MINNOVA INC.

SAMATOSUM PROJECT

825105

PRELIMINARY GEOTECHNICAL ASSESSMENTS AND DESIGN CONSIDERATIONS FOR PIT SLOPES, WASTE DUMPS, MILL AND TAILINGS POND SITES

Prepared by

PITEAU ASSOCIATES ENGINEERING LTD.

MARCH, 1988

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HYDROGEOLOGICAL CONSULTANTS

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PROJECT: 87-946

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1. INTRODUCTION

This report describes the rock mechanics, geotechnical and hydrogeological studies carried out to prepare preliminary open pit slope and waste dump design guidelines, and to evaluate foundation conditions for possible waste dump and tailings pond sites for the proposed Samatosum Project. As shown on Fig. 1, the project site is located about sixty five kilometres northeast of Kamloops, B.C. in the vicinity of Adams Lake. Access to the property is via provincial highway and logging access roads from either Highway 5 or Highway 1.

1.1 TERMS OF REFERENCE

Terms of reference for this work were initially discussed in our proposal dated June 24, 1987. Changes to the terms of reference were discussed at a meeting with Mr. J. Purkis on January 22, 1988. In this regard Piteau Associates was retained .....

> "to carry out preliminary geotechnical and hydrogeological investigations related to the Samatosum Deposit near Barriere, B.C.
>
> "These studies are required to satisfy information requirements in the Stage 1 submission and determine planning parameters for pit wall design, dump design, facility foundations, and groundwater quantity and quality. Implied in this, are general assessments of soils and bedrock in the impact area as they relate to construction materials suitability, facility locations, slope stability and the groundwater regime."

Field work was carried out in the first week of February, 1988. Office analysis, development of design guidelines and report preparation were carried out in the remaining weeks of February. A meeting to discuss the results of the study was held in Minnova's Vancouver office on February 26 and a summary draft report was issued for review on March 3, 1988.

1.2 DESCRIPTION OF THE INVESTIGATION

1.2.1 Field Studies

Field work included field reconnaissance, test pitting, geotechnical core logging, underground geologic structural mapping, point poad index testing of drill core and installation of shallow open standpipes for future water monitoring and sampling.

i) Engineering Geology and Rock Mechanics

Geotechnical core logging, geologic structural mapping and point load index testing of drill core were carried out to assess the engineering geology and rock mechanics characteristics of the proposed open pit.

Detailed geotechnical logging of diamond drill core was previously conducted by Minnova, beginning with the 1987 exploration program. Limited relogging of a few representative drillholes was conducted by Piteau Associates to assess the data previously collected by Minnova and to gain a first hand appreciation for the appearance and mechanical characteristics of the various rock types. Approximately 620m of core from four diamond drillholes was relogged geotechnically.

Limited point load index testing was carried out on core samples to obtain an appreciation for the intact strength of the main rock types that will be encountered in the open pit and will comprise the waste dump.

Underground mapping in the Rea Gold Concession adit was conducted to determine the orientation and nature of the structural discontinuities which may occur in the pit area. Because of the limited number of outcrops, and the snow cover at the time of the field visit, surface mapping was impractical.

ii) Surficial Geology

Surficial geology studies were conducted to assess potential geotechnical hazards and foundation conditions in the vicinity of proposed waste dump, tailings pond and mill sites. Studies included a preliminary airphoto interpretation followed by field reconnaissance, surficial soils mapping, test pitting and soil sampling.

Using a backhoe, sixteen test pits were excavated to depths of up to about 3.1m. The test pits were logged and samples of the various soil strata were obtained for laboratory classification and testing. Preliminary test pit logs were prepared.

iii) Hydrogeology

To evaluate hydrogeologic conditions in the vicinity of the pit and possible waste dump sites, open standpipe piezometers were installed in three test pits where seepage was observed. These standpipes can be used for monitoring water levels in the surficial soils and for obtaining water samples for water quality testing. General observations of seepage in the Rea Gold adit and in other locations around the property were also made during the course of the field work; however, these observations were restricted due to snow cover and site access limitations.

1.2.2 Office Studies

i) Engineering Geology and Pit Design

Geotechnical core logging data collected by Minnova and Piteau Associates were compiled and processed. Based primarily on geological interpretations prepared by Minnova, and on the results of the geotechnical core logging, representative geotechnical sections were prepared for the pit area.

Geological structural data from mapping conducted by both Piteau Associates and Dolmage Campbell and Associates in the Rea Gold adit were analyzed to define the orientation and nature of the structural populations at the site. Surface mapping data obtained by Minnova were also assessed, along with the foliation dip recorded on the drillhole logs.

Based on the results of the geologic structural analysis, assessments were carried out to determine kinematically possible failure modes which could be expected on the pit walls. Slope stability analyses were carried out and preliminary slope design guidelines were established for a full range of possible slope orientations and expected rock mass conditions.

ii) Assessments of Surficial Soils, Waste Dumps and Tailings Sites

Soil samples obtained from test pits were classified according to the Unified Soil Classification System. Pertinent features such as colour, particle size, consistency and density were recorded, as appropriate. Moisture content and gradation tests were conducted on representative samples. Preliminary field classifications were reviewed based on results of laboratory classification and testing, and test pit logs were finalized.

Based on the results of the airphoto, drilling, field and laboratory studies, the approximate extent, types and characteristics of near surface soils were delineated in the vicinity of the proposed waste dump and tailings pond sites. A preliminary surficial geology and terrain classification map was also prepared.

Based on the above, qualitative assessments were made as to the suitability of the proposed waste dump sites. General recommendations for waste dump site development and further investigations that should be completed before dump designs are finalized were prepared. Preliminary assessments of foundation conditions in the vicinity of the proposed tailings pond and mill sites were also carried out.

iii) Hydrogeological Assessment

Hydrogeologic and hydrologic data obtained during the field studies and from others involved with the project were used along with reference material to prepare an initial assessment of regional and local groundwater conditions. Groundwater inflows to the pit were estimated and a preliminary discussion of groundwater quality was prepared.

2. PHYSIOGRAPHY

2.1 TOPOGRAPHY AND DRAINAGE

The Samatosum Project area is located on the flank of a hanging valley immediately downstream from Johnson Lake (see Fig. 1). The area is contained within the Shuswap Highland, which is itself part of the Interior Plateau physiographic region. The Shuswap Highland is generally comprised of gentle to moderate sloping plateau areas dissected by a number of rivers and their tributaries. Most ridges and summits have been rounded by glaciation.

As shown on Fig. 2, ground elevations in the project area range from about 1025m* in Johnson Creek to about 1500m at the height of land above the open pit. The valley side on which the pit and waste dump will be located is characterized by slopes of between about 15° and 20°, with occasional flatter and steeper sections. The steeper portions of the slopes are up to about 30° to 35° and tend to be bedrock controlled. In the vicinity of Tailings Impoundment Site A (see Fig. 2), a number of steep, conical hills are present which appear to be bedrock controlled. These hills create a natural basin which may be suitable for tailings disposal.

As shown on Fig. 2, the project area is crossed by a number of small drainage courses, most of which appear to be intermittent. A small (i.e. about 75m x 75m) pond located in a saddle upslope of Waste Dump Site A appears to drain primarily to the southwest of the immediate mine area, but may also provide

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* All elevations quoted are relative to geodetic datum. Figs. 3 to 9 also reference elevations established according to a Minnova base station which is 185.25m higher than the geodetic datum.

recharge for shallow groundwater seepage into Waste Dump Site A. All of the drainage courses flow into Johnson Creek, which drains Johnson Lake and flows to the southwest, eventually emptying into Sinmax Creek. The project area has in the past been covered by a dense growth of coniferous trees. However, recent logging has removed the vegetation from much of the site.

2.2 CLIMATE

The Samatosum Project area is located along the western edge of the Southeast Wet Interior Climatic Region, immediately east of the Southwest Interior Climatic Region. While no long term monitoring stations are located in the immediate vicinity, it is anticipated that the average temperature and precipitation at the site would be similar (albeit slightly cooler and wetter due to the higher altitude) to that recorded at Barriere. In this regard, the mean annual temperature at Barriere is about 6^{o} C, with the coldest mean daily temperatures occurring in January (i.e. about -8^{o} C) and the warmest mean daily temperatures occurring in July and August (i.e. about 18^{o} C). The mean total precipatation at Barriere is about 442mm/year, with the monthly total precipatation (i.e. rain and snow combined) being relatively evenly distributed throughout the year.

3. ENGINEERING GEOLOGY

3.1 BEDROCK

3.1.1 Regional Geology

In general terms, the site is located along the western flank of the Shuswap Metamorphic Complex and is underlain by rocks of the Devonian or older Eagle Bay Formation. These rocks were complexly deformed and metamorphosed during the Jurasic-Cretaceous Columbian orogeny with the general grade of metamorphism being lower greenschist in many areas.

In the vicinity of the property, the Eagle Bay Formation is generally a medium to dark schistose rock derived from mafic and intermediate volcanic and volcaniclastic rocks. It also includes a major carbonate member, the Tshinakin limestone-dolomite, which is found at the western end of Johnson Lake, as well as widespread and locally dominant intercalations of dark to light grey siliceous and graphitic phyllite, calcareous phyllite, limestone, dolostone, marble, calc-silicate, cherty quartzite and other metasedimentary rocks.

At the Samatosum Project, the ore generally occurs in Muddy Tuff near the contact with overlying altered and weathered Mafic Pyroclastics. The stratigraphic section is isoclinally folded and overturned, with strata dipping to the northeast.

3.1.2 Rock Types

The main rock types in the pit area are illustrated in plan in Fig. 3, on section in Figs. 4 to 9 and are briefly described as follows:

i) Mafic Pyroclastics

Mafic Pyroclastics (including Mafic Tuffs) typically are comprised of basaltic fragmentals (i.e. tuff, lapilli tuff and agglomerate). Quartz clasts, carbonate veinlets and platy minerals are common. These rocks are generally dark green when fresh, becoming bleached to light green with light sericite alteration. Alteration generally increases with depth (i.e. closer to the ore zone). Foliation appears to be the dominent structural feature within the rock, with the rock being relatively weak parallel to foliation. This material, which it is anticipated will form the majority of the pit walls above the ore zone, comprises the stratigraphic footwall and the structural hanging wall of the deposit.

ii) Sericitic Mafic Tuff

As the name implies, these rocks are Mafic Pyroclastics which have undergone moderate to strong sericitic alteration to a sericite-quartz schist. This material is generally found between the Mafic Pyroclastics and the ore zone and is bleached to a light yellowish green. Foliation breaks are dominant in drill core and tend to be relatively smooth, with considerable sericite and other soft minerals.

iii) Cherts

A cherty zone, consisting of a mixed package of chemical chert, argillite and sericitic tuff, has been observed in some areas. This rock, which varies from being relatively massive and hard where the chert is pure, to very weak in numerous gouge zones, shows rapid thickness variations. It is not anticipated that this material will comprise a significant portion of the final pit walls.

iv) Muddy Tuffs

Muddy Tuffs typically underly the Sericitic Mafic Tuffs and are comprised of a complex mix of sediments and tuffs which have been extensively altered to sericite and clays. The rock is relatively homogeneous and grey in color, but alters to a pale yellow or white. Numerous faults have resulted in the formation of soft clay gouge in some zones. The most common texture in this unit is a fine debris flow slump or slump breccia made up of chert, volcanics, sediments and sulphide clasts in a gritty matrix. Muddy Tuffs are the host rocks for the main ore zone and as such will be exposed over much of the footwall.

v) Sediments

Thick bedded to finely laminated greywackes and argillites are present both above and below the ore zone (see Figs. 4 to 9). These rocks, which are locally graphitic and can be a coarse quartzose grit in proximity to the ore zone, are anticipated to be present over only a small portion of the final pit walls.

3.1.3 Structural Geology

What little geologic structural information that was available in the vicinity of the proposed pit was generally confined to records of foliation dip in diamond drill core and to limited surface geological mapping conducted by Minnova. Additional detailed surface mapping was not practical because of snow cover and restricted site access. However, underground mapping by Piteau Associates and Dolmage Campbell and Associates in the adjacent Rea Gold adit yielded geologic structure which appears to be consistent with the limited information available in the pit area. This consistency is not unexpected, as the Minnova and Rea Gold orebodies occur within the same isoclinally folded sequence, albeit within different folds. Therefore, for preliminary assessment purposes, the orientation and characteristics of discontinuity populations are assumed to be the same at both sites. Once site conditions improve and better access is available, additional geologic structural mapping should be conducted to confirm this basic assumption.

Lower hemisphere equal area projections were used to define the peak or average orientation for the main discontinuity sets mapped. While joint data from all three data bases were evaluated separately and collectively during the investigations, only the data collected by Piteau Associates were used for kinematic and stability analyses and preliminary slope design.

i) Joints

The distribution of joints mapped by Piteau Associates in the Rea Gold adit is shown in Fig. 10. Three reasonably tight and well defined joint sets are generally recognized, with only moderate scatter in the data. In the case of foliation joints (i.e. Joint Set F1) it is reasonable to represent the joint set by a single peak orientation. However, the remaining joint sets (i.e. Joint Sets A and B) are more realistically represented by more than one peak (i.e. JB1, JB2, JB3 etc.).

Foliation joints (Joint Set F1) have a peak dip direction/dip of 025^{o} /42^{o}, but appear to range in dip from about 20^{o} to 60^{o}. As illustrated on the geotechnical sections in Figs. 4 to 8, this dip range would appear to continue with depth. However, some erratic variation in both dip direction and dip is to be expected. Foliation joints are very well developed and are the dominant joints in the area. Based on observations in the adit (see Photo 1), these joints, appear relatively continuous and tend to be planar and slightly rough. Foliation joint spacing is difficult to assess, but is likely to be in the range of 0.2 to 1.0m.

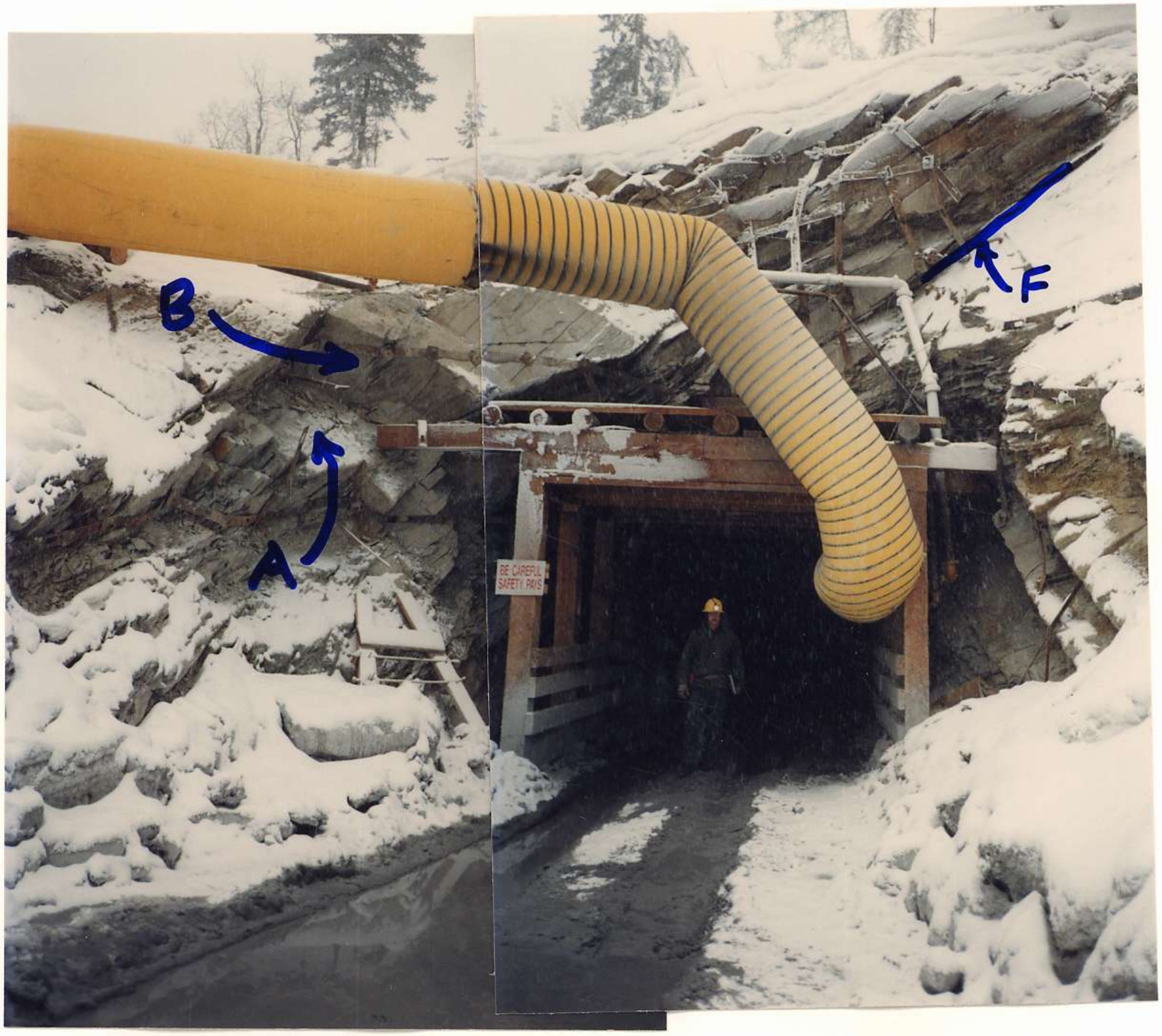

PHOTO 1: View of structural discontinuities at portal of Rea Gold Adit. Note foliation joints (i.e. F) and joints of Joint Sets A and B.

Joints of Joint Sets A and B are referred to as cross joints and are considered to be less continous than foliation joints, often being truncated or offset by throughgoing foliation joints (see Photos 1 and 2). These joints are usually rougher than foliation joints and are often spaced at greater than 1m. Peak orientations of Joint Sets A and B are tabulated on Fig. 10.

ii) Faults

Several relatively continuous faults are indicated on geological plans provided by Minnova. In addition, numerous fault zones have been encountered in diamond drillholes. However, only a very few fault orientations have been obtained to date, with most of these having been measured in the Rea Gold adit. Such measurements, along with some general fault trends noted on the geologic plans, suggest that the primary orientations of faults on the property are similar to those of the joint sets. In addition, it would appear that most faults are fault zones, as opposed to discrete fault planes, and that breccia and soft sericitic gouge are common infilling materials. Further mapping and documentation of faulting is required as the mine is developed to develop a better understanding of the nature and orientation of these structures.

iii) Contacts

Contacts between the various rock units tend to be somewhat obscured and gradational, caused primarily by the various degrees of alteration within the rock mass. Nonetheless, for geotechnical purposes, lithologic contacts can be delineated (see Figs. 4 to 9).

iv) Folds

As discussed above, the stratigraphic sequence at the property has been isoclinally folded and overturned. As a result all, strata dip to the northeast at about 42^{o}.

PHOTO 2: View of Joints of Joint Sets A and B in Rea Gold adit.

3.1.4 Rock Strength

i) Intact Rock Strength

A preliminary assessment of intact rock strength was conducted based on point load index testing of diamond drill core from four drillholes. Results are summarized in Table I and indicate that uniaxial compressive strength varies considerably, depending on rock type, degree of alteration, degree of silicification and direction of testing relative to foliation.

The bulk of the point load index data was obtained from tests conducted parallel to foliation. Testing perpendicular to foliation was very difficult as the samples tended to spall along foliation, indicating the rock types tested are strongly anisotropic. Where testing perpendicular to foliation was successful, Anisotropy Indices (i.e. ratio of point load index perpendicular foliation to point load index parallel foliation) of greater than about three were obtained.

Based on results summarized in Table I, the fresh to lightly altered Mafic Pyroclastics (including Mafic Tuffs) are the strongest rocks tested, with a uniaxial compressive strength (parallel foliation) in the range of about 30 to 70 MPa (4500 to 10000 psi). These rocks would be classified as Hardness R3 to R4 (average to hard rock), which is consistent with field hardness observed in Mafic Tuffs in the Rea Gold adit.

Lightly to moderately altered sericitic Mafic Tuffs and lightly altered Muddy Tuffs appear to have similar uniaxial compressive strengths parallel to foliation, with median values in the range of 12 to 16 MPa (1750 to 2350 psi). These rocks would be classified as Hardness R2 (soft rock), which is generally consistent with observations in Muddy Tuffs in the Rea Gold adit. Point load index testing was also attempted on moderately to strongly altered versions of sericitic Mafic Tuff and Muddy Tuff; however, these rocks were generally too weak to reliably test using the point load machine.

Other rock types (e.g. argillite, greywakes, chert, etc.) were not tested as these were generally minor constituents of the drillholes from which point load samples were obtained; hence, insufficient samples were available for reliable statistical evaluation. Based on field hardness classifications in core and in the Rea Gold adit, however, argillites and greywakes are expected to be as weak or weaker than Muddy Tuff and related rocks. Bedded cherts, cherty zones and sulphide zones throughout the rock mass are expected to be generally harder than the adjacent or host rock. Strength in these zones will depend primarily on the degrees of silicification and sericite alteration.

ii) Strength of Discontinuities

Multistage direct shear testing was conducted on two samples of foliation partings obtained from moderately altered sericitic Mafic Tuff and lightly altered Muddy Tuff. Results of these tests are illustrated in Fig. 11 and indicate lower bound residual friction angles in the range of 21 to 27^{o} and a peak friction angle (based on one test) of 28 to 40^{o}. Based on these results, a friction angle of 25^{o} is considered appropriate for foliation joints for preliminary stability analyses.

No testing of cross joints or faults was conducted; however, cross joints are anticipated to be much rougher and stronger than foliation joints. Based on observed conditions in the Rea Gold adit, for preliminary stability assessments, a friction angle of 35^{o} is considered appropriate for cross joints. Observations in core and in the Rea Gold adit indicate faults may be slickensided and contain a considerable thickness of low friction sericite, talc or graphitic gouge. Hence, for preliminary assessment purposes, a friction angle of 20^{o} is considered appropriate for faults.

3.1.5 Geotechnical Core Logging

Geotechnical core logging of most diamond drill core was conducted by Minnova, beginning with the 1987 exploration program (i.e. Drillholes RG 89 to RG 221). Parameters recorded include: core recovery, RQD, fracture frequency, vein frequency, dip of foliation and Degree of Breakage. It should be noted that no attempt was made to distinguish between naturally occurring fractures and drilling breaks; hence, RQD and fracture frequency measured by Minnova are actually quantitative measures of the more qualitative Degree of Breakage classification, rather than the more traditional definitions of these parameters.

To gain a first hand appreciation of the mechanical properties of the various rock types and to independently check the logging techniques used by Minnova, a limited amount of geotechnical core logging was conducted by Piteau Associates. Approximately 620m of core from four holes was logged for Degree of Breakage, Hardness and Degree of Alteration. Logging data were compared with data from identical holes logged by Minnova. Results are illustrated graphically in Fig. 12, where it can be seen that, in terms of Degree of Breakage, good correlation exists between Minnova and Piteau Associates data for Degrees of Breakage greater than about C, with Minnova's results tending to be one to two catagories lower than Piteau Associates results. For Degree of Breakage less and C, correlation is unclear.

Good correlation is also observed in Fig. 12(b) between Minnova logged RQD and Piteau Associates logged Degree of Breakage, for RQD and Degree of Breakage greater than about 15% and C, respectively. Based on this correlation, RQD and Degree of Breakage data was subdivided into three catagories: Low (RQD $<$ 15%, Degree of Breakage $< C^+$); Moderate (RQD 15 - 50%, Degree of Breakage C^+/D^-); and High (RQD $>$ 50%, Degree of Breakage $> D^-$).

To evaluate the variability of RQD with rock type and location within the rock mass, a statistical assessment of RQD was carried out for those drillholes which project to the five geotechnical sections in Figs. 4 to 8. RQD data were assessed using the Cumulative Sums statistical analysis technique developed by Piteau and Russel (1971). Average values of RQD and corresponding catagories are illustrated on the geotechnical sections.

3.1.6 Rock Mass Competency

The stability of a natural or man-made slope depends on a complex interaction of a wide variety of physical and mechanical characteristics of the slope forming materials, many of which are difficult or impossible to evaluate quantitatively. However, a qualitative assessment or classification of rock mass quality or competency which incorporates several important, relatively easily obtained rock mass parameters is useful in evaluating potential slope behaviour. For the Samatosum Project, an evaluation of general rock mass competency was based on inspection of diamond drill core from a variety of drillholes, detailed evaluation of geotechnical core logging by Minnova and Piteau Associates, results of rock strength testing, geologic interpretations and descriptions of the various rock types by Minnova and observations of general rock mass conditions in the Rea Gold adit.

Based on this information, the rock mass was subdivided into two basic rock mass units, within which the basic mechanical properties and anticipated rock mass behaviour are expected to be similar. Unaltered to lightly altered Mafic Pyroclastics (including Mafic Tuffs), which overly the ore zone and form the structural hanging wall of the deposit, are considered to form one rock mass unit. Assessment of RQD information on the geotechnical sections in Figs. 4 to 8 indicates these rocks have variable RQD/Degree of Breakage (see Fig. 13a). Based on RQD, Degree of Breakage, Degree of Alteration, susceptibility to weathering and deterioration, intact rock strength, etc., these rocks are expected to be the most competent in the pit, exhibiting an overall moderate to good rock mass competency.

All other rock types, including moderately to strongly altered sericitic Mafic Tuffs, Muddy Tuffs, Cherts and Sediments, collectively form the second rock mass unit. Insufficient information was available to allow further definition or subdivision of this rock mass unit based on rock type. While assessment of RQD information in these rocks indicates a similar variability in RQD/Degree of Breakage as the Mafic Pyroclastics and related rocks (see Fig. 13b), these rocks are expected to be generally less competent than the Mafic Pyroclastics, due primarily to their anticipated variable weathering and alteration characteristics and generally lower intact strength. Overall rock mass competency is anticipated to be poor to moderate in these rocks.

3.2 SURFICIAL SOILS

3.2.1 Regional Surficial Geology

Surficial geology of the Samatosum Project area is dominated by glacially deposited soils (moraines). The moderately steep northwestern slope of Samatosum Mountain is covered by a relatively thin veneer of morainal materials (i.e. glacial till) with local accumulations of colluvium derived primarily from reworked glacial deposits and weathered bedrock. Morainal soils generally thicken downslope towards Johnson Creek, forming somewhat hummocky blanket deposits on the lower slopes of Samatosum Mountain.

The Johnson Creek Valley is a "U" shaped hanging valley that was probably formed by a mountain glacier which discharged towards the south into the Sinmax Creek Valley. The current base elevation of the Sinmax Creek Valley is about 250 to 350m lower than the Johnson Creek Valley.

Northwest of Johnson Creek, the terrain is characterized by hummocky morainal veneer and blanket deposits with local accumulations of colluvium and organics in poorly drained depressions. Colluvial and morainal veneers cover the steeper slopes on the northeast flank of Samatosum Mountain and north-northwest of Johnson Lake.

The only clearly identified potential source of clean construction aggregate in the study area is a gravelly kame terrace deposit on the north side of Sinmax Creek, just west of Alex Creek. Other sources may include local kame deposits in Johnson Creek, Haggard/Blomely Creeks north of the site and Samatosum Creek south of Johnson Lake; however none of these potential sources have been confirmed, and substantial deposits of clean sand and gravel in these areas seem unlikely.

3.2.2 Site Surficial Geology

Reconnaissance mapping and limited test pitting was conducted in the vicinity of the proposed waste dump and tailings impoundment sites on the northwest flank of Samatosum Mountain (see Fig. 2). As noted above, reconnaissance work was conducted when the bulk of the site was relatively inaccessible due to snow cover. Additional reconnaissance and possibly additional test pitting are recommended to confirm soil conditions and general site characteristics once the snow has melted.

Limited laboratory testing of samples obtained from the test pits was also conducted. Detailed test pit logs are given in Appendix A, and laboratory test results are summarized in Fig. 14 and on the test pit logs. Based on this information, and in the context of the regional setting, three distinct soil horizons were identified and are summarized in Table II and described below. In addition, thickness of soil overburden in the pit area and proposed waste dump sites was estimated based on depth of casing in diamond drillholes in the vicinity. Approximate overburden isopachs are given on Fig. 2.

i) Organic Soils

Organic soils consist of reddish brown to black, amorphous to fibrous peat, roots and organic silt to sandy silt. These soils are commonly soft and compressible and form a mat or veneer ranging from less than 10cm to greater than 1.0m in thickness. The thickest organic accumulations occur in isolated, poorly drained depressions in the underlying moraine or bedrock. In such areas these soils are commonly saturated and very weak. On shallowly to moderately sloping ground, which forms the bulk of the site, organic soils tend to be relatively dry and less than 30 cm in thickness. Pocket penetrometer testing of organic soils indicated compressive strengths of generally less than 0.5 to 1.5 kg/cm^2.

Occasionally, organic soils may be intermixed with colluvium derived from the weathered bedrock or reworked morainal deposits which commonly underly the organic veneer.

ii) Colluvium/Reworked Morainal Deposits

Beneath the organic veneer and overlying more competent morainal soils or bedrock is a zone of up to about 2.0m of weathered or altered morainal material (i.e. Till) or colluvium. This material varies from light to medium reddish or greyish brown, is commonly compact to dense and ranges from moist to dry. It is generally well graded, ranging from sandy silt to sandy gravel with occasional cobbles and boulders to 30cm in diameter and a trace of clay size particles. Coarser constituents are commonly strongly weathered, subrounded to angular bedrock fragments. Organic fibres and roots are occasionally present. Pocket penetrometer testing in these materials indicates compressive strengths range from 1.0 kg/cm^2 to greater than 5.0 kg/cm^2.

iii) Morainal Deposits

Greyish brown to grey, compact to very dense, competent glacial till commonly underlies the weathered till or colluvial veneer and overlies bedrock. This material is generally well graded, with a similar range of grain sizes as indicated for the weathered morainal deposits described above. Roots and organic fibre are generally absent and coarser constituents are usually less weathered than those described above. Thickness varies from zero at bedrock outcrops to at least 3m (i.e. maximum depth of test pits) and possibly up to about 50m on the lower slopes and in the valley floor (i.e. based on depth of drill casing). These tills are generally hard (pocket penetrometer $>$ 5 kg/cm^2) and dry (water content $<$ 10%). However, wetter zones, generally associated with sand and gravel lenses, and softer, finer grained zones do occur locally.

4. HYDROGEOLOGY

4.1 REGIONAL HYDROGEOLOGY

Eagle Bay Formation rocks which underlie much of the project area are generally metavolcanic and metavolcanoclastic rocks which are expected to have generally low hydraulic conductivities. One major exception to this could be the Tshinakin Limestone, which is exposed in cliffs at the outlet of Johnson Lake. While the limestone could represent a significant aquifer, directing groundwater flow to either Johnson or Adams lakes, other rocks are expected to be aquitards, allowing only very slow rates of groundwater flow. The dominant direction of groundwater flow is expected to be in the northwest/southeast directions, approximately perpendicular to the axes of the Johnson Creek Valley which controls surface drainage in the area.

Surficial sediments in the study area are predominantly very dense tills which are expected to be of low permeability. A thin soil veneer which overlies the dense, unweathered till will conduct groundwater; however, the limited thickness of the soil profile should limit flow quantities to very small amounts. While a major gravel aquifer is known to exist in the Sinmax Valley (based on the log of a well drilled for the Homestake Mine), similar deposits are not expected to exist in the hanging Johnson Creek Valley, or on the valley slopes.

4.2 HYDROGEOLOGICAL UNITS

Very little information is currently available on the hydrogeological properties of the rock and soil units at the site. The following discussion is based on a review of the engineering geology of the site, and on observations of seepage in the Rea Gold adit.

i) Bedrock

The rock mass is expected to be highly anisotropic, having a hydraulic conductivity of about 10^{-9} to 10^{-8}m/s parallel to foliation, and a hydraulic conductivity of about 10^{-11} to 10^{-9}m/s across foliation. As hydrogeologically relevant data is presently very limited, it is not possible to differentiate between various rock types on the basis of hydrogeological properties.

ii) Shallow Soil Horizon

The shallow soil horizon consists of the organic soils and reworked morainal or colluvial deposits described in Section 3.2. This soil horizon is expected to be a moderately permeable (i.e. hydraulic conductivity of 10^{-5} to 10^{-4} m/s) material, but of very limited depth.

iii) Morainal Deposits

As indicated in Section 3.2, morainal deposits of glacial till exist over the entire mine area in thicknesses ranging from a thin veneer on upper bedrock slopes, to about 50m in the centre of the Johnson Creek Valley. The high density and well graded nature of the till are indicative of a low permeability porous media. Hydraulic conductivities of this unit are expected to range from about 10^{-8} to 10^{-6}m/s.

4.3 LOCAL GROUNDWATER FLOW

The local groundwater flow system is expected to be fairly simple, with groundwater recharge occurring on the top and flanks of Samatosum Mountain, and discharge occuring in the major valleys. A portion of the recharge on Samatosum Mountain will therefore flow under the proposed open pit, waste dump and tailings sites, to discharge in the Johnson Creek Valley.

There are many poorly drained areas on the slope in the mine area, as well as a small pond near the top of the mountain. These conditions are indicative of the low permeability material which underlies the area. The majority of the precipitation will ultimately become evapotranspiration from the area, or run off over the ground surface. Groundwater flow is expected to represent only a very small proportion of the precipitation which recharges this area. It is estimated that only about 15% of average annual precipitation will infiltrate as recharge to the groundwater flow system, and that less than one third of the actual recharge (i.e. 5%) will become groundwater flow through bedrock.

4.4 GROUNDWATER IMPACT ON MINE DEVELOPMENT

4.4.1 Groundwater Inflows to the Pit

Groundwater inflows to the pit are expected to be very small. In low permeability rock, slope drainage does not readily occur; hence, inflow calculations based on steady-state flow as controlled by recharge considerations usually provide a reasonable estimate of seepage quantities. The maximum recharge area above the proposed pit is approximately 2.5×10^6 m^2 (see Fig. 1). Annual precipitation is expected to be slightly more than the 442mm/year recorded at Barriere. Assuming a precipitation rate of 500mm/year and an infiltration rate of 5%, steady-state groundwater seepage into the pit is estimated to be about 2 L/s. As the infiltration rate is likely to be less than 5% and the recharge area is likely to be smaller than $2.5 \times 10^6 m^2$, the 2 L/s rate calculated above should be considered as an upper bound estimate.

4.4.2 Piezometric Levels in Pit Slopes

Due to the expected low permeability of the rock mass, very limited natural depressurization of the slopes is expected to occur over the life of the mine. Current piezometric levels will be determined in an upcoming hydrogeological investigation; however, high groundwater conditions should be anticipated.

4.5 GROUNDWATER QUALITY

Groundwater has been sampled from a seep near the top of Waste Dump Site A. This groundwater can be characterized as a hard (238 mg/L as $CaCO_3$), slightly alkaline water (pH = 7.6 to 8.5) with a calcium-magnesium bicarbonate-sulphate chemistry.

Nutrients and metals are present in the groundwater at only very low concentrations and there are no problems anticipated with the disposal of any of this water if it is intercepted outside the waste dump or pit areas.

5. . OPEN PIT SLOPE STABILITY

5.1 BASIC SLOPE DESIGN CONSIDERATIONS

The main objective of any slope design is to provide safe and efficient overall slopes, as well as benches. In rock slopes, instability of individual benches or the overall slope may result from failure along structural discontinuities, such as joints, faults, etc. In fractured, weak or altered rock, slopes may be subject to significant degradation, ravelling and rockfall generation which could result in unsafe working conditions. Overall slopes may also be subject to deep seated instability as a result of failure through the rock mass.

In terms of the proposed Samatosum open pit, we understand that the steepest, highest slopes (i.e to about 150m) would be excavated primarily in the relatively competent Mafic Pyroclastics in the hanging wall. In addition, the principal plane of weakness (i.e. foliation) dips obliquely into the hanging wall slope. Thus, deep seated failure of the rock mass is considered unlikely and detailed deep seated stability assessments were not carried out. Stability assessments described below were directed towards evaluating kinematically possible failures involving discontinuities and the potential for ravelling and rockfall generation on benches. Subsequent preliminary design rationale were based on these evaluations.

5.2 ASSESSMENT OF KINEMATICALLY POSSIBLE FAILURES

When assessing failure mechanisms related to structural discontinuities (i.e. kinematic failure mechanisms), the most important factors to consider are the orientation, geometry and spatial distribution of discontinuities in the slope. To determine which failure modes are kinematically possible, it is necessary to evaluate all possible combinations of discontinuities with respect to both the orientation and alternative possible angles of the proposed pit slope. As conceptual mine plans with proposed slope orientations have yet to be prepared, kinematic assessments were conducted for twelve discrete Design Sectors which cover the full range of possible slope orientations. The first design sector (i.e Design Sector I) was assumed to be excavated as a footwall slope, striking parallel to the peak orientation of foliation and dipping in the same direction as foliation (i.e. slope azimuth 025°). Additional design sectors were chosen at 30° azimuth intervals, clockwise from 025°. Design sector designation, type of slope and slope azimuth are summarized in the first three columns of Table III.

Lower hemisphere equal area projections of planes representing peak orientations of discontinuity sets summarized in Fig. 10 were used to define all kinematically possible failure modes in each design sector (see Appendix B for lower hemisphere projections). Two basic types of kinematically possible failure modes were identified: wedge failures and plane failures. A kinematically possible wedge failure is defined as a block formed by the intersection of two discontinuities and which could slide along the line of intersection if its apparent plunge is undercut by the slope. A kinematically possible plane failure is defined as a block which could slide along a single discontinuity if its apparent dip is undercut by the slope and a suitable lateral release feature exists.

Simple limit equilibrium stability analyses were conducted for each kinematically possible failure mode assuming dry slope conditions and discontinuity shear strengths summarized in Section 3.1.4. Two separate sets of analysis were conducted assuming all discontinuities were joints and faults, respectively. Calculated factors of safety are indicated on the projections in Appendix B.

Not all kinematically possible failure modes have the same impact on slope stability or slope design. Factor of safety of some modes may be such that failure is unlikely even under severe conditions, such as fully saturated slopes. Other failure modes may be so oblique to the slope to be of limited practical importance in terms of size and shape. Still others may be formed on discontinuities which are only weakly developed, and hence, unlikely to occur. Therefore, an evaluation of which kinematically possible failure modes are critical to slope design is necessary.

For purposes of preliminary stability assessment and design, critical failures were defined as kinematically possible failures which have lower bound Factors of Safety of 1.0 or less, based on stability analyses conducted assuming all discontinuities involved are formed on faults. In addition, the line of intersection of wedge failures or dip direction of plane failures must be less than about 45° to 50° oblique to the slope azimuth. Critical plane and wedge failures defined in this manner are indicated on the projections in Appendix B and summarized in Table III.

Based on the defined critical failure modes, and considering the importance or intensity of the various discontinuity sets involved, an assessment of the apparent plunge or dip of failure likely to control bench stability, and a qualitative assessment of the degree of kinematic control were made. Results of these assessments are indicated on the projections in Appendix B and summarized in Table III. It must be appreciated that because no benches have yet been exposed whereby the actual rock mass behavior may be observed, these assessments are based largely on engineering judgement, and must be considered preliminary in nature. Once mining has commenced and benches are available for documentation, confirmation and refinement of preliminary assessments should be conducted.

5.3 SLOPE GEOMETRY BASED ON KINEMATIC ASSESSMENTS

Based on results of kinematic assessments described above, alternative possible slope geometries designed to control kinematically possible failures were prepared. Slope geometry parameters are illustrated in Fig. 15. Alternative slope geometries were prepared for both 10m high single benches and 20m high double benches, assuming 90^{0} design bench face angles. Benches were assumed to breakback based on the apparent dip or plunge of failure considered to control bench stability (i.e. w). Two breakback scenarios were examined. Breakback Scenario A assumes bench crests and effective bench face angles completely breakback to w. This scenario is based on the assumption that all discontinuities involved in a given failure are planar and continuous. Scenario B assumes that only one-half the breakback indicated for Scenario A occurs. Implicit in Scenario B is the assumption that at least some of the discontinuities involved in a given failure are discontinuous or partly healed and that failures occur as stepped features rather than simple blocks. Stepped failure mechanisms are commonly observed in anisotropic rock masses, such as that at Samatosum, where a relatively pervasive plane of weakness (eg. foliation) offsets or otherwise limits the continuity of other discontinuity features.

Design berm width was calculated by adding a mininmum berm width for access and rockfall catchment to the assumed breakback. For 10m high single benches, a minimum berm width of 8m is considered adequate for preliminary assessments. For 20m high double benches, a minimum 10m wide access/rockfall catchment berm is recommended. Based on bench height, bench face angle and design berm width, intermediate (i.e. bench crest to bench crest) slope angles were calculated. Alternative bench geometry parameters are summarized for each design sector in Table III.

5.4 ASSESSMENT OF RAVELLING AND ROCKFALLS

Even the most carefully designed and excavated slope will be subject to occasional small failures, rockfalls and ravelling. The degree to which a given slope will be subject to such failures depends partly on kinematic considerations as described above and partly on the general rock mass competency. Zones of poor rock mass competency can be expected to be more susceptible to deterioration, ravelling, generation of rockfalls, etc., than zones of good rock mass competency.

As described in Section 3.1.6, two basic rock mass units were identified: unaltered to lightly altered Mafic Pyroclastics and Mafic Tuffs of relatively moderate to good rock mass competency; and lightly to strongly altered Muddy Tuff, Sericitic Mafic Tuff, Sediments, Cherts and related rocks of relatively poor to moderate rock mass competency. Based on the likely relative positions of the various pit walls and design sectors with respect to topography and geologic interpretations provided by Minnova, an assessment of which rock mass units may occur in each design sector, together with the anticipated rock mass competency, was carried out. Results of this assessment are summarized in Table III.

5.5. PRELIMINARY OPEN PIT SLOPE DESIGN

5.5.1 Slope Geometry

Recommended preliminary slope designs for each design sector are summarized on the right side of Table III. These slope designs are based on our assessment of kinematically possible failures and alternative bench geometries, rock mass units and rock mass competency for each design sector. Adjacent design sectors which exhibit similar kinematic controls and rock mass competency have been grouped together to simplify the design. Where more than one rock mass unit may occur within a given design sector, separate designs for each rock mass unit have been prepared which reflect the relative difference in anticipated rock mass competency.

As indicated in Table III, slope design in Design Sectors I, II and III is controlled by foliation discontinuities. Rock mass competency in these design sectors is anticipated to be poor to moderate. Single, 10m high benches are recommended to limit the size of potential failures, and inclined (70^{o}) bench face angles are recommended to reduce the potential for rockfall generation and ravelling which may occur due to the relatively incompetent nature of the rock mass in these design sectors. Recommended berm widths of 8m yield 41^{o} intermediate slopes.

In general, 90° design bench face angles and 20m high double benches are recommended in moderate to good competency rock in Design Sectors IV through X. Berm width varies from 15 to 19m and intermediate slope angles range from 47° to 53°. In poor to moderate competency rock, 70° design bench face angles are recommended to reduce potential rockfall generation and allow better control of ravelling. Design berm widths range from 12 to 14m and intermediate slope angles range from 43° to 46° in the less competent rocks in these design sectors.

In Design Sectors XI and XII, little kinematic control is evident; hence, double, 20m high benches are recommended. Inclined bench faces (70°) and 12m wide berms, yielding maximum intermediate slope angles of 46°, are recommended to reduce rockfall generation and ravelling which may occur due to the relatively incompetent nature of the rock mass in these design sectors.

5.5.2 Blending and Modification of Recommended Slope Designs

In general, the recommended slope design for a given design sector should not be exceeded in transition zones between design sectors. However, it is considered feasible to steepen small sections of the slope and accept the possibility of increased failures on berms in small areas where the impact is minimal.

5.5.3 Controlled Blasting and Ripping

Controlled blasting or ripping of final slopes is recommended, particularly in areas of relatively poor rock mass competency, to minimize breakback and damage to the rock mass. In this regard, some provision should be made early in the mine life for blasting and ripping trials to determine the optimum excavation technique.

5.5.4 Scaling and Cleaning of Berms

Benches should be thoroughly scaled immediately following excavation. Depending on slope behavior, additional scaling and periodic cleaning of berms may be required to minimize potential rockfall hazards and maintain berms as effective rockfall catchments.

5.5.5 Remedial Measures

As indicated previously, little is known concerning the characteristics, orientation or continuity of major faults. If such features do occur and are adversely oriented, they could result in failures involving several benches or the whole slope. It is therefore critical that such features be identified as quickly as possible so that remedial measures may be adequately planned. Remedial measures could consist of artificial support, slope flattening, buttressing, or slope depressurization. The need for remedial measures should be assessed as potential stability problems are identified.

5.5.6 Groundwater and Surface Water Control

Stability analyses and slope designs described above are based on fully depressurized or drained slopes. Provided adequate surface water and shallow groundwater seepages are controlled via interception trenches and graded ditches, benches are expected to be generally well drained. Some shallow groundwater depressurization or drainage measures may be required for the lower benches.

In terms of larger scale, deep seated failures involving major faults, groundwater depressurization may be necessary to ensure adequate stability. The need for depressurization should be assessed once potential stability problems are identified.

5.5.7 Slope Monitoring

All slopes should be inspected regularly for signs of distress and overall slope movement. If movements are observed, more sophisticated monitoring techniques should be employed to assess the nature of the movements.

5.5.8 Geotechnical Mapping, Slope Documentation and Ongoing Analyses

An ongoing program of geotechnical mapping, core logging, data assessment, slope documentation and assessment of slope behavior should be conducted throughout mine development and mining. This work is particularly important to identify and assess the occurrence, orientation, and physical characteristics of major faults which may affect slope stability. Mapping and documentation may also serve to warn of possible changes in geologic structural conditions or other unfavourable conditions which may exist in the slope. Updated geotechnical information can be used to confirm or modify the slope designs as mining proceeds.

6. WASTE DUMP, TAILINGS POND AND MILL SITES

Detailed waste dump and mill site layouts were not available for this study. Similarly, final tailings pond and sedimentation pond configurations have not been selected. Alternative site locations illustrated on plan in Fig. 2 are based on preliminary discussions with Minnova personnel. The discussion below is intended to assist with the detailed planning of the waste dumps and other facilities by providing general geotechnical guidelines, concepts and considerations which can be input into the site selection and detailed design process.

6.1 WASTE DUMPS

6.1.1 Foundation Conditions and Material Properties

The stability of the waste dumps will be controlled by the strength of the surficial soils and bedrock materials on which the dump will rest, topography, groundwater conditions, waste rock properties and, to some extent, the method of dump placement. As discussed in Sections 2.1 and 3.2, the proposed waste dump sites illustrated in Fig. 2 are located just west (i.e. Site A) and just north (i.e. Site B) of the proposed pit. The natural slopes in both of these areas are generally favourable, being in the range of 15^{o} to 20^{o} with only a few local areas being as steep as about 25^{o}. The lower approximately 500m of the valley side typically slopes at about 10^{o} to 15^{o} (see Figs. 2, 16 and 17).

Subsurface conditions at Waste Dump Sites A and B are also generally favourable. While an organic "topsoil" layer is present over the sites, this layer is for the most part less than 30 cm thick and should not have a significant affect on the overall stability of a dump. The weathered till/colluvium and underlying hard glacial till described in Section 3.2.2 are both considered to be competent, suitable foundation materials for a waste dump.

Most of the seepage observed in the test pits occured within the surface organic soils, above the weathered till/colluvium. Only occasional evidence of slight seepage at the weathered bedrock contact or in the weathered till/colluvium was observed. Based on the generally dense and well graded nature of these soils, little or no water flow within the soils underlying the organics is expected. Surface water flow within the small drainage courses within the waste dump sites is expected to be intermittent.

Based on the nature of the waste rock and on experience from other waste dumps, a minimum repose angle of 37° may be anticpated for the waste rock. While little difference in behaviour is expected between the two main rock mass units, the Mafic Pyroclastics may be more resistant to general weathering and breakdown. However, some general mechanical degradation or slaking of all rock types is anticipated.

6.1.2 Assessment of Waste Dump Stability and Design Guidelines

Based on the above, it is concluded that either of the two proposed sites would be acceptable for a waste dump. While the exact configuration (i.e. volume, height, etc.) of the waste dump has not been determined, it is anticipated that the dump would be built in a series of terraces or wrap-arounds as opposed to a single dump lift. This method of dump construction, which would probably result in a dump thickness of no greater than about 50m in any one area, is considered favourable in terms of waste dump stability.

To prevent as much surface water or shallow groundwater as possible from entering the dump or the dump foundation, it is recommended that an interceptor ditch be excavated above the dump. Such a ditch should be excavated into the dense and relatively impervious glacial till, and collected water should be directed away from the dump and into an adjacent drainage course.

Surface water and shallow groundwater that is not collected by the interceptor ditch will tend to be concentrated in the few poorly defined drainage courses. To encourage free drainage of the waste dump, it is recommended that the most durable and blocky waste rock be placed in the drainage courses.

Logging, clearing and grubbing of the waste dump sites prior to dumping is not recommended. As has been reported for other waste dump sites in B.C., disturbance of the upper soils by heavy equipment results in remolding of the materials. In the presence of even a relatively small amount of water, these silty remolded soils become very soft, losing virtually all of their strength. Considerations should be given to locating the ultimate waste dump toe within forested sections of the slope as the trees may serve to bind the organic mat or veneer to the underlying soils as well as provide some buttressing effect to the toe, at least in the short term. Vegetation cover also tends to result in drier subsurface conditions due to evapotranspiration.

While overall instability of a waste dump in either of the proposed sites indicated on Fig. 2 is not anticipated, there is always the possibility that small failures could occur. Thus, it is recommended that suitable dump monitoring (eg. wireline extensometers, visual inspections, etc.) be incorporated into the dumping plans. Should a dump failure occur, the flatter slopes below about elevation 1100m (see Figs. 2, 16 and 17) would tend to limit the runout of any such failure.

Notwithstanding the apparent suitability of the site, a further site reconnaissance, and possibly test pitting, should be carried out once the snow cover has melted and before dump plans are finalized.

6.2 ASSESSMENT OF TAILINGS POND SITES

The two conceptual tailings pond sites shown in plan on Fig. 2 and in section on Figs. 17 and 18 are significantly different from a topographic and construction standpoint. Tailings Impoundment Site B, located on a sideslope downhill of the open pit and Waste Dump Site B, would require the construction of a three-sided structure to provide the required containment. Tailings Impoundment Site A, on the other hand, by virtue of its shape, would require only the closing off of one end of a natural draw or basin to provide the required containment. Thus, the borrow material and construction requirements for a tailings dam and pond appear to be far less if Tailings Impoundment Site A is utilized.

Based on observations in test pits in the area of the two sites, it is concluded that subsurface conditions are fairly similar. That is, a thin (i.e. typically $<$ 30cm thick) layer of organic topsoil is typically underlain by a well graded, compact to very dense silty sandy glacial till. While no percolation tests have been completed in this material to date, it is anticipated that the permeability of this material is relatively low (i.e. 10^{-8} to 10^{-6}m/s), indicating that it would form a suitable seal for the base of the pond.

Based on results of the preliminary investigation discussed above, it is concluded that both proposed sites would be geotechnically suitable for tailings ponds. However, Tailings Impoundment Site A has the advantage of requiring considerably less construction to form a fully enclosed impoundment. In addition, Tailings Impoundment Site B has the disadvantage of being located directly downslope of a potential waste dump site. A detailed topographic survey of any tailings pond site is necessary to allow an accurate assessment of the volume of tailings which could be contained behind a dam of a given height. Further geotechnical investigations required to finalize a tailings pond and dam design are discussed below in Section 7.4.

6.3 ASSESSMENT OF PROPOSED MILL SITE

As illustrated on Fig. 2, one proposed mill site is located just west of Tailings Impoundment Site A, on top of a height of land which at present is heavily forested. While it was not possible to determine subsurface conditions in this area during the field program, it is anticipated that bedrock would be very near surface, and that it would be possible to found the mill on bedrock at this site. It is noteworthy that bedrock was observed on the logged, cone-shaped hills just east of Tailings Impoundment Site A. Further reconnaissance of this area should be conducted once the snow has melted. Test pitting or other investigations may also be necessary, depending on the results of the field reconnaissance. Other mill sites may also be suitable, however, none were specifically addressed for this study.

6.4 ASSESSMENT OF POSSIBLE SEDIMENTATION POND SITE

Should a sedimentation pond be required at the site, it would most likely be situated downslope of the open pit and waste dump in the vicinity of, or just west of, Tailings Impoundment Site B. Test pits excavated in the general area (i.e. Test Pits 11, 12 and 13) indicate that favourable foundation soils (i.e. dense glacial till) are present for such a facility. Further field reconnaissance and test pitting are likely required once a final location and configuration for the sedimentation pond has been selected.

7. ADDITIONAL GEOTECHNICAL STUDIES

7.1 ADDITIONAL SITE RECONNAISSANCE

Field invesigations for this study were conducted during a period when the terrain was snow covered. Hence, site access was restricted and limited opportunity was available to observe general site conditions and soil/bedrock characteristics. We therefore recommend that additional site reconnaissance, and possibly additional limited test pitting and geologic structural mapping be conducted by Piteau Associates once the snow cover has melted. Information obtained from this site visit would be assessed and used to verify or update recommendations contained herein, if required.

7.2 ASSESSMENT OF PRELIMINARY MINE PLANS

The preliminary design criteria given herein should be used as a basis for developing detailed mine plans, slope designs and waste dump designs. Once these designs and plans have been prepared, they should be reviewed by Piteau Associates prior to commencement of mining.

7.3 VERIFICATION AND UPDATING OF DESIGN CRITERIA

As indicated previously, many of the assessments described herein are based on engineering judgement and assumptions regarding rock mass and soil foundation behaviour. It is of critical importance that these assumptions and design criteria be verified at an early stage of mine life. In this regard, we recommend that initial mining slopes and waste dump construction be examined and documented by experienced geotechnical personnel. This information should be used as a basis for verifying and updating assumptions and design criteria presented herein. Periodic reviews of slope and waste dump behaviour by qualified geotechnical personnel are recommended throughout mine life.

7.4 DETAILED TAILINGS POND AND MILL SITE INVESTIGATIONS

Once tailings pond and mill sites have been selected and preliminary site layouts have been prepared, detailed investigations of site conditions should be conducted to confirm site suitability and provide information required for detailed facility design. These investigations could include: detailed topographic surveying, site reconnaissance and mapping, test pitting, trenching and possibly drilling.

7.5 ASSESSMENT OF BORROW SOURCES

Further assessments of potential borrow sources will likely be needed once borrow requirements are delineated. As discussed in Section 3.2.1, the only clearly identified potential source of clean construction aggregate in the study area is located in the Sinmax Creek Valley. Based on airphoto interpretation, closer borrow sources of significant volume appear unlikely. However, further field reconnaissance, and possibly test pitting should be carried out to investigate the matter further. With regard to borrow material for a tailings dam, the silty glacial till that is present across the site is felt to be ideal for such a structure.

8. ACKNOWLEDGEMENTS

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Thanks are also extended to Rea Gold Ltd. and Mr. D. Blanchflower of Minorex Consulting Ltd. for permission to inspect and map the adit on the adjacent Rea Gold Concession, and to Mr. M. McFaddyen of Dolmage Campbell & Associates for familiarizing us with the geology and existing documentation of the adit. Mr. Blanchflower also provided copies of recent aerial photography for our inspection.

Respectfully submitted,

PITEAU ASSOCIATES LIMITED

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March 3, 1988

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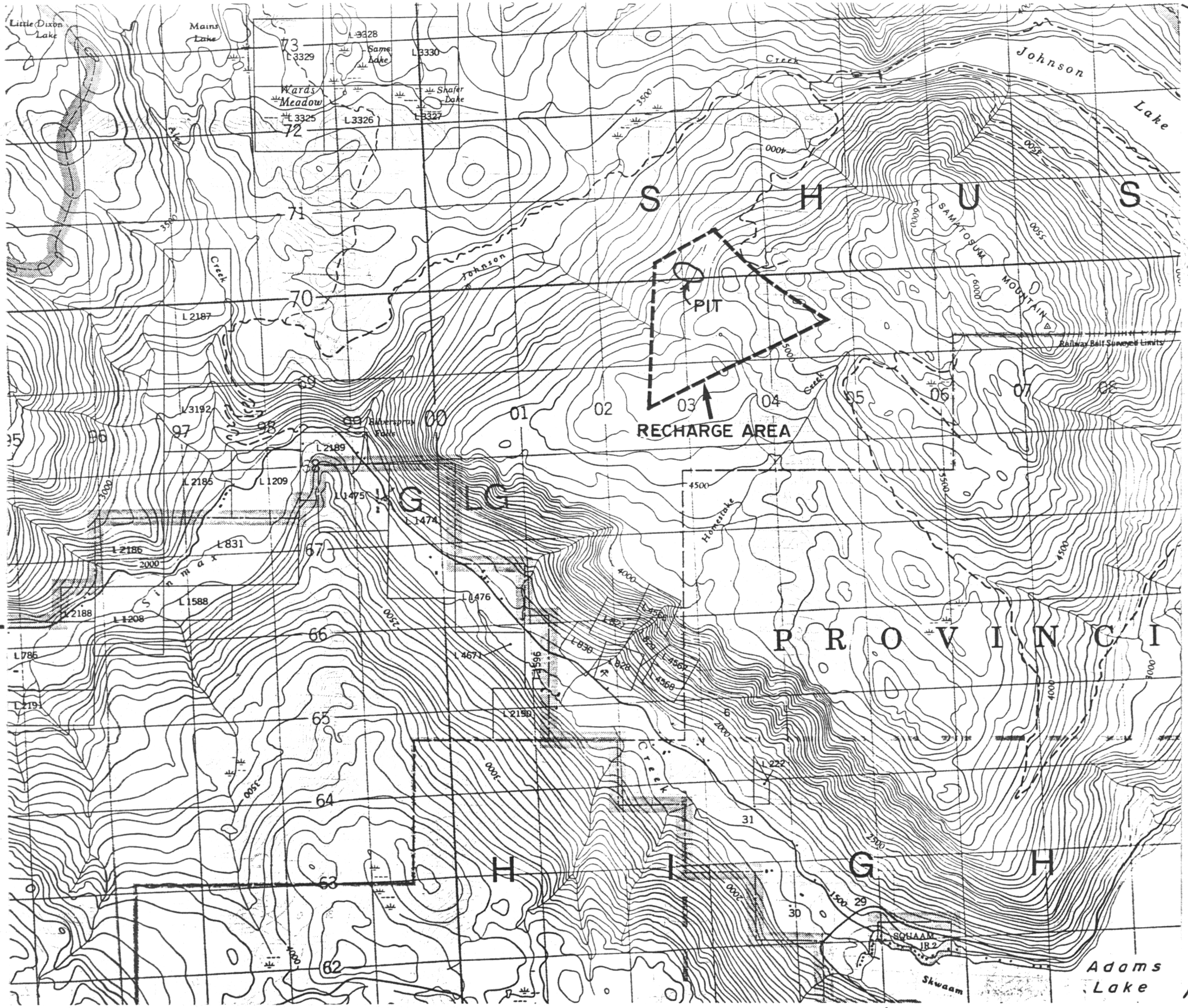
JOB NUMBER



To Hwy. 5

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To Hwy. 1

inches

centimetres

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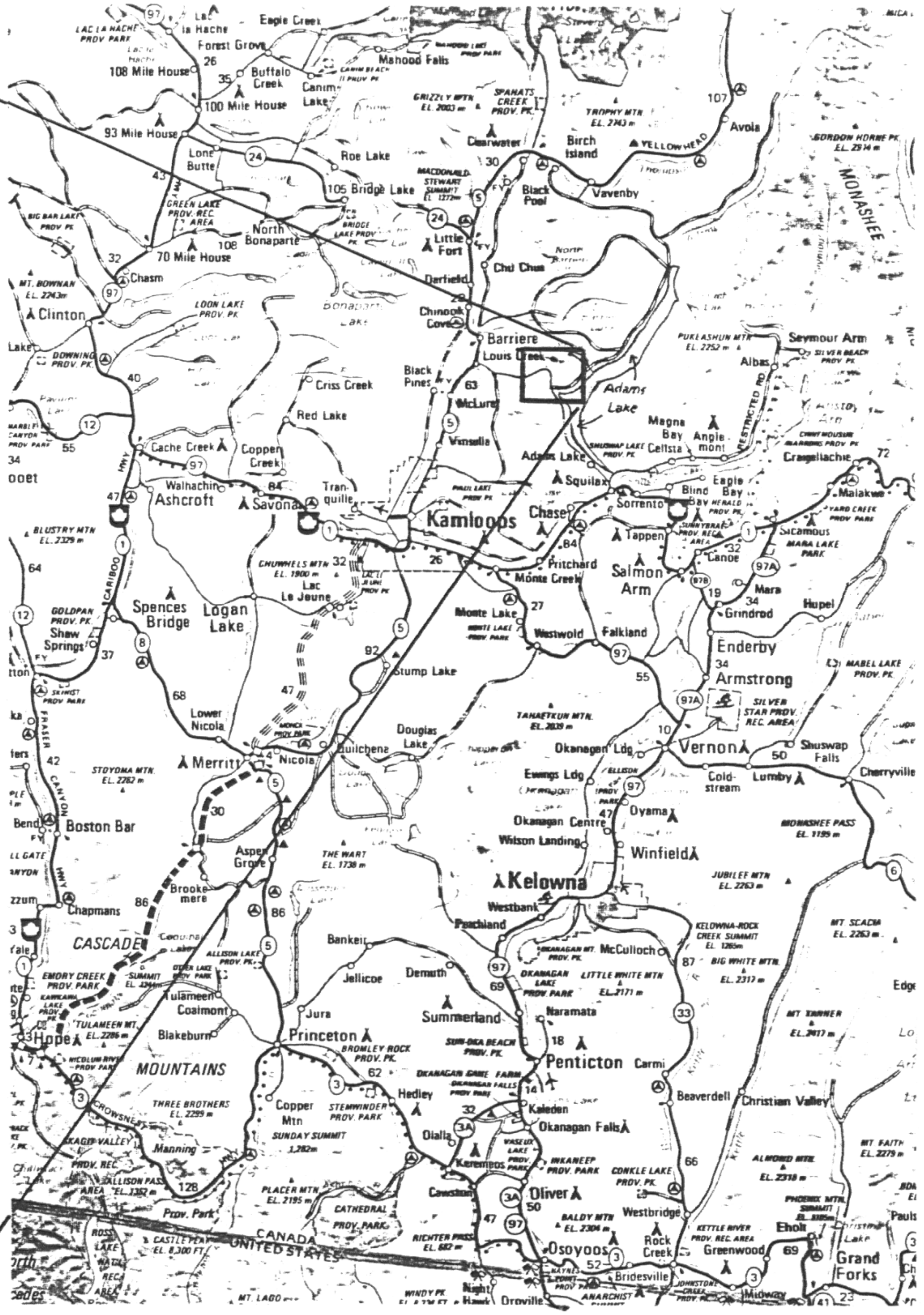
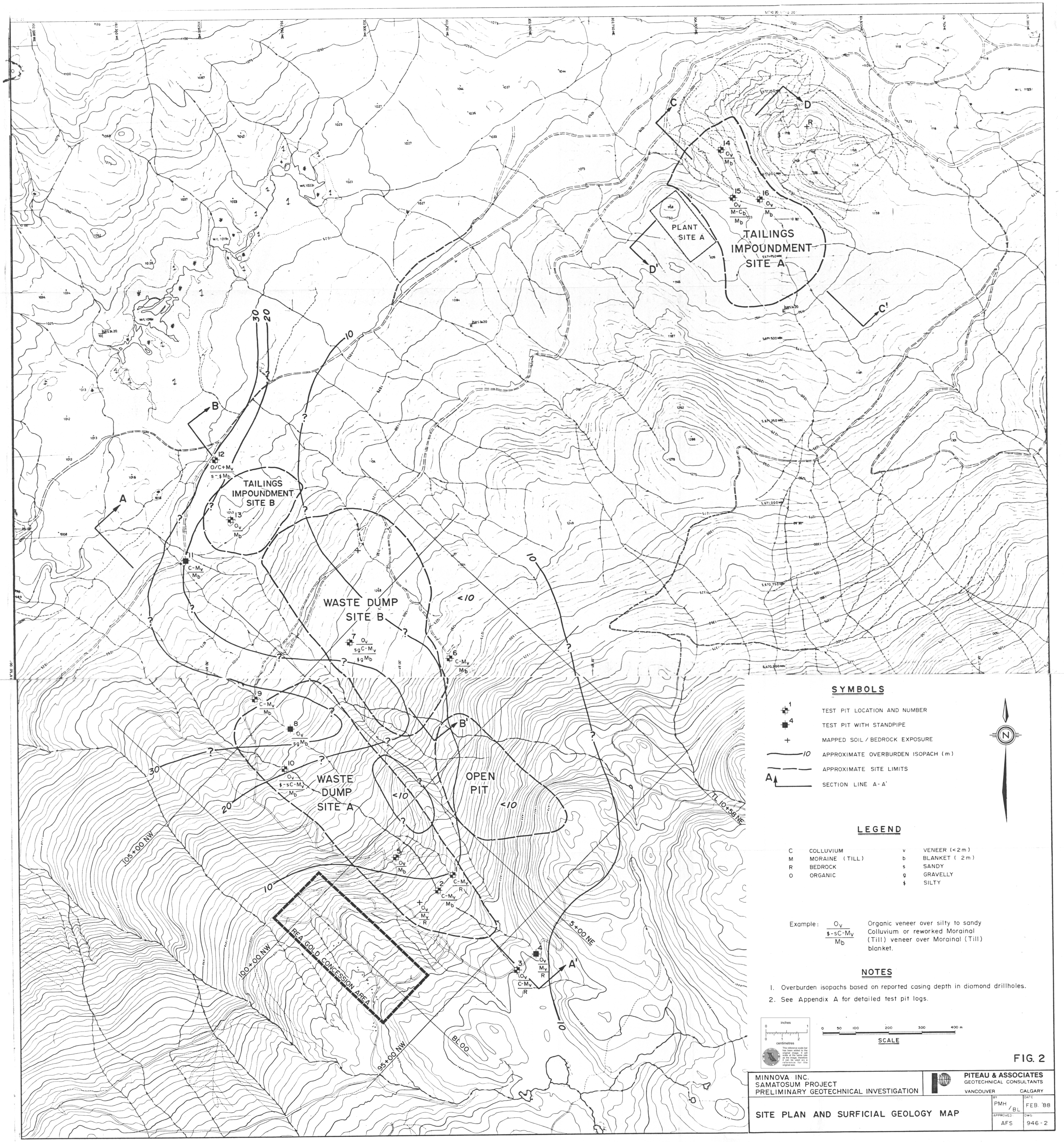


FIG. 1

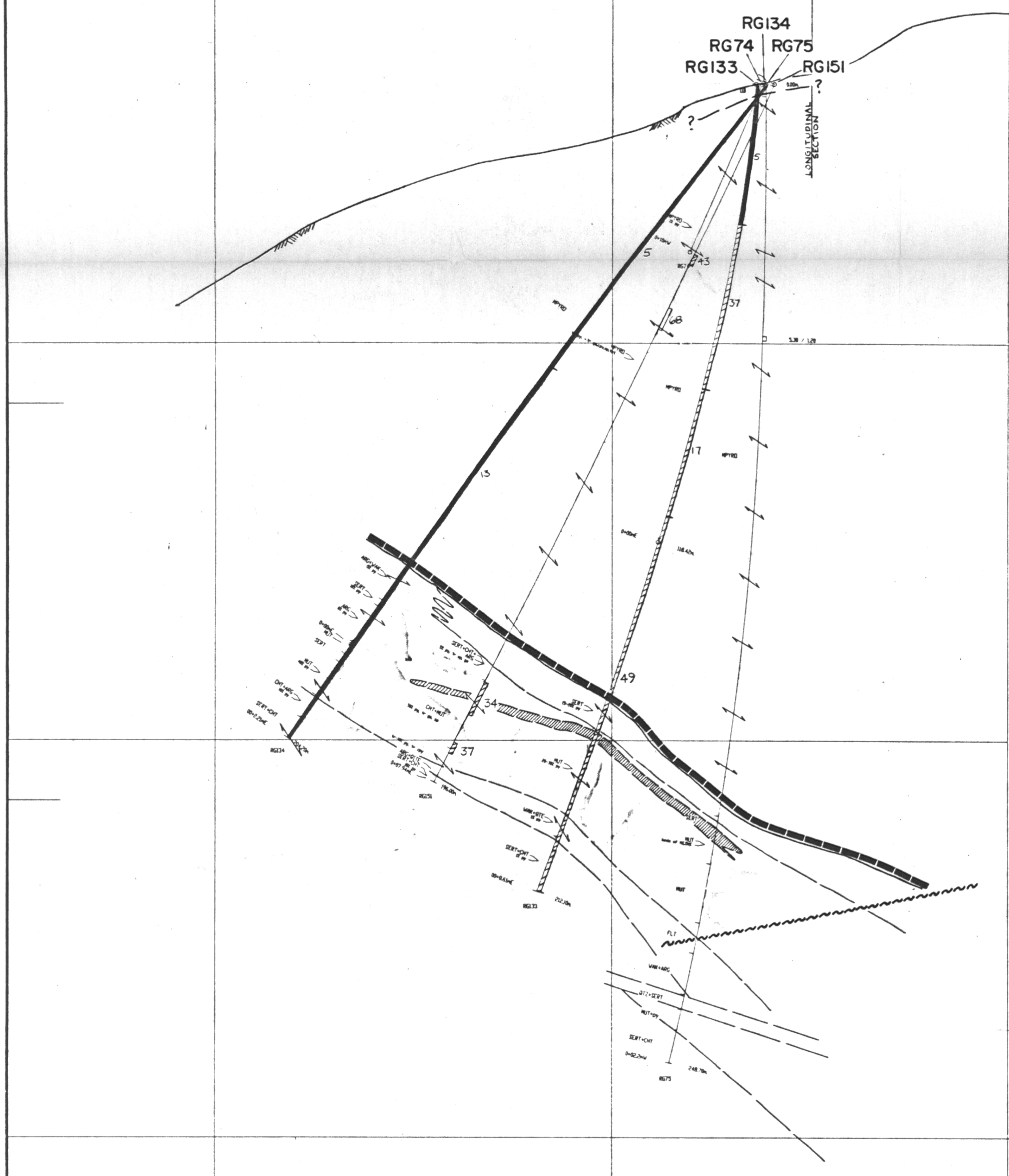
MINNOVA INC. SAMATOSUM PROJECT PRELIMINARY GEOTECHNICAL INVESTIGATION	PITEAU ASSOCIATES GEOTECHNICAL CONSULTANTS VANCOUVER CALGARY	
LOCATION MAP	BY: BL APPROVED:	DATE: FEB. '88 DWG: 946-1



TAILINGS IMPOUNDMENT SITE A
PLANT SITE A
TAILINGS IMPOUNDMENT SITE B
WASTE DUMP SITE B
WASTE DUMP SITE A
OPEN PIT
REA GOLD CONCESSION AREA
105+00 NW
100+00 NW
95+00 NW
5+00 NE
TL 10+58 NE
BL 00
SYMBOLS
TEST PIT LOCATION AND NUMBER
TEST PIT WITH STANDPIPE
MAPPED SOIL / BEDROCK EXPOSURE
APPROXIMATE OVERBURDEN ISOPACH (m)
APPROXIMATE SITE LIMITS
SECTION LINE A-A'
LEGEND
C COLLUVIUM
M MORAINE (TILL)
R BEDROCK
O ORGANIC
v VENEER (<2m)
b BLANKET (2m)
s SANDY
g GRAVELLY
$ SILTY
Example: Organic veneer over silty to sandy Colluvium or reworked Morainal (Till) veneer over Morainal (Till) blanket.
NOTES
1. Overburden isopachs based on reported casing depth in diamond drillholes.
2. See Appendix A for detailed test pit logs.
SCALE
MINNOVA INC.
SAMATOSUM PROJECT
PRELIMINARY GEOTECHNICAL INVESTIGATION
PITEAU & ASSOCIATES
GEOTECHNICAL CONSULTANTS
VANCOUVER CALGARY
PMH / BL
FEB. '88
AFS
946-2
SITE PLAN AND SURFICIAL GEOLOGY MAP

FIG. 2


LONGITUDINAL SECTION
LONGITUDINAL SECTION
100+00W
99+00W
98+00W
97+20W
96+00W
9+00 NE
8+00 NE
7+00 NE
6+00 NE
5+00 NE
4+00 NE
3+00 NE
102+00
101+00
100+00
99+00
98+00
97+00
96+00
GRID NORTH
STEEP REVERSE FAULT
NO OUTCROP
MAIN ACCESS
TO UPPER PADS
SMALL CLIFF
LEGEND
1 Mafic Pyroclastics
2 Sericitic Mafic Tuff
3 Chert (+ argillite)
4 Muddy Tuff
5 Wackes
6 Argillites
VLF Anomaly
Massive Sulphides
Rock Mass Unit Boundary
Approximate Pit Outline
Geotechnical Section Line
ABBREVIATIONS: QFP-quartz feldspar porphyry, bx-breccia chl-chlorite, epi-epidote, arg-argillite, carb-carbonate, py-pyrite, frags-fragments, alt-altered, tr-trace, flt-float, sil-siliceous, stwk-stockwork, Grn-green, vol-volcanic, fgr/mgr/cgr-fine,medium,coarse grained, qtz-quartz, monz-monzonite, frac-fracture, inclus-inclusion, dior-diorite, cpy-chalcopyrite, congl-conglomerate, w-with, t-tuff, lt-lapilli tuff, xt-crystal tuff, x-crystal, dk-dyke, rhy-rhyolite, wk-weak, and-andesite, feld-feldspar, ser-sericite, prop-propylytic, occ-occasional
FIG. 3
NOTE:
CONTOURS DERIVED FROM TOPOGRAPHIC SURVEY AT 10 METRE INTERVALS
MINNOVA Inc.
SAMATOSUM DEPOSIT
GEOLOGY/COMPILATION
0 10 20 30 40 50 100m
SCALE 1:1000
MINNOVA INC.
SAMATOSUM PROJECT
PRELIMINARY GEOTECHNICAL INVESTIGATION
PITEAU ASSOCIATES
GEOTECHNICAL CONSULTANTS
VANCOUVER CALGARY
PIT AREA GEOLOGY PLAN
BY: HWN/BL
DATE: FEB. '88
APPROVED:
DWG: 946-3
DRAWN BY: IP/dm
DATE: 30/06/87
N.T.S. 82M/4W
FIG. NO.:



LEGEND

- OVERBURDEN
- MAFIC PYROCLASTICS
- SERICITIC MAFIC TUFF
- CHERT (+ ARGILLITE)
- MUDDY TUFF
- ORE ZONE
- WACKES
- ARGILLITES
- GEOLOGIC CONTACT (Known, Assumed)
- FAULT
- ROCK MASS UNIT BOUNDARY
- DIAMOND DRILLHOLE WITH ROCK QUALITY DESIGNATION (RQD) 21
  - 0-15% 16-50% 50+%
- GROUND SURFACE
- FOLIATION ORIENTATION

NOTES

1. For location of geotechnical section see Fig. 3.
2. Geological interpretation provided by Minnova Inc.
3. Overburden depth based on reported casing depth in diamond drillholes.

0 10 20 30 40 50 60 70 80 90 100

SCALE: (METRES)

ELEVATION (metres – Geodetic)
ELEVATION (m. – Minnova Datum)

FIG. 4

MINNOVA INC.
SAMATOSUM PROJECT
PRELIMINARY GEOTECHNICAL INVESTIGATION

PITEAU ASSOCIATES
GEOTECHNICAL CONSULTANTS
VANCOUVER CALGARY

MINNOVA Inc.
SAMATOSUM DEPOSIT
CROSS-SECTION VIEWING WEST 96+00mW

GEOTECHNICAL CROSS-SECTION 96+00m W

BY:	DATE:
HWN/BL	FEB. '88
APPROVED:	DWG:
	946-4

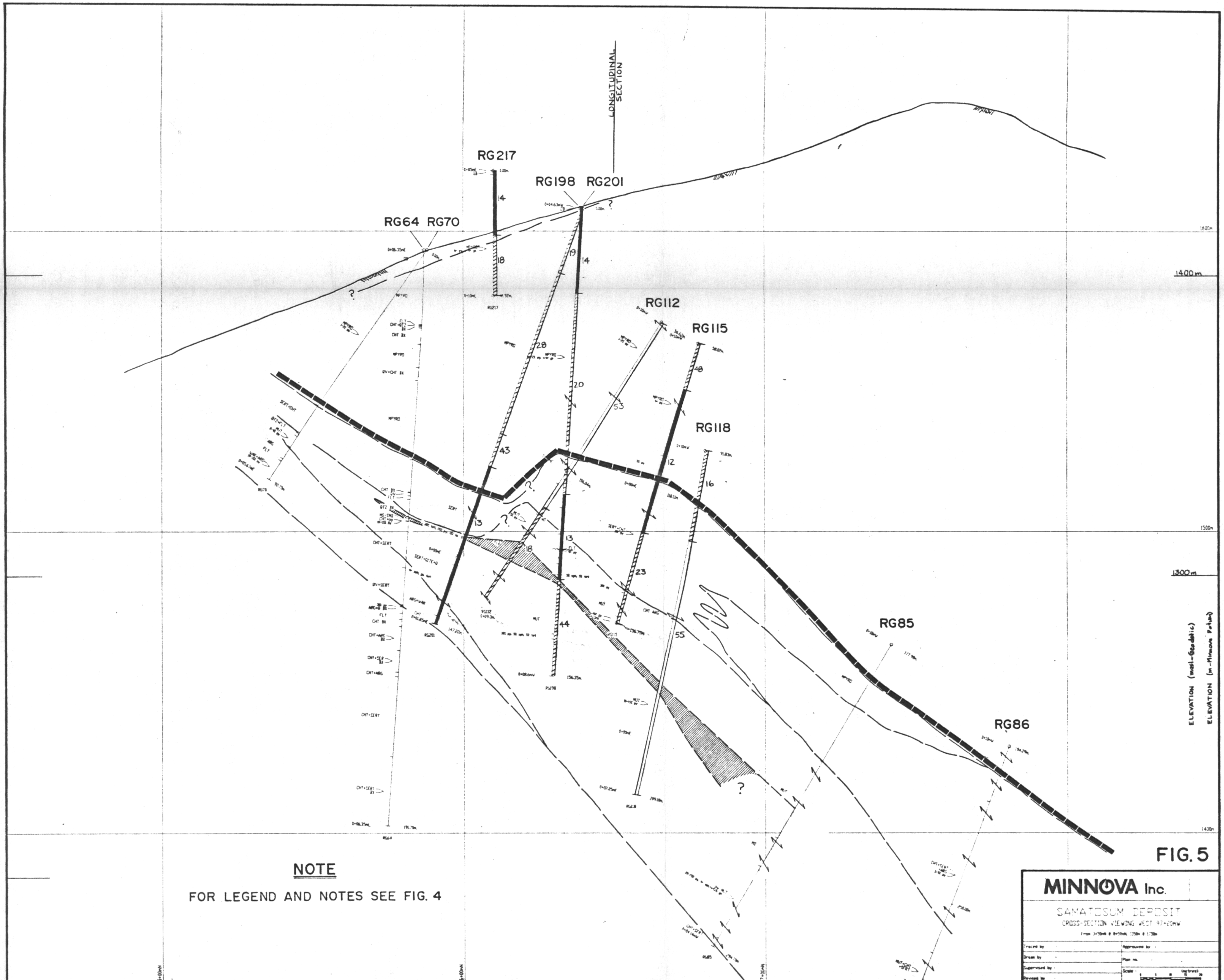


FIG. 5

MINNOVA Inc.

SAMATOSUM DEPOSIT

NOTE

FOR LEGEND AND NOTES SEE FIG. 4

0 10 20 30 40 50 60 70 80 90 100

SCALE: (METRES)

inches

centimetres

This reference scale bar has been added to the original image. It will scale at the same rate as the image, therefore it can be used as a reference for the original size.

BRITISH COLUMBIA GEOLOGICAL SURVEY

MINNOVA INC.
SAMATOSUM PROJECT
PRELIMINARY GEOTECHNICAL INVESTIGATION

PITEAU ASSOCIATES
GEOTECHNICAL CONSULTANTS
VANCOUVER CALGARY

GEOTECHNICAL CROSS-SECTION 97+20mW

BY:	DATE:
HWN/BL	FEB. '88
APPROVED:	DWG:
	946-5

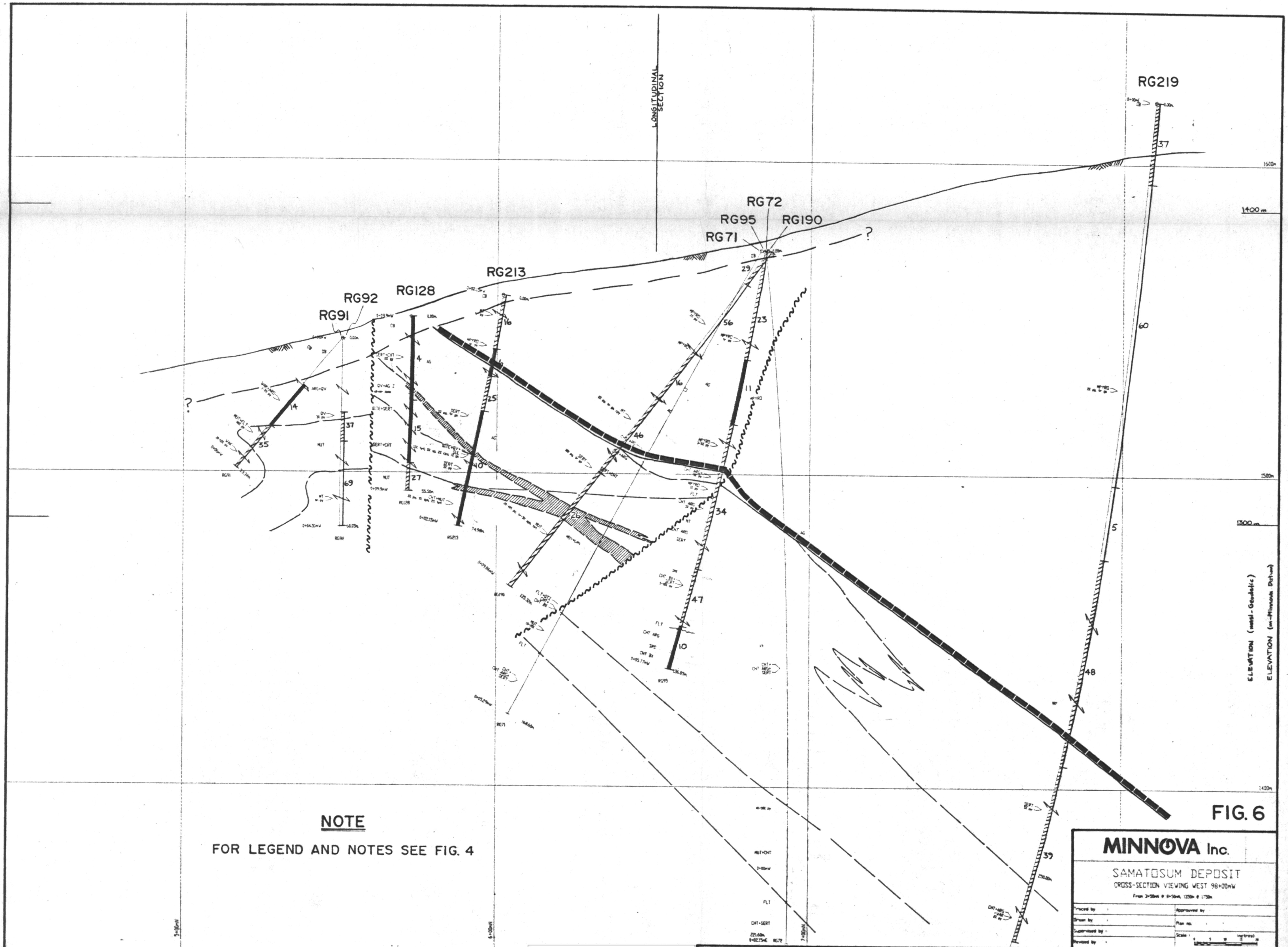


FIG. 6

NOTE

FOR LEGEND AND NOTES SEE FIG. 4

0 10 20 30 40 50 60 70 80 90 100

SCALE: (METRES)

MINNOVA Inc.

SAMATOSUM DEPOSIT

CROSS-SECTION VIEWING WEST 98+00mW

MINNOVA INC.
SAMATOSUM PROJECT
PRELIMINARY GEOTECHNICAL INVESTIGATION

PITEAU ASSOCIATES
GEOTECHNICAL CONSULTANTS
VANCOUVER CALGARY

BY:	DATE:
HWN/BL	FEB. '88
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	946-6

GEOTECHNICAL CROSS-SECTION 98+00mW

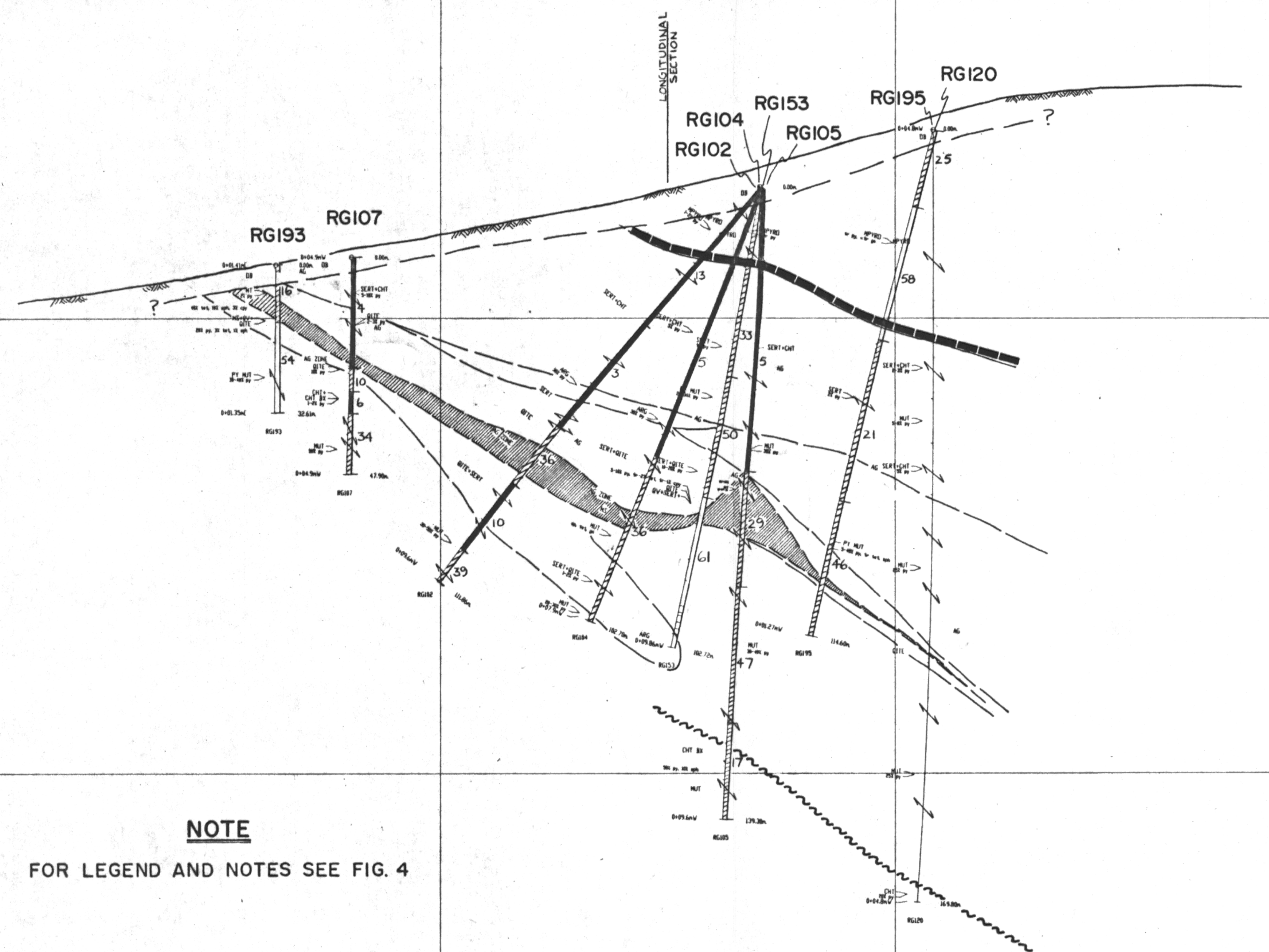


FIG. 7

NOTE

FOR LEGEND AND NOTES SEE FIG. 4

0 10 20 30 40 50 60 70 80 90 100

SCALE: (METRES)

inches 0 1 2

0 1 2 3 4 5 centimetres

MINNOVA Inc.

SAMATOSUM DEPOSIT

CROSS-SECTION VIEWING WEST 99+00mW

MINNOVA INC.
SAMATOSUM PROJECT
PRELIMINARY GEOTECHNICAL INVESTIGATION

PITEAU & ASSOCIATES
GEOTECHNICAL CONSULTANTS
VANCOUVER CALGARY

GEOTECHNICAL CROSS-SECTION 99+00mW

BY:	DATE:
HWN/BL	FEB. '88
APPROVED:	DWG:
	946-7

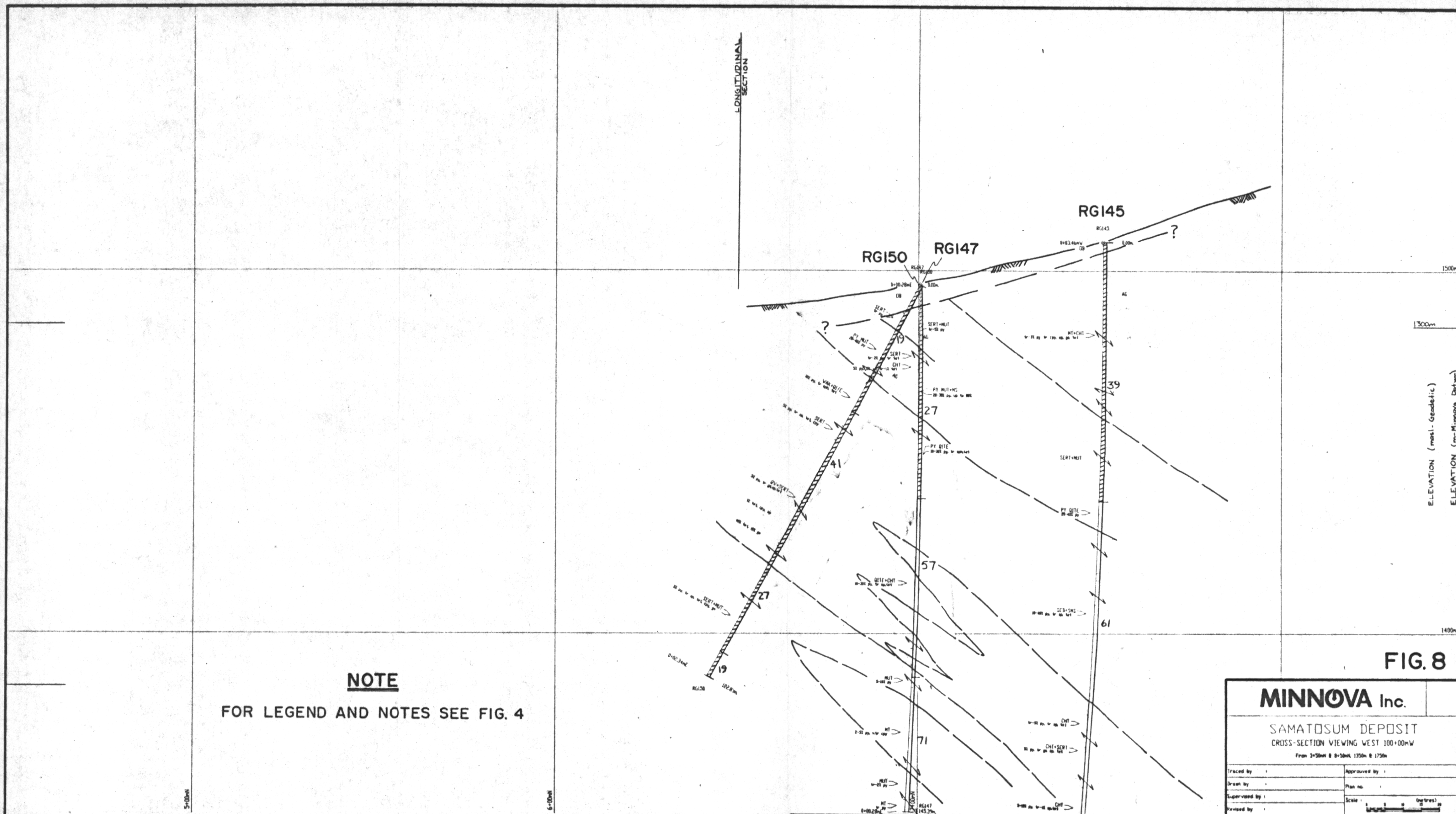


FIG. 8

MINNOVA Inc.

SAMATOSUM DEPOSIT

CROSS-SECTION VIEWING WEST 100+00mW

NOTE

FOR LEGEND AND NOTES SEE FIG. 4

0 10 20 30 40 50 60 70 80 90 100

SCALE: (METRES)

inches 0 1 2

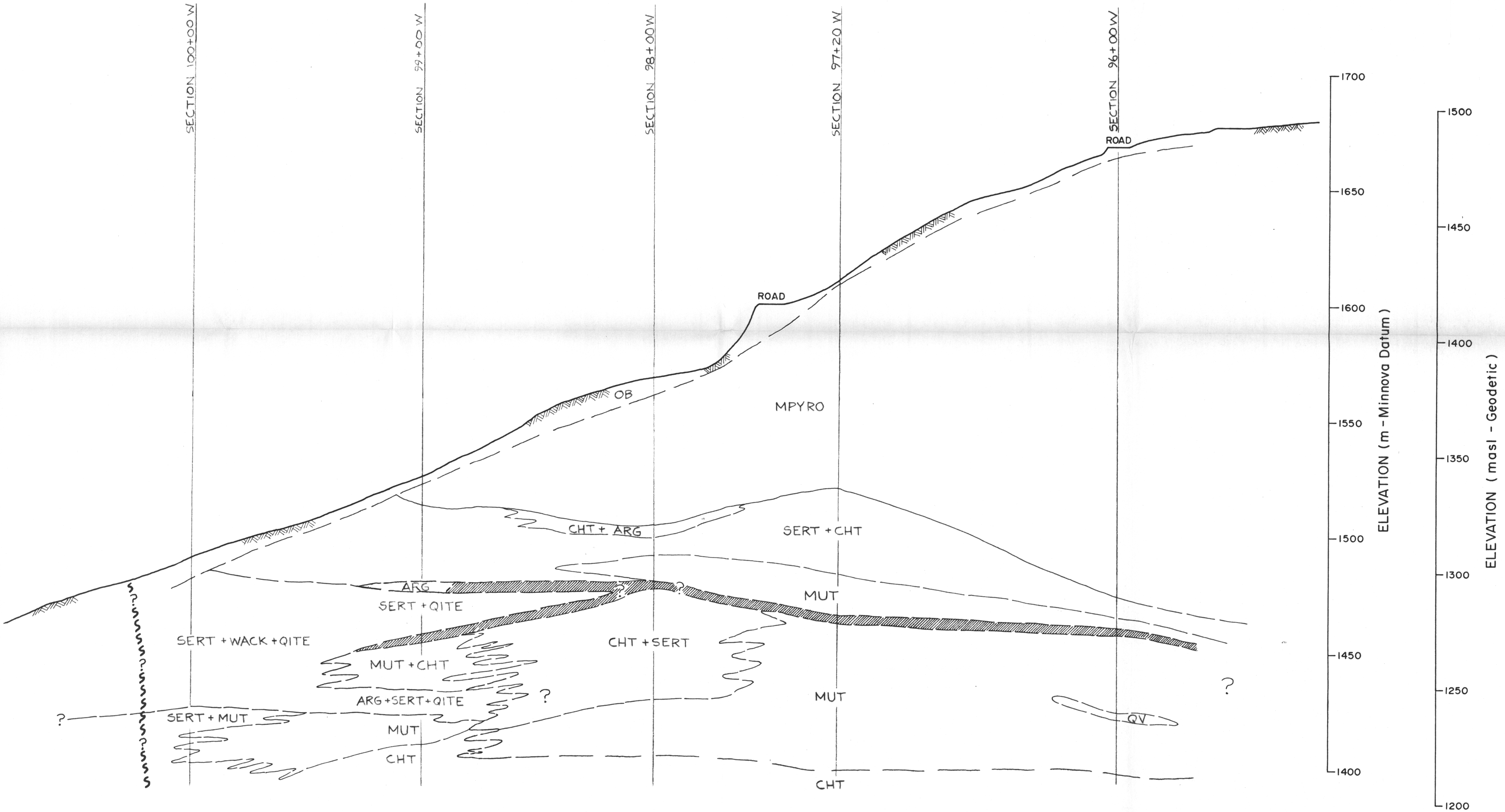
centimetres 0 1 2 3 4 5

MINNOVA INC.
SAMATOSUM PROJECT
PRELIMINARY GEOTECHNICAL INVESTIGATION

PITEAU ASSOCIATES
GEOTECHNICAL CONSULTANTS
VANCOUVER CALGARY

GEOTECHNICAL CROSS-SECTION 100+00mW

BY:	DATE:
HWN/BL	FEB. '88
APPROVED:	DWG: 946-8



NOTES
1. See Fig. 3 for location of longitudinal section.
2. See Fig. 4 for Legend and Notes.
3. Geological interpretation on this longitudinal section is based on the geotechnical sections in Figs 4 to 8.

1 : 1000

0 10 20 30 40 50 100 m

SCALE

FIG. 9

MINNOVA INC.
SAMATOSUM PROJECT
PRELIMINARY GEOTECHNICAL INVESTIGATION

PITEAU ASSOCIATES
GEOTECHNICAL CONSULTANTS
VANCOUVER CALGARY

GEOTECHNICAL LONGITUDINAL SECTION

BY:	DATE:
HWN	FEB '88
APPROVED:	DWG:
	946-9

TABLE III

SUMMARY OF DESIGN SECTORS, KINEMATIC AND ROCK MASS COMPETENCY ASSESSMENTS AND RECOMMENDED PRELIMINARY SLOPE DESIGN

DESIGN SECTOR[1]	WALL TYPE[2]	SLOPE AZIMUTH[1] (°)	CRITICAL PLANE FAILURES[3]: DISCONTINUITY SET[4]	CRITICAL PLANE FAILURES[3]: APPARENT DIP[5] (°)	CRITICAL PLANE FAILURES[3]: FACTOR OF SAFETY[6]	CRITICAL WEDGE FAILURES[7]: DISCONTINUITY SETS INVOLVED	CRITICAL WEDGE FAILURES[7]: APPARENT PLUNGE[8] (°)	CRITICAL WEDGE FAILURES[7]: FACTOR OF SAFETY[6]	DEGREE OF KINEMATIC CONTROL[9]	APPARENT DIP OR PLUNGE OF FAILURE CONSIDERED TO CONTROL BENCH STABILITY, β_w (°)[10]	APPENDIX FIGURE[11]
I	Footwall	025	F1	42	0.4 - 0.5	F1/B3 F1/B2	42 42	0.5 - 0.6 0.7 - 1.0	Very Strong	42	B-1
II	Oblique Footwall	055	F1 (oblique)	45	0.4 - 0.5	F1/B1 F1/B2 F1/B3	29 35 43	1.0 - 1.5 0.7 - 1.0 0.5 - 0.6	Strong	37	B-2
III	Oblique Endwall	085	NONE	-	-	F1/B1 F1/B2 B1/B2	27 37 67	1.0 - 1.5 0.7 - 1.0 0.3 - 0.6	Moderate-Strong	37	B-3
IV	Endwall	115	B2 B3	62 88	0.2 - 0.4 <0.1	F1/B1 B1/B2 F1/B2	32 48 61	1.0 - 1.5 0.7 - 1.0 0.3 - 0.6	Strong	61	B-4
V	Oblique Endwall	145	B2 B1	62 65	0.2 - 0.4 0.2 - 0.3	A2/B2	53	0.6 - 1.2	Strong	62	B-5
VI	Oblique Hanging Wall	175	B1	66	0.2 - 0.3	A2/B2 A1/B1 A2/B3 A2/B1 B1/B3	43 48 54 56 56	0.6 - 1.2 0.7 - 1.3 0.6 - 1.1 0.4 - 0.8 0.8 - 1.5	Moderate-Strong	60	B-6
VII	Hanging Wall	205	NONE	-	-	A1/B1 A2/B2 A2/B3 A2/B1 B1/B3	38 42 49 50 51	0.7 - 1.3 0.6 - 1.2 0.6 - 1.1 0.4 - 0.8 0.8 - 1.5	Weak-Moderate	47	B-7
VIII	Oblique Hanging Wall	235	A2	56	0.2 - 0.5	A1/B1 A2/B2 A2/B3 A2/B1 B1/B3	36 49 52 53 54	0.7 - 1.3 0.6 - 1.2 0.6 - 1.1 0.4 - 0.8 0.8 - 1.5	Weak-Moderate	51	B-8
IX	Oblique Endwall	265	A1 A2	47 57	0.3 - 0.6 0.2 - 0.5	A1/B1 B1/B3	44 66	0.7 - 1.3 0.8 - 1.5	Moderate-Strong	47	B-9
X	Endwall	295	A1 (oblique)	50	0.3 - 0.6	F1/A1	34	0.8 - 1.3	Weak	50	B-10
XI	Oblique Endwall	325	NONE	-	-	F1/A1	28	0.8 - 1.3	Very Weak	-	B-11
XII	Oblique Footwall	355	F1 (oblique)	45	0.4 - 0.5	F1/A1	30	0.8 - 1.3	Weak	46	B-12

ALTERNATIVE BENCH DESIGN BASED ON KINEMATICALLY POSSIBLE FAILURE MODES[12]

DESIGN SECTOR[1]	APPENDIX FIGURE[11]	BENCH HEIGHT, H (m)[13]	DESIGN BENCH FACE ANGLE, β (°)	BREAKBACK[14] ℓ_{bb} (m) A	BREAKBACK[14] ℓ_{bb} (m) B	EFFECTIVE BENCH FACE ANGLE, β_e (°) A	EFFECTIVE BENCH FACE ANGLE, β_e (°) B	DESIGN BERM[15] WIDTH, L (m) A	DESIGN BERM[15] WIDTH, L (m) B	INTERMEDIATE SLOPE ANGLE, θ A	INTERMEDIATE SLOPE ANGLE, θ B
I	B-1	10	90	11.1	5.6	42	61	19.1	13.6	27.6	36.3
		20		22.2	11.1			32.2	21.1	34.6	43.5
II	B-2	10	90	13.3	6.7	37	56	21.3	14.7	25.1	34.2
		20		26.5	13.3			36.5	23.3	28.7	40.6
III	B-3	10	90	13.3	6.7	37	56	21.3	14.7	25.1	34.2
		20		26.5	13.3			36.5	23.3	28.7	40.6
IV	B-4	10	90	5.5	2.8	61	74	13.5	10.8	36.5	42.8
		20		11.1	5.5			21.1	15.5	43.5	52.2
V	B-5	10	90	5.3	2.7	62	75	13.3	10.7	36.9	43.1
		20		10.6	5.3			20.6	15.3	44.2	52.6
VI	B-6	10	90	5.8	2.9	60	74	13.8	10.9	35.9	42.5
		20		11.5	5.8			21.5	15.8	42.9	51.7
VII	B-7	10	90	9.3	4.7	47	65	17.3	12.7	30.0	38.2
		20		18.7	9.4			28.7	19.4	34.9	45.9
VIII	B-8	10	90	8.1	4.1	51	58	16.1	12.1	31.8	39.6
		20		16.2	8.1			26.2	18.1	37.4	47.9
IX	B-9	10	90	9.3	4.7	47	65	17.3	12.7	30.0	38.2
		20		18.7	9.3			28.7	19.4	34.9	45.9
X	B-10	10	90	8.4	4.2	50	67	16.4	12.2	31.4	39.3
		20		16.8	8.4			26.8	18.4	36.7	47.4
XI	B-11	10	90	0	0	90	90	8.0	8.0	51.3	51.3
		20		0	0			10.0	10.0	63.4	63.4
XII	B-12	10	90	9.7	4.9	46	64	17.7	12.9	29.5	37.8
		20		19.3	9.7			29.3	17.7	34.3	45.4

RECOMMENDED PRELIMINARY SLOPE DESIGN[12, 17]

DESIGN SECTOR[1]	PRINCIPAL ROCK TYPES (Rock Mass Units)[16]	ASSESSMENT OF ROCK MASS COMPETENCY[16]	BENCH HEIGHT, H (m)	DESIGN BENCH FACE ANGLE, β (°)	DESIGN BERM WIDTH, L (m)	INTERMEDIATE SLOPE ANGLE, θ (°)	COMMENTS
I II III	Lightly to Strongly Altered Muddy Tuff/Sediments/Chert Mafic Tuff	Poor-Moderate	10	70	8	41	Plane failures on foliation discontinuities (Set F) or wedge failures on foliation and Set B discontinuities could result in loss of berm access or complete loss of berms. Single benches recommended to limit size of potential failures. Inclined bench faces and controlled blasting or ripping recommended to reduce potential rockfall generation and ravelling which may occur due to relatively incompetent nature of the rock mass. If major, continuous faults occur parallel to foliation and/or Discontinuity Set B, large plane or wedge failures involving several benches or the whole slope could result. Remedial measures consisting of artificial support, buttressing, flattening or slope despressurization may be required.
IV V VI	Unaltered to Lighlty Altered Mafic Pyroclastics; Mafic Tuff	Moderate-Good	20 (20)	90 (70)	15 (12)	53 (46)	Plane failures on Discontinuity Set B or wedge failures on Discontinuity Sets B and A control bench design in competent rock (i.e. Mafic Pyroclastics) and may result in loss of access to some berms. Instability of the overall slope not anticipated provided major, continuous faults do not occur which are flatter than the observed discontinuity sets. Design shown in parentheses should be used for incompetent rock masses (i.e. Muddy Tuff, Sericite Altered Mafic Tuff, etc.) where inclined bench faces and controlled blasting or ripping are recommended to reduce potential rockfall generation and ravelling.
VII VIII IX X	Lightly to Strongly Altered Mafic Tuff; Muddy Tuff/Sediments/Chert	Poor-Moderate	20 (20)	90 (70)	19 (14)	47 (43)	Plane failures on Discontinuity Set A or wedge failures on Discontinuity Sets A and B control bench design in competent rock (i.e. Mafic Pyroclastics) and may result in loss of access to some berms. Instability of the overall slope not anticipated provided major, continuous faults do not occur which are flatter than the observed discontinuity sets. Design shown in parentheses should be used for incompetent rock masses (i.e. Muddy Tuff, Sericite Altered Mafic Tuff, etc.) where inclined bench faces and controlled blasting or ripping are recommended to reduce potential rockfall generation and ravelling.
XI XII	Lightly to Strongly Altered Muddy Tuff/Sediments/Chert; Mafic Tuff	Poor-Moderate	20	70	12	46	Oblique plane failures on foliation discontinuities (Set F), or wedges on foliation and Set A discontinuities could result in loss of access to some berms. Inclined bench faces and controlled blasting or ripping recommended to reduce potential rockfall generation and ravelling which may occur due to relatively incompetent nature of the rock mass. If major, continuous faults occur parallel to foliation and/or Discontinuity Set A, large oblique plane or wedge failures involving several benches or the whole slope could result. Remedial measures consisting of artificial support, buttressing, flattening or slope despressurization may be required.

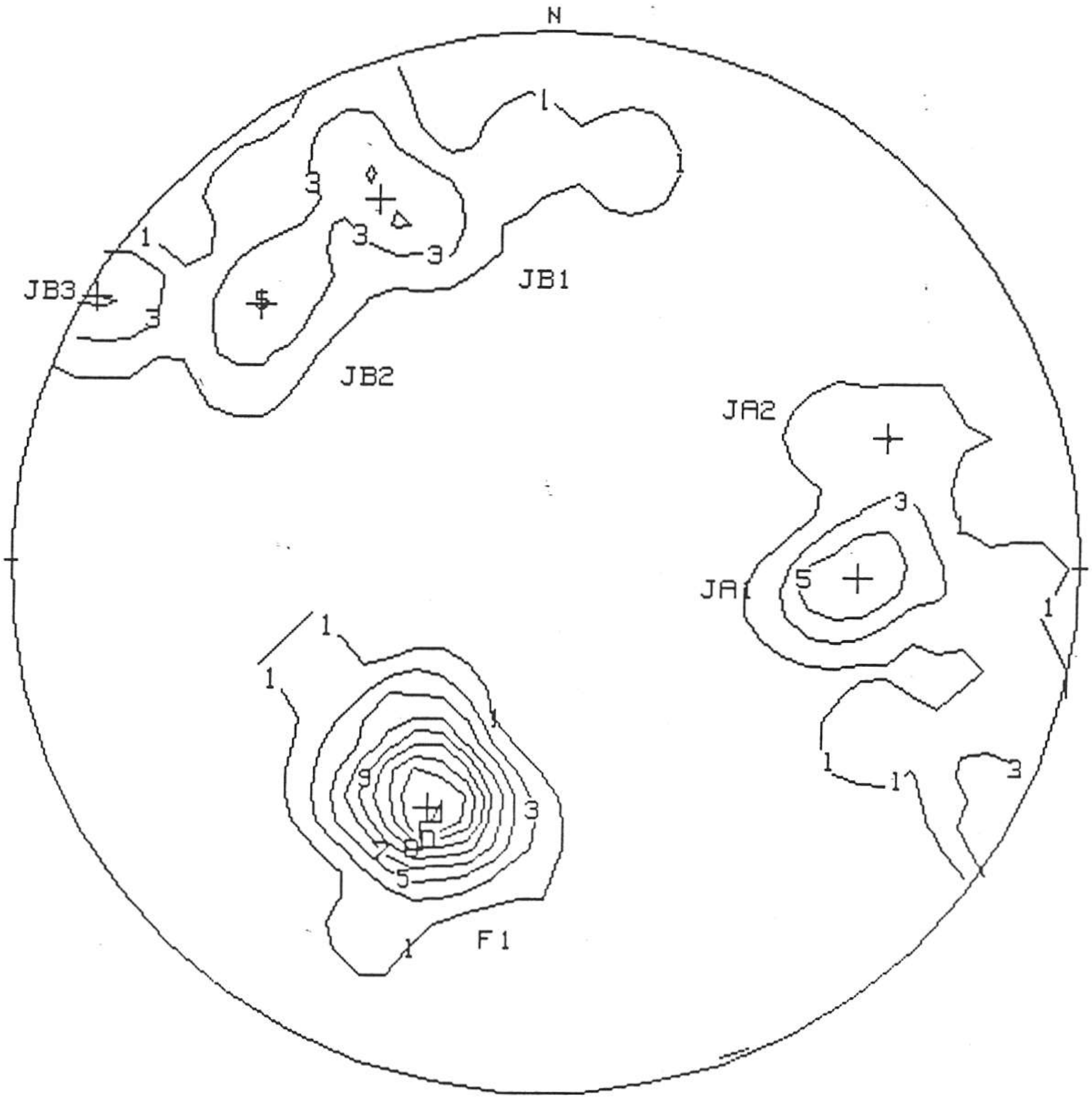
NOTES:

1. Kinematic assessments were conducted for twelve discrete slope orientations or Design Sectors at 30° intervals of slope azimuth (dip direction) assuming a footwall slope azimuth of 025°.
2. Wall Type refers to the general orientation of foliation with respect to the slope (e.g. footwalls strike parallel to foliation and dip in the same direction, endwalls strike normal to foliation, and hanging walls strike parallel to foliation and dip in the opposite direction).
3. Kinematically possible planar failures which generally strike within about 45° parallel to the slope and which have lower bound Factors of Safety of less than 1.0 are designated Critical Plane Failures.
4. Critical Plane Failures which strike about 20° to 45° parallel to the slope are indicated in parentheses as oblique.
5. Apparent Dip is the dip of the plane relative to the slope which must be undercut for the plane to become a kinematically possible plane failure.
6. Factor of Safety is given as a range. The lower bound is the Factor of Safety if all discontinuities involved occur along faults. The upper bound is the Factor of Safety if all discontinuities occur along cross joints or foliation joints.
7. Kinematically possible wedge failures whose line of intersection strikes within about 45° to the slope azimuth and which have lower bound Factors of Safety of less than 1.0 are designated Critical Wedge Failures.
8. Apparent Plunge is the dip of the line of intersection relative to the slope which must be undercut for the wedge to become a kinematically possible wedge failure.
9. Degree of Kinematic Control is a qualitative assessment of the relative importance that kinematically possible failure modes may have on bench stability, and is based on the Factor of Safety, orientation, type of failure and intensity of the discontinuity sets involved.
10. Based on assessment of apparent dips and plunges of the various critical plane and wedge failures and Degree of Kinematic Control.
11. Refers to lower hemisphere projections for each design sector given in Appendix B.
12. Bench geometry parameters are illustrated in Fig. 15.
13. Alternative slope geometries have been prepared for single (i.e. 10m) and double (i.e. 20m) high benches.
14. Detailed assessments of like bench crest breakback were not possible as no benches have as yet been exposed. To evaluate the sensitivity of slope geometry to possible breakback, two possible breakback scenarious have been assessed. Scenario A assumes the benches breakback to the apparent dip or plunge considered to control bench stability (i.e. β_w). Scenario B assumes the benches breakback only half the distance indicated by Scenario A.
15. Design Berm Width is based on the amount of breakback (ℓ_{bb}) plus a minimum berm width required to provide access to the slope and adequate rockfall protection (ℓ_{min}). For 10m single benches, ℓ_{min} is taken as 8m; for 20m double benches, ℓ_{min} is assumed to be 10m.
16. Based on the geologic interpretation and geotechnical core logging conducted by Minnova personnel and our assessment of the rock mass, two basic rock mass units are identified. Lightly to strongly sericite altered Muddy Tuff, Argillite, Greywacke, Chert and related rocks and moderately to strongly sericite altered Mafic Tuff are considered together as one rock mass unit with an overall poor to moderate competency. This unit may occur in all design sectors. Unaltered to lightly altered Mafic Pyroclastics and Mafic Tuff are considered together as the other rock mass unit and have an overall moderate to good competency. These rocks are expected to form significant components of pit walls in Design Sectors IV through X.
17. Recommended Preliminary Slope Designs are based on an assessment of kinematically possible failures, rock types and rock mass competency for each design sector. Based on actual conditions encountered during mining of the initial benches, modification of preliminary designs may be required. Periodic updates and refinement of slope designs throughout mine life are recommended to achieve the optimum overall slope design.
18. Additional comments and recommendations are given in Section 5.5.

JOB NUMBER

OBSERVATIONS: 100
POPULATION: 100

CONTOUR PLOT



LABEL	DIP DIRECTION	DIP	CONCENTRATION (%)
F1	025	42	17
JA1	272	48	6
JA2	249	57	3
JB1	155	65	5
JB2	132	62	5
JB3	121	88	5

NOTE: BASED ON PITEAU ASSOCIATES MAPPING
DATA IN THE REA GOLD ADIT

FIG. 10

MINNOVA INC.
SAMATOSUM PROJECT
PRELIMINARY GEOTECHNICAL INVESTIGATION

PITEAU ASSOCIATES
GEOTECHNICAL CONSULTANTS
VANCOUVER CALGARY

LOWER HEMISPHERE EQUAL AREA PROJECTION OF JOINTS

BY:	DATE:
AFS	FEB/88
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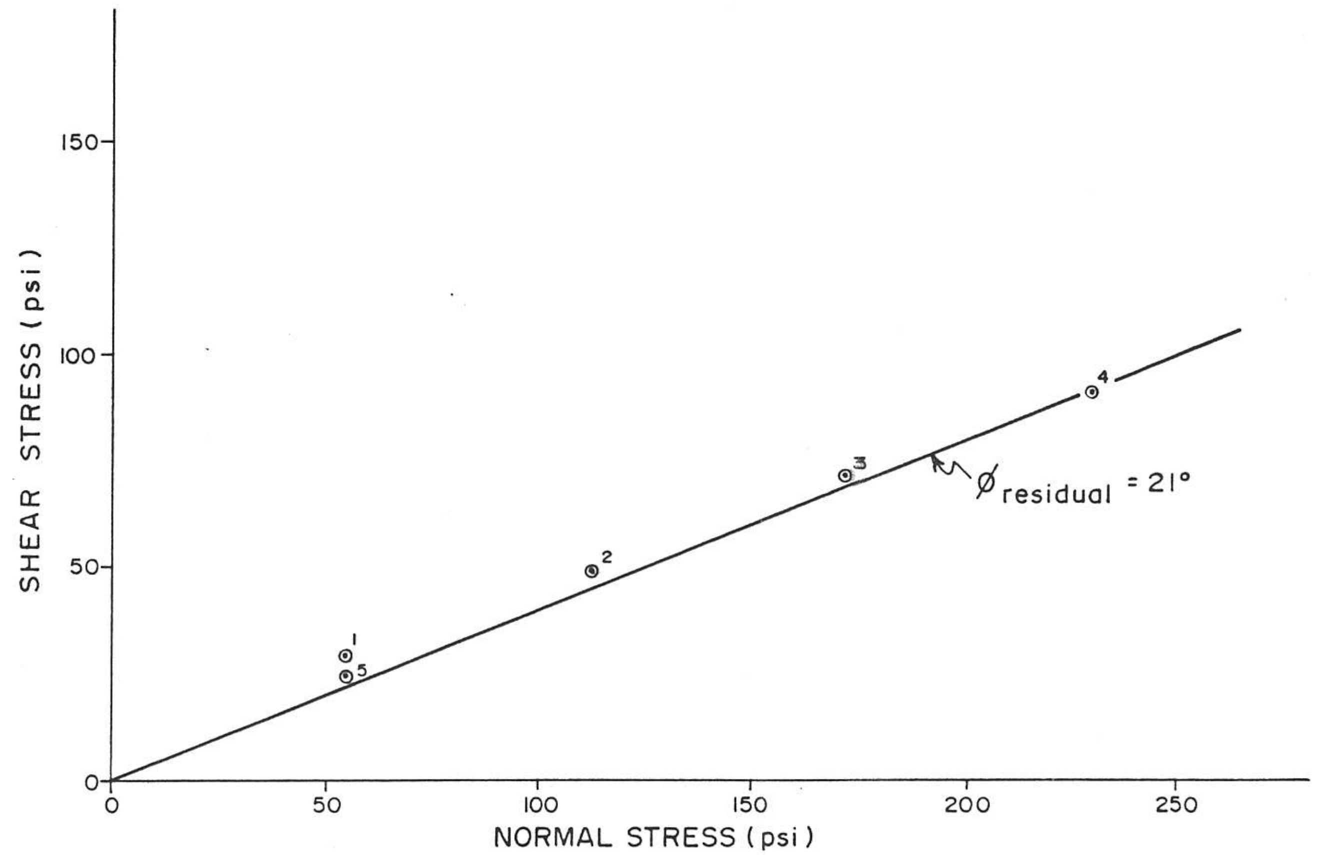
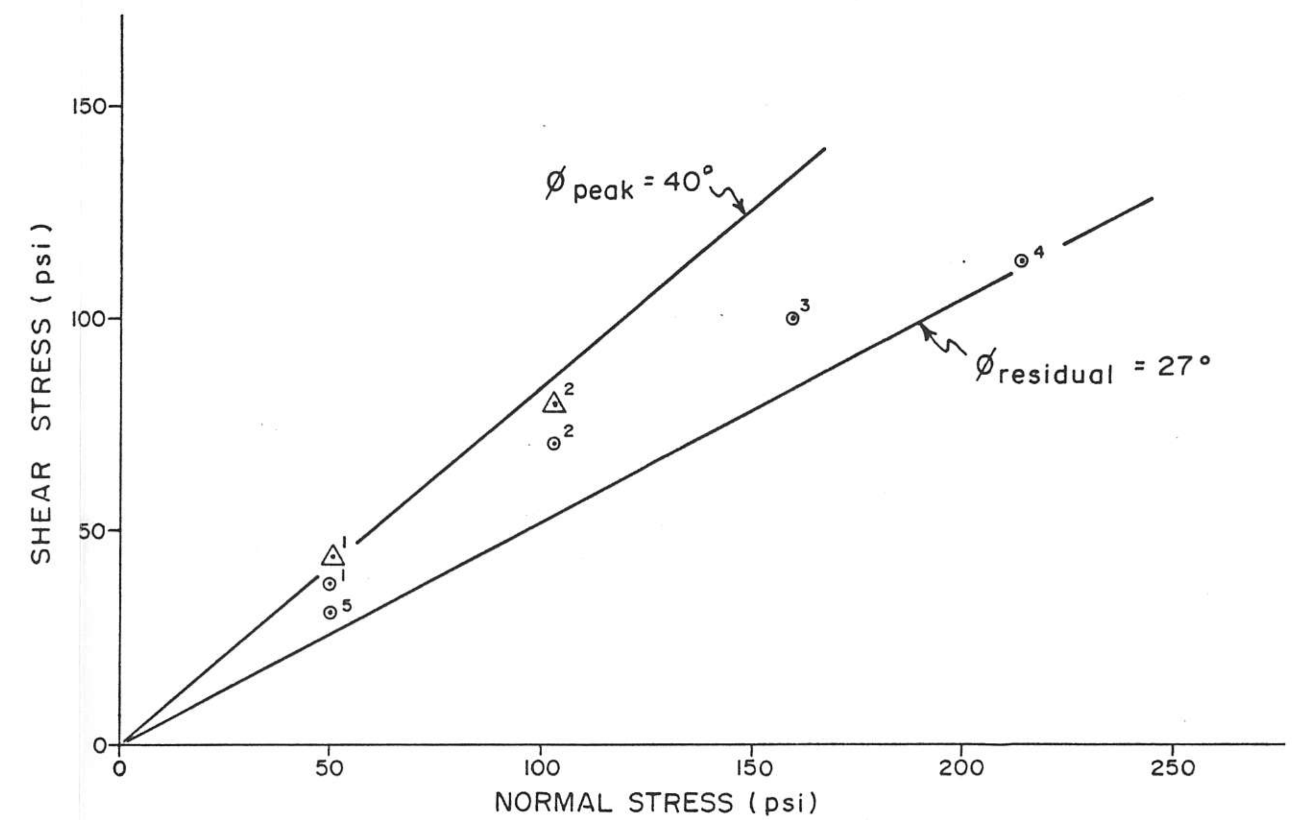
	946-10

JOB NUMBER 946



a) FOLIATION PARTING IN LIGHTLY ALTERED MUDDY TUFF

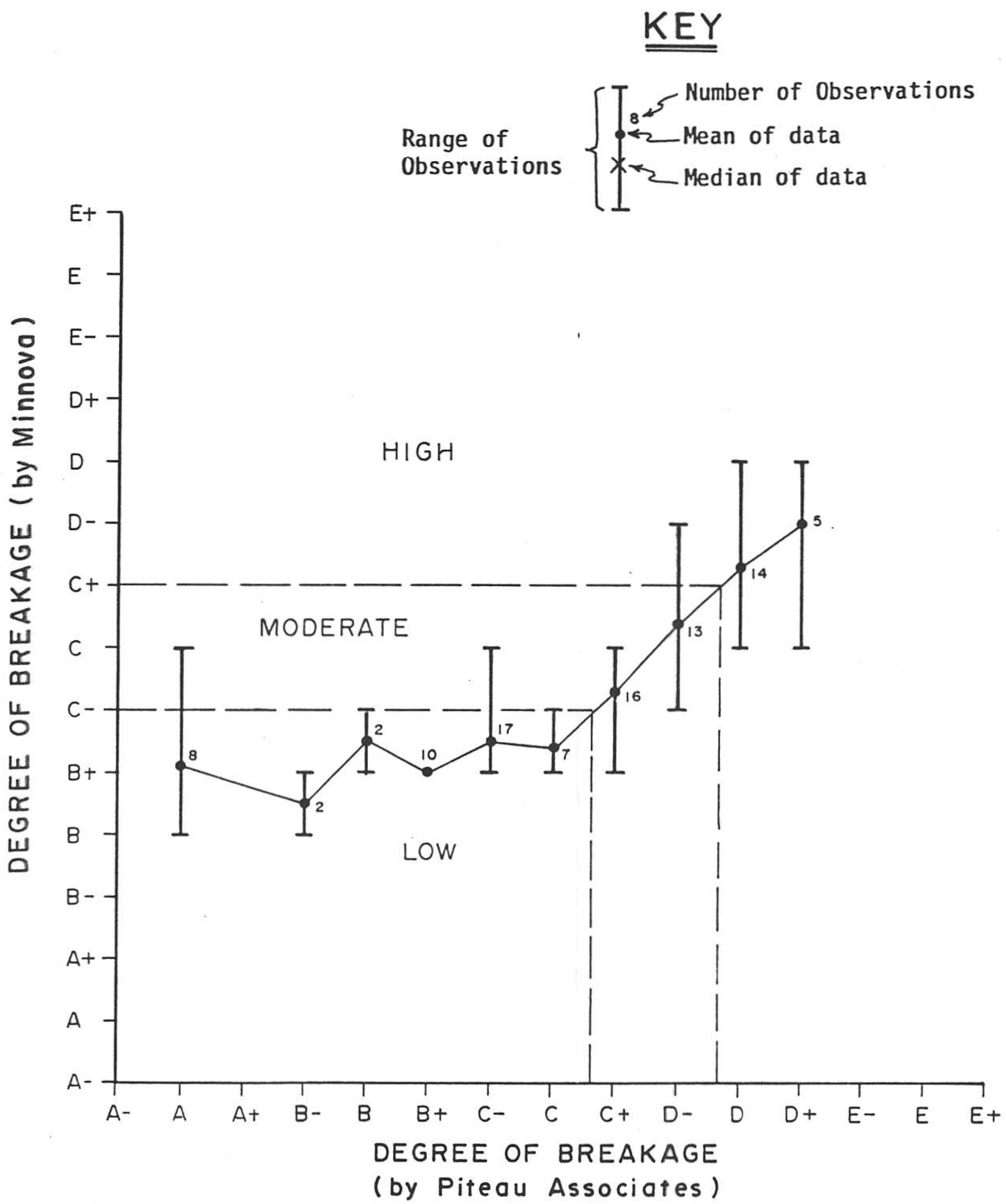


b) FOLIATION PARTING IN MODERATELY ALTERED MAFIC TUFF

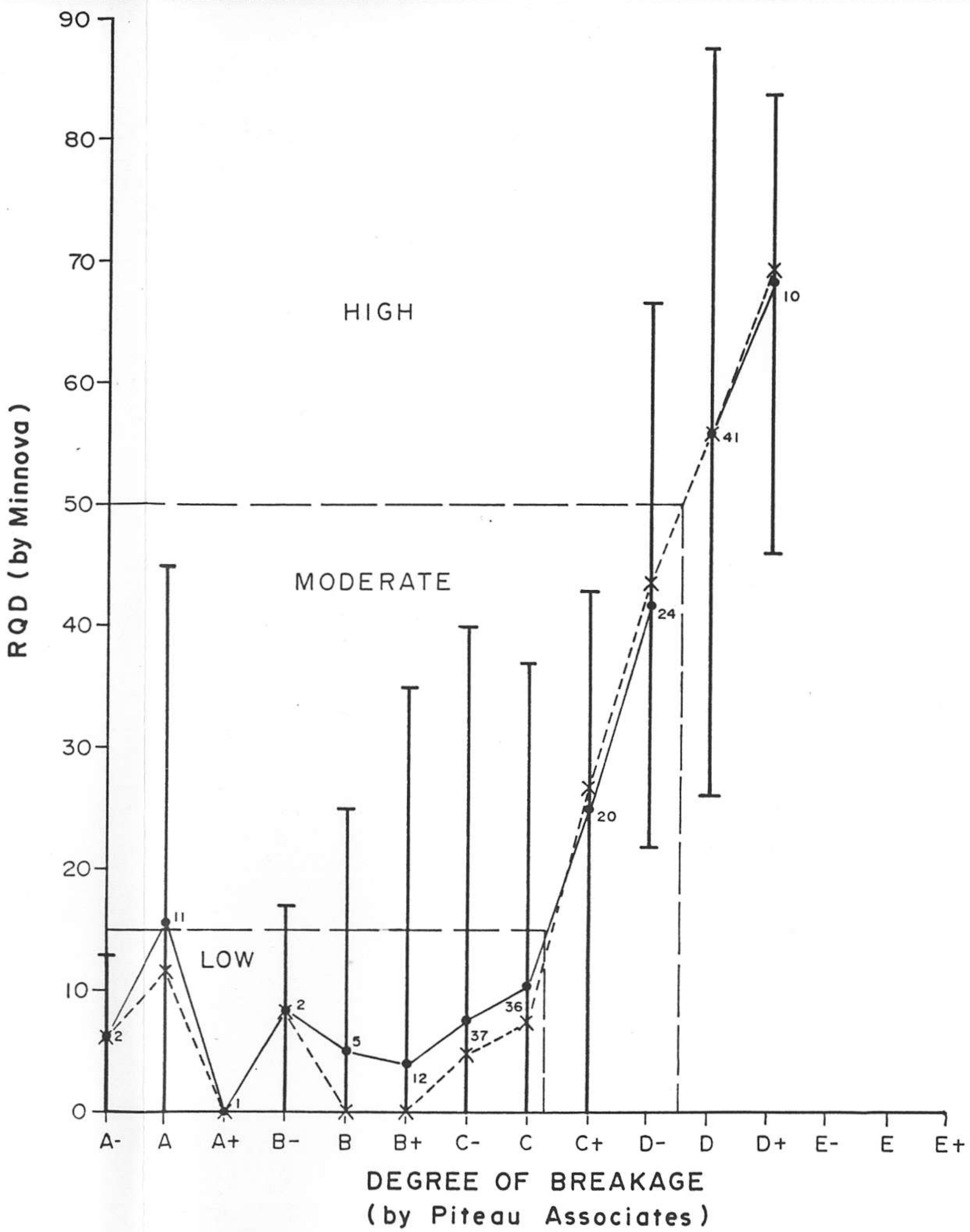
FIG. 11

MINNOVA INC. SAMATOSUM PROJECT PRELIMINARY GEOTECHNICAL INVESTIGATION		PITEAU ASSOCIATES GEOTECHNICAL CONSULTANTS VANCOUVER CALGARY	
RESULTS OF DIRECT SHEAR TESTING ON FOLIATION DISCONTINUITIES		BY: PMH/BL	DATE: FEB. '88
		APPROVED:	DWG: 946-11

JOB No. 946



a) COMPARISON OF DEGREE OF BREAKAGE LOGGING BY PITEAU ASSOCIATES AND MINNOVA.



b) CORRELATION OF RQD LOGGING BY MINNOVA AND DEGREE OF BREAKAGE BY PITEAU ASSOCIATES.

FIG. 12

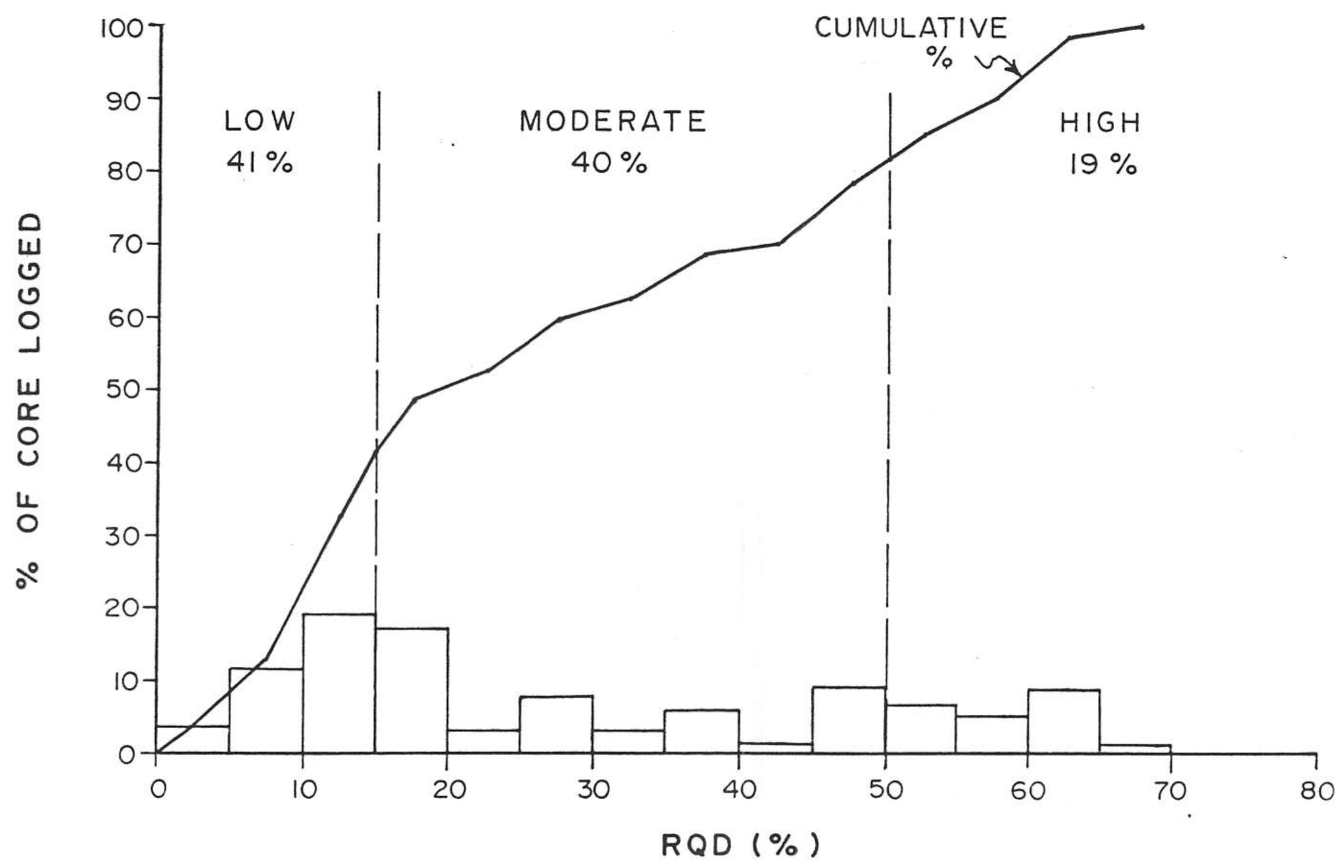
MINNOVA INC.
SAMATOSUM PROJECT
PRELIMINARY GEOTECHNICAL INVESTIGATION

PITEAU ASSOCIATES
GEOTECHNICAL CONSULTANTS
VANCOUVER CALGARY

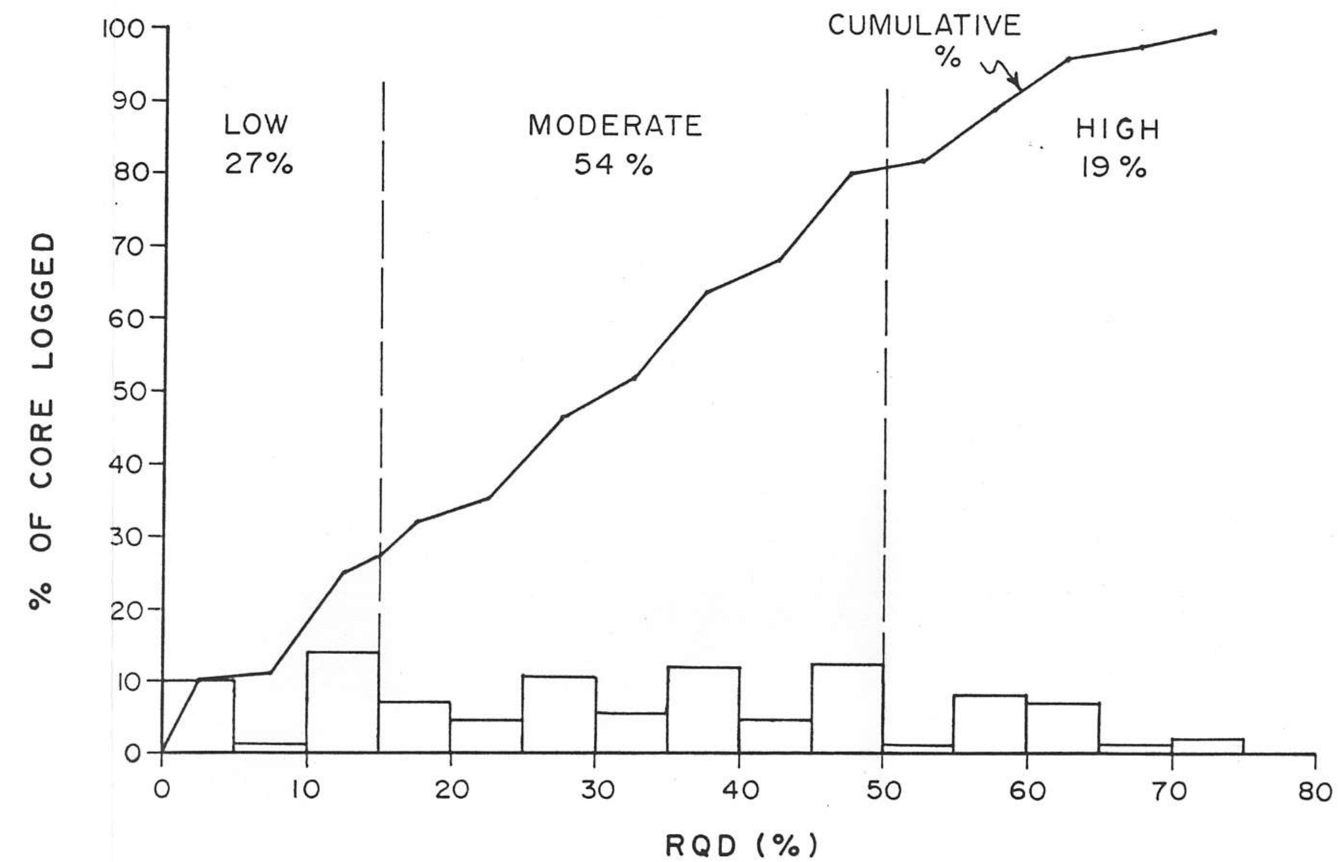
CORRELATION OF GEOTECHNICAL LOGGING PARAMETERS

BY: PMH/BL	DATE: FEB. '88
APPROVED: [signature]	DWG: 946-12

JOB NUMBER 946



a) MAFIC PYROCLASTICS [2]



b) SERICITE ALTERED MAFIC TUFF , MUDDY TUFF, SEDIMENTS, CHERT AND RELATED ROCKS [3]

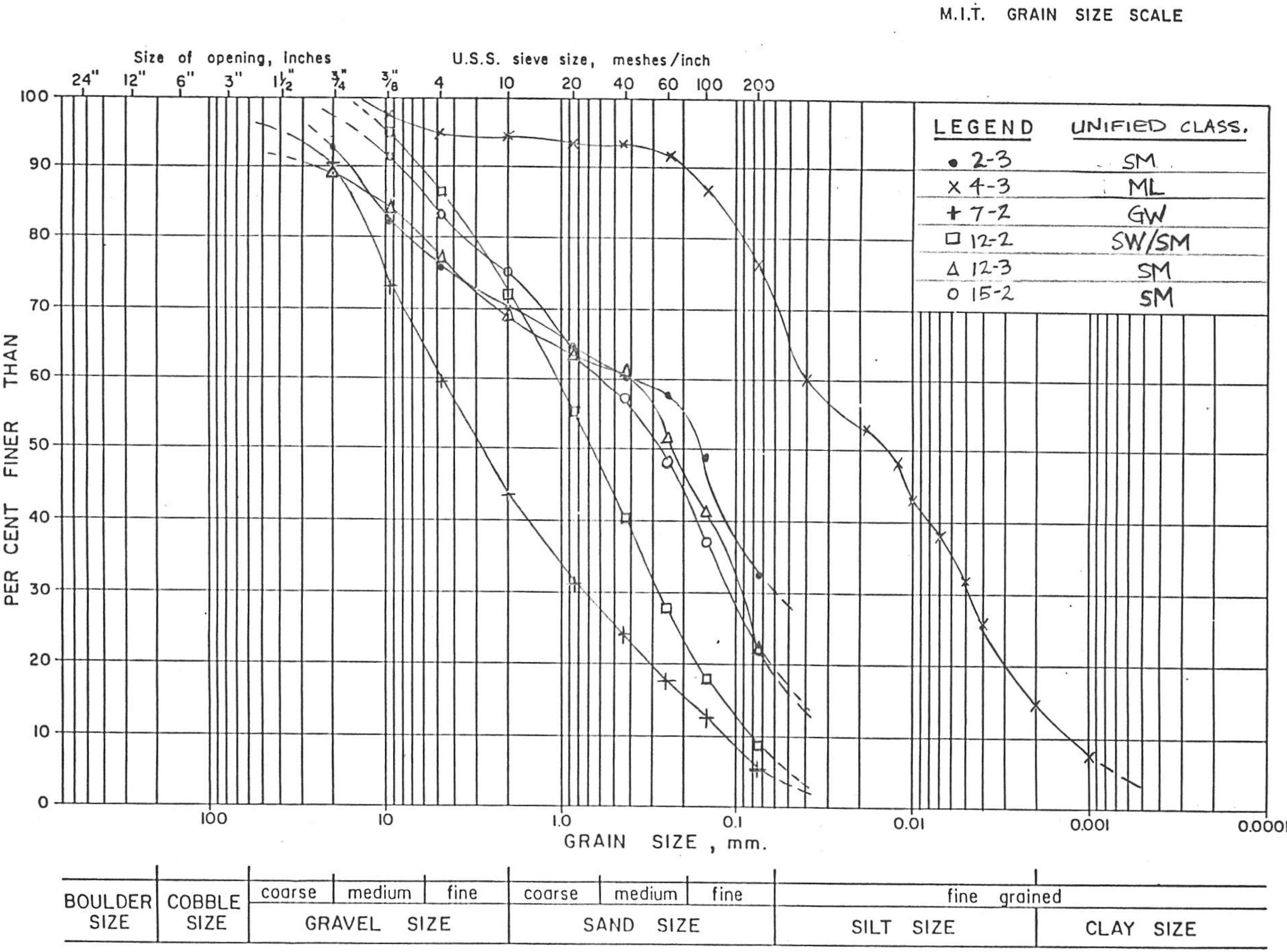
NOTES:

1. Based on cumulative sums analysis of RQD's logged for drillholes which occur on the geotechnical sections in Fig. 4 to 8.
2. Based on 1166 m core from 17 diamond drillholes.
3. Based on 1758 m of core from 24 diamond drillholes.

FIG. 13

MINNOVA INC. SAMATOSUM PROJECT PRELIMINARY GEOTECHNICAL INVESTIGATION	PITEAU ASSOCIATES GEOTECHNICAL CONSULTANTS VANCOUVER CALGARY
DISTRIBUTION OF RQD IN THE TWO BASIC ROCK MASS UNITS	BY: PMH/BL DATE: FEB. '88 APPROVED: [illegible] DWG: 946-13

Drawn Reviewed Date



GRAIN SIZE DISTRIBUTIONS

Figure 14

PITEAU & ASSOCIATES

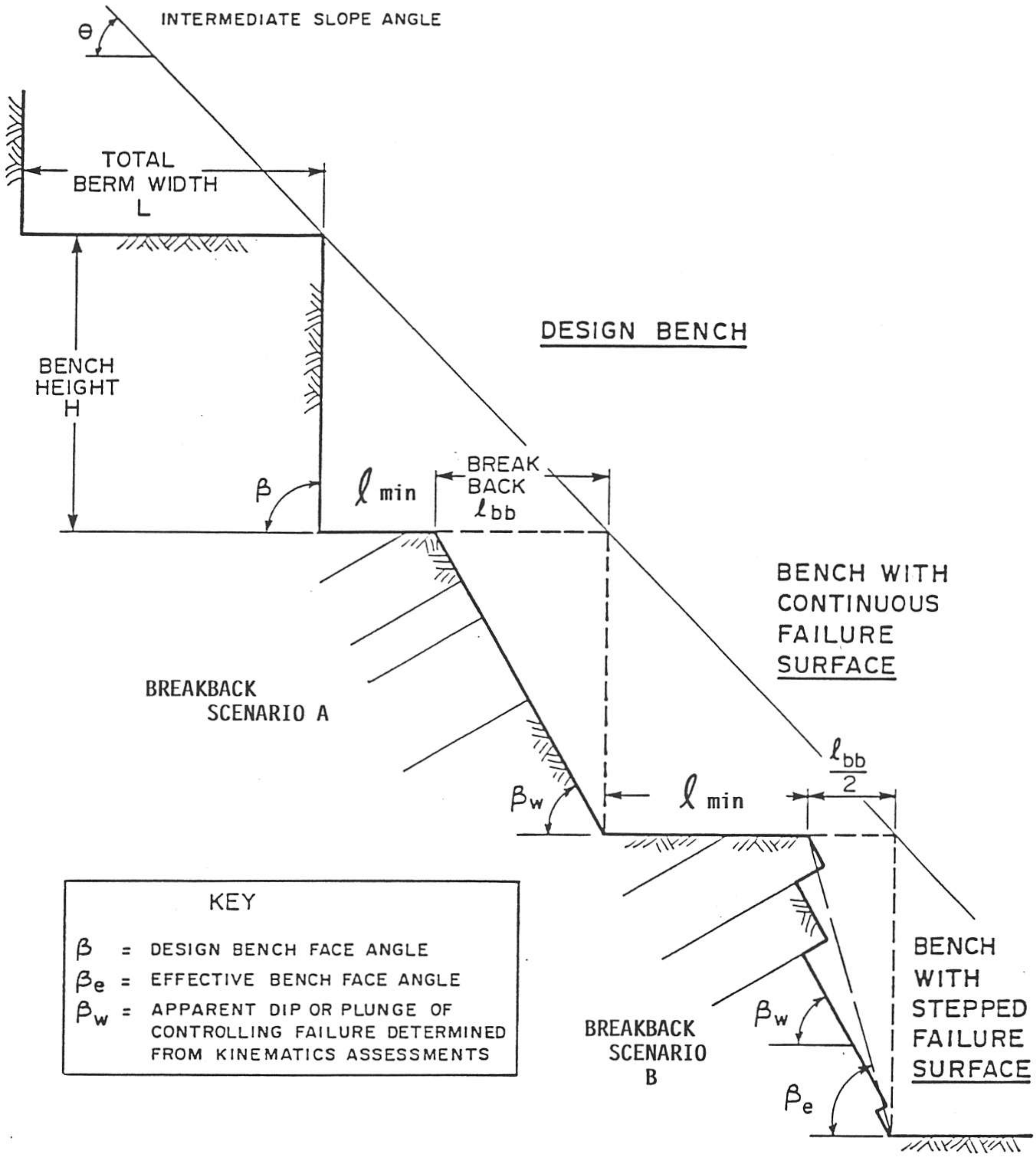


FIG. 15

JOB NUMBER

MINNOVA INC.
SAMATOSUM PROJECT
PRELIMINARY GEOTECHNICAL INVESTIGATION

PITEAU & ASSOCIATES
GEOTECHNICAL CONSULTANTS
VANCOUVER CALGARY

SLOPE GEOMETRY PARAMETERS

BY: B.L.	DATE: 02 88
APPROVED:	DWG: 946-15

JOB NUMBER 946

FIG. 16

NOTE: See Fig. 2 for SYMBOLS, LEGEND, NOTES and Section location.

MINNOVA INC.
SAMATOSUM PROJECT
PRELIMINARY GEOTECHNICAL INVESTIGATION

PITEAU ASSOCIATES
GEOTECHNICAL CONSULTANTS
VANCOUVER CALGARY

CROSS-SECTION THROUGH WASTE DUMP SITE A

BY	DATE
HWN BL	FEB, '88
APPROVED	DWG: 946-16

JOB NUMBER 946

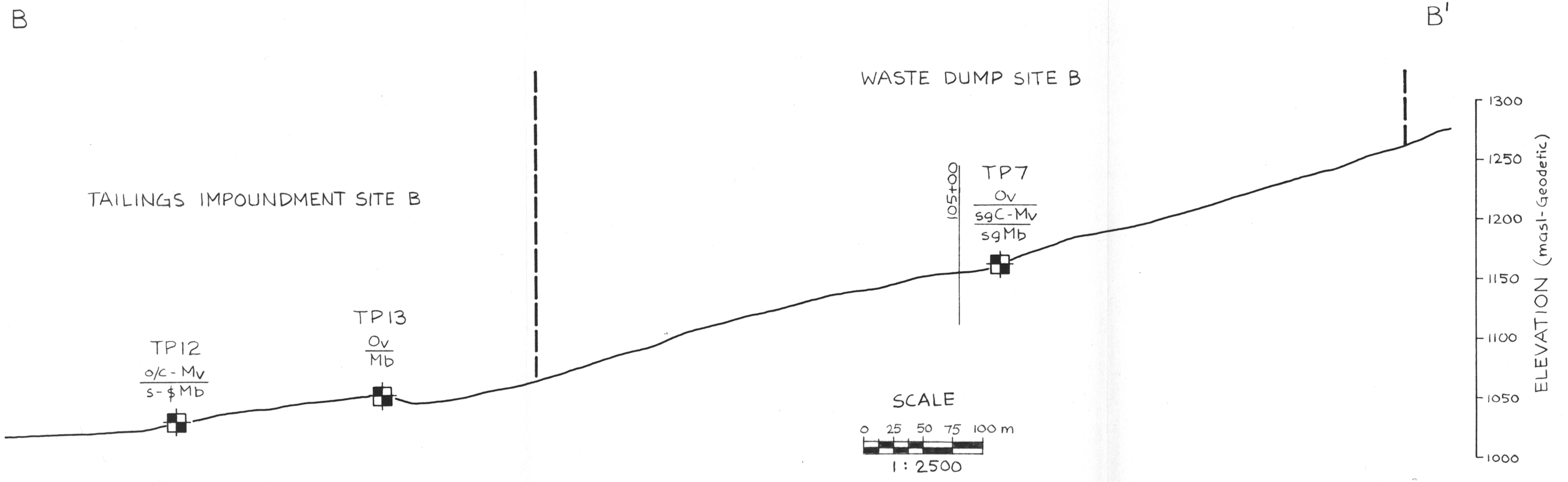


FIG. 17

NOTE: See Fig. 2 for SYMBOLS, LEGEND, NOTES and Section locations.

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CROSS-SECTION THROUGH WASTE DUMP SITE B AND TAILINGS IMPOUNDMENT SITE B

BY	DATE
HWN/BL	FEB '88
APPROVED	DWG:
[illegible]	946-17

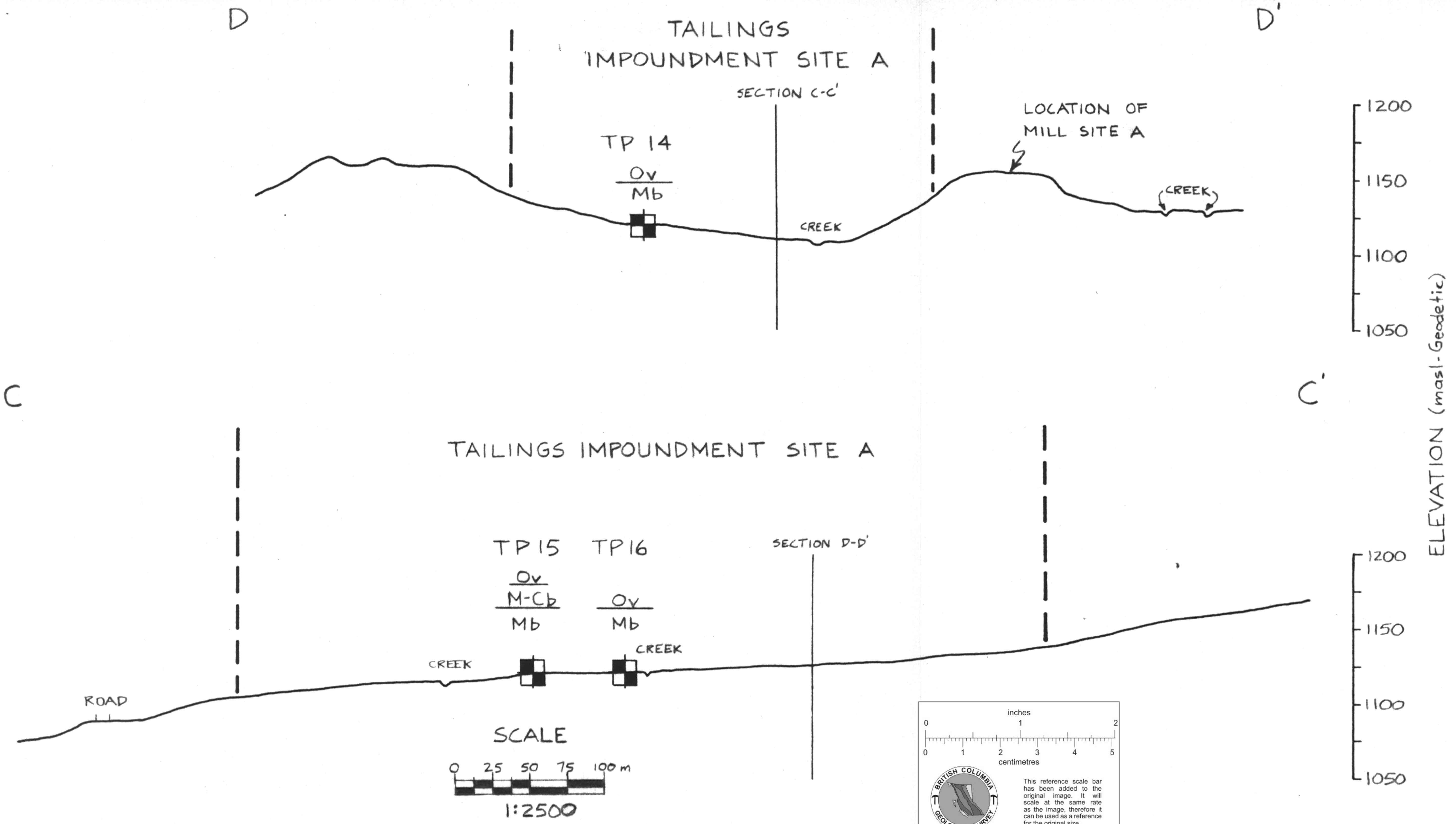

FIG. 18

NOTE: See Fig. 2 for SYMBOLS, LEGEND, NOTES and section locations.

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CROSS-SECTIONS THROUGH TAILINGS IMPOUNDMENT SITE A

BY	DATE
HWN	FEB '88
APPROVED:	DWG: 946-17

TABLE I

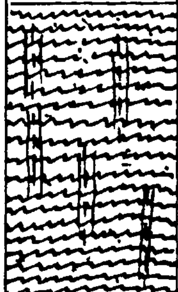
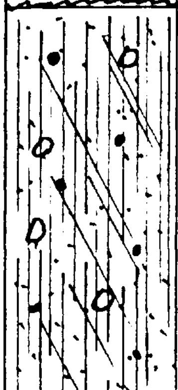
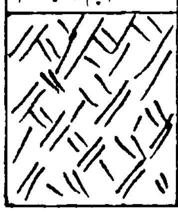
SUMMARY OF POINT LOAD INDEX TESTING RESULTS[1]

ROCK TYPE	DEGREE OF ALTERATION	APPROXIMATE UNIAXIAL COMPRESSIVE STRENGTH[2,3]				ANISOTROPY[4] INDEX
		PARALLEL TO FOLIATION		PERPENDICULAR TO FOLIATION		
		MPa	psi	MPa	psi	
MAFIC PYROCLASTICS	FRESH TO LIGHT	41.9 (12.7-74.4) [20]	6,075	-	-	-
	FRESH WITH IRON STAINING ON FRACTURES	69.6 (10.8-153.6) [10]	10,090	255.6 (111.6-338.4) [10]	37,060	3.7
	LIGHT	42.5 (14.9-91.2) [10]	6,160	-	-	-
MAFIC TUFF	LIGHT	30.9 (5.5-79.2) [20]	4,480	133.8 (115.2-156.0) [5]	19,400	4.3
	LIGHT TO MODERATE	14.0 (10.5-21.6) [4]	2,030	-	-	-
	MODERATE	12.3 (7.2-30.2) [10]	1,780	-	-	-
MUDDY TUFF	LIGHT	16.2 (8.0-42.0) [22]	2,350	-	-	-

NOTES:
1. Based on Point Load Index testing of NQ core from four diamond drillholes.
2. Median Uniaxial Compressive Strength. Range in parentheses (). Number of tests in brackets [ ].
3. Uniaxial compressive strength (U.C.S.) determined by multiplying Point Load Index by 24.
4. Anisotropy Index = Axial U.C.S./Diametral U.C.S.

TABLE II

SUMMARY OF SURFICIAL SOIL TYPES AND STRATIGRAPHY
FROM TESTS PITS AND RECONNAISSANCE MAPPING

GRAPHIC LOG	TERRAIN SYMBOL	UNIFIED SOIL CLASS.	OBSERVED STRATUM THICKNESS (m)	DESCRIPTION
	O_v	PT OL/OH	0~1.0	<u>ORGANIC VENEER</u> Black; dark-reddish brown. Wet-dry. Soft, compressible-firm, compact. Roots, Amorphous-Fibrous PEAT & Organic SILT -SANDY SILT. Occasionally intermixed with Colluvial debris (reworked Till & Bedrock fragments).
	M_v/C_v	ML SM-GM SW-GW	0~2.0	<u>WEATHERED TILL or COLLUVIUM (reworked TILL/WEATHERED BEDROCK)</u> Light, medium, reddish, greyish brown. Compact-dense. Moist-dry. Well graded. Sandy SILT-Silty SAND & GRAVEL-Sandy GRAVEL. Trace clay. Occasional Cobbles & Boulders - 30 cm. Occasional Roots & Organic Fibre. Coarser constituents subround to angular & commonly strongly weathered.
	M_v/M_b	ML SM-GM SW-GW	0>>3.0	<u>TILL</u> Light-medium greyish brown-grey. Compact-very dense, hard. Dry with moist-wet zones generally associated with gravelly lenses. Well graded. Sandy SILT-Silty SAND & GRAVEL-Sandy GRAVEL. Trace Clay. Occasional Cobbles & Boulders - 30m. Coarser constituents subrounded-angular & occasionally strongly weathered.
	R			<u>WEATHERED BEDROCK</u> Moderately-completely weathered/oxidized. Strongly foliated, friable. Hardness R0-R1.

NOTES: 1. See Appendix A for detailed test pit logs.
2. See Fig. 2 for surficial geology map and description of Terrain Classification System.
3. Based on Unified Soil Classification System (Wagner, A.A., 1957).

TABLE III

SUMMARY OF DESIGN SECTORS, KINEMATIC AND ROCK MASS COMPETENCY ASSESSMENTS AND RECOMMENDED PRELIMINARY SLOPE DESIGN

DESIGN SECTOR[1]	WALL TYPE[2]	SLOPE AZIMUTH[1] (°)	CRITICAL PLANE FAILURES[3]: DISCONTINUITY SET[4]	CRITICAL PLANE FAILURES[3]: APPARENT DIP[5] (°)	CRITICAL PLANE FAILURES[3]: FACTOR OF SAFETY[6]	CRITICAL WEDGE FAILURES[7]: DISCONTINUITY SETS INVOLVED	CRITICAL WEDGE FAILURES[7]: APPARENT PLUNGE[8] (°)	CRITICAL WEDGE FAILURES[7]: FACTOR OF SAFETY[6]	DEGREE OF KINEMATIC CONTROL[9]	APPARENT DIP OR PLUNGE OF FAILURE CONSIDERED TO CONTROL BENCH STABILITY, β_w (°)[10]	APPENDIX FIGURE[11]
I	Footwall	025	F1	42	0.4 - 0.5	F1/B3 F1/B2	42 42	0.5 - 0.6 0.7 - 1.0	Very Strong	42	B-1
II	Oblique Footwall	055	F1 (oblique)	45	0.4 - 0.5	F1/B1 F1/B2 F1/B3	29 35 43	1.0 - 1.5 0.7 - 1.0 0.5 - 0.6	Strong	37	B-2
III	Oblique Endwall	085	NONE	-	-	F1/B1 F1/B2 B1/B2	27 37 67	1.0 - 1.5 0.7 - 1.0 0.3 - 0.6	Moderate-Strong	37	B-3
IV	Endwall	115	B2 B3	62 88	0.2 - 0.4 <0.1	F1/B1 B1/B2 F1/B2	32 48 61	1.0 - 1.5 0.7 - 1.0 0.3 - 0.6	Strong	61	B-4
V	Oblique Endwall	145	B2 B1	62 65	0.2 - 0.4 0.2 - 0.3	A2/B2	53	0.6 - 1.2	Strong	62	B-5
VI	Oblique Hanging Wall	175	B1	66	0.2 - 0.3	A2/B2 A1/B1 A2/B3 A2/B1 B1/B3	43 48 54 56 56	0.6 - 1.2 0.7 - 1.3 0.6 - 1.1 0.4 - 0.8 0.8 - 1.5	Moderate-Strong	60	B-6
VII	Hanging Wall	205	NONE	-	-	A1/B1 A2/B2 A2/B3 A2/B1 B1/B3	38 42 49 50 51	0.7 - 1.3 0.6 - 1.2 0.6 - 1.1 0.4 - 0.8 0.8 - 1.5	Weak-Moderate	47	B-7
VIII	Oblique Hanging Wall	235	A2	56	0.2 - 0.5	A1/B1 A2/B2 A2/B3 A2/B1 B1/B3	36 49 52 53 54	0.7 - 1.3 0.6 - 1.2 0.6 - 1.1 0.4 - 0.8 0.8 - 1.5	Weak-Moderate	51	B-8
IX	Oblique Endwall	265	A1 A2	47 57	0.3 - 0.6 0.2 - 0.5	A1/B1 B1/B3	44 66	0.7 - 1.3 0.8 - 1.5	Moderate-Strong	47	B-9
X	Endwall	295	A1 (oblique)	50	0.3 - 0.6	F1/A1	34	0.8 - 1.3	Weak	50	B-10
XI	Oblique Endwall	325	NONE	-	-	F1/A1	28	0.8 - 1.3	Very Weak	-	B-11
XII	Oblique Footwall	355	F1 (oblique)	45	0.4 - 0.5	F1/A1	30	0.8 - 1.3	Weak	46	B-12

ALTERNATIVE BENCH DESIGN BASED ON KINEMATICALLY POSSIBLE FAILURE MODES[12]

DESIGN SECTOR[1]	BENCH HEIGHT, H (m)[13]	DESIGN BENCH FACE ANGLE, β (°)	BREAKBACK[14] ℓ_{bb} (m): A	BREAKBACK[14] ℓ_{bb} (m): B	EFFECTIVE BENCH FACE ANGLE, β_e (°): A	EFFECTIVE BENCH FACE ANGLE, β_e (°): B	DESIGN BERM WIDTH, L (m)[15]: A	DESIGN BERM WIDTH, L (m)[15]: B	INTERMEDIATE SLOPE ANGLE, θ: A	INTERMEDIATE SLOPE ANGLE, θ: B
I	10	90	11.1	5.6	42	61	19.1	13.6	27.6	36.3
	20		22.2	11.1			32.2	21.1	34.6	43.5
II	10	90	13.3	6.7	37	56	21.3	14.7	25.1	34.2
	20		26.5	13.3			36.5	23.3	28.7	40.6
III	10	90	13.3	6.7	37	56	21.3	14.7	25.1	34.2
	20		26.5	13.3			36.5	23.3	28.7	40.6
IV	10	90	5.5	2.8	61	74	13.5	10.8	36.5	42.8
	20		11.1	5.5			21.1	15.5	43.5	52.2
V	10	90	5.3	2.7	62	75	13.3	10.7	36.9	43.1
	20		10.6	5.3			20.6	15.3	44.2	52.6
VI	10	90	5.8	2.9	60	74	13.8	10.9	35.9	42.5
	20		11.5	5.8			21.5	15.8	42.9	51.7
VII	10	90	9.3	4.7	47	65	17.3	12.7	30.0	38.2
	20		18.7	9.4			28.7	19.4	34.9	45.9
VIII	10	90	8.1	4.1	51	58	16.1	12.1	31.8	39.6
	20		16.2	8.1			26.2	18.1	37.4	47.9
IX	10	90	9.3	4.7	47	65	17.3	12.7	30.0	38.2
	20		18.7	9.3			28.7	19.4	34.9	45.9
X	10	90	8.4	4.2	50	67	16.4	12.2	31.4	39.3
	20		16.8	8.4			26.8	18.4	36.7	47.4
XI	10	90	0	0	90	90	8.0	8.0	51.3	51.3
	20		0	0			10.0	10.0	63.4	63.4
XII	10	90	9.7	4.9	46	64	17.7	12.9	29.5	37.8
	20		19.3	9.7			29.3	17.7	34.3	45.4

RECOMMENDED PRELIMINARY SLOPE DESIGN[12, 17]

DESIGN SECTOR[1]	PRINCIPAL ROCK TYPES (Rock Mass Units)[16]	ASSESSMENT OF ROCK MASS COMPETENCY[16]	BENCH HEIGHT, H (m)	DESIGN BENCH FACE ANGLE, β (°)	DESIGN BERM WIDTH, L (m)	INTERMEDIATE SLOPE ANGLE, θ (°)	COMMENTS
I II III	Lightly to Strongly Altered Muddy Tuff/Sediments/Chert Mafic Tuff	Poor-Moderate	10	70	8	41	Plane failures on foliation discontinuities (Set F) or wedge failures on foliation and Set B discontinuities could result in loss of berm access or complete loss of berms. Single benches recommended to limit size of potential failures. Inclined bench faces and controlled blasting or ripping recommended to reduce potential rockfall generation and ravelling which may occur due to relatively incompetent nature of the rock mass. If major, continuous faults occur parallel to foliation and/or Discontinuity Set B, large plane or wedge failures involving several benches or the whole slope could result. Remedial measures consisting of artificial support, buttressing, flattening or slope despressurization may be required.
IV V VI	Unaltered to Lighlty Altered Mafic Pyroclastics; Mafic Tuff	Moderate-Good	20 (20)	90 (70)	15 (12)	53 (46)	Plane failures on Discontinuity Set B or wedge failures on Discontinuity Sets B and A control bench design in competent rock (i.e. Mafic Pyroclastics) and may result in loss of access to some berms. Instability of the overall slope not anticipated provided major, continuous faults do not occur which are flatter than the observed discontinuity sets. Design shown in parentheses should be used for incompetent rock masses (i.e. Muddy Tuff, Sericite Altered Mafic Tuff, etc.) where inclined bench faces and controlled blasting or ripping are recommended to reduce potential rockfall generation and ravelling.
VII VIII IX X	Lightly to Strongly Altered Mafic Tuff; Muddy Tuff/Sediments/Chert	Poor-Moderate	20 (20)	90 (70)	19 (14)	47 (43)	Plane failures on Discontinuity Set A or wedge failures on Discontinuity Sets A and B control bench design in competent rock (i.e. Mafic Pyroclastics) and may result in loss of access to some berms. Instability of the overall slope not anticipated provided major, continuous faults do not occur which are flatter than the observed discontinuity sets. Design shown in parentheses should be used for incompetent rock masses (i.e. Muddy Tuff, Sericite Altered Mafic Tuff, etc.) where inclined bench faces and controlled blasting or ripping are recommended to reduce potential rockfall generation and ravelling.
XI XII	Lightly to Strongly Altered Muddy Tuff/Sediments/Chert, Mafic Tuff	Poor-Moderate	20	70	12	46	Oblique plane failures on foliation discontinuities (Set F), or wedges on foliation and Set A discontinuities could result in loss of access to some berms. Inclined bench faces and controlled blasting or ripping recommended to reduce potential rockfall generation and ravelling which may occur due to relatively incompetent nature of the rock mass. If major, continuous faults occur parallel to foliation and/or Discontinuity Set A, large oblique plane or wedge failures involving several benches or the whole slope could result. Remedial measures consisting of artificial support, buttressing, flattening or slope despressurization may be required.

NOTES:

1. Kinematic assessments were conducted for twelve discrete slope orientations or Design Sectors at 30° intervals of slope azimuth (dip direction) assuming a footwall slope azimuth of 025°.
2. Wall Type refers to the general orientation of foliation with respect to the slope (e.g. footwalls strike parallel to foliation and dip in the same direction, endwalls strike normal to foliation, and hanging walls strike parallel to foliation and dip in the opposite direction).
3. Kinematically possible planar failures which generally strike within about 45° parallel to the slope and which have lower bound Factors of Safety of less than 1.0 are designated Critical Plane Failures.
4. Critical Plane Failures which strike about 20° to 45° parallel to the slope are indicated in parentheses as oblique.
5. Apparent Dip is the dip of the plane relative to the slope which must be undercut for the plane to become a kinematically possible plane failure.
6. Factor of Safety is given as a range. The lower bound is the Factor of Safety if all discontinuities involved occur along faults. The upper bound is the Factor of Safety if all discontinuities occur along cross joints or foliation joints.
7. Kinematically possible wedge failures whose line of intersection strikes within about 45° to the slope azimuth and which have lower bound Factors of Safety of less than 1.0 are designated Critical Wedge Failures.
8. Apparent Plunge is the dip of the line of intersection relative to the slope which must be undercut for the wedge to become a kinematically possible wedge failure.
9. Degree of Kinematic Control is a qualitative assessment of the relative importance that kinematically possible failure modes may have on bench stability, and is based on the Factor of Safety, orientation, type of failure and intensity of the discontinuity sets involved.
10. Based on assessment of apparent dips and plunges of the various critical plane and wedge failures and Degree of Kinematic Control.
11. Refers to lower hemisphere projections for each design sector given in Appendix B.
12. Bench geometry parameters are illustrated in Fig. 15.
13. Alternative slope geometries have been prepared for single (i.e. 10m) and double (i.e. 20m) high benches.
14. Detailed assessments of like bench crest breakback were not possible as no benches have as yet been exposed. To evaluate the sensitivity of slope geometry to possible breakback, two possible breakback scenarious have been assessed. Scenario A assumes the benches breakback to the apparent dip or plunge considered to control bench stability (i.e. β_w). Scenario B assumes the benches breakback only half the distance indicated by Scenario A.
15. Design Berm Width is based on the amount of breakback (ℓ_{bb}) plus a minimum berm width required to provide access to the slope and adequate rockfall protection (ℓ_{min}). For 10m single benches, ℓ_{min} is taken as 8m; for 20m double benches, ℓ_{min} is assumed to be 10m.
16. Based on the geologic interpretation and geotechnical core logging conducted by Minnova personnel and our assessment of the rock mass, two basic rock mass units are identified. Lightly to strongly sericite altered Muddy Tuff, Argillite, Greywacke, Chert and related rocks and moderately to strongly sericite altered Mafic Tuff are considered together as one rock mass unit with an overall poor to moderate competency. This unit may occur in all design sectors. Unaltered to lightly altered Mafic Pyroclastics and Mafic Tuff are considered together as the other rock mass unit and have an overall moderate to good competency. These rocks are expected to form significant components of pit walls in Design Sectors IV through X.
17. Recommended Preliminary Slope Designs are based on an assessment of kinematically possible failures, rock types and rock mass competency for each design sector. Based on actual conditions encountered during mining of the initial benches, modification of preliminary designs may be required. Periodic updates and refinement of slope designs throughout mine life are recommended to achieve the optimum overall slope design.
18. Additional comments and recommendations are given in Section 5.5.

APPENDIX A

TEST PIT LOGS

TEST PIT NO. 1

METHOD OF DIGGING TEST PIT *Case 580 Backhoe*

LOCATION *Waste Dump Site A - gully in mod. sloping ground*

GROUND SURFACE ELEVATION *±1350m*

GROUNDWATER ELEVATION (at time of digging) -

GS - GRAB SAMPLE *(All Samples)*

U - UNDISTURBED SAMPLE

Depth-m	Depth-ft	Symbol	Description	Samples	Comments
		SM/GM	Med. - red brn., compact, dry, well graded, silty SAND + GRAVEL w/ Cobbles -15 cm., subrnd-ang., tr. org. fibre	1-1 (0.3)	str. wx./alt. COLLUVIUM or reworked TILL (C_v or M_v)
		SM	Lt.-med. - red brn. silty SAND + GRAVEL w/ occ. wx B/R clasts, tr. clay, tr. org. fibre	1-2 (0.7)	Mod. wx./alt. COLLUVIUM or TILL (C_v or M_v)
1	5	B/R	Wx., foliated, R1 BEDROCK T.D. 1.1m		(R) Dry Photo A1-14,15
2					
3	10				
4					
5	15				
6	20				
7	25				

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LOG OF TEST PIT NO. 1	By: PMH	Date: 02.88
	Job: 946	Dwg: B-1

TEST PIT NO. 2

METHOD OF DIGGING TEST PIT Case 580 Backhoe

LOCATION Waste Dump Site A - gully in mod. sloping ground

GROUND SURFACE ELEVATION ± 1350 m

GROUNDWATER ELEVATION (at time of digging) -

GS - GRAB SAMPLE (All Samples)

U - UNDISTURBED SAMPLE

Depth-m	Depth-ft	Symbol	Description	Samples	Comments
0–1		OL/OH -PT	Med.-dk. brn., v. wet, Org. SILT & fiberous PEAT w/ tr.-some Sand, Gravel & str. wx. rock frags.	2-4	Frozen. Mixed org. & wx./alt. COLLUVIUM or reworked TILL (C_v or M_v)
1–3	5	SM	Lt.-med. gr. brn., firm to dense w/ occ. softer zones, unstratified, well graded Silty SAND & GRAVEL subrnd-ang, occ. wx. rock frags.	2-1 (1.3) 2-2 (2.0) 2-3 (2.8)	Sl. alt./wx. TILL (M_b) softer, wet zone @ 2.8 W.C. 17.6%, Sieve (Fig.)
3	10		T.D. 3.0 m		Moist only / No measurable seepage or accumulation in base of pit Photo A1-17
4					
5	15				
6	20				
7	25				

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LOG OF TEST PIT NO. 2

By: PMH | Date: 02 88 | Job: 946 | Dwg: B-2

TEST PIT NO. 3

METHOD OF DIGGING TEST PIT CASE 580 Backhoe

LOCATION Saddle above Waste Dump Site A - flat, poorly drained swale

GROUND SURFACE ELEVATION ±1400 m

GROUNDWATER ELEVATION (at time of digging) -

GS - GRAB SAMPLE (All Samples)

U - UNDISTURBED SAMPLE

Depth-m	Depth-ft	Symbol	Description	Samples	Comments
		OL/OH	Med.-dk. brn., v. wet Org. SILT		Org. veneer (Ov)
		SM	Lt. yel.-red. brn, damp, firm Silty SAND & GRAVEL w/ str. wx. B/R clasts, tr. clay & org. fibre, sub ang.-ang.	3-1 (0.3)	Wx./alt COLLUVIUM or TILL (Cv or Mv)
1		B/R	str. wx., foliated BEDROCK		Dry
	5		TD. 0.6 m		Photo A1-18
2					
3	10				
4					
	15				
5					
6	20				
7					
	25				

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LOG OF TEST PIT NO. 3

By: PMH | Date: 02.88
Job: 946 | Dwg: B-3

TEST PIT NO. 4

METHOD OF DIGGING TEST PIT Case 580 Backhoe

LOCATION Saddle above Waste Dump Site A - poorly drained depression

GROUND SURFACE ELEVATION ±1400 m

GROUNDWATER ELEVATION (at time of digging) ±1397 m

GS - GRAB SAMPLE (All Samples)

U - UNDISTURBED SAMPLE

Depth-m	Depth-ft	Symbol	Description	Samples	Comments
		OL/OH -PT	Dk. brn.- blk. soft - firm, amorphous - fiberous PEAT and Organic SILT w/ occ. lenses of lt. yel.-brn. Silty Sand - Sandy Silt	4-1 (0.5)	Pp 1.0-1.5 Kg/cm² (Org. Silt) Pp 4-5 Kg/cm² (Silty Sand lenses) Org. Veneer/Pocket (O_v)
1	5	ML-SM	Mottled, lt. yel. brn. - lt. gr - red brn., firm - dense, well graded clayey, Sandy SILT - Silty SAND + GRAVEL w/ occ. Cobbles - 10cm. subrnd - ang. Becomes softer w/ depth	4-2 (1.1)	Wx./alt. TILL (M_v) Pp 1.5-2.0 Kg/cm²
2				4-3 (2.0)	W.C. 26.5%, Sieve + Hydrometer (Fig.)
				4-4 (2.8)	Pp 1.0 Kg/cm² ▽ Minor Seepage @ B/R sfc.
3	10	B/R	Wx. BEDROCK T.D. 3.1 m		Standpipe installed at bottom of Test Pit
4					
	15				
5					
6	20				
7					
	25				

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LOG OF TEST PIT NO. 4

By: PMH	Date: 02 88
Job: 946	Dwg: B-4

TEST PIT NO. 5

METHOD OF DIGGING TEST PIT Case 580 Backhoe

LOCATION Waste Dump Site A - Mod. sloping ground

GROUND SURFACE ELEVATION ±1300 m

GROUNDWATER ELEVATION (at time of digging) -

GS - GRAB SAMPLE (All Samples)

U - UNDISTURBED SAMPLE

Depth-m	Depth-ft	Symbol	Description	Samples	Comments
		PT	Org. soil, roots		Org. Veneer (Ov)
1	5	SM ↓ GM	Med. gr., dense - v. dense, Silty SAND & GRAVEL, subrnd - ang. grades coarser w/ depth	5-1	Sl. wx./alt. TILL (M_b)
2					
3	10		TD 3.0m		Dry
4					Photo A1-19,20
5	15				
6	20				
7	25				

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LOG OF TEST PIT NO. 5	By: PMH	Date: 02.88
	Job: 946	Dwg: B-5

TEST PIT NO. 6

METHOD OF DIGGING TEST PIT Case 580 Backhoe

LOCATION Waste Dump Site B - Mod. Sloping ground

GROUND SURFACE ELEVATION ±1255m

GROUNDWATER ELEVATION (at time of digging) -

GS - GRAB SAMPLE (All Samples)

U - UNDISTURBED SAMPLE

Depth-m	Depth-ft	Symbol	Description	Samples	Comments
		OL/OH	Red brn., compact, org. Sandy SILT w/ occ Gravel + org. fibre. ang.	6-2 (0.5)	P_p 0.5-1.0 Kg/cm^2 Mixed Org. + Wx./alt. COLLUVIUM or reworked TILL (C_v or M_v)
1 – 2	5	SM/GM	Med. gr. brn., dense, well graded Silty SAND + GRAVEL w/ occ. Cobbles to 10cm. Subang - ang Density > w/depth.	6-1 (1.7)	Pp. > 5.0 Kg/cm^2 Sl. Wx/alt. TILL (M_b)
3	10		TD 3.0m		Dry Photo A1-21
4					
5	15				
6	20				
7	25				

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LOG OF TEST PIT NO. 6

By: PMH | Date: 02.88 | Job: 946 | Dwg: B-6

TEST PIT NO. 7

METHOD OF DIGGING TEST PIT Case 580 Backhoe

LOCATION Waste Dump Site B
-Mod. sloping ground

GROUND SURFACE ELEVATION ±1160 m

GROUNDWATER ELEVATION (at time of digging) -

[GS] - GRAB SAMPLE (All Samples)

[U] - UNDISTURBED SAMPLE

Depth-m	Depth-ft	Symbol	Description	Samples	Comments
		PT	Org. soil + roots		Org. Veneer
		SW-GW	Med. brn, dense, well graded SAND + GRAVEL w/ tr. silt + occ. cobbles. rnd - subang.	7-1	Mod. wx./alt. COLLUVIUM or reworked TILL (sgC_v or sgM_v)
1, 2	5	SW-GW	Med. gr., v.dense, well graded SAND + GRAVEL w/ tr. silt subang-ang. Sl. finer gradation than above.	7-2	Sl. alt TILL (sgM_b) Sieve (Fig.)
3, 4, 5, 6, 7	10, 15, 20, 25		TD 2.6m		Dry

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LOG OF TEST PIT NO. 7

By: PMH	Date: 02 88
Job: 946	Dwg: B-7

TEST PIT NO. 8

METHOD OF DIGGING TEST PIT Case 580 Backhoe

LOCATION Waste Dump Site A
- beside gully in mod. sloping ground

GROUND SURFACE ELEVATION ±1150m

GROUNDWATER ELEVATION
(at time of digging) —

GS – GRAB SAMPLE

U – UNDISTURBED SAMPLE

Depth-m	Depth-ft	Symbol	Description	Samples	Comments
		PT	Org. soil + roots ▽ seepage @ base of organics		Org. Veneer (Ov)
1 2	5	SW-GW	Med. gr. brn., dense, well graded SAND + GRAVEL w/tr. to some Silt + occ. Cobbles subrnd. - ang.	8-1 (1.5) 8-2	Sl. alt./Wx TILL (sgM$_b$) Minor seepage in gravelly zones Gravelly zone below 8-1
3 4 5 6 7	10 15 20 25		T.D. 2.5m		Standpipe installed at base of Test Pit

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LOG OF TEST PIT NO. 8

By: PMH	Date: 02 88
Job: 946	Dwg: B-8

TEST PIT NO. 9

METHOD OF DIGGING TEST PIT Case 580 Backhoe

LOCATION Below Waste Dump Site A - beside gully in mod. sloping ground

GROUND SURFACE ELEVATION ±1125 m

GROUNDWATER ELEVATION (at time of digging) —

[GS] – GRAB SAMPLE (All Samples)

[U] – UNDISTURBED SAMPLE

Depth-m	Depth-ft	Symbol	Description	Samples	Comments
0–1		SM	Med.-dk. brn., loose-compact, well graded Silty SAND + GRAVEL w/ occ. org. fibre/debris. Subang.-ang.	9-1 (1.0)	Wx./alt. COLLUVIUM or TILL (C_v or M_v)
1–3	5	SM ↓ GM	Med. brn.gr., compact-firm, well graded, Silty SAND + GRAVEL w/ occ. Cobbles + Boulders – 20 cm subrnd-ang. Occ. roots. % Cobbles, Boulders increases w/ depth.	9-2 (2.4)	Sl. wx/alt. TILL (M_b)
3	10		TD 3.1 m		– Dry Photo A26
4					
5	15				
6	20				
7	25				

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LOG OF TEST PIT NO. 9

By: PMH | Date: 02 88
Job: 946 | Dwg: B-9

TEST PIT NO. 10

METHOD OF DIGGING TEST PIT Case 580 Backhoe

LOCATION Waste Dump Site A
- gully in mod. sloping ground
GROUND SURFACE ELEVATION ±1180 m
GROUNDWATER ELEVATION
(at time of digging) –

GS – GRAB SAMPLE (All Samples)
U – UNDISTURBED SAMPLE

Depth-m	Depth-ft	Symbol	Description	Samples	Comments
		PT	Org. Soil, roots		Org. Veneer (Ov)
		SM	Med.-red. brn. Silty SAND + GRAVEL w/roots		Wx/alt. COLLUVIUM or reworked TILL (Cv or Mv)
1		SW	Mottled. med. gr.-brn. SAND + GRAVEL w/ tr. Silt V Sandy in places	10-1 (1.0)	sl. wx/alt. TILL or COLLUVIUM (sCv or sMv)
	5	SW-GW	Red brn., dense-v. dense, well graded SAND + GRAVEL w/ occ Cobbles + tr. Silt, Wx. rock frags.	10-2 (1.5)	str. alt./Wx TILL (Mv) ∇ Seepage <0.1 lpm
2		SM/GM	Med. gr. w/ red. brn zones, v. dense Silty SAND + GRAVEL. Subrnd-ang.	10-3 (2.5)	TILL (Mb)
			T.D. 2.3 m		
3	10				Photo A2-7
4					
	15				
5					
6	20				
7					
	25				

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LOG OF TEST PIT NO. 10	By: PMH	Date: 02.88
	Job: 946	Dwg: B-10

TEST PIT NO. 11

METHOD OF DIGGING TEST PIT Case 580 Backhoe

LOCATION Tailings Impoundment
Site B - gully in shallowly sloping ground

GROUND SURFACE ELEVATION ±1035m

GROUNDWATER ELEVATION (at time of digging) -

GS - GRAB SAMPLE (All Samples)

U - UNDISTURBED SAMPLE

Depth-m	Depth-ft	Symbol	Description	Samples	Comments
		SM	DK. brn., org. Silty SAND & GRAVEL w/ roots & cobbles		Mixed org. & COLLUVIUM or reworked TILL (C_v or M_v)
1, 2	5	SM/GM-SW/GW	Lt.-med brn.-gr., dense, Silty SAND & GRAVEL w/ numerous Cobbles & Boulders - 30 cm. Some cleaner zones. Density increases w/ depth	11-1 (1.0)	Sl. wx/alt. TILL (M_b) Seepage from coarser zones in uphill side of test pit - mostly from upper 1 m.
3, 4, 5, 6, 7	10, 15, 20, 25		TD. 2.6 m		Stand pipe installed at base of Test Pit Photo A2-8, 9

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LOG OF TEST PIT NO. 11

By: PMH Date: 02.88
Job: 946 Dwg: B-11

TEST PIT NO. 12

METHOD OF DIGGING TEST PIT Case 580 Backhoe

LOCATION Tailings Impoundment
Site B - swale on shallowly sloping ground

GROUND SURFACE ELEVATION ±1030 m

GROUNDWATER ELEVATION (at time of digging) -

GS - GRAB SAMPLE (All Samples)

U - UNDISTURBED SAMPLE

Depth-m	Depth-ft	Symbol	Description	Samples	Comments
		OL/OH	Red-dk. brn. org. Sandy SILT w/ occ. Gravel, abdt roots + org. fibre	12-1 (0.3)	Mixed org. + COLLUVIUM or reworked TILL (C_v or M_v) Pp. 0.5-1.0 kg/cm²
1		SW	Med. brn.-gr., well graded SAND + GRAVEL w/ tr. Silt. Rnd.-subang	12-2 (1.0)	Sieve (Fig.) Pp. 1.0-2.0 kg/cm² TILL (sM_b)
2	5	SM	Med. gr., v. dense, Well graded Silty SAND + GRAVEL. Subang.-ang. Density increased w/ depth	12-3 (1.5)	W.C. 11.5% Sieve (Fig.) Pp >5 kg/cm² TILL (M_b)
			T.D 2.2 m		Dry
3	10				
4					
	15				
5					
6	20				
7					
	25				

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LOG OF TEST PIT NO. 12

By: PMH | Date: 02 88
Job: 946 | Dwg: B-12

TEST PIT NO. 13

METHOD OF DIGGING TEST PIT Case 580 Backhoe

LOCATION Tailings Impoundment Site B - humocky, flat ground

GROUND SURFACE ELEVATION ± 1050

GROUNDWATER ELEVATION (at time of digging) -

GS - GRAB SAMPLE (All Samples).

U - UNDISTURBED SAMPLE

Depth-m	Depth-ft	Symbol	Description	Samples	Comments
			Red. brn. org. Soil / roots		Org. Veneer (Ov)
1	5	SW ↓ GM	Lt-med. gr-brn., compact - v. dense / hard. SAND + GRAVEL w/ tr. - Some Silt + occ. Cobbles + Boulders. Subrnd. - ang. Grades siltier / cobbly w/ depth Density increases w/ depth Occ. Gravelly zones.	13-1 (0.5) 13-2 (1.2)	TILL (M_b)
2			T.D. 2.2 m)		Dry
3	10				
4	15				
5					
6	20				
7	25				

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LOG OF TEST PIT NO. 13	By: PMH	Date: 02 88
	Job: 946	Dwg: B-13

TEST PIT NO. 14

METHOD OF DIGGING TEST PIT Case 580 Backhoe

LOCATION Tailing Impoundment Site A - hummocky, flat ground

GROUND SURFACE ELEVATION ±1120 m

GROUNDWATER ELEVATION (at time of digging) —

GS - GRAB SAMPLE (All Samples)

U - UNDISTURBED SAMPLE

Depth-m	Depth-ft	Symbol	Description	Samples	Comments
		OL/OH	Red. brn. Org. Sandy SILT w/ roots + org. fibre. Soft	14-1 (0.2)	W.C. 53.6% Org. Veneer (O_v) P_p 1.0-1.5 kg/cm²
1, 2	5	SW/SM ↓ GW/GM	Med. brn., compact - dense SAND + GRAVEL w/ Cobbles, occ. Boulders ~30cm + tr. Silt occ. roots to 1.2 m depth. Boulder/Cobble content increases w/ depth. Subrnd - ang	14-2 (1.2)	TILL (M_b) P_p > 4 kg/cm²
3	10		TD 3.1 m		
4, 5	15				
6, 7	20, 25				

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LOG OF TEST PIT NO. 14

By: PMH
Date: 02 88
Job: 946
Dwg: B-14

TEST PIT NO. 15

METHOD OF DIGGING TEST PIT Case 580 Backhoe

LOCATION Tailings Impoundment Site A - Broad gully - Shallowly sloping

GROUND SURFACE ELEVATION ±1120m

GROUNDWATER ELEVATION (at time of digging) ______

GS - GRAB SAMPLE (All Samples)

U - UNDISTURBED SAMPLE

Depth-m	Depth-ft	Symbol	Description	Samples	Comments
		OL/OH	Red brn. org. SILT + fibre		Org. Veneer (Ov)
1	5	SM/GM (SW/GW)	Med. brn., compact-dense, moist, Silty SAND + GRAVEL Subrnd - ang. Sandy-gravelly zones	15-1	Wx. TILL or COLLUVIUM (Mb or Cb)
2			∇ Seepage <0.1 lpm in gravelly zone		
		— ? —	— ? —		— ? —
3	10	SM/GM	Med. gr., v. dense - hard Silty SAND + GRAVEL w/ occ. cobbles + Boulders	15-2	Sieve (Fig.) TILL (Mb)
			TD. 3.0m		
4					
	15				
5					
6	20				
7					
	25				

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LOG OF TEST PIT NO. 15

By: PMH | Date: 02 88
Job: 946 | Dwg: B-15

TEST PIT NO. 16

METHOD OF DIGGING TEST PIT 580 Case Backhoe

LOCATION Tailing Impoundment
Site A - Broad gully - shallowly sloping

GROUND SURFACE ELEVATION ±1125 m

GROUNDWATER ELEVATION
(at time of digging) —

GS - GRAB SAMPLE (All Samples)

U - UNDISTURBED SAMPLE

Depth-m	Depth-ft	Symbol	Description	Samples	Comments
		OL/OH	Org. Soil, roots		Organic Veneer (Ov)
1	5	SM/GM	Med. yel-gr. brn. Compact - v. dense Silty SAND + GRAVEL w/ occ. Cobbles + Boulders - 25cm Top 30cm sl. Wx. Fines content inc. w/ depth Rnd.-ang.	16-1 (1.5)	Sl. Wx. TILL (M_b) difficult digging @ bottom
2					
3	10		TD. 2.4 m		Dry
4	15				
5					
6	20				
7	25				

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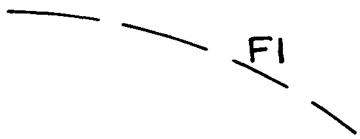
LOG OF TEST PIT NO. 16

By: PMH
Date: 02 88
Job: 946
Dwg: B-16

APPENDIX B

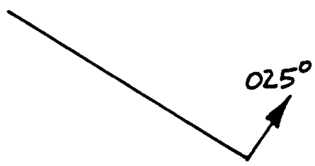
LOWER HEMISPHERE PROJECTIONS ILLUSTRATING KINEMATICALLY POSSIBLE FAILURE MODES

JOB NUMBER

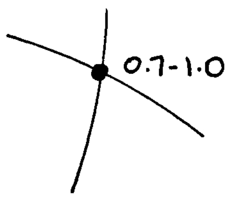


Plane Representing Peak Orientation of Foliation Discontinuities

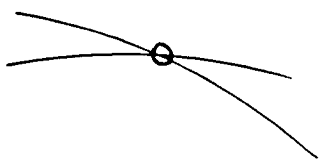
Plane Representing Peak Orientation of Discontinuity Set A1



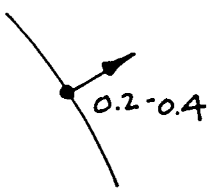
Orientation (Azimuth) of Design Sector



Kinematically Possible Wedge Failure (Range of Factor of Safety Indicated)



Release Feature for Possible Plane Failure



Kinematically Possible Plane Failure (Range of Factor of Safety Indicated)

Critical Kinematically Possible Wedge or Plane Failure

Apparent Plunge or Dip of Failure Considered to Control Bench Stability

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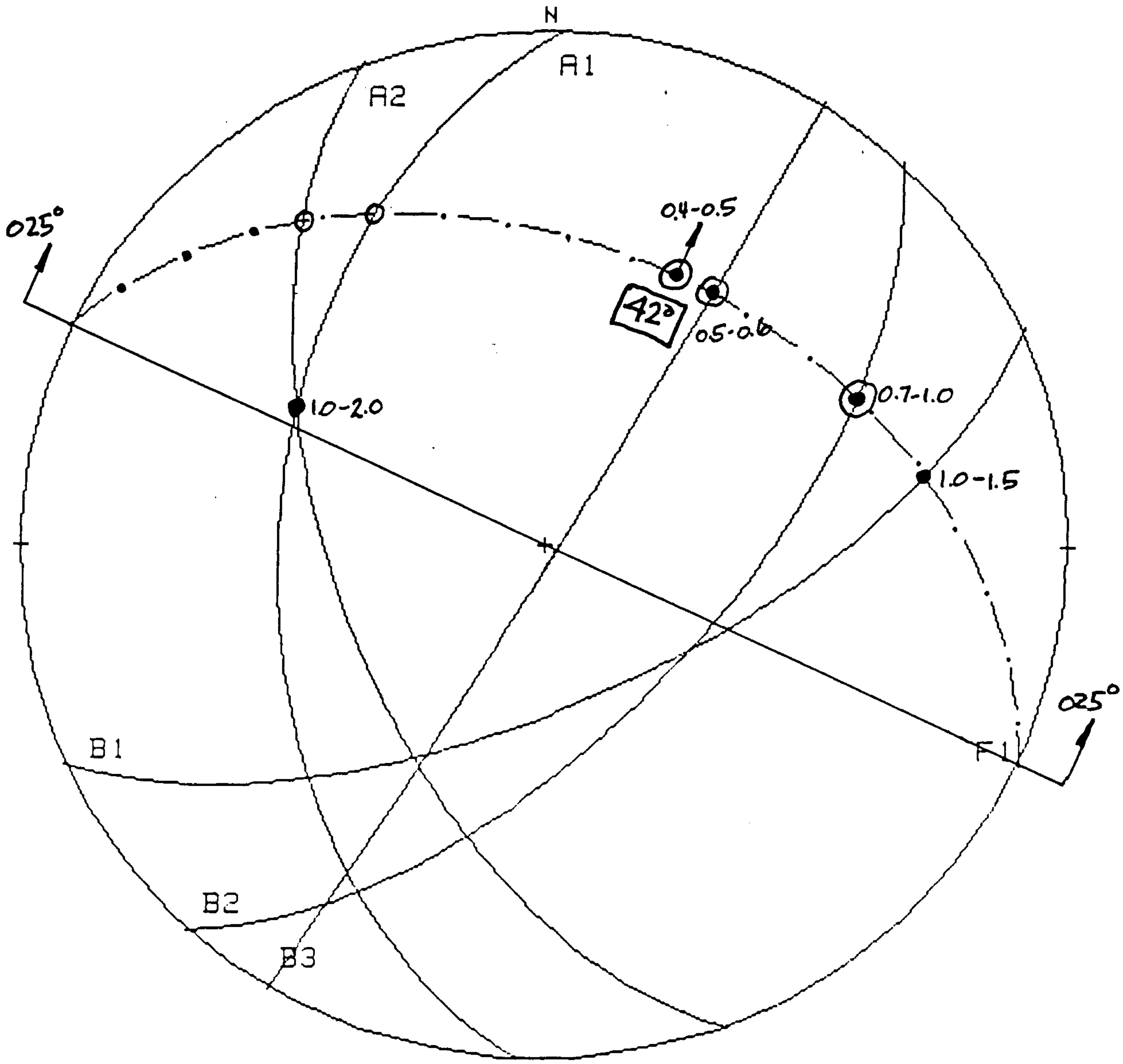
KEY TO KINEMATIC PROJECTIONS		BY: PMH	DATE: 02/88
		APPROVED:	DWG: 946-BO

GEOPLN - LOWER HEMISPHERE EQUAL AREA PROJECTION OF PLANES

PROJECT: MINNOVA - SAMATOSUM PROJECT
DATE: 16/2/88
STRUCTURAL DOMAIN: 1

JOB NUMBER



Very Strong
Kinematic Control

See Fig. B-0 for key

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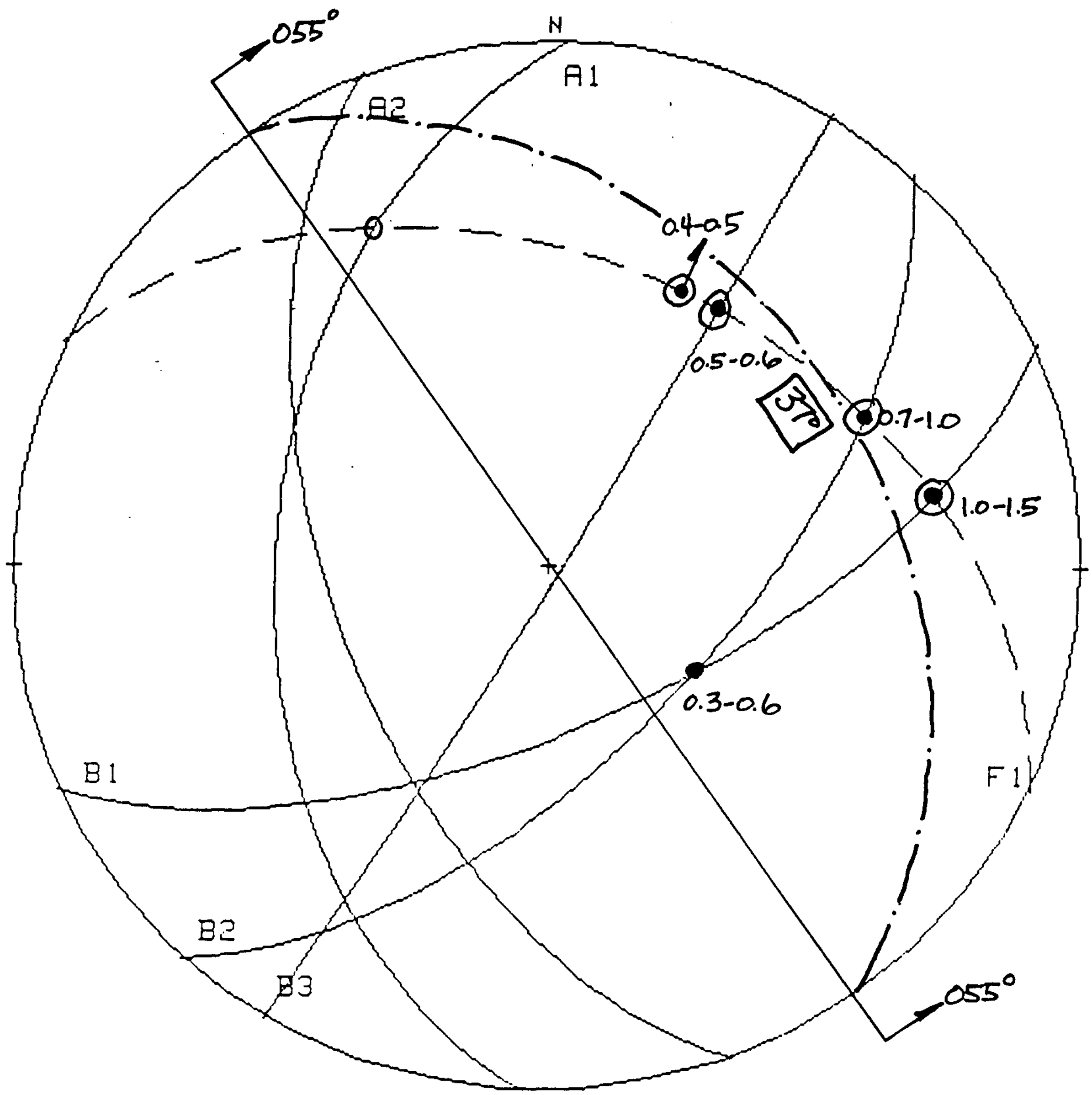
LOWER HEMISPHERE EQUAL AREA PROJECTION OF PLANES REPRESENTING PEAK ORIENTATIONS OF DISCONTINUITY SETS AND ILLUSTRATING KINEMATICALLY POSSIBLE FAILURE MODES IN DESIGN SECTOR I		BY: PMH APPROVED:	DATE: FEB 88 DWG: 946-B1

GEOPLN - LOWER HEMISPHERE EQUAL AREA PROJECTION OF PLANES

PROJECT: MINNOVA - SAMATOSUM PROJECT
DATE: 16/2/88
STRUCTURAL DOMAIN: 1



Strong
Kinematic Control

See Fig. B-0 for key

JOB NUMBER

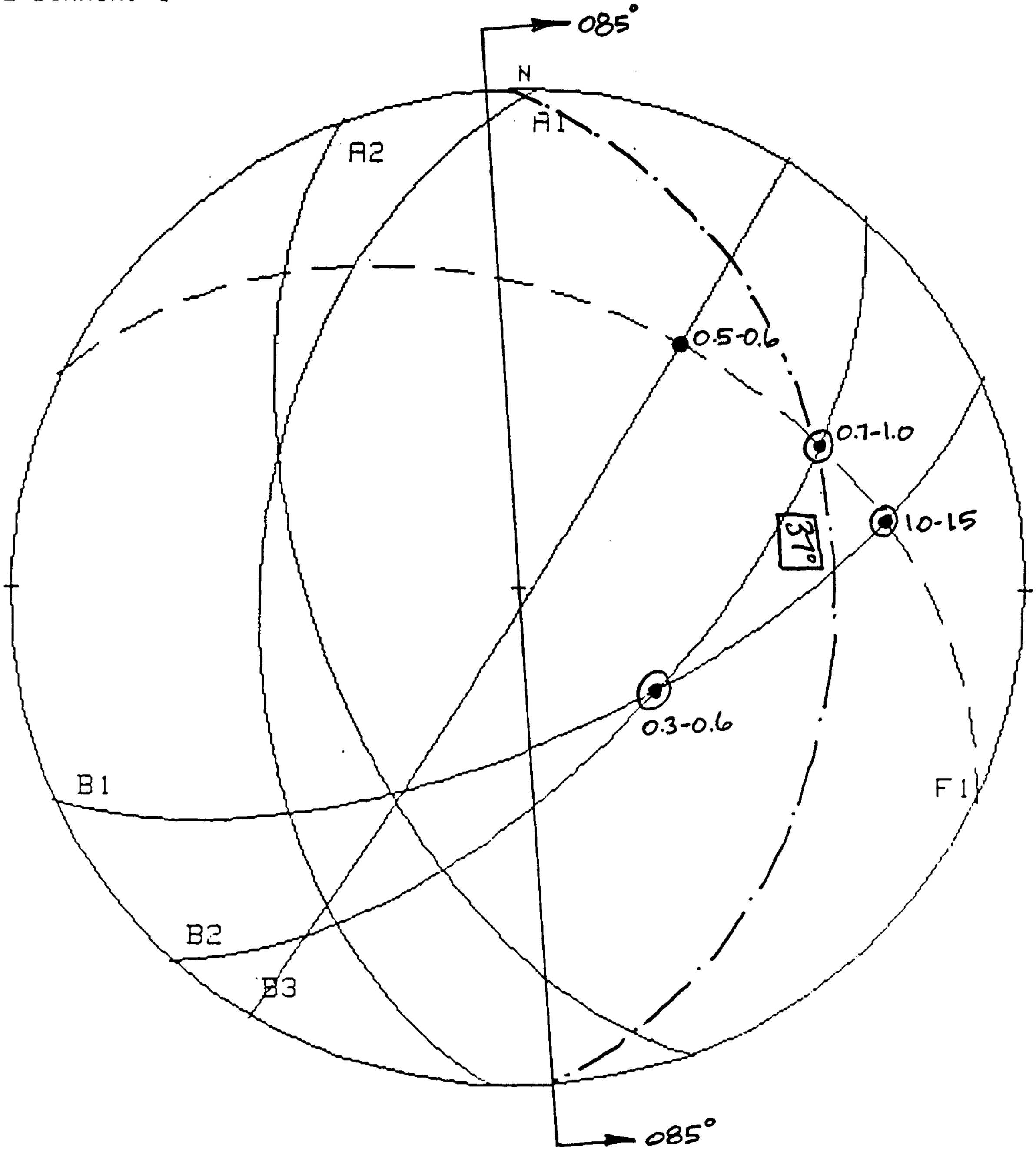
MINNOVA INC. SAMATOSUM PROJECT PRELIMINARY GEOTECHNICAL INVESTIGATION	PITEAU ASSOCIATES GEOTECHNICAL CONSULTANTS VANCOUVER CALGARY

LOWER HEMISPHERE EQUAL AREA PROJECTION OF PLANES REPRESENTING PEAK ORIENTATIONS OF DISCONTINUITY SETS AND ILLUSTRATING KINEMATICALLY POSSIBLE FAILURE MODES IN DESIGN SECTOR II	BY: PMH	DATE: FEB 88
	APPROVED:	DWG: 946-B2

JOB NUMBER

GEOPLN - LOWER HEMISPHERE EQUAL AREA PROJECTION OF PLANES

PROJECT: MINNOVA - SAMATOSUM PROJECT
DATE: 16/2/88
STRUCTURAL DOMAIN: 1



Moderate-Strong
Kinematic Control

See Fig. B-0 for key

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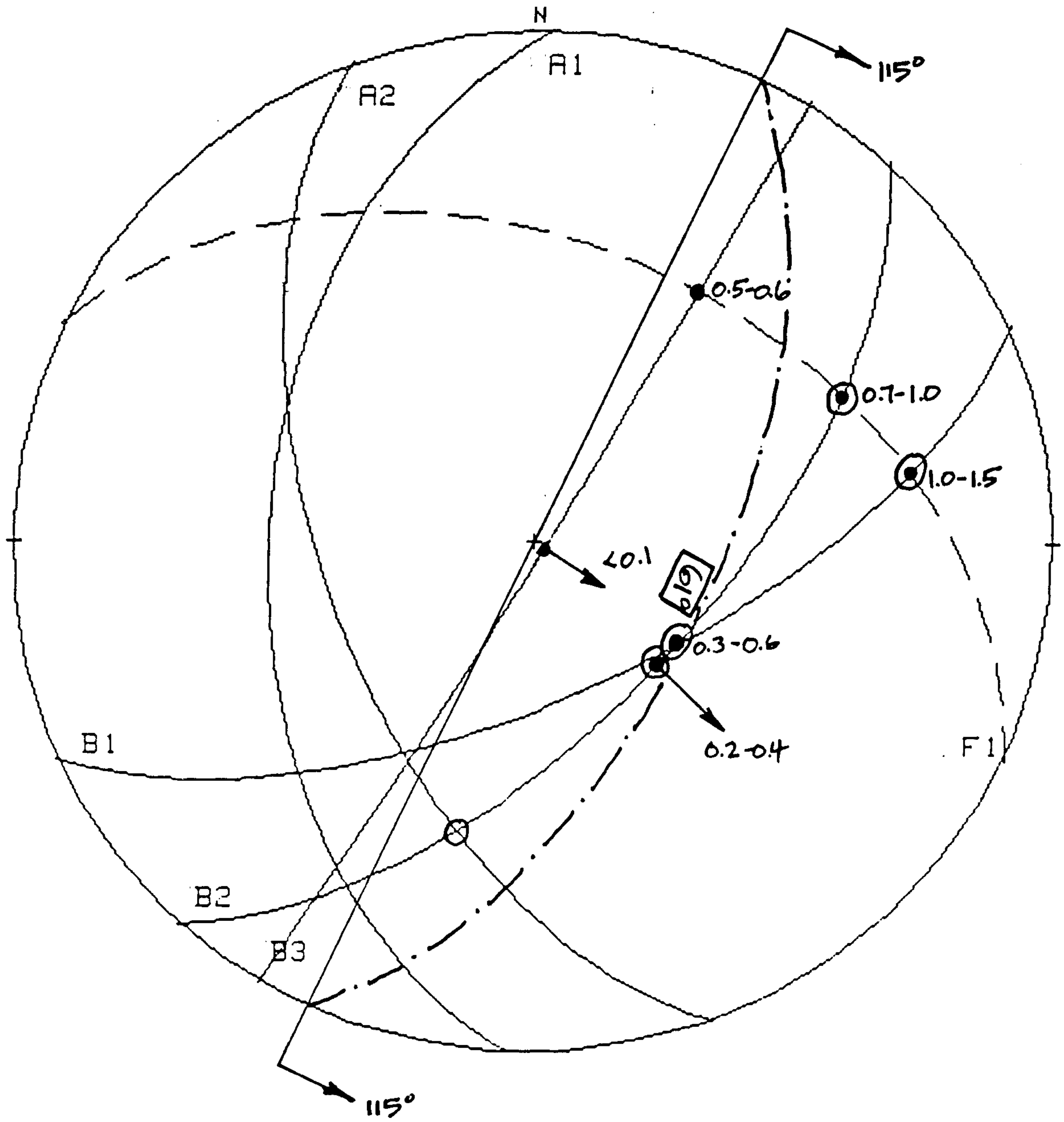
LOWER HEMISPHERE EQUAL AREA PROJECTION OF PLANES REPRESENTING PEAK ORIENTATIONS OF DISCONTINUITY SETS AND ILLUSTRATING KINEMATICALLY POSSIBLE FAILURE MODES IN DESIGN SECTOR III

BY	DATE
PMH	FEB 88
APPROVED	DWG: 946-B3

GEOPLN - LOWER HEMISPHERE EQUAL AREA PROJECTION OF PLANES

PROJECT: MINNOVA - SAMATOSUM PROJECT
DATE: 16/2/88
STRUCTURAL DOMAIN: 1



Strong
Kinematic Control

See Fig. B-0 for key

JOB NUMBER

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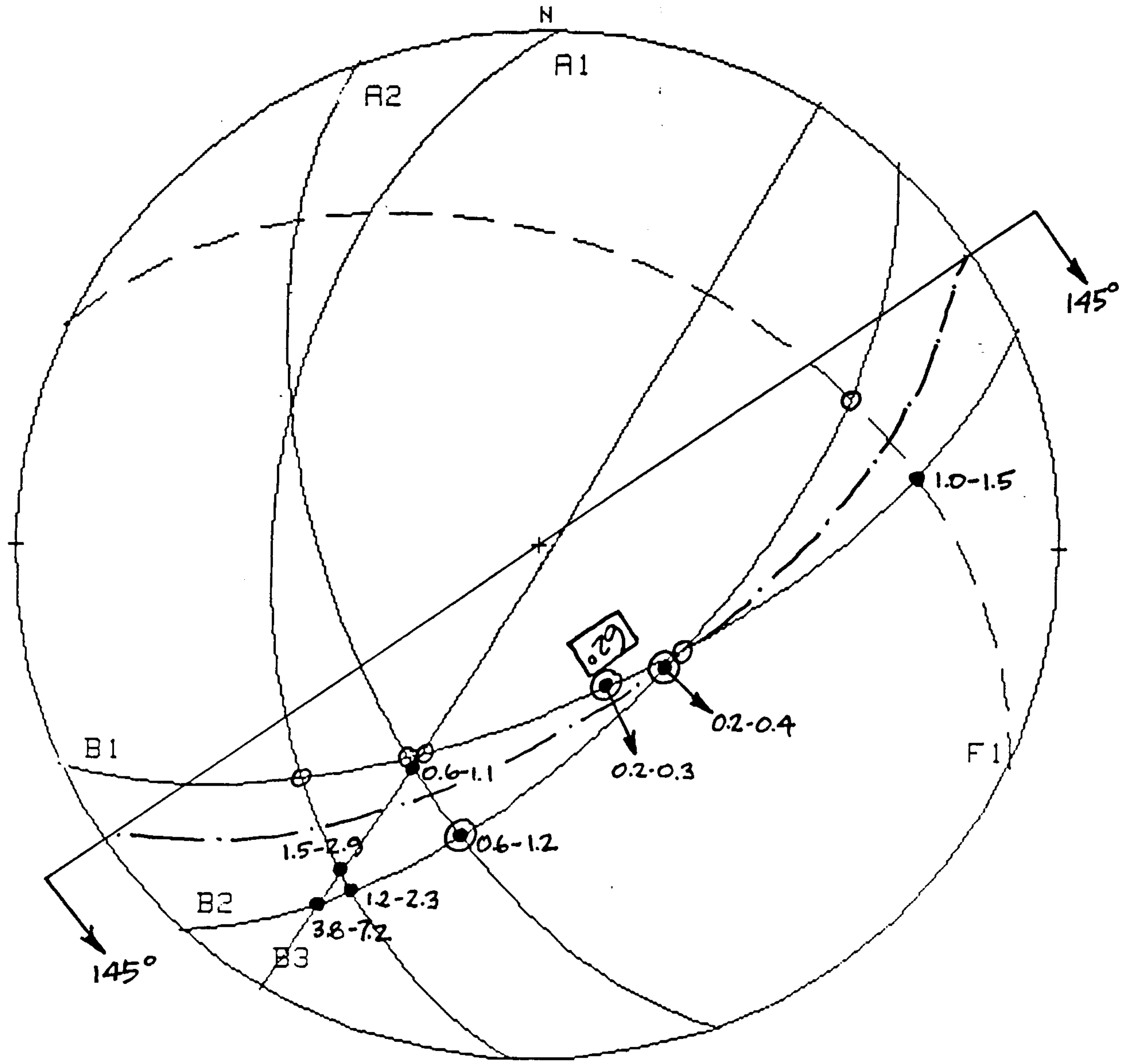
LOWER HEMISPHERE EQUAL AREA PROJECTION OF PLANES REPRESENTING PEAK ORIENTATIONS OF DISCONTINUITY SETS AND ILLUSTRATING KINEMATICALLY POSSIBLE FAILURE MODES IN DESIGN SECTOR IV	BY: PMH	DATE: FEB 88	
	APPROVED:	DWG: 946-84	

JOB NUMBER

GEOPLN - LOWER HEMISPHERE EQUAL AREA PROJECTION OF PLANES

PROJECT: MINNOVA - SAMATOSUM PROJECT
DATE: 16/2/88
STRUCTURAL DOMAIN: 1



Strong
Kinematic Control

See Fig. B-0 for key

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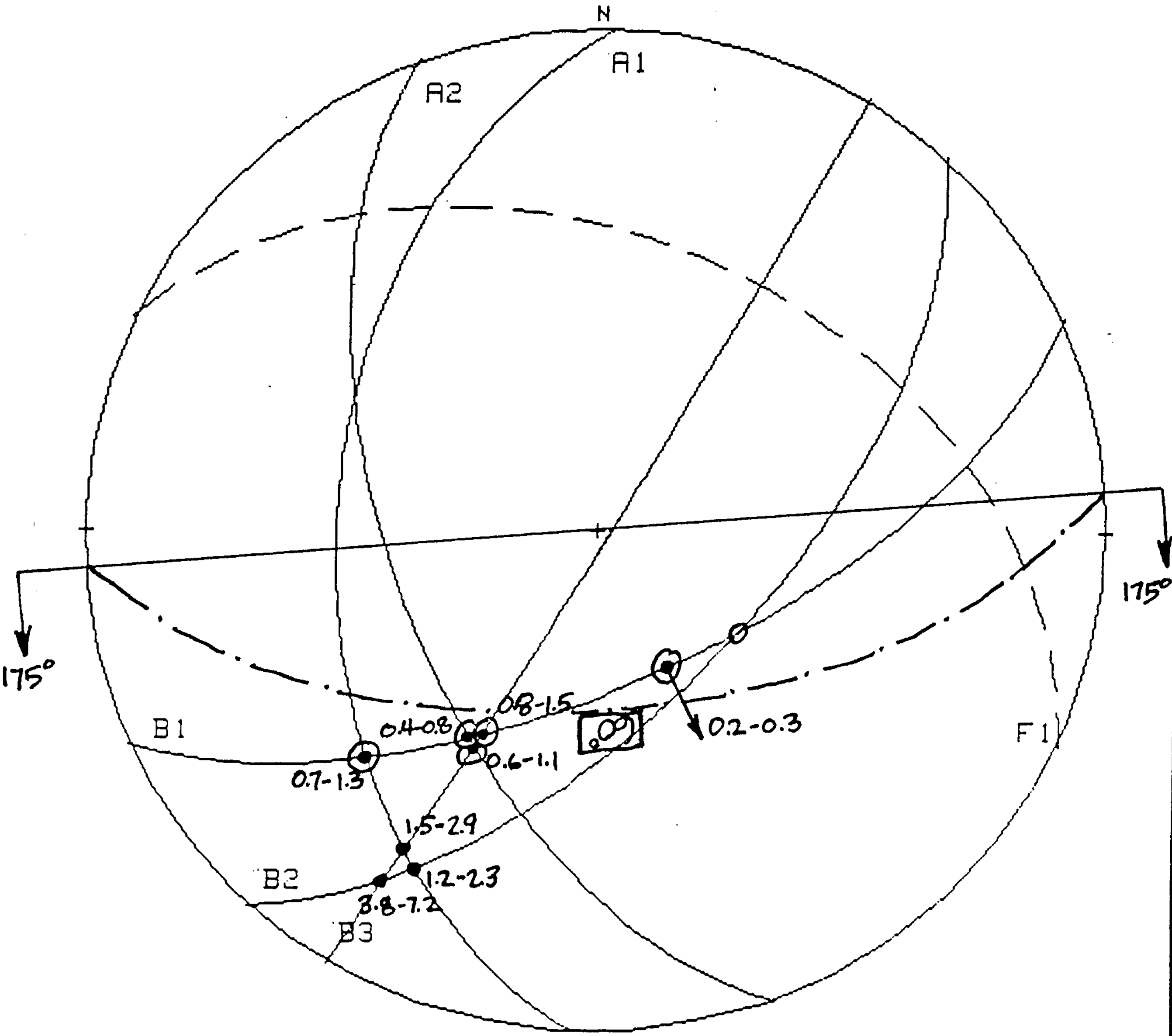
LOWER HEMISPHERE EQUAL AREA PROJECTION OF PLANES REPRESENTING PEAK ORIENTATIONS OF DISCONTINUITY SETS AND ILLUSTRATING KINEMATICALLY POSSIBLE FAILURE MODES IN DESIGN SECTOR V

BY	DATE
PMH	FEB 88
APPROVED	DWG:
	946-B5

GEOPLN - LOWER HEMISPHERE EQUAL AREA PROJECTION OF PLANES

PROJECT: MINNOVA - SAMATOSUM PROJECT
DATE: 16/2/88
STRUCTURAL DOMAIN: 1

JOB NUMBER



Moderate-Strong
Kinematic Control

See Fig. B-0 for key

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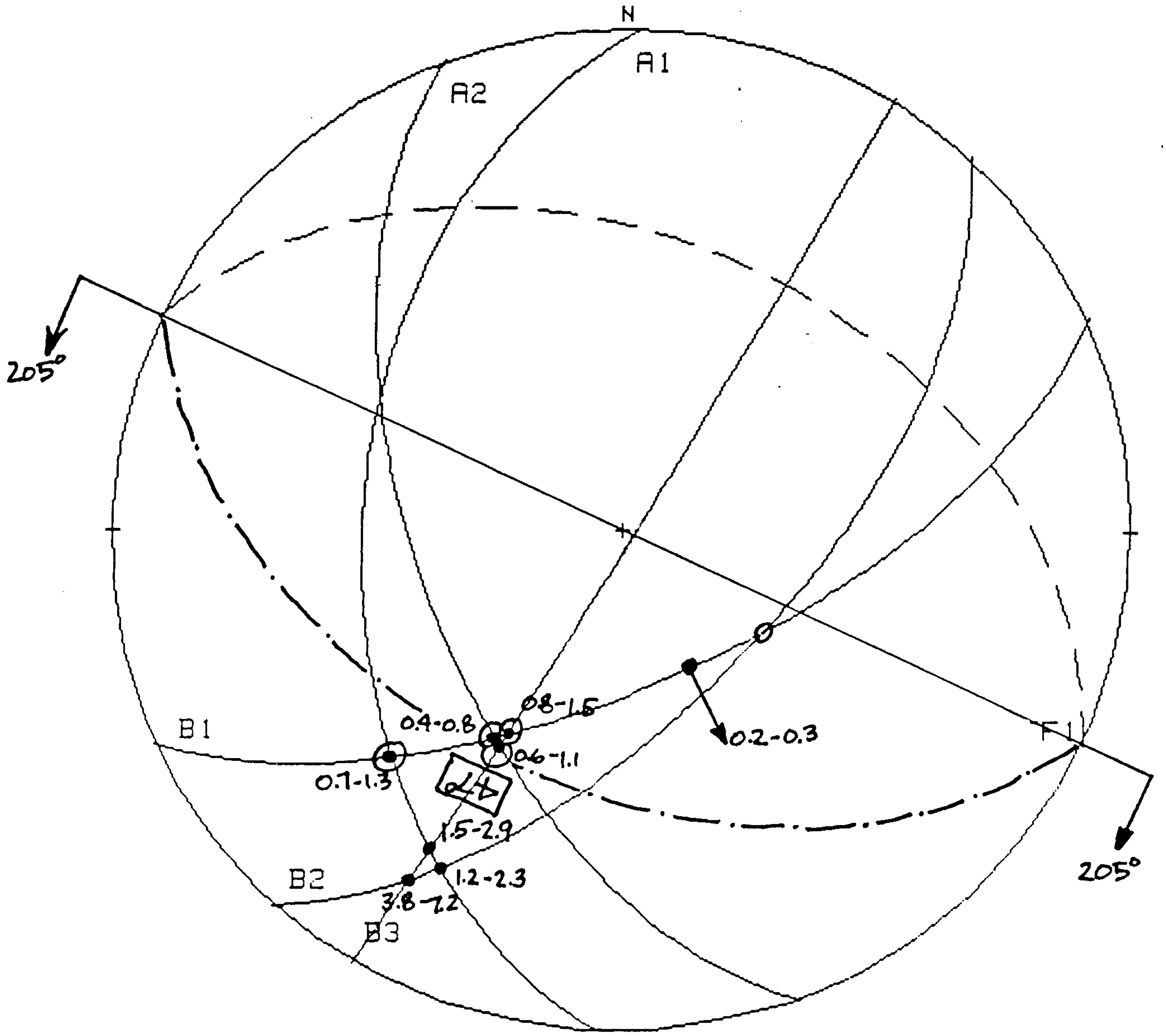
LOWER HEMISPHERE EQUAL AREA PROJECTION OF PLANES REPRESENTING PEAK ORIENTATIONS OF DISCONTINUITY SETS AND ILLUSTRATING KINEMATICALLY POSSIBLE FAILURE MODES IN DESIGN SECTOR VI	BY: PMH	DATE: FEB 88	
	APPROVED:	DWG: 946-B6	

JOB NUMBER

GEOPLN - LOWER HEMISPHERE EQUAL AREA PROJECTION OF PLANES

PROJECT: MINNOVA - SAMATOSUM PROJECT
DATE: 16/2/88
STRUCTURAL DOMAIN: 1



Weak-Moderate
Kinematic Control

See Fig. B-0 for key

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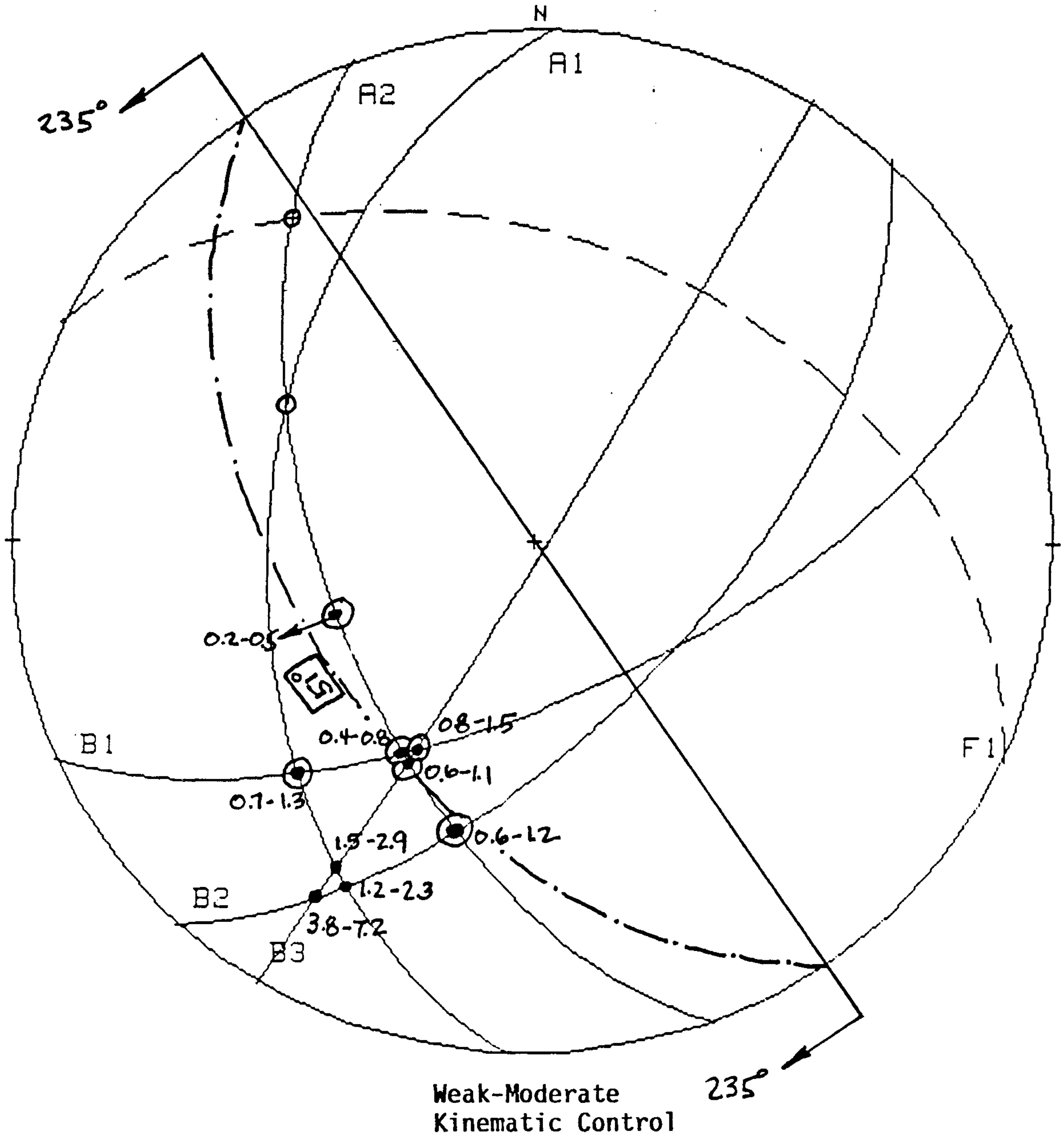
LOWER HEMISPHERE EQUAL AREA PROJECTION OF PLANES REPRESENTING PEAK ORIENTATIONS OF DISCONTINUITY SETS AND ILLUSTRATING KINEMATICALLY POSSIBLE FAILURE MODES IN DESIGN SECTOR VII	BY: PMH	DATE: FEB 88	
	APPROVED:	DWG: 946-B7	

GEOPLN - LOWER HEMISPHERE EQUAL AREA PROJECTION OF PLANES

PROJECT: MINNOVA - SAMATOSUM PROJECT
DATE: 16/2/88
STRUCTURAL DOMAIN: 1

JOB NUMBER



See Fig. B-0 for key

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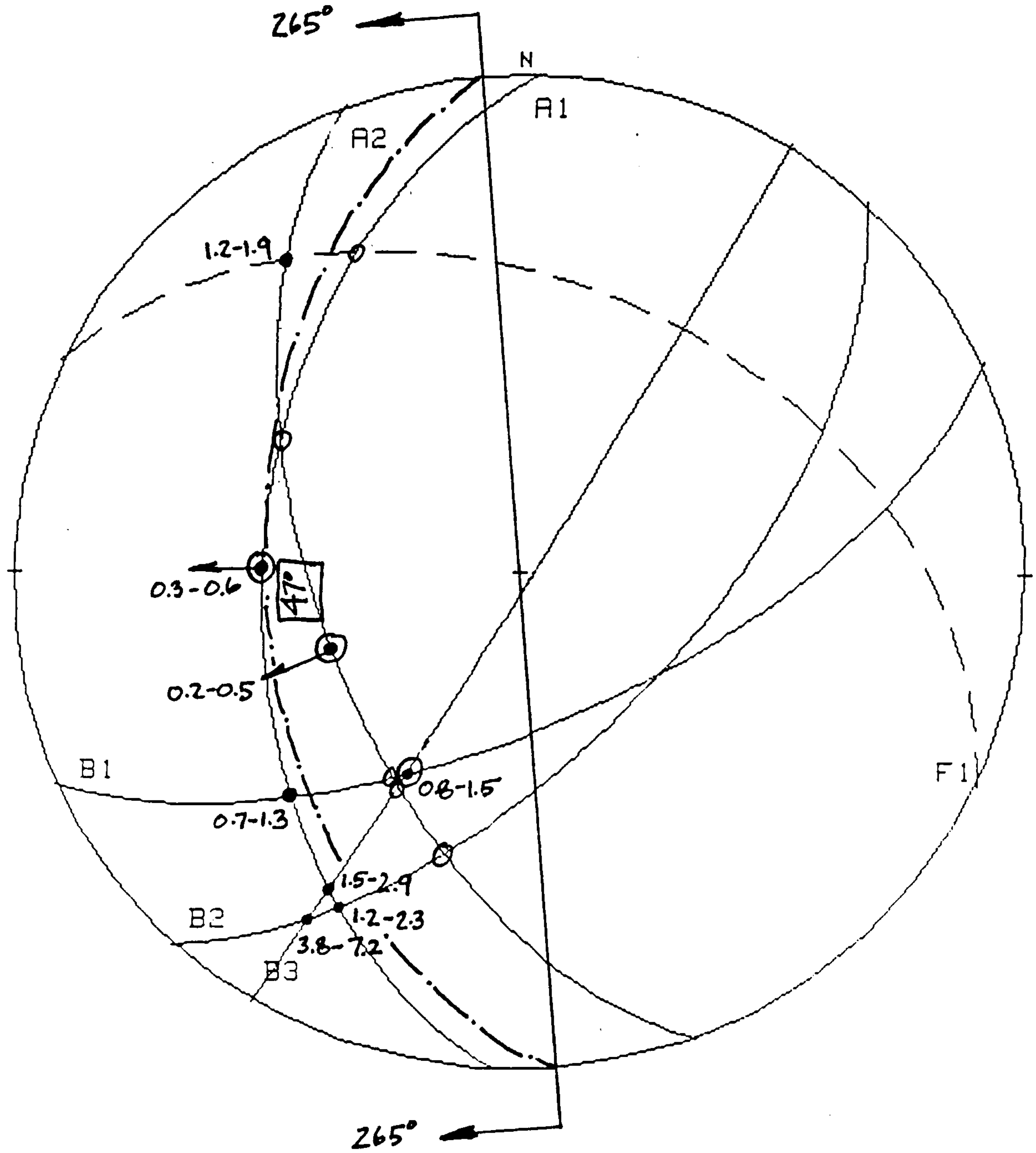
LOWER HEMISPHERE EQUAL AREA PROJECTION OF PLANES REPRESENTING PEAK ORIENTATIONS OF DISCONTINUITY SETS AND ILLUSTRATING KINEMATICALLY POSSIBLE FAILURE MODES IN DESIGN SECTOR VIII	BY: PMH	DATE: FEB 88
	APPROVED:	DWG: 946-B8

GEOPLN - LOWER HEMISPHERE EQUAL AREA PROJECTION OF PLANES

PROJECT: MINNOVA - SAMATOSUM PROJECT
DATE: 16/2/88
STRUCTURAL DOMAIN: 1

JOB NUMBER



See Fig. B-0 for key

Moderate-Strong
Kinematic Control

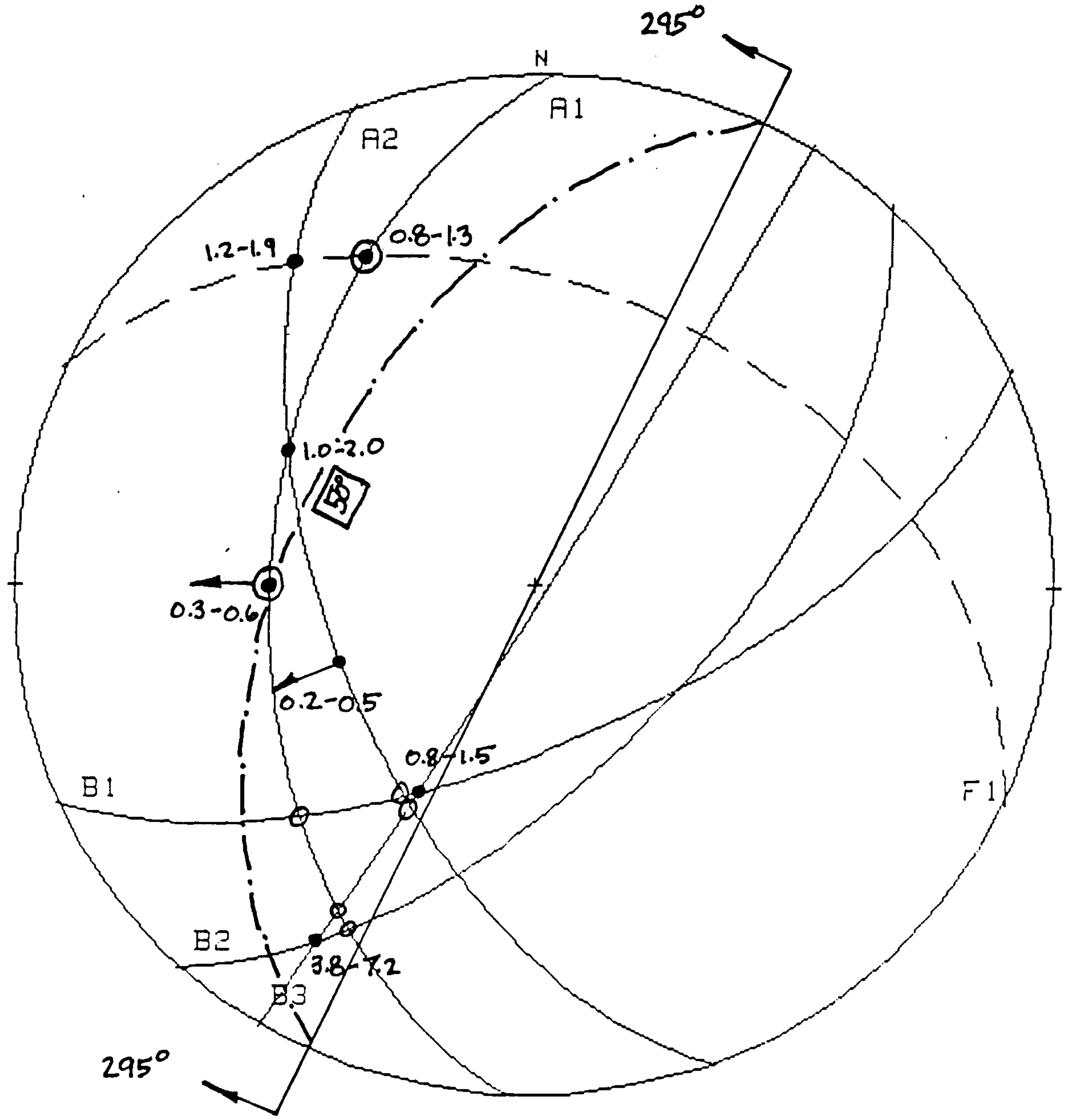
MINNOVA INC. SAMATOSUM PROJECT PRELIMINARY GEOTECHNICAL INVESTIGATION	PITEAU ASSOCIATES GEOTECHNICAL CONSULTANTS VANCOUVER CALGARY

LOWER HEMISPHERE EQUAL AREA PROJECTION OF PLANES REPRESENTING PEAK ORIENTATIONS OF DISCONTINUITY SETS AND ILLUSTRATING KINEMATICALLY POSSIBLE FAILURE MODES IN DESIGN SECTOR IX	BY: PMH	DATE: FEB 88
	APPROVED:	DWG: 946-89

JOB NUMBER

GEOPLN - LOWER HEMISPHERE EQUAL AREA PROJECTION OF PLANES

PROJECT: MINNOVA - SAMATOSUM PROJECT
DATE: 16/2/88
STRUCTURAL DOMAIN: 1



Weak
Kinematic Control

See Fig. B-0 for key

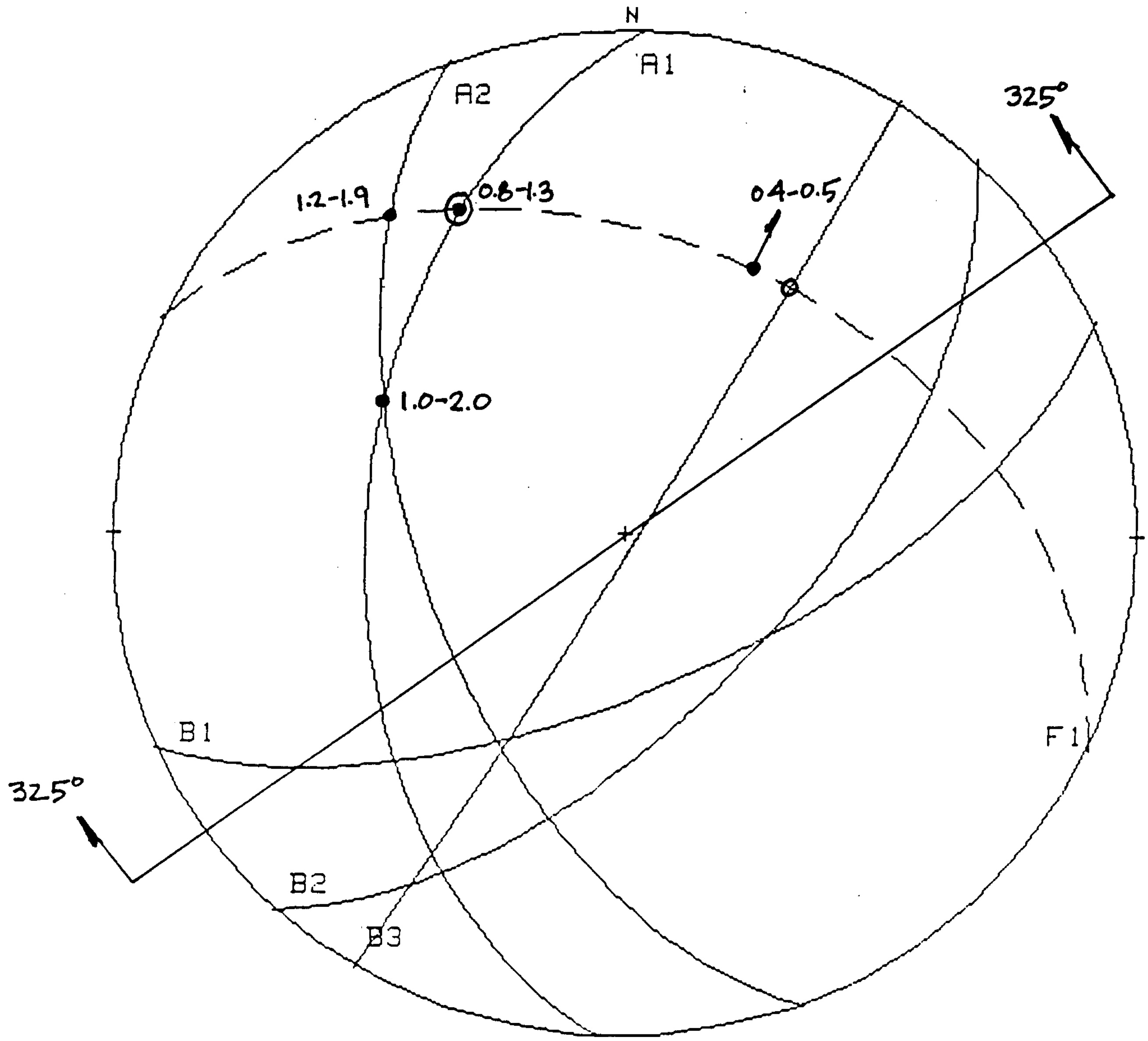
MINNOVA INC. SAMATOSUM PROJECT PRELIMINARY GEOTECHNICAL INVESTIGATION	PITEAU ASSOCIATES GEOTECHNICAL CONSULTANTS VANCOUVER CALGARY

LOWER HEMISPHERE EQUAL AREA PROJECTION OF PLANES REPRESENTING PEAK ORIENTATIONS OF DISCONTINUITY SETS AND ILLUSTRATING KINEMATICALLY POSSIBLE FAILURE MODES IN DESIGN SECTOR X	BY: PMH	DATE: FEB 88
	APPROVED:	DWG: 946-B10

JOB NUMBER

GEOPLN - LOWER HEMISPHERE EQUAL AREA PROJECTION OF PLANES

PROJECT: MINNOVA - SAMATOSUM PROJECT
DATE: 16/2/88
STRUCTURAL DOMAIN: 1



Very Weak
Kinematic Control

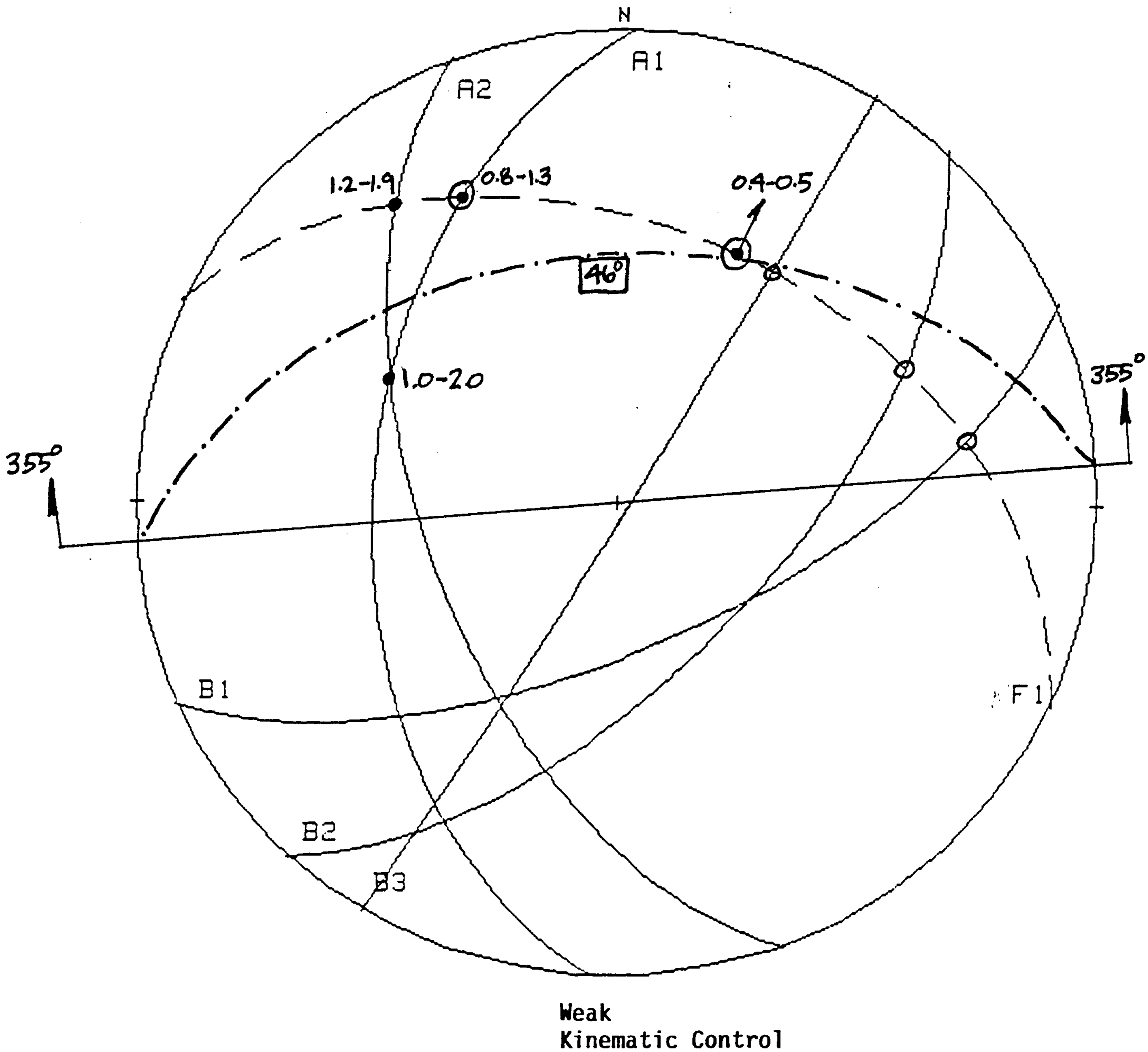
See Fig. B-0 for key

MINNOVA INC. SAMATOSUM PROJECT PRELIMINARY GEOTECHNICAL INVESTIGATION	PITEAU ASSOCIATES GEOTECHNICAL CONSULTANTS VANCOUVER CALGARY		
LOWER HEMISPHERE EQUAL AREA PROJECTION OF PLANES REPRESENTING PEAK ORIENTATIONS OF DISCONTINUITY SETS AND ILLUSTRATING KINEMATICALLY POSSIBLE FAILURE MODES IN DESIGN SECTOR XI	BY: PMH APPROVED:	DATE: FEB 88 DWG: 946-B11	

JOB NUMBER

GEOPLN - LOWER HEMISPHERE EQUAL AREA PROJECTION OF PLANES

PROJECT: MINNOVA - SAMATOSUM PROJECT
DATE: 16/2/88
STRUCTURAL DOMAIN: 1



See Fig. B-0 for key

MINNOVA INC.
SAMATOSUM PROJECT
PRELIMINARY GEOTECHNICAL INVESTIGATION

PITEAU ASSOCIATES
GEOTECHNICAL CONSULTANTS
VANCOUVER CALGARY

LOWER HEMISPHERE EQUAL AREA PROJECTION OF PLANES REPRESENTING PEAK ORIENTATIONS OF DISCONTINUITY SETS AND ILLUSTRATING KINEMATICALLY POSSIBLE FAILURE MODES IN DESIGN SECTOR XII	BY: PMH	DATE: FEB 88
	APPROVED:	DWG: 946-812