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Central Utah Water Conservancy District

Uinta Basin Replacement Project
Big Sand Wash Reservoir Enlargement

Geotechnical Baseline Report for Outlet Works

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Appendix

Earth Mechanics Institute – Colorado School of Mines Laboratory Test Results and Tunneling Report
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1. Introduction

The Big Sand Wash Reservoir Enlargement project forms a major part of the Lake Fork Section 203(a) Uinta Basin Replacement Project. In addition to the reservoir enlargement, the project also includes (1) a new diversion structure on the Lake Fork River to divert water from the Lake Fork River, (2) a new feeder pipeline to deliver water from the Lake Fork River to the Big Sand Wash Reservoir, and (3) a new pipeline to deliver water to Roosevelt and irrigation water to the State Road area near Roosevelt.

The contract documents for the Big Sand Wash Dam Enlargement Project are presented in five volumes:

- Volume 1 contains the general conditions, the Division 1 specifications (special conditions), and instructions to the bidders, including the notice inviting bids, instructions for preparing the bid, etc.
- Volume 2 contains Divisions 2 through 16 of the specifications.
- Volume 3 contains the design drawings.
- Volume 4, the Geotechnical Data Report, contains information on the design and construction of the existing dam and information collected during the CH2M HILL investigations for the new dam.
- Volume 5 (this volume), the Geotechnical Baseline Report for Outlet Works contains a summary of the geologic and geotechnical information, a description of the anticipated ground conditions, and a prediction of the ground behavior during construction of the outlet conduit and vertical shaft.

In addition to these five volumes, three additional design reports were prepared. These design reports are being made available to the Contractor, but are not part of the contract documents and are for the Contractor’s information only. These reports are:

- The Geology Report, which describes the geologic reconnaissance and mapping conducted at the site, provides engineering geologic evaluation of the site conditions, and presents a site geologic map, geologic cross sections, and seismotectonic information, including a regional fault map.
- The Geotechnical Design Report, which includes (1) interpretations of geotechnical data, (2) the results of geotechnical evaluations of the embankment dams, spillway, and outlet works, and (3) recommendations for design.
- The Hydrology and Hydraulic Structures Report, which describes the hydrologic and hydraulic engineering analyses and results, and the hydraulic and structural design of the spillway and the outlet works.
1.1 Project Description

The proposed construction of the Outlet Works for the Big Sand Wash Reservoir Enlargement will involve the following construction activities:

- Development of temporary work areas for shaft and tunnel construction
- Sinking of a construction access shaft approximately 110 feet deep
- Construction of a downstream portal
- Construction of approximately 1,100 lineal feet of tunnel with mortar lined steel pipe of 72 inches minimum inside diameter
- Construction of approximately 110 lineal feet of trenched excavation in the existing reservoir floor
- Construction of an intake structure
- Restoration of the temporary work areas

Figure 1, a plan view of the Outlet Works, shows the outlet conduit, the construction access shaft, the upstream and downstream portals, the intake structure, and the open cut construction connecting the intake structure and the outlet conduits. Figures 2 and 3 are profile views of the upstream and downstream outlet conduits, respectively.

1.2 Purpose of the Geotechnical Baseline Report

This Geotechnical Baseline Report is intended to be the basis for (1) geotechnical assessment of the outlet works by bidders and (2) evaluation of actual geotechnical conditions and determination of conditions that qualify as additional work under the provisions of this contract, as explained in Volume 1: Specifications, General Conditions – Part 5, Article 5-2, “Changes in the Work”; Division 1 – Special Conditions; and Section 01989, “Differing Site Conditions.”
2. Geologic Setting

2.1 Regional Geology

This project is located within a regional geologic feature known as the Uinta Basin in north-central Utah. The site geology is a combination of gravelly glacial outwash and silty-sand to sandy-silt eolian and alluvial deposits overlying near horizontally bedded sedimentary rocks incised by rivers that drain from the Uinta Mountains to the north. The bedrock consists primarily of the Tertiary Age Brennan Basin Member of the Duchesne River Formation.

2.2 Geologic Units

2.2.1 Gravelly Glacial Outwash Deposits

The remnants of the glacial outwash are found primarily on isolated flat-topped buttes and mesas in the area. These deposits consist of rounded quartzite gravels within a fine- to coarse-grained sandy matrix, and containing cobbles and occasional boulders with long dimensions of 2 to 3 feet. These deposits are typically 1 to 3 feet and rarely more than 10 feet thick.

2.2.2 Eolian and Alluvial Deposits

Eolian and alluvial deposits are found in the valley bottoms along Big Sand Wash and its tributaries. The eolian deposits consist primarily of very fine silty sand deposited by wind. The alluvial deposits are composed of both reworked eolian silty sand and clasts of eroded sandstone, siltstone, claystone, and occasional quartzite gravels. These deposits are locally more than 15 feet thick at the upstream end of the reservoir and in the valley below the dam.

2.2.3 Brennan Basin Member of the Duchesne River Formation

This bedrock unit underlies the area around Big Sand Wash Reservoir and underlies all of the previously described unconsolidated geologic units. Bryant (1992) describes the Brennan Basin Member as Tertiary Age, moderate red, grayish-red, reddish-brown, yellowish-brown, and yellowish-orange sandstone and less abundant siltstone and claystone.

Outcrops along Big Sand Wash are reddish to pinkish-purple, fine-grained, slightly to moderately weathered quartzitic sandstone with occasional layers of medium-grained sandstone. Interbedded in the sandstone are layers of fine-grained sandy siltstone, siltstone, silty claystone, and claystone. The siltstone is purple with gray and yellow mottling, massive, fresh to slightly weathered, and soft. The claystone is purple-red with mottled gray and yellow, fresh to slightly weathered, and soft.
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3. Geotechnical Review and Exploration

3.1 Construction Records

Black and white photographs (included in the Geotechnical Data Report) of the dam site during the early stages of construction confirm oral reports of groundwater discharges from the lower part of the right abutment that existed during construction. The fact that these discharges persisted during construction suggests a large, permanent source; the elevation of the discharges suggests that the water source is west of the natural west ridge, and could be partly a result of irrigation of nearby fields.

Construction records from 1965 include the location and take of grout holes along the base of the main dam key trench (included in the Geotechnical Data Report). For the right abutment, the records show grout takes of 29 grout holes drilled into the abutment at a 30-degree angle from the vertical toward the west. The grout holes are 20, 30, 40, 50, 60, and 80 feet deep. The grout takes varied from 0 to 376 sacks per hole and from 0 to 12.5 sacks per foot of grout hole. Grout holes with large grout takes were adjacent to grout holes with small takes and the largest grout takes were near the bottom of the abutment slope. Recent borings indicate that the sandstone in this abutment does not have a high porosity and is somewhat more silty and less porous near the bottom of the abutment. The overall pattern of grout take indicates that most of the grout flowed into open joints.

3.2 Geotechnical Exploration and Testing

Geologic and geotechnical field investigations were conducted at the Big Sand Wash Dam and Reservoir site between February and May 2002, and between November 2002 and January 2003. The investigations consisted of performing geologic mapping, drilling 35 borings, installing piezometers in some of the borings, conducting hydraulic pressure (packer) tests in the borings, and excavating test pits. Soil and rock samples were taken for laboratory testing. The Geotechnical Data Report presents the results of these investigations.

In June 2003, three additional borings were completed along the proposed alignment of the outlet conduit. Core samples from these borings were sent to the Earth Mechanics Institute at the Colorado School of Mines for testing to evaluate conditions for machine tunneling. The Geotechnical Data Report includes the logs of the additional borings, subsequent piezometer measurements, and the laboratory test results.

This report addresses the ground conditions at the location of the proposed tunnel for the outlet conduit through the right abutment of the dam. The tunnel and shaft will be almost entirely in the bedrock. Borings D-10, D-11, D-40, D-41, and D-42 were within the immediate vicinity of the outlet conduit. The information from these and other borings at the Big Sand Wash site, as well as observations during geologic mapping, comprise the basis of the geotechnical documentation for outlet conduit construction.
In July 2003, David Evans and Associates (DEA) performed a bathymetric survey and sub-bottom profiling in the reservoir in the area of the existing and proposed intakes. The bathymetric map has a 1-foot contour interval. The sub-bottom map of the base of soil deposits and the top of the underlying bedrock also has a 1-foot contour interval. In the relatively flat area upstream of the dam, the soil cover is apparently 1 to 4 feet thick. These soils are likely to be loose silts. On the lower slopes of the right abutment, the soil cover is 1 to 10 feet thick. These soils appear to be mainly talus deposits containing materials from sand to boulder size. The Geotechnical Data Report presents the results of the bathymetric survey.

### 3.3 Geologic Structure

Published geologic maps indicate that the site is an area of relatively flat-lying sedimentary rocks with no geologic faults. Geologic mapping confirms that the rock is nearly horizontal and free of geologic faults. The bedrock structural features appear to be limited to joints and bedding planes in the sandstone. Bedding planes appear to have a local dip of approximately 5 degrees toward the northeast. Bedding planes in outcrops are commonly traceable for hundreds of feet. Most of the bedding planes are unfractured. Bedding planes at the contact between sandstone and underlying claystone commonly undulate with a wave length of tens of feet and a wave height of several feet. Along these bedding planes, layers of claystone are commonly truncated by sandstone units.

Joints are prominent in nearly all sandstone outcrops. Most of the observed joints are approximately vertical, widely spaced, and open. Most joints have either a northwest strike or a northeast strike. The northwest-trending joints appear to be responsible for the shape of the distinctive northwest-trending linear ridge that forms the right dam abutment and the oval-shaped northwest-southeast trending butte above the left abutment. The northeast trending joints appear to cross these topographic features. It appears that both joint sets are through-going bedrock features. The open condition noted in outcrops might be, in part, the result of stress relief or block movement caused by sliding on underlying claystone layers. While the width of the joint openings appears to typically decrease with increasing distance behind the cliff face, complete closure was not observed. These and other observations indicate that open joints are present at depth in this bedrock.

### 3.4 Springs

Springs and areas of seepage are present in the valley walls downstream from the Big Sand Wash Dam. The springs originate from the valley wall between elevations of approximately 5,830 and 5,840 feet. The discharge locations appear to be at the contact between an overlying sandstone and an underlying claystone layer. The mechanism for the seepage through the rock mass is less clear. It is likely that the west ridge behaves as a small aquifer storing rainwater and snowmelt, and that the seepage is a release of this groundwater. It is also possible that the seepage is caused by a regional aquifer feeding the lower Big Sand Wash Valley, given that the ground surface elevations outside the valley are approximately 5,850 feet, 10 to 20 feet higher than the noted seepage elevations within the valley.
3.5 Borings

The same general rock conditions were encountered in the 38 borings drilled at the main dam site, the reservoir behind the dam, the natural ridge, the rock in the West Saddle Dam, and along the East Saddle Dam, along with the 3 borings completed for the outlet conduit. Sandstone was the dominant rock type, with thin to thick interbeds of siltstone and mainly thin interbeds of claystone. In boreholes deeper than 50 feet, sandstone was 50 to 90 percent of the rock. In some borings, sandstone was present in layers commonly 10 feet thick, with siltstone and claystone occurring as widely spaced thin interbeds. In other borings, sandstone, siltstone, and claystone alternated as thin interbeds.

Core recovery (i.e., the percentage of core obtained from a cored interval) was commonly 60 to 90 percent, but rarely close to 100 percent, particularly within sandstone layers. In hard, strong rocks, core recovery values were commonly more than 90 percent, with values less than 90 percent typically indicating poorer rock conditions. There were a number of zones in these borings of zero core recovery in vertical intervals of 1 to 2 feet. In these intervals, the in-situ rock was entirely lost by erosion during coring.

The rock quality designation (RQD) is the percentage of a cored interval represented by core cylinders with an axial length of 4 inches or greater. By definition, the RQD values cannot exceed the core recovery. In these borings, the RQD values were generally between 25 and 75 percent, although RQD values less than 25 percent were common. RQD values above 75 percent were not common.

The RQD system was developed for and is commonly used in relatively strong rock with fractures. Based on empirical tunnel observations, an RQD greater than 75 percent indicates good tunneling conditions; an RQD for between 50 and 75 percent indicates fair tunneling conditions; and an RQD less than 50 percent indicates poor tunneling conditions.

The RQD values in this case are not directly related to RQD values and tunneling conditions in strong, fractured rock. The relatively low strength sandstone, siltstone, and claystone in this bedrock are greatly affected by erosion from drilling fluids and mechanical breakage during the coring operation. The low RQD values in the logs reflect extensive damage to the core samples. The in situ rock conditions are more continuous and more massive than the RQD and core recovery values suggest. The low RQD and core recovery values do indicate that the rock mass is susceptible to erosion by moving water and portions of dry bedrock might cave if not fully supported.

The core logs contain estimates of rock core hardness based upon the response of the core to tests of its resistance to scratching or breakage. The rock hardness scale used was (ISRM 1981):

- R0 - Indented by a fingernail; very soft
- R1 - Peeled by a pocketknife and crumbles under a hammer; very soft
- R2 - Peeled with difficulty by a pocketknife; soft
- R3 - Fractured by a hammer; medium hard
- R4 - Fractured by repeated hammer blows; hard
- R5 - Fractured by repeated geologic pick blows; very hard
- R6 - Chipped by a geologic pick; very hard
Zones of R0 to R5 rock were encountered at the site; however, the most common hardness ratings were R0 to R3, very soft to medium hard. Rock with hardness ratings of R0 and R1 was also generally described as having a weak or very weak strength classification. Sandstone that was described as weak to very weak was also often described as friable with weak cementation (i.e., they crumble under finger pressure).

### 3.6 Groundwater Monitoring

Piezometers were installed in most of the borings to monitor groundwater conditions. The piezometers located in the vicinity of the outlet works include D-7, D-10, D-12, D-13, D-25b, D-40, D-41, and D-42 (see Figure 1). Figure 4 summarizes the measured levels in these piezometers. The piezometric levels represent groundwater pressures in the ground at the elevation of the piezometer screen, and might not be representative of the phreatic surface.

### 3.7 Water Pressure Testing

Water pressure testing was conducted within the boreholes during the exploration by forcing water under pressure into an uncased section of the borehole. An inflatable packer was used to seal the borehole at the top of the test zone. This arrangement is referred to as a single packer test. Test zones ranged from 10 to 15 feet long. Each test consisted of measuring flows at three water pressures in a five step sequence: (1) low pressure, (2) medium pressure, (3) high pressure, (4) medium pressure, and (5) low pressure.

A total of 182 zones were subjected to the water pressure tests, which were performed in 37 boreholes across the project. Most of the tests were conducted in vertical boreholes. However, 4 boreholes were advanced 30 degrees to the vertical. A total of 40 packer tests were performed in the 5 borings adjacent to the outlet conduit corridor. The results of the water pressure tests were used to estimate the hydraulic conductivity of each test zone. Table 1 summarizes the calculated hydraulic conductivity values.

<table>
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<th>Hydraulic Conductivity (cm/sec)</th>
<th>All Site Tests</th>
<th>Tunnel Site Tests</th>
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<tr>
<td></td>
<td>Number</td>
<td>Percent</td>
</tr>
<tr>
<td>&lt; 1x10^-6</td>
<td>21</td>
<td>12</td>
</tr>
<tr>
<td>1x10^-6 to 1x10^-5</td>
<td>54</td>
<td>30</td>
</tr>
<tr>
<td>1x10^-5 to 1x10^-4</td>
<td>72</td>
<td>39</td>
</tr>
<tr>
<td>1x10^-4 to 1x10^-3</td>
<td>23</td>
<td>13</td>
</tr>
<tr>
<td>1x10^-3 to 1x10^-2</td>
<td>7</td>
<td>4</td>
</tr>
<tr>
<td>1x10^-2 to 1x10^-1</td>
<td>3</td>
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</tbody>
</table>

The hydraulic conductivity values varied by six orders of magnitude, with the highest values more than 100,000 times the lowest values. Hydraulic conductivity values of less
than 1x10^{-4} \text{ cm/sec} are typical of silt and clay soils and indicate poor drainage. Hydraulic conductivity values greater than 1x10^{-4} \text{ cm/sec} typically indicate good drainage, and are representative of soils such as sand and gravel. In general, the fine-grain sandstone, siltstone, and claystone at the site had hydraulic conductivity values less than 1x10^{-4} \text{ cm/sec}. Sandstone at the site with hydraulic conductivity values from 1x10^{-4} \text{ to } 1x10^{-3} \text{ cm/sec} probably represents porous sandstone with limited jointing. Sandstone at the site with hydraulic conductivity values greater than 1x10^{-3} \text{ cm/sec} probably represents rock with extensive open jointing.

Nineteen percent of the 180 packer tests across the site had hydraulic conductivity values of more than 1x10^{-4} \text{ cm/sec}; six percent had hydraulic conductivity values of more than 1x10^{-3} \text{ cm/sec}. The relatively small percentage of tests with high hydraulic conductivity values suggests that open joints were not commonly encountered. The high hydraulic conductivity values indicate that the widely spaced open joints are continuous. These results are consistent with the observation in outcrops of tight bedding planes, widely spaced near vertical joints.

In the vicinity of the outlet works, ten percent of the tests showed hydraulic conductivity values greater than 1x10^{-4} \text{ cm/sec}; no tests indicated hydraulic conductivities greater than 1x10^{-3} \text{ cm/sec}. We conclude that the absence of values greater than 1x10^{-3} \text{ cm/sec} along the tunnel alignment does not indicate an absence of open joints in this part of the site, but rather it is a function of the small number of tests conducted.

### 3.8 Laboratory Testing

Unconfined compressive strength tests were performed on eight core samples from borings D-12 and D-15 on the right abutment ridge north of the tunnel site. The samples included four claystone, two siltstone, and two sandstone cores. The dry density of the samples ranged from 138.6 to 155.7 pcf and averaged 151 pcf. The strength ranged from 1,670 psi (sandstone) to 5,820 psi (claystone) and averaged 3,549 psi. A comparison of these quantitative results and qualitative observations in the borehole logs suggests that these unconfined compressive strength tests are representative of the stronger rocks.

Four core samples, 6 to 10 inches long, were selected for intact rock testing at the Colorado School of Mines (Earth Mechanics Institute 2003). The samples were taken from borings D-40, D-41, and D-42. These samples were subjected to uniaxial compressive strength, Brazilian tensile strength, Cerchar abrasivity, and punch penetration tests. The report and results of these tests are included in the appendix and in the Geotechnical Data Report.

The report by the Earth Mechanics Institute characterized these rocks as follows: uniaxial compressive strength of 6,706 psi; Brazilian tensile strength of 370 psi; Cerchar abrasivity index of 1.0; punch penetration peak slope index of 21.0. The report also provides an estimate of instantaneous advance rates for a microtunnel machine under assumed operating conditions.

The samples submitted to the Earth Mechanics Institute were, for practical reasons, biased toward core samples that appeared to be suitable for transportation to the laboratory, sample preparation, and testing. A significant part of the recovered core was judged unsuitable for testing because the rock samples were too small or too weak.
3.9 Borehole Summary

Figure 5 summarizes the five borings along the approximate alignment of the outlet conduit. The drawing shows the borings in their relative position from upstream on the tunnel alignment on the left to downstream on the tunnel alignment on the right. The location of each boring is shown by station and is offset in the heading. The borings are correctly shown in the vertical scale but the drawing is not horizontally scaled in order to more completely show the core characteristics. The left bar scale for each boring is a graphic representation of rock hardness. Lithology is presented in the middle of each log. The right bar scale is a graphical representation of RQD. The tunnel zone, elevation 5,790 feet to elevation 5,800 feet is highlighted, as is boring D-41 at the Construction Access Shaft site.

The Construction Access Shaft penetrates a sequence of mainly sandstone with interbeds of siltstone and sandstone. These rocks are mainly very soft to soft, and an RQD that is mainly very poor to poor. The piezometric level on August 25, 2003, was elevation 5,817 feet, approximately 30 feet above the tunnel invert.

The outlet conduit is located mainly in sandstone with some claystone. Rock hardness was noted as soft to hard in D-42, very soft to soft in D-10, soft to very soft in D-41, very soft to soft in D-11, and very soft to hard in D-40. RQD was noted as good to fair at D-42, very poor to poor at D-10, D-42, and D-11, and good to poor at D-40.

The piezometers installed in borings D-40, D-41, and D-42 have been read for a relatively short time period (July 17 to August 25, 2003). The piezometer installed in boring D-10 has been read for over a year (May 20, 2002 to August 25, 2003). Piezometers D-10 and D-42 indicate a piezometric level downstream of the access shaft of 0 to 15 feet above the conduit. Piezometers D-41 and D-40 indicate that the piezometric level varies from approximately 17 to 47 feet above the conduit upstream of the access shaft. However, these data represent monitoring over a fairly short time period. In addition, given that the piezometers were screened at elevations below the valley bottom, it is anticipated that the phreatic surfaces will be higher than the measured piezometric levels, as the claystone/silstone layers behave as aquitards for the local groundwater flow behavior.
4. Baseline Conditions

The remaining sections of this Geotechnical Baseline Report for the Big Sand Wash Reservoir Enlargement project contain the project baseline statements. Baseline conditions are provided for shaft construction, portal construction and tunnel construction.

4.1 Shaft Construction

Boring D-41 indicates that the Construction Access Shaft will be excavated mainly in a low strength, poor quality sandstone rock in the presence of a fairly high hydrostatic pressure. This shaft will be excavated in the spring of 2004 when the reservoir is fairly high, to provide irrigation water. The groundwater level at the time of construction is likely to be higher at borings D-40, D-41 and D-42 than that shown in Figures 4 and 5. Note the limited response of the groundwater level in D-41 at the construction access shaft site to the recent lowering of the reservoir.

The sandstone, siltstone, and claystone are expected to behave as low-strength rocks in the shaft excavation, provided (1) the excavated surfaces are not allowed to move toward the opening as a result of the change in stress and (2) the ground is not exposed to deterioration caused by drying, piping by seepage, or erosion by seepage. Ground deterioration by fracturing or drying can be prevented by application of pattern rock anchors and shotcrete. Control of seepage into the shaft excavation is expected to be the greatest challenge. Seepage from the unjointed sandstone, siltstone, and claystone is expected to be small and, for the most part, benign. Seepage from open joints could be large and could lead to rapid deterioration of the surrounding ground. Control of groundwater flows from open joints is expected to be difficult and uncertain. Dewatering wells outside the shaft and in the bottom of the shaft will probably yield small quantities of water and localized dewatering. Most wells will completely miss vertical open joints and the opportunity to dewater these joints before they are exposed in the excavation. Grouting to fill open joints before they are exposed in the shaft excavation could also yield uncertain results. Once a water-bearing joint is exposed, dewatering and grouting might not be effective tools for managing water inflows and damage to the ground.

It is not possible to quantify the potential impact of water-bearing open joints on shaft construction. The Contractor will have the responsibility for responding to this issue by applying one or more of the following strategies:

- Cut off potential open joints by installing a cut-off system sufficiently below the bottom of the descending shaft excavation to stop inflow from the sidewalls and into the bottom of the shaft.
- Freeze the ground below the descending shaft excavation to stop inflows from the sidewalls and shaft bottom.
- Pattern grout the ground below the descending shaft excavation to stop inflows from the sidewalls and shaft bottom.
• Use dewatering wells outside and perhaps inside the descending shaft to dewater the
  ground and use wells, grouting, or other measures to deal with groundwater inflows
  and ground damage uncovered as the work progresses.

4.2 Portal Construction

The upstream and downstream portal excavations will be made in rock that has not been
sampled in this exploration program. Boring D-42 is the best available indication of the rock
conditions in the downstream portal and boring D-40 is the best available indication of the
rock conditions in the upstream portal.

Geophysical soundings in the reservoir show that the upstream portal site is overlain by
about 10 feet of unconsolidated soil. This soil consists of colluvium ranging from sand to
boulders, overlain by a thin layer of alluvial silt.

It is anticipated that the upstream portal will be excavated in a dewatered area of the
reservoir in September 2004 just prior to a tunnel hole-through. The overburden will be
excavated by earthmoving methods and these saturated soils will assume a relatively flat
angle of repose and daylight high up the slope. Sub-bottom profiling by DEA shows a soil
cover on the relatively flat bottom of the reservoir and on the lower part of the right
abutment slope. Most of this soil will be soft and all of the soil will be wet after the reservoir
is drained. The materials probably will not be capable of supporting heavy equipment,
unless stabilization methods are undertaken. The DEA bathymetric map shows the presence
of depressions in the reservoir bottom that might not drain by gravity. These soil conditions
will be an important factor in the construction and maintenance of facilities to keep the
work area dewatered and in the construction of the portal, pipeline, and intake.

The Contractor will plan and construct an upstream portal for the hole-through of the
tunnel and construction of the pipeline connection to the intake structure. The DEA
bathymetric survey and the sub-bottom survey provide the best available information about
the topography and top of rock in the construction. These indirect measurements should be
considered an approximate representation of the site conditions. The Contractor will
determine the size, shape, and location of the portal excavation and the use of rock
reinforcement and/or ground support as needed to meet its construction requirements. The
upstream portal will be backfilled at the completion of the work.

The downstream portal will be excavated in an existing bedrock exposure. It is expected
that the rock can be excavated to a near vertical slope, but rock bolts and shotcrete will be
required to develop a permanent open excavation above the exposed pipe.

4.3 Tunnel Construction

The tunnel will penetrate rock that ranges in hardness from very soft to hard, has RQD
characterizations of very poor to good, and groundwater conditions ranging from dry to
groundwater heads of 80 feet or more. A significant proportion of the rock is expected to be
low strength. A suitably designed slurry microtunnel boring machine (MTBM) can be
expected to handle this range of conditions. Variations in rock hardness within a single face
of the tunnel bore and at different locations along the tunnel bore will have the potential to
affect line and grade and the effectiveness of rock cutters in the cutterhead and thus future production rates in a single drive.

MTBM excavation has been required on this project mainly for the excavation of the upstream tunnel from the Construction Access Shaft to the bottom of the reservoir because of the presence of weak rock, high groundwater pressures, and the potential for rock deterioration if groundwater inflows enter an unlined tunnel. Groundwater inflows will not occur if the groundwater pressures are balanced in the slurry-filled chamber at the front of the MTBM. Groundwater inflows into the tunnel will be prevented by the advancing string of pipe jacked behind the MTBM. The geotechnical information, and an expectation that the upstream tunnel will be driven in the summer of 2004 while water levels are high in the Big Sand Wash Reservoir, indicates that there would be high risk in performing this tunnel excavation without the benefit of full-face groundwater stabilization.

The decision to lower the Big Sand Wash Reservoir by September 2004 to allow tunnel hole-through into a dewatered part of the reservoir potentially eliminates the need for an underwater excavation of a receiving pit for the tunnel drive and underwater removal of the MTBM. The Contractor will have the option of making this upstream tunnel drive so that hole-through coincides with the completion of dewatering, or making a partial drive and pausing until a completion can be made in the dry. The geotechnical information indicates that the upstream tunnel drive will encounter claystone and weak sandstone. Squeeze of the claystone and collapse of the weak sandstone during a work stoppage could make a second drive difficult or impossible.

The borings indicate that the downstream tunnel, like the upstream tunnel, will encounter a range of rock conditions. Groundwater conditions, however, will be more favorable in the downstream tunnel. As noted in the case of the Construction Access Shaft, it is not possible to quantify the potential impact of water-bearing open joints on the downstream tunnel construction. The Contractor will have the responsibility for responding to this issue by applying one or more of the following strategies:

- Construct the downstream tunnel using an MTBM from the downstream portal.
- Construct the downstream tunnel using an MTBM from the Construction Access Shaft.
- Construct the downstream tunnel from the downstream portal in two-pass construction using an open-face excavation system, steel rib and lagging ground supports, groundwater control ahead of the tunnel face, and a final lining of steel pipe and a grouted annulus between the outside of the pipe and the tunnel supports. Groundwater control ahead of the face may involve (1) pre-drainage by wells from the ground surface, (2) pre-drainage in horizontal wells from the tunnel, (3) grouting ahead of the tunnel face, or (4) other water control method.

It is our opinion that this third method involves greater risk of delay and ground disturbance than the MTBM alternative.
4.4 General

The contractor shall have the capability to treat up to 500,000 gallons per day of water pumped or drained from the shaft and tunnel system.
5. References


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NOTES:
1. FOR ANTICIPATED RANGE IN RESERVOIR
   WATER LEVELS REFER TO SECTION 4.
2. THE RESERVOIR WILL BE KEPT IN SERVICE DURING
   CONSTRUCTION. HOWEVER THE DIVERTED FLOW
   INTO THE RESERVOIR IS UNDER THE CONTROL OF
   DISTRICT AND WILL BE STOPPED BETWEEN
   SEPTEMBER 15 AND DECEMBER 15. THE
   RESERVOIR WILL BE DRAINED AS LOW AS
   POSSIBLE THROUGH THE EXISTING OUTLET WORKS.
3. CONTRACTOR IS RESPONSIBLE FOR PROTECTING
   HIS WORK AND THE EXISTING FACILITY FROM
   FLOODS DURING CONSTRUCTION.

OUTLET CONDUIT PROFILE (UPSTREAM OF SHAFT)

FIGURE 2
OUTLET CONDUIT PROFILE
UPSTREAM OF CONSTRUCTION SHAFT
GEOTECHNICAL BASELINE REPORT
UNITA BASIN REPLACEMENT PROJECT
BIG SAND WASH RESERVOIR ENLARGEMENT
NOTES:
1. FOR ANTICIPATED RANGE IN RESERVOIR WATER LEVELS REFER TO SECTION 4.
2. THE RESERVOIR WILL BE KEPT IN SERVICE DURING CONSTRUCTION. HOWEVER THE DIVERTED FLOW INTO THE RESERVOIR IS UNDER THE CONTROL OF DISTRICT AND WILL BE STopped BETWEEN SEPTEMBER 15 AND DECEMBER 15. THE RESERVOIR WILL BE DRAINED AS LOW AS POSSIBLE THROUGH THE EXISTING OUTLET WORKS.
3. CONTRACTOR IS RESPONSIBLE FOR PROTECTING HIS WORK AND THE EXISTING FACILITY FROM FLOODS DURING CONSTRUCTION.

OUTLET CONDUIT PROFILE (DOWNSTREAM OF SHAFT)

FIGURE 3
OUTLET CONDUIT PROFILE
DOWNSTREAM OF CONSTRUCTION SHAFT
GEOTECHNICAL BASELINE REPORT
UNITA BASIN REPLACEMENT PROJECT
BIG SAND WASH RESERVOIR ENLARGEMENT
FIGURE 4
MEASURED PIEZOMETRIC LEVELS IN THE BORINGS ADJACENT TO THE OUTLET WORKS
GEOTECHNICAL BASELINE REPORT
UINTA BASIN REPLACEMENT PROJECT
BIG SAND WASH RESERVOIR ENLARGEMENT

NOTE: THE MEASURED VALUES ARE PIEZOMETRIC LEVELS, NOT PHREATIC SURFACES
### Figure 5

**Summary of Exploratory Borings**

#### Geotechnical Baseline Report

**Uinta Basin Replacement Project**

**Big Sand Wash Dam Reservoir Enlargement**

<table>
<thead>
<tr>
<th>Location</th>
<th>Rock Hardness</th>
<th>Rock Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>D-40 STA 6+65 (80 ft right)</td>
<td>End of Boring (El. 5762.8 ft)</td>
<td>Sandy Cobbles and Boulders</td>
</tr>
<tr>
<td>D-11 STA 6+44 (46 ft left)</td>
<td>Clastic deposits</td>
<td>Sandstone</td>
</tr>
<tr>
<td>D-41 STA 10+06 (66 ft left)</td>
<td>End of Boring (El. 5754.5 ft)</td>
<td>Claystone</td>
</tr>
<tr>
<td>D-10 STA 11+19 (28 ft right)</td>
<td>End of Boring (El. 5750.7 ft)</td>
<td>Claystone</td>
</tr>
<tr>
<td>D-42 STA 13+92 (28 ft right)</td>
<td>End of Boring (El. 5754.5 ft)</td>
<td>Claystone</td>
</tr>
</tbody>
</table>

#### Notes:

- The maximum and minimum piezometric levels measured between 7/17/03 and 8/25/03 are shown.
- The piezometric levels are measured at different dates as follows:
  - 7/17/03: 5846.8'
  - 8/25/03: 5841.8'
  - 4/21/03: 5815.4'
  - 9/3/02: 5805.0'
  - 7/17/03: 5810.4'
  - 8/4/03: 5841.8'
  - 5/20/02: 5815.4'

#### Rock Hardness Legend

- **Very soft (R0-R1)**
- **Soft (R2-R3)**
- **Hard (R3-R4)**
- **Very hard (R5-R6)**
- **No horizontal scale**

#### RQD Legend

- **Very poor (0-25)**
- **Poor (25-50)**
- **Fair (50-75)**
- **Good-excellent (75-100)**
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<table>
<thead>
<tr>
<th>Sample ID</th>
<th>Rock Type</th>
<th>Average Diameter</th>
<th>Length</th>
<th>Density</th>
<th>Uniaxial Compressive Strength</th>
<th>Average Diameter</th>
<th>Average Length</th>
<th>Brazilian Tensile Strength</th>
<th>Cerchar Abrasivity Index (CAI)</th>
<th>Punch Penetration Test Peak Slope</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>D-40@1092-10911</td>
<td>Sedimentary</td>
<td>2.350</td>
<td>4.842</td>
<td>157</td>
<td>2.51</td>
<td>6,706</td>
<td>46</td>
<td>Non-structural</td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>D-40@10711-1088</td>
<td>Sedimentary</td>
<td>2.334</td>
<td>1.37</td>
<td>488</td>
<td>3.4</td>
<td>0.8</td>
<td>Non-structural</td>
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</tr>
<tr>
<td>D-40@1092-10911</td>
<td>Sedimentary</td>
<td>2.355</td>
<td>1.29</td>
<td>395</td>
<td>2.7</td>
<td>1.2</td>
<td>Non-structural</td>
<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>D-41@9610-976</td>
<td>Sedimentary</td>
<td>2.360</td>
<td>1.39</td>
<td>227</td>
<td>1.6</td>
<td>1.0</td>
<td>Non-structural</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>D-42@782-791</td>
<td>Sedimentary</td>
<td>2.351</td>
<td>1.47</td>
<td>287</td>
<td>2.0</td>
<td>1.1</td>
<td>Structural</td>
<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>D-40@107-108</td>
<td>Sedimentary</td>
<td>23.1</td>
<td></td>
<td></td>
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<td></td>
<td></td>
<td></td>
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</tr>
<tr>
<td>D-40@1092-10911</td>
<td>Sedimentary</td>
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</tr>
<tr>
<td>D-41@9610-976</td>
<td>Sedimentary</td>
<td>16.2</td>
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</tr>
<tr>
<td>D-42@782-791</td>
<td>Sedimentary</td>
<td>32.1</td>
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<td></td>
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</tbody>
</table>
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Preliminary Tunneling Penetration Estimates for Big Sand Wash Dam

Preliminary penetration estimates have been made in support of the tunnels to be constructed on the Big Sand Wash Dam Project. These prediction estimates are for instantaneous production of an assumed pipe jacking machine operating in sedimentary rock. These instantaneous production rate estimates do not take into account any machine availability or utilization, therefore are only for when the machine is fully engaged in cutting.

Two sets of data input were used in these predictions. They were rock properties and machine specifications. The intact rock physical property data was generated by the Colorado School of Mines on core samples provided by CH2M Hill. The summary of the physical property data for this study is presented in Table 1. No results from structural failures were included in the average test results used for performance prediction. The rock described here was the only material provided for rock strength testing.

<table>
<thead>
<tr>
<th>Uniaxial Compressive Strength (psi)</th>
<th>Brazilian Tensile Strength (psi)</th>
<th>Cerchar Abrasivity Index</th>
<th>Punch Penetration Peak Slope Index</th>
</tr>
</thead>
<tbody>
<tr>
<td>6,706</td>
<td>370</td>
<td>1.0</td>
<td>21.0</td>
</tr>
</tbody>
</table>

Table1: Physical Property Summary.

Uniaxial Compressive Strength (UCS) is one of the most basic parameters and the most common test performed for determination of rock strength. In general, UCS provides a measure of what is required for cutters to penetrate the rock. Indirect, or Brazilian, Tensile Strength (BTS) provides a measure of rock tensile strength. The ratio of UCS over BTS (C/T), provides a measure of rock brittleness. The C/T ratios for rock typically range between 10 and 20. A higher C/T ratio indicates a rock that is probably more brittle compared to other rocks of the same compressive strength. The C/T ratio of the rock tested for this estimation was about 18/1.

The Cerchar Abrasivity Index (CAI) provides a measure of rock abrasivity for determining cutter wear. Cutting rate estimates can then be adjusted to account for an estimated amount of cutter wear. CAI values range from zero to six, or possibly higher. Lower values indicate lower abrasivity and lower cutter wear, as long as the tested rock is competent enough to support this test.

The Punch-Penetration index provides a qualitative measure of rock toughness. The slope of the force-penetration curve has been related to the mechanical cut-ability of the rock, i.e., the energy needed for efficient chipping. A baseline slope index value of 175 is used as reference. The rock tested for this estimate is believed to be too soft or weak to be valid for this test, as indicated by the very low result.
The specifications of the machine under consideration were assumed. This is assumed to be a standard micro-tunneling machine, with pipe jacking advance, slurry cuttings removal, and operated from the surface. The assumed basic machine specifications used for this penetration estimate are presented in Table 2.

<p>| | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Cut Diameter</td>
<td>76</td>
<td>in.</td>
</tr>
<tr>
<td>Cutterhead Power</td>
<td>85</td>
<td>hp</td>
</tr>
<tr>
<td>Cutterhead Rotation Rate</td>
<td>8</td>
<td>rpm</td>
</tr>
<tr>
<td>Cutterhead Torque</td>
<td>53,012</td>
<td>ft-lb</td>
</tr>
<tr>
<td>Nominal Line Spacing</td>
<td>1.5</td>
<td>in.</td>
</tr>
<tr>
<td>Number of Cutters</td>
<td>30</td>
<td></td>
</tr>
<tr>
<td>Cutter Type</td>
<td>Conical</td>
<td></td>
</tr>
<tr>
<td>Tip Angle</td>
<td>75</td>
<td>Degrees</td>
</tr>
<tr>
<td>Tip Diameter</td>
<td>0.625</td>
<td>in.</td>
</tr>
</tbody>
</table>

Table 2: Assumed Machine Specifications.

The jacking force capability is intentionally omitted from the machine specification. This is because the thrust force available to the cutterhead, which is a function of jacking force and pipe friction, is constantly changing.

The jacking frame will probably have the capability of producing considerably more force that the conical bits can withstand. Therefore, the machine should not be operated in a manner that does not provide more thrust to the cutterhead than the assumed conical bits can withstand. The additional thrust capability is provided to overcome increasing jacking forces as the micro-tunneling machine advances. Given this excess in thrust capability, the thrust efficiency is assumed to be 100%. A mechanical efficiency of 95% was assumed for the rotational cutting power train on this machine.

Given the nature of pipejacking, available cutterhead thrust will decrease as the machine advances into the tunnel. This is due to friction between the jacking pipe and the walls of the bored tunnel. This friction can vary greatly for many different reasons, including tunnel straightness, effectiveness of the lubrication system, type of jacking pipe and total length of jacking pipe any given moment. For this reason, this study does not have the scope to estimate the thrust available to the cutterhead.

Cutterhead thrust, torque and power requirements, for the assumed machine, are presented as a function of instantaneous penetration rate in Chart 1. The power limitation is presented, for the assumed machine, in Chart 1 as a bold dashed line. This limitation was achieved with a penetration of 0.83 inches per revolution.
Once the thrust available to the cutterhead falls below 41,500 lbf, the instantaneous penetration rate is expected to decrease, as shown in Chart 2. Below this thrust level the assumed machine becomes thrust limited. Above this thrust level, power is expected to be the limiting factor for the assumed machine.

Given that the micro-tunneling machine can provide more than 41,500 lbf of thrust to the cutterhead, the assumed machine is estimated to advance into the stated rock at an instantaneous penetration rate of 0.83 in./revolution. This relates to 33.2 ft./hr. when combined with the cutterhead rotation rate of 8 rpm. This assumes that the power pack driving the thrust jacks can provide enough volume of oil to advance the thrust jacks at that velocity. The instantaneous penetration rate is expected to decrease if the thrust available to the cutterhead decreases below 41,500 lbf.
Chart 2: Instantaneous Penetration Estimates as a Function of Cutterhead Thrust.

These performance prediction estimates are for instantaneous advance rates. That is while the machine is actual cutting and does not account for any utilization, such as time taken for installing jacking pipe or mechanical breakdowns. The results of these instantaneous penetration estimates are based on the stated assumed specification of the micro-tunneling machine and the listed rock physical properties. If any of these items change, different penetration rates should be expected. It is also assumed that the muck collection on this machine is adequate and does leave cut rock between the cutterhead and face, resulting in reduced performance.
Description and Procedures for Physical Property Tests

Performed at the
Earth Mechanics Institute
Department of Mining Engineering
Colorado School of Mines

Golden, Colorado
UNIAXIAL COMPRESSION STRENGTH (UCS)

1. PURPOSE

UCS is one of the most basic parameters and the most common test performed for determination of rock strength. It is measured in accordance with the procedures recommended in ASTM D2938. The samples are prepared to satisfy the requirements of ASTM D4543.

2. EQUIPMENT

1. Rock saw
2. Brown & Sharp No. 2L Grinder (Lap-master Model 15 if needed)
3. Dial Gage with 0.0001 in precision
4. Balance Scale with up to 2 kg range and 0.1 gram accuracy.
5. Caliper with 0.001 inch accuracy.
7. Computer and Data Acquisition system (Integrated into the MTS control system).

3. PROCEDURES

3.1 Sample Preparation

1. Cut the samples to a length of 2.0 to 2.2 times the diameter. The L/D ratio of 2:1 (or up to 2.2:1) must be met where possible. No sample with L/D below 1.5 shall be tested. Strength adjustment to L/D of 2 will be made through proper formulations, as discussed later.

2. Grind specimen ends to required flatness as per ASTM-4543. Make sure that the sample is held by the V-Block to ensure the parallelism of the ends and verticality of the ends with respect to core axis. Do not remove the sample from the V-Block until both ends are ground. This is to maintain the position of the sample with respect to V-Block.
3. Check the smoothness of the ends with the dial gage. The variation must not exceed 0.001 inch. If this value is exceeded, the surfaces must be reground to meet the specified tolerance.

4. Measure all the dimensions of the prepared sample. This includes length, diameters at 3 points (all measurements to the nearest 0.001 in), and weight (to 0.1 gram).

5. Air dry samples for at least 12 hours prior to testing.

6. Log core samples to indicate the position, orientation, and condition of existing joints, fractures, bedding, foliation, inclusions or any other defects. Logging is done with respect to a reference line drawn randomly along the core.

7. Record all pertinent information and measurements of a UCS core sample on the test sheet. This should include general rock and project information, date, sample weight and dimensions.

3.2 Compression Test

1. Place and secure the sample between the platens. Platens must be clean and oil free. The spherical seat must be tight yet moving and rotating freely. Place the sample at the center of the platens assuring that a full contact with both platens is established.

2. Close the shield doors on the test machine to protect the operator from flying pieces of rock in case of violent failure.

3. Turn on the machine, the hydraulic pumps, and the computer control system and rise the lower platen till sample is held firmly by both platens with a pre-load of less than 200 pounds of force. Run the Test-Ware program and select the appropriate template for the test (UCS Soft/Hard).

4. Reset the displacement and start loading. Set the loading rate at 10 kips per minute. Open a new data file with the same name as the core ID. Machine will collect data and detect the increase in load and will retreat when senses a drop of over 500 lbs in load (This assures the termination of loading after failure and prevent unnecessary disintegration of the sample when internal failure is occurred).

5. Record the peak load, make a sketch of the failure on the test sheet, photograph the sample with the ID card (if required), remove the sample and collect all the pieces and put it in a clearly labeled zip lock bag, and clean the machine. Close and back up the data file.
4. DATA REDUCTION AND CALCULATIONS

1. Use the information on the test sheet to calculate the Uniaxial Compressive Strength (UCS), as follows:

\[
UCS = \frac{F_{\text{max}}}{A}
\]

Where:
- \( F_{\text{max}} \) = maximum load on the sample before failure
- \( A \) = cross-sectional area of the sample

INDIRECT (BRAZILIAN) TENSILE STRENGTH

1. PURPOSE

Indirect, or Brazilian, tensile strength provides a measure of rock tensile strength, as well as an indication of rock toughness/brittleness. This parameter is measured following the procedures of ASTM D3967.

2. EQUIPMENT

1. Rock saw
2. Balance Scale with up to 2 kg range and 0.1 gram accuracy.
3. Caliper with 0.001 inch accuracy.
5. Computer and Data Acquisition system (Integrated into the MTS control system).

3. PROCEDURES

3.1 Sample Preparation

1. Cut a disc from the core with an L/D ratio of approximately of 0.5 : 1.
2. Allow the specimen to air dry for at least 12 hours prior to testing.
3. Record all pertinent information and measurements on the test sheet. Measure sample dimensions and weight (to 0.1 gram). This includes length, diameter at 3 points (all measurements to the nearest 0.001 in.).
4. Log the specimen to indicate the position, orientation, and condition of existing discontinuities, such as joints, fractures and bedding/foliation. Make a sketch on the test sheet to show any existing rock discontinuities.

3.2 Testing

1. Place and secure the specimen between the platens of the test machine. Platens must be clean and oil free. The spherical seat must be tight yet able to move and rotate freely. Place cardboard pads between the specimen and the upper and lower platens.

2. Turn on the machine, the hydraulic pumps, and the computer control system and raise the lower platen up till the specimen is held firmly by both platens with a pre-load of about 100 pounds. Run the Test-Ware program and select the appropriate template for the test (Brazilian).

3. Reset the displacement and start loading at a rate of 1.5 kip per minute. Open a new data file with the same name as the core ID. Machine will collect data and detect the increase in load and will retreat when it senses a drop of over 70% in load. This assures the termination of loading after failure and prevents unnecessary disintegration of the sample when internal failure occurs.

4. Record the peak load and make a sketch of the failure on the test sheet, photograph the sample with the ID card (if required), remove the sample and collect all the pieces and put it in a clearly labeled zip lock bag, and clean the machine. Close and back up the data file.

4. CALCULATION AND DATA REDUCTION

Use the information on the test sheet to calculate the Brazilian indirect tensile strength as follows:

\[
\sigma_t = \frac{2P}{\pi LD}
\]

Where:

- \( P \) = Maximum (failure) load
- \( D \) = Diameter of the sample
- \( L \) = Length of the sample

CERCHAR ABRASIVITY INDEX (CAI)

1. PURPOSE

This test a combined measure of rock abrasivity and strength, for determining cutter wear rate and costs. Actual cutter wear data from field projects allows the CAI to be related
directly to expected linear feet of cutter travel, which, when combined with current cutter prices and cutterhead geometry, allows projection of cutter costs per rock volume or linear foot of tunnel.

This test is performed on freshly broken rock surfaces, free of weathering effects. The remaining pieces from indirect (Brazilian) tension tests are used for this purpose. If no Brazilian test was done on sample, any piece of rock sample available can be used for this test.

2. SAMPLE PREPARATION

No particular sample preparation is required for this test. In the case where left over of Brazilian test is not available and other pieces are to be used, a fresh rock surface must be exposed by breaking the edges with a hammer.

3. TESTING

The rock sample is fixed in a holder with the fresh surface facing upward. The sample is held by a vice and secured in place using a layer of wood between the sample and the jaws. A conical 90° hardened steel pin, fastened in a 15 lbs (7.5 kg) head (dead weight), is set carefully on the rock surface and drawn 0.4 in. (1 cm) across it in 1 second. This is repeated for a total of five pins.

The tips of the pins then are examined under a reticular microscope and two perpendicular diameters of the resulting wear flat are recorded for each pin. Coating the pin tips with machinist’s blue dye prior to testing makes the wear flat more visible. A total of 10 measurements are taken and recorded on the test sheet.

4. CALCULATION AND DATA REDUCTION

The Cerchar abrasivity index (CAI) then is calculated by:

\[
CAI = 0.0254 \sum_{i=1}^{10} d_i
\]

Where: \(d_i\) pin diameter (in.)

The lower the CAI, less abrasive the rock is for cutting tools. A CAI of 1 is low abrasivity, while 6 is extremely abrasive.
Table 1. Reference list of cerchar abrasivity indices.

**PUNCH PENETRATION TEST**

1. **PURPOSE**

In this test, a standard conical indentor is pressed into a rock sample that has been cast in a confining steel as shown in schematic drawing. The load and displacement of the indentor are recorded with computer system. The slope of the force-penetration curve has been related to the mechanical cut-ability of the rock, i.e., the energy needed for efficient chipping. This test also provides a qualitative measure of rock brittleness/porosity.
2. EQUIPMENT

1. Rock saw
2. Caliper with 0.001 inch accuracy
3. Steel ring for casting the specimen
4. Hydrostone used as casting material
5. Standard indentor and the support plate
6. Lathe for machining of the casts
7. Servo controlled Hydraulic press, 600 kips MTS Rock Testing Machine
8. Computer and Data Acquisition system (Integrated into the MTS control system).
3. PROCEDURES

3.1 Sample Preparation

1. Record all pertinent information and measurements of core sample on the test sheet. This should include general rock and project information.

2. Log core sample to indicate the position, orientation, and condition of existing joints, fractures, bedding/foliation, inclusions or any other defects.

3. Cut the samples to a length of 1.0 to 1.5 times the diameter. The L/D ratio of 1 : 1 (or up to 1.5 : 1) must be met where possible. No sample with L/D below 1.0 shall be tested.

4. Measure diameter and length of the sample and record it on the test sheet.

5. Use a plastic sheet to cover the table and place the specimen face down, with the flat surface on the plastic sheet. Place the casting ring over the specimen and resting on the plastic sheet.

6. Prepare the casting material by mixing hydrostone with water at a proper ratio.

7. Cast the specimen in the steel ring by pouring the mixture evenly and gently around the specimen. Vibrate the mix with a spatula from time to time to make sure the air bubbles are removed and the mixture fully surrounds the sample. Avoid leaving any voids in the cast. Fill the steel ring with the mixture until it is fully covered.

8. Air dry the cast for at least 24 hours prior to testing.

9. Machine the bottom of the cast on the lathe to remove the extra hydrostone and create a flat surface. Continue machining until a fresh uniform surface is exposed on the bottom of the steel casting ring.

3.2. Punch-Penetration Test

1. Place and secure the sample on the lower platen of the loading machine. The sample must be placed in such a way that the indentor is more or less at the center of the specimen.

2. Close the shield doors on the test machine to protect the operator from flying pieces of rock in case of violent failure.

3. Turn on the machine, the hydraulic pumps, and the computer control system. Run configuration for the 600 kip machine. Raise the lower platen until the surface of the sample is within a few tenth of an inch from the tip of the indentor.
4. Open and run the punch test template. Open a new data file with the same name as the core ID. Reset the displacement and start loading. Machine will raise the cast till it touches the indentor and then continues loading the sample under a controlled displacement rate of 0.001 inches per second. Data is collected automatically and stored in the data file.

5. Continue testing until a penetration of 0.25 inches is reached. At the completion of the testing the machine will automatically retreat the lower platen to its original position.

6. Remove the sample from the machine. Close and back up the data file. Make a sketch of the failure surface on the test sheet, and photograph the sample with the ID. Card (if required).

4. DATA REDUCTION

Use the Punch penetration program to reduce data. Run the Excel file and select the data file. Select the origin and the program will estimate and plot the following slopes:

1. 45 degree slope

2. Average slope (average of slope of a floating point)

3. Peak slope (from origin to peak load)

4. Energy slope (area under the curve)

Record the results in the summary sheet.