Goathill North Final Mitigation Design Report - Part 1

August, 2004

NORWEST CORPORATION

Molycorp

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GOATHILL NORTH FINAL MITIGATION DESIGN REPORT

Submitted to: Molycorp, Inc.

August 11th, 2004

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File: 03-2368

August 11, 2004

Mr. Bill Brancard
Director Mining and Minerals Division
1220 South St. Francis Drive
Santa Fe, NM 87505

RE: Goathill North Final Mitigation Design Report – Part 1

Dear Mr. Brancard:

Molycorp and Norwest are pleased to submit this Final Design report for the Goathill North Rock Pile Mitigation project. This report presents Part 1 of the Final Mitigation Design for the Goathill North rock pile. The purpose of the mitigation is to control sliding movements that are occurring within pre-sheared materials near the weathered bedrock contact with natural colluvium and/or mine rock materials. The Part 1 design is meant to include the major stabilization earthworks and an interim surface drainage network. Following a 12 month period of confirmatory monitoring to demonstrate that the slide has been successfully stabilized a final drainage plan will be developed and submitted for agency approval as Part 2.

The information contained in this submittal is meant to follow Norwest's January, 2004 feasibility level report (Goathill North Investigation, Evaluation and Mitigation Report). There is a complete set of appendices in this report which contains new relevant technical information on the project following submission of the January, 2004 report. During the course of the work program, every effort was made to thoroughly address agency conditions and independent review comments raised over the intervening months. For reference purposes these conditions and independent review comments are summarized in Table 1 (attached to this letter) with references to the location of the responses in the report.

Work has already begun on Phase 1 of the mitigation project, which includes producing and hauling specified drain rock to the project area and placement of this drain rock to construct the under-drain. This has involved upgrades of existing roads and construction of new road segments in order to provide a safe haul route to convey the drain rock. A verbal explanation and field demonstration of the QA/QC process for drain rock quality control, and a field inspection of the initially produced drain rock, was required and completed prior to commencement of full drain rock production.

The ongoing project schedule is essentially the same as the project schedule presented in the May 26, 2004 kickoff meeting, but shifted approximately two weeks later. Therefore, the completion of the under-drain system should occur during the last week of August 2004. Subsequently, Phase 2 re-grading is scheduled to start in the last week of August or first week of September, assuming agency approval. Phase 2 re-grading is scheduled to be complete in mid-December 2004, in time for a winter break scheduled until mid-February; the necessity and duration of this break is dependent on practical considerations of preventing snow and ice accumulation in the fill.

Phase 3 regrading is currently scheduled to start in mid-February 2005 and be complete in mid-June 2005. It is likely that the fundamental components of the Phase 4 temporary drainage will be installed in the highwalls of the Phase 2 and Phase 3 cuts as these progress, however a final completion of Phase 4 is scheduled for June 2005. This schedule results in the following tentative inspection or field visit dates for the project:

- Inspection of completed under drain — last week of August 2004
- Mid point of Phase 2 Regrading — last week of October 2004
- Completion of Phase 2 Regrading — mid December 2004
- Submission of Ph 3 final surface drainage — late January 2005
- Startup of Phase 3 Regrading — mid February 2005
- Mid point of Phase 3 Regrading — mid April 2005
- Completion of Phase 3 Regrading — mid June 2005
- Completion of the construction phase — early July 2005
- Closeout meeting for GHN RPM construction — late July 2005

Please note that a continuous construction process is contingent on a timely review of this report which includes responses to the 10 conditions contained in the June 16, 2004 joint agency approval letter. The report also contains responses to comments raised by the Village of Questa's consultant (Gannett Fleming), Keith Ehlert (NMED) and the most recent stability review board letter of July 2, 2004. Molycorp understands that these are the issues most closely tied to project approvals at this time. In order to provide a coherent report the issues are addressed as part of the completed report rather than as separate items (see Table 1 attached for the appropriate report based reference). Some of the conditions were tied to approvals for Phase 3 and 4 of the project, and for completeness most responses are provided in this Final Report. Any changes resulting from construction experience and subsequent agency review will require review and approval prior to any construction activity related to those potential changes.

The project management framework is also attached to this letter for reference. Molycorp's site based project manager/field engineer is Mr. Mike Ness. Mike will oversee all of the project functions including the construction itself which will be carried out by Nielsons/Skansa, an earthworks contractor. Norwest's resident engineer of record is Mr. Ralph Vail, PE (NM). The design has been carried out by Mr. Tim Peterson and Mr. Sean Ennis, PE (NM) under the direction of Dr. Richard Dawson.

Molycorp appreciates the opportunity to present this package to the state agencies for review. It is hoped that the information is clear and adequate to allow approval to proceed with Phase 2 work on schedule.

Sincerely,

Molycorp, Inc

William L. Sharrer

Bill Sharrer
Vice President, Environmental & Public Policy

Norwest Corporation

R Dawson

Dr. Richard Dawson
Vice President, Geotechnical

Cc:

Terence Foreback (2 copies)
Mike Reed
Keith Ehlert
Charlie Gonzales
Debra Miller
Jim Kuipers
Ralph Vail
Richard Dawson
Tim Peterson
Dave Partridge
Mike Ness

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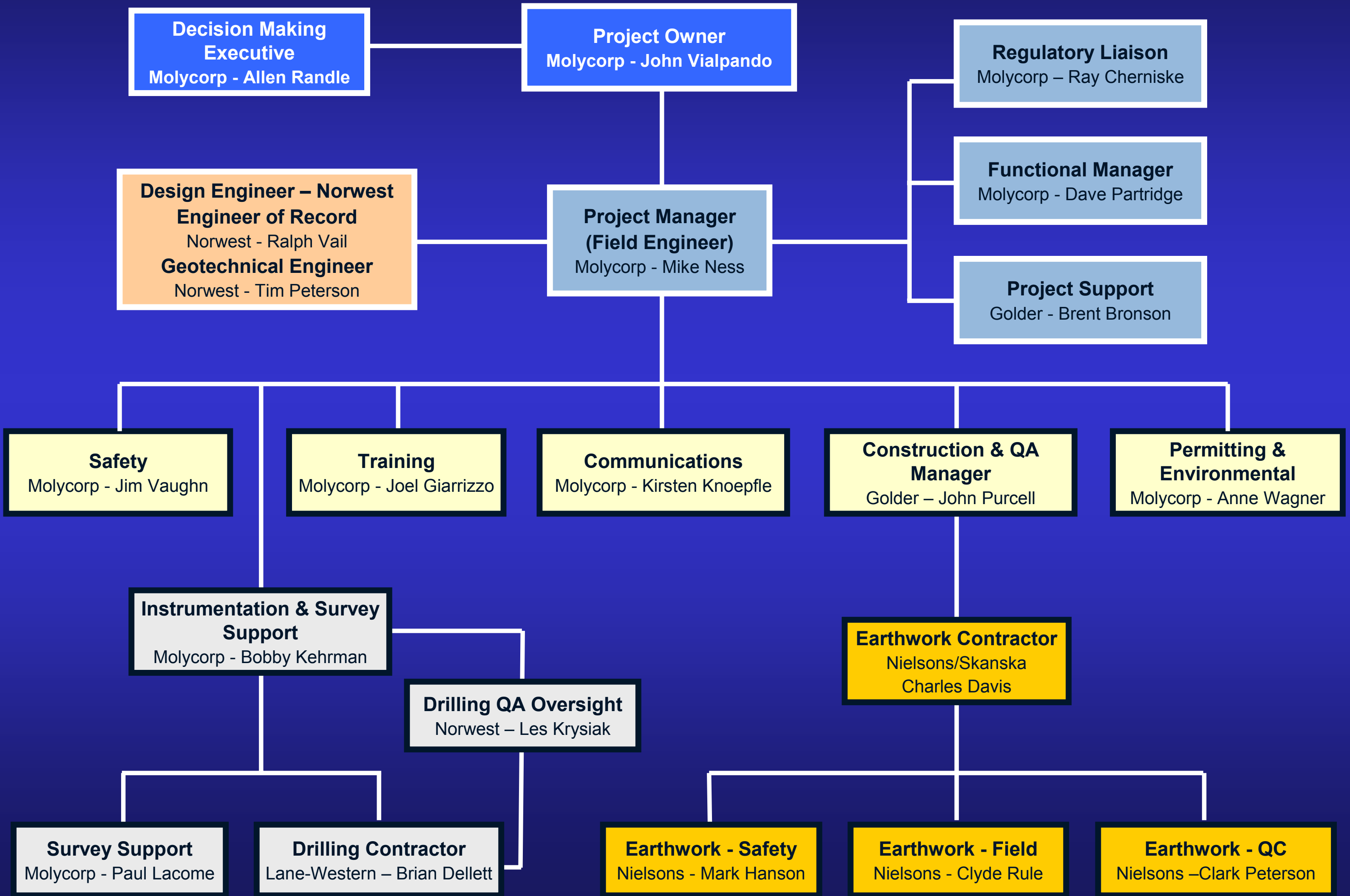
TABLE 1
SUMMARY OF REGULATORY CONDITIONS AND REVIEW COMMENTS

Source Document		Issue	Response Reference Location
Joint Agency Review Letter, Feb 20th, 2004			
Condition	#1	Final design drawings stamped by NM, P.Eng	Appendix B
	#2	Lining of plunge pools	Subsequent detailed storm water management design (part 2)
	#3	Surface water collection and treatment, if necessary	As above
	#4	Drain rock material source and QA/QC	Appendix C
	#5	Ground water collection and treatment, if necessary	Appendix C
	#6	Supplemental toe berm fill option	Appendix E
	#7	Engineer for site supervision	As per organization chart in cover letter
	#8	Contingency plans	Section 2.5.2 in Final Report
	#9	Monitoring plan for construction	Section 2 in Final Report
	#10	MSHA submission verification	MSHA to receive Final Report
	#11	Plans for placement of under drain material and field verification	Appendix C
	#12	Weekly reporting	Appendix D
	#13	As-built drawings and construction summary	Subsequent submission
Joint Agency Approval Letter, June 16, 2004, Request for Additional Information:			
Condition	#1	Minor revisions to drawings	Appendix B
	#2	Upper bench grading and drainage options	Section 2.4.1 in Final Report
	#3	Piezometer maintenance procedures	Section 2.5.1 Final Report
	#4	Trigger level justification	As above
	#5	Project oversight and kickoff meeting	Addressed at May 26th, 2004 kickoff meeting
	#6	Alternate mitigation measures	Section 2.5.2 in Final Report
	#7	Drain rock visual segregation process and qualitative criteria	Appendix C
	#8	Final instrument readings	Appendix F
	#9	Assessment of movement rates for success criteria	Section 4.0 in Final Report
	#10	Analysis of roads and ditches for stability	Section 3.2.3 in Final Report
Gannett Fleming Recommendations, July 16, 2004			
Condition	#1	Phase 2 interim stability	Section 3.2.1 in Final Report
	#2	Alternate water table assumptions	Section 3.4 in Final Report
SRB response to construction documents, July 2, 2004			
	#1	Revised stability analysis for additional cut and fill from stable pile	Section 3.0 Final Report
	#2	Definition of project "success"	Section 4.0 in Final Report

Goathill North Rock Pile Mitigation Project Organizational Chart



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EXECUTIVE SUMMARY

This report presents Part 1 of the final mitigation design for the Goathill North rock pile. The purpose of the design is to control sliding movements that are occurring within pre-sheared materials near the weathered bedrock contact with natural colluvium and/or mine rock materials. The Part 1 design is meant to include the major stabilization earthworks and an interim surface drainage network. Following a 12 month period of confirmatory monitoring to demonstrate that the slide has been successfully stabilized, a final drainage plan will be developed and submitted for agency approval as Part 2.

The work program was initiated in June, 2003 and the design was previously submitted at the feasibility level (Goathill North Investigation, Evaluation and Mitigation Report, January, 2004). The January, 2004 report complete with it's appendices of data presents the slide mechanics and a feasibility level design that provides the basis for the final design presented here.

Design Criteria

The objective of the mitigation plan is to control ongoing foundation sliding movements and other foundation shearing mechanisms related to movements along the weak layer that controls the active slide. This objective is addressed with the following design criteria:

1. Stability criteria based on static limit equilibrium analyses to achieve a safety factor greater than 1.2.

2. An interim water management plan for controlling runoff on the re-graded surface.

3. The design has been developed so that so that slope performance can be closely monitored during and after construction. This monitoring plan forms an integral part of the design.

Due to the re-grading requirements to meet stability requirements and the additional re-grading for accommodating reclamation issues, final slopes are relatively shallow. Thus the mitigation design will also provide benefits for long term stability. Further geotechnical work will be required to check for long term stability against all potential failure modes (including seismic stability) as part of the closure planning process.

Slide Mechanism Overview

Goathill North Rock Pile was constructed in an area characterized by alteration scars. The toe of the pile is founded on a colluvium bench underlain by pre-sheared material. Foundation movement associated with the initial development of the slide occurred between 1969 and 1973, and continues to occur after more than 30 years since their initiation.

Weak foundation conditions associated with pre-shearing are attributed to high bedrock pyrite contents which produce acid drainage and alteration minerals that are more susceptible to weathering. In addition, there is a shallow water table in the weathered zone at Goathill North which contributes pore pressures to trigger slide movements. The seepage also acts as the solution for chemical weathering and the medium for transporting away any dissolved components of the weathered rock.

The total slide volume at Goathill North, based on the slip surface and topographic contours is 2.0 million yds^3, comprised of 1.4 million yds^3 of mine rock and 0.6 million yds^3 of valley colluvium. For comparison with these volumes, the total volume of mine rock material within the Goathill North rock pile is about 4.4 million yds^3. The slide zone is about 50 to 75 feet thick and sliding is occurring along a surface that is dipping at about 20 degrees beneath the colluvium bench and about 30 degrees beneath the rock pile.

Back analysis of the pre-sheared sliding surface, based on current pore pressures and slope indicators to define the sliding geometry, indicate that the residual strength is about 24°. This value is similar to laboratory measured strengths from direct shear tests of slide zone materials and consistent with the characteristics (grain size and plasticity indices) for the weathered materials at residual strength.

Construction Summary

The plan in its entirety involves a balanced cut and fill of approximately 1 million yds^3. All of the cut and fill work will be done with dozers pushing material in a down slope direction. The dozers will work from a platform pushing material out over the crest of the platform. The platform will be sliced away by the dozer activity so that it moves down the slope at successively lower elevations. At the bottom of the slope the fill platform is spread out to achieve the final re-graded surface. Although the fill is re-handled several times with the dozers there is no need to develop access for trucks and to transport material up and down haul ramps and switch backs. This construction method is safer than a truck/shovel operation because there is no equipment working at or near the toe of the landslide.

The final design retains the same overall design concept and phasing sequence as previously presented at a feasibility level in the January, 2004 report. Table 2.1 shows a summary of the cut and fills for each of the 4 phases of the Part 1 construction. Part 2 is the detailed water management plan which will be submitted under separate cover once the Part 1 works are completed. A brief description of each phase is as follows:

1. Phase 1 involves placing the rock drain the erosion gully at the toe of the slide area. The drain will be constructed from 20,000 yd^3 of imported processed (crushed and screened) coarse granular material.

2. Phase 2 involves a large 510,000yd^3 cut on the stable pile with the fill placed as an initial toe buttress in the erosion gully where the shear surface daylights. This phase is designed to achieve some initial stabilization of current slide movements before placing people and equipment on the landslide itself. The cut also significantly improves the stability of the stable pile.

3. Phase 3 involves unloading the landslide by pushing 440,000yd^3 on the upper portion of the slide down to the 9665 ft elevation and then re-grading the slope below this point by dozing down an additional 75,000yd^3 to achieve an overall gradient of between 2H:1V and 3H:1V. The upper unload will remove all the material down to the base of the shear surface leaving behind a final slope in original ground at a gradient of 1.5H:1V.

4. Phase 4 involves constructing an interim drainage system on the mitigated surface in order to minimize erosion while the fill settles and monitoring information is collected (for about 12 months) to demonstrate that the mitigation has been successful.

SUMMARY OF PROPOSED MITIGATION PHASES

Phase	Operation	Location	Cut/Fill	Finished Slope	Volume (yd^3)
1	Rock pile under drain construction	Main valley drainage at toe of rock pile	Imported fill	Follows topography	20,000
2	Stable pile cut and initial toe buttress fill	Stable south rock pile slope	Cut 'A'	2H:1V	510,000
		Initial toe buttress fill	Fill 'B'	1.5H to 2.5H:1V	510,000
3	Slide unloading and regrading and final buttress fill and toe berm	Upper slide unloading	Cut 'C'	Follows shear Plane	440,000
		Upper slide regrading	Cut 'D'	2H and 2.5H:1V	75,000
		Upper slide regrading	Fill 'E'	2H and 2.5H:1V	50,000
		Final buttress toe berm fill	Fill 'F'	1.5H to 2.5H:1V	465,000
4	Interim surface water controls	Rock pile, colluvium and buttress fill slopes	Cut and fill	1.5H to 2.5H:1V	5,000*

* assuming rough grading included in Phase 3.

The construction monitoring program has been developed to record displacements and piezometric pressures at strategic locations over the mitigation site for each phase of the project. Monitoring readings will be taken and evaluated daily for compliance within triggering criteria designed to manage construction safety. Shear displacements will be measured by slope inclinometers with trigger levels based on several months of slide performance data from the investigation program.

Stability Summary

Stability analysis has been carried out to determine the factor of safety (FOS) for interim periods within each phase and at the end of construction. Sliding is currently taking place along a curved trajectory which is normal to the mine rock pile at the head of the slide and rotates out of the down valley direction at the leading edge of the slide. The trend of the daylighted shear plane trace in the erosion gully is a manifestation of this curved sliding trajectory.

The factors of safety for interim construction conditions show that there is a general trend of gradual improvements to stability during construction. The analyses indicated that there was an unfavourable interim geometry during the Phase 3 push down and the profile for this geometry has been adjusted with appropriate construction recommendations.

At the end of construction, the average two dimensional FOS along the current shear surface is 1.32 which represents a 32% improvement in the direction of current sliding. The critical slip surface (determined from a grid search) shows that the FOS is 1.23 which satisfies the minimum 1.2 FOS design criteria.

At the end of construction, the critical (determined from a grid search) two dimensional factor of safety in the down valley direction is 1.12. The mitigation measures provide significant lateral resistance to down valley sliding and three dimensional analysis is used to show that the factor of safety in the down valley direction is 1.30. On this basis the minimum FOS design criteria of 1.2 is exceeded.

Project Success Criteria

The granular colluvium and rock pile materials that currently overly the shear surface and the rock pile fill that will be used to form the buttress are not rigid materials. As a result there will be movements within and above the shear surface for a fairly long period of time (several years) after construction is complete. Norwest has defined three independent criteria for assessing project success, based on slope inclinometer (SI) measurements:

- The SI instrumentation placed beyond the current slide boundary should not show any signs of discrete basal shearing near the colluvium or mine rock contact with weathered bedrock materials.

- The SI shearing rates within the slide boundary should generally show a steadily decreasing trend with time in a manner which is consistent with the sequence and distribution of the fill

- In general, basal shear movement rates after one year should have decreased by about one order of magnitude, based on the SI measurements.

1 INTRODUCTION

1.1 TERMS OF REFERENCE

This report presents Part 1 of the final mitigation design for the Goathill North rock pile. The purpose of the design is to control sliding movements that are occurring within pre-sheared materials near the weathered bedrock contact with natural colluvium and/or mine rock materials. The Part 1 design is meant to include the major stabilization earthworks and an interim surface drainage network. Following a 12 month period of confirmatory monitoring to demonstrate that the slide has been successfully stabilized a final drainage plan will be developed and submitted for agency approval as Part 2.

The work was initiated in June, 2003 after concerns were raised by the state appointed Stability Review Board (SRB) regarding potential down-valley risks associated with the foundation shearing movements. Norwest addressed some of these concerns in a letter report on July 15, 2003 and proposed a program of investigation for assessing the movements. A conceptual mitigation plan was also proposed, subject to results from the field investigations and more detailed analyses and evaluations. Since Norwest's submission of the July, 2003 letter an extensive investigation program was carried out followed by a feasibility level design (Goathill North Investigation, Evaluation and Mitigation Report, January, 2004). The January, 2004 report complete with it's appendices of data presents the slide mechanics and a feasibility level design that provides the basis for the final design presented here.

1.2 DESIGN CRITERIA

The objective of the mitigation plan is to control ongoing foundation sliding movements and other foundation shearing mechanisms related to movements along the weak layer that controls the active slide. This objective is addressed with the following design criteria:

1. Stability criteria have been developed based on static limit equilibrium analyses to achieve a safety factor greater than 1.2. The safety factor's are mostly calculated using shear strengths calculated from back analyses so that the values are directly related to the relative increase in stability f0rom the current unstable to the mitigated stable condition. Thus a safety factor of 1.2 is equivalent to a 20% increase over the limit equilibrium condition. Additional analyses has been carried out to address alternative pore pressure assumptions and currently stable portions of the mine rock pile.

2. The design includes an interim water management plan for controlling runoff on the re-graded surface. Following a 12 month period of confirmatory monitoring to demonstrate

that the slide has been successfully stabilized a final drainage plan will be developed and submitted for agency approval as Part 2. It is planned that the final water management plan will follow the design developed at the feasibility level (Phase 4 of the January, 2004 report).

3. The design has been developed so that so that slope performance can be closely monitored during and after construction. This monitoring plan forms an integral part of the design included herein.

Due to the re-grading requirements to meet stability requirements and the additional re-grading for accommodating reclamation issues, final slopes are relatively shallow. Thus the mitigation will also provide benefits for long term stability. Further geotechnical work will be required to check for long term stability against all potential failure modes (including seismic stability) as part of the closure planning process.

1.3 SLIDE MECHANISM OVERVIEW

The slide mechanics were evaluated and discussed in the January, 2004 report (Sections 2 to 5) and the detail is not repeated here. For completeness a brief overview, excerpted from the previous report, is presented here.

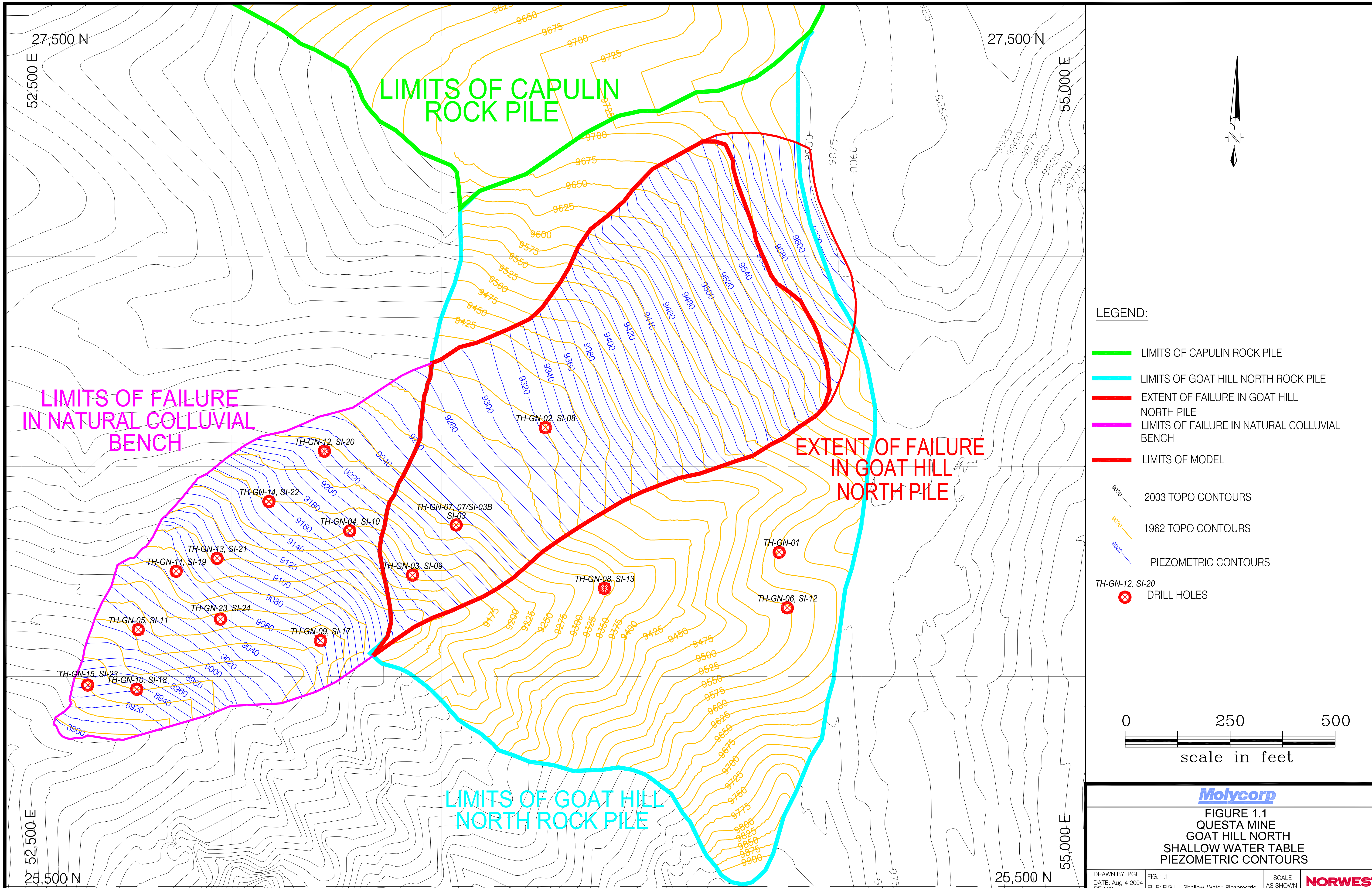
Figures 1.1 and 1.2 illustrate shallow water table contours and shear surface contours respectively that were used to define the current slide mechanism for back analysis and stability of the mitigation works. These surfaces have not been changed since the feasibility level design.

Goathill North Rock Pile was constructed in an area characterized by alteration scars. The toe of the pile is founded on a colluvium bench underlain by pre-sheared material. Foundation movement associated with the initial development of the slide occurred between 1969 and 1973, and continues to occur after more than 30 years since their initiation.

Weak foundation conditions associated with pre-shearing are attributed to high bedrock pyrite contents which produce acid drainage and alteration minerals that are more susceptible to weathering. In addition, there is a shallow water table in the weathered zone at Goathill North which contributes pore pressures to trigger slide movements. The seepage also acts as the solution for chemical weathering and the medium for transporting away any dissolved components of the weathered rock.

The total slide volume at Goathill North, based on the slip surface and topographic contours is 2.0 million yds^3, comprised of 1.4 million yds^3 of mine rock and 0.6 million yds^3 of valley colluvium. For comparison with these volumes, the total volume of mine rock material within the Goathill North rock pile is about 4.4 million yds^3. The slide zone is about 50 to 75 feet thick and sliding is occurring along a surface that is dipping at about 20 degrees beneath the colluvium bench and about 30 degrees beneath the rock pile. The geometry and displacement vectors indicate that the landslide can be classified as a "translational slide" and that the velocity class is "very slow" according to criteria proposed by Cruden and Varnes (1996).

Back analysis of the pre-sheared sliding surface, based on current pore pressures and slope indicators to define the sliding geometry, indicate that the residual strength is about 24°. This value is similar to laboratory measured strengths from direct shear tests of slide zone materials and consistent with the characteristics (grain size and plasticity indices) for the weathered materials at residual strength.


LIMITS OF CAPULIN ROCK PILE
LIMITS OF FAILURE IN NATURAL COLLUVIAL BENCH
EXTENT OF FAILURE IN GOAT HILL NORTH PILE
LIMITS OF GOAT HILL NORTH ROCK PILE
27,500 N
25,500 N
52,500 E
55,000 E
TH-GN-01
TH-GN-02, SI-08
TH-GN-03, SI-09
TH-GN-04, SI-10
TH-GN-05, SI-11
TH-GN-06, SI-12
TH-GN-07, 07/SI-03B SI-03
TH-GN-08, SI-13
TH-GN-09, SI-17
TH-GN-10, SI-18
TH-GN-11, SI-19
TH-GN-12, SI-20
TH-GN-13, SI-21
TH-GN-14, SI-22
TH-GN-15, SI-23
TH-GN-23, SI-24
LEGEND:
LIMITS OF CAPULIN ROCK PILE
LIMITS OF GOAT HILL NORTH ROCK PILE
EXTENT OF FAILURE IN GOAT HILL NORTH PILE
LIMITS OF FAILURE IN NATURAL COLLUVIAL BENCH
LIMITS OF MODEL
2003 TOPO CONTOURS
1962 TOPO CONTOURS
PIEZOMETRIC CONTOURS
TH-GN-12, SI-20
DRILL HOLES
0
250
500
scale in feet
Molycorp
FIGURE 1.1
QUESTA MINE
GOAT HILL NORTH
SHALLOW WATER TABLE
PIEZOMETRIC CONTOURS
DRAWN BY: PGE
DATE: Aug-4-2004
REV.00
FIG. 1.1
FILE: FIG1.1_Shallow_Water_Piezometric
SCALE
AS SHOWN
NORWEST

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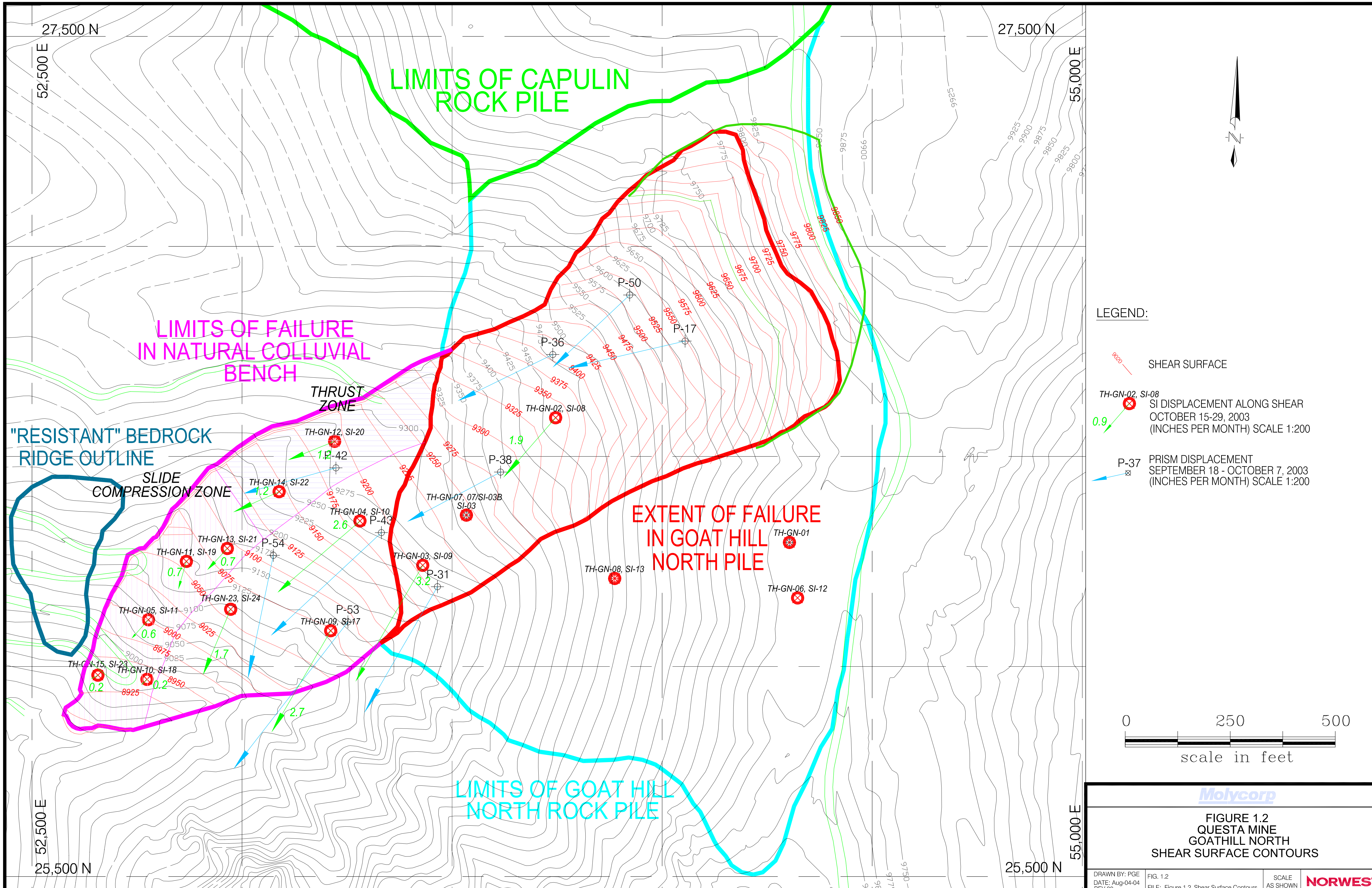
27,500 N
52,500 E
55,000 E
25,500 N
LIMITS OF CAPULIN ROCK PILE
LIMITS OF FAILURE IN NATURAL COLLUVIAL BENCH
THRUST ZONE
"RESISTANT" BEDROCK RIDGE OUTLINE
SLIDE COMPRESSION ZONE
EXTENT OF FAILURE IN GOAT HILL NORTH PILE
LIMITS OF GOAT HILL NORTH ROCK PILE
P-50
P-17
P-36
P-38
P-42
P-43
P-54
P-31
P-53
TH-GN-02, SI-08
TH-GN-12, SI-20
TH-GN-14, SI-22
TH-GN-04, SI-10
TH-GN-07, 07/SI-03B
SI-03
TH-GN-13, SI-21
TH-GN-11, SI-19
TH-GN-03, SI-09
TH-GN-01
TH-GN-08, SI-13
TH-GN-06, SI-12
TH-GN-23, SI-24
TH-GN-05, SI-11
TH-GN-09, SI-17
TH-GN-15, SI-23
TH-GN-10, SI-18
LEGEND:
SHEAR SURFACE
SI DISPLACEMENT ALONG SHEAR OCTOBER 15-29, 2003 (INCHES PER MONTH) SCALE 1:200
PRISM DISPLACEMENT SEPTEMBER 18 - OCTOBER 7, 2003 (INCHES PER MONTH) SCALE 1:200
0
250
500
scale in feet
Molycorp
FIGURE 1.2
QUESTA MINE
GOATHILL NORTH
SHEAR SURFACE CONTOURS
DRAWN BY: PGE
DATE: Aug-04-04
REV.00
FIG. 1.2
FILE: Figure 1.2_Shear Surface Contours
SCALE AS SHOWN
NORWEST

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2 EARTHWORKS CONSTRUCTION AND MONITORING PLAN

This section describes each of the construction phases necessary for completing the design and explains the monitoring program that is an integral part of the construction process. Supporting stability analyses follow in Section 3 and some of the results are referred to here in order to identify interim phases of construction that will require special attention. Considerable effort has been made to balance cut and fill requirements within the Goathill North Valley and ensure that the sequencing of cut and fill maintains stable and safe conditions during construction.

All of the cut and fill work will be done with dozers pushing material in a down slope direction. The dozers will work from a platform pushing material out over the crest of the platform. The platform will be sliced away by the dozer activity so that it moves down the slope at successively lower elevations. At the bottom of the slope the fill platform is spread out to achieve the final re-graded surface. Although the fill is re-handled several times with the dozers there is no need to develop access for trucks and to transport material up and down haul ramps and switch backs. This construction method is safer than a truck/shovel operation because there is no equipment working at or near the toe of the landslide.

The final design retains the same overall design concept and phasing sequence as previously presented at a feasibility level in the January, 2004 report. In accordance with the design criteria, final slope profiles have been designed to increase the stability of the overall sliding mass by a minimum of 20% in order to achieve a factor of safety (FOS) of 1.2 at the end of major earthworks construction. Table 2.1 shows a summary of the cut and fills for each of the four phases of the Part 1 construction. Part 2 is the detailed water management plan which will be submitted under separate cover once the Part 1 works are completed. A brief description of each phase is as follows:

- Phase 1 (Figure 2.1) involves placing the rock drain in the erosion gully at the toe of the slide area. The drain will be constructed from 20,000 yd^3 of imported processed (crushed and screened) coarse granular material.

- Phase 2 (Figure 2.2) involves a large 510,000 yd^3 cut on the stable pile with the fill placed as an initial toe buttress in the erosion gully where the shear surface daylights. This phase is designed to achieve some initial stabilization of current slide movements before placing people and equipment on the landslide itself. The cut also significantly improves the stability of the stable pile. It is noted that this cut is much larger (about 310,000 yd^3) than the Phase 2 cut presented in the January, 2004 report. The main reason is to reduce the whole slope on the stable side to 2H:1V so that it is at the same gradient as the final slopes on the unstable portion. This achieves a more consistent slope profile across the rock pile for reclamation.

There is no decrease in the stability of the Phase 2 interim slopes due to this extra material (see Section 2 for detailed interim construction stability analysis).

- Phase 3 (Figure 2.3) involves unloading the landslide by pushing 440,000 yd^3 from the upper portion of the slide down to the 9665 ft elevation and then re-grading the slope below this point by dozing down an additional 75,000 yd^3 to achieve an overall gradient of between 2H:1V and 3H:1V. The upper unload will remove all the material down to the base of the shear surface leaving behind a final slope in original ground at a gradient of 1.5H:1V.

- Phase 4 (Figure 2.4) involves constructing an interim drainage system on the mitigated surface in order to minimize erosion while the fill settles and monitoring information is collected to demonstrate that the mitigation has been successful. It is expected that this will take about 12 months and after this period a detailed surface water management plan will be developed (Part 2 of the final design) and submitted for approval. At this time it is anticipated that the final plan will follow design concept to that presented as Phase 4 in the January, 2004 report.

TABLE 2.1
SUMMARY OF PROPOSED MITIGATION PHASES

Phase	Operation	Location	Cut/Fill	Finished Slope	Volume (yd^3)
1	Rock pile under drain construction	Main valley drainage at toe of rock pile	Imported fill	Follows topography	20,000
2	Stable pile cut and initial toe buttress fill	Stable south rock pile slope	Cut 'A'	2H:1V	510,000
		Initial toe buttress fill	Fill 'B'	1.5H to 2.5:1V	510,000
3	Slide unloading and regrading and final buttress fill and toe berm	Upper slide unloading	Cut 'C'	Follows shear Plane	440,000
		Upper slide regrading	Cut 'D'	2H and 2.5H:1V	75,000
		Upper slide regrading	Fill 'E'	2H and 2.5H:1V	50,000
		Final buttress toe berm fill	Fill 'F'	1.5H to 2.5:1V	465,000
4	Interim surface water controls	Rock pile, colluvium and buttress fill slopes	Cut and fill	1.5H to 2.5:1V	5,000*

* assuming rough grading included in Phase 3.

The plan in its entirety involves a balanced cut and fill of 1,025,000 yd^3. Further discussion of the construction details for each phase and the construction monitoring program designed to ensure safe operating conditions follows. The detailed design drawing and construction specifications are included as Appendix B.

2.1 CONSTRUCTION OF ROCK UNDER-DRAIN – PHASE 1

2.1.1 Phase 1 Construction Sequence

The first phase of construction will be the installation of an under drain within the existing drainage gully below the toe of the GHN rock pile. Figure 2.1 shows the rock drain footprint and the detailed layout of this rock drain is shown in Drawing 3 of the final design drawings (Appendix B). Appendix C contains detailed information on the rock drain source area, physical and chemical properties, and quality control/placement procedures.

Rock material required for the drain will be processed by the contractor at Spring Gulch as per the quality control procedures specified in Appendix C. Care must be taken to ensure that any potentially acid generating (PAG) mixed volcanics (characteristically yellow from jarosite staining) and any porphyry and Andesite with abundant pyrite, are not processed. This under drain material will be hauled directly to the proposed placement locations with no intermediate transfer points or stockpiling.

During this phase, trees, brush and other vegetation will be cleared from the areas where rock fill will be dozed in Phases 2 and 3.

2.1.2 Monitoring During Phase 1

The gully in which the rock drain is to be constructed is adjacent to a natural scar slope that may be susceptible to isolated rock falls, primarily during storm and high run-off events. This potential hazard will be monitored visually on a daily basis and access will be restricted near the toe of the scar slope. A 20 ft wide catch ditch will be left between the placed drainage rock and the toe of the scar. In addition, a safety berm of drainage rock at least 6 ft high will always be left in place at the crest of the catch ditch. The final design drawings show detailed cross sections for the rock drain that show the configuration of the catch ditch and safety berm (see Drawing 3 of the final design drawings in Appendix B).

As part of the monitoring program integral to this plan, a drill rig and crew will install instrumentation within the Mitigation Site Area. This will include two slope inclinometer (SI) holes placed outside the slide area and three new SI holes inside the slide footprint to replace existing holes where the SI's have sheared off due to slide movements (see Figure 2.1 for hole locations). These instruments must be put in place before the Phase 2 and later phases can begin.

2.2 GRADING OPERATION IN STABLE ROCK PILE AREA – PHASE 2

2.2.1 Phase 2 Construction Sequence

When the full section of the rock drain is complete and all instrumentation required prior to Phase 2 has been installed and baselines set, grading operations on the stable south side of the Goathill North Rock Pile can begin. Rock pile material will be excavated from the upper part of the stable pile and dozed towards the west, directly down the south side of the pile maintaining a rehandle bench and a 2H:1V finished cut slope. At the toe of the rock pile this material will be moved further down slope to cover the rock drain constructed during Phase 1. The rock fill will form an initial buttress with finished slopes varying from 1.5H:1V at the down valley toe to 2.5H:1V and 2H:1V in the upper part of the buttress. Figure 2.2 shows the Phase 2 cut and fill and the detailed design is shown on Drawing 4 of the final design drawings (Appendix B).

The detailed drawings show approximate contours for interim phases of construction. Stability analyses in Section 3 show that, with one exception, there is no significant decrease in the FOS through all of the interim Phase 2 stages. There is a small decrease in the FOS values, from 1.18 to 1.12 (Phase 2a to Phase 2b), as the pile is pushed down to the 9250ft elevation. Although there is still a reserve of stability compared with initial conditions during Phase 2 it is noted that special attention to the monitoring program should be observed during this period. Beyond this point there is a significant increase in the FOS values and the analysis shows that FOS values at the end of Phase 2 are greater than 1.4 for the stable pile slopes.

Figure 2.2 shows two zones where access will be excluded (No Access Zone) or limited (Limited Access Zone) during Phase 2 construction, according to the specification contained in Appendix B. This will prevent and/or limit access to the active landslide area until the initial stabilizing buttress is in place.

2.2.2 Monitoring During Phase 2

Continued attention to rock fall hazards from the natural scar slope are required throughout the Phase 2 period.

Drillhole instrumentation installed prior to Phase 2 construction will be used to provide warning indicating unsafe working conditions on the Phase 2 cut and fill slope. At the start of Phase 2 construction there will be 7 holes available for monitoring (see locations for these holes on the Phase 1 drawings) and at the end of Phase 2 construction there will be 6 monitoring holes available (one hole will be covered up by the Phase 2 fill) as shown on Figure 2.2 and Drawing 3. The rate of shearing and pore pressure information obtained from these holes will be used to govern working conditions on the slope according to the trigger criteria and contingency measures described in Section 2.5 below.

Two additional holes will also be installed prior to initiating Phase 3 construction, also shown on Figure 2.2.

2.3 GRADING OPERATION IN PREVIOUSLY BUTTRESSED ROCK PILE AREA – PHASE 3

2.3.1 Phase 3 Construction Sequence

Following completion of Phase 2, cut and fill operations on the active slide area will begin for Phase 3 construction. Figure 2.3 shows the Phase 3 cut and fill and the detailed design is shown on Drawing 5 of the final design drawings (Appendix B). There are two main construction operations during this phase of construction:

2.3.2 Phase 3 Unload

The first construction operation during Phase 3 is to unload the upper part of the slide down to the 9665 ft elevation. The upper unload will remove all the material down to the base of the shear surface leaving behind a final slope in original ground at a gradient of 1.5H:1V. The contours shown on Figure 2.3 and Drawing 5 are a best estimate of a reasonable final slope configuration based on the location of this failure surface interpreted from drilling, surface crack exposure, and back analysis of the moving rock pile. Field conditions will differ somewhat, and cannot be determined exactly until the re-grading cut begins to expose the failure surface. The best interpretation of bedrock conditions available anticipates altered Amalia tuff for the north and northeast faces of the cut, and this material although soft and weathered stands naturally at face angles of 1.33H:1V and steeper. Therefore, a stable highwall cut of 1.5:1 (shallower than mine rock angle of repose) should be constructible in this area. As the backslope face is exposed, a large excavator will be use to dress the final slope, therefore ample opportunity will be available to construct whatever configuration might be required to ensure stability.

2.3.3 Phase 3 Re-grading

After establishing a bench at the 9665 ft level, cut material will continue to be pushed by dozers directly down the north side of the rock pile, and onto the upper part of the initial buttress fill leaving a regraded final slope of between 2H:1V and 3H:1V. This material shall then be moved further down slope and placed to form the final buttress fill with finished slopes varying from 1.5H to 3H:1V.

The detailed drawings show approximate contours for interim phases of construction. Stability analyses contained in Section 3 show that with one exception, there is no significant decrease in the FOS through all of the interim Phase 3 stages. The analysis did reveal that the 3C interim profile with an angle-of-repose slope could have marginal local stability. Therefore the profile for this phase was modified to 2H:1V to achieve a more stable interim condition for construction. This means that the advancing fill slope below the 9450 ft elevation (i.e. Phase 3b) will need to be pushed out to a lower slope angle to achieve a stable interim condition.

Figure 2.3 shows a Limited Access Zone during Phase 2 construction, where construction activities will be managed according to the specification contained in Appendix B.

2.3.4 Monitoring During Phase 3

Continued attention to rock fall hazards from the natural scar slope are required throughout the Phase 3 period.

Drillhole instrumentation installed prior to Phase 3 construction will be used to provide warning of any unsafe working conditions on the Phase 3 cut and fill slope. Throughout the Phase 3 there will be 8 instrumented holes available for monitoring as shown on Figure 2.3 and Drawing 4. At this stage it is possible that some of the holes will be sheared off. If deemed necessary they will be replaced. The rate of shearing and pore pressure information obtained from these holes will be used to govern working conditions on the slope according to the trigger criteria and contingency measures described in Section 2.5 below.

2.4 GRADING AND CONSTRUCTION OF INTERIM SURFACE WATER CONTROL STRUCTURES – PHASE 4

2.4.1 Phase 4 Construction Sequence

Phase 4 involves constructing an interim drainage system on the mitigated surface in order to minimize erosion while the fill settles and monitoring information is collected to demonstrate that the mitigation has been successful. It is expected that this monitoring will take about 12 months and after this period a detailed surface water management plan will be developed (Part 2 of the final design) and submitted for approval. At this time it is anticipated that the final plan will follow the design concept presented as Phase 4 in the January, 2004 report. Figure 2.4 shows the Phase 4 cut and fill the detailed design is shown on Drawing 6 of the final design drawings (Appendix B).

The main features of the interim drainage plan are the 9665 ft bench with associated runoff control measures and the road cuts which will also be used to direct flow away from the slide area. The philosophy of the current design is to efficiently utilize a minimum number of interceptor ditches to collect and divert surface runoff to virgin ground northwest of the project area. Since these ditches also serve as the roadways to install the final monitoring drill holes, their location is based on more than just hydrological considerations. However the number and location of ditches is a balance between positive and effective runoff interception, and the avoidance of excessive concentration of surface flow. It is likely and expected that as information is gained from the existing re-vegetation test plot programs and the current interim stability surface water control programs on other rock piles, the configuration for surface water control on the Goat Hill North Rock Pile Mitigation project could change to incorporate the latest technology and experience available.

There have been considerable discussions with state agencies and independent reviewer's regarding the control of surface runoff on the 9665 ft bench. The main issues are minimizing infiltration into the shear surface at the back of the bench and preventing erosion in the fill below due to the runoff from the natural slopes above. The current design shows that the bench will be sloped out at about 1% grade and there will be an interception ditch at the bench crest which will carry runoff to the north outside the rock pile area. Additional measures for minimizing infiltration into the shear surface can also be considered once the contact between the bench and the weak zone is at final grade. This could involve placing a fillet of compacted low permeability fill along the contact. It may also be necessary to increase the gradient on the bench to increase runoff. These details are best dealt with during Phase 4 construction. Additional analyses would not be meaningful at this time.

The Phase 4 road cuts are currently designed as 12 foot running surfaces with an upper 1.5H:1V cut slopes. This cut slope is less than the angle-of-repose (1.33H:1V).

2.4.2 Monitoring During Phase 4

Continued attention to rock fall hazards from the natural scar slope are required throughout the Phase 4 period.

There will be up to 8 monitoring holes which will be used to provide warning indicating unsafe working conditions during Phase 4 construction (see Figure 2.4). At this stage it is likely that some of the holes will be sheared off. If deemed necessary they will be replaced. The rate of shearing and pore pressure information obtained from these holes will be used to govern working conditions on the slope according to the trigger criteria and contingency measures described in Section 2.5 below.

Four additional holes will be placed during and/or after the end of Phase 4 construction (see figure 2.4 and Drawing 6). The monitoring holes at the end of Phase 4 will be used to monitor the performance of the mitigation after construction is completed. Discussion of the post construction monitoring program and definition of project success criteria is contained in Section 4.

2.5 CONSTRUCTION MONITORING AND CONTINGENCY MEASURES

2.5.1 Drillhole Monitoring Installations

The primary instrumentation that will be used for construction and post construction monitoring are drillholes containing slope inclinometer (SI) casing and vibrating wire piezometers. The SI's are the best indicator of shearing movements on the landslide, although they may need to be periodically replaced because the casing shears off. Each SI installation will contain at least two vibrating wire piezometers. One piezometer will be located at or near the bottom of the hole in bedrock, and at least one piezometer will be located at the critical elevation of the shear surface or base of colluvium, as appropriate. Details of the installations completed to date are contained in the appendices submitted as part of the January, 2004 report.

There have been some concerns expressed regarding the serviceability of vibrating wire piezometers for use as part of the Goathill North monitoring program. The following notes have been prepared to address these concerns:

The utilization of electrically operated vibrating wire (VW) piezometers remains the best choice for longest life of measurement capability in sliding ground such as in the Goat Hill North Rock Pile Mitigation project (GHN RPM). The cable sending information from the instrument to the surface of the hole can withstand significantly more shearing than alternatives, such as standpipe piezometers, and can still provide information for some time after the drill hole has sheared off completely. Approximately 7 of 44 GHN VW piezometers have failed due to shearing of the cable (~ 15%). This result is superior to what would have been observed with other types of piezometers.

Some lightning damage has occurred with the network of VW piezometers at GHN (a loss of an additional 15% of the units), despite the use of appropriate surge protecting devices on each instrument at the collar of every installation. The incident is still being investigated, but logically, the current flowed through one or more instruments in the ground first, and then up to the surface and through the remainder of one branch of instrument wires to the EWS shed. None of the newly installed VW piezometers for the GHN RPM project have been wired to the EWS shack – and none of these units were damaged in the lightning event. The concept of real time telemetered data acquisition and remote monitoring of groundwater levels appears to have been an example of complication of a system to provide a capability of questionable value, while reducing the durability of the overall system. The GHN RPM project has avoided this defect by utilizing individual battery operated "mini-loggers" installed on each instrument, and these are downloaded daily or weekly depending on their function.

All new piezometer installations have been serviced and checked by soaking the piezometers in water for 24 hours prior to installation to remove all air pockets and bubbles. Readings were taken at the known head during soaking and documented for comparison with later readings. The piezometers are accurate to approximately 0.15 feet of head, and the newly installed piezometers all met that criterion. The purpose of the piezometer installations is to identify changes (particularly increases) in groundwater pressure relative to initial readings, once the drilling record and the initial readings have ascertained the existence and extent of groundwater.

2.5.2 Trigger Criteria and Contingency Measures

The monitoring program has been developed to record displacements and piezometric pressures at strategic locations over the mitigation site as described for each phase of construction above. Monitoring readings will be taken and evaluated daily for compliance within the triggering criteria explained in this section. In addition, a weekly report will be prepared which follows the format presented as Appendix D. The trigger levels and weekly report provide a defendable basis for construction management oversight of the project.

Table 2.2 shows the trigger levels that have been set to provide an early indication of potentially unsafe working conditions. If at any time these criteria are exceeded, work will be stopped and personnel and equipment moved off the site. Once people and equipment are moved off the slide area, the Molycorp site engineer will confer with Norwest's design team to decide on the appropriate course of action.

TABLE 2.2
CONSTRUCTION MONITORING TRIGGER LEVELS

Location	Trigger Levels		
	SI Movement	*Pore Pressure*	*Visual**
1. Outside Slide Area - SI25&26 beyond slide toe, - SI-30 & 32 on stable pile	Development of a new shear plane** **OR** Displacement > 0.03" over 10ft depth	-	Scarp or tension crack development with vertical or horizontal displacement > 1ft
2. Inside Slide Area in the compression zone - SI-18, 23, 27, 28 & 31	Development of a new shear plane** **OR** Displacement rate on existing shear plane > 0.1" over 3 days	Rise in groundwater at specified piezos > 5ft	Scarp or tension crack development with vertical or horizontal displacement > 3ft
3. Inside Slide Area in the active cracking and slumping zone - SI-29, 33, 34 & 35	Development of a new shear plane** **OR** Displacement rate on existing shear plane > 0.1" over 1 day	Rise in groundwater at specified piezos > 5ft	Scarp or tension crack development with vertical or horizontal displacement > 3ft

*Surface movements in in-situ or placed material i.e. not being actively moved by construction equipment.
** A well defined lateral displacement indicated on an inclinometer displacement vs depth plot

The trigger criteria shown in Table 2.2 have been grouped into three areas within the mitigation project site. Each area has different criteria based on location within and adjacent to the slide area. It may be necessary to review and change the trigger levels based on construction experience. The threshold movement levels for the (slope inclinometers) SI's and the piezometers are prescriptive and based on a years worth of data that has been gathered for the slide area. The pore pressure trigger level is set at 5ft above current levels (i.e. water table elevations shown in Figure 1.1); the maximum increase in the levels that has been measured is about 2ft in the Goathill area (see TH-C-05 in Appendix F). Also stability analysis in the January 2004 report shows that there is only a small reduction in the safety factor (about 2%) for a 5ft increase in the water table.

The rationales for the three different SI trigger levels and possible contingency measures that will be considered (in the event of a triggering event) are summarized below:

1. Movements outside the active slide area (either at the toe or beneath the stable pile) area are an indication that the mitigation works have caused a new shearing mechanism to occur. Thus it is possible that these new mechanisms might not be controlled by the current design and there will be a need to revisit the design strategy. The SI trigger level for movements outside the slide area conforms to the precision level of the SI probe. In other words, any detectable basal shearing that occurs beyond the current slide boundary is cause for concern and the work will be stopped until it can be evaluated.

   A contingency option for a new shearing mechanism is to unload the slide area and move the material beyond the slide perimeter. The configuration of the unload and placement options for the excavated material will be dependent on the kinematics of the unexpected movements which occur.

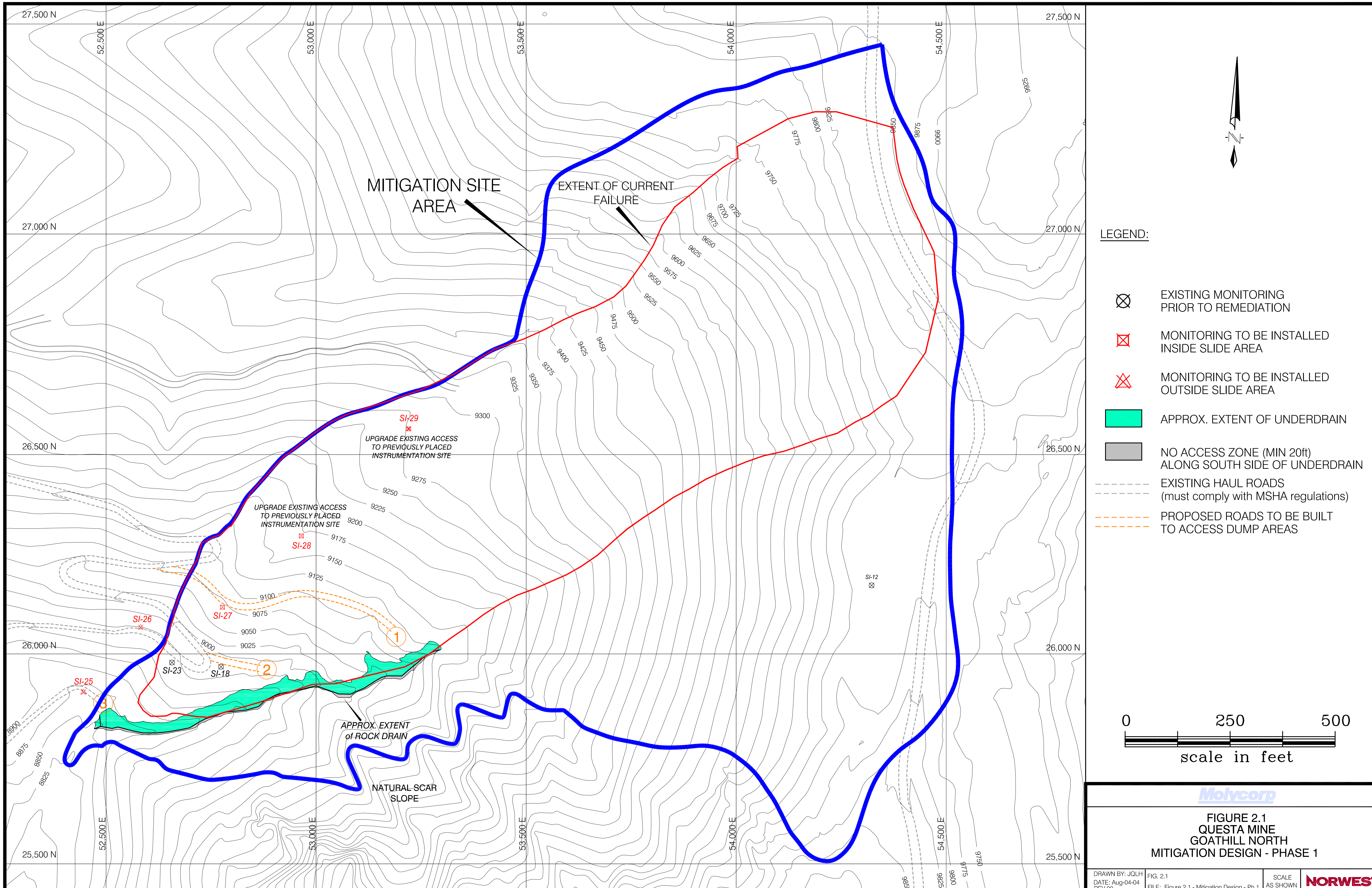
   As a mitigation alternative, the January 2004 report shows that it is possible to mitigate current slide movements by placing about 300,000 yd^3 of material at the slide toe upslope of the area referred to as the "narrows". The "narrows" area could act as a bedrock key for a toe berm. It should be noted that this option was not offered up as a contingency for unforeseen slide movements in the January 2004 report but as an alternative to the current plan. A down valley toe berm placed above the narrows does not eliminate the need for the buttress fill that is planned for stabilizing current slide movements. However, the analysis does provide an indication for the size of the berm that might be required and to confirm that placing an additional large toe buttress fill is a feasible contingency option.

2. Inside the slide area there is a zone of compression where the SI data shows markedly less movement than in the zone characterized as a cracking and slumping zone (see Figure 1.2). The compression zone occurs in the colluvium on the lower part of the slope and is contained within an area which is about 200 feet wide measured from the north boundary of the slide. It appears that the compression zone represents a transition zone between the active slide and a resistant bedrock ridge that is causing the slide to rotate and slump into the erosion gully. Measured movement rates on the SI's within the compression zone range from 0.2 to 0.7 in/month or about 0.007 to 0.023in/day (see Figure 1.2). These current movement rates would not be detectable on a daily basis and the trigger level is set to correspond to a 3 day cumulative value of 0.1in. This compares with the maximum 3 day cumulative rate of about 0.07in currently observed. The trigger value is about 50% higher than the current rates so that the work program is not constantly interrupted unnecessarily. The main issue is to flag movement rates in the compression zone which are trending towards the rates currently measured in the adjacent slumping zone discussed below. Action and contingency measures for this zone are discussed below.

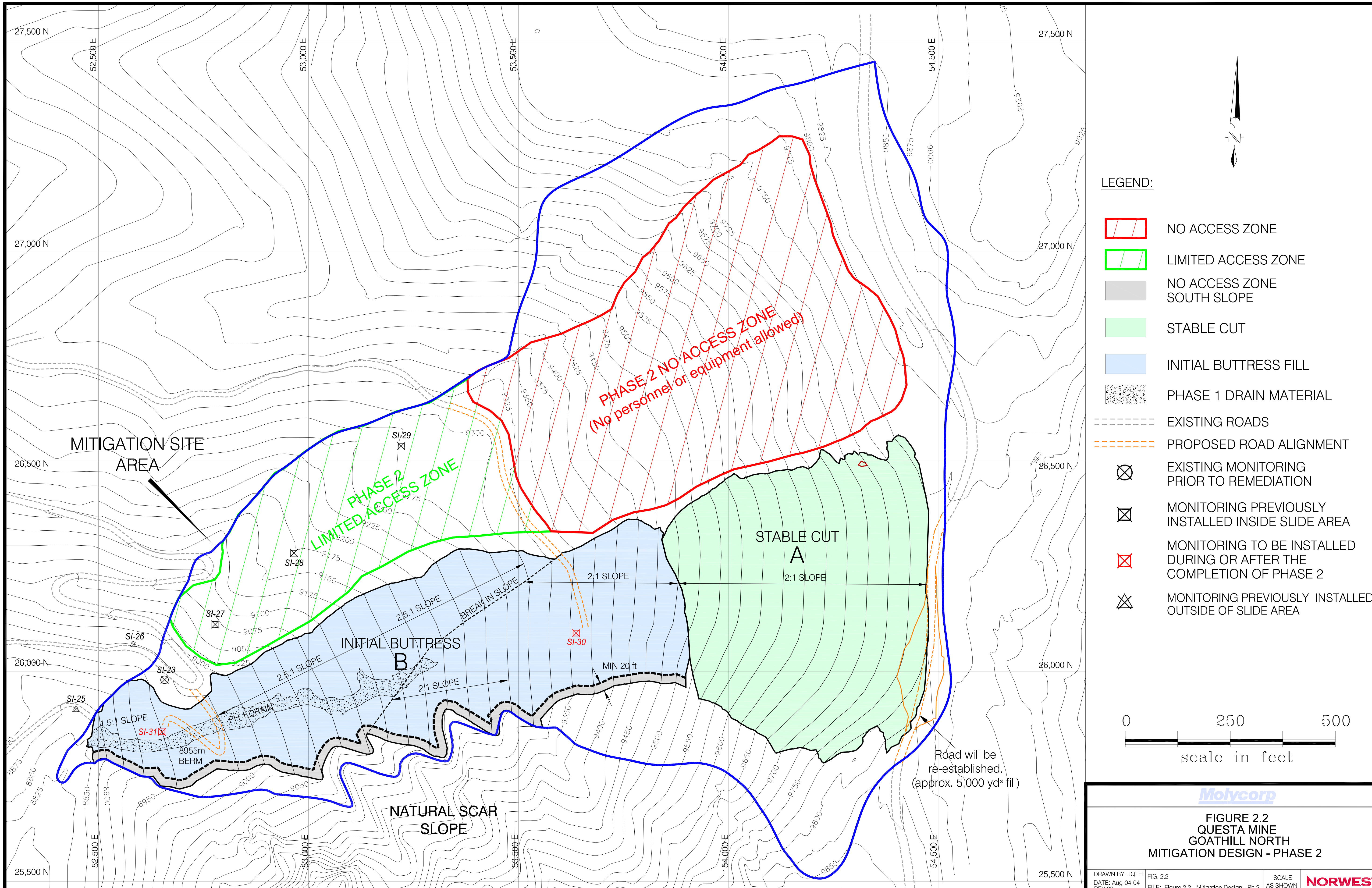
3. The most active movements on the slide occur on the rock pile and in the colluvial bench south of the compression zone. In particular there is active slumping and tension cracking taking place as material sloughs into the erosion gully. Measured movement rates on the SI's within these areas range from 1.7 to 3.2 in/month or about 0.056 to 0.107 in/day (see Figure 1.2). The trigger level set for monitoring holes within this zone is 0.1in/day which corresponds to the higher levels currently seen in these areas. It is anticipated that these higher rates of movements will decrease quickly to lower values once material begins to fill the erosion gully. Action and contingency measures for this zone are discussed below.

Trigger events in Zones 2 and 3 do not necessarily mean that the mitigation plan is not achieving its goal but rather that there may be unsafe working conditions present. In the event that the trigger criteria specified for Zones 2 and 3 are exceeded work will stop and measures to ensure safe working conditions will be considered, including:

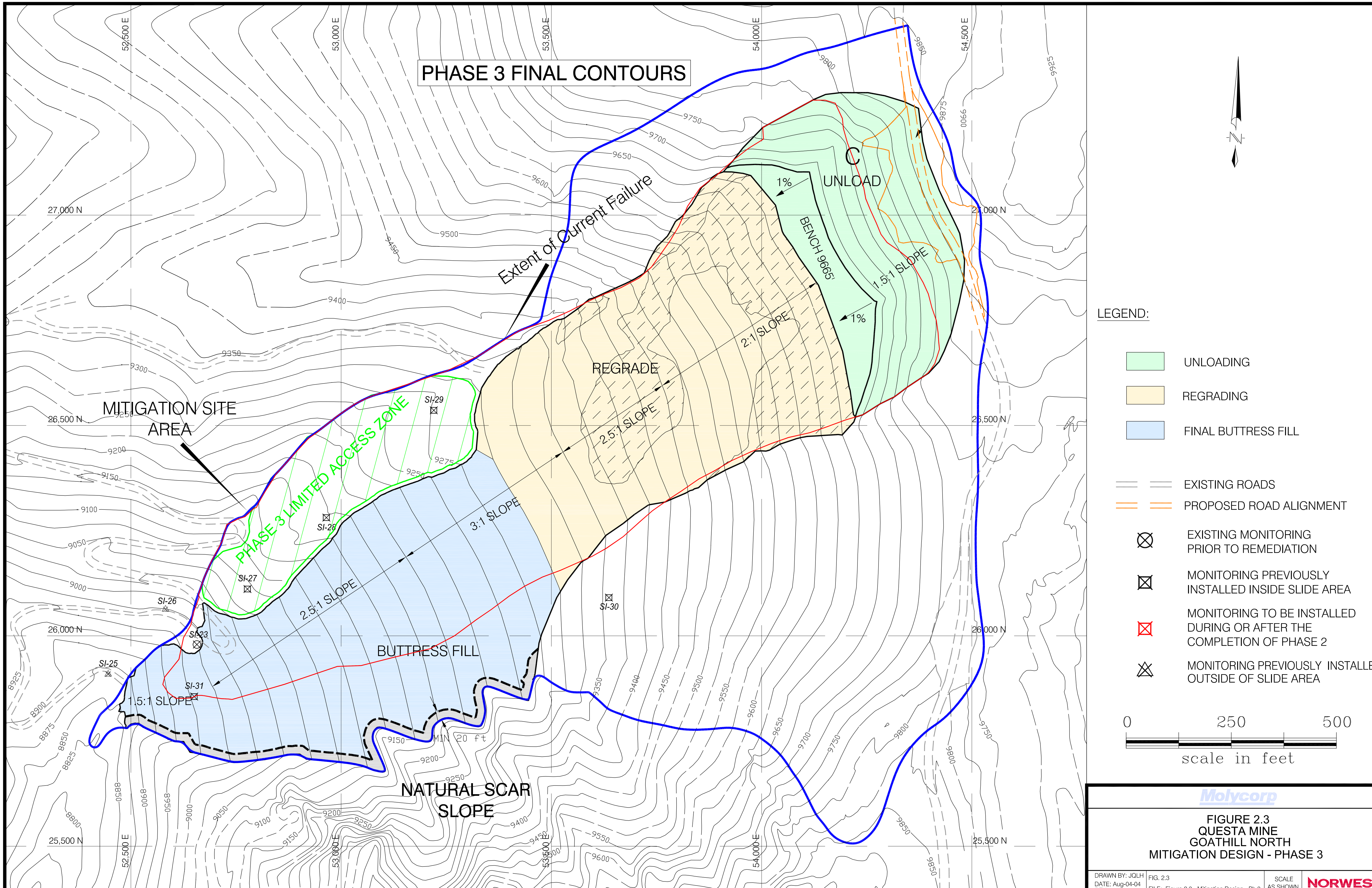
1. Dozing only half the material downslope at a time by working the fill in two parts.
2. Dozing material laterally across the slope towards the stable regraded area in order to unload the upper part of the current slide area.
3. Re-slope the interim angle-of-repose slope by pushing out to a lower slope angle (ex. this is already proposed for Phase 3c).


MITIGATION SITE AREA
EXTENT OF CURRENT FAILURE
SI-29
UPGRADE EXISTING ACCESS TO PREVIOUSLY PLACED INSTRUMENTATION SITE
UPGRADE EXISTING ACCESS TO PREVIOUSLY PLACED INSTRUMENTATION SITE
SI-28
SI-27
SI-26
SI-25
SI-23
SI-18
SI-12
1
2
3
APPROX. EXTENT of ROCK DRAIN
NATURAL SCAR SLOPE
27,500 N
27,000 N
26,500 N
26,000 N
25,500 N
52,500 E
53,000 E
53,500 E
54,000 E
54,500 E
N
LEGEND:
EXISTING MONITORING PRIOR TO REMEDIATION
MONITORING TO BE INSTALLED INSIDE SLIDE AREA
MONITORING TO BE INSTALLED OUTSIDE SLIDE AREA
APPROX. EXTENT OF UNDERDRAIN
NO ACCESS ZONE (MIN 20ft) ALONG SOUTH SIDE OF UNDERDRAIN
EXISTING HAUL ROADS (must comply with MSHA regulations)
PROPOSED ROADS TO BE BUILT TO ACCESS DUMP AREAS
0
250
500
scale in feet
Molycorp
FIGURE 2.1
QUESTA MINE
GOATHILL NORTH
MITIGATION DESIGN - PHASE 1
DRAWN BY: JQLH
DATE: Aug-04-04
REV.00
FIG. 2.1
FILE: Figure 2.1 - Mitigation Design - Ph 1
SCALE AS SHOWN
NORWEST

000155


27,500 N
27,000 N
26,500 N
26,000 N
25,500 N
52,500 E
53,000 E
53,500 E
54,000 E
54,500 E
MITIGATION SITE AREA
PHASE 2 NO ACCESS ZONE
(No personnel or equipment allowed)
PHASE 2 LIMITED ACCESS ZONE
STABLE CUT
A
2:1 SLOPE
INITIAL BUTTRESS
B
2.5:1 SLOPE
BREAK IN SLOPE
1.5:1 SLOPE
PH 1 DRAIN
8955m BERM
MIN 20 ft
NATURAL SCAR SLOPE
SI-23
SI-25
SI-26
SI-27
SI-28
SI-29
SI-30
SI-31
Road will be re-established. (approx. 5,000 yd³ fill)
N
LEGEND:
NO ACCESS ZONE
LIMITED ACCESS ZONE
NO ACCESS ZONE SOUTH SLOPE
STABLE CUT
INITIAL BUTTRESS FILL
PHASE 1 DRAIN MATERIAL
EXISTING ROADS
PROPOSED ROAD ALIGNMENT
EXISTING MONITORING PRIOR TO REMEDIATION
MONITORING PREVIOUSLY INSTALLED INSIDE SLIDE AREA
MONITORING TO BE INSTALLED DURING OR AFTER THE COMPLETION OF PHASE 2
MONITORING PREVIOUSLY INSTALLED OUTSIDE OF SLIDE AREA
0
250
500
scale in feet
Molycorp
FIGURE 2.2
QUESTA MINE
GOATHILL NORTH
MITIGATION DESIGN - PHASE 2
DRAWN BY: JQLH
DATE: Aug-04-04
REV.00
FIG. 2.2
FILE: Figure 2.2 - Mitigation Design - Ph 2
SCALE AS SHOWN
NORWEST

000156



Molycorp

FIGURE 2.3
QUESTA MINE
GOATHILL NORTH
MITIGATION DESIGN - PHASE 3

DRAWN BY: JQLH
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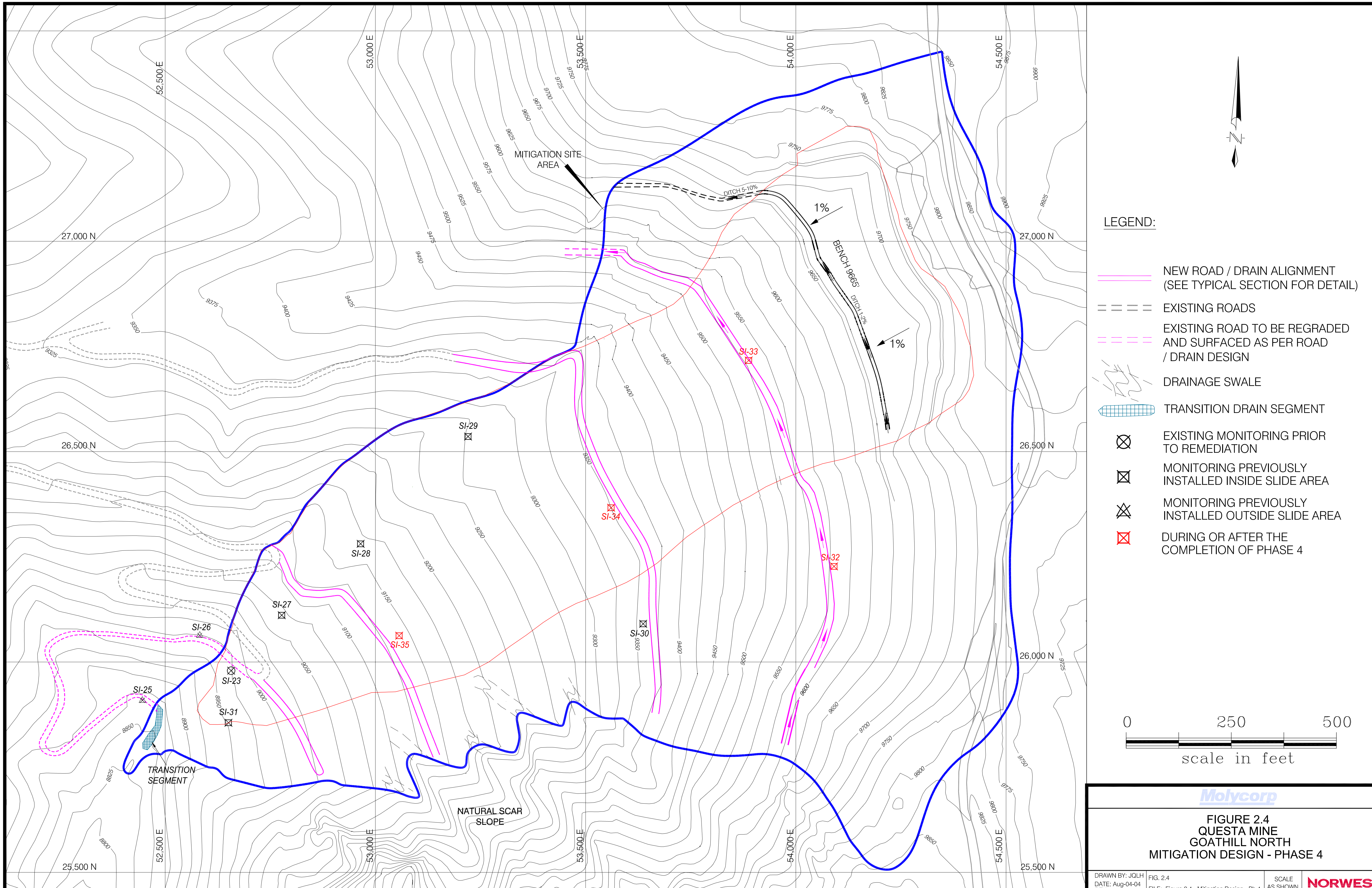
REV.00
FIG. 2.3
FILE: Figure 2.3 - Mitigation Design - Ph 3
SCALE
AS SHOWN
NORWEST

000157



Molycorp

FIGURE 2.4
QUESTA MINE
GOATHILL NORTH
MITIGATION DESIGN - PHASE 4

DRAWN BY: JQLH
DATE: Aug-04-04
REV.00
FIG. 2.4
FILE: Figure 2.4 - Mitigation Design - Ph 4
SCALE AS SHOWN
NORWEST

3 SLOPE STABILITY ANALYSIS

This section provides a discussion of the slope stability analyses that have been carried out to assess the mitigation design. Two and three dimensional back analysis have been carried out to optimize the fill geometry and to assess stability of the design during and after construction:

- Two dimensional stability analyses were carried using the computer program SLOPE/W which uses the method of slices to calculate factor of safety (FOS). Two dimensional FOS values are calculated during and at the end of construction. The two dimensional factors of FOS values are calculated for this report using Spencer's method. For each configuration two FOS values are calculated. One value (termed specified) calculates the FOS for the current shear surface as a measure of the direct improvement (i.e. the increase of FOS above current threshold of unity) that the mitigation provides to current sliding movements. The other value (termed grid search) represents the critical safety factor using a grid search method to find the lowest value. In most cases the grid search value is somewhat lower because the mitigation is specifically designed for stabilizing current movements. A complete package of stability cross sections (program output) are contained in Appendix G.

- Three dimensional stability analysis were carried out using the computer program CLARA-W which uses the method of columns to calculate factor of safety (FOS). Three dimensional FOS values are calculated at the end of construction and the main purpose of the analysis is to demonstrate that the mitigation provides acceptable stability in the "down valley" direction because of the considerable lateral resistance provided by the buttress fill. The three dimensional FOS values are calculated for this report using Bishop's method using a three dimensional specified slip surface.

Figure 3.1 shows the final fill geometry with the locations for all of the cross sections used for stability analysis. Sections A to E are at the same locations that were presented with stability analysis in the January, 2004 report. Sections F and G are new profiles to evaluate stability during the Phase 2 cut and fill. Compared with the January, 2004 report the Phase 2 design includes a much larger cut and fill (mainly to satisfy reclamation objectives) and these extra sections provide additional FOS determinations. In addition there has been more detailed stability analysis for construction stability carried out.

3.1 SOIL UNITS AND SHEAR STRENGTH PARAMETERS

Table 3.1 shows the five soil units used in the stability analyses to calculate FOS values. The rationale for selection of shear strength values is described in the January, 2004 report. For completeness a brief description is provided for each unit below:

1. Mine Rock Fill - The fill is mine rock cut from the rock pile to form the primary buttress fill support system. The friction angle is assumed to be the same as the mine rock pile materials (i.e. 36°) with somewhat higher density due to the trafficking that will occur as the dozer works the material down slope.

2. Mine Rock Pile – Mine rock material shear strength has been assigned a shear strength value of 36°. This value is supported by triaxial test values presented in the January, 2004 report. It is also notable that the angle-of-repose rock pile slopes at the Questa Mine are mostly at an angle of 36 to 38°.

3. Colluvium – The Goathill North colluvium is a sandy gravel material with a similar texture to the finer grained mine rock. As a result of this similarity it is assumed for stability analysis that the friction angle is also at 36°.

4. Weak Zone – The weak zone is the 10 to 30 foot thick zone within which current sliding occurs. This unit is assumed to be present only within the current slide footprint area. The upper contact is gradational and defined by higher clay contents. The lower contact is defined by distinctly higher blow count values and for the purpose of stability analysis the lower contact is considered a hard layer which forms a lower boundary for potential slip surfaces. The shear strength (i.e. friction angle =23.8°) of the weak zone has been determined based on back analysis (see January, 2004 report).

5. Weathered Bedrock - Beyond the slide area the weathered bedrock contact is assumed to be intact and relatively free of continuous pre-sheared surfaces. For the purpose of stability analysis the weathered bedrock zone outside of the slide area is assumed to be 10 feet thick and the lower contact is considered a hard layer which forms a lower boundary for potential slip surfaces. Slope stability analysis for the stable portion of the rock pile indicates that a shear strength value of about 28 to 30° within this unit is necessary for FOS values greater than unity (i.e. at lesser friction angles the stable pile FOS is less than 1 indicating unstable conditions). As a result, a nominal value of 30° has been assigned to the weathered bedrock beyond current slide limits. Assuming that the weak zone is pre-sheared weathered bedrock then the values of 23.8° and 30° represent residual and peak shear strengths respectively.

TABLE 3.1
SHEAR STRENGTH PARAMETERS USED FOR STABILITY ANALYSIS

	Strength Values		Density ($1 lb/ft^3$)
	Friction Angle	Cohesion	
1. Rock Fill	36°	0	131
2. Rock Pile	36°	0	127
3. Colluvium	36°	0	131
4. Weak Zone	23.8°	0	125
5. Weathered Bedrock	30°	0	130

3.2 STABILITY DURING CONSTRUCTION

The mitigation plan requires the placement of material on top of an active landslide. For this reason a number of stability analyses have been carried out at interim stages of construction during Phase 2 and Phase 3. The cross sections used to define the interim profiles follow the interim construction phases shown on the design drawings (see Appendix B drawings). These interim profiles are approximations of the actual fill geometry that will be realized as the dozers push material down slope; as opposed to the final geometry for each phase which are end of construction detailed design geometries.

It is very important to note that the interim stability of the fills does not rely solely on the FOS analysis to ensure safety and adequate fill performance. There is a comprehensive monitoring program (see Section 2.5) that will be in place for managing safe operations and to demonstrate performance. This monitoring program is part of the design criteria for the project as stated in Section 1.2.

3.2.1 Phase 2

Phase 2 construction involves pushing fill from the stable portion of the Goathill North pile into the gully where the current shear surface daylights. This is the only practical way to place fill in the gully prior to working directly on the landslide. It also improves the FOS of the stable pile by re-sloping from angle-of-repose (portions of the top of the pile are at 38-40°) to 27° (2H:1V).

Table 3.2 shows the FOS values that have been calculated for interim phases of Phase 2 construction along Sections D, F and G. The table shows that with one exception there is no significant decrease in the FOS through all of the interim Phase 2 stages. The table shows that Section G is the most critical cross section (i.e. shows the lowest FOS values) and there is a decrease in stability between Phase 2b (grid search FOS =

1.18) and Phase 2c (grid search FOS = 1.12). The Phase 2b FOS value of 1.12 is greater than the initial 1.06 value (the current value is slightly greater than 1.0 because there is no movement on the stable pile) indicating that there is still a reserve of stability compared with initial conditions during this construction stage. Nevertheless, this shows that the interim progress of the cut and fill down to the Phase 2c 9250ft elevation deserves special monitoring attention for adverse interim working slope conditions. Beyond this point it is noted that there is a significant increase in stability for the final Phase 2 configuration along Section G.

TABLE 3.2
PHASE 2 CONSTRUCTION FACTOR OF SAFETY – 2D STABILITY ANALYSIS

	Construction Phase				
	Initial	**2a**	**2b**	**2c**	**2final**
	Specified Slip				
Section D	1.36	1.44	1.71	1.64	1.61
Section F	1.59	1.61	1.60	1.59	1.61
Section G	1.07	1.18	1.29	1.27	1.28
	Grid Search Slip				
Section D	1.23	1.37	1.62	1.60	1.56
Section F	1.18	1.20	1.29	1.53	1.53
Section G	1.06	1.20	1.18	1.12	1.36

As noted by the Village of Questa's consultant (see July 16, 2004 Gannett Fleming letter in Appendix A) there is a cracking and slumping zone in the lower portion of the slide and this tension zone could be particularly sensitive to interim loading during Phase 2. To address this concern, additional analyses have been carried out along Section G assuming that there are tension cracks in the slide below the Phase 2b and 2c interim toes. The tension cracks truncate the slip surfaces so that the slip surface daylights in the vicinity of the mapped tension cracks which have formed in the colluvium. The analysis (see stability sections in Appendix G) shows that the slip surfaces formed in this fashion have higher safety factors than the corresponding slip surfaces that continue up the slope which are represented by the grid search values in Table 3.2. In other words the tension cracks by themselves do not appear to adversely affect the stability along Section G.

3.2.2 Phase 3

Phase 3 construction involves unloading the head-scarp of the slide area down to the 3665 ft elevation followed by re-grading below this elevation to form the final buttress fill. At the start of Phase 3 construction the FOS values on the current shear surface (i.e. specified slip) in the direction of current sliding (i.e. Sections A, B and C) is 1.11 and the FOS in the down valley direction (Section E) is 1.06.

Table 3.3 shows the FOS values that have been calculated for interim phases of Phase 3 construction along Sections B and E. The table shows that there is currently no significant decrease in the FOS through all of the interim Phase 3 stages. The analysis did reveal that the Phase 3c interim profile with an angle-of-repose slope could have marginal local stability. Therefore the profile for this phase was modified to 2H:1V to achieve a more stable interim condition for construction. This means that the advancing fill slope below the 9450ft elevation (i.e. after Phase 3b) will need to be pushed out to a lower slope angle to achieve a more stable interim condition.

TABLE 3.3
PHASE 3 CONSTRUCTION FACTOR OF SAFETY – 2D STABILITY ANALYSIS

	Construction Phase					
	Initial	*End of Phase 2*	*3a*	*3b*	*3c*	*3final*
	Specified Slip					
Section B	1.00	1.14	1.12	1.24	1.25	1.36
Section E	1.05	1.06	1.04	1.16	1.14	1.14
	Grid Search Slip					
Section B	1.00	1.07	1.03	1.14	1.23	1.24
Section E	1.04	1.05	1.03	1.07	1.09	1.12

3.2.3 Phase 4

Phase 4 involves superimposing a temporary road and drainage control network and there are no major cut and fills planned. The roads will be cut into the fill slopes at a 1.5H:1V slope which is flatter than the 1.33H:1V angle-of-repose. An infinite slope analysis shows that FOS value for these road cuts will be about 1.2 assuming well drained conditions.

3.3 END OF CONSTRUCTION STABILITY

3.3.1 Two Dimensional Stability

Table 3.4 shows the final two dimensional FOS values for all the cross sections. The results are similar to those from the January, 2004 report with modest increases along Sections B and C due to the additional Phase 2 fill. There are two cross sections (Section A and E) where the final stability is less than the design criteria of 1.2 and some further discussion provides the appropriate justification:

- Sections A, B and C are used to analyse for stability in the current direction of sliding (i.e. they are curved sections). The back analysed weak zone friction angle for each of these sections differs slightly and ranges from 22.8° for Section A to 25.0° for Section C (see January, 2004 report for back analysis details). It is the average value of 23.8° that is used in all the analysis. The average FOS values for Sections A, B and C are 1.32 for specified slip (i.e. 32% improvement in direction current sliding) and 1.23 for the grid search slip. On this basis it is assumed that the design criteria is satisfied for the sliding direction represented by Sections A, B and C as a combined mass.

- Section E represents down valley stability and is a straight line profile that is aligned differently than the curved direction of current sliding. However it is possible that fill placement could cause a re-distribution of the driving loads on the landslide and initiate down valley movements. The two dimensional FOS along section E improves somewhat from the initial value of 1.04 to the final value of 1.12 (grid search slip) but falls short of the 1.2 design criteria value. From the early stages of the project, the down valley stability has relied on the significant benefits that are provided by the lateral resistance of the buttress fill using three dimensional analysis to assess FOS, as discussed in the next section.

TABLE 3.4
FINAL FACTOR OF SAFETY AT COMPLETION – 2D STABILITY ANALYSIS

	Specified Slip	Grid Search Slip
Section A	1.17	1.14
Section B	1.36	1.24
Section C	1.42	1.31
Average of A, B & C	**1.32**	**1.23**
Section D	1.95	1.76
Section E	1.14	1.12
Section F	1.61	1.55
Section G	1.31	1.35

3.3.2 Three Dimensional Stability

Three dimensional stability was previously (January, 2004 report) carried out to evaluate down valley stability in order to demonstrate that the minimum design FOS criteria of 1.2 is satisfied when the lateral resistance of the buttress fill is taken into account. The analysis has been revisited with the revised fill design.

Figure 3.2 shows the CLARA-W output of the shear surface with the cut and fill design superimposed on it. The additional lateral shear resistance provided by the buttress is illustrated on Section C and local FOS values are shown on the inset figure illustrating the beneficial buttressing effects (i.e. higher local FOS values above the daylighted slip plane area).

The results of the analyses are presented in Table 3.5 to show that the FOS of the total slide in the down valley direction (0º rotation) is 1.30 which represents a 30% increase in the FOS. The FOS values increase with rotation suggesting that after the mitigation is complete the critical slip surface will be in the down-valley direction. This is an important observation for post construction monitoring.

TABLE 3.5
MITIGATION PLAN 3D STABILITY ANALYSIS SUMMARY

Rotation Angle	Limit Equilibrium Weak Layer Shear Strength	Factor of Safety
0º (parallel to upper slope movements)	20.5º	1.30
10º	21.5º	1.34
20º (parallel to lower slope movements)	22.5º	1.39

The limit equilibrium friction angle of 20.5° for 3D conditions is based on the assumption that the down valley stability is at FOS=1. This is a conservative assumption because of the slide rotation. Three dimensional analysis carried out for a weak zone friction angle of 23.8° (i.e. two dimensional back analysed value) result in an FOS value of 1.45.

3.4 STABILITY FOR ALTERNATE WATER TABLE ASSUMPTION

Recent pore pressure data from the drill hole TH-C-05 located on the ridge between the Goathill North and Capulin rock piles has indicated that there is a perched water table located near the base of the rock pile. The Village of Questa's consultant has requested that additional analysis be carried out (see May 12, 2004 Gannett Fleming letter in Appendix A) to assess final design stability assuming a perched water table beneath the upper portion of the slide area.

Figure 3.3 shows the contours for a perched water table that is 10 feet above the base of the weak zone on the steep portion of the slope above the 9330 ft elevation. Below this elevation the contours are the same as the base case assumption in order to honour the data points. The revised contours show a very steep piezometric surface parallel to pre-mine contours.

Stability analyses have been carried out (see cross sections in Appendix G) using the alternate water table assumption and the results are summarized as follows:

- Back analyses have been carried out along Sections A, B and C to analyse the weak zone shear strength based on the alternate water table interpretation, assuming that this is now the pore pressure condition causing slide movements. The back analysed average shear strength for three sections is 24.7° which is about 1° higher than the base case value of 23.8°.

- Table 3.6 shows a comparison of the two dimensional stability for the base case and alternate water table. The differences are small and confirm that the stability analyses produce similar results with either water table assumption.

TABLE 3.6
FINAL FOS AT COMPLETION – ALTERNATE WATER TABLE

	Specified		Grid Search	
	Original Phi=23.8°	Raise WT Phi=24.7°	Original Phi=23.8°	Raise WT Phi=24.7°
Sct A	1.17	1.18	1.14	1.09
Sct B	1.36	1.40	1.24	1.25
Sct C	1.42	1.46	1.31	1.29
Average ABC	**1.32**	**1.35**	**1.23**	**1.21**
Sct E	1.14	1.17	1.12	1.14

- In addition to the two dimensional analysis, the revised water table configuration was used to revisit the three dimensional stability. On this basis the back analysed friction angle is calculated as 21.0° (compared with 20.5° for the base case) and the mitigated FOS is 1.32 (versus 1.30 for the base case). These differences are also small and confirm that the three dimensional stability analyses produce similar results with either water table assumption.

This analysis shows that the water table assumption does not significantly impact the stability analyses. This is partly because the pore pressures are relatively small compared to the total stress of the slide on the shear plane (i.e. small pore pressure ratios). Additionally, changes in the water table assumption result in offsetting changes in the back analysed shear strength.

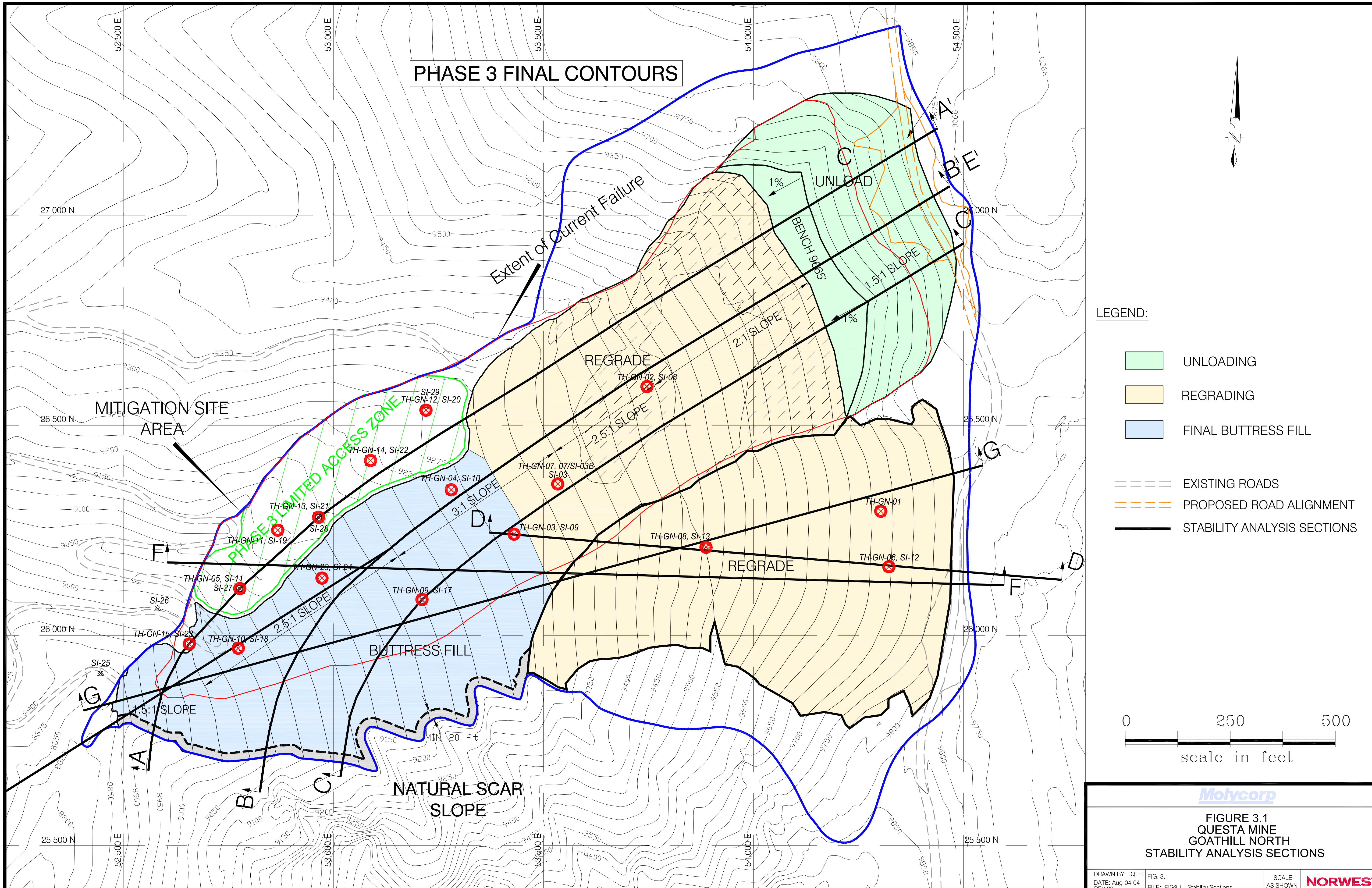
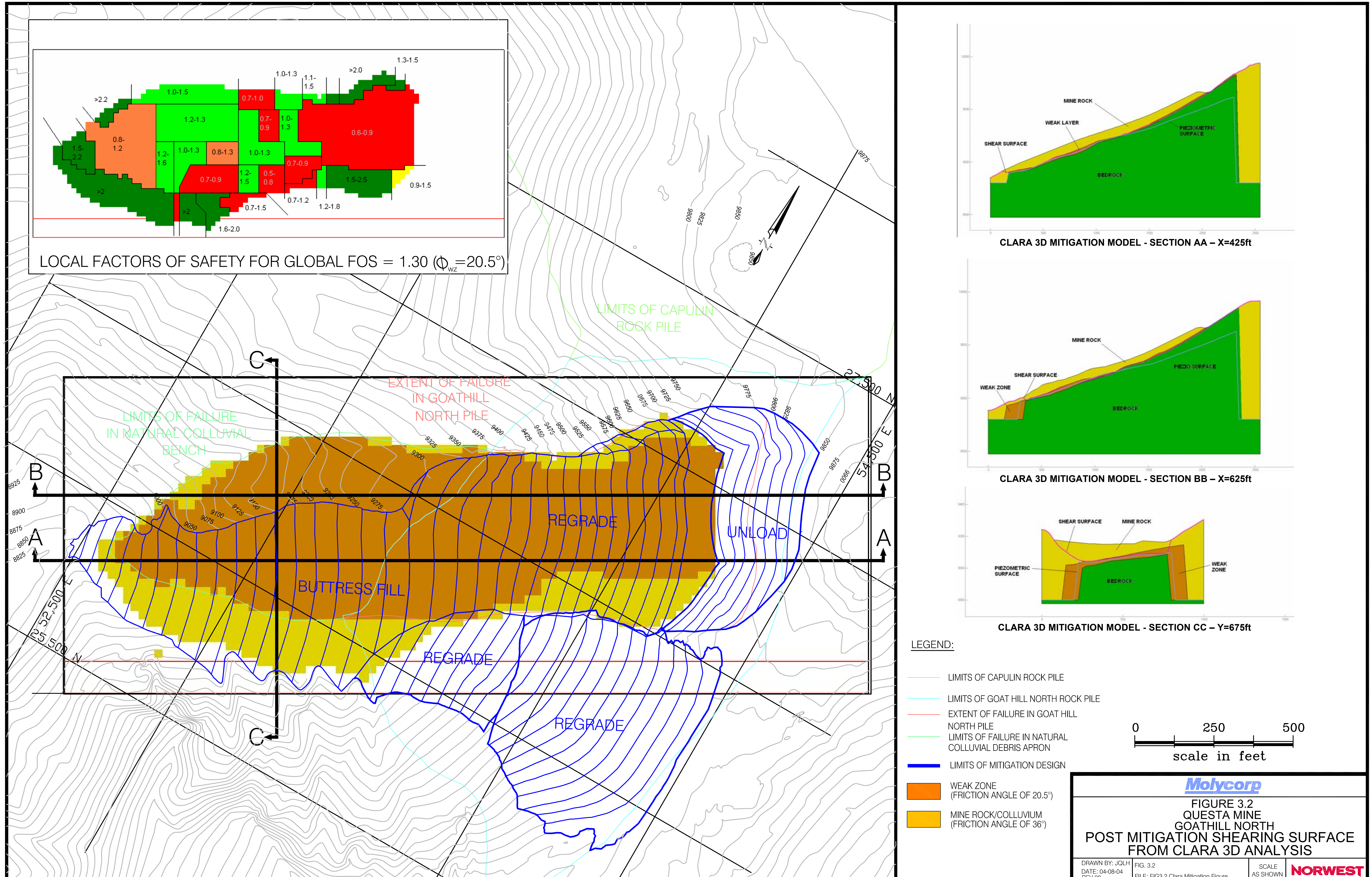
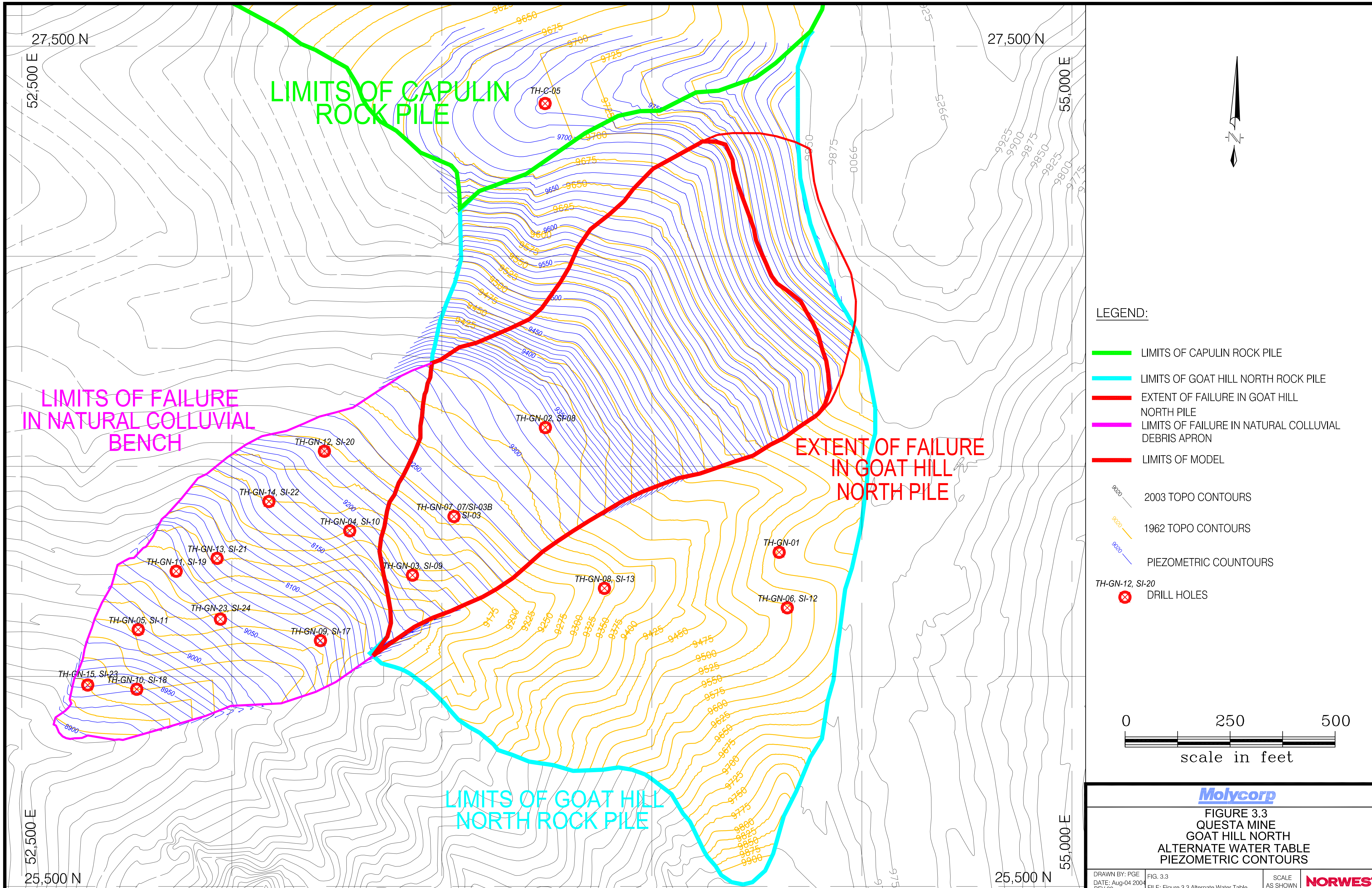


FIGURE 3.1
QUESTA MINE
GOATHILL NORTH
STABILITY ANALYSIS SECTIONS



LOCAL FACTORS OF SAFETY FOR GLOBAL FOS = 1.30 (ϕ_{wz}=20.5°)
>2.2
1.0-1.5
1.0-1.3
1.1-1.5
>2.0
1.3-1.5
0.7-1.0
0.8-1.2
1.2-1.3
0.7-0.9
1.0-1.3
0.6-0.9
1.5-2.2
1.2-1.6
1.0-1.3
0.8-1.3
1.0-1.3
0.7-0.9
0.7-0.9
1.2-1.5
0.5-0.8
1.5-2.5
0.9-1.5
>2
>2
0.7-1.5
0.7-1.2
1.2-1.8
1.6-2.0
LIMITS OF CAPULIN ROCK PILE
EXTENT OF FAILURE IN GOATHILL NORTH PILE
LIMITS OF FAILURE IN NATURAL COLLUVIAL BENCH
REGRADE
UNLOAD
BUTTRESS FILL
REGRADE
REGRADE
A
A
B
B
C
C
27,500 N
54,500 E
52,500 E
25,500 N
N
CLARA 3D MITIGATION MODEL - SECTION AA – X=425ft
MINE ROCK
WEAK LAYER
SHEAR SURFACE
PIEZOMETRIC SURFACE
BEDROCK
CLARA 3D MITIGATION MODEL - SECTION BB – X=625ft
MINE ROCK
SHEAR SURFACE
WEAK ZONE
PIEZO SURFACE
BEDROCK
CLARA 3D MITIGATION MODEL - SECTION CC – Y=675ft
SHEAR SURFACE
MINE ROCK
PIEZOMETRIC SURFACE
BEDROCK
WEAK ZONE
LEGEND:
LIMITS OF CAPULIN ROCK PILE
LIMITS OF GOAT HILL NORTH ROCK PILE
EXTENT OF FAILURE IN GOAT HILL NORTH PILE
LIMITS OF FAILURE IN NATURAL COLLUVIAL DEBRIS APRON
LIMITS OF MITIGATION DESIGN
WEAK ZONE (FRICTION ANGLE OF 20.5°)
MINE ROCK/COLLUVIUM (FRICTION ANGLE OF 36°)
0
250
500
scale in feet
Molycorp
FIGURE 3.2
QUESTA MINE
GOATHILL NORTH
POST MITIGATION SHEARING SURFACE
FROM CLARA 3D ANALYSIS
DRAWN BY: JQLH
DATE: 04-08-04
REV.00
FIG. 3.2
FILE: FIG3.2 Clara Mitigation Figure
SCALE AS SHOWN
NORWEST

000169


27,500 N
52,500 E
55,000 E
25,500 N
LIMITS OF CAPULIN ROCK PILE
LIMITS OF FAILURE IN NATURAL COLLUVIAL BENCH
EXTENT OF FAILURE IN GOAT HILL NORTH PILE
LIMITS OF GOAT HILL NORTH ROCK PILE
TH-C-05
TH-GN-02, SI-08
TH-GN-12, SI-20
TH-GN-14, SI-22
TH-GN-07, 07/SI-03B SI-03
TH-GN-04, SI-10
TH-GN-13, SI-21
TH-GN-11, SI-19
TH-GN-03, SI-09
TH-GN-08, SI-13
TH-GN-01
TH-GN-06, SI-12
TH-GN-23, SI-24
TH-GN-05, SI-11
TH-GN-09, SI-17
TH-GN-15, SI-23
TH-GN-10, SI-18
LEGEND:
LIMITS OF CAPULIN ROCK PILE
LIMITS OF GOAT HILL NORTH ROCK PILE
EXTENT OF FAILURE IN GOAT HILL NORTH PILE
LIMITS OF FAILURE IN NATURAL COLLUVIAL DEBRIS APRON
LIMITS OF MODEL
2003 TOPO CONTOURS
1962 TOPO CONTOURS
PIEZOMETRIC COUNTOURS
TH-GN-12, SI-20
DRILL HOLES
0
250
500
scale in feet
Molycorp
FIGURE 3.3
QUESTA MINE
GOAT HILL NORTH
ALTERNATE WATER TABLE
PIEZOMETRIC CONTOURS
DRAWN BY: PGE
DATE: Aug-04 2004
REV.00
FIG. 3.3
FILE: Figure 3.3 Alternate Water Table
SCALE AS SHOWN
NORWEST

000170

4 PROJECT SUCCESS CRITERIA

Once the construction is completed, the mitigated site will be monitored for a period of about 12 months prior to constructing the channels and erosion protection works that will be necessary for managing surface water over the longer term. This will provide a period of time for demonstrating success. It is unlikely that shearing movements will have been reduced to negligibly low values within a year after completion of the major earthworks, although it is anticipated that there will be a trend of decreasing movement. There is a balance in the timing between allowing for sliding movements to decelerate versus completing the final (long term) mitigation works (Part 2) to defend against the gully erosion that currently daylights the shear surface.

The granular colluvium and rock pile materials that currently overly the shear surface and the rock pile fill that will be used to form the buttress are not rigid materials. As a result there will be movements within and above the shear surface for a fairly long period of time (several years) after construction is complete. Some of these movements will be related to localized strains on the basal shear surface and others will be related to fill settlement (wetting and/or creep induced). For control of sliding movements, only those displacements related to basal shearing (i.e. those with a large lateral component rather than a large vertical component) are considered to be important.

The slide has been largely defined based on SI casing measurements of basal shearing. Thus this instrumentation should be mainly relied on as a source of information for evaluating whether the mitigation has performed an adequate function for control of sliding movements. Norwest has defined three independent criterion for assessing project success:

- The SI instrumentation placed beyond the current slide boundary should not show any signs of discrete basal shearing near the colluvium or mine rock contact with weathered bedrock materials. This criterion is also a construction monitoring trigger because these kinds of movements signal that unforeseen movements are occurring.

- The SI shearing rates within the slide mass should generally show a steadily decreasing trend with time in a manner which is consistent with the sequence and distribution of the fill. For example there should be a fairly large reduction in the movement rates adjacent to the buttress shortly after the fill is placed followed by a smaller decrease afterwards.

- In general, movement rates after one year should have decreased by about one order of magnitude. This means that movement rates within the compression zone should be reduced to within a range of 0.02 to 0.07 in/month and that movements within the zone of active sliding (i.e. within the slide area outside the compression zone) should be reduced to within a range of 0.2 to 0.3 in/month. These rates are approximate and isolated outliers can be excluded if this can be justified.

It is noted that these criteria are not strictly prescriptive and will require the application of engineering judgment. It is expected that the information used to assess project success will constitute a part of the Part 1 as-built drawings that will be required prior to final design of the water management works (i.e. Part 2). In this way the monitoring used to define project success will be provided along with the survey data used to reconcile the design.

Appendix A

Correspondence

Appendix A – Correspondence

Energy, Minerals and Natural Resources Department, Environment Department, Letter**,** Feb 20, 2004: Review of Molycorp's final report on GHN Mitigation dated January 28, 2004, TA001RE, DP-1055

Energy, Minerals and Natural Resources Department, Environment Department, Letter**,** Jun 16, 2004: Joint Agency approval letter regarding Goathill North rock pile mitigation project final design submittal, dated may27, 2004 – TA001RE and DP-1055

Gannett Fleming, Letter**,** Feb 12, 2004: Goathill North rock pile slide investigations and mitigation design – Summary of recommendations to Village of Questa.

Gannett Fleming, Technical Memo**,** Feb 12, 2004: Goathill North rock pile slide investigations and mitigation design.

Gannett Fleming, Letter**,** May 12, 2004: Goathill North rock pile – Emergency Monitoring trigger level exceedence in piezometer TH-C-05.

Gannett Fleming, Letter**,** Jul 16, 2004: Recommendations for analysis to include in the final design report for the Goathill North rock pile mitigation.

Stability Review Board, Letter, Feb 7, 2004

Stability Review Board, Letter, Jul 2, 2004

State of New Mexico
ENERGY, MINERALS and NATURAL RESOURCES DEPARTMENT
ENVIRONMENT DEPARTMENT

Bill Richardson
GOVERNOR

Joanna Prukop
SECRETARY

Ron Curry
SECRETARY

CERTIFIED MAIL –RETURN RECEIPT REQUESTED

February 20, 2004

Mr. Bill Sharrer
Vice President, Environmental and Public Policy
Molycorp, Inc
67750 Bailey Road, P.O. Box 124
Mountain Pass, CA 92366

SUBJECT: Review of Molycorp's Final Report on GHN Mitigation dated January 28, 2004, TA001RE and DP-1055.

Dear Mr. Sharrer:

The Mining and Minerals Division of the Energy Minerals and Natural Resources Department (MMD), and the New Mexico Environment Department (NMED) have reviewed the following Molycorp submittal:

Goathill North Slide Investigation, Evaluation and Mitigation Report (Report), Dated January 28th, 2004.

Norwest Corporation prepared this document for Molycorp in response to the requirement by the State of New Mexico that Molycorp develop a plan for the mitigation of the immediate hazards associated with the Goathill North (GHN) Rockpile.
The main Report itself is stated to be a feasibility level study and MMD and NMED (Agencies) have reviewed the Report in this context. Further work is indicated to be necessary for finalizing the design and bringing it to the detailed construction level. The Agencies have reviewed this Report along with the earlier analyses of the Goathill North Slide prepared by Molycorp and Norwest, and also reviewed the comments of the Stability Review Board (SRB) and the other members of the Stability Review Committee (SRC). The Report was reviewed by the agencies with mitigation of the immediate

hazard, identified in the SRB June 4, 2003 letter, as the only purpose of the mitigation. The report does not address final closeout or reclamation of the Goathill North rockpile.

It is the Agencies opinion that the proposed activities described in the Report, when implemented as presented, will mitigate the immediate public safety concerns expressed in the June 4, 2003 SRB letter; therefore, **the Report is hereby approved with the following conditions:**

1. Prior to construction, all construction final designs and drawings shall be submitted to the Agencies for approval. Included in the construction final designs shall be a schedule for the proposed mitigation activities. All final designs and drawings shall be prepared under the direction of and signed and stamped by a Professional Engineer licensed in New Mexico, with experience in slope stability issues.

2. Included in the construction plans detailing the surface water management controls, Molycorp shall submit to the Agencies a plan to either line the plunge pools or submit justification why the plunge pools should not be lined. Prior to construction activities this plan shall be approved by NMED.

3. Included in the construction plans detailing the surface water management controls, Molycorp shall submit to the Agencies a plan to address how the surface water from the waste rock pile and buttress will be monitored, collected, characterized and treated if necessary. Prior to construction activities this plan shall be approved by NMED.

4. Included in the construction plans, Molycorp shall submit to the Agencies the proposed source of the material that will be used and the QA/QC that will ensure that the rock used for the under drain will contain neutral pH material. Molycorp shall receive approval from NMED of the source material before placement in the under drain.

5. Prior to the construction of the under drain, Molycorp shall submit to the Agencies a plan to address how the ground water from the under drain will be monitored, collected, characterized and treated if necessary. NMED approval of the plan is required before commencing construction activities.

6. The Report discusses supplemental toe berm fill options. One option, to add 40,000 yards to the toe berm, increases the factor of safety (FOS) from 1.11 (Table 6.1) to 1.14 or by 3%, which the Report describes as a "nominal" increase. However this is approximately the same amount of increase that the proposed base case fill provides. The FOS is shown on Table 6.2 from the December Draft Report as 1.08 and has been increased, by the current proposed toe berm, in the proposed base case to 1.11 listed on Table 6.1 of the final report. Comments from the SRB indicate the preference for an increased FOS, which would be accomplished by the additional 40,000 yards of material in the toe berm.

Therefore, as part of the construction plans and drawings, Molycorp shall submit to the Agencies, for approval, plans for the placement of the additional 40,000 yards of material in the toe berm or Molycorp will provide adequate justification why this material is not necessary including a cost estimate for placing the material.

7. Prior to construction Molycorp shall submit to the Agencies, for approval, the identification of the engineer of record, licensed in New Mexico, Who will supervise the remedial grading. The engineer of record shall have experience in the design and construction of slope instability measures. Submittal of engineer's qualifications is required.

8. Prior to construction activities on the GHN Rockpile, Molycorp shall submit to the Agencies, for approval, contingency plans for the construction phases. Contingency plans shall at a minimum include:
   A. The relationship between the ongoing rockpile monitoring activities and the contingency plans. Specifically, identification of the trigger levels that will activate the contingency plans.
   B. Contain action plans for protecting worker safety.
   C. Contain plans for alternative dirt placement for decreasing movement in the event trigger levels are reached.

9. Prior to construction activities on the GHN Rockpile, Molycorp shall submit to the Agencies, for approval, a construction specific rockpile monitoring plan. At a minimum the monitoring plan shall include the following:
   A. A description of the program including rockpile monitoring activities during construction and post- mitigation.
   B. Trigger levels to be used for the construction contingency plans.
   C. Proposed instrumentation levels for post mitigation rockpile monitoring that will be used to define the success of the mitigation measures and anticipated timeframes for obtaining those levels.
   D. A description of how the construction rockpile monitoring plan will be incorporated into the weekly report.

10. Prior to construction activities on the GHN Rockpile Molycorp shall submit to the Agencies verification that construction plans have been submitted to the Federal Mine Safety & Health Administration (MSHA).

11. Prior to construction activities on GHN Rockpile, Molycorp shall submit to the Agencies, for approval, plans for the placement of the rock under drain material. Also, before the Phase 2 buttress material is placed over the under drain, Molycorp will notify the agencies to confirm that the placement and physical properties of the material is as specified in the construction plan.

12. Molycorp shall continue the Weekly GHN Report and shall add, at a minimum, a progress report on construction activities. In the weekly report, Molycorp shall

include monitoring data and shall document any changes from the approved construction plans and drawings.

13. Upon completion of the mitigation construction, Molycorp shall submit to the Agencies the following:
    A. A completion report signed by the engineer of record, which shall include descriptions of problems encountered and their solutions.
    B. A summary of materials test data and construction photographs.
    C. As-built drawings signed by the engineer of record.

In addition, all plans and documents described above in Conditions 1 through 13 shall be submitted by Molycorp to NMED pursuant to Conditions 26 and 29 of DP-1055.

Modification to this Report may be required if ground water contamination occurs as a result of this plan, or if additional information becomes available indicating that the Report is inadequate.

Molycorp's cooperation in these efforts is appreciated. If you have any questions please call Terry Foreback with the Mining and Minerals Division at 505-476-3432, or Mike Reed with the NMED, Mining Environmental Compliance Section at 505-827-2340.

Sincerely,

Bill Brancard, Director
Mining and Minerals Division
NMEMNRD

Charles Lundstrom, Director
Water & Waste Water Management Division
NMED

cc: Holland Shepherd, Program Manager, MARP
Mary Ann Menetrey, Program Manager, MECS
Marcy Leavitt, Bureau Chief, SWQB
Terence Foreback, Permit Lead, MARP
Karen Garcia, Bureau Chief, Mine Regulatory Bureau
Mike Reed, Permit Lead, MECS
Brian Shields, Amigos Bravos
Mark Purcell, USEPA, Region 6
Charlie Gonzales, Mayor, Village of Questa
Al Pasteris, SWQB
Ray Cherniske, Molycorp

Bill Richardson
GOVERNOR

State of New Mexico
ENERGY, MINERALS and NATURAL RESOURCES DEPARTMENT
Joanna Prukop
SECRETARY

State of New Mexico
ENVIRONMENT DEPARTMENT
Ron Curry
SECRETARY

CERTIFIED MAIL –RETURN RECEIPT REQUESTED

June 16, 2004

Mr. William Sharrer
Vice President, Environmental and Public Policy
Molycorp, Inc
P.O. Box 469
Questa, NM 87556

RE: Joint Agency Approval Letter Regarding <u>Goathill North Rock Pile Mitigation Project Final Design Submittal</u>, Dated May 27, 2004 - TA001RE and DP-1055.

Dear Mr. Sharrer:

Both the Mining and Minerals Division (MMD) of the Energy Minerals and Natural Resources Department, under the authority of TA001RE, and the Mining Environmental Compliance Section of the New Mexico Environment Department (NMED), under the authority of DP-1055, have reviewed the *Goathill North Rock Pile Mitigation Project Final Design Submittal*, dated May 27, responding to Conditions of the joint conditional approval of the *Goathill North Slide Investigation, Evaluation and Mitigation Report* dated January 28, 2004.

Molycorp has adequately addressed the conditions described in the conditional approval letter of January 28, 2004 letter. Therefore, Molycorp can proceed with the mitigation of the Goathill North waste rock pile as described in the May 27, 2004 submittal. The agencies have the following comments and request for additional information:

1. Molycorp shall revise all relevant drawings and resubmit the drawings to the Agencies on or before July 31, 2004 to reflect the following:
   a. On Drawing 5 the flat area at the top of the hill is labeled incorrectly as having a 1% grade into the hill. This drawing must be corrected to reflect the proper direction of drainage.
   b. All cross sections shall show the location of the landslide shear surface.
   c. Contour elevations shall be clearly labeled on all contour maps.

d. Molycorp shall identify all instrument stations containing piezometers as well as the inclinometers.

2. Prior to initiation of Phase 3 grading activities, Molycorp shall submit to the Agencies for approval, an analysis of grading options for the upper bench area on Drawing 5. This analysis shall determine the optimum grade that balances stability of the slope with minimal infiltration of runoff. If water is to be directed over the slope, as currently shown, the analysis shall include an evaluation of the potential for slope erosion, or evaluation of other drainage alternatives. The analysis will also contain a justification for the 1.5:1 slope above the bench.

3. Molycorp shall submit on or before July 31, 2004 to the Agencies, for approval, a plan addressing piezometer maintenance procedures and how Molycorp will ensure the accuracy of the instruments.

4. Molycorp shall submit on or before July 31, 2004 to the Agencies, for approval, justification of trigger levels listed in the table labeled Construction Monitoring – Phases 1,2,3, and 4.

5. Before construction activities begin, Molycorp will submit to the Agencies, for approval, a plan for oversight that will include at a minimum, a "project kickoff meeting", formal monthly progress meetings and site inspections.

6. Section 1.7.3 discusses alternative mitigative measures in the event that trigger levels are exceeded during Phase 3 and 4. Mitigation measure number 3 states: "Increase the volume of material placed at the toe of the slide." Prior to the commencement of Phase 2 construction activities, Molycorp shall submit to the Agencies, for approval, a detailed plan that analyzes, at a minimum, volume, placement method, foundation conditions and foundation preparation for the additional volume of material to be placed at the toe, relative to the contingency plan.

7. Molycorp has proposed using visual observation on a daily basis to segregate potentially acid generating waste rock for the Goathill North drain. Prior to commencement of drain rock production, Molycorp shall meet with the Agencies and determine the visual process and the qualitative criteria that will be used to determine the drain rock material.

8. Molycorp shall take a final full set of instrument readings immediately before GHN mitigation construction commences to maximize baseline data prior to mitigation construction.

9. On or before July 31, 2004, Molycorp shall propose to the Agencies, for approval, a quantitative assessment to address movement rates related to the slide area from instrumentation located on the slide. This assessment will aid in defining project

success. In addition, Molycorp shall provide information addressing the expected timeframe for achieving the predicted movement, or deceleration of movement.

10. Before construction of the roads and ditches as shown on Drawing 6 Phase 3, Molycorp shall submit to the Agencies, for approval, an analysis of the effect that these structures have on the overall stability of the Goathill North waste rock pile.

Modification to the Goathill North Rock Pile Mitigation Project Final Design may be required if additional information becomes available indicating that the submittal is inadequate.

If you have any questions please call Terry Foreback with MMD at 505-476-3432, or Mike Reed with the NMED at 505-827-2340.

Sincerely,

Terry Foreback, Permit Lead
Mining and Minerals Division
NMEMNRD

Michael F. Reed, Permit Lead
GWQB-MECS
NMED

cc: Bill Brancard, Director, MMD
Karen Garcia, Bureau Chief, Mine Regulatory Bureau
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Mary Ann Menetrey, Program Manager, MECS
Al Pasteris, SWQB
Brian Shields, Amigos Bravos
Mark Purcell, USEPA, Region 6
Charlie Gonzales, Mayor, Village of Questa
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February 12, 2004

Mr. Marcus Rael, Esq.
Robles and Rael, P.C.
500 Oak Street NE, Suite 201
Albuquerque, NM 87106

Subject: Goathill North Rock Pile Slide Investigations and Mitigation Design – Summary of Recommendations to Village of Questa

Dear Mr. Rael:

This letter summarizes Gannett Fleming's recommendation to the Village of Questa, based on our independent evaluation of geotechnical investigations, technical analyses, and proposed mitigation design concept for the Goathill North waste rock landslide at Molycorp's Questa mine. Per your request, Dr. Debora Miller of Gannett Fleming attended a two-day meeting held at the mine site on December 4-5, 2003, and a one-day meeting in Denver on February 3, 2004, to observe presentations by Molycorp, and to participate in technical discussions with the Stability Review Board (SRB) and the Technical Review Committee (TRC) regarding the slide mechanisms and proposed mitigation plan. Dr. Miller has thoroughly reviewed both the draft (December, 2003) and final (January, 2004) reports provided by Molycorp entitled "Goathill North Slide Investigation, Evaluation and Mitigation Report", and the accompanying appendices and supporting data. A technical memorandum is attached with this letter summarizing the technical findings, and our interpretation of the information that has been provided. The memorandum provides more detailed discussions pertaining to our assessment of Molycorp's mitigation plan and the SRB's comments regarding the proposed action.

RECOMMENDATION

Gannett Fleming recommends that the Village of Questa endorse the preliminary slide mitigation plan as proposed by Molycorp, subject to the following conditions:

1. Recognizing that final design, construction plans and specifications still need to be prepared, and that some details of the proposed design concept may evolve or change, the basic elements of the proposed mitigation plan will be retained. These elements are listed on Table 6.2 of the January, 2004 report and include the following: under-drain system (Phase 1); stable pile cut and initial toe buttress fill (Phase 2); unloading from the head of the slide mass, regrading on the mid –lower slide, and final earth buttress fill (Phase 3); and surface water controls (Phase 4).

2. Ground movements will be appropriately monitored during and after construction of the mitigation measures. The plan for monitoring as outlined in the January 2004 report Section 6.5 appears reasonable.
3. A contingency plan will be developed and implemented if slide movements are not adequately controlled to within specified design criteria. The Village of Questa will have an opportunity to review and comment on the proposed design criteria.

The design criteria have not yet been established. It is our understanding from the discussions at the February 3 meeting that Molycorp will propose design criteria at the time the final design and construction plans are prepared. The criteria will include a time frame (e.g., a number of historically representative wet and dry seasonal cycles) over which ground movements as measured by the monitoring system will be controlled to within specified tolerable limits.

The SRB has presented its comments on the proposed slide mitigation plan in letters dated December 12, 2003, and February 7, 2004. The SRB states “We consider the proposed design to be feasible in principle for Goathill North slide mitigation.” They qualify this statement with certain concerns regarding the technical analyses. In our Technical Memorandum attached with this letter, we try to address these concerns and provide you with our perspective on the technical details.

Please do not hesitate to call with any questions or concerns. It has been a pleasure to work with you on this challenging and interesting assignment.

Sincerely,
GANNETT FLEMING, INC.

Debora J. Miller, P.E., Ph.D.
djmiller@gfnet.com

cc: Jim Langer, Gannett Fleming

TECHNICAL MEMORANDUM

TO: Marcus Rael, Esq., Robles and Rael, P.C.

FROM: Debora J. Miller, Ph.D., P.E., Gannett Fleming

DATE: February 12, 2004

SUBJECT: Goathill North Rock Pile Slide Investigations and Mitigation Design

1.0 Background

Molycorp began work on evaluating the landslide at the Goathill North rock pile in June, 2003, after concerns were raised by the state-appointed SRB (comprising Mr. Steve Vick, Dr. Nigel Skermer, and Dr. James Mitchell) regarding potential down-valley risks associated with the unstable rock pile. Molycorp retained the engineering services of Norwest Corp., a Canadian geotechnical consulting firm, to develop a geotechnical investigation program for evaluating the landslide and a mitigation plan to stabilize the slide movements.

Norwest prepared an initial report dated July 15, 2003, that refuted the SRB's assertion that there is a risk of catastrophic flow-type failure of the rock pile by static liquefaction. Norwest concluded that there is a low probability for a liquefaction flow-slide failure mode at the rock pile, but acknowledged that a slow-moving landslide was present within the rock pile. Norwest proposed a very preliminary design concept to stabilize the slow-moving landslide, along with a preliminary plan for geotechnical investigations to support design of the slide stabilization measures. Gannett Fleming provided review comments on the July 15, 2003 Norwest report to the Village in a letter dated July 28, 2003. In that review, we stated that slide stabilization measures could be similar whether the failure mode is a flow slide or a slow-moving landslide, and that it was prudent to implement slide stabilization measures in either case. However, at that time we had concerns about the preliminary mitigation concept proposed by Norwest, which involved only regrading of the toe of the slide area without offloading from the crest of the slide or importing material to construct an earth buttress or shear key at the toe. (Note: the currently proposed mitigation plan provides both offloading and earth buttress construction.)

Molycorp carried out a comprehensive geotechnical investigation program from July 24 through October 15, 2003. The investigation included geologic mapping, drilling 23 test borings on the Goathill North rock pile and 5 borings on the Capulin rock pile, geophysical surveys, laboratory testing, and installation of field instruments to monitor slide movements and groundwater levels. Meetings were held on August 27-28 (at the mine site) and October 23-24, 2003 (in Denver), to present intermediate findings and discuss progress on the investigations with the SRB, state agencies and stakeholders. Dr. Miller of Gannett Fleming attended both meetings, and provided letter reports dated September 16, 2003, and November 16, 2003, respectively, that summarize the meeting notes and intermediate geotechnical findings.

On December 4-5, 2003, a meeting was convened at the mine site, where Molycorp presented to the SRB and the TRC their draft report entitled "Goathill North Slide Investigation, Evaluation and Mitigation Final Draft Report, December, 2003". Supporting data were provided in separately bound appendices. At the meeting, Norwest presented their findings and interpretations of the landslide movements and mechanics based on the field investigations. Norwest also described their proposed mitigation plan, which involved a combination of offloading from the head of the slide, drainage and earth buttress construction along a portion of the toe of the slide, and surface water controls. Immediately following the December 4-5 presentations, and in a letter dated December 12, 2004, the SRB requested additional analysis by Norwest. The additional analyses were needed to investigate specific concerns that the board had with the interpretation of the slide kinematics, and the effectiveness of the buttress as it was configured in the preliminary concept in preventing down-valley slide movements. Four specific additional work items were requested by the SRB.

Between late December 2003 and January 2004, Norwest carried out the additional work needed to respond to the board's requests. Also during that time period, an error was detected in the topographic mapping that was being used as the base for the analyses and design. The survey error was corrected, which resulted in favorable impacts on both the slide stability analyses and the mitigation plan design. A final report was prepared and was presented February 3, 2004 to the TRC and SRB at a meeting in Denver. The final report is entitled "Goathill North Slide Investigation, Evaluation and Mitigation Report, January, 2004". The data appendices that had been submitted in December were not revised, but additional data on shear strength test results were provided under separate cover.

A complete list of the Goathill North reports and appendices that were reviewed by Gannett Fleming is provided at the end of this memorandum. The December 2003 and February 2004 meetings were attended by representatives from Molycorp, Molycorp's consultants (Norwest, URS, and Golder), the Stability Review Board, NMMMD, NMED, Amigos Bravos, and Dr. Miller (representing the Village of Questa). Attendance lists and meeting agendas are also attached.

2.0 Summary of Geotechnical Findings and Slide Mechanisms

This section provides a summary of the key findings and interpretations of the landslide behavior based on the work to date by Molycorp and their consultants. This information is provided here because (1) it is relevant to understanding the SRB's letters dated December 12, 2003, and February 7, 2004, and (2) it provides the background and basis for our recommendations to the Village of Questa to endorse the proposed mitigation plan.

Based on Gannett Fleming's review of the geotechnical information, presentations by Molycorp and their consultants, and technical discussions between the TRC and SRB, the key technical points regarding the characteristics and mechanics of the landslide are listed as follows:

1. There is evidence based on historic pre-mining and post-mining aerial photographs that **an ancient landslide was present in the current slide area before mining ever occurred**. This recent finding/interpretation was reported by Norwest at the February 3 meeting in Denver. The inferred outline of the ancient slide is shown on Figure 2.1 of the January 2004 report. The possible ancient slide lies almost directly under the footprint of the existing landslide. It is believed that the ancient landsliding created a pre-sheared, weak zone below the ground surface, even before the waste pile was placed.

2. A significant portion of the landslide is underlain by bedrock with high pyrite content. These materials are prone to weathering processes that lead to clay formation. It is significant to note that the pyrite content is highest along the southern edge and mid-slide, and diminishes towards the western edge of the slide and towards the upper east portion under the waste rock materials (see Drawing 1 in the January 2004 report).

3. The pre-sheared, weak zone that the slide is moving on is located near the top of the weathered bedrock, at the interface between the bedrock and the overlying colluvial soils. It is typically about 20 to 30 feet thick, and is distinguished in the field by lower blow counts and white to light grey color. Based on recent soil classification tests (results provided at February 3, 2004 meeting), the "weak zone" appears to be distinctly more clayey and plastic than either the underlying bedrock, or the overlying colluvial and waste rock materials.

4. The groundwater table is very deep, ranging from 500 to 600 feet below the ground surface in the vicinity of the landslide. However, **there is a shallow, perched water table on top of the bedrock surface** that slopes gently down valley, sub-parallel with the ground surface. The perched water table emerges as a spring at about elevation 9100, above the bottom of the gully on the south side (see Figure 3.3 in the January 2004 report).
   - The perched water table has two important influences on the landslide: (1) pore water pressures could trigger slide movements on the shear surface, and (2) on-going seepage plays an important role in weathering and clay formation in the sulphide-rich bedrock.
   - Based on a detailed, site-specific analysis that accounted for the soil properties and climatic data, changes in pore water pressures within the slope due to storm events are expected to be minimal. However, significant long-term increases in the infiltration rate over time would result in an increase of the perched water table elevation. **It is very important to provide and maintain good drainage at the toe of the slide.**

5. The present-day sliding surface is located within the weak zone, and is well defined by slope inclinometer (SI) instruments that were installed within the lower and mid elevations of the slide mass during the 2003 geotechnical investigation. All 12 SI instruments that were installed on the slide have moved, indicating shear displacements are occurring to varying degrees. As of the latest available readings (January, 2004), 5 of the instruments have sheared to the point that they are no longer readable.
   - The shear displacements in the SI instruments indicate that the slide is moving at different rates (inches per month) and in different directions within various areas or zones of the slide mass. The movement vectors have been interpreted to show that

the slide changes direction, from nearly straight down-valley (east to west) in the upper part of the slide, then following a curved path southward, where its leading edge drops into the deep gully near the toe of the slide. Norwest has interpreted three different zones within the slide as follows (refer to Figure 5.1 in the January 2004 report):

(1) "Compression zone" along the western edge of the lower slide where the movement rates are very slow, ranging from 0.2 to 0.7 inch per month. This is where the rotation is occurring as the slide changes direction from down-valley (westward) to cross-valley towards the south.

(2) "Thrust zone" immediately above the compression zone along the north-western edge of the slide, where the SI's show two shear zones. It is believed that the slide mass is "stacking" in this area, which is interpreted to indicate that the failing rock pile on the slope above is applying significant thrust or driving force against the resisting compression zone.

(3) The remainder of the slide is moving at rates ranging between 1.2 and 3.2 inches per month along a rotating path, with higher rates and movements along the southwest "leading" edge as it rotates southward into the drainage.

3.0 Geotechnical Analysis of the Landslide

Slope stability analyses were done using computer models that simulate the landslide geometry, soil properties, and groundwater conditions. The analyses were done for two purposes: (1) to "back-analyze" the slide in its existing condition in order to confirm the general hypothesis of the slide behavior and to estimate the shear strength along the sliding surface, and (2) to design and optimize the stabilization measures.

Slope stability back-analysis is a common technique used in landslide investigations. In back-analyses, the landslide is simulated in a computer model, with the sliding surface defined and all the various soil and bedrock layers and groundwater levels depicted in the simulation as accurately as possible. The computer program solves what are called limit-equilibrium equations, in which the forces tending to drive the sliding mass downhill under gravity on specified failure surfaces are balanced against the resisting forces. The resisting forces in a moving landslide are primarily the friction and shearing resistance along the sliding surface. If the slide geometry, soil properties and groundwater conditions are accurately modeled, this technique can be used to estimate, or "back-calculate" the shear strength on the sliding surface. This technique was used by Norwest to analyze both the existing slide in its current configuration, and the estimated historical (pre-mining) landslide.

Conventional, two-dimensional slope stability models depict the landslide using cross sections constructed parallel to the inferred curved direction of movement. The back-analysis indicated average shear strengths for the entire sliding mass, represented as an angle of internal friction, ranging from 22.8° to 25.0°. Additional analyses were done to back-analyze the shear strength for the lower portion of the slide as a partially separated block moving southward into the gully. The SI readings indicate this portion of the slide is moving at a faster rate, and there are active tension cracks indicating accelerated slumping and displacement of materials into the gully below about elevation 9200. These analyses indicated a back-calculated shear strength on the weak zone of about 23.8°. Back-analysis of the pre-mining slide geometry resulted in a shear strength of 25°.

The results of the back-analysis were compared to laboratory shear strength tests on samples of soil taken from the weak zone. Laboratory test results showed a fairly wide range of shear strengths ranging from 16° to 30°, with an average of 23°. The shear strength of 23.8° for the weak zone was used to design the stabilization measures.

4.0 Mitigation Plan Elements

The basis for the mitigation plan design is to stabilize the landslide through a combination of unloading the driving mass from the upper part of the slide, and increasing the resistance on the lower part of the slide by constructing an earth buttress at the toe of the slide.

The design needs to ensure that the earth buttress does not "dam up" the groundwater seepage in the toe of the slide which could lead to buildup of the perched water table within the slide mass and have a destabilizing effect. Therefore, Phase 1 of the mitigation plan is to construct a gravel under drain within the gully along the south and west toe area of the slide.

Phase 2 will involve constructing an initial buttress by moving material from the upper part of the south (stable) side of the Goathill North rock pile to the toe area to be placed on top of the under drain. The primary purpose of this initial buttress is to improve stability on the slide so that it is safer for workers to access the active sliding mass. An estimated 200,000 cubic yards of material will be moved to construct the initial buttress. A secondary purpose for this work is to re-grade and flatten the upper part of the south side of the rock pile to allow for placement of reclamation cover vegetation.

Phase 3 includes excavating the top portion of the landslide to a bench at about elevation 9665. The waste rock and underlying soil material will be removed to a depth sufficient to expose the slip surface at the top of the bedrock. This will lessen the driving weight acting on the lower part of the landslide. The upper cut slope in bedrock will be approximately 1.5H:1V. Materials derived from this cut will be moved by some method to be determined (e.g., dozers, conveyors, etc.) to the toe area to construct the final buttress. Approximately 435,000 cubic yards of material are estimated to be generated from this cut and fill operation.

Phase 4 involves shaping the cut and fill slopes, and construction of armored channels to control surface water.

5.0 Review of SRB Comments on Slide Mechanisms and Mitigation Plan

The SRB in their letter report No. 7 dated December 12, 2003, expressed concerns about the mitigation plan proposed in the December report. In summary, these concerns were as follows:

1. The board interpreted slide kinematics differently than Norwest. The SRB viewed the slide as being pushed down-valley by the rock pile materials, but with a portion of the south side slipping out laterally (cross-valley). They visualized this cross-valley movement as stress release that was preventing the rock pile on the upper slide from pushing with full force on lower slide.

2. Based on the SRB's interpretation of the slide kinematics, they were concerned that the lateral buttress that was being proposed by Norwest to arrest the cross-valley movements along the southern portion of the slide would stop the stress release. This would then, in their view, cause more stress to be transferred to the lower (western) portion of the slide, which was resisting movement in the down-valley direction. They feared this condition would not improve the stability factor of safety in the down-valley direction along a representative straight down-valley cross-section E, and could potentially worsen the rate of down-valley movement of the slide.

3. The SRB in its December Report No. 7 also expressed concern about evidence from pre-mining photographs that an ancient landslide may have existed on the flank of the Goathill-Capulin ridge. The extent of the ancient landslide was not understood at the time of the December meetings. Additional air photo interpretation by Norwest concluded that the ancient slide was located under the current slide, and does not extend beyond the boundaries of the current slide mass. This was reported by Norwest in their January report, and acknowledged by the SRB in their February 7 report.

In their February 7, 2004, Report No. 8, the SRB states "We consider the proposed design to be feasible in principle for Goathill North slide mitigation." They qualify this statement by adding that they would prefer that the calculated factor of safety against sliding for the straight down-valley cross-section E more closely conform to computed factors of safety for other sections analyzed. The SRB cite 5 reasons for this concern, which we address in the next section.

6.0 Evaluation of Mitigation Plan and SRB Concerns

Based on our independent review of the technical information and participation in the discussions, Gannett Fleming recommends that the proposed mitigation plan be endorsed, with stipulations as noted in the accompanying letter. In our opinion, the proposed offloading in combination with the predominantly lateral toe buttressing is a reasonable approach for stabilizing the slide. If this plan does not work, it is our understanding that Molycorp will develop and implement a contingency mitigation plan that would likely involve additional down-valley buttressing using imported materials.

We would like to see a proposal from Molycorp for the criteria that will be used to evaluate the effectiveness of the mitigation plan. The performance criteria will need to encompass a specified time frame over which the movements are monitored and assessed to determine whether or not the slide has been adequately controlled.

The following commentary is intended to clarify and address the SRB's concerns regarding the slide mitigation plan. In the SRB Report No. 8, the board has expressed their preference that the 2-dimensional model factor of safety on down-valley section E be equivalent to the factors of safety criteria used on other sections. The design was optimized using slope stability calculations, with the goal of achieving a factor of safety equal to 1.2 on critical sections. (The factor of safety expresses the ratio of the resisting forces to the driving forces. A factor of safety equal to 1.2 implies that the forces tending to resist sliding are 20 percent greater than the forces tending to drive the mass downhill.) This goal was achieved on the curved cross sections A, B, and C, which follow the path of the slide based on the SI instrument data. However, the computed 2-D factor of safety on the straight down-valley section E is only 1.12 after completion of the offloading and buttress construction. At intermediate stages during construction, the 2-D factor of safety on section E will be as low as 1.04.

We offer the following comments on the SRB concerns regarding this issue:

1. It is evident from the SI instruments that the lower western and northwestern edge of the slide is moving, but at a much slower rate than the rest of the slide, and is providing substantial resistance to massive failure in a down-valley direction. The SRB refers to this resisting zone of the slide as the "passive block", and Norwest refers to it as the "compression zone". We interpret this behavior of the landslide to indicate that the resisting forces (friction and shear strength) under the lower, western part of the slide are higher than the shear resistance elsewhere under the slide, and offer the following justification for this interpretation:
    a. The higher shearing resistance in the western part of the slide may be partially attributed to the lower pyrite content in the bedrock in that area. It is speculated that the pyrite-rich bedrock under the south edge and middle portions of the landslide is more susceptible to weathering and weak clay formation.
    b. Further evidence that the "passive block" is providing significant resistance to down-valley movement is the "stacking" that is occurring along the north edge of the slide, just above the passive block. This area is called the "thrust zone" by Norwest.
    c. The shear strength along the basal sliding surface was calculated by several different methods, including back-analysis of the current landslide along a curved trajectory, back-analysis of the lower portion of the active slide as a separated block falling southward into the gully, back-analysis of the inferred ancient landslide, and laboratory shear strength tests. The results of the back-analyses are all fairly consistent, and indicate an average shear strength on the failure surface of about 23° to 24°. (The laboratory tests are not considered as reliable as the back-analysis because there is some concern that the direct shear tests were not carried out to sufficient strains to identify the lowest (residual) shear strength. However, the average value from the lab results was also on the order of 23°). Norwest conducted additional analysis for a slide trajectory straight down-valley on Section E, per a request by the SRB after the December meeting. When Norwest analyzed Section E with an assumed shear strength of 23°, the analyses indicated that the slide should be failing in the down-valley direction. However, this is not the case. We interpret this as further support to the argument that the shear strength on the lower western edge of the landslide is currently greater than 23° to 24°, which appears to be the average strength on the remainder of the slide.

2. If the shear strength on the sliding surface is higher than 23.8° under the lower, western part of the slide in the "passive block" or "compression zone", as we suspect it is, the computed 2D factors of safety along Section E, as reported on Table 6.3 in the January, 2004 report, are not correct. **The actual factors of safety on Section E both during and**

after construction may be higher than the numbers reported on Table 6.3. We cannot speculate on how much higher the actual factors of safety may be, because we do not know what the actual shear strength is under the resisting part of the slide, only that it appears to be higher than under the rest of the slide. The slope stability analyses indicate the factor of safety is extremely sensitive to the shear strength assumptions. The actual factor of safety on Section E following construction of the mitigation plan could be closer to or higher than the 1.2 criteria used for design on other sections, if the shear resistance that is currently exhibited by the passive block remains effective.

3. The SRB asserts that restraining lateral movement on the lower slide during Phase 2 could increase stresses on the passive block. (These stresses would ultimately be reduced once the upper slide material is offloaded during Phase 3.) The board is concerned that additional force on the passive block following Phase 2 could cause it to move in a down-valley direction, possibly reducing the shear strength under the resisting western segment of the slide. We offer the following comments on this concern:
    a. This is a compelling and interesting interpretation of the slide kinematics, but it is unclear how it could be evaluated using conventional techniques in geotechnical engineering. Slope stability calculations and failure criteria are based on limiting equilibrium mechanics, in which the slide mass is assumed to be a rigid body and principles of statics are applied to evaluate driving forces and resisting forces acting on that body for specified surfaces of sliding. Design involves altering the slope configuration and re-evaluating the limit equilibrium stability until suitable factors of safety are achieved. The SRB's hypothesis would require a different type of analysis to evaluate internal stress-redistribution within the body of the slide, and its potential effects on the slide stability. This specialized analysis could require significant time and effort to develop in our opinion, and we do not know of any previous cases where this type of analysis has been employed.
    b. The basal shear surface under the slide is believed to be shearing at its lowest possible shear strength, called the residual strength. A significant amount of displacement, or strain, must occur before the residual strength is achieved on the sliding surface. At smaller displacements under shear, the strength is higher than the residual strength. Norwest has modeled the slide as moving on a curved trajectory, constrained by the passive block on the west, with an average residual strength on the active slip surface of 23.8°. For a reduction in strength to occur under the stabilizing passive block, which is the concern of the SRB, it must be assumed that there has not been sufficient shearing in that area to have reached residual strength on the sliding surface. This means the slip surface under the passive block is currently at a strength higher than residual, which is supported by other evidence. We do not know if the current mobilized shear strength under the passive block is sufficient to resist an increased down-valley thrust that the board speculates could occur with the interim buttress. However, the 2-D limit equilibrium analyses predict that the slide would remain marginally stable, even if the shear strength was reduced to residual under the passive block.

4. In addition to the conventional 2-dimensional slope stability analyses, Norwest also performed 3-dimensional analyses of the slide to account for effects of shear resistance

along the entire base and sides of the landslide, and lateral restraint provided by the narrowing down-valley topography near the toe of the slide. The 3-D analyses indicate higher factors of safety than the 2-D analysis, which is typical and expected. The 3-D post-construction factor of safety is equal to 1.26 on Section E. The SRB expressed reluctance to rely on the 3-D analyses due to increased analytical difficulties and uncertainties. We agree that there is less comfort with the 3-D model, but we believe that these results should not be dismissed from consideration in the overall assessment. As with the 2-D analysis, the 3-D factor of safety was computed assuming residual strength everywhere on the basal shear surface, which is a conservative assumption.

5. As stated previously, the design considered stability on Section E using a residual strength assumption, which results in low factors of safety. As reported on Table 6.3 the factor of safety during the intermediate construction condition is as low as 1.04, and the post construction factor of safety on Section E is 1.12. In geotechnical practice, engineers typically try to design for long-term factors of safety on the order of 1.3 to 1.5. Lower factors of safety, such as 1.2, are considered acceptable for non-critical structures, or when more detailed geotechnical evaluations are conducted, thus increasing the confidence in interpretations of material strengths, subsurface geometry, and groundwater conditions. Even lower factors of safety, such as 1.1, are acceptable for unusual loading conditions, such as extreme earthquake events, or for temporary construction conditions. Considering the extensive work that has gone into characterizing the slide, and the conservative strength assumptions used in the analyses, we believe there is acceptable risk associated with factors of safety on the order of 1.2.

6. The SRB has noted that if the current mitigation plan does not successfully arrest the slide movements, it will be difficult to construct additional stabilization measures due to the steep topography. This is true, but it would be equally difficult in our opinion to construct the larger buttress at this time. We see no reason why the current mitigation effort should not go forward as designed, with a larger down-valley buttress option retained as a contingency plan in the event the proposed mitigation proves to be unsuccessful.

Gannett Fleming

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May 12, 2004

Mr. Marcus Rael, Esq.
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Subject: Goathill North Rock Pile – Emergency Monitoring Trigger Level Exceedence in Piezometer TH-C-05

Dear Mr. Rael:

This letter report summarizes Dr. Debora Miller's review and interpretation of piezometer data in the vicinity of the upper, north portion of the Goathill North (GHN) rock pile at the Molycorp mine. This review was initiated as a result of a trigger level being exceeded in one of the piezometers that is being monitored as part of the Emergency Action Plan (EAP) for the GHN landslide.

Background

On Friday, March 5, 2004, Dr. Miller was informed by your office that the water level in piezometer number TH-C-5-67 near the GHN rock pile had exceeded a trigger level that was set as part of the EAP. Piezometer TH-C-5 is located north of the active slide area at the top of the ridge that separates the Goathill and Capulin drainages. There are two piezometer instruments at this location. TH-C-5-67 is 67 feet below the ground surface, and measures water pressures in a zone near the base of the waste rock pile and above the bedrock surface. TH-C-5-172 is 172 feet below the ground surface, and measures water pressures deep within the bedrock.

The exceedence trigger level was set for the upper piezometer (TH-C-5-67) to indicate when water levels build up enough to saturate the lower portion of the waste rock materials. The trigger level at this piezometer was set at 66 ft below ground surface, which is 1.5 feet above the base of the rock pile. The piezometer readings indicate that the water table rose above this trigger level sometime between February 18 and March 3, 2004. This exceedence event was reported in the March 4 Weekly Report that was delivered to the TRC via email.

Molycorp and Norwest made a presentation to the village council on Tuesday, March 9, 2004 regarding this exceedence event. At the council meeting, Dr. Richard Dawson of Norwest presented a topographic map which showed that the ground surface is fairly flat in the vicinity of the piezometer. Dr. Dawson speculated that the buildup of water in the piezometer could be a result of snow melting from a large pile that had been placed on the flat ground adjacent to the instrument. The hypothesis was that the water level rise in the shallow piezometer was due to water infiltrating through the waste rock materials from the melting snow on the ground surface. As the water infiltrated downward it began to mound up on less pervious materials under the waste rock, either on bedrock surface or on the clayey colluvium layer immediately under the waste rock materials.

Two technical memorandums were submitted by Molycorp in reference to this issue: (1) a Preliminary Memorandum dated March 12, 2004 from Nancy Dessenberger of Golder Associates that accompanied the weekly reports that same week, and (2) a memorandum dated March 29, 2004 from Nancy Dessenberger and Richard Dawson to Ray Cherniske and Jim Vaughn that was delivered to the SRC via email along with the April 2, 2004 weekly report. A copy of the final March 29 memorandum is attached for reference with this letter report.

Review Comments on the Preliminary (March 12) Memorandum

After initial review of the Preliminary (March 12) Memorandum, Dr. Miller communicated with Dr. Dawson and Ms. Dessenberger on March 16 and 17, 2004 via email, fax and phone to discuss the possible implications of the piezometer readings. At that time, Dr. Miller provided two sketches to illustrate her interpretation of the data, and a possible perched water table condition in the upper north part of the GHN rock pile and slide area.

The attached sheets labeled Fig. 3.3 and Fig. 5.1 are the sheets that Dr. Miller faxed to Dr. Dawson and Ms. Dessenberger to illustrate the possible perched groundwater table in the upper portion of the GHN rock pile. The perched, water table elevations are shown on the second column on Table 1. These water elevations were calculated from the readings in the shallow piezometers - TH-C-05-67 at the top of the ridge, and TH-GN-02-139, the closest piezometer situated on the landslide - as reported in the March 4, 2004 weekly report. Elevations were calculated from the reported depth to water table readings, referenced to the ground surface elevations as reported in Appendix F, Volume II of the GHN slide mitigation report.

Table 1. Piezometer Data

<table>
<tr><th>Piezometer</th><th>Estimated Perched Water Table Elev[1]</th><th>Estimated Bedrock Water Table Elev.</th><th>Ground Elevation</th><th>Approximate Bedrock Elevation</th></tr>
<tr><td>TH-C-05-97</td><td>9725</td><td></td><td rowspan="2">9790.5</td><td rowspan="2">9717.5</td></tr>
<tr><td>TH-C-05-172</td><td></td><td>9630[2]</td></tr>
<tr><td>TH-GN-02-139</td><td>9320</td><td></td><td rowspan="2">9455.7</td><td rowspan="2">9313.7</td></tr>
<tr><td>TH-GN-02-330</td><td></td><td>9256[3]</td></tr>
</table>

1 From readings reported in March 4, 2004 weekly report

2 From March 29, 2004 Norwest memorandum on TH-C-05-67 exceedence event

3 Estimated from data sheets provided in Appendix G, Volume II "Goathill North Slide Investigation, Evaluation, and Mitigation", dated November 2003. Latest reported reading on TH-GN-02-330 was October 2003.

Contours were sketched on topographic base maps to show the estimated shallow water table surface based on these two piezometric data points. The perched groundwater contours are estimated using a simple linear extrapolation between the piezometric readings and an assumed bedrock surface. The base maps for both of these figures are from the report January 2004 GHN slide mitigation final report. Figure 3.3 in that report shows the pre-dump (1962) ground contours. That figure was used as a rough guide to define the shape of the assumed perched water table. The groundwater surface profile was sketched between the two piezometer locations by generally following the slope of the pre-mining ground surface topographic contours. The assumption is that the perched water is flowing along the top of the bedrock surface, which is assumed to generally mimic the contours of the ground surface. These sketched groundwater contours were then transferred to Figure 5.1, which shows the estimated contours at the base of the landslide. Based on the rough groundwater contours sketched onto Figure 5.1, the perched water table in the upper north part of the GHN slide appeared to very nearly coincide with the elevation of the base of the slide.

This interpretation of the data results in a higher water table condition in the upper, north part of the landslide than was assumed in the stability analysis which was the basis for the mitigation design.

Although this perched water table condition, if it exists, may have little or no impact on the overall slide stability or the mitigation plan, we do believe this alternate interpretation of the data should be considered in future monitoring and analysis of the slide mitigation. The following sections describe in more detail the differences between Gannett Fleming's and Molycorp's interpretation of the piezometric data. We also provide recommendations for supplemental analysis that could be done to evaluate the consequences of our alternate interpretation.

Review Comments on March 29 Memorandum

Perched Water Condition. The March 29 memo from Norwest interpreted the water level in TH-C-05-97 as "…due to a perched water table located at the mine rock/clayey colluvium contact or the colluvium/bedrock contact." Gannett Fleming agrees with this interpretation.

This paragraph is intended to further explain what is meant by a "perched water table". The piezometric data from most piezometers on GHN indicate that a strong downward gradient exists under most of the slide area, as discussed in detail in Addendum A in the January 2004 mitigation plan report. This means that water infiltrating from rainfall and snowmelt works its way downward through the waste rock and native soils and eventually penetrates the bedrock and continues seeping down towards the deep bedrock aquifer that is several hundred feet below the ground surface. While this downward flow is occurring however, the infiltrating water may occasionally "perch" or build up on less pervious layers, notably the bedrock surface or the clayey colluvial materials underlying the more pervious waste rock. This perching of water on less pervious layers may be accelerated at times of heavy rainfall or snowmelt. As the water mounds up on the bedrock surface a shallow water table is formed which "runs off", or flows down-valley through the soils on top of the steeply sloping surface of the bedrock. At the same time the downward percolation into the bedrock is continuing.

Near the head of the valley (near TH-C-05) the shallow perched water table and the deeper bedrock water table are separated, and there is very likely a zone of unsaturated bedrock between the top of bedrock and the deeper bedrock water table. The deeper water table is located approximately 96 feet below the upper, perched water table. It is fairly certain that these water tables are not directly hydraulically interconnected because the response of the deeper piezometer was different from the upper instrument. The deeper instrument indicated a nearly constant reading up until the time when it stopped functioning in late February 2004. It did not register a gradual rise starting in late January as the shallow instrument indicated (see Figure 3 in the Norwest memo).

Further down valley the perched water table may form a continuous saturated zone with the bedrock water table, although there is still a strong downward gradient (downward flow). In the lower slide area we see springs emerging on the slope where the perched water table has developed high enough to flow out on the ground surface.

Comparison between TH-C-05-67 and TH-GN-02-139 readings. Norwest concluded that there is "…no indication that the stability of the mine rock pile at Goathill North has been significantly impacted by the wetting event." Gannett Fleming generally concurs with this conclusion based on observation of a rising water table at TH-C-05-97 and the null response of piezometer TH-GN-02-139 (located on the landslide) during the same time frame. This is explained in the following paragraphs in reference to Figures 1 and 2 attached with this letter report.

Piezometer readings for instruments that have an established trigger elevation have been provided to the TRC on a weekly basis. However, the plots that accompany the weekly reports show the data on such a compressed vertical scale that it is difficult to detect subtle trends. Figure 1 with this letter report shows the piezometer readings for TH-C-05-97 since December 2003 on an expanded vertical scale that allows easier visualization of the piezometer behavior. This graph was created from the tabulated data provided in the weekly reports. Figure 1 clearly shows the rising trend in the water level in that piezometer since late January 2004. The rise continued through mid April, and the last two readings show that the water level in that piezometer is beginning to decline.

Figure 2 shows a graph of the depth of water above bedrock in both TH-C-05-67 and TH-GN-02-139 since December 2003. The depths of the perched water table are similar (6.6 to 8.8 ft above bedrock in TH-C-05-67, and 6.7 to 7.7 feet in TH-GN-02-139). However, the rising trend that was evident in TH-C-05-67 is not apparent in TH-GN-02-139. This is interpreted to indicate that the localized "mounding" of the perched water table in the vicinity of TH-C-05-67 did not have a measurable impact on the perched water table further down the slope. In fact, over the same time period that TH-C-05-67 was rising, the depth of water in TH-GN-02-139 was declining.

Water Table Interpretations. Figure 1 in the final (March 29) technical memorandum from Norwest (attached) shows the location in plan of a cross-section through the GHN slide area that is bent at the location of TH-GN-02 to cut through the north margin of the slide and intersect TH-C-05 at the top of the ridge. For purposes of this discussion, we have enlarged the upper portion of the cross section from Norwest's memo (Figure 4 in the Norwest memo) and attached the enlarged figure as Figure 3 to this letter report.

Norwest's interpretation of the water table profile is labeled "Water Table Assumption by Norwest" on Figure 3 of this letter report. The Norwest interpretation connects the deeper bedrock water table elevation at TH-C-05-172 to the shallow perched water table elevation at TH-GN-02-139.

Gannett Fleming has a somewhat different interpretation of the water table profile in the upper portion of the slide. We interpret the data as showing two separate water tables, represented by lines labeled "Perched Water Table Assumption" and "Bedrock Water Table Assumption", as shown on Figure 3. The perched water table connects the elevations of the shallow piezometers at TH-C-05-97 and TH-GN-02-139. The bedrock water table connects the elevations of the deeper piezometers at those same locations (TH-C-05-172 and TH-GN-02-330). Based on this alternate interpretation, there are two, distinct piezometric surfaces in the upper Goathill North drainage as follows:

- a shallow water table that flows down valley on top of the steep bedrock surface through the colluvium and waste rock materials, and
- a deeper water table within the bedrock that may be hydraulically separated from the shallow water table by an unsaturated zone in the upper part of the drainage.

Potential Implications for Slide Stability
In the stability analysis that was completed for the slide investigation and mitigation plan, Norwest assumed a water table configuration similar to the line labeled "Water Table Assumption by Norwest" shown on Figure 1 of this letter report. This water table assumption indicates pore pressures are present on or near the slide plane on the lower part of the landslide (under the colluvium part of the slide mass). However, the designers assumed no pore pressures on the slide plane in the upper part of the slide (under the waste rock portion of the slide).

With the alternate water table interpretation of a perched condition labeled "Perched Water Table Assumption" on Figure 1, there could be 6 to 8 feet of pore water pressure acting on the upper portion of the slide plane. This is expected to have some impact on the computed slope stability factors of safety, but the overall impact on the mitigated slope stability is unknown.

Summary and Recommendations
Gannett Fleming concurs with the analysis and conclusions presented by Norwest regarding the exceedence event in TH-C-05-67 as follows:

- The rising water level in TH-C-05-67 between late January and late April 2004 is most likely due to localized subsurface mounding of a perched water table in the vicinity, very likely caused by melting of a large stockpile of snow on a flat area adjacent to the piezometer.
- The water table rise in TH-C-05-67 was not observed in a shallow (above bedrock) piezometer on the slide area (TH-GN-02-139). We interpret this to mean that the

mounding effect in the perched water table near TH-C-05-67 did not have a measurable impact on the pore pressures or stability of the GHN slide. This is also in agreement with Norwest's conclusion.

As a result of our review of these piezometer data, however, Gannett Fleming believes that a shallow perched water table may be present above bedrock in the upper part of the GHN rock pile including a portion of the upper north slide area. Based on the data from TH-C-05-67 and TH-GN-02-139, the perched water table is about 6.5 to 8 feet deep above bedrock. As this interpretation of the water profile was not modeled in the original slope stability analysis that was the basis for the slide mitigation plan, we suggest the following actions:

- As part of the monitoring program during and following slide mitigation construction activities, install piezometers in the upper slide area to determine if a perched water table is present, and to monitor pore pressures in the perched water table zone, if it exists.
- If additional instrumentation verifies that a perched water table is present in the upper slide area, conduct additional slope stability analysis with appropriately revised pore pressure assumptions, and report the revised factors of safety.

Please do not hesitate to call with any questions or concerns.

Sincerely,
GANNETT FLEMING, INC.

Debora J. Miller, P.E., Ph.D.
djmiller@gfnet.com

cc: Jim Langer, Gannett Fleming

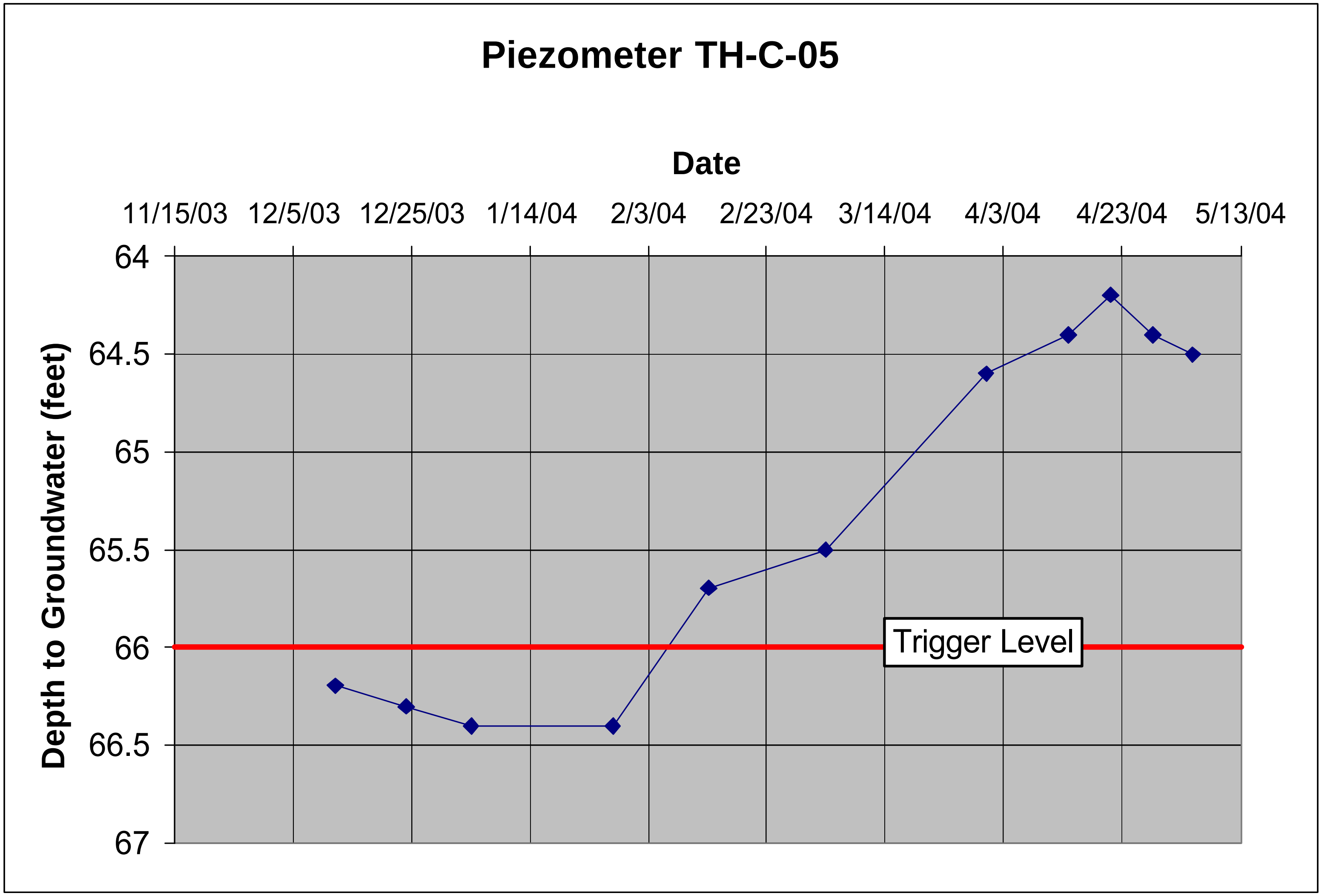

Figure 1. Expanded scale plot of TH-C-05-67 piezometer behavior since December 2003

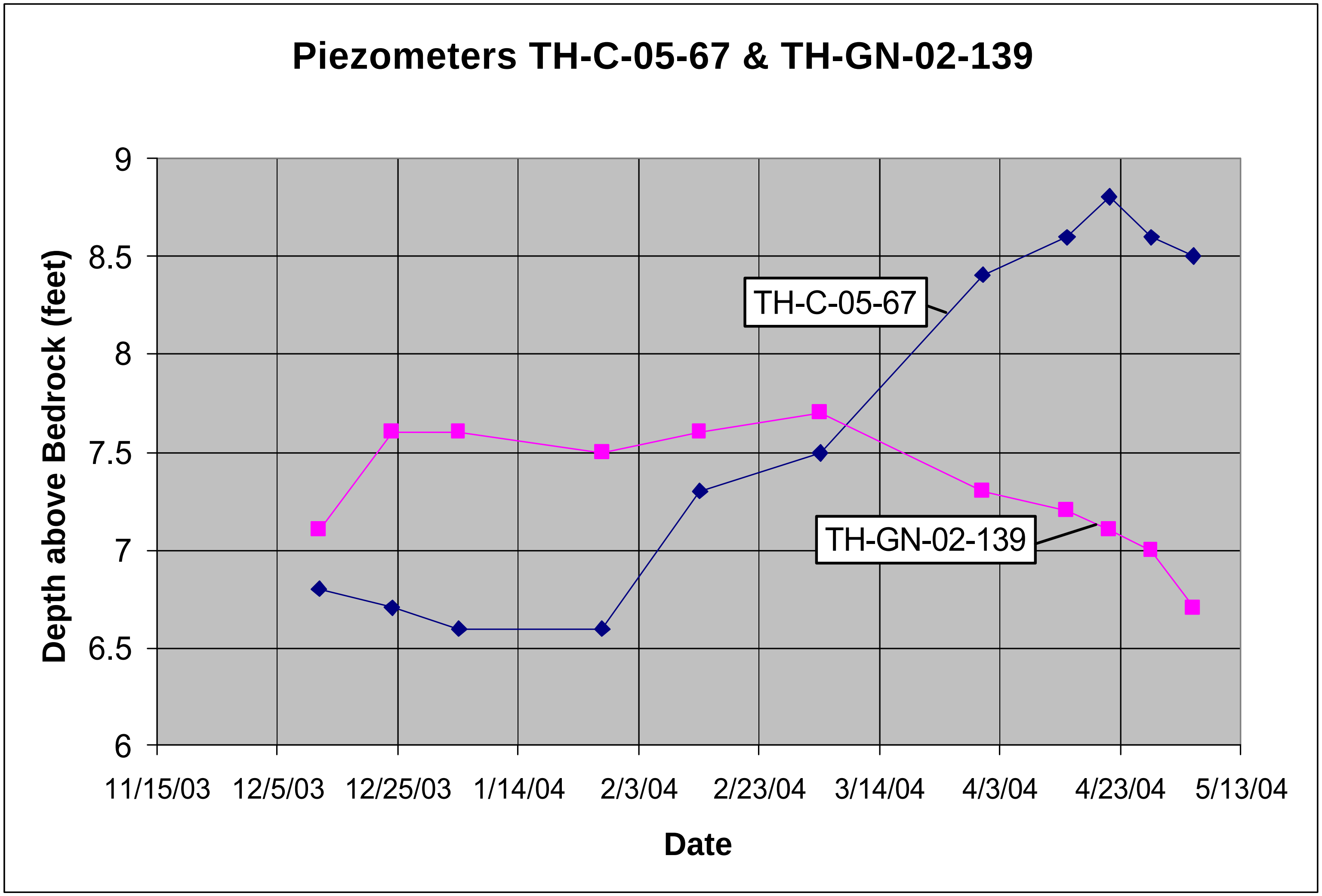

Figure 2. Depth of water above bedrock in TH-C-05-67 and TH-GN-02-139

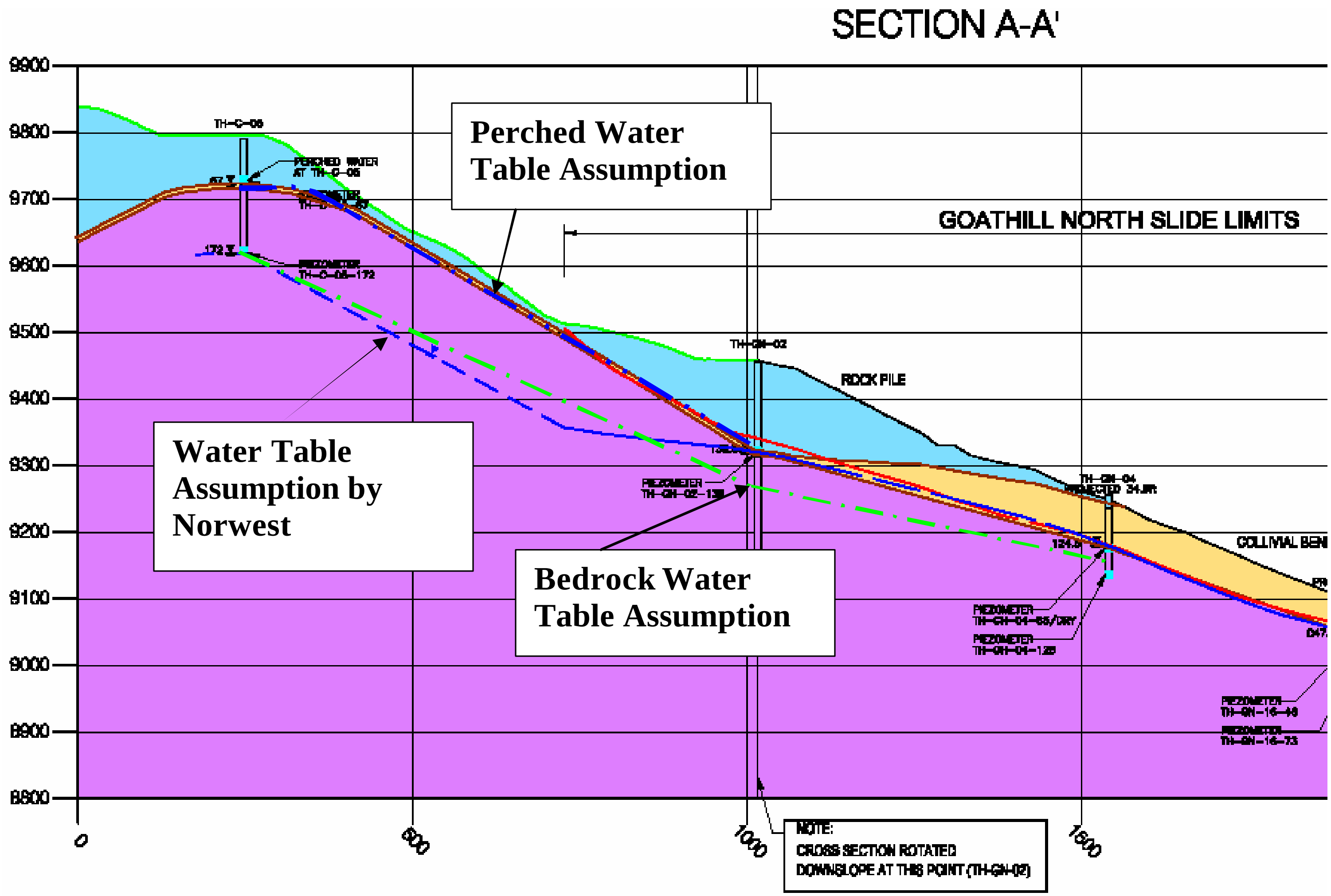

Figure 3. Water Table Assumptions in Upper North Portion of GHN Rock Pile

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July 16, 2004

Mr. Marcus Rael, Esq.
Robles, Rael & Anaya, P.C.
500 4th Street NW, Suite 200
Albuquerque, NM 87102

Mr. Terry Foreback
New Mexico Mining and Minerals Division
1220 South St. Francis Drive
Santa Fe, NM 87505

Subject: Recommendations for analysis to include in the Final Design Report for the Goathill North Rock Pile Mitigation

Dear Mr. Rael and Mr. Foreback:

Molycorp has indicated that they will be providing supporting engineering analysis to accompany the Goathill North Rock Pile Mitigation Project, Final Design Submittal. During our conference call on July 9, 2004, Norwest briefly reviewed the results of their final design analysis through a PowerPoint presentation which focused on the differences between the Final Design and the Feasibility Design. That presentation included a table summarizing anticipated slope stability factors of safety on representative Cross Sections A, B, C, D and E at the conclusions of Phase 2 (Initial) and Phase 3 (final) buttress configurations.

On Friday, July 16, I received from Mr. Foreback by email a CADD drawing developed by Keith Ehlert showing an additional cross section orientation on the stable rock pile slope that he would like to see evaluated for slope stability during Phase 2. That section is oriented east-west through the stable cut-fill area.

Gannett Fleming assumes that the final design report will provide the assumptions used for shear strength and pore water pressures, and methods of analyses for the stability factor of safety results reported in the July 9, 2004 presentation, and for any additional analyses to address Mr. Ehlert's recommended cross section orientation.

In addition to Sections A-E and Mr. Ehlert's cross section, I recommend the following analysis results and discussion be provided as part of final design documentation:

1. ***Slope stability in the area referred to as the "Cracking and Slumping Zone" on the lower portion of the active slide during intermediate stages of Phase 2***. The area of

concern is on the active slide below SI-10, where the basal shear plane tends to steepen towards the toe into the gully, and an "active tension zone" has been identified in the January 2004 report (Section 5.3.1). The area is labeled "Cracking with Slumping Zone" on Figure 3.6 in the January 2004 report. We recommend that Norwest investigate and report factors of safety at intermediate phases of construction of Phase 2 (e.g., 2A, 2B, and/or 2C, as shown on Drawing 4 of the Final Design Submittal) for an appropriate truncated section or sections on this lower zone (e.g. lower Section B). The analyzed section (or sections) should be truncated, or "daylighted" on the upslope side in the vicinity of mapped tension cracks which have formed in the colluvium. The purpose of these analyses is to understand the intermediate stability and risk of initiating localized slumping on the lower slide, which could lead to progressive failure of the larger slide mass during construction of Phase 2.

2. ***Provide documentation of sensitivity analysis for a possible perched water table condition, as described by Debora Miller in a letter to the Village of Questa dated May 12, 2004.*** Document the slope stability factors of safety for representative cross sections (A, B, C and E) following completion of Phase 3, for the alternative, perched ground water assumptions as described in the attached letter. Preliminary results from supplemental back-analysis and forward analysis with the revised water table assumptions were provided to Dr. Miller via email from Norwest on May 20, 2004. These analyses should be fully documented with appropriate discussion, assumptions, and calculations.

Please do not hesitate to call or email with any questions or clarification on these items. I will be in my office through Tuesday, July 20, and will be back in the office August 11 after my vacation.

Sincerely,
GANNETT FLEMING, INC.

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cc: Jim Langer, Gannett Fleming
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February 7, 2004

Mr. Terence Foreback
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1220 S. St. Francis Dr.
Santa Fe, NM 87505

Report No. 8
Goathill North and Capulin Rock Piles
Mine Rock Pile Stability Review Board
Questa Mine

Dear Mr. Foreback:

This letter follows the meeting of the Stability Review Board on February 2 and 3, 2004, during which we met with the Stability Committee and heard technical presentations from Molycorp's consultants concerning the most recent Norwest report entitled *Goathill North Slide Investigation, Evaluation and Mitigation Report* submitted on January 28, 2004. It revises and supplements Norwest's previous *Goathill North Slide Investigation, Evaluation and Mitigation Final Draft Report* submitted on December 1, 2003. Our comments here refer to the January 28, 2004 document.

Like its predecessor, the January 28 report presents a feasibility-level design for Goathill North slide mitigation. It incorporates updated topographic information used in refining the analyses and the proposed mitigation measures. It also includes information to address the four items requested by the Board in our Report No. 7 of December 12, 2003. Among the more significant enhancements is a revised airphoto interpretation that locates an ancient landslide beneath the Goathill North rock pile, making it probable that reactivation of this slide was chiefly responsible for initiating rock pile movements. This fills an important gap in understanding the origin and mechanisms of the current Goathill North slide by providing a more coherent and complete picture of the factors involved in its development. This also lends confidence that the proposed mitigation measures should not affect, or be affected by, any larger unstable mass extending beyond the boundaries of current movements.

We consider the proposed design to be feasible in principle for Goathill North slide mitigation. We would prefer, however, that the calculated factors of safety for Section E conform more closely to the safety factor criteria adopted for the other sections analyzed for the following reasons:

1. As explained in Board Report No. 7, constraining lateral movement of the active block during Phase 2 of the proposed mitigation may increase stresses on portions of the passive block at the toe of the slide, which currently provides the principal restraint on slide movement. Concurrently increasing the factor of safety at Section E will reduce the potential for adverse effects.

2. Increased stress on the passive block could cause future reduction of strength along the existing shear surface that extends beneath it. Present and past movements of the passive block appear to be substantially less than elsewhere, making it unclear whether these movements have been sufficient to reduce the current strength to minimum (residual) values. Increasing the factor of safety at Section E will help accommodate any such future strength reduction.

3. Increasing the factor of safety for Section E will help reduce reliance on three-dimensional effects and the associated analytical uncertainties described in Board Report No. 7.

4. Increasing the factor of safety for Section E will reduce the influence of decreases in calculated stability that are predicted to occur during Phase 3a construction.

5. Should the mitigation measures as proposed be insufficiently effective, topographic and logistical factors would make it difficult to implement any additional stabilization measures that might otherwise be reserved as contingencies. Increasing the factor of safety at Section E will reduce the possibility that any such additional measures could become necessary.

Respectfully submitted,

____
James K. Mitchell

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Nigel Skermer

____
Steven G. Vick, Chairman

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July 2, 2004

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Goathill North Construction Documents

Dear Mr. Foreback:

This letter forwards the Board's remarks on the volume dated May 27, 2004, entitled *Goathill North Rockpile Mitigation Project Final Design Submittal,* and related material that has come to our attention. The May 27 submittal contains construction plans and specifications for Goathill North slide mitigation, together with Molycorp's responses to agency comments. As noted on the drawings, the total fill volume of 1,025,000 yd^3 (exclusive of underdrain material) represents nearly a 50% increase from the 700,000 yd^3 total fill volume in the feasibility-level design submittal of January, 2004. This would amount to more than an additional half-million tons. In particular, Fill Item B in Phase 2 on the drawings has increased by 255%.

If these numbers are reliable, this constitutes a major change from the design that we understood was still in place at the Stability Review Board meeting in Questa on May 25-26, 2004. We do not understand why a revision of this magnitude was not made known to us during the May 25-26 meeting when all parties were present, especially since we were told in the January, 2004 Norwest report (p. 6.5) that any fill increase would be minor and only as a construction expedient:

> *"During the preparation of detailed construction drawings some rationalization of final surface profiles may be necessary to address ease of construction and as a result final fill volumes may slightly increase."*

The May 27, 2004 submittal contains no engineering assessments to substantiate the revised design. We therefore consider it incomplete in this respect, and we are correspondingly unable to fully evaluate the effects of this change. There is reason to believe, however, that it may have the potential to adversely affect movements during construction. In conveying ½ million yd^3 of fill down the slope during Phase 2 operations, we would suggest that the possibility of the onset of "undesirable movements" is not "remote" (Section 1.6.4). It should not be assumed that the Board's previous review and comment on the feasibility-level design necessarily pertain to the revised design presented in the May 27 construction documents, despite the monitoring and contingency options described in Sections 1.6 to 1.7.

We were made aware that the agencies nevertheless approved the revised design in their joint letter of June 16. Notwithstanding its lack of engineering substantiation, the success of the Goathill North slide mitigation project cannot be predicted because the agencies and Molycorp have not yet determined what this means. According to Condition #9 of that letter, this will be decided at some later time while construction is in progress.

Separately, we learned from MMD's website of the existence of the preceding agency letter dated June 7 concerning a 40,000 yd^3 toe berm. In that letter, the agencies jointly expressed their concurrence with the following statement from the Board's report of June 5, 2004 and cited it to support their determination:

> *"Molycorp stated once more during our meeting that their objective is to* ***stop*** *slide movements…"* (emphasis added)

This would appear to leave no ambiguity regarding how project "success" was defined by Molycorp as of the May 25-26 Board meeting, or by the agencies as of June 7. We can see no reason why this definition was superseded in the agency approval letter of June 16 and why the matter is now being deferred. If successful project performance is accepted as something other than stopping slide movements, then the stated premise of the June 7 approval will not be valid.

We would like to be clear that at the time the Board made its June 5, 2004 comments on slide mitigation, including those quoted by the agencies, the revised design had not been shown to us. Neither at that time had we been made aware of any equivocation about stopping slide movements. As of this date, the Committee has not supplied us with the June 7 agency letter. The last Board member did not receive the May 27 submittal until June 25.

Respectfully submitted,

____
James K. Mitchell

____
Nigel Skermer

____
Steven G. Vick, Chairman

Appendix B

Construction Details

Appendix B Construction Details

Work Scope: Goathill North rock pile mitigation scope of work

Specifications: Goathill North rock pile mitigation specifications

Drawing Cover Sheet

Drawing 1: Site Plan

Drawing 2: Phase 1 Rock Drain

Drawing 3: Phase 1 Rock Drain

Drawing 4: Phase 2 Construction

Drawing 5: Phase 3 Construction

Drawing 6: Phase 4 Construction

NORWEST CORPORATION

GOATHILL NORTH - WORK SCOPE

1 GENERAL

1.1 SCOPE OF WORK

The work consists of the supply of all materials, labour and services required for the complete construction of the **GOATHILL NORTH (GHN) ROCK PILE MITIGATION**.

The main features of the work are as follows:

Spring Gulch Quarry:

a) Screening of stockpiled Black Andesite / Aplite rock for under drain rock **(Bid Item TP-005).**

b) Screening of stockpiled Black Andesite / Aplite rock for road base gravel **(Bid Item TP-011).**

Goathill North Rock Pile:

a) Construction and upgrading of access roads **(Bid Items TP-002, 003).**

b) Clearing **(Bid Item TP-004).**

c) Haul and place under drain rock materials **(Bid Items TP-006, 007).**

d) Controlled cut and fill on stable rock pile and slope below **(Bid Item TP-008).**

e) Controlled cut and fill on buttressed rock pile and slope below **(Bid Item TP-009).**

f) Controlled cut and fill on finished slope for interim surface drainage controls **(Bid Item TP-010).**

g) Haul and place road base gravel **(Bid Item TP-012)**.

1.2 CONTRACT AWARD PROCESS

The Contractor is required to bid the work based on the construction methodology and the sequencing specified herein Section 1.5 of this Work Scope as the preferred Option ‘A’. Contractors also have the option of submitting alternate options to any phase of the construction as Options ‘B’, ‘C’ etc. Alternate options may or may not be considered in the award of the Contract based upon their merits as determined by the Owner and Engineer.

Upon award of the contract the Contractor shall meet with the Owner and the Engineer to negotiate contract details and confirm construction methodology. Final construction drawings and specifications will then be prepared by the Engineer and will be submitted to the State Agencies for approval.

1.3 PERFORMANCE OF WORK

The Contractor shall complete the specific items of work on or before certain key dates that will be confirmed during the bid walk meeting on site.

1.4 CONSTRUCTION PROGRAM

The Contractor's construction operations shall be subject at all times to the review of the Owner's Representatives for compliance with the Contract Documents. The capacity of the Contractor's construction equipment, sequence of operations, and methods of operation shall be such as to insure the completion of the work within the period of time specified in Section 1.3 and in accordance with the Construction Sequence specified in Section 1.5.

Prior to the start of construction, the Contractor shall submit a construction schedule to the Construction Manager for approval. This shall take the form of either a bar chart or a critical path method network diagram, which stipulates the amount and kind of labour, material, and equipment resources to be assigned to each performance period of each work item.

The Contractor shall immediately advise the Construction Manager of any proposed changes in the construction schedule and shall update this schedule every two weeks. If, in the opinion of the Construction Manager, any construction schedule as submitted is inadequate to secure the completion of the work within the specified period of time, or is otherwise not in accordance with the specifications, or if the work is not being adequately or properly executed in any respect, the Construction Manager shall have the right to require the Contractor to submit and adhere to a new construction schedule providing for proper and timely completion of the work, and the Contractor shall be entitled to no claim for additional compensation on account of such requirements.

In preparing the construction schedule, the key dates specified in Section 1.3 and the sequence specified in Section 1.5 shall be taken into account, allowing for the deployment of the necessary plant, labour, material and equipment resources to comply with those requirements.

1.5 CONSTRUCTION SEQUENCE AND METHODOLOGY

1.5.1 General

The following construction sequence and methodology were developed to achieve a necessary level of stability during all phases of construction and ensure compliance with current regulatory and industry safety and environmental best practices is maintained. The Contractor shall perform the work in the sequence specified below, or shall propose alternative sequencing that will achieve equivalent stability and safety standards and have equivalent minimal impacts on the environment, subject to the approval of the Engineer and Owner.

All cut and fill operations carried out by the Contractor will be subject to movement monitoring by the Owner and the Construction Manager as outlined in Section 1.6. Should monitoring indicate unacceptable movements are occurring, the Contractor will be required to stop work immediately and follow the contingency plans established for the particular phase of the work as directed by the Construction Manager. Contingency plans for each phase of the cut and fill operations are outlined herein Section 1.7**.**

1.5.2 Access Roads:

The Contractor will be responsible for upgrading existing access roads and construction of additional roads to the Mitigation Site and construction of additional access roads on site as necessary for the duration of the works. Existing access roads to the Mitigation Site are shown on the Drawings, together with prescribed lay-down areas and roads that will require upgrading for access to lower work areas. Upgrading is expected to include regrading of some steep sections of road, provision of run-outs at the bottom of the steeper sections, general widening of some narrow sections, additional widening at suitable passing locations and construction of safety berms based on the size of equipment expected to use the road.

It shall be the responsibility of the Contractor to ensure that access roads are designed and upgraded to meet State and MSHA requirements.

It shall be the Contractor's responsibility to maintain the existing site access roads and construct, as an incidental expense and at its own risk, any temporary haul roads, access roads, bridges and drainage structures that may be required to perform the work within the Site Limits. The Contractor shall submit a plan of the proposed temporary roads for approval by the Engineer and Construction Manager, prior to completing any access road construction or maintenance.

Adequate drainage facilities in the form of ditches, culverts, or other conduits shall be installed as may be necessary to maintain these roads. In the construction of access roads, existing drainage facilities, natural or otherwise, shall not be disturbed to the detriment of properties outside the working area and such facilities shall, unless otherwise provided elsewhere in the specifications, be restored to their original condition on completion of the Work.

1.5.3 Granular Fill Processing:

The Contractor will be responsible for processing Granular Materials from on-site stockpiles at Spring Gulch to produce the materials specified in **Section 02250** of the Specifications **(Granular Materials)** for Drainage Rock and Road Base Gravel.

It is anticipated that selective excavation within the existing stockpiles will provide all the granular materials required for the mitigation work **(Bid Items TP-005, 010)** and no additional quarrying or blasting will be necessary. Selection of the materials will be carried out under the direction of the Construction Manager with ongoing visual inspection of materials to ensure that any potentially acid generating (PAG) mixed volcanics (characteristically yellow from jarosite staining) and any porphyry and Andesite with abundant pyrite, are not processed.

1.5.4 Construction of Rock Under drain – Phase 1:

The first phase of construction will be the installation of an under drain within the existing drainage gully below the toe of the GHN rock pile. Rock material required for the drain will be processed by the Contractor at Spring Gulch as outlined in Section 1.5.3 and will be hauled directly to the proposed placement locations shown on the Drawings with no intermediate transfer points or stockpiling. The Contractor will determine the size and number of trucks required based on the total volume to be hauled, the haul distance, the nature of the haul road and the placement schedule specified in Section 1.3.

A minimum of three locations for the trucks to dump the hauled rock will be prepared by the Contractor as indicated on Drawing 2. Access to these dumping areas will be constructed or upgraded with native material. Drain rock shall not be used for road construction. From these locations the drainage rock will be dozed into place from the north side of the existing gully working from the top down. The Contractor shall determine the type and size of equipment necessary to perform the rock placement in such a manner as to minimize contamination within the drain from native materials.

During this phase of the Work the Contractor will prepare the area of the site to be filled during Phases 2 and 3 as shown on Drawings 4 and 5, by clearing trees brush and vegetation in accordance with **Section 02110 (Clearing)** of the Specifications.

The Contractor is advised that the gully in which the rock drain is to be constructed is adjacent to a natural scar slope that may be susceptible to isolated rock falls, primarily during storm and high run-off events. This potential hazard will be controlled during the work by ongoing monitoring of the condition of the slope in accordance with the Monitoring Program outlined in Section 1.6, and by restricting movement of personnel and equipment near the toe of the scar slope. At no time shall personnel or equipment be permitted to work in or along the gully parallel to the toe of the scar slope and at no point along the gully shall drainage rock be pushed level all the way across the gully to the toe of the scar slope until final grading in Phase 4 is carried out (See Drawing 3). A 20 ft wide catch ditch will at all times be left between the placed drainage rock and the toe of the scar. In addition, a safety berm of drainage rock at least 6 ft high will always be left in place at the crest of the catch ditch.

Prior to and possibly during Phase 1 of the Work a drill rig and crew will be installing instrumentation within the Mitigation Site area at locations shown on Drawing 2 and under the control of the Construction Manager (SI-25, 26, 27, 28 and 29). These installations are required as part of the Monitoring Program outlined in Section 1.6 and they need to be installed before subsequent phases of the work can proceed. The Contractor will provide and maintain access to the drill sites in accordance with the requirements of Section 1.7 and as directed by the Construction Manager.

Two existing instrumentation sites (SI-18 and SI-23) are expected to be functioning when the Works start in the lower part of the Mitigation Site as shown on Drawing 2. The Contractor shall locate, protect and maintain access to these sites to allow ongoing monitoring to be carried out by others during Phase 1. Should the monitoring of the existing instrumentation indicate movements of the slide area that exceed the maximum allowable movement defined in Section 1.6, the contractor shall stop work, move personnel and equipment off the Mitigation Site, and follow the contingency plans outlined in Section 1.7 as directed by the Engineer and/or Construction Manager.

The sequence of the Work requires Phases 1 and 2 to be complete before any operations can proceed on the north part of the Rock Pile. To ensure that this requirement is met the Contractor shall not allow any personnel or equipment to enter the 'NO ACCESS' zone shown on Drawing 4.

1.5.5 Grading Operation in Stable Rock Pile Area – Phase 2:

When the full section of the rock underdrain is complete from Stations 0+900 through to Station 0+300, all instrumentation required prior to Phase 2 has been installed, initial readings have been taken (by others) and all work has been approved by the Construction Manager, grading operations on the stable south side of the Goathill North Rock Pile may commence as directed by the Construction Manager. Rock pile material shall be excavated from the upper part of the stable pile in accordance with **Section 02200 (Earthworks)** of the Specifications and shall be dozed towards the west, directly down the south side of the pile maintaining a rehandle bench and a 2H:1V finished cut slope profile in accordance with Drawing 4. At the toe of the rock pile this material shall be moved further down slope over the rock drain constructed during Phase 1 and shall be placed in accordance with **Section 02200 (Earthworks)** of the Specifications to form the initial buttress fill with finished slopes varying from 1.5H:1V at the down valley toe to 2.5H:1V and 2H:1V in the upper part of the buttress as shown on Drawing 4. The Contractor will determine the type and size of equipment required to move the material based on the volume to be moved, the distance to be moved, the initial and finished design grades and the schedule specified in Section 1.3.

The Contractor is advised that the south side of the initial buttress fill will be adjacent to a natural scar slope that may be susceptible to isolated rock falls, primarily during storm and high run-off events. This potential hazard will be controlled during the work by ongoing monitoring of the condition of the slope in accordance with the Monitoring Program outlined in Section 1.6 and by restricting movement of personnel and equipment near the toe of the scar slope**.** At no time shall personnel or equipment be permitted to work parallel to the toe of the scar slope and at no point along the toe of the scar slope shall fill be pushed level all the way across to the toe of the scar slope (see Drawing 3). A 20 ft wide catch ditch will always be left between the placed fill and the toe of the scar and a safety berm of fill at least 6 ft high will always be left in place at the crest of the catch ditch.

At least seven instrumentation sites are expected to be functioning when the Phase 2 Works start at locations shown on Drawing 2 (SI-18, 23, 25, 26, 27, 28 and 29). The Contractor shall locate, protect and maintain access to these sites to allow ongoing monitoring to be carried out by others during Phase 2. Should the monitoring of the existing instrumentation indicate movements of the slide area that exceed the maximum allowable movement defined in Section 1.6 the Contractor shall stop work, move personnel and equipment off the Mitigation Site and follow the contingency plans outlined in Section 1.7 as directed by the Construction Manager.

The sequence of the Work requires Phases 1 and 2 to be complete before any operations can proceed on the north part of the Rock Pile. To ensure that this requirement is met the Contractor shall not allow any personnel or equipment to enter the ‘NO ACCESS ZONE’ shown on Drawing 4. The lower part of the slide remains a sensitive area during this phase of the Work and the Contractor shall only allow light vehicle access into the ‘LIMITED ACCESS ZONE’ shown on Drawing 4 unless approved otherwise by the Construction Manager. At no time will heavy equipment or concentrations of equipment be allowed in this zone unless approved by the Construction Manager.

On completion of Phase 2 the Contractor will construct and maintain access to the two additional monitoring sites shown on Drawing 4 (SI-30 and SI-31) for others to install and monitor additional instrumentation as Work proceeds into Phase 3.

1.5.6 Grading Operation in Previously Buttressed Rock Pile Area – Phase 3

Following completion of the initial buttress fill and installation of additional instrumentation in Phase 2, and following approval by the Construction Manager, the Contractor shall start cut and fill operations on the previously failed rock pile slopes as shown on Drawing 5. The upper part of the slide shall be excavated in accordance with **Section 02200 (Earthworks)** of the Specification. The Contractor will determine the type and size of equipment required to maintain a 1.5H:1V slope down to an elevation of 9665 ft., based on the volume to be moved, the distance to be moved, the initial and finished design grades and the schedule specified in Section 1.3.

The cut will be continuously monitored by the Construction Manager to ensure that all previously mobile Rock Pile material is removed from the back slope including any shear surfaces evident at the base of the slide. It is intended that original ground under the Rock Pile will be exposed at a 1.5H:1V slope down to a final cut bench elevation of 9665 ft.

The following dozer sequence is to be followed to allow for flexibility in the event that monitoring indicates increased rates of movement and contingency measures are required. The dozer will begin pushing downslope from south advancing north to approximately the midway point of the rehandle bench as indicated on Drawing 5 (Step 1 of Phase 3A). At this time, if rates of movement do not exceed the acceptable levels, the dozer push may continue for the remainder of the material on that lift by dozing downslope from north advancing south, again to the mid-point of the rehandle bench (Step 2 of Phase 3A). . This method will assist in keeping the pushed material within the slide area and reducing spillage. If following Step 1 of Phase 3A, the rates are found to exceed acceptable levels (see Section 1.6.4), the dozer will return to the southern extent of the next lift and confine the push to the south portion of the active slide area, thereby reducing the volume by half that is pushed at one time. See Section 1.7 for an outline of contingency plans for this phase of construction.

The cut material shall continue to be pushed by dozers directly down the north side of the Rock Pile and onto the upper part of the initial buttress fill leaving a regraded final slope of between 2H:1V and 3H:1V as shown on Drawing 5. This material shall then be moved further down slope and shall be placed in accordance with **Section 02200 (Earthworks)** of the Specifications to form the final buttress fill with finished slopes varying from 1.5H to 3H:1V. The Contractor will determine the type and size of equipment required to move the material based on the volume to be moved, the distance to be moved, initial and finished design grades and the schedule specified in Section 1.3.

The Contractor is advised that the south side of the buttress fill will be adjacent to a natural scar slope that may be susceptible to isolated rock falls, primarily during storm and high run-off events. This potential hazard will be controlled during the work by ongoing monitoring of the condition of the slope in accordance with the Monitoring Program outlined in Section 1.6 and by restricting movement of personnel and equipment near the toe of the scar slope**.** At no time shall personnel or equipment be permitted to work parallel to the toe of the scar slope and at no point along the toe of the scar slope shall fill be pushed level all the way across to the toe of the scar slope. A 20 ft wide catch ditch will always be left between the placed fill and the toe of the scar and a berm of fill at least 6 ft high will always be left in place at the crest of the catch ditch.

At least eight instrumentation sites are expected to be functioning when the Phase 3 Works start at locations shown on Drawing 5 (SI-23, 25, 26, 27, 28, 29, 30 and 31). The Contractor shall locate, protect and maintain access to these sites to allow ongoing monitoring to be carried out by others during Phase 3. Should the monitoring of the existing instrumentation indicate movements of the slide area that exceed the maximum allowable movement defined in Section 1.6 the Contractor shall stop work, move personnel and equipment off the Mitigation Site and follow the contingency plans outlined in Section 1.7 as directed by the Construction Manager.

1.5.7 Grading and Construction of Interim Surface Water Control Structures – Phase 4

Following completion of Phase 3, the Contractor will start grading operations for interim surface water control as shown on Drawing 6. The main features of the grading are drainage swales installed between the steep natural drainage chutes entering from the south and lateral interceptor road ditches that channel surface flow down the face of the Rock Pile and the buttress fill. The roads and associated ditches shall be cut into the slope in a cut and fill section and shall be lined with road base gravel to provide access to the slope for maintenance and interim monitoring of instrumentation. In addition to providing access, these roads also serve as sloping gradient terraces that transmit runoff water off the slope.

Road base gravel will be processed by the Contractor at Spring Gulch as outlined in Section 1.5.3 and defined **Section 02250 (Granular Materials)** in the Specifications. The Contractor will determine the type and size of equipment required to move and place the material based on the volume to be moved, the distance to be moved, the initial and finished design grades and the schedule specified in Section 1.3.

The Contractor is advised that the south side of the buttress fill will be adjacent to a natural scar slope that may be susceptible to isolated rock falls, primarily during storm and high run-off events. This potential hazard will be controlled during the work by ongoing monitoring of the condition of the slope in accordance with the Monitoring Program outlined in Section 1.6 and by restricting movement of personnel and equipment near the toe of the scar slope. At no time shall personnel or equipment be permitted to work parallel to the toe of the scar slope. During grading for interim surface water control local areas of fill shall be pushed level all the way across to the toe of the scar slope as shown on Drawings 6. At these points the 20 ft wide catch ditch and 6 ft high berm of fill will be removed. This operation will be carefully controlled by the Contractor and shall only be carried out in good weather conditions by dozers equipped with bush screens under the direction of the Construction Manager.

At least eight instrumentation sites are expected to be functioning during Phase 4 at locations shown on Drawing 6 (SI-23, 25, 26, 27, 28, 29, 30 and 31). The Contractor shall locate, protect and maintain access to these sites to allow replacement if necessary and permit ongoing monitoring to be carried out by others during Phase 4. Should the monitoring of the existing instrumentation indicate movements of the slide area that exceed the maximum allowable movement defined in Section 1.6 the Contractor shall stop work, move personnel and equipment off the Mitigation Site and follow the contingency plans outlined in Section 1.7 as directed by the Construction Manager.

On completion of Phase 4 the Contractor will construct access to an additional four monitoring sites shown on Drawing 6 (SI-32, 33, 34 and 35) for others to install and monitor instrumentation following completion of the Work.

1.6 MONITORING PROGRAM

1.6.1 General

The Contractor is advised that the nature of the mitigation shall require parts of the Work will be carried out on marginally stable ground that is currently creeping at rates of up to 3 inches per month. Rates of movement have been monitored effectively by several slope inclinometers since September 2003. It is important to note that there are no indications from the monitoring and from detailed stability analyses that there is any potential for more rapid or hazardous forms of displacement to occur. However, due diligence dictates that effective slope movement monitoring is maintained during all stages of the Works and that the results of the monitoring be used to protect personnel and equipment on the Mitigation Site by limiting access if necessary.

As the Works proceed, different areas of the Mitigation Site will become accessible for the installation of necessary instrumentation and the Contractor shall at all times give priority to providing and maintaining access to these sites for installation and monitoring by others.

1.6.2 Installations

Slope monitoring installations will typically require conventional rotary truck mounted drill and water truck access for installation and light truck access for ongoing monitoring. The completed installations will have a 3 to 4 inch ABS casing exposed above ground possibly with piezometer cables attached. The Contractor will be required to make every effort to protect these installations to avoid damage from site equipment as directed by the Construction Manager.

1.6.3 Construction Monitoring

Installation and reading of instrumentation shall be carried out under separate contract by others. It is anticipated that during construction Phases 1 to 4 readings from all instruments will be taken daily and that the updated monitoring data will be reviewed daily by the Contractor, Construction Manager and the Engineer prior to work commencing the next morning. The Construction Manager will be responsible for comparing daily monitoring data with the trigger levels specified in Section 1.6.4. If the Construction Manager determines that the acceptable levels are exceeded, the Contractor shall be responsible for moving personnel and equipment off the Mitigation Site and following contingency plans outlined in Section 1.7 as directed by the Construction Manager.

If at any other time during the course of the Work, the Engineer, Construction Manager, Contractor or Owner consider for any reason that acceptable rates of movement have been or may be exceeded, the Contractor will stop work and the Construction Manager will immediately arrange for additional readings to be taken from some or all of the instrumentation. If the Construction Manager determines that the acceptable levels are exceeded, the Contractor shall be responsible for moving personnel and equipment off the Mitigation Site and following contingency plans outlined in Section 1.7 as directed by the Construction Manager.

1.6.4 Construction Monitoring Trigger Criteria

The mitigation plan has been designed to increase stability of the previously failed material through the various phases of construction. However, there is a remote possibility that unforeseen circumstances may cause undesirable movements to occur at any time during the course of the mitigation work. The monitoring program has been developed to record displacements and piezometric pressures at strategic locations over the Mitigation Site and based on previous monitoring and expected improvements during construction the following triggering criteria have been determined. The trigger levels shall be subject to modification based on further monitoring data as dictated by the Construction Manager.

If at any time these criteria are exceeded, the Contractor will stop work, move personnel and equipment off the Mitigation Site and await instructions from the Construction Manager and follow the contingency procedures outlined in Section 1.7.

CONSTRUCTION MONITORING - PHASES 1,2,3 AND 4

Location		Trigger Levels		
		SI Movement	*Pore Pressure*	*Visual**
Outside Slide Area	Rock Pile or Colluvium SI-25,26,30	Development of a new shear plane** **OR** Displacement > 0.03" over 10ft depth	-	Scarp or tension crack development with vertical or horizontal displacement > 1ft
Inside Slide Area	Colluvial bench SI-27, 28, 29	Development of a new shear plane** **OR** Displacement rate on existing shear plane > 0.1"/day	Rise in groundwater at specified piezos > 5ft	Scarp or tension crack development with vertical or horizontal displacement > 3ft
	Toe area of slide SI-23, 31	Development of a new shear plane** **OR** Displacement rate on existing shear plane > 0.1" over 3 days	Rise in groundwater at specified piezos > 5ft	Scarp or tension crack development with vertical or horizontal displacement > 3ft

*Surface movements in in-situ or placed material i.e. not being actively moved by construction equipment.

** A well defined lateral displacement indicated on an inclinometer displacement vs depth plot

1.6.5 Rock Fall Monitoring

At all times during construction the natural scar slope south of the rock pile and slide, will be subject to visual inspection by the Engineer, Construction Manager and the Contractor at least daily and during and following rainfall events. If at any time unstable blocks or any hazardous conditions are identified, the Contractor will stop work within 75ft of the toe of the scar slope and remove personnel and equipment from this zone as directed by the Owner's representatives. At all times the Contractor will keep operations within 75 feet of the toe of the scar slope to a minimum and at no time shall personnel or equipment remain stationary or parked within this zone.

1.7 CONTINGENCY OPTIONS

1.7.1 General

All work carried out on the Mitigation Site will be subject to the monitoring program described in Section 1.6. Should the specified trigger criteria be exceeded at any time during the course of the Work or if the Construction Manager issues the necessary instructions, the Contractor will immediately remove all personnel and equipment from the Mitigation Site and follow the contingency plans outlined in this Section corresponding to the active phase of the work.

1.7.2 Phases 1 and 2

During these phases of the Work, operations are generally concentrated on stable areas of the Rock Pile. In the unlikely event that the trigger criteria specified in Section 1.6.4 are exceeded during these phases, the Engineer shall be required to review the design basis of the mitigation design. No further work will be carried out on the Mitigation Site until Engineer has completed a re-evaluation of the design. During the time that that the Contractor's operations are curtailed, the contractor will be paid in accordance with the Contract Stand-by rate, as approved by the Construction Manager.

1.7.3 Phases 3 and 4

In the event that the trigger criteria specified in Section 1.6.4 are exceeded during these phases, the Engineer shall first specify a period for confirmatory monitoring before recommending any significant change to the mitigation plan. Once the exceeded trigger levels have been confirmed, the Construction Manager shall instruct the Contractor to carry out alternate mitigative measures that may include the following:

1. Push only half the material downslope at a time.
2. Push material laterally across the slope towards the stable regraded area in order to unload the upper part of the active slide area.
3. Increase the overall volume of material placed at the toe of the slide.
4. Increase the rate of placement of material at the toe of the slide.

During the time that that the Contractor's operations are curtailed, the contractor will be paid in accordance with the Contract Stand-by rate, as approved by the Construction Manager. Should the required mitigation strategies result in significant inefficiencies to the Contractor's Work, new unit rates will be negotiated for those work items where the significant inefficiencies to the Contractor's Work occur, or the Contractor will be informed to proceed on a time and materials payment basis in accordance with the out of scope work rates in the Contract.

1.8 SITE ADMINISTRATION

1.8.1 Site Administration by the Construction Manager

The Construction Manager will carry out site administration duties as follows:

1 Record, transcribe and distribute minutes of formal construction progress meetings held during the course of the Contract.

2 Receive from Contractor, for review purposes, the following:
   - Copy of proposed construction schedules and updates.
   - Contract price breakdown and cash flow forecasts.
   - Payment submissions.

Respond to Contractor with respect to these items.

3 Receive Contractor contractual submittals required with respect to the Work, and deal with Contractor in matters arising from submittal review.

4 With respect to contemplated changes in the Work, issue the necessary documentation, receive the related quotations and information and authorize changes in the Work.

1.8.2 Site Administration by the Engineer

The Engineer will carry out site technical duties as follows:

1 Issue clarification of the Drawings and technical Specifications.

Respond to Contractor with respect to these items.

2 Carry out inspection of the Work at the Site and other locations as deemed necessary by the Engineer, and issue instructions to correct observed deficiencies.

1.8.3 Site Administration by the Contractor

1 Provide superintendence of the Work.

2 Provide a site representative who shall be authorized to attend meetings, submit construction schedules and cash flow forecasts, supervise layout of the Work, advise on changes to the Work, maintain project record documents, prepare payment submissions and receive instructions from the Construction Manager.

3 Record, transcribe and distribute minutes of safety meetings and workplace orientation meetings during the course of the Contract.

DOCUMENT 00200

SPECIFICATIONS

SPECIFICATIONS

TABLE OF CONTENTS

DIVISION 1 SPECIFICATIONS

GOATHILL NORTH MITIGATION

SECTION 01100

DEFINITIONS

PART 1: GENERAL

1. 1 SUMMARY:

A. This Section contains definitions and references applicable to the SPECIFICATIONS.

B. Definitions:

- "Bidder" The party (or parties) submitting a Proposal to perform the WORK.
- "Bonds" Includes performance and payment bonds and other instruments of security.
- "Completion" Means that all WORK has been fully completed, (except correction during the Period of Warranty).
- "Contract" The contract entered into by the **OWNER** through the **PROJECT MANAGER** (**OWNER'S** Representative) and the **CONTRACTOR** including, without limitation, all of the documents listed herein, and others, if any, listed in the Construction Services Agreement or in a subsequent Change Agreements signed by the **OWNER** through the **PROJECT MANAGER** (**OWNER'S** Representative) and the **CONTRACTOR**, which specifies the total Contract Price.
- "Construction Services Agreement or Contract Agreement or Agreement" The principal document of the Contract, signed by the **OWNER** through the **PROJECT MANAGER** (**OWNER'S** Representative) and the **CONTRACTOR**.
- "Contract Amendment" (Change Order) The document signed by the **CONTRACTOR** and the **OWNER** through the **PROJECT MANAGER** (**OWNER'S** Representative) to amend the Contract to provide for changed or extra work and, accordingly, an increase or decrease in the Contract Price.
- "Contract Documents" are defined as the Construction Services Agreement, any Addenda (which pertain to the Contract Documents), Cost Proposal Worksheets which constitutes **CONTRACTOR'S** Bid (including documentation accompanying the Bid and any post-Bid documentation submitted prior to the Notice of Award) when attached as an exhibit to the Construction Services Agreement, the Bonds, the General Conditions, the SPECIFICATIONS, and the DRAWINGS, together with all Modifications issued after the execution of the Construction Services Agreement.
- "Contract Price" The total amount of the charges for the WORK ("estimated" or "fixed lump sum") stipulated in the Construction Services Agreement subject to such additions or deductions as may be made under the terms and conditions of the Contract.
- "Contract Unit Prices" The fixed unit prices or rates established by the Proposal which, initially, are applied to estimated measurements of volume, time, or other

units of performance to establish an estimated Contract Price, and, which ultimately, are applied to actual measurements to establish a final Contract Price.

- "**CONTRACTOR**" is defined as the party that has executed a Construction Services Agreement for the specified WORK with **OWNER**.

- "DRAWINGS" is defined as the drawings in conjunction with these SPECIFICATIONS titled, Questa Mine Goathill North Mitigation

- "**ENGINEER**" is defined as Norwest Corporation ("Engineer of Record"), or their representative, and is a representative appointed and authorized by the **OWNER**. The **ENGINEER** of Record shall be a registered Professional Engineer in the State of New Mexico, or a designated site representative under their supervision during construction.

- "**CONSTRUCTION MANAGER**" is defined as the authorized site representative of the **ENGINEER** responsible for the construction management. The individual authorized by **OWNER** is the site representative responsible for administering the Contract.

- "Equal To, or Equal" Means equal in all respects to the specified product and accepted or reviewed for use in the WORK by the **OWNER**, in writing.

- "Final Acceptance" The written Final Acceptance of the WORK issued by the **OWNER** through the **PROJECT MANAGER** (**OWNER'S** Representative) following final inspection, and 100 percent completion of the WORK.

- "Notice" Notices are to be defined as written notice.

- "**OWNER**" Molycorp, Inc., a Delaware corporation with offices in Questa, New Mexico.

- "Products" are defined as new material, machines, components, equipment, fixtures, and systems forming the WORK. This does not include machinery and equipment used for preparation, fabrication, conveying, and erection of the WORK. Products may also include existing material or components required for reuse.

- "Project" is defined as the Questa Mine Goathill North Mitigation, Questa Mine, New Mexico.

- "**PROJECT MANAGER**" is defined as the designee(s) or an authorized representative of Molycorp, Inc. ("**OWNER**") responsible for the project management. The individual designated by **OWNER** is the only person who may execute the Contract and subsequent Construction Services Agreement.

- "Proposal" (or "Bid") the written offer setting forth the price(s) to perform the WORK, as submitted by the Bidder to the **OWNER** through the **PROJECT MANAGER** (**OWNER'S** Representative).

SECTION 01100

DEFINITIONS

- "Quality Assurance Team" is defined as the individuals working under the direction of the **CONSTRUCTION MANAGER** to perform on site quality assurance tasks for **OWNER**.

- "Record Documents" are defined as the documents prepared and certified by a Registered Land SURVEYOR in New Mexico documenting the progress, location, type, and quantity of materials placed to complete the WORK.

- "Revisions" are defined as changes made to the SPECIFICATIONS or the DRAWINGS that are approved by the **PROJECT MANAGER** and the **ENGINEER** in writing after the SPECIFICATIONS on the DRAWINGS have been finalized.

- "Site" The lands of the **OWNER** under, in, or through which the WORK is to be executed.

- "Mitigation Site" The lands of the **OWNER** at which the WORK is to be executed that the **CONTRACTOR** shall remove all personnel and equipment from in the event that monitoring by the **CONSTRUCTION MANAGER** requires contingency plans be followed.

- "SPECIFICATIONS" is defined as this document of technical specifications prepared for **OWNER**.

- "Subcontractor" The party which, with approval of the **OWNER** through the **PROJECT MANAGER** (**OWNER'S** Representative), has executed a subcontract with the **CONTRACTOR** for the performance of a part of the WORK.

- "Substantial Completion" Means the same as and adopts the definition of "Substantial Completion" or "Substantial Performance" contained in the lien legislation in effect in the State in which the WORK is to be performed, and in the event no legislative definition exists for the expression "Substantial Completion" or "Substantial Performance" in the said State, Substantial Completion means that the WORK has been essentially completed, sufficient to permit beneficial use by the **OWNER** for its intended purpose, and that only items of WORK which cannot be completed due to conditions outside the **CONTRACTOR'S** control remain to be done.

- "Supplier" Any party, which with approval of the **PROJECT MANAGER** and **OWNER**, has executed a contract with the **CONTRACTOR** or any Subcontractor to supply materials or equipment in performance of a part of the WORK and includes, but is not limited to, a material man.

- "SURVEYOR" a New Mexico Professional Land Surveyor under the direction of the **OWNER** or **CONSTRUCTION MANAGER** to set control points for the **CONTRACTOR** and to determine pay quantities for the WORK

- "WORK" The work to be performed as specified in the Construction Services Agreement and referred to in the Contract Documents all inclusively as "the WORK."

- All slopes are described in terms of horizontal distance to vertical distance.

C. References

References to known standard specifications, including American Society of Testing Materials (ASTM), American National Standards Institute (ANSI), and Federal Test Method Standards (FTMS), shall mean and intend latest edition of such standards/specifications adopted and published at date of receipt of bids. All materials, fabrication, erection and related work required for this project shall comply with these standards/specifications which form part of these SPECIFICATIONS as applicable, the same as if fully set forth herein.

*** END OF SECTION ***

SECTION 01300

SUBMITTALS

PART 1: GENERAL

1.1 DESCRIPTION:

A. The **CONTRACTOR** shall be responsible for delivering all submittals to the **CONSTRUCTION MANAGER**; checking submittals prior to submission to the **CONSTRUCTION MANAGER** for their review; verification of field measurements, field construction criteria, catalogue numbers and similar data; and ensuring each item submitted clearly shows the Project Name and the Contract Number and Title.

B. Responsibility for errors and omissions in submittals is not relieved by the **CONSTRUCTION MANAGER'S** review of submittals.

C. The **CONTRACTOR** shall submit sufficiently early to provide adequate time for reviews, possible corrections, and resubmittals, placing orders, securing delivery and to avoid construction delays.

D. The **CONTRACTOR** shall accompany each submittal with a letter of transmittal containing all pertinent information required for identification and review of submittals. When submittals are resubmitted for any reason, transmit each resubmittal under a new letter of transmittal.

E. Do not perform any part of the WORK until the submittals for it have been reviewed by the **CONSTRUCTION MANAGER**.

1.2 SAMPLES:

A. Before delivery of materials to the Site, the **CONTRACTOR** shall submit samples of materials as required by sections of the SPECIFICATIONS and as requested by the **CONSTRUCTION MANAGER**. Label samples as to origin and intended use in the WORK and in accordance with the requirements of sections of the SPECIFICATIONS. The **CONTRACTOR** shall ensure samples represent physical examples to illustrate materials, equipment, or workmanship and to establish criteria by which completed WORK is judged.

B. The **CONTRACTOR** shall ensure samples are of sufficient size and quantity, if not otherwise specified, to illustrate the quality and functional characteristics of product or material with integrally related parts and attachment devices, and color.

1.3 SHOP DRAWINGS:

A. The term "shop drawings" means drawings, diagrams, and other data that are provided by the **CONTRACTOR** to illustrate details of portions of the WORK.

B. Prepare shop drawings to include structural details and mark numbers that the **CONSTRUCTION MANAGER** considers necessary to show details of the WORK to be performed. Clearly identify each shop drawings by title and number of the CONTRACT, and reference to applicable Contract DRAWINGS.

C. The **CONTRACTOR** shall submit, in time to suit the Contract Schedule or as otherwise stated in the Agreement, not less than three copies of shop drawings to the **CONSTRUCTION MANAGER** for **ENGINEER** review. One of the copies will be returned by the **CONSTRUCTION MANAGER**, stamped to indicate that the shop drawings has been reviewed and comments added where applicable. If shop drawings are illegible, obscure, or incomplete, they will be returned by the **CONSTRUCTION MANAGER** marked "not reviewed". Redraw and resubmit shop drawings.

D. The **CONTRACTOR** shall make corrections in shop drawings that the **CONSTRUCTION MANAGER** may require consistent with the CONTRACT, and resubmit as before.

When the **CONSTRUCTION MANAGER** review is complete and requested corrections made, provide three copies of certified DRAWINGS incorporating requested corrections, for the use of and distribution by the **CONSTRUCTION MANAGER**. Ensure WORK and units supplied conform to final DRAWINGS which must have the following notation:

Certified for Construction

Signature: ____

Date: ____

E. The **CONSTRUCTION MANAGER'S** review of shop drawings is for the sole purpose of ascertaining conformance with the general arrangement, but no approval is given or responsibility assumed by the **ENGINEER** for the detail design inherent in the shop drawings or for corrections of dimensions or details or conformity to SPECIFICATIONS, which remain the responsibility of the **CONTRACTOR** submitting same.

1.4 PRODUCT DATA:

A. The term "product data" means schematic drawings, catalogue sheets, diagrams, illustrations, brochures, manufacturer's instructions, and other data provided by the manufacturer to illustrate details of a portion of the WORK.

B. The **CONTRACTOR** shall modify schematic drawings if and as necessary to ensure they show all and only the information applicable to the WORK.

C. On catalogue sheets, diagrams, illustrations, brochures and other data, clearly mark each copy to identify materials, products, or models applicable to the WORK. Show dimensions, clearances, performance characteristics, capacities, and controls applicable to the WORK.

D. Product data and manufacturer's instructions only apply to particular requirement relative to the manufacturer's products and are in addition to the SPECIFICATIONS. Do not interpret or apply such instruction to limit the WORK or responsibilities. The Contract

DOCUMENTS take precedence in case of conflict. Inform the **CONSTRUCTION MANAGER** promptly in writing in the event of such conflict.

1.5 WORK PROGRESS SCHEDULE:

A. Scope: The WORK specified in this subsection includes planning, scheduling, and reporting that is required to be performed by the **CONTRACTOR**.

B. Method: A critical path or bar graph type schedule, fully man-loaded and prepared per each Contract item, shall be submitted after the Proposal but prior to the execution of the CONTRACT. Upon **OWNER** review comments, the critical path schedule will be resubmitted to **OWNER** within seven (7) calendar days after the effective date of the Agreement.

C. Schedule Requirements:

1. Distinct items of contract WORK shall be defined and separated on the schedule. As a minimum, the WORK items shall include each contract pay item, mobilization, demobilization, and cleanup. Pay items that are partially subcontracted shall be split up to distinctly show the Subcontracted WORK. These items of WORK shall be plotted on a graph with calendar days duration as a horizontal reference. Anticipated start and finish dates for each WORK stage and for each of the WORK items within a stage, shall be shown.

2. The project name, the **CONTRACTOR'S** name, and the date of the schedule submittal shall be clearly shown on the submittal.

D. Progress Reports:

1. At the end of each week, the **CONTRACTOR** shall submit a summary report of the progress of the various scheduled WORK items stating, for each item, the existing time status, estimated time of completion, and cause of delays, if any. If the WORK is behind the previously submitted schedule, the **CONTRACTOR** shall submit an updated schedule and a written plan acceptable to the **OWNER** for bringing the WORK up to schedule.

2. Updated schedules will be used by the **OWNER** in compiling partial payments and no such computations will be made until the reports have been received and approved by the **OWNER**.

3. The **OWNER** may request reports to be made on a more frequent schedule if he considers the substantial completion date to be in jeopardy because of activities behind schedule or for other valid reasons.

1.6 WARRANTIES:

A. The **CONTRACTOR** shall submit warranties showing the project name and the contract number and title, warranty commencement data and duration of warranty. Clearly indicate what the warranty covers and what remedial action shall be taken under the warranty. The **CONTRACTOR** shall ensure warranty bears the signature and seal of the **CONTRACTOR**.

*** END OF SECTION ***

SECTION 01400

QUALITY CONTROL/QUALITY ASSURANCE

PART 1: GENERAL

1.1 SUMMARY:

The intent of this Section is to define the requirements of the Project Construction Quality Assurance (CQA) Program and the Construction Quality Control (CQC) documentation required by the **CONTRACTOR**. The **CONSTRUCTION MANAGER** will be responsible for all CQA and testing as documented in these SPECIFICATIONS and in Attachment 1 (CQAP) provided at the end of the SPECIFICATION Section, and will compile a construction certification report at the completion of the WORK. **CONTRACTOR** is required to complete all WORK and CQC in accordance with the Project requirements. Prior to approval of WORK, the **PROJECT MANAGER** will coordinate with **CONSTRUCTION MANAGER** to ensure that the WORK has been completed in accordance with the WORK requirements.

1.2 ASSURANCE TESTING AND FREQUENCY:

CQA tests and frequency are discussed throughout the SPECIFICATIONS and Attachment 1. The frequencies indicated are minimums only, and do not include retests of failed materials. Those quality assurance tests and testing frequencies to be conducted in the field by the **PROJECT MANAGER, CONSTRUCTION MANAGER,** or the CQA Team are included in Attachment 1 at the end of the SPECIFICATION Section.

*** END OF SECTION ***

SECTION 01500

ENVIRONMENTAL PROTECTION

PART 1: GENERAL

1.1 SCOPE OF WORK:

A. The WORK covered by this Section consists of furnishing all labor, materials and equipment and performing all WORK required for the prevention of environmental pollution in conformance with applicable laws and regulations, during and as the result of construction operations under this Contract. For the purpose of this SPECIFICATION, environmental pollution is defined as the presence of chemical, physical, or biological elements or agents that adversely affect human health or welfare; unfavorably alter ecological balances of importance to human life; affect other species of importance to man; or degrade the utility of the environment for aesthetic.

B. The control of environmental pollution requires consideration of air, water, and land, and involves management of noise and solid waste, as well as other pollutants.

C. The **CONTRACTOR** shall schedule and control all WORK in a manner that will minimize the erosion of soils in the area of the WORK. The **CONTRACTOR** will provide erosion control measures such as diversion channels, sedimentation systems, berms, staked straw bales, or other special surface treatments as are required to prevent run-on and runoff from the construction area as well as silting and muddying of streams, rivers, etc. Erosion control measures shall be installed prior to commencement of construction activities and shall be maintained throughout the construction period or as dictated by the **CONSTRUCTION MANAGER**. All erosion control measures shall be in place in an area prior to any construction activity in that area.

D. All phases of sedimentation and erosion control shall comply with and may be subject to the approval of the New Mexico Department of Environment.

E. Perform dust control operations, in an approved manner, whenever necessary or when directed by the **CONSTRUCTION MANAGER**, even though other WORK on the project may be suspended. Dust control shall be generally accomplished by the use of water; however, the use of calcium chloride may be used when necessary, as approved by the **CONSTRUCTION MANAGER**, to control dust nuisance. Since utilities are not available at the site, **CONTRACTOR** will use suitable means for obtaining and dispersing water for dust control activities.

F. These SPECIFICATIONS are intended to ensure that construction is achieved with a minimum of disturbance to the existing ecological balance between a water resource and its surroundings. These are general guidelines. It is the **CONTRACTOR'S** responsibility to determine the specific construction techniques to meet these guidelines.

1.2 RELATED SECTIONS:

Section 01300 – Submittals

Section 02110 – Clearing

Section 02200 - Earthworks

1.3 APPLICABLE REGULATIONS:

Comply with all applicable Federal, state and local laws and regulations concerning environmental pollution control.

1.4 SUBMITTALS:

This Section Intentionally Omitted.

1.5 NOTIFICATION:

The **CONSTRUCTION MANAGER** will notify the **CONTRACTOR** in writing of any non-compliance with the foregoing provisions or of any environmentally objectionable acts and corrective action to be taken. If applicable, State or local agencies responsible for verification of certain aspects of the environmental protection requirements shall notify the **CONTRACTOR** in writing, through the **OWNER**, of any non-compliance with State or local requirements. The **CONTRACTOR** shall, after receipt of such notice from the **OWNER** or, as applicable, from the regulatory agency through the **OWNER**, immediately take corrective action. Such notice, when delivered to the **CONTRACTOR** or his/her authorized representative at the site of the WORK, shall be deemed sufficient for the purpose. If the **CONTRACTOR** fails or refuses to comply promptly, the **OWNER**, in consultation with the **CONSTRUCTION MANAGER**, may issue an order stopping all or part of the WORK until satisfactory corrective action has been taken. No part of the time lost due to any such stop orders shall be made the subject of a claim for extension of time or for excess costs or damages by the **CONTRACTOR** unless it is later determined that the **CONTRACTOR** was in compliance.

1.6 IMPLEMENTATION:

A. Prior to commencement of the WORK, meet with the **CONSTRUCTION MANAGER** or **OWNER** to develop mutual understandings relative to compliance with this provision and administration of the environmental pollution control program.

B. Remove temporary environmental control features, when approved by the **OWNER** and incorporate permanent control features into the project at the earliest practicable time.

PART 2: PRODUCTS

2.1 DUST CONTROL:

Dust Control is expected to require a water truck using a water source approved by **OWNER.**

PART 3: EXECUTION

3.1 EROSION CONTROL:

Provide positive means of erosion control such as shallow ditches around construction to carry off surface water. Erosion control measures, such as siltation basins, straw check dams, mulching, jute netting, silt fences, and other equivalent techniques, shall be used as appropriate or as shown on the DRAWINGS. **CONTRACTOR** shall use reasonable care to divert surface water run-on around construction areas. Flow of surface water into excavated, graded and cover areas shall be prevented to the extent practicable. Ditches as shown on the DRAWINGS or others deemed necessary by the **CONTRACTOR** around construction area shall also be used to carry away water resulting from dewatering of excavated areas. At the completion of the WORK, ditches shall be backfilled and the ground surface restored to original condition. **CONTRACTOR** must file Notice of Intent (NOI) for a New Mexico Stormwater Discharge Permit for Construction Activities with the proper authorities and include a copy of the NOI in the **CONTRACTOR'S** Stormwater Control Plan.

3.2 PROTECTION OF STREAMS:

A. Care shall be taken to prevent, or reduce to a minimum, any damage to any stream from pollution by debris, sediment, or other material, or from the manipulation of equipment and/or materials in or near such streams.

B. All preventative measures shall be taken to avoid spillage of petroleum products and other pollutants. In the event of any spillage, prompt remedial action shall be taken in accordance with a contingency action plan approved by the **OWNER**. **CONTRACTOR** shall submit two copies of approved contingency plans to the **CONSTRUCTION MANAGER**, prior to commencing with field activities.

3.3 PROTECTION OF LAND RESOURCES:

A. Land resources within the project boundaries and outside the limits of permanent WORK shall be restored to a condition, after completion of construction that will appear to be natural and not detract from the appearance of the project. Confine all construction activities to areas shown on the DRAWINGS.

B. Outside of areas requiring earthwork, the **CONTRACTOR** shall not deface, injure, or destroy trees or shrubs, nor remove or cut them without prior written approval. No ropes, cables, or guys shall be fastened to or attached to any existing nearby trees for anchorage unless specifically authorized by the **CONSTRUCTION MANAGER**. Where such special emergency use is permitted, first wrap the trunk with a sufficient thickness of burlap or rags over which softwood cleats shall be tied before any rope, cable, or wire is placed. The **CONTRACTOR** shall in any event be responsible for any damage resulting from such use.

C. Where trees may possibly be defaced, bruised, injured, or otherwise damaged by the **CONTRACTOR'S** equipment, dumping or other operations, protect such trees by placing boards, planks, or poles around them. Monuments and markers shall be protected similarly before beginning operations near them.

D. If the **CONTRACTOR** proposes to construct temporary roads or embankments and excavations for WORK areas not shown on the DRAWINGS, they shall submit the following items for approval by the **CONSTRUCTION MANAGER** at least 5 days prior to scheduled start of such temporary WORK:

1. A layout of all temporary roads, excavations, and embankments to be constructed within the WORK area and a plan for restoring these areas.

2. Details of temporary road construction.

3. Removal of any trees and shrubs over 1 inch in diameter outside of the limits of the WORK areas shall be requested in writing and indicated on a drawing prepared by the **CONTRACTOR**. The drawing shall provide for the obliteration of construction scars as such and shall provide for a natural appearing final condition of the area. Modification of the **CONTRACTOR'S** approved drawing shall be made only with the written approval of the **CONSTRUCTION MANAGER**. No unauthorized road construction, excavation, or embankment construction including disposal areas will be permitted.

E. Remove all signs of temporary construction facilities such as haul roads, WORK areas, structures, foundations of temporary structures, stockpiles of excess of waste materials, or any other vestiges of construction as directed by the **CONSTRUCTION MANAGER**.

3.4 PROTECTION OF AIR QUALITY:

A. The use of burning at the project site for the disposal of refuse and debris will not be permitted.

B. Dust Control:

1. The **CONTRACTOR** will be required to maintain all excavations, grading, embankment, stockpiles, access roads, borrow areas, and all other WORK areas within or without the project boundaries free from dust that could cause the standards for air pollution to be exceeded and that would cause a hazard or nuisance to others.

2. Methods of controlling dust shall meet all air pollutant standards as set forth by Federal and State regulatory agencies.

3. An approved method of stabilization consisting of sprinkling or other similar methods will be permitted to control dust. The use of petroleum products is prohibited. The use of chlorides may be permitted with approval from the **CONSTRUCTION MANAGER**.

4. Sprinkling, to be approved, must be repeated at such intervals as to keep all parts of the disturbed area and all unpaved haul roads at least damp at all times, and the **CONTRACTOR** must have sufficient competent equipment

on the job to accomplish this if sprinkling is used. Dust control shall be performed as the WORK proceeds and whenever a dust nuisance or hazard occurs, as determined by the **CONSTRUCTION MANAGER**.

3.5 MAINTENANCE OF POLLUTION CONTROL FACILITIES DURING CONSTRUCTION:

During the life of this Contract, maintain all facilities constructed for pollution control as long as the operations creating the particular pollutant are being carried out or until the material concerned has become stabilized to the extent that pollution is no longer being created.

3.6 NOISE CONTROL:

The **CONTRACTOR** shall make every effort to minimize noises caused by his/her operations. Equipment shall be equipped with silencers or mufflers designed to operate with the least possible noise in compliance with State and Federal regulations.

3.7 FIRE PRECAUTIONS:

A. Smoking and Lunch Fires:

1. Smoking is prohibited except inside a building, vehicle, or while seated in an area of at least 5 feet in diameter than is barren or cleared of all flammable materials.

2. The building of camp, lunch, warming, and other fires within the construction area and vicinity is prohibited.

B. Spark Arrester and Mufflers:

1. Operating or using any internal combustion engine, on any timber, brush, or grass covered land, including trails and roads traversing such land, without a spark arrester, maintained in effective working order, meeting either (I) Department of Agriculture, Forest Service standard 5100, "SPARK ARRESTERS FOR INTERNAL COMBUSTION ENGINES," (current edition); or (II) the Society of Automotive **ENGINEERS** (SAE) recommended Practices J335, "MULTIPOSITION SMALL ENGINE EXHAUST SYSTEM FIRE IGNITION SUPPRESSION," (current revision) and J350, 36 CFR 261.52(j), is prohibited.

2. Passenger carrying vehicles, pickups, medium and large highway trucks (80,000 GVW) will be equipped with a factory designed muffler system that is specified for the make and model of the respective vehicle/truck or with a muffler system that is equivalent or that exceeds factory SPECIFICATIONS.

3. Exhaust systems shall be properly installed and continually maintained in serviceable condition.

C. Fire Extinguishers and Tools on Equipment:

SECTION 01500

ENVIRONMENTAL PROTECTION

1. While in use, each internal combustion engine including tractors, trucks, yarders, loaders, welders, generators, stationary engines, or comparable powered equipment shall be provided with at least the following:

   a. One fire extinguisher, at least 5#ABC with an Underwriters Laboratory (UL) rating of 3A - 40 BC, or greater.

   b. One shovel, sharp, size O or larger, round-pointed with an overall length of at least 48 inches.

   c. One ax, sharp, double bit 3-1/2#, or one sharp pulaski.

2. Extinguishers, shovels, axes, and pulaskis shall be mounted so as to be readily available from the ground. All tools shall be maintained in a serviceable condition.

*** END OF SECTION ***

SECTION 01600

CONTRACT CLOSEOUT

PART 1: GENERAL

1.1 DESCRIPTION:

PROJECT MANAGER shall prepare punch list when notified by **CONTRACTOR** that work is completed. **PROJECT MANAGER** and **CONSTRUCTION MANAGER** will conduct one final inspection only. (Note: Failure of **PROJECT MANAGER** to include any items on punch list does not alter responsibility of **CONTRACTOR** to complete THE WORK in accord with Contract Documents.) Deliver all items called for herein and under various SPECIFICATION sections, and other Contract Documents requirements, to **OWNER** at completion of work.

*** END OF SECTION ***

SECTION 01725

SURVEYING

PART 1: GENERAL

1.1 DESCRIPTION

Requirements for surveying services consists of furnishing all services, labor, equipment, transportation and supervision necessary to provide all Surveying Services required to construct the project.

1.2 RESPONSIBILITIES

A. The **OWNER** is responsible for the following:

1. The **OWNER** will employ a SURVEYOR to provide surveying services to determine pay quantities. The SURVEYOR shall conduct a monthly survey and calculate the quantities completed for each item of work.

2. The SURVEYOR will provide a minimum of three (3) control points and triangulate between the three control points to verify accuracy prior to the **CONTRACTOR** using the points for control of the WORK.

3. The SURVEYOR shall establish the boundary of the WORK areas.

4. The SURVEYOR shall provide topographic survey on the top of the regraded mine waste rock pile to generate digital grid information for use in material quantity determination.

B. The **CONTRACTOR** is responsible for the following:

1. The **CONTRACTOR** shall provide all construction surveying necessary to maintain slopes, specified minimum thickness of layers and grades for control of the WORK.

2. The **CONTRACTOR** shall preserve above-mentioned **OWNER** control and layout points.

3. If in the opinion of the **OWNER** or **CONSTRUCTION MANAGER** surveying control to layout points have been carelessly or willfully disturbed or destroyed by the **CONTRACTOR** or their employees, the cost of replacing such control points shall be incurred by the **CONTRACTOR**.

4. The **CONTRACTOR** may, at their own expense, provide additional surveys conducted by a licensed land surveyor in the State of New Mexico to verify quantities or grades.

1.3 SPECIFIC SURVEY REQUIREMENTS

A. All surveys shall include X, Y, and Z coordinates. Base each survey on the same ground control. Survey cross sections of temporary and permanent drainage channels and haul or access roads at minimum 50-foot intervals. Road cross sections shall include a complete profile of the drainage ditch and road (including top and bottom of ditches, top of berm, and crest and toe of haul and access roads).

B. The **CONTRACTOR** shall be responsible for ensuring that accurate surveys, by the **OWNER'S** SURVEYOR, are obtained for the record (as-built) locations and elevations, and where applicable, shape of grading areas prior to material placement, elevation of subgrade, channels, roads, and any other aspect of the work required by the Contract Documents.

C. The **CONSTRUCTION MANAGER** may require surveys to document critical construction components. The **OWNER**, in accordance with the Contract Documents, will coordinate these survey requirements.

PART 2: PRODUCTS

NOT USED

PART 3: EXECUTION

A. Standards for Control and Accuracy: The control survey and survey monuments shall meet or exceed the following standard deviations:

Horizontal measurements Project Control Points - Standard deviation of 0.1 ft or Third order Class II as defined by Classification and Standards of Accuracy of Geodetic Control Surveys

Vertical measurements Project Control Points – Standard deviation of 0.1 ft or Third order defined by Classification and Standards of Accuracy of Geodetic Control Surveys

B. Equipment and Use

Choice of equipment to be used for surveys is determined strictly by the responsible **CONTRACTOR** or **OWNER'S** SURVEYOR, with a demonstration provided to indicate that the equipment is capable of meeting or exceeding the specified accuracies

*** END OF SECTION ***

DIVISION 2 SPECIFICATIONS

GOATHILL NORTH MITIGATION

SECTION 02110

CLEARING

PART 1: GENERAL

1.1 SUMMARY:

A. This WORK includes all cutting and disposing of brush and vegetation and installation of temporary surface water and erosion controls within the Goathill North Mitigation Site, rock pile, slide area and access roads.

B. Refer to the following Sections for related work:

Section 01300 – Submittals

Section 01400 - Quality Assurance and Construction Documentation

Section 02200 - Earthworks

1.2 QUALITY ASSURANCE:

A. **CONSTRUCTION MANAGER** shall at all times have access to the work during its construction and shall be furnished with every reasonable facility for ascertaining that the materials and workmanship are in accordance with the DRAWINGS and these SPECIFICATIONS.

B. All site preparation operations shall be carried out under the observation of the **CONSTRUCTION MANAGER** or **PROJECT MANAGER**. Testing shall be performed by **CONSTRUCTION MANAGER** in accordance with Attachment 1 (CQAP) provided at the end of the SPECIFICATION Section.

C. Any work found unsatisfactory or any work disturbed by subsequent operations before acceptance is granted shall be corrected by **CONTRACTOR** as directed by **CONSTRUCTION MANAGER**.

PART 2: EXECUTION

2.1 CLEARING:

A. As required, clearing shall be performed in designated areas within the footprint of the regraded area, roads, channels or other components of the WORK where mine waste rock or other ground will be impacted as delineated on the DRAWINGS. As required, clearing shall extend a maximum of fifteen (15) feet and a minimum of five (5) feet outside of the construction limits. Areas for clearing shall be released to the **CONTRACTOR** by the **PROJECT MANAGER**. No pioneering of roads across undisturbed areas shall be allowed without prior written approval of the **CONSTRUCTION MANAGER**.

No clearing shall be performed until written permission is given by the **CONSTRUCTION MANAGER**. Clearing shall consist of cutting trees and brush to the ground level leaving roots in place, removing such material, along with wood, rubbish, and any other vegetation, and disposing of all such material in the accepted manner described below.

B. The **CONTRACTOR** shall clear all vegetative matter, rubbish and other deleterious materials from the delineated areas.

Cleared vegetation shall be removed and disposed of in stockpiles, by controlled burning, or wasted by way of other approved methods in an area designated by the **PROJECT MANAGER** in accordance with permits obtained from the appropriate local, State, and Federal regulatory agencies.

2.2 TEMPORARY EROSION AND SURFACE WATER CONTROLS:

A. The **CONTRACTOR** shall be responsible for providing temporary erosion and surface water controls during construction and shall be responsible for, and shall repair at their own expense, any damage to the structures or other parts of the WORK caused by stormwater run-on or runoff, or failure of any temporary and permanent (as shown on the DRAWINGS) erosion or surface water controls.

B. The **CONTRACTOR** shall be responsible for providing temporary erosion and surface water controls during construction. All temporary surface water controls not part of the permanent facility shall be removed, leveled, and graded. Disturbance of areas beyond the clearing limits shall not be undertaken without prior written approval by **CONSTRUCTION MANAGER.**

C. The **CONTRACTOR** shall have full responsibility for the adequacy of the temporary erosion and surface water controls. The sizing for temporary erosion and surface water controls should consider the duration of the construction activities, the time of the year of construction, characteristics of the storms during the construction seasons, cost of possible damage, cost of delay to the construction completion of the WORK, and the safety of workmen. Historic rainfall data for the Questa Mine Site for various return periods will be made available to **CONTRACTOR** by **PROJECT MANAGER**, upon request. **CONSTRUCTION MANAGER** and **OWNER** assume no responsibility for any interpretations or conclusions made by the **CONTRACTOR** from the supplied data.

*** END OF SECTION ***

SECTION 02200

EARTHWORKS

PART 1: GENERAL

1.1 SUMMARY:

A. This WORK includes all the earthwork activities required to regrade the rock pile and place buttress material. This WORK includes, but is not limited to excavation, dozing, haulage and placement of mine waste rock, and construction of access roads and channels.

B. Refer to the following Sections for related work:

Section 01300 – Submittals

Section 01400 - Quality Assurance and Construction Documentation

Section 02110 - Clearing

1.2 QUALITY ASSURANCE AND QUALITY CONTROL:

A. **CONSTRUCTION MANAGER** shall at all times have access to the work during its construction and shall be furnished with every reasonable facility for ascertaining that the materials and workmanship are in accordance with the DRAWINGS and these SPECIFICATIONS.

B. All excavation, backfill and grading operations shall be carried out under the observation of **CONSTRUCTION MANAGER**. Testing shall be performed by **CONSTRUCTION MANAGER** in accordance with Attachment 1 (CQAP) provided at the end of the SPECIFICATION Section.

C. Any work found unsatisfactory or any work disturbed by subsequent operations before acceptance is granted, shall be corrected by **CONTRACTOR** as directed by **CONSTRUCTION MANAGER**.

1.3 REFERENCES

American Society for Testing and Materials (ASTM) most current version:

A. ASTM D698 - Test Method for Laboratory Compaction Characteristics of Soil Using Standard Effort (12,400 ft-lbf/ft3)

PART 2: PRODUCTS

2.1 MATERIALS:

A. General Fill: Shall be any material excavated from within the mine waste rock area, the mitigation site area or associated access road areas approved by the **CONSTRUCTION MANAGER** and regraded to form the toe buttress, toe berm, roads or channels.

SECTION 02200

EARTHWORKS

PART 3: EXECUTION

3.1 EXCAVATION:

A. Unless specifically noted otherwise, all excavations shall be performed to the lines and grades shown on the DRAWINGS, or to approved field fit modifications made thereto, as approved by the **CONSTRUCTION MANAGER**. Any excavation beyond these limits shall be at the expense of the **CONTRACTOR**, unless approved otherwise by the **CONSTRUCTION MANAGER.** No excavation shall begin until the SURVEYOR has provided existing condition staking for the proposed work. The exposed subgrade shall be inspected and approved by the **CONSTRUCTION MANAGER** prior to any fill placed.

B. Excavations shall be graded and properly maintained to provide adequate drainage at all times. WORK shall be suspended by the **CONTRACTOR** when, in the opinion of the **CONSTRUCTION MANAGER**, the site is overly wet, muddy, or otherwise unsuitable for proper maintenance.

C. All necessary precautions shall be taken to preserve the material below and beyond the lines of excavation in the soundest possible condition. Where required to complete the WORK, all excess excavation or over-excavation shall be refilled with approved materials, placed, and compacted to the satisfaction of the **CONSTRUCTION MANAGER**.

D. Safe temporary construction slopes are the responsibility of the **CONTRACTOR**.

E. The **CONTRACTOR** shall inspect all temporary and permanent excavations on a regular basis for signs of instability. Should signs of instability be noted, the **CONTRACTOR** shall undertake remedial measures immediately and shall notify the **CONSTRUCTION MANAGER** as soon as possible.

F. It will be the **CONTRACTOR'S** responsibility to remove all loose materials from the excavated slopes and to maintain the slopes in a safe and stable condition at all times during the progress of the WORK.

B. G. Construct surface water control channels and erosion control features with uniform gradients between approved control points for the approved channel alignment, without excessive sags and without humps, unless specifically approved otherwise by the **CONSTRUCTION MANAGER**. Cross sectional flow areas for the channels shown on the DRAWINGS are the minimum allowable sections. The channel alignments may require field adjustment to maintain the channel tolerances as defined in this section and on the DRAWINGS. Tolerances of approved channels and erosion control features, unless approved otherwise by the **CONSTRUCTION MANAGER,** will be within minus one-half (1/2) to plus one-half (1/2) percent of the design grades as shown on the DRAWINGS, with a minimum allowable gradient of one-quarter (0.25) percent between fifty- (50) foot stations.

H. Excavation shall be performed to achieve the lines and grades as shown on the DRAWINGS, to tolerance of plus five tenths (+0.5) to minus one (-1.0) feet but not uniformly high or low.

3.2 GENERAL FILL PLACEMENT:

A. General Requirements

General Fill shall be placed to achieve the lines and grades as shown on the DRAWINGS, to tolerance of plus one (+1.0) to minus five tenths (-0.5) feet but not uniformly high or low. The following general guidelines shall be followed except as noted elsewhere in this SPECIFICATION.

1. No Fill shall be placed until clearing activities have been completed. The procedures for fill placement shall be reviewed and approved by **CONSTRUCTION MANAGER** prior to start of fill placement.

2. No brush, roots, sod, or other deleterious or unsuitable materials shall be incorporated in the fills. The suitability of all materials intended for use in the fill shall be subject to approval by **CONSTRUCTION MANAGER**. **CONTRACTOR** shall temporarily stop fill placement due to weather conditions, if materials and installation do not meet these SPECIFICATIONS.

3. At all times during construction, the surface of the fill shall be graded and maintained by the **CONTRACTOR** to prevent ponding of water and for storm water drainage.

4. Except as otherwise specified or approved by **CONSTRUCTION MANAGER**, the **CONTRACTOR** shall place fill in layers of uniform thickness following the grades shown on the DRAWINGS. Thickness of layers shall be no more than three (3) feet on slopes of less than 2H:1V and no more than six (6) feet on slopes steeper than 2H:1V.

5. Except as otherwise specified or approved by **CONSTRUCTION MANAGER**, the **CONTRACTOR** shall compact three (3) feet layers with sufficient passes of tracked earthmoving equipment to remove any obvious voids and achieve a uniform packed surface to each layer.

6. The fill shall be graded to obtain a surface free from depressions.

*** END OF SECTION ***

SECTION 02250
GRANULAR MATERIALS

PART 1: GENERAL

1.1 DESCRIPTION:

This section describes the requirements for the procurement and placement of granular materials for the rock under drain and roadbase gravels shown on the DRAWINGS and associated with the WORK.

1.2 RELATED SECTIONS:

Section 01300 – Submittals

Section 01400 - Quality Assurance and Construction Documentation

Section 02200 Earthworks

1.3 REFERENCES:

American Society for Testing and Materials (ASTM) most current version:

A. ASTM D2938 – Method for Unconfined Compressive Strength of Intact Rock Core Specimens

B. ASTM D1140 – Method for Particle Size Analysis of Gravel.

C. ASTM C131 – Method for Resistance to Degradation of Small-Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine

D. ASTM C535 – Method for Resistance to Degradation of Large-Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine

E. ASTM D4644 – Method for Slake Durability of Shales and Similar Weak Rocks

1.4 TOLERANCES:

A. Grading limits are defined by the lines and elevations shown on the DRAWINGS.

B. **CONTRACTOR** shall maintain uniform gradients between adjacent spot elevations shown on the DRAWINGS, without depressions or humps.

C. **CONTRACTOR** shall grade granular surfaces within one ($\pm$1.0) feet.

1.5 QUALITY CONTROL/QUALITY ASSURANCE:

A. All WORK shall be constructed, monitored, and tested in accordance with the requirements set forth by the **CONSTRUCTION MANAGER**.

B. The **CONTRACTOR** shall be aware of all CQA activities and shall account for these activities in the construction schedule.

C. On-site conformance testing and field quality assurance testing of granular materials will be the responsibility of the **CONSTRUCTION MANAGER**. The **CONSTRUCTION MANAGER** may obtain conformance samples of the granular material upon delivery to the site. The **CONTRACTOR** shall provide equipment and labor to assist the **CONSTRUCTION MANAGER** in sampling, if requested, and shall also provide access to all areas requiring testing. The **CONTRACTOR** shall repair any damage to finished WORK caused by the **CONSTRUCTION MANAGER** sampling or testing activities.

D. Quality control testing of the granular materials at the source and the Site shall be the responsibility of the **CONTRACTOR**.

PART 2: PRODUCTS

2.1 GRAVEL:

A. Gravel for the rock under drain shall be processed on site by the **CONTRACTOR**, and shall consist of sound, hard, durable, inert, non acid generating pit run, crushed or processed gravel or rock with the following properties:

- Preferably igneous or hard metamorphic rock
- Uniaxial compressive strength >7250 psi (50 Mpa)
- Los Angeles Abrasion <40%
- Slake Durability >90%

The under drain material shall conform to the following gradation unless otherwise approved by the **CONSTRUCTION MANAGER**:

U.S. Sieve Size	Percent Passing By Dry Weight
8-inch	100
4-inch	70-100
1½-inch	43-70
3/4-inch	30-50
#4	10-20
#10	0-5

B. Gravel for road base material shall be processed on site by the **CONTRACTOR**, and shall consist of sound, hard, durable, inert, non acid generating pit run, crushed or processed gravel or rock with the following gradation:

U.S. Sieve Size	Percent Passing By Dry Weight
41/2-inch	100
4-inch	50-100
3-inch	20-45
11/2-inch	5-15
3/8	0-5

PART 3: EXECUTION

3.1 PLACEMENT OF GRANULAR MATERIALS:

A. **CONTRACTOR** shall place under drain rock by dozing into place from specific dump locations on the north side of the drain as shown on the DRAWINGS.

B. **CONTRACTOR** shall place drain rock starting at Station 9+00 and proceed down hill to the west. Placing rock from multiple locations at the same time shall not occur.

C. At no time shall equipment work parallel to the drain alignment at the toe of the adjacent scar slope. Drain placement shall incorporate a catch ditch and safety berm along the south edge of the drain rock as shown on the DRAWINGS.

D **CONTRACTOR** shall place road base gravel on access roads in a single layer as shown on the DRAWINGS or as directed by the **CONSTRUCTION MANAGER**.

E **CONTRACTOR** shall determine the type and size of equipment necessary to place granular materials in such a manner as to minimize segregation and contamination from native materials.

3.2 FIELD QUALITY CONTROL/QUALITY ASSURANCE:

A. The **CONTRACTOR** and the **CONSTRUCTION MANAGER** will verify the final thickness of under drain rock to determine compliance with this SPECIFICATION.

B. If the **CONTRACTOR** and **CONSTRUCTION MANAGER** tests indicate WORK does not meet the requirements of the SPECIFICATIONS, the **CONSTRUCTION MANAGER** will establish the extent of the nonconforming area. The nonconforming area shall be reworked by the **CONTRACTOR**, at their own expense, until acceptable test results are obtained.

C. The **CONTRACTOR** shall be aware of all field quality assurance testing activities, as these may affect their schedule, and they shall comply with the requirements of these SPECIFICATIONS.

3.3 PROTECTION OF WORK:

A. After the granular materials have been placed, the **CONTRACTOR** shall maintain them free of ruts, depressions, and damage resulting from the hauling and handling of any material, equipment, tools, etc.

B. The **CONTRACTOR** shall use all means necessary to protect all materials and all partially completed and completed WORK of these SPECIFICATIONS.

C. In the event of damage, the **CONSTRUCTION MANAGER** will identify any areas requiring repair, and the **CONTRACTOR** shall make all repairs and replacements necessary to the satisfaction of the **CONSTRUCTION MANAGER** and at no additional cost to the **OWNER**.

*** END OF SECTION ***

Appendix C

Rock Drain

NORWEST
CORPORATION

Appendix C Rock Drain

Golder Associates, Letter Report, May 14, 2004: Geochemical suitability of Spring Gulch borrow source for Goathill North drain, Questa Minesite.

Goathill North Final Design Submittal: Excerpt (Condition 5): May 27, 2004 Seepage monitoring from rock drain.

Construction Quality Assurance Program

DRAFT

Golder Associates Inc.

44 Union Boulevard, Suite 300
Lakewood, CO USA 80228
Telephone (303) 980-0540
Fax (303) 985-2080

May 14, 2004 Our Ref.: 033-2153

Molycorp, Inc.
Molybdenum Group
P.O. Box 469
Questa, New Mexico 87556-0469

Attention: Messrs. Dave Partridge and John Vialpando

RE: GEOCHEMICAL SUITABILITY OF THE SPRING GULCH BORROW SOURCE FOR THE GOATHILL NORTH DRAIN, QUESTA MINE SITE

Molycorp has requested that Golder Associates Inc. (Golder) evaluate the geochemical suitability of the Spring Gulch mine rock pile borrow source proposed for use in the Goathill North (GHN) reclamation construction as underdrain materials. At Molycorp's request, Golder has reviewed information on the mineralogy of the proposed Spring Gulch borrow materials compiled during the 2003 test plot soil cover evaluation program, conducted limited screening level laboratory testing of borrow samples (as described in a letter dated March 4, 2003), and visually inspected the borrow materials.

The GHN underdrain has been designed by Norwest as part of the overall GHN reclamation program. The drain has been designed to provide a factor of safety of 10 for conveyance of low quality seepage waters. The chemistry of the seepage waters is poor with a pH of approximately 3, a total dissolved solids content of approximately 25,000 mg/L, with a total acidity as high as 20,000 mg/L as $CaCO_3$. Therefore, the effect of drain rock materials on seepage quality is considered secondary to the effect of the drain rock reactivity on the hydraulic performance of the drain. The physical suitability of the Spring Gulch materials for use as drain rock has been evaluated by Norwest.

Suitability Criteria

Golder offers the following criteria for geochemical suitability for the drain rock:

1. The material should exhibit a high quartz content and/or be silicified, which would translate to lower reactivity with the seepage water;
2. The material should exhibit little or no veining of reactive minerals, which, if present, could result in disaggregation and a reduction in total porosity of the drain;
3. The material should contain minimal neutralizing potential (NP), primarily as carbonate minerals which would react with the acidic seepage resulting in dissolution of the drain rock and/or fouling of the drain by the formation of chemical precipitates;
4. The material should contain minimal sulfides, for example, pyrite, which would oxidize in the drain environment resulting in possible further degradation of the seepage water quality, loss of mass from the drain rock itself, and formation of secondary precipitates; and

5. The material should contain minimal amounts of mafic minerals, such as biotite and chlorite, which are more prone to dissolution than feldspars under acidic conditions.

The types of materials within the borrow source were evaluated with respect to these criteria. The predominant rock types in the Spring Gulch mine rock pile include pit porphyry (aplite) and black andesite, which are commingled within the pile. Based on lithologic logs from bore holes in the rock pile, the proportions of the two materials vary, but on average the materials may be 40% andesite and 60% porphyry. These materials cannot be practically segregated during material processing operations. However, they have different geochemical characteristics which warrant individual consideration in this suitability assessment.

Mineralogic Composition

The porphyry is described as a rock containing fine- to medium-grained K-feldspar with disseminated quartz with minor amounts of plagioclase. The porphyry has a very high quartz content, approximately 35 to 39% by weight (based on the petrographic analyses available from previous studies). The porphyry typically has fewer mafic minerals (estimated 3%) such as biotite, chlorite, amphiboles, and magnetite, which may be susceptible to dissolution under acidic conditions. The principal disadvantage of the porphyry is that the K-feldspars may weather in the acidic conditions of the drain over the long term. The porphyry is expected to be on the order of 40 to 50% K-feldspar.

The black andesite is described as a rock containing fine to medium grained plagioclase with quartz, and varying amounts of biotite and chlorite. The quartz content is generally on the order of 5 to 10%, with minor amounts of K-feldspar. Mafic minerals, according to one petrographic analysis for the andesite was 17%.

Both materials include only minor amounts of clay (<5%), calcite, and pyrite. As will be described later, the average calcite and pyrite contents are small for the mixed porphyry and andesite, less than 3% by weight. The pyrite is generally fine grained and disseminated for both materials, with some veining present most notably for the andesite. The calcite present in the porphyry is also disseminated, but tends to be present in veins in the andesite.

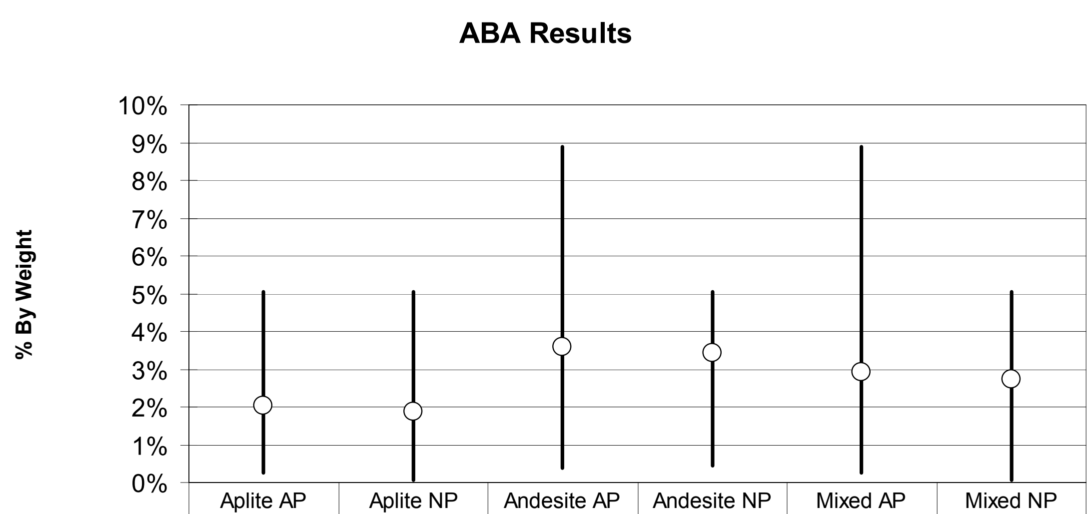
Static Testing

Extensive geochemical testing of the Spring Gulch materials was completed and reviewed as part of the 2003 test plot design (Golder 2003). The intent of the testing for that program was to evaluate the material for use as a final cover for the mine rock piles, and thus the testing was designed to evaluate material behavior under atmospheric conditions, not acidic drain conditions. The information, nevertheless, is useful as a basis for evaluating material reactivity.

A total of 241 samples were tested for paste and rinse pHs and paste EC. Of these, 98% of the samples had measured pH above 5.5 and the average value for both paste and rinse pH was 7.5. Paste EC values ranged from 30 to 2,750 μS/cm and averaged 1,300 μS/cm. The rinse EC values ranged from 90 to 2,040 μS/cm and averaged 1,260 μS/cm. This suggests that under the test conditions, the material is not highly reactive and on average does not generate acidic conditions.

Static Acid Base Accounting (ABA) was performed on 62 samples from Spring Gulch. While standards methods for classifying material behavior from ABA results are not germane to this assessment, the ABA results do provide information on composition. The ABA results for the Spring Gulch material indicate roughly equal potentials for neutralizing (NP) and acid generating (AP), suggesting equal amounts of carbonate minerals and sulfur minerals. Assuming that the NP is calcite and the AP is sulfides (e.g., pyrite), the calcite and pyrite content is less than 3% by weight. The average differences between the two materials is small, typically less than 1% (Figure 1).

Figure 1. ABA results for the aplite, andesite, and mixed material from Spring Gulch.



	Aplite AP	Aplite NP	Andesite AP	Andesite NP	Mixed AP	Mixed NP
Max	5.1%	5.1%	8.9%	5.0%	0.3%	0.1%
Min	0.3%	0.1%	0.4%	0.4%	8.9%	5.1%
○ Arith. Mean	2.0%	1.9%	3.6%	3.4%	2.9%	2.7%

Dissolution of sulfides and calcite by the acidic drain solutions will likely occur rapidly. Since some portion of the calcite is vein oriented in the andesite, some degree of disaggregation of the andesite may occur as the calcite dissolves. However, as noted above, the calcite content is small and the magnitude of the disaggregation is expected to be minimal.

In addition to the static tests described above, leaching tests were also conducted (Golder 2003) using a Synthetic Precipitation Leaching Procedure (SPLP, Method 1312). These tests involved the use of synthetic precipitation and the results are not directly applicable for quantifying the leaching behavior under acidic drain conditions. Therefore, additional laboratory testing was conducted to quantify the short-term reactivity of the Spring Gulch borrow materials when subjected to Goathill North seepage waters and provide a basis for describing the effect on drain water chemistry. The geochemical tests included a chemical analysis of the seepage water, and a modified SPLP using the actual seepage water collected from Goathill Spring, a water to rock ratio of 20:1, and samples of Spring Gulch material crushed to minus 3/8 inch. The grain size reduction was considered to be conservative because the material used for the drain will be screened to include the 3/4 inch to 8 inch size fraction. Three modified SPLPs were completed on three separate samples from Spring Gulch, each containing a mixture of the porphyry and andesite. No complementary ABA testing was conducted, given the large number of test results already available from previous studies. The laboratory test results are included as an attachment to this letter. The analytical results are provided in Table 3.

Table 3. Analytical results from the SPLP testing.

	Al	Ca	Fe	Mg	Mn	K		Si	Na	pH	Cond	TDS		SO4
Sample	mg/L	mg/L	mg/L	mg/L	mg/L	mg/L		mg/L	mg/L		µS/cm	mg/L		mg/L
Influent Diss	1830	447	360	1140	681	3	U	132	13	2.9	11700	24600		18200
Influent Total	1770	437	356	1120	649	6	U	106	15					
Effluent 1	1800	600	210	1110	664	5	B	90	16	2.9	10800	24900	H	18300
Effluent 2	1780	668	108	1110	659	13		100	16	3	10900	24500	H	17900
Effluent 3	1780	648	102	1110	659	16		65	17	3	11000	24400	H	18100

There was no significant change in pH from influent to effluent in the SPLP tests, and overall, minimal change in the chemistry of the effluent composition compared to the influent. This is due to the high water to rock ratio (20:1) used in the test and the very low short-term reactivity of the materials tested. The water to rock ratio in the drain after construction will be substantially higher than that used in the SPLP, therefore, it is likely the drain rock will have no detectable effect on the chemistry of the seepage through the drain beyond the first pore volume. The small changes in chemistry from the SPLP tests were limited to an increase in the Ca concentration. Most other species did not increase significantly between influent and effluent, and in some cases, decreased from influent to effluent concentrations. In summary 1) the mixed andesite and porphyry materials have very low short-term reactivity and 2) will have no detectable effect on drain seepage chemistry.

Long-term Suitability

The carbonates (calcite) present in the Spring Gulch materials are likely to dissolve rapidly upon contact with the acidic drain solutions, but they are only a very small percentage of the rock compositions and are not a significant concern for disaggregation. The sulfides present in both materials will also be reactive, but will not significantly affect drain water chemistry given the initial seepage water chemistry and the large water-to rock ratio. Sulfide oxidation may result in formation of secondary precipitates, however, which may reduce the effective porosity of the drain. Over the longer term, the mafic minerals in the andesite and the K-feldspars in the porphyry are expected to be reactive. Any dissolution of minerals or alternation of mineral faces within the drain will result in formation of secondary precipitates, which will armor the reactive drain material with time. This in turn, will significantly reduce the reaction rates where armoring has occurred.

As described earlier, the Goathill seepage water is an aggressive water with a pH of approximately 3, a total dissolved solids content of 25,000 mg/L, with a total acidity as high as 20,000 mg/L as $CaCO_3$. Quartz is expected to remain stable in this environment. The quartz content of the porphyry is estimated to be on the order of 35 to 39% by weight, with andesite on the order of 5 to 10%. For the mixed materials, the quartz content is expected to be on the order of 23 to 28% quartz. This high quartz content supports long-term suitability of the materials.

The combined effects of the high quartz content, low calcite and sulfide content, the apparent low reactivity of the materials in acidic conditions as indicated by the recent modified SPLPs, and the anticipated armoring of reactive minerals should result in no significant affect on the hydraulic performance of the drain in the short or long term. Over-engineering of design flows by a factor of safety of 10 combined with the anticipated reduction in seepage through other measures, would be expected to provide adequate contingency for any loss of capacity of the drain rock materials in the long term due to geochemical reactions.

Golder strongly suggests that the materials at Spring Gulch be evaluated as part of the Quality Assurance / Quality Control (QA/QC) program during construction to exclude the potentially acid generating (PAG) mixed volcanics (characteristically yellow from jarosite staining) and any porphyry and andesite with abundant pyrite showings. The PAG materials are compartmentalized and easily identifiable within Spring Gulch. These materials are expected to be highly reactive under acidic conditions and would potentially cause fouling of the drain rock. As indicated earlier, sulfides are reactive and can form secondary precipitates within the drain. Therefore, materials with significant visible pyrite should be excluded.

CLOSING

Golder sincerely appreciates the opportunity to support you and Questa on this project. Should you have any questions or comments pertaining to the information provided herein, please contact the undersigned.

Sincerely,

GOLDER ASSOCIATES INC.

Matt Wickham
Senior Hydrogeologist/Geochemist
Associate

Kelly Greaser, Ph.D.
Project Geochemist

cc: Tim Peterson (Norwest)
Mike Ness (Molycorp)

MPW/dls

Reference:

Golder, 2003, Questa Mine Test Plot Work Plan, submitted to Molycorp, Inc.

Kumar & Associates, Inc.
Geotechnical and Materials Engineers
and Environmental Scientists

2390 South Lipan Street
Denver, CO 80223
phone: (303) 742-9700
fax: (303) 742-9666
e-mail: kadenver@kumarusa.com
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Office Locations: Denver, Colorado Springs, Fort Collins, Colorado
Branch Office: Pueblo, Colorado

April 23, 2004

Golder Associates, Inc.
12860 W. Cedar Drive, #300
Lakewood, Colorado 80228

Attention: Matt Barrett

Subject: Laboratory Analyses on a Sample from Golder Associates (Golder Job #033-2153-0007)

Project No. 04-1-230

Dear Mr. Barrett:

As requested, Kumar & Associates, Inc., performed a Los Angeles abrasion (ASTM C-131) and slake durability (ASTM D-4644) on a sample from Spring Gulch submitted by Golder Associates, delivered to our laboratory on March 24, 2004.

Results of this test are listed tabularly and are included with this mailing.

If you have any questions, please feel free to contact us.

Sincerely,

KUMAR & ASSOCIATES, INC.

By____________________
Jim Scott
Laboratory Deputy

jds
Enclosures

UNCONFINED COMPRESSIVE STRENGTH
ASTM D 2938

CLIENT: Golder Associates

LOCATION: Spring Gulch TP-01 Site

PROJECT: 033-2158-007

JOB NO.: 2003-88

DATE TESTED: 3/25/04 HN

Specimen ID Boring, Sample, Depth (ft.)	**Diameter (in.)**	**Length (in.)**	**Mass (gms)**	**Moisture Content (%)**	**Wet Density (pcf)**	**Dry Density (pcf)**	**Failure Load (lb)**	**Failure Types ****	**Compressive Strength (psi)**
TP-01	2.002	3.522	463.68	N/A	159.3	N/A	56,255	C	*17580

Notes and Comments:

* Indicates L/D < 2.0. Correction Factor from ASTM D 2938 used.
C=Ca/[0.88+0.24b/h]
Ca = Failure Load / Surface Area
b = Sample Diameter
h = Sample Length
** Failure Types:
S: Shear Failure, M: Matrix Failure, F/V: Fracture, Bedding/Void Failure, C: Combination

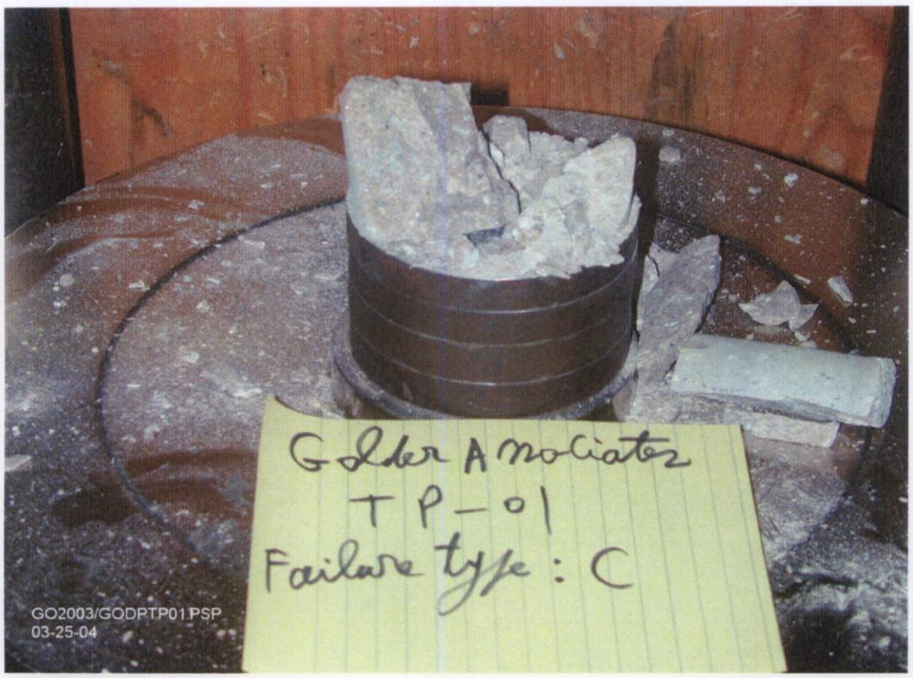
Data Entered By: HN Date: 03/25/2004
Data Checked By: SR Date: 3/29/04
Filename: GOUCSRC6

ADVANCED TERRA TESTING, Inc.

TABLE 1
SUMMARY OF LABORATORY TEST RESULTS

PROJECT NO.: 04-1-230
PROJECT NAME: Golder Associates Inc.
DATE SAMPLED:
DATE RECEIVED: 3/24/04

SAMPLE ID	DATE TESTED	LOS ANGELES ABRASION % LOSS	SLAKE DURABILITY INDEX (%)	SOIL OR BEDROCK TYPE
Spring Gulch TP-01	3/30/04	27	100.0	Coarse Aggregate


Golder Associates
TP-01
Failure type : C
GO2003/GODPTP01.PSP
03-25-04

Excerpt from Goathill North Final Design Submittal

Molycorp will monitor seepage from the under drain as outlined below:

As required in Condition 5 of the letter received from NMED and EMNRD and dated February 20, 2004 regarding the review of Molycorp's Final Report on GHN Mitigation dated January 28, 2004, TA001RE and DP-1055. The Goathill North under drain will be monitored in accordance with the DP-1055 Seep monitoring requirements outlined in conditions 5, 10, and 13 of the discharge permit. The frequency of the monitoring will occur on a quarterly basis. Molycorp will submit results of all monitoring on a quarterly basis to the NMED GWQB. The following is a list of parameters that will be analyzed:

- Flow – The under drain will be inspected and the flow (in GPM) will be estimated on a monthly basis
- Field Parameters – Temperature, pH, electrical conductivity, dissolved oxygen (DO), and reduction-oxidation potential
- General chemistry parameters – Carbonate, bicarbonate, sulfate, chloride, nitrate, fluoride and total dissolved solids.
- Metals parameters (total and dissolved) – Aluminum, arsenic, barium, beryllium, cadmium, chromium, cobalt, copper, iron, lead, manganese, mercury (total concentration only), molybdenum, nickel, selenium, silver, and zinc.

Goathill North Rock Pile Mitigation Project
Construction Quality Assurance Program

Terms of Reference

The QA/QC program will be conducted by the on-site Construction Manager (contracted from Golder Associates) who acts on behalf of Molycorp. As shown on the project organizational chart, the Construction Manager reports directly to the Project Manager. The program is based on the plans and specifications for the project provided by Norwest Corp. Due to the nature of the project, the objectives of the Construction Manager will be, by necessity, somewhat broader than only enforcing project specifications. The main responsibilities specific to this project will be:

1. Inspecting, testing, and controlling quality of the production of drain rock material, including pre-inspection of feed source areas with the assistance of the Molycorp Geology Department.
2. Inspecting, testing, and controlling quality of the production of roadbase material, including pre-inspection of feed source areas with the assistance of the Molycorp Geology Department.
3. Inspecting and documenting survey layout and clearing operations for rock pile and road work.
4. Inspecting cuts, grades and berms for all road upgrade and new road work to assure quality and compliance with standard haul road design. Review road work activities in conjunction with the project Safety Manager for safety performance.
5. Inspecting all work on the rock pile to verify SOP's for quality and safety are being adhered to.
6. Inspecting all cut work for quality, small scale highwall stability, and grade (contour) control.
7. Inspecting all fill work for quality, uncompacted or soft zones, proper construction lift thickness, and grade control.
8. Inspection of all monitoring instrument installation and subsequent monitoring activities.
9. Coordination with all agency inspectors when inspections by non-project personnel are required.
10. Documenting all QA/QC activities, inspection results, and test results.

Program Baseline Testing Schedule

Rock Material Production and Delivery to Project Area

1. Visual inspection for clay and pyrite in rock products feed source areas – daily inspection, plus inspection for any change in source location, and inspection as required by plant operator.
2. Drain rock gradation testing per specification as produced – 1 test per 2500 CY.
3. Roadbase gradation testing per specification as produced – 1 test per 1500 CY.

4. Visual inspection of product for contamination and other deleterious materials in product, in production area and project area stockpiles – daily during these operations.
5. Visual inspection of new haul road construction and existing haul road upgrades upon completion, and weekly after (while in use for hauling product) to verify proper road condition, including safety berm, ditching, and road surface condition.

Rock Material Placement and Other Earthwork Configuration

1. Visual inspection of placement of drain rock to assure proper location and adequate thickness (10 ft) – twice daily during operation. Frequent inspection of placement operation to preclude contamination of material during placement. Final inspection with documentation to submit to MMD, upon completion of drain installation.
2. Visual inspection of rock pile cut and general fill operations to assure adherence to SOP's for quality and safety.
3. Collection of all contractor grade control survey data and management of project survey database documenting adherence to planned grades and elevations on a pre-established 50 ft by 50 ft grid, as dictated in plan specifications:
    a. For cut surfaces – final elevations to be within plus 0.5 ft and minus 1.0 ft of plan, final grades to be within 0.5% of plan.
    b. For fill surfaces – final elevations to be within plus 1.0 ft and minus 0.5 ft of plan, final grades to be within 0.5% of plan.
4. Spot checking of contractor survey results to verify survey accuracy, as required.
5. Visual inspection of maximum lift thickness control during fill operations – four times daily during operations.
6. Visual inspection of fill compaction to prevent voids or soft spots – four times daily during operations.
7. In situ density testing of general fill two times daily.
8. Visual inspection of final highwalls during cut operations to assure that walls are adequately trimmed and free from loose hanging rock.

Appendix D

Proposed Weekly Report

Proposed Weekly Progress and Monitoring Report
Goathill North Mitigation Project
Molycorp, Inc.

Date:

Reporting period from …………to…………

Construction Activities:

- Access road construction
- Under drain
- Phase 2 Earthworks
- Phase 3 Earthworks
- Phase Interim Surface Drainage

Monitoring:

- **Precipitation**
  Maximum 24 hour precipitation ……
  Maximum 48 hour precipitation ……
  Maximum 72 hour precipitation …….

  Total precipitation for the month ………..inches. Total precipitation, to date, for the month is ….inches. Refer to the attached Chart 1.

- **Visual Inspections:**
  Frequency: Daily
  Results:

- **Pore pressure monitoring**: The piezometers recording within the Goathill North and Capulin areas continue to be measured daily. Piezometers without specific trigger levels, those which do not appear to be specifically correlated to groundwater conditions within the rock pile, are not included in the table. All of the piezometers included as part of the emergency warning review and notification program are included in the table. Refer to attached Charts 2 and 3.

Location	Piezometer Depth (ft)	Static Piezometric Level* (ft)	Trigger Level (ft)
TH-C-01-143	141	Dry	140
TH-C-02-201	201	200.1	199
TH-C-04-260	260	Dry	258
TH-C-05-67	67.5	64.5	66
TH-GN-01-205	205	202.6	198
TH-GN-02-139	138.5	135.3	128.5
TH-GN-03-50	50	Dry	40
TH-GN-06-231	231	Dry	229
TH-GN-07-59	59	NA	56
TH-GN-08-168.5	168.5	Dry	167.5

* All reported piezometer static water levels have been adjusted to reflect zero offset values calculated from field data recorded at the time of installation. TH-GN-07-59 could not be read manually because readout unit was sent for maintenance and recalibration. However risk of an undetected event is minimal because this piezometer has exhibited an unchanging dry condition since installation.

- **Displacement monitoring:**

  Lath Crackmeters:
  The two remaining lath crackmeters were damaged April 1st by falling snow and ice; readings are no longer possible from these devices (due to the winter seasonal damage, all of the lath crackmeters are no longer effective). During construction the monitored cracks will be disturbed and monitoring with lath crackmeters will be discontinued.

  Borehole Inclinometers:
  The primary focus of monitoring any slope displacements during construction will shift to daily reading of several new inclinometers to be installed in and around the mitigation site. The following criteria for this monitoring have been set in the Work Scope document.

Location		**Trigger Levels**		
		SI Movement	*Pore Pressure*	*Visual**
Outside Slide Area	Rock Pile or Colluvium SI-25,26,30	Development of a new shear plane** **OR** Displacement > 0.03" over 10ft depth	-	Scarp or tension crack development with vertical or horizontal displacement > 1ft
Inside Slide Area	Colluvial bench SI-27, 28, 29	Development of a new shear plane** **OR** Displacement rate on existing shear plane > 0.1"/day	Rise in groundwater at specified piezos > 5ft	Scarp or tension crack development with vertical or horizontal displacement > 3ft
	Toe area of slide SI-23, 31	Development of a new shear plane** **OR** Displacement rate on existing shear plane > 0.1" over 3 days	Rise in groundwater at specified piezos > 5ft	Scarp or tension crack development with vertical or horizontal displacement > 3ft

  Wire Line Extensometers:
  Displacement monitoring will be superceded by monitoring of new inclinometers and as the rock pile surface area will be disturbed by construction, monitoring wireline extensometers will be discontinued.

  Vibrating Wire Crackmeters:
  During construction the monitored cracks will be disturbed and monitoring with crackmeters will be discontinued.

<u>Investigation Activity:</u>
Covered under construction progress

<u>Surface Water Management at Goathill North and Capulin:</u>
Covered under construction progress

<u>Development of the final mitigation plan:</u>
Covered under construction progress

<u>Emergency Action Plan:</u>
The emergency action plan will be revised to reflect the updated construction monitoring plan.

Appendix E

Evaluation of 40,000 yard supplemental Toe Berm Fill Option:
Letter Report, May 14, 2004

Molycorp, Inc.
Molybdenum Operations
P.O. Box 469
Questa, NM 87556-0469
Telephone (505) 586-7603

Molycorp

May 14, 2004

Terrence Foreback
MMD
Pinon Building
1220 South St. Francis Drive
Santa Fe, NM 87505

Mr. Michael Reed
GWQB-MECS
Harold Runnels Building
1190 St. Francis Drive, P.O. Box 26110
Santa Fe, NM 87502-6110

Subject: Response to Condition No. 6 of the MMD Approval Letter for the Goathill North Rockpile Feasibility Level Design Report – Evaluation of 40,000 Yard Supplemental Toe Berm Fill Option

Dear Sirs:

In response to Condition 6 of your February 20, 2004 conditional approval letter for the feasibility design of the slide mitigation of the Goathill North Rock Pile, please find enclosed a letter from Molycorp's consultant Norwest Corporation that addresses this condition. This submittal is being provided in advance of the detailed design submission, which is currently being prepared for the State's review and approval later this month.

As you are aware, there are 13 conditions of the approval that Molycorp has agreed to meet as part of the design and construction process. Condition 6 specifically requires additional clarification on the 40,000 yard toe berm fill option presented in our January 2004 feasibility design report. The enclosed report provides justification why the additional material is not necessary through use of additional points of clarification and analysis.

Molycorp is committed to working with the State agencies in addressing the matters raised in your letters. If you have any questions regarding this submittal, please call me at 505-586-7603.

Sincerely,

Ray Cherniske
Manager, Remediation Sites

cc: Bill Sharrer - Molycorp
Dr. Richard Dawson, P.Eng. - Norwest

NORWEST
CORPORATION

Suite 1022, 475 Howe Street
Vancouver, British Columbia V6C 2B3
Tel: (604) 602-8992 Fax: (604) 602-8951
Email: Vancouver@norwestcorp.com
Website: www.norwestcorp.com

File #: 03.-2368

May 13, 2004

Molycorp Inc.
PO Box 469
Questa, NM 87556-0469
USA

Attention: Mr. Bill Sharrer
Vice President, Environmental and Public Policy

Dear Mr. Sharrer:

RE: Further Stability Information on Goathill North Mitigation Design Prior to Detailed Design Submission

1.0 Terms of Reference

On February 20, 2004, Molycorp received conditional approval for Goathill North slide mitigation based on Norwest's January, 2004 "feasibility level" design report . There are 13 conditions that need to be attended to during the detailed design and construction process. One of the Conditions of Approval of the mitigation design requires additional clarification on the toe berm fill options presented in the design report. Specifically, Condition #6 states:

> *The Report discusses supplemental toe berm fill options. One option, to add 40,000 yards to the toe berm, increases the factor of safety (FOS) from 1.11 (Table 6.1) to 1.14 or by 3%, which the Report describes as a "nominal" increase. However this is approximately the same amount of increase that the proposed base case fill provides. The FOS is shown on Table 6.2 from the December Draft Report as 1.08 and has been increased, by the current proposed toe berm, in the proposed base case to 1.11 listed on Table 6.1 of the final report. Comments from the SRB indicate the preference for an increased FOS, which would be accomplished by the additional 40,000 yards of material in the toe berm. Therefore, as part of the construction plans and drawings, Molycorp shall submit to the Agencies, for approval, plans for the placement of the additional 40,000 yards of material in the toe berm or Molycorp will provide adequate justification why this material is not necessary including a cost estimate for placing the material.*

The purpose of this letter is to address this issue with additional points of clarification and analysis in order to justify the proposed design. Figure 1 and Figure 2 attached to this letter provide a plan view and a down-valley cross section for reference.

It is recommended that the reader reference the Norwest design report for clarification of design issues not presented here and for background on the slide itself.

2.0 Down Valley Stability and 2D versus 3D Factors of Safety

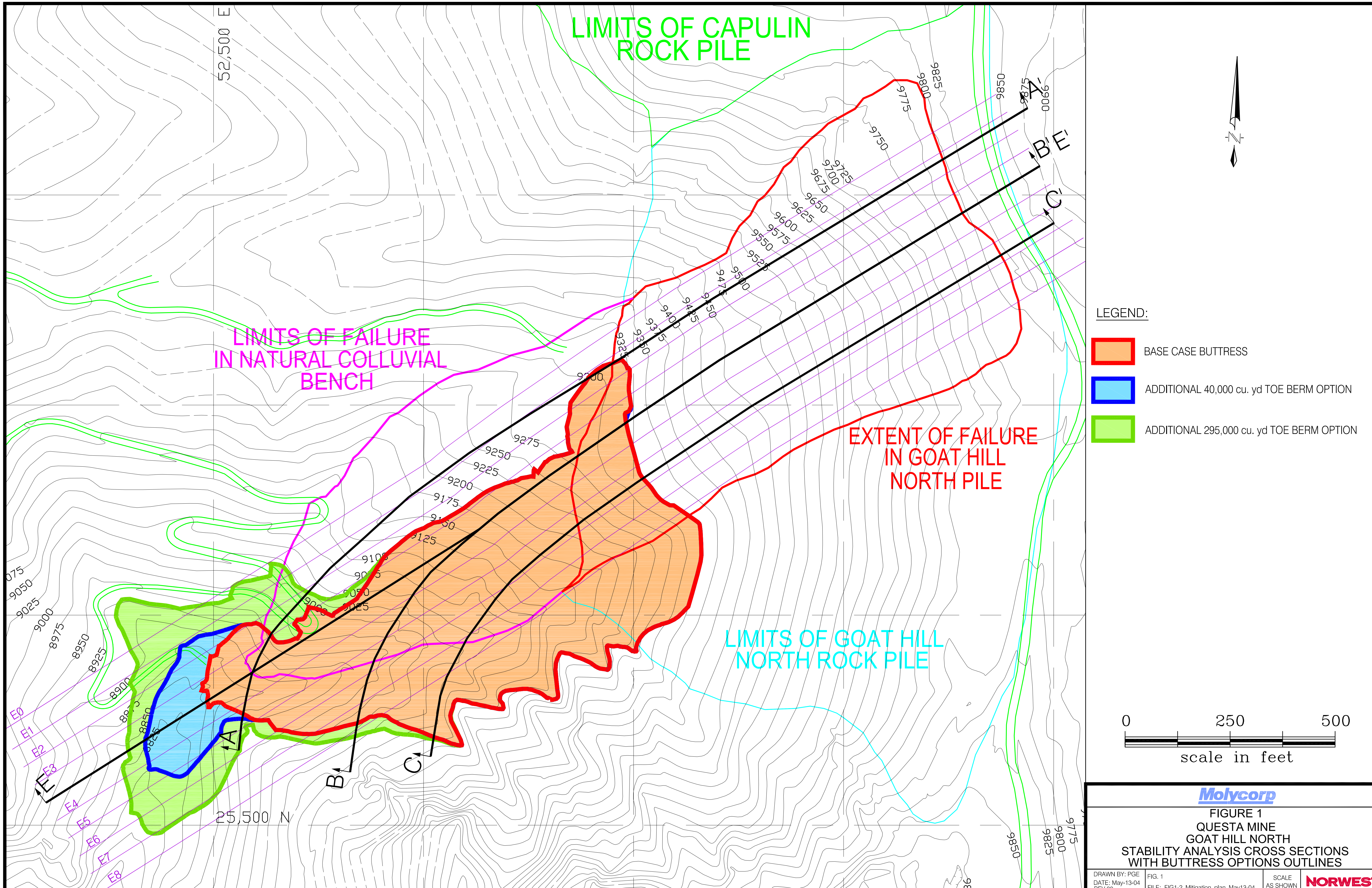
The landslide at Goathill North is not moving in a two dimensional down-valley direction. This is clearly indicated by the shear surface contours and displacement vectors on Figure 5.1 in the feasibility report which shows that the upper portion moves in down-valley direction and that the lower portion is moving across the valley into the gully on the south side below the scar. The shear surface is daylighted in the gully. Stream bed erosion is a key contributing factor to the slide movements which appear to have been occurring for over 30 years. Two dimensional stability analysis on cross sections that rotate in the direction of slide movement (Sections A, B and C in Figure 5.1 of the feasibility report) compute an average slide surface friction angle of 23.8°.

Although the slide is not currently moving entirely in a down valley direction, the mitigation needs to ensure that the whole mass is not directed down valley as a result of the fill placement and mass re-distribution. This is an important issue which has been raised by the Stability Review Board during review of the draft report and it is the primary purpose for the discussion of 2D and 3D down valley stability presented here.

Table 1 below shows a summary of the factors of safety that have been computed for current and mitigated (proposed design) conditions in the down valley direction. For each set of analyses (2D and 3D) there are two different "weak zone" friction angles used. The first is the friction angle calculated for limit equilibrium conditions (factor of safety = 1.0) in the down valley direction and the second is the friction angle which corresponds to the average back calculated value of 23.8° using cross sections A, B, and C. The latter value of 23.8° is considered to be the best estimate for the weak zone because it corresponds to back analysis in the direction of slide movement. There are three important points which emerge from a comparison of values from Table 1:

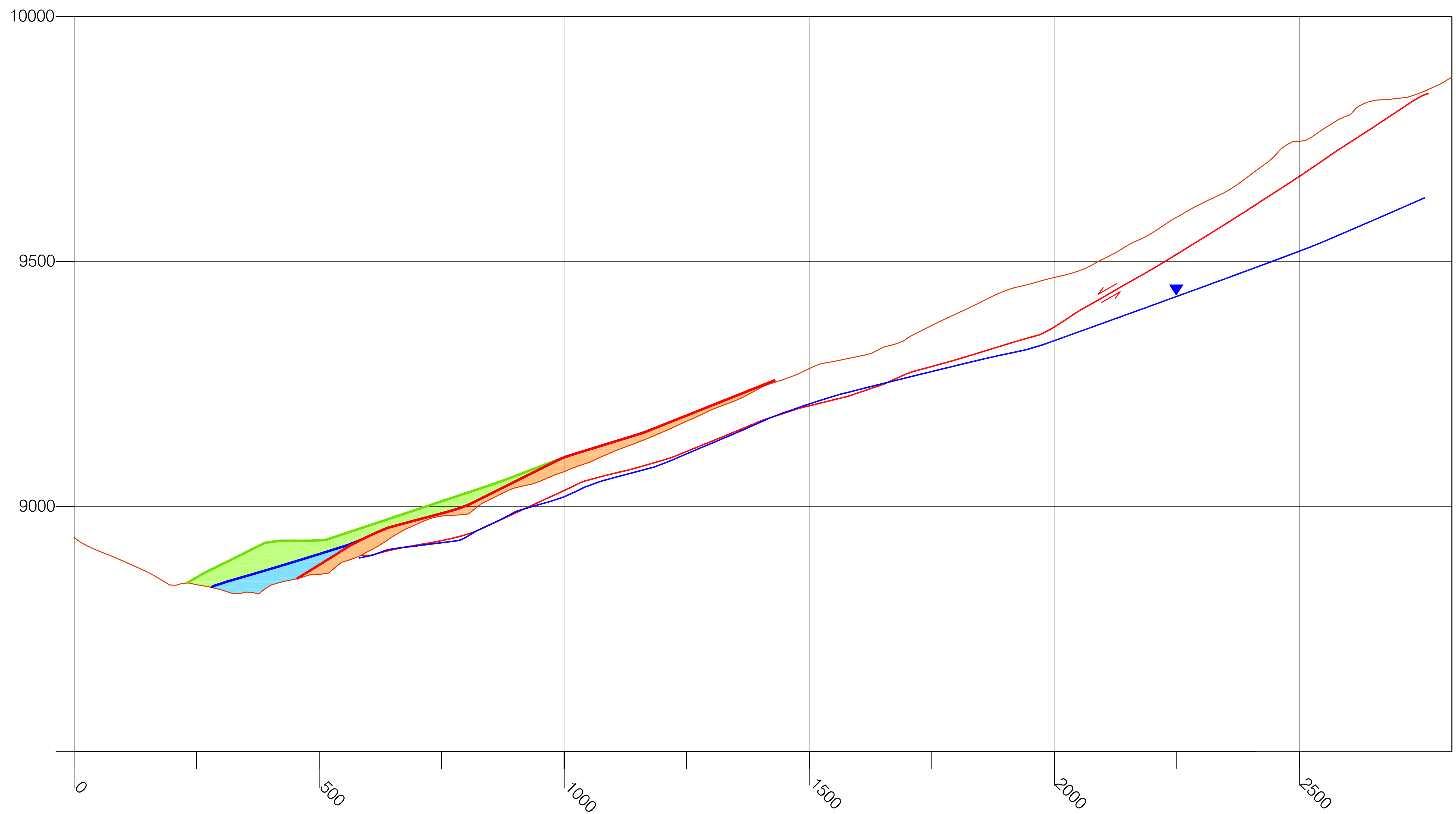
- The 2D and 3D factors of safety in the down valley direction using the best estimate for the average weak zone friction angle (i.e. 23.8°) are greater than unity. This is an indication that there is currently a reserve of stability in the down valley direction, which is analyzed to be between 4% (2D analysis) and 19% (3D analysis).

- The 3D analysis for the proposed design shows a lower bound factor of safety equal to 1.26 and an upper bound equal to 1.40. These values meet the minimum design criteria of 1.2 and both show an increase in stability with the proposed design, which is up to


LIMITS OF CAPULIN ROCK PILE
LIMITS OF FAILURE IN NATURAL COLLUVIAL BENCH
EXTENT OF FAILURE IN GOAT HILL NORTH PILE
LIMITS OF GOAT HILL NORTH ROCK PILE
52,500 E
25,500 N
LEGEND:
BASE CASE BUTTRESS
ADDITIONAL 40,000 cu. yd TOE BERM OPTION
ADDITIONAL 295,000 cu. yd TOE BERM OPTION
0
250
500
scale in feet
Molycorp
FIGURE 1
QUESTA MINE
GOAT HILL NORTH
STABILITY ANALYSIS CROSS SECTIONS
WITH BUTTRESS OPTIONS OUTLINES
DRAWN BY: PGE
DATE: May-13-04
REV.00
FIG. 1
FILE: FIG1-2_Mitigation_plan_May13-04
SCALE
AS SHOWN
NORWEST

000277

Section E-E'



LEGEND:

BASE CASE BUTTRESS

ADDITIONAL 295,000 cu. yd BUTTRESS TOE BERM

ADDITIONAL 40,000 cu. yd TOE BERM

SHEAR SURFACE

PIEZOMETRIC SURFACE

0 250 500
scale in feet

Molycorp

FIGURE 2
QUESTA MINE
GOATHILL NORTH
SECTION E-E'
WITH TOE BERM OPTIONS

DRAWN BY: PGE DATE: May13-04 REV.00	FIG.2 FILE: FIG1-2_Mitigation_plan_May13-04	SCALE 1:200	NORWEST

000278

26% using the best estimate of the average weak zone friction angle (ie.23.8°). This provides an indication for the relative benefit of the proposed mitigation in the down valley direction, regardless of the shear zone friction angle assumption.

- There is a difference between the mitigated 2D versus 3D factors of safety in the range of 18-19%. This reflects the additional lateral shearing resistance that is provided by the buttress fill. A comparison of Figure 5.11 and Figure 6.12 in the feasibility report shows that there is a significant increase in the surface area of the shear surface after the fill is placed. It is difficult to account for this shearing resistance using 2D analysis because of the complex geometry.

TABLE 1
GOATHILL NORTH 2D STABILITY SUMMARY

	2D Analysis on Section E		3D Analysis	
	F = 22.8°*	F' = 23.8°**	F' = 20.5°*	F' = 23.8°**
Current Conditions	1.00	1.04	1.00	1.19
Proposed Design	1.07	1.12	1.26	1.40

* weak zone friction angle for limit equilibrium (FOS = 1) conditions
** average weak zone friction angle from 2D back analysis in the direction of slide movement

These are the primary stability analyses issues which Norwest has relied on to support the design in terms of down valley stability. Stability in the cross valley direction (Sections A, B and C) is assessed in the feasibility report using 2D analysis.

The remaining issue, which the Stability Review Board (SRB) and the state have raised, is whether additional material placed at the toe should be justified in order to further increase the safety factor and provide an additional reserve of stability (either during or after construction) for offsetting uncertainties in the slide kinematics (model uncertainty) and/or in the values used for the back analysis (parameter uncertainty).

3.0 Toe Berm Options

The feasibility study contains a brief overview for three different buttress and toe berm options (proposed design and two other options with larger toe berm configurations). A summary for the factors of safety on section E for each of these options was presented in Table 6.1 of the report. These values are also shown in Table 2 below (under Table 6.1 column) which shows that an extra 40,000yds^3 increases the factor of safety by 3-4% and an additional 295,000yds^3 achieves a 9% increase in the factor of safety. There is a small difference in the safety factors between the detailed design and the feasibility design, as shown in Table 2. The feasibility report concludes that:

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"in order to achieve a significant benefit in stability, a very large volume of fill is required in the toe berm area. It is difficult to justify the additional expense for the large berm especially because there are no current large scale down valley movements. Also, due to the potential complications arising from natural erosion and slumping of the natural valley slopes in this area, it is not advised without carrying out additional investigations and geotechnical evaluations."

TABLE 2
2D ANALYSIS OF TOE BERM OPTIONS

Safety Factor with F = 23.8°			
	Section E		Average of 10 cross sections* in the toe zone with detailed design
	Feasibility Report (Table 6.1)	*Detailed Design*	
Proposed Design	1.11	1.12	1.15
Additional 40,000 yd^3	1.14	1.16	1.15
Additional 295,000 yd^3	1.20	Not included	

* See Figure 2 for cross section locations

The state has requested that the nominal increase in factor of safety (based on 2D stability analysis along Section E) that is provided by the small toe berm increment (40,000yds^3) should be considered as part of the detailed design. Additional 2D analyses have been carried out to evaluate the incremental benefits of this additional material in the toe region of the slide. The analysis was carried out with the detailed slope geometry.

Table 2 shows average safety factors from 10 cross sections constructed parallel to Section E at 50ft spacing. Drawing 2 shows the locations of these sections across the 40,000yds^3 fill option area and the detailed results are compiled in a table attached to this letter, along with all of the stability cross sections. Because of the lateral changes in geometry at the toe of the slide, the average safety factor for the 10 sections is slightly higher (by 3%) than the safety factor on section E by itself. Most importantly, the averaged value of 1.15 for the 10 sections is the same (when rounded to 3 significant figures) for the proposed design and with the additional 40,000yds^3. The 40,000yds^3 fill increment is too small a mass relative to the whole slide to significantly increase the safety factor in the down valley direction.

It is estimated that the additional 40,000yds^3 toe berm fill would cost between $150,000 and $200,000. Norwest is unable to recommend this additional expense, especially considering that the revised and more detailed analysis does not indicate any significant change in stability. There is also a risk that the foundation area below the additional fill, which extends out beyond the toe of the current slide, may not provide for stable conditions.

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4.0 Summary

Norwest recommends that Molycorp advance a detailed mitigation design for Goathill North that follows the “proposed design” in the feasibility report. Some additional clarification and analysis presented in this letter provides the pertinent justification for doing so in order to satisfy Condition #6 in the February 20, 2004 state approval letter. To summarize:

- There is no evidence of large scale slide movements currently acting in the down valley direction in the toe region of the slide mass. Three dimensional stability analyses for the “proposed design” indicates a lower bound factor of safety of 1.26 which exceeds the design criteria of 1.2. Approximately 18-19% of the safety factor increase due to the mitigation is attributed to lateral shearing resistance in the buttress fill.

- In order to achieve a significant benefit in the 2D down valley stability, a very large volume of fill is required in the toe berm area. Revised analysis for the small fill option (40,000yds^3) presented as an alternative in the feasibility design does not show any significant increase in the factor of safety. It is estimated that the additional 40,000yds^3 toe berm fill would cost between \$150,000 and \$200,000. There is also a risk that the foundation area below the additional fill, which extends out beyond the toe of the current slide, may not provide for stable conditions.

Notwithstanding these important points for the completed fill design, the down valley stability during construction remains an important issue. The detailed design will address monitoring and contingency measures that address stability risks during construction.

Yours truly,

Norwest Corporation

Richard Dawson, PhD, P.Eng
Vice President, Geotechnical

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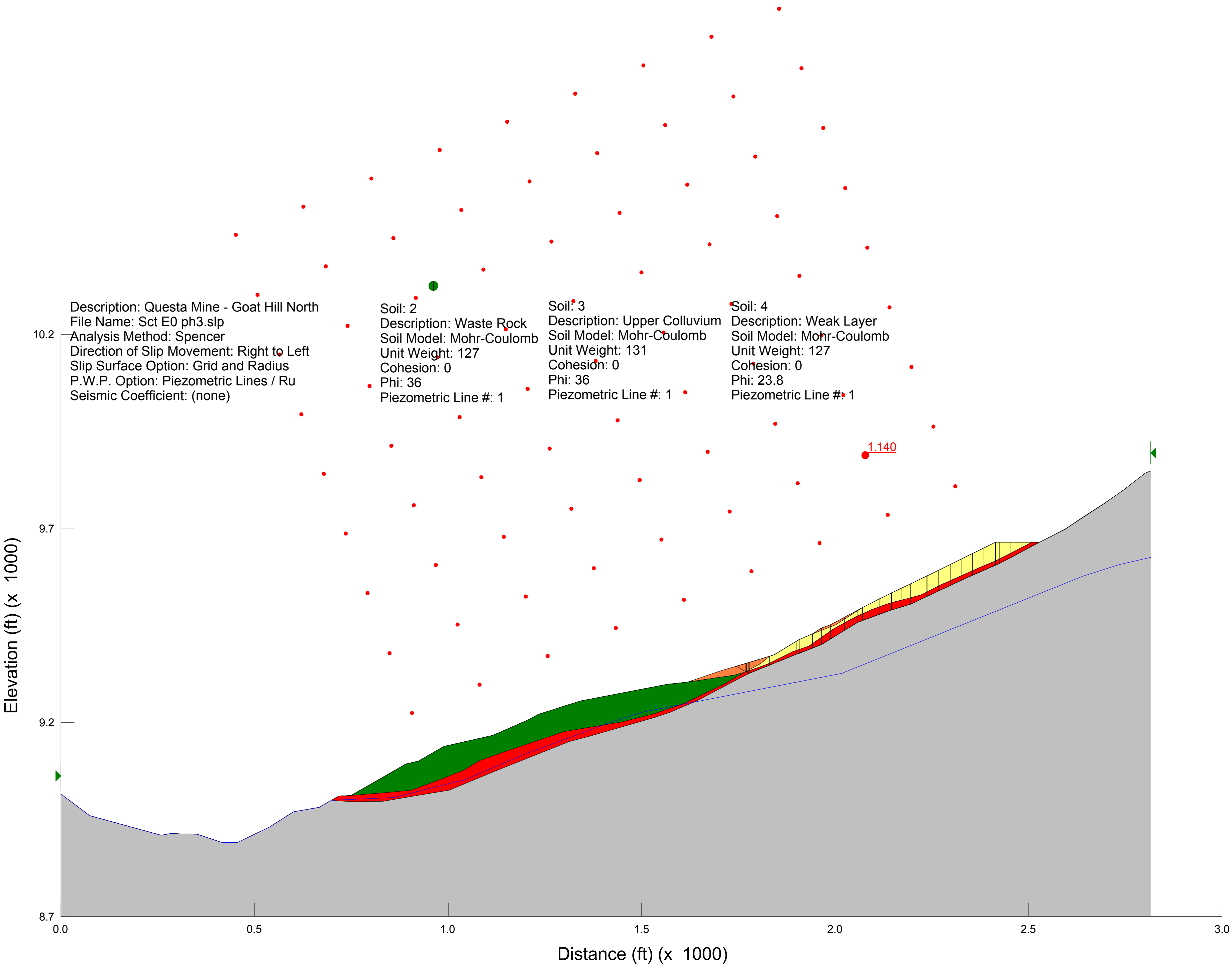
ADDENDUM

Additional Analysis for Toe Berm Option Assessment

Section	φ'average = 23.8°									
	2003 Topo		Ph 2 - Init Buttress		Ph 3 - Final Buttress			Ph 3 - Add 40,000		
	Spec	**Search**	**Spec**	**Search**	**Spec (L)**	**Spec (OV)**	**Search**	**Spec**	**Search**
E0	1.147	0.909	1.147	0.909	1.377	1.277	1.140	1.277	1.140
E1	1.080	0.893	1.080	0.893	1.303	1.180	1.141	1.180	1.141
E2	1.044	0.921	1.044	0.921	1.241	1.132	1.095	1.132	1.095
E3	1.033	1.011	1.031	1.012	1.217	1.121	1.094	1.138	1.115
E	1.051	1.038	1.061	1.047	1.223	1.138	1.122	1.18	1.159
E4	1.029	1.013	1.097	1.075	1.247	1.179	1.155	1.195	1.169
E5	0.973	0.941	1.091	1.068	1.322	1.201	1.169	1.201	1.169
E6	0.939	0.912	1.063	1.026	1.351	1.223	1.177	1.223	1.177
E7	0.967	0.902	1.056	1.003	1.400	1.233	1.191	1.233	1.191
E8	0.838	0.820	1.042	1.033	1.321	1.205	1.187	1.205	1.187
E3-4 Avg	1.038	1.021	1.063	1.045	1.229	1.146	1.124	1.171	1.148
E2-5 Avg	1.026	0.985	1.065	1.025	1.250	1.154	1.127	1.169	1.141
E1-6 Avg	1.021	0.961	1.067	1.006	1.272	1.168	1.136	1.178	1.146
E0-7 Avg	1.029	0.949	1.074	0.995	1.298	1.187	1.143	1.195	1.151
Average	**1.010**	**0.936**	**1.071**	**0.999**	**1.300**	**1.189**	**1.147**	**1.196**	**1.154**

Localized Upper Failure

Localized Lower Failure



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Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 3
Description: Upper Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
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Soil: 4
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Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 23.8
Piezometric Line #: 1
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Distance (ft) (x 1000)

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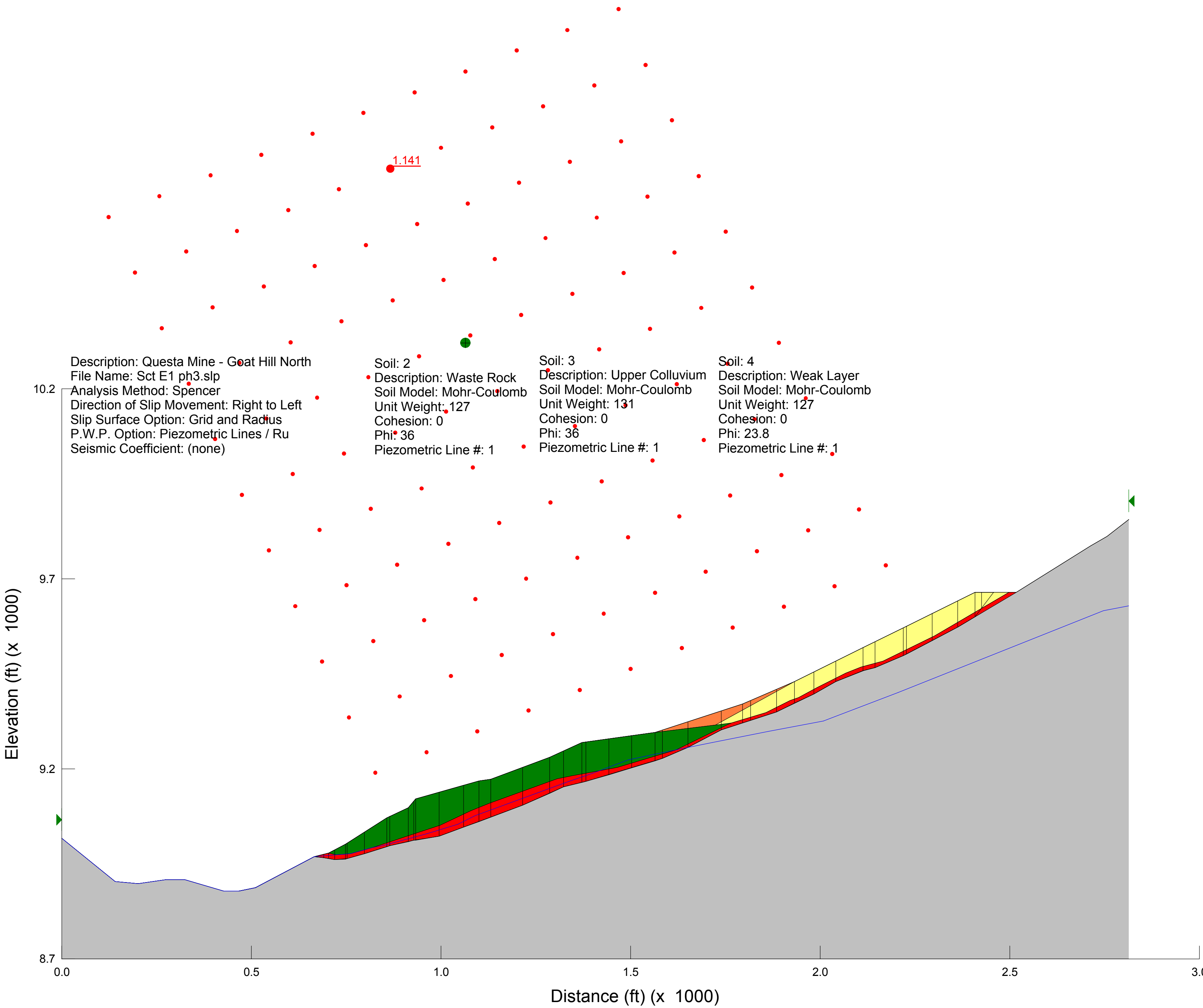
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Phi: 23.8
Piezometric Line #: 1
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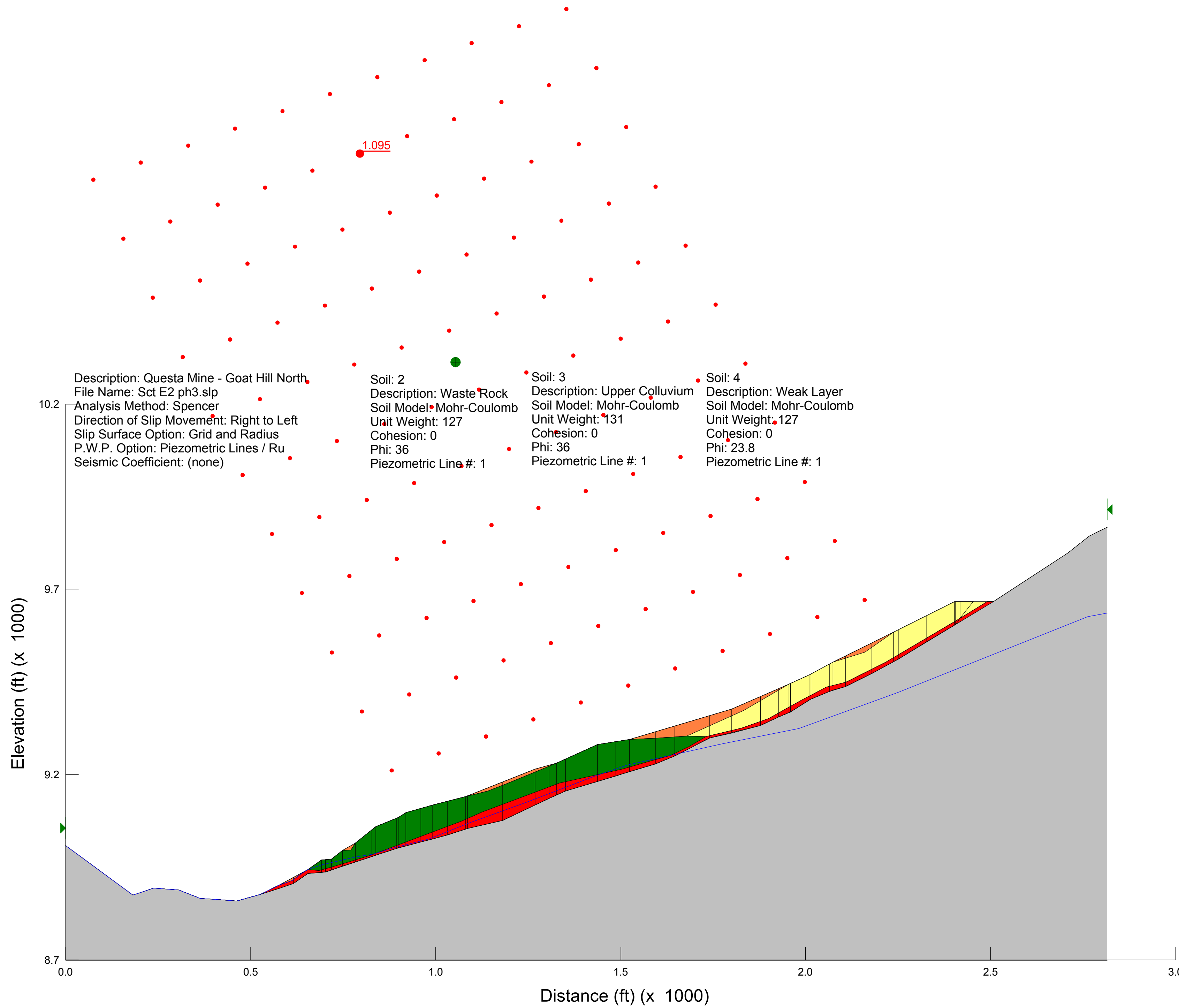
Distance (ft) (x 1000)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Description: Weak Layer
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 23.8
Piezometric Line #: 1
10.2
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8.7
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1.5
2.0
2.5
3.0
Distance (ft) (x 1000)

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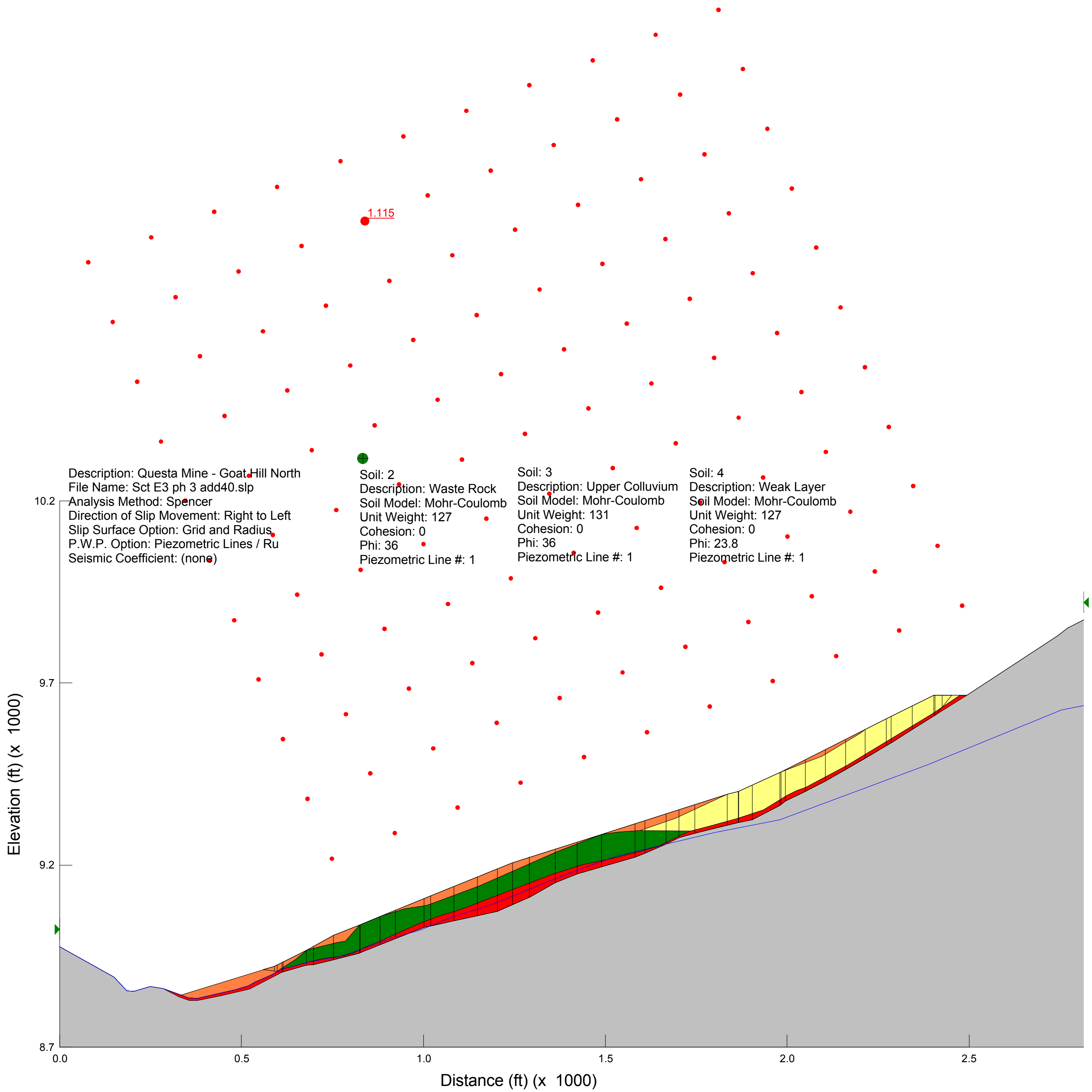
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Phi: 23.8
Piezometric Line #: 1
10.2
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2.0
2.5
Distance (ft) (x 1000)

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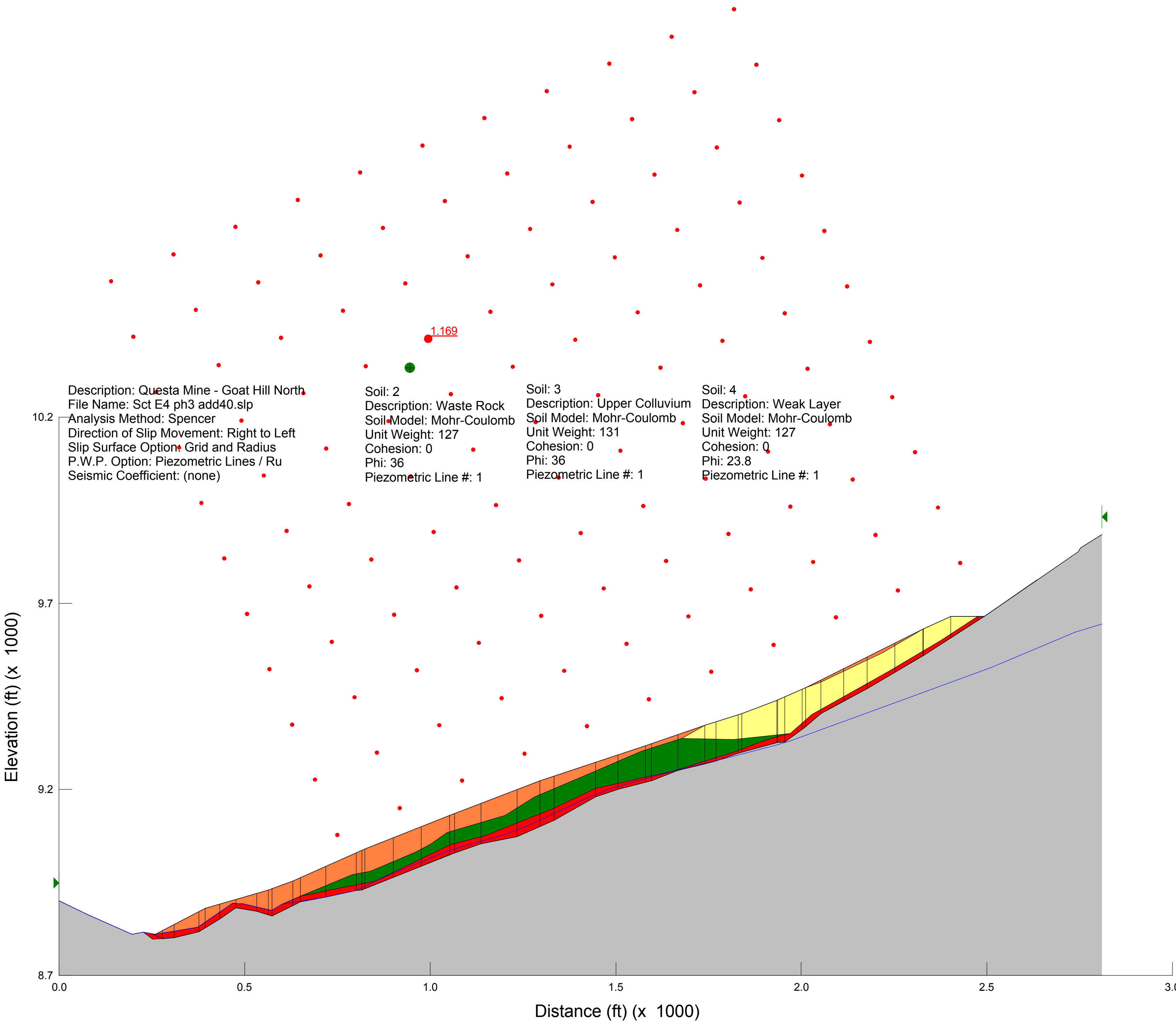
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Soil Model: Mohr-Coulomb
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Description: Weak Layer
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Unit Weight: 127
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Piezometric Line #: 1
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Distance (ft) (x 1000)

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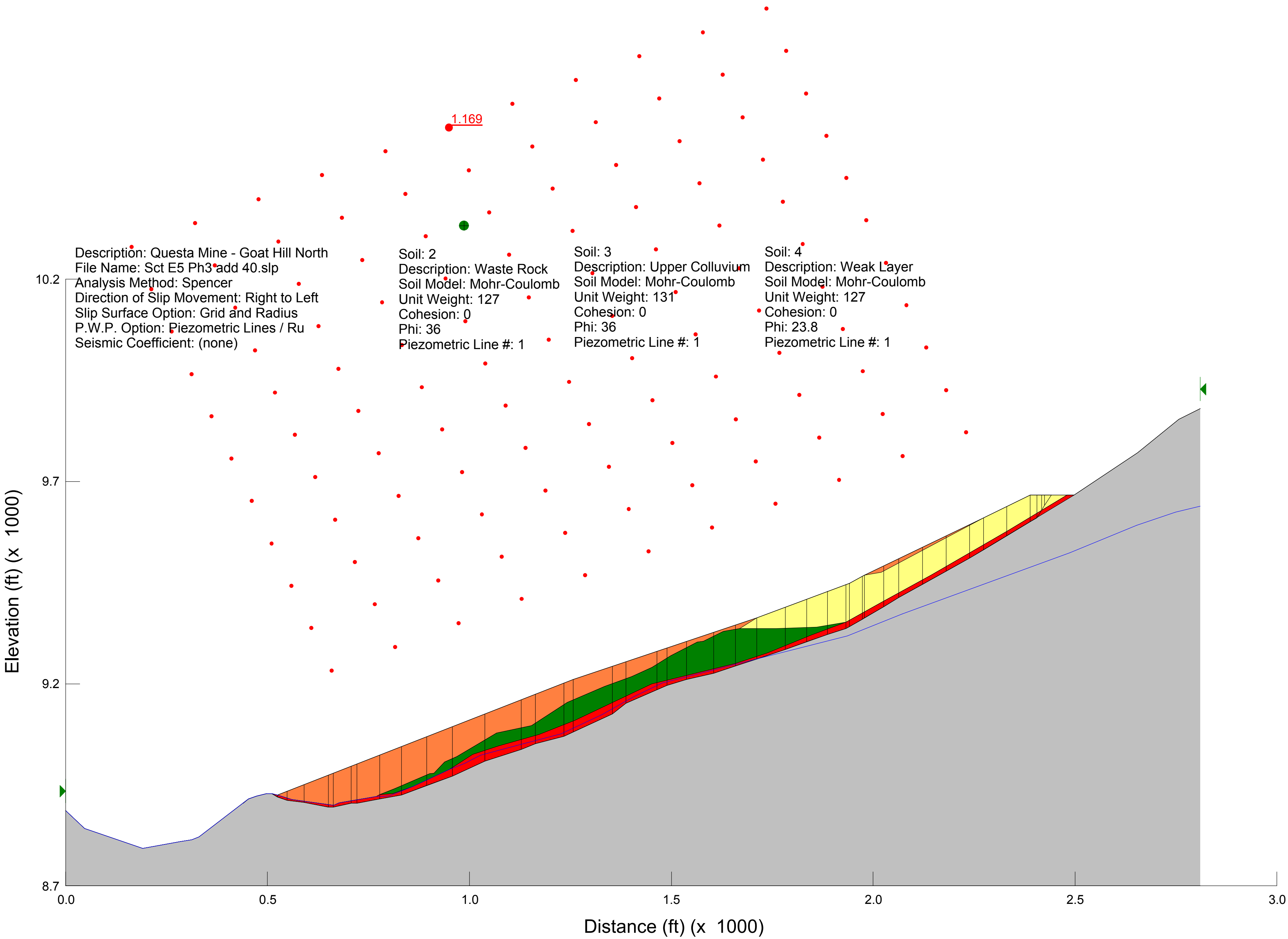
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Description: Weak Layer
Soil Model: Mohr-Coulomb
Unit Weight: 127
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Piezometric Line #: 1
Elevation (ft) (x 1000)
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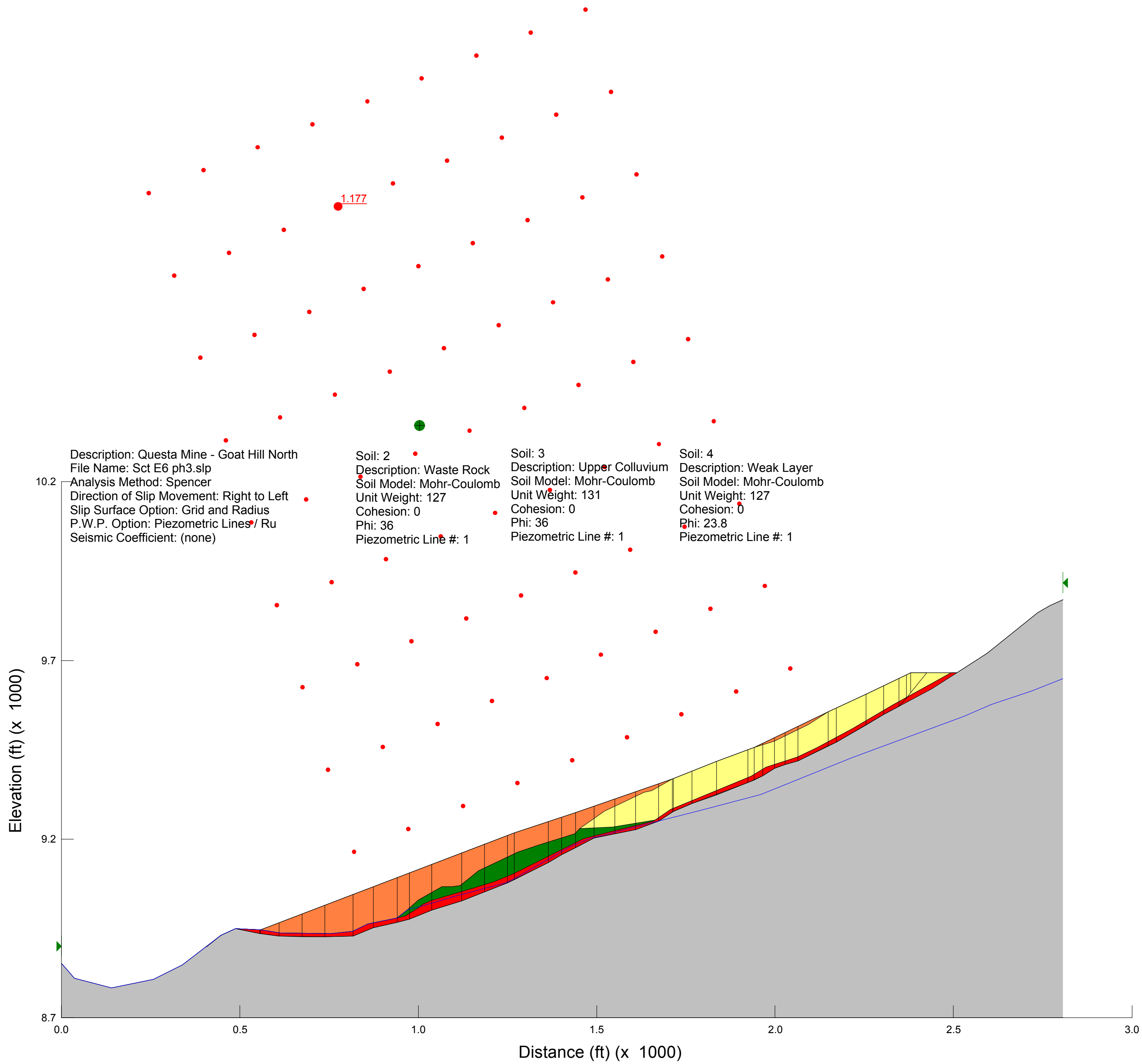

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Soil Model: Mohr-Coulomb
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Soil: 4
Description: Weak Layer
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 23.8
Piezometric Line #: 1
Elevation (ft) (x 1000)
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Distance (ft) (x 1000)

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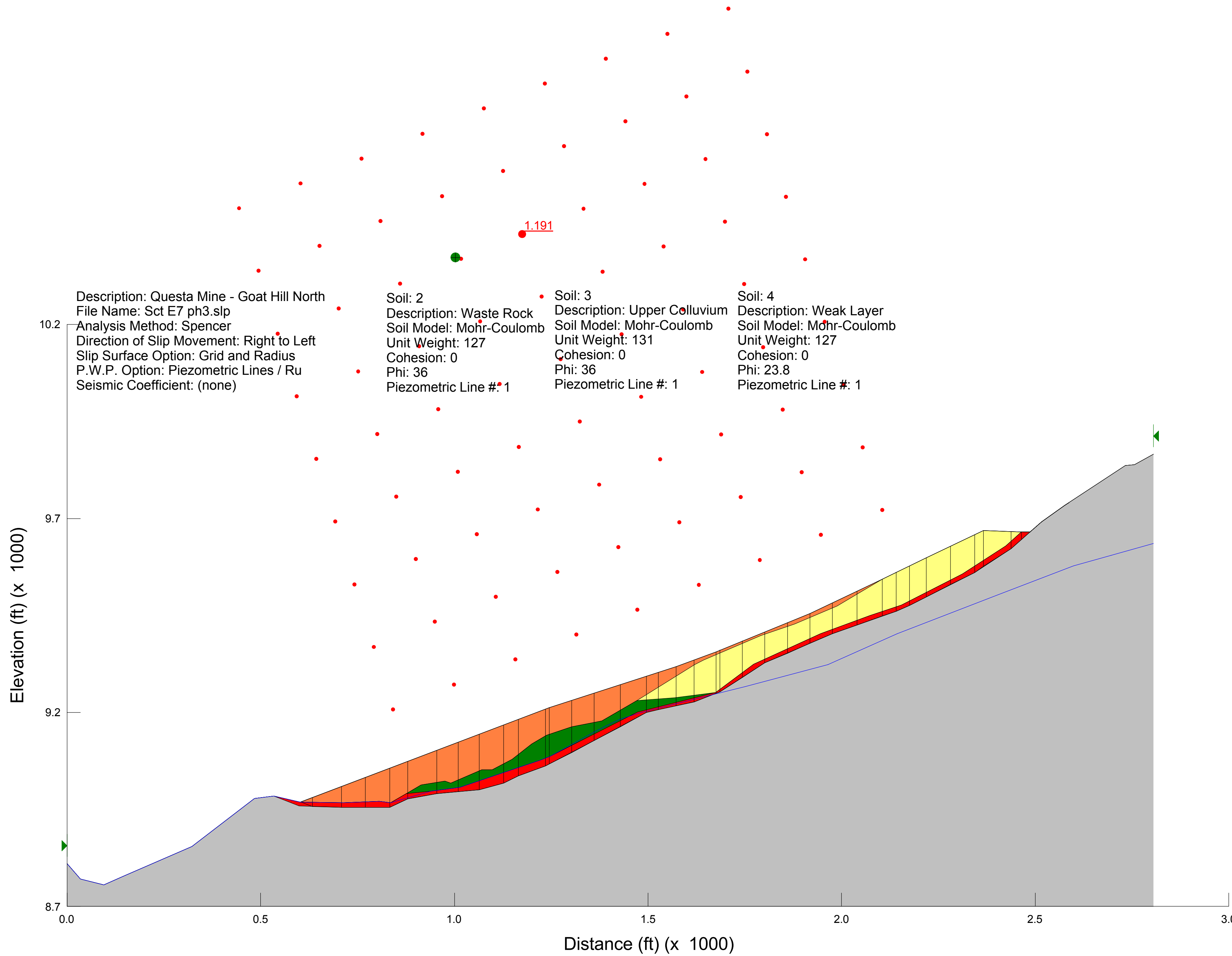

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Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
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Description: Upper Colluvium
Soil Model: Mohr-Coulomb
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Cohesion: 0
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Piezometric Line #: 1
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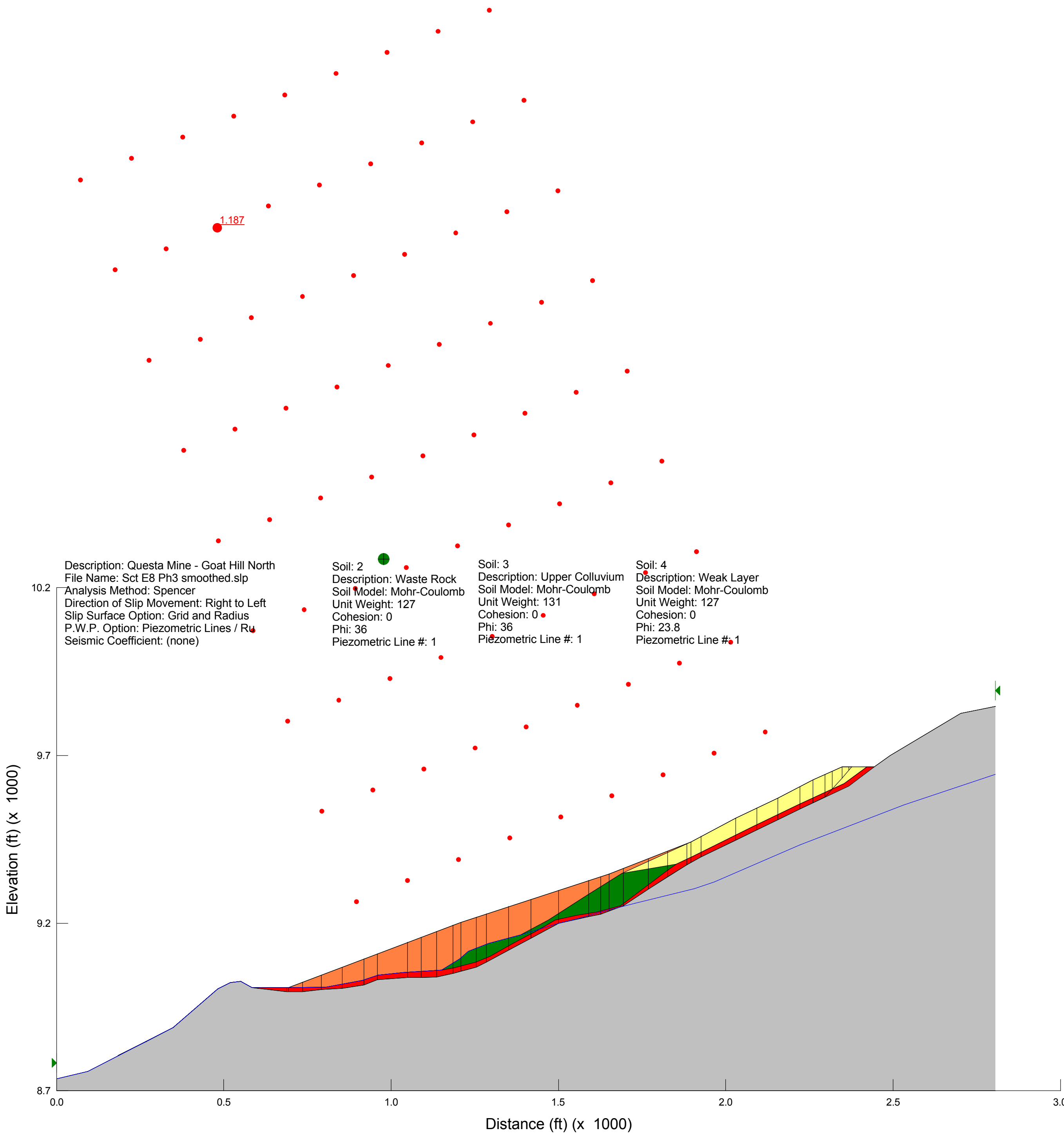
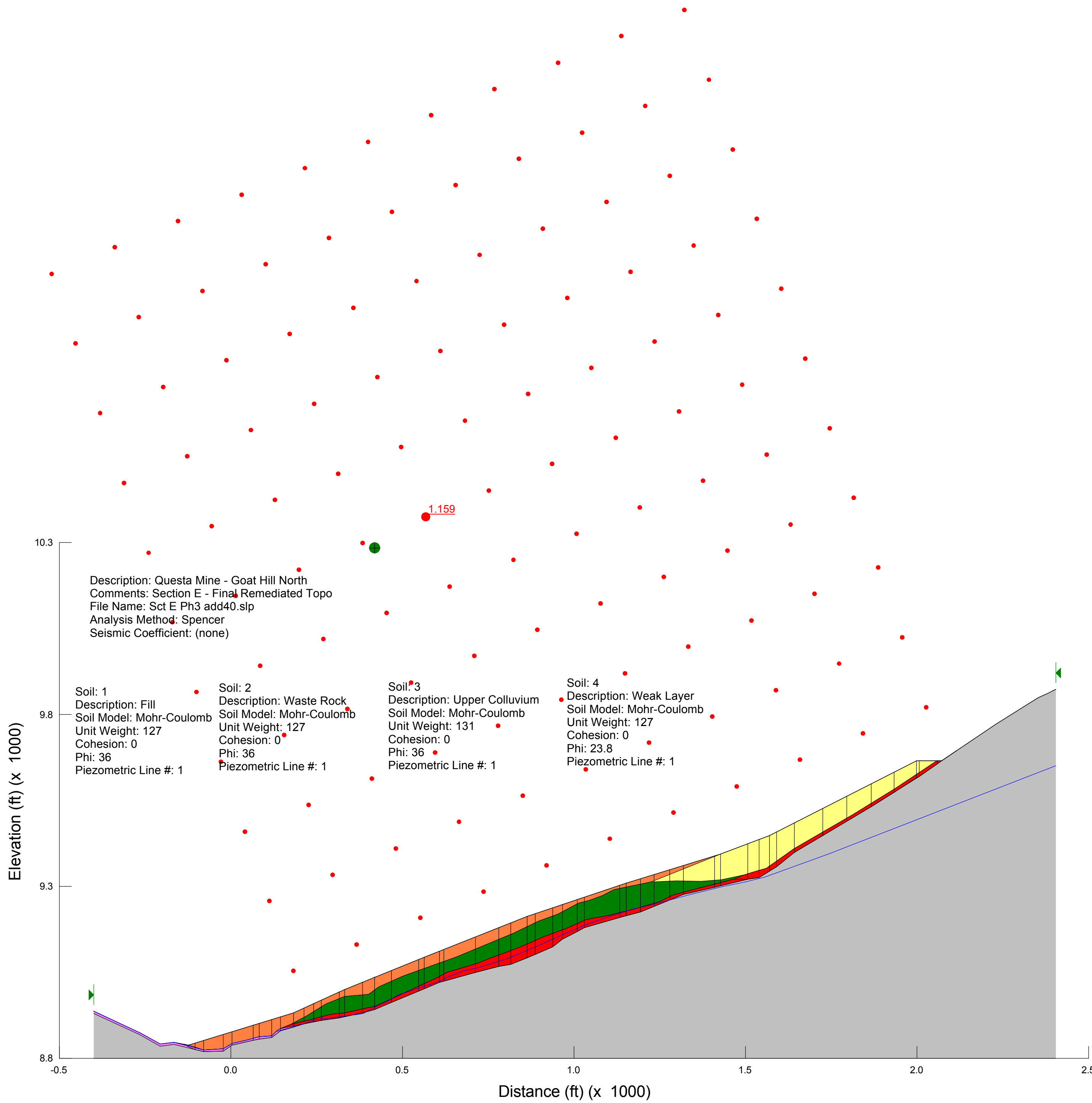
Description: Weak Layer
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 23.8
Piezometric Line #: 1
Elevation (ft) (x 1000)
10.2
9.7
9.2
8.7
0.0
0.5
1.0
1.5
2.0
2.5
3.0
Distance (ft) (x 1000)

000291



1.187
Description: Questa Mine - Goat Hill North
File Name: Sct E8 Ph3 smoothed.slp
Analysis Method: Spencer
Direction of Slip Movement: Right to Left
Slip Surface Option: Grid and Radius
P.W.P. Option: Piezometric Lines / Ru
Seismic Coefficient: (none)
Soil: 2
Description: Waste Rock
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 3
Description: Upper Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 4
Description: Weak Layer
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 23.8
Piezometric Line #: 1
Elevation (ft) (x 1000)
10.2
9.7
9.2
8.7
0.0
0.5
1.0
1.5
2.0
2.5
3.0
Distance (ft) (x 1000)

000292


1.159
Description: Questa Mine - Goat Hill North
Comments: Section E - Final Remediated Topo
File Name: Sct E Ph3 add40.slp
Analysis Method: Spencer
Seismic Coefficient: (none)
Soil: 1
Description: Fill
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 2
Description: Waste Rock
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 3
Description: Upper Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 4
Description: Weak Layer
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 23.8
Piezometric Line #: 1
10.3
9.8
9.3
8.8
Elevation (ft) (x 1000)
-0.5
0.0
0.5
1.0
1.5
2.0
2.5
Distance (ft) (x 1000)

000293

NORWEST CORPORATION

Appendix F

Monitoring

Appendix F Monitoring

Goathill North and Capulin Piezometers, Summary of Methods, Data and Status: Golder Associates: May 18, 2004

Borehole Inclinometer Data: May 27, 2004

Golder Associates Inc.
44 Union Boulevard, Suite 300
Lakewood, CO USA 80228
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GOATHILL NORTH AND CAPULIN PIEZOMETERS, SUMMARY OF METHODS, DATA, AND STATUS

Submitted to:

Molycorp Inc.
P.O. Box 469
Questa, NM 87556-0469

Submitted by:

Golder Associates Inc.
44 Union Blvd., Suite 300
Lakewood, Colorado 80228

May 18, 2004 033-2095

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1.0 INTRODUCTION

In the summer of 2003, a program of subsurface investigation, including the placement of piezometers and inclinometers, was completed at the Goathill North rock pile to support site evaluation and stability design studies. As part of this program, 39 electronic, vibrating-wire piezometers were installed in drill holes, at various depths, to allow monitoring of piezometric levels in the rock pile, and underlying colluvium and bedrock.

This summary document describes the installation of these instruments, data collection and processing, and how the data is used as part of the Goathill North Emergency Action Plan. Failure of a several instruments, which has occurred since their installation, is also evaluated.

Drawing 1 presents a map of the Goathill North and Capulin rock pile areas, showing locations of the drill holes and instrumentation. Table 1 presents a summary of the drill holes and instrument placements.

2.0 INSTALLATION

The piezometers were installed at selected locations in the boreholes for the purpose of monitoring piezometric levels within the various stratagraphic units (rock pile, colluvium, bedrock). In general, the piezometer placements were selected to accomplish the following:

- Monitor piezometric levels within the lower portion of the rock pile. A key purpose of many of these piezometers is to provide an indication of rising water levels in the rock pile, should this occur.

- Monitor piezometric conditions within the colluvium, which was interpreted to include zones of shearing related to movement of the existing landslide.

- Measure and monitor piezometric levels within the underlying bedrock, to achieve an understanding of the hydrologic interaction of the upper units with the bedrock groundwater system.

At each of the 2003 piezometer installations, the sensor unit was placed at the desired depth in the drill hole, and within an interval of sand backfill. Some of the units were also placed within a sand-filled “sock”. Some of the deeper piezometers were placed in the drill hole below the bottom of the SI (inclinometer) casings, installed concurrently with the piezometers. Piezometers within the interval of the SI casings were placed in the same hole after the casing was installed as the backfill was brought up around casing to the desired piezometer level. In some instances it was noted that the backfill would settle and pull the casing downward, sometimes several feet.

The sand backfill interval in each case comprises the zone which is measured by the sensor. The sand backfill was isolated within the drill hole by bentonite plugs. The remainder of the drill hole was backfilled with grouted pea gravel (pea gravel was first used to backfill the annulus, and grout was then washed into the gravel). The cored portions of the hole, in bedrock, were filled with grout only. The nature of these backfills is relatively rigid, and could allow shearing of the instrument cables in response to long term backfill settlement or ground movements.

The piezometers in drill holes TH-GN-09, TH-GN-11, and TH-GN-16 were installed directly in the grout backfill, without sand packs. Each of these sensors was taped to the SI casing for placement in the hole. This practice is endorsed by the manufacturer, and these units appear to be functioning properly.

Each of the SI casings has a protective steel outer casing, and these were finished with concrete pads surrounding the casing at the ground surface. Gravel was placed in the annulus between steel casing and SI casing above ground to stabilize the upper portion of the SI casing. The piezometer cables exit the drill hole inside the steel protective casing. Wring for the piezometers to the data collection system is accomplished through cables suspended on steel posts, converging at the instrumentation shed on the crest of the rock pile. At the instrumentation shed, the individual cables are wired into corresponding terminals of the data collection hardware. Manual readings can also be taken at each of the piezometers by connecting the manual readout unit leads to the corresponding individual wires.

3.0 DATA COLLECTION AND REDUCTION

Each piezometer is connected by cable to a CR10X datalogger installed in a weatherproof shed located on top of the Goathill North waste rock pile. Readings are recorded every 15 minutes to a hard drive also located in the shed. The data is saved as a text file with a .dat extension. This data file also contains readings from all the other instruments connected to the CR10X datalogger.

Readings can be monitored in "real time" using a computer in the Admin building running the MCQ_Rainmon program, and also on several computers in the Dry building that have Proxy Master installed. Proxy Master allows the computers to access the MCQ_Rainmon program on the computer in the Admin building. The "real time" data displayed is actually read and the screen refreshed every two minutes.

The data file on the hard drive in the shed can be transferred to any hard drive on the local network using a computer that has access to the MCQ_Rainmon program. The data file is usually stored on Bobby Kehrman's computer at c:\Documents and Settings\rmkehrm\My Documents\Goathill North Investigation Project\Instrumentation\Datalogger Download Archive\YYMMDD download, where YYMMDD is the date of the download. To reduce the file size, the .dat file is split into arrays using the PC200W program, which creates a comma delimited (.csv) text file for each array. The text file containing the piezometer data is named SMSENSORYYMMDDdl101.csv, where YYMMDD is the download date.

The SMSENSORYYMMDDdl101.csv file is opened in Excel, and any new data is copied and pasted into another Excel spreadsheet named 101_Import_SheetYYMMDD.xls. This spreadsheet is designed to facilitate importing the data into Access, where all the piezometer data is stored.

After the 101_Import_SheetYYMMDD.xls file is imported into Access, a macro is run that appends the data to the data tables. If any functioning piezometers are not reporting to the CR10X datalogger, they are read manually using a VW Data Recorder, and the readings are typed into Access manually. After updating is completed, another macro is run that calculates the downhole depths of the piezometric surfaces. The Access database also contains automatically updating plots of the piezometers that are hooked up to the alarm system. The calculations performed by Access are included in Appendix A.

4.0 CORRECTIONS TO DATA

Temperature and elevation corrections are applied to the piezometer readings during calculation of the downhole piezometric surface depths. The equations used for these corrections are included in Appendix A.

5.0 DATA SUMMARY

Appendix B presents data plots for each of the piezometers. Data for multiple piezometers in the same hole are plotted together on the same chart. The data presented include readings taken through April 28, 2004.

Table 2 presents a summary of the most recent piezometric readings (most for April 28, 2004).

A number of the sensors indicate zero head, or a "dry" condition at the elevation of the sensor. These conditions are found in each of the stratigraphic horizons; bedrock, colluvium, and rock pile. Most of these occur in boreholes drilled from the crest areas of the rock piles, but a few cases also occur near the toe of the Goathill North rock pile at depths less than 70 feet.

Several of the sensors indicate negative pressures. TH-C-04-340 indicates a relatively high negative pressure, on the order of 6 feet, which may indicate a sensor malfunction or initial calibration error. However, two of the sensors have been consistently recording negative pressures on the order of one or two feet. These two piezometers are situated within 10 feet below the top of bedrock. These sensors may be recording true negative pressures induced by downward gradients near the top of the bedrock.

Where multiple piezometers occur at a single borehole location, they generally indicate a downward gradient. One piezometer located in bedrock appears to indicate an upward gradient into the rock pile. The piezometer at TH-C-04-282 is located at the base of the colluvium, immediately above the bedrock contact. This borehole appears to be located within a pre-mine drainageway that may be concentrating groundwater in this area. Also, the colluvium in this borehole was reported to be clay, and may be acting as a confining layer. Bedrock flows may be locally confined in this area due to the geometry of the valley and relative permeability of layers.

6.0 INSTRUMENT RELIABILITY

Several of the piezometers ceased working properly, or entirely, beginning in the late winter 2004. To date, none of the affected units are ones that are part of the EAP emergency monitoring/reporting system. However, failure of instruments has implications that must be addressed in order to maintain an effective monitoring system.

We have discussed the instrument failures with the manufacturer (Slope Indicator Company) to see what insight they might have for the problems. We discussed several possible causes:

- Deterioration/corrosion of the unit due to environmental factors at the depth of burial. The manufacturer agrees that this has been a problem for buried instruments on some projects.

- Damage to the electrical cables during installation, causing eventual shorting, corrosion, or breaking of wires as water enters.

- Damage to the electrical cables in response to settlement/shifting of the borehole backfill or surrounding materials following installation, particularly where angular rock particles are present. Installation of the cables through grout and grouted pea gravel backfill could encourage cable damage if ground movements occur, creating distinct breaks in the grouted backfill.

Table 3 summarizes the piezometers which appear to have been damaged. Only three of the piezometers appear permanently damaged, and these units are each located within the current slide movement area. Two of the three are deeper, placed in bedrock well below the rock pile. It makes sense that the deeper ones are the most exposed to "hazards" that might cause damage to the electrical cables - simply by virtue of their length. At this time, we cannot provide a certain assessment of why these instruments are experiencing problems. As with virtually all types of subsurface instrumentation, a certain degree of attrition and reduced performance may be expected over time and from the conditions inherent in installation. Given the subsurface movements that have occurred in within the slide area, the failure frequency of these instruments due to cable damage seems relatively low. The following paragraphs theorize the causes of failure based on what is know about the failure history.

In two cases (TH-GN-02-360 and TH-GN-04-72), the piezometer recorded unlikely trends prior to failure, and in these cases the failure may have been caused by failure of the pressure transducer.

Failure of TH-GN-02-360 may have been caused by actual heads exceeding the design range of the unit. This unit is still recording data, but reads as zero pressure. The other may have been failure of the transducer through deterioration or manufacturer defect.

The other failure (TH-GN-07-149) is attributed to a damaged cable, due to the sudden cessation of otherwise-normal readings. This damage could be from shifting borehole backfill, slide movement, or moisture penetration via damage experienced during installation.

The two other piezometers listed in the table were inoperative for a period of time. However, they appear to now be functioning normally. The temporary problems may have been due to glitches in the surface cable wiring system which have since been corrected.

One additional piezometer (TH-GN-08-365), not listed in the above table, has been inoperative since its installation, and this failure is attributed to damage incurred during installation. This piezometer was placed in bedrock near the bottom of the hole, but another piezometer (TH-GN-08-168) placed in bedrock within the same hole, is functional. The cable or transducer for TH-GN08-365 was probably damaged during installation.

7.0 METHODS FOR DAILY MONITORING

Automated Monitoring

“Real time” piezometer readings can be viewed at any time from any of the computers with access to MCQ_Rainmon. Each day, usually in the morning, the site instrumentation technician records the readings into a notebook and also types them into an Excel spreadsheet. If any of the piezometers are not reporting to the datalogger she reads them manually, provided she has access to the location. The Excel spreadsheet serves as a backup should any problems occur with the automated data recording system.

Manual Data Reduction Procedures

In the event that the automated data collection system, and computer calculation systems become inoperative, manual readings taken with the manual data readout unit can be hand calculated to obtain piezometric levels. Appendix A presents the methods, equations, calculation factors, and data corrections used for hand calculations. These are the same methods and values used in the automated data collection spreadsheet and database.

Automated Alarm System

Four of the piezometers on Capulin and six of the piezometers on the Goathill North waste rock piles are connected to an alarm system, as provided for in the Goathill North Emergency Action Plan (EAP). When water levels in the piezometers rise above a specified level, a warning is automatically broadcast over the mine radio system. This alarm is received by all personnel carrying mine radios on and off site, and by local fire and emergency response departments that also monitor the frequency.

The piezometers connected to the alarm system, and the alarm trigger levels, are presented in Table 4. Since the purpose of the piezometers included in the emergency monitoring is primarily to detect water levels within the rock pile material, the trigger levels were generally selected to be at or immediately above the base of the rock pile. In some cases the piezometer sensor is somewhat below the base of the rock pile, in underlying bedrock or colluvium, in which case the piezometric pressure measured at the sensor may or may not be directly related to piezometric levels within the rock pile.

However, these levels are conservatively interpreted to be indicative of potential rock pile conditions. In other cases, the piezometer sensor is within the rock pile at some height above the base of the rock pile. In these cases, the trigger level was selected to be nominally above the sensor elevation to accommodate variations in the pressure response of the piezometer unit at very low heads and to prevent non-significant fluctuations of groundwater pressures from creating a “false-trigger” situation.

The interim trigger levels, which are those currently listed in the EAP, were defined while the site investigation program was in progress in the summer of 2003. They were selected based on the best information available at the time, which included raw field records from the drilling and installation of instruments. These trigger levels and other emergency warning criteria are being updated as part of the planning for construction monitoring, based on the results of Norwest’s evaluation of rock pile behavior as described in the January 2004 Goathill North Final Report.

TABLES

TABLE 1
PIEZOMETER INSTALLATION DETAILS

Capulin Drillholes and Installations

test hole location name	**TH-C-01**		**TH-C-02**		**TH-C-03**		**TH-C-04**		**TH-C-05**	
inclinometer/instrumentation name	***(pre-2003)***		***(pre-2003)***		***SI-14***		***SI-15***		***SI-16***	
additional hole name										
Location										
northing (ft)	28,864.9		28,109.0		28,481.1		27,720.0		27,363.6	
easting (ft)	52,512.5		53,367.1		52,824.3		53,455.0		53,745.9	
ground elevation (ft)	9,809.7		9,800.0		9,794.2		9,775.0		9,790.5	
SI casing height (stick-up ft)	3.0		3.7		2.1		1.7		2.8	
top of SI casing elevation (ft)	9,812.7		9,803.7		9,796.3		9,776.7		9,793.3	
Perforation	*depth*	*elevation*	*depth*	*elevation*	*depth*	*elevation*	*depth*	*elevation*	*depth*	*elevation*
Intercept Summary										
depth to rock pile bottom (ft)	137.0	**9,672.7**	185.0	**9,615.0**	180.5	**9,613.7**	263.0	**9,512.0**	67.5	**9,723.0**
depth to top of bedrock (ft)	137.0	**9,672.7**	185.0	**9,615.0**	186.0	**9,608.2**	282.3	**9,492.7**	73.0	**9,717.5**
total depth of borehole (ft)	150.0	**9,659.7**	207.0	**9,593.0**	224.5	**9,569.7**	340.3	**9,434.7**	173.5	**9,617.0**
thickness of rock pile (ft)	137		185		181		263		68	
thickness of colluvium/soil/alt. bedrock (ft)	0		0		6		19		6	
thickness of bedrock interval drilled (ft)	13		22		39		58		101	
Instrumentation Installed										
Inclinometer casing										
A_0-axis azimuth (°)										
top of casing (ft)	-3.0	**9,812.7**	-3.7	**9,803.7**	-2.1	**9,796.3**	-1.7	**9,776.7**	-2.8	**9,793.3**
bottom of casing (ft)					221.0	**9,573.2**	328.8	**9,446.2**	95.0	**9,695.5**
Piezometer - rock pile										
depth / elevation (ft)	141.0	**9,668.7**			150.6	**9,643.6**	260.0	**9,515.0**	67.5	**9,723.0**
type, serial #	77238				77234		77333		77230	
Piezometer - colluvium/soil										
depth / elevation (ft)							282.0	**9,493.0**		
type, serial #							77232			
Piezometer - bedrock										
depth / elevation (ft)			200.0	**9,600.0**			339.0	**9,436.0**	171.8	**9,618.7**
type, serial #			77239				77228		77236	
Piezometer - other										
depth / elevation (ft)										
type, serial #										
Notes	previous study, piezometer installed in previous open standpipe		previous study, piezometer installed in previous open standpipe						piezo at 171.8 ft. not functioning properly since February 2004	

TABLE 1
PPIEZOMETER INSTALLATION DETAILS

Goat Hill N Drillholes and Installations

test hole location name	**TH-GN-01**		**TH-GN-02**		**TH-GN-03**		**TH-GN-04**		**TH-GN-05**		**TH-GN-06**		**SI-03**		**TH-GN-07**		**TH-GN-08**	
inclinometer/instrumentation name	***(pre-2003)***		***SI-08***		***SI-09***		***SI-10***		***SI-11***		***SI-12***		***(pre-2003)***		***07S / SI-03B***		***SI-13***	
additional hole name															**TH-GN-07P**			
Location																		
northing (ft)	26,295.4		26,592.2		26,240.9		26,346.2		26,111.3		26,163.3		26,360.4		26,380.9		26,209.7	
easting (ft)	54,303.0		53,746.5		53,430.1		53,280.7		52,777.2		54,322.4		53,534.1		53,529.3		53,886.9	
ground elevation (ft)	9,765.4		9,455.7		9,253.8		9,256.4		9,073.0		9,765.7		9,328.5		9,327.1		9,474.9	
SI casing height (stick-up ft)	3.0		3.0		3.0		2.1		2.2		2.6		3.0		2.5		2.2	
top of SI casing elevation (ft)	9,768.4		9,458.7		9,256.8		9,258.5		9,075.2		9,768.3		9,331.5		9,330.1		9,477.1	
Perforation	*depth*	*elevation*	*depth*	*elevation*	*depth*	*elevation*	*depth*	*elevation*	*depth*	*elevation*	*depth*	*elevation*	*depth*	*elevation*	*depth*	*elevation*	*depth*	*elevation*
Intercept Summary																		
depth to rock pile bottom (ft)	180.0	**9,585.4**	129.0	**9,326.7**	33.0	**9,220.8**	20.5	**9,235.9**	0.0	**9,073.0**	232.0	**9,533.7**	76.0	**9,252.5**	62.5	**9,264.6**	160.5	**9,314.4**
depth to top of bedrock (ft)	180.0	**9,585.4**	142.0	**9,313.7**	52.3	**9,201.5**	81.0	**9,175.4**	76.5	**8,996.5**	241.0	**9,524.7**	84.0	**9,244.5**	93.5	**9,233.6**	160.5	**9,314.4**
total depth of borehole (ft)	204.5	**9,560.9**	364.0	**9,091.7**	93.5	**9,160.3**	126.0	**9,130.4**	212.5	**8,860.5**	307.0	**9,458.7**	104.0	**9,224.5**	172.0	**9,155.1**	373.2	**9,101.7**
thickness of rock pile (ft)	180		129		33		21		0		232		76		63		161	
thickness of colluvium/soil/alt. bedrock (ft)	0		13		19		61		77		9		8		31		0	
thickness of bedrock interval drilled (ft)	25		222		41		45		136		66		20		79		213	
Instrumentation Installed																		
Inclinometer casing																		
A_0-axis azimuth (°)			230		235		220		220		235		235		245		235	
top of casing (ft)			-3.0	**9,458.7**	-3.0	**9,256.8**	-2.1	**9,258.5**	-2.2	**9,075.2**	-2.6	**9,768.3**	-3.0	**9,331.5**	-2.5	**9,329.6**		
bottom of casing (ft)			157.0	**9,298.7**	81.0	**9,172.8**	115.0	**9,141.4**	202.0	**8,871.0**	287.0	**9,478.7**			134.5	**9,192.6**		
Piezometer - rock pile																		
depth / elevation (ft)											231.0	**9,534.7**			59.0	**9,268.1**		
type, serial #											77241				77229			
Piezometer - colluvium/soil																		
depth / elevation (ft)			138.5	**9,317.2**	50.0	**9,203.8**	72.0	**9,184.4**	76.1	**8,996.9**					93.0	**9,234.1**		
type, serial #			77426		77437		77237		77436						77233			
Piezometer - bedrock																		
depth / elevation (ft)	204.5	**9,560.9**	360.0	**9,095.7**	89.0	**9,164.8**	85.0	**9,171.4**	207.4	**8,865.6**	244.0	**9,521.7**			148.5	**9,178.6**	168.5	**9,306.4**
type, serial #	77240		77427		77425		77231		77334		77235				77227		77332	
Piezometer - other																		
depth / elevation (ft)							124.5	**9,131.9**									365.2	**9,109.7**
type, serial #							77226										77335	
Notes	previous study, piezometer installed in previous open standpipe		piezo at 360 ft. not functioning properly since December 2003				piezo at 72 ft. not functioning properly since January 2004		piezo at 210 ft. not functioning properly since January 2004				previous study, no piezometer		piezo at 93 ft. not functioning properly since January 2004		piezo at 365.2 has not functioned since installation	

000310

TABLE 1
PPIEZOMETER INSTALLATION DETAILS

test hole location name	**TH-GN-09**		**TH-GN-10**		**TH-GN-11**		**TH-GN-12**		**TH-GN-13**		**TH-GN-14**		**TH-GN-15**		**TH-GN-16**	
inclinometer/instrumentation name	***SI-17***		***SI-18***		***SI-19***		***SI-20***		***SI-21***		***SI-22***		***SI-23***		***SI-24***	
additional hole name																
Location																
northing (ft)	26,098.2		25,969.4		26,250.3		26,536.0		26,281.0		26,416.2		25,979.4		26,136.3	
easting (ft)	53,231.4		52,773.7		52,867.5		53,220.4		52,964.9		53,088.3		52,656.9		52,972.9	
ground elevation (ft)	9,140.5		8,978.1		9,148.3		9,277.1		9,165.0		9,240.8		8,968.5		9,090.1	
SI casing height (stick-up ft)	2.9		2.5		3.1		2.2		3.1		2.8		3.2		3.4	
top of SI casing elevation (ft)	9,143.5		8,980.6		9,151.4		9,279.3		9,168.1		9,243.6		8,971.7		9,093.5	
Perforation	*depth*	*elevation*	*depth*	*elevation*	*depth*	*elevation*	*depth*	*elevation*	*depth*	*elevation*	*depth*	*elevation*	*depth*	*elevation*	*depth*	*elevation*
Intercept Summary																
depth to rock pile bottom (ft)	128.7	**9,011.8**	0.0	**8,978.1**	0.0	**9,148.3**	0.0	**9,277.1**	0.0	**9,165.0**	0.0	**9,240.8**	0.0	**8,968.5**	0.0	**9,090.1**
depth to top of bedrock (ft)	128.7	**9,011.8**	52.0	**8,926.1**	122.0	**9,026.3**	84.0	**9,193.1**	44.5	**9,120.5**	96.5	**9,144.3**	37.5	**8,931.0**	48.0	**9,042.1**
total depth of borehole (ft)	143.5	**8,997.0**	122.5	**8,855.6**	165.0	**8,983.3**	193.0	**9,084.1**	128.0	**9,037.0**	138.5	**9,102.3**	78.0	**8,890.5**	73.5	**9,016.6**
thickness of rock pile (ft)	129		0		0		0		0		0		0		0	
thickness of colluvium/soil/alt. bedrock (ft)	58		52		122		84		45		97		38		48	
thickness of bedrock interval drilled (ft)	15		71		43		109		84		42		41		26	
Instrumentation Installed																
Inclinometer casing																
A_0-axis azimuth (°)	225		210		195		225		190		160		230		210	
top of casing (ft)	-2.9	**9,143.4**	-2.5	**8,980.6**	-3.1	**9,151.4**	-2.2	**9,279.3**	-3.1	**9,168.1**	-2.8	**9,243.6**	-3.2	**8,971.7**	-3.4	**9,093.5**
bottom of casing (ft)	113.5	**9,027.0**	108.0	**8,870.1**	163.0	8,985.3	153.0	**9,124.1**	115.0	**9,050.0**	118.5	**9,122.3**	78.0	**8,890.5**	73.0	**9,017.1**
Piezometer - rock pile																
depth / elevation (ft)	67.0	**9,073.5**														
type, serial #	77498															
Piezometer - colluvium/soil																
depth / elevation (ft)			51.0	**8,927.1**	122.0	**9,026.3**	83.0	**9,194.1**			95.0	**9,145.8**			47.0	**9,043.1**
type, serial #			77430		77483		77433				77428				77493	
Piezometer - bedrock																
depth / elevation (ft)			114.0	**8,864.1**	162.0	**8,986.3**	131.0	**9,146.1**	98.0	**9,067.0**	137.0	**9,103.8**			73.0	**9,017.1**
type, serial #			77429		77490		77432		77431		77435				77496	
Piezometer - other																
depth / elevation (ft)							189.0	**9,088.1**	127.0	**9,038.0**						
type, serial #							77434		77424							
Notes																

TABLE 2

PIEZOMETER STATUS SUMMARY APRIL 2004

Piezometer	Measured Piezo Level Below Ground	Stratigraphic Location of Piezometer Sensor	Stratigraphic Location of Piezometric Surface	Indicates Negative Pressure	Trigger Level	Comments
TH-C-01-143	140.7	Bedrock	Dry		140	
TH-C-02-201	199.4	Bedrock	Dry		199	
TH-C-03-151	147.9	Rock pile	Rock pile		NA	
TH-C-04-260	259.9	Rock pile	Dry		258	
TH-C-04-282	237.2	Colluvium	Rock pile		NA	
TH-C-04-340	345.5	Bedrock	Bedrock	√	NA	Indicates unrealistically high negative pressure
TH-C-05-67	64.3	Base of Rock pile	Rock pile		66	Has exceeded trigger level since March 2004
TH-C-05-172	162.8	Bedrock	Bedrock		NA	
TH-GN-01	204.1	Bedrock	Dry		198	
TH-GN-02-139	134.3	Colluvium	Colluvium		128.5	
TH-GN-02-360	389.9	Bedrock	Bedrock		NA	Not functioning properly since January 2003
TH-GN-03-50	49.3	Colluvium	Dry		40	
TH-GN-03-89	75.3	Bedrock	Bedrock		NA	
TH-GN-04-72	66.3	Colluvium	Colluvium		NA	Not functioning properly since January 2003
TH-GN-04-85	87.2	Bedrock	Bedrock	√	NA	
TH-GN-04-125	96.7	Bedrock	Bedrock		NA	
TH-GN-05-76	76.0	Base of Colluvium	Dry		NA	
TH-GN-05-210	126.9	Bedrock	Bedrock		NA	
TH-GN-06-231	230.3	Colluvium	Dry		229	
TH-GN-06-244	243.5	Bedrock	Dry		NA	
TH-GN-07-59	58.8	Rock pile	Dry		56	
TH-GN-07-93	84.7	Colluvium	Colluvium		NA	
TH-GN-07-149	138.8	Bedrock	Bedrock		NA	Not functioning properly since January 2003
TH-GN-08-168.5	169.8	Bedrock	Bedrock	√	167.5	
TH-GN-09	67.2	Rock pile	Dry		NA	
TH-GN-10-51	50.7	Colluvium	Dry		NA	
TH-GN-10-114	66.3	Bedrock	Bedrock		NA	
TH-GN-11-122	103.8	Top of Bedrock	Bedrock		NA	
TH-GN-11-162	103.6	Bedrock	Bedrock		NA	
TH-GN-12-83	55.9	Colluvium	Colluvium		NA	
TH-GN-12-131	81.5	Bedrock	Bedrock		NA	
TH-GN-12-189	119.5	Bedrock	Bedrock		NA	
TH-GN-13-98	79.7	Bedrock	Bedrock		NA	
TH-GN-13-127	116.7	Bedrock	Bedrock		NA	
TH-GN-14-95	87.3	Colluvium	Colluvium		NA	
TH-GN-14-137	98.0	Bedrock	Bedrock		NA	
TH-GN-16-47	45.0	Colluvium	Colluvium		NA	
TH-GN-16-73	57.5	Bedrock	Bedrock		NA	

TABLE 3

SUMMARY OF COMPROMISED PIEZOMETERS

Piezometer Number	Depth	Piezo Type	Stratigraphic Location	Installation Details	Installation Date	Failure History	Probable Cause of Failure
TH-C-05-172	171.8	50 psi	In bedrock near bottom of hole	Near bottom of hole, in sand sock in sand backfill, 77 feet below bottom of SI casing through 71 feet of bentonite pellet backfill.	8/2/03	Indicated slowly declining head about 10 feet above piezo, then went suddenly to zero head 2/20/04	Appears to now be working normally, possible surface cable problem
TH-GN-02-360, (previously called TH-GN-02-330)	360	50 psi	In bedrock near bottom of hole	Near bottom of hole, in 7 ft. sand backfill, through pea gravel backfill.	9/22/03	Declining head about 180 feet above piezo until 12/10/03, then "offset" to higher level followed by sharp decline	Historic head (>>50 psi) exceeds rating of piezometer, pressure transducer likely damaged. Instrument still recording data "zero".
TH-GN-04-72	72	50 psi	In colluvium	Midway in hole, in sand backfill around SI casing, hole above backfilled with alternating pea gravel and cement grout.	8/12/03	Relatively stable at a few feet of head above piezo until 1/16/04, then rising sharply to failure.	Possible transducer failure. This area is also within the slide movement area, and failure could be cable shearing.
TH-GN-05-210	207.4	100 psi	In bedrock near bottom of hole	Near bottom of hole, in sand backfill, about 5 feet below bottom of SI casing through sand backfill and bentonite plug. Upper 202 feet of hole was backfilled with pea gravel and grout.	9/6/03	Slightly declining at about 80 feet of head above piezo until 1/21/04, then not recording until 3/1/04. Now appears to be recording normally.	Appears to now be working normally, possible surface cable problem.
TH-GN-07-149	148.5	100 psi	In bedrock near bottom of hole	In sand backfill, below SI casing in cored portion of hole with grout backfill, hole above core interval backfilled with pea gravel and cement grout.	8/19/03	Relatively steady readings after initial stabilization until sudden failure.	Failure probably due to cable damage. This installation is in known movement area of slide.

TABLE 4

PIEZOMETER TRIGGER DEPTHS FOR EAP ALARM SYSTEM

Piezometer	Location	*Piezometer Depth (ft)	*Depth to Bottom of Rock Pile (ft)	Trigger Depth (ft)	Trigger Head (ft)
	Capulin Rock Pile				
TH-C-01	North crest area	141	137.0	140.0	1.0
TH-C-02	Central crest area	200	185.0	199.0	1.0
TH-C-04 (260)	South crest area	260	263.0	258.0	2.0
TH-C-05 (67)	Ridge between Capulin and GHN crests	67.5	67.5	66.0	1.5
	Goathill North Rock Pile				
TH-GN-01	Central crest area	203.0	180.0	198.0	5.0
TH-GN-02 (139)	Central portion of slope in slide area	138.5	128.5	128.5	10.0
TH-GN-03 (50)	Toe of rock pile in slide area	50.0	40.0	40.0	10.0
TH-GN-06 (231)	Crest of stable portion of rock pile	231.0	241.0	229.0	2.0
TH-GN-07 (59)	Lower slide area of rock pile	59.0	56.0	56.0	3.0
TH-GN-08 (168.5)	Central portion of slope in stable (South) area	168.5	169.0	167.5	1.0

*Referenced from original measured ground elevation at borehole collar

DRAWINGS

APPENDIX A

CALCULATION OF PIEZOMETRIC LEVELS

Appendix A

Piezometer Data Reduction Performed in Microsoft Access

The CR10X datalogger records piezometer frequency readings in kHz. When the Append Data macro is run, Access multiplies the frequency readings by 1000 to convert them to Hz, and also screens out unreasonable frequency and temperature readings.

Access uses the following equation to calculate the downhole depths of the piezometric water levels in feet:

$$\text{Depth} = \text{SD}-2.3067*\{(Ax^2 + Bx + C) + [m*(T - \text{Toffset}) + b] + \text{elevation correction}\}$$

where,

SD = sensor depth in feet
A, B, C, m, Toffset, and b are calibration constants provided by the manufacturer
x is the frequency reading in Hz
T is the temperature reading in degrees C.

The portion of the equation in braces is multiplied by 2.3067 to convert units of pressure head to feet of head. The term in brackets is the temperature correction. The elevation correction is calculated using the following equation:

$$\text{elevation correction} = 14.696*[1-(1-(E/3.2808)/44307.69231]^{5.5328}$$

where E is the sensor elevation in feet.

Calibration constants for each of the piezometers are included in Table A-1.

Manual Calculation of Piezometric Water Levels

The same equations presented above can be used to hand-calculate the piezometric head for any given piezometer. The input required is the raw instrument readings; frequency (expressed in Hz) and temperature in degrees C. The calculation factors are taken from table A-1.

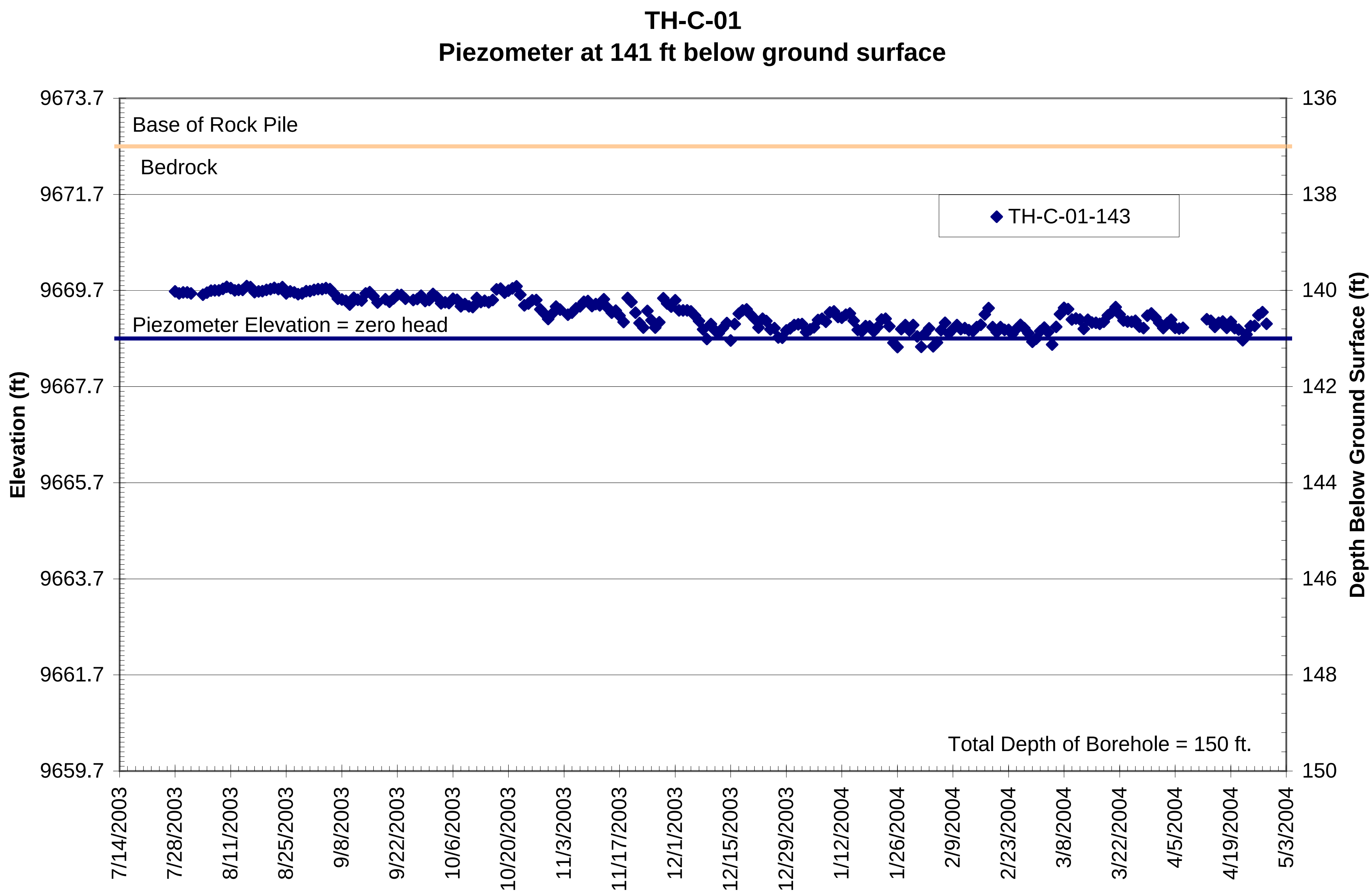
TABLE A-1

PIEZOMETER CALIBRATION CONSTANTS

SN	Range	Installation Information			Manual ABC Factors			Temperature			Elevation	
		Borehole	Depth	Ground Elevation	A (psi)	B (psi)	C (psi)	m (psi/°C)	b (psi)	Offset (°C)	Piezo Elevation	Elevation Correction
77226	100 psi	TH-GN-04S	125.0	9255.0	-0.000024768	0.0086443	198.98	-0.0182	0.428	-0.5	9130'	4.43
77227	100 psi	TH-GN-07S	148.5	9327.9	-0.000029530	0.0139590	222.84	-0.0188	0.444	-0.4	9179'	4.45
77228	50 psi	TH-C-04S	340.3	9770.0	-0.000028972	0.0113300	212.49	-0.0102	0.250	-0.3	9430'	4.56
77229	50 psi	TH-GN-07S	59.0	9327.9	-0.000018444	0.0037381	140.33	-0.0089	0.212	-0.4	9269'	4.49
77230	50 psi	TH-C-05S	66.5	9790.3	-0.000017094	0.0018204	141.06	-0.0154	0.411	-0.3	9724'	4.68
77231	50 psi	TH-GN-04S	85.0	9255.0	-0.000015453	-0.0077358	160.27	-0.0287	0.742	-0.1	9170'	4.45
77232	50 psi	TH-C-04S	282.0	9770.0	-0.000017182	-0.0022795	148.56	-0.0235	0.611	-0.5	9488'	4.58
77233	50 psi	TH-GN-07S	93.0	9327.9	-0.000018249	0.0032547	147.95	-0.0164	0.421	-0.3	9235'	4.48
77234	50 psi	TH-C-03S	151.0	9775.0	-0.000016676	-0.0053321	163.64	-0.0241	0.569	-0.5	9624'	4.64
77235	50 psi	TH-GN-06S	244.0	9765.3	-0.000014588	-0.0031434	136.39	-0.0254	0.599	-0.3	9521'	4.59
77236	50 psi	TH-C-05S	171.8	9790.3	-0.000014673	-0.0024890	135.28	-0.0298	0.698	-0.4	9619'	4.63
77237	50 psi	TH-GN-04S	72.0	9255.0	-0.000015928	-0.0018973	141.40	-0.0275	0.658	-0.1	9183'	4.45
77238	50 psi	TH-C-01	143.0	9809.7	-0.000015133	-0.0040365	143.04	-0.0260	0.603	-0.4	9667'	4.65
77239	50 psi	TH-C-02	201.0	9799.6	-0.000018039	0.0047753	144.35	-0.0204	0.490	-0.4	9599'	4.63
77240	50 psi	TH-GN-01	203.0	9765.4	-0.000014873	-0.0032412	140.53	-0.0228	0.548	-0.5	9562'	4.61
77241	50 psi	TH-GN-06S	231.0	9765.3	-0.000016356	0.0005781	141.84	-0.0395	0.928	-0.2	9534'	4.60
77332	100 psi	TH-GN-08	168.5	9477.2	-0.000028912	0.0162200	217.83	0.0227	-0.519	-0.5	9309'	4.51
77333	100 psi	TH-C-04S	260.0	9770.0	-0.000027617	0.0024178	256.62	0.0020	-0.046	-0.4	9510'	4.59
77334	100 psi	TH-GN-05	210.0	9073.0	-0.000027008	0.0140260	220.28	0.0003	-0.027	-0.4	8863'	4.32
77335	100 psi	LOST IN TH-GN-08	365.0	9477.2	-0.000026146	0.0029704	251.94	0.0125	-0.254	-0.5	9112'	4.42
77424	50 psi	TH-GN-13	127.0	9165.0	-0.000018630	0.0086074	152.39	0.0131	-0.301	-0.4	9038'	4.39
77425	50 psi	TH-GN-03	89.0	9253.8	-0.000019468	0.0094410	162.04	0.0290	-0.642	-0.5	9165'	4.45
77426	50 psi	TH-GN-02	138.5	9455.7	-0.000018952	0.0067931	159.76	0.0245	-0.536	-0.2	9317'	4.51
77427	50 psi	TH-GN-02	330.0	9455.7	-0.000018829	0.0081705	155.92	0.0193	-0.441	-0.4	9126'	4.43
77428	50 psi	TH-GN-14	95.0	9240.5	-0.000019224	0.0104870	156.13	0.0179	-0.385	-0.4	9146'	4.44
77429	50 psi	TH-GN-10	114.0	8978.1	-0.000019644	0.0107460	153.46	0.0207	-0.463	-0.5	8864'	4.32
77430	50 psi	TH-GN-10	51.0	8978.1	-0.000020318	0.0123380	154.65	0.0138	-0.307	-0.5	8927'	4.35
77431	50 psi	TH-GN-13	98.0	9165.0	-0.000018587	0.0071629	153.72	0.0260	-0.586	-0.5	9067'	4.41
77432	50 psi	TH-GN-12	131.0	9277.1	-0.000017071	0.0048453	139.17	0.0271	-0.613	-0.4	9146'	4.44
77433	50 psi	TH-GN-12	83.0	9277.1	-0.000018816	0.0069446	155.85	0.0203	-0.454	-0.3	9194'	4.46
77434	50 psi	TH-GN-12	189.0	9277.1	-0.000019142	0.0022643	176.67	0.0177	-0.396	-0.5	9088'	4.41
77435	50 psi	TH-GN-14	137.0	9240.5	-0.000018686	0.0071410	158.49	0.0139	-0.309	-0.2	9104'	4.42
77436	50 psi	TH-GN-05	76.0	9073.0	-0.000018729	0.0029210	171.80	0.0150	-0.327	-0.4	8997'	4.38
77437	50 psi	TH-GN-03	50.0	9253.8	-0.000019496	0.0083661	165.54	0.0173	-0.371	-0.1	9204'	4.46
77483	50 psi	TH-GN-11	122.0	9148.3	-0.000019485	0.0109060	152.15	0.0153	-0.324	-0.3	9026'	4.39
77490	50 psi	TH-GN-11	162.0	9148.3	-0.000017939	0.0082781	145.30	0.0174	-0.387	-0.4	8986'	4.37
77493	50 psi	TH-GN-16	47.0	9043.1	-0.000019726	0.0090145	159.91	0.0203	-0.452	-0.4	8996.1	4.38
77496	50 psi	TH-GN-16	73.0	9017.1	-0.000020111	0.0083958	164.52	0.0225	-0.490	0.6	8944.1	4.35
77498	50 psi	TH-GN-09	67.0	9140.5	-0.000020327	0.0105300	161.23	0.0247	-0.558	-0.4	9073.5	4.41

APPENDIX B

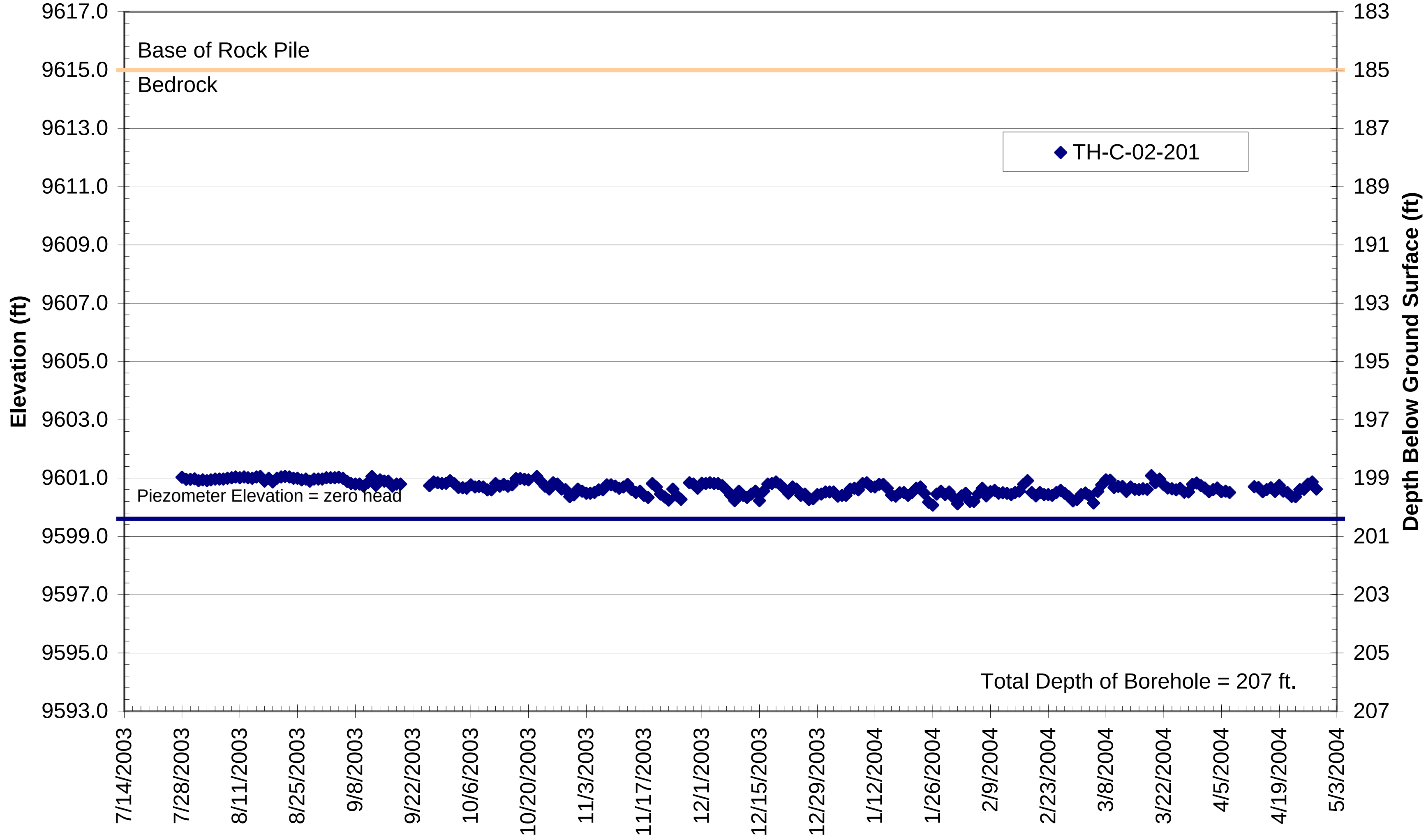
PIEZOMETER DATA PLOTS



TH-C-01
Piezometer at 141 ft below ground surface
Base of Rock Pile
Bedrock
TH-C-01-143
Piezometer Elevation = zero head
Total Depth of Borehole = 150 ft.
Elevation (ft)
Depth Below Ground Surface (ft)
9673.7
9671.7
9669.7
9667.7
9665.7
9663.7
9661.7
9659.7
136
138
140
142
144
146
148
150
7/14/2003
7/28/2003
8/11/2003
8/25/2003
9/8/2003
9/22/2003
10/6/2003
10/20/2003
11/3/2003
11/17/2003
12/1/2003
12/15/2003
12/29/2003
1/12/2004
1/26/2004
2/9/2004
2/23/2004
3/8/2004
3/22/2004
4/5/2004
4/19/2004
5/3/2004

000320

TH-C-02
Piezometer at 200 ft below ground surface



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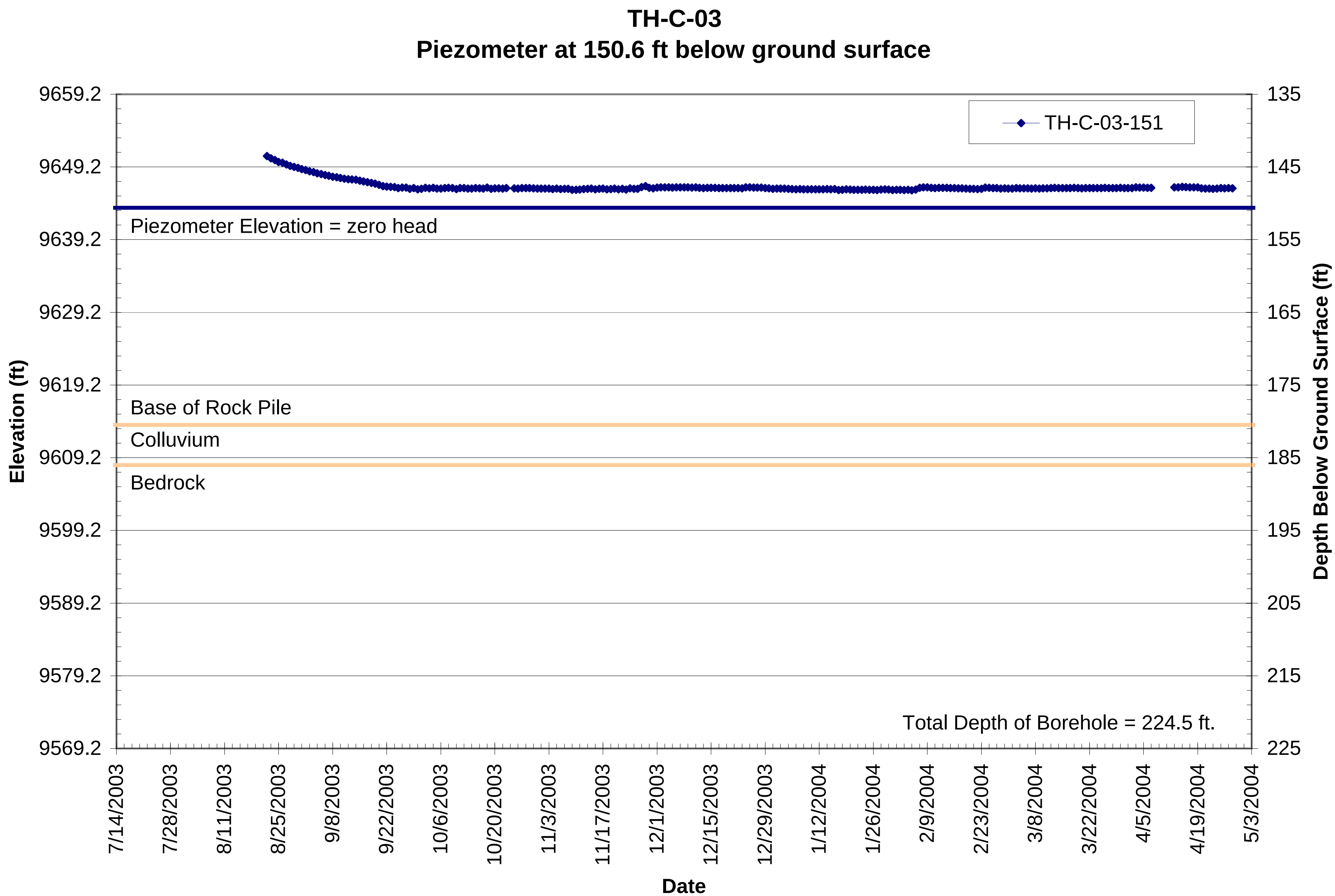
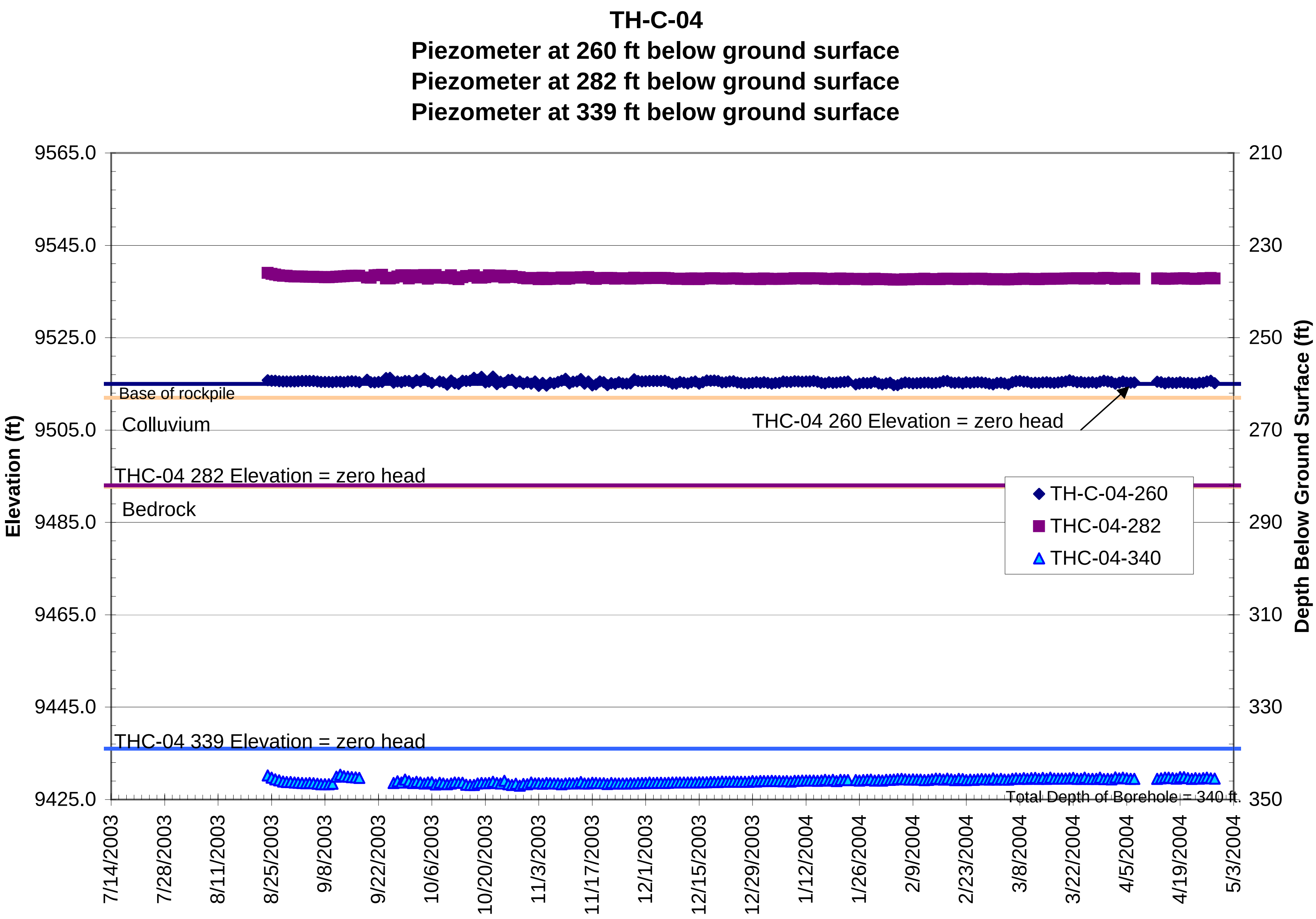
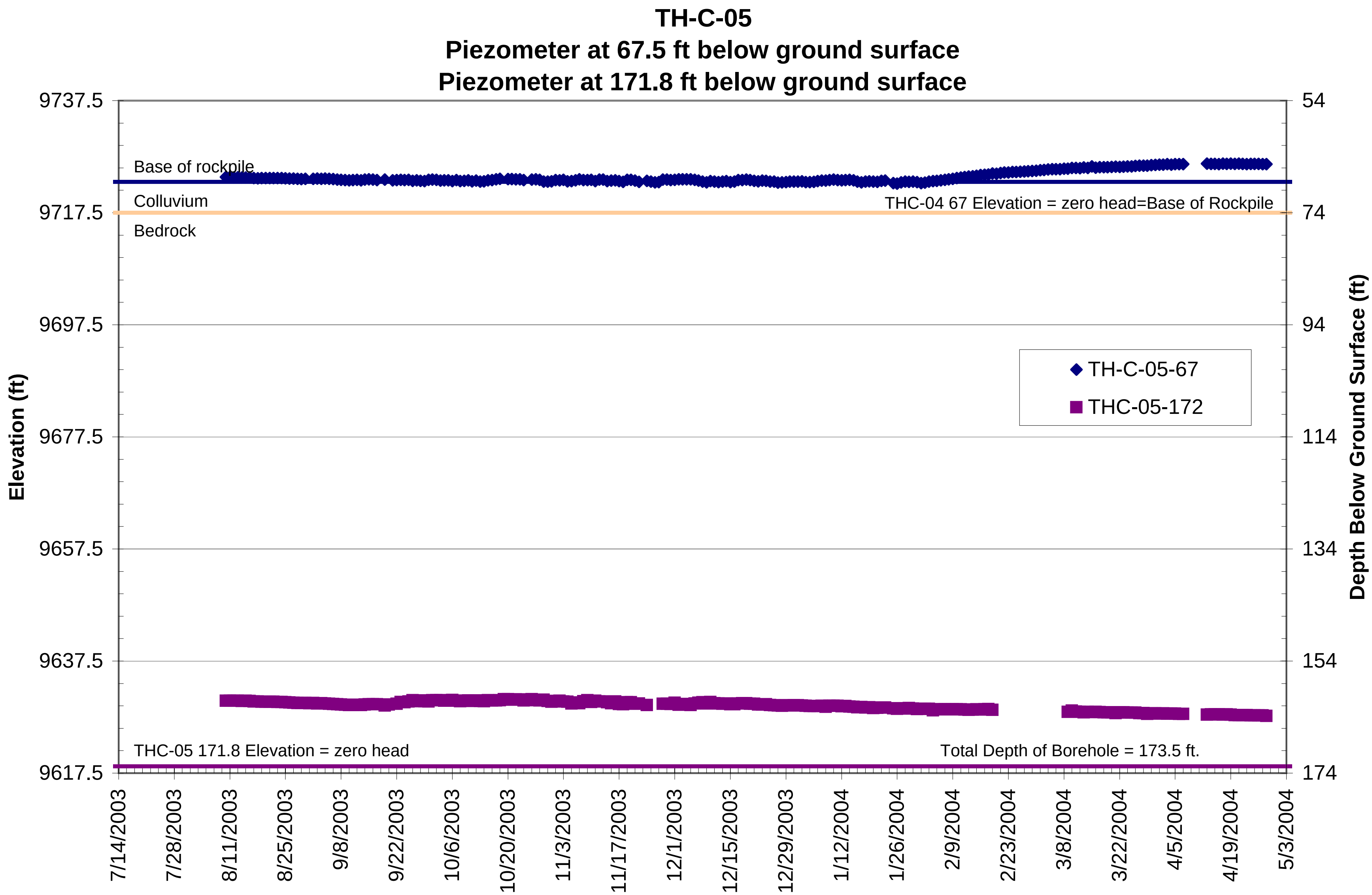

TH-C-03
Piezometer at 150.6 ft below ground surface
TH-C-03-151
Piezometer Elevation = zero head
Base of Rock Pile
Colluvium
Bedrock
Total Depth of Borehole = 224.5 ft.
Elevation (ft)
Depth Below Ground Surface (ft)
Date
9659.2
9649.2
9639.2
9629.2
9619.2
9609.2
9599.2
9589.2
9579.2
9569.2
135
145
155
165
175
185
195
205
215
225
7/14/2003
7/28/2003
8/11/2003
8/25/2003
9/8/2003
9/22/2003
10/6/2003
10/20/2003
11/3/2003
11/17/2003
12/1/2003
12/15/2003
12/29/2003
1/12/2004
1/26/2004
2/9/2004
2/23/2004
3/8/2004
3/22/2004
4/5/2004
4/19/2004
5/3/2004

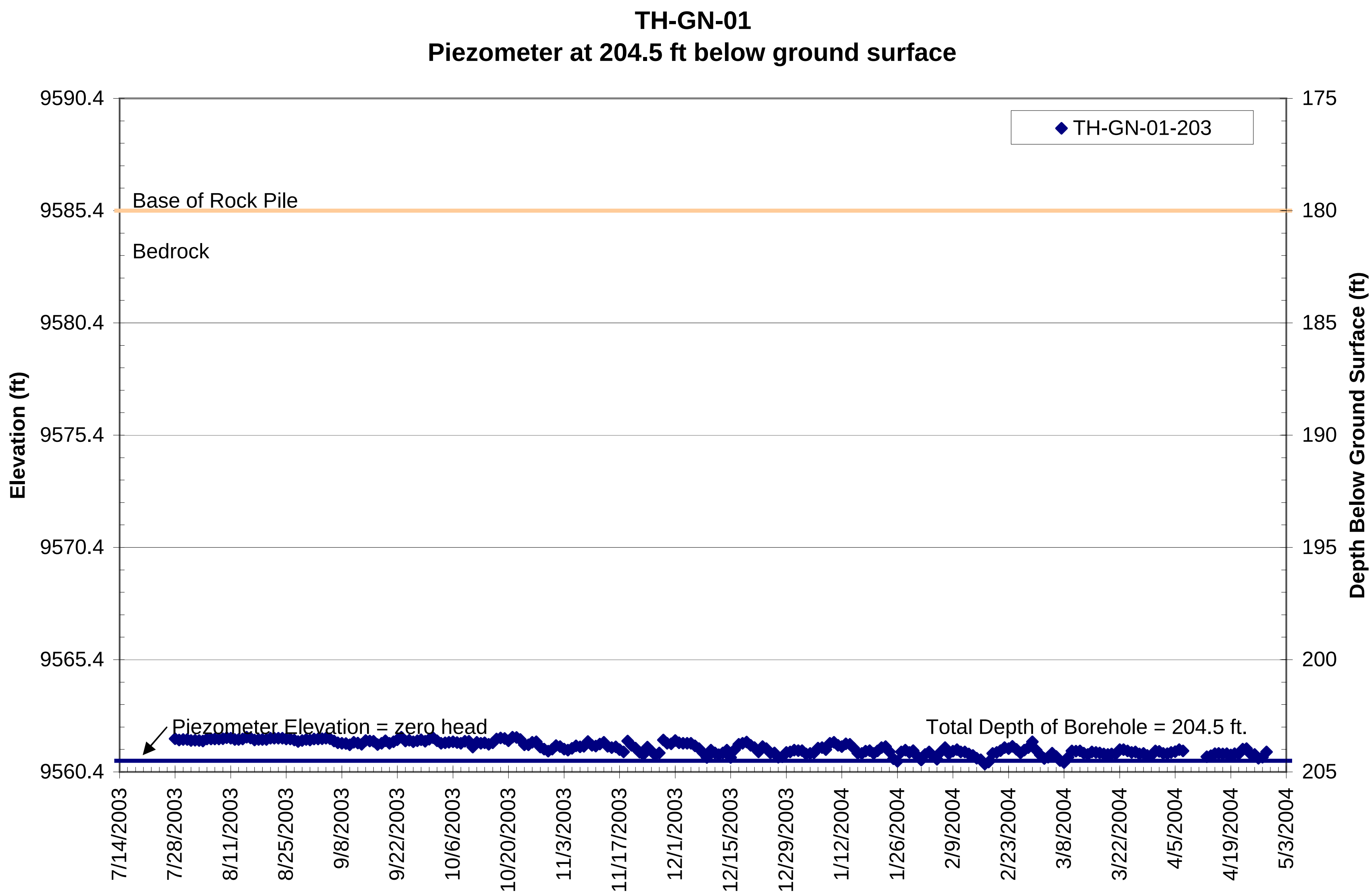
000322

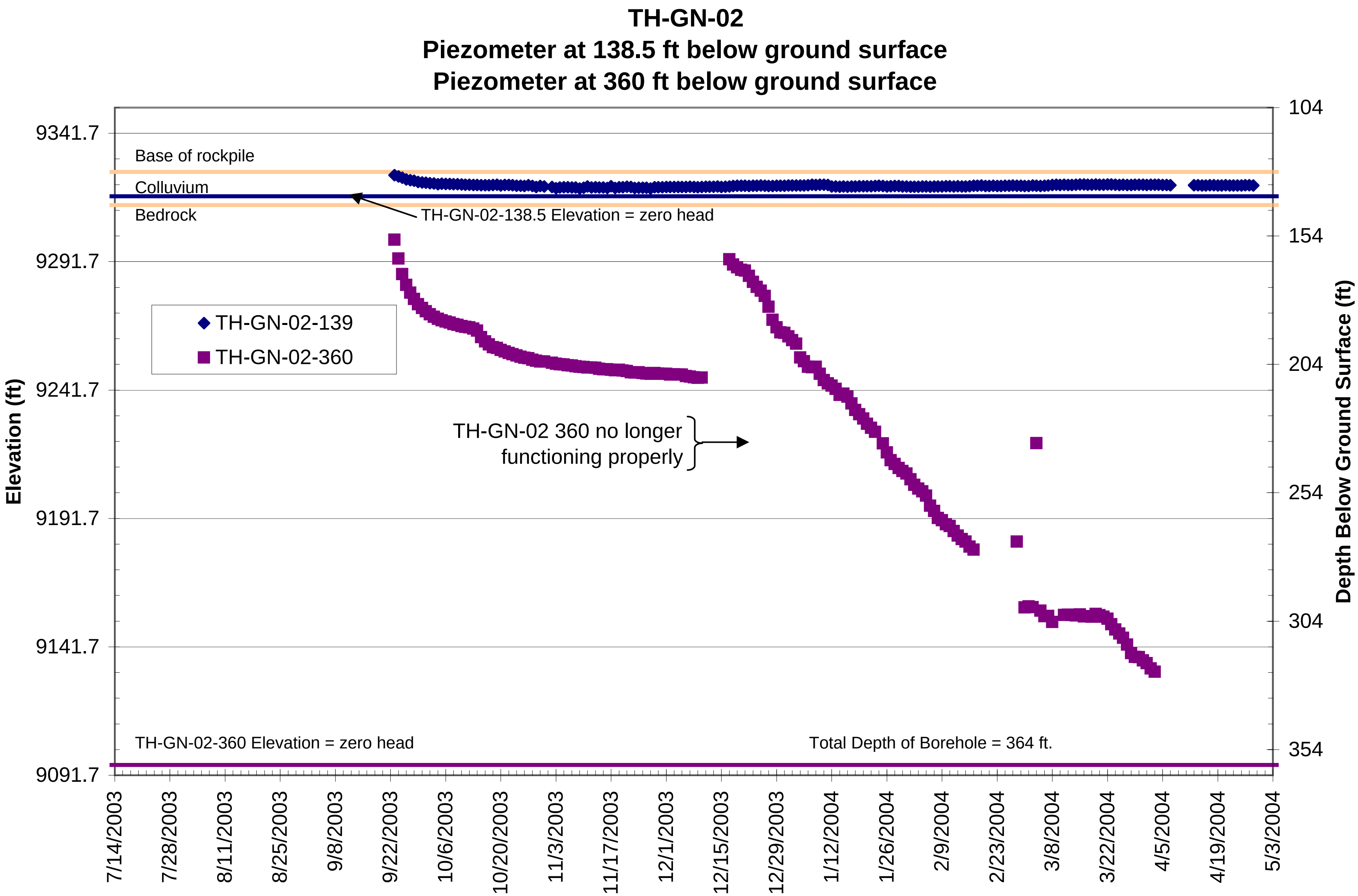

TH-C-04
Piezometer at 260 ft below ground surface
Piezometer at 282 ft below ground surface
Piezometer at 339 ft below ground surface
Elevation (ft)
Depth Below Ground Surface (ft)
9565.0
9545.0
9525.0
9505.0
9485.0
9465.0
9445.0
9425.0
210
230
250
270
290
310
330
350
Base of rockpile
Colluvium
THC-04 260 Elevation = zero head
THC-04 282 Elevation = zero head
Bedrock
THC-04 339 Elevation = zero head
Total Depth of Borehole = 340 ft.
TH-C-04-260
THC-04-282
THC-04-340
7/14/2003
7/28/2003
8/11/2003
8/25/2003
9/8/2003
9/22/2003
10/6/2003
10/20/2003
11/3/2003
11/17/2003
12/1/2003
12/15/2003
12/29/2003
1/12/2004
1/26/2004
2/9/2004
2/23/2004
3/8/2004
3/22/2004
4/5/2004
4/19/2004
5/3/2004

000323


TH-C-05
Piezometer at 67.5 ft below ground surface
Piezometer at 171.8 ft below ground surface
Elevation (ft)
Depth Below Ground Surface (ft)
9737.5
9717.5
9697.5
9677.5
9657.5
9637.5
9617.5
54
74
94
114
134
154
174
Base of rockpile
Colluvium
Bedrock
THC-04 67 Elevation = zero head=Base of Rockpile
TH-C-05-67
THC-05-172
THC-05 171.8 Elevation = zero head
Total Depth of Borehole = 173.5 ft.
7/14/2003
7/28/2003
8/11/2003
8/25/2003
9/8/2003
9/22/2003
10/6/2003
10/20/2003
11/3/2003
11/17/2003
12/1/2003
12/15/2003
12/29/2003
1/12/2004
1/26/2004
2/9/2004
2/23/2004
3/8/2004
3/22/2004
4/5/2004
4/19/2004
5/3/2004

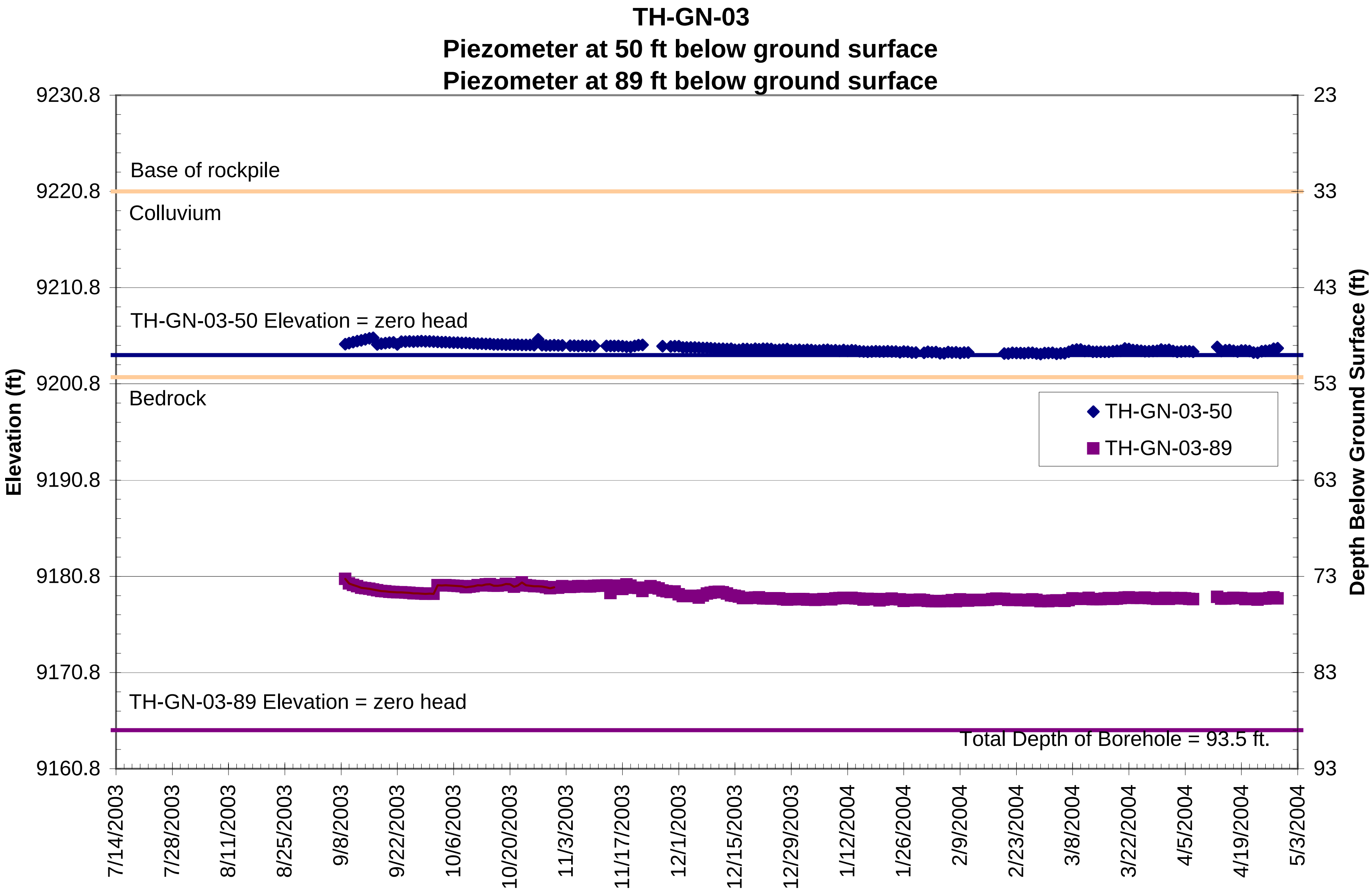
000324


TH-GN-01
Piezometer at 204.5 ft below ground surface
TH-GN-01-203
Elevation (ft)
Depth Below Ground Surface (ft)
Base of Rock Pile
Bedrock
Piezometer Elevation = zero head
Total Depth of Borehole = 204.5 ft.
9590.4
9585.4
9580.4
9575.4
9570.4
9565.4
9560.4
175
180
185
190
195
200
205
7/14/2003
7/28/2003
8/11/2003
8/25/2003
9/8/2003
9/22/2003
10/6/2003
10/20/2003
11/3/2003
11/17/2003
12/1/2003
12/15/2003
12/29/2003
1/12/2004
1/26/2004
2/9/2004
2/23/2004
3/8/2004
3/22/2004
4/5/2004
4/19/2004
5/3/2004



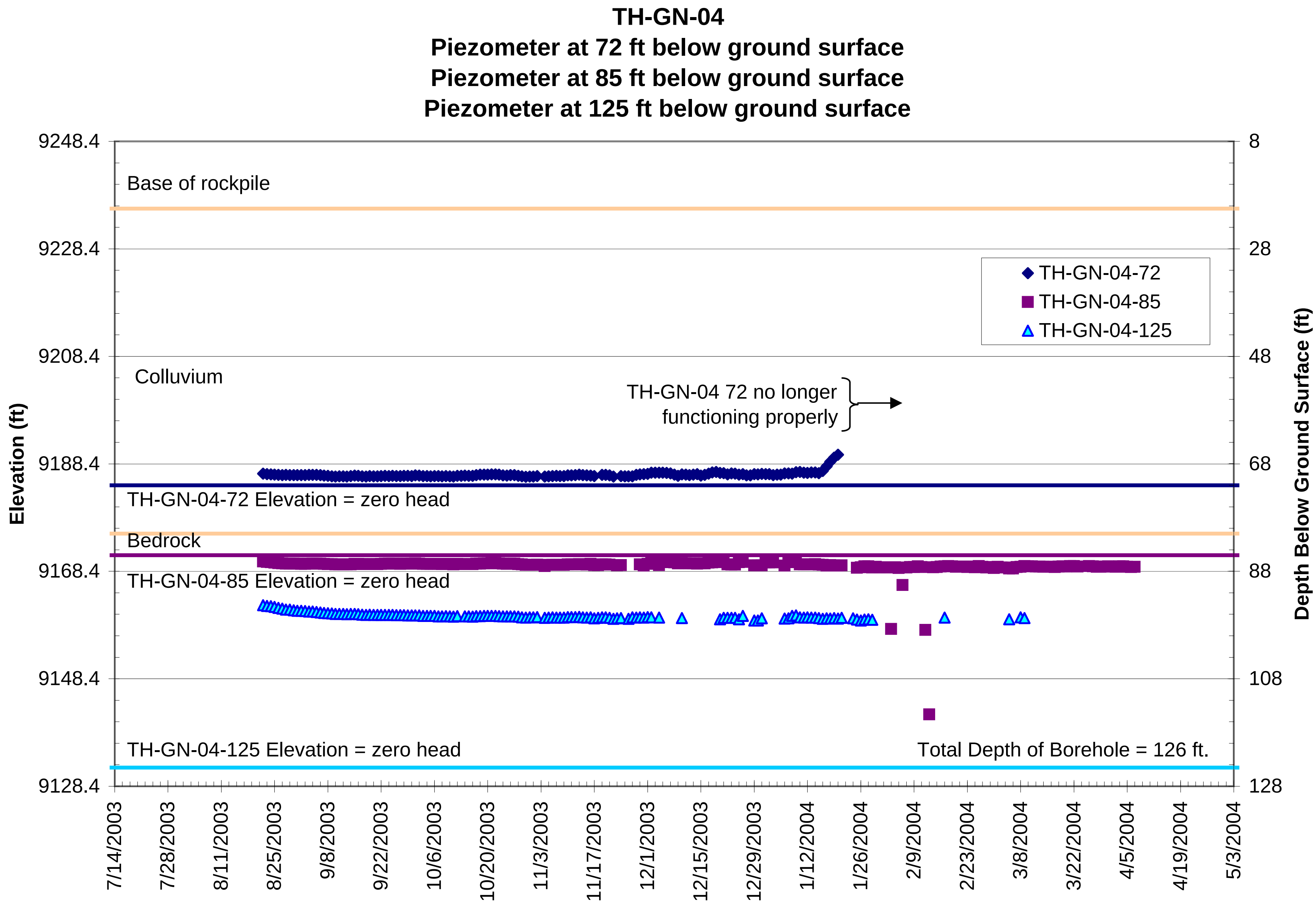
TH-GN-02
Piezometer at 138.5 ft below ground surface
Piezometer at 360 ft below ground surface
Elevation (ft)
Depth Below Ground Surface (ft)
9341.7
9291.7
9241.7
9191.7
9141.7
9091.7
104
154
204
254
304
354
Base of rockpile
Colluvium
Bedrock
TH-GN-02-138.5 Elevation = zero head
TH-GN-02-139
TH-GN-02-360
TH-GN-02 360 no longer functioning properly
TH-GN-02-360 Elevation = zero head
Total Depth of Borehole = 364 ft.
7/14/2003
7/28/2003
8/11/2003
8/25/2003
9/8/2003
9/22/2003
10/6/2003
10/20/2003
11/3/2003
11/17/2003
12/1/2003
12/15/2003
12/29/2003
1/12/2004
1/26/2004
2/9/2004
2/23/2004
3/8/2004
3/22/2004
4/5/2004
4/19/2004
5/3/2004

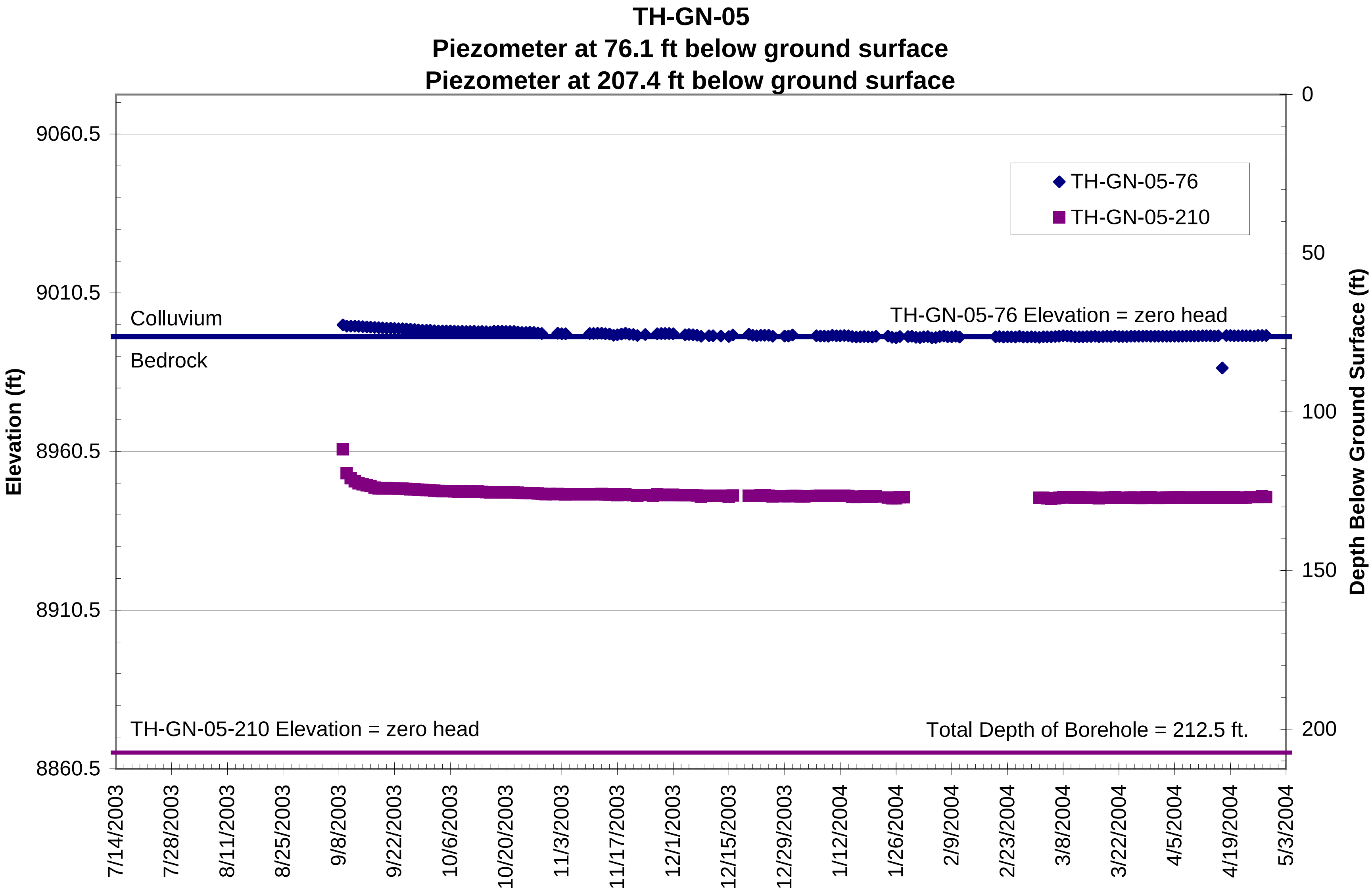
000326



TH-GN-03
Piezometer at 50 ft below ground surface
Piezometer at 89 ft below ground surface
Elevation (ft)
Depth Below Ground Surface (ft)
Base of rockpile
Colluvium
TH-GN-03-50 Elevation = zero head
Bedrock
TH-GN-03-89 Elevation = zero head
Total Depth of Borehole = 93.5 ft.
TH-GN-03-50
TH-GN-03-89
9230.8
9220.8
9210.8
9200.8
9190.8
9180.8
9170.8
9160.8
23
33
43
53
63
73
83
93
7/14/2003
7/28/2003
8/11/2003
8/25/2003
9/8/2003
9/22/2003
10/6/2003
10/20/2003
11/3/2003
11/17/2003
12/1/2003
12/15/2003
12/29/2003
1/12/2004
1/26/2004
2/9/2004
2/23/2004
3/8/2004
3/22/2004
4/5/2004
4/19/2004
5/3/2004

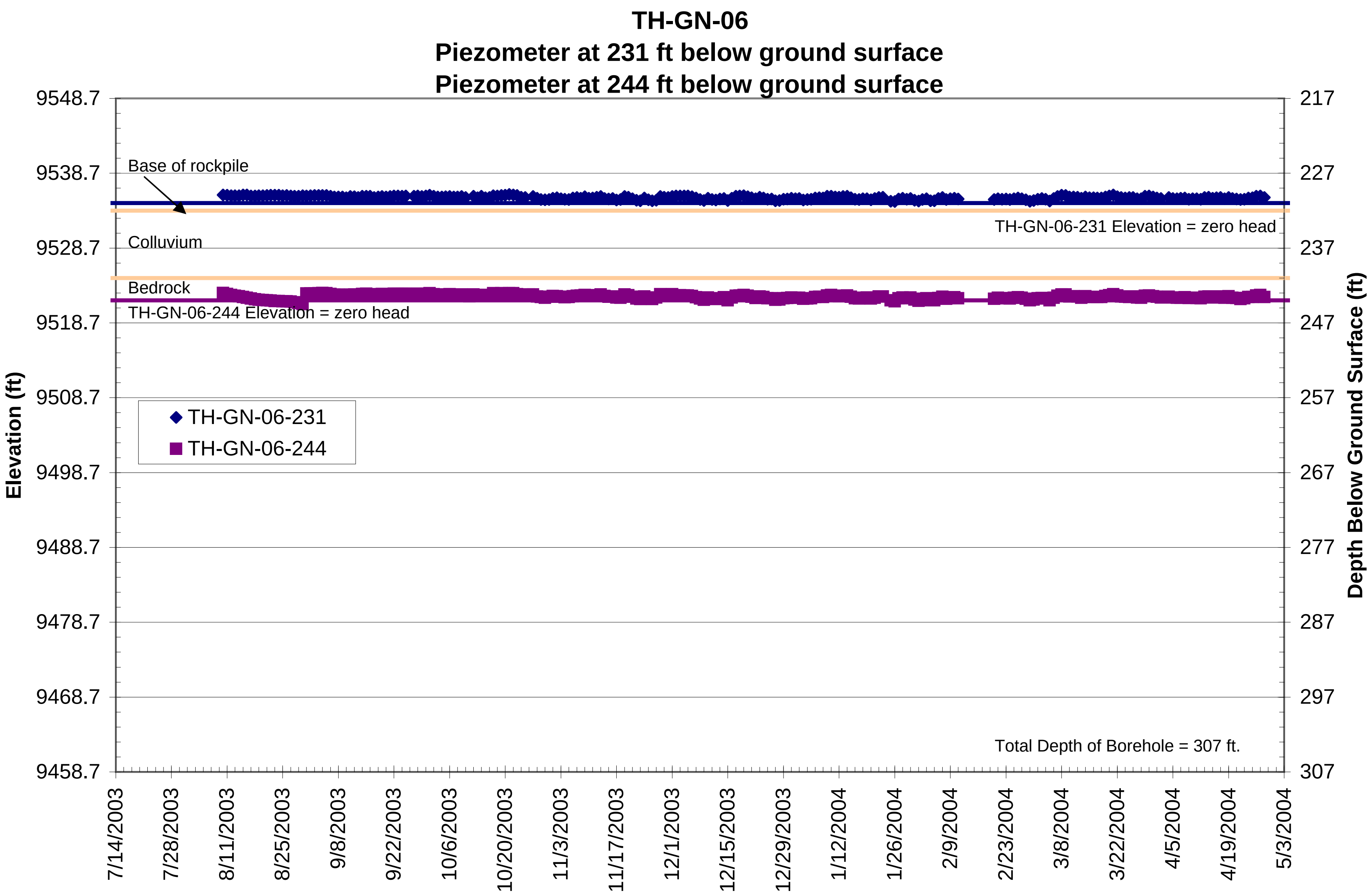
000327


TH-GN-04
Piezometer at 72 ft below ground surface
Piezometer at 85 ft below ground surface
Piezometer at 125 ft below ground surface
Elevation (ft)
Depth Below Ground Surface (ft)
Base of rockpile
Colluvium
TH-GN-04 72 no longer functioning properly
TH-GN-04-72 Elevation = zero head
Bedrock
TH-GN-04-85 Elevation = zero head
TH-GN-04-125 Elevation = zero head
Total Depth of Borehole = 126 ft.
TH-GN-04-72
TH-GN-04-85
TH-GN-04-125
9248.4
9228.4
9208.4
9188.4
9168.4
9148.4
9128.4
8
28
48
68
88
108
128
7/14/2003
7/28/2003
8/11/2003
8/25/2003
9/8/2003
9/22/2003
10/6/2003
10/20/2003
11/3/2003
11/17/2003
12/1/2003
12/15/2003
12/29/2003
1/12/2004
1/26/2004
2/9/2004
2/23/2004
3/8/2004
3/22/2004
4/5/2004
4/19/2004
5/3/2004



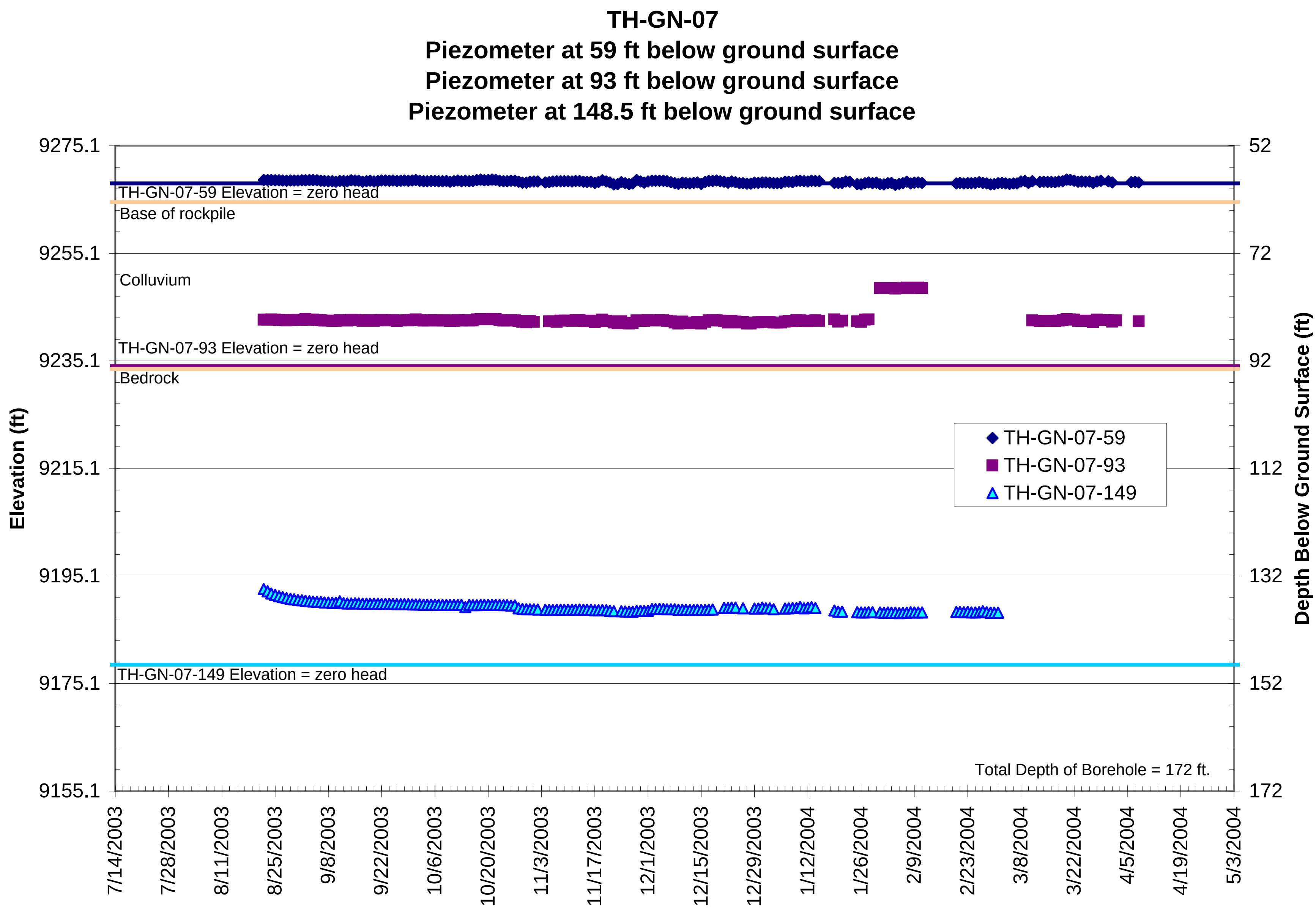
TH-GN-05
Piezometer at 76.1 ft below ground surface
Piezometer at 207.4 ft below ground surface
TH-GN-05-76
TH-GN-05-210
Colluvium
Bedrock
TH-GN-05-76 Elevation = zero head
TH-GN-05-210 Elevation = zero head
Total Depth of Borehole = 212.5 ft.
Elevation (ft)
Depth Below Ground Surface (ft)
9060.5
9010.5
8960.5
8910.5
8860.5
0
50
100
150
200
7/14/2003
7/28/2003
8/11/2003
8/25/2003
9/8/2003
9/22/2003
10/6/2003
10/20/2003
11/3/2003
11/17/2003
12/1/2003
12/15/2003
12/29/2003
1/12/2004
1/26/2004
2/9/2004
2/23/2004
3/8/2004
3/22/2004
4/5/2004
4/19/2004
5/3/2004

000329

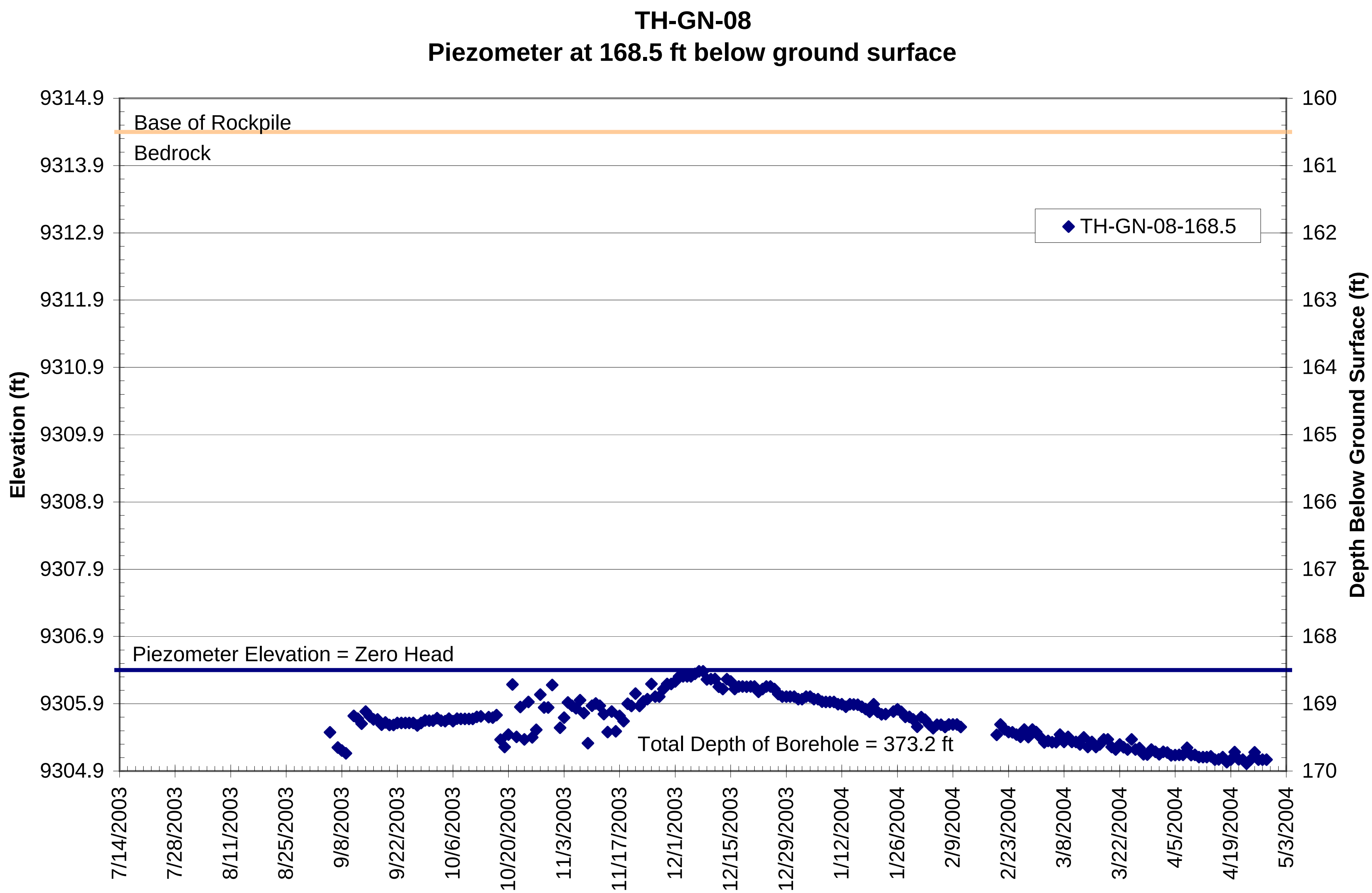


TH-GN-06
Piezometer at 231 ft below ground surface
Piezometer at 244 ft below ground surface
Elevation (ft)
Depth Below Ground Surface (ft)
Base of rockpile
Colluvium
Bedrock
TH-GN-06-231 Elevation = zero head
TH-GN-06-244 Elevation = zero head
TH-GN-06-231
TH-GN-06-244
Total Depth of Borehole = 307 ft.
9548.7
9538.7
9528.7
9518.7
9508.7
9498.7
9488.7
9478.7
9468.7
9458.7
217
227
237
247
257
267
277
287
297
307
7/14/2003
7/28/2003
8/11/2003
8/25/2003
9/8/2003
9/22/2003
10/6/2003
10/20/2003
11/3/2003
11/17/2003
12/1/2003
12/15/2003
12/29/2003
1/12/2004
1/26/2004
2/9/2004
2/23/2004
3/8/2004
3/22/2004
4/5/2004
4/19/2004
5/3/2004

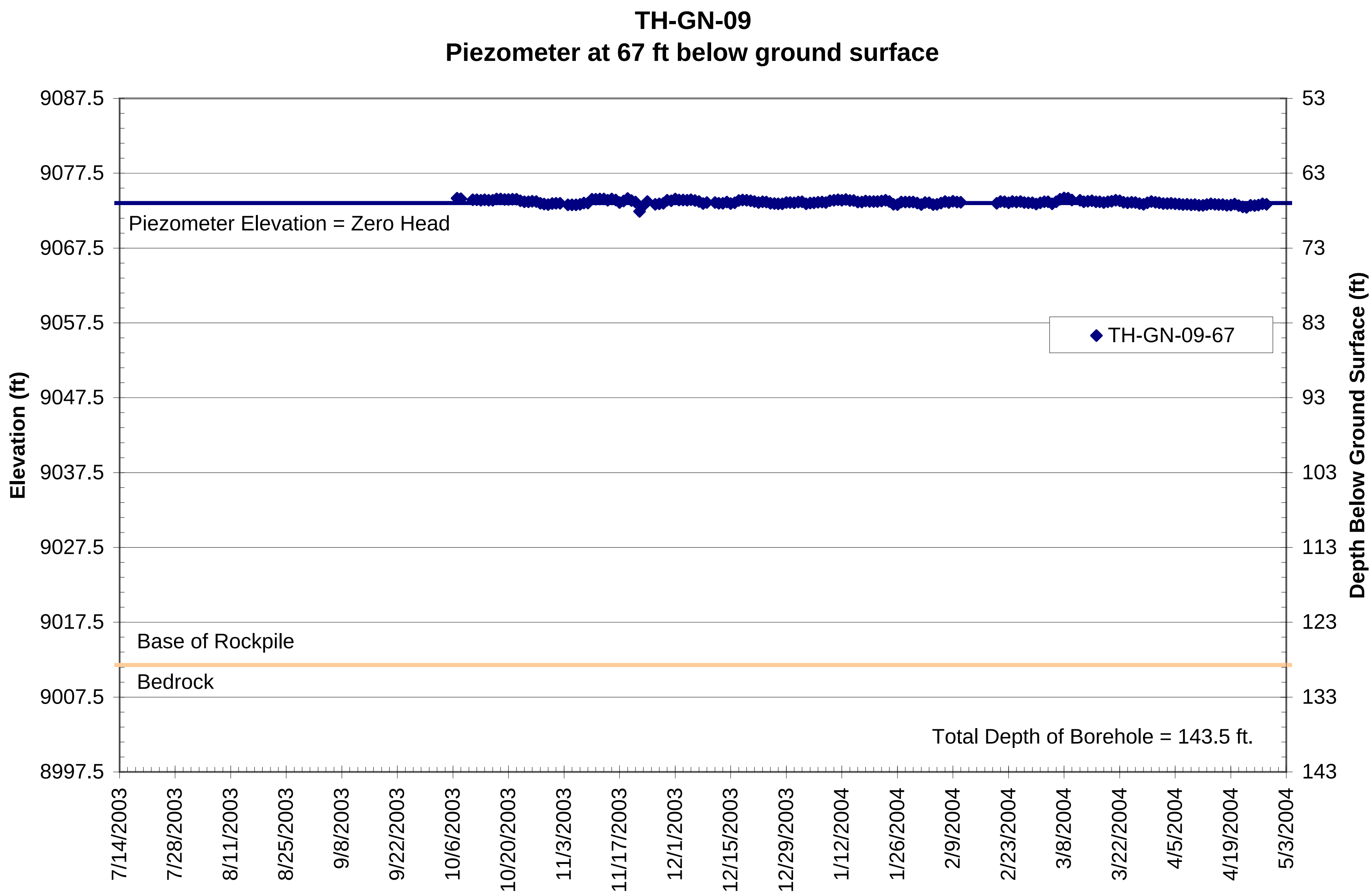
000330



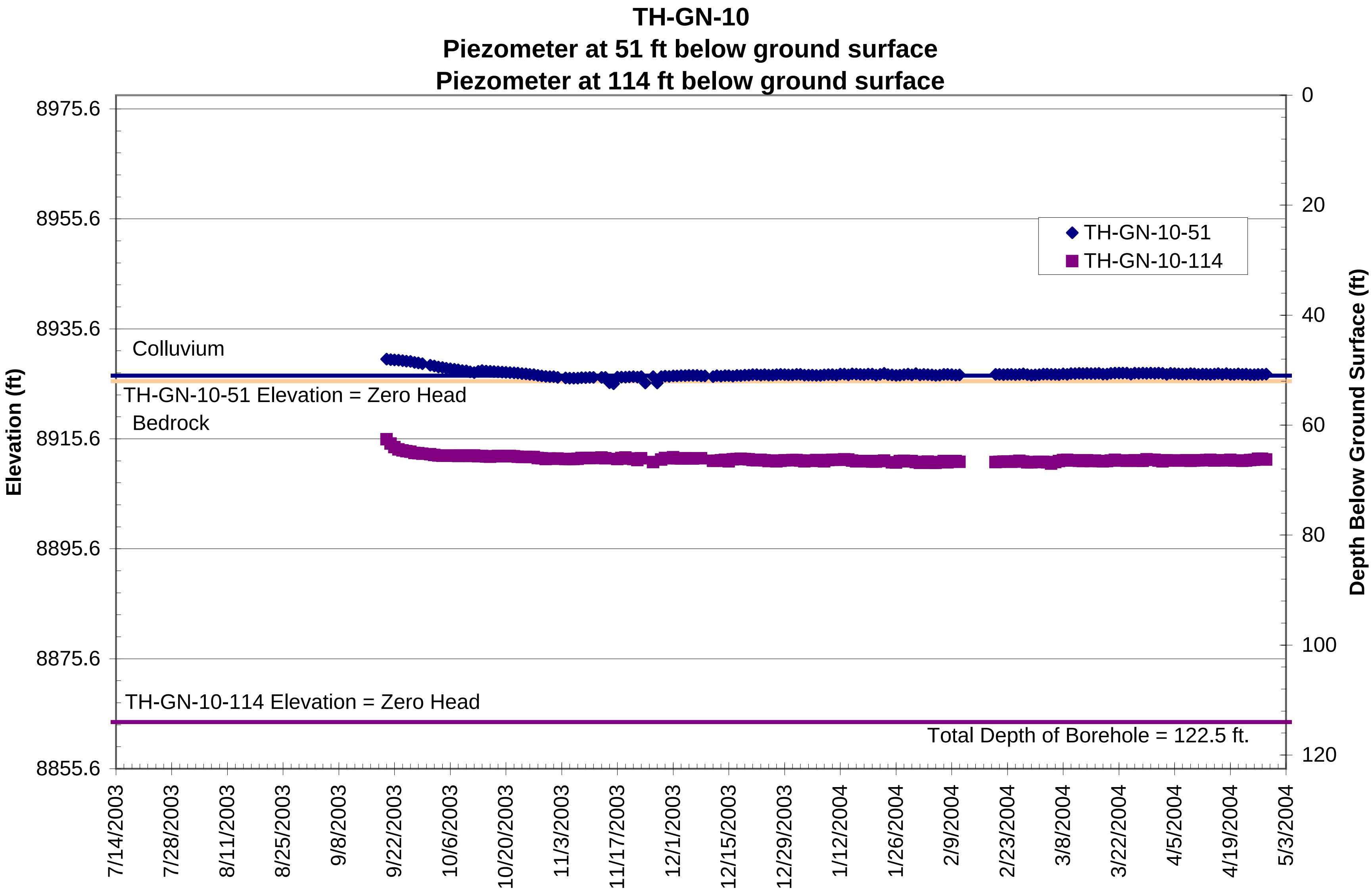
TH-GN-07
Piezometer at 59 ft below ground surface
Piezometer at 93 ft below ground surface
Piezometer at 148.5 ft below ground surface
Elevation (ft)
Depth Below Ground Surface (ft)
9275.1
9255.1
9235.1
9215.1
9195.1
9175.1
9155.1
52
72
92
112
132
152
172
TH-GN-07-59 Elevation = zero head
Base of rockpile
Colluvium
TH-GN-07-93 Elevation = zero head
Bedrock
TH-GN-07-149 Elevation = zero head
Total Depth of Borehole = 172 ft.
TH-GN-07-59
TH-GN-07-93
TH-GN-07-149
7/14/2003
7/28/2003
8/11/2003
8/25/2003
9/8/2003
9/22/2003
10/6/2003
10/20/2003
11/3/2003
11/17/2003
12/1/2003
12/15/2003
12/29/2003
1/12/2004
1/26/2004
2/9/2004
2/23/2004
3/8/2004
3/22/2004
4/5/2004
4/19/2004
5/3/2004


TH-GN-08
Piezometer at 168.5 ft below ground surface
Elevation (ft)
Depth Below Ground Surface (ft)
9314.9
9313.9
9312.9
9311.9
9310.9
9309.9
9308.9
9307.9
9306.9
9305.9
9304.9
160
161
162
163
164
165
166
167
168
169
170
Base of Rockpile
Bedrock
TH-GN-08-168.5
Piezometer Elevation = Zero Head
Total Depth of Borehole = 373.2 ft
7/14/2003
7/28/2003
8/11/2003
8/25/2003
9/8/2003
9/22/2003
10/6/2003
10/20/2003
11/3/2003
11/17/2003
12/1/2003
12/15/2003
12/29/2003
1/12/2004
1/26/2004
2/9/2004
2/23/2004
3/8/2004
3/22/2004
4/5/2004
4/19/2004
5/3/2004

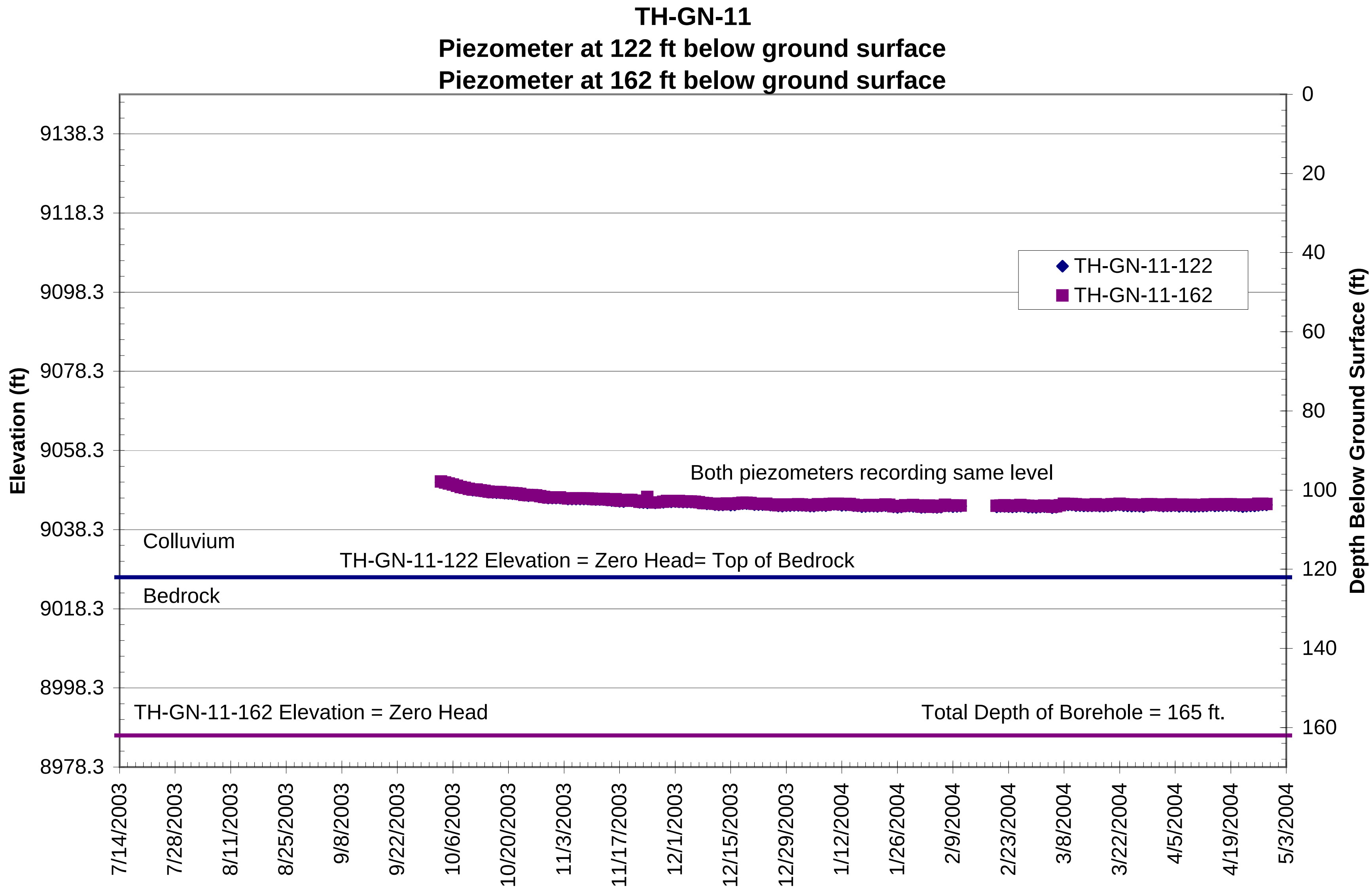
000332



TH-GN-09
Piezometer at 67 ft below ground surface
Elevation (ft)
Depth Below Ground Surface (ft)
9087.5
9077.5
9067.5
9057.5
9047.5
9037.5
9027.5
9017.5
9007.5
8997.5
53
63
73
83
93
103
113
123
133
143
Piezometer Elevation = Zero Head
TH-GN-09-67
Base of Rockpile
Bedrock
Total Depth of Borehole = 143.5 ft.
7/14/2003
7/28/2003
8/11/2003
8/25/2003
9/8/2003
9/22/2003
10/6/2003
10/20/2003
11/3/2003
11/17/2003
12/1/2003
12/15/2003
12/29/2003
1/12/2004
1/26/2004
2/9/2004
2/23/2004
3/8/2004
3/22/2004
4/5/2004
4/19/2004
5/3/2004

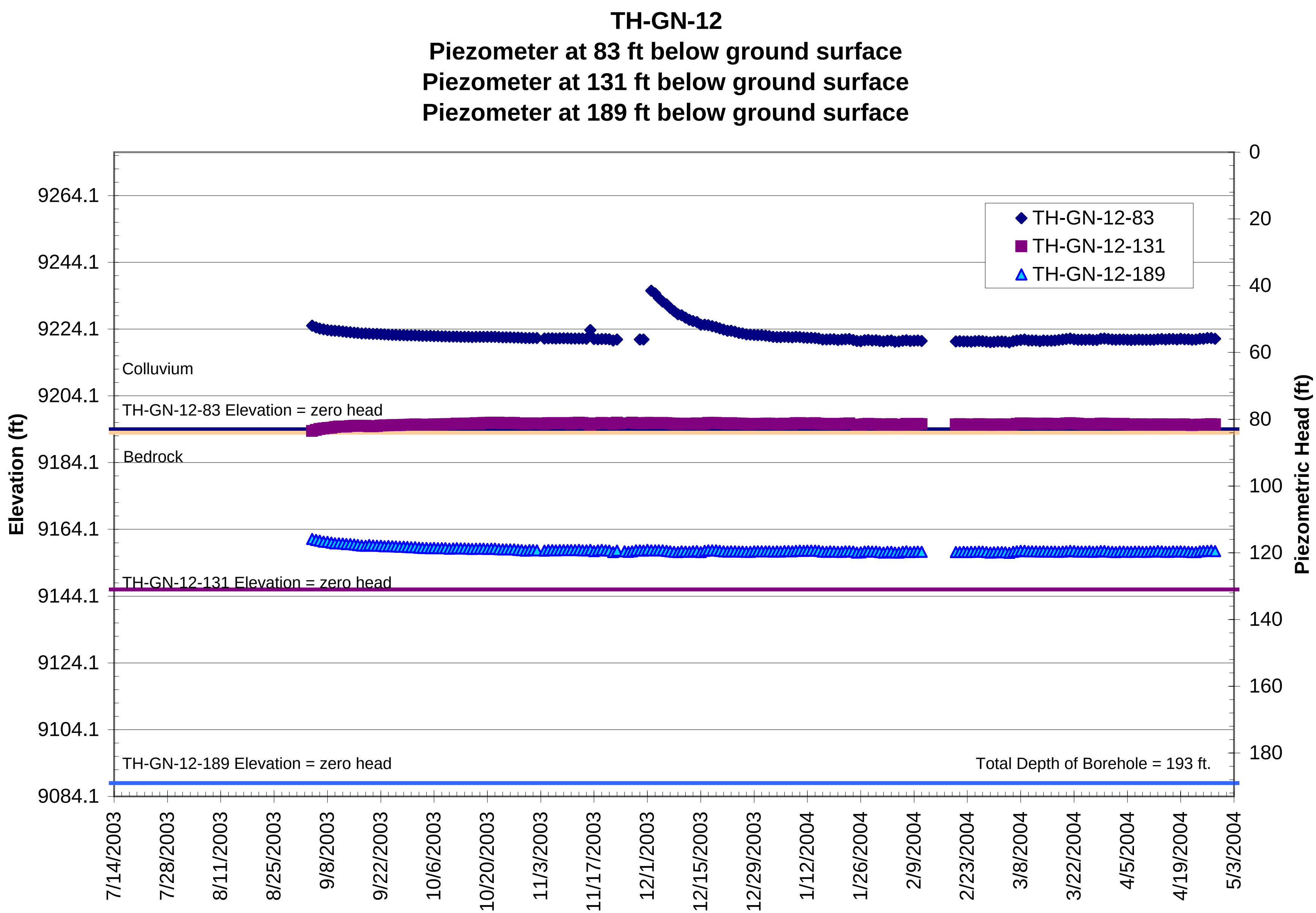

TH-GN-10
Piezometer at 51 ft below ground surface
Piezometer at 114 ft below ground surface
TH-GN-10-51
TH-GN-10-114
Colluvium
TH-GN-10-51 Elevation = Zero Head
Bedrock
TH-GN-10-114 Elevation = Zero Head
Total Depth of Borehole = 122.5 ft.
Elevation (ft)
Depth Below Ground Surface (ft)
8975.6
8955.6
8935.6
8915.6
8895.6
8875.6
8855.6
0
20
40
60
80
100
120
7/14/2003
7/28/2003
8/11/2003
8/25/2003
9/8/2003
9/22/2003
10/6/2003
10/20/2003
11/3/2003
11/17/2003
12/1/2003
12/15/2003
12/29/2003
1/12/2004
1/26/2004
2/9/2004
2/23/2004
3/8/2004
3/22/2004
4/5/2004
4/19/2004
5/3/2004

000334



TH-GN-11
Piezometer at 122 ft below ground surface
Piezometer at 162 ft below ground surface
Elevation (ft)
Depth Below Ground Surface (ft)
TH-GN-11-122
TH-GN-11-162
Both piezometers recording same level
Colluvium
TH-GN-11-122 Elevation = Zero Head= Top of Bedrock
Bedrock
TH-GN-11-162 Elevation = Zero Head
Total Depth of Borehole = 165 ft.
9138.3
9118.3
9098.3
9078.3
9058.3
9038.3
9018.3
8998.3
8978.3
0
20
40
60
80
100
120
140
160
7/14/2003
7/28/2003
8/11/2003
8/25/2003
9/8/2003
9/22/2003
10/6/2003
10/20/2003
11/3/2003
11/17/2003
12/1/2003
12/15/2003
12/29/2003
1/12/2004
1/26/2004
2/9/2004
2/23/2004
3/8/2004
3/22/2004
4/5/2004
4/19/2004
5/3/2004

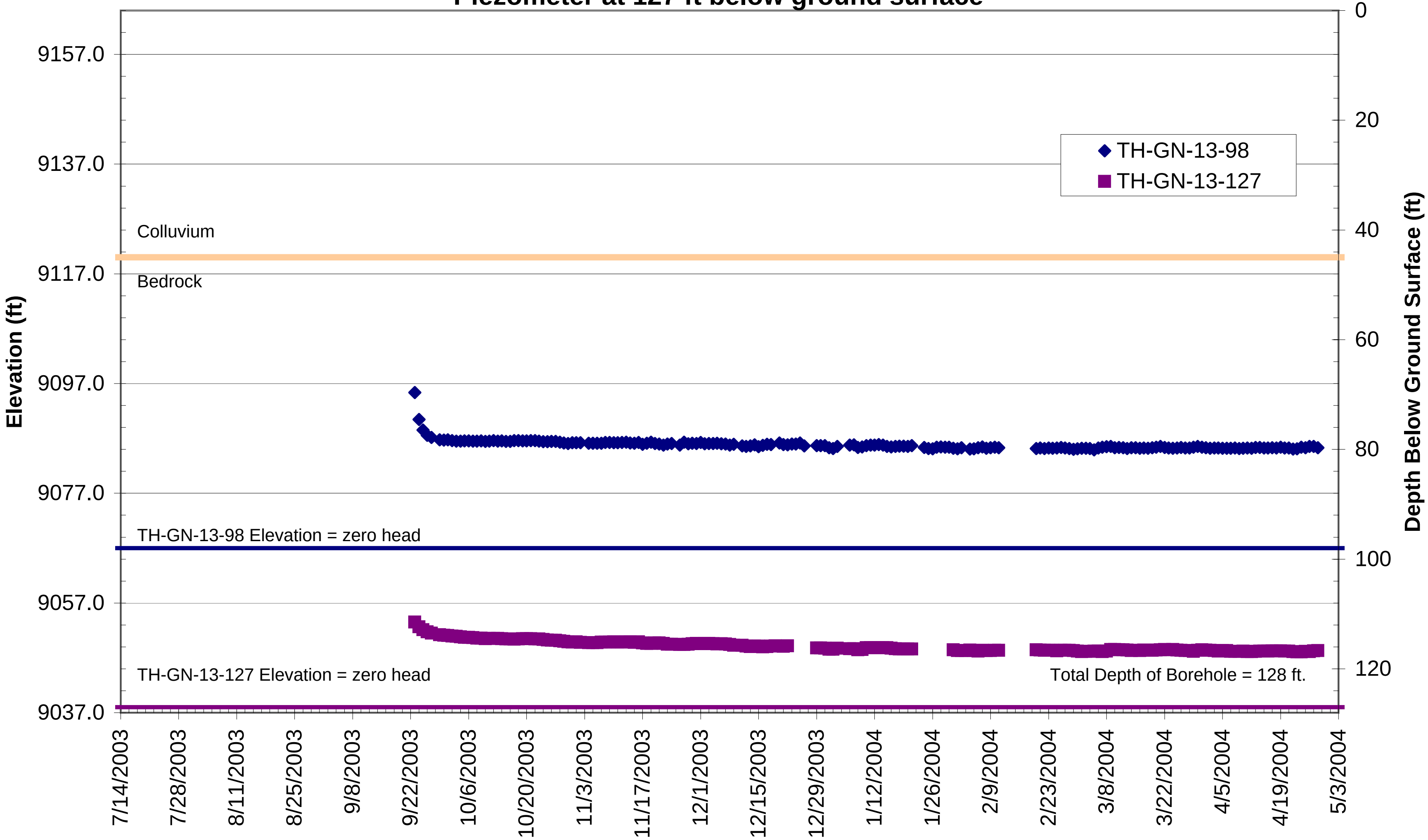
000335

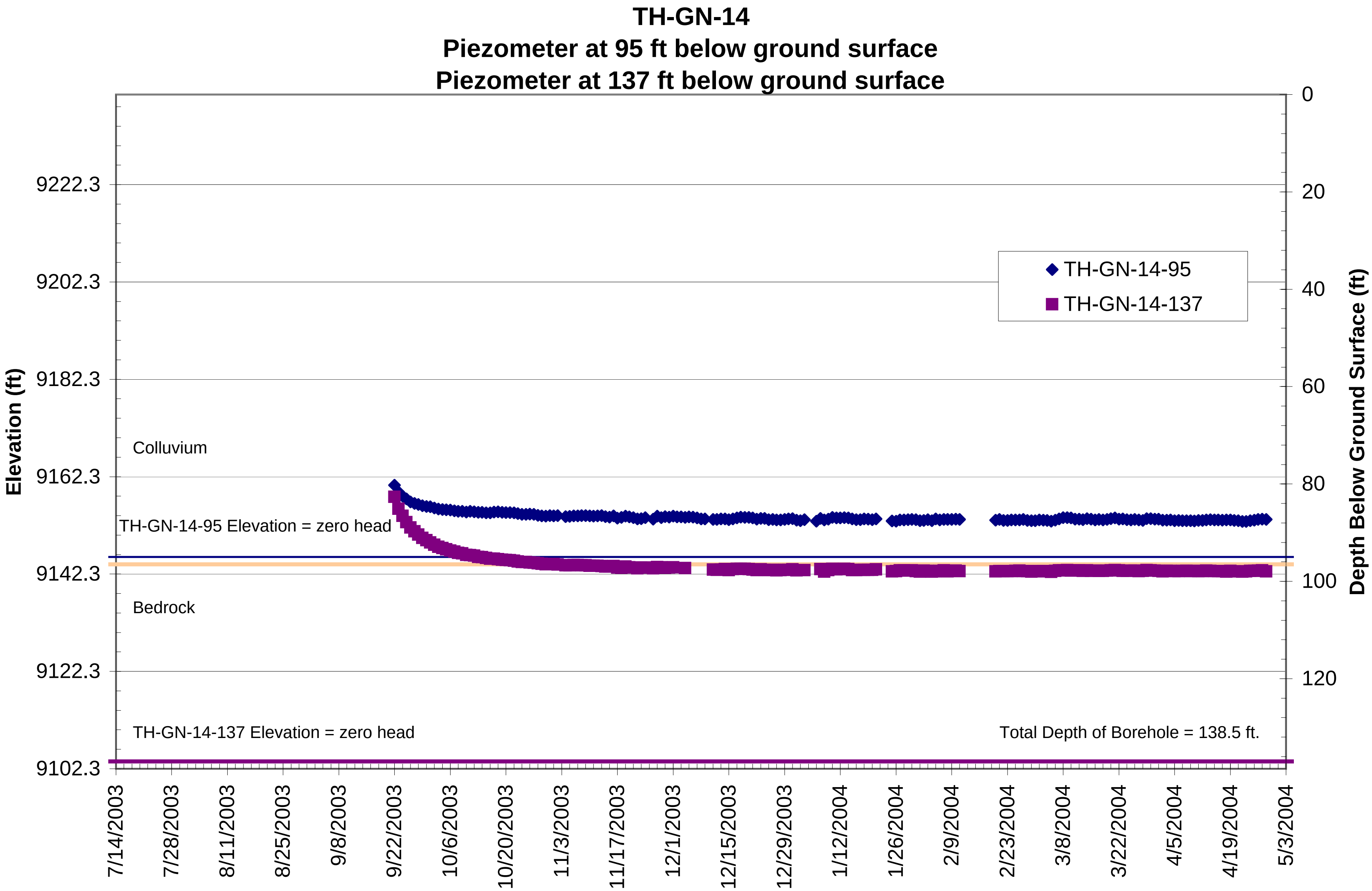


TH-GN-12
Piezometer at 83 ft below ground surface
Piezometer at 131 ft below ground surface
Piezometer at 189 ft below ground surface
TH-GN-12-83
TH-GN-12-131
TH-GN-12-189
Colluvium
TH-GN-12-83 Elevation = zero head
Bedrock
TH-GN-12-131 Elevation = zero head
TH-GN-12-189 Elevation = zero head
Total Depth of Borehole = 193 ft.
Elevation (ft)
Piezometric Head (ft)
9264.1
9244.1
9224.1
9204.1
9184.1
9164.1
9144.1
9124.1
9104.1
9084.1
0
20
40
60
80
100
120
140
160
180
7/14/2003
7/28/2003
8/11/2003
8/25/2003
9/8/2003
9/22/2003
10/6/2003
10/20/2003
11/3/2003
11/17/2003
12/1/2003
12/15/2003
12/29/2003
1/12/2004
1/26/2004
2/9/2004
2/23/2004
3/8/2004
3/22/2004
4/5/2004
4/19/2004
5/3/2004

000336

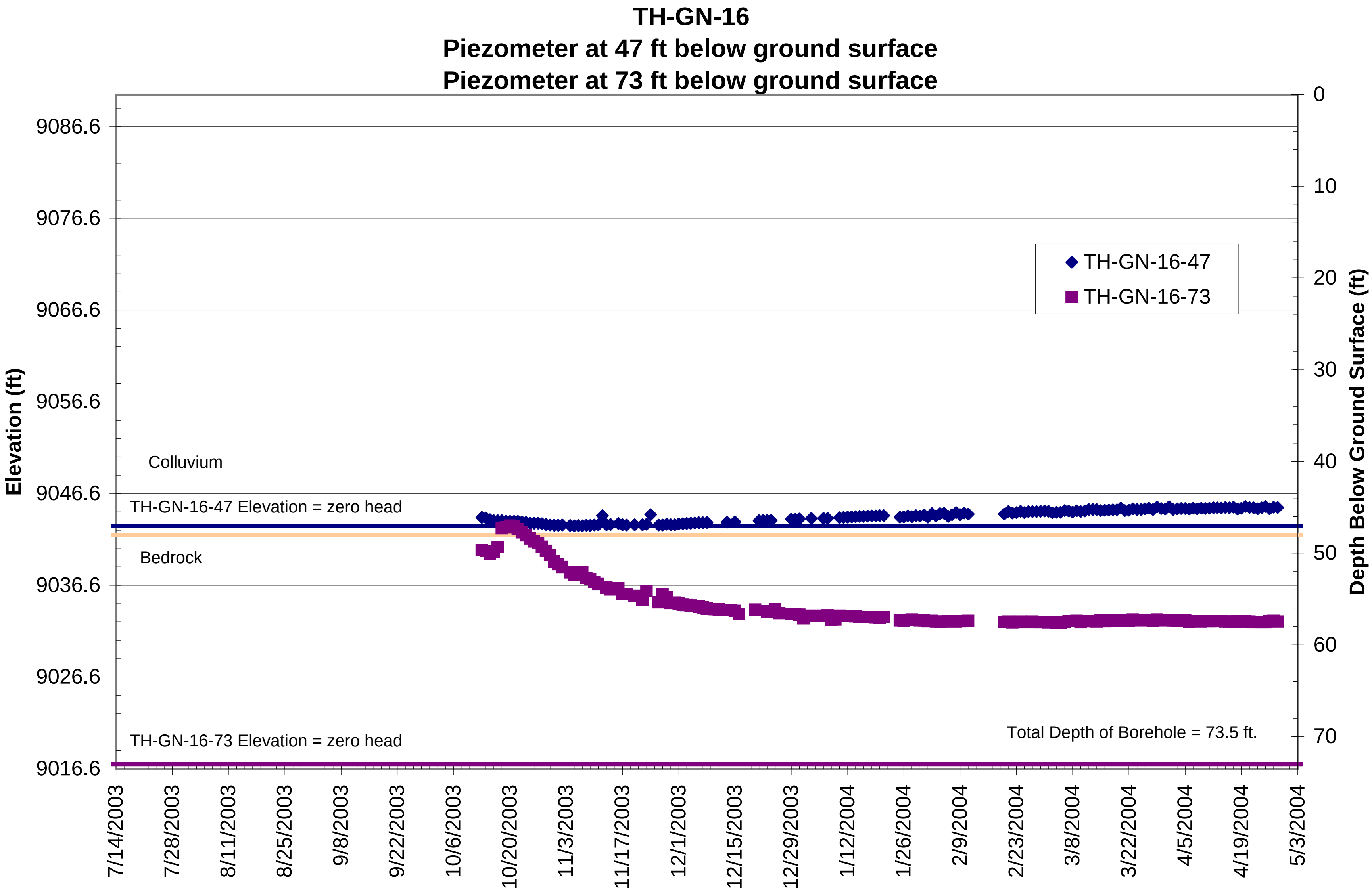
TH-GN-13
Piezometer at 98 ft below ground surface
Piezometer at 127 ft below ground surface





TH-GN-14
Piezometer at 95 ft below ground surface
Piezometer at 137 ft below ground surface
Elevation (ft)
Depth Below Ground Surface (ft)
TH-GN-14-95
TH-GN-14-137
Colluvium
Bedrock
TH-GN-14-95 Elevation = zero head
TH-GN-14-137 Elevation = zero head
Total Depth of Borehole = 138.5 ft.
9222.3
9202.3
9182.3
9162.3
9142.3
9122.3
9102.3
0
20
40
60
80
100
120
7/14/2003
7/28/2003
8/11/2003
8/25/2003
9/8/2003
9/22/2003
10/6/2003
10/20/2003
11/3/2003
11/17/2003
12/1/2003
12/15/2003
12/29/2003
1/12/2004
1/26/2004
2/9/2004
2/23/2004
3/8/2004
3/22/2004
4/5/2004
4/19/2004
5/3/2004

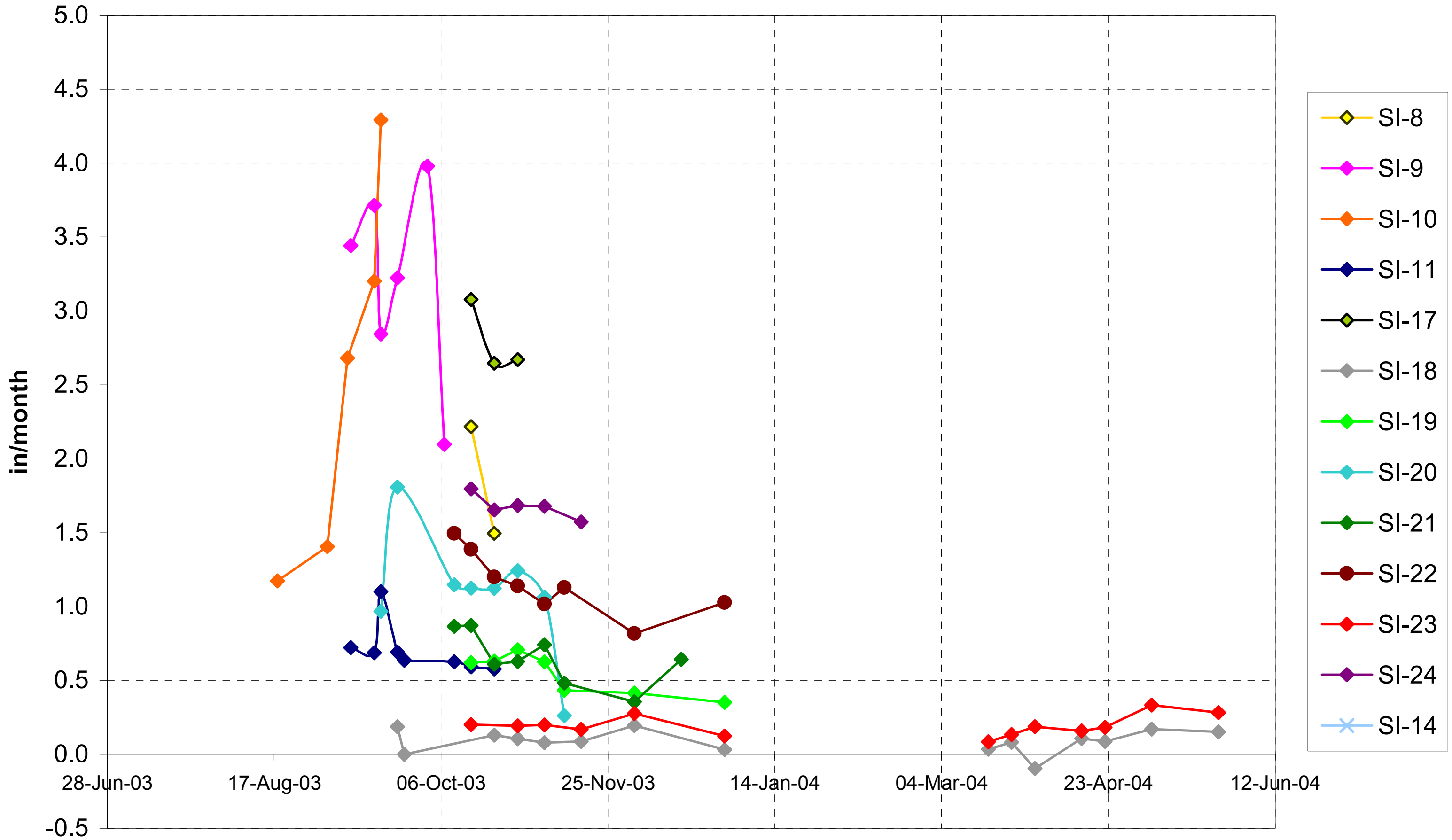
000338



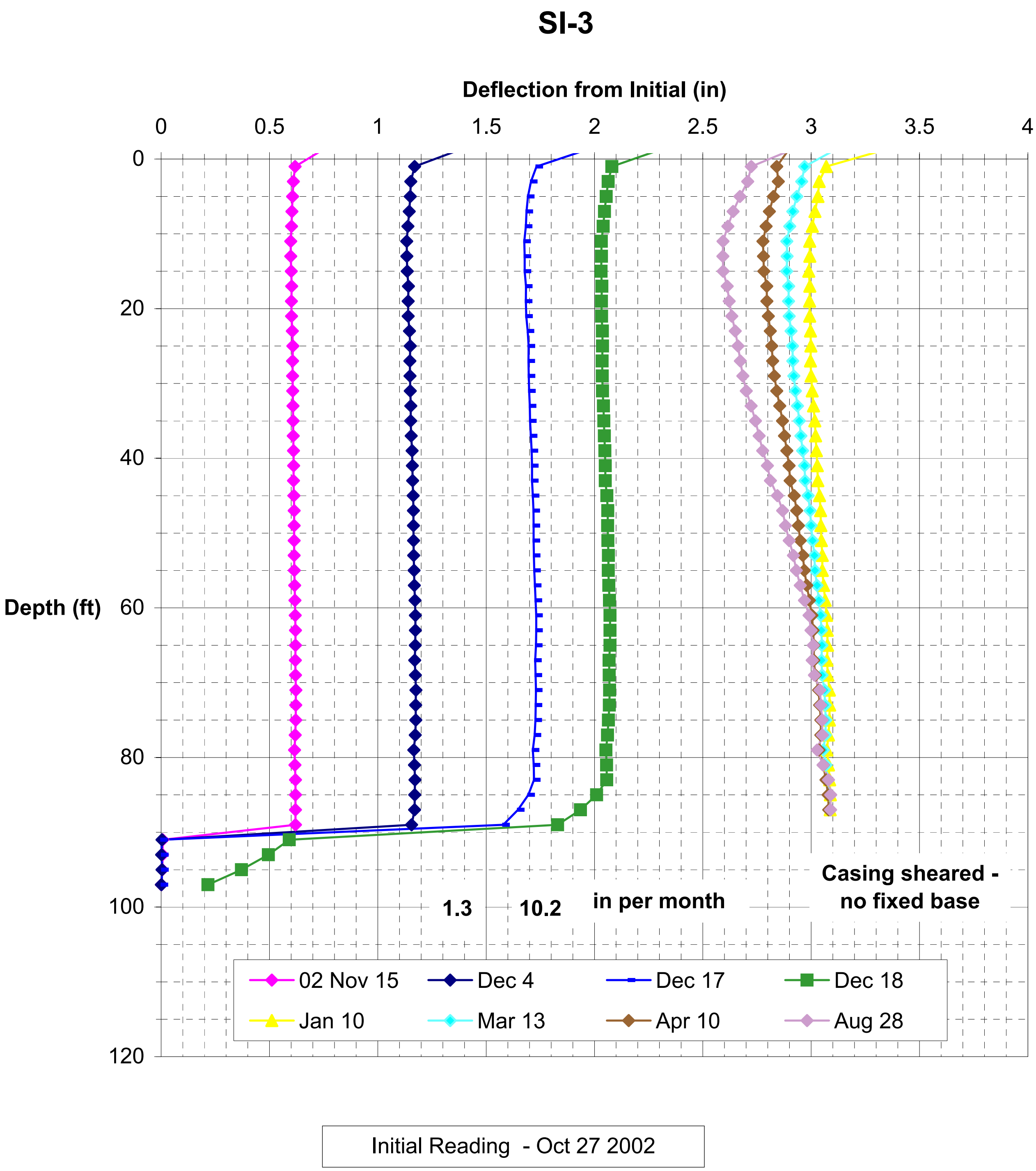
TH-GN-16
Piezometer at 47 ft below ground surface
Piezometer at 73 ft below ground surface
Elevation (ft)
Depth Below Ground Surface (ft)
TH-GN-16-47
TH-GN-16-73
Colluvium
TH-GN-16-47 Elevation = zero head
Bedrock
TH-GN-16-73 Elevation = zero head
Total Depth of Borehole = 73.5 ft.
9086.6
9076.6
9066.6
9056.6
9046.6
9036.6
9026.6
9016.6
0
10
20
30
40
50
60
70
7/14/2003
7/28/2003
8/11/2003
8/25/2003
9/8/2003
9/22/2003
10/6/2003
10/20/2003
11/3/2003
11/17/2003
12/1/2003
12/15/2003
12/29/2003
1/12/2004
1/26/2004
2/9/2004
2/23/2004
3/8/2004
3/22/2004
4/5/2004
4/19/2004
5/3/2004

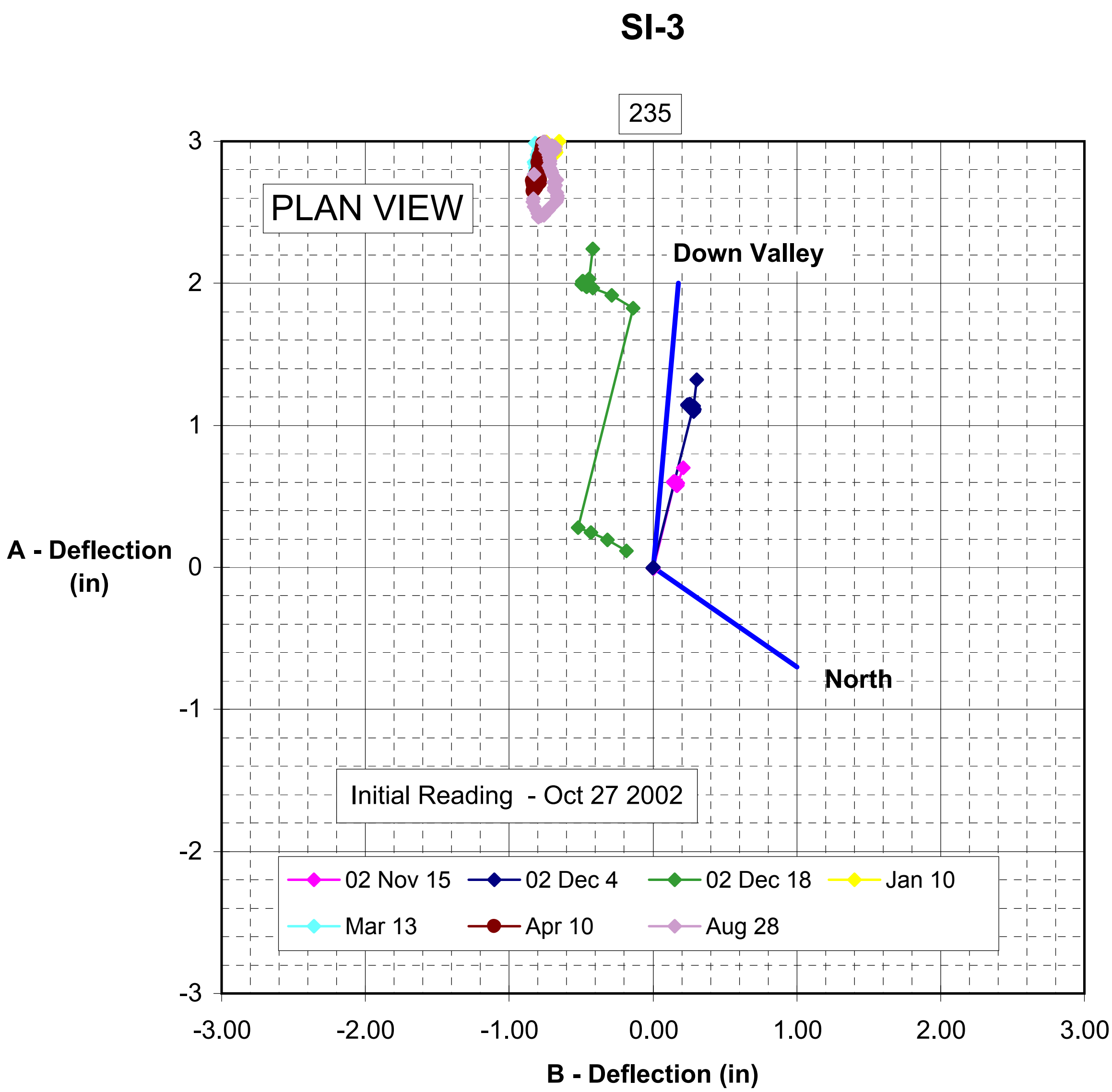
000339

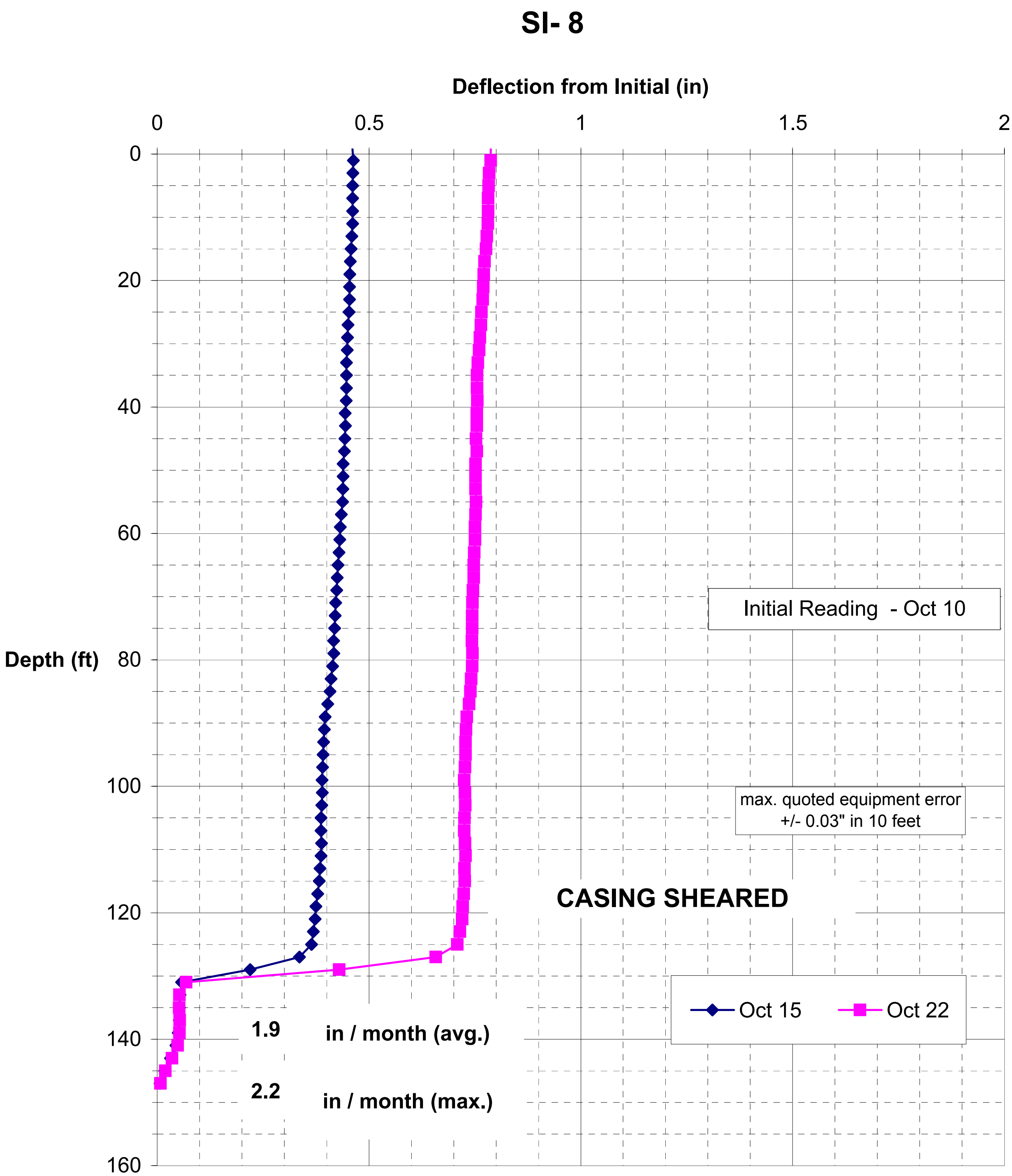
Figure H1
SI Movement Rates along Primary Shear Plane



000340


SI-3
Deflection from Initial (in)
0
0.5
1
1.5
2
2.5
3
3.5
4
Depth (ft)
0
20
40
60
80
100
120
1.3
10.2
in per month
Casing sheared -
no fixed base
02 Nov 15
Dec 4
Dec 17
Dec 18
Jan 10
Mar 13
Apr 10
Aug 28
Initial Reading - Oct 27 2002

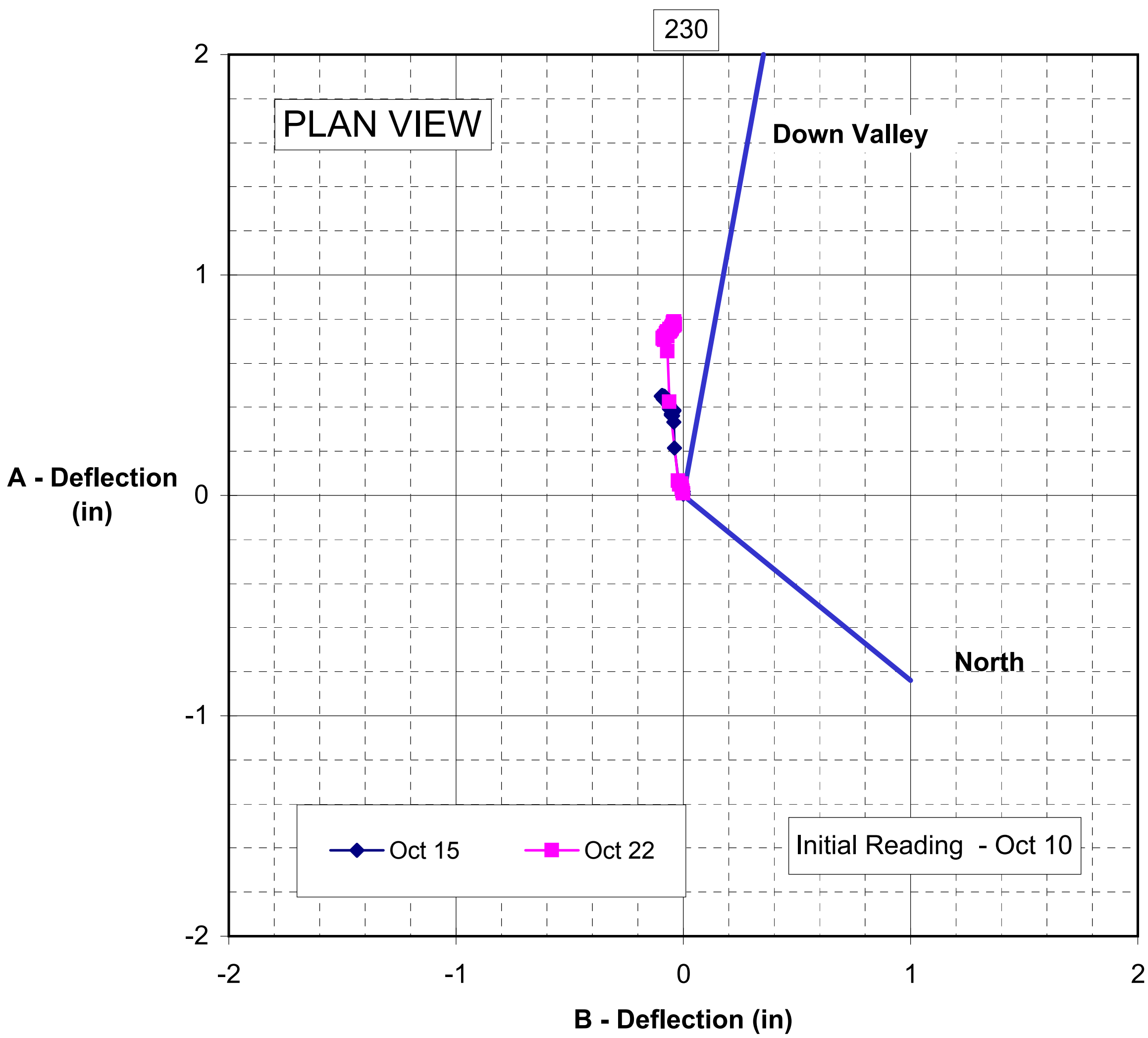

SI-3
235
PLAN VIEW
Down Valley
North
Initial Reading - Oct 27 2002
02 Nov 15
02 Dec 4
02 Dec 18
Jan 10
Mar 13
Apr 10
Aug 28
A - Deflection (in)
B - Deflection (in)
3
2
1
0
-1
-2
-3
-3.00
-2.00
-1.00
0.00
1.00
2.00
3.00



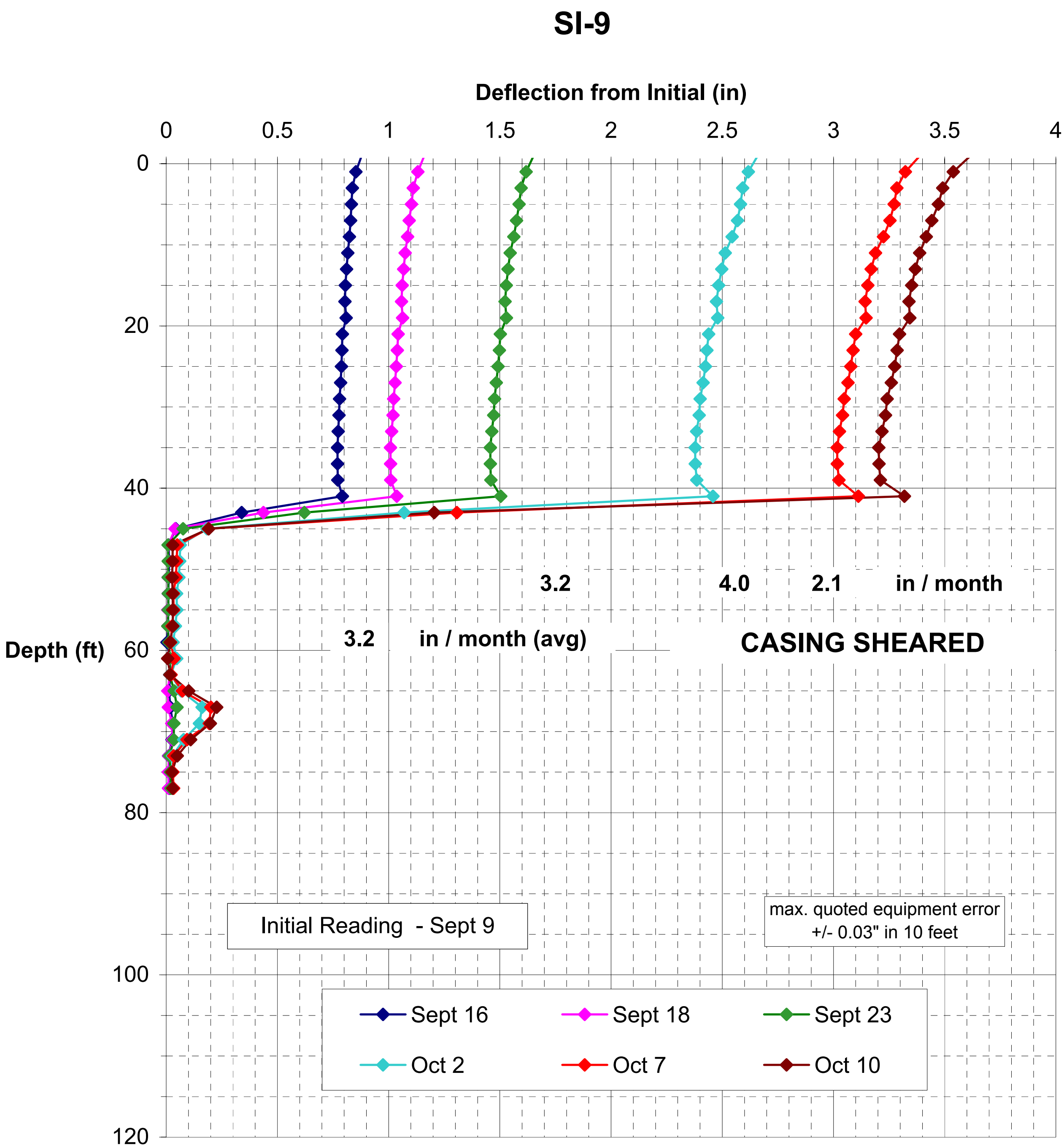
SI- 8
Deflection from Initial (in)
0
0.5
1
1.5
2
Depth (ft)
0
20
40
60
80
100
120
140
160
Initial Reading - Oct 10
max. quoted equipment error
+/- 0.03" in 10 feet
CASING SHEARED
Oct 15
Oct 22
1.9
in / month (avg.)
2.2
in / month (max.)

000343

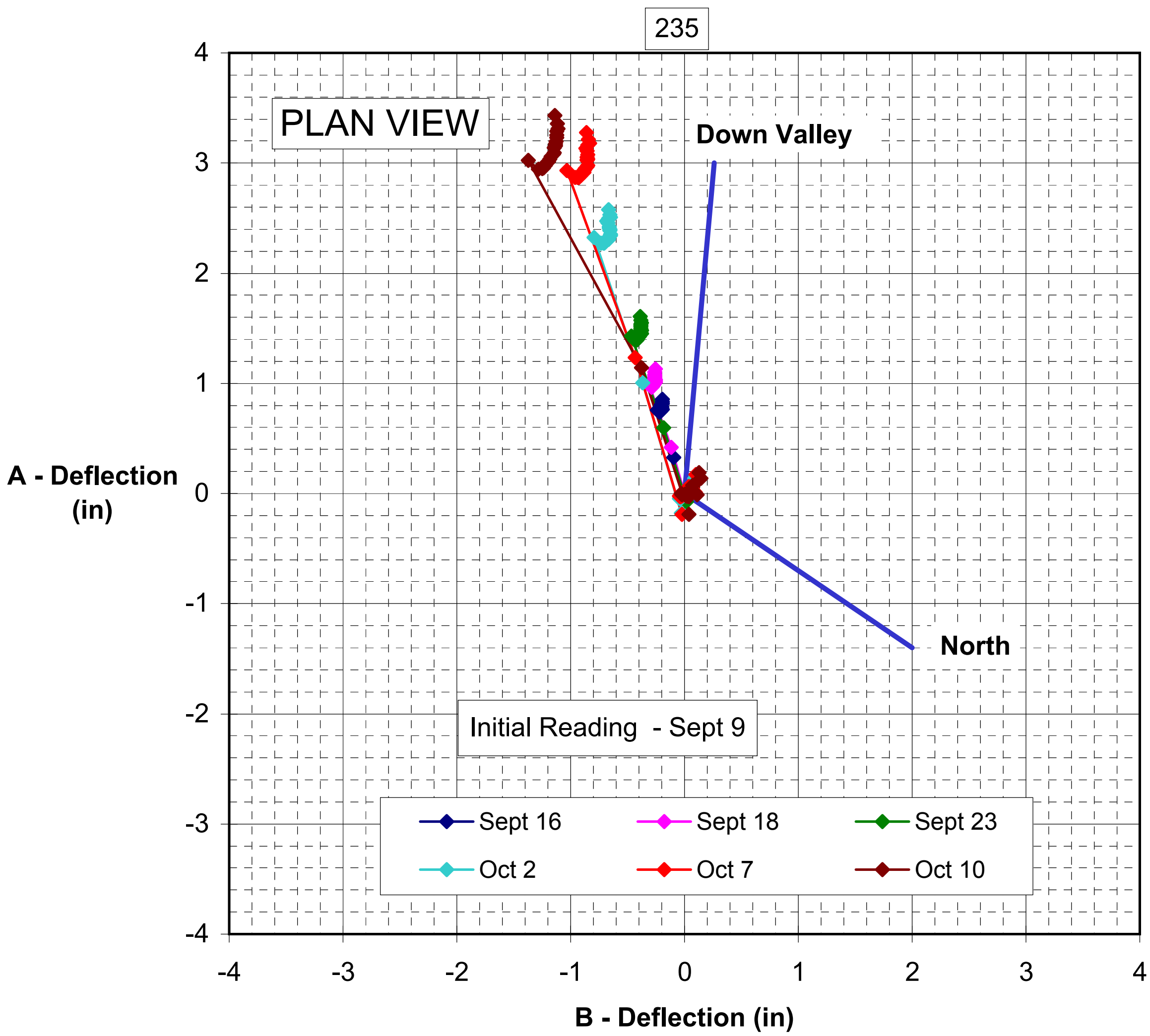
SI- 8

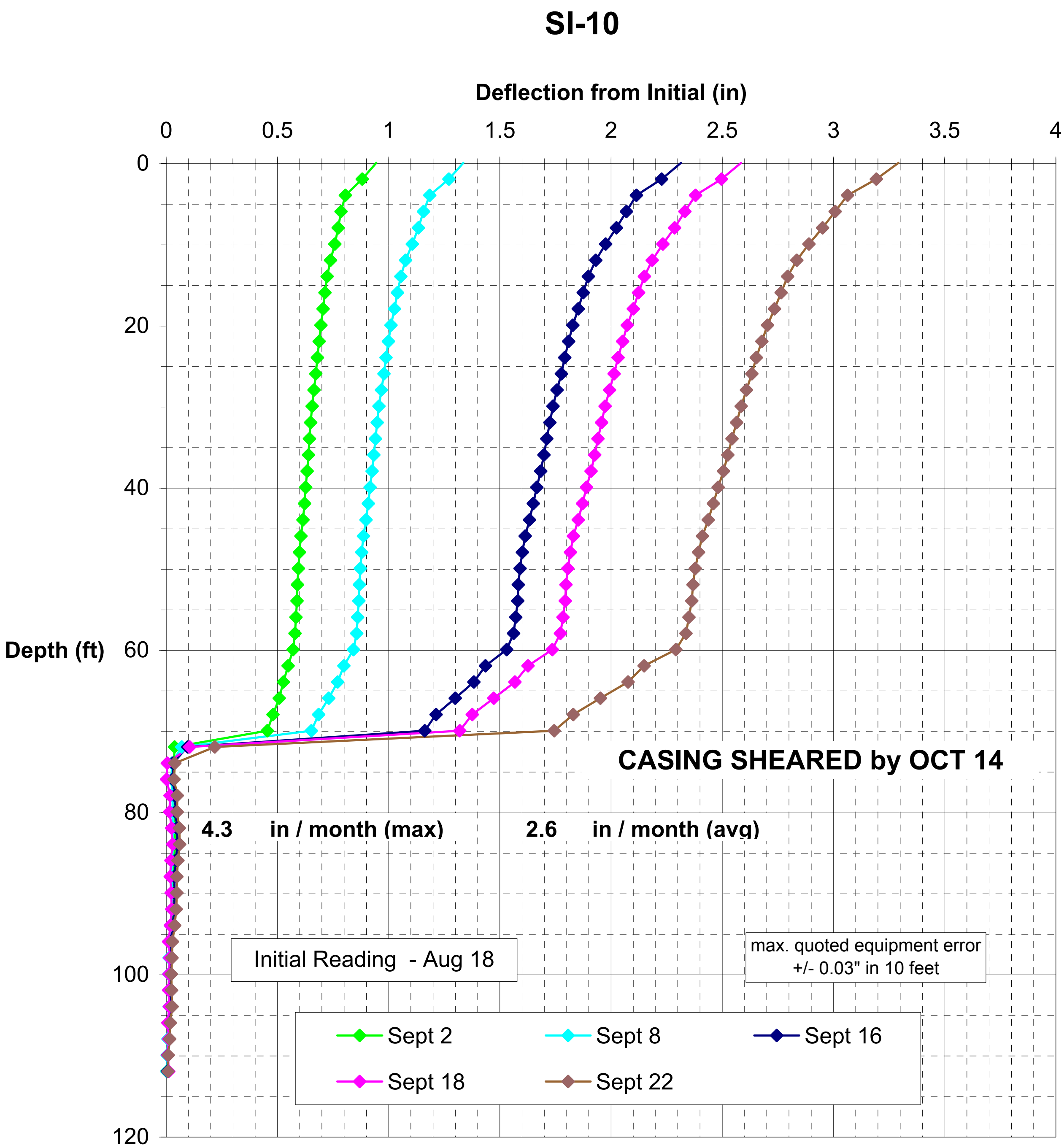


230
PLAN VIEW
Down Valley
North
A - Deflection (in)
B - Deflection (in)
2
1
0
-1
-2
-2
-1
0
1
2
Oct 15
Oct 22
Initial Reading - Oct 10


SI-9
Deflection from Initial (in)
0
0.5
1
1.5
2
2.5
3
3.5
4
Depth (ft)
0
20
40
60
80
100
120
3.2
4.0
2.1
in / month
3.2
in / month (avg)
CASING SHEARED
Initial Reading - Sept 9
max. quoted equipment error
+/- 0.03" in 10 feet
Sept 16
Sept 18
Sept 23
Oct 2
Oct 7
Oct 10

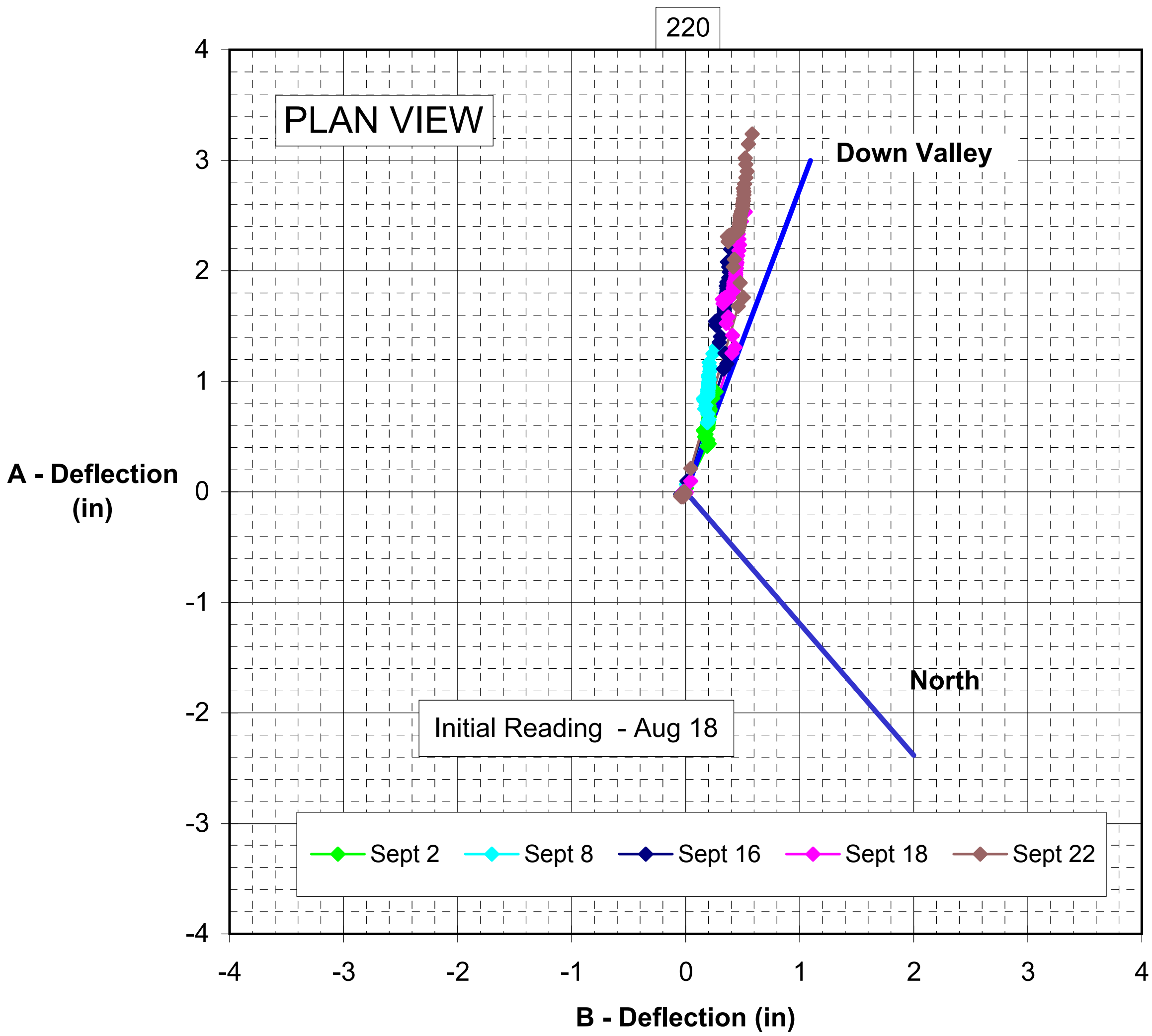
SI-9

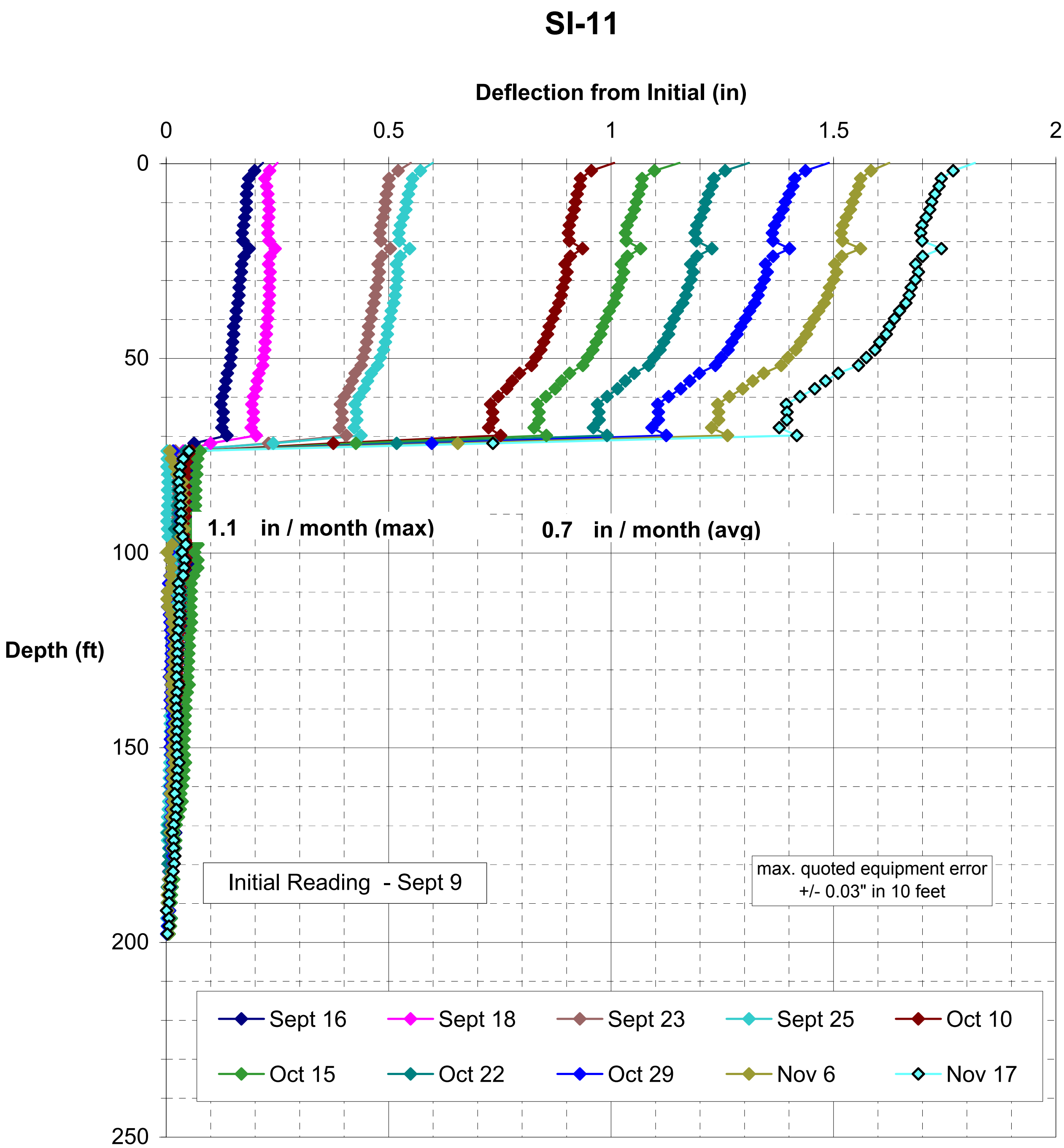

235
PLAN VIEW
Down Valley
North
Initial Reading - Sept 9
Sept 16
Sept 18
Sept 23
Oct 2
Oct 7
Oct 10
A - Deflection (in)
B - Deflection (in)
4
3
2
1
0
-1
-2
-3
-4



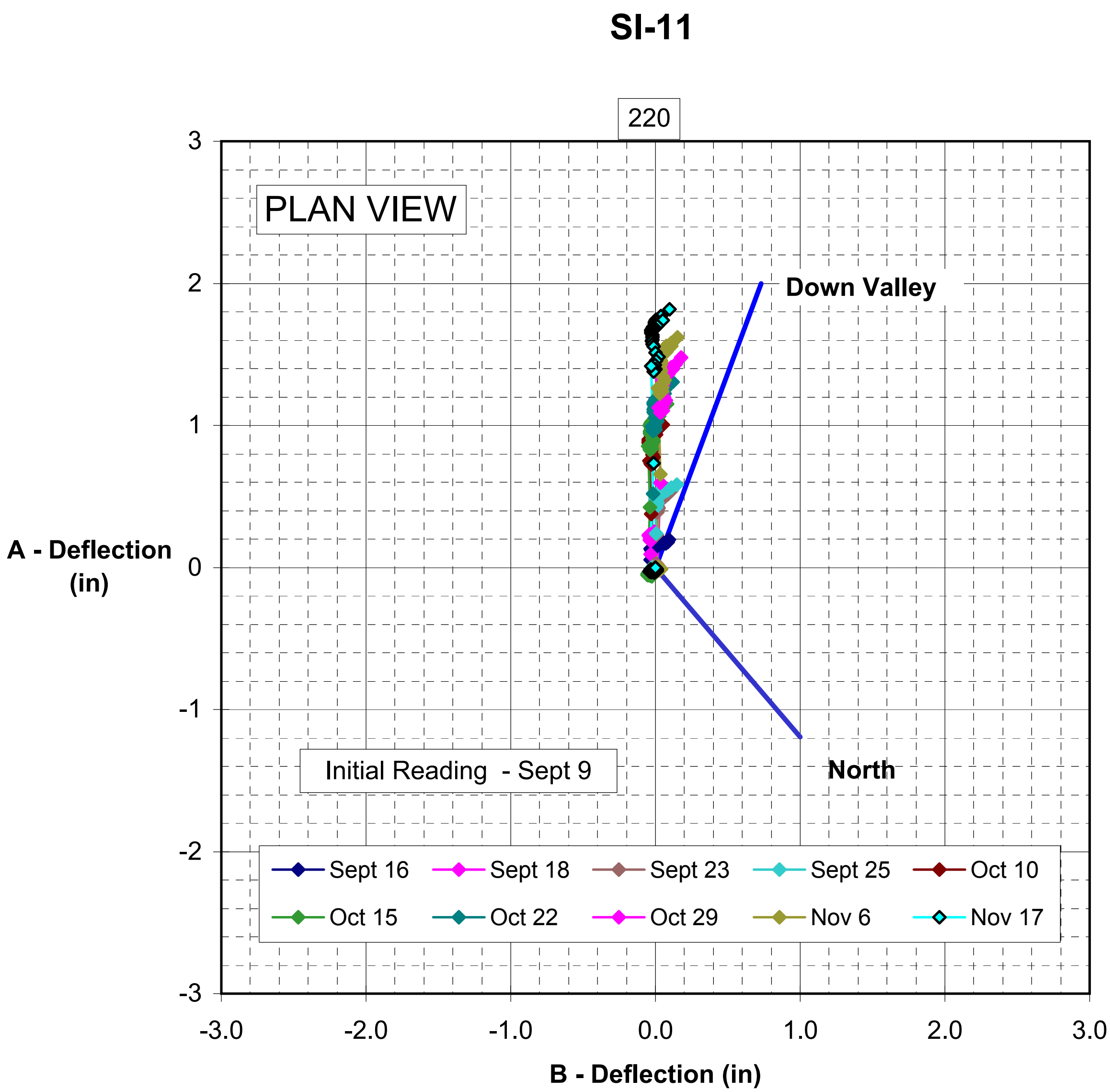
SI-10
Deflection from Initial (in)
0
0.5
1
1.5
2
2.5
3
3.5
4
0
20
40
Depth (ft)
60
80
100
120
CASING SHEARED by OCT 14
4.3 in / month (max)
2.6 in / month (avg)
Initial Reading - Aug 18
max. quoted equipment error
+/- 0.03" in 10 feet
Sept 2
Sept 8
Sept 16
Sept 18
Sept 22

SI-10

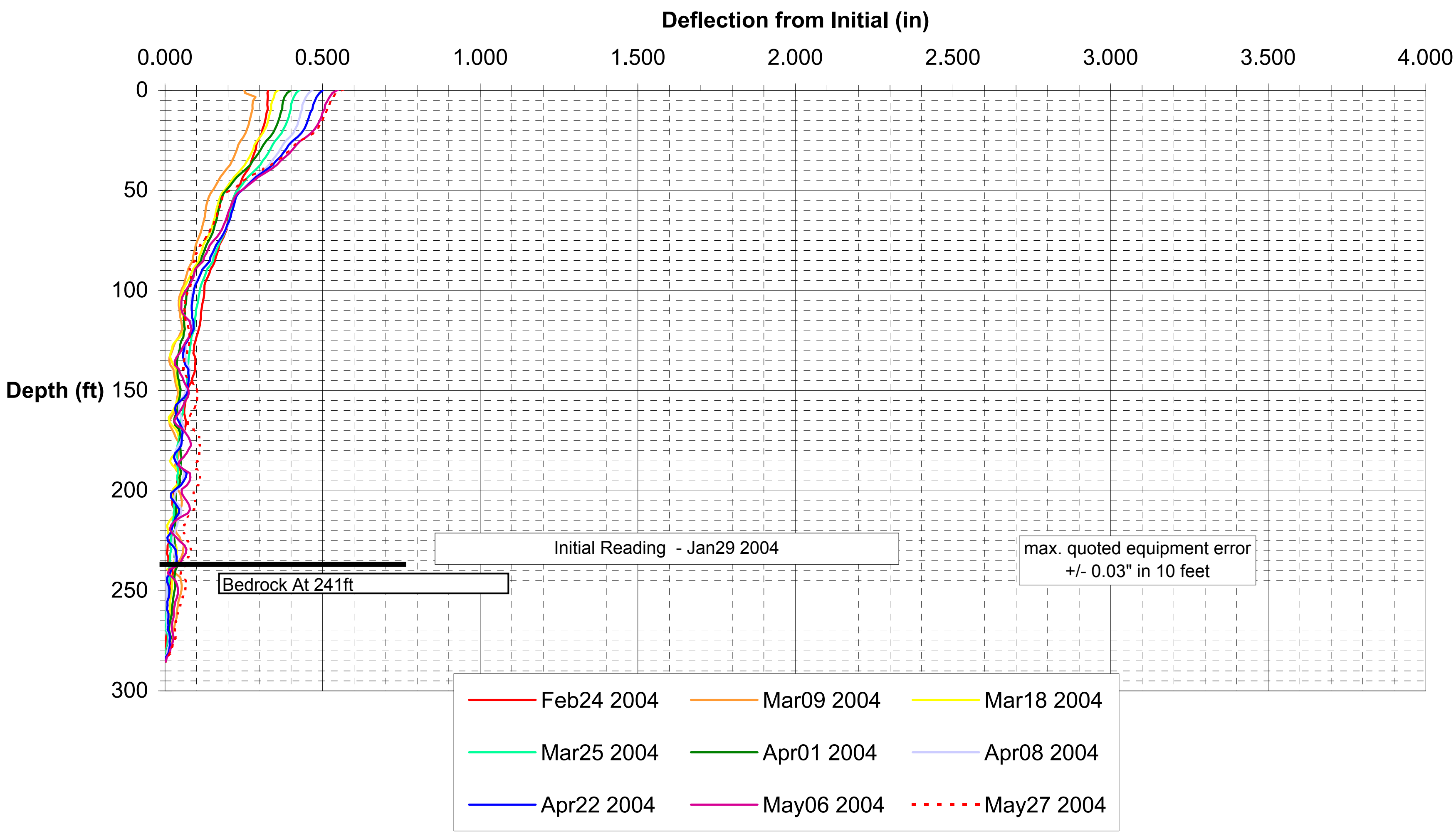

220
PLAN VIEW
Down Valley
North
Initial Reading - Aug 18
Sept 2
Sept 8
Sept 16
Sept 18
Sept 22
A - Deflection (in)
B - Deflection (in)
4
3
2
1
0
-1
-2
-3
-4



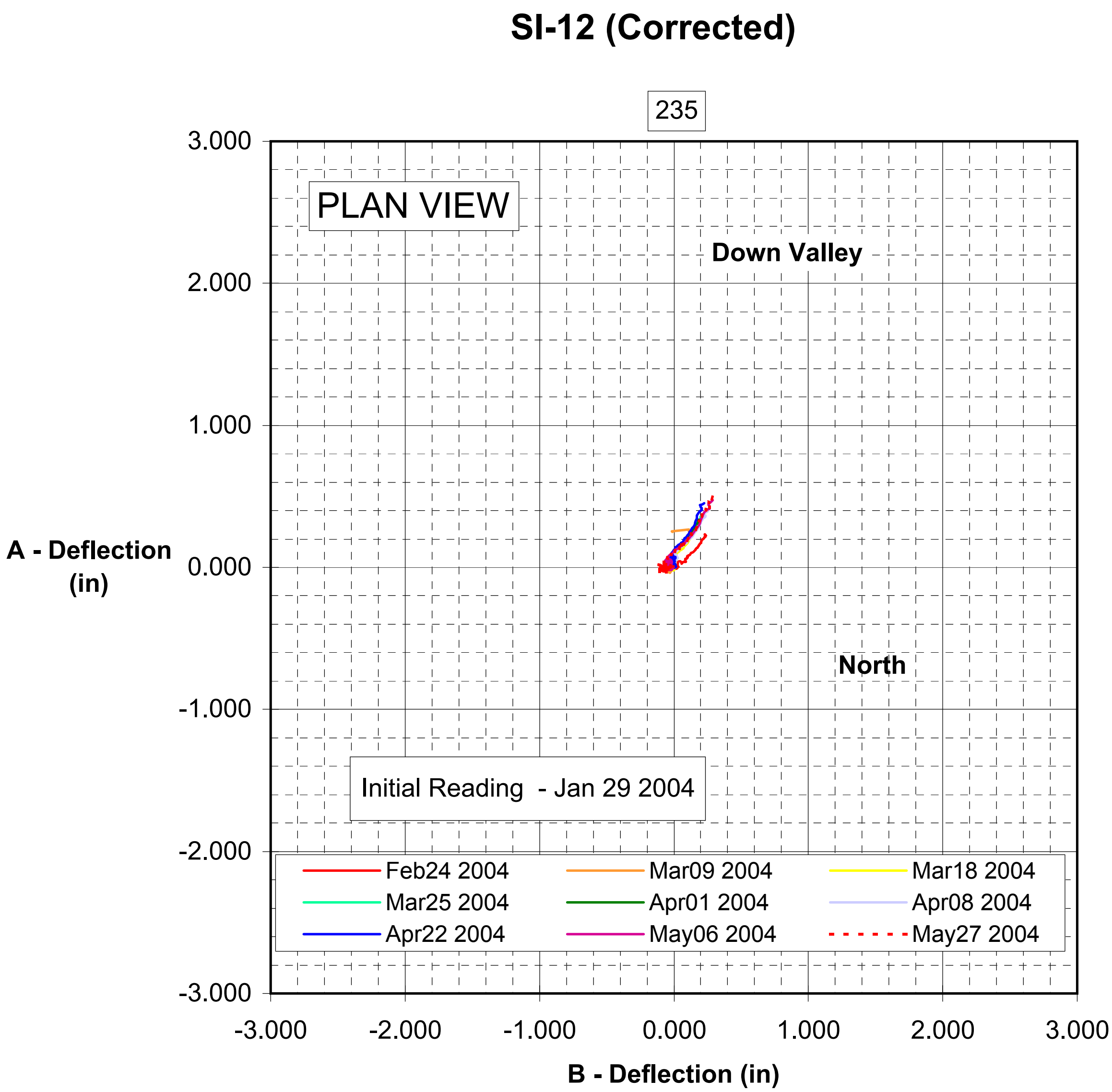
SI-11
Deflection from Initial (in)
0
0.5
1
1.5
2
0
50
100
150
200
250
Depth (ft)
1.1 in / month (max)
0.7 in / month (avg)
Initial Reading - Sept 9
max. quoted equipment error
+/- 0.03" in 10 feet
Sept 16
Sept 18
Sept 23
Sept 25
Oct 10
Oct 15
Oct 22
Oct 29
Nov 6
Nov 17


SI-11
220
PLAN VIEW
Down Valley
North
Initial Reading - Sept 9
A - Deflection (in)
B - Deflection (in)
3
2
1
0
-1
-2
-3
-3.0
-2.0
-1.0
0.0
1.0
2.0
3.0
Sept 16
Sept 18
Sept 23
Sept 25
Oct 10
Oct 15
Oct 22
Oct 29
Nov 6
Nov 17

SI-12 (Corrected)

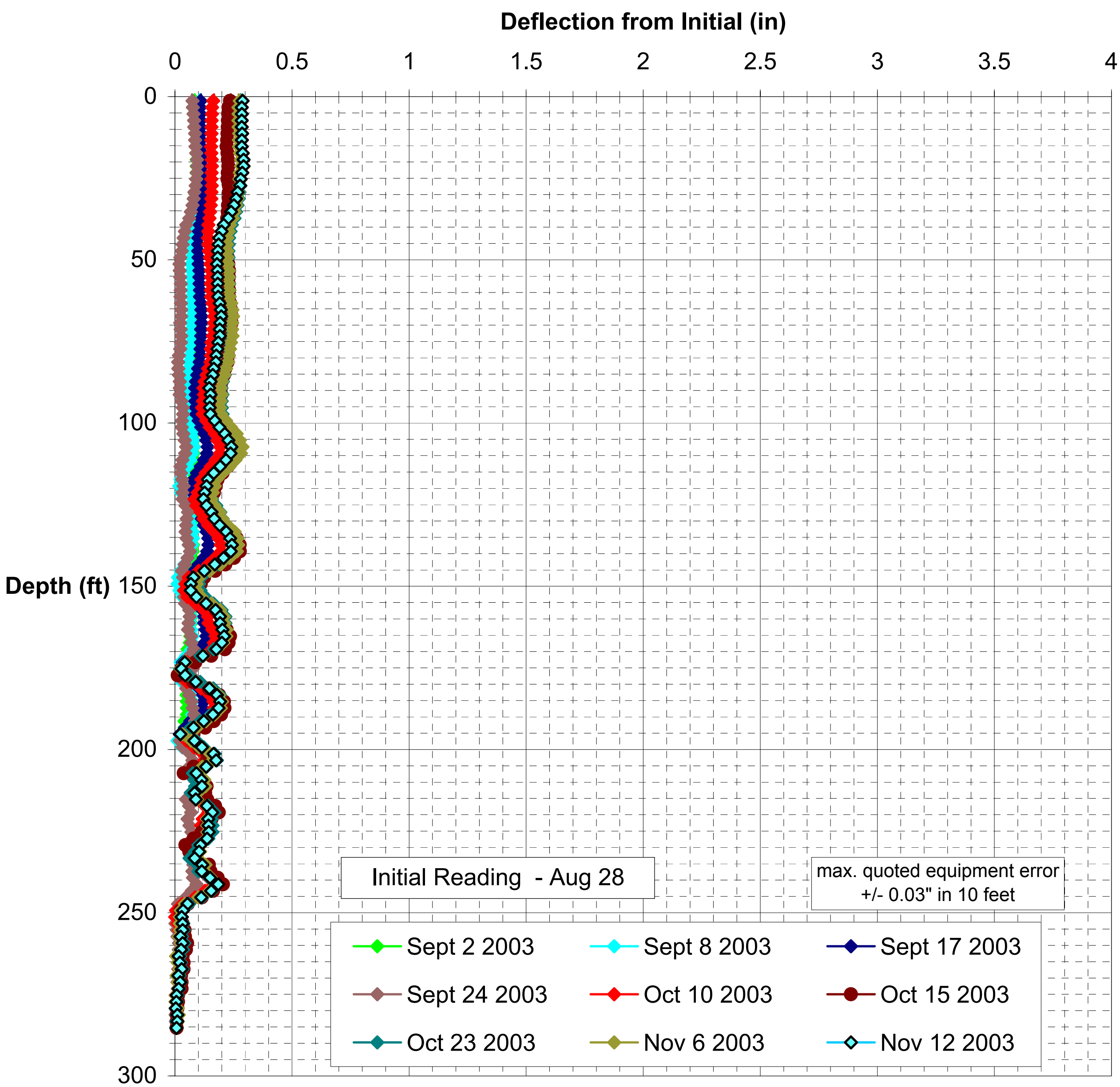


000351



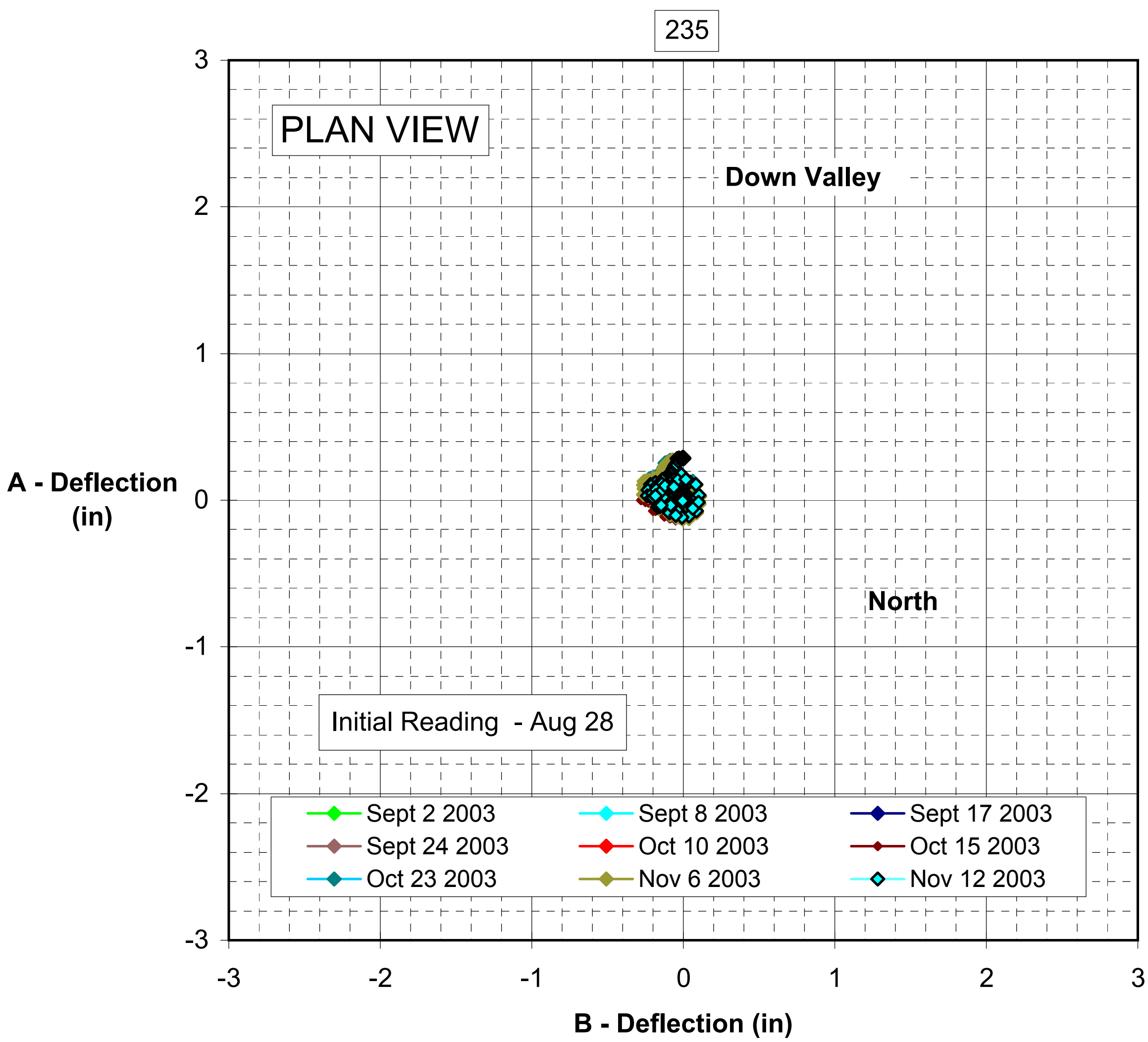
SI-12 (Corrected)
235
PLAN VIEW
Down Valley
North
A - Deflection (in)
B - Deflection (in)
3.000
2.000
1.000
0.000
-1.000
-2.000
-3.000
Initial Reading - Jan 29 2004
Feb24 2004
Mar09 2004
Mar18 2004
Mar25 2004
Apr01 2004
Apr08 2004
Apr22 2004
May06 2004
May27 2004

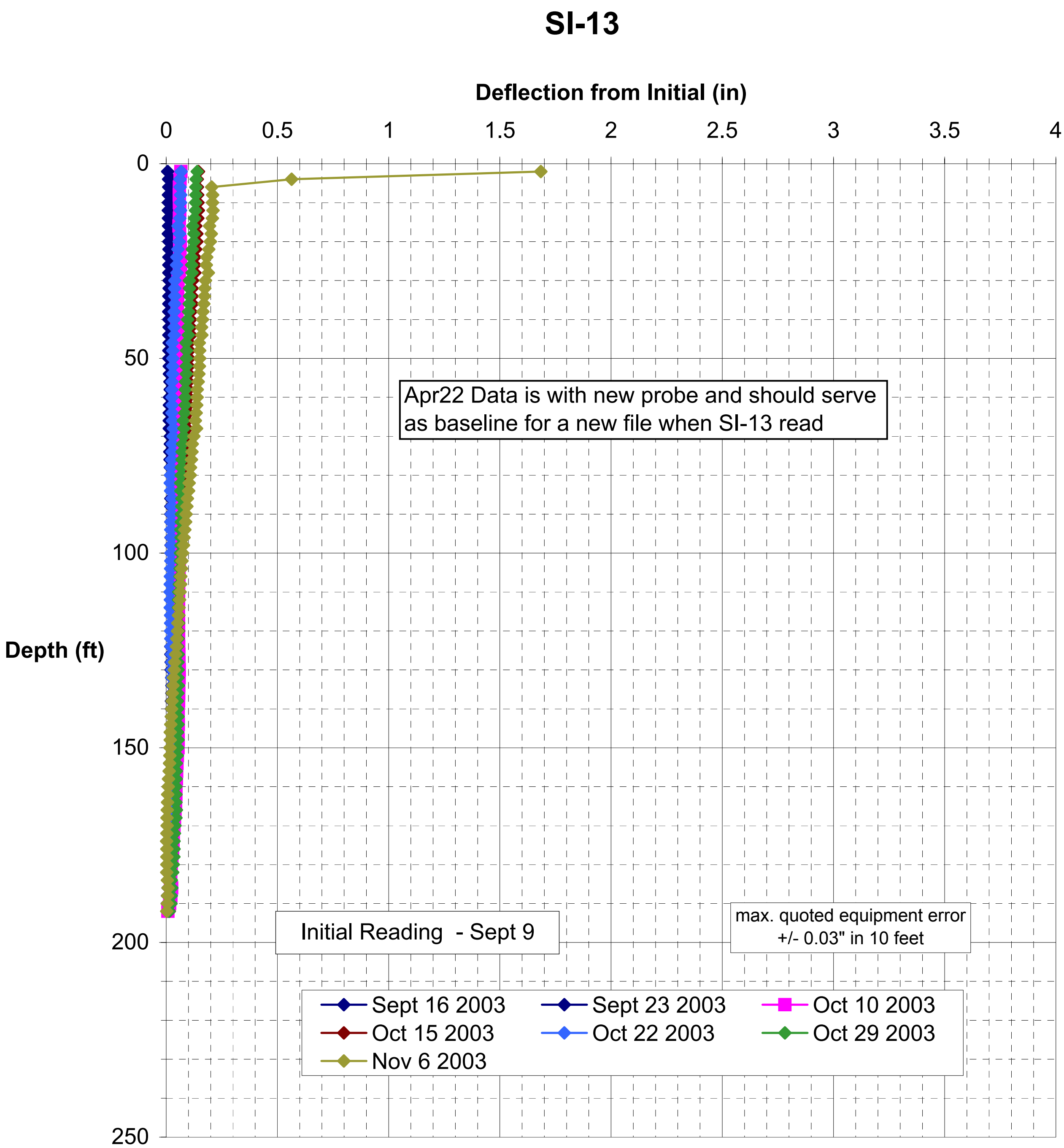
SI-12



Deflection from Initial (in)
0
0.5
1
1.5
2
2.5
3
3.5
4
Depth (ft)
0
50
100
150
200
250
300
Initial Reading - Aug 28
max. quoted equipment error
+/- 0.03" in 10 feet
Sept 2 2003
Sept 8 2003
Sept 17 2003
Sept 24 2003
Oct 10 2003
Oct 15 2003
Oct 23 2003
Nov 6 2003
Nov 12 2003

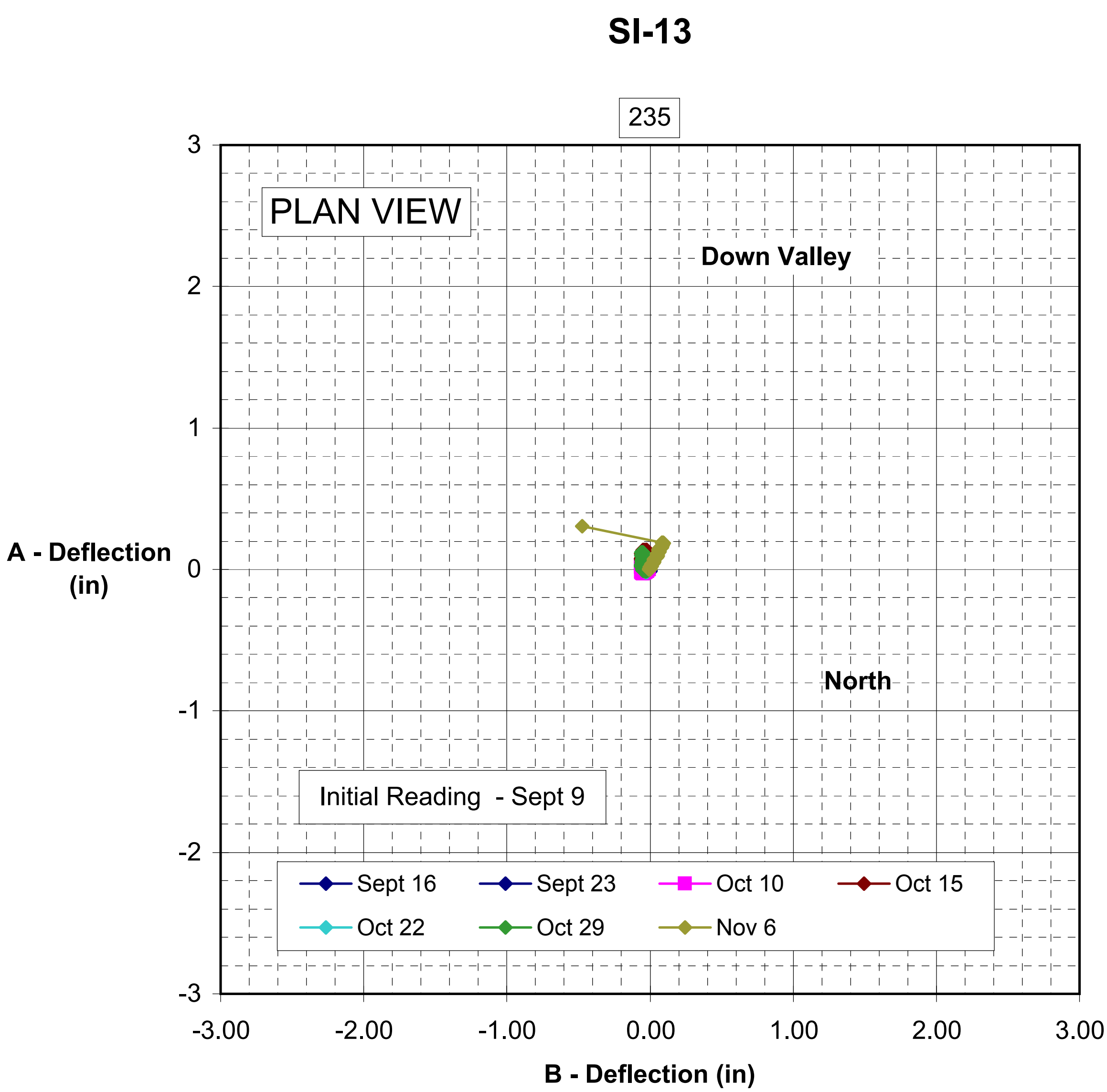
SI-12


235
PLAN VIEW
Down Valley
North
Initial Reading - Aug 28
A - Deflection (in)
B - Deflection (in)
3
2
1
0
-1
-2
-3
Sept 2 2003
Sept 8 2003
Sept 17 2003
Sept 24 2003
Oct 10 2003
Oct 15 2003
Oct 23 2003
Nov 6 2003
Nov 12 2003



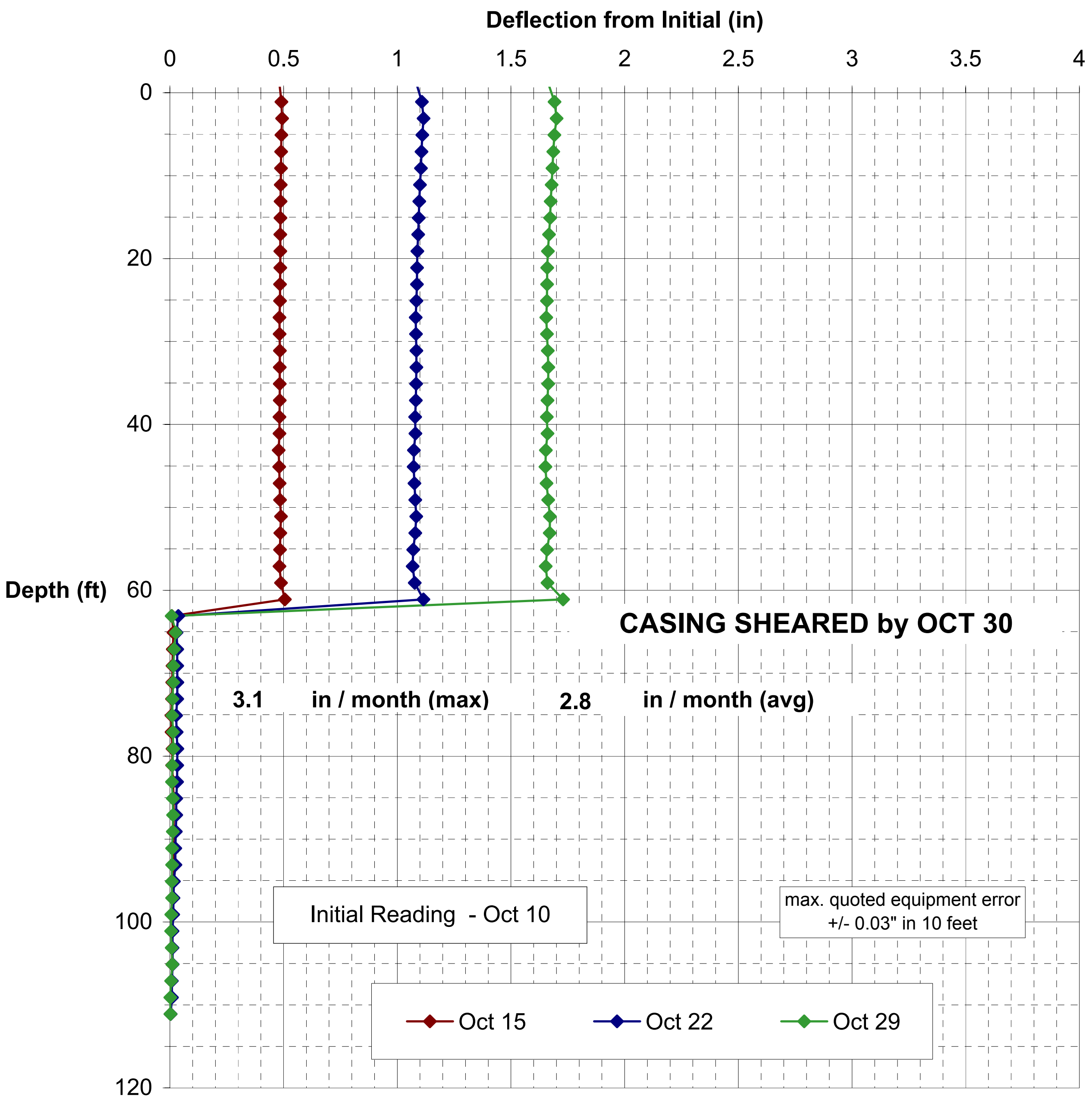
SI-13
Deflection from Initial (in)
0
0.5
1
1.5
2
2.5
3
3.5
4
0
50
100
150
200
250
Depth (ft)
Apr22 Data is with new probe and should serve as baseline for a new file when SI-13 read
Initial Reading - Sept 9
max. quoted equipment error +/- 0.03" in 10 feet
Sept 16 2003
Sept 23 2003
Oct 10 2003
Oct 15 2003
Oct 22 2003
Oct 29 2003
Nov 6 2003

09/08/2004



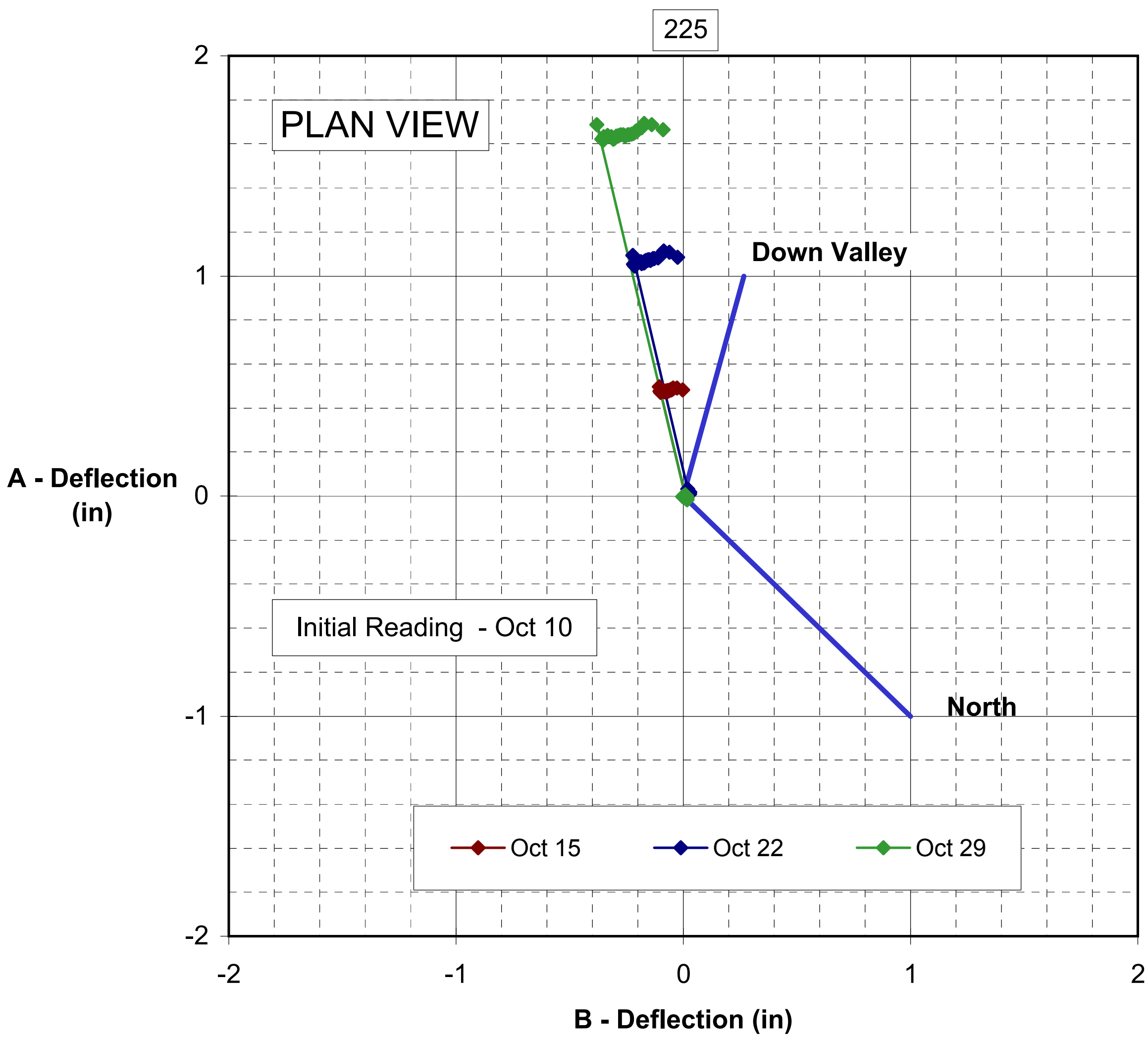
SI-13.xls

SI- 17

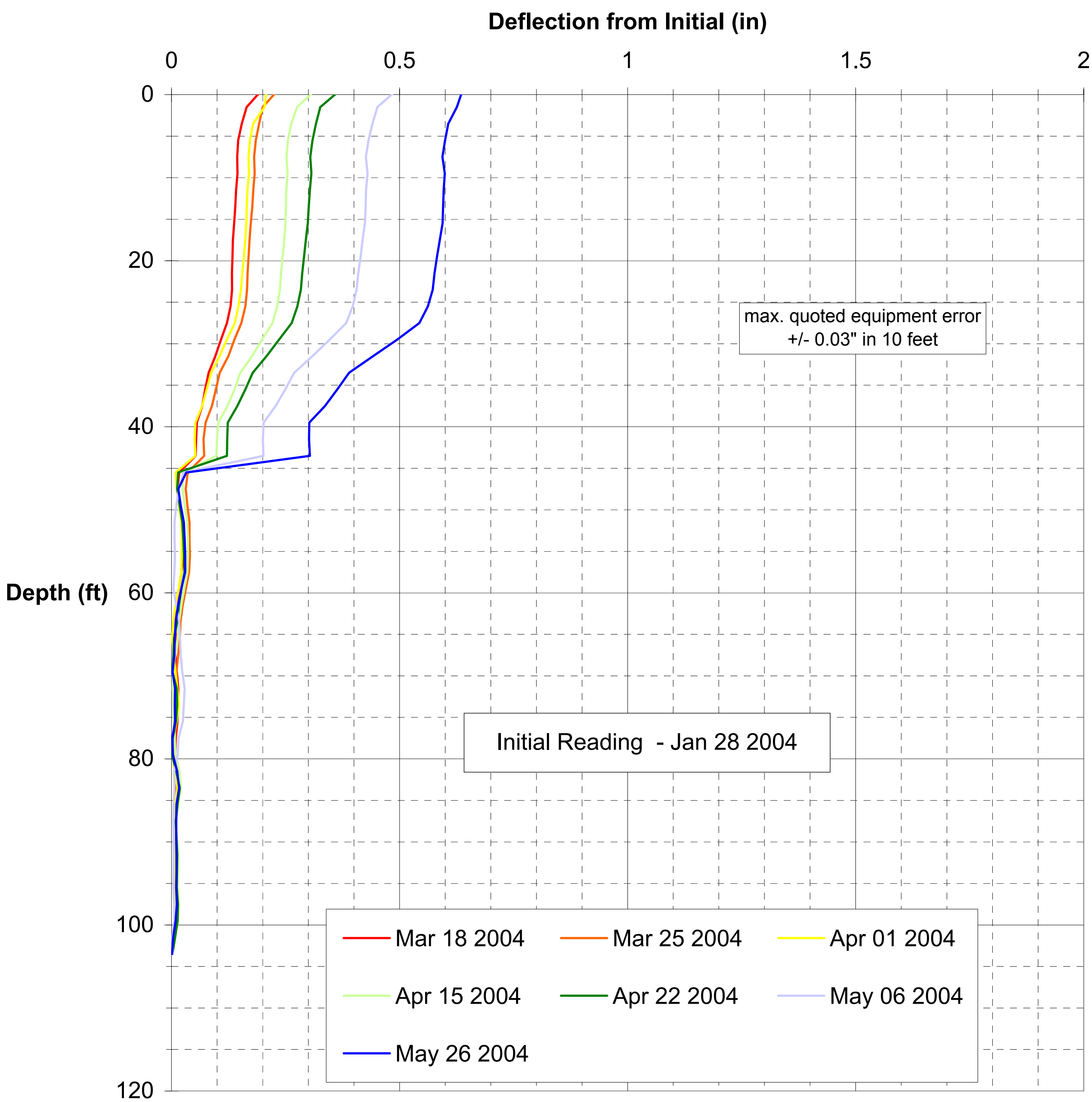


Deflection from Initial (in)
0
0.5
1
1.5
2
2.5
3
3.5
4
Depth (ft)
0
20
40
60
80
100
120
CASING SHEARED by OCT 30
3.1 in / month (max)
2.8 in / month (avg)
Initial Reading - Oct 10
max. quoted equipment error +/- 0.03" in 10 feet
Oct 15
Oct 22
Oct 29

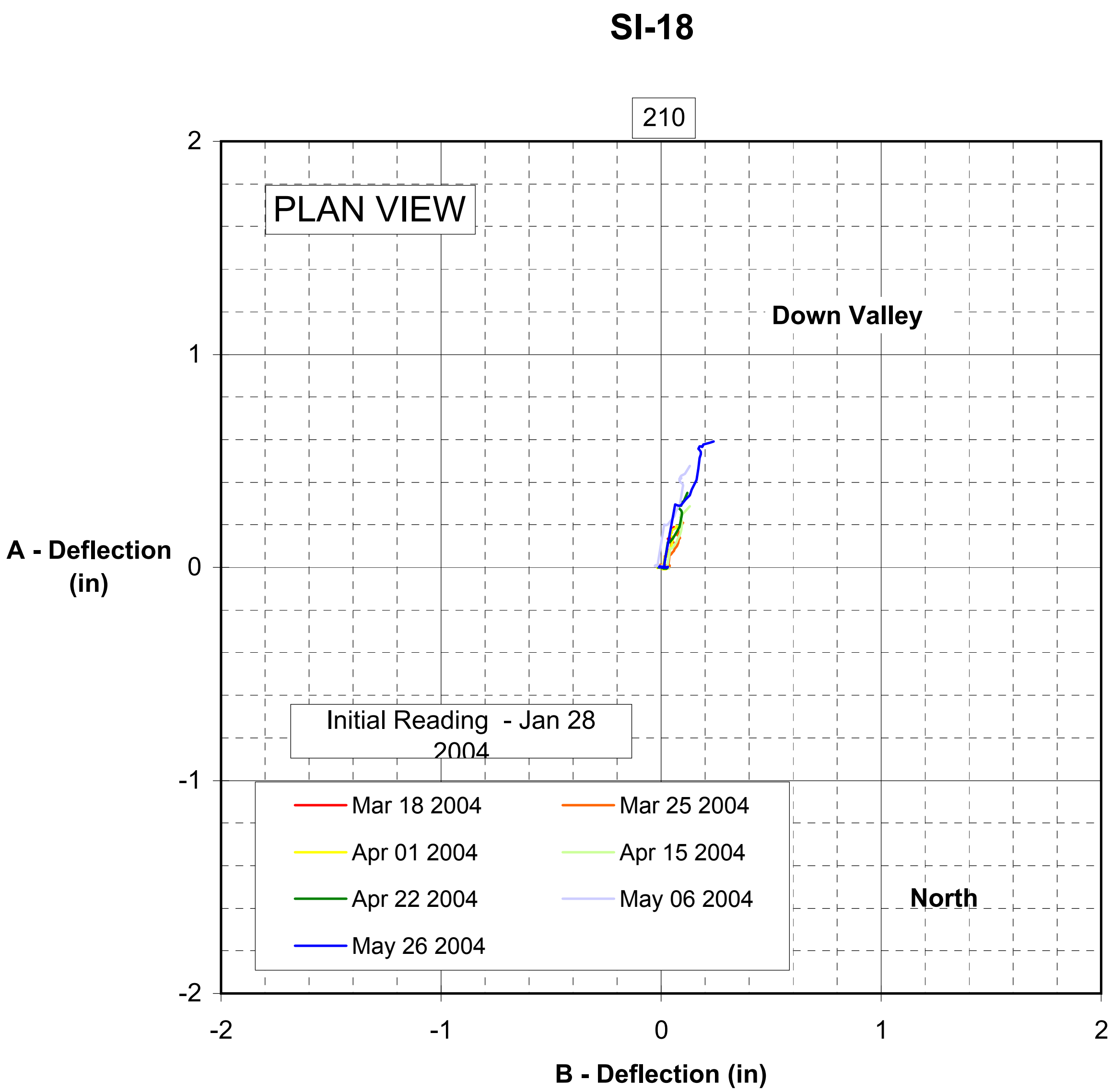
SI- 17

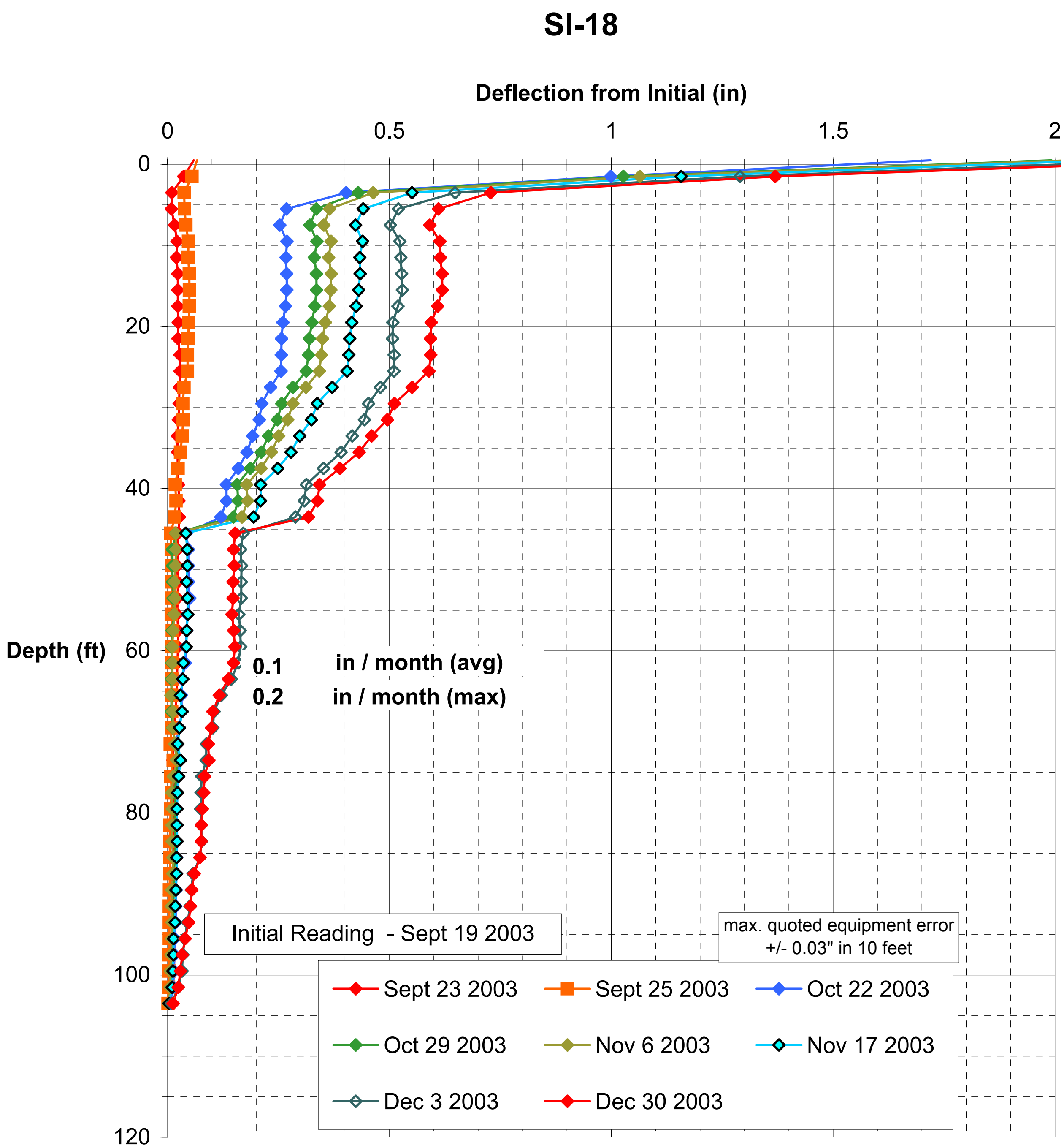

225
2
1
0
-1
-2
PLAN VIEW
Down Valley
A - Deflection (in)
Initial Reading - Oct 10
North
Oct 15
Oct 22
Oct 29
-2
-1
0
1
2
B - Deflection (in)

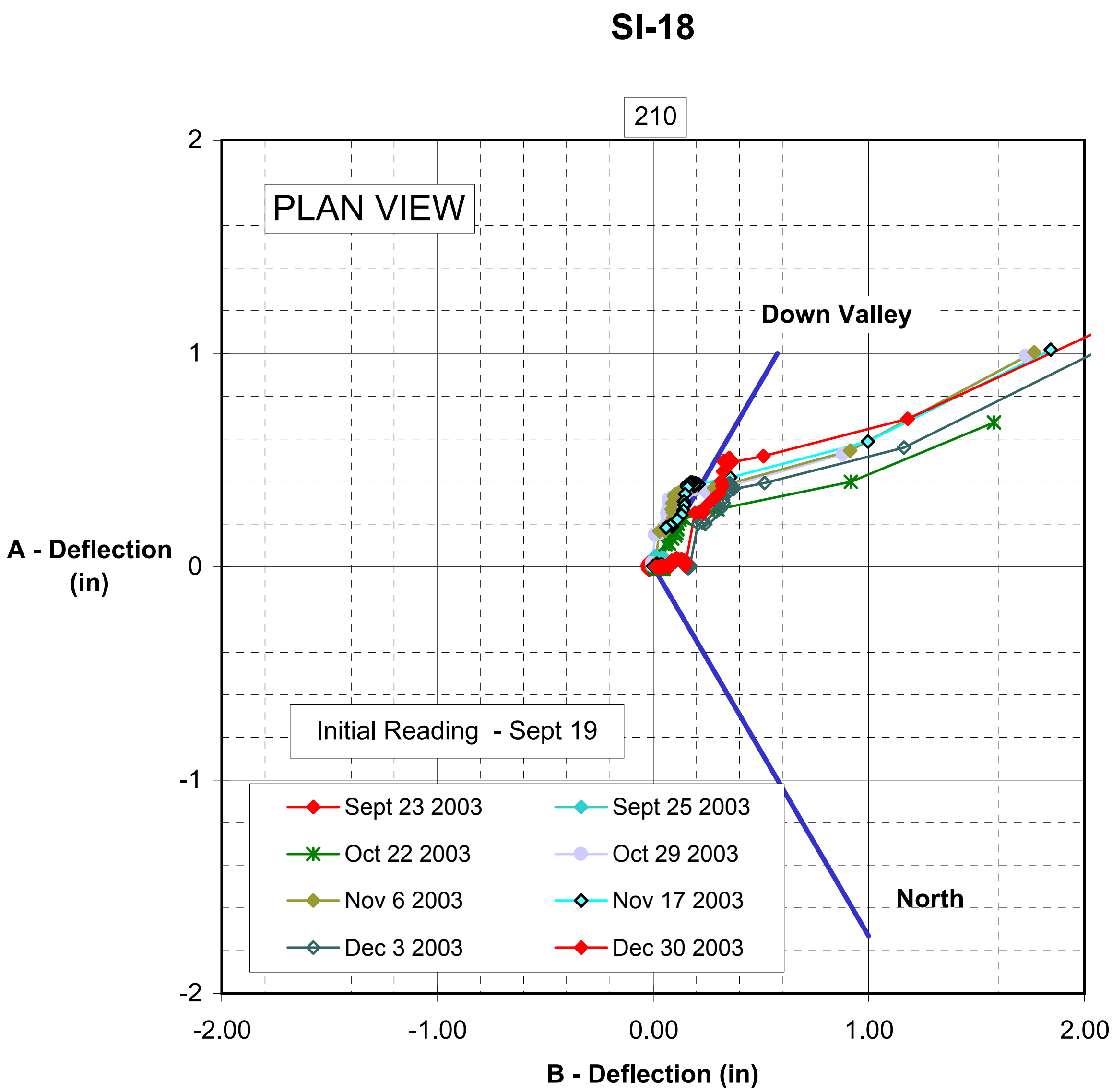
SI-18



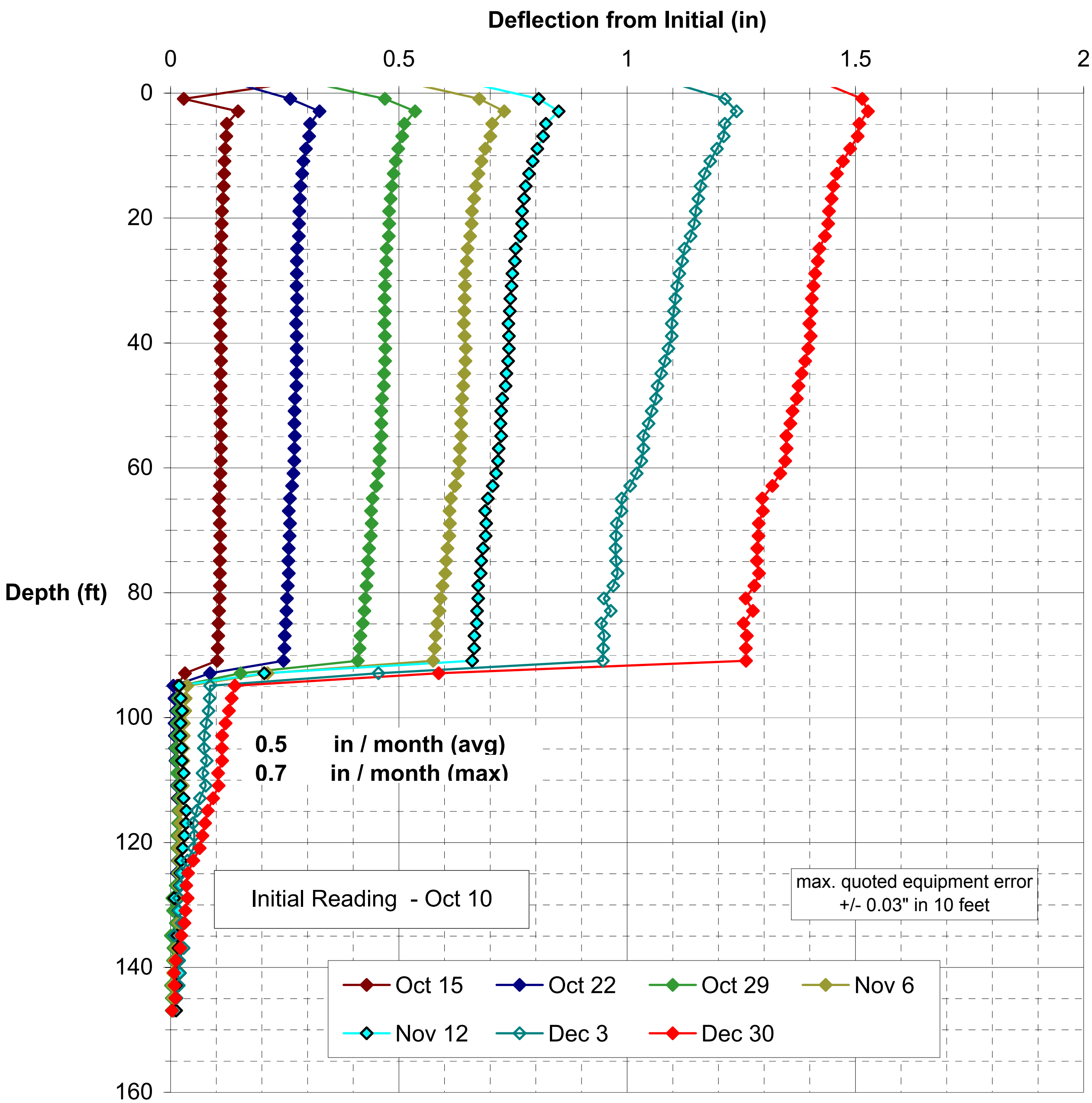
Deflection from Initial (in)
0
0.5
1
1.5
2
Depth (ft)
0
20
40
60
80
100
120
max. quoted equipment error
+/- 0.03" in 10 feet
Initial Reading - Jan 28 2004
Mar 18 2004
Mar 25 2004
Apr 01 2004
Apr 15 2004
Apr 22 2004
May 06 2004
May 26 2004


SI-18
210
PLAN VIEW
Down Valley
A - Deflection (in)
B - Deflection (in)
Initial Reading - Jan 28 2004
Mar 18 2004
Mar 25 2004
Apr 01 2004
Apr 15 2004
Apr 22 2004
May 06 2004
May 26 2004
North
2
1
0
-1
-2
-2
-1
0
1
2

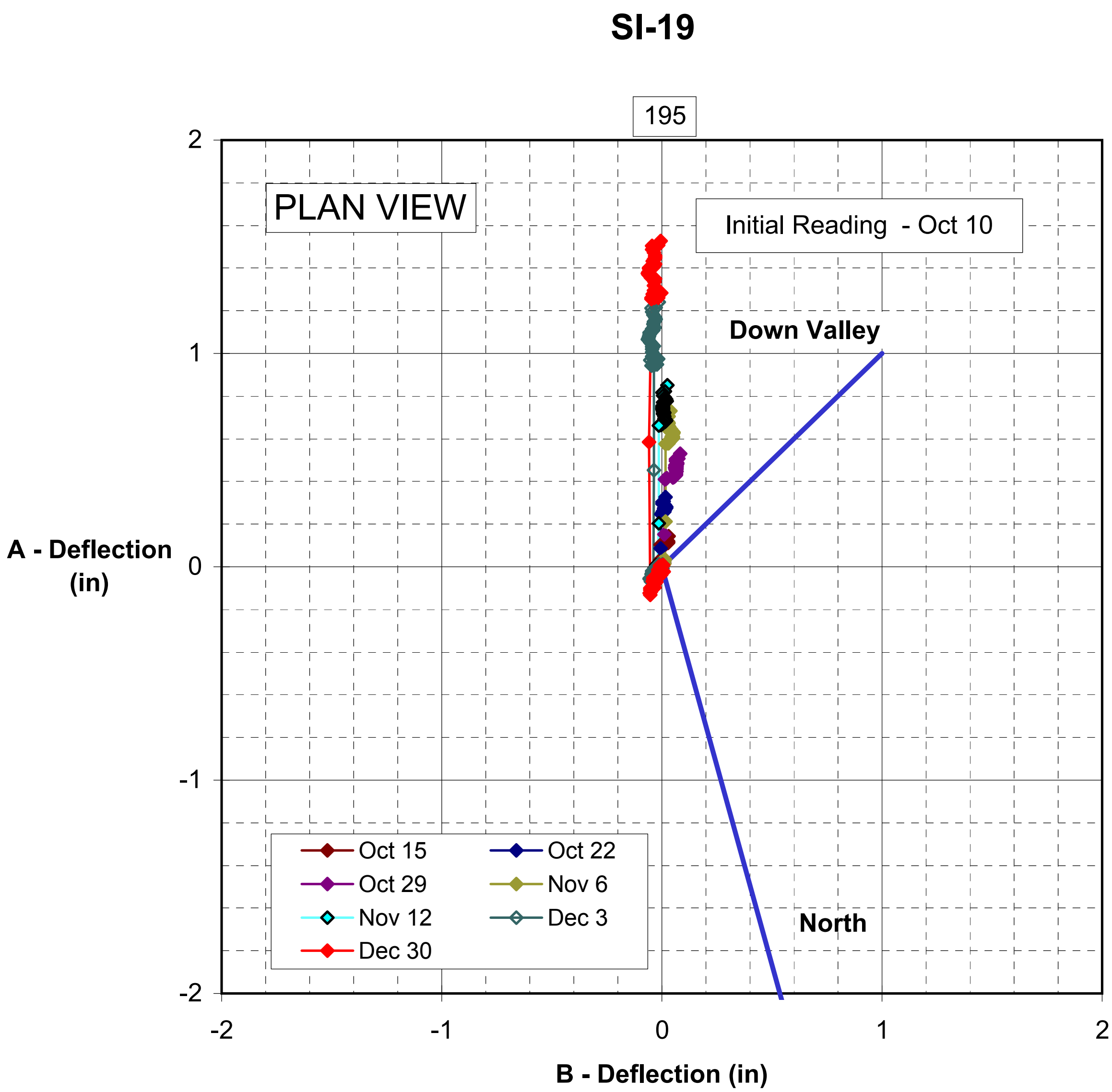

SI-18
Deflection from Initial (in)
0
0.5
1
1.5
2
Depth (ft)
0
20
40
60
80
100
120
0.1 in / month (avg)
0.2 in / month (max)
Initial Reading - Sept 19 2003
max. quoted equipment error
+/- 0.03" in 10 feet
Sept 23 2003
Sept 25 2003
Oct 22 2003
Oct 29 2003
Nov 6 2003
Nov 17 2003
Dec 3 2003
Dec 30 2003


SI-18
210
PLAN VIEW
Down Valley
North
Initial Reading - Sept 19
Sept 23 2003
Sept 25 2003
Oct 22 2003
Oct 29 2003
Nov 6 2003
Nov 17 2003
Dec 3 2003
Dec 30 2003
A - Deflection (in)
B - Deflection (in)
2
1
0
-1
-2
-2.00
-1.00
0.00
1.00
2.00

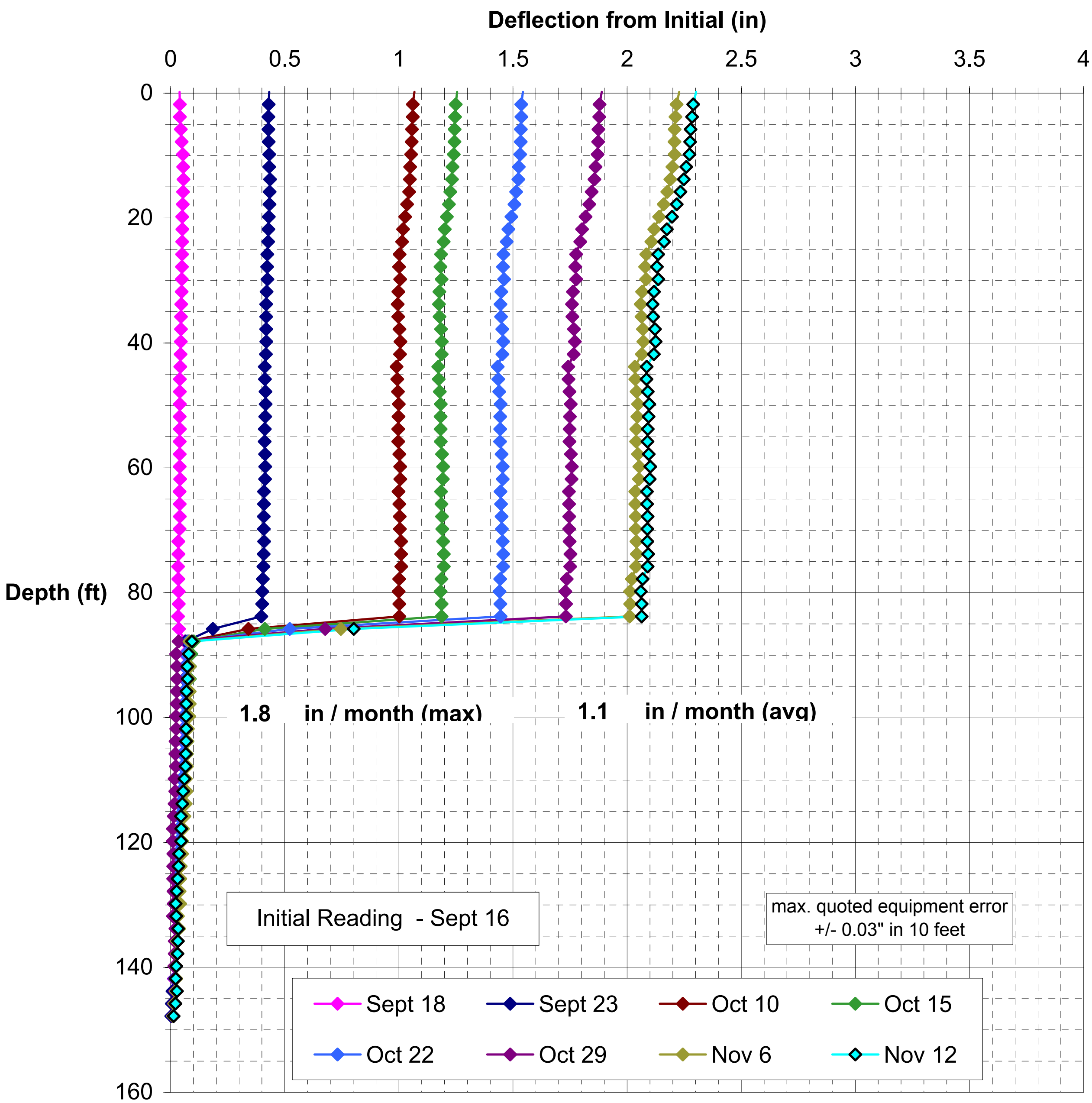
SI-19



Deflection from Initial (in)
0
0.5
1
1.5
2
Depth (ft)
0
20
40
60
80
100
120
140
160
0.5 in / month (avg)
0.7 in / month (max)
Initial Reading - Oct 10
max. quoted equipment error
+/- 0.03" in 10 feet
Oct 15
Oct 22
Oct 29
Nov 6
Nov 12
Dec 3
Dec 30

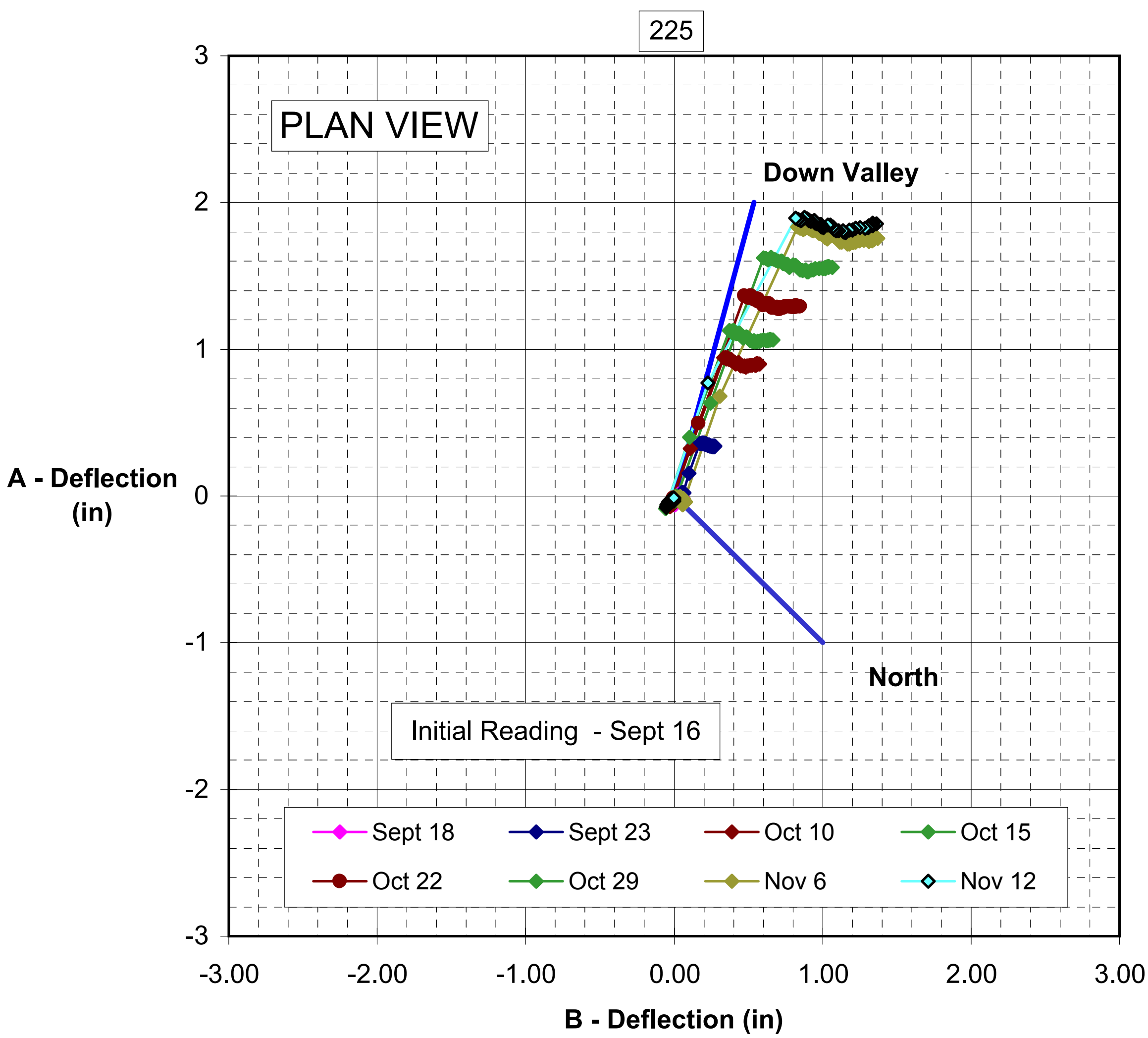

SI-19
195
PLAN VIEW
Initial Reading - Oct 10
Down Valley
North
A - Deflection (in)
B - Deflection (in)
2
1
0
-1
-2
-2
-1
0
1
2
Oct 15
Oct 22
Oct 29
Nov 6
Nov 12
Dec 3
Dec 30

SI-20


Deflection from Initial (in)
0
0.5
1
1.5
2
2.5
3
3.5
4
Depth (ft)
0
20
40
60
80
100
120
140
160
1.8 in / month (max)
1.1 in / month (avg)
Initial Reading - Sept 16
max. quoted equipment error
+/- 0.03" in 10 feet
Sept 18
Sept 23
Oct 10
Oct 15
Oct 22
Oct 29
Nov 6
Nov 12

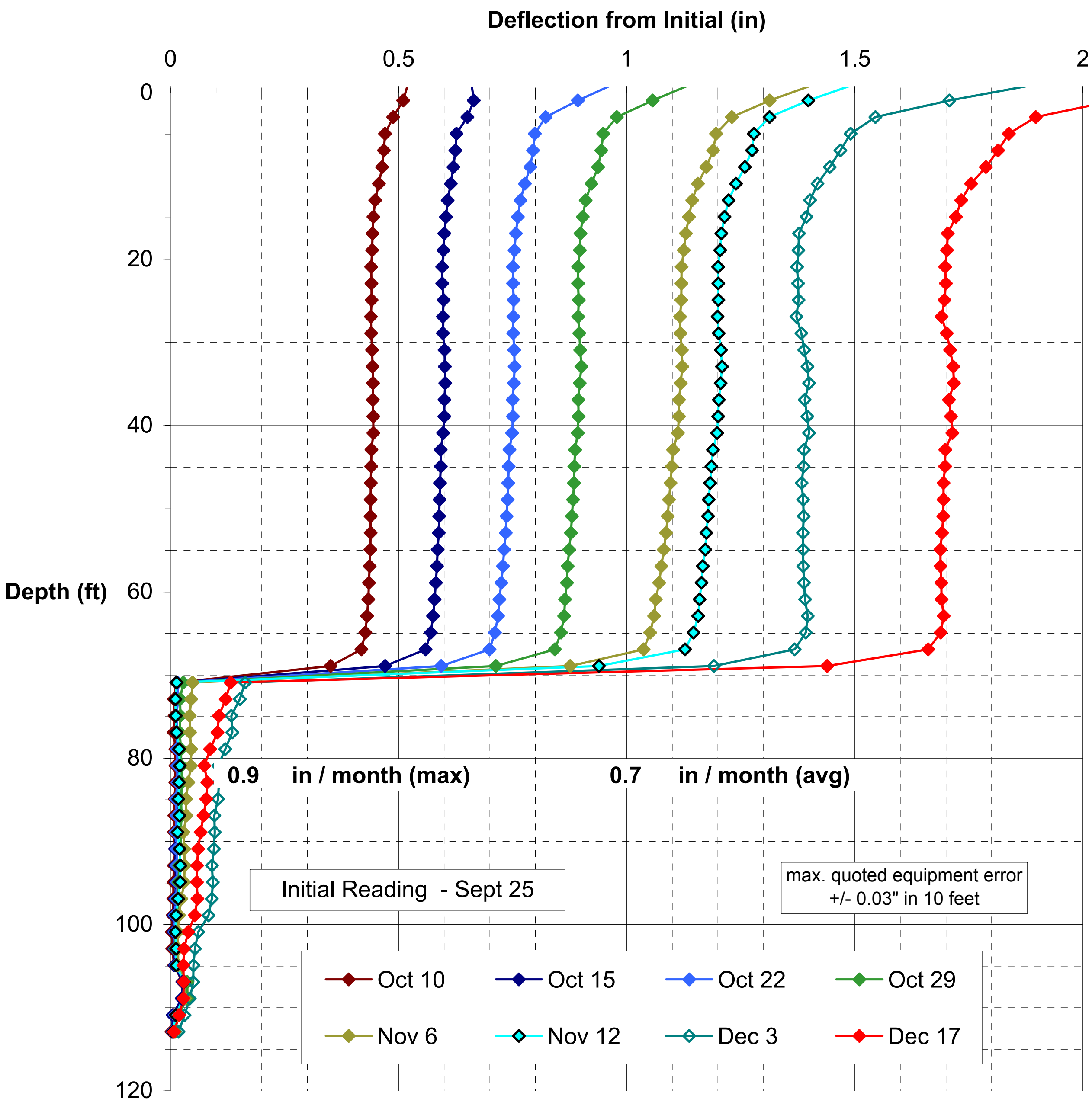
000365

SI-20



225
PLAN VIEW
Down Valley
North
Initial Reading - Sept 16
A - Deflection (in)
B - Deflection (in)
3
2
1
0
-1
-2
-3
-3.00
-2.00
-1.00
0.00
1.00
2.00
3.00
Sept 18
Sept 23
Oct 10
Oct 15
Oct 22
Oct 29
Nov 6
Nov 12

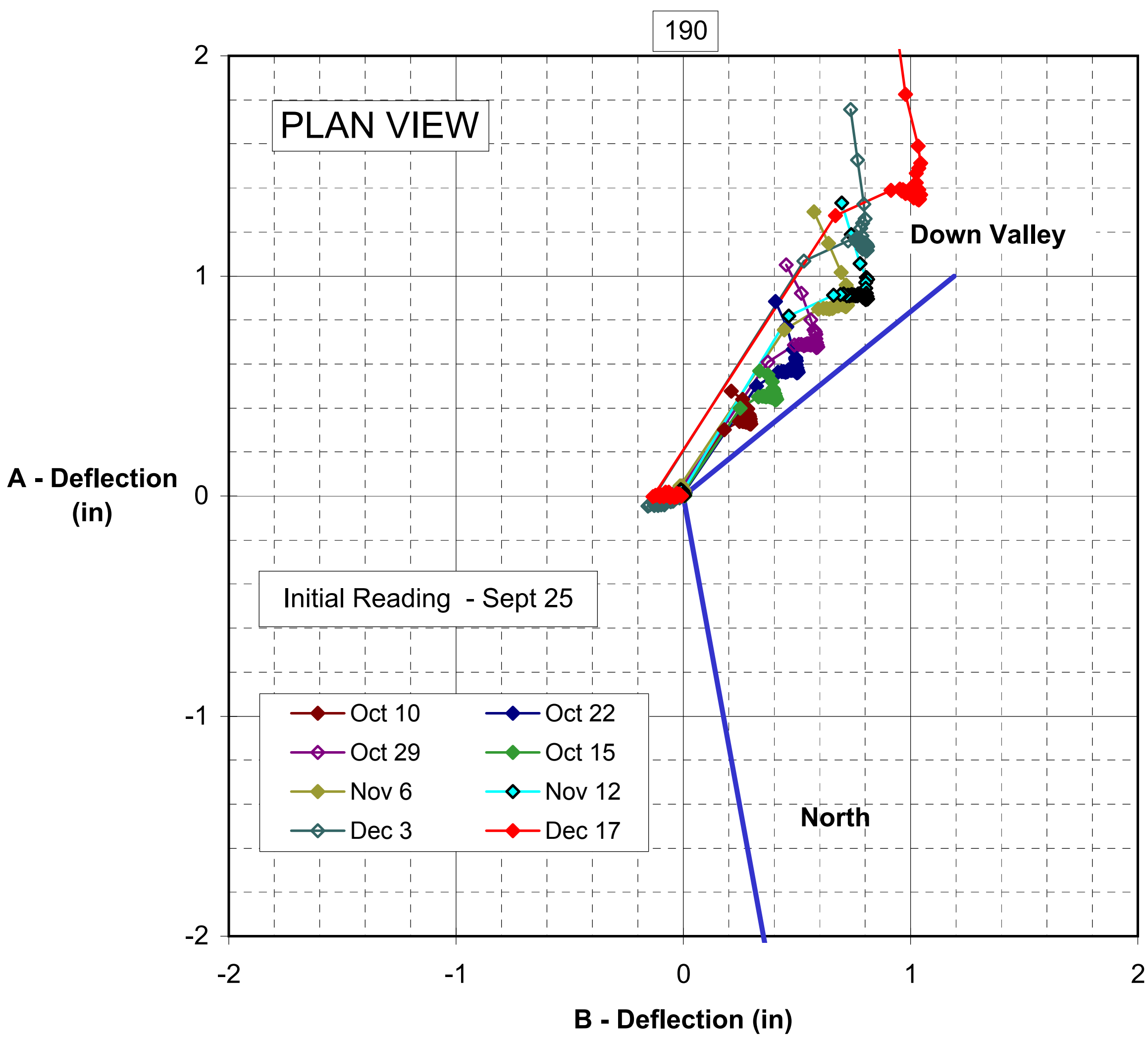
SI- 21



Deflection from Initial (in)
0
0.5
1
1.5
2
Depth (ft)
0
20
40
60
80
100
120
0.9 in / month (max)
0.7 in / month (avg)
Initial Reading - Sept 25
max. quoted equipment error
+/- 0.03" in 10 feet
Oct 10
Oct 15
Oct 22
Oct 29
Nov 6
Nov 12
Dec 3
Dec 17

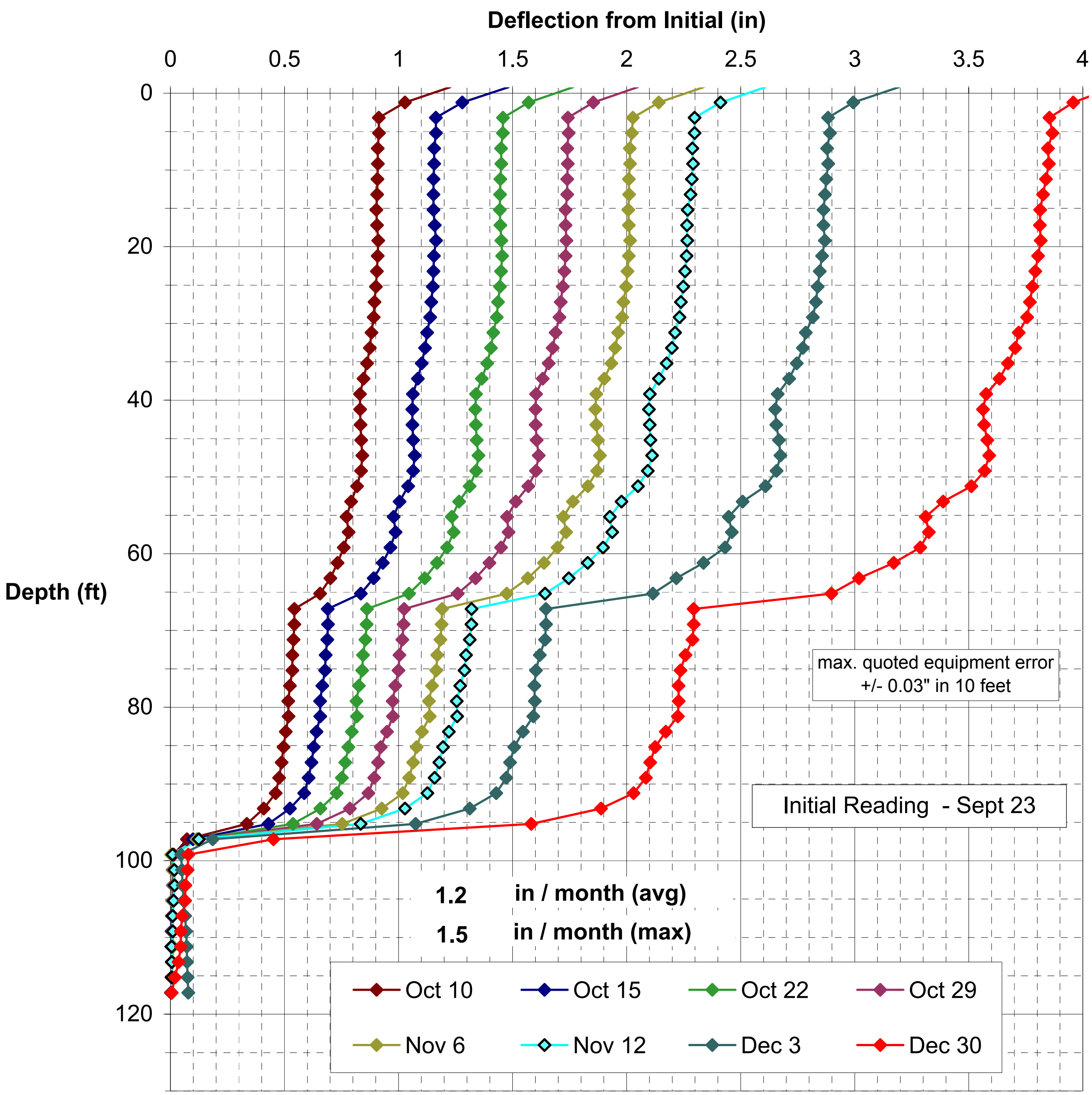
000367

SI- 21

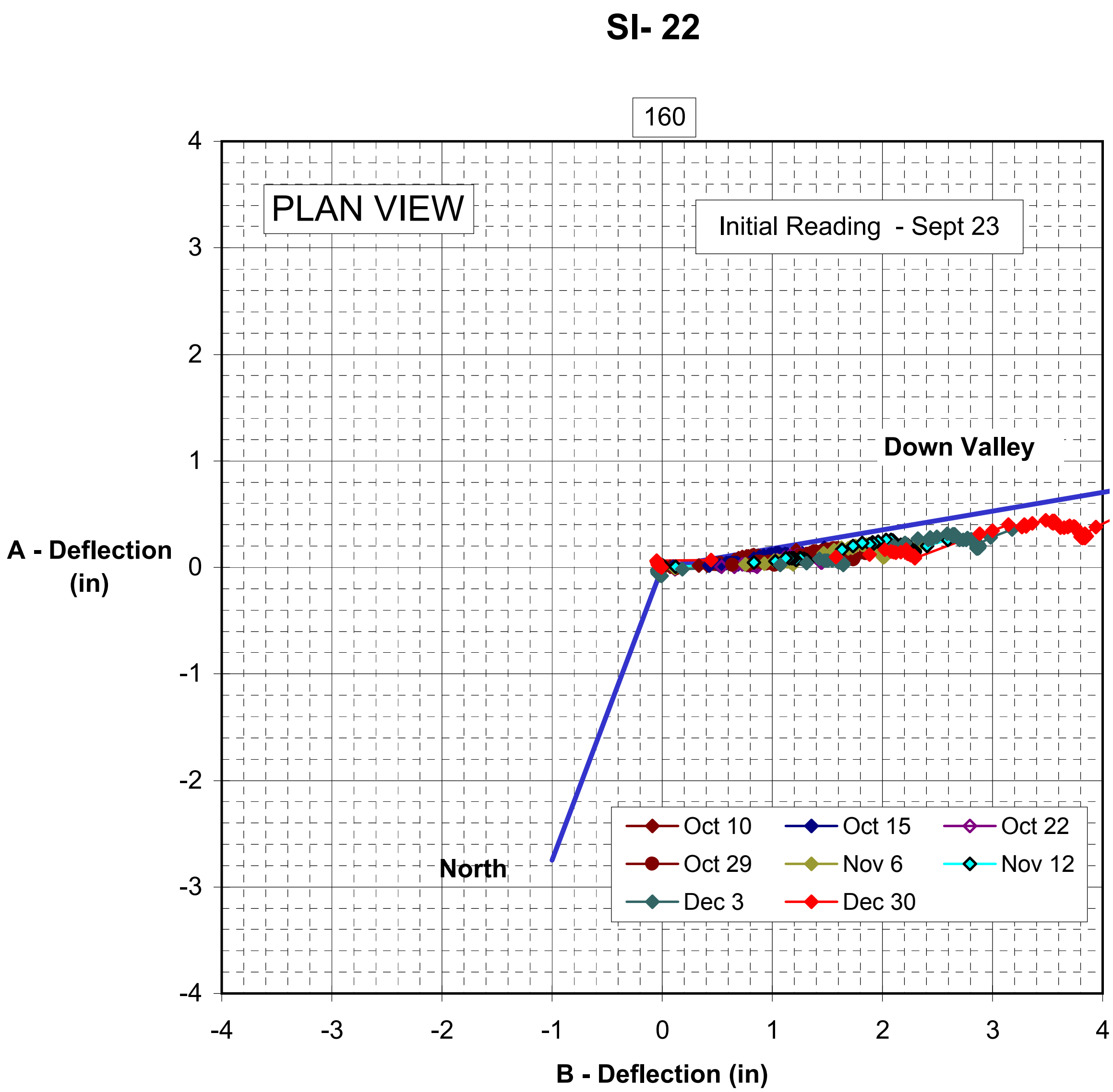


190
PLAN VIEW
Down Valley
North
A - Deflection (in)
B - Deflection (in)
2
1
0
-1
-2
-2
-1
0
1
2
Initial Reading - Sept 25
Oct 10
Oct 22
Oct 29
Oct 15
Nov 6
Nov 12
Dec 3
Dec 17

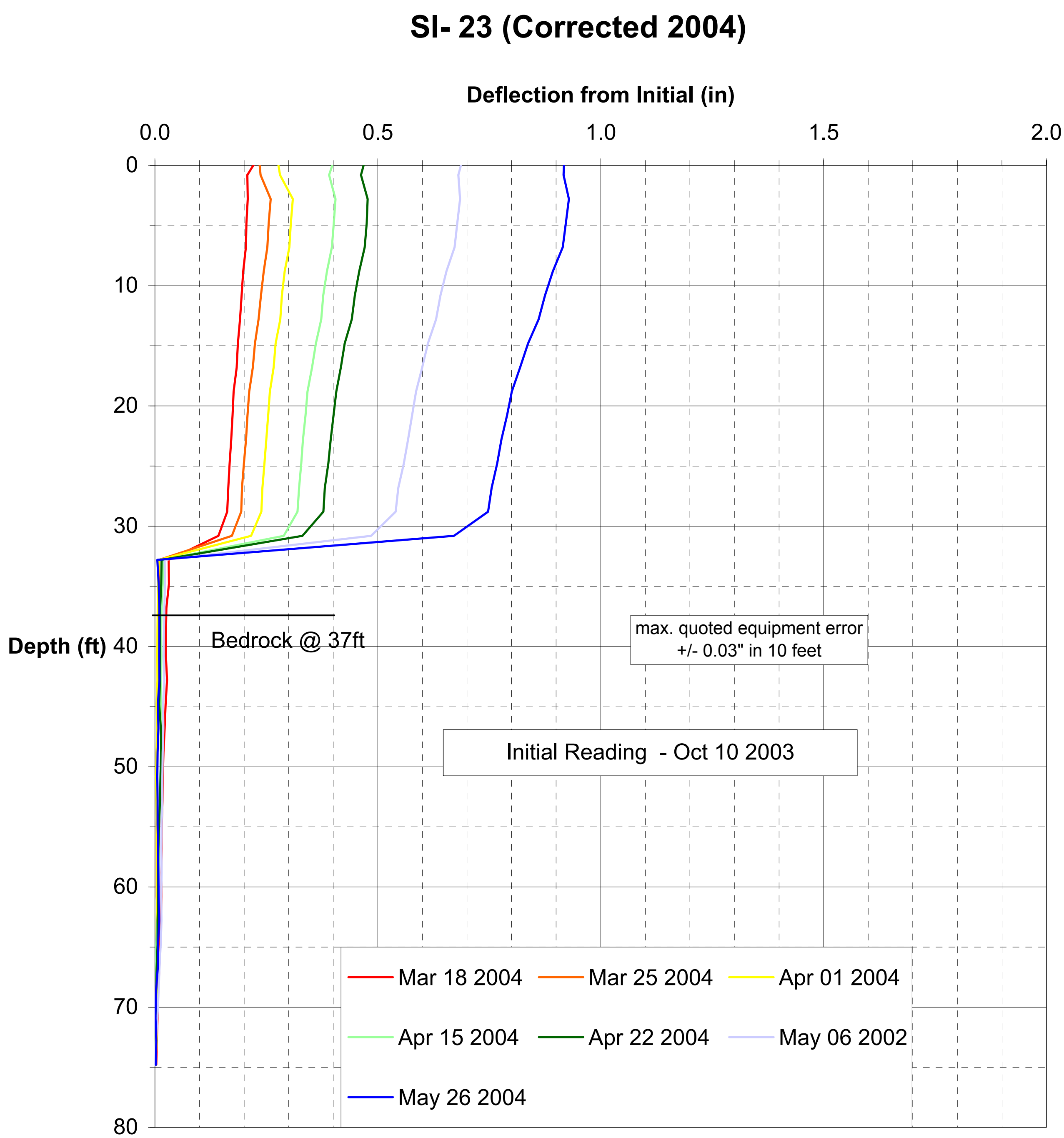
SI- 22



Deflection from Initial (in)
0
0.5
1
1.5
2
2.5
3
3.5
4
Depth (ft)
0
20
40
60
80
100
120
max. quoted equipment error
+/- 0.03" in 10 feet
Initial Reading - Sept 23
1.2 in / month (avg)
1.5 in / month (max)
Oct 10
Oct 15
Oct 22
Oct 29
Nov 6
Nov 12
Dec 3
Dec 30

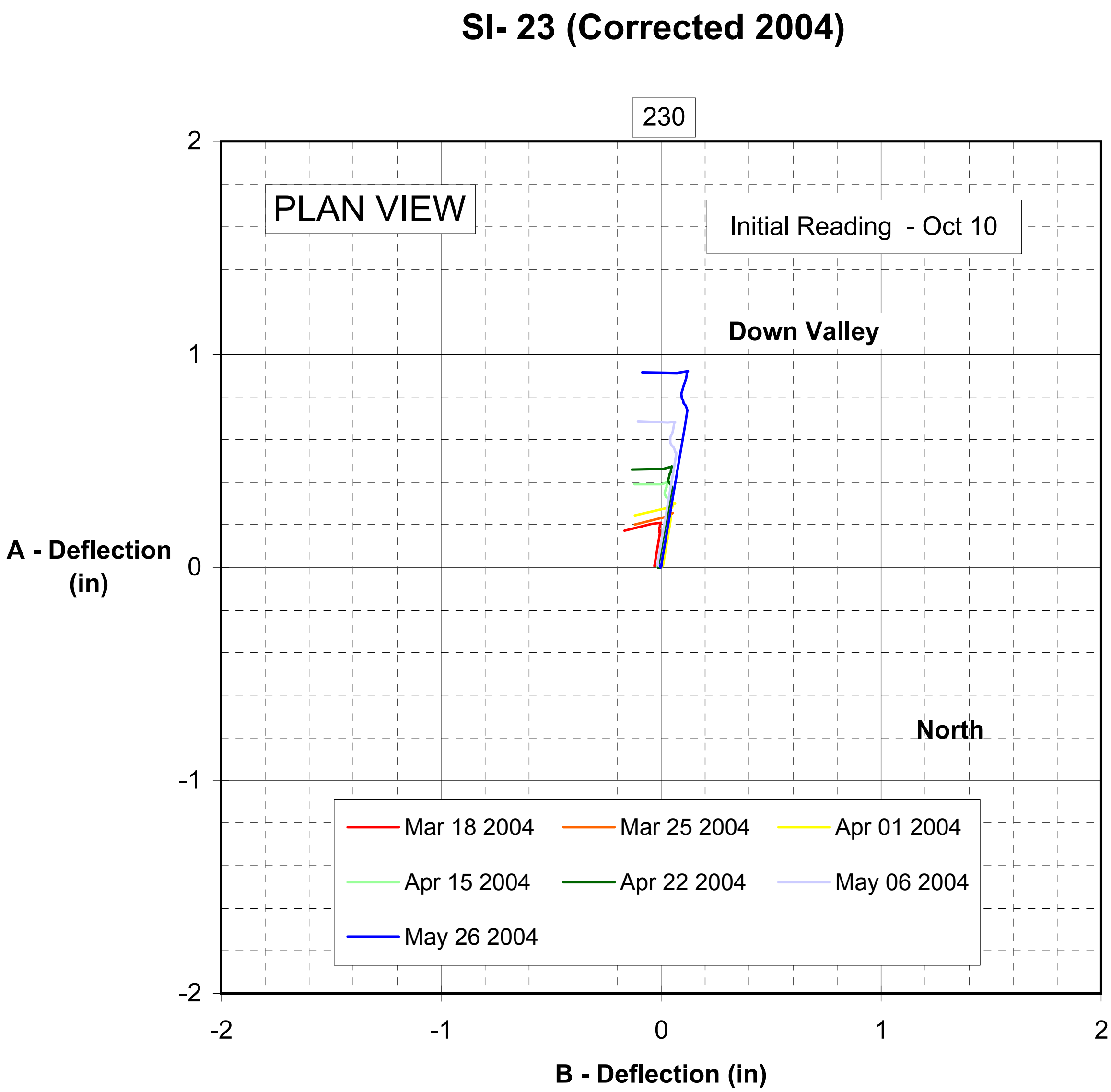


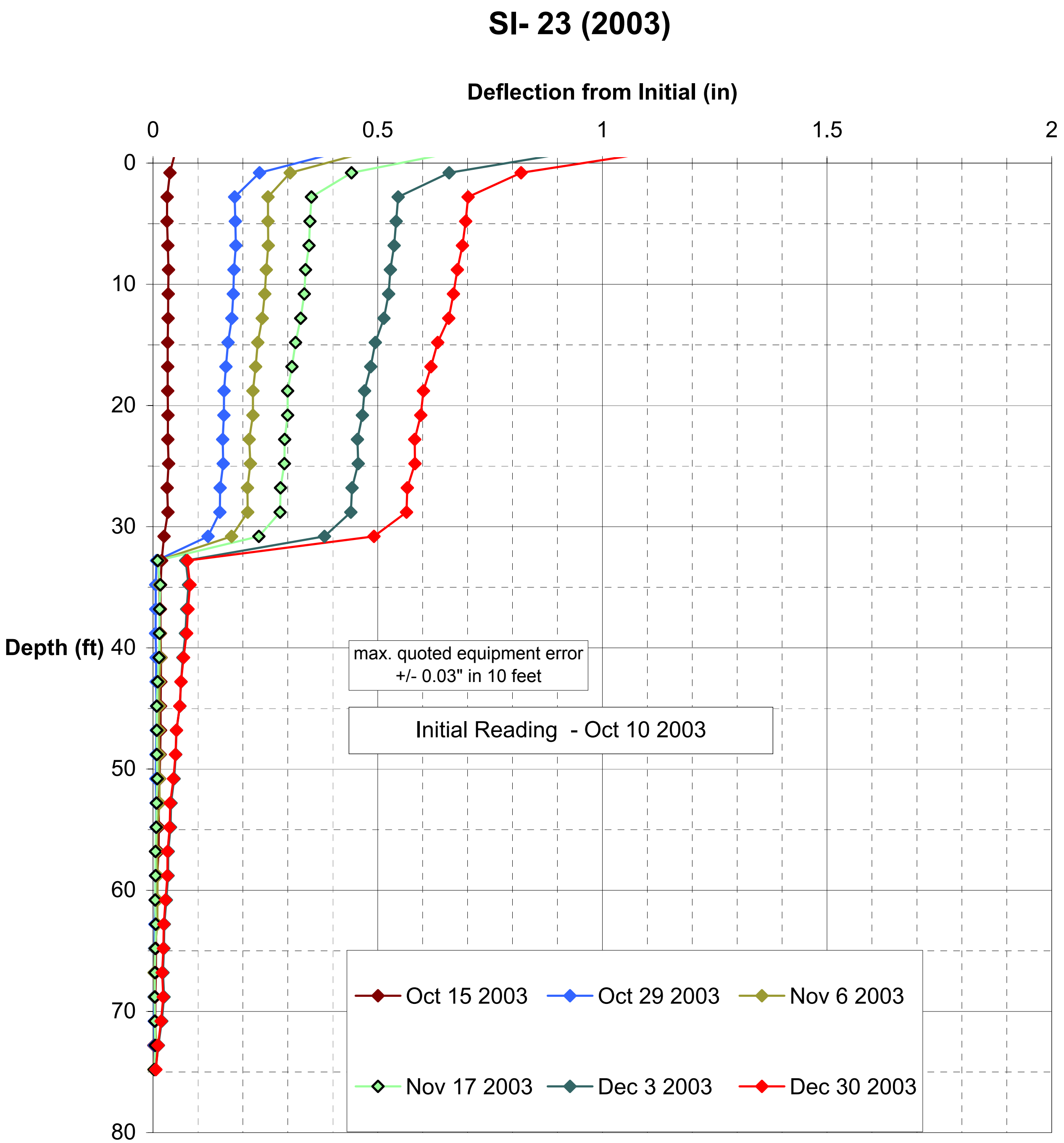
SI- 22
160
PLAN VIEW
Initial Reading - Sept 23
Down Valley
North
A - Deflection (in)
B - Deflection (in)
4
3
2
1
0
-1
-2
-3
-4
-4
-3
-2
-1
0
1
2
3
4
Oct 10
Oct 15
Oct 22
Oct 29
Nov 6
Nov 12
Dec 3
Dec 30

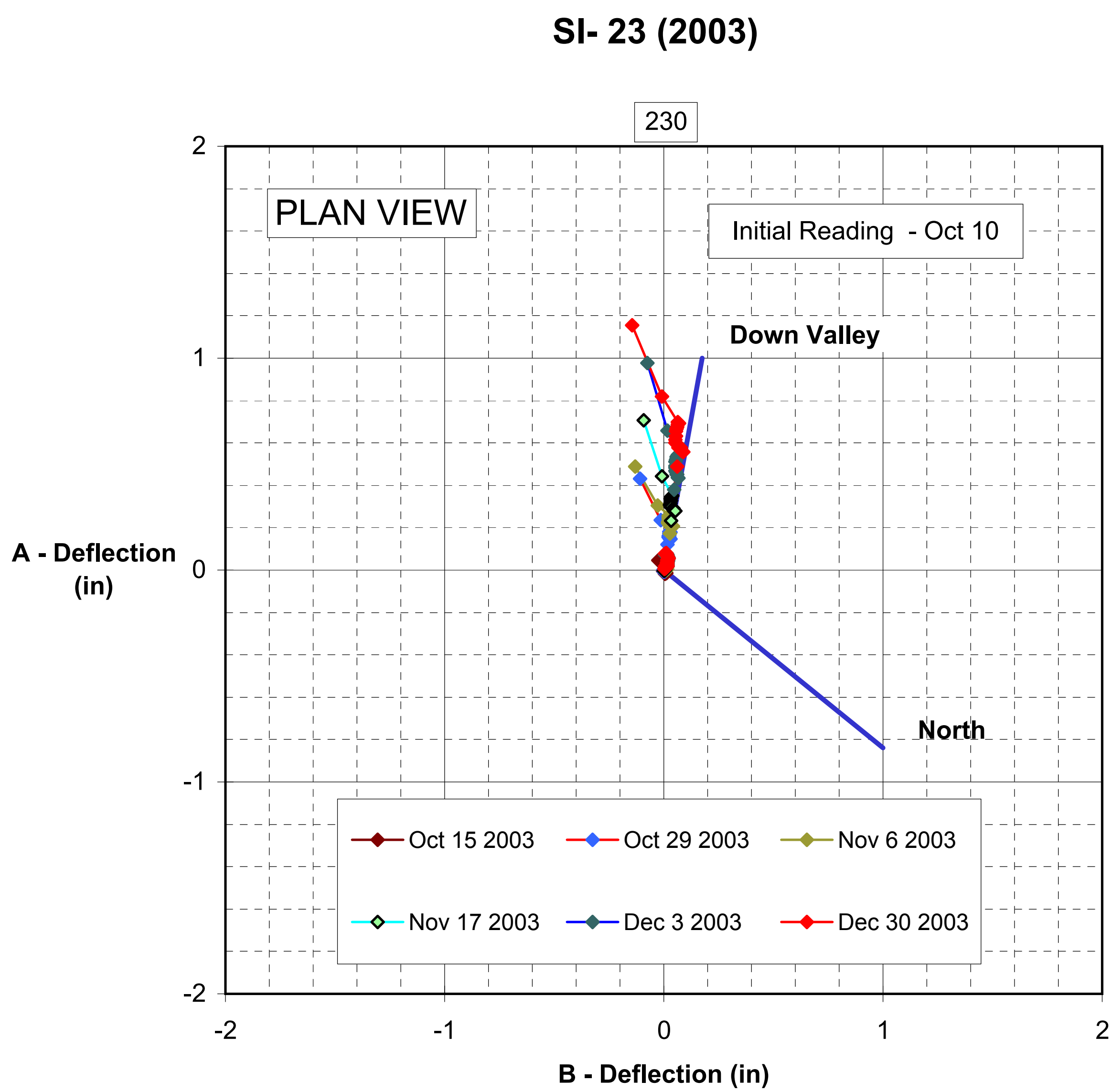


SI- 23 (Corrected 2004)
Deflection from Initial (in)
0.0
0.5
1.0
1.5
2.0
Depth (ft)
0
10
20
30
40
50
60
70
80
Bedrock @ 37ft
max. quoted equipment error
+/- 0.03" in 10 feet
Initial Reading - Oct 10 2003
Mar 18 2004
Mar 25 2004
Apr 01 2004
Apr 15 2004
Apr 22 2004
May 06 2002
May 26 2004

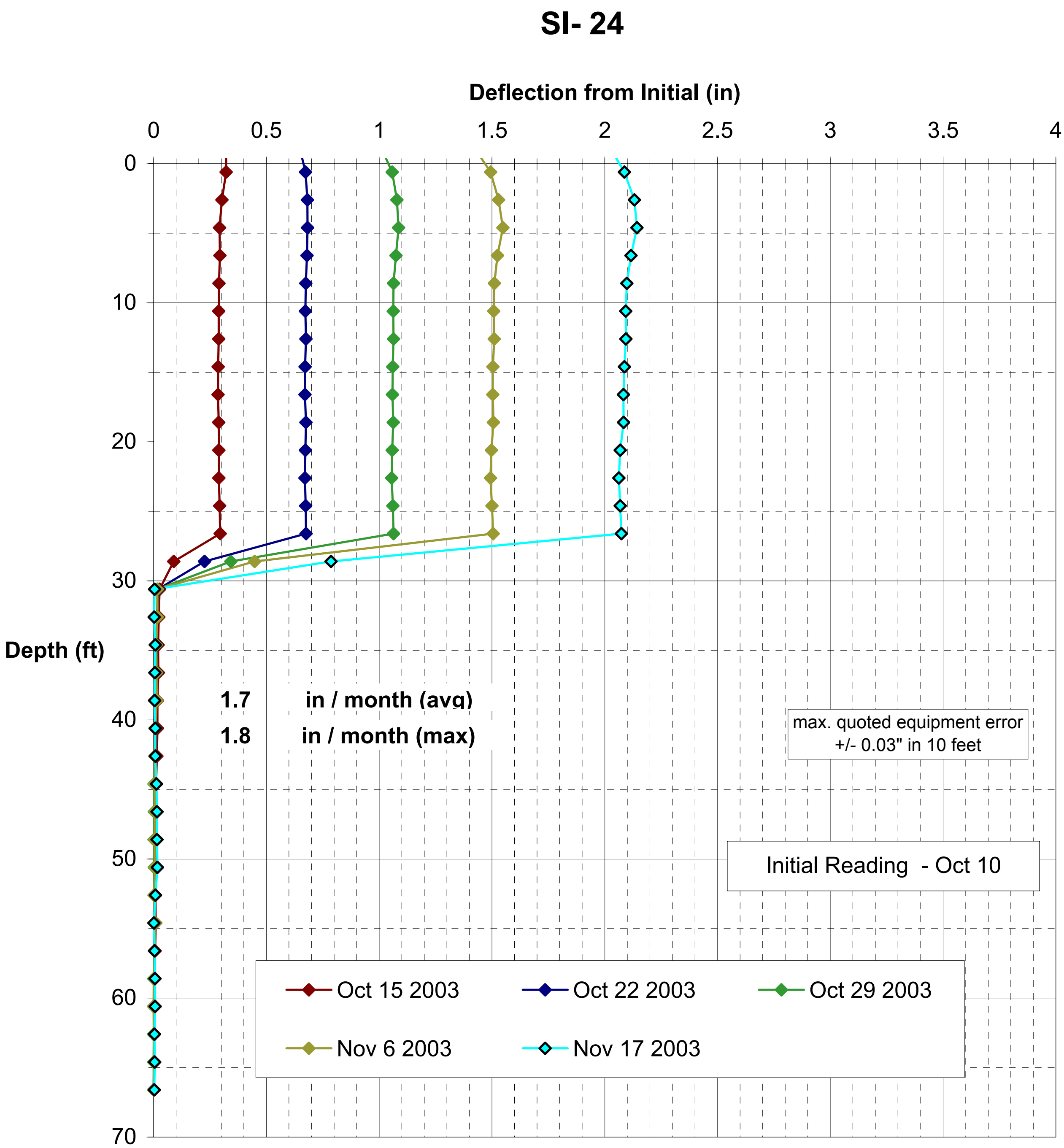
000371


SI- 23 (Corrected 2004)
230
PLAN VIEW
Initial Reading - Oct 10
Down Valley
North
A - Deflection (in)
B - Deflection (in)
2
1
0
-1
-2
-2
-1
0
1
2
Mar 18 2004
Mar 25 2004
Apr 01 2004
Apr 15 2004
Apr 22 2004
May 06 2004
May 26 2004

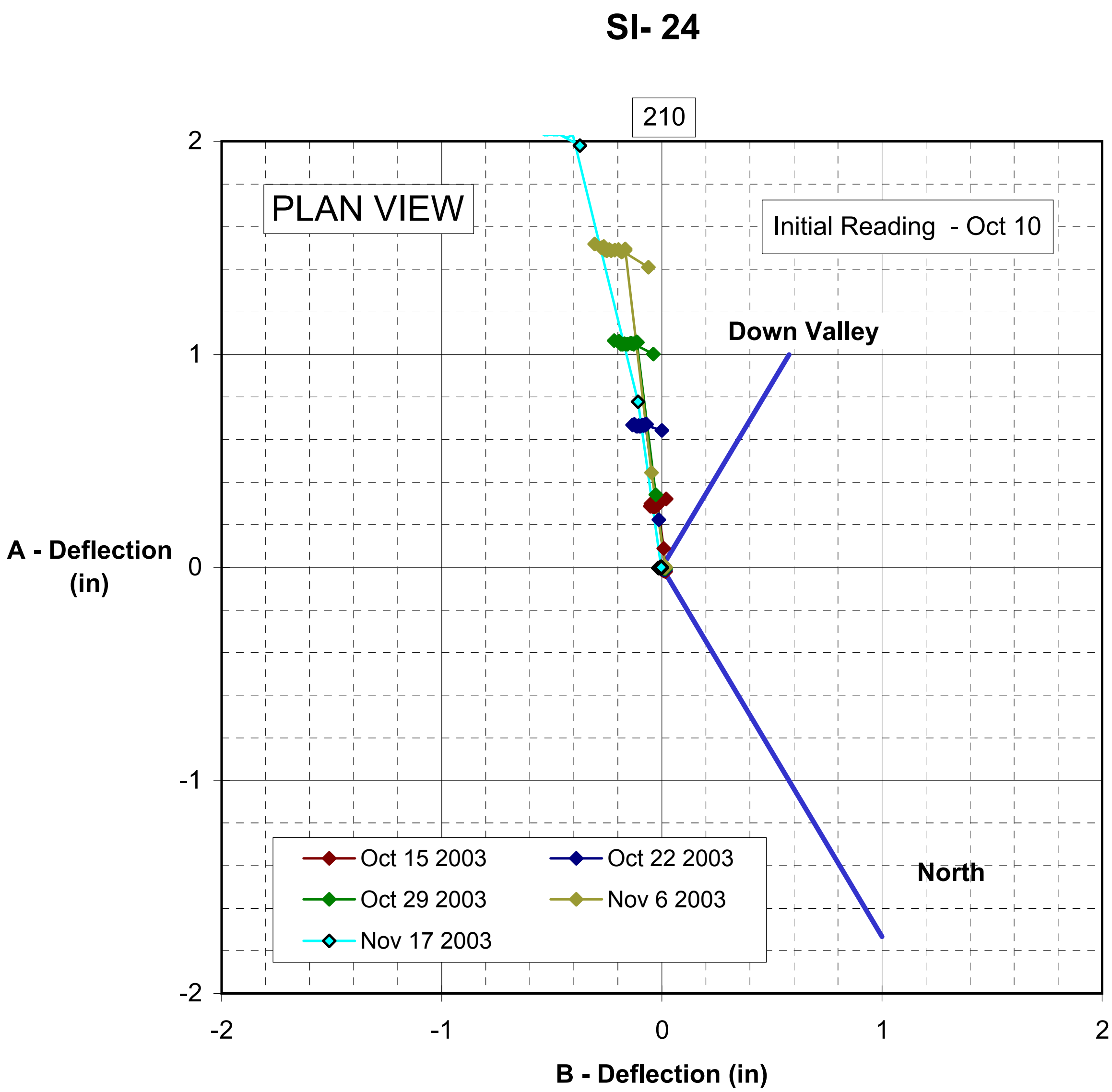

SI- 23 (2003)
Deflection from Initial (in)
0
0.5
1
1.5
2
Depth (ft)
0
10
20
30
40
50
60
70
80
max. quoted equipment error
+/- 0.03" in 10 feet
Initial Reading - Oct 10 2003
Oct 15 2003
Oct 29 2003
Nov 6 2003
Nov 17 2003
Dec 3 2003
Dec 30 2003



SI- 23 (2003)
230
PLAN VIEW
Initial Reading - Oct 10
Down Valley
North
A - Deflection (in)
B - Deflection (in)
2
1
0
-1
-2
Oct 15 2003
Oct 29 2003
Nov 6 2003
Nov 17 2003
Dec 3 2003
Dec 30 2003



SI- 24
Deflection from Initial (in)
0
0.5
1
1.5
2
2.5
3
3.5
4
0
10
20
30
40
50
60
70
Depth (ft)
1.7 in / month (avg)
1.8 in / month (max)
max. quoted equipment error +/- 0.03" in 10 feet
Initial Reading - Oct 10
Oct 15 2003
Oct 22 2003
Oct 29 2003
Nov 6 2003
Nov 17 2003


SI- 24
210
PLAN VIEW
Initial Reading - Oct 10
Down Valley
North
A - Deflection (in)
B - Deflection (in)
2
1
0
-1
-2
-2
-1
0
1
2
Oct 15 2003
Oct 22 2003
Oct 29 2003
Nov 6 2003
Nov 17 2003

Appendix G

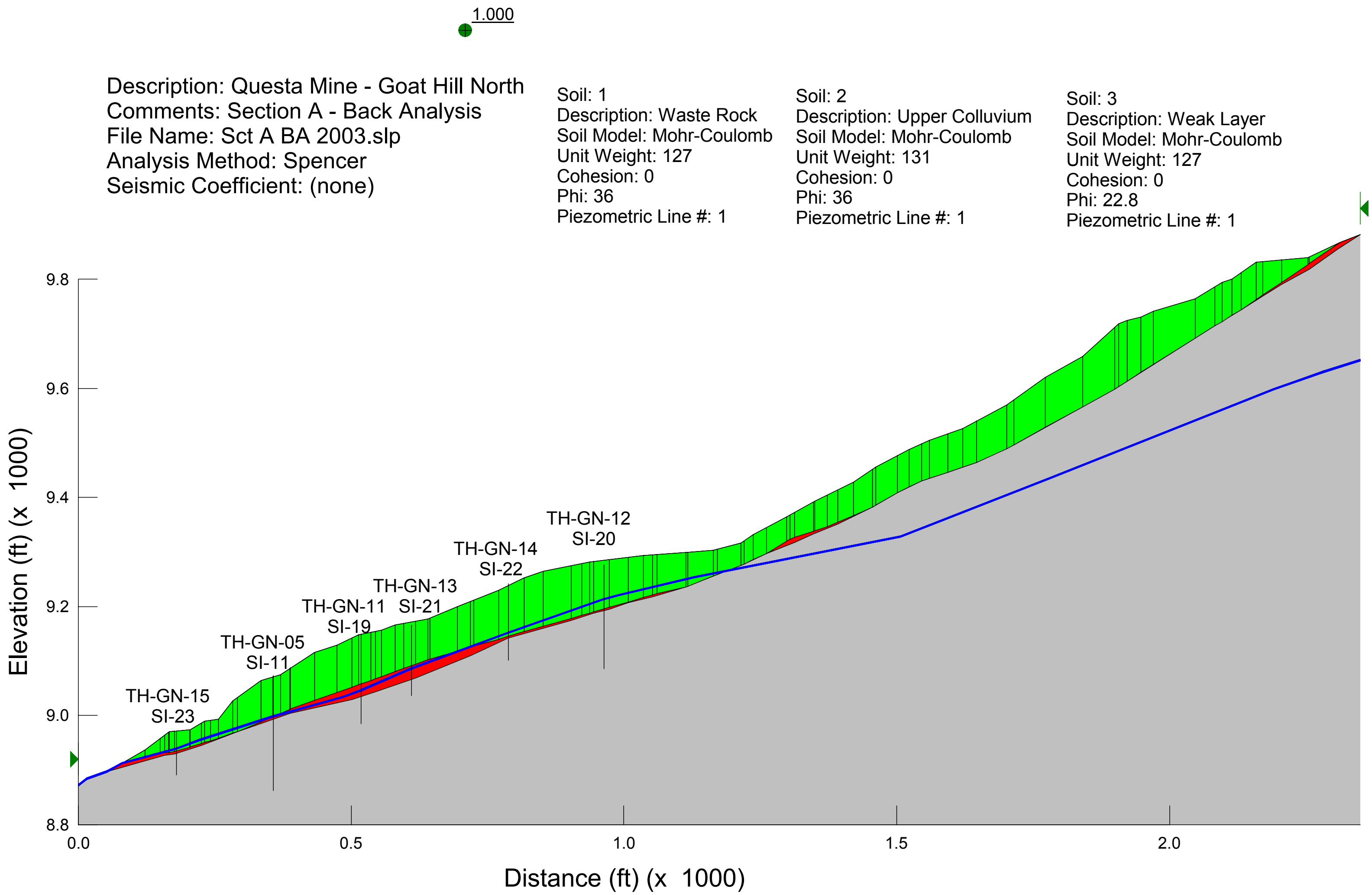
2D Stability Analysis Results

2D STABILITY SUMMARY

	Back Analyzed Friction Angle	Specified Result FOS	Search Result FOS
Back Analysis			
Section A	22.8		
Section B	23.7		
Section C	25.0		
Section D		1.36	1.23
Section E		1.05	1.04
Section F		1.59	1.18
Section G		1.07	1.06
Construction Phase 2			
Phase 2A			
Section D		1.44	1.37
Section F		1.61	1.20
Section G		1.18	1.20
Phase 2B			
Section D		1.71	1.62
Section F		1.60	1.29
Section G		1.29	1.18
Phase 2C			
Section D		1.64	1.60
Section F		1.59	1.53
Section G		1.27	1.12
Phase 2 Completed			
Section A		1.08	0.91
Section B		1.14	1.07
Section C		1.12	1.05
Section D		1.61	1.56
Section E		1.06	1.05
Section F		1.61	1.53
Section G		1.28	1.36
Construction Phase 3			
Phase 3A			
Section B		1.12	1.03
Section E		1.04	1.03
Phase 3B			
Section B		1.24	1.14
Section E		1.16	1.07
Phase 3C			
Section B		1.25	1.23
Section E		1.14	1.09
Final Reslope			
Section A		1.17	1.14
Section B		1.36	1.24
Section C		1.42	1.31
Section D		1.95	1.76
Section E		1.14	1.12
Section F		1.61	1.55
Section G		1.31	1.35
Alternate Water Table			
Back Analysis			
Section A	23.7		
Section B	24.5		
Section C	25.9		
Section E		1.07	1.03
Final Reslope			
Section A		1.18	1.09
Section B		1.40	1.25
Section C		1.46	1.29
Section E		1.17	1.14
Tension Cracks			
Final Reslope			
Section E - Phase 3b			1.19
Section G - Phase 2b			1.56
Section G - Phase 2c			1.43

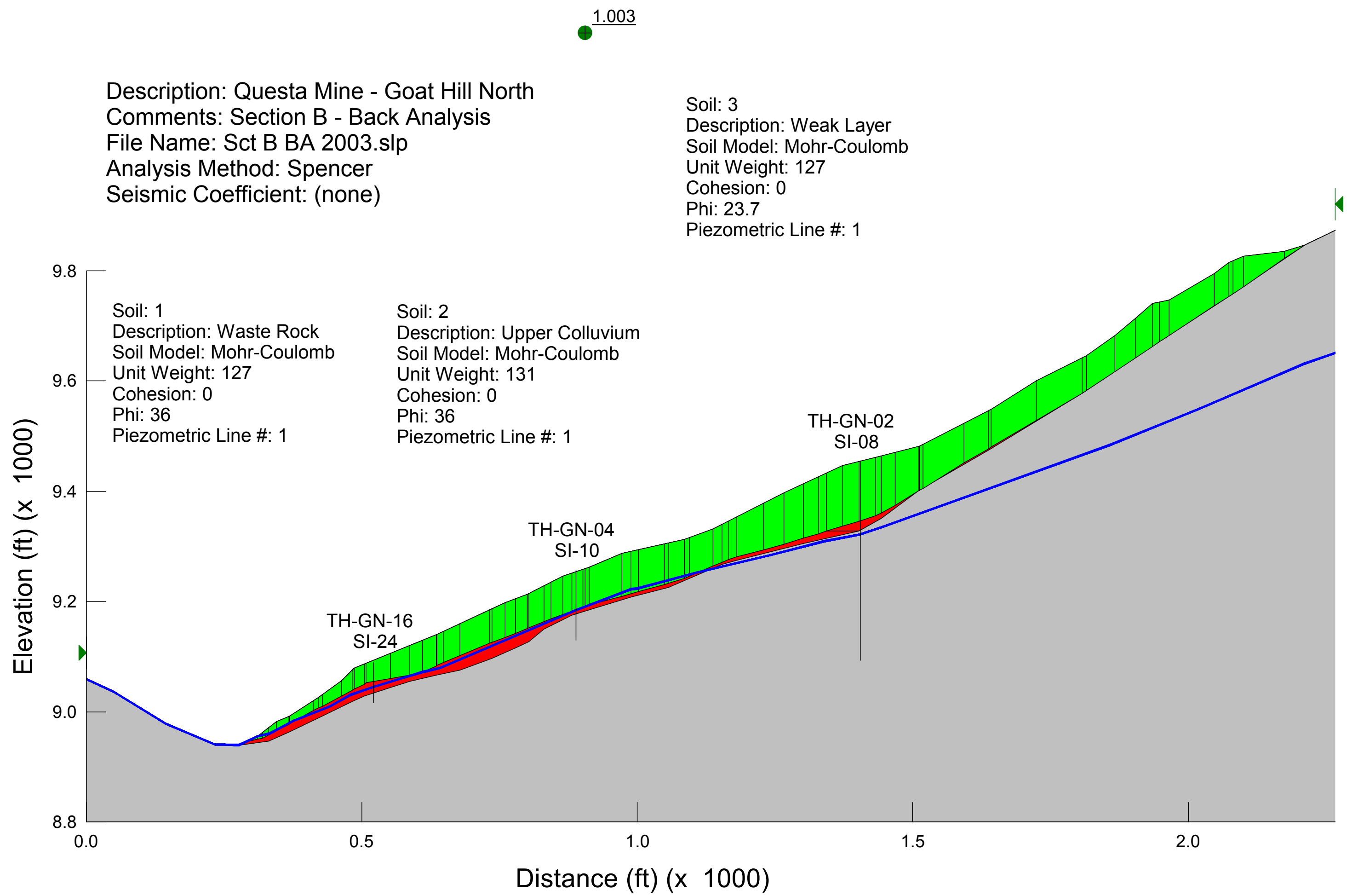
2D Stability Results

Back Analysis

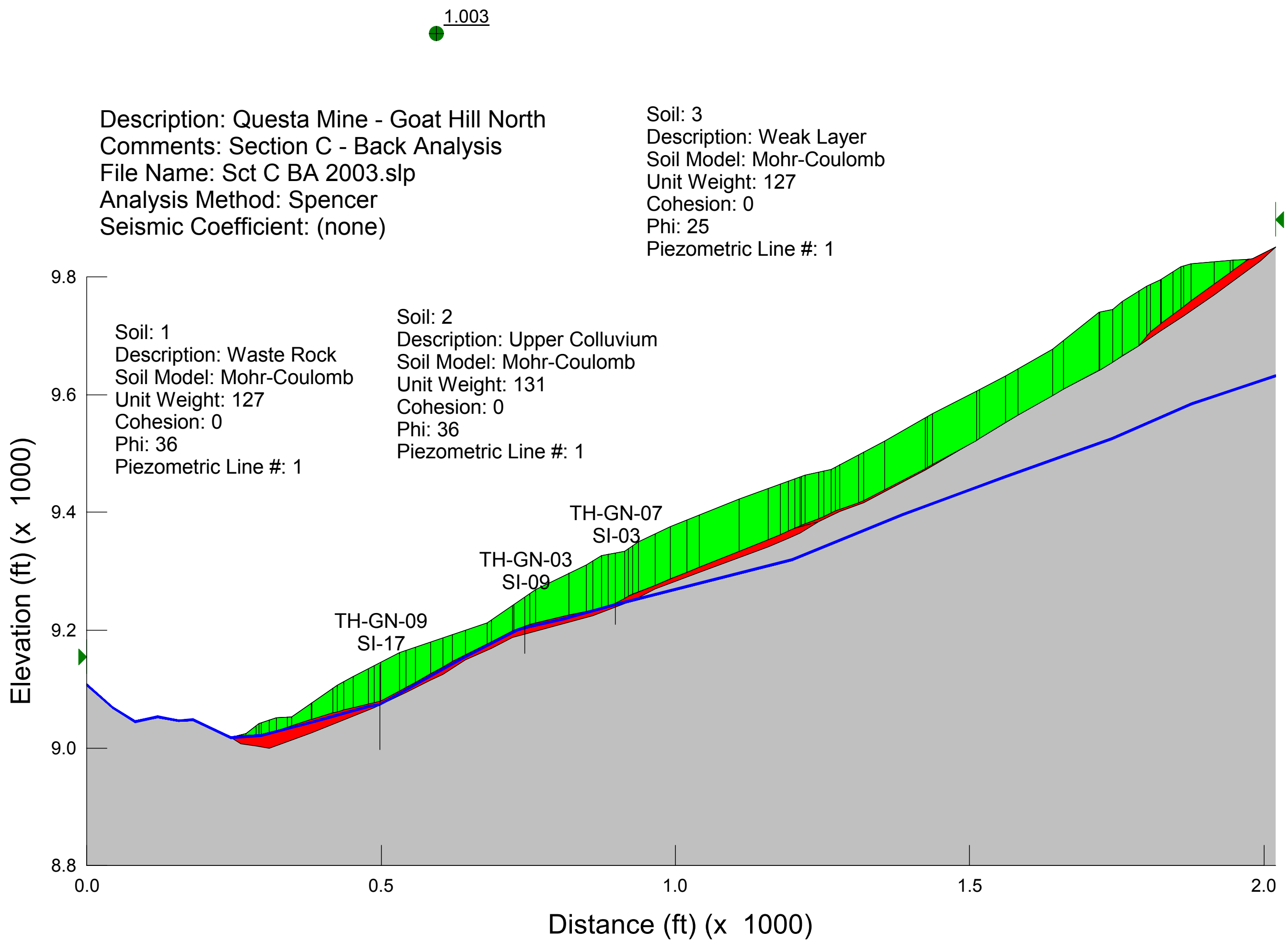


1.000
Description: Questa Mine - Goat Hill North
Comments: Section A - Back Analysis
File Name: Sct A BA 2003.slp
Analysis Method: Spencer
Seismic Coefficient: (none)
Soil: 1
Description: Waste Rock
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 2
Description: Upper Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 3
Description: Weak Layer
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 22.8
Piezometric Line #: 1
TH-GN-15
SI-23
TH-GN-05
SI-11
TH-GN-11
SI-19
TH-GN-13
SI-21
TH-GN-14
SI-22
TH-GN-12
SI-20
Elevation (ft) (x 1000)
Distance (ft) (x 1000)
9.8
9.6
9.4
9.2
9.0
8.8
0.0
0.5
1.0
1.5
2.0

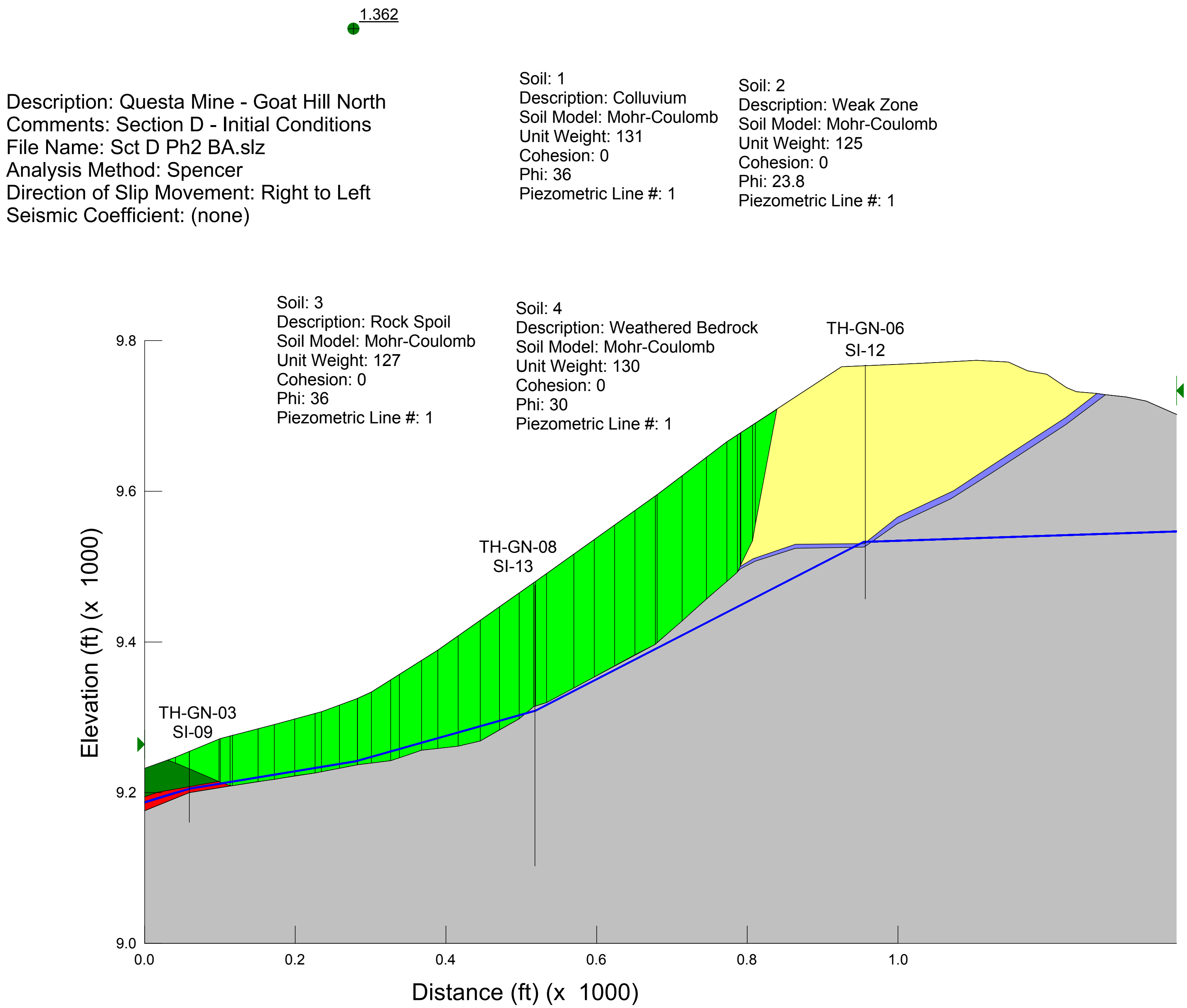
000380


1.003
Description: Questa Mine - Goat Hill North
Comments: Section B - Back Analysis
File Name: Sct B BA 2003.slp
Analysis Method: Spencer
Seismic Coefficient: (none)
Soil: 3
Description: Weak Layer
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 23.7
Piezometric Line #: 1
Soil: 1
Description: Waste Rock
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 2
Description: Upper Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
TH-GN-02
SI-08
TH-GN-04
SI-10
TH-GN-16
SI-24
Elevation (ft) (x 1000)
9.8
9.6
9.4
9.2
9.0
8.8
0.0
0.5
1.0
1.5
2.0
Distance (ft) (x 1000)

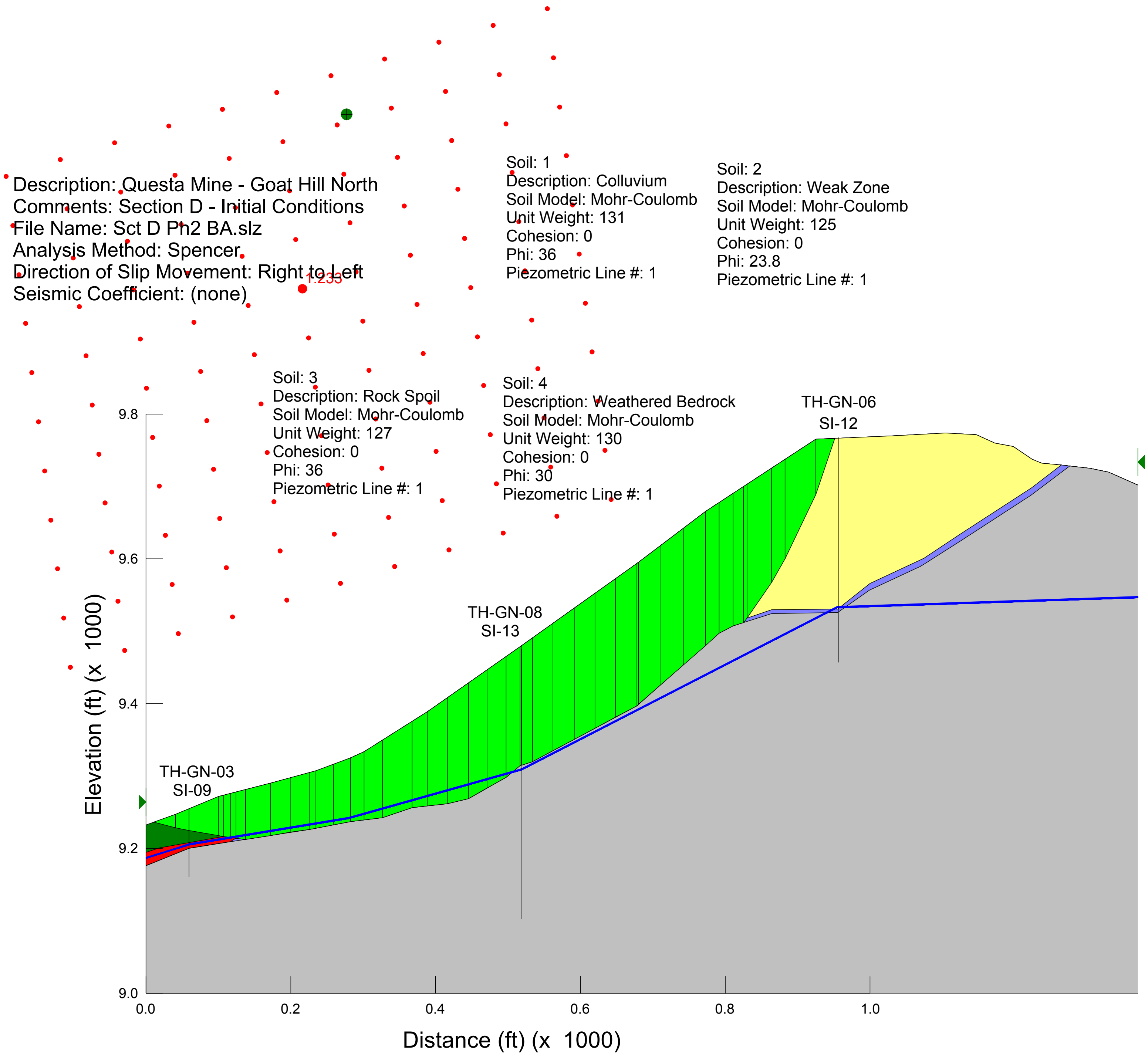
000381


1.003
Description: Questa Mine - Goat Hill North
Comments: Section C - Back Analysis
File Name: Sct C BA 2003.slp
Analysis Method: Spencer
Seismic Coefficient: (none)
Soil: 3
Description: Weak Layer
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 25
Piezometric Line #: 1
Soil: 1
Description: Waste Rock
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 2
Description: Upper Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
TH-GN-07
SI-03
TH-GN-03
SI-09
TH-GN-09
SI-17
Elevation (ft) (x 1000)
9.8
9.6
9.4
9.2
9.0
8.8
0.0
0.5
1.0
1.5
2.0
Distance (ft) (x 1000)

000382

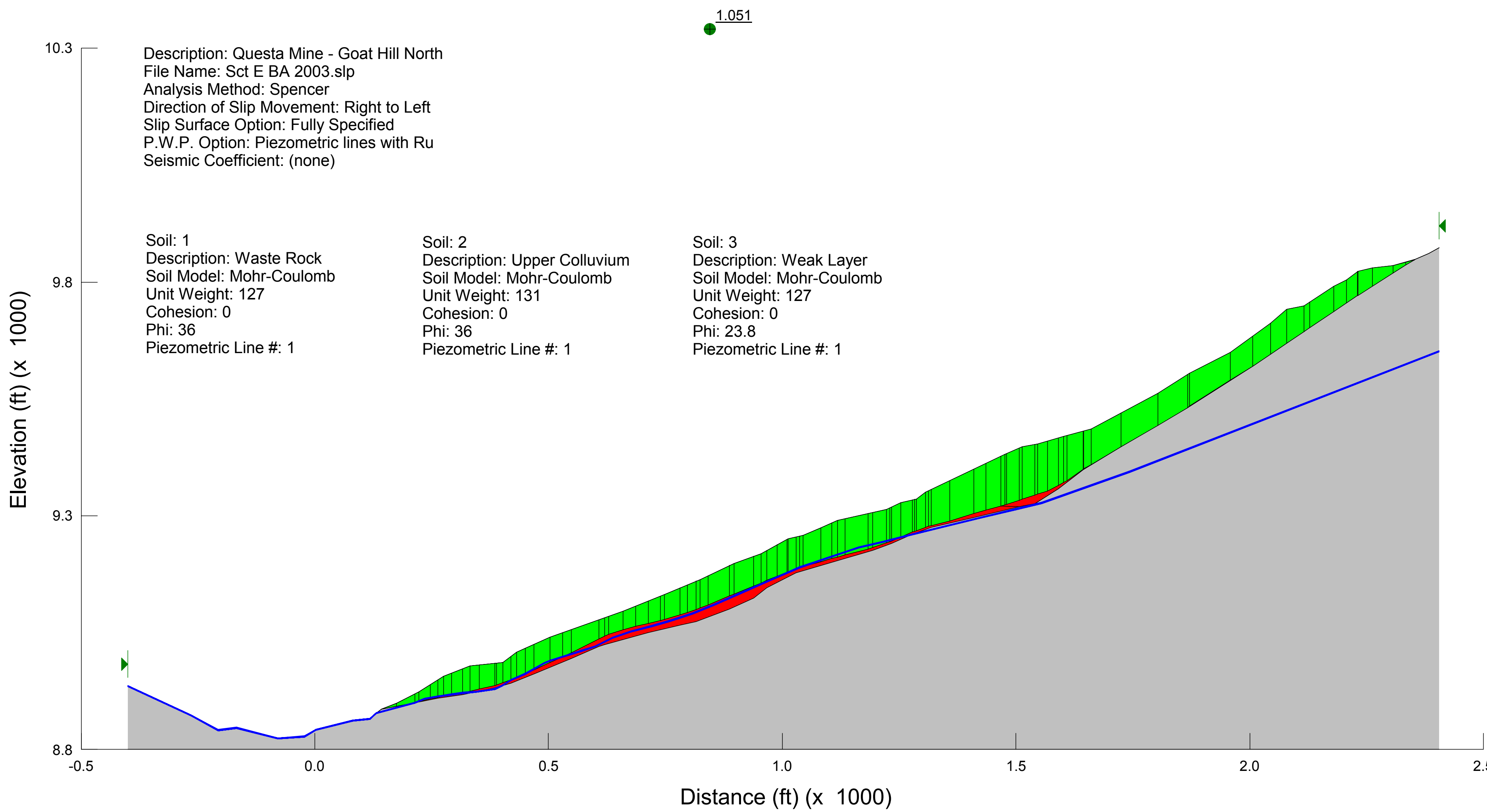

1.362
Description: Questa Mine - Goat Hill North
Comments: Section D - Initial Conditions
File Name: Sct D Ph2 BA.slz
Analysis Method: Spencer
Direction of Slip Movement: Right to Left
Seismic Coefficient: (none)
Soil: 1
Description: Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 2
Description: Weak Zone
Soil Model: Mohr-Coulomb
Unit Weight: 125
Cohesion: 0
Phi: 23.8
Piezometric Line #: 1
Soil: 3
Description: Rock Spoil
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 4
Description: Weathered Bedrock
Soil Model: Mohr-Coulomb
Unit Weight: 130
Cohesion: 0
Phi: 30
Piezometric Line #: 1
TH-GN-06
SI-12
TH-GN-08
SI-13
TH-GN-03
SI-09
9.8
9.6
9.4
9.2
9.0
0.0
0.2
0.4
0.6
0.8
1.0
Elevation (ft) (x 1000)
Distance (ft) (x 1000)

000383



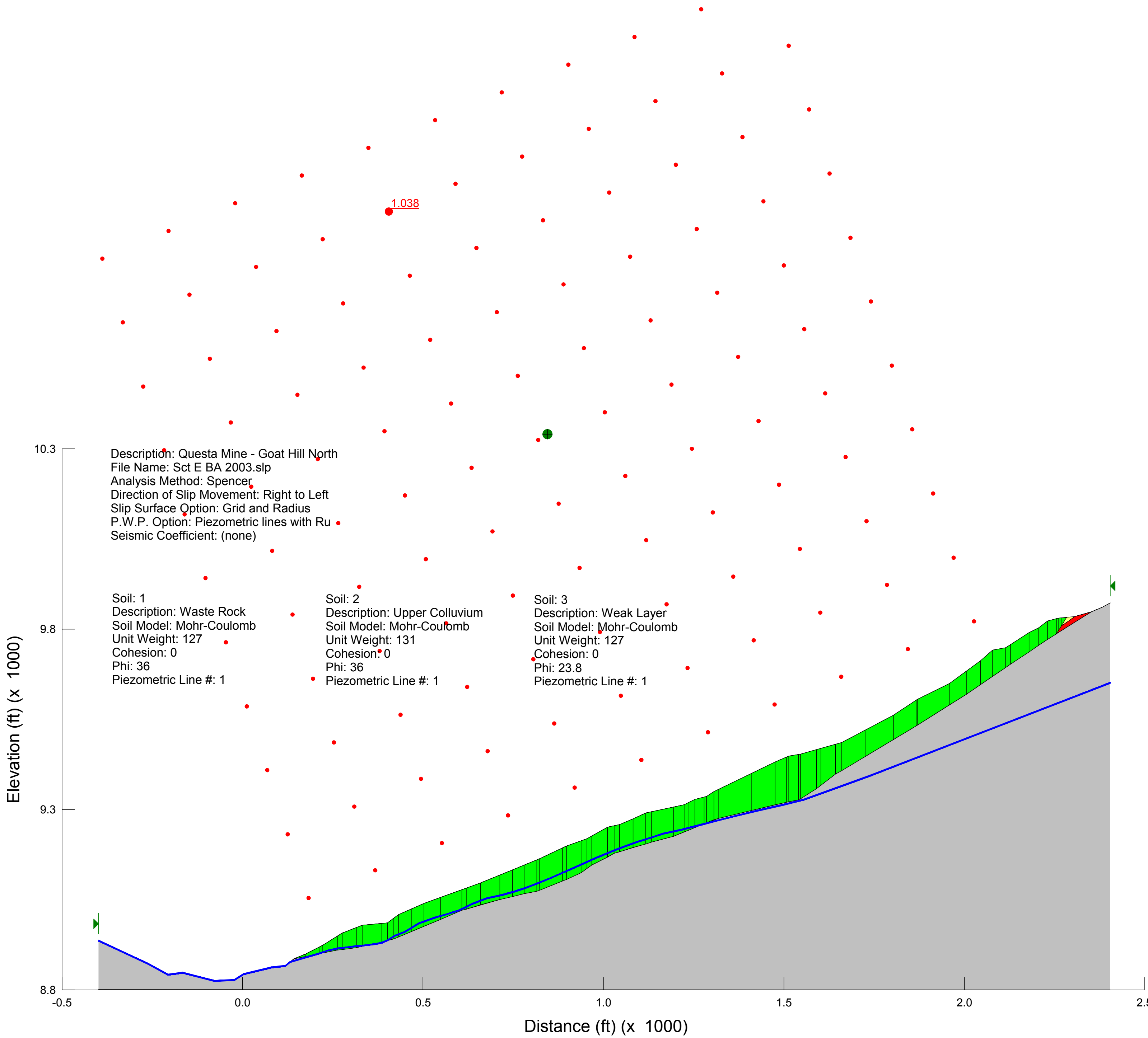
Description: Questa Mine - Goat Hill North
Comments: Section D - Initial Conditions
File Name: Sct D Ph2 BA.slz
Analysis Method: Spencer
Direction of Slip Movement: Right to Left
Seismic Coefficient: (none)
1.233
Soil: 1
Description: Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 2
Description: Weak Zone
Soil Model: Mohr-Coulomb
Unit Weight: 125
Cohesion: 0
Phi: 23.8
Piezometric Line #: 1
Soil: 3
Description: Rock Spoil
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 4
Description: Weathered Bedrock
Soil Model: Mohr-Coulomb
Unit Weight: 130
Cohesion: 0
Phi: 30
Piezometric Line #: 1
TH-GN-06
SI-12
TH-GN-08
SI-13
TH-GN-03
SI-09
Elevation (ft) (x 1000)
9.8
9.6
9.4
9.2
9.0
0.0
0.2
0.4
0.6
0.8
1.0
Distance (ft) (x 1000)

000384

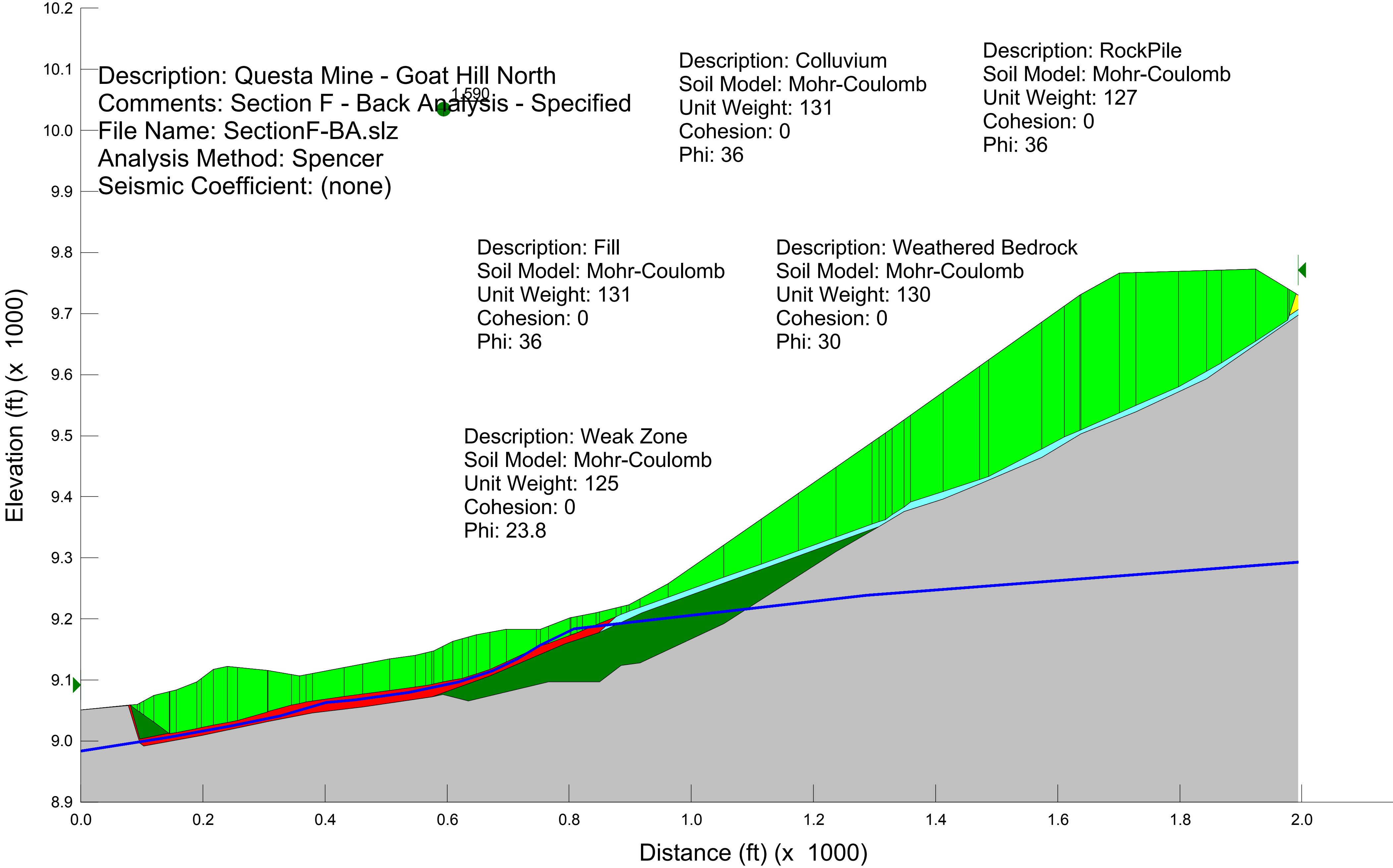


1.051
Description: Questa Mine - Goat Hill North
File Name: Sct E BA 2003.slp
Analysis Method: Spencer
Direction of Slip Movement: Right to Left
Slip Surface Option: Fully Specified
P.W.P. Option: Piezometric lines with Ru
Seismic Coefficient: (none)
Soil: 1
Description: Waste Rock
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 2
Description: Upper Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 3
Description: Weak Layer
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 23.8
Piezometric Line #: 1
10.3
9.8
9.3
8.8
Elevation (ft) (x 1000)
-0.5
0.0
0.5
1.0
1.5
2.0
2.5
Distance (ft) (x 1000)

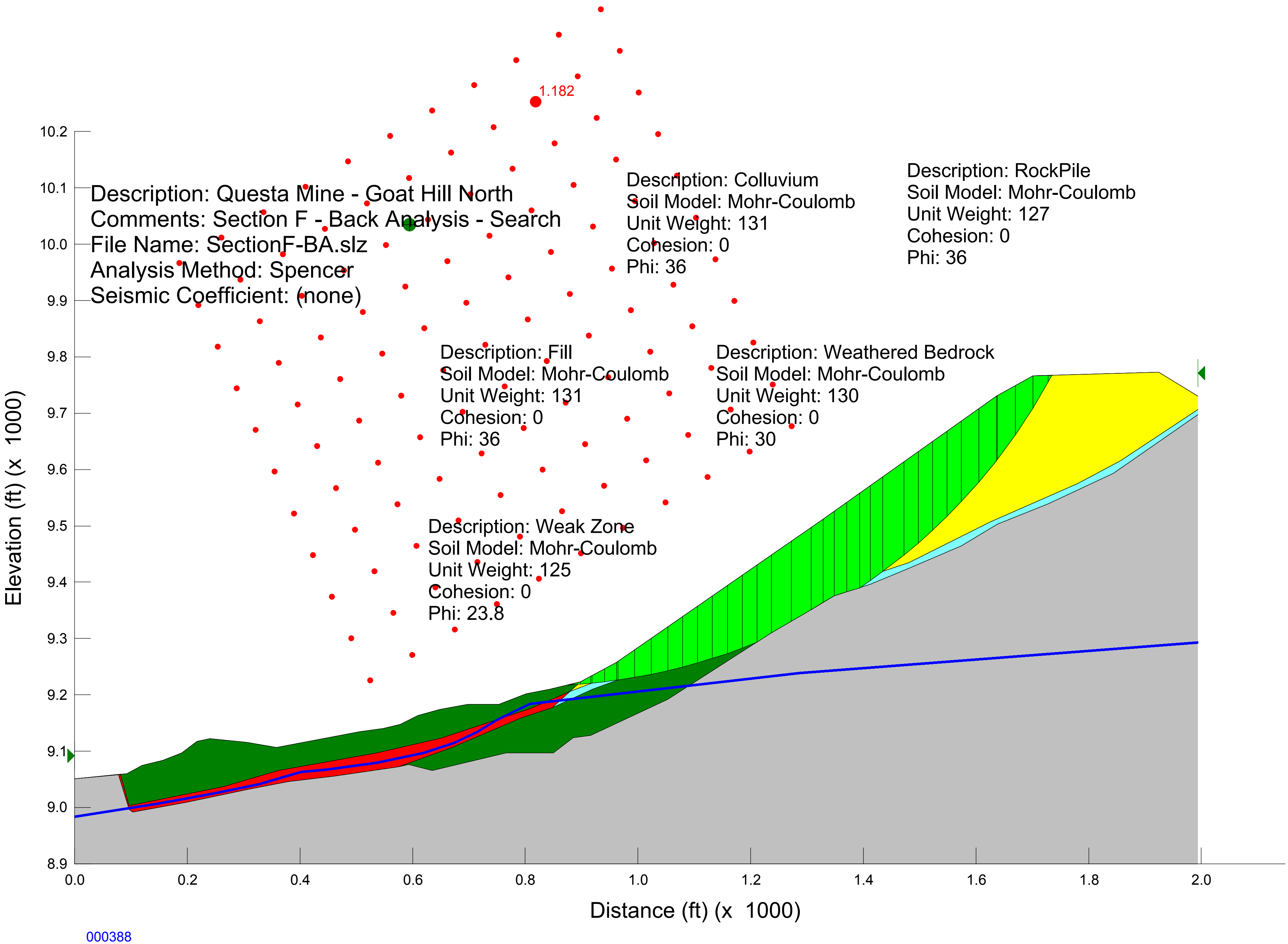
000385

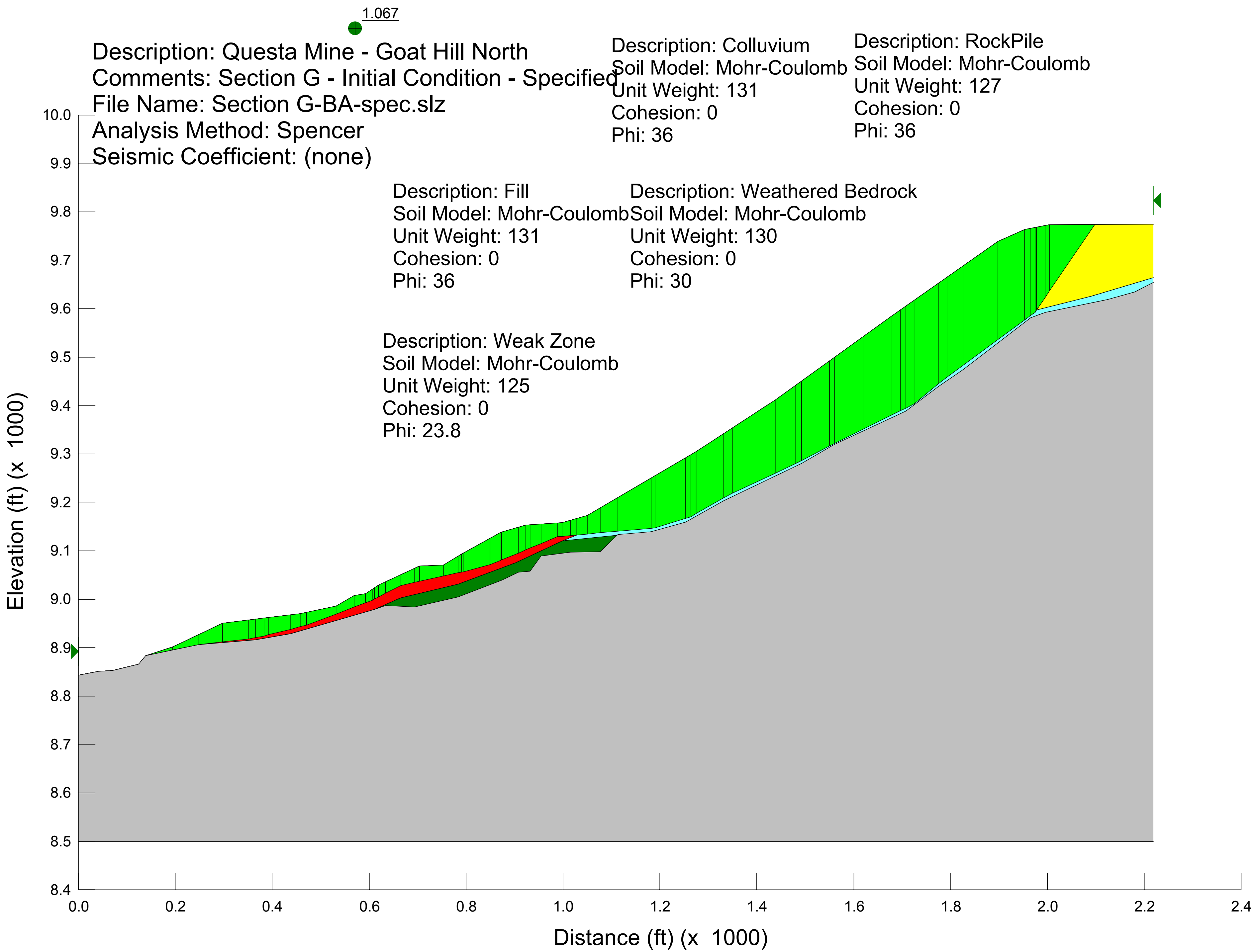

1.038
Description: Questa Mine - Goat Hill North
File Name: Sct E BA 2003.slp
Analysis Method: Spencer
Direction of Slip Movement: Right to Left
Slip Surface Option: Grid and Radius
P.W.P. Option: Piezometric lines with Ru
Seismic Coefficient: (none)
Soil: 1
Description: Waste Rock
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 2
Description: Upper Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 3
Description: Weak Layer
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 23.8
Piezometric Line #: 1
10.3
9.8
9.3
8.8
Elevation (ft) (x 1000)
-0.5
0.0
0.5
1.0
1.5
2.0
2.5
Distance (ft) (x 1000)

000386

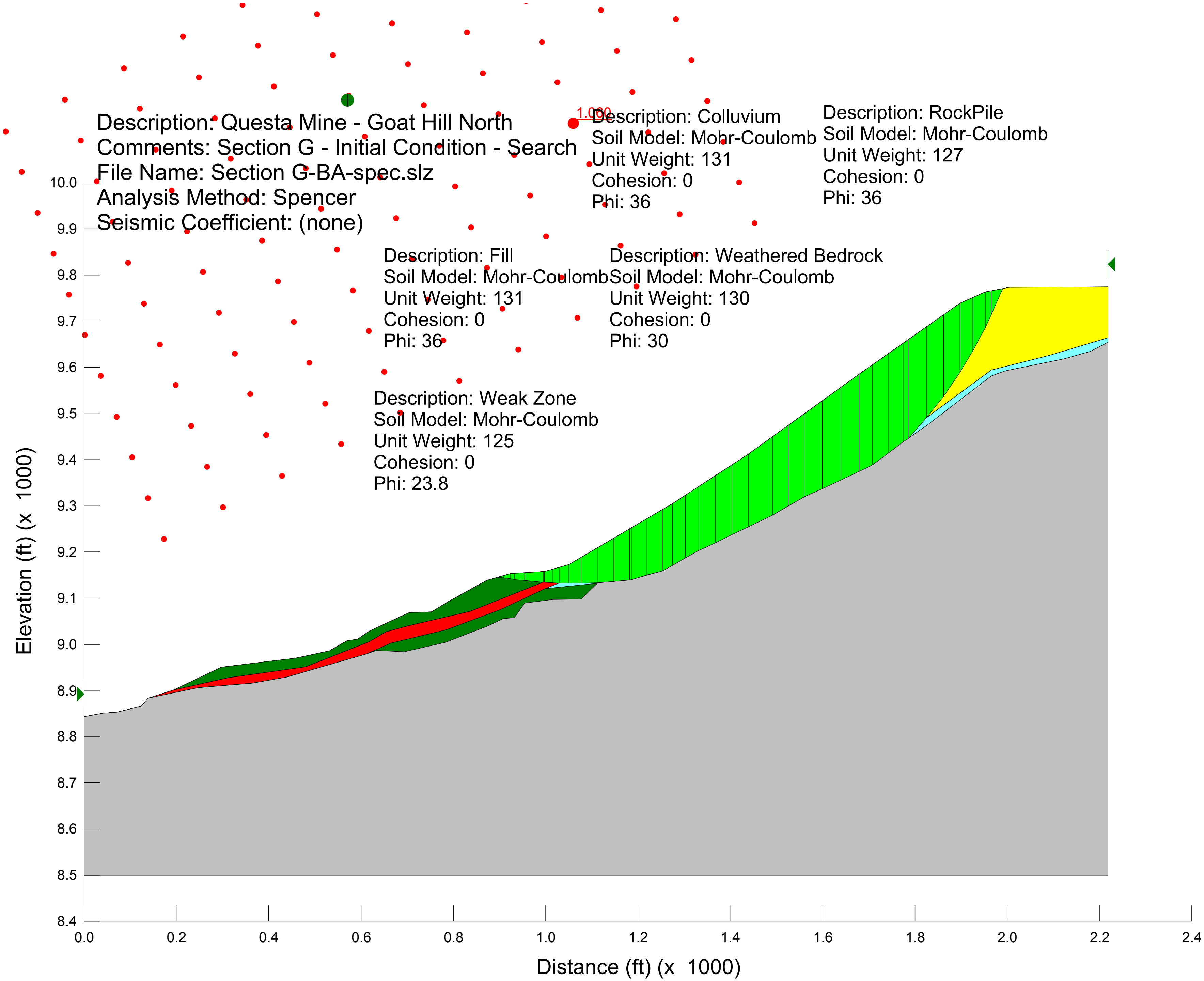

Description: Questa Mine - Goat Hill North
Comments: Section F - Back Analysis - Specified
File Name: SectionF-BA.slz
Analysis Method: Spencer
Seismic Coefficient: (none)
1.590
Description: Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Description: RockPile
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Description: Fill
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Description: Weathered Bedrock
Soil Model: Mohr-Coulomb
Unit Weight: 130
Cohesion: 0
Phi: 30
Description: Weak Zone
Soil Model: Mohr-Coulomb
Unit Weight: 125
Cohesion: 0
Phi: 23.8
Elevation (ft) (x 1000)
Distance (ft) (x 1000)
10.2
10.1
10.0
9.9
9.8
9.7
9.6
9.5
9.4
9.3
9.2
9.1
9.0
8.9
0.0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0

000387


1.182
Description: Questa Mine - Goat Hill North
Comments: Section F - Back Analysis - Search
File Name: SectionF-BA.slz
Analysis Method: Spencer
Seismic Coefficient: (none)
Description: Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Description: RockPile
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Description: Fill
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Description: Weathered Bedrock
Soil Model: Mohr-Coulomb
Unit Weight: 130
Cohesion: 0
Phi: 30
Description: Weak Zone
Soil Model: Mohr-Coulomb
Unit Weight: 125
Cohesion: 0
Phi: 23.8
Elevation (ft) (x 1000)
10.2
10.1
10.0
9.9
9.8
9.7
9.6
9.5
9.4
9.3
9.2
9.1
9.0
8.9
0.0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0
Distance (ft) (x 1000)
000388


1.067
Description: Questa Mine - Goat Hill North
Comments: Section G - Initial Condition - Specified
File Name: Section G-BA-spec.slz
Analysis Method: Spencer
Seismic Coefficient: (none)
Description: Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Description: RockPile
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Description: Fill
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Description: Weathered Bedrock
Soil Model: Mohr-Coulomb
Unit Weight: 130
Cohesion: 0
Phi: 30
Description: Weak Zone
Soil Model: Mohr-Coulomb
Unit Weight: 125
Cohesion: 0
Phi: 23.8
Elevation (ft) (x 1000)
10.0
9.9
9.8
9.7
9.6
9.5
9.4
9.3
9.2
9.1
9.0
8.9
8.8
8.7
8.6
8.5
8.4
0.0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0
2.2
2.4
Distance (ft) (x 1000)

000389

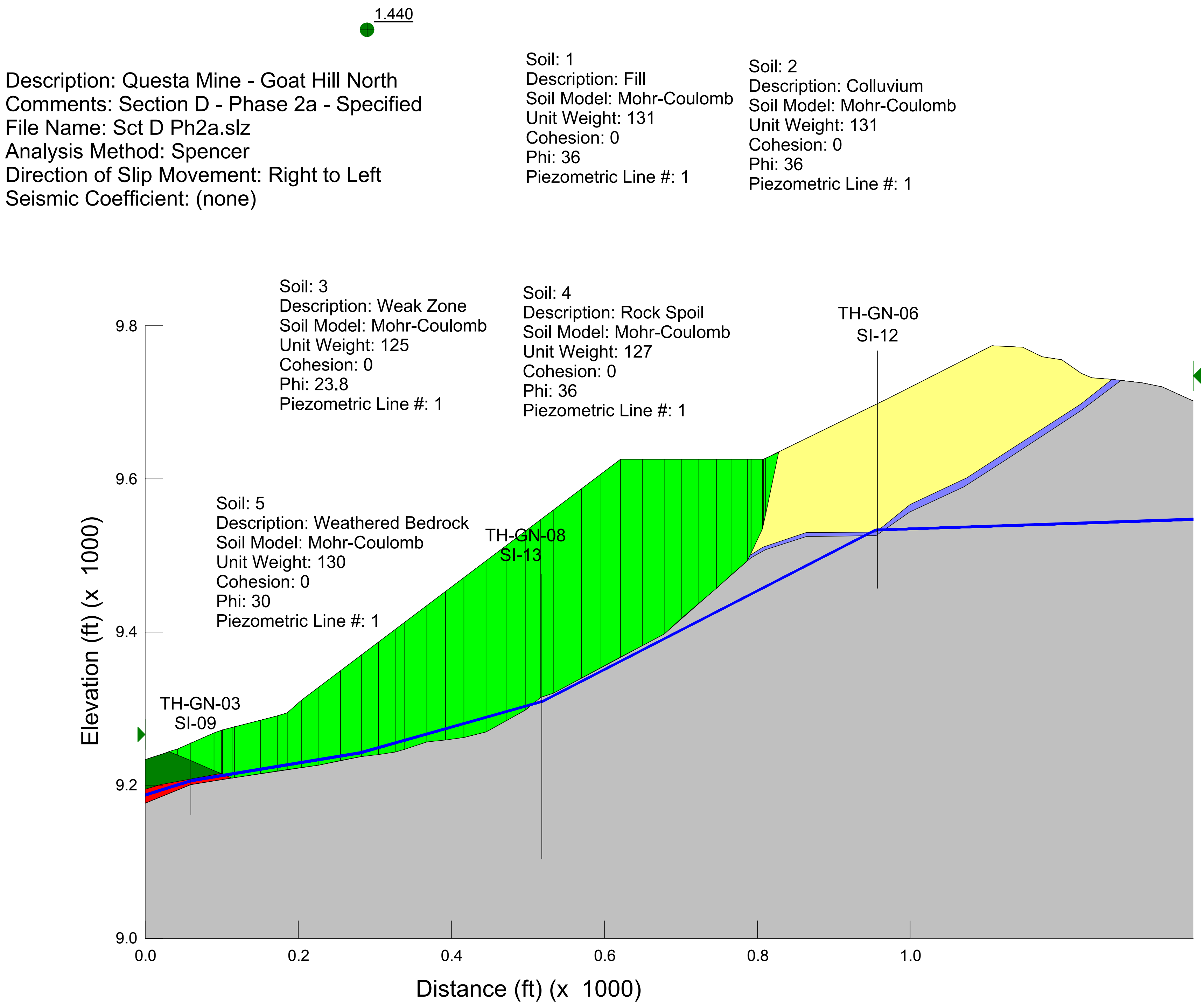


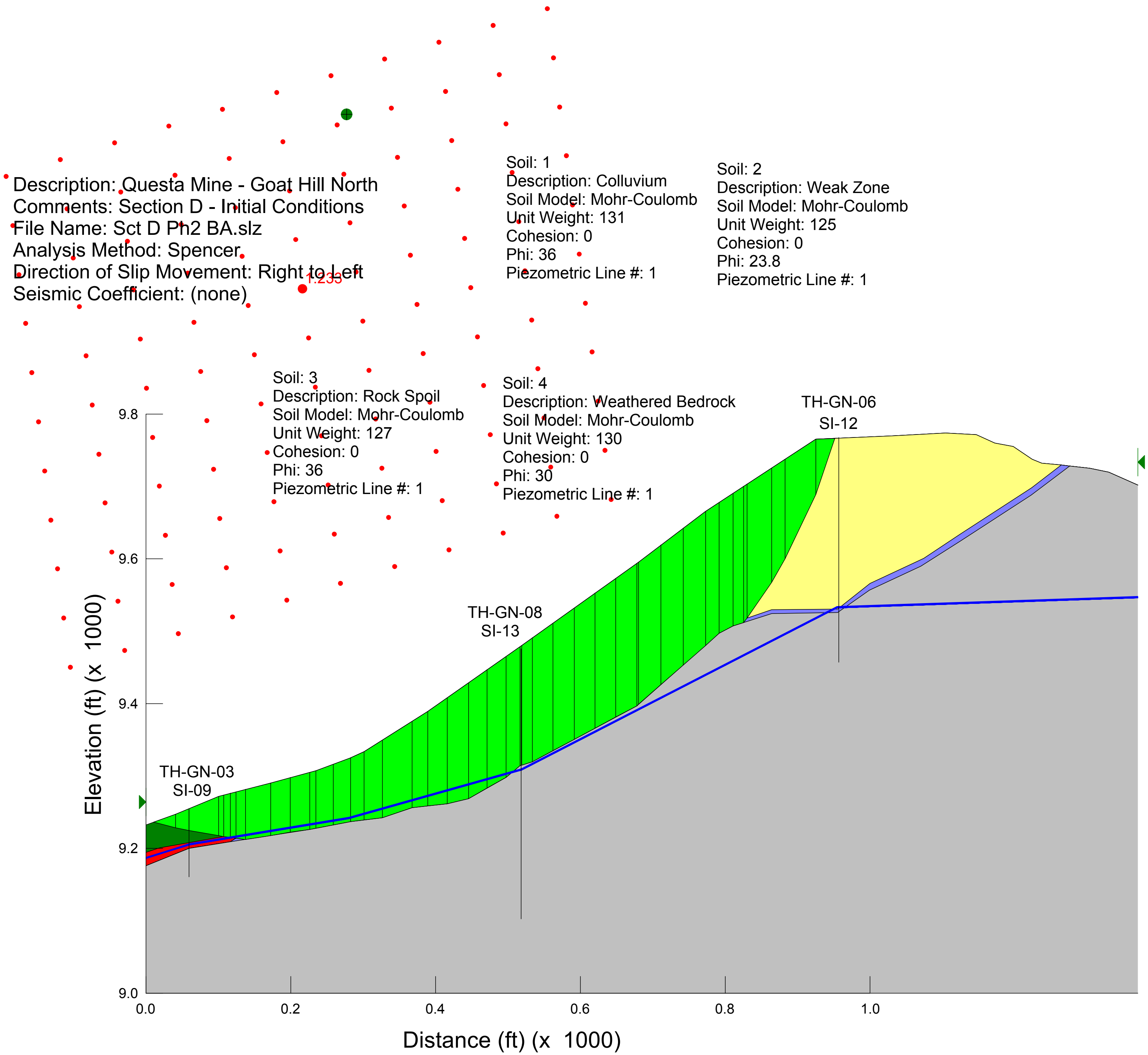
Description: Questa Mine - Goat Hill North
Comments: Section G - Initial Condition - Search
File Name: Section G-BA-spec.slz
Analysis Method: Spencer
Seismic Coefficient: (none)
1.080
Description: Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Description: RockPile
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Description: Fill
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Description: Weathered Bedrock
Soil Model: Mohr-Coulomb
Unit Weight: 130
Cohesion: 0
Phi: 30
Description: Weak Zone
Soil Model: Mohr-Coulomb
Unit Weight: 125
Cohesion: 0
Phi: 23.8
Elevation (ft) (x 1000)
10.0
9.9
9.8
9.7
9.6
9.5
9.4
9.3
9.2
9.1
9.0
8.9
8.8
8.7
8.6
8.5
8.4
0.0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0
2.2
2.4
Distance (ft) (x 1000)

000390

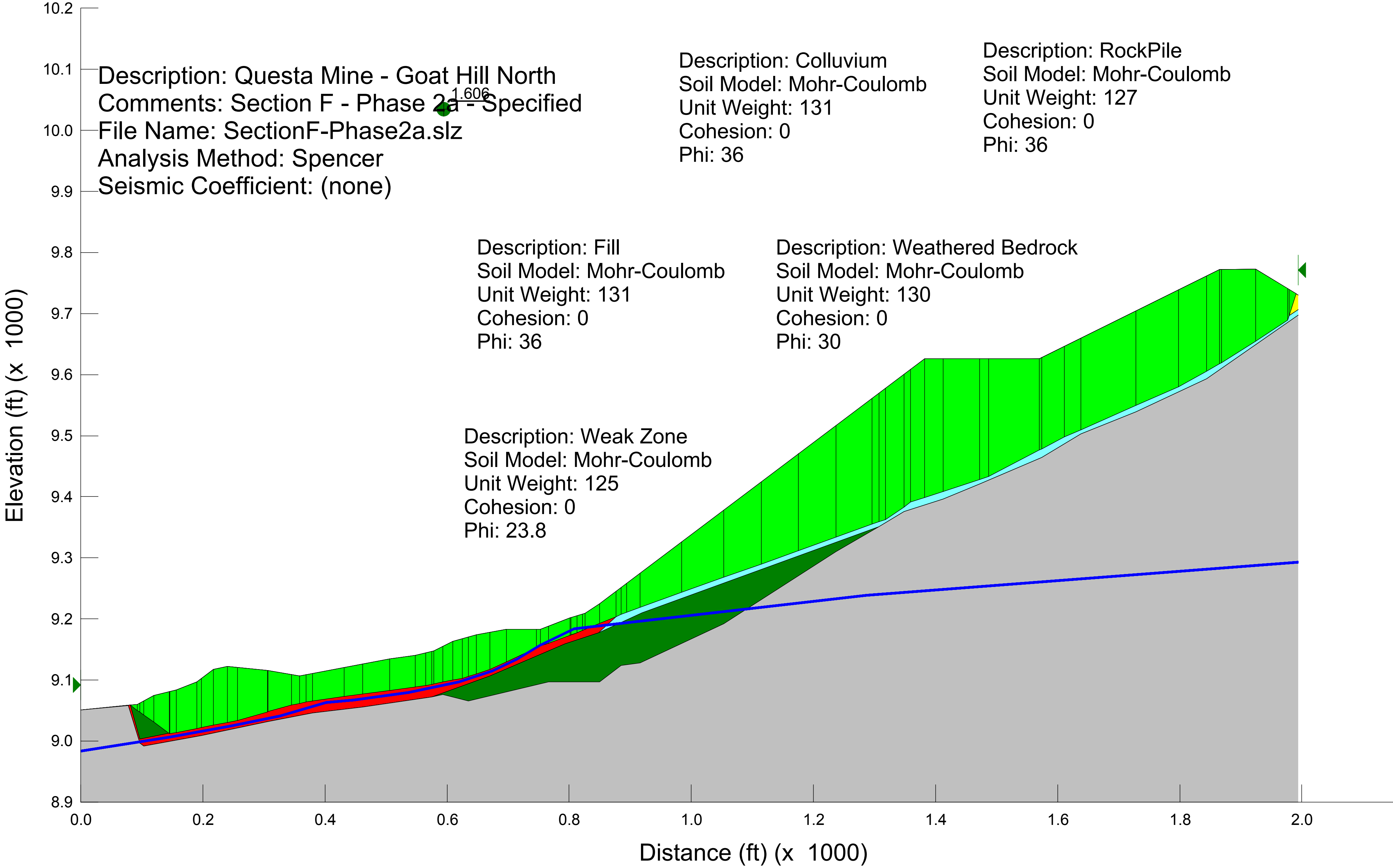
2D Stability Results

Construction Phase 2

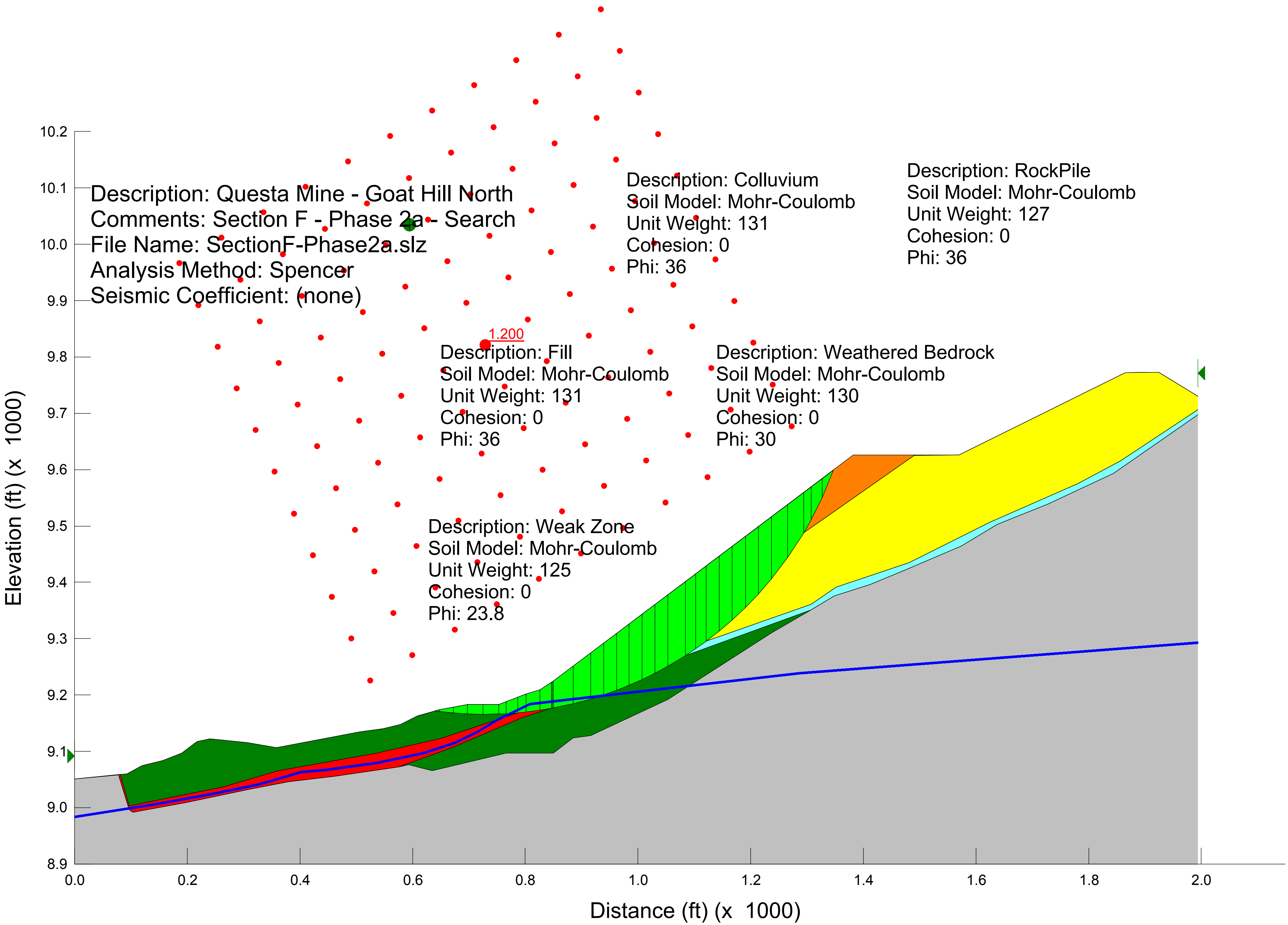

1.440
Description: Questa Mine - Goat Hill North
Comments: Section D - Phase 2a - Specified
File Name: Sct D Ph2a.slz
Analysis Method: Spencer
Direction of Slip Movement: Right to Left
Seismic Coefficient: (none)
Soil: 1
Description: Fill
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 2
Description: Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 3
Description: Weak Zone
Soil Model: Mohr-Coulomb
Unit Weight: 125
Cohesion: 0
Phi: 23.8
Piezometric Line #: 1
Soil: 4
Description: Rock Spoil
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 5
Description: Weathered Bedrock
Soil Model: Mohr-Coulomb
Unit Weight: 130
Cohesion: 0
Phi: 30
Piezometric Line #: 1
TH-GN-06
SI-12
TH-GN-08
SI-13
TH-GN-03
SI-09
Elevation (ft) (x 1000)
9.8
9.6
9.4
9.2
9.0
0.0
0.2
0.4
0.6
0.8
1.0
Distance (ft) (x 1000)

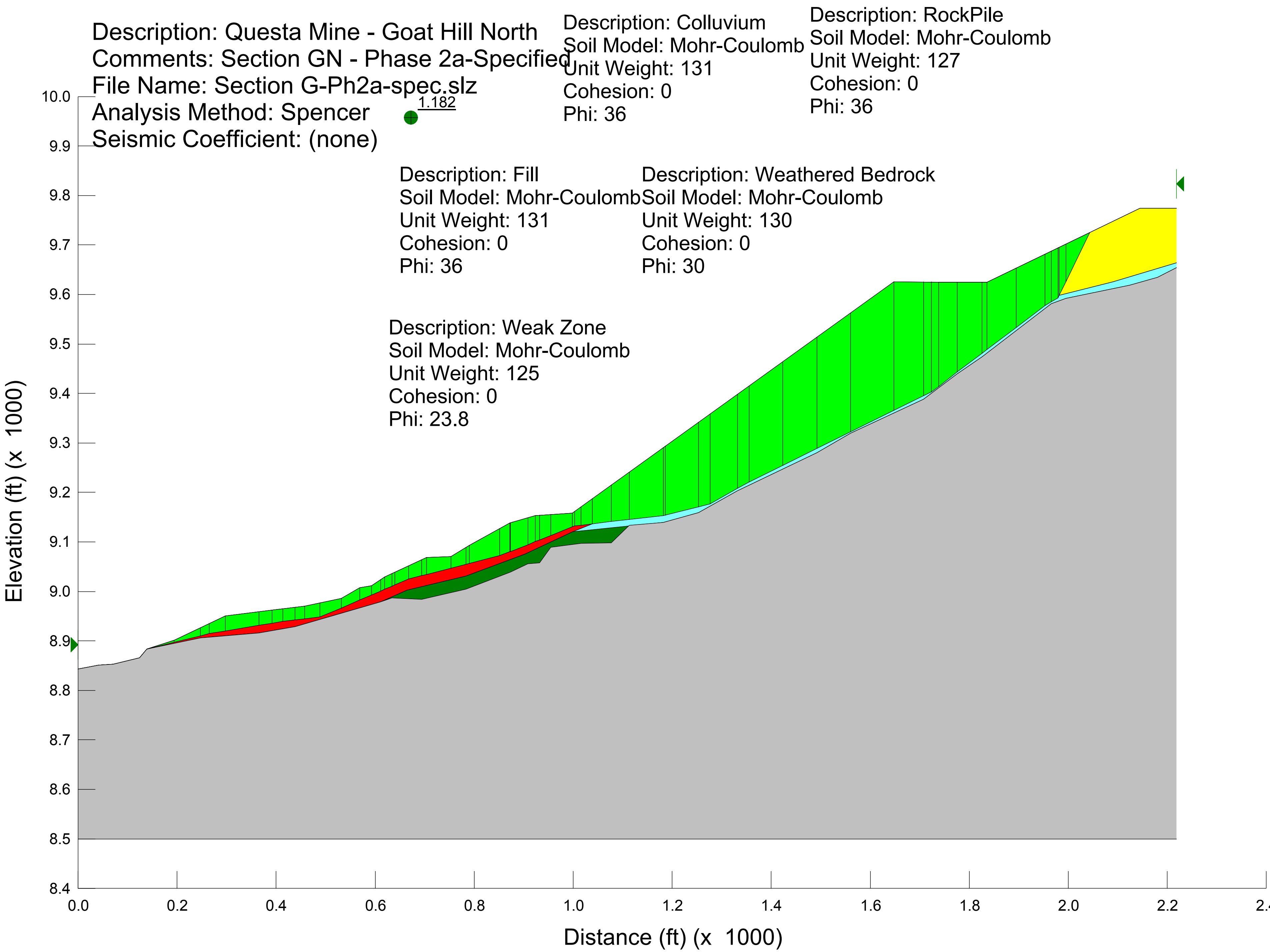


Description: Questa Mine - Goat Hill North
Comments: Section D - Initial Conditions
File Name: Sct D Ph2 BA.slz
Analysis Method: Spencer
Direction of Slip Movement: Right to Left
Seismic Coefficient: (none)
1.233
Soil: 1
Description: Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 2
Description: Weak Zone
Soil Model: Mohr-Coulomb
Unit Weight: 125
Cohesion: 0
Phi: 23.8
Piezometric Line #: 1
Soil: 3
Description: Rock Spoil
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 4
Description: Weathered Bedrock
Soil Model: Mohr-Coulomb
Unit Weight: 130
Cohesion: 0
Phi: 30
Piezometric Line #: 1
TH-GN-06
SI-12
TH-GN-08
SI-13
TH-GN-03
SI-09
Elevation (ft) (x 1000)
9.8
9.6
9.4
9.2
9.0
0.0
0.2
0.4
0.6
0.8
1.0
Distance (ft) (x 1000)


Description: Questa Mine - Goat Hill North
Comments: Section F - Phase 2a - Specified
File Name: SectionF-Phase2a.slz
Analysis Method: Spencer
Seismic Coefficient: (none)
1.606
Description: Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Description: RockPile
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Description: Fill
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Description: Weathered Bedrock
Soil Model: Mohr-Coulomb
Unit Weight: 130
Cohesion: 0
Phi: 30
Description: Weak Zone
Soil Model: Mohr-Coulomb
Unit Weight: 125
Cohesion: 0
Phi: 23.8
Elevation (ft) (x 1000)
10.2
10.1
10.0
9.9
9.8
9.7
9.6
9.5
9.4
9.3
9.2
9.1
9.0
8.9
0.0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0
Distance (ft) (x 1000)

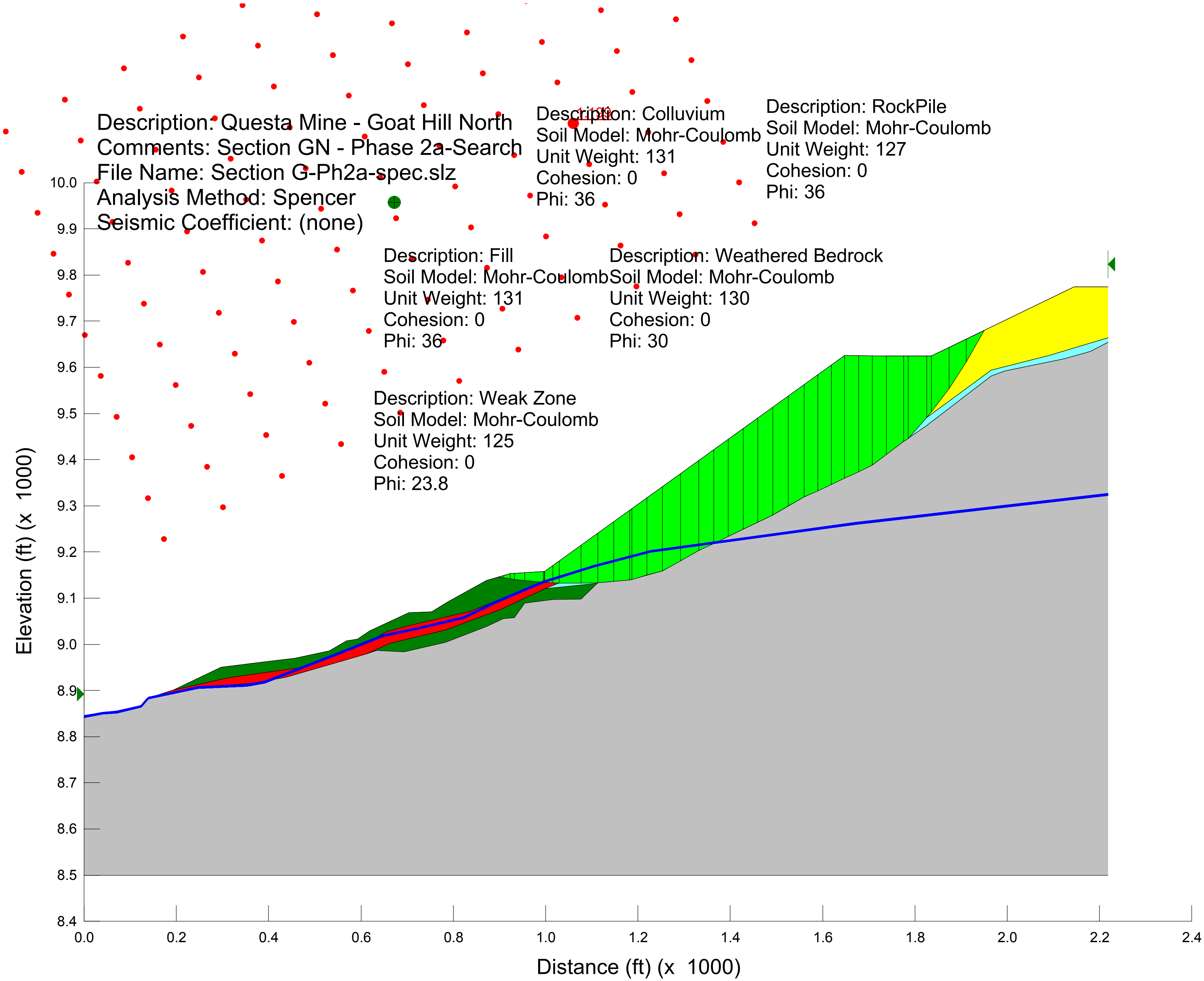
000394


Description: Questa Mine - Goat Hill North
Comments: Section F - Phase 2a - Search
File Name: SectionF-Phase2a.slz
Analysis Method: Spencer
Seismic Coefficient: (none)
Description: Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Description: RockPile
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
1.200
Description: Fill
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Description: Weathered Bedrock
Soil Model: Mohr-Coulomb
Unit Weight: 130
Cohesion: 0
Phi: 30
Description: Weak Zone
Soil Model: Mohr-Coulomb
Unit Weight: 125
Cohesion: 0
Phi: 23.8
Elevation (ft) (x 1000)
10.2
10.1
10.0
9.9
9.8
9.7
9.6
9.5
9.4
9.3
9.2
9.1
9.0
8.9
0.0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0
Distance (ft) (x 1000)
000395



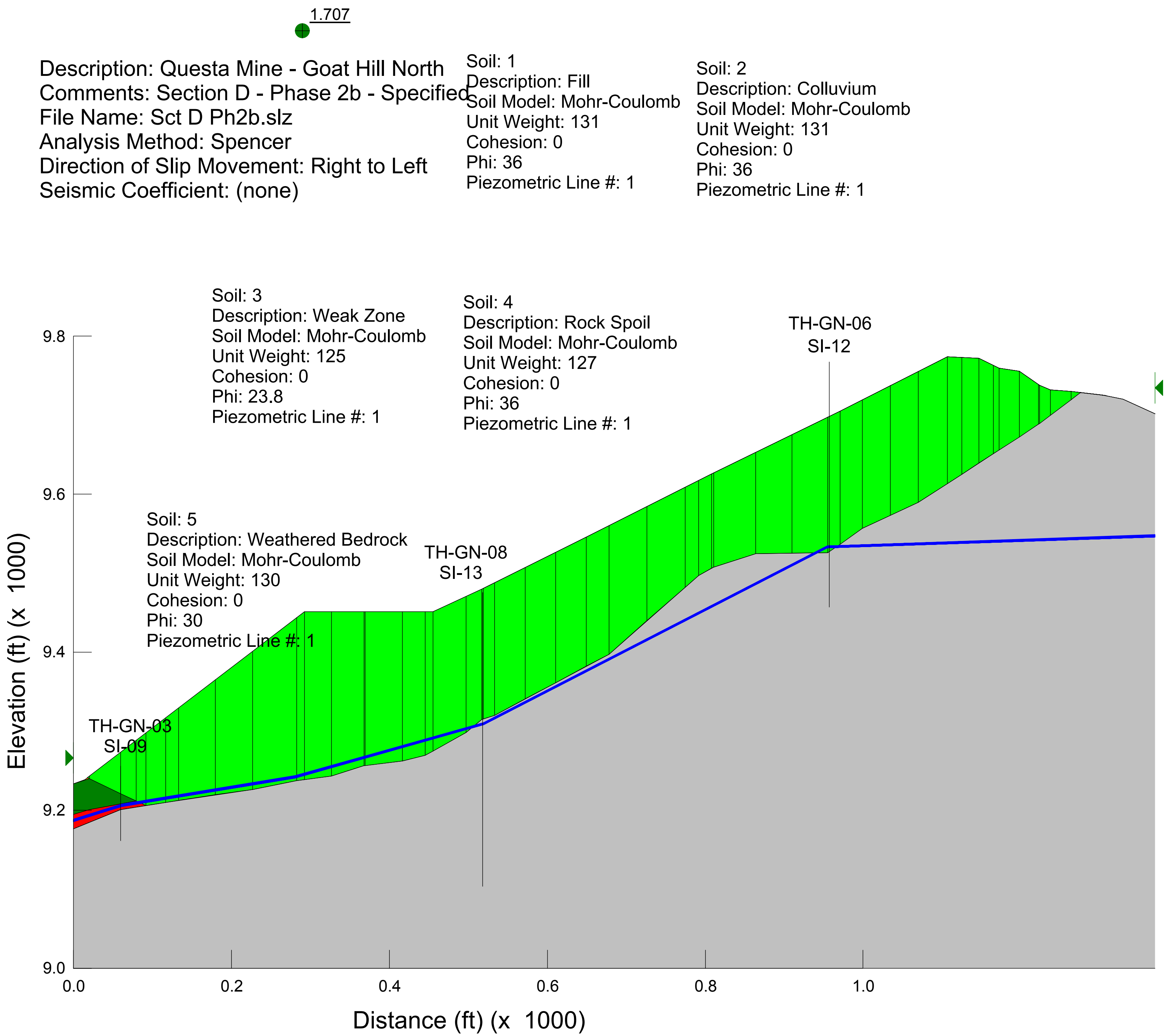
Description: Questa Mine - Goat Hill North
Comments: Section GN - Phase 2a-Specified
File Name: Section G-Ph2a-spec.slz
Analysis Method: Spencer
Seismic Coefficient: (none)
1.182
Description: Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Description: RockPile
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Description: Fill
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Description: Weathered Bedrock
Soil Model: Mohr-Coulomb
Unit Weight: 130
Cohesion: 0
Phi: 30
Description: Weak Zone
Soil Model: Mohr-Coulomb
Unit Weight: 125
Cohesion: 0
Phi: 23.8
Elevation (ft) (x 1000)
10.0
9.9
9.8
9.7
9.6
9.5
9.4
9.3
9.2
9.1
9.0
8.9
8.8
8.7
8.6
8.5
8.4
0.0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0
2.2
2.4
Distance (ft) (x 1000)

000396



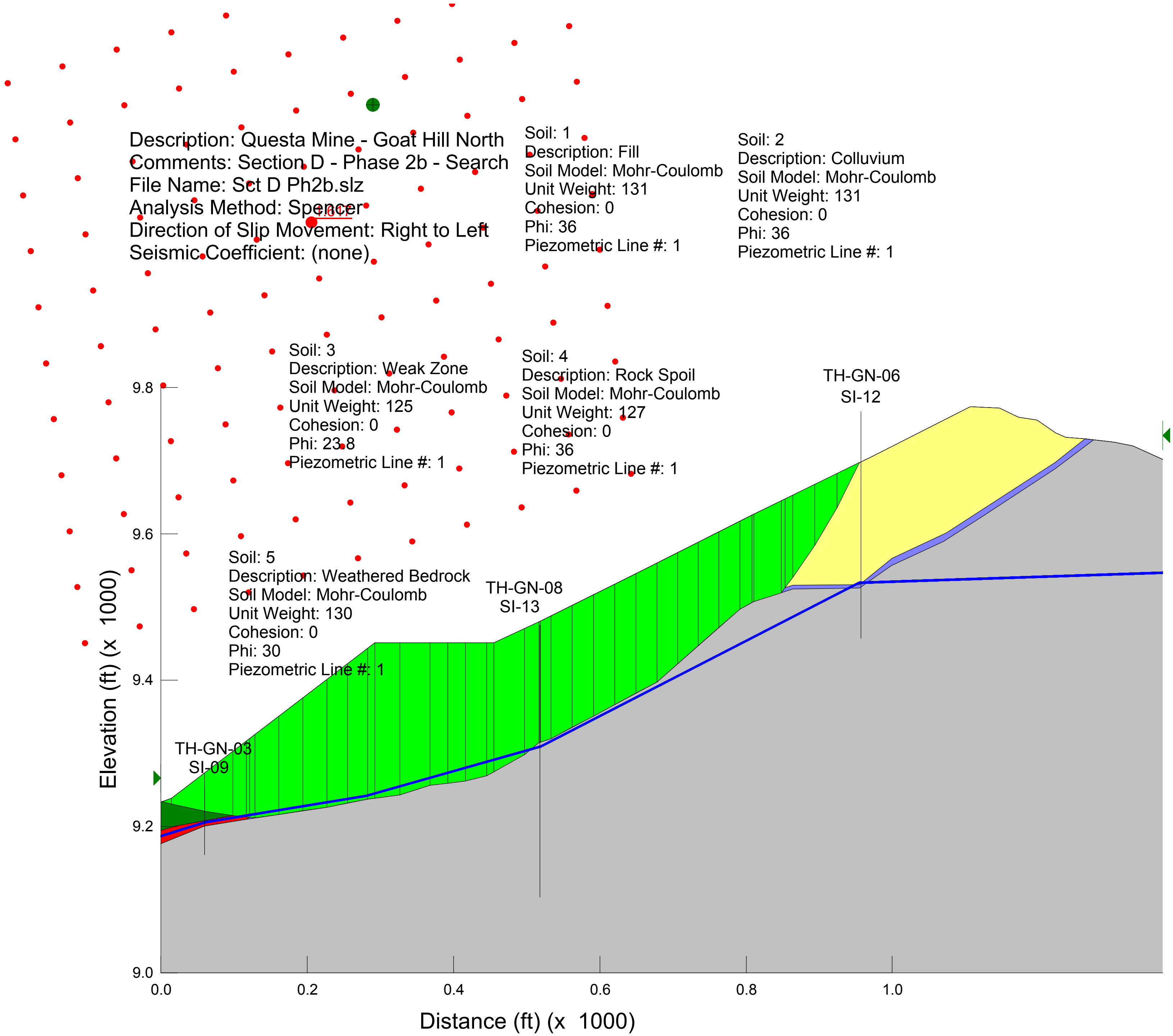
Description: Questa Mine - Goat Hill North
Comments: Section GN - Phase 2a-Search
File Name: Section G-Ph2a-spec.slz
Analysis Method: Spencer
Seismic Coefficient: (none)
Description: Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Description: RockPile
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Description: Fill
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Description: Weathered Bedrock
Soil Model: Mohr-Coulomb
Unit Weight: 130
Cohesion: 0
Phi: 30
Description: Weak Zone
Soil Model: Mohr-Coulomb
Unit Weight: 125
Cohesion: 0
Phi: 23.8
Elevation (ft) (x 1000)
Distance (ft) (x 1000)
10.0
9.9
9.8
9.7
9.6
9.5
9.4
9.3
9.2
9.1
9.0
8.9
8.8
8.7
8.6
8.5
8.4
0.0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0
2.2
2.4

000397

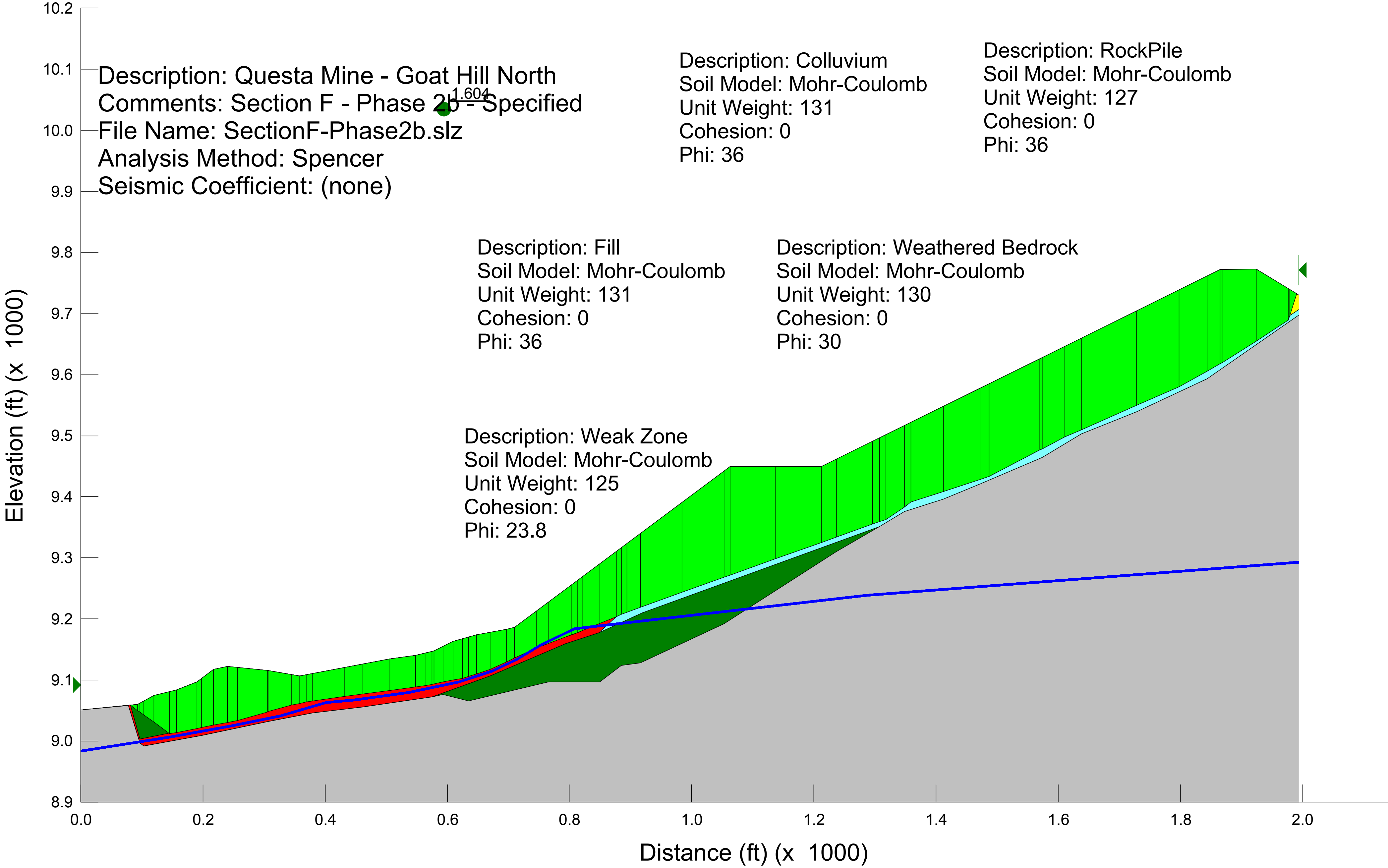


1.707
Description: Questa Mine - Goat Hill North
Comments: Section D - Phase 2b - Specified
File Name: Sct D Ph2b.slz
Analysis Method: Spencer
Direction of Slip Movement: Right to Left
Seismic Coefficient: (none)
Soil: 1
Description: Fill
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 2
Description: Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 3
Description: Weak Zone
Soil Model: Mohr-Coulomb
Unit Weight: 125
Cohesion: 0
Phi: 23.8
Piezometric Line #: 1
Soil: 4
Description: Rock Spoil
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 5
Description: Weathered Bedrock
Soil Model: Mohr-Coulomb
Unit Weight: 130
Cohesion: 0
Phi: 30
Piezometric Line #: 1
TH-GN-06
SI-12
TH-GN-08
SI-13
TH-GN-03
SI-09
Elevation (ft) (x 1000)
9.8
9.6
9.4
9.2
9.0
0.0
0.2
0.4
0.6
0.8
1.0
Distance (ft) (x 1000)

000398

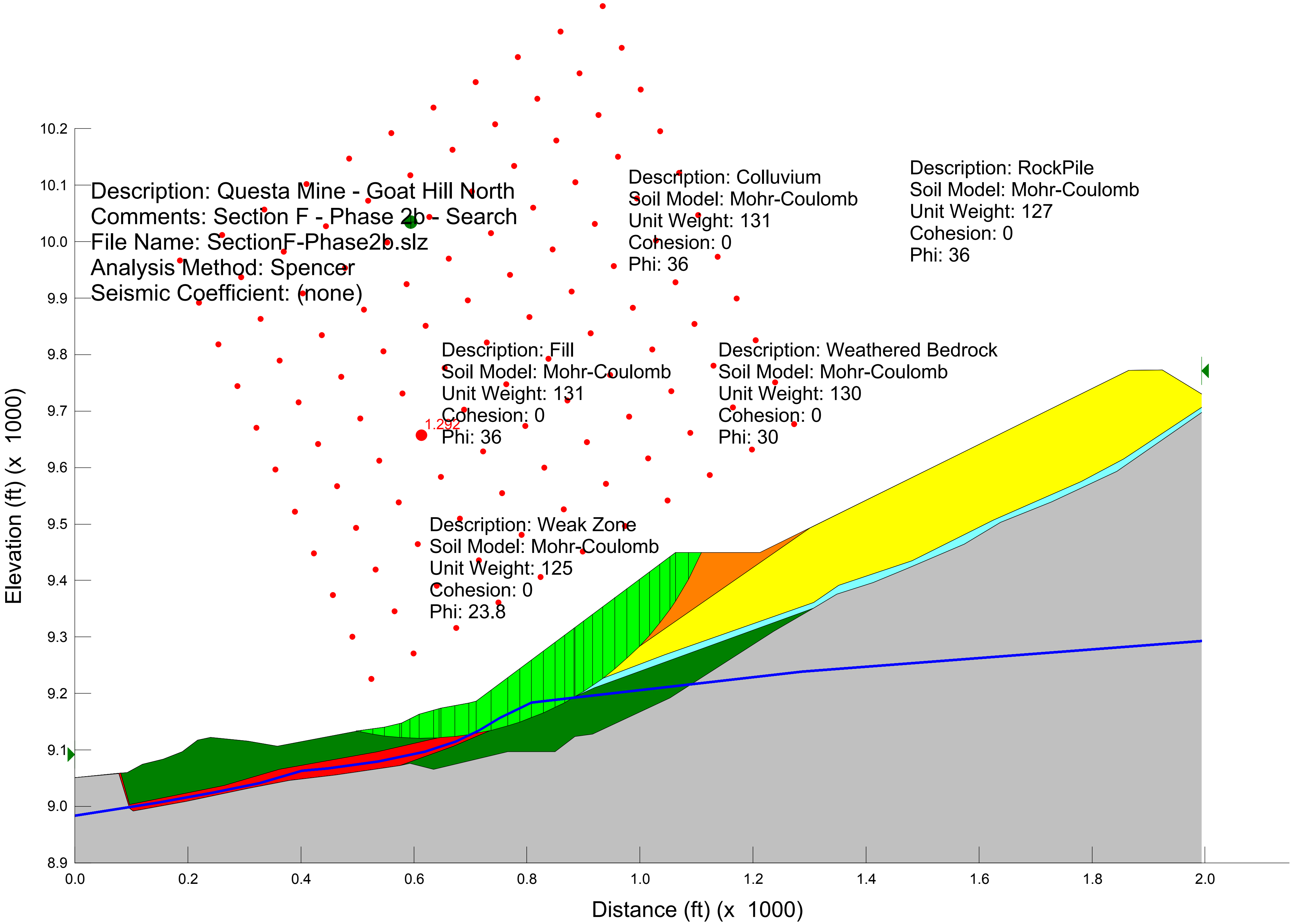

Description: Questa Mine - Goat Hill North
Comments: Section D - Phase 2b - Search
File Name: Sct D Ph2b.slz
Analysis Method: Spencer
Direction of Slip Movement: Right to Left
Seismic Coefficient: (none)
Soil: 1
Description: Fill
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 2
Description: Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 3
Description: Weak Zone
Soil Model: Mohr-Coulomb
Unit Weight: 125
Cohesion: 0
Phi: 23.8
Piezometric Line #: 1
Soil: 4
Description: Rock Spoil
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 5
Description: Weathered Bedrock
Soil Model: Mohr-Coulomb
Unit Weight: 130
Cohesion: 0
Phi: 30
Piezometric Line #: 1
TH-GN-06
SI-12
TH-GN-08
SI-13
TH-GN-03
SI-09
Elevation (ft) (x 1000)
9.8
9.6
9.4
9.2
9.0
0.0
0.2
0.4
0.6
0.8
1.0
Distance (ft) (x 1000)

000399



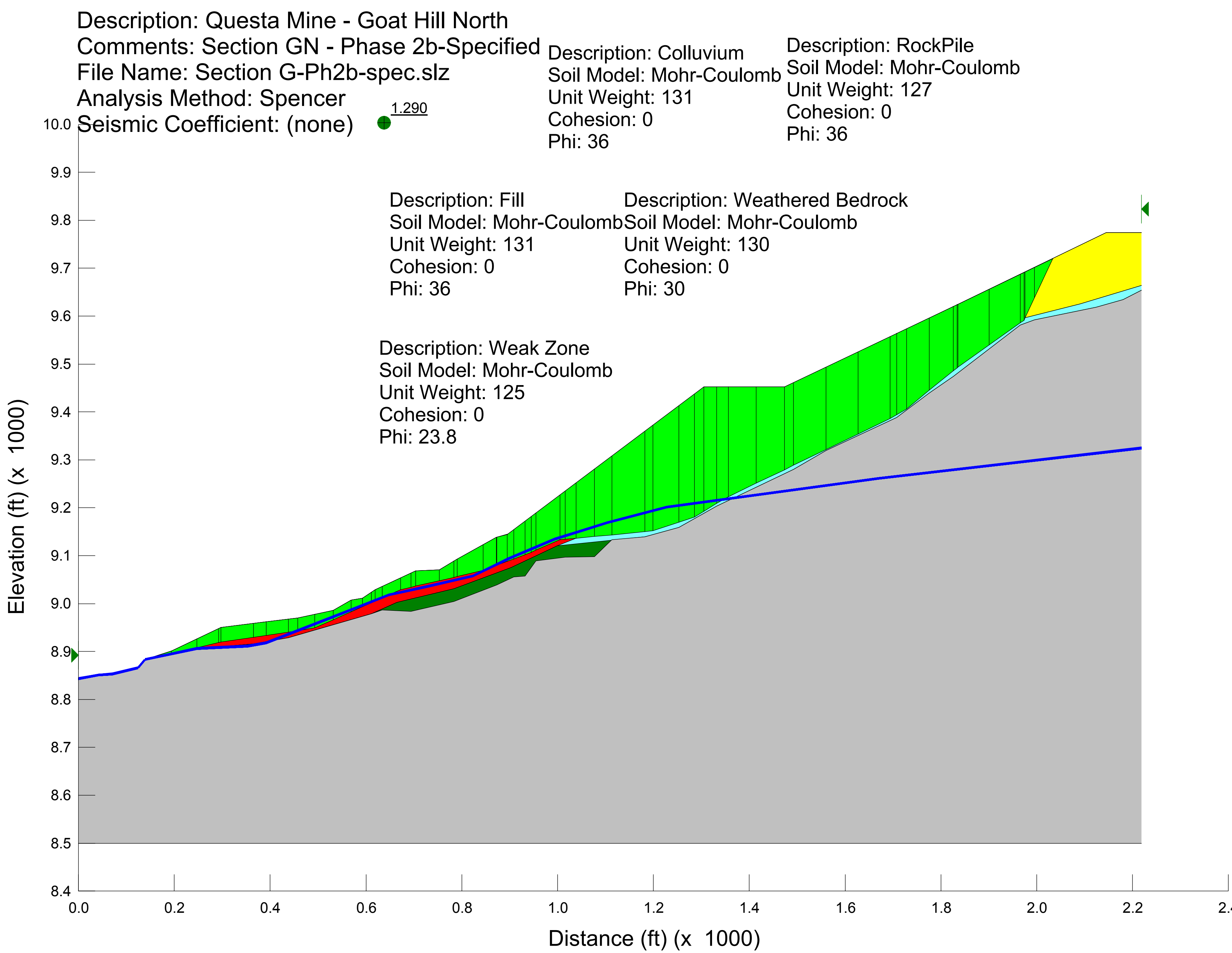
Description: Questa Mine - Goat Hill North
Comments: Section F - Phase 2b - Specified
File Name: SectionF-Phase2b.slz
Analysis Method: Spencer
Seismic Coefficient: (none)
1.604
Description: Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Description: RockPile
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Description: Fill
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Description: Weathered Bedrock
Soil Model: Mohr-Coulomb
Unit Weight: 130
Cohesion: 0
Phi: 30
Description: Weak Zone
Soil Model: Mohr-Coulomb
Unit Weight: 125
Cohesion: 0
Phi: 23.8
Elevation (ft) (x 1000)
10.2
10.1
10.0
9.9
9.8
9.7
9.6
9.5
9.4
9.3
9.2
9.1
9.0
8.9
0.0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0
Distance (ft) (x 1000)

000400



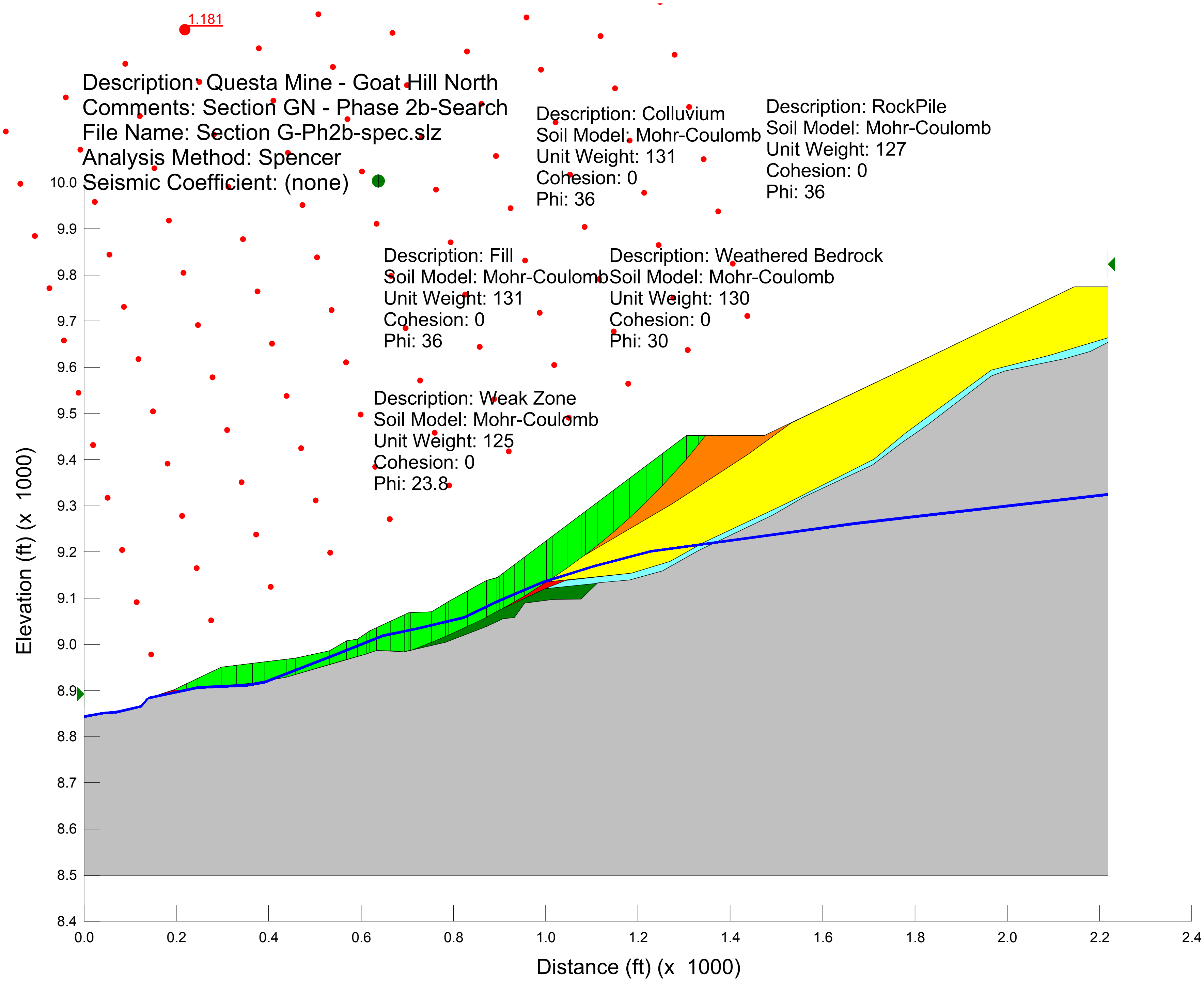
Description: Questa Mine - Goat Hill North
Comments: Section F - Phase 2b - Search
File Name: SectionF-Phase2b.slz
Analysis Method: Spencer
Seismic Coefficient: (none)
Description: Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Description: RockPile
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Description: Fill
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Description: Weathered Bedrock
Soil Model: Mohr-Coulomb
Unit Weight: 130
Cohesion: 0
Phi: 30
1.292
Description: Weak Zone
Soil Model: Mohr-Coulomb
Unit Weight: 125
Cohesion: 0
Phi: 23.8
Elevation (ft) (x 1000)
10.2
10.1
10.0
9.9
9.8
9.7
9.6
9.5
9.4
9.3
9.2
9.1
9.0
8.9
0.0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0
Distance (ft) (x 1000)

000401



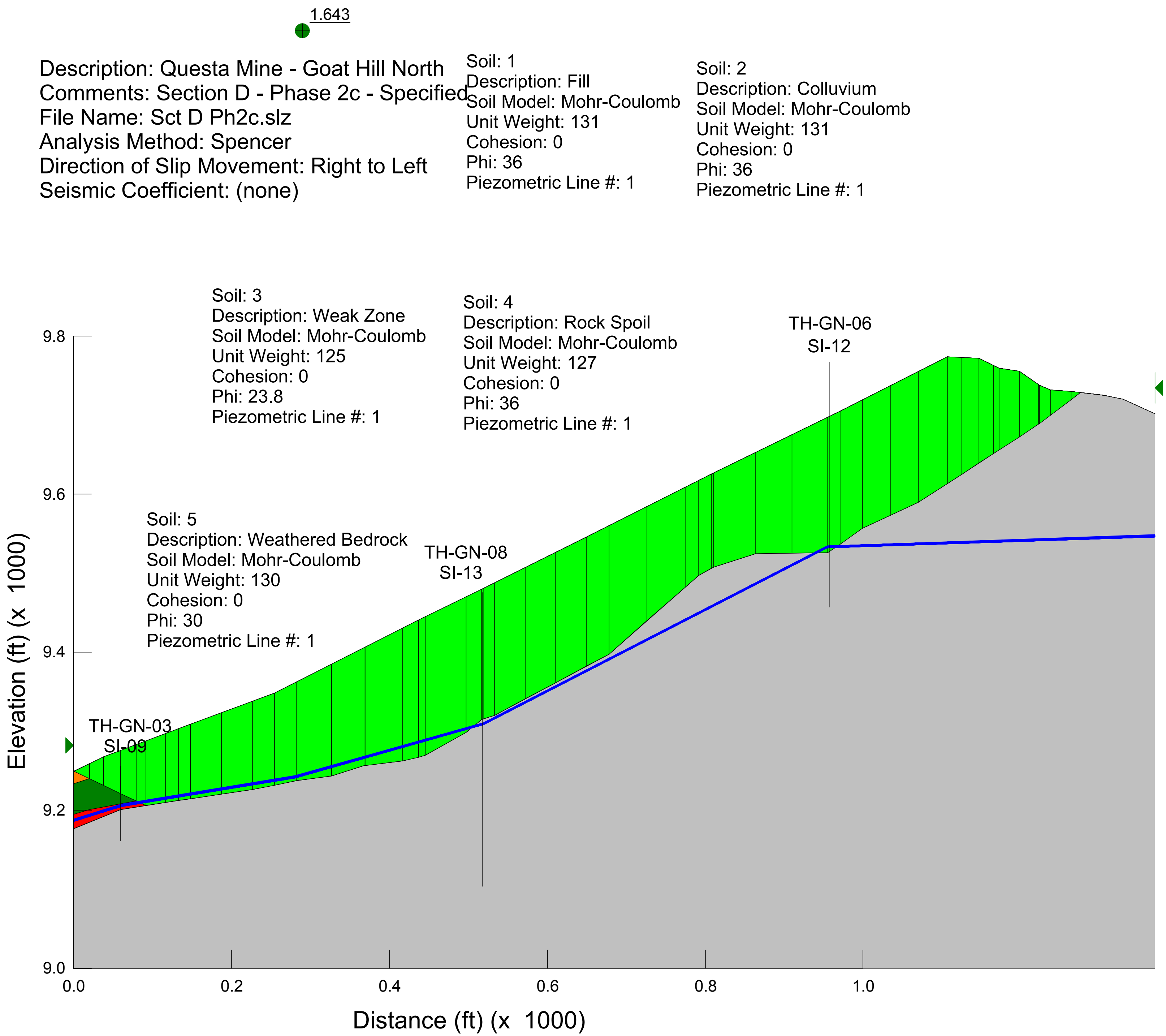
Description: Questa Mine - Goat Hill North
Comments: Section GN - Phase 2b-Specified
File Name: Section G-Ph2b-spec.slz
Analysis Method: Spencer
Seismic Coefficient: (none)
1.290
Description: Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Description: RockPile
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Description: Fill
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Description: Weathered Bedrock
Soil Model: Mohr-Coulomb
Unit Weight: 130
Cohesion: 0
Phi: 30
Description: Weak Zone
Soil Model: Mohr-Coulomb
Unit Weight: 125
Cohesion: 0
Phi: 23.8
Elevation (ft) (x 1000)
10.0
9.9
9.8
9.7
9.6
9.5
9.4
9.3
9.2
9.1
9.0
8.9
8.8
8.7
8.6
8.5
8.4
0.0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0
2.2
2.4
Distance (ft) (x 1000)

000402

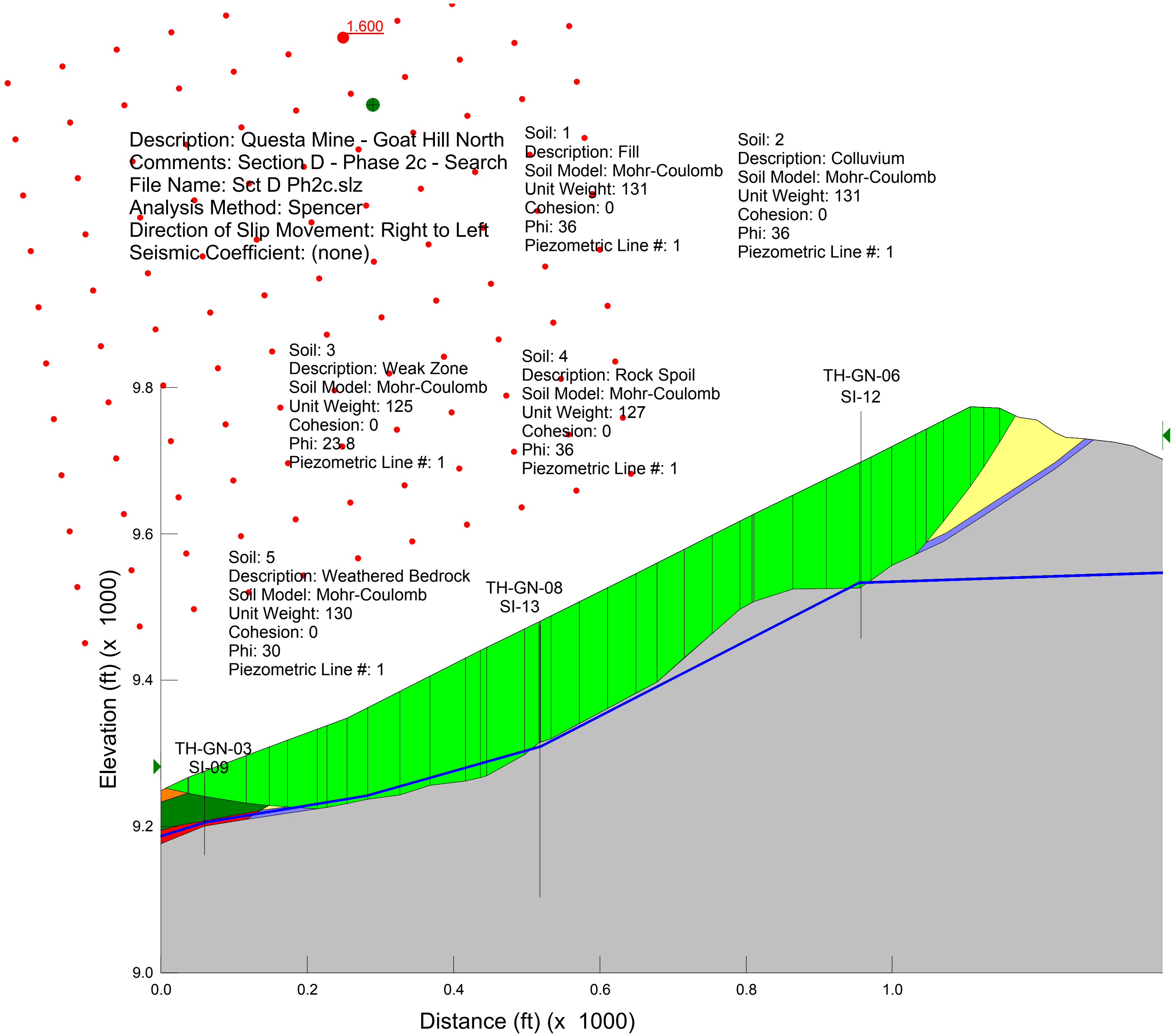


1.181
Description: Questa Mine - Goat Hill North
Comments: Section GN - Phase 2b-Search
File Name: Section G-Ph2b-spec.slz
Analysis Method: Spencer
Seismic Coefficient: (none)
Description: Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Description: RockPile
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Description: Fill
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Description: Weathered Bedrock
Soil Model: Mohr-Coulomb
Unit Weight: 130
Cohesion: 0
Phi: 30
Description: Weak Zone
Soil Model: Mohr-Coulomb
Unit Weight: 125
Cohesion: 0
Phi: 23.8
Elevation (ft) (x 1000)
10.0
9.9
9.8
9.7
9.6
9.5
9.4
9.3
9.2
9.1
9.0
8.9
8.8
8.7
8.6
8.5
8.4
0.0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0
2.2
2.4
Distance (ft) (x 1000)

000403

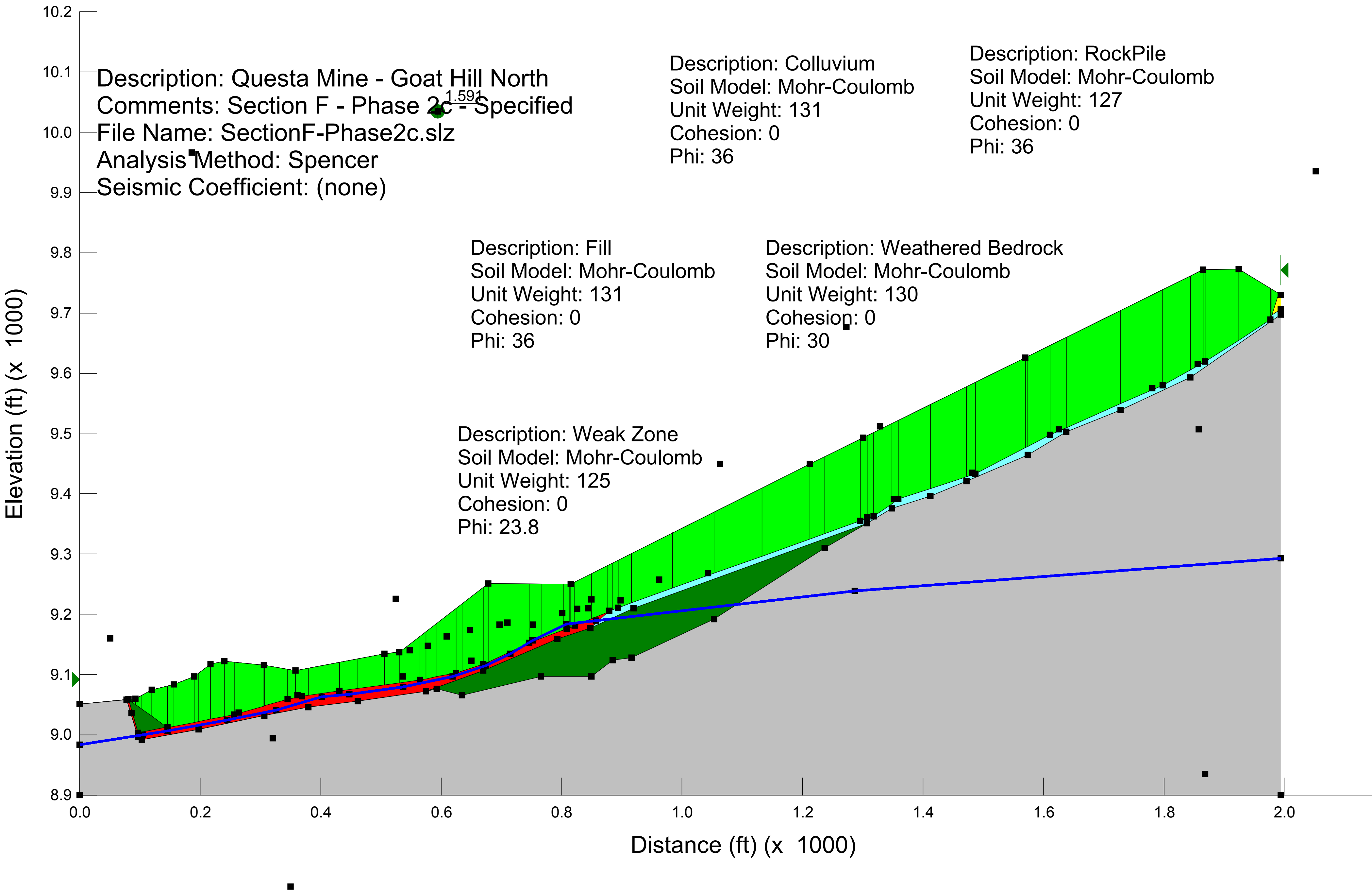

1.643
Description: Questa Mine - Goat Hill North
Comments: Section D - Phase 2c - Specified
File Name: Sct D Ph2c.slz
Analysis Method: Spencer
Direction of Slip Movement: Right to Left
Seismic Coefficient: (none)
Soil: 1
Description: Fill
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 2
Description: Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 3
Description: Weak Zone
Soil Model: Mohr-Coulomb
Unit Weight: 125
Cohesion: 0
Phi: 23.8
Piezometric Line #: 1
Soil: 4
Description: Rock Spoil
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 5
Description: Weathered Bedrock
Soil Model: Mohr-Coulomb
Unit Weight: 130
Cohesion: 0
Phi: 30
Piezometric Line #: 1
TH-GN-06
SI-12
TH-GN-08
SI-13
TH-GN-03
SI-09
9.8
9.6
9.4
9.2
9.0
0.0
0.2
0.4
0.6
0.8
1.0
Elevation (ft) (x 1000)
Distance (ft) (x 1000)

000404



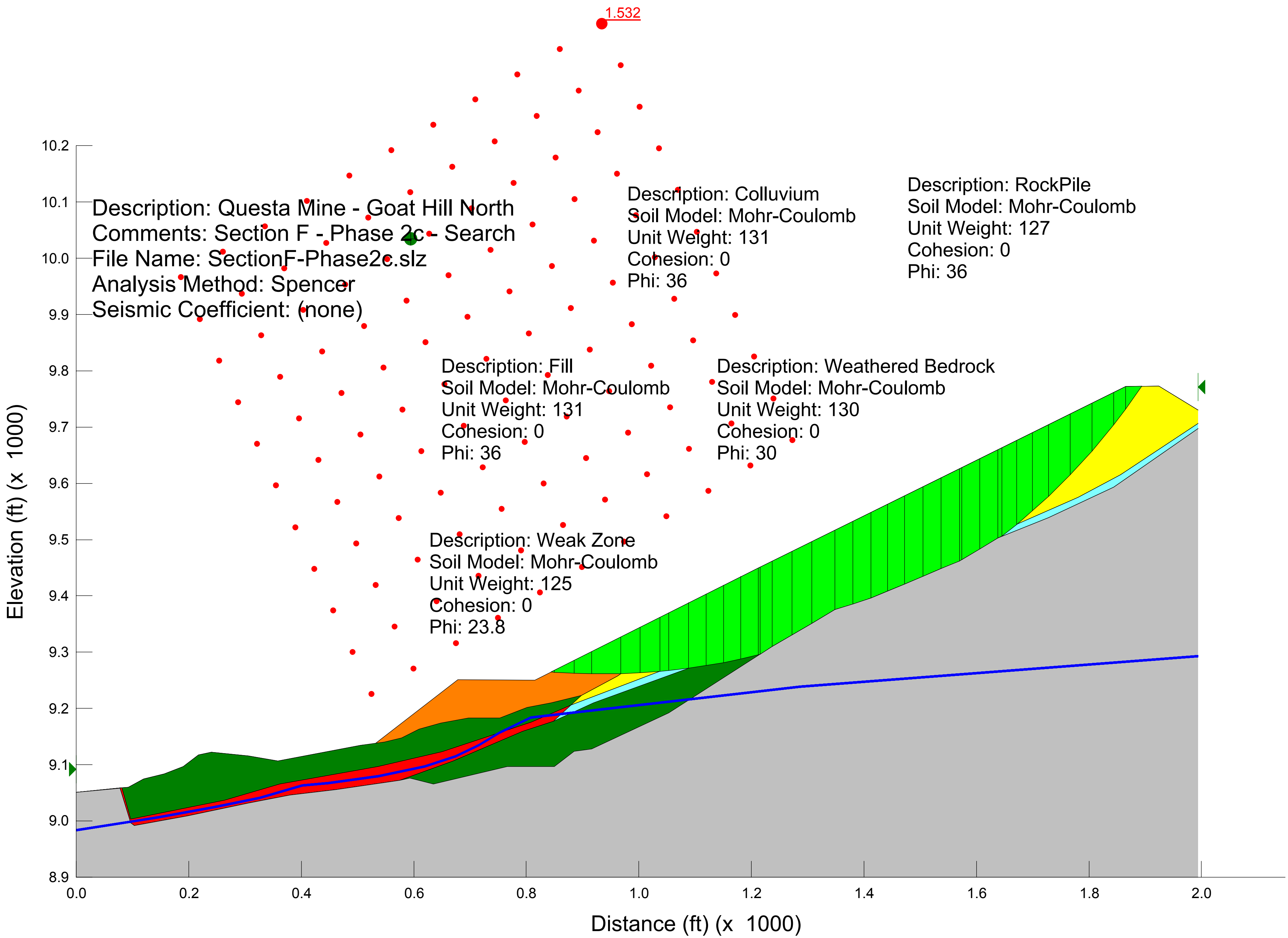
1.600
Description: Questa Mine - Goat Hill North
Comments: Section D - Phase 2c - Search
File Name: Sct D Ph2c.slz
Analysis Method: Spencer
Direction of Slip Movement: Right to Left
Seismic Coefficient: (none)
Soil: 1
Description: Fill
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 2
Description: Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 3
Description: Weak Zone
Soil Model: Mohr-Coulomb
Unit Weight: 125
Cohesion: 0
Phi: 23.8
Piezometric Line #: 1
Soil: 4
Description: Rock Spoil
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 5
Description: Weathered Bedrock
Soil Model: Mohr-Coulomb
Unit Weight: 130
Cohesion: 0
Phi: 30
Piezometric Line #: 1
TH-GN-06
SI-12
TH-GN-08
SI-13
TH-GN-03
SI-09
9.8
9.6
9.4
9.2
9.0
0.0
0.2
0.4
0.6
0.8
1.0
Elevation (ft) (x 1000)
Distance (ft) (x 1000)

000405

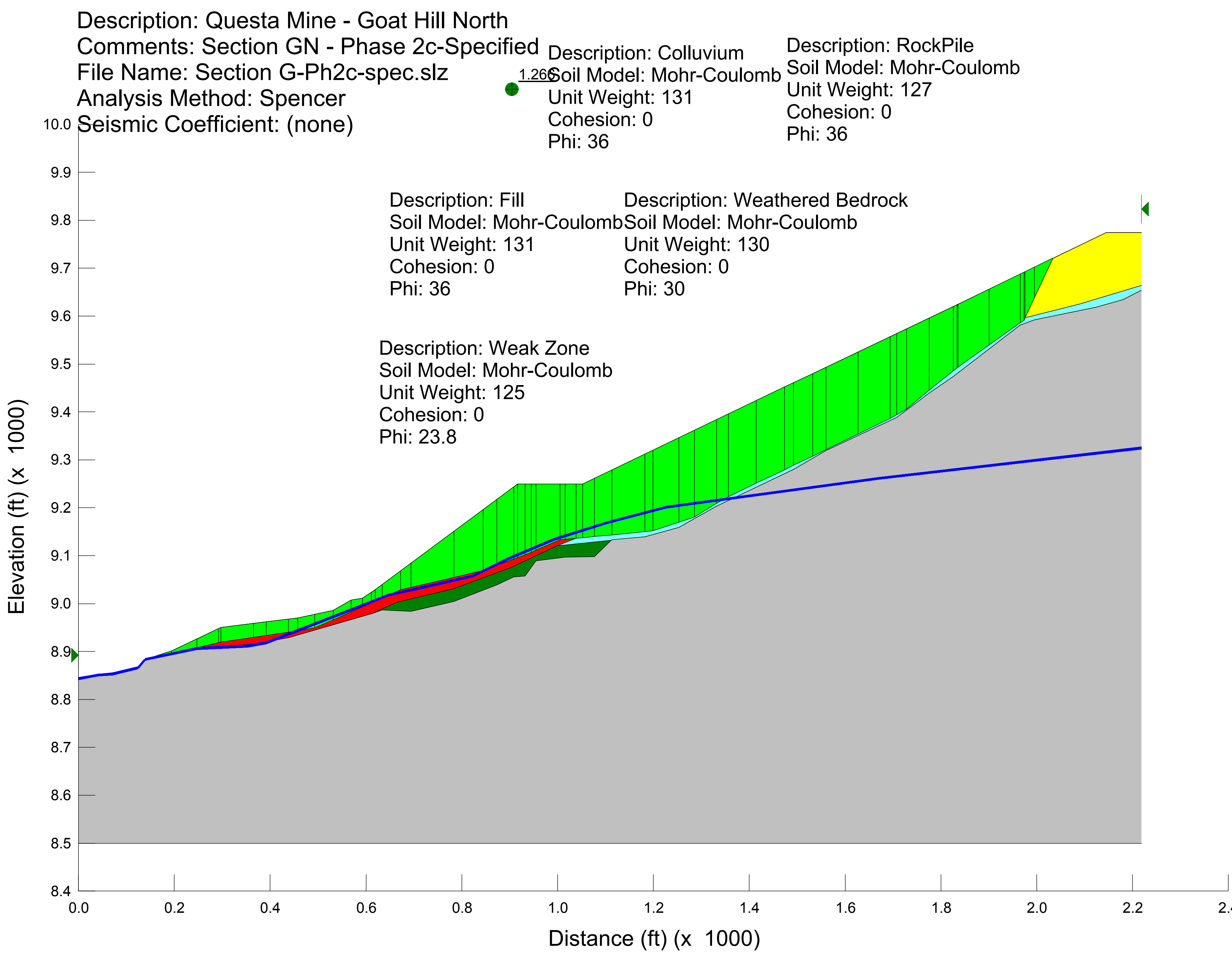


Description: Questa Mine - Goat Hill North
Comments: Section F - Phase 2c - Specified
File Name: SectionF-Phase2c.slz
Analysis Method: Spencer
Seismic Coefficient: (none)
1.591
Description: Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Description: RockPile
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Description: Fill
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Description: Weathered Bedrock
Soil Model: Mohr-Coulomb
Unit Weight: 130
Cohesion: 0
Phi: 30
Description: Weak Zone
Soil Model: Mohr-Coulomb
Unit Weight: 125
Cohesion: 0
Phi: 23.8
Elevation (ft) (x 1000)
10.2
10.1
10.0
9.9
9.8
9.7
9.6
9.5
9.4
9.3
9.2
9.1
9.0
8.9
0.0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0
Distance (ft) (x 1000)

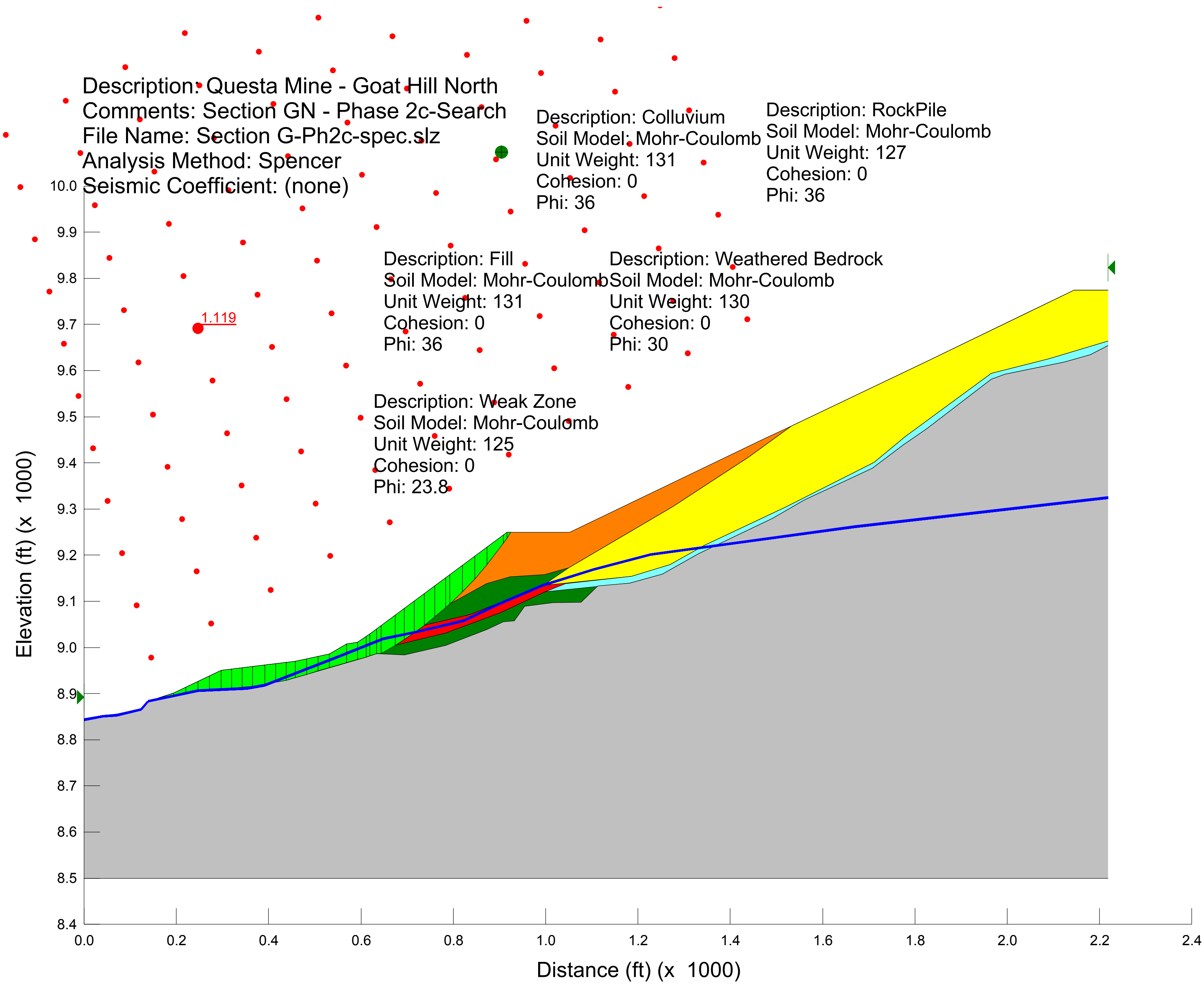
000406


1.532
Description: Questa Mine - Goat Hill North
Comments: Section F - Phase 2c - Search
File Name: SectionF-Phase2c.slz
Analysis Method: Spencer
Seismic Coefficient: (none)
Description: Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Description: RockPile
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Description: Fill
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Description: Weathered Bedrock
Soil Model: Mohr-Coulomb
Unit Weight: 130
Cohesion: 0
Phi: 30
Description: Weak Zone
Soil Model: Mohr-Coulomb
Unit Weight: 125
Cohesion: 0
Phi: 23.8
Elevation (ft) (x 1000)
10.2
10.1
10.0
9.9
9.8
9.7
9.6
9.5
9.4
9.3
9.2
9.1
9.0
8.9
0.0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0
Distance (ft) (x 1000)

000407

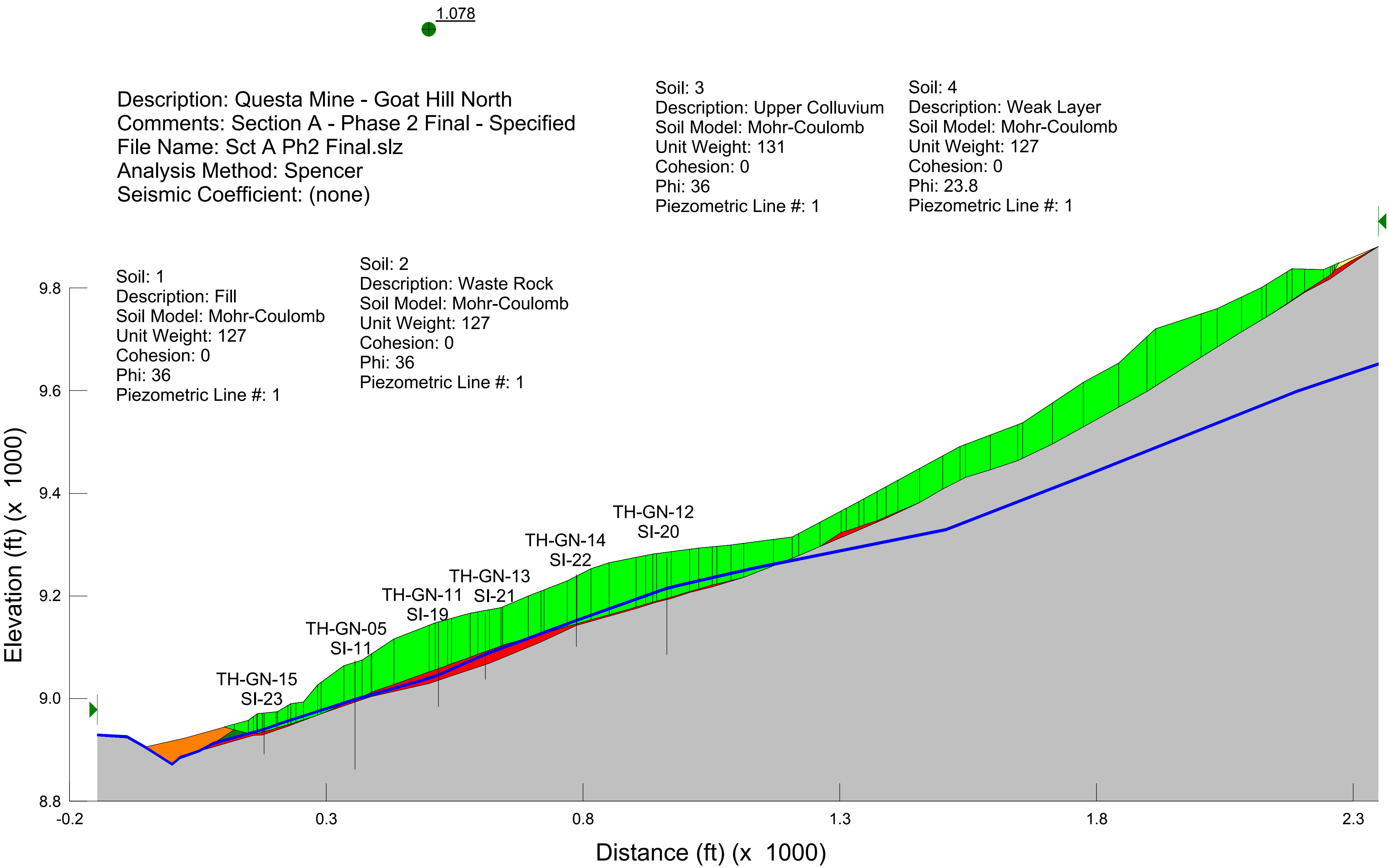

Description: Questa Mine - Goat Hill North
Comments: Section GN - Phase 2c-Specified
File Name: Section G-Ph2c-spec.slz
Analysis Method: Spencer
Seismic Coefficient: (none)
1.266
Description: Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Description: RockPile
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Description: Fill
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Description: Weathered Bedrock
Soil Model: Mohr-Coulomb
Unit Weight: 130
Cohesion: 0
Phi: 30
Description: Weak Zone
Soil Model: Mohr-Coulomb
Unit Weight: 125
Cohesion: 0
Phi: 23.8
Elevation (ft) (x 1000)
Distance (ft) (x 1000)
10.0
9.9
9.8
9.7
9.6
9.5
9.4
9.3
9.2
9.1
9.0
8.9
8.8
8.7
8.6
8.5
8.4
0.0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0
2.2
2.4

000408

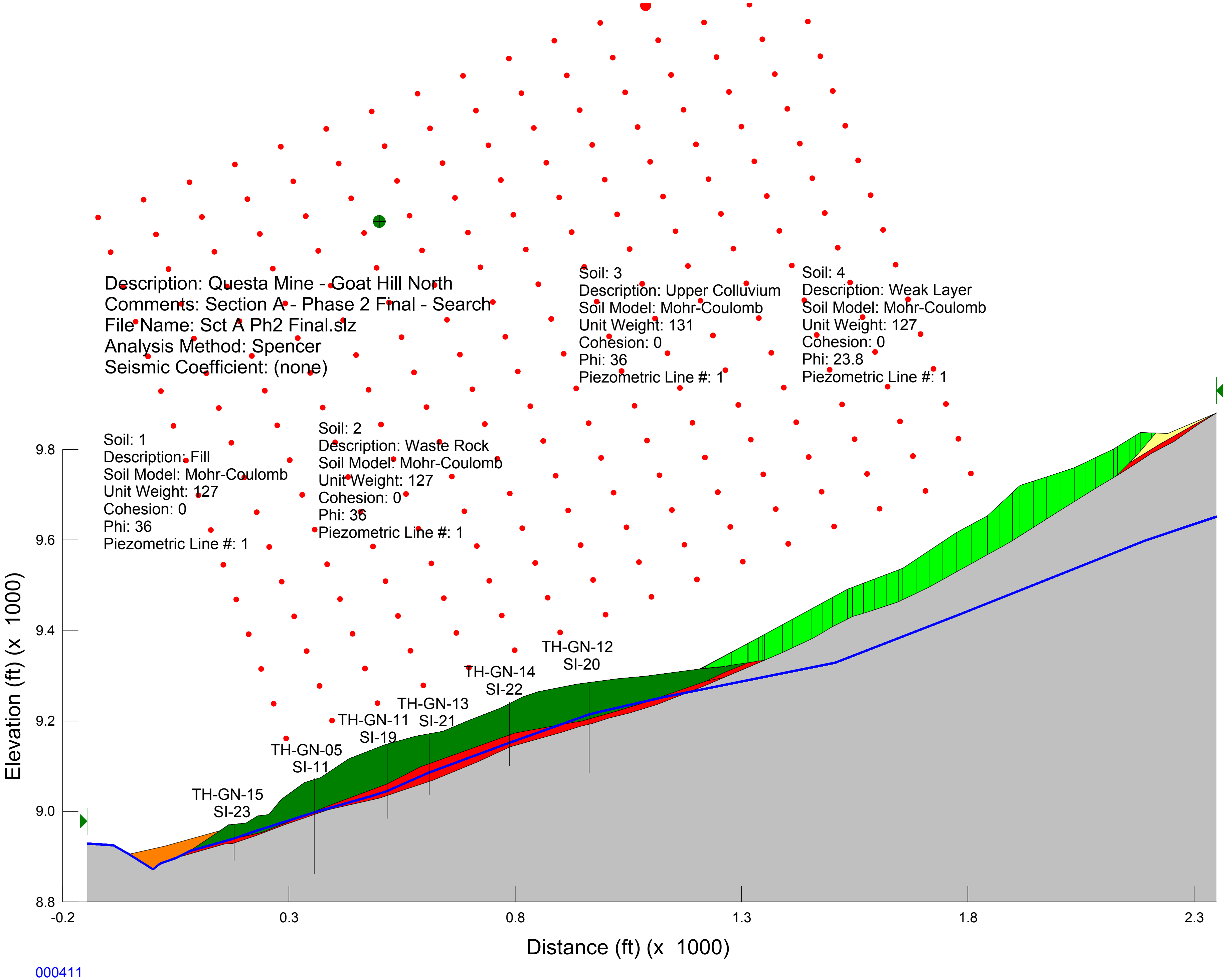


Description: Questa Mine - Goat Hill North
Comments: Section GN - Phase 2c-Search
File Name: Section G-Ph2c-spec.slz
Analysis Method: Spencer
Seismic Coefficient: (none)
Description: Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Description: RockPile
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Description: Fill
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Description: Weathered Bedrock
Soil Model: Mohr-Coulomb
Unit Weight: 130
Cohesion: 0
Phi: 30
Description: Weak Zone
Soil Model: Mohr-Coulomb
Unit Weight: 125
Cohesion: 0
Phi: 23.8
1.119
Elevation (ft) (x 1000)
10.0
9.9
9.8
9.7
9.6
9.5
9.4
9.3
9.2
9.1
9.0
8.9
8.8
8.7
8.6
8.5
8.4
0.0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0
2.2
2.4
Distance (ft) (x 1000)

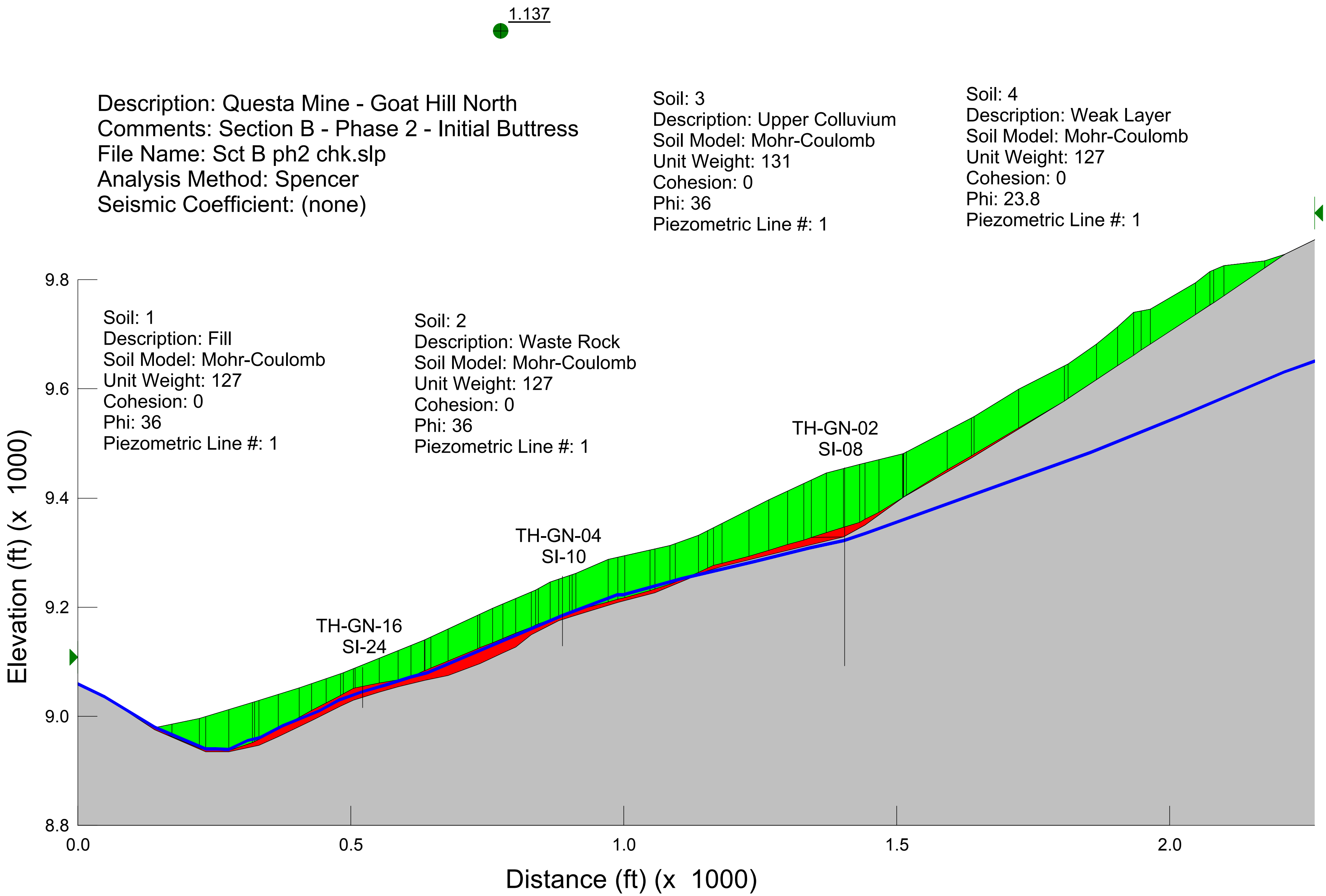
000409


1.078
Description: Questa Mine - Goat Hill North
Comments: Section A - Phase 2 Final - Specified
File Name: Sct A Ph2 Final.slz
Analysis Method: Spencer
Seismic Coefficient: (none)
Soil: 3
Description: Upper Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 4
Description: Weak Layer
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 23.8
Piezometric Line #: 1
Soil: 1
Description: Fill
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 2
Description: Waste Rock
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
TH-GN-12
SI-20
TH-GN-14
SI-22
TH-GN-13
SI-21
TH-GN-11
SI-19
TH-GN-05
SI-11
TH-GN-15
SI-23
9.8
9.6
9.4
9.2
9.0
8.8
-0.2
0.3
0.8
1.3
1.8
2.3
Elevation (ft) (x 1000)
Distance (ft) (x 1000)

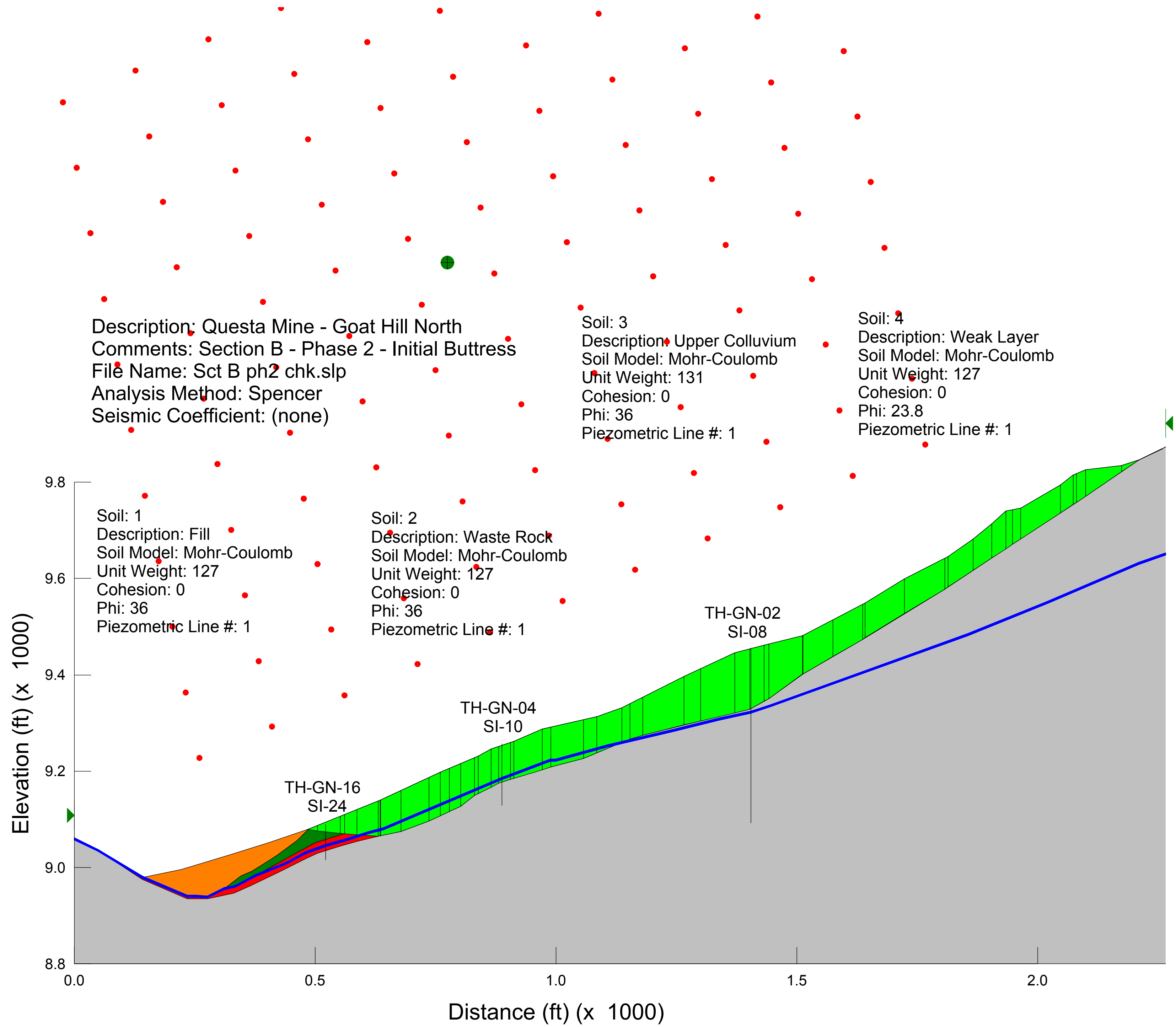
000410



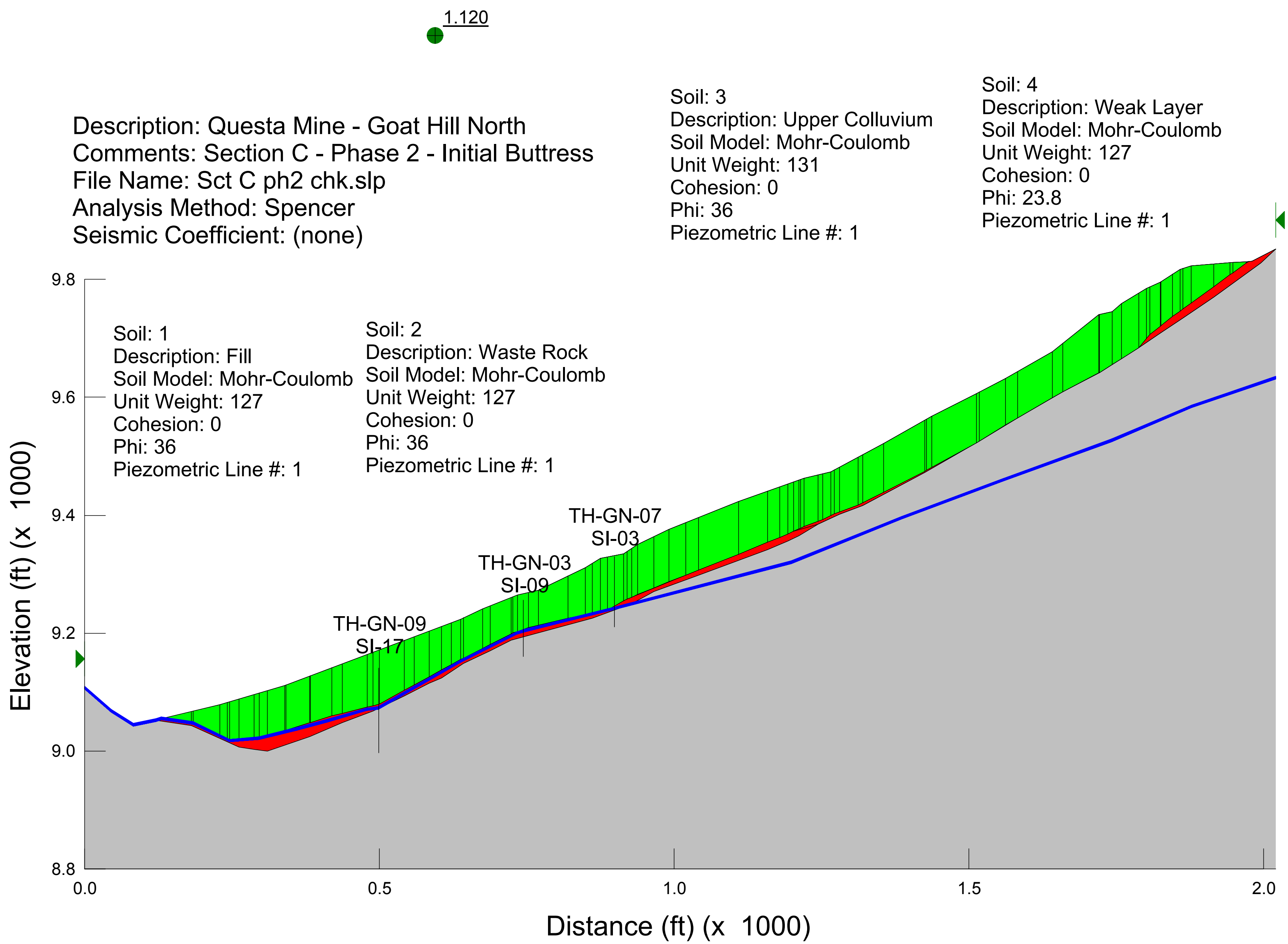
Description: Questa Mine - Goat Hill North
Comments: Section A - Phase 2 Final - Search
File Name: Sct A Ph2 Final.slz
Analysis Method: Spencer
Seismic Coefficient: (none)
Soil: 1
Description: Fill
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 2
Description: Waste Rock
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 3
Description: Upper Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 4
Description: Weak Layer
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 23.8
Piezometric Line #: 1
TH-GN-15
SI-23
TH-GN-05
SI-11
TH-GN-11
SI-19
TH-GN-13
SI-21
TH-GN-14
SI-22
TH-GN-12
SI-20
Elevation (ft) (x 1000)
9.8
9.6
9.4
9.2
9.0
8.8
-0.2
0.3
0.8
1.3
1.8
2.3
Distance (ft) (x 1000)
000411


1.137
Description: Questa Mine - Goat Hill North
Comments: Section B - Phase 2 - Initial Buttress
File Name: Sct B ph2 chk.slp
Analysis Method: Spencer
Seismic Coefficient: (none)
Soil: 3
Description: Upper Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 4
Description: Weak Layer
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 23.8
Piezometric Line #: 1
Soil: 1
Description: Fill
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 2
Description: Waste Rock
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
TH-GN-02
SI-08
TH-GN-04
SI-10
TH-GN-16
SI-24
Elevation (ft) (x 1000)
9.8
9.6
9.4
9.2
9.0
8.8
0.0
0.5
1.0
1.5
2.0
Distance (ft) (x 1000)

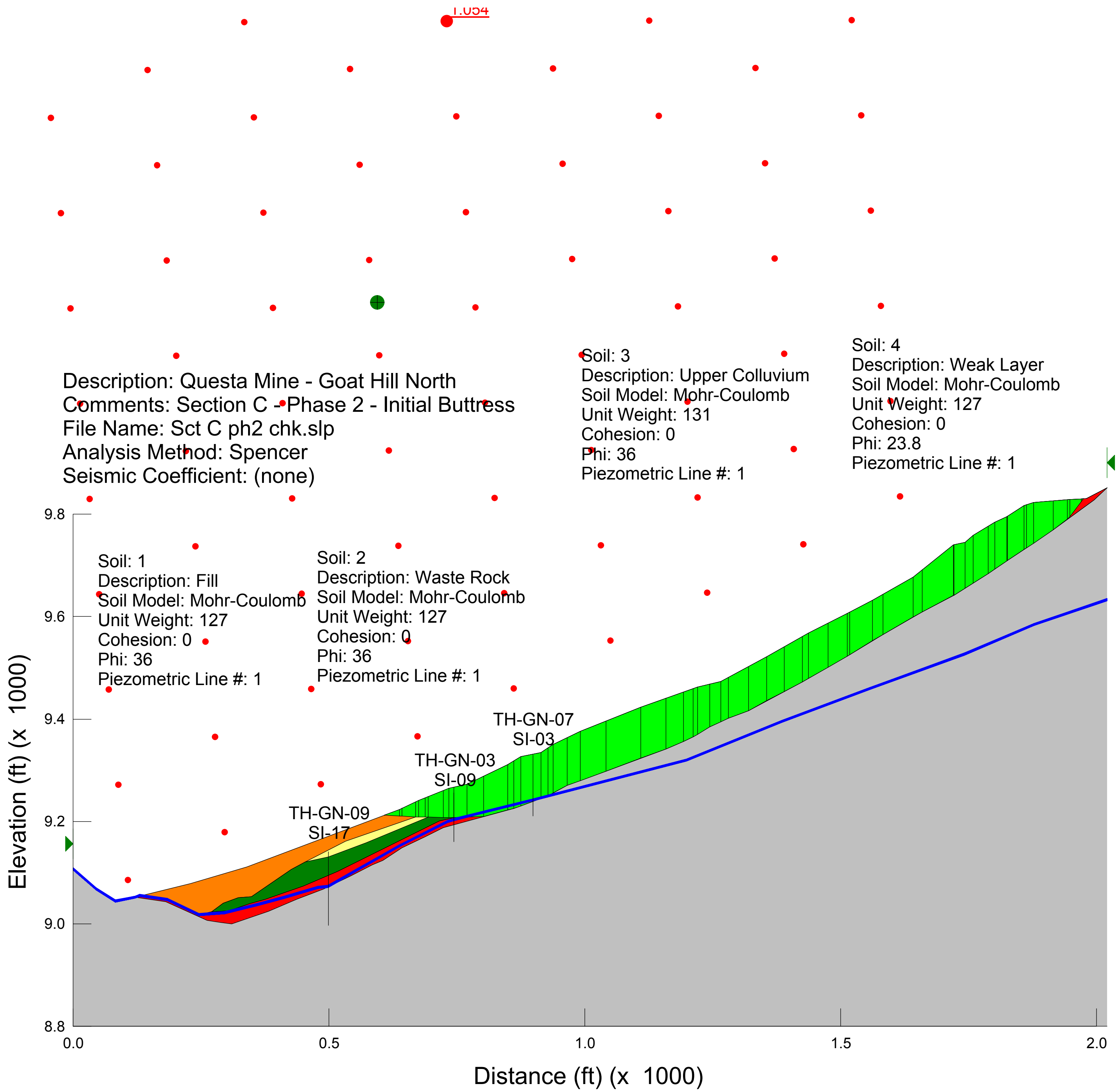
000412


Description: Questa Mine - Goat Hill North
Comments: Section B - Phase 2 - Initial Buttress
File Name: Sct B ph2 chk.slp
Analysis Method: Spencer
Seismic Coefficient: (none)
Soil: 3
Description: Upper Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 4
Description: Weak Layer
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 23.8
Piezometric Line #: 1
Soil: 1
Description: Fill
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 2
Description: Waste Rock
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
TH-GN-02
SI-08
TH-GN-04
SI-10
TH-GN-16
SI-24
Elevation (ft) (x 1000)
9.8
9.6
9.4
9.2
9.0
8.8
0.0
0.5
1.0
1.5
2.0
Distance (ft) (x 1000)

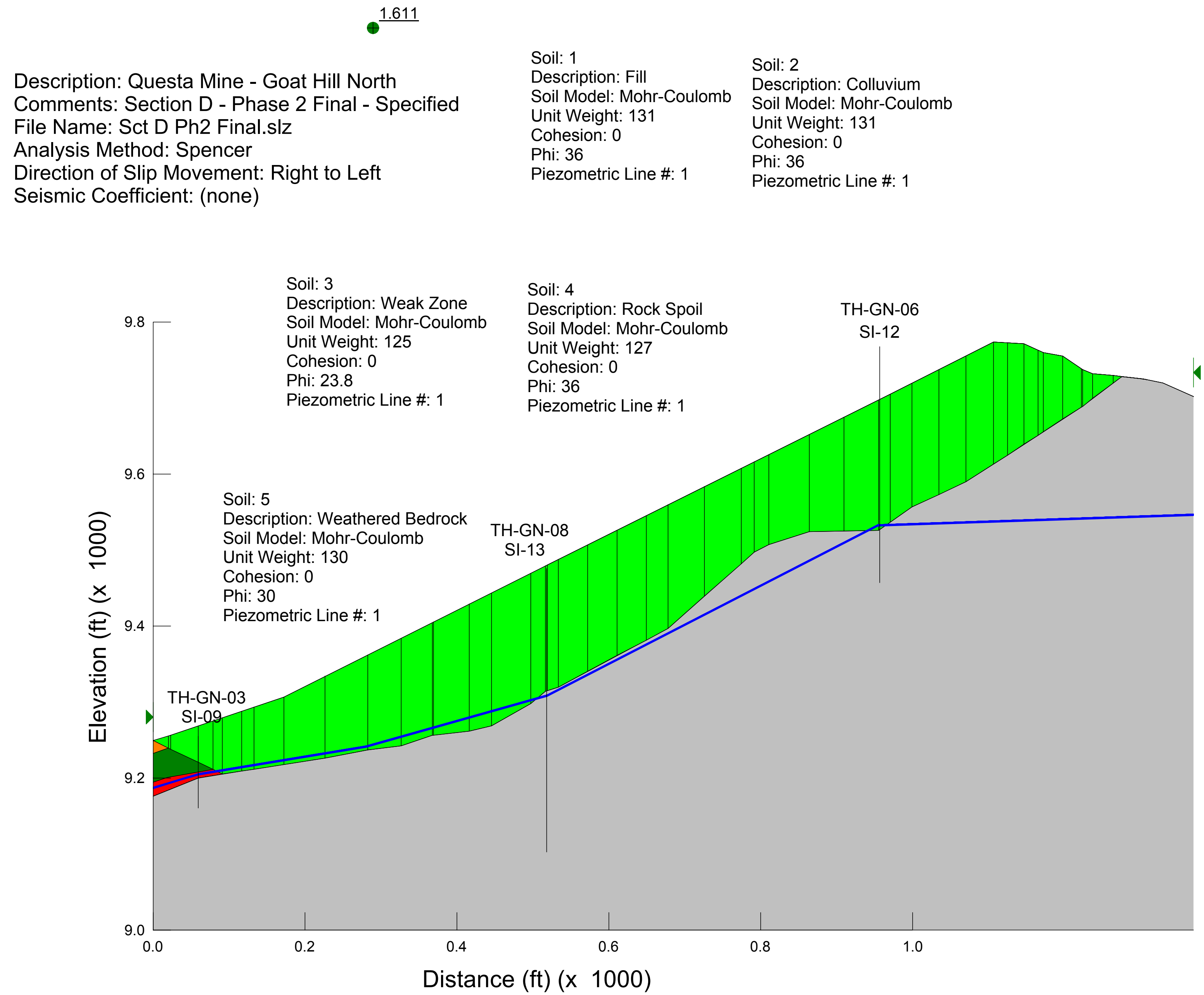
000413

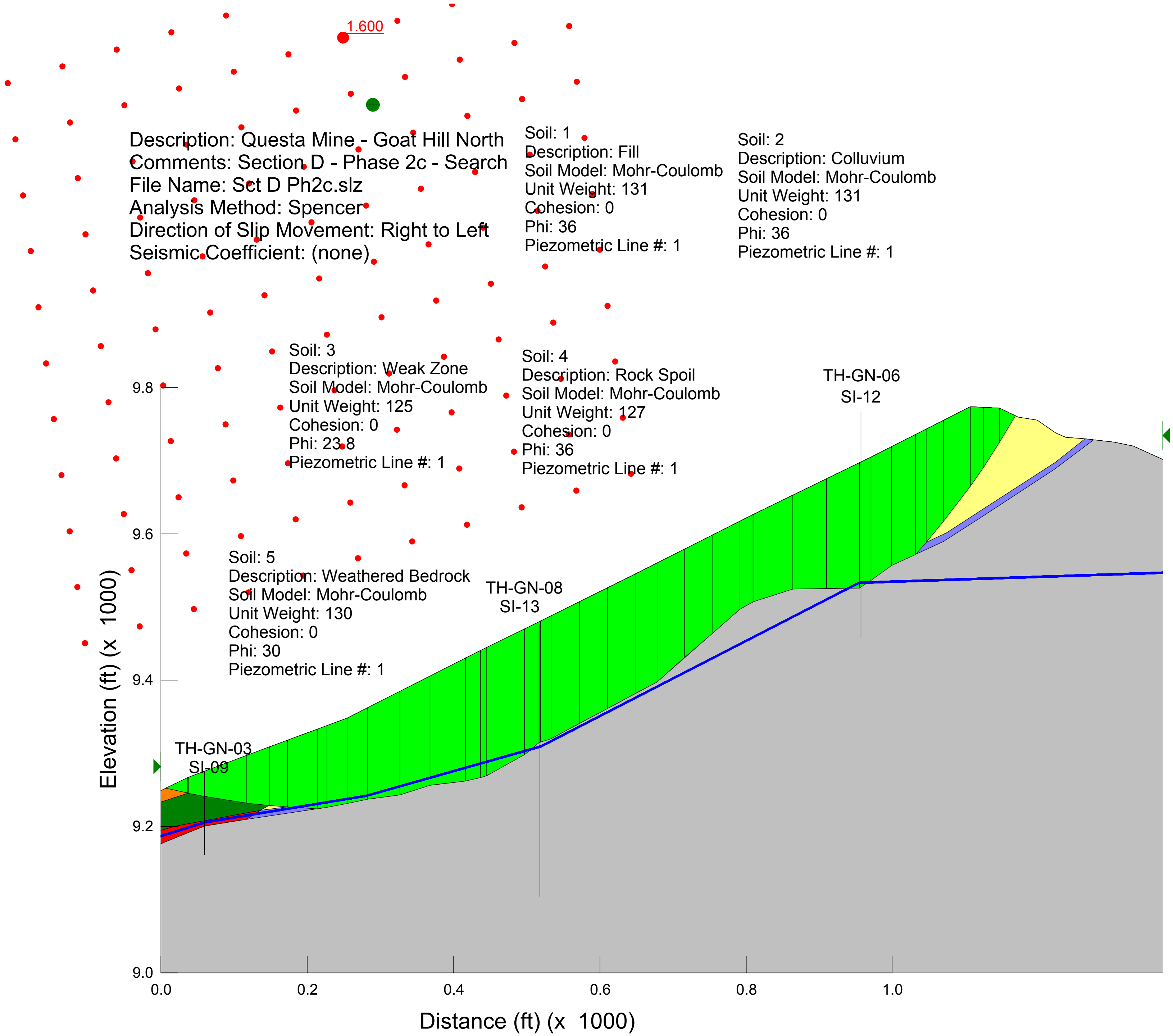

1.120
Description: Questa Mine - Goat Hill North
Comments: Section C - Phase 2 - Initial Buttress
File Name: Sct C ph2 chk.slp
Analysis Method: Spencer
Seismic Coefficient: (none)
Soil: 3
Description: Upper Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 4
Description: Weak Layer
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 23.8
Piezometric Line #: 1
Soil: 1
Description: Fill
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 2
Description: Waste Rock
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
TH-GN-07
SI-03
TH-GN-03
SI-09
TH-GN-09
SI-17
9.8
9.6
9.4
9.2
9.0
8.8
Elevation (ft) (x 1000)
0.0
0.5
1.0
1.5
2.0
Distance (ft) (x 1000)

000414


1.054
Description: Questa Mine - Goat Hill North
Comments: Section C - Phase 2 - Initial Buttress
File Name: Sct C ph2 chk.slp
Analysis Method: Spencer
Seismic Coefficient: (none)
Soil: 3
Description: Upper Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 4
Description: Weak Layer
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 23.8
Piezometric Line #: 1
Soil: 1
Description: Fill
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 2
Description: Waste Rock
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
TH-GN-07
SI-03
TH-GN-03
SI-09
TH-GN-09
SI-17
Elevation (ft) (x 1000)
9.8
9.6
9.4
9.2
9.0
8.8
0.0
0.5
1.0
1.5
2.0
Distance (ft) (x 1000)

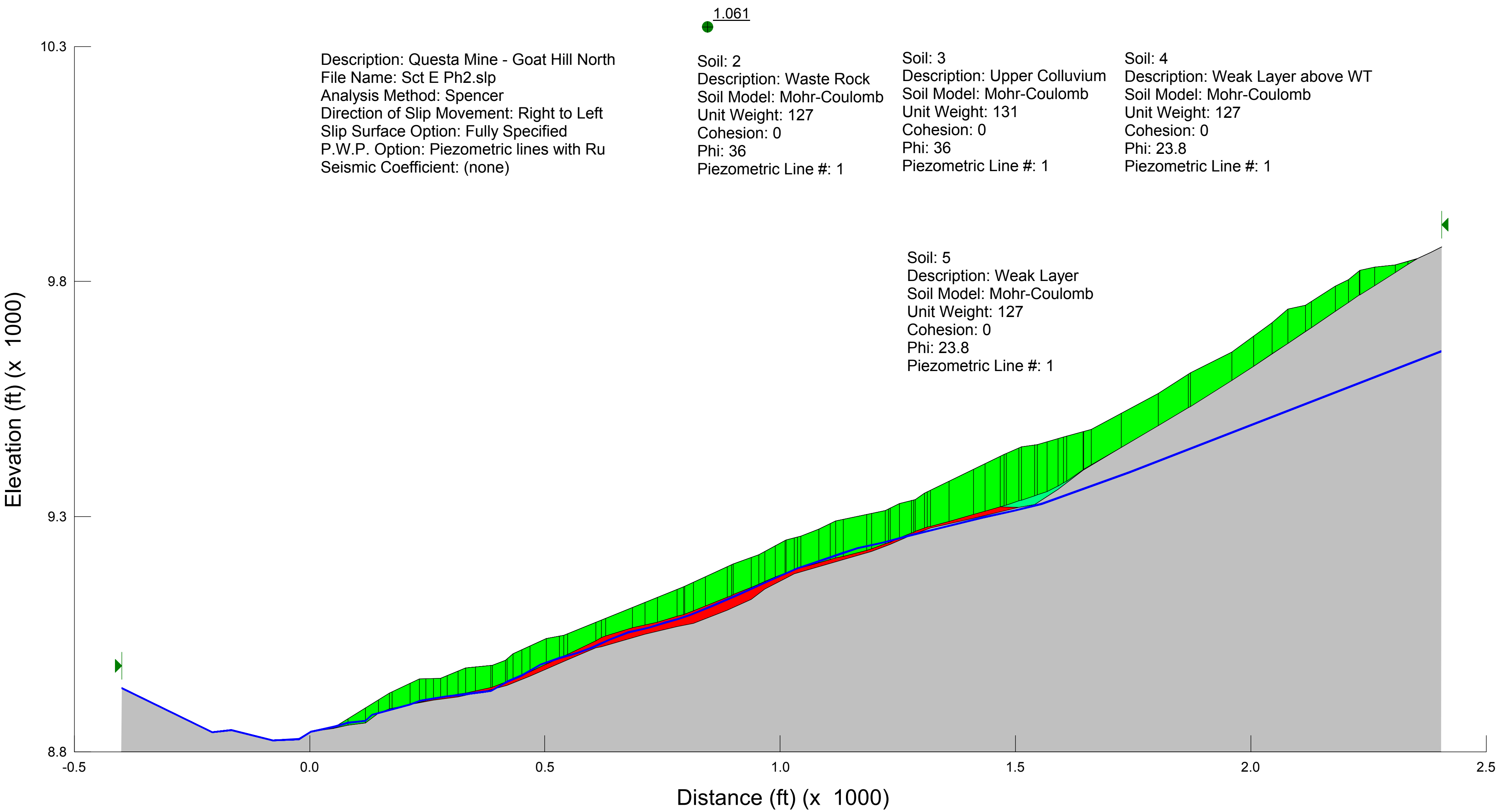
000415


1.611
Description: Questa Mine - Goat Hill North
Comments: Section D - Phase 2 Final - Specified
File Name: Sct D Ph2 Final.slz
Analysis Method: Spencer
Direction of Slip Movement: Right to Left
Seismic Coefficient: (none)
Soil: 1
Description: Fill
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 2
Description: Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 3
Description: Weak Zone
Soil Model: Mohr-Coulomb
Unit Weight: 125
Cohesion: 0
Phi: 23.8
Piezometric Line #: 1
Soil: 4
Description: Rock Spoil
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 5
Description: Weathered Bedrock
Soil Model: Mohr-Coulomb
Unit Weight: 130
Cohesion: 0
Phi: 30
Piezometric Line #: 1
TH-GN-06
SI-12
TH-GN-08
SI-13
TH-GN-03
SI-09
9.8
9.6
9.4
9.2
9.0
Elevation (ft) (x 1000)
0.0
0.2
0.4
0.6
0.8
1.0
Distance (ft) (x 1000)



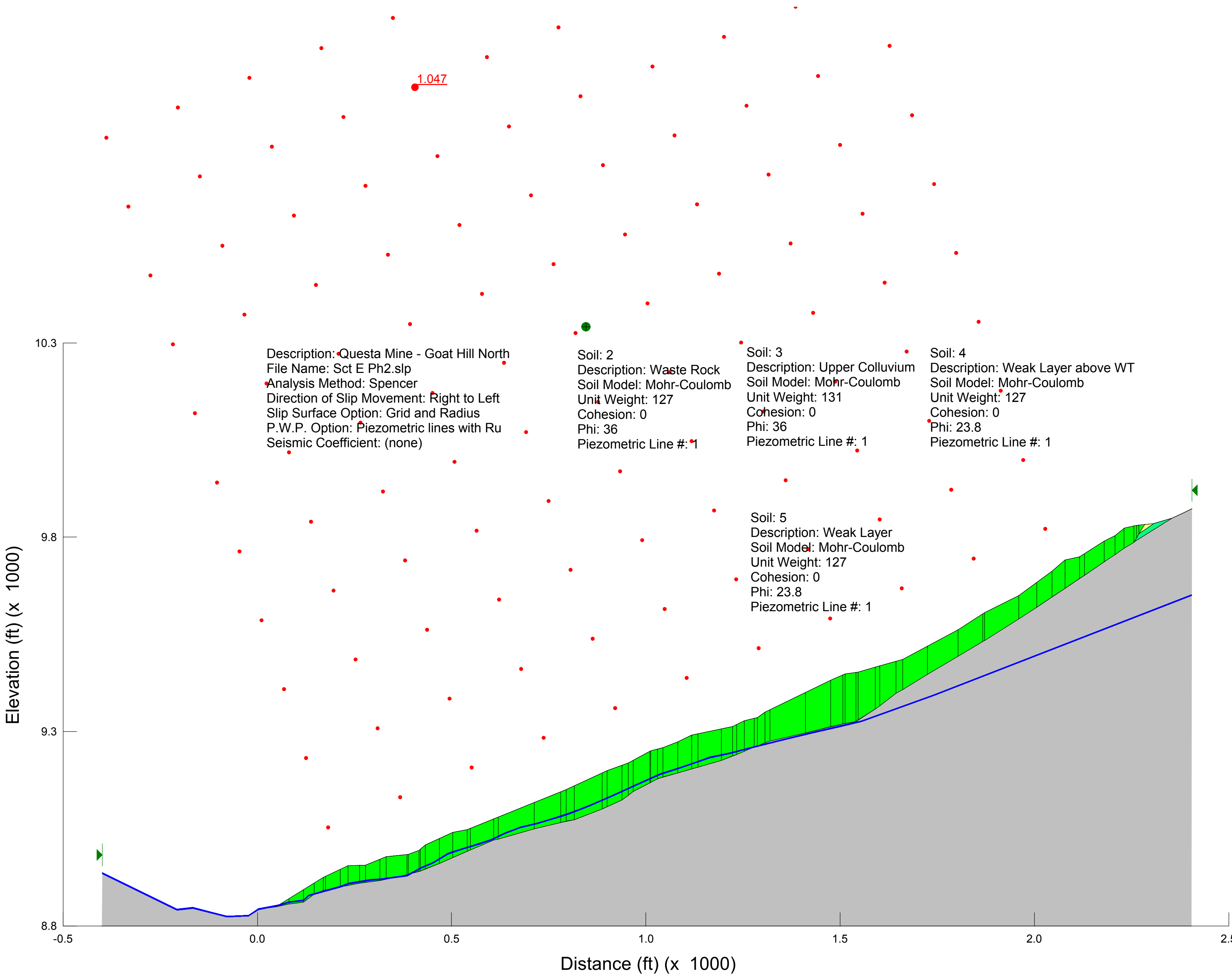
1.600
Description: Questa Mine - Goat Hill North
Comments: Section D - Phase 2c - Search
File Name: Sct D Ph2c.slz
Analysis Method: Spencer
Direction of Slip Movement: Right to Left
Seismic Coefficient: (none)
Soil: 1
Description: Fill
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 2
Description: Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 3
Description: Weak Zone
Soil Model: Mohr-Coulomb
Unit Weight: 125
Cohesion: 0
Phi: 23.8
Piezometric Line #: 1
Soil: 4
Description: Rock Spoil
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 5
Description: Weathered Bedrock
Soil Model: Mohr-Coulomb
Unit Weight: 130
Cohesion: 0
Phi: 30
Piezometric Line #: 1
TH-GN-06
SI-12
TH-GN-08
SI-13
TH-GN-03
SI-09
9.8
9.6
9.4
9.2
9.0
Elevation (ft) (x 1000)
0.0
0.2
0.4
0.6
0.8
1.0
Distance (ft) (x 1000)

000417

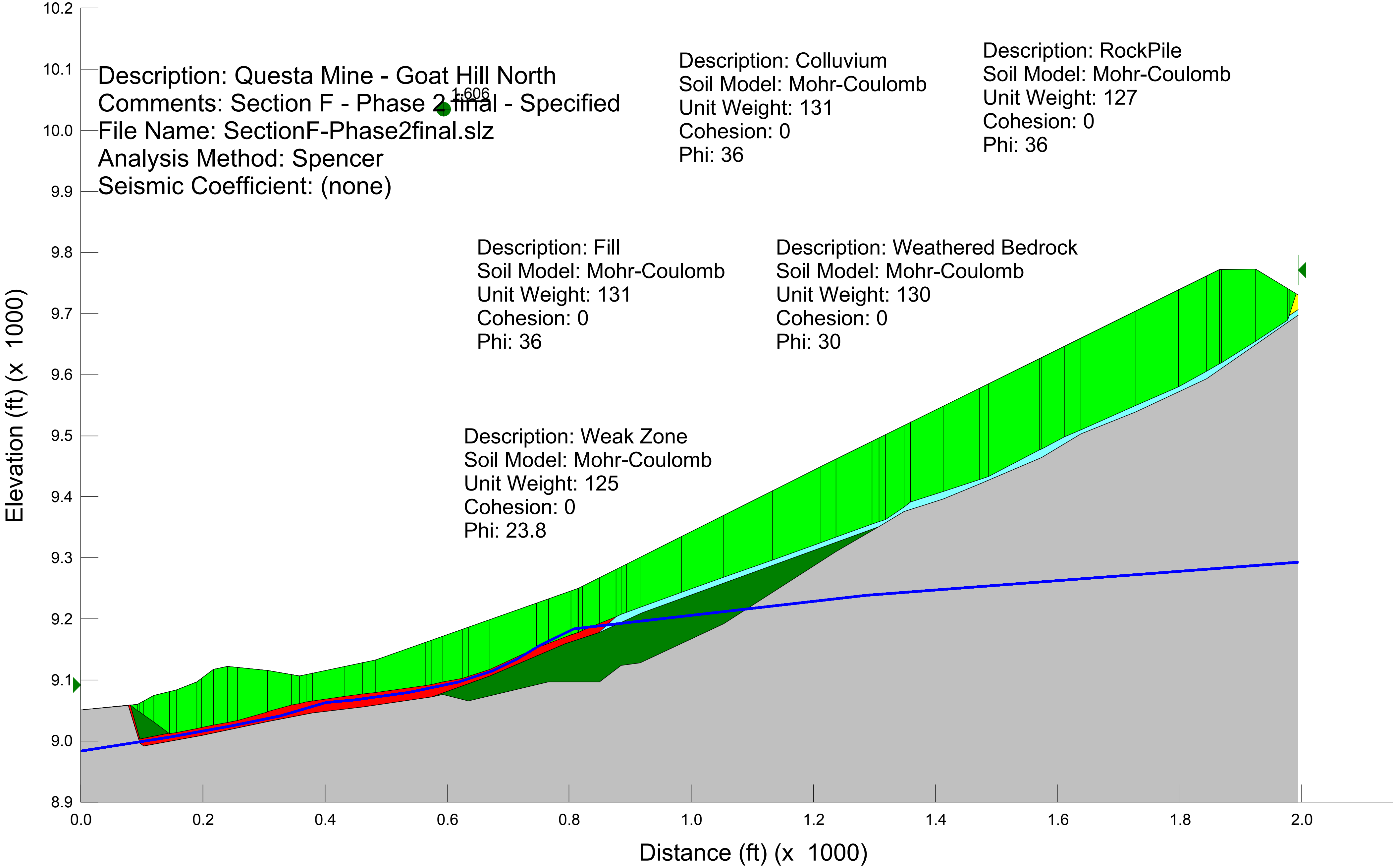


1.061
Description: Questa Mine - Goat Hill North
File Name: Sct E Ph2.slp
Analysis Method: Spencer
Direction of Slip Movement: Right to Left
Slip Surface Option: Fully Specified
P.W.P. Option: Piezometric lines with Ru
Seismic Coefficient: (none)
Soil: 2
Description: Waste Rock
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 3
Description: Upper Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 4
Description: Weak Layer above WT
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 23.8
Piezometric Line #: 1
Soil: 5
Description: Weak Layer
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 23.8
Piezometric Line #: 1
10.3
9.8
9.3
8.8
Elevation (ft) (x 1000)
-0.5
0.0
0.5
1.0
1.5
2.0
2.5
Distance (ft) (x 1000)

000418

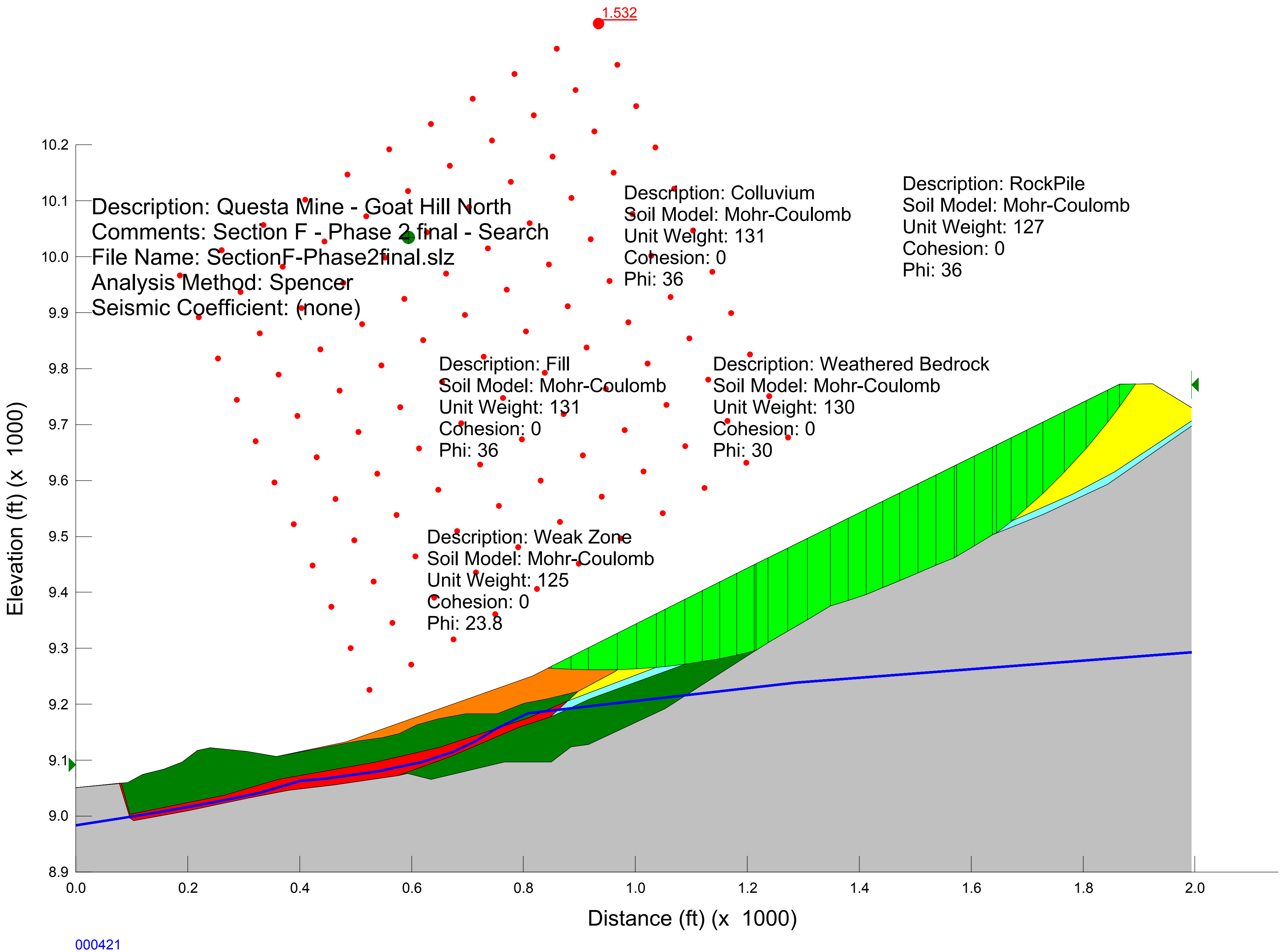

1.047
Description: Questa Mine - Goat Hill North
File Name: Sct E Ph2.slp
Analysis Method: Spencer
Direction of Slip Movement: Right to Left
Slip Surface Option: Grid and Radius
P.W.P. Option: Piezometric lines with Ru
Seismic Coefficient: (none)
Soil: 2
Description: Waste Rock
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 3
Description: Upper Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 4
Description: Weak Layer above WT
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 23.8
Piezometric Line #: 1
Soil: 5
Description: Weak Layer
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 23.8
Piezometric Line #: 1
10.3
9.8
9.3
8.8
Elevation (ft) (x 1000)
-0.5
0.0
0.5
1.0
1.5
2.0
2.5
Distance (ft) (x 1000)

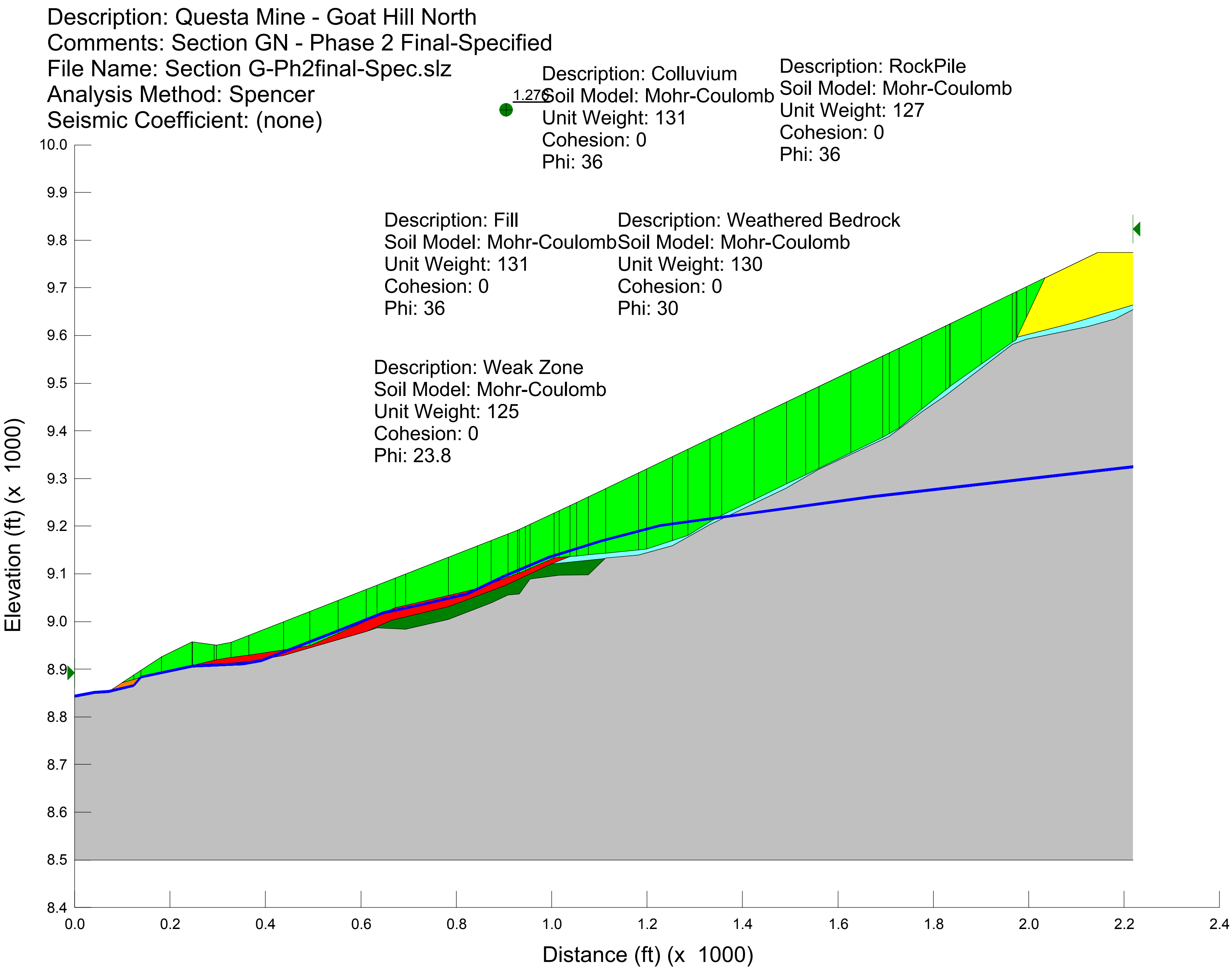
000419



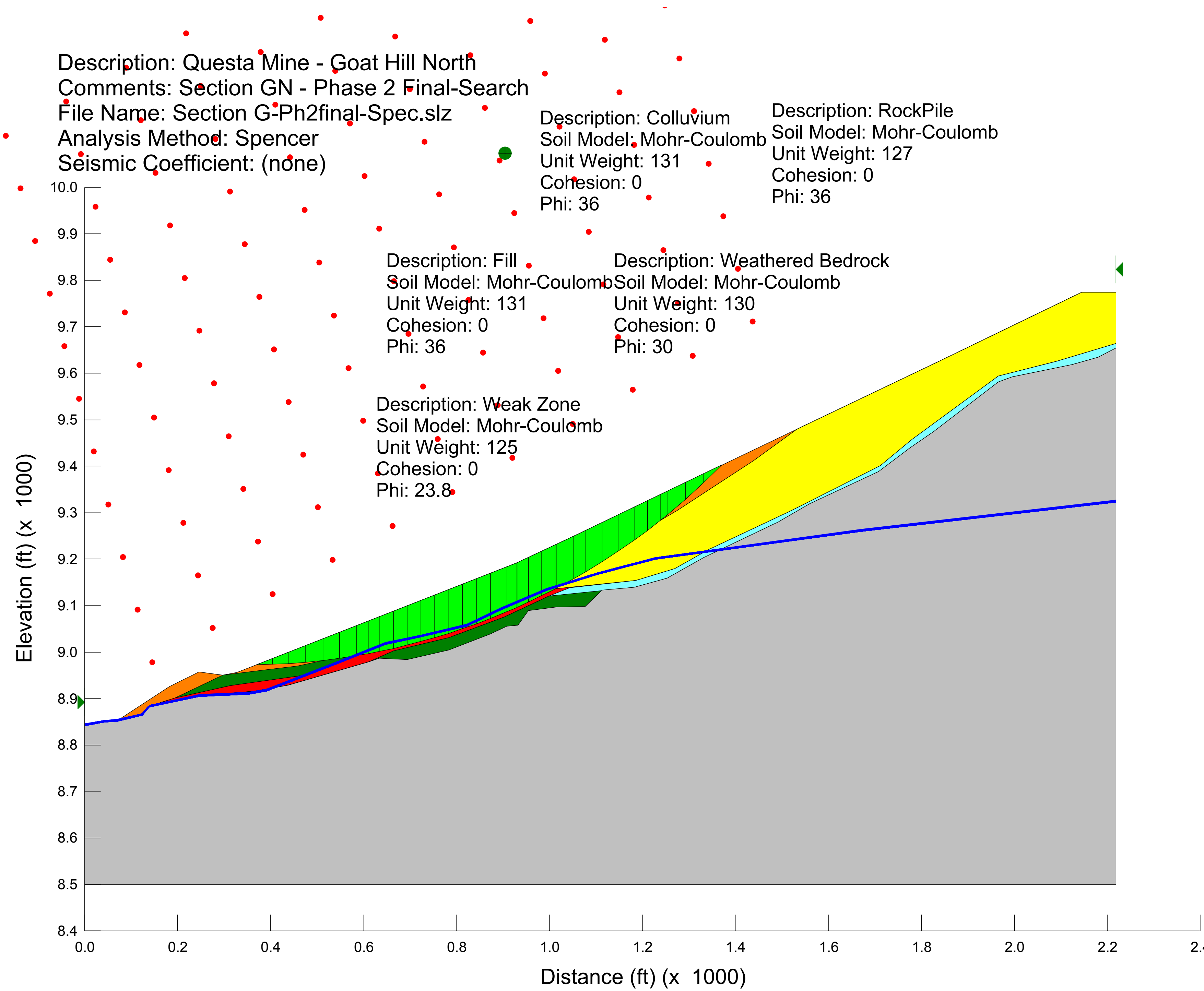
Description: Questa Mine - Goat Hill North
Comments: Section F - Phase 2 final - Specified
File Name: SectionF-Phase2final.slz
Analysis Method: Spencer
Seismic Coefficient: (none)
1.606
Description: Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Description: RockPile
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Description: Fill
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Description: Weathered Bedrock
Soil Model: Mohr-Coulomb
Unit Weight: 130
Cohesion: 0
Phi: 30
Description: Weak Zone
Soil Model: Mohr-Coulomb
Unit Weight: 125
Cohesion: 0
Phi: 23.8
Elevation (ft) (x 1000)
10.2
10.1
10.0
9.9
9.8
9.7
9.6
9.5
9.4
9.3
9.2
9.1
9.0
8.9
0.0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0
Distance (ft) (x 1000)

000420


1.532
Description: Questa Mine - Goat Hill North
Comments: Section F - Phase 2 final - Search
File Name: SectionF-Phase2final.slz
Analysis Method: Spencer
Seismic Coefficient: (none)
Description: Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Description: RockPile
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Description: Fill
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Description: Weathered Bedrock
Soil Model: Mohr-Coulomb
Unit Weight: 130
Cohesion: 0
Phi: 30
Description: Weak Zone
Soil Model: Mohr-Coulomb
Unit Weight: 125
Cohesion: 0
Phi: 23.8
Elevation (ft) (x 1000)
10.2
10.1
10.0
9.9
9.8
9.7
9.6
9.5
9.4
9.3
9.2
9.1
9.0
8.9
0.0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0
Distance (ft) (x 1000)
000421


Description: Questa Mine - Goat Hill North
Comments: Section GN - Phase 2 Final-Specified
File Name: Section G-Ph2final-Spec.slz
Analysis Method: Spencer
Seismic Coefficient: (none)
1.276
Description: Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Description: RockPile
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Description: Fill
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Description: Weathered Bedrock
Soil Model: Mohr-Coulomb
Unit Weight: 130
Cohesion: 0
Phi: 30
Description: Weak Zone
Soil Model: Mohr-Coulomb
Unit Weight: 125
Cohesion: 0
Phi: 23.8
Elevation (ft) (x 1000)
10.0
9.9
9.8
9.7
9.6
9.5
9.4
9.3
9.2
9.1
9.0
8.9
8.8
8.7
8.6
8.5
8.4
0.0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0
2.2
2.4
Distance (ft) (x 1000)

000422

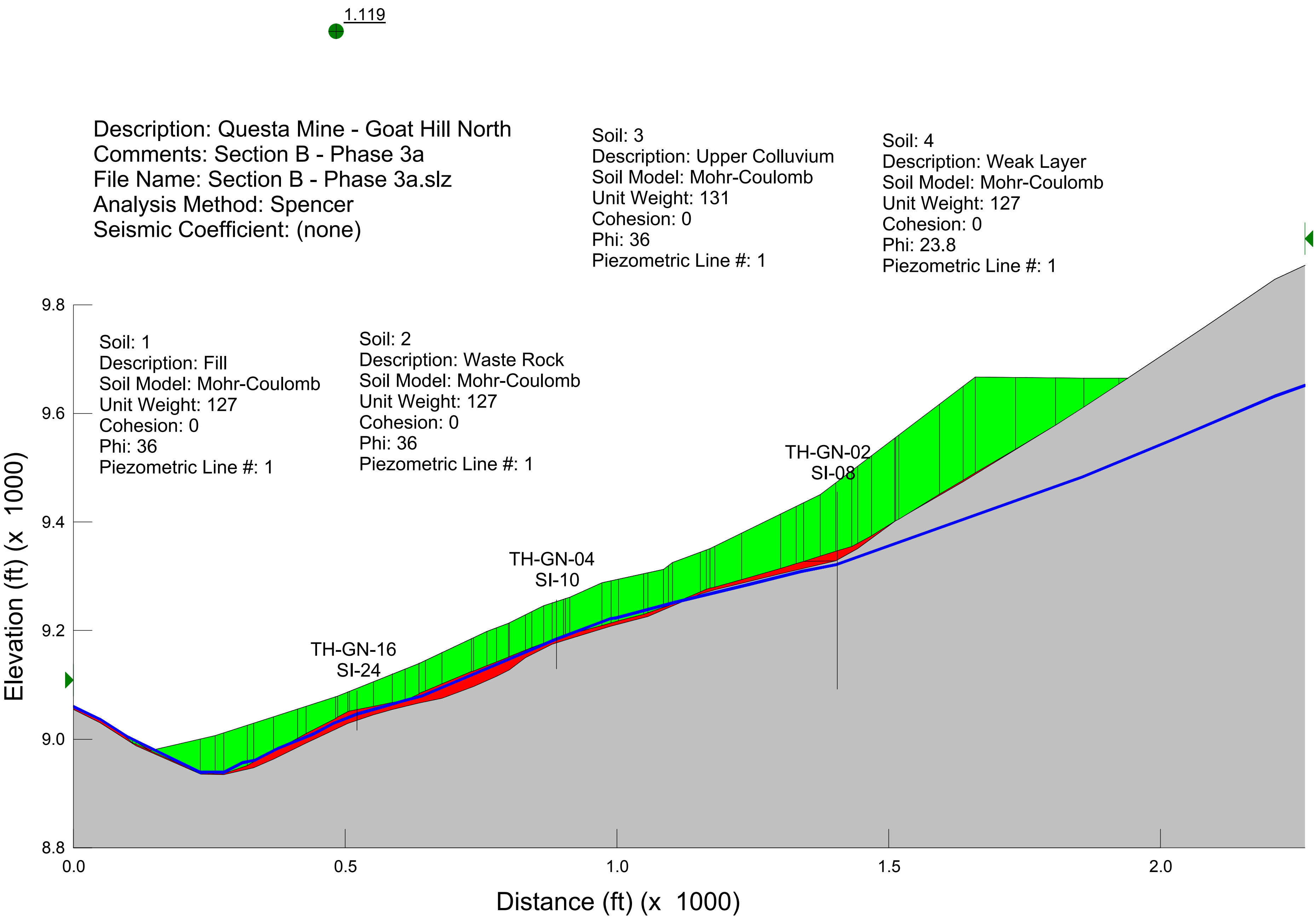


Description: Questa Mine - Goat Hill North
Comments: Section GN - Phase 2 Final-Search
File Name: Section G-Ph2final-Spec.slz
Analysis Method: Spencer
Seismic Coefficient: (none)
Description: Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Description: RockPile
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Description: Fill
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Description: Weathered Bedrock
Soil Model: Mohr-Coulomb
Unit Weight: 130
Cohesion: 0
Phi: 30
Description: Weak Zone
Soil Model: Mohr-Coulomb
Unit Weight: 125
Cohesion: 0
Phi: 23.8
Elevation (ft) (x 1000)
10.0
9.9
9.8
9.7
9.6
9.5
9.4
9.3
9.2
9.1
9.0
8.9
8.8
8.7
8.6
8.5
8.4
0.0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0
2.2
2.4
Distance (ft) (x 1000)

000423

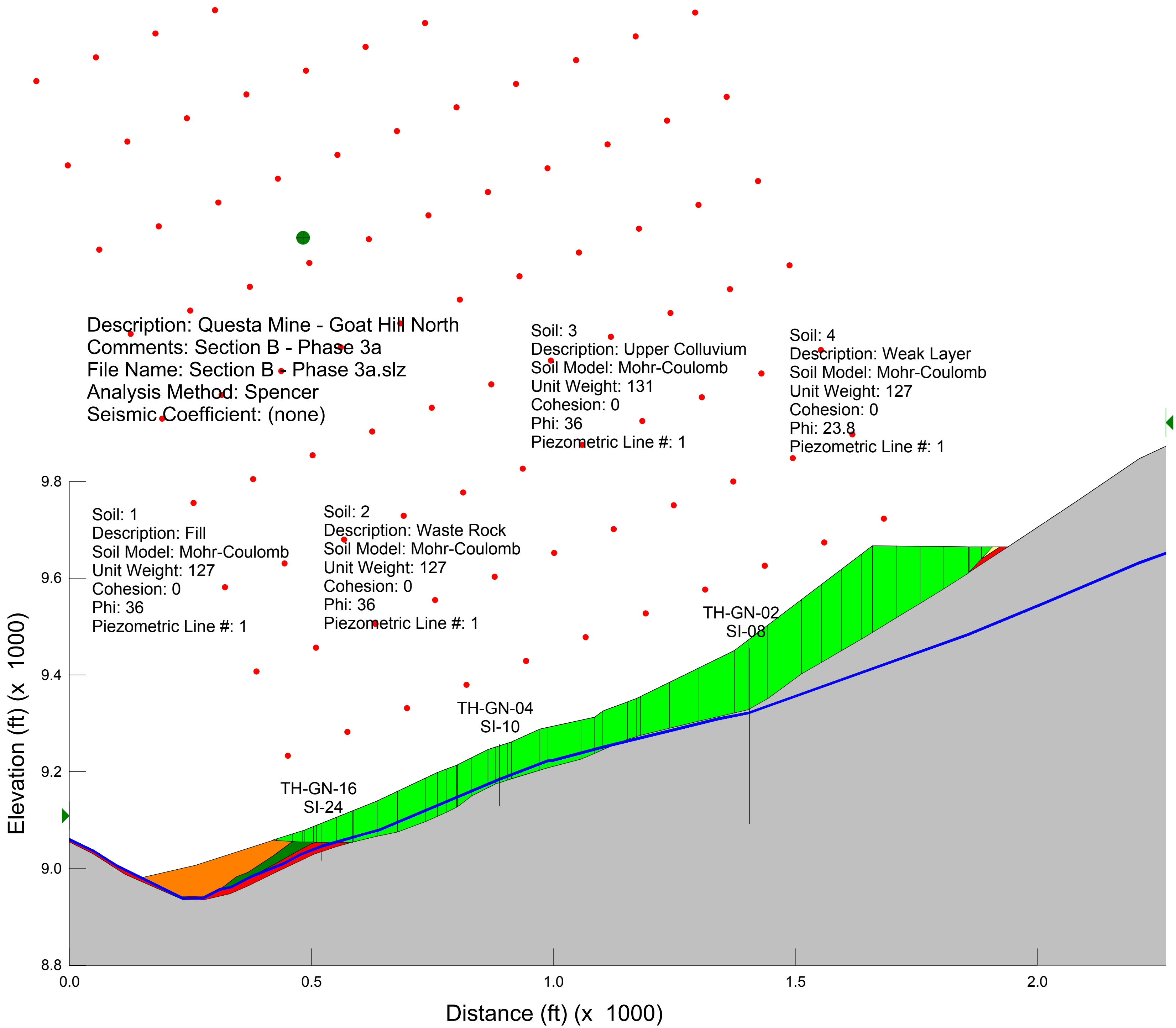
2D Stability Results

Construction Phase 3

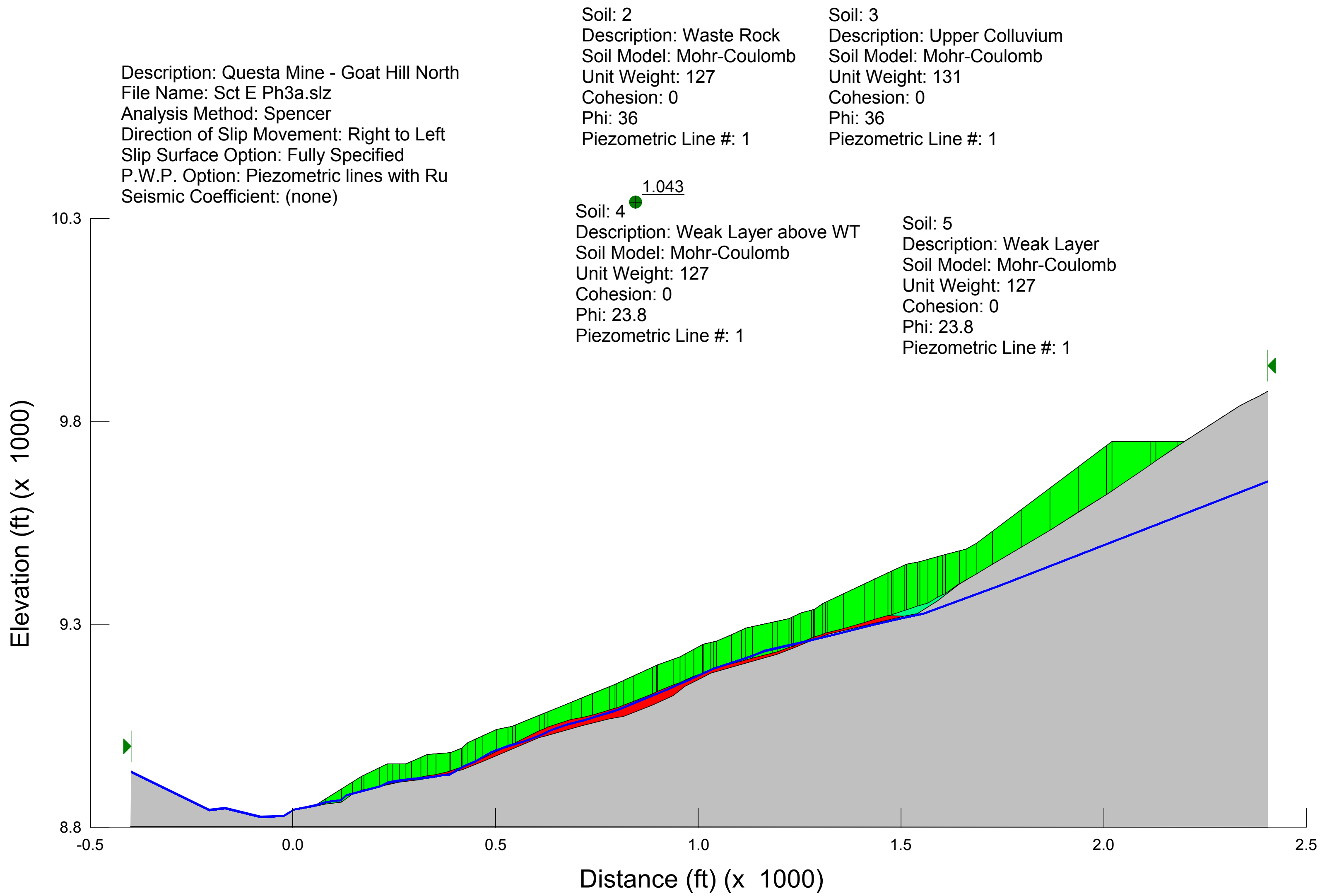


1.119
Description: Questa Mine - Goat Hill North
Comments: Section B - Phase 3a
File Name: Section B - Phase 3a.slz
Analysis Method: Spencer
Seismic Coefficient: (none)
Soil: 3
Description: Upper Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 4
Description: Weak Layer
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 23.8
Piezometric Line #: 1
Soil: 1
Description: Fill
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 2
Description: Waste Rock
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
TH-GN-02
SI-08
TH-GN-04
SI-10
TH-GN-16
SI-24
9.8
9.6
9.4
9.2
9.0
8.8
Elevation (ft) (x 1000)
0.0
0.5
1.0
1.5
2.0
Distance (ft) (x 1000)

000425

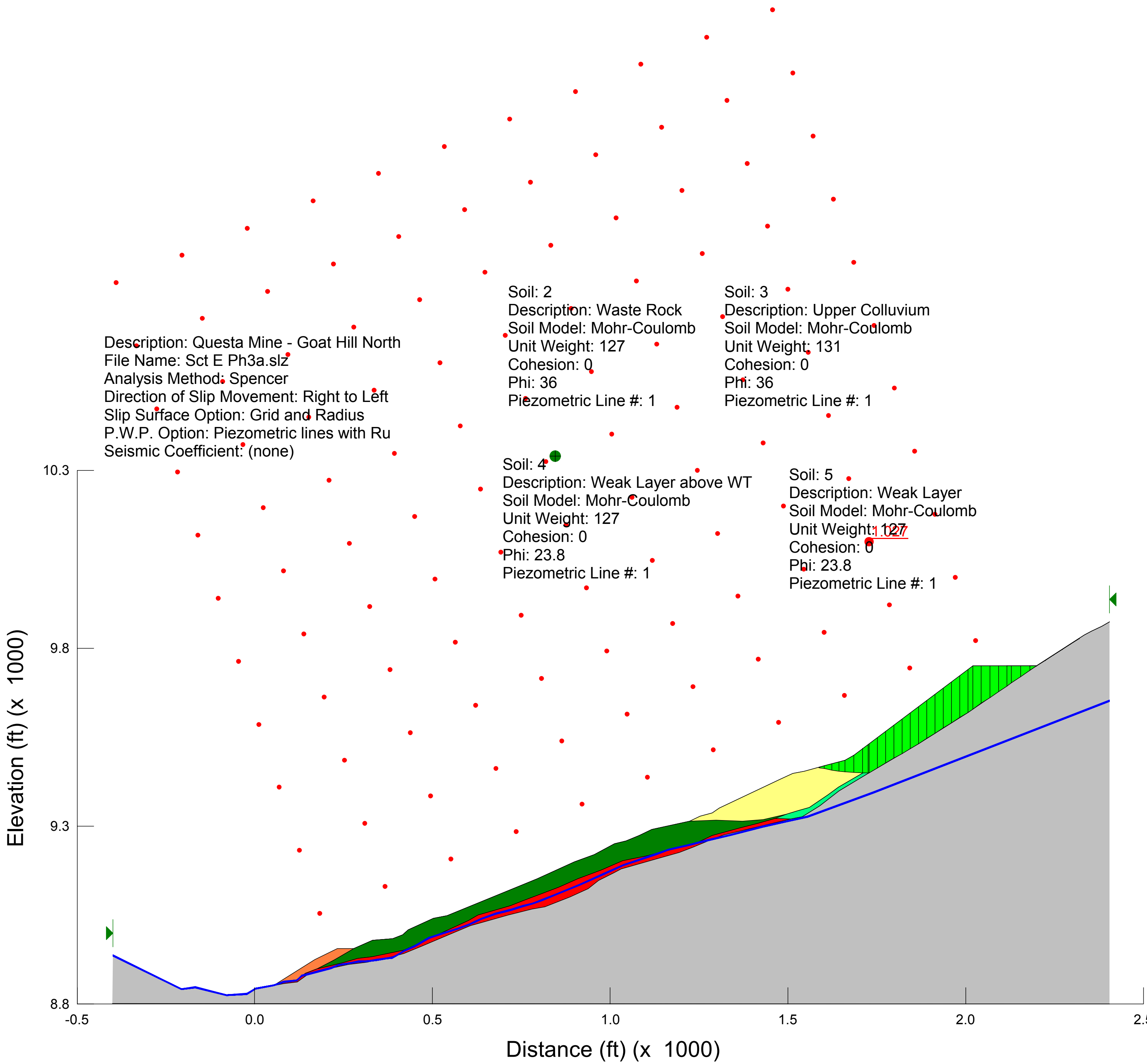


Description: Questa Mine - Goat Hill North
Comments: Section B - Phase 3a
File Name: Section B - Phase 3a.slz
Analysis Method: Spencer
Seismic Coefficient: (none)
Soil: 3
Description: Upper Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 4
Description: Weak Layer
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 23.8
Piezometric Line #: 1
Soil: 1
Description: Fill
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 2
Description: Waste Rock
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
TH-GN-02
SI-08
TH-GN-04
SI-10
TH-GN-16
SI-24
Elevation (ft) (x 1000)
9.8
9.6
9.4
9.2
9.0
8.8
0.0
0.5
1.0
1.5
2.0
Distance (ft) (x 1000)

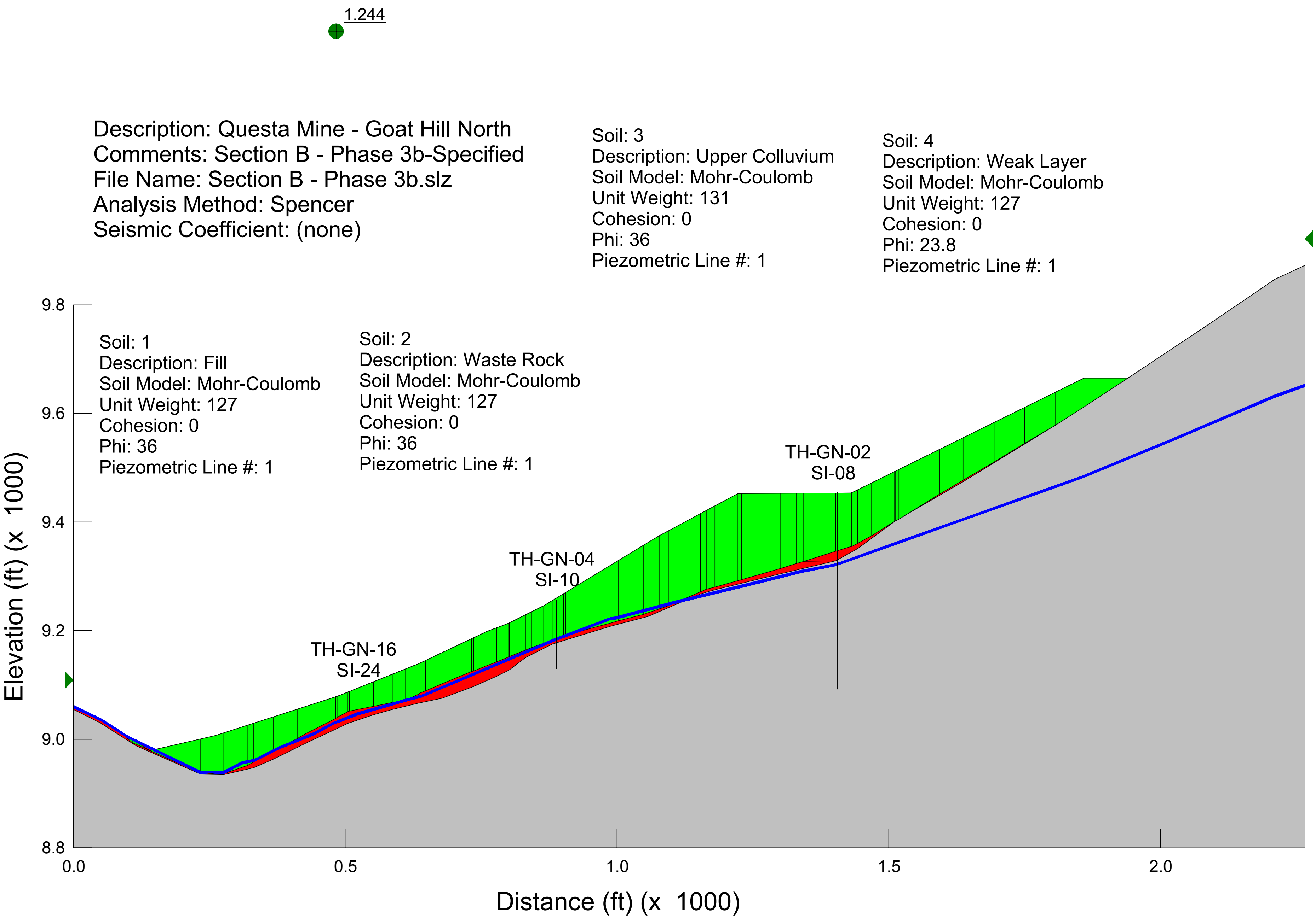


Description: Questa Mine - Goat Hill North
File Name: Sct E Ph3a.slz
Analysis Method: Spencer
Direction of Slip Movement: Right to Left
Slip Surface Option: Fully Specified
P.W.P. Option: Piezometric lines with Ru
Seismic Coefficient: (none)
Soil: 2
Description: Waste Rock
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 3
Description: Upper Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
1.043
Soil: 4
Description: Weak Layer above WT
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 23.8
Piezometric Line #: 1
Soil: 5
Description: Weak Layer
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 23.8
Piezometric Line #: 1
10.3
9.8
9.3
8.8
Elevation (ft) (x 1000)
-0.5
0.0
0.5
1.0
1.5
2.0
2.5
Distance (ft) (x 1000)

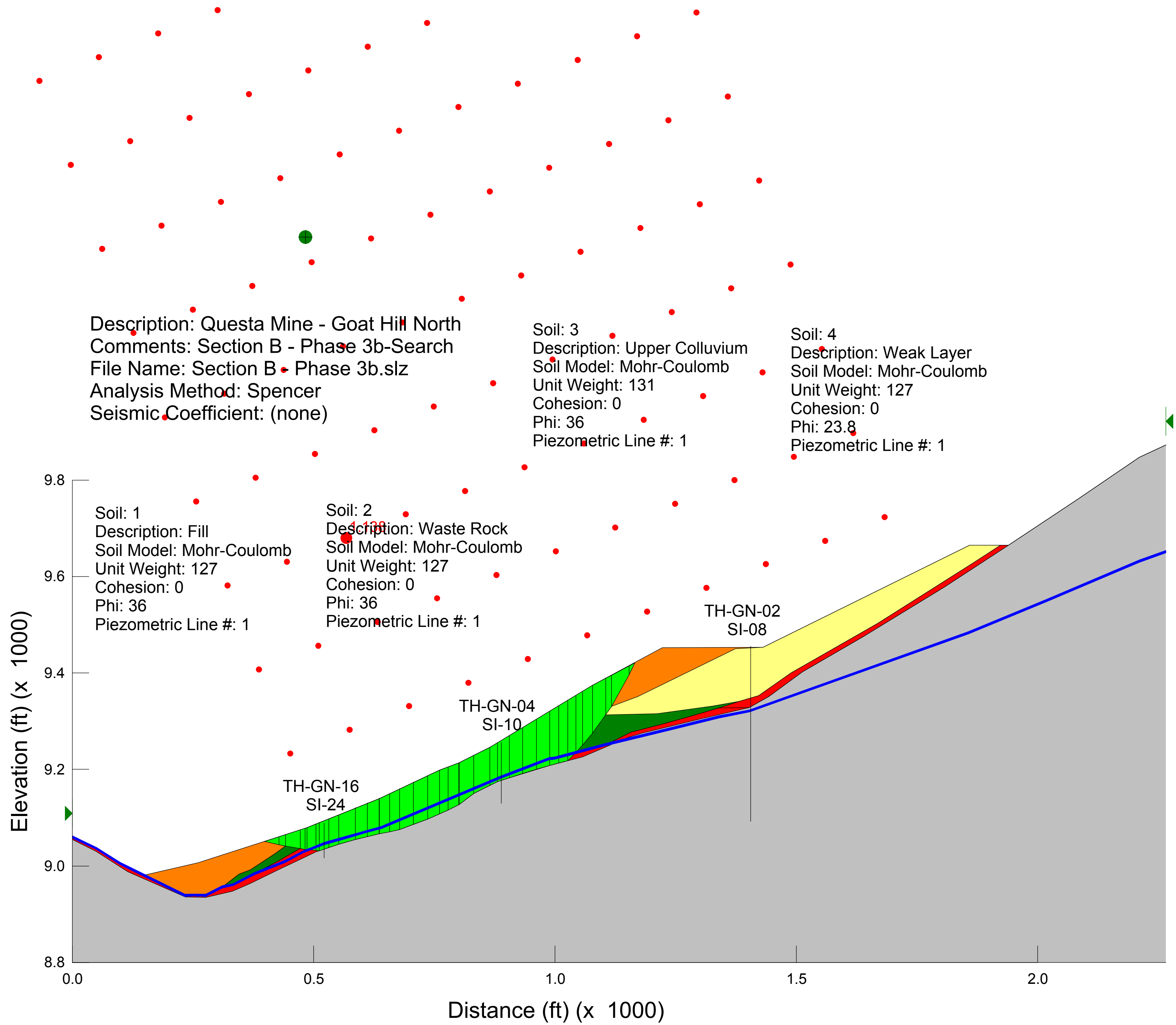
000427


Description: Questa Mine - Goat Hill North
File Name: Sct E Ph3a.slz
Analysis Method: Spencer
Direction of Slip Movement: Right to Left
Slip Surface Option: Grid and Radius
P.W.P. Option: Piezometric lines with Ru
Seismic Coefficient: (none)
Soil: 2
Description: Waste Rock
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 3
Description: Upper Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 4
Description: Weak Layer above WT
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 23.8
Piezometric Line #: 1
Soil: 5
Description: Weak Layer
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 23.8
Piezometric Line #: 1
Elevation (ft) (x 1000)
10.3
9.8
9.3
8.8
-0.5
0.0
0.5
1.0
1.5
2.0
2.5
Distance (ft) (x 1000)

000428

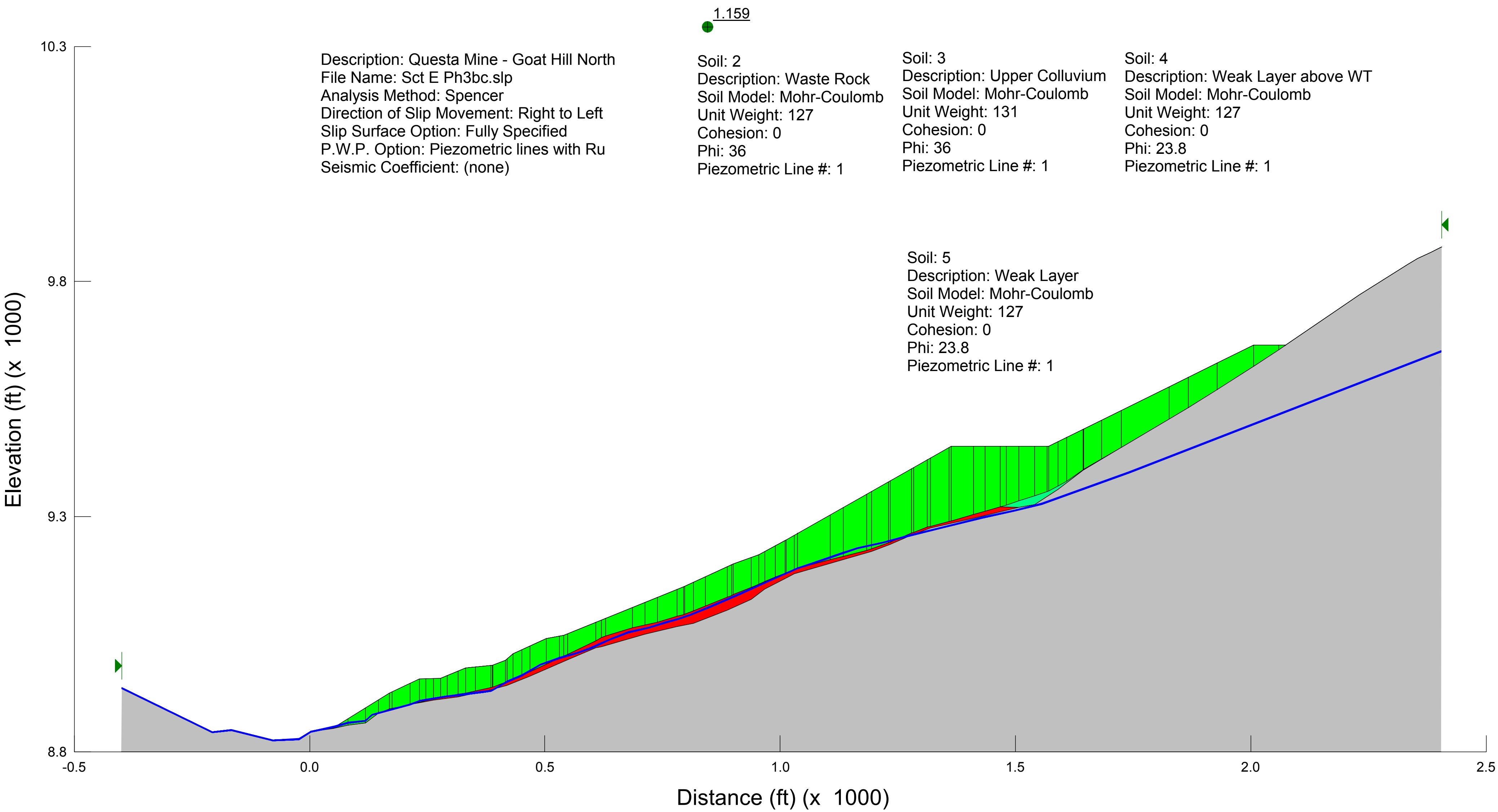

1.244
Description: Questa Mine - Goat Hill North
Comments: Section B - Phase 3b-Specified
File Name: Section B - Phase 3b.slz
Analysis Method: Spencer
Seismic Coefficient: (none)
Soil: 3
Description: Upper Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 4
Description: Weak Layer
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 23.8
Piezometric Line #: 1
Soil: 1
Description: Fill
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 2
Description: Waste Rock
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
TH-GN-02
SI-08
TH-GN-04
SI-10
TH-GN-16
SI-24
9.8
9.6
9.4
9.2
9.0
8.8
0.0
0.5
1.0
1.5
2.0
Elevation (ft) (x 1000)
Distance (ft) (x 1000)

000429

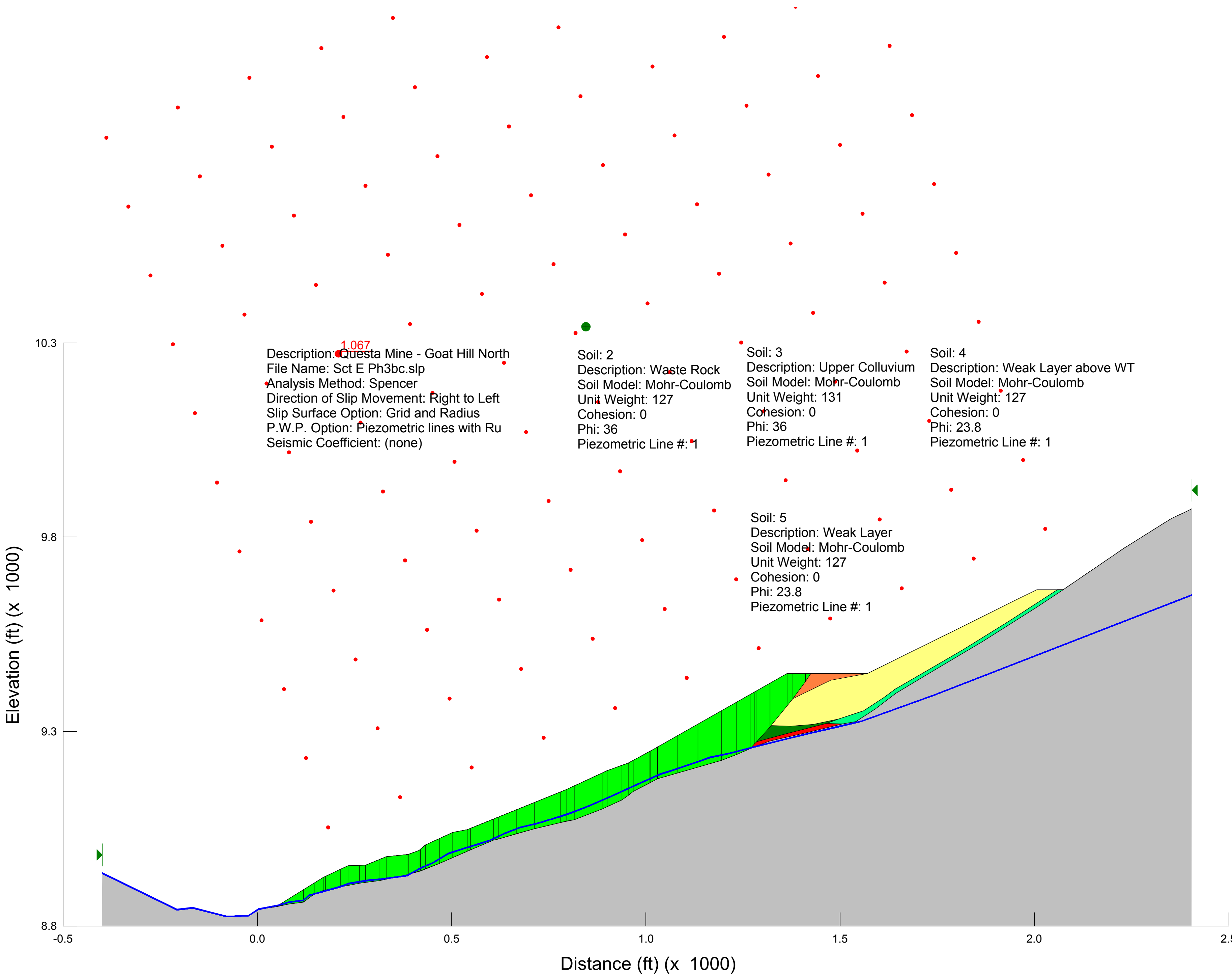


Description: Questa Mine - Goat Hill North
Comments: Section B - Phase 3b-Search
File Name: Section B - Phase 3b.slz
Analysis Method: Spencer
Seismic Coefficient: (none)
Soil: 3
Description: Upper Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 4
Description: Weak Layer
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 23.8
Piezometric Line #: 1
Soil: 1
Description: Fill
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 2
Description: Waste Rock
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
1.138
TH-GN-02
SI-08
TH-GN-04
SI-10
TH-GN-16
SI-24
9.8
9.6
9.4
9.2
9.0
8.8
Elevation (ft) (x 1000)
0.0
0.5
1.0
1.5
2.0
Distance (ft) (x 1000)

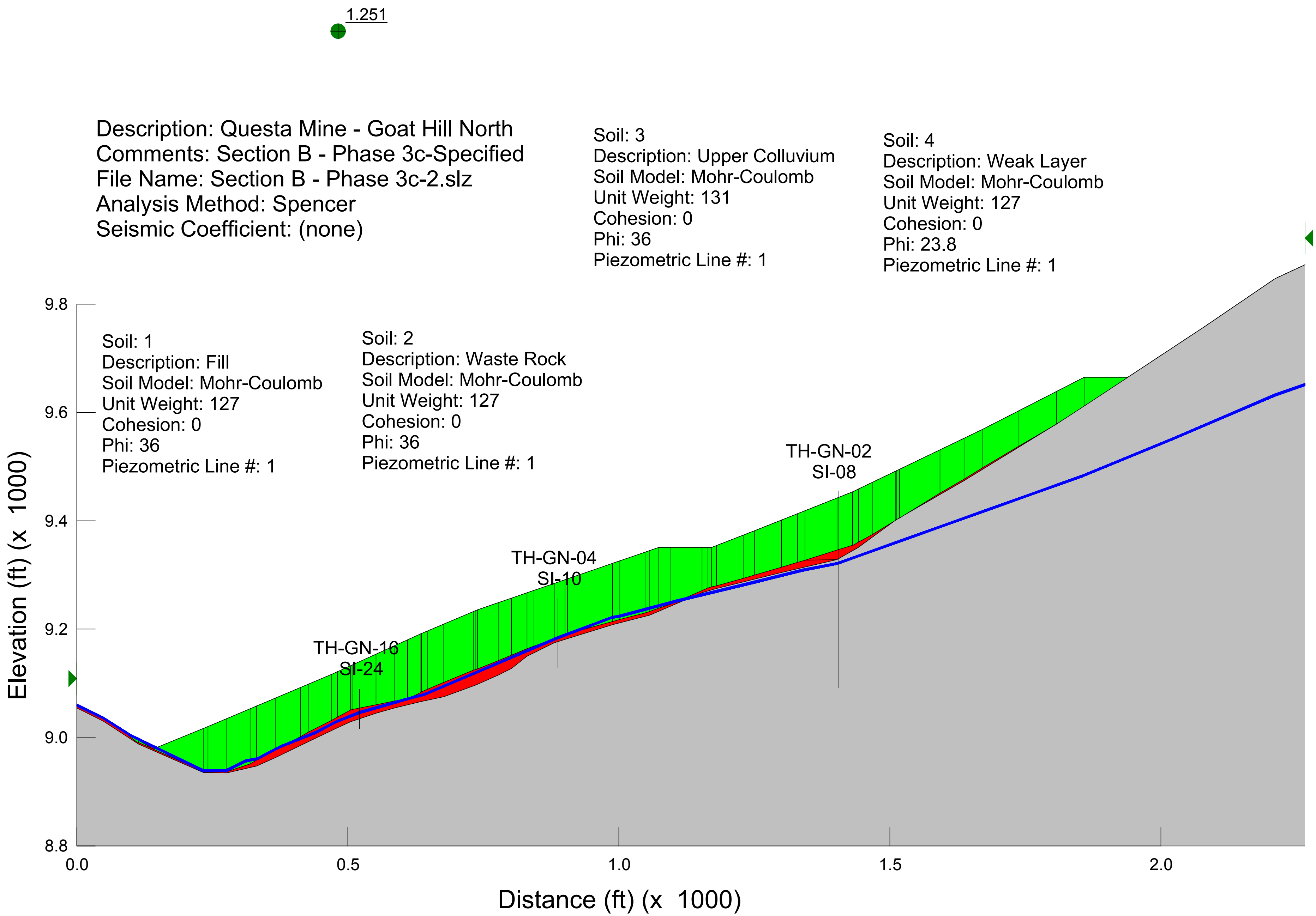
000430


1.159
Description: Questa Mine - Goat Hill North
File Name: Sct E Ph3bc.slp
Analysis Method: Spencer
Direction of Slip Movement: Right to Left
Slip Surface Option: Fully Specified
P.W.P. Option: Piezometric lines with Ru
Seismic Coefficient: (none)
Soil: 2
Description: Waste Rock
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 3
Description: Upper Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 4
Description: Weak Layer above WT
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 23.8
Piezometric Line #: 1
Soil: 5
Description: Weak Layer
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 23.8
Piezometric Line #: 1
10.3
9.8
9.3
8.8
Elevation (ft) (x 1000)
-0.5
0.0
0.5
1.0
1.5
2.0
2.5
Distance (ft) (x 1000)

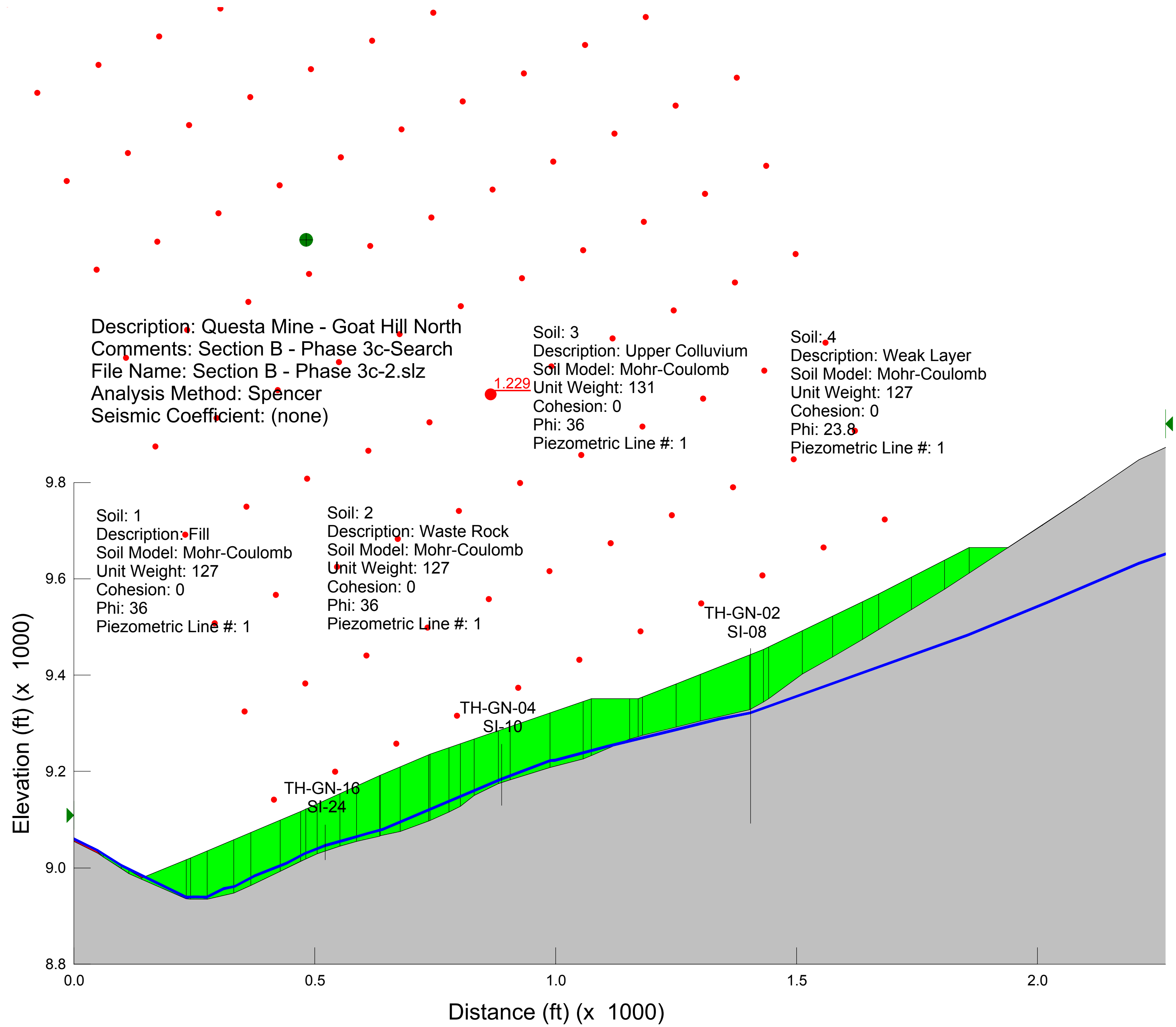
000431


1.067
Description: Questa Mine - Goat Hill North
File Name: Sct E Ph3bc.slp
Analysis Method: Spencer
Direction of Slip Movement: Right to Left
Slip Surface Option: Grid and Radius
P.W.P. Option: Piezometric lines with Ru
Seismic Coefficient: (none)
Soil: 2
Description: Waste Rock
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 3
Description: Upper Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 4
Description: Weak Layer above WT
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 23.8
Piezometric Line #: 1
Soil: 5
Description: Weak Layer
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 23.8
Piezometric Line #: 1
10.3
9.8
9.3
8.8
Elevation (ft) (x 1000)
-0.5
0.0
0.5
1.0
1.5
2.0
2.5
Distance (ft) (x 1000)

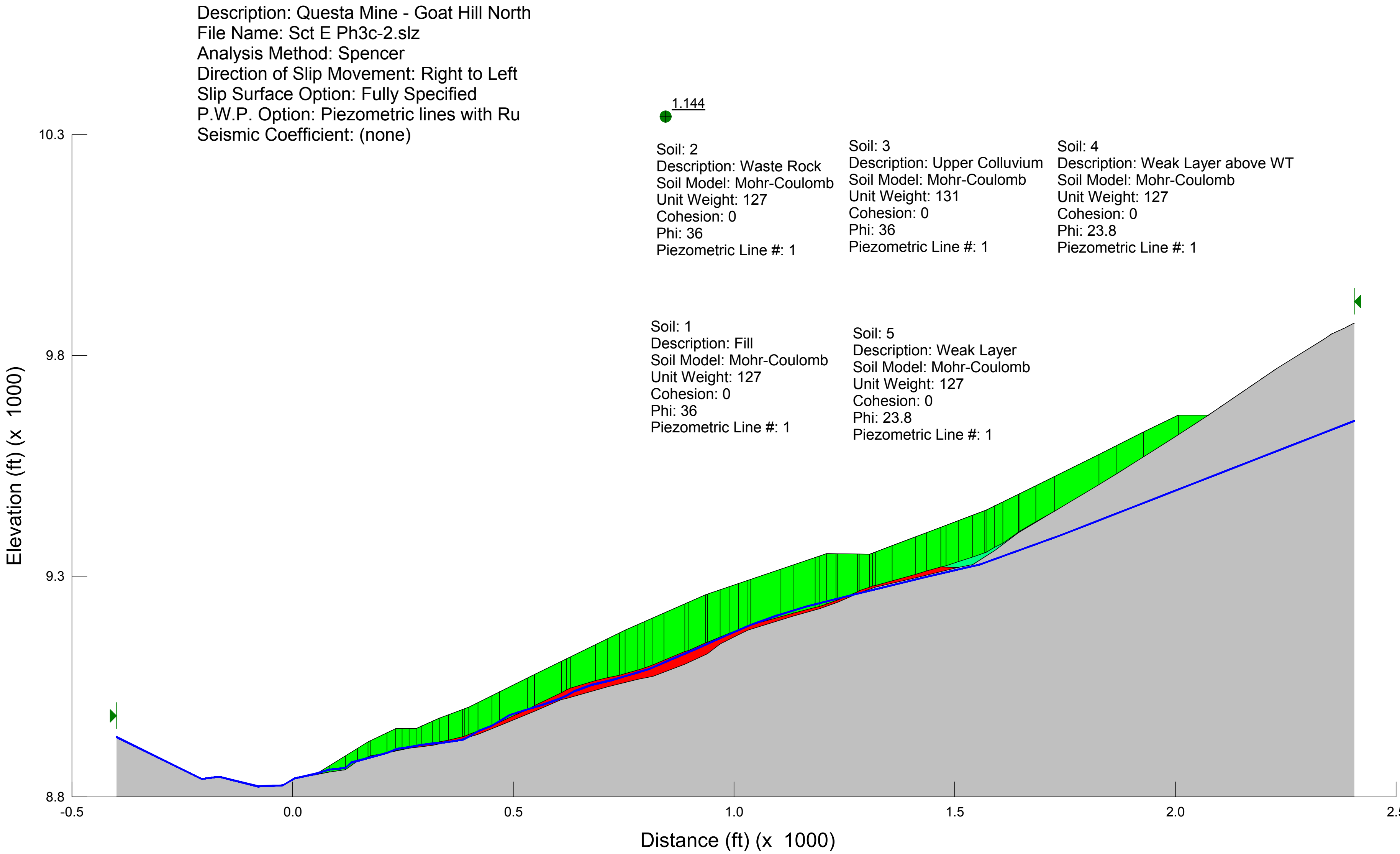
000432


1.251
Description: Questa Mine - Goat Hill North
Comments: Section B - Phase 3c-Specified
File Name: Section B - Phase 3c-2.slz
Analysis Method: Spencer
Seismic Coefficient: (none)
Soil: 3
Description: Upper Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 4
Description: Weak Layer
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 23.8
Piezometric Line #: 1
Soil: 1
Description: Fill
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 2
Description: Waste Rock
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
TH-GN-02
SI-08
TH-GN-04
SI-10
TH-GN-16
SI-24
Elevation (ft) (x 1000)
9.8
9.6
9.4
9.2
9.0
8.8
0.0
0.5
1.0
1.5
2.0
Distance (ft) (x 1000)

000433

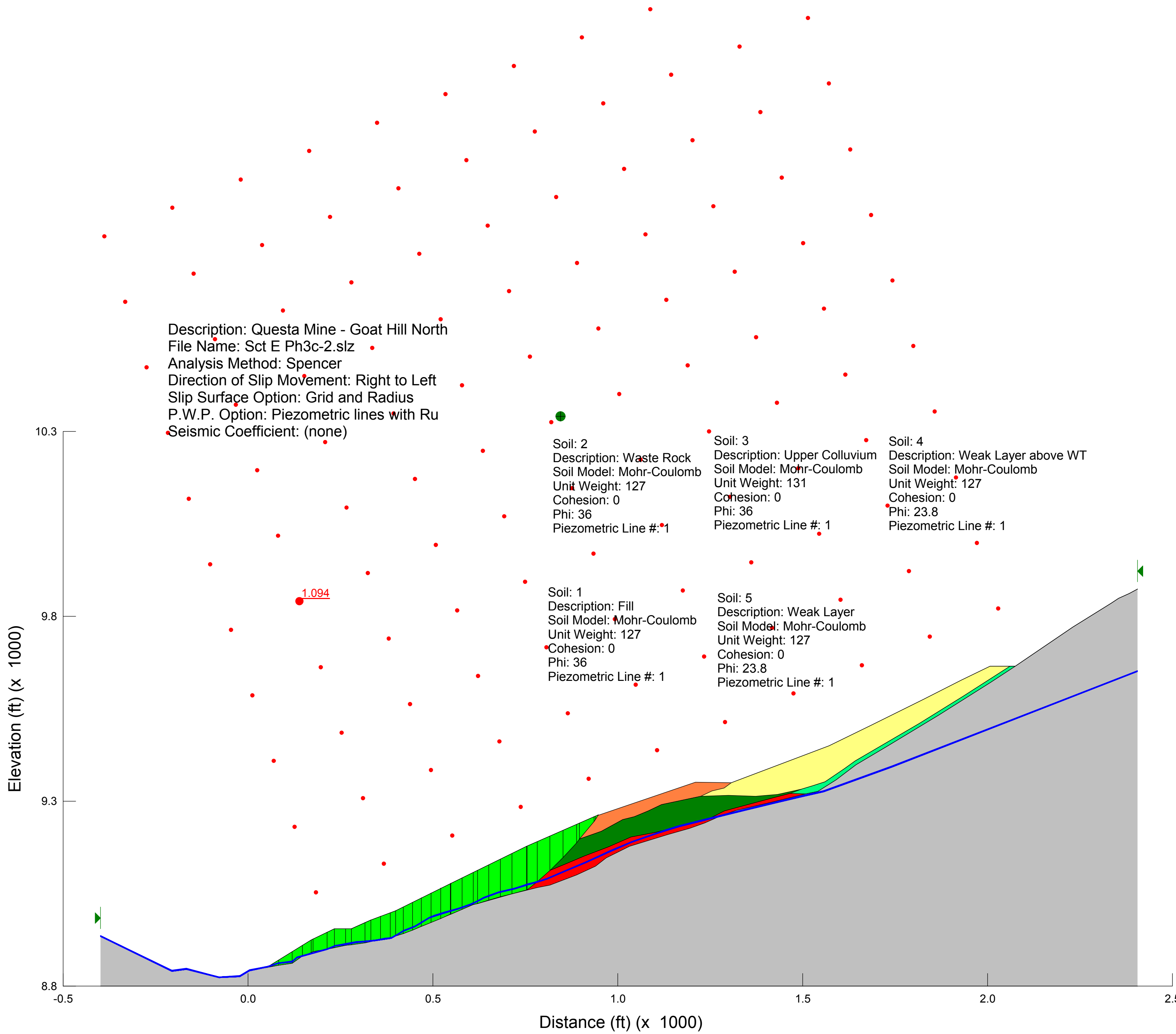

Description: Questa Mine - Goat Hill North
Comments: Section B - Phase 3c-Search
File Name: Section B - Phase 3c-2.slz
Analysis Method: Spencer
Seismic Coefficient: (none)
1.229
Soil: 3
Description: Upper Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 4
Description: Weak Layer
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 23.8
Piezometric Line #: 1
Soil: 1
Description: Fill
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 2
Description: Waste Rock
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
TH-GN-02
SI-08
TH-GN-04
SI-10
TH-GN-16
SI-24
Elevation (ft) (x 1000)
9.8
9.6
9.4
9.2
9.0
8.8
0.0
0.5
1.0
1.5
2.0
Distance (ft) (x 1000)

000434



Description: Questa Mine - Goat Hill North
File Name: Sct E Ph3c-2.slz
Analysis Method: Spencer
Direction of Slip Movement: Right to Left
Slip Surface Option: Fully Specified
P.W.P. Option: Piezometric lines with Ru
Seismic Coefficient: (none)
1.144
Soil: 2
Description: Waste Rock
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 3
Description: Upper Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 4
Description: Weak Layer above WT
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 23.8
Piezometric Line #: 1
Soil: 1
Description: Fill
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 5
Description: Weak Layer
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 23.8
Piezometric Line #: 1
Elevation (ft) (x 1000)
10.3
9.8
9.3
8.8
-0.5
0.0
0.5
1.0
1.5
2.0
2.5
Distance (ft) (x 1000)

000435

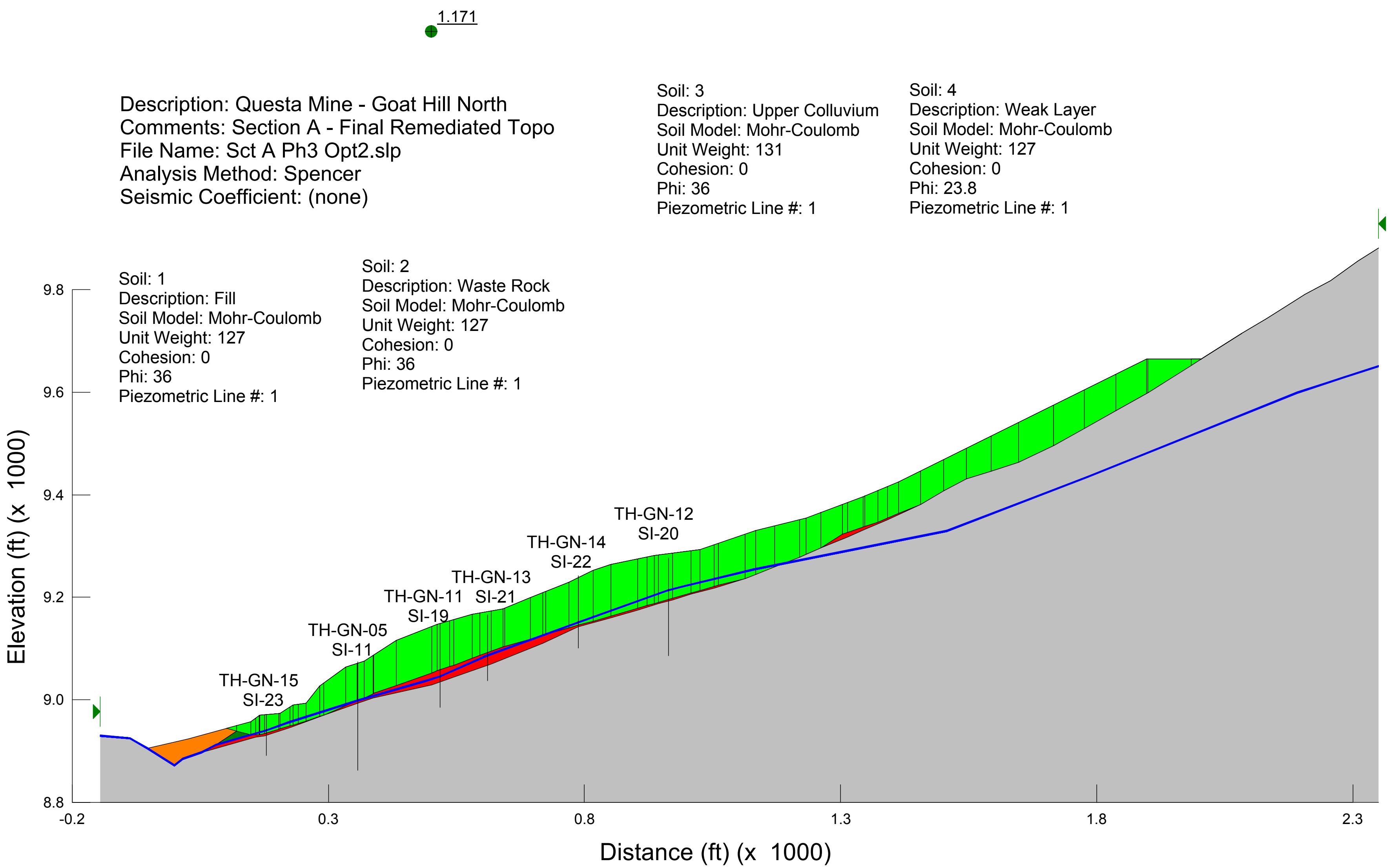


Description: Questa Mine - Goat Hill North
File Name: Sct E Ph3c-2.slz
Analysis Method: Spencer
Direction of Slip Movement: Right to Left
Slip Surface Option: Grid and Radius
P.W.P. Option: Piezometric lines with Ru
Seismic Coefficient: (none)
Soil: 2
Description: Waste Rock
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 3
Description: Upper Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 4
Description: Weak Layer above WT
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 23.8
Piezometric Line #: 1
Soil: 1
Description: Fill
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 5
Description: Weak Layer
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 23.8
Piezometric Line #: 1
1.094
10.3
9.8
9.3
8.8
-0.5
0.0
0.5
1.0
1.5
2.0
2.5
Elevation (ft) (x 1000)
Distance (ft) (x 1000)

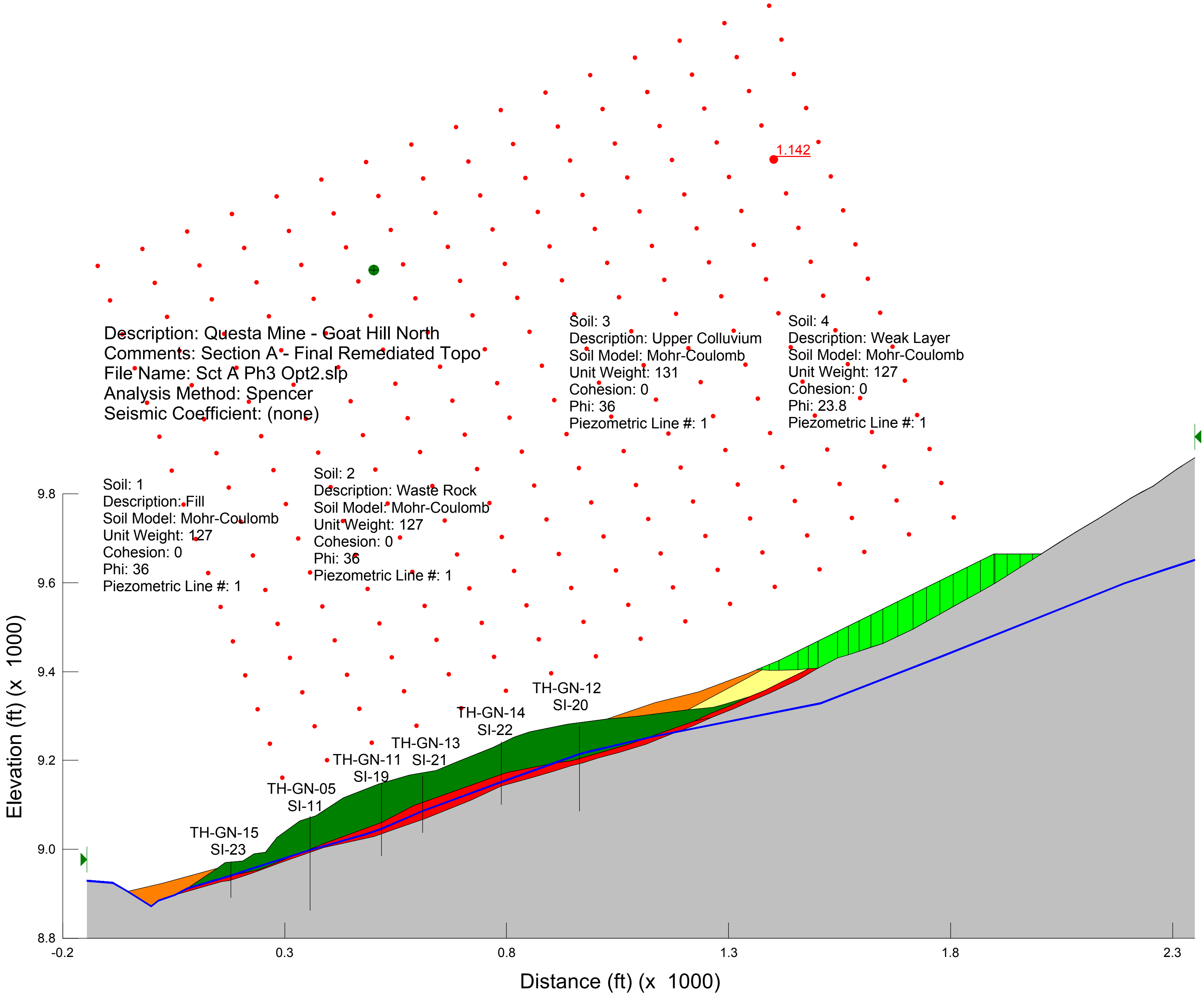
000436

2D Stability Results

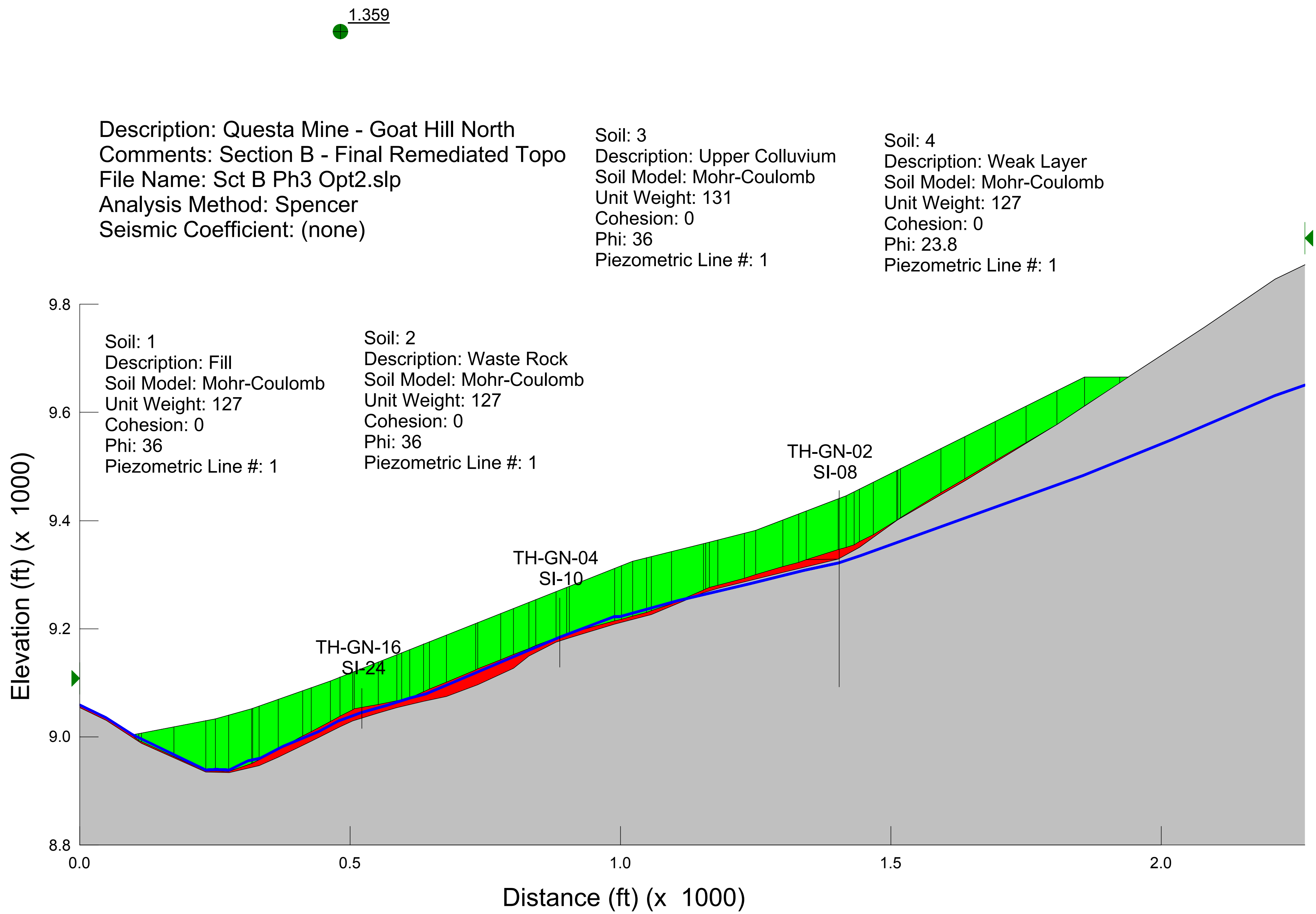
Final Mitigated


1.171
Description: Questa Mine - Goat Hill North
Comments: Section A - Final Remediated Topo
File Name: Sct A Ph3 Opt2.slp
Analysis Method: Spencer
Seismic Coefficient: (none)
Soil: 3
Description: Upper Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 4
Description: Weak Layer
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 23.8
Piezometric Line #: 1
Soil: 1
Description: Fill
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 2
Description: Waste Rock
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
TH-GN-12
SI-20
TH-GN-14
SI-22
TH-GN-13
SI-21
TH-GN-11
SI-19
TH-GN-05
SI-11
TH-GN-15
SI-23
Elevation (ft) (x 1000)
9.8
9.6
9.4
9.2
9.0
8.8
-0.2
0.3
0.8
1.3
1.8
2.3
Distance (ft) (x 1000)

000438

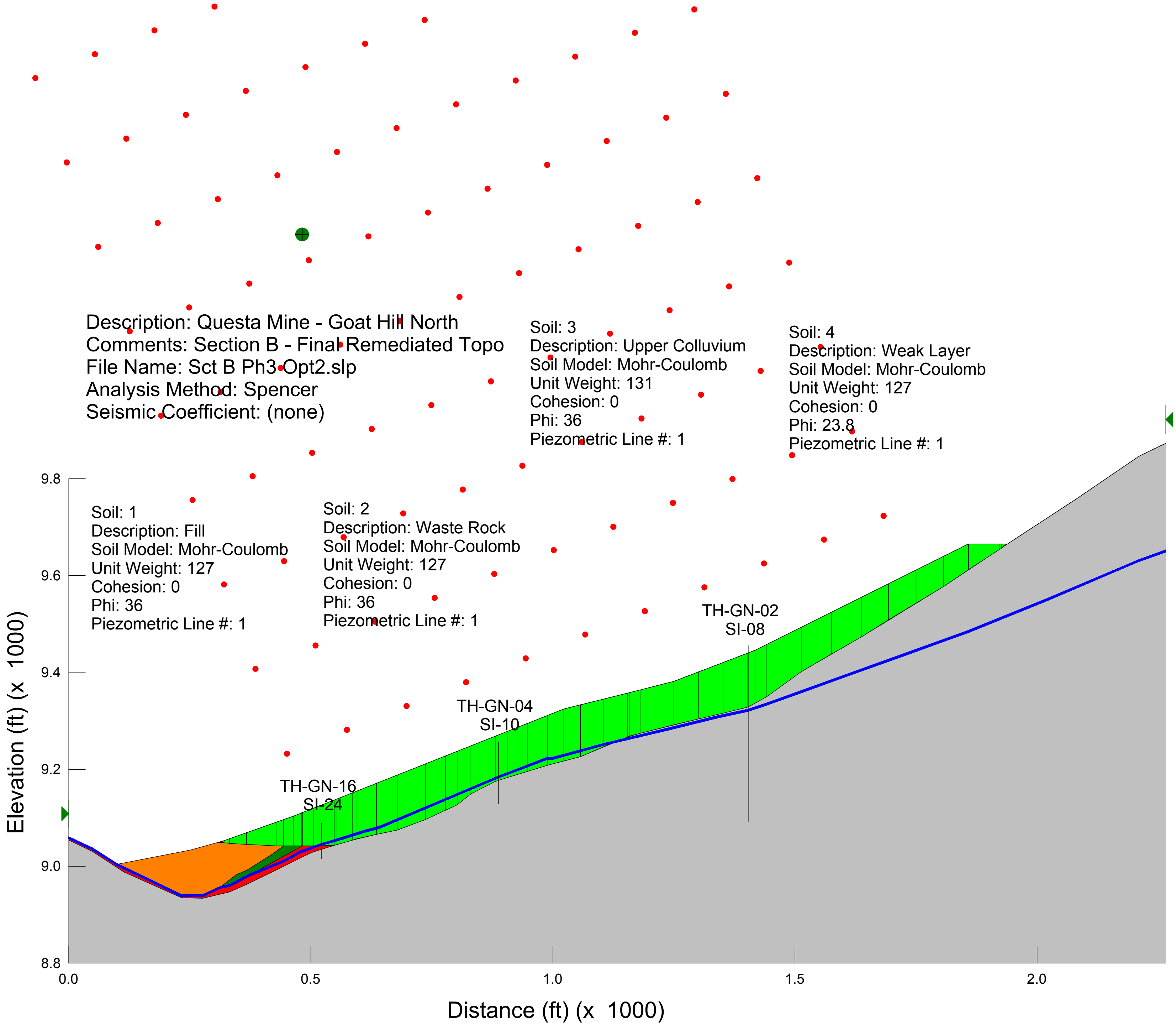

1.142
Description: Questa Mine - Goat Hill North
Comments: Section A - Final Remediated Topo
File Name: Sct A Ph3 Opt2.slp
Analysis Method: Spencer
Seismic Coefficient: (none)
Soil: 3
Description: Upper Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 4
Description: Weak Layer
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 23.8
Piezometric Line #: 1
Soil: 1
Description: Fill
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 2
Description: Waste Rock
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
TH-GN-12
SI-20
TH-GN-14
SI-22
TH-GN-13
SI-21
TH-GN-11
SI-19
TH-GN-05
SI-11
TH-GN-15
SI-23
9.8
9.6
9.4
9.2
9.0
8.8
Elevation (ft) (x 1000)
-0.2
0.3
0.8
1.3
1.8
2.3
Distance (ft) (x 1000)

000439

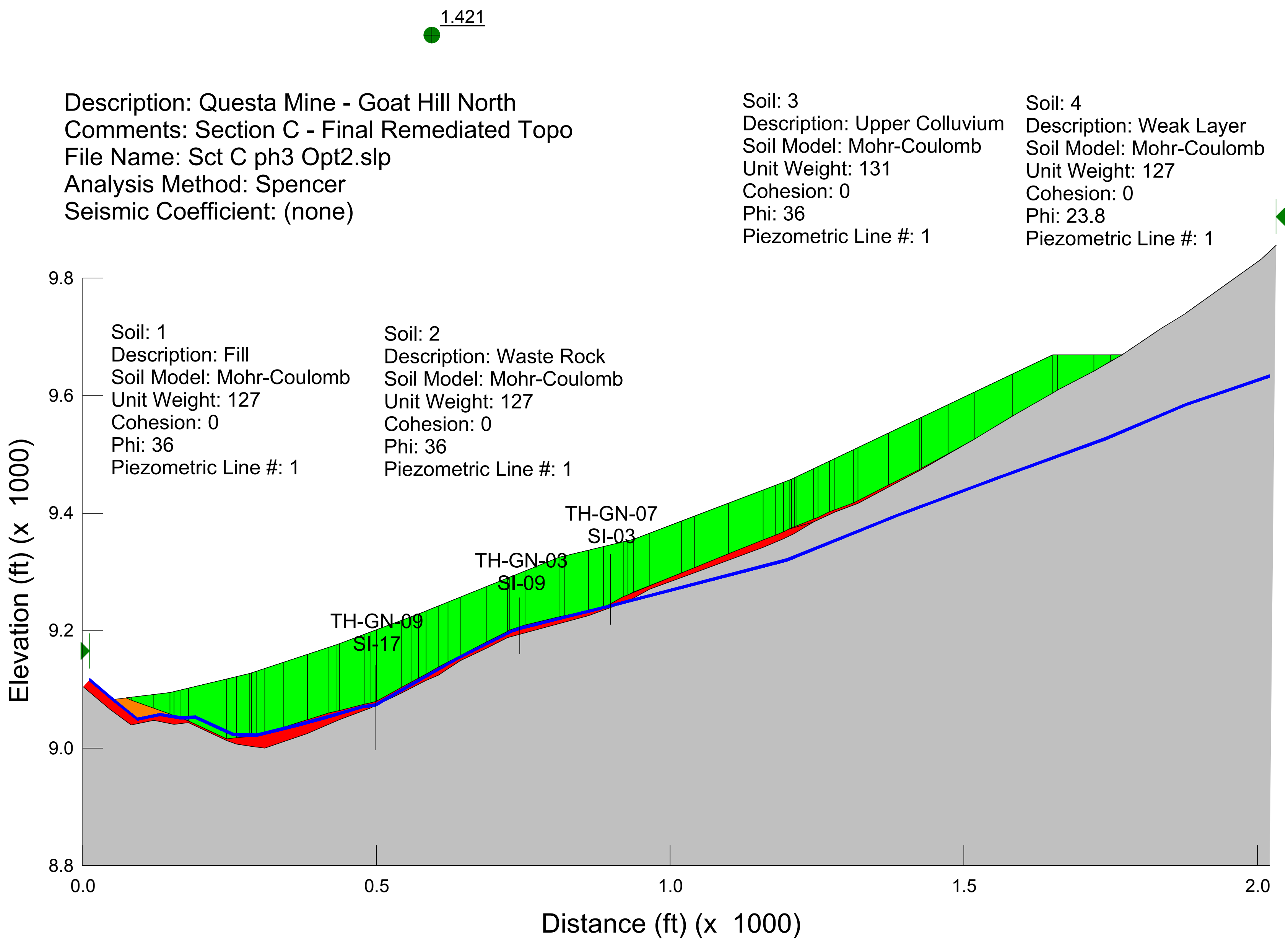


1.359
Description: Questa Mine - Goat Hill North
Comments: Section B - Final Remediated Topo
File Name: Sct B Ph3 Opt2.slp
Analysis Method: Spencer
Seismic Coefficient: (none)
Soil: 3
Description: Upper Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 4
Description: Weak Layer
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 23.8
Piezometric Line #: 1
Soil: 1
Description: Fill
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 2
Description: Waste Rock
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
TH-GN-02
SI-08
TH-GN-04
SI-10
TH-GN-16
SI-24
Elevation (ft) (x 1000)
9.8
9.6
9.4
9.2
9.0
8.8
0.0
0.5
1.0
1.5
2.0
Distance (ft) (x 1000)

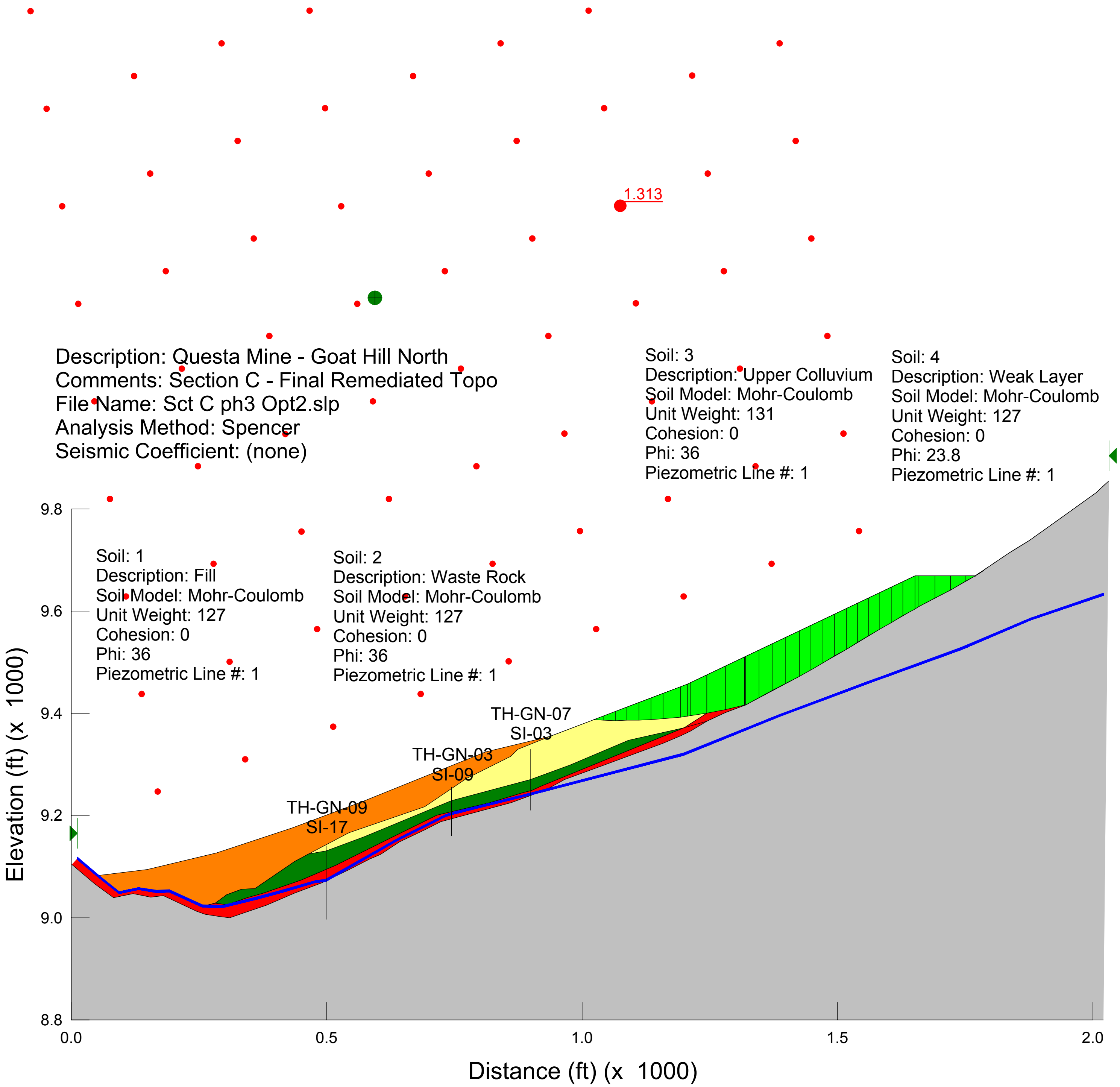
000440


Description: Questa Mine - Goat Hill North
Comments: Section B - Final Remediated Topo
File Name: Sct B Ph3 Opt2.slp
Analysis Method: Spencer
Seismic Coefficient: (none)
Soil: 3
Description: Upper Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 4
Description: Weak Layer
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 23.8
Piezometric Line #: 1
Soil: 1
Description: Fill
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 2
Description: Waste Rock
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
TH-GN-02
SI-08
TH-GN-04
SI-10
TH-GN-16
SI-24
9.8
9.6
9.4
9.2
9.0
8.8
Elevation (ft) (x 1000)
0.0
0.5
1.0
1.5
2.0
Distance (ft) (x 1000)

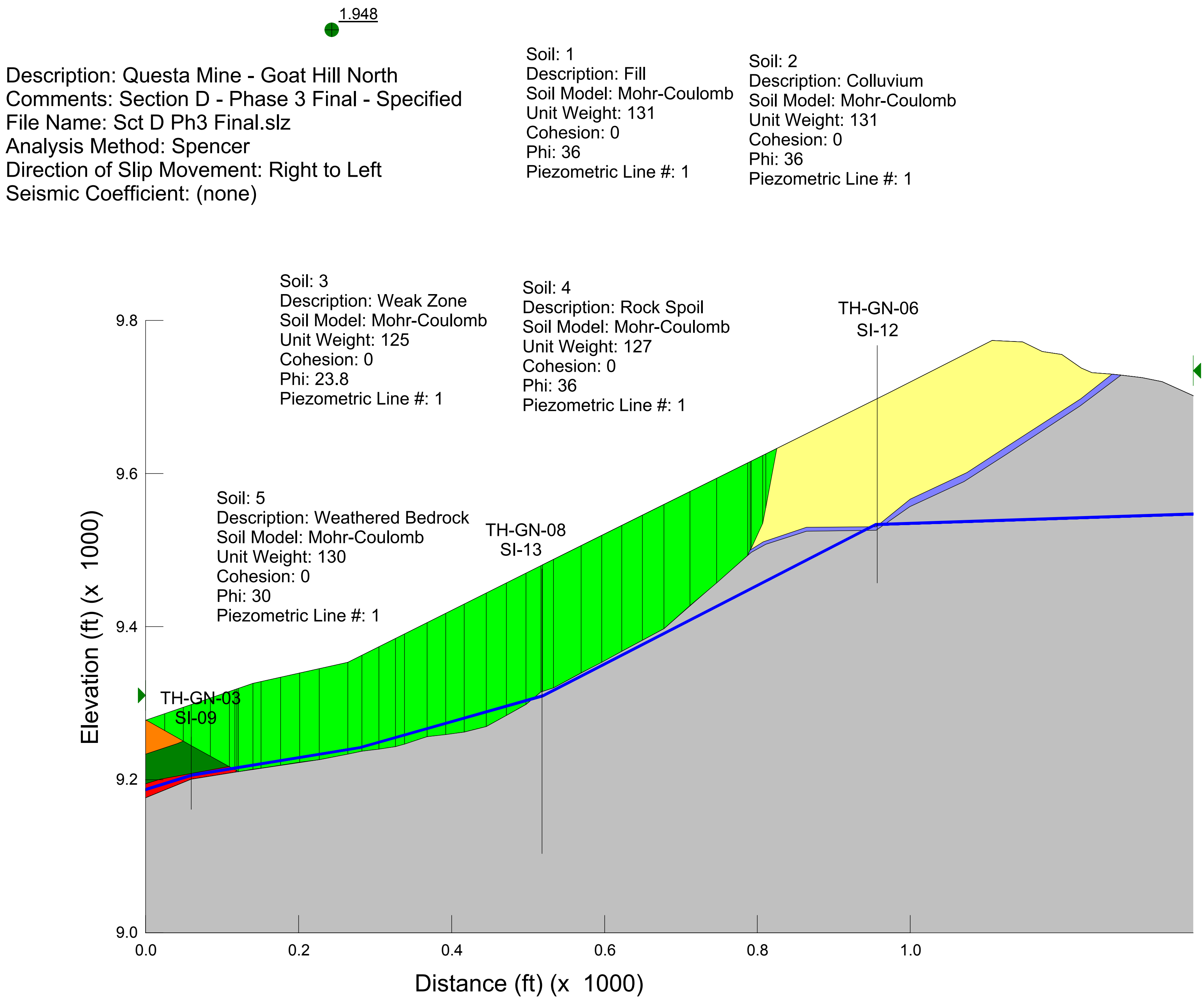
000441


1.421
Description: Questa Mine - Goat Hill North
Comments: Section C - Final Remediated Topo
File Name: Sct C ph3 Opt2.slp
Analysis Method: Spencer
Seismic Coefficient: (none)
Soil: 3
Description: Upper Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 4
Description: Weak Layer
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 23.8
Piezometric Line #: 1
Soil: 1
Description: Fill
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 2
Description: Waste Rock
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
TH-GN-07
SI-03
TH-GN-03
SI-09
TH-GN-09
SI-17
9.8
9.6
9.4
9.2
9.0
8.8
Elevation (ft) (x 1000)
0.0
0.5
1.0
1.5
2.0
Distance (ft) (x 1000)

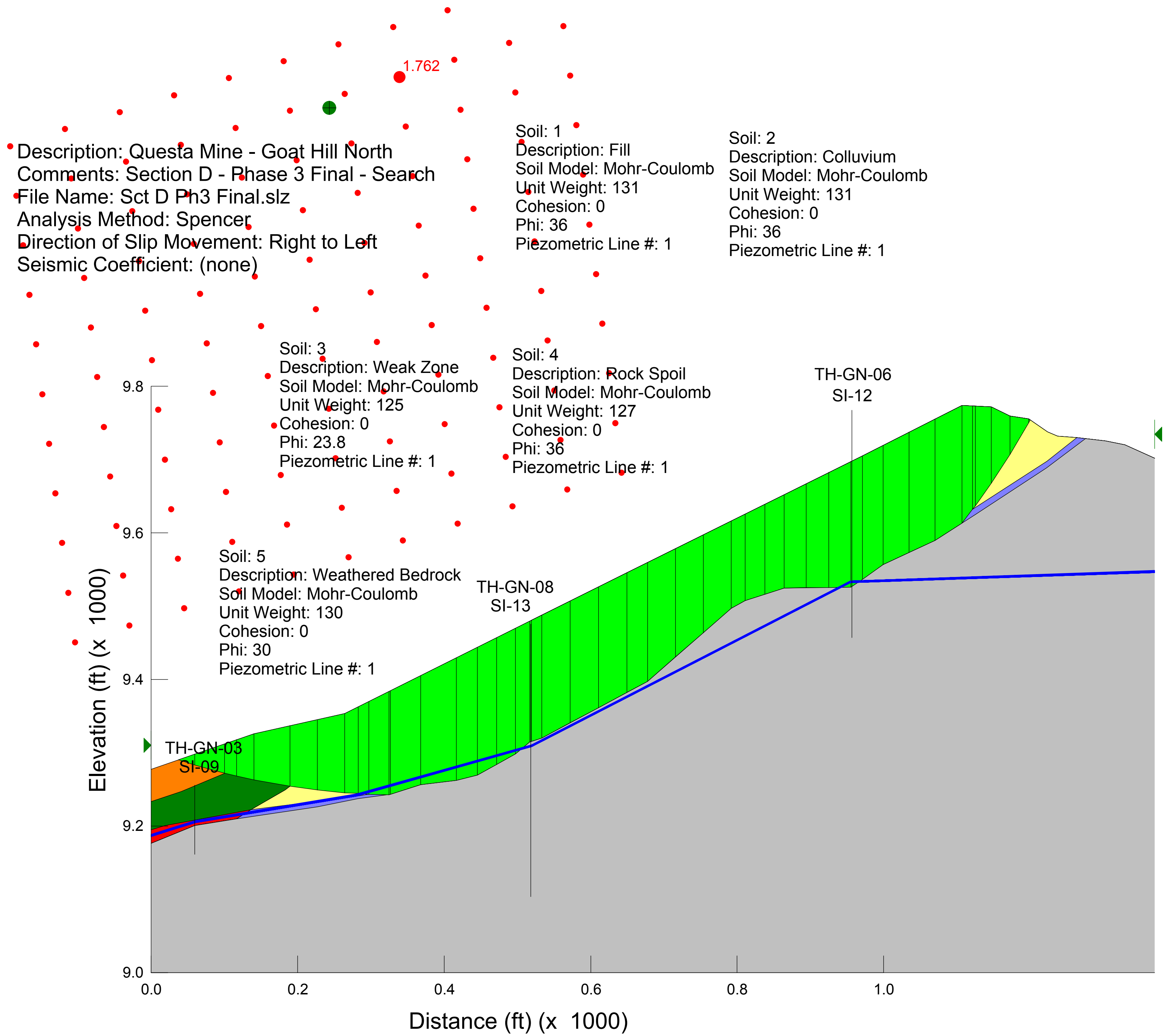
000442


1.313
Description: Questa Mine - Goat Hill North
Comments: Section C - Final Remediated Topo
File Name: Sct C ph3 Opt2.slp
Analysis Method: Spencer
Seismic Coefficient: (none)
Soil: 3
Description: Upper Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 4
Description: Weak Layer
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 23.8
Piezometric Line #: 1
Soil: 1
Description: Fill
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 2
Description: Waste Rock
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
TH-GN-07
SI-03
TH-GN-03
SI-09
TH-GN-09
SI-17
9.8
9.6
9.4
9.2
9.0
8.8
0.0
0.5
1.0
1.5
2.0
Elevation (ft) (x 1000)
Distance (ft) (x 1000)

000443

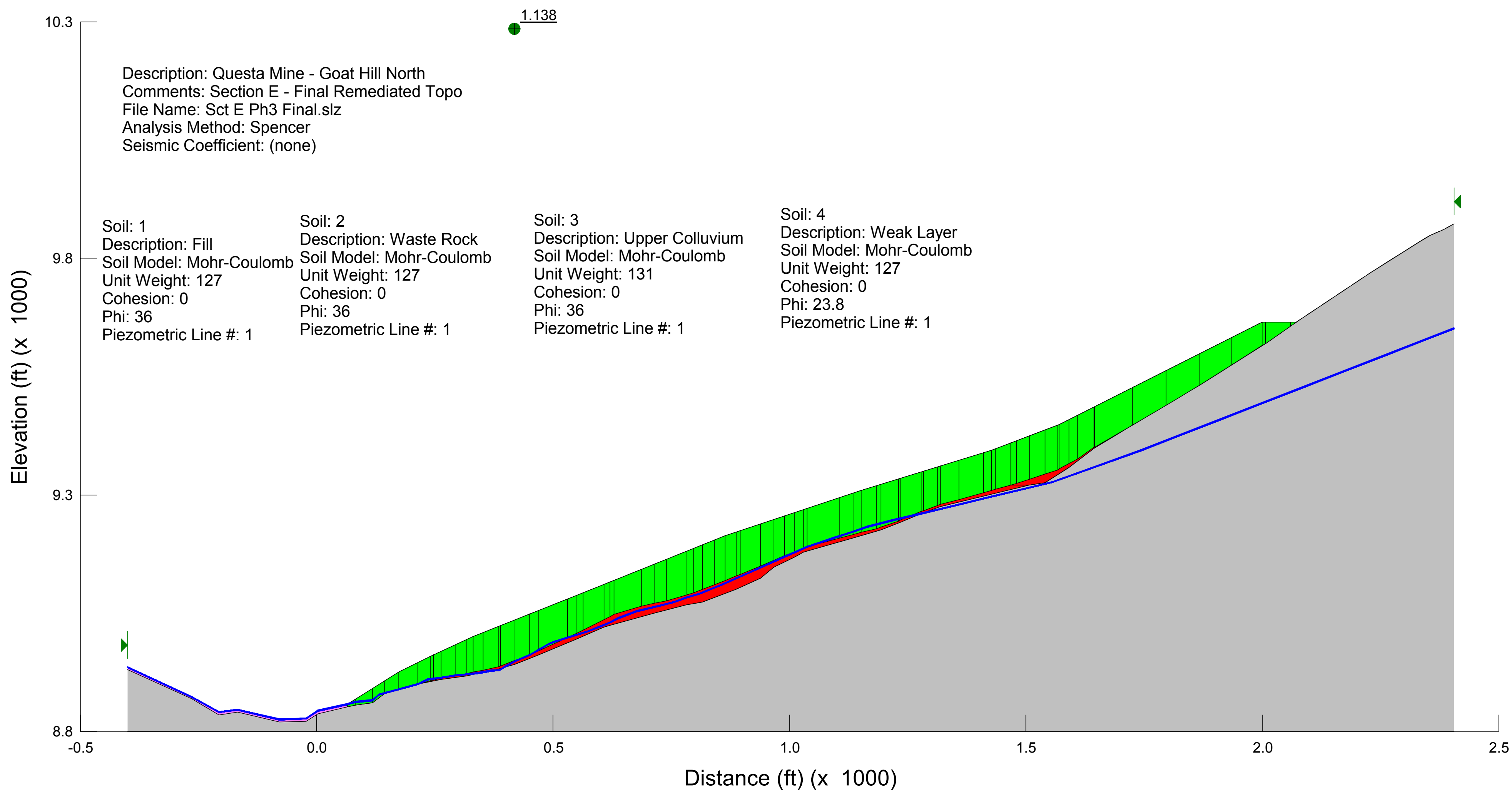

1.948
Description: Questa Mine - Goat Hill North
Comments: Section D - Phase 3 Final - Specified
File Name: Sct D Ph3 Final.slz
Analysis Method: Spencer
Direction of Slip Movement: Right to Left
Seismic Coefficient: (none)
Soil: 1
Description: Fill
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 2
Description: Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 3
Description: Weak Zone
Soil Model: Mohr-Coulomb
Unit Weight: 125
Cohesion: 0
Phi: 23.8
Piezometric Line #: 1
Soil: 4
Description: Rock Spoil
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
TH-GN-06
SI-12
Soil: 5
Description: Weathered Bedrock
Soil Model: Mohr-Coulomb
Unit Weight: 130
Cohesion: 0
Phi: 30
Piezometric Line #: 1
TH-GN-08
SI-13
TH-GN-03
SI-09
Elevation (ft) (x 1000)
9.8
9.6
9.4
9.2
9.0
0.0
0.2
0.4
0.6
0.8
1.0
Distance (ft) (x 1000)

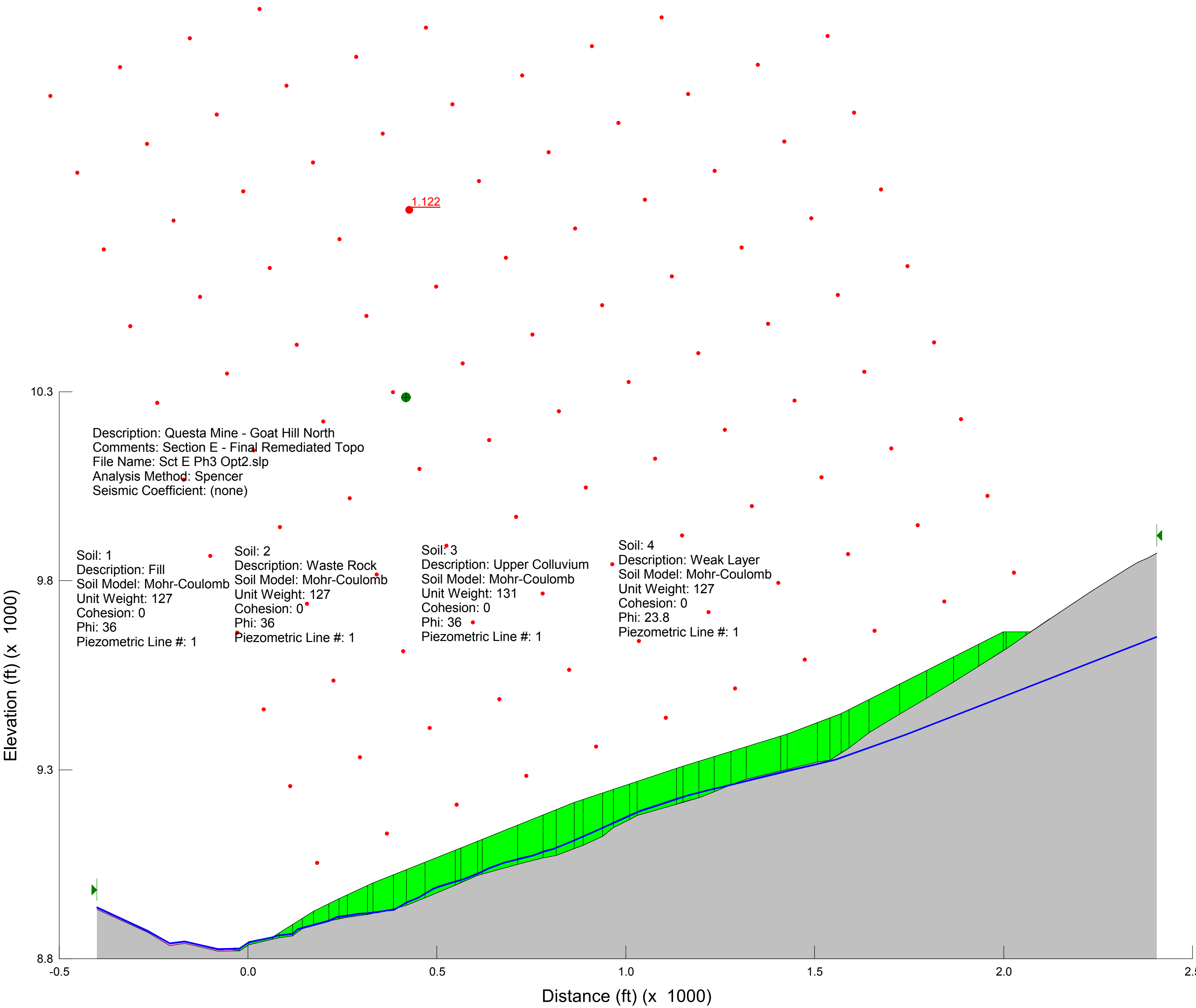
000444



1.762
Description: Questa Mine - Goat Hill North
Comments: Section D - Phase 3 Final - Search
File Name: Sct D Ph3 Final.slz
Analysis Method: Spencer
Direction of Slip Movement: Right to Left
Seismic Coefficient: (none)
Soil: 1
Description: Fill
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 2
Description: Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 3
Description: Weak Zone
Soil Model: Mohr-Coulomb
Unit Weight: 125
Cohesion: 0
Phi: 23.8
Piezometric Line #: 1
Soil: 4
Description: Rock Spoil
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 5
Description: Weathered Bedrock
Soil Model: Mohr-Coulomb
Unit Weight: 130
Cohesion: 0
Phi: 30
Piezometric Line #: 1
TH-GN-06
SI-12
TH-GN-08
SI-13
TH-GN-03
SI-09
Elevation (ft) (x 1000)
9.8
9.6
9.4
9.2
9.0
0.0
0.2
0.4
0.6
0.8
1.0
Distance (ft) (x 1000)

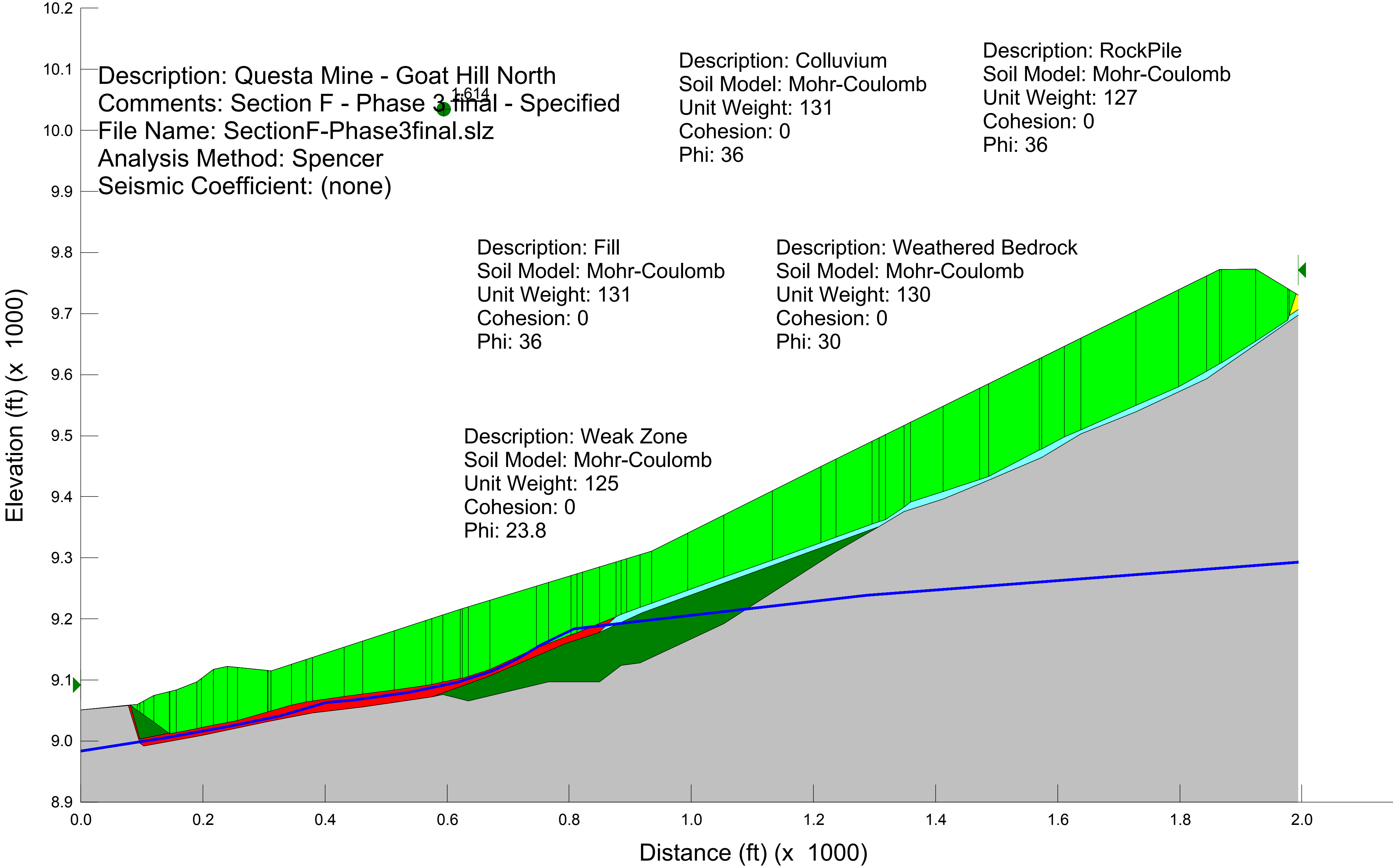
000445


1.138
Description: Questa Mine - Goat Hill North
Comments: Section E - Final Remediated Topo
File Name: Sct E Ph3 Final.slz
Analysis Method: Spencer
Seismic Coefficient: (none)
Soil: 1
Description: Fill
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 2
Description: Waste Rock
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 3
Description: Upper Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 4
Description: Weak Layer
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 23.8
Piezometric Line #: 1
Elevation (ft) (x 1000)
10.3
9.8
9.3
8.8
-0.5
0.0
0.5
1.0
1.5
2.0
2.5
Distance (ft) (x 1000)

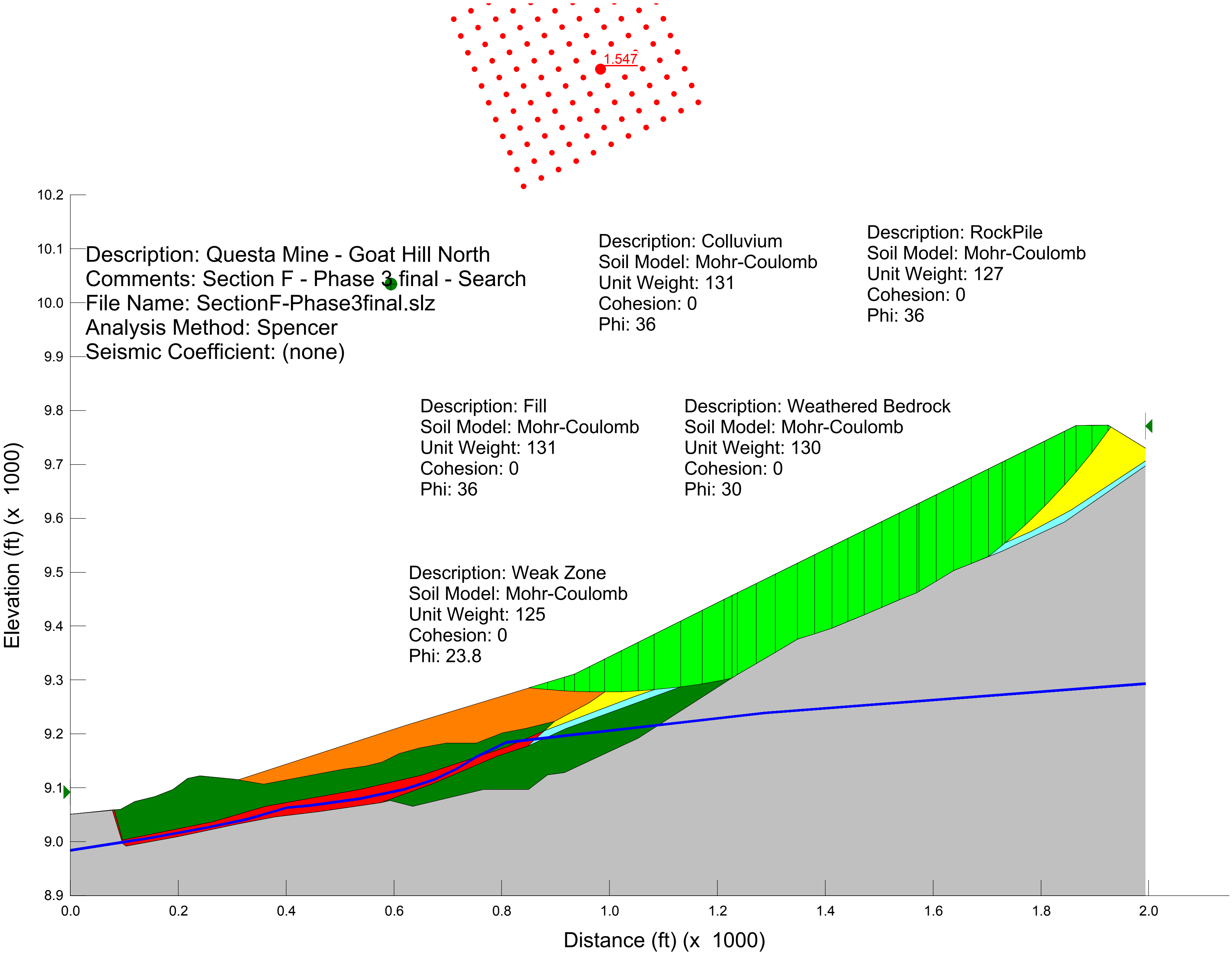


1.122
Description: Questa Mine - Goat Hill North
Comments: Section E - Final Remediated Topo
File Name: Sct E Ph3 Opt2.slp
Analysis Method: Spencer
Seismic Coefficient: (none)
Soil: 1
Description: Fill
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 2
Description: Waste Rock
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 3
Description: Upper Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 4
Description: Weak Layer
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 23.8
Piezometric Line #: 1
10.3
9.8
9.3
8.8
Elevation (ft) (x 1000)
-0.5
0.0
0.5
1.0
1.5
2.0
2.5
Distance (ft) (x 1000)

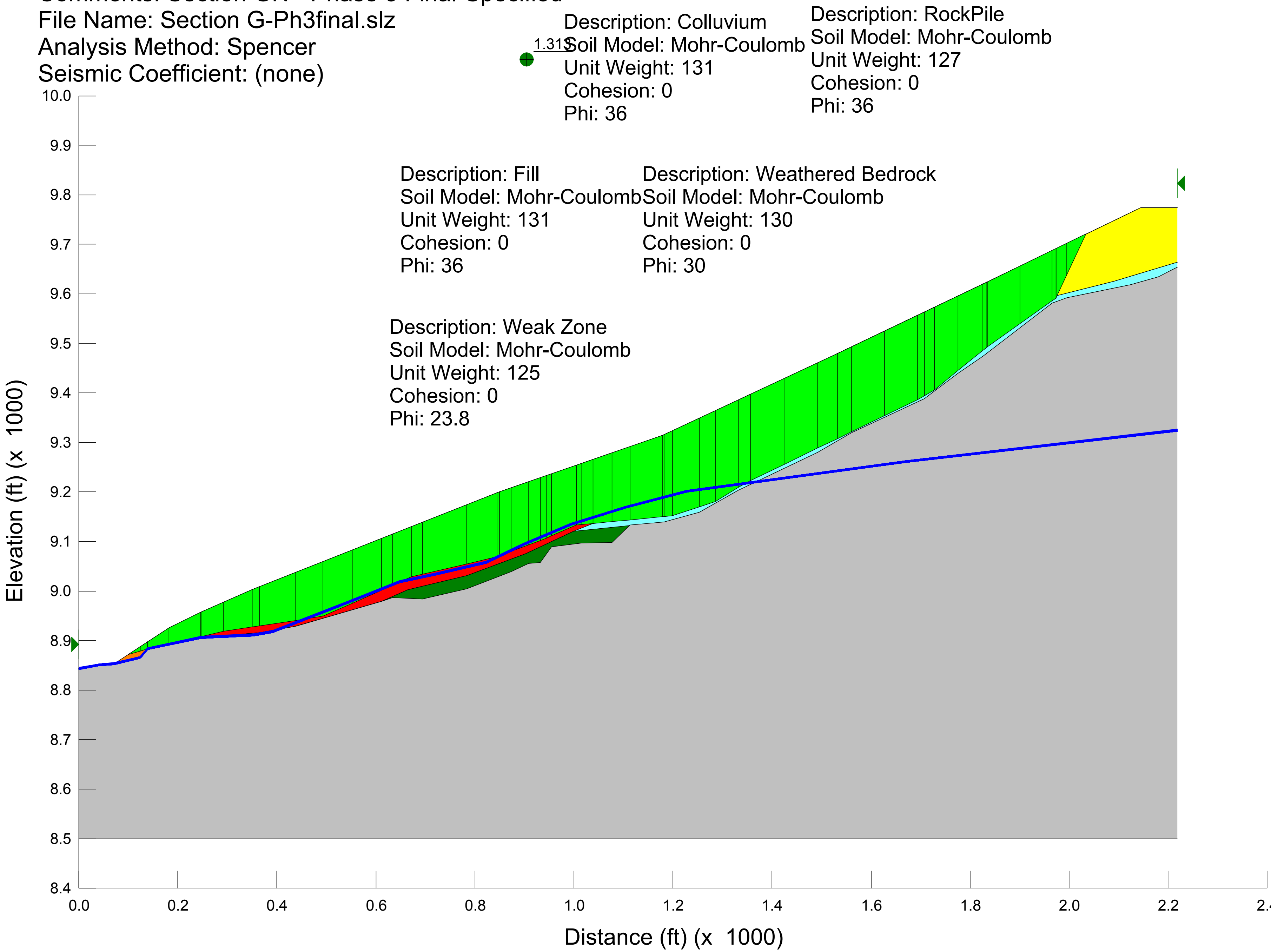
000447

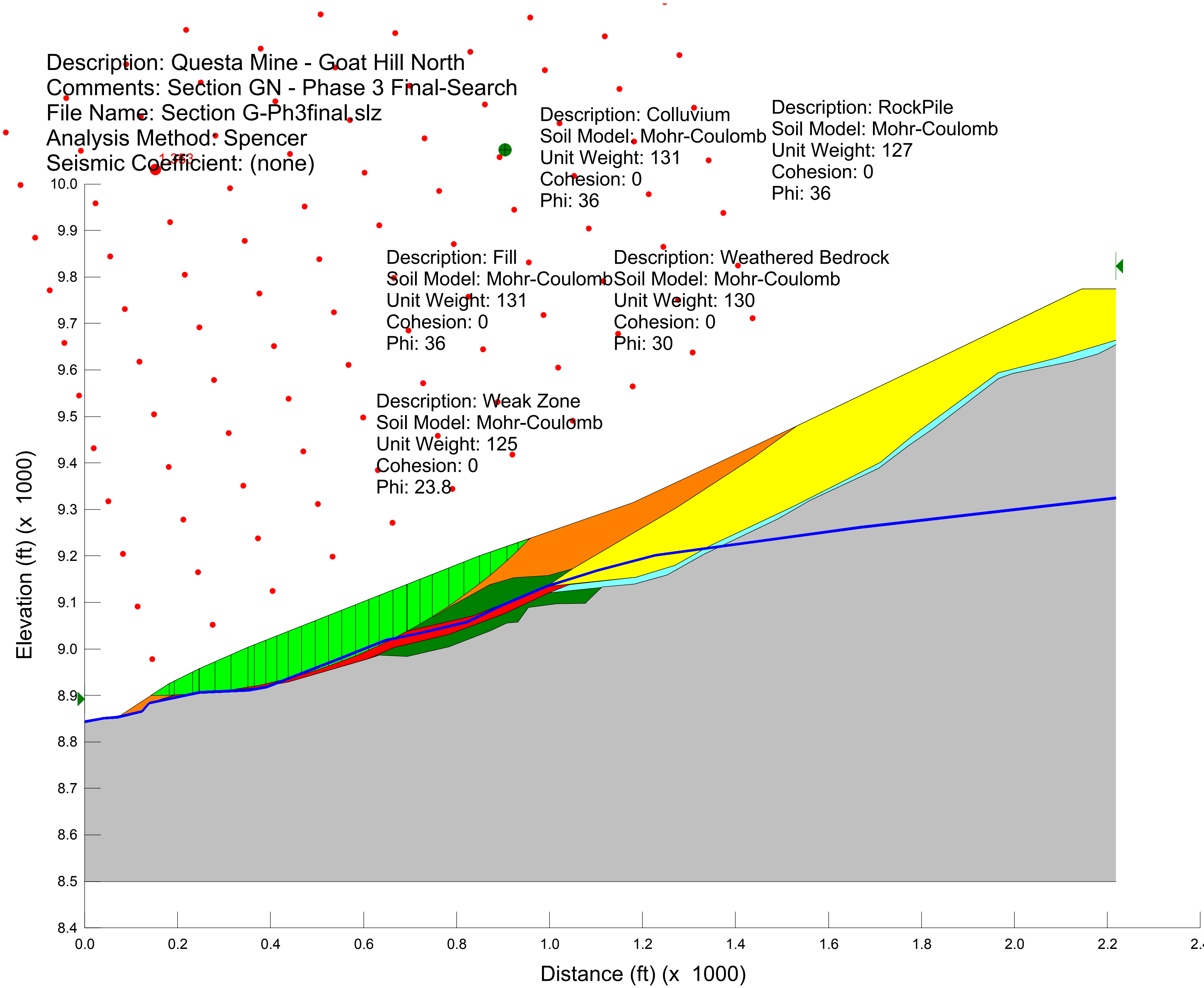

Description: Questa Mine - Goat Hill North
Comments: Section F - Phase 3 final - Specified
File Name: SectionF-Phase3final.slz
Analysis Method: Spencer
Seismic Coefficient: (none)
1.614
Description: Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Description: RockPile
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Description: Fill
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Description: Weathered Bedrock
Soil Model: Mohr-Coulomb
Unit Weight: 130
Cohesion: 0
Phi: 30
Description: Weak Zone
Soil Model: Mohr-Coulomb
Unit Weight: 125
Cohesion: 0
Phi: 23.8
Elevation (ft) (x 1000)
Distance (ft) (x 1000)
10.2
10.1
10.0
9.9
9.8
9.7
9.6
9.5
9.4
9.3
9.2
9.1
9.0
8.9
0.0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0

000448


1.547
Description: Questa Mine - Goat Hill North
Comments: Section F - Phase 3 final - Search
File Name: SectionF-Phase3final.slz
Analysis Method: Spencer
Seismic Coefficient: (none)
Description: Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Description: RockPile
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Description: Fill
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Description: Weathered Bedrock
Soil Model: Mohr-Coulomb
Unit Weight: 130
Cohesion: 0
Phi: 30
Description: Weak Zone
Soil Model: Mohr-Coulomb
Unit Weight: 125
Cohesion: 0
Phi: 23.8
Elevation (ft) (x 1000)
10.2
10.1
10.0
9.9
9.8
9.7
9.6
9.5
9.4
9.3
9.2
9.1
9.0
8.9
0.0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0
Distance (ft) (x 1000)

Description: Questa Mine - Goat Hill North
Comments: Section GN - Phase 3 Final-Specified
File Name: Section G-Ph3final.slz
Analysis Method: Spencer
Seismic Coefficient: (none)





Description: Questa Mine - Goat Hill North
Comments: Section GN - Phase 3 Final-Search
File Name: Section G-Ph3final.slz
Analysis Method: Spencer
Seismic Coefficient: (none)
Description: Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Description: RockPile
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Description: Fill
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Description: Weathered Bedrock
Soil Model: Mohr-Coulomb
Unit Weight: 130
Cohesion: 0
Phi: 30
Description: Weak Zone
Soil Model: Mohr-Coulomb
Unit Weight: 125
Cohesion: 0
Phi: 23.8
Elevation (ft) (x 1000)
Distance (ft) (x 1000)
10.0
9.9
9.8
9.7
9.6
9.5
9.4
9.3
9.2
9.1
9.0
8.9
8.8
8.7
8.6
8.5
8.4
0.0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0
2.2
2.4

000451

2D Stability Results

Alternate Water Table

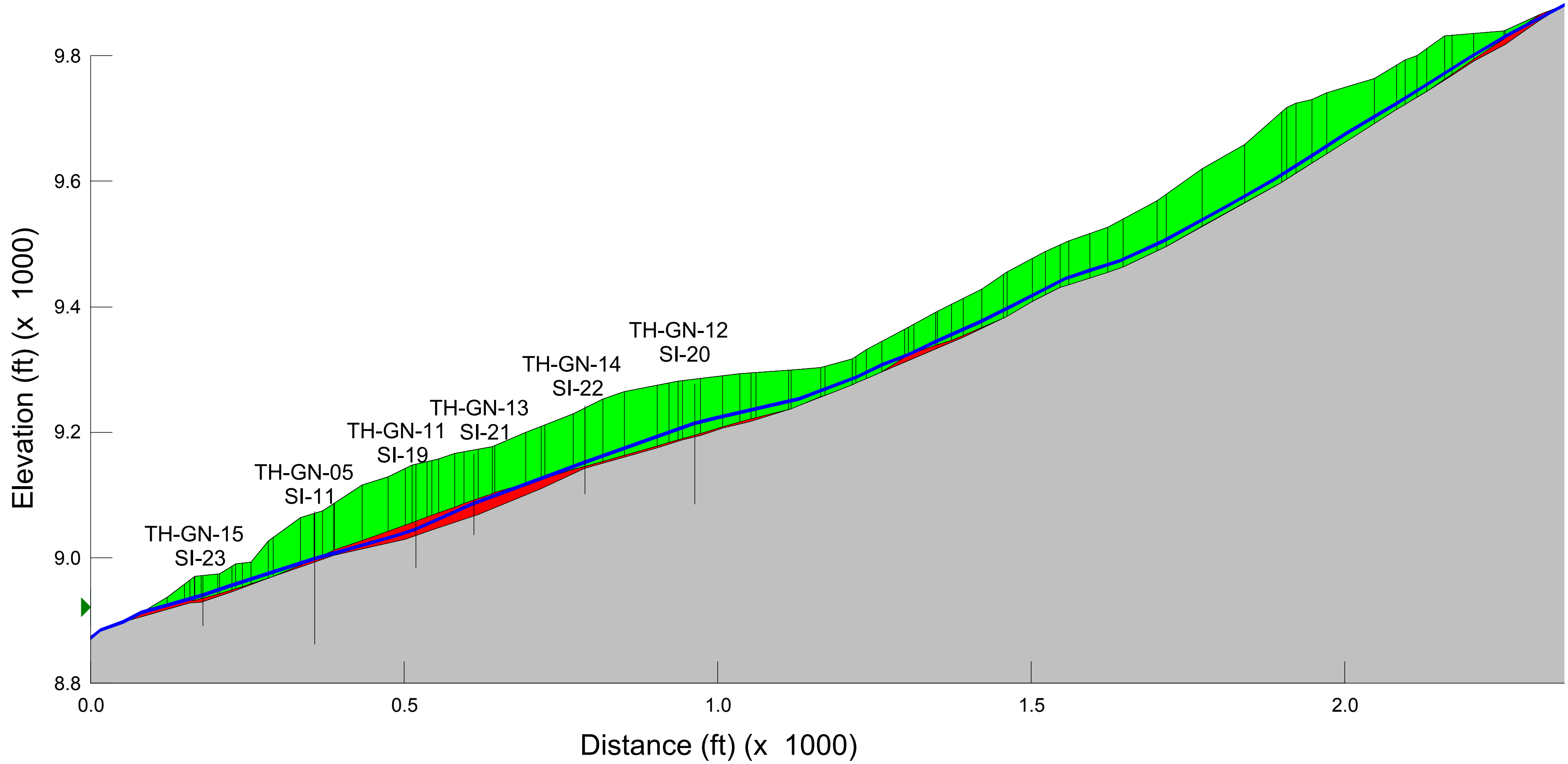
0.999

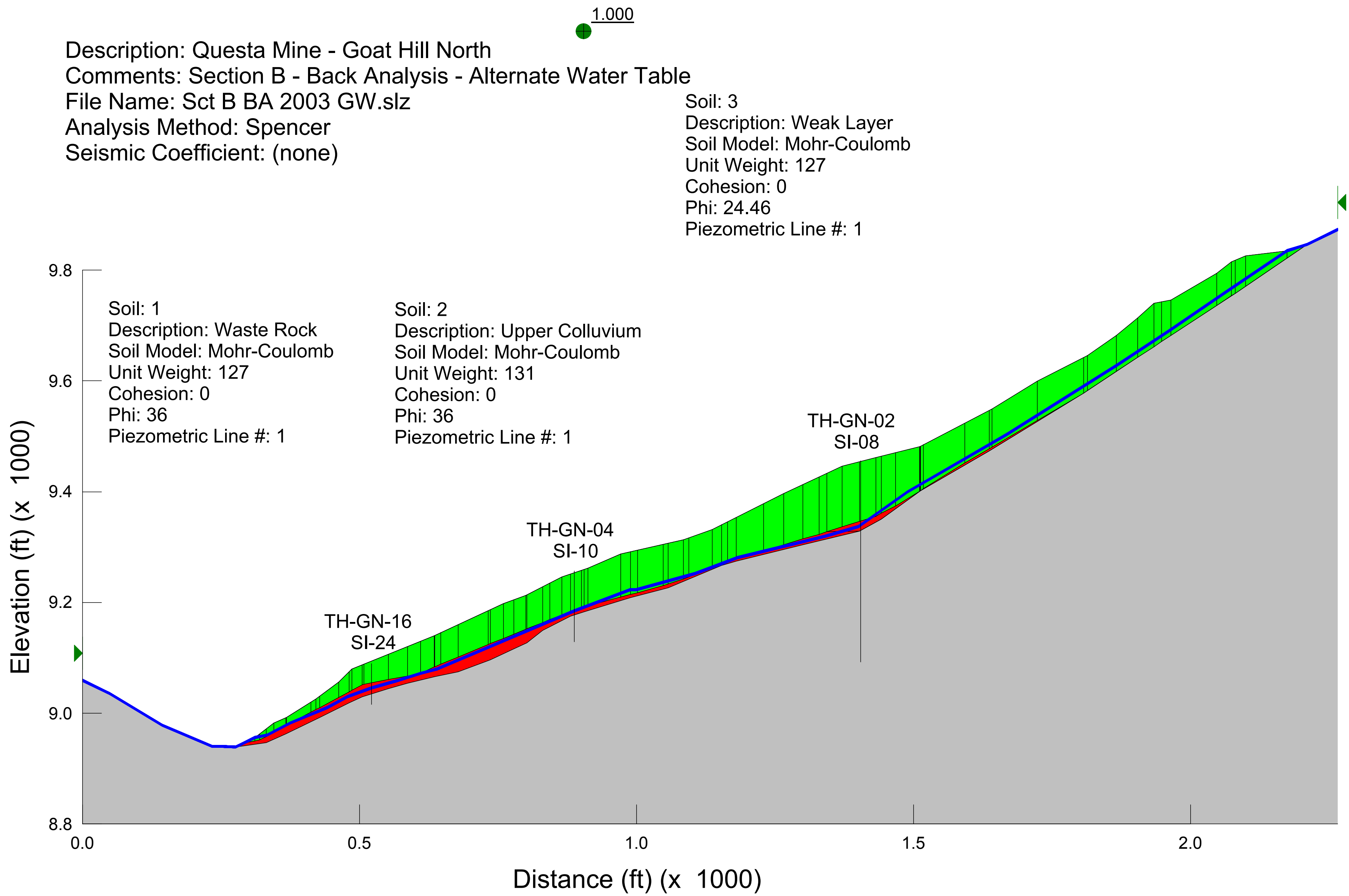
Description: Questa Mine - Goat Hill North
Comments: Section A - Back Analysis - Alternate Water Table
File Name: Sct A BA 2003 GW.slz
Analysis Method: Spencer
Seismic Coefficient: (none)

Soil: 1
Description: Waste Rock
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1

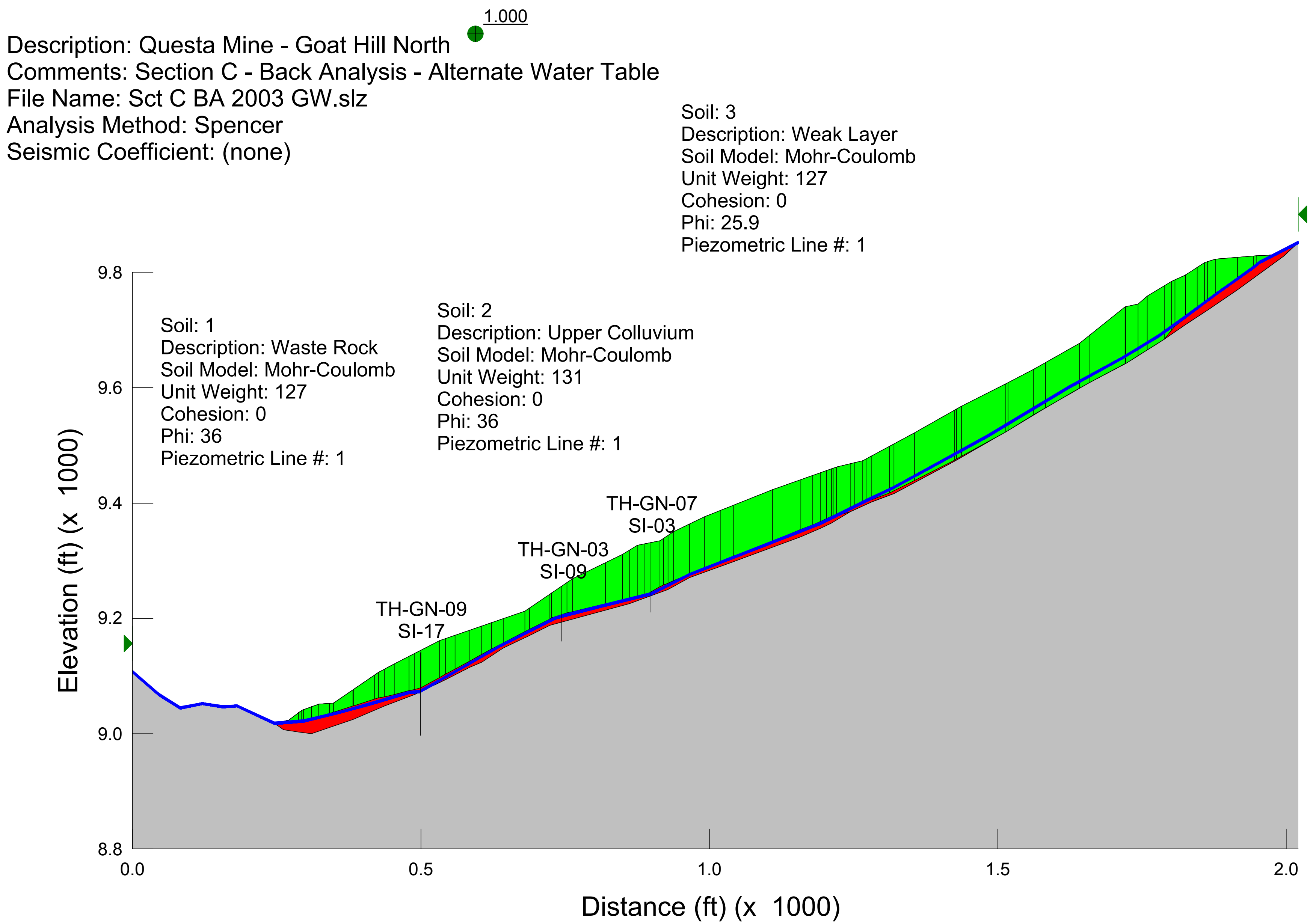
Soil: 2
Description: Upper Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1

Soil: 3
Description: Weak Layer
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 23.7
Piezometric Line #: 1



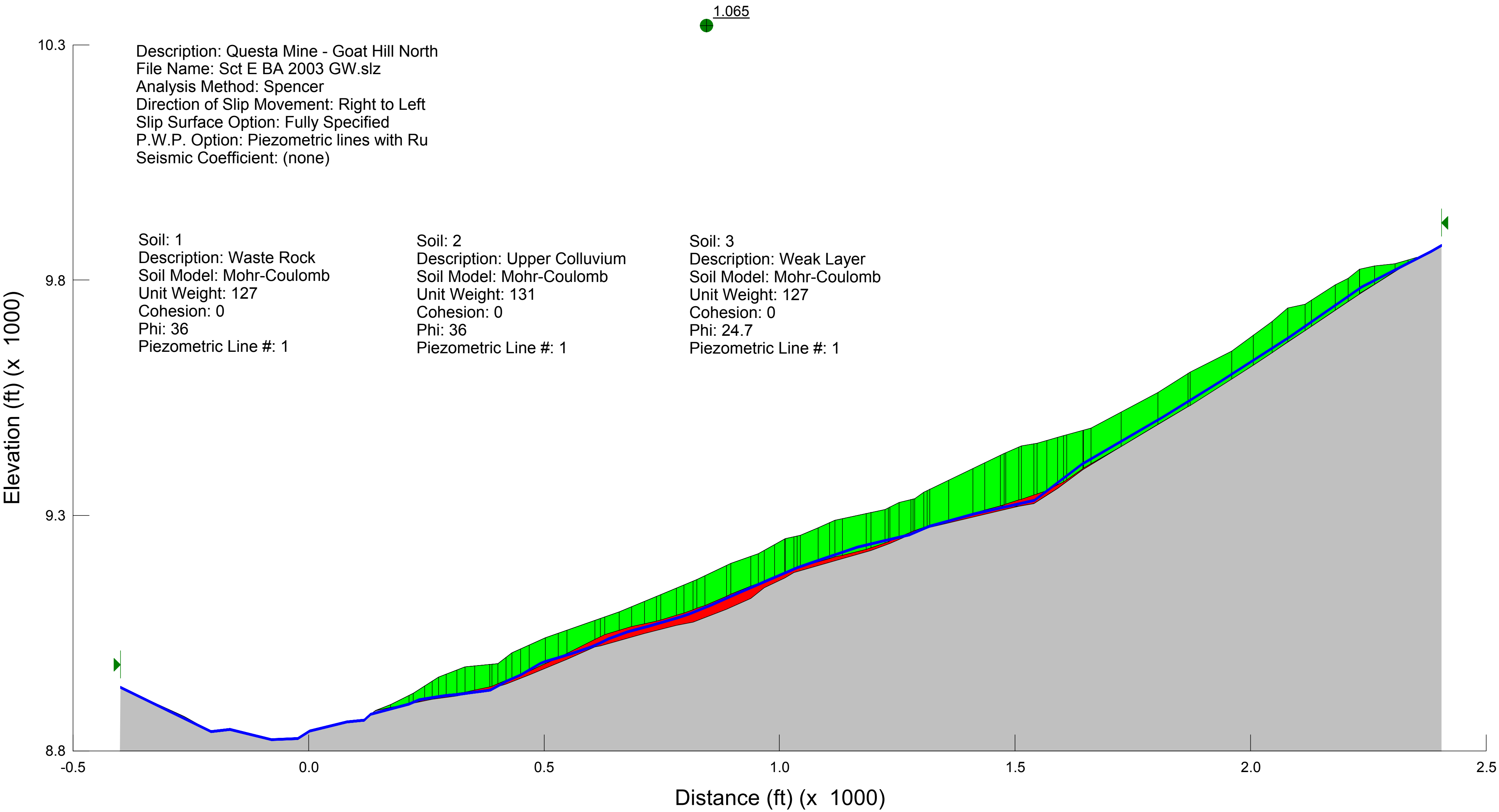

1.000
Description: Questa Mine - Goat Hill North
Comments: Section B - Back Analysis - Alternate Water Table
File Name: Sct B BA 2003 GW.slz
Analysis Method: Spencer
Seismic Coefficient: (none)
Soil: 3
Description: Weak Layer
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 24.46
Piezometric Line #: 1
Soil: 1
Description: Waste Rock
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 2
Description: Upper Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
TH-GN-02
SI-08
TH-GN-04
SI-10
TH-GN-16
SI-24
9.8
9.6
9.4
9.2
9.0
8.8
0.0
0.5
1.0
1.5
2.0
Elevation (ft) (x 1000)
Distance (ft) (x 1000)

000454

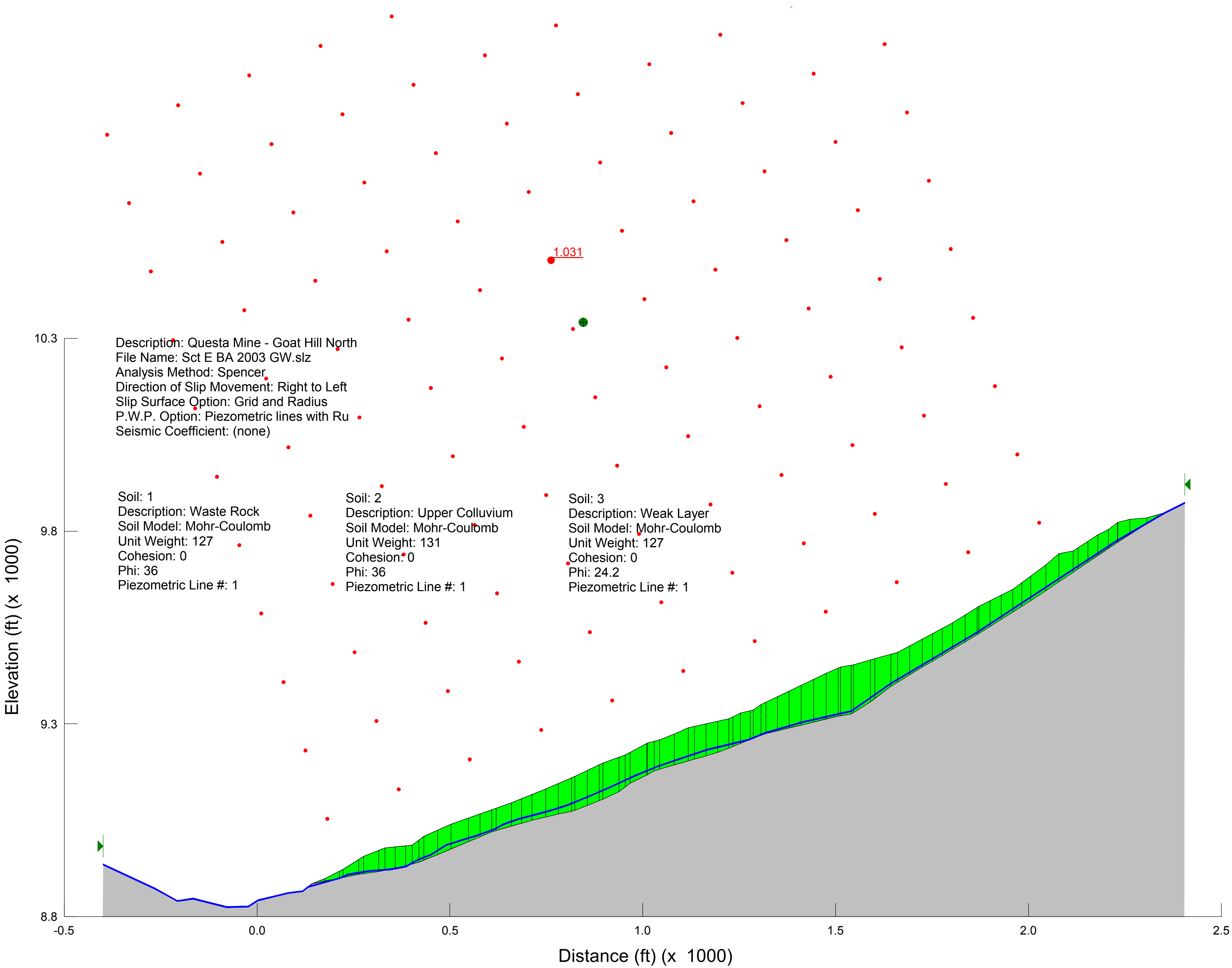


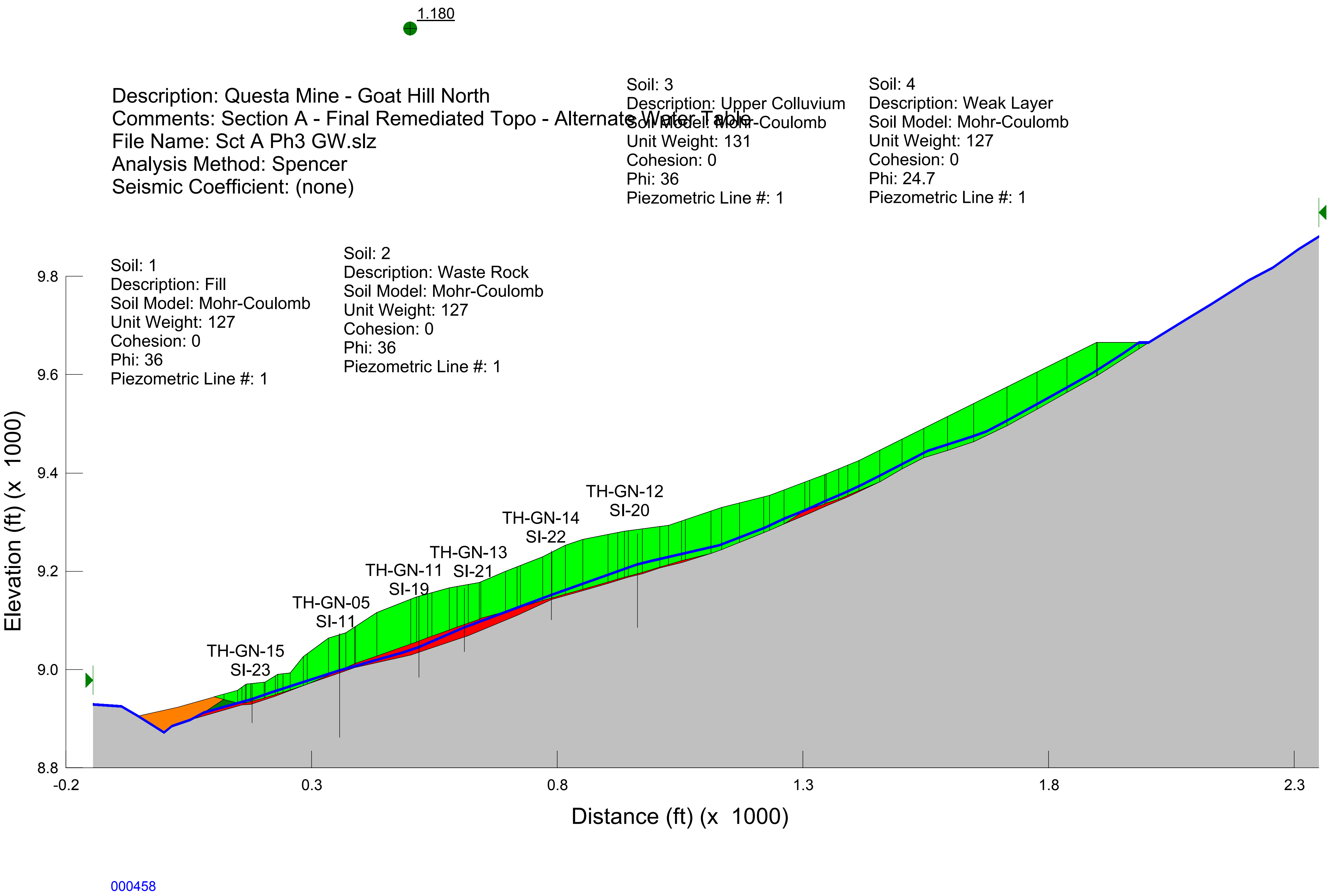
1.000
Description: Questa Mine - Goat Hill North
Comments: Section C - Back Analysis - Alternate Water Table
File Name: Sct C BA 2003 GW.slz
Analysis Method: Spencer
Seismic Coefficient: (none)
Soil: 3
Description: Weak Layer
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 25.9
Piezometric Line #: 1
Soil: 1
Description: Waste Rock
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 2
Description: Upper Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
TH-GN-07
SI-03
TH-GN-03
SI-09
TH-GN-09
SI-17
Elevation (ft) (x 1000)
9.8
9.6
9.4
9.2
9.0
8.8
0.0
0.5
1.0
1.5
2.0
Distance (ft) (x 1000)

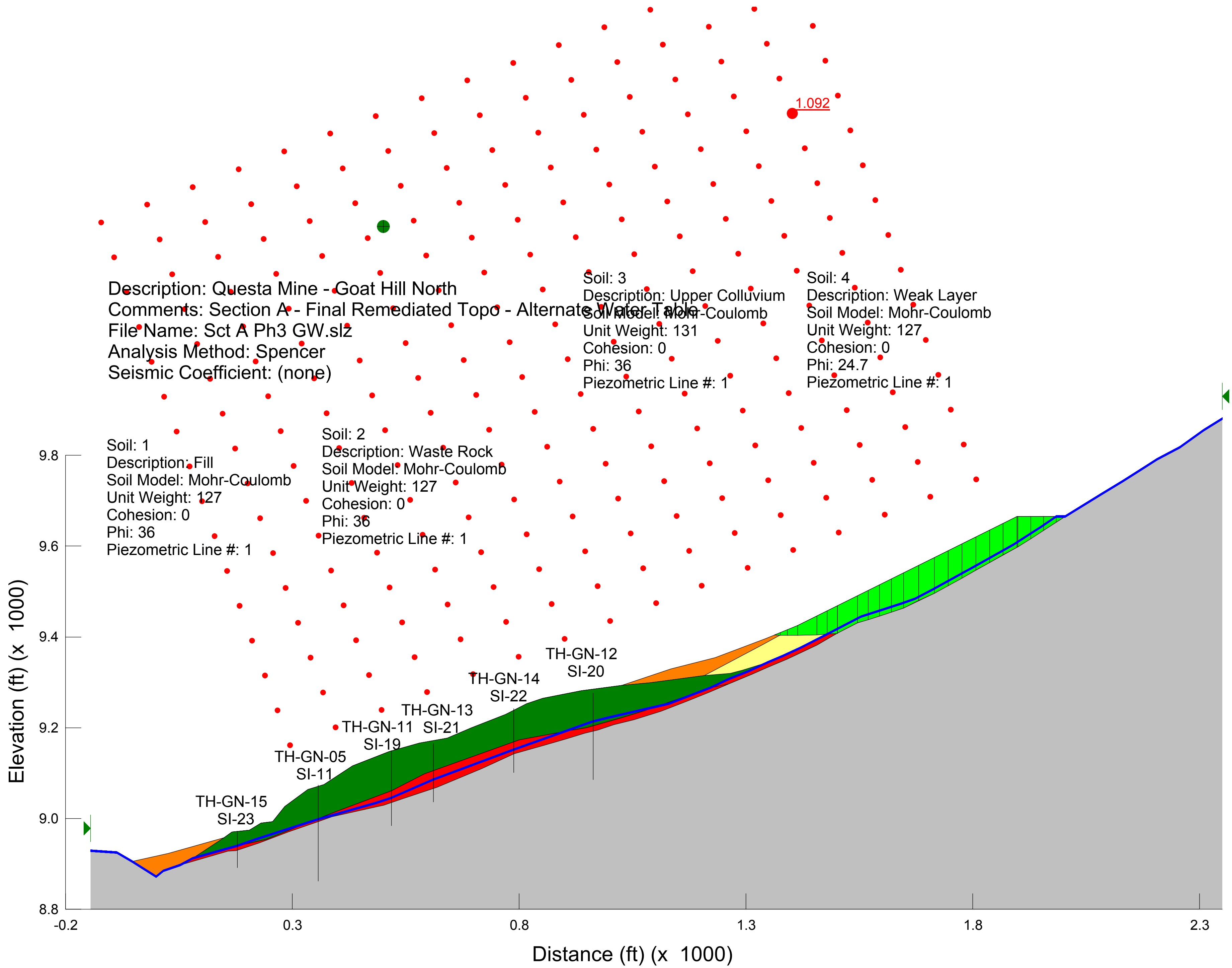
000455


1.065
Description: Questa Mine - Goat Hill North
File Name: Sct E BA 2003 GW.slz
Analysis Method: Spencer
Direction of Slip Movement: Right to Left
Slip Surface Option: Fully Specified
P.W.P. Option: Piezometric lines with Ru
Seismic Coefficient: (none)
Soil: 1
Description: Waste Rock
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 2
Description: Upper Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 3
Description: Weak Layer
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 24.7
Piezometric Line #: 1
10.3
9.8
9.3
8.8
Elevation (ft) (x 1000)
-0.5
0.0
0.5
1.0
1.5
2.0
2.5
Distance (ft) (x 1000)

000456

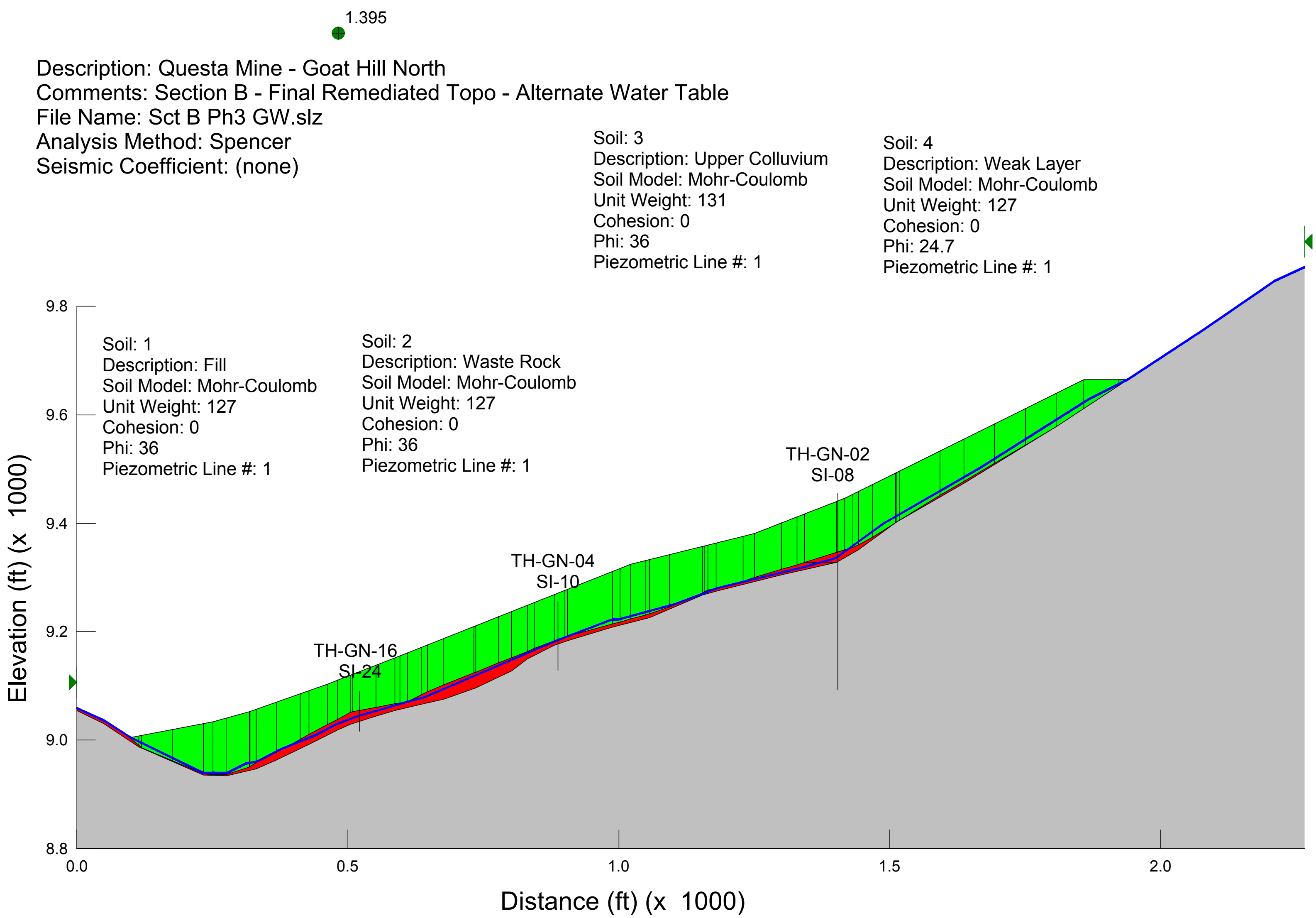

1.031
Description: Questa Mine - Goat Hill North
File Name: Sct E BA 2003 GW.slz
Analysis Method: Spencer
Direction of Slip Movement: Right to Left
Slip Surface Option: Grid and Radius
P.W.P. Option: Piezometric lines with Ru
Seismic Coefficient: (none)
Soil: 1
Description: Waste Rock
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 2
Description: Upper Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 3
Description: Weak Layer
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 24.2
Piezometric Line #: 1
10.3
9.8
9.3
8.8
Elevation (ft) (x 1000)
-0.5
0.0
0.5
1.0
1.5
2.0
2.5
Distance (ft) (x 1000)


1.180
Description: Questa Mine - Goat Hill North
Comments: Section A - Final Remediated Topo - Alternate Water Table
File Name: Sct A Ph3 GW.slz
Analysis Method: Spencer
Seismic Coefficient: (none)
Soil: 3
Description: Upper Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 4
Description: Weak Layer
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 24.7
Piezometric Line #: 1
Soil: 1
Description: Fill
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 2
Description: Waste Rock
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
TH-GN-15
SI-23
TH-GN-05
SI-11
TH-GN-11
SI-19
TH-GN-13
SI-21
TH-GN-14
SI-22
TH-GN-12
SI-20
Elevation (ft) (x 1000)
9.8
9.6
9.4
9.2
9.0
8.8
-0.2
0.3
0.8
1.3
1.8
2.3
Distance (ft) (x 1000)
000458



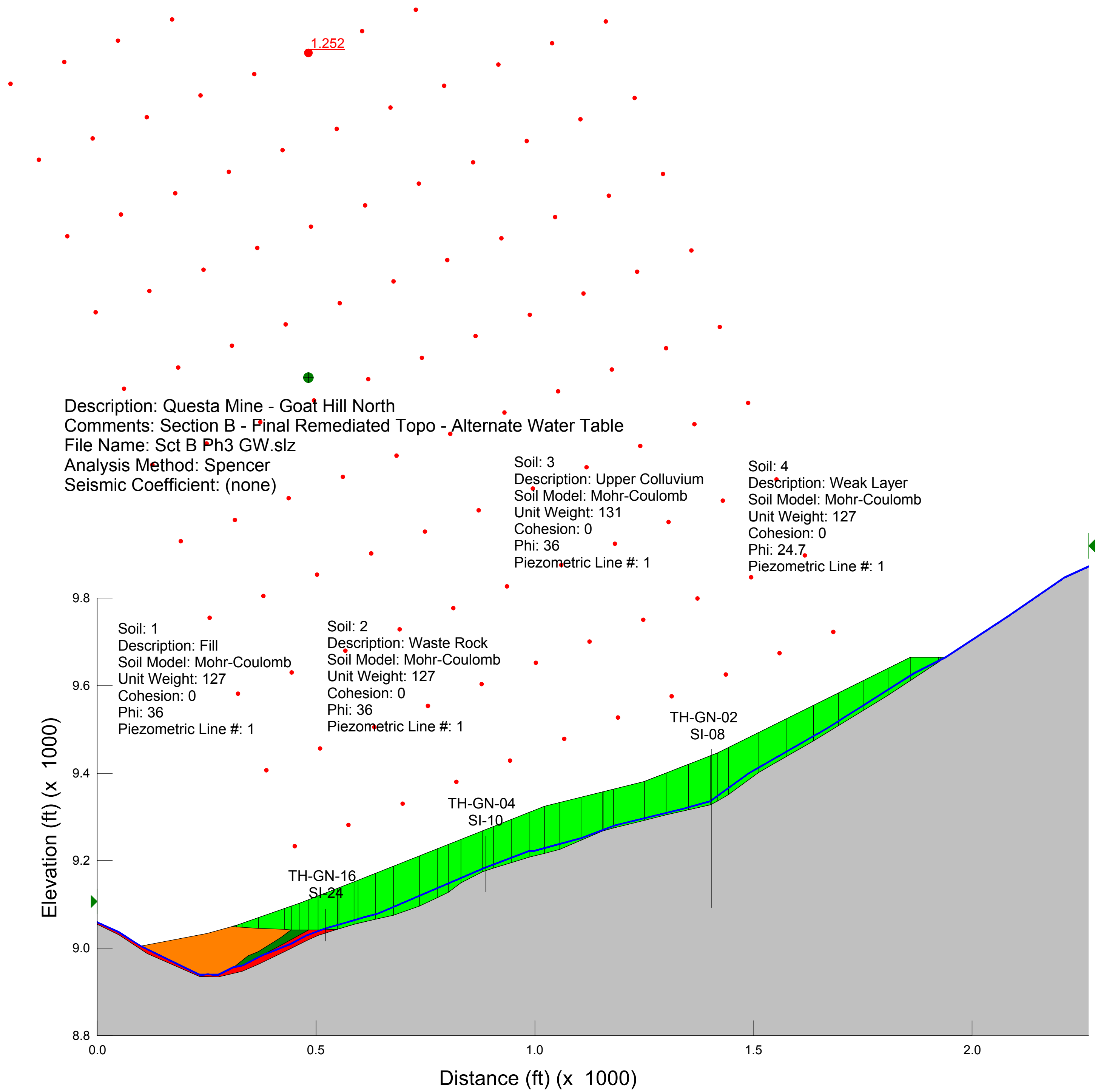
1.092
Description: Questa Mine - Goat Hill North
Comments: Section A - Final Remediated Topo - Alternate Water Table
File Name: Sct A Ph3 GW.slz
Analysis Method: Spencer
Seismic Coefficient: (none)
Soil: 3
Description: Upper Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 4
Description: Weak Layer
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 24.7
Piezometric Line #: 1
Soil: 1
Description: Fill
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 2
Description: Waste Rock
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
TH-GN-12
SI-20
TH-GN-14
SI-22
TH-GN-13
SI-21
TH-GN-11
SI-19
TH-GN-05
SI-11
TH-GN-15
SI-23
Elevation (ft) (x 1000)
9.8
9.6
9.4
9.2
9.0
8.8
-0.2
0.3
0.8
1.3
1.8
2.3
Distance (ft) (x 1000)

000459



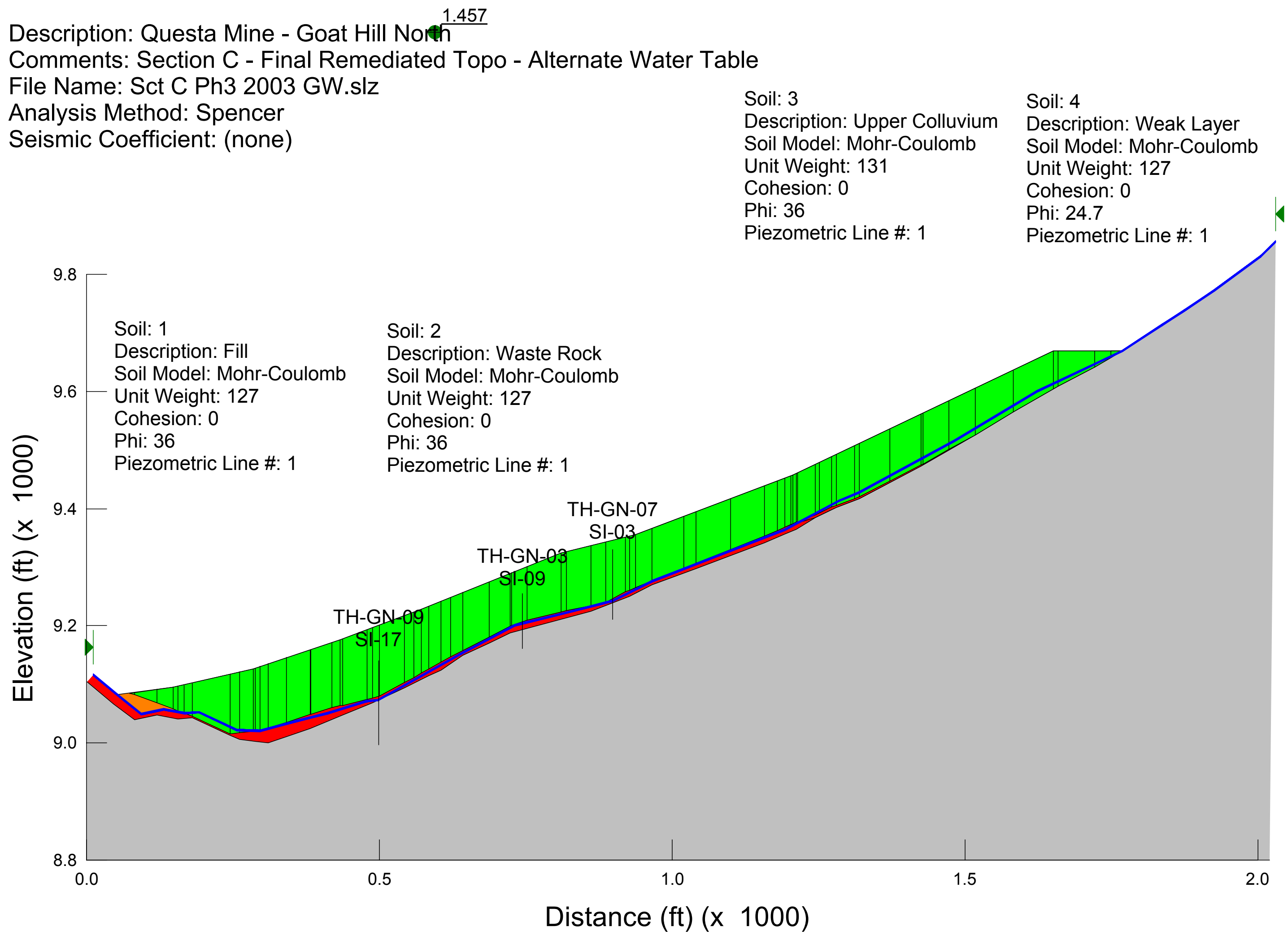
1.395
Description: Questa Mine - Goat Hill North
Comments: Section B - Final Remediated Topo - Alternate Water Table
File Name: Sct B Ph3 GW.slz
Analysis Method: Spencer
Seismic Coefficient: (none)
Soil: 3
Description: Upper Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 4
Description: Weak Layer
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 24.7
Piezometric Line #: 1
Soil: 1
Description: Fill
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 2
Description: Waste Rock
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
TH-GN-02
SI-08
TH-GN-04
SI-10
TH-GN-16
SI-24
Elevation (ft) (x 1000)
9.8
9.6
9.4
9.2
9.0
8.8
0.0
0.5
1.0
1.5
2.0
Distance (ft) (x 1000)

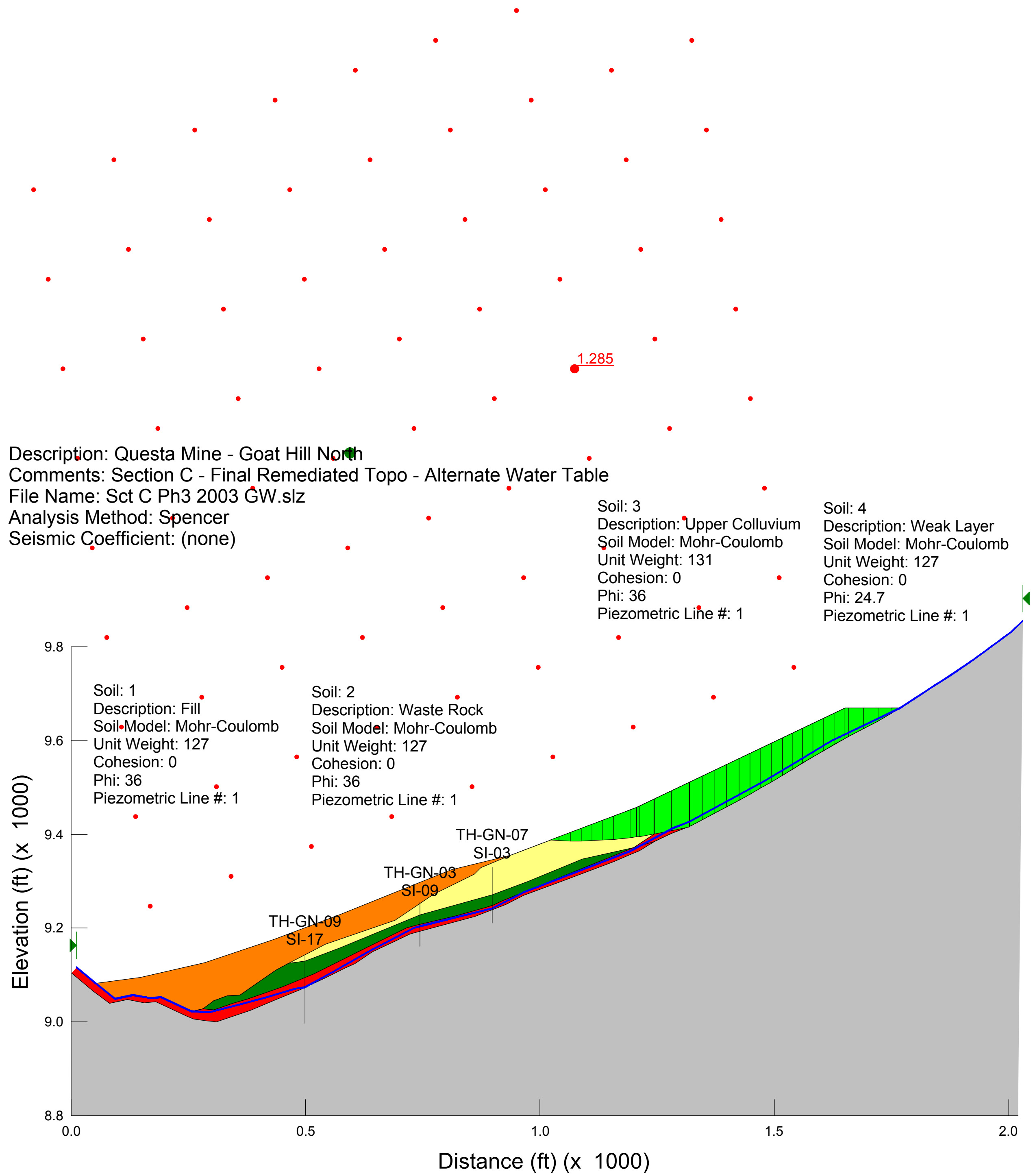
000460



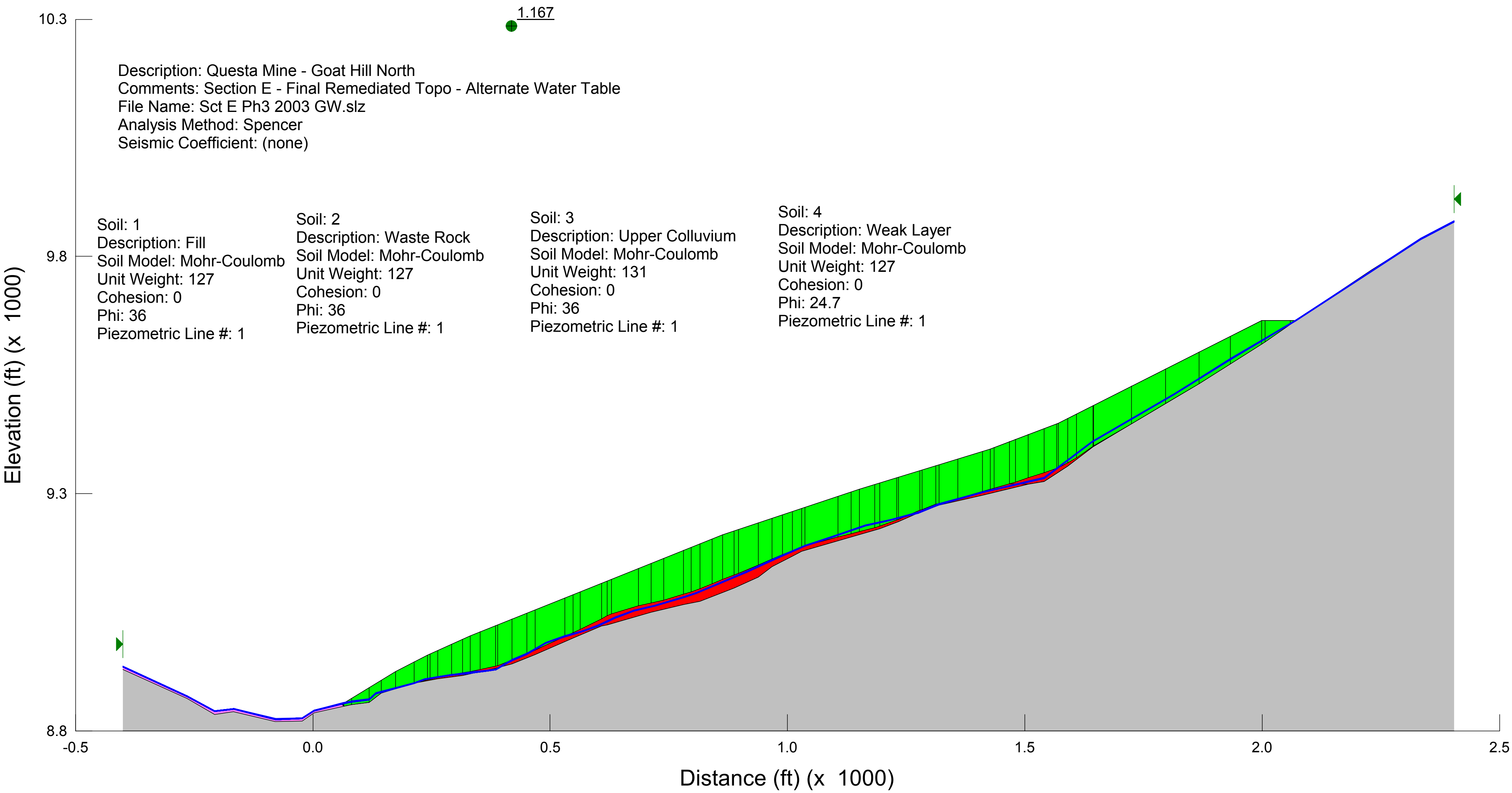
1.252
Description: Questa Mine - Goat Hill North
Comments: Section B - Final Remediated Topo - Alternate Water Table
File Name: Sct B Ph3 GW.slz
Analysis Method: Spencer
Seismic Coefficient: (none)
Soil: 3
Description: Upper Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 4
Description: Weak Layer
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 24.7
Piezometric Line #: 1
Soil: 1
Description: Fill
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 2
Description: Waste Rock
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
TH-GN-02
SI-08
TH-GN-04
SI-10
TH-GN-16
SI-24
9.8
9.6
9.4
9.2
9.0
8.8
Elevation (ft) (x 1000)
0.0
0.5
1.0
1.5
2.0
Distance (ft) (x 1000)

000461

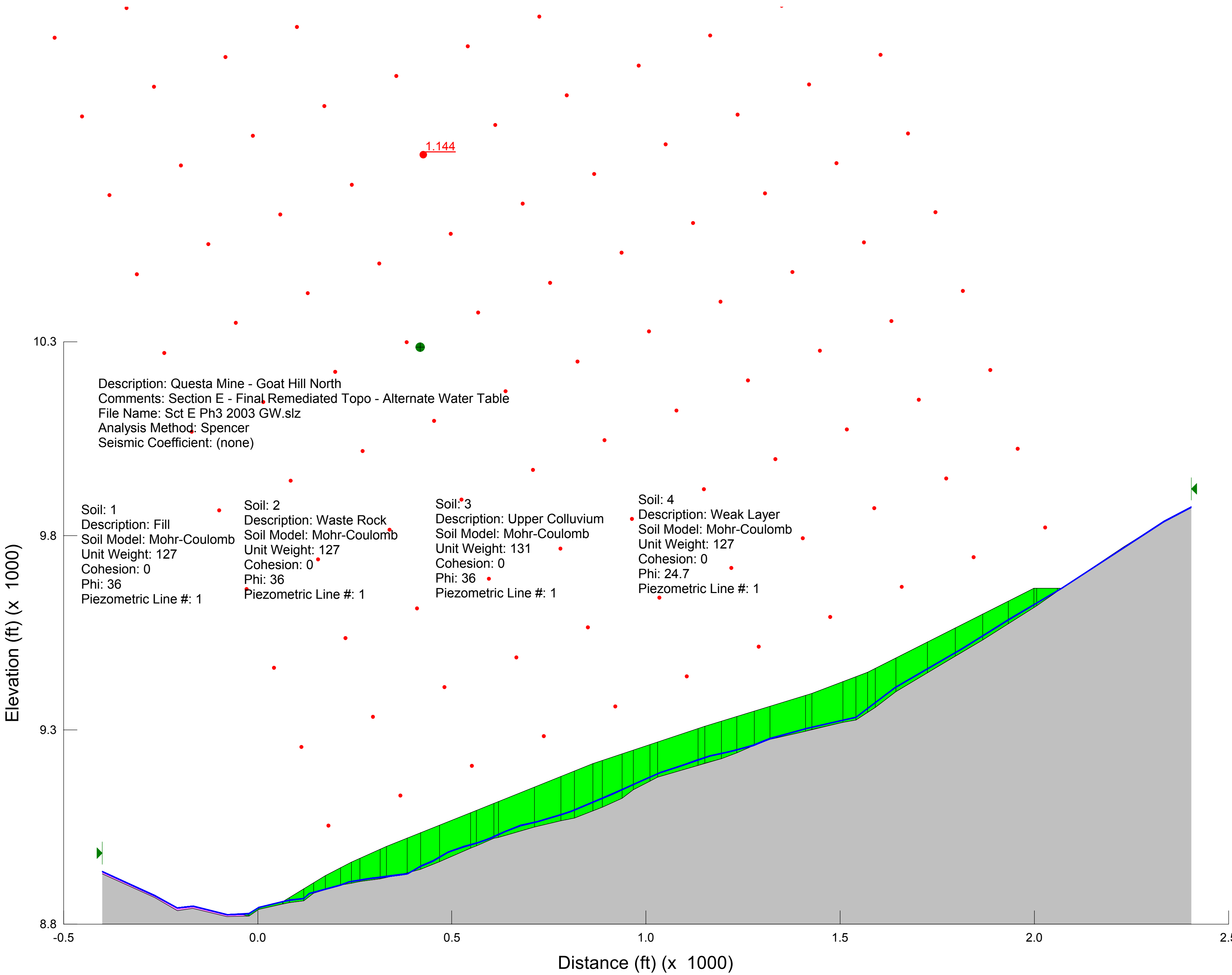

1.457
Description: Questa Mine - Goat Hill North
Comments: Section C - Final Remediated Topo - Alternate Water Table
File Name: Sct C Ph3 2003 GW.slz
Analysis Method: Spencer
Seismic Coefficient: (none)
Soil: 3
Description: Upper Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 4
Description: Weak Layer
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 24.7
Piezometric Line #: 1
Soil: 1
Description: Fill
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 2
Description: Waste Rock
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
TH-GN-07
SI-03
TH-GN-03
SI-09
TH-GN-09
SI-17
9.8
9.6
9.4
9.2
9.0
8.8
Elevation (ft) (x 1000)
0.0
0.5
1.0
1.5
2.0
Distance (ft) (x 1000)


1.285
Description: Questa Mine - Goat Hill North
Comments: Section C - Final Remediated Topo - Alternate Water Table
File Name: Sct C Ph3 2003 GW.slz
Analysis Method: Spencer
Seismic Coefficient: (none)
Soil: 3
Description: Upper Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 4
Description: Weak Layer
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 24.7
Piezometric Line #: 1
Soil: 1
Description: Fill
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 2
Description: Waste Rock
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
TH-GN-07
SI-03
TH-GN-03
SI-09
TH-GN-09
SI-17
Elevation (ft) (x 1000)
9.8
9.6
9.4
9.2
9.0
8.8
0.0
0.5
1.0
1.5
2.0
Distance (ft) (x 1000)

000463


1.167
Description: Questa Mine - Goat Hill North
Comments: Section E - Final Remediated Topo - Alternate Water Table
File Name: Sct E Ph3 2003 GW.slz
Analysis Method: Spencer
Seismic Coefficient: (none)
Soil: 1
Description: Fill
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 2
Description: Waste Rock
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 3
Description: Upper Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 4
Description: Weak Layer
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 24.7
Piezometric Line #: 1
10.3
9.8
9.3
8.8
Elevation (ft) (x 1000)
-0.5
0.0
0.5
1.0
1.5
2.0
2.5
Distance (ft) (x 1000)

000464

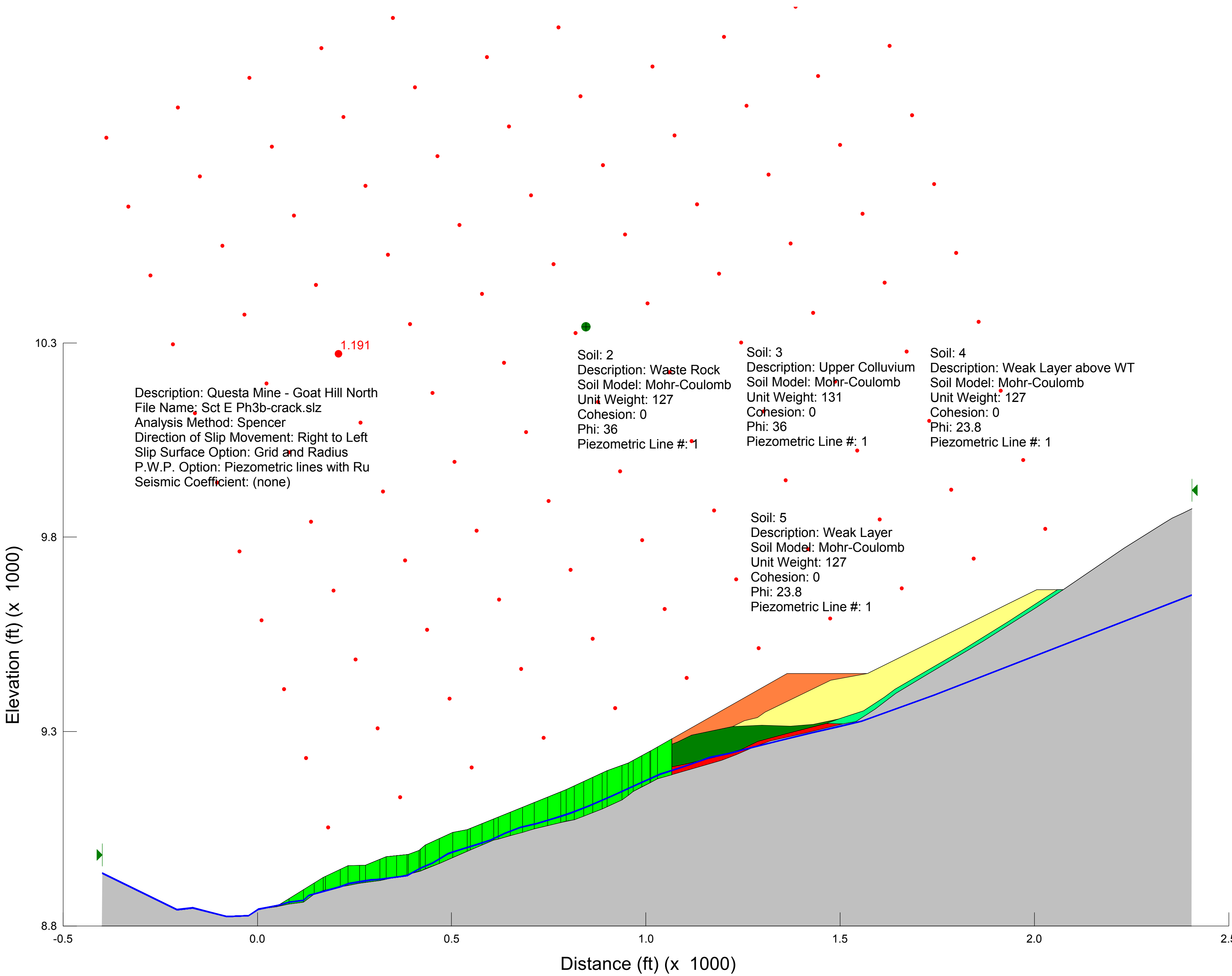


1.144
10.3
Description: Questa Mine - Goat Hill North
Comments: Section E - Final Remediated Topo - Alternate Water Table
File Name: Sct E Ph3 2003 GW.slz
Analysis Method: Spencer
Seismic Coefficient: (none)
Soil: 1
Description: Fill
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 2
Description: Waste Rock
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 3
Description: Upper Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 4
Description: Weak Layer
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 24.7
Piezometric Line #: 1
9.8
9.3
8.8
Elevation (ft) (x 1000)
-0.5
0.0
0.5
1.0
1.5
2.0
2.5
Distance (ft) (x 1000)

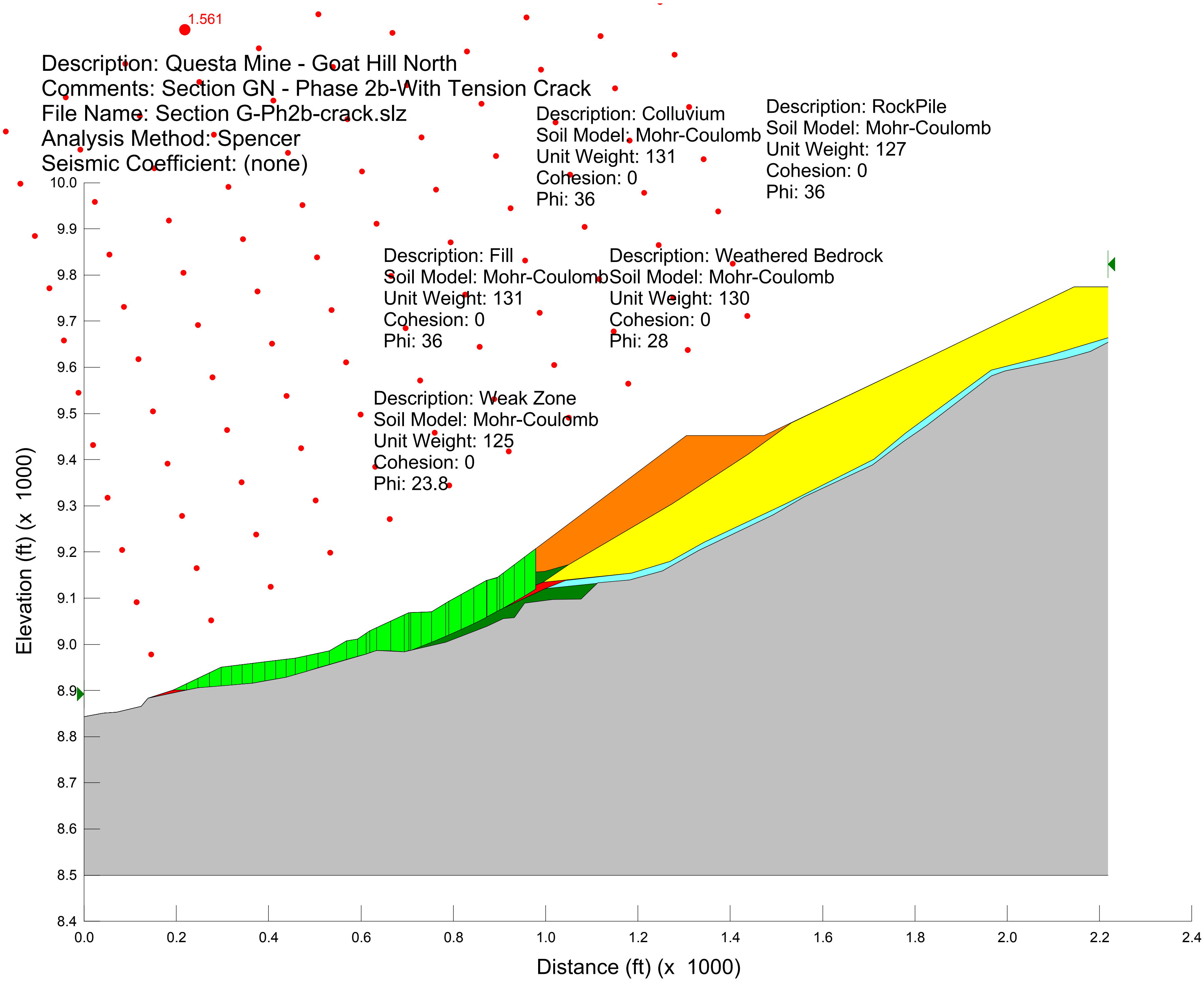
000465

2D Stability Results

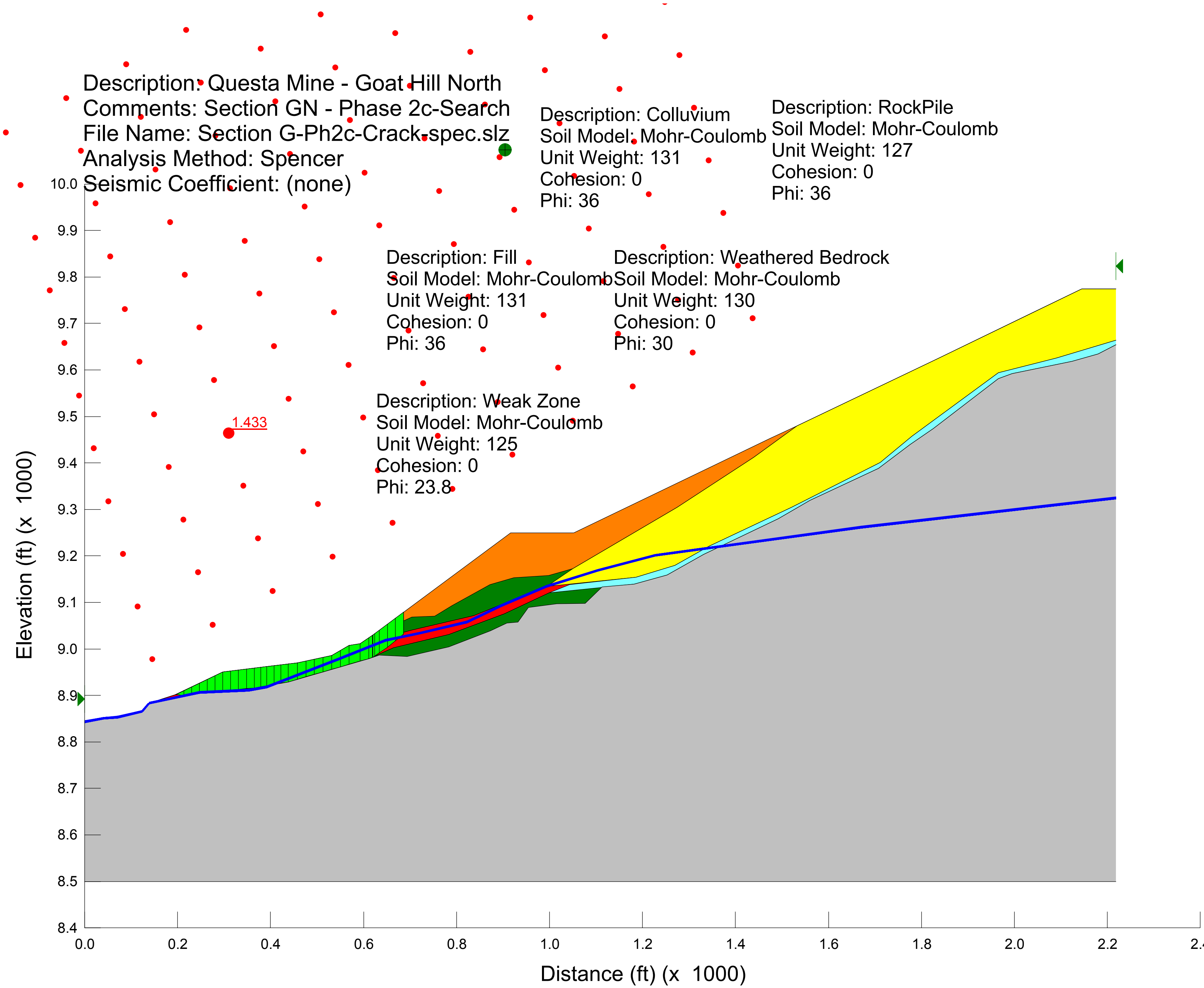
Tension Cracks


1.191
Description: Questa Mine - Goat Hill North
File Name: Sct E Ph3b-crack.slz
Analysis Method: Spencer
Direction of Slip Movement: Right to Left
Slip Surface Option: Grid and Radius
P.W.P. Option: Piezometric lines with Ru
Seismic Coefficient: (none)
Soil: 2
Description: Waste Rock
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 3
Description: Upper Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Piezometric Line #: 1
Soil: 4
Description: Weak Layer above WT
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 23.8
Piezometric Line #: 1
Soil: 5
Description: Weak Layer
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 23.8
Piezometric Line #: 1
10.3
9.8
9.3
8.8
Elevation (ft) (x 1000)
-0.5
0.0
0.5
1.0
1.5
2.0
2.5
Distance (ft) (x 1000)

000467


1.561
Description: Questa Mine - Goat Hill North
Comments: Section GN - Phase 2b-With Tension Crack
File Name: Section G-Ph2b-crack.slz
Analysis Method: Spencer
Seismic Coefficient: (none)
Description: Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Description: RockPile
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Description: Fill
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Description: Weathered Bedrock
Soil Model: Mohr-Coulomb
Unit Weight: 130
Cohesion: 0
Phi: 28
Description: Weak Zone
Soil Model: Mohr-Coulomb
Unit Weight: 125
Cohesion: 0
Phi: 23.8
Elevation (ft) (x 1000)
10.0
9.9
9.8
9.7
9.6
9.5
9.4
9.3
9.2
9.1
9.0
8.9
8.8
8.7
8.6
8.5
8.4
0.0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0
2.2
2.4
Distance (ft) (x 1000)

000468


Description: Questa Mine - Goat Hill North
Comments: Section GN - Phase 2c-Search
File Name: Section G-Ph2c-Crack-spec.slz
Analysis Method: Spencer
Seismic Coefficient: (none)
Description: Colluvium
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Description: RockPile
Soil Model: Mohr-Coulomb
Unit Weight: 127
Cohesion: 0
Phi: 36
Description: Fill
Soil Model: Mohr-Coulomb
Unit Weight: 131
Cohesion: 0
Phi: 36
Description: Weathered Bedrock
Soil Model: Mohr-Coulomb
Unit Weight: 130
Cohesion: 0
Phi: 30
Description: Weak Zone
Soil Model: Mohr-Coulomb
Unit Weight: 125
Cohesion: 0
Phi: 23.8
1.433
Elevation (ft) (x 1000)
10.0
9.9
9.8
9.7
9.6
9.5
9.4
9.3
9.2
9.1
9.0
8.9
8.8
8.7
8.6
8.5
8.4
0.0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0
2.2
2.4
Distance (ft) (x 1000)
000469