**COLORADO DIVISION OF RECLAMATION, MINING AND SAFETY**
1313 Sherman Street, Room 215, Denver, Colorado 80203 ph(303) 866-3567

## REQUEST FOR TECHNICAL REVISION (TR) COVER SHEET

File No.: M- 1974-086 Site Name: Ralston Quarry

County Jefferson TR# *(DRMS Use only)*

Permittee: Asphalt Paving Co., Inc.

Operator (If Other than Permittee):

Permittee Representative: Ben Langenfeld

Please provide a brief description of the proposed revision:

Revisions to slide repair designs and supporting slope stability analysis.

As defined by the Minerals Rules, a Technical Revision (TR) is: "a change in the permit or application which does not have more than a minor effect upon the approved or proposed Reclamation or Environmental Protection Plan." The Division is charged with determining if the revision as submitted meets this definition. If the Division determines that the proposed revision is beyond the scope of a TR, the Division may require the submittal of a permit amendment to make the required or desired changes to the permit.

The request for a TR is not considered "filed for review" until the appropriate fee is received by the Division (as listed below by permit type). Please submit the appropriate fee with your request to expedite the review process. After the TR is submitted with the appropriate fee, the Division will determine if it is approvable within 30 days. If the Division requires additional information to approve a TR, you will be notified of specific deficiencies that will need to be addressed. If at the end of the 30 day review period there are still outstanding deficiencies, the Division must deny the TR unless the permittee requests additional time, in writing, to provide the required information.

There is no pre-defined format for the submittal of a TR; however, it is up to the permittee to provide sufficient information to the Division to approve the TR request, including updated mining and reclamation plan maps that accurately depict the changes proposed in the requested TR.

Required Fees for Technical Revision by Permit Type - Please mark the correct fee and submit it with your request for a Technical Revision.

| **Permit Type** | **Required TR Fee** | **Submitted** (mark only one) |
|---|---|---|
| 110c, 111, 112 construction materials, and 112 quarries | $216 | ☑ |
| 112 hard rock (not DMO) | $175 | ☐ |
| 110d, 112d(1, 2 or 3) | $1006 | ☐ |

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January 28, 2020

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**Asphalt Paving Co., Ralston Quarry M-1974-086**
**Technical Revision – Modifications to Slide Repair Designs and Plans**

Mr. Scott:

Asphalt Paving Co., the permittee of the Ralston Quarry, is submitting this technical revision to incorporate a modified plan to repair the slide area west of the quarry. Following geologic investigation of the slide area, it was determined that the extent of excavation required to implement the approved repair plan would create undesirable conditions.

Mainly, the depth of excavation necessary would make the slope susceptible to sliding. Construction safety was brought into question. There were concerns that the excavated slope for some buttress areas could slide down onto equipment during construction. The second undesirable condition was that the deeper the excavation went for buttress construction, the larger the horizontal disturbance needed. APC determined that it was better to re-evaluate the buttress designs and seek an effective, but less disturbance intensive, solution than to continue with the designs as they were.

This technical revision contains those new designs and revised slope stability analysis that demonstrates their effectiveness.

## 1. New Slide Repair Designs

The slide area at the Ralston Quarry has been divided into five zones in terms of repair designs. Each zone was evaluated to determine if a revised slope stability plan was needed following the geologic investigations in 2019. Ground Engineering evaluated each of the zones and their slope stability needs in a report that is attached. This report reiterates the previous Ground Engineering report from January 2019 as well as adding new information from additional data gathering and analysis.

The primary effect of the analysis conducted by Ground Engineering was to improve the slope stability designs in the southern half of Zone 3, shown on the maps and reports as Zone 3A and to better define the position of subsurface pipes and the extent of earthwork needed in Zone 4.

1.1 Zone 1
Zone 1 consists of the material pile built from slide material that is located south of the main drainage in this area. Its location can be seen on the attached Map F-2. No changes have been made to this zone's construction or maintenance plans. Ground Engineering's revised report reflects the same recommendations as the original 2019 report.

1.2 Zone 2
Zone 2 is the southernmost area along the western facing slope of the quarry. It can be seen on Map F-2. This zone consists mostly of long revegetated fill material and has not seen any slope stability issues at any point in its existence. Additional fill that has occurred along the toe of the slope since Ground Engineering's 2019 report has led to an increase in the stability of that slope from 1.25 to 1.3.

1.3 Zone 3
Zone 3 consists of the 2018 slide and surrounding ground. This area was evaluated in detail by Ground Engineering to determine the most effective and efficient way to accomplish slope stability without construction safety risk or unnecessary ground disturbance. Cross sections were developed perpendicular to the slope and different designs were evaluated along these cross sections in an iterative process to determine the appropriate repair design. Ground Engineering's attached report shows the slope stability that is achieved with these designs. Map F-2 shows the existing and final slope cross sections.

Portions of this zone will require only regrading to reach stable slopes. The areas around the 2018 slide will have buttresses and subsurface drainage installed to achieve stable slopes, in accordance with the existing approved plan. The net effect of additional test pits and slope evaluation has been to better define the extent to which buttresses and subsurface drains will be needed to achieve stable slopes in the area

Zone 3A will not require the installation of subsurface drains, as regrading of this section of slope will be sufficient to achieve a safety factor of 1.3. Two cross sections through this area were analyzed and can be seen on Map F-2.

Zone 3B will see the installation of buttresses and subsurface drainage as previously approved in order to meet the safety factor of 1.3.

1.4 Zone 4
Zone 4 consists of areas north of the 2018 slide that contain a previous, smaller slide area. This area was evaluated in detail by Ground Engineering to determine the most effective and efficient way to accomplish slope stability without construction safety risk or unnecessary ground disturbance. Cross sections (X1-X1' and X2-X2' on Map F-2) were developed perpendicular to the slope and different designs were evaluated along these cross sections in an iterative process to determine the appropriate repair design. Ground Engineering's attached report shows the slope stability that is achieved with these designs is 1.4 and 1.2. Map F-2 shows the existing and final slope cross sections.

Subsurface drainage systems will be installed in this zone. There arrangement can be seen on Map F-2.

1.5 Zone 5

Zone 5 consists of areas north of all other zones that have been previously determined to not require any regrading or modification for stability. No earthwork is planned in this area.

## 2. Subsurface Drainage

All subsurface drainage systems will consist of a minimum of 12-inch diameter pipe installed as shown on Map C-5. This pipe size was determined using the design standards from the Natural Resource Conservation Service Part NJ650.14[1] applied to a smooth HDPE pipe. The drainage area is ~17 acres. Inflow pipes (transverse to surface grade) will be perforated while transport pipes (longitudinal to surface grade) will be solid to convey drainage water to the slope toe. Additionally, this pipe size is in excess of Ground Engineering's recommendations for pipe size and flow capacity.

Map C-4 and F-2 show the revised subsurface pipe locations. None of the stormwater designs on Map C-4 have been changed from the approved permit designs.

## 3. Revegetation

All disturbed areas will be revegetated according to the approved permit. Stripped topsoil will be salvaged for replacement in stockpiles. Temporary sediment control structures will be installed downgrade of the disturbance areas as shown in the approved permit.

Upon successful revegetation of the disturbance areas related to the 2018 slide, the sediment control structures will be removed to allow a return of natural drainage paths.

## 4. Summary

The earthwork needed to create stable slopes on the west side of the Ralston Quarry was re-evaluated. New designs were developed to reduce construction scale and safety risk. These new designs were modelled by Ground Engineering and determined to produce stable slopes. Therefore, Asphalt Paving Co. is submitting this technical revision to implement these new designs at the Ralston Quarry.

[1] Figure 3-3 Subsurface drain discharge from drainage coefficient

**Table 1 - Attachments**

| Documents | Maps |
| --- | --- |
| **NJ650.14 Fig. 3-3** | C-4 |
| **Ground Engineering Report** | C-5 |
| | F-2 |
| | |

Please contact me if you have any further questions or concerns.

Sincerely,

Ben Langenfeld, P.E.
Greg Lewicki and Associates

# Limited Geotechnical Evaluation
# Slope Stability and Remediation Approach
# Ralston Quarry
# Jefferson County, Colorado
## Revised6

Prepared For:

**Asphalt Paving Company**

**14802 W. 44th Avenue**
**Golden, Colorado 80403**

**Attention: Mr. Stan Opperman**

**Job Number: 19-1235** **January 28, 2020**

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## PURPOSE AND SCOPE OF STUDY

This report presents the results of a geotechnical and slope stability evaluation performed by GROUND Engineering Consultants, Inc. (GROUND) of the west-facing slope on the Asphalt Paving Company aggregate quarry property north of Golden, Colorado. Our study was conducted in general accordance with GROUND's Proposal No. 1807-1305, dated July 23, 2018.

We understand that Asphalt Paving Company has been required to develop a remedial approach to provide long term stability of the west-facing slope of the ridge facing Ralston Reservoir. We also understand that Asphalt Paving Company has been asked to evaluate the long term stability of the stockpile where material displaced in a recent slope failure has been placed.

A field exploration program was conducted to obtain information on the surface and subsurface conditions. Material samples obtained from several test holes were tested in the laboratory to provide data on the classification and engineering characteristics of the on-site soils. Subsequently, a number of test pits were excavated to obtain additional subsurface data. This report has been prepared to summarize the data obtained and to present our findings and conclusions based on the subsurface conditions encountered to develop approaches to stabilize the slope in question. This revised study includes slope stability analyses in addition to those previously performed for GROUND's January 18, 2019, report.[1]

This report should be understood and utilized in its entirety; specific sections of the text, drawings, graphs, tables, and other information contained within this report are intended to be understood in the context of the entire report. This includes the *Closure* section of the report which outlines important limitations on the information contained herein.

This report was prepared for the planning purposes of the Asphalt Paving Company based on our understanding of the project at the time of preparation of this report. The

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[1] GROUND Engineering Consultants, Inc., 2018, Limited Geotechnical Evaluation, Slope Stability and Remediation Approach, Ralston Quarry, Jefferson County, Colorado, Revised2, Job No. 18-3046, prepared for Asphalt Paving Company, dated January 18.

data, conclusions, opinions, and geotechnical parameters provided herein should not be construed to be sufficient for other purposes, including the use by any other parties for any reason not specifically related to the design of the project. Furthermore, the information provided in this report was based on the exploration and testing methods as described. Deviations between what was reported herein and the actual surface and/or subsurface conditions may exist, and in some cases those deviations may be significant.

## INTRODUCTION

The project site is underlain generally by bedrock of the Pierre Shale on which residual (weathered in place) and colluvial ("slope wash") soils have developed. The clay and silt shales of that formation, however, were intruded by a sill of shoshonite, a dark gray, igneous rock.

The Asphalt Paving Company has mined aggregate from that shoshonite sill exposed on the eastern side of a north-trending ridge in Jefferson County north of Golden for several decades. The mining property straddles the ridge with Ralston Reservoir occupying the drainage to the west of the mining property and Upper Long Lake immediately to the east. The relative positions of these features can be seen in an historical Google Earth® images below.


*Asphalt Paving Co. site*


*Area of Tailings Placed on West-facing Slope*

Actual mining has been from the eastern side of the ridge. We understand that until approximately 20 years ago, tailings from the mining on the east side of the ridge were placed as fill on the west-facing side. The filled slope was graded at an approximately 2½ : 1 (horizontal : vertical) angle. In the early 2000's, several retention ponds also were constructed along the toe of the fill slope. These features are indicated in the historical Google Earth® image (above right).

## SUBSURFACE EXPLORATION

Subsurface exploration for the project was conducted in August and October, 2018. Twelve (12) test holes were drilled with a conventional, track-mounted drilling rig advancing 4-inch diameter auger to evaluate the subsurface conditions and retrieve samples for laboratory testing. The test holes were advanced at the approximate locations shown on the Google Earth® image in Figures 1 and 2 following the text, to depths of approximately 19 to 35 feet below existing grade. (Test Hole 5, however, met refusal on coarse materials at a depth of about 5 feet. Test Hole 9 met refusal at a depth of about 16 feet; and Test Hole 11 at a depth of about 6 feet.) A GROUND engineer directed the subsurface exploration, logged the test holes in the field, and prepared the samples for transport to our laboratory.

Samples of the subsurface materials were retrieved with a 2-inch I.D. California liner sampler. The samplers were driven into the substrata with blows from a 140-pound hammer falling 30 inches, a procedure similar to that described by ASTM Method D1586. Penetration resistance values, when properly evaluated, indicate the relative density or consistency of soils. Depths at which the samples were obtained and associated penetration resistance values are shown on the test hole logs.

In addition, in 2019, 16 test pits were excavated with a track-mounted excavator. The pits were advanced to depths of 8 to 22 feet at the approximate locations shown in Figures 1 and 2. Logs of the test holes are presented in Figures 3, 4 and 5. The logs of Test Holes 1 through 4 are re-drawn to elevation in Figure 6. Construction details of the groundwater monitoring and observation points installed at Test Holes 1 through 4 are provided in Figures 7 through 10. Logs of the test pits are provided in Figures 11 and 12 Explanatory notes and a legend are provided in Figures 13 and 14.

## LABORATORY TESTING

Samples retrieved from our test holes were examined and visually classified in the laboratory by the project engineer. Laboratory testing of soil samples included standard property tests, such as natural moisture contents, dry unit weights, grain size analyses, and Atterberg limits. Direct shear strength testing was performed on both relatively undisturbed samples and remolded disturbed (bulk) samples. Laboratory tests were performed in general accordance with applicable ASTM protocols. Results of the laboratory index testing are summarized in Table 1. Gradation plots are provided in Figures 15, 16 and 17. Plots of the direct shear strength data obtained are provided in Figures 18, 19 and 20.

## SUBSURFACE CONDITIONS

***Soils and Bedrock*** The test holes penetrated from about 1 foot to at least 16 feet of mine tailings placed on the west-facing side of the ridge. Due to encountering refusal at several locations, the tailings likely are significantly deeper, commonly. Up to about 25 feet of tailings were present along much of the central portion of the slope prior to the 2018 slope failure. Similar soils, relocated to the stockpile, were encountered in Test Holes 10 and 11. The tailings consist of fragments of shale and igneous rock (shosohonite) up to cobble- and boulder size, as well as clay and sand. Some of the tailings remain as placed, other volumes were displaced by prior slumping and some in the relative recent slope failures. A few inches of poorly developed topsoil had developed on the tailings at Test Hole 1.

The tailings had been placed on about 6 to 12 feet of native, clayey colluvial soils. At Test Hole 2, however, some displaced tailings overlay the Pierre Shale materials.

Beneath the surficial fill and native soils, the test holes encountered Pierre Shale materials to the depths explored. Locally, the Pierre Shale consisted of clean to sandy clay shales that were hard to very hard. The uppermost several feet of the shales typically were severely weathered.

More detailed descriptions of these materials are provided on Figures 13 and 14.

***Groundwater*** was encountered in the test holes at the time of drilling and then re-measured several times. Water levels and measurement dates are shown on the test hole logs. At the upper test holes (Test Holes 1 and 4) depth to groundwater typically was on the order of 30 feet whereas at the test holes at lower elevation (Test Holes 2 and 3) groundwater depths were more variable, but generally on the order of 18 feet. Groundwater levels can be expected to fluctuate, however, in response to annual and longer-term cycles of precipitation and surface drainage. Transient, local zones of perched groundwater likely develop commonly within the tailings and at the contact between the tailings and the native soils. Transient seeps and springs have been reported on the slope at various times, particularly in the areas where slope failures have occurred.

## SLOPE STABILITY MODELING

The slope stability analysis was performed by using the Slope-W® computer program. The Morgenstern-Price method was used to estimate strength parameters for the assumed factor of safety.

To supplement the laboratory strength data, slope stability analysis was performed on a critical slope through the area of the 2018 slope failure. This allowed the strength parameters of the mine tailings and colluvium to be back calculated, assuming a factor of safety of just below 1.0 for that case. Precipitation and snowmelt were assumed to have saturated the colluvial soils. (These assumptions are supported by previous slope failures, where groundwater seepage was reportedly observed near the toe of the failures shortly prior to the slope failures.) The results of this analysis yielded values for the angles of internal friction similar to those obtained from laboratory testing.

Based on those combined findings, the parameters tabulated below were used to in the further stability modeling. Note that the unit weights listed are moist but un-saturated unit weights (i.e., not buoyant unit weights).

| **Material** | **Unit Weight** $\gamma'$ (pcf) | **Friction Angle** $\phi$ (degrees) | **Cohesion** **c** (psf) |
|---|---|---|---|
| Tailings | 125 | 35 | 100 |
| Colluvium | 125 | 22 | 135 |
| Shale Bedrock | 130 | 20 | 1,000 |

We understand that the tailings were placed on approximately the natural ground surface that had received little preparation for filling and had not been provided with benches or drains. Subsequently, water accumulated at the contact between the tailings and the underlying clayey, colluvial soils, wetting those underlying soils.

Our observations and the results of our analyses suggest that the most likely failure mode of the mine tailing slope is large-radius, rotational movements through both the tailings and colluvial soils. The stability of the tailings and colluvial soils appear to be controlled by the locations and degree to which perched groundwater affected the tailings and colluvium shear strength. (The failed areas largely corresponded to areas where retention ponds had been constructed, suggesting that the presence of those ponds was a significant factor in the development of the slope failures at the site.)

Additional analyses were performed to model conditions following remedial construction, particularly grading to re-locate tailings from the higher to the lower portions of the slope and construct somewhat flatter finished grades, as well as with stabilization embankments constructed in one area and slope drains in another. Again, the Morgenstern-Price method was used to estimate the factors of safety against deep-seated failure for the as-remediated conditions of the several zones along the slope.

The remediated conditions for the area of the 2018 slope failure also were modeled under seismic loading conditions. A peak horizontal ground acceleration of 0.05g was used. The US Geological Survey indicates a peak horizontal ground acceleration of about 0.065g for the site, but that value is based on the 'maximum credible earthquake' which generally is based on a return period of 1,000 years and sometimes longer. The 'maximum probable earthquake' for the site, based on a return period of 100 years would yield a significantly reduced ground shaking. Therefore, a peak horizontal ground acceleration of 0.05g is considered conservative for the site, given low likelihood of a major earthquake event in the area.

## DISCUSSION

For clarity, we have divided the study area into six zones generally from south to north:

**Zone 1** is the stockpile of relocated mine tailings constructed in 2017 and 2018.

**Zone 2** is the southern portion of the slope on which tailings were placed, and where additional filling appears to have buttressed the slope. No retention ponds were constructed at the toe of the slope in this zone. No slope failures have been reported in this zone.

**Zone 3a** is the south-central portion of the slope, south of the 2018 failure.

**Zone 3b** is the central portion of the slope where the 2018 slope failure developed.

**Zone 4** is the portion of the slope north of Zone 4 where the earliest, significant slope failure began to develop in 2003 and where remedial earthwork was performed by Asphalt Paving Company in 2010.

**Zone 5** is the portion of the west-facing ridge slope along which tailings were placed only on the top 25 feet (±) vertically, and is therefore remains in close to its pre-mining condition. No slope failures have been reported in this zone.

These zones are shown on the Google Earth® image on the next page.

The results of the stability modeling and the remedial grading proposed by GROUND to yield factors of safety against slope failure in the future are addressed below for each zone.

GROUND will be available to assist the civil engineer in preparing grading plans to implement the measures outlined herein.


Zone 5
Zone 4
Zone 3b
Zone 3a
Zone 2
Zone 1
(Stockpile)
Approximate Zone Boundaries

A table summarizing the approximate factors of safety against deep-seated slope failures based on the proposed remediation is presented below. Each zone subsequently is discussed in more detail.

| Zone | Factor of Safety | Remediation to Achieve Factor of Safety |
|---|---|---|
| 1 | **2.4** | None |
| 2 | **1.25** | None |
| 3a | **1.3** | Re-grading, shifting tailings down slope. |
| 3b | **1.3** | Stability embankments with heel drains. |
| 4 | **1.2 & 1.4** | Re-grading, shifting tailings down slope + interceptor drains. |
| 5 | N/A | None |

***Zone 1 – Relocated Tailings Stockpile*** In this area, displaced material – primarily tailings – removed from the slope to northeast were stockpiled. The stockpile was constructed on Asphalt Paving Company property that, we understand, now has been incorporated into the ground permitted for mining.

The stockpile extends over an area about 360 feet by 420 feet and is up to approximately 25 feet in thickness. The stockpile margin slopes descending to the north and to the west were graded at an angle of about 3 : 1 (horizontal : vertical) or slightly flatter as shown in the photographs below.


*Stockpile of Relocated Tailings, seen from the northeast*

*Northern Slope of Stockpile of Relocated Tailings*

Scrapers were used to construct the stockpile, bringing material from the slope. An average penetration resistance value (sampler blow count, **N**) of 14 was obtained from the test holes advanced into the stockpile (Test Holes 10 and 11) indicating that the fill materials comprising the stockpile were reasonably well compacted.

Stability analyses of the stockpile slope (Section E – E') on the Google Earth® image on Figure 21 indicated a minimum, Morgenstern-Price factor of safety against failure of about **2.4**, as shown on the SlopeW® output below, even with the groundwater at the base of the tailings stockpile which we consider unlikely.

*Remedial Measures* Any additions to the stockpile also should be placed so that a maximum slope angle of 3:1 (horizontal : vertical) is maintained. The stockpile slopes should be protected from erosion. Minor raveling or surficial sloughing should be anticipated on the slopes until vegetation is well re-established.

The final configuration of the stockpile should be designed to direct surface water away from the taller northern and western slope faces and into appropriate drainage pathways. Periodic maintenance of the stockpile should be performed to correct ponded areas and re-establish effective surface drainage as well as repair sloughed or eroded areas on the slopes.

***Zone 2 – The Southern Slope*** The extent of this zone is shown on a September 12, 2019, Google Earth® image in Figure 22. Tailings were placed on this portion of the original, west-facing slope over the same general time frame as the portions of the slope farther north.

*Zone 2 Slope seen from the southwest*

No retention pond was constructed below this reach, however, and addition tailings and other excavated materials have been placed against the lower portion of the slope as shown in the photograph above.

Slope stability analyses of this zone of the slope in the proposed (re-graded) condition (Section A – A') indicated a Morgenstern-Price factor of safety against failure of about **1.25**, as shown on the SlopeW® output on Figure 22, even with the water at the contact between the tailings and the underlying, native soils.

The additional fill appears to act as a buttress to increase the factor of safety of this reach of slope. No slope failures have been reported in this zone, or noted on historic aerial photographs reviewed for this study. This area also is south of the reservoir with the Zone 1 fill to the west of it. A failure here, should it develop, appears unable to affect off-site areas or the reservoir adversely.

*Remedial Measures* Because of the apparent stability, historically, of this zone of the slope and the factor of safety obtained for Zone 2 by slope stability modeling, no further remedial measures appear necessary at this time. The slope should be observed periodically for indications of instability or excessive erosion, particularly after heavy precipitation events.

***Zone 3a – The South-Central Slope*** The extent of this zone is shown on a September 12, 2019, Google Earth® image in Figure 23. Tailings also were placed on this portion of the original, west-facing slope over the same general time frame as the portions of the slope farther north. This zone has a smaller volume of fill on the lower slope to act as a buttress and previously a retention pond was present at the toe of the slope in this zone. A small slope failure developed in the central portion of this zone in 2005. Additional movement of the failed area took place in 2007. In 2010, that area was re-graded and the retention pond removed. No subsequent failures have developed in Zone 3a.

Slope stability analyses of this zone of the slope in the proposed (re-graded) condition (Sections X3 – X3' and X4 – X4') indicated a Morgenstern-Price factor of safety against failure of about **1.3** for both sections, as shown on the SlopeW® output on Figure 23, even with the water at the contact between the tailings and the underlying, native soils.

*Remedial Measures* Based on the factor of safety obtained for Zone 3a by slope stability modeling, the proposed re-grading should be implemented in this zone. Minor raveling or surficial sloughing should be anticipated on the slopes until vegetation is well re-established. The slope should be observed periodically for indications of instability or excessive erosion, particularly after heavy precipitation events.

***Zone 3b – The Central Slope Slope*** In Zone 3b (shown on a September 12, 2019, Google Earth® image in Figure 24a) tailings extended the farthest down slope, but exhibited no slope failures prior to construction of the retention ponds which appear to have been constructed in 2002. The slope failures in Zone 3b appear to have been moderate to low velocity failures.

Slope stability analyses of this zone of the slope indicated a Morgenstern-Price factor of safety against failure of less than 1.0, with the groundwater at the contact between the tailings and the underlying, native soils as suggested by local, transient seepage from the slope.

*Remedial Measures* In GROUND's opinion, in order to increase the factor of safety against failure in Zone 3b, two stability embankments should be constructed along the slope across the full north to south width of the zone. The excavations to construct the embankments should be extended through the colluvium and into in-place shale (into firm weathered shale is sufficient) to interrupt the low-strength layer of colluvium underlying the tailings. This geometry is shown on the SlopeW® output in Figure 24a. The stability embankments are shown in green. Stability analyses of this geometry indicated a calculated Morgenstern-Price factor of safety against failure of about **1.3**. With seismic loading, the calculated minimum factor of safety was about **1.1** as shown on Figure 24b.

The stability embankments should be constructed of properly compacted granular material, similar to the tailings, exhibiting an angle of internal friction (**ϕ**) of at least 35 degrees and an effective cohesion (**c**) of at least 100 psf.

Embankment construction will include

- cutting near horizontal benches through the tailings and the underlying colluvial soils to reach the underlying shale.
- Relatively coarse, granular materials (the tailings or similar) would be placed as compacted fill to backfill the excavation to approximate original grade.
- Prior to backfilling, a back drain would be installed at the heel of each stability embankment excavation to limit future saturation of the colluvial soils beneath the tailings. Geotechnical parameters for back drain design are provided in the *Embankment Back Drains* section of this report.
- The lower portions of the slope, including the stability embankments then would be graded to a uniform slope angle of 3 : 1 (horizontal : vertical). This would include removal of the existing retention ponds.

Geotechnical parameters for embankment earthwork are provided in the *Project Earthwork* section of this report. GROUND will be available to observe the embankment excavations, particularly to assess whether bedrock has been reached and to address questions and conditions that may arise. We also are available to provide quality assurance testing of the materials during earthwork.

A schematic layout of the embankment drains relative to the failure is shown in Figure 25. The approximate locations of the embankment drains are shown on Figure 26.

Minor raveling or surficial sloughing should be anticipated on the slopes until vegetation is well re-established. Asphalt Paving Company should provide for periodic observation of the completed earthworks to identify areas of ground cracking, water discharge, etc. This should be performed quarterly and after major precipitation events. GROUND will be available to assist with that, as well.

***Zone 4 – The Earliest Failure*** The boundaries of Zone 4 shown on a September 12, 2019, Google Earth® image in Figure 27a. The outline of a previous slope failure is visible within the zone, indicated by the red arrow. This slope failure in Zone 4 also appears to have been a moderate to low velocity failure. In 2010, the area was re-graded by Asphalt Paving Company to construct a buttress and directing surface water away from the toe area of the failure. There has been little or no further movement in the area of that slope failure area, and the vegetation has re-grown extensively since that time.

Slope stability analyses of Zone 4 of the slope were made of the proposed (re-graded) condition with drains installed to keep water from accumulating in the tailings. The sections analyzed for slope stability for this study are indicated approximately in Figure 27a (X1 – X1' and X2 – X2'). Analyses indicated a Morgenstern-Price factor of safety against failure of about **1.23** for X1 – X1' and **1.40** for X2 – X2' as shown on the SlopeW® output on Figure 27a and b, respectively, for this condition.

*Remedial Measures* Based on the factor of safety obtained for Zone 4 by slope stability modeling, the proposed re-grading should be implemented in this zone, and two transverse (roughly north – south) slope drains should be installed to keep water from accumulating in the tailings section. The slope drains should be installed at mid-slope and relatively low on the slope. A typical slope drain section is provided in Figure 28. Minor raveling or surficial sloughing should be anticipated on the slopes until vegetation is well re-established. The slope should be observed periodically for indications of instability or excessive erosion, particularly after heavy precipitation events.

***Zone 5 – The Northern Slope*** The extent of this reach is shown on a September 12, 2019, Google Earth® image in Figure 29. The blue zone boundary line also indicates the approximate limit of the tailings adjacent to this reach.

A recent photograph of the slope in Zone 5 is shown below.

*Zone 5 Native Slope seen from the south*

Tailings were not placed on this portion of the west-facing slope except on the uppermost approximately 25 vertical feet, as can be seen also in the November 22, 2003 Google Earth® image below. Native soils were encountered downslope of the tailings.


*Zone 5 without Tailings*

An exposure of the native colluvial soils in Zone 5 is shown in the recent photograph below.

*Colluvial Soils Exposed in Trail Cut in Zone 5*

No retention pond was constructed at the toe of slope in this zone and no slope failures have developed on this reach, based on reports from Asphalt Paving Company personnel and review of historic aerial photographs as old as 1999.

*Remedial Measures* Because of the apparent stability, historically, of this zone of the slope and because it remains nearly in its undisturbed configuration, we do not consider remedial earthwork or other remedial measures necessary for this zone. The slope should be observed periodically for indications of instability or excessive erosion, particularly after heavy precipitation events.

**PROJECT EARTHWORK**

***Use of On-Slope Tailings*** The existing tailings on the subject slope are anticipated to comprise a significant proportion of the materials placed as compacted fill either in the stability embankments or stockpiles. We anticipate that the majority of the on-slope tailings will be suitable for re-use as compacted fill. Fragments of rock and cobbles up to **12 inches** in maximum dimension may be included in the stability embankment fills, as long as the coarse clasts are not nested with resultant voids, and the surrounding soils can be compacted effectively.

However, deleterious materials such as trash, organic material, and boulders, or construction debris likely will be encountered in the on-slope tailings and should not be included in stability embankment fills.

Excess tailings from the slope should be placed in the stockpile (Zone 1) moisture-conditioned and compacted.

***Use of Existing Native Colluvial Soils*** Based on the samples retrieved from the test holes, we anticipate that the existing native soils consist largely of clays that should not be used to construct the embankment fills. (We anticipate that scattered pockets of clayey soil may become incorporated into the stability embankments, but this should be avoided to the extent possible.) The clay soils may be placed as fill in other locations.

***Imported Fill Materials*** We understand that most or all of the fill soils imported to the slope will be granular materials mined elsewhere on the Asphalt Paving Company property. Materials brought to the slope for use to construct the stability embankments. Imported material should exhibit **15 percent or less** passing the No. 200 Sieve and a plasticity index of **5 or less**. Materials proposed for import should be approved prior to transport to the slope.

***Fill Platform Preparation*** Prior to filling, the top **12 inches** of in-place materials on which fill soils will be placed (except for utility trench bottoms where bedding will be placed) should be scarified, moisture conditioned and properly compacted in accordance with the criteria below to provide a uniform base for fill placement.

If surfaces to receive fill expose organic materials or loose, wet, soft, or otherwise deleterious material, additional material should be excavated, or other measures taken to establish a firm platform for filling. A surface to receive fill should be effectively stable prior to placement of fill.

***General Considerations for Fill Placement*** No fill materials should be placed, worked, rolled while they are frozen, thawing, or during poor/inclement weather conditions.

Care should be taken with regard to achieving and maintaining proper moisture contents during placement and compaction.

Compaction areas should be kept separate, and no lift should be covered by another until relative compaction and moisture content within the specified ranges are obtained.

***Compaction Criteria for the Stability Embankments*** Fill soils should be thoroughly mixed to achieve a uniform moisture content, placed in uniform lifts not exceeding **8 inches** in loose thickness, and properly compacted.

Granular materials placed as embankment fill should be compacted to **95 or more percent** of the maximum dry density at moisture contents **within 3 percent** of the optimum moisture content as determined by ASTM D1557, the 'modified Proctor.'

***Compaction Criteria for Other Fills*** Fill soils should be thoroughly mixed to achieve a uniform moisture content, placed in uniform lifts not exceeding **12 inches** in loose thickness, and properly compacted.

Soils that classify as **GP, GW, GM, GC, SP, SW, SM, or SC** in accordance with the USCS classification system (granular materials) should be compacted to **92 or more percent** of the maximum dry density at moisture contents **within 3 percent** of the optimum moisture content as determined by ASTM D1557, the 'modified Proctor.'

Soils that classify as **ML, MH, CL, or CH** should be compacted to **at least 95 percent** of the maximum dry density at moisture contents **within 2 percent of** the optimum moisture content as determined by ASTM D698, the 'standard Proctor.'

***Settlements*** Settlements will occur in newly filled ground, typically on the order of 2 percent of the fill depth. If fill placement is performed properly, in GROUND's experience the majority of settlement likely will take place during earthwork construction.

## EXCAVATION CONSIDERATIONS

***Excavation Difficulty*** Test holes for the subsurface exploration were advanced to the depths indicated on the test hole logs by means of conventional, truck-mounted, geotechnical drilling equipment. Therefore, in general, we anticipate no unusual excavation difficulties in these materials, in general, for the proposed construction with conventional, heavy duty, excavating equipment.

However, refusal was encountered locally in the on-slope tailings. The contractor should anticipate encountering coarse cobble- and boulder-sized fragments of rock within the tailings that may require extra effort or specialized equipment to excavate, handle and process.

Likewise excavations that are extended into the shale bedrock generally will encounter increasingly difficult excavation conditions with depth.

***Temporary Excavations and Personnel Safety*** Excavations in which personnel will be working must comply with all applicable OSHA Standards and Regulations, particularly CFR 29 Part 1926, OSHA Standards-Excavations, adopted March 5, 1990. The contractor's "responsible person" should evaluate the soil exposed in the excavations as part of the contractor's safety procedures. GROUND has provided the information in this report solely as a service to Asphalt Paving Company and is not assuming responsibility for construction site safety or the construction activities.

***Groundwater and Surface Water*** Groundwater was encountered during subsurface exploration at various depths and elevations. Locally, groundwater was measured to be as shallow as about 11 feet below existing grade. Therefore, groundwater likely will be encountered in project excavations. Temporary drainage measures will benefit project earthwork.

The contractor should take pro-active measures to control surface waters during construction and maintain good surface drainage conditions to direct waters away from excavations and into appropriate drainage structures.

Temporary slopes should be protected against erosion.

## SURFACE DRAINAGE

Effective surface drainage should be established following remedial construction. The graded area should be observed and maintained, re-establishing effective drainage as necessary so that water is not allowed to pond on or below the slope.

In GROUND's opinion, the retention ponds constructed along the toes of the slope in the early 2000's reportedly for water quality purposes were major factors contributing to the subsequent slope failures. Those retention should be removed and the approximate prior surface gradients should be re-established.

## EMBANKMENT BACK DRAINS

As indicated above, the benches excavated for construction of the stability embankments should be provided with back drains at the heels of the excavation. A conceptual layout of the back drain systems is shown in Figure 25 and their approximate locations in Figure 26.

Multiple high points and discharge lines should be constructed, as appropriate.

***Geotechnical Parameters for Back Drain Design*** Back drain design should incorporate the parameters below. The actual back drain layout and outlets should be developed by a civil engineer. Typical, cross-section detail of back drains for this project are provided in Figure 30.

Each back drain array should be tested by the contractor after installation and after placement and compaction of the overlying backfill to verify that the system functions properly.

1) The back drain systems should consist of perforated, rigid, PVC collection pipe at least **6 inches** in diameter, non-perforated, rigid, PVC discharge pipe at least **6 inches** in diameter, free-draining gravel, and filter fabric.

2) The free-draining gravel should have a maximum particle size of **2 inches**, **50 percent or more** retained on the No. 4 Sieve, and **5 percent or less** passing the No. 200 Sieve.

3) Each collection pipe should be surrounded on the sides and top (only) with **12 or more inches** of free-draining gravel.

The gravel surrounding the collection pipe(s) should be wrapped with filter fabric (Mirafi 140N® or the equivalent) to reduce the migration of fines into the drain system.

4) Each back drain array should be designed to discharge at least **25 gallons per minute** of collected water.

5) The flow lines of the collection pipes should be placed at a gradient of at least 1 percent.

The flow lines of the discharge pipes should be placed at gradients as determined by the civil engineer but presumably generally will follow the descent of the overall slope.

6) The underdrain discharge pipes should be routed to outlet(s) for gravity discharge at appropriate locations and/or drainage structures.

7) Back drain ‘clean-outs’ should be provided at appropriate intervals and locations to facilitate maintenance.

8) Regular maintenance of the back drain systems should be performed to insure that they continue to work properly.

Observing the drains weekly (±) for the first year after completion and then monthly for at least 2 years after that appears appropriate. We suggest quarterly observation subsequently.

## CLOSURE

***Materials Testing*** Asphalt Paving Company or the contractor should consider retaining a geotechnical engineer to perform materials testing during construction. The performance of such testing or lack thereof, however, in no way alleviates the burden of the contractor or subcontractor from constructing in a manner that conforms to applicable project documents and industry standards. The contractor or pertinent subcontractor is ultimately responsible for managing the quality of his work; furthermore, testing by the geotechnical engineer does not preclude the contractor from obtaining or providing whatever services that he deems necessary to complete the project in accordance with applicable documents.

***Limitations*** This report has been prepared for Asphalt Paving Company as it pertains to design of improvements to the subject slope to enhance gross slope stability as described herein. It may not contain sufficient information for other parties or other purposes.

In addition, GROUND has assumed that project construction will commence by Summer 2019. Any changes in project plans or schedule should be brought to the attention of a geotechnical engineer, in order that the geotechnical conclusions in this report may be re-evaluated and, as necessary, modified.

The geotechnical conclusions in this report relied upon subsurface exploration at a limited number of exploration points, as shown in Figure 2, as well as the means and methods described herein. Subsurface conditions were interpolated between and extrapolated beyond these locations. It is not possible to guarantee the subsurface conditions are as indicated in this report. Actual conditions exposed during construction may differ from those encountered during site exploration.

If during construction, surface, soil, bedrock, or groundwater conditions appear to be at variance with those described herein, a geotechnical engineer should be retained at once, so that re-evaluation of the conclusions for this site may be made in a timely manner. In addition, a contractor who obtains information from this report for development of his scope of work or cost estimates may find the geotechnical information in this report to be inadequate for his purposes or find the geotechnical conditions described herein to be at variance with his experience in the greater project area. The contractor is responsible for obtaining the additional geotechnical information that is necessary to develop his workscope and cost estimates with sufficient precision. This includes current depths to groundwater, etc.

It is important that ALL aspects of this report, as well as the estimated performance (and limitations with any such estimations) of proposed improvements are understood by the

Asphalt Paving Company. If any information referred to herein is not well understood, then Asphalt Paving Company or other members of the design team, should contact the author or a GROUND principal immediately. We will be available to meet to discuss the risks and remedial approaches presented in this report, as well as other potential approaches, upon request.

This report was prepared in accordance with generally accepted soil and foundation engineering practice in the Jefferson County, Colorado, area at the date of preparation. GROUND makes no warranties, either expressed or implied, as to the professional data, opinions or conclusions contained herein. This document, together with the concepts and conclusions presented herein, as an instrument of service, is intended only for the specific purpose and client for which it was prepared. Re-use of, or improper reliance on this document without written authorization and adaption by GROUND Engineering Consultants, Inc., shall be without liability to GROUND Engineering Consultants, Inc.

GROUND appreciates the opportunity to complete this portion of the project and welcomes the opportunity to provide Asphalt Paving Company with a proposal for construction observation and materials testing.

Sincerely,

**GROUND Engineering Consultants, Inc.**

Ben Fellbaum, P.G.

Reviewed by Brian H. Reck, P.G., C.E.G., P.E.


GOOGLE EARTH AERIAL IMAGE (06/09/2017)

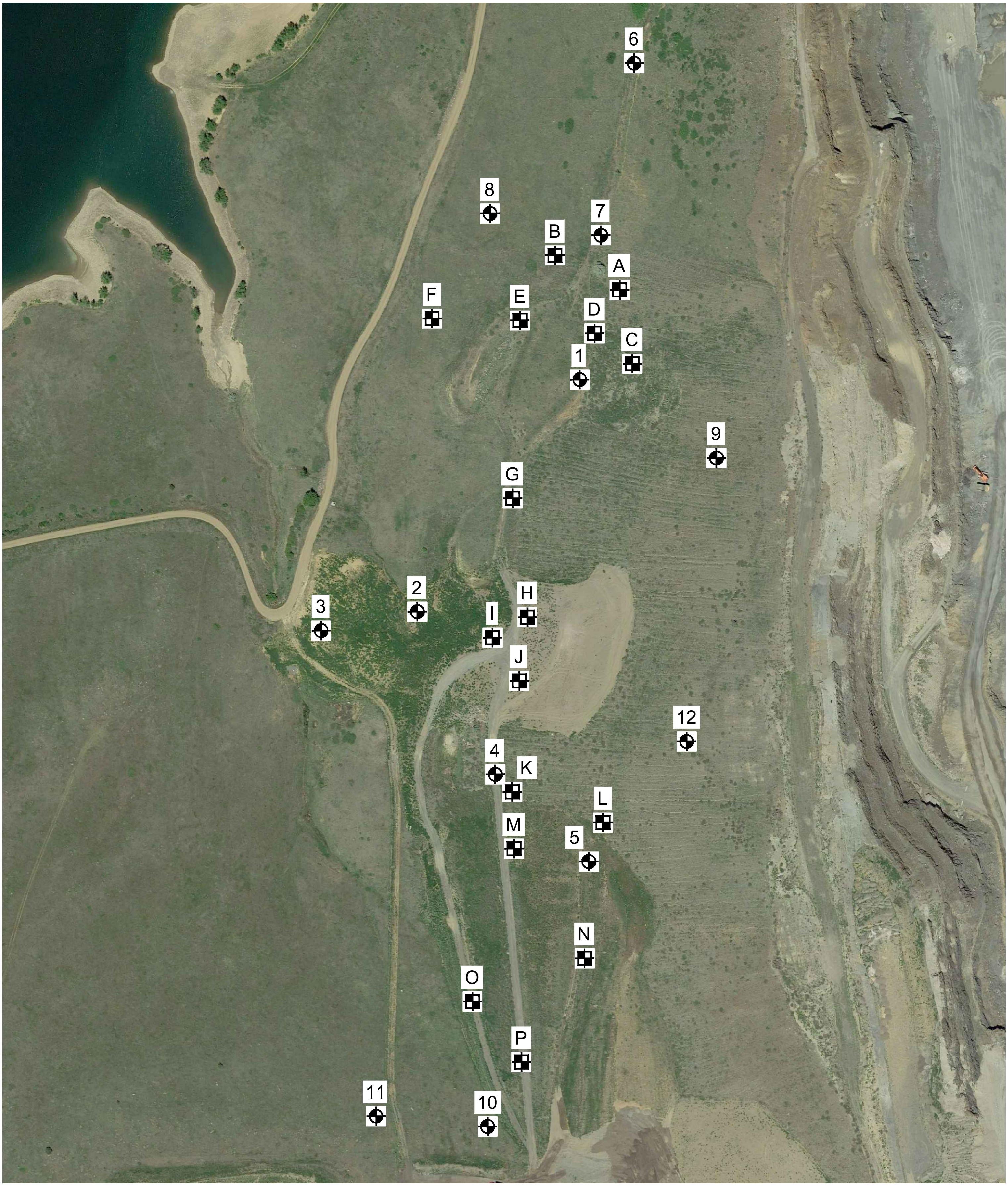
Indicates approximate area of work.

(Not to Scale)

**GROUND**
ENGINEERING CONSULTANTS

VICINITY MAP

| | |
|---|---|
| JOB NO.: 19-1235 | FIGURE: 1 |
| CADFILE NAME: 1235SITE.DWG | |




GOOGLE EARTH AERIAL IMAGE (05/31/2018)

1 — Indicates test hole number and approximate location. (Drilled under separate scope)

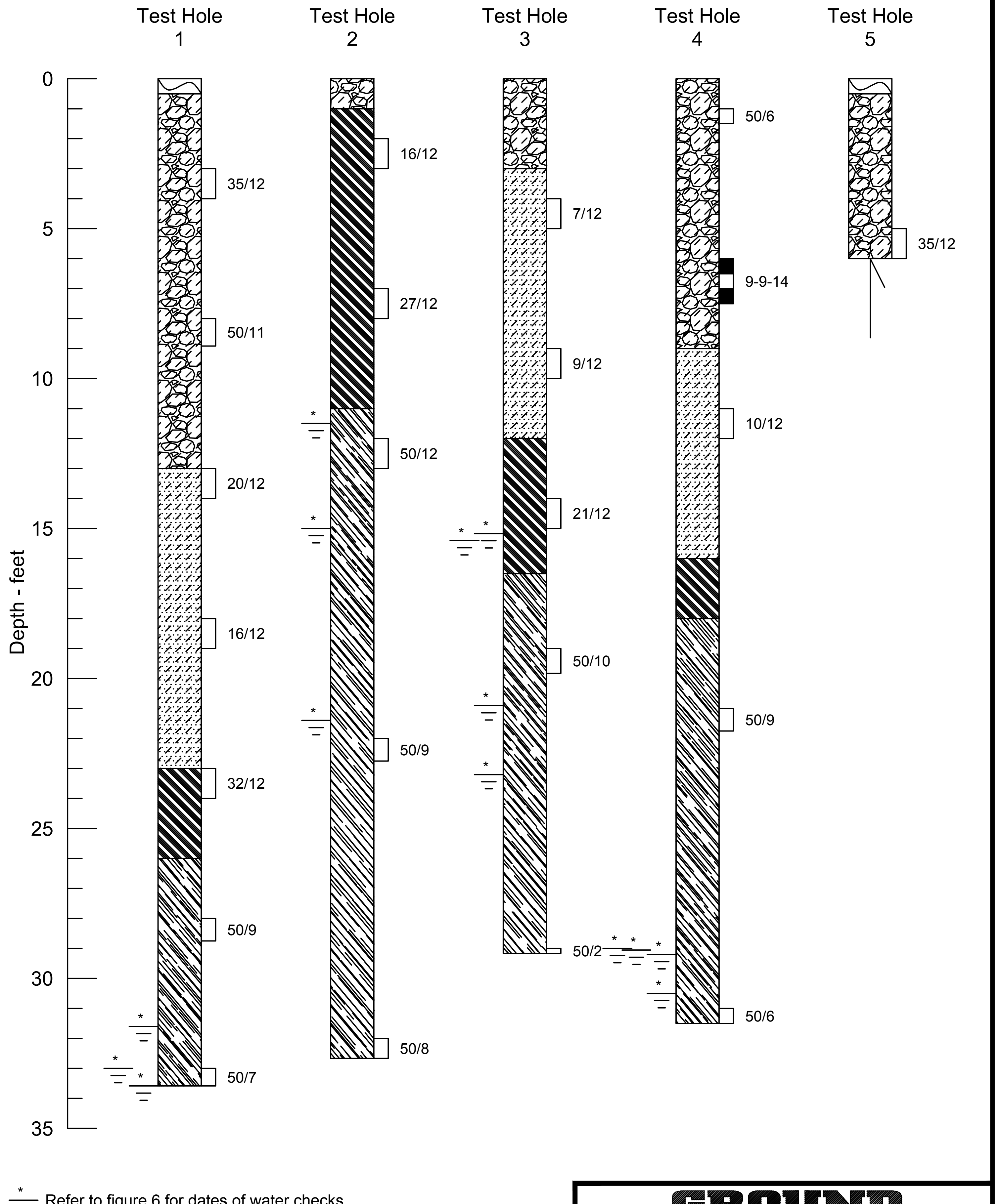
P — Indicates test pit number and approximate location.

(Not to Scale)

GROUND ENGINEERING CONSULTANTS

LOCATION OF TEST HOLES/PITS

| | |
|---|---|
| JOB NO.: 19-1235 | FIGURE: 2 |
| CADFILE NAME: 1235SITE.DWG | |

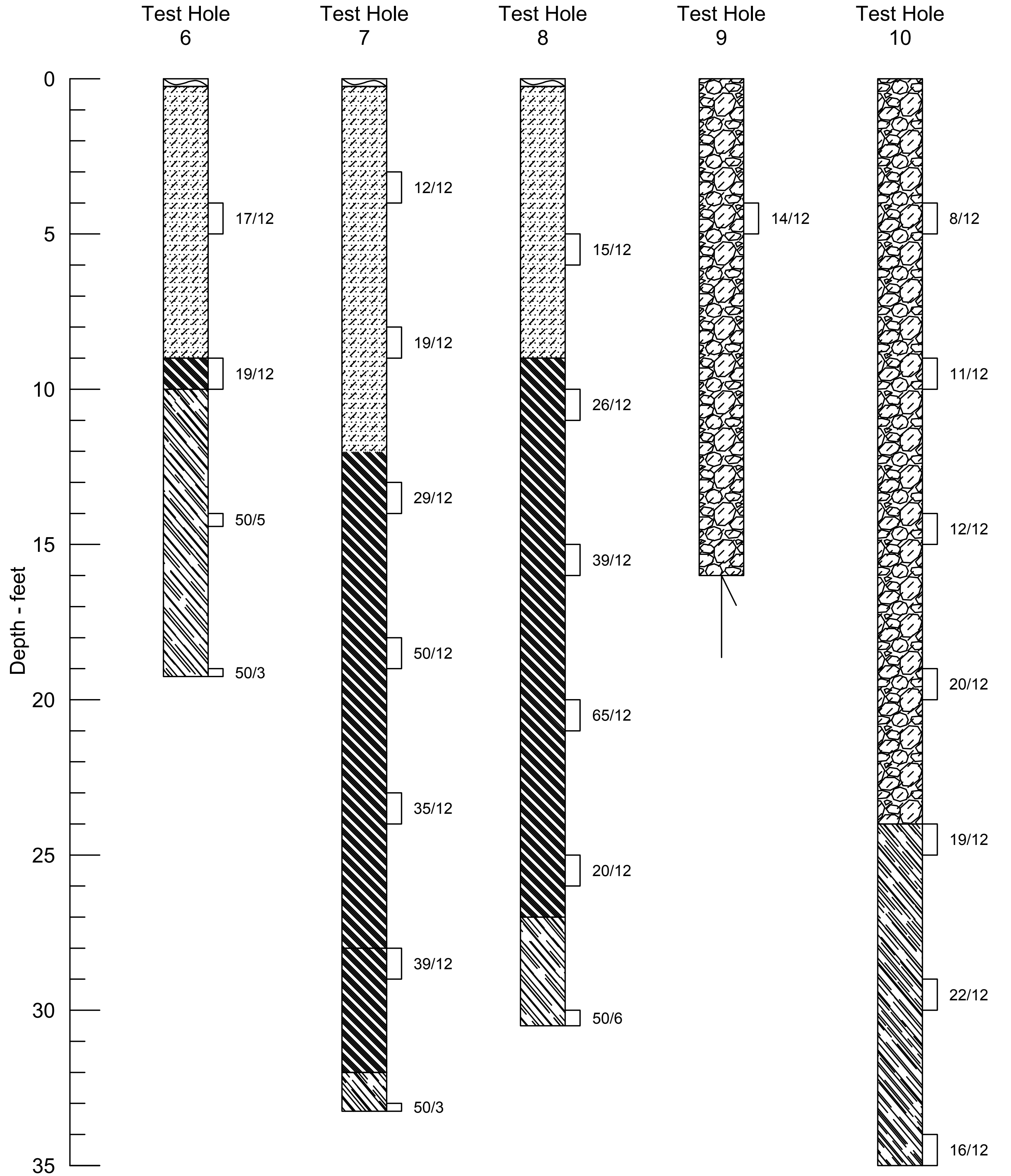



GROUND ENGINEERING CONSULTANTS

LOGS OF TEST HOLES

| JOB NO.: 19-1235 | FIGURE: 3 |
|---|---|

CADFILE NAME: 1235LOG01.DWG

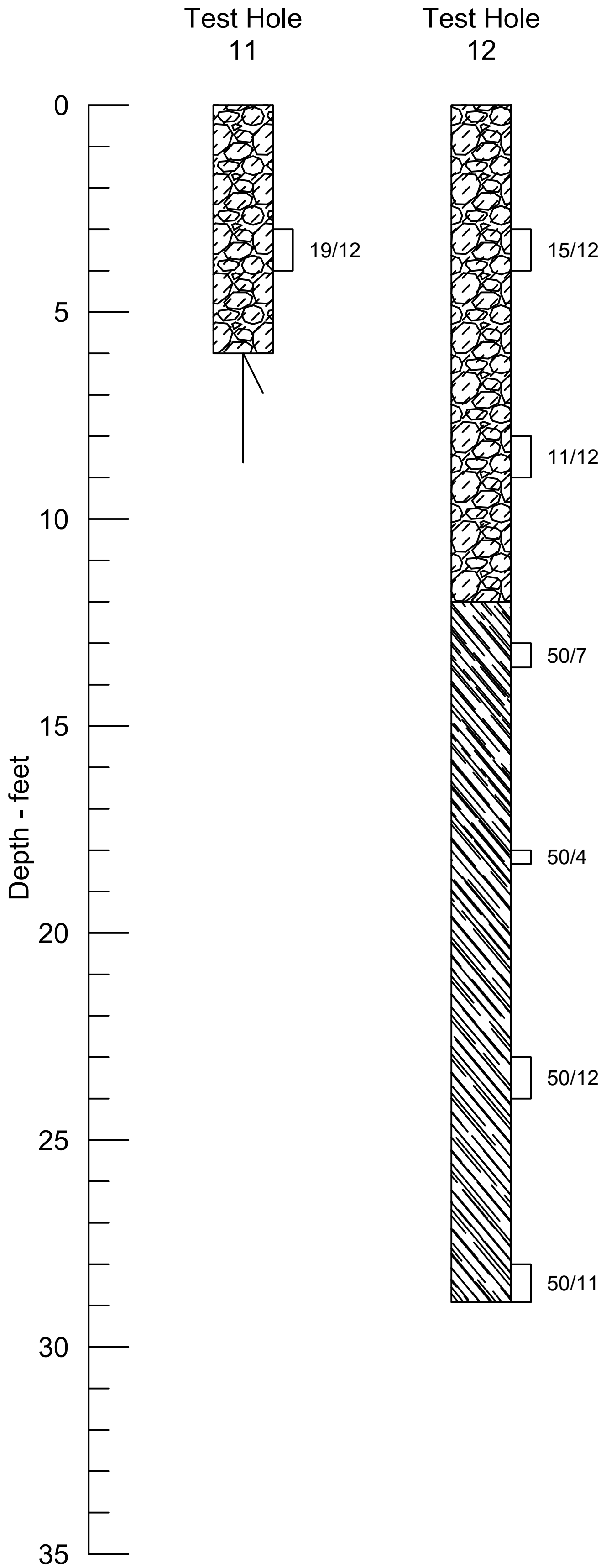



GROUND ENGINEERING CONSULTANTS

LOGS OF TEST HOLES

| JOB NO.: 19-1235 | FIGURE: 4 |
|---|---|

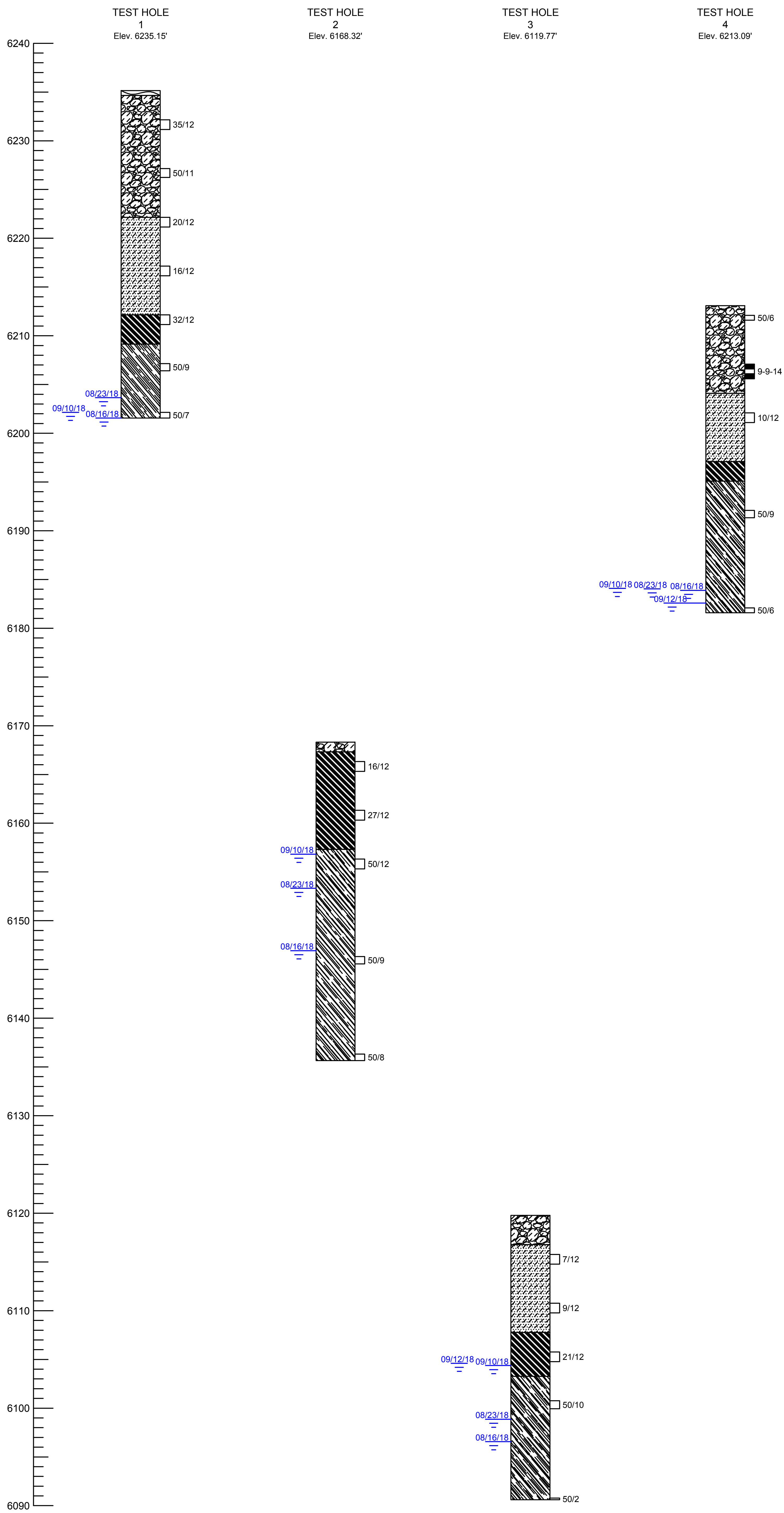
CADFILE NAME: 1235LOG02.DWG




GROUND ENGINEERING CONSULTANTS

LOGS OF TEST HOLES

| JOB NO.: 19-1235 | FIGURE: 5 |
| --- | --- |
| CADFILE NAME: 1235LOG03.DWG | |

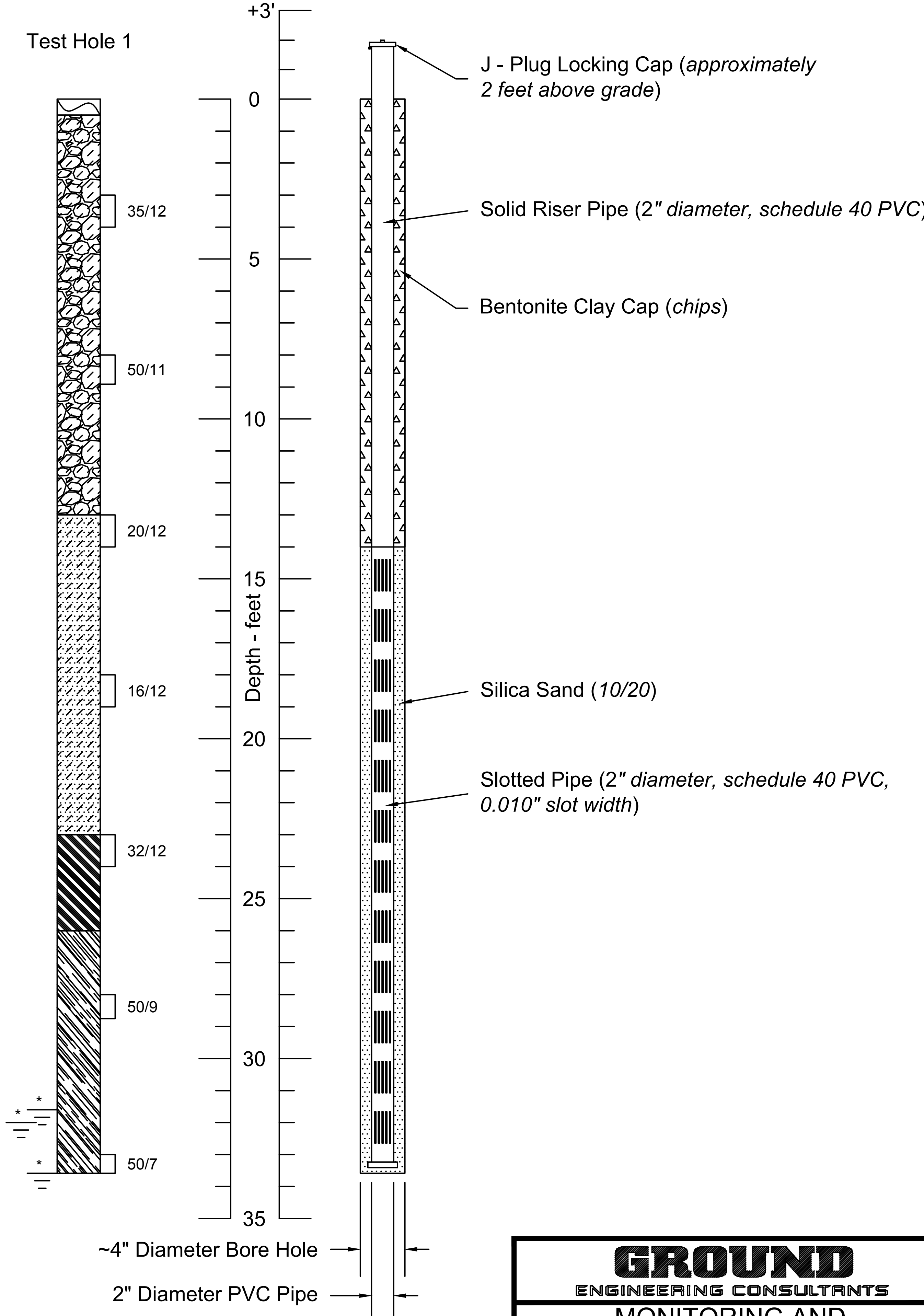



GROUND ENGINEERING CONSULTANTS

LOG OF TEST HOLES 1-4
AT ELEVATION

| | |
|---|---|
| JOB NO.: 19-1235 | FIGURE: 6 |

CADFILE NAME: 1235LOG04.DWG

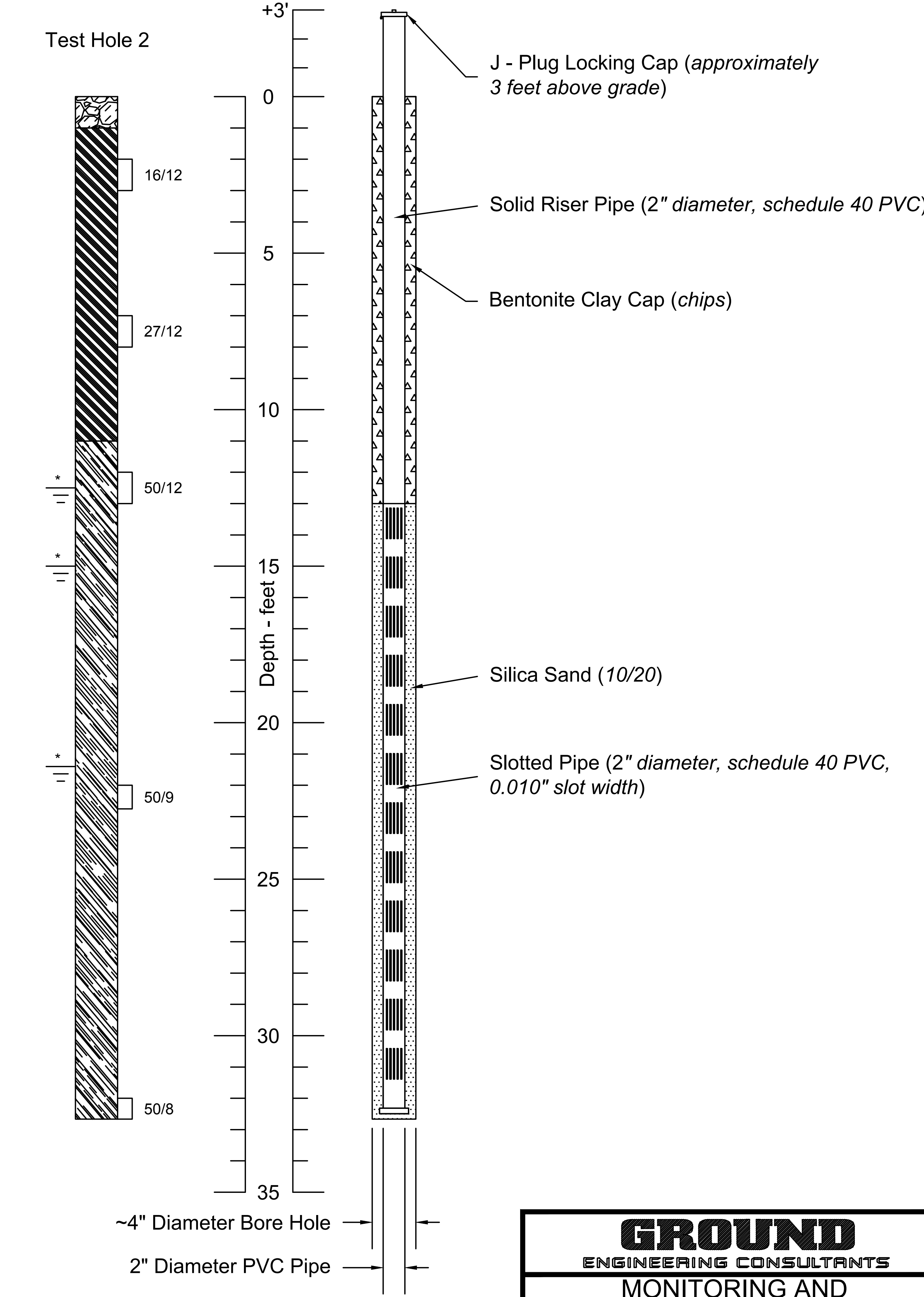



* Refer to Figure 6 for dates of water checks.

**GROUND**
ENGINEERING CONSULTANTS

MONITORING AND OBSERVATION POINT

| | |
|---|---|
| JOB NO.: 19-1235 | FIGURE: 7 |
| CADFILE NAME: 1235MOP01.DWG | |

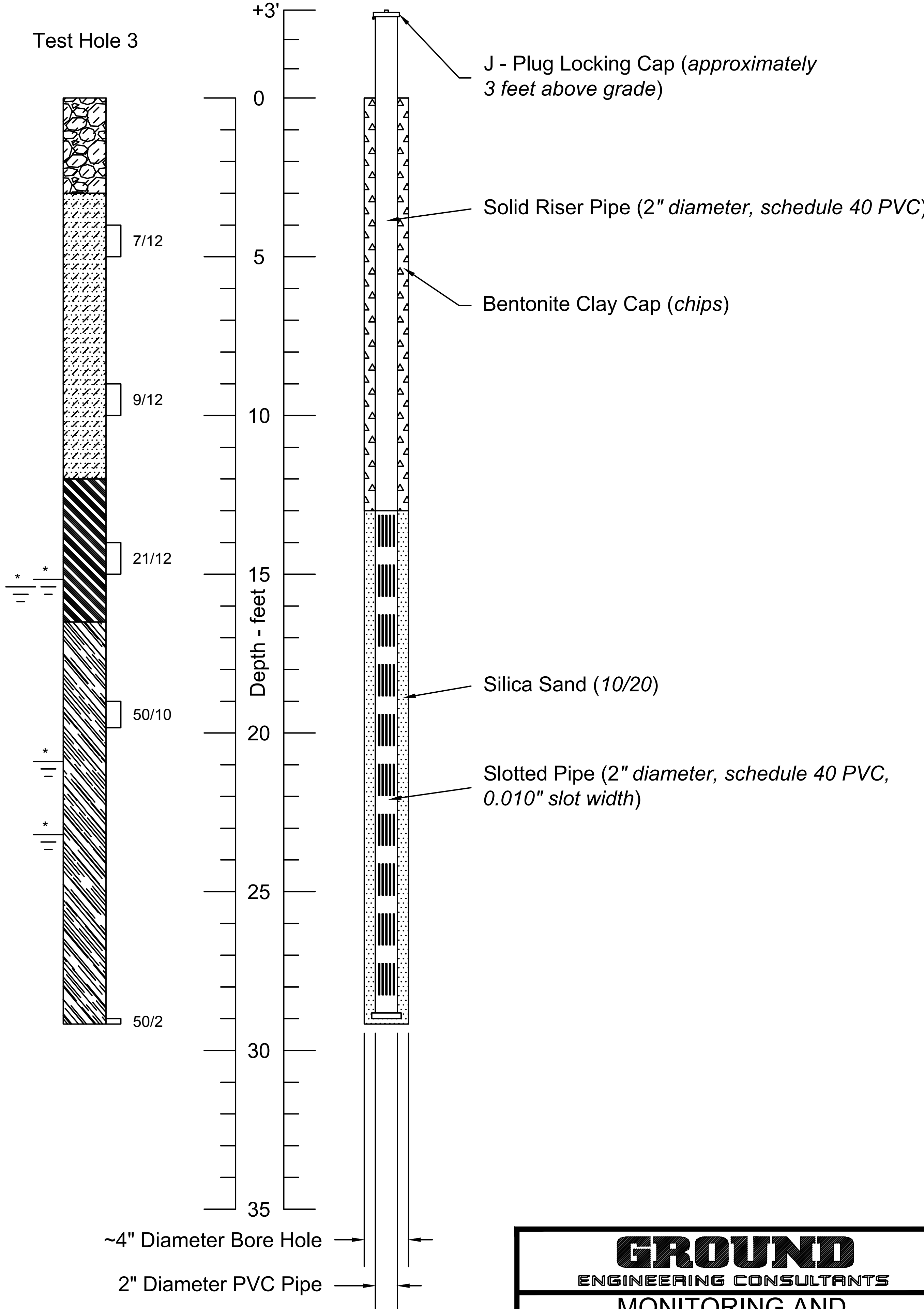



* Refer to Figure 6 for dates of water checks.

GROUND ENGINEERING CONSULTANTS

MONITORING AND OBSERVATION POINT

| | |
|---|---|
| JOB NO.: 19-1235 | FIGURE: 8 |
| CADFILE NAME: 1235MOP02.DWG | |

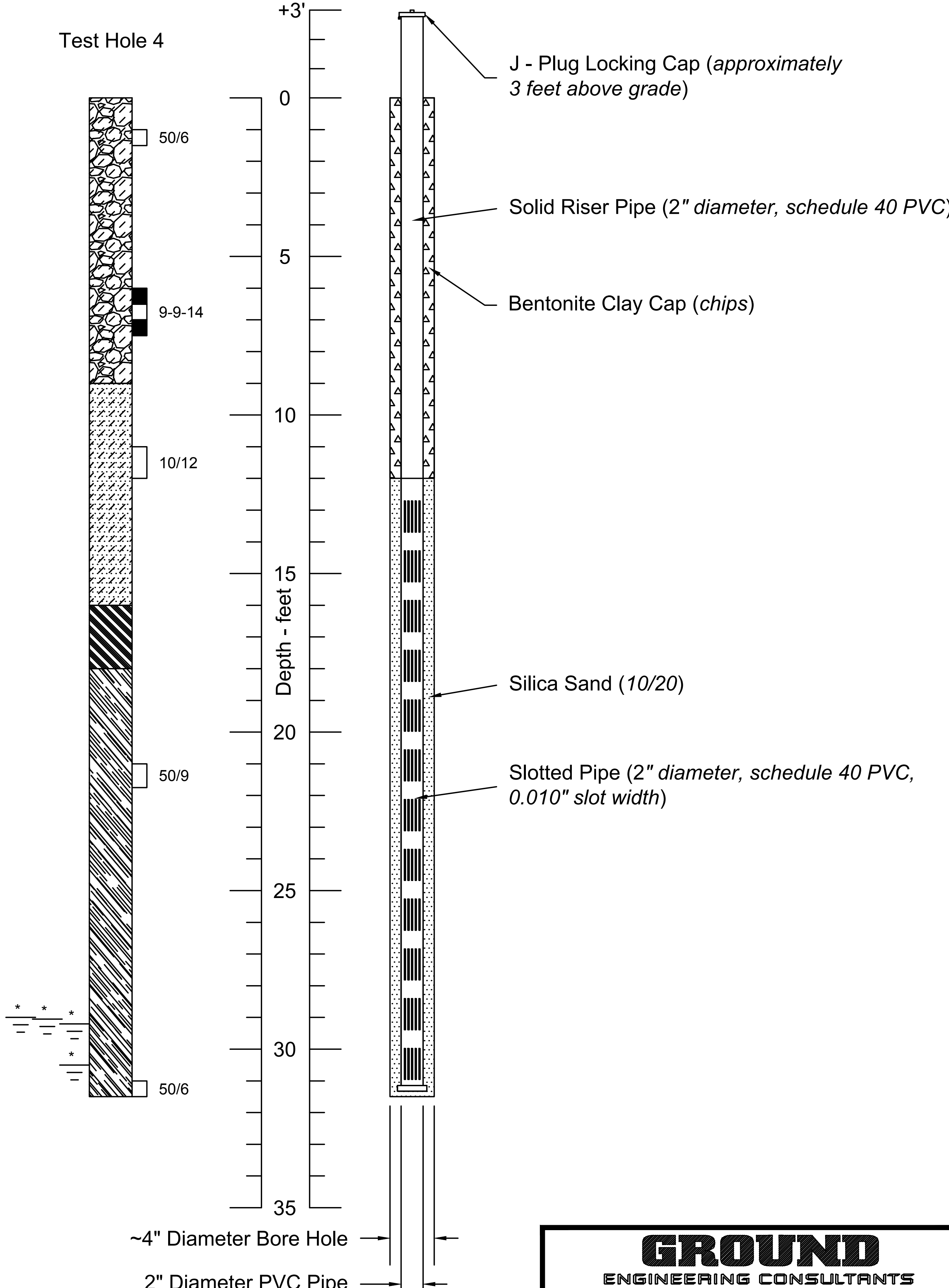



* Refer to Figure 6 for dates of water checks.

GROUND ENGINEERING CONSULTANTS

MONITORING AND OBSERVATION POINT

| JOB NO.: 19-1235 | FIGURE: 9 |
|---|---|
| CADFILE NAME: 1235MOP03.DWG | |

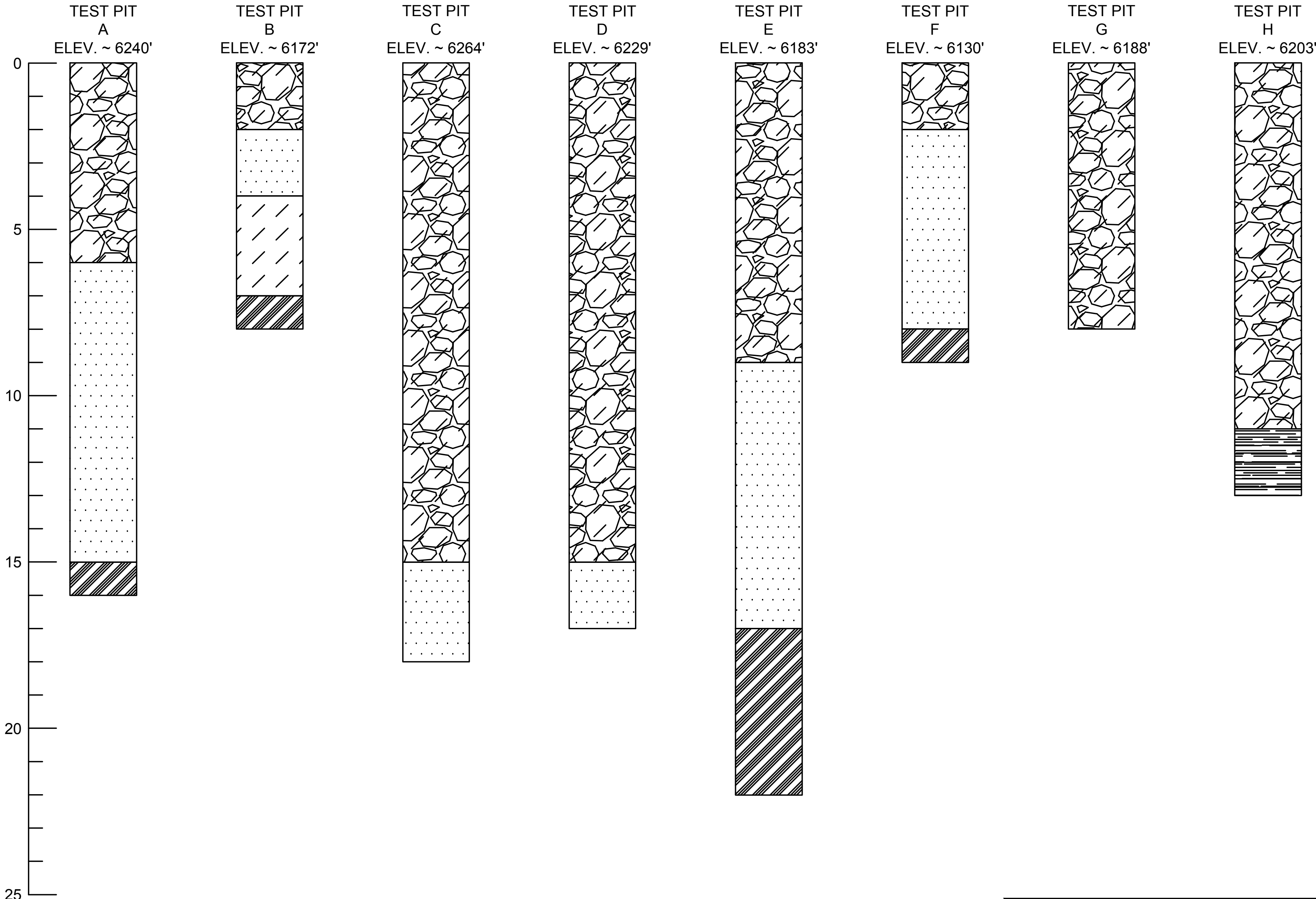



* Refer to Figure 6 for dates of water checks.

GROUND ENGINEERING CONSULTANTS

MONITORING AND OBSERVATION POINT

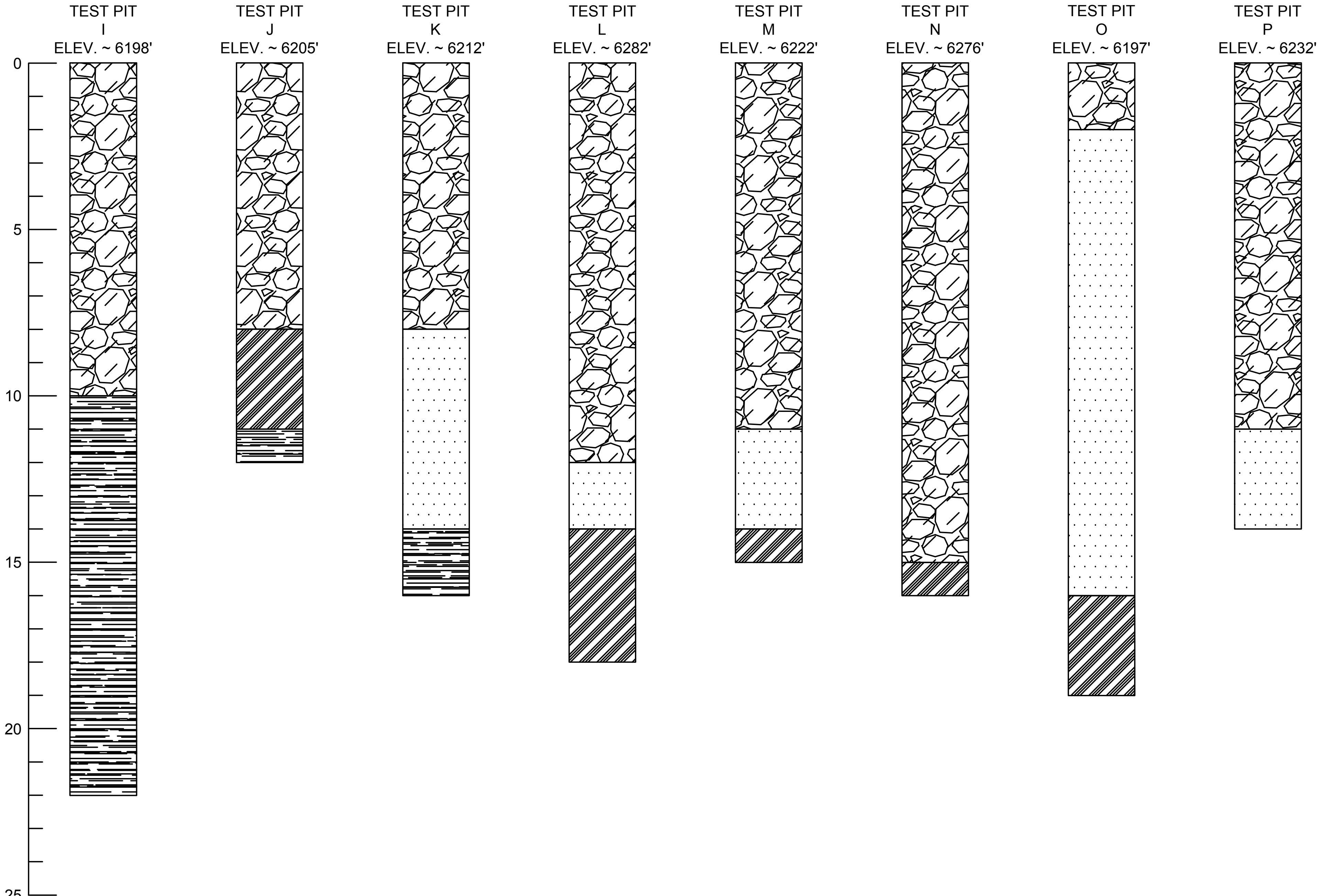
| | |
|---|---|
| JOB NO.: 19-1235 | FIGURE: 10 |
| CADFILE NAME: 1235MOP04.DWG | |




GROUND ENGINEERING CONSULTANTS

# LOGS OF TEST HOLES

| JOB NO.: 19-1235 | FIGURE: 11 |
|---|---|
| CADFILE NAME: 1235LOG.DWG | |




GROUND ENGINEERING CONSULTANTS

LOGS OF TEST HOLES

| | |
|---|---|
| JOB NO.: 19-1235 | FIGURE: 12 |
| CADFILE NAME: 1235LOG.DWG | |

LEGEND:

Topsoil

**Tailings**: Cobble- to sand-sized clasts of shale and igneous rock, with clay. Boulder sized clasts were present locally. They were slightly moist to locally wet, low to moderately plastic, dense, to very dense, and gray-brown to dark gray-brown in color, with local iron oxide staining. In some cases these materials have been displaced since they were placed as fill, and not in others.

**Colluvium**: Sandy clays with gravel. They were moist to very moist, moderately to highly plastic, stiff to very stiff, and red-brown to brown to gray-brown in color. Iron oxide staining and caliche were noted locally.

**Weathered Clay Shale**: Severely weathered, clean to sandy clay shales. They were moist, moderately to highly plastic, moderately hard, and gray-brown in color. Iron oxide staining was noted commonly.

**Clay Shale Bedrock**: Clean to sandy clay shales. They were moist, moderately to highly plastic, hard to very hard, and gray-brown to gray to dark gray in color. Iron oxide staining was noted commonly.

Drive sample, 2-inch I.D. California liner sample

23/12 Drive sample blow count, indicates 23 blows of a 140-pound hammer falling 30 inches were required to drive the sampler 12 inches.

Drive sample, 1-3/8 inch I.D. standard sample

20-25-30 Drive sample blow count, indicates 20, 25, and 30 blows of a 140-pound hammer falling 30 inches were required to drive the sampler 18 inches.

Practical Rig Refusal

* Depth to water level, refer to figure 6 for the date the measurements were taken.

GROUND ENGINEERING CONSULTANTS

LEGEND AND NOTES

| | |
|---|---|
| JOB NO.: 19-1235 | FIGURE: 13 |
| CADFILE NAME: 1235LEG01.DWG | |

NOTES:

1) Test holes were drilled on 08/07/2018 with 4-inch diameter continuous flight augers.

2) Locations of the test holes were surveyed by a representative of the client and shown on the site plan provided.

3) Elevations of the test holes were surveyed by a representative of the client and the logs of the test holes are hung to elevation.

4) The test hole locations and elevations should be considered accurate only to the degree implied by the method used.

5) The lines between materials shown on the test hole logs represent the approximate boundaries between material types and the transitions may be gradual.

6) Groundwater level readings shown on the logs were made at the time and under the conditions indicated. Fluctuations in the water level may occur with time.

7) The material descriptions on this legend are for general classification purposes only. See the full text of this report for descriptions of the site materials and related information.

8) All test holes were immediately backfilled upon completion of drilling, unless otherwise specified in this report.

GROUND ENGINEERING CONSULTANTS

LEGEND AND NOTES

| | |
|---|---|
| JOB NO.: 19-1235 | FIGURE: 14 |
| CADFILE NAME: 1235LEG02.DWG | |

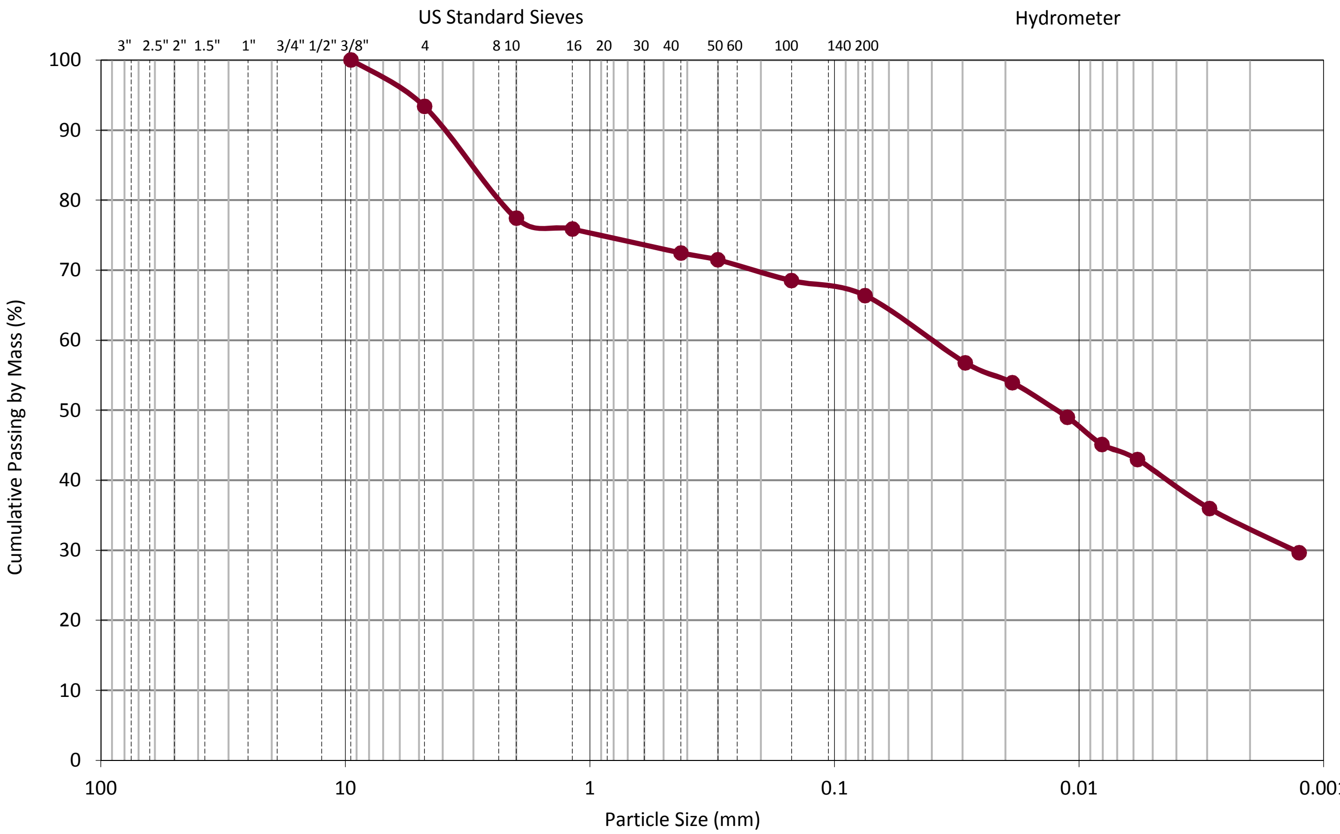
GROUND ENGINEERING

Client: Asphalt Paving Company

Project No.: 19-1235

## Ralston Quarry Slope Stability

## Gradation and Hydrometer (ASTM D422-63[2007])




| Coarse Gradation | | | Fine Gradation | | | Hydrometer | | Grading | |
|---|---|---|---|---|---|---|---|---|---|
| US Standard Sieve | Particle Size (mm) | Passing by Mass (%) | US Standard Sieve | Particle Size (mm) | Passing by Mass (%) | Particle Size (mm) | Passing by Mass (%) | Coefficient | Value |
| 6 in | 150 | - | No. 4 | 4.75 | 93 | 0.029 | 57 | D90 | 3.958 |
| 5 in | 125 | - | No. 8 | 2.36 | - | 0.019 | 54 | D85 | 3.018 |
| 4 in | 100 | - | No. 10 | 2.00 | 77 | 0.011 | 49 | D80 | 2.301 |
| 3 in | 75 | - | No. 16 | 1.18 | 76 | 0.008 | 45 | D60 | - |
| 2.5 in | 63 | - | No. 20 | 0.85 | - | 0.006 | 43 | D50 | - |
| 2 in | 50 | - | No. 30 | 0.60 | - | 0.003 | 36 | D40 | - |
| 1.5 in | 37.5 | - | No. 40 | 0.425 | 72 | 0.001 | 30 | D30 | - |
| 1 in | 25.0 | - | No. 50 | 0.300 | 71 | - | - | D15 | - |
| 3/4 in | 19.0 | - | No. 60 | 0.250 | - | - | - | D10 | - |
| 1/2 in | 12.5 | - | No. 100 | 0.150 | 68 | - | - | D05 | - |
| 3/8 in | 9.5 | 100 | No. 140 | 0.106 | - | - | - | Cu | - |
| No. 4 | 4.75 | 93 | No. 200 | 0.075 | 66.4 | - | - | Cc | - |

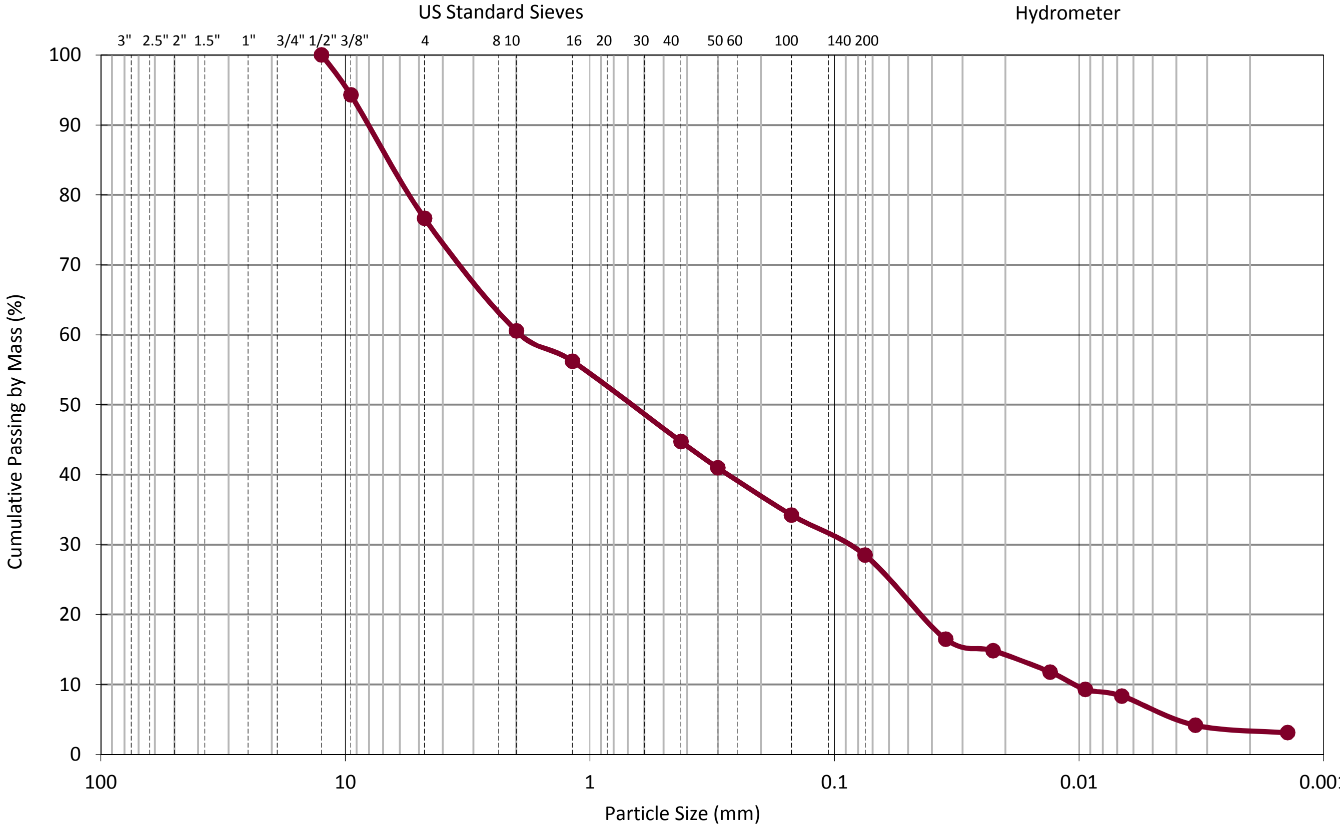
*Location:* BULK 1
*Description:* Sandy CLAY

| | | | |
|---|---|---|---|
| *Classification:* | s(CL)/A-7-6(16) | *Gravel (%):* | 7 |
| *Liquid Limit:* | 50 | *Sand (%):* | 27 |
| *Plasticity Index:* | 26 | *Silt/Clay (%):* | 66 |
| *Activity:* | 0.8 | *< .002 mm (%):* | 33 |

Results apply only to the specific items and locations referenced and at the time of testing. For the hydrometer portion of the test, a composite temperature correction and meniscus correction were applied to each reading. This report should not be reproduced, except in full, without the written permission of GROUND Engineering Consultants, Inc.

## Ralston Quarry Slope Stability

## Gradation and Hydrometer (ASTM D422-63[2007])




| Coarse Gradation | | | Fine Gradation | | | Hydrometer | | Grading | |
|---|---|---|---|---|---|---|---|---|---|
| US Standard Sieve | Particle Size (mm) | Passing by Mass (%) | US Standard Sieve | Particle Size (mm) | Passing by Mass (%) | Particle Size (mm) | Passing by Mass (%) | Coefficient | Value |
| 6 in | 150 | - | No. 4 | 4.75 | 77 | 0.035 | 16 | D90 | 8.022 |
| 5 in | 125 | - | No. 8 | 2.36 | - | 0.022 | 15 | D85 | 6.592 |
| 4 in | 100 | - | No. 10 | 2.00 | 61 | 0.013 | 12 | D80 | 5.416 |
| 3 in | 75 | - | No. 16 | 1.18 | 56 | 0.009 | 9 | D60 | 1.872 |
| 2.5 in | 63 | - | No. 20 | 0.85 | - | 0.007 | 8 | D50 | 0.679 |
| 2 in | 50 | - | No. 30 | 0.60 | - | 0.003 | 4 | D40 | 0.272 |
| 1.5 in | 37.5 | - | No. 40 | 0.425 | 45 | 0.001 | 3 | D30 | 0.090 |
| 1 in | 25.0 | - | No. 50 | 0.300 | 41 | - | - | D15 | - |
| 3/4 in | 19.0 | - | No. 60 | 0.250 | - | - | - | D10 | - |
| 1/2 in | 12.5 | 100 | No. 100 | 0.150 | 34 | - | - | D05 | - |
| 3/8 in | 9.5 | 94 | No. 140 | 0.106 | - | - | - | Cu | - |
| No. 4 | 4.75 | 77 | No. 200 | 0.075 | 28.5 | - | - | Cc | - |

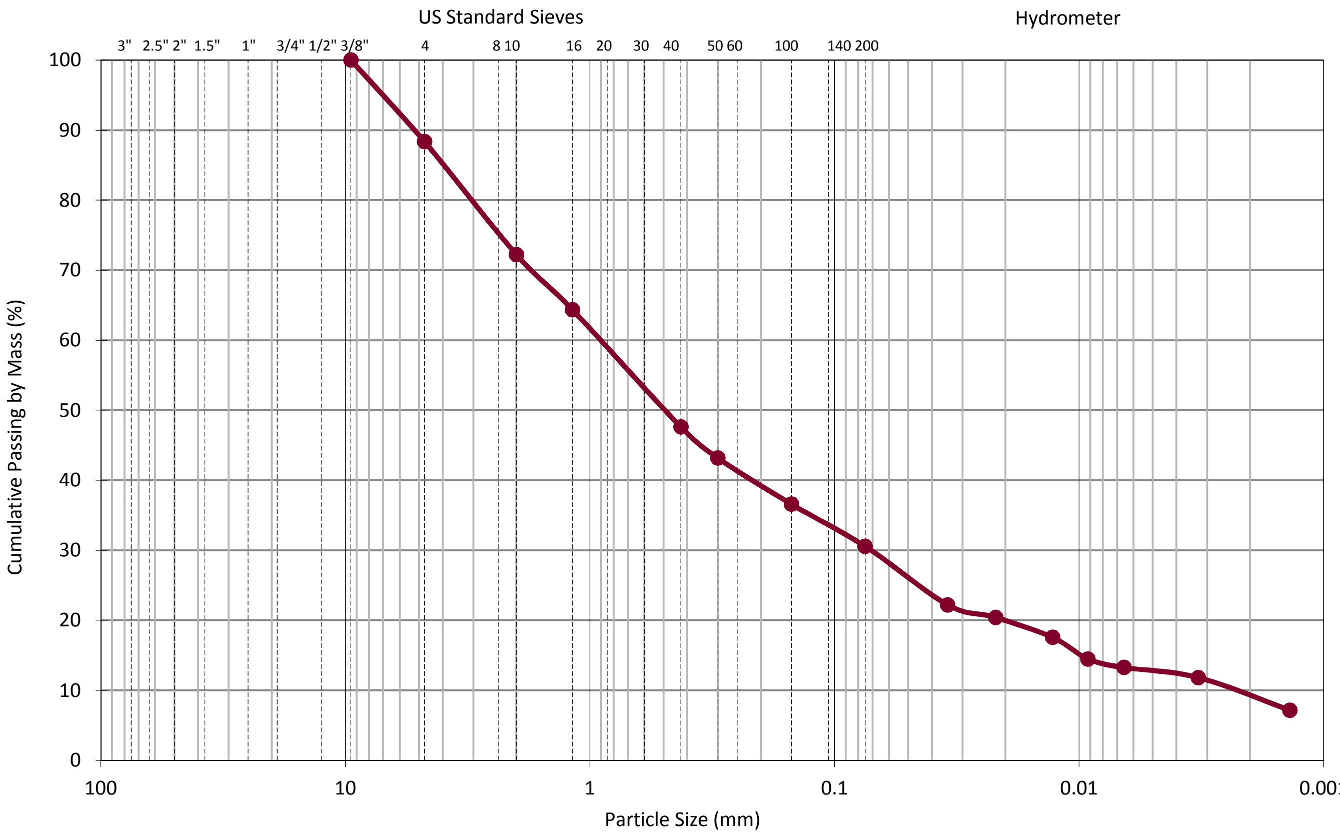
*Location:* BULK 2
*Description:* Clayey SAND with gravel

| | | | |
|---|---|---|---|
| *Classification:* | (SC)g/A-2-6(1) | *Gravel (%):* | 23 |
| *Liquid Limit:* | 31 | *Sand (%):* | 49 |
| *Plasticity Index:* | 14 | *Silt/Clay (%):* | 28 |
| *Activity:* | 4.0 | *< .002 mm (%):* | 4 |

Results apply only to the specific items and locations referenced and at the time of testing. For the hydrometer portion of the test, a composite temperature correction and meniscus correction were applied to each reading. This report should not be reproduced, except in full, without the written permission of GROUND Engineering Consultants, Inc.

## Ralston Quarry Slope Stability

## Gradation and Hydrometer (ASTM D422-63[2007])




| Coarse Gradation | | | Fine Gradation | | | Hydrometer | | Grading | |
|---|---|---|---|---|---|---|---|---|---|
| US Standard Sieve | Particle Size (mm) | Passing by Mass (%) | US Standard Sieve | Particle Size (mm) | Passing by Mass (%) | Particle Size (mm) | Passing by Mass (%) | Coefficient | Value |
| 6 in | 150 | - | No. 4 | 4.75 | 88 | 0.034 | 22 | D90 | 5.240 |
| 5 in | 125 | - | No. 8 | 2.36 | - | 0.022 | 20 | D85 | 3.968 |
| 4 in | 100 | - | No. 10 | 2.00 | 72 | 0.013 | 18 | D80 | 3.034 |
| 3 in | 75 | - | No. 16 | 1.18 | 64 | 0.009 | 14 | D60 | 0.905 |
| 2.5 in | 63 | - | No. 20 | 0.85 | - | 0.007 | 13 | D50 | 0.492 |
| 2 in | 50 | - | No. 30 | 0.60 | - | 0.003 | 12 | D40 | 0.215 |
| 1.5 in | 37.5 | - | No. 40 | 0.425 | 48 | 0.001 | 7 | D30 | - |
| 1 in | 25.0 | - | No. 50 | 0.300 | 43 | - | - | D15 | - |
| 3/4 in | 19.0 | - | No. 60 | 0.250 | - | - | - | D10 | - |
| 1/2 in | 12.5 | - | No. 100 | 0.150 | 37 | - | - | D05 | - |
| 3/8 in | 9.5 | 100 | No. 140 | 0.106 | - | - | - | Cu | - |
| No. 4 | 4.75 | 88 | No. 200 | 0.075 | 30.6 | - | - | Cc | - |

*Location:* HD 1
*Description:* Clayey SAND

*Classification:* SC/A-2-6(1)
*Liquid Limit:* 35
*Plasticity Index:* 14
*Activity:* 1.5

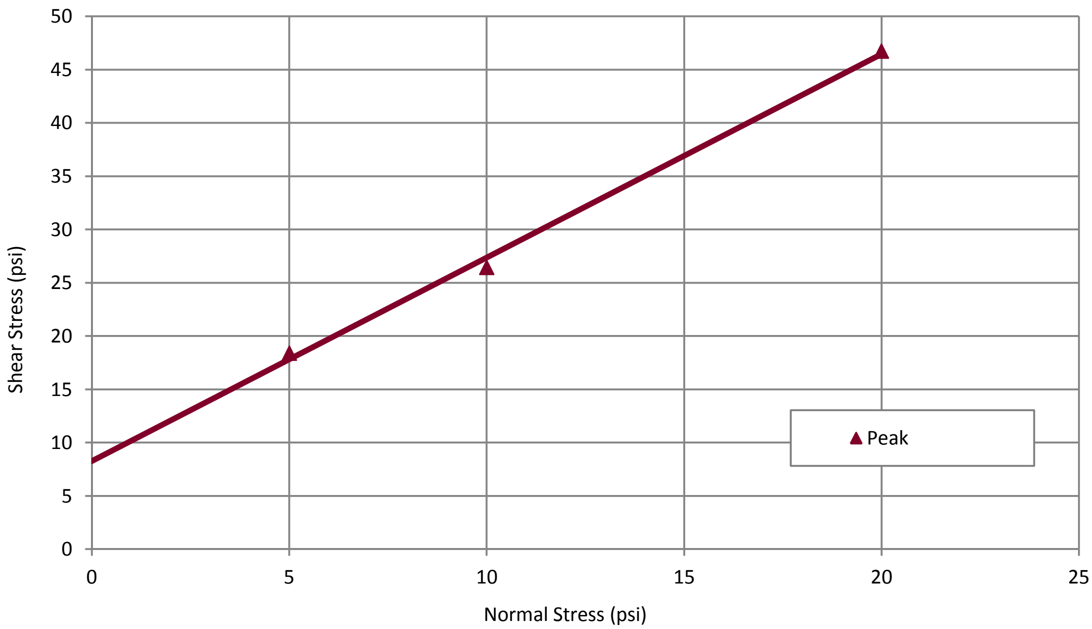
*Gravel (%):* 12
*Sand (%):* 57
*Silt/Clay (%):* 31
*< .002 mm (%):* 9

Client: Asphalt Paving Company

Project No.: 19-1235

## Ralston Quarry Slope Stability

## Direct Shear




| Tested By | Shear Date | Device | Prep. | Specific Gravity |
|---|---|---|---|---|
| DS | 8/15/2018 | Geotest S2215A | Liner | 2.65 Estimated |

| **Point Normal Stress (psi)** | **5.0** | | **10.0** | | **20.0** | |
|---|---|---|---|---|---|---|
| **Point Properties** | **Initial** | **Pre-shear** | **Initial** | **Pre-shear** | **Initial** | **Pre-shear** |
| Diameter (in) | 1.94 | | 1.94 | | 1.94 | |
| Height (in) | 1.00 | 0.99 | 1.00 | 0.99 | 1.00 | 0.99 |
| Wet Density (pcf) | 113.0 | 114.2 | 121.8 | 122.6 | 126.9 | 128.6 |
| Moisture (%) | 10.1 | 10.1 | 9.7 | 9.7 | 9.3 | 9.3 |
| Dry Density (pcf) | 102.6 | 103.7 | 111.0 | 111.8 | 116.2 | 117.7 |
| Void Ratio | 0.607 | 0.591 | 0.485 | 0.475 | 0.420 | 0.401 |
| Saturation (%) | 44.2 | 45.5 | 52.9 | 54.0 | 58.5 | 61.3 |
| Shear Rate (in/min) | 0.0025 | | 0.0025 | | 0.0025 | |
| **Point Results** | **Peak** | | **Peak** | | **Peak** | |
| Shear Stress (psi) | 18.4 | | 26.5 | | 46.8 | |
| Horizontal Strain (%) | 4.6 | | 6.2 | | 7.2 | |

| **Test Results** | **Peak** |
|---|---|
| Friction Angle (deg) | **62** |
| Cohesion (psi) | **8** |

*Sample:* TH-1 at 8 ft

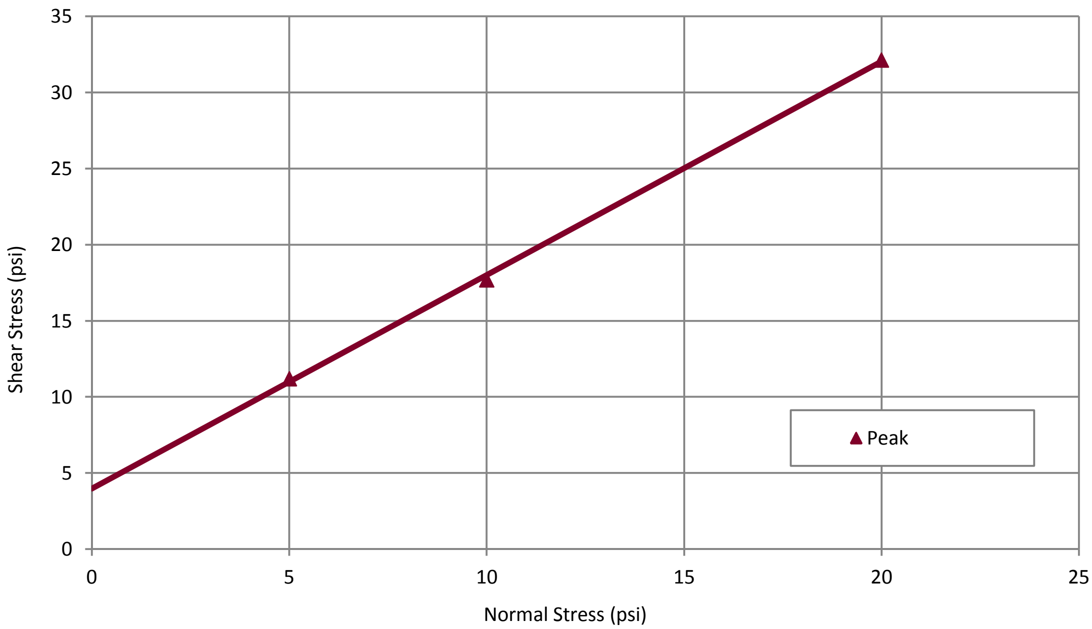
*Description:*

Figure 18

Results apply only to the specific items and locations referenced and at the time of testing. Testing was performed in general accordance with ASTM D3080 / AASHTO T236. This report should not be reproduced, except in full, without the written permission of GROUND Engineering Consultants, Inc.

# Ralston Quarry Slope Stability

## Direct Shear




| Tested By | Shear Date | Device | Prep. | Specific Gravity |
|---|---|---|---|---|
| DS | 8/15/2018 | Geotest S2215A | Liner | 2.65 Estimated |

| Point Normal Stress (psi) | 5.0 | | 10.0 | | 20.0 | |
|---|---|---|---|---|---|---|
| **Point Properties** | **Initial** | **Pre-shear** | **Initial** | **Pre-shear** | **Initial** | **Pre-shear** |
| Diameter (in) | 1.94 | | 1.94 | | 1.94 | |
| Height (in) | 1.00 | 0.98 | 1.00 | 0.98 | 1.00 | 0.97 |
| Wet Density (pcf) | 107.7 | 110.1 | 112.0 | 113.9 | 119.9 | 123.2 |
| Moisture (%) | 24.4 | 24.4 | 24.1 | 24.1 | 20.2 | 20.2 |
| Dry Density (pcf) | 86.6 | 88.5 | 90.2 | 91.8 | 99.7 | 102.5 |
| Void Ratio | 0.904 | 0.863 | 0.828 | 0.797 | 0.654 | 0.609 |
| Saturation (%) | 71.5 | 74.9 | 77.2 | 80.2 | 81.8 | 87.8 |
| Shear Rate (in/min) | 0.0025 | | 0.0025 | | 0.0025 | |
| **Point Results** | **Peak** | | **Peak** | | **Peak** | |
| Shear Stress (psi) | 11.2 | | 17.7 | | 32.1 | |
| Horizontal Strain (%) | 8.2 | | 9.8 | | 11.3 | |

| Test Results | Peak |
|---|---|
| Friction Angle (deg) | **55** |
| Cohesion (psi) | **4** |

*Sample:* TH-3 at 4 ft

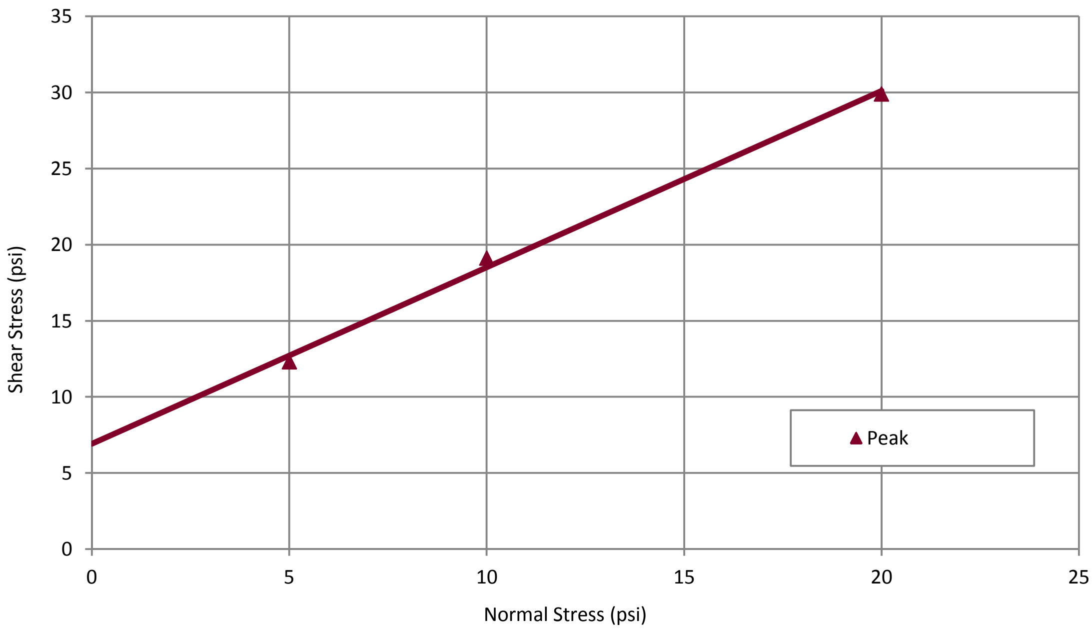
*Description:*

Figure 19

Results apply only to the specific items and locations referenced and at the time of testing. Testing was performed in general accordance with ASTM D3080 / AASHTO T236. This report should not be reproduced, except in full, without the written permission of GROUND Engineering Consultants, Inc.

## Ralston Quarry Slope Stability

## Direct Shear




| Tested By | Shear Date | Device | Prep. | Specific Gravity |
|---|---|---|---|---|
| DS | 8/16/2018 | Geotest S2215A | Liner | 2.65 Estimated |

| **Point Normal Stress (psi)** | **5.0** | | **10.0** | | **20.0** | |
|---|---|---|---|---|---|---|
| **Point Properties** | **Initial** | **Pre-shear** | **Initial** | **Pre-shear** | **Initial** | **Pre-shear** |
| Diameter (in) | 1.94 | | 1.94 | | 1.94 | |
| Height (in) | 1.00 | 0.99 | 1.00 | 0.99 | 1.00 | 0.96 |
| Wet Density (pcf) | 112.4 | 113.6 | 113.9 | 115.4 | 114.1 | 118.8 |
| Moisture (%) | 24.3 | 24.3 | 18.3 | 18.3 | 20.9 | 20.9 |
| Dry Density (pcf) | 90.4 | 91.4 | 96.3 | 97.5 | 94.3 | 98.2 |
| Void Ratio | 0.824 | 0.805 | 0.712 | 0.691 | 0.749 | 0.679 |
| Saturation (%) | 78.1 | 79.9 | 68.0 | 70.1 | 74.0 | 81.6 |
| Shear Rate (in/min) | 0.0025 | | 0.0025 | | 0.0025 | |
| **Point Results** | **Peak** | | **Peak** | | **Peak** | |
| Shear Stress (psi) | 12.3 | | 19.1 | | 29.9 | |
| Horizontal Strain (%) | 6.2 | | 7.7 | | 9.3 | |

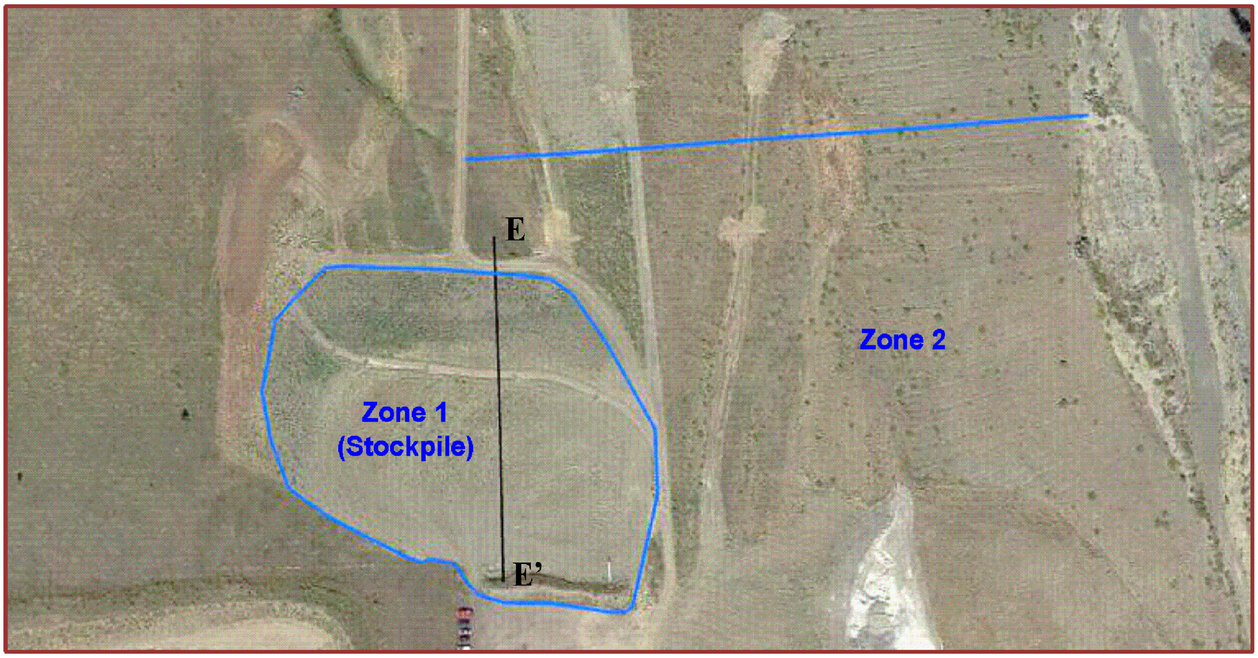
| **Test Results** | **Peak** |
|---|---|
| Friction Angle (deg) | **49** |
| Cohesion (psi) | **7** |

*Sample:* TH-4 at 11 ft

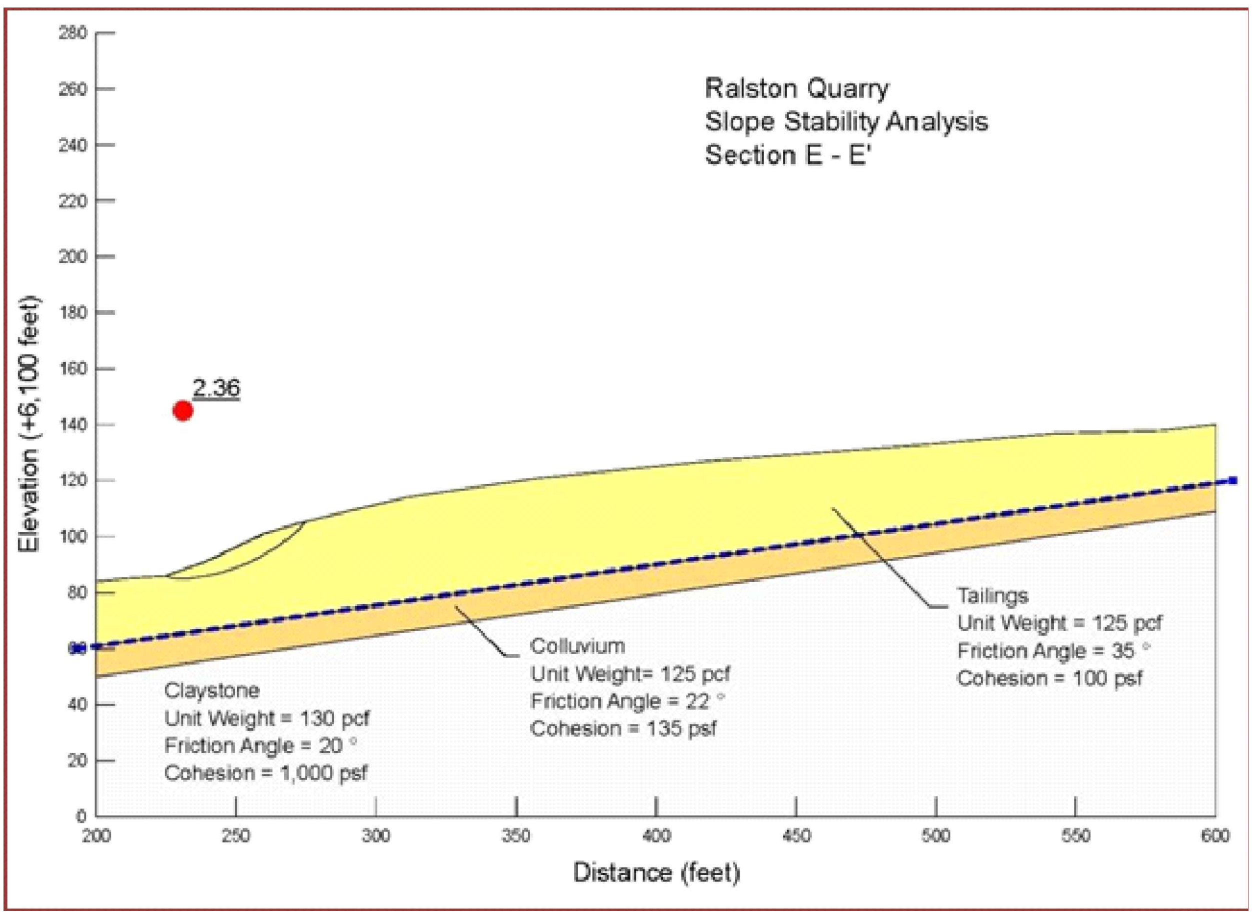
*Description:*

**Figure 20**

Results apply only to the specific items and locations referenced and at the time of testing. Testing was performed in general accordance with ASTM D3080 / AASHTO T236. This report should not be reproduced, except in full, without the written permission of GROUND Engineering Consultants, Inc.



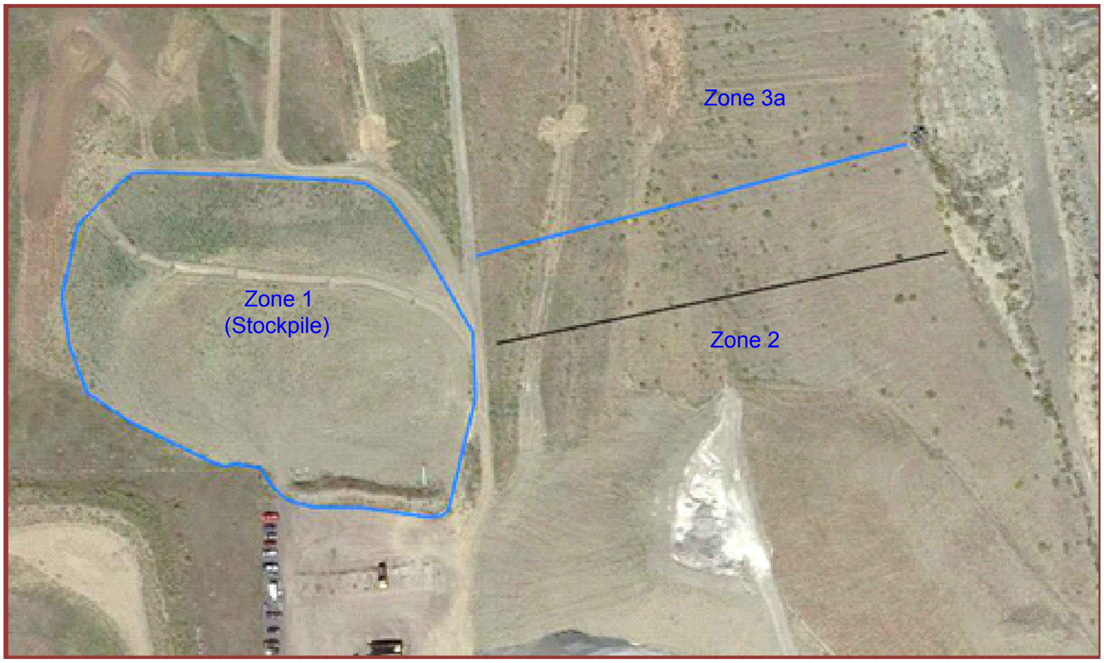

*Zone 1 and Section E – E'*

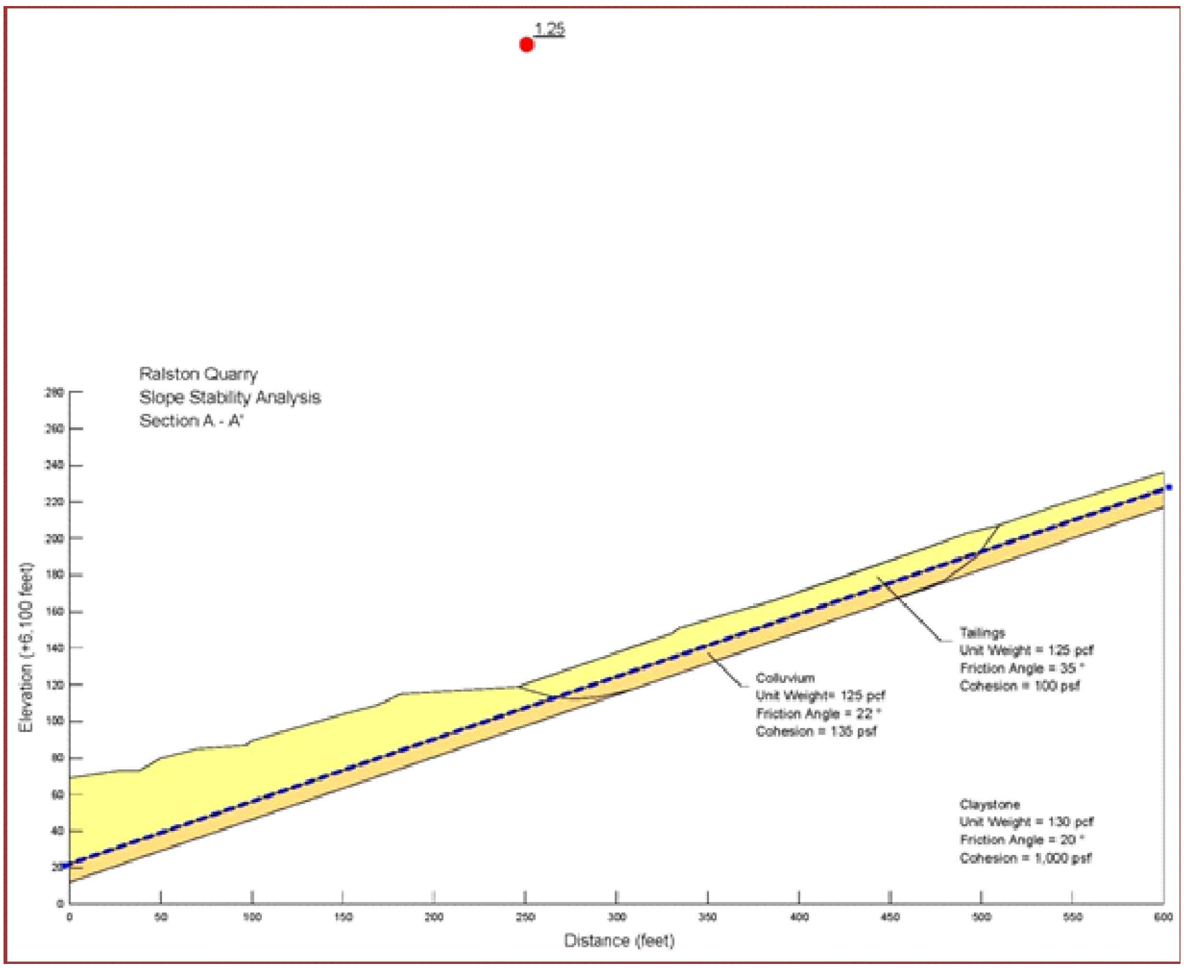



**GROUND**
ENGINEERING CONSULTANTS

SLOPE STABILITY ANALYSIS

| JOB NO.: 19-1235 | FIGURE: 21 |
| --- | --- |
| CADFILE NAME: 1235SSA.DWG | |

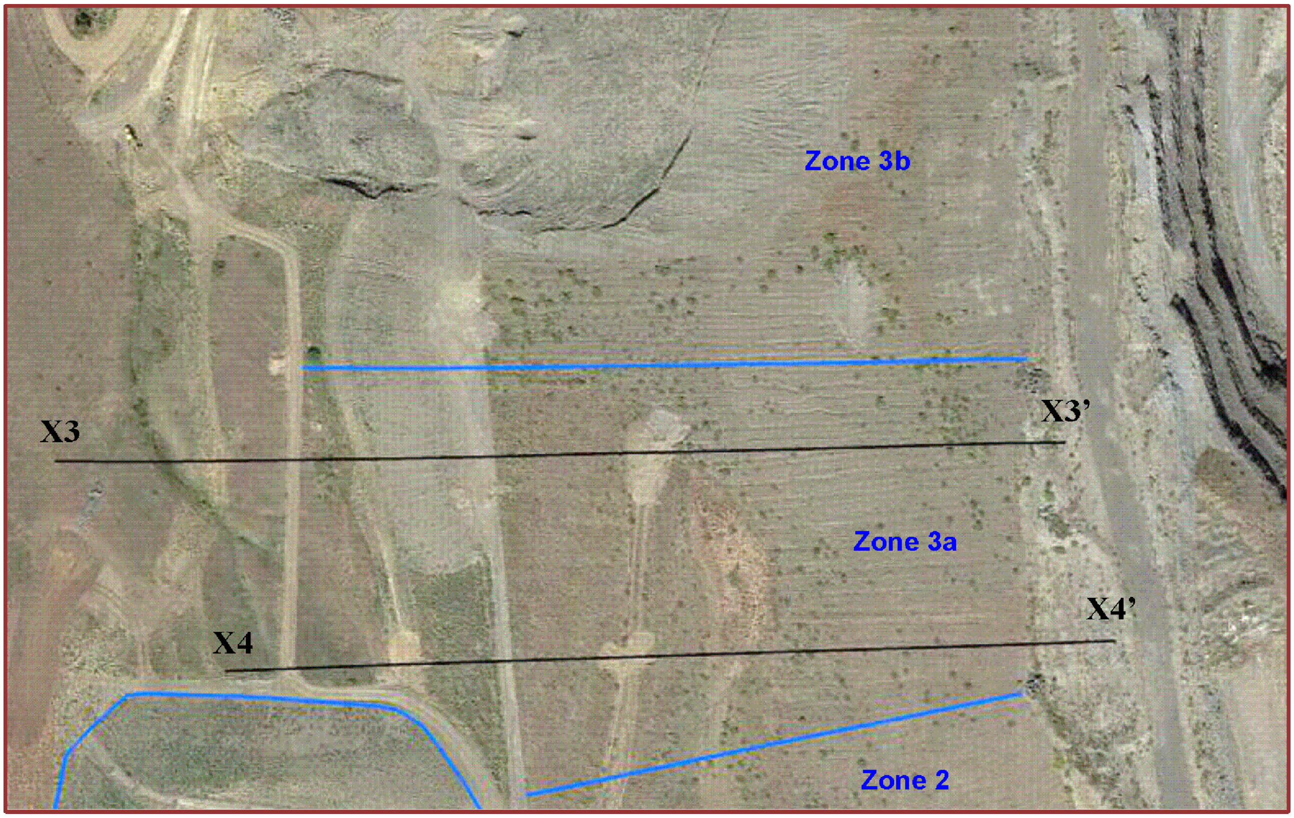


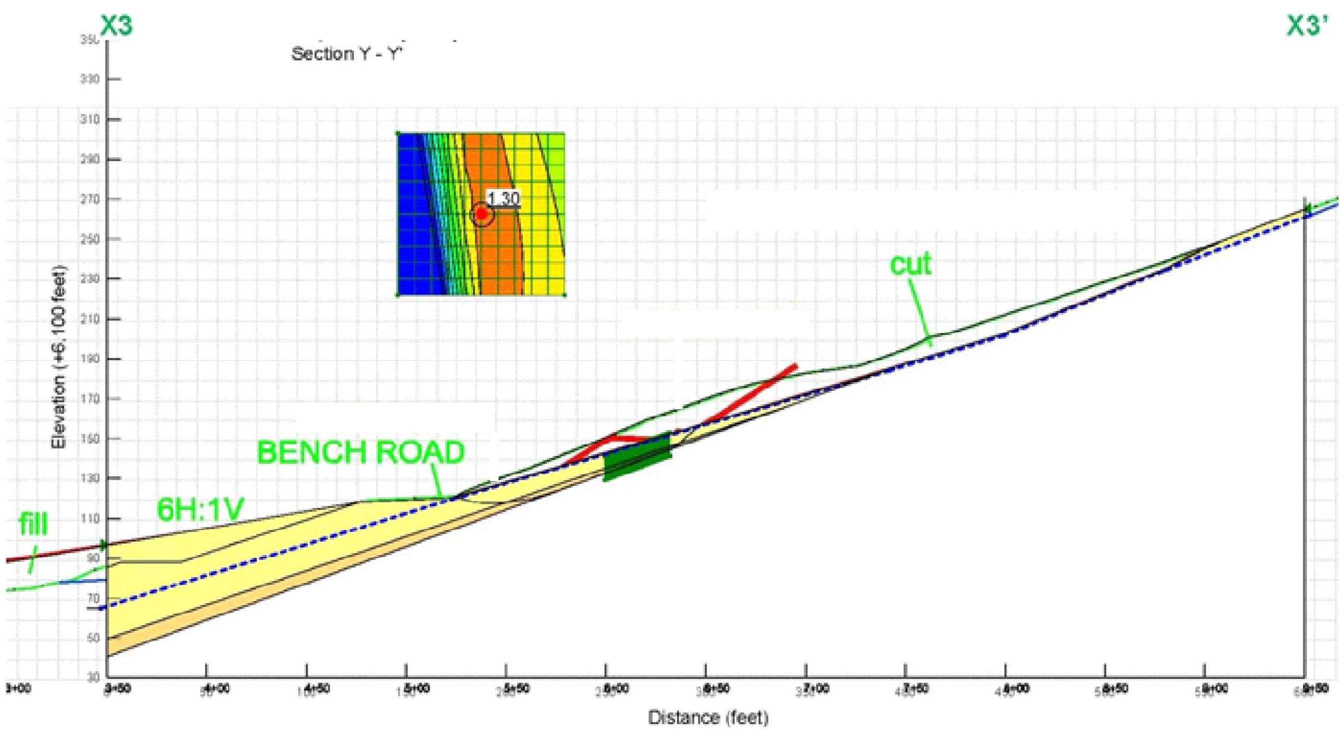




GROUND
ENGINEERING CONSULTANTS

SLOPE STABILITY ANALYSIS

| | |
|---|---|
| JOB NO.: 19-1235 | FIGURE: 22 |
| CADFILE NAME: 1235SSA.DWG | |

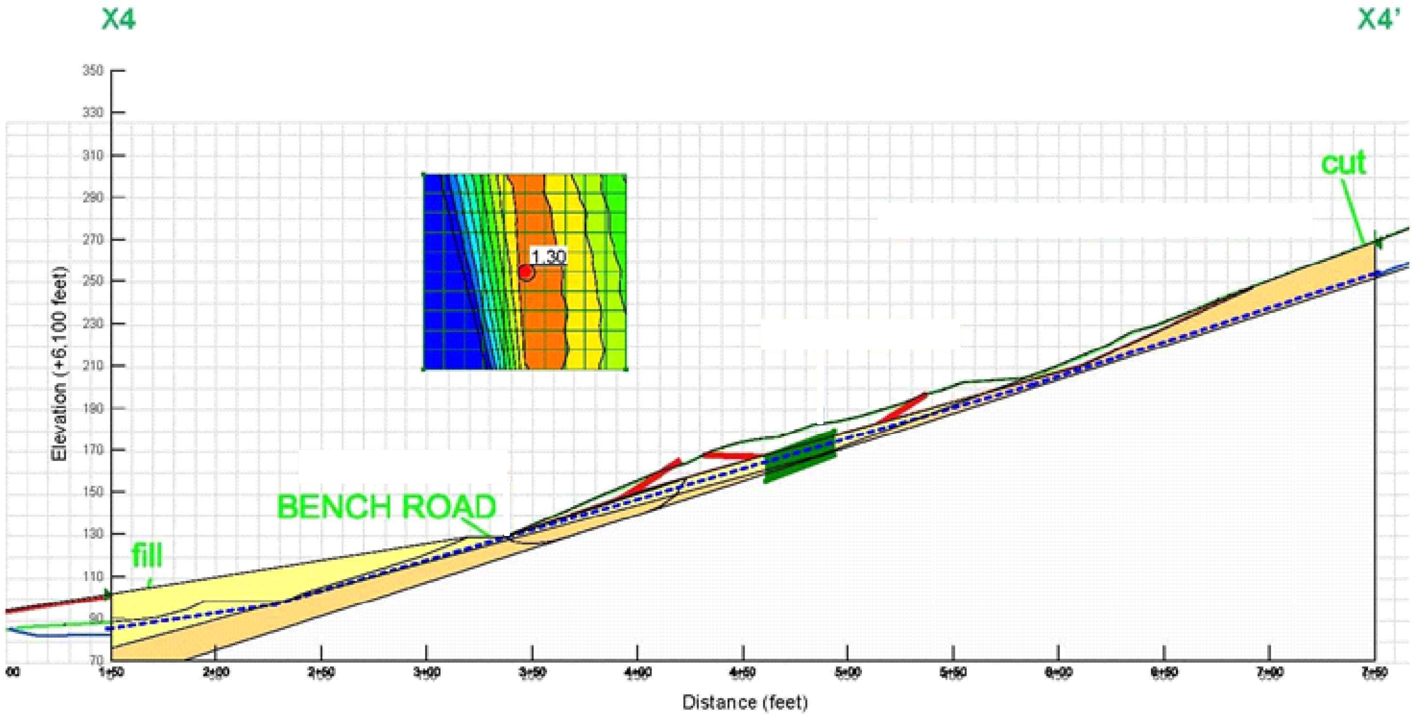



GROUND ENGINEERING CONSULTANTS

SLOPE STABILITY ANALYSIS

| JOB NO.: 19-1235 | FIGURE: 23A |
|---|---|
| CADFILE NAME: 1235SSA.DWG | |

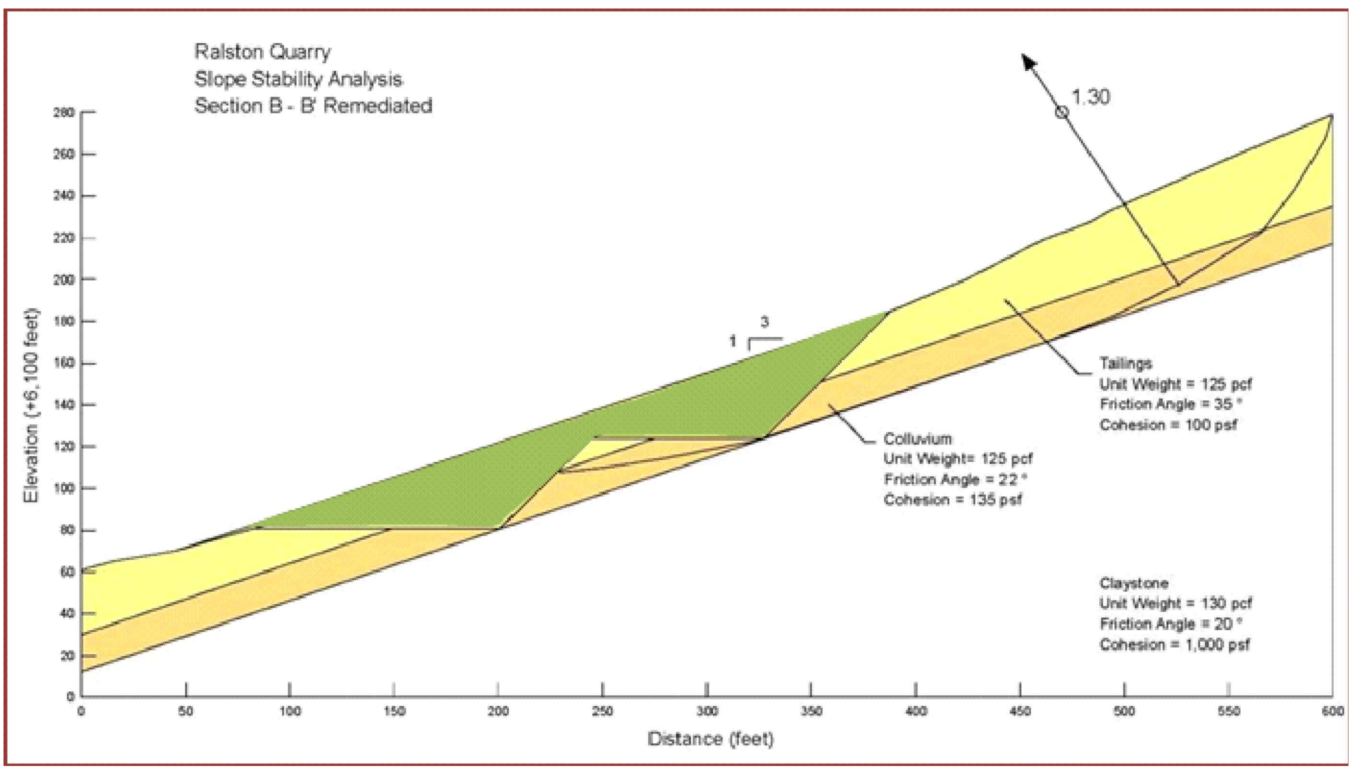



GROUND
ENGINEERING CONSULTANTS

SLOPE STABILITY ANALYSIS

| | |
|---|---|
| JOB NO.: 19-1235 | FIGURE: 23B |
| CADFILE NAME: 1235SSA.DWG | |

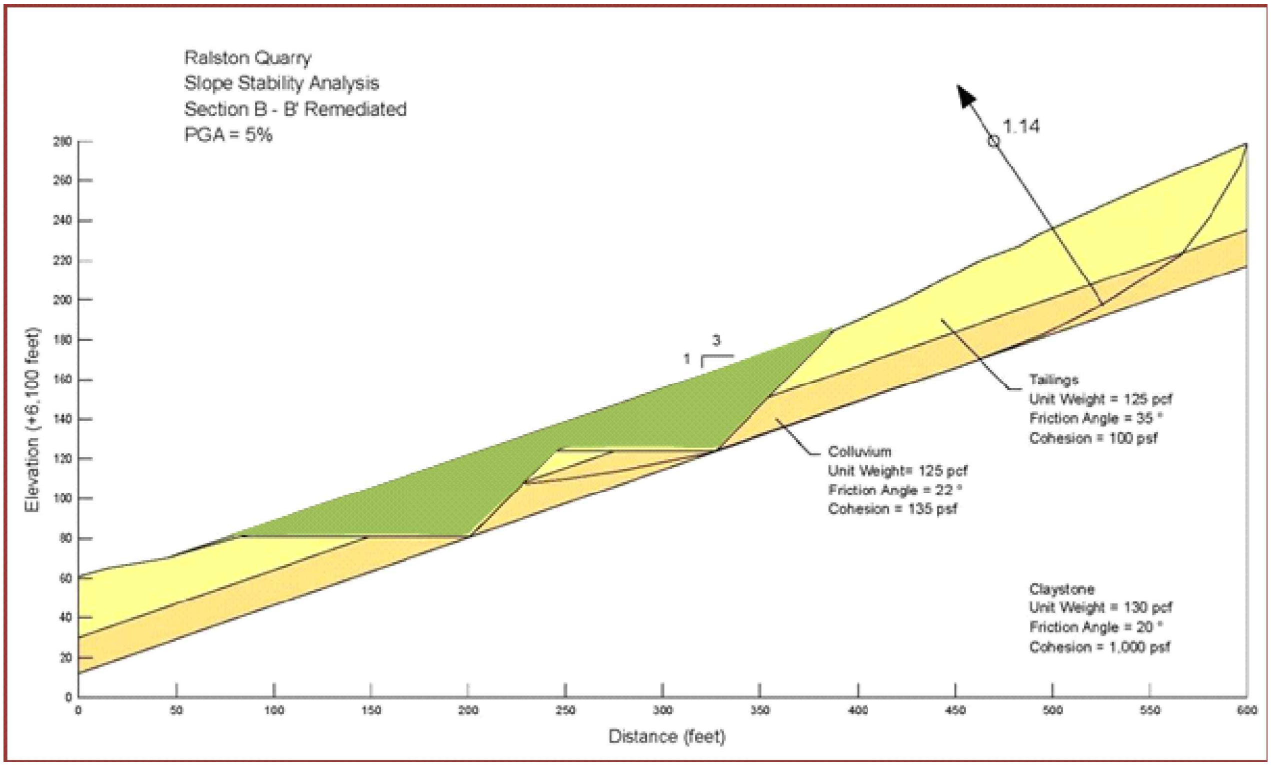

*Zone 3b and Section B – B'*




**GROUND**
ENGINEERING CONSULTANTS

SLOPE STABILITY ANALYSIS

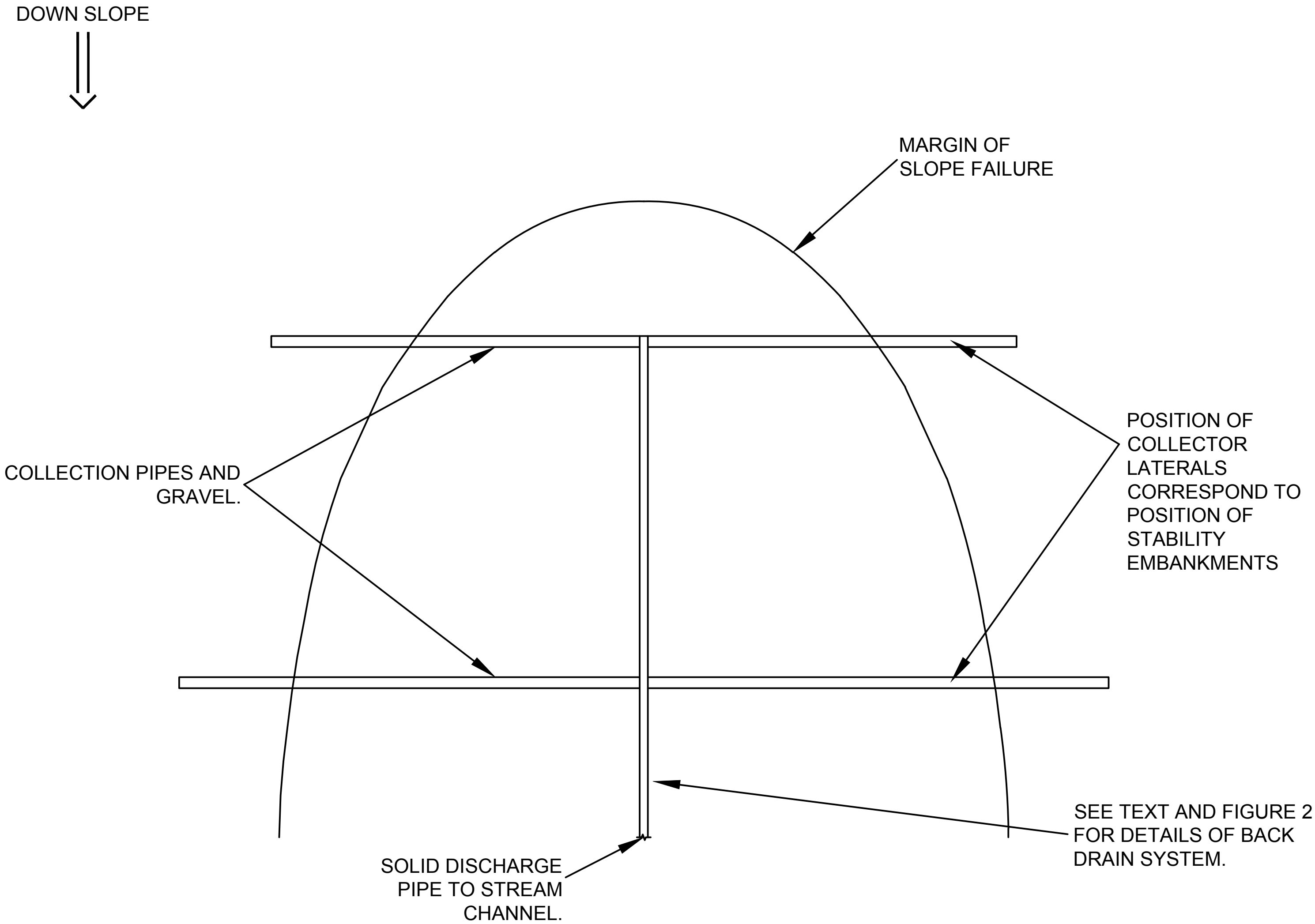
| | |
|---|---|
| JOB NO.: 19-1235 | FIGURE: 24A |
| CADFILE NAME: 1235SSA.DWG | |




GROUND ENGINEERING CONSULTANTS

SLOPE STABILITY ANALYSIS

| | |
|---|---|
| JOB NO.: 19-1235 | FIGURE: 24B |
| CADFILE NAME: 1235SSA.DWG | |



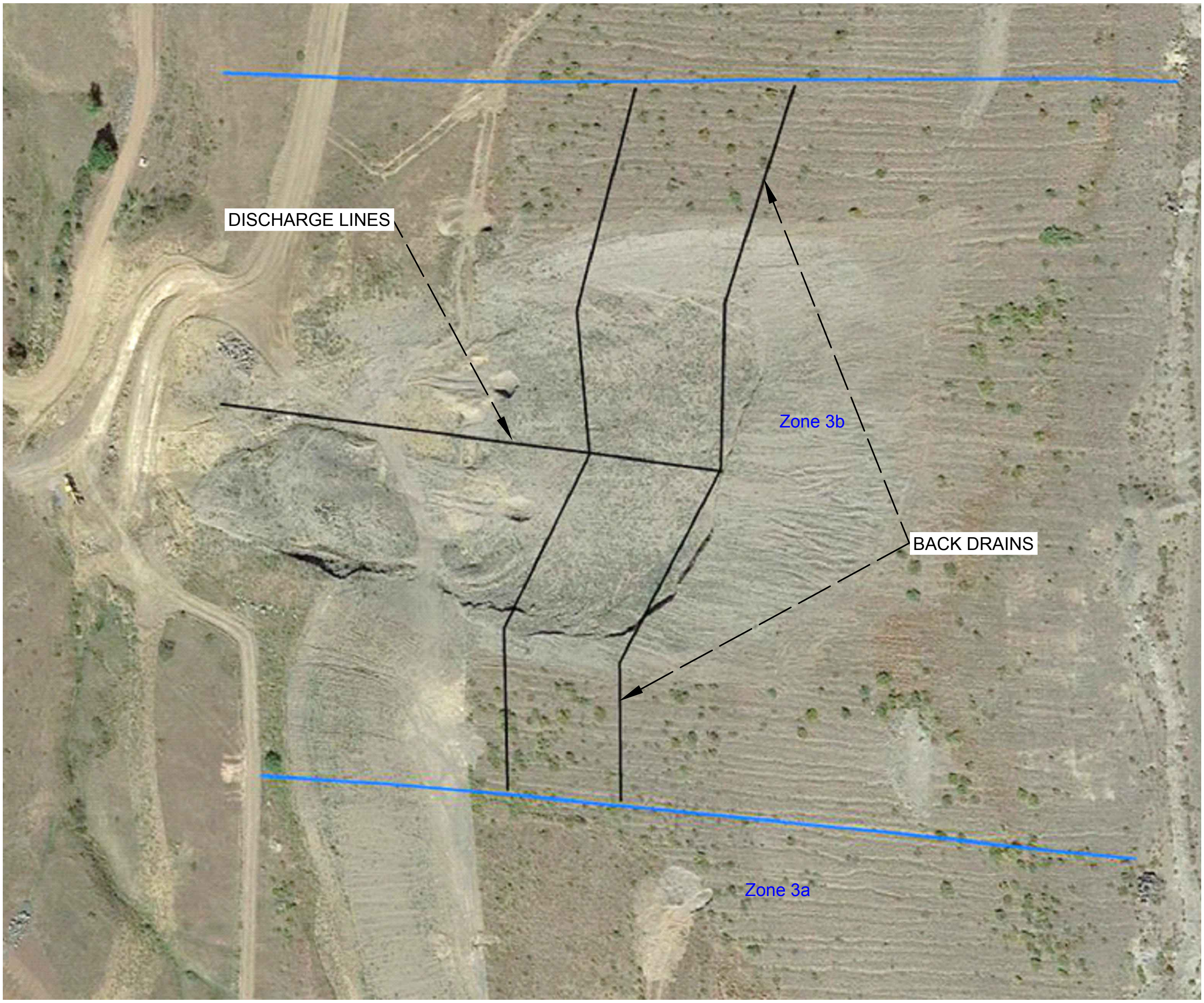

SEE TEXT FOR ADDITIONAL INFORMATION.

(Not to Scale)

GROUND ENGINEERING CONSULTANTS

GENERAL BACK DRAIN LAYOUT

| | |
|---|---|
| JOB NO.: 19-1235 | FIGURE: 25 |
| CADFILE NAME: 1235DRAIN01.DWG | |

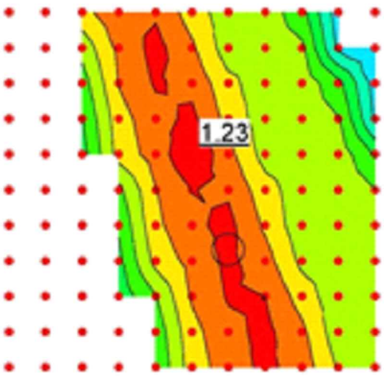


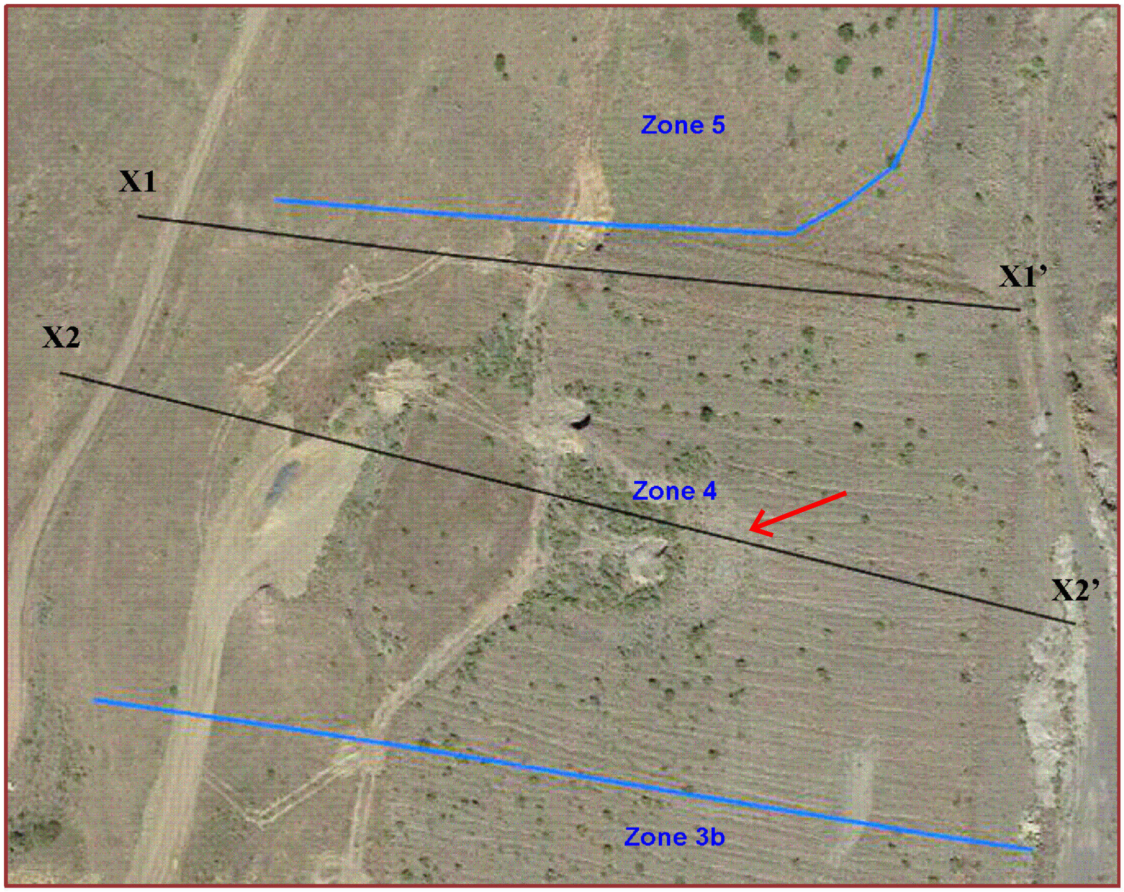

(Not to Scale)

**GROUND**
ENGINEERING CONSULTANTS

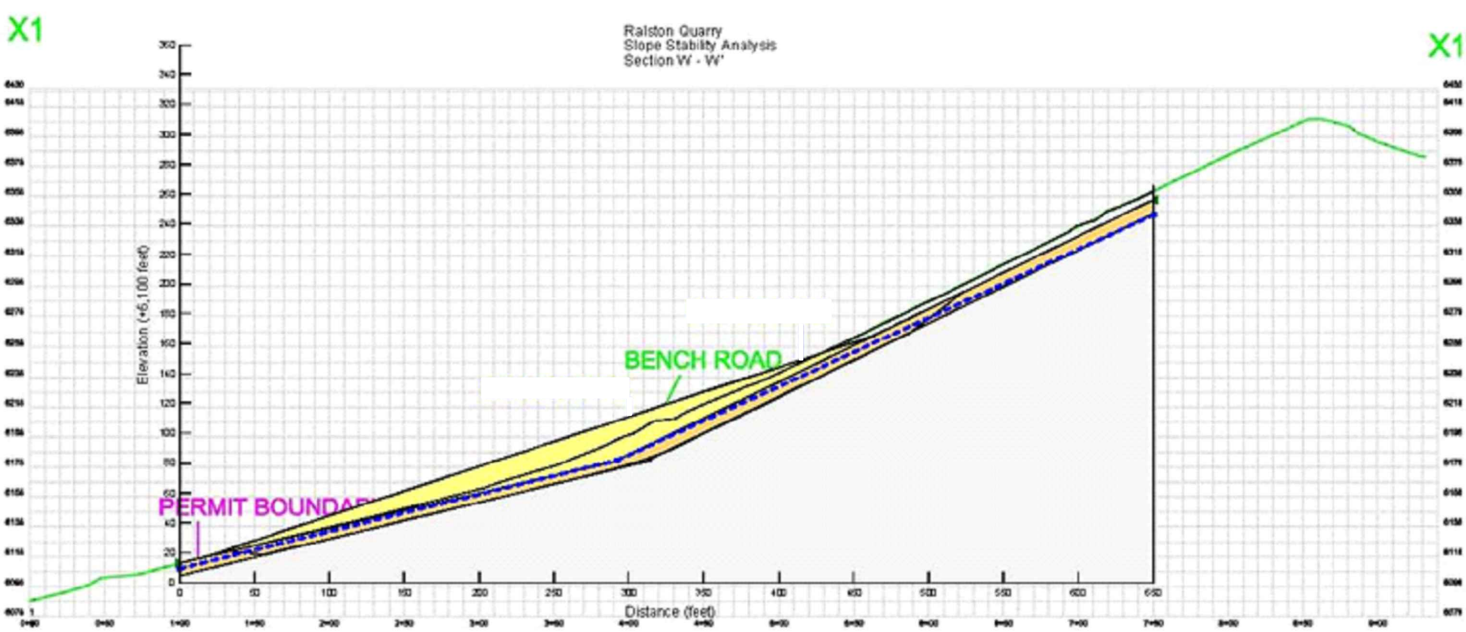
CONCEPTUAL EMBANKMENT
BACK DRAIN LAYOUT

| | |
|---|---|
| JOB NO.: 19-1235 | FIGURE: 26 |
| CADFILE NAME: 1235DRAINLAYOUT.DWG | |



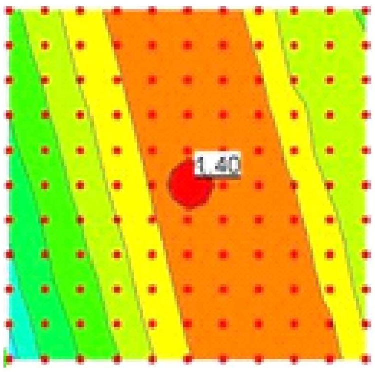

*Zone 4 and Sections X1 – X1' and X2 – X2'*
(Red arrow indicates (top of) older slope failure.)

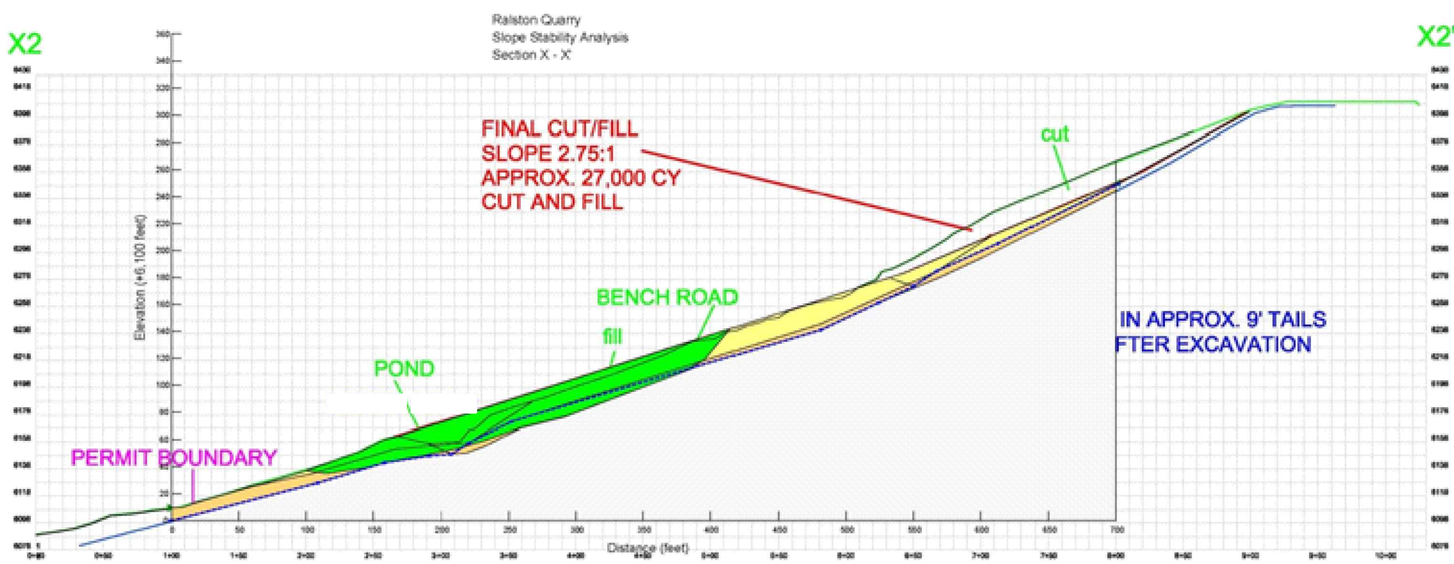



GROUND
ENGINEERING CONSULTANTS

SLOPE STABILITY ANALYSIS

| JOB NO.: 19-1235 | FIGURE: 27A |
|---|---|
| CADFILE NAME: 1235SSA.DWG | |

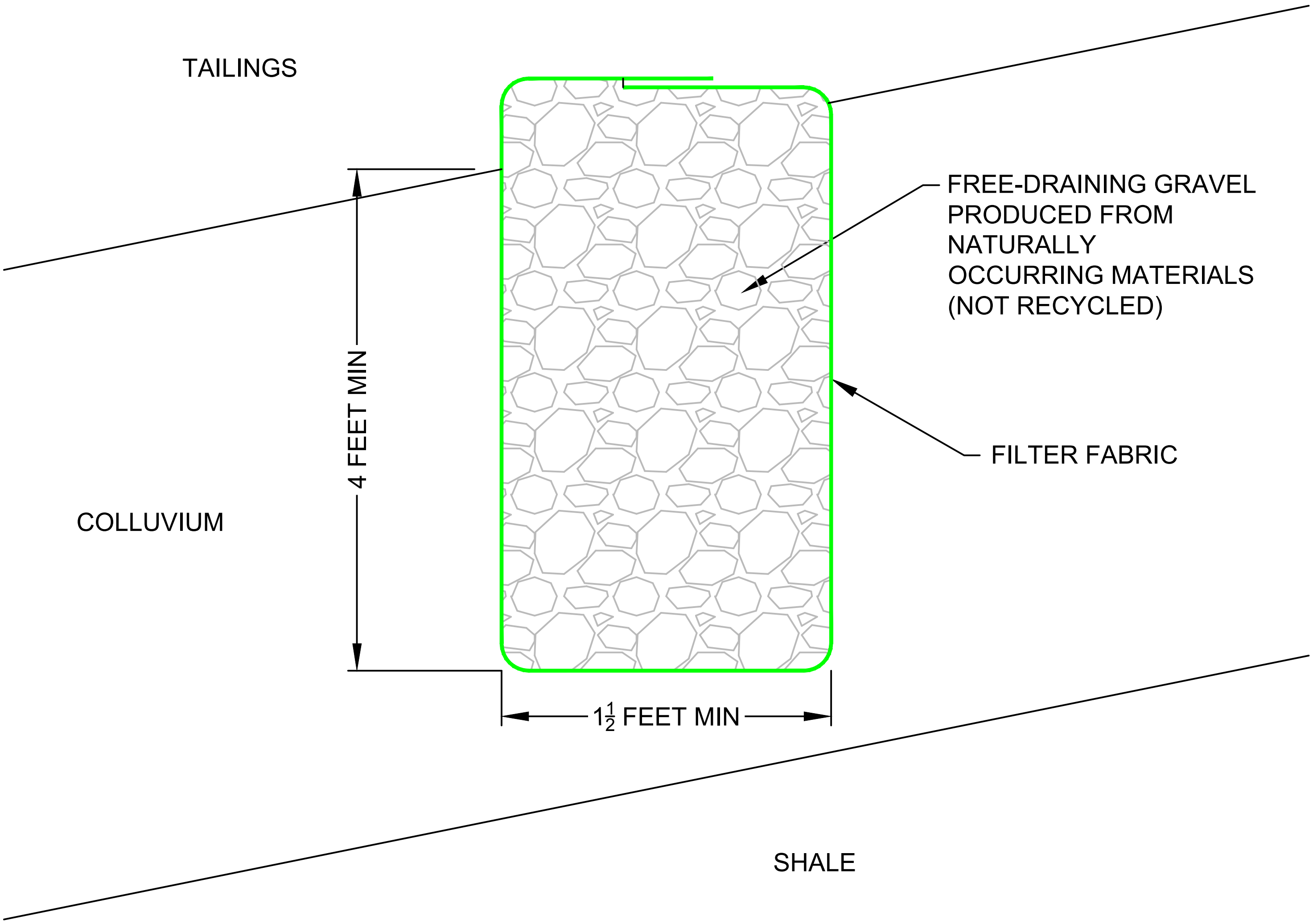



GROUND ENGINEERING CONSULTANTS

SLOPE STABILITY ANALYSIS

| | |
|---|---|
| JOB NO.: 19-1235 | FIGURE: 27B |
| CADFILE NAME: 1235SSA.DWG | |




SEE TEXT FOR ADDITIONAL INFORMATION

NOTES:
1. THIS IS NOT A DESIGN-LEVEL DRAWING. IT SHOULD BE USED SOLELY FOR GENERAL INFORMATIONAL PURPOSES ONLY. ACTUAL UNDERDRAIN DESIGN SHOULD BE COMPLETED BY OTHERS.
2. THE UNDERDRAIN SYSTEM MUST BE TESTED BY THE CONTRACTOR AFTER INSTALLATION AND BACKFILLING TO VERIFY THAT IT FUNCTIONS PROPERLY.
3. INCLUSION OF THIS FIGURE IN CONSTRUCTION DOCUMENTS IS DONE SO AT THE DOCUMENT PREPARER'S RISK.
4. REPRODUCTION OF THIS DOCUMENT SHOULD BE IN COLOR.

(Not to Scale)

GROUND ENGINEERING CONSULTANTS

TYPICAL SLOPE DRAIN DETAIL

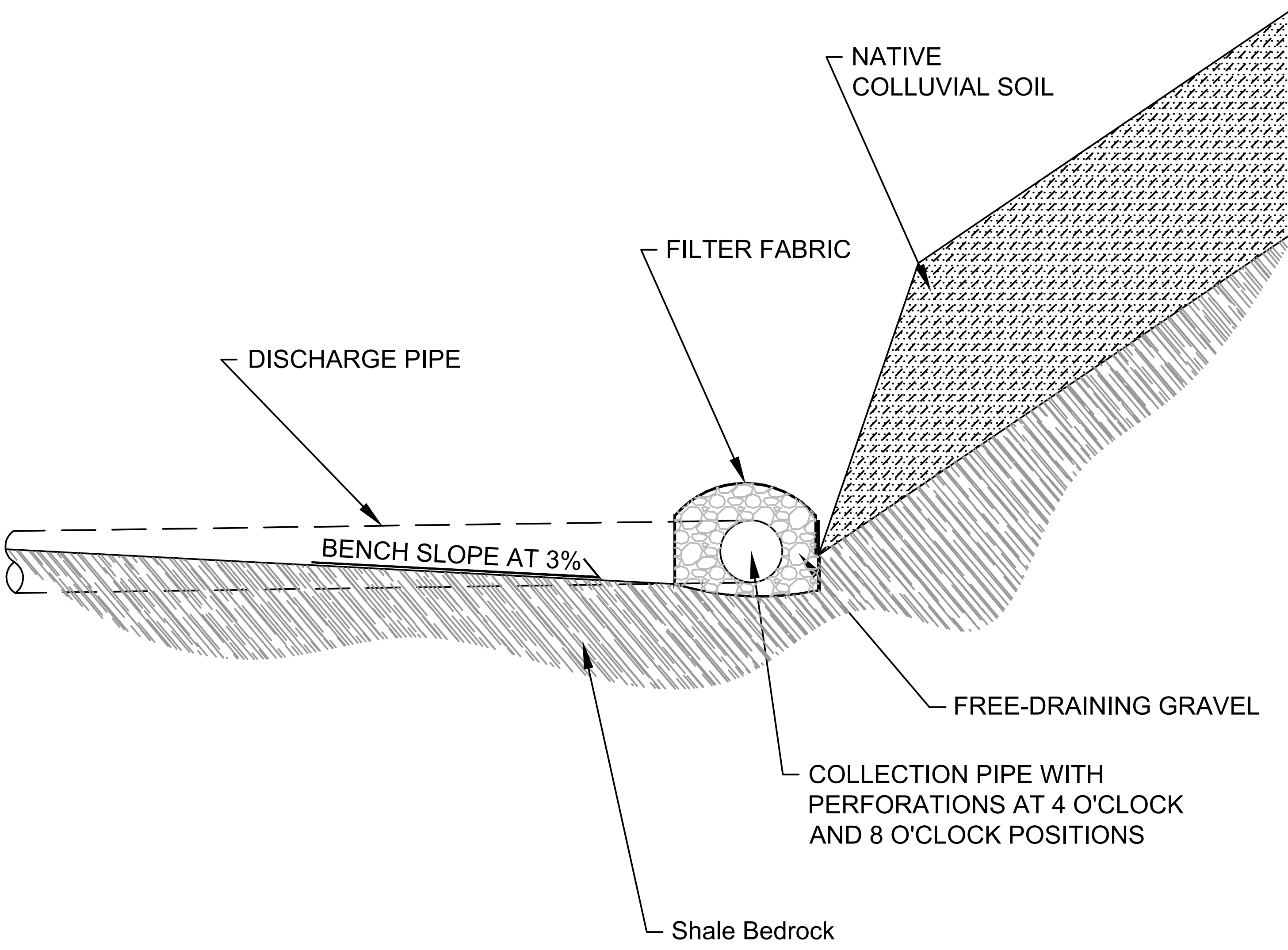
| JOB NO.: 19-1235 | FIGURE: 28 |
|---|---|
| CADFILE NAME: 1235DRAIN02.DWG | |

Zone 5

GROUND
ENGINEERING CONSULTANTS

SLOPE STABILITY ANALYSIS

| JOB NO.: 19-1235 | FIGURE: 29 |
|---|---|
| CADFILE NAME: 1235SSA.DWG | |




**SEE TEXT FOR ADDITIONAL INFORMATION**

NOTES:
1. THIS IS NOT A DESIGN-LEVEL DRAWING. IT SHOULD BE USED SOLELY FOR GENERAL INFORMATION PURPOSES ONLY. ACTUAL UNDERDRAIN DESIGN SHOULD BE COMPLETED BY OTHERS.
2. THE UNDERDRAIN SYSTEM MUST BE TESTED BY THE CONTRACTOR AFTER INSTALLATION AND BACKFILLING TO VERIFY THAT IT FUNCTIONS PROPERLY.
3. INCLUSION OF THIS FIGURE IN CONSTRUCTION DOCUMENTS IS DONE SO AT THE DOCUMENT PREPARER'S RISK.
4. REPRODUCTION OF THIS DOCUMENT SHOULD BE IN COLOR.

(Not to Scale)

GROUND
ENGINEERING CONSULTANTS

TYPICAL BACK DRAIN DETAIL

| | |
|---|---|
| JOB NO.: 19-1235 | FIGURE: 30 |
| CADFILE NAME: 1235DRAIN03.DWG | |

GROUND ENGINEERING

Asphalt Paving Company

19-1235

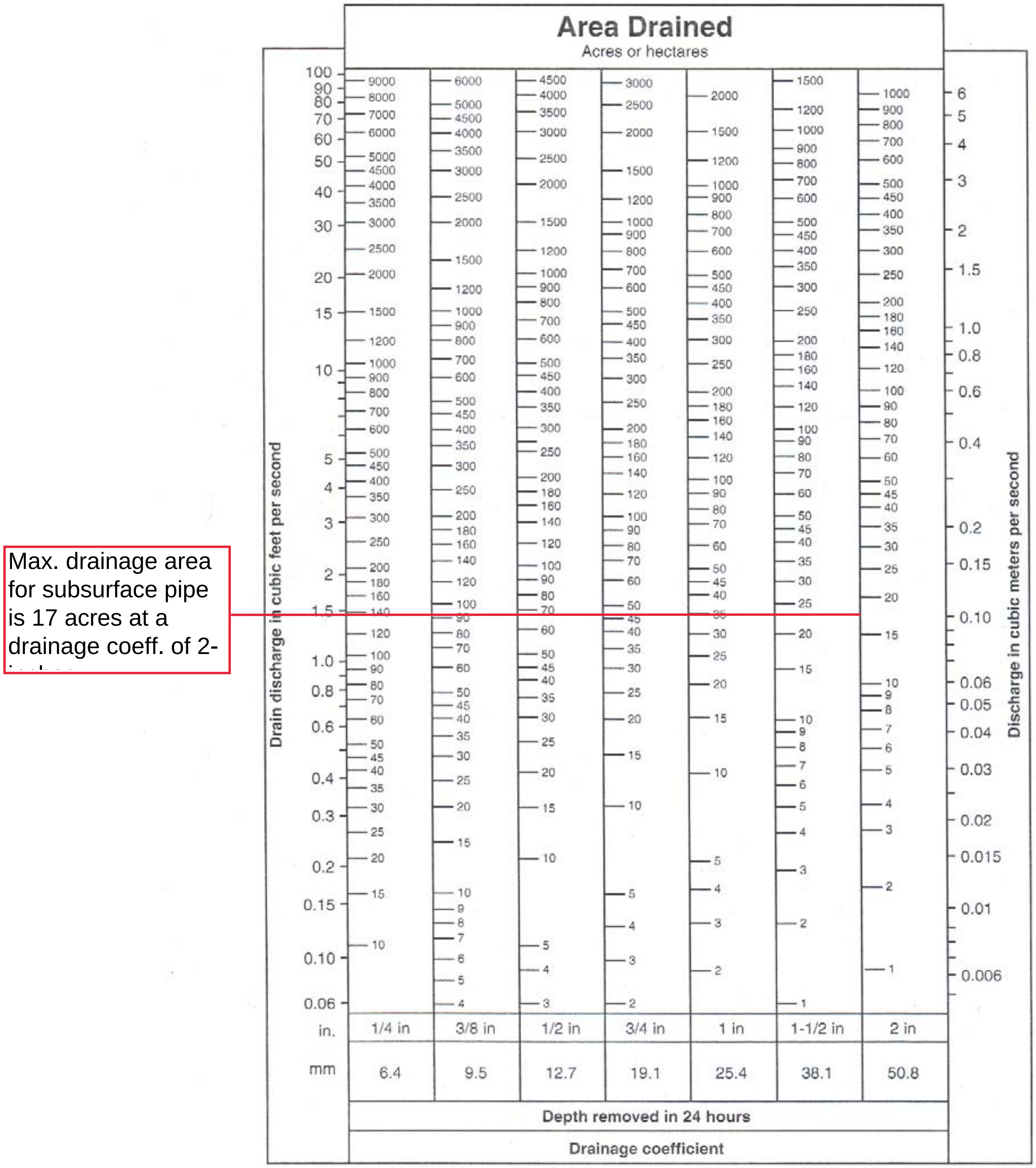
## Ralston Quarry Slope Stability

## TABLE 1: SUMMARY OF LABORATORY TEST RESULTS

| Sample Location | | Natural Moisture Content (%) | Natural Dry Density (pcf) | Gradation | | | Atterberg Limits | | USCS Equivalent Classification | Sample Description |
|---|---|---|---|---|---|---|---|---|---|---|
| Test Hole No. | Depth (feet) | | | Gravel (%) | Sand (%) | Fines (%) | Liquid Limit | Plasticity Index | | |
| 1 | 13 | 10.0 | 121.2 | | | 46 | 34 | 14 | SC | Clayey SAND |
| 1 | 18 | 24.5 | 95.2 | | | 80 | 59 | 33 | (CH)s | CLAY with Sand |
| 2 | 2 | 19.5 | 108.3 | | | 90 | 53 | 30 | CH | CLAY |
| 2 | 12 | 15.3 | 118.8 | | | 99 | 51 | 27 | CH | CLAY SHALE |
| 3 | 9 | 25.1 | 93.4 | | | 76 | 62 | 38 | (CH)s | CLAY with Sand |
| 3 | 19 | 16.2 | 114.8 | | | 95 | 52 | 29 | CH | CLAY SHALE |
| 4 | 1 | 6.5 | SD | | | 28 | 30 | 11 | SC | Clayey SAND |
| Lower Slope | | | | 0 | 17 | 83 | 48 | 25 | (CL)s | CLAY with Sand |
| Mid-Slope | | | | 8 | 63 | 30 | 35 | 14 | SC | Clayey SAND |

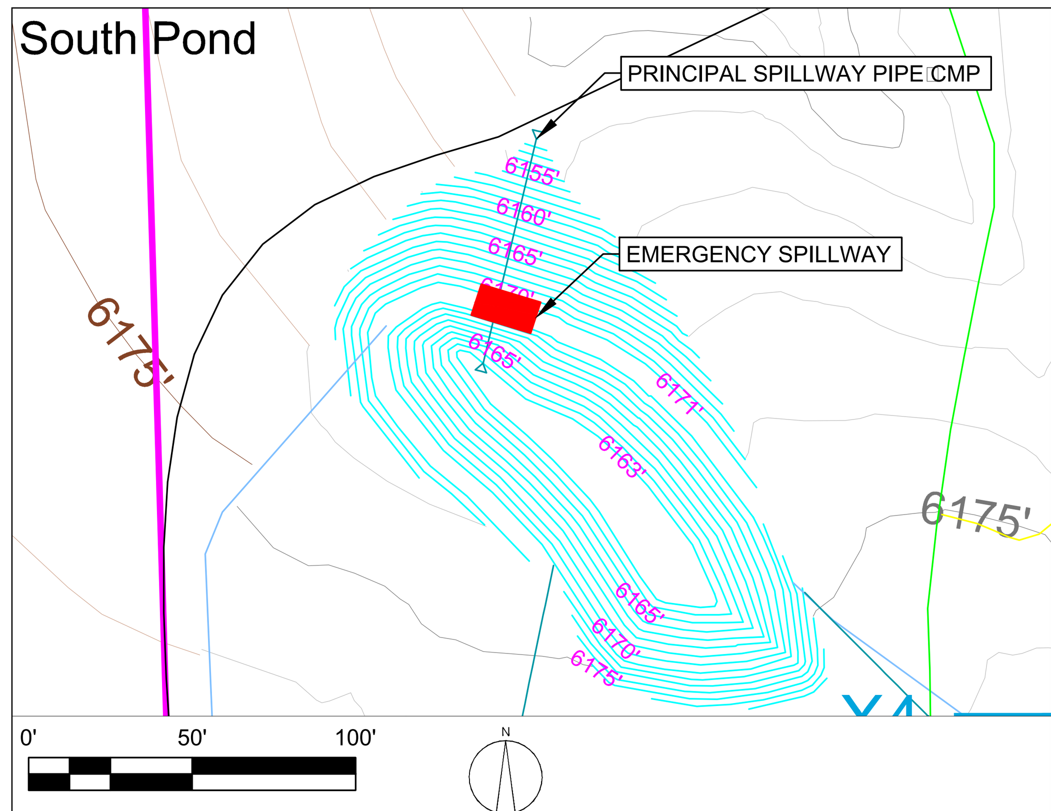
SD indicates 'Sample disturbed.'

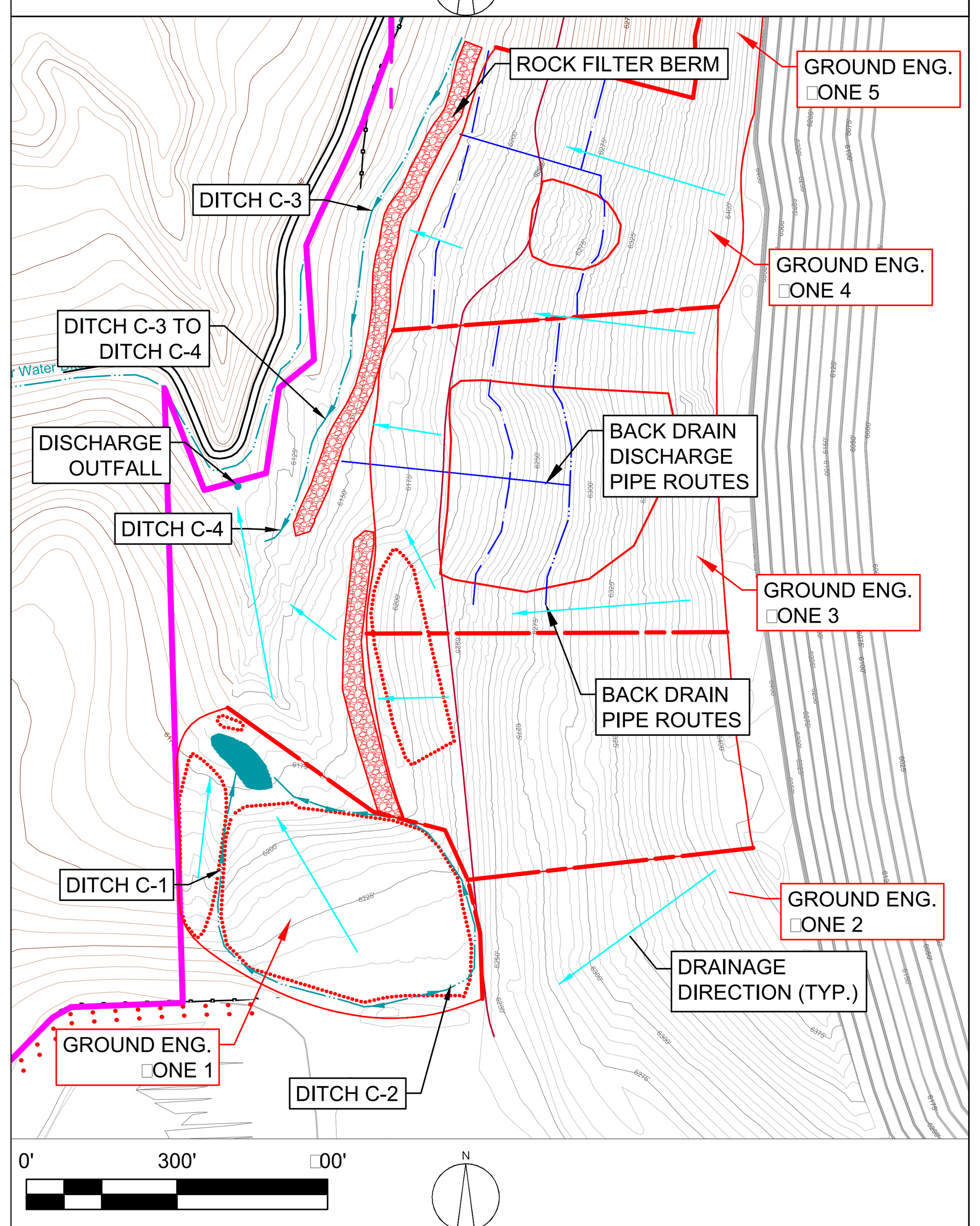
**Figure 3-3** Subsurface drain discharge from drainage coefficient




Note: Use acres with $ft^3/s$ and hectares with $m^3/s$
(Source-ASAE Standard EP260.4)

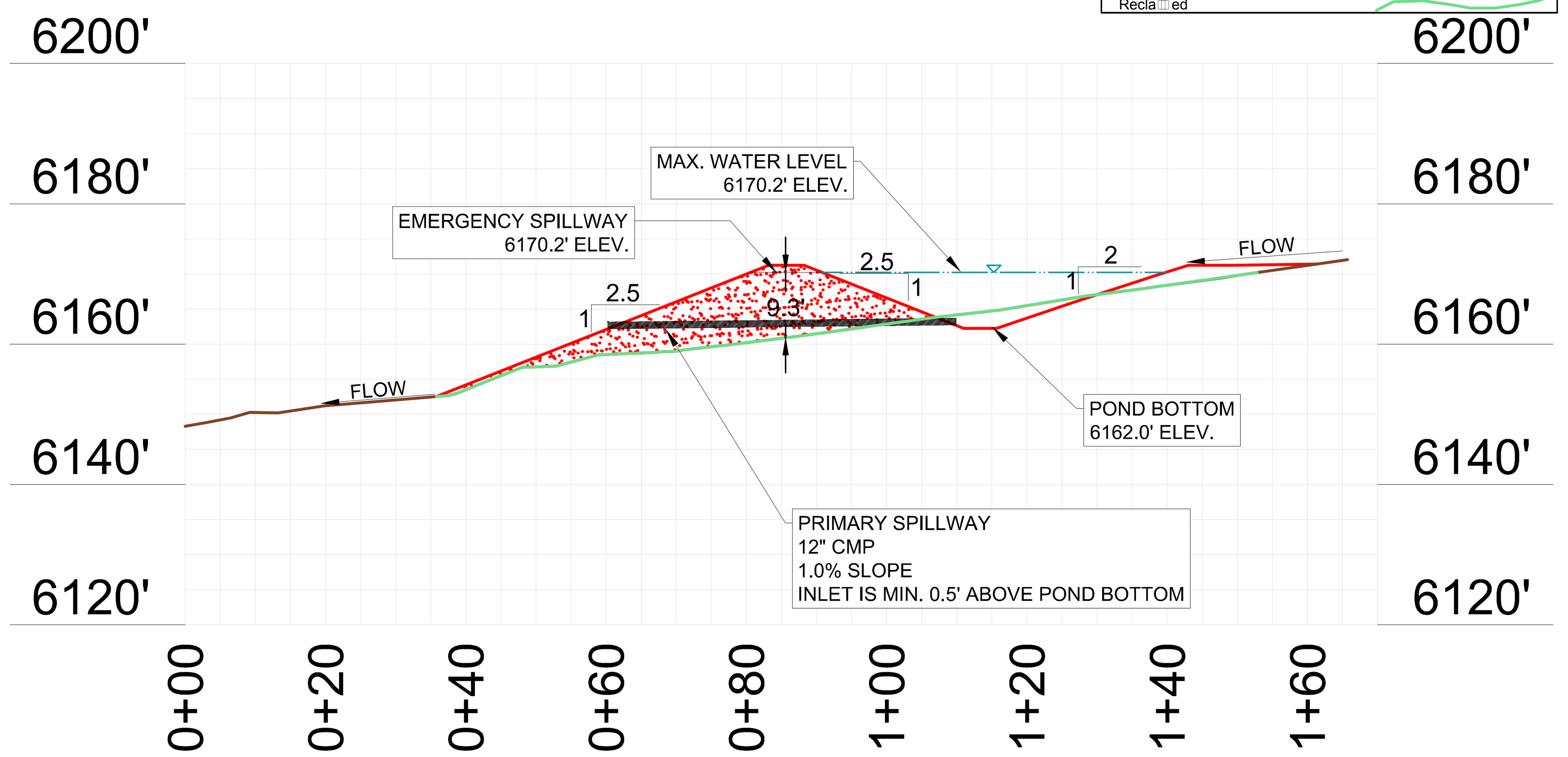
# South Pond

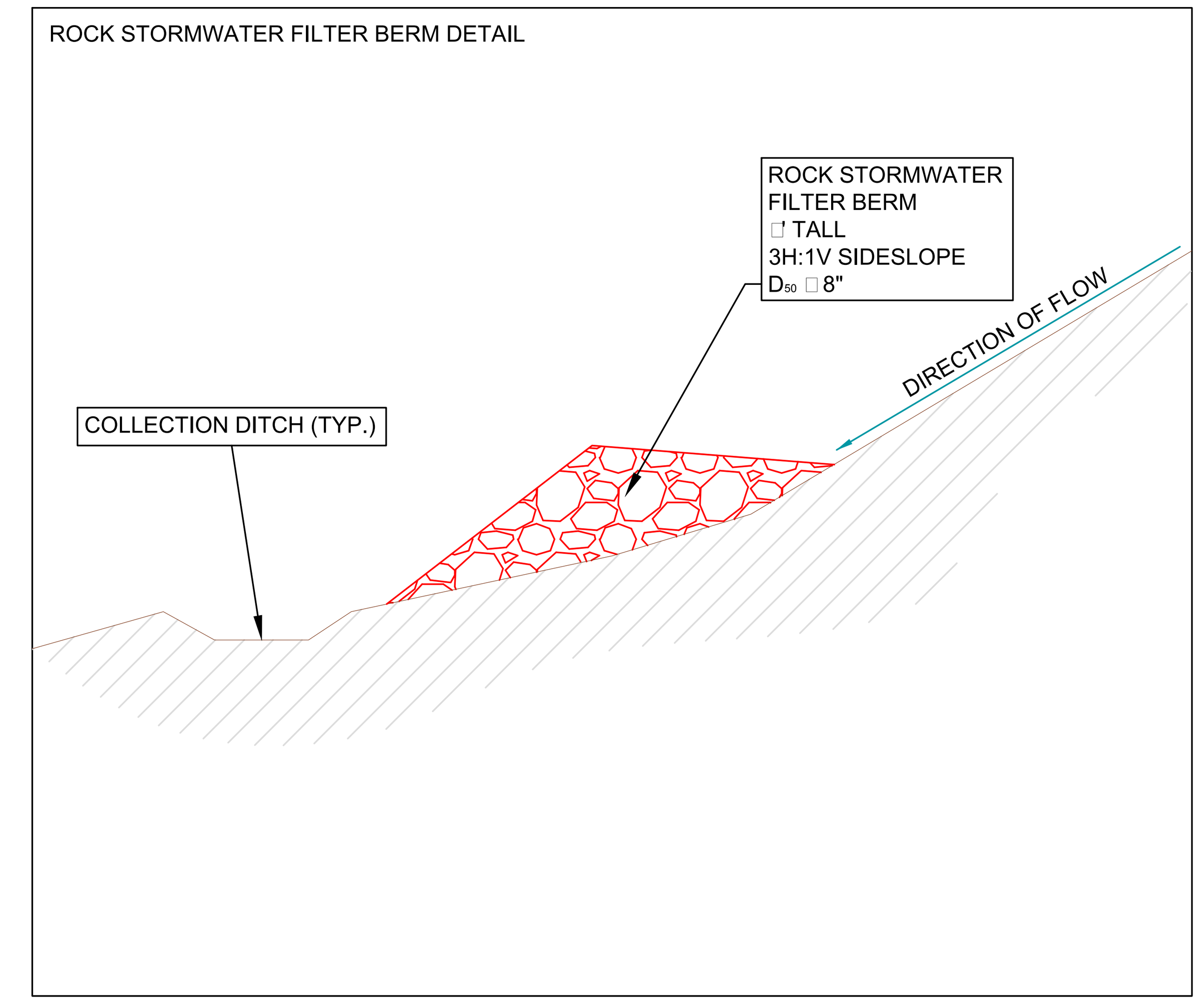



## South Pond

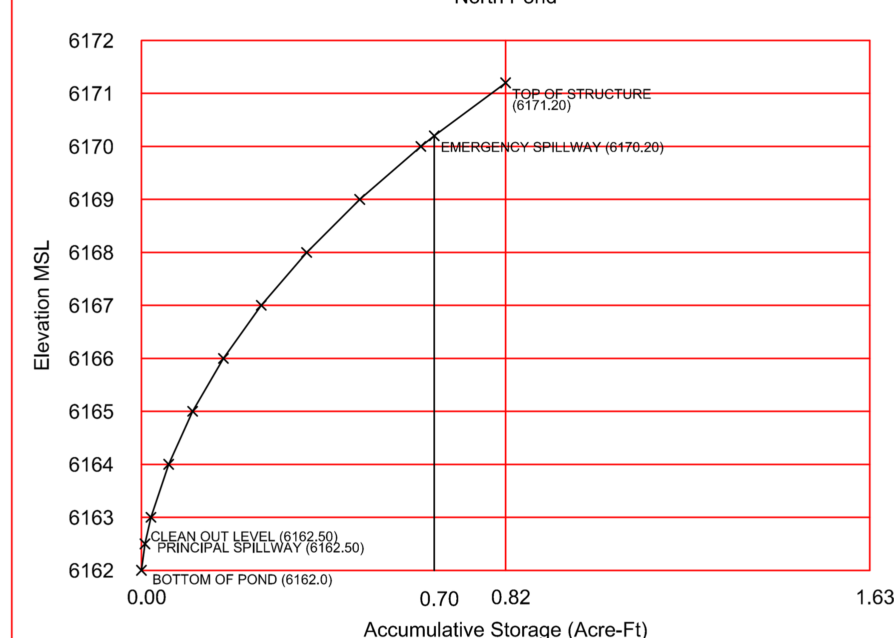
| LEGEND | |
|---|---|
| BOUNDARIES | |
| DRMS Per□□t Bo□□dar□ | |
| CROSS SECTION TOPOGRAPHY | |
| Ba□el□e | |
| M□□□ | |
| Recla□□ed | |




M-1974-083

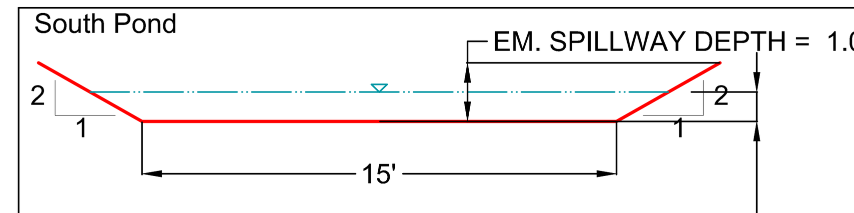
Stage Storage Curve
North Pond




Storage volume computations
M-1974-083
North Pond

| ELEV. (ft) | Width (ft) | LENGTH (ft) | AREA (ac) | AVG. AREA (ac) | INTERVAL (ft) | STORAGE (ac-ft) | ACC. STORAGE (ac-ft) | STAGE INTERVAL (ft) |
|---|---|---|---|---|---|---|---|---|
| 6162.00 | 6162.00 | 6162.00 | 0.0100 | | | | | |
| 6162.50 | N/A | N/A | 0.0213 | 0.0156 | 0.50 | 0.0078 | 0.0078 | 0.50 |
| 6163.00 | 6163.00 | 6163.00 | 0.0326 | 0.0213 | 0.50 | 0.0135 | 0.0213 | 1.00 |
| 6164.00 | 6164.00 | 6164.00 | 0.0466 | 0.0396 | 1.00 | 0.0396 | 0.0609 | 2.00 |
| 6165.00 | 6165.00 | 6165.00 | 0.0613 | 0.0540 | 1.00 | 0.0540 | 0.1148 | 3.00 |
| 6166.00 | 6166.00 | 6166.00 | 0.0769 | 0.0691 | 1.00 | 0.0691 | 0.1839 | 4.00 |
| 6167.00 | 6167.00 | 6167.00 | 0.0931 | 0.0850 | 1.00 | 0.0850 | 0.2689 | 5.00 |
| 6168.00 | 6168.00 | 6168.00 | 0.1102 | 0.1016 | 1.00 | 0.1017 | 0.3706 | 6.00 |
| 6169.00 | 6169.00 | 6169.00 | 0.1280 | 0.1191 | 1.00 | 0.1191 | 0.4897 | 7.00 |
| 6170.00 | 6170.00 | 6170.00 | 0.1466 | 0.1373 | 1.00 | 0.1373 | 0.6270 | 8.00 |
| 6170.20 | N/A | N/A | 0.1505 | 0.1485 | 0.20 | 0.0297 | 0.6567 | 8.20 |
| 6171.20 | 6171.20 | 6171.20 | 0.1700 | 0.1583 | 1.00 | 0.1603 | 0.8170 | 9.20 |

South Pond




DEPTH OF FLOW = 0.5'
DISCHARGE CAPACITY = 15 CFS

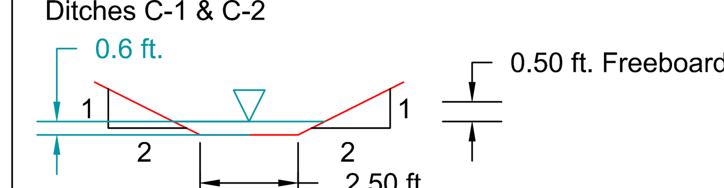
10-YR 24-HR EVENT:
0.69 ac-ft, 12.7 cfs

## D□tch Flo□ Re□□□re□e□t□

| D□tch | I□flo□ Area (acre□) | D□□char□e (CFS) |
|---|---|---|
| C-1 | 2.3 | □.0 |
| C-2 | 2.5 | □.□ |
| C-3 | 10.8 | 1□.1 |
| C-4 | □.□ | 28.2 (1□.1□12.1) |

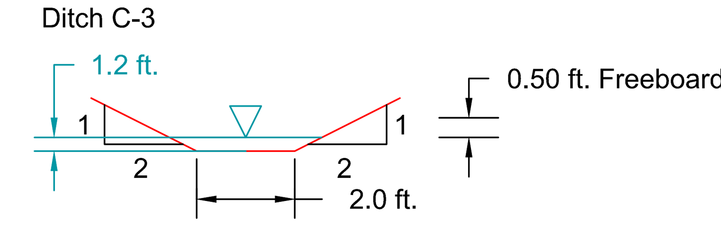
### Collection Ditch Designs

Ditches C-1 & C-2



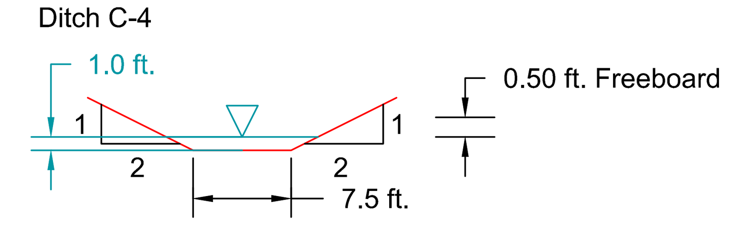

Ditches C-1 & C-2
Trapezoidal Channel
Equal Side Slopes
Channel Lining: Graded silts to cobbles
(excavated in existing material)
Capacity: 8.3 CFS on 20% grade (3.75ft/s)

Ditch C-3




Ditch C-3
Trapezoidal Channel
Equal Side Slopes
Channel Lining: Riprap $D_{50}$ = 6-inch
Capacity: 23 CFS on 1.7% grade (4.3 ft/s)

Ditch C-4




Ditch C-4
Trapezoidal Channel
Equal Side Slopes
Channel Lining: Riprap $D_{50}$ = 12-inch
Capacity: 60 CFS on 4.5% grade (6.7 ft/s)

NOTES:
- DITCH C-3 MUST HAVE CAPACITY OF ITS OWN DRAINAGE AND A SUBSURFACE DRAINAGE SYSTEM
- DITCH C-4 MUST HAVE CAPACITY OF ITS OWN DRAINAGE AREA PLUS DITCH C-3 AND SUBSURFACE DRAINAGE SYSTEM
- CURVE NUMBER OF □9 USED IN ALL RUNOFF CALCS.
- DITCHES C-3 THROUGH C-4 ARE DESIGNED TO BE NON-EROSIVE BUT TEMPORARY
- 10-YR EVENT USED FOR ALL DITCH DESIGNS AND POND DESIGN

## Ma□ C-4 Stor□ □ater De□□□□□
## Ral□to□ □ □arr□
## A□□halt Pa□□□□ Co.

DRMS Per□ □t N□□ □er: M-19□4-08□

| | | | |
|---|---|---|---|
| M□□e E□tr□ Locat□o□: | Lat□t□de: 39.81222 | Lo□□□t□de: 105.23139 | |
| State: CO | Co□□t□: Je□□er□o□ | Neare□t To□□: Ar□ada | |
| Sect□o□: 4 | To□□□□h□□: 3S | Ra□□e: □0W | PM: □th |
| Ma□or Water□hed: ---- | | | |
| Ma□ Scale: 100 | 0' | 500' | 1000' |

Ma□ Geore□ere□c□□□ I□□or□at□o□: Dat□□: NAD83 Pro□ect□o□: CO Ce□tral
S□r□e□ So□rce: Goo□le □ Dro□e S□r□e□ Date: 2010 □ 2018
I□a□er□ So□rce: Goo□le □ Dro□e I□a□er□ Date: 08□1□□18
Dra□□ □□: BEL Date: 01□22□20
Checked □□: BEL Date: 01□24□20
A□□ro□ed □□: GL Date: 01□24□20

F□le Na□e: E:□Work□Dro□□o□(GLA)□A□□halt Pa□□□□ Co□ □a□□□Ral□to□ □ □arr□□A□toCAD□Ral□to□ □ □arr□200120.d□□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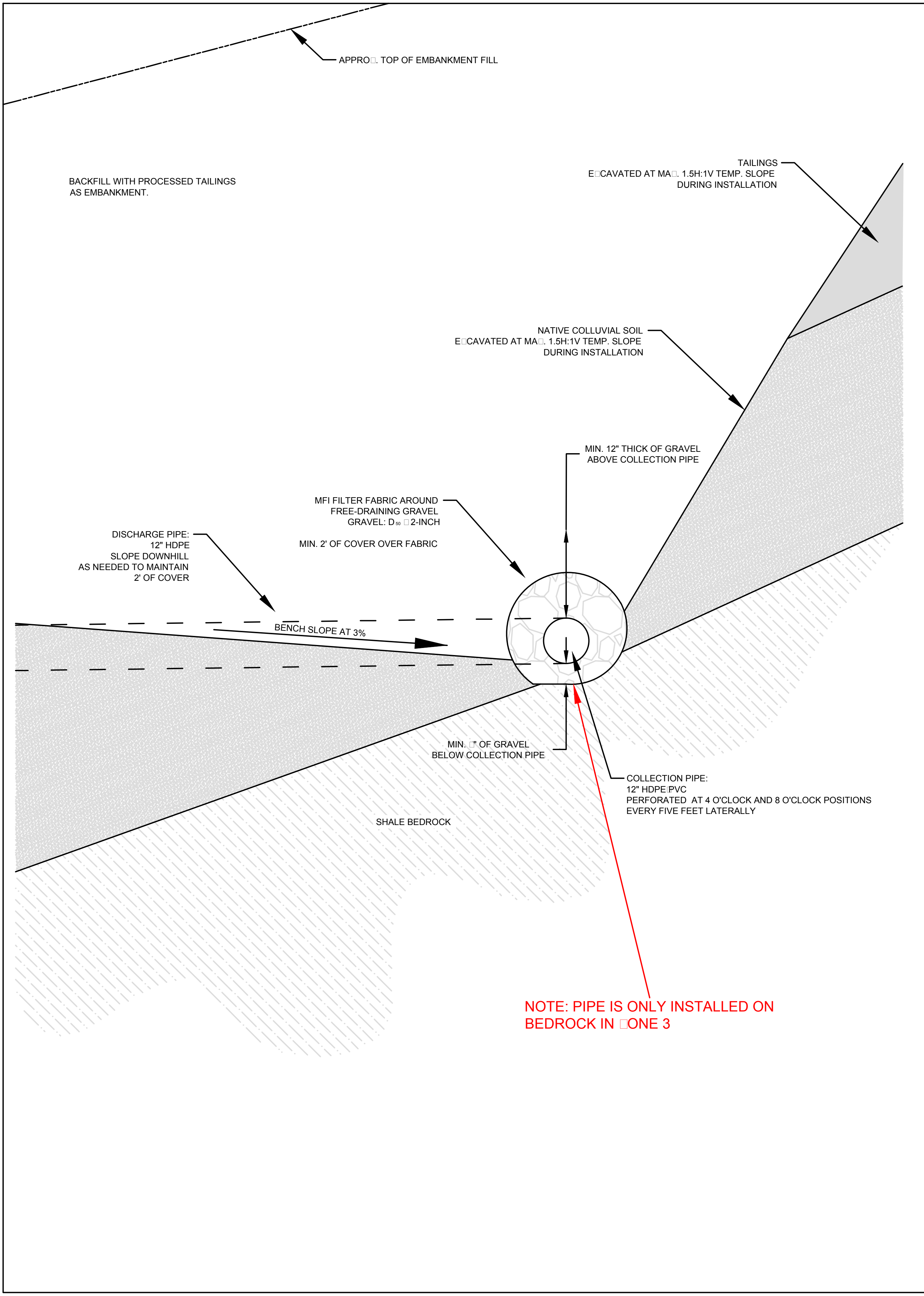


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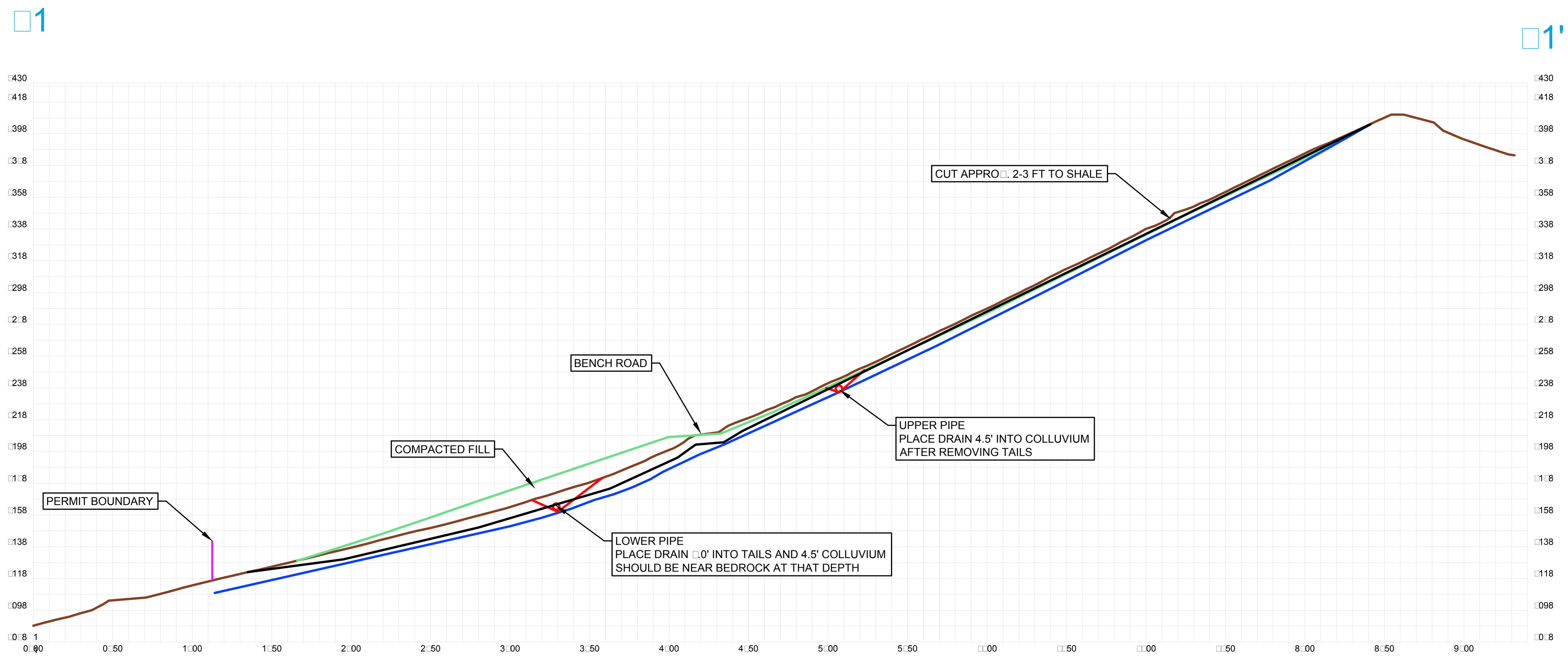
Map Georeferencing Information:
Survey Source: Google & Drone
Imagery Source: Google & Drone
Drawn by: BEL
Checked by: BEL
Approved by: GL

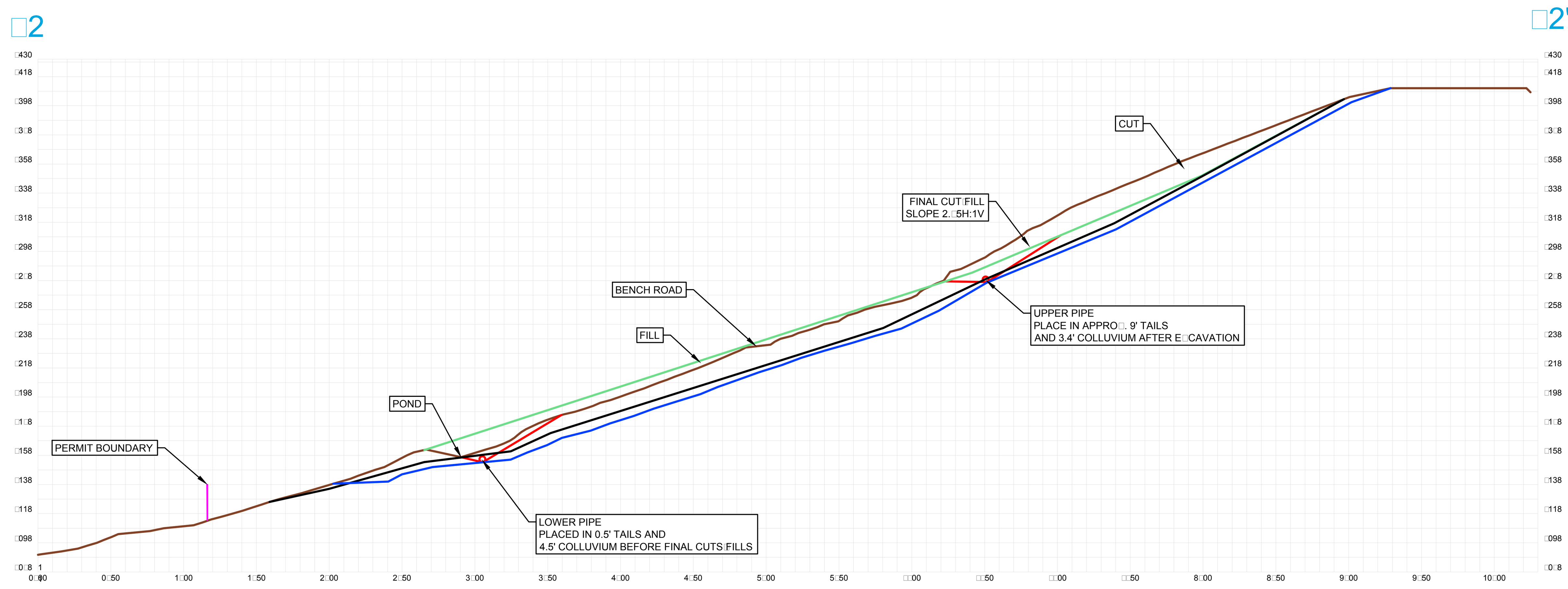
Datum: NAD83 Projection: CO Central
Survey Date: 2010 & 2018
Imagery Date: 08/16/18

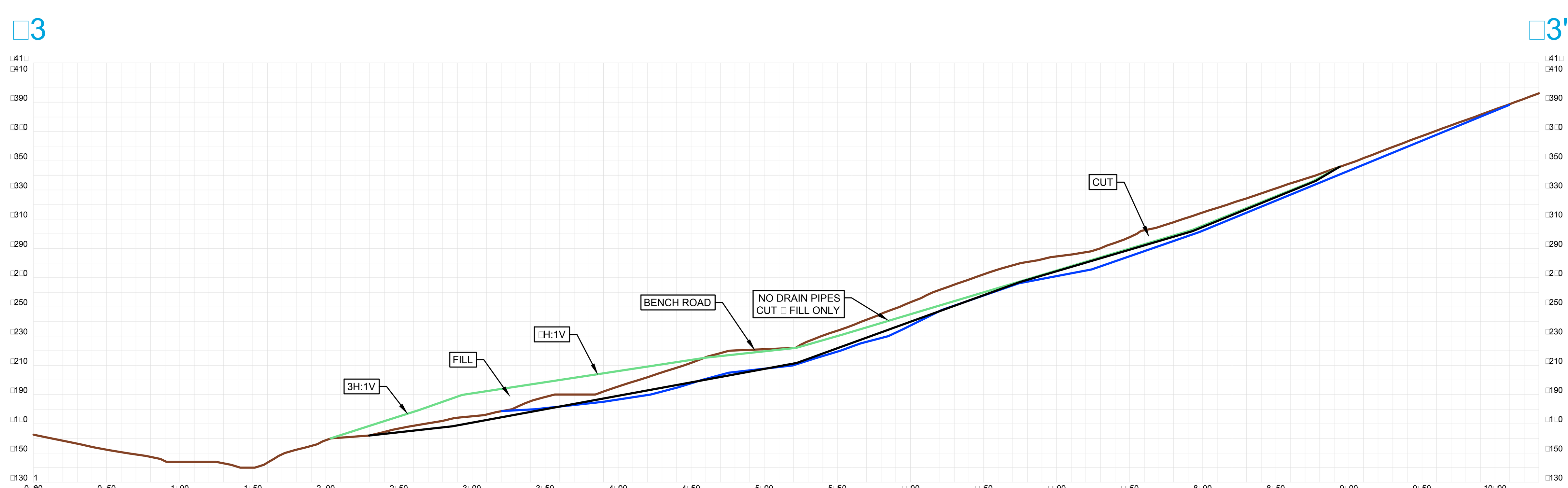
Date: 01/22/20
Date: 01/24/20
Date: 01/24/20

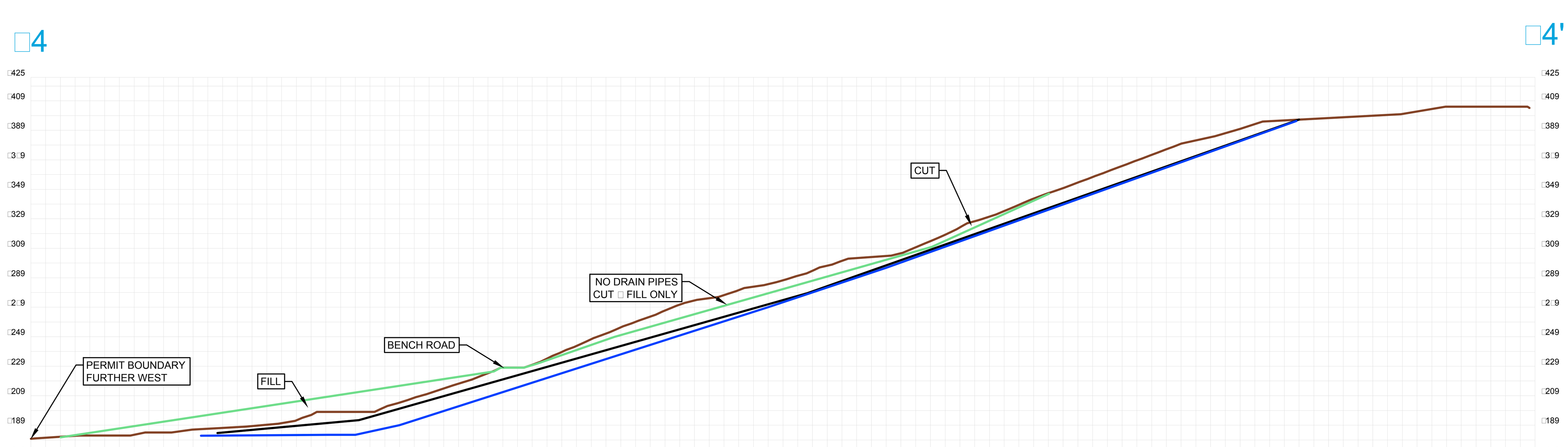
DRMS Permit Number: M-1974-086
Mine Entry Location: Latitude: 39.81222 Longitude: 105.23139
State: CO County: Jefferson Nearest Town: Arvada
Section: 4 Township: 3S Range: 70W PM: 6th
Major Watershed: ----
NOT TO SCALE

# Map C-5 Back Drain Design
## Ralston Quarry
## Asphalt Paving Co.



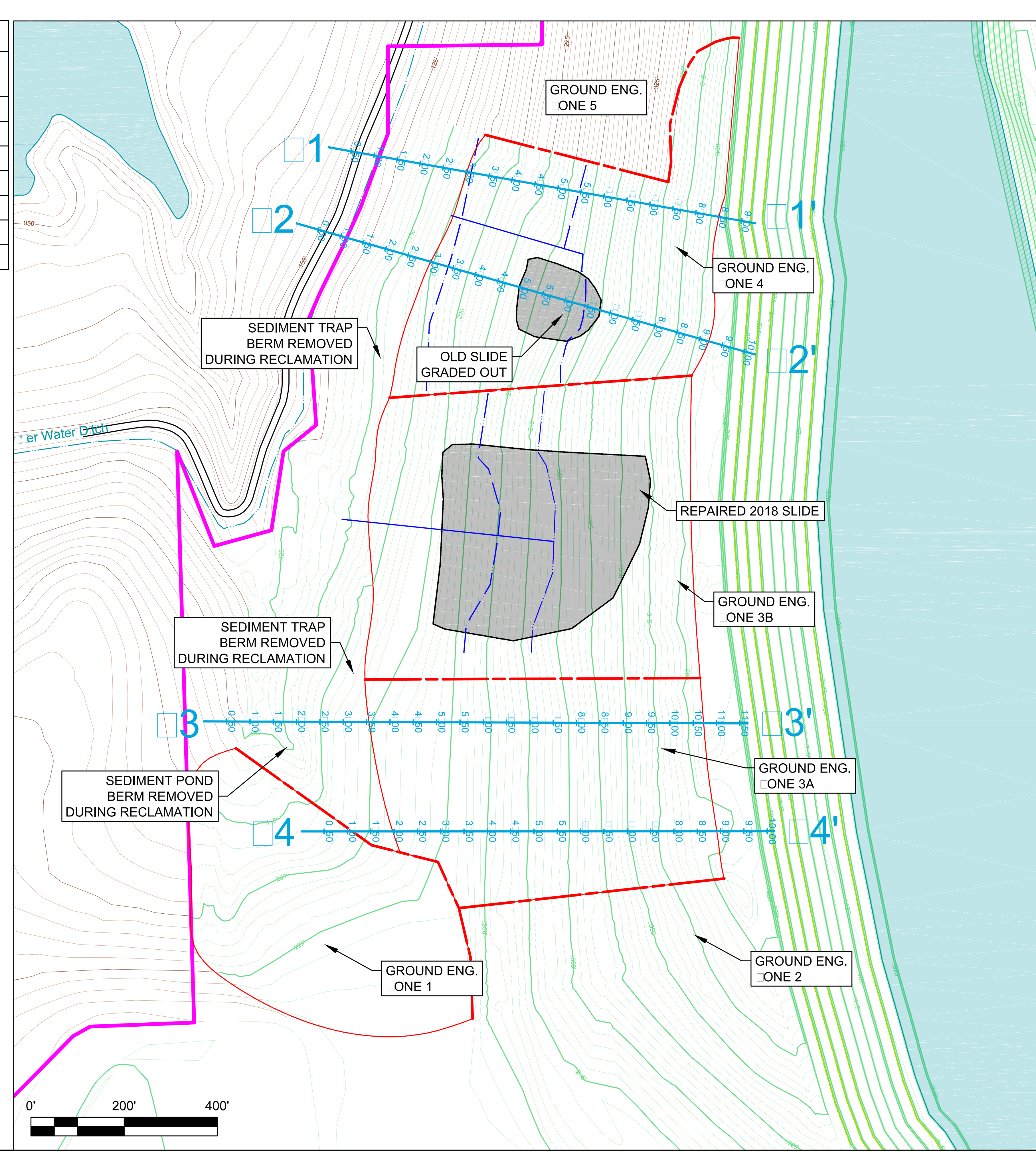




## Repair Zones

| Zone | Cross Section | Factor of Safety[1] | Regrade | Buttress | Subsurface Drain Pipe |
|---|---|---|---|---|---|
| 1 | N/A | 2.3 | No | No | No |
| 2 | N/A | 1.3 | No | No | No |
| 3A | Z3 / Z4 | 1.3 | Yes | No | No |
| 3B | N/A | 1.3 | Yes | Yes | Yes |
| 4 | Z1 / Z2 | 1.2 | Yes | No | Yes |
| 5 | N/A | N/A | No | No | No |

[1] From Ground Engineering Report dated Jan. 2020




# Map F-2 West Slope Regrade
# Ralston Quarry
# Asphalt Paving Co.

DRMS Permit Number: M-19[illegible]4-08[illegible]

Mine Entry Location: Latitude: 39.81222 Longitude: 105.23139
State: CO County: Jefferson Nearest Town: Arvada
Section: 4 Township: 3S Range: [illegible]0W PM: [illegible]th
Major Watershed: ----

Map Georeferencing Information: Datum: NAD83 Projection: CO Central
Survey Source: Google / Drone Survey Date: 2010 / 2018
Imagery Source: Google / Drone Imagery Date: 08/1[illegible]/18
Drawn By: BEL Date: 01/22/20
Checked By: BEL Date: 01/24/20
Approved By: GL Date: 01/24/20

File Name: E:\Work\Dropbox\(GLA)\Asphalt Paving Company\Ralston Quarry\AutoCAD\Ralston Quarry 200120.dwg

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## LEGEND

BASELINE CONTOURS
- Index
  - Approximate 20' Intervals
- Intermediate
  - Approximate 5' Intervals
- Sublevel Contours

RIVERS, LAKES, SHORELINES, AND CANALS
- Perennial lake/pond
- Intermittent lake/pond
- Subsurface drain pipe

RECLAIMED CONTOURS
- Index
  - Approximate 20' Intervals
- Intermediate
  - Approximate 5' Intervals
- Sublevel Contours

POST MINE FEATURES
- DRMS Permit Boundary

CROSS SECTION TOPOGRAPHY
- Baseline
- Mined
- Reclaimed
- Shale (approx.)
- Colluvium (approx.)