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CURRAGH RESOURCES INC.

GEOTECHNICAL, HYDROLOGICAL AND HYDROGEOLOGICAL REVIEW FOR THE PROPOSED VANGORDA AND GRUM OPEN PITS

AUGUST, 1987

PITEAU ASSOCIATES
GEOTECHNICAL AND
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HYDROGEOLOGICAL CONSULTANTS

KAPILANO 100, SUITE 408
WEST VANCOUVER, B.C.
CANADA V7T 1A2
TELEPHONE (604) 926-8551
TELEX 04-352896

DENNIS C. MARTIN
R. ALLAN DAKIN
ALAN F. STEWART
FREDERIC B. CLARIDGE
TADEUSZ L. DABROWSKI

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by

Andrew T. Holmes

and

Alan F. Stewart

87-947

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1. INTRODUCTION

1.1 TERMS OF REFERENCE

As part of Curragh Resources' continuing work toward developing the Vangorda and Grum deposits over the next few years, Piteau Associates Engineering Ltd. was requested to review and comment on existing data and the proposed geological, geotechnical, hydrological and hydrogeological investigation programs for these deposits. The scope of this study was discussed in a series of telephone calls between Mr. A. Stewart of Piteau Associates and Messrs. M. Pelly and G. Jilson of Curragh Resources in mid-July, 1987. Specifically, Piteau Associates was to review: i) available reports prepared for Kerr Addison Explorations between 1975 and 1978; ii) reports prepared for Cyprus Anvil between 1979 and 1985; and iii) reports recently prepared for Curragh Resources. Other geological and exploration data relevant to geotechnical studies were also to be reviewed. All the above information was to be considered in the context of the proposed site work which is to be carried out later this summer and fall. As most of the proposed site work is related to dewatering the overburden, the emphasis of our review was to be on the hydrological and hydrogeological aspects of the project.

The field aspects of the review were undertaken by Mr. A.T. Holmes during the week of July 27 to 31, 1987. Initially, Mr. Holmes spent approximately three days in Curragh Resources' Whitehorse office, during which time he reviewed the available documents on file. Following this, approximately two days were spent at the site conducting a reconnaissance of field conditions. On returning to Vancouver, further review of relevant data, reports and the proposed field program was undertaken and a report summarizing our findings was prepared.

1.2 DESCRIPTION OF PROPOSED DEVELOPMENT

Two open pits are to be developed on the Vangorda Plateau (see Fig. 1). The Vangorda Pit, with open pit reserves of approximately 6.5 million tonnes, would be the first developed. It is a relatively small pit with a proposed depth of about 70m. In order to mine this pit, Vangorda Creek will have to be diverted around the pit, and two small tributaries will have to be conveyed around the pit in interception trenches. The large diversion and small interception trenches could possibly be combined as one structure.

The Grum Pit, which would start producing ore shortly after Vangorda, is a much larger pit, having open pit reserves totalling 25 million tonnes, plus 1.7 million tonnes in the adjoining Champ Zone. Production from this pit would carry mining and milling operations through to the year 1999. Although there are no major diversions required for this pit, there may be water related problems due to the 270m depth of the pit, and the 100m plus depth of surficial soils identified on the proposed southeast wall.

1.3 DESCRIPTION OF CURRENTLY PROPOSED WORK

Drilling and excavation work are proposed for the Vangorda and Grum pit areas in the late summer and fall of 1987. Small diversion ditches and shallow finger ditches (i.e. trenches) are proposed in both pit areas to allow drainage of shallow surficial soils. Doal Lake is also to be drained. Trenching is to be carried out along possible diversion alignments and in potential waste dump areas. Drilling of about six holes in overburden, and extending a short distance into rock, is also proposed. Piezometers will be installed in these holes, and a pumping test is planned if sufficient quantities of water are encountered during drilling.

1.4 GEOLOGY

1.4.1 Surficial Geology

There appears to be three basic surficial soils overlying the Grum/Vangorda area. The most widespread is a very dense, well graded (i.e. gravelly sandy silt with some clay) cohesive till-like material which appears to cover most of the Vangorda Pit area, and may be present at thicknesses of greater than 100m over the southeast portion of the Grum orebody. This material also covers most of the potential waste dump areas for the Vangorda Pit, and some of the dump areas for the Grum Pit.

Sand and gravel (possibly deposited as a kame terrace with upper elevation at approximately 1305m) covers the portion of the Grum deposit south and west of the mine access road. It is present as a thin veneer near the proposed Vangorda diversion on the west side of Vangorda Creek, and underlies a portion of the potential waste dump area for the Grum Pit. This material is generally a well graded, very fine sand to fine gravel. Its thickness is not accurately known, but it probably overlies till in most areas.

Colluvium is present as a thin veneer on bedrock over much of the potential waste dump area for the Grum Pit. As this deposit is derived from phyllite, it is generally a silty material with platey gravel.

1.4.2 Bedrock Geology

Calcareous and carbonaceous phyllites of the Vangorda formation will be the dominant rock types exposed in the walls of the proposed pits. Sulphide bearing quartzites and massive sulphides will primarily be mined as ore, but some of these lower grade ore rocks will be left in the walls. Altered Mt. Mye phyllites will be exposed near the base of the east wall in the Grum Pit.

The dominant plane of weakness present in the rock mass is the S2 foliation, which, in the Grum pit area, generally appears to dip shallowly (i.e. between about 10° to 35°) to the southwest (G. Jilson, March 29, 1985). However, relatively little detailed interpretation of foliation dip has been completed, and there is considerable evidence that both the strike and dip of the S2 foliation may vary considerably. In addition, there is concern that assumptions made regarding the dip of the S2 foliation during a previous geotechnical study (Montreal Engineering Company Limited, December, 1979) are sufficiently in error such that the results of this previous study must be seriously questioned. Little assessment of faulting in the proposed pit areas has been completed, although it is anticipated that a major fault zone, named the Bankruptcy Break (striking north/south and dipping about 40^{0} west), will be encountered in the northeast wall of the Grum Pit.

2. VANGORDA PIT

The proposed Vangorda Pit is to cover an area approximately 900m x 400m (see Fig. 2). A maximum slope height of 120m will be developed in the north corner of the pit, with slopes on the east side of the pit averaging about 70 to 80m in height. Slopes in surficial soils will range in height from a few metres up to about 35m.

2.1 DIVERSION OF VANGORDA CREEK

Vangorda Creek must be diverted around the pit. The catchment area for the upper portion of the creek which must be diverted is approximately 22 Km^2. A study of a proposed diversion dam and channel has already been conducted by Golder Geotechnical Consultants Ltd. (December, 1979), in conjunction with Hydrocon Engineering (Continental) Ltd. (flood frequency analysis and hydraulic design) and Atmex Geophysics Inc. (seismic survey).

A diversion dam site was identified on Vangorda Creek at the existing Vangorda Plateau access road (see Fig. 1). Based on seismic survey and test pit data, the proposed dam axis is underlain by till overlying bedrock, with some alluvium associated with the present creek channel. If an approximately 15m high diversion dam is to be constructed at this location, drilling will be required along the dam axis to determine foundation conditions to verify that low permeability till does exist down to bedrock, and to delineate the extent of alluvium present. Provided till does exist to bedrock, as is indicated by presently available data, an effective diversion dam, with minimal subsurface leakage, could be constructed at the proposed location.

An alternate diversion structure, located slightly upstream from the proposed dam, is also being considered (see Fig. 1). As this diversion would be located at a higher elevation on the creek, a much smaller structure would be required. This site was dismissed by Golder Associates due to the bedrock scarp on the west side of the creek, which would result in high construction costs for the initial portion of the diversion channel. However, as the cost of constructing the proposed dam along the access road could be very high, it would be worthwhile to investigate the feasibility of locating a diversion structure at a narrower point in the valley. Mr. Holmes reconnoitred Vangorda Creek upstream of the mine access road on July 30, 1987, and a potential dam site, with a rock slope for an east abutment, was identified.

The diversion route initially recommended by Golder Associates follows the 1182m contour around the north end of the pit to an existing drainage channel which, in turn, flows into the existing Vangorda channel. Advantages and disadvantages to this route are:

Advantages	Disadvantages
- Well away from the northeast and east walls of the proposed pit where seepage could cause stability problems.	- Water is being diverted through area of mining activity, hence water quality could be affected by waste dumps, haulroads, etc.
- Diversion empties back into original Vangorda Creek channel.	- Much of the alignment will be constructed in shallow, weathered bedrock which, in highly fractured zones, could be more permeable than till.
	- The outlet to the diversion is very steep; thus a carefully engineered channel would be required to prevent erosion.

An alternate diversion route around the southeast end of the pit is also being considered. This would require a diversion starting at about the same elevation, but would direct water to Dixon Creek, a tributary of Vangorda Creek which is located well outside the area of mining activity. Advantages and disadvantages of this route are:

Advantages	Disadvantages
- Diverted water is kept well away from area of mining activity.	- Channel is located above the east wall of the pit, hence seepage could cause stability problems on the east wall.
- A much gentler grade could be achieved at the outlet end, such that a channel cut into phyllite would probably be all that is required.	- Work may be required on small creek into which Vangorda Creek water is to be diverted. This could represent a large cost due to the length of the creek involved.
- The diversion would intercept two small tributaries to Vangorda Creek that might otherwise flow into the pit.	

As will be discussed in Section 5.1.1, we recommend that the proposed trenching program along both of the potential diversion routes be carried out later this summer. Some minor trenching should also be performed in the upper tributary of Dixon Creek. If dense till exists along most of the alignment above the east wall, leakage is not likely to be a major problem and a channel along this route would warrant serious consideration.

A third diversion option, which is also being considered, involves the following procedure:

i) Moving Vangorda Creek to the northern side of its present flood plain.
ii) Mining the recoverable ore to the south of the relocated creek.
iii) Establishing a final wall on the north side of the creek.
iv) Rerouting Vangorda Creek along a bench on this final north wall.
v) Complete mining in Vangorda Pit.
vi) Flood or backfill pit with waste and re-establish creek in its original channel.

This diversion option would have the advantage of being a short diversion channel which would only convey the creek for a relatively short period of time. However, mill feed from the pit would be disrupted while the creek was being relocated on the north wall of the pit, and leakage from the diversion channel would recharge directly into the pit wall. The mill feed problem may be overcome by scheduling production in the Faro pit or Faro underground operations around the ore gap from the Vangorda pit. Leakage of the diversion may have to be prevented by lining the diversion channel along the north wall of the pit; however, for this relatively short diversion, the cost should not be prohibitive.

If the third diversion option is selected, it would have to be initiated at about the 1145m elevation. Some soils information should be collected along this proposed alignment. In order to obtain this information, two or three trenches should be excavated along the 1145m contour between the points where this contour intersects Vangorda Creek and the northeast wall of the proposed pit.

2.2 DRAINAGE OF SURFACE WATER FROM PIT AREA

A small diversion of a tributary creek above the east wall of the proposed pit, and some ditching in the pit area, are to be carried out later this year. As the flow in the tributary creek is very small (estimated at less than 1 L/s on July 30, 1986), there should be no problem diverting it to the Dixon Creek tributary.

A finger drain ditch system proposed for the pit area this year should drain the surficial soils quite well. Rates of seepage into these ditches are expected to be very small.

Once mining is underway, it would be desirable to collect local runoff plus any seepage from the Vangorda Creek diversion on a bench on the east wall of the pit. As this bench should be at or slightly below the bedrock/surficial soils contact, the 1130 or 1120 benches (see Fig. 2) are probably the most suitable. Pit planning should make provision for constructing small trenches on one of these benches. As gravity drainage from the pit area would be impossible for a trench on the 1120 bench, and very difficult for one on the 1130 bench, a sump near the present location of Vangorda Creek would be required.

2.3 GROUNDWATER IN PIT SLOPES

Two piezometers installed in drillholes along Section 4E (in the Vangorda Valley), plus a flowing hole on Section 12, indicate that the water table is currently very high, being near or slightly above ground surface over the entire pit area.

Gravity drainage towards the pit is expected to result in partial dewatering of the slopes in surficial soils and in bedrock, but high groundwater conditions should be anticipated when assessing pit wall stability. While depressurization along specific structures, such as faults or shear zones, may be achieved with wells or drainholes, it is highly unlikely that measures to reduce pore pressures along foliation planes could be successfully implemented. A piezometer nest, outside the east limit of the pit, should be installed prior to mining, so that pore pressure can be monitored as the pit is excavated. This piezometer nest should be located such that pore pressure information can be obtained on either side of any major faults which may daylight in the northeast wall of the proposed pit.

2.4 SLOPE STABILITY ASSESSMENT

A report describing a preliminary geotechnical investigation and slope design for the Vangorda Pit was prepared by R. Lopaschuk of Cyprus Anvil on October 1, 1980. While the study was based on all data available at the time, and the recommended overall slope angles of 40° for the northeast wall (i.e. where the foliation dips out of the wall) and 45° for the southwest wall (i.e. where the foliation dips into the wall) do not seem unreasonable, it is noted that the analysis and design study had a number of limiting constraints. As discussed in the report, no drillhole information was available in the area of the proposed pit walls and none of the available core was oriented or mechanically logged.

In addition, no structural mapping data or groundwater information was available. However, the strength data that were used in the study, which were based on previous studies done for the Faro and Grum deposits, would seem to be reasonable.

As discussed in the Lopaschuk report, and has been seen at the Faro Pit, S2 foliation is considered to be the most prominent discontinuity affecting slope design. Thus, the determination of the orientation of this structural weakness on a few typical cross-sections through the Vangorda Pit, along with the determination of the mechanical properties of the rock, is considered important for rational slope design. The presence of high groundwater pressures in the slope can have a significant effect on slope stability. Information obtained to date (see Section 2.3, above) indicates that such groundwater conditions do exist and will, in all likelihood, be difficult to alter.

The overburden slopes in the Vangorda Pit are estimated to be up to about 35m high in glacial till. Preliminary design recommendations in the 1980 Lopaschuk report indicate that overall slope angles of 37^{o} could be stable. Based on our brief review of the information provided, and considering the relatively low slope height (i.e. compared to Grum) of the surficial soils, such an overall slope angle is not unreasonable. However, further assessment of the overburden slope angle should be conducted.

Based on the above, it is recommended that further data collection, analysis and design is required for both the overburden and bedrock pit slopes. As will be discussed in more detail in Section 5.1.3, this work should initially involve obtaining oriented core to allow detailed structural interpretation of S2 foliation and other joints (particularly on the northeast wall) which, in turn, will allow a more rational slope design to be carried out. Sampling of the bedrock and overburden materials should also be completed to allow the mechanical and strength properties of these materials to be assessed, and to allow appropriate slope designs to be completed.

2.5 WASTE DUMPS

The most recent plan for the Vangorda pit waste is to create dumps on the slopes above Vangorda Creek or in the small tributary creek valley which exists immediately southwest of the pit. Curragh Resources' personnel will evaluate possible dump sites based on operational constraints, and select one or two potential sites which will have to be investigated in detail. As discussed in Section 3.5, it may be necessary to construct separate dumps for the soil and rock waste.

No waste dump site investigations have been carried out for the Vangorda Pit to date. However, once potential sites have been selected, field reconnaissance and test pitting should be conducted to determine foundation conditions, including type of surficial soils, strength and permeability, and position of the water table. Documentation of conditions in the Faro Pit waste dumps, at least some of which are expected to behave in a similar manner to those for the Vangorda Pit, is also recommended. Further comments and recommendations with regard to siting and investigating the waste dumps are given in Sections 3.5 and 5.1.4.

3. GRUM PIT

The Grum Pit is to be much larger than the Vangorda Pit. As currently planned, it extends over an area of about 1150m x 850m and is about 300m deep as measured from the highest point on the perimeter (see Fig. 3). Slopes in surficial soils will exceed 100m in height in the extreme east corner of the pit, and will average about 60m in height along the northeast and southeast walls.

3.1 DIVERSIONS AND PIT DRAINAGE

There are no major diversions required for the Grum Pit. Two small creeks that feed Doal Lake are the only surface flows that must be diverted around the pit. Current plans are to divert both of these creeks to a small tributary of Vangorda Creek (see Fig. 1), although consideration is being given to diverting the more northerly of these two creeks into another Vangorda tributary further to the west.

The main creek that flows into the east end of Doal Lake is to be diverted later this summer to allow for drainage of Doal Lake. Based on test pits excavated for a foundation investigation (Montreal Engineering Company Limited, October, 1977), it is anticipated that the alignment of the proposed diversion will be underlain by a thin layer of organic material, over about 0.5 to 3.0m of gravelly sand. The gravelly sand overlies a silty sand or sandy silt till. If possible, the invert of the diversion trench should be excavated into the till along the entire length of the diversion alignment. However, this may not be a practicable recommendation for the diversion trench that is to be installed this summer, as it could involve trenching to a very great depth in some areas, making grading difficult. Therefore, the diversion ditch should be excavated to a nominal 1.5m depth, and should be graded as uniformly as possible to the point of discharge. If the ditch, which will be excavated this summer, is intended to be permanent for the life of the mine, the trench should be wide enough to ensure adequate cross-sectional area for the 50 year storm flow. This flow would have to be determined prior to construction. The flow frequency relationship for the drainage basin could be determined using the same equations and procedure as was used for estimating flows in Vangorda Creek (Golder Associates, December, 1979).

Once stripping of overburden from the pit starts, a shallow trench could be located closer to the pit crest, in an area where most of the shallow gravelly sand will have been removed. This trench could intercept local surface runoff, plus any groundwater which seeps under the diversion trench, and convey it by gravity back into the diversion trench at some point further downstream. The tributary creek to Vangorda Creek, which is to receive the diverted water, has coarse channel deposits exposed along its length and should be capable of carrying the expected flows, provided Doal Lake is not drained too quickly.

An in-pit ditch may be required to intercept water on a bench at the base of the slope in surficial soils (see Fig. 5). This ditch would collect any seepage which enters the pit through the surficial soils or along the surficial soils/bedrock contact. A sump would have to be maintained at some point on the bench to enable the collected seepage to be pumped from the pit. Present indications are that this quantity of seepage will be small, as the bulk of the surficial soils are expected to be a dense silty till with low permeability. An investigation program, as discussed in Section 5.2, is to be carried out later this year to determine the nature of these soils.

3.2 GROUNDWATER IN PIT SLOPES

Four piezometers have been installed in the pit area by Montreal Engineering personnel (Montreal Engineering Company Ltd, December, 1979). Three of these were installed in bedrock, while the fourth was installed in overburden. Piezometric heads are all near ground surface and relative hydraulic heads measured between piezometers show that the groundwater flow has a strong southerly component, with the probable sources of recharge being the high ground to the northeast and east of the pit. Groundwater conditions are therefore expected to be worst on the northeast and east walls of the pit.

Some natural depressurization of the slopes in surficial soils is expected to occur due to gravity drainage towards the pit. However, based on observations in the Faro Pit, depressurization of the bedrock slopes is expected to be minimal, particularly on the recharge side of the pit (i.e. northeast and east). Some depressurization of fault structures (eg. Doal Lake Fault, Bankruptcy Break) could probably be achieved by drilling wells or drainholes to intersect them, but depressurization of the entire rock mass would be extremely difficult due to the low permeability of the rock mass and the height of the slopes. Stability analyses should be performed to assess the affect that the major structures may have on the slopes, so that the benefits of depressurizing the structures can be evaluated. Similar analyses should also be performed to determine the benefits of depressurizing foliation planes, although a 30% reduction in pore pressure should be considered as the best depressurization which could likely be achieved.

Results of falling head tests to be run in piezometers, and possibly results of a pump test in a bedrock well, will be available later this year. However, unless these results indicate that the rock mass is reasonably permeable, mine planning should realistically be based on high groundwater conditions in the pit slopes.

3.3 GROUNDWATER IN THE BASE OF THE PIT

An exploration decline was constructed in the Grum Pit area in the mid 1970's. The exploration adit developed from this decline extended down to about the 1130m elevation, with many drillholes fanning out into the ore. Based on a proposal from Canadian Mine Services to Kerr Addison Exploration regarding a pumping system, dated June, 1976, total seepage inflow to the adit during the exploration period was estimated to be about 3 to 5 L/s. Most of this flow evidently came from the ore (Montreal Engineering, December, 1979), which seems reasonable, based on our knowledge of groundwater in the Faro Pit.

The observed flows into the decline were apparently quite low, indicating that either the ore is of low permeability (which is doubtful) or the ore was not being recharged quickly. If the latter is the case, the exploration adit would make an ideal drainage gallery to dewater the ore in the Stage I pit and early phases of the ultimate pit. Rehabilitation of the adit would not be required, as all that is needed is a means of installing a pump near the lowest point in the exploration adit. This could be achieved with a 150mm or 200mm diameter well drilled from ground surface or from a bench on the pit wall. Pilot holes could be drilled until one successfully intersected the adit, and then a larger diameter hole could be reamed out and cased. An initial pumping rate of about 15 to 20 L/s would be desirable to dewater the adit, but a pumping rate of between about 2.5 and 5 L/s would probably be sufficient to maintain a dewatered adit once initial dewatering is complete.

3.4 SLOPE STABILITY ASSESSMENT

The Grum Pit is to be developed in a number of phases or pushbacks. As for the Vangorda Pit, the regional geology in the area of the Grum Pit indicates that the S2 foliation generally dips moderately toward the southwest. Therefore, the design of the northeast wall of the pit will likely be controlled by the foliation. In a report to Cyprus Anvil Mining Corporation dated December, 1979, Montreal Engineering indicated that four diamond drillholes had been drilled and logged and that the S2 foliation dip had been found to dip to the west at between about 33° and 70°. Rock slope design recommendations given in the December, 1979 report for the eastern walls of the pit indicated that overall slope angles of about 30° to 40° would be required to ensure stability. Based on an apparent lack of adversely oriented throughgoing discontinuities on the western side of the proposed pit, is was recommended that the west pit slope be designed at an overall slope angle of 45°.

Subsequent to the studies by Montreal Engineering, a report was prepared by Mr. G. Jilson in March, 1985 that provides an updated assessment of the orientation of the critical S2 foliation planes. In this report, it is concluded that there is considerable doubt as to the validity of previous attempts at correctly defining the orientation of the foliation from drill core and that it is possible that the S2 foliation may dip as flat as 10° to the west near the bottom of the proposed pit wall. In addition, in the upper portion of the pit wall, foliation may actually dip into the wall (i.e. to the east). Considering the concerns with respect to the orientation of the S2 foliation, the preliminary slope designs presented in the December, 1979 report must be questioned. Recommendations for further work, including obtaining and logging oriented core, mapping pit walls as they are developed, etc. are included in Section 5.

Recommendations concerning the stability of slopes excavated in surficial soils were included in a report by Montreal Engineering dated August, 1979. In this report, it was noted that three holes had been drilled into the deep overburden on the eastern side of the proposed pit and that the surficial soils consisted primarily of a gravelly sandy silt with some clay. This material, which was referred to as a till, was categorized as having a high density and low plasticity. Preliminary slope design recommendations for long term slopes cut in this material were: i) 30° overall angles for slopes where seepage is minimal; and ii) $\leq$ 25° overall slope angles where seepage is significant. While, in general terms, these overall slope angles do not appear unreasonable, the soil slopes should probably be benched with 5 to 10m berms at suitable intervals. Revegetation of the slopes may also be desirable. In any event, further work is recommended to arrive at final slope design recommendations in the surficial soils. Such work should be focused on substantiating the findings of the August, 1979 report, investigating the possibility of encountering saturated sand and/or gravel zones in the overburden that might be subject to instability (and may have to be buttressed or otherwise protected), and obtaining accurate strength parameters for the surficial soils. During mining, assessment of the exposed first phase slopes in overburden should be undertaken to obtain further data that will help to optimize the overburden slope design. Further comments with regard to geotechnical work recommended for the surficial soils slopes are included in Section 5.

3.5 WASTE DUMPS

Waste dumps, as initially planned by Kerr Addison Mines, are addressed in the Montreal Engineering report dated August, 1979. While preliminary assessments of safe waste dump angles in the 1979 report indicate overall slope angles of 28° for rock waste, and 23° for surficial soil waste should be acceptable, it is difficult to comment precisely on the accuracy of these recommendatins. Depending on the exact dump location and method of placement of dump materials, the above recommended dumping angles may be either too conservative or too optimistic.

It is our understanding that Curragh Resources has not made any final plans regarding waste dump locations and that the location shown on Fig. 1 is only tentative and likely to change. With regard to siting the waste dumps for the Grum area (and the Vangorda area for that matter), it is suggested that separate dumps for the rock and surficial soils will likely be required and that the best location for any dump material, particularly for the surficial waste, would be in a depression that is surrounded on all sides by sound rock or soil. A mined out pit would be ideal. If this cannot be done, consideration should then be given to a valley or draw that has confinement on at least two sides or, failing this, to locate the waste dumps on as flat topography as possible. While these siting considerations may seem rather extreme for most waste dumps, the generally poor quality of the soil and rock waste may rule out locating and building the waste dumps in a more conventional manner on somewhat steeper, more open terrain.

Depending on the selected locations for the waste dumps, and depending on the properties of the waste materials, consideration may have to be given to impounding and revegetating the dump material. This could be particularly relevant to the unusually large volume of surficial soils which may have a tendency to flow or otherwise fail if loose dumped and allowed to become saturated. Such problems could possibly be overcome by compacting the waste dump in lifts as the material is dumped, or at least by compacting a dam-like structure or buttress of waste, behind which the remaining waste could be loose dumped. Recommendations regarding the investigation and design of waste dumps are given in Sections 5.1.4 and 5.2.4.

4. ENVIRONMENTAL CONCERNS

The following discussion on environmental concerns is limited to potential impacts on water quality, as that is the main facet of the environment which is affected by mine drainage and waste dumps.

4.1 SURFACE WATER QUALITY

Surface water quality is currently monitored at nine locations along creeks which drain the Vangorda Plateau area. One of the water quality monitoring stations is on a tributary of Rose Creek, while the rest are on Vangorda Creek or its tributaries. Discharge from the decline in the Grum deposit, from Doal Lake and from a flowing hole near Doal Lake are also monitored. It is recommended that one more station be added on Dixon Creek, just upstream of its confluence with Vangorda Creek, as Dixon Creek would receive the main Vangorda Creek flow if the easterly (i.e. alternate) diversion option is chosen.

Recent water quality data for the above monitoring network shows that all metal concentrations analyzed for are extremely low, with the exception of the 2.3 mg/L zinc concentration in the Grum decline. The 2.3 mg/L zinc concentration in the water from the Grum decline is above the 0.5 mg/L effluent regulation set by Indian and Northern Affairs; however, as the decline discharge settles and aerates in a pond before it flows into the Creek, zinc concentrations are generally near the allowable concentration where the flow reaches the creek. If the decline is pumped after mining starts, suspended solids and total metals concentrations are expected to increase. This would require close monitoring of the settling pond system to ensure effluent regulations are being met.

4.2 ACID MINE DRAINAGE

Acid mine drainage from the exploration decline has not been a problem to date. This is probably due to the high alkalinity of the groundwater in the area.

However, if sulphide rich waste rock is placed in a dump through which unbuffered, well oxygenated water derived from precipitation can infiltrate, there is a good potential that acid drainage could develop. Waste dump planning and scheduling should consider the problem of acid generating waste. Possible solutions which should be investigated include sandwiching acid generating waste between calcareous phyllites to neutralize the sulphur, or placing it in the bottom of the Vangorda pit, and covering it with other wastes, to ensure that there is no supply of oxygen to the bacterial processes that are required to initiate acid mine drainage.

4.3 GROUNDWATER QUALITY

A flowing hole in the Vangorda pit area (V-322), plus a spring near the discovery outcrop beside Vangorda Creek, should be sampled. These samples would provide some baseline groundwater quality data which could prove to be useful when assessing environmental impacts in the future.

5. RECOMMENDATIONS REGARDING THE UPCOMING FIELD INVESTIGATION AND POSSIBLE FUTURE INVESTIGATIONS

A program involving ditching, drilling and trenching is proposed for later this summer and continuing into the fall. The objectives of this program are to:

i) Investigate possible diversion channel alignments around the Vangorda Pit.

ii) Divert a small creek which flows through the Vangorda Pit area, and drain the shallow surficial soils in the pit area.

iii) Divert a small creek which flows into Doal Lake and construct a ditch to drain the lake.

iv) Drain the shallow surficial soils in the Grum Pit area.

v) Determine the character and permeability of the surficial soils and shallow bedrock in the east area of the Grum Pit, and investigate the feasibility of dewatering these soils prior to mining.

Our recommendations regarding data that should be collected during this program are given in the following, along with our comments on the program as currently proposed. Also included are additional comments concerning anticipated geotechnical matters that will have to be addressed before final mine designs can be prepared.

5.1 VANGORDA PIT AREA

5.1.1 Trenching and Diversion Works

It is recommended that the proposed test trenching program along the potential Vangorda Creek diversion routes be carried out this year. Some trenching should also be carried out to assess valley bottom soils in the tributary of Dixon Creek which would receive diverted water if the eastern route is selected. Surficial soils exposed in the test trenches should be carefully logged, and the strength of the soils should be estimated (see descriptive terms in Appendix A). Two or three percolation tests should also be performed in each representative soil type encountered, to provide an indication of the in-situ permeability. These tests should be performed using the standard methods employed when evaluating effluent drainfield sites (see Appendix A). Some photographs of typical soils should be taken and a few samples should be selected for grain size analysis. Any bedrock exposed in the trenches should be described in terms of lithology and mechanical properties (rippability and fracturing).

The preparation of a limited number of typical cross sections through Dixon Creek should also be carried out to allow an assessment to be made of the capability of Dixon Creek to carry the flow from Vangorda Creek. In conjunction with this, if more recent stream flow data than that available for Hydrocon's assessment of Vangorda Creek in 1979 is available, a review of Hydrocon's study, and possible updating of the flood flows, should be completed.

Once the above information has been assessed, the Vangorda Creek diversion route should be selected and a preliminary design prepared. Following this, additional, more detailed work will be required to provide a better understanding of foundation conditions in critical areas, such as in the area of the proposed diversion structure across Vangorda Creek. If a large dam is to be constructed (as proposed by Golder Associates), and if bedrock is too deep to trench to, or groundwater seepage is too severe, drilling and soil sampling would likely be required to determine foundation conditions and delineate borrow material areas and properties. After the surficial sediment and bedrock characteristics along the chosen diversion route are accurately known, the proposed hydraulic design should be re-evaluated.

The diversion trench that is proposed to be excavated this summer to divert a small creek that flows through the Vangorda pit area should be carefully logged to obtain information on surficial soils. Consideration should also be given to constructing weirs on either end of this diversion to measure seepage losses. This information could prove to be very useful when planning the Vangorda Creek diversion and should be relatively simple to collect. However, this should only be done if little or no seepage into the trench is noted during construction. If too much seepage into the trench is occurring, it would be difficult to estimate true seepage losses between the weirs.

The finger drains that are proposed for the mine area should proceed as planned. While they should do an adequate job of draining the shallow surficial soils, it is expected that this will only involve very low rates of flow.

5.1.2 Investigations for Assessment of Dewatering Requirements

It is understood that some diamond drilling is to be carried out for further ore evaluation in the Vangorda Pit area. If a hole is drilled near V-322 (flowing hole), and a similar water-bearing zone is encountered, a standpipe piezometer should be installed and sealed into the zone making water. If a piezometer is installed, Drillhole V-322 should be pumped and a mini pump test should be performed. Monitoring of the new piezometer would give some indication of the hydraulic connection along the structure which is making water. As this structure must extend well outside the pit area for the artesian head to be maintained, an interception well outside the pit perimeter should be considered when pit excavation begins.

5.1.3 Investigations for Pit Slope Stability

As discussed in Section 2.4, further data collection (i.e. including structural and mechanical logging of oriented core and sampling of soil overburden) should be undertaken before final pit slope designs are prepared for both the bedrock and surficial soils. With reference to the diamond drilling program referred to above in Section 5.1.2, it is recommended that soil sampling and testing of the overburden portion of a few of these holes be carried out to allow the character of the soils at depth to be determined. In this regard, it is suggested that standard penetration tests be performed at regular (i.e. at least every 15m) intervals and when a different soil type is encountered in the drillholes. Sampling of soils using a thick walled drive sampler or by coring should also be undertaken at the same intervals in the drillholes. Depending on the materials obtained, suitable laboratory tests (i.e. such as grain size analyses, Atterberg Limits, etc.) should then be carried out. If downhole geophysics is planned as a regular investigative tool, it is suggested that electrical resistivity (run in an open hole) or neutron porosity or natural gamma (run in a cased hole) could be utilized to delineate various stratigraphies in the surficial soils. These methods may be beneficial in locating saturated sandy, gravelly or cobbly layers which could be the source of stability problems on the soils portion of the pit slope. As a supplement to the above sampling and testing, it is suggested that the tills that seem to comprise much of the surficial material at the site be compared to the tills that are exposed along a portion of the Faro Pit wall. If these soils are considered to be similar, it would be advantageous to document the behaviour of the tills at the Faro Pit with a view toward extrapolating and/or estimating likely soils behaviour at the Vangorda Pit. By doing this, there may be some possible savings in the amount of testing required in the Vangorda area.

As discussed in Section 2.4, the drilling of oriented core is recommended to obtain a more accurate and detailed structural interpretation of the S2 foliation and other relevant structural discontinuities. All core should be mechanically logged as per the core logging techniques outlined in

Appendix B. Sufficient data should be obtained such that at least two or three "typical design sections" through the deposit can be constructed. At this time, it is anticipated that sufficient index and shear strength testing of the rock has been undertaken for a preliminary design of the pit walls and that further such testing may not be required.

5.1.4 Investigations for Waste Dumps

As discussed in Section 2.5, once potential waste dump sites have been selected, field reconnaissance, test pitting and sampling should be conducted to determine foundation conditions. The surficial soils encountered should be classified in terms of soil type, strength and permeability, and the location of the water table, if known, should be recorded. Appropriate laboratory tests, such as grain size analyses and Atterberg Limits, should be conducted, as required.

With regard to investigating the properties and stability of the surficial soil dump materials for purposes of designing stable overburden waste dumps, it is recommended that representative samples of the appropriate soils be obtained and that grain size analyses and Atterberg Limit tests be conducted. Strength and permeability testing at various levels of compaction should also be conducted, the end result being the determination of a relationship between strength and density. If possible, it would also be desirable to develop a relationship between lift thickness and density for loose dumped material.

For the waste rock dump(s), freeze-thaw tests, slake durability tests and swelling tests may have to be conducted to attempt to evaluate the long term behaviour of the waste rock. It is further recommended that, because it is anticipated that the waste rock in the Vangorda and Grum rock dumps will behave in a similar manner to the Faro waste dumps that are predominantly comprised of schist, the relevant Faro waste dumps should be documented with respect to their behaviour and stability. In addition, it is suggested that the Faro waste dumps be test pitted and sampled, that grain size analyses be carried out and that percolation tests be performed in the test pits.

Following the above field and laboratory assessments of the soil and bedrock waste materials, appropriate stability analyses would be completed and design recommendations made for the waste dumps.

5.2 GRUM PIT AREA

5.2.1 Diversions and Ditching

Soils encountered while excavating the diversion and ditches proposed for the Grum area should be carefully logged, as discussed above in Section 5.1.1. A few shallow piezometers (i.e. simple standpipes installed with a backhoe) should be installed throughout the pit area so that the effect of the drains can be evaluated. Finger drains should be excavated from the drains currently proposed for this area to improve not only the rate at which drainage will occur, but also the amount of drainage which will occur.

As discussed in Section 3.1, the diversion ditch to intercept the creek flowing into Doal Lake should be excavated to a depth of about 1.5m. While this may not be sufficiently deep to fully penetrate the sand and gravel which overlies till throughout much of this area, it will be deep enough to cut off most of the seepage through the sand and gravel that flows towards the mine area. Seepage under this diversion could be picked up in another trench, located nearer to the pit crest, after stripping for the pit is underway. If this diversion is to be part of the permanent trench system around the northeast perimeter of the pit, it should be sufficiently wide to handle the 50 year storm flow.

The ditching and diversion works should proceed as soon as possible so that the effectiveness of the diversion and drainage ditches can be evaluated on both a long term and seasonal basis. If any problems are encountered with draining the area, they could then be addressed in a rational manner before mining commences, rather than having to contend with wet conditions during the early phases of stripping.

5.2.2 Investigations for Assessment of Dewatering Requirements

Drilling in the deep surficial soils in the eastern area of the pit is to be carried out later this summer. This drilling is to be performed by a contractor with water well experience, as the main purpose of the program is to obtain hydrogeological data. The budget, as it currently stands, allows for six rotary holes to be completed with piezometers, plus one pumping well.

We recommend that five rotary holes for piezometer installations be drilled at the locations shown on Fig. 4. Each hole should be completed with one piezometer in bedrock, and two or three in surficial soils. All piezometers should be properly sealed with cement so that falling head tests can be performed to provide estimates of hydraulic conductivity. Procedures for conducting falling head tests in sealed standpipe piezometers are included in Appendix C.

The drillers or a supervising geologist should log the soils carefully and keep notes on water flows from the drill. If zones of coarse, clean sediments are encountered which appear fairly permeable, they should be developed briefly so that flow rates can be accurately measured (in a bucket) and recorded. Representative samples of the till and any sand and gravel should be bagged for future reference and possible laboratory testing (see Section 5.2.3 for further comments on sampling and testing).

Once the five piezometer holes have been completed, a decision can be made as to whether or not a pumping well should be constructed. It should only be constructed if significant water bearing zones are encountered during drilling, or if falling head tests indicate there are moderately to highly permeable zones in the till or in the bedrock. The most likely location for a successful pumping well is shown in Figs. 4 and 5. There is the possibility of encountering coarse sediments in the bedrock channel located here, or alternatively, encountering a fracture zone in bedrock associated with the Doal Lake Fault. While it is unlikely that a well, developed in either the bedrock channel or in a fractured zone in rock, would produce more than about 2 L/s, a pump test run at even a low pumping rate can provide valuable hydrogeological data. If a pumping test is performed, it should be continued for a period of a few days to a week, as there could be boundaries encountered many hours after testing begins, due to the expected low permeability of the sediments and rock. All piezometers should be monitored during any pumping tests which are performed.

Data from the drilling program should be evaluated so that an estimate of groundwater conditions which will be encountered when excavating the overburden in the Grum Pit can be made. Planning of groundwater control measures could then be carried out in conjunction with the detailed mine planning.

5.2.3 Investigations for Pit Slope Stability

Recommendations concerning the investigation and design of pit slopes in the Grum Pit are considered to be virtually identical to those discussed in Section 5.1.3 for the Vangorda Pit. However, the substantially increased slope heights in both the overburden and bedrock portions of the Grum Pit serve to increase the need for and the importance of such studies.

It is noteworthy that the phased approach to mining the pit will allow trial interim slopes to be mined and documented before final slope designs are prepared.

With regard to the rotary drilling program to be carried out this summer, it is recommended that, if possible, the driller be equipped to conduct soil sampling and penetration testing as discussed for the Vangorda Pit in Section 5.1.3. He should also be instructed to collect samples of the cuttings every 1.5m in the holes so that visual and laboratory classifications of the materials can be made by an experienced engineer. Such a sampling program, along with the recommended penetration tests, drive sampling (and possibly geophysical logging) and suitable laboratory testing, should give a good indication of the properties of the soils that will be encountered in the pit walls. However, it may still be necessary at some time in the future (i.e. before final slope designs are prepared) to drill and sample the soils using a conventional soil drilling rig. Although the need for downhole pressuremeter tests is uncertain, such testing cannot be ruled out at this time.

Previous recommendations for developing more accurate structural interpretations of the bedrock and developing "typical design sections" through the use of oriented core are also important for the Grum Pit. Such work will allow a more rational initial design for the high pit walls that eventually will be developed at Grum.

An additional source of data that should be utilized in the assessment and determination of final slope design recommendations (for both the rock and soil slopes) will be the exposed interim mining slopes. The phased approach to mining the Grum Pit should uncover a wealth of geotechnical information that should be recorded and analyzed for use in updating the final pit wall design.

5.2.4 Investigations for Waste Dumps

Investigations for waste dumps for the Grum deposit should be carried out in an identical manner to that described in Section 5.1.4 for the Vangorda deposit. These investigations will be even more critical than that for the Vangorda area, due to the increased volume of material that will be mined and the larger size of the waste dumps that will be created.

Respectfully submitted,

PITEAU ASSOCIATES ENGINEERING LTD.

Alan F. Stewart, P.Eng.

for

Andrew T. Holmes, P.Eng.

August 20, 1987

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FIGURES

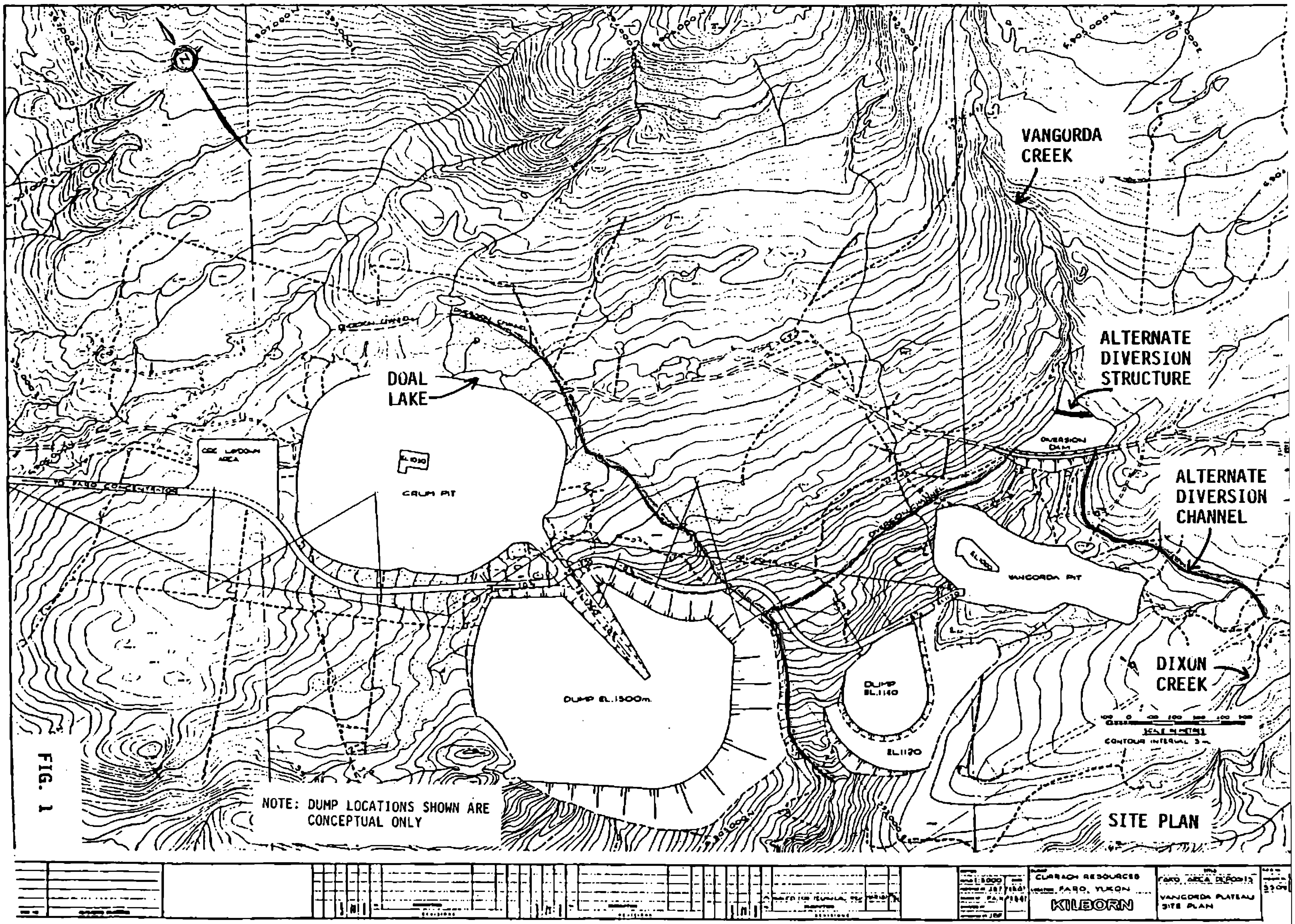

VANGORDA
CREEK
DOAL
LAKE
ALTERNATE
DIVERSION
STRUCTURE
ALTERNATE
DIVERSION
CHANNEL
GRUM PIT
VANGORDA PIT
DUMP EL.1500m.
DUMP
EL.1140
EL.1120
DIXON
CREEK
CONTOUR INTERVAL 3 m.
NOTE: DUMP LOCATIONS SHOWN ARE
CONCEPTUAL ONLY
SITE PLAN
CURRAGH RESOURCES
FARO, YUKON
KILBORN
VANGORDA PLATEAU
SITE PLAN

FIG. 1

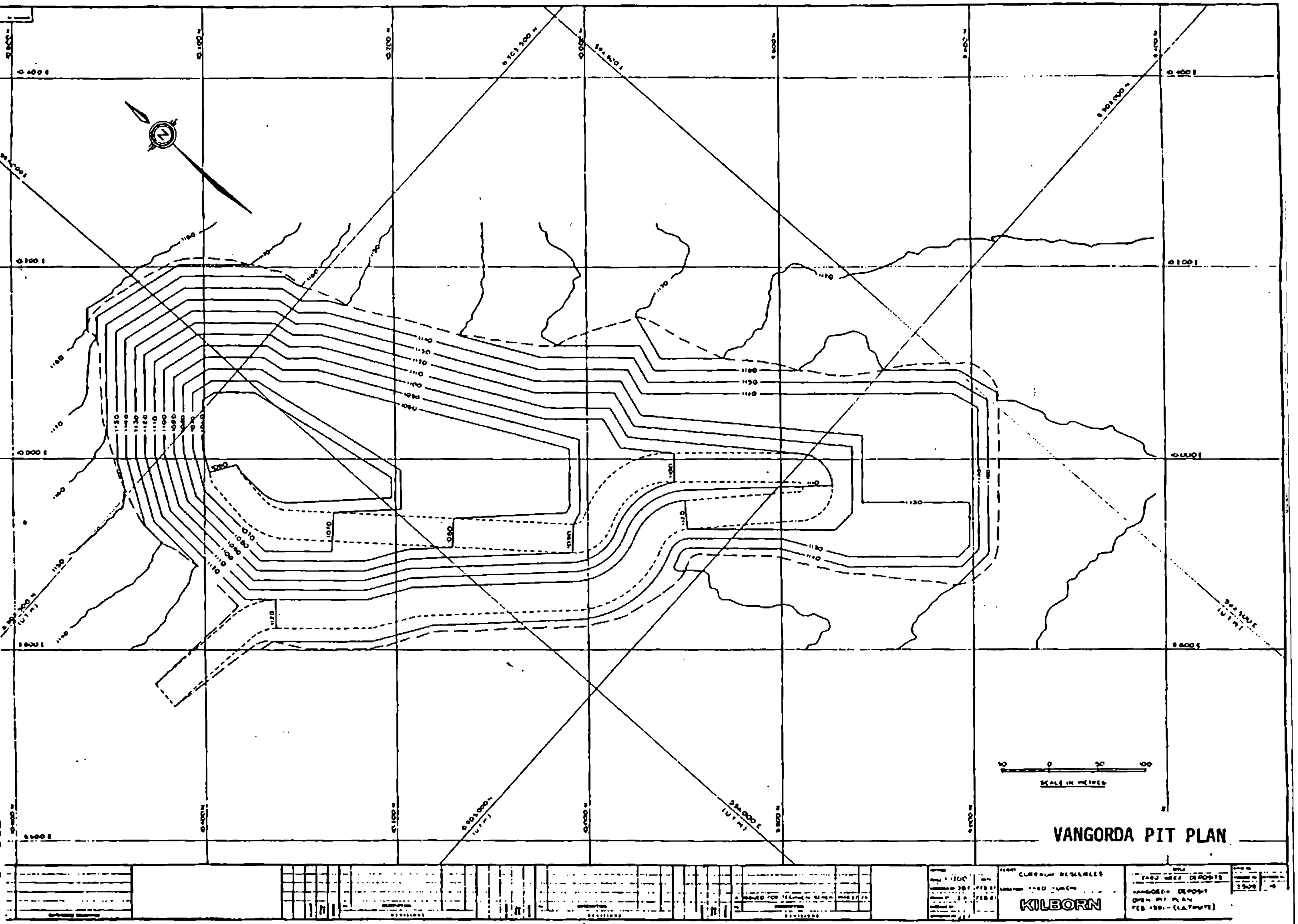
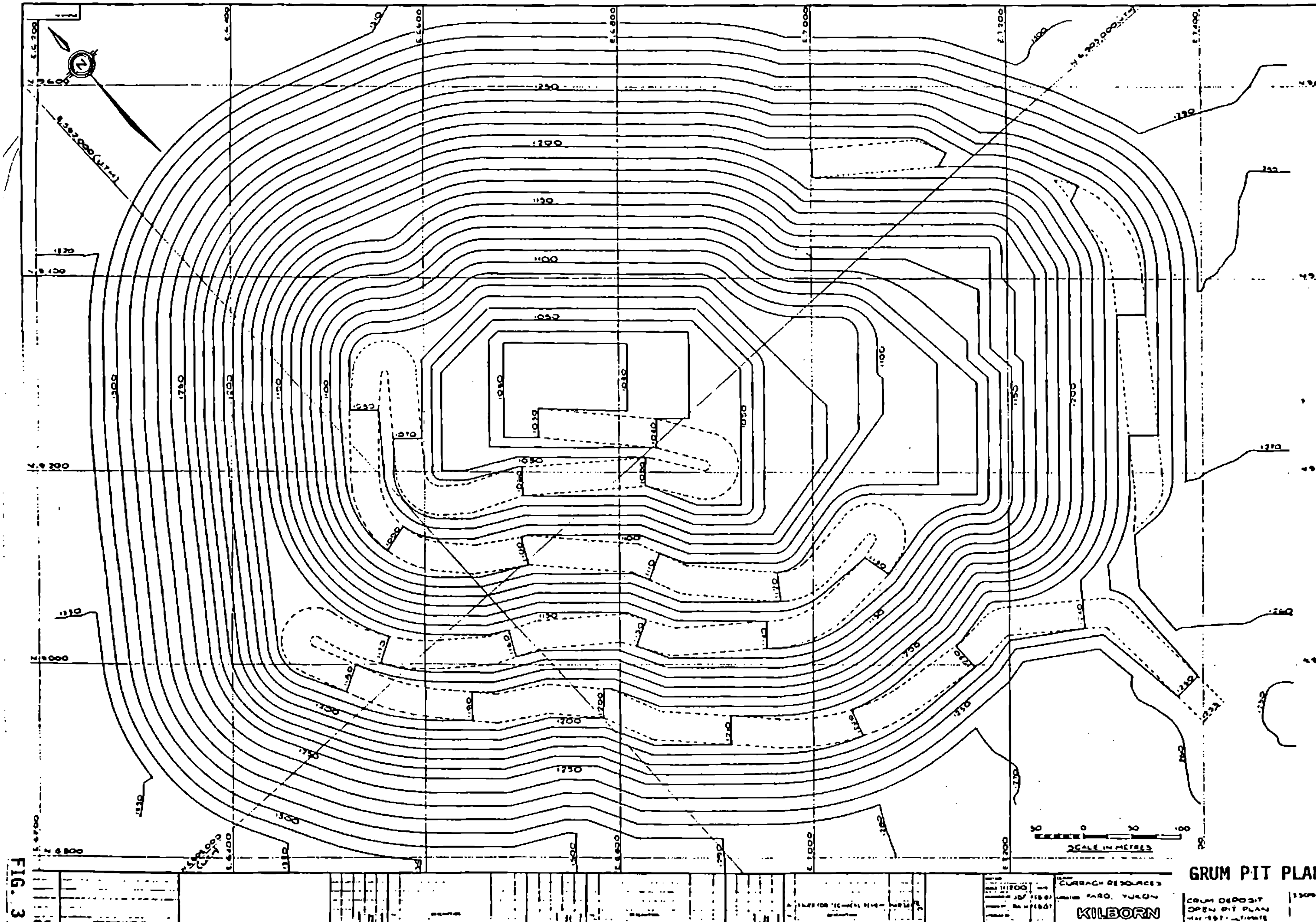
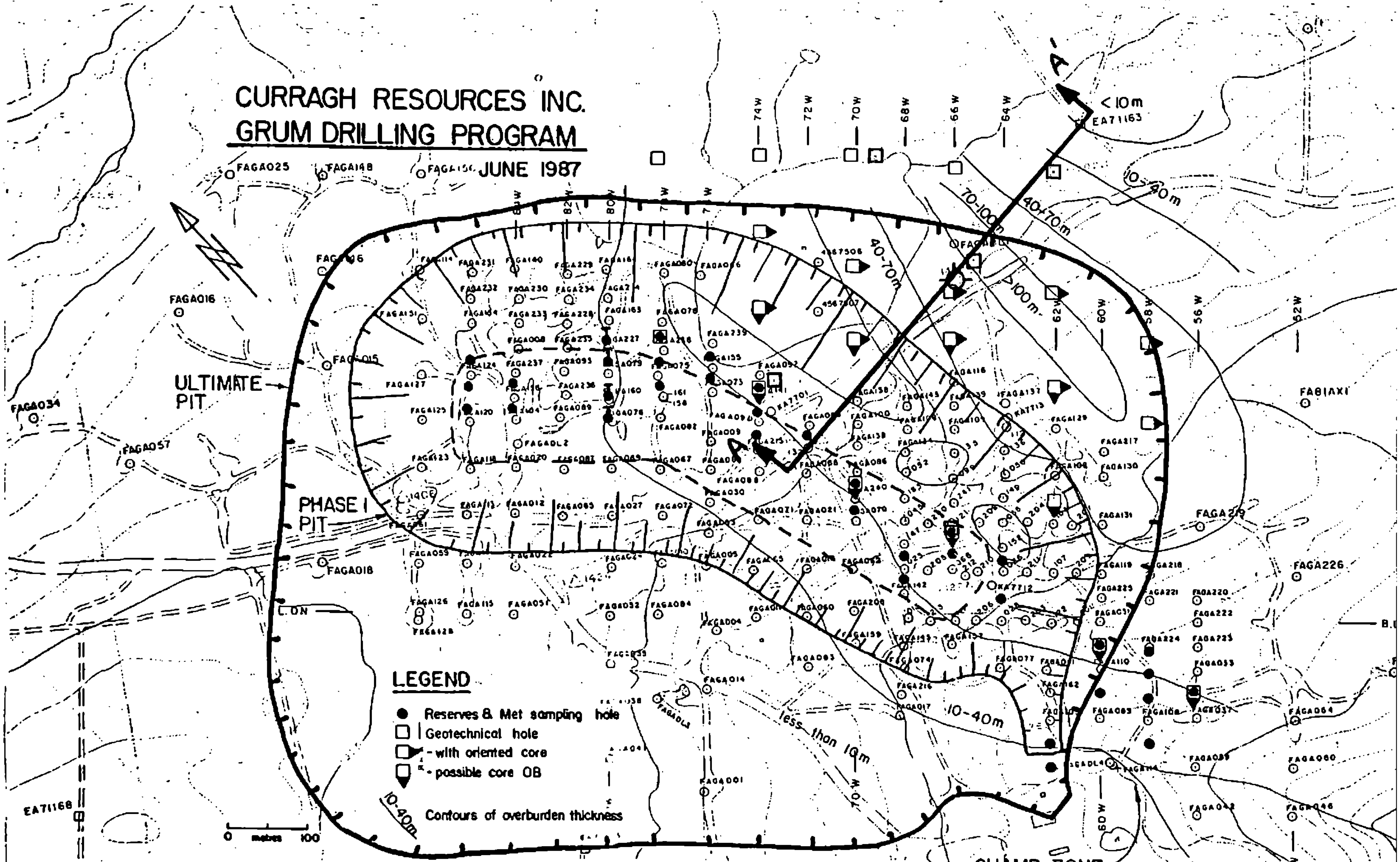


FIG. 2



GRUM PIT PLAN

FIG. 3



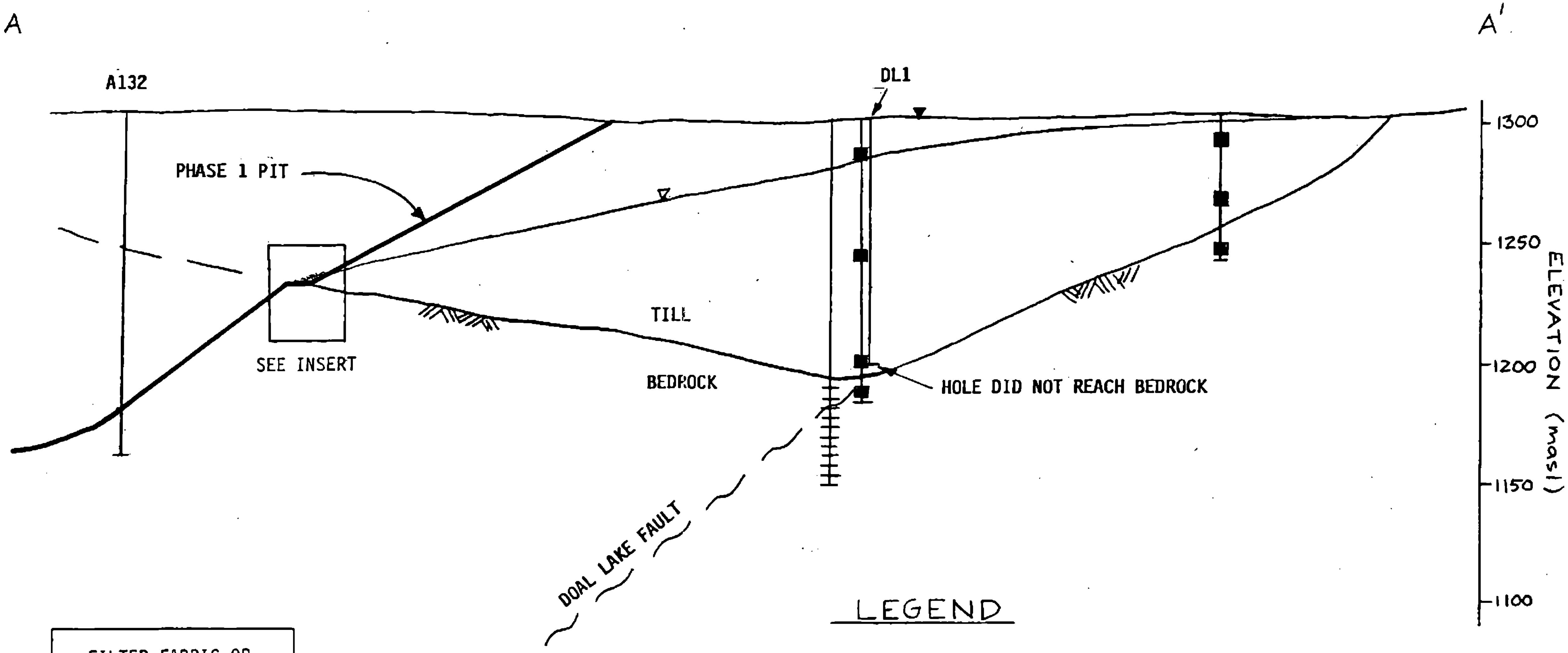
Rotary holes to be instrumented with one piezometer in bedrock and two or three in till (proposed).

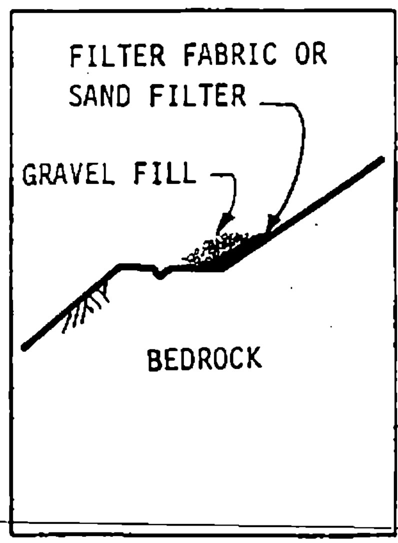
Pumping well (proposed).

PLAN SHOWING PROPOSED DRILLING PROGRAM

FIG. 4

JOB NO 947





POSSIBLE DESIGN FOR TOE BERM TO
STABILIZE TOE OF OVERBURDEN SLOPE

LEGEND

PRESENT WATER TABLE

ESTIMATED WATER TABLE WHEN PIT IS EXCAVATED

EXISTING HOLES

PROPOSED MONITORING HOLE SHOWING ONE STANDPIPE PIEZOMETER COMPLETED IN BEDROCK AND TWO OR THREE COMPLETED IN TILL

PROPOSED PUMPING WELL-SCREEN TO BE INSTALLED IN WATER-BEARING ZONE WITHIN TILL UNIT, IF ENCOUNTERED. HOLE TO BE DRILLED INTO BEDROCK AFTER SHALLOW PUMP TEST. OPEN HOLE IN BEDROCK TO BE TESTED IF WATER ENCOUNTERED IN FAULT OR FRACTURE ZONE.

SCALE = 1:2000
NO VERTICAL EXAGGERATION

FIG.5

NOTE: For location of section see Fig. 4

CURRAGH RESOURCES
VANGORDA/GRUM DEVELOPMENT

PITEAU ASSOCIATES
GEOTECHNICAL CONSULTANTS
VANCOUVER CALGARY

BY: HWN
DATE: AUG '87
APPROVED: AH
DWG:

HYDROGEOLOGICAL SECTION A-A'

APPENDICES

APPENDIX A

SOILS LOGGING INFORMATION AND PERCOLATION TEST PROCEDURE

1. INTRODUCTION

Samples of unconsolidated sediments are collected from drillholes and test pits which were drilled or dug in order to determine subsurface conditions. Based on many years of experience and laboratory testing, the geotechnical/hydrogeological engineering discipline has been able to assign material properties to various classifications of surficial sediments. These material properties can be used to determine whether surface conditions are suitable for any proposed construction, whether it be a foundation, well, drainfield etc. However, if this experience gained from many years of working with various surficial sediments is to be applied to a particular job, it is important that these sediments are logged using a standard classification system. A number of classification systems have been developed, but the UNIFIED SOIL CLASSIFICATION SYSTEM, which is one of the simplest and probably most widely used systems, will be presented in this technical note.

2. TEXTURAL CLASSIFICATION

Textural classifications group surficial sediments according to their grain size characteristics. The divisions for the various particle sizes are shown on Fig. 1. A surficial sediment is described according to percentage of the grain size present. Based on the grain size distribution, the sample can be given a descriptive name (eg. well graded sand and gravel), and a group symbol from the Unified Soil Classification System (Table I).

Group symbols for the Unified Classification System consist of the following:

Primary Letter	Secondary Letter	Grain Size
G: gravel	W: well graded	C: coarse
S: sand	P: poorly graded	m: medium
M: silt	M: non-plastic fines	f: fine
C: clay	C: plastic fines	f-m: fine to medium, etc.
O: organic	L: low plasticity	
P: peat	H: high plasticity	

Descriptive terms for grain size distribution include:

- and (35 to 50%) eg. silt and sand
- y (20 to 35%) eg. silty sand
- little (10 to 20%) eg. sand with little silt
- trace (1 to 10%) eg. sand with trace silt

3. COLOUR

Samples should all be given a colour description. This can be done with the aid of colour charts if correlation is expected to be of great importance, but generally a simple colour description is adequate, eg. light brown or reddish brown.

4. STRENGTH

It is important that an estimate of strength is made, particularly if any foundations are likely to be constructed in the area. Strength classifications are applied to cohesive soils only (clayey and some silty sediments). A commonly used strength classification is given in Table II.

Other descriptive terms should also be included, if appropriate. If the sample is brittle, elastic, friable or sensitive (loses strength on remolding), these terms should also be included in the description.

It should be noted that the above strength scale is applicable to undisturbed samples, and is not relevant to drill cutting samples. However, an estimate of strength based on the drill cuttings is usually adequate in water well drilling, as the strength of fine grained sediments is not an important consideration. If, however, accurate strength parameters are required, appropriate sample procedures (shelby tubes) would be necessary.

5. DENSITY

Density is a term which is applied to non cohesive soils. This parameter is often important in describing aquifer materials, as permeability is a function of density. The Standard Penetration Test (SPT) is the most widely used method for determining the density of sand and gravel deposits. Density descriptions based on SPT results are given in Table III.

As SPT's are seldom carried out in well drilling, the above table is not likely to be very useful. However, it is possible to apply the relative density terms to samples, from drilled holes based on the rate of drill bit or casing advance through sand and gravel, or on the drillers estimate of the density of the formations drilled.

6. PARTICLE DESCRIPTION

In granular materials, it is also necessary to describe the appearance of individual grains. Wherever relevant, the following descriptive categories should be used:

i) Shape - rounded, subrounded, subangular, angular (see shape classification code in Fig. 2 attached).
ii) Mineralogy - of larger grains only, eg micaceous, quartzitic etc.

7. SOIL STRUCTURE

Any pertinent soil structures should also be described. This applies more to test pit and other undisturbed samples, as drill cuttings can only provide indications of large scale structure (eg. thickly bedded). Terms which could be used include:

- homogeneous
- laminated (very thinly bedded)
- stratified (or thickly bedded)
- varved (alternating laminations of silt and clay)
- fissured (dessication cracks)

- weathered (various degrees)
- roots (present to what depth)
- slickensided (fractured planes appear glossy)
- blocky (material can be broken in small and hard angular lumps)
- lensed (small pockets of different texture)

9. SOME USEFUL FIELD TESTS

9.1 Shaking Test

A small amount of wet soil (of silt-clay grain size) is worked until it has the consistency of soft putty. It is then placed in the palm of the hand and shaken slowly. If it becomes soft and glossy with shaking, and then becomes hard and cracked when squeezed slightly, the sample has "dilatancy". If the dilation occurs quickly, the sample is likely a non-plastic silt. If it occurs slowly, the sample is an organic silt, a slightly clayey silt or a nonplastic silt with a high liquid limit. No reaction indicates a clay or silty clay.

9.2 Grit Test

A very fine sand can be differentiated from a silt by putting a small portion between your tooth and biting. Any grittiness indicates the presence of fine sand.

10. SAMPLE DESCRIPTION

Based on examination in the field, it should be possible to fit each soil sample into one of the Unified Soil Classifications shown in Table I. In the case of fine grained soils, some laboratory analyses are generally necessary to accurately determine plasticity, which is one of the main criteria for classifying silty and/or clayey soils (Table IV). However, a reasonable estimate, based on field examination, is often all that is required, particularly once the observer has required some experience in the classification of soils.

Two examples of complete soil descriptions are given below:

'Reddish - brown, dense, homogeneous, well graded, clean sand consisting of subangular particles - SW'

'Dark grey, firm, silty clay of low plasticity with small fissures and silt inclusions - CL'

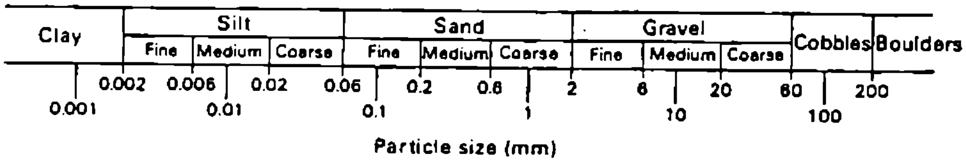


FIG. 1 Particle Size Ranges

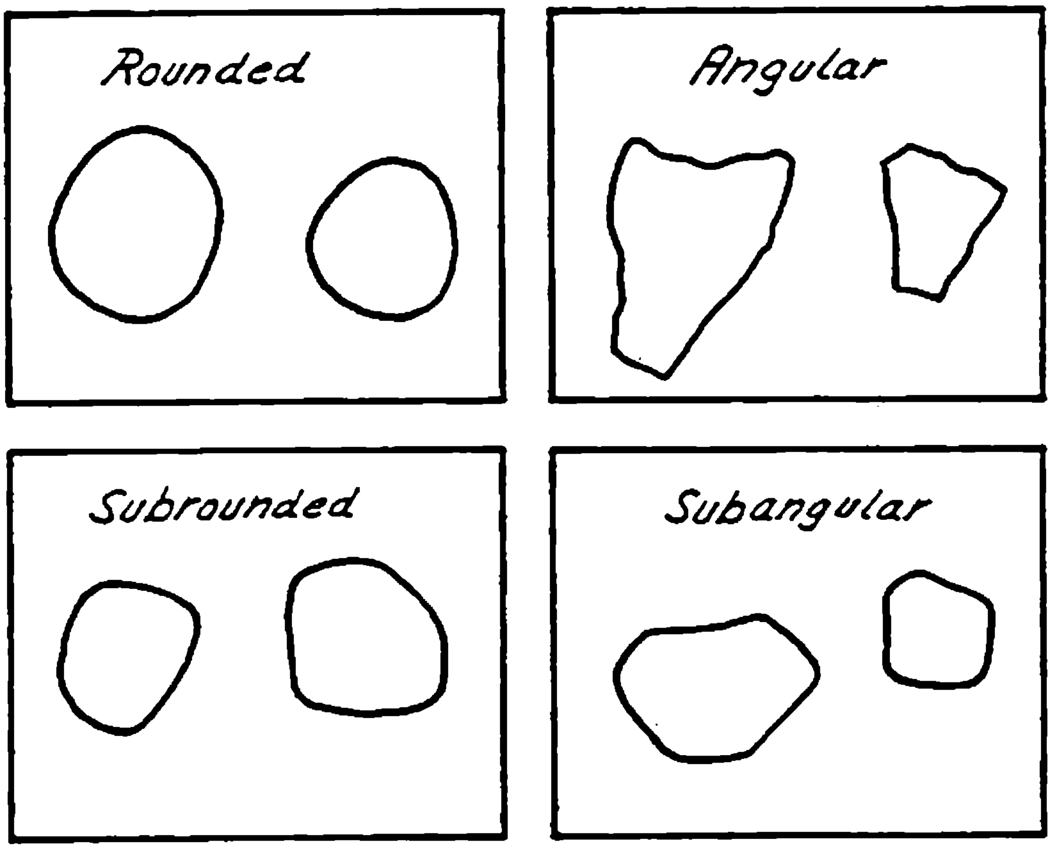


FIG. 2 Soil Grain Shapes

TABLE I

UNIFIED SOIL CLASSIFICATION SYSTEM

Description			Group symbols	Laboratory criteria			
				Fines (%)	Grading	Plasticity	Notes
Coarse grained (More than 50% larger than 63 μm BS sieve size)	Gravels (More than 50% of coarse fraction of gravel size)	Well-graded gravels, sandy gravels, with little or no fines	GW	0–5	$C_U > 4$ $1 < C_C < 3$		Dual symbols if 5–12% fines. Dual symbols if above 'A' line and $4 < PI < 7$
		Poorly-graded gravels, sandy gravels, with little or no fines	GP	0–5	Not satisfying GW requirements		
		Silty gravels, silty sandy gravels	GM	> 12		Below 'A' line or $PI < 4$	
		Clayey gravels, clayey sandy gravels	GC	> 12		Above 'A' line and $PI > 7$	
	Sands (More than 50% of coarse fraction of sand size)	Well-graded sands, gravelly sands, with little or no fines	SW	0–5	$C_U > 6$ $1 < C_C < 3$		
		Poorly-graded sands, gravelly sands, with little or no fines	SP	0–5	Not satisfying SW requirements		
		Silty sands	SM	> 12		Below 'A' line or $PI < 4$	
		Clayey sands	SC	> 12		Above 'A' line and $PI > 7$	
Fine grained (More than 50% smaller than 63 μm BS sieve size)	Silts and clays (Liquid limit less than 50)	Inorganic silts, silty or clayey fine sands, with slight plasticity	ML	Use Plasticity Chart			
		Inorganic clays, silty clays, sandy clays of low plasticity	CL	Use Plasticity Chart			
		Organic silts and organic silty clays of low plasticity	OL	Use Plasticity Chart			
	Silts and clays (Liquid limit greater than 50)	Inorganic silts of high plasticity	MH	Use Plasticity Chart			
		Inorganic clays of high plasticity	CH	Use Plasticity Chart			
		Organic clays of high plasticity	OH	Use Plasticity Chart			
Highly organic soils		Peat and other highly organic soils	Pt				

TABLE II

CONSISTENCIES OF COHESIVE SOILS

Consistency	Field Identification	Approximate undrained shear strength*	
		kPa	lb/sq ft
Very soft	Easily penetrated several centimetres by the fist	<12	<250
Soft	Easily penetrated several centimetres by the thumb	12-25	250-500
Firm	Can be penetrated several centimetres by the thumb with moderate effort	25-50	500-1000
Stiff	Readily indented by the thumb but penetrated only with great effort	50-100	1000-2000
Very Stiff	Readily indented by the thumbnail	100-200	2000-4000
Hard	Indented with difficulty by the thumbnail	>200	>4000

*The undrained shear strength is taken as 1/2 of the compressive strength.

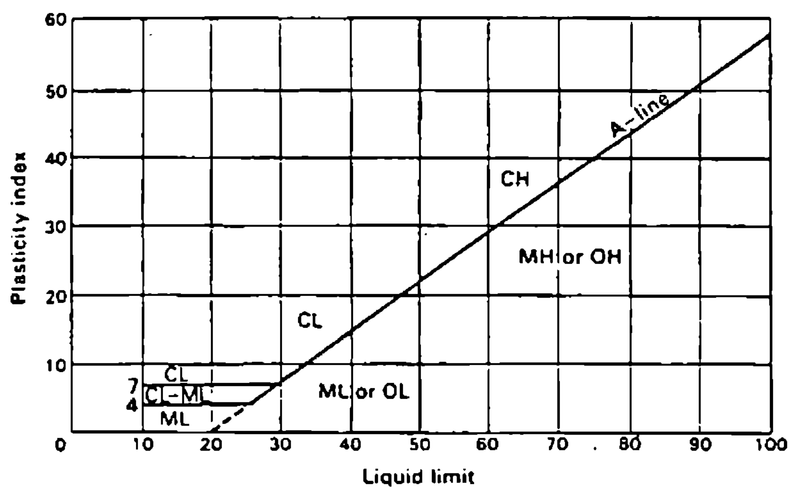
TABLE III

RELATIVE DENSITY OF GRANULAR SEDIMENTS

Blows Required to Drive sampler 1 Foot	Relative Density
0 - 4	Very Loose
5 - 11	Loose
11 - 20	Firm
21 - 30	Very Firm
31 - 50	Dense
Over 50	Very Dense

TABLE IV

PLASTICITY CHART



PERCOLATION TEST PROCEDURE

determine the suitability of the soil to absorb effluent by conducting percolation tests as follows:

(i) Percolation test holes shall be made at points and elevations selected as typical in the area of the proposed absorption field.

(ii) One of these test holes shall be dug at each end of the area of the absorption field. Further holes may be required, depending upon the nature of the soil, the results of the first tests and the size of the proposed absorption field.

(iii) Test holes shall be 12 inches square and excavated to the proposed depth of the absorption field.

(iv) To make the percolation test more accurate, any smeared soil should be removed from the walls of the test holes.

(v) If the soil contains considerable amounts of silt and/or clay, the test holes must be presoaked before proceeding with the test. Presoaking is accomplished by keeping the hole filled with water for four hours or more. Proceed with the test immediately after presoaking.

(vi) To undertake the test, fill the test hole with water. When the water level is 5 inches or less from the bottom of the hole, refill the hole to the top. No recording of time need be done for these two fillings.

(vii) When the water level, after the second filling (step vi) is 5 inches or less from the bottom of the hole, add enough water to bring the depth of water to 6 inches or more.

(viii) Observe the water level until it drops to the 6 inch depth. At precisely 6 inches, commence timing. When the water level reaches the 5 inch depth, stop timing. Record the time in minutes.

(ix) Repeat procedures (vii) and (viii) until the last two rates of fall do not vary more than 2 minutes per inch.

(x) Record and report all rates of fall in minutes per inch.

(xi) Determine the percolation rate for the proposed sewage disposal system by averaging the slowest rate determined for each of the test holes.

(xii) Backfill the holes with the excavated soil and flag their locations.

From B.C. Public Health Act
dated September 3, 1975

APPENDIX B

DESCRIPTION OF ROCK MECHANICS

CORE LOGGING TECHNIQUE

DESCRIPTION OF ROCK MECHANICS

CORE LOGGING TECHNIQUE

Prepared

by

PITEAU ASSOCIATES ENGINEERING LTD.

AUGUST, 1986

APPENDIX III

DESCRIPTION OF ROCK MECHANICS CORE LOGGING TECHNIQUE

The basic parameters measured from the rock core are as follows:

1. Core recovery
2. Rock hardness
3. Degree of fracturing (breakage)
4. Degree of weathering
5. Core size

It is noteworthy that the best data on core competency can be collected by the drill inspector at the drill site before the core becomes broken or data lost from excessive handling, splitting, or drying out.

The data on the various parameters may be tabulated on appropriate recording forms (see Fig. 1) and presented graphically for specific boreholes on geological sections or plans.

A detailed description of each of the parameters recorded is given in the following:

1. <u>CORE RECOVERY AND RQD</u>

   Core recovery is expressed as a percentage of the total length drilled for each core run which is marked by wooden blocks in the core boxes. Recovery gives an indication of the quality of the ground being drilled and the general competency of the rock. Low recovery may also be indicative of faults.

2. <u>RQD (ROCK QUALITY DESIGNATION)</u>

   The RQD is defined as the percentage of core in each run in which the spacing between natural fractures is greater than 10cm (4 in.).

3. <u>HARDNESS</u>

   A simple scheme for classifying soil or rock according to its consistency or hardness is given in Table I. Using this scheme, a reasonble first estimate of the unconfined compressive strength (q_u) of the material may be made. Classifications are based on simple mechanical tests which can be easily performed in the field. By the use of fingers, a pocket knife and geologic pick and with a minimum amount of experience, the complete range of classifications can be established in the field.

TABLE I

QUALITATIVE & QUANTITATIVE EXPRESSIONS

FOR CONSISTENCY OF COHESIVE SOIL AND ROCK*

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

*. Modified Rock Hardness Classification

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

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

4. DEGREE OF BREAKAGE

Degree of Breakage is a visual and thus somewhat subjective estimation of the quality of the rock in terms of the number of fractures or breaks. General categories, numerical equivalents and qualifying descriptions are given below. Photographic illustrations of the Degree of Breakage Classifications are given in Fig. 2.

CATEGORY	NUMERICAL EQUIVALENT	MEAN SPACING OF BREAKS OR DIAMETER OF FRAGMENTS (in.)	QUALITY DESCRIPTIONS
A-	1		Mostly fault gouge with/without minor rock fragments
A	2	<½	Gouge and crushed rock
A+	3		Crushed rock with/without minor gouge
B-	4		Crushed rock - no gouge
B	5	½ to 2	Crushed rock - diameter of pieces <2 in.
B+	6		Broken rock - fracture spacing <2 in.
C-	7		Mean spacing 2 to 3 in.
C	8	2 - 4	Mean spacing 3 in.
C+	9		Mean spacing 3 to 4 in.
D-	10		Mean spacing 4 to 6 in.
D	11	4 - 8	Mean spacing 6 in.
D+	12		Mean spacing 6 to 8 in.
E-	13		Mean spacing 8 to 12 in.
E	14	>8	Mean spacing 12 to 14 in.
E+	15		Mean spacing >24 in.

NOTE: Care should be taken to identify all fault/shear zones (Category A). However, for other Degrees of Breakage, the category should be averaged over the length of the core run.

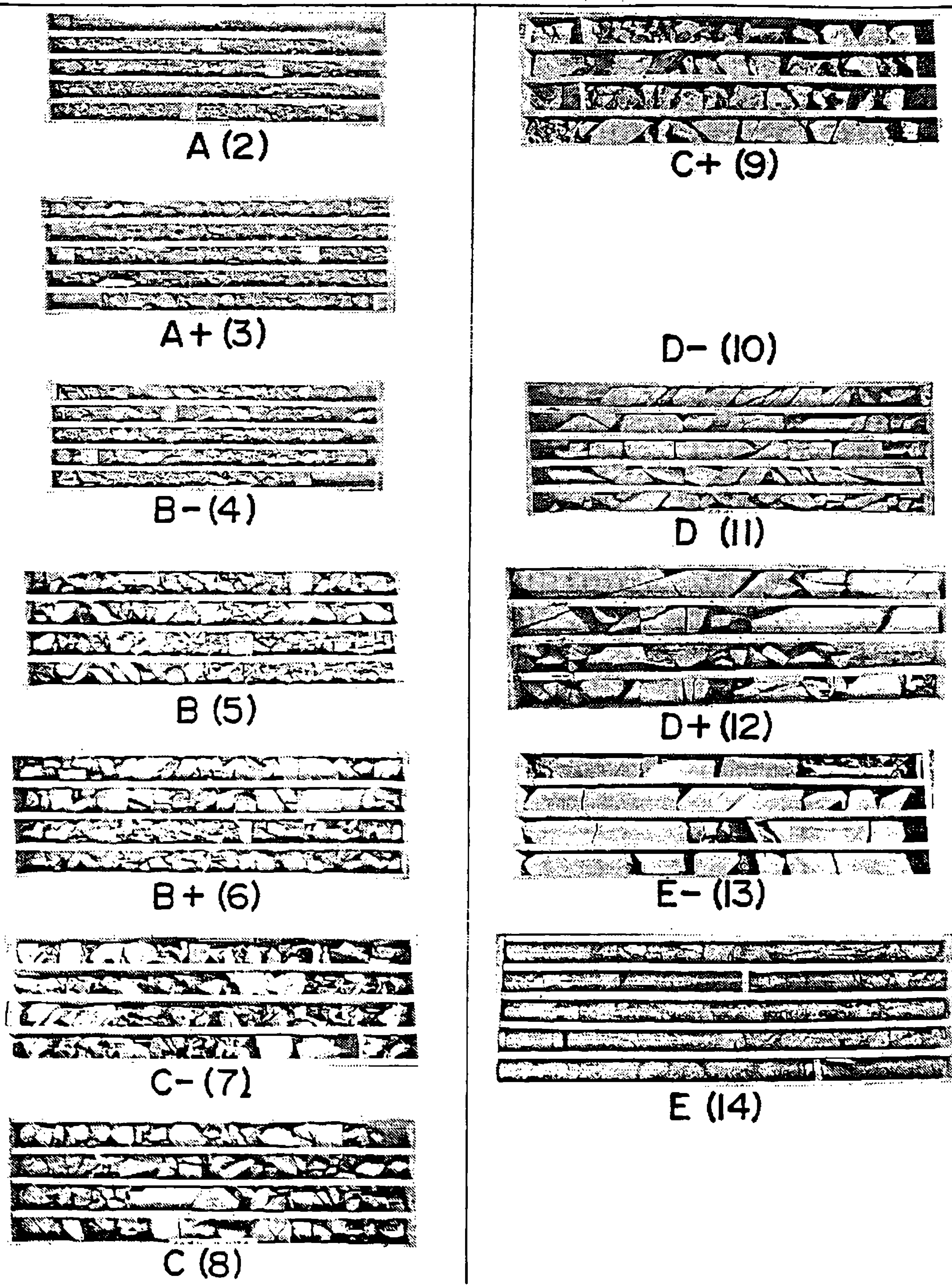


FIG. 2 DEGREE OF BREAKAGE CLASSIFICATION

5. DEGREE OF WEATHERING

The degree of weathering or oxidation of the rock core is used to define the upper boundary of unweathered bedrock and to delineate faults and other zones of intense weathering. The degree of weathering is estimated visually to give a qualitative feel for this parameter. The classification for degree of weathering is as follows:

A - Residual Soil - original fabric destroyed.
B - Completely Weathered - original fabric and relict structures remain, but rock is decomposed and friable.
C - Highly Weathered - rock is discoloured and strength is significantly reduced by weathering.
D - Moderately Weathered - rock is discoloured, but rock strength only slightly affected, discontinuities weathered.
E - Slightly weathered - rock strength unchanged - weathering on joints only.
F - Fresh and unweathered.

6. CORE SIZE

Core size has a direct effect on the quality of core recovered. It is generally recognized that larger diameter core will give better core recovery and a better sample of the geological structures. Accordingly, a record of the core size is kept in conjunction with the core competency study to consider these aspects.

7. JOINT FREQUENCY

The number of natural joints or fractures in each core run is used to calculate the joint frequency. In sedimentary rocks, the number of bedding joints/m and number of cross joints/m are recorded separately. Frequency of drill induced breaks or fractures in the core may also be recorded as an index for assessment purposes.

PROJECT ______________ DRILLHOLE NO. ________ COORDINATES: N ________ DATE ________ 19__

LOCATION ______________ HOLE SIZE ________ E ________ PAGE __ of __

LOGGER ______________ INCLINATION ________ ELEVATION ________

PITEAU & ASSOCIATES
GEOTECHNICAL CONSULTANTS
VANCOUVER CALGARY

GEOTECHNICAL CORE LOG

DEPTH (TO)	LENGTH OF RUN	CORE RECOVERY		RQD		HARDNESS	DEGREE OF BREAKAGE		DEGREE OF WEATHERING	ROCK TYPE	BEDDING DIP		BEDDING JOINTS		CROSS JOINTS		COMMENTS
		LENGTH	%	LENGTH	%		CATEGORY	NO.			DEPTH	ANGLE	NO.	FREQ.	NO.	FREQ.	

Fig. 1. Typical rock mechanics core log.

APPENDIX C

FALLING HEAD TEST PROCEDURE

FIELD PROCEDURE FOR PERFORMING FALLING OR RISING HEAD TESTS IN COMPLETED STANDPIPE PIEZOMETERS

D R PITEAU & ASSOCIATES LIMITED

Piteau Associates - Procedure Guide No. 9

FIELD PROCEDURE FOR PERFORMING FALLING OR RISING HEAD TESTS IN COMPLETED STANDPIPE PIEZOMETERS

1. OBJECTIVE: To obtain a value of hydraulic conductivity for a formation in which a piezometer tip has been sealed.

2. METHOD: After a static water level has been measured, water is added to or withdrawn from a standpipe piezometer. Depths to water are measured at appropriate time intervals and a record of the depths and times is kept. A hydraulic conductivity value is then calculated using the recorded data and a simple formula.

3. MATERIALS:
   - Piteau Associates field sheets PA4, PA5 and PA6
   - an electric water sounder
   - a watch with a second hand
   - a small container of water
   - for flowing piezometers, or piezometers with very shallow depths to static water level, a pump, airline tube or bar for sucking or displacing water from the piezometer to allow a rising head test to be performed

4. PROCEDURE:

4.1 The pertinent data is entered on form PA5. This includes diameter of the standpipe, depth of the piezometer pocket, length of piezometer pocket, diameter of drillhole and a description of the formation over the length of the piezometer pocket.

4.2 The static water level is measured. If the depth is greater than 1 or 2 metres, a falling head test can be run. If the depth is shallow, a rising head test should be performed. If the piezometer is flowing, a pressure gauge should be affixed to the top to determine what the static pressure head is. The static head is recorded on sheet PA5.

4.3 To start a falling head test, 1 to 2 litres of water are added to a 19 mm standpipe. To start a rising head test, water is sucked fropm the pipe using a small tube, blown from the pipe using compressed gas and a tube or displaced from the pipe by forcing a bar down the pipe. Ideally, the water level should be raised or lowered to create an upset head of about 3 to 10 metres measured from the original static level.

4.4 After the water has been added or displaced, a measurement of the water level is made and the time at which it is measured is called time = 0. Subsequent depths are measured and recorded along with the times at which the measurement was performed. This data is entered on sheet PA5. Ideally, about 10 measurements should be made at approximately equal changes in depth. However, in practice a minimum of 3 readings could be used and sometimes up to 25 readings is justified.

4.5 The length of time between each reading will depend upon the permeability of the rock. In order to obtain reliable results, the test should be monitored until the excess head (the difference between the static head and the measured head) has dropped to 37% of the initial excess head. Where the rock is very permeable, this may only take a few seconds and it is possible that only a few measurements may be obtained. Where the rock is very impermeable, it could take weeks for a 63% drop in excess head to occur. The following table provides approximate times for a 63% drop in excess head in 19 mm piezometers installed in various hole sizes:

TIMES TO ACHIEVE 63% DROP IN EXCESS HEAD IN VARIOUS HOLE SIZES

HYDRAULIC COND. (m/s)	BQ (60 mm)	NQ (76 mm)	HQ (96 mm)	6 INCH (150 mm)
10^{-6}*	30 sec	38 sec	36 sec	33 sec
10^{-8}	65 min	63 min	61 min	55 min
10^{-10}	108 hr.	106 hr.	100 hr.	92 hr.
10^{-12}	450 days	440 days	420 days	383 days

* Note: Tests in rock or sediments more permeable than this value, cannot be performed using this procedure.

In some cases it may be impractical to monitor the test for the required length of time, however, the monitoring should continue until at least a 63% head drop has been achieved whenever possible.

4.6 Once the data has been collected, the calculations are carried out as per sheet PA6. If the units used are seconds and cm, the resulting hydraulic conductivity will be in cm/sec. In choosing a value for anistropy, any structure in the bedrock should be considered. Generally this value will be .1, 1 or 10 depending on the presence and orientation of any bedding or fracturing.

The plot on sheet PA6 should form a straight line. If it doesn't, an error in the static head or a leaky seal is indicated. If the static head is in error, new values for the static head can be estimated and a straighter line can be achieved by trial and error.

4.7 The particulars of the hydraulic conductivity tests performed in standpipes in each hole are then recorded on a summary sheet (PA4).

SUMMARY OF INFORMATION ON PERMEABILITY TESTS

BOREHOLE NO. ____________

Project No.: ________________ Date: from ____/____/____ to ____/____/____
day mo yr day mo yr

Project Name: ________________

Client: ________________ Tested by ______________ ____________________

Location: ________________

Borehole Drilled by: _________________________

Type of Test: Falling head ☐ Rising head ☐ Both types ☐

Hole Completion: Standpipe piez-meter ☐
Inflatable packer ☐
Open hole ☐

Packer arrangement in hole:

Through-the-bit ☐
Double packer ☐
Single packer ☐

Hole Depth: _______ m (_______ ft)

Hole Diameter:

Depth Interval	Diameter
_________ m (______ ft)(1)	__________ mm (_________ ins)
_________ m (______ ft)	__________ mm (_________ ins)
_________ M (______ ft)	__________ mm (_________ ins)

No. of tests ran in hole: __________________

Summary of Hydraulic Conductivity Values: ______________

Test No	Depth Interval		Depth to Water ( )	Elevation (asl)		Description of Material	Hydraulic Conductivity ( )
	Depth Range	Length ( )		Pocket metres (feet)	Piezometric metres (feet)		

Note: (2) Elevation is for mid point of test interval
(1) fill in either metric in English Units

Permeability Test Sheet For Test No.____

Borehole No.

Project No.: Tested Interval ________ to ________ metres

Location: ________ to ________ feet

System Data

Hole diameter _____ cm (____ins) Length tested _________ m (________ft)

Hole Inclination _______ ° from horizontal Azimuth ______ °

Hole depth _______m (_______ft)

Ground Elev _______m (_______ft)

Height of measuring point above ground _______m (______ft)

Bubbler depth _______m(_______ft)

Height of water added _______m (______ft)

Volume of water added _______m (______ft)

Diameter of stem _______mm (_______ins)

Soil/Rock Description

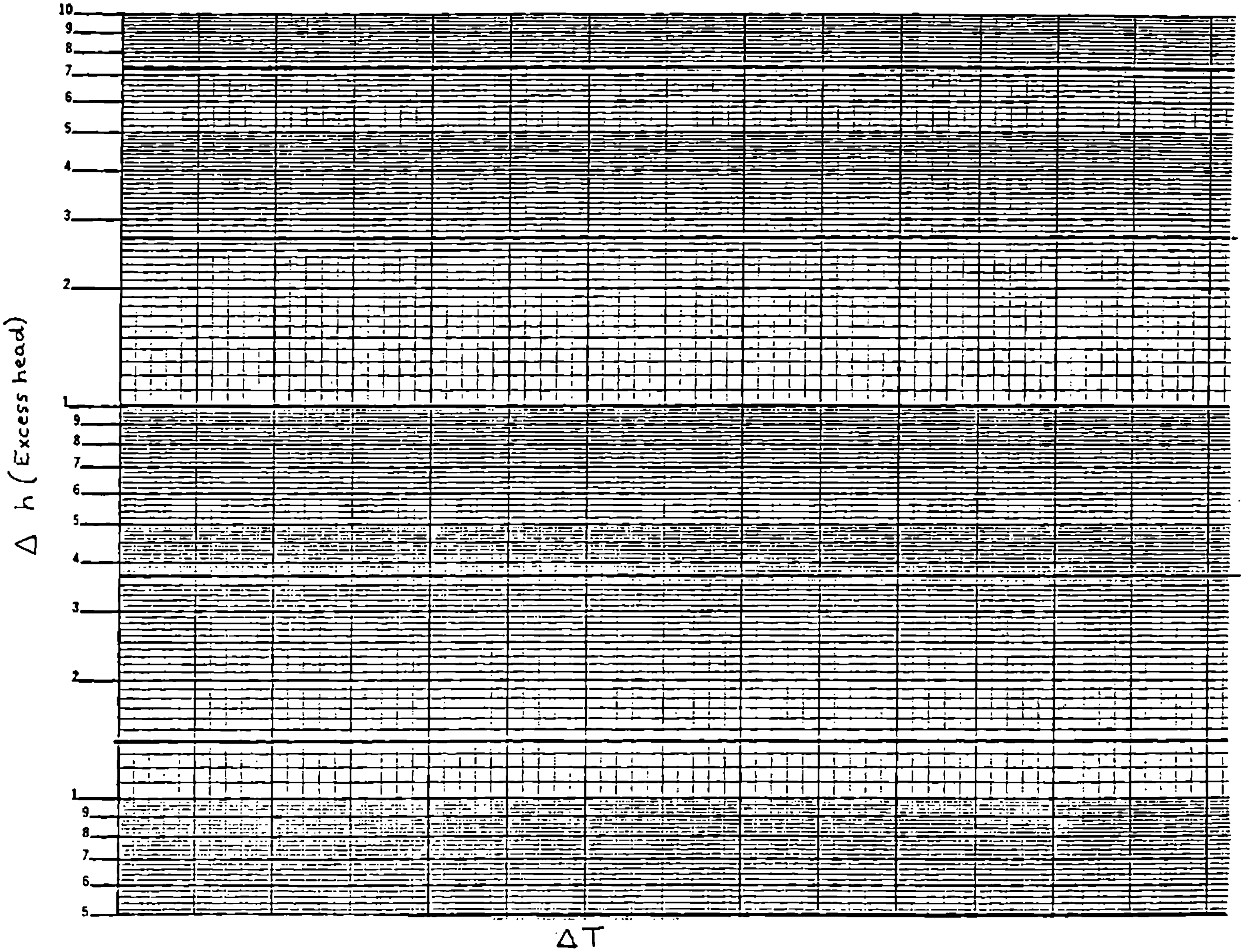
Estimated Static Head _________ (depth) _______(elev.)

Time	Elapsed Time	Depth	Elev.	Excess Head	Time	Elapsed Time	Depth	Elev.	Excess Head

PA-6 Permeability Test Sheet for Test No ____ Sheet __of__

Test Interval ______ to ______ ___(metres)

______ to __________(feet)



CALCULATIONS: $K = \frac{d^2}{8LTi} \; Ln \left(\frac{2\sqrt{M}}{D} \; xL \right)$

where: D = Hole diameter ___cm (____in)

d = Tube diameter ___cm (____in)

$M = K_H/K_V$ = ANISOTROPY

K = Hydraulic Conductivity

L = Test Length ____cm (____in)

Ti = Time Interval for Excess Head (h) to Decay by 63%

= Time for Excess to Drop from One Bold Line to the Next Bold Line

= _______ seconds