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MINEREX RESOURCES LTD.

AURORA PROJECT

GEOTECHNICAL ASSESSMENTS AND PRELIMINARY DESIGN CONSIDERATIONS RELATING TO OPEN PIT SLOPES AND EXCAVATION METHODS

FEBRUARY 1988

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Prepared by

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CONTENTS (cont'd)

FIGURES

TABLES

PHOTOS

1. INTRODUCTION

This report summarizes the investigation and analyses carried out to prepare a preliminary design of open pit slopes for the Aurora Gold Project and to prepare an assessment of blasting and related excavation practices at the mine. The terms of reference for the study were discussed in a meeting with Mr. R. Duncan of Minerex Resources Limited and Mr. J.C. Devitt of Canada Tungsten Mining Corporation Limited on October 19, 1987, and are outlined in a letter from Mr. D.C. Martin of Piteau Associates Engineering Ltd. to Mr. R. Duncan dated October 21, 1987.

Results of the investigation, analyses and design recommendations were discussed at a meeting with Mr. Duncan and Messrs. J. Devitt and S. Bartlett of Canada Tungsten Mining Corporation Limited on December 22, 1987. A draft copy of the report was submitted for review on January 8, 1987.

Assessments of the engineering geology, structural geology and rock competency are presented in Section 3. Slope stability analyses and alternative slope designs are presented in Section 4. Assessments and recommendations relating to blasting, rock rippability and excavation methods are presented in Section 5. Conclusions and recommendations for all aspects of the study are summarized in Section 6.

2. DESCRIPTION OF THE INVESTIGATION

2.1 FIELD INVESTIGATION

Mr. A.F. Stewart of Piteau Associates Engineering Ltd. (PAEL) visited the site from October 22 to 27, 1987, and Mr. P.M. Gannon of PAEL visited the site from October 24 to November 2, 1987. During this time, background information concerning the general geology, mineralogy, alteration, geologic structure, etc. was obtained from Mr. A. Glatiotis of Minerex, along with copies of relevant plans, geologic sections, etc. Detailed inspection of the site and geologic structural mapping and documentation of existing rock slopes was carried out. Chip samples from selected percussion drillholes were examined, along with a limited amount of split drill core from an old diamond drilling program. Representative samples of the various rock types were obtained for later point load index testing.

Information relating to groundwater conditions was obtained through discussions with mining personnel. Information relating to the blasting parameters being utilized in different areas of the pit was obtained through discussions with mining personnel and the mining contractor. Further information documenting actual blasts was compiled by mine staff and forwarded to PAEL in Vancouver in December.

2.2 STRENGTH TESTS

To obtain approximate unconfined compressive strengths for the various rock units present at the mine, point load index tests were conducted. Limited direct shear testing of heavily altered andesite was performed to assess the strength characteristics of this material. Shear strengths of joint surfaces were estimated based on visual classification.

2.3 OFFICE STUDIES

Geologic structural analyses and slope stability analyses were carried out using the geologic data and strength test results. The geological mapping data were processed and plotted on lower hemisphere equal area projections using a desktop computer. Statistical methods were used to define the basic geologic structural parameters and to develop an appreciation of the nature and distribution of these features in the rock mass.

Based on the results of the structural analysis, and on a review of geologic sections, the pit was divided into areas wherein the general rock type and geologic structure are similar (i.e. structural domains). The proposed final pit was divided into design sectors within which the geologic structure, rock strength and orientation of the proposed ultimate slope are similar. Separate slope stability assessments were carried out for each design sector. Equal area projections were used to determine the kinematically possible failure modes which are likely to control slope design in each design sector. Mechanical stability analyses were carried out on all possible failure modes and alternative slope design configurations were established. Slope design configurations were developed with due consideration of the effects of strength, groundwater conditions, etc., and with the object of providing sufficient information to prepare preliminary slope designs.

Based on the results of the strength testing, slope documentation, assessment of the rock mechanics properties of the rock units and the documentation of present blasting methods, an assessment of blasting and related excavation practices was carried out.

3. ENGINEERING GEOLOGY

3.1 REGIONAL SETTING

The Aurora property is located in western Nevada, approximately 34 miles southwest of Hawthorne in Mineral County. Ore was first discovered in the area in 1860, with gold having been mined intermittently ever since.

The property is located in the Bodie Hills region in the Walker Belt at the western edge of the Basin and Range province, between about the 7100 and 7500 foot elevations (see Fig. 1). Topography is somewhat rounded. Vegetation is sparse to moderate and is comprised largely of pine trees. Average annual precipitation in the area is about 6 inches, most of which is received in brief summer rainstorms and winter snow storms.

As noted in a report by Questore Consultants Ltd. (1984), the Aurora district is predominantly underlain by a series of Tertiary volcanic and sub-volcanic rocks. Pre-Esmeralda Aurora volcanics, which host the gold lodes, are comprised of a sequence of flows and pyroclastics of andesitic to latitic composition. Adjacent to quartz-gold veins, these volcanics have been variably altered. Unconformably overlying the Aurora volcanics are a sequence of flow-banded rhyolites known as the Bodie Canyon volcanics.

Quartz-gold mineralization was deposited within a series of northeast trending faults related to mid-Tertiary deformation. These mineralized veins have experienced offset in a right-lateral sense by a series of northerly trending shear zones. One such shear zone, referred to as the Prospectus Fault, displaces the main vein at the Aurora Gold Project (i.e. the Humboldt West Vein) about 1400 feet to the north.

3.2 BEDROCK LITHOLOGY AND ALTERATION

The bulk of the rock in the mine area consists of fine grained porphyritic andesite, quartz veins and rhyolite. As shown on section in Figs. 2 to 4, the ore bearing main quartz vein has intruded the andesite at a very steep angle, dipping about 80° to 90° to the northwest. On the footwall side of the quartz vein, the andesite is, for the most part, fresh and hard with some propylitic zones adjacent to the main quartz vein (see Photo 1). This rock will form the southeastern wall of the final pit. The gold bearing main quartz vein, which varies from less than 20 feet up to about 100 feet in width, is comprised of two types of quartz; a banded and contorted variety, and a fine grained vuggy quartz of somewhat reduced grade. The main quartz vein will occur in the final pit bottom and in a narrow zone intersecting the southwestern end of the final pit.

On the hanging wall side of the main quartz vein, the andesite has been hydrothermally altered to varying degrees. Immediately adjacent to the main vein, and near vein contacts of other smaller veins in the hanging wall, silicified or bleached pyritic andesite (i.e. lightly altered andesite) predominates (see Photo 2). The degree of alteration tends to increase with distance from the hanging wall contact, with the moderately to heavily altered rock being referred to as clay altered andesite. This rock is significantly weaker than the lightly altered andesite. The heavily altered material, which has the consistency of a dense fine grained soil with sand to gravel size particles of predominantly quartz, will occupy about the middle third of the northwestern wall of the final pit. Within this material are layers or zones, referred to as "clay zones" (i.e. "maroon clay" and "blue clay"), where the rock has been completely altered.

Rhyolite, which overlies the altered andesite (see Photo 3), will likely be exposed in the upper portion of the northwestern wall of the open pit. Along the contact with the moderately to heavily altered andesite, which appears to be a fault contact in some locations, the rhyolite is typically moderately to heavily altered, with the degree of alteration diminishing with distance from

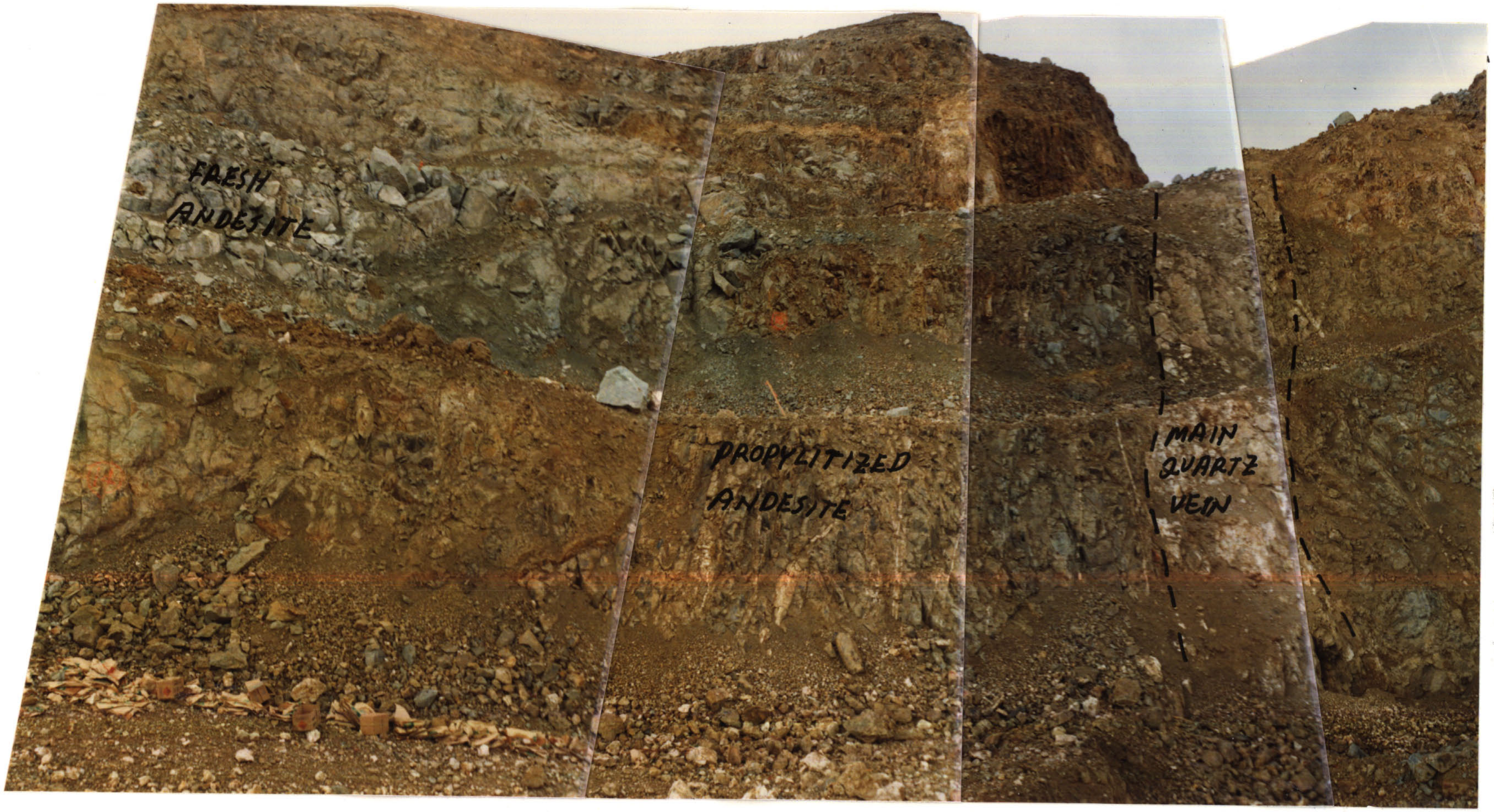

PHOTO 1. Looking at fresh and propylitized andesite on footwall of main quartz vein from 7180 bench.

PHOTO 2. Looking at altered andesite and other rocks on hanging wall of main quartz vein from 7180 foot level.

PHOTO 3. Lightly altered rhyolite on the 7180 foot bench.

the contact. Fresh, unaltered rhyolite appears to be almost as strong and competent as fresh, unaltered andesite. It is noteworthy that, where the rock is heavily bleached and altered, it is sometimes very difficult to distinguish the original rock type (i.e. andesite or rhyolite).

Besides being hydrothermally altered, rock at the Aurora Mine has undergone surface weathering. This weathering is most noticeable and significant in the rhyolite, where the rock has a rather blocky, loose appearance (see Photo 2). Selected near surface sections within the footwall andesite also show significant weathering effects. Here the rock exhibits a similar consistency and strength to the moderately clay altered andesites found on the hanging wall side of the main quartz vein. Fine, ravelled material is more abundant in these areas.

3.3 SURFICIAL SOILS

Surficial materials in the pit area consist largely of mixed grain soils up to about fifteen to twenty feet thick. The particle size ranges from silt to boulder sized, with the coarse material being subangular to subrounded. It is noteworthy that previous mining activity has redistributed the surficial soils over the site, increasing the thickness of the deposits in some areas, particularly on the northwest side of the pit.

3.4 STRUCTURAL GEOLOGY

A rational slope stability analysis and slope design requires that the mine area be subdivided into areas of approximately similar geologic structural characteristics. The engineering behaviour of the slope forming materials can be expected to differ in areas of the rock mass which have different geologic structural characteristics. Hence, extrapolation of stability analysis results and slope design criteria is only valid within areas of the rock mass having similar geologic structural characteristics. Such areas with similar geological structure are called structural domains.

At the Aurora Mine, four basic structural domains were defined, as follows:

1. andesite on the footwall side of the main quartz vein
2. main quartz vein
3. altered andesite and altered rhyolite on the hanging wall side of the main quartz vein
4. unaltered rhyolite

In addition, some rotation of structures within the basic structural domains appears to occur across a fault which transects the pit in the vicinity of Section 1100, forming a secondary structural domain boundary (see Fig. 1). This fault divides the existing mine area into two sub pits: the "Lower" pit to the northeast (designated A) and the "Upper" pit to the southwest (designated B). Thus, a total of eight structural domains (i.e. 1A, 1B, 2A, 2B, 3A, 3B, 4A, and 4B) were defined and structurally assessed.

Attitude of geological structures is the most important consideration in determining whether geological structural populations within a structural domain, or between structural domains, are similar or dissimilar. Other parameters, such as continuity (joint extent or size), roughness, infilling, etc. were considered in evaluating the engineering properties and nature of the joint sets, but were not used for designation of structural domains. Lower hemisphere equal area projections were used to define the peak or average orientation for each main discontinuity set mapped in each structural domain.

3.4.1 Joints

Attitudes of the main joint sets are summarized in Table I and shown on lower hemisphere equal area projections in Appendix A. Throughout the mine area, joint sets are generally seen as reasonably well defined clusters. In some cases, it is reasonable to represent each joint set by a single peak orientation. However, other joint sets are diffuse enough that they may be more readily represented by two or three different peaks. In such cases, the joint set is represented by a number of subsets (e.g. Joint Set JA1, Joint Set JA2, etc.)

As summarized in Table I, up to six joint sets were recognized in the mine area (i.e. Joint Sets A to F). However, only three sets (i.e. Joint Sets A, B and C) are recognizable and well developed throughout the property, with the remaining three sets being somewhat less common and less well developed. Subtle differences in attitude of joint sets are evident from one structural domain to another. For example, a 20^{o} to 25^{o} rotation of the structure occurs between Structural Domains 1A and 1B (i.e. across the fault described in Section 3.4), with Structural Domain 1B being rotated counterclockwise around a vertical axis relative to Domain 1A. This structural rotation does not seem to be quite as pronounced between Structural Domains 2A and 2B, 3A and 3B, and 4A and 4B, respectively. There may also be some counterclockwise rotation around a horizontal axis trending northeast; however, this second rotation is poorly defined.

The joint sets mapped at the mine are described as follows:

i) Joint Set A

Joints of Joint Set A generally strike northeast/southwest and dip very steeply to the northwest or southeast (see Table I). These joints are very well developed, and are parallel to the regional fault set which contains the quartz veining. Joints of Joint Set A are generally continuous over at least one bench.

ii) Joint Set B

Joints of Joint Set B are nearly as well developed as joints of Joint Set A. These joints strike approximately north/south and dip very steeply to the west or east. They tend to be continuous over at least one bench height and are parallel to the younger (i.e. north/south striking) regional fault set which has cut through and offset the quartz veins.

iii) Joint Set C

Joints of Joint Set C strike north/northwest and dip moderately to very steeply to the west-southwest/east-northeast. These joints are developed as well as, and are approximately as continuous as, joints of Joint Set B.

iv) Joint Set D

Joints of Joint Set D, which are relatively poorly developed and were not mapped in all structural domains, strike west-northwest and dip moderately to steeply to the south-southwest. These joints are less continuous than joints of Joint Sets A, B and C.

v) Joint Set E

Joints of Joint Set E strike approximately east-west and dip moderately to the north. These joints have an adverse orientation for the southeast pit wall that will be excavated in relatively fresh andesite on the foot-wall side of the main quartz vein. It is noteworthy that their presence was most noticeable in the recently excavated benches within the andesite, where the rock had been overblasted and significant breakback of the bench crests had occurred (see Photo 4). In the areas of the pit that have been mined by previous operators, little evidence of these structures is present and very little breakback on these structures has occurred. Based on this, and observations of the joint surfaces, it is felt that the potential adverse effects of this joint set can be minimized by careful excavation techniques.

PHOTO 4. Overblasted fresh andesite. Note joints of Joint Set E.

vi) Joint Set F

Joints of Joint Set F strike approximately northeast and dip moderately to the southeast. These joints are considered to be the least well developed and least continuous joints that were mapped during the field investigation.

3.4.2 Faults

With the exception of Structural Domain 1A, relatively few faults were mapped. However, on the basis of the limited data which were collected, fault sets appear to exist parallel to each of the Joint Sets described above. Distribution of faults and veins (which occur as fault infillings) mapped during the field program are shown on lower hemisphere equal area projections in Appendix A. Peak orientations of the various sets are summarized in Table II.

As discussed in Sections 3.1 and 3.4.1, two regional fault systems are present in the mine area, one which trends northeast that has been infilled with gold-bearing quartz veins (Fault Set A), and a second, younger system which trends approximately north-south (Fault Set B).

It can be seen in Appendix A that faults of Fault Set A are the most dominant and well developed. Relatively few faults from other fault sets were observed; however, their presence cannot be completely discounted. The fault that appears to form the structural domain boundary between Domains 1A and 1B, 2A and 2B, etc. is thought to belong to Fault Set D.

The faults that were observed are of various widths and contain a variety of infill materials, ranging from a thin (i.e. <1 inch thick) zone of breccia to a thick (i.e. up to 5 feet thick) zone of stiff clay gouge. Most fault zones were in the order of a few inches thick.

3.5 ROCK COMPETENCY

In addition to the structural geology aspects described above, a rational prediction of the likely behaviour of a rock mass also requires an assessment of the general physical and mechanical properties of the rock mass. Such parameters as susceptibility to mechanical deterioration, alteration, and strength characteristics may be of key importance in evaluating rock mass behaviour. In this regard, a preliminary rock mass quality/competency study was conducted for each of the main rock types based on simple field observations of bedrock exposures. In conjunction with the field mapping program, a number of typical bedrock exposures were inspected and assessed for the following index properties, where applicable:

- rock type
- rock hardness
- estimated RQD of rock mass
- average block size (or average joint spacing) of rock mass
- degree of alteration

Assessed index properties are summarized for each main rock type on Table III.

Most of the accessible bench slopes were documented for bench height and bench face angle. As will be discussed in Section 4, these parameters, which are also a measure of the behaviour and competency of the rock mass in question, were used to predict the likely behaviour of slopes to be excavated in the future.

3.5.1 Unconfined Compressive Strength

Analysis results and field mapping indicate that there is a definite relationship between rock type, degree of alteration, unconfined compressive strength and rock competency. Results of field hardness classifications and point load index tests are sumarized in Table III. These data provide a basis for estimating the unconfined compressive strength of the various rock types which will be encountered in the open pit. Based on Table III, unconfined compressive strengths of about 30,000 psi or greater for fresh andesite or quartz are indicated. As the degree of alteration increases, the estimated unconfined compressive strength drops rapidly to less than about 3,000 psi. At some stage, it may be worthwhile to correlate the point load strength data with results of actual unconfined compressive strength tests on selected intact drillcore, thereby obtaining a more accurate determination of the relationship between point load test results and unconfined compressive strength. This would be particularly important for the more altered materials.

3.5.2 Discontinuity Strength

Little is known concerning the shear strength of discontinuities within the rock mass. However, as most of the discontinuities that are likely to control the stability of the slope are steeply dipping (i.e. $>60^{\circ}$), detailed assessments of discontinuity shear strength are considered unnecessary at this time. For purposes of stability analyses, joints in fresh to moderately weathered rock are assumed to have negligible cohesion and a friction angle of 35°.

3.5.3 Strength of Heavily Altered Rock

Assessments of slope stability in the heavily altered rock which occurs on the hanging wall of the deposit (i.e. on the northwest wall of the proposed final pit) requires an estimate of the rock mass strength. An initial estimate was based on the results of two direct shear tests conducted on samples of intact, heavily clay altered andesite obtained from a bench face and from a muck pile. The results of these shear tests are presented in Fig. 5, where it can be seen that the clay altered andesite has a peak strength that is considerably higher than its eventual residual strength. For preliminary stability analyses, residual strength criteria are considered appropriate. As shown in Fig. 5, the residual strength is open to some interpretation. Residual strengths appear to range from a friction angle of 27° and a cohesion of 20 psi, to a friction angle of 33° and a cohesion of 15 psi, to a friction angle of 36° and a cohesion of 10 psi. All these strengths were assessed and the results compared in the stability analyses.

3.6 HYDROGEOLOGY

Relatively little is known concerning the hydrogeology of the site. No piezometers or other forms of groundwater instrumentation have been installed. However, in discussions with Mr. A. Glatiotis, it was learned that water was encountered in exploration drillholes below about the 7150 foot elevation in the main quartz vein. A water supply well located in the main quartz vein to the northeast of the pit has a similar static water elevation. This well, which has an 8 inch diameter casing installed in it, has been pumped at 250 gpm for 24 hours with a maximum apparent drawdown of about 125 feet.

More recent information concerning the hydrogeology was obtained when a sinking cut was made at the northeastern end of the area that is presently being mined. Following the development of the cut, which was the first cut excavated below the apparent water table elevation of about the 7150 foot elevation, water was observed to be flowing into the excavation from the quartz vein from both the northeast and southwest directions. Furthermore, it is understood that after some time, seepage from the southwest side of the excavation dissipated and blastholes that had been drilled to below the original water level on the southwest side of the excavation and that were initially water filled, dried up. However, on the northeast side of the excavation, seepage continued at a relatively constant rate with little or no decrease in apparent water level.

Based on the above, it would appear that the main quartz vein is relatively permeable with a present peizometric level at about the 7150 foot elevation. Furthermore, it would appear that there is a source of recharge to the quartz vein from the northeast end of the pit. The water in the quartz vein to the southwest appears to be in storage with little, if any, recharge from the southwest. Monitoring of seepage into the sinking cut (i.e. maintaining comprehensive pumping records and visually observing seepage occurrences), along with monitoring of water levels within the main quartz vein to the northeast and southwest of the excavation, should be implemented to enable a better understanding of the hydrogeology. Additional discussions with regard to hydrogeological aspects are included in Section 6.3.

4. SLOPE STABILITY ANALYSES AND SLOPE DESIGN

Slope stability analyses involved investigating all kinematically possible failure modes (i.e. failure modes involving discrete blocks, formed by discontinuities, which are free to slide or topple into the excavation) which could lead to shallow failure of individual benches, and/or deep seated failure involving large sections of the overall slope. Consideration was also given to the past performance of the slope with respect to evaluation of basic failure modes as well as general rock mass behaviour.

4.1 BASIC SLOPE DESIGN CONSIDERATIONS

In rock slopes, instability may result from failure along structural discontinuities such as bedding, joints, geological contacts, faults, etc. (i.e. kinematic failures). In high slopes or slopes in relatively weak rock or altered rock, instability may also develop as a result of failure through intact rock or along a deep seated failure surface consisting of a combination of discontinuities and intact rock. In analyses of deep seated failure, an assessment of the rock mass strength is usually required.

When assessing failure mechanisms related to structural discontinuities, the most important factors are the orientation, geometry and spatial distribution of discontinuities in the slope. It is also important to evaluate these discontinuities with respect to both the orientation and alternative possible angles of the proposed pit slope. It is following these basic principles that the slope stability analyses were carried out.

Slope control can be basically accomplished in two ways:

a) to design the slope so that no failures occur or

b) to excavate the pit under controlled conditions and to design the slope with adequate access so that failures can be caught on berms and accordingly removed, if necessary.

The first solution is usually too conservative to be economically feasible. The second solution requires thorough consideration of slope geometry so that failures are contained on berms and safe access is available to the berms to allow removal of collected debris, if required. This solution provides adequate safety at minimal cost, although special design or remedial measures may be required to ensure the stability of haulroads or critical installations on benches.

The parameters which govern the geometry of a slope are shown in Fig. 6. These parameters are primarily controlled by the strength and nature of the rock. Bench height should be selected to provide a safe working slope as well as an optimum overall slope angle. It should be noted that, without affecting the slope angle, higher benches will allow wider berms for better protection and more reliable and easier access, if this is desirable, although the size of possible failures may increase.

Berm width should be controlled by the access required to the slope, as well as by the optimum width to accomodate failures. It must be accepted that, even with careful perimeter blasting techniques, some breakback may occur. In general, slopes should have berms wide enough to trap falling debris and, if and where desired, provide sufficient access for equipment to keep berms clean and effective as catchments.

By inclining the bench faces, blasting damage is reduced and high stresses are less likely to develop near the bench crests. Hence, tension cracks and overhangs are minimized. Avoiding these problems accordingly reduces the amount of rockfall and increases the safety of the slope.

4.2 ENGINEERING GEOLOGY AND DESIGN SECTORS ON THE FINAL SLOPES

Rational slope stability analysis and slope design requires prediction of the geologic structural conditions which will occur on the final pit walls. Such a prediction includes determining the distribution and location of lithologic units, major structures, etc. (i.e. Structural Domain boundaries). At Aurora, the basic approach to preliminary slope design was to consider slopes in the various structural domains separately, with Structural Domains 3A and 3B being further subdivided into those areas of light to moderate alteration (i.e. Structural Domains 3A and 3B) and moderate to heavy alteration (i.e. Structural Domains 3A' and 3B'), respectively.

Not only must structural domains be considered for individual analysis; the overall orientation of the final pit walls must also be considered. Different pit wall orientations may require different design considerations. Hence, it is usually necessary to define zones which contain one structural domain and one general slope orientation. These zones are designated design sectors.

All available geological information was used to project the structural domain boundaries onto the proposed final wall. The structural domain and straight slope segments on the proposed final wall were used to determine the design sector boundaries and delineate the design sectors shown in Fig. 7. Information relating to pit wall orientations, rock type and related slope information for each design sector is given in Table IV. In each design sector, consideration was given to deep seated failure (i.e. rock mass failure), as well as to bench failure, to evaluate the optimum geometry required to control failure and ravelling of individual benches on the slope.

4.3 ASSESSMENT OF POSSIBLE DEEP SEATED FAILURE MECHANISMS

In general, deep seated rock mass failure is not anticipated in the fresh or lightly altered andesite and rhyolite or the main quartz vein. However, rotational failure of moderately to heavily altered rocks on the hanging wall side of the main quartz vein in Design Sectors 3A-1', 3A-2', 3B-1' and 3B-2' is of concern. Surficial soils could also fail by this mechanism.

For preliminary slope design, deep seated instability was assessed using simple stability charts developed by Hoek & Bray (1977), based on standard slip circle analyses.

For the moderately to heavily altered andesite and rhyolite at Aurora, analyses were conducted for the range of preliminary estimates of rock mass strength obtained from the two direct shear tests discussed in Section 3.5.3 and presented in Fig. 5. Assuming a piezometric level at about the 7150 foot elevation, analyses were carried out assuming a dry or dewatered slope in Design Sectors 3B-1' and 3B-2', and a high groundwater condition in Design Sectors 3A-1' and 3A-2'. A Factor of Safety of 1.2 was considered adequate for preliminary assessments for both overall or interramp slopes as well as for bench scale slopes of 40 and 60 feet high.

Analyses results are summarized in Table IV, where it can be seen that for undrained slopes up to about 100 feet high in Design Sectors 3A-1' and 3A-2', overall slope angles up to about 50° are acceptable. In these same design sectors, maximum bench face angles of 80° for 40 foot high benches and 70° for 60 foot high benches are considered reasonable in the altered rock. In Design Sectors 3B-1' and 3B-2', where drained slope heights in moderately to heavily altered rock are expected to be up to about 200 and 120 feet high respectively, overall slope angles of about 50° and 60°, respectively, appear feasible. Bench face angles of 80° for 40 foot high benches and 75° for 60 foot high benches should be attainable in these design sectors.

4.4 STABILITY ANALYSES AND ASSESSMENTS OF KINEMATICALLY POSSIBLE FAILURES

4.4.1 Slope Design Based on Orientation of Geological Structure

As discussed above, individual discontinuities or combinations of discontinuities may form discrete blocks which could result in failure of the slope or benches (i.e. kinematically possible failure modes).

Because there are variations in intensity, orientation and dip of the joint and fault sets throughout the mine, the significance of one particular failure mode on the stability of a particular slope may be low. However, the combined significance of all possible failure modes, with respect to both the actual and predicted performance of the slope may be high and should be evaluated to prepare a rational slope design.

Within most areas of the mine, many faults and joints are continuous over at least one to two bench heights (i.e. 20 to 40 feet). This is particularly true of Joint Sets A, B and C. However, because of the generally steep dipping nature of most structures in the mine area, it is unlikely that failure would develop over more than one bench height.

4.4.2 Determination of Kinematically Possible Modes of Failures

Lower hemisphere equal area projections of planes representing the peak orientations of the various fault sets and joint sets in each design sector were used to define possible failure modes. Failure modes which are considered to be kinematically possible are illustrated on the equal area projections in Appendix C.

4.4.3 Mechanical Stability Analyses

Simple limit equilibrium stability analyses were carried out for each possible failure mode using computer techniques. As discussed in Section 3.5.2, neglible cohesion and a friction angle of 35^{o} were assumed for all discontinuities.

To simplify the analysis, drained slopes were assumed, and a Factor of Safety greater than or equal to 1.2 was considered adequate for stability of dry (i.e. drained or dewatered slopes. A Factor of Safety greater than or equal to 2.0 (assuming the dry condition) was considered adequate for stability of slopes subject to adverse groundwater conditions (i.e. undrained or fully saturated slopes).

4.4.4 Assessment of Possible Failures

In most areas, with the exception of Design Sectors 3A-1', 3A-2', 3B-1' and 3B-2', the principal kinematic controls are wedge and planar failures on benches. As shown in Table IV, these potential failures vary in terms of their intensity or importance. That is, the relative degree of development of individual discontinuity sets or combinations of discontinuity sets which form potential failures, and therefore the likelihood of bench faces consistently breaking back to or forming along such failures, is variable. Based on the estimated intensity/importance of the principal kinematic controls summarized in Table IV, and on engineering judgement, the apparent plunge or dip of potential failure modes considered to control stability of individual benches was selected (see Table IV).

For example, in Design Sector 1A-1, the numerous strongly developed wedges at an apparent plunge of 83° are felt to control the stability of the benches. While the potential for planar failures on Joints of Joint Set E which dip at about 40° is recognized, this joint set is not felt to be strongly enough developed to control the design of the bench. In this case, breakback along joints of Joint Set E is only considered to be a local problem, unless overblasting results in opening and/or extension of these joints.

In general, because most of the strongly developed kinematic controls are steeply dipping, it is unlikely that large deep seated failures could develop in the unaltered to moderately altered rocks in the pit. Hence, in terms of design of interramp slopes, bench geometry will control the overall slope design in most areas.

4.4.5 Bench Breakback Analysis

A brief study of bench breakback was carried out to evaluate the behaviour of existing benches at the mine. Bench breakback was obtained by direct measurement of representative bench face angles in the mine for benches with similar orientations to those of the proposed final walls. It is noteworthy that bench face angle measurements were made on recently excavated benches (i.e. on those benches excavated by Minerex) as well as on benches that were excavated by previous operators in the last few years.

Results of this study are summarized in Table IV and indicate that average bench face angles of about 72^{o} have been excavated in the past in both the andesite and moderately to heavily altered rocks. Present day mining has resulted in average bench face angles of about 63^{o} to 64^{o} in similar geologic conditions. As will be discussed in Section 5, present day mining has led to considerable blast damage to the benches.

In general, bench faces have been excavated at angles that are slightly flatter than the apparent plunge or dip of failure considered to control bench stability and summarized in Table IV. Only in Design Sector 1B-1, where the principal kinematic controls are not as strong as in other areas, has the average excavated bench face angle been steeper than that indicated by kinematic analysis.

4.5 ALTERNATIVE INTERRAMP SLOPE DESIGNS

Except within the moderately to heavily altered rocks on the hanging wall (i.e. northwest) side of the pit, the possibility of deep seated failure involving multiple benches on large portions of the slope is low. Consequently, slope design has for the most part been carried out by assessing possible bench failures and related alternative bench geometries. These analyses can then be used to select optimum bench design and, hence optimum design of overall slopes. Consideration was also given to the results of the bench breakback study and other relevant aspects.

The slope design illustrated on plan in Fig. 7 and on section in Figs. 2 to 4 represents the ultimate pit design developed by Minerex. This design, which is uniform throughout all areas of the pit, consists of 40 foot high benches with 70° bench face angles and 19 foot wide berms. All 40 foot high benches are developed in two 20 foot high lifts. The interramp slope angle is 50°. Alternative slope designs based on the results of the above work are summarized in the following and on the right hand side of Table IV.

4.5.1 Bench Geometry

i) Bench Height

Based on assessment of the engineering geology and the kinematic controls, and because of the advantages of achieving bench heights as high as possible (with due consideration of safety and efficiency), 40, 60 and 80 foot high benches were evaluated in most design sectors. Maximum bench heights of 60 feet were assessed in the weaker, moderately to heavily altered rocks.

ii) Bench Face Angle

In most design sectors, it is anticipated that with careful controlled excavation, it should be possible to achieve bench face angles of 70° to 80°. Generally, bench face angles have been equated to the apparent plunge or dip of the principal kinematic control or to the documented bench face angle from previous mining, whichever is greater. Within the moderately to heavily altered rocks, bench face angles have been based on the results of bench scale rock mass failure analyses as discussed in Section 4.3. A maximum bench face angle of 80° is assumed for all benches. Although some small failures will no doubt occur with these steep bench face angles, large scale consistent breakback of final wall benches is not anticipated if appropriate perimeter excavation techniques are employed.

iii) Berm Width

As shown in Fig. 6, total berm width is defined as being the sum of the breakback of the bench crest (i.e. between vertical and the design bench face angle) and the effective berm width required to provide access, contain small failures and rockfalls, etc. At the Aurora Mine, effective berm widths of 25 feet for 40 foot high benches and 30 feet for 60 and 80 foot high benches are recommended. These berm widths provide greater catchment width than is provided in the design shown in Fig. 7, and allow for some additional breakback of the benches without the berms becoming so narrow as to be ineffective.

4.5.2 Interramp Slope Angles

Based on the assessments carried out and the bench geometries discussed above, the range of slope angles which appears feasible in each design sector is summarized on the right hand side of Table IV. As can be seen, significantly steeper intermediate (i.e. interramp) slope angles can be achieved if the bench height is increased. However, increased care and caution with drilling, blasting and excavating must also be taken when increasing the bench height. As will be discussed below, improved excavating procedures will have to be developed with trial slopes to ensure that steeper and higher slopes can be safely mined.

The overall significance of steeper slope angles is best seen in Design Sector 1A-1, which encompasses the full height of the southeast (i.e. footwall) slope for most of its length. The present pit design, shown on a typical section in Fig. 8, includes the pit bottom at the 6980 foot elevation and a 50^{o} interramp slope angle. Increasing the bench height to 60 feet and the bench face angle to 80^{o}, with an effective berm width of 30 feet, yields an interramp slope angle of 56^{o}. More importantly, at the 6980 foot elevation, the width of the pit bottom would increase by about 50 feet, thereby allowing deepening of the pit and an increase in mineable reserves. Similarly, increasing the bench height to 80 feet would gain about 78 feet in width at the 6980 foot elevation, possibly allowing even further deepening of the pit. Final determination of the extent to which the pit could be deepened would also involve an evaluation of the overall increase in slope angle that could be achieved on the northwest side of the pit.

The consequences of not maintaining bench face angles as steep as possible can also be signficant. As shown in Table V, if effective berm widths of 25 feet for 40 foot high benches and 30 feet for 60 and 80 foot high benches are maintained, the effect of flattening the bench faces to 72^o (i.e. the bench face angle documented from previous mining) would be to reduce the interramp slope angle by about 5^o to 6^o. Similarly, if bench face angles were reduced to about 63^o to 64^o (i.e. the angle documented as a result of recent mining) the intermediate slope angle would be reduced by a further 5^o to 7^o. Such decreases in slope angle could have a considerable economic impact on mining.

5. EXCAVATION METHODS

5.1 ASSESSMENT OF ROCK RIPPABILITY

5.1.1 General Discussion

Rippability, or ease of excavation of a rock mass, is primarily a function of intact rock strength and joint spacing. Other parameters of concern are degree of weathering/alteration, joint type, joint orientation, joint length and joint infilling materials.

One of the most accepted methods of assessing rippability is to determine the seismic velocity of the rock mass using the seismic refraction technique. Caterpillar Tractor company has developed charts which relate ripper performance to seismic velocity for various tractors. However, accuracy of results of seismic refraction studies decreases with depth of penetration, particularly if hard beds or zones with higher seismic velocity are likely to overly soft beds or lower velocity zones. In these cases, down-the-hole or cross-hole seismic techniques may be more applicable.

A number of workers have developed or proposed rock mass classification systems to assess rippability, excavation methods and equipment requirements. Some of these systems have been field tested. These systems should only be used as a guide for preliminary assessment, as local variation in ground conditions and actual engineering geology conditions at a particular site will dictate the optimum excavation method.

5.1.2 Rock Rippability at Aurora

A preliminary assessment of the rippability of the various rock types at Aurora was carried out using the results of the field investigation and point load testing as input into four rock mass classification systems. Because there is a strong correlation between rock type, degree of alteration and rock mass competency, separate rock mass classifications were prepared for each of the seven general rock types or categories that were point load tested and summarized in Table III, including: unaltered andesite, quartz vein, lightly altered andesite, moderately clay altered andesite, heavily clay altered andesite and rhyolite, lightly altered rhyolite and rhyolite/silicified rhyolite. This approach allowed a reasonably comprehensive assessment of the main rock types and degrees of alteration found throughout the pit. As shown in Table VI, it also allowed a range of anticipated excavation conditions to be assessed for each of the design sectors in the open pit.

Each rock mass classification system was reviewed and the relevant rock mass parameters were estimated for each rock type and alteration category. This information was used to prepare an estimate of the rock mass rating or index, the excavation method and the equipment requirements indicated for each classification system. In some cases, due to uncertainties regarding the rating of certain parameters, values had to be estimated as a range of possible ratings.

Results of the estimates for the various classification systems are summarized in Table VI. While it can be seen that there is some variation in the results between classification systems, the following general preliminary conclusions are presented:

i) Moderately to heavily clay altered andesite and rhyolite and related rocks (i.e. Design Sectors 3A-1', 3A-2', 3B-1' and 3B-2') are probably rippable using a D7, D8 or D9 or equivalent tractor.

ii) Lightly to moderately clay altered andesite and rhyolite and related rocks (i.e. Design Sectors 3A-1, 3A-2, 3B-1 and 3B-2) may be rippable with a D8, D9 or D10 or equivalent. However, the more competent rocks may require blasting to loosen or fracture the rock.

iii) Unaltered or very lightly altered andesite, rhyolite or quartz (i.e. Design Sectors 1A-1, 1B-1, 1B-2, 2B-1, 4A-1, 4A-2 and 4B-1) will probably require blasting to fracture or loosen the rock. Some ripping with a D9 or D10 or equivalent tractor may be possible in a portion of the lightly altered rhyolite.

Results of the preliminary assessment of rippability clearly indicate the variability of rock competency in the pit area. Moreover, it appears that a portion of the rock on the hanging wall is rippable. However, the actual amount of rock which is rippable will depend on the equipment used and possible variations in rock strength, geological structure and rock competency throughout the mine area. While the actual distribution of strong and weak, altered and unaltered zones may vary significantly on a local basis, and a detailed and accurate assessment of rippability is difficult to obtain, the general conclusions presented above with respect to rippability are considered valid.

Depending on the mining method and excavation philosophy chosen, equipment selection must recognize the variability of the rock mass and the fact that equipment requirements could change over the mine life. It is clear that at different stages of mine development, different percentages of the rock will require drilling and blasting versus ripping. Further discussions, with regard to the potential for ripping, ripping trials, etc., is included below in section 6.2.

5.2 ASSESSMENT OF BLASTING

5.2.1 General Blast Documentation

Blasthole drilling at Aurora is carried out in 20 foot lifts plus four feet of subgrade. All holes are vertical and 4.5 inches in diameter. Anfo is the normal blasting agent with drilling cuttings used for stemming. Charges are not decoupled. Where possible, blasting is to a free face. However, many of the blasts are at least partially choked. Both row-by-row blasting and "Vee" cut blasting are utilized. Up to 15 delays at 25 to 50 msec/delay are typical.

Within the ore, 10 foot x 10 foot staggered patterns are typical for production blasts. Holes are loaded with 100 lbs of Anfo/hole, yielding a powder factor of 0.6 lb/ton. The stemming depth is about nine feet. It was observed during the site visit that in one particular blast, a considerable amount of oversized quartz ore was generated. Secondary blasting was necessary to reduce the rock to a suitable size for the crusher.

Within the hard andesite on the footwall, 12 foot x 12 foot staggered patterns have been utilized for production blasts. Typically, the back row of such blasts has been loaded with only about 50 lbs of Anfo/hole. All other production rows are loaded with one 4 inch x 32 inch cartridge of "T805 Anfo" (i.e. a high strength, high velocity hexamine nitrate slurry with aluminum) plus 75 lbs of Anfo/hole, yielding an overall powder factor of 0.6 lb/ton. Stemming depth is about 9 feet.

On the hanging wall side of the main quartz vein (i.e. in the altered andesite, rhyolite, etc.), production blasting is apparently carried out using a 14 foot x 14 foot staggered pattern. As for the andesite, the back row is loaded with 50 lbs of Anfo/hole. The remaining production holes are generally loaded with 100 lbs. of Anfo/hole with about 9 feet of stemming, yielding a powder factor of about 0.3 lb/ton. However, because the competency of the hanging wall rocks is so variable, one cartridge of "T805 Anfo" is inserted at the bottom of blastholes that have encountered hard drilling. It is understood that up to 40% to 50% of the blastholes in a pattern may be loaded with "T805 Anfo" in this manner.

5.2.2 Documentation of Specific Blasts

More recent data, including actual blast plans and comments relating to six blasts in quartz and hanging wall rocks on the 7120 bench, were received on December 22, 1987. From this information, which has been summarized on Table VII, a number of general observations can be made concerning the approximately 10 foot x 10 foot staggered patterns that were used. First of all, it would appear that a row-by-row firing sequence is being used within these rocks and that the powder factor has been reduced to 0.5 lb/ton for the most recent blasts. It would also appear that fragmentation and easier digging has been achieved when the blast is not choked and/or when the overall width of the blast pattern is greater than or equal to the overall pattern depth. Where the blast pattern is choked, is considerably deeper than it is wide and has a large number of delays, such as in Blasts B and F, it is very difficult to propogate adequate breakage and loosening of the rock mass.

It would seem that Blast E achieved the best results in terms of degree of fragmentation, ease of digging and minimal overbreak. Changing the firing sequence from a row-by-row blast to a "Vee" blast would probably improve blasting results to some degree. However, other basic changes, such as modifying the relative dimensions of the blast, ensuring that blasting is to a free face, etc., may also be required to optimize these blast designs. Further discussion and recommendations with regard to improving blasting practices at the mine are included below in Section 6.2.

6. CONCLUSIONS AND RECOMMENDATIONS

6.1 SLOPE DESIGN AND TRIAL SLOPES

Based on the results of the slope stability analyses summarized in Section 4 and the maximum overall slope height of about 350 to 400 feet, it is recommended that interramp slope designs incorporating 60 foot high benches or 80 foot high benches, as summarized on the right hand side of Table IV, be used for preliminary slope design. Both of these design options will result in generally steeper (i.e. $\geq 50^{o}$) interramp slope angles, with the end result that it should be possible to increase ore production without moving the pit crest beyond that illustrated in Fig. 7.

Because of the nature of the pit and geological conditions at Aurora, theoretical final slopes include small design sectors with different designs than adjacent design sectors. Zones of transition between adjacent design sectors have not been included in these slope designs. In general, recommended slope angles should not be exceeded in transition zones. However, because of the amount of berm width provided in the slope designs (i.e. to account for bench breakback) it is considered reasonable to allow slight deviations in transition areas (i.e. within about 2^{o}) in interramp slope angles from those recommended in Table IV. While it is recognized that steepening could result in additional breakback and narrower bench widths, increasing the possibility of rockfalls, steepening or flattening the interramp slope in small areas would allow more rational incorporation and blending of the recommended slope designs into the overall pit plan. Taking these possible adjustments into account, areas of the pit with similar interramp slope angles have been combined and are illustrated on Fig. 9 for slope designs incorporating both 60 foot high benches and 80 foot high benches (with 60 foot high benches in the moderately to highly altered rocks).

While both of the above mentioned designs are felt to be possible from a kinematic and slope stability standpoint, there may be some operational or other constraints that will preclude the use of 80 foot high benches. In addition, it will no doubt be more difficult and require more care with all phases of the excavation process to develop 80 foot high benches, as opposed to 60 foot high benches. Thus, to fully assess the two preliminary slope designs, it is recommended that trial slopes be developed before final walls are excavated. These slopes should incorporate trial perimeter blasts, ripping trials in weaker rocks, etc., to evaluate all aspects for potential wall steepening.

Trial slopes should be developed in areas where failures can be allowed to occur without affecting the efficiency and safety of the operation. If failures or other problems do occur on a particular trial section, provision should be made so that the failure can be controlled and the mining geometry modified without jeopardizing the safety of the mine.

If future exploration and mine planning indicates deepening of the pit by more than about 100 feet is feasible, a geotechnical review of slope design recommendations should be carried out.

6.2 EXCAVATION METHODS

The hazard of local instability, rockfalls and general ravelling increases as both the jointing and blasting damage increases. Other than thorough scaling and possibly local application of mesh and rock bolts, little can be done to control unfavourable effects of jointing. However, blasting damage can be minimized by control perimeter blasting on the final wall, careful production blasting and, where practical, the use of ripping instead of blasting. The following comments are intended to provide guidance in minimizing blasting damage throughout the pit.

6.2.1 Ripping

The preliminary geotechnical investigation for the Aurora Project has indicated that there is a significant variation in rock strength, degree of alteration and rock competency in different areas of the rock mass. Moreover, it is concluded that a portion of the rock, particularly the moderately to heavily altered rock, will be rippable. Thus, ripping trials are recommended during initial mining stages to assess which portions of the rock mass can be most efficiently excavated by ripping. Even if the productivity from ripping is shown to be inferior to that of conventional drill and blast methods, it is recommended that, where possible, ripping be used to excavate the final walls to minimize the possible effects of blasting on the final wall and to preserve the inherent strength within the altered rock mass. Chip logging, core logging and point load index testing are recommended on subsequent exploration boreholes to obtain a better appreciation of the variation in rock quality throughout the mine area.

6.2.2 Perimeter Blasting

Results of the preliminary assessment of blasting at Aurora indicate that significant changes from present practices will have to be made if steep interramp slope angles (i.e. in the order of 50° to 60°) are to be established on the final pit walls. In this regard, control blasting (or ripping) is mandatory in all areas of the final pit slope to maintain the inherent strength within the slope and to control excessive bench breakback.

The exact type of control blasting technique to be used on the final slopes depends on the results of blasting trials. In this regard, two different blast layouts are recommended and presented schematically in Fig. 10 as being reasonable starting points for perimeter blasting trials. Both of these layouts are for pre-split blasting in the hard andesite that will form the entire footwall (i.e. southeast) slope of the pit. The presplit row should be fired at least 100 msec before initiation of the main perimeter blast, with the main blast being fired "en-echelon" to reduce the destructive energy that would otherwise be transmitted instantaneously into the final wall if the perimeter blast were fired row-by-row. Twenty five millisecond delays are recommended between each "en echelon" row.

High velocity, decoupled charges of an appropriate pre-splitting explosive strung in the holes would probably provide the best results for the pre-split (i.e. back) row. It is also recommended that a reduced powder factor of about 0.3 lb Anfo/ton be tried in the perimeter blasts.

The main difference between the trial blast layouts in Figs. 10a and 10b lies in the blasthole diameter and the associated drilling pattern. As can be seen in Fig. 10a, where it is assumed that the present 4½" diameter blastholes are maintained, the pre-split row and the main perimeter blast pattern are much more closely spaced than in Fig. 10b, where it is assumed that a 6" diameter blasthole can be drilled. While it is not known whether it would be possible for the present drilling equipment to drill 6" diameter holes, it is felt that the hole depth to hole diameter ratio is more favourable for 6" diameter blastholes than for 4½" diameter blastholes. In addition, the total drilling footage required for 6" diameter holes is considerably less than for 4½" holes, and the charge in the larger holes will tend to have a lower centre of gravity.

To increase the effectiveness of the perimeter blasts, it is recommended that, where possible, two free faces be present (see Fig. 10a). Two free faces will generally provide better movement of the muck and less damage to the final wall. If only a single free face is available, the firing sequence illustrated in Fig. 10b should help to reduce blast damage to the final wall. In addition, planning the blasts such that subgrade drilling is reduced in the perimeter blast, and ensuring that blastholes span the crests of final benches, will assist in reducing damage to final walls.

With regard to perimeter blasting in the quartz, hard rhyolite and slightly altered andesite on the hanging wall side of the main quartz vein, it is recommended that similar patterns to those illustrated in Fig. 10 be used as a starting point for blasting trials. As discussed in Section 6.2.1, in the moderately to heavily altered rocks, ripping should be used, where possible, to excavate the final slopes.

While it is felt that the above perimeter blasting recommendations will help to control excessive breakback and increase the stability of final slopes, further reduction in blasting damage and consequent steepening of bench face angles possibly could be achieved with other control blasting methods. Effects of using inclined preshear holes, cushion blasting, buffer blasting, different powder factors, etc. should be considered in blasting trials on interim slopes where possible.

6.2.3 Production Blasting

Production blasting should be designed so that blasting damage to the final wall and the area of the perimeter blast is minimized. For the trial perimeter blast patterns illustrated in Figs. 10a and 10b it is recommended that distances of at least 12 feet and 15 feet, respectively, should be allowed between the last production row and the first perimeter row of blastholes to help minimize blast damage. This, combined with a powder factor of about 0.5 lbs/ton in 4½" diameter production blastholes and about 0.4 lbs/ton in the 6" diameter production blastholes is recommended as a starting point for improved production blasting. In addition, utilizing "en echelon" or "Vee" blasting will likely lessen the amount of overblasting, without reducing fragmentation. Blasting to at least one free face should also help to improve the overall quality of the blasts.

As observed during the site visit, a considerable amount of oversized quartz was produced in one particular blast. It is recommended that better fragmentation could be obtained in this very hard rock by using an explosive with a higher velocity than that of Anfo.

Notwithstanding the above, optimum results (i.e. with respect to fragmentation, eliminating overblasting damage to final walls, eliminating dilution in ore, etc.) can only be obtained by varying blasting techniques and correctly supervising and designing field trials where loads, spacing, burden, delays, etc. are varied to obtain the best possible results. It is sound engineering to make special provision for a series of trial blasts to ensure optimum results.

6.3 GROUNDWATER AND SURFACE WATER CONTROL

As discussed in Section 3.6, relatively little is known concerning the hydrogeology of the site. However, based on the available information, it is concluded that the main quartz vein is relatively permeable, with the existing piezometric level in this vein being about the 7150 foot elevation. It is also concluded that the surrounding rocks, which almost certainly are less permeable than the main quartz vein, probably have a piezometric level equal to or slightly higher than that of the quartz.

Within most areas of the pit, stability analyses indicate that little or no slope steepening could be achieved by depressurizing the slopes. However, in some areas, such as in Design Sector 1A-1, depressurizing of moderately dipping, but relatively discontinuous joints (i.e. such as Joint Set E) could improve the stability of the benches. The nature and behaviour of the moderately to heavily altered rocks below the water table are unknown at this time, but may be significantly different than that assumed in the stability analyses conducted for this study. Hydrogeological conditions and material properties should be investigated during the upcoming drilling program to enable a more accurate assessment of the overall stability of the moderately to heavily altered rocks and to help determine if depressurization of these rocks is practical or even necessary.

With regard to dewatering the quartz vein and maintaining dry blastholes for the production and perimeter blasts, it would appear from available information that a sump should be maintained at the northeastern end of the pit in the main quartz vein. In addition, it is recommended that the water flow into the sump be monitored (i.e. by maintaining comprehensive pumping records) to determine such factors as approximate rate of flow, preferential direction of flow, change of flow rate with time, etc. The installation of at least one standpipe piezometer in the quartz vein in at least one drillhole during the upcoming exploration program would allow monitoring of the response of the piezometric level to drawdown in the sump. Consideration should also be given to installing piezometers in the altered hanging wall rock above the quartz vein.

An efficient system of surface drainage ditches should be maintained to control surface water behind and in the pit, with the surface water directed away from the pit. Ditches on haulroads and specific benches would also help to control runoff from both precipitation and snowmelt.

6.4 CLEANING BERMS AND SCALING

All benches should be adequately scaled to minimize rockfalls. This is particularly important if 80 foot high benches are utilized. Debris buildups may require cleanup at a later date in certain areas. If possible, berms should be kept relatively free of excessive buildup of rockfalls and ravelling material to maintain adequate catchments. Ideally, if berms are accessible from both ends, access will not be lost if a bench failure occurs.

6.5 MONITORING SLOPES FOR MOVEMENT

Performance of all slopes should be carefully evaluated on an ongoing basis by regular visual field surveys and/or other forms of monitoring systems to detect slope displacements. Movement of the top, as well as the bottom, of slopes should be recorded. It may be feasible to install a straight line of survey hubs at a few key locations on the slopes. Direct distance measuring equipment can be used to considerable advantage in this work as well.

Because of the difficulty of making absolute predictions about the behaviour of the slope, there is always the possibility that the most carefully engineered slope will become unstable and will require remedial measures. Thus, the purpose of monitoring would be to provide data about current and anticipated stability of the slope. Also, for purposes of efficiency and safety, this data would assist in making rational design modifications as mining proceeds.

6.6 SUMMARY OF RECOMMENDATIONS FOR FURTHER GEOTECHNICAL WORK

A number of additional tasks which should be carried out by mine personnel to further evaluate and update the preliminary slope design studies summarized herein are outlined in the following. Piteau Associates would be pleased to assist Minerex with planning and initiating this work.

i) Geologic Structural Mapping

Because of the importance of the geological structure to slope design, detailed geologic structural mapping should be carried out on an ongoing basis by mine personnel as mining proceeds. Mapping of major faults and/or fault sets and joint sets should be carried out to update the geologic structural mapping, to determine if the structural populations are changing, and thus to determine if any slope design modifications are required.

ii) Core Logging and Strength Testing

As only chip samples are presently available from which to estimate the competency of the various rock types at depth, it is recommended that all drillholes in the upcoming exploration program be carefully logged. Core from the diamond drillholes should be drilled and handled with care and geotechnically logged immediately after being drilled and before being split. A limited number of core samples should be taken for further strength testing, as required. Careful sampling and strength testing is likely to be most important in the moderately to heavily altered rocks. For the percussion holes, the chip samples should be logged for lithology and degree of alteration. All of the above information should be summarized and compiled on section to update the existing geotechnical data base.

iii) Hydrogeological Investigations

Monitoring of all blastholes should be continued to determine when water bearing rock is being encountered in the main quartz vein, and to determine if and when rock other than the main quartz vein is subject to undrained conditions. In addition, and as mentioned in Section 6.3, the installation of piezometers in at least one drillhole during the upcoming drilling program would allow monitoring of the response of the piezometric level to drawdown in the sump. For maximum benefit the piezometers should be located southwest of the sump in an area where they will not be destroyed by mining activity in the near future.

iv) Documentation and Back Analysis of Failures

Documentation and back analysis of any slope failures should be carried out to obtain additional information on the strength of the discontinuities or rock mass. A periodic visual inspection of the pit should be carried out to determine visible signs of movement.

7. ACKNOWLEDGEMENTS

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Respectfully submitted,

PITEAU ASSOCIATES ENGINEERING LTD.

Alan F. Stewart, P.Eng.

February 18, 1988

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FIGURES

JOB NUMBER 87-975

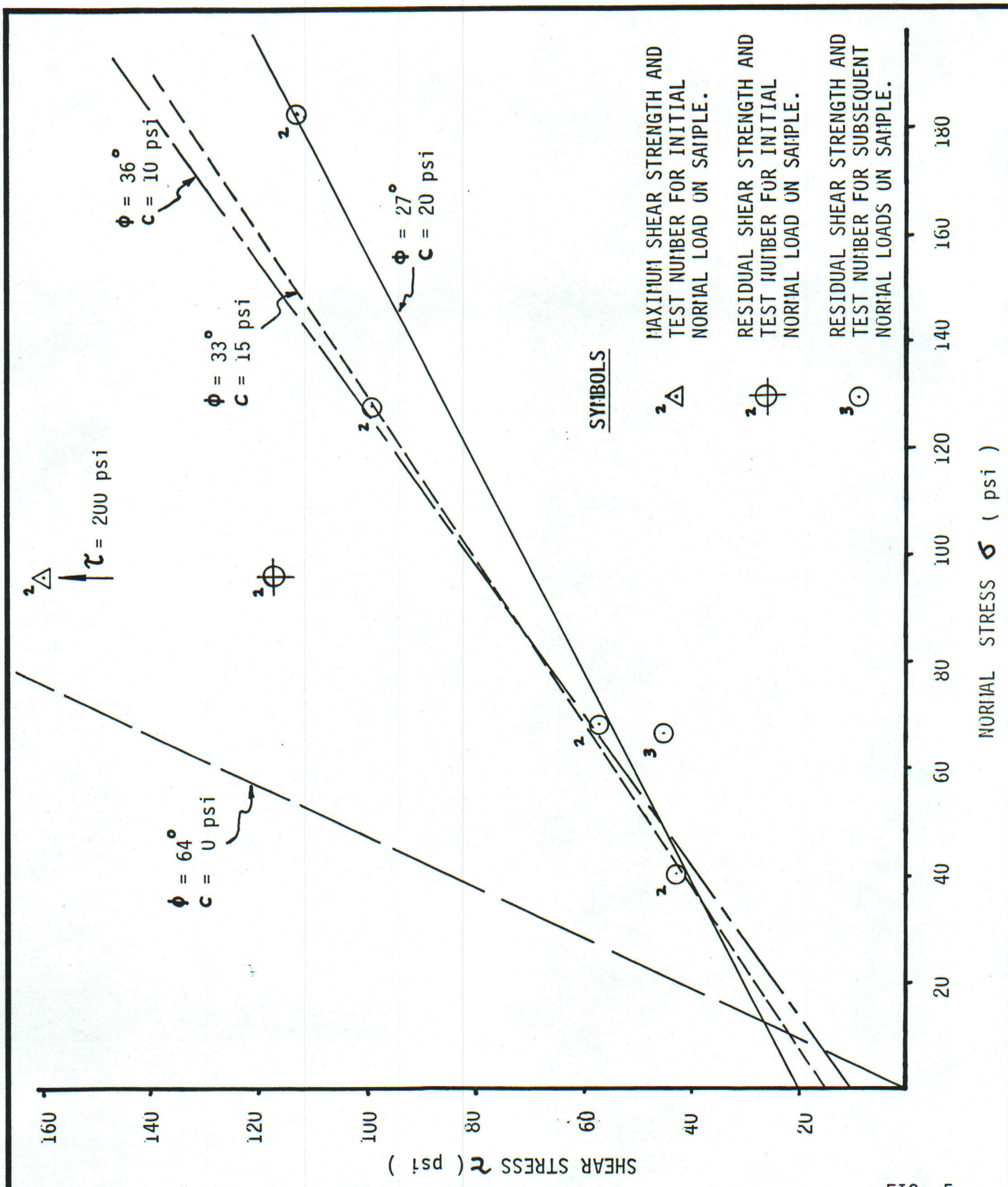
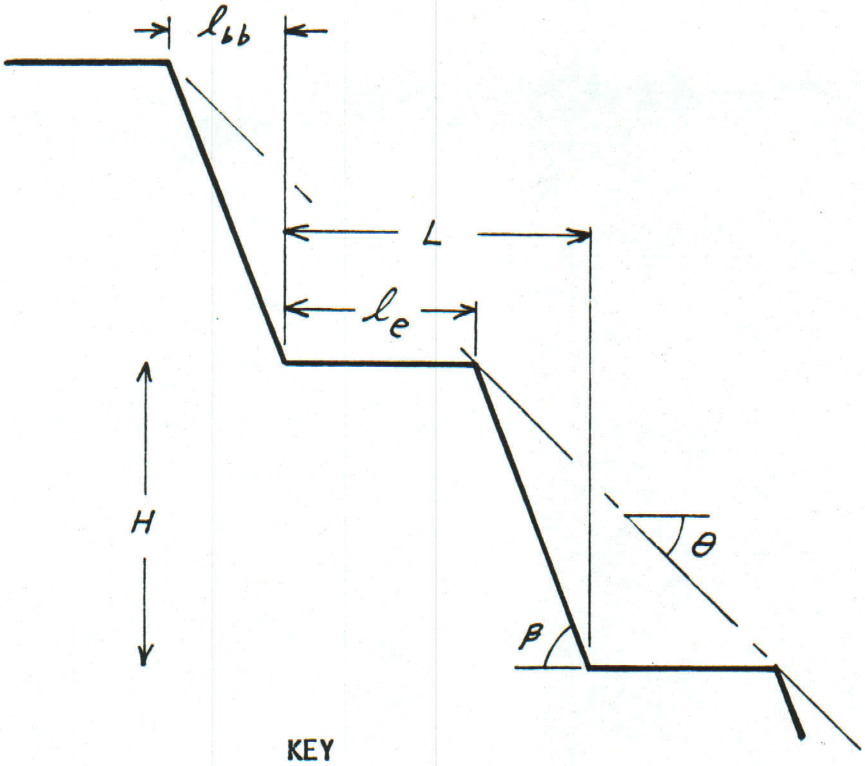


FIG. 5

MINEREX RESOURCES LIMITED AURORA PROJECT GEOTECHNICAL ASSESSMENT	PITEAU ASSOCIATES GEOTECHNICAL CONSULTANTS VANCOUVER CALGARY		
RESULTS OF DIRECT SHEAR TESTS ON HEAVILY CLAY ALTERED ANDESITE	BY: PMG	DATE: DEC 87	
	APPROVED: AFS	DWG:	

JOB NUMBER 87-975



KEY

H = Bench Height

l_{bb} = Breakback of Bench Crest

l_e = Effective Berm Width

L = Total Berm Width ($l_e + l_{bb}$)

β = Bench Face Angle

θ = Interramp Slope Angle

FIG. 6

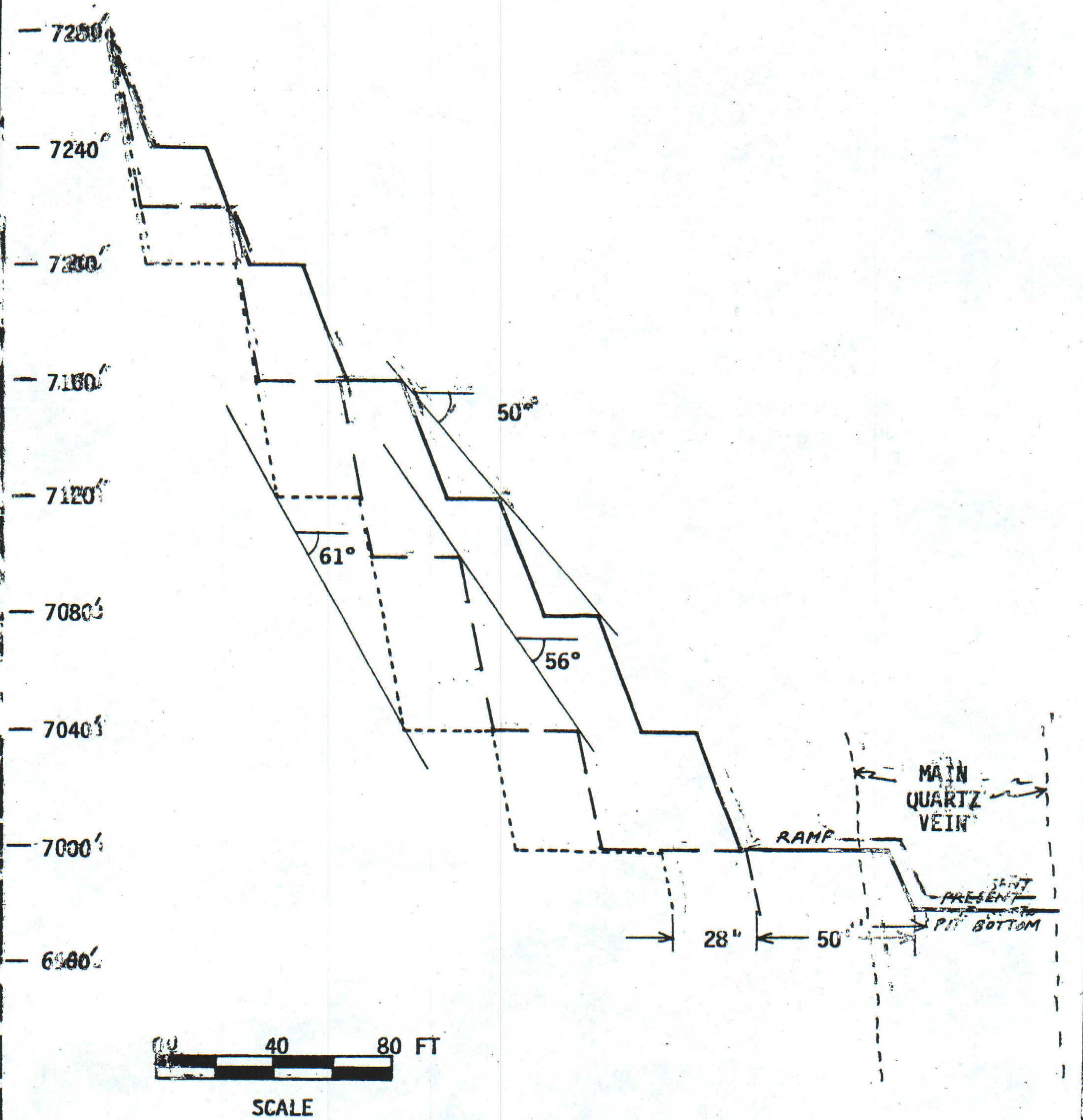
MINEREX RESOURCES LTD.
AURORA PROJECT
GEOTECHNICAL ASSESSMENT

PITEAU ASSOCIATES
GEOTECHNICAL CONSULTANTS
VANCOUVER CALGARY

BENCH GEOMETRY PARAMETERS

BY:	DATE:
AFS	12/87
APPROVED: A.S.	DWG:

JOB NUMBER 87-275



Note: See Table IV for details of alternative bench configurations.

FIG. 8

MINNEREX RESOURCES LTD.
AURORA PROJECT
GEOTECHNICAL ASSESSMENT

PITEAU ASSOCIATES
GEOTECHNICAL CONSULTANTS
VANCOUVER CALGARY

SCHEMATIC OF ALTERNATIVE BENCH CONFIGURATIONS
FOR DESIGN SECTOR 1A-1

BY:	DATE:
AFS	12/87
APPROVED:	DWG:

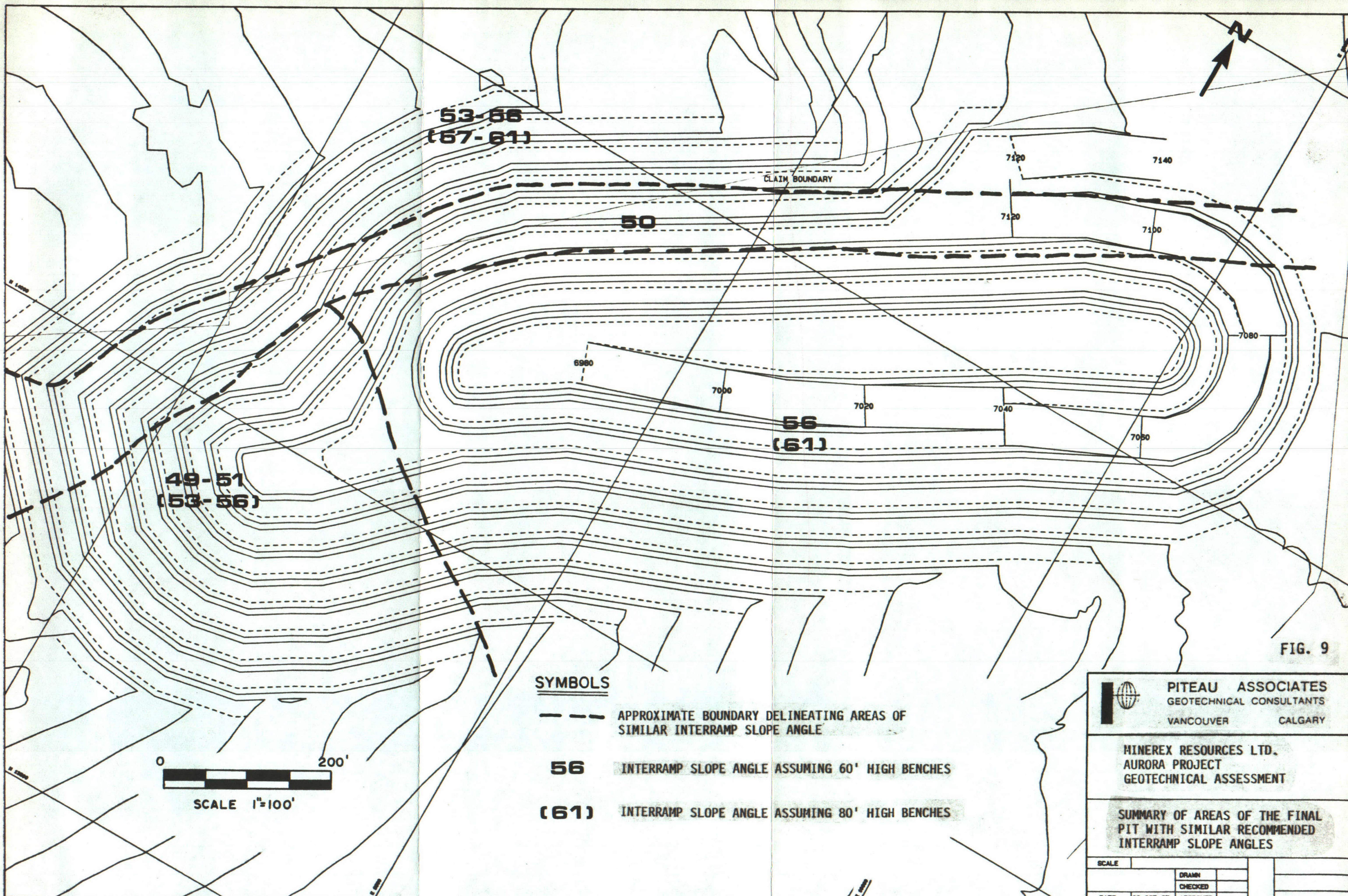


FIG. 9

SYMBOLS

— — APPROXIMATE BOUNDARY DELINEATING AREAS OF SIMILAR INTERRAMP SLOPE ANGLE

56 INTERRAMP SLOPE ANGLE ASSUMING 60' HIGH BENCHES

(61) INTERRAMP SLOPE ANGLE ASSUMING 80' HIGH BENCHES

PITEAU ASSOCIATES
GEOTECHNICAL CONSULTANTS
VANCOUVER CALGARY

MINEREX RESOURCES LTD.
AURORA PROJECT
GEOTECHNICAL ASSESSMENT

SUMMARY OF AREAS OF THE FINAL PIT WITH SIMILAR RECOMMENDED INTERRAMP SLOPE ANGLES

SCALE			
		DRAWN	
		CHECKED	
DATE	28-OCT-87	APPROVED	

TABLE I

ORIENTATION OF JOINT SETS WITHIN STRUCTURAL DOMAINS[1]

BASED ON SURFACE MAPPING

STRUCTURAL DOMAINS	AREA AND ROCK TYPE	POPULATION	JOINT SET A				JOINT SET B				JOINT SET C				JOINT SET D				JOINT SET E				JOINT SET F			
			SET	DIP DIRECTION	DIP	%/NO.	SET	DIP DIRECTION	DIP	%/NO.	SET	DIP DIRECTION	DIP	%/NO.	SET	DIP DIRECTION	DIP	%/NO.	SET	DIP DIRECTION	DIP	%/NO.	SET	DIP DIRECTION	DIP	%/NO.
1A	Lower Pit - Footwall Andesite	320	JA1 JA2	337 315	87 86	8/26 5/16	JB1 JB2	287 271	87 80	7/22 5/16	JC1	235	65	2/6	JD1	203	72	3/10	JE1	355	37	4/13	-	-	-	-
1B	Upper Pit - Footwall Andesite	96	JA1 JA2	357 340	85 90	6/6 4/4	JB1	301	79	3/3	JC1	265	82	8/8	JD1	234	85	8/8	JE1	351	66	4/4	JF1	152	58	5/5
2A	Lower Pit - Main Quartz Vein	20	JA1	334	83	25/4	JB1	289	66	10/2	JC1	065	81	35/7	-	-	-	-	-	-	-	-	-	-	-	-
2B	Upper Pit - Main Quartz Vein	16	JA1	321	70	19/3	*	-	-	-	JC1	078	89	16/2	*	-	-	-	-	-	-	-	-	-	-	-
3A	Lower Pit - Altered Andesite and Altered Rhyolite	105	JA1 JA2 JA3	313 159 329	74 84 75	9/9 9/9 8/8	JB1 JB2	265 081	88 78	7/7 6/6	JC12	244	86	7/7	*	-	-	-	-	-	-	-	JF1	147	33	7/7
3B	Upper Pit - Altered Andesite and Altered Rhyolite	45	JA1 JA2	340 308	73 71	9/4 9/4	JB1	113	87	7/3	JC1 JC2 JC3	253 251 237	85 51 61	15/7 6/3 6/3	JD1	191	61	4/2	-	-	-	-	*	-	-	-
4A	Lower Pit - Rhyolite	68	JA1	143	88	7/5	JB1 JB2	106 084	86 82	8/5 6/4	JC1 JC2	068 047	79 62	9/6 7/5	JD1	195	46	7/5	JE1	016	34	4/3	JF1	140	40	7/5
4B	Upper Pit - Rhyolite	20	JA1 JA2	343 318	66 69	20/4 10/2	JB1	267	85	15/3	-	-	-	-	-	-	-	-	JE1	345	34	12/2	-	-	-	-

NOTES:
1. The main orientations of the discontinuity sets were determined from contoured lower hemisphere equal area projections of joints mapped in the structural domains.
2. "%/NO." refers to the percent concentration and corresponding number of joints in a one percent area of the lower hemisphere for the average or peak orientation of the population.
3. "*" indicates that there were some structures mapped that appeared to belong to the joint set in question, but that there was insufficient data to determine a statistically significant peak orientation.

TABLE II

ORIENTATION OF FAULT SETS WITHIN STRUCTURAL DOMAINS[1]

BASED ON SURFACE MAPPING

STRUCTURAL DOMAINS	AREA AND ROCK TYPE	POPULATION	FAULT SET A				FAULT SET B				FAULT SET C				FAULT SET D				FAULT SET E				FAULT SET F			
			SET	DIP DIRECTION	DIP	%/NO.	SET	DIP DIRECTION	DIP	%/NO.	SET	DIP DIRECTION	DIP	%/NO.	SET	DIP DIRECTION	DIP	%/NO.	SET	DIP DIRECTION	DIP	%/NO.	SET	DIP DIRECTION	DIP	%/NO.
1A	Lower Pit - Footwall Andesite	65	FA1 FA2	330 307	88 88	15/10 10/6	-	-	-	-	-	-	-	-	FD1	211	63	5/3	-	-	-	-	FF1	167	61	7/5
1B	Upper Pit - Footwall Andesite	9	*[3]	-	-	-	-	-	-	-	-	-	-	-	FD1	054	87	22/2	-	-	-	-	*	-	-	-
3A, 4A	Lower Pit - Altered Andesite, Altered Rhyolite and Rhyolite in Hanging Wall	25	FA1 FA2	326 300	78 68	14/3 18/4	-	-	-	-	*	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
3B, 4B	Upper Pit - Altered Andesite, Altered Rhyolite and Rhyolite in Hanging Wall	12	FA1 FA2	335 318	81 62	17/2 17/2	-	-	-	-	FC1	241	68	17/2	-	-	-	-	-	-	-	-	-	-	-	-

NOTES:
1. The main orientations of the discontinuity sets were determined from contoured lower hemisphere equal area projections of faults mapped in the structural domains.
2. "%/NO." refers to the percent concentration and corresponding number of faults in a one percent area of the lower hemisphere for the average or peak orientation of the population.
3. "*" indicates that there were some structures mapped that appeared to belong to the fault set in question, but that there was insufficient data to determine a statistically significant peak orientation.

TABLE III

SUMMARY OF POINT LOAD INDEX TESTING RESULTS AND ESTIMATED ROCK COMPETENCY PARAMETERS

ROCK TYPE	HARDNESS DESIGNATION[1]		ESTIMATED UNCONFINED[2] COMPRESSIVE STRENGTH FROM POINT LOAD TESTING (psi)	ESTIMATED RQD OF ROCK MASS (%)	ESTIMATED AVERAGE BLOCK SIZE OR AVERAGE JOINT SPACING OF ROCK MASS (ft)
	FROM MAPPING	FROM POINT LOAD TESTING			
UNALTERED ANDESITE	R4 - R5	R5	29,400 (8)	80 - 100	1.5 - 3
QUARTZ VEIN	R5	R6	53,200 (3)	75 - 100	2
LIGHTLY ALTERED ANDESITE					
- propylitized andesite (Footwall)	R4	R4	13,900 (12)	75 - 100	1 - 2
- propylitized andesite (Hanging Wall)	R4	R4	15,200 (6)	75 - 100	1 - 2
- bleached andesite	R4	R4	15,900 (3)	75 - 100	1 - 2
MODERATELY CLAY ALTERED ANDESITE	R2 - R4	R2	3,300 (7)	25 - 50	<.15
HEAVILY CLAY ALTERED ANDESITE and RHYOLITE	R1 - R2	-	-	0 - 25	<.15
LIGHTLY ALTERED RHYOLITE	R4	R4	11,200 (5)	50 - 75	0.5 - 2
RHYOLITE/SILICIFIED RHYOLITE	R4 - R5	R5	24,000 (10)	75 - 100	1 - 3

NOTES: 1. The relationship between hardness and unconfined compressive strength is given in Appendix B.

2. The unconfined compressive strength of each rock type was estimated from the mean and the median of results of point load tests on a limited number of samples. Number of samples tested is given in brackets.

TABLE V

ASSESSMENT OF INTERMEDIATE SLOPE ANGLES

FOR VARIOUS BENCH FACE ANGLES

BENCH HEIGHT (ft)	EFFECTIVE BERM WIDTH (ft)	BENCH FACE ANGLE (°)	INTERMEDIATE SLOPE ANGLE (°)
40	25	80 72 63	51 46 41
60	30	80 72 63	56 50 45
80	30	80 72 63	61 55 48

TABLES

TABLE IV

SUMMARY OF SLOPE INFORMATION AND ALTERNATIVE INTERRAMP SLOPE DESIGNS

DESIGN SECTOR	DESCRIPTION OF DESIGN SECTORS			PRINCIPAL KINEMATIC CONTROLS				CALCULATED AND DOCUMENTED BENCH FACE ANGLES			ASSESSMENT OF POTENTIAL FOR ROCK MASS FAILURE USING STABILITY CHARTS DEVELOPED BY HOEK & BRAY (1977)					ALTERNATIVE INTERRAMP SLOPE DESIGN[5]						COMMENTS[8,9]	DESIGN SECTOR
									DOCUMENTED BENCH FACE ANGLES		OVERALL SLOPE			BENCH SCALE SLOPE									
												ESTIMATED MAXIMUM[4] OVERALL SLOPE ANGLE FOR F.S. = 1.2 (°)		MAXIMUM BENCH FACE ANGLE (°)									
	ROCK TYPE	SLOPE DIP DIRECTION (°)	MAXIMUM SLOPE HEIGHT (ft)	TYPE OF FAILURE	DISCONTINUITY SETS INVOLVED	INTENSITY/[1] IMPORTANCE	APPARENT PLUNGE OR DIP OF FAILURE (°)	BASED ON APPARENT[2] PLUNGE OR DIP OF FAILURE CONSIDERED TO CONTROL BENCH STABILITY (°)	FROM PREVIOUS MINING (°)	FROM PRESENT MINING (°)	ESTIMATED SLOPE[3] HEIGHT OVER WHICH FAILURE COULD OCCUR (ft)	DRAINED	UNDRAINED	40 FOOT HIGH BENCH	60 FOOT HIGH BENCH	BENCH HEIGHT (ft)	BENCH[6] FACE ANGLE (°)	CREST BREAKBACK (ft)	EFFECTIVE BERM WIDTH (ft)	TOTAL BERM WIDTH (ft)	INTERMEDIATE[7] SLOPE ANGLE (°)		
1A-1	Footwall Andesite	331	340	Planes Wedges	JE FA, FD, JA, JB, JC	Weak/Moderate Strong	40 83	83	72	63	N/A	N/A		N/A	N/A	40 60 80	80	7 10.6 14.1	25 30 30	32 40.6 44.1	51 56 61	- See Fig. 8, Table V and Section 5 for further discussion and comparison of possible bench configurations	1A-1
1B-1	Footwall Andesite	317	260	Planes Wedges	JE FD, JC, JE	Weak/Moderate Moderate/Strong	69 65	65	71	-	N/A	N/A		N/A	N/A	40 60 80	70	14.5 21.8 29.1	25 30 30	39.5 51.8 59.1	45 49 53	- Structural analysis and kinematic assessments indicate controlling wedge failures are flatter than 70°, but slope documentation of existing benches indicates 70° bench faces should be attainable.	1B-1
1B-2	Footwall Andesite	006	300	Planes Wedges	JE FD, JA, JB, JC	Weak/Moderate Strong	67 70	70	-	-	N/A	N/A		N/A	N/A								1B-2
2B-1	Main Quartz Vein	033	340	Wedges	JA, JC	Strong	73	73	-	-	N/A	N/A		N/A	N/A	40 60 80	73	12.2 18.3 24.5	25 30 30	37.2 48.3 54.5	47 51 56		2B-1
3A-1	Light to Moderately Altered Andesite and Rhyolite	115	200	Planes	JF	Weak/Moderate	37	78	-	-	N/A	N/A	N/A	N/A	N/A	40 60 80	78	8.5 12.8 17.0	25 30 30	33.5 42.8 47.0	50 54 59		3A-1
3A-1'	Moderate to Heavily Altered Andesite and Rhyolite			Wedges	JA, JB, JC	Strong	78		72	64	100	>60	50	80	70	40 60	80 70	7 21.8	26.5 28.2	33.5 50	50 50	- It is anticipated that most of this rock, particularly the moderately to heavily altered portion, can be ripped. Thus, ripping trials as discussed in Section 5 should be implemented. - Undrained conditions may be encountered below about the 7160 elevation.	3A-1'
3A-2	Light to Moderately Altered Andesite and Rhyolite	149	200	Planes	JF	Weak/Moderate	33	82	-	-	N/A	N/A	N/A	N/A	N/A	40 60 80	80	7 10.6 14.1	25 30 30	32 40.6 44.1	51 56 61		3A-2
3A-2'	Moderate to Heavily Altered Andesite and Rhyolite			Wedges	JA, JB	Strong	82		72	64	100	>60	50	80	70	40 60	80 70	7 21.8	26.5 28.2	33.5 50	50 50		3A-2'
3B-1	Light to Moderately Altered Andesite and Rhyolite	048	320	Planes	JA	Strong	83	65	-	-	N/A	N/A	N/A	N/A	N/A	40 60 80	70	14.5 21.8 29.1	25 30 30	39.5 51.8 59.1	45 49 53		3B-1
3B-1'	Moderate to Heavily Altered Andesite and Rhyolite			Wedges	FA, JA, JB, JC	Strong	65		-	-	200	50	35	80	75	40 60	80 75	7 16	26.5 34	33.5 50	50 50	- It is anticipated that most of this rock, particularly the moderately to heavily altered portion, can be ripped. Thus, ripping trials as discussed in Section 5 should be implemented. - Drained conditions are anticipated throughout. - Structural analysis and kinematic assessments indicate that controlling wedge failures are flatter (i.e. 65°) in Design Sector 3B-1, but location and extent of Design Sector should allow additional breakback to 65° without compromising safety.	3B-1'
3B-2	Light to Moderately Altered Andesite and Rhyolite	111	240	Planes	JB	Strong	87	74	-	-	N/A	N/A	N/A	N/A	N/A	40 60 80	74	11.5 17.2 22.9	25 30 30	36.5 47.2 52.9	48 52 56		3B-2
3B-2'	Moderate to Heavily Altered Andesite and Rhyolite			Wedges	FA, JA, JB, JC, JD	Strong	74		-	-	120	60	50	80	75	40 60	80 75	7 16	25 30	32 46	51 52		3B-2'
4A-1	Unaltered Rhyolite	117	140	Planes Wedges	JF JA, JB, JC, JD	Weak/Moderate Strong	42 75	75	-	-	N/A	N/A		N/A	N/A	40 60 80	75	10.7 16 21.4	25 30 30	35.7 46 51.4	48 53 57	- Near surface rhyolite may be somewhat less competent due to surface weathering. Bench faces may break-back to about 70° in this rock.	4A-1
4A-2	Unaltered Rhyolite	149	120	Planes Wedges	JF JB, JD	Weak/Moderate Weak/Moderate	40 50	*	-	-	N/A	N/A		N/A	N/A	40 60 80	80	7 10.6 14.1	25 30 30	32 40.6 44.1	51 56 61		4A-2
4B-1	Unaltered Rhyolite	111	120	-	-	-	-	*	-	-	N/A	N/A		N/A	N/A								4B-1

NOTES:
1. Intensity/importance refers to the relative degree of development of individual joint sets in a given failure mode and the estimated likli-hood of excavated bench faces breaking back to or forming along this failure mode.
2. "*" indicates no strong kinematic control is present. Maximum effective bench face angles of 80° are assumed.
3. Estimated slope height over which rock is heavily altered.
4. Overall slope angles that are circled are anticipated to be critical for design.
5. See Fig. 6 for definitions of bench geometry parameters.
6. Based on previous mining, minimum bench face angle is assumed to be 70o.
7. Intermediate slope angle is overall slope angle between haulroads.
8. Control blasting or other controlled method of excavation is recommended in all areas of the final slope to ensure minimal disturbance to the rock. The exact type of control blasting should be determined by field trials.
9. Remedial measures are recommended to control surface water. This should consist of graded berms and drainage ditches.

TABLE IV

TABLE VI

SUMMARY OF ROCK MASS CLASSIFICATION [1]

AND PRELIMINARY RIPPABILITY ASSESSMENTS

ROCK TYPE	RELEVANT DESIGN SECTORS	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (psi)	JOINT SPACING (ft)	DIGGABILITY INDEX AFTER SCOBLE AND MUFTUOGLU (1984) [2]		CLASSIFICATION SYSTEM AFTER KIRSTEN (1982) [3]			RIPPABILITY RATING AFTER WEAVER (1975) [4]			CLASSIFICATION AFTER FRANKLIN ET AL (1981) [5]	PRELIMINARY ASSESSMENT OF ROCK MASS [6]
				CLASSIFICATION INDEX	DESCRIPTION OF EXCAVATION EQUIPMENT	INDEX	DESCRIPTION	EXCAVATION EQUIPMENT	RATING	DESCRIPTION	EXCAVATION EQUIPMENT		
Unaltered Andesite	1A-1 1B-1 1B-2	30,000	1.5 - 3	75 - 100	Blast	14,000 to 18,750	Blast	Blast	90	Blasting	Blast	Blast to Fracture	Blast to Fracture
Quartz Vein	2B-1	>50,000	2	75 - 100	Blast	14,000 to 18,750	Blast	Blast	90	Blasting	Blast	Blast to Fracture	Blast to Fracture
Propylitized Andesite/ Bleached Andesite (Lightly Altered Andesite)	3A-1 3A-2 3B-1 3B-2	15,000	1 - 2	65 - 70	D10	1,800 to 9,400	Extremely Hard Ripping/Blasting	D10 or Blast	70 - 75	Extremely Hard Ripping and Blasting	D9G or Blasting	Blast to Loosen or Blast to Fracture	D10 or Blast to Loosen or Fracture
Moderately Clay Altered Andesite		3,500	<.15	10 - 35	D8	100 to 600	Very Hard Ripping	D9H	27 - 48	Hard Ripping	D8 or D7	Blast to Loosen or Rip	D8 or D9
Heavily Clay Altered Andesite and Rhyolite	3A-1' 3A-2' 3B-1' 3B-2'	<1,000	<.15	0 - 10	D8	2 - 10	Easy Ripping	D7G	15 - 29	Easy to Hard Ripping	D7 or D8	Rip or Dig	D7 or D8
Lightly Altered Rhyolite	4A-1 4A-2 4B-1	11,000	0.5 - 2	45 - 65	D9 or D10	780 to 1,800	Very Hard Ripping to Extremely Hard Ripping/Blasting	D9H to D10 or Blast	64 - 86	Very Hard Ripping or Extremely Hard Ripping and Blasting	D9, D9G or Blasting	Blast to Loosen	D9, D10 or Blast to Loosen
Rhyolite/Silicified Rhyolite		24,000	1 - 3	75 - 100	Blast	3,500 to 9,400	Extremely Hard Ripping/Blasting	D10 or Blast	85 - 90	Blasting and Extremely Hard Ripping	Blasting or D9G	Blast to Fracture	Blast to Fracture

NOTES:
1. All parameters and values required for the various classification systems were estimated based on the field investigation and analysis of geological mapping and core logging data provided.

2. The Diggability Index of Scoble and Muftuoglu (1984) applies a rating to four parameters: Weathering (W), Strength (S), Joint Spacing (J) and Bedding Spacing (B). The index is determined from the sum of these parameters and has been related to excavation equipment used in British coal mines.

3. The system developed by Kirsten (1982) determines an excavatibility index based on a modification of the rock mass classification system of Barton et al (1974) which assigns values for strength (Ms), RQD, joint set number (Jn), the structural orientations (Js), joint roughness (Jr) and joint alteration (Ja). The excavatibility index is defined as:

$$N = Ms\ \frac{RQD}{Jn}\ .\ Js\ .\ \frac{Jr}{Ja}$$

4. The rippability rating of Weaver (1975) assigns values to seismic velocity, rock hardness, rock weathering, joint spacing, joint continuity, joint gouge and joint orientation. The total rating is the sum of the values for these parameters and is used to select equipment.

5. Classification systems of Franklin (1971) relate joint spacing and unconfined compressive strength to excavation technique.

6. The preliminary assessment of rock rippability is based on results of all classification systems, experience and engineering judgment. Field ripping trials are recommended to further refine the assessments presented above.

TABLE VII

BLAST DOCUMENTATION FOR BLASTS IN QUARTZ[1] AND HANGING WALL ROCKS ON 7120 FOOT LEVEL

BLAST	APPROXIMATE[2] OVERALL WIDTH AND DEPTH (FT.)	ESTIMATED[2] WEIGHT OF ROCK BLASTED (TONS)	APPROXIMATE[2] BLASTHOLE PATTERN (BURDEN AND SPACING) (FT.)	NUMBER OF[2] DELAYS	EXPLOSIVE[3] USED	ESTIMATED AVERAGE LBS/DELAY	ESTIMATED POWDER FACTOR (LBS/TON)	FIRING SEQUENCE	COMMENTS[4]
A	70 x 30	3,500	8 x 12	3	Anfo	350	0.3	Row-by-Row	- Most of blast in compact soil. - Blast choked. - Easy digging.
B	60 x 135	13,500	10 x 10	15	Deta Gel	600	0.6	Row-by-Row	- Holes in quartz full of water. - Blast choked. - Rock broke well and muck heaved about 5 feet. - Muck pile tight, resulting in difficult digging.
C	50 x 120	10,000	10 x 10	12	Anfo	825	1.0	Row-by-Row	- Much of blast in soil. - Blast choked. - Overblasted as evidenced by considerable cratering.
D	120 x 50	10,000	10 x 10	8	Anfo and Deta Gel	825	0.5	Row-by-Row	- Quartz loaded with Deta Gel and hanging wall rocks loaded with Anfo. - Blast choked. - Quartz muck heaved about 5 feet and altered andesite muck heaved about 10 feet. - Rock broke well with no over-break into footwall.
E	100 x 100	16,500	10 x 10	13	Anfo and Deta Gel	750	0.5	Row-by-Row	- Quartz loaded with Deta Gel and hanging wall rocks loaded with Anfo. - Blasted to free face. - Quartz muck heaved about 5 feet and altered andesite muck heaved about 10 feet. - Rock broke well, resulting in easy digging and clean, even floor.
F	100 x 150	25,000	10 x 10	15	Anfo and Deta Gel	825 ?	0.5 ?	Row-by-Row	- Quartz loaded with Deta Gel and hanging wall rocks loaded with Anfo. - Blast choked. - Quartz muck heaved about 5 feet and altered andesite muck heaved about 10 feet. - Rock did not break as well as in Blast E, resulting in more oversize and difficult digging.

NOTES:
1. Information provided by Minerex Resources Ltd.
2. Based on drawing entitled "Blast Hole Plan - U Bench 7120 - Delay Pattern", dated December 16, 1987.
3. Anfo used in dry holes. Data gel used in wet holes.
4. Comments provided by A. Glatiotis, chief geologist.

APPENDIX A

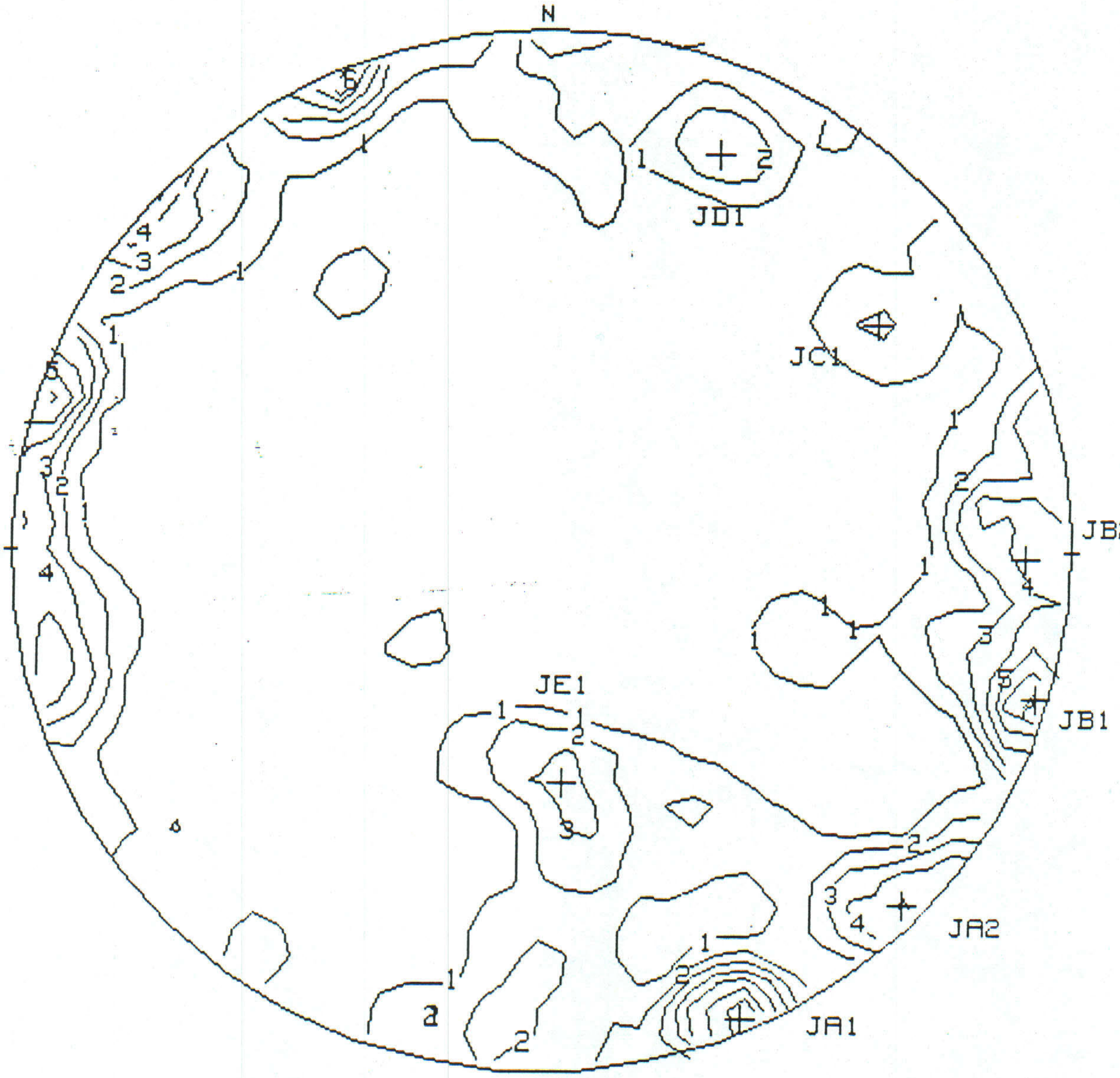
APPENDIX A

LOWER HEMISPHERE EQUAL AREA

PROJECTIONS OF DISCONTINUITIES

OBSERVATIONS: 195
POPULATION: 320

CONTOUR PLOT



LABEL	DIP DIRECTION	DIP	CONCENTRATION (%)
JA1	337	87	8
JA2	315	86	5
JB1	287	87	7
JB2	271	80	5
JC1	235	65	2
JD1	203	72	3
JE1	355	37	4

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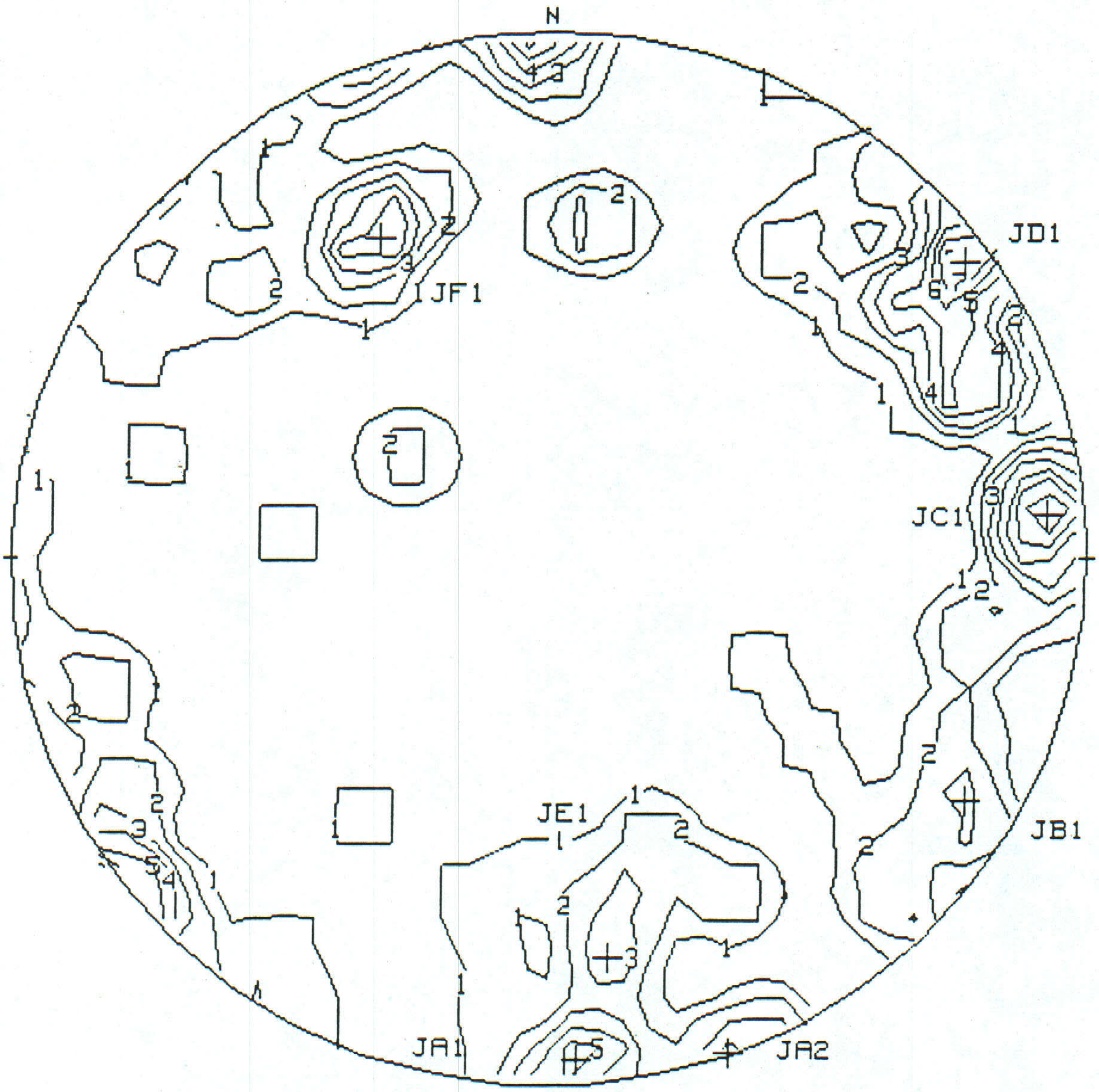
GEODAT: LOWER HEMISPHERE EQUAL AREA PROJECTION

ALL JOINTS IN LOWER FOOTWALL
(STRUCTURAL DOMAIN 1A)

By: AFS
Date: 12/87
Page: A-1
Job#:

OBSERVATIONS: 76
POPULATION: 96

CONTOUR PLOT

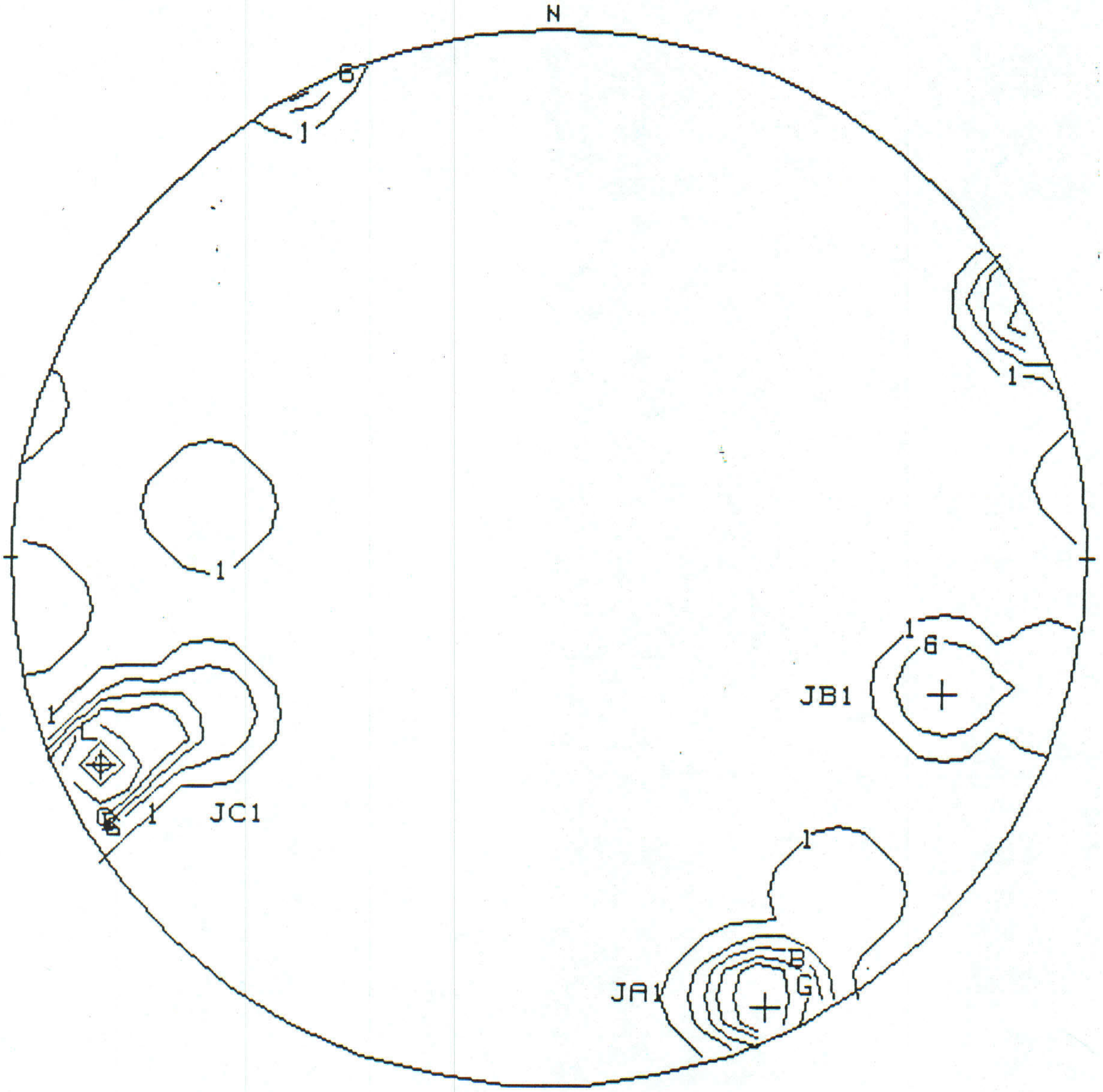


LABEL	DIP DIRECTION	DIP	CONCENTRATION (%)
JA1	357	85	6
JA2	340	90	4
JB1	301	79	3
JC1	265	82	8
JD1	234	85	8
JE1	351	66	4
JF1	152	58	5

MINEREX RESOURCES LTD. AURORA PROJECT GEOTECHNICAL ASSESSMENT	PITEAU ASSOCIATES ENGINEERING LTD.		
GEODAT: LOWER HEMISPHERE EQUAL AREA PROJECTION ALL JOINTS IN UPPER FOOTWALL (STRUCTURAL DOMAIN 1B)		By: AFS	Date: 12/87
		Page: A-2	Job#:

OBSERVATIONS: 13
POPULATION: 20

CONTOUR PLOT

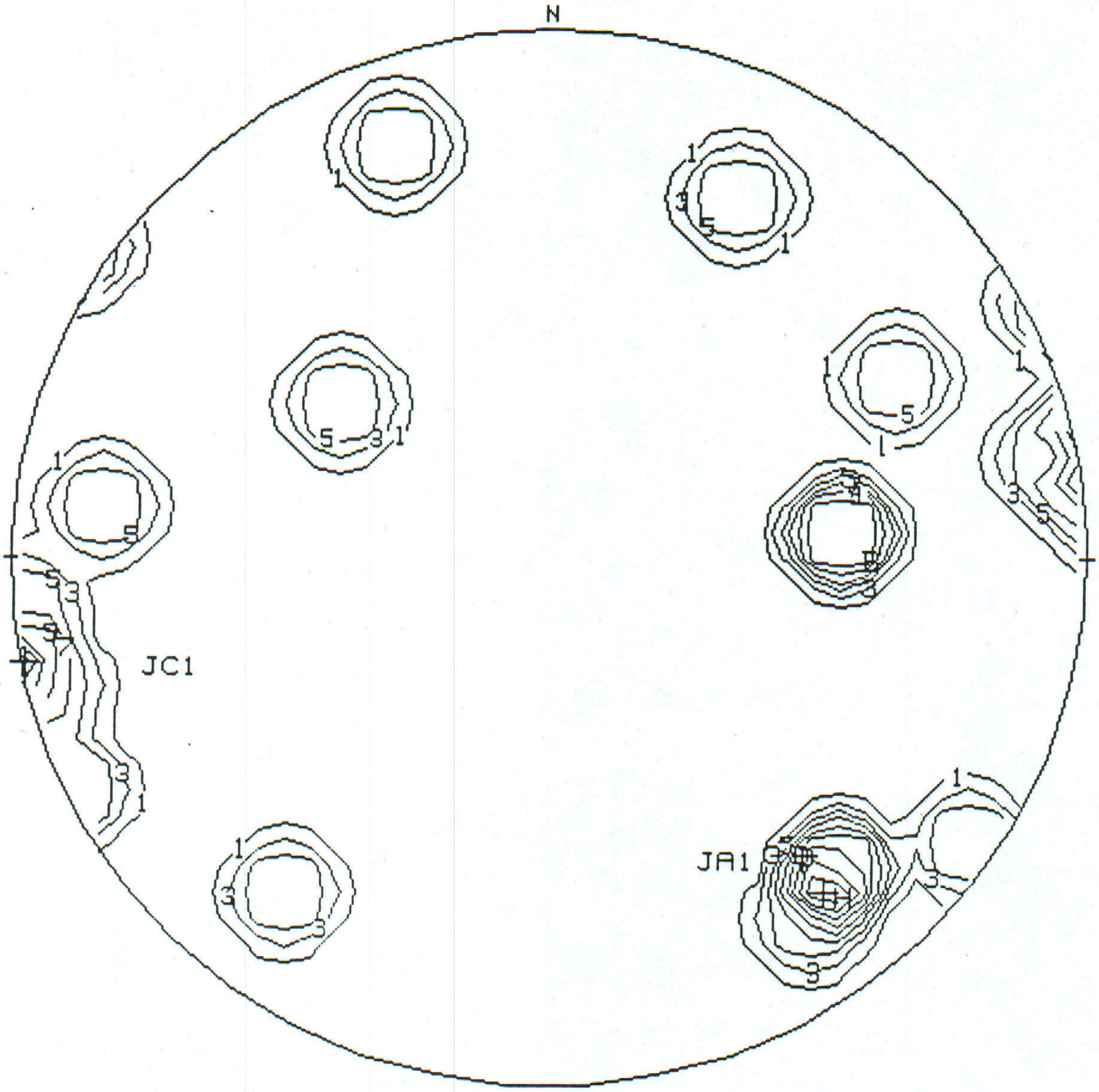


LABEL	DIP DIRECTION	DIP	CONCENTRATION (%)
JA1	334	83	25
JB1	289	66	10
JC1	065	81	35

MINEREX RESOURCES LTD. AURORA PROJECT GEOTECHNICAL ASSESSMENT	PITEAU ASSOCIATES ENGINEERING LTD.		
GEODAT: LOWER HEMISPHERE EQUAL AREA PROJECTION		By: AFS	Date: 12/87
ALL JOINTS IN MAIN QUARTZ VEIN IN LOWER PIT (STRUCTURAL DOMAIN 2A)		Page: A-3	Job#:

OBSERVATIONS: 15
POPULATION: 16

CONTOUR PLOT



LABEL	DIP DIRECTION	DIP	CONCENTRATION (%)
JA1	321	70	19
JC1	078	89	16

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GEODAT: LOWER HEMISPHERE EQUAL AREA PROJECTION

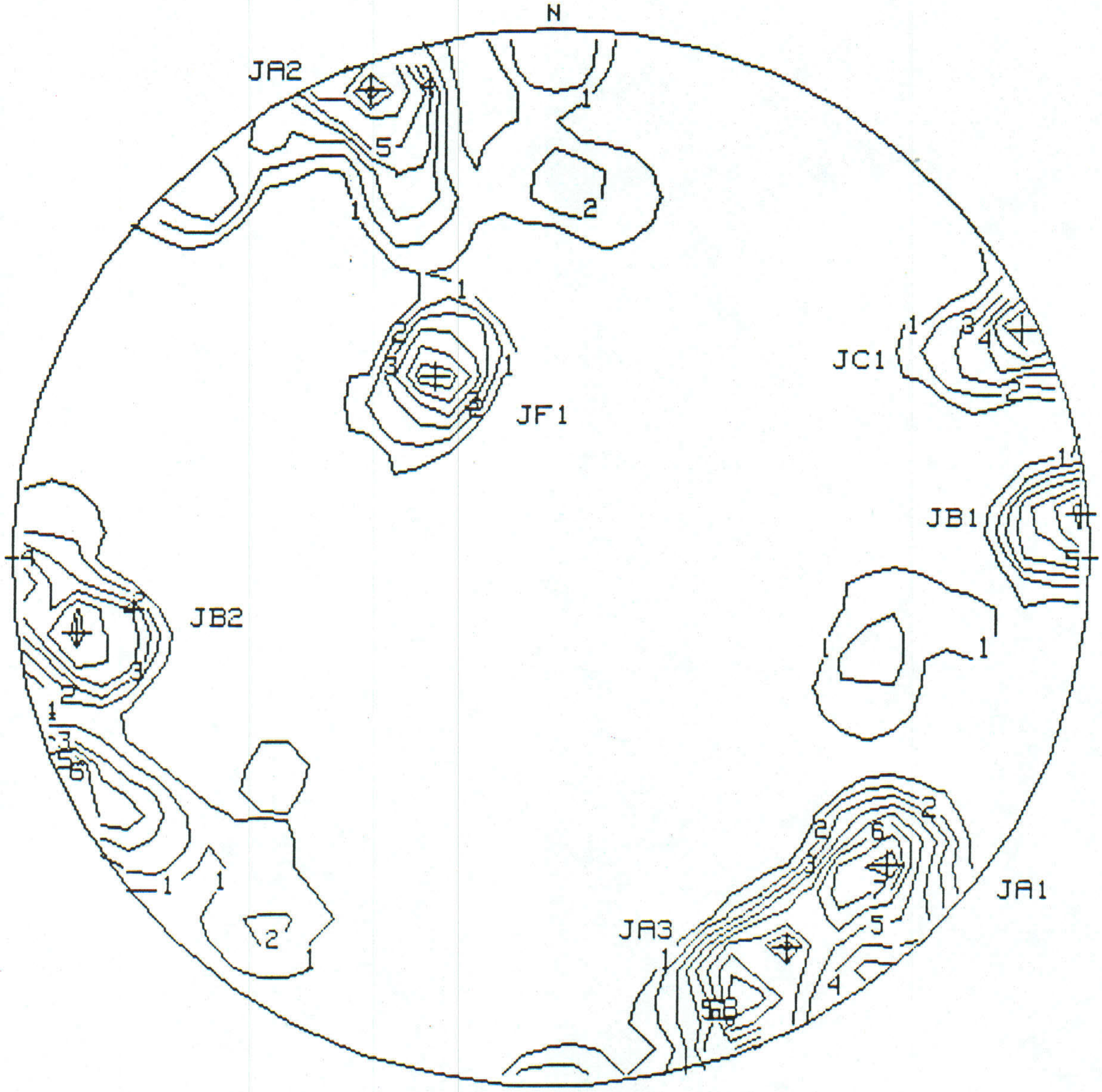
ALL JOINTS IN MAIN QUARTZ VEIN IN UPPER PIT

(STRUCTURAL DOMAIN 2B)

By:	Date:
AFS	12/87
Page:	Job#:
A-4	

OBSERVATIONS: 63
POPULATION: 105

CONTOUR PLOT



LABEL	DIP DIRECTION	DIP	CONCENTRATION (%)
JA1	313	74	9
JA2	159	84	9
JA3	329	75	8
JB1	265	88	7
JB2	081	78	6
JC1	244	86	7
JF1	147	33	7

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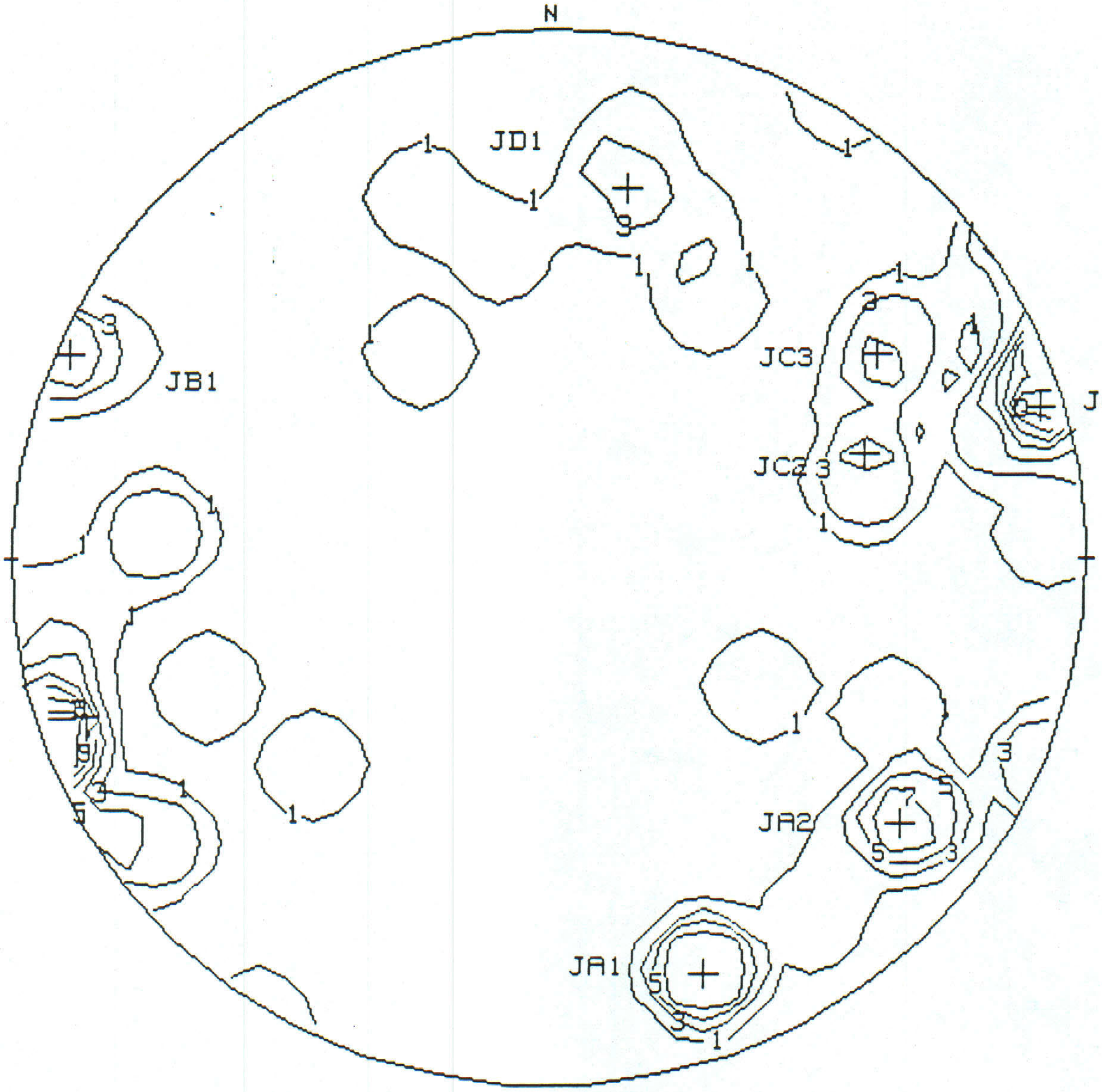
GEODAT: LOWER HEMISPHERE EQUAL AREA PROJECTION

ALL JOINTS IN ALTERED AND.& RHY. IN LOWER H/WALL
(STRUCTURAL DOMAIN 3A)

By: AFS
Date: 12/87
Page: A-5
Job#:

OBSERVATIONS: 32
POPULATION: 45

CONTOUR PLOT

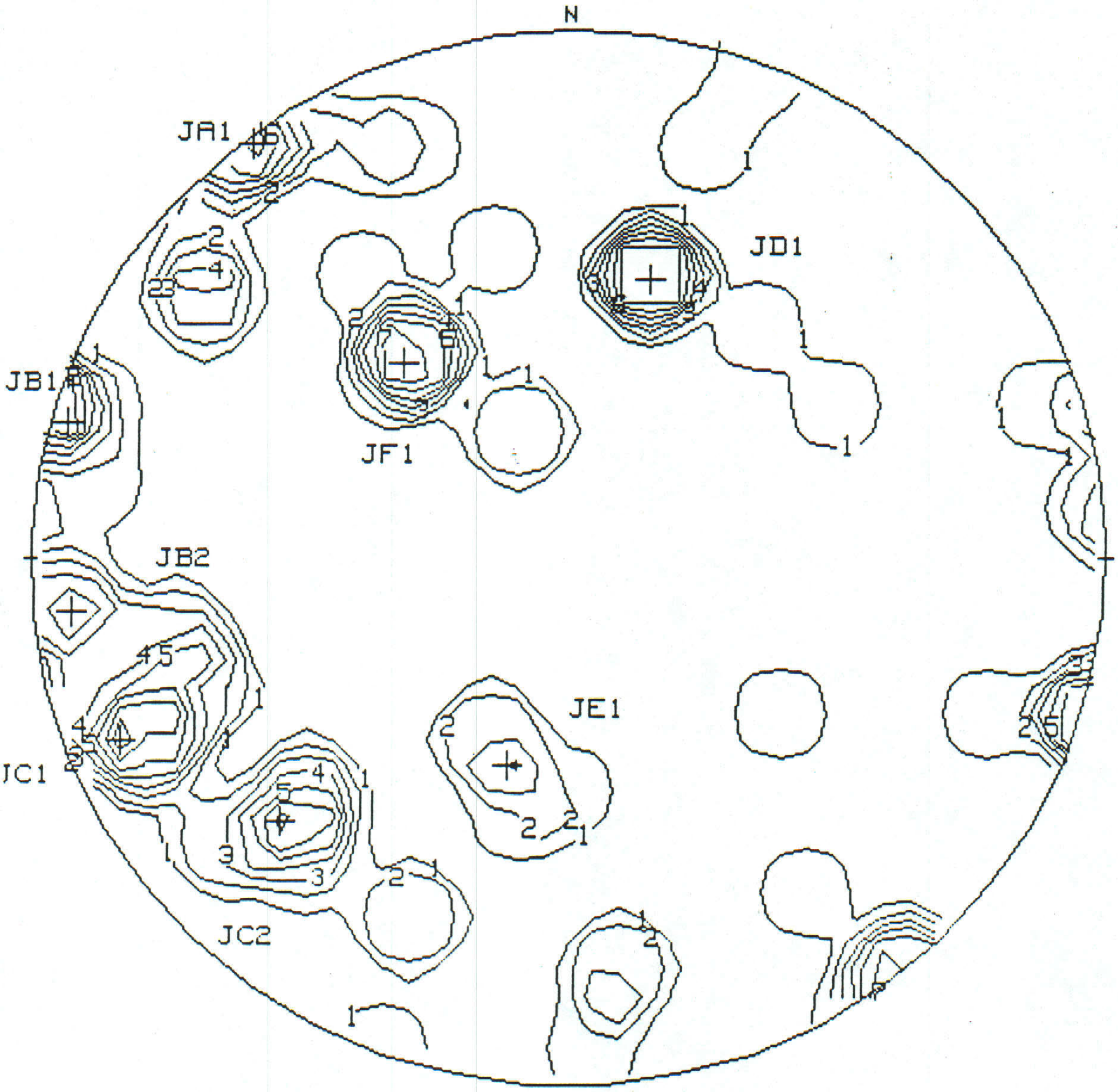


LABEL	DIP DIRECTION	DIP	CONCENTRATION (%)
JA1	340	73	9
JA2	308	71	9
JC1	253	85	15
JC2	251	51	6
JC3	237	61	6
JB1	113	87	7
JD1	191	61	4

MINEREX RESOURCES LTD. AURORA PROJECT GEOTECHNICAL ASSESSMENT	PITEAU ASSOCIATES ENGINEERING LTD.		
GEODAT: LOWER HEMISPHERE EQUAL AREA PROJECTION		By: AFS	Date: 12/87
ALL JOINTS IN ALTERED AND.& RHY. IN UPPER H/WALL (STRUCTURAL DOMAIN 3B)		Page: A-6	Job#:

OBSERVATIONS: 45
POPULATION: 68

CONTOUR PLOT



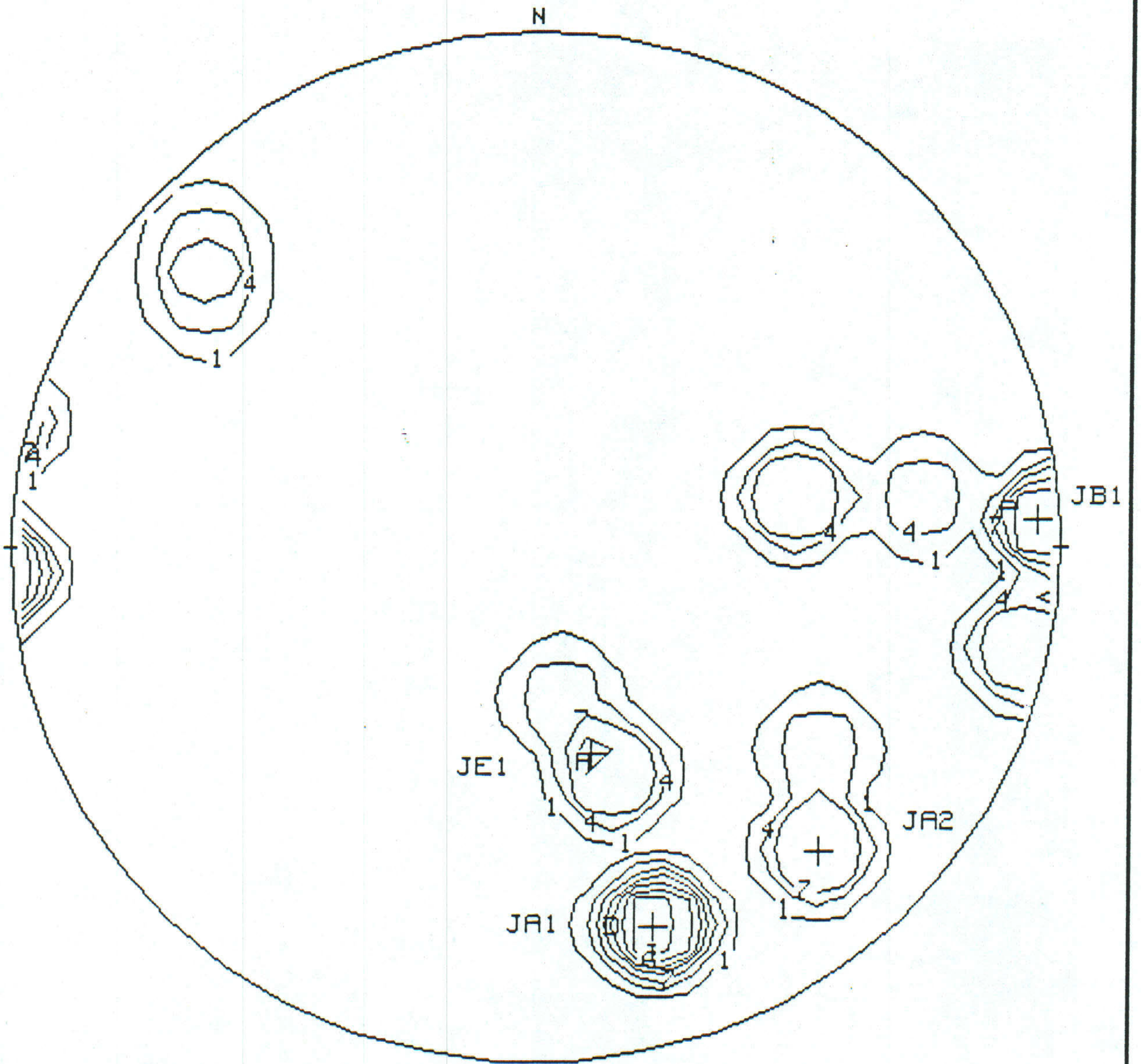
LABEL	DIP DIRECTION	DIP	CONCENTRATION (%)
JA1	143	88	7
JB1	106	86	8
JB2	084	82	6
JC1	068	79	9
JC2	047	62	7
JD1	195	46	7
JE1	016	34	4
JF1	140	40	7

MINEREX RESOURCES LTD. **AURORA PROJECT** **GEOTECHNICAL ASSESSMENT**	**PITEAU ASSOCIATES ENGINEERING LTD.**

GEODAT: LOWER HEMISPHERE EQUAL AREA PROJECTION	By: AFS	Date: 12/87
ALL JOINTS IN RHYOLITE IN LOWER HANGING WALL (STRUCTURAL DOMAIN 4A)	Page: A-7	Job#:

OBSERVATIONS: 13
POPULATION: 20

CONTOUR PLOT



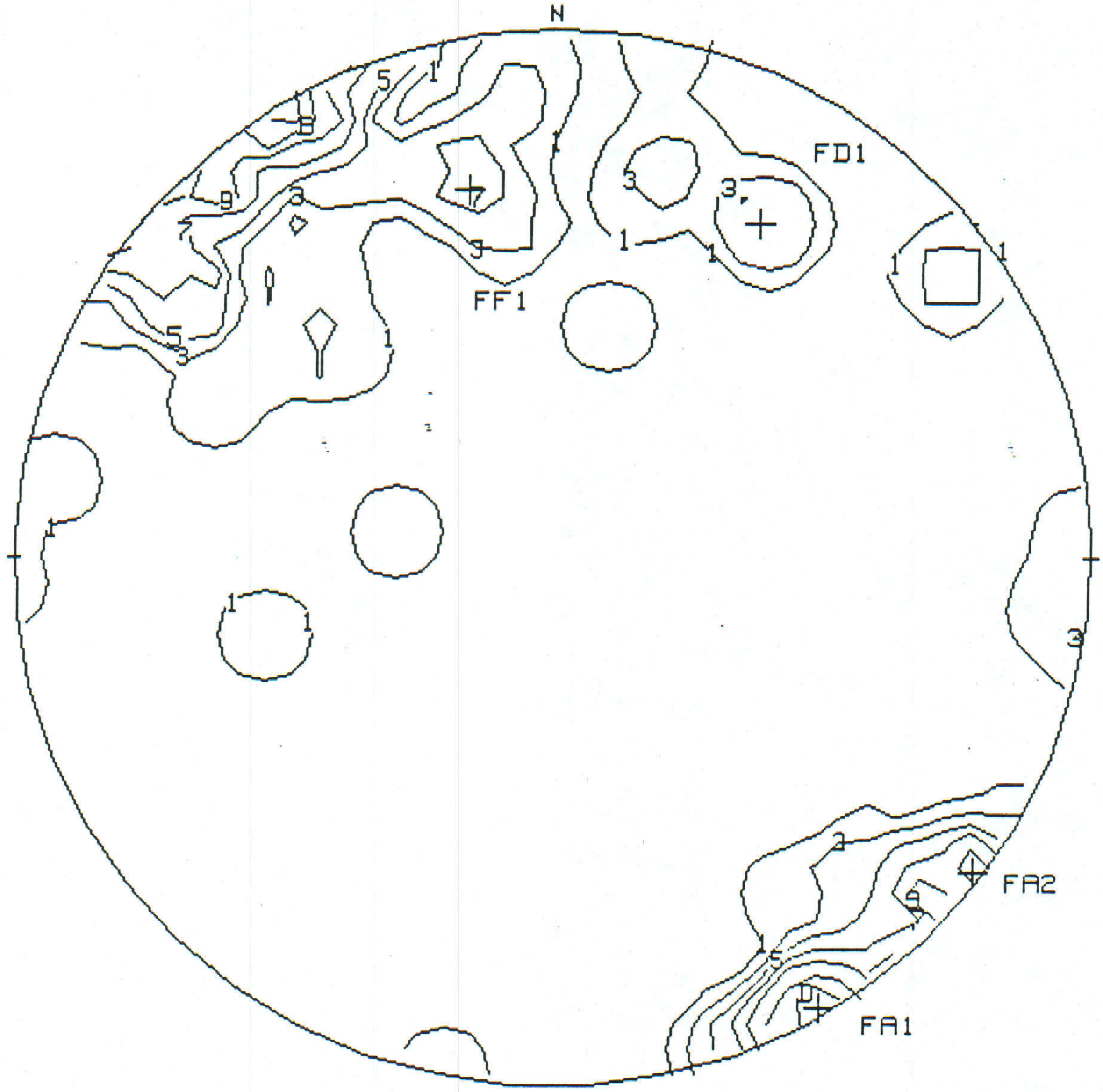
LABEL	DIP DIRECTION	DIP	CONCENTRATION (%)
JA1	343	66	20
JA2	318	69	10
JB1	267	85	15
JE1	345	34	12

MINEREX RESOURCES LTD. AURORA PROJECT GEOTECHNICAL ASSESSMENT	PITEAU ASSOCIATES ENGINEERING LTD.

GEODAT: LOWER HEMISPHERE EQUAL AREA PROJECTION ALL JOINTS IN RHYOLITE IN UPPER HANGING WALL (STRUCTURAL DOMAIN 4B)	By: AFS Page: A-8	Date: 12/87 Job#:

OBSERVATIONS: 45
POPULATION: 65

CONTOUR PLOT



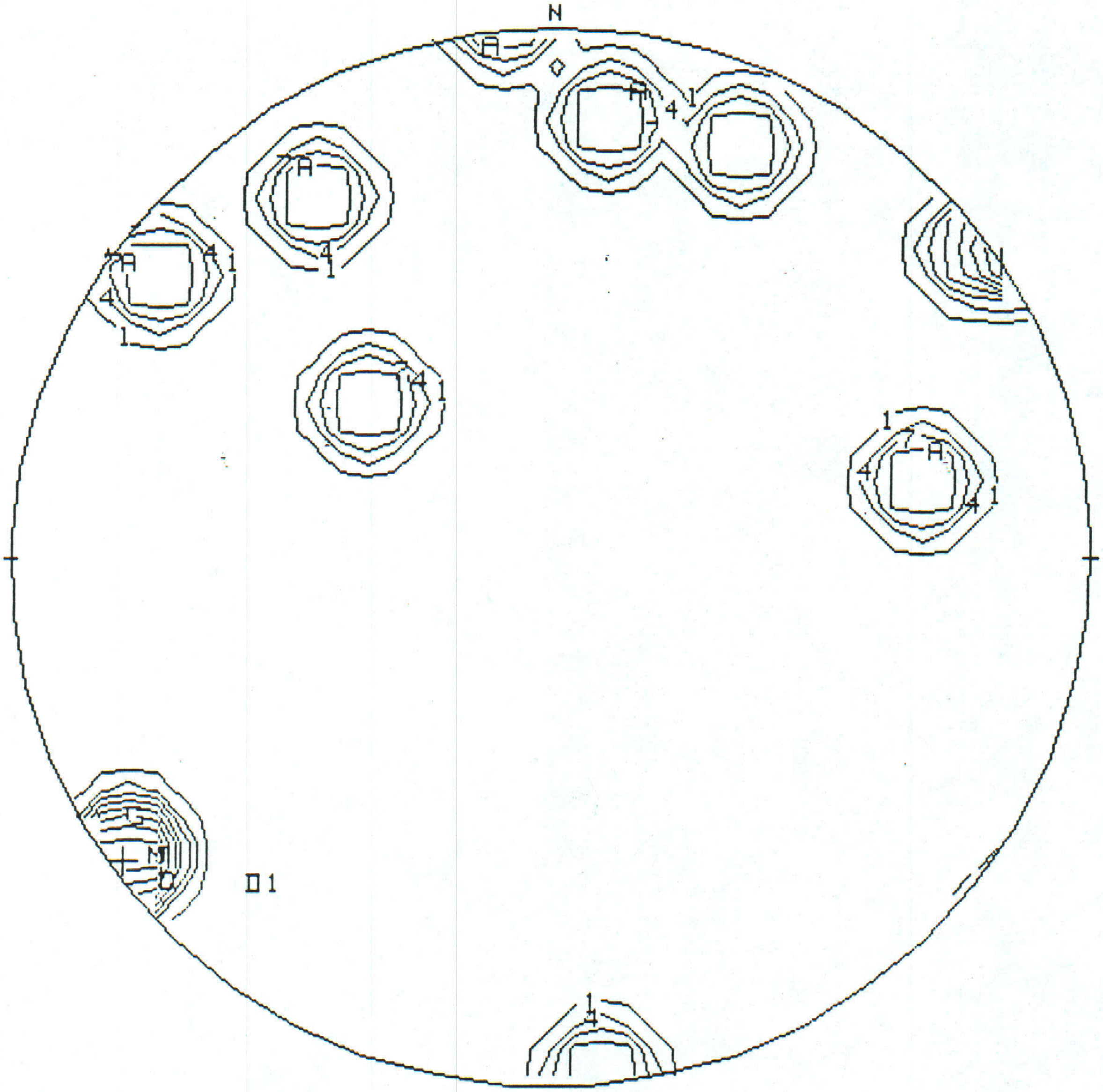
LABEL	DIP DIRECTION	DIP	CONCENTRATION (%)
FA1	330	88	15
FA2	307	88	10
FD1	211	63	5
FF1	167	61	7

MINEREX RESOURCES LTD. AURORA PROJECT GEOTECHNICAL ASSESSMENT	PITEAU ASSOCIATES ENGINEERING LTD.

GEODAT: LOWER HEMISPHERE EQUAL AREA PROJECTION	By: AFS	Date: 12/87
ALL FAULTS & VEINS IN LOWER FOOTWALL (STRUCTURAL DOMAIN 1A)	Page: A-9	Job#:

OBSERVATIONS: 9
POPULATION: 9

CONTOUR PLOT



LABEL	DIP DIRECTION	DIP	CONCENTRATION (%)
D1	054	87	22

MINEREX RESOURCES LTD.
AURORA PROJECT
GEOTECHNICAL ASSESSMENT

PITEAU ASSOCIATES ENGINEERING LTD.

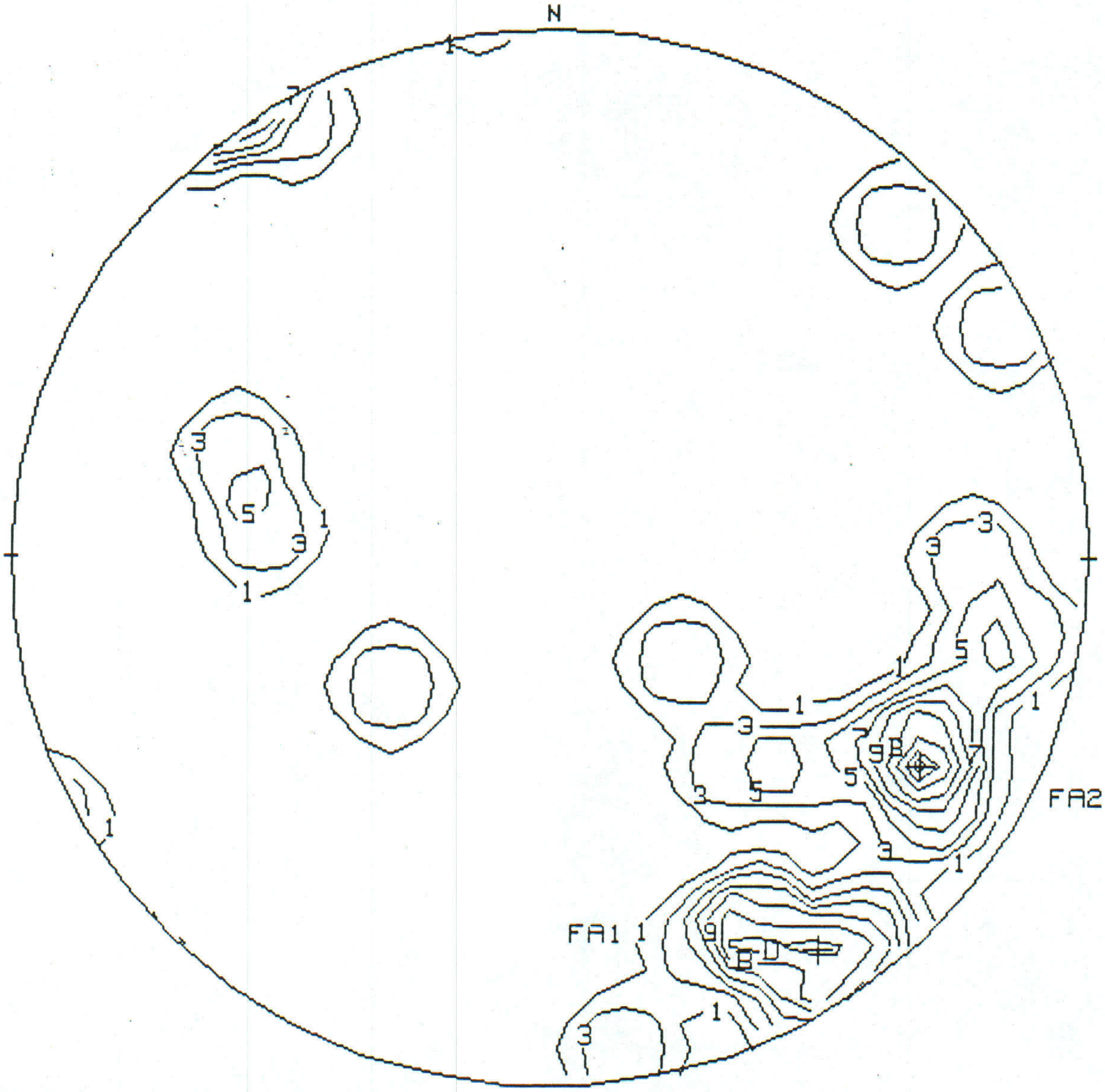
GEODAT: LOWER HEMISPHERE EQUAL AREA PROJECTION

ALL FAULTS & VEINS IN UPPER FOOTWALL
(STRUCTURAL DOMAIN 1B)

By:	Date:
AFS	12/87
Page:	Job#:
A-10	

OBSERVATIONS: 25
POPULATION: 25

CONTOUR PLOT



LABEL	DIP DIRECTION	DIP	CONCENTRATION (%)
FA1	326	78	14
FA2	300	68	18

MINEREX RESOURCES LTD.
AURORA PROJECT
GEOTECHNICAL ASSESSMENT

PITEAU ASSOCIATES ENGINEERING LTD.

GEODAT: LOWER HEMISPHERE EQUAL AREA PROJECTION

ALL FAULTS & VEINS IN LOWER HANGING WALL
(STRUCTURAL DOMAINS 3A and 4A)

By: AFS

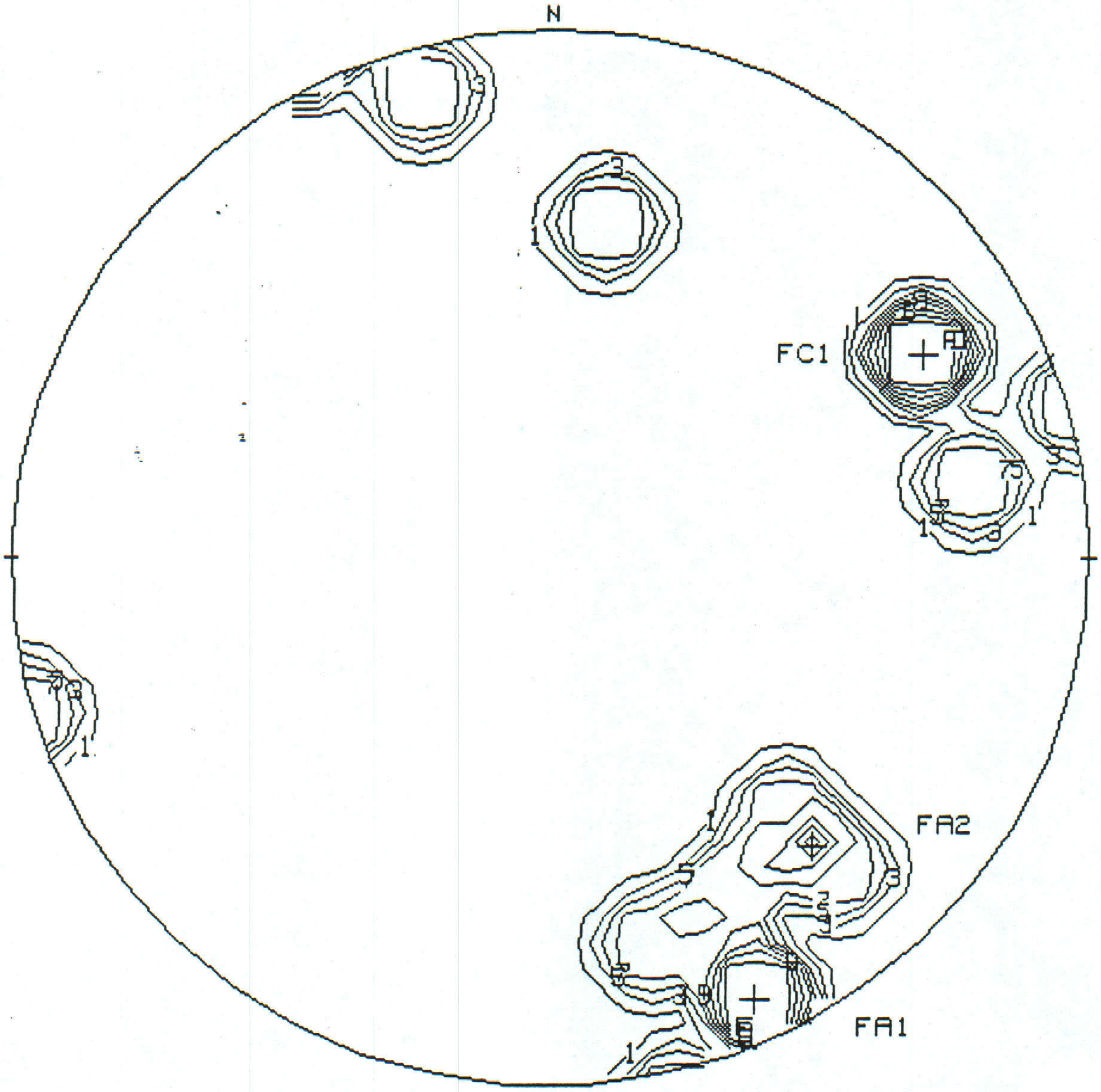
Date: 12/87

Page: A-11

Job#:

OBSERVATIONS: 11
POPULATION: 12

CONTOUR PLOT



LABEL	DIP DIRECTION	DIP	CONCENTRATION (%)
FA1	335	81	17
FA2	318	62	17
FC1	241	68	17

MINEREX RESOURCES LTD. AURORA PROJECT GEOTECHNICAL ASSESSMENT	PITEAU ASSOCIATES ENGINEERING LTD.		
GEODAT: LOWER HEMISPHERE EQUAL AREA PROJECTION		By: AFS	Date: 12/87
ALL FAULTS & VEINS IN UPPER HANGING WALL (STRUCTURAL DOMAINS 3B and 4B)		Page: A-12	Job#:

APPENDIX B

APPENDIX B

RELATIONSHIP BETWEEN HARDNESS AND UNCONFINED COMPRESSIVE STRENGTH

QUALITATIVE & QUANTITATIVE EXPRESSIONS

FOR CONSISTENCY OF COHESIVE SOIL AND ROCK*

HARDNESS	CONSISTENCY	FIELD IDENTIFICATION	APPROXIMATE RANGE OF UNCONFINED COMPRESSIVE STRENGTH	
			MPa	p.s.i.
S1	very soft soil	Easily penetrated several inches by fist; shows distinct heel marks.	<0.025	<3.5
S2	soft soil	Easily penetrated several inches by thumb; faint heel marks.	0.025 - 0.05	3.5 - 7
S3	firm soil	Can be penetrated by thumb with moderate effort; difficult to cut with hand spade.	0.05 - 0.10	7 - 14
S4	stiff soil	Readily indented by thumb but penetrated only with great effort; cannot be cut with hand spade.	0.1 - 0.2	14 - 28
S5	very stiff soil	Readily indented by thumbnail; requires pneumatic spade for excavation.	0.20 - 0.4	28 - 56
S6	hard soil	Indented with difficulty by thumbnail.	>0.4	>56
R0	extremely soft rock	Indented by thumbnail.	0.2 - 0.7	28 - 100
R1	very soft rock	Crumbles under firm blows with point of geological pick; can be peeled by a pocket knife.	0.7 - 7.0	100 - 1,000
R2	soft rock	Can be peeled by a pocket knife with difficulty; shallow indentations made by firm blow of geological pick.	7.0 - 28	1,000 - 4,000
R3	average rock	Cannot be scraped or peeled with a pocket knife; specimen can be fractured with single firm blow of hammer end of geological pick.	28 - 56	4,000 - 8,000
R4	hard rock	Specimen requires more than one blow with hammer end of geological pick to fracture it.	56 - 112	8,000 - 16,000
R5	very hard rock	Specimen requires many blows of hammer end of geological pick to fracture it.	112 - 224	16,000 - 32,000
R6	extremely hard rock	Specimen can only be chipped with geological pick.	>224	>32,000

* Modified Rock Hardness Classification

S1 to S6 Modified after Terzaghi, K. and Peck, R.B., 1967. "Soil Mechanics in Engineering Practice, 2nd Edition, John Wiley and Sons Inc., New York. p.30.

R1 to R5 Modified after Piteau, D.R., 1970. "Geological Factors Significant to the Stability of Slopes Cut in Rock" in Planning Open Pit Mines, Van Rensburg Ed. Aug. 29-Sept. 4, 1970. Balkema. p.51 and 68.

APPENDIX C

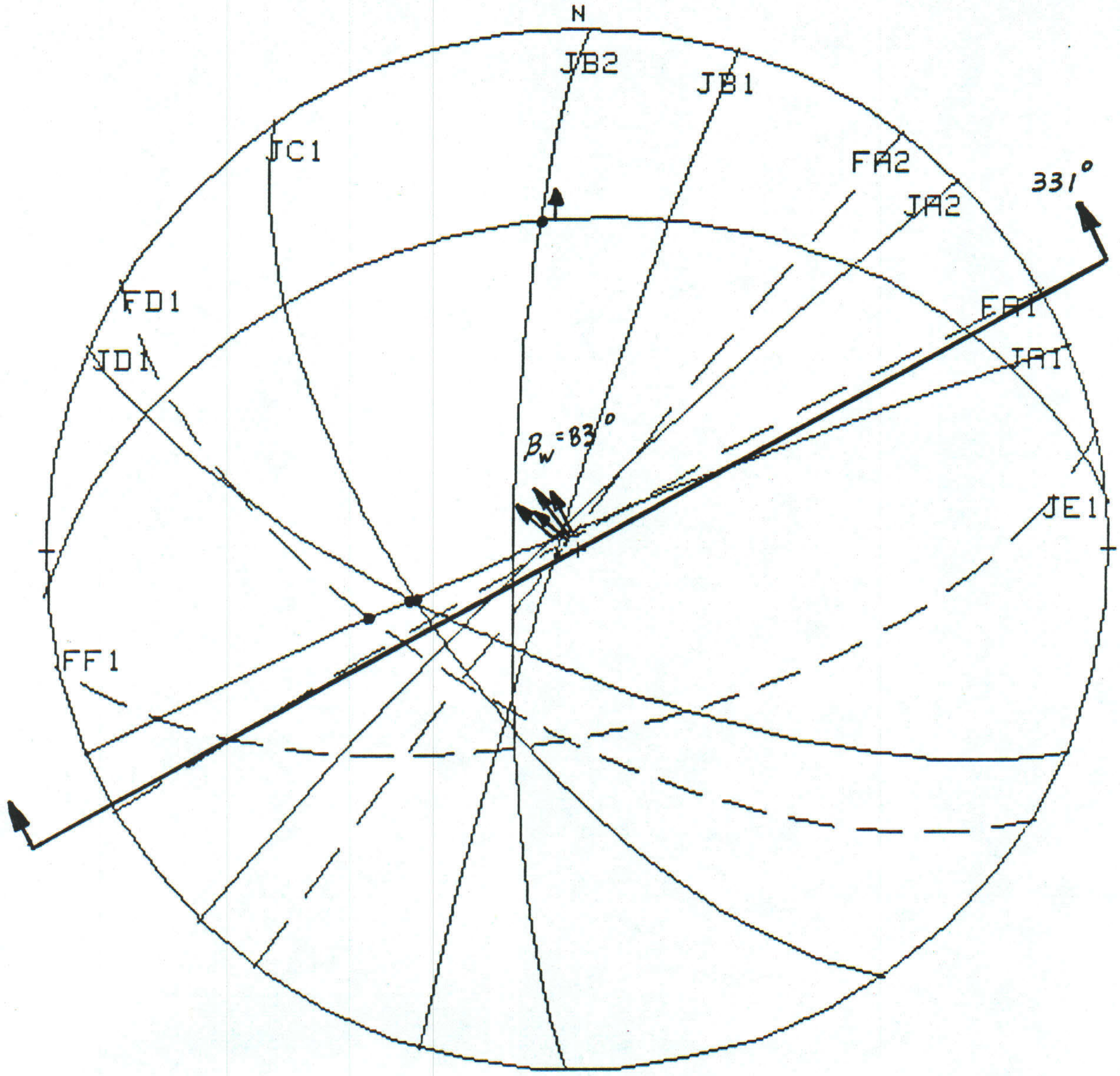
APPENDIX C

LOWER HEMISPHERE EQUAL AREA PROJECTIONS
OF PLANES FOR KINEMATIC ASSESSMENT
OF POSSIBLE MODES OF FAILURE

JOB NUMBER 87-975

GEOPLN - LOWER HEMISPHERE EQUAL AREA PROJECTION OF PLANES

PROJECT: AURORA PROJECT
DATE: 1,12,87
STRUCTURAL DOMAIN: 1A



KEY

033° PIT SLOPE ORIENTATION & DIP DIRECTION

PEAK ORIENTATION OF FAULT SET

PEAK ORIENTATION OF JOINT SET

POTENTIAL PLANE FAILURE

POTENTIAL WEDGE FAILURE

β_w:83° APPARENT PLUNGE OR DIP OF FAILURE CONSIDERED TO CONTROL BENCH STABILITY

MINEREX RESOURCES LTD.
AURORA PROJECT
GEOTECHNICAL ASSESSMENT

PITEAU ASSOCIATES
GEOTECHNICAL CONSULTANTS
VANCOUVER CALGARY

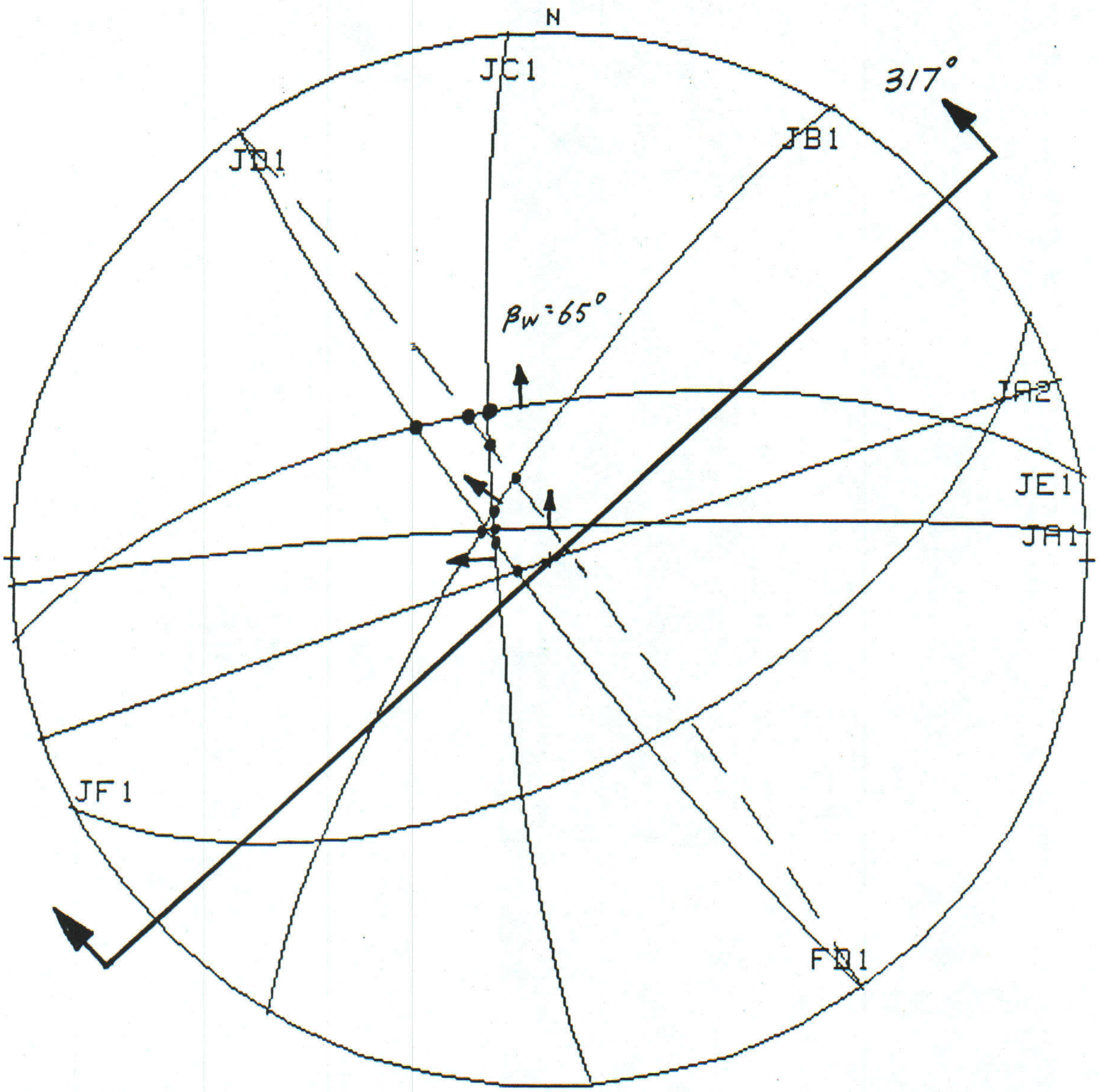
LOWER HEMISPHERE EQUAL AREA PROJECTIONS OF PLANES REPRESENTING PEAK ORIENTATIONS OF DISCONTINUITY SETS IN DESIGN SECTOR 1A-1

BY:	DATE:
AFS	12/87
APPROVED:	DWG:
[signature]	C-1

JOB NUMBER 87-975

GEOPLN - LOWER HEMISPHERE EQUAL AREA PROJECTION OF PLANES

PROJECT: AURORA PROJECT
DATE: 1,12,87
STRUCTURAL DOMAIN: 1B



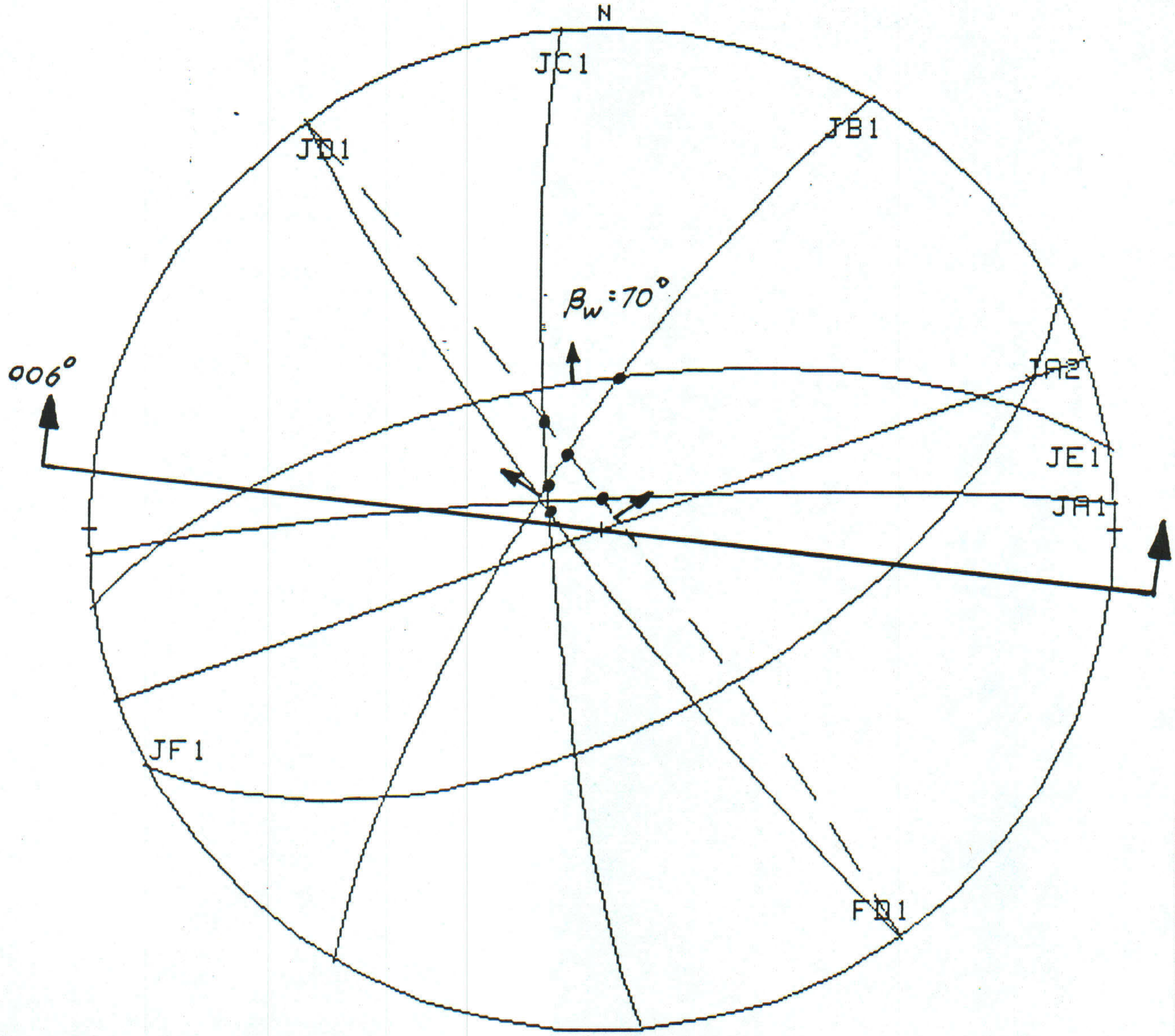
NOTE: SEE FIG. C-1 FOR KEY

MINEREX RESOURCES LTD. AURORA PROJECT GEOTECHNICAL ASSESSMENT	PITEAU ASSOCIATES GEOTECHNICAL CONSULTANTS VANCOUVER CALGARY		
LOWER HEMISPHERE EQUAL AREA PROJECTIONS OF PLANES REPRESENTING PEAK ORIENTATIONS OF DISCONTINUITY SETS IN DESIGN SECTOR 1B-1	BY: AFS	DATE: 12/87	
	APPROVED: a.f.	DWG: C-2	

GEOPLN - LOWER HEMISPHERE EQUAL AREA PROJECTION OF PLANES

PROJECT: AURORA PROJECT
DATE: 1,12,87
STRUCTURAL DOMAIN: 1B

JOB NUMBER 87-975



NOTE: SEE FIG. C-1 FOR KEY

MINEREX RESOURCES LTD.
AURORA PROJECT
GEOTECHNICAL ASSESSMENT

PITEAU ASSOCIATES
GEOTECHNICAL CONSULTANTS
VANCOUVER CALGARY

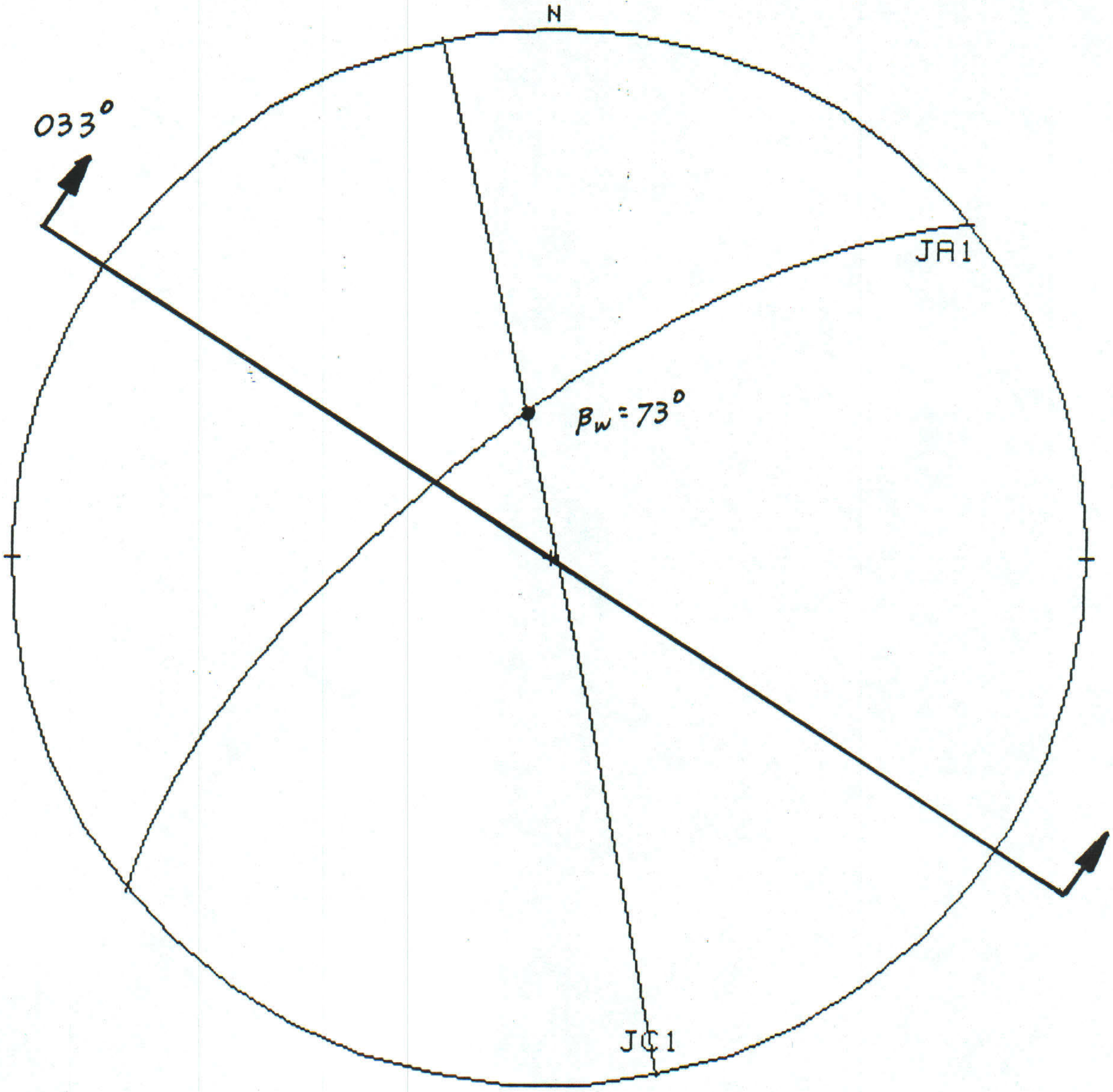
LOWER HEMISPHERE EQUAL AREA PROJECTIONS OF PLANES REPRESENTING PEAK ORIENTATIONS OF DISCONTINUITY SETS IN DESIGN SECTOR 1B-2

BY:	DATE:
AFS	12/87
APPROVED: a.f.	DWG: C-3

JOB NUMBER 87-975

GEOPLN - LOWER HEMISPHERE EQUAL AREA PROJECTION OF PLANES

PROJECT: AURORA PROJECT
DATE: 1,12,87
STRUCTURAL DOMAIN: 2B



NOTE: SEE FIG. C-1 FOR KEY

MINEREX RESOURCES LTD.
AURORA PROJECT
GEOTECHNICAL ASSESSMENT

PITEAU ASSOCIATES
GEOTECHNICAL CONSULTANTS
VANCOUVER CALGARY

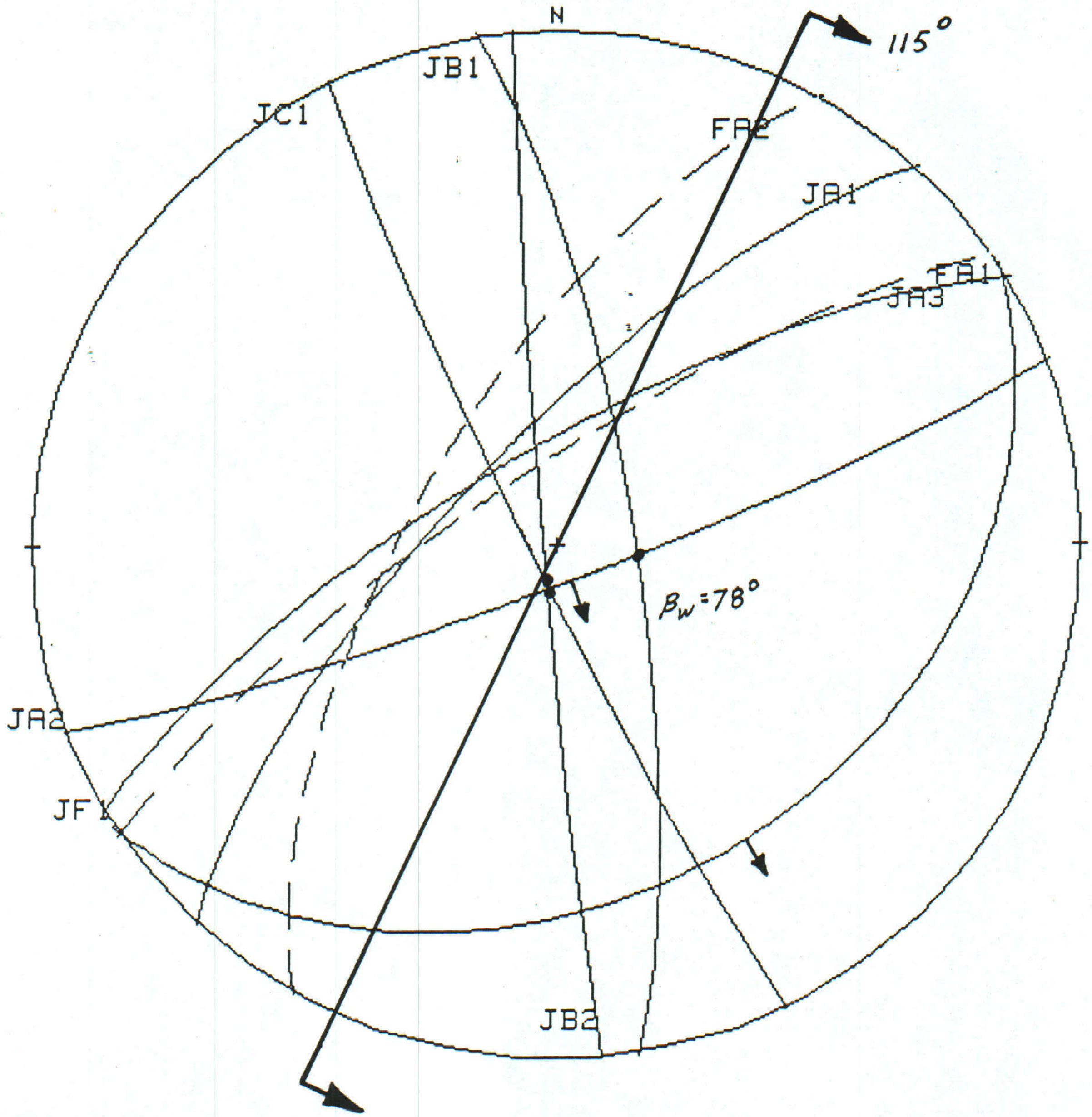
LOWER HEMISPHERE EQUAL AREA PROJECTIONS OF PLANES REPRESENTING PEAK ORIENTATIONS OF DISCONTINUITY SETS IN DESIGN SECTOR 2B-1

BY:	DATE:
AFS	12/87
APPROVED:	DWG:
a.s.	C-4

JOB NUMBER 87-975

GEOPLN - LOWER HEMISPHERE EQUAL AREA PROJECTION OF PLANES

PROJECT: AURORA PROJECT
DATE: 1,12,87
STRUCTURAL DOMAIN: 3A



NOTE: SEE FIG. C-1 FOR KEY

MINEREX RESOURCES LTD.
AURORA PROJECT
GEOTECHNICAL ASSESSMENT

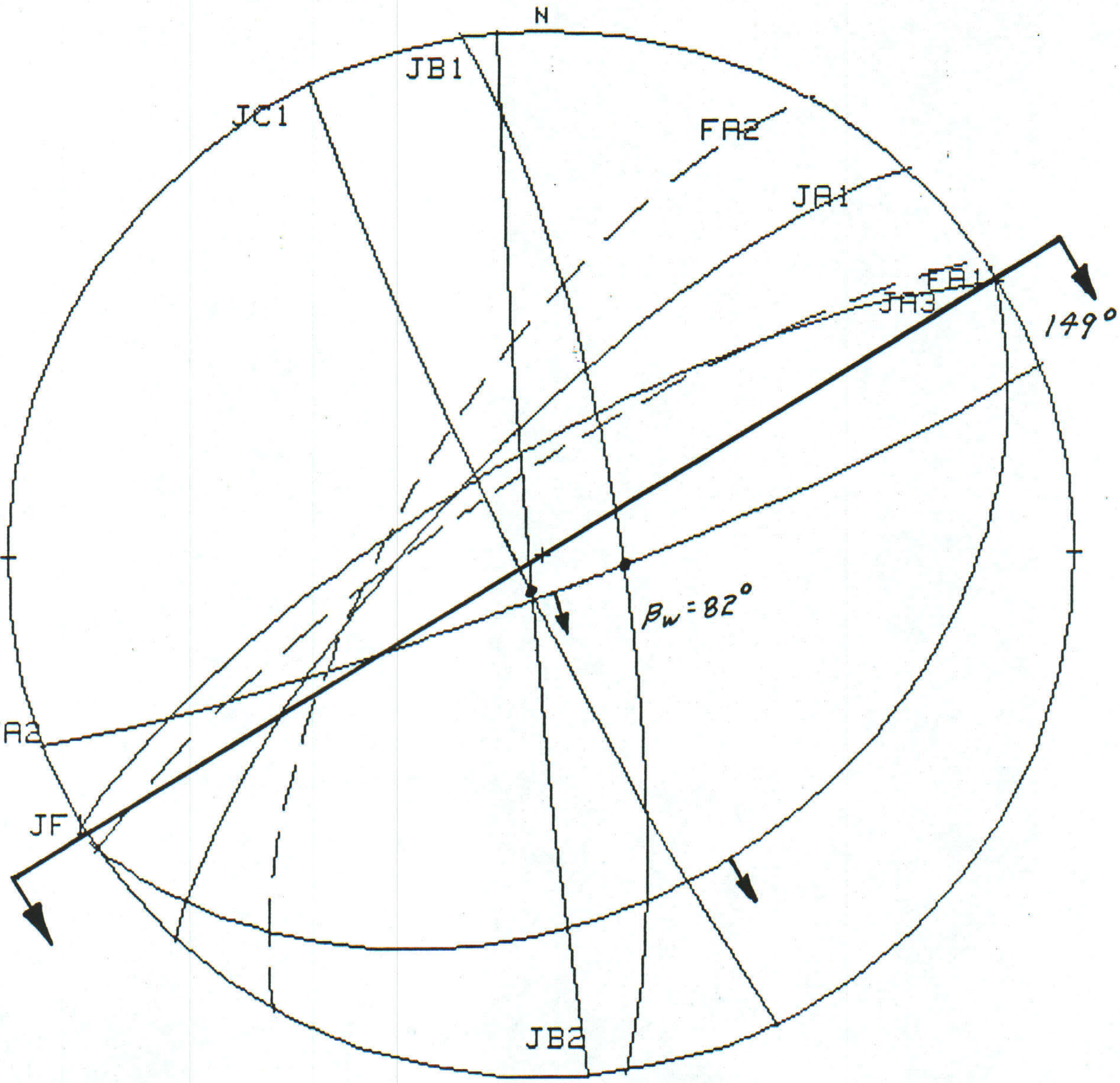
PITEAU ASSOCIATES
GEOTECHNICAL CONSULTANTS
VANCOUVER CALGARY

LOWER HEMISPHERE EQUAL AREA PROJECTIONS OF PLANES REPRESENTING PEAK ORIENTATIONS OF DISCONTINUITY SETS IN DESIGN SECTOR 3A-1 & 3A-1'	BY: AFS	DATE: 12/87
	APPROVED: A.S.	DWG: C-5

JOB NUMBER 87-975

GEOPLN - LOWER HEMISPHERE EQUAL AREA PROJECTION OF PLANES

PROJECT: AURORA PROJECT
DATE: 1,12,87
STRUCTURAL DOMAIN: 3A



NOTE: SEE FIG. C-1 FOR KEY

MINEREX RESOURCES LTD.
AURORA PROJECT
GEOTECHNICAL ASSESSMENT

PITEAU ASSOCIATES
GEOTECHNICAL CONSULTANTS
VANCOUVER CALGARY

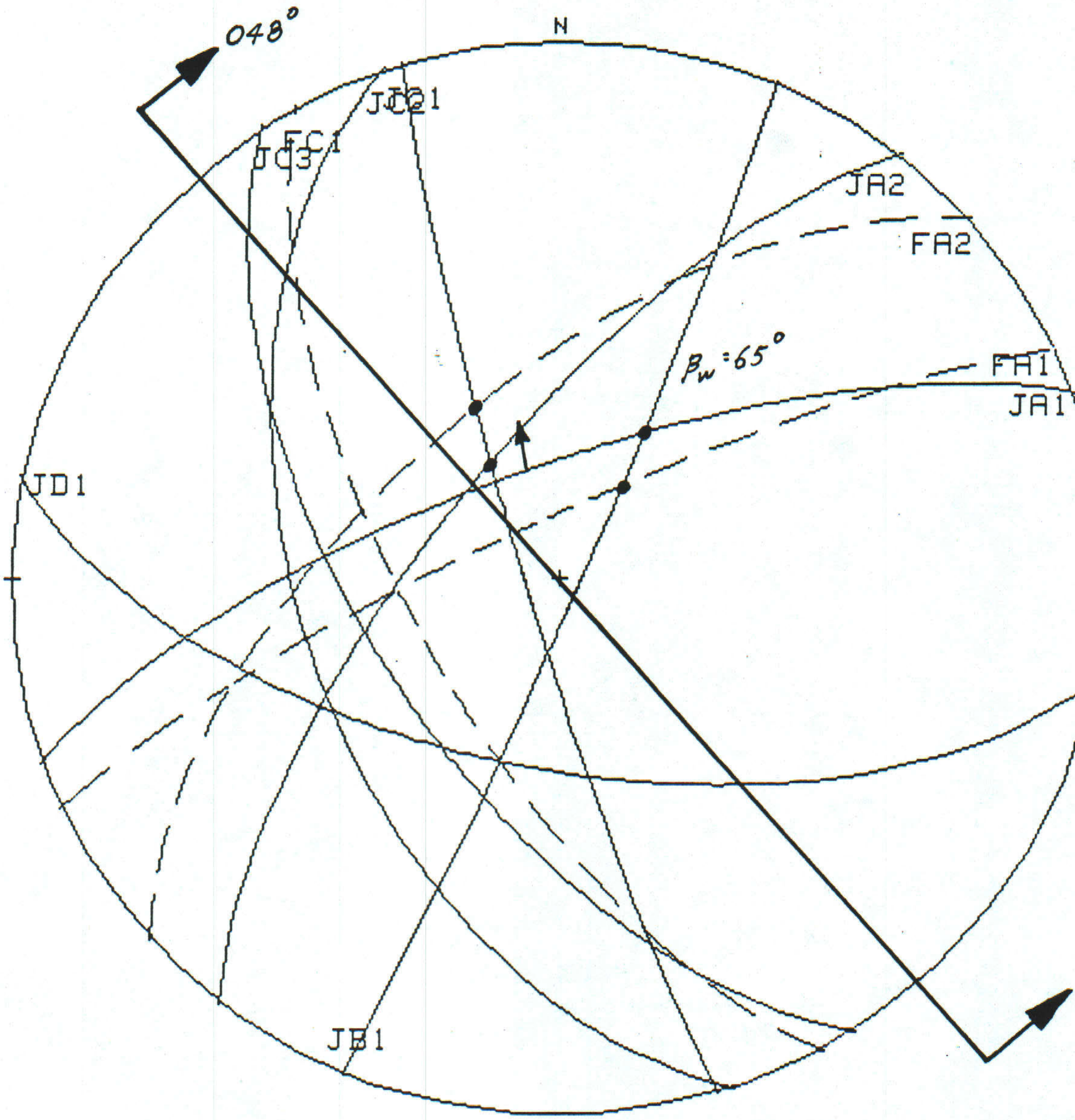
LOWER HEMISPHERE EQUAL AREA PROJECTIONS OF PLANES REPRESENTING PEAK ORIENTATIONS OF DISCONTINUITY SETS IN DESIGN SECTOR 3A-2 & 3A-2'

BY:	DATE:
AFS	12/87
APPROVED:	DWG:
A.S.	C-6

JOB NUMBER 87-975

GEOPLN - LOWER HEMISPHERE EQUAL AREA PROJECTION OF PLANES

PROJECT: AURORA PROJECT
DATE: 1,12,87
STRUCTURAL DOMAIN: 3B



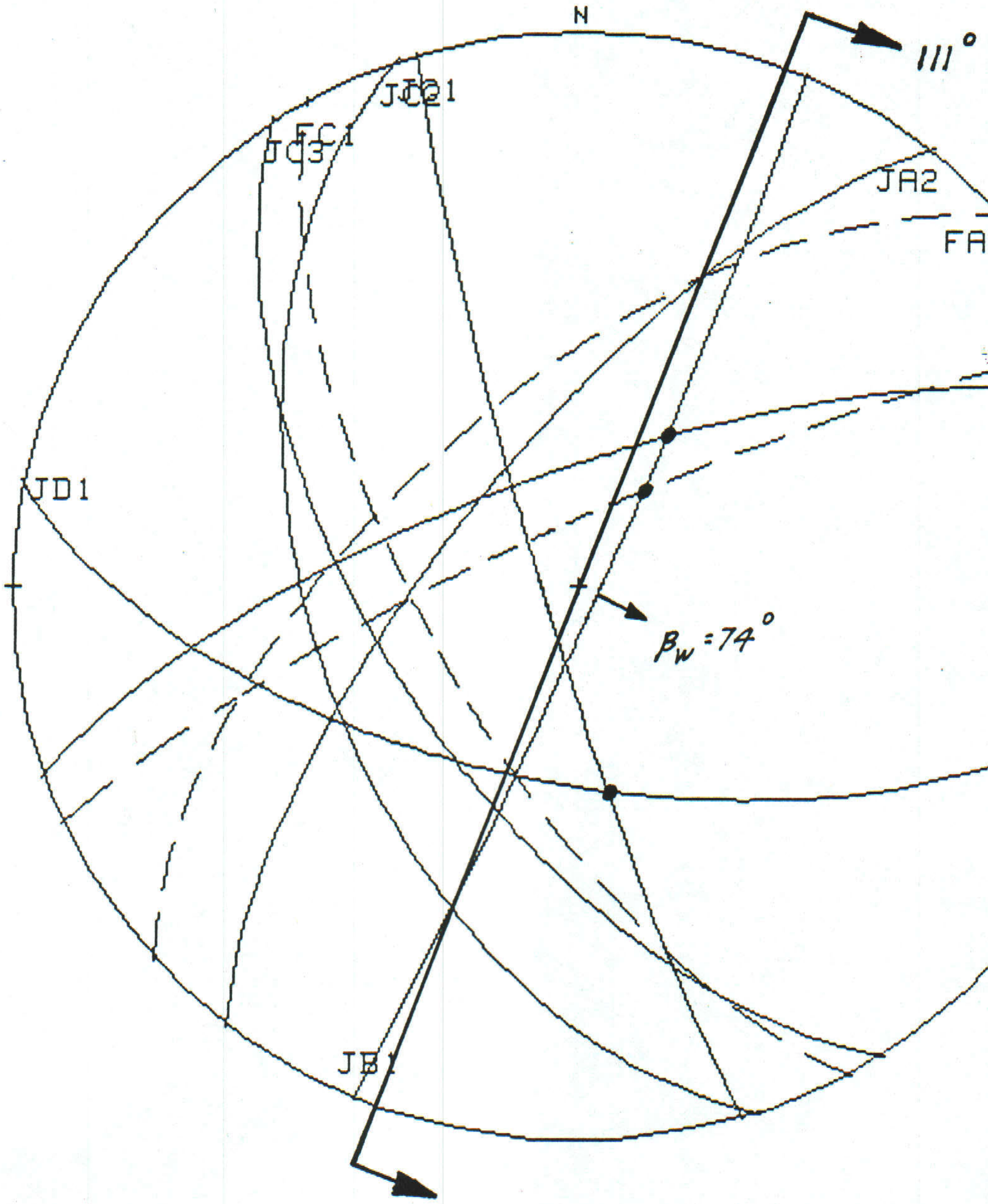
NOTE: SEE FIG. C-1 FOR KEY

MINEREX RESOURCES LTD. AURORA PROJECT GEOTECHNICAL ASSESSMENT	PITEAU ASSOCIATES GEOTECHNICAL CONSULTANTS VANCOUVER CALGARY		
LOWER HEMISPHERE EQUAL AREA PROJECTIONS OF PLANES REPRESENTING PEAK ORIENTATIONS OF DISCONTINUITY SETS IN DESIGN SECTOR 3B-1438-1'	BY: AFS	DATE: 12/87	
	APPROVED: A.S.	DWG: C-7	

GEOPLN - LOWER HEMISPHERE EQUAL AREA PROJECTION OF PLANES

PROJECT: AURORA PROJECT
DATE: 1,12,87
STRUCTURAL DOMAIN: 3B

JOB NUMBER 87-975



NOTE: SEE FIG. C-1 FOR KEY

MINEREX RESOURCES LTD.
AURORA PROJECT
GEOTECHNICAL ASSESSMENT

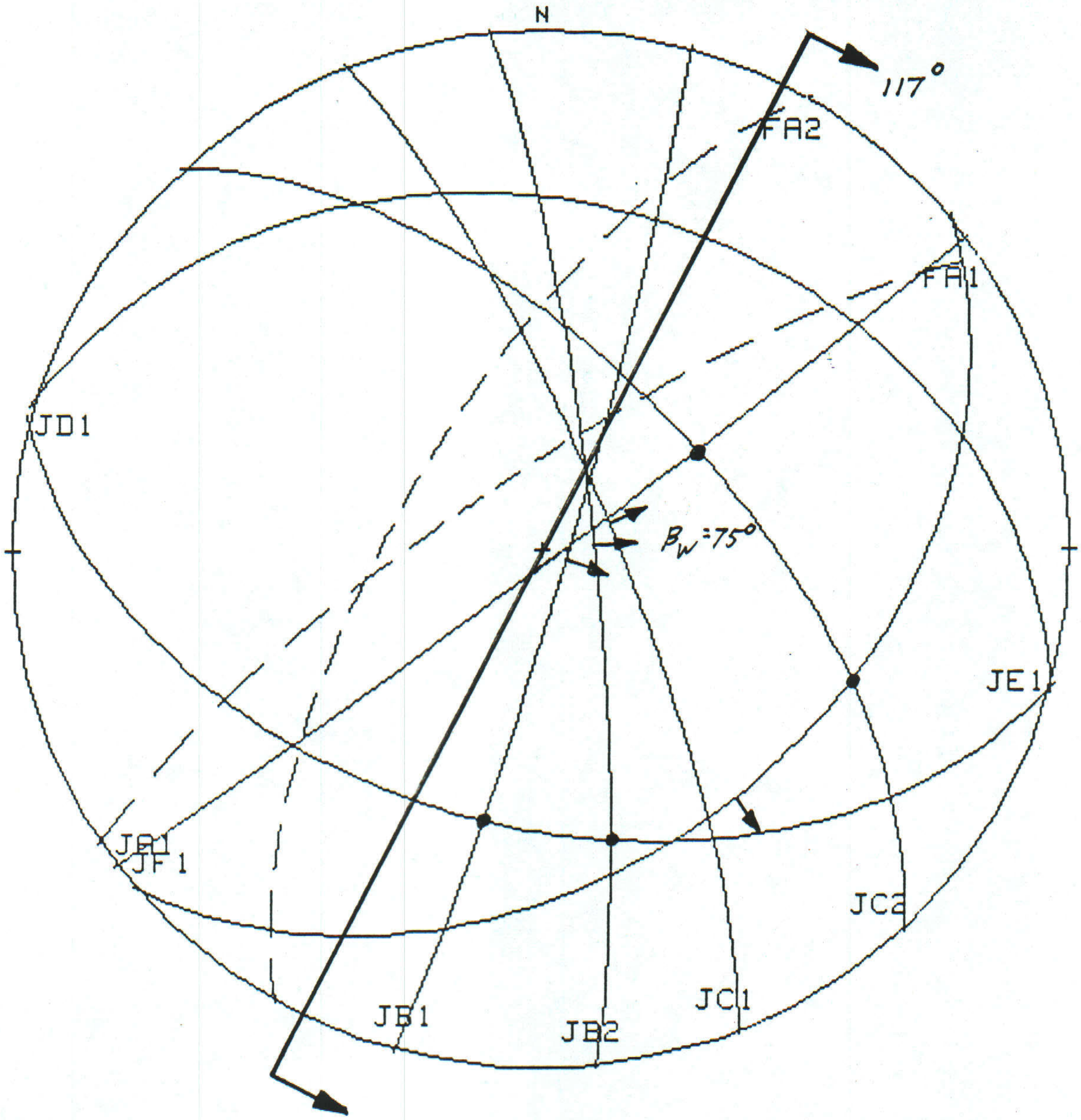
PITEAU ASSOCIATES
GEOTECHNICAL CONSULTANTS
VANCOUVER CALGARY

LOWER HEMISPHERE EQUAL AREA PROJECTIONS OF PLANES REPRESENTING PEAK ORIENTATIONS OF DISCONTINUITY SETS IN DESIGN SECTOR 3B-2 + 3B-2'	BY: AFS	DATE: 12/87
	APPROVED: A.S.	DWG: C-B

JOB NUMBER 87-975

GEOPLN - LOWER HEMISPHERE EQUAL AREA PROJECTION OF PLANES

PROJECT: AURORA PROJECT
DATE: 1,12,87
STRUCTURAL DOMAIN: 4A



NOTE: SEE FIG. C-1 FOR KEY

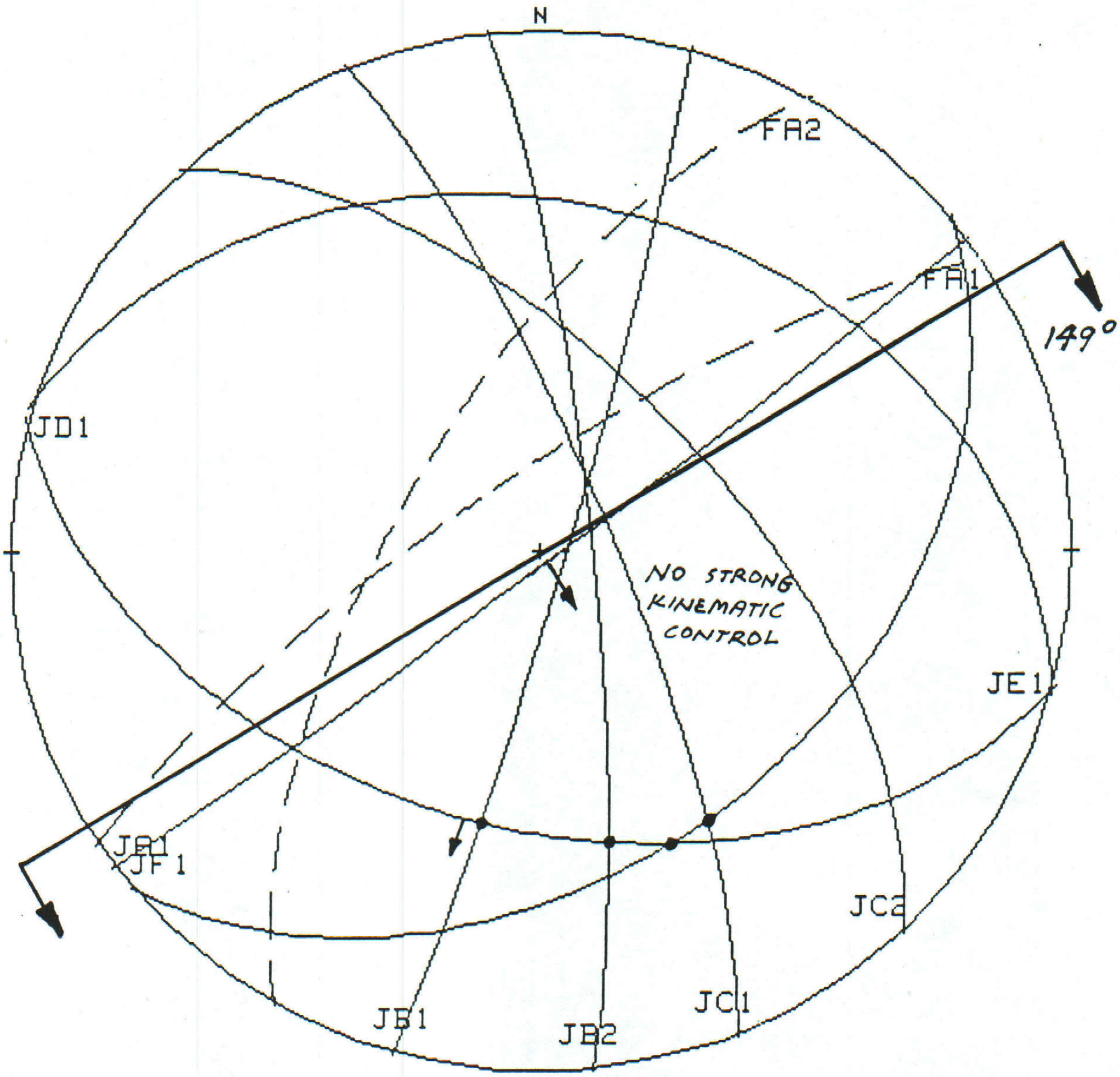
MINEREX RESOURCES LTD. AURORA PROJECT GEOTECHNICAL ASSESSMENT	PITEAU ASSOCIATES GEOTECHNICAL CONSULTANTS VANCOUVER CALGARY	

LOWER HEMISPHERE EQUAL AREA PROJECTIONS OF PLANES REPRESENTING PEAK ORIENTATIONS OF DISCONTINUITY SETS IN DESIGN SECTOR 4A-1	BY: AFS	DATE: 12/87
	APPROVED: A.S.	DWG: C-9

JOB NUMBER 87-975

GEOPLN - LOWER HEMISPHERE EQUAL AREA PROJECTION OF PLANES

PROJECT: AURORA PROJECT
DATE: 1,12,87
STRUCTURAL DOMAIN: 4A



NOTE: SEE FIG. C-1 FOR KEY

MINEREX RESOURCES LTD.
AURORA PROJECT
GEOTECHNICAL ASSESSMENT

PITEAU ASSOCIATES
GEOTECHNICAL CONSULTANTS
VANCOUVER CALGARY

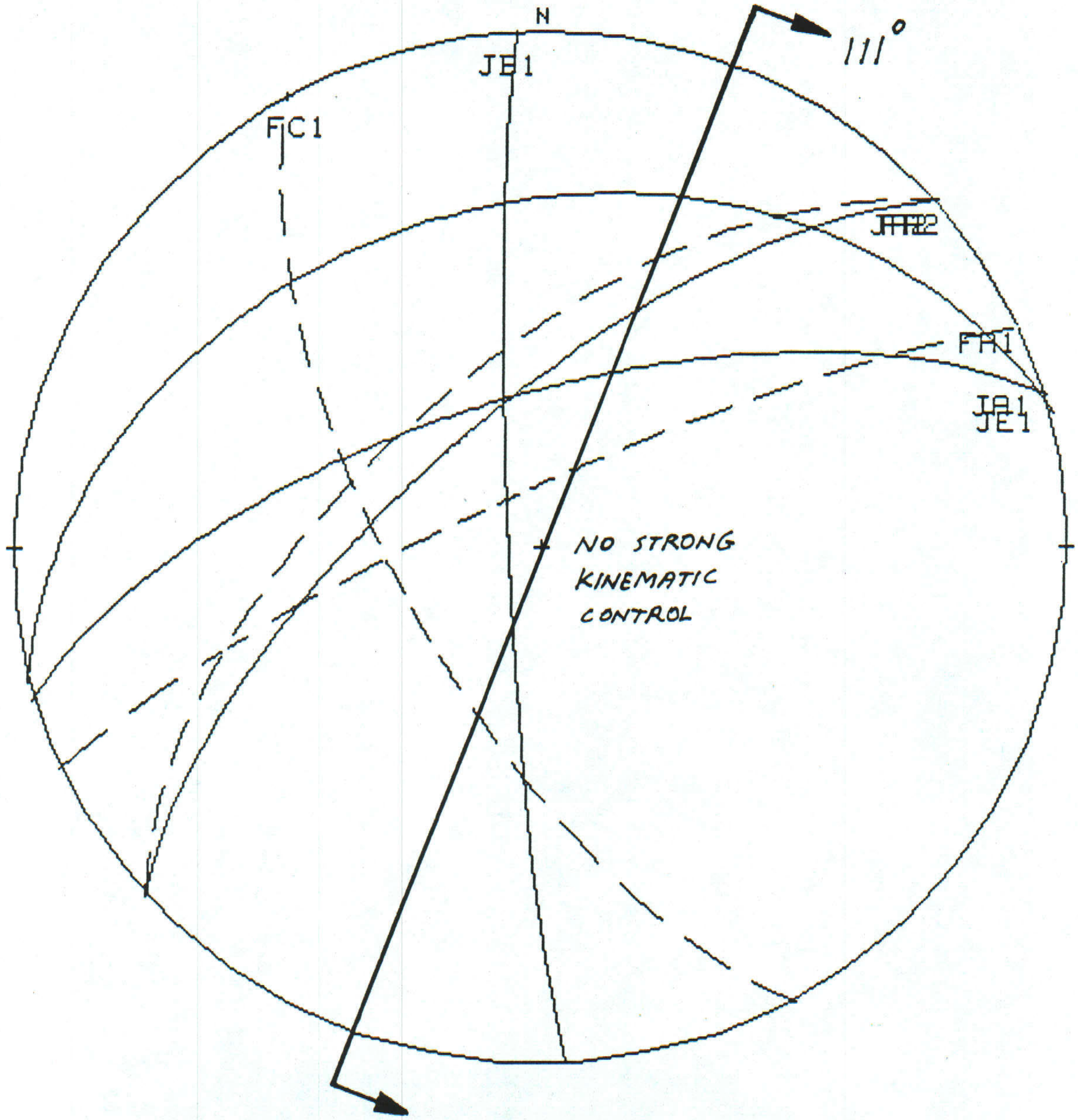
LOWER HEMISPHERE EQUAL AREA PROJECTIONS OF PLANES REPRESENTING PEAK ORIENTATIONS OF DISCONTINUITY SETS IN DESIGN SECTOR 4A-2

BY:	DATE:
AFS	12/87
APPROVED:	DWG:
A.S.	C-10

JOB NUMBER 87-975

GEOPLN - LOWER HEMISPHERE EQUAL AREA PROJECTION OF PLANES

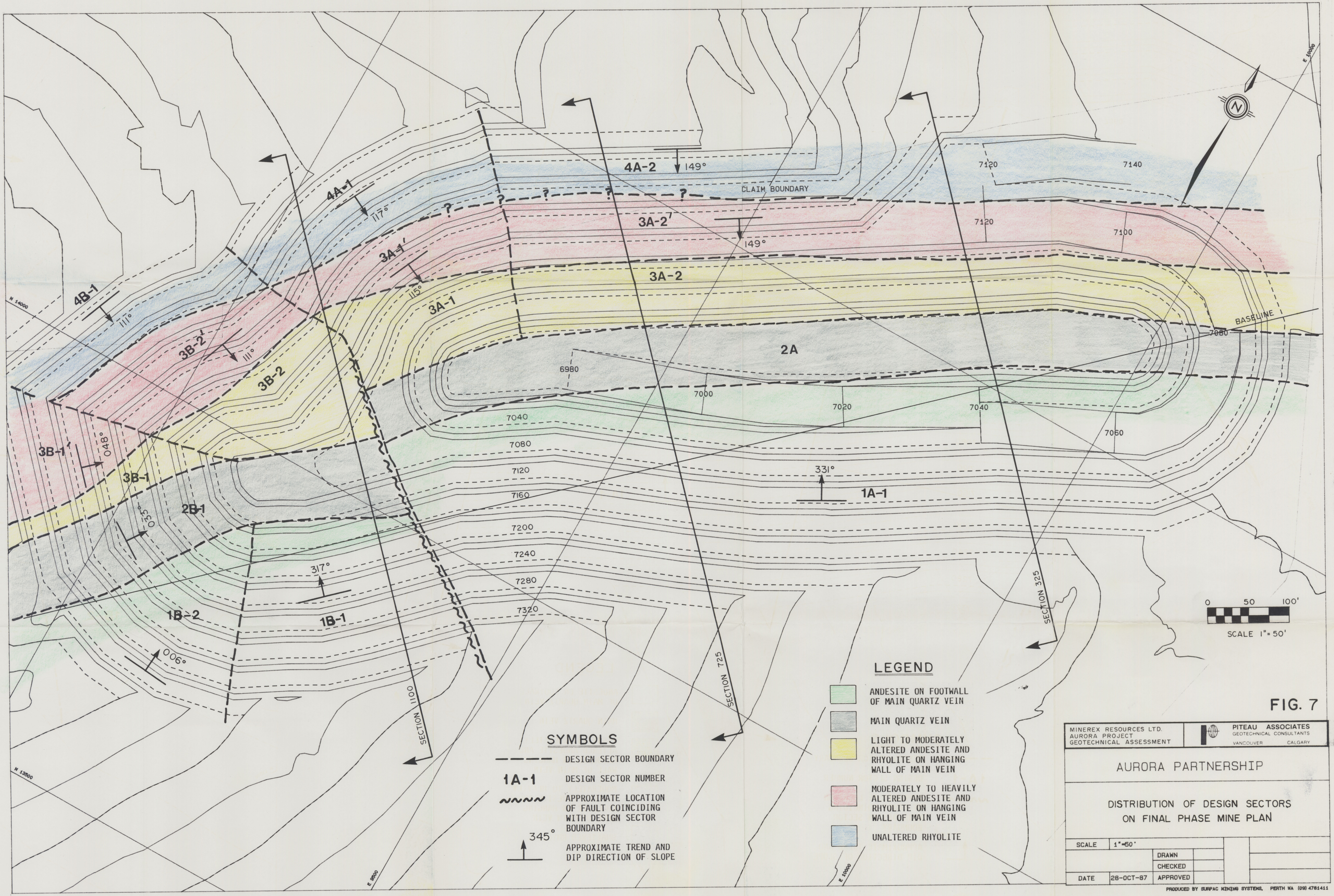
PROJECT: AURORA PROJECT
DATE: 1/12/87
STRUCTURAL DOMAIN: 4B



NOTE: SEE FIG. C-1 FOR KEY

MINEREX RESOURCES LTD. AURORA PROJECT GEOTECHNICAL ASSESSMENT	PITEAU ASSOCIATES GEOTECHNICAL CONSULTANTS VANCOUVER CALGARY

	BY:	DATE:
LOWER HEMISPHERE EQUAL AREA PROJECTIONS OF PLANES REPRESENTING PEAK ORIENTATIONS OF DISCONTINUITY SETS IN DESIGN SECTOR 4B-1	AFS	12/87
	APPROVED:	DWG:
	a.v.	C-11



0410 0068

0410 0068


SECTION 325
SECTION 725
SECTION 1100
4A
3A
2A
1A
3B
2B
1B
APPROXIMATE ULTIMATE PIT CREST
BASELINE
LEGEND
1A,1B ANDESITE ON FOOTWALL OF MAIN QUARTZ VEIN
2A,2B MAIN QUARTZ VEIN
3A,3B ALTERED ANDESITE & ALTERED RHYOLITE ON HANGING WALL OF MAIN QUARTZ VEIN
4A,4B UNALTERED RHYOLITE
SYMBOLS
STRUCTURAL DOMAIN BOUNDARY
1A STRUCTURAL DOMAIN NUMBER
APPROXIMATE LOCATION OF FAULT COINCIDING WITH STRUCTURAL DOMAIN BOUNDARY
0 50' 100'
SCALE 1"=50'
FIG. 1
MINEREX RESOURCES LTD.
AURORA PROJECT
GEOTECHNICAL ASSESSMENT
PITEAU ASSOCIATES
GEOTECHNICAL CONSULTANTS
VANCOUVER CALGARY
PMG
DEC87
SITE PLAN

0410 0068

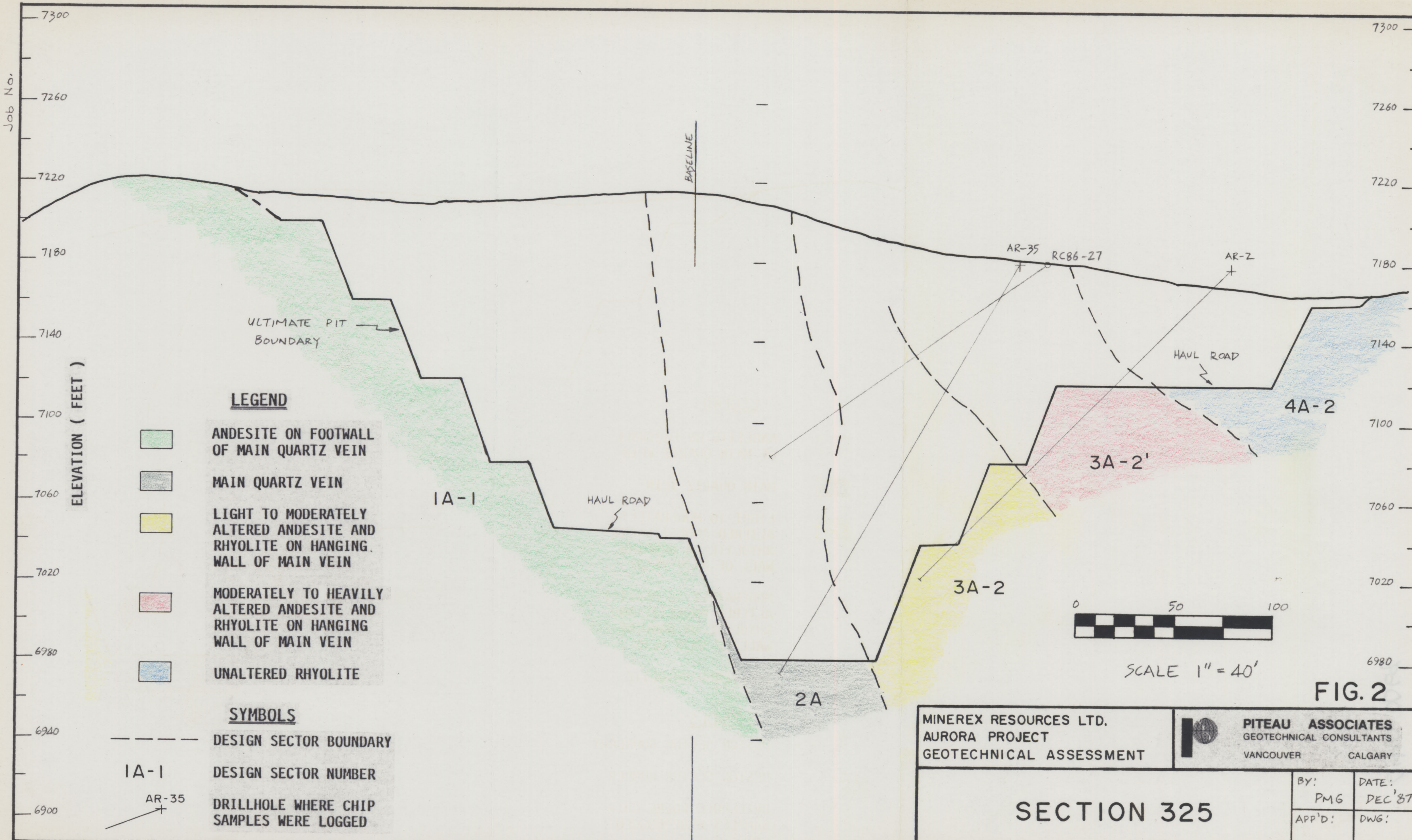


FIG. 2

MINEREX RESOURCES LTD.
AURORA PROJECT
GEOTECHNICAL ASSESSMENT

PITEAU ASSOCIATES
GEOTECHNICAL CONSULTANTS
VANCOUVER CALGARY

SECTION 325

BY:	DATE:
PMG	DEC '87
APP'D:	DWG:

Job No.

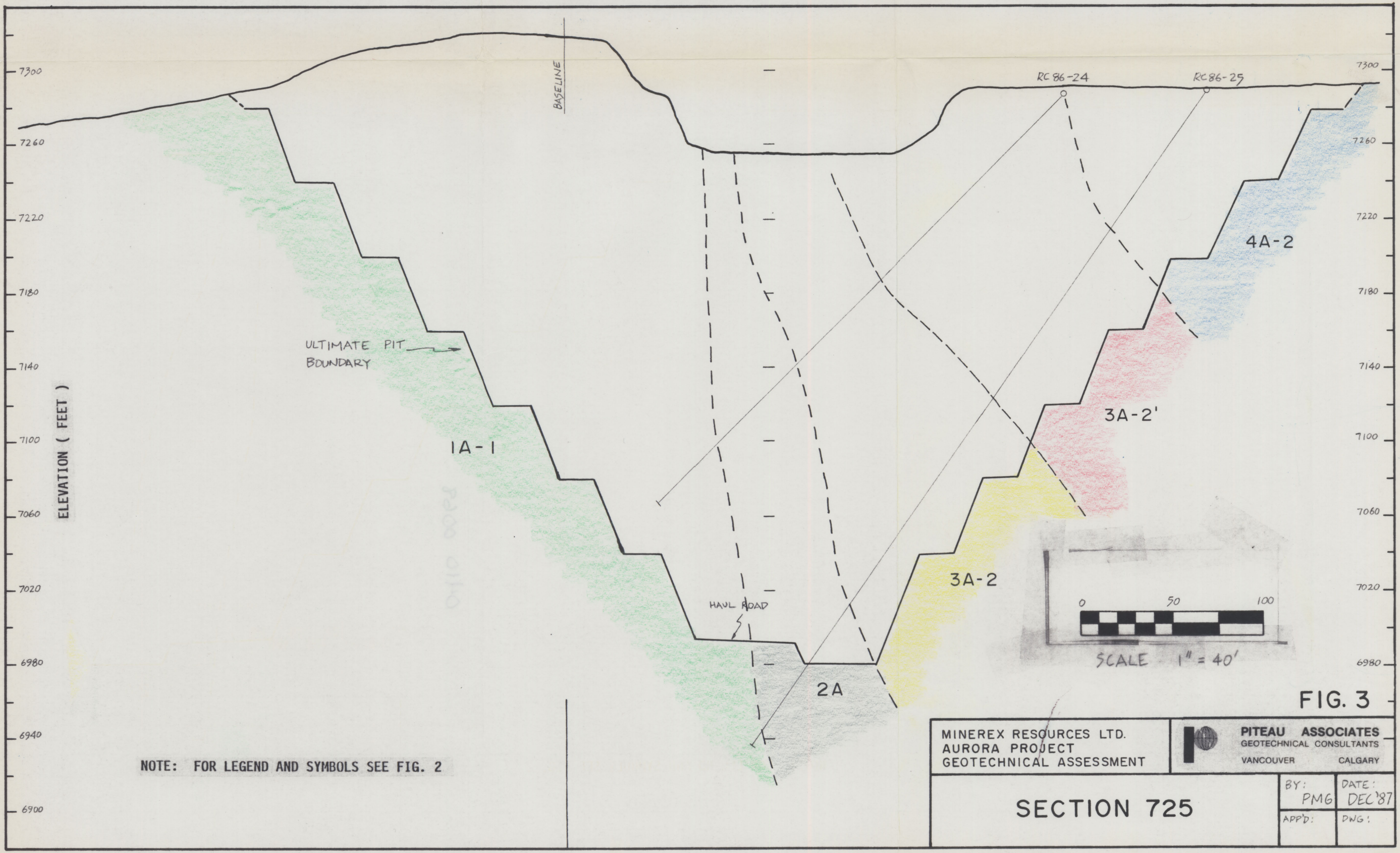
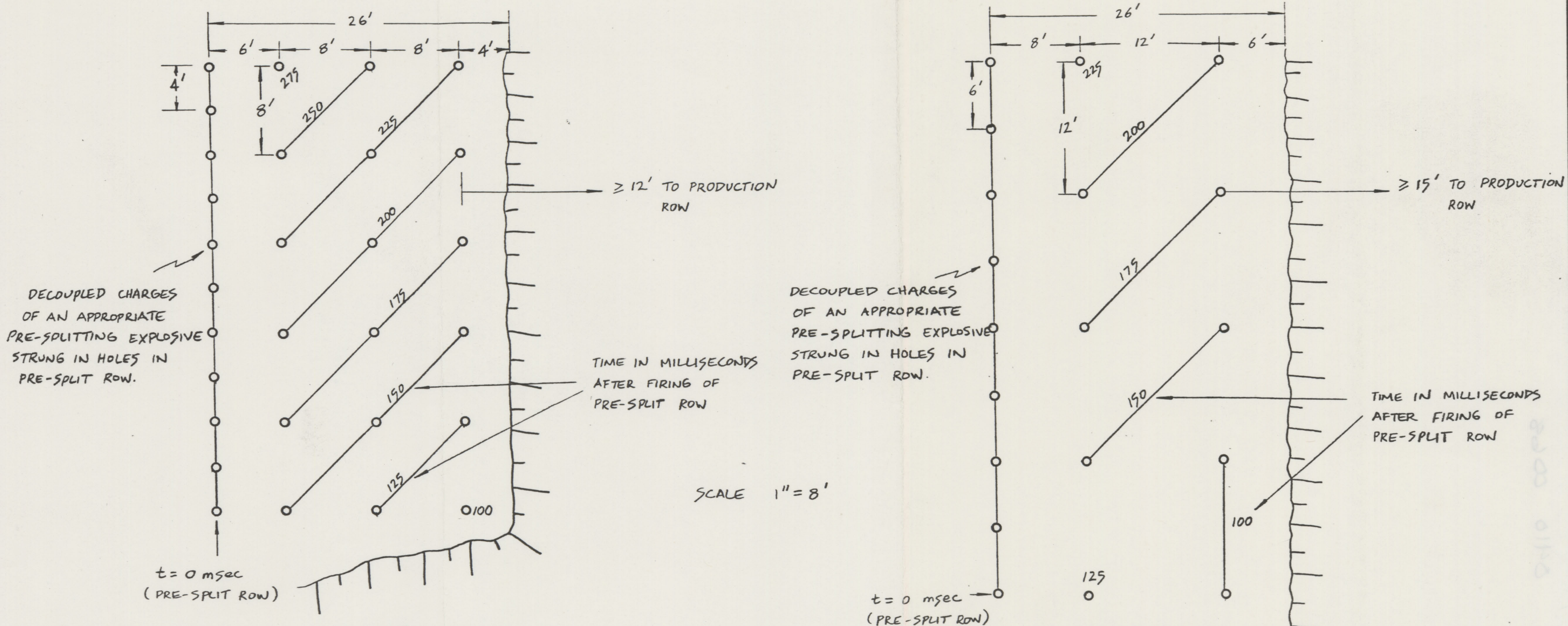

ELEVATION (FEET)
7300
7260
7220
7180
7140
7100
7060
7020
6980
6940
6900
BASELINE
RC86-24
RC86-25
ULTIMATE PIT BOUNDARY
1A-1
HAUL ROAD
2A
3A-2
3A-2'
4A-2
0
50
100
SCALE 1" = 40'
NOTE: FOR LEGEND AND SYMBOLS SEE FIG. 2
MINEREX RESOURCES LTD.
AURORA PROJECT
GEOTECHNICAL ASSESSMENT
PITEAU ASSOCIATES
GEOTECHNICAL CONSULTANTS
VANCOUVER
CALGARY
SECTION 725
BY: PMG
DATE: DEC '87
APP'D:
DWG:

FIG. 3

JOB # 87-975



10a) 4½" DIAMETER VERTICAL HOLES

10b) 6" DIAMETER VERTICAL HOLES

FIG. 10

NOTE:
1) TWO FREE FACES
2) ~ 0.3 lb/TON POWDER FACTOR IN PERIMETER BLAST
3) ~ 0.5 lb/TON POWDER FACTOR IN 12'x12' PRODUCTION BLAST

NOTE:
1) ONE FREE FACE
2) ~ 0.3 lb/TON POWDER FACTOR IN PERIMETER BLAST
3) ~ 0.4 lb/TON POWDER FACTOR IN 12'x12' PRODUCTION BLAST

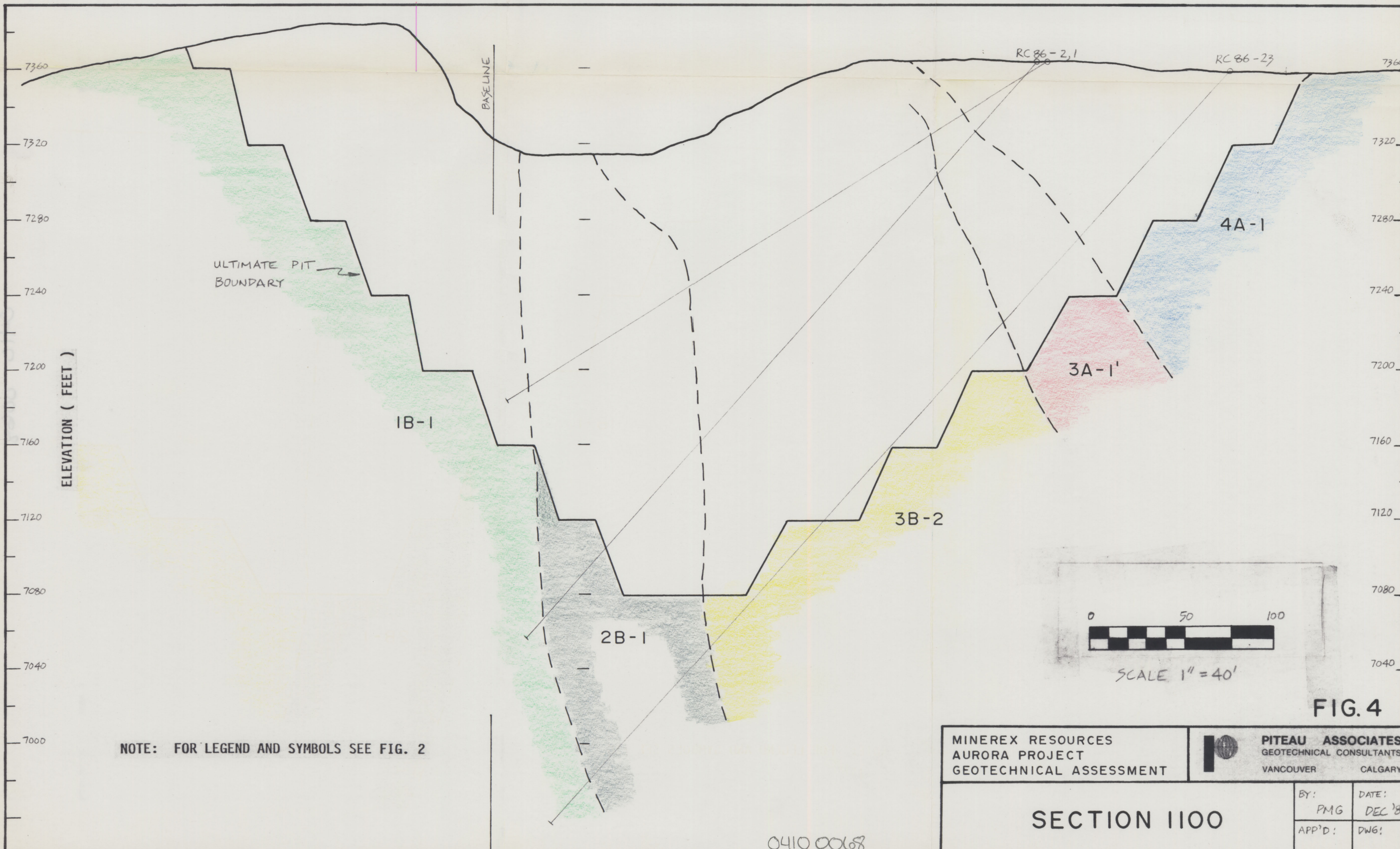
MINEREX RESOURCES LTD.
AURORA PROJECT
GEOTECHNICAL ASSESSMENT

PITEAU ASSOCIATES
GEOTECHNICAL CONSULTANTS
VANCOUVER CALGARY

SCHEMATIC OF SUGGESTED TRIAL PERIMETER BLASTS IN ANDESITE ON FOOTWALL OF MAIN QUARTZ VEIN

BY:	DATE
PMG	JAN 88
APPROVED:	DWG:

04100068



NOTE: FOR LEGEND AND SYMBOLS SEE FIG. 2

FIG. 4

MINEREX RESOURCES AURORA PROJECT GEOTECHNICAL ASSESSMENT	PITEAU ASSOCIATES GEOTECHNICAL CONSULTANTS VANCOUVER CALGARY	
SECTION 1100	BY: PMG	DATE: DEC '87
	APP'D:	DWG:

0410 0068