TRAUTNER GEOTECH LLC
GEOTECHNICAL ENGINEERING, MATERIAL TESTING
AND ENGINEERING GEOLOGY

# GEOTECHNICAL ENGINEERING STUDY

# GROUNDHOG RESERVOIR INTAKE GATED BULKHEAD STRUCTURE

Dolores County, Colorado

November 28, 2018

Prepared For:
Mr. Brandon Johnson
Montezuma Valley Irrigation Company
Project Number: 55415GE

## 1.0 REPORT INTRODUCTION

This report presents our geotechnical engineering recommendations for the proposed Goundhog Reservoir Intake Gated Bulkhead Structure Project. This report was requested by Mr. Brandon Johnson, Montezuma Valley Irrigation Company, and was prepared in accordance with our proposal dated September 26, 2018, Proposal No. 18231P. The field study was completed on October 12, 2018. The laboratory study was completed on November 19, 2018.

We understand that a design engineer for the proposed structure has not been selected at the date of issue of this report. Particular details of the structure are not known at this time. We must be contacted as the project design progresses to verify that the recommendations provided in this report are applicable for the final project structure design. The following general details regarding the project are assumed at this time;

- The new bulkhead structure will likely consist of a gated structure that is supported on a new foundation system.
- The new steel-reinforced concrete bulkhead structure will either be constructed immediately upstream of the existing intake structure, or the existing intake structure concrete flatwork and trash gate support structure will be removed and replaced with the new bulkhead structure.

We must be consulted throughout the design and construction process to verify the implementation of the geotechnical engineering recommendations provided in this report. The recommendations and technical aspects of this report are intended for design and construction personnel who are familiar with construction concepts and techniques, and understand the terminology presented below.

It is common for unforeseen, or otherwise variable subsurface soil and water conditions to be encountered during construction. As discussed in our proposal for our services, it is imperative that we be contacted during the foundation excavation stage of the project to verify that the conditions encountered in our field exploration are representative of those encountered during construction. Compaction testing of fill material and testing of foundation concrete are equally important tasks that should be performed by the geotechnical engineering consultant during construction. We should be contacted during the construction phase of the project if any questions or comments arise as a result of the information presented below.

The following outline provides a synopsis of the various portions of this report;

- ❖ Sections 1.0 and 2.0 provide an introduction and an establishment of our scope of service.
- ❖ Sections 3.0 and 4.0 of this report present our geotechnical engineering field and laboratory studies.

- Section 5.0 presents our geotechnical engineering design parameters and recommendations which are based on our engineering analysis of the data obtained.
- Section 6.0 provides a brief discussion of construction sequencing and strategies which may influence the geotechnical engineering characteristics of the site.

The discussion and construction recommendations presented in Section 6.0 are intended to help develop site soil conditions that are consistent with the geotechnical engineering recommendations presented in the report. The construction considerations section is not intended to address all of the construction planning and needs for the project site, but is intended to provide an overview to aid the owner, design team, and contractor in understanding some construction concepts that may influence some of the geotechnical engineering aspects of the site and proposed development.

The data used to generate our recommendations are presented throughout this report and in the attached figures.

### 1.1 *Scope of Project*

We understand that the purpose of the proposed bulkhead structure is to allow for repairs of the reservoir outlet works, such as the outlet tunnel or existing service gates without draining the reservoir. The proposed bulkhead structure will block water flow into the existing inlet area of the reservoir outlet works. The structure will be designed to withstand the forces associated with all potential loading conditions that may act on the structure (normal reservoir operating conditions, low pool conditions, potential seismic loading, etc.).

As discussed above, we understand that the structure will be supported by a steel reinforced concrete foundation system, likely consisting of a steel reinforced mat slab. Construction of the structure will require draining the reservoir. Water flow into the construction area should be expected. Dewatering of the construction area will likely be necessary during the project construction phase.

## 2.0 GEOTECHNICAL ENGINEERING STUDY

Our services include a geotechnical engineering study of the subsurface soil and water conditions for development of the proposed project. We have provided basic/limited seismic information in Section 3.3 of this report.

## 2.1 *Geotechnical Engineering Study Scope of Service*

The scope of our study which was delineated in our proposal for services, and the order of presentation of the information within this report, is outlined below.

Field Study

- We advanced two test borings in the vicinity of the proposed structure. We were limited in areas that we could advance our test borings due to soft and saturated soil conditions in the vicinity of the existing intake area of the outlet works.

- Select driven sleeve and bulk soil samples were obtained from the test borings and returned to our laboratory for testing.

Laboratory Study

- The laboratory testing and analysis of the samples obtained included;
    - Moisture content and dry density characteristics of Modified California Barrel samples obtained from the weak formational materials located in the vicinity of the proposed structure.
    - Estimates of soil strength parameters including direct shear strength tests, to help establish a basis for development of lateral earth pressures that may act on the structure. The direct shear strength tests were performed on the existing shallow soil materials that overlie the upstream face of the dam in the vicinity of the existing intake area of the outlet works.
    - Sieve analyses and Atterberg Limits tests performed on the existing shallow soil materials that overlie the upstream face of the dam in the vicinity of the existing intake area of the outlet works.
    - Unconfined compressive strength tests of rock core obtained from the formational materials that underlie the existing intake area. The results of the unconfined compressive strength tests were used to estimate allowable foundation bearing capacity values, and estimated allowable capacities for grouted anchors (micropiles) that will likely be required to resist various uplift and lateral forces that may act on the structure.
    - Swell/consolidation tests to help assess the expansion and consolidation potential of the weathered formational shale materials that will be located in the vicinity of the new structure.
    - Water soluble sulfate testing to assess the corrosion potential of the formational materials on Portland cement concrete and grout.

The laboratory test results are provided in Appendix B of this report and generally discussed in Section 4.0 below.

Geotechnical Engineering Recommendations

- This report addresses the geotechnical engineering aspects of the site and provides recommendations including;

  *Geotechnical Engineering Section(s)*

  - Subsurface soil and water conditions that may influence the project design and construction conditions. The subsurface conditions are generally discussed in Section 3.3 below. The logs of the subsurface conditions are presented in Appendix A.
  - Geotechnical engineering design parameters that may be used for the project design are discussed in Section 5 below.

  *Construction Consideration Section*

  - Anticipated excavation characteristics of the formational materials that underlie the project site.
  - Considerations for temporary excavation cut slopes.
  - Compaction recommendations for various types of backfill that may be placed against the structure retaining/wing walls (if used).

- This report provides design parameters, but does not provide foundation design or design of structure components. The project structural engineer should be contacted to provide a design based on the information presented in this report.

- Our subsurface exploration, laboratory study and engineering analysis do not address environmental or geologic hazard issues.

## 3.0 FIELD STUDY

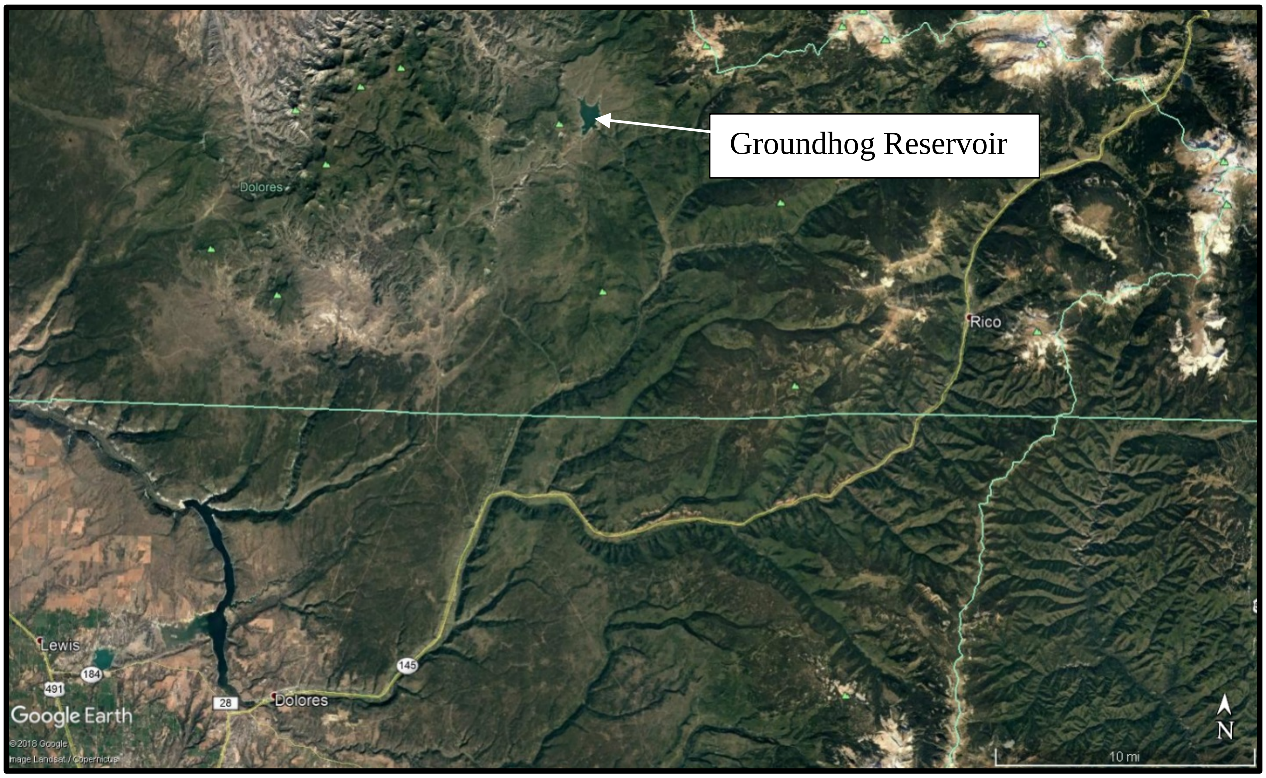
### 3.1 *Project Location*

Groundhog Reservoir is located in Dolores County, Colorado, approximately mid-distance from Dolores, Colorado (located in Montezuma County to the south of the reservoir) and Norwood, Colorado (located in San Miguel County to the north of the reservoir). General directions to Groundhog Reservoir from the Town of Dolores, Colorado are provided below;

- From Dolores, travel north on Forest Service Road 526 (Montezuma County Road 31) for a distance of approximately 25 miles to Forest Service Road 533.
- From the intersection of Forest Service Roads 526 and 533, travel northwest on Forest Service Road 533 for a distance of approximately 5.25 miles at which point the road crosses the crest of the dam.

The approximate location of the reservoir is provided on Figure 3.1 presented below (Google Earth imagery).

*Figure 3.1: Reservoir Location*



The existing service gates for the outlet works are located at the intake side (upstream side) of the outlet works. We have indicated the approximate location of the intake area of the outlet works on Figure 3.2 below (the actual location of the intake area is obscured by impounded water). The aerial image used for Figure 3.2 was obtained from Google Earth (imagery date 10/6/2012), and represents the imagery with the lowest impounded water level available from various Google Earth imagery.

*Figure 3.2:  Approximate Location for the Intake Area of the Outlet Works*

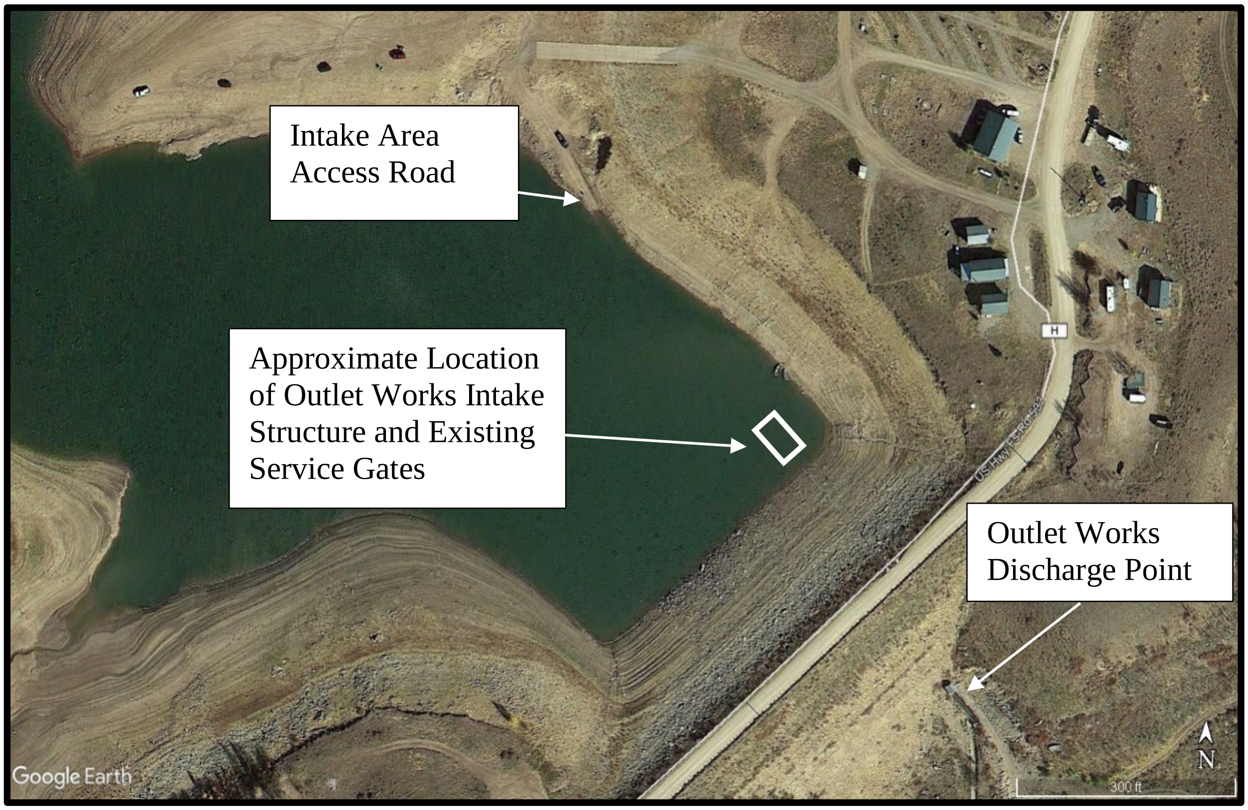


Figure 3.3 presented below indicates the characteristics of the intake structure at the time of our field study.  The upstream face of the dam is in the background of the photograph.

*Figure 3.3: Characteristics of the Existing Intake Structure*

### 3.2 *Existing Intake Structure General Description*

The existing intake structure consists of a concrete box structure (likely steel reinforced). Based on the information obtained from our Test Boring TB-1, we suspect that the concrete mat slab is likely bearing on formational sandstone and shale materials. Trash racks are located around the left, right and upstream sides, as well as the top of the intake structure. The existing service gates are located in the rear (downstream) side of the intake structure at the inlet to the concrete outlet tunnel. The white colored cylinders shown in Figure 3.3 presented above are the housing structures around the hydraulic actuated cylinders that control the service gates. Our track mounted drilling equipment is set up at our Test Boring TB-1 location.

The general surface characteristics of the existing concrete intake structure appeared to be good. We did not observe evidence that would indicate that substantial degradation to the intake structure concrete from phenomena such as sulfate attack or alkali-silica reactions has occurred in the past. The compressive strength characteristics of the concrete, or characteristics and condition of the reinforcement steel is not known at this time.

### 3.3 *Geomorphology and Basic Seismic Considerations for the Project Site*

Groundhog Reservoir is primarily situated within the Mancos Shale Formation. We observed shale exposed throughout many areas of the reservoir basin (the reservoir had been drained at the time of our field study). The transition between the Mancos Shale and Dakota Sandstone Formations is mapped as occurring in the general vicinity of the intake structure. The transitional area between the Mancos Shale and Dakota Sandstone/Burro Canyon Formations often consists of interbedded layers of brown to gray colored shales and claystone material (likely associated with the Mancos Shale Formation) with layers of brown to tan colored sandstone (associated with the Dakota Sandstone Formation). As discussed in more detail in Section 3.4 below, we encountered interbedded layers of shale, claystone, and sandstones in the upper portions of Test Boring TB-1. Based on the results of our field study and site observations, the existing intake structure is likely situated over the transitional zone between the Mancos Shale and Dakota Sandstone Formations.

We referenced USGS seismic mapping data to provide basic level seismic information for the project. The 2014 long-term peak ground surface acceleration (two percent probability of exceedance in 50 years) indicates a peak ground surface acceleration (PGA) of about 0.1g. The link to the mapping that we reviewed is provided below.

https://earthquake.usgs.gov/static/lfs/nshm/conterminous/2014/2014pga2pct.pdf

### 3.4 *Subsurface Soil and Water Conditions*

We advanced two test borings in the vicinity of the proposed structure. We were limited in areas that we could access our drilling equipment due to the slope surfaces of the dam embankment and soft/saturated soil conditions on the dam embankment. Figure 3.4 indicates the approximate locations of our test borings relatively to Google Earth imagery (imagery date 10/6/2012). Figures 3.5 and 3.6 provide photographs that indicate the locations of our test borings. The drill rig shown on Figure 3.6 is mobilizing away from the Test Boring TB-2 location and became buried in the soft/saturated surface soils of the upstream dam face. The logs of the soils encountered in our test borings are presented in Appendix A.

*Figure 3.4: Approximate Test Boring Locations Relative to Google Earth Imagery*

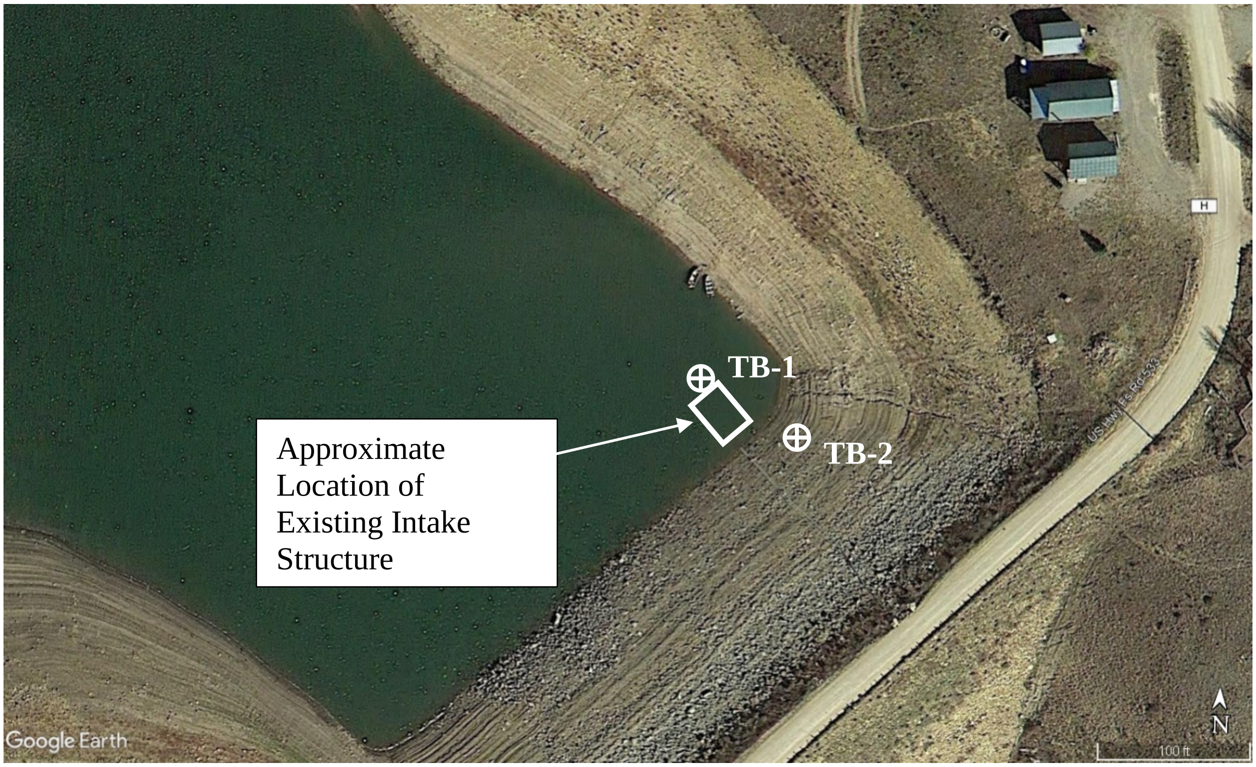


*Figure 3.5: Photographic Location of Test Boring TB-1*

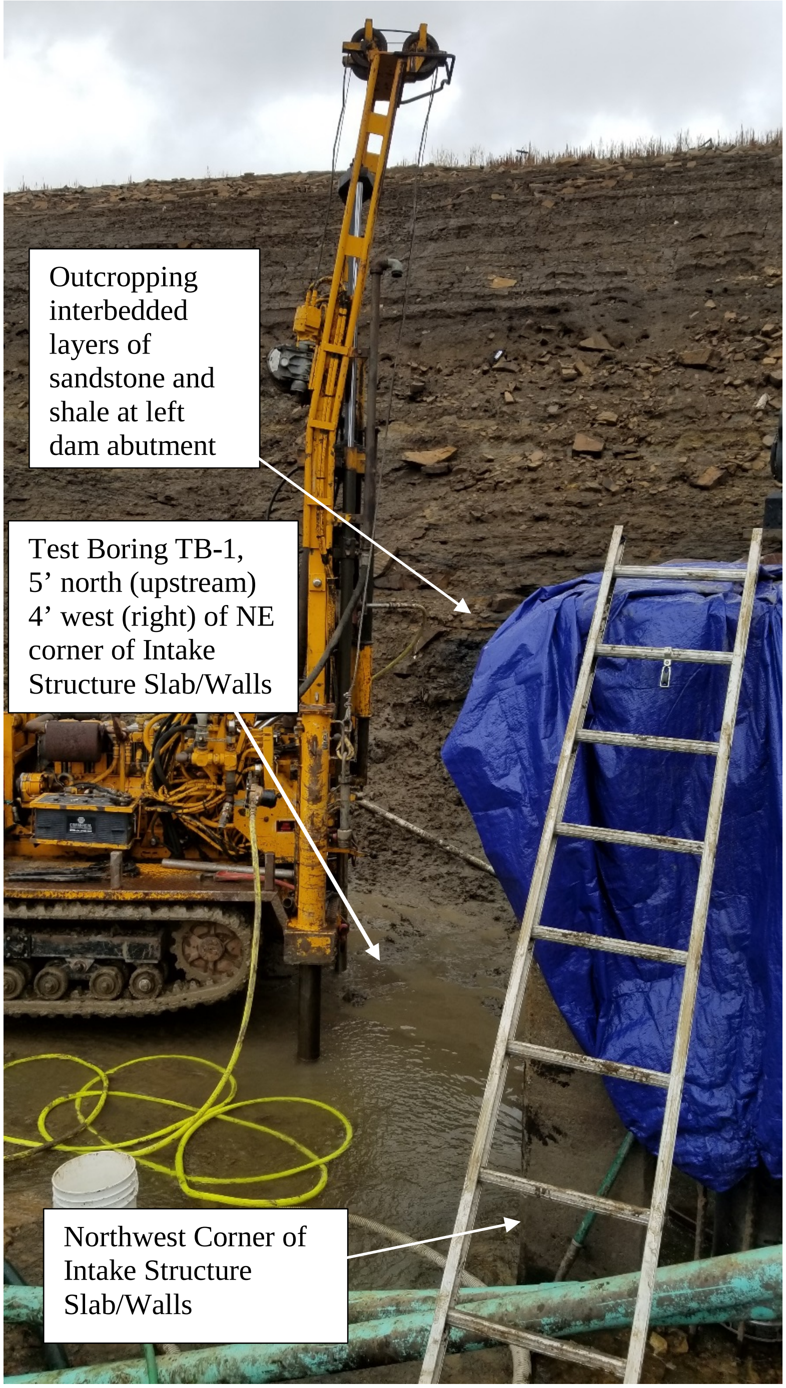
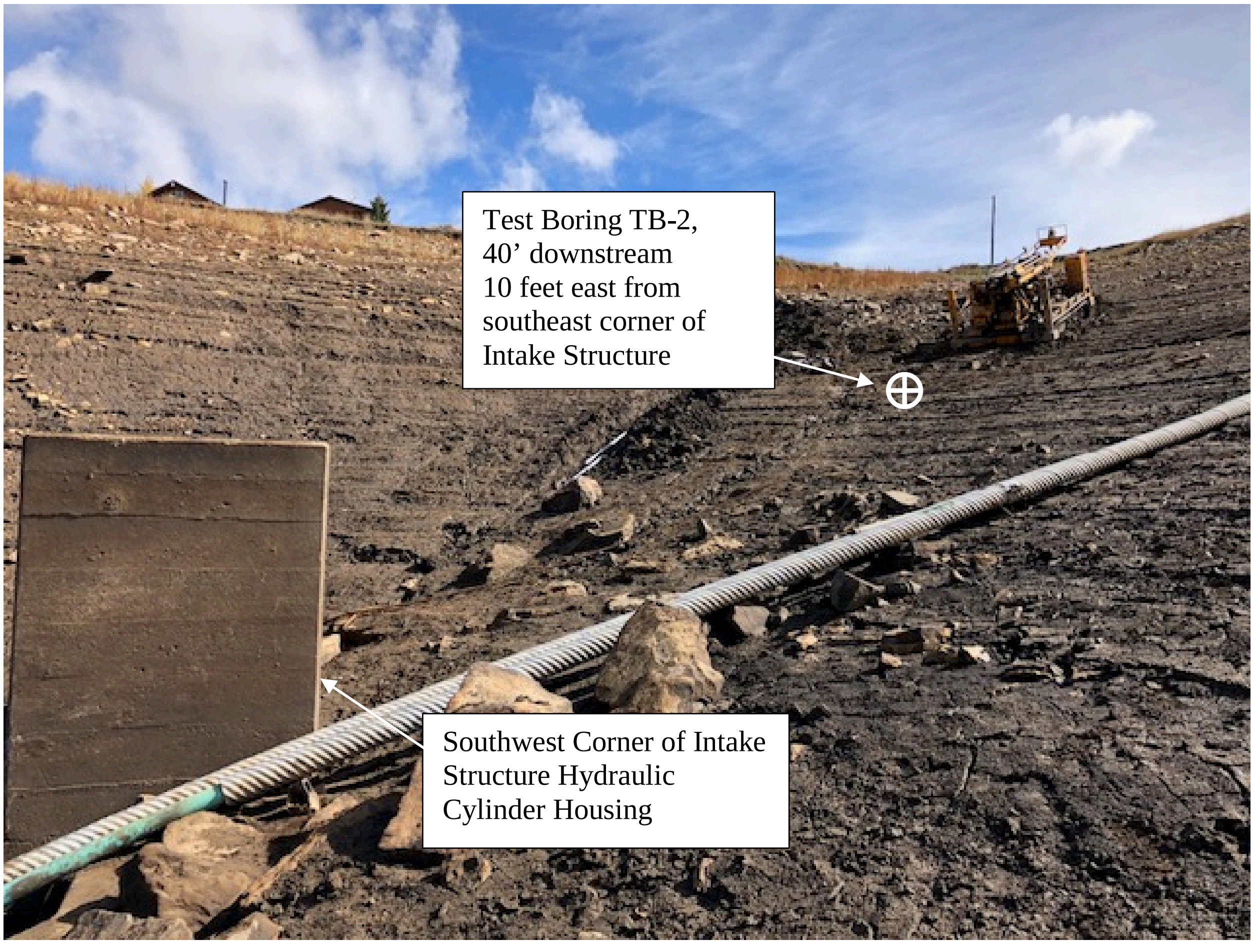


*Figure 3.6: Photographic Location of Test Boring TB-2*



### *Test Boring TB-1 Textual Description*

Test Boring TB-1 was advanced on October 11, 2018. The drilling equipment was accessed to the boring location from the intake area structure access road (indicated on Figure 3.2 above). The test boring was located approximately 5 feet north (upstream) and 4 feet west (right) of the northeast corner of the existing intake structure slab and supported trash rack walls. The elevation of the boring was at the flowline elevation of the intake structure slab.

We initially used 4 inch diameter solid stem auger to advance the boring. We encountered sandstone cobbles with a sandy clay matrix from the intake flowline elevation to a depth of about 2 feet below the intake flowline elevation. We suspect that this first two feet of material may have been rip-rap type material placed around the intake structure slab. It should be noted that water was actively flowing at our test boring location, obscuring observations of the surface materials. At a depth of approximately 2 feet below the intake slab flowline elevation we encountered the Dakota/Burro Canyon Sandstone Formation. The initial formational materials encountered were fractured and consisted of saturated interbedded layers of sandstone, shale, and claystone materials. Standard penetration values (N-Values) at a depth of about 0.5 feet into the formational materials were about N=57 with a Modified California Barrel (2.5 feet below the intake flowline elevation).

We encountered auger refusal in a very hard sandstone layer of the formational material at a depth of about 4.5 feet below the flowline elevation of the intake structure. At this point we switched to NWL wireline core drilling techniques (NQ diameter core). At depths ranging from about 4.5 to 15.5 feet below the intake flowline elevation we encountered highly fracture interbedded layers of sandstone and shale. Rock Quality Designation (RQD) ranged from about 0 to 32 percent within this depth. At depths below about 15.5 feet below the intake flowline elevation we predominately encountered gray to white colored and moderately to slightly fractured sandstone to the bottom of the boring advanced to a depth of about 24 feet below intake flowline elevation. RQD for this zone of material (15.5 to 24 feet below intake flowline elevation) ranged from about 73 to 85 percent. A low-grade coal seam was encountered at depths ranging from about 19 to 20 feet below the intake flowline elevation. The test boring was backfilled with fluid grout pumped via a tremie tube placed to the bottom of the boring.

Figures 3.7 and 3.8 presented below are photographs that were obtained from the rock core. It should be noted that some sections of rock core had been removed for unconfined compressive strength testing prior to the time the photographs were obtained.

*Figure 3.7: Rock Core Obtained from Test Boring TB-1, 4.5 to 15.5 feet below the Existing Intake Structure Flowline Elevation (Core Runs 1 through 3)*

*Figure 3.8: Rock Core Obtained from Test Boring TB-1, 15.5 to 24.5 feet below the Existing Intake Structure Flowline Elevation (Core Runs 4 and 5)*

### *Test Boring TB-2 Textual Description*

Test Boring TB-2 was advanced on October 12, 2018 on the upstream face of the dam at a location of about 40 feet south (downstream) and 10 feet east from the southeast corner of the intake structure. The test boring was advanced about 20 feet above the flowline elevation of the intake structure. We were not able to access closer to the intake structure due to very soft/saturated surface soil conditions, and constraints from the somewhat unknown alignment of the outlet works tunnel structure. The location of this test boring was selected to gain information regarding the characteristics of the soil materials that are located upslope and retained by the existing intake structure wing walls.

In Test Boring TB-2 we encountered very soft and saturated sandy clay soil materials from the dam face elevation to a depth of about 2 feet where we encountered a mixture of medium stiff and saturated sandy clay soil with gravel to a depth of 5 feet below the dam face elevation. At a depth of 5 feet below the dam face elevation we encountered dense/stiff sandy clay soil with gravel and cobbles to a depth of about 10.5 feet below the ground surface elevation where we encountered the Dakota/Burro Canyon Sandstone Formation. We suspect that the upper approximate 10.5 feet of material encountered in our boring was previously placed fill material within the concrete tunnel excavation sidewalls.

The elevation of the formational materials encountered in our test boring appeared to coincide with exposed outcrops of the formational materials within the dam basin sidewalls northeast of the intake structure (indicated on Figure 3.5 presented above). The upper portions of the formational materials (at depths ranging from about 10.5 to 16 feet below the dam face elevation at our test boring location) consisted of hard to very hard interbedded layers of sandstone, claystone, and shale. We encountered a very hard layer of sandstone at depths ranging from about 16 to 17 feet below the dam face elevation. Auger refusal occurred at a depth of 17 feet below the dam elevation.

Subsurface free water was measured at a depth of about 16.5 feet below the ground surface elevation after we completed the boring. We suspect that the subsurface free water elevation would have risen above the depth that we measured given additional time. The test boring was backfilled with fluid grout pumped down a tremie tube placed to the bottom of the boring.

The logs of the subsurface soil conditions encountered in our test borings are presented in Appendix A. The logs present our interpretation of the subsurface conditions encountered exposed in the test borings at the time of our field work. Subsurface soil and water conditions are often variable across relatively short distances. It is likely that variable subsurface soil and water conditions will be encountered during construction. Laboratory soil classifications of samples obtained may differ from field classifications.

## 4.0  LABORATORY STUDY

The laboratory study included tests to estimate the strength, swell and consolidation potential of the soils tested.  We performed the following tests on select samples obtained from the test borings.

***Moisture content and Dry Density****;* the moisture content and in-situ dry density of a Modified California Barrel liner sample was tested as part of the swell-consolidation testing for the project.  The results of this test are tabulated  in Table 4.1 below.  We have also provided unit weights of select section of rock core that were tested for unconfined compressive strength (tabulated results provided in Table 4.2 below).

*Table 4.1: Moisture Content and Dry Density of Weathered Formational Materials*

| **Sample Location** | **Sample Description** | **Sample Moisture Content** | **Sample Dry Density** |
|---|---|---|---|
| TB-1, 2.5 feet below Intake Flowline Elevation | Shale | 11.6 | 126.2 |
| TB-1, 3 feet below Intake Flowline Elevation | Shale with interbedded Sandstone | 7.1 | 134.8 |

***Unconfined Compressive Strength of Rock Core;*** the unit weight and unconfined compressive strength characteristics of select sections of rock core obtained from Test Boring TB-1 was performed.  The results of the unconfined compressive strength tests are tabulated in Table 4.2 below.  The tabulated depth of the samples is based on the depth below the flowline elevation of the existing intake structure.

*Table 4.2: Unconfined Compressive Strength Test Results for Rock Core*

| **Test Sample Location and Depth** | **Sample Description** | **Unit Weight (pcf)** | **Unconfined Compressive Strength (psi)** |
|---|---|---|---|
| TB-1, 8 feet | Shale | 154.6 | 6,650 |
| TB-1, 12 feet | Medium Grain white to gray sandstone | 160.1 | 13,600 |
| TB-1, 16 feet | Medium Grain white to gray sandstone | 145.6 | 5,630 |
| TB-1, 21 feet | Fine Grain white to gray sandstone | 157.9 | 8,400 |

***Swell-Consolidation Tests;*** the one-dimensional swell-consolidation potential of the driven Modified California liner samples obtained from Test Boring TB-1 at depths ranging from about 2.5 to 3.5 below the flowline elevation the existing intake structure was assessed. We suspect that these samples likely represent the formational materials with the highest potential for swell that could be located under the new bulkhead structure and are likely currently located under the existing intake structure.

The samples were inundated with water at surcharge loads of 250 pounds per square foot and 500 pounds per square foot. The one-dimensional swell-consolidation response of the samples tested are provided on Figures 4.2 and 4.3 of Appendix A. A summary of the pertinent information related to the swell potential of the test samples is tabulated in Table 4.3 presented below.

*Table 4.3: Measured Swell Pressure and Potential of the Weathered Formational Shale*

| Sample Designation | Moisture Content (percent) | Dry Density (PCF) | Measured Swell Pressure* (PSF) | Swell Potential |
|---|---|---|---|---|
| TB-1, 2.5 feet below flowline of Existing Intake Structure | 11.6 | 126.2 | 1,470 | 0.3 %<br>(500 psf surcharge) |
| TB-1, 3 feet below flowline of Existing Intake Structure | 7.1 | 134.8 | 1,880 | 1.7%<br>(250 psf surcharge) |

*NOTE: We determine the swell pressure as measured in our laboratory using the constant volume method. The graphically determined swell pressure may be different from that measured in the laboratory.

The samples tested exhibit a low to moderate swell potential when surcharged to loads ranging from about 250 to 500 pounds per square foot. Based on our laboratory test results, we anticipate that up to about ¼ to ½ inch of post construction heave could occur within the upper approximate 2 feet of the weathered formational materials provided the dead load of the structure is in the range of about 250 to 500 pounds per square foot. The formational materials encountered at depths below about 4.5 feet below the existing intake structure flowline elevation primarily consist of sandstone material which will not exhibit a significant swell potential.

***Sieve Analysis and Atterberg Limits;*** we performed sieve analysis (in accordance with ASTM D422 and ASTM C136) and Atterberg Limits tests (in accordance with ASTM D4318) for the soil materials encountered in Test Boring TB-2 at depths ranging from the dam face surface to a depth of 5 feet. This sample was selected as we anticipate this is the type of material that will eventually accumulate and/or be located in areas adjacent to potential retaining/wing walls associated with the new bulkhead structure. The samples tested classify as USCS type “CL” sandy lean clay. The sample tested exhibits about 58 percent passing the #200 sieve screen with a liquid limit of 33, plastic limit of 18, and a corresponding plasticity index of 15. The results of the sieve analysis and Atterberg Limits tests are provided on Figure 4.1 of Appendix B.

***Direct Shear Strength tests;*** residual strength direct shear tests were performed on select soil samples to estimate the soil strength characteristics in general accordance with ASTM D3080. The selected test sample was obtained from Test Boring TB-2 from the dam face elevation to a depth of about 5 feet below the dam face elevation. As with the sieve analysis and Atterberg Limits testing, we anticipate that this type of material is the moist likely to collect form sediment transportation against/behind potential retaining structures associated with the project. The results of the direct shear strength tests are provided on Figure 4.4 of Appendix B. We obtained an estimated angle of internal friction (phi) of about 31 degrees and a cohesion of about 160 pounds per square foot for drained conditions. We are available to provide consolidated-drained (CD) or consolidated-undrained (CU) triaxial shear testing for the project if a select structure backfill material is selected.

***Soluble Sulfates Tests;*** the soluble sulfate content of the weathered formational materials encountered in Test Boring TB-1 at a depth of about 3.5 to 4.5 feet below the existing intake structure flowline elevation were assessed. We obtained a soluble sulfate concentration of about 400 parts per million which constitutes a moderate sulfate exposure level.

The American Concrete Institute recommends a maximum water/cement ratio of 0.50 and either a type II, IP(MS), IS(MS), P(MS), I(PM)(MS), or a I(SM)(MS) cement for soils with a moderate sulfate exposure level. We recommend that Portland cement concrete and/or grout exhibit a minimum compressive strength of at least 4,000 pounds per square inch for this project.

As discussed in Section 3.2 above, we did not observe evidence that would indicate that substantial degradation to the intake structure concrete from phenomena such as sulfate attack or alkali-silica reactions has occurred in the past. The characteristics of the reinforcement steel in the existing intake structure is not known.

## 5.0 FOUNDATION RECOMMENDATIONS

The details of the proposed intake bulkhead structure are not known at this time. We anticipate that the foundation system for the structure will consist of a new steel reinforced mat slab structure, and that the dimensions of the mat slab will be roughly equal to the existing intake structure slab area. The recommendations provided below are based on these general assumptions. We should be contacted once the proposed structure concept has been developed to verify our recommendations

The structural interface or connection between the new bulkhead structure and the existing intake structure or intake tunnel (existing outlet work components) will need to be examined by the project structural engineer to verify that potential forces that act on the new bulkhead structure, such as buoyant forces, do not negatively influence the structural integrity of the existing outlet work structures.

The integrity and long-term performance of any type of system is influenced by the quality of workmanship which is implemented during construction. It is imperative that all excavation and fill placement operations be conducted by qualified personnel using appropriate equipment and techniques to provide suitable support conditions for the foundation system.

### 5.1 *Recommended Mat Slab Support Elevation and Allowable Bearing Capacity*

The anticipated steel reinforced mat slab should be supported directly over the more competent formational materials that we encountered in Test Boring TB-1 at a depth of about 4.5 feet below the flowline elevation of the existing intake structure. This support elevation will provide the following benefits to the structure;

- High bearing capacity conditions with negligible potential for post construction settlement.
- Decreased potential for heave due to the expansive conditions of the shallow formational shale materials that we encountered at depths ranging from about 2.5 to 3.5 feet below the existing intake structure flowline elevation.
- Additional embedment to help resist potential scour below and around the foundation system.
- Decreased potential for frost heave in the event the reservoir is drained during freezing ambient air conditions.

Based on the subsurface conditions encountered at our Test Boring TB-1 location, we anticipate that this bearing elevation can be established using conventional large excavation equipment. It may be necessary to utilize hydraulic or pneumatic percussion hammers to assist with the excavation. Blasting must not be used for the project excavation. If needed expansive grout or other non-explosive means may be incorporated into the excavation effort. We should be contacted to observe the foundation excavation process to verify the competency of the formational materials. It is possible that refusal of the excavation equipment may occur at shallower depths than discussed above in some areas of the foundation excavation. We recommend that some flexibility regarding the bearing elevation of the foundation system be designed for.

The foundation excavation for the mat slab should be excavated as carefully as possible to the designed outside dimensions of the mat slab. The foundation concrete should be placed directly against the undisturbed sidewalls of the excavation. We do not recommend that soil type backfill materials be used to backfill between the foundation system concrete and structure excavation, due to the probability that the backfill materials will be compromised in the future from forces associated with waterflow into the intake structure.

The new foundation system may be designed using an allowable bearing capacity of 7,500 pounds per square foot. We should be contacted if additional bearing capacity is needed for the project.

We anticipate that utilizing the existing intake structure foundation system for the new bulkhead structure may be considered. At this time, we do not know the support elevation/characteristics of the existing intake structure foundation system. In addition, we anticipate that the steel reinforcement design for the existing intake structure is not known. We are available to assist with determining the support characteristics of the existing intake structure foundation system at your request. This will likely require angled core drilling through the floor slab of the existing intake structure due to the geometry of the existing structure.

### 5.2 *Recommendations for Micropile Anchors to Resist Uplift Forces*

Micropiles may be used to resolve uplift forces such as buoyant and hydraulic uplift forces. The anchors may also be used to resolve lateral forces that may act on the structure.

Based on the RQD data obtained from Test Boring TB-1, the upper approximate 13 feet of the formational materials are highly fractures with RQD percentages ranging from 0 to about 32 percent. At a depth of about 15 feet below the flowline elevation of the existing intake structure the formational materials become much more competent with RQD percentages ranging from about 85 to 73 percent. We recommend that the subsurface materials to a depth of 15 feet below the flowing elevation of the existing intake structure be neglected from contributing to bond capacity of the micropile elements. The upper materials should be grouted during the installation of the micorpile elements, but not accounted for contribution to bond capacity.

The micropile elements should be embedded to a minimum depth of at least 25 feet below the flowline elevation of the existing intake structure. An estimated allowable capacity of 5 kips per foot of embedment (embedment beyond 15 feet below the flowline elevation of the existing intake structure) may be used for the initial micropile design for boring diameters ranging from 3.5 to 4.5 inches. This estimated allowable capacity may be used for both tensional and compression forces. We are available to provide estimated tensional and compression capacities for larger diameter micropile elements at your request. The estimated allowable capacity must be verified with testing performed on both sacrificial and production anchors. This micropile testing is discussed in more detain later in this section of the report.

We recommend that the following general design and construction procedures be used for micropiles;

- The micropile steel reinforcement should consist of minimum 150 ksi solid bar steel, and at minimum should be epoxy coated to resist corrosion. The steel manufacturer should be contacted to discuss additional corrosion protection measures for the steel. We recommend that an aggressive corrosion protection criteria be used for the micropile reinforcement steel.

- A minimum boring diameter of 3.5 inches should be used to construct the micropiles.
- The formational materials encountered and tested exhibit unconfined compressive strengths up to at least 13,600 psi based on our laboratory testing. The selected drilling equipment/drilling type must be capable of advancing the micropile borings through very hard sandstone materials. If possible, we recommend using water and/or air to clean/flush the boring. We do not recommend using bentonite type drilling fluid as this may reduce the grout bond capacity of the micropiles. Subsurface free water within the micropile borings will likely exist.
- The boring diameter must be appropriately sized for the selected reinforcement steel such that a minimum 1-inch thick grout column between the steel reinforcement and circumference of the boring exists.
- The micropile reinforcement steel should be placed in the boring with appropriately sized centralizers to ensure that the steel reinforcement is centered within the boring.
- Cement grout should be pumped through a tremie tube that is located at the bottom of the micropile boring to verify that the grout column extends from the bottom of the boring to the surface of the boring. The subsurface free water that will likely exist in the micropile boring may be displaced by the tremied grout. The tremie tube should be removed after the grout has been placed.
- The cement grout should exhibit a maximum water to cement ratio of 0.45 and achieve a minimum compressive strength of at least 5,000 psi at 28 days. The project structural engineer may have additional grout related requirements. Sufficient grout should be pumped into the borings such that the exiting grout exhibits the appropriate water to cement ratio.
- The micropile center-to-center spacing should be at least 30 inches or 3 micropile boring diameters, whichever is greater.
- Due to the relatively high strength characteristics that will likely develop for the micropile elements, we anticipate that minimal elements will be necessary to resist potential uplift forces. However, we recommend that a high level of redundancy be designed for this project. The micropile elements should be appropriately spaced/patterned to verify that uplift forces are resolved equally across the mat slab foundation area.

We recommend that at least one sacrificial micropile be tested for creep and failure at a load of at least 200 percent of the determined design load for the micropiles. It will likely be necessary to only grout the lower portion of the sacrificial micropile within the design bond depth (15 feet below the flowline elevation of the existing intake structure) to achieve failure of the element. The sacrificial test load must not exceed 80 percent of the yield strength of the reinforcement steel. Due to the anticipated relatively small size of the structure and critical nature of the structure, we recommend that every production pile be proof tested at a minimum load of 160% of the determined structure design load for the micropiles. The project structural engineer should determine the actual load testing criteria for the micropiles. We are available to assist with the development of the load testing schedule for the project.

The project structural engineer should be contacted to assess the lateral capacity of the micropile reinforcement steel. We do not recommend accounting for lateral capacity of vertical micropile elements unless the foundation mat slab is bearing on the competent formational materials as discussed in Section 5.1 above, and the micropiles are thoroughly grouted to the surface of the micropile boring which is located directly at the base elevation of the mat slab foundation structure. Battered micropile components may also be used to resolve lateral forces that may act on the structure.

### 5.3 *Lateral Earth Pressure Values for the Existing Dam Embankment Near-Surface Soils*

We anticipate that laterally loaded walls may be included with the new proposed bulkhead structure. We have provided lateral earth pressure values for the sandy clay soil materials that we encountered at the existing upstream dam face to a depth of about 5 feet in Test Boring TB-2 below. Based on our understanding of the project, the highest lateral loads associated with soil materials acting against retaining structures will occur immediately during/after an event where the reservoir is drained and the backfill soils are still fully saturated. Lateral earth pressure values for the existing near surface soils encountered at our Test Boring TB-2 location are provided in Table 5.1 below. We have also provided values for an imported granular structure fill such as CDOT Class 2 or Class 6 material in the table below. The tabulated values reflect fully saturated soil conditions.

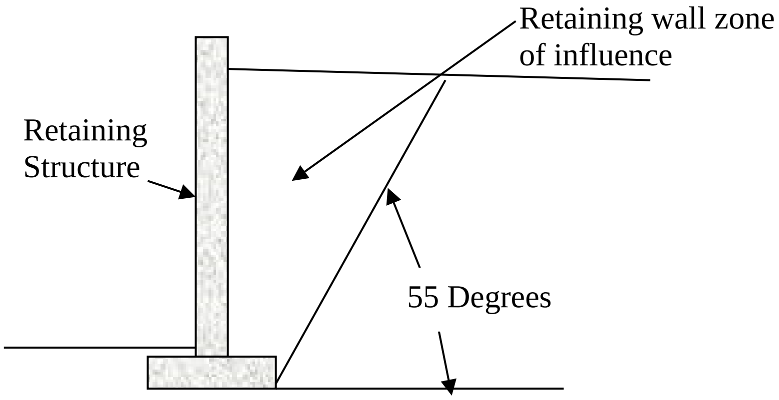
*Table 5.1: Lateral Earth Pressure Values (Saturated Conditions)*

| | Saturated Lateral Earth Pressure Values | |
|---|---|---|
| Type of Lateral Earth Pressure | Level Native Soil Backfill (pounds per cubic foot/foot) | Level Granular Soil Backfill (pounds per cubic foot/foot) |
| Active | 92 | 82 |
| At-rest | 105 | 95 |
| Passive* | 240 | 330 |

*The passive pressures tabulated above are applicable for the types of soil materials listed. The passive resistance provided by the formational sandstone materials against the edges of the mat slab foundation system will be higher. The passive resistance values for the formational sandstone materials are discussed below.

A passive pressure of 750 pounds per cubic foot per foot may be assumed for the portion of the mat slab concrete that is place directly against the undisturbed formational materials in the sidewall of the foundation excavation. This capacity is only valid if the mat slab concrete is placed directly against the undisturbed formational materials. A coefficient of friction of 0.50 may be used to resist sliding provided the mat slab foundation concrete is placed directly over the clean competent formational materials.

The granular imported soil backfill values tabulated above are appropriate for material with an angle of internal friction of 35 degrees, or greater. The granular backfill must be placed within the retaining structure zone of influence as shown below in order for the lateral earth pressure values tabulated above for the granular material to be appropriate.



Retaining Structure Zone of Influence Concept, No Scale

Backfill should not be placed and compacted behind the retaining structure unless approved by the project structural engineer. Backfill placed prior to construction of all appropriate structural members such as floors, or prior to appropriate curing of the retaining wall concrete (if used) may result in severe damage and/or failure of the retaining structure.

## 6.0 CONSTRUCTION CONSIDERATIONS

This section of the report provides comments, considerations and recommendations for aspects of the site construction which may influence, or be influenced by the geotechnical engineering considerations discussed above. The information presented below is not intended to discuss all aspects of the site construction conditions and considerations that may be encountered as the project progresses. If any questions arise as a result of our recommendations presented above, or if unexpected subsurface conditions are encountered during construction we should be contacted immediately.

### 6.1 *Fill Placement Recommendations*

There are several references throughout this report regarding both natural soil and compacted structural fill recommendations for backfill against potential retaining structures. The recommendations presented below are appropriate for the fill placement considerations discussed throughout the report above.

All areas to receive fill, structural components, or other site improvements should be properly prepared and grubbed at the initiation of the project construction.  The grubbing operations should include scarification and removal of organic material and soil.  No fill material or concrete should be placed in areas where existing vegetation or fill material exist.

### 6.1.1 *Natural Soil Fill*

Any natural soil used for any fill purpose should be free of all deleterious material, such as organic material and construction debris.  Natural soil fill includes excavated and replaced material or in-place scarified material.

The natural soils should be moisture conditioned, either by addition of water to dry soils, or by processing to allow drying of wet soils.  The proposed fill materials should be moisture conditioned to between about optimum and about 2 percent above optimum soil moisture content.

The moisture conditioned soil should be placed in lifts that do not exceed the capabilities of the compaction equipment used and compacted to at least 95 percent of maximum dry density as defined by ASTM D698, standard Proctor test.  We typically recommend a maximum fill lift thickness of 6 inches for hand operated equipment and 8 to 10 inches for larger equipment.

### 6.1.2 *Granular Compacted Structural Fill*

Granular compacted structural fill should be constructed using an imported commercially produced rock product such as aggregate road base.  Many products other than road base, such as select crusher fines may be suitable, depending on the intended use.  If a specification is needed by the design professional for development of project specifications, a material conforming to the Colorado Department of Transportation (CDOT) “Class 6” aggregate road base material can be specified.  This specification can include an option for testing and approval in the event the contractor’s desired material does not conform to the Class 6 aggregate specifications.  We have provided the CDOT Specifications for Class 6 material below

| Grading of CDOT  Class 6 Aggregate Base-Course Material | |
|---|---|
| Sieve Size | Percent Passing Each Sieve |
| ¾ inch | 100 |
| #4 | 30 – 65 |
| #8 | 25 – 55 |
| #200 | 3 – 12 |

Liquid Limit less than 30

All compacted structural fill should be moisture conditioned and compacted to at least 95 percent of maximum dry density as defined by ASTM D698, standard Proctor test.

### 6.2 *Excavation Considerations*

Unless a specific classification is performed, the site soils should be considered as an Occupational Safety and Health Administration (OSHA) Type C soil and should be sloped and/or benched according to the current OSHA regulations. Excavations should be sloped and benched to prevent wall collapse. Any soil can release suddenly and cave unexpectedly from excavation walls, particularly if the soils is very moist, or if fractures within the soil are present. Daily observations of the excavations should be conducted by OSHA competent site personnel to assess safety considerations.

It will likely be necessary to dewater excavations during the construction phase of the project. The dewatering system must be carefully design to provide suitable working/construction conditions for the project.

Excavations into the formational materials on the project site will require large excavation equipment and possibly substantial effort. Hydraulic or pneumatic excavator mounted hammers may be required in the harder layers of the formational materials. Blasting techniques must not be used for the project excavations. If needed expansive grout or other non-explosive means may be incorporated into the excavation effort.

#### 6.2.1 *Excavation Cut Slopes*

We anticipate that some permanent excavation cut slopes may be included in the site development. Temporary cut slopes should not exceed 5 feet in height and should not be steeper than about 1:1, horizontal to vertical. Permanent cut slopes may need to be analyzed on a site specific basis. In general, permanent excavation cut slopes should not be steeper than the approximate 3:1, h:v existing interior dam face embankment slopes.

#### 6.2.2 *General Site Subgrade Stabilization Techniques for the Project Construction*

We suspect that very soft and saturated soil conditions will be encountered in the project area during construction. This section provides a general concept that may be considered to help stabilize the subgrade soils around the project site construction area to facilitate access of heavy equipment and personnel.

Chemical stabilization using Portland cement is effective for most soils. Generally dry Portland cement powder may be placed on the surface of the soft yielding material and subsequently mixed into the soil. The effectiveness of this technique is partially dependent upon the thoroughness of the mixing. We suspect that an application rate of about 10 to 20 percent of Portland cement will help dry and stabilize the subgrade soils in the area of the project site.

After mixing, the material should be allowed to “rest” for about two of more hours prior to proof compaction.  The treated material will often yield some during initial compaction, but will generally increase in rigidity as the process of hydration begins takes place.  If yielding under compaction is excessive, the material should be allowed “cure” additionally prior to continued compaction effort being applied.  Often it takes more time, such as overnight, to allow the cement to fully stabilize the material so this strategy is often implement in an area at the end of a work day and allowed to cure overnight followed by subsequent fill placement on the following day.

### 6.3 *Utility Considerations*

Some movement of all structural components is normal and expected.  The amount of movement may be greater on sites with problematic soil conditions.  Utility line penetrations through any walls or floor slabs should be sleeved so that movement of the walls or slabs does not induce movement or stress in the utility line.  Utility connections should be flexible to allow for some movement of the structure.

## 7.0  CONSTRUCTION MONITORING AND TESTING

Construction monitoring including engineering observations and materials testing during construction is a critical aspect of the geotechnical engineering contribution to any project. Unexpected subsurface conditions are often encountered during construction. The site foundation excavation should be observed by the geotechnical engineer or a representative during the early stages of the site construction to verify that the actual subsurface soil and water conditions were properly characterized as part of field exploration, laboratory testing and engineering analysis.  If the subsurface conditions encountered during construction are different than those that were the basis of the geotechnical engineering report then modifications to the design may be implemented prior to placement of fill materials or foundation concrete.

Compaction testing of fill material should be performed throughout the project construction so that the engineer and contractor may monitor the quality of the fill placement techniques being used at the site.  We recommend that compaction testing be performed for any fill material that is placed as part of the site development.  Compaction tests should be performed on each lift of material placed for critical areas of the project such as retaining wall backfill.  In addition to compaction testing we recommend that the grain size distribution, clay content and swell potential be evaluated for any imported materials that are planned for use on the site.  Concrete tests should be performed on foundation concrete and flatwork.  We are available to develop a testing program for soil, aggregate materials, and concrete for the project.

## 8.0  CONCLUSIONS AND CONSIDERATIONS

The information presented in this report is based on our understanding of the proposed construction that was provided to us and on the data obtained from our field and laboratory studies.  We recommend that we be contacted during the design and construction phase of this project to aid in the implementation of our recommendations.  Please contact us immediately if you have any questions, or if any of the information presented above is not appropriate for the proposed site construction.

The recommendations presented above are intended to be used only for this project site and our understanding of the proposed project construction.  The recommendations presented above are not suitable for adjacent project sites, or for proposed construction that is different than that outlined for this study.

Our recommendations are based on limited field and laboratory sampling and testing. Unexpected subsurface conditions encountered during construction may alter our recommendations.  We should be contacted during construction to observe the exposed subsurface soil conditions to provide comments and verification of our recommendations. We are available to review and tailor our recommendations as the project progresses and additional information which may influence our recommendations becomes available.

Please contact us if you have any questions, or if we may be of additional service.

Respectfully,
**TRAUTNER GEOTECH**

COLORADO REGISTERED PROFESSIONAL ENGINEER
JONATHAN P. BUTLER
11/28/18

Reviewed

COLORADO LICENSED PROFESSIONAL ENGINEER
DAVID LEE TRAUTNER
29165

Jonathan P. Butler, P.E.
Senior Engineer

David L. Trautner, P.E.
Principal Geotechnical Engineer

# APPENDIX A

## Logs of Test Borings

**TRAUTNER GEOTECH LLC**

# LOG OF BORING TB-1

Field Engineer : J. Butler
Drilling Method : 4" Solid/NWL wireline
Date Drilled : 10/11/2018
Total Depth : 24 feet
Location : 5' Upstream (North),
: 4' West (right) of Northeast
: corner of Intake Structure
Elevation : ~Flowline of Intake
Water Table : 6" above drill elevation

Groundhog Reservoir Intake Bulkhead Structure
(Intake structure for the Reservoir Outlet Tunnel)
Dolores (area), Colorado
Mr. Brandon Johnson, M.V.I.

PN: 55415GE

☒ Bag Sample
▨ Core Run
▦ Mod. California Sampler

| Depth in feet | DESCRIPTION | USCS | GRAPHIC | Samples | Run | Water Level | RECOVERY, R.Q.D. |
|---|---|---|---|---|---|---|---|
| 0<br>1 | POSSIBLE RIP-RAP MATERIAL, COBBLES, clayey, medium dense, wet, brown | GC | | | | ▽ | Water level at 6 inches above drill elevation |
| 2<br>3<br>4 | DAKOTA AND BURRO CANYON FORMATION at 2 feet, Shale/Claystone with interbedded Sandstone lenses, hard to very hard, wet, brown/gray | Formation | | ▦<br>☒ | 20/6<br>37/6 | | |
| 5<br>6<br>7<br>8<br>9<br>10<br>11<br>12<br>13<br>14<br>15 | Auger refusal at 4.5 feet, Sandstone and Shale, highly fractured, very hard, white to gray | Formation | | ▨ | Run One<br>Run Two<br>Run Three | | Run One<br>4.5 feet to 5.5 feet<br>Recovery=63%<br>R.Q.D.=0%<br>Run Two<br>5.5 feet to 10.5 feet<br>Recovery=88%<br>R.Q.D.=20%<br>Run Three<br>10.5 feet to 15.5 feet<br>Recovery=100%<br>R.Q.D.=32% |
| 16<br>17<br>18 | Sandstone, moderate to low fracturing, very hard, gray to white | Formation | | ▨ | Run Four | | Run Four<br>15.5 feet to 20 feet<br>Recovery=100%<br>R.Q.D.=85% |
| 19 | Coal Layer 19' to 20' | Formation | ■ | ▨ | | | |
| 20<br>21<br>22<br>23 | Sandstone, moderate fracturing, very hard, gray to white | Formation | | ▨ | Run Five | | Run Five<br>20 feet to 24 feet<br>Recovery=97%<br>R.Q.D.=73% |
| 24<br>25 | Bottom of Test Core at 24 feet | | | | | | |

**TRAUTNER GEOTECH LLC**

Field Engineer : J. Butler
Drilling Method : 4" Solid
Sampling Method : Mod. California Sampler
Date Drilled : 10/12/2018
Total Depth : 17 feet
Location : 40' Downstream
: 10' East (left) from
: Southeast corner of
: Intake Structure
Elevation : ~20' above Flowline

# LOG OF BORING TB-2

Groundhog Reservoir Intake Bulkhead Structure
(Intake structure for the Reservoir Outlet Tunnel)
Dolores (area), Colorado
Mr. Brandon Johnson, M.V.I.

PN: 55415GE

Sample Type
- ▦ Mod. California Sampler
- ▨ Bag Sample
- ■ Standard Split Spoon

Water Level
- ▼ Water Level During Drilling
- ▽ Water Level After Drilling

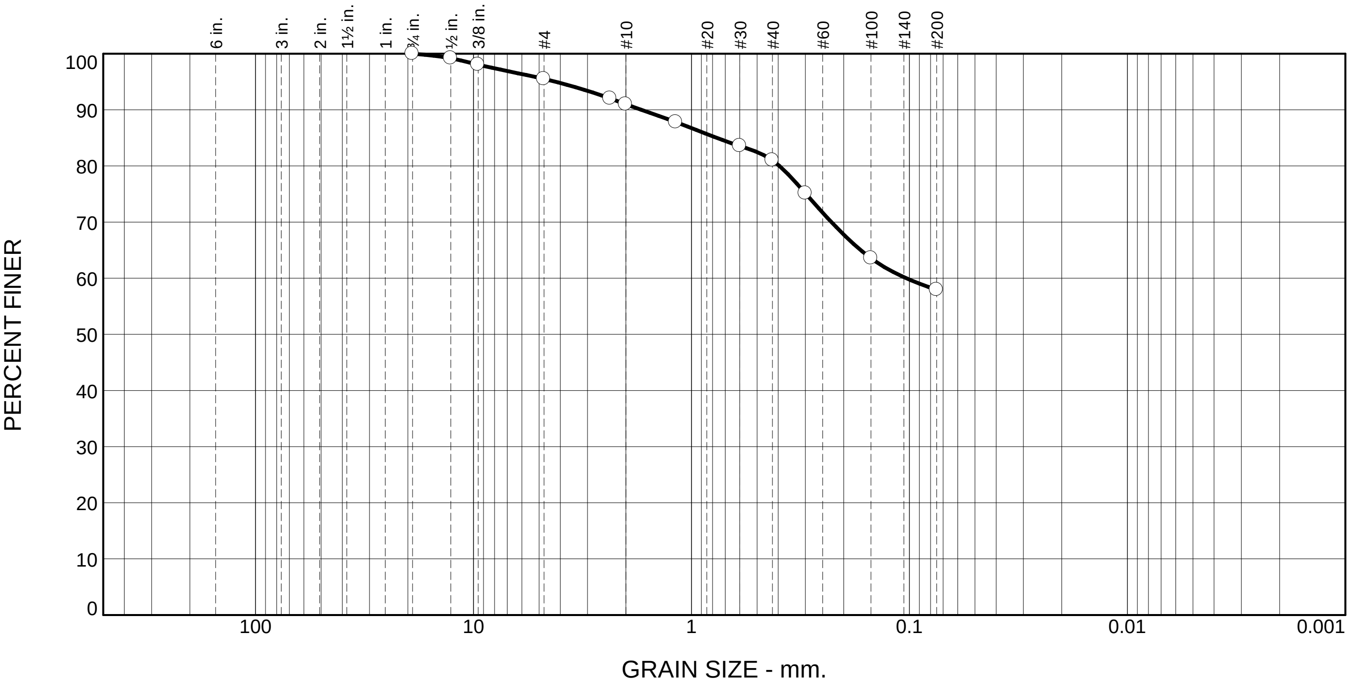
| Depth in feet | DESCRIPTION | USCS | GRAPHIC | Samples | Blow Count | Water Level | REMARKS |
|---|---|---|---|---|---|---|---|
| 0 | CLAY, sandy, very soft, wet, dark brown | CL | ▨ | ▨ | | | Possible Tunnel, (Man-placed back-fill material to 10.5 feet) |
| 2 | CLAY, GRAVEL, sandy, cobbles, medium stiff, wet, dark brown | CL/GC | ▨ | ▨ | | | |
| 5 | CLAY, GRAVEL, sandy, cobbles, dense, wet, brown | CL/GC | ▨ | ▨<br>■<br>▨ | 5/6<br>9/6<br>12/6 | | |
| | DAKOTA AND BURRO CANYON FORMATION at 10.5 feet, Sandstone, Shale, with Claystone lenses, hard to very hard | Formation | | ▨<br>▦ | 16/6<br>28/6 | | |
| 16 | Sandstone 16'-17', very hard | Formation | | | | ▽ | Water Level After Drilling |
| 17 | Auger refusal at 17 feet | | | | | | |
| 18 | | | | | | | |
| 19 | | | | | | | |
| 20 | | | | | | | |

# APPENDIX B

## Laboratory Test Result

Figure 4.1: Sieve Analysis and Atterberg Limits
Figure 4.2 and 4.3: Swell Consolidation Test Results
Figure 4.4: Direct Shear Strength Test Results

# Particle Size Distribution Report



| % +3" | % Gravel | | % Sand | | | % Fines | |
|---|---|---|---|---|---|---|---|
| | Coarse | Fine | Coarse | Medium | Fine | Silt | Clay |
| 0.0 | 0.0 | 4.5 | 4.5 | 10.0 | 23.1 | 57.9 | |

| TEST RESULTS | | | |
|---|---|---|---|
| Opening Size | Percent Finer | Spec.* (Percent) | Pass? (X=Fail) |
| .75 | 100.0 | | |
| .50 | 99.2 | | |
| .375 | 98.1 | | |
| #4 | 95.5 | | |
| #8 | 92.0 | | |
| #10 | 91.0 | | |
| #16 | 87.8 | | |
| #30 | 83.6 | | |
| #40 | 81.0 | | |
| #50 | 75.1 | | |
| #100 | 63.6 | | |
| #200 | 57.9 | | |

* (no specification provided)

## Material Description

CL Sandy Lean Clay

### Atterberg Limits (ASTM D 4318)

**PL=** 18 **LL=** 32 **PI=** 15

### Classification

**USCS (D 2487)=** CL **AASHTO (M 145)=** A-6(5)

### Coefficients

| | | | | | |
|---|---|---|---|---|---|
| $D_{90}$= | 1.6984 | $D_{85}$= | 0.7636 | $D_{60}$= | 0.1033 |
| $D_{50}$= | | $D_{30}$= | | $D_{15}$= | |
| $D_{10}$= | | $C_u$= | | $C_c$= | |

### Remarks

**Date Received:** 10/12/18 **Date Tested:** 11/6/18

**Tested By:** R. Barrett

**Checked By:** J. Butler

**Title:** P.E.

**Location:** TB-2

**Sample Number:** C10195-E **Depth:** 0'-5'

**Date Sampled:** 10/12/18

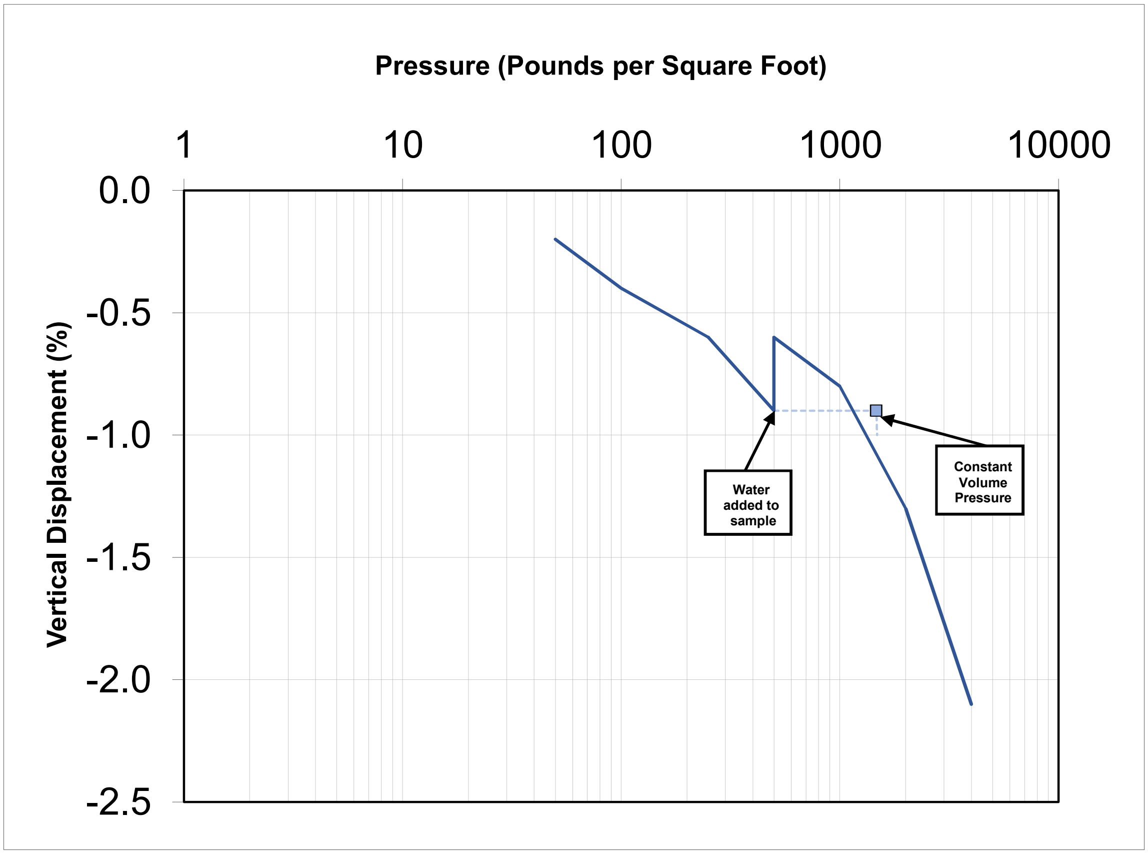
TRAUTNER GEOTECH LLC

**Client:** Montezuma Valley Irrigation, Mr. Brandon Johnson

**Project:** Groundhog Reservoir Intake Bulkhead Structure, Dolores, CO

**Project No:** 55415GE **Figure** 4.1

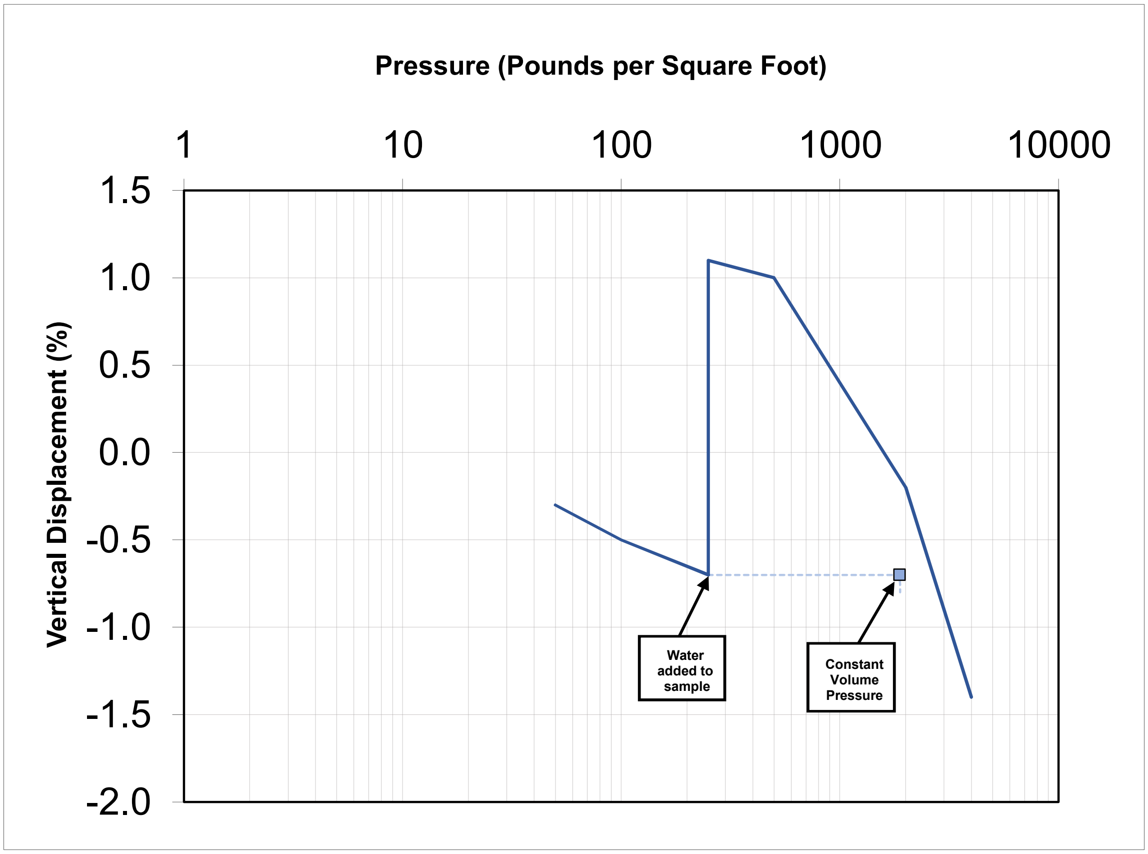
## SWELL - CONSOLIDATION TEST



| SUMMARY OF TEST RESULTS | | |
|---|---|---|
| **Sample Source:** | TB-1@2.5' | |
| **Visual Soil Description:** | Shale | |
| **Constant Voume Swell Pressure (lb/ft$^2$):** | 1,470 | |
| | **Initial** | **Final** |
| **Moisture Content (%):** | 11.6 | 13.3 |
| **Dry Density (lb/ft$^3$):** | 126.2 | 128.1 |
| **Height (in.):** | 1.000 | 0.979 |
| **Diameter (in.):** | 1.94 | 1.94 |

| | |
|---|---|
| **Project Number:** | 55419GE |
| **Sample ID:** | C10195-B |
| **Test Date:** | October 23, 2018 |
| **Figure:** | 4.2 |

## SWELL - CONSOLIDATION TEST



| SUMMARY OF TEST RESULTS | | |
|---|---|---|
| **Sample Source:** | TB-1@3' | |
| **Visual Soil Description:** | Shale/Sandstone | |
| **Constant Voume Swell Pressure (lb/ft$^2$):** | 1,880 | |
| | **Initial** | **Final** |
| **Moisture Content (%):** | 7.1 | 12.3 |
| **Dry Density (lb/ft$^3$):** | 134.8 | 132.0 |
| **Height (in.):** | 1.000 | 0.986 |
| **Diameter (in.):** | 1.94 | 1.94 |

| | |
|---|---|
| **Project Number:** | 55419GE |
| **Sample ID:** | C10195-B |
| **Test Date:** | October 23, 2018 |
| **Figure:** | 4.3 |

# Direct Shear Test Results

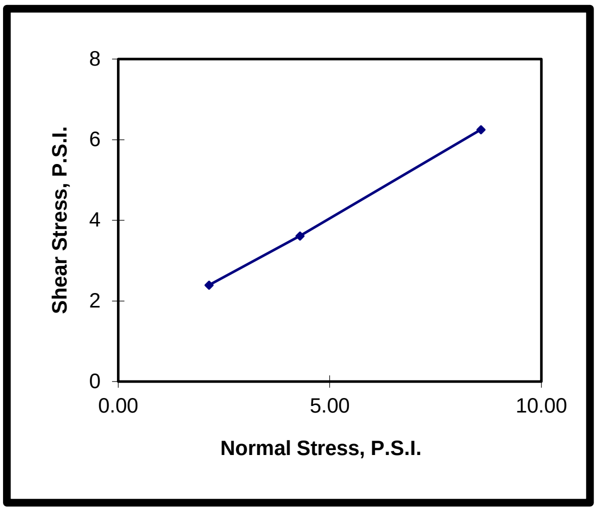
## ASTM D3080-90

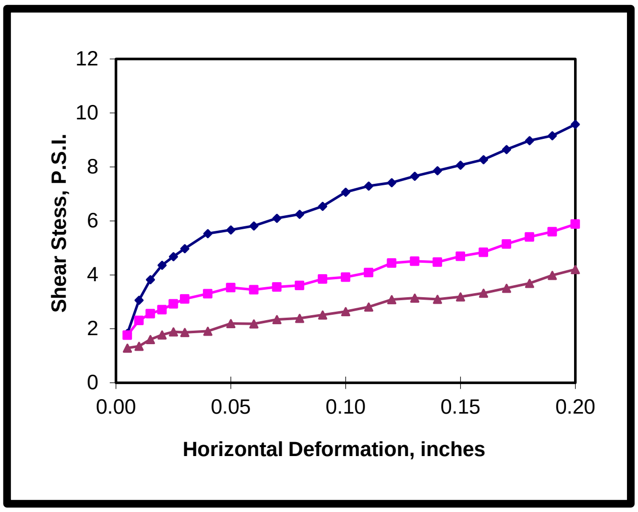
**Project:** Groundhog Reservoir Intake Bulkhead
**Project Number:** 55415GE
**Laboratory Number:** C10195-E
**Date:** 10/11/2018
**Project Technician:** RB
**Figure: 4.4**

**Visual Soil Description:** CL Sandy Lean Clay
**Type of Specimen:** Remolded
Diameter 1.946 in.
Thickness 2.0 in
**Sample Source:** TB-2@0'-5'

| Summary of Sample Data: | |
|---|---|
| Initial Moisture Content (%) | 14.4 |
| Intial Dry Density (P.C.F) | 113.2 |
| Final Moisture Content (%) | 17.0 |
| Final Dry Density (P.C.F) | 112.6 |

| Residual Direct Shear Test Results: | | | |
|---|---|---|---|
| Normal Stress (P.S.I) | 2.14 | 4.29 | 8.57 |
| Max. Shear Stress (P.S.I) | 2.39 | 3.61 | 6.25 |





| ESTIMATED STRENGTH PARAMETERS | |
|---|---|
| Angle of Internal Friction, phi | 31 |
| Cohesion, P.S.F. | 160 |