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Eagle Project Geotechnical Study

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REPORT ON

**EAGLE PROJECT
GEOTECHNICAL STUDY**

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EXECUTIVE SUMMARY

A geotechnical study has been completed for Kennecott Exploration's Eagle Nickel deposit in the Upper Peninsula of Michigan, USA. Included in this geotechnical study is information regarding a data review of two geotechnical borehole databases for the Eagle Project. A geotechnical model was created using the information in the databases to infer rock mass quality, rock strength, and major discontinuities in the semi-massive/massive sulphide ore lens, the peridotite hangingwall and footwall and the sediments in the main decline and portal area. The geotechnical model was built using the GoCAD software. Representative horizontal and vertical contoured sections of rock mass quality and rock quality designation (RQD) were created in the GoCAD geotechnical model.

Based on these sections, a ground support assessment was completed on the various underground drift infrastructures. The results from the ground support assessment recommended a three-class support system which was based on rock mass quality and drift dimension. The three-class support system was defined as: systematic pattern bolting support for $Q > 4$; systematic pattern bolting support and 4-10 cm of fibre reinforced shotcrete for $Q < 4$ and $Q > 1$; and systematic pattern bolting support, metal screen and 4-10 cm of plain shotcrete for $Q < 1$. Details of the rock support assessment are presented in Section 4.0.

A slope stability assessment was completed using the Mathews Method based on the initial slope dimensions for transverse (15-60 m by 20 m by 30 m high) and longitudinal (5-15 m by 60 m by 30 m high) longhole mining as proposed in the AMEC Scoping Study. The results from the slope stability assessment indicated that the initial slope dimensions for transverse and longitudinal longhole mining are stable. A second slope stability assessment was completed by using 10% and 20% dilution along the walls of the transverse and longitudinal slopes and indicated that these diluted slopes were stable but may require support as well. This slope stability assessment was based on the inferred rock mass quality. The actual in situ rock mass conditions need to be incorporated in the final slope designs.

Several crown pillar assessments were completed using the empirical Scaled Span Method and a limit equilibrium assessment using the CPillar software. The last assessment considered slope geometries supplied by McIntosh within an AutoCAD sequencing model. The results of this assessment were that if the rock mass quality in the crown had a Rock Mass Rating (RMR) = 75, then no slopes could be mined in Level 1 in order to get a factor of safety of 1.9. Alternatively, if the rock mass quality in the crown had an RMR = 85, then three slopes (i.e., all of the high grade zones) could be mined in Level 1 with a factor of safety of 1.8. Details of the crown pillar assessments are presented in Sections 6.0 through 6.4. A discussion on additional crown pillar information requirements and assessment recommendations as the project proceeds underground is presented in Section 6.5.

An initial backfill strength assessment was completed for the primary/secondary stope panels proposed by McIntosh Engineering. This assessment identified the minimum strength required for the primary panels to be self-supporting when they are exposed on the east and west walls. Two methods were used to determine the minimum strengths: 2D vertical slope method and the Mitchell method. Using a factor of safety of 2, a strength requirement of 1.5 MPa (218 psi) was considered the minimum strength for the primary stopes panels (maximum stope dimensions 70 m north-south, 20 m east-west and 34.5 m high).

A review was completed for the proposed mining extraction sequence provided by McIntosh Engineering. A 3-dimensional numerical model was created using the Map3D software based on the proposed stope panel dimensions, extraction sequence and rock mass properties of the ore, waste and backfill types and an estimated stress regime. Two stress parameters were reviewed in the numerical model results: the minor principal stress and the difference between major and minor principal stress (deviatoric stress). The minor principal stress and deviatoric stress were considered to indicate potential issues due to relaxation and high stress, respectively. The Map3D results indicated that major stability problems due to induced stress conditions are not expected with the proposed mining sequence. Details of the modelling are presented in Section 8.0.

TABLE OF CONTENTS

<u>SECTION</u>	<u>PAGE</u>
1.0 INTRODUCTION.....	1
1.1 Objective.....	1
2.0 DATA REVIEW	2
2.1 Borehole Data Review.....	2
3.0 GEOTECHNICAL MODEL.....	4
3.1 Eagle Deposit General Lithology	4
3.2 GoCAD Model.....	4
3.3 Rock Strength	5
3.4 RQD.....	7
3.5 Rock Mass Classification Systems.....	7
3.5.1 RMR Parameter of A1	8
3.5.2 RMR Parameter A2	8
3.5.3 RMR Parameter A3	9
3.5.4 RMR Parameter A4	9
3.5.5 RMR Parameter A5	10
3.6 RMR Summary.....	10
3.7 Major Discontinuity Sets.....	11
3.8 Discrete Features.....	13
4.0 ROCK SUPPORT ASSESSMENT	14
4.1 Empirical Design Chart.....	14
4.2 Mining Methods.....	15
4.3 Support Classification	15
4.4 Bolt Length and Spacing	17
4.5 Ground Support Pressure	17
4.6 Pattern Support.....	18
4.6.1 Ground Support for Main Ramp.....	18
4.7 Kinematic Unwedge Analysis	18
5.0 STOPE SIZING	21
5.1 Matthews Stability Graph Method.....	21
5.2 Longitudinal Longhole Mining.....	22
5.3 Transverse Longhole Mining	22
5.4 Rock Stress Factor (A).....	23
5.5 Joint Orientation (B)	24
5.6 Gravity Adjustment Factor (C).....	25
5.7 Stability Number (N').....	26
5.8 Stope Dilution.....	27
5.9 Stope Sizing Discussion.....	27

TABLE OF CONTENTS (CONTINUED)

6.0	CROWN PILLAR.....	29
6.1	Crown Pillar Stability Assessment	29
6.2	Scaled Span C_s	29
	6.2.1 Eagle Crown Pillar Scaled Span Assessment.....	30
6.3	CPillar Analysis	32
	6.3.1 Eagle CPillar Analysis.....	32
6.4	Additional Crown Pillar Assessment.....	34
6.5	Crown Pillar Discussion and Recommendations	34
7.0	BACKFILL DESIGN	36
8.0	MINING SEQUENCE	38
8.1	Model Geometry and Mining Sequence	38
8.2	Modelling Material Parameters and Stress Regime.....	39
8.3	Modelling Results.....	40
	8.3.1 Minor Principal Stress.....	40
	8.3.2 Deviatoric Stress Results.....	41
8.4	Mining Sequence Discussion	42
9.0	HYDROGEOLOGY	43
10.0	CLOSURE	44

TABLE OF CONTENTS (CONTINUED)**LIST OF TABLES**

Table 1	Boreholes Used in GoCAD Model
Table 2	ISRM Rock Strength Chart
Table 3	Point Load Testing Data and UCS for Eagle Project Database
Table 4	A1 Chart for RMR
Table 5	RQD Rating for RMR
Table 6	Discontinuity Spacing Rating for RMR
Table 7	Discontinuity Condition for RMR
Table 8	Eagle Project Rock Mass Classification
Table 9	Discontinuity Sets for Semi-massive and Massive Sulphides
Table 10	Discontinuity Sets for Peridotite and Feldspathic Peridotite
Table 11	Discontinuity Sets for Siltstone and Sandstone
Table 12	Major and Minor Discontinuity Sets
Table 13	ESR Values for Excavation Categories after Barton et Al. (1974)
Table 14	Drift Dimensions by Mining Method
Table 15	Eagle Project Rock Mass Classification
Table 16	HR for Longitudinal Longhole Mining
Table 17	HR for Transverse Longhole Mining
Table 18	Stability Number A Value
Table 19	Stability Number B Value
Table 20	Stability Number N' vs. Q_{equiv}
Table 21	Eagle Crown Pillar Assessment - Scaled Span (RMR = 75)
Table 22	Eagle Crown Pillar Assessment - Scaled Span (RMR = 85)
Table 23	Eagle Crown Pillar CPillar Analysis (RMR = 75)
Table 24	Eagle Crown Pillar CPillar Analysis (RMR = 85)
Table 25	Eagle Crown Pillar Assessment – Scaled Span Optimized Geometry
Table 26	Minimum Strength of Backfill Design for Primary Stopes
Table 27	Map3D Model Material Properties
Table 28	Map3D Model Stress Regime Properties

TABLE OF CONTENTS (CONTINUED)

LIST OF FIGURES

Figure 1	Eagle Deposit Site Plan
Figure 2	Eagle Deposit: Isometric View of Geotechnical Drillhole Coverage
Figure 3	Plan: RQD and RMR Contouring for 405 Elev.
Figure 4	Plan: RQD and RMR Contouring for 355 Elev.
Figure 5	Plan: RQD and RMR Contouring for 280 Elev.
Figure 6	Plan: RQD and RMR Contouring for 220 Elev.
Figure 7	Plan: RQD and RMR Contouring for 160 Elev.
Figure 8	Plan: RQD and RMR Contouring for 100 Elev.
Figure 9	Traverse Section: RQD and RMR Contouring for 431420E Facing East
Figure 10	Traverse Section: RQD and RMR Contouring for 431457E Facing East
Figure 11	Traverse Section: RQD and RMR Contouring for 431500E Facing East
Figure 12	Traverse Section: RQD and RMR Contouring for 431545E Facing East
Figure 13	Traverse Section: RQD and RMR Contouring for 431585E Facing East
Figure 14	Longitudinal Section: RQD and RMR Contouring for 5177500N Facing North
Figure 15	Longitudinal Section: RQD and RMR Contouring for 5177560N Facing North
Figure 16	Major Discontinuity Sets (All Features) – Semi-Massive Sulphide and Massive Sulphide
Figure 17	Major Discontinuity Sets (All Features) – Peridotite and Feldspar Peridotite
Figure 18	Major Discontinuity Sets (All Features) – Sandstone and Siltstone
Figure 19	Rock Support Design Chart for Spans 5 m to 8 m
Figure 20	Rock Support Design Chart for Intersection Spans 6.2 m to 8.8 m
Figure 21	Unwedge Assessment 5 m High by 5 m Wide
Figure 22	Unwedge Assessment 5 m High by 6.2 m Wide (Intersections)
Figure 23	Unwedge Assessment 4.7 m High by 8.0 and 8.8 m Wide (Intersection)
Figure 24	Transverse and Longitudinal Longhole Stope Face Stability Plot
Figure 25	Transverse and Longitudinal Longhole End Walls Stability Plot
Figure 26	Transverse and Longitudinal Longhole Stope Back Stability Plot
Figure 27	Transverse and Longitudinal Stopping Dilution
Figure 28	C_s Versus Q_{Equiv} for the Scaled Span Assessments Eagle Crown Pillar
Figure 29	CPillar Output for the Eagle Crown Pillar Assessments
Figure 30	Eagle Deposit – Map3D Model – Stope Labelling and Sequencing, View – South
Figure 31	Eagle Project – Map3D Model – Isometric View
Figure 32	Eagle Project – Map3D – Sigma 3 Results, Grid 1 – Steps 11, 13, 16, 20, 23, 27, View – South
Figure 33	Eagle Project – Map3D – Sigma 3 Results – Secondary Panels - Grid 2 – Steps 11, 13, 14, 17, 19, 27 – View – West
Figure 34	Eagle Project – Map3D – Sigma 3 Results – Grid 1, 2, 3 and 4 – North & South Wall
Figure 35	Eagle Project – Map3D – Deviatoric Stress Results – Grid 1, 2, 3 and 4 – North and South Wall

TABLE OF CONTENTS (CONTINUED)

LIST OF APPENDICES

Appendix A	Lithological Sections A1 to A12 Provided by KEX
Appendix B	Rock Mass Classification Systems

1.0 INTRODUCTION

The Eagle Project is a nickel deposit owned by Kennecott Exploration Company (KEX) located in the Upper Peninsula of Michigan, USA. Golder Associates Ltd. (Golder) has been retained by KEX to provide a geotechnical design study as part of an overall prefeasibility study for the Eagle Project. Illustrated on Figure 1 is a plan drawing indicating the location of the portal, the main ramp and main mining zones.

1.1 Objective

The objective of this study is to provide geotechnical design input to assist in the preparation of a prefeasibility study, with an accuracy of $\pm 25\%$, to evaluate the alternatives for developing the Eagle Project.

2.0 DATA REVIEW

The geotechnical logging database (dated January 3, 2005) from the ongoing drilling program at the Eagle deposit has been reviewed. Currently, the data is organised into two Microsoft Access databases based on when they were drilled; pre-2004 and 2004. The databases were created using custom entry forms and consist of tables including those describing cemented joints, open joints, basic geotechnical measurements, major structures and point load tests. Some entries were found to be deficient in information. A list of deficient data has been formulated and communicated to KEX personnel on site.

Geotechnical logging is completed in Marquette by a team of geologists. Drill core is processed by the geologists after it has been boxed on site by the drillers. Logging procedures have been formulated and revised from 2001 to present. This has resulted in a marked improvement in the quality and completeness of the data over this period. Currently, the geologists are logging according to the procedures outlined in “EAGLE PROJECT: DATA COLLECTION AND ANALYSIS PROCEDURES, 2001 – 2003” (Coombes, 2004) which follows standard Rio Tinto and industry recognised geotechnical core logging practices.

Basic geotechnical measurements are given over a 3 m interval (based on run length) and consist of rock type, total core recovery (TCR), solid core recovery (SCR), intact rock strength (R-value) and Rock Quality Designation (RQD). Cemented joint data is based on sets. Data for each set includes the depth interval (from surface along borehole), the number of features, the alpha angle, the beta angle (when available), the type of joint infill and the relative strength of the joint(s). Open joint data is based on sets and the data collected is the same as that for cemented joints with the exception that joint wall condition is logged rather than relative joint strength. Three separate parameters are used to log the joint wall condition: joint roughness, joint alteration and joint infill. Axial and diametric point load test data exists for the majority of the basic rock types. A procedure is currently in place to photograph and catalogue core photos (wet and dry).

2.1 Borehole Data Review

The Microsoft Access databases contain a total of approximately 92 boreholes. The pre-2004 data is comprised of 43 NQ-sized boreholes representing just over 8,700 m of core of which less than 50% is orientated. The 2004 data is comprised of 49¹ boreholes representing over 11,200 m of core of which more than 75% is orientated.

A review of the spatial distribution of the borehole data in GoCAD (Figure 2) indicates the following:

¹ Borehole YD02-07 appears in the pre-2004 database and is updated with more data in the 2004 database.

- Area 1 – 7 boreholes are collared at surface, north of the hangingwall. The boreholes are fan drilled between 50° and 60° toward the south with a 50 m to 250 m spacing between borehole set-ups. These holes provided coverage for the hangingwall at depth and the centre of the deposit;
- Area 2 – 12 boreholes are collared at surface in the footwall of the deposit. The boreholes are fan drilled between 45° and 70° toward the north with a 30 m spacing between borehole set-ups. These holes provided coverage in the footwall, hangingwall and centre of the deposit;
- Area 3 – 30 boreholes are collared at surface in the north side of the crown. The boreholes are fan drilled between 45° and 90° toward the south with a 30 m spacing between borehole set-ups. These boreholes provided coverage of the hangingwall side of the crown, centre of the orebody and the footwall at depth;
- Area 4 – 18 boreholes are collared at surface along the east side of the deposit. The boreholes are fan drilled toward the south with a varying spacing between borehole set-ups. These holes provide coverage along the eastern end of the deposit; and
- Area 5 – 2 boreholes are drilled at surface, which intersect the main decline and 1 borehole was drilled into the portal area.

Based on the drill coverage currently in the database, there is limited data for the south end of the crown pillar, for the main decline (approximate 800 linear metres) and for the portal area. More information in these areas would be prudent when advancing this project beyond the prefeasibility stage. Additional information on the crown pillar is required to plan the upper extent of the mining and to more accurately determine the crown pillar rock quality. Additional information is also required to assess the location and ground condition for the portal and main decline.

It should be noted that soil and rock formations are variable to a greater or lesser extent. The data from individual borehole logs in the database indicate approximate subsurface conditions only at the individual borehole locations. Boundaries between zones on the logs are often not distinct, but rather are transitional and have been interpreted. Subsurface conditions between boreholes are inferred and may vary significantly from conditions encountered at the boreholes.

3.0 GEOTECHNICAL MODEL

The geotechnical model created for the Eagle Project is the base for the majority of the work conducted as part of this geotechnical study. The data used in the geotechnical model is based primarily on the data provided by KEX from their two Microsoft Access databases which contain the exploration drill core information, the Eagle Project Scoping Study Report (Scoping Study) by AMEC (2004) and the lithological block model data provided by KEX.

The geotechnical model defines the rock strength parameters, classifies the rock mass quality and identifies the structural fabric in the footwall, hangingwall and ore zone of the Eagle deposit.

3.1 Eagle Deposit General Lithology

The Eagle deposit ore zone is composed of two semi-massive (SMS) and one massive sulphide (MS) bodies hosted in a peridotite intrusive as illustrated on Figures A1 to A12 in Appendix A. These figures were provided by KEX from their lithological block model and are cut on the same sections as the GoCAD model. Low grade ore zones surrounding the sulphide bodies have been included in the ore zone deposit. Between the upper (355 m to 400 m) and lower mining zones (100 m to 280 m) is a lower grade ore zone which is planned as a sill pillar between the two zones (AMEC Scoping Study). The actual thickness of the sill pillar has not been determined and will be defined when more drilling information is available. In the crown area of the deposit, the ore zone (above 400 m) pinches out into the peridotite intrusive. The peridotite intrusive body is hosted in sedimentary units composed of sandstones and siltstones. For the purpose of this geotechnical study, the rock types for the hangingwall, footwall, and crown pillar are considered to be peridotite and feldspathic peridotite. However, there is a portion of the MS lens that intersects the sediments along the footwall of the peridotite intrusive between 431450E and 431550E and between the 200 m and 250 m elevation. The rock types in the access drifts, main declines, internal ramps and specific underground facilities (underground rock breaker and ore bins) are considered to be siltstone and sandstone.

3.2 GoCAD Model

The rock mass rating portion of the geotechnical model has been completed using the GoCAD software based on the pre-2004 and 2004 Microsoft Access databases. The model contains two fields, RQD and Rock Mass Rating (RMR), which can be queried to produce contoured sections in GoCAD. Thirteen sections were contoured in GoCAD for RQD and RMR and are illustrated on Figures 3 to 15.

A simple algorithm to calculate RQD (on a 3 m interval) for each borehole has been written. The structure of the two databases (pre-2004 and 2004) is the same for the fields required to calculate RQD. The majority of the data (pre-2004 and 2004) contained sufficient data to calculate RQD.

A series of algorithms have been created to calculate four of the five parameters summed to give an RMR rating. RMR has been calculated on a 3 m interval for each hole. The majority of the data in the 2004 database was found to adequately calculate RMR. Of the pre-2004 data, roughly half of the data is lacking all or some of the data required to calculate RMR. Table 1 lists the boreholes that were used in the GoCAD model in order to define the RMR for the Eagle deposit.

TABLE 1
BOREHOLES USED IN GOCAD MODEL

RMR Calculated for Entire Hole	RMR Calculated for Part of Hole	Insufficient Data to Calculate RMR	Database
03EA029 to 03EA043, YD02-24 to YD02-28, YD02-06	YD02-02, YD02-04, YD02-09, YD02-11, YD02-13, YD02-14, YD02-16, YD02-17, YD02-20	YD02-01, YD02-03, YD02-05, YD02-08, YD02-10, YD02-12, YD02-15, YD02-18, YD02-19, YD02-21, YD02-22, YD02-23	Pre-2004
04EA044 to 04EA088, YD02-07	-	-	2004

Thirteen RQD and RMR contour plots have been created in GoCAD to indicate the rock mass quality for the various rock types in the ore, hangingwall and footwall of the deposit. Seven of the plots are horizontal planes created at specific elevations: 100 m, 160 m, 220 m, 280 m, 355 m, and 405 m. Three east-west vertical sections were created along the 5177500 and 5177560 Easting and 6 north-south plots were creating along the 431585, 431545, 431500, 431457 and 431420 Northing. For the ground support requirements and stope stability assessments, the RMR values were converted to Q_{equiv} .

The following relationship was used to determine Q_{equiv} from RMR (Bieniawski, 1976):

- $Q_{equiv} = \text{Exp}[(\text{RMR}-44)/9]$

3.3 Rock Strength

A large number of methods are available for measuring the strength of rock. The most often used and the one that provides a basic means of classifying rock, is the uniaxial compressive strength

(UCS) test. This test is normally carried out in the laboratory using selected core samples. Other methods that are generally employed during the collection of field data include point load testing and qualitative estimates based on the ISRM charts listed in Table 2.

Both point load testing and qualitative estimates of rock strength were completed on drill core from the Eagle Project. Point load testing results from drill core can be converted to UCS using the following formula (Hoek and Brown, 1980):

- $Is = P / D^2$

Where: Is is the point load index;

P is the force required to break the specimen;

D is the diameter of the core;

and:

- $UCS = Is (14 + 0.175D)$

TABLE 2
ISRM ROCK STRENGTH CHART

GRADE	DESCRIPTION	FIELD IDENTIFICATION	APPROX. RANGE OF UCS (MPa)
R0	Extremely weak rock	Indented by thumbnail.	0.25 - 1.0
R1	Very weak rock	Crumbles under firm blows with point of geological hammer, can be peeled by a pocket knife.	1.0 - 5.0
R2	Weak rock	Can be peeled by a pocket knife with difficulty, shallow indentations made by firm blow with point of geological hammer.	5.0 - 25
R3	Medium strong rock	Cannot be scraped or peeled with a pocket knife, specimen can be fractured with single firm blow of geological hammer.	25 - 50
R4	Strong rock	Specimen requires more than one blow of geological hammer to fracture it.	50 - 100
R5	Very strong rock	Specimen requires many blows of geological hammer to fracture it.	100 - 250
R6	Extremely strong rock	Specimen can only be chipped with geological hammer.	>250

Table 3 summarizes the point load test data for various rock types.

TABLE 3
POINT LOAD TESTING DATA AND UCS FOR EAGLE PROJECT DATABASE

ROCK TYPE	AVERAGE Is	STD DEV	QUANTITY OF TESTS	UCS (MPa)
Feldspathic Peridotite	6.6	2.5	151	92
Gabbro	8.5	3.0	2	119
Hornfels	10.4	2.40	17	146
Missourite	1.8	1.0	5	25
Massive Sulphide	4.1	1.9	221	57
Semi-Massive Sulphide	7.9	1.8	262	111
Peridotite	8.6	2.8	272	120
Pyroxenite	9.7	2.8	114	136
Sandstone	4.5	3.0	154	63
Siltstone	5.3	2.6	543	74

3.4 RQD

RQD is determined from the following expression:

$$RQD(\%) = 100 \times \frac{\text{the sum of the lengths of core in pieces equal to or longer than 10 cm}}{\text{length of core run}}$$

Using the GoCAD software, thirteen contour plots were generated throughout the Eagle deposit in order to define the general RQD conditions in the hangingwall, footwall and ore zone. These thirteen RQD plots are illustrated on Figures 3 through 15.

3.5 Rock Mass Classification Systems

Two rock mass classification schemes have been widely used in industry. These are Barton's Q system and Bieniawski/Laubscher's RMR system. Although the systems differ, they rely on similar data to be collected in order to classify rock mass quality. They are based on measurement of intact rock strength, fracture spacing and condition, groundwater pressure and orientation of structures with respect to excavations. The main differences between the Q and RMR system is the Q system places more emphasis on rock structure, while the RMR system places more emphasis on rock strength. It is standard practice to use both systems in parallel in order to improve the definition of rock mass quality.

RMR was used to classify the rock mass based on the Eagle drill core data. The formula for RMR is given as:

$$\text{RMR} = \text{A1} + \text{A2} + \text{A3} + \text{A4} + \text{A5}$$

With the parameters defined as:

A1 – Uniaxial Compressive Strength (UCS) or Point Load Strength;

A2 – RQD;

A3 – Spacing of discontinuity features;

A4 – Condition of discontinuity feature; and

A5 – Groundwater.

References describing RMR in more detailed can be found in Appendix B.

Thirteen RMR contour sections were created in GoCAD in order to indicate the minimum and typical rock mass quality in the Eagle deposit. These contour plots are illustrated on Figures 3 to 15.

3.5.1 RMR Parameter of A1

A1 or the intact rock strength rating was assigned according to the rock type of each run. The point-load index has been calculated and averaged for each of the rock types tested (Table 3). A1 was assigned according to Table 4.

TABLE 4
A1 CHART FOR RMR

Point-load strength index (MPa)	>8	4 – 8	2 – 4	1 – 2
Rating	15	12	7	4

Point-load testing data indicates that the intact rock strength of most rock types within and immediately around the Eagle deposit is moderately high with an A1 rating of 12 or 15. Some minor rock types, assigned less frequently, were not point load tested. These were assigned an A1 rating of rock types that were point load tested which were similar in lithology.

3.5.2 RMR Parameter A2

The RQD rating (A2) was assigned for each run using a smoothing algorithm as defined in Table 5.

TABLE 5
RQD RATING FOR RMR

RQD	<i>A2</i>
< 25	3
25 – 40	$ (RQD - 16) / 3 $
40 – 100	$ RQD / 5 $

3.5.3 RMR Parameter A3

The discontinuity spacing (S) was calculated by dividing the number of discontinuities per run by the length of the run. A3 was then assigned for each run using a smoothing algorithm as defined in Table 6.

TABLE 6
DISCONTINUITY SPACING RATING FOR RMR

<i>A3</i>	
Spacing, S	<i>R₃</i>
0 – 0.05 m	$R_3 = 5$
0.05 m – 0.5 m	$R_3 = 30S^{0.6} $
0.5 m – 3 m	$R_3 = 18 + 4S $
> 3 m	$R_3 = 30$

A change in the field structure within the pre-2004 database complicated the process of calculating the discontinuity spacing in the RMR field of the geotechnical model. The consistency and completeness of the 2004 database simplified the process of calculating RMR.

3.5.4 RMR Parameter A4

Discontinuity condition was described using three parameters in the database: joint infilling (Oji), joint alteration (Oja) and joint roughness (Ojr). The combination of these three parameters defined the A4 rating as listed in Table 7.

TABLE 7
DISCONTINUITY CONDITION FOR RMR

Precedence	Oji	defer	defer	1, 2, 4	3, 5, 6, 7	8
	Oja	defer	defer	1	2	-
	Ojr	1, 4	2, 5, 7	2, 5, 7	3, 6, 8, 9	-
	Rating	25	20	12	6	0

3.5.5 RMR Parameter A5

The RMR parameter A5 represents the joint water condition. Based on current information, the A5 parameter has been estimated to represent a dry joint water condition which corresponds to a rating of 15.

3.6 RMR Summary

Thirteen RMR contour plots have been generated in GoCAD as illustrated on Figures 3 to 15. A summary of the minimum RMR and typical RMR values for the hangingwall, footwall and ore zone for drift and fill mining, transverse and longitudinal longhole mining and the development for the main decline and access drifts are summarized in Table 8 including their Q_{equiv} .

TABLE 8
EAGLE PROJECT ROCK MASS CLASSIFICATION

Mining Method	RMR Minimum	RMR Typical	Q_{Equiv} Minimum	Q_{Equiv} Maximum
Drift and Fill Ore	60	75	5.9	31.3
Drift and Fill F/W	60	70	5.9	18.0
Drift and Fill H/W	70	75	18.0	31.3
Transverse Ore	70	75	18.0	31.3
Transverse F/W	70	80	18.0	54.6
Transverse H/W	70	75	18.0	31.3
Longitudinal Ore	70	80	18.0	54.6
Longitudinal H/W	70	80	18.0	54.6
Longitudinal F/W	70	80	18.0	54.6
Decline and Access Drifts	60	75	5.9	31.3

Typical rock mass conditions for the ore zone, footwall and hangingwall, based on the current geotechnical database, indicate rock quality in the “Good” to “Very Good” category. There are some isolated lower rock mass quality areas, “Fair”, located within the ore zone and the footwall of the UZ predominantly on the eastern side of the deposit possible related to the fault zone defined in that area as illustrated on Figures 3 and 4.

Rock mass classification has been calculated at specific data locations (at individual drills runs) and all information between data locations has been interpreted. The interpretation between data locations has been contoured in plans and sections in GoCAD using kriging as the interpretation method. Therefore, actual conditions encountered during excavation will vary from the interpretation between data locations.

3.7 Major Discontinuity Sets

Discontinuity data has been reviewed from the geotechnical database in order to identify major and minor discontinuity sets in each rock type. The discontinuity data was analysed using the DIPS (Rocscience, 2003) software which plots discontinuity data in two dimensional stereographic projection plots. Six rock types were analysed: semi-massive sulphide, massive sulphide, peridotite, feldspathic peridotite, sandstone and siltstone. Stereographic pole contour plots for each rock type are illustrated on Figures 16 to 18 and listed in Tables 9 to 11.

TABLE 9
DISCONTINUITY SETS FOR SEMI-MASSIVE AND MASSIVE SULPHIDES

SMS			MS		
ID No.	Dip	Dip Direction	ID No.	Dip	Dip Direction
1	2°	272°	1	38°	178°
2	83°	360°	2	30°	084°
3	70°	021°	3	87°	357°
4	80°	344°	4	88°	145°
			5	55°	348°
			6	13°	280°

TABLE 10
DISCONTINUITY SETS FOR PERIDOTITE AND FELDSPATHIC PERIDOTITE

Peridotite			Feldspathic Peridotite		
ID No.	Dip	Dip Direction	ID No.	Dip	Dip Direction
1	12°	069°	1	3°	261°
2	77°	009°	2	74°	184°
3	81°	341°	3	72°	337°
4	42°	038°	4	53°	034°
5	44°	268°			
6	74°	312°			

TABLE 11
DISCONTINUITY SETS FOR SILTSTONE AND SANDSTONE

Sandstone			Siltstone		
ID No.	Dip	Dip Direction	ID No.	Dip	Dip Direction
1	2°	115°	1	2°	123°
2	46°	196°	2	51°	201°
3	43°	074°	3	60°	161°
4	70°	295°			

Based on above assessment the following major and minor discontinuity sets have been summarized in Table 12.

TABLE 12
MAJOR AND MINOR DISCONTINUITY SETS

Set Number	Set Type	Dip	Dip Direction	Rock Types
J1	Major	2°-12°	069°-272°	All Rock Types
J2	Major	70°-87°	295°-021°	All Rock Types
J3	Minor	38°-51°	178°-201°	MS, Sandstone and Siltstone
J4	Minor	30°-53°	034°-084°	Massive Sulphide, Peridotite, Feldspathic Peridotite and Sandstone

The dip direction for both the horizontal set, J1, and the near vertical set, J2, is quite varied because of the scatter in the data. Additionally, the siltstone and sandstone data sets are highly variable with only two well defined discontinuity sets. The dominant feature type (>95% of data)

for each rock type is joints with a small amount of the feature types classified as bedding and foliation which occur primarily in siltstone, sandstone and semi-massive sulphide. However, when these features were plotted separately, they did not create any new discontinuity sets.

3.8 Discrete Features

A discrete fault plane feature has been included in the GoCAD model which was provided by AMEC study. This fault plane is located in the eastern and center portions of the deposit approximately between the 431520E and 431560E and is represented as a green linear feature on Figures 3 to 8. The estimated strike of the fault plane is 010°/190° azimuth and the dip is subvertical. RMR values around the fault plane are 60 in the Upper Zone (UZ) and between 70 and 80 in the Lower Zone (LZ).

Based on the information in the two Microsoft Access databases, there have been other discrete structural features identified in the Eagle deposit. These discrete features have been stored in a separate table of the database instead of being included in the main database. A review of these discrete features indicated that there are three types of structural features: broken core zones, shear zones and fault gouge zones. The broken core zones (1 to 7 m core length) make up the majority of the discrete features compared to the gouge zones (0.1 to 0.4 m core lengths) and the shear zones (1 to 4 m core length).

These structural features identified during the logging have not been incorporated into the GoCAD model. The current data density is not sufficient to interpolate these features. This should be considered at a later date when more data is available or when mapping becomes available during underground development.

4.0 ROCK SUPPORT ASSESSMENT

The design methodology used by Golder for the development of the recommended ground support for the Eagle Project included the following:

- Empirical and analytical support design using a rock mass quality approach;
- Analytical and empirical formulae to determine suitable bolt length and spacing; and
- Analysis of the stability of potential wedges using the recommended ground support.

Based on this assessment, a minimum rock support class system will be recommended that will use different support type categories on the measured rock mass conditions and the drift dimensions.

4.1 Empirical Design Chart

The empirical support design approach based on Barton et al. (1974) was used for the Eagle Project and employs a chart that relates rock mass quality and excavation span to suitable ground support which is based on engineering experience and precedence (Figure 19). The horizontal axis of the chart is the rock mass rating based on the Q system. The vertical axis of the chart is the Equivalent Dimension (De) which is the ratio of the excavation span (in metres) and the Excavation Support Ratio (ESR). The ESR is a general factor of safety (FOS) term and is lowest for permanent openings and highest for temporary mine openings. Table 13 lists the ESR values suggested by Barton et al. (1974) for design of various types of underground openings. The ESR for the Eagle Project was classified as 1.6, which is the rating for permanent mine openings.

TABLE 13
ESR VALUES FOR EXCAVATION CATEGORIES
AFTER BARTON ET AL. (1974)

Excavation Category	ESR Value
Temporary mine openings	3 to 5
Permanent mine openings, water tunnels for hydro power (excluding high pressure penstocks), pilot tunnels, drifts and heading for large excavations	1.6
Storage rooms, water treatment plants, minor road and railway tunnels, surge chambers and access tunnels	1.3
Power stations, major road and railway tunnels, civil defence chambers and portal intersections	1.0
Underground nuclear power stations, railway stations, sports and public facilities and factories	0.8

4.2 Mining Methods

The Eagle Nickel Deposit is divided into the UZ and the LZ. The UZ is a flat-lying tabular deposit located between the 355 m and 385 m Levels. The approximate ore width of the UZ is 80 m to 90 m and the strike is 170 m. The LZ is a sub-vertical wedge-shaped deposit located between the 100 m and 280 m Levels. The approximate ore width of the LZ between 160 m and 280 m Levels is from 15 m to 60 m, which then narrows below the 160 m Levels from 5 to 15 m.

The mining method that has been proposed (AMEC Scoping Study Support, 2004) for the UZ is drift and fill. The mining methods that have been proposed for the LZ are longitudinal longhole mining below the 160 m Level and transverse longhole mining above the 160 m Level.

The ramp, access and development drift dimensions that have been proposed for the development of the Eagle Project are summarised in Table 14.

TABLE 14
DRIFT DIMENSIONS BY MINING METHOD

Mining Method	Longitudinal Longhole	Transverse Longhole	Drift and Fill
Location	LZ (lower one-third)	LZ (upper two-third)	UZ
Mining Levels	100 m to 160 m	160 m to 280 m	355 m to 385 m
Ore Thickness	5 m to 15 m	15 to 60 m	80 m to 90 m
Strike Length	135 m	135 m	170 m
Mining Panel Strike Length	60 m (with a temporary central rib 15 m long)	15-20 m primary and 20 m secondary	170 m (60 west primary, 80 m east primary and 30 m pillar)
Main Ramp Decline Dimension	5 m high by 5 m wide	5 m high by 5 m wide	5 m high by 5 m wide
Access Drift Dimensions	4.7 m high by 5 m wide	4.7 m high by 5 m wide	4.7 m high by 5 m wide
Development Drift Dimensions	4.7 m high by 8 m wide (nominal)	4.7 m high by 5 m wide	5 m high by 5 m wide
Maximum 3-way Intersection Dimension	4.7 m high by 8.8 m diameter wide	4.7 m high by 6.2 m diameter wide	4.7 m high by 6.2 m diameter wide

4.3 Support Classification

Based on the proposed mining methods for the Eagle deposit, the typical spans that will be developed underground for access drifts, ramps (internal and main decline) and development

drifts will be 5 m wide to a maximum of 8 m in the longitudinal stopes and a height of 4.7 m to 5.0 m. Additionally, three-way intersections will be required for access to the levels and stopping areas with estimated maximum excavation spans between 6.2 m (two 5 m wide intersection drifts) and 8.8 m (5 and 8 m wide intersecting drifts) diameter and heights between 4.7 m and 5.0 m.

Based on the above excavation dimensions, the D_e is estimated between 3.1 (5 m/1.6) and 5 (8 m/1.6) for the majority of the underground drifts and between 3.9 and 5.5 for the three-way intersections. Primarily, ore development will be in SMS, MS and peridotites and waste development will be in peridotites and sedimentary units (sandstone and siltstone). The current database is primarily focused on the ore deposit and therefore has a majority of information pertaining to the ore, footwall and hangingwall and is summarized in Table 8. Geotechnical data for the waste development (sedimentary units) is very limited. There are currently only three boreholes in the main decline and portal area. For the purpose of this geotechnical study, the lower rock mass property values in the footwall ($RMR = 60$ or $Q_{Equiv} = 5.9$) has been used as an initial estimate of the rock mass quality for the waste development drifts.

Illustrated on Figures 19 and 20 are ground support design charts based on Q_{Equiv} versus D_e for drifts with 5 m and 8 m spans and intersections with 6.2 m and 8.8 m spans.

Three ground support classes have been outlined on each chart and are defined as follows:

- Class 1 ($Q > 4$) – Spot bolting or Pattern Support Bolting;
- Class 2 ($Q < 4$ and $Q > 1$) – Pattern Support Bolting and 4-10 cm of fibre reinforced shotcrete; and
- Class 3 ($Q < 1$) – Pattern Support Bolting, metal screen and 4-10 cm of plain shotcrete.

Based on the current interpretation, the majority of the drift development (sediments with a $Q_{Equiv} = 5.9$ or “Fair”) is in the minimum ground support Class 1. The current data for the drift development (i.e., main decline and internal ramp) area is composed of three boreholes. Since unknown ground conditions exist between these boreholes, ground support classes have also been included for lower rock mass qualities. Therefore, excavations with rock qualities with a $Q < 4$ and $Q > 1$ (“Poor”) will be classified as Class 2 and rock qualities with a $Q < 1$ (“Very Poor” to “Exceptionally Poor”) will be classified as Class 3. For the purposes of the prefeasibility study, it would be reasonable to consider that approximately 70% of the underground development will be Class 1, 20 % will be Class 2 and 10% will be Class 3.

During excavation, actual ground conditions will need to be determined on a regular basis in order to assign the required ground support class. Also, the support system may need to be modified in specific locations in order to support rock wedges in the wall or back that may be

present due to intersecting discontinuities. The generation of rock wedges from the intersecting discontinuities in the rock mass and required support is discussed in the UNWEDGE assessment section (Section 4.7).

Additional underground infrastructures will be constructed during the development of the Eagle Project and include: the main ramp; raises for ventilation; ore haulage; backfill; emergency egress and production mining (slot raises); crusher chambers; and ventilation fan chambers. The ground support requirements for these various underground infrastructure facilities will need to be designed individually based on their dimensions, rock mass conditions, excavation type and expected duration of use.

4.4 Bolt Length and Spacing

In order to determine the necessary bolting requirements for the support classes the 1/3 bolting span and the 1/2 bolt spacing rules have been employed. The 1/3 bolting span rule suggests that the ground support length should be at least 1/3 of the maximum span of the excavation. The 1/2 bolt spacing rule suggests that the maximum spacing of bolts be half the length of the bolts. Therefore, based on these relationships, bolting patterns for 5 m and 8 m span drifts and 6.2 m and 8.8 m diameter drift intersections are presented in Table 15.

TABLE 15
EAGLE PROJECT ROCK MASS CLASSIFICATION

Drift Dimensions	Support Length (m)	Support Spacing (m)
5 m	1.7	0.85
6.2 m	2.1	1.05
8 m	2.7	1.35
8.8 m	2.9	1.45

For operational efficiency, it is recommended that two types of bolt lengths be considered: 1.8 m for 5 m drifts (including intersections) and 2.7 m for 8 m drifts.

4.5 Ground Support Pressure

To identify the required support pressure necessary for the primary support, the following support pressure relationship developed by Barton (1974) was used:

$$\text{Roof Support Pressure (tonnes/m}^2\text{)} = 20/J_r \times Q^{-1/3}$$

The J_r is the joint roughness variable used in the calculation of Q . Based on a review of the Eagle database, the J_r value varies between slickensided and planar (0.5) to very rough and undulating (3) with a typical value being rough and planar (1.5). Using the range of minimum Q_{Equiv} values summarized in Table 8, 5.9 to 18.0, the range of minimum roof support pressures vary between 5.1 and 7.4 tonnes/m².

4.6 Pattern Support

Therefore, the pattern support bolting recommended for Classes 1, 2 and 3 are 1.8 m (6 ft) long resin rebar bolts for 5 m wide and 4.7 to 5 m high excavation drifts on a 1.5 m by 1.5 m pattern. Drifts 8 m wide will require 2.7 m (9 ft) long resin rebar bolts on a 1.5 m by 1.5 m pattern. Bolt support capacity has been considered to be a minimum of 20 tonnes. Bolts installed half-way up the wall to the back may be necessary depending on the ground conditions and the probability of rock blocks sliding from the walls as described in Section 4.7. Spans in excess of 8 m should be evaluated on a site-specific basis.

4.6.1 Ground Support for Main Ramp

Based on the Scoping Study (AMEC, 2004), a ground support program was recommended for the main ramp. This proposed main ramp dimensions will be 5.0 m wide by 5.0 m high in order to accommodate the mobile equipment. The ground support recommended in the AMEC report for the main ramp included 1.8 m long rockbolts in the back and 1.5 m long bolts in the walls using a 1.5 m by 1.5 m pattern. In the intersection areas, ground support will include 2.4 m long resin grouted rebar. Galvanized wire mesh screen will be installed as required.

Based on the previous discussion, the pattern support bolting (Section 4.6) recommended for Classes 1, 2 and 3 could also be used in the ramp. Again, wire mesh can be installed as necessary or fibre reinforced or steel mesh and plain shotcrete as outlined in support Classes 2 and 3.

4.7 Kinematic Unwedge Analysis

A kinematic assessment was conducted in order to check the empirical support predictions for site-specific joint data and to identify preferential drift development orientation. This was done using the UNWEDGE (Rocscience, 2004) software. The main objective of this kinematic assessment was to identify the possible bedrock wedge combinations that could occur using the major and minor discontinuity sets outlined in Table 12. The UNWEDGE program is designed such that wedges are defined by three intersecting joints and are subject to gravitational loading and sliding only. The UNWEDGE program gives the size, shape, weight and FOS for each potential rock wedge, based on the size and the orientation of the excavation. Ground support, in

the form of pattern and spot bolting, can then be applied to the wedge to determine possible support requirements.

The following parameters were used in the UNWEDGE analysis:

- Joints were ubiquitous with their surfaces perfectly planar;
- Wedges behaved as rigid bodies;
- The angle of friction of the joint surfaces was taken as 30°;
- No cohesion was attributed to any joint surface;
- No effect of clamping due to stresses generated around the excavation was considered;
- Tunnel excavation sizes were 5 m wide by 5 m high and 6.2 m wide by 5 m high for intersections;
- Rock support consisted of pattern support bolting with 1.8 m long bolts for 5 m wide by 5 m high drifts; and
- 12 drift orientations were used in the assessment between 000° and 180° azimuths on 15° increments.

Three of the joint sets in Table 12 were considered for the UNWEDGE analysis: 80°/360°, 45°/190° and 42°/60°. The sub-horizontal joint was not included in the analysis since it would not generate any sizeable wedges.

Plots of drift orientation versus wedge tonnage were developed to identify which drift orientation produced the largest wedges. Based on the three joint sets analysed for all drift dimensions, the largest wedges were generated when drifts were oriented between 075° and 105°. The largest wedges were produced at the 090° or 270° drift orientation as illustrated on Figures 21 to 23. Also illustrated on Figures 21 to 23 is a 2D profile of the type of wedges generated in the walls and back for drifts oriented between 075° and 105°. The largest wedges generated occurred in the intersections with a weight of 753 tonnes and 1002 tonnes for 8 m and 8.8 m wide drifts, respectively. The FOS for any wedge that formed in the back, prior to support being added was 0.

A second set of analysis was complete for the same sets of drift dimensions and orientations that included support to the back and walls. The type, length and spacing of support added to each wedge was based on the pattern support for drifts and intersection outlined in Section 4.6 as illustrated on Figures 22 and 23. All wedges were supportable with a FOS >1.2 when support was added to each wedge with 1.8 m (5 m and 6.2 m wide drifts) and 2.7 m (8 m and 8.8 m wide drifts) long bolts using a 1.5 m by 1.5 m pattern and support capacity of 20 tonnes assuming 100% bond capacity for the resin rebar support.

It should be noted that, where possible, drifts should not be preferentially designed along the 090° orientation in order to reduce the likelihood of large tonnage wedges. However, since the strike of the ore deposit is roughly in this direction, the requirement to develop along this orientation is acknowledged. It should also be noted that underground mapping is required to verify the joint sets in Table 12.

5.0 STOPE SIZING

5.1 Matthews Stability Graph Method

The stope sizing analysis that has been conducted for the Eagle Project has been based on the stability graph method. The stability graph method (Mathews et al., 1981; Potvin, 1988; Potvin and Milne, 1992 and Nickson, 1992) is an empirical relationship that has been developed for open stope design based on the depth of mining, rock mass quality and stope span. The stability graph (Figure 24) is a plot of stope Hydraulic Radius (HR) versus Modified Stability Number (N'). Stopes plotted on the graph are classified as stable, unstable, stable with support or caved. A stable stope will exhibit little or no wall deterioration during its mining cycle, while an unstable stope will exhibit limited wall deterioration (30% of face area) and a caved stope will exhibit unacceptable failure (Hutchinson and Diederichs, 1996). The stability number, N' , is defined as:

$$N' = Q' \times A \times B \times C$$

Where:

Q' = the modified Q rock mass classification;

A = the rock stress factor (value between 0.1 to 1.0);

B = the joint orientation adjustment factor (value between 0.2 to 1.0); and

C = the gravity adjustment factor (values between 0 and 10).

The Q' is a modified version of the Q rock mass classification formula that has the stress reduction factor (SRF) equal to one, which represents a moderately clamped but not overstressed rock mass and the joint water reduction (J_w) factor equal to one, representing a dry excavation for underground stopes. Therefore, the Q' represents the inherent characteristic of the rock mass (block size and joint properties). The rock mass rating that has been calculated for the Eagle Project has been based on the RMR rock mass classification system. The groundwater factor in the RMR calculation assumed a dry condition. Therefore, the calculated Q based on the formula in Section 3 is the same in this case.

The HR used in the stability graph is defined as the stope area/stope perimeter. The HR is calculated for the stope face (width = strike) and the end walls (width = hangingwall to footwall) and is defined by the following formula:

$$HR = \text{stope width} \times \text{stope height} / 2(\text{stope width} + \text{stope height})$$

5.2 Longitudinal Longhole Mining

Longitudinal longhole mining has been selected as the mining method between the 100 m and 160 m Levels due to the narrow ore widths in this section of the LZ. This method requires upper and lower access drift developed from the main haulage ramp currently proposed at a vertical interval of 30 m. Therefore, two main stoping panels will be used: one between 100 m and 130 m and the second between 130 m and 160 m. The cross-cutting access drifts will intersect the ore approximately half-way along the strike and mining will occur along the east and west limbs. A 15 m wide temporary pillar is proposed between the east and west panels in order to permit backfilling after longhole mining is completed. The approximate length of the east and west limbs is 60 m. The approximate width of the ore (hangingwall to footwall) is between 5 and 15 m. The dip of the hangingwall and footwall is sub-vertical.

The estimated HR for the longitudinal longhole mining stopes including 10% and 20% wall dilution is summarized in Table 16.

TABLE 16
HR FOR LONGITUDINAL LONGHOLE MINING

Stope Location	Stope Width (m)	Stope Height (m)	HR	HR 10% Dilution	HR 20% Dilution
Stope Face	60 (strike width)	30	10	10.3	10.6
End Walls	15 (ore width)	30	5	5.3	5.6
Back	60 (strike width)	15 (ore width)	6	6.6	7.2

5.3 Transverse Longhole Mining

Transverse longhole mining has been the mining method between the 160 m and 280 m Levels due to the increased hangingwall to footwall ore thickness, 15 m to 60 m, respectively. This method requires upper and lower access drifts to be developed from the main ramp with a proposed 30 m vertical spacing between levels. The access drifts will be parallel to the strike of the ore and have perpendicular cross-cuts into the ore zone. This method will use a primary/secondary panel extraction method. The strike length of the primary and secondary panels, based on the Scoping Study, is 20 m. A review of the 3D mining model, created for the Scoping Study, has shown the majority of the primary panel strike lengths being 15 m and the secondary panels strike length being 20 m. The width of the panels from hangingwall to footwall will depend on the ore thickness at each level and will therefore vary between 15 m and 60 m. The dip angle of the footwall varies between 55° and vertical while the dip of the hangingwall is vertical. McIntosh identified that the largest stope geometry in the mine will occur between the 200 m and 240 m Level. The dimensions of this stope panel are 70 m long, 34.5 m high

(including the 4.5 high upper level drift) and 20 m wide. The estimated HR for the transverse longhole mining stopes including 10% and 20% wall dilution is summarized in Table 17.

TABLE 17
HR FOR TRANSVERSE LONGHOLE MINING

Stope Location	Stope Width (m)	Stope Height (m)	HR	HR 10% Dilution	HR 20% Dilution
Primary Stope Face	60 (ore width)	30	10	10.3	10.6
Primary End Walls	15-20 (strike width)	30	5-6	6.3	6.6
Secondary Stope Face	60 (ore width)	30	10	10.3	10.6
Secondary End Walls	20 (strike width)	30	6	6.3	6.6
Largest Stope Face	70 (ore width)	34.5	11.5	11.9	12.2
Largest Stope End Walls	20 (ore width)	34.5	6.3	6.5	6.7
Back	60 (ore width)	20 (strike)	7.5	8.3	9.0
Back Largest Stope	70 (ore width)	20 (strike)	7.8	8.5	9.3

5.4 Rock Stress Factor (A)

The rock stress factor (A) replaces the SRF factor calculated in Q. The A factor is the ratio of intact rock strength (UCS) to induced stress (stress acting parallel to the exposed stope wall or roof). The UCS values for the ore, hangingwall and footwall rocks of the Eagle deposit are based on converted point-load testing data summarized in Table 3. The SMS ore is located in the UZ and the LZ while the MS ore is located only in the LZ. The average UCS value for the SMS is 111 MPa. The average UCS value for the MS is 57 MPa. The hangingwall rocks are comprised of sedimentary rocks (sandstone and siltstones) that have average UCS values equal to 63 MPa and 74 MPa, respectively. The footwall rocks are comprised of unaltered and altered peridotite intrusives with average UCS values equal to 120 MPa and 92 MPa.

Since there is no in situ stress data information available for the Eagle deposit, the vertical stress is estimated to be the weight of the overlying rock (0.027 MN/m^3) which is defined as follows:

- Vertical Stress, $\sigma_v = 0.027 \text{ MN/m}^3 \times Z$ (Vertical Depth in m)

The horizontal stress values (σ_{H1} and σ_{H2}) have been estimated to be equal and a ratio of the vertical stress using the following relationship:

- Horizontal Stress, $\sigma_{H1} = K_{H1} \times \sigma_v$ or $\sigma_{H2} = K_{H2} \times \sigma_v$

The K_{H1} and K_{H2} are a ratio of the horizontal to vertical stress and are typically between 1.5 and 3.0. The K value has been estimated using the following relationship (Herget, 1988):

- $K_{H1} = K_{H2} = 251.68/Z$ (Vertical Depth in m) + 1.14 = 2.0

The vertical depth has been assumed to be 300 m which is the approximate depth to the lowest level in the mine (100 m Level). Therefore, at the same depth, the vertical and horizontal stresses are:

- $\sigma_V = 8.1$ MPa
- $\sigma_{H1} = \sigma_{H2} = 16.2$ MPa

Depending on the rock types encountered in the end walls, footwall and hangingwall and back of the stope panel, the following (A) values have been estimated in Table 18 for the back and the walls of the stopes.

TABLE 18
STABILITY NUMBER A VALUE

Rock Type	Rock Strength/ σ_V	A value Back	Rock Strength/ σ_{H1}	A value Walls
SMS	13.7	1.0	6.9	0.63
MS	7.0	0.6	3.5	0.27
Peridotite	14.8	1	7.4	0.70
Feldspathic Peridotite	11.4	1	5.7	0.51

The UZ of the Eagle orebody is composed primarily of SMS, while the LZ is composed of two SMS and one MS zones.

5.5 Joint Orientation (B)

The joint orientation factor (B) accounts for the effect of persistent discontinuity features on exposed stope faces (back and walls). Structures that are orientated parallel (both dip and dip direction) to the exposed surface are rated the lowest (0.3), while structures that are orientated perpendicular (dip) to the exposed surfaces are the highest (1.0). The dip direction of the longest and shortest dimensions for the longitudinal longhole stope is between 000°- 033° and 090°-123°, respectively. The dip direction of the longest and shortest dimensions for the transverse longhole stope (primary and secondary) is between 090°-095° and 000°-005°, respectively. The orientation of the longest and shortest dimensions for the drift and fill mining panels is 150° and

60°, respectively. These wall orientations were based on the 3D mining geometries provided in the Scoping Study model.

Four pole plots have been plotted on each of the various rock type discontinuity stereonet plots to represent the longitudinal and transverse longhole stope face orientations (see Figures 16 to 18). The angle between the discontinuity pole plot and the stope face pole plot is the alpha angle. Alpha angles = 90° have a joint orientation B factor = 1.0 and alpha angles = 0° have a joint orientation factor = 0.3. The dip and dip direction of the stope faces (90°/000°, 90°/033°, 90°/090° and 90°/123°) were vertical and east-west or north-south trending. The lowest alpha value and corresponding B value for the stope walls and back have been listed in Table 19.

TABLE 19
STABILITY NUMBER B VALUE

Rock Type	Mining Method	Alpha Angle Stope Back	B Value Stope Back	Alpha Angle Longest Wall	B Value Longest Wall	Alpha Angle Shortest Wall	B Value Shortest Wall
SMS	Longitudinal	0°	0.3	10°-20°	0.2	80°	0.9
SMS	Transverse	0°	0.3	80°	0.9	10°-20°	0.2
MS	Longitudinal	0°	0.3	10°	0.2	20	0.2
MS	Transverse	0°	0.3	20	0.2	10°	0.2
Peridotite	Longitudinal	0°	0.3	20°	0.2	65°	0.85
Peridotite	Transverse	0°	0.3	65°	0.85	20°	0.2
Feldspathic Peridotite	Longitudinal	0°	0.3	30°	0.2	60°	0.8
Feldspathic Peridotite	Transverse	0°	0.3	60°	0.8	30°	0.2

Since one of the dominant joint sets, J1, is sub-horizontal, the stope backs for both longitudinal and transverse longhole mining were estimated with a B value = 0.3.

5.6 Gravity Adjustment Factor (C)

The gravity adjustment factor identifies the likely mode of structural failure that will occur in the stope walls and back. Three types of failure are considered: gravity falls, slabbing and sliding, but only the dominant failure category is considered. Based on the current data available for the Eagle deposit, the likely failure mode will be gravity and slabbing falls in the back and the walls of the stopes.

The following formula is used to determine the C value for gravity and slabbing type failures (Hutchinson and Diederichs, 1996):

- $C = 8 - 6 \times \text{Cosine (Dip of Stope Wall)}$

Since the walls of the longitudinal and transverse stopes have been estimated to have a dip of 90°, the corresponding C values for the stope walls and back are 8 and 2, respectively.

5.7 Stability Number (N')

Based on the estimated values for the A, B and C factors and the Q' values from Tables 8 and 16 to 19, the stability numbers N' are summarized in Table 20 for the longitudinal and transverse longhole mining stopes.

TABLE 20
STABILITY NUMBER N' VS. Q_{EQUIV}

Mining Method	RMR Typical	Q'_{equiv} Typical	A value	B Value	C Value	N'	HR	HR 10% Dilution	HR 20% Dilution
Longitudinal Longhole Stope Long Wall (H/W)	80	54.6	0.7	0.2	8	61.2	10	10.3	10.6
Longitudinal Longhole Stope Short Wall (ore)	80	54.6	0.63	0.9	8	247.7	5	5.3	5.6
Longitudinal Longhole Stope Back (ore)	80	54.6	1.0	0.3	2	32.8	6	6.1	6.2
Transverse Stope Long Wall (ore)	75	31.3	0.27	0.9	8	60.8	10	10.3	10.6
Transverse Stope Short Wall (H/W)	75	31.3	0.7	0.2	8	35.1	6	6.3	6.3
Transverse Stope Back (ore)	75	31.3	0.6	0.3	2	11.3	7.5	7.6	7.7

Plots of N' vs. HR are illustrated on Figures 24 to 26 for the short wall, long wall and the stope back. Based on these plots, both the longitudinal and transverse longhole stopes plot on the stable side of the curve.

5.8 Stope Dilution

Two types of dilution have been defined in the Scoping Study (AMEC, 2004): internal dilution and external dilution. The internal dilution will represent the portion of material (low grade) that is planned for the design of the stope panel for each mining method. The external dilution represents the portion of material beyond the planned panel dimensions from the stope walls and back.

The Scoping Study estimated that the external dilution for longitudinal and transverse longhole mining will be 5% and 10%, respectively. This would increase the maximum mining dimensions of the transverse stopes (parallel ore strike) from 20 m to 22 m and increase the longitudinal stopes dimension (perpendicular ore strike) from 15 m to 16 m.

However, similar dilution should be expected along the end walls of stopes as well. Therefore, for this analysis, we have considered that some total dilution (10% to 20%) will occur along the hangingwall and footwall of the transverse mining panels and some total dilution along the east and west end walls of the longitudinal stopes as illustrated on Figure 27. Adding a 10% and 20% dilution to the original stope dimensions increases the HR marginally. No mining dilution has been added to the height portion of the stope.

The 10% and 20% diluted HR values are plotted for the stope faces, end walls and stope backs for the transverse stopes (highest HR values) on Figures 24 to 26.

The only notable change in stability conditions in the transverse stopes is after a 20% dilution is applied to all walls and the stope back stability condition moves from an unsupported stable transition zone to a stable with support zone. Applying a similar dilution to the largest stope moves the stope face from the stable zone to the unsupported transition zone and the stope back from unsupported transition zone to stable with support transition zone.

5.9 Stope Sizing Discussion

Based on the results of the stability assessment, there should be no stability issues associated with the end walls and the stope faces for the longitudinal and the transverse stopes and the stope back of the longitudinal stope after a 10% or 20% dilution is applied to the walls. The stability conditions for the transverse stope back plot in the unsupported transition zone when 0 to 10% dilution is applied to the walls but changes to stable with support after 20% dilution is applied to the walls (see Figure 26). Therefore, support (i.e., cable bolting) may be necessary in the primary or secondary transverse stope backs depending on final stope size and rock mass quality. It should be noted that these stope are considered ‘non-entry’ and unsafe for workers to enter and may need to be mucked using remote access mining techniques.

The stability assessment of the longitudinal and transverse stope panels is based on the rock mass quality data at a prefeasibility level. Actual rock mass quality conditions in the stope walls and back will vary as more data becomes available. A decrease in the rock mass quality will have a greater impact on the wall and back stability than an increase in the stope panel dimensions (HR).

For example, if the rock mass quality of the walls or the back is actually a Q value of 3 or less ("Poor" quality), the stability conditions of the stope walls and the back change from a stable with support to a supported transition zone or "quasi stable" condition as is illustrated on Figures 24 to 26 (see red diamonds and triangle data points). Also, if 10% or 20% wall dilution occurs in these rock mass conditions, these stopes can change to a caving stability condition.

Therefore, as more data becomes available during the development of this project, a stope sizing optimization assessment needs to be completed for the longitudinal and transverse mining panels.

Based on the current assessment, the stope sizes considered by AMEC are reasonable (60 m wide by 20 m long by 30 m high and 15 m wide by 60 m long by 30 m high). Obviously, these stope sizes will need to be reassessed once more data becomes available and the long range layouts (i.e., locations) are created. Based on the current stope size range and expected wall and back conditions, stope support will likely consist of backfilling (tight). A contingency of cable bolt support should be considered if a more aggressive design is considered or lower than expected rock quality or adverse rock structure is encountered.

6.0 CROWN PILLAR

6.1 Crown Pillar Stability Assessment

Crown pillar stability assessments were conducted on the Eagle crown pillar where the geometry of the crown pillars was amenable to the analytical methods available. The two methods that have been used are (i) the empirical Scaled Span Concept Method and (ii) limit equilibrium analyses utilizing the CPillar program (Rocscience, 2001).

The crown pillar geometries used in the original assessment were based on five AutoCAD plan drawings provided by McIntosh Engineering. The AutoCAD drawings illustrated the estimated mining boundaries for the 360 m, 365 m, 370 m, 375 m and 380 m elevations and are illustrated in Appendix A. A second assessment was completed with specific stope geometries based on the revised mining plan provided by McIntosh (refer to Section 6.4). The rock mass quality for the bedrock in the crown pillar was based on the average RMR and Q_{Equiv} values discussed in Section 3.2. The RMR for the crown pillar ranged from 70 to 90. The RMR in the centre of the crown pillar ranged from 80 to 90, while on the east and west limbs, it was approximately 70. The typical RMR for the crown pillar was 75.

6.2 Scaled Span C_s

The concept of a scaled span for defining crown pillar stability was originally proposed in Golder's report for a CANMET research project (Golder, 1990). The concept was formulated based on back-analysis of old failures and review of precedent experience.

For any given geometry and rock quality, the crown pillar stability is estimated empirically by plotting rock mass quality, using the Q against a scaled crown pillar span index, C_s , calculated as follows:

$$C_s = S \left\{ \frac{\gamma}{t (1 + S_R) (1 - 0.4 \cos (\text{dip}))} \right\}^{0.5}$$

where: S = crown pillar span (m);

γ = density of the rock mass (t/m^3);

t = thickness of crown pillar (m);

S_R = span ratio = S / L (crown pillar span / crown pillar strike length); and

dip = dip of the orebody or foliation (degrees).

In this equation, all the parameters are related to the geometry of the crown pillar. The effects of groundwater and clamping stresses are included within the determination of the rock mass quality (Carter, 1992).

With the rock quality, Q , known, an estimate of the maximum stable span can be made by reference to the critical span (S_c) calculated as follows:

$$S_c = 3.3 \times Q^{0.43} \times \sinh^{0.0016}(Q)$$

where: S_c = critical span (m)

When $C_s < S_c$, the crown pillar is considered stable and when the $C_s > S_c$, the likelihood of failure is high, assuming the crown has not been artificially reinforced by bolting or fill. The ratio of $S_c/C_s = F_c$ can be considered to be an expression of the crown pillar stability FOS. The scaled span method has been based predominantly on $S = 3$ to 4 m. The estimated S for the Eagle crown pillar is between 70 m and 73 m.

6.2.1 Eagle Crown Pillar Scaled Span Assessment

The crown pillar dimensions have been based on the mining boundaries for the 360 m to 380 m elevation (see Appendix A) and orebody dip was based on the contoured GoCAD drawings as discussed in Section 3.0. The Q_{Equiv} values and crown pillar thickness (t) used for the assessment were based on these drawings as well as the analyses discussed above. The dip of the orebody in the upper level of the mine is vertical to sub-vertical. For this assessment, the worst case of vertical (-90°) was used. The typical RMR in the area of the crown pillar was estimated as 75 ($Q=31.3$) with an upper range of 85 (95.2). The following crown pillar geometries were considered for the assessment:

- S between 70 and 73 m;
- L between 107 and 115 m; and
- t between 25 and 65 m.

The S and L dimensions for the crown pillars below the 360 m elevation were based on the 360 m mining boundary. The top of bedrock has been estimated to be located at the 405 m elevation. The F_c values were calculated for the above crown pillar geometries using the typical and upper range of RMR values and are listed in Tables 21 and 22.

TABLE 21
EAGLE CROWN PILLAR ASSESSMENT - SCALED SPAN (RMR = 75)

Top of Crown Elevation (m)	H/W Dip (°)	t (m)	S (m)	L (m)	C _s	RMR	Q _{EQUIV}	Sc	Fc (FOS)
380	90	25	71	113	18.6	75	31.3	15.2	0.82
375	90	30	73	115	17.4	75	31.3	15.2	0.87
370	90	35	72	114	15.9	75	31.3	15.2	0.96
365	90	40	72	111	14.8	75	31.3	15.2	1.03
360	90	45	70	107	13.6	75	31.3	15.2	1.12
355	90	50	70	107	12.9	75	31.3	15.2	1.18
350	90	55	70	107	12.3	75	31.3	15.2	1.24
345	90	60	70	107	11.8	75	31.3	15.2	1.30
340	90	65	70	107	11.3	75	31.3	15.2	1.35

TABLE 22
EAGLE CROWN PILLAR ASSESSMENT - SCALED SPAN (RMR = 85)

Top of Crown Elevation (m)	H/W Dip (°)	t (m)	S (m)	L (m)	C _s	RMR	Q _{EQUIV}	Sc	Fc (FOS)
380	90	25	71	113	18.6	85	95.2	27.2	1.46
375	90	30	73	115	17.4	85	95.2	27.2	1.56
370	90	35	72	114	15.9	85	95.2	27.2	1.71
365	90	40	72	111	14.8	85	95.2	27.2	1.83
360	90	45	70	107	13.6	85	95.2	27.2	2.01
355	90	50	70	107	12.9	85	95.2	27.2	2.11
350	90	55	70	107	12.3	85	95.2	27.2	2.22
345	90	60	70	107	11.8	85	95.2	27.2	2.32
340	90	65	70	107	11.3	85	95.2	27.2	2.41

Based on the above assessment, a crown pillar thickness of 40 m is required for a FOS > 1 and a RMR of 75 and greater. The minimum crown pillar thickness for a FOS = 2.0 is a 45 m thick crown with a RMR = 85 and 145 m thick crown with a RMR = 75. A plot of C_s versus Q for the Eagle Mine crown pillar assessment is illustrated on Figure 28. This scaled span assessment indicates that the crown pillar is stable (FOS = 1.0) when the t = 40 m and the RMR is 75 or greater. The crown pillar becomes unstable if the RMR in the crown is below 75 when the t = 40 m. As the rock mass quality decreases, the crown pillar thickness will need to increase in

order to maintain a stable crown. It is important to note that this assessment considers that all the stopes are open beneath the crown pillar (i.e., that there is no fill present). However, the current mining plan is to tight fill all the stopes below the crown pillar.

6.3 CPillar Analysis

The program CPillar (Rocscience, 2001) was used for checking the empirical scaled span procedure assessment of stability state. This program uses limit equilibrium techniques to compute a FOS and probability of failure (POF) against several modes of failure, including various cracking modes and vertical downward sliding of a rectangular, horizontal crown pillar (assuming a rigid block model). Input, in the form of geometry, rock mass strength, in situ stresses and groundwater conditions are entered (with the option to input standard deviations of controlling parameters, if known) and, based on these values, a mean FOS and POF is calculated.

A number of basic assumptions were made for the CPillar analyses performed to evaluate the stability of the Eagle Mine crown pillar. These assumptions are as summarized below:

- (i) Since no accurate water levels are known, the water levels were estimated to be coincident with the ground surface;
- (ii) The extent of the in situ horizontal stresses, which affect the crown pillars, is not known with any degree of certainty. Although most of the analyses were conducted with horizontal and vertical stress ratios $K = 1$, several runs were conducted with reduced ratios of 0.7. This was done to demonstrate the beneficial effects of horizontal stress on crown pillar stability;
- (iii) The dip of the orebody in the region of the crown pillar were estimated to be 90° ;
- (iv) All the stopes are assumed open beneath the crown pillar (i.e., the assumption is made that there is no fill present);
- (v) For comparative purposes, because statistical variability was often lacking, a standard deviation of 10 percent of the mean value for most input parameters has been used allowing generation of spread statistics when insufficient raw factual data was available to define the full distribution;
- (vi) RMR values that were used for the analysis were based on the contour plots shown on Figures 3 and 9 to 14; and
- (vii) m values were determined using $m = m_i \cdot \exp((\text{RMR} - 100)/28)$ from Hoek and Brown, 1988 with m_i estimated = 25 ± 5 . For the Eagle Mine, the crown pillar $m = 7.1$.

6.3.1 Eagle CPillar Analysis

The same crown pillar geometries that were used in the Scaled Span assessment were used in the CPillar analysis. Crown pillar dimensions have been based on the mining boundaries for the 360 m to 380 m elevation which are outlined in Section 6.2.1. The water depth that was used in

the analysis assumed the crown pillar was completely saturated with water and (crown pillar t + overburden depth–water depth from surface) was measured from the bottom of the crown pillar. Point-load tests conducted on drill core was compiled and correlation was made between rock type and strength. This has been summarized in Table 3 (Section 3.3) and a sample illustration on Figure 29. Based on drill core data in the crown pillar region, the dominant rock type is a peridotite. Thus, the strength of the bedrock in the crown was designated to be 120 MPa. A UCS rock strength of 120 MPa (standard deviation = 39 MPa) was used for the analysis. The rock mass quality used for the analysis was based on the average RMR = 75 and a RMR = 85, as discussed in Section 3.6. The overburden depth was estimated at 12 metres. This was based on data from drill hole logs in the crown pillar area. The value of m_i was estimated at 25 which represents a peridotite bedrock with a 75 rock mass rating. Summarised in Tables 23 and 24 are the results of the CPILLAR analysis for RMR = 75 and 85.

TABLE 23
EAGLE CROWN PILLAR CPILLAR ANALYSIS (RMR = 75)

Top of Crown Elevation (m)	x (m)	y (m)	t (m)	OB (m)	H ₂ O Depth (m)	H ₂ O Height (m)	k	RMR	m_i	Fc (FOS)	POF (%)
380	113	71	25	12	0	37	1.0	75	25	2.7	15.0
375	115	73	30	12	0	42	1.0	75	25	2.8	12.9
370	114	72	35	12	0	47	1.0	75	25	3.1	10.8
365	111	72	40	12	0	52	1.0	75	25	3.36	9.1
360	107	70	45	12	0	57	0.7	75	25	3.7	7.6

TABLE 24
EAGLE CROWN PILLAR CPILLAR ANALYSIS (RMR = 85)

Top of Crown Elevation (m)	x (m)	y (m)	t (m)	OB (m)	H ₂ O Depth (m)	H ₂ O Height (m)	k	RMR	m_i	Fc (FOS)	POF (%)
380	113	71	25	12	0	37	1.0	85	25	6.3	11.7
375	115	73	30	12	0	42	1.0	85	25	6.5	11.0
370	114	72	35	12	0	47	1.0	85	25	6.9	10.3
365	111	72	40	12	0	52	1.0	85	25	7.3	9.7
360	107	70	45	12	0	57	0.7	85	25	7.9	9.0

Results from the CPillar analysis using a $K=1$ indicate that a minimum crown thickness of 25 m is required for a $FOS > 2.0$ ($POF = 15.0\%$) and a RMR of 75. When the RMR is increased to 85, the FOS is increased to 6.3 and higher. However, if we consider the worst case rock mass quality RMR of 70 in the crown pillar, and a K value reduced to 0.7 (which represents a lower confining stress condition), it requires a $t = 40$ to have a $FOS > 2.0$ ($POF = 17.1\%$).

6.4 Additional Crown Pillar Assessment

An additional crown pillar assessment was completed using an AutoCAD sequencing model provided by McIntosh (see Figure 30). This crown pillar assessment considered mining various thicknesses between the 340 m and 360 m Levels (22 m to 45 m thick). Based on these crown pillar geometries, a Scaled Span assessment was completed and is summarized in Table 25:

TABLE 25
EAGLE CROWN PILLAR ASSESSMENT - SCALED SPAN OPTIMIZED GEOMETRY

Stopes Panels Mined Level 1	Top of Crown Elevation (m)	H/W Dip (°)	t (m)	S (m)	L (m)	C _s	RMR	Q _{EQUIV}	Sc	F _c (FOS)
0106	383	90	22	47	20	9.2	85	95.2	27.2	3.0
0106 and 0107	383	90	22	47	40	11.4	85	95.2	27.2	2.4
0106, 0107 and 0108	383	90	22	61	60	15.3	85	95.2	27.2	1.8
No Panels Mined on Level 1	360	90	45	42	60	8.04	85	95.2	27.2	3.4
0106	383	90	22	47	20	9.2	75	31.3	15.2	1.7
0106 and 0107	383	90	22	47	40	11.4	75	31.3	15.2	1.3
0106, 0107 and 0108	383	90	22	61	60	15.3	75	31.3	15.2	1.0
No Panels Mined on Level 1	360	90	45	42	60	8.04	75	31.3	15.2	1.9

Based on the results presented in the previous table, if the rock mass quality in the crown has an RMR = 75, then no stopes could be mined in Level 1 in order to get a FOS of 1.9. Alternatively, if the rock mass quality in the crown has an RMR = 85, then three stopes (i.e., all of the high grade zones) could be mined in Level 1 with a FOS of 1.8. This table shows the relationship between rock quality, geometry and FOS. It also demonstrates the need to revisit this assessment once underground information becomes available.

6.5 Crown Pillar Discussion and Recommendations

The long-term stability of the crown pillar will be dependent on the following parameters:

- Rock mass quality of the crown pre-mining and post-mining;
- Crown pillar dimensions; and,
- Void size beneath crown pillar.

It will be required that additional rock mass quality information be collected underground, when access becomes available, and the crown pillar stability re-assessed. The density of drilling required to define the rock mass quality in the crown should be on the order of 15 m by 15m spacing (approximately 9 boreholes equally spaced in a 40 m by 40 m or 60 m by 60 m crown pillar).

If the crown pillar is determined to be 'marginally stable' (i.e., FOS between 1.0 and 2.0) or 'unstable' (i.e., FOS less than 1.0), it will be critical that all the void areas beneath the crown be filled with consolidated material (i.e., cemented fill) when mining is complete. The mining sequence should also be designed such that a minimal amount of stope area is open and blast damage beneath the crown is minimized.

7.0 BACKFILL DESIGN

The mining methods that have been proposed for the Eagle Project were a combination of drift and fill mining in the UZ, longitudinal and transverse longhole mining in the LZ (AMEC Scoping Study Support, 2004). Revisions to the initial design have been proposed that only include transverse longhole mining for the Eagle Project. Based on the revised mining plan (3D block model from McIntosh), there are approximately 50 mining panels that are typically 20 m long (east-west), 30 m high (34.5 m if including upper level and lower level access drifts) and 7.5 m to 70 m wide (ore width north-south). Mining of the stope panels will use a primary and secondary mining sequence. Primary stope panels will be mined first and filled with a cemented backfill. Secondary stopes will be mined on the east and west sides of the primary stopes. The purpose of the cemented backfill in the primary stopes will be to reduce wall dilution and improve the regional ground support after mining of that area is complete. The cemented backfill must also be self-supporting when it is exposed on the east and west walls of the primary stope. It is planned to fill secondary stopes with unconsolidated material (i.e., mine waste) to maintain stability.

Two methods have been used to determine the minimal UCS of the cemented backfill in order to be self-supporting.

The first method is the two-dimensional vertical slope method which is:

- $UCS_{FILL} = \gamma H$

Where:

γ = Bulk unit weight of the fill (estimated at 2,000 kg/m³ or 19.6 kN/m³); and

H = Height of the fill.

The second method is the three-dimensional Mitchell method (Mitchell et al, 1981) which is:

- $UCS_{FILL} = \gamma H(1 + H/L)$

Where:

γ = Bulk unit weight of the fill;

H = Height of the fill; and

L = Strike length of exposed face.

Based on above equations and the dimensions of the primary panels, the following minimum UCS backfill values have been calculated and are listed in Table 26.

TABLE 26
MINIMUM STRENGTH OF BACKFILL DESIGN FOR PRIMARY STOPEs

L (m)	H (m)	W (m)	γ (kN/m ³)	UCS Vertical Slope Method (kPa)		UCS Mitchell Method (kPa)	
				FOS=1	FOS=2	FOS=1	FOS=2
70	34.5	20	19.6	676.2	1352.1	453.0	906.0
60	34.5	20	19.6	676.2	1352.1	429.3	857.6
50	34.5	20	19.6	676.2	1352.1	400.1	800.2
40	34.5	20	19.6	676.2	1352.1	363.1	726.2
30	34.5	20	19.6	676.2	1352.1	314.5	629.0
20	34.5	20	19.6	676.2	1352.1	248.1	496.2
10	34.5	20	19.6	676.2	1352.1	152.0	304.0

W = width of panel (east-west)

The minimum backfill strength required for the primary stopes will be dependent on the FOS required, density of the backfill material, fill height and length of exposed stope face. For initial design purposes, the minimum self-supporting backfill strength considered for primary stope panels is 1.5 MPa.

8.0 MINING SEQUENCE

A review of the revised mining sequence was completed using Map3D, a 3D rock stability analysis package. The Map3D program is used to construct models, analyse and display displacements, strains, stresses and strength factors. Models can include underground excavations with yielding (non-linear) zones of different moduli (e.g., stiff dykes or soft ore zones) and can be intersected by multiple discrete faults (non-planar and gouge filled) that slip and open.

A base model was constructed using the revised mining plan geometries and mining sequence created by McIntosh. Material properties for the host rock and the ore were based on the Eagle deposit data and backfill properties were based on typical industry standards. Stress regime data for the Eagle deposit was based on published values for the regional stresses in the Canadian Shield.

8.1 Model Geometry and Mining Sequence

Based on the revised mining plan (i.e., 3D block model from McIntosh), there are approximately 50 primary and secondary mining panels to be mined. Illustrated on Figure 30 is a longitudinal section of the Eagle deposit (south view) showing the primary and secondary stope panels. Each stope is labelled a mining level (row) and a panel (column) number. At the top and bottom of each panel, there is a 4.5 m high access drift.

A three-dimensional model was created in Map3D based on the block model geometry illustrated on Figure 30. The stope panels in the Map3D model include the 4.5 m high access drifts at the top and bottom of each stope panel to give a total vertical height of 34.5 m.

The mining excavation sequence used in the Map3D model was based on the sequence provided by McIntosh and included the following:

- Primary stope panels were excavated and filled before secondary stope panels were excavated. Also, underlying primary stopes were excavated and filled before overlying primary stopes were excavated;
- A maximum of three stope panels were excavated in a single step where possible. Early on and later on in the mining sequence, less than three stope panels were mined because overlying primary stopes were mined in succession; and
- A 30-day set-up time was required in the primary stopes before adjacent secondary stopes could be excavated.

In order to simulate the excavation sequence proposed for the Eagle deposit, there were 27 steps included in the Map3D model. A typical mining step included a combination of stope

excavations and backfilling occurring simultaneously at different parts of the mine. All stopes were excavated using the overhand method. Two types of backfill were used in the model: a higher strength backfill for the primary stopes and a lower strength backfill for the secondary stopes.

8.2 Modelling Material Parameters and Stress Regime

Four material parameters were used in the Map3D model. Material 1 was the host material surrounding the ore deposit which was peridotite only for model building simplicity. Material 2 was the ore deposit and was composed of massive sulphides only. The material properties for these rock types (Hoek-Brown) were based on a combination of the Eagle deposit data and published estimates of similar rock types. Material 3 was the higher strength cemented backfill (UCS = 600 kPa) and Material 4 was the lower strength unconsolidated backfill (UCS = 200 kPa). For initial modelling, the higher strength backfill was designed to be self supporting with a FOS at 1.0. A second model was completed using a higher strength backfill of 1.5 MPa and indicated no significant difference in the results when reviewing minor principal and deviatoric stresses. The backfill material properties (Mohr-Coulomb) were based on typical industry standards. The material properties used in the Map3D model are listed in Table 27.

TABLE 27
MAP3D MODEL MATERIAL PROPERTIES

Material	UCS (MPa)	Young's Modulus (MPa)	m	s	Friction Angle	Cohesion	Poisson's Ratio
Peridotite	120	56,000	12.2	0.0108	-	-	0.25
Massive Sulphide	57	32,000	13.1	0.0622	-	-	0.25
High Strength Backfill	0.6 or 1.5	1000	-	-	38°	0	0.2
Low Strength Backfill	0.2	220	-	-	30°	0	0.2

The stress regime data for the Eagle deposit is unknown. An estimate of the stress regime data has been based on the regional stresses in the Canadian Shield from Herget (1988) and was discussed in Section 5.4. The vertical stress, σ_v , and horizontal stresses ($\sigma_{h1} = \sigma_{h2}$) for the four material properties are listed in Table 28. The stress regime ratio for the massive sulphide material is slightly higher than the peridotite material because the unit weight is higher.

TABLE 28
MAP3D MODEL STRESS REGIME PROPERTIES

Material	σ_v (MPa/m)	σ_{h1} (MPa/m)	σ_{h2} (MPa/m)
Peridotite	0.027	0.054	0.054
Massive Sulphide	0.030	0.060	0.060
High Strength Rockfill	0.020	0.020	0.020
Low Strength Rockfill	0.027	0.027	0.027

The stress regime data for the backfill material is based on weight of the material.

8.3 Modelling Results

Four stress analysis grids were used to review the Map3D model results. Grid 1 was oriented east-west through the centre of the deposit and Grid 2 was orientated north-south through secondary stope panels (Panel 6). Grids 3 and 4 were horizontal planes at the 252 m and 310 m elevations. An isometric view of the stress analysis grid planes and the Map3D model are illustrated on Figure 31.

Two types of stress data were reviewed in the Map3D model: minor principal stress (σ_3) and deviatoric stress ($\sigma_1 - \sigma_3$).

8.3.1 Minor Principal Stress

Minor principal stress results were reviewed for the proposed mining sequence in order to identify zones with low confining stress (i.e., low minor principal stress). These areas indicate zones of relaxation and, if left unsupported (i.e., backfilled), may be locations where sloughage and gravity falls can occur in the stope backs and walls. Illustrated on Figures 32 to 33 are screen captures from Grids 1 and 2 from various excavation steps. Illustrated on Figure 34 is an isometric view of the north and south walls after all the stope panels have been mined and backfilled (step 27). The minor principal results from the Map3D model indicated the following:

- Minor $\sigma_3 < 0$ contours begin to occur in the walls and the back of the primary stopes after a primary stope is mined over a backfilled primary stope (steps 11-13 – Figure 32). The depth of the $\sigma_3 < 0$ contours were 3-5 m into the back and the east and west walls;
- Minor to Intermediate $\sigma_3 < 0$ contours begin to occur in the perimeter east and west walls of the deposit after the adjacent stope panels are removed (steps 16-17– Figure 31). The depth of the $\sigma_3 < 0$ contours was 5-10 m in the west wall (between 220 m and 320 m elevations) to step 27 and 30-40 m in the east wall (between the 270 m to 320 m elevations) between

steps 17 to 27. At step 16, thirty-two of the fifty stopes have been mined and are all below the 260 m Level; and

- Major $\sigma_3 < 0$ contours occur in the north wall after step 17 (see Figure 33) and increase into the north and south walls between steps 19 and 27. Initial $\sigma_3 < 0$ contour depths in the north wall were 10 m at step 14 and increase to 20-45 m between steps 19 and 27 (210 m to 350 m elevations). $\sigma_3 < 0$ contour depths in the south wall were 10 m or less to step 19 and increased to 10-45 m between steps 20 and 27 (270 m to 370 m elevations).

The low confinement zones identified in the modelling indicate that there is a potential for sloughage and gravity falls around the perimeter of the deposit, particularly in the upper east and west walls (220 m to 320 m elevations) and in the north and south walls between the 210 m and 370 m elevations. These low confining zones begin to develop and increase after the majority of the mining is completed below the 260 m elevation. Backfilling the stopes and applying ground support to the perimeter walls will minimize potential sliding and ground fall instabilities.

8.3.2 Deviatoric Stress Results

Deviatoric stress ($\sigma_1 - \sigma_3$) results were reviewed for the proposed mining sequence in order to identify potential failure zones. Two thresholds were generally considered: deviatoric stress of approximately 0.3 UCS (intact rock mass) when the rock mass may begin to yield and approximately 0.6 UCS (intact rock mass) when the yielded rock mass may be sufficiently yielded to begin sloughing.

The UCS values of the host rock have been estimated to be 120 MPa (peridotite) and 57 MPa for the ore (massive sulphides). Semi-massive sulphides were not included in the model but occur in the ore zone and have a UCS equal to 111 MPa. Therefore, the rock mass may begin to yield between 17 MPa (massive sulphides) and 36 MPa (peridotite) and begin to slough between 34 MPa (massive sulphides) and 72 MPa (peridotite) depending on the rock type. These thresholds need to be calibrated to field conditions when the mine is being developed.

Illustrated on Figure 35 are the deviatoric stress contours for the north and south walls of the Map3D model. Typical deviatoric stress contours values to a depth of 10-20 m into the north and south wall were between 18 MPa and 30 MPa when all stopes have been excavated in the model (step 27). A small concentration of deviatoric stress contours above 42 MPa is predicted in the centre of the south wall at the 252 m elevation.

The deviatoric stress results from the Map3D modelling indicate that some of the rock mass may yield around the perimeter of the deposit in the north and south walls. The effect of deviatoric stress will be governed by the in situ stress regime and rock mass properties. Both of these parameters have been estimated based on values from other mine sites. Therefore, even though

major deviatoric stress issues are not expected, it would be pertinent to revisit this issue again once more information is available when the project proceeds underground.

8.4 Mining Sequence Discussion

The current mining sequence proposed for the Eagle deposit does not indicate major rock mechanics issues. The following observations have been identified from the data provided:

- The sequence considers that all full sized (i.e., 20 m strike length by 30 m high) secondary stope panels are mined only after the primary stopes on either side are filled;
- Mining begins at the 140 m elevation (i.e., Level 8 – see Figure 30) and works upward;
- A typical mining extraction cycle consists of three stope panels (primary or secondary) open and one stope panel filled. No more than three stope panels are open at one time during the total extraction sequence;
- The three stope panels open at one time are spread over two or three levels; and
- The first full sized secondary stope panel (Panel 0706 – see Figure 30) is extracted when approximately 70% of the primary panels are excavated and filled. These primary panels are located below the 260 m elevation (i.e., Levels 5 through 8).

This approach is reasonable from a rock mechanics perspective and balances production requirements and allows for flexibility. A sequencing question was raised with the regards to mining primaries concurrently on both sides of secondaries. Although not ideal, this is likely achievable with attention to local geologic conditions (rock mass quality and structure) and with controlled blasting techniques (to reduce blast vibrations). While concurrent mining is acknowledged, it is recommended that the production from these stopes be staggered as much as practical. This will minimize unfavourable interactions due to stress and blasting, while also allowing timely filling of the first production should instabilities be encountered.

9.0 HYDROGEOLOGY

A detailed hydrogeology study of the bedrock and shallow overburden groundwater has been undertaken by the Golder Calgary office (in Alberta, Canada) and a draft study report has been issued to Warner, Norcross and Judd, and also to Kennecott Minerals Company in Marcotte, Michigan. The draft report is dated November 2004, and covers the work completed to date by Golder.

The objectives of the study were to characterize the hydraulic properties of the bedrock at six selected cored borehole locations, evaluate the hydraulic communication between the overburden and bedrock aquifer, and to estimate mine inflows for an idealized tunnel.

Six existing representative NQ-cored boreholes were selected for further testing, to cover the reported lithology and a range in orientations and inclinations. Geophysical and flow logging was performed to identify localized zones of elevated hydraulic conductivity. Hydraulic testing was subsequently carried out on zones of interest using packer testing methods. Based on the results, a conceptual model of the groundwater system was developed, and groundwater inflows into an idealized mine were estimated.

The study determined that the uppermost 300 feet of the tested rock mass is weathered, and the hydraulic conductivity was determined to be on the order of $2\text{E-}6$ cm/s. The underlying rock (to a depth of 1,000 ft) exhibited a lower hydraulic conductivity, on the order of $5\text{E-}8$ cm/s. Monitoring of the overburden aquifer suggests that there is negligible hydraulic communication between the overburden aquifer and the bedrock mass.

Some mine inflow estimates for the project have been developed. However, additional analysis is being performed to refine inflow estimates to account for any bias of the borehole orientation and dip relative to the dominant fracture patterns. This analysis is being completed in January, and the conceptual model and inflow estimates will be revised.

The study data generated as part of this report has been conducted on a scale that should be adequate to make estimates of parameters at a prefeasibility level.

10.0 CLOSURE

We trust that this report meets your needs at the present time. Should you have any questions, please do not hesitate to contact the undersigned.

GOLDER ASSOCIATES LTD.

Paul Palmer, P.Eng.
Geological Engineer

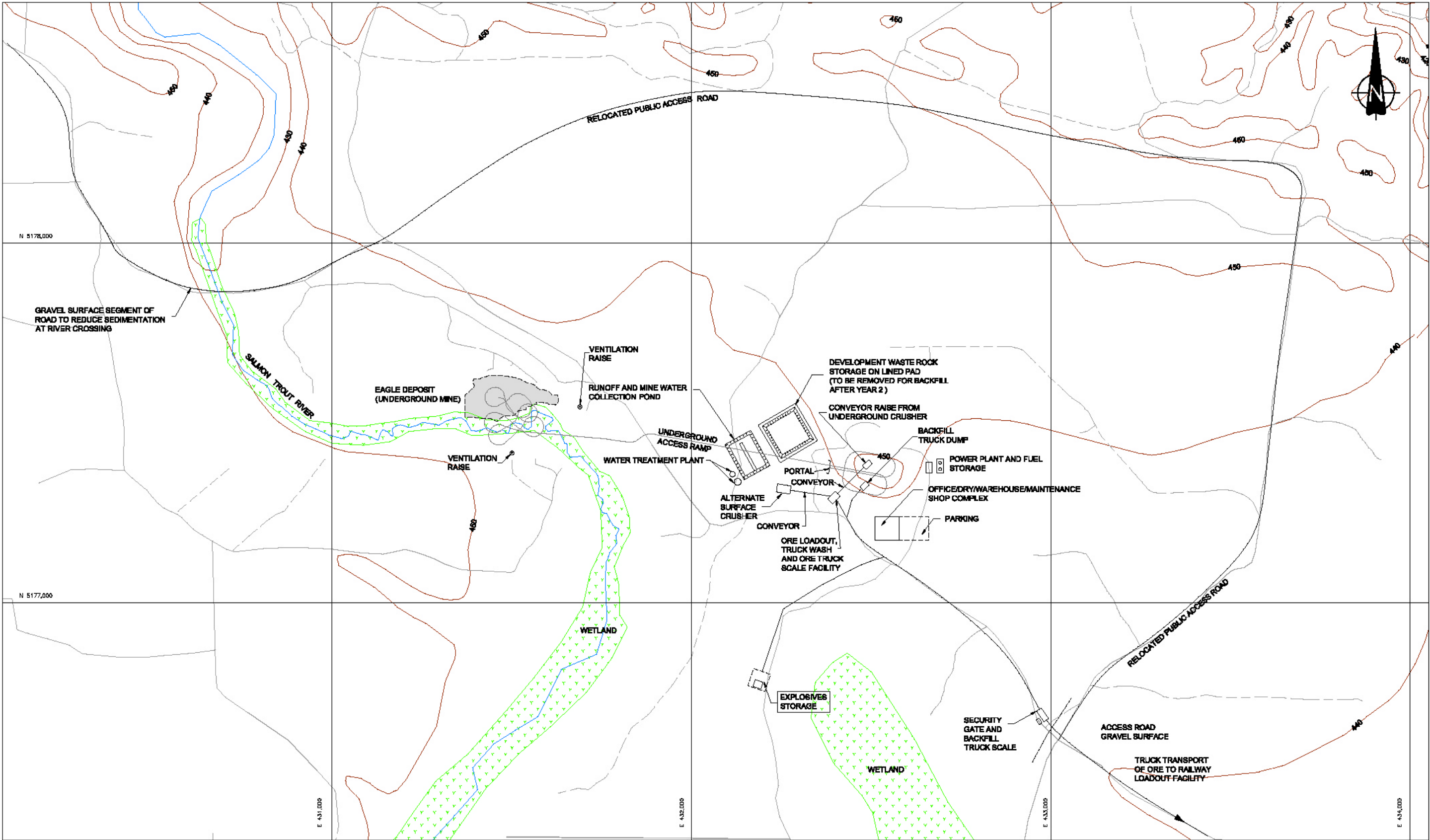
K.J. Beauchamp, P.Eng.
Associate

PP/KJB/lb

File Location: S:\Active\2004\1190_Sudbury\1193\04-1193-020 Kennecott Eagle Geotech Michigan

Eagle Deposit Site Plan

FIGURE 1

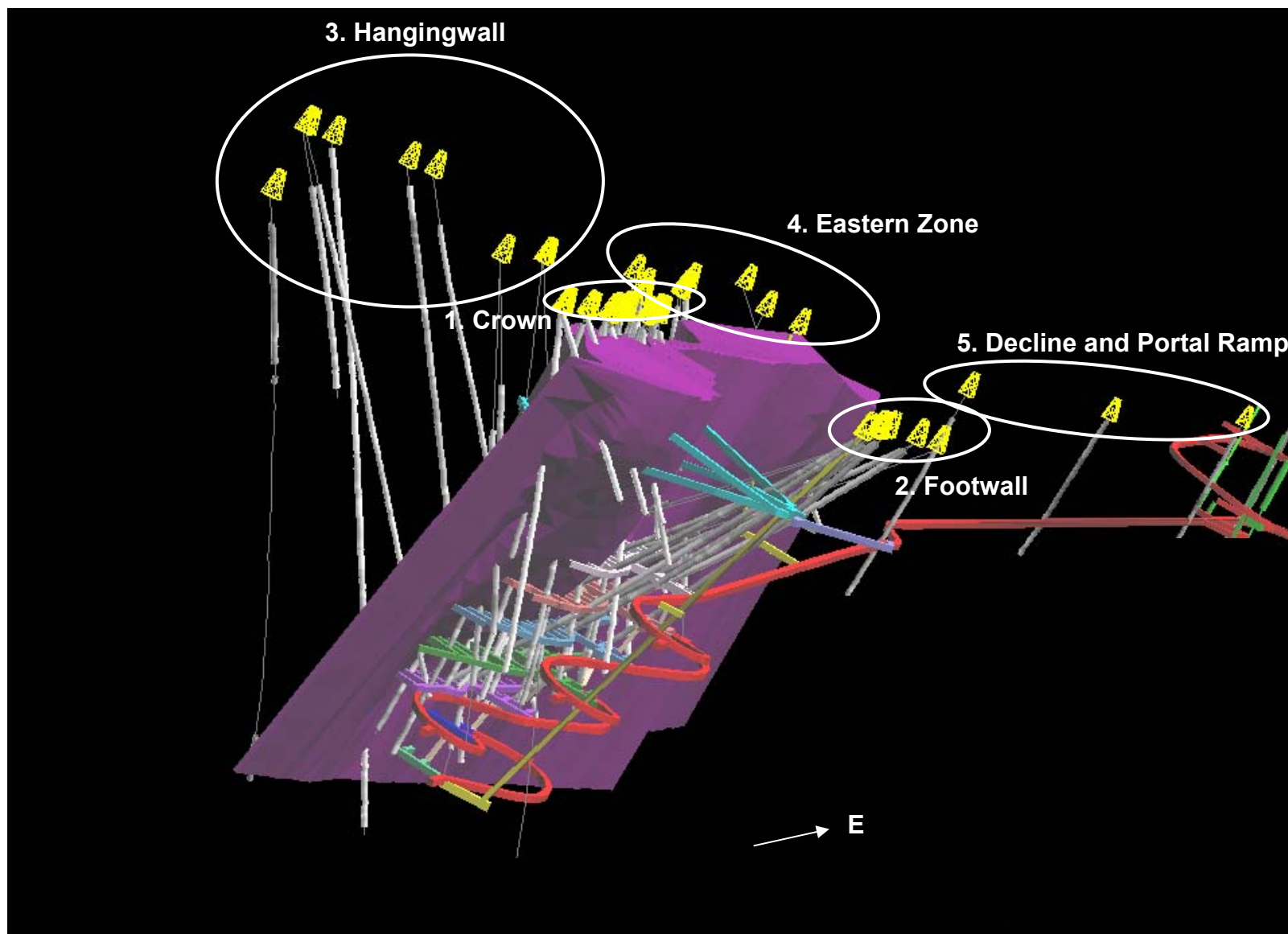


Taken from: Eagle Nickel Project – Scoping Study Report, AMEC 2004

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CHKD...PP

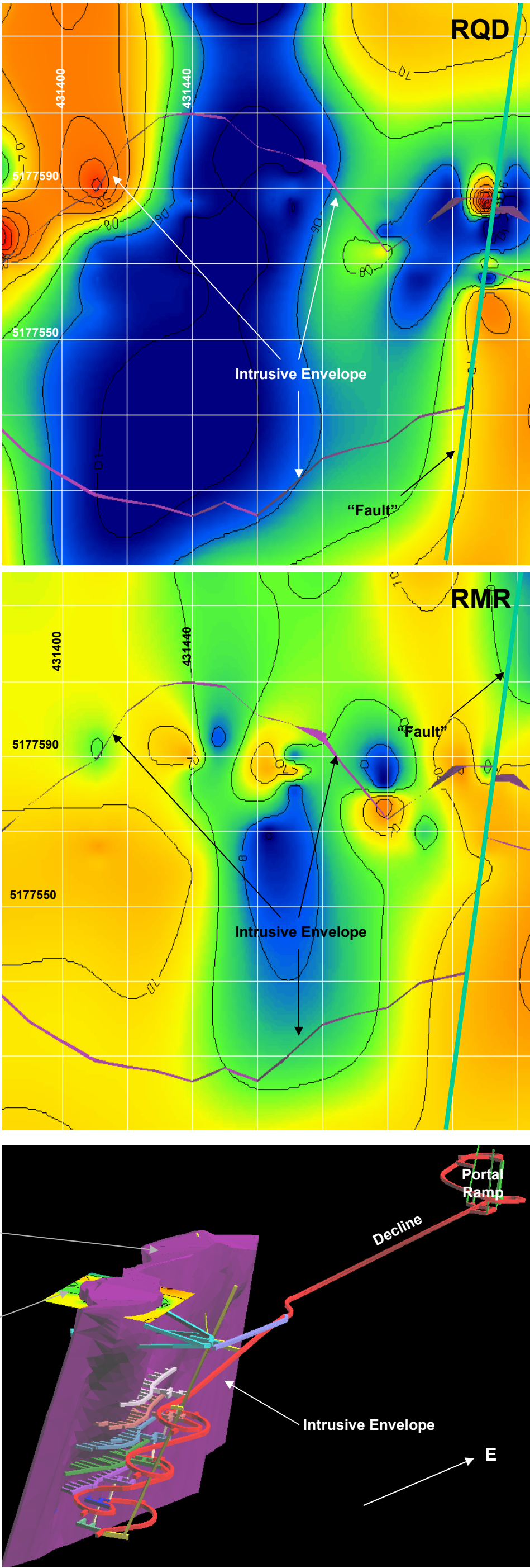


Project: 04-1193-020 Drawn: _____ Reviewed: _____

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PROJECT 04-1193-020



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Plan: RQD and RMR Contouring for 405 Elev.

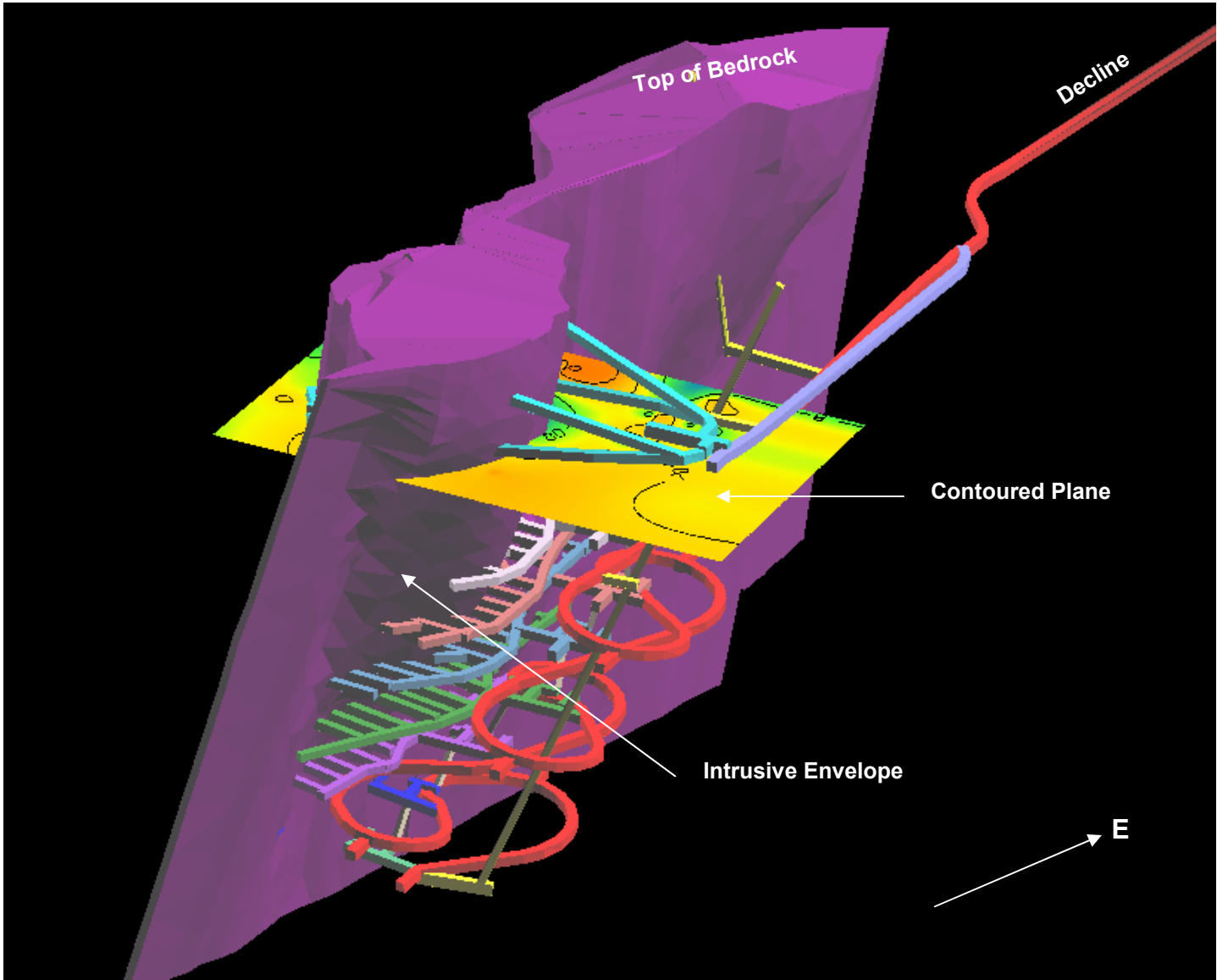
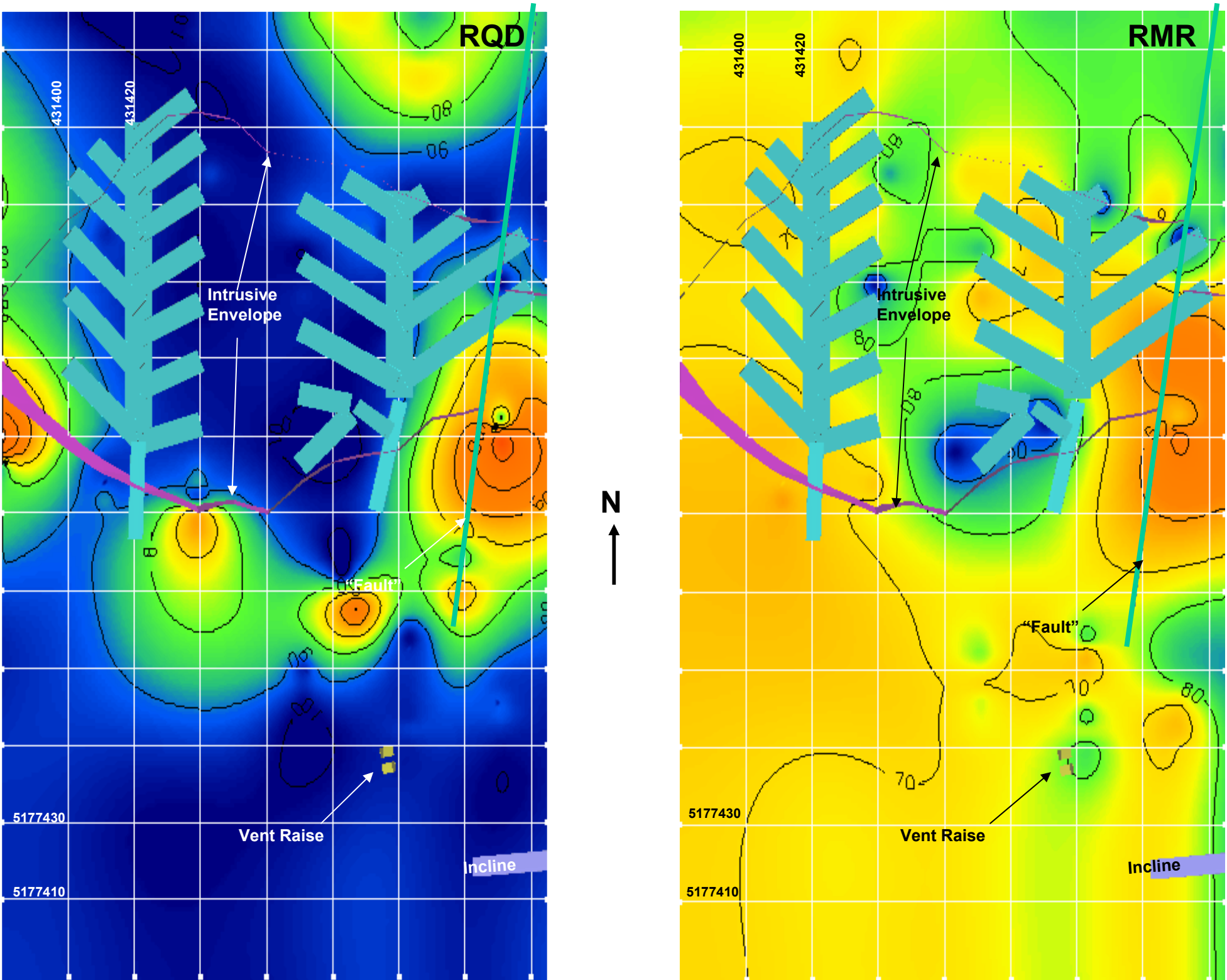


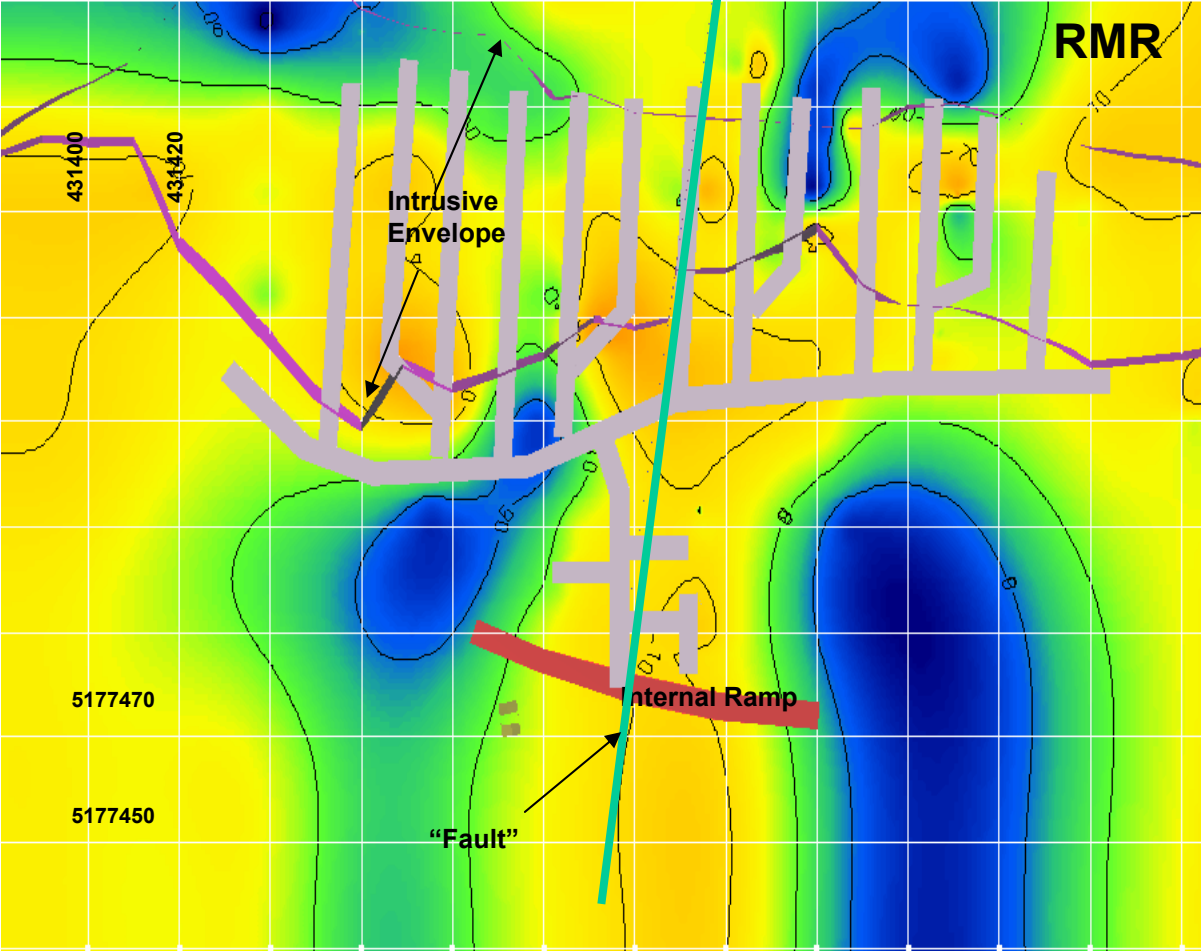
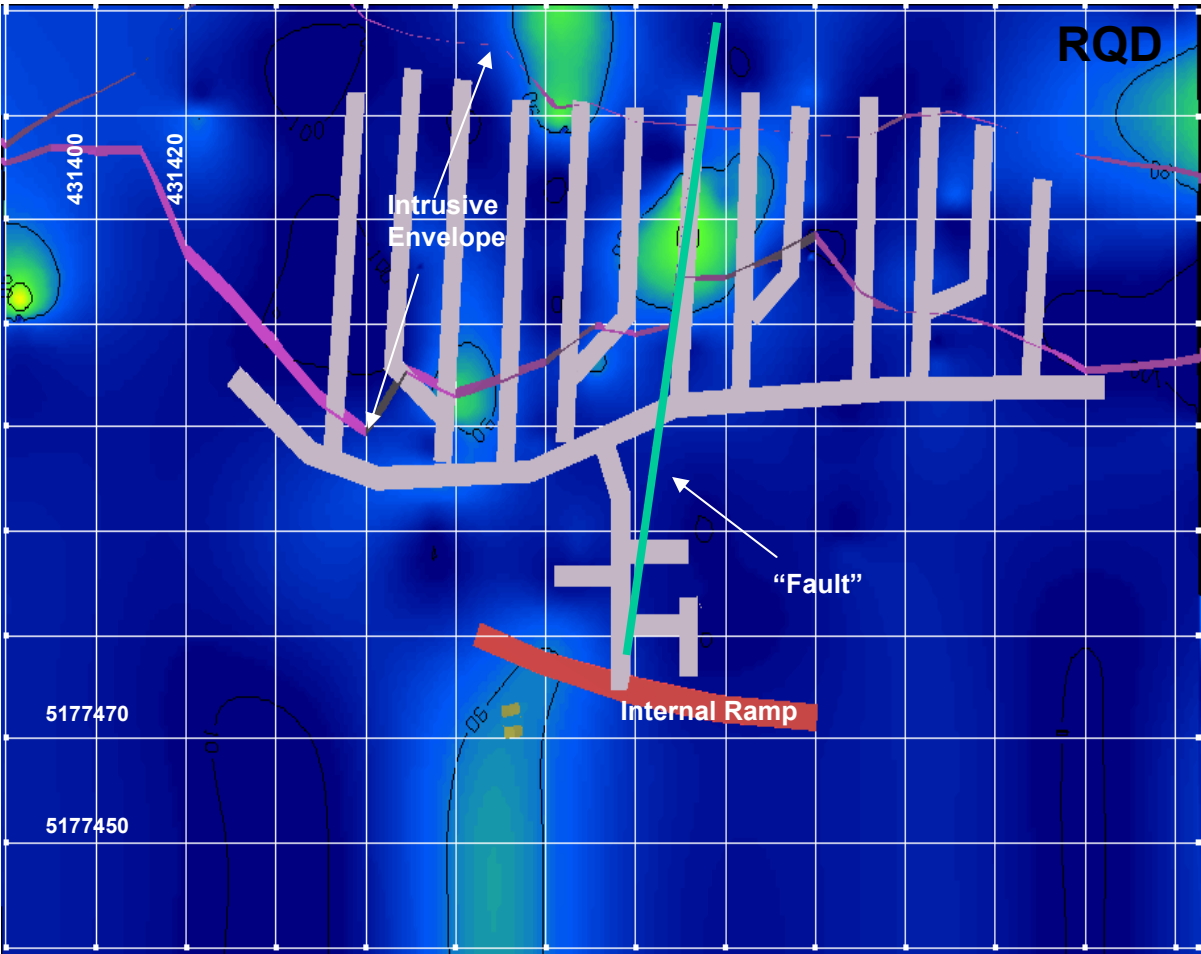
FIGURE 4

Plan: RQD and RMR Contouring for 355 Elev.

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PROJECT: 04-1193-020



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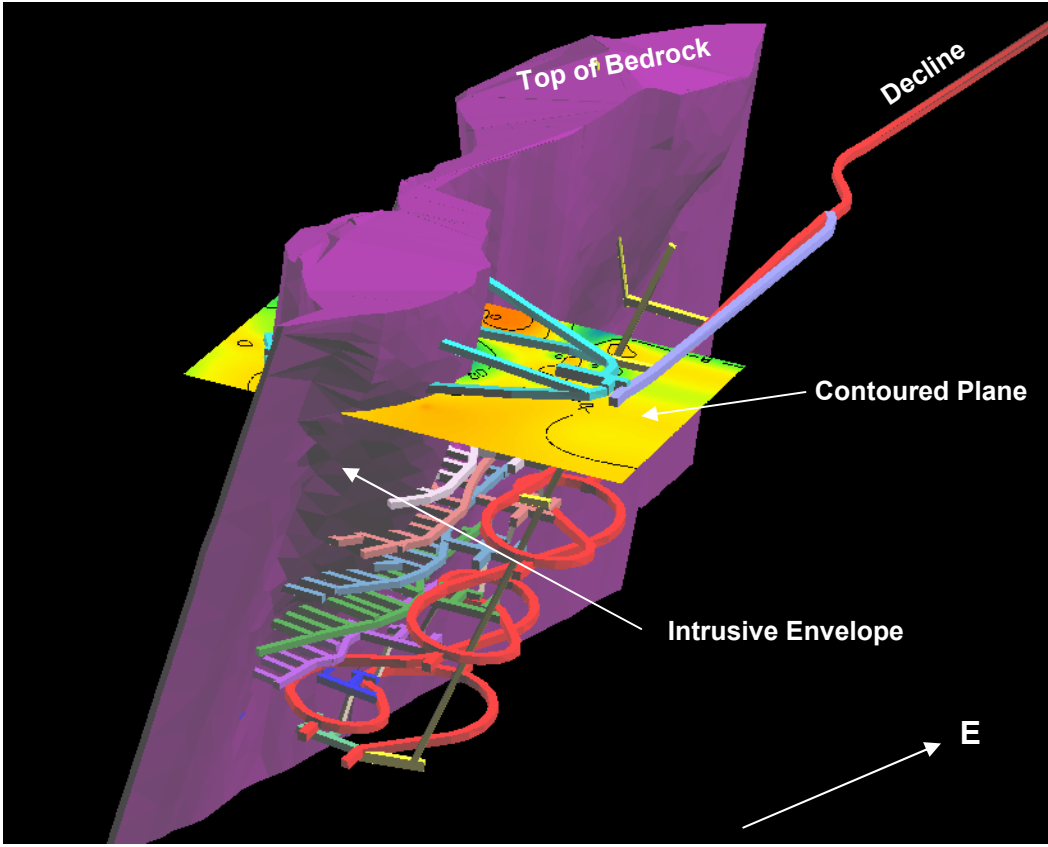


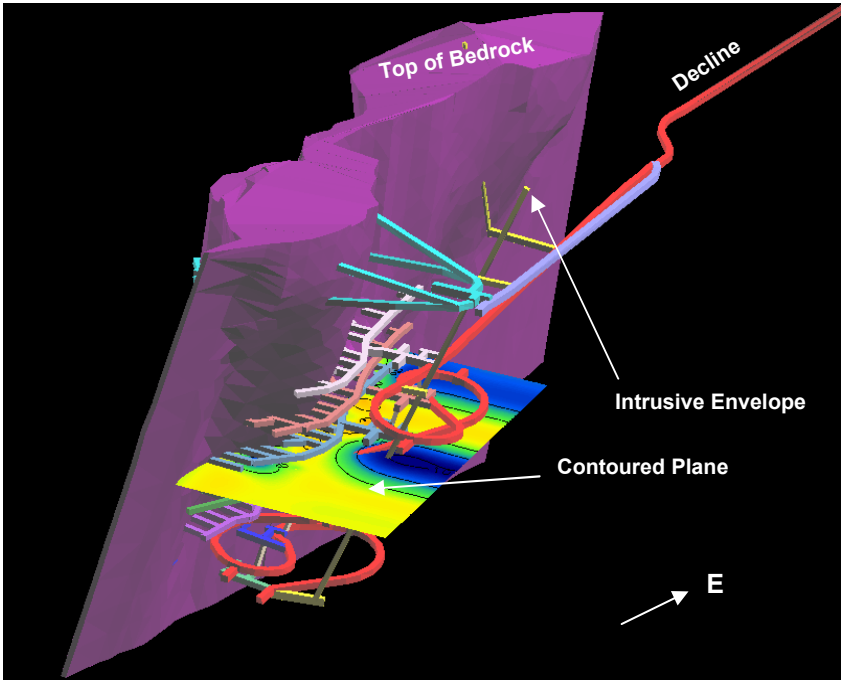
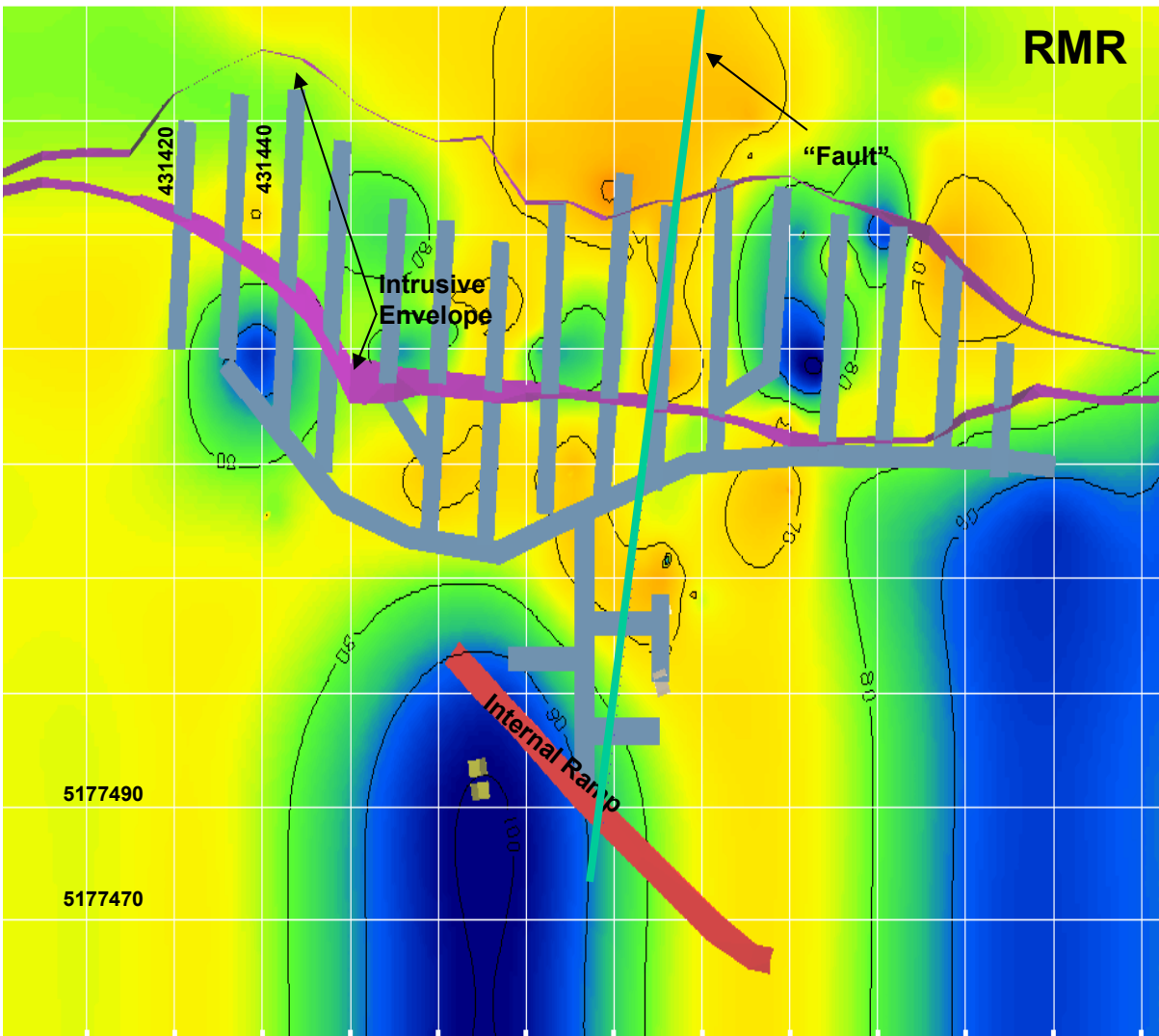
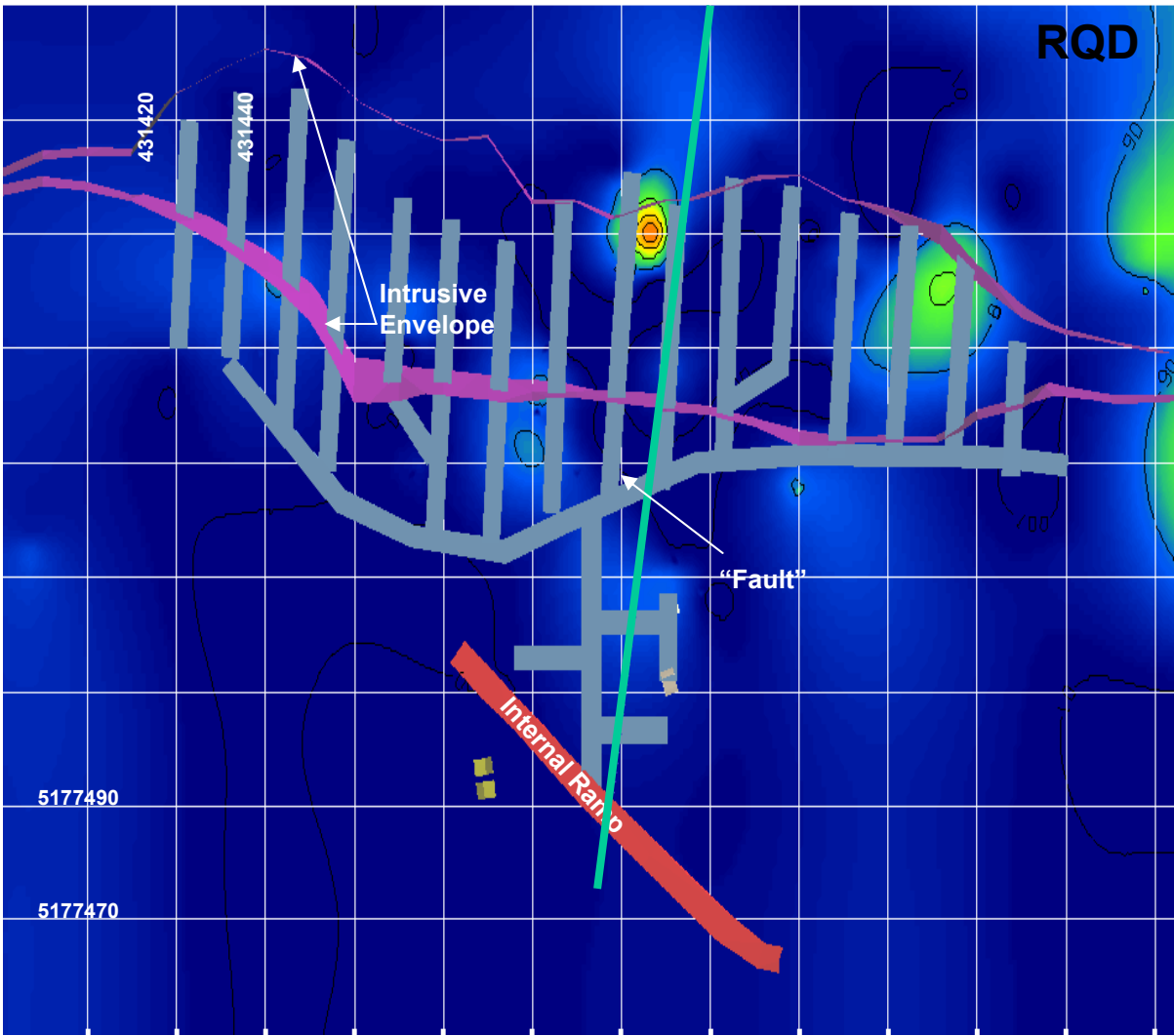
FIGURE 5

Plan: RQD and RMR Contouring for 280 Elev.

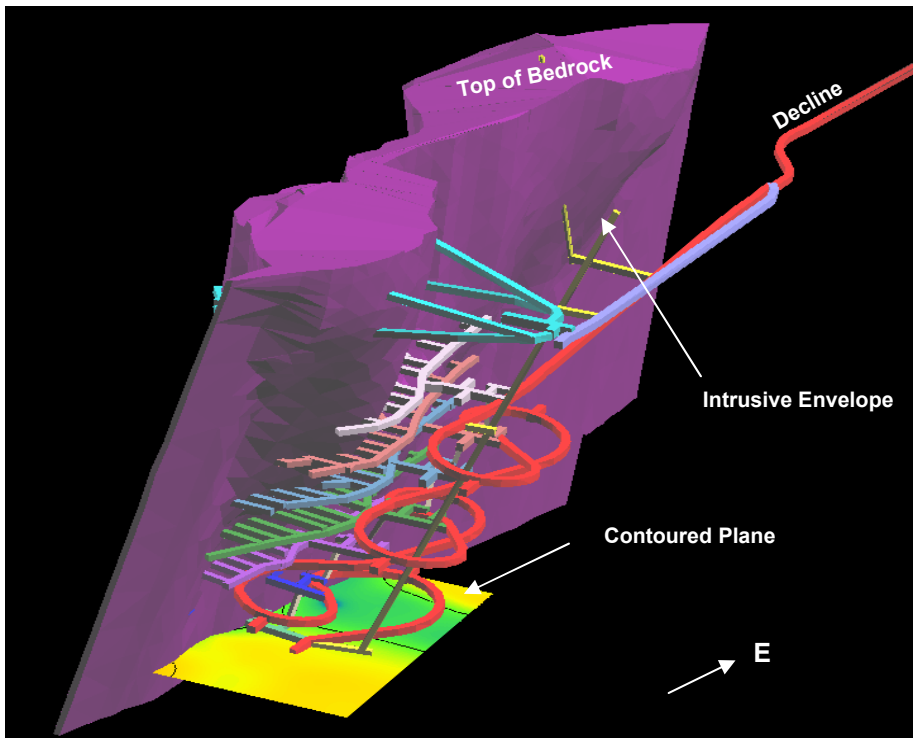
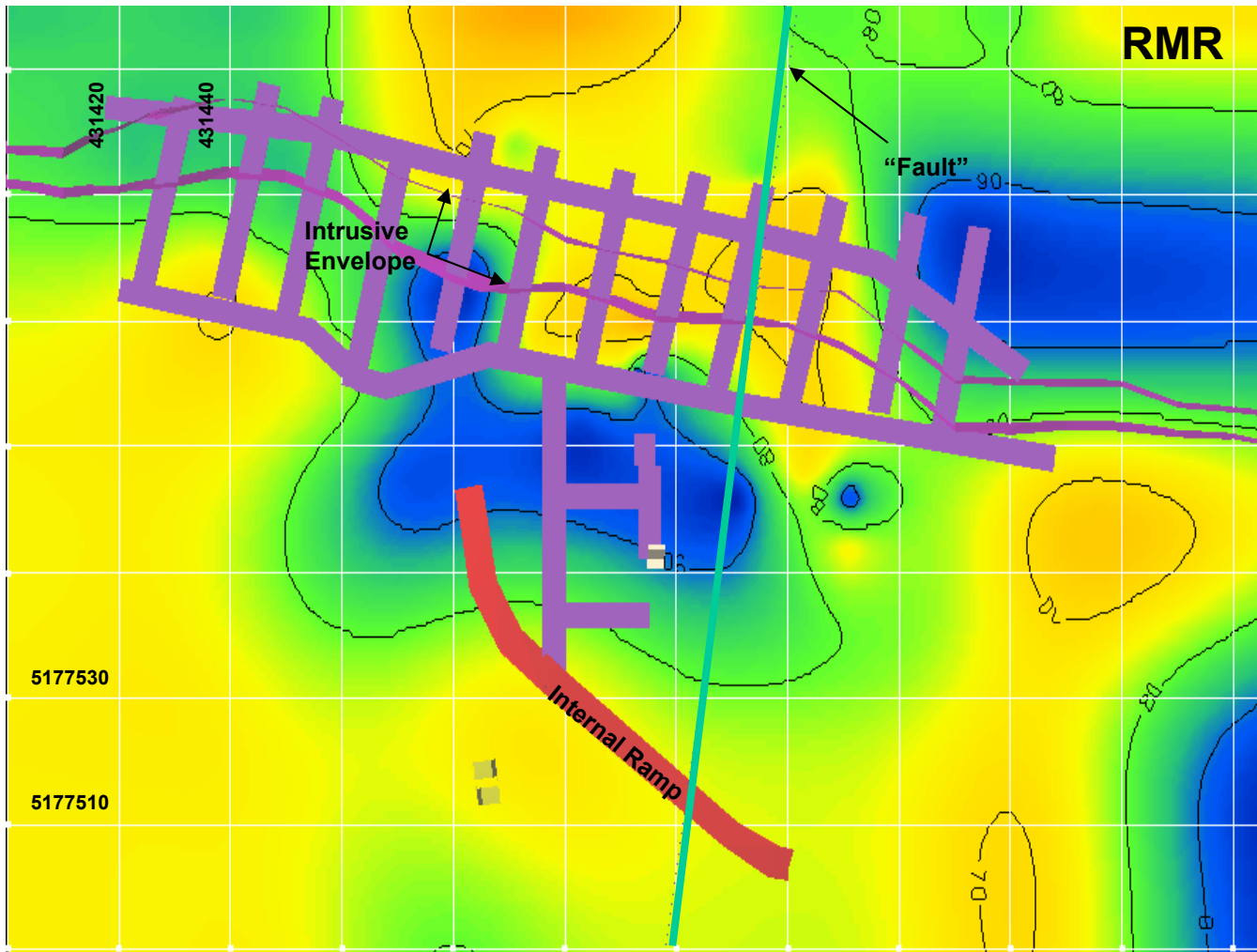
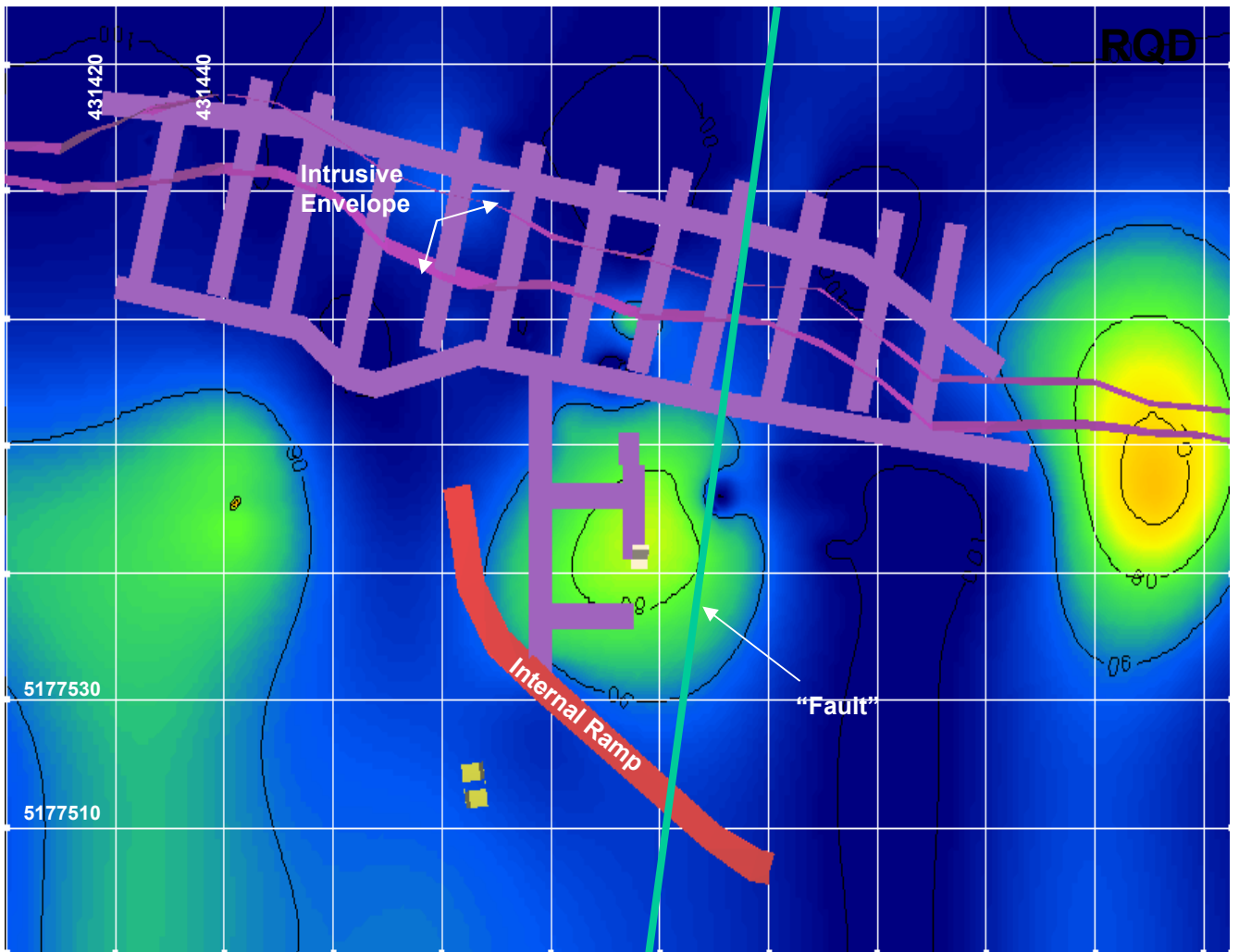
DATE: JANUARY 2005
PROJECT: 04-1193-020



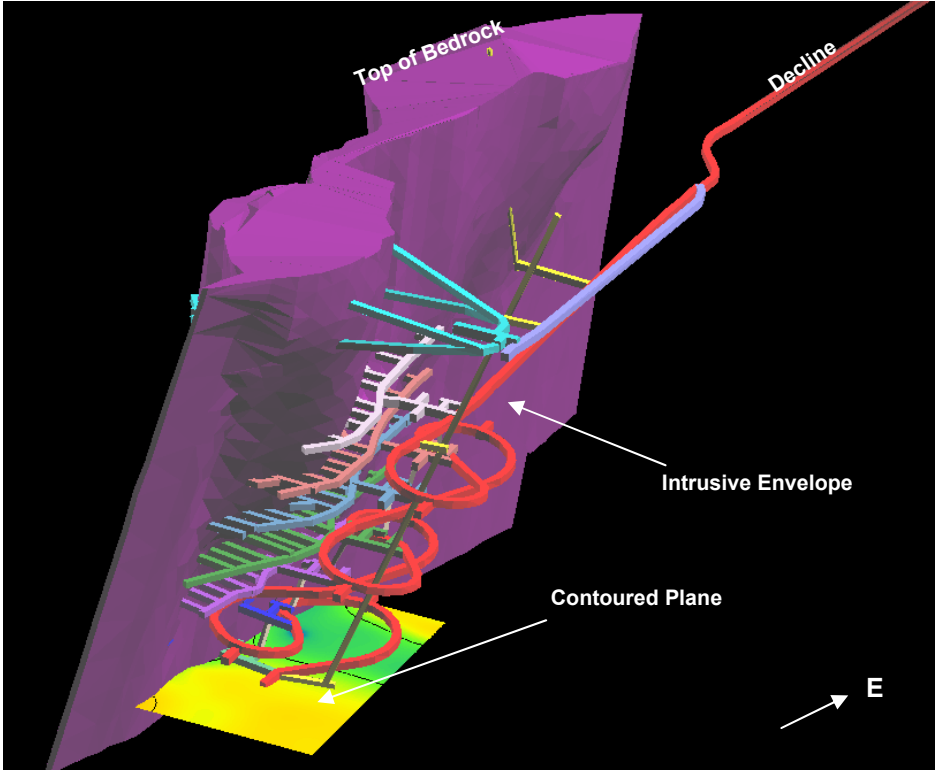
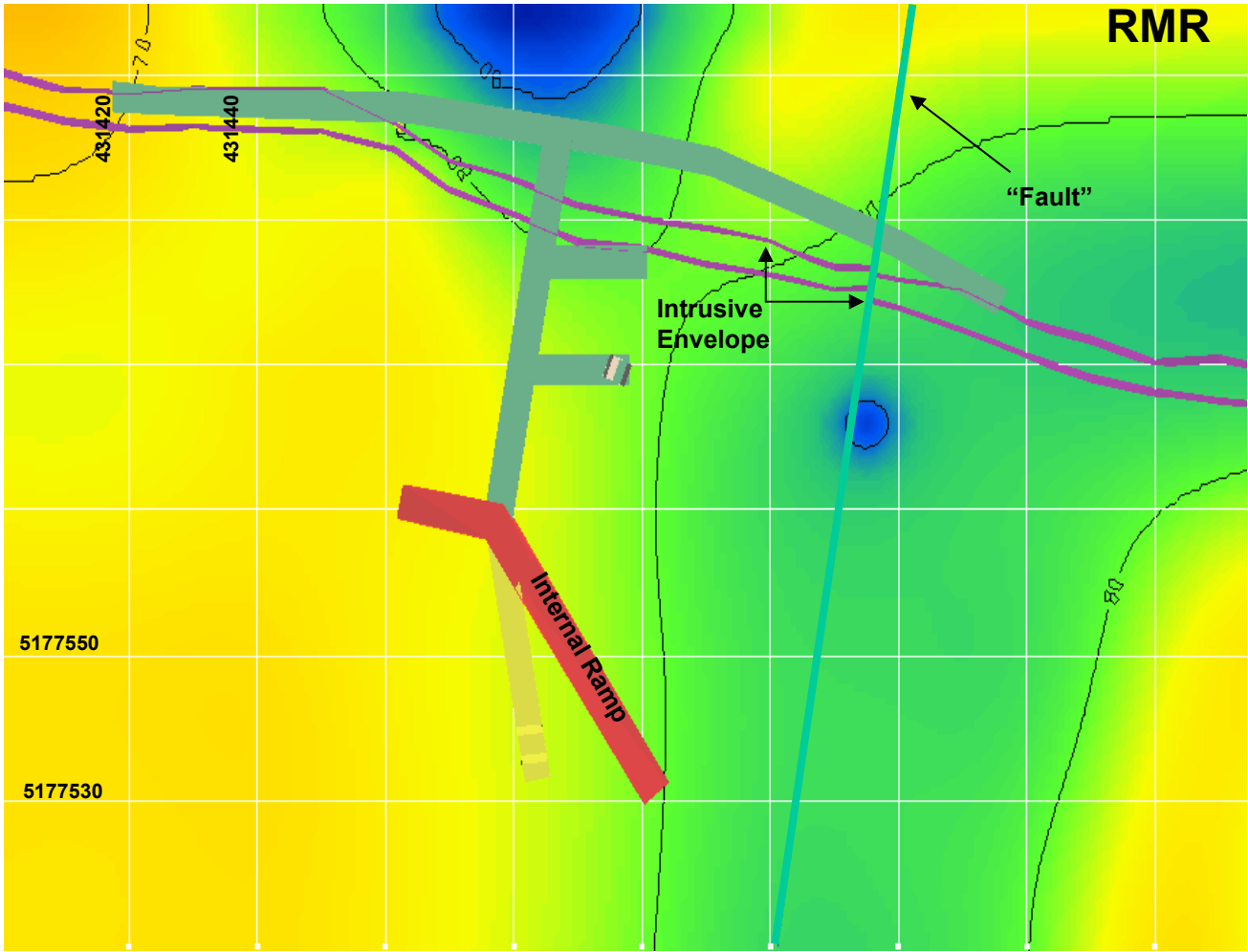
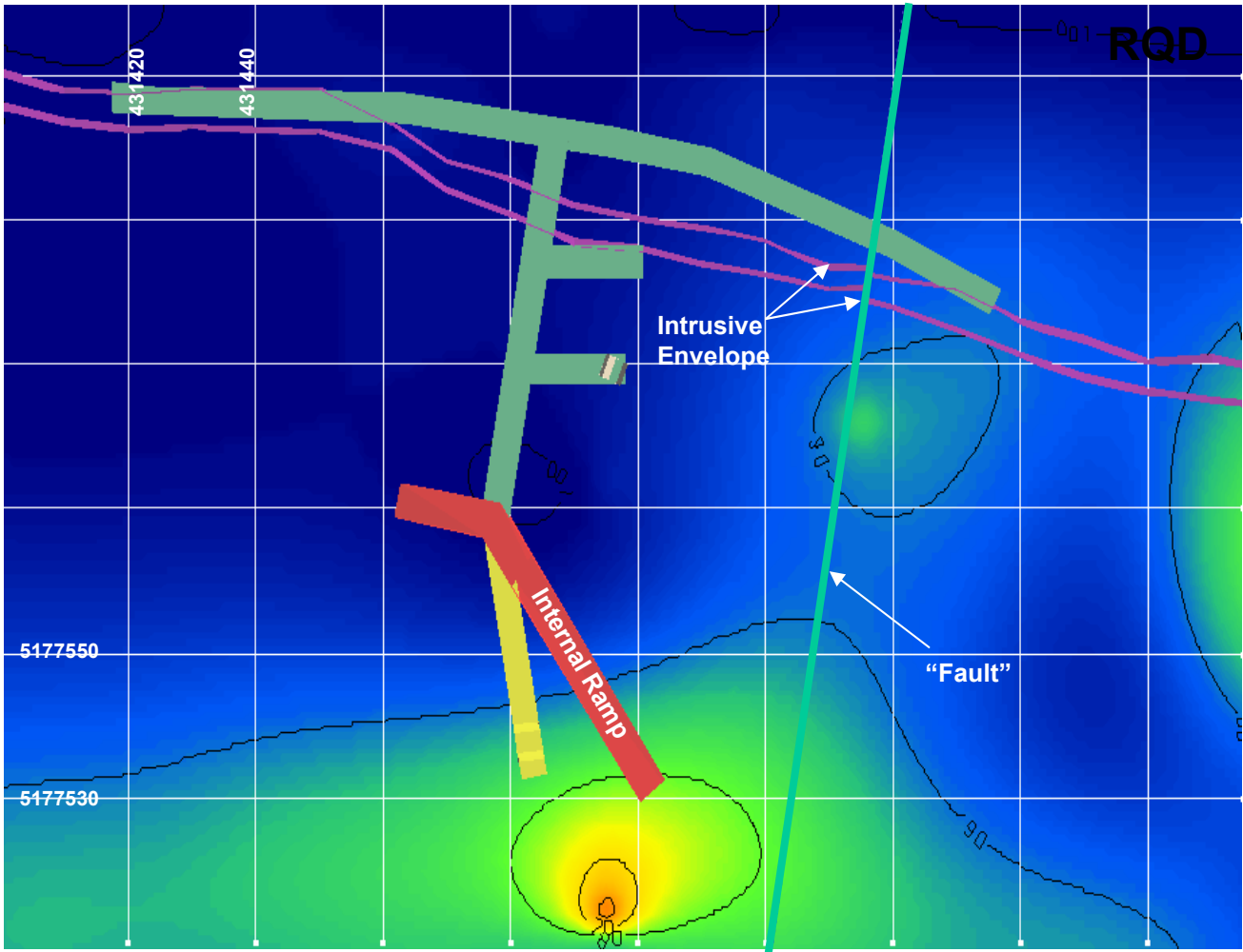
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CHKD..... pp



Plan: RQD and RMR Contouring for 220 Elev.



Plan: RQD and RMR Contouring for 160 Elev.



Plan: RQD and RMR Contouring for 100 Elev.

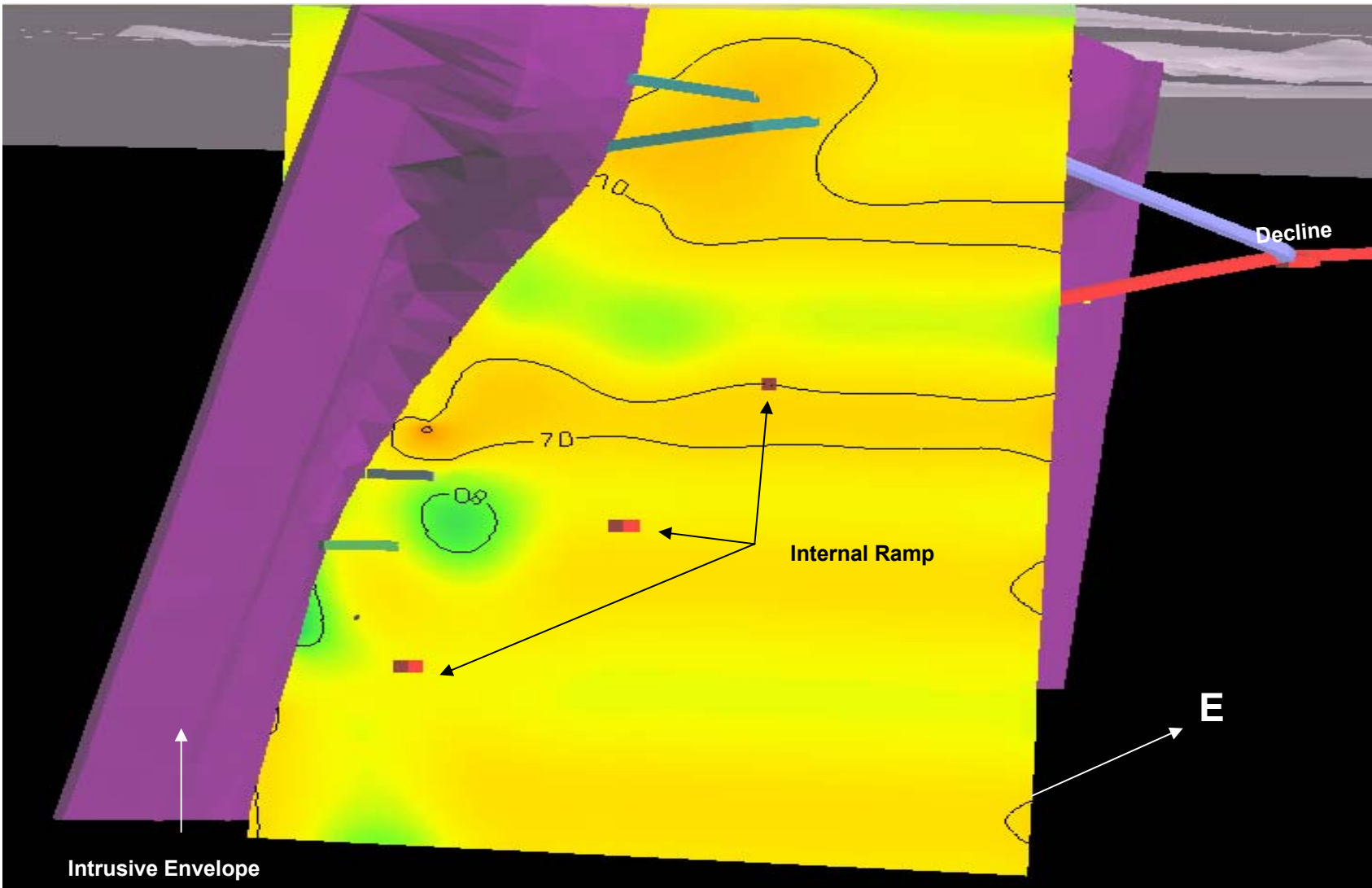
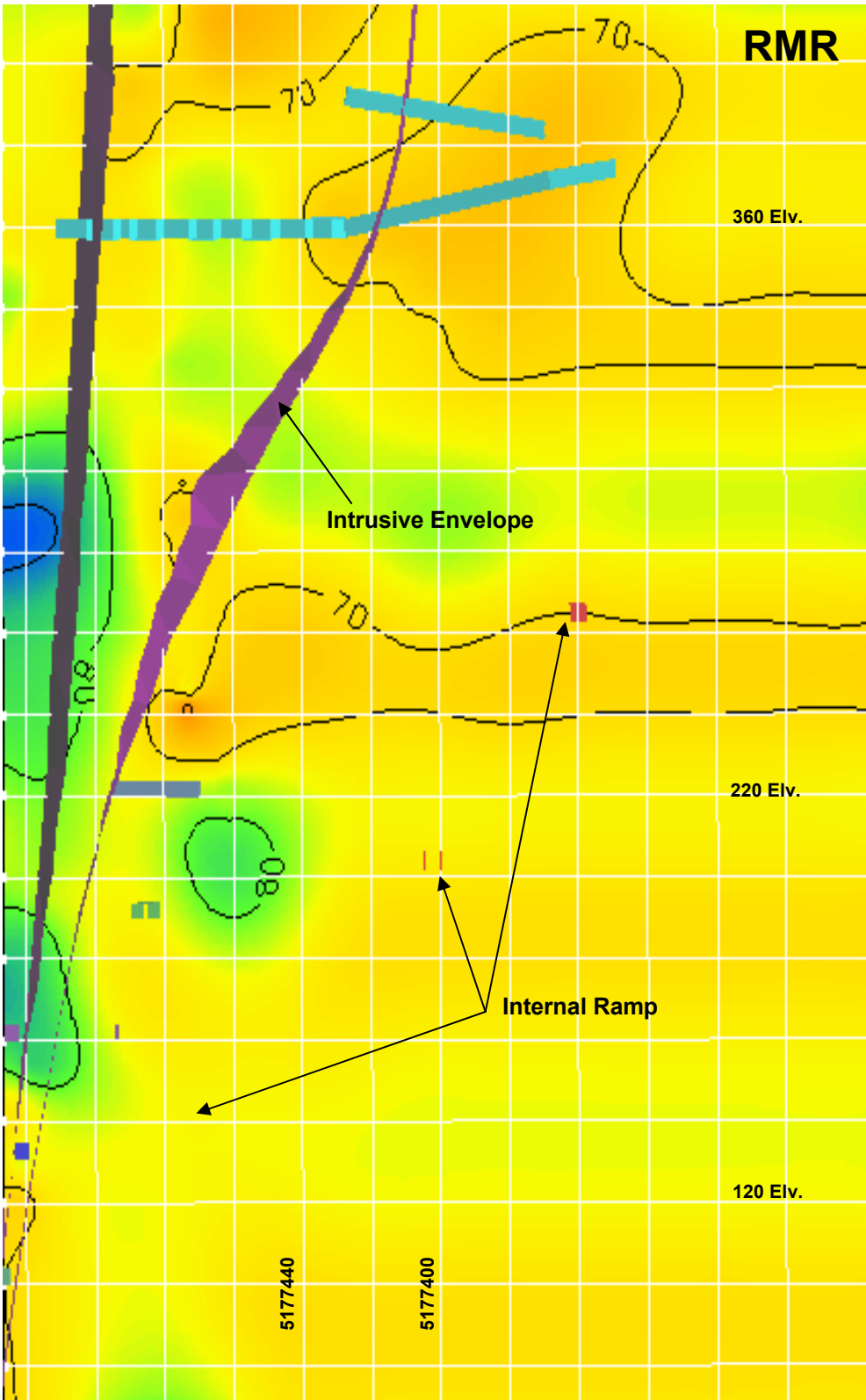
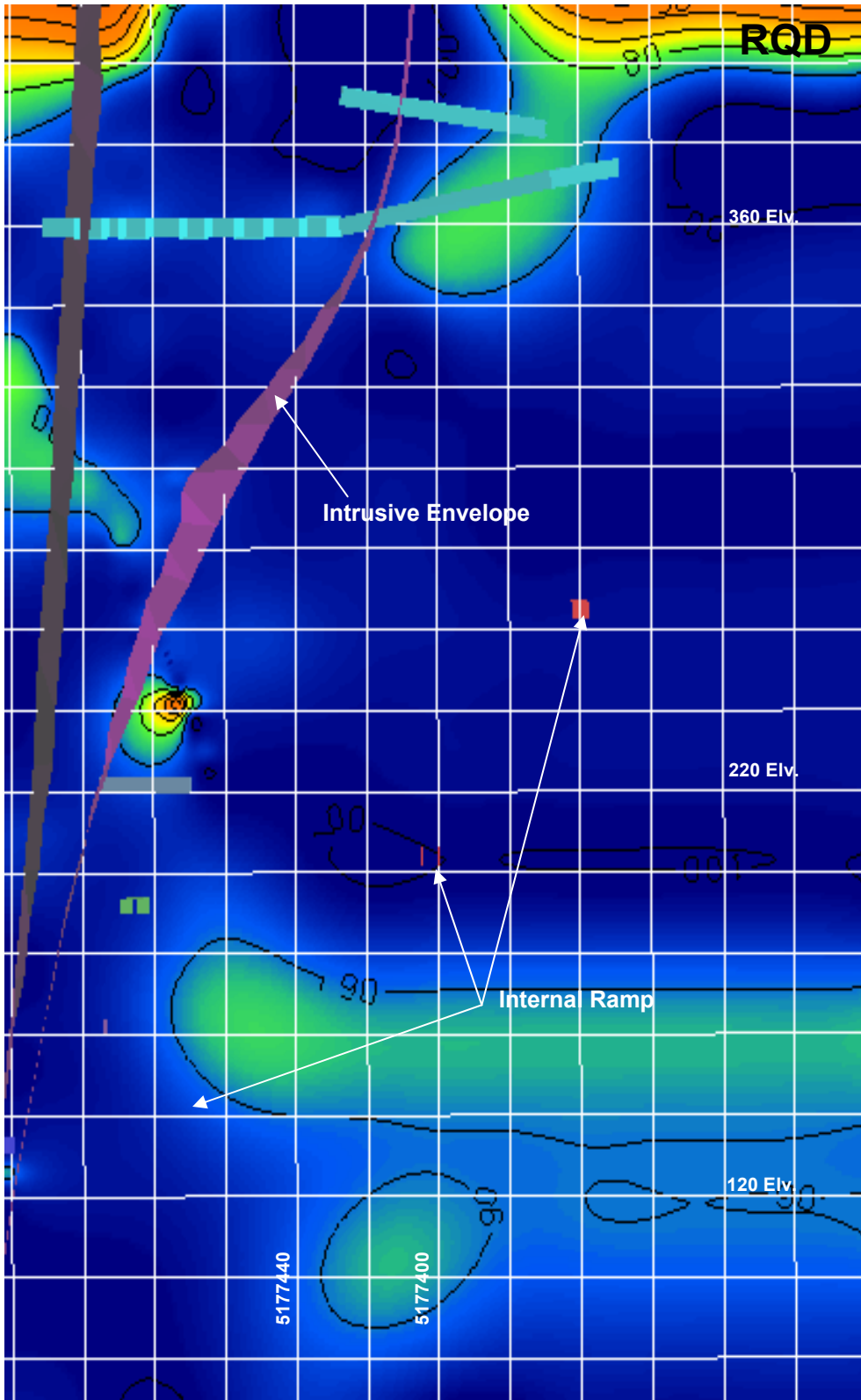


FIGURE 9

Traverse Section: RQD and RMR Contouring for 431420E Facing East

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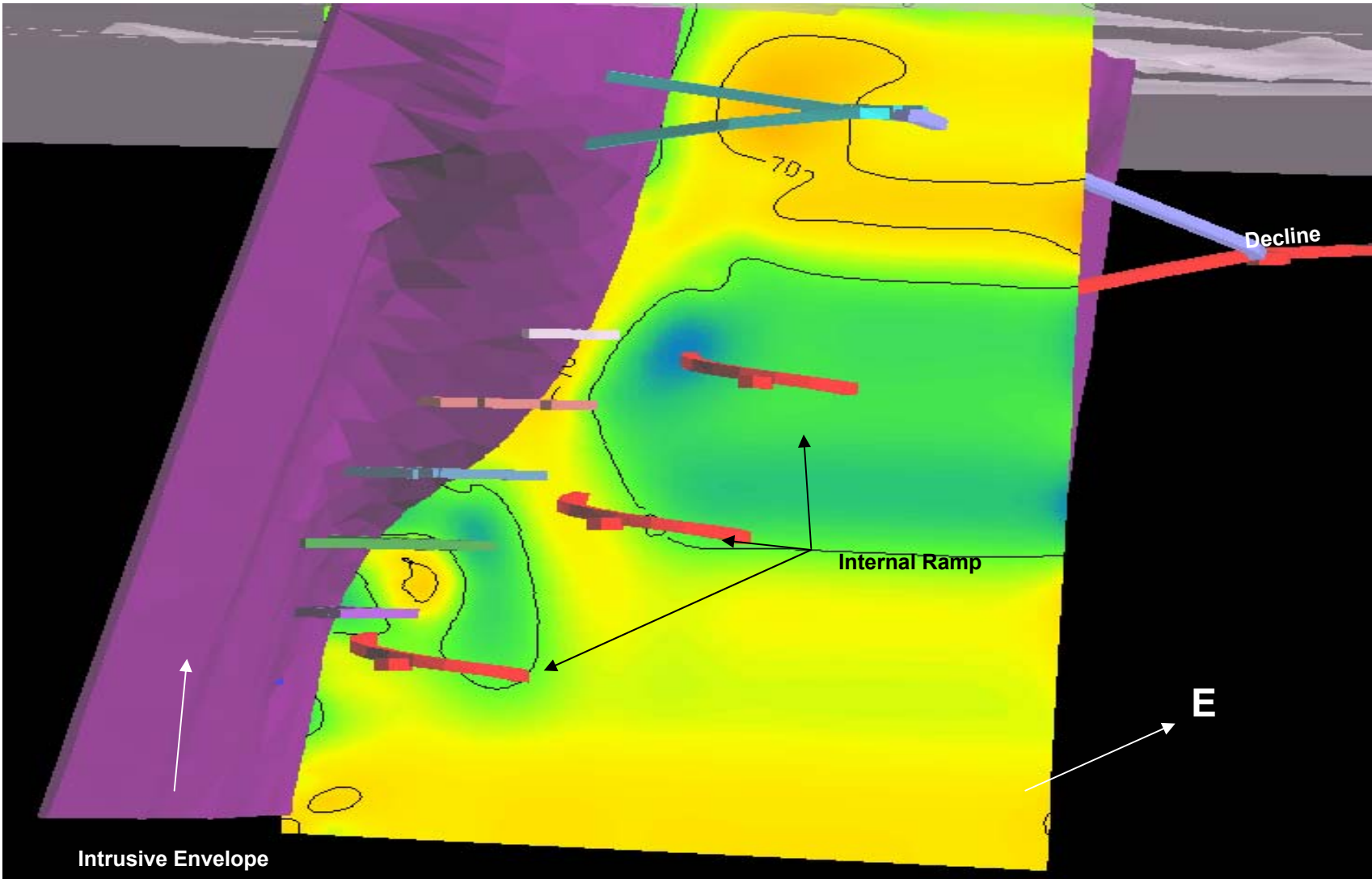
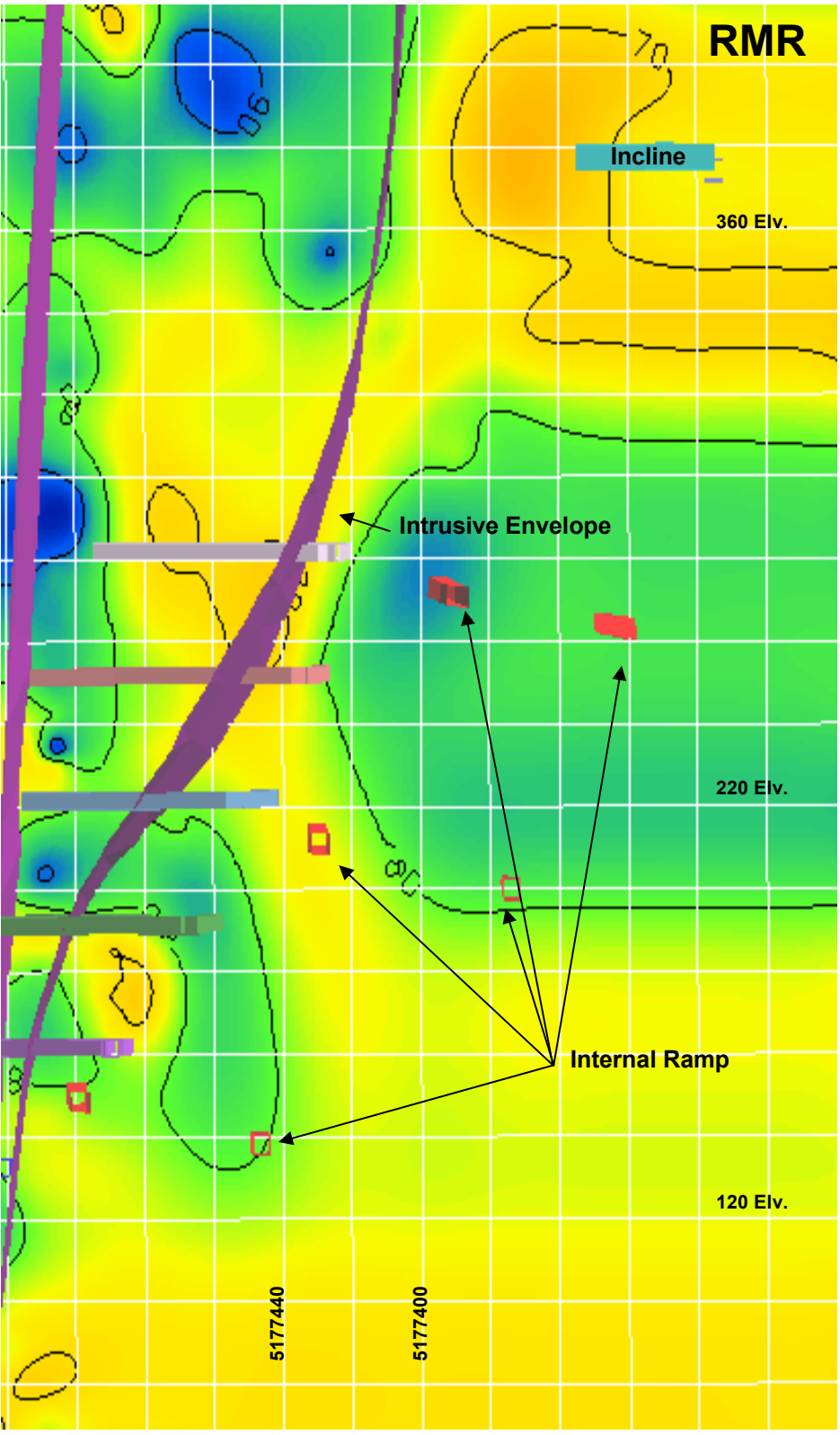
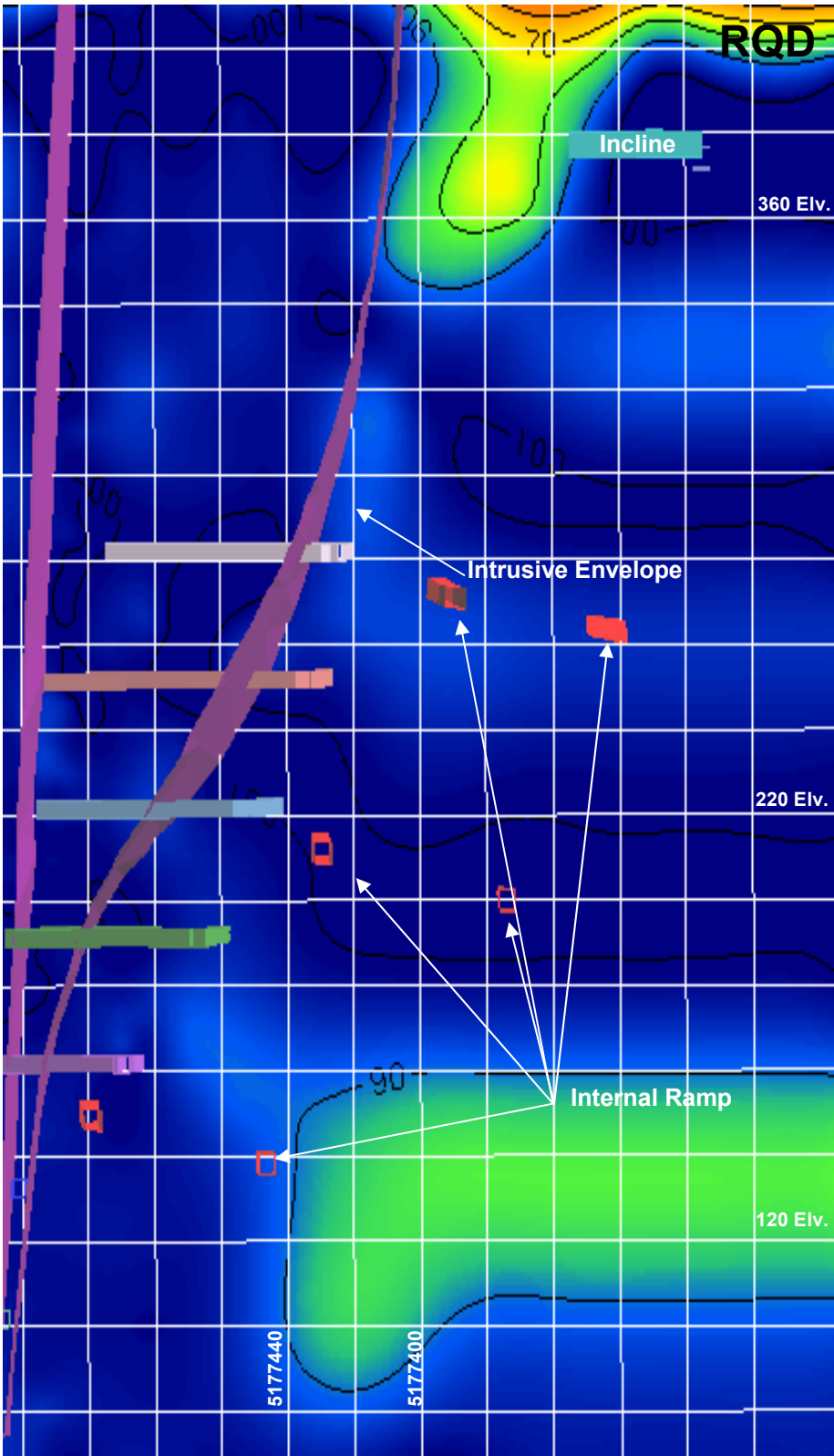


FIGURE 10

Traverse Section: RQD and RMR Contouring for 431457E Facing East

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PROJECT: 04-1193-020



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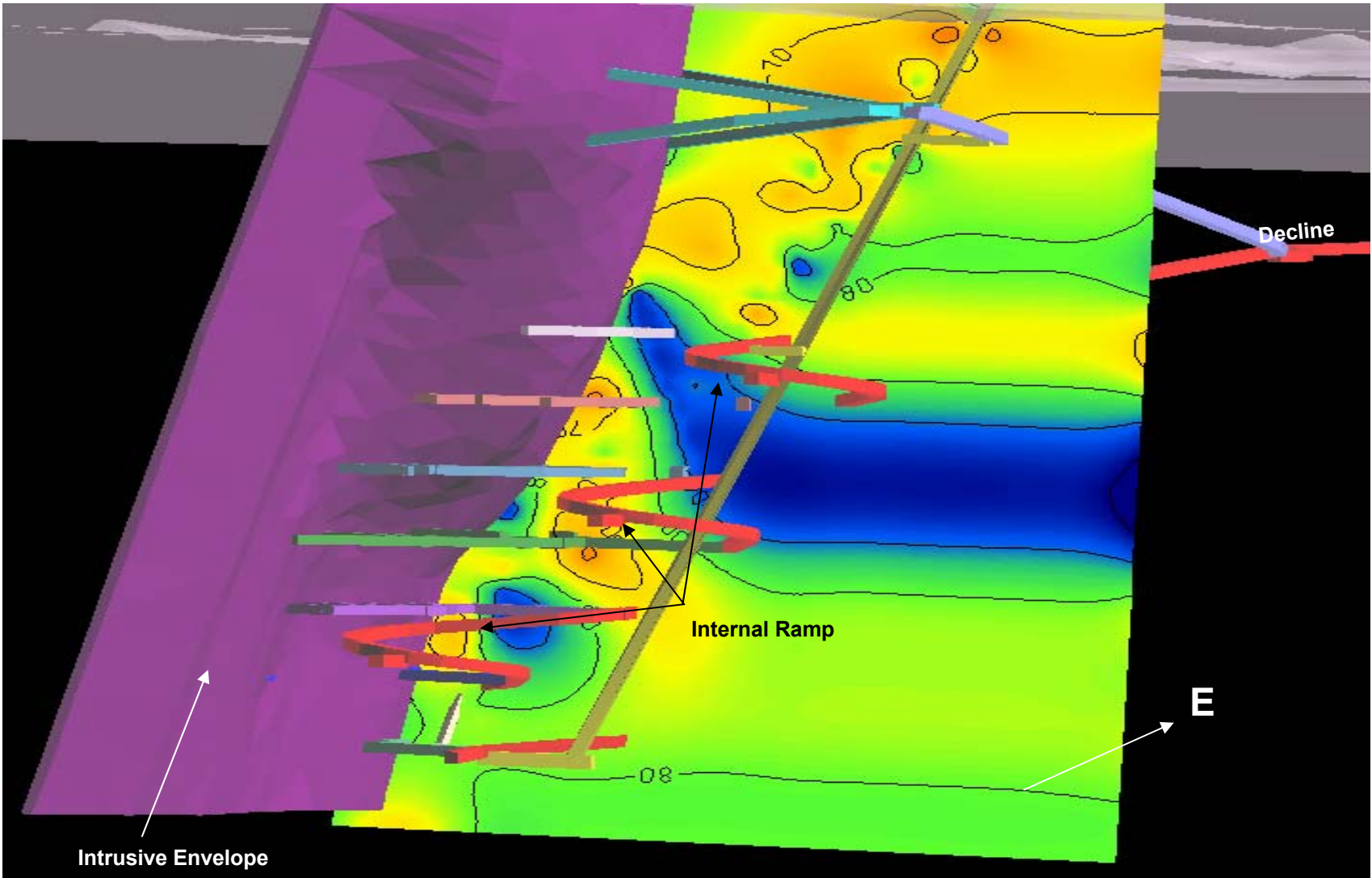
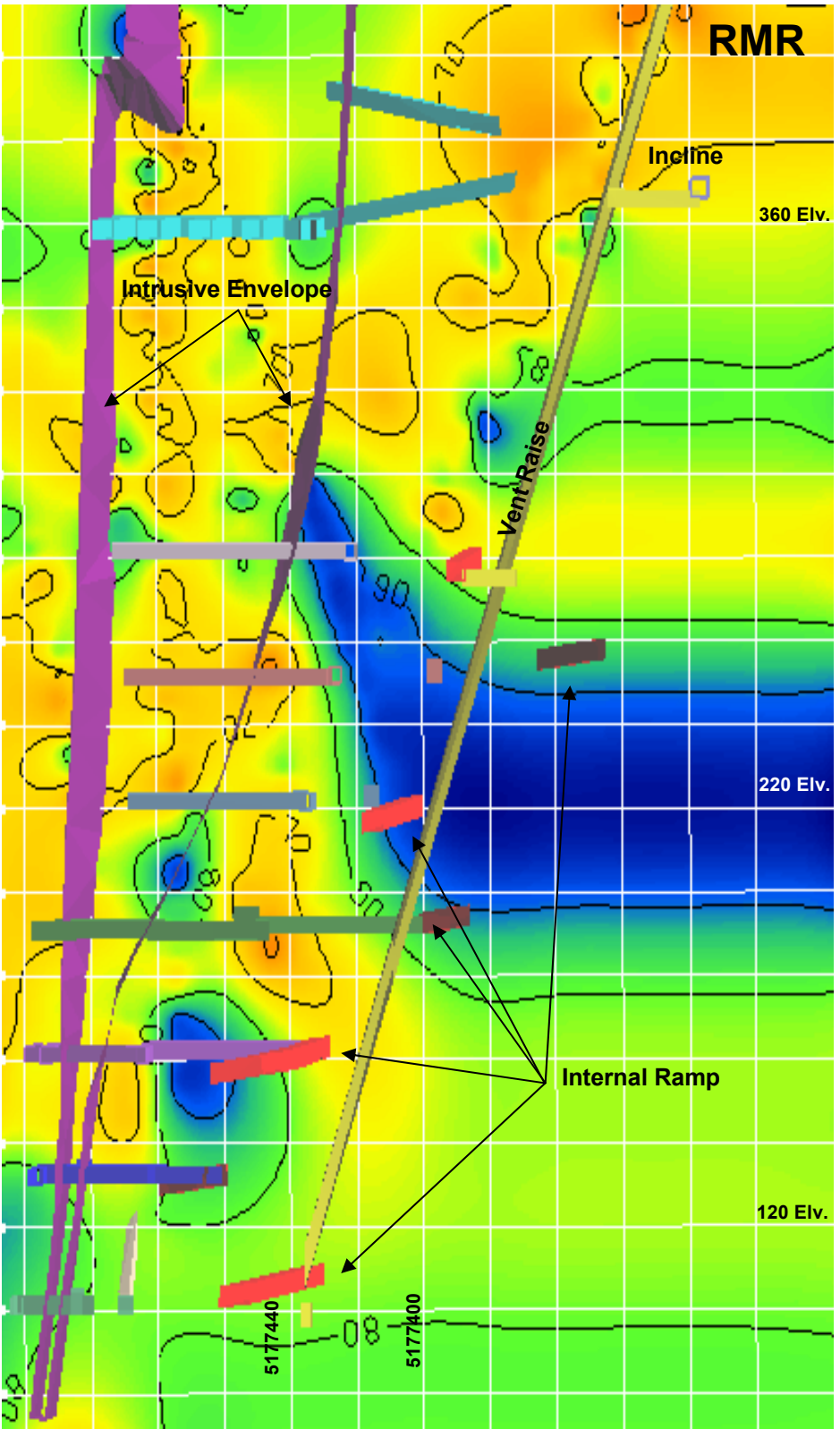
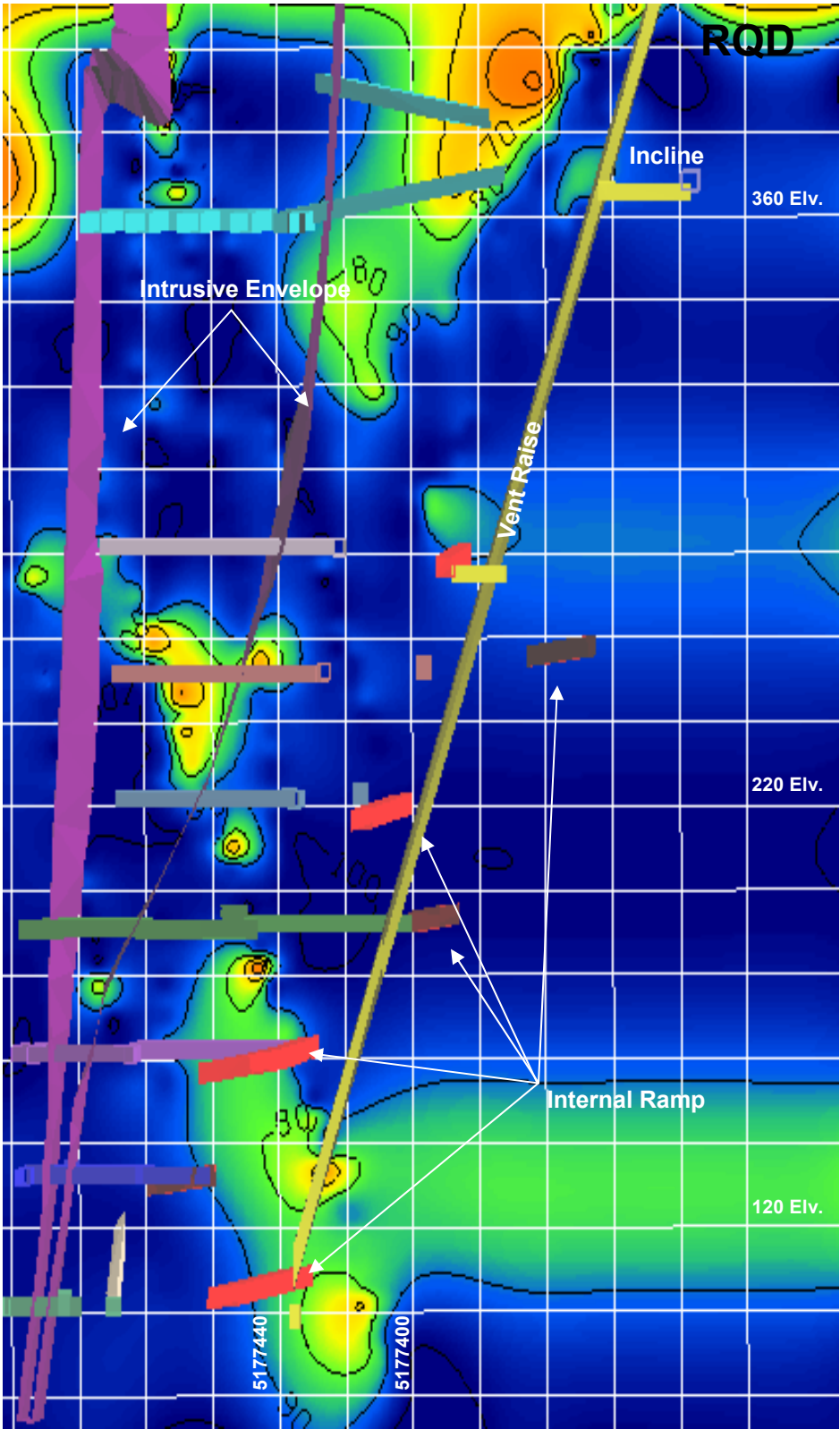


FIGURE 11	Traverse Section: RQD and RMR Contouring for 431500E Facing East		
	DATE: JANUARY 2005 PROJECT: 04-1193-020	Golder Associates	DRAWN..... TV CHKD..... pp

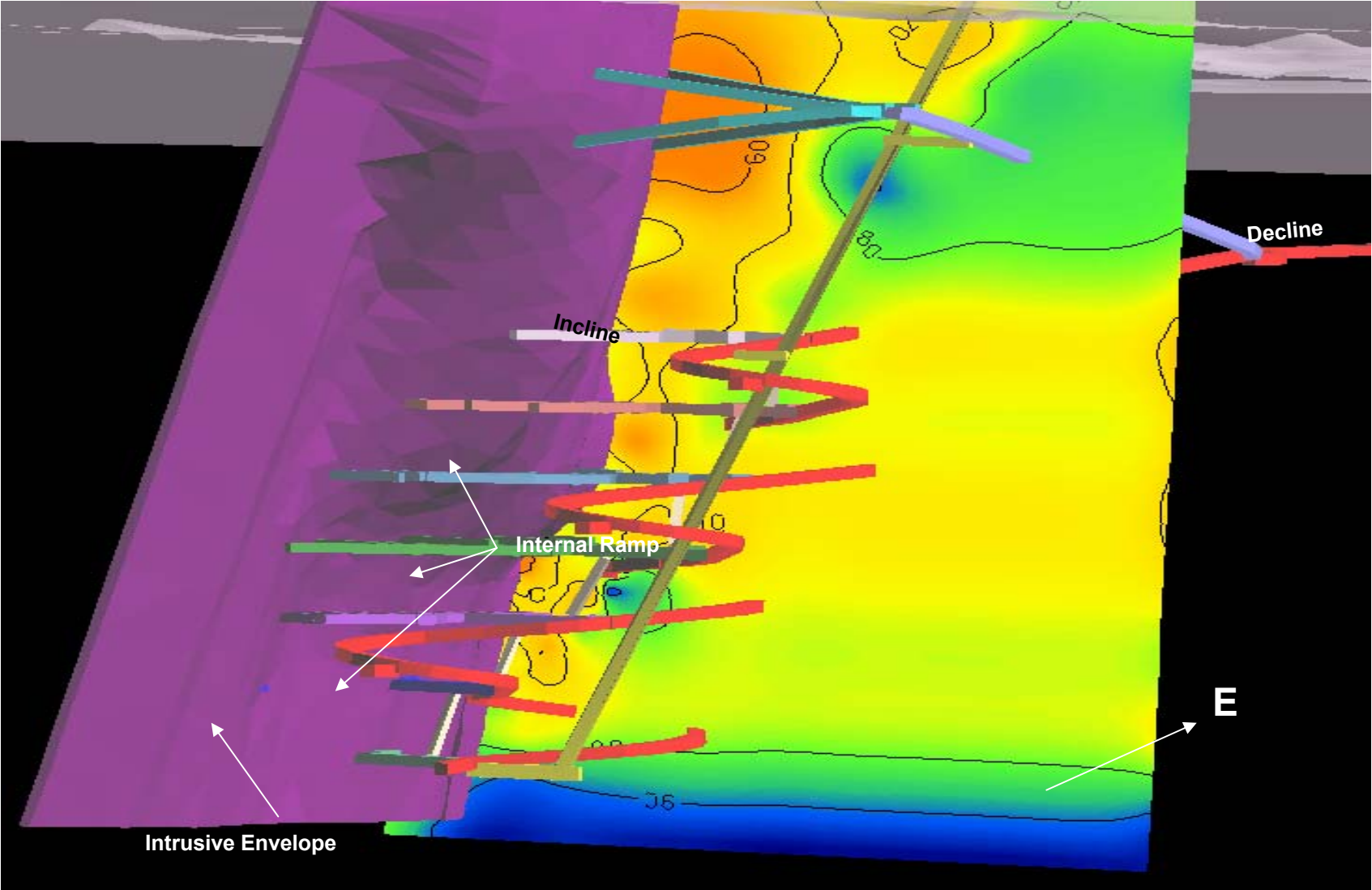
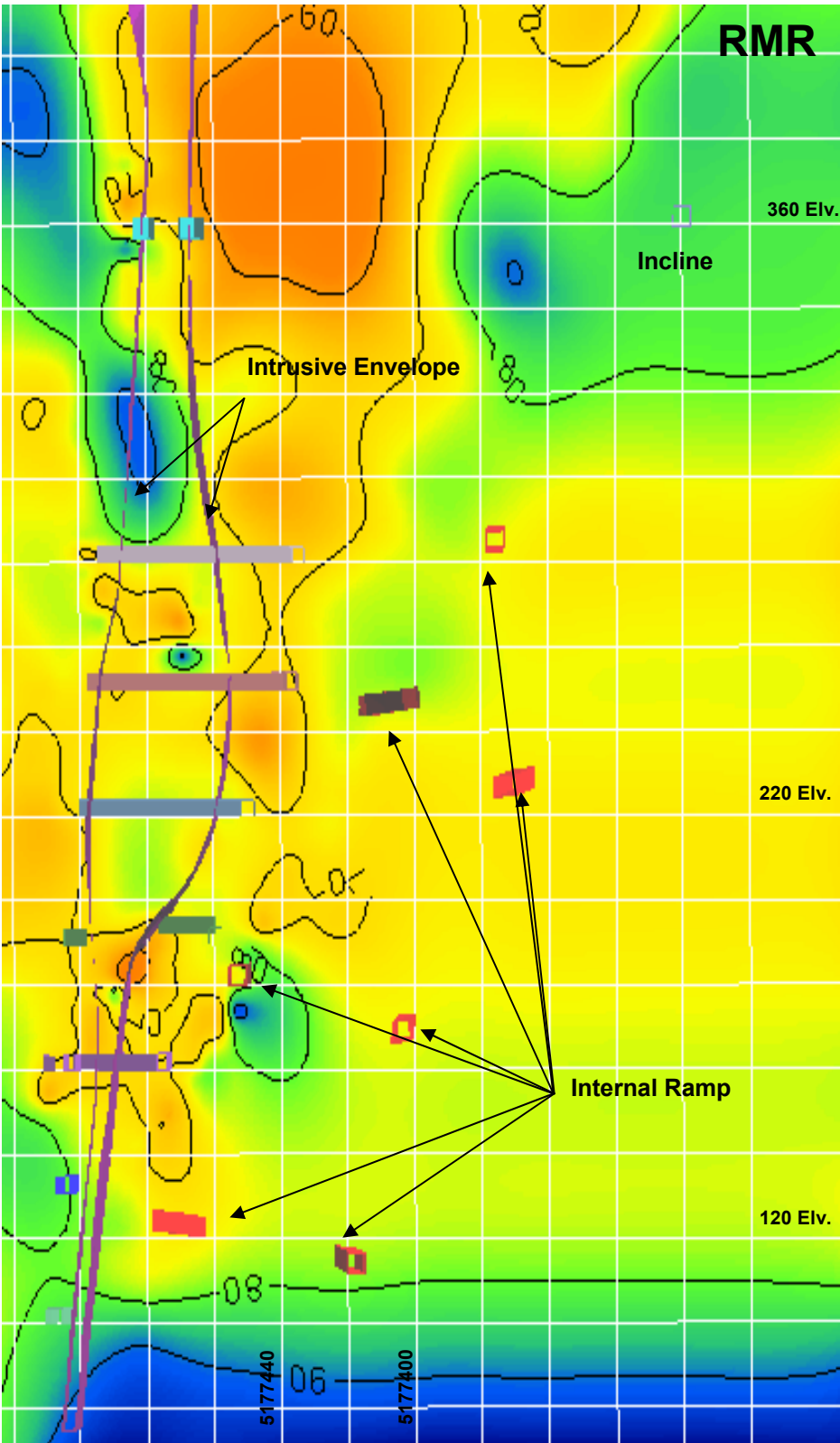
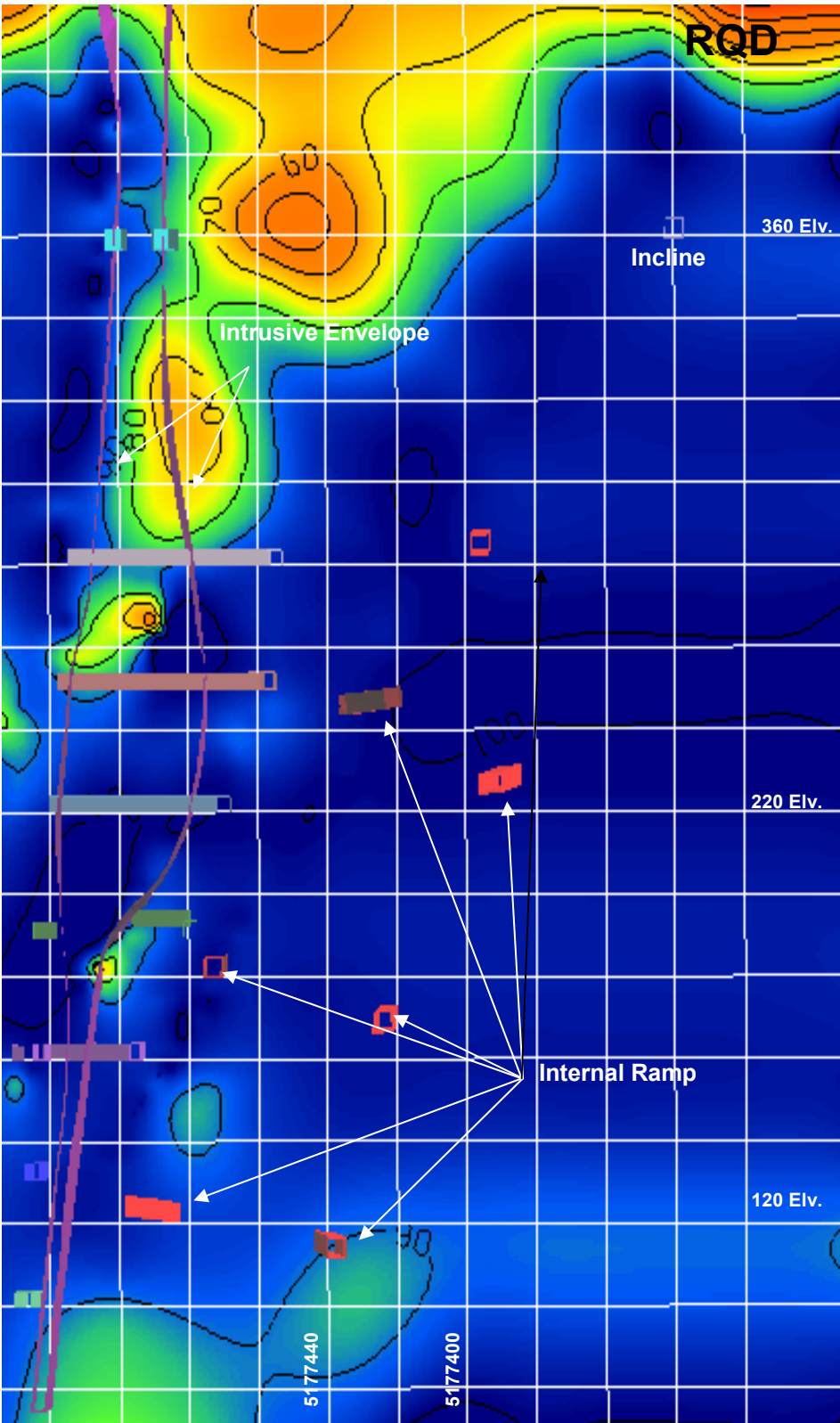


FIGURE 12

Traverse Section: RQD and RMR Contouring for 431545E Facing East

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PROJECT: 04-1193-020



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CHKD..... pp

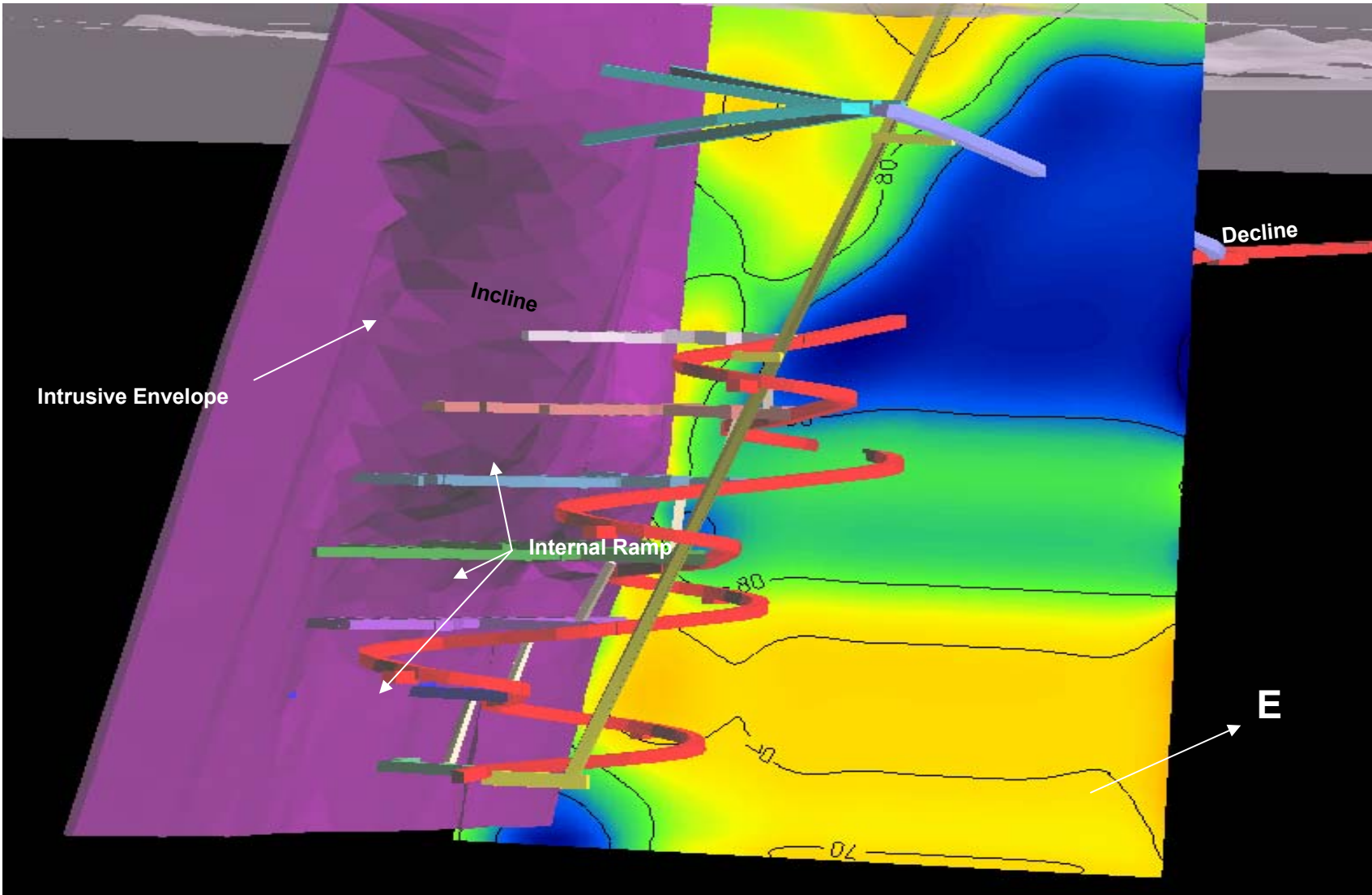
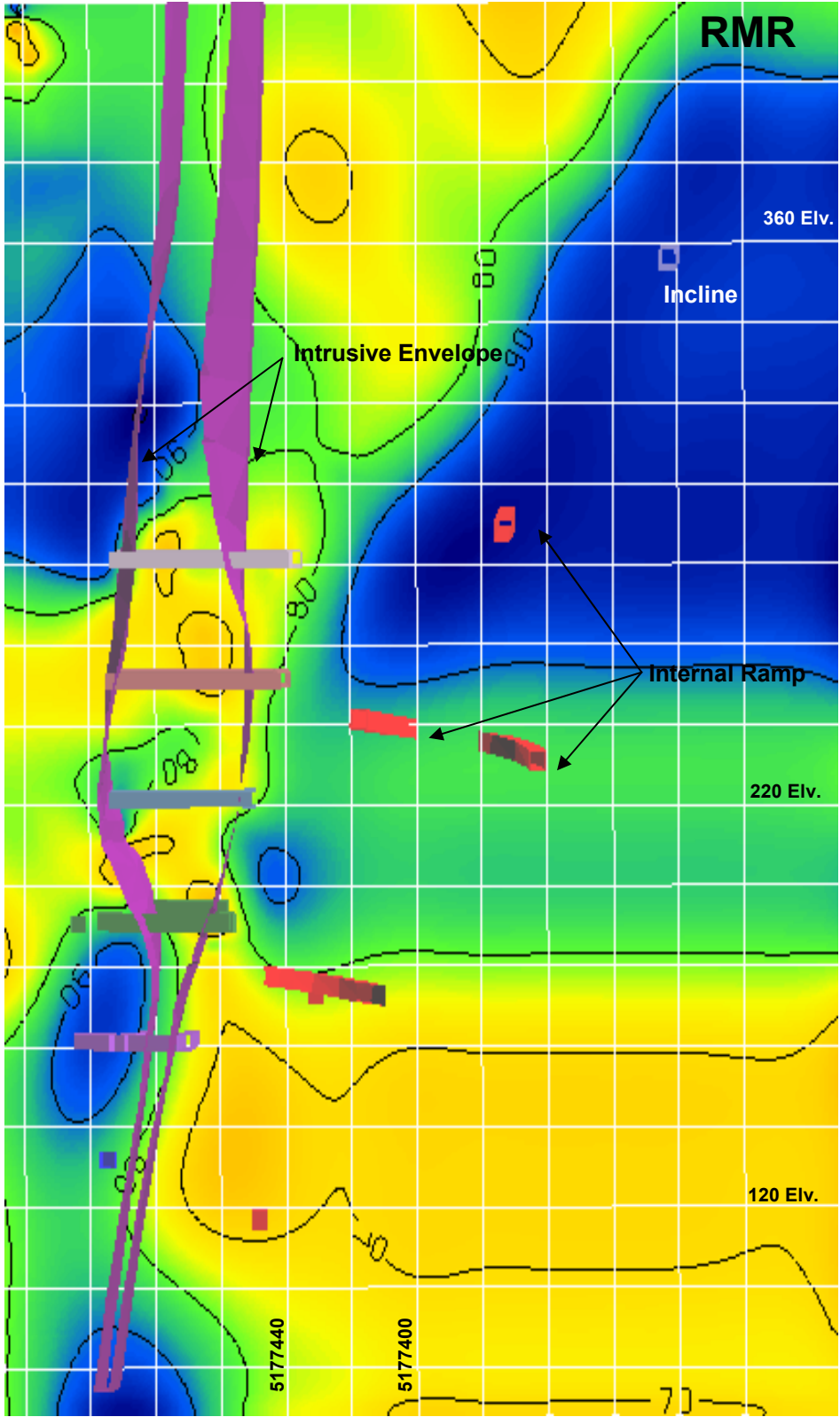
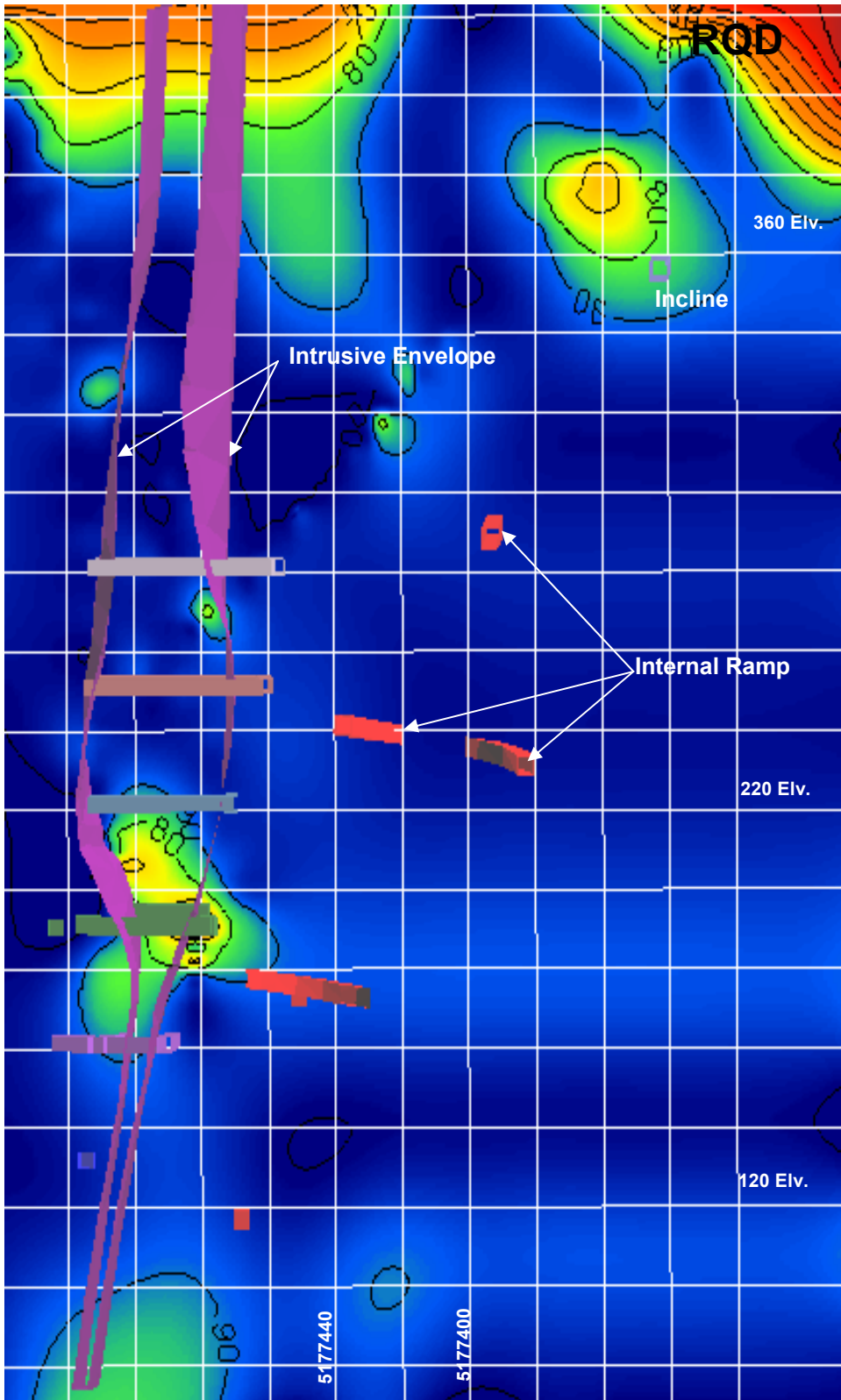


FIGURE 13	Traverse Section: RQD and RMR Contouring for 431585E Facing East		
	DATE: JANUARY 2005 PROJECT: 04-1193-020		DRAWN..... TV CHKD..... pp

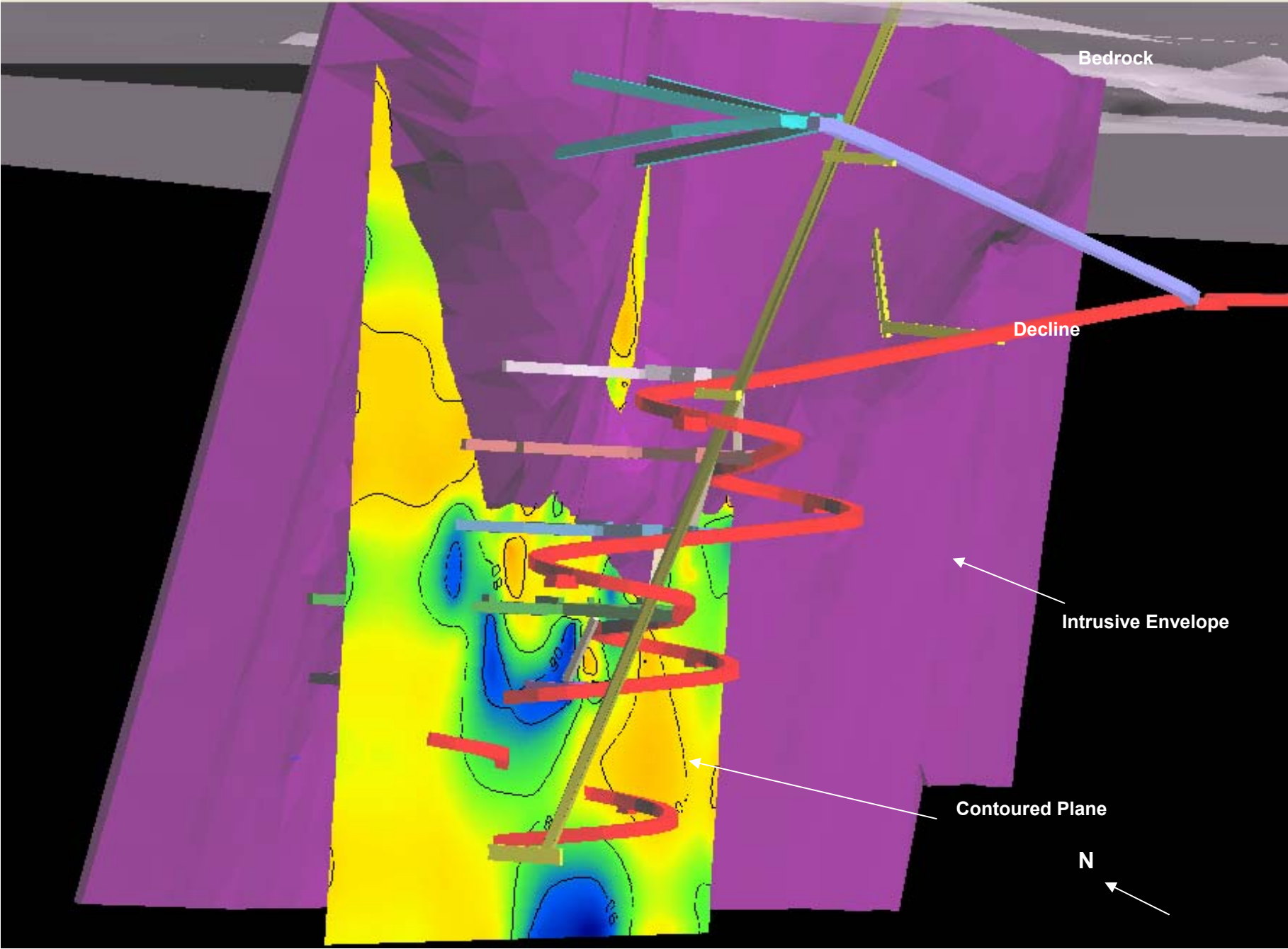
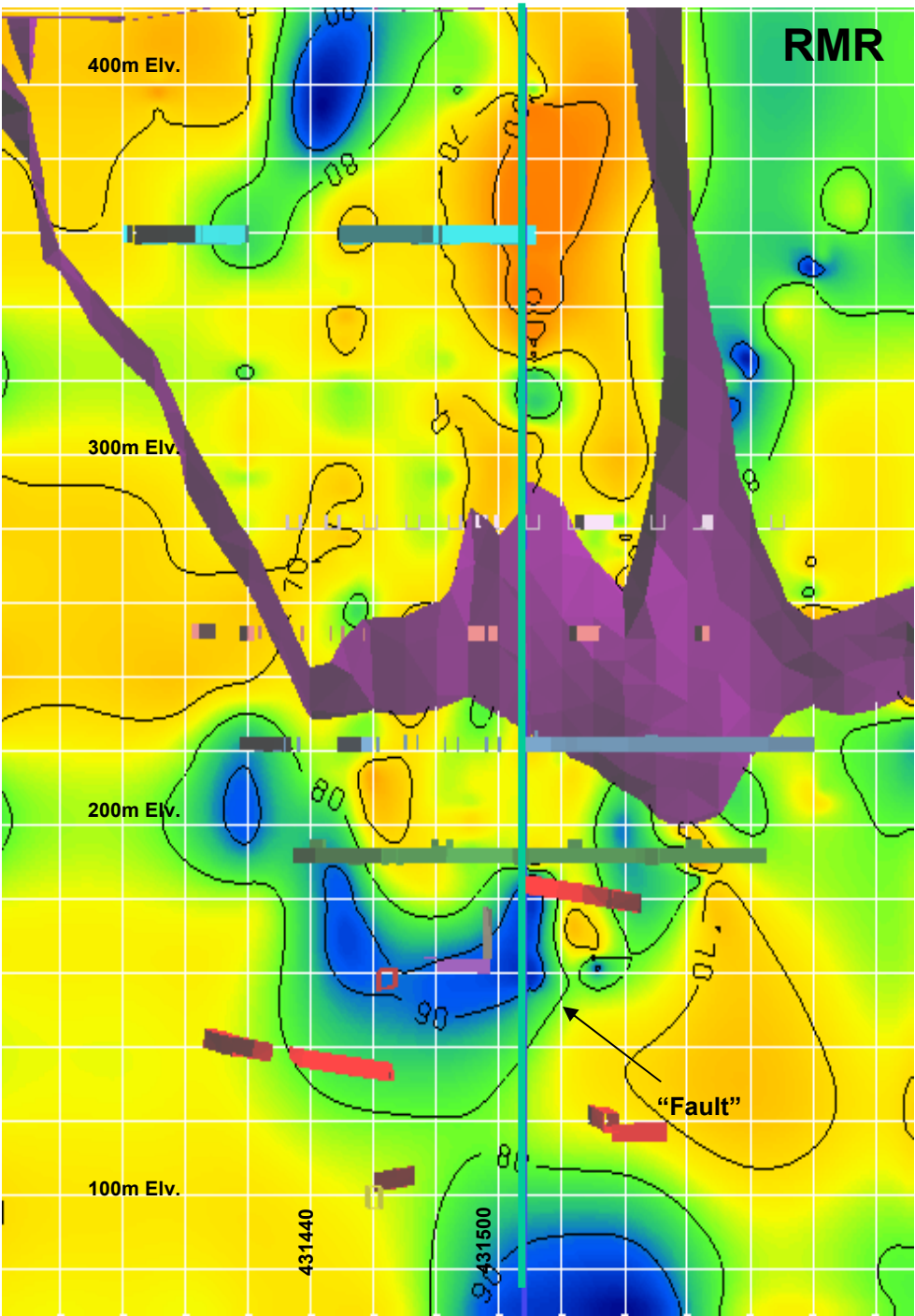
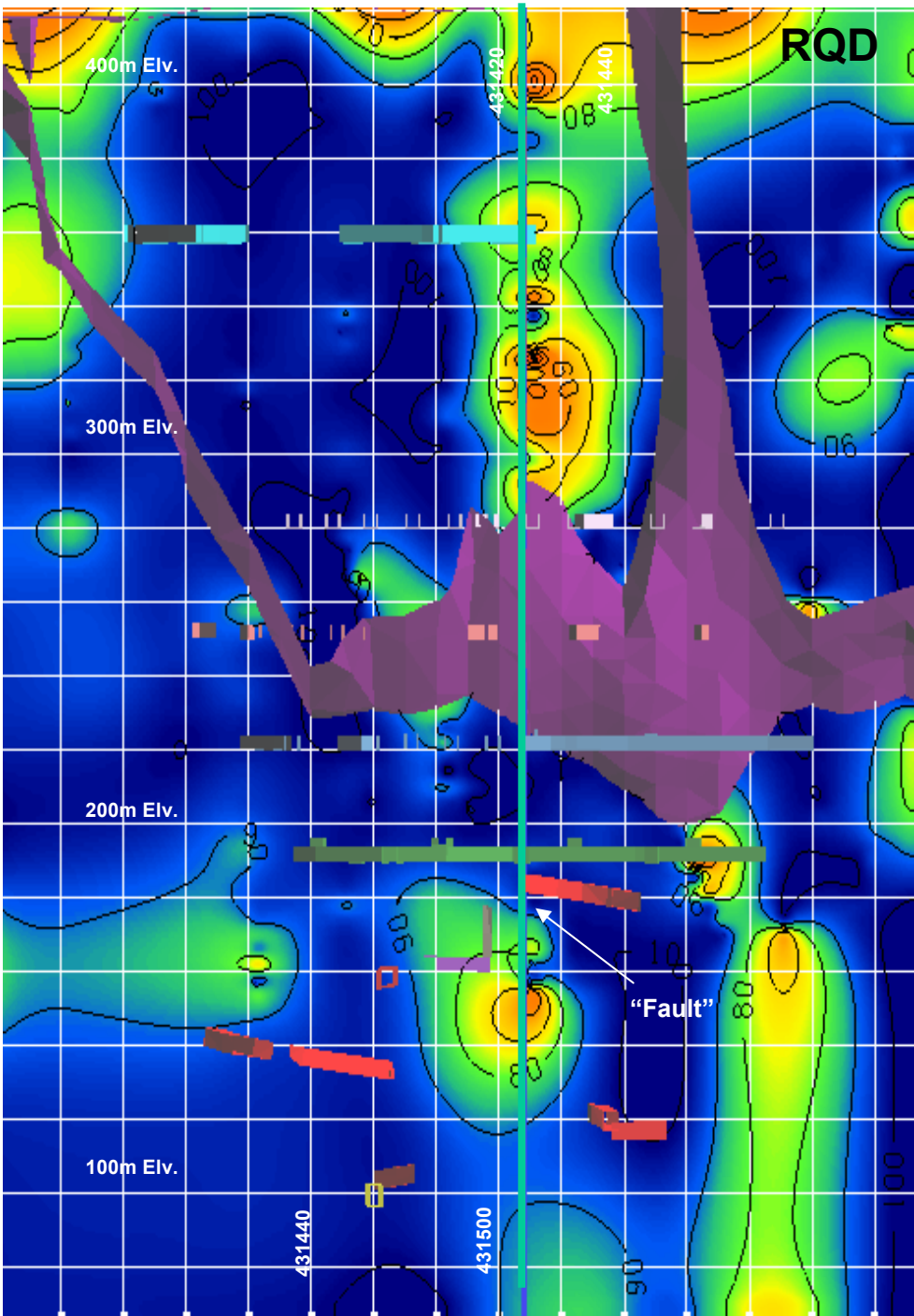


FIGURE 15

Longitudinal Section: RQD and RMR Contouring for 5177560N Facing North

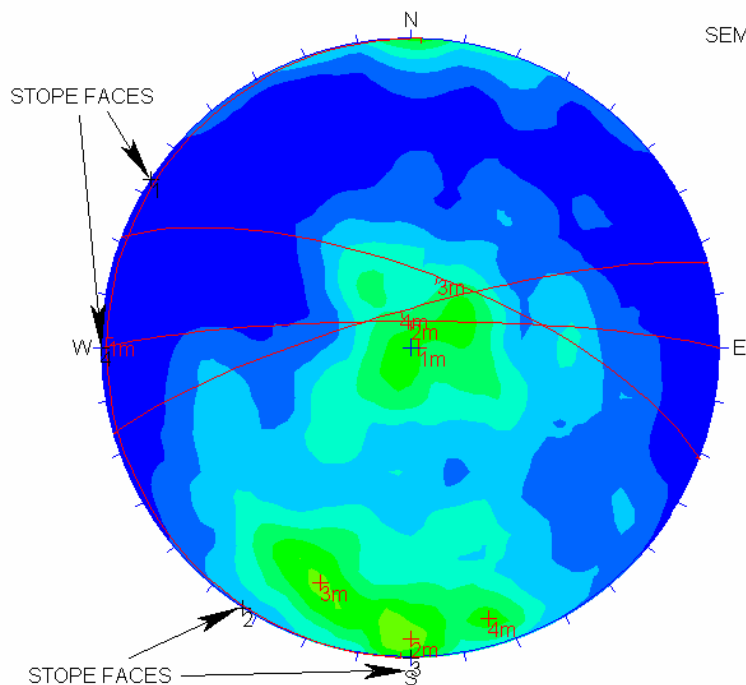
DATE: JANUARY 2005
PROJECT: 04-1193-020



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CHKD..... pp

MAJOR DISCONTINUITY SETS (ALL FEATURES) SEMI-MASSIVE SUPHIDE AND MASSIVE SUPHIDE

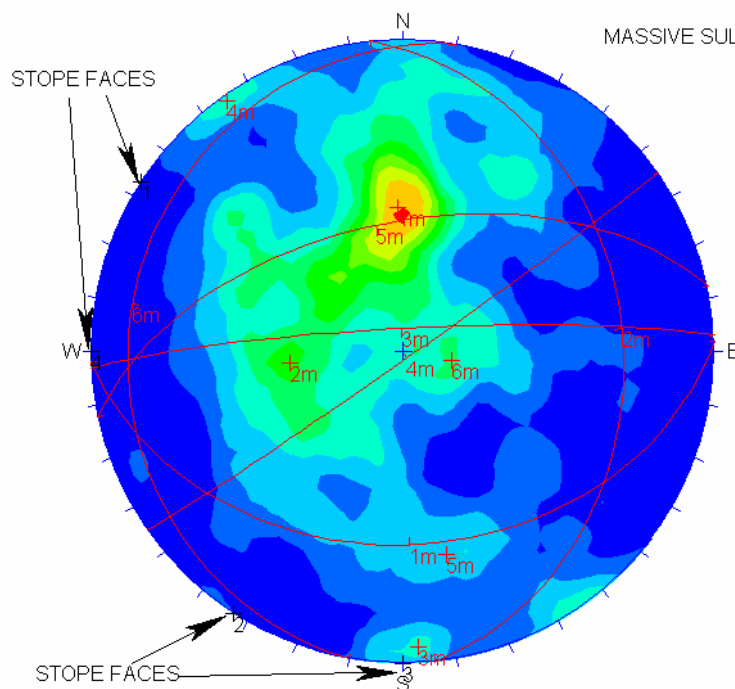
FIGURE 16



SEMI-MASSIVE SULPHIDE

Orientations	
ID	Dip / Direction
1	90 / 123
2	90 / 033
3	90 / 360
4	90 / 090
1 m	02 / 272
2 m	83 / 360
3 m	70 / 021
4 m	80 / 344

Equal Area
Lower Hemisphere
1857 Poles
1637 Entries



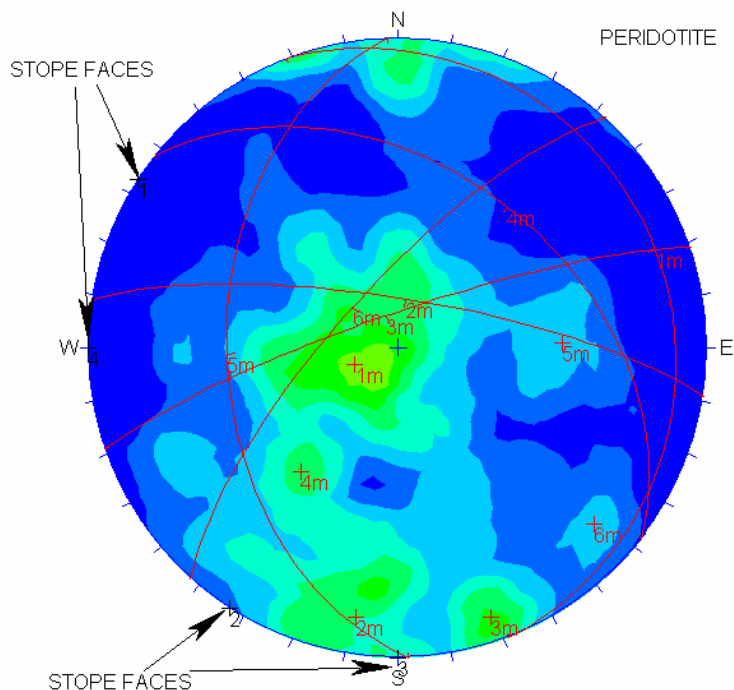
MASSIVE SULPHIDE

Orientations	
ID	Dip / Direction
1	90 / 123
2	90 / 033
3	90 / 360
4	90 / 090
1 m	38 / 178
2 m	30 / 084
3 m	84 / 357
4 m	88 / 145
5 m	56 / 348
6 m	13 / 280

Equal Area
Lower Hemisphere
625 Poles
621 Entries

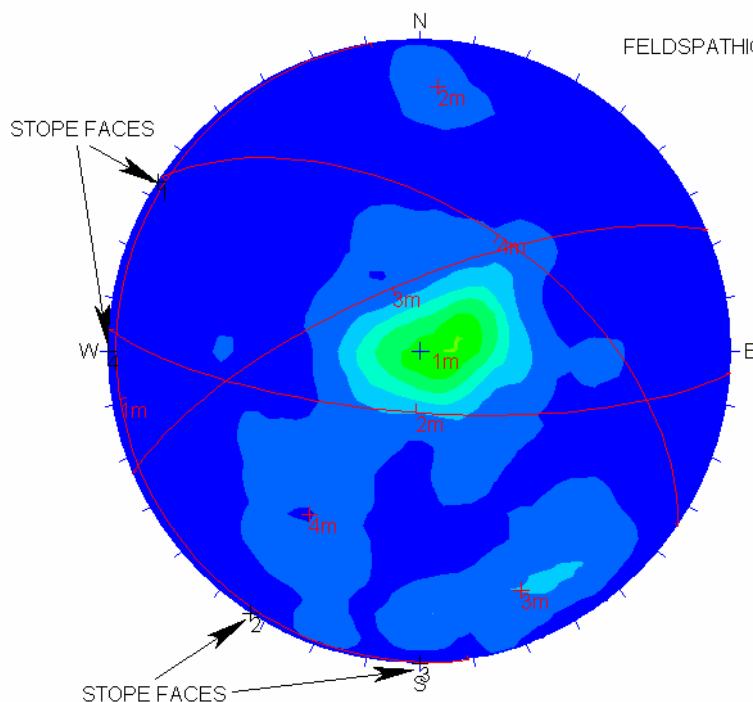
MAJOR DISCONTINUITY SETS (ALL FEATURES) PERIDOTITE AND FELDSPAR PERIDOTITE

FIGURE 17



Orientations	
ID	Dip / Direction
1	90 / 123
2	90 / 033
3	90 / 360
4	90 / 090
1 m	12 / 069
2 m	77 / 009
3 m	81 / 341
4 m	42 / 038
5 m	44 / 268
6 m	74 / 312

Equal Area
Lower Hemisphere
848 Poles
827 Entries

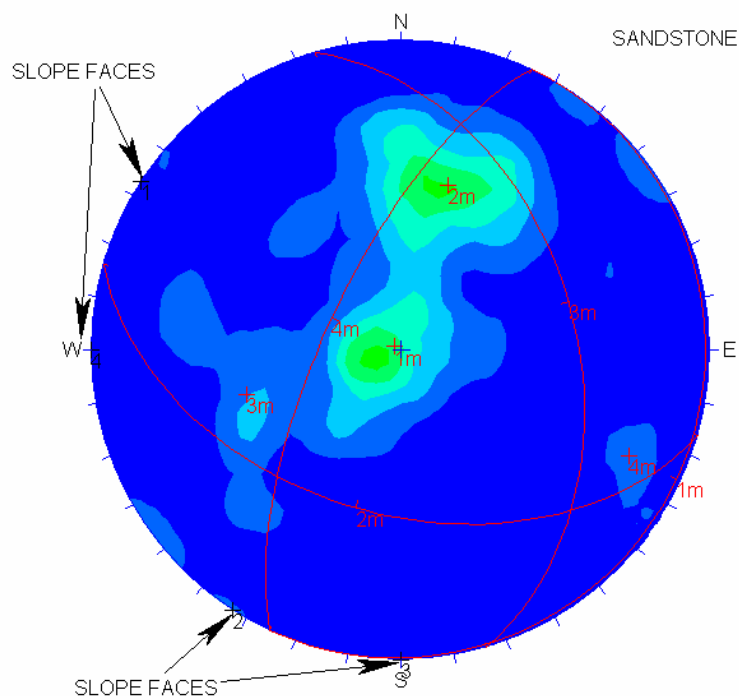


Orientations	
ID	Dip / Direction
1	90 / 123
2	90 / 033
3	90 / 360
4	90 / 090
1 m	03 / 261
2 m	74 / 184
3 m	72 / 337
4 m	53 / 034

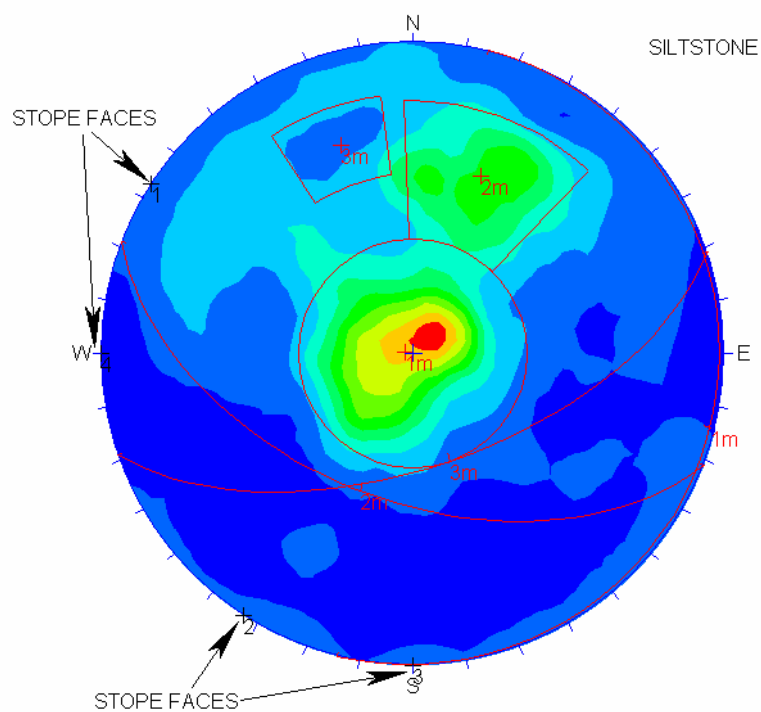
Equal Area
Lower Hemisphere
1337 Poles
1337 Entries

MAJOR DISCONTINUITY SETS (ALL FEATURES) SANDSTONE AND SILTSTONE

FIGURE 18



Orientations	
ID	Dip / Direction
1	90 / 123
2	90 / 033
3	90 / 360
4	90 / 090
1 m	02 / 115
2 m	46 / 196
3 m	43 / 074
4 m	70 / 295



Orientations	
ID	Dip / Direction
1	90 / 123
2	90 / 033
3	90 / 360
4	90 / 090
1 m	02 / 104
2 m	51 / 201
3 m	60 / 161

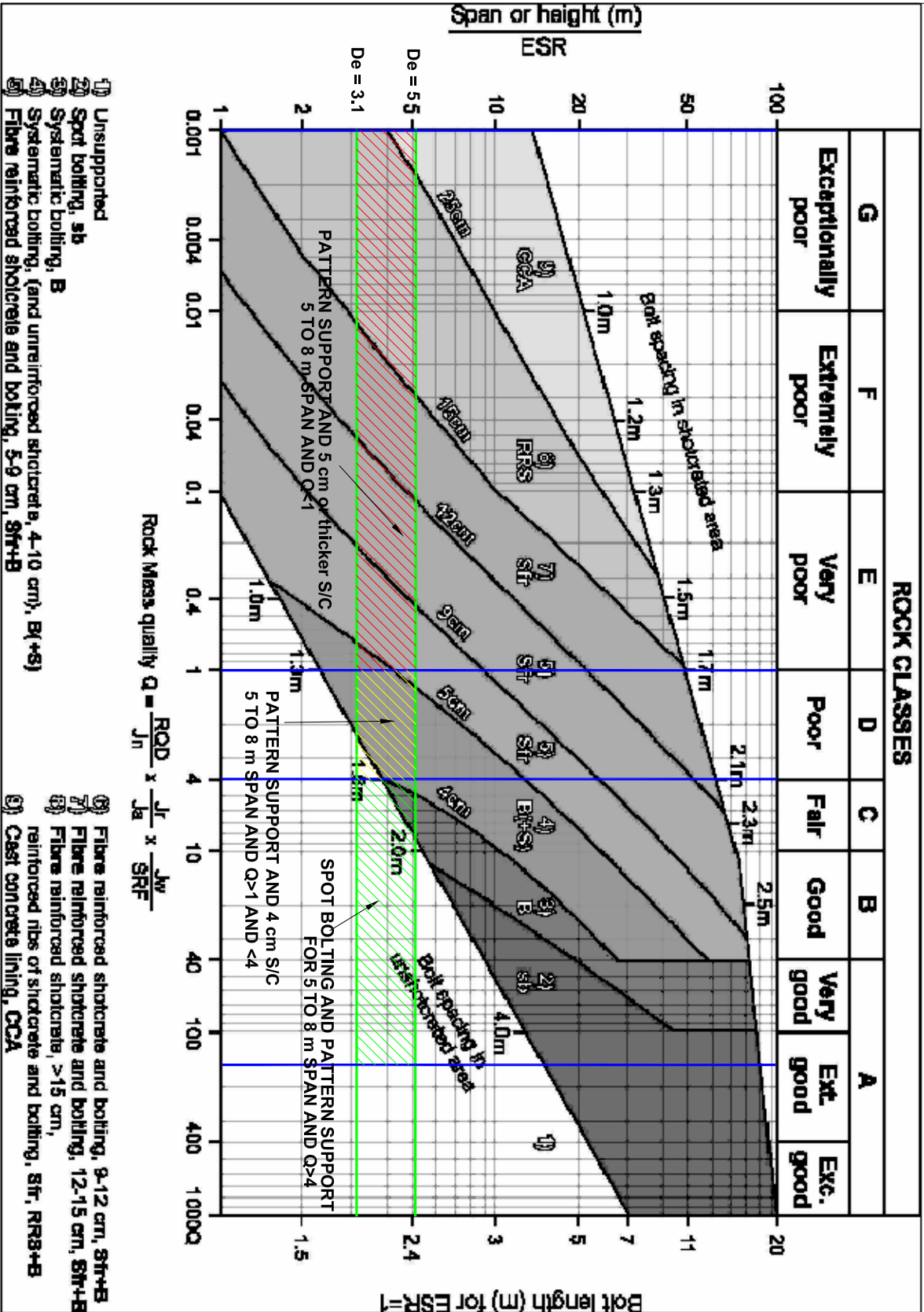


FIGURE 19

PROJECT	FILE No.	REV.	DESIGN	CADD	CHECK	REVIEW
No. 04-1193-020	NTS	0	SCALE	PGP	PGP	KJB
						JAN. 2005
						JAN. 2005
						JAN. 2005

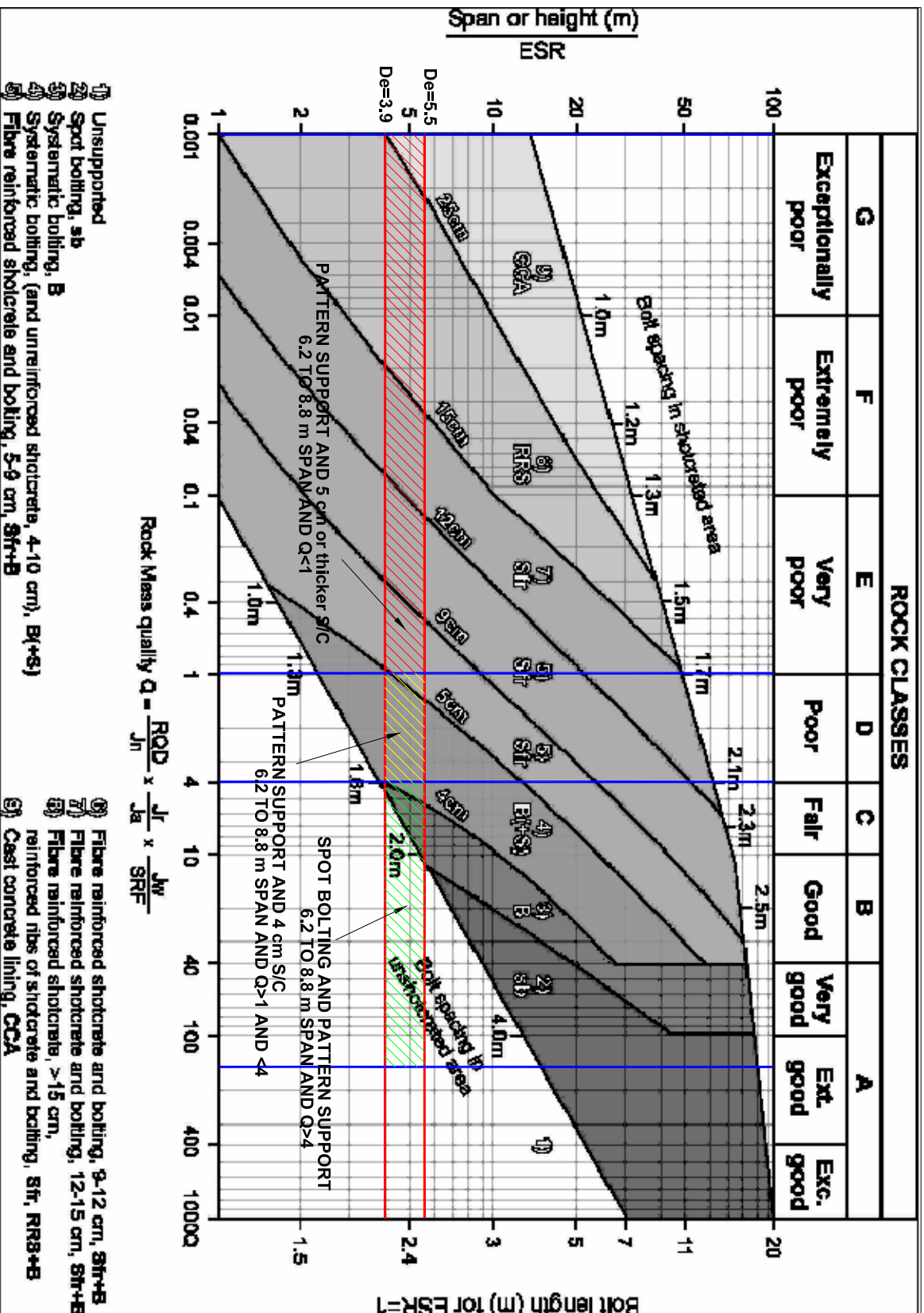
TITLE

**ROCK SUPPORT
DESIGN CHART
FOR SPANS 5 m TO 8 m**

PROJECT

**KENNECOTT EXPLORATION
COMPANY
EAGLE PROJECT**





ROCK SUPPORT DESIGN CHART FOR INTERSECTION SPANS 6.2 m TO 8.8 m

PROJECT No. 04-1193-020	FILE No.	REV. 0	SCALE	NTS
	DESIGN	CADD	PGP	JAN. 2005
	CHECK	PGP	JAN. 2005	
	REVIEW	KJB	JAN. 2005	

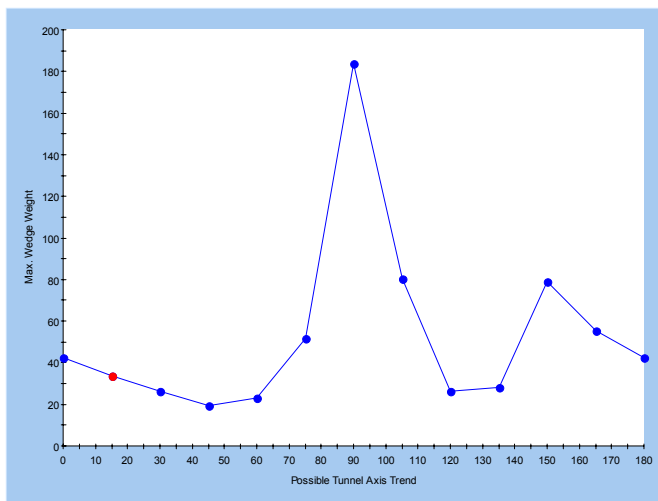
PROJECT

KENNECOTT EXPLORATION
COMPANY
EAGLE PROJECT



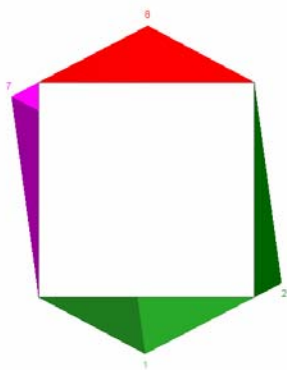
FIGURE 20

Optimization for Tunnel Axis Plunge = 0°



Current Tunnel Axis Trend = 15°

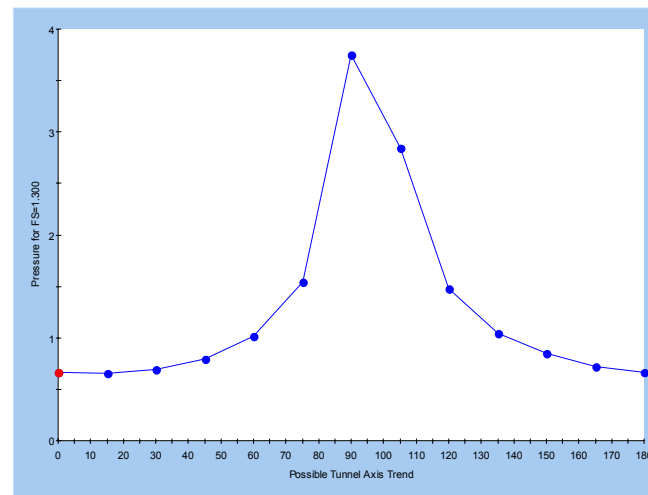
MAXIMUM WEIGHT vs ORIENTATION



Front *

75° Orientation

Optimization for Tunnel Axis Plunge = 0°



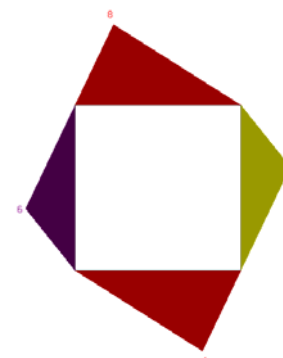
Current Tunnel Axis Trend = 0°

SUPPORT PRESSURE vs ORIENTATION



Front *

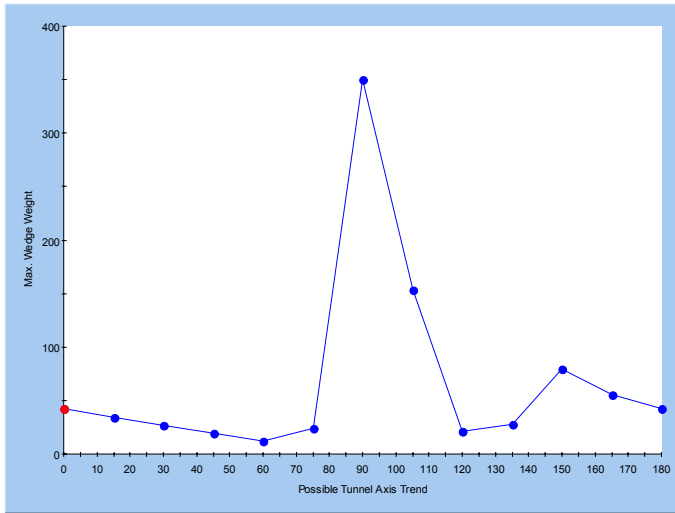
90° Orientation



Front *

105° Orientation

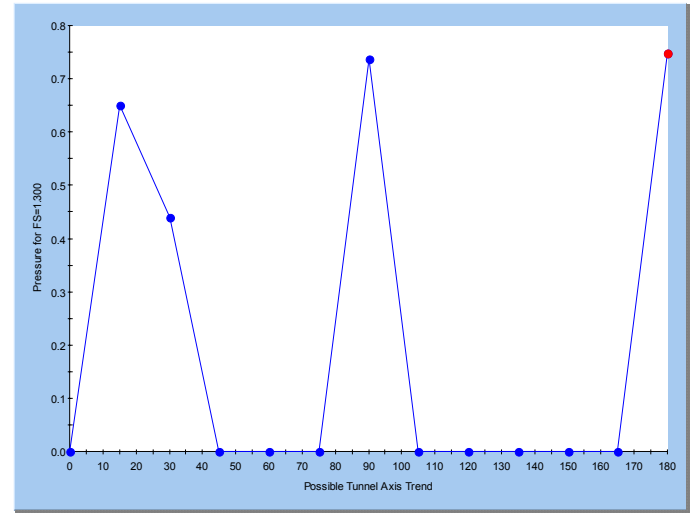
Optimization for Tunnel Axis Plunge = 0°



Current Tunnel Axis Trend = 0°

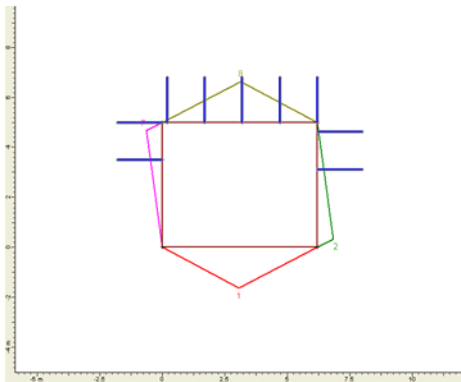
MAXIMUM WEIGHT vs ORIENTATION

Optimization for Tunnel Axis Plunge = 0°

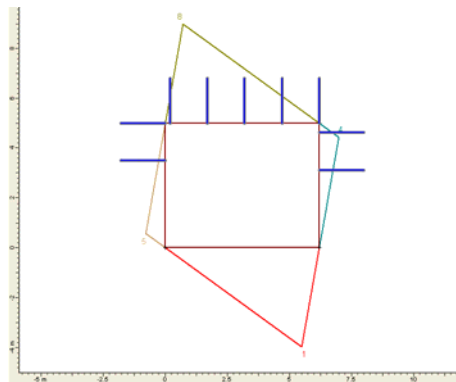


Current Tunnel Axis Trend = 180°

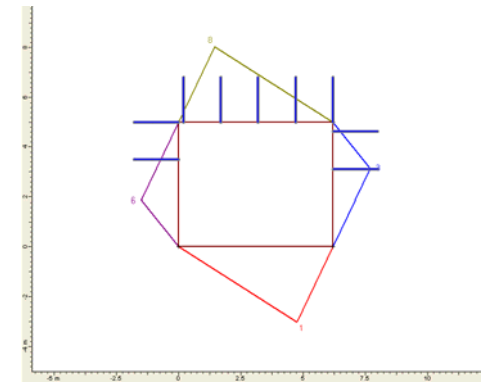
SUPPORT PRESSURE vs ORIENTATION



75° Orientation

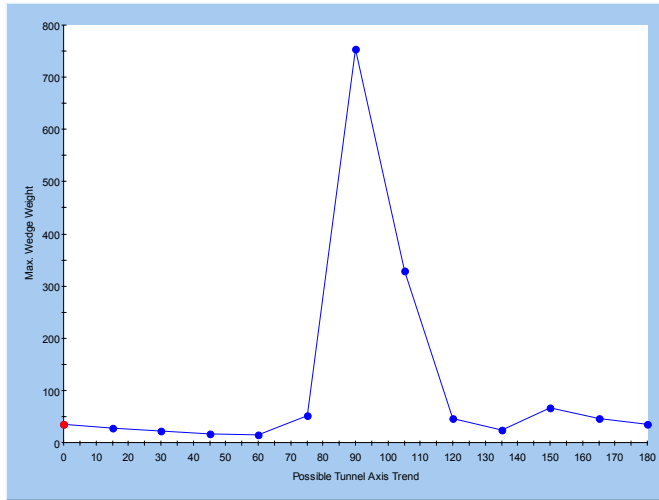


90° Orientation



105° Orientation

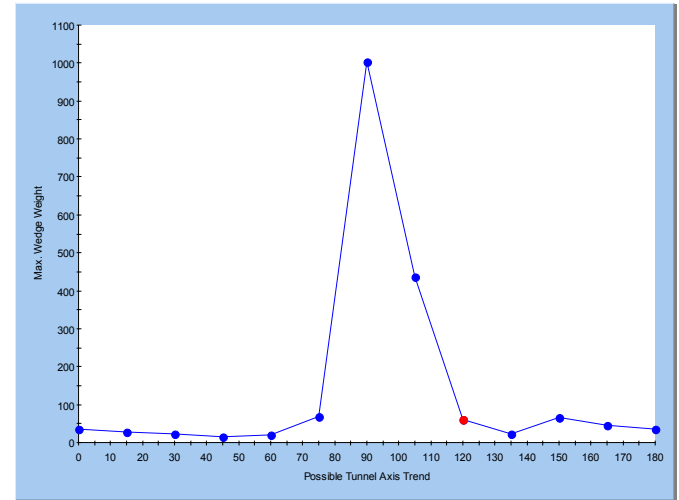
Optimization for Tunnel Axis Plunge = 0°



Current Tunnel Axis Trend = 0°

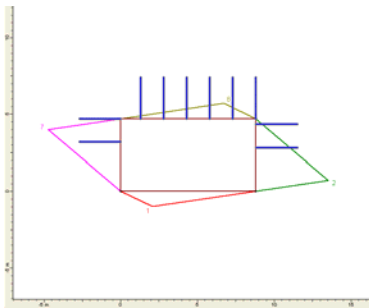
**MAXIMUM WEIGHT vs ORIENTATION
(4.7m BY 8.0m)**

Optimization for Tunnel Axis Plunge = 0°

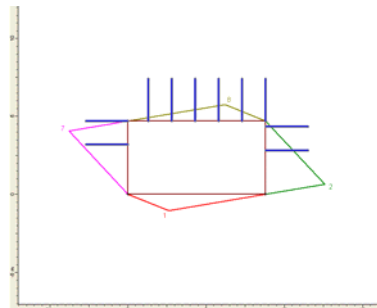


Current Tunnel Axis Trend = 120°

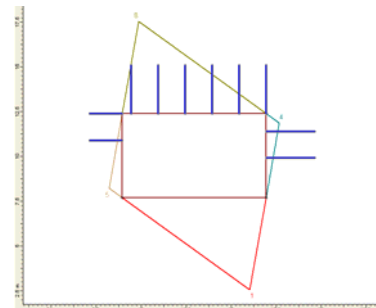
**MAXIMUM WEIGHT vs ORIENTATION
(4.7m BY 8.8m)**



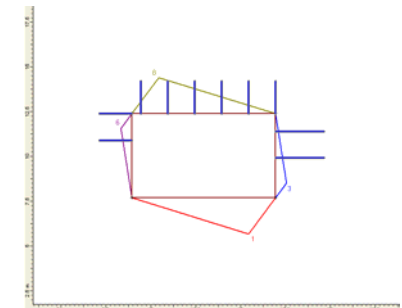
0° Orientation



30° Orientation



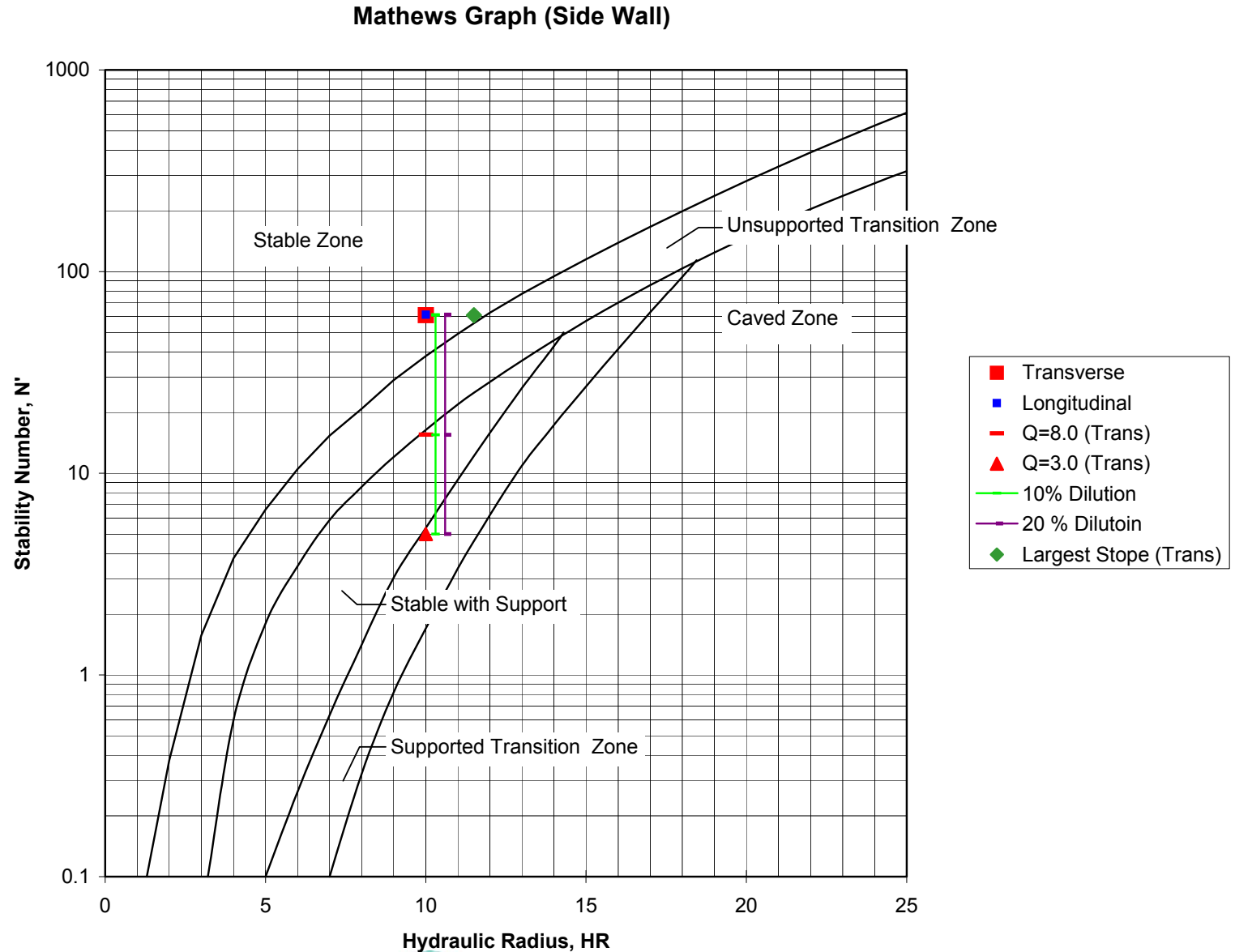
90° Orientation



120° Orientation

TRANSVERSE AND LONGITUDINAL LONGHOLE STOPE FACE STABILITY PLOT

FIGURE 24



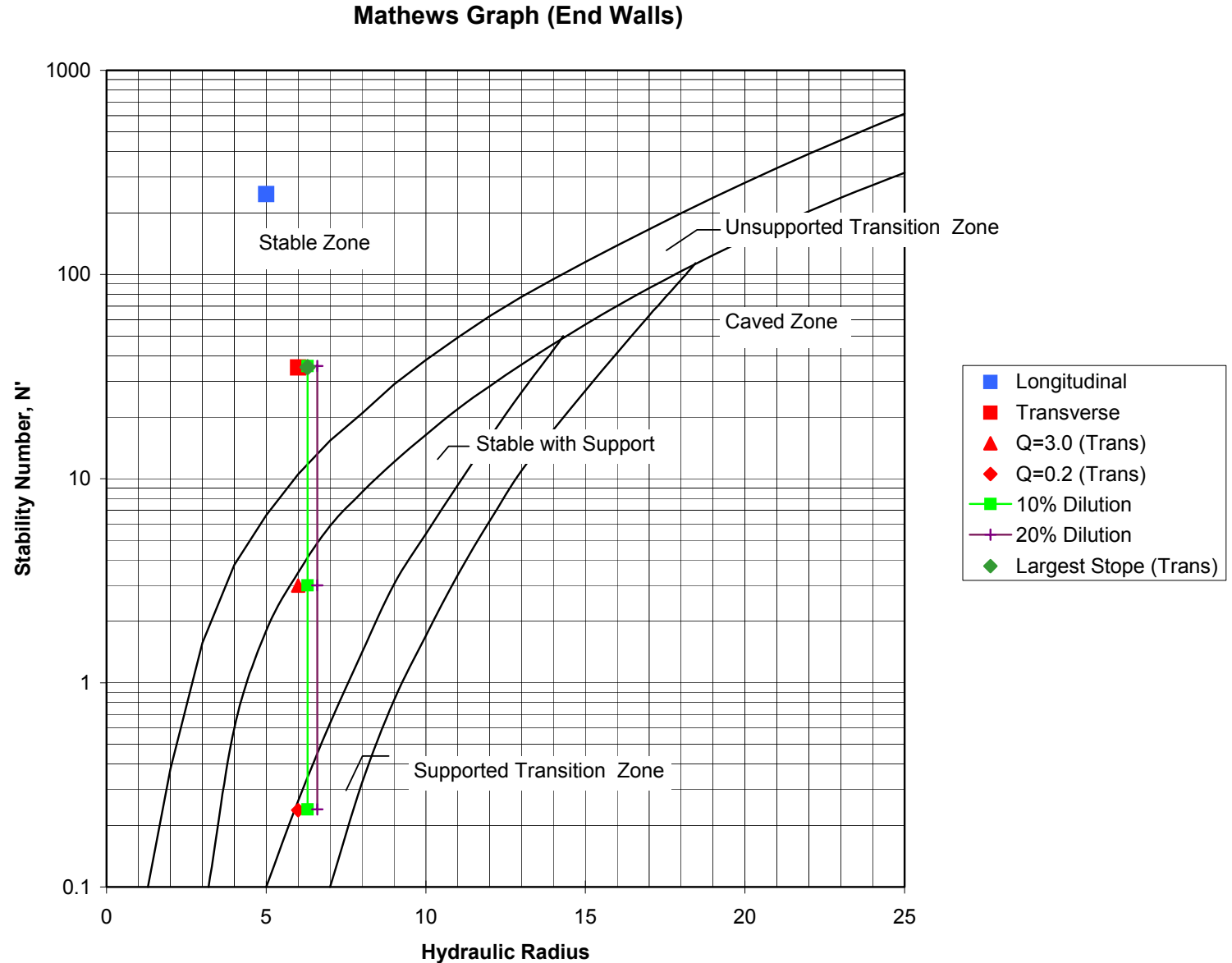
DATE **APRIL 2005**
PROJECT 04-1193-020



DRAWN..PGP..
CHKD...KJB...

TRANSVERSE AND LONGITUDINAL LONGHOLE END WALLS STABILITY PLOT

FIGURE 25



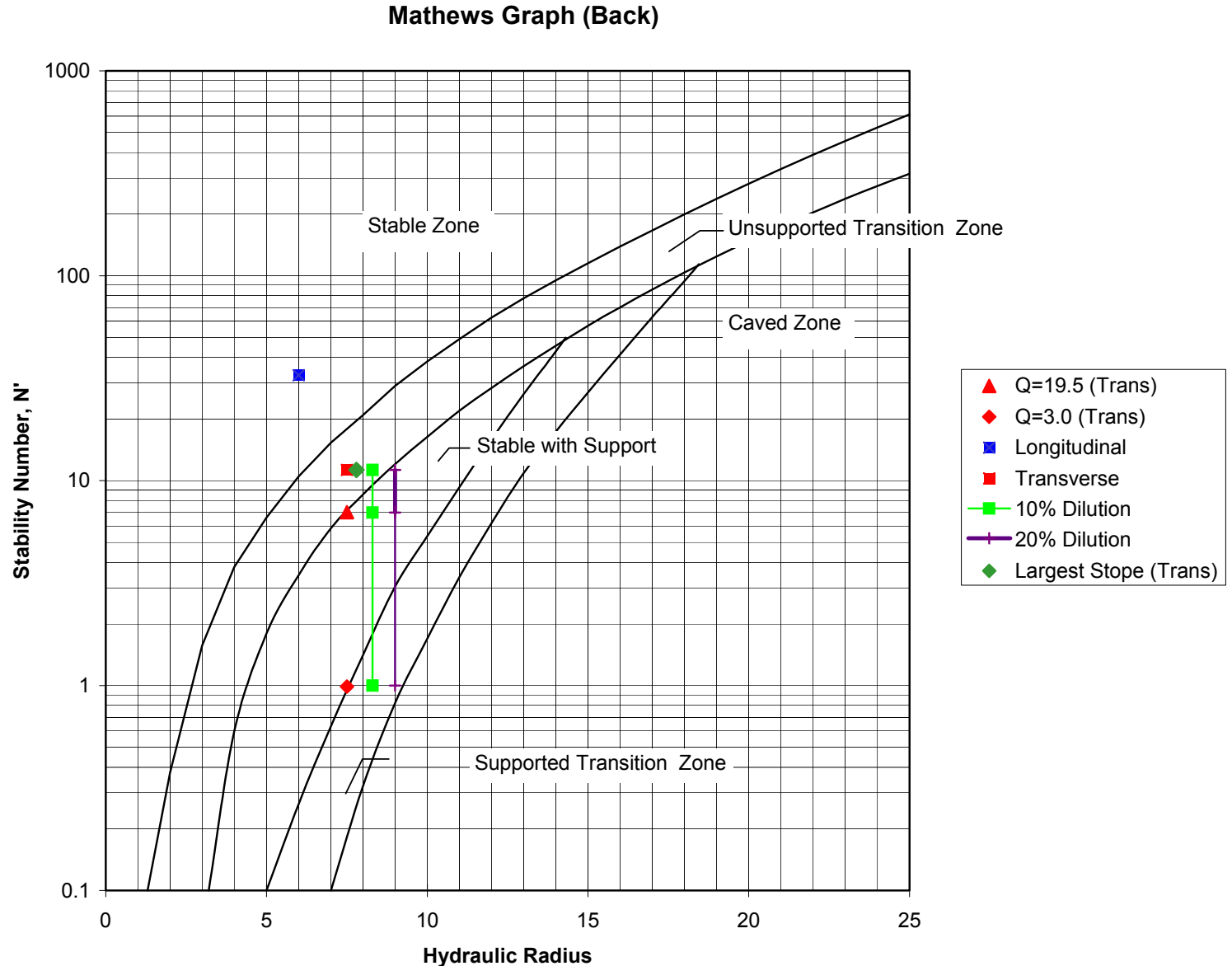
DATE **APRIL 2005**
PROJECT 04-1193-020



DRAWN.PGP..
CHKD...KJB...

TRANSVERSE AND LONGITUDINAL LONGHOLE STOPE BACK STABILITY PLOT

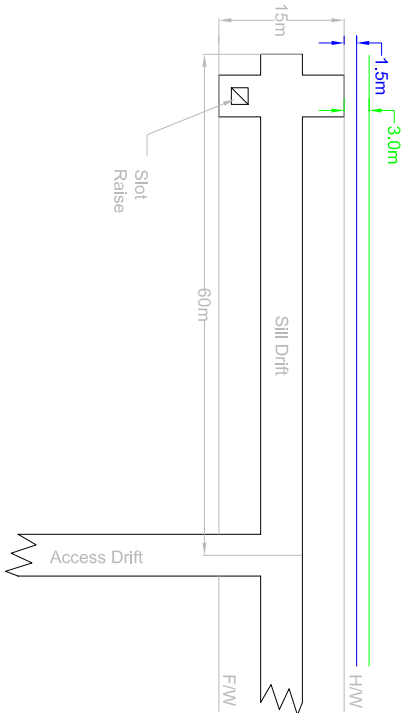
FIGURE 26



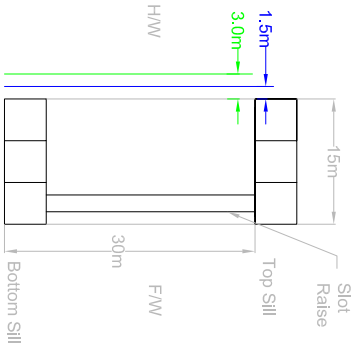
DATE **APRIL 2005**
PROJECT 04-1193-020



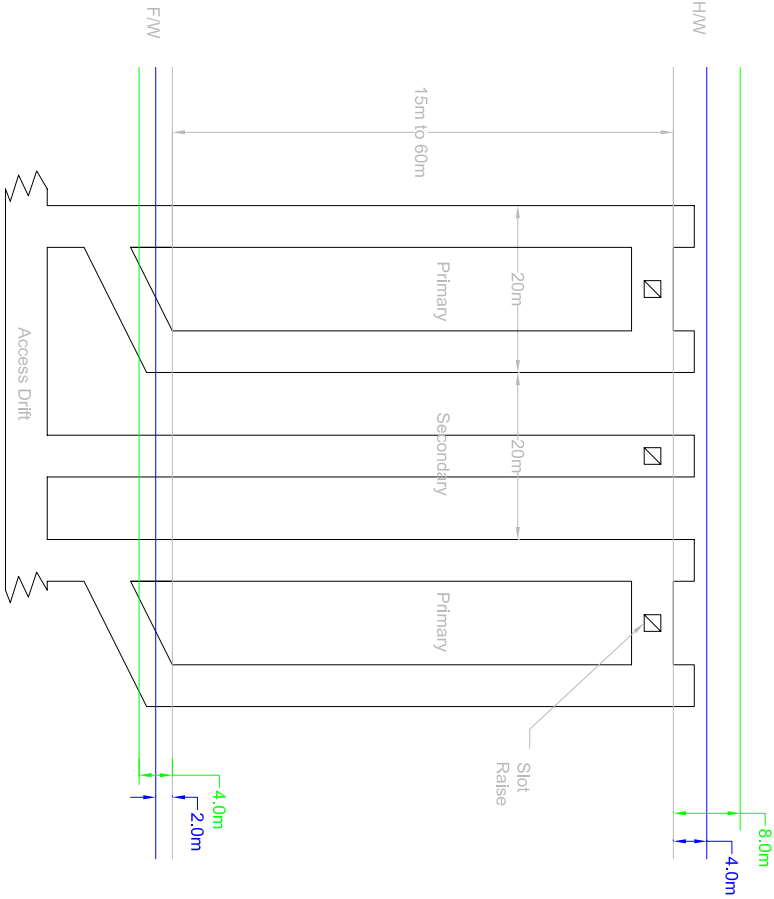
DRAWN.PGP..
CHKD...KJB...



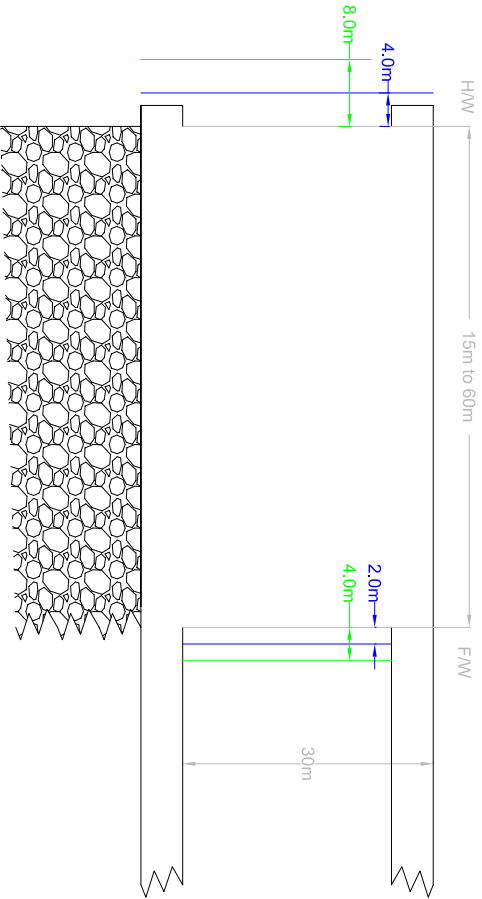
Longitudinal Mining, Plan View



Longitudinal Mining, Vertical Section (facing east)



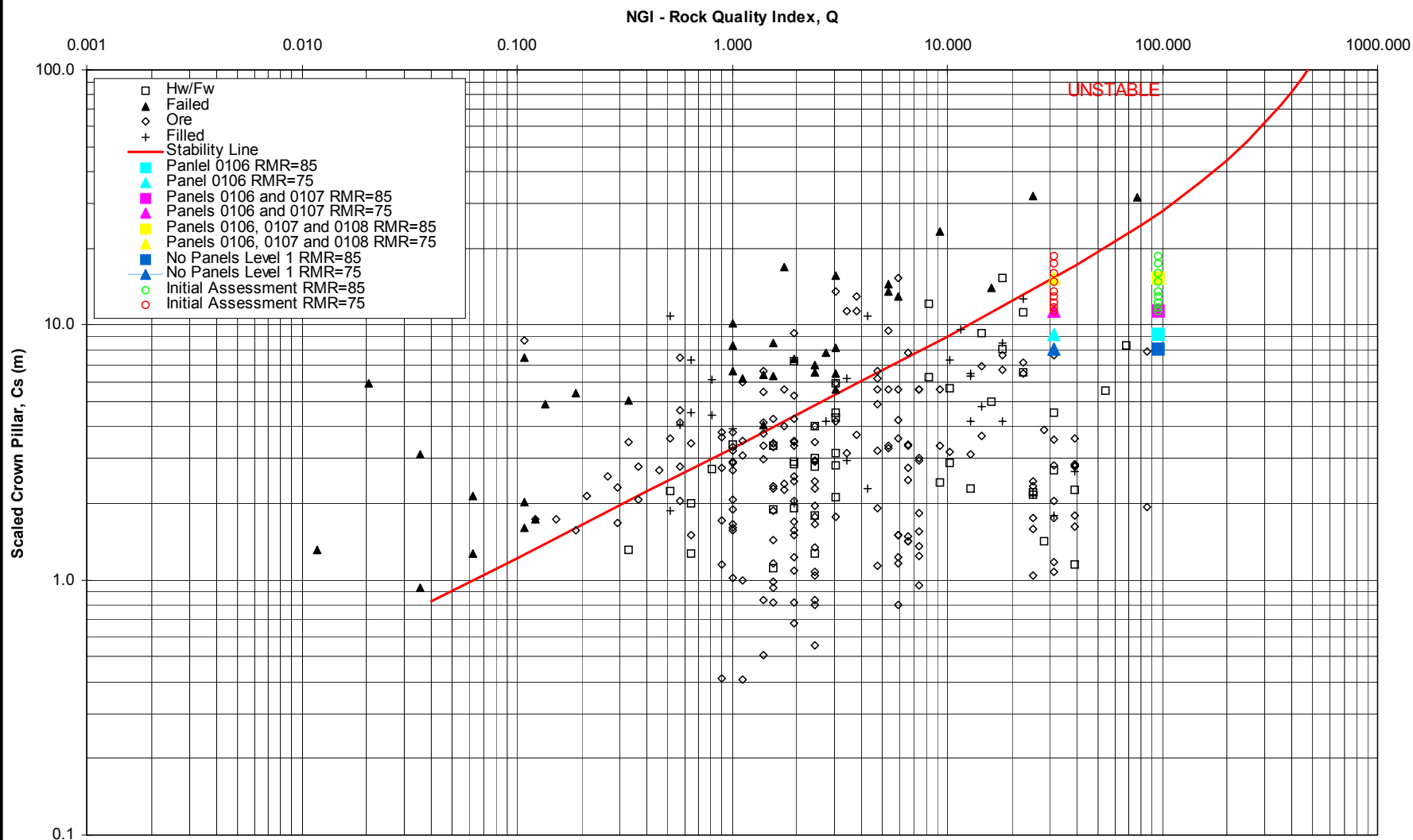
Transverse Mining, Plan View

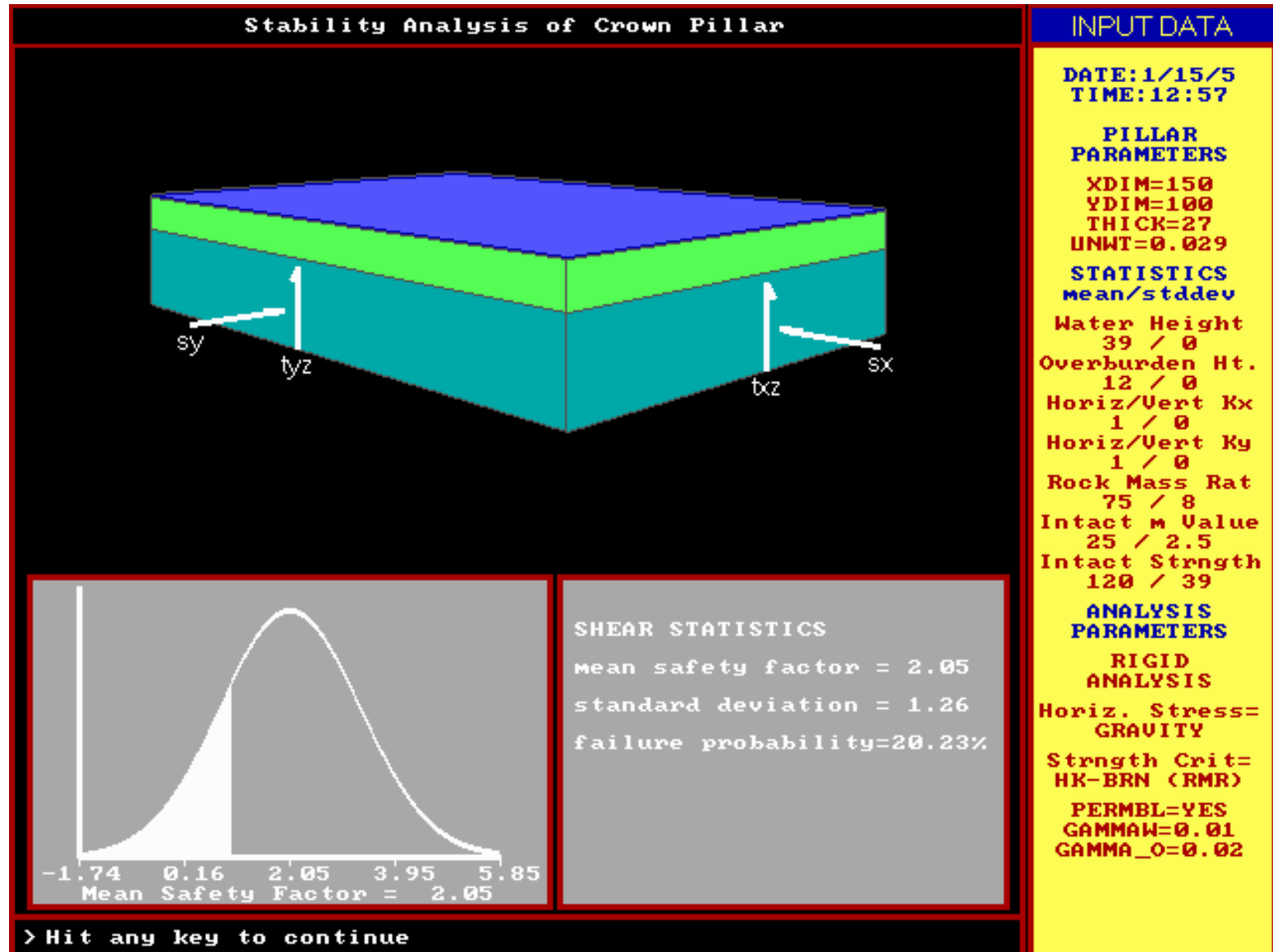


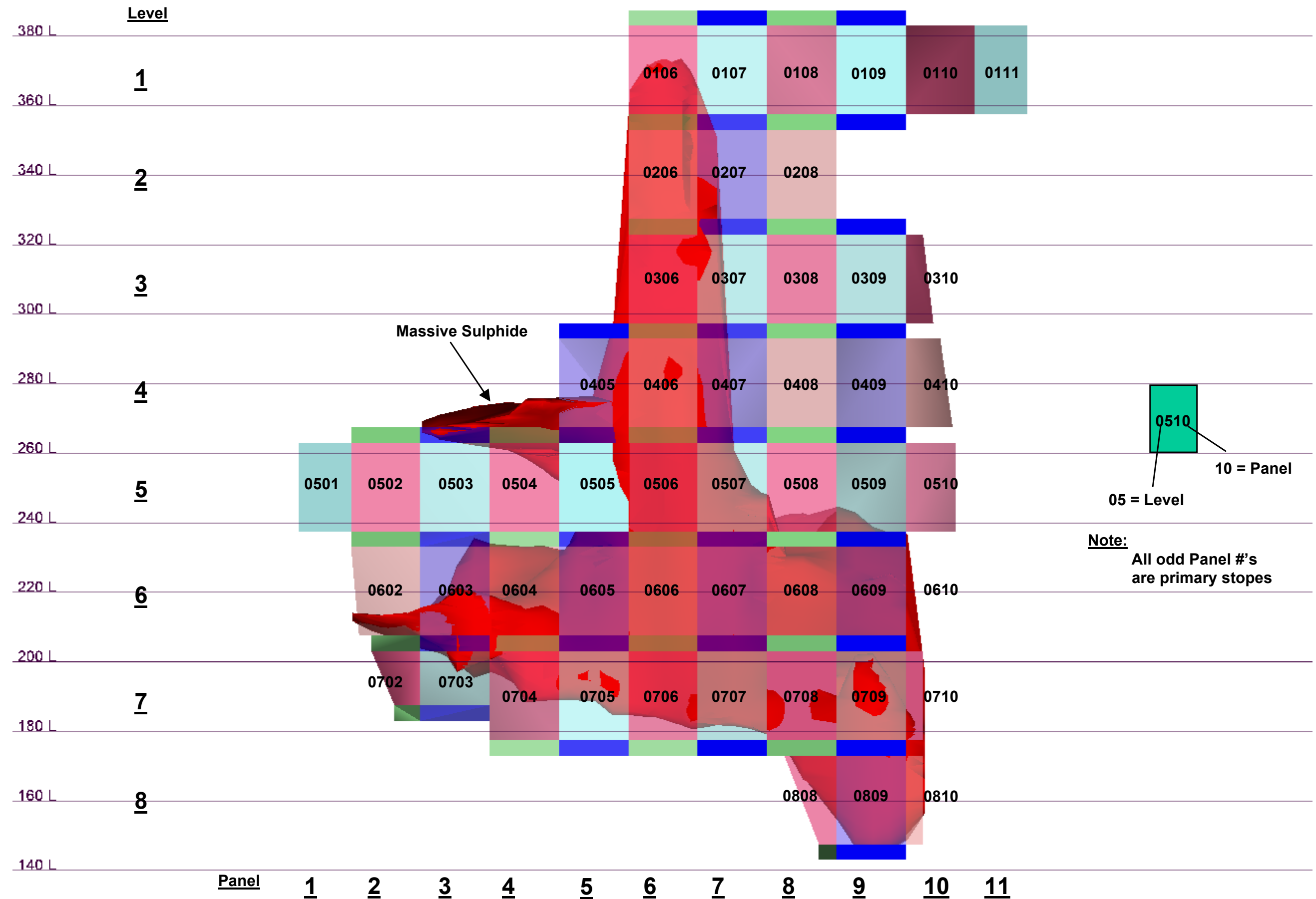
Transverse Mining, Vertical Section (facing east)

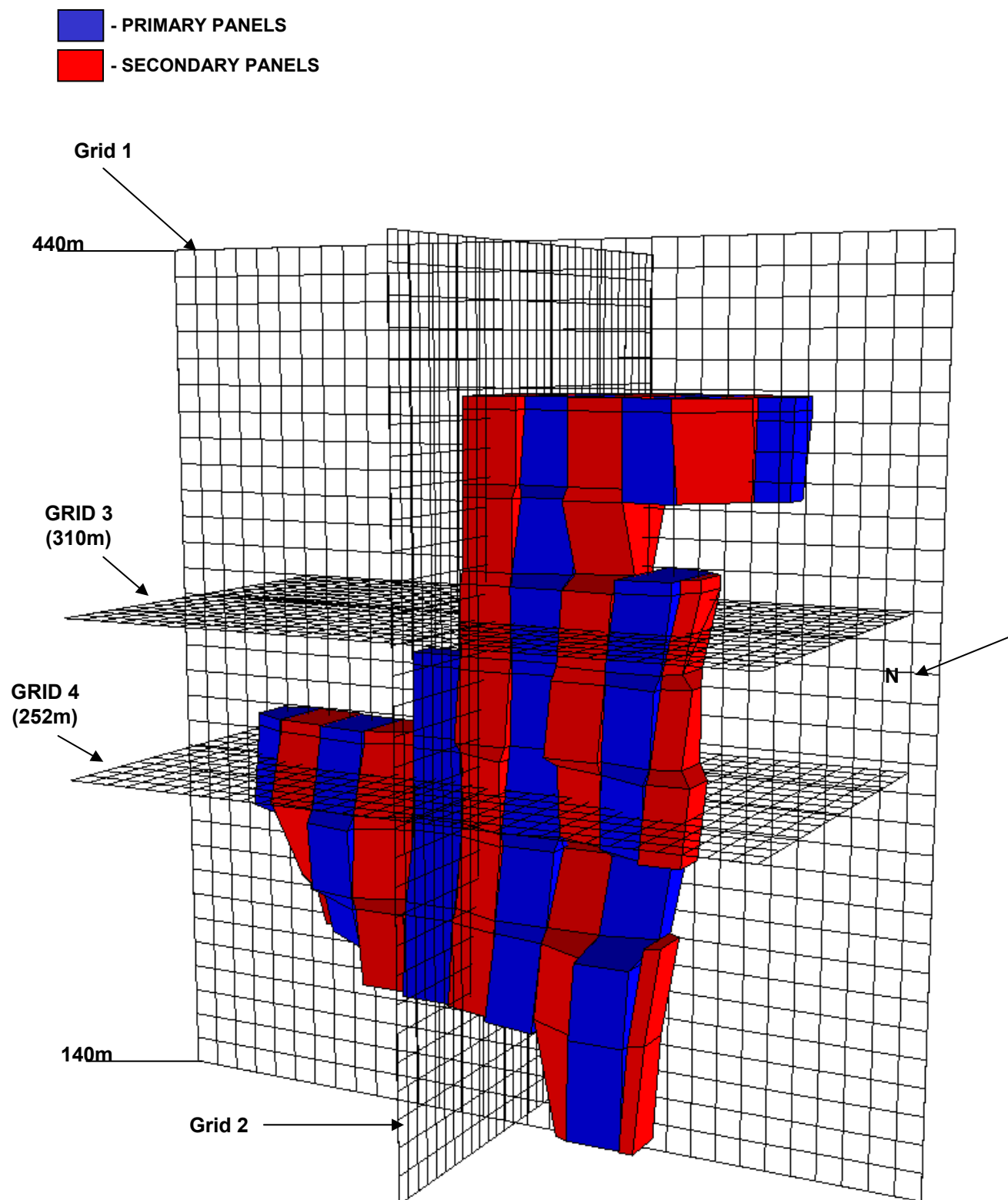
LEGEND

- 10% Dilution
- 20% Dilution



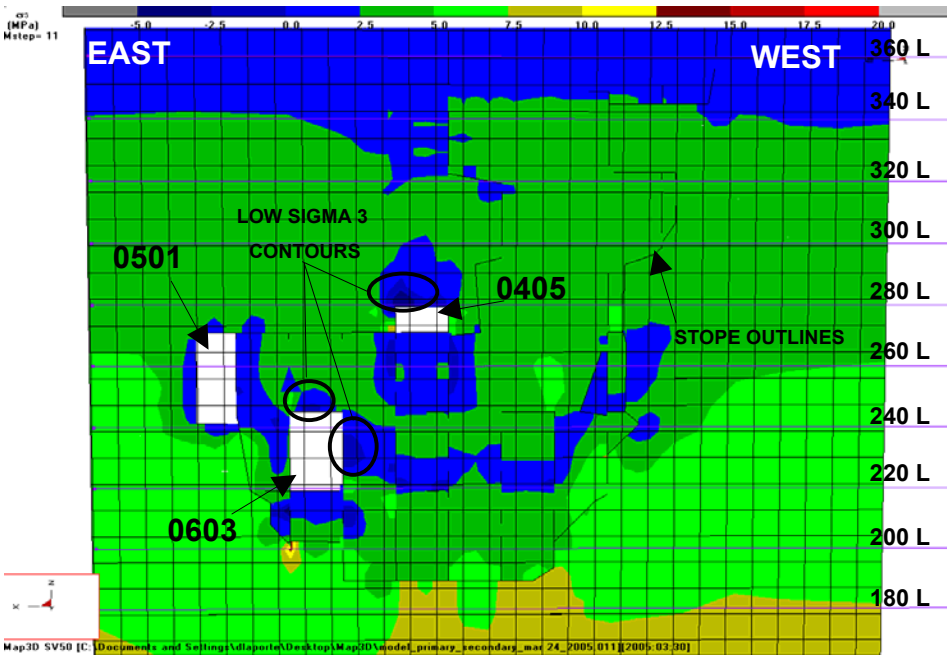




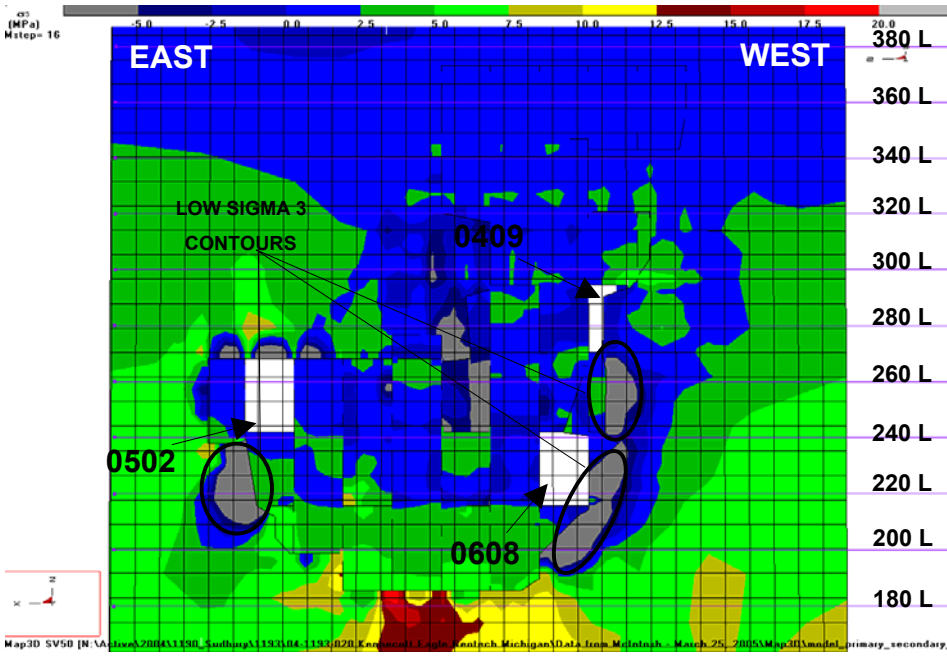


\\004\1190_Sudbury\1193\04-1193-020 Kennecott Eagle Geotech Michigan\Data from McIntosh - March 25, 2005\Map31

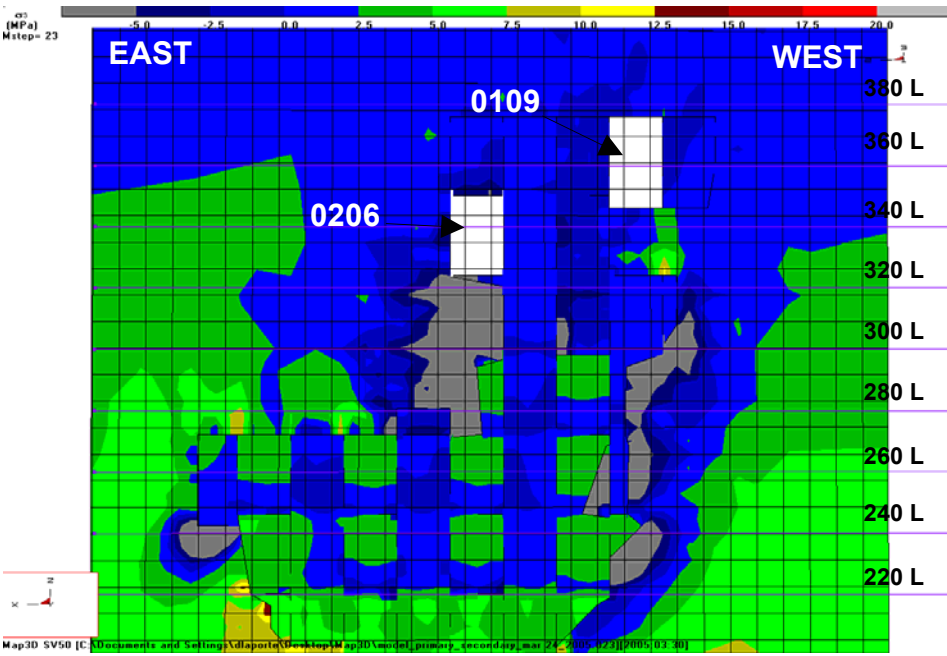
GRID 1 – STEP 11



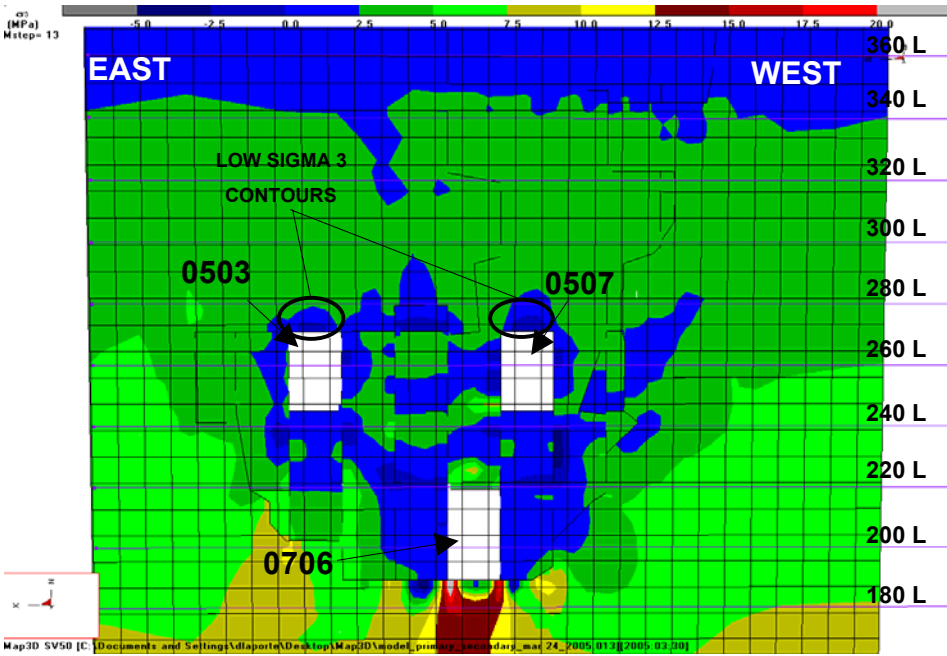
GRID 1 – STEP 16



GRID 1 – STEP 23

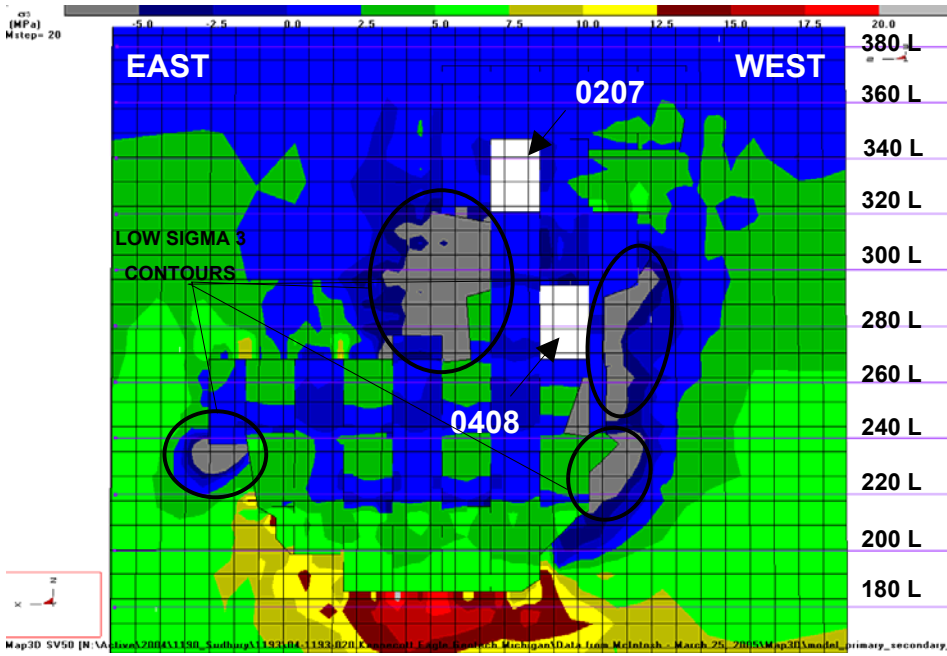


GRID 1 – STEP 13

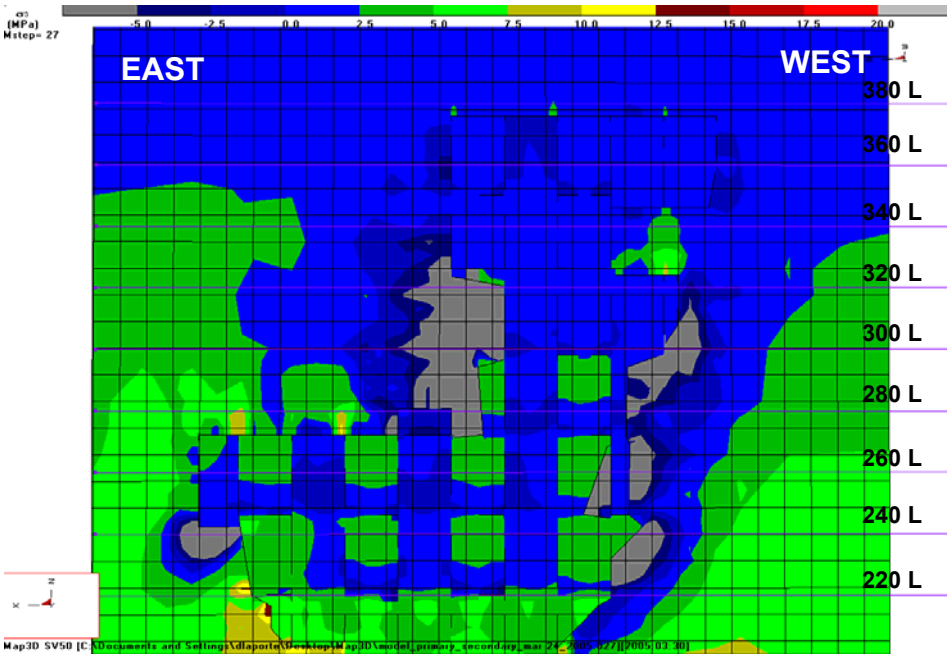


Surface = 440m

GRID 1 – STEP 20

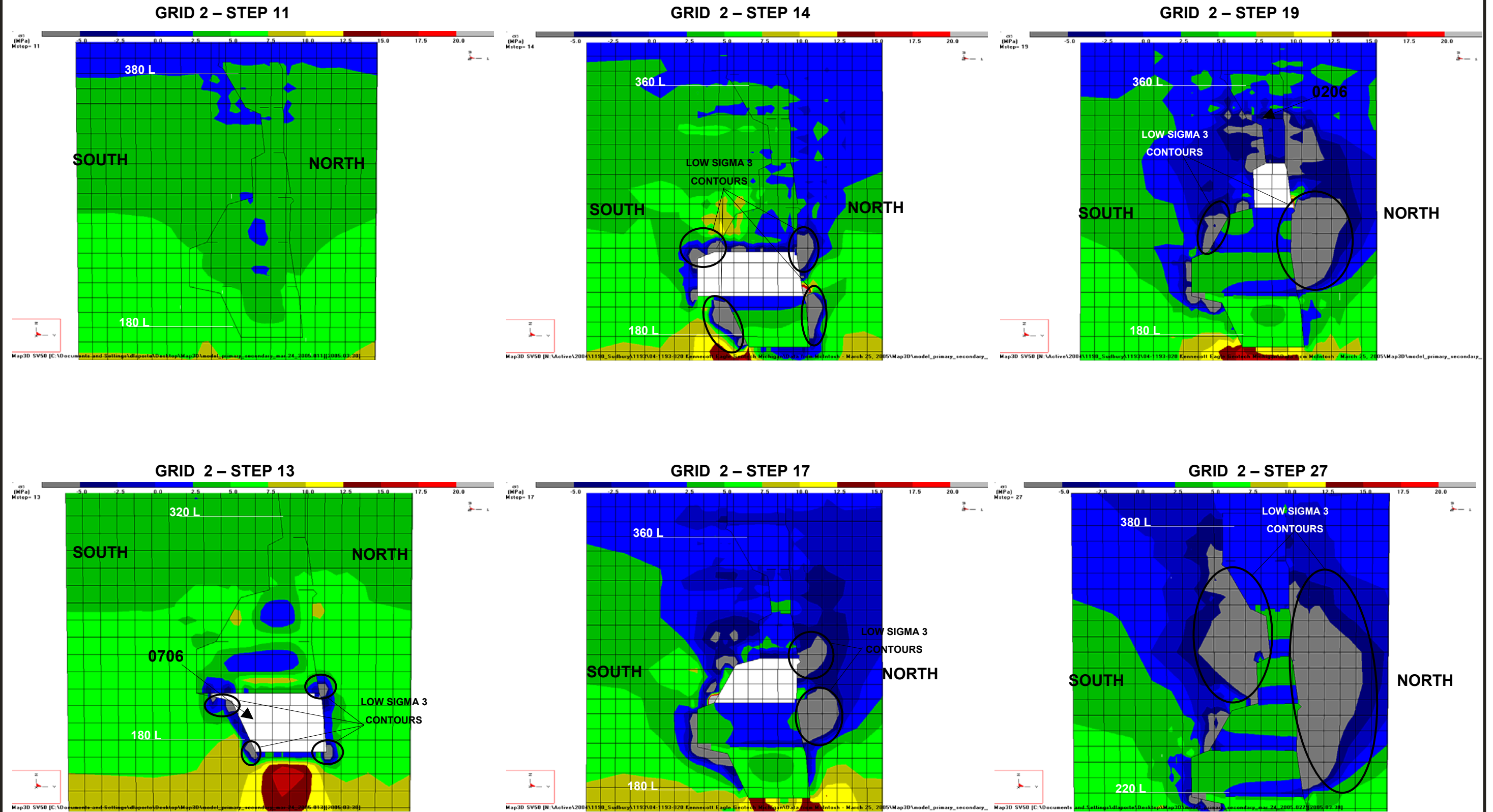


GRID 1 – STEP 27

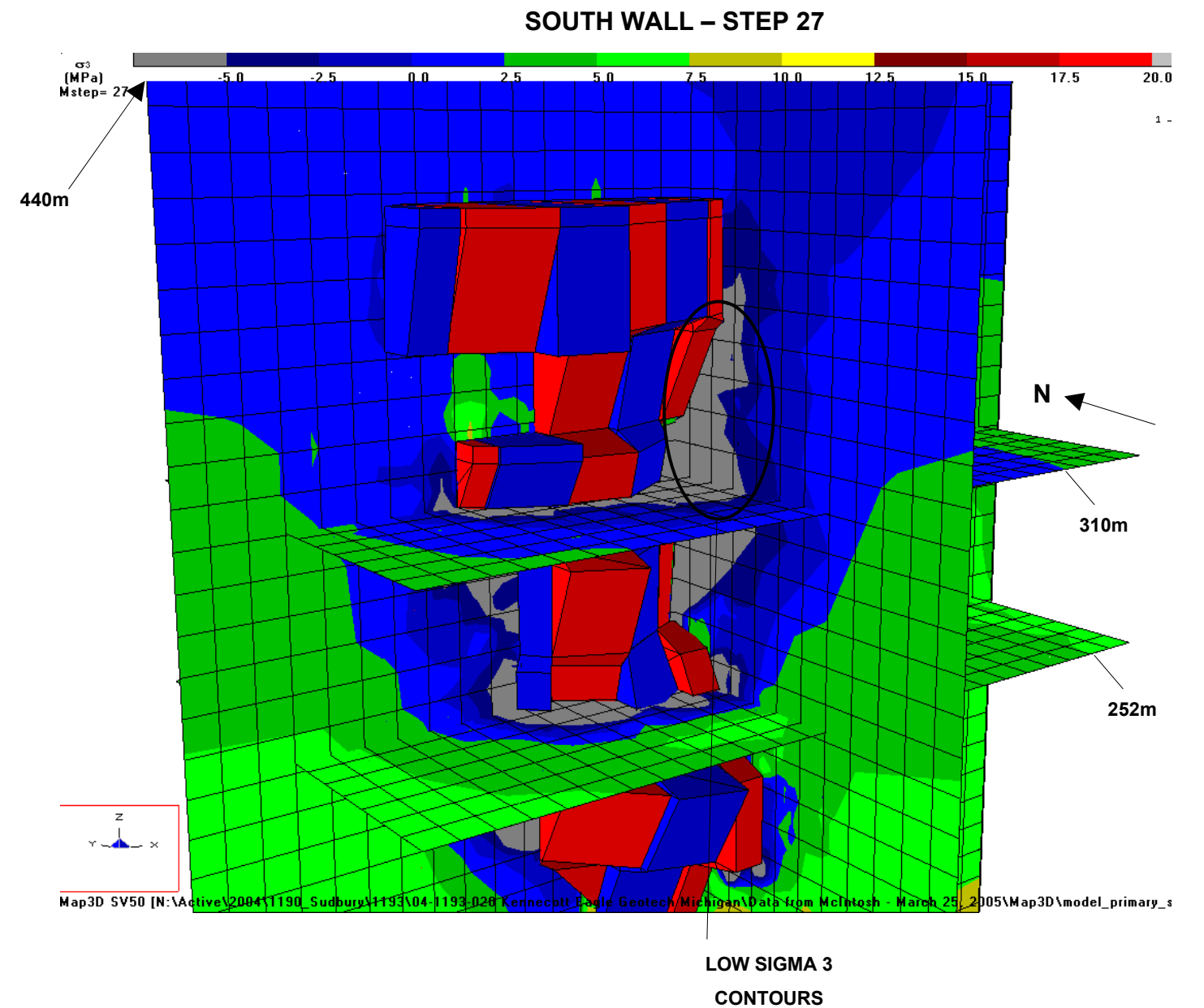
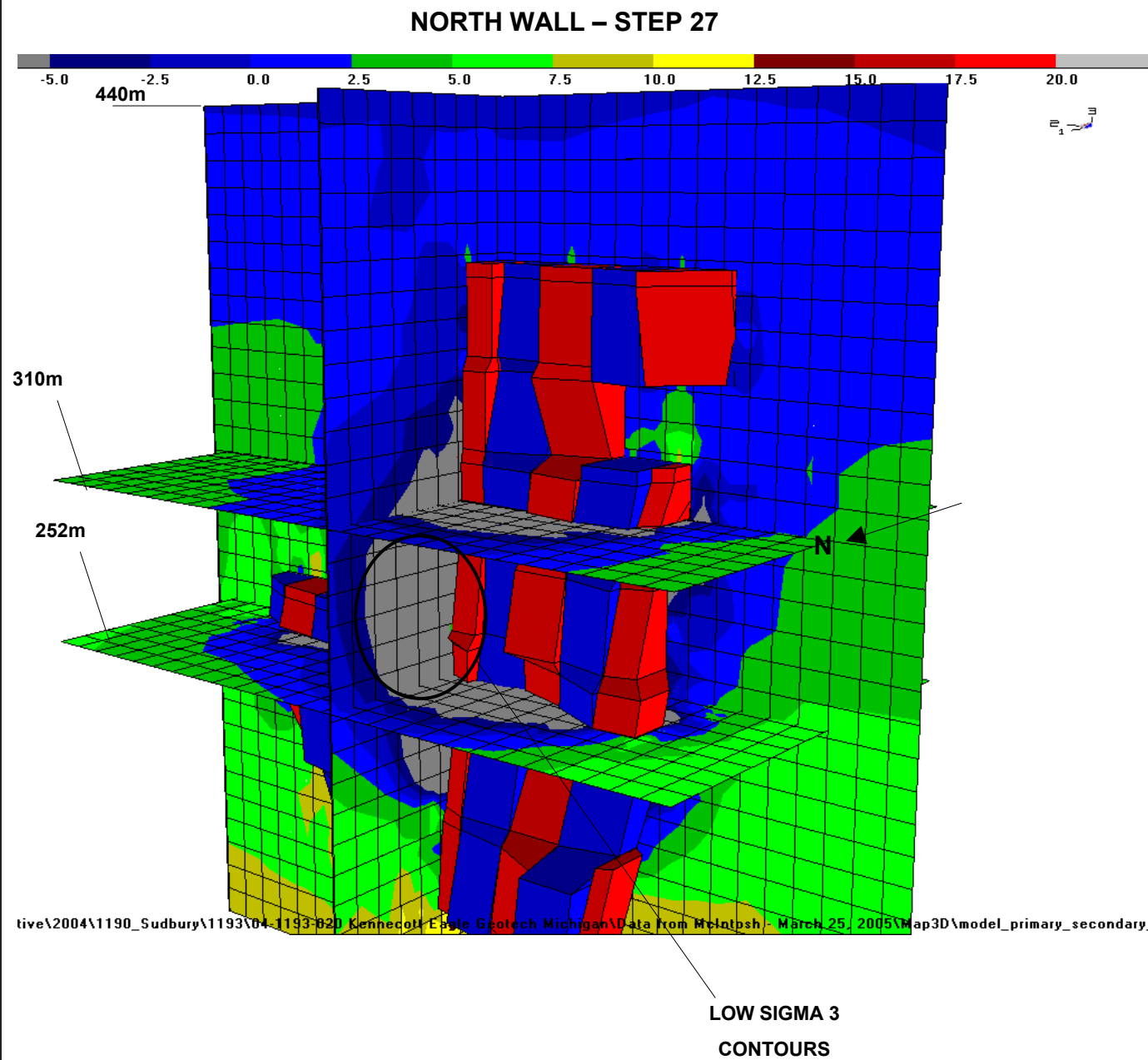


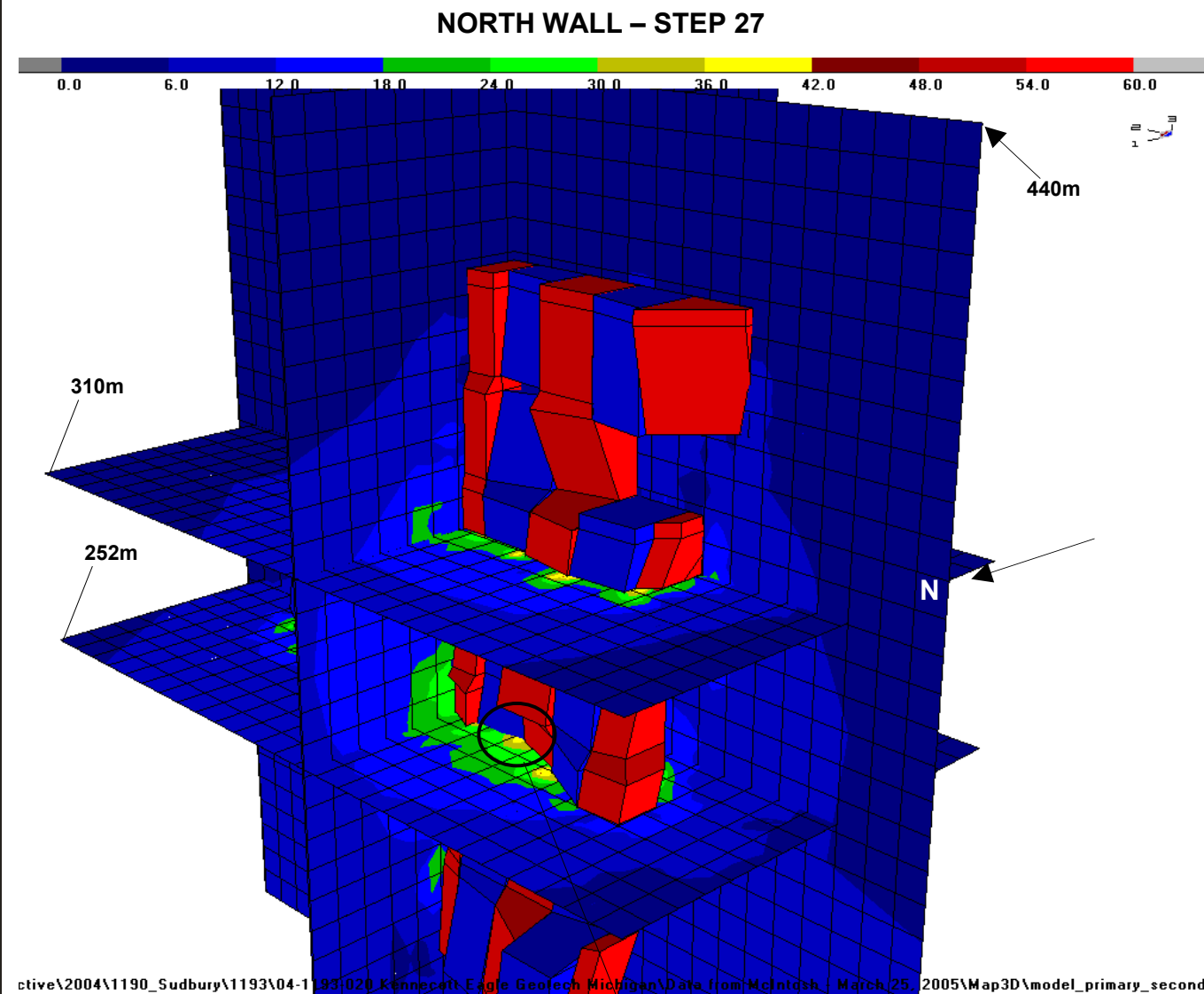
EAGLE PROJECT – MAP3D – SIGMA 3 RESULTS
GRID 1 – STEPS 11, 13, 16, 20, 23 ,27
VIEW - SOUTH



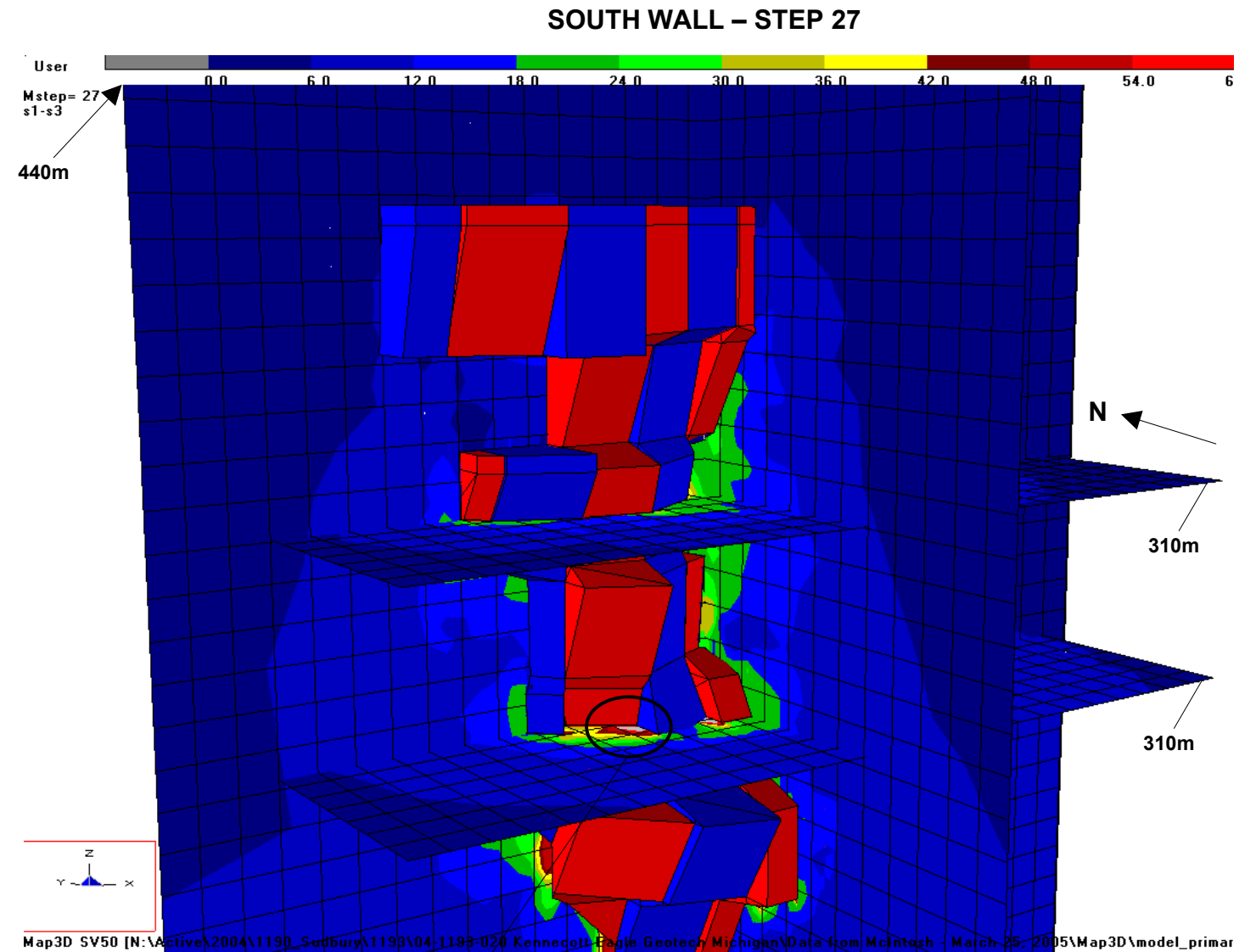


Surface = 440m





DEVIATORIC STRESS CONTOURS
BETWEEN 18-36 MPa

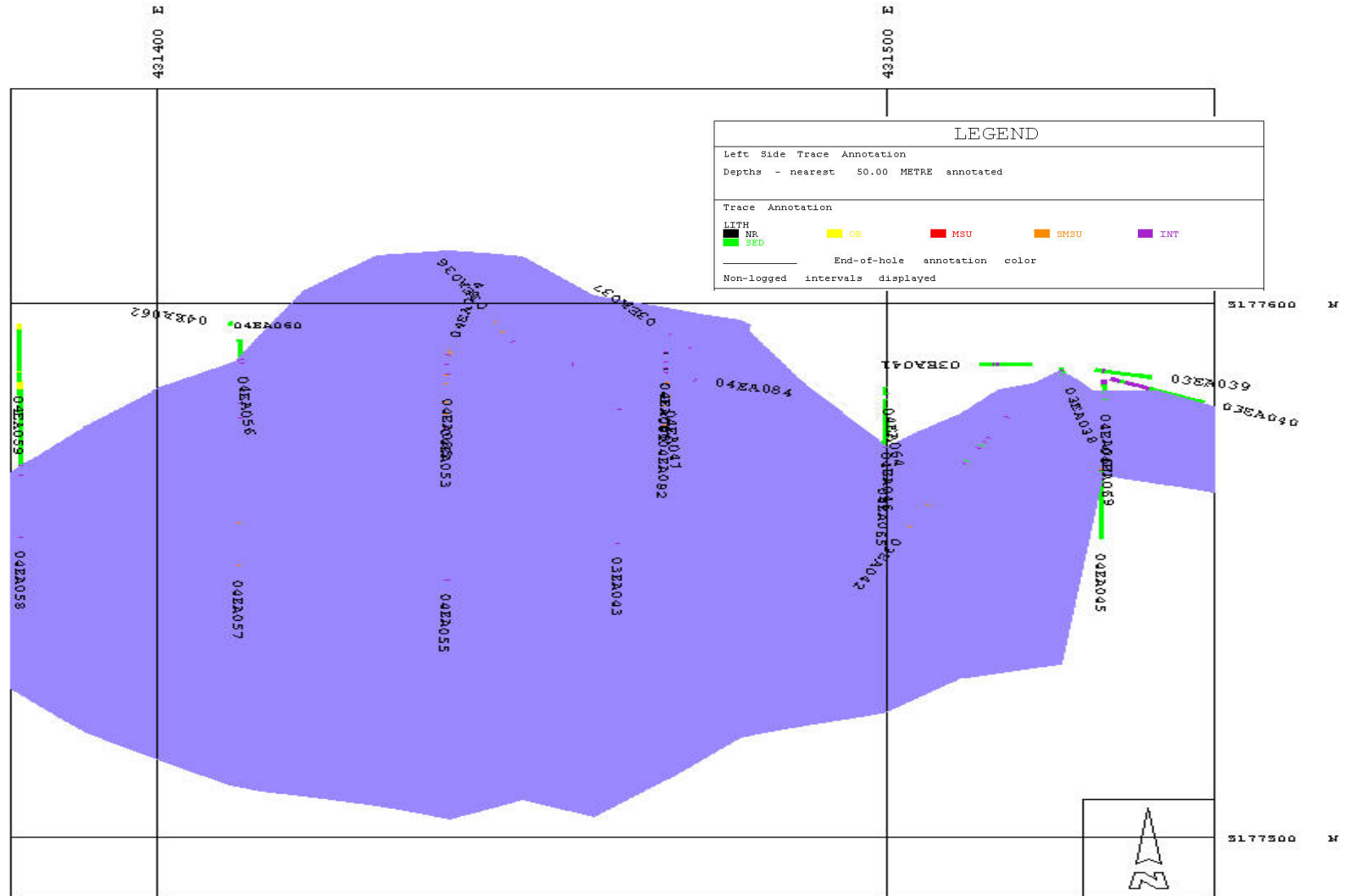


DEVIATORIC STRESS CONTOURS > 42MPa

APPENDIX A
LITHOLOGICAL SECTIONS A1 TO A12

LITHOLOGY – LEVEL 405

FIGURE A1



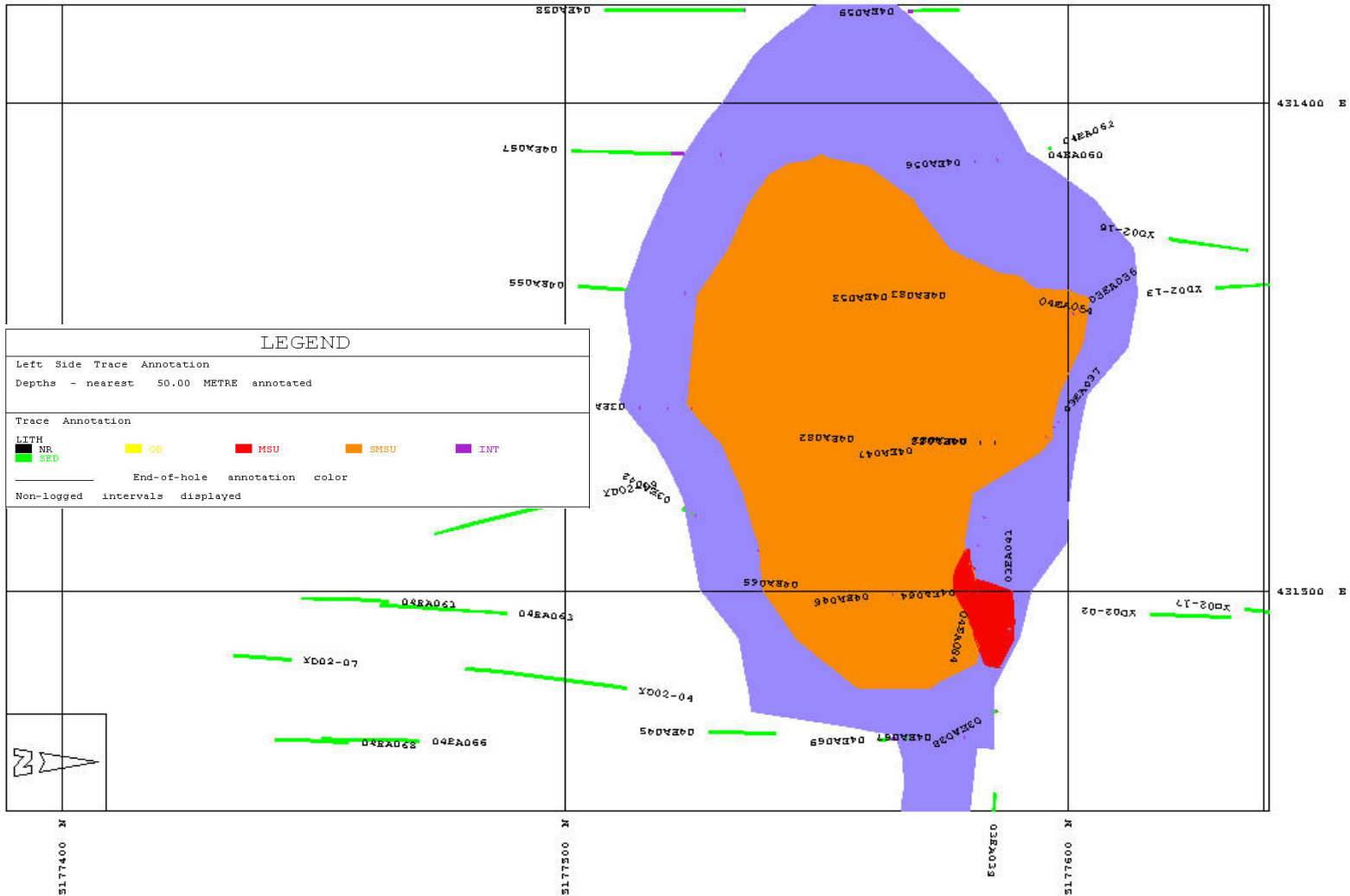
DATE JAN 2005
 PROJECT 04-1193-020



DRAWN..PGP...
 CHKD...KJB...

LITHOLOGY – LEVEL 355

FIGURE A2



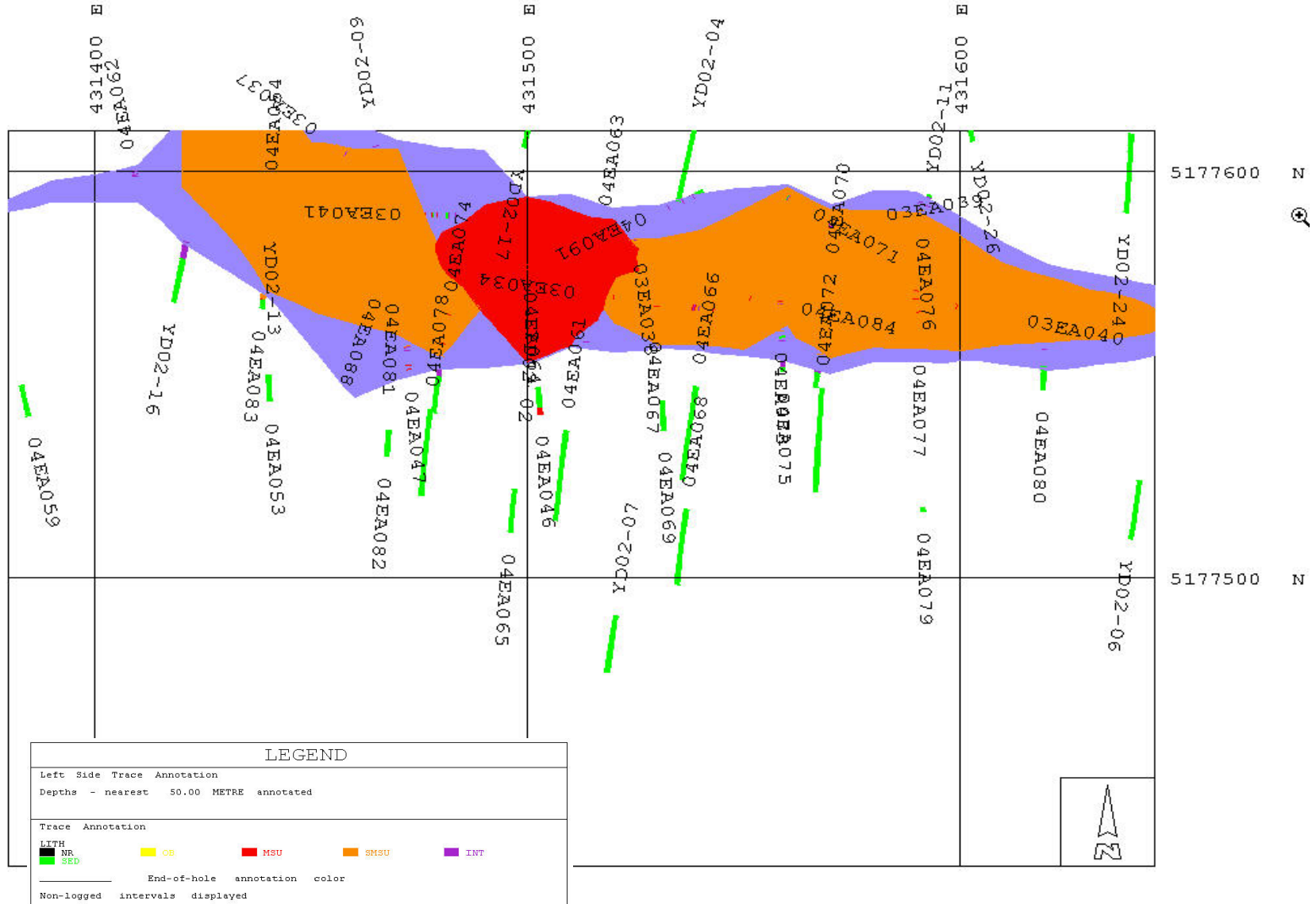
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DRAWN.PGP..
 CHKD...KJB...

LITHOLOGY - LEVEL 250

FIGURE A3



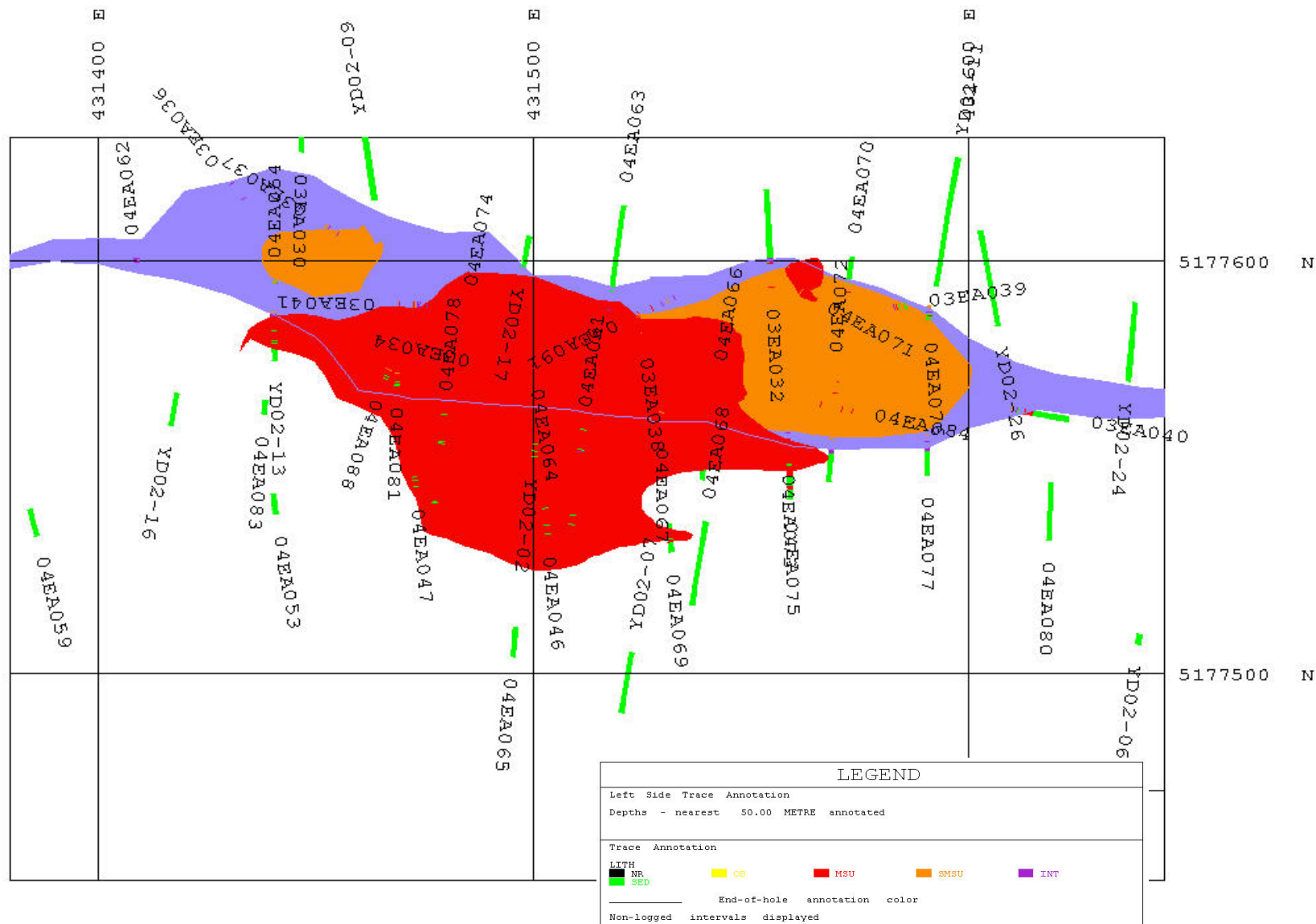
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PROJECT 04-1193-020

30 Orient 

DRAWN..PGP..
CHKD...KJB...

LITHOLOGY - LEVEL 220

FIGURE A4



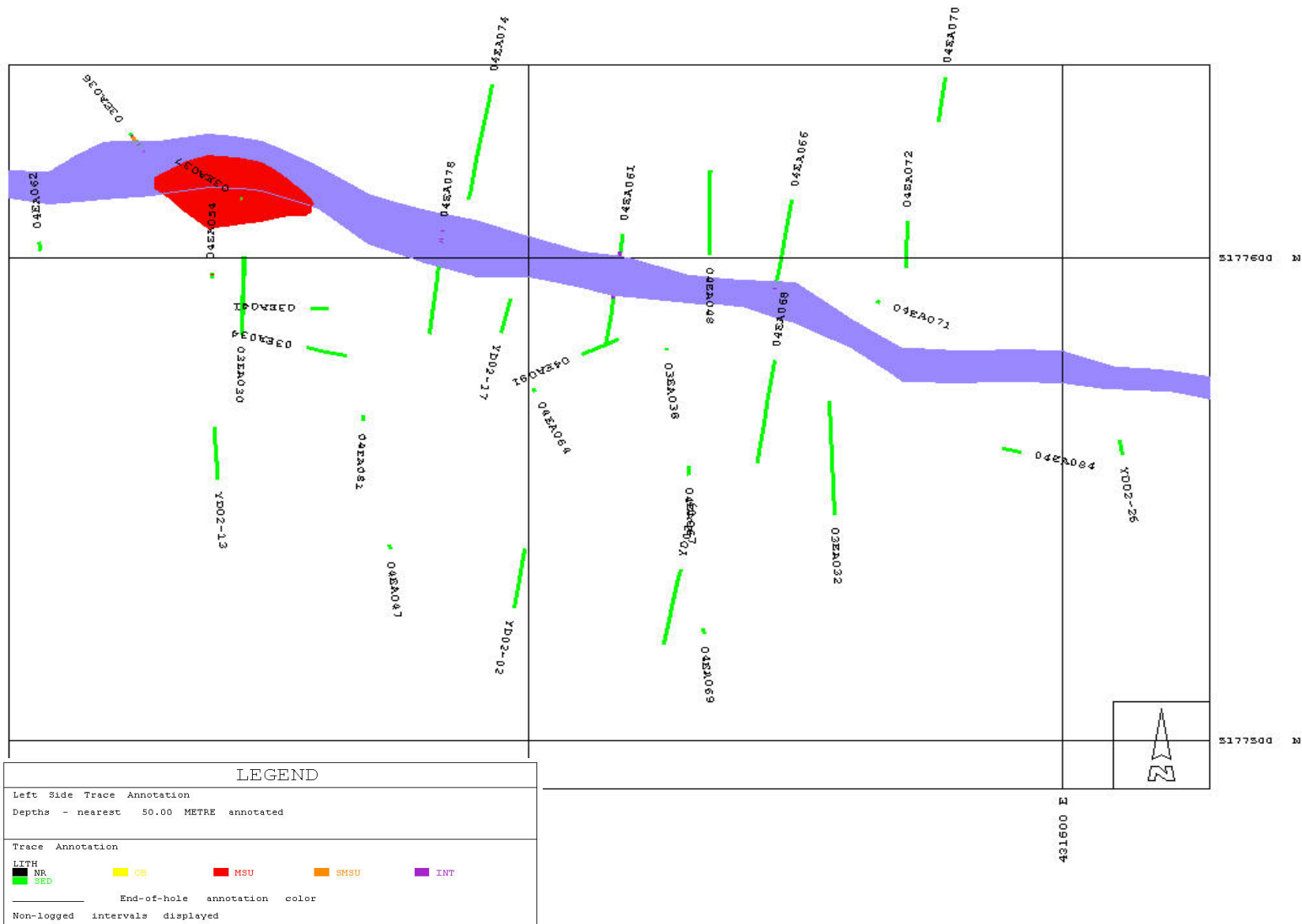
DATE JAN 2005
PROJECT 04-1193-020



DRAWN..PGP..
CHKD...KJB...

LITHOLOGY – LEVEL 160

FIGURE A5



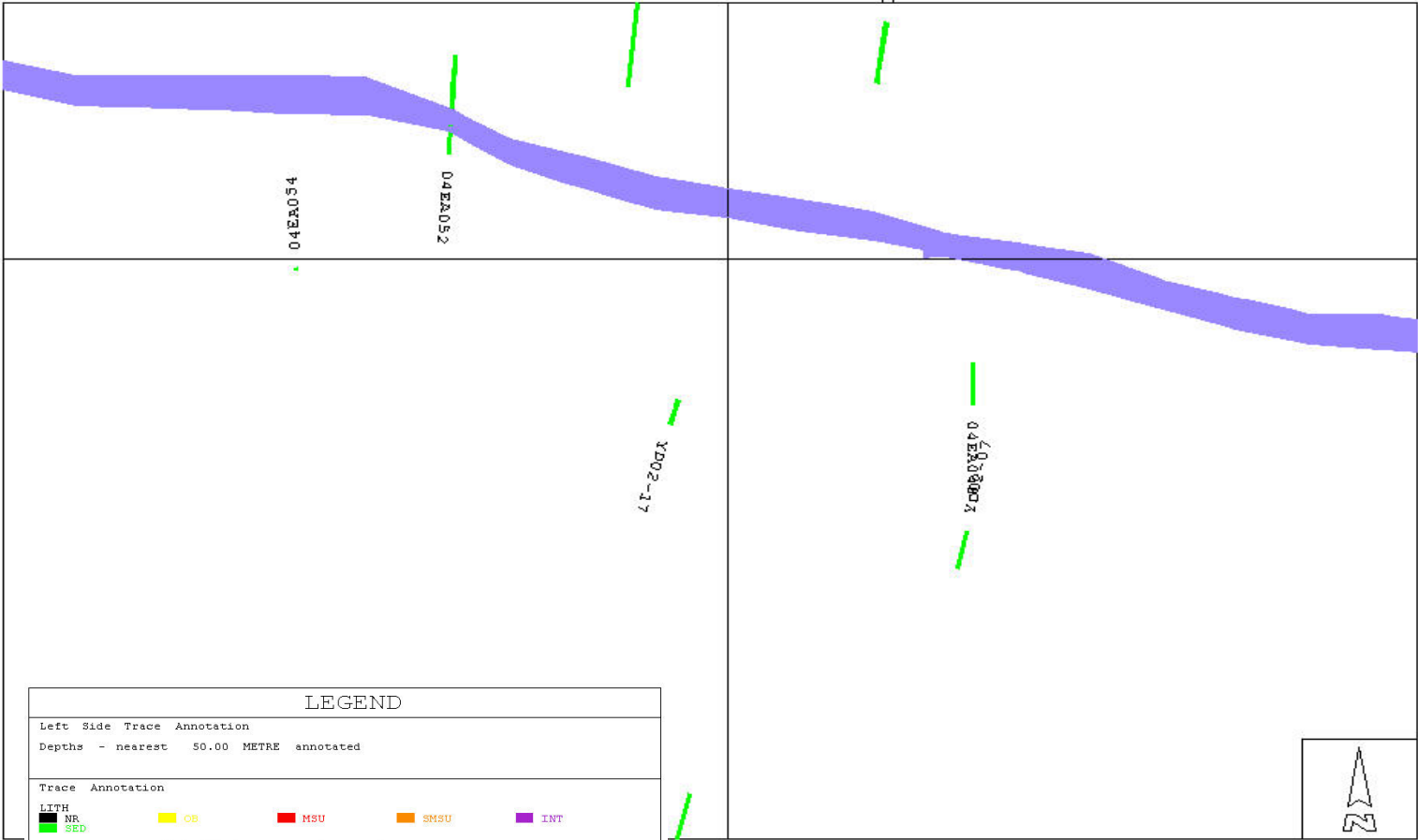
DATE JAN 2005
PROJECT 04-1193-020



DRAWN...PGP...
CHKD...KJB...

LITHOLOGY – LEVEL 100

FIGURE A6



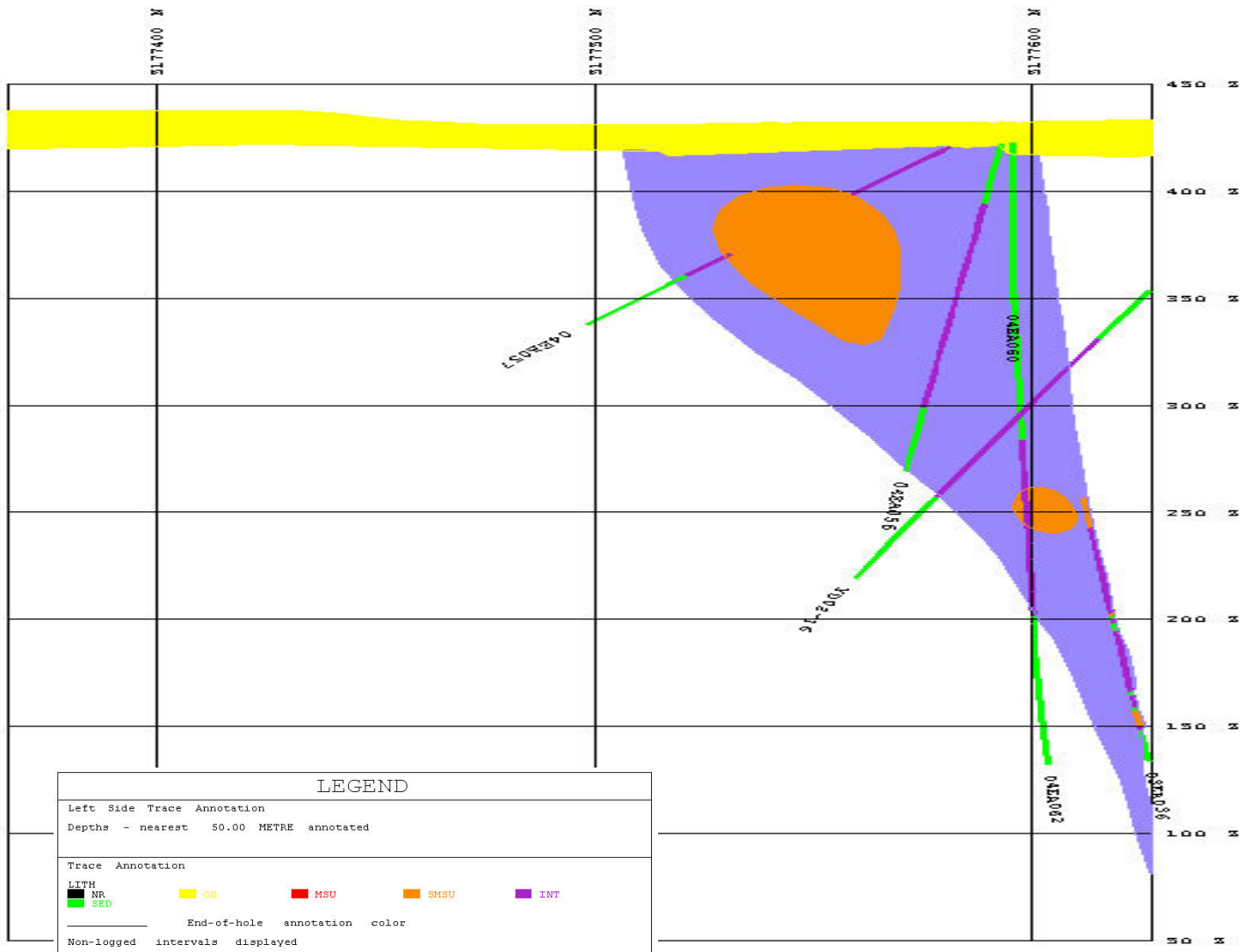
DATE JAN 2005
PROJECT 04-1193-020



DRAWN...PGP...
CHKD...KJB...

LITHOLOGY – SECTION 431420E

FIGURE A7



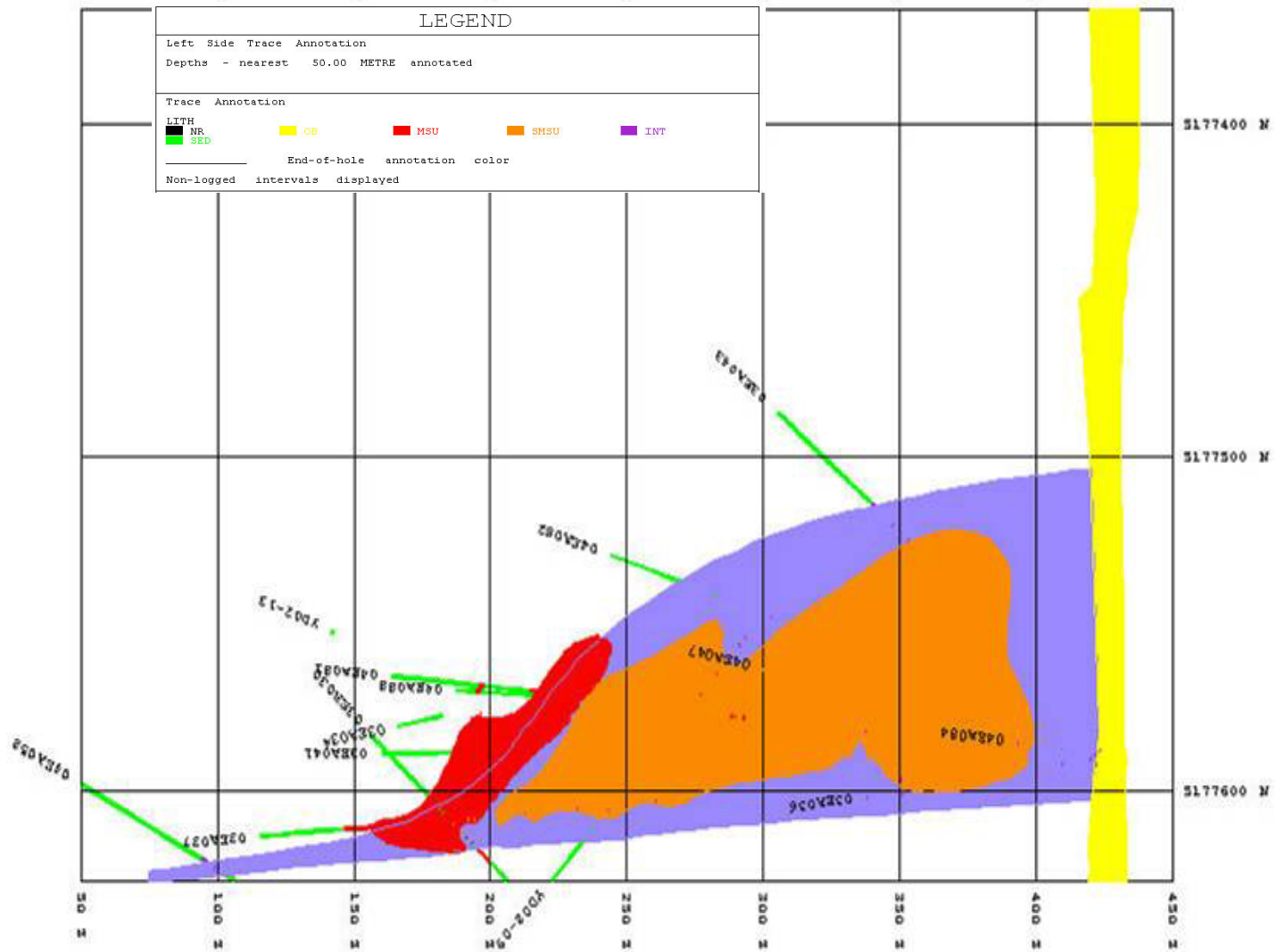
DATE **JAN 2005**
 PROJECT 04-1193-020



DRAWN..PGP..
 CHKD...KJB...

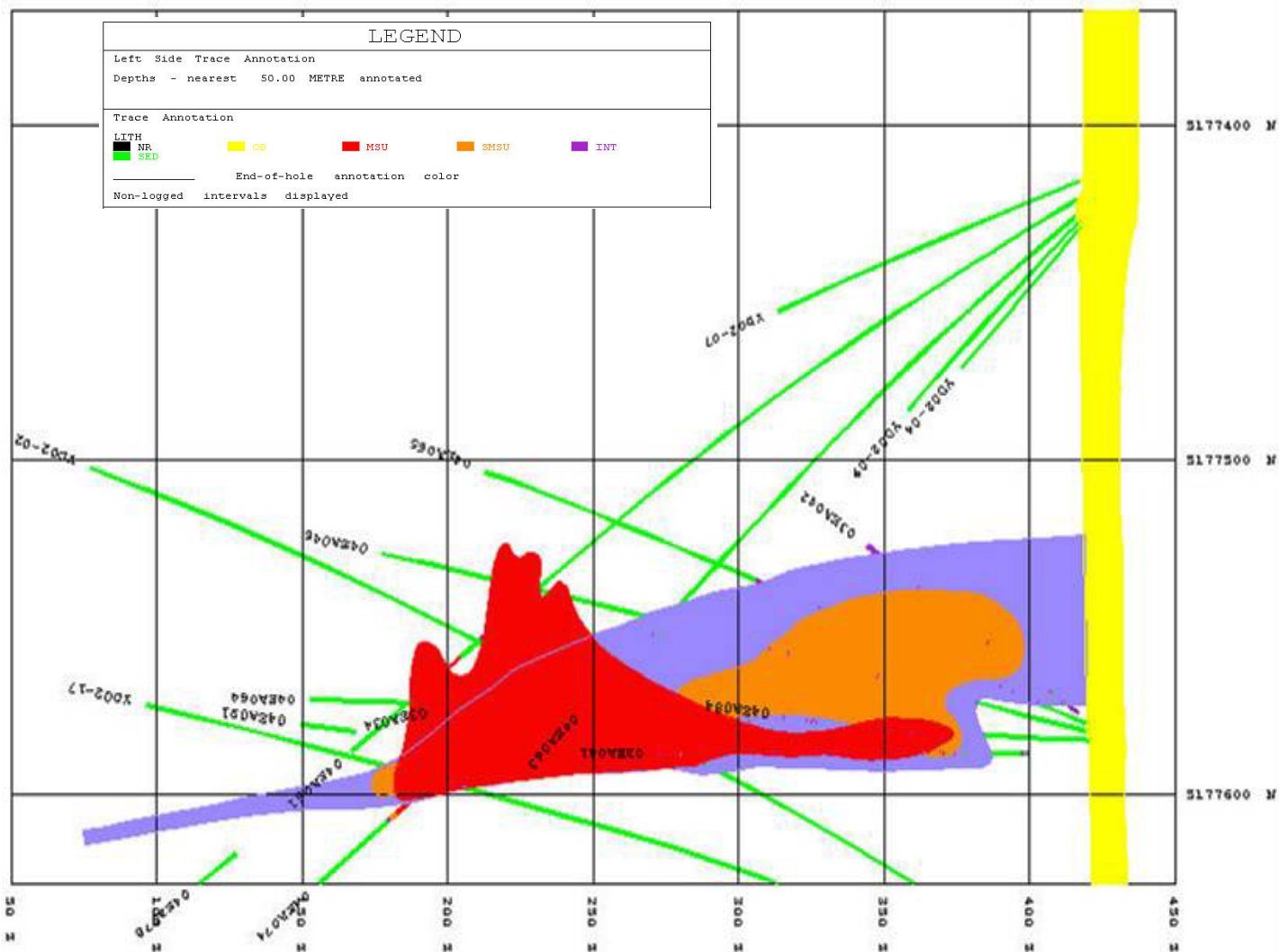
LITHOLOGY – SECTION 431457E

FIGURE A8



LITHOLOGY – SECTION 431500E

FIGURE A9



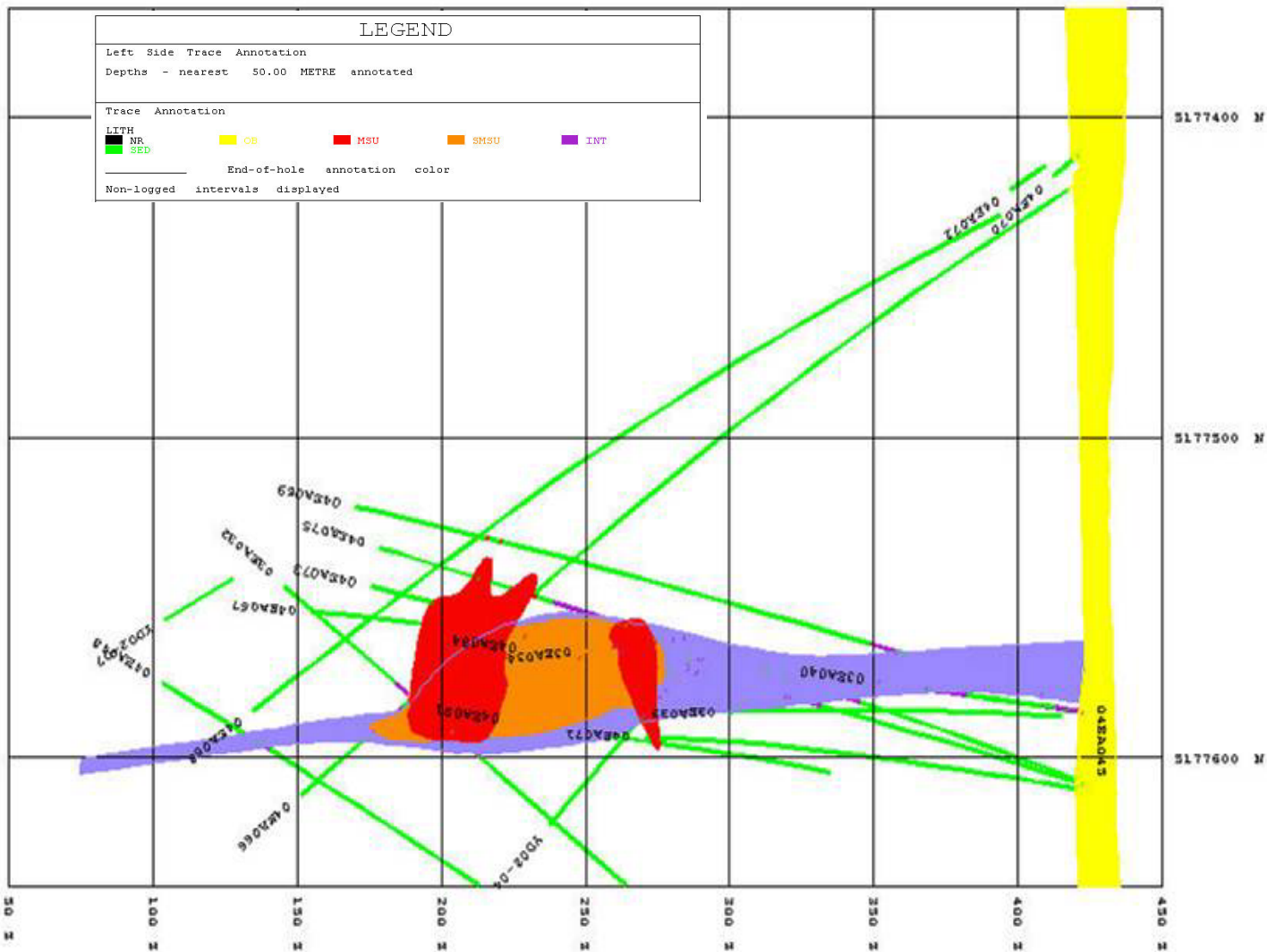
DATE JAN 2005
 PROJECT 04-1193-020



DRAWN.PGP...
 CHKD...KJB...

LITHOLOGY - SECTION 431545E

FIGURE A10

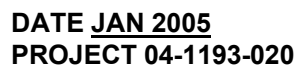


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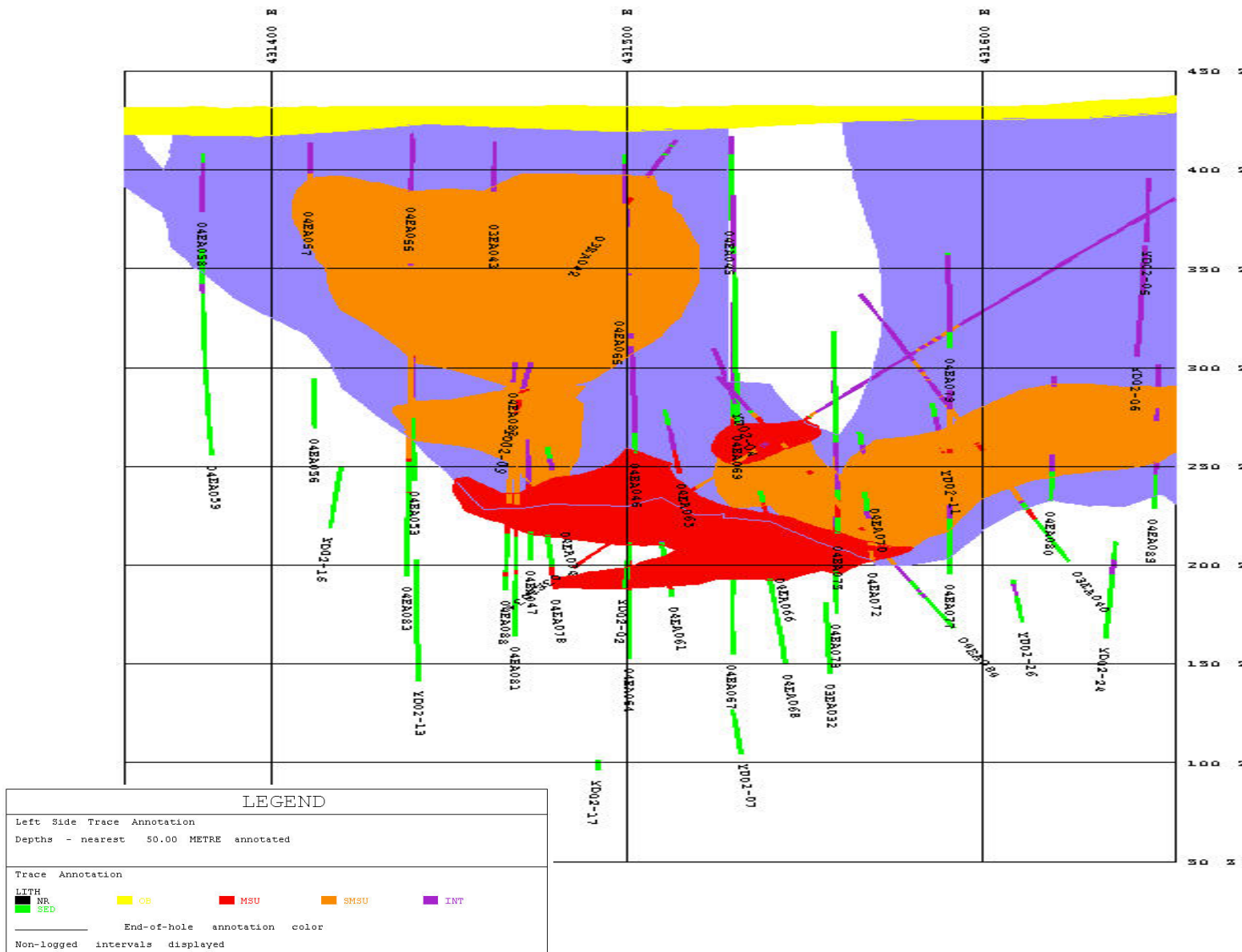
DRAWN.PGP...
CHKD...KJB...

FIGURE A11



LITHOLOGY – SECTION 5177560N

FIGURE A12



DATE JAN 2005
PROJECT 04-1193-020



DRAWN...PGP...
CHKD...KJB...

APPENDIX B

ROCK MASS CLASSIFICATION SYSTEMS

**(EXCERPTS FROM *PREDICTION AND UNCERTAINTIES IN
GEOLOGICAL ENGINEERING AND ROCK MASS
CHARACTERIZATION ASSESSMENTS* -
T.G. CARTER, 1992. PP 1-9 TO 1-12)**

situation a normal shear test may be sufficiently useful to allow representative data to be collected. However such tests may not be suited to the determination of the stress-displacement behaviour of discontinuities isolating a rock block that may potentially slide or fall from the periphery of an underground excavation (Figure 12 lower right diagram). In the former case, dilation is permitted to occur freely, but in the latter case, dilation may be inhibited by surrounding rock such that the normal stress may increase with shear displacement. In this case normal compressive stress - displacement behaviour is highly non-linear and at high values of normal stress σ_n , closure becomes asymptotic to the original aperture of the discontinuity. Accurately determining realistic ranges for shear strength properties thus demands considerable attention to detail, not only in the laboratory but also in describing discontinuity surfaces in the field.

The precise control on shear strength of aperture, is difficult to evaluate. Large apertures can result from shear displacement of discontinuities having appreciable roughness, from outwash of filling materials (e.g. clay), or from solution or from tensile opening of the fracture. In most subsurface rock masses, apertures will be small, usually well less than half a millimetre, but actual values will be difficult, if not impossible, to measure although estimates can be made through measurements of the permeability or hydraulic conductivity of the discontinuity within the rock mass.

This measurement and hence the influence of aperture may also be subject to further error due to the fact that many discontinuities contain full or partial filling comprising, for example calcite, chlorite, clay, silt, fault gouge, breccia, quartz or other minerals. Such filling may also therefore exert a major influence on shear strength. With the exception of those discontinuities filled with relatively strong vein materials (calcite, quartz, pyrite), filled joints will generally have lower shear strengths than comparable clean, closed discontinuities. However, again, although the behaviour of filled discontinuities, which largely depends on the properties of the filling materials, is beyond the scope of this paper, it is important here to note that the lack of a simple definition of controlling shear strength constitutes yet another level of uncertainty within the basic geological database that forms the foundation for any design.

3. ROCK MASS CLASSIFICATION

Some steps towards quantifying the uncertainties inherent in all of these discrete measurements can be achieved by careful comparative classification.

It will be appreciated, by now that evaluating the controlling strength of the rock mass that will surround a proposed excavation or will form a cut slope, on a scale that is meaningful for design is difficult. Laboratory tests of rock specimens provide data relevant only of a very small, highly selective sample of the rock mass from which they were removed. In fact, for most typical projects the samples tested in the laboratory represent well less than 1% of the volume of the rock mass required to be evaluated. In addition, since only those specimens which survive the collection and preparation process are tested, the results of these tests also represent a highly biased sample,

usually representative only of the better rock. Thus again this sampling process, of itself introduces some further level of uncertainty into the data-base.

Several rock mass classification systems have been developed, specifically in an attempt to quantify precedent experience; however, by their very formulation they provide a means for rationalizing some of the vagaries in basic rock mass property definition. Typical of these classifications are those published by Bieniawski (1973, 1976) and by Barton and co-workers (1974). These empirical methods of design, which are based on observations of rock mass and support behaviour in similar situations were essentially developed for use in civil excavations where geometry is typically much simpler than in most mine layouts. Due care therefore must be taken when applying them to excavation design for mining applications. In consequence several modified schemes have been specifically developed for mining use, namely those by Mathews et al. (1981) Potvin et al. (1989) and by Laubscher (1977).

Despite the fact that many simplifying assumptions exist with the use of classification approaches, it is generally accepted that these, essentially empirical design methods provide a good starting point for support design. They are, however, only as reliable as the assessment of the basic parameters. Sound engineering judgement must still be used in selecting the input parameters, bearing in mind the uncertainties in the raw data discussed previously. When reliable representativeness can be assured from detailed evaluation of uncertainties in input data, these methods of assessment are most useful. For preliminary design, when only limited geotechnical data is available, these rock mass classification systems themselves provide a convenient way of comparing different rock mass conditions across a site.

In Civil practise, Barton's Q-system and Bieniawski's RMR method are most commonly used. The mining industry by contrast, generally avoids direct use of either of these schemes preferring use of modified schemes for deep stope design, such as Laubscher's MRMR and/or the Mathews or Potvin's methods. Laubscher's MRMR system is a modification of Bieniawski's RMR system while both the Mathews and Potvin methods are based on Barton's Q. For shallow opening design, recent development work has formulated a further modified classification based mainly on the Q system (Carter, 1992b). Parameters for each of these systems just like the straightforward original schemes, can be estimated from surface mapping and from core logging information then verified as excavation proceeds.

Although each of the classification and evaluation systems differ, they all rely on collection of similar data. However, there are minor differences in the basic suite of parameters defined by the two principal classification approaches. In consequence, there is advantage in applying both of the original schemes for classifying the rock mass and then comparing the deduced RMR and Q values. This procedure not only provides confirmation of the reliability of the field classification data, it also allows inherent rock mass variability and uncertainty in parameter definition to be better defined.

3.1 Barton's Q System

Barton et al (1974) analyzed extensive data relating rock mass quality to the behaviour and support requirements of underground excavations. Barton's original work provided guidelines for evaluating the structure of the rock mass, the frictional characteristics of fractures and the stresses surrounding an opening. It was based on 212 case records of excavations in igneous, metamorphic and sedimentary rocks. Opening spans ranged from 5 - 30 m, and depths ranged from very shallow (<50 m) to 500 m. To date, hundreds of additional openings have been evaluated using the system, which is expressed as follows:

$$Q = \frac{RQD}{J_n} \cdot \frac{J_r}{J_a} \cdot \frac{J_w}{SRF}$$

Where:

RQD = rock quality designation (Deere, 1964)

J_n = joint set number

J_r = joint roughness number

J_a = joint alteration number

J_w = joint water reduction factor

SRF = stress reduction factor

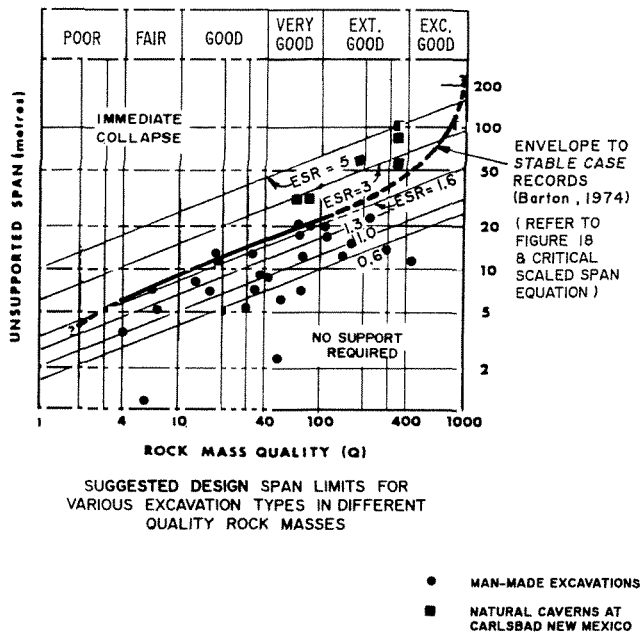


Figure 13 : Basic Q-System Classification Chart Relating Unsupported Span to Rock Mass Quality

As indicated on Figure 13, which shows plotted values from Barton's original database for unsupported stable openings, the value of Q, which varies logarithmically, ranges from 1 for poor ground to 1,000 for exceptionally good quality rock which is practically unjointed. This chart has been further extended by Barton down to 0.001 for exceptionally poor, squeezing ground.

Examination of the terms in the Q relationship presented above, is instructive in considering uncertainty in defining rock mass properties as each term has some physical significance. The term RQD/J_n represents the average block size of the mass; J_r/J_a the peak shear strength of controlling joints and the J_w/SRF term the relative importance of water pressure and the effects of high stress, squeezing, swelling, or loosening of the rock mass. Specific parameter descriptions are given on Table 2.

Table 2 : NGI Q-System Parameter Codes

Item and Description	Parameter & Value	
Number of Sets of Discontinuities		
Massive	J_n	0.5
One set		2.0
Two sets		4.0
Three sets		9.0
Four or more sets		15.0
Crushed rock		20.0
Roughness of Discontinuities		
Noncontinuous joints	J_r	4.0*
Rough and wavy		3.0*
Smooth and wavy		2.0*
Rough and planar		1.5*
Smooth and planar		1.0*
Slick and planar		0.5*
Filled discontinuities		1.0*
Filling and Wall-Rock Alteration, Essentially Unfilled		
Healed	J_a	0.75
Staining only; no alteration		1.0
Silty or sandy coatings		3.0
Clay coatings		4.0
Filling and Wall-Rock Alteration, Filled		
Sand or crushed rock filling	J_a	4.0
Stiff clay filling less than 5 mm thick		6.0
Soft clay filling less than 5 mm thick		8.0
Swelling clay filling less than 5 mm thick		12.0
Stiff clay filling more than 5 mm thick		10.0
Soft clay filling more than 5 mm thick		15.0
Swelling clay filling more than 5 mm thick		20.0
Water Conditions		
Dry	J_w	1.0
Medium water inflow		0.66
Large inflow; unfilled joints		0.5
Large inflow; filled joints with washout		0.33
Large / high transient inflow; filled joints		0.2 to 0.1
Large / continuous inflow; filled joints		0.1 to 0.05
Stress Reduction Class		
Loose rock with clay-filled discontinuities	SRF	10.0
Loose rock with open discontinuities		5.0
Shallow depth (50 m or less) rock with clay-filled discontinuities		2.5
Rock with tight unfilled discontinuities; medium stress		1.0
Rock-Mass Quality for Tunnelling		
Exceptionally poor	Q	< 0.01
Extremely poor		0.01 to 0.10
Very poor		0.10 to 1.00
Poor		1.00 to 4.00
Fair		4.00 to 10.0
Good		10.0 to 40.0
Very good		40 to 100
Extremely good		100 to 400
Exceptionally good		400 to 1000

Adapted from: Barton, N., Lien, R., and Lunde, J., 1974

* Add 1.0 if the mean joint spacing exceeds 3.0 m.

3.2 Bieniawski's RMR System

The Rock Mass Rating (RMR) system was developed by Bieniawski in 1973 and modified over subsequent years as the database of case histories increased. In its current form it is based on data from 189 case histories. It uses six parameters:

- Uniaxial compressive strength of intact rock.
- RQD (rock quality designation).
- Spacing of discontinuities.
- Condition of discontinuities.
- Groundwater conditions.
- Orientation of discontinuities.

Various values are assigned for these parameter as outlined in Table 3 below, such that the final adjusted RMR value can be rated into one of 5 classes which describe the relative quality of the rock mass. Although intended primarily for evaluation with respect to mapped exposures, because of the primary dependance of this classification system on intact strength and RQD measurement, evaluation of borehole core data is often somewhat easier with this method than with the Q approach. Nevertheless both systems can be readily compared as the initial database of 111 case histories worldwide used for the RMR system have not only been correlated with stand-up time and maximum stable spans for underground excavations but also have been correlated with reference to the Q system codes.

Table 3 : Geomechanics System Rating Values

Classification Parameters and Their Ratings									
Strength of intact rock material		Point-load strength index	> 8 MPa	4-8 MPa	2-4 MPa	1-2 MPa	Use of uniaxial compressive test preferred		
		Uniaxial compressive strength	< 200 MPa	100-200 MPa	50-100 MPa	25-50 MPa	10-25 MPa	3-10 MPa	1-3 MPa
Rating			15	12	7	4	2	1	0
Drill core quality-RQD			90-100%	75-90%	50-75%	25-50%	25%		
Rating			20	17	13	8	3		
Spacing of Joints			> 3 m	1-3 m	0.3-1 m	50-300 mm	< 50 mm		
Rating			30	25	20	10	5		
Conditions of joints			Very rough surfaces; not continuous; no separation; hard joint wall rock	Slightly rough surfaces; separation < 1 mm; hard joint wall rock	Slightly rough surfaces; separation < 1 mm; soft joint wall rock	Slickensided surfaces or gouge < 5 mm thick or joints open about 1-5 mm; continuous joints	Soft gouge > 5 mm thick or joints open > 5 mm; continuous joints		
Rating			25	20	12	6	0		
Ground water	Inflow per 10 m tunnel length	Joint water pressure	None		25 L/min	25-125 L/min	> 125 L/min		
	0		0.0-0.2	0.2-0.5	> 0.5				
	Ratio	Major principal stress							
	General conditions		Completely dry		Moist only	Water under moderate pressure	Severe water problems		
	Rating		10		7	4	0		
Adjustments For Joint Orientation									
Strike and dip orientations of joints			Very favourable	Favourable	Fair	Unfavourable	Very unfavourable		
Ratings	Tunnels		0	-2	-5	-10	-12		
	Foundations		0	-2	-7	-15	-25		
	Slopes		0	-2	-25	-50	-60		

Rock Mass Classes And Their Ratings					
Class No.	I	II	III	IV	V
Description	V. Good rock	Good rock	Fair rock	Poor rock	V. Poor rock
Rating	90-100	70-90	50-70	25-50	25
Meaning of Rock Mass Classes					
Class No.	I	II	III	IV	V
Average stand-up time	10 years for 5 m span	6 months for 4 m span	1 week for 3 m span	5 hr for 1.5 m span	10 min for 0.5 m span
Cohesion of the rock mass	300 kPa	200-300 kPa	150-200 kPa	100-150 kPa	100 kPa
Rock mass friction angle	45°	40-45°	35-40°	30-35°	30°
Caveability of ore	Very poor	Will not cave readily; large fragments	Fair	Will cave readily; good fragmentation	Very good

The correlation relationship first proposed by Bieniawski (1976) and which has been widely applied and found to fit most situations, is as follows:

$$RMR = 9 \ln Q + 44$$

or

$$Q = \exp \left(\frac{RMR - 44}{9} \right)$$

Although there is quite considerable scatter in the data that was used to erect this correlation, the overall error band is quite small, thus the correlation has wide applicability. That there is some degree of variation inherent between the two classification systems should be of no surprise, in view of the heterogeneity of most rock masses and the fact that there are several fundamental differences between the approaches used by each classification system to evaluate the rock mass. However, even with such correlations available, considerable caution should be exercised when applying the results for excavation design as accurate application of the rating systems for both is somewhat subjective, certainly requiring experience and judgement if single value results are to be given credibility.

Because of the variability of natural rock masses, there is merit in replacing the subjectivity normally associated with picking single value parameters, with use of probabilistic sampling approaches. These approaches also have additional value in that they tend to provide some insight into the degree of uncertainty in the basic data on which the classifications are based.

In this regard, it should be appreciated that the single most important, and almost, by definition the most statistically variable, of the parameters in both of the main classifications is **Rock Quality Designation (RQD)**. This parameter, which was introduced by Deere in 1964 in an attempt to quantify discontinuity spacing is itself a complex qualitative index of rock competence. In fact the very definition of RQD has been a subject of debate for years. So much so that at least four attempts have been made at developing standard definitions that are not open to interpretation. Some of these differences in interpretation are discussed by Hawkins, 1986, others are addressed by Heuze, 1971 and by Goodman, 1976. They all revolve around definitions of the types of fractures to be included and the precise method by which RQD is calculated.

As initially defined, RQD was to be determined from N sized drill core, counting only the lengths of intact individual pieces of core in the drill run having lengths of 0.1 m or greater, separated by naturally occurring fractures, such that where L is the total length of the drill run and $\sum x_i$ the total length of the acceptable core sticks:

$$RQD = \frac{100 \sum x_i}{L}$$

RQD right from its conception was meant to be an index property, which was defined by Deere as applying universally to NX rock core (Diameter of 54.7 mm or 2.16 in.). Values of RQD determined using a 100 mm (4 in.) reference length for core of other sizes can be misleading as the larger the core, the

greater the potential for maintaining the presence of unopened or marginally healed discontinuities. For instance, H-series, (100mm diameter) core of a particular RQD may represent an artificially higher number than would have been obtained for NX core. Conversely, very small core would tend to reduce the calculated value. Heuze, 1971 and Goodman, 1976 indicate that to maintain similar RQD values as would be measured on N-sized core, the reference length should be altered for different core diameters, such that the reference length would be maintained at about twice the core diameter, as illustrated by the data plotted on Figure 14.

Deere and Deere, 1989 confirmed Heuze's indications that using a 100 mm reference length for determining RQD on

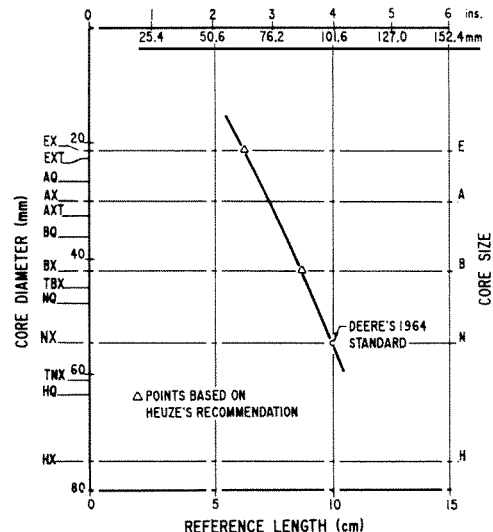


Figure 14 : Relationships between RQD Reference Lengths for Various Core Diameters (data from Heuze, 1971 and elsewhere)

significantly smaller core (EX etc) may give rise to differing values than might have been determined using N-sized core. They however indicate that within a reasonable spread of diameters either side of N, no significant differences in value exist. This suggests that both marginally smaller core (such as NQ, 47.6 mm diameter) as well as marginally larger diameter core are acceptable for computation of RQD using the standard 100 mm. reference length. (Hatheway, 1990) makes the very practical suggestion that the profession should start making prominent notation on drill logs of the defined reference length when RQD is computed for core of a different diameter than N (54 mm) so that the reader may reflect on possible size effects.

It is important to note that determination of values of RQD are also subject to some interpretation with respect to defining reference length related to discontinuity angles intersecting the core. The control exerted by the drilling process on the *correct* determination of RQD values also cannot be over-emphasised as, in many rocks (especially the shaly units), when fresh, or only slightly weathered, it is just about impossible to differentiate between a drilling-induced fracture and a natural discontinuity surface. This then becomes a judgemental issue, relying on the experience of the observer, logging the core.