SHAW
ENGINEERING

October 10, 2014
14026.10D

Scott Benedict, Senior Engineer
Truckee Meadows Water Authority
1355 Capital Blvd.
Reno, NV 89502

Re.: Geotechnical Engineering Report for
TMWA Sutro #2 Pump Station

Dear Scott:

The attached report presents the results of our geotechnical engineering investigation and recommendations for the proposed Sutro #2 Pump Station in Reno, Nevada. We appreciate having been selected to perform this study and trust the results will satisfy your requirements. If you have any questions concerning our evaluations or recommendations, please call our office.

Very truly yours,

Dennis Keely

Dennis Keely, P.E.

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GEOTECHNICAL ENGINEERING REPORT

FOR THE PROPOSED

SUTRO #2 PUMP STATION

AT

THE SOUTHEAST CORNER OF SUTRO STREET AND SELMI DRIVE

IN

RENO, NEVADA

FOR THE

TRUCKEE MEADOWS WATER AUTHORITY

RENO, NEVADA

PREPARED BY

SHAW ENGINEERING, LTD.

RENO, NEVADA

OCTOBER 2014
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INTRODUCTION AND SCOPE

During September and October 2014, a geotechnical engineering investigation was conducted for a proposed pump station for the Truckee Meadows Water Authority (TMWA) in Reno, Nevada. The work was performed at the request of Scott Benedict, Senior Engineer for TMWA.

The purpose of the investigation was to determine the general soil conditions and other subsurface soil data related to the design and construction of the proposed pump station. Our scope of work included digging test pits, conducting field resistivity testing, and providing this report addressing the general soil types, their condition, and providing foundation engineering recommendations for supporting the proposed structure and its paved parking area.

The test pits were logged in the field by a civil engineer from Shaw Engineering and the soil field classified in accordance with visual manual procedures.

LIMITATIONS

This report has been prepared for exclusive application to the specific project and locations discussed herein. In the event that any changes are implemented in the general design or preliminary location of the structure as shown on our Test Pit Plan or as described in this report, we might need to determine if any modifications to our recommendations are required or if any additional field studies are required to confirm the subsurface conditions and our original design recommendations.

We have prepared this report in accordance with locally accepted soil and geotechnical engineering practices and make no other warranties either expressed or implied. The analyses and evaluations presented in this report are based upon the necessary interpolation and extrapolation of the data and soil conditions determined from the test holes as shown on the attached Test Pit Plan and Log of Test Pits. The test pit logs present the soil conditions only at the locations where the test pits were excavated and are not intended to guarantee similar subsurface conditions between, nor beyond, the test pit locations. The actual existing subsurface soil conditions between and beyond the test pits will not be known until construction excavations reveal them. If soil conditions different than those we describe become evident during construction, it may be necessary to review the variations and modify the recommendations to accommodate the soil conditions revealed during construction. The test pits were excavated with a John Deere 310SG rubber-tired 4X4 backhoe using a 2 foot wide bucket at the locations shown and to the depths indicated on the Test Pit Plan and Log of Test Pits sheet. However, this does not imply or express any warranties as to the excavation characteristics of the subsurface soil or bedrock. An evaluation for the presence of toxic waste or other health hazards was not part of this investigation.

Ground water was not encountered during this investigation. However, it must be noted that water level fluctuations will occur due to infiltration, seasons, diurnal barometric pressure changes, local irrigation practices, the level of water in drainage ditches, and any future developments or improvements in the vicinity of the project. Other factors not evident at the time of our investigation may also affect water level changes.

If our report or Log of Test Pits is included in the contract specifications or plans, we advise that they be included in their entirety and not be modified or redrafted. Bidders should examine copies of our full report, and we recommend that they investigate the site conditions of the project and fully satisfy themselves of both the surface and subsurface conditions there.

PROJECT DESCRIPTION AND BACKGROUND INFORMATION

The proposed project is located at the southeast corner of Sutro Street and Selmi Drive in Reno, Nevada. We understand that the proposed project consists of a single story masonry block building with approximate dimensions of 24 feet 8 inches by 24 feet 8 inches. The building will have a slab-on-grade floor. A small paved parking area and a drive way off Selmi Drive are part of the project. The parking area has concrete curbs and gutters. The building will house electrical equipment, electric motors, water pumps and piping. The finish floor elevation is reported to be approximately 4654.1, which ranges from about 0.6 to 3.2 feet above the ground surface. The ground surface slopes approximately 8 percent from the northwest downward to the southeast. We understand that the new water pipes to the pump station will be buried approximate 5 to 6 feet below the ground surface.

Test pits are reported to have been excavated in the past either on the site or near it for other purposes. The locations of these test pits are not known and they are not clearly evident on the site.

GENERAL SITE AND SOIL CONDITIONS

The project site is covered with fill and sparse vegetation. There are small piles of debris and wasted soil on the site. The test pits we excavated revealed that the fill varies from 0.6 to 1.8 feet thick and is composed of loose, silty sand and gravel with debris to soft, brown, (CH) sandy clay with debris. The debris consists of asphalt concrete, rusted metal, nails, glass, red brick, and decayed wood, plastic, and shaped timber. Beneath the fill is very stiff, dark brown, dry to slightly damp, (CH) clay that is highly expansive. This fat, expansive clay varies from approximately 1 to 3.5 feet thick and predominately transitions into an undetermined thickness of intensely hydrothermally altered andesite bedrock that is equally highly expansive if it were to imbibe water. In test pit TP-1, the (CH) clay had the most variable thickness and the clay was absent at the southwest corner of the test pit where the existing fill rested on moderately soft to hard andesite bedrock. The (CH) clay became thicker and underlain by the intensely altered bedrock toward the east end of the test pit. Where the bedrock was hard, it was close fractured and jointed such that the backhoe could excavate it to the depths of the test pits. Descriptions of rock hardness and other rock characteristics are presented in Tables 2 and 3 at the end of this report. Where the bedrock has a soft rock hardness, it is usually intensely hydrothermally altered to a soil texture of expansive clay and expansive elastic silt. Due to the three dimensional variations of alteration of the bedrock, its degree of expansiveness is also variable within short horizontal and vertical distances. Engineering, construction, and laboratory experience has shown that the hydrothermally altered bedrock complex, when converted to montmorillonite clay, exhibit volume expansion and swell pressure similar to the overlying dark brown, (CH) clay. The author of this report conducted the geotechnical investigations for the commercial retail complex near the site at the southwest corner of Clearacre Lane and North McCarran Boulevard. A swell pressure of 16,680 pounds per square foot and an expansion of 12.9 percent under a seating load of 125 pounds per square foot were measured on a sample of the in situ dark brown (CH) clay from the Sonic Drive-In Restaurant at the retail complex. This is the same (CH) clay present at the proposed pump building site.

We excavated a shallow test pit approximately 30 feet east of the proposed pump building and found approximately a foot of existing very soft fill composed of expansive sandy clay with debris overlying the in situ dark brown, expansive (CH) clay. No ground water was encountered in the test pits. Specific details of the soil conditions encountered are shown on the log of test pits.

Near the project site, is a road cut just west of the intersection of Sutro Street and North McCarran Boulevard that shows an approximately similar profile of the in situ (CH) clay and intensely to moderately hydrothermally altered bedrock complex.

Expansive Clay Background Comments

1. The single most important factor affecting swelling characteristics of expansive soils is density.
2. By far the most important element, and of the most engineering concern, is the effect of water on expansive soils. With the introduction of water, volumetric expansion takes place.
3. The thickness of expansive soil affects the magnitude of total heave.
4. Total heave depends on environmental conditions such as extent of wetting, duration of wetting and the pattern of moisture migration. Such variables cannot be ascertained, and consequently total heave predictions can be entirely erroneous.
5. Volume change increases in direct proportion to the degree of saturation; swelling pressure is relatively constant. A short duration of wetting can cause equally severe damage to lightly loaded structures as long duration wetting.
6. Expansive soils will not shrink or swell unless there is a change in moisture content. A drier soil will swell more than a wet soil.
7. The swelling pressure of a clay is independent of the surcharge pressure, the initial moisture content, the degree of saturation, and the thickness of the stratum.

8. The swelling pressure increases with the increase in initial dry density.

9. Expansive clay soils expand very little when compacted at low densities and high moisture content but expand greatly when compacted at high densities and low moisture contents.

10. In theory, the swelling potential of an expansive clay can be minimized or eliminated by one or more of the following methods:

    a. Flood the in-place soil to achieve swelling prior to construction, but the technique is unpredictable and uncontrollable.

    b. Decrease the density of the soil by compaction control.

    c. Replaced the swelling soils with non-swelling soils.

    d. Change the properties of the expansive soil by chemical injection or chemical mixing.

    e. Isolate the soil so there will be no moisture change.

11. A drilled pier foundation system is a solution to offset the effects of expansive soils where the expansive soil is of limited thickness and the piers can be founded and anchored in a non-expansive soil or bedrock. Design and construction of the piers requires careful attention and quality control.

12. With present technology on expansive soil, and where the expansive soil is thick, partial soil replacement is the best method to use in obtaining a stabilized foundation soil:

    a. It is possible to compact the replaced non-expansive soil to a high degree of compaction and therefore support heavily loaded slabs and footings.

b. The cost of soil replacement is relatively inexpensive compared to chemically treating the soil and can be carried out without delay as is encountered in pre-wetting, and the work does not require a specialty subcontractor.

c. The non-expansive soil cushion will distribute the swell from deep seated expansive soil more uniformly and any heave movement is consequently more tolerable.

Warnings

1. Good surface drainage will reduce the risk of foundation movement from the expansive soils. However, other factors such as adequate structural design and proper construction techniques are equally important. Changing the grade to improve the appearance of landscape features will have a damaging effect if surface drainage is directed toward the foundations.
2. Any downspouts should be long enough to drain water away from the building onto surface drainage features to rapidly carry off the water.
3. Lawn and sprinkler systems should not be constructed adjacent to the building, but should be kept at least 10 feet away from the building with the sprinkler nozzles directed away from the structure.
4. Expansive soil used as backfill can exert swell pressures on a wall or footing stemwall and can cause cracking. Horizontal swelling pressure is approximately equal in magnitude to the vertical swelling pressure.
5. A desert landscape theme is recommended that minimizes water applications to expansive soil.

6. It should be understood that dry expansive soil beneath an impervious membrane, concrete slab, or asphalt pavement, will in time become moist or wet because evaporation can no longer take place or is substantially retarded. However, using membranes, Portland cement concrete slabs and/or asphalt concrete pavement around the building will increase the time required for the moisture penetration and should make the moisture distribution and heave more uniform. Membranes and pavements will retard swelling but will not prevent it.

DISCUSSION

The in situ (CH) clay and hydrothermally altered andesite have a high bearing capacity but they also have a high potential to swell when they absorb water. **Foundations and slabs-on-grade should not be placed directly on the expansive soil and intensely altered bedrock.** The existing fill on the site should be removed and wasted off-site. Imported granular non-expansive fill should replace the existing fill and the in situ expansive (CH) clay and altered bedrock to a depth of at least 4 feet beneath the footings and concrete floor slab. (On a level site, the excavation would extend to approximate 6 feet below the ground surface.) The 4 feet of granular fill beneath the footings and floor slabs will make potential swell from the foundation subgrade soil more uniform and tolerable for the building. In addition to the recommended aggregate base course and asphalt pavement section the structural pavement section should be underlain by at least 2 feet of imported granular subbase that has an "R" value of at least 20. Exterior concrete flat work, such as sidewalks, curbs and gutters, should be underlain by at least 2 feet of imported granular subbase and Type 2 Class B aggregate base course to aid control of potentially swelling soil.

As was noted in test pit TP-1, the andesite bedrock was near the ground surface at the south end of the test pit and has a moderately hard to hard rock hardness. Due to the unpredictable nature of the hydrothermal alteration and the acidic water and steam vent locations that existed when the bedrock was undergoing alteration, there can be areas under the site where the bedrock is less altered or fresh where it was not altered by hydrothermal processes, and the bedrock may be very hard and massive such that hydraulic rock hammers, large excavators, or large dozers with a ripper, or drill holes and expansive grout compounds might be necessary to excavate the bedrock to install the pipelines to and from the pump building.

GENERAL SITE GRADING RECOMMENDATIONS

The general site grading should be conducted in accordance with the grading plan, the grading specifications, the attached "General Engineered Fill Recommendations" in the Appendix of this report, and the following recommendations.

The existing fill should be removed from the entire site and wasted off-site. Further excavating should be done to provide at least 4 feet of imported granular fill beneath the footings and floor slab of the proposed building and the excavation should horizontally extend at least 5 feet from the perimeter of the building to provide a buffer between the structure and the expansive soil. This might require excavating approximately 6 feet below the ground surface depending on the building’s finish floor elevation and bottom elevation of the footings. In situ soil beneath the proposed paved areas and under exterior concrete flat work should be excavated to provide at least 2 feet of granular fill under these improvements and for a horizontal distance of at least 2 feet beyond these improvements. We speculate from the test pits, topography, and the fill on the ground surface that existing loose fill might extend to 2 or 3 feet deep along the south edge of Selmi Drive and east edge of Sutro Street. The loose fill should be removed and replaced with imported granular fill that is compacted to 95 percent or better per ASTM D1557. We recommend that an engineer from our office be retained during the excavating to verify the existing fill has been removed. All excavations should be made in accordance with OSHA regulations.

Densification of the in situ clay at the bottom of the excavations is not necessary because of its very stiff consistency. The addition of water can also induce long term swelling of the expansive clays. However, the bottom of the excavations should be proof rolled to compress any loose soil left by the excavating equipment. An engineer from our office should inspect the site during the proof rolling to verify that any uncompacted fill has been removed and to confirm the condition of the foundation soil upon which structural fill is to be placed.

The excavations should be backfilled and the site raised to approximate rough finish subgrade elevations using imported silty sand consisting of decomposed granite (DG). This includes proposed paved areas and exterior concrete flat work areas. Other types of import soil are not recommended as decomposed granite from various commercial sources in the area is non-expansive and has relatively low permeability when compacted. These characteristics are necessary to preclude rapid infiltration of water, yet provide a non-expansive cushion of soil beneath the footings, floor slabs, pavements, and exterior concrete flat work. DG from the Donovan Pit in Spanish Springs and the pit in Golden Valley have been used successfully. The imported fill should be free of wood, organics, deleterious debris and it should approximately conform to the requirements of Table 1 and will require approval by an engineer from our office prior to being imported to the site.

Table 1 – General Requirements for Imported Structural Fill Beneath the Improvements.

SIEVE SIZE	PERCENT PASSING
1 Inch	100
No.4	90 - 100
No. 200	10 – 20

LIQUID LIMIT - 35 MAX
PLASTICITY INDEX - 6 MAX

A five gallon sample of the proposed import fill should be delivered to an approved materials testing laboratory for analysis for these recommended properties and the results should be submitted for review and approval prior to importing the soil to the site. The imported granular fill should be placed in maximum 12 inch loose lifts and each lift compacted to at least 95 percent per ASTM D1557. Field density tests should be made on every vertical foot of fill placed and each lift should be tested prior to placing additional lifts of fill.

Where the finished surface of the fill is higher than the existing topography, the top of the fill should extend at least 5 feet beyond the outer edge of the perimeter footings, before sloping off at 4 horizontal to 1 vertical or flatter slope as might be required by the grading plan.

Floor slabs and exterior concrete flat work should be underlain by at least 6 inches of Type 2 Class B aggregate base per Section 200.01.03 of the Standard Specifications for Public Works Construction. The aggregate base should be compacted to at least 95 percent per ASTM D1557.

Exterior concrete flat work (i.e. sidewalks not contiguous to the building, driveways, curbs & gutters) should be underlain by 24 inches of non-expansive granular fill to reduce the potential heave from the clayey subgrade soils. The upper 6 inches of fill should consist of Type 2, Class B aggregate base per the Standard Specifications for Public Works Construction and should be compacted to 95 percent per ASTM D1557. The lower 18 inches or more of granular fill should consist of decomposed granite conforming to the requirements of Table 1 and it should be compacted to 95 percent per ASTM D1557.

Grading should be performed in a manner that will provide drainage away from the walls and footing excavations so that surface water will not enter into the excavations. Soil that is allowed to become wet in footing excavations should be removed and replaced with compacted granular fill. The building site should be final graded to drain surface water away from the proposed structure.

FOUNDATION ENGINEERING RECOMMENDATIONS

The proposed building should be supported by continuous footings that are founded on at least 4 feet of non-expansive granular structural fill. Perimeter footings should be founded at least 24 inches below the adjacent exterior final grade for frost protection. Prior to placing reinforcing steel and concrete, the footing excavations should be compacted with hand operated Wackers (not vibratory plates) to compact soil disturbed by the excavating equipment. Footings may be designed for an allowable bearing pressure up to 4000 pounds per square foot. Footings should be at least 16 inches wide. Vertical bearing pressures may be increased by a factor of 1.33 for wind or seismic lateral forces. For mass concrete on granular structural fill we recommend a coefficient of friction up to 0.45 be used at the bottom of the footings. An equivalent fluid active earth pressure of at least 35 pounds per square foot per foot of depth and a passive earth pressure up to 300 pounds per square foot per foot of depth may be used for the compacted horizontal granular fill adjacent to the footings.

Based on experience, settlement of footings founded on compacted granular structural fill is anticipated to be 3/4 inch or less and differential settlement is expected to be less than 3/8 inch. Since the foundation support soil will be granular, settlements are expected to occur rapidly as the loads are applied and long term settlement is anticipated to be negligible.

In our opinion the pump building site is approximately represented by a Site Class D soil profile per the 2012 International Building Code. The pump building site is located at approximately 39.558091 degrees north latitude and 119.798307 degrees west longitude for use with the IBC spectral acceleration maps or the USGS Seismic Design Maps[1].

CORROSIVE SOIL

Field resistivity testing was conducted on the site using the Wenner Four-Point configuration. The testing was done with a SoilTest Stratameter. One alignment was approximately east-west, located approximately 10.5 feet south of the proposed building, and another alignment was approximately north-south, approximately centered on the south side of the proposed building. The results are as follows:

Test No.	Line Direction/Location	Electrode Spacing (Ft)	Resistivity (ohm-cm)
1	E-W, 10.5’ S. of Building	5	837
2	Same	10	740
3	Same	15	637
4	N-S, centered on south side of proposed building.	5	919
5	Same	10	97
6	Same	15	724

The results for test number 5 in the north-south alignment, appears to be anomalous and might represent better contact with the (CH) clay that underlies the existing fill. The average apparent resistivity in the east-west alignment is 738 ohm-centimeters. The average apparent resistivity in

[1] http://earthquake.usgs.gov/hazards/designmaps/

the north-south alignment is 822 ohm-centimeters with the results from test No. 5 thrown out. The overall resistivity for the approximate building site is 780 ohm-centimeters for the in situ soil.

A sample of the on-site dark brown (CH) clay and another of the intensely altered andesite were also analytically tested for their resistivity under laboratory conditions. The results of these tests and other corrosive soil suite of tests are presented in the Appendix of this report. The testing was conducted by Western Environmental Testing Laboratory (WETLAB) in Sparks, Nevada.

PAVED AREAS

As previously recommended a 2 foot layer of decomposed granite with an 'R' value of at least 20 should underlie the pavement sections to control heave from the subgrade soil. Depending on the site grading, the 2 feet of granular fill might reduce the thickness of the expansive clay in part of the paved area and its potential heave, but will not prevent it. Removing a greater thickness of the expansive soil is thought to be not commensurate for the potential cost for additional excavating and placement of imported decomposed granite in the paved area. Some heaving and “bird-bath” depressions might develop at the pavement surface and exterior concrete flat work with time as the expansive clay absorbs water. Predicated on this approach, it is our opinion that the structural pavement section for the paved areas should consist of 3 inches of asphalt concrete over 6 inches of aggregate base. This pavement section is based on a 10 year design life, a traffic index of 4.5, and the recommended 2 foot thick layer of decomposed granite with an ‘R’ Value of at least 20. A traffic index (TI) of 4.5 is equal to approximately 2900 equivalent 18-kip axle loads (EAL). The actual life of the pavement is affected by such factors as the aging of the asphalt concrete, the quality of the asphalt concrete mix, the density of the compacted mix, the quality and frequency of maintenance, the actual number of equivalent 18-kip axle loads applied to the pavement over a period of time, and the stability of the subgrade soil.

Aggregate base should consist of Type 2, Class B aggregate base per Section 200.01.03 of the local Standard Specifications for Public Works Construction (i.e. the Orange Book), and it should be compacted to at least 95 percent per ASTM D1557. The asphalt concrete should conform to the Standard Specifications for Public Works Construction, Section 200.02 and 320, for Type 3 asphalt concrete with PG64-22 asphalt, and 2 to 4 percent air voids in the mix using 50 blows per side of the Marshall biscuits. The asphalt concrete should be compacted from 92 to 97 percent of its Rice theoretical maximum density per ASTM D2041.

Vertical and horizontal surfaces should be tack coated with SS1 asphalt emulsion per Section 316 of the SSPWC, and the tack coat should cure (break) prior to paving. We recommend that the ground temperature be at least 40^{o}F, and the air temperature should be 40^{o}F and rising before paving begins. We recommend that the density and thickness of the aggregate base and asphalt pavement be verified during construction, and that the pavement surface be fog sealed in three to five years to recoat surface aggregates and to enhance the pavement life.

Cracks will occur in the asphalt concrete pavement from thermal expansion, shrinkage and aging which will permit rapid infiltration of surface water to the expansive subgrade soil and induce early differential heave of the pavement. We recommend that a yearly maintenance program include cleaning the cracks of soil and debris, and filling them with an appropriate rubberized crack filler (e.g. Crafco’s Parking Lot Sealant No. 34200, or No. 34202, or Crafco’s

PolyFlex Type 2, Crafco Part No. 34518) to prolong the life of the pavement and to especially reduce infiltration of surface water to the aggregate base, subbase, and expansive clay subgrade.

SPECIAL PROVISIONS

We recommend that the proposed building be completely surrounded by the asphalt concrete pavement and/or Portland cement concrete sidewalk to rapidly shed surface water away from the building and to provide additional control of water infiltration into the compacted decomposed granite subbase. The pavement surface and/or sidewalk must slope away from the building.

Roof gutter downspouts should not dump their collected water adjacent to the building footings and stemwalls. Downspouts should be connected to pipes that pass through sidewalks contiguous to the building so collected water is dumped at the sidewalk edge and onto the pavement surface.

The joints in exterior sidewalks contiguous to the building and the joints between these sidewalks and the building stemwalls should be caulked with a durable rubberized elastomeric or silicon sealant to prevent surface water infiltration through the joints. A product such as Sikaflex-1a manufactured by the SIKA Corporation is suggested. Other sealants that adhere well to concrete should also be satisfactory.

The cushion of decomposed granite under the building is thought to be sufficient to control differential heave from the expansive subgrade clay. Flexible pipe joint connections are recommended for consideration at critical locations as a precaution to allow for some movement from potential heave.

Pipes should be embedded and backfilled in accordance with TMWA standards.

Test pit TP-1 is located in the paved area north of the proposed building. We recommend that the loose backfill soil be removed from the test pit and replaced with the imported decomposed granite that is compacted to at least 95 percent per ASTM D1557. The decomposed granite backfill is recommended because improvements will be constructed above test pit and the granular fill can be easily compacted to high density and it will not swell. The loose backfill in test pit TP-2 should also be removed, but the excavated soil here can be replaced as compacted backfill that is compacted from 88 to 90 percent per ASTM D1557 and between approximately one to two percent over its optimum moisture content to preclude rapid infiltration of surface water and to provide compacted soil for a probable thrust block for the 90 degree pipe elbow shown on the test pit plan. Compaction at higher moisture contents might be permitted provided that the compacted expansive soil is demonstrated to be relatively stable under the compaction equipment.

As was mentioned earlier in this report, there are other backfilled test pits reported in the vicinity of the project site. The locations of these other test pits are not known. Should these other test pits be found during construction, we recommend that we be notified to review their location relative the proposed site improvements (i.e. the proposed building, paved areas, exterior concrete flat work, and underground pipes) and to provide recommendations regarding their disposition.

CONCLUSIONS

In order to permit correlation between the soil data obtained from the two test pits and the actual soil conditions encountered during construction, and so as to facilitate conformance with the plans and specifications as contemplated, we recommend that our firm be retained to perform on-site construction review during the excavating, foundation preparation and backfilling portions of the work. We will assume no responsibility for construction compliance with the design concepts, specifications or recommendations, unless we are retained to perform construction review, or full time observation where generally accepted soil and geotechnical engineering practices deem it necessary. Review of any plans, specifications, or on-site inspections, testing and associated consultation are a scope of work and fee separate from the preparation of this report.

If there are questions concerning our evaluation or recommendations, please call for an explanation.

Respectfully submitted,
Shaw Engineering, Ltd.

PROFESSIONAL ENGINEER - STATE OF NEVADA
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TABLE 2
DEFINITIONS OF DESCRIPTIVE TERMS FOR ROCKS BASED ON MEGASCOPIC EXAMINATION

SPACING OF DISCONTINUITIES:

A. Nomenclature of Joints and Fractures

Very Close	-	Less than two inches
Close	-	Two inches to one foot
Moderately Close	-	One foot to three feet
Wide	-	Three feet to six feet
Very Wide	-	Greater than six feet

ROCK HARDNESS

A. SOFT or PLASTIC - Slightly harder than very hard overburden; may have soil-like consistency or rock-like character, but crumbles or breaks easily by hand; can be scratched by fingernail.

B. MODERATELY SOFT - Cannot be crumbled between fingers, but can be easily picked with light blows of a geologic pick; can be cut with a knife, but cannot be scratched by fingernail.

C. MODERATELY HARD - Can be picked with moderate blows of a geologic pick and can be scratched with a knife, but cannot be cut by a knife.

D. HARD - Cannot be picked with geologic pick, but can be chipped with moderate blows of a hammer and is difficult to scratch with a knife.

E. VERY HARD - Chips can be broken off only with heavy blows of a hammer and cannot be scratched with a knife.

DEGREE OF WEATHERING

A. UNWEATHERED - Fresh.

B. SLIGHTLY WEATHERED - Stained joints and fractures up to depths of 1 to 2 inches.

C. MODERATELY WEATHERED - Staining, discoloration and some loosened mineral grains.

D. INTENSELY WEATHERED - Mineral grains loosened, rock volume change, spalling, by-products due to hydration, oxidation and carbonation.

TABLE 3
ROCK HARDNESS CLASSIFICATION / UNCONFINED COMPRESSIVE STRENGTH / ROCK DESCRIPTION[2]

Hardness	Unconfined Compressive Strength (tsf)	Rock Description
Very Soft	10 to 250	The rock can be readily indented, grooved, or gouged with the fingernail, or carved with a knife. Breaks with light manual pressure.
Soft	250 to 500	The rock can be grooved or gouged easily with a knife or sharp pick with light pressure. Can be scratched with a fingernail. Breaks with light to moderate manual pressure.
Hard	500 to 1,000	The rock can be scratched with a knife or sharp pick with great difficulty (heavy pressure). A heavy hammer blow is required to break the rock.
Very Hard	1,000 to 2,000	The rock cannot be scratched with a knife or sharp pick. The rock can be broken with several solid blows of a geologic hammer.
Extremely Hard	>2,000	The rock cannot be scratched with a knife or sharp pick. The rock can only be chipped with repeated heavy hammer blows.

[2] Reproduced from "Horizontal Directional Drilling Utility and Pipeline Applications" by David A. Willoughby, 2005, Table 2-3, Page 35.