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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 TRØ3

File No.: M- 1985-043 Site Name: Red Canyon Quarry

County Fremont TR# _______ *(DRMS Use only)*

Permittee: Rocky Mountain Materials and Asphalt, Inc.

RECEIVED
AUG 24 2015
DIVISION OF RECLAMATION MINING AND SAFETY

Operator (If Other than Permittee): _______

Permittee Representative: Robert Mangone

Please provide a brief description of the proposed revision:
Provide Geotechnical Stability Exhibit

√ Annual Fee/Report (no Map)
√ Violations
MV1987034 5/27/1987

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	☐

GEOTECHNICAL STABILITY STUDY

Red Canyon Quarry
Fremont County, CO

Rocky Mountain Materials & Asphalt
1910 Rand Avenue
Colorado Springs, CO 80905

August, 2015

Prepared by: Azurite, Inc
POBox 338
Cotopaxi, CO 81223

Site Description and Rock Type

The Red Canyon Quarry is located in extreme eastern Fremont County, CO, near the boundary between Fremont and El Paso Counties approximately 3 miles west of State Highway 115 in the Mount Pittsburg quadrangle. The rock units exposed in the mine site are primarily Proterozoic (Precambrian) granite gneiss of the Pike's Peak Complex, with minor exposures of Paleozoic meta-sediments that can be viewed the southeastern corner of the permit area. Due to the fact that Paleozoic sediments exposed in the permit area are limited to those along the east permit boundary and that mining will not progress any further in this direction, the stability study will focus on the Precambrian granite gneiss rock units. The Pike's Peak granite complex outcrops extensively throughout the central front-range area, with numerous examples of a Type A granite/granite gneiss composition, a feldspar (potassium) rich granite batholith containing primarily feldspar and quartz with ample muscovite, biotite and hornblende. A number of intrusive events following the original granite pluton placement resulted in a number of intrusive structures (as well as a number of joint and cleavage patterns) carrying iron enriched mafic materials interior to the alkali granite ore body. Post granite placement episodes of injection and compression have impacted the Pike's Peak granite units with a variety of grain size and textures. Typical at this location is vertical or nearly vertical foliation of the granite gneiss along a strike path of north by 30 degrees east. The orientation and near vertical foliation of the rock is reflected in the steep, near vertical terrain surrounding the site. The highly resistant bedrock naturally weathers to spires of granite rock where vertical runs of 50'or more are typical. The mining plan is also aligned with the geologic and geographic parameters described above, with the primary face development normal to the strike of foliation and direction of mining north by 30 east. The cross section A–A' shown on the site map locates the typical cross section and maximum benches of the Red Canyon highwall development plan. The mine site covers an elevation range of just under 6900'to just over 7400'at top of pit.

Highwall excavation will take place at elevations above 7100' in dry bedrock. No water table (phreatic surface) or potentiometric surface interface is anticipated.

Highwall Stability Study Methodology

The site was visited several times during June, 2015, to scrutinize and sample granite gneiss rock and neighboring meta-sediments outcropping along the southeastern margin of the permit areas. Granite gneiss outcrop differed in physical and geologic habit through a large range of rock type, grain size, texture and orientation. Physical characteristics of rock encountered were recorded, ranging from fine to medium grained K-feldspar granites to heavily foliated biotite granite gneiss. Measurements were made and documented at three locations within the active mining zone reflecting the metamorphic rock's foliation direction and orientation as well as primary, secondary, and in some locations, tertiary joint and cleavage patterns. At some locations, rock failure along preferred joint directions controlled the mining bench geometry. At the least, joint patterning controls rock breakage during blasting as well as crushing operations.

Rock samples were extracted from bedrock exposures in the vicinity of active mining areas at three locations of large enough (12"x12"x12"minimum) size to bore in preparation for Unconfined Compression Testing and Indirect Brazilian Tensile Strength Testing. Care was taken to select rock samples portraying minimum impact from blasting and weathering. Eighteen large rock samples were transported to Golden, Colorado, to the Earth Mechanics Institute at Colorado School of Mines for boring and testing. EMI performed 27 individual failure tests with details of the test results included in this report. The resulting data, specifically median values for Unconfined Compressive Strength was input to Galena software for stability analysis. Indirect (Brazilian)Tensile Strength results were utilized to guide final highwall configuration dimensions proposed in the final highwall configuration design plan.

Joint and Cleavage control of fracture planes in the granite gneiss rock is complex and dynamic as one traverses active mining areas. Joint patterns were documented at several locations as shown on the site map (Survey Sites 1-3). Joint patterns can have considerable impact on mining operations as conditions at the mining face can become unstable and possibly a danger to work around if prominent joint patterns are not considered during drilling and blasting. Dominant joint patterns over a large area can continue to the extent of a predominant fault plane with a larger scale impact than at that specific mining face or even one mine bench. These potentials were noted to be at minimal level at this stage in mine development, with relatively small scale examples of failure plane control on the scale of a mining bench. Jointing patterns as well as identification of potential failure planes along clay mineral zones were documented and taken into account in the final highwall design dimensions. While some locations within the active mining area were identified as reflecting joint patterning that could, if found to continue, demand adjustment in direction of mining or consume more working space to insure a safe work area, no major fault planes, potential or former failure planes, or suspicious ground conditions were noted that might raise concerns of bedrock stability.

Galena Stability Analysis

The rock strength parameters gained from lab testing were input to Galena software models reflecting proposed final maximum bench build-out potential. Due to present conditions of top of pit elevation (~7300') vs. projection of bottom of eventual highwall excavation elevation at the 7000'elevation (about 30'higher than present crushing system elevation), the maximum number of potential mine benches used in overall slope stability analysis was eight. In reality, the number of final benches will be less due to mining removal of as many as five of the upper benches as mining proceeds northeast towards the higher ground. At a point approximately 1600'northeast of the southern terminus point of cross section A-A', the present ground elevation is app. 7100', or 100'above the south terminus point elevation. However, for purposes of modeling the maximum possible exposure of mined rock face, this study projected a maximum possible bench height of 320', or 8 - 40' in high benches.
Galena software allows for multiple analysis including Bishop Simplified Single and Multiple failure analysis, circular and non-circular failure surfaces, Spencer-Wright Single and Multiple Surface analysis, and Sarma Non-Vertical Single and Multiple analysis. Input parameters were obtained using Hoek-Brown criteria and approximate relationships between constants *m* and *s* and the rock mass classification developed by Bieniawski for both disturbed and undisturbed rock masses. A short paper including a table describing the approximate relationships is included in this study.

Potential seismic stress factor of 0.15 was used for all runs.

All water encountered on site is meteoric derived surface water with no evidence of springs or other ground water expressions. No phreatic surface or piezometric surface was included in the modeling for this location.

Results of Slope Stability Analysis

All of the model runs of slope stability resulted in risk factors of 1.3 or higher, reflecting a stable slope at a conservative 1:1 overall final slope. Failure analysis confirms stability regardless of bench dimension as long as overall gross slope is 1:1. This overall final slope was then scrutinized for preference of bench dimensions to optimize ability to address changes in joint and cleavage patterns, maintaining drainage for storm flows and berm construction where necessary for containing localized rock fall.

Highwall Design Criteria and Mine Plan Considerations

Final bench configuration dimensions proposed for the highwall plan is 40'vertical run and 40'horizontal run, with an outslope bench gradient of 0.25/1.0, H:V, resulting in a 30'horizontal bench run from edge of bench to toe of next higher bench outslope.

Cross Section A'- A included in this study shows a generalized cross section of Red Canyon granite gneiss outcrop presently being mined with projected mining benches and possible operating level elevations drawn in to show that as mining progresses over time, the northern end of the deposit will be reduced in elevation by mining to approximately the 7100'elevation. The mining of the upper elevations of the granite gneiss will result in an eventual one hundred feet of elevation difference between the south end of the mine site and northern end of the current mining footprint, or three mining benches left from south to north end of the current work area. Along the eastern flank of the pit, five benches will be constructed including the top two benches which will terminate the pit limit at the northeastern end of the current work area. Top benches will follow the disturbed zone west across the north end of the current pit disturbance. However, future mining will remove most of the granite gneiss bedrock to the 7100'elevation across the bulk of the existing mine footprint.

A cross section portraying the final bench configuration along the eastern flank of the pit is also included in this report, showing the predominant foliation direction when oriented looking northeast. Foliation dip will appear nearly vertical to high angle to the west. The top benches referenced in the paragraph above will terminate the northernmost reach of disturbance and will turn to a more southern aspect prior to merging with grading operations north of the current mine area.

The bench dimensions were not determined so much by overall slope stability as much as consideration of rock conditions due to localized foliation control, joint, and cleavage within the working faces of the existing mine operation. Primary joint parallels general foliation, which strikes N30E at most measured locations and dips at vertical or near vertical primary to the west. Secondary joint patterns strike North by West and dip vertical or at high degree to the west. Tertiary joint patterns reflect north by east and north by west strikes with lower angle dips. Joint planes to the south and southwest can have serious repercussion regarding bench face stability and may require larger work areas if the joint pattern is found to continue over more than a localized area.

Forty feet of horizontal run on the benches (actually 30' with outslope at 0.25:1, H:V) leaves room to excavate storm flow ditches along the back of the horizontal run of the bench as well as placement of a small berm along the front of the bench run to help control rock fall. In addition, the thirty feet of run will also allow for safer equipment operation in the event that rock removal and or maintenance of the bench condition over time.

Existing Highwall(Southeast) Reclamation

At the southeast corner of the pit area, a highwall exposure of metamorphic sediments can be viewed. The highwall exposure reflects incomplete bench construction and inadequate erosion control with unstable sediments perched at the apex of the highwall exposure. Existing mine benches need to have loose material removed via excavator work and the pile of soil left at the top needs to be cast down and re-graded to keep storm flows away from the highwall edge. While some vegetation removal may be necessary to attain surface material stability, the overall stability of the meta-sediment highwall underlying the soil covering the outcrop is relatively good, in that the meta-sediments appear to be dipping gently away from the highwall edge and should not be a factor regarding long range stability of the area. As mining proceeds northeast from this area, granite gneiss will be the primary rock type that will be mined and benched for final land form left along the eastern flank of the existing mine footprint. (See idealized cross section, Red Canyon Pit). Soils left on the surface can be used to direct storm flows away from the highwall edge and towards the north and west to eventually drain to the southwest in the manner of the main drainage direction south of the mining area.

Summary and Conclusion

The Red Canyon granite gneiss ore body is a Very Good Quality Rock Mass (Hoek-Brown) unit portraying a relatively high degree of weathering resistance and vertical stability. The Red Canyon Pit development will entail a highwall bench system located along the eastern flank of the existing disturbed area. Highwall benches of 40'height and 40'runs will be excavated in the granite gneiss. Face gradient will be approximately 0.25:1, H:V, with some variation expected due to foliation control.

ELEV POINTS

SAMPLE LOCATION

PERMIT BOUNDARY

CROSS SECTION BEARING

SURROUNDING AREA CONTOUR = 30 FT.

PIT CONTOURS = 10 FT.

HIGHWALL SURVEY MAP

RED CANYON QUARRY
Rocky Mountain Minerals

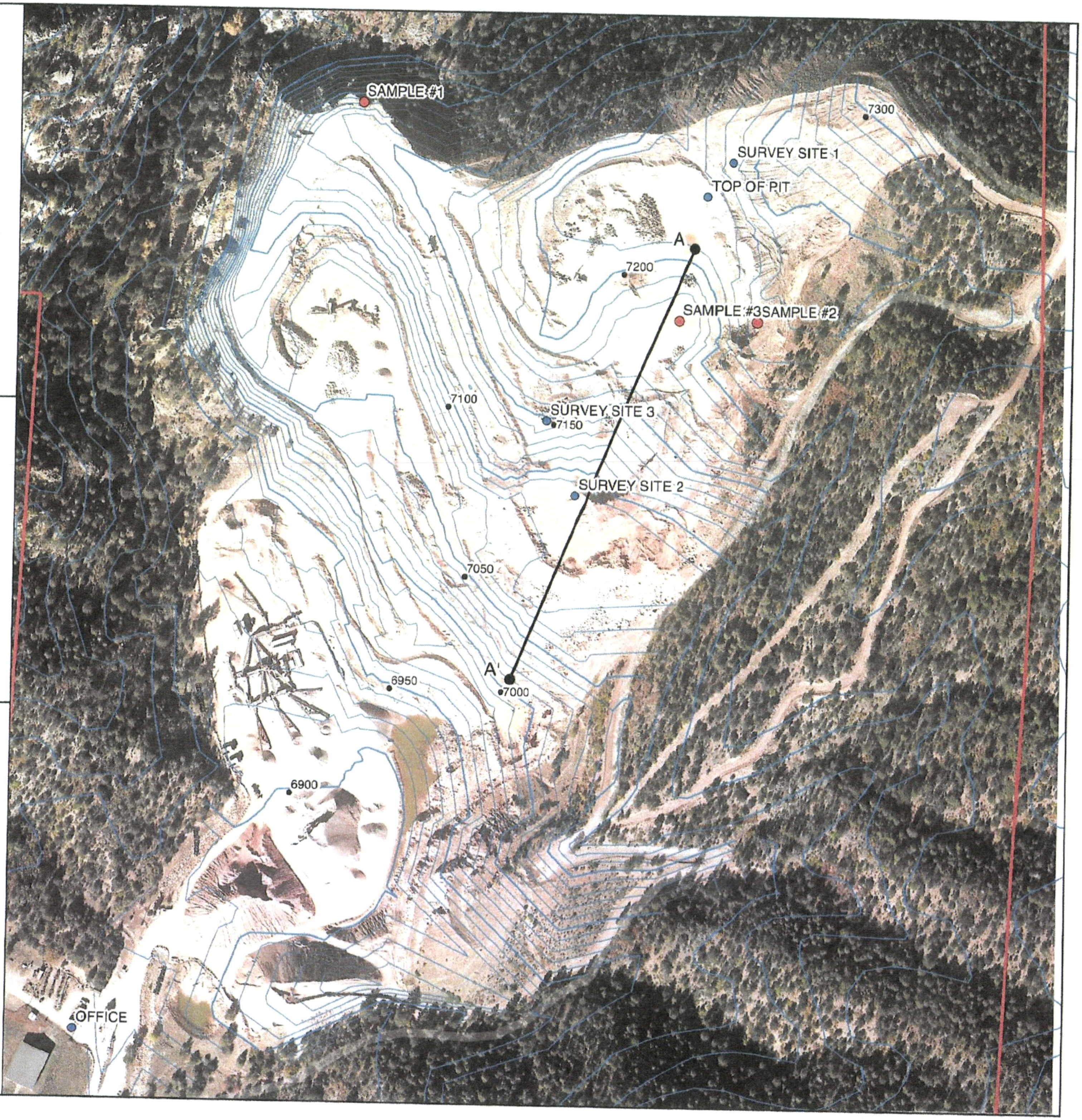
Section 36, T 16S, R 68 W
Section 1, T17S, R 68 W
Fremont County, CO

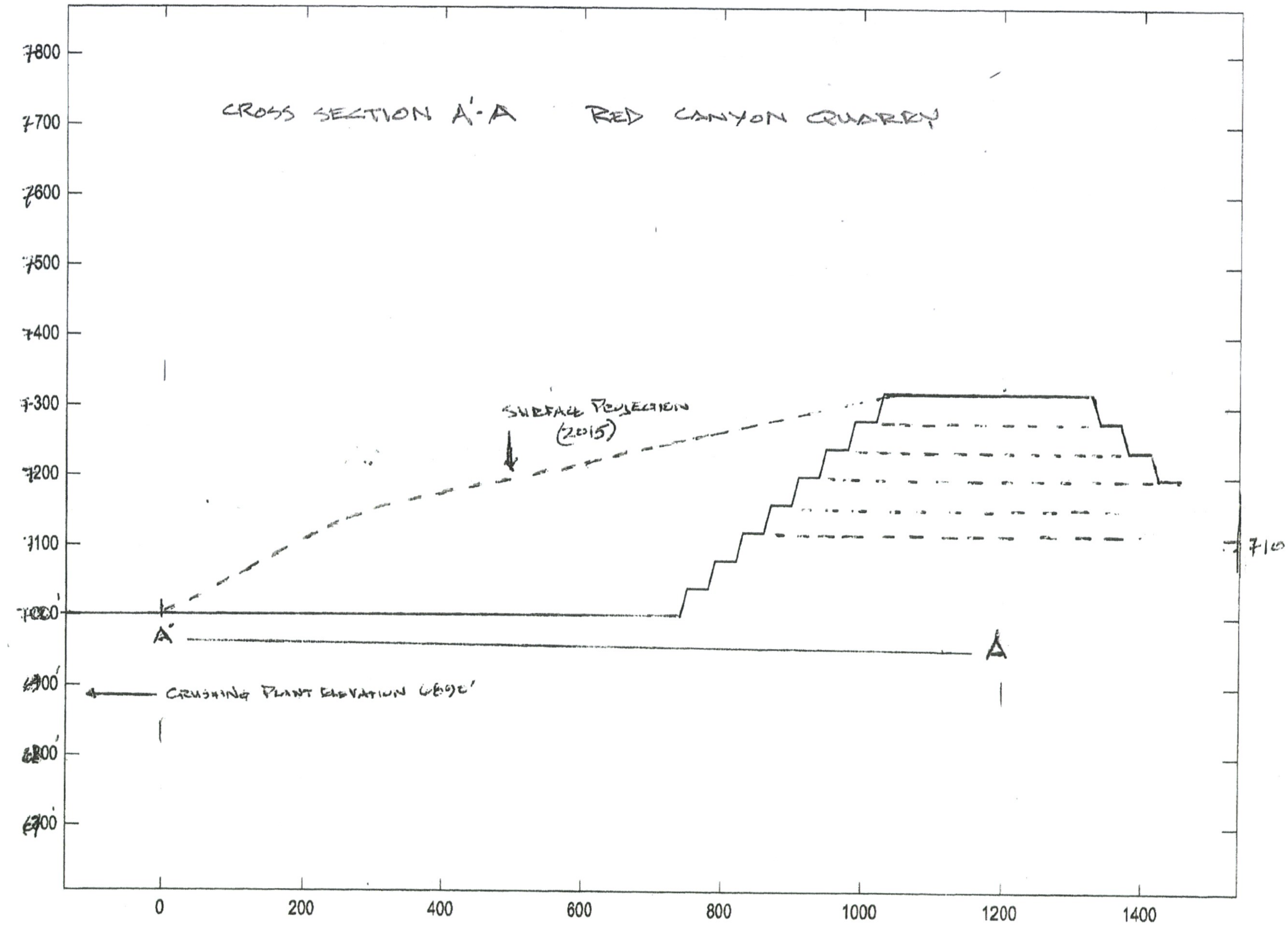
July 28, 2015

Prepared by Azurite Inc.

N

0 250 500 ft.





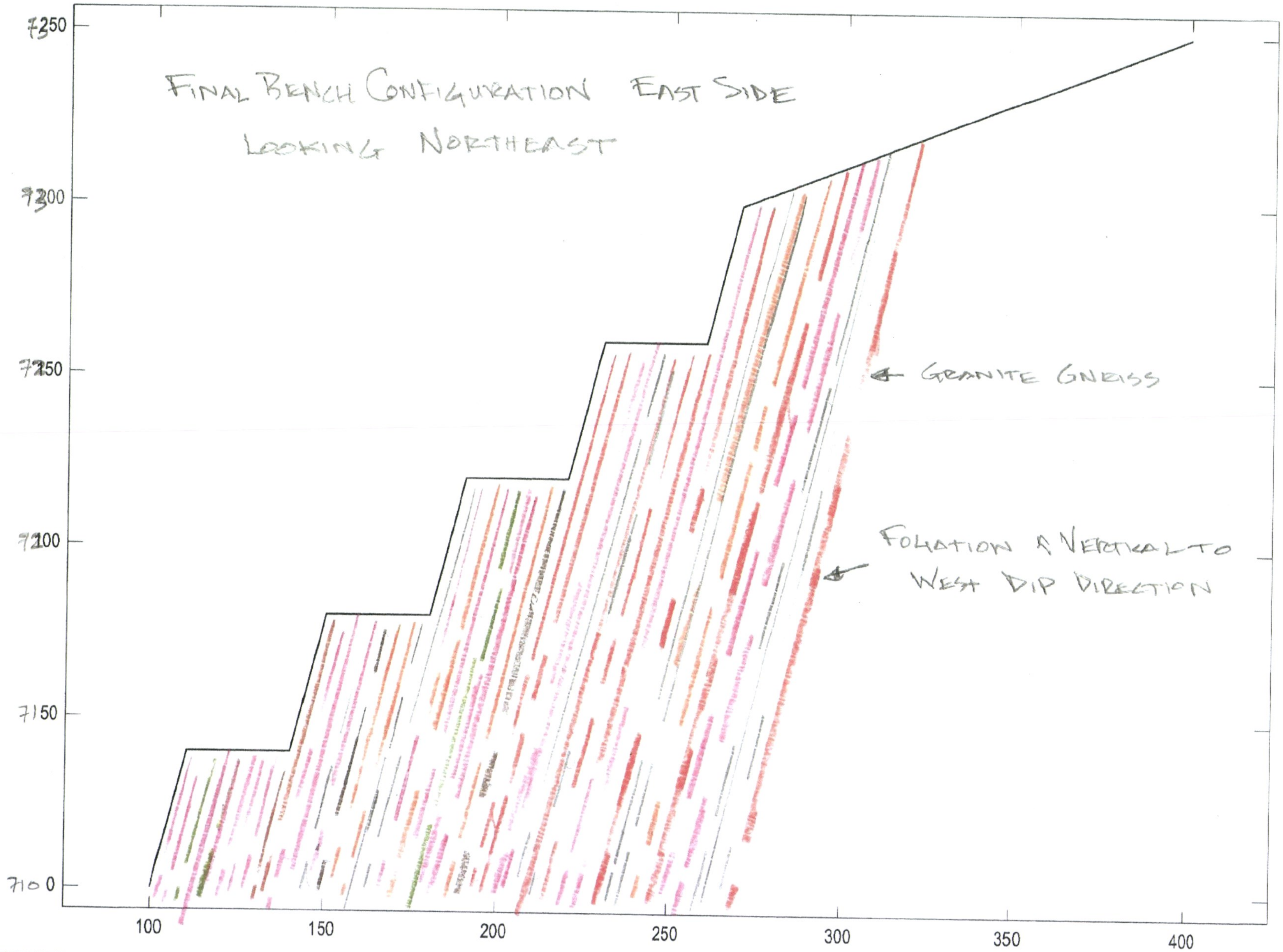
Project Rocky Mtn Materials
RMMSiteSection

8/1/2015

File: C:\Users\Ken\Documents\My Documents\Galena\Test Results\RMMsitesection.gmf

AZURITE, INC

ROCKY MOUNTAIN MATERIALS RED CANYON PIT

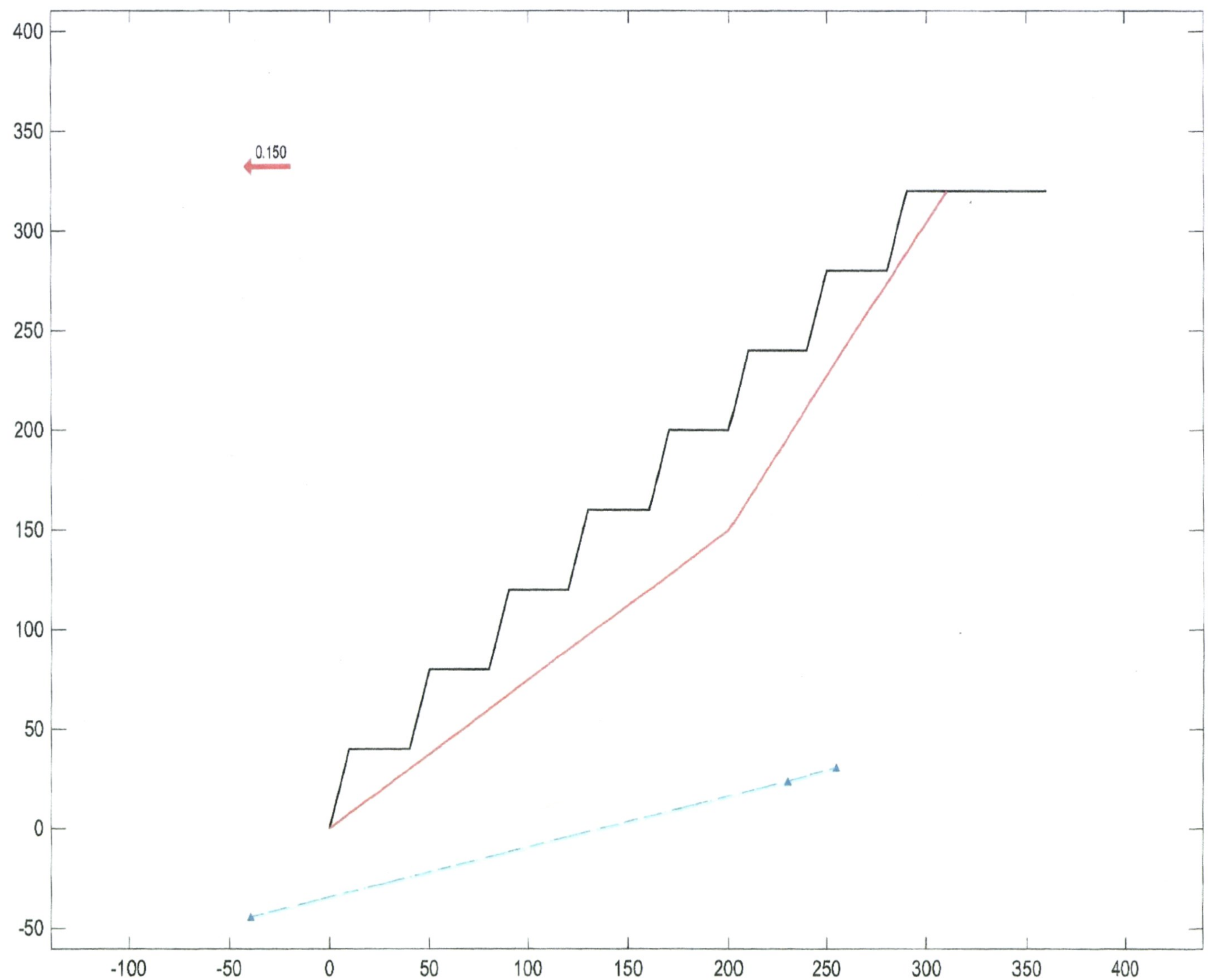


Project RMM xsecloolkingNE
RMM1

Analysis 1

File: C:\Users\Ken\Documents\My Documents\Galena\Test Results\east highwall view.gmf

AZURITE, INC. KSKLCO 8/1/15



GALENA Version 6.10

Material Keys

granite gneiss

Analysis 1

Single Stability Analysis

Method: Spencer-Wright

Surface: Non-Circular

Results

Factor of Safety: 1.51

Interslice Force (Final) Angle: 41.2 °

Edited: 2 Aug 2015 *Processed:* 2 Aug 2015

Project Rocky Mtn Materials

File: C:\Users\Ken\Documents\My Documents\Galena\Test Results\RMMspencermultfailure.gmf

Azurite, Inc.

Project: Rocky Mtn Materials
File: C:\Users\Ken\Documents\My Documents\Galena\Test Results\RMMSarma02.gmf Processed: 02 Aug 2015 12:20:45

DATA: Analysis 1 - RMMSpencerMulti-Surf01

Material Properties (1 material)

Material: 1 (Hoek-Brown(83)) - granite gneiss

m	s	UCS	UnitWeight	Ru
14.6960001	0.1889000	17990	165.00	Auto

Water Properties

Unit weight of water: 9.810 Unit weight of water/medium above ground: 9.810

Material Profiles (1 profile)

Profile: 1 (4 points) Material beneath: 1 - granite gneiss
-100.00 -50.00 -100.00 400.00 0.00 400.00 400.00 400.00

Slope Surface (2 points)

0.00 0.00 320.00 320.00

Phreatic Surface (3 points)

-39.00 -44.00 230.00 24.00 255.00 31.00

Piezometric Surfaces (1 surface)

Surface within profile: 1 (3 points) - granite gneiss
-39.00 -44.00 230.00 24.00 255.00 31.00

Failure Surface

Non-circular surface (3 points)
0.00 0.00 220.00 150.00 320.00 320.00

Earthquake Force

Pseudo-static earthquake (seismic) coefficient: 0.150

RESULTS: Analysis 1 - RMMSpencerMulti-Surf01

Sarma Non-Vertical Slice Method of Analysis - Non-Circular Failure Surface

Factor of Safety: 1.89

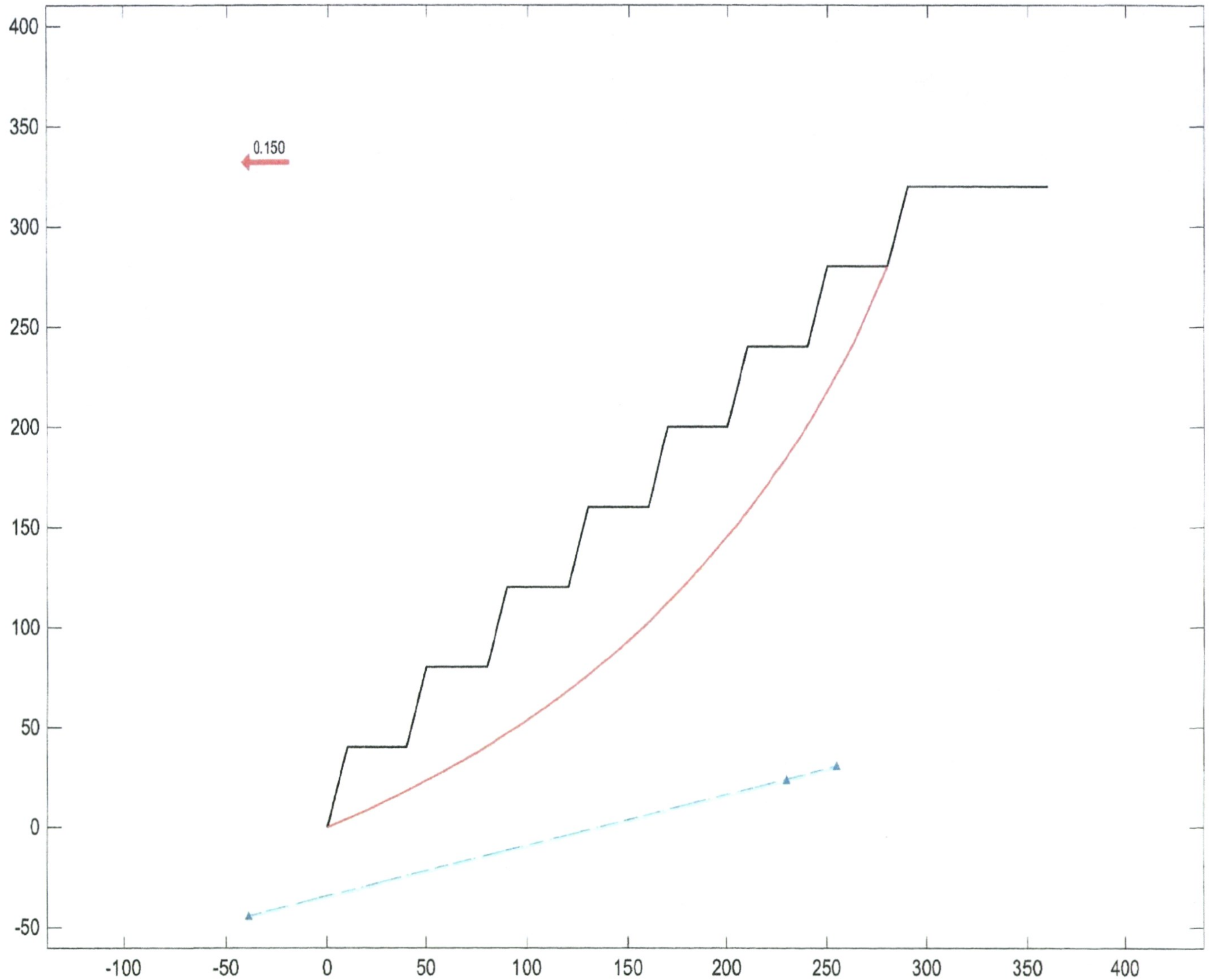
Critical Acceleration (Kc): 0.093

Non-Vertical Slice Geometry (2 slices)

Slice	Left Hand Side X-Top	Y-Top	X-Base	Y-Base	Angle	Length	X-S Area	Base Angle	Width	Length
1	0.00	0.00	0.00	0.00	0.0	0.00	6444.22	34.3	220.00	266.27
2	184.12	184.12	220.00	150.00	-46.4	49.51	4755.78	59.5	100.00	197.23
RHS	320.00	320.00	320.00	320.00	0.0	0.00				
						X-S Area:	11200.00		Path Length:	463.50

Non-Vertical Slice Properties (2 slices)

Slice	Left-Hand-Side Cohesion	Phi	Base Cohesion	Phi	Weight	Total-Extrnl-Force Vert	Horiz	Water-Force Side	Base	Effect-Normal-Stress Side	Base
1	0.00	0.0	2171.57	51.0	1063296.50	0.00	-159494.48	0.00	0.00	0.00	4767.67
2	1788.68	52.3	1326.16	57.6	784703.44	0.00	-117705.52	0.00	0.00	9999.35	172.13
RHS	0.00	0.0						0.00		0.00	
				X-S Weight:	1848000.00						



GALENA Version 6.10

Material Keys

granite gneiss

Analysis 1

Single Stability Analysis

Method: Spencer-Wright

Surface: Non-Circular

Results

Factor of Safety: 1.39

Interslice Force (Final) Angle: 40.2 °

Edited: 29 Jul 2015 *Processed:* 29 Jul 2015

Project Rocky Mtn Materials

File: C:\Users\Ken\Documents\My Documents\Galena\Test Results\RMMcircfailuremodel01.gmf

Azurite, Inc.

Project: Rocky Mtn Materials
File: C:\Users\Ken\Documents\My Documents\Galena\Test Results\RMMcircfailuremodel01.gmf
Processed: 29 Jul 2015 13:24:03

DATA: Analysis 1

Material Properties (1 material)

Material: 1 (Hoek-Brown(83)) - granite gneiss

m	s	UCS	UnitWeight	Ru
14.6960001	0.1889000	16500	165.00	Auto

Water Properties

Unit weight of water: 9.810 Unit weight of water/medium above ground: 9.810

Material Profiles (1 profile)

Profile: 1 (4 points) Material beneath: 1 - granite gneiss

-100.00	-50.00	-100.00	400.00	0.00	400.00	400.00	400.00

Slope Surface (18 points)

0.00	0.00	10.00	40.00	40.00	40.00	50.00	80.00	80.00	80.00
90.00	120.00	120.00	120.00	130.00	160.00	160.00	160.00	170.00	200.00
200.00	200.00	210.00	240.00	240.00	240.00	250.00	280.00	280.00	280.00
290.00	320.00	320.01	320.00	360.00	320.00				

Phreatic Surface (3 points)

-39.00	-44.00	230.00	24.00	255.00	31.00

Piezometric Surfaces (1 surface)

Surface within profile: 1 (3 points) - granite gneiss

-39.00	-44.00	230.00	24.00	255.00	31.00

Failure Surface

Non-circular surface (20 points)

0.00	0.00	19.74	8.34	39.10	17.51	58.05	27.51	76.56	38.31
94.58	49.89	112.10	62.24	129.07	75.32	145.46	89.12	161.24	103.61
176.39	118.76	190.88	134.54	204.68	150.94	217.76	167.90	230.11	185.42
241.69	203.44	252.49	221.95	262.49	240.90	271.66	260.26	280.00	280.00

Earthquake Force

Pseudo-static earthquake (seismic) coefficient: 0.150

RESULTS: Analysis 1

Spencer-Wright Method of Analysis - Non-Circular Failure Surface

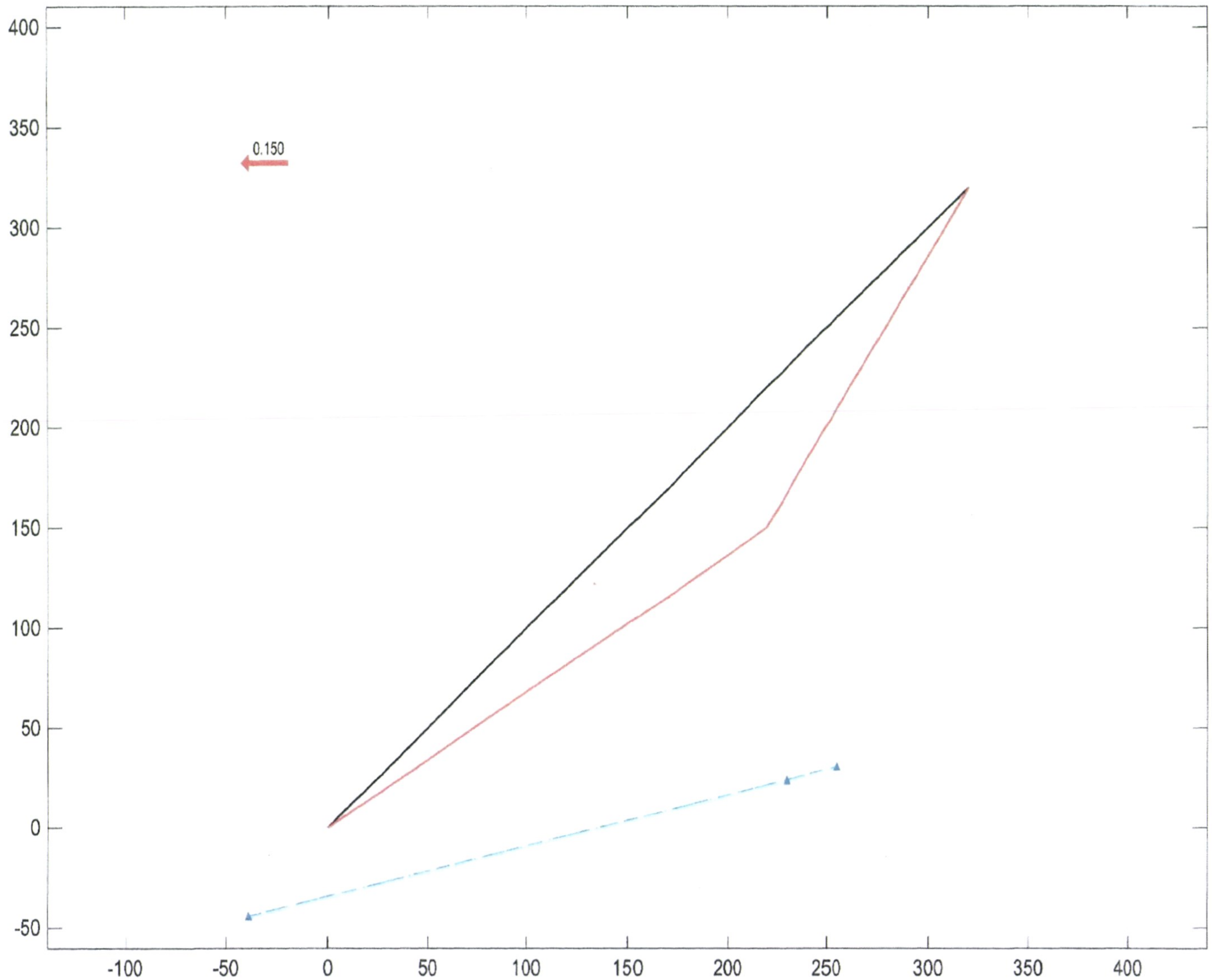
Factor of Safety: 1.39

Final Angle of Interslice Forces: 40.2 degrees
Negative interslice forces exist on one or more slices - examine slice data and consult the GALENA Help utility
Effective stress line of thrust is not within one or more slices - examine slice data and consult the GALENA Help utility

Slice Geometry and Properties (44 slices)

Slice		X-S	Base							PoreWater	Left Hand Side		
	X-Left	Area	Angle	Width	Length	Matl	Cohesion	Phi	Weight	Force	Side Force	l/h	l'/h
1	0.00	44.72	22.9	5.00	5.43	1	1176.06	57.2	7378.61	0.00	0.00	0.00	0.00
2	5.00	134.16	22.9	5.00	5.43	1	2069.28	49.1	22135.83	0.00	12486.81	0.06	0.06
3	10.00	169.21	22.9	4.87	5.29	1	2433.49	46.9	27920.39	0.00	29756.44	0.05	0.05
4	14.87	159.19	22.9	4.87	5.29	1	2340.28	47.5	26267.06	0.00	47680.64	0.09	0.09
5	19.74	194.45	25.3	6.45	7.14	1	2170.80	48.5	32084.20	0.00	65354.97	0.12	0.12
6	26.19	174.72	25.3	6.45	7.14	1	2032.98	49.3	28829.47	0.00	84968.02	0.18	0.18
7	32.65	155.00	25.3	6.45	7.14	1	1892.73	50.3	25574.71	0.00	104135.36	0.25	0.25
8	39.10	154.52	27.8	5.45	6.16	1	2036.47	49.3	25495.53	0.00	122770.41	0.34	0.34
9	44.55	257.65	27.8	5.45	6.16	1	2823.47	44.9	42512.97	0.00	136662.11	0.22	0.22
10	50.00	439.64	27.8	8.05	9.10	1	3112.35	43.7	72541.05	0.00	151034.06	0.16	0.16
11	58.05	460.81	30.3	9.26	10.72	1	2831.94	44.9	76033.08	0.00	171928.66	0.20	0.20
12	67.31	410.83	30.3	9.25	10.72	1	2624.46	45.9	67786.84	0.00	191095.45	0.25	0.25
13	76.56	139.61	32.7	3.44	4.09	1	2393.01	47.2	23035.89	0.00	210772.09	0.31	0.31
14	80.00	239.36	32.7	5.00	5.94	1	2664.19	45.7	39495.09	0.00	216634.00	0.34	0.34
15	85.00	323.30	32.7	5.00	5.94	1	3262.52	43.0	53344.29	0.00	224605.78	0.25	0.25
16	90.00	327.84	32.7	4.58	5.44	1	3500.27	42.1	54094.24	0.00	230725.80	0.20	0.20
17	94.58	587.12	35.2	8.76	10.72	1	3214.01	43.2	96874.27	0.00	235457.94	0.22	0.22
18	103.34	533.02	35.2	8.76	10.72	1	3009.09	44.1	87948.99	0.00	240615.95	0.25	0.25
19	112.10	432.25	37.6	7.90	9.97	1	2687.12	45.6	71321.60	0.00	247495.28	0.29	0.29
20	120.00	601.48	37.6	9.07	11.45	1	3056.70	43.9	99244.49	0.00	251497.89	0.33	0.33

21	129.07	665.68	40.1	8.20	10.71	1	3349.01	42.7	109837.27	0.00	252301.58	0.22	0.22
22	137.27	609.13	40.1	8.19	10.71	1	3150.87	43.5	100507.05	0.00	243314.02	0.24	0.24
23	145.46	491.03	42.6	7.27	9.87	1	2806.40	45.0	81020.24	0.00	237030.08	0.27	0.27
24	152.73	442.50	42.6	7.27	9.87	1	2619.58	45.9	73012.27	0.00	230339.80	0.29	0.29
25	160.00	73.71	42.6	1.24	1.68	1	2579.18	46.2	12161.34	0.00	225992.16	0.32	0.32
26	161.24	652.53	45.0	8.76	12.39	1	2837.05	44.9	107667.77	0.00	225333.25	0.30	0.30
27	170.00	539.54	45.0	6.39	9.04	1	3087.45	43.8	89024.02	0.00	210142.14	0.21	0.21
28	176.39	560.00	47.4	7.25	10.71	1	2740.52	45.3	92400.50	0.00	195369.23	0.23	0.23
29	183.64	502.84	47.4	7.24	10.71	1	2553.10	46.3	82968.41	0.00	178419.70	0.26	0.26
30	190.88	547.57	49.9	9.12	14.16	1	2186.05	48.4	90349.47	0.00	164948.28	0.28	0.28
31	200.00	286.42	49.9	4.68	7.27	1	2212.29	48.2	47259.22	0.00	149780.53	0.31	0.31
32	204.68	398.85	52.4	5.32	8.71	1	2352.26	47.4	65809.58	0.00	141665.50	0.24	0.24
33	210.00	598.54	52.4	7.76	12.71	1	2395.33	47.1	98758.38	0.00	125753.26	0.19	0.19
34	217.76	418.17	54.8	6.18	10.72	1	2053.33	49.2	68998.25	0.00	101365.95	0.22	0.22
35	223.93	364.08	54.8	6.18	10.72	1	1890.16	50.3	60072.91	0.00	83940.52	0.24	0.24
36	230.11	250.87	57.3	4.94	9.15	1	1616.01	52.5	41393.84	0.00	70320.22	0.26	0.26
37	235.05	212.82	57.3	4.95	9.15	1	1482.24	53.7	35115.34	0.00	60963.02	0.27	0.27
38	240.00	69.72	57.3	1.69	3.13	1	1450.87	54.0	11503.95	0.00	54191.79	0.27	0.27
39	241.69	438.92	59.7	8.31	16.49	1	1532.28	53.3	72422.48	0.00	52075.75	0.23	0.23
40	250.00	149.86	59.7	2.49	4.94	1	1642.92	52.3	24726.57	0.00	33233.80	0.10	0.10
41	252.49	266.56	62.2	5.00	10.71	1	1423.96	54.3	43982.70	0.00	26202.97	0.09	0.09
42	257.49	219.19	62.2	5.00	10.71	1	1297.82	55.7	36165.95	0.00	13643.43	0.05	0.05
43	262.49	269.78	64.7	9.17	21.42	1	1037.80	59.3	44513.95	0.00	4683.84	-0.08	-0.08
44	271.66	82.32	67.1	8.34	21.43	1	803.16	65.0	13582.05	0.00	-3407.92	0.32	0.32
RHS	280.00	--------			-------				----------		-702.52	0.00	0.00
X-S Area:		15049.53	Path Length:		407.10		X-S Weight:		2483172.25				



GALENA Version 6.10

Material Keys

granite gneiss

Analysis 1

Multiple Stability Analysis

Method: Spencer-Wright

Surface: Non-Circular

Results

Critical Factor of Safety: 1.61

Interslice Force (Final) Angle: 39.1 °

Edited: 29 Jul 2015 *Processed:* 29 Jul 2015

Project Rocky Mtn Materials
RMMSpencerMulti-Surf01

File: C:\Users\Ken\Documents\My Documents\Galena\Test Results\Rocky Mountain Highwall Model 03.gmf

Azurite, Inc.

Project: Rocky Mtn Materials
File: C:\Users\Ken\Documents\My Documents\Galena\Test Results\RMMspencermultfailure.gmf
Processed: 02 Aug 2015 12:11:58

DATA: Analysis 1

Material Properties (1 material)

Material: 1 (Hoek-Brown(83)) - granite gneiss

m	s	UCS	UnitWeight	Ru
14.6960001	0.1889000	17990	165.00	Auto

Water Properties

Unit weight of water: 9.810 Unit weight of water/medium above ground: 9.810

Material Profiles (1 profile)

Profile: 1 (4 points) Material beneath: 1 - granite gneiss

-100.00	-50.00	-100.00	400.00	0.00	400.00	400.00	400.00

Slope Surface (18 points)

0.00	0.00	10.00	40.00	40.00	40.00	50.00	80.00	80.00	80.00
90.00	120.00	120.00	120.00	130.00	160.00	160.00	160.00	170.00	200.00
200.00	200.00	210.00	240.00	240.00	240.00	250.00	280.00	280.00	280.00
290.00	320.00	320.01	320.00	360.00	320.00				

Phreatic Surface (3 points)

-39.00	-44.00	230.00	24.00	255.00	31.00

Piezometric Surfaces (1 surface)

Surface within profile: 1 (3 points) - granite gneiss

-39.00	-44.00	230.00	24.00	255.00	31.00

Failure Surface

Non-circular surface (3 points)

0.00	0.00	200.00	150.00	310.00	320.00

Earthquake Force

Pseudo-static earthquake (seismic) coefficient: 0.150

RESULTS: Analysis 1

Spencer-Wright Method of Analysis - Non-Circular Failure Surface

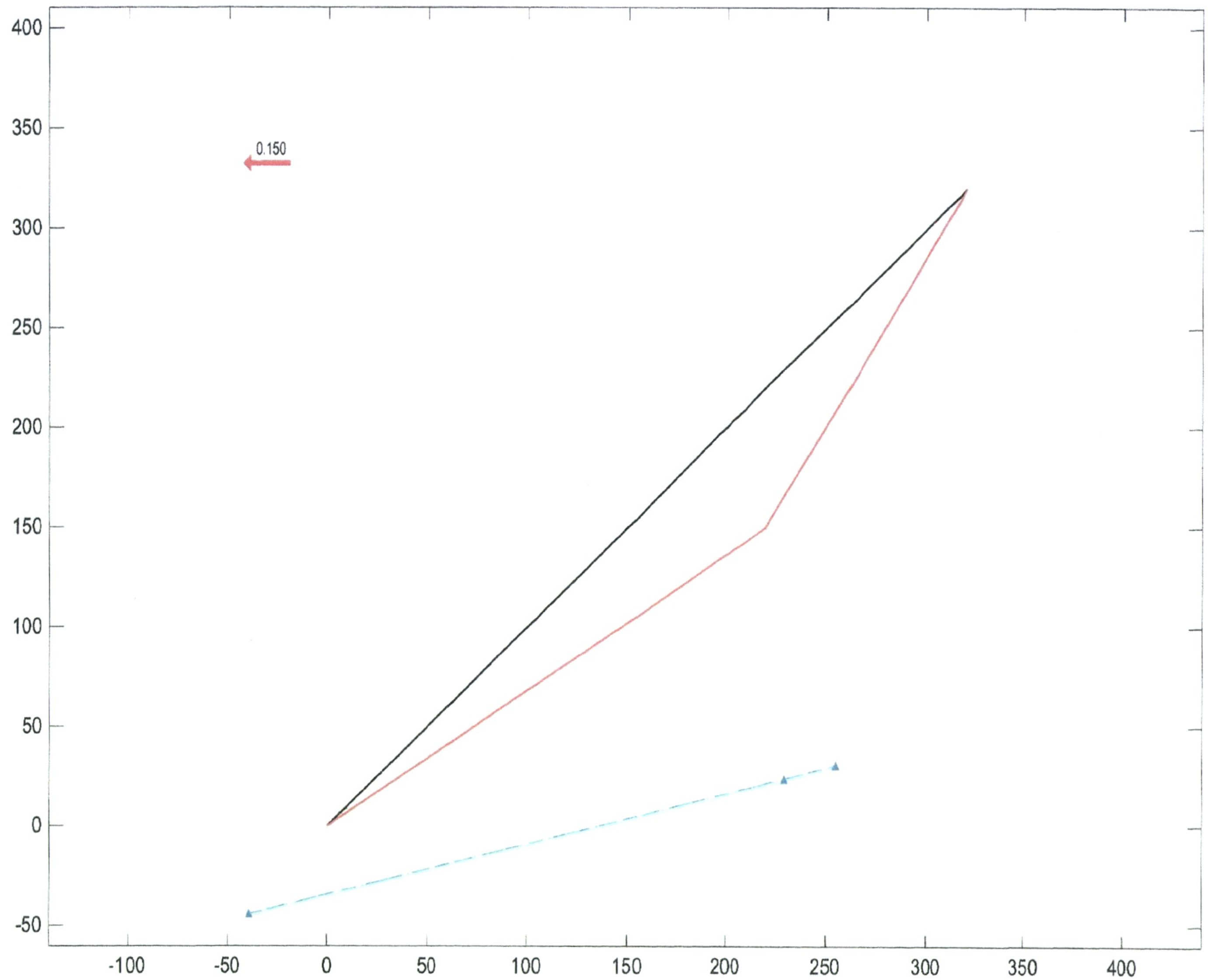
Factor of Safety: 1.51

Final Angle of Interslice Forces: 41.2 degrees
Negative interslice forces exist on one or more slices - examine slice data and consult the GALENA Help utility
Effective stress line of thrust is not within one or more slices - examine slice data and consult the GALENA Help utility

Slice Geometry and Properties (31 slices)

Slice	X-Left	X-S Area	Base Angle	Base Width	Base Length	Base Matl	Base Cohesion	Base Phi	Weight	PoreWater Force	Left Hand Side Side Force	Left Hand Side l/h	Left Hand Side l'/h
1	0.00	162.50	36.9	10.00	12.50	1	1414.35	55.7	26812.50	0.00	0.00	0.00	0.00
2	10.00	287.50	36.9	10.00	12.50	1	1892.41	51.5	47437.50	0.00	11708.36	0.02	0.02
3	20.00	212.50	36.9	10.00	12.50	1	1609.22	53.8	35062.50	0.00	21862.56	0.05	0.05
4	30.00	137.50	36.9	10.00	12.50	1	1315.10	56.8	22687.50	0.00	33199.89	0.11	0.11
5	40.00	262.50	36.9	10.00	12.50	1	1799.18	52.2	43312.50	0.00	44928.66	0.25	0.25
6	50.00	387.50	36.9	10.00	12.50	1	2254.86	49.1	63937.50	0.00	55547.82	0.07	0.07
7	60.00	312.50	36.9	10.00	12.50	1	1984.53	50.9	51562.50	0.00	63282.06	0.11	0.11
8	70.00	237.50	36.9	10.00	12.50	1	1704.80	53.0	39187.50	0.00	72910.10	0.17	0.17
9	80.00	362.50	36.9	10.00	12.50	1	2165.71	49.7	59812.50	0.00	83926.37	0.26	0.26
10	90.00	487.50	36.9	10.00	12.50	1	2602.87	47.2	80437.50	0.00	92340.09	0.11	0.11
11	100.00	412.50	36.9	10.00	12.50	1	2343.10	48.6	68062.50	0.00	96952.24	0.15	0.15
12	110.00	337.50	36.9	10.00	12.50	1	2075.62	50.2	55687.50	0.00	103963.74	0.20	0.20
13	120.00	462.50	36.9	10.00	12.50	1	2517.08	47.6	76312.50	0.00	113010.02	0.27	0.27
14	130.00	587.50	36.9	10.00	12.50	1	2938.90	45.6	96937.50	0.00	118457.97	0.14	0.14
15	140.00	512.50	36.9	10.00	12.50	1	2687.92	46.8	84562.50	0.00	119414.37	0.18	0.18
16	150.00	437.50	36.9	10.00	12.50	1	2430.49	48.1	72187.50	0.00	123157.65	0.23	0.23
17	160.00	562.50	36.9	10.00	12.50	1	2855.90	45.9	92812.50	0.00	129406.13	0.29	0.29
18	170.00	687.50	36.9	10.00	12.50	1	3264.78	44.2	113437.50	0.00	131319.92	0.18	0.18
19	180.00	612.50	36.9	10.00	12.50	1	3021.26	45.2	101062.50	0.00	128194.05	0.22	0.22
20	190.00	537.50	36.9	10.00	12.50	1	2772.25	46.3	88687.50	0.00	128166.56	0.27	0.27
21	200.00	622.73	57.1	10.00	18.41	1	1891.13	51.5	102749.99	0.00	131009.82	0.33	0.33
22	210.00	668.18	57.1	10.00	18.41	1	1968.46	51.0	110249.99	0.00	100785.46	0.18	0.18
23	220.00	513.64	57.1	10.00	18.41	1	1702.33	53.0	84749.99	0.00	66977.67	0.21	0.21

24	230.00	359.09	57.1	10.00	18.41	1	1426.53	55.6	59250.00	0.00	45080.21	0.23	0.23
25	240.00	404.55	57.1	10.00	18.41	1	1508.70	54.7	66750.00	0.00	34166.90	0.19	0.19
26	250.00	450.00	57.1	10.00	18.41	1	1589.99	54.0	74250.00	0.00	20138.63	0.00	0.00
27	260.00	295.45	57.1	10.00	18.41	1	1310.08	56.9	48750.00	0.00	2894.13	-0.69	-0.69
28	270.00	140.91	57.1	10.00	18.41	1	1023.68	61.1	23250.00	0.00	-3853.81	0.85	0.85
29	280.00	186.36	57.1	10.00	18.41	1	1107.71	59.6	30749.98	0.00	-1781.40	4.66	4.66
30	290.00	231.82	57.1	10.00	18.41	1	1192.26	58.4	38250.01	0.00	-2064.75	0.63	0.63
31	300.00	77.27	57.1	10.00	18.41	1	910.95	63.8	12749.99	0.00	-4920.92	0.22	0.22
RHS	310.00	---------			-------				----------		-42.71	0.00	0.00
	X-S Area:	11950.00	Path Length:		452.48		X-S Weight:		1971750.00				



GALENA Version 6.10

Material Keys

granite gneiss

Analysis 1

Multiple Stability Analysis

Method: Sarma

Surface: Non-Circular

Results

Critical Factor of Safety: 1.89

Critical Acceleration (Kc): 0.093

Edited: 2 Aug 2015 *Processed:* 2 Aug 2015

Azurite, Inc.

Project Rocky Mtn Materials
Sarma Multiple

File: C:\Users\Ken\Documents\My Documents\Galena\Test Results\RMMSarma02.gmf

DATA: Analysis 1 - Sarma Multiple

Material Properties (1 material)

Material: 1 (Hoek-Brown(83)) - granite gneiss

m	s	UCS	UnitWeight	Ru
14.6960001	0.1889000	17990	165.00	Auto

Water Properties

Unit weight of water: 9.810 Unit weight of water/medium above ground: 9.810

Material Profiles (1 profile)

Profile: 1 (4 points) Material beneath: 1 - granite gneiss

-100.00	-50.00	-100.00	400.00	0.00	400.00	400.00	400.00

Slope Surface (2 points)

0.00	0.00	320.00	320.00

Phreatic Surface (3 points)

-39.00	-44.00	230.00	24.00	255.00	31.00

Piezometric Surfaces (1 surface)

Surface within profile: 1 (3 points) - granite gneiss

-39.00	-44.00	230.00	24.00	255.00	31.00

Failure Surface

Initial non-circular surface for critical search (3 points)

0.00	0.00	220.00	150.00	320.00	320.00

Earthquake Force

Pseudo-static earthquake (seismic) coefficient: 0.150

Variable Restraints

Horizontal range around X-Left: 0.00 Trial positions within range: 1
Horizontal range around X-Right: 0.00 Trial positions within range: 1
Vertical range around Mid-Point: 4.00 Trial positions within range: 1

RESULTS: Analysis 1 - Sarma Multiple

Sarma Non-Vertical Slice Method of Analysis - Non-Circular Failure Surface

Critical Failure Surface Search using Multiple Surface Generation Techniques

Factor of Safety for initial failure surface approximation: 1.89

There were: 1 successful analyses from a total of 1 trial surfaces

Critical (minimum) Factor of Safety: 1.89

Critical Acceleration (Kc): 0.093

Surface and Results Summary (Lowest 1 Factor of Safety surfaces)

Surface	X-Left	Y-Left	X-Right	Y-Right	Y-Deflection	FoS	Kc
1	0.00	0.00	320.00	320.00	0.00	1.894	0.093

Note: Y-Deflection values are failure surface mid-point vertical distances from the initial failure surface mid-point

Critical Failure Surface (3 points)

0.00	0.00	220.00	150.00	320.00	320.00

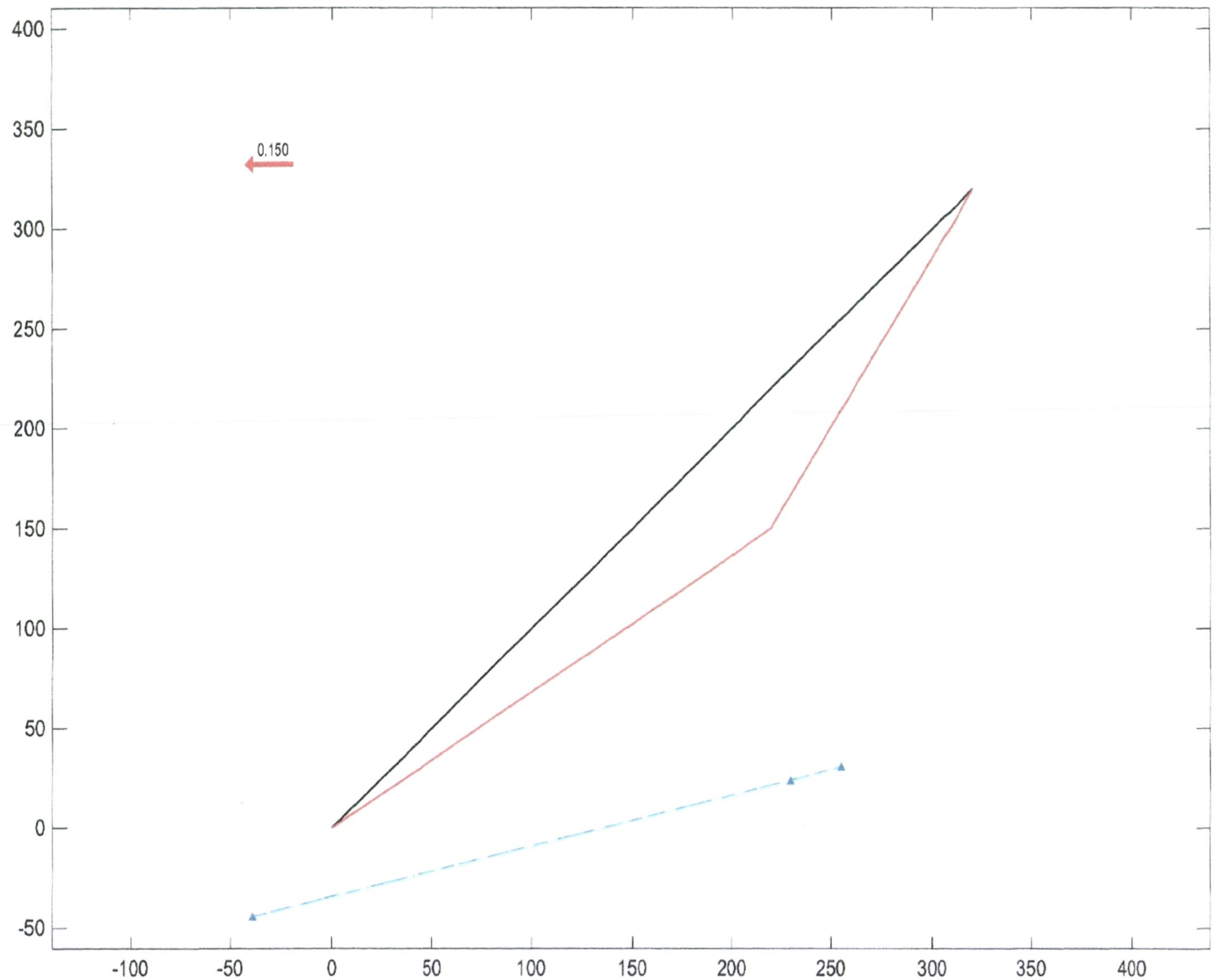
Non-Vertical Slice Geometry (2 slices)

Slice	Left Hand Side						X-S	Base		
	X-Top	Y-Top	X-Base	Y-Base	Angle	Length	Area	Angle	Width	Length
1	0.00	0.00	0.00	0.00	0.0	0.00	6444.22	34.3	220.00	266.27
2	184.12	184.12	220.00	150.00	-46.4	49.51	4755.78	59.5	100.00	197.23
RHS	320.00	320.00	320.00	320.00	0.0	0.00				
						X-S Area:	11200.00		Path Length:	463.50

Non-Vertical Slice Properties (2 slices)

Slice	Left-Hand-Side Cohesion	Left-Hand-Side Phi	Base Cohesion	Base Phi	Weight	Total-Extrnl-Force Vert	Total-Extrnl-Force Horiz	Water-Force Side	Water-Force Base	Effect-Normal-Stress Side	Effect-Normal-Stress Base

1	0.00	0.0	2171.57	51.0	1063296.50	0.00	-159494.48	0.00	0.00	0.00	4767.67
2	1788.68	52.3	1326.16	57.6	784703.44	0.00	-117705.52	0.00	0.00	9999.35	172.13
RHS	0.00	0.0			-----------			0.00		0.00	
				X-S Weight:	1848000.00						



GALENA Version 6.10

Material Keys

granite gneiss

Analysis 1

Single Stability Analysis

Method: Sarma

Surface: Non-Circular

Results

Factor of Safety: 1.89

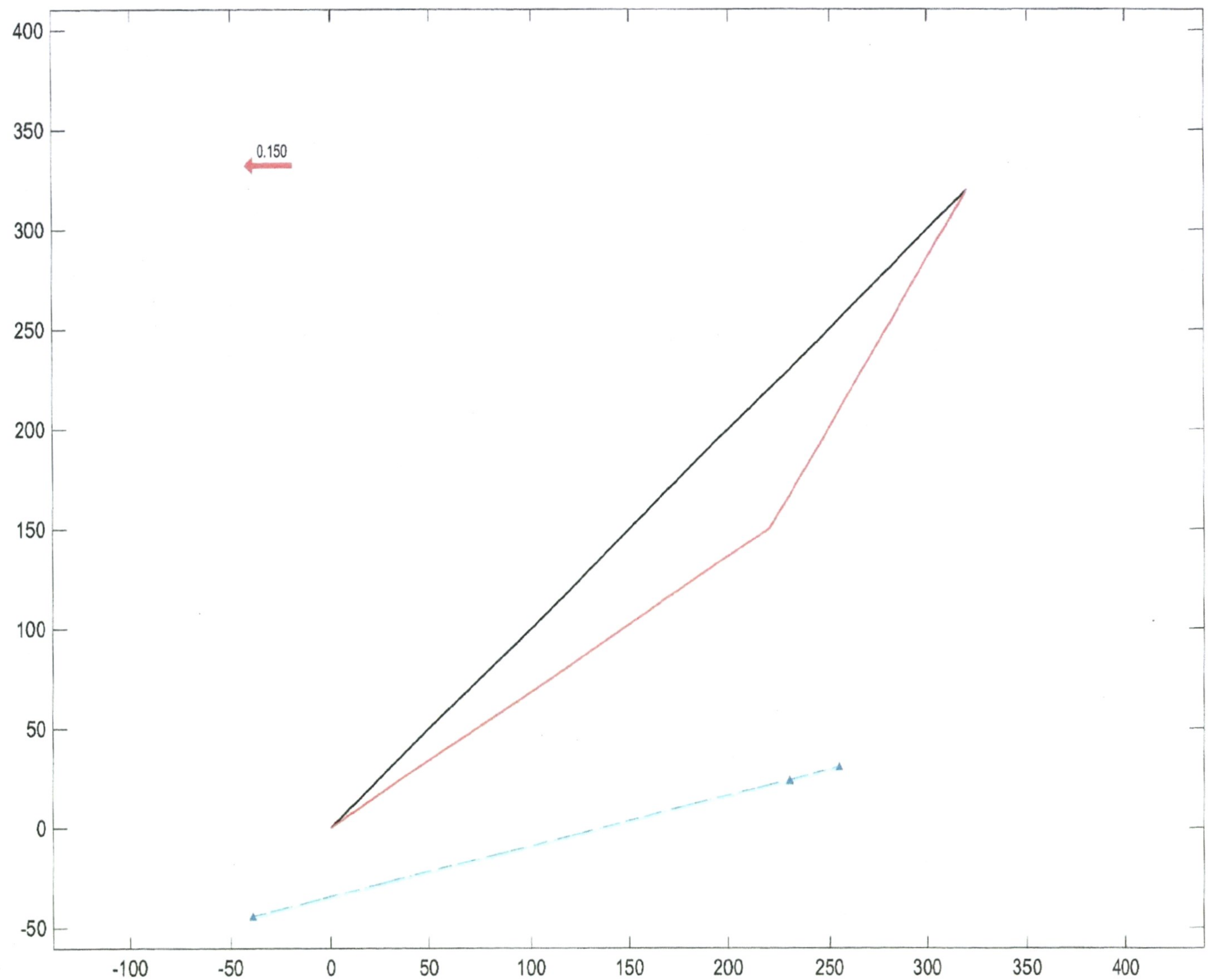
Critical Acceleration (Kc): 0.093

Edited: 29 Jul 2015 *Processed:* 29 Jul 2015

Project Rocky Mtn Materials
RMMSpencerMulti-Surf01

File: C:\Users\Ken\Documents\My Documents\Galena\Test Results\Rocky Mountain Highwall Model 04.gmf

Azurite, Inc.



GALENA Version 6.10

Material Keys

granite gneiss

Analysis 1

Single Stability Analysis

Method: Sarma

Surface: Non-Circular

Results

Factor of Safety:	1.89
Critical Acceleration (Kc):	0.093

Edited: 2 Aug 2015 *Processed* 2 Aug 2015

Project Rocky Mtn Materials
RMMSpencerMulti-Surf01

File: C:\Users\Ken\Documents\My Documents\Galena\Test Results\RMMSarma02.gmf

Azurite, Inc.

Earth Mechanics Institute

Colorado School of Mines

Mining Engineering Department

Client: Azurite

Location: CO Hwy 115

Project Name: Rocky Mountain Minerals

Date: 6/22/2015 Sample ID	Rock Type	Average Length (in)	Average Diameter (in)	Length to Diameter Ratio	Density (lbs/ft^3)	Failure Load (lbs)	Uniaxial Compressive Strength: Failure Stress σ_c (psi)	UCS (2:1) (psi)	UCS (2:1) (MPa)	Notes *(Failure type)*
T1-R1-S1	Metamorphic	4.002	1.938	2.07	162	26,272	8,906	9,019	62.2	*Non-Structural
T1-R3-S11	Metamorphic	3.995	1.939	2.06	161	29,154	9,873	9,996	68.9	*Non-Structural
T1-R3-S12	Metamorphic	3.989	1.936	2.06	162	27,413	9,312	9,428	65.0	*Non-Structural
T2-R5-S19	Metamorphic	4.020	1.940	2.07	163	41,073	13,902	14,084	97.1	Structural
T2-R5-S20	Metamorphic	4.026	1.935	2.08	166	27,681	9,418	9,545	65.8	Non-Structural
T2-R5-S21	Metamorphic	4.066	1.936	2.10	164	36,152	12,287	12,466	85.9	Non-Structural
T3-R2-S7	Metamorphic	3.927	1.933	2.03	166	45,125	15,377	15,543	107.2	Non-Structural
T3-R2-S8	Metamorphic	4.009	1.932	2.08	165	55,116	18,801	19,049	131.3	Non-Structural
T3-R2-S9	Metamorphic	3.840	1.936	1.98	164	56,589	19,223	19,380	133.6	Non-Structural

* Samples failed along foliation planes.

$$UCS_{21correction} = \frac{\sigma_c}{0.88 + 0.222(\frac{d}{l})}$$

Earth Mechanics Institute

Client: Azurite

Location: CO Hwy 115

Project Name: Rocky Mountain Minerals

Colorado School of Mines

Mining Engineering Department

Date: 6/24/2015	Rock Type	Load Direction	Average Length	Average Diameter	Failure Load	Indirect (Brazilian) Tensile Strength		Notes *(Failure type)*
ID			(in)	(in)	(lbs)	(psi)	(MPa)	
T1-R1-S2	Metamorphic	Parallel	1.29	1.933	1,397	355	2.4	*Non-Structural
T1-R1-S4	Metamorphic	Parallel	1.22	1.933	1,444	389	2.7	*Non-Structural
T1-R3-S11	Metamorphic	Parallel	1.08	1.932	1,048	321	2.2	*Non-Structural
T1-R1-S1	Metamorphic	Perpendicular	1.06	1.933	1,456	453	3.1	Non-Structural
T1-R1-S3	Metamorphic	Perpendicular	1.01	1.931	1,666	547	3.8	Non-Structural
T1-R1-S5	Metamorphic	Perpendicular	1.15	1.933	1,926	550	3.8	Non-Structural
T2-R5-S25	Metamorphic	Parallel	1.24	1.931	2,190	583	4.0	*Non-Structural
T2-R5-S26	Metamorphic	Parallel	1.24	1.932	2,216	589	4.1	*Non-Structural
T2-R5-S27	Metamorphic	Parallel	1.24	1.932	2,381	635	4.4	*Non-Structural
T2-R5-S22	Metamorphic	Perpendicular	1.22	1.931	4,053	1,097	7.6	Non-Structural
T2-R5-S23	Metamorphic	Perpendicular	1.24	1.932	3,494	925	6.4	Non-Structural
T2-R5-S24	Metamorphic	Perpendicular	1.21	1.931	2,389	653	4.5	Structural
T3-R2-S9	Metamorphic	Parallel	1.17	1.930	3,048	860	5.9	Non-Structural
T3-R4-S13	Metamorphic	Parallel	1.27	1.932	941	244	1.7	Non-Structural
T3-R4-S15	Metamorphic	Parallel	1.17	1.928	3,062	863	5.9	Non-Structural
T3-R2-S10	Metamorphic	Perpendicular	0.96	1.931	2,009	688	4.7	Non-Structural
T3-R4-S12	Metamorphic	Perpendicular	0.94	1.933	1,571	548	3.8	Non-Structural
T3-R4-S14	Metamorphic	Perpendicular	0.916	1.932	2,162	778	5.4	Non-Structural

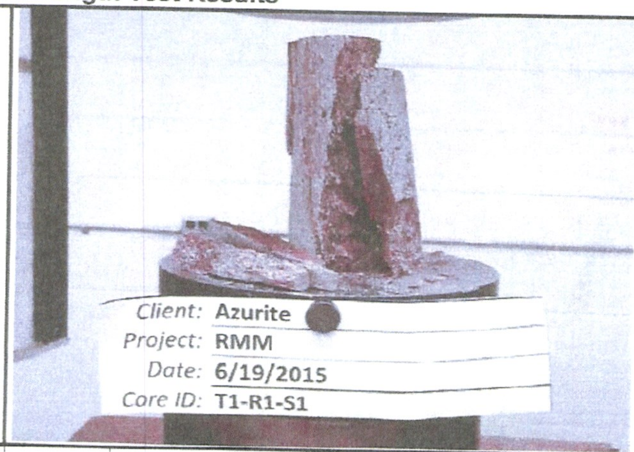
* Samples failed in direction of foliation planes

Earth Mechanics Institute

Mining Engineering Department, CSM

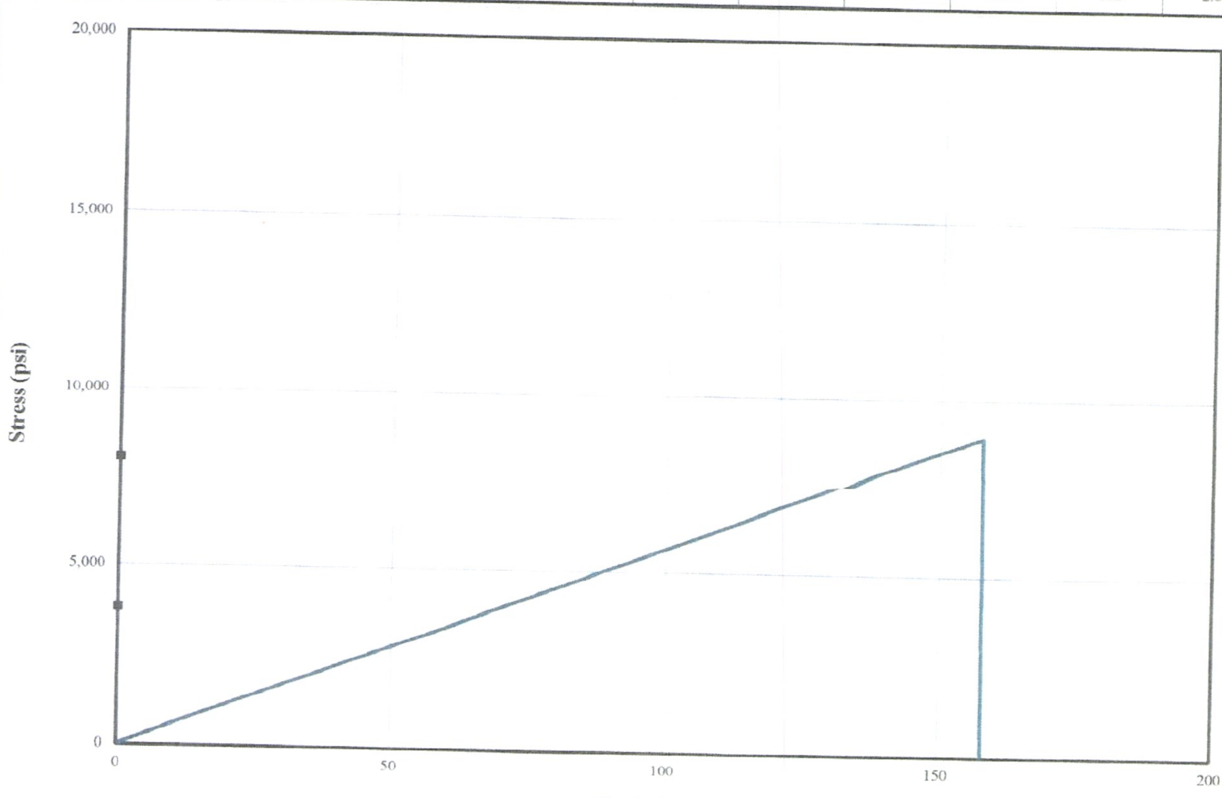
Uniaxial Compressive Strength Test Results

Client: Azurite
Project: Rocky Mountain Minerals
Location: CO Hwy 115
Rock Type: Metamorphic
Rock Name:
Characteristics: Core axis verticle
Core ID: T1-R1-S1
File Name: T1-R1-S1_UCS
Test Performed By: PK
Date Tested: 6/19/2015
Data Reduced By: PK
Date Reduced: 6/19/2015



Core Length		Diameter		L/D Ratio	Failure Load	Failure Stress	UCS 2:1		Failure Mode
in	cm	in	cm		lbs	psi	psi	MPa	
4.002	10.165	1.938	4.923	2.07	26,272	8,906	9,019	62.2	Non-Structural, along foliation plane

P-Wave		S-Wave		Dynamic E		Dynamic ν	Static E		Static ν	Density	
ft/sec	m/sec	ft/sec	m/sec	ksi	GPa		ksi	GPa		lb/ft³	g/cm³
N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	162	2.60

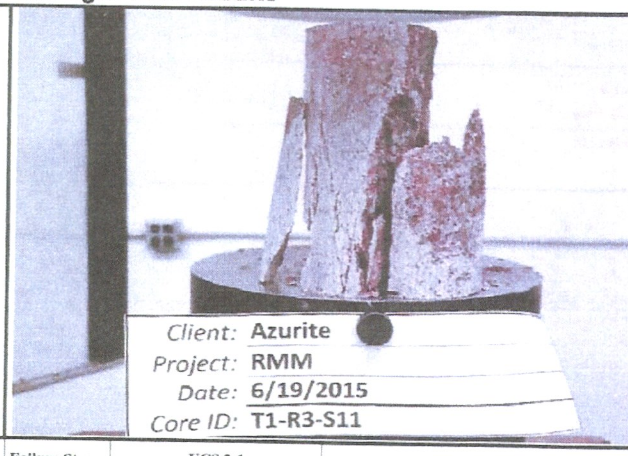


Earth Mechanics Institute

Mining Engineering Department, CSM

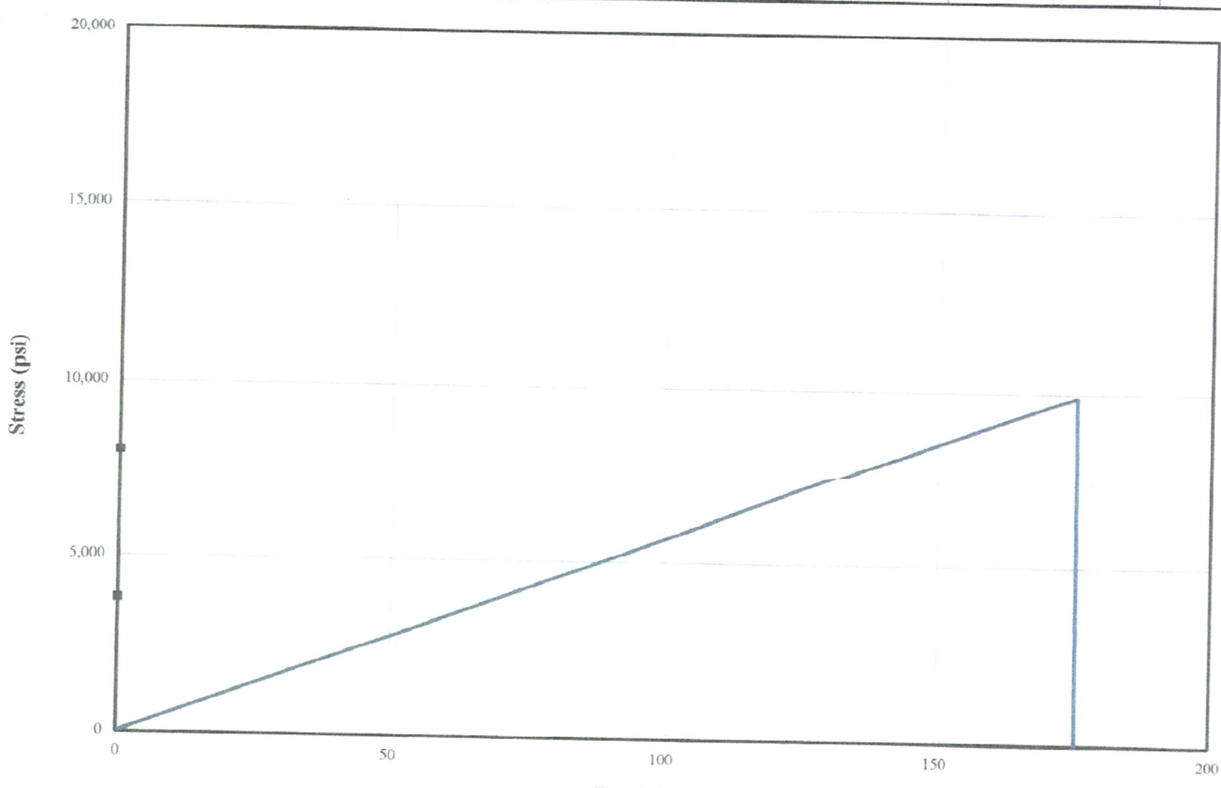
Uniaxial Compressive Strength Test Results

Client: Azurite
Project: Rocky Mountain Minerals
Location: CO Hwy 115
Rock Type: Metamorphic
Rock Name:
Characteristics: Core axis verticle
Core ID: T1-R3-S11
File Name: T1-R3-S11_UCS
Test Performed By: PK
Date Tested: 6/19/2015
Data Reduced By: PK
Date Reduced: 6/19/2015



Core Length		Diameter		L/D Ratio	Failure Load	Failure Stress	UCS 2:1		Failure Mode
in	cm	in	cm		lbs	psi	psi	MPa	
3.995	10.147	1.939	4.925	2.06	29,154	9,873	9,996	68.9	Non-Structural, along foliation plane

P-Wave		S-Wave		Dynamic E		Dynamic v	Static E		Static v	Density	
ft/sec	m/sec	ft/sec	m/sec	ksi	GPa		ksi	GPa		lb/ft³	g/cm³
N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	161	2.58

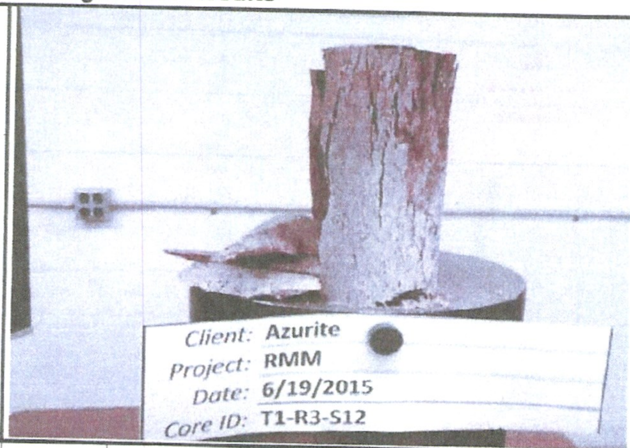


Earth Mechanics Institute

Mining Engineering Department, CSM

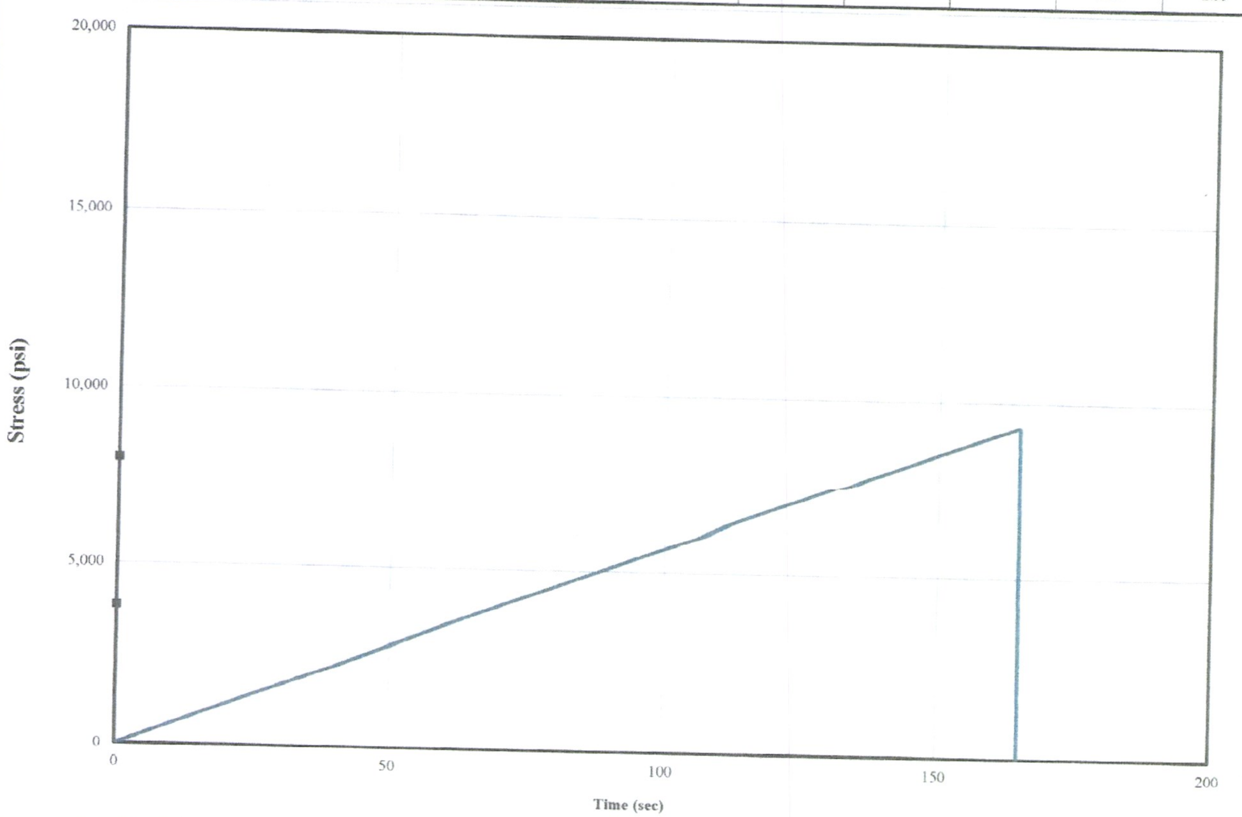
Uniaxial Compressive Strength Test Results

Client: Azurite
Project: Rocky Mountain Minerals
Location: CO Hwy 115
Rock Type: Metamorphic
Rock Name:
Characteristics: Core axis verticle
Core ID: T1-R3-S12
File Name: T1-R3-S12_UCS
Test Performed By: PK
Date Tested: 6/19/2015
Data Reduced By: PK
Date Reduced: 6/19/2015



Core Length		Diameter		L/D Ratio	Failure Load	Failure Stress	UCS 2:1		Failure Mode
in	cm	in	cm		lbs	psi	psi	MPa	
3.989	10.132	1.936	4.917	2.06	27,413	9,312	9,428	65.0	Non-Structural, along foliation plane

P-Wave		S-Wave		Dynamic E		Dynamic v	Static E		Static v	Density	
ft/sec	m/sec	ft/sec	m/sec	ksi	GPa		ksi	GPa		lb/ft^3	g/cm^3
N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	162	2.59

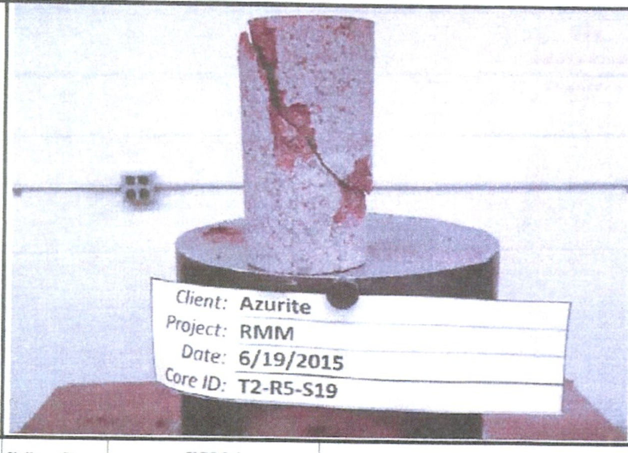


Earth Mechanics Institute

Mining Engineering Department, CSM

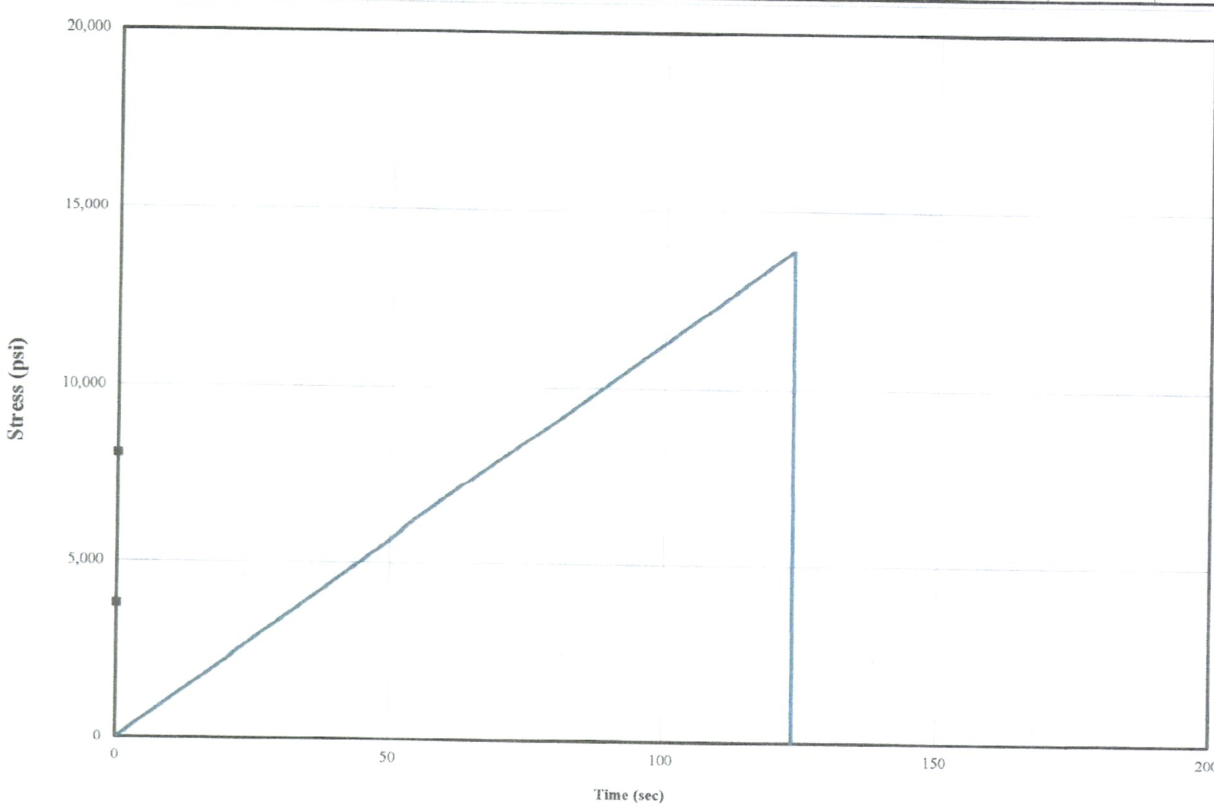
Uniaxial Compressive Strength Test Results

Client: Azurite
Project: Rocky Mountain Minerals
Location: CO Hwy 115
Rock Type: Metamorphic
Rock Name:
Characteristics: Core axis verticle
Core ID: T2-R5-S19
File Name: T2-R5-S19_UCS
Test Performed By: PK
Date Tested: 6/19/2015
Data Reduced By: PK
Date Reduced: 6/19/2015



Core Length		Diameter		L/D Ratio	Failure Load	Failure Stress	UCS 2:1		Failure Mode
in	cm	in	cm		lbs	psi	psi	MPa	
4.020	10.211	1.940	4.926	2.07	41,073	13,902	14,084	97.1	Structural

P-Wave		S-Wave		Dynamic E		Dynamic v	Static E		Static v	Density	
ft/sec	m/sec	ft/sec	m/sec	ksi	GPa		ksi	GPa		lb/ft^3	g/cm^3
N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	163	2.62

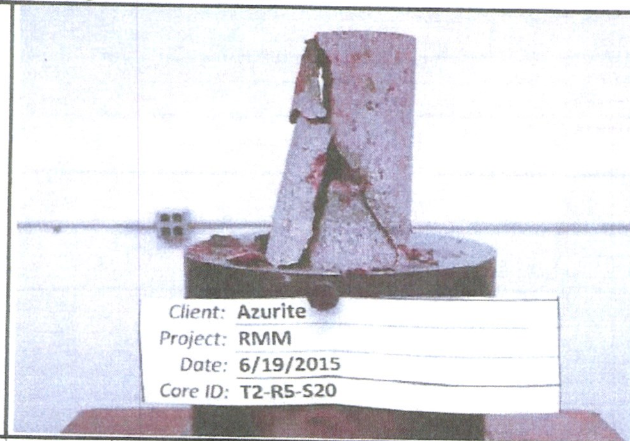


Earth Mechanics Institute

Mining Engineering Department, CSM

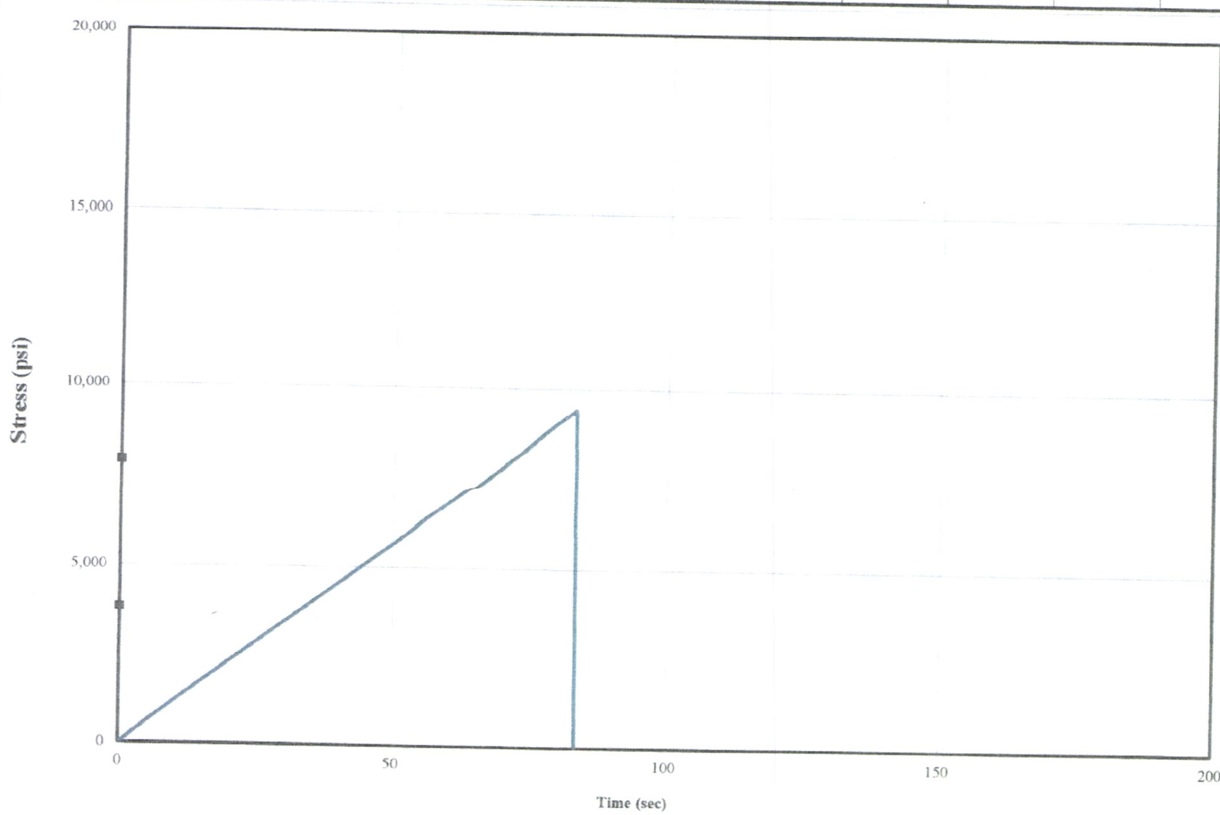
Uniaxial Compressive Strength Test Results

Client: Azurite
Project: Rocky Mountain Minerals
Location: CO Hwy 115
Rock Type: Metamorphic
Rock Name:
Characteristics: Core axis verticle
Core ID: T2-R5-S20
File Name: T2-R5-S20_UCS
Test Performed By: PK
Date Tested: 6/19/2015
Data Reduced By: PK
Date Reduced: 6/19/2015



Core Length		Diameter		L/D Ratio	Failure Load	Failure Stress	UCS 2:1		Failure Mode
in	cm	in	cm		lbs	psi	psi	MPa	
4.026	10.226	1.935	4.914	2.08	27,681	9,418	9,545	65.8	Non-Structural

P-Wave		S-Wave		Dynamic E		Dynamic v	Static E		Static v	Density	
ft/sec	m/sec	ft/sec	m/sec	ksi	GPa		ksi	GPa		lb/ft^3	g/cm^3
N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	166	2.66

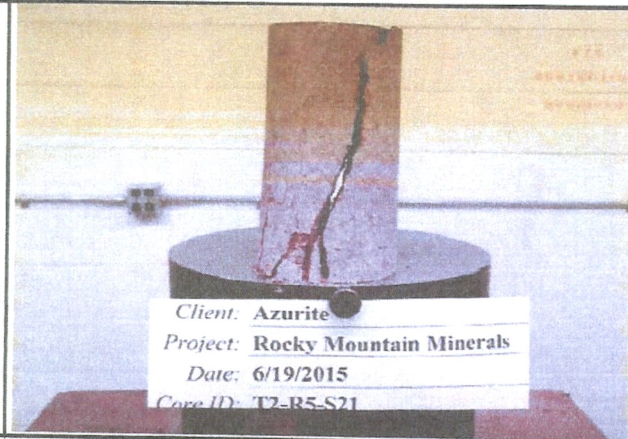


Earth Mechanics Institute

Mining Engineering Department, CSM

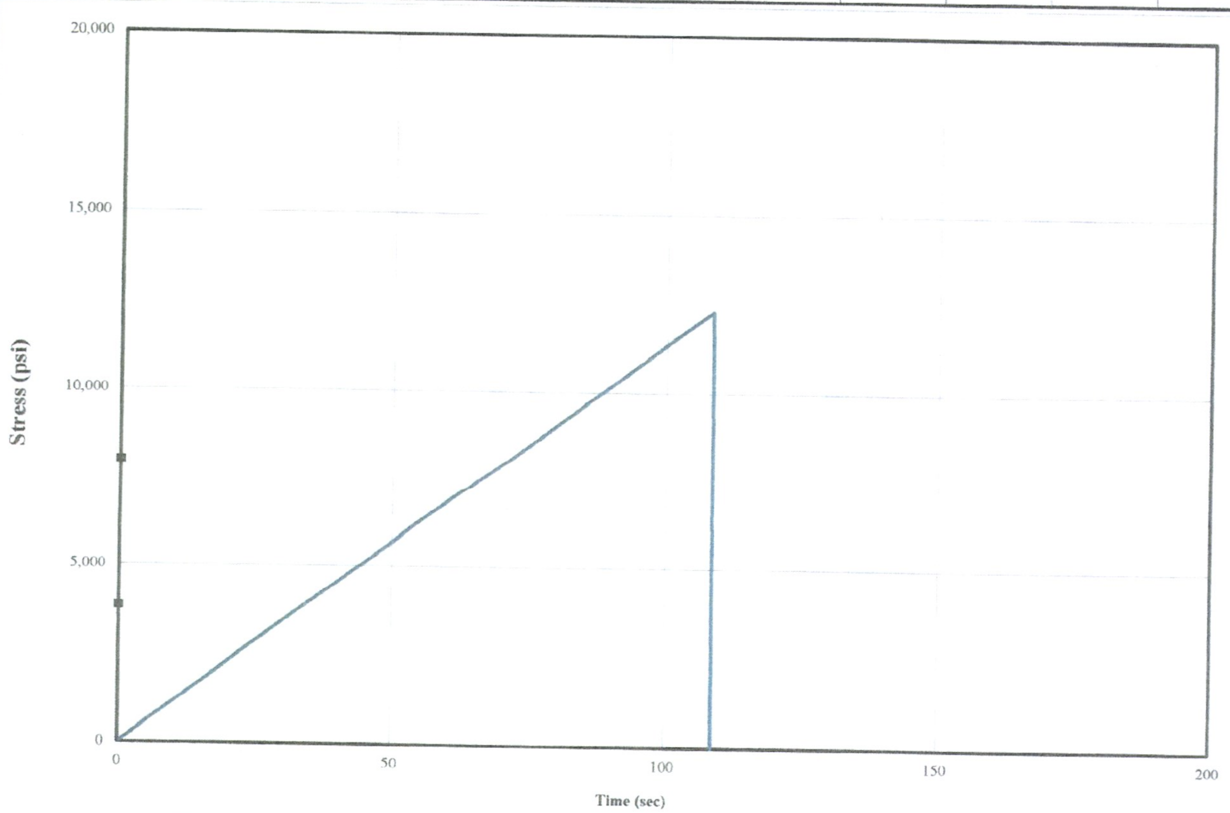
Uniaxial Compressive Strength Test Results

Client: Azurite
Project: Rocky Mountain Minerals
Location: CO Hwy 115
Rock Type: Metamorphic
Rock Name:
Characteristics: Core axis verticle
Core ID: T2-R5-S21
File Name: T2-R5-S21_UCS
Test Performed By: PK
Date Tested: 6/19/2015
Data Reduced By: PK
Date Reduced: 6/19/2015



Core Length		Diameter		L/D Ratio	Failure Load	Failure Stress	UCS 2:1		Failure Mode
in	cm	in	cm		lbs	psi	psi	MPa	
4.066	10.328	1.936	4.916	2.10	36,152	12,287	12,466	85.9	Non-Structural

P-Wave		S-Wave		Dynamic E		Dynamic v	Static E		Static v	Density	
ft/sec	m/sec	ft/sec	m/sec	ksi	GPa		ksi	GPa		lb/ft³	g/cm³
N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	164	2.62

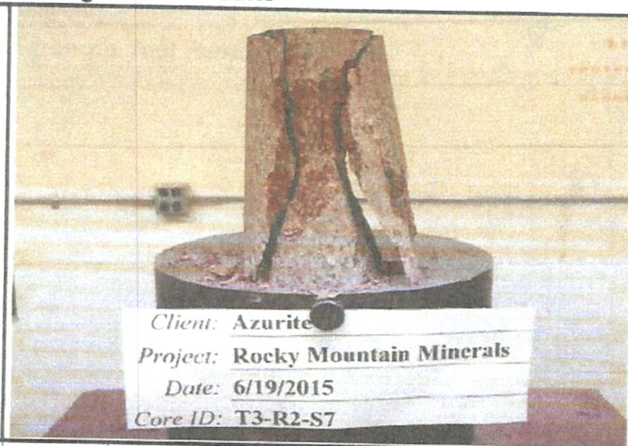


Earth Mechanics Institute

Mining Engineering Department, CSM

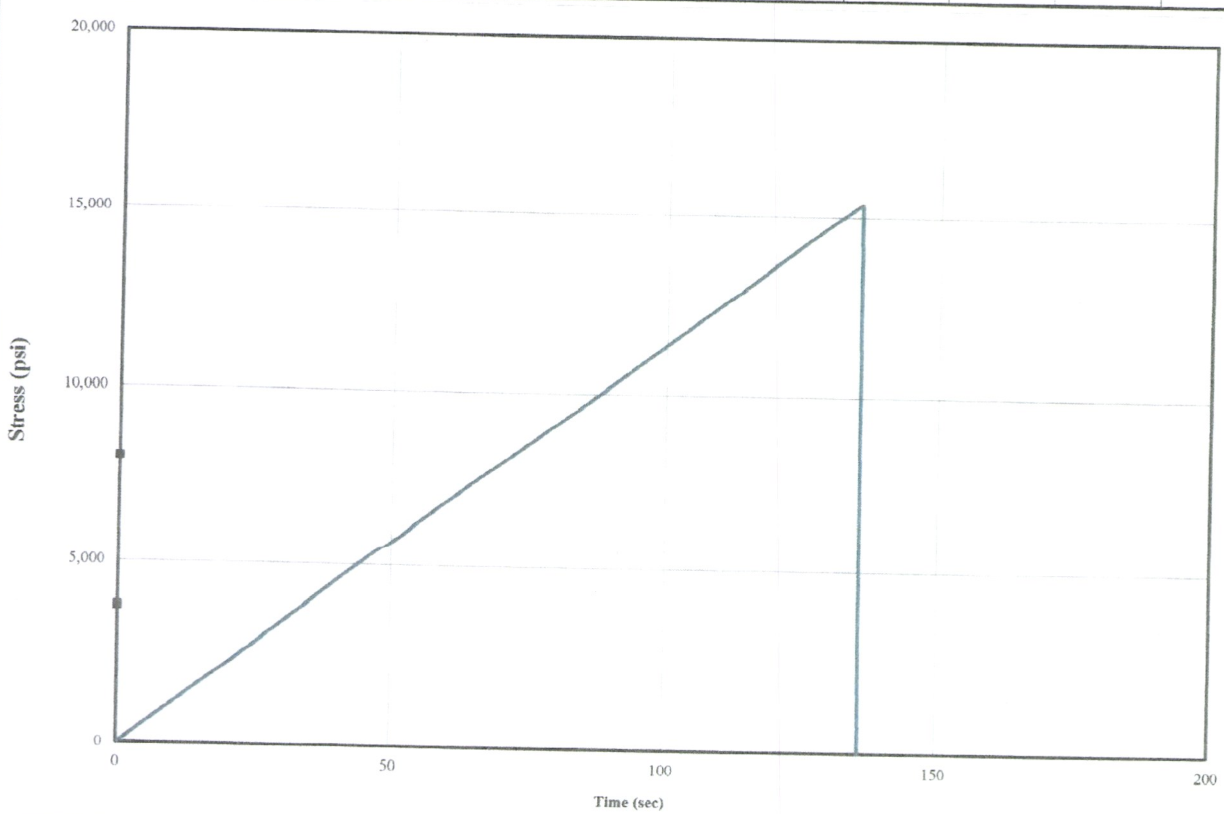
Uniaxial Compressive Strength Test Results

Client: Azurite
Project: Rocky Mountain Minerals
Location: CO Hwy 115
Rock Type: Metamorphic
Rock Name:
Characteristics: Core axis verticle
Core ID: T3-R2-S7
File Name: T3-R2-S7_UCS
Test Performed By: PK
Date Tested: 6/19/2015
Data Reduced By: PK
Date Reduced: 6/19/2015



Core Length		Diameter		L/D Ratio	Failure Load	Failure Stress	UCS 2:1		Failure Mode
in	cm	in	cm		lbs	psi	psi	MPa	
3.927	9.975	1.933	4.910	2.03	45,125	15,377	15,543	107.2	Non-Structural

P-Wave		S-Wave		Dynamic E		Dynamic v	Static E		Static v	Density	
ft/sec	m/sec	ft/sec	m/sec	ksi	GPa		ksi	GPa		lb/ft^3	g/cm^3
N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	166	2.66

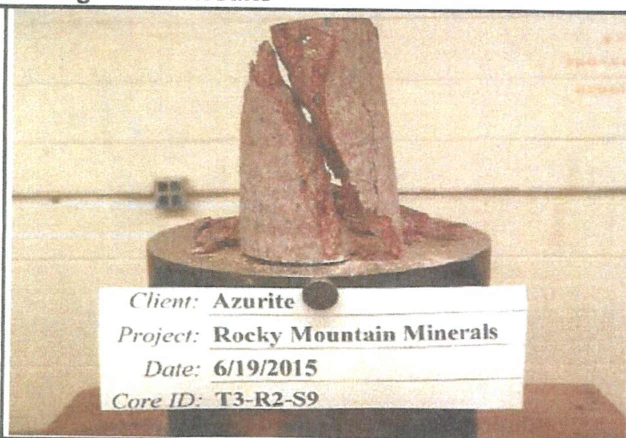


Earth Mechanics Institute

Mining Engineering Department, CSM

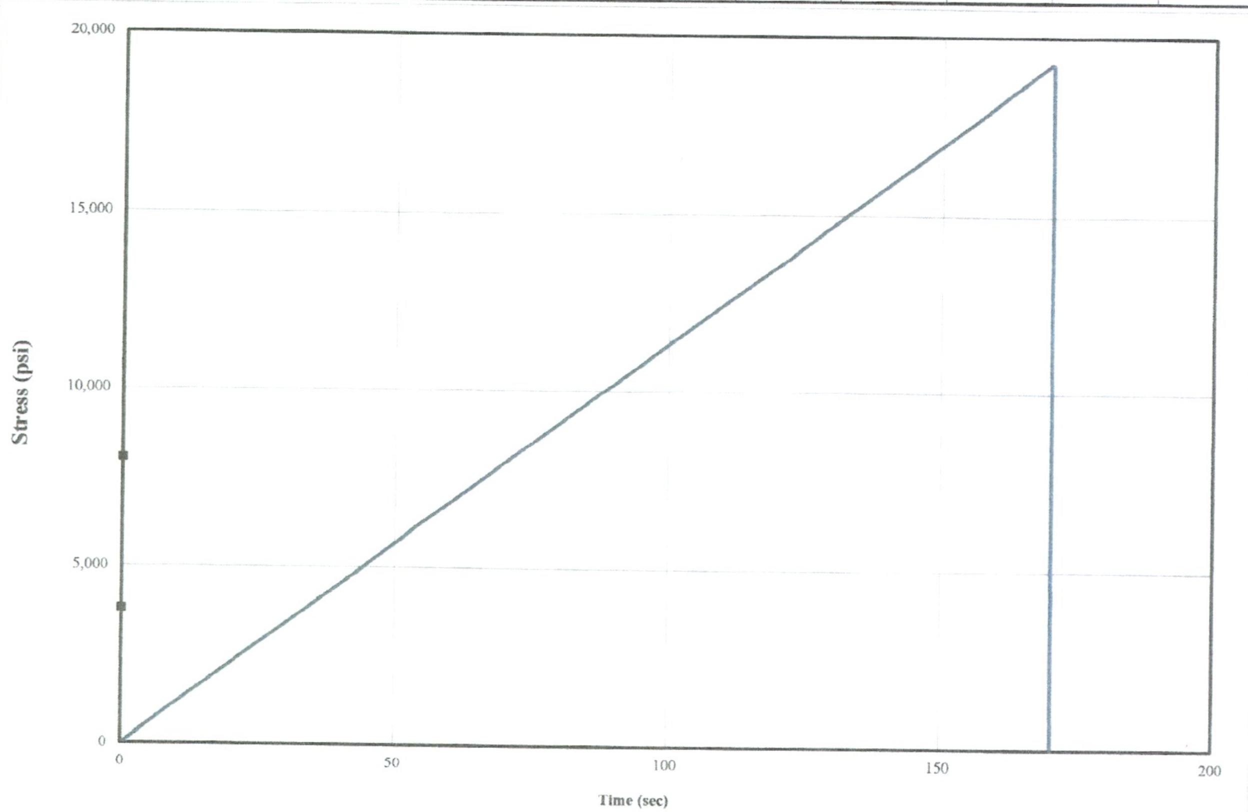
Uniaxial Compressive Strength Test Results

Client: Azurite
Project: Rocky Mountain Minerals
Location: CO Hwy 115
Rock Type: Metamorphic
Rock Name:
Characteristics: Core axis verticle
Core ID: T3-R2-S9
File Name: T3-R2-S9_UCS
Test Performed By: PK
Date Tested: 6/19/2015
Data Reduced By: PK
Date Reduced: 6/19/2015



Core Length		Diameter		L/D Ratio	Failure Load	Failure Stress	UCS 2:1		Failure Mode
in	cm	in	cm		lbs	psi	psi	MPa	
3.840	9.754	1.936	4.917	1.98	56,589	19,223	19,380	133.6	Non-Structural

P-Wave		S-Wave		Dynamic E		Dynamic v	Static E		Static v	Density	
ft/sec	m/sec	ft/sec	m/sec	ksi	GPa		ksi	GPa		lb/ft^3	g/cm^3
N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	164	2.63



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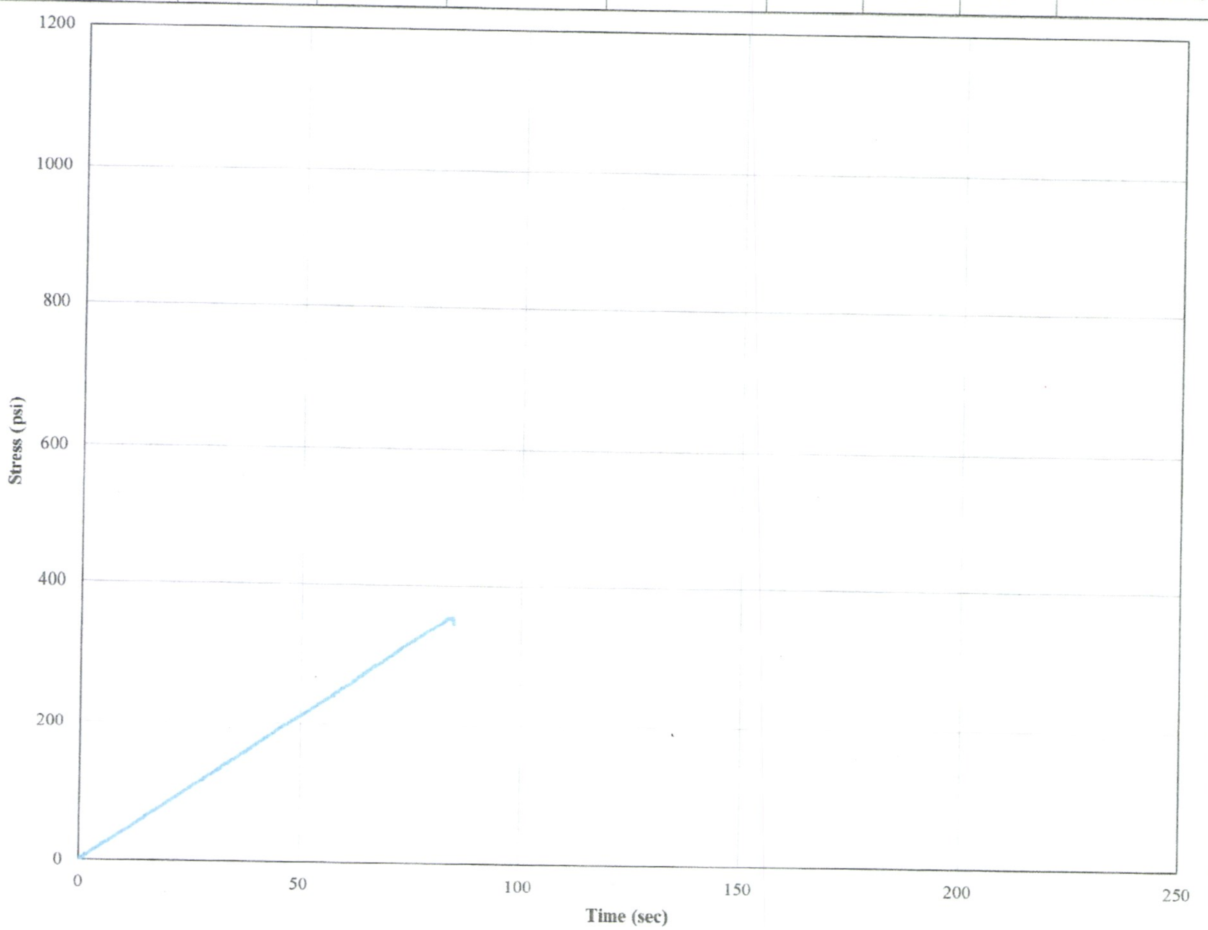
Mining Engineering Department, CSM

Brazilian Tensile Strength

Project Name : Rocky Mountain Minerals
Location : CO Hwy 115
Rock Type : Metamorphic
Characteristics : Failed in direction of foliation
Loading Direction Parallel to foliation
Test Performed by : JT
Date Tested : 6/23/2015
Data Reduced by : PK
Date Reduced : 6/24/2015
Rock Source : N/A
Core ID : T1-R1-S2
File Name : T1-R1-S2_BTS
Comments : N/A

Disc Length		Disc Diameter		L/D Ratio	Failure Load		BTS		Failure Mode
in	cm	in	cm		lbs	N	psi	MPa	
1.29	3.29	1.933	4.911	0.67	1,397	6,214	355	2.4	Non-Structural



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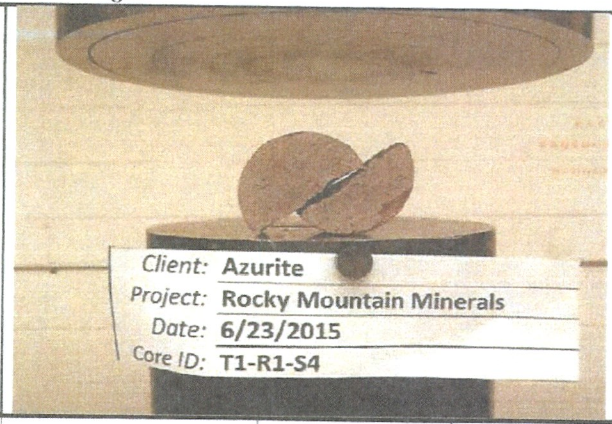
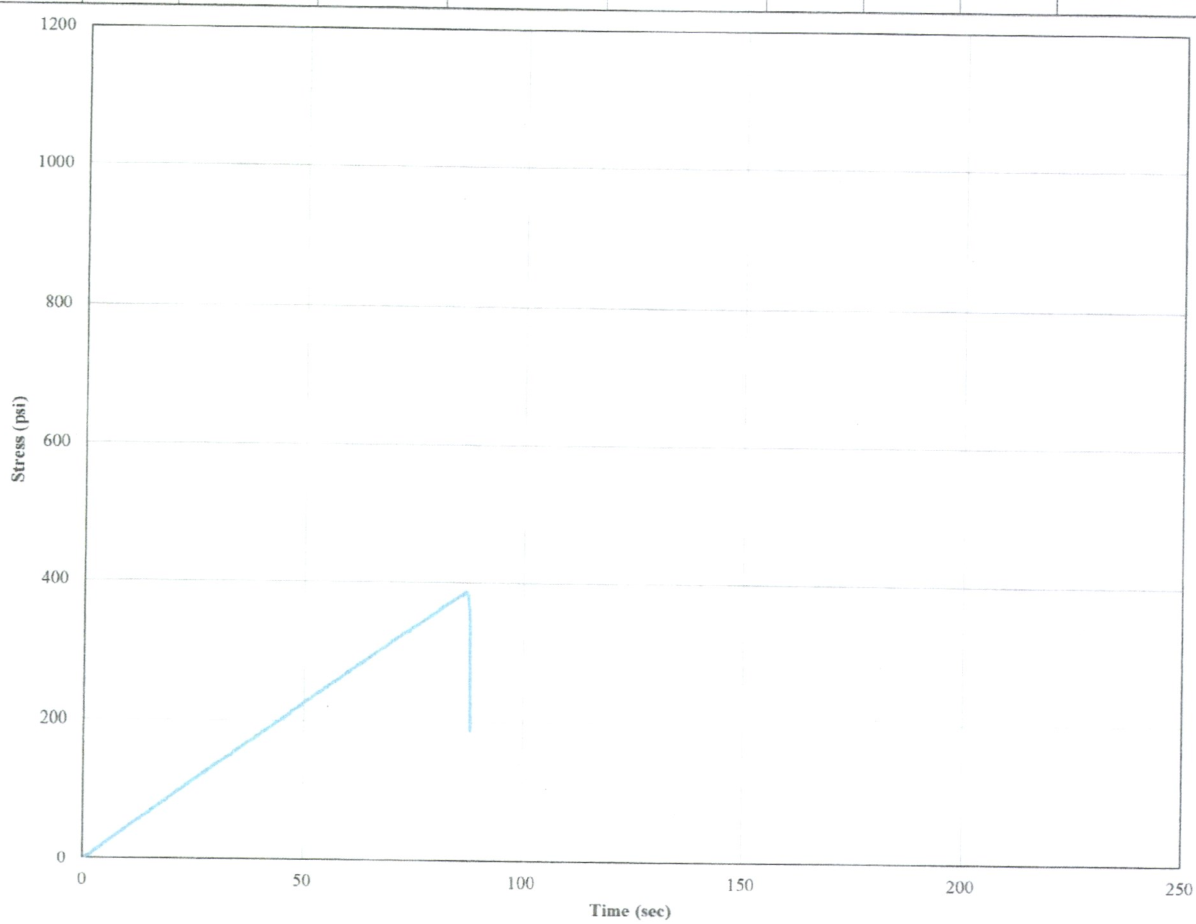
Mining Engineering Department, CSM

Brazilian Tensile Strength

Project Name : Rocky Mountain Minerals
Location : CO Hwy 115
Rock Type : Metamorphic
Characteristics : Failed in direction of foliation
Loading Direction Parallel to foliation
Test Performed by : JT
Date Tested : 6/23/2015
Data Reduced by : PK
Date Reduced : 6/24/2015
Rock Source : N/A
Core ID : T1-R1-S4
File Name : T1-R1-S4_BTS
Comments : N/A



Disc Length		Disc Diameter		L/D Ratio	Failure Load		BTS		Failure Mode
in	cm	in	cm		lbs	N	psi	MPa	
1.22	3.11	1.933	4.909	0.63	1,444	6,423	389	2.7	Non-Structural



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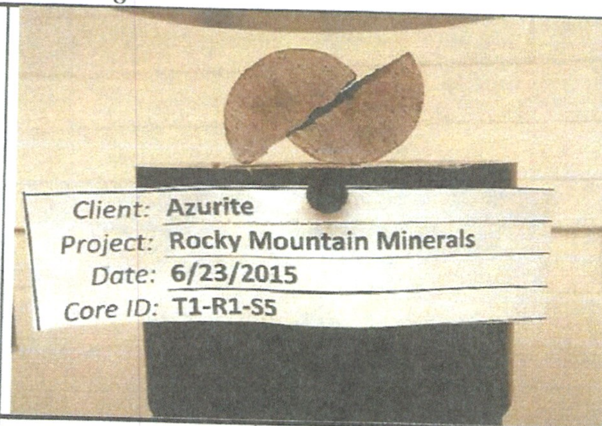
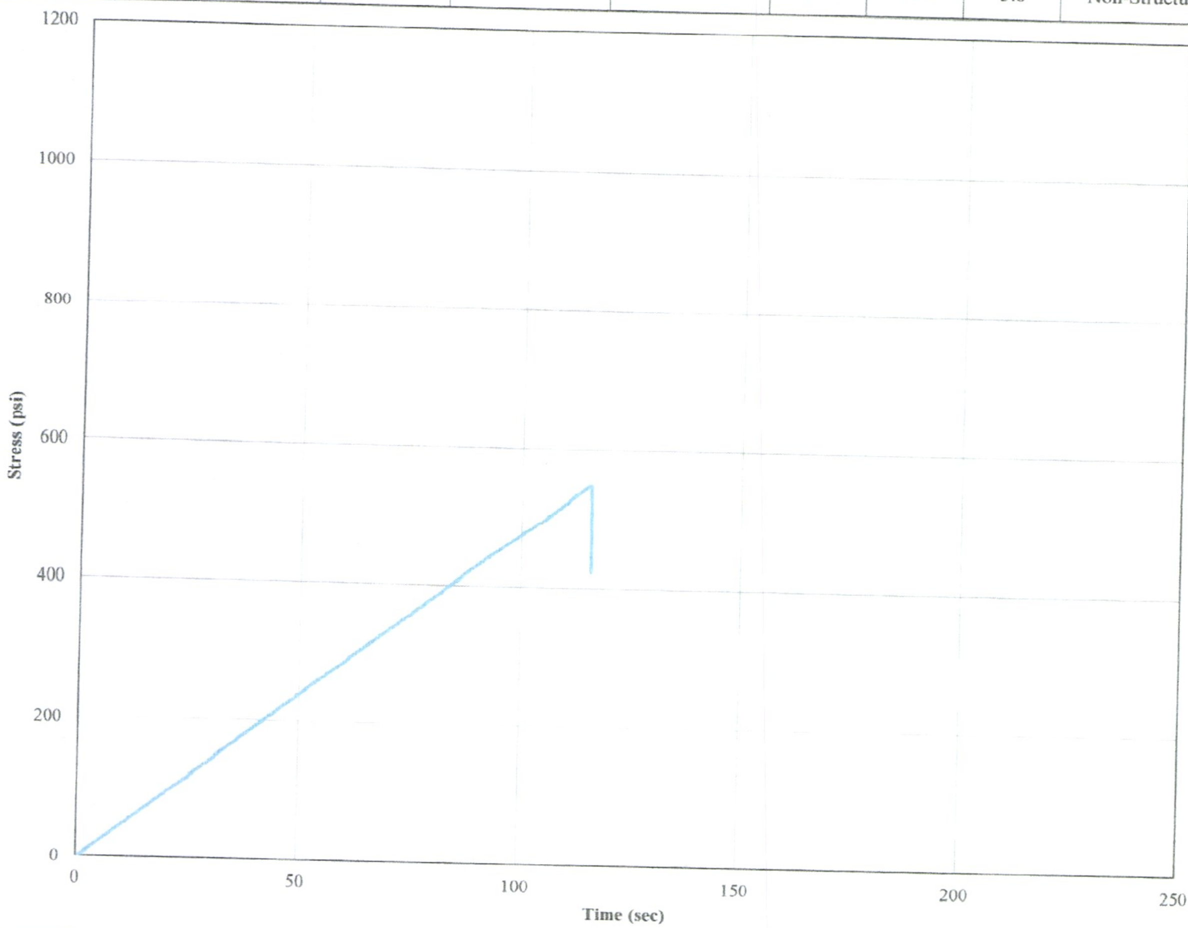
Brazilian Tensile Strength

Project Name : Rocky Mountain Minerals
Location : CO Hwy 115
Rock Type : Metamorphic
Characteristics :
Loading Direction Perpendicular to foliation
Test Performed by : JT
Date Tested : 6/23/2015
Data Reduced by : PK
Date Reduced : 6/24/2015
Rock Source : N/A
Core ID : T1-R1-S5
File Name : T1-R1-S5_BTS
Comments : N/A



Disc Length		Disc Diameter		L/D Ratio	Failure Load		BTS		Failure Mode
in	cm	in	cm		lbs	N	psi	MPa	
1.15	2.93	1.933	4.910	0.60	1,926	8,567	550	3.8	Non-Structural



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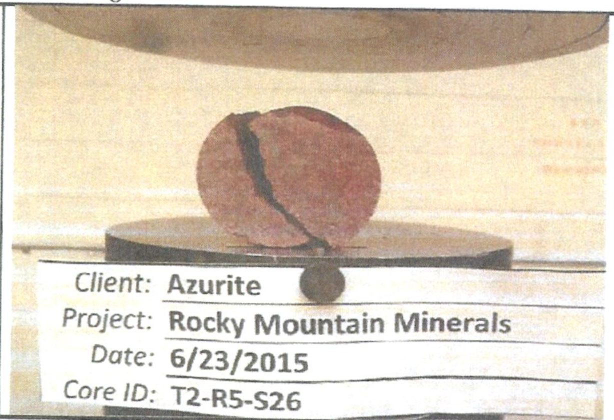
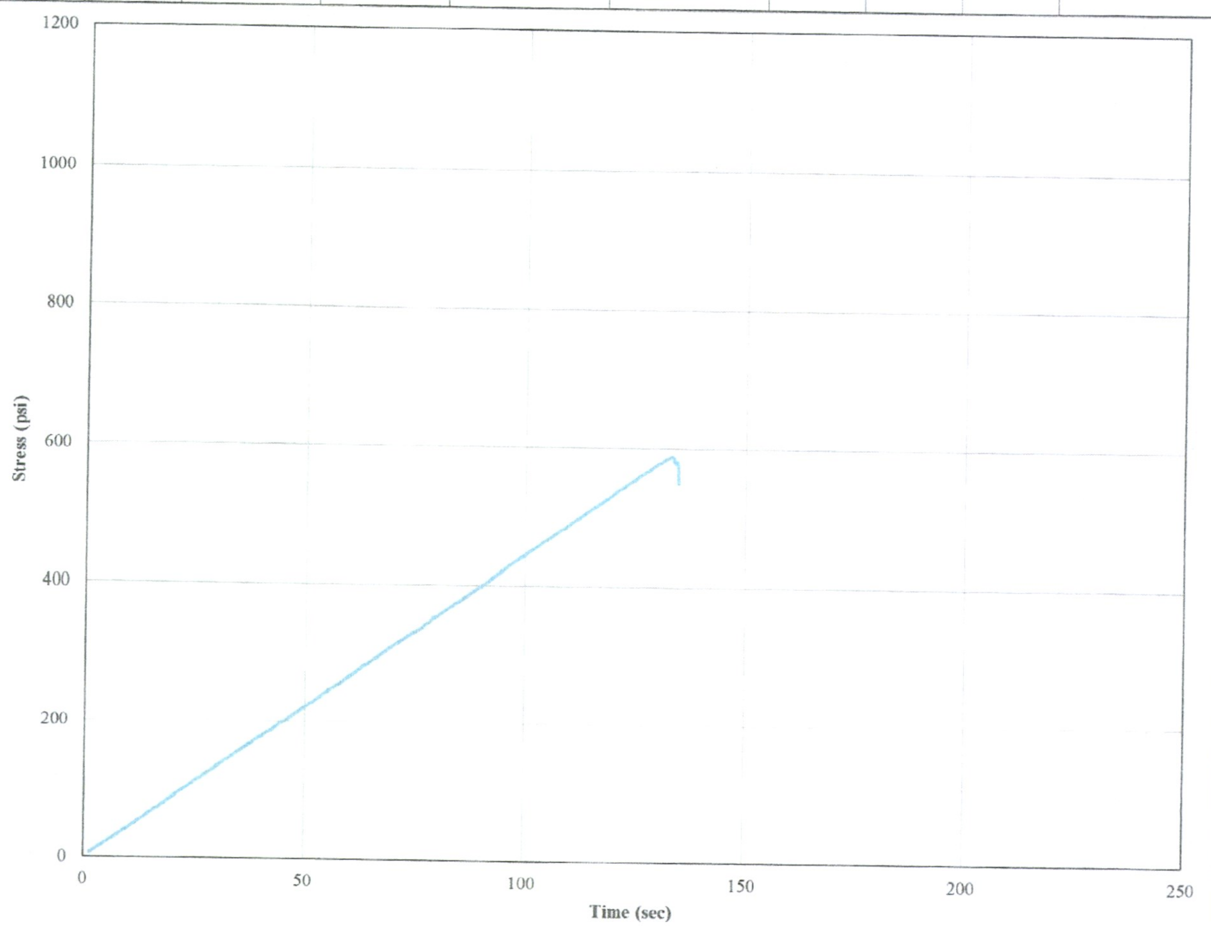
Brazilian Tensile Strength

Project Name : Rocky Mountain Minerals
Location : CO Hwy 115
Rock Type : Metamorphic
Characteristics : Failed in direction of foliation
Loading Direction Parallel to foliation
Test Performed by : JT
Date Tested : 6/23/2015
Data Reduced by : PK
Date Reduced : 6/24/2015
Rock Source : N/A
Core ID : T2-R5-S26
File Name : T2-R5-S26_BTS
Comments : N/A



Disc Length		Disc Diameter		L/D Ratio	Failure Load		BTS		Failure Mode
in	cm	in	cm		lbs	N	psi	MPa	
1.24	3.15	1.932	4.908	0.64	2,216	9,857	589	4.1	Non-Structural



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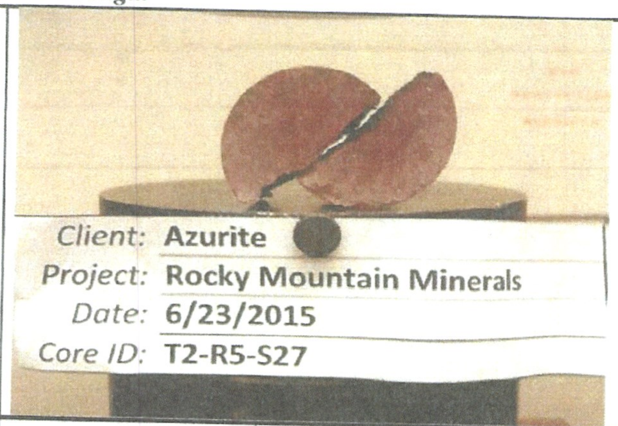
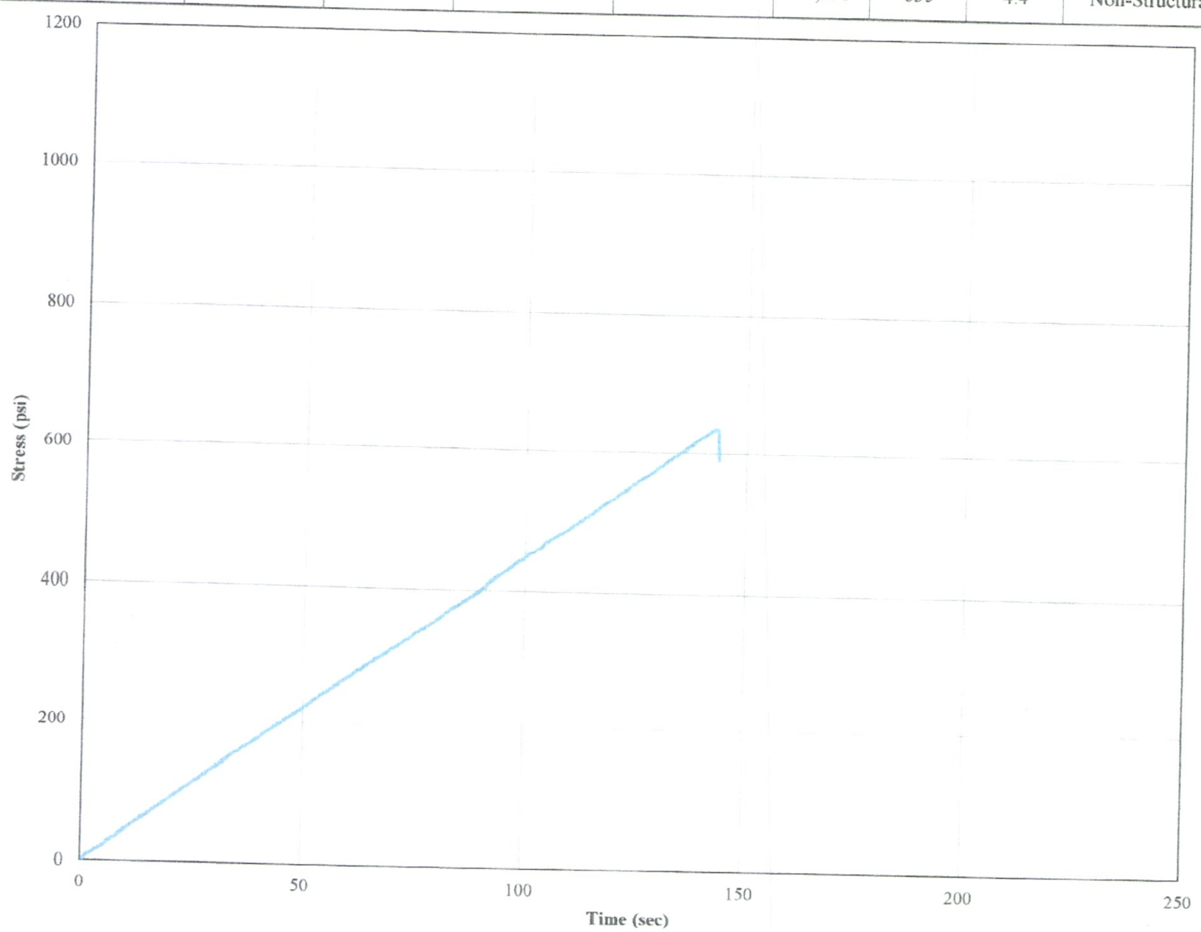
Mining Engineering Department, CSM

Brazilian Tensile Strength

Project Name : Rocky Mountain Minerals
Location : CO Hwy 115
Rock Type : Metamorphic
Characteristics : Failed in direction of foliation
Loading Direction Parallel to foliation
Test Performed by : JT
Date Tested : 6/23/2015
Data Reduced by : PK
Date Reduced : 6/24/2015
Rock Source : N/A
Core ID : T2-R5-S27
File Name : T2-R5-S27_BTS
Comments : N/A



Disc Length		Disc Diameter		L/D Ratio	Failure Load		BTS		Failure Mode
in	cm	in	cm		lbs	N	psi	MPa	
1.24	3.14	1.932	4.907	0.64	2,381	10,591	635	4.4	Non-Structural



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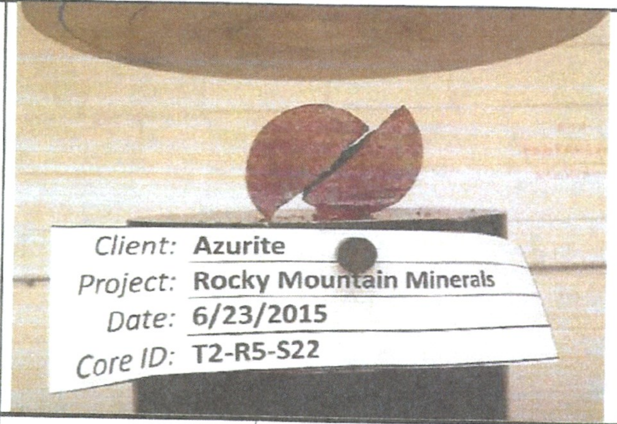
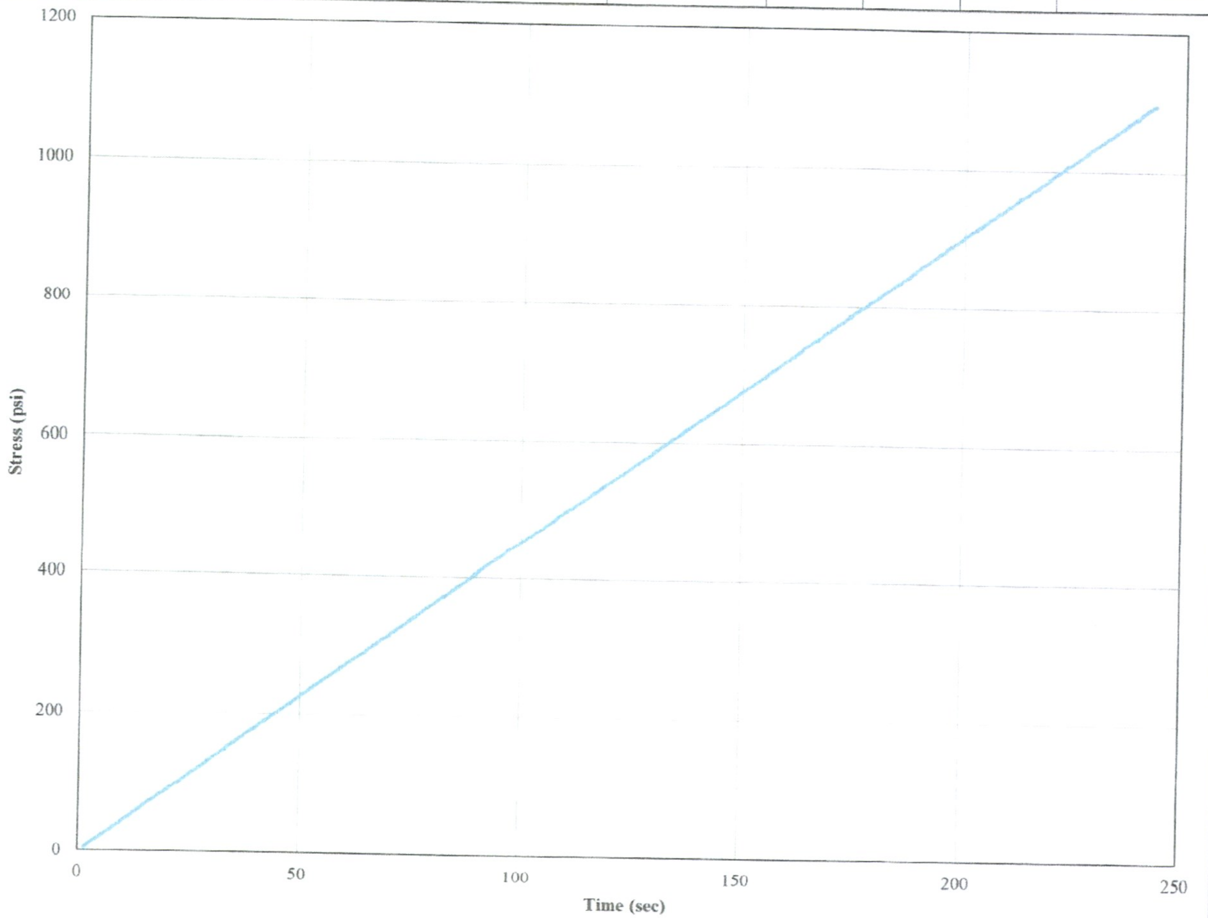
Brazilian Tensile Strength

Project Name : Rocky Mountain Minerals
Location : CO Hwy 115
Rock Type : Metamorphic
Characteristics :
Loading Direction Perpendicular to foliation
Test Performed by : JT
Date Tested : 6/23/2015
Data Reduced by : PK
Date Reduced : 6/24/2015
Rock Source : N/A
Core ID : T2-R5-S22
File Name : T2-R5-S22_BTS
Comments : N/A



Disc Length		Disc Diameter		L/D Ratio	Failure Load		BTS		Failure Mode
in	cm	in	cm		lbs	N	psi	MPa	
1.22	3.09	1.931	4.906	0.63	4,053	18,029	1,097	7.6	Non-Structural



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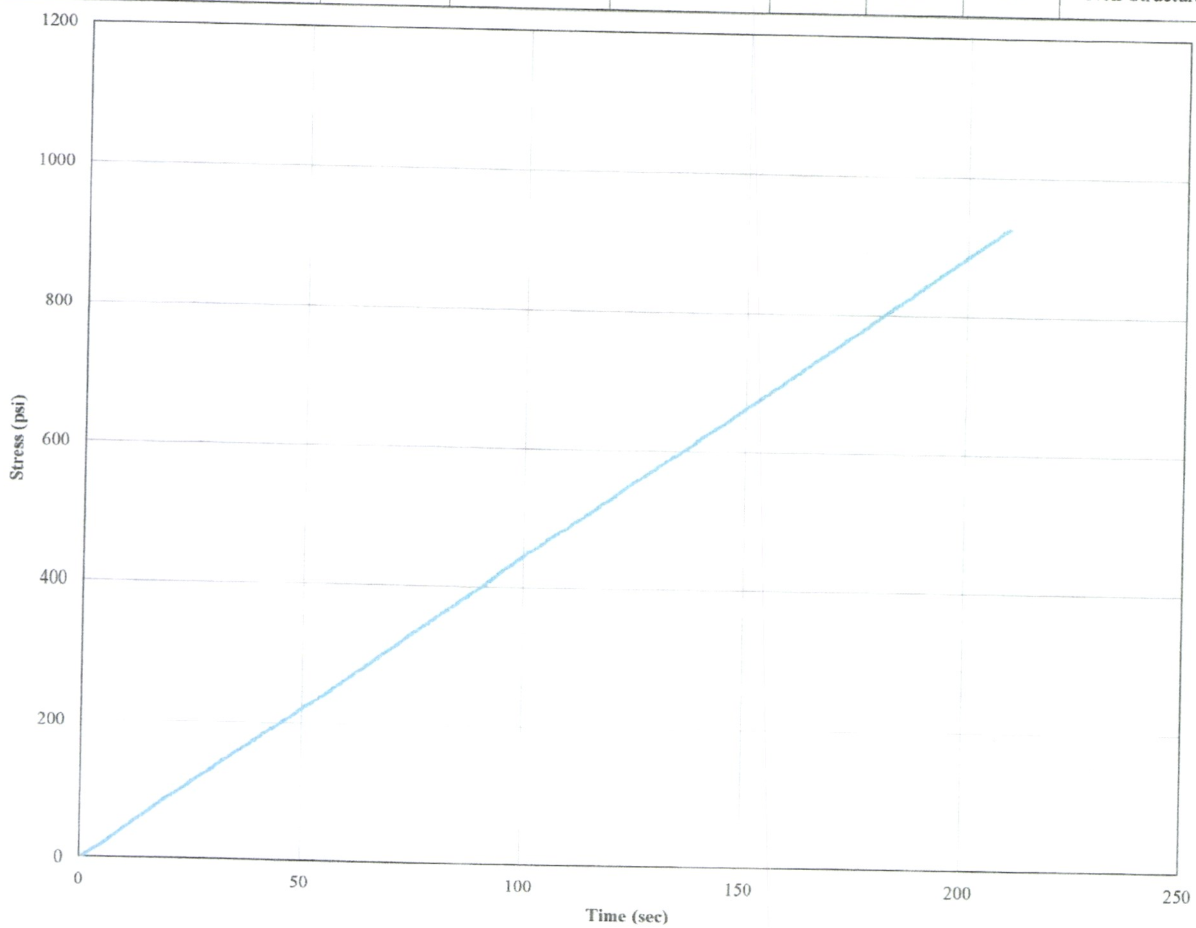
Brazilian Tensile Strength

Project Name : Rocky Mountain Minerals
Location : CO Hwy 115
Rock Type : Metamorphic
Characteristics :
Loading Direction Perpendicular to foliation
Test Performed by : JT
Date Tested : 6/23/2015
Data Reduced by : PK
Date Reduced : 6/24/2015
Rock Source : N/A
Core ID : T2-R5-S23
File Name : T2-R5-S23_BTS
Comments : N/A

Disc Length		Disc Diameter		L/D Ratio	Failure Load		BTS		Failure Mode
in	cm	in	cm		lbs	N	psi	MPa	
1.24	3.16	1.932	4.907	0.64	3,494	15,542	925	6.4	Non-Structural



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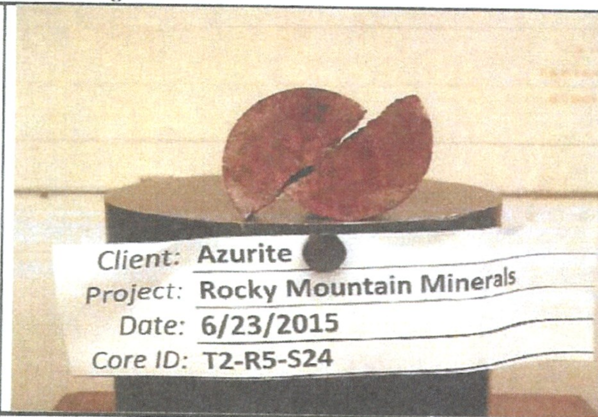
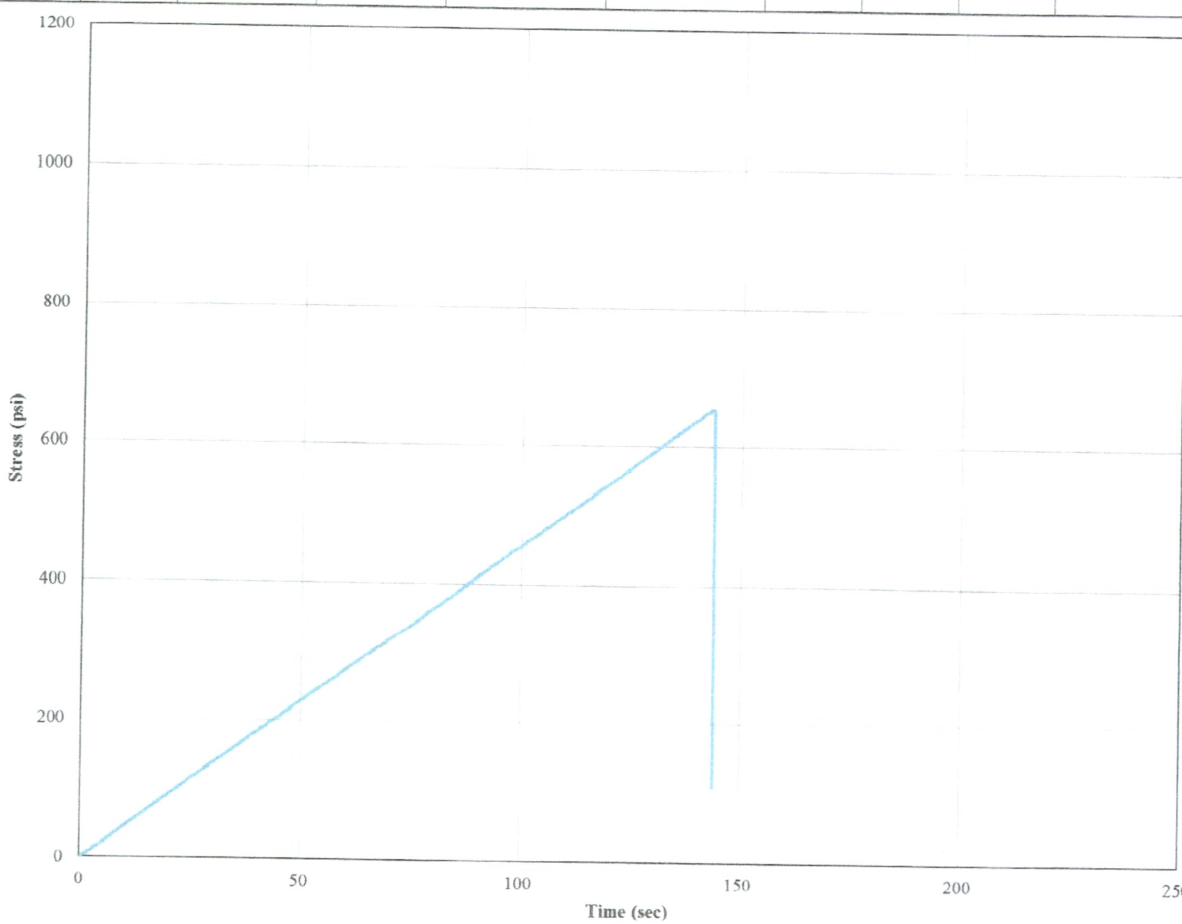
Brazilian Tensile Strength

Project Name : Rocky Mountain Minerals
Location : CO Hwy 115
Rock Type : Metamorphic
Characteristics :
Loading Direction Perpendicular to foliation
Test Performed by : JT
Date Tested : 6/23/2015
Data Reduced by : PK
Date Reduced : 6/24/2015
Rock Source : N/A
Core ID : T2-R5-S24
File Name : T2-R5-S24_BTS
Comments : N/A



Disc Length		Disc Diameter		L/D Ratio	Failure Load		BTS		Failure Mode
in	cm	in	cm		lbs	N	psi	MPa	
1.21	3.06	1.931	4.905	0.62	2,389	10,627	653	4.5	Structural



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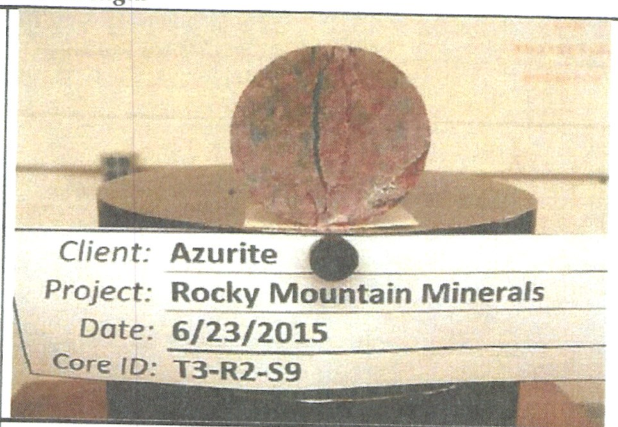
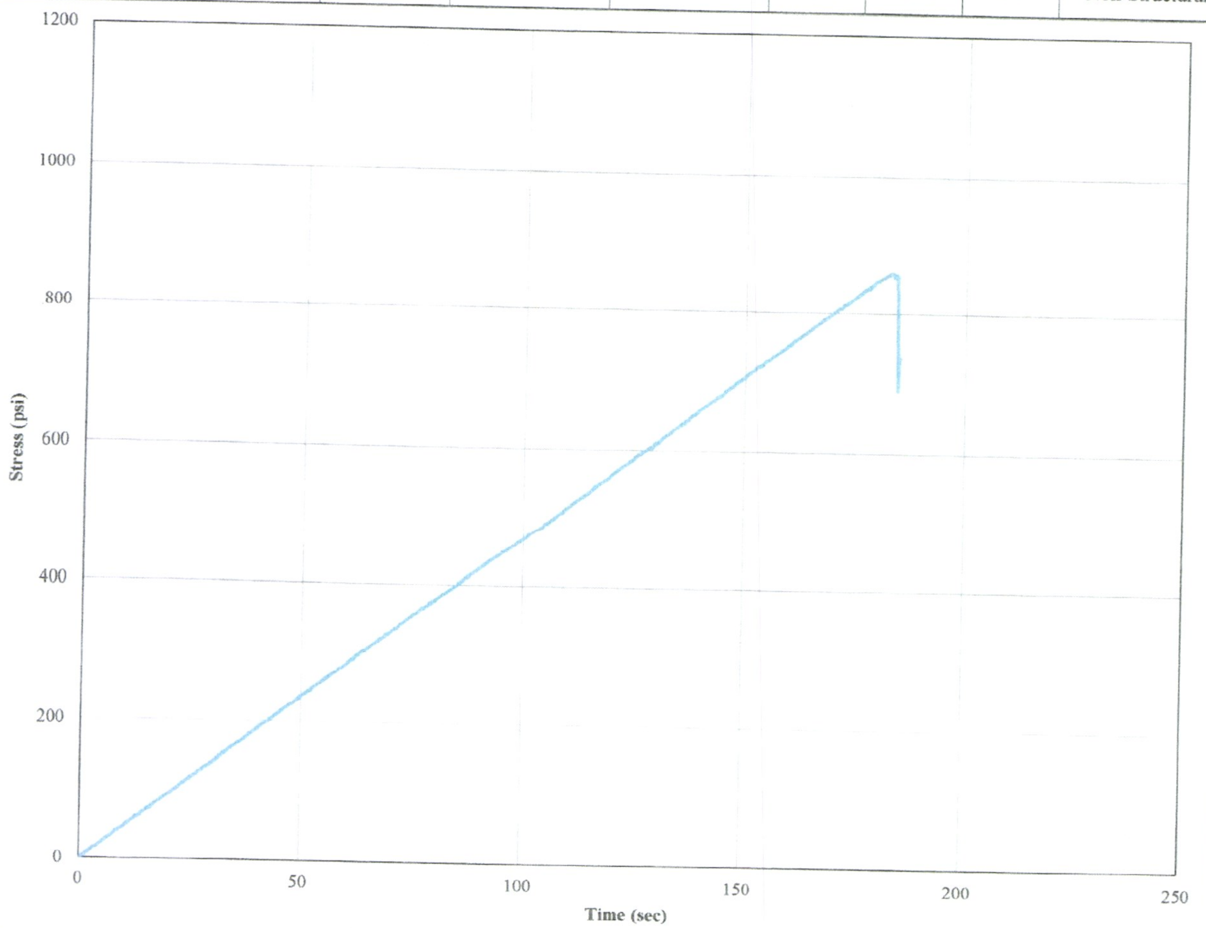
Brazilian Tensile Strength

Project Name : Rocky Mountain Minerals
Location : CO Hwy 115
Rock Type : Metamorphic
Characteristics :
Loading Direction Parallel to foliation
Test Performed by : JT
Date Tested : 6/23/2015
Data Reduced by : PK
Date Reduced : 6/24/2015
Rock Source : N/A
Core ID : T3-R2-S9
File Name : T3-R2-S9_BTS
Comments : N/A



Disc Length		Disc Diameter		L/D Ratio	Failure Load		BTS		Failure Mode
in	cm	in	cm		lbs	N	psi	MPa	
1.17	2.97	1.930	4.903	0.61	3,048	13,558	860	5.9	Non-Structural



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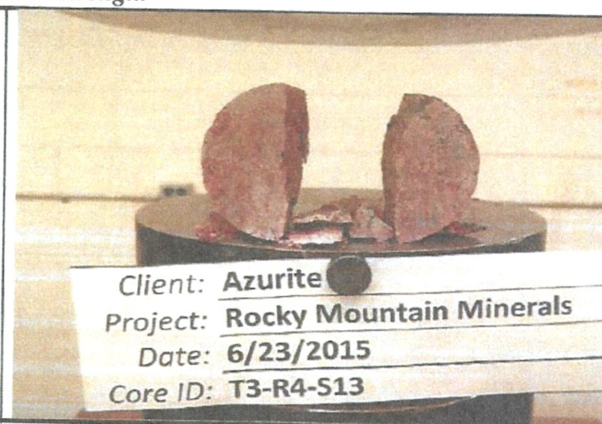
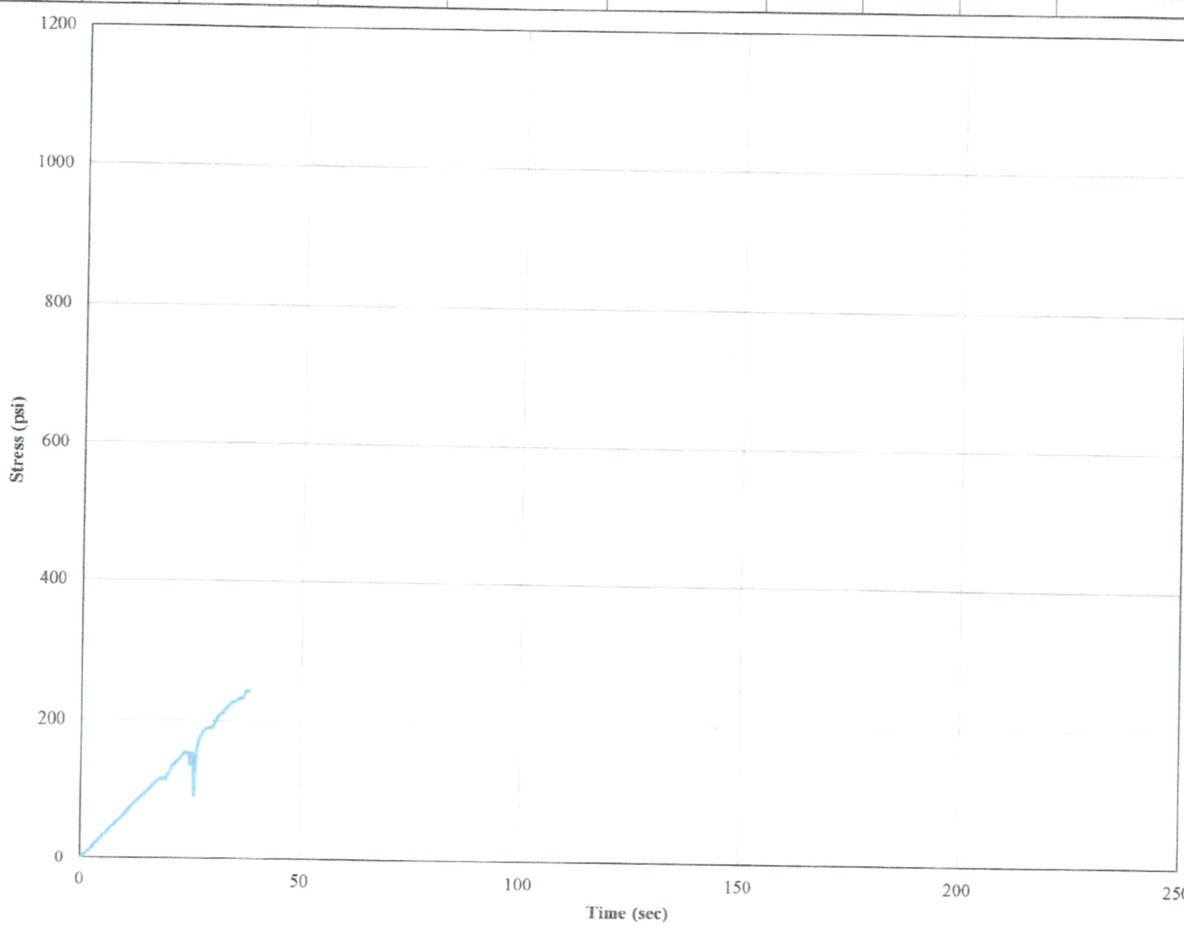
Brazilian Tensile Strength

Project Name : Rocky Mountain Minerals
Location : CO Hwy 115
Rock Type : Metamorphic
Characteristics :
Loading Direction Parallel to foliation
Test Performed by : JT
Date Tested : 6/23/2015
Data Reduced by : PK
Date Reduced : 6/24/2015
Rock Source : N/A
Core ID : T3-R4-S13
File Name : T3-R4-S13_BTS
Comments : N/A



Disc Length		Disc Diameter		L/D Ratio	Failure Load		BTS		Failure Mode
in	cm	in	cm		lbs	N	psi	MPa	
1.27	3.23	1.932	4.907	0.66	941	4,186	244	1.7	Non-Structural



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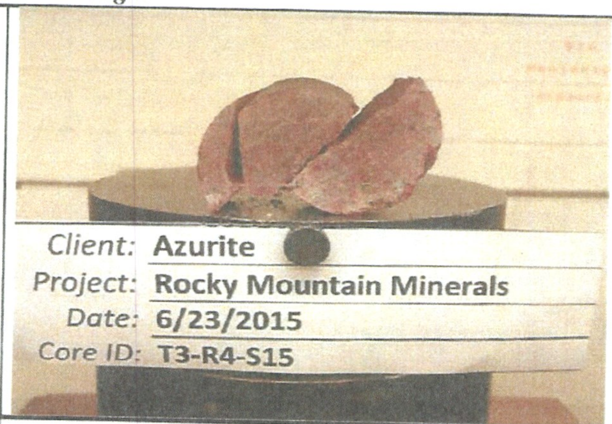
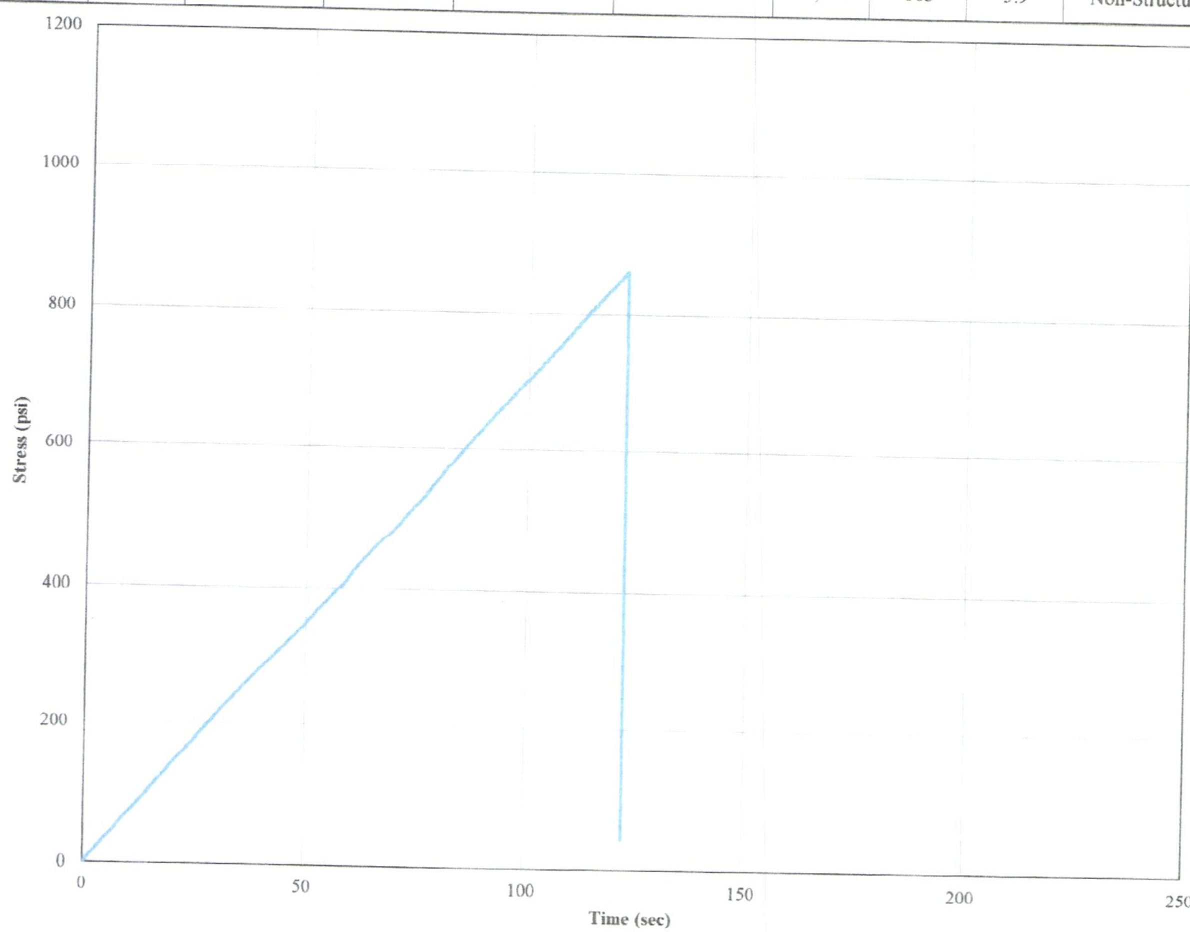
Brazilian Tensile Strength

Project Name : Rocky Mountain Minerals
Location : CO Hwy 115
Rock Type : Metamorphic
Characteristics :
Loading Direction Parallel to foliation
Test Performed by : JT
Date Tested : 6/23/2015
Data Reduced by : PK
Date Reduced : 6/24/2015
Rock Source : N/A
Core ID : T3-R4-S15
File Name : T3-R4-S15_BTS
Comments : N/A



Disc Length		Disc Diameter		L/D Ratio	Failure Load		BTS		Failure Mode
in	cm	in	cm		lbs	N	psi	MPa	
1.17	2.98	1.928	4.898	0.61	3,062	13,620	863	5.9	Non-Structural



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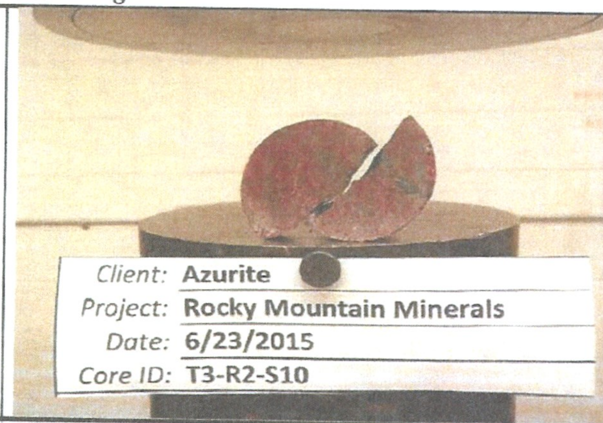
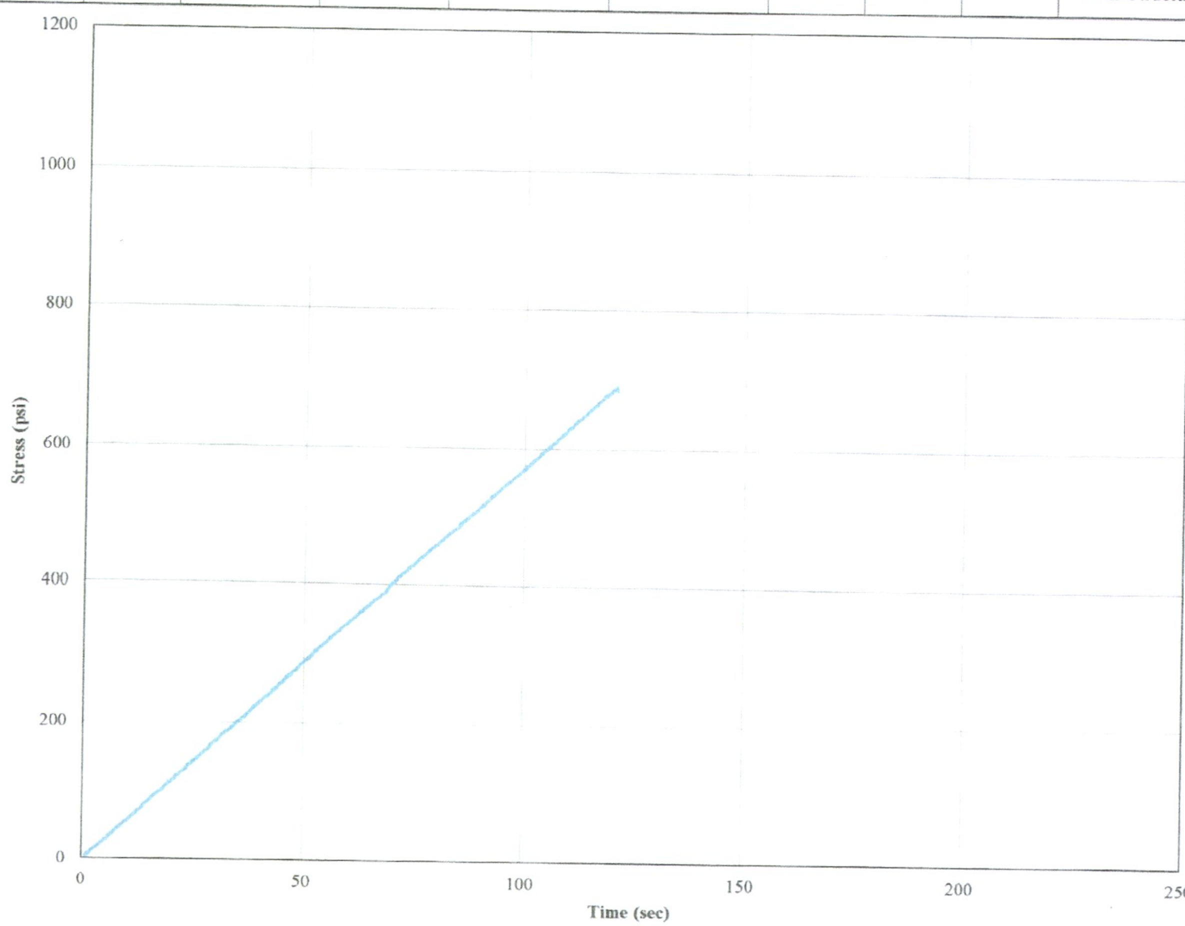
Mining Engineering Department, CSM

Brazilian Tensile Strength

Project Name : Rocky Mountain Minerals
Location : CO Hwy 115
Rock Type : Metamorphic
Characteristics :
Loading Direction Perpendicular to foliation
Test Performed by : JT
Date Tested : 6/23/2015
Data Reduced by : PK
Date Reduced : 6/24/2015
Rock Source : N/A
Core ID : T3-R2-S10
File Name : T3-R2-S10_BTS
Comments : N/A



Disc Length		Disc Diameter		L/D Ratio	Failure Load		BTS		Failure Mode
in	cm	in	cm		lbs	N	psi	MPa	
0.96	2.44	1.931	4.905	0.50	2,009	8,936	688	4.7	Non-Structural



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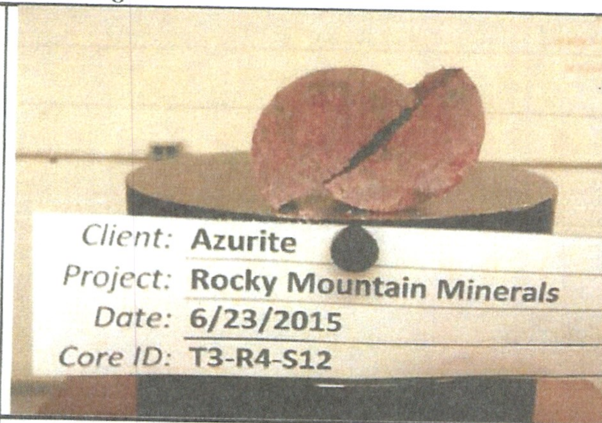
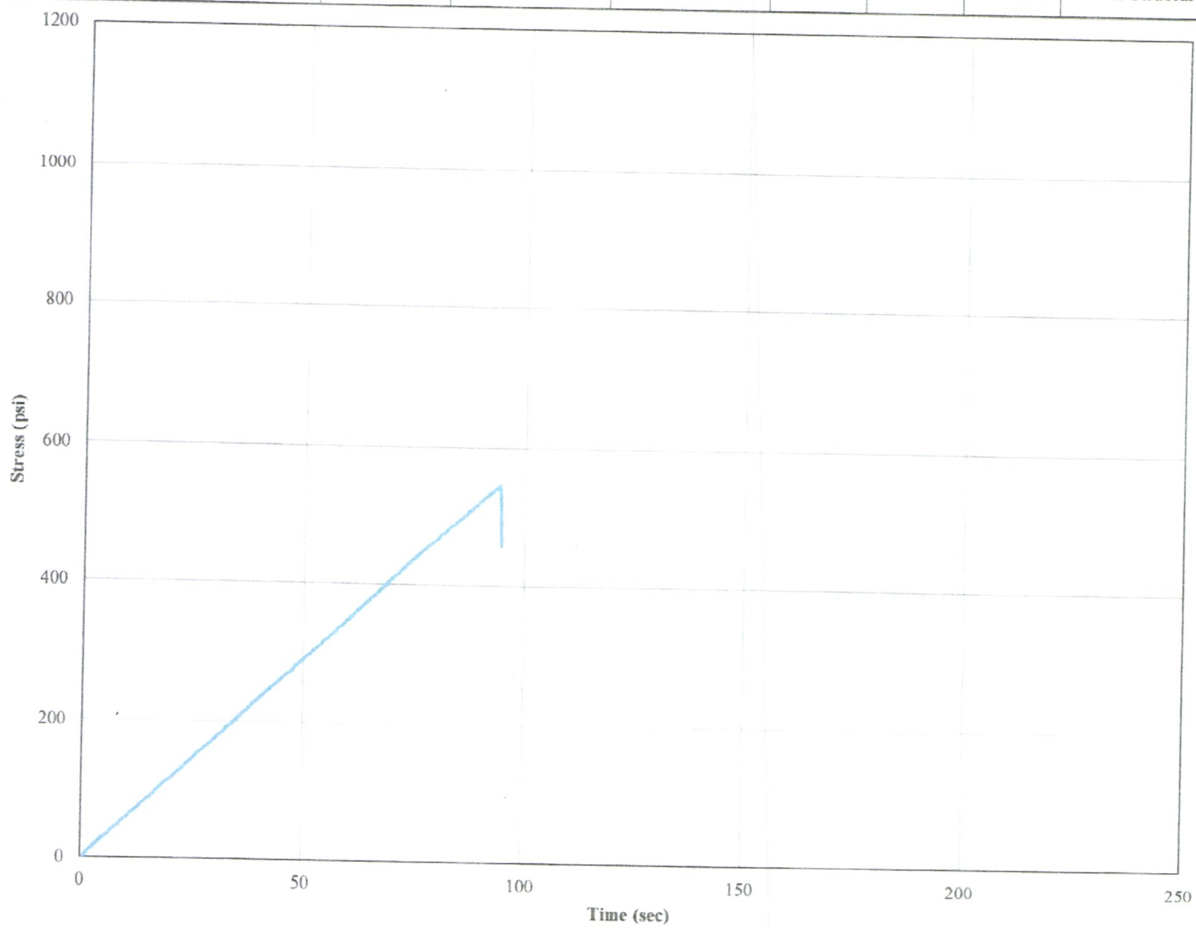
Brazilian Tensile Strength

Project Name : Rocky Mountain Minerals
Location : CO Hwy 115
Rock Type : Metamorphic
Characteristics :
Loading Direction Perpendicular to foliation
Test Performed by : JT
Date Tested : 6/23/2015
Data Reduced by : PK
Date Reduced : 6/24/2015
Rock Source : N/A
Core ID : T3-R4-S12
File Name : T3-R4-S12_BTS
Comments : N/A



Disc Length		Disc Diameter		L/D Ratio	Failure Load		BTS		Failure Mode
in	cm	in	cm		lbs	N	psi	MPa	
0.94	2.40	1.933	4.909	0.49	1,571	6,988	548	3.8	Non-Structural

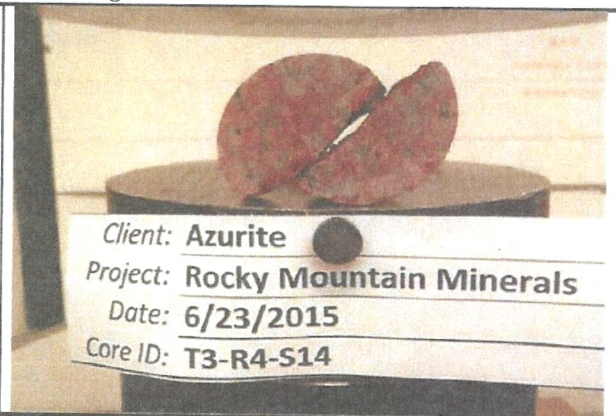


Earth Mechanics Institute

Mining Engineering Department, CSM

Brazilian Tensile Strength

Project Name : Rocky Mountain Minerals
Location : CO Hwy 115
Rock Type : Metamorphic
Characteristics :
Loading Direction Perpendicular to foliation
Test Performed by : JT
Date Tested : 6/23/2015
Data Reduced by : PK
Date Reduced : 6/24/2015
Rock Source : N/A
Core ID : T3-R4-S14
File Name : T3-R4-S14_BTS
Comments : N/A



Disc Length		Disc Diameter		L/D Ratio	Failure Load		BTS		Failure Mode
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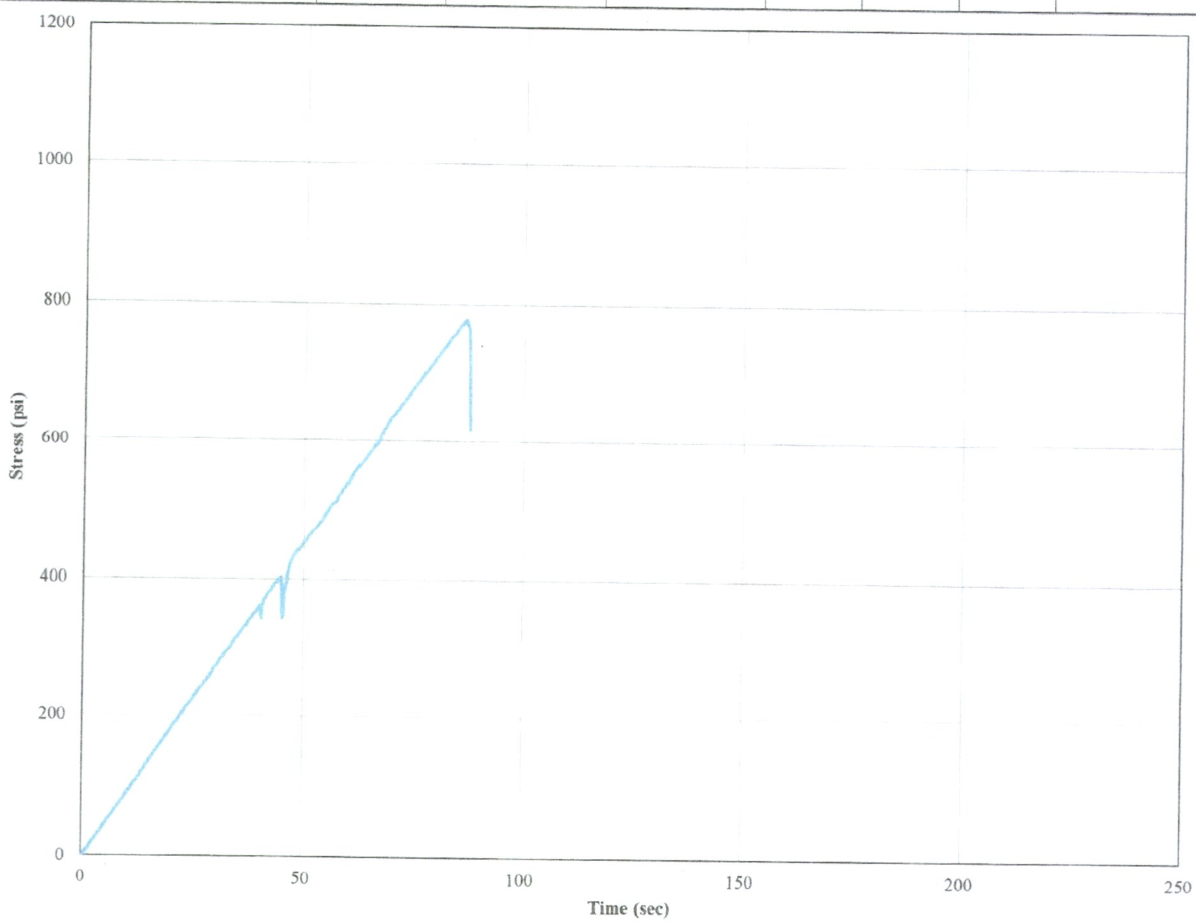
0.92	2.33	1.932	4.907	0.47	2,162	9,617	778	5.4	Non-Structural

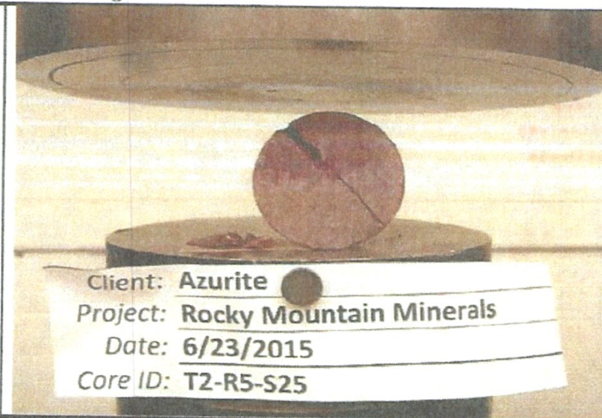


Earth Mechanics Institute

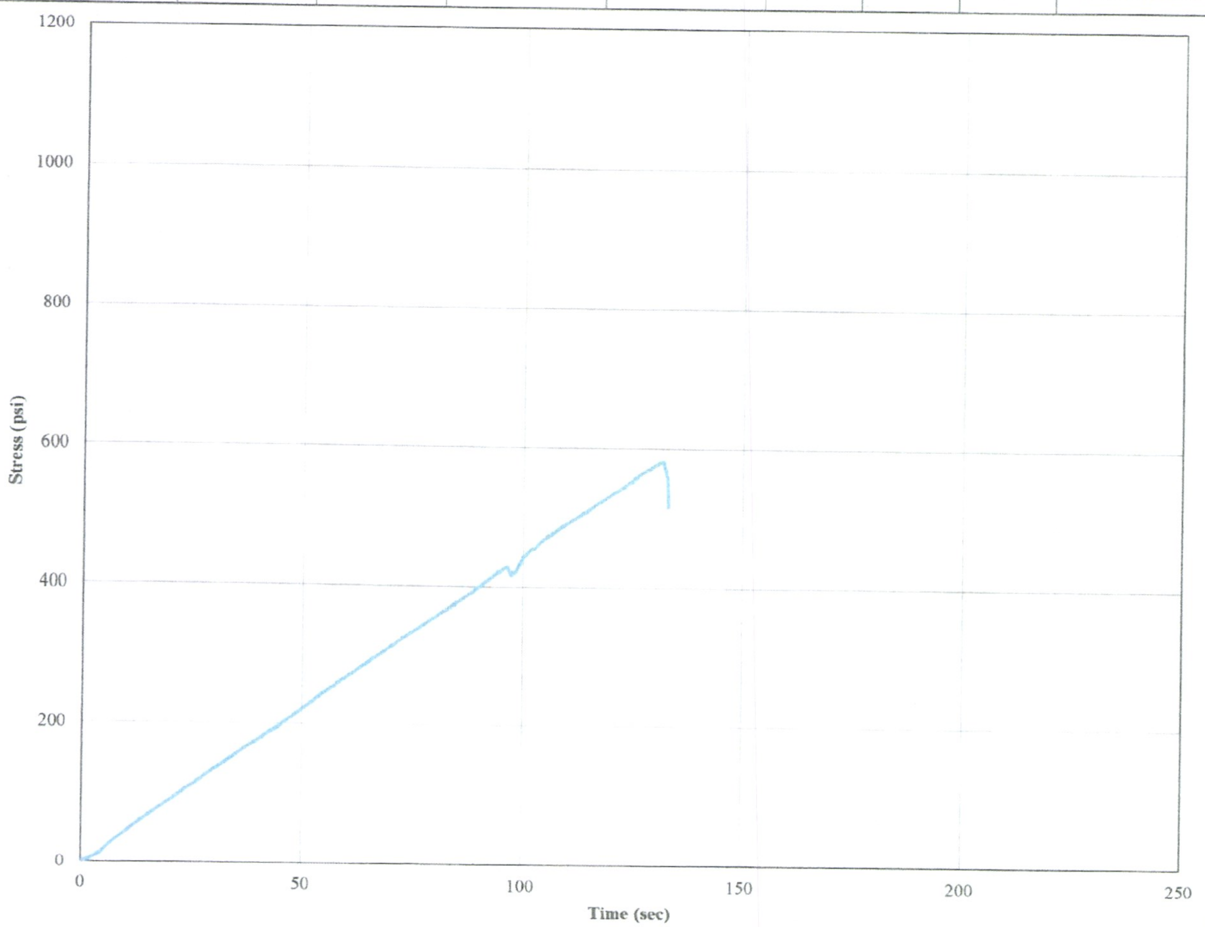
Mining Engineering Department, CSM

Brazilian Tensile Strength

Project Name : Rocky Mountain Minerals
Location : CO Hwy 115
Rock Type : Metamorphic
Characteristics : Failed in direction of foliation
Loading Direction Parallel to foliation
Test Performed by : JT
Date Tested : 6/23/2015
Data Reduced by : PK
Date Reduced : 6/24/2015
Rock Source : N/A
Core ID : T2-R5-S25
File Name : T2-R5-S25_BTS
Comments : N/A



Disc Length		Disc Diameter		L/D Ratio	Failure Load		BTS		Failure Mode
in	cm	in	cm		lbs	N	psi	MPa	
1.24	3.14	1.931	4.906	0.64	2,190	9,742	583	4.0	Non-Structural

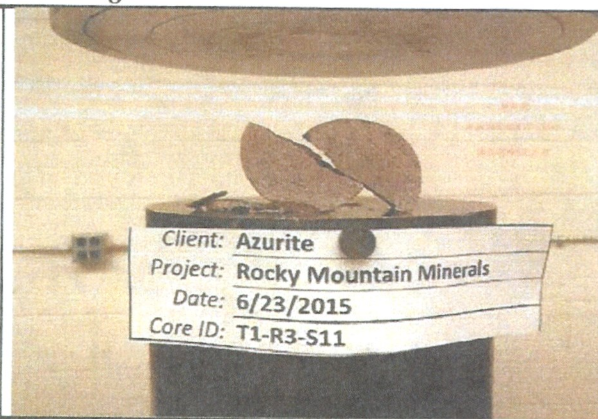


Earth Mechanics Institute

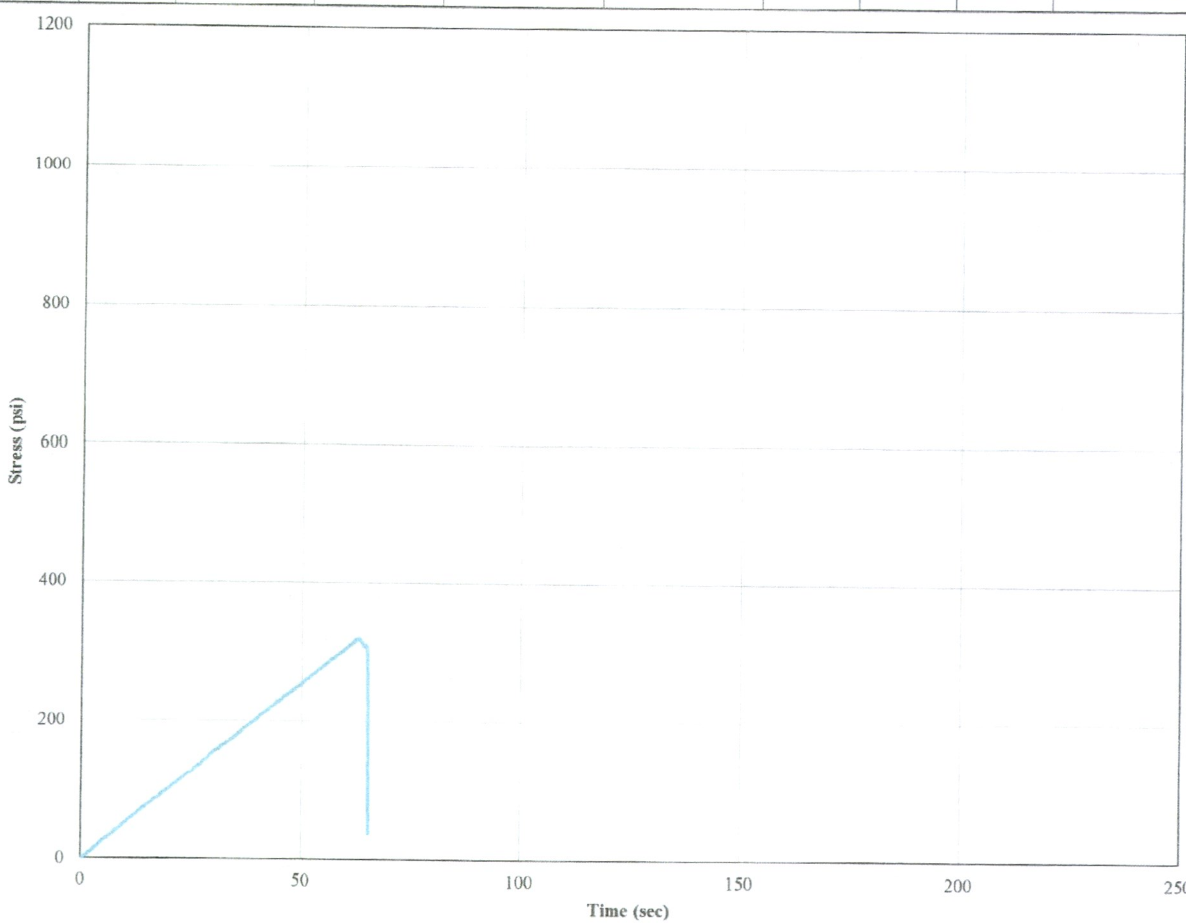
Mining Engineering Department, CSM

Brazilian Tensile Strength

Project Name : Rocky Mountain Minerals
Location : CO Hwy 115
Rock Type : Metamorphic
Characteristics : Failed in direction of foliation
Loading Direction Parallel to foliation
Test Performed by : JT
Date Tested : 6/23/2015
Data Reduced by : PK
Date Reduced : 6/24/2015
Rock Source : N/A
Core ID : T1-R3-S11
File Name : T1-R3-S11_BTS
Comments : N/A



Disc Length		Disc Diameter		L/D Ratio	Failure Load		BTS		Failure Mode
in	cm	in	cm		lbs	N	psi	MPa	
1.08	2.73	1.932	4.906	0.56	1,048	4,662	321	2.2	Non-Structural

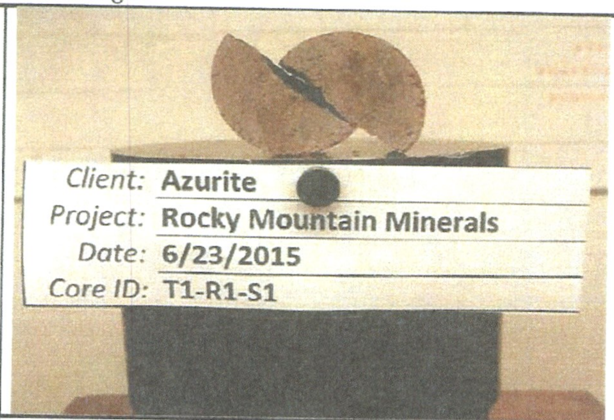


Earth Mechanics Institute

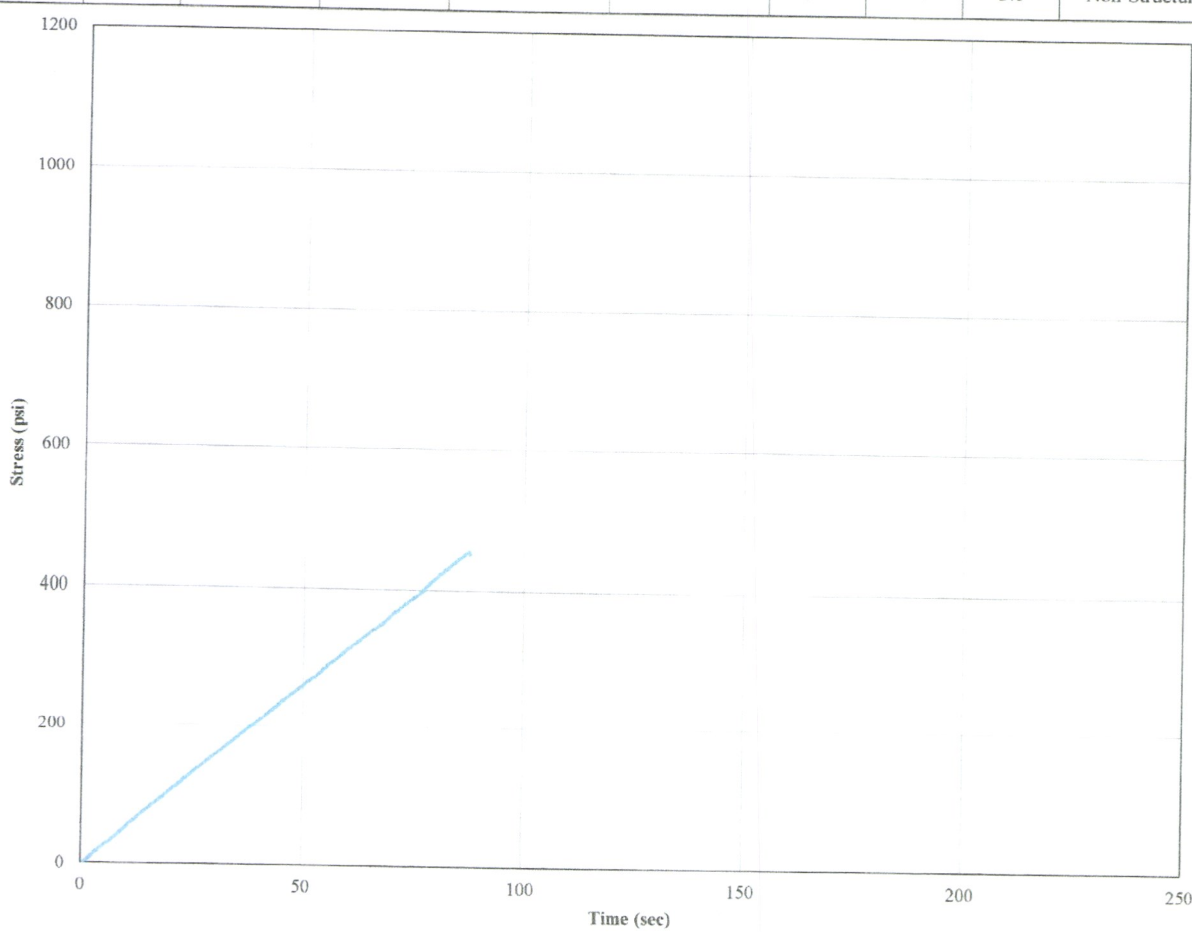
Mining Engineering Department, CSM

Brazilian Tensile Strength

Project Name : Rocky Mountain Minerals
Location : CO Hwy 115
Rock Type : Metamorphic
Characteristics :
Loading Direction Perpendicular to foliation
Test Performed by : JT
Date Tested : 6/23/2015
Data Reduced by : PK
Date Reduced : 6/24/2015
Rock Source : N/A
Core ID : T1-R1-S1
File Name : T1-R1-S1_BTS
Comments : N/A



Disc Length		Disc Diameter		L/D Ratio	Failure Load		BTS		Failure Mode
in	cm	in	cm		lbs	N	psi	MPa	
1.06	2.69	1.933	4.910	0.55	1,456	6,477	453	3.1	Non-Structural

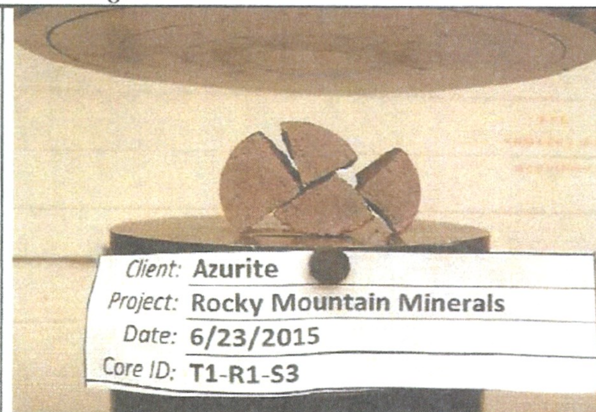


Earth Mechanics Institute

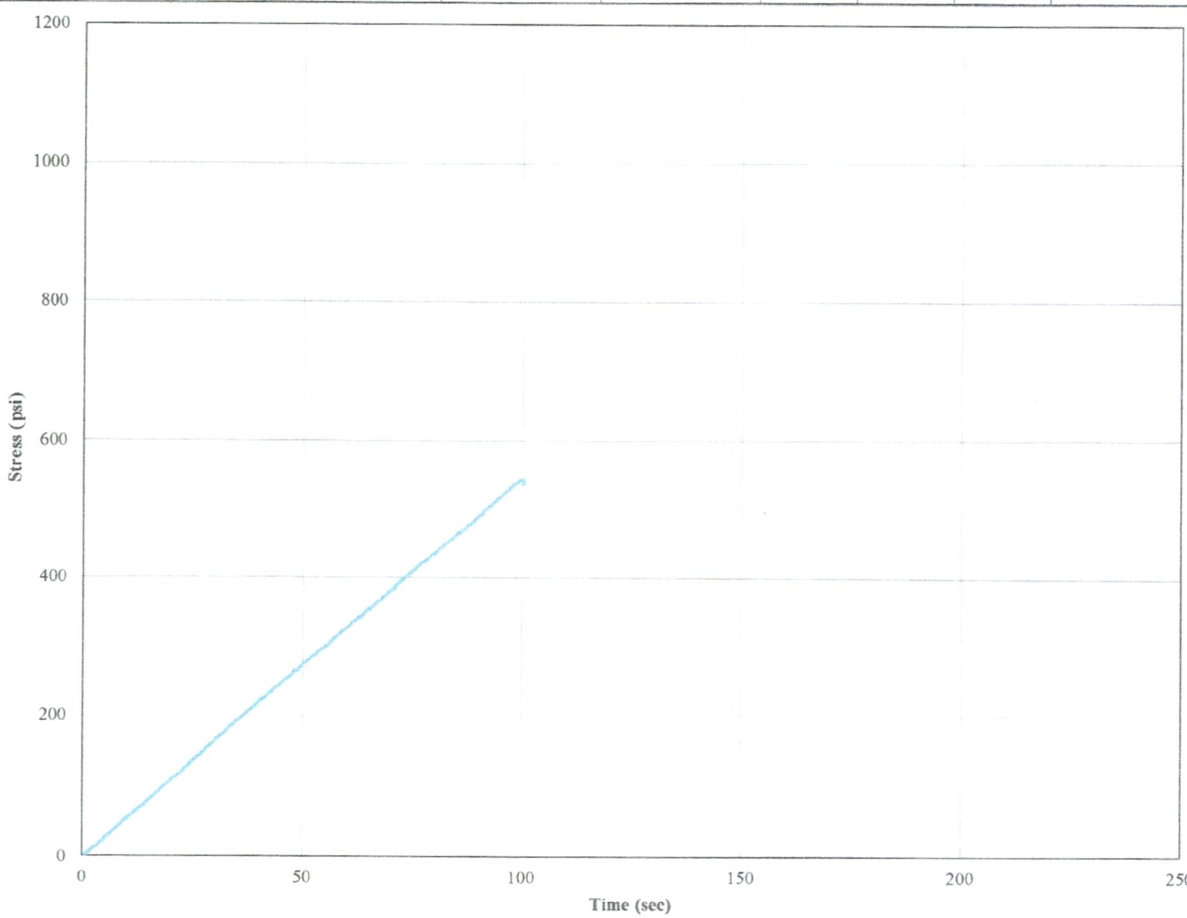
Mining Engineering Department, CSM

Brazilian Tensile Strength

Project Name : Rocky Mountain Minerals
Location : CO Hwy 115
Rock Type : Metamorphic
Characteristics :
Loading Direction Perpendicular to foliation
Test Performed by : JT
Date Tested : 6/23/2015
Data Reduced by : PK
Date Reduced : 6/24/2015
Rock Source : N/A
Core ID : T1-R1-S3
File Name : T1-R1-S3_BTS
Comments : N/A



Disc Length		Disc Diameter		L/D Ratio	Failure Load		BTS		Failure Mode
in	cm	in	cm		lbs	N	psi	MPa	
1.01	2.55	1.931	4.905	0.52	1,666	7,411	547	3.8	Non-Structural



The Hoek-Brown Failure Criterion – a 1988 Update

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Abstract

The brittle failure criterion for rock, described by Hoek and Brown (1980 a,b) and Hoek (1983), is widely used for estimating the strength of jointed rock masses. Publications describing applications of the criterion and correspondence received by the authors suggests that there is confusion on some of the details of the criterion and upon its limitations. This paper gives a brief summary of the equations which define the failure criterion in terms of major and minor principal stresses and normal and shear stresses and then discusses approximate relationships between the constants m and s and the rock mass classification developed by Bieniawski (1974) for both disturbed and undisturbed rock masses. Limitations on the use of the criterion and guidance on the selection of the the empirical constants are discussed in the final section of the paper.

Definition of failure criterion

The most detailed description of the Hoek-Brown failure criterion is contained in the Rankine lecture by Hoek (1983) and this will be taken as the reference upon which the up-dates presented in this paper are based.

The criterion was originally derived for applications in underground excavation design and it was therefore expressed in terms of the major and minor principal effective stresses acting upon an element of the rock mass. The basic equation defining the criterion is:

$$\sigma_1' = \sigma_3' + \sqrt{m\sigma_c\sigma_3' + s\sigma_c^2} \qquad (1)$$

where

σ_1' is the major principal effective stress at failure

σ_3' is the minor principal effective stress or confining pressure

m and s are material constants

σ_c is the uniaxial compressive strength of the *intact* rock.

Note that the uniaxial compressive strength of the intact rock refers to the strength which would be determined on a laboratory sized specimen (say a 50mm diameter by 100mm long core) which is free from discontinuities such as joints or bedding planes. This value is a measure of the contribution of the rock material to the overall strength of the rock mass. The uniaxial compressive strength of the *rock mass* is given by substituting $\sigma_3' = 0$ into equation 1:

$$\sigma_{cmass} = \sqrt{s}\,\sigma_c \qquad (2)$$

Similarly, substituting $\sigma_1' = 0$ into equation 1 and solving the resulting quadratic equation gives the uniaxial tensile strength of the rock or rock mass as:

$$\sigma_t = \frac{\sigma_c}{2}\left(m - \sqrt{m^2 + 4s}\right) \qquad (3)$$

Although the original failure criterion was developed for use in underground excavation design, there has been considerable interest in applying it to the design of slopes in heavily jointed rock. This led to a number of attempts to derive a corresponding relationship between the normal and shear stresses at failure. A solution was obtained by Dr John Bray at Imperial College (reported by Hoek, 1983) and similar relationships were developed by Ucar (1986) and Londe (1988). While these three sets of equations are different in appearance they all yield identical results and, to avoid confusion, the equations derived by Bray are presented here in a slightly modified form which the authors have found to be most convenient for incorporation into computer programs.

The shear strength τ for a specified effectove normal stress σ' is found by solving the following set of equations:

$$\tau = (Cot\phi'_i - Cos\phi'_i)\frac{m\sigma_c}{8} \quad (4)$$

where

$$\phi'_i = Arctan\frac{1}{\sqrt{4h\,Cos^2\theta - 1}} \quad (5)$$

$$\theta = \frac{1}{3}\left(90 + Arctan\frac{1}{\sqrt{h^3 - 1}}\right) \quad (6)$$

$$h = 1 + \frac{16\,(m\sigma' + s\sigma_c)}{3m^2\sigma_c} \quad (7)$$

Note that the angles ϕ'_i and θ are in degrees.

The slope of the tangent to the Mohr failure envelope at a normal effective stress of σ' is given by the instantaneous friction angle ϕ'_i. The corresponding instantaneous cohesion c'_i, the intercept of the tangent on the τ axis, is :

$$c'_i = \tau - \sigma'\,Tan\phi'_i \quad (8)$$

The failure angle β, measured from the direction of σ'_1, the major principal effective stress, is :

$$\beta = 45 - \frac{\phi'_i}{2} \quad (9)$$

As an example of the application of the equations relating the shear strength τ to the normal effective stress σ', consider a rock mass defined by the material constants σ_c = 100 MPa, m = 3.5 and s = 0.1. The calculated values for h, ϕ'_i and τ, for a range of normal effective stress σ' values, are as follows:

σ' MPa	h	ϕ'_i °	τ MPa
0	1.0435	54.88	5.60
5	1.1197	46.30	11.58
10	1.1959	41.74	16.39
25	1.4245	34.32	27.95
50	1.8054	28.23	42.95
75	2.1864	24.71	55.34
100	2.5673	22.30	66.18

The uniaxial tensile strength of this rock mass is given by equation 3 as σ_t = -2.834 MPa. Calculation of the values of ϕ'_i and τ using this value for the normal effective stress σ' will not give the expected values of ϕ'_i = 90 and τ = 0. These conditions are satisfied when h = 1 in equation 7, ie when $\sigma_{t.env} = s\sigma_c/m$, giving $\sigma_{t.env}$ = -2.857 MPa. This difference arises because the radius of curvature of the 'nose' of the Mohr envelope is not necessarily the same as the radius of the Mohr circle defining the uniaxial tensile strength of the rock mass. This problem is similar to that which occurs in Griffith's theory of brittle failure, discussed by Hoek (1968), and it results in a slight truncation of either the principal stress plot or, in this case, the 'nose' of the Mohr envelope.

For most engineering analyses it is assumed that a jointed rock mass is incapable of carrying any tensile stress and hence a tension cut-off is usually imposed at σ'_3=0 for the principal stress plot or σ'=0 for the Mohr envelope. Hence the difference in calculated tensile strengths of less than 1% in the example given above has no practical significance. It has been included in this discussion because it can lead to confusion when checking the performance of a computer program.

Analysis of laboratory data

Where the results of laboratory triaxial or shear strength tests are available, the constants m, s and σ_c can be determined as follows.

Triaxial tests on intact rock

For intact rock the value of the constant s = 1 and the values of σ_c and m are given by:

$$\sigma_c = \sqrt{\frac{\Sigma y}{n} - \frac{\Sigma x}{n}\left(\frac{\Sigma xy - (\Sigma x \Sigma y)/n}{\Sigma x^2 - (\Sigma x)^2/n}\right)} \quad (10)$$

$$m = \frac{1}{\sigma_c}\left(\frac{\Sigma xy - (\Sigma x \Sigma y)/n}{\Sigma x^2 - (\Sigma x)^2/n}\right) \quad (11)$$

where

$x = \sigma'_3$

$y = (\sigma'_1 - \sigma'_3)^2$

n = number of σ'_1, σ'_3 data pairs.

The coefficient of determination r^2 is

$$r^2 = \frac{(\Sigma xy - (\Sigma x \Sigma y)/n)^2}{(\Sigma x^2 - (\Sigma x)^2/n)(\Sigma y^2 - (\Sigma y)^2/n)} \qquad (12)$$

Triaxial tests on broken rock

For broken or heavily jointed rock, the uniaxial compressive strength of the intact pieces is determined from equation 10 or from uniaxial compression or point load tests on specimens of the rock. The value of m is found from equation 11 and the value of s is:

$$s = \frac{\Sigma y/n - m\sigma_c \Sigma x/n}{\sigma_c^2} \qquad (13)$$

When the value of s is very close to zero, equation 13 will sometimes give a small negative s. This problem usually arises when there is a deficiency of experimental data in the region of $\sigma_3' \leq 0$ for triaxial tests or $\sigma' \leq 0$ for shear tests. In such cases, put the value of $s = 0$ and calculate a new value for m from:

$$m = \frac{\Sigma y}{\sigma_c \, \Sigma x} \qquad (14)$$

Analysis of shear test data

The major and minor principal effective stresses σ_1' and σ_3' corresponding to each shear and normal effective stress (τ, σ') pair can be calculated from :

$$\sigma_1' = \sigma' + \frac{\tau}{\sigma'}\left((\tau - c_s') + \sqrt{\sigma'^2 + (\tau - c_s')^2}\right) \qquad (15)$$

$$\sigma_3' = \sigma' + \frac{\tau}{\sigma'}\left((\tau - c_s') - \sqrt{\sigma'^2 + (\tau - c_s')^2}\right) \qquad (16)$$

where c_s', the cohesion intercept for the τ, σ' data set, can be estimated from:

$$c_s' = \frac{\Sigma\sigma'}{n} - \frac{\Sigma\tau}{n}\left(\frac{\Sigma\tau\sigma' - (\Sigma\tau\Sigma\sigma')/n}{\Sigma\tau^2 - (\Sigma\tau)^2/n}\right) \qquad (17)$$

The values of σ_1' and σ_3' calculated from equations 15 and 16 are substituted into equations 11 and 13 to find the values of m and s. The Mohr failure envelope corresponding to these values of m and s can be calculated by means of equations 4 to 7.

Limitations of data analysis

The regression analyses described in the previous section will give excellent results for data sets which are well spaced over the stress range of interest and which do not exhibit an excessive amount of scatter. Fortunately, many such data sets are available and some of these were used in the original derivation of the failure criterion by Hoek and Brown (1980 a,b).

When the test results exhibit a large amount of scatter or when these results are concentrated at one end of the stress range of interest, the curve fitting processes defined by equations 10 to 17 will not give satisfactory results. The results may be particularly misleading when one attempts to extrapolate from a limited amount of experimental data from tests in which the confining pressures (σ_3') or the normal effective stresses (σ') are compressive to a region in which these stresses are very small or negative. This situation can arise when analyzing the stability of slopes or near surface underground excavations where the stress levels are very low and where it is necessary to make an estimate of the rock mass strength at these very low stresses.

The Lowness algorithm (Cleveland, 1979) was developed to overcome some of these problems and, where it is essential that results be extracted or extrapolations be made from a limited or scattered data set, consideration should be given to using this technique instead of the analysis described in the previous section.

Field estimates of m and s

It is practically impossible to carry out triaxial or shear tests on rock masses at a scale which is of the same order of magnitude as surface or underground excavations used in mining or civil engineering. Numerous attempts have been made to overcome this problem by testing small scale models made up from assemblages of blocks or elements of rock or of carefully designed model materials. While these model studies have provided a great deal of valuable information, they generally suffer from limitations arising from the assumptions and simplifications which have to be made in order to permit construction of the models. Consequently, our ability to predict the strength of jointed rock masses on the basis of direct tests or of model studies is severely limited.

In searching for a solution to this problem in order to provide a basis for the design of underground excavations in rock, Hoek and Brown (1980a) felt that some attempt had to be made to link the constants m and s of their criterion to measurements or observations which could be carried out by any competent geologist in the field. Recognizing that the characteristics of the rock mass which control its strength and deformation behaviour are similar to the characteristics which had been adopted by Bieniawski (1974) and by Barton, Lien and Lunde (1974) for their rock mass classifications, Hoek and Brown (1980a) proposed that these rock mass classifications could be used for estimating the material constants m and s.

Because of the lack of suitable methods for estimating the strength of rock masses, the first table relating rock mass classifications to material properties published by Hoek and Brown (1980a) was widely accepted by the geotechnical community and has been used on a large number of projects. Experience gained from these applications showed that the estimated rock mass strengths were reasonable when used for slope stability studies in which the rock mass is usually disturbed and loosened by relaxation due to excavation of the slope. However, the estimated rock mass strengths generally appeared to be too low in applications involving underground excavations where the confining stresses do not permit the same degree of loosening as would occur in a slope.

In order to incorporate the lessons learned from practical applications, Brown and Hoek (1988) proposed a revised set of relationships between the rock mass rating (RMR) from Bieniawski's (1974) rock mass classification and the constants m and s. Following Priest and Brown (1983), the relationships were presented in the form of the following equations:

Disturbed rock masses :

$$\frac{m}{m_i} = \exp\left(\frac{\mathrm{RMR} - 100}{14}\right) \tag{18}$$

$$s = \exp\left(\frac{\mathrm{RMR} - 100}{6}\right) \tag{19}$$

Undisturbed or interlocking rock masses:

$$\frac{m}{m_i} = \exp\left(\frac{\mathrm{RMR} - 100}{28}\right) \tag{20}$$

$$s = \exp\left(\frac{\mathrm{RMR} - 100}{9}\right) \tag{21}$$

where

m and s are the rock mass constants and
m_i is the value of m for the *intact* rock.

Equations 18 to 21 have been used to construct Table 1 which shows the approximate relationship between rock mass quality and the Hoek-Brown material constants. Note that the value of the Tunnelling Quality Index Q from the NGI rock mass classification by Barton, Lien and Lunde (1974) has been calculated from the relationship proposed by Bieniawski (1976) :

$$\mathrm{RMR} = 9\,\mathrm{Log}_e\,\mathrm{Q} + 44 \tag{22}$$

Limitations on using failure criterion

Figure 1 illustrates a jointed rock mass in to which a tunnel has been mined. The circles adjacent to the right hand wall of the tunnel enclose different rock mass volumes and the comments on the right hand side of the drawing indicate situations to which the Hoek-Brown failure criterion can be applied.

When the volume of rock under consideration is small enough that it does not contain any structural discontinuities, equation 1 can be applied, using the m and s values for *intact* rock. This condition would apply to small scale specimens which has been extracted for laboratory testing or to the analysis of concentrated forces such as those which may be exerted by an individual pick on a tunnel boring machine cutter.

When the volume of rock being considered is such that only a few structural discontinuities are contained in this volume, the Hoek-Brown criterion should not be used. The behaviour of this rock is likely to be highly anisotropic and the Hoek-Brown failure criterion, which is only applicable to isotropic rock, will give erroneous results.

Table 1 : Approximate relationship between rock mass quality and material constants

Disturbed rock mass m and s values

undisturbed rock mass m and s values

EMPIRICAL FAILURE CRITERION $\sigma_1' = \sigma_3' + \sqrt{m\sigma_c\sigma_3' + s\sigma_c^2}$ σ_1' = major principal effective stress σ_3' = minor principal effective stress σ_c = uniaxial compressive strength of intact rock, and m and s are empirical constants.		CARBONATE ROCKS WITH WELL DEVELOPED CRYSTAL CLEAVAGE *dolomite, limestone and marble*	LITHIFIED ARGILLACEOUS ROCKS *mudstone, siltstone, shale and slate (normal to cleavage)*	ARENACEOUS ROCKS WITH STRONG CRYSTALS AND POORLY DEVELOPED CRYSTAL CLEAVAGE *sandstone and quartzite*	FINE GRAINED POLYMINERALLIC IGNEOUS CRYSTALLINE ROCKS *andesite, dolerite, diabase and rhyolite*	COARSE GRAINED POLYMINERALLIC IGNEOUS & METAMORPHIC CRYSTAL-LINE ROCKS – *amphibolite, gabbro gneiss, granite, norite, quartz-diorite*
INTACT ROCK SAMPLES *Laboratory size specimens free from discontinuities* CSIR rating: RMR = 100 NGI rating: Q = 500	m s *m* *s*	7.00 1.00 *7.00* *1.00*	10.00 1.00 *10.00* *1.00*	15.00 1.00 *15.00* *1.00*	17.00 1.00 *17.00* *1.00*	25.00 1.00 *25.00* *1.00*
VERY GOOD QUALITY ROCK MASS *Tightly interlocking undisturbed rock with unweathered joints at 1 to 3m.* CSIR rating: RMR = 85 NGI rating: Q = 100	m s *m* *s*	2.40 0.082 *4.10* *0.189*	3.43 0.082 *5.85* *0.189*	5.14 0.082 *8.78* *0.189*	5.82 0.082 *9.95* *0.189*	8.56 0.082 *14.63* *0.189*
GOOD QUALITY ROCK MASS *Fresh to slightly weathered rock, slightly disturbed with joints at 1 to 3m.* CSIR rating: RMR = 65 NGI rating: Q = 10	m s *m* *s*	0.575 0.00293 *2.006* *0.0205*	0.821 0.00293 *2.865* *0.0205*	1.231 0.00293 *4.298* *0.0205*	1.395 0.00293 *4.871* *0.0205*	2.052 0.00293 *7.163* *0.0205*
FAIR QUALITY ROCK MASS *Several sets of moderately weathered joints spaced at 0.3 to 1m.* CSIR rating: RMR = 44 NGI rating: Q = 1	m s *m* *s*	0.128 0.00009 *0.947* *0.00198*	0.183 0.00009 *1.353* *0.00198*	0.275 0.00009 *2.030* *0.00198*	0.311 0.00009 *2.301* *0.00198*	0.458 0.00009 *3.383* *0.00198*
POOR QUALITY ROCK MASS *Numerous weathered joints at 30-500mm, some gouge. Clean compacted waste rock* CSIR rating: RMR = 23 NGI rating: Q = 0.1	m s *m* *s*	0.029 0.000003 *0.447* *0.00019*	0.041 0.000003 *0.639* *0.00019*	0.061 0.000003 *0.959* *0.00019*	0.069 0.000003 *1.087* *0.00019*	0.102 0.000003 *1.598* *0.00019*
VERY POOR QUALITY ROCK MASS *Numerous heavily weathered joints spaced <50mm with gouge. Waste rock with fines.* CSIR rating: RMR = 3 NGI rating: Q = 0.01	m s *m* *s*	0.007 0.0000001 *0.219* *0.00002*	0.010 0.0000001 *0.313* *0.00002*	0.015 0.0000001 *0.469* *0.00002*	0.017 0.0000001 *0.532* *0.00002*	0.025 0.0000001 *0.782* *0.00002*

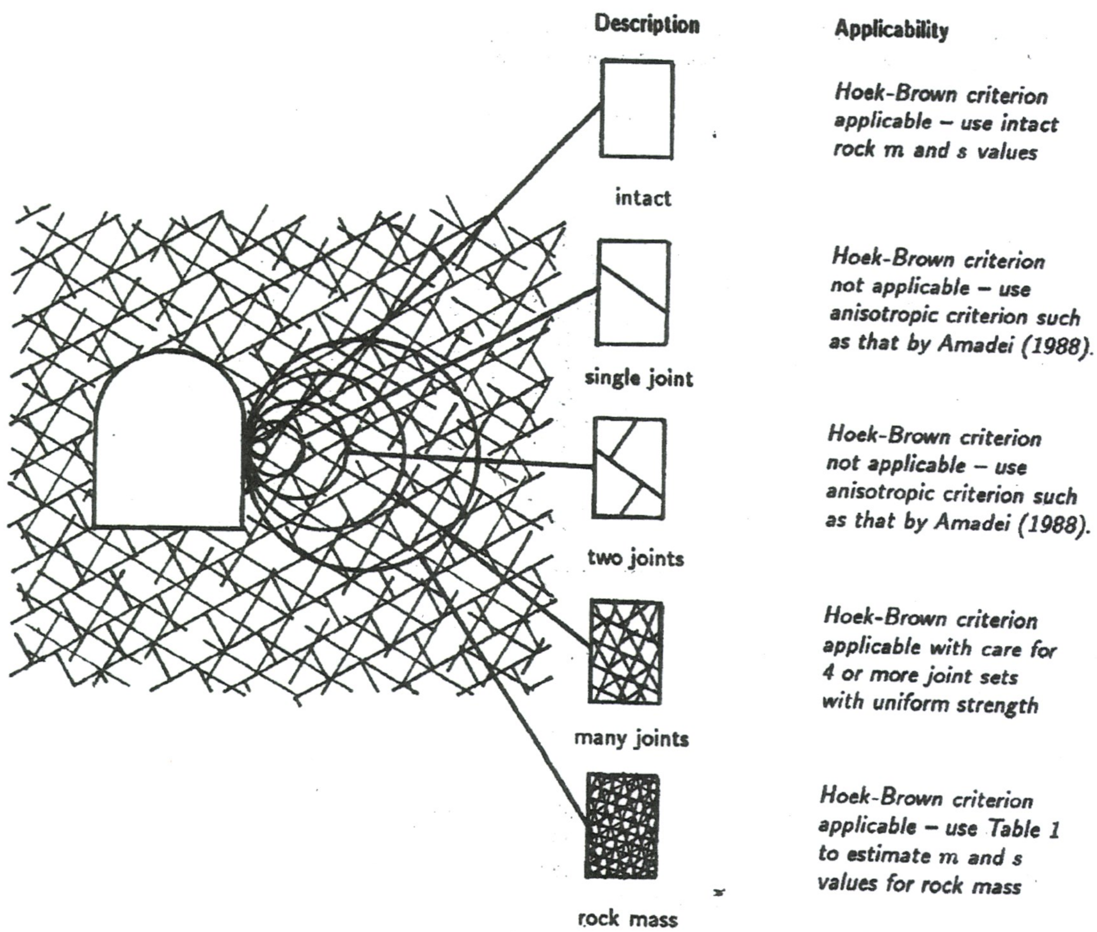

Figure 1 : Applicability of the Hoek-Brown criterion to different scales of rock mass.

Hoek (1983) showed that the criterion could be modified to allow for two-dimensional anisotropy and, more recently, Amadei (1988) published a detailed discussion on the strength of a regularly jointed rock mass subjected to three-dimensional stresses. When analyzing the stability of a tunnel where the span or height of the tunnel is only two or three times the spacing of the discontinuities in the rock mass, an anisotropic criterion such as that discussed by Amadei (1988) must be used. The stability of a small structurally defined wedge or block in the roof or sidewall of a tunnel will be controlled by the shear strength of the individual discontinuities and the Hoek-Brown failure criterion should not be used for the analysis of this type of problem.

When the volume of rock under consideration contains four or more closely spaced discontinuity sets and where none of these discontinuity sets is significantly weaker than any of the others, the Hoek-Brown criterion can be used and the m and s values can be estimated from Table 1. If one of the discontinuities is very weak as compared with the others, the rock mass should be treated as anisotropic and the Hoek-Brown criterion should not be used unless allowance is made for this anisotropy. This would be the case when dealing with a fault passing through a heavily jointed rock mass. The rock mass may be treated as an isotropic medium to which the Hoek-Brown criterion applies but the fault must be treated as an anisotropic weakness plane along which slip can occur at a much lower stress level than that which would cause failure in the rock mass.

The Hoek-Brown failure criterion does not contain a parameter which depends upon the size of the opening or the spacing of the discontinuities. The user is left to decide upon the applicability of the criterion on the basis of considerations such as those presented in the preceding paragraphs and illustrated in Figure 1.

Use of rock mass classifications

The rock mass classifications by Bieniawski (1974) and Barton, Lien and Lunde (1974) were developed for the estimation of tunnel support and they have been adopted by these authors for estimating m and s values because they were already available and well established in 1980 and because there appeared to be no justification for proposing yet another classification system. If one examines two alternative approaches to the design of tunnel support it becomes clear that there is a potential problem in using these existing rock mass classification systems as a basis for estimating the strength of a rock mass.

Consider a tunnel in a highly jointed rock mass subjected to an in situ stress field such that failure can occur in the rock surrounding the tunnel. When using the Tunnelling Quality Index Q proposed by Barton, Lien and Lunde (1974) for estimating the support required for the tunnel, the in situ stress field is allowed for by means of a Stress Reduction Factor. This factor can have a significant influence upon the level of support recommended on the basis of the calculated value of Q. An alternative approach to support design is to estimate the strength of the rock mass from Table 1 and to apply this estimated strength to the results of an analysis of the stress distribution around the tunnel in order to estimate the extent of zones of overstressed rock requiring support. If the Barton, Lien and Lunde classification has been used to estimate the values of m and s from Table 1, and if the Stress Reduction Factor has been used in calculating the value of Q, it is clear that the influence of the in situ stress level will be accounted for twice in the analysis.

Similar considerations apply to the Joint Water Reduction Factor in Barton, Lien and Lunde's classification and to the Ground Water term and the Rating Adjustment for Joint Orientations in Bieniawski's classification. In all cases there is a potential for double counting if these factors are not treated with care when using these classifications as a basis for estimating the strength of rock masses.

In order to minimize potential problems of the type described above, the following guidelines are offered for the selection of parameters when using rock mass classifications as a basis for estimating m and s values for the Hoek-Brown failure criterion:

Bieniawski's classification

Ratings for strength of intact rock material, RQD, spacing of joints and condition of joints – use exactly as defined in the table published by Bieniawski (1974).

Rating for groundwater – use a value of 10, equivalent to completely dry conditions. The influence of groundwater *pressure* should be taken into account in the analysis of stresses acting on the rock mass.

Rating adjustment for joint orientations – use a value of *zero* for all cases, equivalent to a very favourable joint orientation. The influence of joint orientation should be taken into account in deciding whether or not the Hoek-Brown failure criterion is applicable, as defined in Figure 1.

Barton, Lien and Lunde's classification

Rock Quality Designation (RQD), joint set number (Jn), joint roughness number (Jr) and joint alteration number (Ja) – use exactly as defined in the table published by Barton, Lien and Lunde (1974).

Joint water reduction factor (Jw) and stress reduction factor (SRF) – use a value of 1 for both of these parameters, equivalent to a dry rock mass subjected to medium stress conditions. The influence of both water pressure and stress should be included in the analysis of stresses acting on the rock mass for which failure is defined in terms of the Hoek-Brown failure criterion.

Estimate of rock mass deformability

The Hoek-Brown failure criterion deals only with failure and it does not provide any basis for estimating the deformability of a rock mass. Since deformation is sometimes as important as failure of a rock mass, the reader is referred to a publication by Serafim and Pereira (1983) in which the in situ modulus of deformation (E, in GPa) is related to Bieniawski's rock mass rating (RMR) by the following equation:

$$E = 10^{\left(\frac{RMR-10}{40}\right)} \tag{23}$$

These authors have found that equation 23 provides a reliable basis for estimating the modulus of deformation of rock masses, particularly during the early stage of a project when relatively little field information is available.

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Receipt #: 20144

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