elastic silt encountered at Boring BH-4, these soils should not be used for trench backfill. Imported backfill soils if required shall meet the specifications of a Class E backfill material (SSPWC, 2016).

Intermediate trench backfill[7] shall be placed in (loose) lifts not exceeding 8-inches thick, and densified to at least 90 percent relative compaction.

8.2.4 Bottom of Trench Preparation

Granular materials consisting of silty sands and poorly graded sands with cobbles and boulders will be encountered at the bottom of the trench across the majority the alignment. Bottom of trench excavation preparation in areas with firm, unyielding soils, as encountered in the majority of the trench alignment, shall consist of removing all loose soils from the bottom of the excavation prior to placing bedding material.

These soils should provide adequate support during densification of the bedding materials. Large cobbles and boulders located at the bottom of the trench may need to be removed prior to placement of pipe zone bedding, creating a void at the bottom of the trench. Voids left from isolated boulder removal shall be backfilled with granular material meeting the requirements of a Class E backfill or can be filled with additional bedding material. The resulting void should be widened as necessary to permit access to compaction equipment. The backfill shall be uniformly moisture conditioned within two 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. Larger voids in confined excavations may be backfilled with an excavatable sand-cement slurry.

Potentially unstable bottom of trench soils are anticipated to be encountered near Station 47+00 of the waterline alignment. Unstable soil remediation may be completed using a geotextile/gravel system (refer to Section 8.2.4.1-Stabilizing Construction Methods). Alternate stabilization methods may be suitable, and can be discussed during construction.

8.2.4.1 Stabilization Construction Methods

Bottom of trench stabilization may be achieved by removing the unstable soils and replacing them with Class C backfill (Section 200.03.04, SPPWC). The extent of the removal and replacement will be determined in the field, however a minimum replacement thickness of about 12 to 18 inches is recommended. Stabilization fill should extend the entire width of the trench excavation. Additional thicknesses may be required to adequately bridge unstable soils and will need to be determined in the field using a test section.

A high-performance geotextile (HPG) combines strength and permeability and is recommended for use during stabilization. The geotextile shall be placed directly below the stabilizing fill to provide separation and stabilization. The geotextile should be woven and meet or exceed the minimum properties presented in Table2 (Stabilizing Fill Geotextile).

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[7] Material located directly above the pipe zone bedding extending to the proposed finished subgrade.

Table 2- Stabilizing Fill Geotextile		
Mechanical Properties	**Minimum Average Roll Value (MARV)**	
	MD (#/ft)	**CD (#/ft)**
Tensile Strength at ultimate (ASTM D 4595)	3600	3200
Tensile Strength at 5% strain (ASTM D 4595)	1400	1400
Minimum permittivity (ASTM D 4491)	0.5 sec^{-1}	
Apparent Opening Size (AOS)	0.60 mm maximum	

Products such as a Mirafi HP370, Terra Tex HPG-37, or approved equal can be utilized for this project. Geotextile shall be placed on a ground surface that is smooth without sharp particles or abrupt edges. Geotextile should be laid in accordance with the manufacturer's recommendations with a minimum joint overlap of 18 inches. Construction equipment is prohibited from traveling directly over the geotextile surface.

Stabilizing fill should be densified using lightweight equipment such as a vibratory plate trench compactor.

8.2.5 Pipeline Restraint Design

Design recommendations for two common types of pipeline restraint methods will be considered for project design. Restrained joints will be the primary method of restraint for the proposed water main. Restrained joints provide thrust restraint by the use of a push-on or mechanical joint that is designed to provide longitudinal restraint.

Alternatively, the use of thrust blocks, may be considered for thrust restraint in areas along the alignment where trench sidewall soils consist of medium dense to dense granular material. Thrust blocking restrains the pipeline when a pressurized system is activated by transferring the dynamic thrust force to the bearing soil. Thrust blocks should not be considered for project design where bearing soils consist of low strength elastic silts similar to those encountered near Station 47+00 of the waterline alignment. Design recommendations for both pipe restraint methods are included in the following section.

8.2.5.1 Restrained Joints

As part of this design methodology, pipeline/bedding material interface friction is used in the design calculation of pipeline restraints.

The total unit friction resistance (F_s) is based on two primary components:

1. Unit normal force (earth pressure, pipe load, and water weight); and

2. Pipeline/bedding material interface friction.

The design F_s value used as part of the project design is based on ductile iron piping and recommendations from the Ductile Iron Pipe Research Association (DIPRA).

The pipeline/bedding interface friction is based on bedding material type, pipeline laying condition, and pipeline coating. Table 3 (Pipeline/Bedding Interface Design Parameters) presents the pipeline/bedding interface design parameters:

Table 3 – Pipeline/Bedding Interface Design Parameters for Restrained Joint Design	
Minimum recommended pipe zone backfill unit weight (above the groundwater table)	120 pcf (1)
Minimum recommended pipe zone backfill unit weight (below the groundwater table)	57 pcf (1)
Pipeline laying condition	Type 5 (2)
Pipeline/bedding Interface friction (**tan (d**(3)**)**) (ductile Iron coated with Asphalt)	0.51
Pipeline/bedding Interface friction (**tan (d)**) (Polyethylene Encasement)	0.36 (4)

1. Pipe zone backfill is assumed to be granular material with a minimum friction angle of 36 degrees and a unit weight of 120 pounds per cubic foot (buoyant unit weight of 57 pcf).
2. Type 5-bedding material densified to at least 90 percent relative compaction and fully encapsulates the pipe.
3. Obtained from DIPRA Table 3 (Suggested Values for Soil Parameters and Reduction Constant, K_n), where K_n=1.0 assumed Type 5 laying condition, f_f=0.75 for bedding sand meeting the requirements of this report, Pipe friction angle determined from the following equation. $d=K_n*f_f*f_{soil}$.
4. It is recommended by DIPRA that a reduction factor of 0.7 is applied to the pipeline/bedding interface value if polyethylene encasement is used.

8.2.5.2 Thrust Blocks

The design bearing pressure for thrust blocks is based on the passive pressure lateral loading at the proposed design depths for the thrust blocks. The allowable bearing pressure will be dependent on the bearing soil type and depth of installation. Table 4 summarizes recommended allowable bearing pressures for project design with an assumed installation depth of at least 4½ feet.

Table 4 – Allowable Bearing Pressures for Thrust Blocks

Soil Type	Unit Passive Pressure Value[1,3] (psf/ft of depth)	Allowable Bearing Pressure (psf) [2,3]
Granular Native Silty Sand, Poorly Graded Sands, and Gravels (SP, SM, GP, GM)	300	1,300
Sandy Elastic Silt (MH)	NR[4]	NR

Note:

1) The passive pressure is based on the following equation:
   i. $Pp = \gamma N_{\Phi} + 2Cs\sqrt{N_{\Phi}}$
      1. γ=unit weight (pcf)
      2. N_{Φ} =coefficient of passive pressure (Kp)
      3. Cs = Assumed soil cohesion (#/ft^2) calculated based on field pocket penetrometer readings.
2) Based on a depth to centerline of pipe of 5 feet. Other depths can be given, upon request.
3) Assumes a factor of safety of 3.
4) **NR:** Not Recommended, alternative pipe restraint methods are recommended in areas where sandy elastic will serve as the primary bearing surface.

Where possible, the bearing surface of the thrust block should be placed against undisturbed native soil. If the bearing surface of the thrust block will be on granular fill, the fill material should be compacted to at least 90 percent relative compaction.

8.2.6 Soil Corrosion

Soil chemistry testing was completed on select soils samples and included pH, soluble sulfates, and resistivity. It is recommended that these test results be reviewed by a corrosion engineer to determine soil corrosion potential. A brief summary of corrosion potential is presented below:

- **Soluble sulfates (ASTM 1050C):** Soluble sulfate levels in the tested samples was less than 0.02 percent by weight. Soluble sulfate levels less than 150 ppm (<0.10 percent by weight) indicate a negligible potential for sulfate exposure. Therefore, Type II cement can be used for project design (Concrete International, August 2005).

- **Chlorides:** Corrosion of embedded rebar is of major concern with reinforced concrete exposed to soils containing chlorides. The presence of chloride ions in soils could result in reduced resistivity. Additionally, high chloride content can make zinc coating of galvanized surface more susceptible to corrosion. Test results ranged from 6.8 to 29 mg/Kg. In general the majority of the site soils have a low chloride content (<20 ppm). The highest chloride content was from Boring B-1 at a depth of 6 to 7 feet bgs.

- **pH (SW-846 9045D):** The pH test results ranged from about 6.8 to 8.5, which indicates the site soils range from slightly acidic to alkaline across the profile. In general, the pH test results have a low to moderate potential for corrosivity of ferrous metals in contact with the alignment soils (Baboian, 2006).

- **Resistivity (ASTM G-57):** Resistivity test results ranged from 1,400 to 4,623 ohms x cm. In general, soils with resistivity below 3,000 are moderately to severely corrosive to metal pipes. In general, the site soils have a moderate to severe corrosive potential for ferrous metal (Baboian, 2006).

- **Redox potential:** The redox potential indicates the degree of aeration in the soil. Testing is currently being completed. Soils with a high redox potential indicate that there is free oxygen available in the soil (i.e. aerobic soil conditions) and are generally non-corrosive to metal pipes.

- **Sulfides:** The presence of sulfides indicates that sulfate-reducing bacteria may be present, which can be corrosive to metal pipes. Sulfide content within the tested samples was negative, indicating sulfides were not detected within the tested samples.

Corrosion test results are included in Appendix B (Plate B-3).

A 10-point soil evaluation system (AWWA,2005) can be used to determine if soils are corrosive to ductile iron piping. The laboratory test with the greatest influence for corrosion potential is the resistivity test, which provides a measurement of the soil's conductivity potential. The lower soil resistivity, the more corrosive it is to ductile iron piping. The total points for the representative soil samples are presented in Table 5 (Corrosion Test Results and Corrosion Potential Total Points).

Table 5 – Corrosion Test Results and Corrosion Potential Total Points

Boring Sample Number	Laboratory Tests					
	Resistivity (ohm-cm)	Redox potential (mV)	Sulfide	pH	Field moisture conditions	Corrosion Potential Total points
BH-1 (1C)	3,065	495	Negative	8.49	Moist, well drained	0
BH-3 (3B)	4,009	416	Negative	8.12	Moist, well drained	0
BH-4 (4B)	1,450	513	Negative	6.87	Moist, poorly drained	12
BH-5 (5C)	4,623	441	Negative	7.83	moist	0

Based on the total points obtained, a majority of the project soils are generally non-corrosive to ductile iron pipe. However, soils tested from Boring BH-4 indicate that soils have a high corrosive potential to iron pipe and corrosion protection should be considered. In areas where the potential for corrosion is high or where saturated soils are present, polyethylene encasement should be considered for project design.

8.3 Structural Section Design and Construction Recommendations

The proposed subsurface waterline will include asphalt trench patching with potential curb and gutter replacement on a portion of the following roads within the River Oak Subdivision:

- Riveredge Drive;
- Stoneridge Drive; and
- Fallbrook Drive.

The existing roads are narrow, approximately 25 feet wide. Minor areas of transverse and longitudinal cracking are visible within the pavement along the proposed waterline alignment. Some existing utility trench edge seams are located along Riveredge Drive extending to Stoneridge Drive. One longitudinal crack is visible along the centerline of Riveredge Drive extending the entire length of the roadway. Minimal cracking in Fallbrook Drive was observed.

It is assumed that the structural section replacement will be limited to the area disturbed by the installation. One boring was completed within the River Oak Subdivision near the proposed waterline Station 41+00. The structural section encountered consisted of about 4½ inches of asphalt concrete pavement placed directly on granular subgrade soils. The recommended replacement structural section is included as Table 5 (Recommended Structural Section). The following recommended structural section assumes subgrade soils will have an R-Value of at least 50. Pavement section is based on Washoe County standards. The aggregate base layer will also be a leveling course over coarse grained granular subgrade soils with cobbles and boulders.

Table 6 – Recommended Structural Section

Layer Description	Layer Thickness (inches)
Asphalt Concrete Pavement *(Section 8.3.1 Asphaltic Pavement Construction)*	3
Aggregate Base *(Section 8.3.2-Aggregate Base Material)*	4 to 6[2]
Total Replacement Structural Section Thickness[3]	7 to 9

1) Materials shall meet the guideline specifications as outlined in the SSPWC unless otherwise stated in this report.
2) Four inch thickness provided subgrade soils consist of granular soils exhibiting with a low plasticity (PI≤10). The base thickness may be less than 4 inches, but shall have a minimum thickness of 2 inches. The structural section directly over the trench shall have an increased base aggregate thickness of 6 inches. Subgrade soils shall have a minimum R-Value of 50.

For partial pavement removal, it is recommended the existing asphalt is saw cut. The saw cut shall extend at least 9 inches beyond the limits of the trench sidewall.

Wider sections may require removal in areas where cracked or damaged asphalt is present. The permanent trench patch shall comply with Drawing No: W-2.2 (Permanent Pavement Patch), Standard Details for Public Works Construction, Washoe County.

Because of the presence of cobbles and boulders in the subgrade soils, pulverization is likely not a viable construction option.

8.3.1 Asphaltic Pavement Construction

Type 2 Plantmix Aggregate in accordance with Section 200.02 of the SSPWC, 2016 should be utilized for the asphalt. Asphalt pavement compaction requirements should be in accordance with the SSPWC, 2016. A pavement mix design should be submitted to the owner by the Contractor at least five 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.

8.3.1.1 Pavement Maintenance

Maintenance is ***mandatory*** to long-term pavement performance. Maintenance refers to any activity performed on the pavement that is intended to preserve its original service life or load-carrying capacity. Examples of maintenance activities include patching, crack or joint sealing, and seal coats. If these maintenance activities are ignored or deferred, premature failure of the pavement **will occur**.

The cost associated with proper maintenance is generally much less than the cost for reconstruction due to premature failure of the pavement. Therefore, since pavement quality is an integral consideration in the formulation of our design recommendations, we strongly recommend the owner/project manager implement a pavement management program.

8.3.2 Aggregate Base Material

Aggregate base material, shall conform to Section 200.01 of the SSPWC, 2016. Aggregate base material should be densified to at least 95 percent relative compaction.

9.0 CONSTRUCTION OBSERVATION AND TESTING SERVICES

The recommendations presented in this report are based on the assumption that the owner/project manager provide adequate field testing and construction review during all phases of construction. These tests and observations[8] should include, but not be limited to:

- Earthwork observation and materials testing;
- Observation and testing of construction utility trench backfill; and
- Special Inspection as required by the design engineer.

It is also recommended that the project geotechnical engineer conduct a general review of the project plans and specifications to determine if the recommendations presented in this report have been properly interpreted and implemented during design.

Prior to construction, the owner/project manager should schedule a preconstruction conference to include, the owner, design engineer, the general contractor, earthwork and materials subcontractors, and geotechnical engineer. It is the owner's/project manager 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.

[8] CME maintains one of the region's largest accredited labs and employs a full staff of qualified inspectors and can provide additional information concerning the scope and cost of these services upon request.

10.0 LIMITATIONS

The recommendations provided herein, particularly under Section 8.0 are intended to reduce the risks of structural distress. These recommendations along with proper design and construction of the Verdi Water Main Extension as currently proposed are intended to improve the overall performance. If any part of this system is ignored or poorly implemented, the performance and quality of the project will be reduced. Sufficient construction observation and testing should be performed to document that the recommendations presented in this report are adhered to.

This report has been prepared in accordance with generally accepted local geotechnical practices. The analysis and recommendations submitted are based upon the field exploration performed at the locations shown on Plate A-1. This report does not reflect soils variations that may become evident during construction. Re-evaluation of the recommendations may be necessary if subsurface conditions vary from those presented in this report.

This report has been prepared to provide information allowing the engineer to design the project as currently proposed. The owner/project manager is responsible for distribution of this report to all designers and contractors whose work is affected by the geotechnical recommendations contained herein. 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 project geotechnical engineer. If the geotechnical engineer is not accorded the privilege of making this recommended review, they can assume no responsibility for misinterpretation or misapplication of recommendations or their validity in the event changes have been made in the original design concept without 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 for Shaw Engineering. The material in it reflects our best judgment based on the information available to us at the time of preparation. Any use which a third party makes of this report, any reliance on, or decisions to be made based upon it, shall be done at their own risk. CME accepts no responsibility for damages suffered by any third party as a result of decisions made or implemented based on this report.

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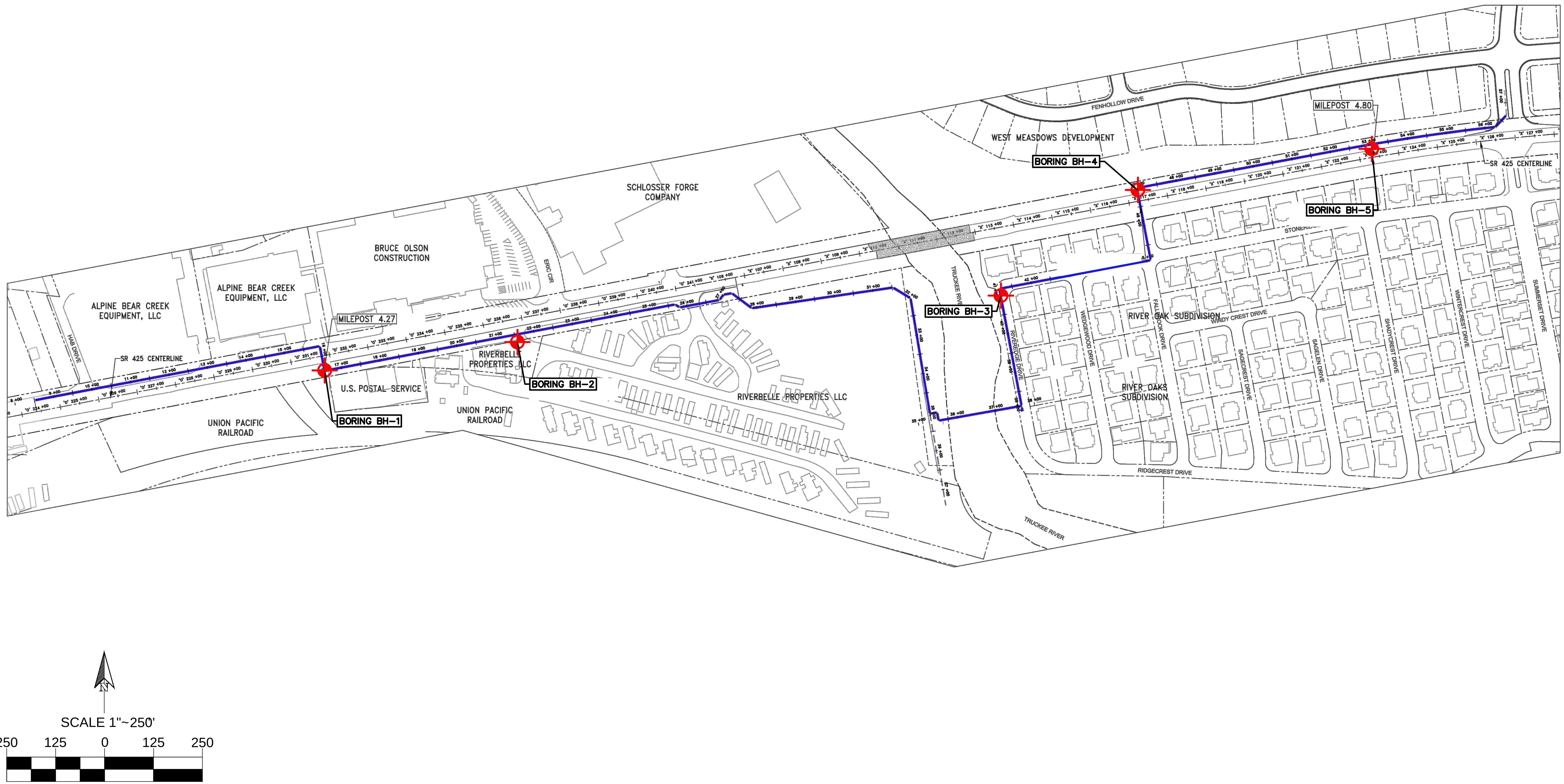
APPENDIX A



REFERENCE: BASE MAP SITE PLAN BY SHAW ENGINEERING, DATED MAY 2017.

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Reno, NV 89511

SHAW ENGINEERING
VERDI WATER MAIN EXTENSION- RIVER BELL
FIELD EXPLORATION LOCATION MAP
VERDI, NEVADA

PROJECT NO.: 2008 DATE: 7/24/2017

LEGEND
- APPROXIMATE BORING LOCATION
- PROPOSED WATER LINE

PLATE
A-1

LOG OF TEST BORING NO. BH-1

PROJECT VERDI WATER MAIN EXTENSION-RIVER BELL **RIG & BORING TYPE** Air Knife/Sonic Rig
LOCATION South side of US Hwy 40; Waterline Sta. 16+55
CLIENT: Shaw Engineering **DATE** 6/14/17
PROJECT NO. 2008 **LOGGED BY:** AH **SURFACE ELEVATION** 4848 ft
BLOW COUNTS: Corrected N/A Not Corrected N/A **HAMMER TYP.:** N/A

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									0.0'-0.5': ASPHALT CONCRETE.								
	GC							SL. MOIST	0.5'-10.0': CLAYEY GRAVEL WITH SAND, mostly fine to coarse gravel, little fine to coarse sand, dark brown.								
2.5				B	1A												
									Note: Air knife terminated a 5.0', cobbles up to 12-inches nominal diameter encountered.								
5				B	1B												
				B	1C					21.8	27	9				5.2	A, G
7.5				B	1D												
10									Terminated at 10.0'. No free water encountered.								
12.5																	
15																	

GROUNDWATER

DEPTH	HOUR	DATE
NE		

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. BH-2

PROJECT VERDI WATER MAIN EXTENSION-RIVER BELL **RIG & BORING TYPE** Air Knife/Sonic Rig
LOCATION South side of US Hwy 40; Waterline Sta. 21+50
CLIENT: Shaw Engineering **DATE** 6/13-14/17
PROJECT NO. 2008 **LOGGED BY:** AH **SURFACE ELEVATION** 4837 ft

BLOW COUNTS: Corrected N/A Not Corrected N/A **HAMMER TYP.:** N/A

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	SP-SM							SL. MOIST	0.0'-10.0': POORLY GRADED SAND WITH SILT, GRAVEL AND COBBLES, mostly fine to coarse sand, some fine to coarse gravel and cobbles, brown to dark brown.								
2.5				B	2A				Note: Air Knife terminated at 5.0', cobbles up to 10-inches nominal diameter encountered.								
5																	
				B	2B												
7.5																	
				B	3B												
10									Terminated at 10.0'. No free water encountered.								
12.5																	
15																	

GROUNDWATER

DEPTH	HOUR	DATE
NE		

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

CONSTRUCTION MATERIALS ENGINEERS, INC.

LOG OF TEST BORING NO. BH-3

PROJECT VERDI WATER MAIN EXTENSION-RIVER BELL **RIG & BORING TYPE** Air Knife/Sonic Rig
LOCATION South of US Hwy 40; Waterline Sta. 41+00
CLIENT: Shaw Engineering **DATE** 6/13-14/17
PROJECT NO. 2008 **LOGGED BY:** AH **SURFACE ELEVATION** 4815 ft

BLOW COUNTS: **Corrected** N/A **Not Corrected** N/A **HAMMER TYP.:** N/A

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									0.0'-0.4': ASPHALT CONCRETE.								
	GP-GC			B	3A			SL. MOIST	0.4'-10.0': POORLY GRADED GRAVEL WITH CLAY AND SAND, mostly fine to coarse gravel, some fine to coarse sand, few low plasticity fines, brown to dark brown.								
2.5									Note: Due to high concentration of cobbles Air Knife terminated at 4.0', cobbles up to 12-inches nominal diameter encountered.								
5				B	3B					8.4	26	9				3.3	A, G
				B	3C												
7.5																	
				B	3D												
10									Terminated at 10.0'. No free water encountered.								
12.5																	
15																	

GROUNDWATER

DEPTH	HOUR	DATE
NE		

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-2c

CME CONSTRUCTION MATERIALS ENGINEERS, INC.

LOG OF TEST BORING NO. BH-4

PROJECT VERDI WATER MAIN EXTENSION-RIVER BELL **RIG & BORING TYPE** Air Knife/Sonic Rig
LOCATION North side of US Hwy 40; Waterline Sta. 47+00
CLIENT: Shaw Engineering **DATE** 6/14/17
PROJECT NO. 2008 **LOGGED BY:** AH **SURFACE ELEVATION** 4832 ft
BLOW COUNTS: **Corrected** N/A **Not Corrected** N/A **HAMMER TYP.:** N/A

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									0.0'-0.5': ASPHALT CONCRETE.								
	SM							SL. MOIST	0.5'-4.0': SILTY SAND WITH GRAVEL AND COBBLES, mostly fine to coarse sand, some fine to coarse gravels and cobbles, little low plasticity fines, dark brown.								
2.5				B	4A				Note: Due to high concentration of cobbles Air Knife terminated at 4.0', cobbles up to 10-inches nominal diameter encountered.								
	MH							VERY MOIST	4.0'-10.0': SANDY ELASTIC SILT, mostly medium to high plasticity fines, some fine to medium sand, dark brown.								
5				B	4B					55.1	75	27				42.3	A, G, MD
				B	4C												
7.5				B	4D												
10									Terminated at 10.0'. No free water encountered.								
12.5																	
15																	

GROUNDWATER

	DEPTH	HOUR	DATE
▽	NE		
▼			

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-2d

CME CONSTRUCTION MATERIALS ENGINEERS, INC.

LOG OF TEST BORING NO. BH-5

PROJECT VERDI WATER MAIN EXTENSION-RIVER BELL **RIG & BORING TYPE** Air Knife/Sonic Rig
LOCATION North side of US Hwy 40; Waterline Sta. 53+00
CLIENT: Shaw Engineering **DATE** 6/13-14/17
PROJECT NO. 2008 **LOGGED BY:** AH **SURFACE ELEVATION** 4842 ft

BLOW COUNTS: Corrected N/A Not Corrected N/A **HAMMER TYP.:** N/A

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	SM							SL. MOIST	0.0'-10.0': SILTY SAND WITH GRAVEL AND COBBLES, mostly very fine to coarse sand, some fine to coarse gravel and cobble, brown to dark brown.								
				B	5A												
2.5									Note: Due to high concentration of cobbles Air Knife terminated at 4.0', cobbles up to 12-inches nominal diameter encountered.								
5				B	5B												
				B	5C												
7.5																	
				B	5D												
10									Terminated at 10.0'. No free water encountered.								
12.5																	
15																	

GROUNDWATER

DEPTH	HOUR	DATE
NE		

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-2e

CME CONSTRUCTION MATERIALS ENGINEERS, INC.

UNIFIED SOIL CLASSIFICATION CHART

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

		Clean Gravels (Less than 5% fines)
GRAVELS More than 50% of coarse fraction larger than No. 4 sieve size	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
		Clean Sands (Less than 5% fines)
SANDS 50% or more of coarse fraction smaller than No. 4 sieve size	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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6980 Sierra Center Parkway, Suite 90
Reno, NV 89511

VERDI WATER MAIN EXTENSION- RIVER BELL
SOIL CLASSIFICATION CHART
VERDI, NEVADA

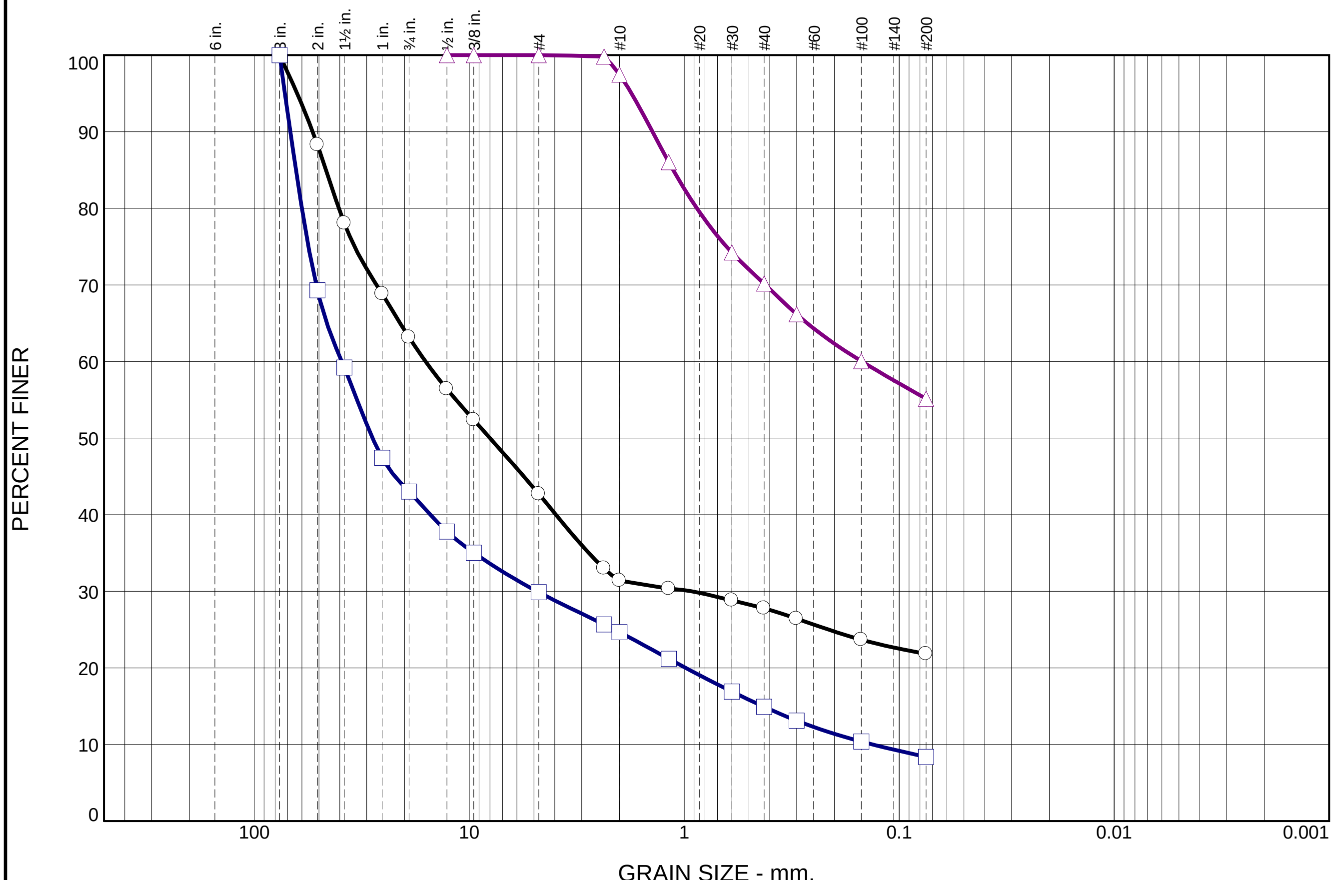
PROJECT NO.:2008 DATE: 07/25/2017

PLATE
A-3

V:\Active\2008\Shaw-River Bella Waterline\CAD\soil classification chart and rock description.dwg

APPENDIX B

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	36.8	20.5	11.3	3.6	6.0	21.8	
□	0.0	57.0	13.1	5.2	9.8	6.5	8.4	
△	0.0	0.0	0.0	2.5	27.3	15.1	55.1	

╳	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	27	18	46.3828	15.9582	8.0072	0.9265				
□	26	17	63.9734	39.1491	28.1706	4.8401	0.4312	0.1331	4.50	294.22
△	75	48	1.1255	0.1490						

	MATERIAL DESCRIPTION	TEST DATE	USCS	NM
○	clayey gravel with sand	6/14/17	GC	5.2
□	poorly graded gravel with clay and sand	6/14/17	GP-GC	3.3
△	sandy elastic silt	6/14/17	MH	42.3

Project No. 2008 **Client:** SHAW ENGINEERING

Project: VERDI WATER MAIN EXTENSION- RIVER BELL

○ **Location:** B-1 **Depth:** 6.0'-8.0' **Sample Number:** 1C

□ **Location:** B-3 **Depth:** 4.0'-6.0' **Sample Number:** 3B

△ **Location:** B-4 **Depth:** 4.0'-6.0' **Sample Number:** 4B

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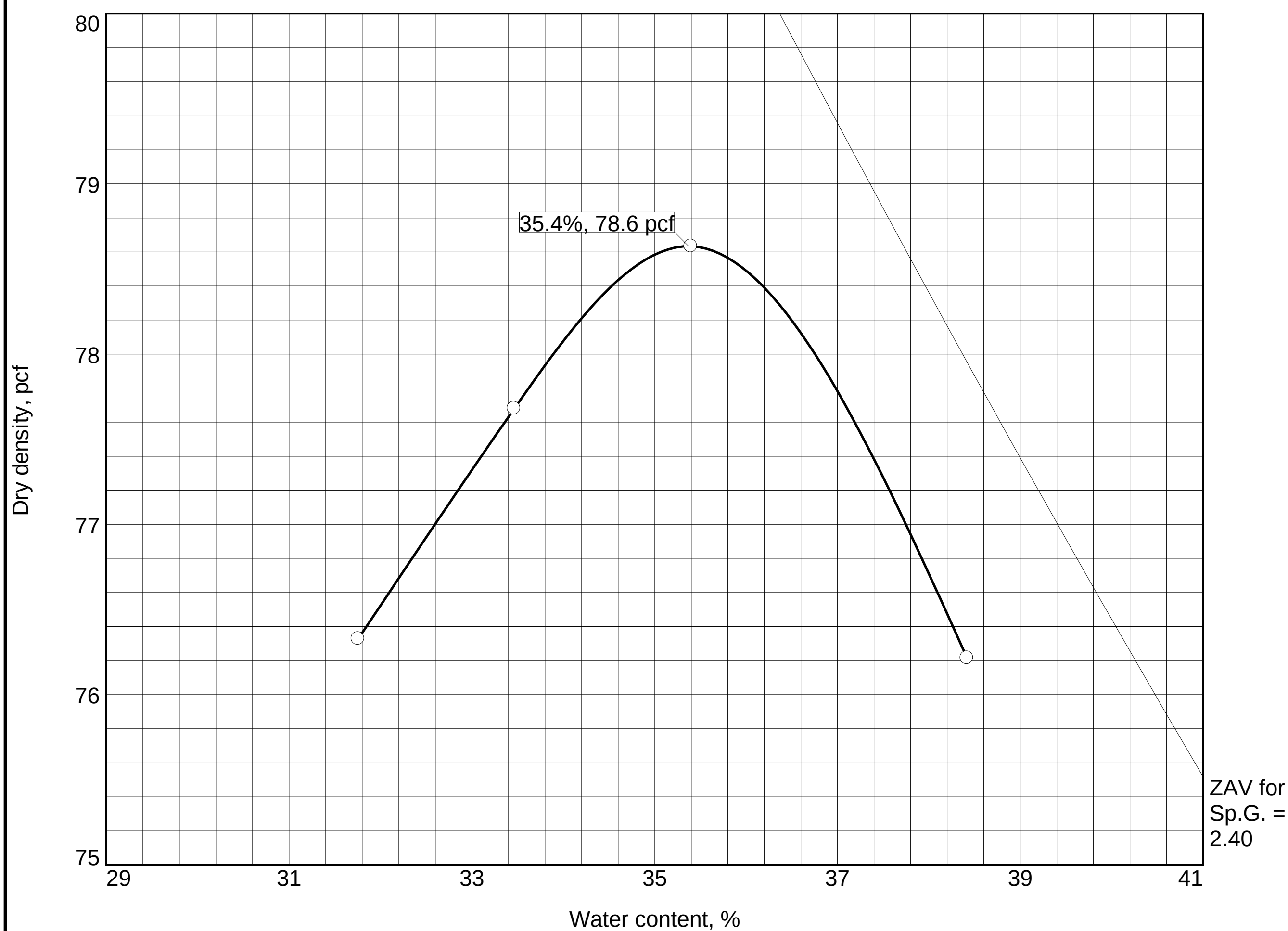
Remarks:

Figure B-1

Tested By: ○ A. SALAZAR □ A. SALAZAR △ S. BRUKETTA **Checked By:** S. HEIN

COMPACTION TEST REPORT



Test specification: ASTM D 1557-12 Method A Modified

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > #4	% < No.200
	USCS	AASHTO						
4.0'-6.0'	MH	A-7-5(14)	42.3		75	27	0.0	55.1

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 78.6 pcf Optimum moisture = 35.4 %	sandy elastic silt

Project No. 2008 **Client:** SHAW ENGINEERING
Project: VERDI WATER MAIN EXTENSION- RIVER BELL

○**Location:** B-4 **Sample Number:** 4B

CME CONSTRUCTION MATERIALS ENGINEERS, INC.

Remarks:

Figure B-2

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

Silver State Analytical Laboratories
Sierra Environmental Monitoring

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

Analytical Report

Workorder#: 17060883
Date Reported: 8/8/2017

Client: CME-Construction Materials Engineers, Inc
Project Name: TMWA Verdi Main
PO #: 2008
Sampled By: Client

Laboratory Accreditation Number: NV015/CA2990

Laboratory ID	Client Sample ID	Date/Time Sampled	Date Received
17060883-01	B-1 1C 6'-8'	06/14/2017 12:00	6/16/2017

Parameter	Method	Result	Units	PQL	Analyst	Date/Time Analyzed	Data Flag
Chloride	EPA 300.0	29	mg/Kg	5	JF	08/03/2017 16:45	
Oxidation-Reduction Potential	SM 2580B	495	mV		LRB	08/04/2017 9:39	
pH	SW-846 9045D	8.49	pH Units		LRB	06/20/2017 16:11	
pH Temperature	SW-846 9045D	24.0	°C		LRB	06/20/2017 16:11	
Resistivity	ASTM G-57	3065	Ohms-cm		LRB	06/21/2017 0:00	
SO4	ASTM 1580C	< 0.02	%	0.02	LRB	06/20/2017 9:55	
Sulfide	AWWA C105	Negative	POS/NEG		LRB	08/02/2017 16:58	

Laboratory Accreditation Number: NV015/CA2990

Laboratory ID	Client Sample ID	Date/Time Sampled	Date Received
17060883-02	B-3 3B 4'-6'	06/14/2017 12:00	6/16/2017

Parameter	Method	Result	Units	PQL	Analyst	Date/Time Analyzed	Data Flag
Chloride	EPA 300.0	6.8	mg/Kg	5	JF	08/03/2017 17:12	
Oxidation-Reduction Potential	SM 2580B	416	mV		LRB	08/04/2017 9:39	
pH	SW-846 9045D	8.12	pH Units		LRB	06/20/2017 16:11	
pH Temperature	SW-846 9045D	24.0	°C		LRB	06/20/2017 16:11	
Resistivity	ASTM G-57	4009	Ohms-cm		LRB	06/21/2017 0:00	
SO4	ASTM 1580C	< 0.02	%	0.02	LRB	06/20/2017 9:55	
Sulfide	AWWA C105	Negative	POS/NEG		LRB	08/02/2017 16:58	

Laboratory Accreditation Number: NV015/CA2990

Laboratory ID	Client Sample ID	Date/Time Sampled	Date Received
17060883-03	B-4 4B 4-6'	06/14/2017 12:00	6/16/2017

Parameter	Method	Result	Units	PQL	Analyst	Date/Time Analyzed	Data Flag
Chloride	EPA 300.0	16	mg/Kg	5	JF	08/03/2017 17:40	
Oxidation-Reduction Potential	SM 2580B	513	mV		LRB	08/04/2017 9:39	
pH	SW-846 9045D	6.87	pH Units		LRB	06/20/2017 16:11	
pH Temperature	SW-846 9045D	24.0	°C		LRB	06/20/2017 16:11	
Resistivity	ASTM G-57	1450	Ohms-cm		LRB	06/21/2017 0:00	
SO4	ASTM 1580C	< 0.02	%	0.02	LRB	06/20/2017 9:55	
Sulfide	AWWA C105	Negative	POS/NEG		LRB	08/02/2017 16:58	

Laboratory ID	Client Sample ID	Date/Time Sampled	Date Received
17060883-04	B-5 5C 6-8'	06/14/2017 12:00	6/16/2017

Parameter	Method	Result	Units	PQL	Analyst	Date/Time Analyzed	Data Flag
Chloride	EPA 300.0	7.0	mg/Kg	5	JF	08/03/2017 18:07	
Oxidation-Reduction Potential	SM 2580B	441	mV		LRB	08/04/2017 9:39	
pH	SW-846 9045D	7.83	pH Units		LRB	06/20/2017 16:11	
pH Temperature	SW-846 9045D	24.0	°C		LRB	06/20/2017 16:11	
Resistivity	ASTM G-57	4623	Ohms-cm		LRB	06/21/2017 0:00	
SO4	ASTM 1580C	< 0.02	%	0.02	LRB	06/20/2017 9:55	
Sulfide	AWWA C105	Negative	POS/NEG		LRB	08/02/2017 16:58	

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

VERDI WATER MAIN EXTENSION- RIVER BELL
CORROSION TEST RESULTS
VERDI, NEVADA

PROJECT NO.:2008 DATE: 09/08/2017

PLATE
B-4

V:\Active\2008\Shaw-River Bella Waterline\CAD\soil classification chart and rock description.dwg