SLOPE STABILITY AND GROUNDWATER HYDROLOGY RESEARCH FOR PITWALL DESIGN AT EQUITY SILVER MINES LTD. HOUSTON, BRITISH COLUMBIA

By

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ABSTRACT

Structural geology, groundwater, shear strength and blasting control pitwall stability at Equity Silver Mines, Houston, British Columbia. A geotechnical investigation of these parameters was carried out in the Main Zone pit during the summer of 1984. The objective of the study was to develop a pitwall design based on geologic and groundwater conditions observed in each design sector. This thesis presents the results of the investigation; methods of improving stability by drainage and control blasting are also discussed.

Information on structural geology was obtained by line mapping of existing berms. The discodat package of computer programs was used to process the structural data and to identify trends in orientation of discontinuities. Based on this information, the Main Zone pit was divided into ten design sectors, each sector having a consistent pattern of discontinuity orientations, rock type, groundwater conditions and pit wall orientation.

Kinematically possible failure modes were identified in each design sector. Failure modes that were expected to present stability problems were analyzed to calculate factor of safety. Pit wall and berm face angles were then selected such that only a small number of potential failure modes will daylight.

The stability evaluation has shown that it should be possible to increase pitwall angles by 5° in the west half of the pit. However, the data base in this area of the pit is presently limited because only a small number of berms are exposed. Therefore, additional line mapping will be required before the west wall design can be finalized.

Groundwater will reduce pitwall stability, especially in the east half of the Main Zone pit. Multi-berm failures are very sensitive to groundwater conditions. A dewatering system should be installed in the Main Zone pit to minimize the possibility of such failures occuring.

Wet blastholes dictate that expensive water resistant slurry explosives be used in many areas of the Main Zone pit. The dewatering system should also draw down the water table so blastholes will become dryer and less expensive ANFO can be utilized.

The magnitude of shear strength on failure surfaces is required in order to evaluate stability of potential failures. Slip tests, point load tests and back analyses of existing failures were used to determine the shear strength parameters. Further studies should be carried out to better define the parameters at higher stress levels that will develop in a multi-berm failure.

Further potential for pit steepening exists if the berm face in the volcanics can be maintained at a slightly steeper angle, e.g. 70 instead of the present 66°. It may be possible to achieve this goal if trim blasting procedures are modified to reduce blast damage to the final wall.

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1.0 INTRODUCTION

This report presents the findings of a geotechnical study carried out in the summer of 1984 in the Main Zone pit at Equity Silver Mines. The investigation had five goals:

1. To design overall slope angles in the Main Zone pit.
2. To evaluate the influence of structural geology, groundwater, shear strength, and monitoring on stability.
3. To study the existing groundwater conditions in the Main Zone pit in order to determine whether pit dewatering will be possible.
4. To introduce a multi-stage monitoring program for the Main Zone pit that will ensure early detection of any instabilities and adequate monitoring of the rates of movement once the slides are identified.
5. To recommend operating procedures in the areas of blasting, dewatering, and monitoring that will improve stability in the Main Zone pit, make it safer, and possibly allow for further increase in pit wall angle.

1.1 TERMS OF REFERENCE

The geotechnical study summarized in this report was a joint effort carried out by the Mine Engineering Department at Equity Silver Mines and The Department of Mining and Mineral Process Engineering at The University of British Columbia. A preliminary structural stability analysis was carried out by Equity engineers in 1983. The study concluded that there was potential for steepening sections of the pit, but further investigation was required to confirm the observed trends.

Prof. C.O. Brawner of the Department of Mining and Mineral Process Engineering submitted a proposal to have a graduate student assist the Mine Engineering Department in the advanced stage of the geotechnical investigation. This proposal was accepted.

Field work in all areas of the geotechnical investigation was carried out during the summer of 1984, early May to mid-September, by the author and Equity Mine Engineering personnel. Progress was reviewed on a periodic basis by Professor Brawner.

Analysis of data was carried out in part during the summer at Equity; most of the design work was completed at The University of B.C. This thesis summarizes the findings of the geotechnical studies.

1.2 PURPOSE OF REPORT

The purpose of this thesis is to present the findings of the geotechnical investigations carried out in the Main Zone pit during the summer of 1984. The most important goal of the program was to design ultimate pit wall angles according to the geologic structure observed in each design sector in the Main Zone. Procedures used to develop the pitwall design are also presented.

The second objective of the report is to present the results of the hydrologic investigation. The purpose of the hydrology study was to determine the existing groundwater conditions in the Main Zone and to find out whether the Main Zone pit could be successfully dewatered. The hydrologic section of this thesis also reviews existing dewatering technology and makes recommendations as to which systems could be the most effective in dewatering the Main Zone pit.

The third objective of the thesis is to summarize the results of the shear strength testing program. Representative magnitudes of shear strength parameters are required in the stability evaluation of any kinematically possible wedges and in the design of support systems.

Many methods of improving slope stability have been developed in recent years, especially in the area of control blasting. Several control blasting procedures that have potential for improving stability in the Main Zone pit, that appear practical, and that should be of economic benefit to the operation are introduced.

Monitoring to detect pit wall movement is an important part of an overall open pit stability program. This report reviews existing technology and outlines monitoring procedures that should be implemented in the Main Zone pit to ensure rapid detection of any instabilities.

1.3 SCOPE OF WORK

The geotechnical investigation that was carried out to develop the pit wall design and to evaluate the potential for Main Zone pit dewatering consisted of:

1 - STRUCTURAL GEOLOGY
- line mapping of exposed interior and ultimate pit walls
- structural drill hole logging and core orientation
- analysis of structural data using Discodat System
- designation of structural domains

2 - SHEAR STRENGTH
- point load testing of drill core
- slip tests for ϕ
- back analysis of small wedge failures
- assessment of shear strength parameters

3 - GROUNDWATER HYDROLOGY
- field reconnaissance
- weekly piezometer monitoring
- completion of 11 piezometers
- falling head permeability tests
- assessment of groundwater hydrology in the Main Zone
- computer modelling of dewatering systems
- evaluation of dewatering systems

4 - BLASTING
- evaluation of Equity's trim blasting program

5 - PIT WALL DESIGN
- selection of design sectors
- assessment of stability in each sector
- design of slope angle in each sector

6 - MONITORING
- development of guidelines for a slope stability monitoring program

With the exception of the diamond drilling phase of the program that was completed by J.T. Thomas Ltd; all drilling, instrumentation installation, and testing was carried out by Equity Mine Department personnel.

2.0 SITE CONDITIONS

2.1 LOCATION

Equity Silver Mine is located in central British Columbia, 54° 12′ N latitude and 126° 16′ W longitude. It is situated in the uplands of the Nechako Plateau, 35 km southeast of Houston, the nearest town. Access to the mine site is by a 37 km all weather, gravel surface road from Houston that follows the Dungate Creek drainage. Figure 2.1 shows the location of the mine site on provincial and regional maps.

2.2 PHYSIOGRAPHY

Topography at Equity consists of rolling hills and broad valleys. Elevation changes are for the most part gradual as topography has been subdued and rounded by tertiary lava flows that flowed in near horizontal sheets, filling existing topographic lows, and pleistocene glaciation that has rounded the hill tops and deposited a thick blanket of glacial till and glaciofluvial deposits in much of the lowland.

Relief in the immediate area is approximately 725 m, from a low of 900 m at Goosly Lake (5 km southwest of the mine) to a high of 1625 m at a prominent ridge top 3 km east of the Main Zone pit. This topographic high is formed by rocks of the gabbro monzonite intrusive complex that are relatively resistant to erosion. The gabbro intruded into the overlying volcanics some 48 million years ago.

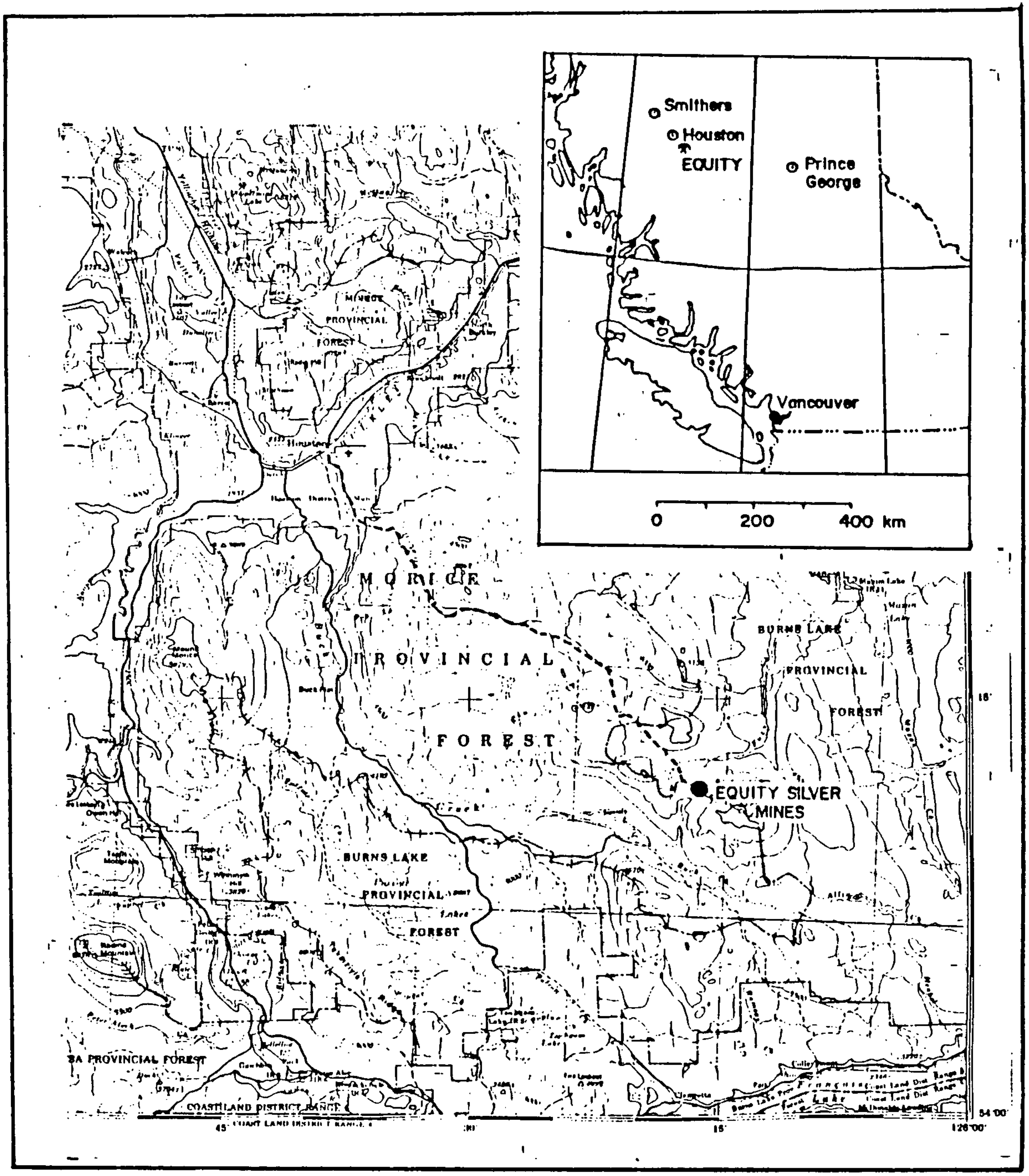

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Figure 2.1 LOCATION OF EQUITY SILVER MINES

2.3 SURFACE HYDROLOGY

The major drainage systems that drain the Nechako Plateau in the area of Equity exhibit a northwest - southeast lineation (e.g. Buck, Parrot, Maxan, and Owen Creeks). The tributary drainage pattern between these creeks is dendritic.

Several surface water catchments drain the mine property: 1) Lu Creek drains the flats west of the mine facilities, 2) Foxy Creek collects runoff from a low relief basin north of the mine, 3) Berzelius Creek flows from the highlands northeast of the pit, and 4) Bessemer Creek drainage covers most of the hillside east of the Main Zone pit. Location of the above creeks and diversion channels is illustrated in relation to the mine facilities in Figure 2.2.

Water is an important factor in the stability of the Main Zone pit; therefore, it is important to have a good understanding of the location of catchment basins and groundwater recharge areas in the immediate vicinity.

The largest source area for groundwater seepage and surface runoff into the pit is the Bessemer Creek drainage. The original stream bed followed a westerly course through the center of the Main Zone ore body and then turned sharply south, eventually emptying into Buck Creek above Goosly Lake. The creek has since been diverted northward by a diversion ditch that also collects surface water from Berzelius Creek. The upper Bessemer catchment basin covers an area of 3 km. Most water that infiltrates into the groundwater system in the basin will eventually discharge into the pit. Some groundwater seepage will also originate in the Berzelius Creek and Lu Creek catchments. However, inflows are expected to be small because the recharge areas are much

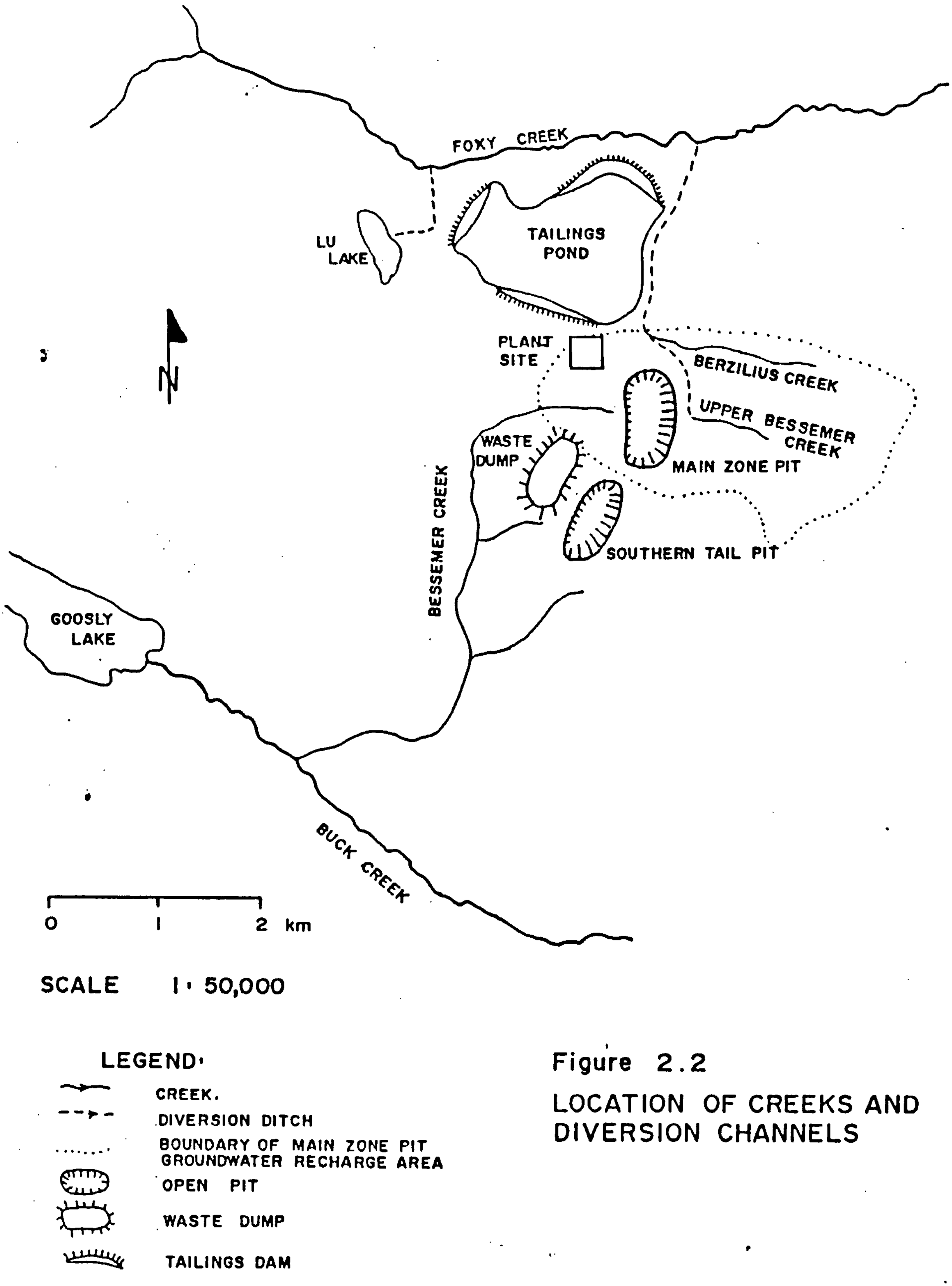


Figure 2.2

LOCATION OF CREEKS AND DIVERSION CHANNELS smaller, and for the most part, covered with a blanket of low permeability glacial till. The till promotes surface runoff.

2.4 CLIMATE

Climate at the mine site is influenced by the high elevation and proximity to the Pacific west coast. Temperatures average about 13 C during the summer months and about -12 C during the winter. The property receives an average of 51 cm of precipitation annually. Most precipitation statistically falls during the winter months, but the past several summers have been abnormally cool and wet. Annual snowfall exceeds 2 m; with much of the property remaining snow covered until mid-June.

GEOLOGY

The Equity Ag-Cu deposit is situated in an inlier of Cretaceous (65-71 m.y.) volcanic and sedimentary rocks called the Goosly Sequence. Rocks of this sequence are exposed at surface only in the area around the Equity Property. Outside the inlier, they are covered by Tertiary volcanic flows.

The contact between the Goosly Sequence and the Tertiary flows is unconformable; the Goosly Sequence was tilted before the near horizontal lava flows covered the landscape. Two intrusions are also present within the inlier: 1) A quartz-monzonite stock is situated 1 km west of the Main Zone ore body, and 2) a gabbro-monzonite complex has intruded just east of the ore zone, the contact forms the the footwall of the of the ore body.

3.1 GEOLOCIC HISTORY

The geology at the Equity Property has been divided into 7 units by site geologists (Pease et al. 1983). These units are summarized below. Table 3.1.

Relative Age	Period	Unit	Name
youngest	Tertiary	7	Andesitic Flows & Flow Breccias
	"	6	Gabbro Monzonite Intrusive
	"	5	Quartz Monzonite Intrusive
	Cretaceous	4	Volcanic Flow Division
	"	3	Sedimentary Volcanic Division
	"	2	Pyroclastic Division
oldest	"	1	Clastic Division

The oldest rocks on the property are the Clastic Division, a transgressive series of conglomerate, sandstone, and argillite that were deposited in a subaqeous environment.

Violent volcanic activity north of the mine then generated vast quantities of pyroclastic material that accumulated over time to cover the area with as much as 975 m of ash and coarser ejecta. Material of Unit 2 is coarser in the north portion of the property (i.e. the Main Zone). There, the dominant lithology is lapilli tuff (tuff that has fragments 4-32 mm in diameter) with minor zones of volcanic breccia and dust tuff. To the south, Unit 2 becomes much finer grained; dust tuff is the dominant rock type in the Southern Tail pit.

After the major eruptions additional ash and coarser volcanic rock were transported into the area by fluvial and mass wasting processes to deposit Unit 3, the Sedimentary-Volcanic Division.

Another episode of volcanic activity covered the area with lava flows of Andesitic to Dacitic composition. This was the last event in the formation of the Goosly Sequence.

Tectonic activity then continued with the intrusion of the Quartz-Monzonite Stock approximately 60 million years ago and the emplacement of the Gabbro-Monzonite Complex 48-49 million years ago (ages based on data of several workers summarized by Pease et al. 1983). The Goosly Sequence was tilted during this period to its present orientation of: strike 015°, dip 45° - 80° W.

Andesitic lava flows covered much of the low lying areas in the final depositional event shortly after intrusion of the Gabbro Complex. Church (1970), suggests that this intrusive was the feeder for the flows, citing mineralogical and chemical similarities between units as evidence.

A geologic map of the Equity Property (Figure 3.1) shows the location of the 7 lithologic units.

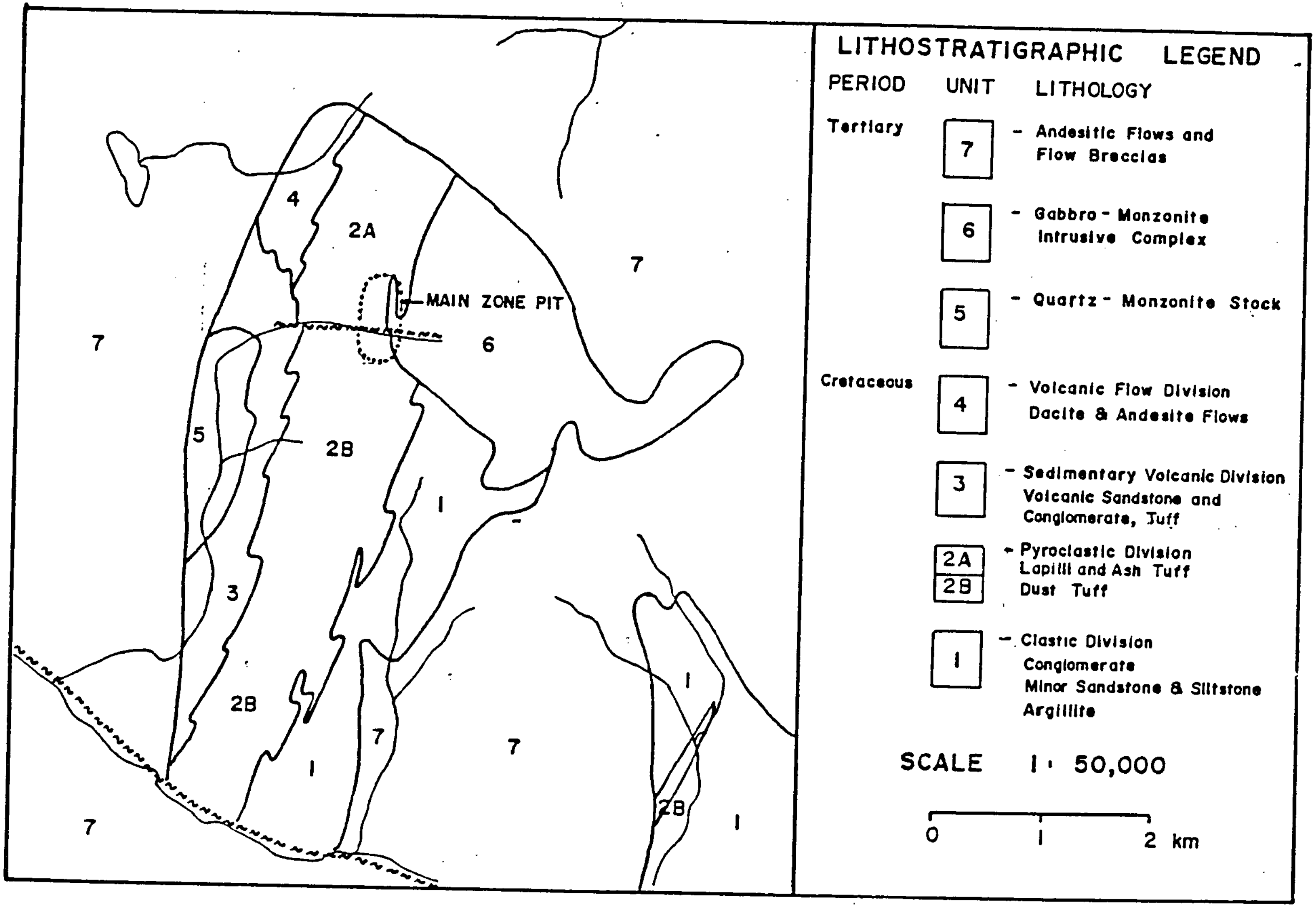


Figure 3.1 PROPERTY GEOLOGY - EQUITY SILVER MINE
(Modified after Pease, 1983)

3.2 PLEISTOCENE GEOLOGY

Much of the Equity property is covered with glacial till that was deposited during the last major glacial advance of the late pleistocene, approximately 10,000 years ago. During the earliest period of the Fraser glaciation alpine glaciers grew, eventually coalescing to form a continental ice sheet. During the climax of the glaciation regional ice movement was to the northeast. The flowing ice scoured the bedrock, plucking up any loose fragments. Evidence of the northeasterly flow can be observed on striated surfaces of several outcrops in the vicinity of the mine.

Topographic highs were rounded by the erosive forces of the flowing ice of several glaciations. Rock that was plucked up was then carried along with the ice, much of it broken down to glacial flour. The reworked material was deposited as ground moraine at the bottom of the glacier and highly compacted by the weight of the overlying ice.

The thickest deposits of glacial till occur in topographic lows. In one drill hole on the property 45 m of till were triconed before bedrock was encountered (Wetherell, 1979); however, the normal thickness of till ranges from 10 to 20 m in the valleys and 0 to 5 m at higher elevations.

Composition of the till is usually silty clay with trace to some gravel, but can vary from location to location. Because the till has a very high clay content and is well compacted it is very impermeable. As a result, in till covered areas most precipitation drains as surface runoff before it can seep into the groundwater system. The till blanket also confines several underlying aquifers of fluvial sands and fractured bedrock. Artesian conditions have been observed in several boreholes on the property where the confined aquifers are located in steep terrain.

Hydrologic investigations by Golder Associates (1983) have confirmed the presence of fluvially deposited sands and gravels below the till in the lower reaches of Lu and Bessemer Creeks. The sands and gravels are the bed load of pre-glacial streams that were burried by the advancing glaciers.

In the final stages of the Fraser glaciation the direction of ice movement reversed because local topography again began to influence the flow direction of the much shrunken ice sheet. Evidence indicating this final episode of southwest movement includes: 1) southwesterly offsets in the Ag geochemical anomaly over the ore zones, 2) southwest transport of a granitic boulder train from a well defined source area, and 3) roches moutonnées structures with glacial striae that clearly indicate a flow direction of 240° (Wojdak, 1974 & Wetherell 1979).

3.3 GEOLOGIC UNITS IN THE MAIN ZONE

Only two of the seven geologic units occur in the Main Zone. Pyroclastic Division 2A is the host rock for the economic mineralization, and the gabbro-monzonite intrusive complex (Division 6) will form much of the east ultimate pit wall. The gabbro is also being actively mined for non-acid generating rockfill that is used in the construction of the tailings dams. Rocktype influences virtually all aspects of pit design and mining operations. Therefore, it is important to have a good understanding of the physical properties and characteristics of the rockmass.

3.3.1 Pyroclastic Division 2A

The dominant rock type in the pyroclastic unit is lapilli tuff. The lapilli fragments are usually subangular to subrounded and composed of aphanitic groundmass. The matrix is finer grained ash. Colour of the lapilli tuff is dark grey but can deviate to a dark olive green if chlorite alteration is present. The hardness classification ranges between R3 to R4, depending on degree of alteration. Point load tests were also carried out on the drill core to determine the uniaxial compressive strength. Results of these tests are discussed in Section 7.1. The lapilli tuff, with an average uniaxial compressive strength of 112 MPa is classified as "strong rock".[1] Joint set spacings of 0.2 to 0.6 m (DISCODAT classification i) were observed most often in line mapping of pit walls. Most joints were not continuous (i.e. less than 5 m in length).

Other rock types that are present in Unit 2A include flow breccia, ash tuff, dust tuff, and minor volcanic conglomerate.

The breccia has rockmass characteristics that are very similar to those of the lapilli tuff except that the clasts are angular and often larger in size.

The ash tuff occurs in irregular zones within the lapilli, the contacts are generally gradational. Ash tuff is defined as a pyroclastic rock with grains smaller than 4 mm in diameter, but sufficiently large to be visible to the naked eye. Colour of the ash tuff is also dark grey to olive green. Hardness classification is

1. Based on classification system proposed by Hoek, 1981 that is listed in Appendix D.1.

usually R3. Uniaxial strengths obtained from the point load tests were approximately 30% lower than for the lapilli, averaging 88 Mpa, or "moderately strong rock". Jointing is also more common in the ash tuff than in the lapilli.

Dust tuff is rare in the Main Zone pit. When it does occur it seems to be in localized lenses that span less than 100 m in the longest dimension. The rocktype can be identified easily in hand specimens because it is aphanitic (individual grains too fine to be distinguished by the naked eye). Colour is usually a lighter shade of grey. In the pit wall, zones of dust tuff can be recognized by the blocky, crumbling nature of the pit wall. This characteristic is caused by very closely spaced, intersecting sets of joints. The joints are once again discontinuous. Some of the joints observed at surface may actually be fractures opened up by blasting because the rock is moderately weak (R3) and brittle. It is therefore susceptible to blast damage. However, discontinuities were also much more prevalent in dust tuff drill core from structural drill hole DDH 84-167 that was not damaged by blasting than core of the other rock types. Uniaxial compressive strength was about 44 mPa and rock quality designation indices (RQD) often dropped below 50 percent in the dust tuff.

In summary, engineering properties of the intact pyroclastic rocks are related to grainsize. Strength decreases and degree of jointing increases with decreasing grainsize. Overall, pyroclastic unit 2A is sufficiently competent and intact that failures will be controlled by throughgoing discontinuities, not by exceeding the shear strength of the intact rock.

3.3.2 Gabbro Monzonite Intrusive - Division 6

The gabbro-monzonite intrusive complex is situated just east of the Main Zone ore body. The intrusive-volcanic contact dips westward, into the pit, at 40° to 45°. The intrusive complex covers an extensive area of uplands. Detailed petrographic work by Ney et al. (1972) has identified six separate intrusive phases. All of the intrusive rocks in the eastern section of the Main Zone pit consist of phase 6C, monzonite.

Monzonite is an intrusive rock composed primarly of plagioclase and potassium feldspar, with minor amounts of quartz, biotite, and other common accessory minerals. In the Main Zone the monzonite is coarse grained, some feldspar phenocrysts exceed 1 cm in size. Colour is medium speckled grey.

Intrusive rocks are generally very strong because their grains are all interlocking and no planes of weakness (i.e. sedimentary bedding or metamorphic foliation) are present when the rock forms. The average uniaxial compressive strength of the gabbro-monzonite was 455 MPa, a "very strong rock". In field mapping the rock type was assigned an R5 rating, as numerous blows with a rock hammer were required to break a sample.

Discontinuities were widely spaced in the gabbro , spacings usually ranged from 0.6 to 6.0 m (DISCODAT codes j and k). Many of the joints were very continuous in the gabbro[2], exceeding the length of double benches (i.e. longer than 20 m).

2. The group of intrusive rocks of Division 6 are generally referred to as "Gabbro" at the mine site; therefore, this name will be used in the remainder of the report.

When mined along the east wall of the Main Zone pit, the gabbro has a tendency to break along a set of continuous joints that dip 50° to 55° into the pit. This trend is clearly seen in Figure 3.2.

Figure 3.2 PHOTOGRAPH SHOWING TENDENCY OF GABBRO TO BREAK ALONG CONTINUOUS WEST DIPPING JOINTS.

3.3.3 DYKES

Dykes are common in the Main Zone pit. Three principal types have been recognized: 1) andesite, 2) quartz latite, and 3) trachytic andesite.

1. Andesite dykes are the most common variety in the Main Zone. They are dark green to black in colour, aphanitic, and occasionally vesicular. Intact andesite dyke is rated "strong" (R4, uniaxial compressive strength of 176 MPa), but considerably weaker specimens of altered dyke material have been tested. Orientations vary, but two trends have been recognized on the geologic plan of the Main Zone pit (Plan 1, located envelope): 1) a southeast strike dipping 50°-60° to the southwest is prevalent in the central portion of the east wall, and 2) an easterly strike dipping 70°-90° to the south. Most andesite dykes are relatively thin (0.5-2.0 m). Despite being narrow, they are very continuous and can be traced over several benches in the pit. Random jointing is always present, often closely spaced. The joints are discontinuous and are best described as conchoidal (rounded fractures, similar to fractures in broken glass).

2. Quartz latite dykes are less common in the pit, but are very prominent because of their cream colour and considerable thickness. Two latite dykes in the central portion of the east wall exceed 5 m in thickness. The remaining dykes are thinner, (1-3 m). Insufficient dykes exist to identify any structural trends, but the two thick latite dykes in the central portion of the east wall dip moderately to the southwest, parallel to the andesite dykes that are also present in the area. Perhaps all dykes within the "central dyke package" preferentially invaded along some weakness in the rockmass, e.g. an old fault zone. Quartz latite is rated as strong (R4, 200 MPa uniaxial compressive strength).

3. The trachyandesite dykes (commonly called trachyte) are very similar to the gabbro-monzonite intrusive rocks in appearance. They are coarse grained, containing up to 15% bladed plagioclase phenocrysts that are very distinct when present. Colour is dark speckled grey. This dyke type is relatively rare in the Main Zone pit; as a result, no structural trends have been identified. Core samples of the dyke were not available for testing, but the trachyte can be classified as R5, very strong rock. Uniaxial strengths will likely be similar to those obtained from gabbro specimens.

Dykes are very important in the overall wall stability evaluation because altered gouge is often present along one or both contacts. Any such surface must be considered as a low strength throughgoing discontinuity that could provide a release surface for a major wall failure if unfavourably oriented.

Dykes also control groundwater seepage in the Main Zone pit. Most groundwater seepage in the pit walls exits at dyke contacts (see Figure 3.3). Whether this seepage is caused by the low permeability clay gouge that forces water to flow along the discontinuity or the dykes fractured the adjacent rockmass during intrusion to provide drainage paths of higher permeability is yet to be determined. If the first hypothesis dominates dykes could have a very unfavourable influence on pit drainage, wells would have to be located at closer spacings and carefully positioned in the central areas of dyke isolated blocks.

3.4 MINERALIZATION

Equity Silver Mines Ltd. produces concentrates of silver, copper and gold. The principal source of these metals is chalcopyrite, tetrahedrite and arsenopyrite mineralization. In the Main Zone, the economic minerals occur as fine grained disseminations within the pyroclastic rocks (Unit 2A). Locally, the mineralization grades to massive sulfide, and rarely occurs as veins.

The ore genesis model for the Main Zone orebody is not fully understood. Work by Wetherell (1979) indicates that the ore body is discordant to the stratigraphy. Therefore, the sulfides must have been emplaced after deposition of the pyroclastics, but before gabbro monzonite intrusion, because the gabbro does not contain significant amounts of sulfides.

The mineralized zone strikes approximately north-south and dips at 45° to 60° to the west. The zone extends 700 m along strike, with a maximum thickness of 90 m. The ore body is open to depth.

Figure 3.3 A THICK ANDESITE DYKE. Notice gouge and groundwater seepage at the contacts.

4.0 MINING PROGRAM

4.1 SUMMARY OF PRESENT MINE PLAN

The Main Zone pit design has been updated in the fall of 1984 to reflect changes in metal prices and to incorporate the results of the geotechnical slope design program. The current ultimate pit design is illustrated in Plan 5 (in map envelope).

The Main Zone pit is oval in shape, the long axis is 830 m in length and strikes north - south, parallel to the strike of the ore body. The pit will have a maximum width of 530 m crest to crest. The highest elevation on the pit crest is 1360 m, the ultimate pit floor will be at 1130 m elevation. As a result, the east ultimate pit wall will be 230 m in height.

Access into the ultimate pit will be maintained near the present position, midway along the west wall through the notch of the old Bessemer Creek channel. The main haul road will be maintained on the west side of the pit because the geotechnical study indicates that the west wall will be the most stable, as the majority of discontinuities dip into the pit wall.

Average pit wall angles used in the current design are summarized in Table 4.1. They are the end result of a geotechnical study carried out by the mine engineering department from 1983 to 1984. The methodology used to determine the optimum pit wall angle in each design sector is discussed in Sections 5 and 9 of this report.

Table 4.1

PIT WALL	MAXIMUM HEIGHT (m)	AVERAGE WALL ANGLE (deg)
east	230	45
north	180	46
south	190	48
west	170	48
west	170	40 (with ramps)

Mining will progress on 5 m benches for maximum ore - waste selectivity. Eight meter wide berms will be maintained every 20 m in accordance with the Mining Regulations Act.

4.2 INNER PIT / PUSHBACK

To increase the ore/waste ratio during the early years of mine life the Main Zone pit is being excavated in two stages. An interior pit is being developed in the core of the Main Zone. This pit will be completed to a depth of 1190 m. Waste rock is being mined simultaneously on the upper benches of the ultimate pit; however, the interior pit will be completed several years before the ultimate pit reaches the 1190 m elevation.

The interior pit will provide excellent exposures for collection of structural data close to the ultimate pit wall. The present design of the west ultimate pit is based on observations that were at times projected as much as 400 m to where they were applied. The present ultimate pit design must be re-evaluated once the interior pit is completed and all structural information has been collected.

A second major advantage of the interior pit is that a failure encompassing 2 to 3 berms can be tolerated provided it occurs in an area where it will not influence production. Two areas should be oversteepened in the interior pit in an attempt to induce failure.

One trial should be carried out to steepen the berm face angle to 70° on the southern end of the interior pit west wall. If the berm can be maintained at 70° without failure then the design of the ultimate pit west wall should be reevaluated and possibly steepened to an overall pit wall angle of 53° (before haul road).

The second trial should be on the east wall of the interior pit. A wedge structure formed by two major discontinuities should be identified and the slope steepened until that structure daylights. By performing a back analysis on any induced failure a better estimate of friction angle can be achieved, and will result in a more accurate assesment of the factor of safety in the east wall.

5.0 MAIN ZONE STRUCTURAL GEOLOGY

5.1 PAST PROGRAMME

The mine engineering department at Equity carried out a preliminary pit design investigation in 1983. The goal of this study was to determine whether steepening of the pit walls in the Main Zone appeared feasible and whether a more detailed investigation was warranted.

The study consisted of line mapping and structural analysis of the data. Line mapping information was entered into the Placer computer system. The data was processed with the Discodat program package. Stereonets generated by the program were analyzed to define structural domains and design sectors in the Main Zone. The study concluded that steepening of the pit walls appeared feasible on the west side of the pit and that a more detailed geotechnical slope design program was required to further define the observed structural trends.

5.2 STRUCTURAL MAPPING PROGRAM

A detailed line mapping program was completed in the summer of 1984. All safely accessible berms in the Main Zone pit that were not mapped in 1983 were carefully examined. In all, 64 line mapping traverses were completed, each approximately 30 m in length. Approximately 1920 m of berm were mapped in total.

Information that was collected during the traverses included: 1) location, 2) discontinuity type, 3) discontinuity orientation, 4) length, 5) width, 6) spacing, 7) lithology, 8) rockmass hardness, and 9) groundwater conditions. The data was recorded on a standard Discodat coding form and later entered into the computer.

Because the Main Zone pit is located on a westerly sloping hillside and is in the early stages of development very little rock has been exposed on the west pit wall. Approximately 50% of structural data used in this study was collected on the east wall of the ultimate pit and 50% on the south, east, and west walls of the interior pit. The location of each traverse is plotted on Plan 3.

5.3 STRUCTURAL DOMAINS

Structural domains are areas within a pit that have consistent rock type and structural orientations. Based on results of the line mapping programs the Main Zone has been divided into four structural domains. The characteristics of each structural domain are summarized in the following four subsections. The information for each structural domain is illustrated on four stereo nets that include:

1) Orientations of "major structures" including faults, shears and dykes. These structures are very continuous and gouge is usually present on the discontinuities; therefore, they will have the greatest influence on full wall stability.

2) The "all major structures" stereonet shows the distribution of poles to faults, shears, dykes and major joints. This stereonet is used in the majority of design work because it shows the peak orientations of all continuous planes that could form release surfaces. Because in most areas of the pit there are many more major joints than any other major structures the joints can mask very important structural trends in the larger major discontinuities. That is why the faults, dykes and shears are treated separately in stereonet 1. The first table in each structural domain section summarizes the most important peak orientations

from the two stereonets.

3) The peak orientations of each discontinuity type are summarized in the third figure in each structural domain section. A unique symbol is used to identify each type of discontinuity and a number indicates the relative size of each peak on that stereonet (e.g. 1 implies the largest peak). A legend that explains what discontinuity each symbol represents is presented in Table 5.1. The stereonets for each discontinuity type from which the "poles to major structures by type" figures were constructed are presented in Appendix A.

4) All wedge, plane, block, and toppling failure modes that are formed by planes with orientations of the peak major discontinuities as determined in 1) and 2) are shown in the fourth figure in each structural domain. This figure is titled "failure modes". It must be realized that many of these failure modes will not be dangerous because they may dip into the pit wall, or too steeply to daylight. They may also dip so flat that the dip angle of the intersection will be shallower than the angle of friction and sliding will not be possible. A stability assessment that considers pit wall geometry and shear strength characteristics is carried out for each design sector in section 9 of this report. The type of failure described by the terminology used in this paragraph is explained on the next page and sketches of a typical failure in each category are shown in Figure 5.0.

A plane failure is the simplest failure mechanism. A mass of rock slides out on a single plane. For this failure to occur the failure surface must strike nearly parallel to the wall (+/- 10°) of friction and cohesion. Some form of lateral release surfaces must

Table 5.1 SYMBOLS AND CODES USED IN STEREONETS

Symbol	Discontinuity Type	Symbol	Size	Symbol	Size
○	fault	1	> 1	G	> 16
		2	> 2	H	> 17
□	shear	3	> 3	I	> 18
		4	> 4	J	> 19
⬡	major joint	5	> 5	K	> 20
		6	> 6	L	> 21
△	dyke	7	> 7	M	> 22
		8	> 8	N	> 23
Symbol	Failure Mode	9	> 9	O	> 24
		A	> 10	P	> 25
○	plane	B	> 11	Q	> 26
		C	> 12	R	> 27
△	wedge	D	> 13	S	> 28
		E	> 14	T	> 29
⬡	toppling	F	> 15	U	> 30

1. size indicates the total weight of poles within a circle that has an area equal to 1% of the stereonet and centered on the peak of the of the distribution.

be present to allow the rockmass to fall out unless the plane is situated on a convexity so the sides also daylight out of the wall as was the case on one of the large failures in the Southern Tail pit.

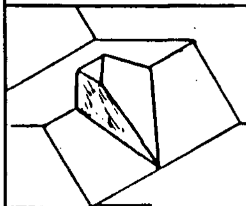
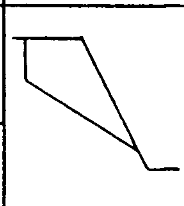
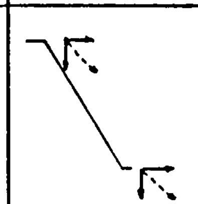
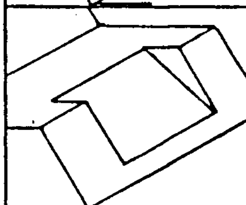
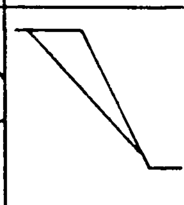
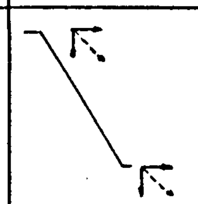
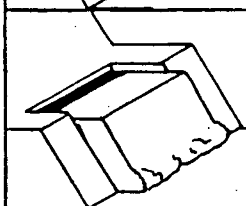
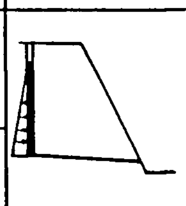
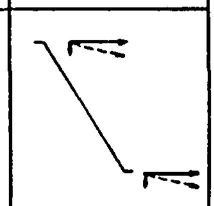
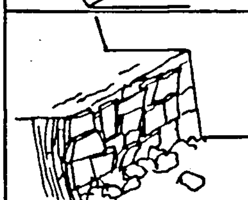
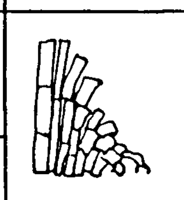
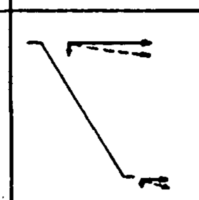
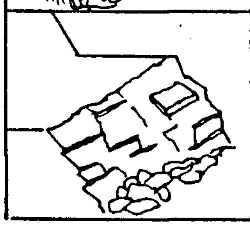
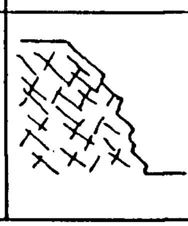
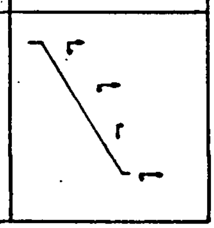
A wedge failure is formed by two intersecting discontinuities. For a wedge to be unstable it must daylight; the line of intersection of the two planes must dip shallower than the face of the slope. As a rough rule of thumb the line of intersection must also dip steeper than the friction angle for sliding to occur. A detailed stability evaluation that considers wedge geometry, shear strength parameters on each failure surface, and pore pressure will usually show that a wedge will be stable even when it plunges considerably steeper than the angle of friction because a large component of the stabilizing force is derived from the increased surface area on the failure plane per unit mass. This is especially true when the wedge becomes very tight.

A block failure is really a plane failure with a vertical tension crack. The failure plane is usually flat lying (less than 20°) and the failure is driven by pore water pressures developed in the tension crack. This failure mode is much less common in hardrock mines than the first two modes discussed.

Toppling occurs when discontinuities dip very steeply and strike nearly parallel to the pit wall face. If the sheets of rock are overhanging or sufficiently large water pressures exist the top of the sheet can become unstable and little by little the entire berm or pit wall can loose its intactness and become a pile of boulders. Toppling is a progressive failure that usually takes some time to develop and is fairly uncommon in mines, but may occur in isolated zones on the west wall of the Main Zone pit.

The final failure mode that has been observed in the Main Zone is step failure. This failure occurs only in heavily jointed rockmasses where joint sets are closely spaced and discontinuous. Numerous cross joints must also be present. The actual failure plane steps across from one joint to another and can dip considerably steeper than the joint set. This failure mode will not result in large failures, but appears to limit the steepness of the berm face in the volcanics to 66°.

Figure 5.0 COMMON FAILURE MODES IN OPEN PITS

ISOMETRIC	X-SECTION	FAILURE MODE	DISPLACEMENT VECTORS
		**WEDGE** - REQUIRES TWO INTERSECTING PLANES - TENSION CRACK MAY DEVELOP - STABILITY FUNCTION OF WEDGE GEOMETRY, SHEAR STRENGTH AND GROUNDWATER	
		**PLANE** - MOVEMENT ON ONE PLANE - REQUIRES LATERAL RELEASE SURFACES - PLANE PARALLEL TO FACE +/- 20 - MUST DAYLIGHT	
		**BLOCK** - RARE IN HARD ROCK MINES - DRIVEN BY WATER PRESSURE IN TENSION CRACK - USUALLY REQUIRES LOW STRENGTH BASAL PLANE, E.G. CLAY	
		**TOPPLING** - REQUIRES STEEPLY DIPPING JOINTS - WILL OCCUR ONLY IN LOCAL AREAS IN MAIN ZONE	
		**STEP** - WILL OCCUR ONLY IN VOLCANICS - REQUIRES CLOSELY SPACED AND DISCONTINUOUS JOINTS - CONTROLS BERM FACE ANGLE - STABILITY LIKELY A FUNCTION OF TRIM BLASTING METHOD	

5.3.1 Structural Domain D1

LOCATION:

Structural Domain D1 comprises the south and west walls of the Main Zone pit. To date, this domain has been defined along exposed walls of both Interior and Ultimate pits between elevations 1285 to 1360 m. It is likely that the domain extends across most of the current pit floor on 1285 bench and to the westerly dipping gabbro contact at depth.

GEOLOGY:

Lapilli tuff is the dominant lithologic unit in this domain. Localized zones of ash and dust tuff are also present. Dykes of the three main compositions penetrate the volcanics. Most of these dykes are less than 2 m thick, abnormally thin in comparison to dykes in other areas of the Main Zone.

SELECTION:

Structural domain D1 was defined on the basis of lithology, (lapilli tuff), and a strong structural trend in the major discontinuities (striking east-west, and nearly vertical).

ORIENTATIONS:

Five clusters of orientation trends were identified in domain D1. The peak orientations of these trends are summarized in Table 5.2; the stereonets used to define the trends are shown in Figures 5.1 & 5.2. The dominant orientations of major structures in this domain strike easterly, dipping steeply to the south. This orientation trend is observed in all the major structures including faults, contacts, shears, and major joints; as can be seen in Figure 5.3. This figure shows the peak orientations of each major structure by type.

Because much of the line mapping in this domain was carried out on bench walls running north south, most of the structures were coming straight out of the wall. The Terzaghi correction factor for these structures was therefore very low as the true spacing was observed. The few structures that had strikes near parallel to the traverse line were assigned very high correction factors and artificially dominated the corrected stereonet plots. Because field observations do not indicate that the structures are as dominant as the corrected plot suggests all analysis in this structural domain is based on uncorrected stereonet plots.

Table 5.2 Orientations of Major Structures

Peak	Dip Direction (deg)	Dip (deg)	Size Code	Weighted Percentage of Population in Peak
A	157	67	7	7
B	179	77	7	7
C	256	55	5	5
D	224	55	3	3
E	76	88	3	3

Table 5.3 Orientations of Major Structures by Type

Type	Peak	Dip Direction (deg)	Dip (deg)	Size Code	Weighted Percentage of Population in Peak
FL	1	152	70	H	17
FL	2	246	62	H	17
FL	3	189	85	A	10
CN	1	176	74	B	11
CN	2	74	88	B	11
CN	3	251	53	8	8
SR	1	178	86	F	15
SR	2	200	45	A	10
SR	3	219	42	A	10
MJ	1	175	78	9	9
MJ	2	256	68	7	7
MJ	3	006	82	6	6

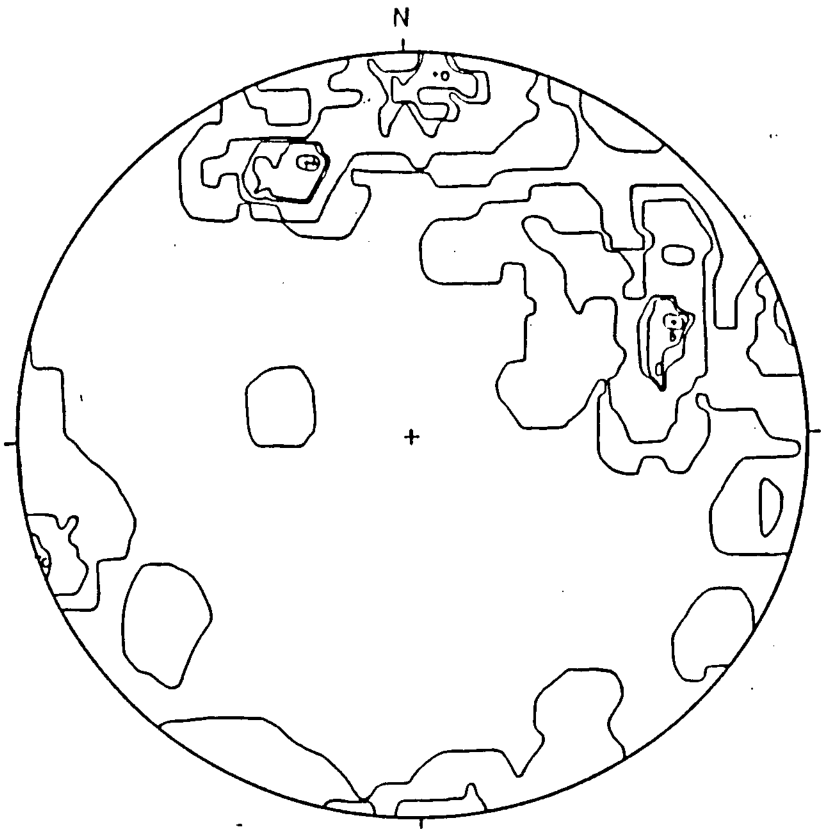


Figure 5.1
DOMAIN DI
MAJOR STRUCTURES
FL CN SR
UNWEIGHTED

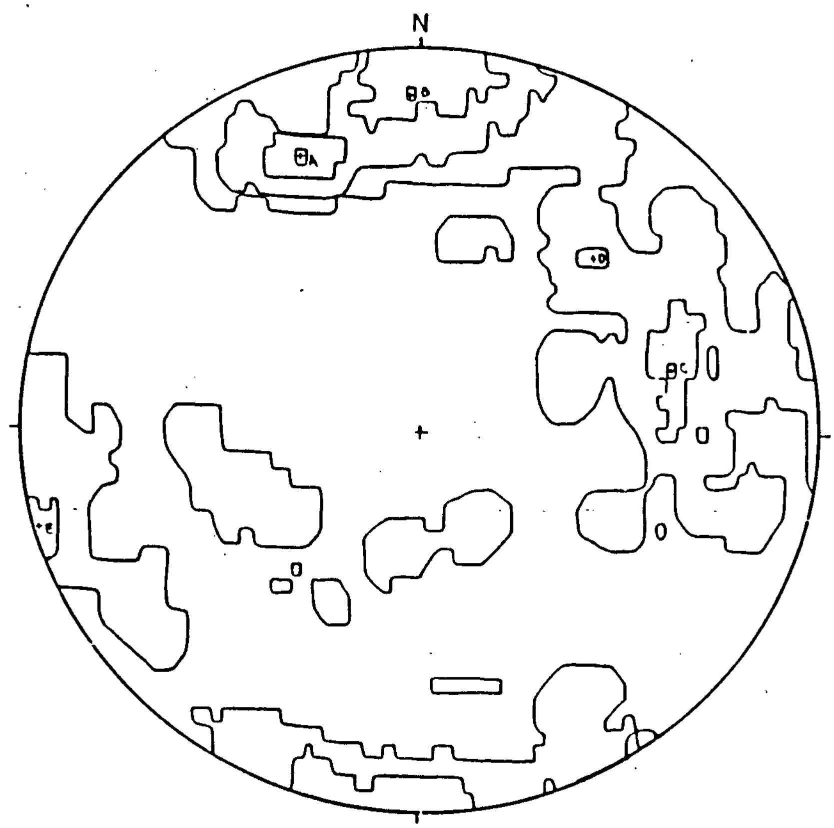


Figure 5.2
DOMAIN DI
ALL MAJOR STRUCTURES
FL CN SR MJ.
UNWEIGHTED

FAILURE MODES:

The four groups of major discontinuities combine to form ten possible failure modes. The failure types and orientations are summarized in Table 5.4. A stereonet showing the possible failure modes in this domain is shown in Figure 5.4.

Table 5.4 Failure Modes - Domain D1

Failure #	Mode	Direction of Slip (deg)	Plunge (deg)
1	wedge	252	48
2	wedge	247	55
3	wedge	228	52
4	wedge	217	48
5	wedge	212	52
6	wedge	157	74
7	wedge	163	32
8	wedge	112	57
9	plane	156	55
10	toppling	265	3 over

Which failure modes in Table 5.4 will be kinematically unstable will depend on pit wall geometry. Figure 5.4 indicates that most stability problems in this domain will be encountered on walls that trend 320° (NW-SE). Fortunately, no pit walls have this unfavourable orientation in this structural domain.

For the current pit design failure modes #8 and #10 are the most unfavourable and will influence pit design. Mode #8 is a wedge failure that plunges at 57° to the south east. It could result in single berm wedge failures on west to north-west pit walls. Large, full wall failures may result if overall pit angle is steepened to undercut the wedges. Mode #10, toppling, may occur on benches of the west and south-west walls. As the pit is concave in this area toppling should be confined to failures less than 100 m in length and should not pose significant stability problems.

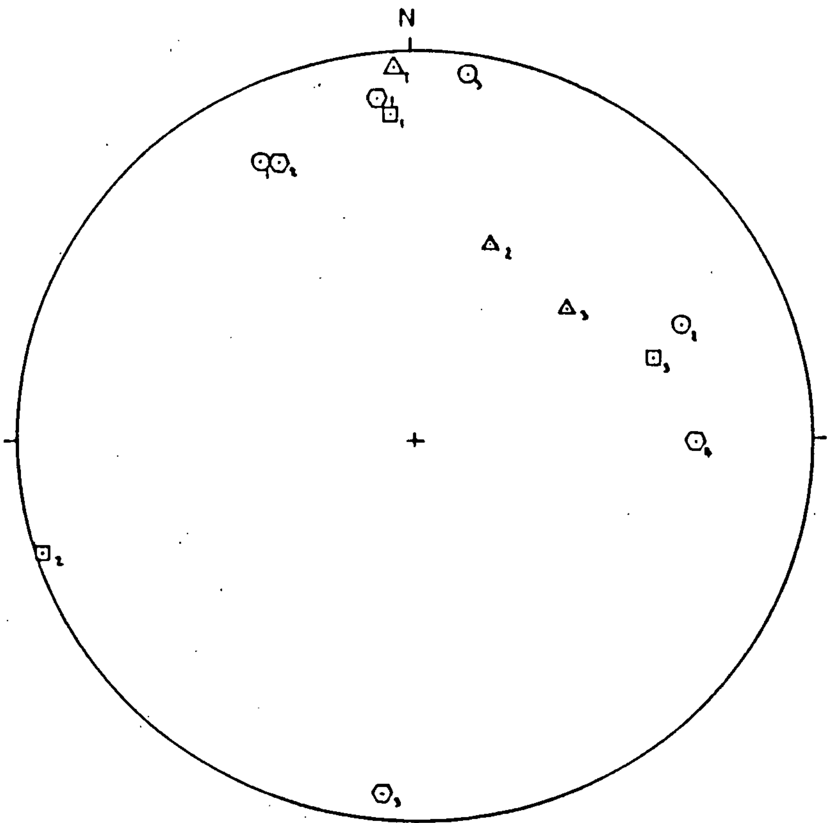


Figure 5.3
DOMAIN DI
POLES TO MAJOR
STRUCTURES BY TYPE

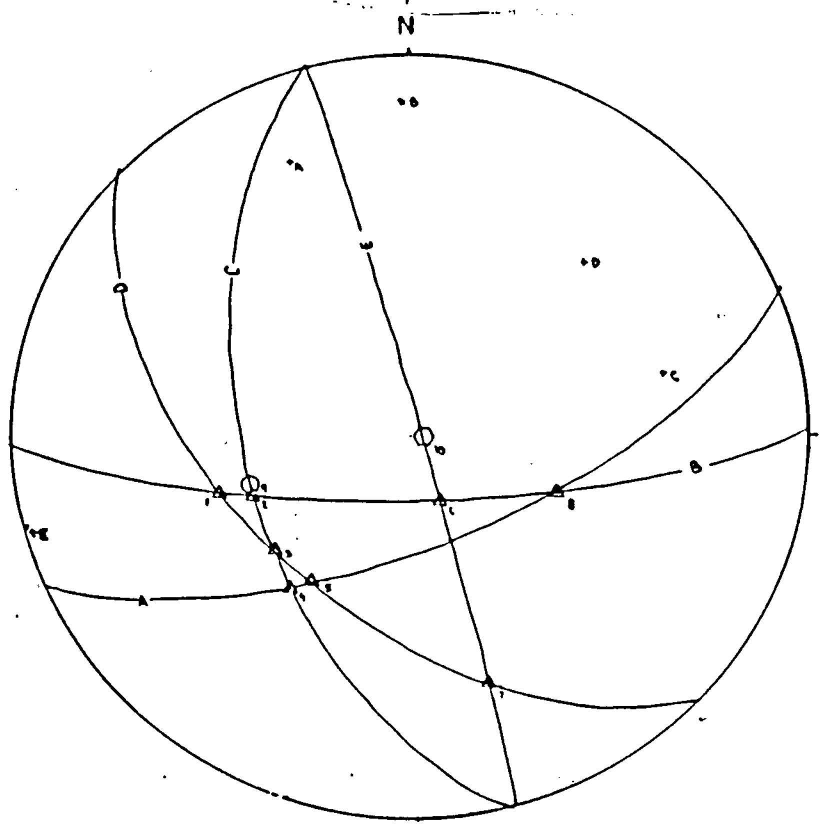


Figure 5.4
DOMAIN DI
FAILURE MODES

5.3.2 Sructural Domain D2

LOCATION:

Structural domain D2 is located in the southeastern corner of the Main Zone pit. The northern boundary of the domain is formed by a series of thick dykes, the southern by the gabbro-volcanic contact. Pit walls in this domain trend north - south to northeast - southwest.

To date this domain has been defined along exposed walls of both interior and ultimate pits between elevations of 1285 and 1360 m. It is likely that the domain extends along the gabbro volcanic contact to the south and to depth.

GEOLOGY:

Structural domain D2 is formed by the gabbro rocks close to the gabbro-volcanic contact that dips at approximately 50° to the west, into the pit. Lapilli tuff is also present in small areas within D2, especially below 1320 m elevation.

A thick, east-west trending quartz latite dyke bisects the domain. It dips steeply to the south. Several thinner (2 to 5 m) andesite dykes are also present, some have sub-parallel orientations to the quartz latite, a second set trends northwest - southeast.

SELECTION:

The main selection criteria in this domain are the gabbro lithology and proximity to the gabbro volcanic contact. A very strong easterly dipping trend in joint orientation is also unique to this domain. The boundaries of D2 are formed by a series of several thick andesite and quartz latite dykes to the north and the gabbro - volcanic contact to the south. In both cases the boundaries are marked by changes in structural orientations, especially in the joint population.

ORIENTATIONS:

Six clusters of orientation trends were identified in domain D2. Four are strong primary peaks while peaks D and E are secondary highs on the flanks of the primary clusters. The contoured stereonet plot of the major discontinuities (without major joints) is shown in Figure 5.5. Figure 5.6 is a plot of all major discontinuities including major joints. It is not used as the principal design distribution because the very strong trend in major joints masks all other major structures.

Weighted stereonets were used for analysis in this domain because the weighting did not attach excessive importance to a few structures of minor significance as was the case in domain D1.

The dominant orientation in this domain is a northerly strike, dipping moderately to steeply (45° - 80°) to the west. It is observed in all major structures but is most prominent in the joint population. Table 5.5 summarizes the six peak orientations of major structures in domain D2. Table 5.6 lists the peak orientations by structure type. These orientations are also plotted on a stereonet in Figure 5.7.

Table 5.5 Orientations of Major Structures - Domain D2

Peak	Dip Direction (deg)	Dip (deg)	Size Code	Weighted Percentage of Population in Peak
A	267	51	H	17
B	136	47	A	10
C	173	83	A	10
D	146	62	7	7
E	295	69	5	5
F	253	67	5	5

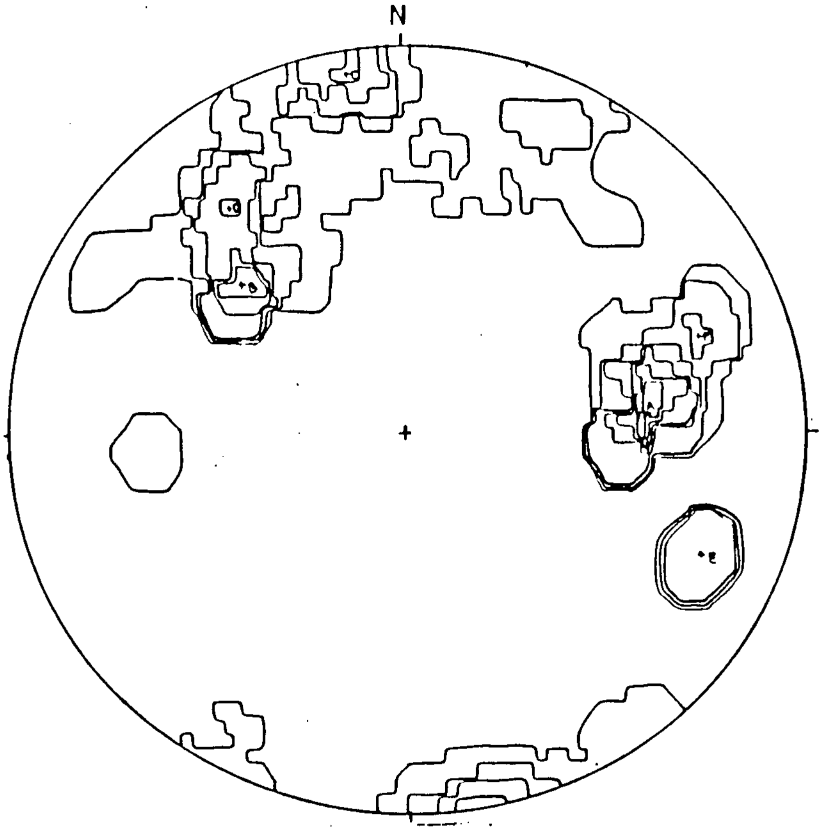


Figure 5.5

DOMAIN D2
MAJOR STRUCTURES
FL CN SR
WEIGHTED

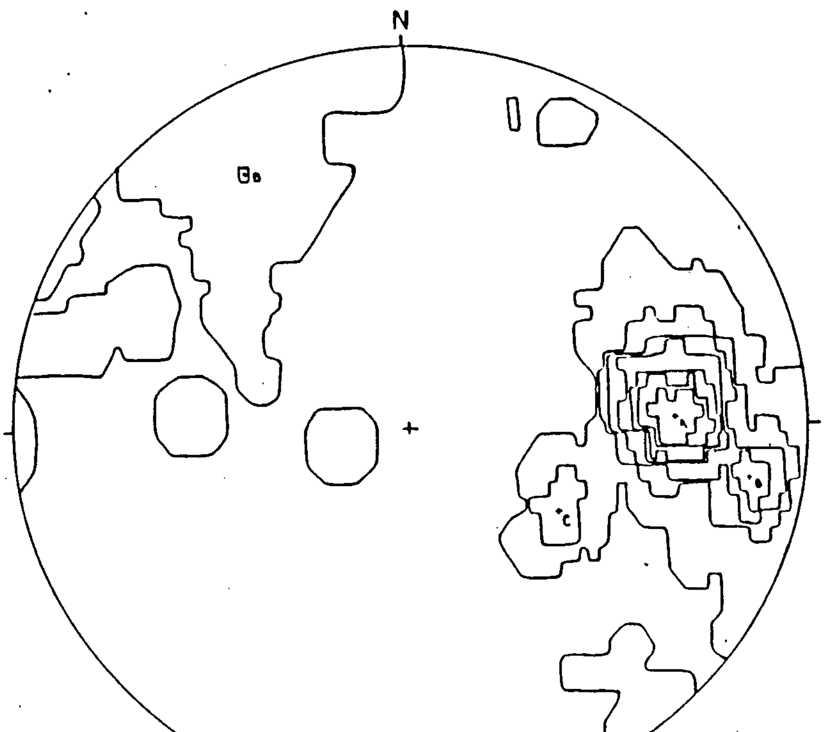


Figure 5.6

DOMAIN D2
ALL MAJOR STRUCTURES
FL CN SR MJ
WEIGHTED

Table 5.6 Orientations of Major Structures by Type - Domain D2

Type	Peak	Dip Direction (deg)	Dip (deg)	Size Code	Weighted Percentage of Population in Peak
FL	1	292	69	I	18
FL	2	253	58	F	15
FL	3	172	83	D	13
CN	1	259	54	U	30
CN	2	172	83	E	14
CN	3	198	74	A	10
SR	1	135	48	M	22
SR	2	272	44	H	17
SR	3	347	82	8	8
MJ	1	256	58	I	18

FAILURE MODES:

The six groups of major discontinuities combine to form 21 possible failure modes. Each failure type and orientation is listed in Table 5.7. The failure planes and orientations of lines of intersection are plotted in Figure 5.8.

Table 5.7 Failure Modes - Domain D2

Failure #	Mode	Direction of Slip (deg)	Plunge (deg)
1	wedge	324	29
2	wedge	265	66
3	wedge	252	50
4	wedge	248	62
5	wedge	244	66
6	wedge	224	41
7	wedge	206	30
8	wedge	212	35
9	wedge	209	15
10	wedge	198	24
11	wedge	194	50
12	wedge	180	35
13	wedge	90	48
14	wedge	87	33
15	wedge	67	22
16	plane	292	69
17	plane	266	50
18	plane	292	69
19	plane	145	62
20	plane	135	44
21	toppling	352	8 over

Because the pit walls in this domain will trend north - south and northeast - southwest the unfavourable failure modes will have directions of plunge between 170° & 330° and plunge between 30° & 70°. Wedge failures 1 - 8 and 11 - 12, plane failures 16 - 18, and toppling failure 21 have kinematically unstable orientations.

For the eastern pit wall in domain D2 the most important kinematically possible failures are:

wedge 3 - bench failures, wall failures if pit steeper than 51°
wedge 4 - bench failures, wall failures if pit steeper than 62°
wedge 6 - bench failures, wall failures if pit steeper than 52°
wedge 7 - bench failures, wall failures if pit steeper than 43°
plane 17 - bench failures, wall failures if pit steeper than 50°

Wedge 7 plunges obliquely to the pit wall and will not be as significant as the 43 degree angle of plunge suggests. Failure modes 2, 5, 16, and 18 plunge too steeply to affect full wall stability, but may result in small berm failures.

Far fewer kinematically unstable failure modes can be identified along the southeastern walls of domain D2. Wedge 1 is the only full wall failure mode possible on this wall, as it will plunge shallower than the slope angle. However, because the angle of plunge is only 29° shear strength should be sufficient to prevent this wedge from failing. Failure mode 3 may result in minor berm failures.

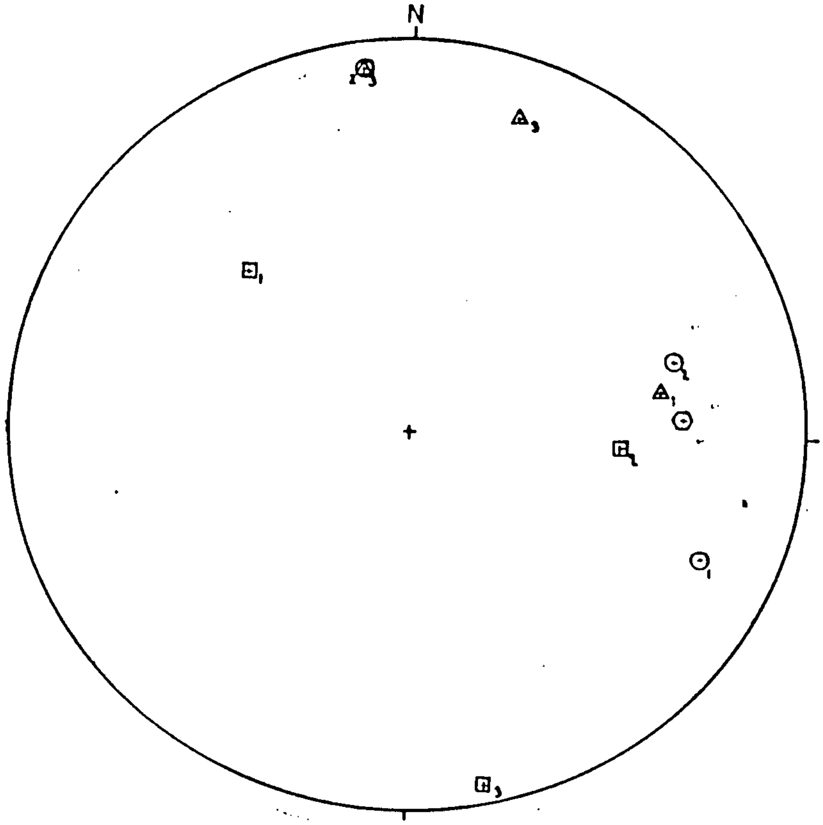


Figure 5.7

DOMAIN D2
POLES TO MAJOR
STRUCTURES BY TY

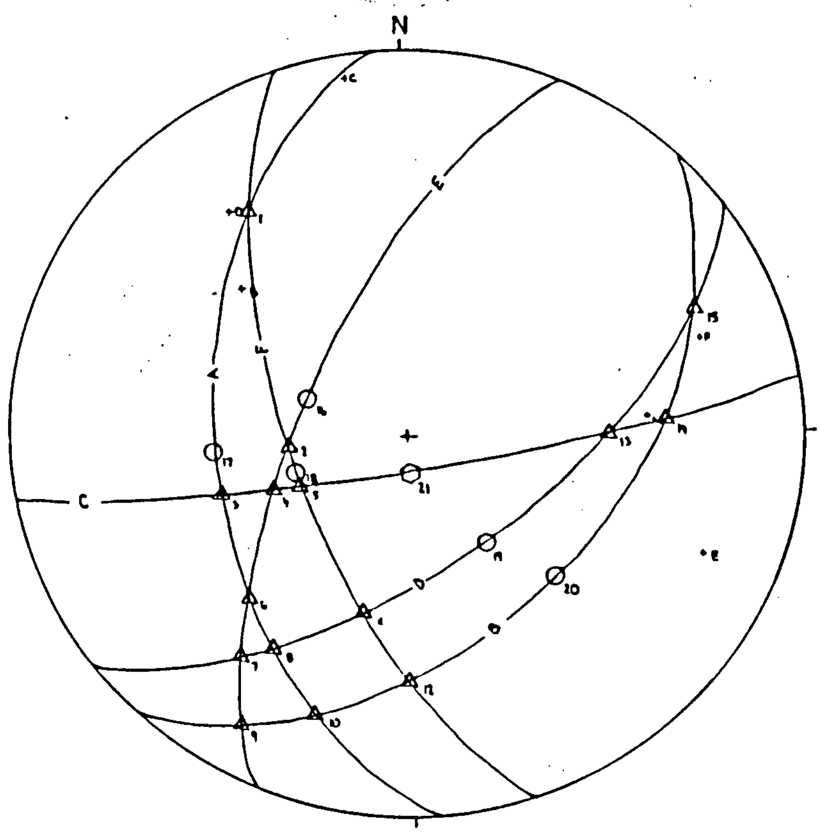


Figure 5.8

DOMAIN D2
FAILURE MODES

5.3.3 Structural Domain D3

LOCATION:

Domain D3 is located in the east half of the Main Zone pit below 1320 m elevation. To the south the domain extends beyond the ultimate pit boundary, to the north it is terminated by the gabbro contact at 7750 m North. At present, the western boundary of this domain is not clearly defined; it is assumed to extend beyond the ultimate pit walls.

GEOLOGY:

The dominant rock unit in D3 is lapilli tuff. Minor dust tuff is also present. A thick package of dykes cuts across the central part of the domain. These dykes plunge steeply (60°-70°) to the southwest. A five meter wide quartz latite dyke runs along the southern domain boundary. Pit walls in D3 have formed along major discontinuities, numerous berm size plane and wedge failures can be seen in the wall. This is in contrast with the volcanics of domain D1 where step failures along minor discontinuities control stability of the berms.

SELECTION:

Domain D3 has been defined by the volcanic lithology and a very strong orientation trend in the major discontinuities (i.e. faults, shears, and contacts), dipping steeply to the southwest. The east and south domain boundaries are defined by the gabbro - volcanic lithology change. The southern boundary between D3 and D1 has been identified by a change in discontinuity orientation. At present this boundary is assumed to extend in a vertical plane trending east - west. Further structural mapping and analysis is required in the west half of the Main zone to clearly define this boundary.

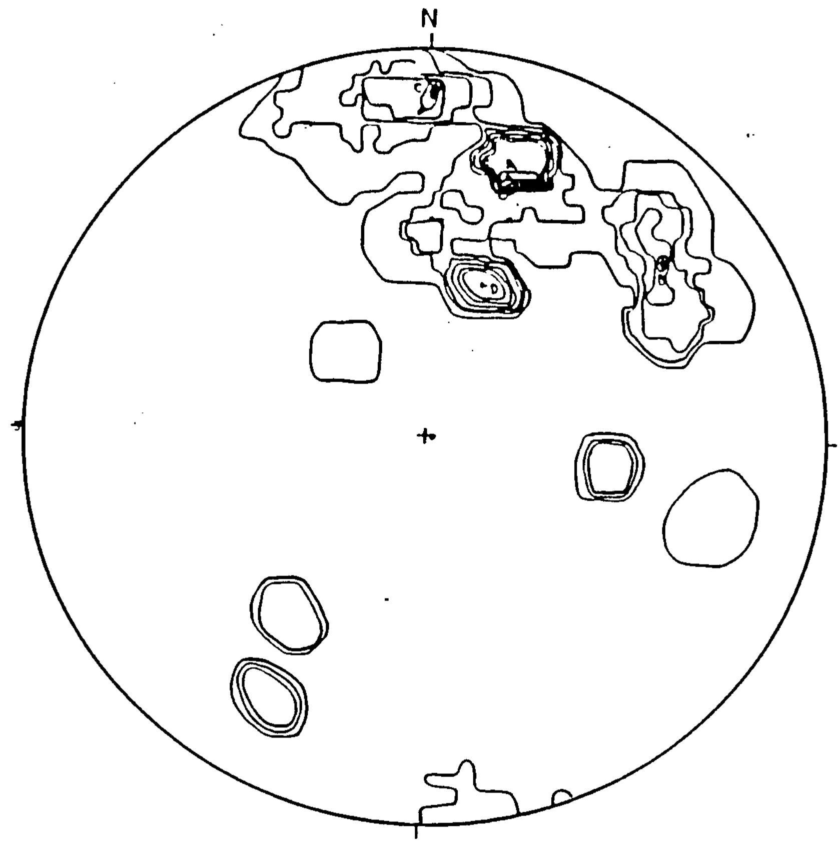


Figure 5.9
DOMAIN D3
MAJOR STRUCTURES
FL CN SR

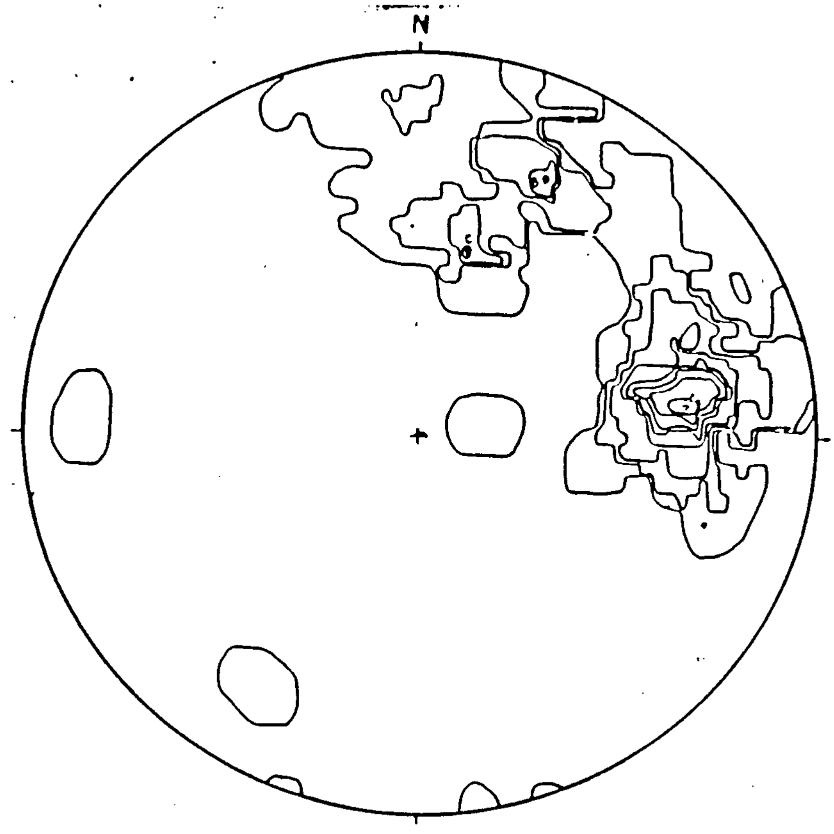


Figure 5.10
DOMAIN D3
ALL MAJOR STRUCTURES
FL CN SR MJ

ORIENTATIONS:

Two principal orientation groups are observed in D3. The majority of faults, shears, and contacts plunge steeply to the south, while the principal major joint orientation is a plunge of approximately 50° to the west. Table 5.8 summarizes the peak orientations of the five largest clusters. The stereonets used to define the peaks are presented in Figures 5.9 & 5.10.

Table 5.8 Orientations of Major Structures - Domain D3

Peak	Dip Direction (deg)	Dip (deg)	Size Code	Weighted Percentage of Population in Peak
A´	262	58	E	14
A	196	60	E	14
B	232	64	E	14
C	182	78	B	11
D	195	35	A	10

Table 5.9 Orientations of Major Structures by Type - Domain D3

Type	Peak	Dip Direction (deg)	Dip (deg)	Size Code	Weighted Percentage of Population in Peak
FL	1	181	50	P	25
FL	2	287	61	H	17
FL	3	138	26	G	16
CN	1	190	39	J	19
CN	2	232	66	I	18
CN	3	031	66	A	10
SR	1	176	74	J	19
SR	2	283	66	A	10
MJ	1	260	56	J	19
MJ	2	204	63	H	17

FAILURE MODES:

Thirteen failure modes have been identified in domain D3. The type and orientation of each failure mode is presented in Table 5.10. A stereoplot of the controlling discontinuities is shown in Figure 5.12.

A majority of the failures plunge steeply. As a result, they will not daylight out of the overall slope so they will not affect the slope angle directly. However, four failure modes do have unfavourable orientations. In the east wall of the interior pit only wedge failure 3 could result in a full slope failure. As the wedge is very tight and has a shallow plunge it proves stable in a detailed analysis, (f.o.s = 5.0 dry). Wall stability on east walls will therefore be governed by bermface angle. Numerous failure modes daylight out of the east and south walls once the berm angle is increased above 50°. Therefore, bench failures must be expected on the east wall in this domain and adequate berms left to catch slide debris.

On the south walls plane failure 11 is unfavourably oriented, plunging out of the wall at 34°. The same plane can combine with planes A´ and A to form wedges 8 & 9 that also plunge at shallow angles. Fortunately, plane D represents the smallest cluster used in the analysis; therefore, only a small number of these unfavourable planes are expected in the south end of the Main Zone.

Table 5.9 Failure Modes - Domain D3

Failure #	Mode	Direction of Slip (deg)	Plunge (deg)
1	wedge	272	58
2	wedge	270	12
3	wedge	264	34
4	wedge	254	57
5	wedge	246	64
6	wedge	235	56
7	wedge	198	60
8	wedge	198	34
9	wedge	158	28
10	plane	231	64
11	plane	197	34
12	plane	196	70
13	toppling	003	12 over

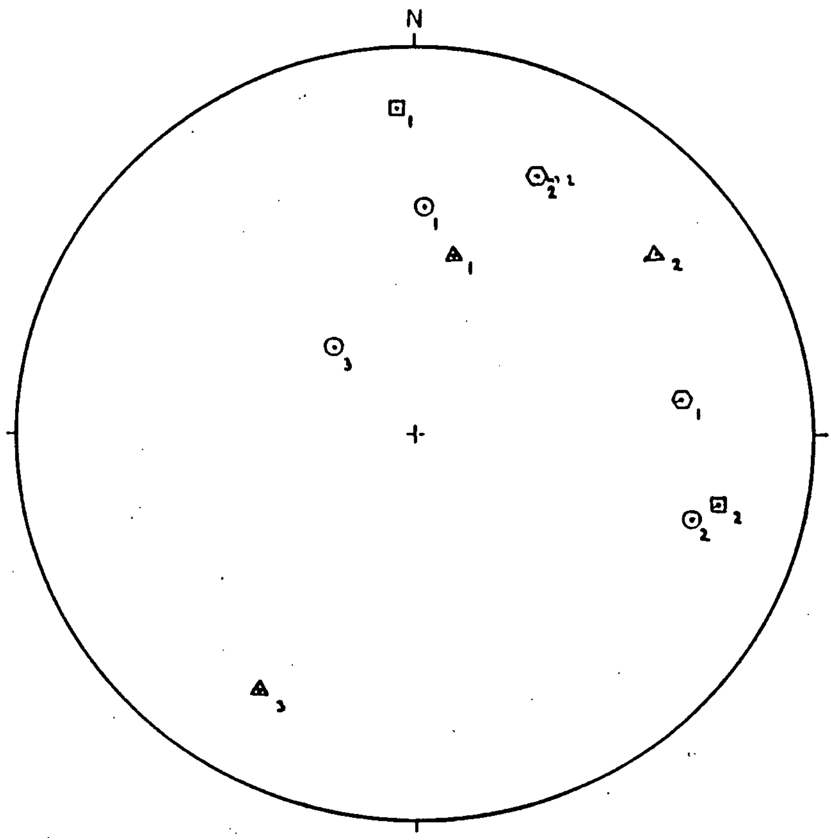


Figure 5.11

DOMAIN D3
POLES TO MAJOR
STRUCTURES BY TYPE

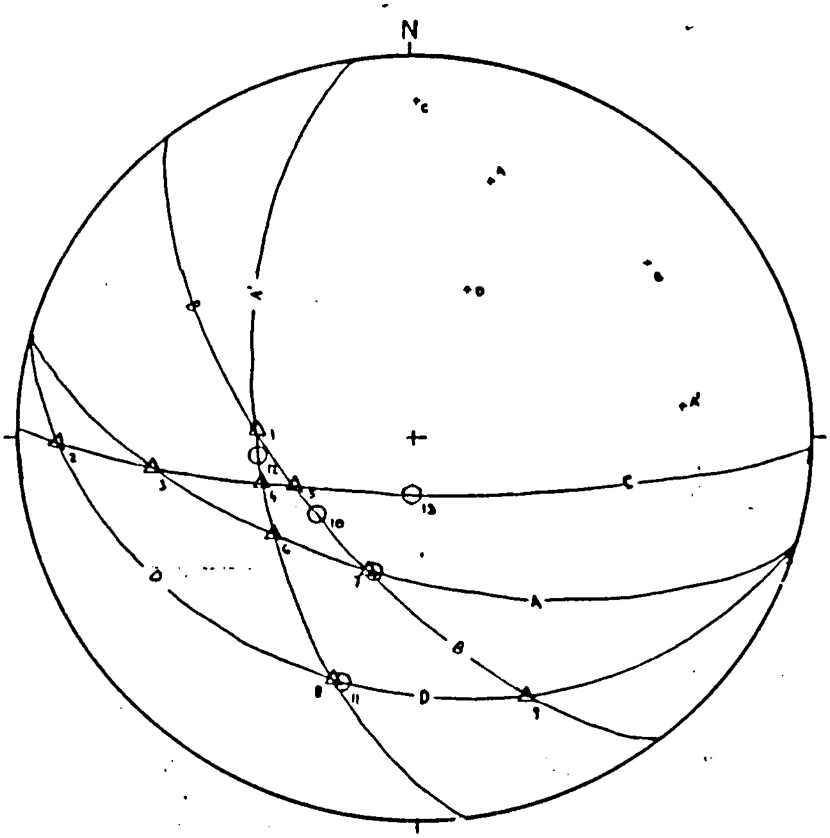


Figure 5.12

DOMAIN D3
FAILURE MODES

5.3.4 Structural Domain D4

LOCATION:

Structural Domain D4 is located in the northeast corner of the Main Zone. To the south the domain extends to the old Bessemer Creek drainage. The north boundary of the domain is beyond the ultimate pit wall and has not been defined. The west boundary between D4 and D3 is formed by the gabbro - volcanic contact that runs approximately north - south along grid coordinate 8750 E. To the east, the domain also extends beyond the ultimate pit wall.

GEOLOGY:

Gabbro is the dominant lithologic unit in D4. It occurs as a large and elongated tongue like intrusion that extends northward from the main gabbro pluton. To the east and west the tongue is surrounded by volcanic rocks, primarly lapilli tuff. Unique to this domain is a 30 m wide band of volcanic conglomerate. The orientation and continuity of this stratum remains to be defined. Moderately thick (3-5 m) quartz latite and andesite dykes have intruded into both rock types. The dominant orientation of these structures is a south westerly plunge.

SELECTION:

Boundaries for this domain were selected to fully contain the gabbro tongue. To the south, the domain boundary was selected at the Bessemer Creek dyke package because a fault is suspected in the area and the structural orientations south of the dyke zone differ from those observed to the north. A strong east-west trend in the orientation of faults, contacts, and shears is dominant in this domain.

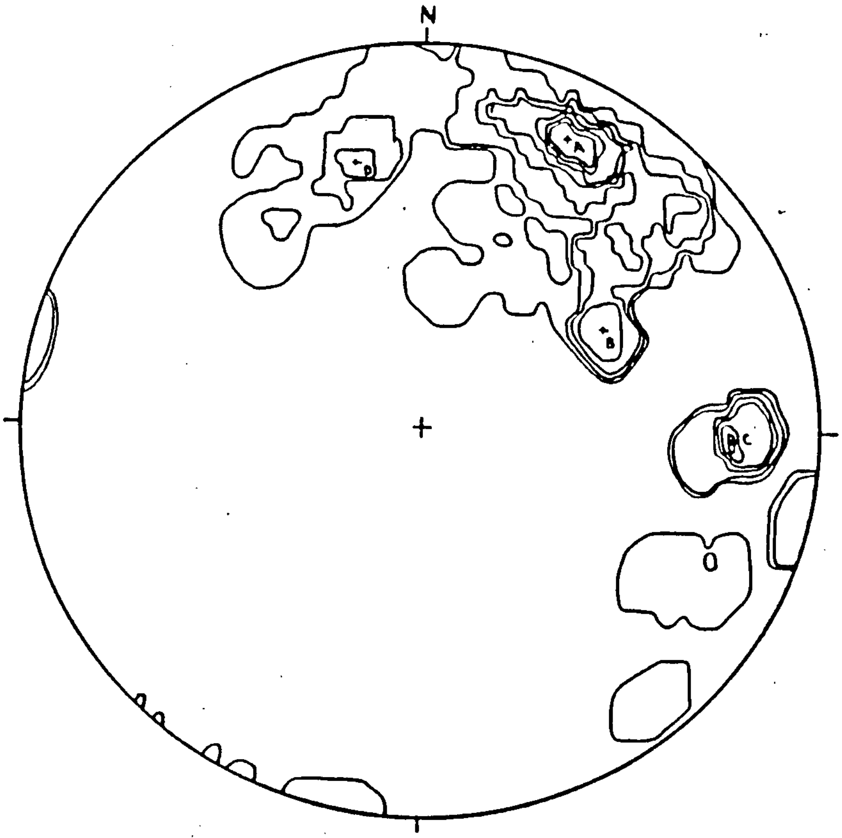


Figure 5.13
DOMAIN D4
MAJOR STRUCTURES
FL CN SR

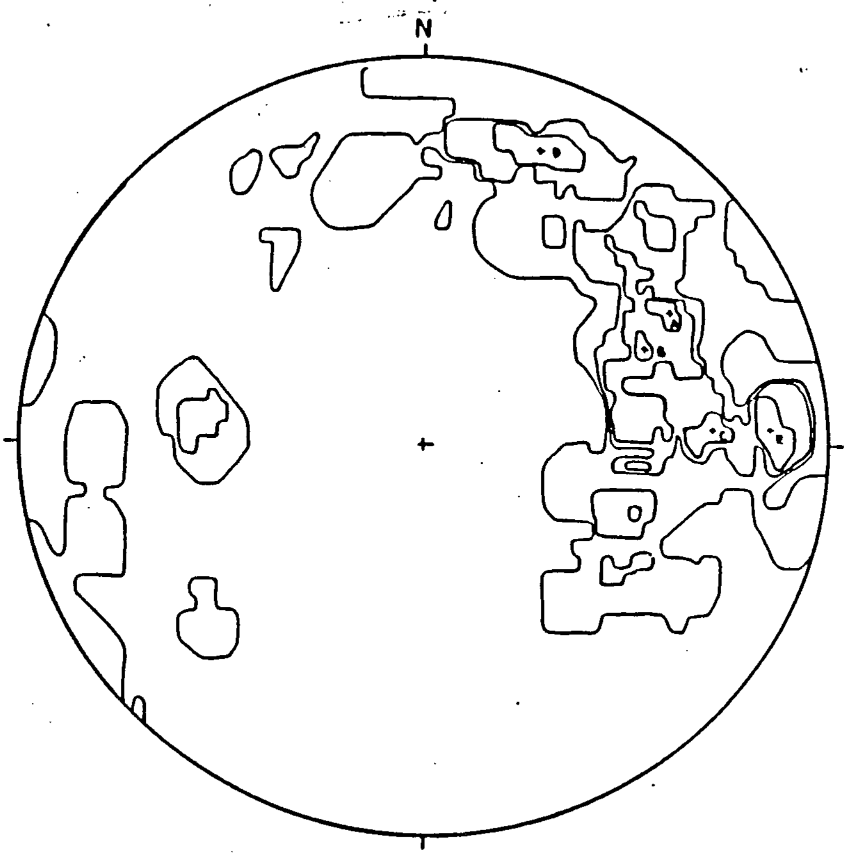


Figure 5.14
DOMAIN D4
ALL MAJOR STRUCTURES
FL CN SR MJ

ORIENTATIONS:

Two strong structural trends are evident in Figures 5.13 and 5.14. First, most of the major structures plunge steeply to the south, generating a very strong cluster (A) in the north corner of Figure 5.13. The second trend, a moderate westerly plunge is very evident in the joint population (see Figure 5.14).

Weighted stereonets were used in the analysis of this domain as they appeared to best represent the structural data observed in the structural geology plan (Plan 2.).

Table 5.11 summarizes the peak orientations of the four largest clusters in the structural fabric. Table 5.12 lists the dominant orientations of each major discontinuity type.

Table 5.11 Orientations of Major Structures - Domain D4

Peak	Dip Direction (deg)	Dip (deg)	Size Code	Weighted Percentage of Population in Peak
A	206	73	F	15
B	240	44	8	8
C	272	67	7	7
D	166	61	5	5

Table 5.12 Orientations of Major Structures by Type - Domain D4

Type	Peak	Dip Direction (deg)	Dip (deg)	Size Code	Weighted Percentage of Population in Peak
FL	1	271	66	H	17
FL	2	226	60	E	14
FL	3	205	71	C	12
CN	1	208	74	L	21
CN	2	240	44	K	20
SR	1	194	79	O	24
SR	2	267	74	F	15
MJ	1	238	60	B	11
MJ	2	262	46	A	10
MJ	3	287	46	7	7

FAILURE MODES:

The four dominant discontinuities combine to form 10 possible failure modes (see Figure 5.16). As domain D4 is in the northeast corner of the ultimate pit only failure modes that plunge into the the southeast and southwest quadrants of the stereoplot must be considered dangerous. In the east wall failure modes 1, 3, and 9 will be kinenatically unstable. Wedge 1 plunges at only 37 degrees to the west. As shear strength tests indicate that the friction angle is less than 37° it is very likely that any wedges that have this orientation will be unstable and may fail on plane B if undercut. Plane B will also control stability on southwest facing walls. Plane failure 9 will daylight if walls become steeper than 45°. Lateral release surfaces will be formed by planes C & D (wedges 5 & 3). No full wall stability problems are anticipated on south facing walls as long as overall wall angle does not exceed 50°. Planar berm failures can be expected on plane D (failure modes 3, 4, & 10) if berm face angles exceed 55°.

Table 5.13 Failure Modes - Domain D4

Failure #	Mode	Direction of Slip (deg)	Plunge (deg)
1	wedge	282	36
2	wedge	250	65
3	wedge	224	44
4	wedge	212	50
5	wedge	199	37
6	wedge	147	58
7	wedge	270	66
8	plane	204	72
9	plane	239	44
10	plane	166	60

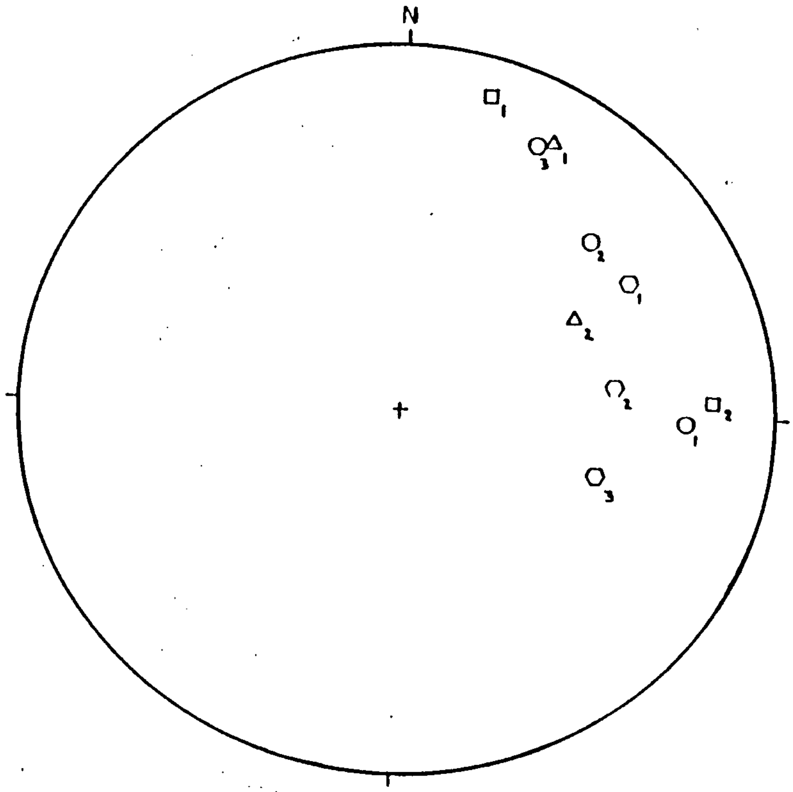


Figure 5.15
DOMAIN D4
POLES TO MAJOR STRUCTURES

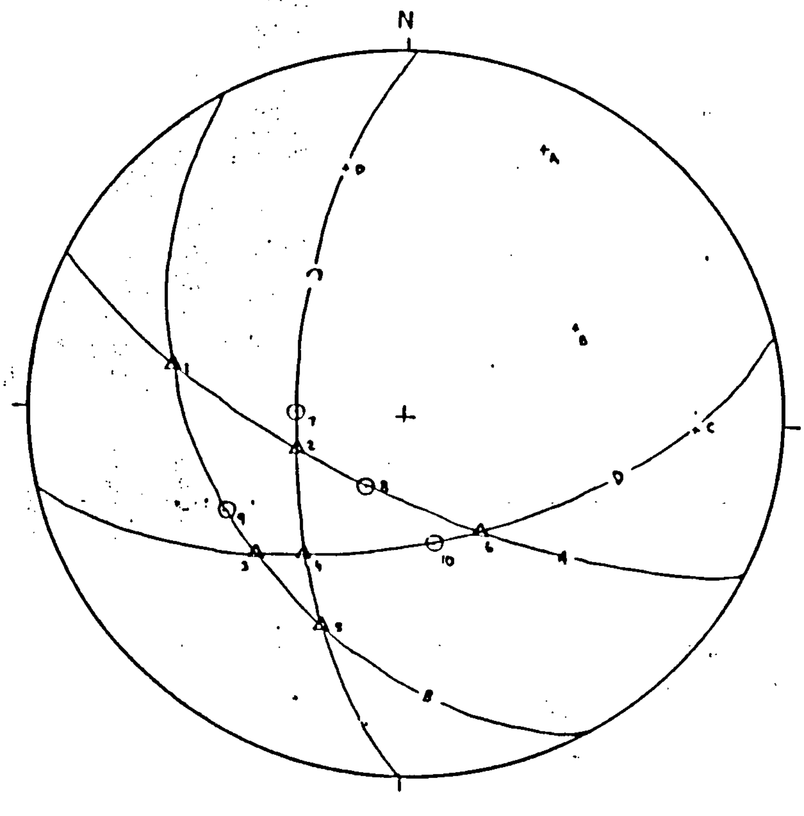


Figure 5.16
DOMAIN D4
FAILURE MODES

6.0 GROUNDWATER HYDROLOGY

The problem of groundwater seepage into the Main Zone pit is becoming more troublesome as mining progresses to depth. Larger quantities of water have to be pumped from the pit floor sump, a greater percentage of blast holes have to be loaded with more expensive slurry explosives, and tire life is likely becoming shorter because equipment has to operate on wetter ramps within the pit. Groundwater also has a a destabilizing influence on any potential slope failures. Therefore, in areas where high water pressures are anticipated, the pit walls have to be designed at shallower angles then in a dry wall to attain the same factor of safety against failure.

Because dewatering of the Main Zone pit would reduce operational costs in explosives, tires, and waste rock transported; a preliminary study was carried out in the summer of 1984 to determine whether pit dewatering is technically possible.

Permeability testing and piezometer monitoring programs were developed to provide information on hydrologic parameters. In combination with available geologic data and observations of surficial water conditions, the test results were used to develop a greater understanding of the hydrologic regime in the vicinity of the pit. Based on this understanding it was possible to identify areas where the greatest water problems can be expected, and where some form of dewatering would prove of most benefit to operations. Several methods of dewatering that would likely prove very effective in the Main Zone are then introduced.

A numerical model was developed to simulate the performance of a dewatering well in the Main Zone environment. The model was used to test whether the rock mass permeabilities are sufficiently high to allow successful dewatering. A sensitivity study was also carried out to determine which hydrologic parameters have the greatest influence on well behaviour and should therefore be established before a pumping system is designed.

The expected performance of the horizontal drainage systems is also discussed, but in a qualitative manner, because the complex geometry of the pit wall and horizontal drains cannot be evaluated by a simple analytical solution. A more detailed numerical simulation utilizing finite difference or finite element techniques would be required to study this problem. A more practical approach would be to perform an in-pit trial of the system to test its effectiveness and then calibrate a numerical model with the observed results for further analysis and sensitivity studies.

6.1 LOCATION OF TEST SITES

All piezometer monitoring sites have been located in the east half of the Main Zone pit, along berms of the east ultimate pit wall and the ultimate pit crest. Figure 6.1 is a plan of the Main Zone pit that shows the position of all piezometer and permeability test locations utilized during the 1984 investigation.

The reasons for the site selection are as follows:

1) Westward sloping topography induces a regional hydraulic gradient to the west. Water will flow down the gradient from recharge areas east of the Main Zone. Much of this water will flow towards the Main Zone pit because the pit has created a large trough in the phreatic surface. Because the east side of the pit will be recharged continuously from a fairly large area the water table is expected to remain close to surface and will result in high pore pressures and significant amounts of seepage into the pit. This behaviour is already being observed and will increase as mining progresses to depth, increasing the hydraulic gradient driving flow. The west half of the pit should not have water problems because the gradient is away from the pit walls in both directions, into the pit to the east and down the Bessemer Creek valley to the west. Therefore, the west wall should eventually become dry. All activity has focused on the east half of the pit because it is the area where the the greatest water problems are anticipated.

2) Geotechnical investigations indicate that approximately five percent of major discontinuities observed on the east wall have unfavourable orientations that could result in multiple berm failures. In this report, the term unfavourable is used to describe any plane that

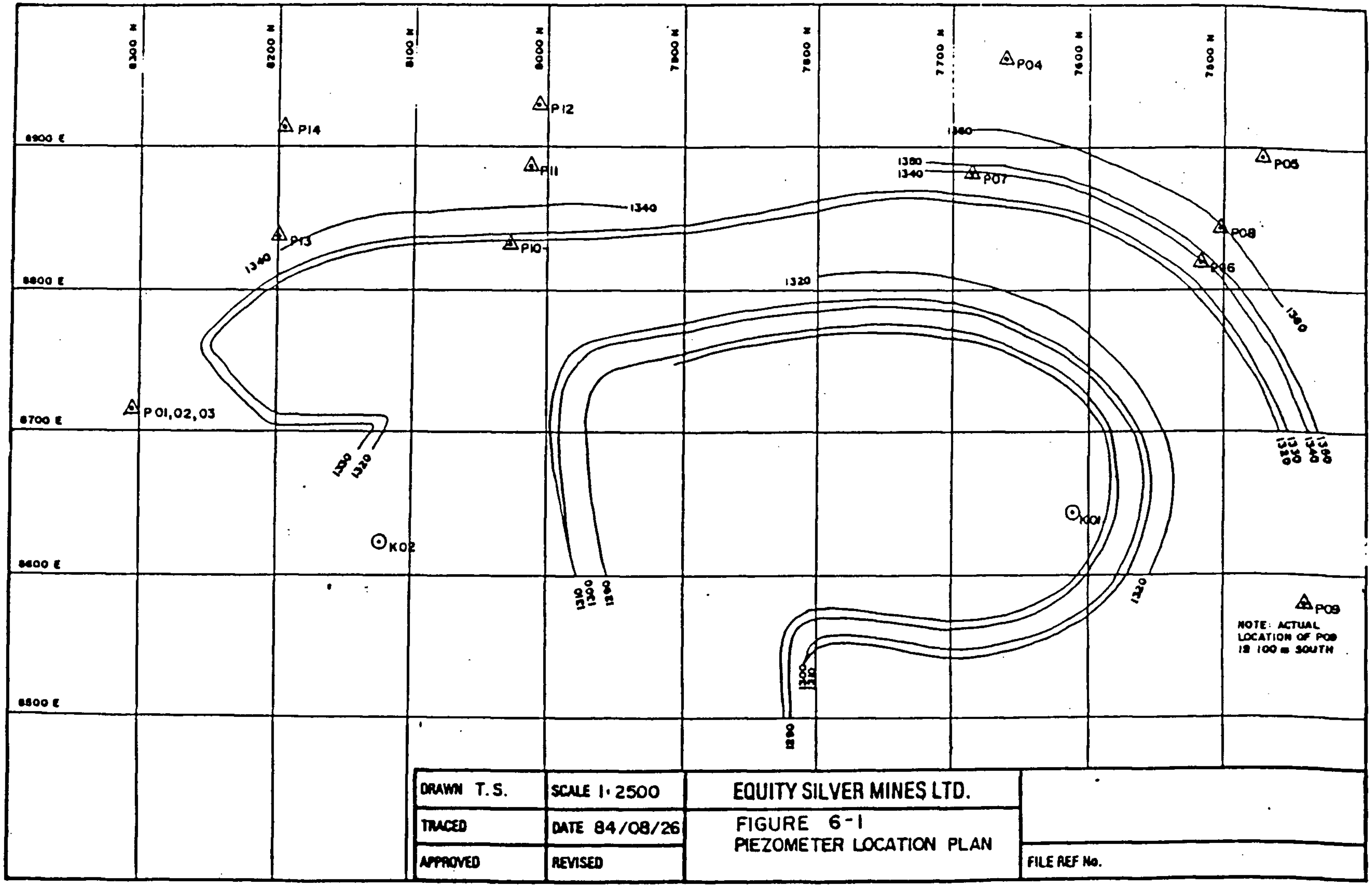


FIGURE 6-1
PIEZOMETER LOCATION PLAN

daylights out of the overall pit slope at an angle that could result in multiple berm failure or in the loss of a major portion of the catchment berm. The magnitude of pore pressure in the wall will significantly affect the stability of any potential failure blocks and must therefore be established.

3) Inspection of drill core (Golder Associates, 1983) has indicated that the gabbro intrusive complex that is situated east of the Main Zone may be highly impermeable and is unlikely to yield large amounts of discharge into the pit. In essence, the gabbro is suspected to act as a dam. Several drill holes were located in the gabbro to confirm that the entire unit is highly impermeable and no zones of fractured or otherwise pervious material exist. Other holes were located in areas that are suspected to provide the principal flow paths for groundwater seepage into the pit. Permeability testing in these holes will assist in designing the most effective dewatering system to intercept and remove the water.

4) With the exception of one hole for permeability testing purposes in the bottom of the Interior pit, all holes were located in areas where no further mining activity is planned. All piezometers will therefore be permanent installations and can be monitored periodically during the entire life of the mine.

6.2 HYDRAULIC CONDUCTIVITY TESTING

6.2.1 Background

Hydraulic conductivity is a rockmass parameter that indicates the rate at which water will flow through the rockmass under a specified hydraulic gradient. The selection and subsequent success of any dewatering scheme is highly dependent on the coefficient of hydraulic conductivity, K. If K is low (e.g. 1.0×10^{-8} cm/s) there will be large resistance to water flow toward the well; therefore, self priming pumps that automatically turn off when the water is drawn near the bottom of the hole will have to be used to prevent the wells from being sucked dry. Also, the radius of pumping influence will be small. If K is large, (e.g. 1.0×10^{-3} cm/s) the rockmass is considered highly permeable. Water will flow toward the wells easily and from large distances. As a result, the drawdown cone will be very broad, but shallow. A very large area must therefore be dewatered to drop the water level in the well a significant amount. Table 6.1 summarizes the normal range of hydraulic conductivity in rock and indicates when dewatering can be successful.

Table 6.1

HYDRAULIC CONDUCTIVITY		DEWATERING CONSIDERATIONS
Quantitative (cm/s)	Qualitative	
1.0×10^{-4} - 1.0×10^{-2}	moderate to high	can be drained easily by well pumps.
1.0×10^{-6} - 1.0×10^{-4}	low to moderate	can be drained over a period of time.
1.0×10^{-8} - 1.0×10^{-6}	nearly impermeable	cannot be drained by conventional methods. Wells must be under vacuum.

6.2.2 Method

Falling head permeability tests were used in the Main Zone to measure the coefficient of hydraulic conductivity. The tests were performed in vertical air trac holes drilled to depths of 30 m. Two pneumatic packers, separated by a 3.07 m perforated pipe were lowered down the hole to desired depth and the assembly was inflated, sealing the test section. Water was then poured into the rod until the water level came up to surface or a steady state condition was attained where water flowed out as quickly as it was poured in. By raising the water level in the rods an excess pressure head was created in the test interval. This head induced water flow into the surrounding rock. The rate of head dissipitation in the rod once the flow is shut off is indicative of the rockmass hydraulic conductivity. This rate was precisely monitored by an electronic water level probe and recorded.

The coefficient of hydraulic conductivity can be calculated from the solution of the boundary value problem (B.V.P.) that governs the falling head test. The solution to this B.V.P. was first presented by Hvorslev (1935) and is derived in detail in Appendix E. The resulting formula for the calculation of K is also shown below:

$$K = \frac{r^2 \ln(L/R)}{2\ L\ To}$$

K = hydraulic conductivity
r = radius of standpipe
L = length of test section
R = radius of borehole
To= time factor

Computer program EQFHEAD was developed to reduce the time required to carry out the Hvorslev analysis and increase computational accuracy. The program is fully documented in Appendix F.

6.2.3 Results

The 1984 falling head test program was highly successful, defining hydraulic conductivities of all major rock units in the Main Zone. Testing also showed that blasting and surficial weathering have a significant effect on permeability. Table 6.2 summarizes the falling head test results.

Table 6.2 Results of Falling Head Permeability Tests

PIEZOMETER NUMBER	TEST SECTION FROM	TO (m)	ROCK TYPE	K (cm/s)
P5	10.43	13.50	gabbro	3.0×10^{-5}
P5	16.57	19.64	gabbro	1.2×10^{-7}
P5	19.64	22.71	gabbro	2.2×10^{-7}
P6	16.57	19.64	lapilli	6.9×10^{-5}
P6	19.64	22.71	lapilli	2.9×10^{-5}
P6	22.71	25.78	lapilli	4.3×10^{-5}
P7	16.57	19.64	gabbro	1.7×10^{-6}
P7	22.71	25.78	gabbro	9.6×10^{-4}
P8	19.64	22.71	gabbro	2.9×10^{-7}
P8	22.71	25.78	gabbro	2.2×10^{-6}
K1	7.36	10.31	lapilli	1.3×10^{-5}
K2	22.71	25.78	dust tuff	1.2×10^{-5}
K2	25.78	28.85	dust tuff	7.7×10^{-6}
K2	31.92	34.99	dust tuff	2.5×10^{-6}

Because the air trac is a percussion drill, a large amount of fine cuttings are generated during drilling. Compressed air is then circulated down the drill rod to flush the cuttings out of the hole through the small clearance between the drill rod and the wall of the drill hole. Some of the cuttings are forced into any open cracks and fissures in the drill hole. As a direct result, hydraulic conductivities measured in air trac holes are generally lower than the true value. Therefore, average values of K for each rock unit that were obtained from falling head tests in air trac holes have been multiplied by a factor of 5 to account for the artificial decrease in rock mass permeability. Permeability tests in hole K2 were not corrected because K2 is an old vertical diamond drill hole, and should not be clogged by cuttings to the same degree. The average, corrected values of hydraulic conductivity are listed in Table 6.3. They are the best available estimates of K, and should be used for all subsequent investigations.

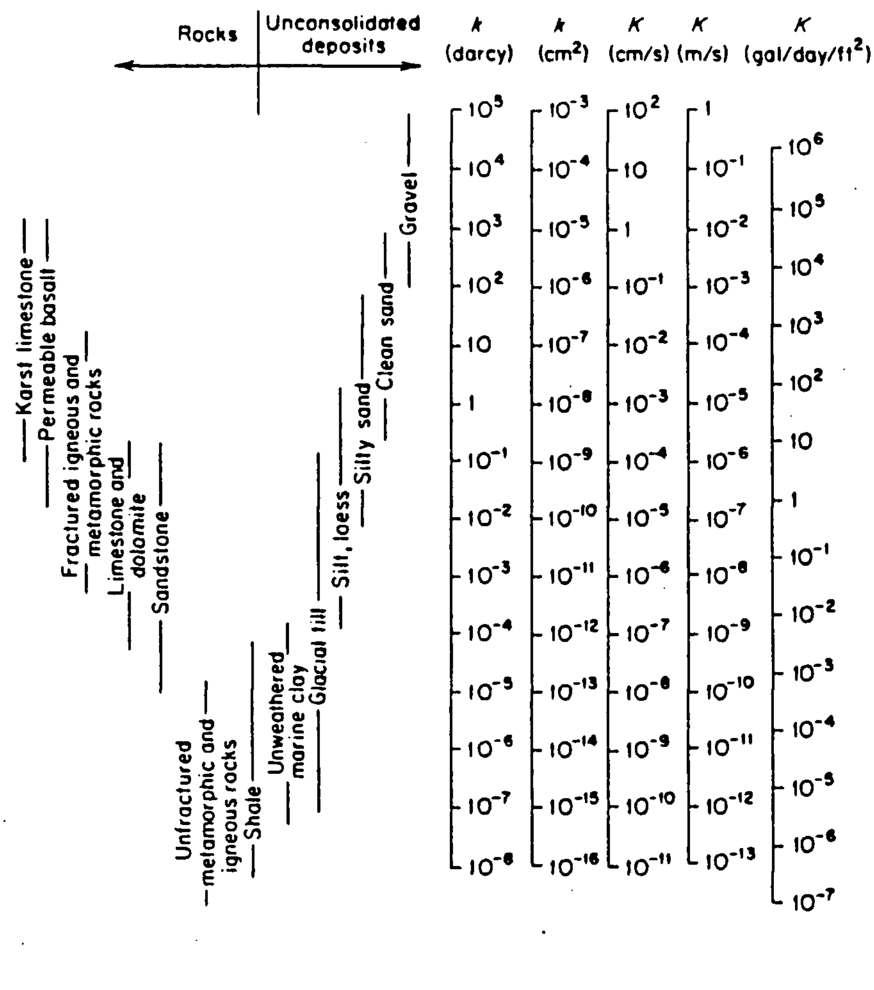
Table 6.3 Representative Values for Hydraulic Conductivity

ROCKTYPE	CONDITION	K (cm/s)
Gabbro	intact	2.0×10^{-6}
Gabbro	blasted or weathered	2.0×10^{-4}
Lapilli Tuff	intact	2.0×10^{-4}
Lapilli Tuff	blasted or weathered	1.0×10^{-3}
Dust Tuff	intact	7.0×10^{-6}

The hydraulic conductivities obtained in the 1984 testing correspond closely to results from earlier tests in the vicinity of the Main Zone (Golder Associates, 1983) and in the Southern Tail pit (Beaudoin, 1981). Results of the earlier tests are tabulated in Appendix H.3. Table 6.4 indicates the normal range of permeabilities that can be expected for specific rock types. The test results also correlate well with these quidelines.

The permeability testing program has confirmed that the gabbro is nearly impermeable while the volcanics have a moderately high K. Within 10 to 20 m of production blasts the hydraulic conductivity of all rock units increases, possibly by as much as two orders of magnitude. The increase can be directly attributed to fracturing of the rockmass and opening of healed or gouge filled joints that occurs close to the blasted area. Dewatering of the Main Zone pit should be possible as hydraulic conductivities in the volcanics are sufficiently high to allow flow towards the wells. A preliminary evaluation of dewatering potential in the Main Zone is presented in Section 6.6.

Table 6.4 Normal Range of Permeabilities in Soil and Rock (after Freeze, 1979)



6.3 PIEZOMETER MONITORING

Fourteen standpipe piezometers have been installed in the east half of the Main Zone pit to provide information on pore pressures in the pit walls. Piezometers P1 to P4, completed by Golder Associates in 1983, were monitored weekly during the spring and summer to determine seasonal fluctuations in pore pressures. Ten additional piezometers were installed in late August, 1984 in key areas of the pit. Appendix J summarizes existing piezometer information including: 1. method of installation, 2. location, and 3. monitoring records.

The highest water levels, within 6 m of surface, were observed in the south end of the pit. Water levels in the east wall, south of Bessemer Creek ranged from 10 to 20 m below surface. Above the Gabbro pit the water levels were relatively low, from 15 m below surface in P 10 to more than 27 m in P 14, as the hole remains dry. In summary, the water table generally follows topography, but is found deeper below suface from south to north. It is also found at shallower depth below surface as elevation is decreased in any given section, from 10 to 20 m below surface at the top of the pit to zero near the pit floor. There, seepage occurs so the water table must be at the surface.

Weekly monitoring of P1 to P4 has indicated that there is a strong seasonal fluctuation in pore pressure. Figure 6.2 summarizes the monitoring records in a graph of water level vs. time. The highest levels in each of the piezometers were observed in late May, then gradually decreased in June and July. By early August the levels were once again rising and have continued to increase slowly to date. The maximum seasonal fluctuation appears to be about 10 m. There does not appear to be a correlation between short term climatic events and

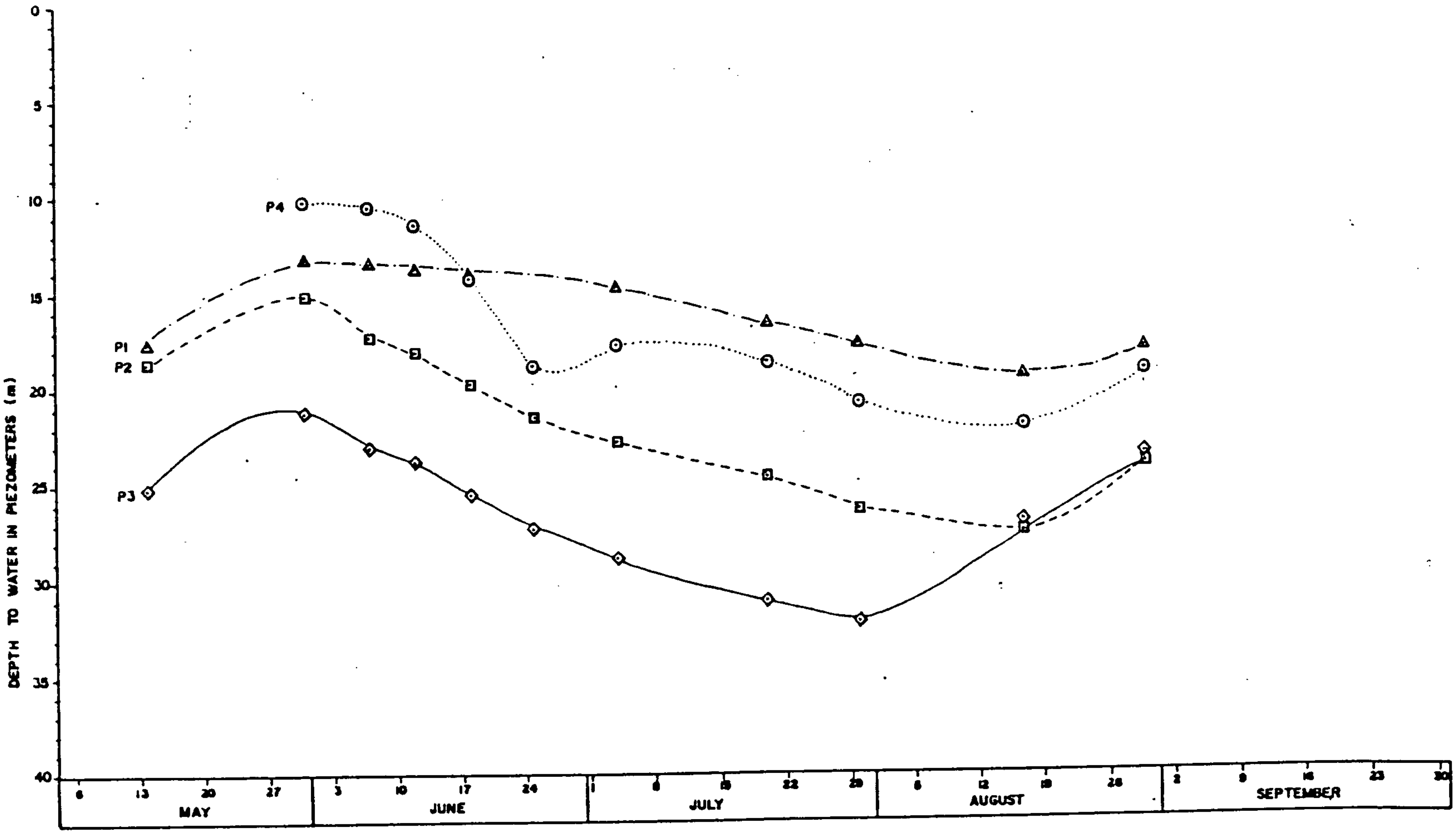


FIGURE 6-2
PIEZOMETER MONITORING

the water levels.

Piezometers P1 to P3 are nested in a single bore hole south of the Gabbro pit. The vertical component of the hydraulic gradient has generally been upward. Gradients as high as 0.15 m/m were observed between P1 at 148 m and P2 at 67 m. This response indicates that there may be significant amounts of flow of water toward the pit from depth, as well as the expected drainage of groundwater from the hills above the Main Zone.

6.4 INTERPRETATION OF MAIN ZONE HYDROLOGY

Groundwater flow patterns in the Main Zone are influenced by geology, topography and the presence of the open pit. The recharge area that drains toward the pit is outlined in Figure 6.3. Most of the surface runoff is diverted away from the pit, but groundwater flow is not impeded. Steady state groundwater inflow into the Main Zone pit is estimated at 1.27×10^{-2} m^3/s (Golder Associates, 1983). A flownet (Figure 6.4) shows the likely flow pattern in vertical section. The majority of groundwater that drains into the pit originates east of the Main Zone. As much as 30% of the water may be flowing into the pit from below the pit floor under an upward hydraulic gradient. This upward flow has been observed at depth in the southern portion of the Main Zone in piezometers P1 to P3 and must be considered in the design of the pit dewatering system.

The permeability testing program has confirmed that the gabbro intrusive complex east of the Main Zone is nearly impermeable. It acts as a barrier to groundwater recharge. The majority of groundwater inflow must occur through the volcanic rocks, found in the south end of the Main Zone and in the east wall of the Gabbro Pit. Whether a significant amount of water flows through the highly fractured rocks in the vicinity of Bessemer Creek remains to be established. Figure 6.5, a plan of the Main Zone pit, shows the location of the principal flowpaths discharging into the pit.

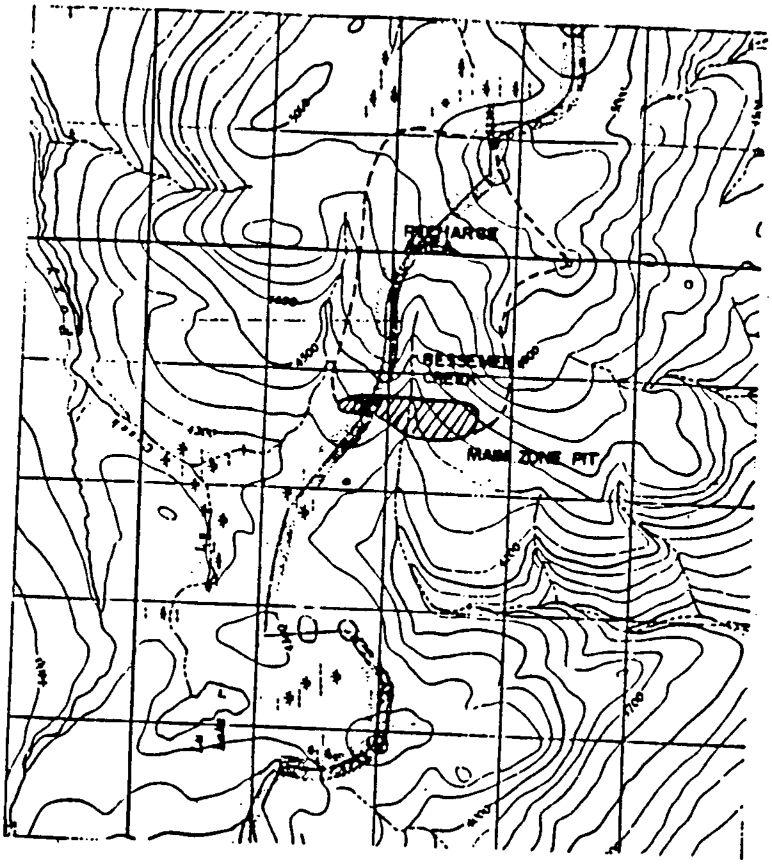


FIGURE 6-3
WATERSHED RECHARGING
MAIN ZONE PIT

SCALE 1:40,000

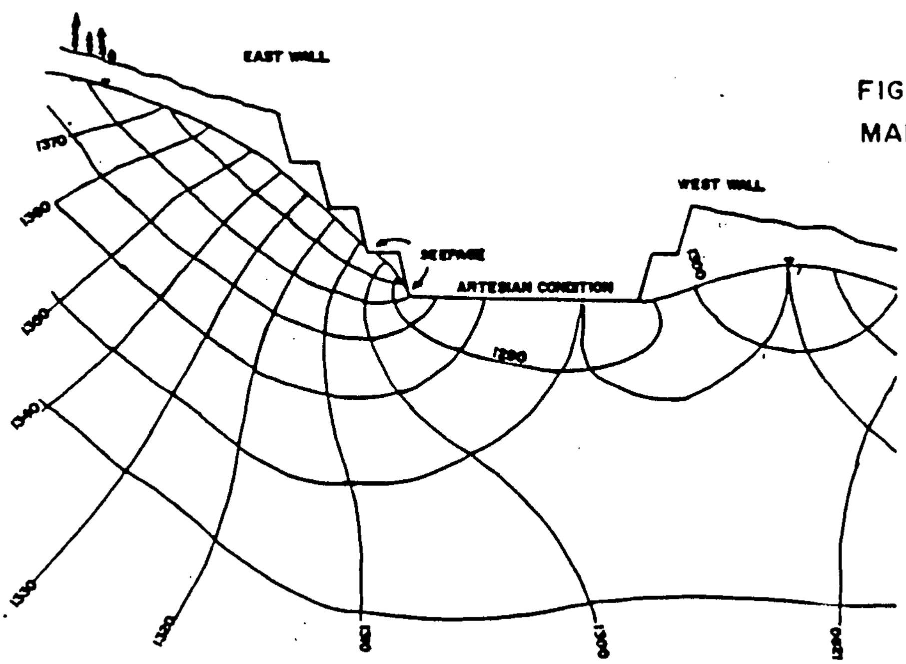


FIGURE 6-4
MAIN ZONE FLOWNET

NOTE: NUMBERED CONTOURS ARE EQUIPOTENTIALS

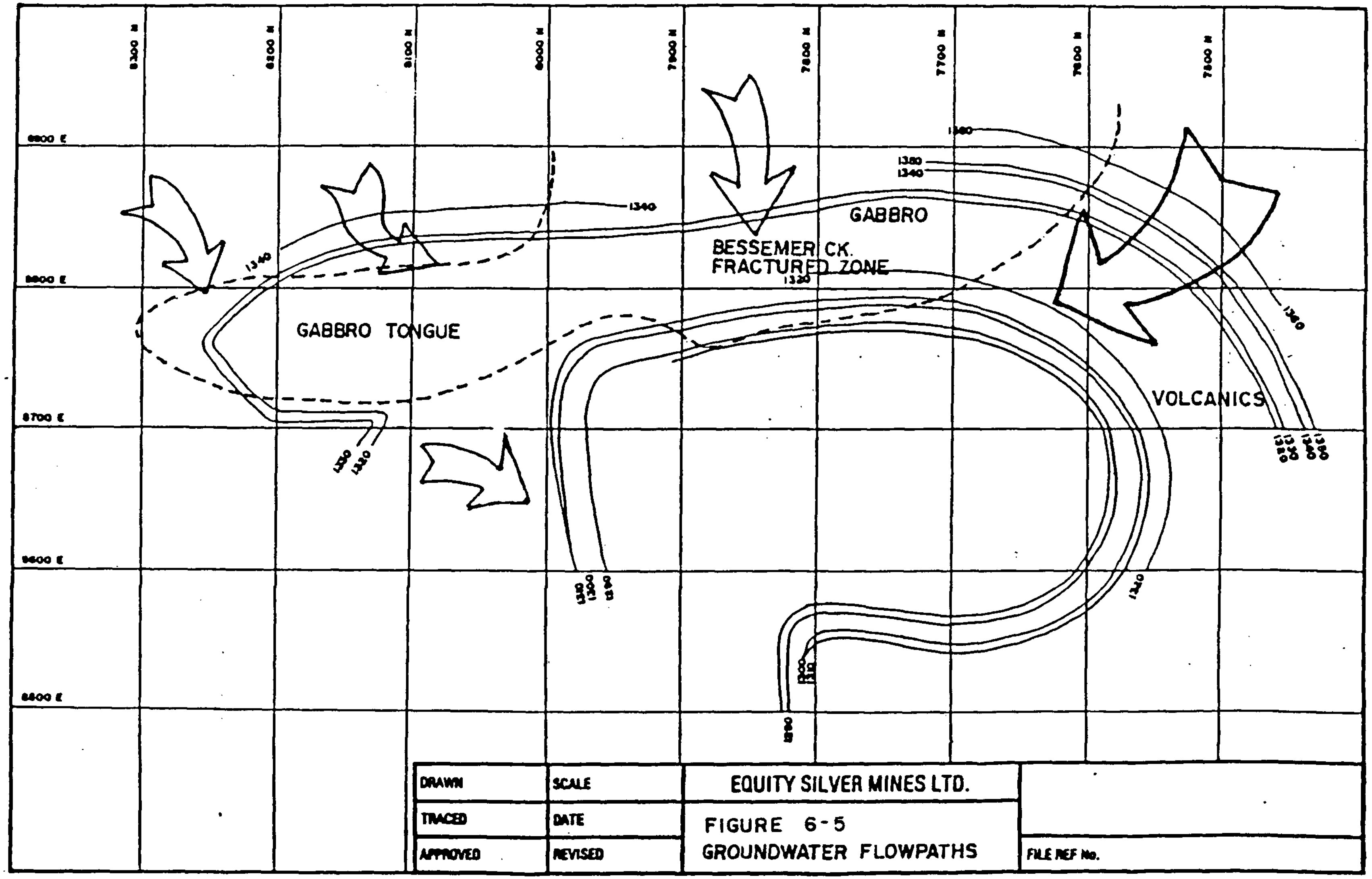


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FIGURE 6-5
GROUNDWATER FLOWPATHS

The gabbro tongue is expected to have some influence on rates of inflow of groundwater into the pit and on pore pressures in the east ultimate pit wall in the area of the existing gabbro pit. Because of the low permeability, the gabbro tongue will act as a dam, preventing drainage of the more pervious pyroclastics in the "notch" between the tongue and the main gabbro pluton. However, because the pyroclastics within the notch are surrounded on three sides by low permeability gabbro, recharge into the area will be slow in the short term, provided precautions are taken to ensure that all surface runoff is prevented from seeping into the area (i.e. no seepage is occuring from the Bessemer Creek diversion). Water that does enter the notch can likely drain sufficiently fast through the gabbro tongue or southward around it. Piezometers located in the pyroclastics within the notch indicate that adequate drainage is occuring as pressures are quite low, some piezometers remain dry.

When mining in the gabbro pit progresses to greater depth the gradient in the notch will become fairly steep toward the south and larger quantities of water can be expected to flow toward the pit from the vast area of pyroclastics situated north of the tongue. As a precaution, piezometers should be completed in the pyroclastics behind the dam as mining progresses to depth. If these piezometers are indicating high pressures then horizontal drains should be installed to bleed them off. Figure 6.6 is an illustration of the anticipated hydrologic problem.

Visible seepage on south walls of the interior pit and lack of such seeps in the gabbro confirms that much more water is flowing in the volcanics. Most of the seeps occur adjacent to dykes, faults, and shears. The gouge zones associated with these major discontinuities have a high clay content; therefore, they act as impervious membranes. Because water cannot penetrate these planar structures it is forced to flow along them. A seep is created wherever these structures daylight. The rock directly adjacent to the discontinuity may also be more fractured by tectonic activity, providing a flow path of less resistance. The influence of these major structures on pit dewatering must be considered in the design. The structures may isolate groundwater into structurally bounded blocks that cannot be dewatered unless the pumping wells are located directly within the blocks. Such a situation was recently experienced at Gibraltar Mines (Carpenter, 1980).

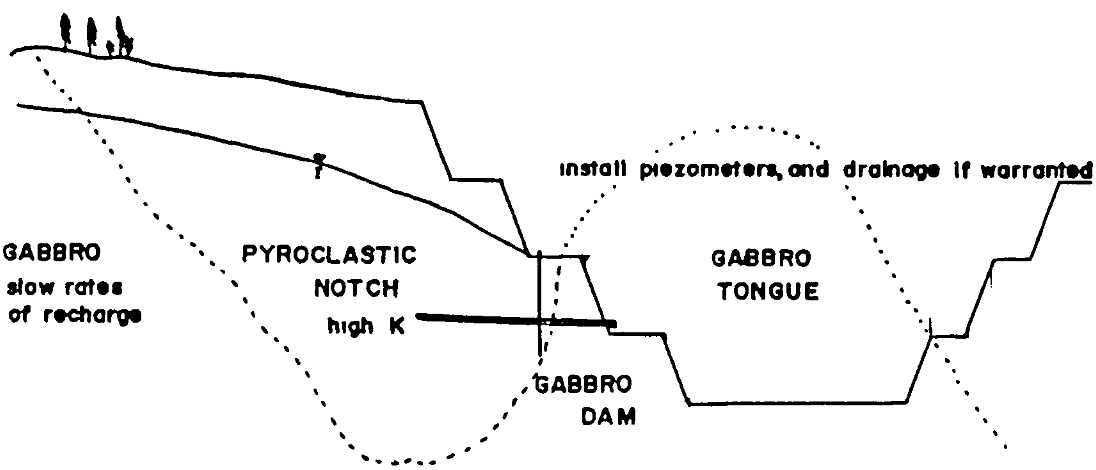


FIGURE 6-6

6.5 SURFACE RUNOFF

Two diversion ditches capture the majority of surface run-off flowing toward the Main Zone pit from the eastern hillside. Bessemer Creek is diverted to the south along a ditch that also intercepts all run-off south of the creek bed. A second ditch, starting at the base of the Bessemer Creek diversion dam and draining north, is currently under construction. When completed, this ditch will divert any flows north of the creek bed. To complete this ditch, obstructions blasted this summer must be mucked out and areas where bedrock is exposed should be lined with compacted glacial till to prevent seepage.

During times of heavy rainfall a flow of approximately 50 l/min develops down the old Bessemer Creek bed downstream of the diversion dam. The water discharges into the pit at 1320 m elevation. The flow originates as seepage through the diversion dam and as groundwater discharge. Near the diversion dam, most of the seepage water is flowing below ground so it is not intercepted by the north diversion ditch. To prevent the water from seeping into the pit a catchment dam and diversion system will be required near the pit crest. A sketch of the problem areas and suggested improvements is presented in Figure 6.7.

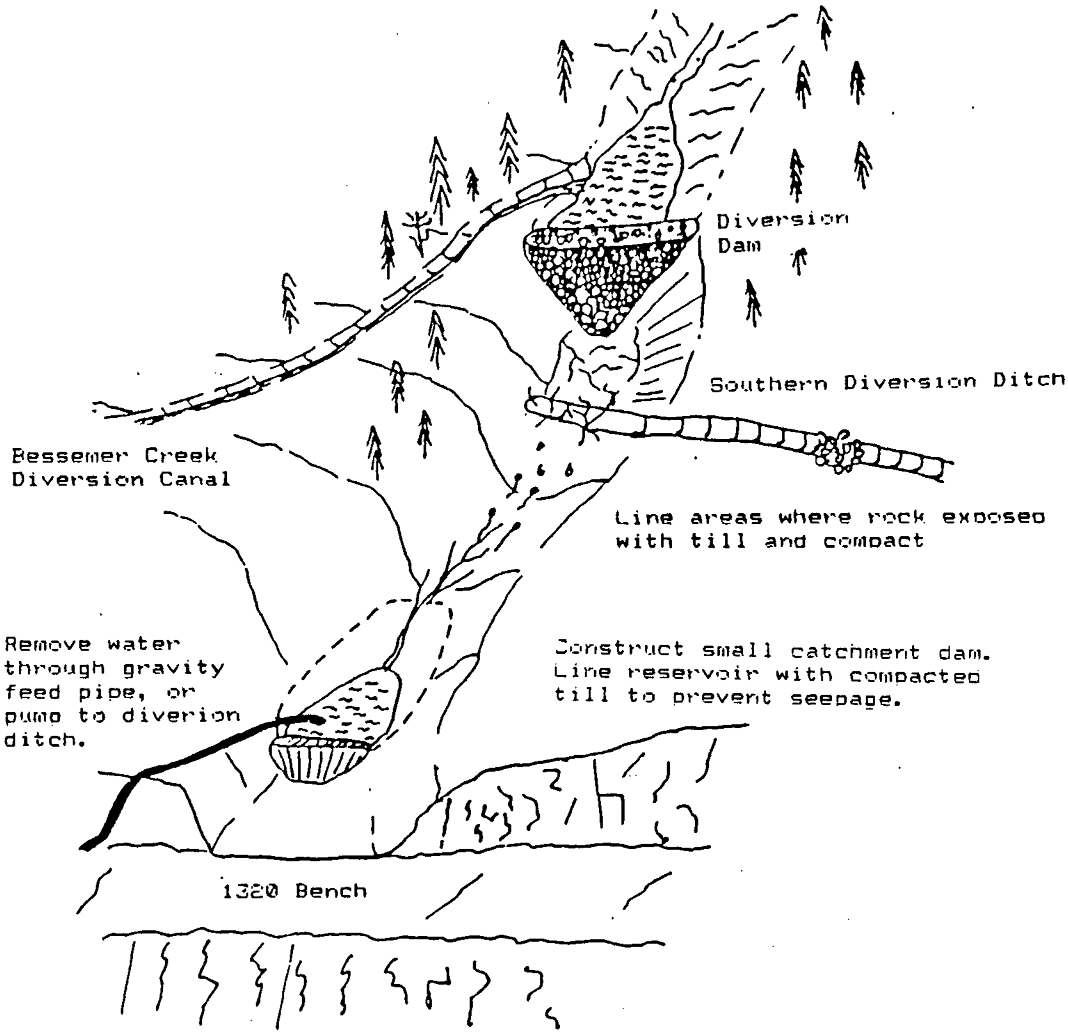


FIGURE 6-7
RUN-OFF CONTROL

6.6 CONSIDERATIONS FOR PIT DEWATERING

6.6.1 Background

The falling head permeability testing program has established the magnitude of the hydraulic conductivity coefficient for each geologic unit. Experience suggests that the volcanic rocks can be dewatered effectively as their K is fairly high. The gabbro, especially when intact, will require numerous closely spaced wells if dewatering is to be successful. The purpose of this section is to explore in greater detail how the controlling hydrologic parameters (hydraulic conductivity, aquifer thickness, specific yield, and pumping rate) influence the performance of the dewatering systems. By studying the shape of the drawdown cone about a pumping well and the rate of drawdown as one parameter is varied while the others are held constant the most influential parameters can be identified. Further work can then focus on those important parameters while the less influential parameters can be estimated in the analysis without seriously affecting the validity of the results.

The second part of this section presents the "most likely" model of drawdown behaviour that can be expected in the Main Zone if in-pit wells are selected as the dewatering system. This simulation provides a rough idea of the pumping rates, well depths, and spacings that will be required to attain the desired goal of reducing the inflows into drill holes to levels where most holes can be loaded with ANFO, provided the holes are first pumped dry, then lined with a waterproof membrane.

A computer program was developed to simulate the behaviour of a single pumping well dewatering an unconfined aquifer. The program is based on the Theis Solution, an analytical solution to the single pumping well boundary value problem. Theoretical concepts of the Theis Solution and the procedures used to evaluate the mathematically complex equations are presented in detail in Appendix I.

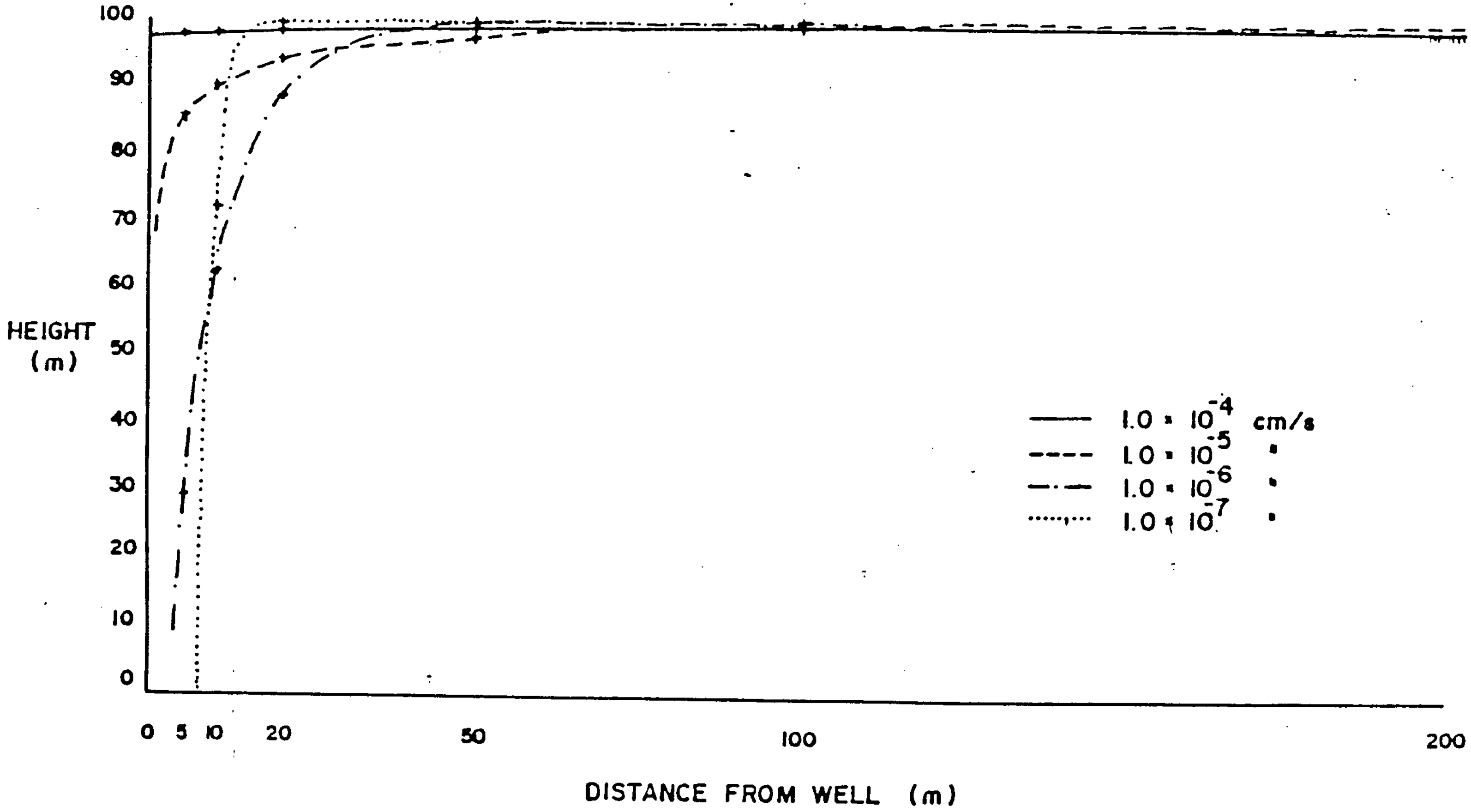
The third part of this section briefly discusses the various systems that could be used for dewatering, including the advantages and faults of each.

6.6.2 Sensitivity Study

The sensitivity study consisted of four parts. During each part one parameter was varied while the remaining three were held constant at reasonable values. Three or four different magnitudes spanning the expected range of the parameter were entered into the simulation. Results for each simulation are tabulated in Appendix K. To aid in interpretation, one specific time was selected during each simulation and the drawdown curves for each value of the variable parameter were drawn. The spread of the curves is a direct indication of the sensitivity of the system to that particular variable.

6.6.2.1 Hydraulic Conductivity

Hydraulic conductivity is a very important parameter because if K is too low the rock cannot be successfully dewatered, no matter what pumping rate is used. This is clearly evident in Figure 6.8. The two low K simulations ($K=1x10^{-6}$ & $1x10^{-8}$ cm/s) have very tight cones about the well while the high K cones are very extensive but shallow. This behaviour can be understood by considering hydraulic conductivity as a quantitative measure of resistance to flow. If K is high water flows easily so it can flow from great distances under a low gradient; hence, the cone is broad but shallow. When K is low there is a lot



TIME = 200.00 days
THICKNESS = 50.00 m
SPECIFIC YIELD = 0.05
PUMPING RATE = 10.00 l/min

FIGURE 6-8

INFLUENCE OF HYDRAULIC CONDUCTIVITY

of resistance to flow. Water has a hard time flowing any distance. Any water close to the well is removed first and a very steep cone is established. The hydraulic gradient toward the well is very high. Figure 6.8 confirms that rocks with K smaller than $1x10^{-6}$ cm/s cannot be dewatered easily because even at the very low pumping rate of 10 l/min the well is quickly pumped dry and an equilibrium condition is developed with a radius of influence of only 10 m. Although the depth of the high K drawdown cone in Figure 6.8 appears too shallow to justify dewatering, it must be remembered that if the pumping rate is increased the cone will become much deeper.

6.6.2.2 Aquifer Thickness

Aquifer thickness can affect the size of the drawdown cone considerably if it is allowed to vary over a large range. However, at Equity, it is very likely that the "aquifer" consists of the artificially fractured rocks that extend from surface to a depth of 10 to 20 m. Because there is not much difference in the size of the drawdown cone over this range of t, thickness is not a highly influential variable. In the Theis analysis, the solution is developed in terms of the parameter "transmissivity", which is the product of hydraulic conductivity and thickness. When one considers that the range in K is three or four orders of magnitude while thickness most likely varies by a factor of five, it becomes obvious that it is much more important to get a valid value for the parameter K then to accurately define thickness.

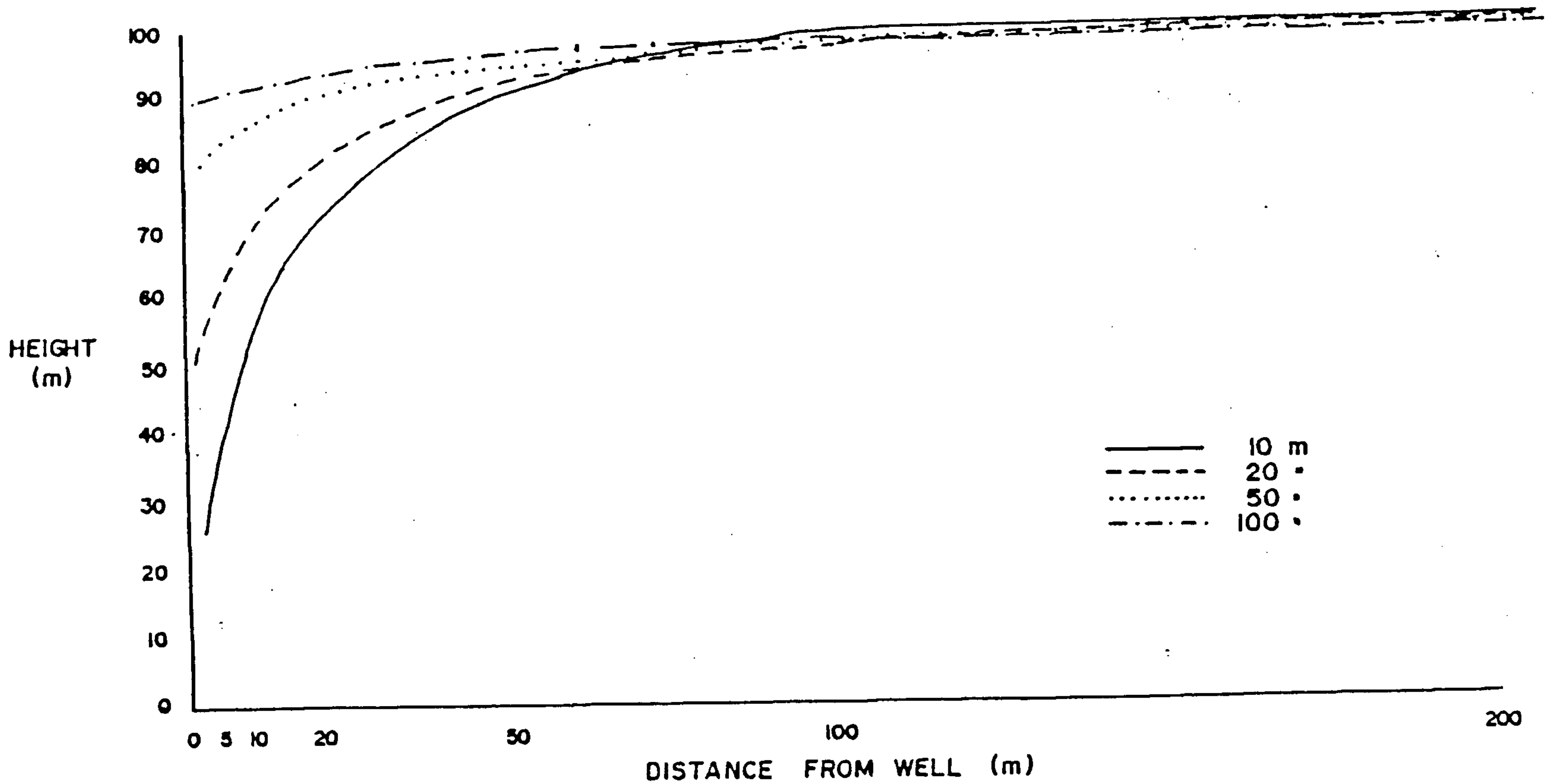


FIGURE 6-9
INFLUENCE OF AQUIFER THICKNESS

TIME =1000.00 days
PUMPING RATE = 10.00 l/min
HYDRAUIC COND. = 1.00 x 10^{-5} cm/s
SPECIFIC YIELD = 0.05

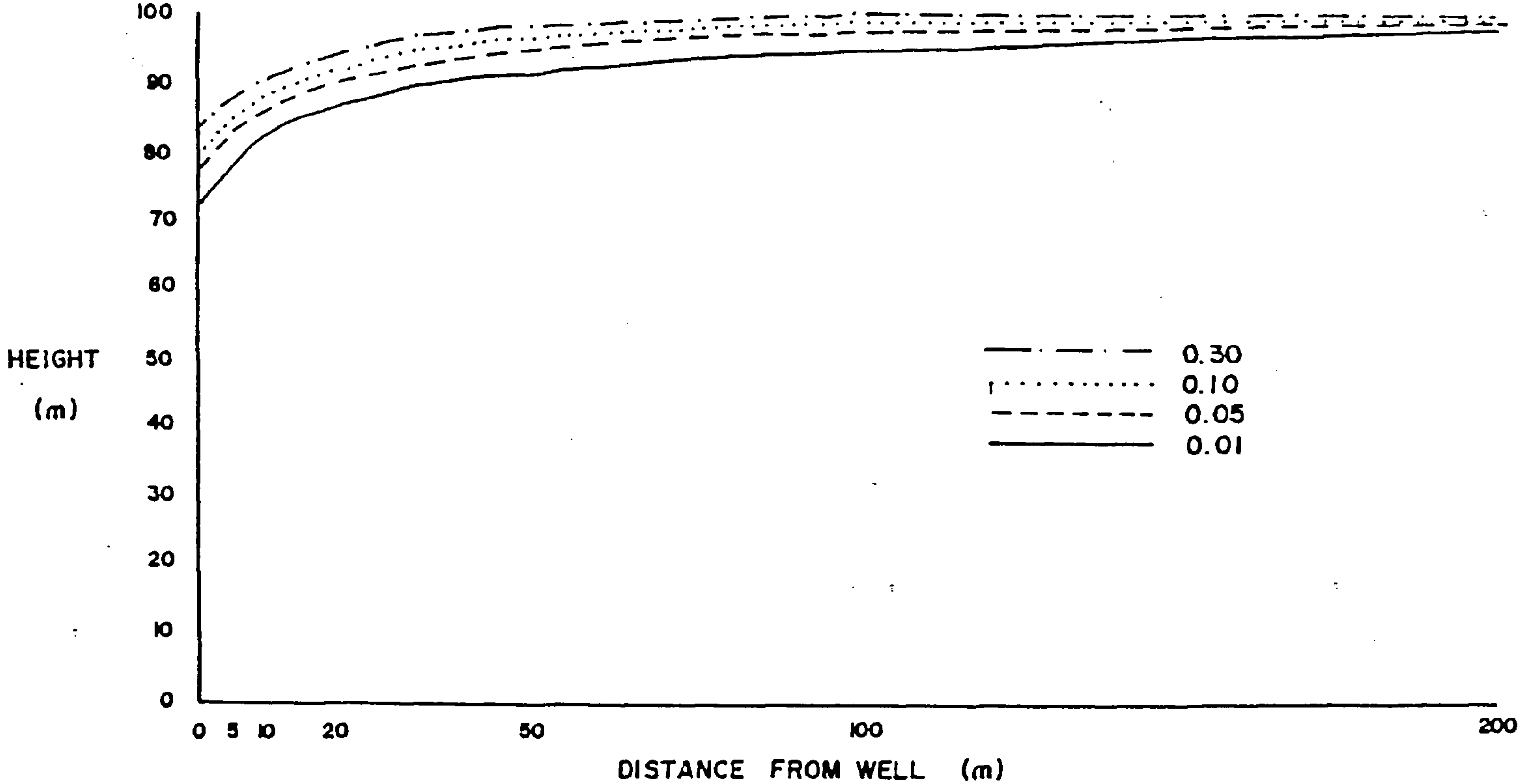
6.6.2.3 Specific Yield

Specific yield is synonymous to storativity in the case of a confined aquifer. The parameter specifies the volume of water that will be released from storage over a unit area of aquifer for a unit decline in the water table. Hence, the parameter is dimmensionless. In practical terms, specific yield is really an indicator of porosity, because when the water table is lowered most water that drains comes directly from the pores, while much smaller amounts are generated by expansion of the water due to reduced pressure and expansion of the rock into the pore space. Therefore, specific yield has a fairly narrow range, from 0.01 to 0.30. In the fractured rocks within the fragmented zone the porosity is expected to be about 0.05.

Specific yield is the least sensitive parameter. The curves from the four simulations spanning the expected range of SY all fit within a very narrow envelope. Therefore, it is valid to select a reasonable estimate of SY for the simulations and focus on accurate definition of hydraulic conductivity K.

6.6.2.4 Pumping Rate

Unlike the hydrologic variables, pumping rate is a parameter that can be controlled. Figure 6.11 indicates that the size of the drawdown cone and rate of dewatering can be controlled by the rate of pumping. In general, the larger the pumping rate, the more extensive and deeper will be the drawdown cone. However, it can also be seen in Figure 6.11 that if an excessively large pumping rate is used the well will quickly be pumped dry. This type of problem was encountered at Gibraltar Mines



TIME	=	1000.00 days
THICKNESS	=	50.00 m
HYDRAUIC COND.	=	1.00×10^{-5} cm/s
PUMPING RATE	=	10.00 l/min

FIGURE 6-10

INFLUENCE OF SPECIFIC YIELD

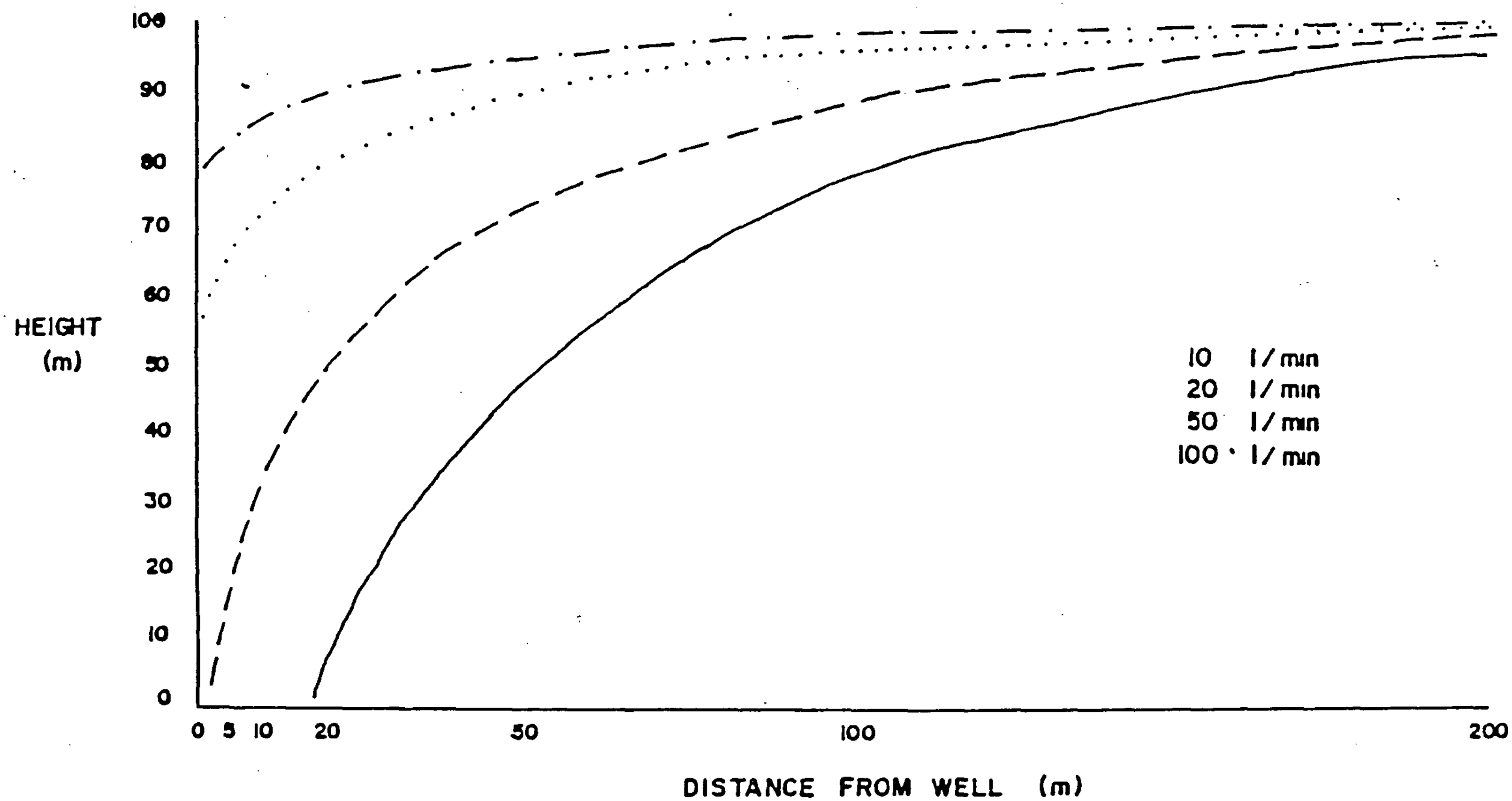


FIGURE 6-11

INFLUENCE OF PUMPING RATE

```
TIME              =1000.00 days
THICKNESS         =   50.00 m
HYDRAUIC COND.    =    1.00 x 10  cm/s
SPECIFIC YIELD    =    0.05
```

where it was overcome by installing limit switches in the wells so the pumps automatically stopped when the water level in the well reached a specified cut-off point.

6.6.2.5 Most Likely Simulation

A hydraulic conductivity value of 1.0×10^{-4} cm/s was selected as the representative K because it is intermediate between the fractured K for lapilli tuffs as measured near the surface and the intact K value for the same rock unit that was averaged over several measurements at depth.

The aquifer thickness was assumed to be 30 m. This estimate is conservative because a thinner aquifer has a deeper drawdown cone so more pumping will be required to get the desired water table drawdown with the thicker aquifer.

A specific yield of 0.05 was chosen as representative of the fractured rock.

The only variable that still required selection was the pumping rate. After several trials it was discovered that for the fairly high K value selected a very large pumping rate would have to be used to achieve a reasonably sized drawdown cone. A pumping rate of 100 l/min was input into the program.

With the above parameters a desirable drawdown was achieved after about 200 days. Figure 6.12 illustrates the simulated drawdown after 500 days with all variables set to the above mentioned values. By using the principle of superposition that states drawdowns from two individual wells can simply be added together to obtain the resultant, a composite drawdown curve was constructed for two wells spaced 100 m apart. The net drawdown exceeds 30 m everywhere between the two pumping wells so it is likely that a lower pumping rate of 50 to 75 l/min could adequately dewater the fractured aquifer.

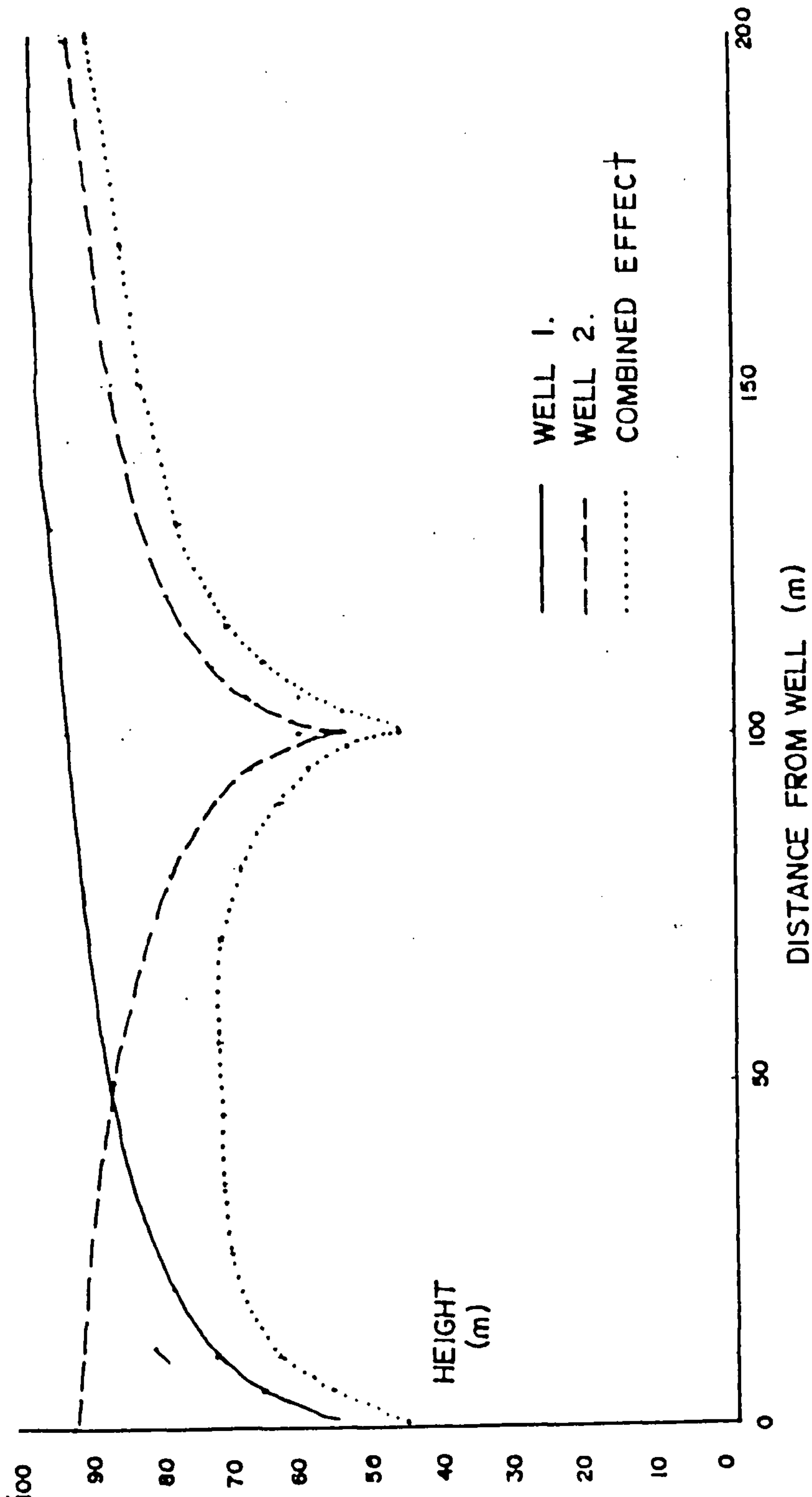


FIGURE 6-12
EXPECTED DRAWDOWN CONDITION

TIME = 500.00 days
THICKNESS = 30.00 m
HYDRAUIC COND. = 1.00 x 10 cm/s
SPECIFIC YIELD = 0.05

6.7 DEWATERING SYSTEMS

In this section six possible dewatering systems are presented and the advantages and faults of each system are listed in point form.

The methods are:
1. existing sump method
2. modified sump trench
3. pit perimeter wells
4. in-pit well point system
5. horizontal drains
6. self draining wells

6.7.1 Existing Sump Method

The sump method consists of a single submersible pump that is placed in a sump excavated several meters below the pit floor. Water is pumped through a thick pipe to the tailings area. A second, and and perhaps third sump could be excavated so a drainage channel does not have to be maintained on the pit floor to collect water and direct it to the sump.

Advantages	Disadvantages
1. Relatively inexpensive.	1. Water level remains near surface.
2. Easy to maintain.	2. Slurry explosives often required.
3. Very mobile.	3. pore pressures in wall remain high.
4. Easy to install.	4. In production areas, gets in way.
5. Accessible.	
6. Works in any rock condition.	

6.7.2 Modified Sump Trench

A simple alternative to the sump method that could decrease the amount of slurry explosives used would be to always start the sinking cut of a new bench in an area of high water inflow, e.g. the south east side. In this way the principal flow paths of water into the pit would be intercepted. The pit floor would progressively become dryer as water that was present in the rock would drain off.

Advantages	Disadvantages
1. Relatively inexpensive.	1. Not thoroughly tested.
2. Easy to maintain.	2. Pore pressures in wall remain high.
3. Works in any rock condition.	3. Affects mining sequence on bench.
4. Accessible.	4. Water level will not be pulled down sufficiently to make all holes dry.

6.7.3 Pit Perimeter Wells

Deep pumping wells could be installed around the pit perimeter. The wells would have downhole submersible pumps capable of pumping under very high pressure head.

Advantages	Disadvantages
1. Permanent installation.	1. Expensive to install.
2. Minimal maintenance required.	2. Wells may have to be spaced. fairly closely (e.g. 20-50 m). to attain sufficient drawdowns.
3. All equipment out of the way.	3. Very large cone would have to be dewatered to pull down W.T.
4. Water would be drawn far away from pit wall, stability would be increased.	4. May not work in gabbro as permeability simply too low.
5. Water table in bottom of pit could be drawn down adequately if K sufficiently high.	5. Specialized drill required for installation of deep wells.
6. Slurry explosives would be required in far fewer holes.	6. Large energy consumption expense.

6.7.4 In-Pit Well Point System

The in-pit well point dewatering system is probably the most effective way of drawing the water table down to a sufficient depth to allow for the use of ANFO explosives in most drill holes. Nine inch diameter holes could be drilled to a depth of 30 m with the production drills. Submersible pumps could then be lowered into the holes. The wells should be located along ramps so that they would remain accessible for maintenance and eventual relocation once mining progressed to the level of the well points.

Advantages	Disadvantages
1. Relatively small area has to be dewatered. Less water produced.	1. Pumps have to be relocated in new wells as mining progresses.
2. Installation and service can be performed by Equity staff and equipment.	2. Water lines in pit may get in way or get damaged.
3. Beneficial effect on wall stability, but less than peripheral pumps.	3. Pumps will be high cost items.
4. System more flexible than peripheral pumps.	4. Fairly labour intesive method.
5. Less wells required for equal coverage.	

6.7.5 Horizontal Drains

Horizontal drains are without doubt the single most effective and efficient method of groundwater control for wall stability. When properly installed, the drains reduce the pore pressures in the pit walls sufficiently to have a very dramatic increase on stability. Installation of the drains is expensive as a specialized drill rig is required to obtain a sufficient depth of penetration (the Aardvark system from Seattle has an excellent track record and should be considered during the contract bid process). However, because down hole pumps are not required, the initial investment is considerably lower than equivalent drainage with vertical pumping wells. In the long term, horizontal drains become even more lucrative because operating costs are very low as the only pumping required is the removal of water from a central collection sump.

To increase the initial effectiveness of the drainage system in rocks of low permeability, a seal can be developed in the outer 5-10 m and the entire drain placed under vacuum.

Advantages

1. Relatively inexpensive to install.
2. Improves wall stability by reducing pore pressures.
3. Very economic in long term as pumping and maintenance costs nominal.

Disadvantages

1. Does not draw water table below pit floor. Requirements for slurry based explosives remain.
2. Specialized contractor required for installation.
3. Water must be collected at face, collection system in way of operations.

6.7.6 Gravity Well Method

The gravity well method posseses many of the favourable attributes of both the in-pit wells and the horizontal drains. The method consists of vertical, 9" diameter holes, drilled to maximum depth with the 40-R production drills. The holes are then backfilled with a high permeability coarse sand to keep them from caving. Horizontal drains are drilled precisely to intercept the base of the vertical wells, providing a flow path by which water can escape to surface. The drains must be lined, and only the inner two thirds of the casing perforated. Experience at Highland Uranium Mines in Wyoming has shown that approximately 50% of the drains successfully intercept the vertical holes. To ensure an adequate flow path between the wells and drains in all holes, a small explosive charge (5-10 kg) can be detonated to fracture the rock at the site of intersection.

Because a much larger sink of atmospheric pressure is introduced well behind the pit wall then would be the case with horizontal drains, dewatering will be much more rapid. This is highly advantageous at Equity, where hydraulic conductivities are quite low, especially in the intact rock at depth.

As with the horizontal drains, operating costs of this system are again very low because of nominal pumping and maintenance costs.

Advantages

1. Drainage by gravity, no pumps in wells.
2. Larger area of influence than with conventional horizontal drains.
3. Major reduction in pore pressures leads to increased wall stability.
4. Most work can be carried out by Equity staff and equipment.

Disadvantages

1. Does not fully dewater pit.
2. Requires specialized equipment for drilling of drains.
3. High degree of precision required for successful installation.
4. Water table not pulled below pit floor.
5. Collection system in way of pit operations.

6.7.7 System Evaluation

Of the six systems introduced in this report in pit wells and gravity drains have the greatest potential for improving the groundwater situation in the Main Zone and should be studied in further detail.

The pit sump methods have no influence on wall stability and very little influence on the inflow of water into blast holes; therefore, increasing problems with blasting and wall stability can be expected as mining progresses to depth if these methods of drainage are selected.

Because of the relatively low permeability of the intact rock in the Main Zone deep wells would have to be spaced very closely together to achieve the desired rate of drawdown. Even then, the drawdown cone would be quite steep and narrow so very little drainage would occur from the pit floor.

In-pit wells will drawdown the water table on the pit floor to allow for increased use of ANFO. However, pore pressures in the pit walls will not be reduced significantly if pumping wells are located only on the bottom of the pit. Therefore, groundwater will continue to have a strong destabilizing influence on any potential failures.

The gravity drainage system would reduce pore pressures in the pit walls to a favourable level at minimum expense. But this system does not have the capacity to pull the water table down below the pit floor to increase the number of dry blast holes.

The optimum drainage system in the Main Zone should have the capacity to achieve both a reduction in pore pressure in the walls and drawdown the water table on the pit floor. With carefully planned drainage design this goal can be achieved. The recommended system would consist of 30 m deep wells located on the pit floor. These would be pumped to draw down the water table prior to drilling of production blastholes. Then, as mining progressed down another 30 m, a new set of wells would again be completed on the pit floor. At the same time, the old wells could be intercepted by horizontal drains to form gravity drains. These would continue to drain the pit walls and maintain pore pressures at very favourable levels. The proposed WIP/GraD system (Wells In Pit / GRAvity Drainage) is illustrated in Figure 6.13.

Note that if 20 m high double benches are used in the area to be dewatered then the system will have to be modified to maintain access to both the top of the well (required during pumping stage) and to the horizontal drains (for maintenance of water collection system). The options are: 1) intercept wells at 20 m depth from every double bench, 2) drill horizontal drain inclined at 12° from 40 m below the well collar, and 3) drill 40 m deep well and intercept with horizontal drain. Option 3 is the optimum technical solution because it drains the largest area of the pit wall with a minimum number of wells. However, because such deep holes may be beyond the capacity of the 40-R drill option 3 may not be operationally feasible. In that case, option 1 would be the next best practical alternative.

It is recommended that the WIP/GraD system be evaluated by the mine engineering department to establish whether it will satisfy all operational and economic requirements. If the system passes the feasibility evaluation a trial dewatering program should be initiated to determine whether the expected level of performance can be attained in practice.

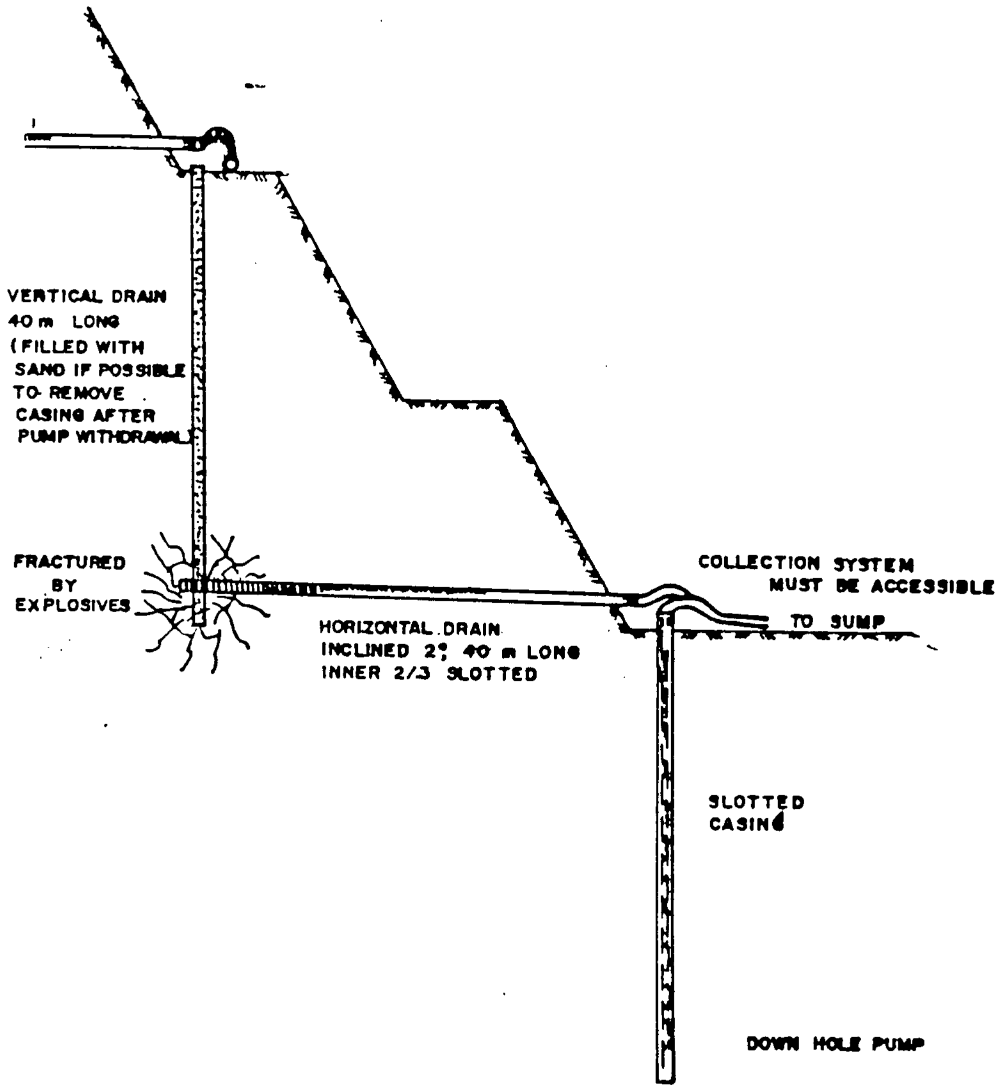


FIGURE 6-13 WIP/GraD SYSTEM OF PIT DEWATERING

6.8 CONSIDERATIONS FOR WALL STABILITY

Groundwater destabilizes open pit walls through several mechanisms. By far the most important is the reduction in shear strength that is associated with increasing pore pressure. Shear strength on a discontinuity is governed by the equation:

$$S = c + (\sigma - u) * \tan \phi$$

S = shear strength
c = cohesion
u = pore pressure
ϕ = friction angle
σ = total normal stress

In analogy, it becomes more difficult to slide a book on a table when it is firmly pressed down then when no weight is placed on it. Pore pressure acts as the lifting or buoyancy force, reducing the normal effective stress.

Water in a tension crack on the uphill side of a loose planar block or wedge can also induce a large destabilizing force out of the hill side. The force is equal in magnitude to the average hydrostatic pressure in the crack times the submerged surface area of the discontinuity.

Figure 6.14 is a scale drawing of Equity´s east wall that illustrates the position of the water table as observed in three piezometers located on the section. The magnitudes and directions of the destabilizing pressures that act on the potential failure block are also indicated.

Seepage force also induces a destabilizing force in the direction of flow. The magnitude of this force is:

$$F = i * L * A$$

where:

F = seepage force
i = hydraulic gradient
L = length of flow path
A = cross sectional area of failure wedge

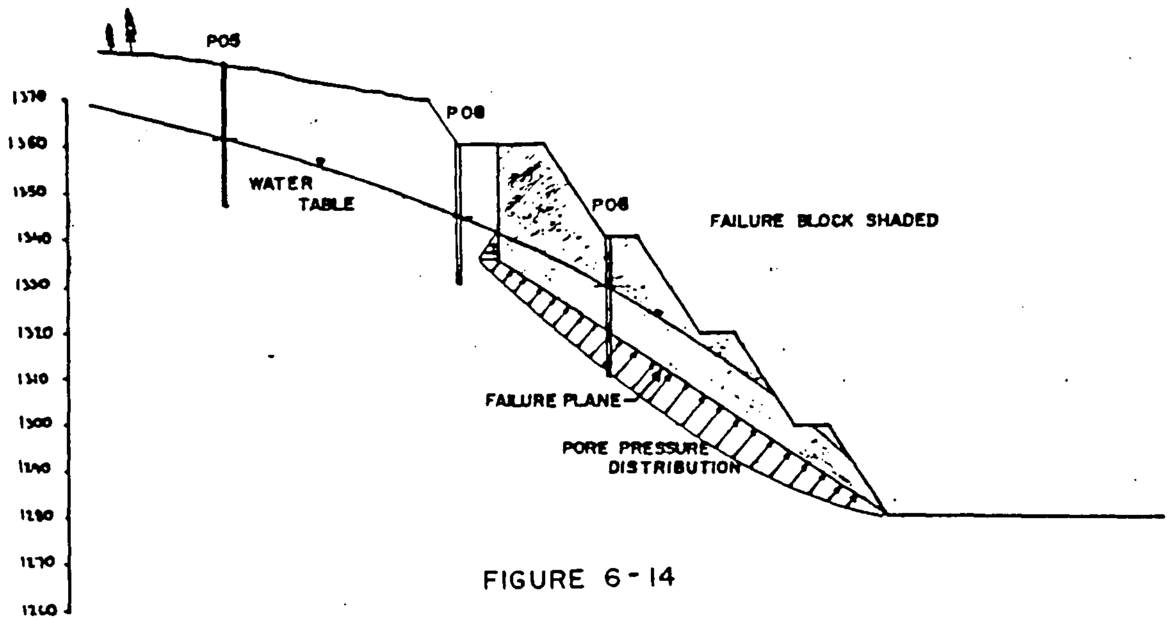


FIGURE 6-14

HYPOTHETICAL PLANE FAILURE ILLUSTRATES INFLUENCE OF GROUNDWATER ON SLOPE STABILITY

Figure 6.4, the flownet of the east wall, suggests that the gradients are not excessive except at the very bottom of the wall so seepage forces are likely less important than the shear strength reduction discussed above.

Finally, the presence of groundwater can destabilize smaller blocks on surface by freeze - thaw wedging action or washing out of gouge or cement out of joints, effectively loosening the blocks and eventually triggering a failure.

Reduction of the pore pressures by some form of drainage is the only way of reducing the destabilizing effect of groundwater. The most successful drainage technique today is the use of horizontal drains. Other forms of dewatering such as the deep well dewatering scheme or drainage adits will also improve stability because the pore pressures in the pit walls will be reduced as the water table is lowered.

Any surficial water should be directed away from the walls because it is primarly this source of water that quickly fills tension cracks and triggers failures.

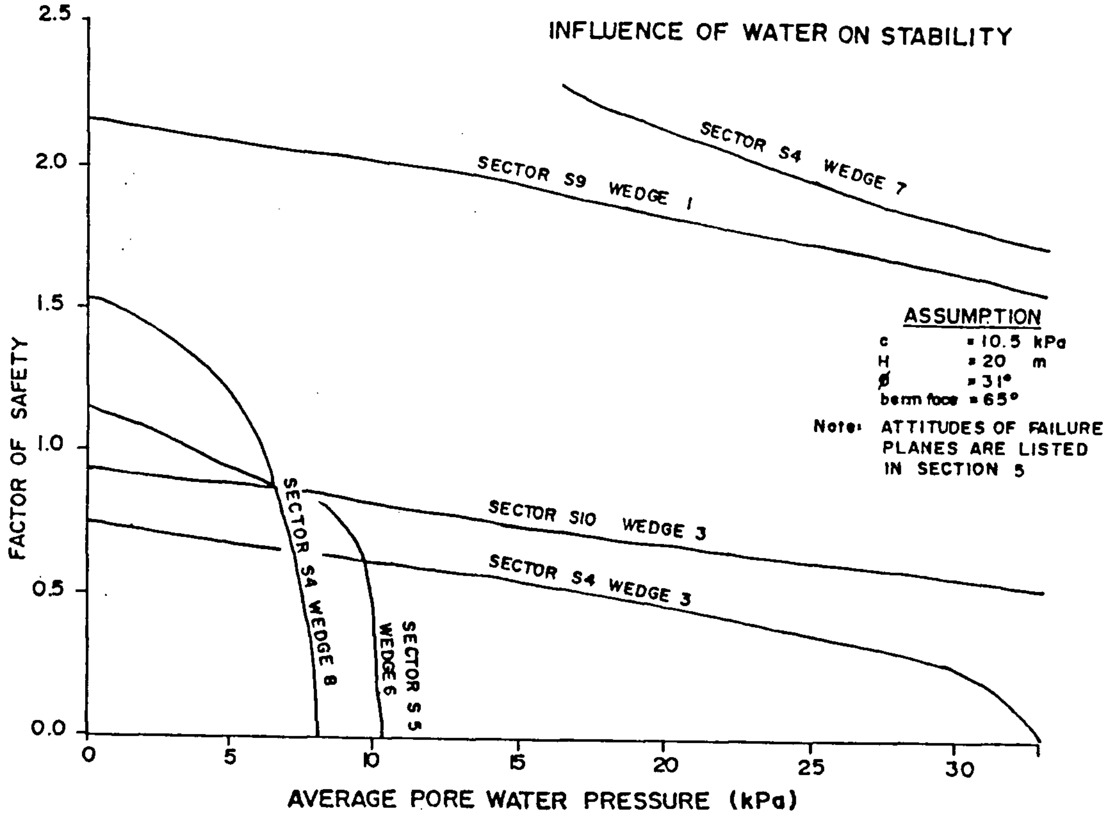
The most severe groundwater induced destabilizing forces are expected in the east and south walls of the Main Zone pit because most of the water that flows into the pit is believed to originate in the hills east of the pit. These two walls will be continuously recharged from the hillside so the water table and pore pressures will remain high. In the west wall all water should eventually drain out of the wall as no major recharge system can be identified at surface. Therefore, any specific failure mode will be less likely to fail in the west wall then the east. This additional stabilizing factor must be considered in the design of the west wall.

To illustrate the importance of a dewatering program at Equity stability analyses were carried out on the most likely failure modes in design sectors S4, S5, S6, S9, and S10. Each of these design sectors is located on the east side of the Main Zone pit where the greatest stability problems are anticipated.

Orientations of failure modes for the analyses were obtained from Section 5 of this report. Shear strength estimates of $\phi=31^{\circ}$, c=10.5 kPa were used. Details of how these values were selected are provided in Section 7. Groundwater conditions were varied from dry slope (pore water pressure u=0) to the theoretical maximum, u=33 kPa ($0.5 \cdot \gamma_w \cdot H/6$ equal to 33 kPa for 20 m high wedge). The results of the analyses are illustrated in Figure 6.15.

FIGURE 6-15

INFLUENCE OF WATER ON STABILITY



It is evident that the factor of safety decreases as water pressure increases. Wedges S9-1 and S4-7 remain stable even at full water pressure because they plunge at shallow angles (30°-35°). On the other hand, wedge S4-3 has a very steep plunge (50°) and remains unstable even under dry conditions. Because the line of intersection plunges at 50° the wedge will not daylight as a multiple berm failure unless the overall pit wall is cut steeper than 50°.

Wedges with intersections plunging between 35° and 60° out of the slope are of greatest concern in a stability evaluation because only these wedges can result in large multi-berm failures (i.e. they will daylight on the pit wall and may be sufficiently steep to be unstable). These wedges are also the most sensitive to changes in water pressure because they are generally close to limiting equilibrium when dry. As water pressure rises the factor of safety quickly drops below unity and failure occurs.

The theoretical factor of safety of wedges S4-9 and S5-6 drops very quickly to zero because they are both tight wedges and the plunge direction is oblique to the berm face. As a result pore water pressures quickly exceed gravitational forces and a buoyant condition is reached. In reality, slight movement of the wedge would allow the excess pore water pressures to dissipate and the factor of safety would again increase to some value near unity.

In summary, reduction of water pressures will reduce the probability of major failures in the Main Zone pit by minimizing the destabilizing forces acting on the wedge. Steep, single berm failures that are common on the east ultimate pit wall will not benefit from drainage because they are very near limiting equilibrium under dry slope conditions, or possibly unstable as soon as they are undercut.

6.9 RECOMMENDATIONS FOR FURTHER WORK

The preliminary investigation into groundwater hydrology conditions at Equity has indicated that it should be possible to dewater the Main Zone pit with a well engineered dewatering system. The investigation should now proceed to the next level, a two hole trial dewatering program. Pump tests should be attempted at the wells to confirm that the localized hydraulic conductivity measurements and assumptions regarding the air trac influence on K are valid on a large scale.

The gravity well drainage method should be evaluated by the mining engineering department. If proven operationally and economically feasible, the system should be promptly tested as it appears to have considerable potential for improving pit wall stability and reducing the need for slurry explosives. The dewatering trials could be incorporated into the first stage of the actual dewatering program.

Additional work is also required in continuing the piezometer monitoring portion of the preliminary program and in the completing of several small jobs that did not get finished during the summer.

6.9.1 Completion of Preliminary Study

A total of fourteen piezometer sites now exist in the Main Zone. Piezometers P09 and P13 were not completed during the summer. If possible, the standpipes should be installed before the locations are buried by snow. The steel drum protective covers should now be completed. They belong over the standpipes to protect them from damage and make them easier to find for unfamiliar monitors. Monitoring of the piezometers should be continued to better define the peak seasonal pore pressures, because these will have the greatest impact on slope stability. Monitoring will be required on a weekly basis from mid April to June to attain this goal. It is also desirable to further define the seasonal fluctuations of the water table in order to to accurately establish whether future observed changes in water levels are due to dewatering or seasonal fluctuations. Ideally, monitoring of existing piezometers should continue on a monthly basis, but because many of the sites will not be easily accessible or even locatable during the winter, it is recommended that only one easily accessible piezometer location be monitored during the winter. This program will provide sufficient information on seasonal fluctuations as these changes in piezometric levels appear to be fairly consistent at the two locations monitored extensively during the summer, and it is hoped that a similar response can be expected over the entire Main Zone area.

6.9.2 Initial Dewatering / Pump Tests

Two 30 to 40 m deep pumping wells should be installed in the bottom of the interior pit as the first stage of the dewatering program. One of the wells should be completed in the volcanics along the south wall, the other should be attempted along the east wall in the area of the suspected Bessemer Creek fault. The drill holes should be lined with perforated casing (vertical slots cut into pipe with torch).

Pumping tests should be carried out during the initial operation of these wells to establish the hydrologic parameters and to monitor the size and shape of the drawdown cones. Monitoring wells should be installed at 5, 10, 20 & 50 m along two lines radiating from the wells into the pit. The response in these wells will indicate whether the zone being dewatered is behaving as an inhomogeneous rockmass with impermeable zones of fault gouge, or whether blasting has fractured these zones sufficiently to create a relatively homogeneous medium of high transmissivity. If it is discovered that gouge zones are limiting the radius of influence of the pumping wells, then further field work will be required to identify any such structures before additional wells are installed and then locate the wells in the central portions of the gouge bounded blocks.

6.9.3 Gravity Well Drainage

The gravity well method of drainage should be carefully evaluated in terms of cost and practical operation in the Main Zone mining environment. This method has many advantages over the other drainage methods discussed, especially operating costs, improved slope stability, and possibly, increased use of ANFO. The method may also be successful in dewatering the gabbro unit because a close spacing of wells is practical with this method whereas it would be prohibitevly expensive if pumps would have to be purchased and maintained. Dewatering is recommended in that area because high groundwater pressures are known to exist. The pressures will exert a destabilizing force on all unfavourably oriented discontinuities. Recall that approximately five percent of major discontinuities in the east wall dip shallower than the overall slope and could result in a multiple berm failure.

The test should consist of four vertical 9" holes drilled to 30 m depth. A casing is not required unless the hole cannot be maintained open to full depth. The well should be filled with coarse sand. Sand placed near the bottom of the hole should be brightly painted. A 10 m spacing should be maintained between holes. Horizontal drainholes inclined slightly upward should be drilled to intercept the vertical wells from a berm 30 m below the well collars. The drain should be cased. The casing should be fully perforated for one third of its length, the second third should be perforated only on the top surface, the final third closest to the wall should not be perforated at all. The reasoning is to capture as much water as possible and then prevent it from discharging back into the rock face above the water table. If the drain does not intercept the drill hole (no coloured cuttings are observed) then a light charge should be detonated as close as possible to the vertical well to open up more flowpaths and improve drainage.

The initial test should be located in the volcanic units, near the southeast corner of the interior pit if access can be attained to benches of ideal geometry. If this is not possible an alternate site will have to be selected.

7.0 SHEAR STRENGTH CONSIDERATIONS

Analysis of the structural data has identified the most likely failure modes in each design sector. Whether a wedge or plane block that daylights out of the pit wall will actually fail will depend on the shear strength parameters and stress conditions acting on the failure surfaces. Groundwater pressures are also very important. It is beneficial to have a rough idea of how close are potential wedges within each design sector to failure, i.e. what is their factor of safety. If the factor of safety is below unity then the risk of undercutting the structures should be carefully evaluated. If any wedges or blocks begin to move in the pit, stabilization may be required to ensure safe working conditions below, especially if the failure is above a haul road or other key services in the pit. Shear strengths on the failure surface will have to be known to identify the method of stabilization, e.g. will drainage be sufficient, or what magnitude of support will be needed to stabilize the wedge.

An investigation into shear strength was carried out during the summer of 1984 as part of the Slope Design Program. The investigation had three phases: 1) point load testing to determine uniaxial compressive strength, 2) measurements of plunge angle of berm failures and slip tests to estimate friction angle ϕ, and 3) back analysis of berm scale wedge failures to compute ϕ and cohesion from actual slides.

This section summarizes the results of the shear strength investigations. Recommended values of cohesion and friction angle are presented for conditions of low confining stress (e.g. berm scale failures). Additional work should be undertaken to define the failure criterion at higher stress levels that would arise during a major pit wall failure.

7.1 POINT LOAD TESTING

Point load tests were performed on a total of 112 core samples from the 1984 exploration drilling program. Each test was carefully observed and pertinent data recorded. This included rock type, sample location, point load index I , failure mode, and any additional observations such as presence of weathering or alteration. A computer program was developed to statistically determine a representative uniaxial shear strength for each rock type. A report titled "Point Load Testing Program and Results" dated 84/06/19 discusses all aspects of the point load testing program and methods of analysis.

Results of the testing program indicate that the major rock units in the Main Zone have reasonably high intact rock strengths. Therefore, failure will be controlled by unfavourably oriented discontinuities. The gabbro proved to be the strongest unit, lapilli tuff was generally rated strong, ash tuff of moderate strength, and dust tuff proved moderately weak. Quantitative results are summarized in Table 7.1.

Table 7.1

Rock Type	Load to Failure (kN)	Design Strength (MPa)	Description	Rating
dust tuff	79	44.3	moderately weak	R2
ash tuff	156	87.6	moderately strong	R3
lapilli tuff	200	112.3	strong	R4
gabbro	455	255.4	very strong	R5
andesite dyke	313	175.7	strong	R4
quartz latite dike	359	201.5	strong	R4

7.2 ESTIMATION OF FRICTION ANGLE

Angles of plunge were measured on 17 single bench plane and near planar wedge failures in the Main Zone pit to get an approximate idea of the minimum angle of friction. The observations are plotted in Figure 7.1. No failures were observed until plunge angles exceeded 32°. It can be concluded that a failure is unlikely if the plunge of a wedge intersection or failure plane is shallower than 30°.

Very crude slip tests were carried out by placing rocks on the pit wall and measuring the plunge angle at the onset of slip. Slip test angles varied between 32° and 44°. Fresh, gouge free, reasonably smooth surfaces were used for the tests. The measured angles are indicative of the effective friction angle ϕ' (ϕ + roughness angle i).

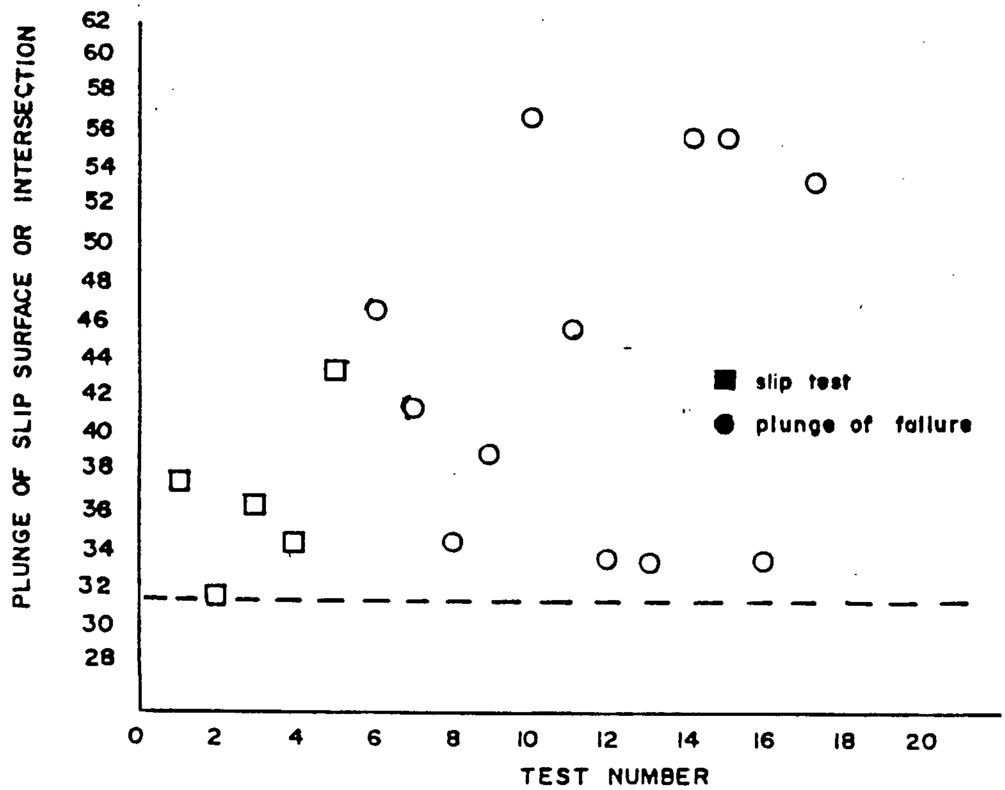


Figure 7.1 DETERMINATION OF APPROXIMATE BASIC FRICTION ANGLE

A first order estimate of ϕ can be made by analyzing the above failures and tests as simple plane failures and making several assumptions to reduce the number of unknowns. In the most general case the factor of safety for a plane failure can be expressed as:

Eq. 7.1 $$F = \frac{\text{resisting forces}}{\text{destabilizing forces}} = \frac{cA+(W\cos(\theta)-u-v\sin(\theta))\tan(\phi)}{W\sin(\theta)-V\cos(\theta)+S}$$

F = factor of Safety
c = cohesion
A = area of failure plane
W = weight of sliding block
θ = angle of plunge of failure surface
ϕ = friction angle
V = force due to water in tension crack
U = uplift force due to pore pressure
S = seismic force

The water table in the upper portion of the Main Zone pit where the observations were made is 10-20 m below surface so the berms are dry. Therefore, U and V can be dropped from Equation 7.1. If the failures occur well after detonation S will also be equal to zero. This assumption is made because it is conservative, generating low values of ϕ if a seismic force was present. Assuming U, V and S are zero, equation 7.1 can be simplified to:

Eq. 7.2 $$\tan(\phi) = F \tan(\theta)$$

At the onset of failure the destabilizing forces are exactly equal to the driving forces and F is equal to unity (condition of limiting equilibrium). At this moment ϕ is equal to the plunge angle θ. Many of the steeper wedges were probably stable only because they were keyed in at the toe. As soon as this support was removed they slipped down. For these failures F is less than unity but the exact magnitude is indeterminate so the only conclusion that can be made is that ϕ is smaller than θ in magnitude. Therefore, Equation 7.2 was applied only to the minimum observed plunge angle in estimating ϕ.

It should be noted that the only wedge failure that occurred during the summer fell out some time after the face was mucked out. The slide was triggered by vibrations from a shovel working nearby. The wedge must have therefore been very close to limiting equilibrium. The plunge angle was 44°. In this instance Equation 7.2 can be applied with confidence with F=1 so the friction angle for that surface must be near 44°. The relatively high friction angle is probably due to the roughness that was observed on the failure planes. It is also possible that some cohesion was present.

In summary, the friction angles in the Main Zone pit are likely in the range 30°-45°, and vary with the roughness characteristics of the failure plane.

7.3 BACK ANALYSIS OF BERM FAILURES

The factor of safety can be computed for a wedge in much the same manner as for the plane failure discussed in the previous section, although the formulation is more complex because of geometric constraints. In principle, the factor of safety is a ratio of resisting forces developed on the two failure surfaces to the driving forces of gravity and pore pressure acting on the wedge. The F.O.S. can be calculated if wedge dimensions, groundwater pore pressure, shear strength parameters c and ϕ, and rockmass density are known. Wedge dimensions and rockmass density are easily measured. If no seepage is observed on the berm it is likely that water pressure close to the pit face is zero. If water is seeping out of the berm then a higher value of u must be specified in the analysis (see Appendix B.2 for guidelines to groundwater pressure assumptions). Thus, the only parameters still required to calculate the factor of safety are c and ϕ.

It is difficult to establish accurate shear strength parameters to input into the stability analysis. Several methods of determining them are available, including: empirical correlations, laboratory tests, in-situ tests, and back analyses of actual failures. The recommended approach is to use all four methods to obtain several estimates of c and ϕ, and then use engineering judgement to select the most reasonable values.

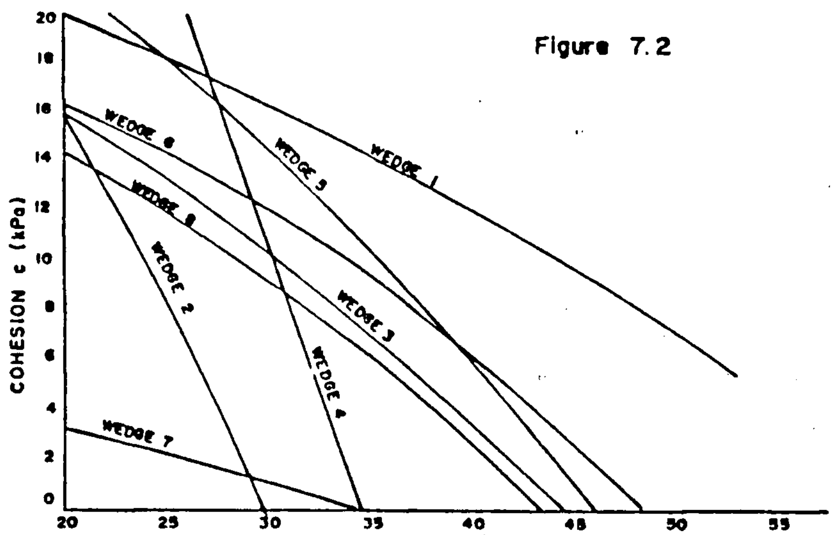
In the back analysis approach the assumption is made that the wedge attained limiting equilibrium just before failure so F.O.S.=1.0. The factor of safety equation can then be solved for c and ϕ. Because only one equation is available for two unknowns a unique solution is not possible, but a range of c/ϕ pairs can be determined.

A computer program was developed as part of the slope design project to calculate wedge stability and carry out a back analysis to determine a range of c/ϕ pairs that satisfy the condition of limiting equilibrium. The program SWEDGE is based on the "short solution for rapid computation of wedge stability" (Hoek, 1981). The program is fully documented in Appendix B.

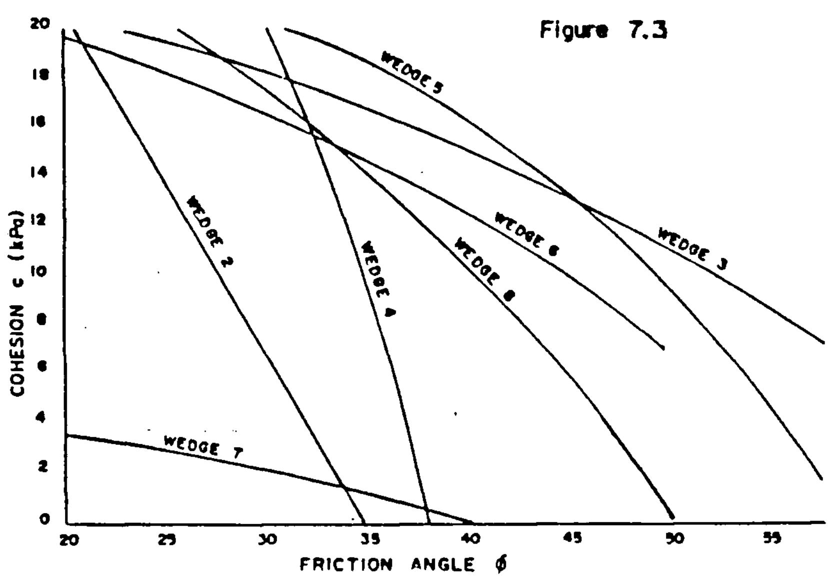
Eight larger berm failures were selected for the back analyses. Wedge dimensions and orientations varied. Field data for the analyses is tabulated in Appendix C.1. Appendix C.2 lists the output from SWEDGE for each back analysis.

Figures 7.2 and 7.3 summarize the results of the back analyses. Each graph is a plot of cohesion vs. friction angle that satisfy limiting equilibrium for the eight wedges. In developing Figure 7.2 water pressure was assumed to be zero, i.e. the slope was assumed dry.

In Figure 7.3 the average groundwater condition was assumed to be 0.5(γ_wH/6) or one half of the theoretical maximum value. Because all slip planes examined in this study were dry (DISCODAT code 3 or lower) Figure 7.2 is used for subsequent analyses as it most likely represents the conditions at failure.



Friction Angle and Cohesion Required for Limiting Equilibrium (F.O.S.=1) Under Dry Slope Groundwater Condition.



Friction Angle and Cohesion Required for Limiting Equilibrium (F.O.S.=1) Under Maximum Expected Groundwater Conditions

A conservative interpretation of Figure 7.2 would be to assume zero cohesion and to take the minimum friction angle required to maintain stability of the most stable wedge, e.g. 30° for wedge 2, as representative. All other wedges require greater shear strength, so they are assumed to be unstable and to have slipped as soon as the toe was exposed. This approach results in shear strength parameters of $c=0$, $\phi=30°$.

The conservative approach should not be used in open pit mine design because some failures can be tolerated; even desired to indicate that the pit is not overdesigned.

A probability function approach has been developed to obtain more realistic values of shear strength parameters based on the back analysis data. In this approach the relative probability of satisfying limiting equilibrium of all wedges is determined for each c/ϕ data point. The probability function can be generated by manually contouring a discritized version of Figure 7.2 using a counting circle. More elegant and time saving methods of doing this task can be developed.

Figure 7.4 is a contoured plot of the probability function for the eight wedges studied. There is a well defined maximum at $\phi=31°$, $c=10.5$ kPa. These shear strength parameters result in conditions nearing limiting equilibrium in the largest number of wedges tested and are the best estimate for c and ϕ based on existing back analysis data.

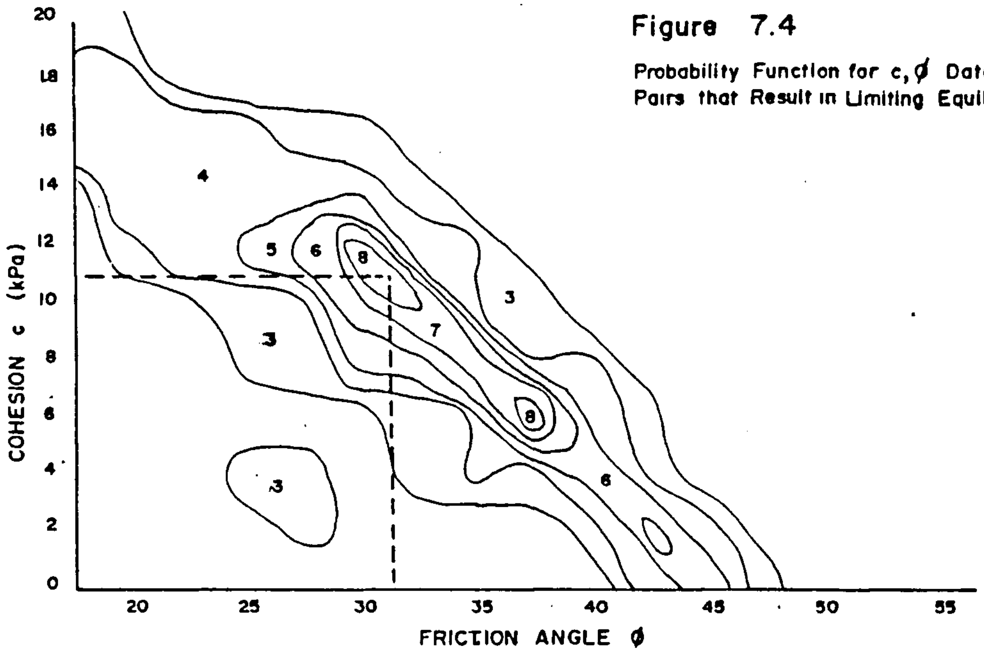


Figure 7.4

Probability Function for c, ϕ Data Pairs that Result in Limiting Equilibrium

7.4 SHEAR STRENGTH SUMMARY

Shear strength is a very important parameter in pit design. A structural study (as described in sections 5 and 9) can define the most probable failure modes in each design sector. The design criterion for numerous pits has been to minimize the number of failure modes that daylight. Such an approach is certainly more effective then designing to an overall slope angle of 45° regardless of geologic structure; however, if stability analyses indicate that the potential failure modes are stable if allowed to daylight (F.O.S.>1.2) then the pit wall angle can be safely steepened even further.

Shear strength parameters have been estimated for the Main Zone pit from slip tests, minimum angle of failure plunge, and back analyses.

All of these analyses indicate that the parameters can vary considerably if limiting equilibrium is assumed. By using the probability function a narrower range of values for c and ϕ have been identified. The mean results of this approach are c=10.5 kPa, ϕ=31°. They are consistent with results from slip tests and plunge measurements (ϕ=30° to 35°), and with results cited in published literature for similar conditions.

The results summarized above have been generated from a limited data base. In addition, the methods used required several assumptions that may not necessarily be valid. Further data collection, testing, and engineering analysis must be carried out before the shear strength/stability evaluation approach can be used for final design of Equity's pit slopes. Preliminary results using this approach are presented in section 9 and achieve favourable results but the "minimimum failure modes daylighting" concept is still used as the final design criterion in this report.

The following program is recommended to better define the shear strength parameters in the Main Zone pit:

1. Slip Tests (30)
2. Plunge Angle Measurements (30)
3. Back Analyses as slides occur
4. Direct Shear Tests on Major Discontinuities (10)
5. Comparison of Results to Emprical Failure Criteria

Note that the bracketed numbers are rough guidelines to the quantities of tests required. Testing should continue until the stability engineer is confident that further testing would not alter his best estimate. If results are consistent in all above tests then it is likely that fewer tests will be required than the suggested quantities listed above. Test methods 1 and 2 will likely yield similar results as previous tests, but are so easy to carry out that the data base can be increased without too much effort. The c/ϕ probability function will be much more reliable with a larger data base, especially if the back analyses are carried out on failures that occured well after the slope was mucked out, indicating conditions near limiting equilibrium. The direct shear tests will provide friction angles and cohesion values that can be used to verify the results of the back analyses and provide sound evidence that the assumptions made in the analyses are valid.

8.0 BLASTING CONSIDERATIONS

8.1 INFLUENCE OF BLASTING ON WALL STABILITY

Detonation of explosives close to the pit wall can cause structural damage to the rock behind the final dig line, reducing overall wall stability by developing fracture cracks in the slope and by increasing the frequency of ravelling and small berm failures in the pit. By carefully optimizing the trim blasting design several operational advantages can be realized. Foremost, when the rock forming the final wall is intact it will stand at a steeper bermface angle. Where stability of the benches controls overall pit angle there is good potential for steepening the pit wall if the bermface can be excavated to 70° from the current 65°. Less frequent ravelling and berm failures will reduce the chance of broken power cables, drainage pipes (especially when the drainage system is installed), and rock on the haul roads. Berms will require less clearing and will be more effective in catching any ravelling rock that does fall. Greater safety will be realized.

8.2 PARAMETERS THAT CONTROL BLAST PERFORMANCE

Breakage of intact rock is caused by two mechanisms during a blast. The detonation of an explosive generates a large quantity of gases that are initially confined in the rockmass under extremelly high pressure. The explosion transfers vast amounts of energy to the rockmass as if a big hammer hit the rock. Seismic waves (Primary, Secondary and Raleigh) transport the energy radially from the blast. When the wavefront passes through a point in the rockmass a force is exerted on the rock particles at that point and they are displaced. The level of strain and peak particle velocities are directly dependent on the type and strength of the wavefront. Fracture of the rockmass occurs when the maximum elastic strain of the rockmass is exceeded; onset of fracturing by tensile waves generally begins when peak particle velocities exceed 250 cm/s. The first goal of the trim blast design is to reduce the amount of energy per delay so peak particle velocities exceeding 250 cm/s are attained only within a small distance beyond the row of line holes.

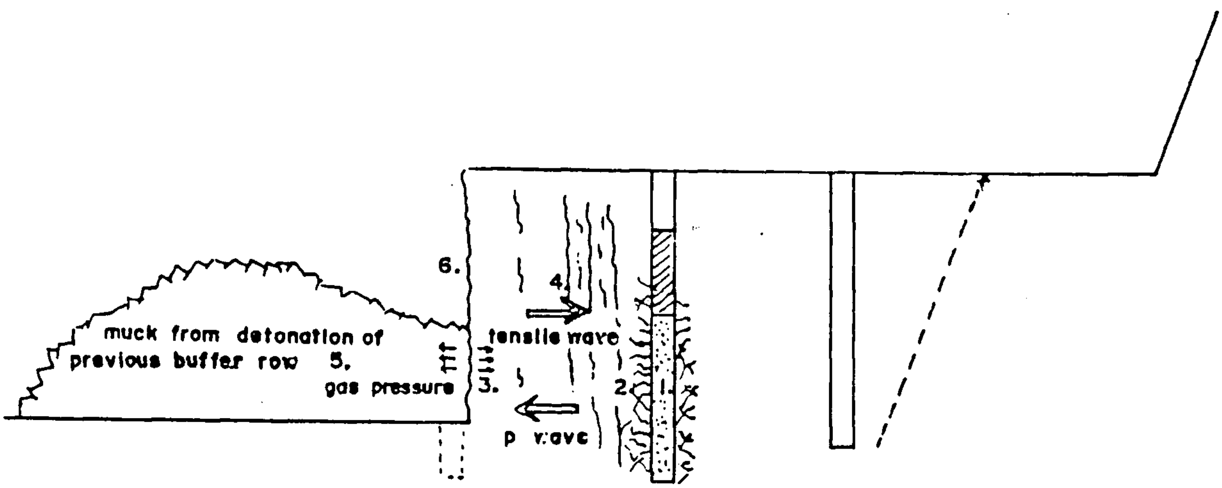


Figure 8.1

1. Detonation - high pressure gas created.
2. P wave has adequate energy to fracture rock in compression for 4 diam.
3. P wave reflected from free face, forms tensile wave parallel to face.
4. Tensile wave forms cracks since tensile strength of rock low.
5. Gases expand into tensile cracks, heave muck (illustrated on previous row.
6. New free face created (illustrated on previous buffer row).

Expansion of the explosive gases does most of the actual displacement of the rockmass. Gases penetrate into existing weaknesses and cracks opened up by the passing wavefronts. Differential gas pressures exert a very substantial force on individual blocks of rock, pushing them apart and outward toward the free face. If insufficiently confined, escaping gases can trigger cratering and flyrock. The second goal of trim blast design is to minimize the quantity of gases that penetrate the rockmass beyond the final digline. The mechanics of a blast are illustrated in Figure 8.1.

The following parameters can be systematically varied in the optimization of trim blast performance:

1. Powder Factor (mass ratio of explosive used/rock broken)
   Goal: - as low as possible while maintaining adequate fragmentation.

2. Explosive Charge per Hole
   Goal: - as low as possible while ensuring adequate quantities for detonation and compatibility with loading equipment.

3. Hole Spacing (especially trim row)
   Goal: - dictated by powder factor and charge per hole.

4. Burden (distance between rows)
   Goal: - well balanced, providing adequate confinement without choking.

5. Firing Order
   Goal: - Depends on control blasting technique. Should ensure that every hole detonates adjacent to a free face.

6. Delays
   Goal: - Adequate delay to prevent reinforcement of individual wavefronts.

7. Power of Explosive/Detonating Speed
   Goal: - Dictated by economics and water conditions. Low strength explosive should be used in detonation of line holes in cushion blasts.

8. Control Blasting Method (pre-split or cushion blasting)
   Goal: - Method should be dictated by rock conditions.

9. Blast Hole Diameter
   Goal: - Smallest diameter production drill should be used in drilling of line holes to maximize height of explosive column.

10. Sub-Drill Depth
    Goal: - As shallow as possible while developing adequate fragmentation on rocks at bottom of blast.

To be effective, the optimization trials must be well structured, docummented, and carefully evaluated. Any modifications should also be discussed with the blasting personnel before they are implemented to determine whether they would pose any operational problems. Sufficient observations of current trim blast performance must be made prior to the trials so that any changes can be adequately evaluated.

8.3 CURRENT BLASTING PRACTICE

At present, all trim blasts at Equity are cushioned. The trim pattern consists of two buffer rows and a trim line. The charge per hole is reduced in the trim pattern. To maintain the powder factor at the production blast level of 0.22 Kg/tonne the spacings are tightened up from the standard 4x5 and 5x5 m production patterns to dimensions illustrated in Figure 8.2. Damage in the final wall is a function of the energy contained in each seismic wave that is generated during an explosion. A delay is placed between each hole in the trim pattern to prevent reinforcement of several low energy waves into one high energy wave front. 15 ms and 25 ms delays are used between each line hole, 25 ms delays between each buffer hole, and 100 ms delays between each

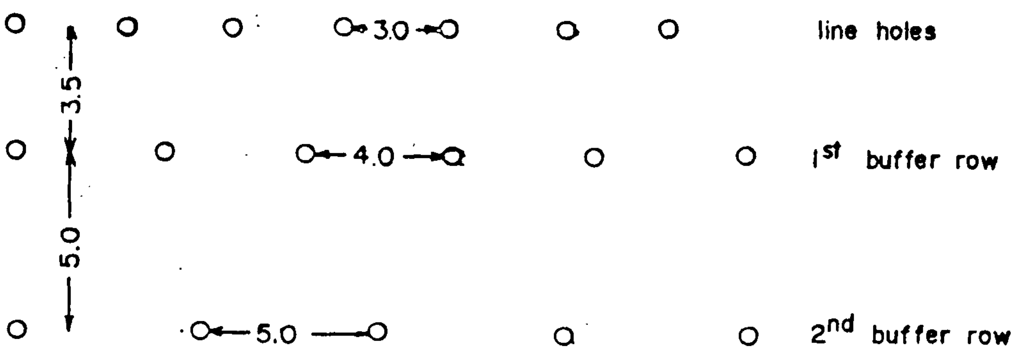


Figure 8.2 PRESENT TRIM BLAST PATTERN
all dimensions in m

row. The detonation sequence is as follows: 1) second buffer row, 2) first buffer row, and 3) line holes. The powder factor in the line holes is reduced to 0.148 kg/tonne to further reduce the amount of explosive per delay. The line holes are always loaded with higher strength slurry explosives, the buffer rows are charged with ANFO unless water conditions are severe, in which case slurry has to be used. The standard trim blast pattern outlined above is used in all rock conditions.

8.4 AREAS OF POTENTIAL IMPROVEMENT

Equity Silver Mines has a good control blasting program that reduces the damage to the final wall substantially from levels of damage that could be expected if production blasts were used throughout. Substantial work remains to be done on additional refinement of the trim blast design to further improve rock conditions in the final wall. Research should focus on: 1) use of lower strength ANFO in the line holes where possible, 2) further reduction in weight of explosive per delay, 3) reduction in line hole burden, 4) variation in trim blast design to match rock conditions (particularly rock type), 5) inclusion of "Hercudet" initiation system into trim blast design, and 6) design of detonation sequence to maximize formation of free face.

8.4.1 Use of ANFO in Line Holes

In the optimum cushion blast narrow diameter line holes are drilled on a tight spacing. The line holes have a reduced burden. Each hole is delayed. A low strength explosive is used to "peel" the final rock off the wall, leaving the rock behind the final digline with minimal damage.

In a mining environment the ideal cushion blast cannot be justified because of equipment and cost/return considerations, but the concept can be applied to reduce blast damage.

An equivalent mass charge of a lower strength explosive, e.g. ANFO, will do less damage to the final wall because the seismic wave will be weaker, attenuating to below the 250 cm/sec damage threshold in a shorter distance.

The use of ANFO in line holes has additional advantages that: 1) the charge per hole can be precisely controlled (i.e. not limited to 30 lb. shots as with slurry). As a result there will be greater flexibility in changing spacing or the powder factor in the line holes. 2) Less expensive explosives will be utilized whenever groundwater conditions permit. 3) Only one type of explosive will be required in dry areas so loading will be simplified. When wet holes are encountered plastic liners in sonnet tubes or slurry will have to be utilized. Far fewer wet holes should be encountered after a pit dewatering system is installed.

8.4.2 Reduction of Charge per Hole

One of the most reliable methods of predicting blast damage is the U.S. Bureau of Mines empirical formula that relates peak particle velocity of the rockmass to radial distance from detonation and weight of explosive charge per delay:

Eq. 8.1 $$V = K*(R/\sqrt{W})^{\beta}$$

V = peak particle velocity (in/s)
R = radial distance from blast (ft)
W = weight explosive per delay (lb)
K = constant, function of rockmass
β = constant, function of rockmass

The constants K and B are dependent on the elastic properties of the rockmass. Typical values are K=26 to 260 and B=-1.6. Given that rock begins to fracture at V=100 in/s and line holes are loaded with 60 lb of explosive per delay a range of distances to which the rock will be fractured can be predicted by solving equation 8.1 for R.

Eq. 8.2 $$R = \sqrt{W} * (V/K)^{1/\beta}$$

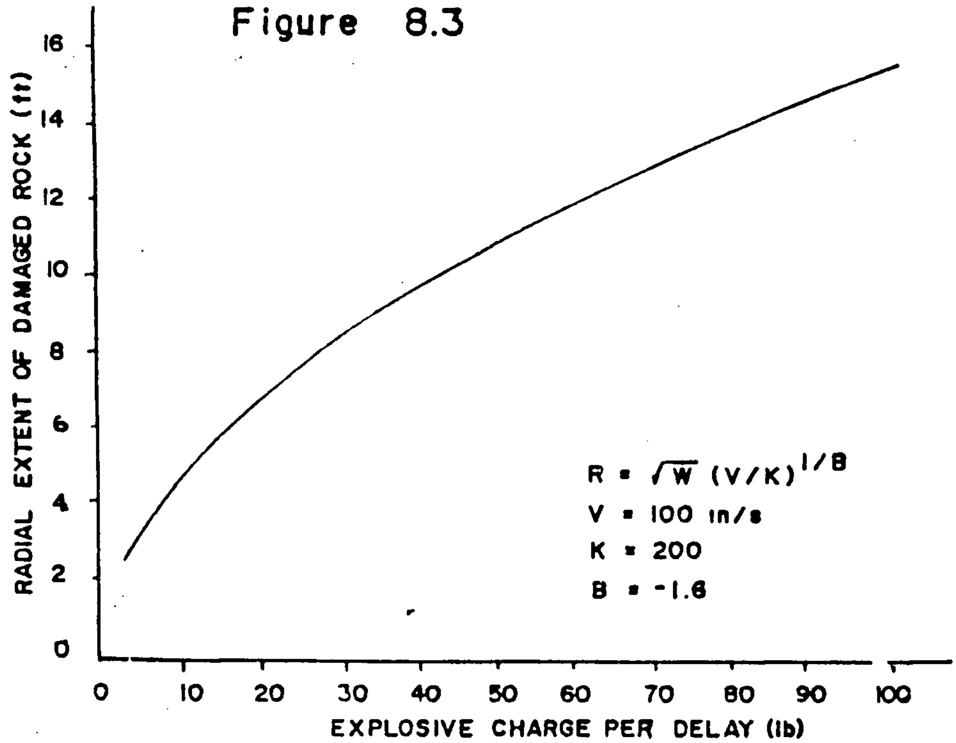
The results of several calculations are tabulated below.

Table 8.1

Assumption	K	β	W(lb)	R(ft)
current conservative	26	-1.6	60	3.34
current worst case	300	-1.7	60	14.78
current most likely	200	-1.6	60	11.95

The U.S. Bureau of Mines formula indicates that at present the damage from detonation of the trim blast extends well beyond the final digline, perhaps by as much as 15 ft. (assuming K and β values chosen are representative of the rockmass).

The trim blast optimization tests should focus on reducing the charge per delay. However, the charge cannot be reduced too much because it would only fill the very bottom of the hole. A reasonable goal would be to attain a 50 lb charge per line hole. This modification would require a 2.5 m line hole spacing to maintain the linehole powder factor at the present level of 0.148 Kg/tonne. A 2.5 m line hole spacing was successfully tested in one trial blast in 1984. Guidelines in the literature also suggest that spacing of the lineholes should be approximately 1/2 of the production spacing for maximum practical cushion effect. Figure 8.3 illustrates the influence of charge weight per delay on the lateral extent of damage to the rockmass.



If blasting trials indicate that a significant improvement in bermface angle can be attained by modification of the blasting pattern then economic rewards of a steeper pit may justify the purchase of a small diameter drill for line holes. Such a drill should have the capacity to drill holes up to 25° off vertical. If all holes in the trim blast were inclined parallel to the final face the burden would be of constant thickness from top to bottom. As a result, the reflected tensile wave would form cracks that would be parallel to the final face, resulting in a smoother ultimate wall.

The D-2 drill should be used on all trim patterns because it has the narrow 7 7/8" drill steel. The limited charge in the trim blast will form a 31% higher explosive column in the blast hole than if the 9" drill holes were utilized. This is desirable because the explosive force is distributed over a greater area of the rockmass.

8.4.3 Reduction of Burden in Line Holes

Because the line holes should peel away from the rock between the final buffer row they must not be heavily burdened. Burden thicknesses quoted in literature are consistently smaller than the line hole spacing, e.g. 0.5 to 0.8 of spacing. The influence of reduced burden should also be evaluated in the optimization trials. A distance of 2.0 m between the line hole and the first buffer row will be a good starting point for the trial.

8.4.4 Influence of Rock Conditions

The performance of an explosive is very dependent on the mechanical properties of the rockmass, especially on shear strength, elastic modulus, and frequency of discontinuities. There are two principal rock types in the Main Zone pit: intrusive gabbro and pyroclastics. The point load testing program has shown that the intact gabbro is 2 to 4 times stronger than the volcanic rocks. Uniaxial compression tests would likely indicate that the gabbro is also much stiffer. In-pit structural mapping and observations of R.Q.D. in bore holes has shown that discontinuities in the volcanics are closer spaced.

Blasting design should reflect these differences in the rockmass. The stronger, stiffer rock usually requires more explosive energy to attain equivalent fragmentation. The same pattern and powder factor is used in all production blasts in the Main Zone. Blasting tests should be carried out to determine the optimum powder factor for each rock type. The same powder factors should then be used in the trim blasts.

The pre-split controlled blasting technique has proven very effective in controlling blast damage to the final wall on numerous civil projects. The pre-split blasting pattern is similar to the cushion blast pattern illustrated in Figure 8.2, but the initiation sequence is quite different. Closely spaced, decoupled charges in the line holes are detonated first without any delay. Note that the Hercudet initiation system cannot be used in the line holes because the relatively low burning speed of the gas mixture causes a delay between each blasthole. Reinforcement of P-waves from adjacent blast holes and high gas pressures fail the rock in tension along a plane parallel to the row of line holes. Because the blast is greatly overburdened extremely high gas pressures are generated in the line holes. The gases penetrate the pre-split crack and open it slightly as they escape to surface.

After the pre-split crack is established the remaining buffer lines are detonated in the standard sequence. The pre-split crack will not prevent the compressive P-wave from penetrating into the walls, but it will provide a vent by which explosive gases can escape. As a result, radial cracks generated by hoop stresses induced by the expanding gases will not propagate beyond the pre-split plane nor will the gases open up any existing discontinuities. The overall result will be a much more intact rockmass.

It is recommended that the pre-split detonation method be tested in the gabbro. A line of twenty holes should be drilled. The initial test should not be carried out in the ultimate pit wall in case the gases vent along existing joints instead of opening the pre-split plane. The holes should be lightly loaded. Decoupling should be achieved with 2 inch internal diameter plastic pipe. The type of explosive should be selected with the assistance of the blasting contractor. Spacing of the line holes should be between 1.5 and 2.1 m. The blast should be inspected prior to detonation of the buffer rows to see if the pre-split crack has formed.

If the pre-split method creates a better final wall then the traditional cushion blasting it should be utilized because the stability studies indicate that there are numerous continuous joints in the rockmass that could act as release surface for berm scale failures. By reducing blast damage on the failure planes the shear strength can be maintained at near peak levels, increasing the stability of the potential failures.

8.4.5 Hercudet Initiation System

The Hercudet initiation system is presently being evaluated for use in all production blasts. The system has several operational advantages over conventional safety fuse delay systems including: simplicity and ease of operation, cost, testing of the circuit prior to detonation, and the potential of desensitizing the system after it has been charged.

The most favourable characteristic of the system from a stability point of view is the inherent delay between every blast hole. In the Hercudet system blasting caps are connected by plastic tubes that are charged with an explosive gas mixture. The flame front travels at 2500 m/s through the plastic tubing, and initiates every cap hooked into the circuit. In a standard production blast pattern approximately 15 m of tubing will extend between caps in adjacent holes. As a result, the detonation of consecutive blast holes will be delayed by a minimum of 6 ms. If additional delay is required extra tubing can be placed between the holes, e.g. 2.5 m per 1 ms.

The Hercudet system should also be used in trim blasts if it proves effective in production blasts because adequate delays are especially important near the final face. The goal of any controlled blasting procedure is to reduce the explosive energy released to one hole per delay (unless pre-splitting). The Hercudet system will guarantee that this goal is achieved.

8.4.6 Firing Order and Confinement

To minimize the amount of blast energy going into the final wall and to attain maximum fragmentation it is important that all holes be free faced at the time of explosion. The free face reflects the P-wave into a tensile wave that does most of the fragmentation. If a hole is overburdened the gases and fragmented muck cannot expand outward toward the free face. Higher gas pressures are developed, opening radial cracks that may extend well beyond the final digline. Energy that would have been dissipated during the expansion is instead redirected into the final wall as a higher energy seismic wave.

In summary, blasting next to a free face results in better fragmentation of muck and less gas and vibration damage to the final wall. Therefore, it is important that no ultimate wall trims are choked blasts or sinking cuts.

The recommended detonation sequencing is illustrated in Figure 8.4A. An alternate detonation sequence that establishes the free face at 90° to the ultimate wall should also be tested. This firing sequence has the advantage that any constructive reinforcement of seismic waves and propagation of the reflected tensile wave that causes most of the damage will be parallel to the final wall and not into it. The firing order is illustrated in Figure 8.4B. This technique is effective if a large number of buffer rows is used because a well defined free face will be established. Because only two buffer rows are used at Equity the technique may not prove as successful as the standard initiation method.

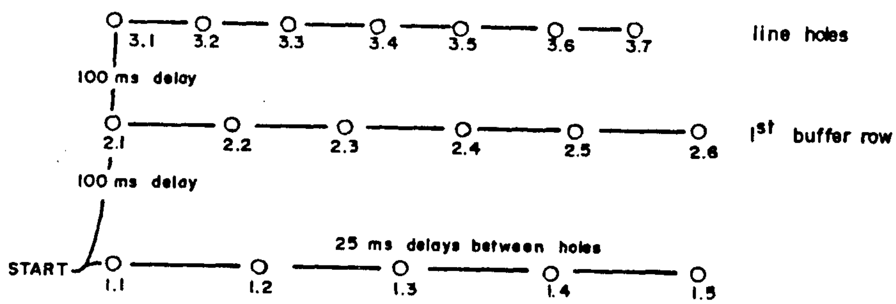


Figure 8.4.A PRESENT DETONATION SEQUENCE

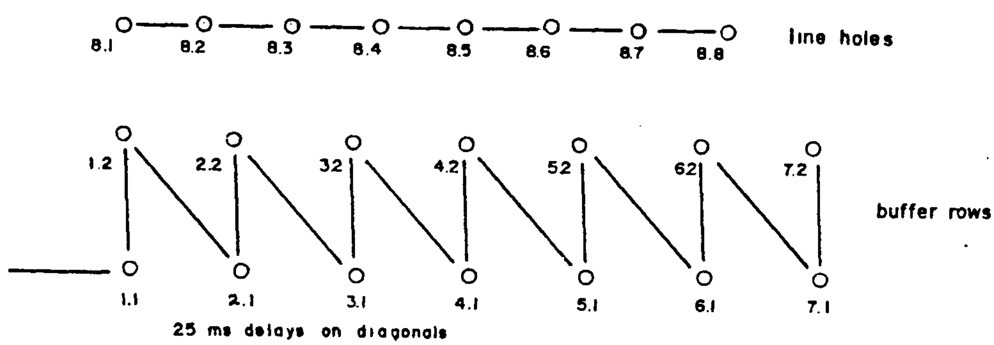


Figure 8.4.B DETONATION SEQUENCE THAT CREATES FREE FACE PERPENDICULAR TO WALL

9.0 EVALUATION OF PIT SLOPE STABILITY

9.1 PARAMETERS THAT INFLUENCE STABILITY

Throughgoing discontinuities, water conditions, shear strength, and blasting all influence pit wall stability. The most important variable in this list is geologic structure. In all but the weakest rocks failure can only occur if some pre-existing weakness is present in the rockmass on which the failure can occur.

A wedge bounded by two discontinuities is the most common failure type. Failure will only occur if the line of intersection of the two planes plunges at an angle shallower than the angle of the slope, i.e. the failure daylights.

To determine whether there is potential for a failure the structural fabric of the rockmass must be well understood. In the Main Zone the dominant discontinuity orientations in each of the four structural domains have been accurately established by line mapping and statistical analysis.

At this point, all possible combinations of planes must be evaluated to see if they will daylight out of a slope of given geometry. The Main Zone pit has been divided into 10 zones of consistent geologic structure and pitwall geometry. These zones are called Design Sectors. Any discontinuities that daylight out of the slope have potential for failure. They are called "kinematically possible" failure modes. Failure can occur by several different failure mechanisms. These include: wedge plane, toppling, block, active-passive, circular, and step. Only the first three mechanisms have been used to identify kinematically possible failure modes in this study because they are the most common failure types in open pit mine environments.

A pit could be designed on the criterion that "no failure modes shall daylight out of the slope". This approach is extremely conservative because not all kinematically possible failure modes will be unstable and a small number of failures can be tolerated in a mine environment, if under controlled conditions. Stability will depend on wedge geometry, shear strength on the failure plane, and groundwater conditions. The effect of these parameters was discussed in detail in in earlier sections of this report.

A more reasonable design approach is to identify all kinematically possible failure modes and determine their approximate stability. When a kinematically possible failure is identified as very stable, marginally stable, or unstable a better decision can be made on the slope angle. If the failure is stable the slope can be steepened further. If it will be unstable the slope should be designed so the failure does not daylight unless the probability of an actual failure is sufficiently remote to justify the risk of letting it daylight. If it is marginally stable the slope can either be flattened or some other remedial measure taken to increase stability.

9.2 Methods and Assumptions Used in Design

The design process used to determine the maximum safe slope and berm face angle in each design sector is summarized below in point form:

1. Determine average plunge direction of pit wall.

2. Place pit geometry overlay on Failure Modes figure for appropriate structural domain.

3. Determine maximum overall slope for which no failures will daylight.

   - if failures daylight at slopes $< 50^\circ$ calculate F.O.S. for failure mode. Assume c=10.5 kPa, $\phi=31^\circ$, failure height of 50 m, dry condition.

     - if F.O.S. > 2 failure is stable under all conditions and can be allowed to daylight.

     - if 1 < F.O.S. < 2 failure is marginally stable and sensitivity of water must be calculated. Assume maximum average water pressure of 33 kPa for 50 m high slope.

     - if groundwater causes F.O.S. to drop below 1.1 recommend dewatering in design sector.

     - if F.O.S. < 1 failure will be unstable. Evaluate probability of occurence based on size of discontinuity groups forming the two failure surfaces.

       - if probability high failure should not daylight.

       - if probability low potential hazard should be noted but the slope can be steepened to allow failure to to daylight.

4. If no failure modes daylight below 50° stability will be controlled by berm failure.

   - if any failure modes daylight below 65° berm face evaluate probability of those berm failures occuring based on size of discontinuity group.

     - if probability high flatten berm face.

     - if probability low maintain berm face at present value and note possibility of berm failures.

     - some berm failures can be tolerated as long as adequate catchment is provided by benches below.

5. If no berm failures exist stability will be controlled by maximum berm face angle that can be maintained by the rockmass.

    - examine orientation of minor joints.

    - evaluate potential for step failure on minor joints.

    - design berm at maximum angle that can be maintained, presently 60°-66°.

6. Check compatibility of design.

    - The berm face angle must be sufficiently steep to allow an 8 m wide berm every 20 m in elevation. If this cannot be achieved the overall slope angle must be flattened.

9.3 DESIGN SECTORS

This section presents a brief stability evaluation of each design sector. Recommended slope angles, most likely failure modes, and expected groundwater conditions are briefly discussed. If stability of the design sector could improve from drainage then some form of a dewatering system is recommended. The most important design conclusions presented in the following 10 subsections are summarized in Table 9.1 on the following page. Figure 9.1 shows the location of each design sector in the Main Zone.

Table 9.1 SUMMARY OF DESIGN PARAMETERS

Sector	Wall Angle (deg)	Berm Face (deg)	Controlling Failure Mode	Groundwater Condition	Drainage
1	50	66	step	very favourable	no
2	49	64	step	mod. unfavourable	yes
3	49	64	step/berm wedge	very unfavourable	yes
4	45	59	full wall wedge	unfavourable	yes
5	45	59	berm wedge	unfavourable	yes
6	46	60	berm plane	unfavourable	yes
7	50	66	step	mod. favourable	no
8	50	66	step	favourable	no
9	46	60	full wall wedge	unfavourable	yes
10	45	59	full wall wedge	mod. unfavourable	yes

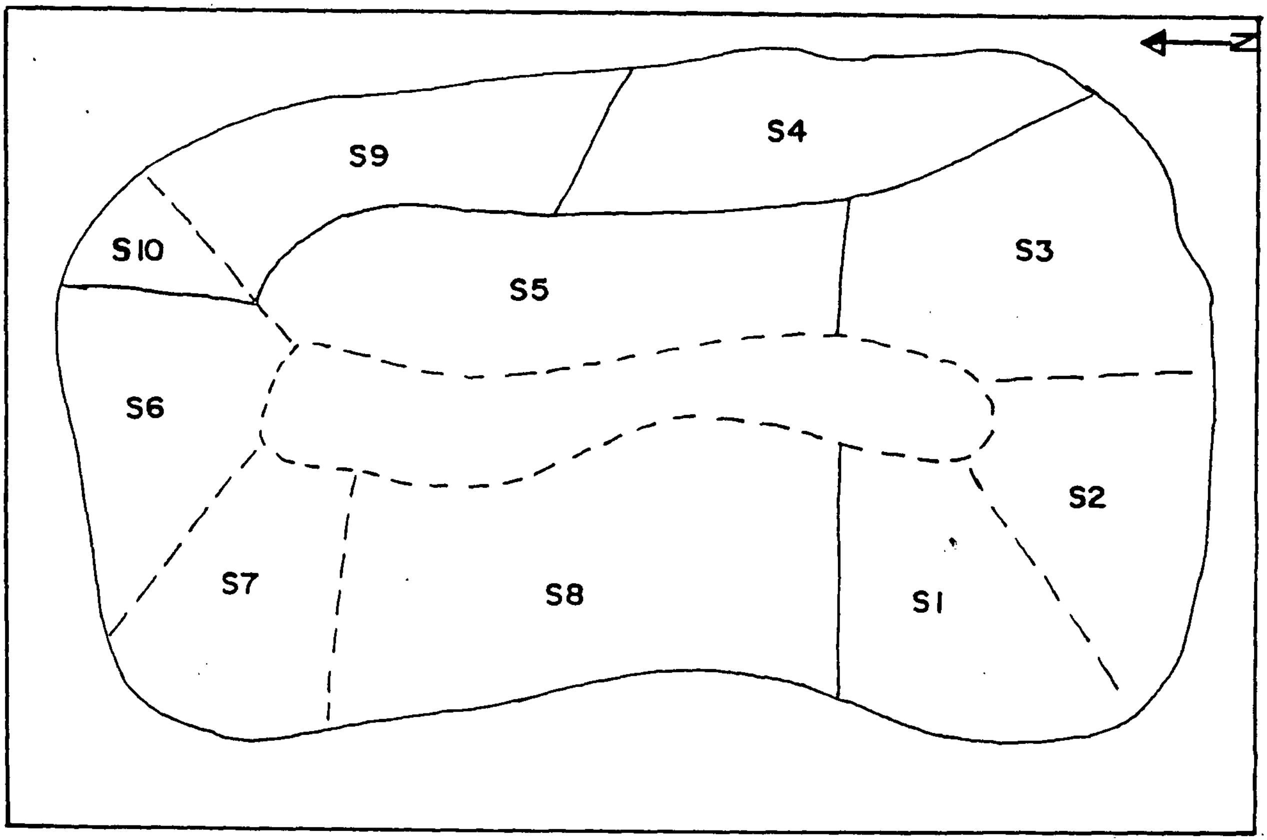


Figure 9.1 MAIN ZONE DESIGN SECTORS

9.3.1 Design Sector S-1

STRUCTURAL DOMAIN	-	D1
DIP DIRECTION	-	045°
ROCK TYPE	-	volcanics
WATER CONDITION	-	very favourable
OVERALL PIT ANGLE	-	50°
BERM FACE ANGLE	-	66°

STABILITY EVALUATION:

Stability of berm face controls the overall pitwall angle in this design sector. No kinematically possible failure modes controlled by major discontinuities have been identified. Stability of the berm will be controlled by persistent sets of minor joints that dip approximately 35° out of the slope. The joints are not continuous; as a result a step failure condition develops. In areas where step failure occurs berm angles of 60° to 65° degrees have been measured in the field. The recommended pit wall angle is based on maintaining a 66° berm face. Figure 9.2 is a stereographic plot of geological structure and pit geometry in this sector.

COMMENTS:

Because this sector is located on the west side of the pit, opposite the major groundwater recharge areas, the groundwater conditions are expected to be very favourable. Dewatering should not be necessary. Only limited pit wall exposures were available in this sector in 1984 (a total of only four traverses). As a result, the design is also based on structural information collected in S-2 and S-3. Additional line mapping and analysis will be required to verify the present design as soon as adequate exposures are excavated.

9.3.2 Design Sector S-2

STRUCTURAL DOMAIN	-	D1
DIP DIRECTION	-	020°
ROCK TYPE	-	volcanics
WATER CONDITION	-	moderate to unfavourable
OVERALL PIT ANGLE	-	49°
BERM FACE ANGLE	-	64°

STABILITY EVALUATION:

Most major structures strike perpendicular to wall or dip steeply into it. No large scale failures on major discontinuities are anticipated. Step failure and localized toppling failure on berm scale will control slope stability in this design sector. The pit wall design is based on a maximum bermface angle of 64° that is presently maintained in this sector.

COMMENTS:

Modifications to trim blasting procedures may improve rockmass condition in the final wall. If berms can be maintained at 70° the overall slope can be steepened to 53°. Concavity of the pit in this sector will increase overall stability. Poor groundwater conditions will be encountered at depth; therefore, it is recommended that drainage be installed to reduce the destabilizing forces of excess fluid pressure.

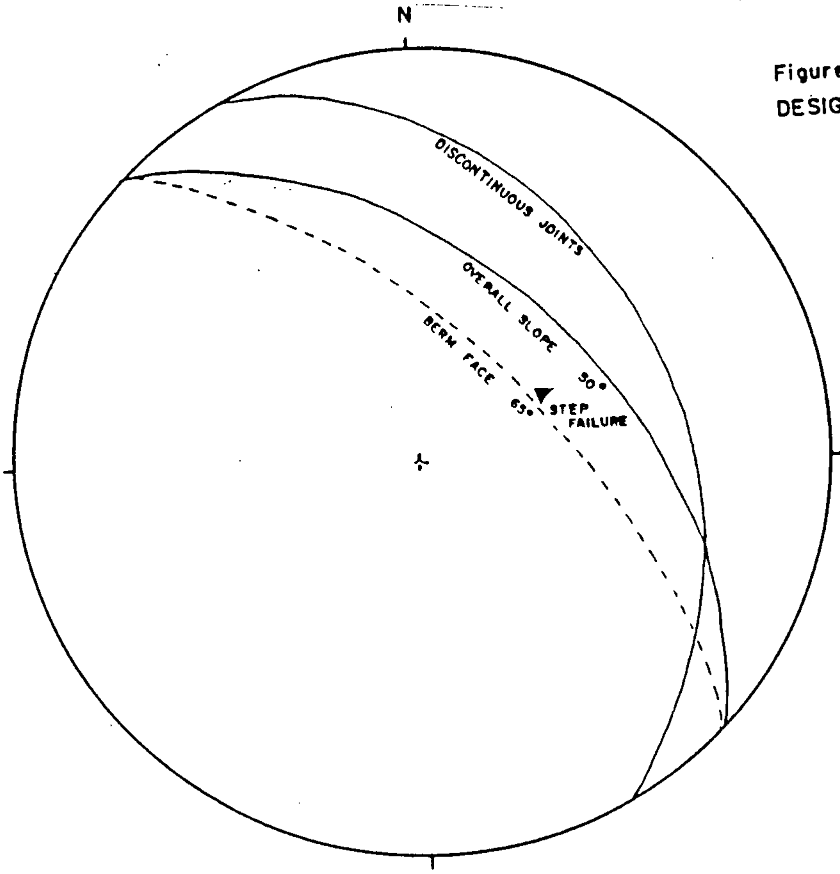


Figure 9.2
DESIGN SECTOR S-1

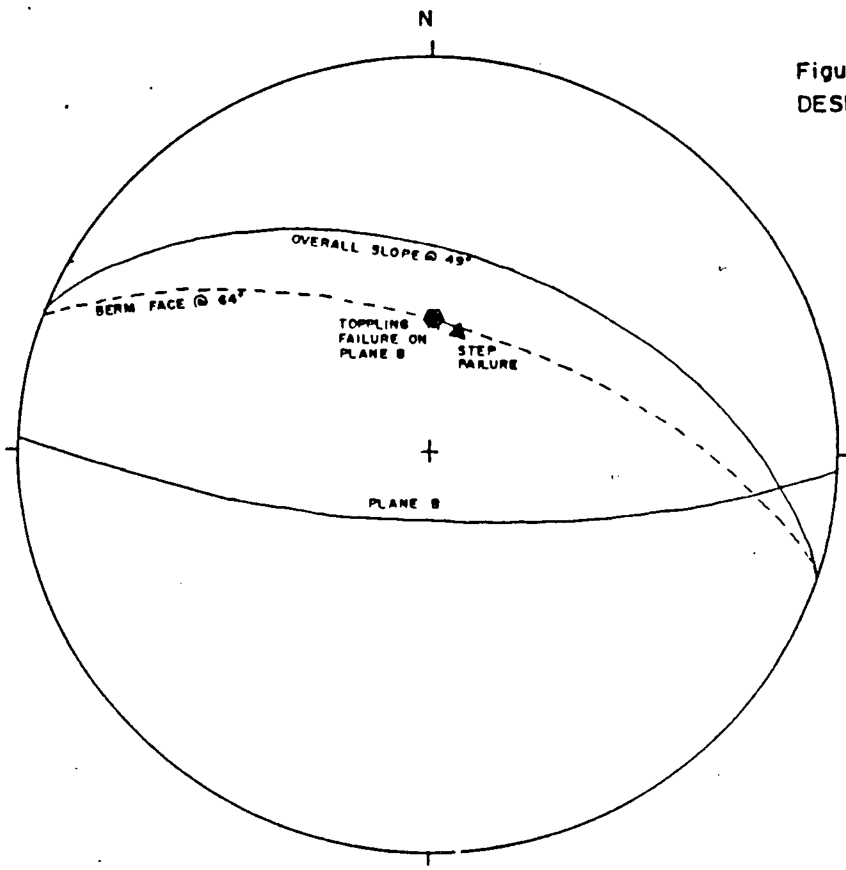


Figure 9.3
DESIGN SECTOR S-2

9.3.3 Design Sector S-3

STRUCTURAL DOMAIN	-	D1
DIP DIRECTION	-	320°
ROCK TYPE	-	volcanics
WATER CONDITION	-	very unfavourable
OVERALL PIT ANGLE	-	49°
BERM FACE ANGLE	-	64°

STABILITY EVALUATION:

Pit wall angle is controlled by the maximum angle of berm face that can be maintained in the volcanic rocks except in the northern portion of the design sector. There, steeply dipping wedges formed by planes B, C, and D begin to daylight out of the berm if it is steeper than 60 . Because no discontinuous jointing is observed parallel to the slope in this design sector step failure problems are not anticipated and the berms will remain stable at 64°. In the northern portion of the sector wedges 1 and 2 will start to daylight if berm face angles exceed 48°. To reduce the volume of any berm failures and to minimize the potential for full wall failure on these planes the berm face angle should be reduced to 60°, resulting in a 46° overall slope in that area. A slope reduction is also required in this sector to serve as a transition zone between S-2 at 49° and S-4 at 45°. Figure 9.4 illustrates pit geometry and expected failure modes in this design sector.

COMMENTS:

Groundwater conditions are very unfavourable in this domain. Large quantities of recharge are expected to flow into the pit through this sector because the volcanics are much more permeable than the gabbro. The water table in the upper portion of this sector is within 20 m of surface. In the pit seepage has been observed at 1290 m elevation, suggesting that the water table is at surface at this depth in the pit. Because higher water pressures will occur as mining progresses to depth a drainage system should be installed in this design sector. The drainage will intercept a large quantity of inflows into the pit. It will also reduce the destabilizing forces of water on any potential failures. Concavity of this design sector will contribute to overall stability of major failures.

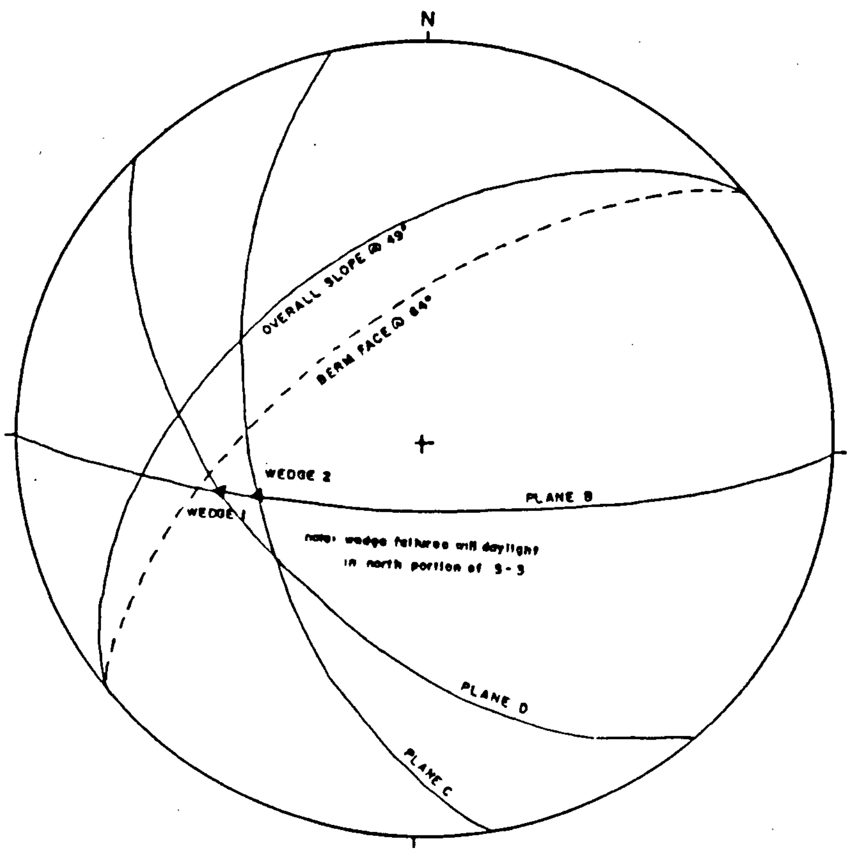


Figure 9.4
DESIGN SECTOR S-3

9.3.4 Design Sector S-4

STRUCTURAL DOMAIN	-	D2
DIP DIRECTION	-	275°
ROCK TYPE	-	gabbro
WATER CONDITION	-	unfavourable (very unfavourable at depth)
OVERALL PIT ANGLE	-	45°
BERM FACE ANGLE	-	59°

STABILITY EVALUATION:

Design sector S-4 has the greatest potential for developing instability in the Main Zone pit. Failure will occur on plane A that strikes 267°, parallel to the pit walls, The mean dip is 51° into the pit. The mode of failure will be either planar or an assymetrical wedge formed by plane A and one of the steeply dipping discontinuity sets that strike near 90° to the pit wall (e.g. C & D in Figure 9.5). Several berm failures that have this orientation have already been observed.

Because the mean angle of plunge for wedge 3 is 50° there is no way to prevent berm failures unless the face angle is reduced to unrealistic levels (e.g. 45°). The design goal in this sector is to minimize the potential of a multiberm failure. This can be achieved if the overall pit slope angle is maintained at 45° and the berm face is excavated at 59°. An adequate berm 8 m wide must be maintained every 20 m in elevation to confine any berm failures. The three most likely failure modes in this domain are shown in Figure 9.5.

COMMENTS:

Because the statistical distribution of dips on planes in group A is broad (see Figure 9.6), there is a possibility of a multi-berm failure in this sector even at the 45° overall slope angle. The failure would of course occur on one of the flatter discontinuities in group A.

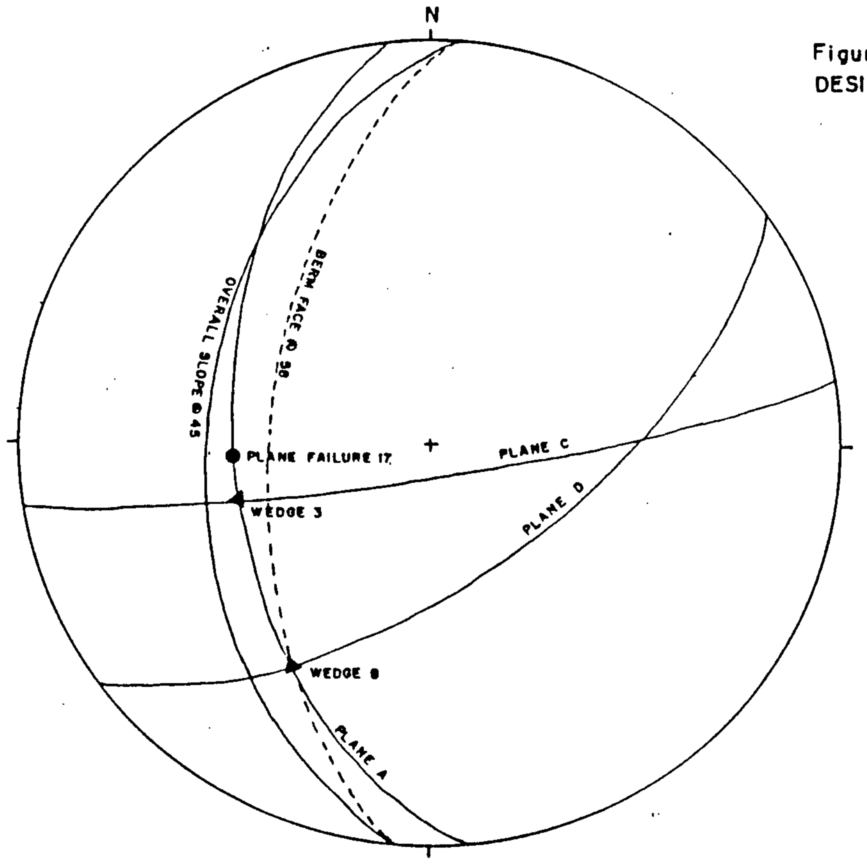
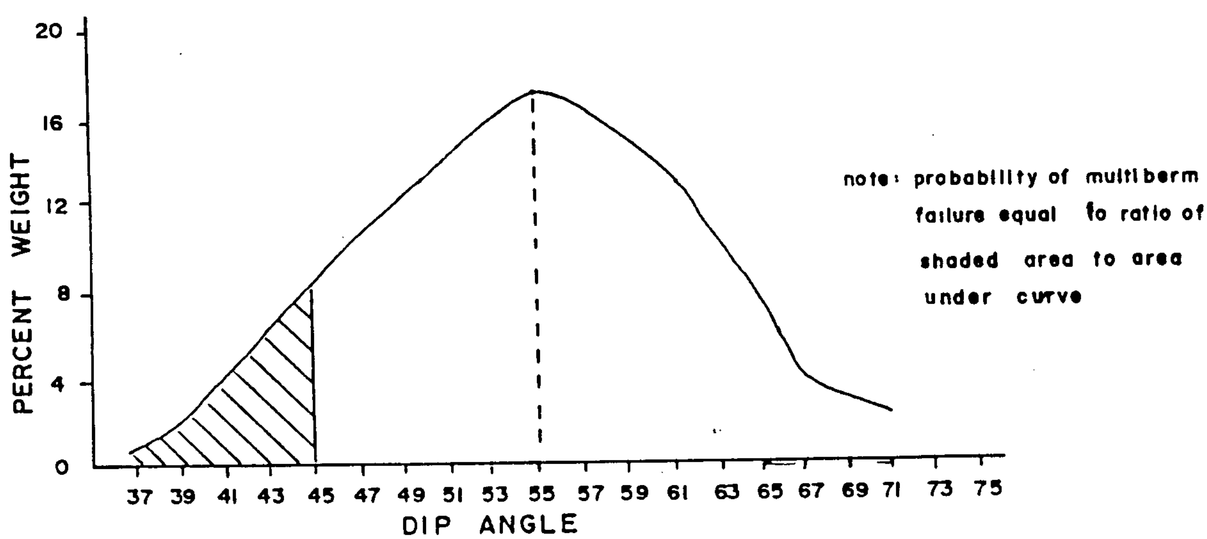


Figure 9.5
DESIGN SECTOR S-4

Figure 9.6

STATISTICAL DISTRIBUTION OF DIP ON GROUP A PLANES
ALL MAJOR STRUCTURES IN DOMAIN D-2.



A drainage system that reduces pore pressure behind the ultimate wall will significantly decrease the probability of a large failure. For example, a 50 m high wedge bounded by one of the flatter planes in group A ($40^\circ/267^\circ$) and plane C ($83^\circ/173^\circ$) will be very unstable (F.O.S.=0.81) if the average water pressure on the failure surfaces exceeds 16.5 kPa, only one fifth of the theoretical maximum value and equivalent to a maximum pressure head of only 10 m somewhere near the center of the wedge. If the water can be drained the wedge will be stable with a factor of safety of 1.08. The stability analysis assumed shear strength parameters presented in Section 7.3.

The WIP GraD drainage system will be the most effective method of reducing water pressures in this design sector because closely spaced wells will be required to achieve sufficiently rapid drainage in the low permeability gabbro.

The current east ultimate pit wall is slighlty convex. Stability will be reduced somewhat because less lateral confinement will be provided on any planar failure. As the convexity is very broad the destabilizing influence will not be as severe as in the Southern Tail pit where large failures occured on both convex lobes.

Design Sector S-5

STRUCTURAL DOMAIN	-	D3
DIP DIRECTION	-	270°
ROCK TYPE	-	volcanics
WATER CONDITION	-	unfavourable
OVERALL PIT ANGLE	-	45°
BERM FACE ANGLE	-	59°

STABILITY EVALUATION:

Stability in this design sector is controlled by stability of the berms. Berm failures are expected on failure modes 4, 6 and 12 (see Figure 9.7). All of these failure modes plunge quite steeply (peak plunges 56°- 60°) to daylight out of the overall slope so multi-berm failures will not occur on these discontinuities. Wedge 3 is the only failure mode in this sector that can daylight out of the overall slope. Because it is a very thin, overhanging wedge plunging only 35°, it will not pose stability problems (F.O.S. under dry condition = 5.03). Pit wall design is based on minimizing the number of berm failures by reducing the berm face angle to 59°. With the constraint of a 59° berm face and an 8 m wide berm every 20 m in elevation to contain ravelling the maximum pit slope angle that can be maintained is 45°.

COMMENTS:

Groundwater conditions are expected to be unfavourable in this sector because the water table will be at or close to the surface deep in the pit. The Bessemer Creek dyke package is located in the center of this sector. The dyke package is expected to act as a substantial groundwater discharge area. To minimize the amount of surface water entering this sector all east wall diversion ditches must be fully lined. Drainage should be considered in this sector to improve stability of any multi-berm failure modes that could develop and to capture as much of the incoming water so the water table on the pit floor can be pulled down.

The slight convexity of the ultimate pit wall in this sector will have a minor destabilizing influence.

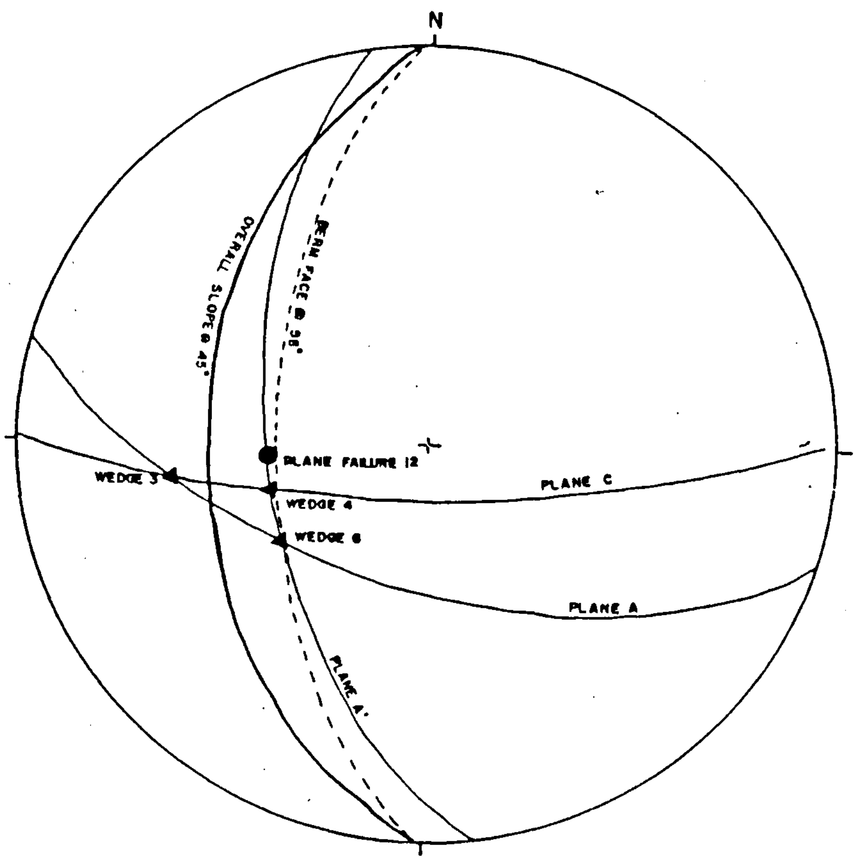


Figure 9.7
DESIGN SECTOR S-5

9.3.6 Design Sector S-6

STRUCTURAL DOMAIN	-	D3
DIP DIRECTION	-	180°
ROCK TYPE	-	volcanics
WATER CONDITION	-	unfavourable
OVERALL PIT ANGLE	-	46°
BERM FACE ANGLE	-	60°

STABILITY EVALUATION:

Some stability problems must be expected in design sector S-6. The most dominant discontinuity trend strikes sub-parallel to the pit wall. The peak dip is 60° so berm failures must be expected on the flatter discontinuities within this group. The berm face was designed at the same inclination as the peak dip of plane A so any failures that do form will be contained easily by the catchment berms because they will be thin slivers.

Overall pit wall stability will be controlled by failure modes 8 and 11 that will release on plane D (see Figure 9.8). Plane D is very unfavourably oriented, dipping at 35° out of the slope. As a result, any continuous discontinuities with this orientation will daylight out of the overall pit slope and could result in multi-berm failures. Reduction of the overall slope angle below 45° to limit failures 8 and 11 is not practical because plane D is not a dominant orientation. Rather, the sector should be designed at 45° and any loose wedges should be removed or some form of remedial measures should be applied to increase the stability.

COMMENTS:

This sector has considerable potential for a multiberm failure on plane D. A stability evaluation of wedge 8 gives a factor of safety of 1.04 under dry conditions and only 0.88 with water present (assuming $\phi=31^{\circ}$, c=10.5 kPa, slope height 50 m, average water pressure = 16.5 kPa, equivalent to a maximum head of 10 m). The analysis clearly indicates that reduction of water pressure will lower the potential for a multiberm failure considerably. Any instability that does develop will then require only minimal support to increase the F.O.S. above 1.1.

The pit walls in this design sector will be tightly concave. The concavity will also help to increase stability of the larger failures because it will provide an element of lateral confinement.

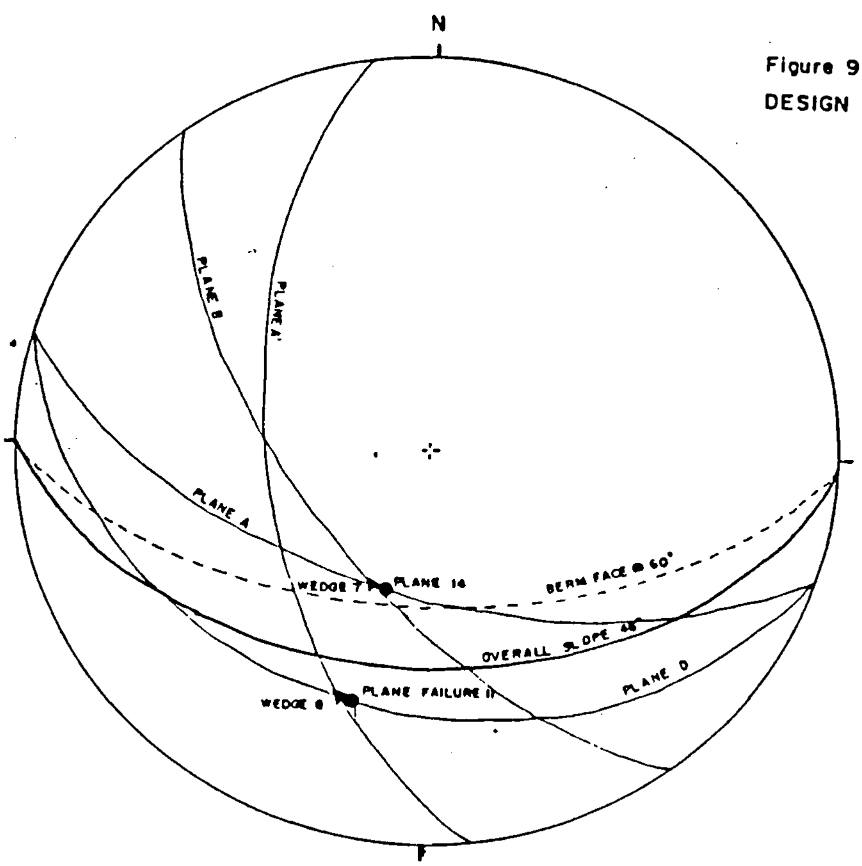


Figure 9.8
DESIGN SECTOR S-6

9.3.7 Design Sector S-7

STRUCTURAL DOMAIN	-	D3
DIP DIRECTION	-	115°
ROCK TYPE	-	volcanics
WATER CONDITION	-	moderately favourable
OVERALL PIT ANGLE	-	50°
BERM FACE ANGLE	-	66°

STABILITY EVALUATION:

Design in this sector is controlled by the maximum berm angle that can be maintained. Figure 9.9 shows the orientations of the major structural trends in this sector. As all major structures strike very obliquely to the wall there appears to be no danger of a major berm failure. The only kinematically possible failure wedge (9) was analyzed by program SWEDGE. The factors of safety were 2.43 for a dry slope and 1.13 for a slope with an average water pressure of 33 mPa (20 m maximum head). The wedge should not present any stability problems.

The discontinuity groups do not combine to form any berm scale failures. Pit walls in this design sector should be very stable. As a result the pit wall design is based on maintaining the maximum berm face angle that appears to be stable in the volcanic rocks of the Main Zone. This angle is 66°.

COMMENTS:

Favourable groundwater conditions and concavity will both improve overall stability. Drainage should not be required unless monitoring indicates that high pore pressures are developing. According to the latest pit design the main haul road will be located on the west wall of the pit. The location is very favourable in terms of stability considerations and no failures should threaten the haul road during the life of the mine according to currently available data. It is important to note that the design in this sector was based on information collected several hundred meters away in design sector S-5. Line mapping and analysis must be carried out in S-7 when one or two benches become exposed to confirm the validity of the present design before a large portion of the ultimate pit wall is excavated.

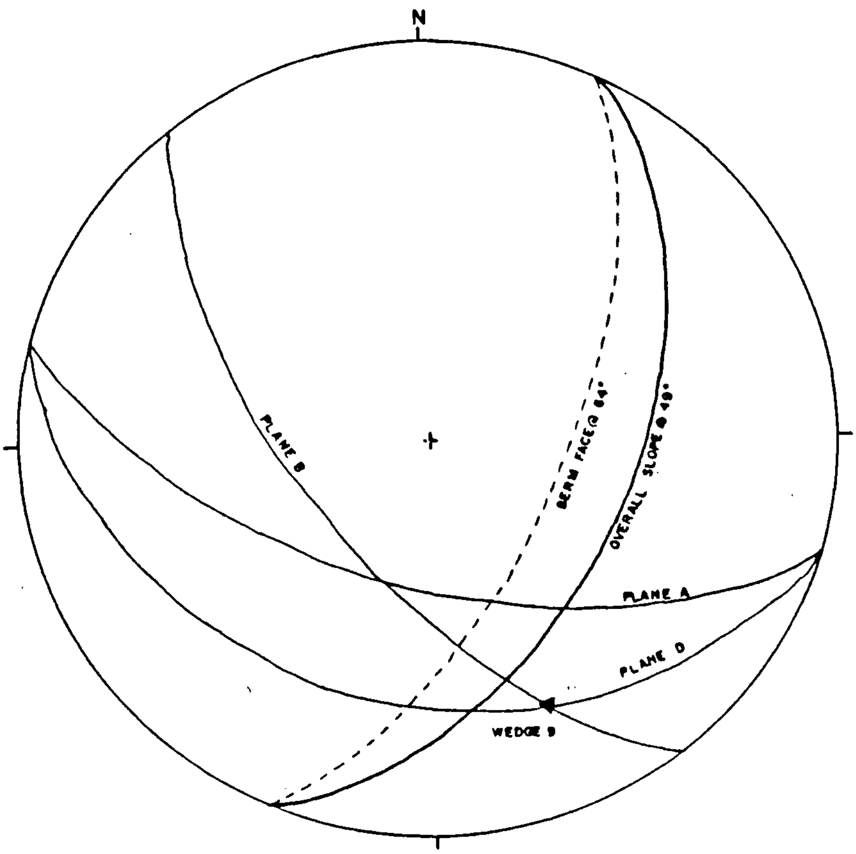


Figure 9.9
DESIGN SECTOR S-7

9.3.8 Design Sector S-8

STRUCTURAL DOMAIN	-	D3
DIP DIRECTION	-	090°
ROCK TYPE	-	volcanics
WATER CONDITION	-	favourable
OVERALL PIT ANGLE	-	50°
BERM FACE ANGLE	-	66°

STABILITY EVALUATION:

Design sector S-8 is expected to be the most stable sector in the Main Zone pit. Available data indicates that all major structures plunge into the wall. No multi-berm failures are expected. Some berm failures will likely develop on random jointing but no unfavourable structural trends that would combine to form steeply dipping wedges plunging into the pit have been identified. Pit wall angle is going to be controlled by the maximum angle that can be maintained on the berms. The design calls for a 66 berm face.

COMMENTS:

Groundwater conditions in this sector are expected to be very favourable, the entire wall should drain after a period of time. The main haul road will be developed in this design sector. The location is ideal because the probability of a multi-berm failure that would disrupt operations is remote. If refinements to the present trim blasting procedure result in a more intact rockmass that can maintain a 70° berm face then there will be further potential for steepening the overall slope in this sector.

The structural data used for the design was collected in sector S-5, several hundred meters away on the opposite side of the pit because no exposures were available in S-8 in 1984. Because the structural fabric can vary significantly over this distance additional line mapping must be carried out to finalize the ultimate pitwall design in S-8.

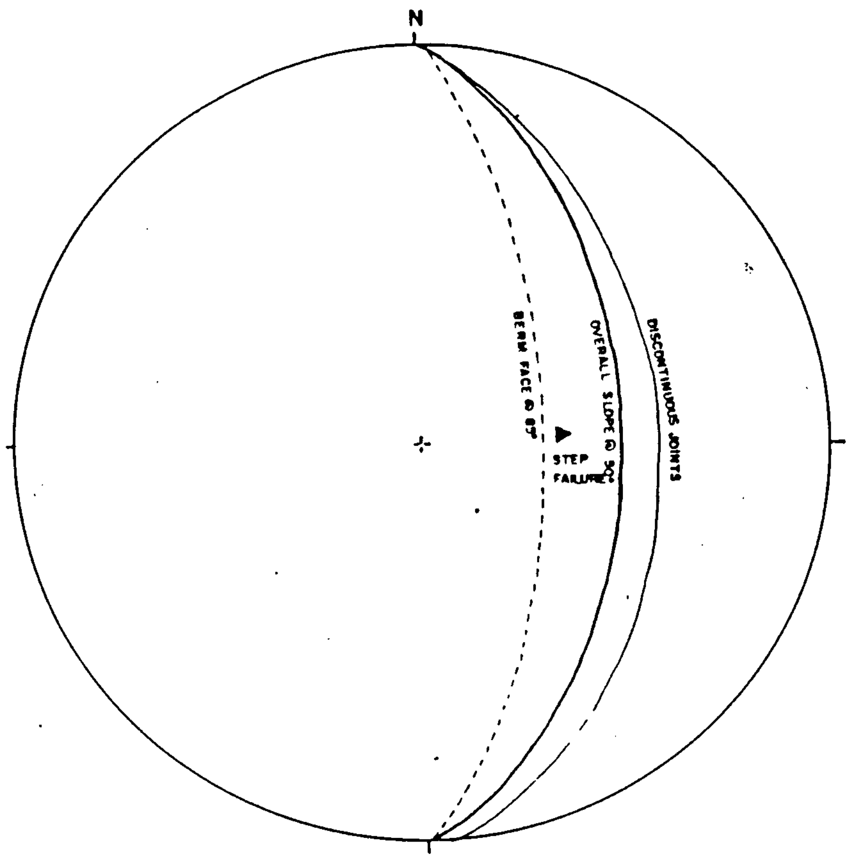


Figure 9.10
DESIGN SECTOR S-8

9.3.9 Design Sector S-9

STRUCTURAL DOMAIN	-	D4
DIP DIRECTION	-	270°
ROCK TYPE	-	gabbro/volcanics
WATER CONDITION	-	unfavourable
OVERALL PIT ANGLE	-	45°
BERM FACE ANGLE	-	60°

STABILITY EVALUATION:

Unlike in all other sectors where stability is controlled by either berm stability or potential for multi-berm failure, in sector S-9 both mechanisms have to be considered. Overall slope angle has to be maintained below 50 to prevent multi-berm wedge 3 from daylighting. Wedge 1 will daylight out of the overall slope; but will remain stable because it plunges at a shallow angle of 35 .

Berm stability will be controlled by failures on plane C . All three failure modes on plane C will daylight if the berm face exceeds 65°. Even at the shallower berm face angle of 60° several of the shallower planes in group C will daylight and cause failures, but these will be contained on the catchment berms. Because group B discontinuities strike almost dead parallel to the pit wall there is a natural tendency for the bermface to form along these planes of weakness. The shovel operators should make an effort to excavate to these joints as the final berm will then be smoother and there will be much less potential for ravelling.

COMMENTS:

Piezometer monitoring in the upper portions of this design sector indicates that the water table is 20 to 30 m below surface, a favourable water condition. However, deeper in the pit the water table will come to surface and drainage should then be installed to improve the stability of any multi-berm failure modes.

Groundwater inflows will not be large in this sector because it is bounded by relatively impervious gabbro on three sides. Pit walls in the lower portion of this sector will be in gabbro. Well points will have to be closely spaced to achieve adequate depressurization if piezometers indicate that dewatering is warranted.

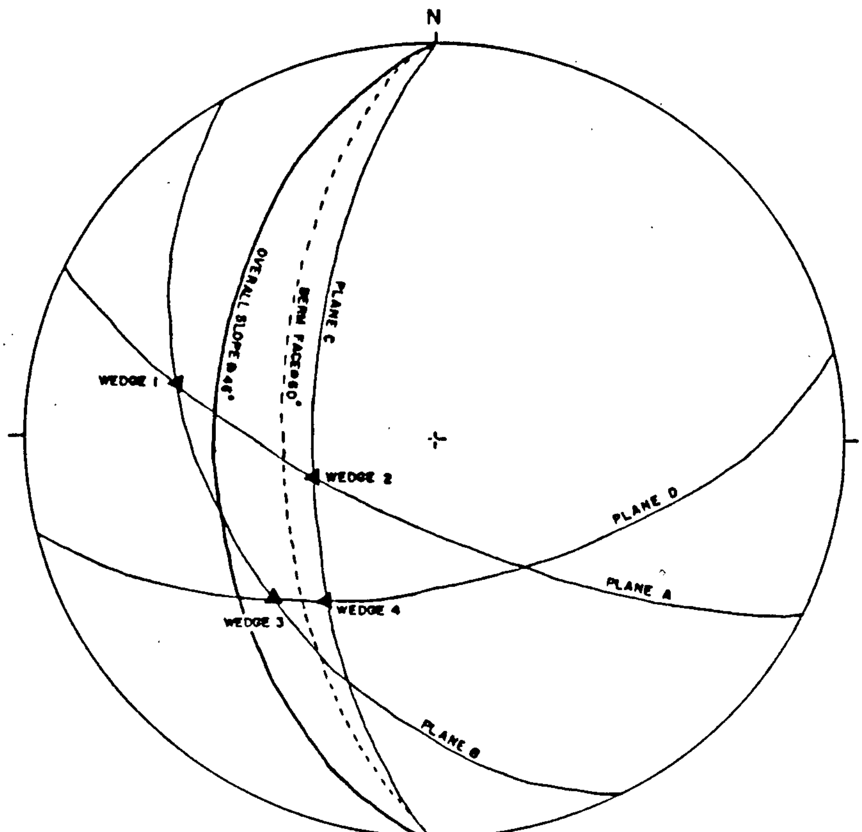


Figure 9.11
DESIGN SECTOR S-9

9.3.10 Design Sector S-10

STRUCTURAL DOMAIN	-	D4
DIP DIRECTION	-	225°
ROCK TYPE	-	volcanics
WATER CONDITION	-	moderately unfavourable
OVERALL PIT ANGLE	-	45°
BERM FACE ANGLE	-	59°

STABILITY EVALUATION:

Slope angle in this design sector is controlled exclusively by wedge failure 4. This "classical" wedge plunges directly out of the slope at 44°. A stability analysis on this wedge indicates that it is marginally stable when dry, F.O.S.=1.07 (again assuming ϕ=31°, c=10.5 kPa, height of 50 m). If the slope face exceeds 47° the wedge becomes unstable even when dry. Because the wedge is so marginally stable the overall slope in this design sector should be flattened to 44°, parallel to the peak angle of intersection.

Berm scale failures on wedges 3 and 4 must be expected. As the plunge angles of both wedges are shallow, wedges that daylight near the bottom of the berms will result in failures that will take out nearly the entire berm. The berm face has been designed at 59° to limit the volume of material generated by the berm failures.

COMMENTS:

Drainage is essential in this sector to increase the stability of wedges that will daylight out of the overall slope. As these wedges will plunge shallower than wedge 3 they are likely to be stable if the pit wall is maintained dry but will fail if significant water pressures are allowed to build up on the failure planes.

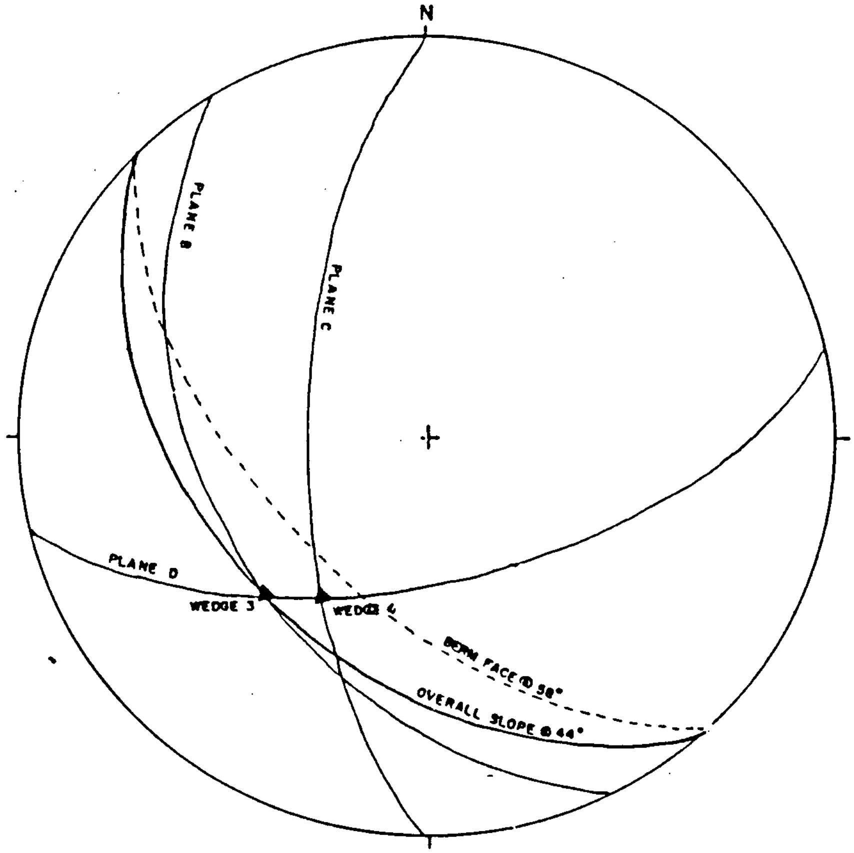


Figure 9.12
DESIGN SECTOR S-10

10.0 MONITORING

Monitoring of pit walls can be subdivided into three principal levels: 1) detection of instability, 2) determination of mechanics, and 3) mine and monitor. Monitoring techniques, equipment and personnel requirements vary considerably from level to level; therefore, each is discussed separately below.

10.1 LEVEL 1 MONITORING

The goal of Level 1 monitoring is to detect a major pitwall failure very soon after movement commences. This is most important for safety reasons as workers must not be exposed below an active failure unless it is being carefully monitored. Also, shear strength on the failure surface decreases with movement (from peak to residual). When stabilization is required, any stabilizing measures should be undertaken as soon after movement starts as possible. Any loss in shear strength due to movement must be replaced by additional artificial support, unnecessesarily increasing the cost of the stabilizing measures.

The most practical method of detecting instability is observation of the walls for tension cracks. Cracks will always appear long before the actual slide occurs if it is large, unless the slide is triggered by a large seismic event, e.g. an earthquake or a large non-delayed production blast. To increase the odds of detecting a failure all mine department staff should be instructed about the importance of reporting any observed crack to the pit shifter and stability engineer immediately!

Some areas of the pit may not be accessed for long periods of time during operations, especially the upper slopes of the pit when mining progresses to greater depth. In order to detect any failures in these areas a regular inspection should be carried out by the stability engineer, especially along all infrequently travelled pit benches, and on the pit crest. During the inspection, he should be looking for cracks, evidence of increased ravelling, abnormal seepage out of the walls, and leakage from surface diversion ditches. The inspection should be carried out weekly during the spring run-off and times of abnormally heavy rainfall. During more favourable climatic conditions the inspection can be carried out on a monthly basis.

Level 1 EDM (electronic distance measuring) is being used in several larger mines in British Columbia. Prisms are placed around the pit crest and along a bench mid-slope and monitored on a daily basis from permanent monitoring installations. This approach is very effective but requires a full time stability technician to carry out the surveys. A Level 1 EDM program proved practical in the Southern Tail pit and it is recommended that the same procedures be used in the Main Zone. These include monitoring of strategically placed prisms on a twice weekly basis.

Groundwater pressures should also be monitored as part of Level 1 stability monitoring to establish the magnitude of destabilizing pressures and to evaluate the effectiveness of any dewatering programs. All accessible piezometers should be monitored on a weekly basis during the spring run-off period when piezometric heads will be greatest. During the summer and fall months of 1985 the piezometers should be monitored on a monthly basis to verify the seasonal trends observed in the limited 1984 piezometer monitoring program. During the winter, most piezometers will not be accessible so only one piezometer should be measured to gain at least some information about piezometric response in that season. Six additional piezometers should be installed on 1280 bench of the Main Zone ultimate pit to determine pore pressures in the wall near the bottom of the pit. The piezometers should lie in vertical section below existing piezometers on slopes above.

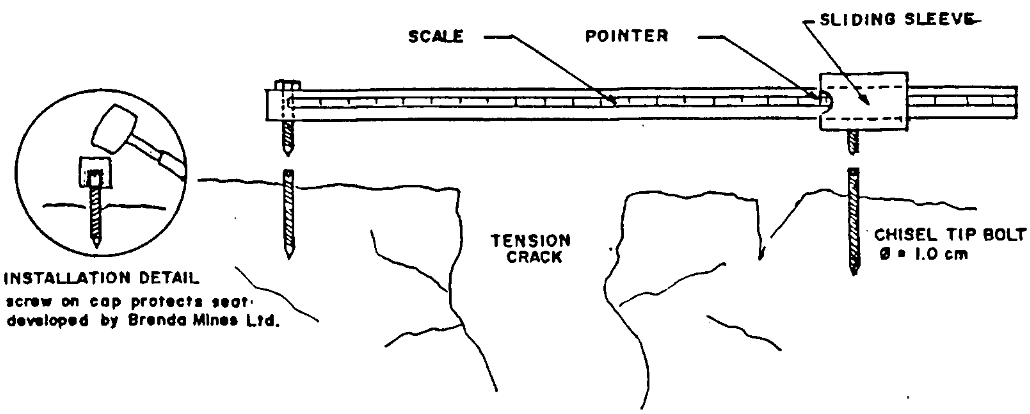
10.2 LEVEL 2 MONITORING

The objectives of Level 2 monitoring are 1) to determine the size of the slide, 2) to establish the failure mode and location of failure surfaces, 3) to obtain approximate magnitudes of groundwater pressures acting on the failure surfaces, and 4) to determine the rate of movement of the slide. At completion of the Level 2 program there should be adequate information available to make a decision on whether a mine and monitor program can be carried out below the failure or whether the area should be closed until the slide comes down. If in a critical area, e.g. above the main haul road, the Level 2 monitoring should provide sufficient data for the design of a stabilization program. The complete monitoring program should be completed in one week. Access to the slide area should be restricted to engineering staff carrying out the study until it is determined that there is no immediate danger of collapse. Stabilization should commence as soon as this assessment is made if the failure is in a critical area.

Level 2 monitoring should begin immediately after a major crack observation is reported. The stability engineer and mine geologist should examine the area looking for additional cracks, evidence of groundwater seepage, and any geologic evidence that will indicate what geologic structures are controlling the failure. All observations should be recorded.

A simple displacement monitoring station should be set up at the uppermost tension crack. The required apparatus is illustrated in Figure 10.1. Readings should be taken every 12 hours or less if the rate of movement exceeds 1 cm/day. If it is less than 1 cm/day daily readings will provide sufficient information. The results should be plotted on daily displacement vs. time and cummulative displacement vs.

Figure 10.1 TENSION CRACK DISPLACEMENT MONITORING TOOL



time graphs after every reading. Results should be interpreted by the stability engineer on a daily basis.

An EDM monitoring program should also be started. The program will consist of an adequate number of prisms located at regular spacings along the center line of the failure. One of the prisms should be located well above the tension crack where no movement is expected. The displacement monitoring guidelines should also be followed in deciding how often EDM readings should be taken. Results should again be plotted and evaluated as soon as they are taken.

Piezometers in the vicinity of the failure should be read to establish the magnitude of water pressures acting on the failure surface. If existing piezometers are not ideally situated, one or two holes should be drilled along the centerline of the slide if possible. The holes should be drilled sufficiently deep to penetrate 5m below the expected location of the failure surface.

Sonde soundings should be taken in the holes to establish the exact position of the failure plane. A sonde is a stiff steel rod at least 2 m in length that is lowered down the bore hole. It will jam when sufficient offset has occured to prevent it from going down or coming back up the hole. The piezometer can be used as a sonde casing, with the understanding that water level readings may be affected slightly by the sonde.

At the completion of the Level 2 monitoring program the stability engineer will be able to advise management on the size of the slide, whether a mine and monitor program can be safely completed, if stabilization is possible; and what is the recommended stabilization method. A safe, cost efficient mining program can then be developed by the mine engineering department.

Figure 10.2 is a cross section through a slide that illustrates each component of the Level 2 monitoring program.

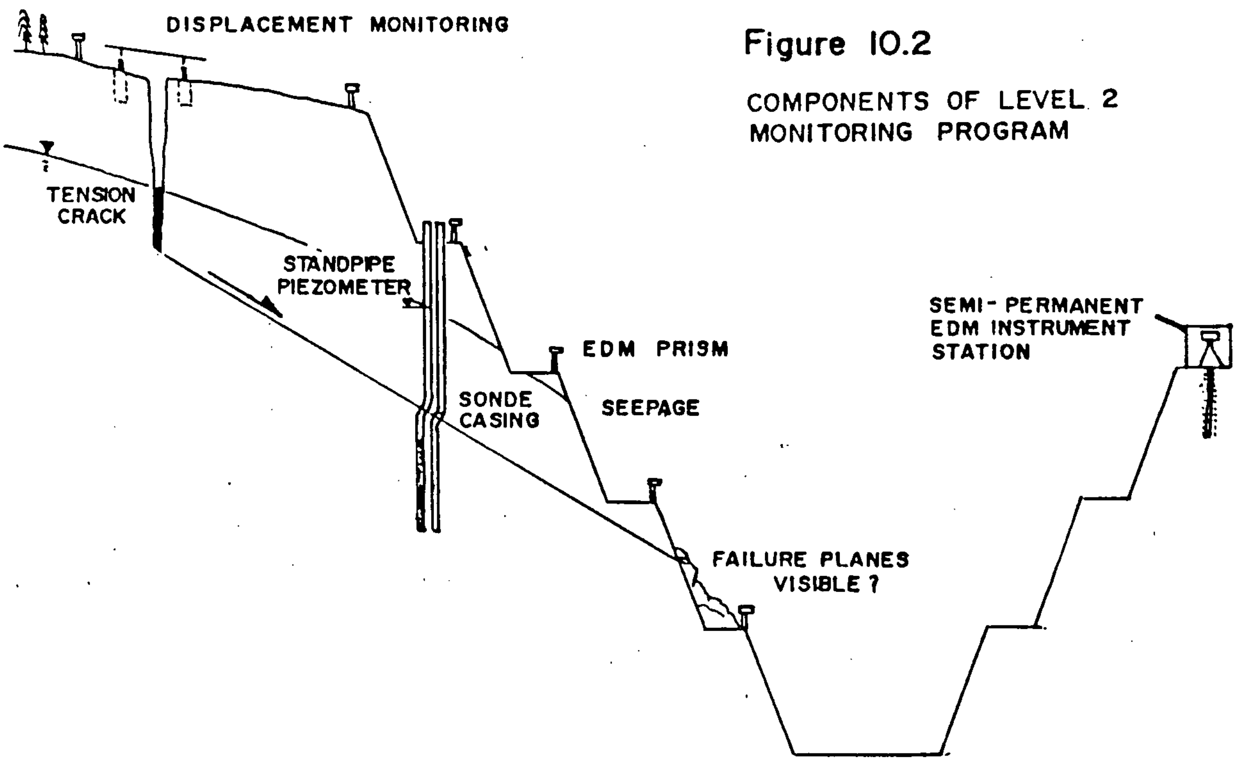


Figure 10.2

COMPONENTS OF LEVEL 2 MONITORING PROGRAM

10.3 LEVEL 3 MONITORING

Mining can be carried out safely under a failing rock slope to within several days of failure provided a number of precautions are taken. These precautions form the Level 3 monitoring program. Called "Mine & Monitor", Level 3 monitoring provides detailed information on the rates of movement in the failing rockmass. Level 3 monitoring requires precise instrumentation that may include several of the following: 1) pulley monitoring system with limit switches, 2) EDM, 3) potentiometers, 4) inclonometers, 5) shear strips, and 6) telemmetry.

A large slide will give warning before actual failure occurs. The objective of Level 3 monitoring is to detect the warning signal and sound the alarm well before failure occurs. The warning comes as a gradual increase in the rate of movement of the unstable rockmass. To detect the acceleration displacement readings must be taken and evaluated on a daily basis. The readings should be plotted on a daily displacement vs. time graph and on a cummulative displacement vs. time graph (figures 10.3 & 10.4). If the slopes of the two graphs become steeper the unstable rockmass is accelerating; becoming more unstable and approaching closer to failure. "It is giving the ALARM".

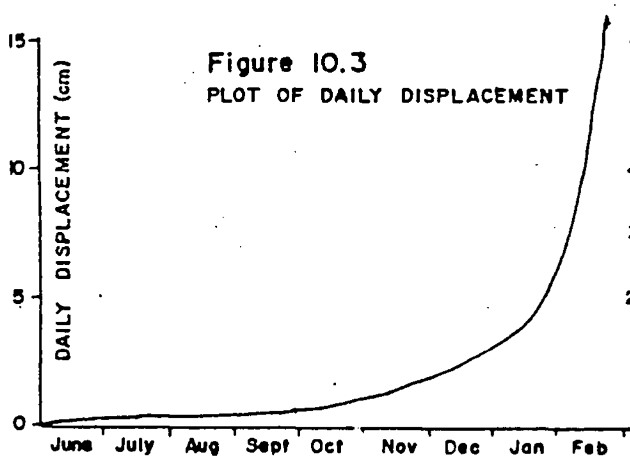


Figure 10.3
PLOT OF DAILY DISPLACEMENT

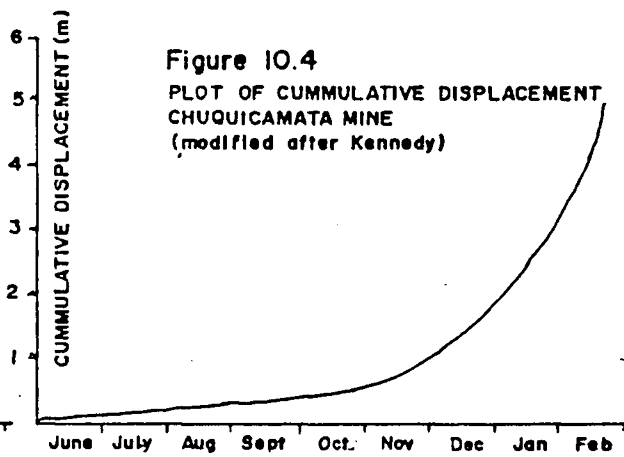


Figure 10.4
PLOT OF CUMMULATIVE DISPLACEMENT
CHUQUICAMATA MINE
(modified after Kennedy)

If the rate of movement exceeds 7.5 cm per day the slide path and runout zone should be cleared of all workers and access to the area is to be forbidden until failure occurs or the rate of movement again drops well below the critical level. It is very important to realize that the above rule only applies to slides exceeding 100,000 m^3 in volume. Smaller slides can occur very rapidly and with little warning.

Because the principal aim of Level 3 monitoring is to protect the equipment operators and support staff working below the slide it is important that all pit workers have a good understanding of how the monitoring systems work, what type of alarm is given, and what to do if the alarm is sounded. Employees working in the pit should be involved in the mine and monitor program from the very start and kept fully informed on daily monitoring results and changes to the systems. To create a feeling of trust most of the instrumentation should be kept simple and all readings from sophisticated instrumentation should be interpreted and plotted on a simple displacement vs. time graph.

The pulley monitoring system should be used as the primary method of Level 3 monitoring because it is simple, effective, and can be used and observed by the pit employees. The system consists of a steel wire that leads from a sound anchor in the unstable rockmass over one or more pulleys to a limit switch anchored in stable rock. The wire is kept taught with a counter weight. If movement occurs the weight is pulled upward. The limit switch is closed once displacement exceeds a predetermined magnitude, e.g. 1.0 cm per 8 hour shift. The limit switch should activate some form of alarm, preferably a flashing light and a siren. The limit switch should be reset at the start of every shift and a pointer should be visible on the mechanism to indicate the amount of displacement to any interested employee.

EDM monitoring of the hubs established during Level 2 should be continued on a daily basis. The displacements must be evaluated and plotted the same day as the readings are taken for the program to be effective. If more than one area of the pit becomes unstable at one time then monitoring will become a major task and may require the appointment of a full time stability technician. Alternately, a computerized data acquisition system can be added to the AGA Geodimeter so readings can be stored electronically and then downloaded to the computer. Software can be developed by the mine engineering department to automatically reduce the data and calculate the displacement of each target, update the displacement graphs, and give warning of excessive movement. Again, the data must be analyzed and plotted on the day it is taken.

More sophisticated monitoring systems that utilize rotating potentiometers, inclonometers, telemmetry, and computerized data reduction have been developed by several mines. Such systems require a highly specialized workforce during development of the system, and often during maintenance. Because Equity Silver is in a remote location and in an area of severe climate it is recommended that a sophisticated electronic monitoring system not be used at present. Recent advances in microprocessor technology may make such a system more reliable and affordable in the near future in which case it should be evaluated. A remote monitoring - telemmetry system was used to monitor slides at Brenda Mines Ltd. The system is discussed in detail by Blackwell et.al., 1984.

One of the best examples of mine and monitor technique occured in 1969 at Chuquicamata Copper Mine, Chile (Kennedy, 1969). It became evident in late 1968 that a major slope failure was developing that would take out the only haulage railway out of the pit. Ore was stockpiled and work began on rerouting of the railway. Displacement measurements indicated that the rate of movement was increasing (see Figs. 10.3 & 10.4). On January 13, 1969 a failure date of February 18 was predicted. Subsequent monitoring indicated that rates of movement continued to increase and a decision was made to shut down the pit on February 17 as failure appeared imminent. The failure occured only hours later. In all the mine was shut down for only 65 hours. Such precision cannot be expected in most cases but work can continue safely under an active slide for a considerable period of time provided adequate Level 3 monitoring is also performed.

A more local example of the mine and monitor technique as applied at Steep Rock Mines Ltd. in Ontario is described by Brawner et. al. (1975). A large toppling failure was discovered above an active mining area. Because valuable ore was located below the unstable mass the mine and monitor technique was applied in an attempt to safely recover the ore before the failure occured. Instrumentation that was used included triangulation, EDM, wire extensometers with limit switches, crack separation callipers and a seismic unit. The EDM and wire extensometers proved to be the most effective instrumentation. The seismic unit did not work because of background noise due to mining activity.

However, the most important lesson from this example is not about instrumentation. The paper describes in detail how the mine staff and the Ontario Department of Mines were kept informed and involved in the mine and monitor program. A union member maintained a lookout near the failure on a 24 hour basis. His responsibility was to detect any ravelling or other sign of impending failure. A movement chart was was kept in the mine dry and updated on a daily basis to keep all staff well informed on the status of stability of the slide.

As a result of the excellent cooperation between the consultant, the mine and the mines´ inspector the mine and monitor program at Steep Rock proved very successful.

11.0 CONTINUING PROGRAM

The geotechnical investigation into improving pit wall stability in the Main Zone must not end with the completion of this report. So far the investigation has examined each of the five geotechnical categories that have the greatest impact on stability and safety in the pit. The categories are: 1) influence of discontinuities, 2) groundwater, 3) shear strength parameters, 4) trim blasting, and 5) monitoring. The subjects that have the greatest potential for improving stability were examined in great detail while others were reviewed only briefly because of time constraints imposed on the research. Valuable work remains to be done in each of the categories. An attempt was made to provide guidelines in each of the report sections as to the direction that further investigation should follow. This section is a summary of those guidelines and briefly outlines the goals that the particular research should achieve. The sections are reviewed in the same numerical order listed above.

11.1 DISCONTINUITIES

The single most important topic that must be investigated in the continuing program is the definition of geologic structure along the west wall of the ultimate pit. So far, the pit design has been based on data collected along exposures of the east, and to a lesser extent, north and south walls. Very little of the west wall was exposed in 1984 so the design is based on the assumption that the same structural trends observed elsewhere in the pit continue in that area. Although there is geologic evidence to suggest that this is likely it is very important that all exposures on the west wall be mapped and the structural data evaluated to confirm that the trends do continue.

Structural mapping should also continue on all other benches in the Main Zone. The program does not have to be as detailed as the line mapping carried out during 1984. Mapping should only be carried out on every second pit berm (i.e. every 40 m). Because the major failures that will influence pit stability are going to be controlled by "major discontinuities" collection of structural data should focus on faults, shears, dykes and joints exceeding 6 m in length.

11.2 GROUNDWATER

The investigation into groundwater conditions in the Main Zone has indicated that it should be possible to dewater the pit, thereby achieving improved slope stability and reduced operating costs, especially costs of blasting and equipment maintenance.

Several dewatering systems were reviewed in the groundwater hydrology report and the most promising drainage system was identified. The WIP/GraD or alternate drainage system will have to be optimized to get maximum drawdown at minimum cost. This program will require detailed field observation of the influence of well spacing, pumping rate and well location in relation to geologic structure on the rate of drawdown. Piezometer installation and weekly monitoring will form a major component of the study. Pump tests should also be carried out in the first few wells to better define the hydrologic variables: transmissivity, specific storage, and aquifer geometry.

11.3 SHEAR STRENGTH PARAMETERS

Very little work has been carried out in the study of shear parameters in the Main Zone with the exception of a detailed point load testing program. The initial studies indicate that friction angle is in the range of 30 to 35 degrees and a small amount of cohesion is present under low stress conditions. By further investigation into shear strength it may be possible to narrow the range of observed values, and more importantly, place greater confidence in the results. Once the shear strength parameters are well defined it will be possible to do a stability analysis on the potential failure modes in each design sector. At present, the east wall of the ultimate pit is designed to minimize the number of kinematically possible failure modes from daylighting. If the shear studies and subsequent analyses indicate that many of these wedges are stable then there may be limited potential for steepening. The potential is limited because berm scale failures are presently being observed and further steepening would result in additional failures with greater volume. As a result, the catchment capacity of the berms would soon be exceeded because larger volumes of debris would have to be stored in a smaller area. The reduction in storage area is due to the fact that parts of the lower berm would also fail.

It is important to know the magnitude of shear strength that will develop on the failure planes when a slide has to be stabilized. The quantity of support required must exceed the difference between the driving forces of gravity and water pressure and the stabilizing force of shear strength on the failure planes. The support requirement can be estimated accurately only if shear strength is known. Otherwise the support system must be based on worst case assumptions and will likely be overdesigned and unnecessarily expensive.

11.4 TRIM BLASTING

Considerable work has been done to date on refinement of the trim blast pattern to reduce the amount of blast induced damage to the rockmass in the final wall; however, further refinement is possible. It is recommended that a well organized trial program be initiated to to determine whether changes to the trim blast suggested in section 8.4 do indeed improve the condition of the final wall.

The program would consist of systematic changes to the trim blast pattern and careful docummentation of the results. The present condition of the walls would have to be evaluated first to serve as a reference. Still photography would be used extensively to record the condition of the final wall. High speed photography would be applied to study the mechanics of the blast and to evaluate behaviour of the muck pile. The researcher would have to work closely with the blasting crew to gather all neccessary information and ensure that each trim blast is drilled and loaded according to design. The goal of this program is to reduce blast damage to the volcanic rockmass so it will remain intact and maintain steeper berm face angles. If berm face angle can be inceased from the current 66° to 70° there is excellent potential for additional pit steepening in areas where the pit wall angle is controlled by stability of the individual berms. The possibility also exists that many of the berm scale wedge failures along the east wall berms in gabbro have been triggered by expansion of blast gases into the cracks. Pre-split blasting may reduce this damage, creating a cleaner and more stable ultimate wall.

11.5 MONITORING

Level 1 monitoring will be an ongoing part of the continuing geotechnical program. Initially, all pit personnel must be educated to keep an eye out for signs of instability and to report any observations. The monthly stability inspection and report procedures should be carefully organized so that they will require the very minimum amount of time to complete.

Level 2 monitoring will only be required when a failure occurs; however, response must be fast when it does occur so the stability engineer and surveyors should develop a set of guidelines that they will follow and all necessary equipment should be available for use on site.

Level 3 monitoring will begin several weeks after the failure is first reported so there will be adequate lead up time to it. It is therefore not practical to make extensive preparations for a Level 3 monitoring program because such a program is very site and failure type dependent. The monitoring procedure should be reviewed by the stability engineer and surveyors so that they are aware of the type and quantity of work that such a program will require.

12.0 CONCLUSION

The slope stability study of the Main Zone has established that steepening of the pit by as much as 5° should be possible in the west north, and south walls. Numerous unfavourably oriented discontinuities occur in the east wall; it is therefore recommended that it remain at 45°. The pit wall design program is based on an analysis of structural geology, shear strength of discontinuities, and groundwater conditions in the Main Zone.

All accessible benches in the Main Zone were line mapped in detail to collect sufficient information on the orientations of discontinuities that will control failure. Based on the data collected the Main Zone was divided into four structural domains, each domain having consistent rock type and consistent trends in the orientation of discontinuities. Because limited exposures existed on the west wall of the pit at the time of mapping design work in Domain 3 was based primarly on structural trends observed in adjacent domains. As a result, the structural trends used to determine the optimum pit wall angles are not sufficiently reliable to be used for a final design of the west wall. They do indicate that the geologic structure will be favourable, as does the regional structural geology; however, the structural trends and pit wall designs must be verified by further line mapping and analysis once additional exposures are uncovered in the west half of the interior pit. Only then can there be a commitment to the final ultimate pit wall angle in the west wall.

The Main Zone pit has been divided into 10 design sectors. The potential for full wall, berm scale, and step failure was carefully analyzed in each sector. Overall and berm face angles were selected to minimize the possibility of developing large areas of instability in the pit. However, as some controlled failures can be tolerated the pit wall angles were not designed to eliminate the possibility of a failure. Rather, all failure modes that appeared to have potential for causing stability problems were evaluated to determine the factor of safety and the probability of the two planes actually intersecting in the sector to form a wedge. The pit wall angle was reduced to prevent a particular failure mode from daylighting only if the analyses indicated that the wedge was unstable and a large number of the controlling discontinuities were observed in the design sector. By adopting this design approach the pit walls will not be unnecessarily overdesigned while maintaining a sufficiently high degree of stability.

Groundwater reduces stability of the pit walls. Presence of water in the pit also increases operating costs of blasting and equipment maintenance. To improve groundwater conditions and reduce the chance of failures in the pit it is recommended that an aggressive dewatering program be immplemented in the Main Zone. A detailed study of pit dewatering was carried out as part of this geotechnical investigation. Results indicate that some form of in-pit well system will be required to dewater the rockmass because of its relatively low permeability. In theory, the WIP/GraD drainage system that consists of free flowing gravity wells in the pit walls and pumping wells on the pit floor appears to be the technically optimum dewatering method for the Main Zone pit. However, a detailed evaluation and testing program will be required before a final decision is made on selecting the most practical drainage method.

The stability of most slope failures is very sensitive to the magnitude of shear strength developed on the failure surfaces. In order to carry out the stability analyses for pit design, reasonable values of friction angle and cohesion were required. A study was undertaken in the summer of 1984 to establish these parameters. The study consisted of point load tests, inclined slip tests, and back analyses of berm failures. The test results indicate that c=10.5 kPa, $\phi=31^\circ$ appear to be reasonable estimates of shear strength. A more detailed shear strength study should be undertaken to better define these important parameters.

Blast damage to the rockmass behind the ultimate wall can result in a much less stable pit because numerous cracks are opened up, the rockmass looses its intactness, and shear strength on existing discontinuities can drop from peak to residual levels. To reduce blast damage a good trim blasting program has been developed at Equity. A study of existing literature suggests that there is further potential for reducing blast damage. Key changes that deserve additional study include: 1) use of ANFO in line holes, 2) reduction of charge per hole, 3) reduction of burden, 4) changing trim pattern to match rock conditions, 5) inclusion of the Hercudet intiation system, and 6) changing the firing order.

As some small failures of the pit wall are anticipated an efficient monitoring program must be developed so the failures will be detected quickly and will not endanger regular operations in the pit, especially the wellfare of the workers. The monitoring program shall consists of three levels. Level 1 is designed to detect any instability in the pit. Level 2 will determine the nature of the failure and the degree of hazard that the failure presents. Level 3, or "mine and monitor" will allow safe operation under an active unstable slope.

Substantial work remains to be done in the geotechnical investigation of slope stability in the near future, especially in the areas of pit dewatering, structural design of the west wall, and improvements to the control blasting program. It has been the purpose of this report to provide some guidance as to the directions that the geotechnical work should follow in order to obtain maximum improvement of pit wall stability.

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APPENDIX A

A.1 STEREONETS FOR DOMAIN D1

D1A
433 OBSERVATIONS WITH TOTAL WEIGHT OF 433.0
COUNTING CIRCLE IS 1.% OF TOTAL AREA

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D1FL
  27 OBSERVATIONS WITH TOTAL WEIGHT OF     27.0
COUNTING CIRCLE IS  1.%  OF TOTAL AREA
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                                       3336 6AAAAA666663
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                    33666533  33333333633333333 33333333
                  36666AA6D66AAA6333333333333333 333333333
                  36AAAAAHD6AADDA3333    3333      3333333
                  36AA65DDDAAADD663
                  3666636DAADDA663333                 33333
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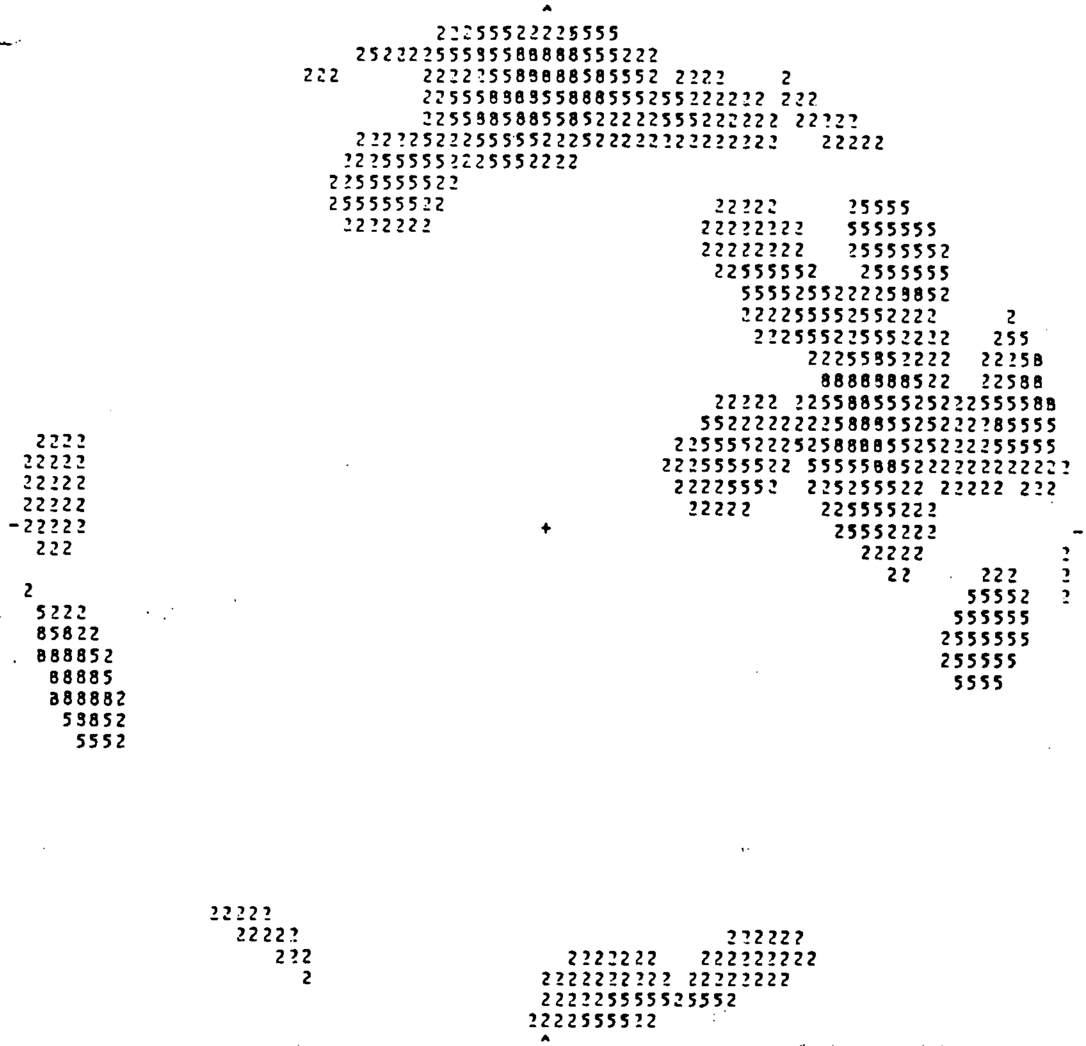
COUNTING CIRCLE IS 1.% OF TOTAL AREA



D15R
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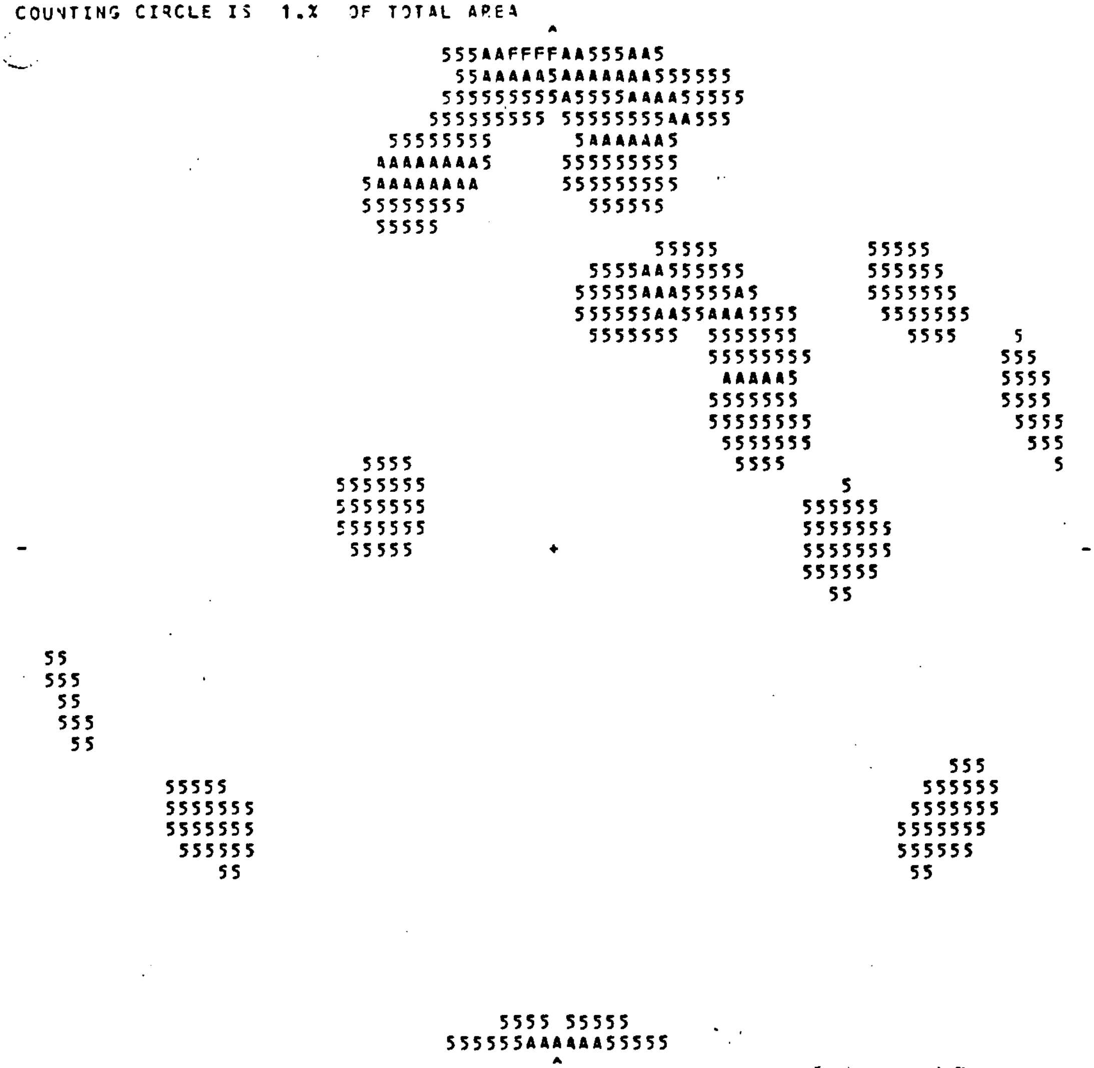

D1JN

349 OBSERVATIONS WITH TOTAL WEIGHT OF 349.0

COUNTING CIRCLE IS 1.% OF TOTAL AREA

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D1MJ
 105 OBSERVATIONS WITH TOTAL WEIGHT OF    105.0
COUNTING CIRCLE IS  1.%  OF TOTAL AREA
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DIMS
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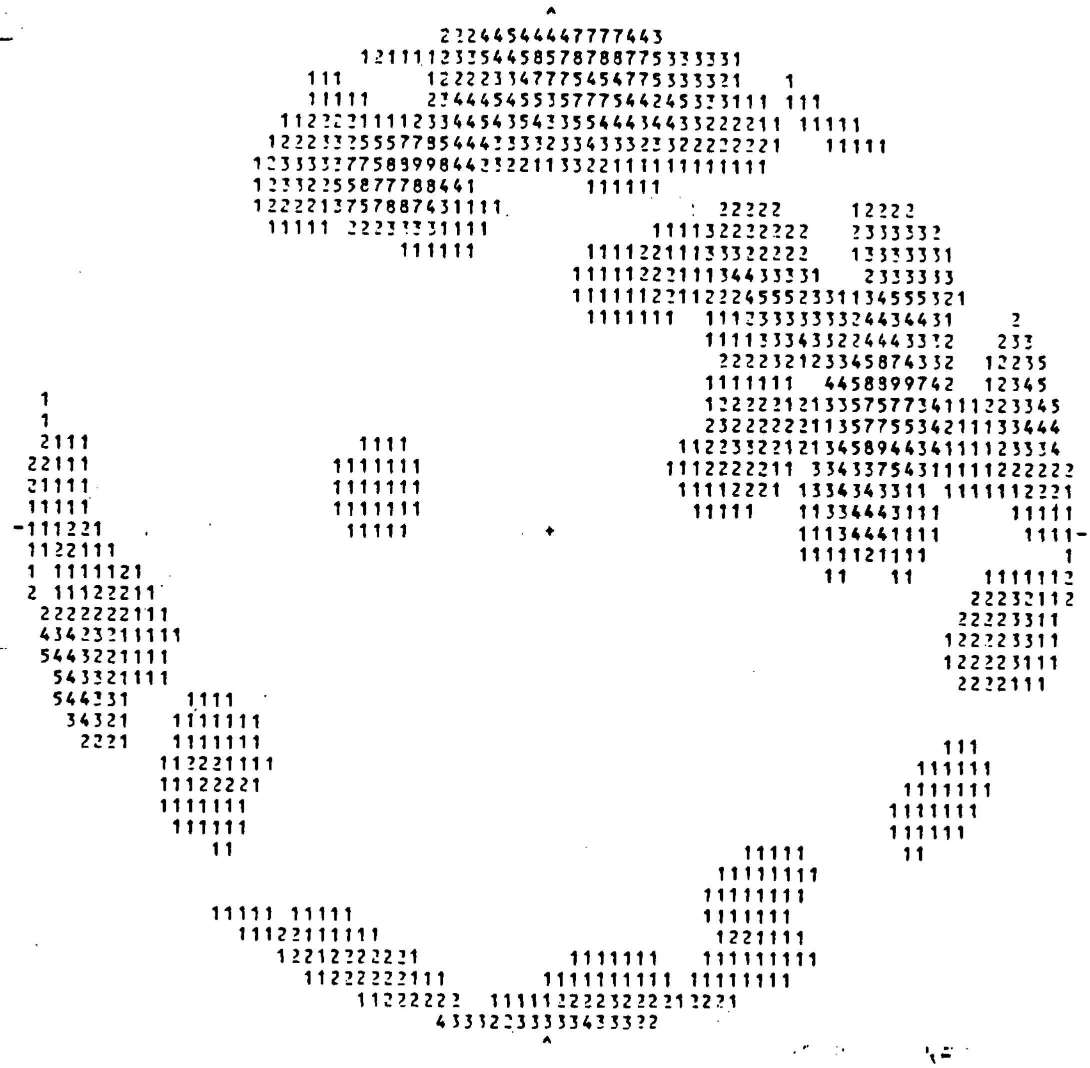
COUNTING CIRCLE IS 1.% OF TOTAL AREA



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D1MA
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COUNTING CIRCLE IS  1.%  OF TOTAL AREA
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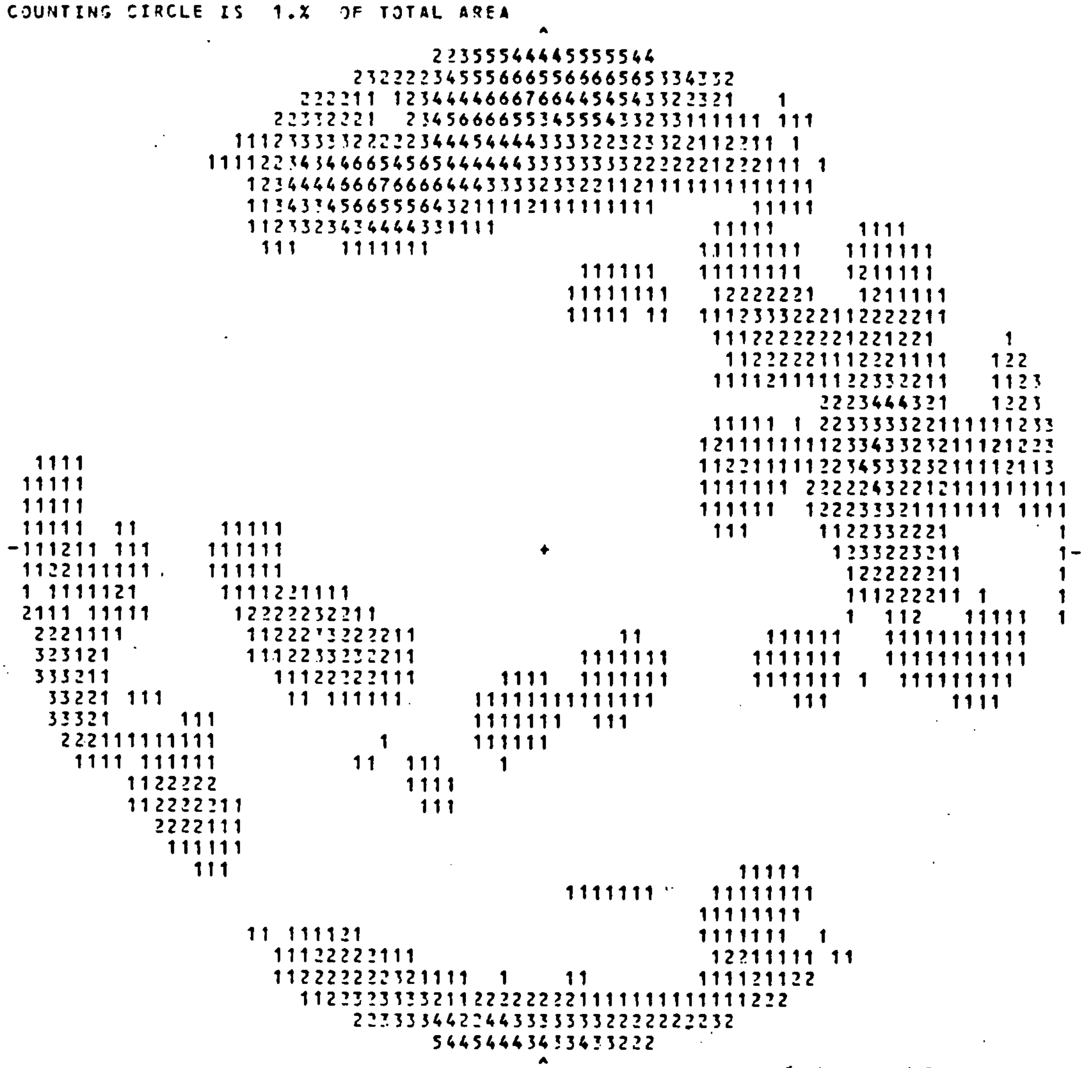

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A.2 STEREONETS FOR DOMAIN D2

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[illegible]24 OBSERVATIONS WITH TOTAL WEIGHT OF 1696.7
COUNTING CIRCLE IS 1.X OF TOTAL AREA

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  29 OBSERVATIONS WITH TOTAL WEIGHT OF     57.8
COUNTING CIRCLE IS  1.X  OF TOTAL AREA
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                         57899988807775511             2
                     3333335635555775333111           22222
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                   5998896662  22222 222222222     111133333
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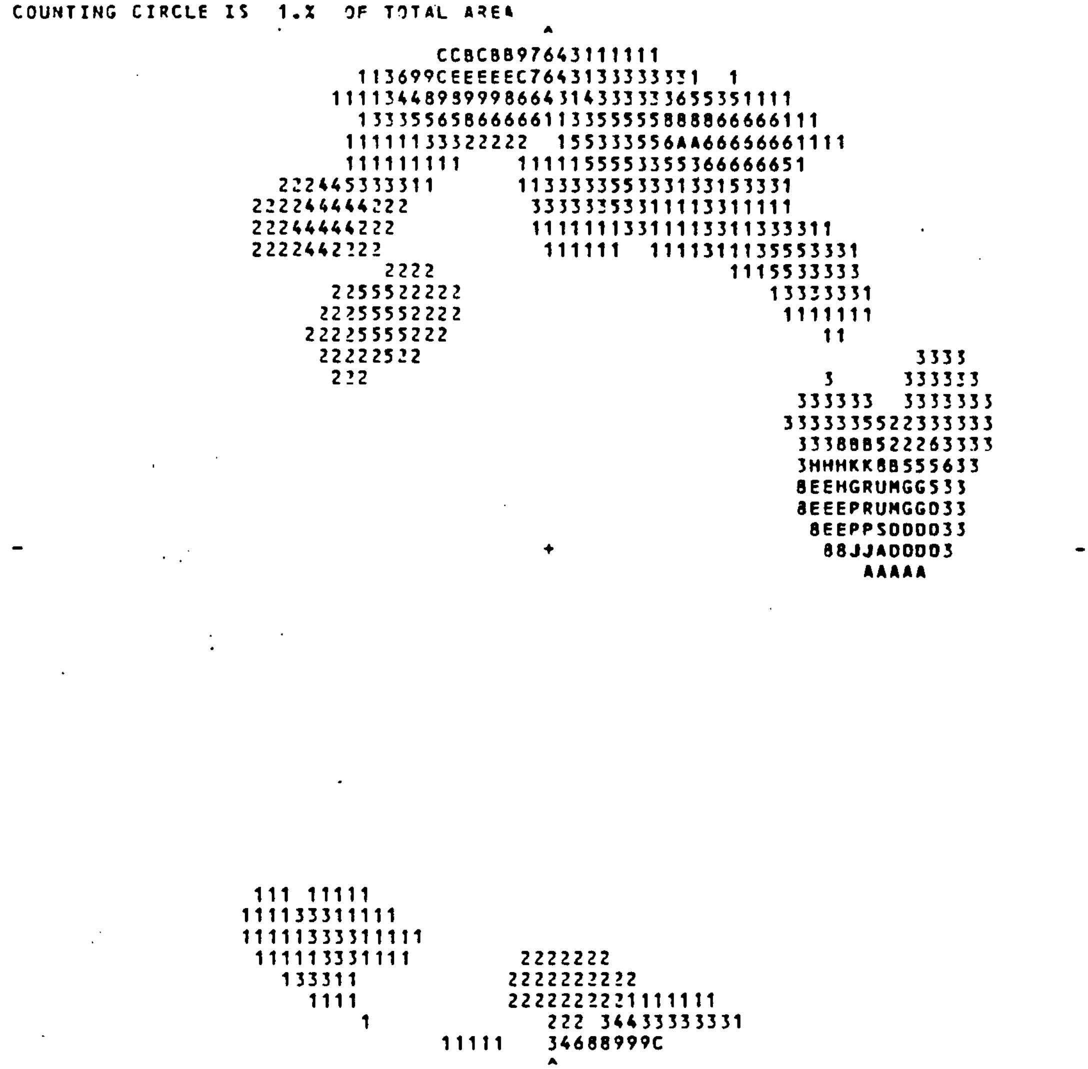

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COUNTING CIRCLE IS  1.X  OF TOTAL AREA
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                    2222256666445566777763111     111111111
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...15 OBSERVATIONS WITH TOTAL WEIGHT OF  1497.6
COUNTING CIRCLE IS  1.X  OF TOTAL AREA
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  11  1  11222222 111111
  2  11 1222233211111111
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 1  11  121222211111111
 1      112222121   11
          111111111
 2       112112211  1 11111
 4      11111111     1111
643     1 11111    1111
 31     111111   111 111
 211    111111      111
43211   11111
 21 111
```

```
            1
           1111
          11111111 1
          1111112111   1
        11111112211111
         1 122222222111
        1112233333322111
        1122224466544221111
       11145556666555531122222
        1234577866876542111455
        123467789AB987531 23666
        2246777ACCDC9643124766
       11145677BBCCBA843246776-
   111111213356ABBAA9854334666
   1111212233567 89AA6544454665
   1222231223453556554555 55442
 1223333212123333112324545431
 1334332323233422111123454431
112333433322222221 2234453322
 113444332222222111122234322
11112211111112221112112122231
 11121111111211 11212122222
    111     11   11 1121222
                  111 1111
                      111
        111111       111
      11111111  11 11
      1111111        1
      111111 11111
           1111111
          11 111
          1111
          1
```

```
 :MJ
 222 OBSERVATIONS WITH TOTAL WEIGHT OF    328.1
COUNTING CIRCLE IS  1.X  OF TOTAL AREA
                                        ^
                                111


                     1
                 2211111
               221112111111   111
             1211112221111111111
            1111122211112121111211
            11121 11111 11111111111
         111 11111       11111211
        2 1  11            111111
       11                                                     11 11
      11                                                     11111 1
     1      1111                                            11111111
    1      1122221                                          111222211
       11111222222                                       111112223322211
   21122113222222                                        111 1223222322121
  211221111222221                                        11113344444443211
  211221111 11111     1111                               1112233548776543211
  21 11111             11111                              1257778897678952111
 1               111   11111                              135678B899BA99632111111
 2             1111112  11                                14568CCDCEECBA64211111
 21            1111111         11111                       25898CFIHIGB954222221
-11            1111111        1111111   +                   579BCGGFGEDB55355322-
  1             11111         1111111                111    346AFHFEDCA75444532
 11                           1111111               111111   236AABCCC6868736652
 1                              111                11124422    44578767689997632
                                                   1224444211 12333222448989762
                                                   1345443321 113322113357A7652
                                                  112444332    21111 1236664422
                                                 11213555321          12234432
                                                 1111122111                33
                                                   11111                1221
                                                                       12221
                                                                       1111
                                                                       122
                                                             11       122
                                                        11  1111 11  111
                                                       11111111 11    1
                                                       1111111 21111
                                                             1111111
                                                            1 1111
                                                           11111
                                                            1

                                               111
                                              111
                                        ^
```

```
 2MS
 103 OBSERVATIONS WITH TOTAL WEIGHT OF    190.2
COUNTING CIRCLE IS  1.X  OF TOTAL AREA
                                        ^
                                7779886442211
                          444676899A8885431 1111111   1
                      1333445545766776554312111111222121111
                     1111334543455666665212211111244433444111
                 1   11133322234322333212331111123444444442111
                      21223322111111112223333311121344444431
                    2335543332112222211123322331111111132222
                  22336666521211111122 2222223111 11111111
                  22566665221 11111111      112211  11111111
                  3455775321  11211        111  111 1   1111111
               11115545553  1111                         1111111
         11111111113346565522211111                       111111
        11111111112446676444332221
       1111111111113456AAA5433222                                    111
       111111      2279AAAA332211                                   333311
       1111        1667788761111                             1   1133333311
                    6666655                                 1111122235533331
                     55555                                 11111144334553331
                                                           111348655353333
                                                           1555AA78664422
                                                           24458CDAAA511
                                                           2444ABDA88711
             1111                                          68AAEHC777411
            111111                      +                 66688CC344441         -
           1111111                                        66666693333
            111111                                         666666
             1111                                           666

                                                                     55
                                                                   555555
                                                                  5555555
                                                                  5555555
                                                                  555555
                                                                   555

                                                              1111
                     111                                   21111111
                      1111                                21111111
                  1    111                               1111111
                    1111                    222111111 11  111
                      1111              1112222332444211121
                          1            11111456565555433
                                     111334667879
                                        ^
```

TMA
325 OBSERVATIONS WITH TOTAL WEIGHT OF 1013.3
COUNTING CIRCLE IS 1.% OF TOTAL AREA

A.3 STEREONETS FOR DOMAIN D3

```
D3A
 357 OBSERVATIONS WITH TOTAL WEIGHT OF  1162.4
COUNTING CIRCLE IS  1.X  OF TOTAL AREA
                                        ^
                                11 1 1
                           1111111111111 1      1111111
                          1 11111111111111 1     111111111
                           1111111111211111  1111112111111
                             11 1111112122111222222211
                              1    1111111111434333332
                                    1111111112344444443       1
                                        111222344434443221    11111
                                         122111133444322111 1111111  11
                                       111123433333222211111 11111111  1
                                     11111 35544463 111111  1111111111111
                                      11111333333331       1112211  11 111
                                           144444331       1112221111111111
                                          11 233321        1122222   1111211
                                          1  111111         122332111 1122221
                                           1111111       111112332221112233221
                                                         112221222122233433321
                11                                  11  11222223221146344454321
   1111                                              11  1222322214555886545434
  11111  1   11       1111                               12234434356599A9755345
  11112211                11                             11336453456739977545567
 2311121121      11 11   22211            1 1             1222444556777864544677
 4422232111     2111111 1221111            11             1122224334677434423767
-6542222331     1111111 1 11111         +  11             1111123223432233 12666-
 655332233      1111111     111                             2222112221    11456
 664232221      11222                                   11 11111222221    111333
 774331211      1111                                    11111  1111111    11111
  443332311     11111                                          11211111   11111
  2232221111    111111                                       1     111      1 1
  1122222111      111                                              1
   2121122111  1                                     1
   12111211111
    2  11 11111
           1111

         1
        1
          1           11
          11         1111
                     111





                          111
                                                1
                                        ^
```

```
D3FL
   6 OBSERVATIONS WITH TOTAL WEIGHT OF     16.2
COUNTING CIRCLE IS  1.%  OF TOTAL AREA
                                         ^



                                            88
                                         88888888
                                      88 8HHH88888
                                   88888HHPHHHH888
                                   8888HHHH9HHHH
                                   88888HHH99999
                                     8888 9999


                               GGG3G
                              GGGG3GG
                              GGGG3GG
                              GGGG3GG


-                                        +                                        -


                                                                   HHHH
                                                                  HHHHHH
                                                                 HHHHHHH
                                                                 HHHHHHH
                                                                  HHHHH









                                         ^
```

```
D3CN
  32 OBSERVATIONS WITH TOTAL WEIGHT OF     67.1
COUNTING CIRCLE IS  1.X  OF TOTAL AREA
                                       ^
                               44244557554211111
                         11112245577877587554421111
                       133333332145577788575442221
                      111344433114444577752211111
                     11133333313311113333  111
                      113333333331131111111
                        11133331113311111111    1111111   222222
                           11111111111111111111 11144444  2222222
                             111        244111113466644445668995222
                                      22244441144466666458669CC99522
                                   22444424444144224444455990D99953
                                  22224442222   22222244266990DD6333
                                  22222244JJHHHH  2222  33669II677744
                                   222222HHHHHHHH        3399E99944444
                                         HHHHHHHHH         EEEEB9944444
                                          HHHHHHH         55BBBBB994444
                                                          55BBBBBB4444
                                                           5555BB
                                                              55


 -                                     +                                      -



                                                                 11111
                        55555                                   111111
                       5555555                                 1111111
                       55555555                                111111
                        5555555                                 1111
                          555
                      AAA
                     AAAAAAA
                     AAAAAAAA
                      AAAAAAA
                        AAAA


                                           12211111111
                               11111   122222224
                                       ^
```

```
D3SR
  17 OBSERVATIONS WITH TOTAL WEIGHT OF     30.5
COUNTING CIRCLE IS  1.X  OF TOTAL AREA
                                             ^
                                     333966633
                               36999369DDDDD99D6663333
                           3336999999GDGDGGGGD666633333
                           336666696699DDDJJJDAA6633333
                           336563333 6566AAA6633333  ****
                                   33356AAA6333333**     .*
                                  3333333373333344         3
                                  33333333     44444      33
                                    3333       444444888333
                                                 444444



-                                            +                                             -
                                                                              AAA
                                                                             AAAAA
                                                                            AAAAAAA
                                                                            AAAAAA
                                                                            AAAAAA
                                                                             AAA




                                            333333
                                          333333333
                                          333333366
                                           33666663333
                                            3333333333    33
                                             333333333
                                             ^
```

D3JN
298 OBSERVATIONS WITH TOTAL WEIGHT OF 1043.4
COUNTING CIRCLE IS 1.% OF TOTAL AREA

```
D3MJ
  73 OBSERVATIONS WITH TOTAL WEIGHT OF    285.5
COUNTING CIRCLE IS  1.X  OF TOTAL AREA
                                        ^
                                111
                          111    11                3333333
                                   11 11            643333333
                                  1111111  111        4444444
                                 1111122211 11        1
                                   122222111144445444      11111
                      1             111111111133444444    11111111
                    1   111          2211111  3334468844411111111
                       111     1111111132222    3363844444211111
                     11111     1111111223333653333113333442  111
                                1111132222266633333 1333332
                                      3222266543333                 111
                                       11111553333            11111     1111
                                          1                  1111444    11111
                                                              11444442   11111
                                                              125555522   11111
                                                             3315555486333  11
                                                           33334888448633354444
                                                           33377885GCD3877744444
                                                           337AAAGDHHC3F87984444
      222                                                   AAEEIBGGGC3GF77744444
     22222                                    333333       3777FFFJJJFCC83384444
    222222                                   33333333      33777BJEEFF9C73444443
-   222222                              +    33333333      33377BEAACG9 344
    222222                                    333333       11557766777411
     2222                                                1111114367777422111
                                                         1111111434441222111 1
                                                         1111111 33411221111
                                                           1111     11 11111
                                                                       111

   2                                                   1111
    1                                                 111
     1
      1









                                                     11
                          333                       11111
                                        ^
```

```
D3MS
  55 OBSERVATIONS WITH TOTAL WEIGHT OF   113.9
COUNTING CIRCLE IS  1.%  OF TOTAL AREA
                                        ^
                                332545554321
                          12334246777877686553321
                        13444444667778889956543222
                       122344533336667897763311111
                      112332222 3322334332  11B9AAAA
                         1111122232343211111 AABBBBBBA
                            12221112221111133BCCCCCCCC21    111111
                             11111111   2233333ECDDCD3322   1111111
                               11    11 1366333344444622222335553111
                                  11111334446633432333332253357755311
                                  1122555524444122112222222557755532
                                  1222255542222  11111122133557774222
                                   12222144CCAAAA  1111   2233588444422
                                    111111AAAAAAAA         2255855522222
                                          AAAAAAAAA          988865522222
                                           AAAAAAA          3366666552222
                              22222                         336666662222
                             2222222                         333366
                             2222222                            33
                             2222222



-                                       +               55555                    -
                                                       5555555
                                                       5555555       222
                                                       555555     2255222
                                                                 225555222
                                                                222555522
                                                                222555522
                                                                 222552

                        33333
                       3333333
                       33333333
                        3333333
                          333
                      555
                     5555555
                     55555555
                      5555555
                        5555

                                            11
                                        11111
                                            12211    11
                                        122222223
                                        ^
```

```
D3MA
 123 OBSERVATIONS WITH TOTAL WEIGHT OF    402.4
COUNTING CIRCLE IS  1.%  OF TOTAL AREA
                                         ^
                                 1111111111
                           11111112222222321111 3333222
                         1111111112232333121111332222222
                         1111111111222333322111111 3333333
                          11  11 11112232221 11333333
                               1111113222211115566666663   11
                                    11111111226666666661   1111111
                                 11111121111221136656897433  111111
                                1111  112123311  44577433332112221
                                   11112333336533432233333442112221
                                 1111233333266643333 23333321122111
                                   111133333355432233      11112221
                                     1111111147655522        112331111
                                           33222222          1223444    1
                                           222222222          33554442   1  11
                                            2222222           135555532     111
                                                             3325555465332  11
                                                           22233667425422223333
                                                           222555648895855533333
                                                           22577789CC89885953333
      111                                                   77AAD8BBBB89B855533333
     11111                                 222222          2555B98EEEB9882253333
    111111                                22222222         225558DAABB9852333332
-   111111                           +    22222222      11444557A778B6 233         -
    111111                                 222222      111225555445553
     1111                                              12222223245555422 11
                                                       122222  323332322111
                                                                 2232233111
                                                                    222211
                                                                    112211
                                                                     11

   1
    1



                       111
                      1111111
                      11111111
                       1111111
                         1111

                           222                 1     111
                                              1111
                                         ^
```

A.4 STEREONETS FOR DOMAIN D4

D4A
275 OBSERVATIONS WITH TOTAL WEIGHT OF 343.4
COUNTING CIRCLE IS 1.% OF TOTAL AREA

```
D4FL
  47 OBSERVATIONS WITH TOTAL WEIGHT OF      73.2
COUNTING CIRCLE IS  1.%  OF TOTAL AREA
                                        ^
                                           111111
                                             11111111
                                        22223331221   33331
                               1111111 222233565855546A844431
                    11       1115555522 2244556897559BB8A5844433
                 11111331111114577775541  4444545777ACCA8778A63111
                1111133311111 5889875541   1111114666AACB79753111111
               11111133111111347788755         113555657B766244444111
               1111111111111344444441      111113313666687752474277774
                      11111333334331     11111113313446655687997866663
                    11133333111111        11111111111333447E89997B866633
                    133333331              111111      AAAAEE5747B663333
                     33333311           2222           69999EEB444 3333
                     1111111          22222222         2699999B22
                       111            22222222       222599666
                                       2222222       2222222
                                         222         2222222
                                                      222222


                                                                        AAA
                                                                    66 AAAAA
                                                                  6665HAAAAAA
-                                       +                        66665HHAAAAA   -
                                                                 666656HAAAA
                                                                  666666AAA
                                                                   665


                                                               222222
                                                             222444222
                                                            2222444222
                                                            2224444222
                                                            222444222
                                                                2


                                                                1111
                                                              333333
                                                            33333333
             1                                              3333333
               1                                           1333331

                    11
                      111
                            11111111
                                111111
                                        .
```

```
04CN
  25 OBSERVATIONS WITH TOTAL WEIGHT OF     44.1
COUNTING CIRCLE IS  1.X  OF TOTAL AREA
```

```
D4SR
  10 OBSERVATIONS WITH TOTAL WEIGHT OF     13.6
COUNTING CIRCLE IS  1.%  OF TOTAL AREA
                                        ^
                                  7777777  7FFFFF
                                7777777777FFF000000G
                                777777777 7FF0000000GB
                                   77        F000008888
                                                 888K888
                                                   88888888
                                               9999LL8888888
                                              9999999LL88888
                          999999              999999999 88
                         99999999               999999
                        99999999
                         999999                DDDD
                                              DDDDDDD
                                             DDDDDDDD
                                              DDDDDDD
                                                DDDD

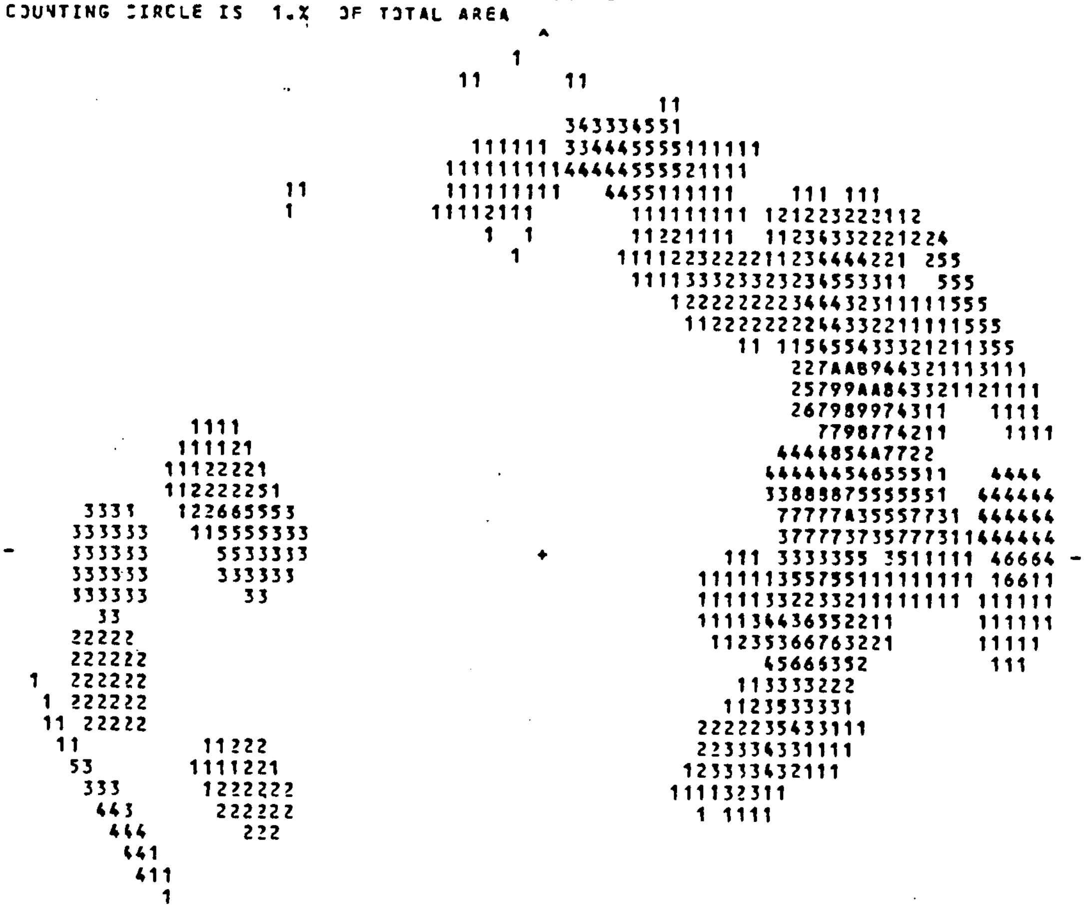



                                                                       FFFFF
                                                                      FFFFFF
                                                                      FFFFFFF
-                                       +                              FFFFFF   -
                                                                       FFFFF

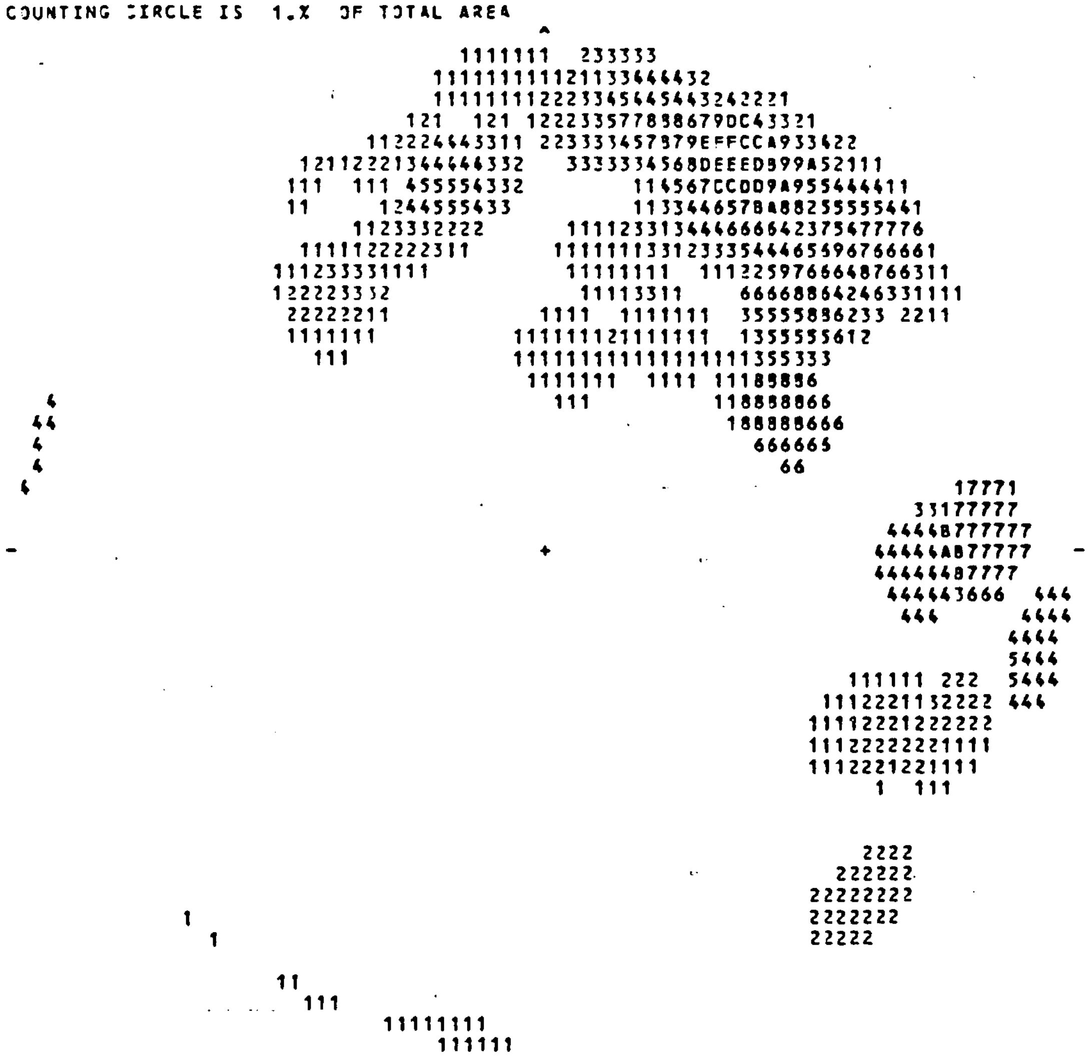
                                                                         8
                                                                       8888
                                                                      888888
                                                                     8888888
                                                                     888888
                                                                      8888
```

D4JN
194 OBSERVATIONS WITH TOTAL WEIGHT OF 707.5
COUNTING CIRCLE IS 1.% OF TOTAL AREA

```
D4MJ
 105 OBSERVATIONS WITH TOTAL WEIGHT OF    319.3
COUNTING CIRCLE IS  1.%  OF TOTAL AREA
```



D4MS
82 OBSERVATIONS WITH TOTAL WEIGHT OF 135.9
COUNTING CIRCLE IS 1.% OF TOTAL AREA



D4MA
188 OBSERVATIONS WITH TOTAL WEIGHT OF 455.2
COUNTING CIRCLE IS 1.X OF TOTAL AREA

APPENDIX B. PROGRAM SWEDGE

B.1 OBJECTIVE

Program SWEDGE was developed for two specific tasks. The first goal is to be able to evaluate the stability of wedges of known or assumed geometry, shear strength parameters, and water pressures by calculating a factor of safety. This capacity is required to determine whether wedges identified in the analysis of structural data are likely to fail if they are allowed to daylight.

The second objective is to carry out a back analysis study to determine a range of shear strength parameters that would satisfy the condition of limiting equilibrium for a wedge of specified geometry assuming that FOS=1 and water pressures are known.

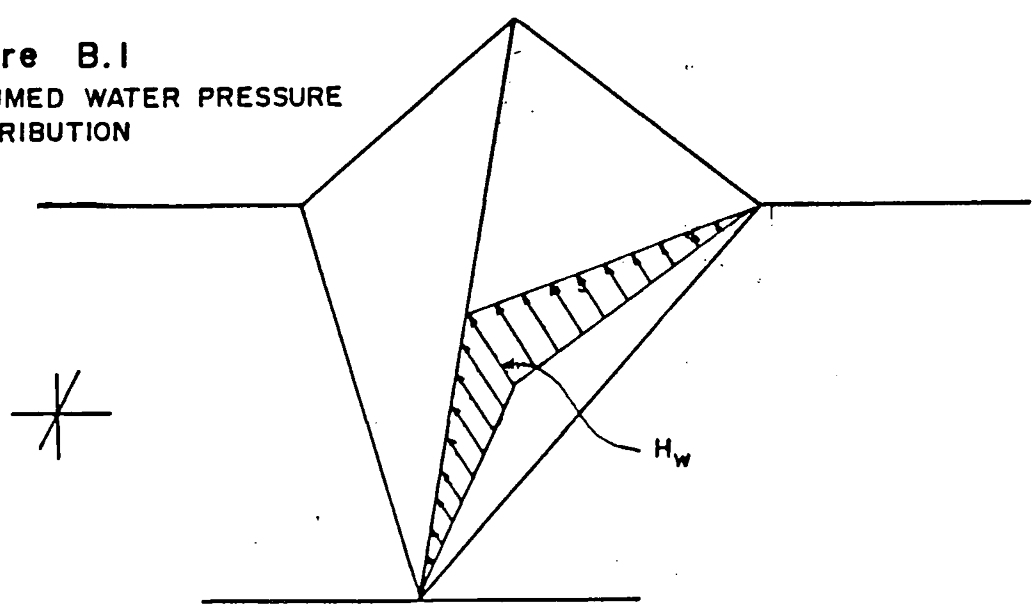
B.2 THEORY

A closed form mathematical solution has been developed for the calculation of factor of safety by Hoek, 1981. This solution uses a vector algebra approach that is computationally more efficient than other analytical methods. The appendix of Rock Slope Engineering that presents this solution is reproduced on the following pages.

A brief discussion on groundwater assumptions used in this model is required. The program uses a linear pore pressure distribution to calculate uplift forces due to groundwater. That is to say, pressures are assumed to be zero at all free faces and then increase linearly to a maximum value H_w at some point along the line of intersection of the two failure planes. The maximum possible value of H_w is assumed to be the full height of the wedge, H. The groundwater pressure distribution is illustrated in an isometric diagram in Figure B.1.

The maximum pore pressure achieved in the berms at Equity is likely less than the maximum theoretical value because most berms are somewhat fractured by blasting so any excess pore pressures usually disipate rapidly because of high permeability. If freezing occurs at the face, water is not allowed to drain and pore pressures may exceed the theoretical maximum value by significant amounts. For back analysis of the small berm scale failures it is recommended that u=0 be input if the berm appears dry or piezometer monitoring indicates that the water table is well below the berm face. If water is present in the berm u should be assigned a value of $0.5H\,\gamma_w/6$. If the opportunity arises to back analyze a full scale pit wall failure then pore pressures should be measured with piezometers in the failure or in a nearby section and the average pore pressure distribution should be calculated before it is input into the stability analysis.

Figure B.1
ASSUMED WATER PRESSURE DISTRIBUTION



Listing of "WEDGE SOLUTION FOR RAPID COMPUTATION" from Hoek and Bray, 1981.

SHORT SOLUTION

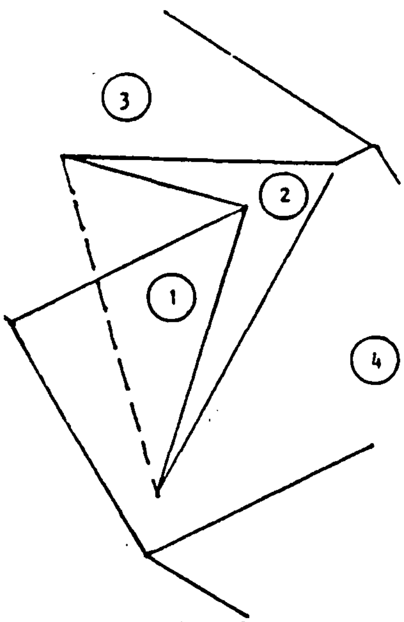
Scope of solution

The solution presented is for the computation of the factor of safety for translational slip of a tetrahedral wedge formed in a rock slope by two intersecting discontinuities, the slope face and the upper ground surface. It does not take account of rotational slip or toppling, nor does it include a consideration of those cases in which more than two intersecting discontinuities isolate tetrahedral or tapered wedges of rock. In other words, the influence of a tension crack is not considered in this solution.

The solution allows for different strength parameters and water pressures on the two planes of weakness. It is assumed that the slope crest is horizontal, ie the upper ground surface is either horizontal or dips in the same direction as the slope face or at 180° to this direction.

When a pair of discontinuities are selected at random from a set of field data, it is not known whether :

a) the planes could form a wedge (the line of intersection may plunge too steeply to daylight in the slope face or it may be too flat to intersect the upper ground surface).



Plane 1 overlies plane 2

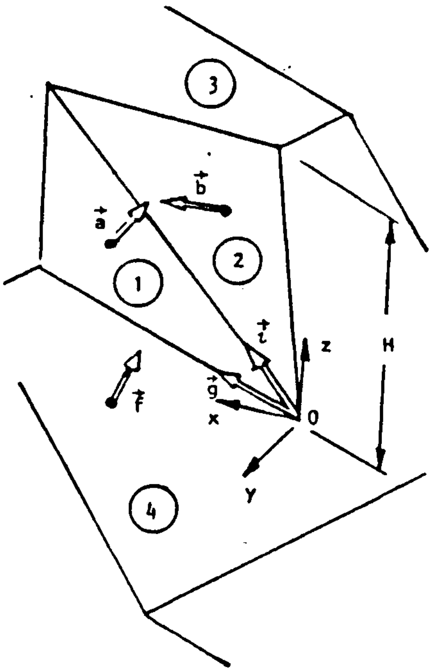
b) one of the planes overlies the other (this affects the calculation of the normal reactions on the planes)

c) one of the planes lies to the right or the left of the other plane when viewed from the bottom of the slope.

In order to resolve these uncertainties, the solution has been derived in such a way that either of the planes may be labelled 1 (or 2) and allowance has been made for one plane overlying the other. In addition, a check on whether the two planes do form a wedge is included in the solution at an early stage. Depending upon the geometry of the wedge and the magnitude of the water pressure acting on each plane, contact may be lost on either plane and this contingency is provided for in the solution.

Notation

The geometry of the problem is illustrated in the margin sketch. The discontinuities are denoted by 1 and 2, the upper ground surface by 3 and the slope face by 4. The data required for the solution of the problem are the unit weight of the rock γ, the height H of the crest of the slope above the intersection O, the dip ψ and dip direction α of each plane , the cohesion c and the friction angle ϕ for planes 1 and 2 and the average water pressure u on each of the planes 1 and 2*. If the slope face overhangs the toe of the slope, the index η is assigned the value of -1; if the slope does not overhang, η = +1.



Other terms used in the solution are :

F = factor of safety against wedge sliding calculated as the ratio of the resisting to the actuating shear forces

A = area of a face of the wedge

W = weight of the wedge

N = effective normal reaction on a plane

S = actuating shear force on a plane

x, y, z = co-ordinate axes with origin at 0. The z axis is directed vertically upwards, the y axis is in the dip direction of plane 2

$\vec{a}$ = unit vector in the direction of the normal to plane 1 with components (a_x, a_y, a_z)

$\vec{b}$ = unit vector in the direction of the normal to plane 2 with components (b_x, b_y, b_z)

$\vec{f}$ = unit vector in the direction of the normal to plane 4 with components (f_x, f_y, f_z)

$\vec{g}$ = vector in the direction of the line of intersection of planes 1 and 4 with components (g_x, g_y, g_z)

$\vec{i}$ = vector in the direction of the line of intersection of planes 1 and 2 with components (i_x, i_y, i_z)

If it is assumed that the discontinuities are completely filled with water and that the water pressure varies from zero at the free faces at a maximum at some point on the line of intersection, then $u_1 = u_2 = \gamma_w H_w/6$ where H_w is the overall height of the wedge.

$i = -i_z$

q = component of $\vec{g}$ in the direction of $\vec{b}$

r = component of $\vec{a}$ in the direction of $\vec{b}$

$k = |\vec{i}|^2 = i_x^2 + i_y^2 + i_z^2$

$l = W/A_2$

$p = A_1/A_2$

$$\left.\begin{array}{l} n_1 = N_1/A_2 \\ n_2 = N_2/A_2 \\ |li|/\sqrt{k} = SA_2 \end{array}\right\} \text{Assuming contact on both planes}$$

$$\left.\begin{array}{l} m_1 = N_1/A_2 \\ \text{denominator of } F = S_1/A_2 \end{array}\right\} \text{contact on plane 1 only}$$

$$\left.\begin{array}{l} m_2 = N_2/A_2 \\ \text{denominator of } F = S_2/A_2 \end{array}\right\} \text{contact on plane 2 only}$$

Sequence of calculations

The factor of safety of a tetrahedral wedge against sliding along a line of intersection may be calculated as follows :

1. $(a_x, a_y, a_z) = (\mathrm{Sin}\psi_1 . \mathrm{Sin}(\alpha_1 - \alpha_2),\ \mathrm{Sin}\psi_1 . \mathrm{Cos}(\alpha_1 - \alpha_2),\ \mathrm{Cos}\psi_1)$

2. $(f_x, f_y, f_z) = (\mathrm{Sin}\psi_4 . \mathrm{Sin}(\alpha_4 - \alpha_2),\ \mathrm{Sin}\psi_4 . \mathrm{Cos}(\alpha_4 - \alpha_2),\ \mathrm{Cos}\psi_4)$

3. $b_y = \mathrm{Sin}\psi_2$

4. $b_z = \mathrm{Cos}\psi_2$

5. $i = a_x b_y$

6. $g_z = f_x a_y - f_y a_x$

7. $q = b_y(f_z a_x - f_x a_z) + b_z g_z$

8. If $\eta q/i > 0$, or if $\eta(f_z - q/i)\ \mathrm{Tan}\psi_3 > \sqrt{1 - f_z^2}$ and $\alpha_3 = \alpha_4 \pm (1 - \eta)\ \pi/2$, no wedge is formed and the calculations should be terminated.

9. $r = a_y b_y + a_z b_z$

10. $k = 1 - r^2$

11. $l = (\gamma H q)/(3 g_z)$

12. $p = -b_y f_x / g_z$

13. $n_1 = \{(l/k)(a_z - r b_z) - p u_1\} . p/|p|$

14. $n_2 = \{(l/k)(b_z - r a_z) - u_2\}$

15. $m_1 = (l a_z - r u_2 - p u_1) . p/|p|$

16. $m_2 = (l b_z - r p u_1 - u_2)$

17. a) If $n_1 > 0$ and $n_2 > 0$, there is contact on both planes and

$$F = (n_1 . \mathrm{Tan}\phi_1 + n_2 . \mathrm{Tan}\phi_2 + |p|c_1 + c_2)\sqrt{k}/|l i|$$

b) If $n_2 < 0$ and $m_1 > 0$, there is contact on plane 1

only and

$$F = \frac{m_1 . \mathrm{Tan}\,\phi_1 + |p| c_1}{\{l^2(1 - a_z^2) + k u_2^2 + 2(r a_z - b_z) l u_2\}^{\frac{1}{2}}}$$

c) If $n_1 < 0$ and $m_2 > 0$, there is contact on plane 2 only and

$$F = \frac{m_2 . \mathrm{Tan}\phi_2 + c_2}{\{l^2 b_y^2 + k p^2 u_1^2 + 2(r b_z - a_z) p l u_1\}^{\frac{1}{2}}}$$

d) If $m_1 < 0$ and $m_2 < 0$, contact is lost on both planes and the wedge floats as a result of water pressure acting on planes 1 and 2. In this case, the factor of safety falls to zero.

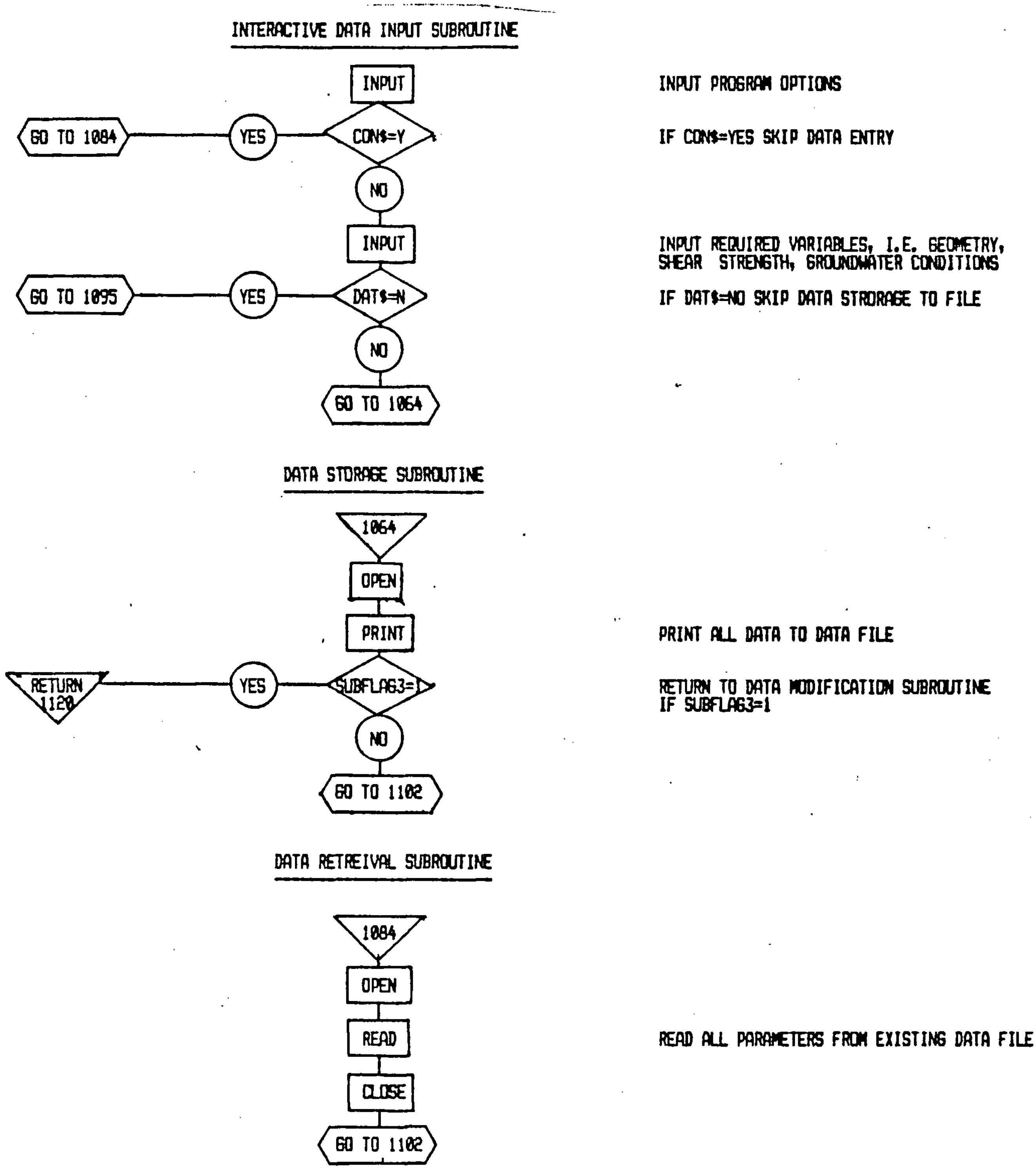
APPENDIX B.3 LIST OF VARIABLES

VARIABLE	FUNCTION	TYPE
AX	x component of unit vector perp. to plane 1	r
AY	y component of unit vector perp. to plane 1	r
AZ	z component of unit vector perp. to plane 1	r
A$	buffer for printing of final title	s
AUTOWAT$	control variable for groundwater assumption	s
BY	y component of unit vector perp. to plane 2	r
BZ	z component of unit vector perp. to plane 2	r
CON$	control variable for input, screen/data file	s
C1	cohesion on plane 1	r
C2	cohesion on plane 2	r
CHANGE	indicator for which variable to change	i
CFLAG	indicator for failure type	i
CLOW	initial cohesion in sensitivity subroutine	r
C(II)	cohesion at loop II of sensitivity analysis	r
COH$	string variable holding cohesion for print	s
DIP1	dip angle on plane 1 (failure surface)	r
DIP2	dip angle on plane 2 (failure surface)	r
DIP3	dip angle on plane 3 (crest)	r
DIP4	dip angle on plane 4 (face)	r
DIR1	dip direction on plane 1 (failure surface)	r
DIR2	dip direction on plane 2 (failure surface)	r
DIR3	dip direction on plane 3 (crest)	r
DIR4	dip direction on plane 4 (face)	r
DELTAC	incremental change in cohesion per loop	r
DAT$	variable indicates whether data to be stored	s
DATFILE$	name of data file	s
DATFILE2$	name of data file used in input	s
DEGRAD	conversion factor, degrees to radians	r
DELTAPHI	incremental change in friction angle	r
DOT	position of decimal point in fos	i
FX	x component of unit vector perp. to plane 4	r
FY	y component of unit vector perp. to plane 4	r
FZ	z component of unit vector perp. to plane 4	r
FLAG	control variable used if no wedge formaed	i
F	factor of safety	r
FOS(II,J)	factor of safety in sensitivity analysis	r
FOS$	string variable holding factor of safety	s
GAMMAD	dry density of rock	r
GZ	z component of vector along intersection 1,4	r
H	height of wedge, toe to crest	r
H$(I)	variable holds failure mode header	s
I	z component of unit vector i	r
II	counter	i
J	counter	i
K	length vector i squared	r
L	used	r
LIN$	buffer for printing sensitivity line	s
LWIDTH	line width	i
M1	stress on plane 1 if contact only on plane 1	r
M2	stress on plane 2 if contact only on plane 2	r

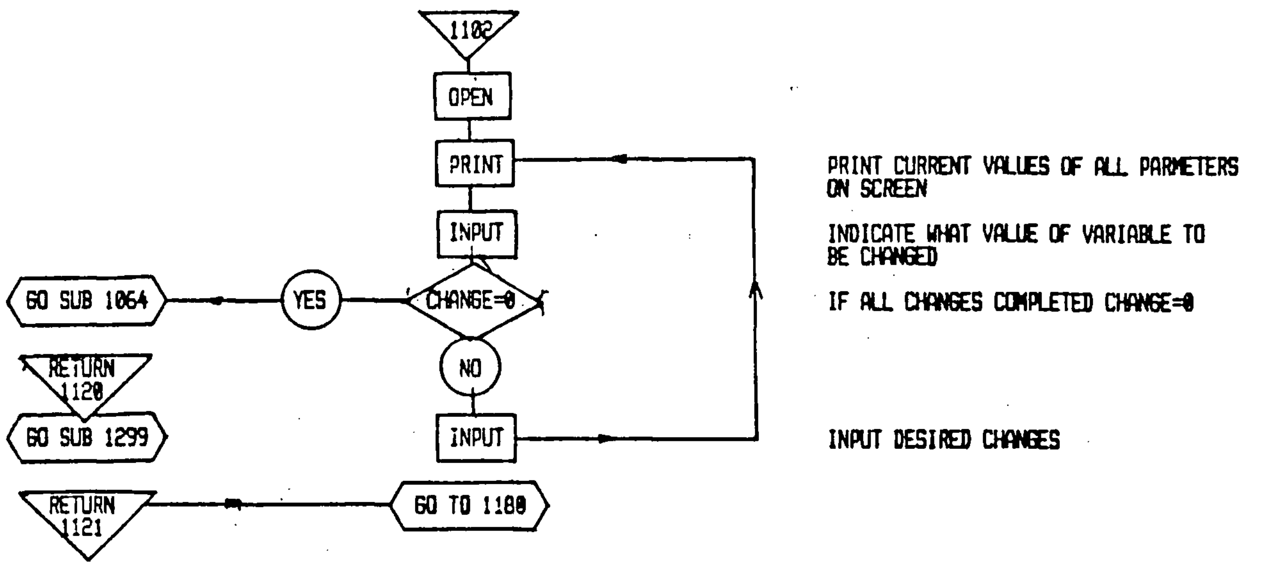
VARIABLE	FUNCTION	TYPE
NETA	-1 if face overhangs, else 1	i
N1	stress on plane 1 if contact on both planes	r
N2	stress on plane 2 if contact on both planes	r
PCON$	controls hardcopy output	s
PHI1	friction angle on plane 1	r
PHI2	friction angle on plane 2	r
PHILOW	minimum friction angle in sensitivity anal.	r
PHIINC	friction angle increment	r
PHI(J)	friction angle in current loop of sens. study	r
PAD	controls padding of buffer for left justify	i
PHI$	string variable holds value of phi	s
Q	component of vector g in direction of b	r
R	component of vector a in direction of b	r
SENS$	controls whether sensitivity study desired	s
SPEED$	controls detail of screen output durinc calcs	s
SUBFLAG2	used in return from subroutine	i
SUBFLAG3	used in return from subroutine	i
SPEC$	controls whether c-0 limits input or assigned	s
U1	pore pressure on plane 1	r
U2	pore pressure on plane 2	r
UMAX	maximum theoretical pore pressure	r
UNDER$	string variable used for underlining	s
WRATIO	ratio of avg. water pressure to maximum	r

Note: TYPE indicates variable type, i.e. r=real variable (e.g. 1.73) i=integer variable (e.g. 33) and s=string variable (e.g. "S-1")

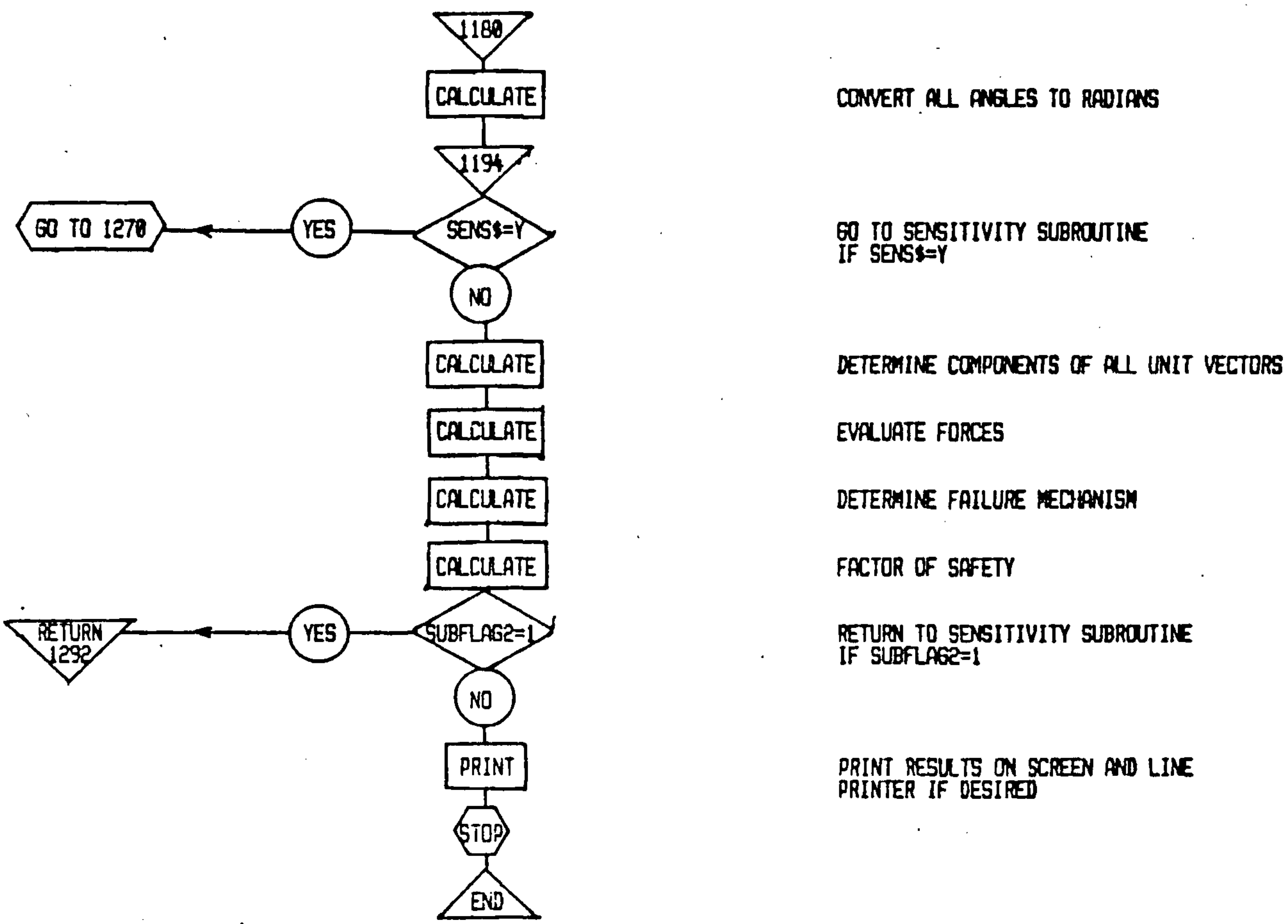
APPENDIX B.4 FLOW CHART FOR PROGRAM SWEDGE



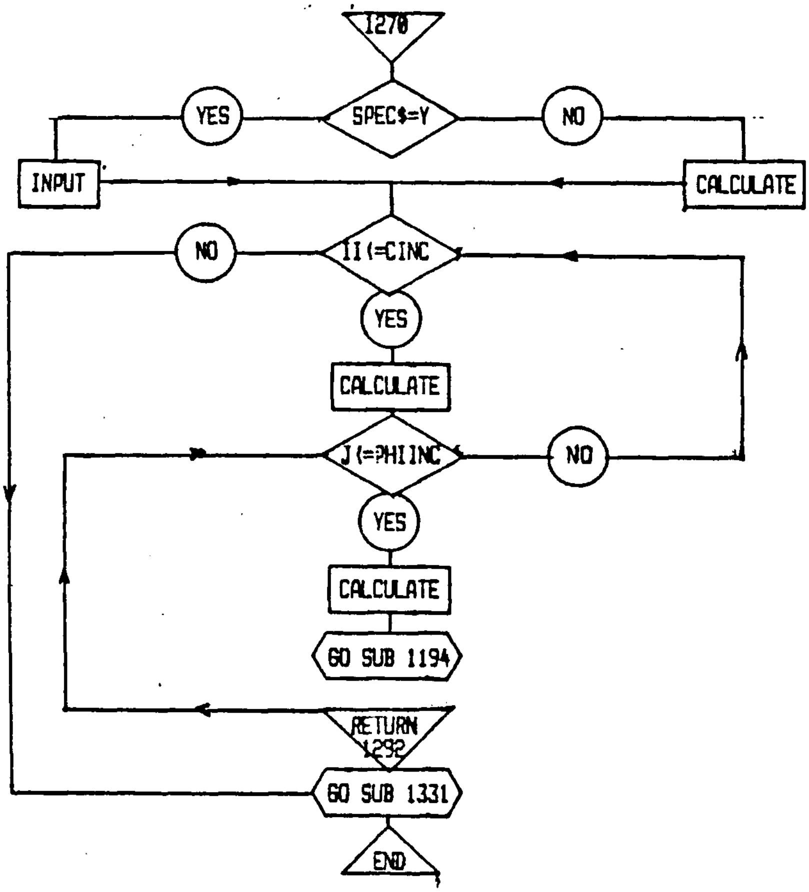
DATA ECHO AND MODIFICATION SUBROUTINE



MAIN CALCULATION SUBROUTINE



SENSITIVITY STUDY SUBROUTINE



INPUT C & PHI IF SPEC$=Y ELSE ASSIGN STANDARD RANGE

LOOP THROUGH ALL COHESIONS

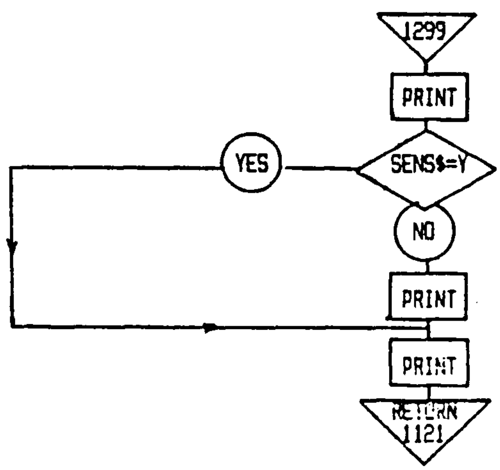
INCREMENT C

INCREMENT C

GO TO MAIN CALCULATION ROUTINE

GO TO PRINTING SUBROUTINE

ECHO PRINT INPUT DATA SUBROUTINE



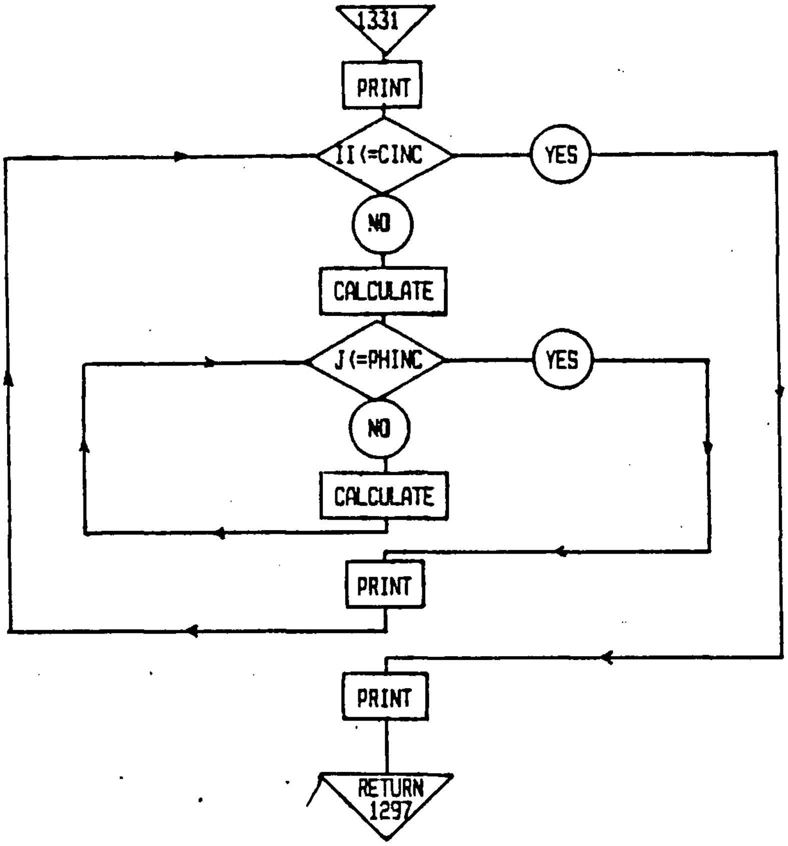
PRINT ALL DIMENSION PARAMETERS ON LINE PRINTER

BYPASS PRINTING SHEAR STRENGTH PARAMETERS IF SENS$=Y

PRINT SHEAR STRENGTH PARAMETERS

PRINT PORE PRESSURES

PRINT SENSITIVITY RESULTS SUBROUTINE



PRINT HEADERS

LOOP THROUGH ALL COHESIONS

ADD C(II) TO BUFFER

LOOP THROUGH ALL FRICTION ANGLES

LINE PRINT BUFFER

PRINT FAILURE MECHANISM, STRESS CONDITIONS AND TITLE

B.5 PROCEDURE FOR USE

Program SWEDGE is menu driven, with the capability to enter data from a screen or a data file on disk. Values of all parameters can be selectively altered before each run. The program can calculate a single factor of safety for a wedge of specified geometry, shear strength and water pressure, or it can carry out a sensitivity study over a full range of c and ∅. Because the procedure for use varies with the type of analysis and options selected it is not possible to give step by step instructions. Instead, each option is explained below.

1. "Do you want to use existing data?"
   - Yes if data already on file, note that each entry can be altered.
   - No if new data deck has to be entered.

2. "Do you want to do sensitivity study?"
   - Yes if a table with F.O.S. for expected range of c and ∅ is desired, e.g. for back analysis.
   - No if only one F.O.S. is desired for specified paramemeters

3. "Do you want hardcopy?"
   - Yes if listing of input data and results on printer desired.
   - No if output desired only on screen.

4. "Do you want to observe?"
   - Yes if c,∅, and F.O.S. to be printed on screeen during each iteration so progress can be observed. Slightly slower, but valuable to see if shear strength in right ball park.

5. "Name of file in which data is stored?"
   - Enter name of data file from which existing data is to be taken. Should be format "________.dat".

6. "Indicate Parameter to be Changed! If all OK type 0."
   - at this point values of all parameters listed on screen. Simply input number that precedes desired parameter or 0 if no further changes.

7. "Name of storage file if new? Else hit return"
   - name of data file to which new or modified data is to be written. If it is the same file from which data was read then hit return.

8. At this point data and results are printed if in single F.O.S. mode and program is terminated.

9. "Do you want to specify c and phi range?"
   - Yes if range other than default is desired.
   - No if default range c=0 to 20 kPa, incremented by 2
     ϕ=20 to 50 deg, incremented by 2

10. "Back analysis results printed.

11. "Analysis title?"
   - Input desired title. e.g. "Wedge 4 - Dry water conditions"

12. "Do you want to specify water pressures?"
   - yes if default value printed on screen is not desired. Default is theoretical maximum, i.e. $H_w \gamma_w/6$.

APPENDIX B.6 PROGRAM LISTING

```
1000 '************************************************************************
1001 '                               SHORT WEDGE
1002 '************************************************************************
1003 'A PROGRAM TO EVALUATE THE STABILITY OF A WEDGE USING DR. HOEK'S VECTOR
1004 'SOLUTION IN ROCK SLOPE ENG. APPENDIX 2.  FOR DETAILS SEE DOCUMENTATION.
1005 '
1006 '++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++
1007 '                              INTERACTIVE DATA INPUT SUBROUTINE
1008 '++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++
1009 'THIS SUBROUTINE PROMPTS FOR ALL REQUIRED PARAMETERS IN ANALYSIS.
1010  CLS
1011  PRINT"    **************************SHORT WEDGE PROGRAM***************************"
1012  PRINT CHR$(10)"                                    DATA INPUT"
1013  PRINT"
1014  INPUT"DO YOU WANT TO USE EXISTING DATA FILE? Y/N";CON$
1015  INPUT"DO YOU WANT TO DO SENSITIVITY STUDY?   Y/N";SENS$
1016  INPUT"DO YOU WANT HARDCOPY ON PRINTER?       Y/N";PCON$
1017  IF SENS$="Y" THEN INPUT"DO YOU WANT TO OBSERVE?                    Y/N";SPEED$
1018  IF CON$="Y" GOTO 1084
1019  INPUT"DRY UNIT WEIGHT OF ROCK (n/m**3)    ";GAMMAD
1020  INPUT"VERTICAL HEIGHT CREST ABOVE TOE (m)";H
1021  PRINT"
1022  PRINT"PROPERTIES OF 1ST SIDE OF WEDGE: PLANE 1"
1023  PRINT"
1024  INPUT"DIP FOR PLANE 1                     ";DIP1
1025  INPUT"DIP DIRECTION FOR PLANE 1           ";DIR1
1026  INPUT"COHESION FOR PLANE 1                ";C1
1027  INPUT"FRICTION ANGLE FOR PLANE 1          ";PHI1
1028  PRINT"
1029  PRINT"PROPERTIES OF 2ND SIDE OF WEDGE: PLANE 2"
1030  PRINT"SPECIFY NETA: 1 IF DOES NOT OVERHANG, 1 IF OVERHANGING"
1031  PRINT"
1032  INPUT"DIP FOR PLANE 2                     ";DIP2
1033  INPUT"DIP DIRECTION FOR PLANE 2           ";DIR2
1034  INPUT"COHESION FOR PLANE 2                ";C2
1035  INPUT"FRICTION ANGLE FOR PLANE 2          ";PHI2
1036  PRINT"
1037  PRINT"ORIENTATION OF TOP PLANE: PLANE 3"
1038  PRINT"
1039  INPUT"DIP FOR PLANE 3                     ";DIP3
1040  INPUT"DIP DIRECTION FOR PLANE 3           ";DIR3
1041  PRINT"
1042  PRINT"ORIENTATION OF FACE: PLANE 4"
1043  PRINT"
1044  INPUT"DIP FOR PLANE 4                     ";DIP4
1045  INPUT"DIP DIRECTION FOR PLANE 4           ";DIR4
1046  PRINT"
1047  UMAX=H*9.810001/6
1048  PRINT USING"UMAX ON WEDGE = ####";UMAX
1049  INPUT"DO YOU WANT TO SPECIFY WATER PRESSURES? Y/N";AUTOWAT$
1050  IF AUTOWAT$="N" THEN U1=UMAX
1051  IF AUTOWAT$="N" THEN U2=UMAX
1052  IF AUTOWAT$="N" GOTO 1055
1053  INPUT"U1=?                                ";U1
1054  INPUT"U2=?                                ";U2
1055  PRINT"
1056  INPUT"NETA=                               ";NETA
1057  INPUT"DO YOU WANT TO STORE DATA           ";DAT$
1058  IF DAT$="N"GOTO 1095 ELSE GOTO 1064
1059 '
1060 '++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++
1061 '                                    DATA STORAGE SUBROUTINE
1062 '++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++
1063 'THIS SUBROUTINE STORES ALL INPUT DATA IN A FILE FOR FUTURE ACCESS.
1064  CLS
1065  LOCATE 10,10
1066  PRINT USING "CURRENT DATA FILE =    \                  \";DATFILE$
1067  LOCATE 12,10
1068  INPUT"NAME OF STORAGE FILE IF NEW? ELSE HIT RETURN";DATFILE2$
1069  IF LEN(DATFILE2$)()0 THEN DATFILE$=DATFILE2$
```

```
1070  OPEN "O",#1,DATFILE$
1071  WRITE #1,GAMMAD,H
1072  WRITE #1,DIP1,DIR1,C1,PHI1
1073  WRITE #1,DIP2,DIR2,C2,PHI2
1074  WRITE #1,DIP3,DIR3,DIP4,DIR4
1075  WRITE #1,U1,U2,NETA
1076  CLOSE #1
1077  IF SUBFLAG3=1 THEN RETURN
1078  GOTO 1102
1079 '
1080 '++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++
1081 '                           DATA RETREIVAL SUBROUTINE
1082 '++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++
1083 'THIS SUBROUTINE RETREIVES ALL INPUT DATA FROM A STORAGE FILE.
1084  CLS
1085  LOCATE 10,10
1086  INPUT"NAME OF FILE IN WHICH DATA IS STORED";DATFILE$
1087  OPEN "I",#1,DATFILE$
1088  INPUT #1,GAMMAD,H
1089  INPUT #1,DIP1,DIR1,C1,PHI1
1090  INPUT #1,DIP2,DIR2,C2,PHI2
1091  INPUT #1,DIP3,DIR3,DIP4,DIR4
1092  INPUT #1,U1,U2,NETA
1093  CLOSE #1
1094  GOTO 1102
1095 '
1096 '++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++
1097 '                   DATA ECHO AND CORRECTION SUBROUTINE
1098 '++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++
1099 'THIS SUBROUTINE PRINTS ALL DATA THAT WILL BE USED IN ANALYSIS ON SCREEN.
1100 'A PROVISION IS MADE TO CORRECT ALL DATA.
1101 '
1102  CLS
1103  PRINT"                        INPUT PARAMETERS
1104  PRINT"
1105  PRINT USING " 1. GAMMAD     = ####      2. HEIGHT H          = ###";GAMMAD,H
1106  PRINT"
1107  PRINT USING " 3. DIP 1      =   ##      4. DIP DIRECTION 1 = ###";DIP1,DIR1
1108  PRINT USING " 5. DIP 2      =   ##      6. DIP DIRECTION 2 = ###";DIP2,DIR2
1109  PRINT USING " 7. DIP 3      =   ##      8. DIP DIRECTION 3 = ###";DIP3,DIR3
1110  PRINT USING " 9. DIP 4      =   ##     10. DIP DIRECTION 4 = ###";DIP4,DIR4
1111  PRINT"
1112  PRINT USING "11. COHESION 1 = ####     12. PHI 1             =  ##";C1,PHI1
1113  PRINT USING "13. COHESION 2 = ####     14. PHI 2             =  ##";C2,PHI2
1114  PRINT"
1115  PRINT USING "15. U1         = ####     16. U2                =  ##";U1,U2
1116  PRINT"
1117  INPUT"INDICATE PARAMETER TO BE CHANGED! IF ALL OK ENTER 0!";CHANGE
1118  IF CHANGE=0 THEN SUBFLAG3=1
1119  IF CHANGE=0 THEN GOSUB 1064
1120  IF CHANGE=0 AND PCON$="Y" THEN GOSUB 1301
1121  IF CHANGE=0 GOTO 1180
1122 '
1123 'MAKE ANY DESIRED CHANGES
1124  IF CHANGE=1 GOTO 1140
1125  IF CHANGE=2 GOTO 1142
1126  IF CHANGE=3 GOTO 1144
1127  IF CHANGE=4 GOTO 1152
1128  IF CHANGE=5 GOTO 1146
1129  IF CHANGE=6 GOTO 1154
1130  IF CHANGE=7 GOTO 1148
1131  IF CHANGE=8 GOTO 1156
1132  IF CHANGE=9 GOTO 1150
1133  IF CHANGE=10 GOTO 1158
1134  IF CHANGE=11 GOTO 1160
1135  IF CHANGE=12 GOTO 1164
1136  IF CHANGE=13 GOTO 1162
1137  IF CHANGE=14 GOTO 1166
1138  IF CHANGE=15 GOTO 1168
1139  IF CHANGE=16 GOTO 1170
```

```
1140  INPUT"GAMMAD       =";GAMMAD
1141  GOTO 1102
1142  INPUT"H            =";H
1143  GOTO 1102
1144  INPUT"DIP1         =";DIP1
1145  GOTO 1102
1146  INPUT"DIP2         =";DIP2
1147  GOTO 1102
1148  INPUT"DIP3         =";DIP3
1149  GOTO 1102
1150  INPUT"DIP4         =";DIP4
1151  GOTO 1102
1152  INPUT"DIR1         =";DIR1
1153  GOTO 1102
1154  INPUT"DIR2         =";DIR2
1155  GOTO 1102
1156  INPUT"DIR3         =";DIR3
1157  GOTO 1102
1158  INPUT"DIR4         =";DIR4
1159  GOTO 1102
1160  INPUT"COHESION 1 =";C1
1161  GOTO 1102
1162  INPUT"COHESION 2 =";C2
1163  GOTO 1102
1164  INPUT"PHI 1        =";PHI1
1165  GOTO 1102
1166  INPUT"PHI 2        =";PHI2
1167  GOTO 1102
1168  INPUT"U1           =";U1
1169  GOTO 1102
1170  INPUT"U2           =";U2
1171  GOTO 1102
1172 '
1173 '++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++
1174 '                    MAIN CALCULATION SUBROUTINE
1175 '++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++
1176 'THIS SUBROUTINE DOES ALL REQUIRED CALCULATIONS TO COMPUTE FACTOR OF
1177 'SAFETY
1178 '
1179 'CONVERT ANGLES TO RADIANS
1180  DEGRAD=3.142/180
1181  PHI1=DEGRAD*PHI1
1182  PHI2=DEGRAD*PHI2
1183  DIP1=DEGRAD*DIP1
1184  DIP2=DEGRAD*DIP2
1185  DIP3=DEGRAD*DIP3
1186  DIP4=DEGRAD*DIP4
1187  DIR1=DEGRAD*DIR1
1188  DIR2=DEGRAD*DIR2
1189  DIR3=DEGRAD*DIR3
1190  DIR4=DEGRAD*DIR4
1191  IF SENS$="Y" GOTO 1272
1192 '
1193 'CALCULATE COMPONENTS OF UNIT VECTORS
1194  AX=SIN(DIP1)*SIN(DIR1-DIR2)
1195  AY=SIN(DIP1)*COS(DIR1-DIR2)
1196  AZ=COS(DIP1)
1197  FX=SIN(DIP4)*SIN(DIR4-DIR2)
1198  FY=SIN(DIP4)*COS(DIR4-DIR2)
1199  FZ=COS(DIP4)
1200  BY=SIN(DIP2)
1201  BZ=COS(DIP2)
1202  I=AX*BY
1203  GZ=FX*AY-FY*AX
1204  Q=BY*(FZ*AX-FX*AZ)+BZ*GZ
1205 'CHECK IF GEOMETRY ACTUALLY FORMS A WEDGE
1206  IF NETA*I/Q)0 THEN FLAG=-1
1207  IF NETA*(FZ-Q/I)*TAN(DIP3))SQR(1-FZ^2) THEN CHECK=-1
1208  IF DIR3=DIR4+(1-NETA)*3.1416/2 AND CHECK=-1 THEN FLAG=-1
1209  IF FLAG()-1 GOTO 1215
```

```
1210  CLS
1211  LOCATE 10,1
1212  PRINT"                         NO WEDGE FORMED WITH INPUT GEOMETRY"
1213  SOUND 100,10
1214  STOP
1215  R=AY*BY+AZ*BZ
1216  K=1-R^2
1217  L=(GAMMAD*H*Q)/(3*GZ)
1218  P=-BY*FX/GZ
1219  N1=((L/K)*(AZ-R*BZ)-P*U1)*P/ABS(P)
1220  N2=((L/K)*(BZ-R*AZ)-U2)
1221  M1=(L*AZ-R*U2-P*U1)*P/ABS(P)
1222  M2=(L*BZ-R*P*U1-U2)
1223 '
1224 'EVALUATE WHETHER SLIDING ON ONE OR BOTH PLANES AND COMPUTE APPROPRIATE
1225 'F.O.S.
1226 '
1227 'CONTACT IS ON BOTH PLANES: CFLAG=3
1228  IF N1)0 AND N2)0 GOTO 1229 ELSE GOTO 1233
1229  F=(N1*TAN(PHI1)+N2*TAN(PHI2)+ABS(P)*C1+C2)*SQR(K)/ABS(L*I)
1230  CFLAG=3
1231  GOTO 1252
1232 '
1233 'CONTACT IS ON PLANE 1: CFLAG=1
1234  IF N2(0 AND M1)0 THEN GOTO 1235 ELSE GOTO 1241
1235  F=M1*TAN(PHI1)+ABS(P)*C1
1236  F=F/SQR(L^2*(1-AZ^2)+K*U2^2+2*(R*AZ-BZ)*L*U2)
1237  CFLAG=1
1238  GOTO 1252
1239 '
1240 'CONTACT IS ON PLANE 2: CFLAG=2
1241  IF N1(0 AND M2)0 THEN GOTO 1242 ELSE GOTO 1248
1242  F=M2*TAN(PHI2)+C2
1243  F=F/SQR(L^2*BY^2+K*P^2*U1^2+2*(R*BZ-AZ)*P*L*U1)
1244  CFLAG=2
1245  GOTO 1252
1246 '
1247 'WEDGE IS FLOATED DUE TO WATER PRESSURE CONTACT IS LOST
1248  IF M1(0 AND M2(0 THEN CFLAG=0
1249  F=0
1250 '
1251 'PRINT RESULTS OF ANALYSIS
1252  H$(0)="WEDGE FLOATED DUE TO PORE PRESSURES"
1253  H$(1)="CONTACT ON PLANE 1 ONLY"
1254  H$(2)="CONTACT ON PLANE 2 ONLY"
1255  H$(3)="CONTACT ON BOTH PLANES"
1256  IF SUBFLAG2=1 THEN RETURN
1257  CLS
1258  LOCATE 10,10
1259  PRINT USING "\                              \  F.O.S = ##.###";H$(CFLAG),F
1260  IF PCON$()"Y" THEN GOTO 1262
1261  LPRINT USING "\                              \  F.O.S = ##.###";H$(CFLAG),F
1262  STOP
1263  END
1264 '
1265 '+++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++
1266 '                         SENSITIVITY STUDY SUBROUTINE
1267 '+++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++
1268 'THIS SUBROUTINE IS ONLY ACTIVATED IF THE INFLUENCE OF C AND PHI ON
1269 'WEDGE STABILITY IS DESIRED.
1270 '
1271 'INPUT LIMITS ON C AND PHI
1272  INPUT"DO YOU WANT TO SPECIFY C AND PHI RANGE? Y/N";SPEC$
1273  IF SPEC$="Y" THEN GOTO 1276
1274  CLOW=0:DELTAC=2:CINC=10
1275  PHILOW=20:DELTAPHI=2:PHIINC=15
1276  GOTO 1281
1277  INPUT"CLOW,DELTA C, NUMBER OF INCREMENTS       ";CLOW,DELTAC,CINC
1278  INPUT"PHILOW, DELTA PHI, NUMBER OF INCREMENTS  ";PHILOW,DELTAPHI,PHIINC
1279 '
```

```
1280 'LOOP THROUGH PHI HOLDING C CONSTANT
1281  CLS
1282  DIM PHI(20),FOS(10,20)
1283  FOR II=0 TO CINC
1284      IF SPEED$="N" THEN PRINT II
1285      C1=CLOW+II*DELTAC
1286      C2=C1
1287      C(II)=C1
1288      FOR J=0 TO PHIINC
1289         PHI(J)=PHILOW+J*DELTAPHI
1290         PHI1=PHI(J)*DEGRAD
1291         PHI2=PHI1
1292         SUBFLAG2=1
1293         GOSUB 1194
1294         FOS(II,J)=F
1295         IF SPEED$="Y" THEN PRINT USING" ## ##   ####   ####   ##.##";II,J,C(II),PHI(J),F
1296      NEXT J
1297  NEXT II
1298  GOSUB 1333
1299  END
1300 '
1301 '+++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++
1302 '                          ECHO PRINT INPUT DATA SUBROUTINE
1303 '+++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++
1304 'THIS ROUTINE PRINTS ALL INPUT DATA ON THE LINE PRINTER.
1305  LPRINT"                                  SHORT WEDGE STABILITY ANALYSIS"
1306  LPRINT"                                  ______________________________"
1307  LPRINT CHR$(10),CHR$(10)
1308  WIDTH LPRINT 120
1309  LPRINT"                                  INPUT PARAMETERS
1310  LPRINT"
1311  LPRINT USING "        1. GAMMAD      = ####    2.  HEIGHT H       = ###";GAMMAD,H
1312  LPRINT"
1313  LPRINT USING "        3. DIP 1       =   ##    4. DIP DIRECTION 1 = ###";DIP1,DIR1
1314  LPRINT USING "        5. DIP 2       =   ##    6. DIP DIRECTION 2 = ###";DIP2,DIR2
1315  LPRINT USING "        7. DIP 3       =   ##    8. DIP DIRECTION 3 = ###";DIP3,DIR3
1316  LPRINT USING "        9. DIP 4       =   ##   10. DIP DIRECTION 4 = ###";DIP4,DIR4
1317  LPRINT"
1318  IF SENS$="Y" GOTO 1322
1319  LPRINT USING "       11. COHESION 1 = ####   12. PHI 1          =  ##";C1,PHI1
1320  LPRINT USING "       13. COHESION 2 = ####   14. PHI 2          =  ##";C2,PHI2
1321  LPRINT"
1322  LPRINT USING "       15. U1         = ####   16. U2             =  ##";U1,U2
1323  LPRINT"
1324  RETURN
1325 '
1326 '+++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++
1327 '                         PRINT SENSITIVITY RESULTS SUBROUTINE
1328 '+++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++
1329 'THIS SUBROUTINE PRINTS A TABLE OF F.O.S. FOR VARIOUS C AND PHI ON THE
1330 'LINE PRINTER
1331 '
1332 'PRINT FRICTION ANGLE COLUMN HEADER
1333  LPRINT"                          TABLE OF FACTOR OF SAFETY"
1334  IF PHIINC>9 THEN LPRINT CHR$(29)
1335  LPRINT TAB(40),"FRICTION ANGLES"
1336  LIN$="      COHESION  "
1337  FOR J=0 TO PHIINC
1338      PHI$=STR$(PHI(J))
1339      PAD=6-LEN(PHI$)
1340      PHI$=SPACE$(PAD)+PHI$
1341      LIN$=LIN$+PHI$
1342  NEXT J
1343  LPRINT LIN$
1344  LWIDTH=6*(PHIINC+1)+10
1345  UNDER$="        "+STRING$(LWIDTH,"-")
1346  LPRINT UNDER$
1347 'PRINT FACTOR OF SAFETY FOR CURRENT COHESION AND VARIOUS PHI
1348  FOR II=0 TO CINC
1349      COH$="        "+STR$(C(II))+"   !"
```

```
1350      PAD=17-LEN(COH$)
1351      COH$=SPACE$(PAD)+COH$
1352      LIN$=COH$
1353      FOR J=0 TO PHIINC
1354         IF FOS(II,J)<.1 THEN FOS(II,J)=0!
1355         FOS$=STR$(FOS(II,J))
1356         DOT=INSTR(FOS$,".")+2
1357         FOS$=LEFT$(FOS$,DOT)
1358         PAD=6-LEN(FOS$)
1359         FOS$=SPACE$(PAD)+FOS$
1360         LIN$=LIN$+FOS$
1361      NEXT J
1362      LPRINT LIN$
1363   NEXT II
1364 '
1365 'PRINT FAILURE MECHANISM
1366  LPRINT CHR$(30)
1367  H$(0)="WEDGE FLOATED DUE TO PORE PRESSURES"
1368  H$(1)="CONTACT ON PLANE 1 ONLY"
1369  H$(2)="CONTACT ON PLANE 2 ONLY"
1370  H$(3)="CONTACT ON BOTH PLANES"
1371  LPRINT USING"       \                                   \";H$(CFLAG)
1372 '
1373 'PRINT STRESS CONDITIONS ON FAILURE SURFACES
1374  IF CFLAG=1 THEN LPRINT USING"       TOTAL STRESS ON PLANE 1     = ####.## ";M1
1375  IF CFLAG=2 THEN LPRINT USING"       TOTAL STRESS ON PLANE 2     = ####.## ";M2
1376  IF CFLAG=3 THEN LPRINT USING"       TOTAL STRESS ON PLANE 1     = ####.## ";N1
1377  IF CFLAG=3 THEN LPRINT USING"       TOTAL STRESS ON PLANE 2     = ####.## ";N2
1378  UMAX=H*9.810001/6
1379  WRATIO=(U1+U2)/2/UMAX
1380  LPRINT USING "       WATER PRESSURE RATIO U/UMAX = ####.##";WRATIO
1381  SOUND 100,10
1382  CLS
1383  INPUT"ANALYSIS TITLE";TITLE$
1384  LPRINT
1385  A$="         \                                                \"
1386  LPRINT USING A$;TITLE$
1387  LPRINT CHR$(12)
1388  RETURN
1389 '
```

BERM FAILURE BACK ANALYSIS

A. LOCATION:

PIT:	MNZN ULT	NORTHING:	
BENCH:	1320	EASTING:	
WALL:	EAST	ELEVATION:	
DOMAIN:	D2		

FAILURE MODE: PLANE + LATERAL RELEASE

B. ORIENTATION:

PLANE 1		PLANE 2		PLANE 3(CREST)		PLANE 4(FACE)	
DIP:	56	DIP:	90	DIP:	0	DIP:	70
DIP DIR:	243	DIP DIR:	323	DIP DIR:	270	DIP DIR:	270

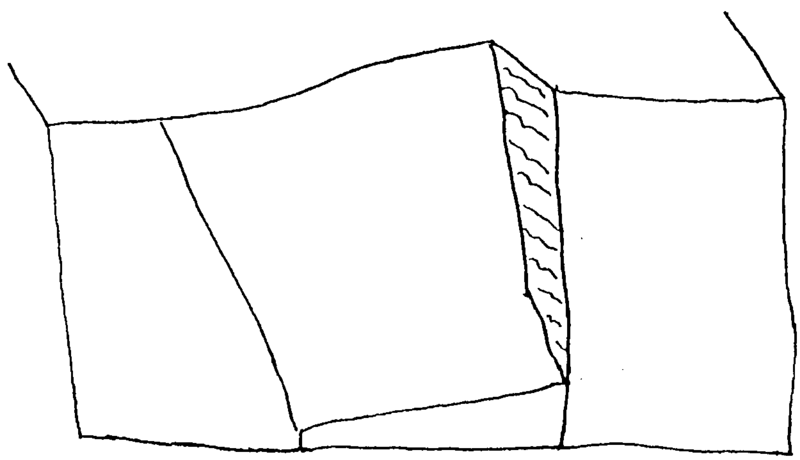
C. DIMENSIONS:

WIDTH ALONG CREST:	12.0	HEIGHT TOE/CREST:	15.0
LENGTH INTERSECTION:	20.0	ESTIMATED VOLUME:	

D. GEOLOGY:

PLANE 1		PLANE 2	
DISCONTINUITY TYPE:	MJ	DISCONTINUITY TYPE:	SR
ROUGHNESS ANGLE:	5	ROUGHNESS ANGLE:	10
GOUGE TYPE:	NONE	GOUGE TYPE:	2 cm.
WATER CODE:	3	WATER CODE:	3
ROCK TYPE:	GBR		

E. SKETCH:



DATE: 85/08/28 RECORDED BY: TS / JM FAILURE NUMBER: 1

SHORT WEDGE STABILITY ANALYSIS

INPUT PARAMETERS

1.	GAMMAD	= 34	2.	HEIGHT H	=	15
3.	DIP 1	= 56	4.	DIP DIRECTION 1	=	243
5.	DIP 2	= 90	6.	DIP DIRECTION 2	=	323
7.	DIP 3	= 0	8.	DIP DIRECTION 3	=	270
9.	DIP 4	= 70	10.	DIP DIRECTION 4	=	270
15.	U1	= 0	16.	U2	=	0

TABLE OF FACTOR OF SAFETY

		FRICTION ANGLES										
COHESION		20	23	26	29	32	35	38	41	44	47	50
0	!	.24	.28	.32	.37	.42	.47	.52	.58	.65	.72	.80
2	!	.32	.36	.40	.44	.49	.54	.60	.66	.72	.79	.87
4	!	.39	.43	.48	.52	.57	.62	.67	.73	.80	.87	.95
6	!	.47	.51	.55	.60	.64	.69	.75	.81	.87	.95	1.03
8	!	.54	.58	.63	.67	.72	.77	.83	.88	.95	1.02	1.10
10	!	.62	.66	.70	.75	.80	.85	.90	.96	1.03	1.10	1.18
12	!	.70	.74	.78	.82	.87	.92	.98	1.04	1.10	1.17	1.25
14	!	.77	.81	.85	.90	.95	1.00	1.05	1.11	1.18	1.25	1.33
16	!	.85	.89	.93	.98	1.02	1.07	1.13	1.19	1.25	1.33	1.41
18	!	.92	.96	1.01	1.05	1.10	1.15	1.20	1.26	1.33	1.40	1.48
20	!	1.00	1.04	1.08	1.13	1.17	1.23	1.28	1.34	1.40	1.48	1.56

CONTACT ON PLANE 1 ONLY
TOTAL STRESS ON PLANE 1 = 37.73
WATER PRESSURE RATIO U/UMAX = 0.00

WEDGE 1 - DRY CONDITION

SHORT WEDGE STABILITY ANALYSIS

INPUT PARAMETERS

1. GAMMAD	= 34	2. HEIGHT H	= 15
3. DIP 1	= 56	4. DIP DIRECTION 1	= 243
5. DIP 2	= 90	6. DIP DIRECTION 2	= 323
7. DIP 3	= 0	8. DIP DIRECTION 3	= 270
9. DIP 4	= 70	10. DIP DIRECTION 4	= 270
15. U1	= 13	16. U2	= 13

TABLE OF FACTOR OF SAFETY

	FRICTION ANGLES										
COHESION	20	23	26	29	32	35	38	41	44	47	50
0 !	0	0	0	0	0	0	.11	.12	.13	.15	.16
2 !	.12	.13	.14	.15	.16	.17	.18	.19	.20	.22	.24
4 !	.19	.20	.21	.22	.23	.24	.25	.26	.28	.29	.31
6 !	.26	.27	.28	.29	.30	.31	.32	.33	.35	.36	.38
8 !	.34	.34	.35	.36	.37	.38	.39	.41	.42	.44	.45
10 !	.41	.42	.43	.44	.45	.46	.47	.48	.49	.51	.53
12 !	.48	.49	.50	.51	.52	.53	.54	.55	.57	.58	.60
14 !	.55	.56	.57	.58	.59	.60	.61	.62	.64	.65	.67
16 !	.63	.63	.64	.65	.66	.67	.68	.70	.71	.73	.74
18 !	.70	.71	.72	.72	.73	.75	.76	.77	.78	.80	.81
20 !	.77	.78	.79	.80	.81	.82	.83	.84	.86	.87	.89

CONTACT ON PLANE 1 ONLY
TOTAL STRESS ON PLANE 1 = 8.28
WATER PRESSURE RATIO U/UMAX = 0.53

WEDGE 1 - WITH WATER - NORMAL RANGE OF C AND PHI

SHORT WEDGE STABILITY ANALYSIS

INPUT PARAMETERS

1.	GAMMAD =	34	2.	HEIGHT H =	15
3.	DIP 1 =	56	4.	DIP DIRECTION 1 =	243
5.	DIP 2 =	90	6.	DIP DIRECTION 2 =	323
7.	DIP 3 =	0	8.	DIP DIRECTION 3 =	270
9.	DIP 4 =	70	10.	DIP DIRECTION 4 =	270
15.	U1 =	13	16.	U2 =	13

TABLE OF FACTOR OF SAFETY

	FRICTION ANGLES										
COHESION	30	33	36	39	42	45	48	51	54	57	60
10 !	.44	.45	.46	.47	.48	.50	.51	.53	.55	.57	.60
12 !	.51	.52	.53	.54	.56	.57	.59	.60	.62	.65	.67
14 !	.58	.59	.60	.62	.63	.64	.66	.68	.70	.72	.75
16 !	.66	.67	.68	.69	.70	.72	.73	.75	.77	.79	.82
18 !	.73	.74	.75	.76	.77	.79	.80	.82	.84	.86	.89
20 !	.80	.81	.82	.83	.85	.86	.88	.89	.91	.94	.96
22 !	.87	.88	.89	.91	.92	.93	.95	.97	.99	1.01	1.04
24 !	.95	.96	.97	.98	.99	1.00	1.02	1.04	1.06	1.08	1.11
26 !	1.02	1.03	1.04	1.05	1.06	1.08	1.09	1.11	1.13	1.15	1.18
28 !	1.09	1.10	1.11	1.12	1.14	1.15	1.17	1.18	1.20	1.23	1.25
30 !	1.16	1.17	1.18	1.20	1.21	1.22	1.24	1.26	1.28	1.30	1.33

CONTACT ON PLANE 1 ONLY
TOTAL STRESS ON PLANE 1 = 8.28
WATER PRESSURE RATIO U/UMAX = 0.53

WEDGE 1 - WITH WATER - HIGH LEVELS OF C AND PHI

BERM FAILURE BACK ANALYSIS

A. LOCATION:

PIT:	MNZN ULT	NORTHING:	
BENCH:	1340	EASTING:	
WALL:	EAST	ELEVATION:	
DOMAIN:	D2		

FAILURE MODE: WEDGE, MOSTLY SLIDING ON PLANE 1.

B. ORIENTATION:

	PLANE 1	PLANE 2	PLANE 3(CREST)	PLANE 4(FACE)
DIP:	43	81	0	70
DIP DIR:	227	343	270	270

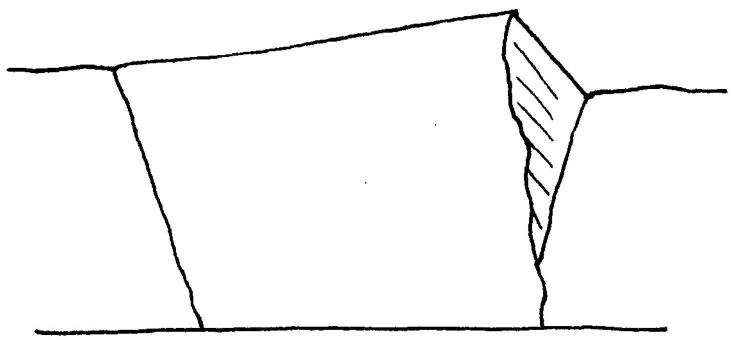
C. DIMENSIONS:

WIDTH ALONG CREST:	12.0	HEIGHT TOE/CREST:	16.0
LENGTH INTERSECTION:	25.0	ESTIMATED VOLUME:	

D. GEOLOGY:

	PLANE 1	PLANE 2
DISCONTINUITY TYPE:	MJ	JN
ROUGHNESS ANGLE:	5	3
GOUGE TYPE:	NONE	NONE
WATER CODE:	3	2
ROCK TYPE:	GBR	

E. SKETCH:



DATE: 84/08/28 RECORDED BY: TS /JM FAILURE NUMBER: 2

SHORT WEDGE STABILITY ANALYSIS

INPUT PARAMETERS

1.	GAMMAD	= 34	2.	HEIGHT H	=	16
3.	DIP 1	= 43	4.	DIP DIRECTION 1	=	227
5.	DIP 2	= 81	6.	DIP DIRECTION 2	=	343
7.	DIP 3	= 0	8.	DIP DIRECTION 3	=	270
9.	DIP 4	= 70	10.	DIP DIRECTION 4	=	270
15.	U1	= 0	16.	U2	=	0

TABLE OF FACTOR OF SAFETY

		FRICTION ANGLES										
COHESION		20	22	24	26	28	30	32	34	36	38	40
0	!	.64	.71	.78	.85	.93	1.01	1.10	1.18	1.27	1.37	1.47
2	!	.68	.75	.83	.90	.98	1.06	1.14	1.23	1.32	1.42	1.52
4	!	.73	.80	.87	.95	1.02	1.10	1.19	1.28	1.37	1.46	1.57
6	!	.78	.85	.92	.99	1.07	1.15	1.23	1.32	1.41	1.51	1.61
8	!	.82	.89	.97	1.04	1.12	1.20	1.28	1.37	1.46	1.56	1.66
10	!	.87	.94	1.01	1.09	1.16	1.24	1.33	1.42	1.51	1.60	1.71
12	!	.91	.99	1.06	1.13	1.21	1.29	1.37	1.46	1.55	1.65	1.75
14	!	.96	1.03	1.10	1.18	1.26	1.34	1.42	1.51	1.60	1.70	1.80
16	!	1.01	1.08	1.15	1.23	1.30	1.38	1.47	1.55	1.65	1.74	1.84
18	!	1.05	1.12	1.20	1.27	1.35	1.43	1.51	1.60	1.69	1.79	1.89
20	!	1.10	1.17	1.24	1.32	1.40	1.48	1.56	1.65	1.74	1.84	1.94

CONTACT ON BOTH PLANES
TOTAL STRESS ON PLANE 1 = 166.30
TOTAL STRESS ON PLANE 2 = 63.22
WATER PRESSURE RATIO U/UMAX = 0.00

WEDGE 2 - DRY CONDITION

SHORT WEDGE STABILITY ANALYSIS

INPUT PARAMETERS

1.	GAMMAD =	34	2.	HEIGHT H =	16
3.	DIP 1 =	43	4.	DIP DIRECTION 1 =	227
5.	DIP 2 =	81	6.	DIP DIRECTION 2 =	343
7.	DIP 3 =	0	8.	DIP DIRECTION 3 =	270
9.	DIP 4 =	70	10.	DIP DIRECTION 4 =	270
15.	U1 =	13	16.	U2 =	13

TABLE OF FACTOR OF SAFETY

	FRICTION ANGLES										
COHESION	20	22	24	26	28	30	32	34	36	38	40
0 !	.53	.58	.64	.71	.77	.84	.91	.98	1.05	1.13	1.22
2 !	.57	.63	.69	.75	.82	.88	.95	1.03	1.10	1.18	1.27
4 !	.62	.68	.74	.80	.86	.93	1.00	1.07	1.15	1.23	1.31
6 !	.67	.72	.78	.85	.91	.98	1.05	1.12	1.19	1.27	1.36
8 !	.71	.77	.83	.89	.96	1.02	1.09	1.16	1.24	1.32	1.41
10 !	.76	.82	.88	.94	1.00	1.07	1.14	1.21	1.29	1.37	1.45
12 !	.80	.86	.92	.99	1.05	1.12	1.19	1.26	1.33	1.41	1.50
14 !	.85	.91	.97	1.03	1.10	1.16	1.23	1.30	1.38	1.46	1.54
16 !	.90	.96	1.02	1.08	1.14	1.21	1.28	1.35	1.43	1.51	1.59
18 !	.94	1.00	1.06	1.12	1.19	1.26	1.33	1.40	1.47	1.55	1.64
20 !	.99	1.05	1.11	1.17	1.24	1.30	1.37	1.44	1.52	1.60	1.68

CONTACT ON BOTH PLANES
TOTAL STRESS ON PLANE 1 = 139.90
TOTAL STRESS ON PLANE 2 = 50.22
WATER PRESSURE RATIO U/UMAX = 0.50

WEDGE 2 - WITH WATER - NORMAL RANGE OF C AND PHI

BERM FAILURE BACK ANALYSIS

A. LOCATION:

PIT:	MNZN ULT	NORTHING:	
BENCH:	1340	EASTING:	
WALL:	EAST	ELEVATION:	
DOMAIN:	D2		

FAILURE MODE: WEDGE

B. ORIENTATION:

PLANE 1		PLANE 2		PLANE 3(CREST)		PLANE 4(FACE)	
DIP:	62	DIP:	84	DIP:	0	DIP:	70
DIP DIR:	236	DIP DIR:	352	DIP DIR:	270	DIP DIR:	270

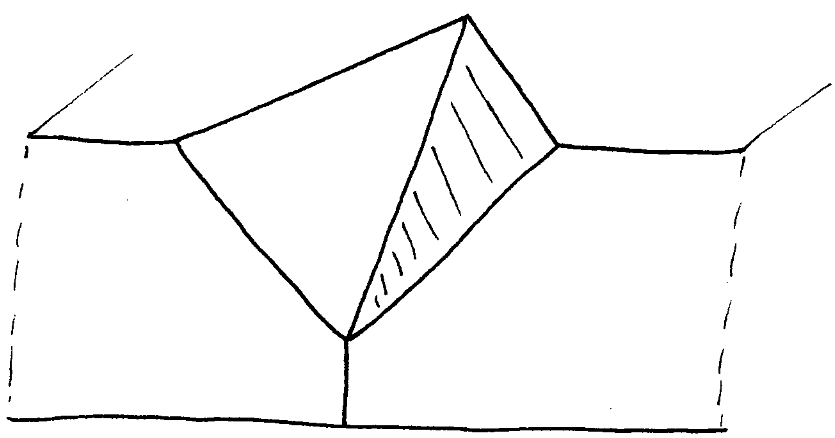
C. DIMENSIONS:

WIDTH ALONG CREST:	8.0	HEIGHT TOE/CREST:	17.0
LENGTH INTERSECTION:	25.0	ESTIMATED VOLUME:	

D. GEOLOGY:

PLANE 1		PLANE 2	
DISCONTINUITY TYPE:	MJ	DISCONTINUITY TYPE:	MJ
ROUGHNESS ANGLE:	5	ROUGHNESS ANGLE:	5
GOUGE TYPE:	NONE	GOUGE TYPE:	NONE
WATER CODE:	3	WATER CODE:	3
ROCK TYPE:	GBR		

E. SKETCH:



DATE: 85/08/28 RECORDED BY: TS / JM FAILURE NUMBER: 3

SHORT WEDGE STABILITY ANALYSIS

INPUT PARAMETERS

1. GAMMAD	=	34	2. HEIGHT H	=	17
3. DIP 1	=	62	4. DIP DIRECTION 1	=	236
5. DIP 2	=	84	6. DIP DIRECTION 2	=	352
7. DIP 3	=	0	8. DIP DIRECTION 3	=	270
9. DIP 4	=	70	10. DIP DIRECTION 4	=	270
15. U1	=	0	16. U2	=	0

TABLE OF FACTOR OF SAFETY

	FRICTION ANGLES										
COHESION	20	22	24	26	28	30	32	34	36	38	40
0 !	.37	.41	.45	.50	.54	.59	.64	.69	.74	.80	.86
2 !	.45	.49	.54	.58	.62	.67	.72	.77	.83	.88	.94
4 !	.53	.57	.62	.66	.71	.75	.80	.85	.91	.96	1.02
6 !	.61	.65	.70	.74	.79	.83	.88	.93	.99	1.04	1.10
8 !	.69	.73	.78	.82	.87	.91	.96	1.01	1.07	1.12	1.18
10 !	.77	.82	.86	.90	.95	.99	1.04	1.09	1.15	1.20	1.26
12 !	.86	.90	.94	.98	1.03	1.08	1.12	1.18	1.23	1.29	1.35
14 !	.94	.98	1.02	1.06	1.11	1.16	1.21	1.26	1.31	1.37	1.43
16 !	1.02	1.06	1.10	1.14	1.19	1.24	1.29	1.34	1.39	1.45	1.51
18 !	1.10	1.14	1.18	1.23	1.27	1.32	1.37	1.42	1.47	1.53	1.59
20 !	1.18	1.22	1.26	1.31	1.35	1.40	1.45	1.50	1.55	1.61	1.67

CONTACT ON BOTH PLANES
TOTAL STRESS ON PLANE 1 = 50.29
TOTAL STRESS ON PLANE 2 = 26.14
WATER PRESSURE RATIO U/UMAX = 0.00

WEDGE 3 - DRY CONDITION - NORMAL RANGE OF C AND PHI

SHORT WEDGE STABILITY ANALYSIS

INPUT PARAMETERS

1.	GAMMAD = 34		2.	HEIGHT H = 17	
3.	DIP 1 = 62		4.	DIP DIRECTION 1 = 236	
5.	DIP 2 = 84		6.	DIP DIRECTION 2 = 352	
7.	DIP 3 = 0		8.	DIP DIRECTION 3 = 270	
9.	DIP 4 = 70		10.	DIP DIRECTION 4 = 270	
15.	U1 = 0		16.	U2 = 0	

TABLE OF FACTOR OF SAFETY

		FRICTION ANGLES										
COHESION		30	32	34	36	38	40	42	44	46	48	50
0	!	.59	.64	.69	.74	.80	.86	.92	.99	1.06	1.14	1.22
1	!	.63	.68	.73	.78	.84	.90	.96	1.03	1.10	1.18	1.26
2	!	.67	.72	.77	.83	.88	.94	1.00	1.07	1.14	1.22	1.31
3	!	.71	.76	.81	.87	.92	.98	1.05	1.11	1.18	1.26	1.35
4	!	.75	.80	.85	.91	.96	1.02	1.09	1.15	1.22	1.30	1.39
5	!	.79	.84	.89	.95	1.00	1.06	1.13	1.19	1.27	1.34	1.43
6	!	.83	.88	.93	.99	1.04	1.10	1.17	1.23	1.31	1.38	1.47
7	!	.87	.92	.97	1.03	1.08	1.14	1.21	1.27	1.35	1.42	1.51
8	!	.91	.96	1.01	1.07	1.12	1.18	1.25	1.31	1.39	1.46	1.55
9	!	.95	1.00	1.05	1.11	1.16	1.22	1.29	1.35	1.43	1.50	1.59
10	!	.99	1.04	1.09	1.15	1.20	1.26	1.33	1.40	1.47	1.54	1.63

CONTACT ON BOTH PLANES
TOTAL STRESS ON PLANE 1 = 50.29
TOTAL STRESS ON PLANE 2 = 26.14
WATER PRESSURE RATIO U/UMAX = 0.00

WEDGE 3 - DRY CONDITION - EXTENDED RANGE OF C AND PHI

SHORT WEDGE STABILITY ANALYSIS

INPUT PARAMETERS

1.	GAMMAD = 34	2.	HEIGHT H = 17		
3.	DIP 1 = 62	4.	DIP DIRECTION 1 = 236		
5.	DIP 2 = 84	6.	DIP DIRECTION 2 = 352		
7.	DIP 3 = 0	8.	DIP DIRECTION 3 = 270		
9.	DIP 4 = 70	10.	DIP DIRECTION 4 = 270		
15.	U1 = 14	16.	U2 = 14		

TABLE OF FACTOR OF SAFETY

	FRICTION ANGLES										
COHESION	20	22	24	26	28	30	32	34	36	38	40
0 !	.16	.18	.20	.22	.24	.26	.29	.31	.33	.36	.39
2 !	.25	.26	.28	.30	.32	.34	.37	.39	.41	.44	.47
4 !	.33	.34	.36	.38	.40	.43	.45	.47	.50	.52	.55
6 !	.41	.43	.44	.46	.49	.51	.53	.55	.58	.60	.63
8 !	.49	.51	.53	.55	.57	.59	.61	.63	.66	.68	.71
10 !	.57	.59	.61	.63	.65	.67	.69	.71	.74	.76	.79
12 !	.65	.67	.69	.71	.73	.75	.77	.79	.82	.84	.87
14 !	.73	.75	.77	.79	.81	.83	.85	.87	.90	.92	.95
16 !	.81	.83	.85	.87	.89	.91	.93	.96	.98	1.01	1.03
18 !	.89	.91	.93	.95	.97	.99	1.01	1.04	1.06	1.09	1.11
20 !	.97	.99	1.01	1.03	1.05	1.07	1.09	1.12	1.14	1.17	1.19

CONTACT ON BOTH PLANES
TOTAL STRESS ON PLANE 1 = 22.37
TOTAL STRESS ON PLANE 2 = 12.14
WATER PRESSURE RATIO U/UMAX = 0.50

WEDGE 3 - WITH WATER - NORMAL RANGE OF C AND PHI

SHORT WEDGE STABILITY ANALYSIS

INPUT PARAMETERS

1.	GAMMAD =	34	2.	HEIGHT H =	17
3.	DIP 1 =	62	4.	DIP DIRECTION 1 =	236
5.	DIP 2 =	84	6.	DIP DIRECTION 2 =	352
7.	DIP 3 =	0	8.	DIP DIRECTION 3 =	270
9.	DIP 4 =	70	10.	DIP DIRECTION 4 =	270
15.	U1 =	14	16.	U2 =	14

TABLE OF FACTOR OF SAFETY

	FRICTION ANGLES										
COHESION	30	33	36	39	42	45	48	51	54	57	60
0 !	.26	.30	.33	.37	.41	.46	.51	.57	.64	.71	.80
2 !	.34	.38	.41	.45	.50	.54	.59	.65	.72	.79	.88
4 !	.43	.46	.50	.53	.58	.62	.67	.73	.80	.87	.96
6 !	.51	.54	.58	.61	.66	.70	.75	.81	.88	.95	1.04
8 !	.59	.62	.66	.70	.74	.78	.84	.89	.96	1.04	1.13
10 !	.67	.70	.74	.78	.82	.86	.92	.97	1.04	1.12	1.21
12 !	.75	.78	.82	.86	.90	.95	1.00	1.06	1.12	1.20	1.29
14 !	.83	.86	.90	.94	.98	1.03	1.08	1.14	1.20	1.28	1.37
16 !	.91	.94	.98	1.02	1.06	1.11	1.16	1.22	1.28	1.36	1.45
18 !	.99	1.02	1.06	1.10	1.14	1.19	1.24	1.30	1.36	1.44	1.53
20 !	1.07	1.11	1.14	1.18	1.22	1.27	1.32	1.38	1.44	1.52	1.61

CONTACT ON BOTH PLANES
TOTAL STRESS ON PLANE 1 = 22.37
TOTAL STRESS ON PLANE 2 = 12.14
WATER PRESSURE RATIO U/UMAX = 0.50

WEDGE 3 - WITH WATER - EXTENDED RANGE OF C AND PHI

BERM FAILURE BACK ANALYSIS

A. LOCATION:

PIT:	MNZN ULT	NORTHING:	
BENCH:	1340	EASTING:	
WALL:	EAST	ELEVATION:	
DOMAIN:	D2		

FAILURE MODE: PLANE

B. ORIENTATION:

PLANE 1		PLANE 2		PLANE 3(CREST)		PLANE 4(FACE)	
DIP:	34	DIP:	256	DIP:	0	DIP:	70
DIP DIR:	256	DIP DIR:	346	DIP DIR:	270	DIP DIR:	270

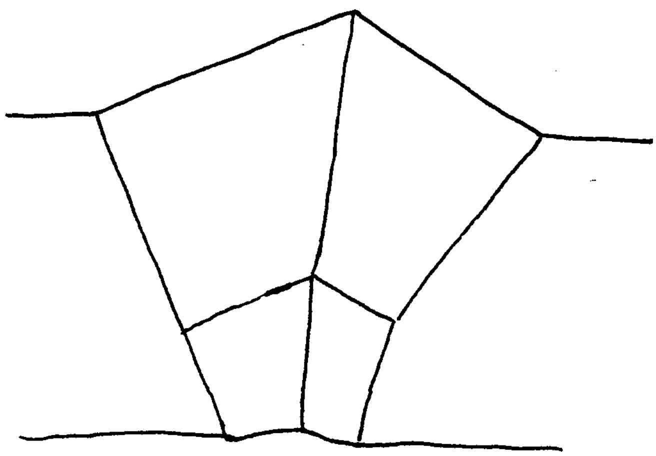
C. DIMENSIONS:

WIDTH ALONG CREST:	10.0	HEIGHT TOE/CREST:	20.0
LENGTH INTERSECTION:	25.0	ESTIMATED VOLUME:	

D. GEOLOGY:

PLANE 1		PLANE 2	
DISCONTINUITY TYPE:	MJ	DISCONTINUITY TYPE:	JN
ROUGHNESS ANGLE:	10	ROUGHNESS ANGLE:	0
GOUGE TYPE:	NONE	GOUGE TYPE:	NONE
WATER CODE:	3	WATER CODE:	1
ROCK TYPE:	GBR		

E. SKETCH:



DATE: 85/08/28 RECORDED BY: TS / JM FAILURE NUMBER: 4

SHORT WEDGE STABILITY ANALYSIS

INPUT PARAMETERS

1.	GAMMAD	= 34	2.	HEIGHT H	=	20
3.	DIP 1	= 34	4.	DIP DIRECTION 1	=	256
5.	DIP 2	= 90	6.	DIP DIRECTION 2	=	346
7.	DIP 3	= 0	8.	DIP DIRECTION 3	=	270
9.	DIP 4	= 70	10.	DIP DIRECTION 4	=	270
15.	U1	= 0	16.	U2	=	0

TABLE OF FACTOR OF SAFETY

COHESION		FRICTION ANGLES 20	23	26	29	32	35	38	41	44	47	50
0	!	.53	.62	.72	.82	.92	1.03	1.15	1.28	1.43	1.58	1.76
2	!	.56	.65	.75	.85	.95	1.06	1.18	1.31	1.46	1.61	1.79
4	!	.59	.68	.78	.87	.98	1.09	1.21	1.34	1.48	1.64	1.82
6	!	.62	.71	.81	.90	1.01	1.12	1.24	1.37	1.51	1.67	1.85
8	!	.65	.74	.83	.93	1.04	1.15	1.27	1.40	1.54	1.70	1.88
10	!	.68	.77	.86	.96	1.07	1.18	1.30	1.43	1.57	1.73	1.91
12	!	.71	.80	.89	.99	1.10	1.21	1.33	1.46	1.60	1.76	1.94
14	!	.74	.83	.92	1.02	1.12	1.24	1.36	1.49	1.63	1.79	1.97
16	!	.77	.86	.95	1.05	1.15	1.27	1.39	1.52	1.66	1.82	1.99
18	!	.80	.89	.98	1.08	1.18	1.29	1.41	1.55	1.69	1.85	2.02
20	!	.82	.91	1.01	1.11	1.21	1.32	1.44	1.57	1.72	1.88	2.05

CONTACT ON BOTH PLANES
TOTAL STRESS ON PLANE 1 = 834.48
TOTAL STRESS ON PLANE 2 = 0.03
WATER PRESSURE RATIO U/UMAX = 0.00

WEDGE 4 - DRY CONDITION - NORMAL LEVEL OF C AND PHI

SHORT WEDGE STABILITY ANALYSIS

INPUT PARAMETERS

1.	GAMMAD	=	34	2.	HEIGHT H	=	20
3.	DIP 1	=	34	4.	DIP DIRECTION 1	=	256
5.	DIP 2	=	90	6.	DIP DIRECTION 2	=	346
7.	DIP 3	=	0	8.	DIP DIRECTION 3	=	270
9.	DIP 4	=	70	10.	DIP DIRECTION 4	=	270
15.	U1	=	16	16.	U2	=	16

TABLE OF FACTOR OF SAFETY

		FRICTION ANGLES										
COHESION		20	23	26	29	32	35	38	41	44	47	50
0	!	.46	.54	.62	.70	.79	.89	.99	1.11	1.23	1.37	1.52
2	!	.49	.56	.64	.73	.82	.92	1.02	1.13	1.25	1.39	1.54
4	!	.51	.59	.67	.75	.84	.94	1.04	1.16	1.28	1.42	1.57
6	!	.54	.61	.69	.78	.87	.97	1.07	1.18	1.31	1.44	1.59
8	!	.56	.64	.72	.81	.90	.99	1.10	1.21	1.33	1.47	1.62
10	!	.59	.66	.75	.83	.92	1.02	1.12	1.23	1.36	1.49	1.65
12	!	.61	.69	.77	.86	.95	1.04	1.15	1.26	1.38	1.52	1.67
14	!	.64	.72	.80	.88	.97	1.07	1.17	1.28	1.41	1.54	1.70
16	!	.66	.74	.82	.91	1.00	1.09	1.20	1.31	1.43	1.57	1.72
18	!	.69	.77	.85	.93	1.02	1.12	1.22	1.34	1.46	1.59	1.75
20	!	.71	.79	.87	.96	1.05	1.14	1.25	1.36	1.48	1.62	1.77

CONTACT ON PLANE 1 ONLY
TOTAL STRESS ON PLANE 1 = 719.75
WATER PRESSURE RATIO U/UMAX = 0.49

WEDGE 4 - WITH WATER - NORMAL RANGE OF C AND PHI

BERM FAILURE BACK ANALYSIS

A. LOCATION:

PIT:	MNZN ULT	NORTHING:	
BENCH:	1340	EASTING:	
WALL:	EAST	ELEVATION:	
DOMAIN:	D2		

FAILURE MODE: WEDGE

B. ORIENTATION:

	PLANE 1	PLANE 2	PLANE 3(CREST)	PLANE 4(FACE)
DIP:	57	69	0	70
DIP DIR:	229	321	270	270

C. DIMENSIONS:

WIDTH ALONG CREST:	5.0	HEIGHT TOE/CREST:	15.0
LENGTH INTERSECTION:	7.0	ESTIMATED VOLUME:	

D. GEOLOGY:

	PLANE 1	PLANE 2
DISCONTINUITY TYPE:	JN	JN
ROUGHNESS ANGLE:	0	7
GOUGE TYPE:	NONE	NONE
WATER CODE:	2	2
ROCK TYPE:	GBR	

E. SKETCH:

DATE: 85/08/28 RECORDED BY: TS / JM FAILURE NUMBER: 5

SHORT WEDGE STABILITY ANALYSIS

INPUT PARAMETERS

1. GAMMAD	=	34	2. HEIGHT H	=	15
3. DIP 1	=	57	4. DIP DIRECTION 1	=	229
5. DIP 2	=	69	6. DIP DIRECTION 2	=	321
7. DIP 3	=	0	8. DIP DIRECTION 3	=	270
9. DIP 4	=	70	10. DIP DIRECTION 4	=	270
15. U1	=	0	16. U2	=	0

TABLE OF FACTOR OF SAFETY

COHESION		FRICTION ANGLES 20	23	26	29	32	35	38	41	44	47	50
0	!	.35	.41	.47	.53	.60	.68	.76	.84	.94	1.04	1.16
2	!	.41	.47	.53	.60	.67	.74	.82	.90	1.00	1.10	1.22
4	!	.47	.53	.59	.66	.73	.80	.88	.96	1.06	1.16	1.28
6	!	.53	.59	.65	.72	.79	.86	.94	1.03	1.12	1.22	1.34
8	!	.60	.65	.72	.78	.85	.92	1.00	1.09	1.18	1.29	1.40
10	!	.66	.72	.78	.84	.91	.98	1.06	1.15	1.24	1.35	1.46
12	!	.72	.78	.84	.90	.97	1.05	1.13	1.21	1.30	1.41	1.52
14	!	.78	.84	.90	.97	1.03	1.11	1.19	1.27	1.37	1.47	1.59
16	!	.84	.90	.96	1.03	1.10	1.17	1.25	1.33	1.43	1.53	1.65
18	!	.90	.96	1.02	1.09	1.16	1.23	1.31	1.40	1.49	1.59	1.71
20	!	.96	1.02	1.08	1.15	1.22	1.29	1.37	1.46	1.55	1.65	1.77

CONTACT ON BOTH PLANES
TOTAL STRESS ON PLANE 1 = 47.36
TOTAL STRESS ON PLANE 2 = 26.10
WATER PRESSURE RATIO U/UMAX = 0.00

WEDGE 5 - DRY CONDITION - NORMAL RANGE OF C AND PHI

SHORT WEDGE STABILITY ANALYSIS

INPUT PARAMETERS

1. GAMMAD	=	34	2. HEIGHT H	=	15
3. DIP 1	=	57	4. DIP DIRECTION 1	=	229
5. DIP 2	=	69	6. DIP DIRECTION 2	=	321
7. DIP 3	=	0	8. DIP DIRECTION 3	=	270
9. DIP 4	=	70	10. DIP DIRECTION 4	=	270
15. U1	=	12	16. U2	=	12

TABLE OF FACTOR OF SAFETY

	FRICTION ANGLES										
COHESION	20	23	26	29	32	35	38	41	44	47	50
0 !	.22	.25	.29	.33	.37	.42	.47	.52	.58	.64	.72
2 !	.28	.31	.35	.39	.43	.48	.53	.58	.64	.71	.78
4 !	.34	.37	.41	.45	.50	.54	.59	.64	.70	.77	.84
6 !	.40	.44	.47	.51	.56	.60	.65	.71	.76	.83	.90
8 !	.46	.50	.54	.58	.62	.66	.71	.77	.83	.89	.96
10 !	.52	.56	.60	.64	.68	.73	.78	.83	.89	.95	1.02
12 !	.58	.62	.66	.70	.74	.79	.84	.89	.95	1.01	1.09
14 !	.65	.68	.72	.76	.80	.85	.90	.95	1.01	1.07	1.15
16 !	.71	.74	.78	.82	.87	.91	.96	1.01	1.07	1.14	1.21
18 !	.77	.81	.84	.88	.93	.97	1.02	1.07	1.13	1.20	1.27
20 !	.83	.87	.91	.95	.99	1.03	1.08	1.14	1.19	1.26	1.33

CONTACT ON BOTH PLANES
TOTAL STRESS ON PLANE 1 = 31.54
TOTAL STRESS ON PLANE 2 = 14.10
WATER PRESSURE RATIO U/UMAX = 0.49

WEDGE 5 - WITH WATER - NORMAL RANGE OF C AND PHI

SHORT WEDGE STABILITY ANALYSIS

INPUT PARAMETERS

1.	GAMMAD	= 34	2.	HEIGHT H	= 15
3.	DIP 1	= 57	4.	DIP DIRECTION 1	= 229
5.	DIP 2	= 69	6.	DIP DIRECTION 2	= 321
7.	DIP 3	= 0	8.	DIP DIRECTION 3	= 270
9.	DIP 4	= 70	10.	DIP DIRECTION 4	= 270
15.	U1	= 12	16.	U2	= 12

TABLE OF FACTOR OF SAFETY

COHESION	FRICTION ANGLES 40	42	44	46	48	50	52	54	56	58	60
0 !	.50	.54	.58	.62	.67	.72	.77	.83	.89	.96	1.04
2 !	.56	.60	.64	.68	.73	.78	.83	.89	.95	1.03	1.10
4 !	.63	.66	.70	.74	.79	.84	.89	.95	1.02	1.09	1.17
6 !	.69	.72	.76	.81	.85	.90	.95	1.01	1.08	1.15	1.23
8 !	.75	.79	.83	.87	.91	.96	1.02	1.07	1.14	1.21	1.29
10 !	.81	.85	.89	.93	.97	1.02	1.08	1.14	1.20	1.27	1.35
12 !	.87	.91	.95	.99	1.04	1.09	1.14	1.20	1.26	1.33	1.41
14 !	.93	.97	1.01	1.05	1.10	1.15	1.20	1.26	1.32	1.39	1.47
16 !	.99	1.03	1.07	1.11	1.16	1.21	1.26	1.32	1.38	1.46	1.54
18 !	1.06	1.09	1.13	1.18	1.22	1.27	1.32	1.38	1.45	1.52	1.60
20 !	1.12	1.15	1.19	1.24	1.28	1.33	1.38	1.44	1.51	1.58	1.66

CONTACT ON BOTH PLANES
TOTAL STRESS ON PLANE 1 = 31.54
TOTAL STRESS ON PLANE 2 = 14.10
WATER PRESSURE RATIO U/UMAX = 0.49

WEDGE 5 - WITH WATER - EXTENDED RANGE OF C AND PHI

BERM FAILURE BACK ANALYSIS

A. LOCATION:

PIT:	MNZN ULT	NORTHING:	
BENCH:	1340	EASTING:	
WALL:	EAST	ELEVATION:	
DOMAIN:	D2		

FAILURE MODE: WEDGE

B. ORIENTATION:

PLANE 1		PLANE 2		PLANE 3(CREST)		PLANE 4(FACE)	
DIP:	48	DIP:	87	DIP:	0	DIP:	70
DIP DIR:	276	DIP DIR:	341	DIP DIR:	290	DIP DIR:	290

C. DIMENSIONS:

WIDTH ALONG CREST:	5.0	HEIGHT TOE/CREST:	8.0
LENGTH INTERSECTION:	10.0	ESTIMATED VOLUME:	

D. GEOLOGY:

PLANE 1		PLANE 2	
DISCONTINUITY TYPE:	JN	DISCONTINUITY TYPE:	JN
ROUGHNESS ANGLE:	2	ROUGHNESS ANGLE:	2
GOUGE TYPE:	NONE	GOUGE TYPE:	NONE
WATER CODE:	2	WATER CODE:	2
ROCK TYPE:	LAPILLI		

E. SKETCH:

DATE: 85/08/28 RECORDED BY: TS / JM FAILURE NUMBER: 6

SHORT WEDGE STABILITY ANALYSIS

INPUT PARAMETERS

1. GAMMAD	= 34	2. HEIGHT H		=	8
3. DIP 1	= 48	4. DIP DIRECTION 1		=	276
5. DIP 2	= 87	6. DIP DIRECTION 2		=	341
7. DIP 3	= 0	8. DIP DIRECTION 3		=	290
9. DIP 4	= 70	10. DIP DIRECTION 4		=	290
15. U1	= 0	16. U2		=	0

TABLE OF FACTOR OF SAFETY

		FRICTION ANGLES										
COHESION		20	23	26	29	32	35	38	41	44	47	50
0	!	.32	.38	.43	.49	.56	.63	.70	.78	.86	.96	1.07
2	!	.40	.46	.52	.58	.64	.71	.78	.86	.95	1.04	1.15
4	!	.48	.54	.60	.66	.72	.79	.86	.94	1.03	1.12	1.23
6	!	.57	.62	.68	.74	.80	.87	.94	1.02	1.11	1.20	1.31
8	!	.65	.70	.76	.82	.88	.95	1.02	1.10	1.19	1.29	1.39
10	!	.73	.78	.84	.90	.96	1.03	1.10	1.18	1.27	1.37	1.47
12	!	.81	.86	.92	.98	1.04	1.11	1.19	1.26	1.35	1.45	1.55
14	!	.89	.95	1.00	1.06	1.13	1.19	1.27	1.35	1.43	1.53	1.64
16	!	.97	1.03	1.08	1.14	1.21	1.27	1.35	1.43	1.51	1.61	1.72
18	!	1.05	1.11	1.16	1.22	1.29	1.36	1.43	1.51	1.59	1.69	1.80
20	!	1.13	1.19	1.25	1.31	1.37	1.44	1.51	1.59	1.68	1.77	1.88

CONTACT ON PLANE 1 ONLY
TOTAL STRESS ON PLANE 1 = 95.78
WATER PRESSURE RATIO U/UMAX = 0.00

WEDGE 6 - DRY CONDITION - NORMAL RANGE OF C AND PHI

SHORT WEDGE STABILITY ANALYSIS

INPUT PARAMETERS

1. GAMMAD	= 34	2.	HEIGHT H	= 8
3. DIP 1	= 48	4.	DIP DIRECTION 1	= 276
5. DIP 2	= 87	6.	DIP DIRECTION 2	= 341
7. DIP 3	= 0	8.	DIP DIRECTION 3	= 290
9. DIP 4	= 70	10.	DIP DIRECTION 4	= 290
15. U1	= 7	16.	U2	= 7

TABLE OF FACTOR OF SAFETY

		FRICTION ANGLES										
COHESION		20	23	26	29	32	35	38	41	44	47	50
0	!	.21	.24	.28	.32	.36	.40	.45	.50	.56	.62	.69
2	!	.29	.32	.36	.40	.44	.48	.53	.58	.64	.70	.77
4	!	.37	.40	.44	.48	.52	.56	.61	.66	.72	.78	.85
6	!	.45	.48	.52	.56	.60	.64	.69	.74	.80	.86	.93
8	!	.53	.56	.60	.64	.68	.72	.77	.82	.88	.94	1.01
10	!	.61	.64	.68	.72	.76	.80	.85	.90	.96	1.02	1.09
12	!	.69	.72	.76	.80	.84	.88	.93	.98	1.04	1.10	1.17
14	!	.77	.80	.84	.88	.92	.96	1.01	1.06	1.12	1.18	1.25
16	!	.84	.88	.92	.96	1.00	1.04	1.09	1.14	1.20	1.26	1.33
18	!	.92	.96	1.00	1.04	1.08	1.12	1.17	1.22	1.28	1.34	1.41
20	!	1.00	1.04	1.08	1.12	1.16	1.20	1.25	1.30	1.36	1.42	1.49

CONTACT ON PLANE 1 ONLY
TOTAL STRESS ON PLANE 1 = 63.13
WATER PRESSURE RATIO U/UMAX = 0.54

WEDGE 6 - WITH WATER - NORMAL RANGE OF C AND PHI

BERM FAILURE BACK ANALYSIS

A. LOCATION:

PIT:	MNZN ULT	NORTHING:	
BENCH:	1360	EASTING:	
WALL:	EAST	ELEVATION:	
DOMAIN:	D2		

FAILURE MODE: WEDGE

B. ORIENTATION:

PLANE 1		PLANE 2		PLANE 3(CREST)		PLANE 4(FACE)	
DIP:	38	DIP:	79	DIP:	0	DIP:	70
DIP DIR:	296	DIP DIR:	204	DIP DIR:	270	DIP DIR:	270

C. DIMENSIONS:

WIDTH ALONG CREST:	3.0	HEIGHT TOE/CREST:	2.0
LENGTH INTERSECTION:	2.0	ESTIMATED VOLUME:	

D. GEOLOGY:

PLANE 1		PLANE 2	
DISCONTINUITY TYPE:	JN	DISCONTINUITY TYPE:	JN
ROUGHNESS ANGLE:	3	ROUGHNESS ANGLE:	10
GOUGE TYPE:	NONE	GOUGE TYPE:	NONE
WATER CODE:	2	WATER CODE:	2
ROCK TYPE:	LAPILLI		

E. SKETCH:

DATE: 85/08/28 RECORDED BY: TS / JM FAILURE NUMBER: 7

SHORT WEDGE STABILITY ANALYSIS

INPUT PARAMETERS

1.	GAMMAD	=	34	2.	HEIGHT H	=	2
3.	DIP 1	=	38	4.	DIP DIRECTION 1	=	296
5.	DIP 2	=	79	6.	DIP DIRECTION 2	=	204
7.	DIP 3	=	0	8.	DIP DIRECTION 3	=	270
9.	DIP 4	=	70	10.	DIP DIRECTION 4	=	270
15.	U1	=	0	16.	U2	=	0

TABLE OF FACTOR OF SAFETY

COHESION	FRICTION ANGLES 20	23	26	29	32	35	38	41	44	47	50
0 !	.51	.60	.69	.78	.88	.99	1.11	1.23	1.37	1.52	1.69
2 !	.83	.91	1.00	1.10	1.20	1.31	1.42	1.55	1.68	1.84	2.01
4 !	1.14	1.23	1.32	1.41	1.51	1.62	1.73	1.86	2.00	2.15	2.32
6 !	1.45	1.54	1.63	1.73	1.83	1.93	2.05	2.17	2.31	2.46	2.63
8 !	1.77	1.85	1.94	2.04	2.14	2.25	2.36	2.49	2.63	2.78	2.95
10 !	2.08	2.17	2.26	2.35	2.45	2.56	2.68	2.80	2.94	3.09	3.26
12 !	2.40	2.48	2.57	2.67	2.77	2.88	2.99	3.12	3.25	3.40	3.57
14 !	2.71	2.80	2.89	2.98	3.08	3.19	3.30	3.43	3.57	3.72	3.89
16 !	3.02	3.11	3.20	3.30	3.40	3.50	3.62	3.74	3.88	4.03	4.20
18 !	3.34	3.42	3.51	3.61	3.71	3.82	3.93	4.06	4.19	4.35	4.52
20 !	3.65	3.74	3.83	3.92	4.02	4.13	4.25	4.37	4.51	4.66	4.83

CONTACT ON BOTH PLANES
TOTAL STRESS ON PLANE 1 = 35.09
TOTAL STRESS ON PLANE 2 = 4.09
WATER PRESSURE RATIO U/UMAX = 0.00

WEDGE 7 - DRY CONDITION - NORMAL RANGE OF C AND PHI

SHORT WEDGE STABILITY ANALYSIS

INPUT PARAMETERS

1. GAMMAD = 34		2. HEIGHT H = 2	
3. DIP 1 = 38		4. DIP DIRECTION 1 = 296	
5. DIP 2 = 79		6. DIP DIRECTION 2 = 204	
7. DIP 3 = 0		8. DIP DIRECTION 3 = 270	
9. DIP 4 = 70		10. DIP DIRECTION 4 = 270	
15. U1 = 2		16. U2 = 2	

TABLE OF FACTOR OF SAFETY

	FRICTION ANGLES										
COHESION	20	23	26	29	32	35	38	41	44	47	50
0 !	.40	.47	.54	.61	.69	.77	.86	.96	1.07	1.18	1.32
2 !	.71	.78	.85	.92	1.00	1.09	1.18	1.27	1.38	1.50	1.63
4 !	1.03	1.09	1.16	1.24	1.32	1.40	1.49	1.59	1.69	1.81	1.94
6 !	1.34	1.41	1.48	1.55	1.63	1.71	1.80	1.90	2.01	2.13	2.26
8 !	1.65	1.72	1.79	1.87	1.94	2.03	2.12	2.21	2.32	2.44	2.57
10 !	1.97	2.04	2.11	2.18	2.26	2.34	2.43	2.53	2.64	2.75	2.89
12 !	2.28	2.35	2.42	2.49	2.57	2.66	2.75	2.84	2.95	3.07	3.20
14 !	2.60	2.66	2.73	2.81	2.89	2.97	3.06	3.16	3.26	3.38	3.51
16 !	2.91	2.98	3.05	3.12	3.20	3.28	3.37	3.47	3.58	3.70	3.83
18 !	3.22	3.29	3.36	3.44	3.51	3.60	3.69	3.78	3.89	4.01	4.14
20 !	3.54	3.60	3.68	3.75	3.83	3.91	4.00	4.10	4.21	4.32	4.46

CONTACT ON BOTH PLANES
TOTAL STRESS ON PLANE 1 = 28.45
TOTAL STRESS ON PLANE 2 = 2.09
WATER PRESSURE RATIO U/UMAX = 0.61

WEDGE 7 - WITH WATER - NORMAL RANGE OF C AND PHI

BERM FAILURE BACK ANALYSIS

A. LOCATION:

PIT:	MNZN ULT	NORTHING:	
BENCH:	1360	EASTING:	
WALL:	EAST	ELEVATION:	
DOMAIN:	D2		

FAILURE MODE: WEDGE

B. ORIENTATION:

PLANE 1		PLANE 2		PLANE 3(CREST)		PLANE 4(FACE)	
DIP:	43	DIP:	90	DIP:	0	DIP:	70
DIP DIR:	327	DIP DIR:	327	DIP DIR:	270	DIP DIR:	270

C. DIMENSIONS:

WIDTH ALONG CREST:	10.0	HEIGHT TOE/CREST:	10.0
LENGTH INTERSECTION:	15.0	ESTIMATED VOLUME:	

D. GEOLOGY:

PLANE 1		PLANE 2	
DISCONTINUITY TYPE:	MJ	DISCONTINUITY TYPE:	JN
ROUGHNESS ANGLE:	3	ROUGHNESS ANGLE:	2 cm.
GOUGE TYPE:	NONE	GOUGE TYPE:	NONE
WATER CODE:	3	WATER CODE:	3
ROCK TYPE:	GBR		

E. SKETCH:

DATE: 85/08/28 RECORDED BY: TS / JM FAILURE NUMBER: 8

SHORT WEDGE STABILITY ANALYSIS

INPUT PARAMETERS

1.	GAMMAD =	34	2.	HEIGHT H =	10
3.	DIP 1 =	43	4.	DIP DIRECTION 1 =	237
5.	DIP 2 =	90	6.	DIP DIRECTION 2 =	327
7.	DIP 3 =	0	8.	DIP DIRECTION 3 =	270
9.	DIP 4 =	70	10.	DIP DIRECTION 4 =	270
15.	U1 =	0	16.	U2 =	0

TABLE OF FACTOR OF SAFETY

	FRICTION ANGLES										
COHESION	20	23	26	29	32	35	38	41	44	47	50
0 !	.39	.45	.52	.59	.67	.75	.83	.93	1.03	1.15	1.27
2 !	.47	.54	.60	.68	.75	.83	.92	1.01	1.12	1.23	1.36
4 !	.56	.62	.69	.76	.84	.92	1.00	1.10	1.20	1.32	1.44
6 !	.64	.71	.78	.85	.92	1.00	1.09	1.18	1.29	1.40	1.53
8 !	.73	.79	.86	.93	1.01	1.09	1.18	1.27	1.37	1.49	1.62
10 !	.81	.88	.95	1.02	1.09	1.17	1.26	1.36	1.46	1.57	1.70
12 !	.90	.96	1.03	1.10	1.18	1.26	1.35	1.44	1.55	1.66	1.79
14 !	.99	1.05	1.12	1.19	1.27	1.35	1.43	1.53	1.63	1.75	1.87
16 !	1.07	1.14	1.20	1.28	1.35	1.43	1.52	1.61	1.72	1.83	1.96
18 !	1.16	1.22	1.29	1.36	1.44	1.52	1.60	1.70	1.80	1.92	2.04
20 !	1.24	1.31	1.38	1.45	1.52	1.60	1.69	1.78	1.89	2.00	2.13

CONTACT ON BOTH PLANES
TOTAL STRESS ON PLANE 1 = 81.46
TOTAL STRESS ON PLANE 2 = 0.00
WATER PRESSURE RATIO U/UMAX = 0.00

WEDGE 8 - DRY CONDITION - NORMAL RANGE OF C AND PHI

SHORT WEDGE STABILITY ANALYSIS

INPUT PARAMETERS

1.	GAMMAD	= 34	2.	HEIGHT H	= 10
3.	DIP 1	= 43	4.	DIP DIRECTION 1	= 237
5.	DIP 2	= 90	6.	DIP DIRECTION 2	= 327
7.	DIP 3	= 0	8.	DIP DIRECTION 3	= 270
9.	DIP 4	= 70	10.	DIP DIRECTION 4	= 270
15.	U1	= 8	16.	U2	= 8

TABLE OF FACTOR OF SAFETY

	FRICTION ANGLES										
COHESION	20	23	26	29	32	35	38	41	44	47	50
0 !	.30	.35	.40	.46	.51	.58	.64	.72	.80	.89	.98
2 !	.36	.41	.46	.51	.57	.64	.70	.78	.86	.94	1.04
4 !	.42	.47	.52	.57	.63	.69	.76	.83	.91	1.00	1.10
6 !	.47	.52	.58	.63	.69	.75	.82	.89	.97	1.06	1.16
8 !	.53	.58	.64	.69	.75	.81	.88	.95	1.03	1.12	1.22
10 !	.59	.64	.70	.75	.81	.87	.94	1.01	1.09	1.18	1.28
12 !	.65	.70	.75	.81	.87	.93	1.00	1.07	1.15	1.24	1.34
14 !	.71	.76	.81	.87	.93	.99	1.06	1.13	1.21	1.30	1.40
16 !	.77	.82	.87	.93	.99	1.05	1.12	1.19	1.27	1.36	1.46
18 !	.83	.88	.93	.99	1.05	1.11	1.18	1.25	1.33	1.42	1.52
20 !	.89	.94	.99	1.05	1.10	1.17	1.23	1.31	1.39	1.48	1.58

CONTACT ON PLANE 1 ONLY
TOTAL STRESS ON PLANE 1 = 63.40
WATER PRESSURE RATIO U/UMAX = 0.49

WEDGE 8 - WITH WATER - NORMAL RANGE OF C AND PHI

APPENDIX D. APPROXIMATE ROCK STRENGTH CLASSIFICATION

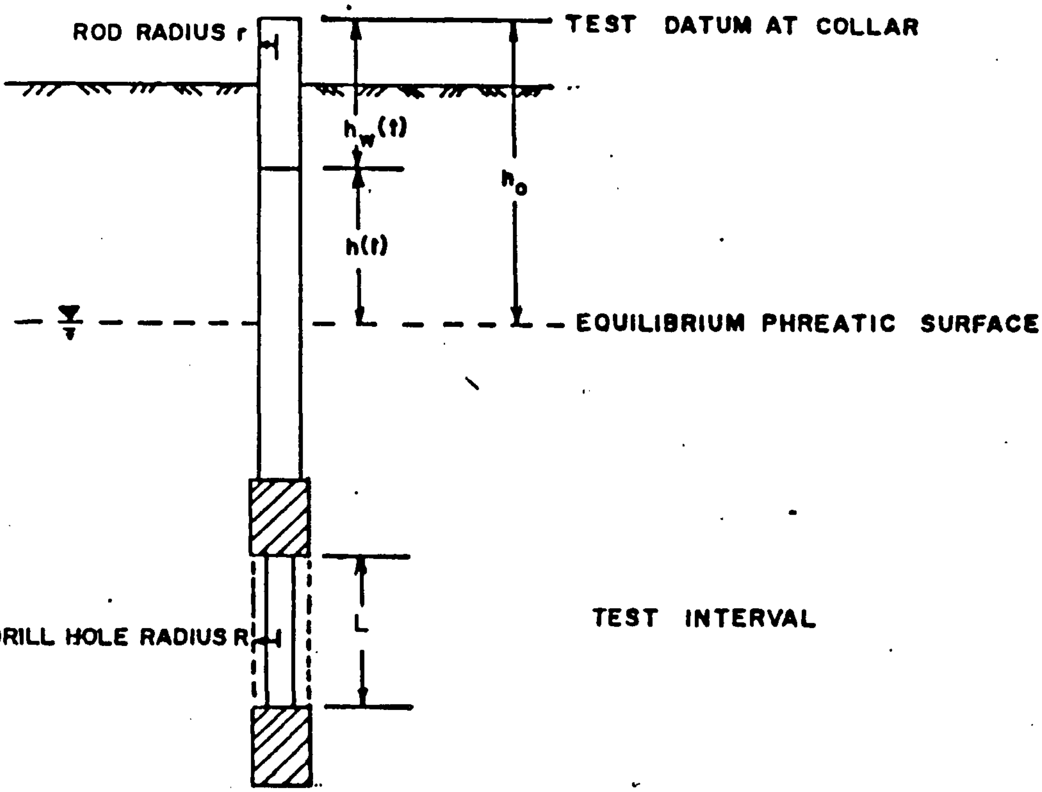
(chart after Hoek & Bray, 1981)

No.	*Description*	*Uniaxial compressive strength* lb/in^2	kg/cm^2	*MPa*	*Examples*
S1	VERY SOFT SOIL - easily moulded with fingers, shows distinct heel marks.	<5	<0.4	<0.04	
S2	SOFT SOIL - moulds with strong pressure from fingers, shows faint heel marks.	5-10	0.4-0.8	0.04-0.08	
S3	FIRM SOIL - very difficult to mould with fingers, indented with finger nail, difficult to cut with hand spade.	10-20	0.8-1.5	0.08-0.15	
S4	STIFF SOIL - cannot be moulded with fingers, cannot be cut with hand spade, requires hand picking for excavation .	20-80	1.5-6.0	0.15-0.60	
S5	VERY STIFF SOIL - very tough , difficult to move with hand pick, pneumatic spade required for excavation.	80-150	6-10	0.6-1.0	
R1	VERY WEAK ROCK - crumbles under sharp blows with geological pick point, can be cut with pocket knife.	150-3500	10-250	1-25.	Chalk, rocksalt
R2	MODERATELY WEAK ROCK - shallow cuts or scraping with pocket knife with difficulty, pick point indents deeply with firm blow.	3500-7500	250-500	25-50	Coal, schist, siltstone
R3	MODERATELY STRONG ROCK - knife cannot be used to scrape or peel surface, shallow indentations under firm blow from pick point.	7500-15000	500-1000	50-100	Sandstone, slate, shale
R4	STRONG ROCK - hand-held sample breaks with one firm blow from hammer end of geological pick.	15000-30000	1000-2000	100-200	Marble, granite, gneiss
R5	VERY STRONG ROCK - requires many blows from geological pick to break intact sample.	> 30000	> 2000	> 200	Quartzite, dolerite, gabbro, basalt

APPENDIX E

E.1 FALLING HEAD TEST THEORY

Hydraulic conductivities can be calculated from falling head test data by solving the governing differential equation. The method, first introduced by Hvorslev is presented in Freeze, 1979. This appendix presents the solution to the differential equation used to determine the hydraulic conductivity coefficient and the regression method used to obtain a representative semilogarithmic relationship between excess head and time. Figure A.1 defines the parameters used in the test.



The variables used in this development are:

h_w	- depth to water table	m
$h(t)$	- excess pressure head driving flow	m
h	- depth to equilibrium phreatic surface	m
L	- length of test section	m
r	- inner radius of rod	m
R	- radius of drill hole	m
$q(t)$	- volumetric rate of flow into rock	m^3/s
F	- shape factor	m
T	- time factor	s
K	- hydraulic conductivity	cm/s

The differential equation is obtained by equating rate of flow into the rock and the flow down the rods.

$$q(t) = -\pi \cdot r^2 \cdot dh/dt = F \cdot K \cdot h(t) \qquad 1.$$

where K = hydraulic conductivity
F = shape factor, depends on shape & dimension of test interval.

Define basic time lag T_o:

$$T_o = \frac{\pi \cdot r^2}{F \cdot K} \qquad 2.$$

Then substituting 2 into 1.

$$T_o \cdot dh/dt = h(t) \qquad 3.$$

$$dh/h = (-1/T_o)\ dt \qquad 4.$$

and integrating:

$$\int_{h_1}^{h_2} dh/h = \int_{t_1}^{t_2} (-1/T_o)\ dt \qquad 5.$$

$$\ln(h)\Big|_{h_1}^{h_2} = (-1/T_o)\ (t_2 - t_1) \qquad 6.$$

At $t_1 = 0$ $h_1 = h_o$; therefore:

$$\ln(h_2 - h_o) = (-1/T_o)\ (t_2 - 0\ ) \qquad 7.$$

$$\boxed{\ln(h_2/h_o) = (-1/T_o)\ t_2} \qquad 8.$$

Equation 8 illustrates the semilogarithmic relationship between excess pressure head and time.

BOUNDARY CONDITIONS:

Checking validity of equation 8 by evaluating boundary conditions.

at $t_2 = 0$ $\quad h_2 = h_o \quad \therefore \quad \ln(h_2/h_o) = (-1/T_o)\cdot t_2 = 0$

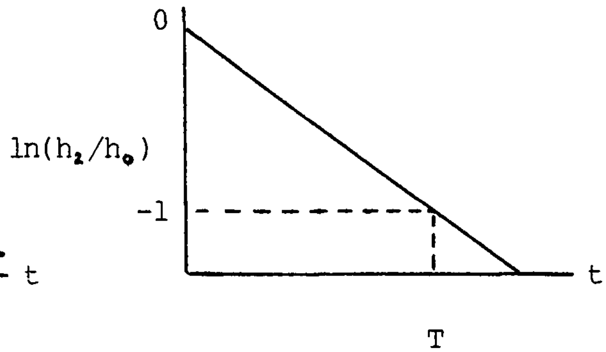
at $t_1 = \infty$ $\quad h_2 = 0 \quad \therefore \quad \ln(h_2/h_o) = (-1/T_o)\cdot t_2 = -\infty$

DETERMINING T :

$$\ln(h_2/h_o) = (-1/T_o)\cdot t$$

$\ln(0.368) = -1 \quad \therefore$ when $h_2/h_o = 0.368 \quad -t_2/T_o = -1 \quad .. \quad t_2 = T_o$

so T can be determined by plotting $\ln(h_2/h_o)$ vs. t_2 and determining the t_2 value at $h_2 = 0.368\ h_o$.



DETERMINING K:

Hvorslev established an empirical relationship that expresses the shape factor F. This relationship is valid if $L/R > 8$.

$$F = \frac{2\cdot L\cdot \pi}{\ln(L/R)}$$

Substituting in Equation 2:

$$K = \frac{r^2\cdot \ln(L/R)}{2\cdot L\cdot T}$$

REGRESSION:

A straight line relationship should exist when the falling head data is plotted on a semilogarithmic graph, (i.e. $\ln(h_t/h_o)$ vs. t_i) as it is governed by Equation 8. The slope of this line will have the value $-1/T_o$. By using linear regression to determine the best values for a & b in the straight line equation y = a + bx the best fit slope can be established. The goodness of fit or "linearity" is tested by evaluating the coefficient of determination R that is equal to one for perfectly linear data and approaches zero if the data is non linear.

The two regression equations are:

$$A\,n + B\;X = Y \qquad 11.$$

$$A\cdot\Sigma X_i + B\cdot\Sigma(X_i^2) = \Sigma X_i\cdot Y_i \qquad 12.$$

Solving 11 for A:

$$A = \frac{\Sigma Y_i - B\cdot\Sigma X_i}{n} \qquad 13.$$

Solving 12 for B:

$$B = \frac{\left[\Sigma X_i\cdot Y_i - \left\{\left(\frac{\Sigma Y_i - B\Sigma X}{n}\right)\cdot\Sigma X_i\right\}\right]}{\Sigma(X_i^2)} \qquad 14.$$

$$B\cdot n\cdot\Sigma(X_i^2) = n\cdot\Sigma(X_i\cdot Y_i) - \Sigma Y_i\cdot\Sigma X_i + B\cdot\Sigma X_i\,\Sigma X_i \qquad 15.$$

$$B = \frac{n\,\Sigma(X_i\cdot Y_i) - \Sigma Y_i\cdot\Sigma X_i}{n\cdot\Sigma(X_i^2) - \Sigma X_i\cdot\Sigma X_i} \qquad 16.$$

Equation 16 is first solved for B, then equation 13 is solved for A. Finally, the regression coefficient R is calculated by:

$$R^2 = \frac{A\cdot\Sigma Y_i + B\cdot\Sigma(X_i\cdot Y_i) - 1/n\,(\Sigma Y_i)^2}{(Y_i^2) - 1/n\cdot(\Sigma Y_i)^2}$$

APPENDIX F. PROGRAM EQFHEAD

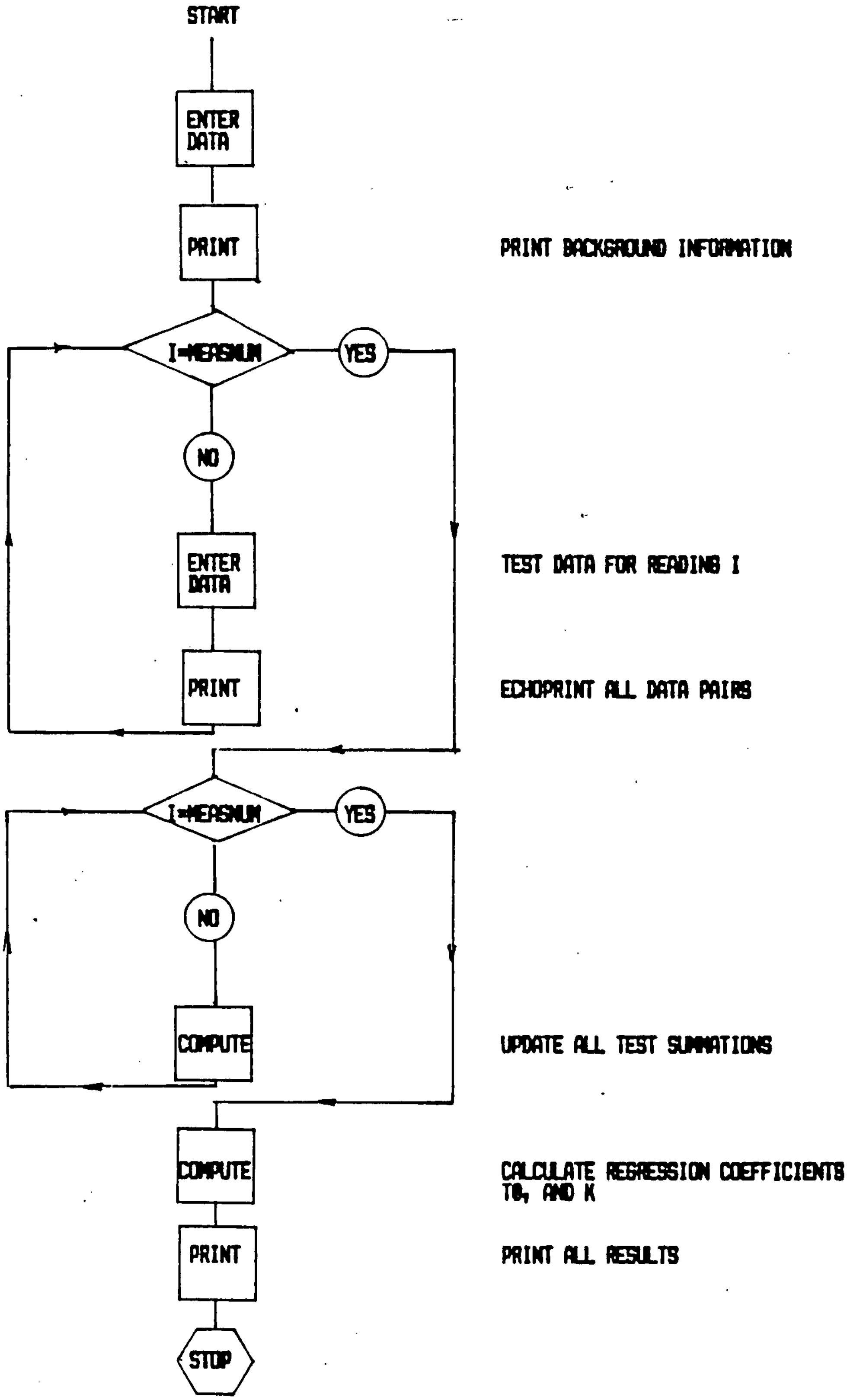
F.1 OBJECTIVE

Program EQFHEAD was developed to evaluate falling head test data and determine the coefficient of hydraulic conductivity. The most time consuming portion of the falling head test analysis is constructing a semilogarithmic plot of excess head vs. time. A linear regression subroutine was incorporated into the program to eliminate the need for construction of the plot.

F.2 THEORY

The program uses linear regression to find a best fitting straight line through the data points. The equations that are used in the regression are presented in Appendix E. The goodness of fit of the data to a straight line is quantitatively expressed in terms of the coefficient of determination. This parameter varies between 0 and 1, the latter being a perfect fit. Data that has a coefficient value between 0.9 and 1.0 can be considered sufficiently linear to be used for the analysis. If the coefficient falls below 0.9 the data should be plotted on a semilogarithmic graph and engineering judgement should be used in accepting the results and subsequent evaluation.

F.3 FLOWCHART



F.4 LIST OF VARIABLES

Variable Name	Function	Type
A	Y intercept in regression	r
B	slope in regression	r
B$(I)	variables used to contain printing headers	s
BDEPTH	depth to bottom of test section	r
C$	variable used to contain printing headers	s
DAT$	date	s
HRAT(I)	excess head ratio	r
HREF	excess head value at T0	r
K	hydraulic conductivity	r
L	length of test section	r
LOGRAT(I)	natural logarithm of HRAT(I)	r
LEVEL(I)	depth to water in rod at TIME(I)	r
MEASNUM	number of readings	r
MINTIME(I)	time in minutes . seconds	r
MIN	minutes portion of time reading	i
PIEZO$	piezometer number	s
RPIPE	inner radius of rods	r
RHOLE	radius of drill hole being tested	r
RR	codfficient of determination	r
SEC	seconds portion of time reading	i
SIGX	summation of TIME(I)	r
SIGY	summation of LOGRAT(I)	r
SIGXY	summation of TIME(I)*LOGRAT(I)	r
SIGXX	summation of TIME(I)^2	r
SIGYY	summation of LOGRAT(I)^2	r
TDEPTH	depth to top of test section	r
TIME(I)	time in seconds	i
T0	time factor	r
WT	depth to water table	r

F.5 PROCEDURE FOR USE

Program EQFHEAD is fully interactive, prompting for all required input, computing all parameters, and printing all results. The order of data entry corresponds exactly to the order that data is recorded on the field data sheet.

Procedure:

- printer on line
- run EQFHEAD
- enter test information
- enter time-depth data pairs in sequence
- output is printed
- check coefficient of determination for linearity

F.6 PROGRAM CODE

```
1000 '*****************************************************************
1001 '                          PROGRAM EQFHEAD
1002 '*****************************************************************
1003 'THIS PROGRAM CALCULATES THE HYDRAULIC CONDUCTIVITY FROM FALLING
1004 'HEAD TEST DATA BASED ON THE HVORSLEV PIEZOMETER TEST METHOD.
1005 '
1006 'DECLARE ARRAY SIZES
1007  DIM MINTIME(30),TIME(30),HRAT(30),LOGRAT(30),LEVEL(30)
1008 '
1009 'DECLARE TEST CONSTANTS (CHANGE IN PROGRAM IF REQUIRED)
1010  RPIPE=.00775
1011  RHOLE=.03493
1012  L=307!
1013 '
1014  CLS
1015 'ENTER TEST SPECIFIIC INFORMATION
1016  INPUT"DATE                              =";DAT$
1017  INPUT"PIEZOMETER NUMBER                 =";PIEZO$
1018  INPUT"NUMBER OF RODS ON                 =";TDEPTH
1019  INPUT"WATER TABLE @ (in m)              =";WT
1020  INPUT"PACKER PRESSURE (in psi)          =";PACKP
1021  INPUT"NUMBER OF MEASUREMENTS            =";MEASNUM
1022  TDEPTH=TDEPTH*3.07+1.219
1023  BDEPTH=TDEPTH+(L/100)
1024 'ENTER TIME AND LEVEL FOR EACH MEASUREMENT
1025  FOR I=1 TO MEASNUM
1026     INPUT"TIME (min) , LEVEL (meters)";MINTIME(I),LEVEL(I)
1027     MIN=INT(MINTIME(I))
1028     SEC=(MINTIME(I)-MIN)*100
1029     TIME(I)=MIN*60+SEC
1030 '   CALCULATE (HT-HE)/(H0-HE)
1031     HRAT(I)=(WT-LEVEL(I))/(WT-LEVEL(1))
1032     LOGRAT(I)=LOG(HRAT(I))
1033  NEXT I
1034  HREF=.632*WT
1035 '
1036 'PRINT ALL INFORMATION
1037  LPRINT "                          FALLING HEAD TEST CALCULATIONS"
1038  LPRINT "                          ------------------------------"
1039  B$(1)="       TEST DATE                        = \          \"
1040  B$(2)="       PIEZOMETER NUMBER                = \       \"
1041  B$(3)="       TEST INTERVAL:               FROM ####.## (m)"
1042  B$(7)="                                    TO   ####.## (m)"
1043  B$(4)="       PACKER PRESSURE                  = ####.## (psi)"
1044  B$(5)="       WATER TABLE                      = ####.## (m)"
1045  B$(6)="       NO. OF MEASUREMENTS              =    ##"
1046  LPRINT CHR$(10)
1047  LPRINT USING B$(1);DAT$
1048  LPRINT USING B$(2);PIEZO$
1049  LPRINT USING B$(3);TDEPTH
1050  LPRINT USING B$(7);BDEPTH
1051  LPRINT USING B$(4);PACKP
1052  LPRINT USING B$(5);WT
```

```
1053  LPRINT USING B$(6);MEASNUM
1054  LPRINT CHR$(27)CHR$(11)CHR$(48)CHR$(53)
1055  LPRINT"        TIME            LEVEL            RATIO            LOG-RATIO"
1056  LPRINT"        --------------------------------------------------------------"
1057  C$=  "        ###.##          ###.##           ##.####          ##.####"
1058  FOR I=1 TO MEASNUM
1059     LPRINT USING C$;MINTIME(I),LEVEL(I),HRAT(I),LOGRAT(I)
1060  NEXT I
1061  LPRINT
1062  LPRINT USING "                      REFERENCE DEPTH = ###.##";HREF
1063  LPRINT CHR$(27)CHR$(11)CHR$(48)CHR$(53)
1064 '
1065 'SUM UP ALL REQUIRED COEFFICIENTS
1066  SIGX=0
1067  SIGY=0
1068  SIGXX=0
1069  SIGYY=0
1070  SIGXY=0
1071  FOR I=1 TO MEASNUM
1072     SIGX=SIGX+TIME(I)
1073     SIGY=SIGY+LOGRAT(I)
1074     SIGXX=SIGXX+TIME(I)^2
1075     SIGYY=SIGYY+LOGRAT(I)^2
1076     SIGXY=SIGXY+TIME(I)*LOGRAT(I)
1077  NEXT I
1078 '
1079 'CALCULATE SLOPE B, Y INTERCEPT A, AND REGRESSION COEFFICIENT RR
1080  B=(MEASNUM*SIGXY-SIGX*SIGY)/(MEASNUM*SIGXX-SIGX^2)
1081  A=(SIGY-B*SIGX)/MEASNUM
1082  RR=(A*SIGY+B*SIGXY-(1/MEASNUM)*(SIGY^2))/(SIGYY-(1/MEASNUM)*SIGY^2)
1083 '
1084 'CALCULATE T0 AND HYDRAULIC CONDUCTIVITY K
1085  T0=(-1-A)/B
1086  K=RPIPE^2*LOG(L/RHOLE)/(2*L*T0)*100
1087 '
1088 'PRINT ALL CALCULATED VALUES
1089  LPRINT
1090  LPRINT USING"      SLOPE B                          =##.###^^^^";B
1091  LPRINT USING"      Y INTERCEPT A                    =##.###^^^^";A
1092  LPRINT USING"      RR COEFFICIENT                   =##.###";RR
1093  LPRINT USING"      T0                               =##.###^^^^";T0
1094  LPRINT CHR$(10)
1095  C$="       HYDRAULIC CONDUCTIVITY K        =##.###^^^^ (cm/s)"
1096  LPRINT USING C$;K
1097  LPRINT CHR$(12)
1098  STOP
1099  END
```

APPENDIX G TESTING EQUIPMENT

G.1 EQUIPMENT LIST

This section serves as a checklist of all equipment required for successful testing with the pneumatic packer apparatus.

TRIPOD ASSEMBLY
- tripod legs, 3
- tripod braces, 3
- nuts and bolts for binding tripod, 7, 3/8" diameter, 4" long
- polypropelene rope for lashing tripod braces 3, 2 m. long
- polypropelene rope for tying of base, 1, 15 m. long

PACKER ASSEMBLY
- bottom pneumatic packer
- top pneumatic packer
- brass reducers, 2
- steel pipe reducer, 1
- perforated rod between packers, 1
- spaghetti tubing coupler for connecting packers, 1, 3.15 m. long

NITROGEN SUPPLY
- nitrogen (N2) cylinder
- regulator
- T coupler and bleeder valve
- spaghetti tubing on spool
- additional spaghetti tubing for rising head tests (optional)

WATER SUPPLY
- fourty five gallon barrels, 2 or 3
- plastic syphon hose, 2 cm. diameter, 4 m. long
- funnel

TOOLS
- horseshoe plate
- steel hoisting cable with swivel attached, 5 m.
- polypropelene hoisting rope, 10 m.
- pipe wrenches, 2, 12"
- crescent wrench, 1, 8"
- airline wrench, 1, 7/16" or 11 mm
- hack saw
- pully with rope loop attached
- come along (optional to 100 ft. unless rods stuck)
- shackles, 3, 3/8"
- electricians tape
- bucket

MONITORING EQUIPMENT
- water level probe
- stop watch
- data sheet on clip board

G.2 EQUIPMENT SETUP

This section lists the sequential steps for setting up the falling head test apparatus. The proper assembly of the various systems is illustrated in Figures G.1 to G.3.

CONSTRUCTION OF TRIPOD:

- place two rusty legs side by side shackles up and bolt together at top.
- bolt black cross brace to legs so one side of cross brace projects out. The A frame should be completely bolted, not lashed.
- bolt third leg to lower hole in rusty tripod leg so shackle faces inside. Use upper hole on third leg.
- loop pulley over tripod.
- raise tripod.
- bolt one end of each remaining brace to legs. Lash other end.
- lash bottom of tripod with rope.

CONNECTING NITROGEN SUPPLY:

- remove safety cap from cylinder.
- clean out threads in bottle by blowing out and wiping any visible dirt.
- screw in regulator. Tighten snuggly.
- turn main valve on, listen for leaks. If leaking tighten regulator bolt more snuggly if possible.
- check pressure in bottle. Pressure should be at least 500 p.s.i.

WATER SUPPLY:

- place one barrel upside down near tripod.
- place survey stakes across barrel.
- place second barrel on top.
- fill by syphoning from drill service truck.

ASSEMBLY OF PACKERS:

- screw brass reducer into bottom packer and tighten.
- screw perforated rod into bottom reducer and tighten.
- screw brass reducer into top packer and tighten.
- attach spaghetti tubing coupler to bottom packer.
- tape tubing to perforated rod.
- place lower assembly into hole. Hold onto it carefully.
- attach upper packer to assembly.
- attach tubing coupler to upper packer.
- lower slightly and attach main airline to top of upper packer.
- tape airline twice to maintain it in the groove on the upper packer. Make sure that the airline can move through the groove as the packer shrinks several centimeters on inflation and would rip the airline if it were taped firmly.
- attach swivel to first drill rod and raise up tower.
- screw the rod into the packer assembly.
- tape airline to center of rod.
- lower packers down the hole, adding additional rods as required.

Figure G-1 Packer Assembly

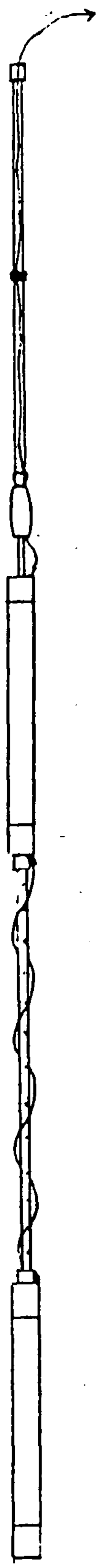


Figure G-2 PNEUMATIC PACKER SYSTEM

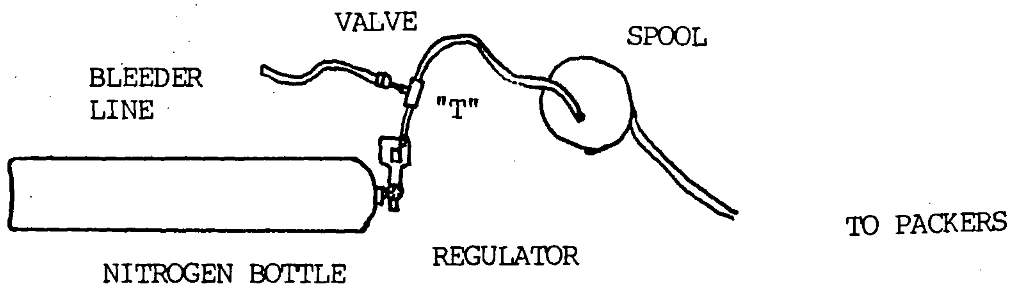
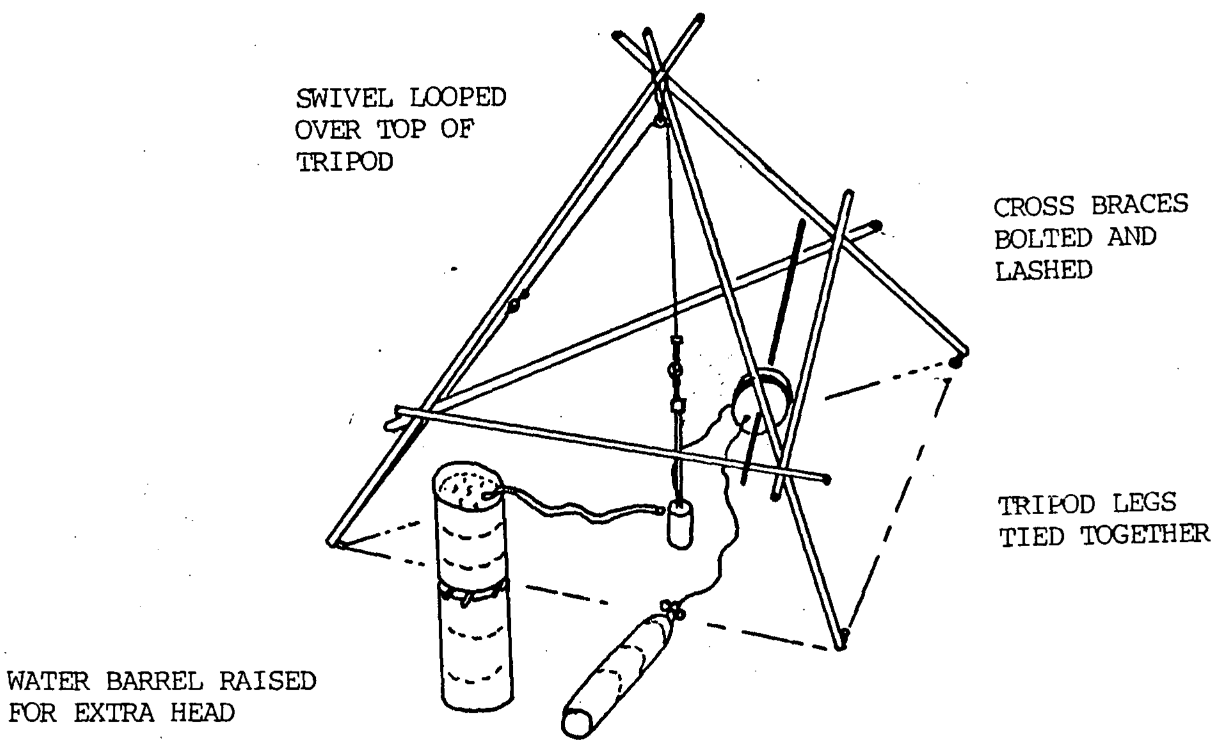


Figure G-3 TESTING TRIPOD SET-UP



G.3 TESTING PROCEDURE

This section lists the sequential steps for completing a falling head test. The steps are as follows;

- lower packer assembly to desired test interval, (must be below water table)
- check equilibrium water level in rods and outside casing.
- attach airline to regulator.
- start inflating packers, first at 50 psi, then gradually increasing pressure to 175 psi over 5 to 10 minutes.
- at this point rods should rise up slightly so no weight remains on horseshoe.
- check that packers are holding by lifting up on horseshoe.
- listen for air leaks in hole.
- check water levels inside and outside of rod. If levels differ then it is likely that a good seal has been established.
- start filling the rod with water. Syphon from the top barrel.
- if level cannot be brought up to surface after 1/3 of a barrel has been poured down the hole it is likely that the rock is sufficiently permeable to allow water to drain as fast as it is added. In that case the test can be started at the maximum level that can be obtained after pouring another 1/4 barrel down the hole.
- measure water level and record time.
- repeat measurements approximately every 50 cm. of drop in water level.
- continue monitoring until excess head has droped to 1/3 of initial value.
- upon completion of test turn nitrogen supply off, bleed pressure, and disconect airline from regulator so spool can turn freely.

EQUITY SILVER MINES LTD.

FALLING HEAD PERMEABILITY TEST DATA SHEET

DRILL HOLE NUMBER: P-5

DATE: 84/08/04

TEST INTERVAL - FROM: 10.43 (m)

TECHNICIAN: TS /EC

- TO: 13.50 (m)

NUMBER OF RODS ON: 3

WATER TABLE @: 8.42 (m)

PACKER PRESSURE: 150 (psi)

ROCK TYPE: GABBRO

STRUCTURAL DOMAIN: D2

TEST RECORD

READING #	TIME	WATER LEVEL
1	0.00	0.55
2	1.05	0.78
3	1.13	0.98
4	1.18	1.25
5	1.27	1.77
6	1.40	2.01
7	1.51	2.63
8	2.31	6.38
9	2.40	6.93

EQUITY SILVER MINES LTD.

FALLING HEAD PERMEABILITY TEST DATA SHEET

DRILL HOLE NUMBER: P-5 DATE: 84/08/04

TEST INTERVAL - FROM: 16.57 (m) TECHNICIAN: TS /EC

- TO: 19.64 (m)

NUMBER OF RODS ON: 5

WATER TABLE @: 8.55 (m)

PACKER PRESSURE: 175 (psi)

ROCK TYPE: GABBRO

STRUCTURAL DOMAIN: D2

TEST RECORD

READING #	TIME	WATER LEVEL
1	0.00	0.00
2	10.00	0.16
3	20.00	0.29
4	30.00	0.42
5	40.00	0.55

EQUITY SILVER MINES LTD.

FALLING HEAD PERMEABILITY TEST DATA SHEET

DRILL HOLE NUMBER:	P-5		DATE:	84/08/04
TEST INTERVAL - FROM:	19.64	(m)	TECHNICIAN:	TS /EC
- TO:	22.71	(m)		
NUMBER OF RODS ON:	6			
WATER TABLE @:	8.55	(m)		
PACKER PRESSURE:	175	(psi)		
ROCK TYPE:	GABBRO			
STRUCTURAL DOMAIN:	D2			

TEST RECORD

READING #	TIME	WATER LEVEL
1	0.00	0.00
2	10.00	0.37
3	20.00	0.66
4	30.00	0.91
5	40.00	1.10
6	50.00	1.30
7	60.00	1.46
8	70.00	1.60

EQUITY SILVER MINES LTD.

FALLING HEAD PERMEABILITY TEST DATA SHEET

DRILL HOLE NUMBER: P-6

DATE: 84/08/14

TEST INTERVAL - FROM: 16.57 (m)

TECHNICIAN: TS /EC

- TO: 19.64 (m)

NUMBER OF RODS ON: 5

WATER TABLE @: 10.46 (m)

PACKER PRESSURE: 175 (psi)

ROCK TYPE: LAPILLI

STRUCTURAL DOMAIN: D2

TEST RECORD

READING #	TIME	WATER LEVEL
1	0.42	9.55
2	1.08	10.13
3	1.26	10.26
4	1.42	10.31
5	2.05	10.35
6	2.39	10.39
7	3.00	10.40
8	3.37	10.42
9	4.04	10.42
10	4.56	10.43
11	6.00	10.46

EQUITY SILVER MINES LTD.

FALLING HEAD PERMEABILITY TEST DATA SHEET

DRILL HOLE NUMBER: P-6

DATE: 84/08/14

TEST INTERVAL - FROM: 19.64 (m)

TECHNICIAN: TS /EC

- TO: 22.71 (m)

NUMBER OF RODS ON: 6

WATER TABLE @: 10.46 (m)

PACKER PRESSURE: 175 (psi)

ROCK TYPE: LAPILLI

STRUCTURAL DOMAIN: D2

TEST RECORD

READING #	TIME	WATER LEVEL
1	0.00	8.00
2	0.14	8.13
3	0.26	8.58
4	0.38	8.74
5	0.54	8.91
6	1.11	9.04
7	1.29	9.20
8	1.54	9.38
9	2.27	9.53
10	2.53	9.63
11	3.30	9.86
12	4.25	10.02
13	5.30	10.15
14	6.45	10.24
15	8.34	10.34

EQUITY SILVER MINES LTD.

FALLING HEAD PERMEABILITY TEST DATA SHEET

DRILL HOLE NUMBER: P-6

DATE: 84/08/14

TEST INTERVAL - FROM: 22.71 (m)

TECHNICIAN: TS /EC

- TO: 25.78 (m)

NUMBER OF RODS ON: 7

WATER TABLE @: 10.49 (m)

PACKER PRESSURE: 175 (psi)

ROCK TYPE: LAPILLI

STRUCTURAL DOMAIN: D2

TEST RECORD

READING #	TIME	WATER LEVEL
1	0.45	6.38
2	1.06	7.73
3	1.27	8.52
4	1.55	9.27
5	2.15	9.61
6	2.45	9.93
7	3.07	10.09
8	3.35	10.21
9	5.00	10.43
10	5.31	10.48
11	7.02	10.47

EQUITY SILVER MINES LTD.

FALLING HEAD PERMEABILITY TEST DATA SHEET

DRILL HOLE NUMBER: P-7

DATE: 84/08/15

TEST INTERVAL - FROM: 16.57 (m)

TECHNICIAN: TS /EC

- TO: 19.64 (m)

NUMBER OF RODS ON: 5

WATER TABLE @: 15.42 (m)

PACKER PRESSURE: 150 (psi)

ROCK TYPE: LAPILLI

STRUCTURAL DOMAIN: D2

TEST RECORD

READING #	TIME	WATER LEVEL
1	0.29	0.42
2	1.05	0.74
3	2.03	1.10
4	3.15	1.64
5	6.36	2.97
6	9.07	3.84
7	10.36	4.28
8	15.40	5.68
9	20.30	6.85
10	28.04	8.31
11	37.05	9.68
12	39.47	9.99

EQUITY SILVER MINES LTD.

FALLING HEAD PERMEABILITY TEST DATA SHEET

DRILL HOLE NUMBER: P-7 DATE: 84/08/15

TEST INTERVAL - FROM: 16.57 (m) TECHNICIAN: TS /EC

- TO: 19.64 (m)

NUMBER OF RODS ON: 7

WATER TABLE @: 14.62 (m)

PACKER PRESSURE: 150 (psi)

ROCK TYPE: GABBRO

STRUCTURAL DOMAIN: D2

TEST RECORD

READING #	TIME	WATER LEVEL
1	0.04	5.20
2	0.15	7.18
3	0.23	8.39
4	0.35	9.43
5	0.47	10.17
6	1.09	11.90
7	1.33	12.48
8	1.45	12.79
9	1.55	13.01
10	2.38	13.29
11	3.02	13.39
12	3.23	13.67
13	3.36	13.73
14	4.03	13.84
15	4.26	13.92
16	4.55	13.99
17	5.27	14.06
18	6.42	14.20
19	7.52	14.29
20	9.06	14.34
21	9.59	14.37
22	12.12	14.46

EQUITY SILVER MINES LTD.

FALLING HEAD PERMEABILITY TEST DATA SHEET

DRILL HOLE NUMBER: P-8

DATE: 84/08/17

TEST INTERVAL - FROM: 19.64 (m)

TECHNICIAN: TS /EC

- TO: 22.71 (m)

NUMBER OF RODS ON: 6

WATER TABLE @: 17.14 (m)

PACKER PRESSURE: 175 (psi)

ROCK TYPE: GABBRO

STRUCTURAL DOMAIN: D2

TEST RECORD

READING #	TIME	WATER LEVEL
1	0.00	0.00
2	2.53	0.23
3	5.08	0.38
4	10.32	0.77
5	16.12	1.09
6	23.25	1.51
7	29.35	1.88
8	49.02	3.09

EQUITY SILVER MINES LTD.

FALLING HEAD PERMEABILITY TEST DATA SHEET

DRILL HOLE NUMBER:	P-8		DATE:	84/08/20
TEST INTERVAL - FROM:	22.71	(m)	TECHNICIAN:	TS /EC
- TO:	25.78	(m)		
NUMBER OF RODS ON:	7			
WATER TABLE @:	16.42	(m)		
PACKER PRESSURE:	175	(psi)		
ROCK TYPE:	GABBRO			
STRUCTURAL DOMAIN:	D2			

TEST RECORD

READING #	TIME	WATER LEVEL
1	0.33	0.44
2	1.05	0.70
3	1.27	0.87
4	1.57	1.13
5	2.39	1.44
6	3.03	1.63
7	4.38	2.32
8	6.45	3.13
9	9.27	4.29
10	13.22	6.30
11	23.09	8.23
12	28.00	9.46
13	34.21	10.91

EQUITY SILVER MINES LTD.

FALLING HEAD PERMEABILITY TEST DATA SHEET

DRILL HOLE NUMBER:	K-1		DATE:	84/08/01
TEST INTERVAL - FROM:	7.36	(m)	TECHNICIAN:	TS /EC
- TO:	10.43	(m)		
NUMBER OF RODS ON:	1			
WATER TABLE @:	1.20	(m)		
PACKER PRESSURE:	100	(psi)		
ROCK TYPE:	LAPILLI			
STRUCTURAL DOMAIN:	D2			

TEST RECORD

READING #	TIME	WATER LEVEL
1	0.00	0.00
2	1.16	1.19

EQUITY SILVER MINES LTD.

FALLING HEAD PERMEABILITY TEST DATA SHEET

DRILL HOLE NUMBER:	K-2		DATE:	84/08/30
TEST INTERVAL - FROM:	22.71	(m)	TECHNICIAN:	TS /EC
- TO:	25.78	(m)		
NUMBER OF RODS ON:	7			
WATER TABLE @:	17.80	(m)		
PACKER PRESSURE:	100	(psi)		
ROCK TYPE:	LAPILLI			
STRUCTURAL DOMAIN:	D2			

TEST RECORD

READING #	TIME	WATER LEVEL
1	0.16	1.50
2	0.27	2.15
3	0.43	3.20
4	1.02	4.25
5	1.31	5.80
6	1.51	6.30
7	2.20	7.20
8	2.40	8.00
9	3.23	9.00
10	4.26	10.10
11	6.35	12.00
12	8.54	13.65
13	12.06	15.20
14	16.20	16.60

EQUITY SILVER MINES LTD.

FALLING HEAD PERMEABILITY TEST DATA SHEET

DRILL HOLE NUMBER: K-2

DATE: 84/09/03

TEST INTERVAL - FROM: 25.78 (m)

TECHNICIAN: TS /EC

- TO: 28.85 (m)

NUMBER OF RODS ON: 8

WATER TABLE @: 18.10 (m)

PACKER PRESSURE: 125 (psi)

ROCK TYPE: LAPILLI

STRUCTURAL DOMAIN: D2

TEST RECORD

READING #	TIME	WATER LEVEL
1	0.00	0.00
2	0.29	2.45
3	0.52	3.30
4	1.23	4.20
5	2.00	5.10
6	2.54	6.30
7	4.35	8.00
8	7.25	10.20
9	10.56	12.30
10	12.28	13.05
11	13.32	13.45

EQUITY SILVER MINES LTD.

FALLING HEAD PERMEABILITY TEST DATA SHEET

DRILL HOLE NUMBER:	K-2		DATE:	84/09/03
TEST INTERVAL - FROM:	31.92	(m)	TECHNICIAN:	TS /EC
- TO:	34.99	(m)		
NUMBER OF RODS ON:	10			
WATER TABLE @:	18.10	(m)		
PACKER PRESSURE:	125	(psi)		
ROCK TYPE:	LAPILLI			
STRUCTURAL DOMAIN:	D2			

TEST RECORD

READING #	TIME	WATER LEVEL
1	0.00	0.00
2	0.44	0.60
3	1.24	1.00
4	2.45	1.90
5	5.30	3.35
6	7.32	4.30
7	8.24	5.00
8	12.20	6.48
9	15.43	7.70
10	17.30	8.15

FALLING HEAD TEST CALCULATIONS

TEST DATE	= 84/08/04	
PIEZOMETER NUMBER	= P5	
TEST INTERVAL:	FROM	16.57 (m)
	TO	19.64 (m)
PACKER PRESSURE	=	175.00 (psi)
WATER TABLE	=	8.42 (m)
NO. OF MEASUREMENTS	=	5

TIME	LEVEL	RATIO	LOG-RATIO
0.00	0.00	1.0000	0.0000
10.00	0.16	0.9810	-0.0192
20.00	0.29	0.9656	-0.0350
30.00	0.42	0.9501	-0.0512
40.00	0.55	0.9347	-0.0676

REFERENCE DEPTH = 5.32

SLOPE B	=-2.785E-05
Y INTERCEPT A	=-1.174E-03
RR COEFFICIENT	= 0.999
T0	= 3.587E+04

HYDRAULIC CONDUCTIVITY K = 1.221E-07 (cm/s)

FALLING HEAD TEST CALCULATIONS

TEST DATE	=	84/08/04
PIEZOMETER NUMBER	=	P5
TEST INTERVAL:	FROM	19.64 (m)
	TO	22.71 (m)
PACKER PRESSURE	=	175.00 (psi)
WATER TABLE	=	8.55 (m)
NO. OF MEASUREMENTS	=	8

TIME	LEVEL	RATIO	LOG-RATIO
0.00	0.00	1.0000	0.0000
10.00	0.37	0.9567	-0.0442
20.00	0.66	0.9228	-0.0803
30.00	0.91	0.8936	-0.1125
40.00	1.10	0.8713	-0.1377
50.00	1.30	0.8480	-0.1649
60.00	1.46	0.8292	-0.1872
70.00	1.60	0.8129	-0.2072

REFERENCE DEPTH = 5.40

SLOPE B	=-4.850E-05
Y INTERCEPT A	=-1.493E-02
RR COEFFICIENT	= 0.984
T0	= 2.031E+04

HYDRAULIC CONDUCTIVITY K = 2.156E-07 (cm/s)

FALLING HEAD TEST CALCULATIONS

TEST DATE	=	84/08/14
PIEZOMETER NUMBER	=	P6
TEST INTERVAL:	FROM	16.57 (m)
	TO	19.64 (m)
PACKER PRESSURE	=	175.00 (psi)
WATER TABLE	=	10.46 (m)
NO. OF MEASUREMENTS	=	11

TIME	LEVEL	RATIO	LOG-RATIO
0.42	9.55	1.0000	0.0000
1.08	10.13	0.3626	-1.0144
1.26	10.26	0.2198	-1.5151
1.42	10.31	0.1648	-1.8028
2.05	10.35	0.1209	-2.1130
2.39	10.39	0.0769	-2.5650
3.00	10.40	0.0659	-2.7191
3.37	10.42	0.0495	-3.0068
4.04	10.42	0.0440	-3.1246
4.56	10.43	0.0330	-3.4123
6.00	10.46	0.0055	-5.2040

REFERENCE DEPTH = 6.61

SLOPE B	=-1.315E-02
Y INTERCEPT A	=-1.602E-01
RR COEFFICIENT	= 0.923
T0	= 6.385E+01

HYDRAULIC CONDUCTIVITY K = 6.858E-05 (cm/s)

FALLING HEAD TEST CALCULATIONS

TEST DATE	= 84/08/14
PIEZOMETER NUMBER	= P6
TEST INTERVAL:	FROM 19.64 (m)
	TO 22.71 (m)
PACKER PRESSURE	= 175.00 (psi)
WATER TABLE	= 10.46 (m)
NO. OF MEASUREMENTS	= 15

TIME	LEVEL	RATIO	LOG-RATIO
0.00	8.00	1.0000	0.0000
0.14	8.13	0.9472	-0.0543
0.26	8.58	0.7642	-0.2689
0.38	8.74	0.6992	-0.3578
0.54	8.91	0.6301	-0.4619
1.11	9.04	0.5772	-0.5495
1.29	9.20	0.5122	-0.6690
1.54	9.38	0.4390	-0.8232
2.27	9.53	0.3780	-0.9727
2.53	9.63	0.3374	-1.0865
3.30	9.86	0.2439	-1.4110
4.25	10.02	0.1789	-1.7211
5.30	10.15	0.1260	-2.0713
6.45	10.24	0.0894	-2.4143
8.34	10.34	0.0488	-3.0204

REFERENCE DEPTH = 6.61

SLOPE B	=-5.814E-03
Y INTERCEPT A	=-1.092E-01
RR COEFFICIENT	= 0.995
T0	= 1.532E+02

HYDRAULIC CONDUCTIVITY K = 2.858E-05 (cm/s)

FALLING HEAD TEST CALCULATIONS

TEST DATE		= 84/08/14
PIEZOMETER NUMBER		= P6
TEST INTERVAL:	FROM	22.71 (m)
	TO	25.78 (m)
PACKER PRESSURE		= 175.00 (psi)
WATER TABLE		= 10.49 (m)
NO. OF MEASUREMENTS		= 11

TIME	LEVEL	RATIO	LOG-RATIO
0.45	6.38	1.0000	0.0000
1.06	7.73	0.6715	-0.3982
1.27	8.52	0.4793	-0.7354
1.55	9.27	0.2968	-1.2146
2.15	9.61	0.2141	-1.5413
2.45	9.93	0.1363	-1.9932
3.07	10.09	0.0973	-2.3297
3.35	10.21	0.0681	-2.6864
5.00	10.43	0.0146	-4.2268
5.31	10.48	0.0024	-6.0186
7.02	10.47	0.0049	-5.3255

REFERENCE DEPTH = 6.63

SLOPE B	=-1.620E-02
Y INTERCEPT A	= 6.392E-01
RR COEFFICIENT	= 0.938
T0	= 1.012E+02

HYDRAULIC CONDUCTIVITY K = 4.327E-05 (cm/s)

FALLING HEAD TEST CALCULATIONS

TEST DATE	=	84/08/15
PIEZOMETER NUMBER	=	P7
TEST INTERVAL:	FROM	16.57 (m)
	TO	19.64 (m)
PACKER PRESSURE	=	150.00 (psi)
WATER TABLE	=	15.42 (m)
NO. OF MEASUREMENTS	=	12

TIME	LEVEL	RATIO	LOG-RATIO
0.29	0.42	1.0000	0.0000
1.05	0.74	0.9787	-0.0216
2.03	1.10	0.9547	-0.0464
3.15	1.64	0.9187	-0.0848
6.36	2.97	0.8300	-0.1863
9.07	3.84	0.7720	-0.2588
10.36	4.28	0.7427	-0.2975
15.40	5.68	0.6493	-0.4318
20.30	1.85	0.9047	-0.1002
28.04	8.31	0.4740	-0.7465
37.05	9.68	0.3827	-0.9606
39.47	9.99	0.3620	-1.0161

REFERENCE DEPTH = 9.75

SLOPE B	=-4.111E-04
Y INTERCEPT A	= 1.237E-02
RR COEFFICIENT	= 0.882
T0	= 2.462E+03

HYDRAULIC CONDUCTIVITY K = 1.778E-06 (cm/s)

FALLING HEAD TEST CALCULATIONS

TEST DATE	= 84/08/17
PIEZOMETER NUMBER	= P7
TEST INTERVAL:	FROM 22.71 (m)
	TO 25.78 (m)
PACKER PRESSURE	= 150.00 (psi)
WATER TABLE	= 14.62 (m)
NO. OF MEASUREMENTS	= 22

TIME	LEVEL	RATIO	LOG-RATIO
0.04	5.20	1.0000	0.0000
0.15	7.18	0.7898	-0.2360
0.23	8.39	0.6614	-0.4135
0.35	9.43	0.5510	-0.5961
0.47	10.17	0.4724	-0.7499
1.09	11.90	0.2887	-1.2422
1.33	12.48	0.2272	-1.4820
1.45	12.79	0.1943	-1.6385
1.55	13.01	0.1709	-1.7666
2.38	13.29	0.1412	-1.9577
3.02	13.39	0.1306	-2.0358
3.23	13.67	0.1008	-2.2941
3.36	13.73	0.0945	-2.3594
4.03	13.84	0.0828	-2.4913
4.26	13.92	0.0743	-2.5995
4.55	13.99	0.0669	-2.7049
5.27	14.06	0.0594	-2.8227
6.42	14.20	0.0446	-3.1103
7.52	14.29	0.0350	-3.3515
9.06	14.34	0.0297	-3.5158
9.59	14.37	0.0265	-3.6291
12.12	14.46	0.0170	-4.0754

REFERENCE DEPTH = 9.24

SLOPE B	=-5.319E-03
Y INTERCEPT A	=-8.043E-01
RR COEFFICIENT	= 0.880
T0	= 3.679E+01

HYDRAULIC CONDUCTIVITY K = 1.190E-04 (cm/s)

FALLING HEAD TEST CALCULATIONS

TEST DATE	=	84/08/17
PIEZOMETER NUMBER	=	P8
TEST INTERVAL:	FROM	19.64 (m)
	TO	22.71 (m)
PACKER PRESSURE	=	150.00 (psi)
WATER TABLE	=	17.14 (m)
NO. OF MEASUREMENTS	=	8

TIME	LEVEL	RATIO	LOG-RATIO
0.00	0.00	1.0000	0.0000
2.53	0.23	0.9866	-0.0135
5.08	0.38	0.9778	-0.0224
10.32	0.77	0.9551	-0.0460
16.12	1.09	0.9364	-0.0657
23.25	1.51	0.9119	-0.0922
29.35	1.88	0.8903	-0.1162
49.02	3.09	0.8197	-0.1988

REFERENCE DEPTH = 10.83

SLOPE B	=-6.642E-05
Y INTERCEPT A	=-1.208E-03
RR COEFFICIENT	= 0.999
T0	= 1.504E+04

HYDRAULIC CONDUCTIVITY K = 2.912E-07 (cm/s)

FALLING HEAD TEST CALCULATIONS

TEST DATE	=	84/08/20
PIEZOMETER NUMBER	=	P8
TEST INTERVAL:	FROM	22.71 (m)
	TO	25.78 (m)
PACKER PRESSURE	=	175.00 (psi)
WATER TABLE	=	16.42 (m)
NO. OF MEASUREMENTS	=	13

TIME	LEVEL	RATIO	LOG-RATIO
0.33	0.44	1.0000	0.0000
1.05	0.70	0.9837	-0.0164
1.27	0.87	0.9731	-0.0273
1.57	1.13	0.9568	-0.0441
2.39	1.44	0.9374	-0.0646
3.03	1.63	0.9255	-0.0774
4.38	2.32	0.8824	-0.1252
6.45	3.13	0.8317	-0.1843
9.57	4.29	0.7591	-0.2757
13.22	6.30	0.6333	-0.4568
23.09	8.23	0.5125	-0.6684
28.00	9.46	0.4355	-0.8312
34.21	10.91	0.3448	-1.0648

REFERENCE DEPTH = 10.38

SLOPE B	=-5.153E-04
Y INTERCEPT A	= 1.631E-02
RR COEFFICIENT	= 0.996
T0	= 1.972E+03

HYDRAULIC CONDUCTIVITY K = 2.220E-06 (cm/s)

FALLING HEAD TEST CALCULATIONS

TEST DATE	= 84/08/01	
PIEZOMETER NUMBER	= K1	
TEST INTERVAL:	FROM 7.36 (m)	
	TO 10.43 (m)	
PACKER PRESSURE	= 100.00 (psi)	
WATER TABLE	= 1.20 (m)	
NO. OF MEASUREMENTS	= 2	

TIME	LEVEL	RATIO	LOG-RATIO
0.00	0.00	1.0000	0.0000
0.16	1.19	0.0083	-4.7875

REFERENCE DEPTH = 0.76

SLOPE B	=-2.992E-01
Y INTERCEPT A	= 0.000E+00
RR COEFFICIENT	= 1.000
T0	= 3.342E+00

HYDRAULIC CONDUCTIVITY K = 1.310E-03 (cm/s)

FALLING HEAD TEST CALCULATIONS

TEST DATE	=	84/08/30
PIEZOMETER NUMBER	=	K2
TEST INTERVAL:	FROM	22.71 (m)
	TO	25.78 (m)
PACKER PRESSURE	=	100.00 (psi)
WATER TABLE	=	17.80 (m)
NO. OF MEASUREMENTS	=	14

TIME	LEVEL	RATIO	LOG-RATIO
0.16	1.50	1.0000	0.0000
0.27	2.15	0.9601	-0.0407
0.43	3.20	0.8957	-0.1101
1.02	4.25	0.8313	-0.1848
1.31	5.80	0.7362	-0.3063
1.51	6.30	0.7055	-0.3488
2.20	7.20	0.6503	-0.4303
2.40	8.00	0.6012	-0.5088
3.23	9.00	0.5399	-0.6164
4.26	10.10	0.4724	-0.7499
6.35	12.00	0.3558	-1.0333
8.54	13.65	0.2546	-1.3681
12.06	15.20	0.1595	-1.8357
16.20	16.60	0.0736	-2.6088

REFERENCE DEPTH = 11.25

SLOPE B	=-2.584E-03
Y INTERCEPT A	=-3.167E-02
RR COEFFICIENT	= 0.996
T0	= 3.748E+02

HYDRAULIC CONDUCTIVITY K = 1.168E-05 (cm/s)

FALLING HEAD TEST CALCULATIONS

TEST DATE		= 84/09/03
PIEZOMETER NUMBER		= K2
TEST INTERVAL:	FROM	25.78 (m)
	TO	28.85 (m)
PACKER PRESSURE		= 125.00 (psi)
WATER TABLE		= 18.10 (m)
NO. OF MEASUREMENTS		= 11

TIME	LEVEL	RATIO	LOG-RATIO
0.00	0.00	1.0000	0.0000
0.29	2.45	0.8646	-0.1454
0.52	3.30	0.8177	-0.2013
1.23	4.20	0.7680	-0.2640
2.00	5.10	0.7182	-0.3310
2.54	6.30	0.6519	-0.4278
4.35	8.00	0.5580	-0.5834
7.25	10.20	0.4365	-0.8290
10.56	12.30	0.3204	-1.1381
12.28	13.05	0.2790	-1.2765
13.32	13.45	0.2569	-1.3590

REFERENCE DEPTH = 11.44

SLOPE B	=-1.571E-03
Y INTERCEPT A	=-1.112E-01
RR COEFFICIENT	= 0.992
T0	= 5.657E+02

HYDRAULIC CONDUCTIVITY K = 7.740E-06 (cm/s)

FALLING HEAD TEST CALCULATIONS

TEST DATE	= 84/09/03	
PIEZOMETER NUMBER	= K2	
TEST INTERVAL:	FROM	31.92 (m)
	TO	34.99 (m)
PACKER PRESSURE	=	125.00 (psi)
WATER TABLE	=	18.10 (m)
NO. OF MEASUREMENTS	=	10

TIME	LEVEL	RATIO	LOG-RATIO
0.00	0.00	1.0000	0.0000
0.44	0.60	0.9669	-0.0337
1.24	1.00	0.9448	-0.0568
2.45	1.90	0.8950	-0.1109
5.30	3.35	0.8149	-0.2047
7.32	4.30	0.7624	-0.2712
8.24	5.00	0.7238	-0.3233
12.20	6.48	0.6420	-0.4432
15.43	7.70	0.5746	-0.5541
17.30	8.15	0.5497	-0.5983

REFERENCE DEPTH = 11.44

SLOPE B	=-5.741E-04
Y INTERCEPT A	=-1.207E-02
RR COEFFICIENT	= 0.998
T0	= 1.721E+03

HYDRAULIC CONDUCTIVITY K = 2.545E-06 (cm/s)

H.3 SUMMARY OF PREVIOUS PERMEABILITY TESTING

A. TESTING BY GOLDER ASSOCIATES, 1982

PIEZOMETER NUMBER	TYPE OF TEST	BASIC TIME LAG (sec)	LENGTH OF GRAVEL PACK (m)	DIAMETER OF GRAVEL PACK (m)	HYDRAULIC CONDUCTIVITY (m/sec)	LITHOLOGY AND FORMATION
RH-82-01-2	RHT	16	1.4	0.152	8.1×10^{-6}	Andesite (Goosly Lake/Buck Creek)
-3	RHT	570	2.7	0.152	1.0×10^{-7}	Andesite (Goosly Lake?Buck Creek)
RH-82-02-2	RHT	351,000	2.7	0.152	2.2×10^{-10}	Till
RH-82-03-02	RHT	580	2.4	0.152	1.0×10^{-7}	Silty Gravel
RH-82-05-01	FHT	72,000	3.4	0.152	9.2×10^{-10}	Undiferentiated Volcanics
RH-82-06-01	RHT	77	6.8	0.152	5.0×10^{-7}	Andesite (Goosly Lake/Buck Creek)
RH-82-08-01	RHT	1,440	16.0	0.152	1.3×10^{-8}	Diorite (?)
-02	FHT	830	8.8	0.152	3.7×10^{-8}	Ash Tuff

B. TESTING BY EQUITY SILVER MINES (Southern Tail Pit)

HOLE NUMBER	TEST NUMBER	BASIC TIME LAG (sec)	LENGTH OF OPEN HOLE (m)	HOLE DIAMETER (m)	HYDRAULIC CONDUCTIVITY (m/s)
1B	1	94	3.05	0.07	7.0×10^{-7}
1B	2	1,040	1.52	0.07	1.0×10^{-7}
2A	1	158	3.20	0.07	4.0×10^{-7}
2A	2	630	6.10	0.07	5.5×10^{-8}
2B	2	800	1.20	0.07	2.0×10^{-7}
3A	1	635	3.05	0.07	9.8×10^{-8}
3B	1	112	2.13	0.07	8.0×10^{-7}
4B	1	395	1.98	0.07	2.0×10^{-7}
5B	1	92	1.31	0.07	1.3×10^{-6}
6B	1	68	1.82	0.07	1.4×10^{-6}

EQUITY SILVER MINES LTD.

FALLING HEAD PERMEABILITY TEST DATA SHEET

DRILL HOLE NUMBER: 2-A SOUTHERN TAIL   DATE: 81/03/19

TEST INTERVAL - FROM: 7.36 (m)   TECHNICIAN: PB

- TO: 10.43 (m)

NUMBER OF RODS ON: 2

WATER TABLE @: 2.41 (m)

PACKER PRESSURE: 150 (psi)

ROCK TYPE: ?

STRUCTURAL DOMAIN: ?

TEST RECORD

READING #	TIME	WATER LEVEL
1	0.30	0.46
2	1.00	0.75
3	1.32	1.03
4	2.08	1.28
5	2.36	1.46
6	3.01	1.61
7	3.25	1.73
8	4.00	1.85
9	4.28	1.94
10	5.00	2.01
11	6.15	2.13
12	6.56	2.20
13	7.44	2.24
14	8.21	2.26
15	9.35	2.27
16	10.05	2.30
17	11.06	2.32
18	12.28	2.33
19	16.15	2.31
20	20.30	2.33
21	26.00	2.31
22	41.45	2.35
23	52.40	2.39
24	57.50	2.39
25	62.00	2.40

FALLING HEAD TEST CALCULATIONS

TEST DATE		= 81/03/19
PIEZOMETER NUMBER		= 2A S.T.
TEST INTERVAL:	FROM	7.36 (m)
	TO	10.43 (m)
PACKER PRESSURE	=	150.00 (psi)
WATER TABLE	=	2.41 (m)
NO. OF MEASUREMENTS	=	25

TIME	LEVEL	RATIO	LOG-RATIO
0.30	0.46	1.0000	0.0000
1.00	0.75	0.8513	-0.1610
1.32	1.03	0.7077	-0.3457
2.08	1.28	0.5795	-0.5456
2.36	1.46	0.4872	-0.7191
3.01	1.61	0.4103	-0.8910
3.25	1.73	0.3487	-1.0535
4.00	1.85	0.2872	-1.2476
4.28	1.94	0.2410	-1.4229
5.00	2.01	0.2051	-1.5841
6.15	2.13	0.1436	-1.9408
6.56	2.20	0.1077	-2.2285
7.44	2.24	0.0872	-2.4398
8.21	2.26	0.0769	-2.5649
9.35	2.27	0.0718	-2.6339
10.05	2.30	0.0564	-2.8751
11.06	2.30	0.0564	-2.8751
12.28	2.30	0.0564	-2.8751
16.15	2.32	0.0462	-3.0758
20.30	2.33	0.0410	-3.1936
26.00	2.31	0.0513	-2.9704
41.45	2.35	0.0308	-3.4812
52.40	2.39	0.0103	-4.5799
57.50	2.39	0.0103	-4.5799
62.00	2.40	0.0051	-5.2730

REFERENCE DEPTH = 1.52

SLOPE B	=-1.127E-03
Y INTERCEPT A	=-1.202E+00
RR COEFFICIENT	= 0.764
T0	=-1.791E+02

HYDRAULIC CONDUCTIVITY K =-2.445E-05 (cm/s)

EQUITY SILVER MINES LTD.
FALLING HEAD PERMEABILITY TEST DATA SHEET

DRILL HOLE NUMBER: ________ DATE: _________

TEST INTERVAL - FROM: ________ (m) TECHNICIAN: _________

- TO: ________ (m)

NUMBER OF RODS ON: ________

WATER TABLE @: ________ (m)

PACKER PRESSURE: ________ (psi)

ROCK TYPE: ________

STRUCTURAL DOMAIN: ________

TEST RECORD

READING #	TIME	WATER LEVEL
1	__________	___________
2	__________	___________
3	__________	___________
4	__________	___________
5	__________	___________
6	__________	___________
7	__________	___________
8	__________	___________
9	__________	___________
10	__________	___________
11	__________	___________
12	__________	___________
13	__________	___________
14	__________	___________
15	__________	___________
16	__________	___________
17	__________	___________
18	__________	___________
19	__________	___________
20	__________	___________

APPENDIX I. PROGRAM EQDRAWDOWN

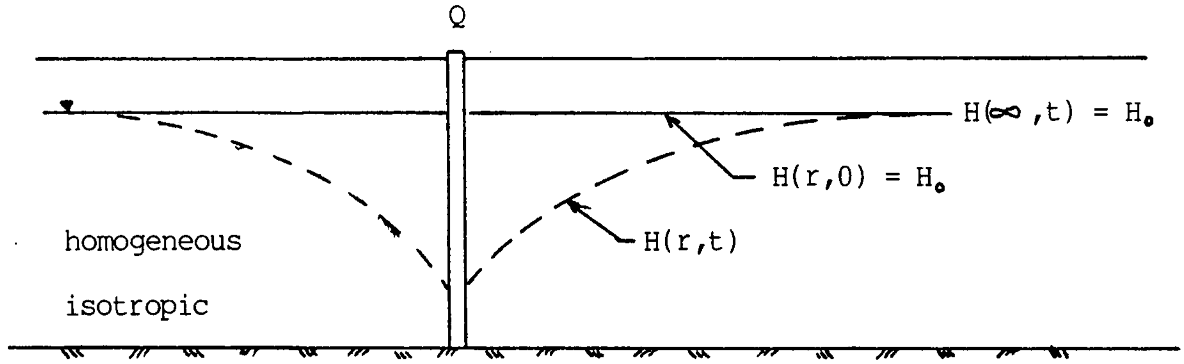
I.1 OBJECTIVE

Program EQDRAWDN was developed to model the response of the Main Zone water table to well point dewatering. The program uses the Theis solution to the boundary value problem of a single well dewatering an unconfined horizontal aquifer of constant thickness and homogeneous and isotropic hydraulic parameters. The program can model the transient response of the water table at any position and time.

The purpose of the numerical modelling is to determine whether: 1. a sufficiently large drawdown cone will be developed around each pumping well, 2. the rate of water table drawdown will be fast enough to dewater in advance of mining activity, 3. to study the sensitivity of the system to changes in aquifer properties and pumping rates, and 4. to assist in the design of the dewatering system after all important hydraulic parameters (as identified from sensitivity study in step 3.) have been adequately established from in-situ tests.

I-2 THEORY OF SIMULATING PUMPING WELL DRAWDOWN

The boundary value problem of a single well dewatering an unconfined aquifer is illustrated below:



The governing differential equation for this boundary value probem is the flow equation:

$$\frac{d^2h}{dx^2} + \frac{d^2h}{dy^2} = \frac{S}{T} \cdot \frac{dh}{dt} \qquad 1.$$

where: h = total head

S = specific yield - The volume of water that an unconfined aquifer releases from storage per unit surface area of aquifer per unit decline in water table.

T = transmissivity - Product of hydraulic conductivity and saturated thickness of the aquifer.

Equation 1 can be simplified by transforming to radial coordinates:

$$\frac{d^2h}{dr} + \frac{1}{r} \cdot \frac{dh}{dr} = \frac{S}{T} \cdot \frac{dh}{dt}$$

The initial condition to this B.V.P. is:

$h(r,0) = h_0$ initial water table at h .

The boundary conditions are:

$h(\infty,t) = h_0$ water table unaffected by pumping at large distances.

$\lim_{r \to 0} \frac{r\,dh}{dr} = \frac{Q}{2 \cdot \pi \cdot T}$ constant pumping rate at the well point. (from Darcy's law)

The solution to this B.V.P. was first worked through by Theis (1935).

$$h \;\; - h(r,t) = \frac{Q}{4\cdot\pi\cdot T} \int_{u}^{\infty} \frac{e^{-U}\, dU}{U}$$

where:

$$U = \frac{r^{2}\cdot S}{4\cdot T\cdot t}$$

The integral portion of this function is often called the well function in hydrology. An analytical solution does not exist to this integral, but an infinite series solution is available and converges rapidly.

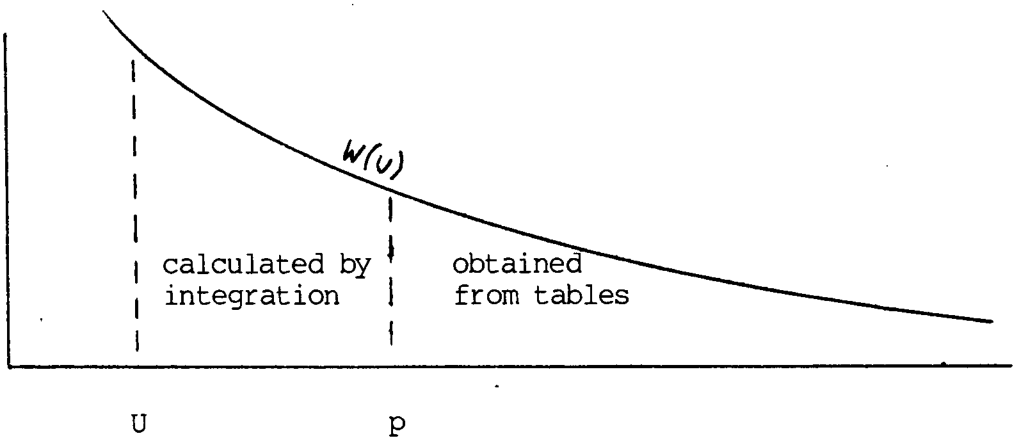
Evaluation of Well Function:

The well function can be evaluated at a point x by the series:

$$\int \frac{e^{ax}\;\; dx}{x} = \ln(x) + \frac{a\cdot x}{1!} + \frac{a^{2}\cdot x^{2}}{2\cdot 2!} + \frac{a^{3}\cdot x^{3}}{3\cdot 3!} + \frac{a^{4}\cdot x^{4}}{4\cdot 4!} \;.............$$

(from CRC Handbook, A-83)

The Theis solution requires the definite integral from U to . This can be calculated by evaluating the intergral between U and an intermediate point P and adding this value to the integral between p and . The latter value can be obtained from well function tables (Freeze, 1981).



$$W(U) = \int_{U}^{\infty} \frac{e^{-x}\cdot dx}{x} = \int_{U}^{p} \frac{e^{-x}\cdot dx}{x} + \int_{p}^{\infty} \frac{e^{-x}\cdot dx}{x}$$

$$W(U) = \ln(p) + \sum_{n=1}^{Z} \frac{(-1)^n \cdot p^n}{n\, n!} - \ln(U) - \sum_{n=1}^{Z} \frac{(-1)^n \cdot (U)^n}{n\, n!} \qquad \int_p^{\infty} \frac{e^{-x} \cdot dx}{x}$$

The above expression is used to evaluate the well function in program eqdrawdn, presented in appendix H.

When U becomes greater than 0.5 the series fails to converge rapidly. When U exceeds 1.0 the series becomes divergent and the series solution cannot be used. For values of U greater than 0.5 the well function can be calculated by evaluating the integral by Simpson´s rule. This approach is computationally less efficient so the series approach is used whenever possible.

By Simpson´s rule:

$$\int_A^B f(x)\, dx = \frac{\Delta}{3}\Big\{ f_0 + 4\,(f_1 + f_3 + f_5 \cdots f_{n-1}) + 2\,(f_2 + f_4 + f_6 \cdots f_{n-2}) + f_n \Big\}$$

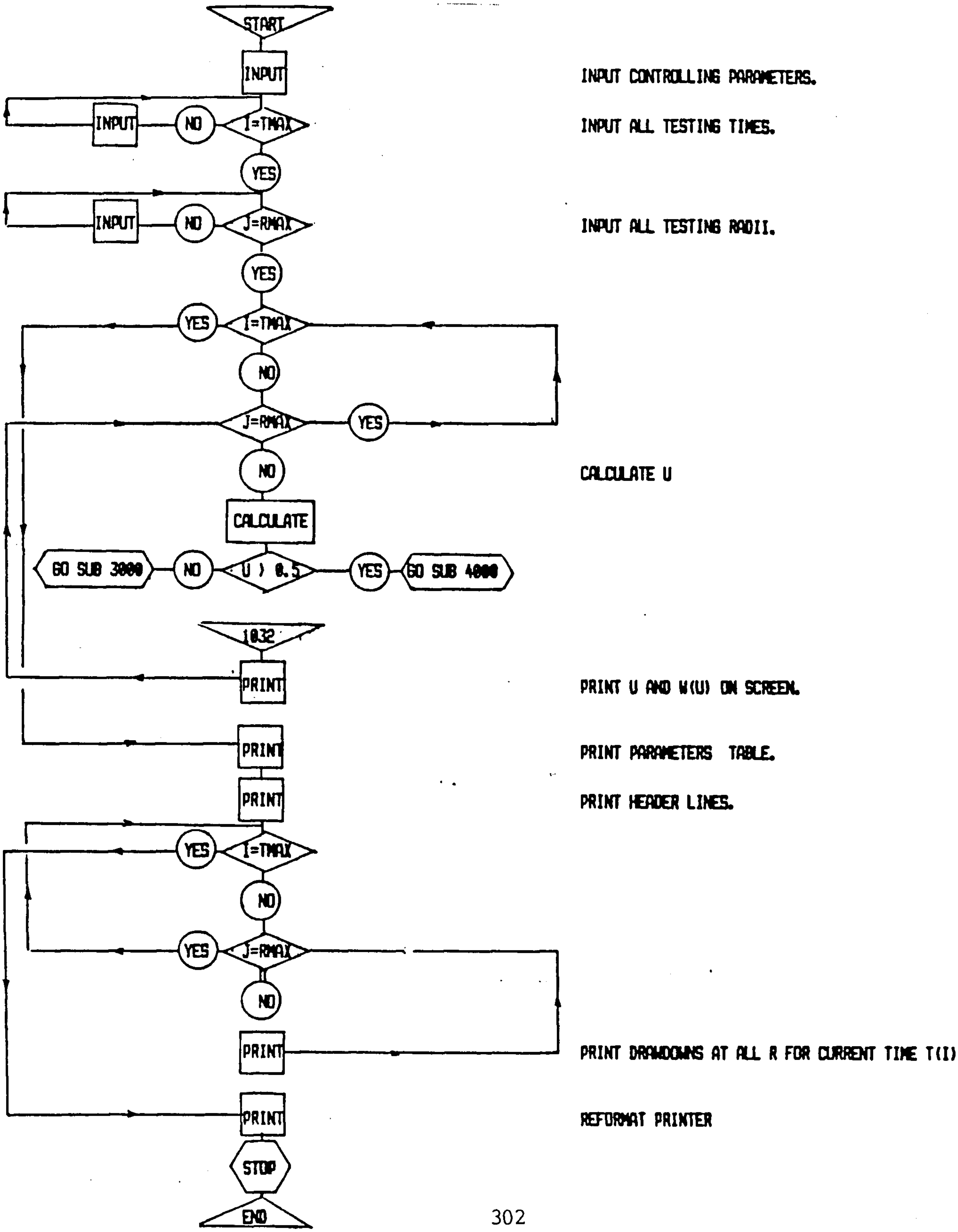
$$= \Delta/3 \cdot \{\text{ends} + 4 \text{ odds} + 2 \text{ evens}\}$$

Therefore:

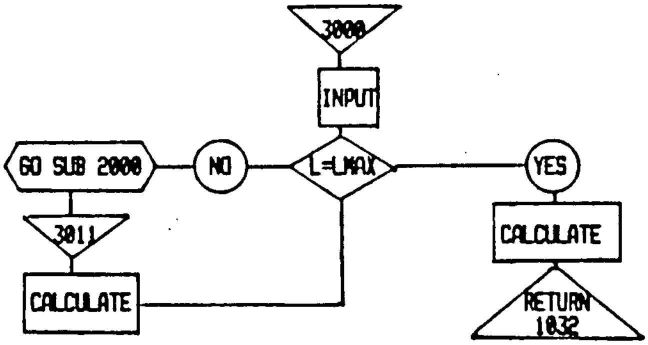
$$W(U) = \int_U^{0.5} \frac{e^{-x} \cdot dx}{x} + \int_{0.5}^{\infty} \frac{e^{-x} \cdot dx}{x}$$

by Simpson´s Rule From Tables

I-3 EQDRAWDN FLOWCHART



SUBROUTINE WSERIES

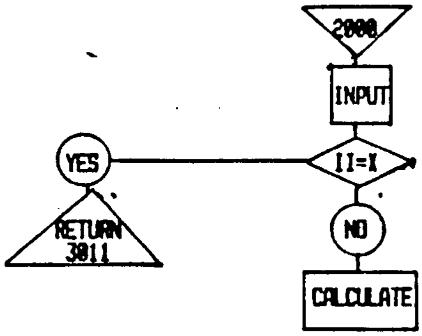


RECEIVE INPUT FROM CALLING PROGRAM
REFRESH SUMMATION VARIABLES.

CALCULATE SERIES TERM
FOR CURRENT L

CALCULATE W(U)

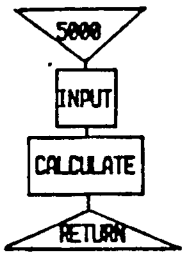
SUBROUTINE FACTORIAL



RECEIVE INPUT FROM CALLING SUBROUTINE.

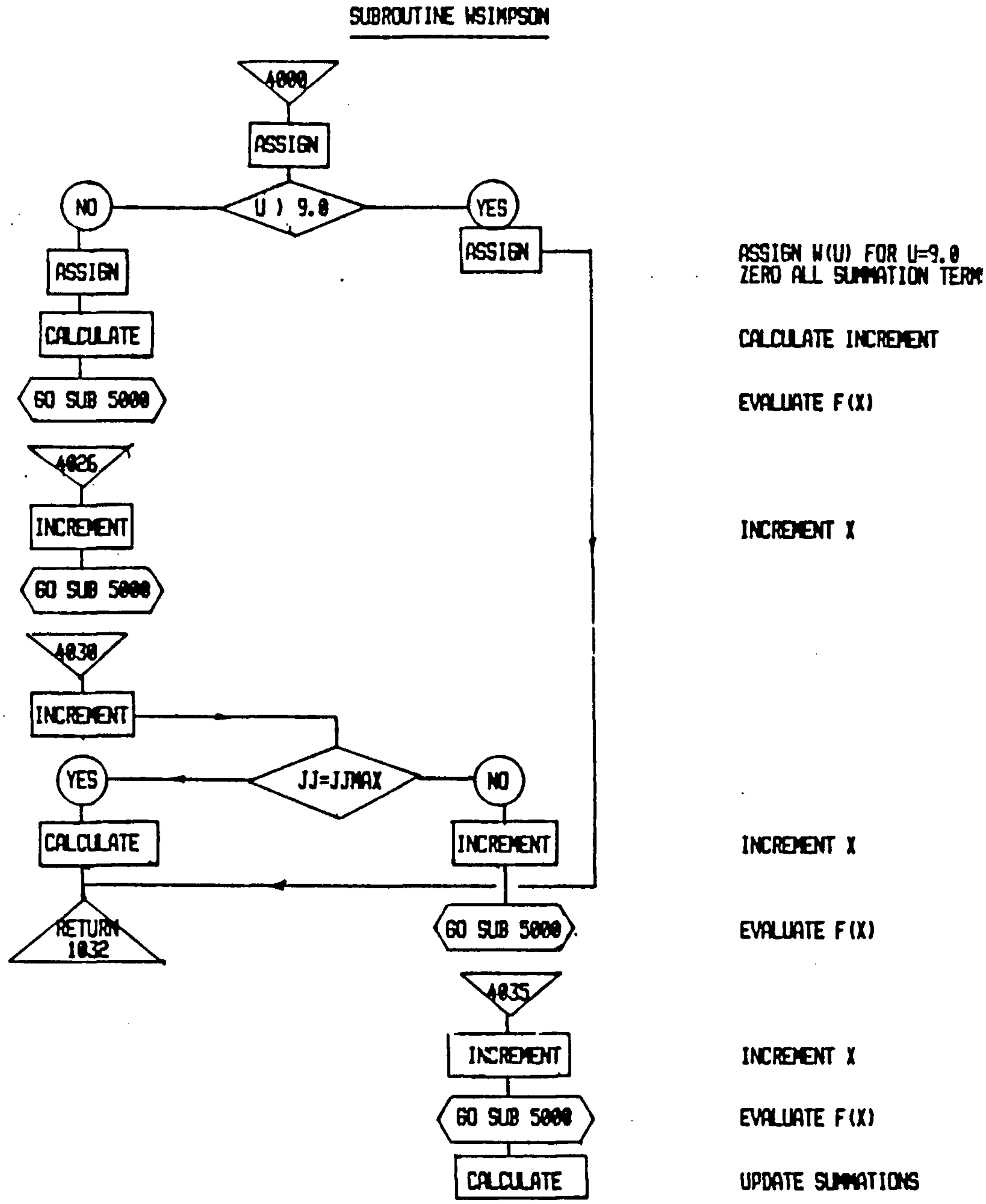
MULTIPLY PRODUCT BY CURRENT II

SUBROUTINE FUNCTION



RECEIVE INPUT FROM CALLING SUBROUTINE

EVALUATE F(X) FOR CURRENT X


SUBROUTINE WSIMPSON
4000
ASSIGN
U) 9.0
NO
YES
ASSIGN
ASSIGN
CALCULATE
GO SUB 5000
4026
INCREMENT
GO SUB 5000
4030
INCREMENT
JJ=JJMAX
YES
NO
CALCULATE
INCREMENT
RETURN 1032
GO SUB 5000
4035
INCREMENT
GO SUB 5000
CALCULATE
ASSIGN W(U) FOR U=9.0
ZERO ALL SUMMATION TERM
CALCULATE INCREMENT
EVALUATE F(X)
INCREMENT X
INCREMENT X
EVALUATE F(X)
INCREMENT X
EVALUATE F(X)
UPDATE SUMMATIONS

I.4 LIST OF VARIABLES

Variable Name	Function	Type
A$	used as printer header	s
B$	used as printer header	s
BLANK	used to insert blanks for left justification of numbers.	i
C$	used as printer header	s
DELH	drop in water level from initial condition	r
F$	string variable acts as line buffer for printing	s
FACT	factorial of X	i
G$	holds decimal character	s
H(I.J)	water level at T(i),R(j)	r
H0	initial elevation of water level	r
I	counter	i
II	counter	i
J	counter	i
K	hydraulic conductivity	r
L	counter	i
LENGTH	length of buffer, used for inserting blanks	i
LIN$	line buffer variable	s
LMAX	number of increments in series	i
NUM$	string variable used in time conversion	s
PI	constant	r
POSI	counter used to insert blanks in line buffer	i
PLOW	low bound for numerical integration	r
PHIGH	upper bound for numerical integration	r
PINF	value of integral between phigh and infinity	r
Q	pumping rate	r
R(J)	distance from well	r
RMAX	number of positions R(j) to be evaluated	i
SY	specific yield	r
SUM#	sum of series in numerical integration	d
T(I)	time	r
TMAX	number of times at which drawdowns are calculated	i
THICK	thickness of unconfined aquifer	r
TIMES$	used to print time variable	s
TERM#	one term of series	d
U	input value for well function	r
UU#	double precision value of U	d
W	W(U)	r
X	argument for factorial subroutine	i

I.5 PROCEDURE FOR USE

Program EQFHEAD was developed to study the sensitivity of the aquifer system to changes in hydraulic parameters and pumping rates; therefore, all parameters are entered into the program at the beginning of the first run. Then, in subsequent runs the user is asked to identify which parameters he wishes to change. All others remain as in the previous run. The program is fully interactive, and results are released directly to the line printer.

Procedure:

- Printer on line.
- Load EQDRAWDN
- Input appropriate initial values for the following parameters in the data lines of the program.

  Line 1120: TMAX,RMAX,K(cm/s),THICK(m),SY,Q(l/min),H0
  Line 1130: T(I) (days) I=1 to TMAX
  Line 1140: R(J) (m)   J=1 to RMAX

- Run
- Indicate which parameters to be changed when prompted, code 0 on first run. Note that more than one parameter can be changed during a run.
  0 = no changes
  1 = change K
  2 = change aquifer thickness
  3 = change specific head
  4 = change pumping rate
  5 = change times of simulation
  6 = change radii where drawdown computed
- Input desired parameter as prompted
- Results printed on line printer

I.6 EQDRAWDN PROGRAM CODE

```
1000 '**********************************************************************
1005 '                         PROGRAM EQDRAWDN
1010 '**********************************************************************
1015 'THIS PROGRAM CALCULATES THE DRAWDOWN IN A HORIZONTAL ISOTROPIC AQUIFER
1020 'ACCORDING TO THE THEIS SOLUTION.  DRAWDOWN IS CALCULATED AS A FUNCTION
1025 'OF RADIAL POSITION R AND TIME T.
1050 '
1055 'DIMENSION ALL ARRAYS AND ASSIGN ALL CONSTANTS
1060  DIM H(20,10),R(10),T(20)
1065  PI=3.141592
1080 '
1081 'DATA DECK
1082  DATA 8,6,1E-07,100,0.05,10,100
1083  DATA 1,5,10,50,100,200,500,1000
1084  DATA 5,10,20,50,100,200
1085 'INPUT CONTROLLING PARAMETERS
1090  READ TMAX,RMAX,K,THICK,SY,Q,H0
1095  K=K/100
1096  Q=Q/60/1000
1100  FOR I=1 TO TMAX
1105     READ T(I)
1106     T(I)=T(I)*24*60*60
1110  NEXT I
1115  FOR J=1 TO RMAX
1120     READ R(J)
1125  NEXT J
1130 '
1135 'CALCULATE DRAWDOWNS AT ALL POSITIONS R FOR A GIVEN TIME T
1140  FOR I=1 TO TMAX
1145     FOR J=1 TO RMAX
1150        U=R(J)^2*SY/(4*K*THICK*T(I))
1151        IF U <.5 THEN GOSUB 1994 ELSE GOSUB 3031
1152        PRINT USING "U=##.##^^^^ W(U)=###.####";U,W
1190        DELH=Q/(4*PI*K*THICK)*W
1195        H(I,J)=H0-DELH
1200     NEXT J
1210  NEXT I
1215 '
1220 'PRINT OUTPUT ON LINE PRINTER
1225  LPRINT"                               PUMPING WELL DRAWDOWN SIMULATION"
1230  LPRINT
1235  LPRINT"        PARAMETERS"
1240  LPRINT"        ----------"
1245  LPRINT
1250  A$="       K      =  ##.##^^^^ cm/s"
1255  B$="       THICK  = ###.##     m"
1260  C$="       SY     =   #.###"
1265  D$="       Q      = ###.##     l/min"
1266  Q=Q*60*1000
1267  K=K*100
1270  LPRINT USING A$;K
1275  LPRINT USING B$;THICK
1280  LPRINT USING C$;SY
1285  LPRINT USING D$;Q
1290  LPRINT CHR$(10)"      DRAWDOWN RESULTS"
1295  LPRINT         "      ----------------"
1300  LPRINT CHR$(29)
1301 '
1302 'PRINT OUT HEADER FOR TABLE
1303  LPRINT "         TIME                          RADIAL POSITION (m)"
1305  LIN$="         (days)"
1306  FOR I=1 TO RMAX
1307     NUM$=STR$(R(I))
1308     BLANK=10-LEN(NUM$)
1309     FOR L=1 TO BLANK
1310        LIN$=LIN$+" "
1311     NEXT L
1312     LIN$=LIN$+NUM$
1313  NEXT I
1314  LPRINT LIN$
```

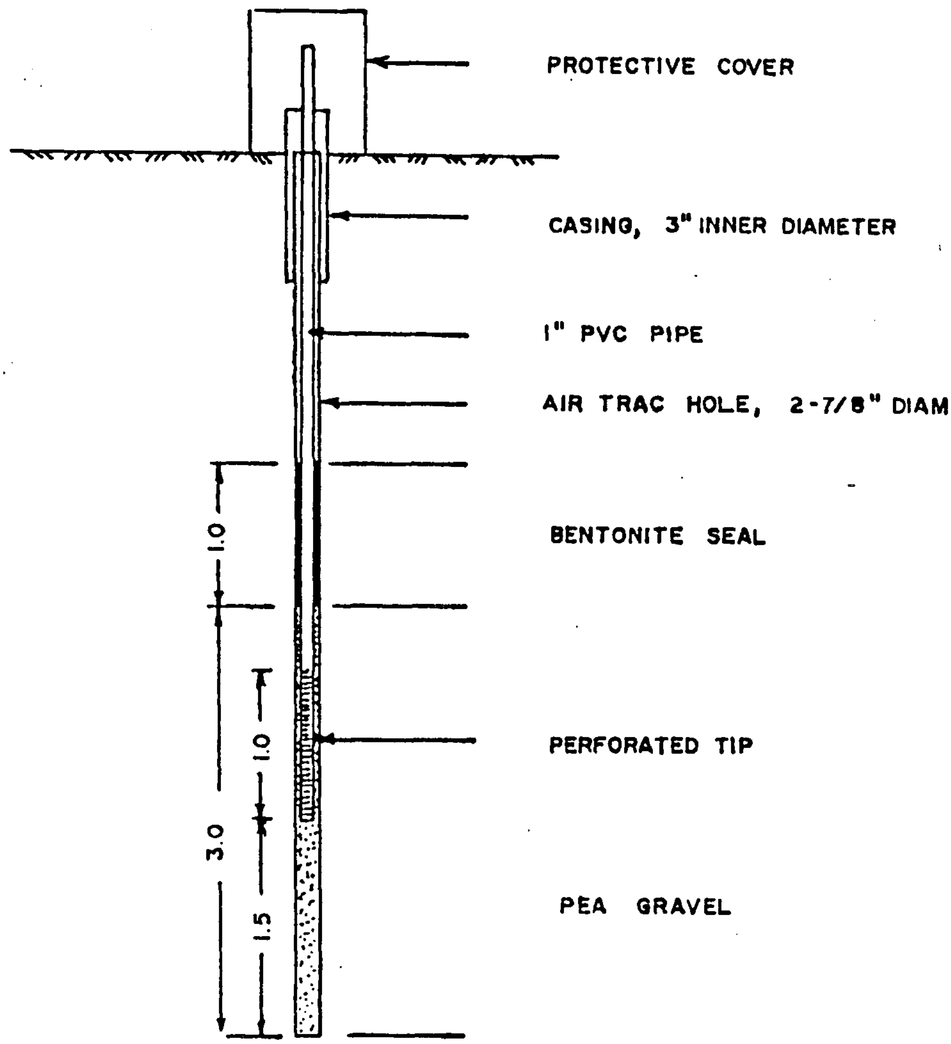
```
1315 LENGTH=LEN(LIN$)
1316 FOR I=1 TO TMAX
1317    F$="          "+STRING$(LENGTH,"-")
1318 NEXT I
1320 LPRINT F$
1321 '
1322 'PRINT DRAWDOWNS FOR TIME T(I)
1323 FOR I=1 TO TMAX
1324   T(I)=T(I)/24/60/60
1325   TIMES$=STR$(T(I))
1326   BLANK=5-LEN(TIMES$)
1327   TIMES$=SPACE$(BLANK)+TIMES$
1331   LIN$="       "+TIMES$+"  "
1332   FOR J=1 TO RMAX
1333      NUM$=STR$(H(I,J))
1334      G$="."
1335      POSI=INSTR(NUM$,G$)
1336      POSI=POSI+2
1337      LNUM$=MID$(NUM$,1,POSI)
1338      BLANK=10-LEN(LNUM$)
1340      FOR L=1 TO BLANK
1345         LIN$=LIN$+" "
1350      NEXT L
1355      LIN$=LIN$+LNUM$
1360   NEXT J
1365   LPRINT LIN$
1370 NEXT I
1372 LPRINT CHR$(30),CHR$(27)CHR$(11)CHR$(48)CHR$(52)
1375 STOP
1380 END
1385 '
1390 '####################################################################
1395 '                     SUBROUTINE FACTORIAL
1400 '####################################################################
1405 'THIS SUBROUTINE CALCULATES THE FACTORIAL OF A NUMBER INPUT AS X AND
1410 'ASSIGNS THE RESULT TO VARIABLE FACT. VARIABLE COUNTER = II.
1415 FACT=1
1420 FOR II=1 TO X
1425    FACT=FACT*II
1430 NEXT II
1435 RETURN
1436 '
1440 '####################################################################
1990 '                     SUBROUTINE WSERIES
1991 '####################################################################
1992 'THIS SUBROUTINE EVALUATES THE WELL FUNCTION BY THE FIRST TEN TERMS OF AN
1993 'INFINITE TAYLOR SERIES FOR THE INTEGRAL.
1994 LMAX=10
1999 UU#=U
2000 SUM#=LOG(UU#)
2001 FOR L=1 TO LMAX
2002    X=L
2003    GOSUB 1405
2004    TERM#=(-UU#^L)/(L*FACT)
2005    SUM#=SUM#+TERM#
2007 NEXT L
2008 W=-1.263298-SUM#+.56
2009 RETURN
2010 '
2014 '####################################################################
3000 '                     SUBROUTINE WSIMPSON
3005 '####################################################################
3010 'THIS SUBROUTINE USES THE SIMPSON RULE TO EVALUATE THE WELL FUNCTION
3015 'INTEGRAL FOR VALUES OF U GREATER THAN 0.5 BECAUSE THE TAYLOR SERIES DOES
3020 'NOT CONVERGE RAPIDLY ABOVE 0.5.
3025 '
3026 'DETERMINE SMALLEST UPPER BOUND FOR INTEGRAL TO INCREASE ACCURACY
3031 IF U <=3! THEN PHIGH=3!
3032 IF U <=3! THEN PINF=.013
3033 IF U <=6! THEN PHIGH=6!
```

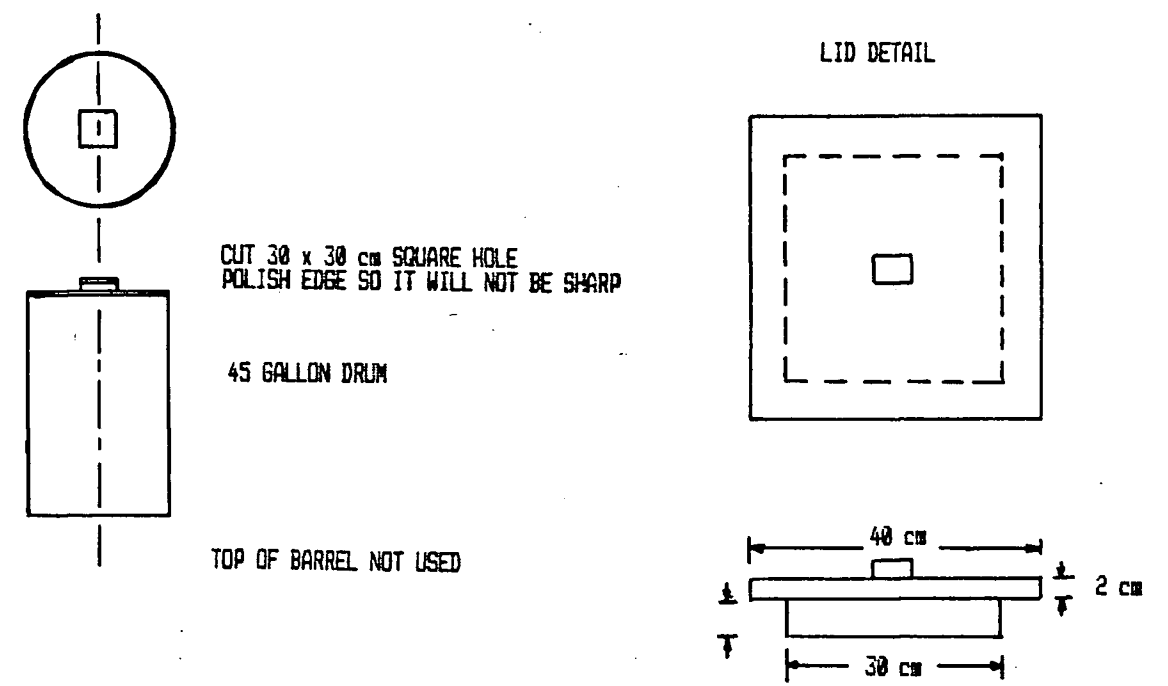
```
3034  IF U <=6! THEN PINF=.00036
3035  IF U <=9! THEN PHIGH=9!
3036  IF U <=9! THEN PINF=.000012
3042  IF U <=9! GOTO 3080
3065  W=.000012
3066  GOTO 3200
3070 '
3075 'EVALUATE THE INTEGRAL
3080  PLOW=U
3085  LMAX=100
3090  DEL=(PHIGH-PLOW)/(LMAX-1)
3095  SUM=0
3100  ODD=0
3105  EVEN=0
3110  X=PHIGH
3115  GOSUB 3230
3120  SUM=SUM+FX
3125  X=PLOW
3130  GOSUB 3230
3135  SUM=SUM+FX
3140  JJMAX=LMAX/2-1
3145  FOR JJ=1 TO JJMAX
3155     X=X+DEL
3160     GOSUB 3230
3165     EVEN=EVEN+FX
3170     X=X+DEL
3175     GOSUB 3230
3180     ODD=ODD+FX
3185  NEXT JJ
3190  SUM=DEL/3*(SUM+4*ODD+2*EVEN)
3195  W=SUM+PINF
3200  RETURN
3205 '
3210 '**********************************************************************
3215 '                            SUBROUTINE FUNCTION
3220 '**********************************************************************
3225 'THIS SUBROUTINE EVALUATES THE FUNCTION TO BE INTEGRATED.
3230  FX=EXP(-X)/X
3235  RETURN
3240 '
3245 '**********************************************************************
```

J.1 PIEZOMETER INSTALLATION

This appendix describes the method of piezometer installation used in all piezometers completed in 1984. Figure J.1 is a generalized illustration of a completed piezometer, showing approximate dimensions of fill obtained using the following procedure:

- check depth to bottom of hole and record.
- slowly pour 3/4 of bucket of pea gravel
- perforate bottom 1.0 m of 1" pvc pipe
- glue up pipe to sufficient length to allow 1.5 m stick up
- lower pipe down the hole, caution perforated section breaks easily
- pour 1/2 bucket of pea gravel, shake pvc pipe continuously while pouring
- slowly pour 1/2 bucket of bentonite down hole
- back fill hole with cuttings
- cut off remaining pipe, leave 1.5 m stick up
- install protective cover (see Figure J.2 for design)





note:
- 8 COVERS REQUIRED
- PAINT BRIGHT RED FOR VISIBILITY
- PAINT LIDS BLACK
- LABEL BARRELS ON SIDES AS FOLLOWS:
  P1-3, P4, P5, P6, P7, P8, P9, P10

EQUITY SILVER MINES LTD.

PIEZOMETER INSTALLATION COVER

FIGURE J.2

J.2 PIEZOMETER LOCATION

Piezometer Number	Northing	Easting	Elevation at Ground	Depth	Pipe Stickup
P01	8305.02	8715.83	1330.28	147.50	
P02	"	"	"	61.80	
P03	"	"	"	38.20	
P04	7663.43	8961.48	1358.99		
P05	7475.67	8893.77	1377.28	23.50	1.00
P06	7523.34	8821.34	1339.52	26.50	0.60
P07	7686.51	8885.23	1340.18	28.00	0.90
P08	7505.50	8844.54	1359.88	26.90	1.00
P09	7343.36	8579.31	1340.33		
P10	8028.30	8833.00	1320.09		
P11	8013.38	8887.08	1351.85	27.70	
P12	8006.88	8929.90	1359.30	26.40	
P13	8200.84	8837.02	1346.25		
P14	8198.63	8914.05	1353.42	27.70	
K01	7612.50	8643.75	1280.00	9.00	
K02	8119.06	8624.06	1318.55	151.49	

J.3 PIEZOMETER MONITORING

DATE	P1	P2	P3	P4	P5	P6	P7	P8	P9	P10	P11	P12	P13	P14
84/05/14	17.60	18.65	25.30											
84/05/30	13.35	15.23	21.36	10.21										
84/06/07	13.60	17.38	23.13	10.61										
84/06/12	13.83	18.20	23.91	11.52										
84/06/18	14.10	19.63	25.51	14.38										
84/06/25	21.50	21.54	27.41	18.96										
84/07/04	14.92	22.93	28.82	17.82										
84/07/20	16.62	24.70	31.05	18.86										
84/07/30	17.93	26.44	32.23	20.84										
84/08/17	19.46	27.52	27.08	22.01										
84/08/30	18.10	24.20	23.60	19.35				15.80		15.35	17.50	18.20		27.10
84/09/06					16.70	10.50	15.80	16.55	6.50					

PUMPING WELL DRAWDOWN SIMULATION

PARAMETERS

INFLUENCE OF HYDRAULIC CONDUCTIVITY

K = 1.00E-04 cm/s
THICK = 50.00 m
SY = 0.050
Q = 10.00 l/min

DRAWDOWN RESULTS

TIME	RADIAL POSITION (m)					
(days)	5	10	20	50	100	200
1	99.47	99.77	99.95	99.99	100.00	100.00
5	99.05	99.41	99.73	99.97	99.99	100.00
10	98.87	99.23	99.58	99.90	99.99	100.00
50	98.45	98.81	99.18	99.63	99.88	99.99
100	98.26	98.63	99.00	99.47	99.77	99.95
200	98.08	98.45	98.81	99.29	99.63	99.88
500	97.84	98.20	98.57	99.05	99.41	99.73
1000	97.65	98.02	98.39	98.87	99.23	99.58

PUMPING WELL DRAWDOWN SIMULATION

INFLUENCE OF HYDRAULIC CONDUCTIVITY

PARAMETERS

K = 1.00E-05 cm/s
THICK = 50.00 m
SY = 0.050
Q = 10.00 l/min

DRAWDOWN RESULTS

TIME	RADIAL POSITION (m)					
(days)	5	10	20	50	100	200
1	99.09	99.96	99.99	99.99	99.99	99.99
5	96.33	98.80	99.91	99.99	99.99	99.99
10	94.70	97.74	99.56	99.99	99.99	99.99
50	90.59	94.15	97.33	99.72	99.99	99.99
100	88.77	92.39	95.82	99.09	99.96	99.99
200	86.94	90.59	94.15	98.11	99.72	99.99
500	84.51	88.18	91.81	96.33	98.80	99.91
1000	82.68	86.35	90.00	94.70	97.74	99.56

PUMPING WELL DRAWDOWN SIMULATION

INFLUENCE OF HYDRAULIC CONDUCTIVITY

PARAMETERS

```
K     =   1.00E-06  cm/s
THICK =  50.00       m
SY    =   0.050
Q     =  10.00       l/min
```

DRAWDOWN RESULTS

TIME (days)	RADIAL POSITION (m) 5	10	20	50	100	200
1	99.99	99.99	99.99	99.99	99.99	99.99
5	97.22	99.98	99.99	99.99	99.99	99.99
10	90.95	99.61	99.99	99.99	99.99	99.99
50	63.39	88.06	99.17	99.99	99.99	99.99
100	47.03	77.49	95.67	99.99	99.99	99.99
200	29.63	63.39	88.06	99.84	99.99	99.99
500	5.91	41.51	73.32	97.22	99.98	99.99
1000	-12.28	23.90	58.29	90.95	99.61	99.99

PUMPING WELL DRAWDOWN SIMULATION

PARAMETERS INFLUENCE OF HYDRAULIC CONDUCTIVITY

K = 1.00E-07 cm/s
THICK = 50.00 m
SY = 0.050
Q = 10.00 l/min

DRAWDOWN RESULTS

TIME (days)	RADIAL POSITION (m) 5	10	20	50	100	200
1	99.99	99.99	99.99	99.99	99.99	99.99
5	99.99	99.99	99.99	99.99	99.99	99.99
10	99.97	99.99	99.99	99.99	99.99	99.99
50	72.29	99.87	99.99	99.99	99.99	99.99
100	9.59	96.13	99.99	99.99	99.99	99.99
200	-88.56	72.29	99.87	99.99	99.99	99.99
500	-266.06	-19.39	91.79	99.99	99.99	99.99
1000	-429.66	-125.07	56.79	99.97	99.99	99.99

PUMPING WELL DRAWDOWN SIMULATION

INFLUENCE OF THICKNESS

PARAMETERS

K = 1.00E-05 cm/s
THICK = 10.00 m
SY = 0.050
Q = 10.00 l/min

DRAWDOWN RESULTS

TIME	RADIAL POSITION (m)					
(days)	5	10	20	50	100	200
1	99.92	99.99	99.99	99.99	99.99	99.99
5	95.47	99.80	99.99	99.99	99.99	99.99
10	90.57	98.61	99.99	99.99	99.99	99.99
50	73.51	88.74	97.83	99.99	99.99	99.99
100	64.81	81.69	94.03	99.92	99.99	99.99
200	55.86	73.51	88.74	99.17	99.99	99.99
500	43.85	61.95	79.14	95.47	99.80	99.99
1000	34.71	52.95	70.75	90.57	98.61	99.99

PUMPING WELL DRAWDOWN SIMULATION

INFLUENCE OF THICKNESS

PARAMETERS

K = 1.00E-05 cm/s
THICK = 20.00 m
SY = 0.050
Q = 10.00 l/min

DRAWDOWN RESULTS

TIME (days)	RADIAL POSITION (m) 5	10	20	50	100	200
1	99.58	99.99	99.99	99.99	99.99	99.99
5	95.28	99.30	99.99	99.99	99.99	99.99
10	92.06	97.73	99.90	99.99	99.99	99.99
50	82.40	90.84	97.01	99.96	99.99	99.99
100	77.93	86.75	94.37	99.58	99.99	99.99
200	73.39	82.40	90.84	98.36	99.36	99.99
500	67.35	76.47	85.37	95.28	99.30	99.99
1000	62.77	71.92	80.97	92.06	97.73	99.30

PUMPING WELL DRAWDOWN SIMULATION

INFLUENCE OF THICKNESS

PARAMETERS

```
K      =   1.00E-05 cm/s
THICK  =  50.00      m
SY     =   0.050
Q      =  10.00      l/min
```

DRAWDOWN RESULTS

TIME	RADIAL POSITION (m)					
(days)	5	10	20	50	100	200
1	99.09	99.96	99.99	99.99	99.99	99.99
5	96.33	98.80	99.91	99.99	99.99	99.99
10	94.70	97.74	99.56	99.99	99.99	99.99
50	90.59	94.15	97.33	99.72	99.99	99.99
100	88.77	92.39	95.82	99.09	99.96	99.99
200	86.94	90.59	94.15	98.11	99.72	99.99
500	84.51	88.18	91.81	96.33	98.80	99.91
1000	82.68	86.35	90.00	94.70	97.74	99.56

PUMPING WELL DRAWDOWN SIMULATION
INFLUENCE OF THICKNESS

PARAMETERS

K	=	1.00E-05	cm/s
THICK	=	100.00	m
SY	=	0.050	
Q	=	10.00	l/min

DRAWDOWN RESULTS

TIME	RADIAL POSITION (m)					
(days)	5	10	20	50	100	200
1	99.05	99.86	99.99	99.99	99.99	99.99
5	97.35	98.87	99.78	99.99	99.99	99.99
10	96.48	98.16	99.40	99.99	99.99	99.99
50	94.38	96.19	97.91	99.54	99.98	99.99
100	93.47	95.29	97.07	99.05	99.86	99.99
200	92.55	94.38	96.19	98.41	99.54	99.98
500	91.34	93.17	95.00	97.35	98.87	99.78
1000	90.42	92.25	94.09	96.48	98.16	99.40

PUMPING WELL DRAWDOWN SIMULATION

INFLUENCE OF SPECIFIC YIELD

PARAMETERS

K = 1.00E-05 cm/s
THICK = 50.00 m
SY = 0.010
Q = 10.00 l/min

DRAWDOWN RESULTS

TIME (days)	RADIAL POSITION (m) 5	10	20	50	100	200
1	96.33	98.80	99.91	99.99	99.99	99.99
5	92.39	95.82	98.43	99.96	99.99	99.99
10	90.59	94.15	97.33	99.72	99.99	99.99
50	86.35	90.00	93.59	97.74	99.56	99.99
100	84.51	88.18	91.81	96.33	98.80	99.91
200	82.68	86.35	90.00	94.70	97.74	99.56
500	80.25	83.92	87.59	92.39	95.82	98.43
1000	78.41	82.08	85.76	90.59	94.15	97.33

PUMPING WELL DRAWDOWN SIMULATION

PARAMETERS

K = 1.00E-05 cm/s
THICK = 50.00 m
SY = 0.050
Q = 10.00 l/min

DRAWDOWN RESULTS

TIME (days)	RADIAL POSITION (m) 5	10	20	50	100	200
1	99.09	99.96	99.99	99.99	99.99	99.99
5	96.33	98.80	99.91	99.99	99.99	99.99
10	94.70	97.74	99.56	99.99	99.99	99.99
50	90.59	94.15	97.33	99.72	99.99	99.99
100	88.77	92.39	95.82	99.09	99.96	99.99
200	86.94	90.59	94.15	98.11	99.72	99.99
500	84.51	88.18	91.81	96.33	98.80	99.91
1000	82.68	86.35	90.00	94.70	97.74	99.56

PUMPING WELL DRAWDOWN SIMULATION

PARAMETERS

K = 1.00E-05 cm/s
THICK = 50.00 m
SY = 0.100
Q = 10.00 l/min

DRAWDOWN RESULTS

TIME (days)	RADIAL POSITION (m) 5	10	20	50	100	200
1	99.72	99.99	99.99	99.99	99.99	99.99
5	97.74	99.56	99.99	99.99	99.99	99.99
10	96.33	98.80	99.91	99.99	99.99	99.99
50	92.39	95.82	98.43	99.96	99.99	99.99
100	90.59	94.15	97.33	99.72	99.99	99.99
200	88.77	92.39	95.82	99.09	99.96	99.99
500	86.35	90.00	93.59	97.74	99.56	99.99
1000	84.51	88.18	91.81	96.33	98.80	99.91

PUMPING WELL DRAWDOWN SIMULATION

INFLUENCE OF SPECIFIC YIELD

PARAMETERS

K = 1.00E-05 cm/s
THICK = 50.00 m
SY = 0.300
Q = 10.00 l/min

DRAWDOWN RESULTS

TIME	RADIAL POSITION (m)					
(days)	5	10	20	50	100	200
1	99.99	99.99	99.99	99.99	99.99	99.99
5	99.30	99.98	99.99	99.99	99.99	99.99
10	98.36	99.81	99.99	99.99	99.99	99.99
50	95.14	98.05	99.63	99.99	99.99	99.99
100	93.42	96.73	99.04	99.99	99.99	99.99
200	91.64	95.14	98.05	99.90	99.99	99.99
500	89.25	92.85	96.24	99.30	99.98	99.99
1000	87.42	91.06	94.60	98.36	99.81	99.99

PUMPING WELL DRAWDOWN SIMULATION

PARAMETERS   INFLUENCE OF SPECIFIC YIELD

K = 1.00E-05 cm/s
THICK = 50.00 m
SY = 0.050
Q = 10.00 l/min

DRAWDOWN RESULTS

TIME (days)	RADIAL POSITION (m) 5	10	20	50	100	200
1	99.09	99.96	99.99	99.99	99.99	99.99
5	96.33	98.80	99.91	99.99	99.99	99.99
10	94.70	97.74	99.56	99.99	99.99	99.99
50	90.59	94.15	97.33	99.72	99.99	99.99
100	88.77	92.39	95.82	99.09	99.96	99.99
200	86.94	90.59	94.15	98.11	99.72	99.99
500	84.51	88.18	91.81	96.33	98.80	99.91
1000	82.68	86.35	90.00	94.70	97.74	99.56

PUMPING WELL DRAWDOWN SIMULATION

PARAMETERS

INFLUENCE OF SPECIFIC YIELD

K = 1.00E-05 cm/s
THICK = 50.00 m
SY = 0.300
Q = 10.00 l/min

DRAWDOWN RESULTS

TIME (days)	RADIAL POSITION (m) 5	10	20	50	100	200
1	99.99	99.99	99.99	99.99	99.99	99.99
5	99.30	99.98	99.99	99.99	99.99	99.99
10	98.36	99.81	99.99	99.99	99.99	99.99
50	95.14	98.05	99.63	99.99	99.99	99.99
100	93.42	96.73	99.04	99.99	99.99	99.99
200	91.64	95.14	98.05	99.90	99.99	99.99
500	89.25	92.85	96.24	99.30	99.98	99.99
1000	87.42	91.06	94.60	98.36	99.81	99.99

PUMPING WELL DRAWDOWN SIMULATION

PARAMETERS

INFLUENCE OF PUMPING RATE

K = 1.00E-05 cm/s
THICK = 50.00 m
SY = 0.050
Q = 10.00 l/min

DRAWDOWN RESULTS

TIME (days)	RADIAL POSITION (m) 5	10	20	50	100	200
1	99.09	99.36	99.99	99.99	99.99	99.99
5	96.33	98.80	99.91	99.99	99.99	99.99
10	94.70	97.74	99.56	99.99	99.99	99.99
50	90.59	94.15	97.33	99.72	99.99	99.99
100	88.77	92.39	95.82	99.09	99.96	99.99
200	86.94	90.59	94.15	98.11	99.72	99.99
500	84.51	88.18	91.81	96.33	98.80	99.91
1000	82.68	86.35	90.00	94.70	97.74	99.56

PUMPING WELL DRAWDOWN SIMULATION

PARAMETERS

INFLUENCE OF PUMPING RATE

K = 1.00E-05 cm/s
THICK = 50.00 m
SY = 0.050
Q = 20.00 l/min

DRAWDOWN RESULTS

TIME (days)	RADIAL POSITION (m) 5	10	20	50	100	200
1	98.19	99.92	99.99	99.99	99.99	99.99
5	92.67	97.61	99.83	99.99	99.99	99.99
10	89.40	95.49	99.13	99.99	99.99	99.99
50	81.18	88.30	94.66	99.44	99.93	99.99
100	77.54	84.78	91.65	98.19	99.92	99.99
200	73.88	81.18	88.30	96.22	99.44	99.99
500	69.03	76.36	83.62	92.67	97.61	99.83
1000	65.36	72.70	80.01	89.40	95.49	99.13

PUMPING WELL DRAWDOWN SIMULATION

PARAMETERS   INFLUENCE OF PUMPING RATE

K = 1.00E-05 cm/s
THICK = 50.00 m
SY = 0.050
Q = 50.00 l/min

DRAWDOWN RESULTS

TIME (days)	RADIAL POSITION (m) 5	10	20	50	100	200
1	95.47	99.80	99.99	99.99	99.99	99.99
5	81.69	94.03	99.58	99.99	99.99	99.99
10	73.51	88.74	97.83	99.99	99.99	99.99
50	52.95	70.75	86.66	98.61	99.99	99.99
100	43.85	61.95	79.14	95.47	99.80	99.99
200	34.71	52.95	70.75	90.57	98.61	99.99
500	22.58	40.91	59.07	81.69	94.03	99.58
1000	13.40	31.76	50.03	73.51	88.74	97.83

PUMPING WELL DRAWDOWN SIMULATION

INFLUENCE OF PUMPING RATE

PARAMETERS

K = 1.00E-05 cm/s
THICK = 50.00 m
SY = 0.050
Q = 100.00 l/min

DRAWDOWN RESULTS

TIME (days)	RADIAL POSITION (m) 5	10	20	50	100	200
1	90.95	99.61	99.99	99.99	99.99	99.99
5	63.39	88.06	99.17	99.99	99.99	99.99
10	47.03	77.49	95.67	99.99	99.99	99.99
50	5.91	41.51	73.32	97.22	99.38	99.99
100	-12.28	23.90	58.29	90.95	99.61	99.99
200	-30.57	5.91	41.51	81.14	97.22	99.98
500	-54.82	-18.16	18.14	63.29	88.06	99.17
1000	-73.18	-36.47	6.93	47.03	77.49	95.67

PUMPING WELL DRAWDOWN SIMULATION

PARAMETERS

K = 1.00E-04 cm/s
THICK = 30.00 m
SY = 0.050
Q = 100.00 l/min

DRAWDOWN RESULTS

TIME (days)	RADIAL POSITION (m) 5	10	20	50	100	200
1	93.20	97.46	99.76	99.99	99.99	99.99
5	86.53	92.33	97.01	99.87	99.99	99.99
10	83.52	89.48	94.93	99.33	99.99	99.99
50	76.45	82.54	88.54	95.68	99.01	99.98
100	73.39	79.50	85.56	93.20	97.46	99.76
200	70.32	76.45	82.54	90.42	95.68	99.01
500	66.28	72.40	78.52	86.53	92.33	97.01
1000	63.21	69.34	75.46	83.52	89.48	94.93