Golder Associates
CONSULTING GEOTECHNICAL ENGINEERS

A REPORT
to
CYPRUS ANVIL MINING
Providing

FINAL DESIGN RECOMMENDATIONS
For The
DOWN VALLEY TAILINGS DISPOSAL PROJECT
Faro Yukon Territory

VOLUME ONE
OF
THREE VOLUMES

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June, 1980

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June 6, 1980
File No. 792-2025

Cyprus Anvil Mining Corporation
P.O. Box 1000
Faro, Yukon Territory
Y0B 1K0

Attn: Mr. N.G. Cornish,
Superintendent Environmental Control

Dear Sir:

Re: Down Valley Tailings Containment Facility

We take pleasure in providing with this letter 12 copies of a three-volume report presenting our final design for the Cyprus Anvil Down Valley Tailings project. Distribution has been noted on the title page.

We wish to acknowledge the substantial help, direction, constructive criticism and cooperation that you and other members of the Cyprus Anvil team have provided throughout the study.

We look forward to approval of the Down Valley Project by the appropriate governmental agencies and to participation in the construction of this project.

Respectfully submitted,
GOLDER GEOTECHNICAL CONSULTANTS LTD.

PROFESSIONAL ENGINEER
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1.0 SUMMARY

The Down Valley Tailings Project was conceived in May, 1979 and during the past year detailed study has been undertaken in anticipation of construction beginning in October, 1980 and carrying through to the fall of 1981. Golder Associates has conducted a number of field investigations and office studies on behalf of Cyprus Anvil with the purpose of developing an appropriate design for the project. This report provides Golder Associates' final design recommendations.

The Down Valley Tailings Project consists of a Diversion Canal to bypass Rose Creek around the tailings deposits which will fill the bottom of the Rose Creek valley, a Cross Valley Dam to retain spent tailings water for a 60 day retention time, and a cross valley Intermediate Dam which will be constructed in stages to retain tailings in the valley bottom. Additional project elements of lesser importance include a North Valley Wall Interceptor to prevent uncontaminated drainage runoff from entering the tailings system, a Diversion Dam which is necessary to achieve creek diversion from the existing canal to the new canal and the spillways and decants necessary to ensure the integrity of the cross valley dams.

The proposed Rose Creek Diversion Canal has a design hydraulic capacity equivalent to a 50 year return period flood and contingency capacity for the 500 year return period flood, the latter assuming no freeboard on the engineered section. In addition, a low flow pilot channel is provided to diminish the tendancy for winter glaciation and low flow meandering. The proposed 3.6 km alignment follows along the south valley wall of the Rose Creek valley. Because the canal channel grade of 0.19 percent is flatter than the natural valley bottom grade of 2 percent, tailings storage capacity is generated below the elevation of the diversion canal. At the location of the Cross Valley Dam the Diversion Canal invert is about 25 m above the valley bottom. At this point the return of the diverted creek to its natural channel is initiated using a

rock drop weir system similar in construction to that presently operating in the existing diversion works.

The canal excavation will traverse a variety of materials ranging from bedrock to glacial till and creek alluvium. Much of the route is through permafrost and, because construction will induce thawing of these sometimes ice rich materials, a filtering and insulating gravel layer will be required. It will be necessary to construct the canal during the winter months to avoid thaw degradation during construction. Total earthworks volume for the canal is estimated to be 950,000 m^3 plus some 84,000 m^3 of erosion protective rock work.

Both the Cross Valley and the Intermediate Dams are to be of zoned earthfill and the former will have an upstream blanket attached to the glacial till core to reduce foundation seepage to an acceptable level. The alignment of the Cross Valley Dam has been chosen to minimize foundation interaction with a permafrost area but some excavation of unsuitable and frozen materilas will still be necessary. Seepage reduction at the Intermediate Dam will involve excavation through the terrace gravels to creek level and use of a sloping glacial till blanket on this borrow pit face. Combined Stage I earthworks for these dams is approximately 870,000 m^3.

Emergency spillways are provided for each dam as an insurance against overtopping of the dams in the event of failure of the diversion canal system or the occurrence of flows in excess of canal capacity. The design capacity of these spillways is for a 50 year return period Rose Creek flood flow and their intended function is to protect and maintain the crest of the dams. Decant channels are located adjacent to the spillways to pass on a routine basis the tailings spigot water flows.

The Diversion Dam is a relatively small structure required to divert the creek from its existing route into the new diversion canal. It is intended that this structure will be built in the wet using a rockfill

starter dyke with progressively finer materials being dumped upstream of the rockfill.

The North Valley Wall interceptor consists of a simple unarmoured channel cut in the generally unfrozen till deposits of the North Valley wall. Hydraulically the required cross section is based on a 50 year design return period.

This report discusses the various stages and procedures of the investigation, the findings of the investigation and studies, and the recommended designs for construction. A summary of project construction quantities is provided in Chapter 8. A general project summary is presented below.

GENERAL PROJECT SUMMARY

Primary Construction Elements

Length of Diversion Canal:	3600 m
Length of Cross Valley Dam:	500 m
Height of Cross Valley Dam:	17 m (max)
Length of Intermediate Dam:	500 m
Height of Intermediate Dam:	17 m (max)
Ultimate Height of Intermediate Dam:	31 m (max)

Retention Pond

Area of Retention Pond:	32×10^4 m^2
Volume of Retention Pond:	1.4×10^6 m^3

Tailings Storage

Area of Tailings Storage:	128×10^4 m^2
Approximate Stage I Volume:	11×10^6 m^3
Ultimate Volume:	20×10^6 m^3

(based on 2 percent tailings slope)

2.0 INTRODUCTION

Cyprus Anvil Mining Corporation plans to expand the tailings facility at their Faro, Yukon lead-zinc mine (for location see Figure 2-1) to provide storage capacity related to milling of recently acquired reserves. The proposed tailings storage expansion is referred to as the "Down Valley Project".

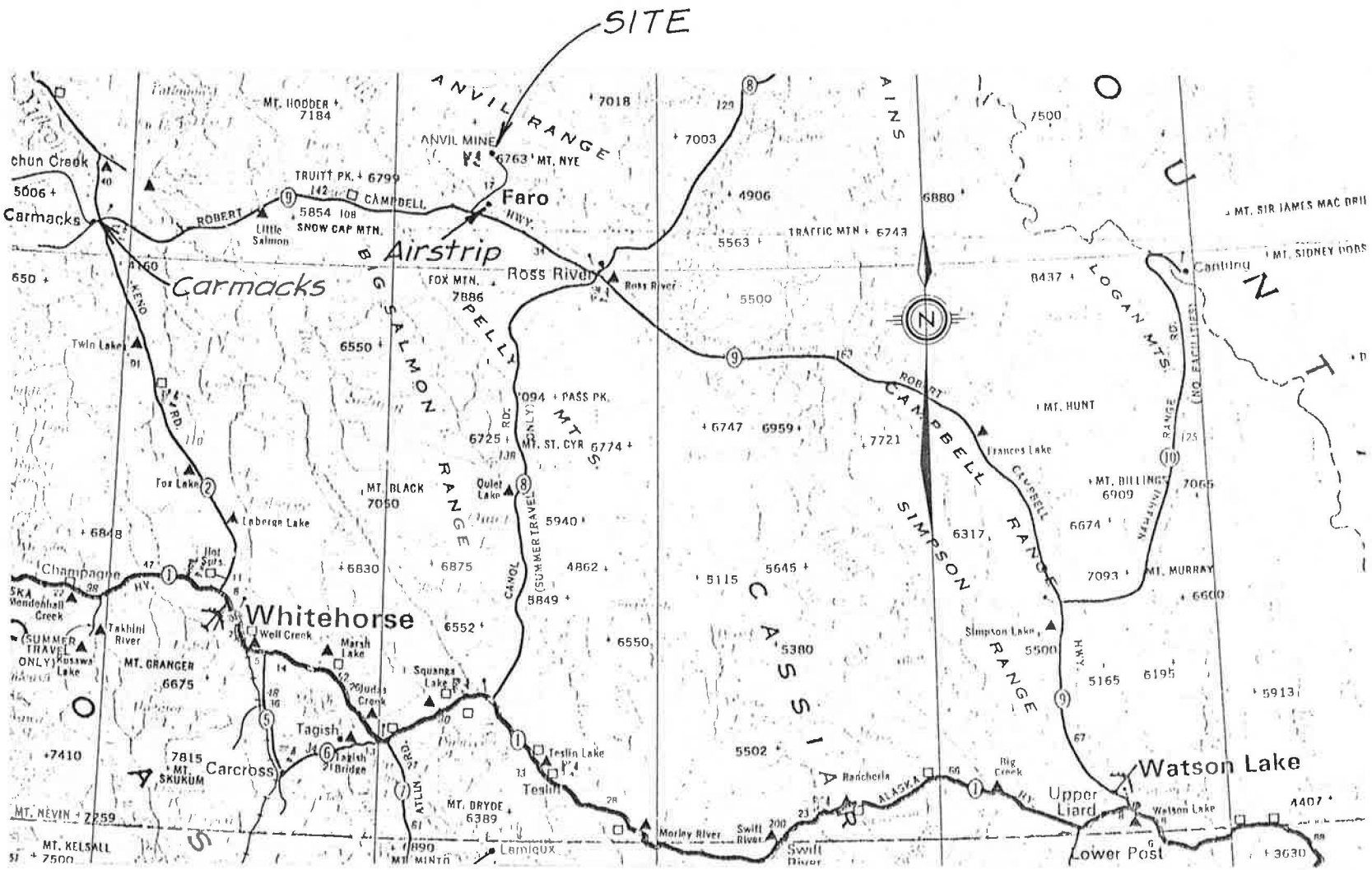
The Down Valley Project utilizes the Rose Creek Valley downstream of the existing tailings facility and as such is an extension to the existing facility which has been in use since plant operations began in 1969. The proposed expansion is expected to provide storage for about 20 million m^3 or about 40 million tonnes of tailings, depending on deposited tailings density in place which in turn depends on spigotting practice. This available volume does not include an additional 4 million m^3 which could be included if tailings were to be stored to the crest of the canal dyke; this will be possible if the seepage through and beneath the canal dyke is within acceptable limits. The actual tonnage which may be stored in the available volume depends on spigotting practice, mill grind, and ore source. Dry solids densities observed at the Lakefield Research test grind tailings pond ranged from 2.4 tonnes/m^3 for above water deposition down to 0.9 tonnes/m^3 (measured indirectly) for below water deposition. These observations suggest a need to understand and control the process of tailings deposition to optimize available storage.

As an outgrowth of its standing involvement in Cyprus Anvil's tailings disposal since 1973, Golder Associates' terms of reference for the Down Valley Project describe full involvement with respect to the design and construction supervision of the various project elements in accordance with the design objectives defined by Cyprus Anvil. Primary project elements consist of a 3.6 km long "diversion canal" along the south valley wall to isolate Rose Creek flow from the tailings deposits, an "intermediate" cross valley dam to retain spigotted tailings and a second "cross valley" dam to provide a 60 day retention time for spent tailings water. Secondary elements of the proposed scheme include a

relatively small diversion dam, spillways and decants across the cross valley dams, and a north valley wall interceptor to collect uncontaminated surface flows which would otherwise flow down the north valley wall and enter the tailings disposal area. All project elements are illustrated on Figure 2-2.

Project No. 792-2025 Drawn RK Reviewed [illegible] Date 18 Sep 79

LOCATION PLAN



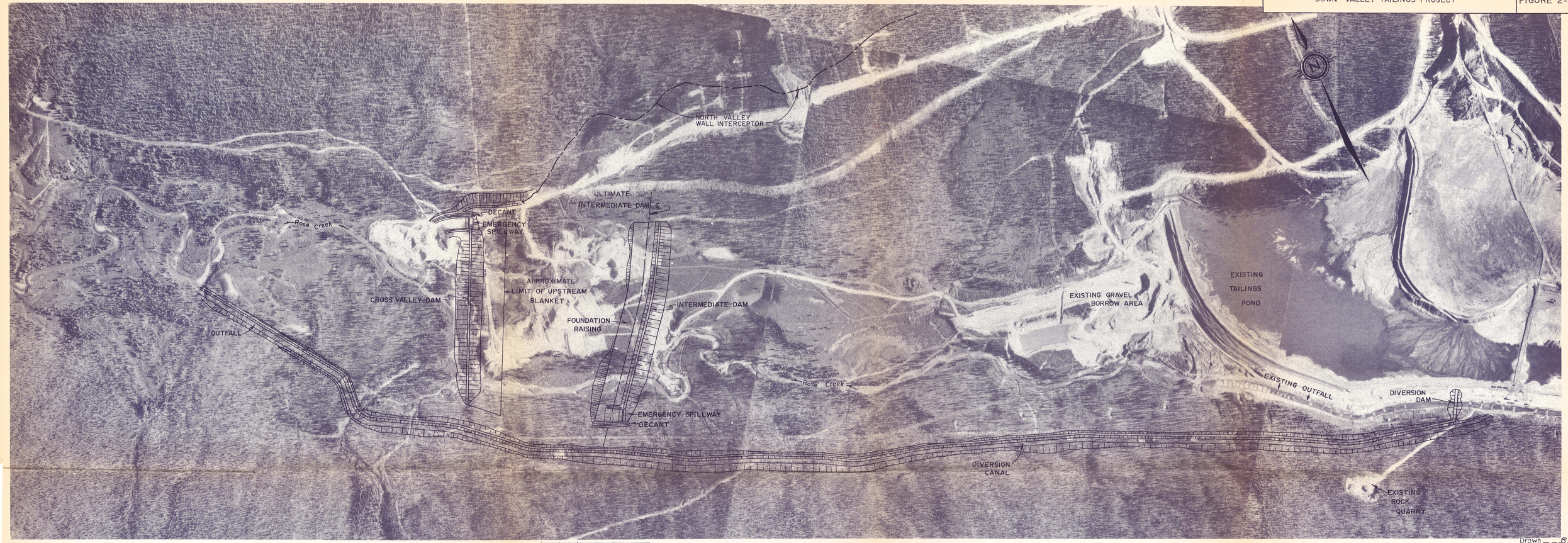
SCALE: 1 in. to 40 mi.

Figure 2-1

Golder Associates

PHOTOMOSAIC SHOWING PROPOSED DOWN VALLEY TAILINGS PROJECT

FIGURE 2-2



REFERENCE: AERIAL PHOTOGRAPHS OBTAINED FOR CYPRUS ANVIL MINING CORPORATION BY NORTHWEST SURVEY CORPORATION LTD., AUGUST 15, 1979.

100 50 0 100 200 Metres

SCALE: 1:5000

Golder Associates

Drawn R.W.
Rev'd J.D.G.
Date MAY 1980

3.0 PROJECT SETTING

The project is located in the Rose Creek Valley near Faro, Yukon Territory. Rose Creek is an underfit stream which meanders its way along the valley bottom eventually entering Anvil Creek which in turn flows into the Pelly River. The bottom of the valley is about 400 m wide and it has been infilled by gravel deposits up to 40 m thick. Rose Creek winter flows are about 0.3 m^3/sec. whereas summer flows are typically 3 m^3/sec. Variable peak spring flows of up to 33 m^3/sec. occur during the late May, June, or early July periods. Detailed flow data may be found in Appendix I.

Climate in the area of interest is somewhat influenced by the mountain coastal factors. Temperatures at Anvil minesite on the mountain are often warmer than at Faro or in the valley bottoms during winter as a result of inversion phenomena. In December, 1979, temperatures in the Rose Creek valley bottom were often observed to be about 5°C lower than mine temperatures. The mean annual air temperature at the minesite is about -3.7°C. The coldest month is January with a mean daily temperature of about -20.8°C. The warmest month is July with a mean daily temperature of 11.1°C. Mean annual precipitation is 410 mm of which 211 mm is snowfall. Average daily temperatures are estimated to be below 0°C between September 5 and May 2. Supplementary climatic data is included in the Appendix II.

Discontinuous permafrost is present in the valley bottom and on the valley walls. Most of the south valley wall (see Figure 2-2 for orientation) consists of frozen coarse glacial till whereas much of the north valley wall is unfrozen similar material. Isolated pockets of occasionally ice rich frozen sands and silts may be found on the valley floor. The permafrost is characteristically "warm" with temperatures in the 0° to -1°C range.

The valley walls, particularly the south valley wall, are mantled with a moss cover about 0.3 m thick. Spruce trees range in diameter up to a maximum of about 0.4 m with typical sizes being about 0.2 m or less.

Tree heights are typically less than 10 m. Tree densities on the valley walls range up to 220 trees per hectare and are typically about 100 trees per hectare. On the valley floor barren areas occur where the gravel terrace deposits adjacent to Rose Creek are well drained.

4.0 DESIGN OBJECTIVES

In addition to the requirement for design of stable, economical and practical project elements, consultations with Cyprus Anvil have defined the following design objectives:

(i) that all Down Valley Project structures be designed to resist earthquake, it being assumed that all other structures not constructed as a part of the tailings project such as the fresh water impoundment dam will remain stable;

(ii) that the diversion canal water level be above that of the tailings storage pond;

(iii) that the drop weir energy dissipation system which has been operating successfully in the outfall of the existing diversion canal and has previously been approved by the Yukon Territorial Water Board and Department of Fisheries be adopted in principle for the new diversion canal outfall;

(iv) that the seepage losses from the canal into the new tailings pond should not exceed the lesser of 0.15 m^3/sec. (2000 i.g.p.m.) or 10 percent of the Rose Creek flow at the inlet to the facility (flows greater than the 50 year return period flood being excluded);

(v) that the alignment of the canal be chosen such that a 50 year return period flood of Rose Creek would be contained within the cut banks of the canal, the existing ground surface being used as a reference;

(vi) that the design capacity of the canal would be for a 50 year return period Rose Creek flood with contingency capacity for a 500 year return period flood;

(vii) that the outfall portion of the canal be designed for the same flows as the canal proper;

(viii) that the Cross Valley Dam be constructed to an ultimate elevation such that 60 days of retention time or approximately 1.4 million m^3 of storage would be available for chemical aging of tailings water;

(ix) that the seepage through the Intermediate Dam and its foundation should not exceed 0.22 m^3/sec. (3000 i.g.p.m.) to impound water behind this dam and therefore eliminate winter glaciation.

(x) that the seepage through the Cross Valley Dam and its foundation should not exceed 0.15 m^3/sec. (2000 i.g.p.m.) to ensure adequate retention time and permit some water reclaim for plant use.

5.0 STUDIES AND INVESTIGATIONS

5.1 General

The Down Valley Project was conceptually outlined by Mr. N.G. Cornish of Cyprus Anvil in a report entitled "A New Tailing Containment Proposal" dated May 15, 1979. Review of this document and a site inspection on June 1, 1979 by Messrs. H.G. Gilchrist and D.C. Goodall lead to the submission to Cyprus Anvil of a letter proposal dated June 15, 1979 by Golder Associates. This proposal presented a conceptual design and recommendations for field investigations pertinent to more detailed design. It was submitted to the Territorial Water Board at a meeting in Whitehorse on July 20, 1979 by Mr. N.G. Cornish of Cyprus Anvil and by Messrs. Gilchrist and Goodall of Golder Associates.

Data gathering and field investigation was initiated on July 25, 1979 according to the recommendations of the above-mentioned proposals and under the field supervision of Mr. D.C. Goodall of Golder Associates. This primary investigation was completed by about September 1, 1979. Supplementary field investigations were conducted in December, 1979 and February, 1980 to investigate the foundation of the "intermediate" dam which was not examined in the previous fieldwork, to investigate borrow potential and to reduce uncertainties concerning stratigraphy along the proposed canal route. In addition, a full scale canal test section was constructed in March 1980 to test Golder Associates' recommended canal design in view of the difficulties inherent to prediction of permafrost behaviour and the general lack of documented precedents or experience for such construction.

All field investigation sites and elements are located on the site topographic plan, Figure 5-1.

5.2 July-August, 1979 Field Investigation

Activities included surveying, drilling, test pit excavation, geophysical studies, both surface and downhole, and aerial photography. All activities were conducted under the supervision of Mr. D.C. Goodall of

Golder Associates except the aerial photography which was performed by Northwest Survey Corporation International Ltd. of Edmonton as part of their regional photography for Cyprus Anvil.

Surveying was conducted by Underhill Engineering Ltd. of Whitehorse and by Cyprus Anvil survey staff. Control was established by Underhill Engineering using spirit levelling and electronic distance measuring equipment and was based on the metric mine grid system coordinates provided by Cyprus Anvil for monument 78-17. Elevations are based on the elevation of the northwest corner of the floor slab inside the Cyprus Anvil pumphouse, this also being provided by Cyprus Anvil. Underhill Engineering also performed the following work:

- established baselines for the Cross Valley Dam and the Diversion Canal.
- obtained cross-sections for the Diversion Canal at about 100 m stations.
- obtained cross-sections for the Cross Valley Dam at 60 m stations.
- obtained cross-sections of the proposed core borrow pit (as defined at that time) at 60 m stations.

The Cyprus Anvil survey crew determined the project coordinates for all boreholes and test pits (except most of those along the canal route which were located conveniently by Underhill Engineering while obtaining cross-section information). The mine survey crew also staked all seismic investigation lines on 10 m intervals and determined the elevations of these stakes.

Most of the drilling was performed using a Failing 1000 rotary rig owned and operated by Midnight Sun Drilling Co. Ltd. of Whitehorse. Boreholes in the unfrozen valley bottom deposits were normally drilled

with water and/or drilling mud as a circulation fluid whereas many of the boreholes in the frozen till deposits forming the valley wall were most successfully drilled using air. An air rotary "Shram" rig also owned and operated by Midnight Sun Drilling was mobilized to drill Borehole No. 79-20 to the required depth of investigation because the caving of frozen gravels into the borehole precluded use of the wet rotary rig.

A total of 27 borehole investigation sites were drilled. Relatively deep boreholes to a maximum depth of 41 m were drilled in the valley bottom or in the Cross Valley Dam abutments to determine bedrock topography. Pneumatic piezometers, standpipe piezometers and/or thermistors were installed in most boreholes. All boreholes were grouted with a cement/bentonite grout or backfilled with selected sand and gravel with intermittent "Peltonite" seals. Details of all boreholes may be found on the Borehole Schedule, Table 5-1 or on the Record of Borehole sheets in Appendix III (July-August boreholes include Borehole Nos. 79-1 to 79-26 inclusive).

Sampling was conducted according to the purposes of the various boreholes and the variation of stratigraphy. Samples were obtained using the standard penetration test procedure with both 50 and 75 mm split spoons, the latter being more suitable for recovery of sufficient sample quantity. A few Shelby tube samples were obtained from the finer grained deposits. Cuttings samples from the air or mud return were examined regularly but were generally discarded. All recovered samples, as designated on the Record of Borehole sheets in Appendix III, were forwarded to Golder Associates' Calgary office for laboratory testing and storage.

Test pits were excavated using ripper equipped D-8 bulldozers owned and operated by General Enterprises Ltd. of Whitehorse. Most test pits could be excavated to the maximum practical depth for this method of excavation; exceptions to this occurred at the following test pits which were terminated for the reasons noted (for location see Figure 5-1):

Test Pit No.	Reason for Termination
0+195	Seepage
0+560	Rock
0+760	Rock
1+390	Seepage
2+155	Rock
79-8	Rock
79-9	Rock
79-12	Seepage
79-22	Rock

Access to many of the test pits along the canal baseline was difficult as a result of the thawing of near-surface ice-rich permafrost. Test pits were the only means of direct investigation in many areas of the canal where drill rig access was not possible. Commonly, the bulldozers were bogged down locally in thawed areas near small creeks. The area in the vicinity of the test excavation was inaccessible to the bulldozers after the initial survey cutlines were in place. These site features were instrumental in the decision to construct the proposed diversion canal during freezing conditions.

Up to two 20 kg bag samples were recovered from each test pit. These samples are designated on the Record of Test Pit sheets in Appendix III and were forwarded to Golder Associates' Calgary office for appropriate testing and storage. Similar samples were also taken from sidehill cuts and old borrow pits in areas of interest.

A total of 25 test pits were excavated during the July/August investigation. These include Test Pit Nos. 1 to 12 inclusive and and all test pits along the canal except the Test Excavation, T.P. 1+900 to 1+930 and T.P. -0+060 to -0+080. Details pertinent to these test pits may be found on Tables 5-2 and 5-3.

Seismic refraction studies were conducted along the baselines of the Cross Valley Dam and the canal and on selected cross-sections to these baselines. All seismic studies were conducted by Atmex Geophysics Inc. of Montreal, Quebec using a 24 channel recorder with geophones spaced at 10 m intervals. In addition, downhole seismic, nuclear and resistivity logging were performed on Borehole Nos. 79-15, 79-16, 79-18, 79-19, 79-20, and 79-21. The results of this downhole logging are reported in Appendix III.

Aerial photographs pertaining to the area of interest were taken on August 15, 1979 by Northwest Survey at the time that the field investigation was nearing completion. The resulting photographs, taken from a height above the ground of about 1500 m, are at a scale of about 1:10,000. These photographs were used by Northwest Survey to produce the photogrammetric 3 m contour map shown on Figure 5-1 at a scale of 1:2,000.

5.3 November - December, 1979 Field Investigation

The Intermediate Dam was introduced formally into the Down Valley Project in Golder Associates' October 1979 reports at the request of Mr. N.G. Cornish of Cyprus Anvil. It was decided that an investigation of the foundation area of the dam would be appropriate particularly in the vicinity of Rose Creek.

Seismic refraction and electromagnetic induction surveys were conducted by Geo-Physi-Con Co. Ltd. of Calgary in mid-November, 1979 along lines designated H-H and I-I on Figure 5-1. A 12-channel recorder was used with geophones being spaced at 20 m intervals. The purpose of line I-I was to investigate the foundation of the Intermediate Dam; line H-H was located to examine possible dam foundation conditions upstream of the Cross Valley Dam and the feasibility of a slurry trench seepage cutoff.

In early December a borehole and test pit program was completed under the supervision of Mr. D.C. Goodall of Golder Associates. A total of 8 boreholes were drilled using Midnight Sun's Shram air rotary rig. These

included Borehole Nos. 27 to 34 inclusive with all except Borehole No. 34 being drilled in the Intermediate Dam foundation area. Borehole No. 34 was drilled in the Cross Valley Dam area to assess the hydraulic characteristics of the Cross Valley Dam foundation. All boreholes are located on Figure 5-1 and pertinent details may be found on Table 5-1. Piezometers were installed in most boreholes. One thermistor was successfully installed in Borehole No. 79-29.

Cuttings samples were normally taken at 1.5 m intervals. In addition, some split spoon samples were taken from Borehole No. 79-33. All soil samples were forwarded to Golder Associate's Calgary office. Several samples of the water return gathered during drilling were provided to Cyprus Anvil for analysis. Soil and water sampling details may be found on the Record of Borehole sheets in Appendix III.

The December drilling program had to be terminated prematurely because of extremely cold weather and the resulting inoperative condition of the drill rig hydraulic system.

Ten test pits were excavated in December using General Enterprises' ripper-equipped D-8 bulldozer to take advantage of substantially improved access to the canal area and to better define the nature and extent of till and gravel borrow areas. In addition, Test Pit No. 79-20 was excavated near Borehole No. 79-18 in the Cross Valley Dam area to investigate the nature of near surface permafrost known to exist at this location.

Several frozen samples were taken from the test pit locations. These were packed in specially constructed, insulated boxes and forwarded to Golder's Calgary officewhere they were preserved in a frozen condition. Test Pit sampling and other details may be found in Appendix III and on Tables 5-2 and 5-3. The December series of test pits includes Test Pit Nos. 79-13 to 79-22.

Note that Test Pit Nos. 21 and 22 were excavated along the canal and have been designated chainages 1+900 to 1+930 and 0-060 to 0-080 respectively on Table 5-3. All test pits are located on Figure 5-1.

5.4 February, 1980 Field Investigations

The December 1979 field program revealed some unexpected stratigraphy. Bedrock had been encountered at a depth of about 3 m at the location of Borehole No. 79-32 which was located near the canal baseline. This meant that some rock excavation would be required to complete the canal and that seepage losses through disturbed bedrock was a potential concern in this area. In addition, there were a number of specific design questions which were considered by Golder Associates to be of both economic and geotechnical relevance to the project. Foundation preparation was considered to be an important cost item for both the Intermediate and Cross Valley dams and therefore a program of shallow drilling was recommended for these areas. The quality and distribution of till core materials was a concern and therefore exploration beyond the initial limits of the till borrow area was recommended in the vicinity of December Test Pit No. 79-19 where good borrow potential had been found.

These considerations together with the fact that the decision was made not to construct the Down Valley Project in the winter-spring seasons of 1980 prompted the February 1980 investigation. The reasons for the investigation being recommended are not considered to be related to the initial investigation deficiencies but rather to better define construction quantities.

The Down Valley February drilling program consisted of 14 boreholes to a maximum depth of 14.2 m. Many of the boreholes were drilled with the air rotary auxiliary of Midnight Sun's Failing 1000. Very few samples were taken, the objective of the work being to determine stratigraphy and the occurrence of water in the borehole, the latter defining the limit of "dry" excavation which might be done by bulldozers. Standard Penetration tests were conducted at 1.5 m intervals in Borehole No. 80-46 in the Intermediate Dam area to provide additional detailed stratigraphic and

geotechnical information. Standpipes were installed in most boreholes and a thermistor was installed in Borehole No. 80-51. Borehole details are provided on Table 5-1 and on the Record of Borehole sheets in Appendix III (Borehole Nos. 35 to 41 and 46 to 52 inclusive). The locations are shown on Figure 5-1.

Test Pits Nos. 80-23, 80-24 and 80-25, as shown on Figure 5-1, were excavated in a potential core borrow extension area. Some samples were recovered for gradation purposes and forwarded to Golder Associates' Calgary laboratory. Logs of the test pits may be found in Appendix III and general details are on Table 5-2. Test pits were excavated using General Enterprises' D-8 bulldozer and ripper.

All work was carried out under the supervision and inspection of Mr. D.C. Goodall of Golder Associates.

5.5 March 1980 Test Excavation

A full scale test section of the proposed canal was constructed to assess geotechnical performance, construction cost and practicality of the proposed design. In addition useful information was obtained with respect to the definition of the design to the contractor and the nature of the constructed product which will result.

The test excavation was constructed by General Enterprises Ltd. of Whitehorse on an hourly basis. D-8 and D-9 Caterpiller tractors equipped with rippers were used exclusively for initial excavation with Caterpillar 631C motor scrapers being used for removal of material when it became practical to do so. A Dynapac CA 25 PD vibratory pad foot packer was used to compact the materials under freezing condition. (See photographs in Appendix IV). As mentioned previously, the test excavation was constructed in the area (see Figure 5-1, canal chainage approximately 1+850) which was most inaccessible during the summer as a consequence of thawing ice rich ground and the local occurrence of small downhill surface stream flows. Furthermore, the test excavation was located in an area of

relatively steep cross-slope and where the geophysical seismic velocities suggested that relatively ice rich permafrost existed.

The maximum depth of vertical excavation below ground surface was about 8.5 m with the maximum elevation difference between the head of the cut and the base of excavation being about 13 m. The excavation was blanketed with gravel taken from a borrow pit typical of the terrace gravel borrow pits which will be developed as the source of gravel for full scale lining of the canal. Some 20,000 m was excavated and 8,000 m of gravel was backfilled for thermal protection.

The work was supervised by Golder Associates personnel including Mr. D.C. Goodall. Sampling and monitoring of materials was an important aspect of the work. Density, water content and compaction tests were conducted in the field. Layout and survey control was maintained by Underhill Engineering Ltd. Thermistors and pneumatic piezometers were installed along the central cross-section of the cut using Cyprus Anvil's pneumatic percussion drill mounted on a military tracked carrier. Surface movement monitors were installed in some boreholes and located with respect to a control station on the gravel terrace in the valley bottom opposite the test excavation using electronic distance measuring equipment.

A list of sampling details and the location of installations may be found in Appendix V.

Since work was completed Cyprus Anvil and Golder Associates have been jointly monitoring the performance of the test excavation on a weekly basis. This monitoring presently pertains to thermistors but piezometer and movement monitors are intended to be observed when appropriate.

5.6 Laboratory Testing

Water content, gradation, specific gravity, organic content, unit weight and plasticity tests were performed to classify the project materials and their variability.

Specialized testing included drained triaxial strength tests, consolidation and thaw consolidation tests on the Cross Valley Dam foundation materials, permeability of canal till, unit weights of frozen samples and standard Proctor density tests. Most tests were performed using standard procedures.

The permeability test was conducted on minus 19 mm diameter till material compacted at its natural water content in a 102 mm diameter compaction mould using standard Proctor effort. The permeabililty sample was also subjected to several freeze-thaw cycles to estimate the effects of this factor on canal till liner performance.

Frozen unit weights were determined by direct measurement of weight and volume of the sample. Sample volumes ranged from 5000 to 14,000 cm^3 and were determined by the overflow of a garbage pail of antifreeze when the sample was submerged.

5.7 Personnel

For a project of this size personnel from several companies have been involved. The duties and responsibilities of the primary individuals have been as follows:

i) General

- Mr. N.G. Cornish, Superintendent, Environmental Control for Cyprus Anvil, Faro, client contact, liaison and review.
- Mr. H.G. Gilchrist of Golder Associates, Calgary, initial proposal, oversight and direction of the project as well as review.
- Mr. D.C. Goodall of Golder Associates, Calgary, supervision of all project studies and reporting.

ii) Field Studies

- Mr. Bruce McLean of Underhill Engineering Ltd., Whitehorse, surveying party chief.
- Mr. Merv Tull of Midnight Sun Drilling, Whitehorse, driller.

- Mr. Alvin Vance of General Enterprises Ltd., Whitehorse, dozer operator.

- Messrs. Richard Reid and Michael David of Atmex Geophysics Inc., Montreal, geophysical studies.

- Messrs. Colin MacDonald and Pieter Hoekstra of Geo-Physi-Con, Calgary, geophysical studies.

- Mr. Jim Masson of General Enterprises, Whitehorse, site superintendent for test excavation.

iii) Office Studies

- Mr. Wim Veldman of Hydrocon Engineering (Continental) Ltd., Calgary, hydrologic and hydraulic design (including on site inspection).

- Dr. Bryan Fletcher of Golder Associates, Vancouver, geothermal and stability studies for the diversion canal.

- Mr. Bill Hodge of Golder Associates, Cross Valley Dam design.

- Dr. Peter Byrne of the University of British Columbia, earthquake stability consultation.

Reports were received from each of the above and from Atmex Geophysics and Geo-Physi-Con. These reports are retained on file by Golder Associates, Calgary and appended to this report as appropriate.

Other staff include:

- Mr. Shawn McKeown of Golder Associates, Calgary, geothermal and stability studies, design assistant to Mr. Goodall.

- Mr. Jim Murphy of Golder Associates, Calgary, Intermediate Dam investigation.

- Dr. J.I. Clark of Golder Associates, Calgary, review.

TABLE 5-1
BOREHOLE SCHEDULE

BOREHOLE	LOCATION NORTH(m)	LOCATION EAST(m)	ELEVATION	DEPTH OF BOREHOLE (m)	PIEZOMETER DETAILS TYPE	PIEZOMETER DETAILS TIP ELEV.	NUMBER OF THERMISTORS INSTALLED	REMARKS
79-1	2320.85	122.32	1049.38	17.84	Standpipe	1034.42		℄ Dam - Cross Valley Dam
								Borrow pit
79-3	2361.99	814.80	1132.74	5.18				Borrow pit
79-4	2418.36	833.67	1140.35	9.15				Borrow pit
79-5	2439.05	147.66	1077.37	12.20				North Abutment - Cross Valley Dam
					Standpipe	1011.28		℄ Dam - Cross Valley Dam
79-7	2081.44	-43.55	1048.34	25.91	Standpipe	1022.43		Dam Toe
79-8	1806.18	-92.71	1078.80	16.15				South Abutment - Cross Valley Dam
79-9	2305.00*	851.00*		5.18				Borrow pit
79-9A	2394.65	913.28	1141.82	4.57				Borrow pit
79-10	2378.12	979.62	1142.82	4.57				Borrow pit
79-11	2386.95	1039.04	1148.19	4.57				Borrow pit
79-12	2290.54	1267.85	1151.91	3.96				Borrow pit
79-13	2284.47	1372.13	1157.36	4.57				Borrow pit
79-14	2237.31	1249.75	1144.76	4.57				Borrow pit
79-15**	2374.12	57.74	1056.10	38.11	Standpipe	1028.27		Dam Toe - Cross Valley Dam
79-16**	2256.12	5.14	1048.25	30.49	Standpipe	1015.20	5	Dam Toe - Cross Valley Dam
79-17	2448.63	109.94	1076.83	12.19				North Abutment - Cross Valley Dam
79-18**	2138.41	44.98	1049.23	41.14	Pneumatic	1019.13	5	℄ Dam - Cross Valley Dam
79-19**	2216.56	73.73	1048.26	24.38	Standpipe	1030.73		℄ Dam - Cross Valley Dam
79-20**	2447.49	114.36	1076.80	37.19	Standpipe	1039.80	9	North Abutment - Cross Valley Dam
79-21**	1974.58	-119.73	1050.07	36.50	Standpipe	1013.49		Dam Toe - Cross Valley Dam
79-22	1869.83	-198.86	1079.20	9.14	Pneumatic	1067.31		South Abutment - Cross Valley Dam
79-22A	1840.84	-185.90	1081.90	18.29			5	South Abutment - Cross Valley Dam
79-23	1910.41	-387.72	1083.17	18.29				Sta.3+000 Canal
79-24	1161.40	1222.52	1084.64	22.86	Pneumatic	1063.47	5	Sta.1+230 Canal
79-25	1050.57	1432.59	1087.00	12.19	Pneumatic	1025.00	9	Sta.1+000 Canal
79-26	2379.13	126.90	1061.99	7.01				℄ Dam (N.Abut) - Cross Valley Dam
79-27	1946.49	427.73	1066.71	51.83	Standpipe	1016.41		Intermediate Dam
79-28	2056.32	490.48	1065.17	39.63	Standpipe	1025.54		Intermediate Dam
79-29	2250.56	559.17	1084.95	17.07	Standpipe	1068.19	5	Intermediate Dam
79-30	2138.07	521.03	1060.73	24.39	Standpipe	1036.34		Intermediate Dam
79-31	1796.72	334.19	1053.62	48.78	Standpipe	1004.84		Intermediate Dam
79-32	1578.76	235.42	1085.37	7.93	-	-		Intermediate Dam
79-33	1747.61	301.65	1054.07	25.00	Destroyed			Intermediate Dam
79-34	2177.44	-23.00	1048.54	26.83	-	-		Cross Valley Dam
80-35	1709.74	258.45	1056.08	8.10	Standpipe	1047.98		Intermediate Dam
80-36	1776.67	250.93	1052.73	12.20				Intermediate Dam

TABLE 5-1 (Cont'd)
BOREHOLE SCHEDULE

BOREHOLE	LOCATION NORTH(m)	LOCATION EAST(m)	ELEVATION	DEPTH OF BOREHOLE (m)	PIEZOMETER DETAILS TYPE	PIEZOMETER DETAILS TIP ELEV.	NUMBER OF THERMISTORS INSTALLED	REMARKS
80-37	1711.65	300.85	1053.41	8.10	Standpipe	1045.31		Intermeidate Dam
80-38	2215.32	41.17	1048.85	4.60				Cross Valley Dam
80-38A	2198.99	41.94	1048.86	5.00				Cross Valley Dam
80-39	2153.38	21.98	1048.84	6.50	Standpipe	1046.24		Cross Valley Dam
80-40	2113.48	7.51	1049.11	3.20	Standpipe	1045.81		Cross Valley Dam
80-41	2069.45	-8.56	1049.15	2.30	Standpipe	1046.85		Cross Valley Dam
80-46	1807.56	303.86	1054.11	14.20	Standpipe	1040.31		Intermediate Dam
80-47	1600.21	164.83	1090.66	10.70	Standpipe	1079.96		Sta. 2+370 Canal
80-48	1568.45	227.72	1089.89	9.10	Standpipe	1080.79		Sta. 2+300 Canal
80-49	1528.80	299.21	1088.83	10.70	Standpipe	1078.13		Sta. 2+220 Canal
80-50	1475.77	404.44	1085.66	9.10	Standpipe	1076.56		Sta. 2+100 Canal
80-51	1437.99	482.46	1089.54	13.70	Standpipe	1076.04	5	Sta. 2+000 Canal
80-52	1369.51	618.50	1089.41	9.10	Standpipe	1080.31		Sta. 1+890 Canal

*Estimated from aerial photos
**Downhole geophysical logging performed.

TABLE 5-2
TEST PIT SCHEDULE (Other Than Along Canal)

TEST PIT	LOCATION NORTH(m)	LOCATION EAST(m)	ELEVATION (m)	DEPTH OF TEST PIT (m)	REMARKS
79-1	2420.13	735.96	1133.45	3.96	Till Borrow Pit
	2411.37	715.25	1131.86		
79-2	2367.89	705.27	1123.44	3.81	Till Borrow Pit
	2368.50	673.86	1119.91		
79-3	3125.82	(-813.81)	1069.14	5.33	Downstream Gravel Pit
	3160.17	(-835.68)	1068.71		
79-4	2483.29	1004.77	1157.04	5.18	Till Borrow Pit
	2519.95	1038.50	1162.52		
79-5	2486.88	(-590.94)	1055.96	6.00	Downstream Terrace
	2507.10	(-587.73)	1055.69		
79-6	2308.52	(-476.58)	1055.60	5.60	Downstream Terrace
	2291.83	(-459.18)	1055.78		
79-7	2291.95	(-537.02)	1057.61	3.35	Downstream Terrace
	2307.82	(-553.33)	1057.54		
79-8	2342.95	1030.72	1144.45	3.90	Till Borrow Pit
	2349.13	1005.11	1143.01		
79-9	2272.79	1294.52	1151.80	4.27	Till Borrow Area
	2282.62	1320.80	1153.53		
79-10	2482.69	69.92	1074.41	2.83	North Abutment
	2522.43	33.36	1075.91		
79-11	2113.39	(-413.29)	1060.72	3.66	Outfall Area
	2149.41	(-469.62)	1060.92		
79-12	2065.01	(-239.99)	1055.22	1.37	Outfall Parallel to Creek
	2055.37	(-334.41)	1059.49		
79-13					
℄ Top	1945.19	836.86	1069.07	3.0	Gravel Terrace
℄ Bottom	1943.70	843.91	1066.56		
℄ Top	1938.48	858.06	1069.66		
79-14					
℄ Top	1713.13	1028.00	1069.88	6.5	Gravel Terrace
℄ Bottom	1722.47	1011.67	1064.57		
℄ Top	1738.86	989.13	1069.23		
79-15	1967.08	600.85	1065.77	5.8	Gravel Terrace
79-16	2096.80	414.91	1060.61	8.2	Old Borrow Pit
79-17	2041.66	380.74	1064.34	6.4	Old Borrow Pit
79-18	2144.77	524.84	1062.13	4.1	North Abutment, Intermed. Dam
	2133.57	553.92	1062.39		
79-19	2124.35	787.81	1081.89	5.2	Core Borrow Area

TABLE 5-2 (Cont'd)

TEST PIT SCHEDULE (Other Than Along Canal)

TEST PIT	LOCATION NORTH(m)	LOCATION EAST(m)	ELEVATION (m)	DEPTH OF TEST PIT (m)	REMARKS
79-20					
℄ Top	2143.99	31.36	1049.46	4.6	Cross Valley Dam
℄ Bottom	2130.97	28.36	1046.16		
℄ Top	2111.20	24.07	1049.24	3.0	Sta. 0+070 East Canal
80-23					
℄ Top	2178.55	874.12	1109.20	4.0	Core Borrow Area
℄ Bottom	2181.30	857.64	1104.13		
Side	2178.84	854.16	1105.54		
℄ Top	2189.56	840.81	1107.87		
Side	2188.31	858.70	1108.63		
80-24				4.0	Core Borrow Area
℄ Top	2160.66	1158.30	1128.98		
℄ Bottom	2157.78	1169.93	1125.56		
℄ Top	2150.58	1185.41	1129.35		
Side	2153.09	1170.01	1129.34		
Side	2164.23	1170.80	1130.66		
80-25				4.0	Core Borrow Area
℄ Top	2105.12	1252.65	1124.27		
℄ Bottom	2102.13	1229.24	1118.04		
℄ Top	2102.72	1215.42	1121.11		
Side	2097.68	1228.99	1120.57		
Side	2111.52	1230.56	1122.54		

TABLE 5-3
TEST PIT SCHEDULE ALONG CANAL

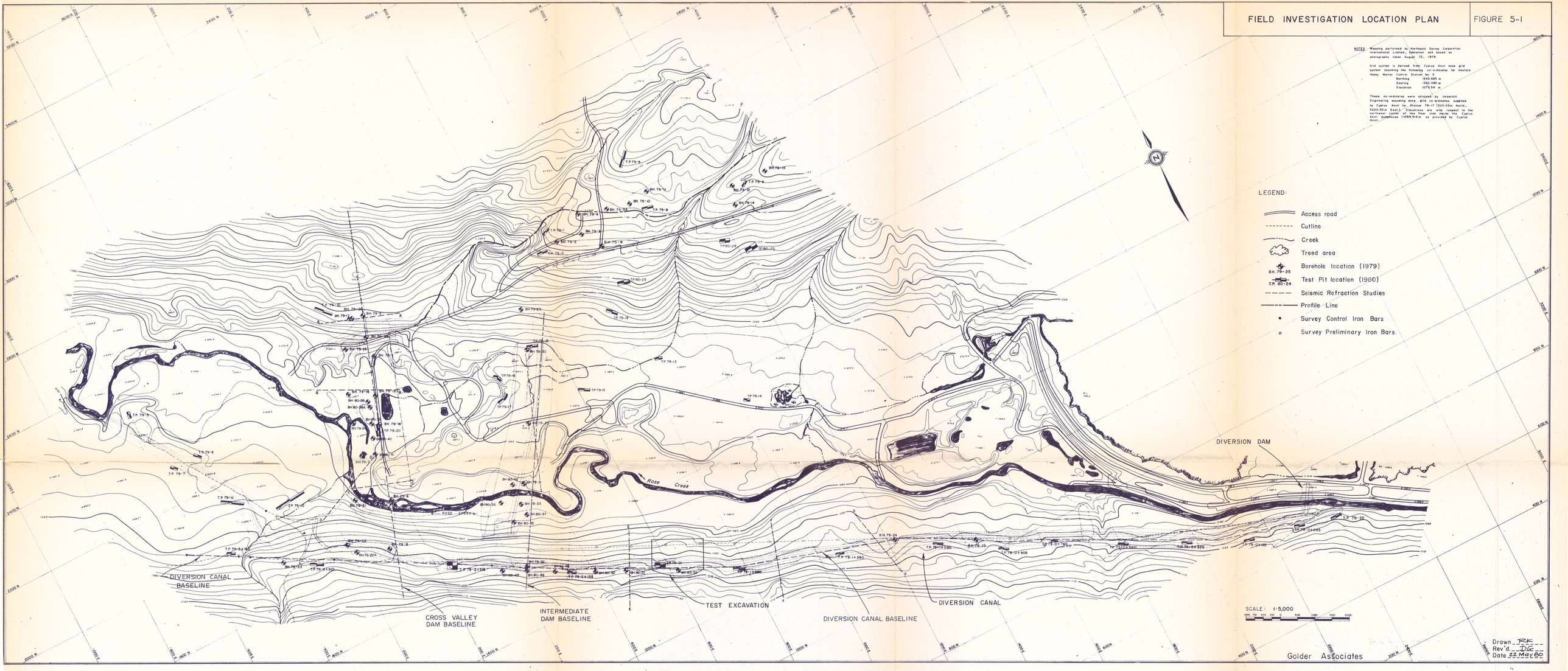
STATION FROM(m)	STATION TO(m)	ELEVATION (m)	DEPTH OF TEST PIT (m)	REMARKS
0-060	0-080	1091.09	3.00	Excavated December, 1979 (T.P. 79-22)
0+030	0+060	1091.3	3.66	Excavated August, 1979
0+180	0+210	1092.3	2.44	"
0+360	0+390	1089.3	4.57	"
0+545	0+575	1090.8	4.27	"
0+750	0+770	1091.9	3.96	"
0+890	0+920	1088.9	4.88	"
1+080	1+110	1089.5	4.88	"
1+380	1+400	1087.3	2.44	"
1+665	1+700	1086.5	4.27	"
1+900	1+930	1087.4*	4.00	Excavated December, 1979 (T.P. 79-21)
2+155	2+180	1084.0	4.27	"
2+500	2+530	1083.5	6.83	"
2+890	2+930	1086.0*	5.18	"
3+150	3+175	1072.3*	3.66	"
1+772	1+936	1086.8*	8.50	Test Excavation March, 1980

* Elevations averaged from canal baseline profile. All other obtained by stadia survey of pit periphery and averaging.

NOTE: For the most part the test pits are located on the uphill side of the canal baseline. See Figure 5 - 1 for plan location.

FIELD INVESTIGATION LOCATION PLAN

FIGURE 5-1



6.0 GEOTECHNICAL SETTING

6.1 Location of Information

The factual findings of the geotechnical studies for the Down Valley Project are documented for the boreholes and test pits in Appendix III and for the test excavation in Appendix V. This data, together with geophysical and monitoring data, has been summarized on profiles and cross-sections as follows:

Canal — Profile Sta.
- 0+000 to 0+700 - Figure 6-1
- 0+700 to 1+400 - Figure 6-2
- 1+400 to 2+100 - Figure 6-3
- 2+100 to 2+800 - Figure 6-4
- 2+800 to 3+290 - Figure 6-5
- Outfall - Figure 6-6

Cross Valley Dam

Profile - Figure 6-7

Intermediate Dam

Profile - Figure 6-8

Information not presented here but available from Golder Associates files includes the following:

(i) Seismic refraction data for cross-sections A,B,C,D,E,F,G,H shown in plan on Figure 5-1.

(ii) Electromagnetic induction measurements along lines H and I shown on Figure 5-1.

(iii) Stadia survey cross-sections along the canal baseline from Sta. 0+000 to 2+600 at 100 m intervals, along the Cross Valley Dam baseline at 60 m intervals, and through the initially proposed core borrow pit at 60 m intervals.

6.2 Findings

6.2.1 Diversion Canal -

About 0.3 m of moss and organic debris generally blankets the ground along the canal route. This organic cover is instrumental in maintaining permafrost conditions. This stratum freezes seasonally.

Beneath the organic cover are black or brown organic silts and/or colluvium to a depth of about 1.2 m with exceptions occurring locally and in the outfall area where such materials are found to depths of 1.5 m (an extreme depth of 2.4 m was observed away from the canal route). Often this organic silt persists as permafrost and may have water contents as high as 120 percent. In the vicinity of creeks and groundwater flows this material may remain unfrozen on an annual basis and these areas can present trafficability problems. Some white volcanic ash may also be found dispersed within the organic silt layer.

Beneath the organic silt stratum a form of till occurs. The till may be green or brown, the green colour probably indicating the top of the stratum as a result of chemical reduction or gleying at the base of the surficial organic layers. The upper surface of the till is often pebble rich and it is apparent that some erosion of the till surface may have occurred at one time. In the vicinity of the test excavation boulders up to 1.8 m in diameter were encountered at the top of the till stratum. Evidence of some colluvial action is also present in that the upper till may be interlayered with organic layers and/or white volcanic ash.

There is a substantial variation in the till over the canal route from the point of view of both condition and composition. In general, frozen till may occur throughout the route although more continuously frozen, ice rich, fine-grained till tends to occur beyond station 0+900 on the canal baseline. It is considered that the seismic refraction velocities presented on the canal profile sheets (Figures 6-1 to 6-5 inclusive)

provide an indication of the frozen or unfrozen condition of the till. A report prepared by Atmex Geophysics Inc., which performed the field studies, indicated that "the permafrost horizon may correspond to a velocity between 2000 and 3500 m/sec". Furthermore, it was suggested that partially frozen, unsaturated till could correspond to a velocity between 900 and 1500 m/sec and that velocities between 1500 m/sec and 2100 m/sec correspond to unfrozen, saturated till.

Thermistors installed along the canal baseline confirm the discontinuous nature of the permafrost. The thermistor installed in Borehole No. 79-24 indicates no permafrost. This borehole is located in a relatively flat topographic low where it is evident that appreciable surface and subsurface water flow occurs and that this flow has a geothermal warming effect. It is noted that this is the only thermistor along the canal route which shows no frozen ground.

Aside from seasonal freezing the thermistors generally indicate minimum temperatures of about -1° C. It is not surprising then that the permafrost is not well defined by the thermistors, by seismic refraction or even by direct excavation. A significant amount of unfrozen water can exist at temperatures between 0° C and -1°C in fine grained materials and this unfrozen water impacts the rippability of the material. In the course of construction of the test excavation the till was found to be discontinuously frozen on a small scale in that certain chunks of till contained some ice stratification whereas adjacent material was completely granular. It is also of interest that the frozen chunks were plastic and could be broken down depending on the ambient air temperature, overall rate of excavation and time to placement.

Till water contents normally range from a low of about 7 percent to a maximum of about 39 percent. The average water content observed for materials from the test excavation was about 18 percent with the higher values being observed near the ground surface. A standard Proctor compac-

tion test on a till sample from the test excavation yielded a maximum dry density of 2100 kg/m^3 at a water content of 7.8 percent.

Till composition is also highly variable. Gradations for selected samples taken along the canal route are illustrated on Figure 6-9. The outstanding feature of the tills is that they are extremely broadly graded with only a few exceptions. Typically the materials contain at least 9 percent by weight of silt and clay size particles. Examination of Figure 6-9 indicates that most of the samples containing more than 40 percent silt and clay size particles occur in the vicinity of the south abutment of the Cross Valley Dam at Borehole No. 79-8 and at Borehole No. 79-24 near canal Sta. 1+230. Some of these samples exhibit low plasticity. More typically the till contains between 10 and 30 percent fine grained material.

Of some concern from the point of view of seepage losses are the materials near the inlet to the proposed canal. The occurrence of alluvial desposits and gap graded schist-rich gravel tills has been noted on the logs such as for Test Pit Nos. 79-0+195 and 79-0+045 (referred to as 1+80 and 0+30 on Figure 6-9 legend). These coarser materials would be expected to have a permeability between 10^{-5} and 10^{-6} cm/sec. although the stability of the fine grained portion of the gap graded gravel till could be questioned when subjected to seepage gradients. During the field investigation it was noted that standing water in Test Pit No. 0+195 disappeared after excavation.

A permeability test was conducted on till material obtained from the test excavation and compacted with standard Proctor effort at natural water content. The resulting permeability was about 3×10^{-7} cm/sec. The sample contained about 22 percent silt and 7 percent clay sizes on a dry weight basis. Although crude in its representation of actual field conditions the permeability increased by a factor of about 15 when the sample was subjected to several freeze/thaw cycles.

Shear strength parameters for the till are discussed in Appendix VI. The effective angle of internal shear resistance for nonplastic tills is expected to be about 36 degrees as was obtained for till from the proposed till borrow pit. Tills which contain higher proportions of fines and display some plasticity may have effective angles of internal friction as low as 32.5 degrees as was obtained for material from Borehole No. 79-24.

Some densities for the till in situ were obtained by determining the density of frozen chunk samples. These ranged from 1780 to 2260 kg/m^3 with an average of 1989 kg/m^3. The particle specific gravity for sample No. 1 from Borehole No. 79-24 was determined to be 2.73.

In the outfall portion of the proposed canal, alluvial deposits underlie the surficial organic silts (see Figure 6-6). Some of these gravel and sand deposits are marginally frozen and will require ripping for excavation under freezing conditions. In general these alluvial deposits become sandier with depth. Typical gradations may be found on Figure 6-10.

Bedrock will be encountered within the proposed canal depth of excavation as indicated by seismic data, borehole and test pit information. The reaches of the canal baseline where bedrock is likely to be encountered are:

(i) at the inlet (bedrock encountered at depth 2.4 m in Test Pit No. 79-22 [canal baseline Sta. 0-070])
(ii) Sta. 0+050 to 0+230
(iii) Sta. 0+330 to 0+350
(iv) Sta. 0+500 to 0+820
(v) Sta. 1+490 to 1+600
(vi) Sta. 1+940 to 1+980
(vii) Sta. 2+200 to 2+410

In most of these areas the rock is indicated to be nonrippable with seismic velocities generally exceeding 4000 m/sec. Test Pits which have encountered bedrock have also confirmed this.

Geologic evidence suggests that the bedrock in the Rose Creek diversion area dips to the south-southwest at about 20 to 30 degrees. This means that the primary bedrock structure dips into the hillside along the proposed diversion canal route and this has been confirmed in test pit exposures of subcrops, particularly in the vicinity of Sta 0+600. The rock outcrops which occur upslope of the proposed canal are postulated to be relatively erosion resistant bedrock and these outcrops are probably related to shallow bedrock occurrences along the canal baseline. The 1974 quarry was developed in such rock and it is also probable that such outcrops represent additional quarry potential in the vicinity of Sta. 1+500 or Sta. 2+500 as necessary.

The specific gravity of a rock chunk sample recovered from Test Pit No. 79-22 was found to be 2.80.

It should be expected that some of the rock near the bedrock surface is disturbed and fractured. The permeability of such rock is unknown but it is anticipated that some till lining of the canal will be required to minimize seepage losses. There are indications that some bedrock will be frozen and therefore use of blasted rock as a potential source of riprap must be based on unfrozen fragment sizes. Some photographs from the existing quarry may be found in Appendix IV.

6.2.2 Cross Valley Dam

At the location of the Cross Valley Dam (see Figures 2-2 and 5-1), the Rose Creek Valley has been infilled mainly by granular alluvium to a maximum depth of about 50 m. The Cross Valley Dam site is somewhat unique in that only a small gravel ridge exists to the elevation of the upper level terrace which is characteristic of the Rose Creek Valley. Terrace level is generally about 10 m above the elevation of the creek.

The valley floor deposits above creek level in the "gravel" ridge consist of mainly sand and gravel but also there are extensive lenses of frozen, nonorganic fine sands and silts (see for example curve #7, Figure 6-10) and, further upstream, organic silt pockets which can be seen on aerial photographs of the area because the ridge was stripped in July/August, 1979. The gravel ridge deposits are be discussed below under the heading, Borrow Sources.

The alluvial fill materials at or near creek level generally are loose fine sands mixed with organics. In the vicinity of the closure section of Rose Creek (i.e. Borehole No. 79-21) several large boulders were encountered to depths of about 2.1 m. In some areas in the vicinity of the upstream toe of the proposed dam, organic silts and muskeg occur to depths of 4.5 m and, in the vicinity of Borehole No. 79-18 these high water content materials are frozen. Test Pit No. 79-20 yielded frozen densities in the organic silt ranging from 1181 to 1485 kg/m^3. Water contents ranged as high as 90 percent and thaw consolidation tests on the material indicated that the settlement on thaw alone would be about 29 percent of the thickness of the deposit. The identifiable wood content of the material was found to be 7.5 percent.

The occurrence of permafrost in the vicinity of the upstream toe of the proposed dam is shown by the seismic data presented on Figure 6-7. The near surface stratum showing a seismic velocity of 3500 m/sec corresponds to permafrost and confirmation of this is shown by the thermistor data obtained for Borehole No. 79-18 and the downhole geophysical log which are contained in Appendix III. It is noted, however, that no permafrost was observed during drilling at Boreholes No. 79-7, 79-19, 80-38, 80-38A, 80-41 and 79-34. Furthermore, it appears that the permafrost thins from the dam baseline toward the cross valley arm of Rose Creek because both Borehole Nos. 80-39 and 80-40 had to be terminated due to inflow of water into the air rotary hole and these termination levels are believed to define the lower limit of permafrost. Seismic and

electromagnetic induction measurements along line H-H of Figure 5-1 indicate the 10 m thickness of permafrost encountered at Borehole No. 79-18 probably extends up valley beneath the proposed upstream blanket (discussed later in text).

The alluvium which forms the unconsolidated foundation of the Cross Valley Dam is highly varible. The variety of gradations is demonstrated on Figure 6-11 which clearly indicates that fine grained materials do occur, particularly in the vicinity of the current Rose Creek channel. Some of these materials appear to be gap graded silty gravels. On the north side of the valley floor occurrences of schist rich "till" were noted within the alluvium at Borehole Nos. 79-1 and 79-19. This observation is believed to be related to the relatively shallow bedrock elevation and is compatible with the till-like deposits which are wedged within upper level terrace gravels upstream and downstream of the proposed Cross Valley Dam.

Standard Penetration Test "N" values indicate that the alluvium is in a marginally compact condition in some local areas with a minimum value of 10 being observed. More typically it is compact with "N" values ranging between 15 and 30. Abnormally high "N" values reported on the logs reflect the occurrence of particle sizes much larger than the split spoon diameters.

A loss of circulation zone was encountered at about El. 1030 in Borehole Nos. 79-15 and 79-16 and corresponded to a relatively high permeability zone in Borehole 79-1. Piezometer tests in the alluvium indicate permeabilities up to 10^{-4} cm/sec. Computations relating to water production and depressed head for the air rotary Borehole 79-34 suggests a maximum permeability of about 10^{-2} cm/sec. It should be noted that this is a local permeability and does not represent the alluvium as a whole. In fact the alluvium is interlayered with fine sandy silts at Borehole 79-34 such that the overall vertical permeability may be less than 10^{-5} cm/sec.

However, the occurrence of relatively permeable sand and gravel at the base of the dam must be recognized in seepage estimates.

The bedrock which forms the valley bottom is a greenish to grey schist which varies in hardness to drilling. Permeabilities of the bedrock as determined by piezometer rising/falling head tests are approximately 10^{-5} cm/sec.

The abutments of the cross valley dam are transitional between the valley bottom alluvium and the valley wall tills, the latter often occurring as ice stratified permafrost near surface. Borehole No. 79-26 on the north abutment encountered alluvium exclusively. Alternatively the till which was found in Borehole Nos. 79-5, 79-17 and 79-20 was frozen to a depth of about 13 m and contained significant pockets of alluvium. The excavation for the North Valley Wall Interceptor will most certainly include some permafrost excavation. At depth it is believed that a deposit of rock flour was encountered at Borehole No. 79-20.

The south abutment will be keyed into frozen tills and possibly some fine grained colluvium. It is expected that the upper 1.5 m of this material will be relatively ice rich. Downstream of the Cross Valley Dam the alluvial fan of the valley wall creek occurs and some unfrozen fine grained alluvium should be anticipated there. Gradations of selected samples taken from the Cross Valley Dam abutments are presented on Figure 6-12.

6.2.3 Intermediate Dam

The Intermediate Dam is located about 400 m upstream of the Cross Valley Dam and differs from the Cross Valley Dam in that much of the dam traverses the upper level terrace gravels. The unconsolidated foundation also differs in that no extensive occurrences of permafrost occur in the valley bottom area.

As for the Cross Valley Dam, the Intermediate Dam foundation is fine grained in the proximity of the current Rose Creek channel. The current creek banks consist of near surface loose fine sand alluvium mixed with organic debris which is anticipated to be about 1 m thick. In fact several distributary flood channels dissect this surficial alluvium adjacent to Rose Creek to depths of about 1 m and this alluvium is precarious to surface traffic. This is also true for the Cross Valley Dam foundation area.

The fine grained foundation materials generally exist in a compact condition with "N" values ranging fom 14 to 67 (see Record of Borehole sheets for Borehole Nos. 80-33 and 80-46). Some colluvial action is apparent at the location of Borehole No. 80-37 in the south abutment area where organic seams occur within a till-like mass; here an "N" value of 10 was recorded. The organic silt observed in the field in Borehole No. 79-31 contained only 0.8 percent organic matter with respect to air dry weight. It appears that this stratum becomes coarser and more competent with increasing distance downstream of the Intermediate Dam baseline as evidenced by contrasting observations between Borehole Nos. 79-31, 79-33, 80-37 and 80-46, 80-35,80-36.

Typical moisture contents of the fine grained alluvium range from about 18.5 to 28.8 percent with an average of about 26 percent. Gradations for the fine grained alluvium occurring in Boreholes Nos. 79-31, 79-33, and 80-46 are reported in Figure 6-11.

The bulk of the Intermediate Dam foundation will interface with sand and gravel alluvium similar to that reported on Figure 6-11 for the Cross Valley Dam. The permeabilities, as indicated by the air rotary rig water returns, could be expected to range as high as 10^{-2} cm/sec. However, the layering of the natural materials precludes the direct use of this parameter in estimates of seepage beneath the dam.

The bedrock forming the bottom of the valley at the site of the Intermediate Dam is a greenish grey schist. Normally little or no water return was obtained when drilling with the air rotary rig indicating a permeability lower than 10^{-5} cm/sec. However, significant flows were observed in Borehole No. 80-33 suggesting that bedrock disturbance may exist at this location and therefore that the permeability may be as high as 10^{-4} cm/sec. This is consistent with the relatively low seismic velocities which were observed in the vicinity of Rose Creek at the Cross Valley Dam.

The north abutment of the Intermediate Dam consists of relatively coarse sand and gravel till and some alluvium as indicated by Test Pit No. 79-18 and Borehole No. 79-29. The materials are quite dry with a moisture content of only 8 percent being observed at the Test Pit which was also completely frozen. However, Borehole No. 79-29 revealed no permafrost at that location.

The topographic low which occurs in the vicinity of Test Pit No. 79-18 indicates that there has been some near surface fluvial action and this is confirmed by the silty sand lensing which was evident on one wall of the test pit and noted on the log.

Bedrock occurs at a relatively shallow depth of about 9 m at the north abutment.

In the south abutment area it is expected that frozen till will be encountered with the possibility of some colluvium and organic deposits. The occurrence of shallow bedrock is also a possibility judging from the occurrence of bedrock at a depth of about 2 m at Borehole 79-32. An irregular, possibly disturbed, bedrock surface should be anticipated in the south abutment area.

6.2.4 Borrow Sources

There are a number of areas which have been investigated as potential sources of gravel, filter material, and till core. Specific quarry sites

for production of riprap have not been investigated but the experience gained in the existing quarry is documented on the photographs included in Appendix IV.

The most outstanding source of relatively clean gravel deposits is the upper level terrace which is prevalent north of Rose Creek and upstream of the Cross Valley Dam. Typically these materials have less than 5 percent passing the No. 200 sieve and are relatively dry with moisture contents of about 3 to 5 percent. Numerous exposures in old borrow pits and the current test pit program have demonstrated that the terrace is typically blanketed by 0.3 m of moss, organic silt and volcanic ash where no tree population is present. Where trees occur, up to 1.2 m of frozen organic silt may overlie the gravel which may or may not be locally frozen. Beneath the stripping the upper 1.5 m of gravel is found to be relatively silty and boulder rich (see P. 11, Appendix IV). Once below this upper gravel stratum the alluvium tends to become finer and lenses of medium to coarse sand are common. In some areas the gravel is relatively fine and yet clean and therefore it is considered generally suitable as a filter material against both general gravel borrow and the glacial till (see P.15, Appendix IV).

Gravel terrace deposits also occur downstream of the Cross Valley Dam in the outfall area of the canal, further downstream in an extension of the upper level terraces, and in the North Abutment area of the Cross Valley Dam. It appears that the comments made above for the up valley deposits are also appropriate for the downstream terraces. A summary of the gradations obtained for the various gravel deposits is provided on Figure 6-10.

Anomalous terrace deposits occur in the vicinity of and upstream of the north abutment of the Cross Valley Dam. These deposits appear to be till-like wedges within the normal terrace deposits, are schist rich and contain larger proportions of fines than the typical terrace gravels. No samples of this material were taken although photographs are on file.

Glacial till is available on both valley walls. However, the north valley wall tills are more accessible and are less extensively frozen. It is proposed therefore that the north valley wall tills be used for till borrow with supplementary potential arising from the till waste resulting from excavation of the diversion canal. This is also recommended to minimize activity on the south valley wall where unnecessary activity on the slope would risk the stability of the canal. Nevertheless the area in the vicinity of Borehole No. 79-24 is flat, relatively unfrozen and represents a possible source of till borrow on the south valley wall.

Example gradations of the till on the North Valley wall are provided on Figure 6-13. The outstanding feature of these tills are their gradation variability. The spectrum ranges from tills containing 60 percent gravel to those containing more than 50 percent passing the No. 200 sieve. The coarse tills are schist rich and handling of such materials would probably result in some segregation and locally high permeabilities (see photograph of schist rich till on P.4, Appendix IV). The occurrence of schist rich till is not directly related to the proximity of bedrock because, at several locations, it overlies finer grained tills below.

Till thicknesses range from as low as 1.5 m to as high as 5.1 m with an average thickness of about 3.5 m. Stripping depths of 0.3 to 0.5 m should be expected to expose the till thicknesses described above.

A more thorough discussion of borrow pit quantities is provided in Appendix VII.

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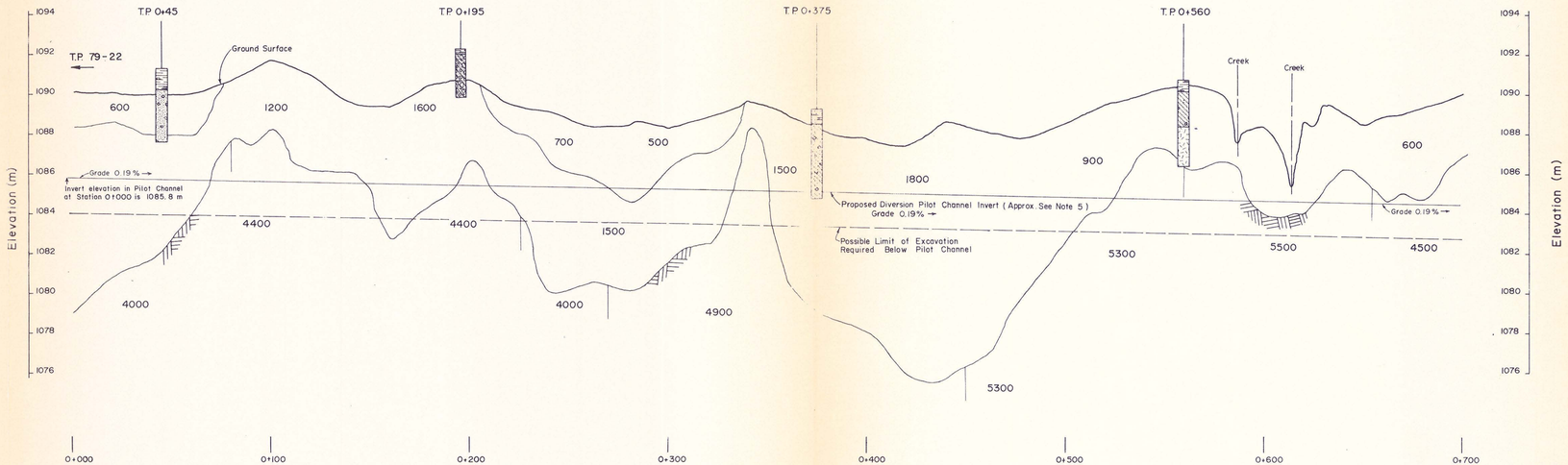
CANAL BASELINE GEOTECHNICAL PROFILE
STA. 0 + 000 TO STA. 0 + 700

FIGURE 6-1

EAST

WEST



Horizontal Chainage (m) Along Canal Baseline

LEGEND

- Fill
- Organics & Debris
- Organic Silt
- Volcanic Ash
- Boulders & Cobbles
- Gravel
- Sand & Gravel (Alluvium)
- Sand
- Silt
- Clay
- Till or Colluvium
- Bedrock
- Water level
- Piezometer tip elevation
- T.P. Test Pit
- B.H. Borehole
- Thermistor Installation
- frozen
- unfrozen
- Sept. 27/79
- 2000 - Seismic Velocity m/sec
- Seismically inferred bedrock surface
- Seismic boundary

SCALE: Vertical - 1:200
Horizontal - 1:2000

NOTES :

1) Ground surface profile obtained by Underhill Engineering Ltd., Whitehorse, Yukon using ground survey techniques based on co-ordinates for Cyprus Anvil station 78-17 of 320.09 m N., 3220.50 m E and a NW corner pumphouse slab elevation 1089.515 m.
2) Seismic data obtained by Atmex Geophysics Inc., Vancouver, B.C. Seismic boundaries are based on observed velocities which may result from the integrated effect of several layers.
3) Test Pit and Borehole information projected onto profile (See plan location on Figure 5-1. Detailed Test Pit and Borehole logs may be found in Appendix III Test Pit elevations obtained by averaging stadia survey elevations of pit periphery except T.P. 1+915 (T.P. 79-21), T.P. 2+155, T.P. 2+910, and T.P. 3+160 which are estimated from canal baseline Ground Surface profile.
4) Interpolation of stratigraphy between boreholes and test pits using seismic data is based on conditions at the boreholes and the test pits and therefore should not be considered conclusive.
5) Canal Alignment does not follow Canal Baseline precisely. See Figure 5-1

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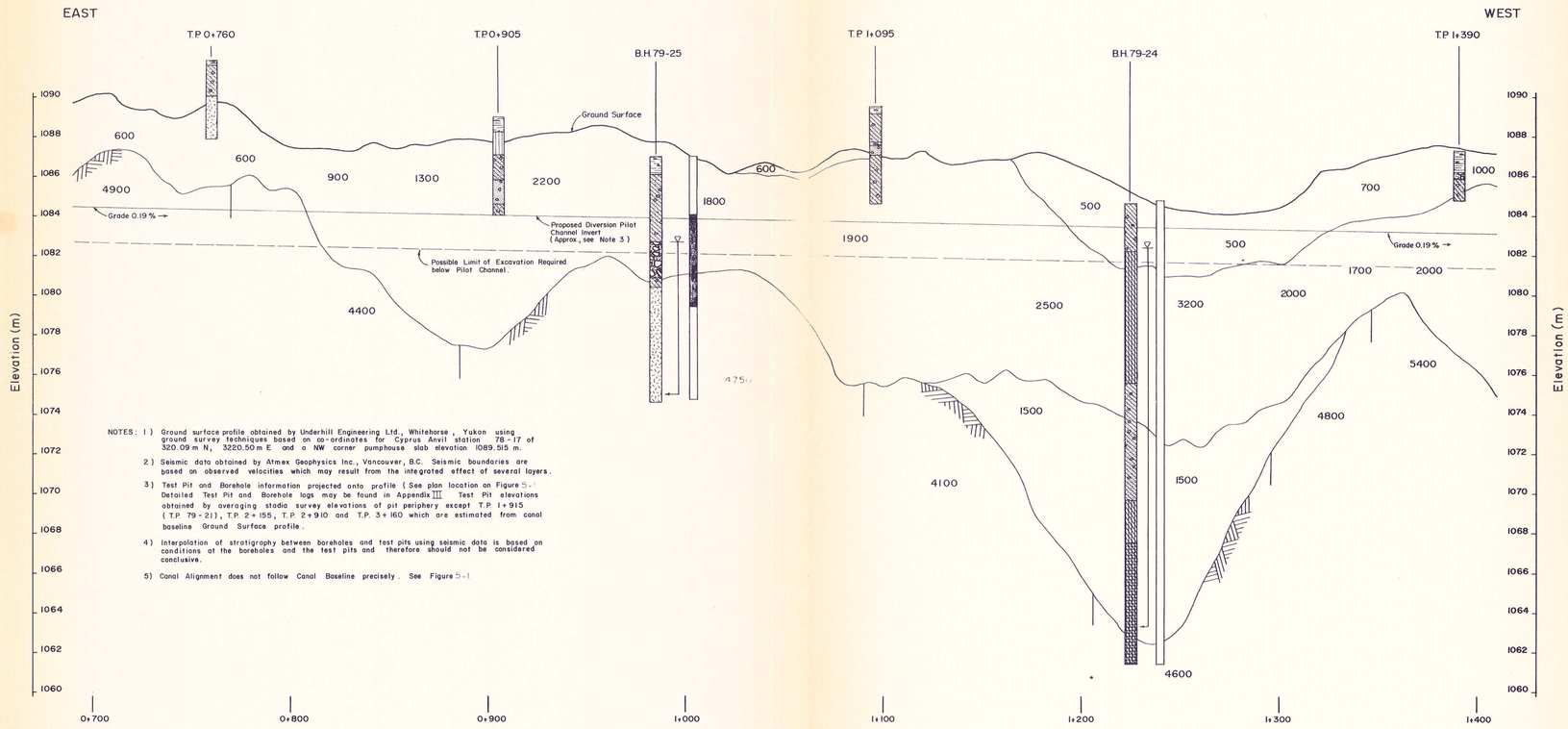
Rev'd: DG
Date: May 1980

Project No. 792-2025

CANAL BASELINE GEOTECHNICAL PROFILE
STA. 0 + 700 TO STA. 1+400

FIGURE 6-2



NOTES: 1) Ground surface profile obtained by Underhill Engineering Ltd., Whitehorse, Yukon using ground survey techniques based on co-ordinates for Cyprus Anvil station 78-17 of 320.09 m N, 3220.50 m E and a NW corner pumphouse slab elevation 1089.515 m.

2) Seismic data obtained by Atmex Geophysics Inc., Vancouver, B.C. Seismic boundaries are based on observed velocities which may result from the integrated effect of several layers.

3) Test Pit and Borehole information projected onto profile (See plan location on Figure 5-). Detailed Test Pit and Borehole logs may be found in Appendix III. Test Pit elevations obtained by averaging stadia survey elevations of pit periphery except T.P. 1+915 (T.P. 79-21), T.P. 2+155, T.P. 2+910 and T.P. 3+160 which are estimated from canal baseline Ground Surface profile.

4) Interpolation of stratigraphy between boreholes and test pits using seismic data is based on conditions at the boreholes and the test pits and therefore should not be considered conclusive.

5) Canal Alignment does not follow Canal Baseline precisely. See Figure 5-1

LEGEND

- Fill
- Organics & Debris
- Organic Silt
- Volcanic Ash
- Boulders & Cobbles
- Gravel
- Sand & Gravel (Alluvium)
- Sand
- Silt
- Clay
- Till or Colluvium
- Bedrock
- Water level
- Piezometer tip elevation
- T.P. Test Pit
- B.H. Borehole
- Thermistor Installation (frozen / unfrozen) Sept. 27/79
- 2000 Seismic Velocity m/sec
- Seismically inferred bedrock surface
- Seismic boundary

Scale: Vertical - 1:200
Horizontal - 1:2000

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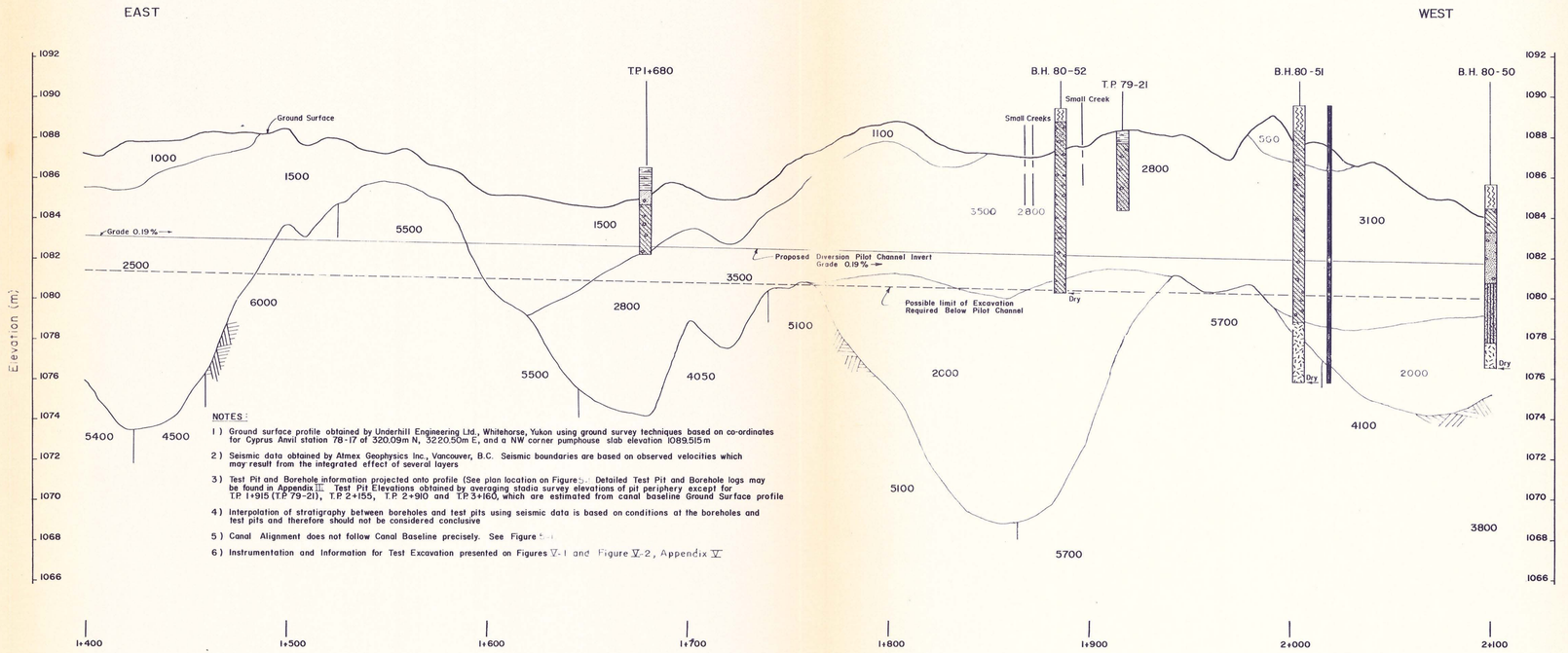
Revd: DG
Date: May 80

project 792-2025

CANAL BASELINE GEOTECHNICAL PROFILE STA. 1+400 TO STA. 2+100

FIGURE 6-3



NOTES:

1) Ground surface profile obtained by Underhill Engineering Ltd., Whitehorse, Yukon using ground survey techniques based on co-ordinates for Cyprus Anvil station 78-17 of 320.09m N, 3220.50m E, and a NW corner pumphouse slab elevation 1089.515m
2) Seismic data obtained by Almex Geophysics Inc., Vancouver, B.C. Seismic boundaries are based on observed velocities which may result from the integrated effect of several layers
3) Test Pit and Borehole information projected onto profile (See plan location on Figure 5-1). Detailed Test Pit and Borehole logs may be found in Appendix II. Test Pit Elevations obtained by averaging stadia survey elevations of pit periphery except for T.P. 1+915 (T.P. 79-21), T.P. 2+155, T.P. 2+910 and T.P. 3+160, which are estimated from canal baseline Ground Surface profile
4) Interpolation of stratigraphy between boreholes and test pits using seismic data is based on conditions at the boreholes and test pits and therefore should not be considered conclusive
5) Canal Alignment does not follow Canal Baseline precisely. See Figure 5-1
6) Instrumentation and Information for Test Excavation presented on Figures V-1 and Figure V-2, Appendix V

LEGEND

- Fill
- Organics & Debris
- Organic Silt
- Volcanic Ash
- Boulders & Cobbles
- Gravel
- Sand & Gravel (Alluvium)
- Sand
- Silt
- Clay
- Till or Colluvium
- Bedrock
- Water level
- Piezometer tip elevation
- T.P. Test Pit
- B.H. Borehole
- Thermistor Installation
- frozen
- unfrozen
- April 23, 1980
- 2000 - Seismic Velocity m/sec
- Seismic boundary
- Seismically inferred bedrock surface

SCALE: Vertical - 1:200
Horizontal - 1:2000

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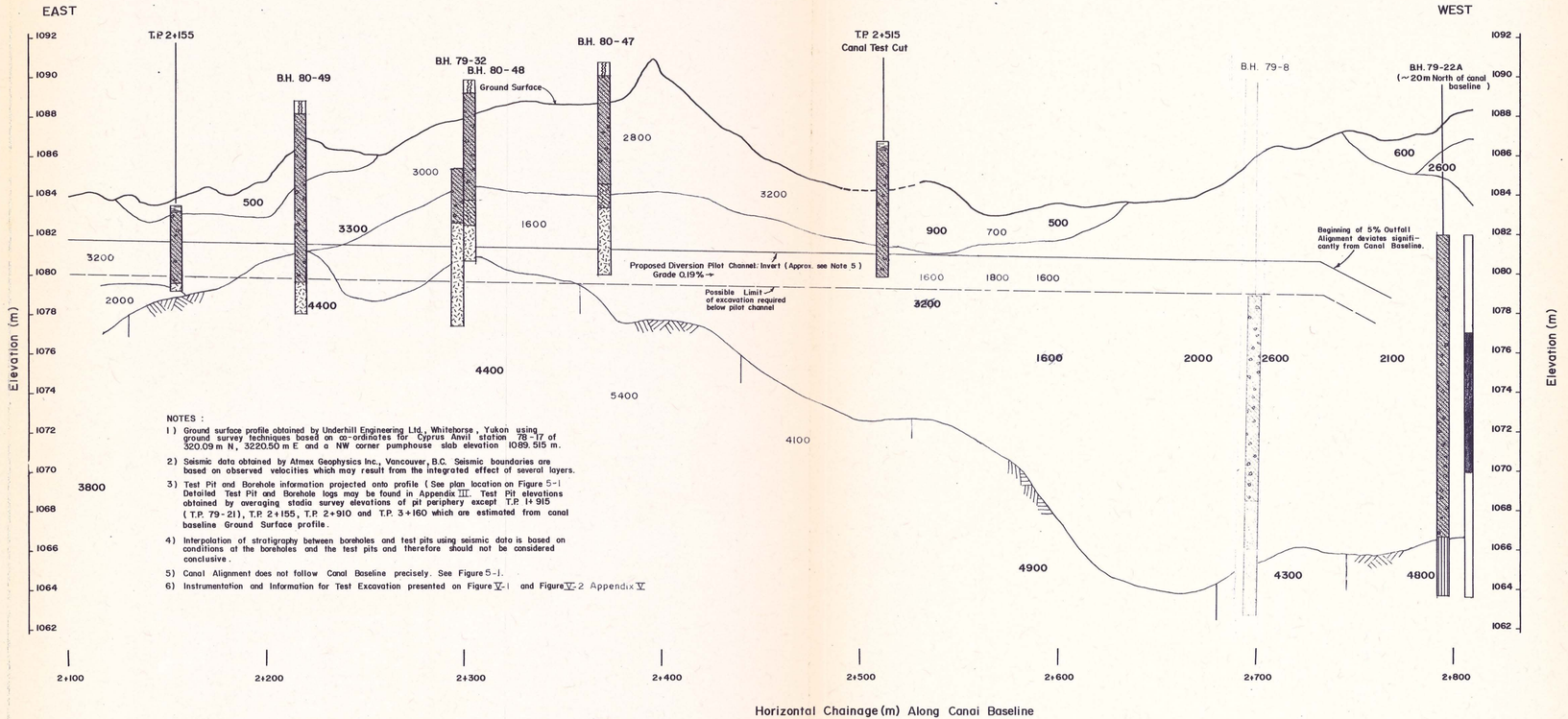
Date Jan., 1980

Project 792-2025

CANAL BASELINE GEOTECHNICAL PROFILE
STA. 2 + 100 TO STA. 2+800

FIGURE 6-4



NOTES :

1) Ground surface profile obtained by Underhill Engineering Ltd., Whitehorse, Yukon using ground survey techniques based on co-ordinates for Cyprus Anvil station 79-17 of 320.09 m N, 3220.50 m E and a NW corner pumphouse slab elevation 1089.515 m.
2) Seismic data obtained by Atmex Geophysics Inc., Vancouver, B.C. Seismic boundaries are based on observed velocities which may result from the integrated effect of several layers.
3) Test Pit and Borehole information projected onto profile (See plan location on Figure 5-1 Detailed Test Pit and Borehole logs may be found in Appendix III. Test Pit elevations obtained by averaging stadia survey elevations of pit periphery except T.P. 1+915 (T.P. 79-21), T.P. 2+155, T.P. 2+910 and T.P. 3+160 which are estimated from canal baseline Ground Surface profile.
4) Interpolation of stratigraphy between boreholes and test pits using seismic data is based on conditions at the boreholes and the test pits and therefore should not be considered conclusive.
5) Canal Alignment does not follow Canal Baseline precisely. See Figure 5-1.
6) Instrumentation and Information for Test Excavation presented on Figure V-1 and Figure V-2 Appendix V

LEGEND

- Fill
- Organics & Debris
- Organic Silt
- Volcanic Ash
- Boulders & Cobbles
- Gravel
- Sand & Gravel (Alluvium)
- Sand
- Silt
- Clay
- Till or Colluvium
- Bedrock
- Water level
- Piezometer tip elevation
- T.P. Test Pit
- B.H. Borehole
- Thermistor Installation
  - frozen
  - unfrozen
  - Sept. 27/79
- 2000 - Seismic Velocity m/sec
- Seismically inferred bedrock surface
- Seismic boundary

SCALE: Vertical - 1:200
Horizontal - 1:2000

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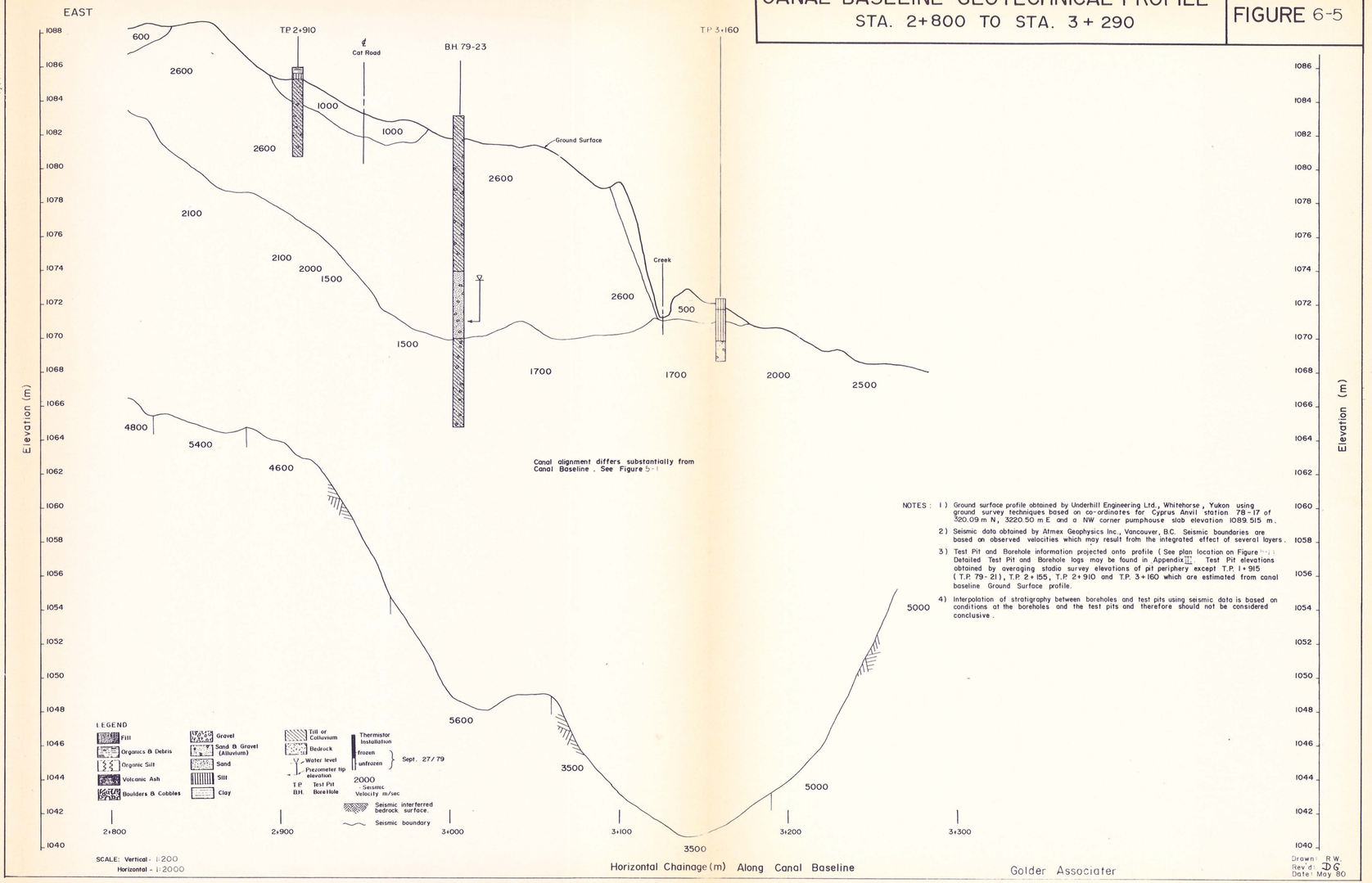
Date Jan, 1980

Project No 792-2025

CANAL BASELINE GEOTECHNICAL PROFILE
STA. 2+800 TO STA. 3+290

FIGURE 6-5



NOTES :

1) Ground surface profile obtained by Underhill Engineering Ltd., Whitehorse, Yukon using ground survey techniques based on co-ordinates for Cyprus Anvil station 78-17 of 320.09 m N, 3220.50 m E and a NW corner pumphouse slab elevation 1089.515 m.

2) Seismic data obtained by Atmex Geophysics Inc., Vancouver, B.C. Seismic boundaries are based on observed velocities which may result from the integrated effect of several layers.

3) Test Pit and Borehole information projected onto profile (See plan location on Figure). Detailed Test Pit and Borehole logs may be found in Appendix . Test Pit elevations obtained by averaging stadia survey elevations of pit periphery except T.P. 1+915 (T.P. 79-21), T.P. 2+155, T.P. 2+910 and T.P. 3+160 which are estimated from canal baseline Ground Surface profile.

4) Interpolation of stratigraphy between boreholes and test pits using seismic data is based on conditions at the boreholes and the test pits and therefore should not be considered conclusive.

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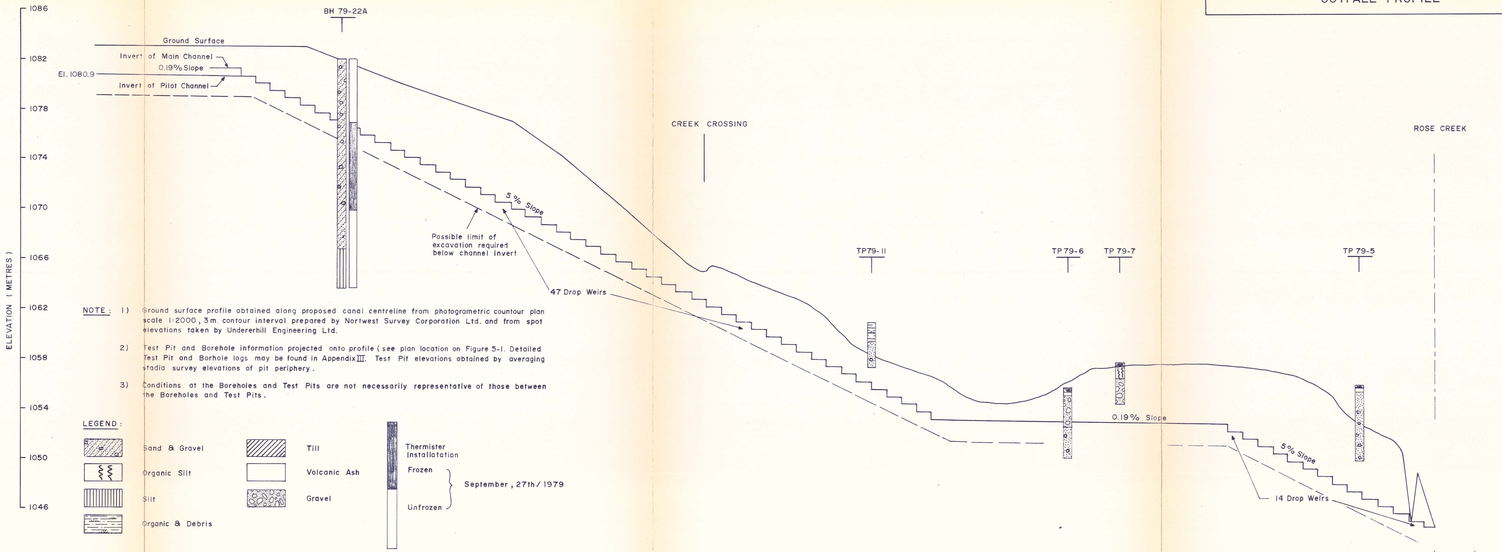
Rev'd: DG
Date: May 80

Project No. 792-2025

PROPOSED DIVERSION CANAL OUTFALL PROFILE

FIGURE 6-6



NOTE: 1) Ground surface profile obtained along proposed canal centreline from photogrametric countour plan scale 1:2000, 3m. contour interval prepared by Northwest Survey Corporation Ltd. and from spot elevations taken by Undererhill Engineering Ltd.

2) Test Pit and Borehole information projected onto profile (see plan location on Figure 5-1. Detailed Test Pit and Borhole logs may be found in Appendix III. Test Pit elevations obtained by averaging stadia survey elevations of pit periphery.

3) Conditions at the Boreholes and Test Pits are not necessarily representative of those between the Boreholes and Test Pits.

LEGEND:

- Sand & Gravel
- Organic Silt
- Silt
- Organic & Debris
- Till
- Volcanic Ash
- Gravel
- Thermister Installation
- Frozen
- Unfrozen

September, 27th/1979

APPROXIMATE HORIZONTAL CHAINAGE (METRES) ALONG PROPOSED CANAL CENTRELINE

SCALE = HOR. 1:2000 VERT. 1:200

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Drawn
Rev'd
Date May 1980

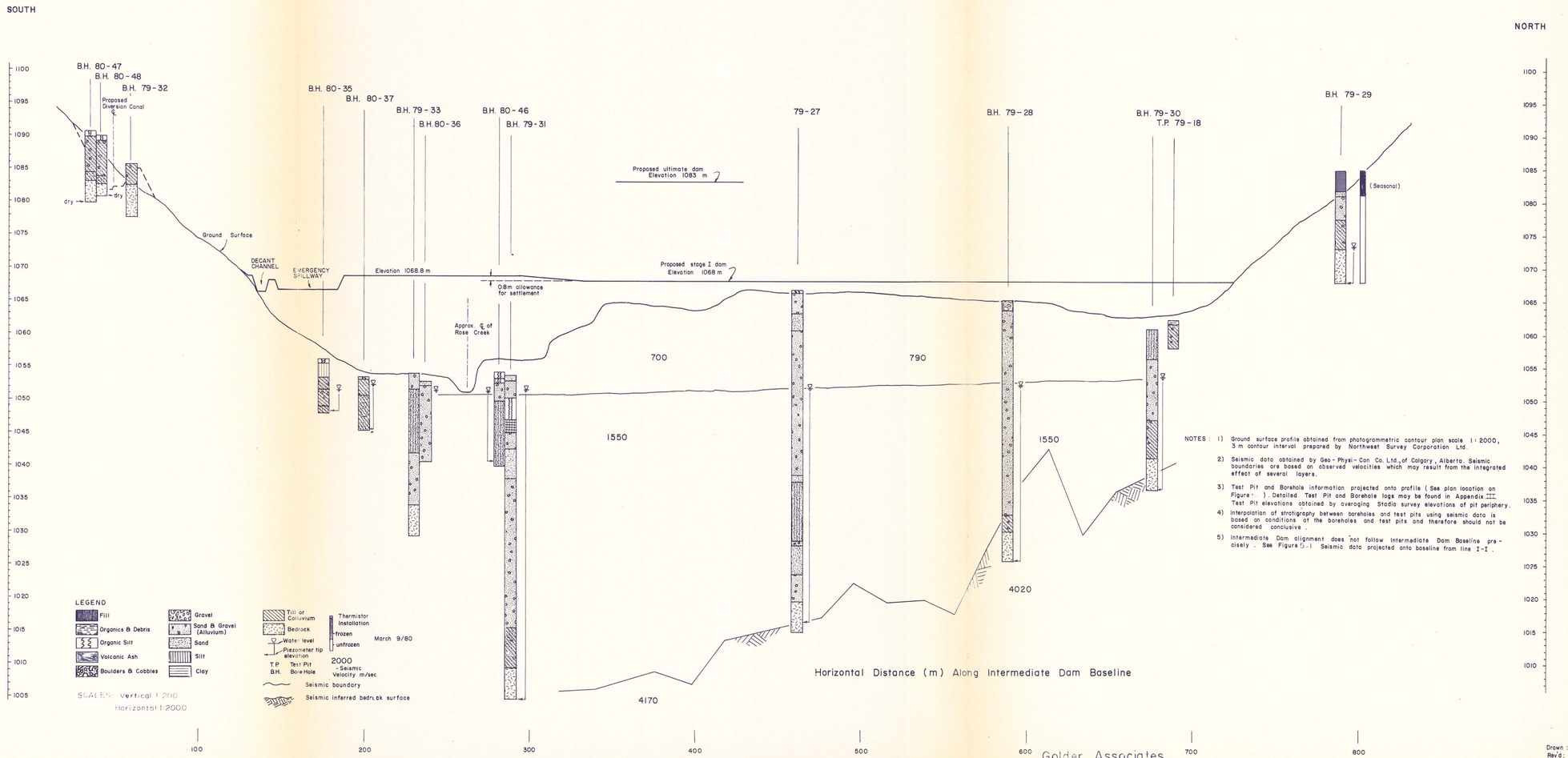
GEOTECHNICAL PROFILE ALONG CROSS VALLEY DAM BASELINE

FIGURE 6-7

Project No. 792-2025

GEOTECHNICAL PROFILE ALONG INTERMEDIATE DAM BASELINE

FIGURE 6-8



NOTES:
1) Ground surface profile obtained from photogrammetric contour plan scale 1:2000, 3 m contour interval prepared by Northwest Survey Corporation Ltd.
2) Seismic data obtained by Geo-Physi-Con Co. Ltd., of Calgary, Alberta. Seismic boundaries are based on observed velocities which may result from the integrated effect of several layers.
3) Test Pit and Borehole information projected onto profile (See plan location on Figure). Detailed Test Pit and Borehole logs may be found in Appendix III. Test Pit elevations obtained by averaging Stadia survey elevations of pit periphery.
4) Interpolation of stratigraphy between boreholes and test pits using seismic data is based on conditions at the boreholes and test pits and therefore should not be considered conclusive.
5) Intermediate Dam alignment does not follow Intermediate Dam Baseline precisely. See Figure 5-1. Seismic data projected onto baseline from line I-I.

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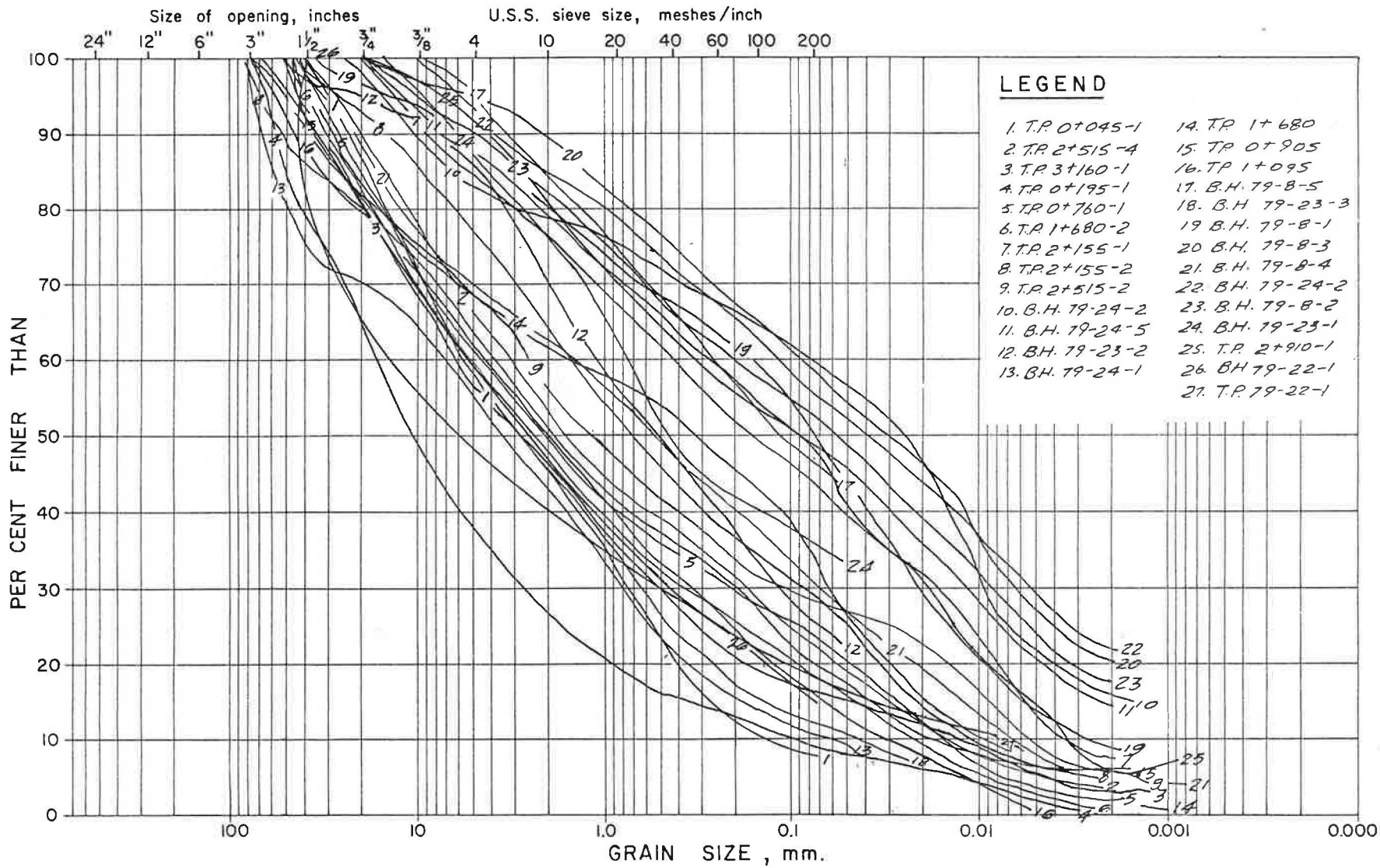
Date: May 1980

Project No. 792-2025 Drawn P.D. Reviewed DG Date 9 NOV. 1979

M.I.T. GRAIN SIZE SCALE



BOULDER SIZE	COBBLE SIZE	GRAVEL SIZE: coarse	GRAVEL SIZE: medium	GRAVEL SIZE: fine	SAND SIZE: coarse	SAND SIZE: medium	SAND SIZE: fine	fine grained: SILT SIZE	fine grained: CLAY SIZE

GRAIN SIZE DISTRIBUTION
DIVERSION CANAL

Figure 6-9

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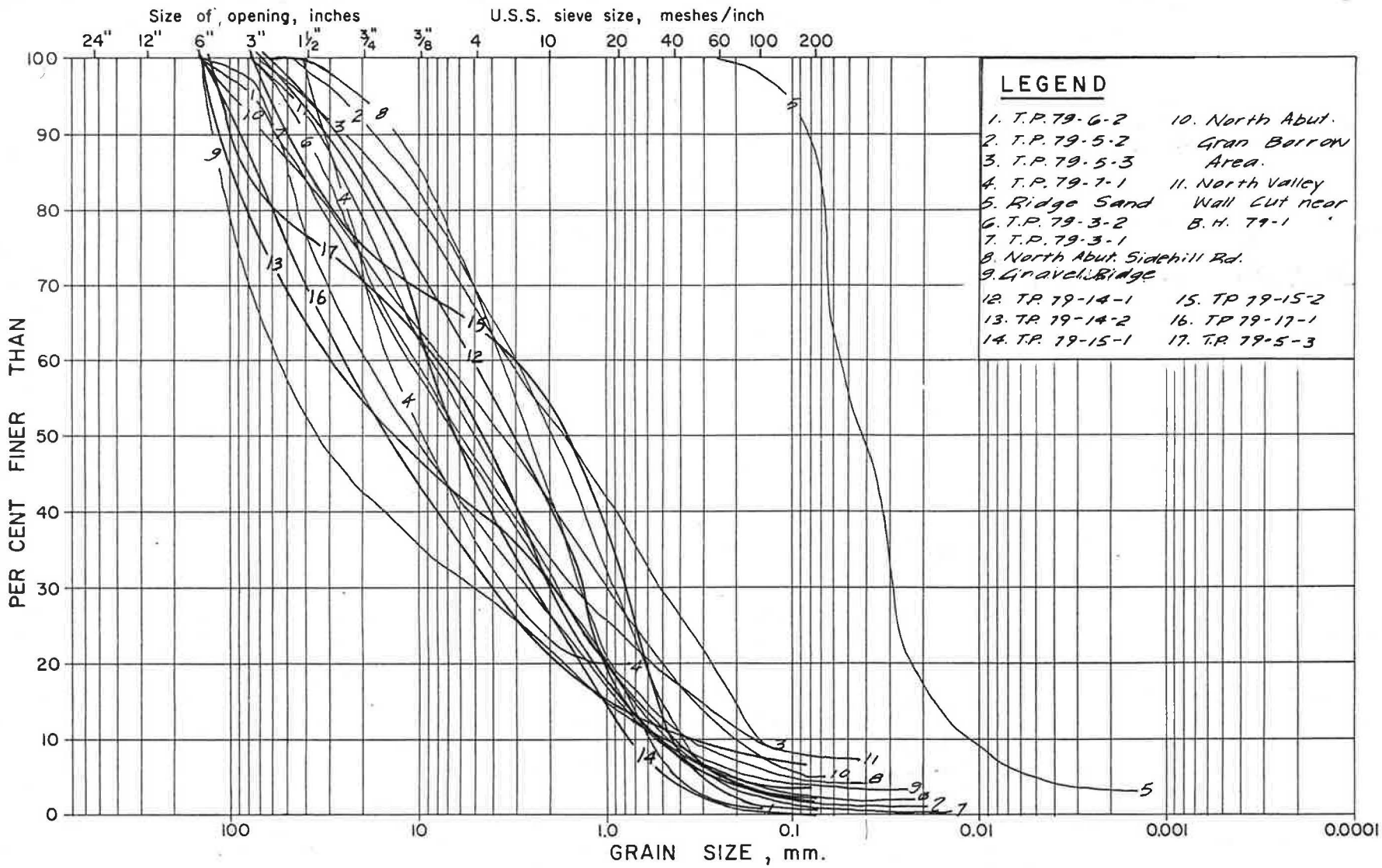
GRAIN SIZE DISTRIBUTION

GRAVEL BORROW AND OUTFALL GRAVELS

M.I.T. GRAIN SIZE SCALE



BOULDER SIZE	COBBLE SIZE	GRAVEL SIZE			SAND SIZE			fine grained	
		coarse	medium	fine	coarse	medium	fine	SILT SIZE	CLAY SIZE

Figure 6-10

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GRAIN SIZE DISTRIBUTION

CROSS VALLY AND INTERMEDIATE DAM FOUNDATIONS

M.I.T. GRAIN SIZE SCALE

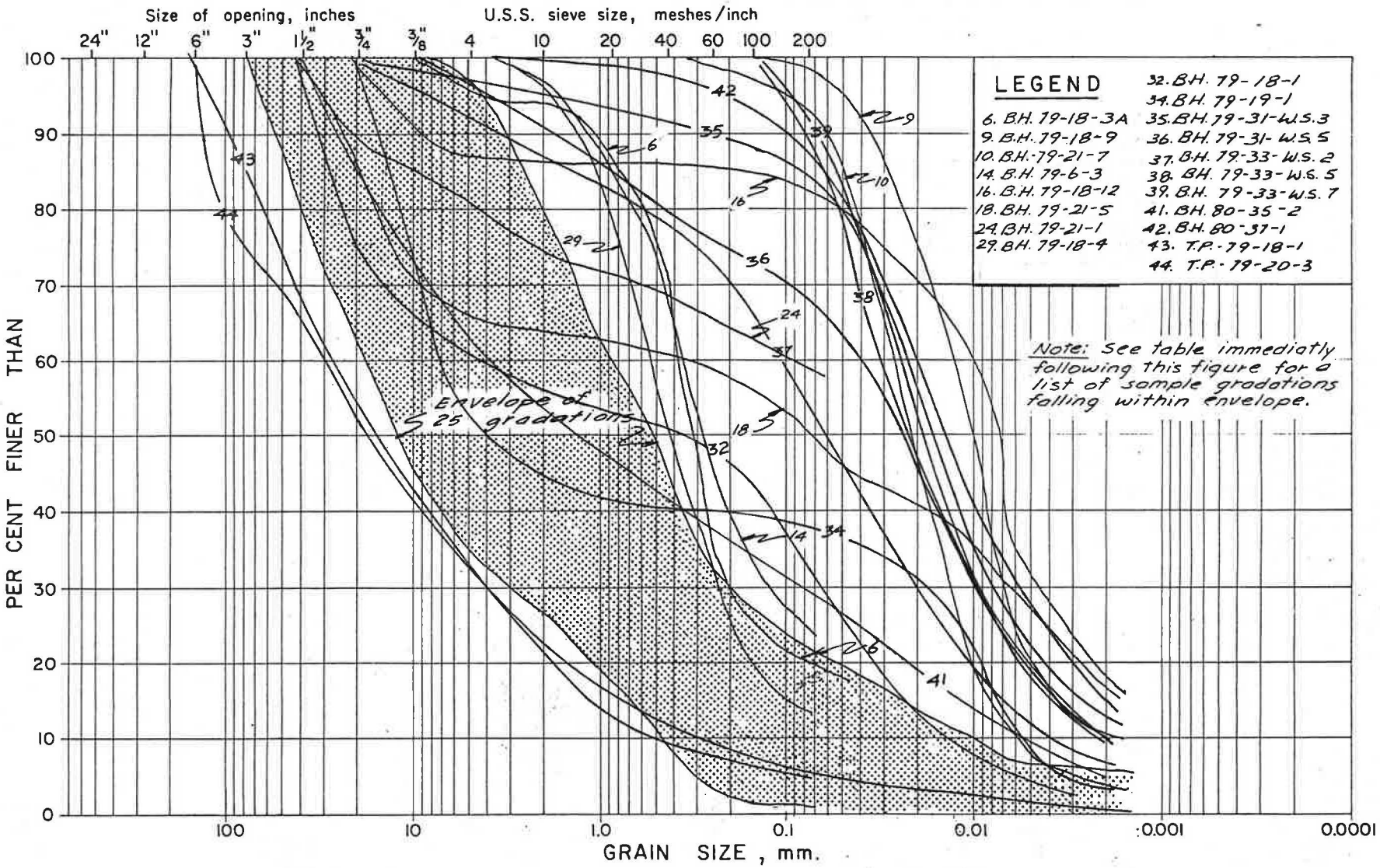

Figure 6-11

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LIST OF CROSS VALLEY AND INTERMEDIATE DAM SAMPLE GRADATIONS

List of Gradations in Envelope

1. BH-79-16-5
2. BH-79-7-13
3. BH-79-16-9
4. BH-79-16-7
5. BH-79-15-13
7. BH-79-18-3B
8. BH-79-15-12
11. BH-79-7-11
12. BH-79-7-5
13. BH-79-6-9
15. BH-79-18-15
17. BH-79-16-13
19. BH-79-21-12
20. BH-79-15-6&7
21. BH-79-18-6
22. BH-79-18-2
23. BH-79-16-2
25. BH-79-18-5A
26. BH-79-18-13
27. BH-79-18-5B
28. BH-79-15-3
30. BH-79-18-15
31. BH-79-16-15
33. BH-79-18-2
40. BH-80-35-1

List of Gradations Outside Envelope

6. BH-79-18-3A
9. BH-79-18-9
10. BH-79-21-7
14. BH-79-6-3
16. BH-79-18-12
18. BH-79-21-5
24. BH-79-21-1
29. BH-79-18-4
32. BH-79-18-1
34. BH-79-19-1
35. BH-79-31-WS3
36. BH-79-31-WS5
37. BH-79-33-WS2
38. BH-79-33-WS5
39. BH-79-33-WS7
41. BH-80-35-2
42. BH-80-37-1
43. TP-79-18-1
44. TP-79-20-3

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GRAIN SIZE DISTRIBUTION

CROSS VALLEY DAM - ABUTMENT MATERIALS

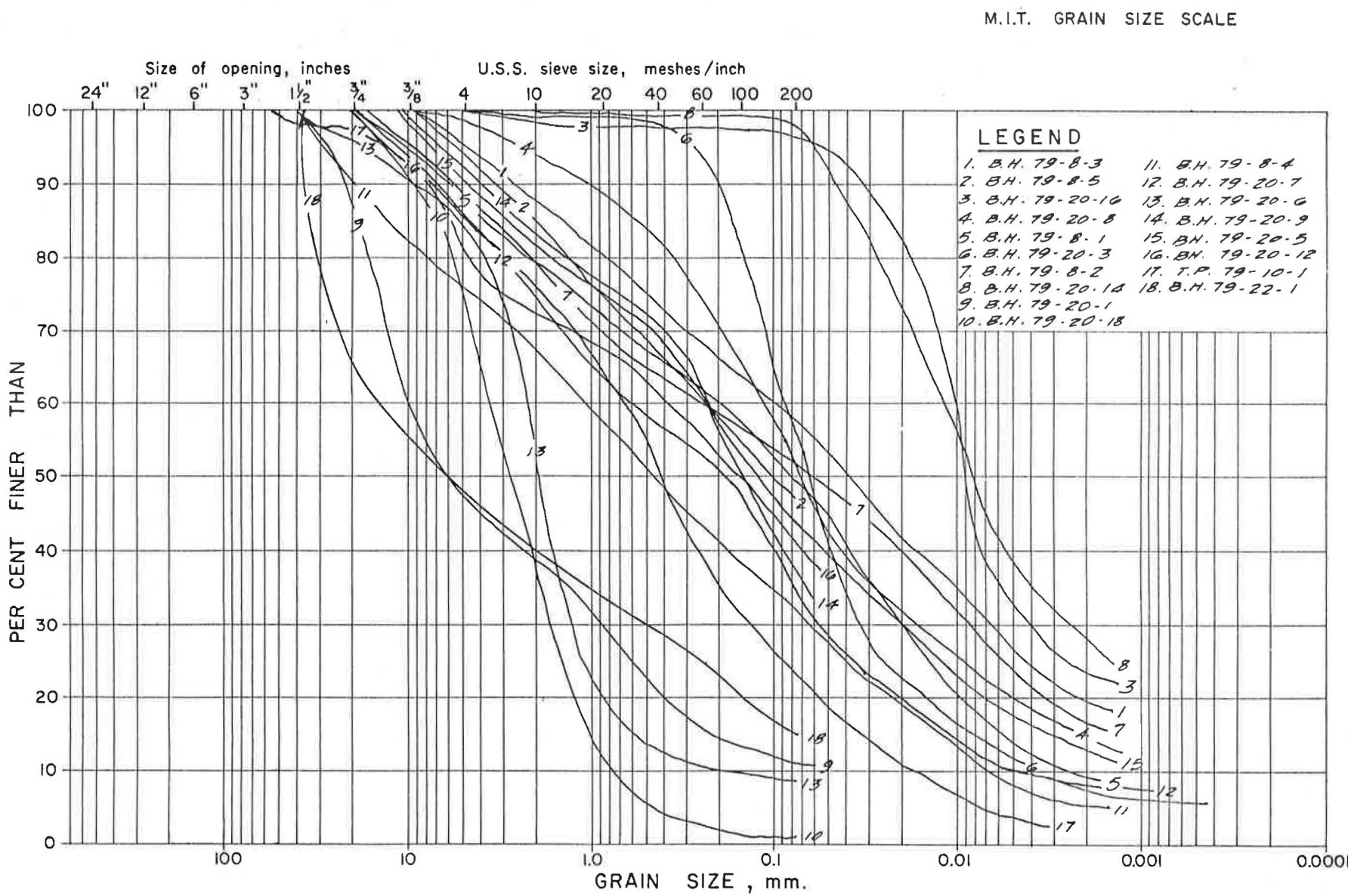

Figure 6-12

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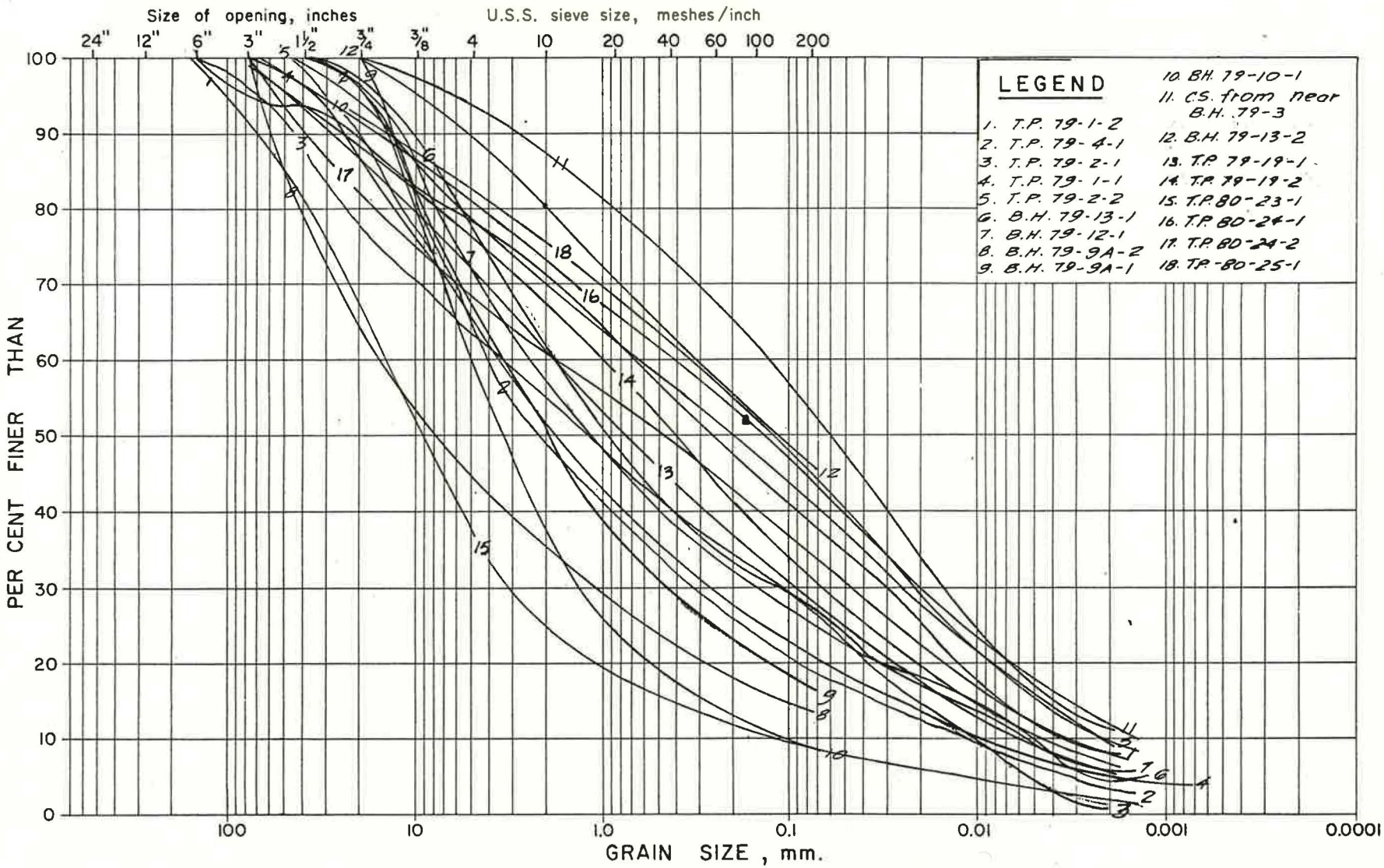
Project No. 792-2025 Drawn D.D. Reviewed DG Date 9 Nov. 1979

M.I.T. GRAIN SIZE SCALE



BOULDER SIZE	COBBLE SIZE	GRAVEL SIZE			SAND SIZE			SILT SIZE	CLAY SIZE
		coarse	medium	fine	coarse	medium	fine	fine grained	

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GRAIN SIZE DISTRIBUTION
TILL BORROW PIT

Figure 6 - 13

7.0 PROPOSED DESIGN AND CONSTRUCTION

Each of the elements of the Down Valley Project is located on Figure 7-1. The investigation profile baselines are also located on this plan for reference.

The proposed designs of each one of the project elements are presented here. It is noted that the technical support data for these designs may be found appended to this report as follows:

Appendix VI - Cross Valley Dam Design
Appendix VIII - Diversion Canal Geothermal, Seepage and Stability Considerations
Appendix IX - Hydrologic and Hydraulic Design
Appendix X - Earthquake Data, Peter Byrne Review

Important discussions included in these appendices pertinent to construction of the various project elements, to post construction monitoring and instrumentation, and to maintenance are not reiterated here for brevity.

7.1 Diversion Canal

The alignment of the proposed canal was chosen, based on the design assumption that it is desirable to keep the canal in cut as much as possible because postulated winter construction would probably preclude the building of dependable dykes. Sidehill canals are generally most economical if balanced cut/fill is attempted but such a solution is anticipated to result in unacceptable seepage losses through poorly compacted dykes. Therefore a 50 year flood in cut criteria was adopted to locate the canal route meaning that the centreline of the canal was located such that the downhill channel cut was 2.4 m with respect to the natural ground surface. Use of this criteria has resulted in a ratio of excavation volume to fill volume of about 4 to 1 and, in spite of this, some significant fills are required locally to maintain an acceptably smooth alignment. Where these fills are required it is recommended that

the dykes be constructed in thawing conditions to permit proper compaction. This constraint will mean that the diversion canal will not be functional before the spring flood. Alternatively it may be desirable to accept larger excavation volume or a more tortuous alignment and thus to avoid the potential seepage problems of locally shallow cuts and locally large fills.

The standing canal alignment is shown on Figure 7-1. It is expected that this alignment will require some modification when the final survey cross-sections have been obtained.

The proposed cross-sections for construction of the canal are presented on Figures 7-2, 7-3 and 7-4. The hydraulic cross-section of the canal is based on the recommendations provided by Hydrocon Engineering Ltd. and their report contained in Appendix IX for reference. With 1 m of freeboard the section is capable of carrying a 50 year design flood. With no freeboard the section is anticipated capable of passing a 500 year flood and this is referred to as the contingency design flood.

There are three possible basic cross-sections which are anticipated necessary to accommodate the expected range of soil and rock conditions along the route. These sections will reflect:

(i) full thermal protection only
(ii) seepage control and thermal protection
(iii) channel excavated in rock

The full thermal protection alternative shown on Figure 7-2 requires that a 1.7 m thick gravel blanket be placed against the 2 horizontal to 1 vertical design sideslopes. This gravel blanket is intended to insulate and thus reduce the rate of thaw of otherwise problematic permafrost. Reduced rate of thaw permits drainage to occur in which case the thawing soil retains strength. The gravel blanket also serves to filter locally ice rich pockets which would otherwise flow and possibly lead to

retrogressive degradation of the slope. Lining the diversion canal with gravel will also improve the erosion stability of the canal at most locations because it is normally coarser than the tills in which the channel will be cut.

At some locations along the canal route the materials are not continuously frozen or frozen ice rich. This does not preclude the use of the gravel blanket because areas which overlie these thaw-stable or thawed zones are usually ice rich and therefore require thermal and seepage filtering protection. Furthermore, permafrost may underlie thaw-stable zones. The practical solution to these broadly variable soil conditions is to simply blanket the entire cut slope with gravel and accept that, where thermal insulation and filtering are not critically important, some additional erosion protection is provided against both stream flow and down slope surface drainage.

The base of the excavation is generally flat and therefore virtually no soil shear strength is required to maintain stability. It is also noted that ice rich soils appear to be uncommon at depth. For these reasons the canal bottom blanket thickness is reduced to 0.8 m, consisting of 0.5 m of Class A rock for erosion control and 0.3 m of gravel filter between the rock and the natural soils.

The dyke which forms the downslope bank of the canal and is common to all 3 cross-section alternatives should be compacted according to a procedural specification. Relatively dry selected canal waste excavation should be used in these dykes and compacted immediately after excavation to take advantage of the relatively warm permafrost when compared to ambient air temperature. As stated above, where relatively high dykes occur the construction should be undertaken under thawing conditions.

The seepage control and thermal protection alternative shown on Figure 7-3 utilizes a 1 m thickness of compacted till as a seepage reduction liner. The need for such seepage control should be assessed

according to the seepage control criteria established in Appendix VIII, namely, that the canal should be lined in areas where the soil exhibits a permeability greater than 10^{-4} cm/sec. The lining till material should be selected from the waste excavation materials, should be unfrozen, should be placed in thawing conditions at about standard Proctor optimum moisture content (7.8 percent) and should be compacted to maximum standard Proctor dry density (2100 kg/m^3) in maximum lift thicknesses of 0.3 m. The till is considered suitable if it has a minimum of 20 percent by weight passing the No. 200 sieve.

Between Sta. 0+000 and 0+700 of the Diversion Canal occurrences of relatively coarse till or alluvial deposits have been noted. Although all laboratory gradations of these materials are such that a permeability lower than 10^{-4} cm/sec. is expected, it is possible that local occurrences of coarser materials will be exposed during excavation that will require lining. For this reason it is important to monitor the excavation materials. Of principal concern are the bedrock occurrences which may form the bottom of the canal channel. While the seismic refraction surveys have generally indicated the rock to be of good quality, some disturbed and jointed bedrock has been noted. The area in the vicinity of the prominent creek at Sta.0+600 is one area where lining may be required. Others include those areas where rock may be encountered within the depth of excavation, namely:

(i) in the vicinity of the canal inlet
(ii) Sta. 0+050 to 0+230
(iii) Sta. 0+330 to 0+350
(iv) Sta. 0+500 to 0+820
(v) Sta. 1+490 to 1+600
(vi) Sta. 1+940 to 1+980
(vii) Sta. 2+200 to 2+980

As an alternative to till lining in the rock, it is also possible that grouting of the fractures would successfully control seepage. The

success of this labour intensive technique will depend on the degree of rock disturbance. Grouting operations should be completed in nonfreezing conditions.

If the base of the canal excavation is ice rich or extensive ice lensing occurs within the 50 year design flood depth, seepage losses may become a concern upon thaw. In such cases it would be adviseable to construct the seepage control cross-section (as discussed previously and during thawing conditions). Over-ripping of the downslope cut bank of the canal is another undesirable situation which may require the placement of a till blanket on the downslope cut face.

Where intact bedrock is encountered the excavation sideslopes may be steepened to 0.5 horizontal to 1 vertical as shown on Figure 7-4. If the rock is disturbed and fractured either the seepage control cross-section described above will be required, necessitating flattening of the sideslopes to 2 horizontal to 1 vertical, or the rock may be grouted if a successful product results.

Where the depth to bedrock is uncertain the canal should be staked as though seepage control or thermal protection is required. When bedrock is encountered a bench may be exposed and the excavation completed with steeper sidelopes. If poor quality rock is revealed the possibility remains that the rock bench can be cut back and lined with till. The rock excavation will require blasting and it is important that the rock not be unduly disturbed outside the excavation for reasons of seepage loss control and possible deficient stability.

It is probable that the canal lowside cutslope will expose bedrock having some planes of weakness dipping into the excavation. The attitudes of these planes of weakness are not known but it is expected that the 0.5 horizontal to 1 vertical sideslopes can be used at the hazard of having only small block failures which would daylight on the exposed bench where

rock pick up could be treated as a maintenance item if it occurs after construction is complete.

If there is no doubt that the upside cut of the canal can be made in intact rock below the 50 year flood level it may be possible to stake the upside cut according to rock cut only. Furthermore, cut slopes steeper than 0.5 horizontal to 1.0 vertical may be acceptable on the upslope side of the canal.

Along the route of the proposed canal there are 2 relatively major valley wall creeks which must be intercepted. These are at Sta. 0+600 and 3+100. The flow in these creeks varies substantially from more than 0.2 m^3/sec in the spring and summer down to virtually zero flow at certain times in the winter. The proper interception of these flows is important because sizeable glaciations across the diversion channel might eventually choke off the Diversion Canal. In general, the interception design approach should be to prevent heat loss from the flowing water and to keep velocities high and the flow concentrated. Possible designs for these interceptions are presented in Appendix IX where it is also recommended that the interceptions be field engineered.

Appendix IX also discusses the occurrence of relatively small and intermittent valley wall streams and possible designs for their interception. It is noted that one of these streams occurs in the vicinity of the test excavation and that, in the course of construction, a slope icing up to 0.4 m thick was encountered in the vicinity of Sta. 1+900. No special interception installations were made and the behaviour of the excavation in this area should better define the need for and requirements of small stream interceptions. Artificial stimulation of glaciation uphill of the proposed canal could serve as a short term solution if canal icing problems develop and this would permit evolution of optimal interceptions as necessary.

Erosion control in the canal along the 0.19 percent grade reaches should be provided by lining the channel bottom and wells to the 500 year

flood elevation with Class A riprap as discussed in Appendix IX. Initially it was intended to windrow cobbles from the gravel fill lining and place these on the channel slopes but this operation was found to be somewhat unsuccessful when constructing the test excavation. The resulting riprap was not continuous and the gravel borrow pit was sufficiently variable to render impractical the generation of slope protection by this method. Furthermore, gravel segregation was not possible on the bottom of the canal.

Grade control buried weirs or sills are also discussed in Appendix IX. The necessity and frequency of these structures will be determined in the field; they are recommended where the distance between canal rock excavations exceeds 250 m.

The proposed drop weir and rock lining to be used in the canal outfall is illustrated on Figure 7-5. The design shown is considered to be a practical extension of the recommendations of Appendix IX and the weir section is considered to be more robust than that which is now employed for the smaller flood flows carried by the existing channel. Parts of the canal outfall will be excavated in permafrost and generally the rock fill generated will serve as an insulation substitute for gravel. Nevertheless, it is particularly important to place a gravel filter between fine grained ice rich soils exposed by the excavation and the rock filter lining to ensure that sinkholes do not develop through unwanted internal erosion of the channel walls.

7.2 Cross Valley Dam

Design considerations for the construction of the Cross Valley Dam are documented in Appendix VI. The proposed cross-section is shown on Figure 7-6.

The Cross Valley Dam will be constructed to its final design crest elevation of 1065 m during the summer 1981 Stage I construction. This will provide a water storage between the Cross Valley and Intermediate

Dams of about 1.4 million m^3, equivalent to about 60 days of inflow at 0.27 m^3/sec.

The alignment of the Cross Valley Dam has been shifted downstream in the south abutment area from that which was initially designed. This has become possible because the final crest elevation of the dam has been reduced and desirable because ice rich permafrost tends to feather out as one moves downstream of Borehole No. 79-18. The new alignment reduces significantly the amount of difficult frozen foundation subexcavation required. Extrapolation of the February, 1980 borehole information suggests a fine-grained permafrost limit as shown on Figure 7-1 and suggests that relatively minor amounts of permafrost removal will be required near the upstream toe of the dam.

The proposed cross-section is a zoned earthfill dam with 2 horizontal to 1 vertical upstream and downstream slopes and consists of an upstream seepage control blanket and a central till core extending below ground surface by 3 m to cut off any near-surface abandoned creek beds or other pervious zones. The proposed dam will be constructed under spring/summer conditions and all materials placed above water level must be compacted in maximum 0.3 m lifts. Materials placed below water level should be placed in a single lift with precautions being taken to ensure that loose material is displaced at the advancing face of the fill and that post construction settlement is minimized. Where two fills are advanced in water towards one another it is mandatory to make provisions before closure for removal of the accumulated loose material which has been displaced by the advancing fill fronts.

The depth of foundation preparation required will be determined in the field. To avoid costly well point dewatering of the excavation, it is proposed that the subexcavation be done in the wet with as much dewatering as possible with simple sumps and that the subexcavation materials be carefully monitored by a qualified engineer as they are excavated.

The amount of abutment preparation required will also be field determined. Excavation of ice rich frozen soils within the foundation area of the dam will be necessary and gravel blanketing of the cut slopes and permafrost area 20 m downstream of the dam toe is recommended.

The upstream blanket has been divided into primary and secondary components. The primary component will be a good quality till properly compacted and constructed above water. The secondary component will consist of initially frozen waste excavation from the Diversion Canal, compacted as a single lift after thaw using the most practicable means available.

Upstream of the core will be a granular zone which, although not excessively coarse grained, must be as cohesionless as possible so that any core cracks will be healed by seepage induced transfer of this cohesionless material. Materials which best meet this requirement are silty sands which occur locally, such as in the gravel ridge deposit.

A filter zone downstream of the core is intended to prevent piping of core materials into segregations of the coarse material of the adjacent zone. The filter material should be clean and sandy and devoid of cobbles and coarse gravel.

Shell Type I and Shell Type II materials should be considered as general fills, the difference between them being that Type I materials may be fine grained whereas Type II should contain less than 5 percent of minus No. 200 sieve. Type I materials could consist of reject core borrow. Type II materials would be terrace sand and gravel.

In view of the nature of the Cross Valley Dam foundation, settlements of about 0.8 m are expected. Such a settlement would be expected to occur over most of the length of the dam and therefore a uniform dam crest elevation of 1065.8 m has been selected for construction.

Assuming the seepage control elements discussed above are in place, the seepage through and beneath the Cross Valley Dam is computed to be

about 0.03 m^3 /sec. If the secondary blanket is ruptured the seepage loss would be about 0.04 m^3/sec. These computed seepage flows are judged indicative of acceptable seepage reduction in view of the allowable design quantity of 0.15 m^3/sec. Precise prediction of seepage losses from the retention pond are not possible because the local stratigraphy and geology of the dam foundation are not known with complete certainty. For example, the occurrence of abandoned creek beds at depth below the seepage cutoff elevation cannot be precluded based on existing knowledge of valley bottom conditions in the vicinity of the Cross Valley Dam and its associated upstream blanket. And such abandoned creek beds could also daylight upstream of the proposed upstream blanket. The seepage estimates documented in Appendix VI are derived from an assessment of permeability based on experience with other water retaining structures involved in Cyprus Anvil's tailings disposal, the gradation of samples recovered in the field investigation, and from the information obtained when drilling air rotary boreholes.

7.3 Intermediate Dam

The Intermediate Dam differs from the Cross Valley Dam because its primary purpose is to retain tailings. It is expected that the dam will be built in two or more stages with the design crest elevation of the first stage being 1068 m. The proposed ultimate elevation is to be 1083 m, the retained tailings pond being lower than the invert elevation of the diversion canal at this location.

Because the Intermediate Dam will be constructed in at least two stages and a tail water will develop on the downstream side as a result of the retention pond between the Intermediate and Cross Valley Dams, the foundation of the ultimate Intermediate Dam must be completed to El. 1064 m to allow subsequent construction activity. All materials must be compacted according to the specifications for dam fill placement. The extent of ultimate section construction assumes that the ultimate dam will be raised using downstream construction techniques with a downstream slope of 2 horizontal to 1 vertical. The initial construction must assume

this design approach although it is conceivable that a centreline construction could be used if tailings are spigotted against the upstream face of the dam. This latter method is presently being used for raising the existing dam and is quite economical because readily available tailings materials are used to form a significant proportion of the embankment volume and to keep the beach edge well away from the face of the constructed section.

Cross-sections of the proposed Intermediate Dam are presented on Figures 7-7 and 7-8. In general, the remarks stated previously for the Cross Valley Dam concerning fill placement and construction also apply to the Intermediate Dam. The proposed cross-section in the vicinity of Rose Creek (Figure 7-7) is similar to that recommended for the Cross Valley Dam except that the base raising for the ultimate dam is required and the upstream seepage control blanket is not required. The recommended construction crest elevation for this reach of the dam is 1068.8, allowing 0.8 m of settlement which will result from compression of the embankment, its foundation, and the foundation subexcavation replacement materials. As for the Cross Valley Dam, it is expected that foundation subexcavation below creek level will be necessary to remove unacceptable recent alluvium and organic debris.

Figure 7-8 presents the recommended Intermediate Dam cross-section in the upper level terrace reach. The seepage requirements of the Intermediate Dam, namely that no more than 0.23 m^3/sec should seep through the dam and its foundation, dictate that seepage control measures be installed in the terrace area down to about creek elevation. Rather than place a central core within the terrace gravels and the associated excavation and replacement of much of the dam foundation, it is recommended that a 5 m wide inclined blanket of glacial till be placed against the trimmed wall of a terrace gravel borrow pit excavated such that the till blanket can be easily incorporated into the dam section on the top of the terrace as shown on Figure 7-8. The borrow pit wall should be trimmed back to a 2 horizontal to 1 vertical slope to avoid disturbance and loosening of the natural terrace deposits and to permit replacement of unsuitably coarse materials adjacent to the till.

It is expected that the Intermediate Dam will be constructed as soon as possible to allow disposal of tailings in the new facility and so that the closure of the Cross Valley Dam can be effected in relatively dry conditions. This is relevant to the performance of the inclined till blanket because freezing and dessication of the till will reduce its value as a seepage reduction zone.

Because the Intermediate Dam in the upper terrace area will be completed to a relatively low height and no permafrost is present, very little crest settlement is anticipated and the proposed construction elevation is 1068 m.

The remarks made above regarding the abutments of Cross-Valley Dam are also appropriate to those of the Intermediate Dam.

7.4 Emergency Spillways and Decants

Details of the proposed emergency spillways are presented on Figures 7-9 and 7-10 according to the hydraulic recommendations of Appendix IX. The profile and cross-sections shown pertain strictly to the Cross Valley Dam but similar construction is proposed for the Intermediate Dam. The locations of the spillways are shown on Figure 7-1. Tailings wastewater decants are also shown adjacent to the emergency spillways. The purpose of the emergency spillway is to provide a controlled dam overflow capability consequent to possible temporary inability of the diversion channel to totally convey the Rose Creek flows.

The spillways consist essentially of an abutment sand and gravel fill downstream of the dam embankments. The fills are graded at a 5 percent slope until the retention pond elevation is reached in the case of the Intermediate Dam spillway or until the natural ground surface is reached for the Cross Valley Dam. The decant channels are incised into the top of the spillway fills and should be continuously riprapped within their channel sections.

The spillway crest is designed to be non-erodible during the 50 year return Rose Creek flood. Some erosion of the unprotected fill forming the

5 percent outfall should be expected but the riprapped spillway crest weir is considered to adequately protect the main dam embankment. Guidebanks along the spillway sides should also be riprapped.

7.5 Diversion Dam

The dam necessary to achieve redirection of the Rose Creek flow to the new channel is located on Figure 7-1 and the proposed cross-section is shown on Figure 7-11. The diversion dam will likely be constructed in early July, 1981 after the new diversion canal is completed. This relatively small dam will probably be constructed in the wet across the existing channel of Rose Creek. Zoned earthfill would be dumped as necessary upstream of a starter rockfill dyke to reduce the seepage through and around the dyke to acceptable levels. Embankment fills outside the closure section and above the water level in the closure section should be compacted according to normal dam specifications.

The rockfill aspect of the closure section shown on Figure 7-11 is not necessary outside the closure section. The south end of the diversion dam will be continuous with the downhill dyke of the diversion canal. The north end will abut the tailings dyke of the existing tailings facility.

The design elevation of the Diversion Dam should be marginally lower than the diversion canal dyke elevation. The Diversion Dam can then be used as a fuse plug protection for the Diversion Canal in the event of larger than design floods occurring. Therefore it is recommended that the Diversion Dam be constructed to an elevation such that the desired flow capacity of the diversion channel will be utilized and the excess flow will overtop the diversion dam. The surface of the diversion dam should therefore be riprapped.

If necessary, the Diversion Dam can be easily raised at the end of tailings disposal and when the performance of the canal will have been adequately established.

7.6 North Valley Wall Interceptor

The required hyraulic cross-sections for a 50 year return period flood are presented on Figure 8, Appendix IX. The alignment of the proposed interceptor is shown on Figure 7-1.

Because most of the North Valley wall overburden is unfrozen there is no concern given here to lining the channel section. It is recommended that the proposed cross-sections in Appendix IX be constructed without armouring because it is believed that channel erosion can be treated as a routine maintenance item, and that the channel will be fully stabilized by the end of mining.

It is noted that much of the alignment follows existing creek beds and that channel excavation is required only to transfer flow westward from creek to creek. Small diversion structures will be required to divert the flows from these creek beds as indicated on Figure 8, Appendix IX.

It is proposed that the North Valley Wall Interceptor join the decant outfall of the Cross Valley Dam just beyond the centreline of this dam as shown on Figure 7-1. This will require a significant cut into permafrost in the North Valley wall which should be lined with gravel for filter and thermal insulation purposes.

7.7 Post-Construction Maintenance

Earthworks construction is traditionally accompanied by a future need for maintenance and the usual objective is that the amount of maintenance decrease with time as the works become fully attuned to the local environment. The amount of maintenance is related to the degree of conservatism adopted in design and this is partly reflected by design factors of safety. However, some aspects of design are not easily quantified and both climatic factors and variability of construction material play an important role. The final design thus becomes a melding

of quantitative aspects, experience, judgement and the ability to provide maintenance on a timely basis.

Previous involvement with Cyprus Anvil's tailings facility at Faro has demonstrated that a maintenance free design is very conservative and is not generally warranted because the tailings facility can be routinely inspected by responsible mine staff who can initiate maintenance as necessary. For this reason the final design of the Down Valley Project is not a maintenance-free design. The objectives have been to provide a safe design for the project, to advantageously use available materials, and to utilize maintenance to attune the works to the environment. Decreasing annual maintenance requirements are anticipated, as has been the case with the existing tailings containment complex.

Other factors making an acceptance of maintenance appropriate pertain to the natural geological variability of materials, particularly permafrost, and the practicability of restraining the number of design variations necessary for response to the variable geology. Furthermore design compromise is necessary to accommodate practical construction. For example, the intention that the proposed diversion canal be constructed in winter conditions introduces the desirability for compaction of materials under subzero temperatures. The success of this compaction is contingent on weather, construction procedure, and variability of excavation materials, all of which can only be specified, identified, or controlled within practical limits. The need for good compaction also depends on the location within the dyke configuration.

We expect that, for certain sections of the canal dyke, acceptable compaction within the winter months will not be possible and that some remedial work will be necessary during the thawing months for the first few years of operation. Remedial work could involve reworking of poorly compacted materials to the extent of 40,000 m^3.

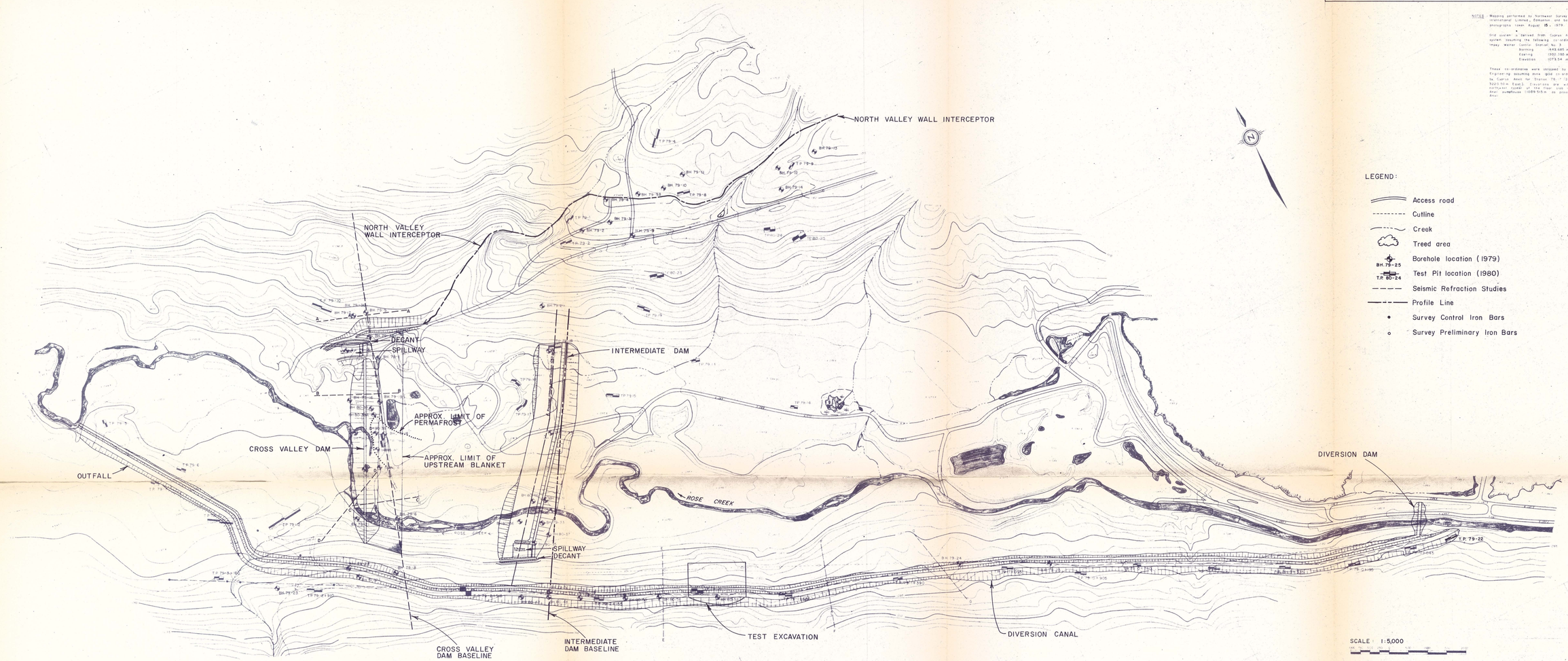
Other maintenance items which we expect pertain to degradation of permafrost along the canal and erosion of the canal, the dam spillways, the decant channels, and the North Valley Wall Interceptor. The canal has been designed to accommodate permafrost thaw at acceptable rates but settlements related to consolidation of ice rich fine-grained materials will inevitably occur on a local basis. Although the proposed canal is to be completely armoured, we anticipate a need to react to local conditions of glaciation and erosion, particularly in the vicinity of valley wall creek interceptions where some maintenance experience will be gained during the early operation of the canal. The emergency spillways and decants have been designed to protect the dam cross-sections from erosion by short term large flows over the dams and some repair of these structures may be recommended after such flows occur. Periodic upgrading of the decant channels which are continuously exposed to erosion may also be necessary on a scale comparable to the maintenance of the existing decant channel. The North Valley Wall Interceptor should be allowed to develop a natural armouring just as the existing creeks have done in the past. Maintenance should therefore only involve correction of problems which could lead to breaching.

The dams are expected to perform much as the existing dam. The North Abutment of the existing West Dam consists of frozen tills and colluvium and it has performed acceptably to date. The abutments of the new dams should behave similarly with allowance for gravel blanketing of the slope downstream of the dam as necessary. It is expected that thaw of permafrost will occur upstream of the Cross Valley Dam and that the secondary upstream blanket will suffer some loss of integrity in consequence; this should not affect the dam performance except that seepage quantities would be expected to increase with time. The seepage at the downstream toe of the Cross Valley Dam should be monitored with respect to quantity and turbidity and a gravel filter could be required.

Although the true extent of remedial work will only become apparent after the project is in operation we do not anticipate extensive maintenance other than that documented above. It is important to note that all of these maintenance items can be rectified within the proposed remaining 20 to 30 year operating life of the mine and most of the maintenance needs will have occurred and will have been dealt with within the first five years. Maintenance required as a result of higher than design floods and other unusual occurrences is beyond the scope of this discussion.

PROJECT LOCATION PLAN

FIGURE 7-1



NOTES: Mapping performed by Northwest Survey Corporation International Limited, Edmonton and based on photographs taken August 15, 1979.

Grid system is derived from Cyprus Anvil mine grid system, assuming the following co-ordinates for control impact Water Control Station No. 3:

Northing 443,685 m
Easting 1902.180 m
Elevation 1073.54 m

These co-ordinates were obtained by Underhill Engineering assuming mine grid co-ordinates supplied by Cyprus Anvil for Station 78.1+ 1222.03m North, 1670.17m East, 1.73m fore and aft with respect to the North Portal and 1.73m high on house and Cyprus Anvil surveyor (1089.515m as provided by Cyprus Anvil.

LEGEND:

- Access road
- Cutline
- Creek
- Treed area
- BH.79-25 Borehole location (1979)
- T.P. 80-24 Test Pit location (1980)
- Seismic Refraction Studies
- Profile Line
- Survey Control Iron Bars
- Survey Preliminary Iron Bars

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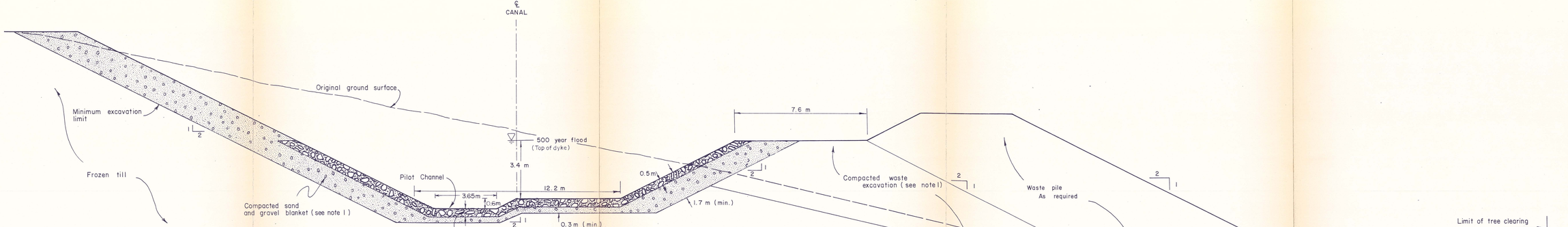
Rev'd JDE
Date 22 May 80

Project: 792-2025

PROPOSED DIVERSION CANAL CROSS-SECTION FULL THERMAL PROTECTION ONLY

FIGURE 7-2



NOTE : 1) Excavation and gravel lining of slopes to be done under freezing conditions. Procedural specifications to be assigned for compaction of frozen fill.

2) This drawing to be read in conjunction with the text of Golder Associates Final Design Report.

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Date: May 1980

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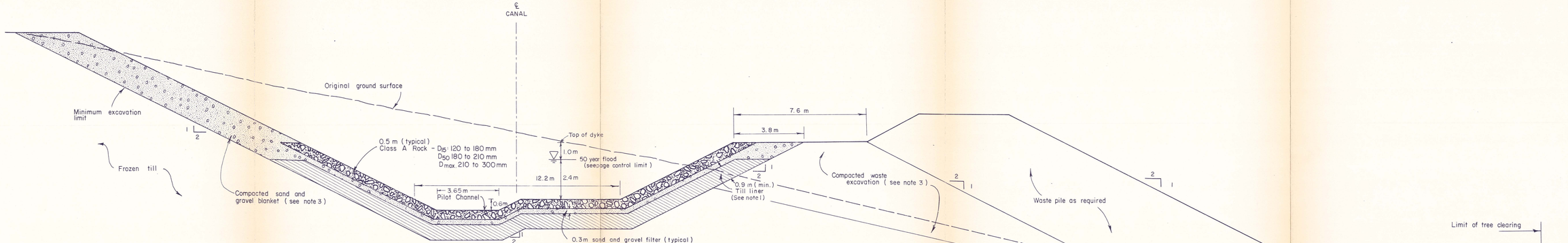
PROPOSED DIVERSION CANAL CROSS-SECTION SEEPAGE CONTROL AND THERMAL PROTECTION

FIGURE 7-3



NOTE: 1) Till seepage liner to be compacted to Standard Proctor maximum dry density under thawing conditions.

2) This drawing to be read in conjunction with the text of Golder Associates, Final Design Report.

3) Procedural specifications to be assigned for compaction of materials other than till seepage liner.

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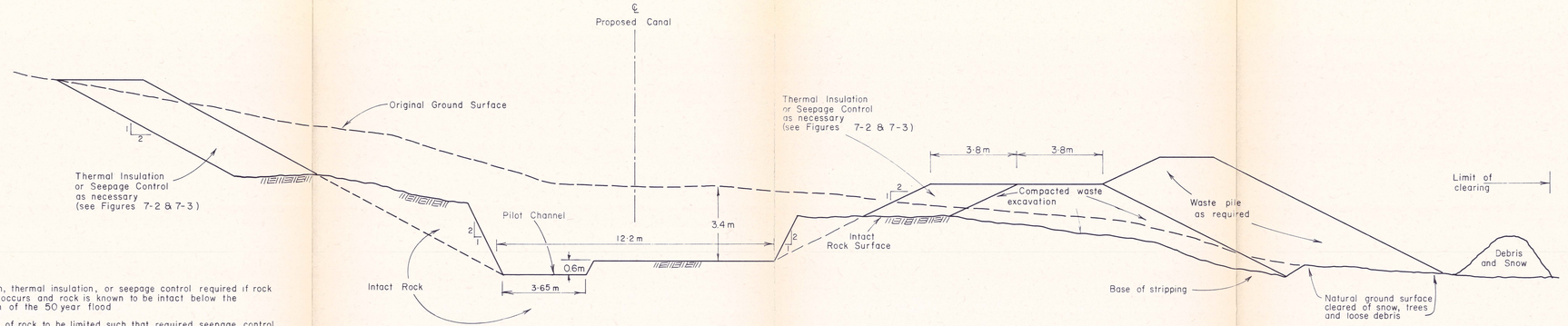
PROPOSED DIVERSION CANAL CROSS-SECTION INTACT ROCK EXPOSURES

FIGURE 7-4



NOTES:

1. No bench, thermal insulation, or seepage control required if rock outcrop occurs and rock is known to be intact below the elevation of the 50 year flood
2. Blasting of rock to be limited such that required seepage control or thermal insulation elements may be constructed as shown, and such that integrity of the intact rock forming the channel is maintained
3. All fractured or broken bedrock forming the walls of the excavation to be treated or lined to the satisfaction of the Resident Engineer
4. This drawing to be read in conjunction with the text of Golder Associates' Final Design Report

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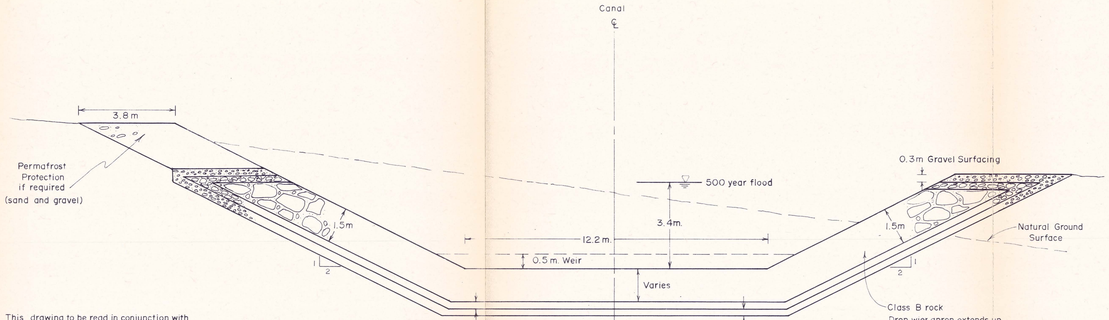
Date May 1980

Project No. 792-2015

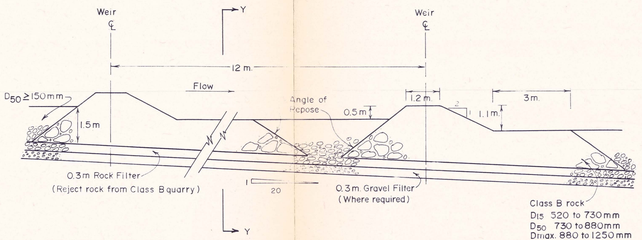
PROPOSED DROP WEIR SYSTEM

FIGURE 7-5



CROSS SECTION Y-Y

Note: This drawing to be read in conjunction with the text of Golder Associates Final Design Report.



TYPICAL DROP WEIR PROFILE

SCALE: 1:100 Horizontal and Vertical

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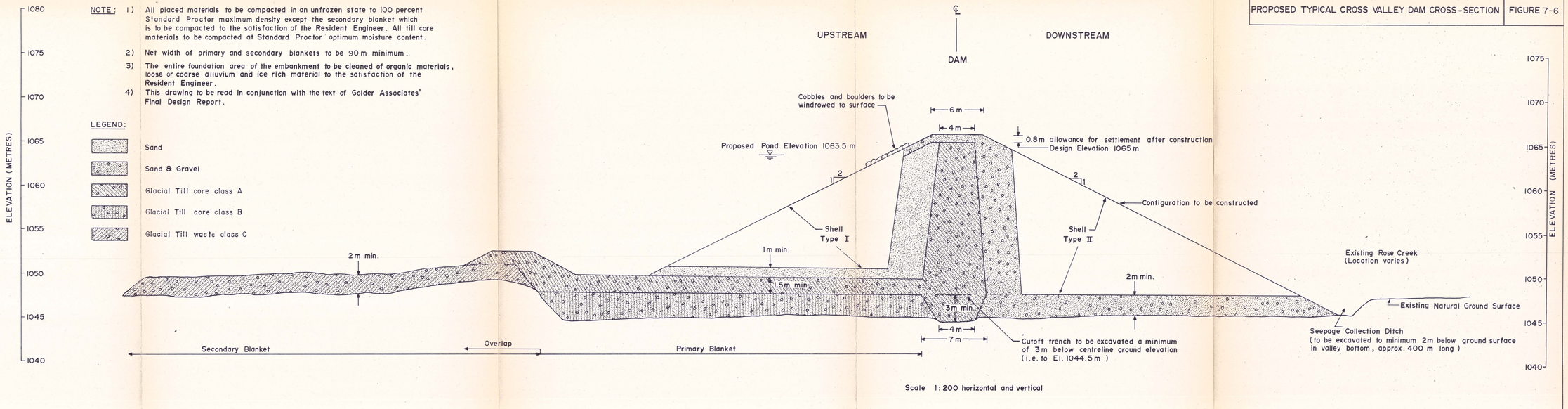
PROPOSED TYPICAL CROSS VALLEY DAM CROSS-SECTION

FIGURE 7-6

NOTE:
1) All placed materials to be compacted in an unfrozen state to 100 percent Standard Proctor maximum density except the secondary blanket which is to be compacted to the satisfaction of the Resident Engineer. All till core materials to be compacted at Standard Proctor optimum moisture content.
2) Net width of primary and secondary blankets to be 90 m minimum.
3) The entire foundation area of the embankment to be cleaned of organic materials, loose or coarse alluvium and ice rich material to the satisfaction of the Resident Engineer.
4) This drawing to be read in conjunction with the text of Golder Associates' Final Design Report.

LEGEND:

- Sand
- Sand & Gravel
- Glacial Till core class A
- Glacial Till core class B
- Glacial Till waste class C



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PROPOSED TYPICAL INTERMEDIATE DAM CROSS-SECTION IN ROSE CREEK VICINITY

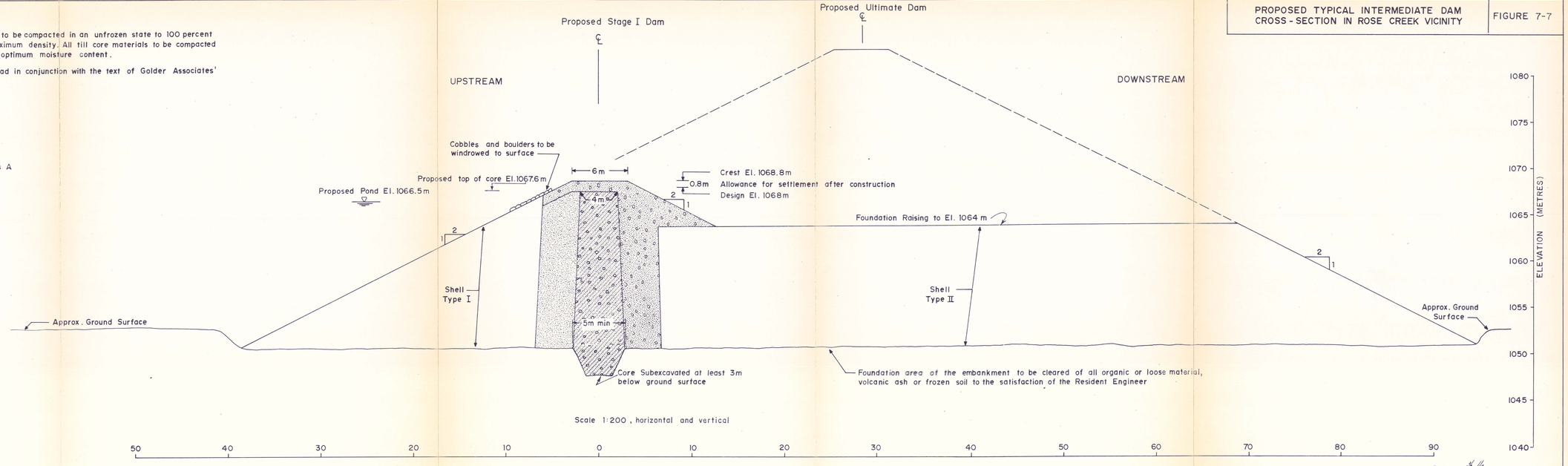
FIGURE 7-7

NOTE: 1) All placed materials to be compacted in an unfrozen state to 100 percent Standard Proctor maximum density. All till core materials to be compacted at Standard Proctor optimum moisture content.

2) This drawing to be read in conjunction with the text of Golder Associates' Final Design Report.

LEGEND:

- Sand
- Sand & Gravel
- Glacial Till core class A



Scale 1:200, horizontal and vertical

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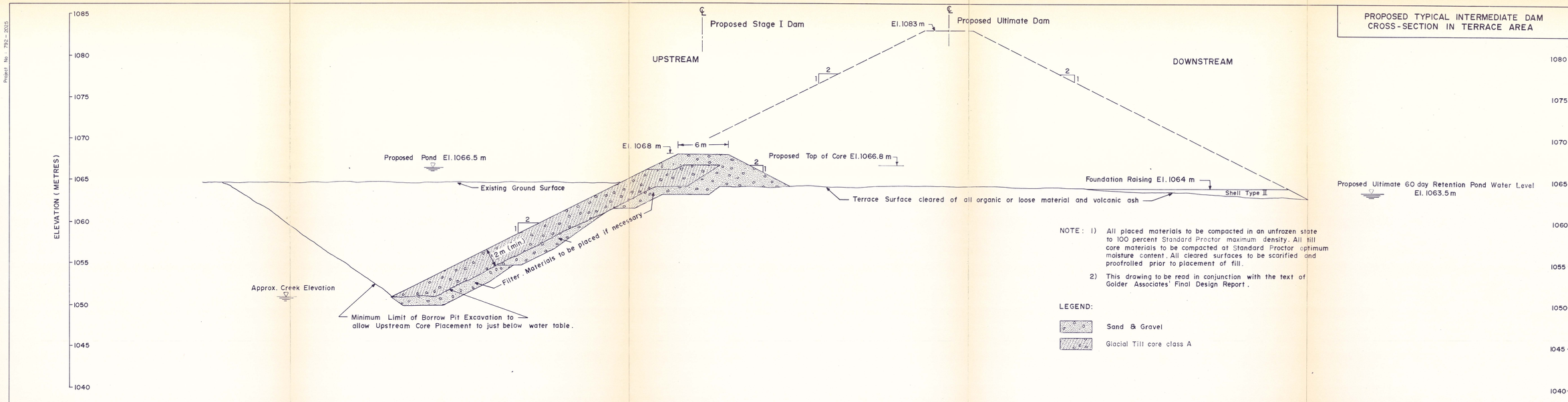
Project No. 792-2015

PROPOSED TYPICAL INTERMEDIATE DAM CROSS-SECTION IN TERRACE AREA

FIGURE 7-8



NOTE: 1) All placed materials to be compacted in an unfrozen state to 100 percent Standard Proctor maximum density. All till core materials to be compacted at Standard Proctor optimum moisture content. All cleared surfaces to be scarified and proofrolled prior to placement of fill.

2) This drawing to be read in conjunction with the text of Golder Associates' Final Design Report.

LEGEND:

Sand & Gravel

Glacial Till core class A

Scale 1:200 Horizontal and vertical

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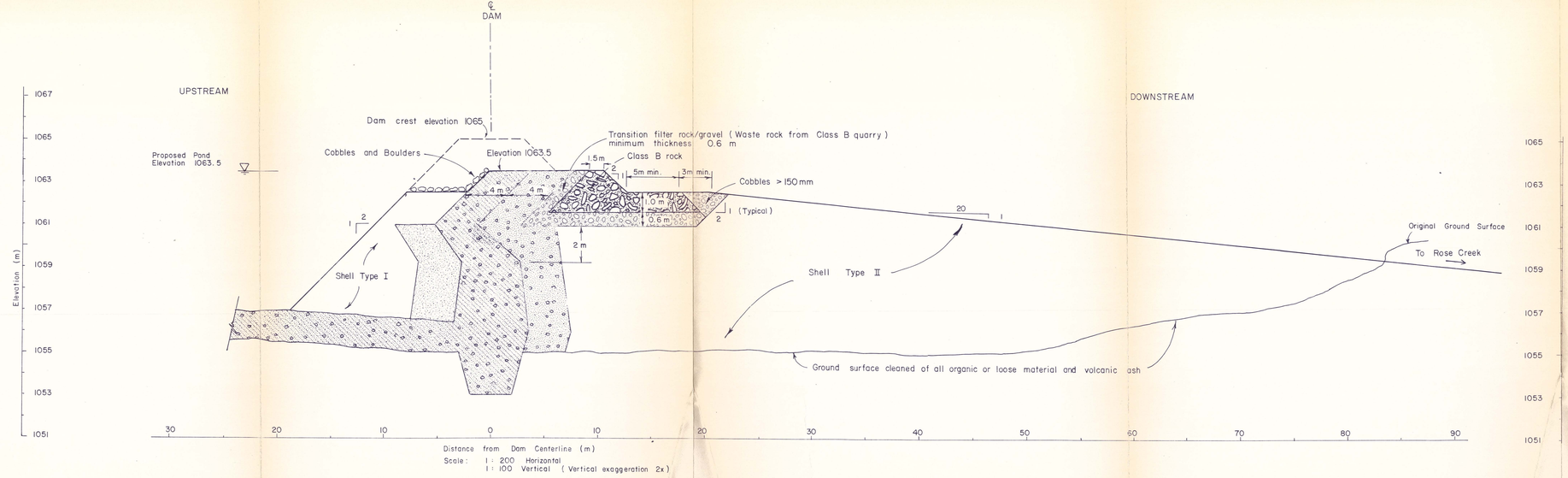
Rev'd DG
Date May 1980

Project : 792 - 2025

PROPOSED EMERGENCY SPILLWAY PROFILE

FIGURE 7-9



NOTES : 1) No settlement considerations made in selection of spillway elevations.

2) Spillway shown is for Cross Valley Dam (For location of spillway see Figure 7-1). Intermediate Dam to be similar in construction except all elevations must be increased by 3 meters.

3) For Cross - section of spillway see Figure 7-10

4) All placed materials to be compacted where possible according to dam specifications. Rip-rap to be intermeshed as much as possible.

5) This drawing is to be read in conjunction with the text of Golder Associates, Final Design Report.

LEGEND :

- Sand
- Sand & Gravel
- Glacial Till core class A

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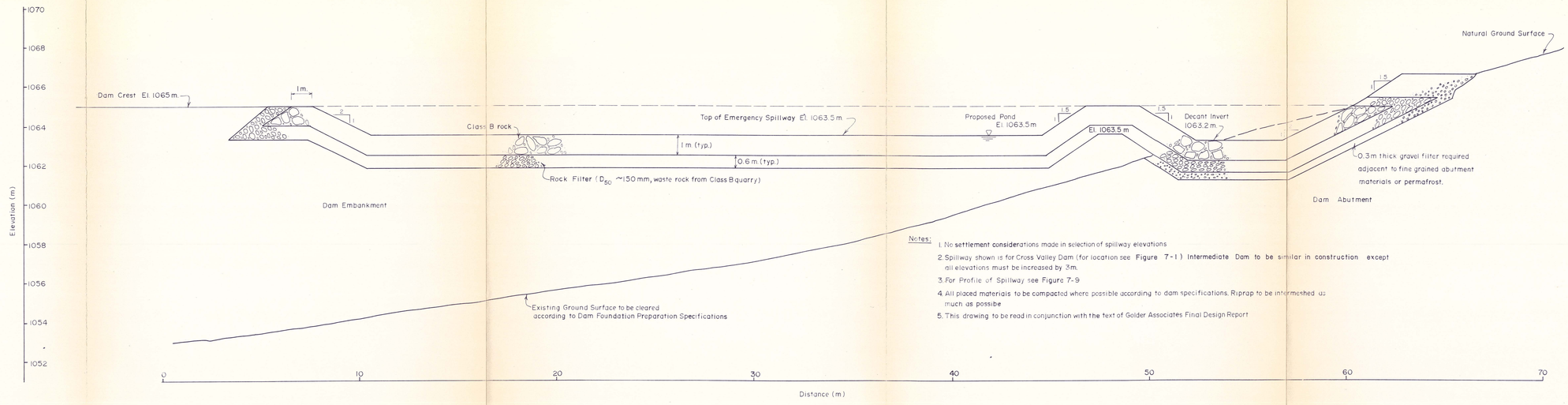
Date : May 1980

Project No. 792 - 2025

PROPOSED EMERGENCY SPILLWAY
CROSS-SECTION AT CREST

FIGURE 7-10



Notes:

1. No settlement considerations made in selection of spillway elevations
2. Spillway shown is for Cross Valley Dam (for location see Figure 7-1) Intermediate Dam to be similar in construction except all elevations must be increased by 3m.
3. For Profile of Spillway see Figure 7-9
4. All placed materials to be compacted where possible according to dam specifications. Riprap to be intermeshed as much as possible
5. This drawing to be read in conjunction with the text of Golder Associates Final Design Report

Scale: 1:1000 Horizontal and Vertical

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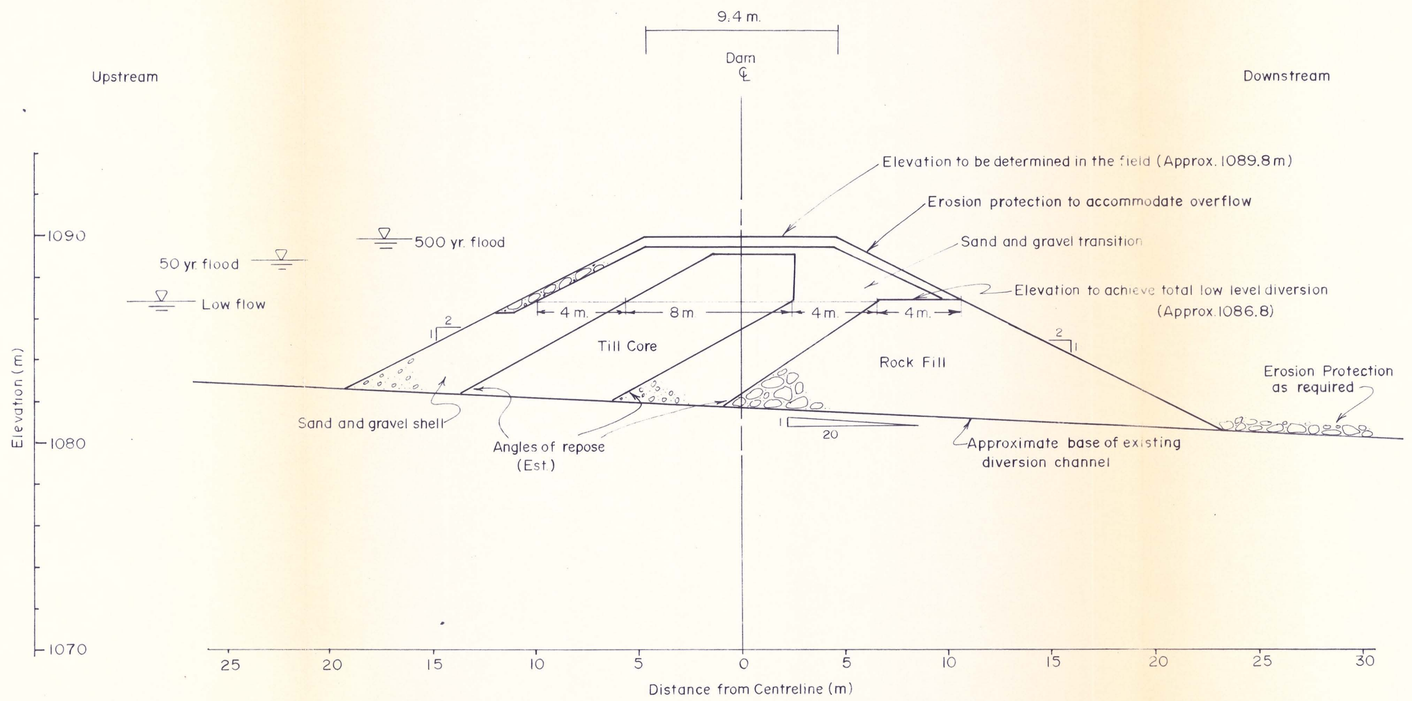
Project No. 792 - 2025

PROPOSED DIVERSION DAM CROSS-SECTION

FIGURE 7-11



Notes:

1. Materials placed above water level to be compacted as per dam specifications for fill placement.
2. For location of Diversion Dam see Figure 7-1
3. Dam foundation to be cleaned of all loose recent alluvium and organic materials.
4. Final design cross-section to be determined in the field.
5. This drawing to be read in conjunction with text of Golder Associates final design report.

Scale : 1 : 200 Horizontal and Vertical

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8.0 PROJECT QUANTITIES SUMMARY

(i) Diversion Canal

Excavation Overburden:	650,000 m^3
Rock:	5,000 to 10,000 m^3
Stripping Beneath Dyke:	55,000 m^3
Gravel Lining:	172,000 to 185,000 m^3
Ersion Lining Class A Rock:	37,000 m^3
Till Lining:	16,000 to 26,000 m^3
Outfall Rock Class B:	30,000 m^3
Outfall Filter Rock and Lining:	17,000 m^3

(ii) Cross-Valley Dam

Class "A" Core (good quality till)	86,000 m^3
Class "B" Core (poor quality till)	37,000 m^3
Class "C" Core (canal waste excavaton)	63,000 m^3
Sand Zone	40,000 m^3
Sand and Gravel Filter	64,000 m^3
Shell Type I	75,000 m^3
Shell Type II	113,000 m^3
Foundation Subexcavation (fill included)	89,000 m^3

(iii) Intermediate Dam

Core (good quality till)	56,000 m^3
Shell Type I	34,000 m^3
Shell Type II	207,000 m^3
Foundation Subexcavation	53,000 m^3
Sand Zone	11,000 m^3
Sand and Gravel Filter	30,000 m^3
Foundation Raising (included in Shell Type II)	130,000 m^3

(iv) Emergency Spillways

	Cross Valley Dam	Intermediate Dam
Sand and Gravel Abutment Fill:	10,000 m^3	2200 m^3
Rock Crest Weir Class B:	1,000 m^3	1000 m^3
Rock Filter	500 m^3	500 m^3

Decant Rock lining: not computed

(v) Diversion Dam

Total Fill:	8500 m^3 consisting of
	1900 m^3 Rock Fill
	4200 m^3 Sand and Gravel
	2500 m^3 Till

(vi) North Valley Wall Interceptor

Excavation:	60,000 m^3
Gravel Lining at North Abutment:	16,000 m^3
Diversion Fills:	2,000 m^3

(vii) *Retention Pond Storage: 1.4×10^6 m^3

(viii) *Tailings Volume Storage Stage I 11×10^6 m^3
(assuming 2 percent spigotted slopes)

Ultimate Tailings Volume Storage 20×10^6 m^3

LIST OF ABBREVIATIONS

The abbreviations commonly employed on each "Record of Borehole," on the figures and in the text of the report, are as follows:

I. SAMPLE TYPES

AS auger sample
CS chunk sample
DO drive open
DS Denison type sample
FS foil sample
RC rock core
ST slotted tube
TO thin-walled, open
TP thin-walled, piston
WS wash sample

II. PENETRATION RESISTANCES

Dynamic Penetration Resistance: The number of blows by a 140-pound hammer dropped 30 inches required to drive a 2-inch diameter, 60 degree cone one foot, where the cone is attached to 'A' size drill rods and casing is not used.

Standard Penetration Resistance, N: The number of blows by a 140-pound hammer dropped 30 inches required to drive a 2-inch drive open sampler one foot.

WH sampler advanced by static weight—weight, hammer

PH sampler advanced by pressure—pressure, hydraulic

PM sampler advanced by pressure—pressure, manual

III. SOIL DESCRIPTION

(a) *Cohesionless Soils*

Relative Density	*N, blows/ft.*
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) *Cohesive Soils*

Consistency	c_u, *lb./sq. ft.*
Very soft	Less than 250
Soft	250 to 500
Firm	500 to 1,000
Stiff	1,000 to 2,000
Very stiff	2,000 to 4,000
Hard	over 4,000

IV. SOIL TESTS

C consolidation test
H hydrometer analysis
M sieve analysis
MH combined analysis, sieve and hydrometer[1]
Q undrained triaxial[2]
R consolidated undrained triaxial[2]
S drained triaxial
U unconfined compression
V field vane test

NOTES:

[1]Combined analyses when 5 to 95 per cent of the material passes the No. 200 sieve.

[2]Undrained triaxial tests in which pore pressures are measured are shown as $\bar{Q}$ or $\bar{R}$.

LIST OF SYMBOLS

I. GENERAL

$\pi = 3.1416$

e = base of natural logarithms 2.7183

$\log_e a$ or $\ln a$, natural logarithm of a

$\log_{10} a$ or $\log a$, logarithm of a to base 10

t	time
g	acceleration due to gravity
V	volume
W	weight
M	moment
F	factor of safety

II. STRESS AND STRAIN

u	pore pressure
σ	normal stress
σ'	normal effective stress ($\bar{\sigma}$ is also used)
τ	shear stress
ϵ	linear strain
ϵ_{xy}	shear strain
ν	Poisson's ratio (μ is also used)
E	modulus of linear deformation (Young's modulus)
G	modulus of shear deformation
K	modulus of compressibility
η	coefficient of viscosity

III. SOIL PROPERTIES

(a) *Unit weight*

γ	unit weight of soil (bulk density)
γ_s	unit weight of solid particles
γ_w	unit weight of water
γ_d	unit dry weight of soil (dry density)
γ'	unit weight of submerged soil
G_s	specific gravity of solid particles $G_s = \gamma_s/\gamma_w$
e	void ratio
n	porosity
w	water content
S_r	degree of saturation

(b) *Consistency*

w_L	liquid limit
w_P	plastic limit
I_P	plasticity index
w_S	shrinkage limit
I_L	liquidity index $= (w - w_P)/I_P$
I_C	consistency index $= (w_L - w)/I_P$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
D_r	relative density $= (e_{max} - e)/(e_{max} - e_{min})$

(c) *Permeability*

h	hydraulic head or potential
q	rate of discharge
v	velocity of flow
i	hydraulic gradient
k	coefficient of permeability
j	seepage force per unit volume

(d) *Consolidation (one-dimensional)*

m_v	coefficient of volume change $= -\Delta e/(1+e)\Delta\sigma'$
C_c	compression index $= -\Delta e/\Delta \log_{10} \sigma'$
c_v	coefficient of consolidation
T_v	time factor $= c_v t/d^2$ (d, drainage path)
U	degree of consolidation

(e) *Shear strength*

τ_f	shear strength	
c'	effective cohesion intercept	in terms of effective stress $\tau_f = c' + \sigma' \tan \phi'$
ϕ'	effective angle of shearing resistance, or friction	
c_u	apparent cohesion*	in terms of total stress $\tau_f = c_u + \sigma \tan \phi_u$
ϕ_u	apparent angle of shearing resistance, or friction	
μ	coefficient of friction	
S_t	sensitivity	

*For the case of a saturated cohesive soil, $\phi_u = 0$ and the undrained shear strength $\tau_f = c_u$ is taken as half the undrained compressive strength.

Golder Associates
CONSULTING GEOTECHNICAL ENGINEERS

A REPORT
to
CYPRUS ANVIL MINING
Providing

FINAL DESIGN RECOMMENDATIONS
For The
DOWN VALLEY TAILINGS DISPOSAL PROJECT
Faro Yukon Territory

VOLUME TWO
OF
THREE VOLUMES

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792-2025

June, 1980

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VOLUME II

VOLUME III

Rose Creek Flow Data

ROSE CREEK BELOW FARO CREEK - STATION No. 9BC-3

Location: Lat. 62° 20' 30", long. 133° 24' 30", (revised), Yukon Territory, approximately one thousand feet below Faro Creek and forty-five miles west-northwest of Ross River P.O.
Gauge: Manual.
Period of Record: Mainly continuous, August and September 1966 and December 1966 to September 1967.
Extremes Recorded: Maximum daily discharge, 1,150 cfs on May 31, 1967. Minimum daily discharge, 6.1 cfs on March 10, 1967.
Remarks: Records fair.

Daily Discharge in Cubic Feet per Second for the 1967 Water Year

Day	Oct.	Nov.	Dec.	Jan.	Feb.	March	April	May	June	July	Aug.	Sept.
1.....	-	-	-	-	-	-	-	-	1,070	105	108	41.0
2.....	-	-	-	-	-	-	-	-	810	99.0	93.0	38.2
3.....	-	-	-	-	-	-	-	-	616	155	87.8	79.4
4.....	-	-	-	-	-	-	6.3 b	-	732	110	82.5	60.5
5.....	-	-	-	-	-	-	-	-	801	129	77.2	59.5
6.....	-	-	12.3 b	-	-	-	-	-	783	121	77.2	62.6
7.....	-	-	-	-	-	-	-	-	512	100	78.3	68.8
8.....	-	-	12.4 b	-	-	-	-	-	323	90.9	82.5	78.3
9.....	-	-	-	-	-	-	-	-	334	90.9	77.2	93.0
10.....	-	-	-	-	-	6.1 b	-	-	591	105	77.2	79.4
11.....	-	-	-	-	-	-	-	-	543	95.4	74.1	71.0
12.....	-	-	-	-	-	-	-	-	292	115	77.2	79.4
13.....	-	-	-	-	-	-	-	-	246	93.0	71.0	71.0
14.....	-	-	-	-	-	-	-	-	292	94.2	68.8	68.8
15.....	-	-	-	-	-	-	-	-	431	135	64.6	72.0
16.....	-	-	-	-	-	-	-	-	467	115	61.5	73.0
17.....	-	-	-	-	-	-	-	-	414	110	59.5	71.0
18.....	-	-	-	-	-	-	-	-	467	105	59.5	77.2
19.....	-	-	-	-	-	-	-	-	337	105	61.5	80.4
20.....	-	-	-	-	-	-	-	-	273	82.5	53.5	72.0
21.....	-	-	-	-	-	-	-	-	278	101	46.8	69.9
22.....	-	-	-	-	-	-	-	-	204	72.0	51.5	66.8
23.....	-	-	-	-	-	-	-	-	200	80.4	53.5	67.8
24.....	-	-	-	-	-	-	-	-	180	86.7	45.8	66.8
25.....	-	-	-	-	-	-	-	-	169	82.5	42.0	64.6
26.....	-	-	-	-	-	-	-	-	196	86.7	42.0	61.5
27.....	-	-	-	-	-	-	-	-	202	82.5	36.3	51.5
28.....	-	-	-	-	-	-	-	792	200	83.6	32.5	61.5
29.....	-	-	-	-	-	-	-	595	163	105	34.4	59.5
30.....	-	-	-	-	-	-	-	1,110	121	173	36.3	57.5
31.....	-	-	-	-	-	-	-	1,150	-	163	34.4	-
Total	-	-	-	-	-	-	-	-	12,247	3,272.3	1,947.6	2,023.9
Mean	-	-	-	-	-	-	-	-	408	106	62.8	67.5
Acre-feet	-	-	-	-	-	-	-	-	24,290	6,490	3,860	4,010

For the Period..... Maximum daily discharge, 1,150 cfs on May 31
Minimum daily discharge, 6.1 cfs on March 10

b - Ice conditions.

SOURCE: Water Survey of Canada

Location: Lat. 62° 20' 00", long. 133° 24' 00", Yukon Territory, approximately one thousand feet below Faro Creek, and forty-five miles west-northwest of Ross River P.O.
Gauge: Manual.
Period of Record: August and September 1966.
Extremes Recorded: Maximum daily discharge, 45.2 cfs on September 6, 1966. Minimum daily discharge, 34.0 cfs at various times in 1966.
Remarks: Records fair.

Daily Discharge in Cubic Feet per Second for the 1966 Water Year

Day	Oct.	Nov.	Dec.	Jan.	Feb.	March	April	May	June	July	Aug.	Sept.
1.....	-	-	-	-	-	-	-	-	-	-	-	34.0
2.....	-	-	-	-	-	-	-	-	-	-	-	36.0
3.....	-	-	-	-	-	-	-	-	-	-	-	38.4
4.....	-	-	-	-	-	-	-	-	-	-	-	40.2
5.....	-	-	-	-	-	-	-	-	-	-	-	41.4
6.....	-	-	-	-	-	-	-	-	-	-	-	45.2
7.....	-	-	-	-	-	-	-	-	-	-	-	43.6
8.....	-	-	-	-	-	-	-	-	-	-	-	40.8
9.....	-	-	-	-	-	-	-	-	-	-	-	38.4
10.....	-	-	-	-	-	-	-	-	-	-	-	38.4
11.....	-	-	-	-	-	-	-	-	-	-	-	36.6
12.....	-	-	-	-	-	-	-	-	-	-	44.4	35.6
13.....	-	-	-	-	-	-	-	-	-	-	42.0	35.2
14.....	-	-	-	-	-	-	-	-	-	-	41.4	36.0
15.....	-	-	-	-	-	-	-	-	-	-	38.4	37.2
16.....	-	-	-	-	-	-	-	-	-	-	39.6	37.2
17.....	-	-	-	-	-	-	-	-	-	-	41.4	37.2
18.....	-	-	-	-	-	-	-	-	-	-	42.0	36.0
19.....	-	-	-	-	-	-	-	-	-	-	39.0	36.0
20.....	-	-	-	-	-	-	-	-	-	-	40.2	35.6
21.....	-	-	-	-	-	-	-	-	-	-	42.0	35.2
22.....	-	-	-	-	-	-	-	-	-	-	38.4	35.2
23.....	-	-	-	-	-	-	-	-	-	-	38.4	37.8
24.....	-	-	-	-	-	-	-	-	-	-	38.4	37.8
25.....	-	-	-	-	-	-	-	-	-	-	38.4	36.0
26.....	-	-	-	-	-	-	-	-	-	-	35.6	35.6
27.....	-	-	-	-	-	-	-	-	-	-	35.2	35.2
28.....	-	-	-	-	-	-	-	-	-	-	35.2	34.0
29.....	-	-	-	-	-	-	-	-	-	-	34.4	34.0
30.....	-	-	-	-	-	-	-	-	-	-	34.0	34.8
31.....	-	-	-	-	-	-	-	-	-	-	34.0	-
Total	-	-	-	-	-	-	-	-	-	-	-	1,114.6
Mean	-	-	-	-	-	-	-	-	-	-	-	37.2
Acre-feet	-	-	-	-	-	-	-	-	-	-	-	2,210

For the Period..... Maximum daily discharge, 45.2 cfs on September 6
Minimum daily discharge, 34.0 cfs at various times

SOURCE: Water Survey of Canada

Location: Lat. 62° 20' 30", long. 133° 24' 30", Yukon Territory, approximately one thousand feet below Faro Creek and forty-five miles west-northwest of Ross River P.O.
Gauge: Manual.
Period of Record: Mainly continuous, August and September 1966 and December 1966 to December 1968.
Extremes Recorded: Maximum daily discharge, 1,150 cfs on May 31, 1967.
Minimum daily discharge, 6.1 cfs on March 10, 1967.
Remarks: Records fair. Since October 4, 1968, flows have been affected by dam on south fork of Rose Creek.

Daily Discharges in Cubic Feet per Second for the Period October to December 1967

Day	Oct.	Nov.	Dec.	Day	Oct.	Nov.	Dec.	Day	Oct.	Nov.	Dec.
1	56.5	23.9	18.2	11	49.6	22.3	16.2	21	30.7	20.3	14.0
2	55.5	23.7	18.1	12	49.6	22.1	16.0	22	27.1	20.1	13.8
3	42.0	23.5	17.9	13	46.8	21.9	15.8	23	26.8	19.9	13.6
4	42.0	23.3	17.7	14	40.1	21.7	15.6	24	26.5	19.7	13.4
5	40.1	23.2	17.4	15	40.1	21.5	15.2	25	26.0	19.5	13.1
6	41.0	23.1	17.2	16	41.0	21.3	15.0	26	25.7	19.3	13.0
7	40.1	23.0	17.0	17	40.1	21.1	14.8	27	25.4	19.1	12.9
8	61.5	22.9	16.8	18	32.5	20.9	14.6	28	25.1	18.9	12.8
9	40.1	22.7	16.6	19	32.5	20.7	14.4	29	24.8	18.9	12.6
10	42.0	22.5	16.4	20	32.5	20.5	14.2	30	24.5	18.9	12.3
								31	24.0	-	12.0
Ice conditions October 22 to December 31.								Total	1,152.2	640.4	468.6
								Mean	37.2	21.3	15.1
								Acre-feet	2,290	1,270	929

Daily Discharges in Cubic Feet per Second for the Year 1968

Day	Jan.	Feb.	March	April	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
1	11.8	8.0	7.1	7.1	12.4	212 E	152	261	218	115	36.5	28.0
2	11.6	7.9	7.1	7.1	13.5	230 E	139	316	280	111	36.1	27.7
3	11.4	7.8	7.1	7.1	14.5	205 E	133	316	229	90.0	35.7	27.4
4	11.2	7.7	7.1	7.2	16.0	185 E	114	351	253	70.0	35.3	27.0
5	11.0	7.7	7.1	7.2	17.5	181	198	236	241	57.0	35.0	26.5
6	10.9	7.6	7.1	7.2	19.5	185 E	548	170 E	218	54.0	34.7	26.2
7	10.8	7.5	7.1	7.2	22.0	190 E	372	133	174	53.0	34.4	25.9
8	10.7	7.4	7.1	7.3	27.0	204 E	211	115	196	47.1	34.1	25.6
9	10.6	7.4	7.1	7.3	32.0 E	220 E	178	106	210 E	46.0	33.8	25.3
10	10.4	7.4	7.1	7.3	35.8	250 E	168	97.0	220 E	45.0	33.8	25.0
11	10.3	7.4	7.1	7.3	44.0 E	236 E	159	88.0 E	234	44.0	33.6	24.6
12	10.2	7.3	7.1	7.4	52.0 E	228 E	140	84.0 E	227	43.0	33.4	24.2
13	10.0	7.3	7.1	7.4	60.0 E	213	170	83.0	211	42.0	33.0	23.8
14	9.8	7.2	7.1	7.5	64.0 E	201	150	80.6	198	42.0	32.8	23.4
15	9.7	7.2	7.1	7.6	68.0 E	190 E	160	78.2	181	42.0	32.5	23.0
16	9.6	7.2	7.1	7.7	70.0 E	210 E	150	84.0 E	202	42.0	32.3	22.2
17	9.5	7.2	7.1	7.8	74.6	220 E	150	90.0 E	200	42.0	32.1	22.4
18	9.4	7.1	7.1	7.9	52.0	240 E	176	96.0 E	200	41.0	31.9	22.1
19	9.2	7.1	7.1	8.0	220 E	260 E	166	100 E	198	41.0	31.7	21.8
20	9.0	7.1	7.1	8.2	270 E	295 E	160	108	183	40.0	31.5	21.5
21	8.9	7.1	7.1	8.5	330 E	302	140	106	168	39.5	31.2	21.1
22	8.8	7.1	7.1	8.8	390 E	274	129	105	168	39.0	30.9	20.7
23	8.7	7.1	7.1	9.0	440	236	131	97.0	160	38.5	30.6	20.3
24	8.6	7.1	7.1	9.2	348	220 E	114	148	156	38.0	30.3	19.9
25	8.5	7.1	7.1	9.5	300 E	206 E	114	148	148	38.0	30.0	19.5
26	8.5	7.1	7.1	10.0	280 E	200 E	106	158	146	38.0	29.3	19.0
27	8.4	7.0	7.1	10.5	260 E	194 E	98.5	168	139	38.0	29.2	18.5
28	8.3	7.0	7.1	11.0	230 E	185 E	98.5	168	142	37.5	29.1	18.0
29	8.2	7.0	7.1	11.5	215 E	178	84.4	168	131	37.0	28.8	17.5
30	8.1	-	7.1	11.9	210 E	160 E	91.4	180 E	122	37.0	28.5	17.0
31	8.1	-	7.1	-	205 E	-	114	198 E	-	37.0	-	16.5
Total	300.2	212.1	220.1	248.7	4,392.8	6,510	5,014.8	4,636.8	5,753	1,524.6	972.1	701.6
Mean	9.7	7.3	7.1	8.3	142	217	162	150	192	49.2	32.4	22.6
Acre-feet	595	421	437	493	8,710	12,900	9,950	9,200	11,400	3,020	1,930	1,390

For the Year 1968..... Maximum daily discharge, 548 cfs on July 6
Minimum daily discharge, 7.0 cfs, February 27 to 29
Mean discharge, 83.3 cfs
Total discharge, 60,400 ac-ft

Ice conditions January 1 to May 8 and October 5 to December 31. E - Estimated.

SOURCE: Water Survey of Canada

ROSE CREEK BELOW FARO CREEK - STATION NO. 09BC003

DAILY DISCHARGE IN CUBIC FEET PER SECOND FOR 1969

DAY	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	DAY
1	16.5 B	7.6 B	7.6 B	7.4 B	14.3 B	117	42.1	---	---	---	---	---	1
2	15.8 B	7.6 B	7.5 B	7.4 B	15.0 B	84.4	45.7	---	---	---	---	---	2
3	15.1 B	7.6 B	7.5 B	7.4 B	15.7 B	75.8	49.3	---	---	---	---	---	3
4	14.8 B	7.6 B	7.5 B	7.4 B	16.5 B	127	52.0	---	---	---	---	19.4	4
5	13.7 B	7.6 B	7.5 B	7.4 B	17.3 B	160	63.0	---	---	---	---	---	5
6	13.3 B	7.6 B	7.5 B	7.4 B	19.4 B	179	103	---	---	---	---	---	6
7	12.9 B	7.6 B	7.5 B	7.4 B	19.3 B	216	122	---	---	---	---	---	7
8	12.5 B	7.6 B	7.5 B	7.5 B	21.0 B	200	122	---	---	---	---	---	8
9	12.0 B	7.6 B	7.5 B	7.6 B	23.0 B	239	90.0	---	---	---	---	---	9
10	11.5 B	7.6 B	7.5 B	7.6 B	27.6	354	72.2	---	---	---	---	---	10
11	11.2 B	7.6 B	7.4 B	7.6 B	38.5	277	72.2	---	---	---	---	---	11
12	10.9 B	7.6 B	7.4 B	7.7 B	46.6	213	179	---	---	---	---	---	12
13	10.6 B	7.6 B	7.4 B	7.8 B	49.3	202	198	---	---	---	---	---	13
14	10.2 B	7.6 B	7.4 B	7.9 B	58.3	193	139	---	---	---	---	---	14
15	9.8 B	7.6 B	7.4 B	8.0 B	65.0	174	102	---	---	---	---	---	15
16	9.6 B	7.6 B	7.4 B	8.2 B	95.6	164	94.2	---	---	---	---	---	16
17	9.4 B	7.6 B	7.4 B	8.4 B	105	140	84.4	---	---	---	---	---	17
18	9.2 B	7.6 B	7.4 B	8.6 B	109	127	98.5	---	---	---	---	---	18
19	9.0 B	7.6 B	7.4 B	8.7 B	98.5	139	81.8	---	---	---	---	---	19
20	8.7 B	7.6 B	7.4 B	8.8 B	105	111	73.4	---	---	---	---	---	20
21	8.7 B	7.6 B	7.3 B	9.1 B	144	90.0	68.0	---	---	---	---	---	21
22	8.6 B	7.6 B	7.3 B	9.3 B	196	77.0	66.0	---	---	---	---	---	22
23	8.4 B	7.6 B	7.3 B	9.6 B	277	39.4	71.0	---	---	---	---	---	23
24	8.2 B	7.6 B	7.3 B	10.0 B	305	53.8	69.0	---	---	---	---	---	24
25	8.0 B	7.6 B	7.3 B	10.5 B	328	50.2	90.0	---	---	---	---	---	25
26	7.9 B	7.6 B	7.3 B	11.0 B	158	47.5	124	---	---	---	---	---	26
27	7.8 B	7.6 B	7.3 B	11.7 B	81.8	45.7	103	---	---	---	---	---	27
28	7.7 B	7.6 B	7.3 B	12.4 B	56.5	43.9	90.0	---	---	---	---	---	28
29	7.6 B		7.3 B	13.1 B	61.0	41.2	81.8	---	---	---	---	---	29
30	7.6 B		7.3 B	13.8 B	102	42.1	76.0 E	---	---	---	---	---	30
31	7.6 B		7.4 B		139		74.0 E	---		---		---	31
TOTAL	324.4	212.8	229.5	266.7	2808.2	4023.0	2796.6	---	---	---	---	---	TOTAL
MEAN	10.5	7.6	7.4	8.9	90.6	134	90.2	---	---	---	---	---	MEAN
AC-FT	643	422	455	529	5570	7980	5550	---	---	---	---	---	AC-FT
MAX	16.5	7.6	7.6	13.8	328	354	198	---	---	---	---	---	MAX
MIN	7.6	7.6	7.3	7.4	14.3	39.4	42.1	---	---	---	---	---	MIN

SUMMARY FOR THE MONTHS JAN TO JUL

MEAN DISCHARGE, 50.3 CFS
TOTAL DISCHARGE, 21100 AC-FT
MAXIMUM DAILY DISCHARGE, 354 CFS ON JUN 10
MINIMUM DAILY DISCHARGE, 7.3 CFS ON MAR 21

TYPE OF GAUGE - MANUAL
LOCATION - LAT 62 20 30 N
LONG 133 24 30 W

B-ICE CONDITIONS
E-ESTIMATED

REGULATED

ROSS RIVER AT ROSS RIVER - STATION NO. 09BA001

DAILY DISCHARGE IN CUBIC FEET PER SECOND FOR 1969

DAY	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	DAY
1	312 B	184 B	181 B	182 B	630 B	5810	1910 E	3660	3890	2650	320 B	600 B	1
2	304 B	184 B	182 B	184 B	680 B	5570	2100 E	4010	3740	2550	350 B	600 B	2
3	296 B	184 B	182 B	186 B	740 B	5540	2300 E	4180	3710	2450	400 B	594 B	3
4	288 B	184 B	181 B	189 B	820 B	5740	2500 E	4090	4070	2400	430 B	570 B	4
5	278 B	184 B	181 B	191 B	920 B	5910	2750 E	3820	4500	2340	440 B	550 B	5
6	270 B	184 B	180 B	195 B	1010 B	6320	2930 E	3560	4740	2270	435 B	520 B	6
7	262 B	184 B	180 B	198 B	1110 B	7390	3050 E	3380	4740	2230	426 B	490 B	7
8	254 B	184 B	180 B	200 B	1270 B	7930	3340 E	3250	4650	2230	415 B	470 B	8
9	246 B	184 B	180 B	205 B	1450 B	8360	3700 E	2700	4500	2250	407 B	460 B	9
10	240 B	184 B	179 B	212 B	1650 B	8900	4150 E	2880	4440	2300	400 B	450 B	10
11	234 B	184 B	179 B	216 B	1810	8980 A	5000 E	3000	4620	2320	410 B	450 B	11
12	228 B	184 B	179 B	222 B	2060	8480 E	6350 E	3130	4800	2270	430 B	445 B	12
13	222 B	184 B	178 B	228 B	2500	7700 E	8300 E	3300	5370	2230	430 B	440 B	13
14	216 B	184 B	178 B	235 B	3280	6950 E	10400 E	3300	6490	2190	420 B	430 B	14
15	212 B	184 B	178 B	243 B	3870	6380 E	11600 A	3980	6860	2090	380 B	425 B	15
16	210 B	183 B	178 B	251 B	4150	5900 E	9540	4210	6590	1920	365 B	420 B	16
17	206 B	183 B	178 B	261 B	4270	5200 E	7570	4470	6080	1770	360 B	395 B	17
18	202 B	183 B	178 B	274 B	4740	4650 E	6350	4650	5540	1830	375 B	380 B	18
19	198 B	183 B	178 B	289 B	5040	4200 E	5370	4620	5100	1830	415 B	370 B	19
20	196 B	183 B	178 B	305 B	5190	3900 E	4680	4500	4710	1710	460 B	350 B	20
21	194 B	183 B	178 B	320 B	5250	3570 E	4120	4560	4380	1690	500 B	335 B	21
22	192 B	183 B	177 B	337 B	5160	3340 E	3710	4950	4090	1350	520 B	320 B	22
23	190 B	183 B	177 B	355 B	5100	3100 E	3380	5470	3840	908	560 B	310 B	23
24	189 B	183 B	178 B	375 B	5470	2850 E	3180	5640	3660	654	550 B	305 B	24
25	188 B	182 B	178 B	391 B	6350	2650 E	3030	5430	3480	524	560 B	300 B	25
26	188 B	181 B	178 B	420 B	7170	2450 E	3000	5070	3350	372	580 B	300 B	26
27	187 B	181 B	179 B	450 B	7390	2250 E	3150	4740	3230	290 A	580 B	298 B	27
28	186 B	180 B	179 B	488 B	7070	2150 E	3400	4590	3030	288 B	580 B	298 B	28
29	185 B		179 B	530 B	6790	2010 E	3690	4530	2850	290 B	600 B	298 B	29
30	185 B		180 B	588 B	6280	1940 E	3710	4360	2750	295 B	590 B	300 B	30
31	184 B		180 B		6010		3630	4120		300 B		300 B	31
TOTAL	6942	5131	5551	8720	115230	156120	141890	128150	133800	50791	13688	12773	TOTAL
MEAN	224	183	179	291	3720	5200	4580	4130	4460	1640	456	412	MEAN
AC-FT	13800	10200	11000	17300	229000	310000	281000	254000	265000	101000	27200	25300	AC-FT
MAX	312	184	182	588	7390	8980	11600	5640	6860	2650	600	600	MAX
MIN	184	180	177	182	630	1940	1910	2700	2750	288	320	298	MIN

SUMMARY FOR THE YEAR 1969

MEAN DISCHARGE, 2130 CFS
TOTAL DISCHARGE, 1540000 AC-FT
MAXIMUM DAILY DISCHARGE, 11600 CFS ON JUL 15
MINIMUM DAILY DISCHARGE, 177 CFS ON MAR 22

TYPE OF GAUGE - RECORDING
LOCATION - LAT 61 59 40 N
LONG 132 22 40 W
DRAINAGE AREA 2800 SQ MILES

A-MANUAL GAUGE
B-ICE CONDITIONS
E-ESTIMATED

NATURAL FLOW

SOURCE: Water Survey of Canada

ROSE CREEK BELOW FARO CREEK - STATION NO. 09BC003

MONTHLY AND ANNUAL MEAN DISCHARGES IN CUBIC FEET PER SECOND FOR THE PERIOD OF RECORD

YEAR	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	MEAN	YEAR
1966	---	---	---	---	---	---	---	---	37.2	---	---	---	---	1966
1967	---	---	---	---	---	408	106	62.8	67.5	37.2	21.3	15.1	---	1967
1968	9.7	7.3	7.1	8.3	142	217	162	150	192	49.2	32.4	22.6	83.3	1968
1969	10.5	7.6	7.4	8.9	90.6	134	90.2	---	---	---	---	---	---	1969
MEAN	10.1	7.5	7.3	8.6	116	253	119	106	98.9	43.2	26.9	18.9	83.3	MEAN

LOCATION - LAT 62 20 30 N
LONG 133 24 30 W

REGULATED

ROSE CREEK BELOW FARO CREEK - STATION NO. 09BC003

ANNUAL EXTREMES OF DISCHARGE IN CFS AND ANNUAL TOTAL DISCHARGE IN AC-FT FOR THE PERIOD OF RECORD

YEAR	MAXIMUM INSTANTANEOUS DISCHARGE	MAXIMUM DAILY DISCHARGE	MINIMUM DAILY DISCHARGE	TOTAL DISCHARGE	YEAR
1966	---	---	---	---	1966
1967	---	1150 CFS ON MAY 31 *	6.1 CFS ON MAR 10 *	---	1967
1968	---	548 CFS ON JUL 6	7.0 CFS ON FEB 27	60400 AC-FT	1968
1969	---	354 CFS ON JUN 10	7.3 CFS ON MAR 21	---	1969
		* - EXTREME RECORDED FOR THE PERIOD OF RECORD		60400 AC-FT	MEAN

SOURCE: Water Survey of Canada

FLOW HYDROGRAPH

10 MAY to 4 JUNE, 1975.

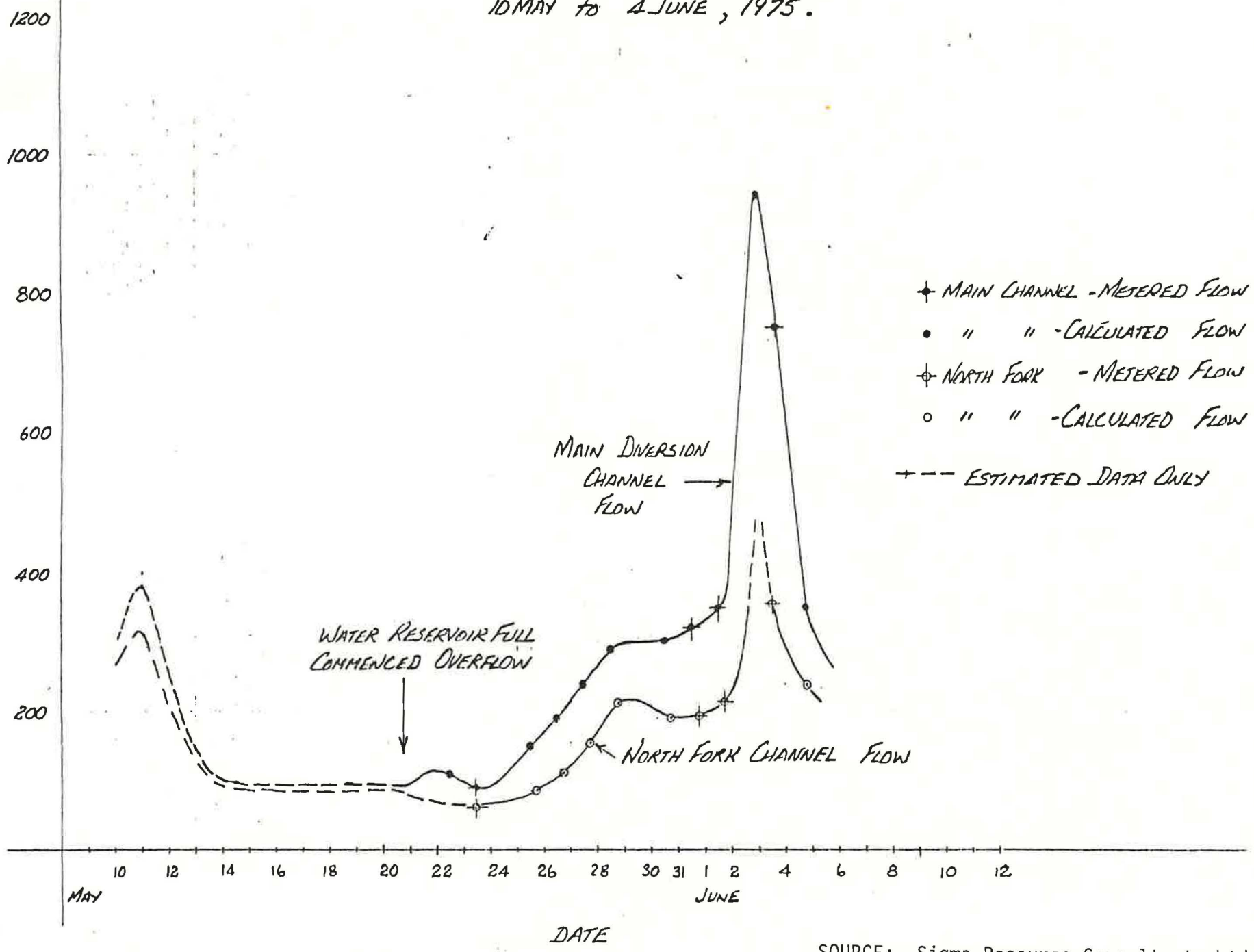


FIGURE 2

SOURCE: Sigma Resource Consultants Ltd.

DATE ..... SUBJECT .....
BY ..... DATE .....
JOB NO. ..... SHEET NO. ..... OF .....

Supplementary Climatic Data for
Faro, Yukon and Anvil, Yukon Stations

Golder Associates

FARO, YUKON LAT 62° 14'N LONG 133° 20' W ELEV 2380'

59-73 calc

Temperatures in degrees Celsius

Snowfall in centimetres

Rainfall and total precipitation in millimetres

	JAN.	FEB.	MAR.	APRIL	MAY	JUNE	JULY	AUG.	SEPT.	OCT.	NOV.	DEC.	YEAR
TEMPERATURES													
Mean Daily	-26.7	-15.4	-9.7	0.3	7.8	13.6	15.4	12.7	7.0	-1.9	-15.5	-20.8	-2.8
Mean Daily Max	21.2	-9.4	-3.2	6.2	14.3	19.8	21.6	18.3	13.2	2.1	-11.4	-16.3	2.8
Mean Daily Min	-32.1	-21.4	-16.2	-5.7	1.5	7.3	9.1	7.2	0.8	-5.9	-19.6	-24.8	- 8.3
Extreme Max	7.2	11.1	12.8	21.1	26.7	35.0	35.0	32.2	26.7	18.3	12.8	4.4	35.0
DATE/YEAR													
No of Years													
Extreme Min	-56.7	-53.9	-42.8	-28.9	-12.2	-3.9	-1.1	-3.9	-12.2	-28.9	-42.8	-51.1	-56.7
Date/Year													
No of years													
Precipitation													
Mean Rainfall	TR	TR	0.8	0.8	19.3	35.1	40.6	34.5	32.3	11.4	1.3	TR	176.0
Mean Snowfall	30.5	16.5	17.8	8.1	6.4	NIL	NIL	NIL	2.5	15.2	24.1	19.3	140.5
Mean Ttl Pcpn	30.5	16.5	18.5	8.9	25.7	35.1	40.6	34.5	34.8	26.7	25.4	19.3	316.5
No days with Measurable PCPN													

ANVIL YUKON Lat. 62°22'N Long. 133°23'W Elevation 3800

	JAN.	FEB.	MAR.	APRIL	MAY	JUNE	JULY	AUG.	SEPT.	OCT.	NOV.	DEC.	YEAR
59-73 calc.													
	Temperature in degrees Celsius												
	Precipitation in millimeters except snowfall which is in centimeters.												
Temperature													
Mean Daily	-20.8	-14.3	-11.3	-3.9	3.6	10.6	11.1	9.6	3.9	-3.1	-12.3	-17.8	-3.7
Mean Max.	-15.9	-8.4	-5.6	1.8	9.1	17.2	16.9	14.7	8.6	1.7	-7.7	-12.6	1.7
Mean Min.	-25.9	-20.3	-17.1	-8.6	-1.9	4.2	4.9	2.8	-0.7	-7.8	-17.1	-22.9	-9.2
Extreme Max.	1.7	9.4	5.0	11.1	20.0	29.4	25.0	26.1	20.0	14.4	11.1	3.3	29.4
Date/Year	6/71	27/68	1/68	18/70	9/72	13/69	27/71	1/71	3/72	14/69	2/70	24/70	
# Years record	6	6	6	6	5	5	5	5	5	5	6	5	
Extreme Min.	-46.1	-43.3	-41.1	-25.6	-11.7	-2.8	-2.8	-6.7	-13.3	-23.3	-39.4	-43.9	-46.1
Date/Year	25/71	3/68	8/72	1/72	16/72	7/69	1/70	12/69	23/72	25/71	30/70	12/71/76	
# yrs. record	6	6	6	6	5	5	5	5	5	5	6	5	
Precipitation													
Mean Rainfall	Nil	Nil	Nil	1.0	9.7	43.2	56.6	50.3	34.5	3.1	TR	Nil	198.4
Mean Snowfall	27.9	21.1	24.1	11.9	22.4	2.5	Nil	4.1	8.1	32.8	36.6	19.8	211.3
Mean Precipitat.	27.9	21.1	24.1	13.0	32.0	45.7	56.6	54.4	42.7	35.8	36.6	19.8	409.7
# days Msrbl pcpn	9	8	9	7	9	8	17	14	13	12	14	11	131

Test Pit and Borehole Information

Contents

General Description of Logging Format

Record of Borehole Sheets -

Boreholes Nos. 1 to 42 incl.

46 to 52 incl.

Record of Test Pit Sheets Other Than Along Diversion Canal

Test Pit No. 1 to 20 incl.

23 to 25 incl.

Record of Test Pit Sheets Along Diversion Canal

Test Pit No. 79-22 [0-060 to 0-80]

Chainage Test Pit 0+030 to 0+060

0+180 to 1+210

0+360 to 0+390

0+545 to 0+575

0+750 to 0+770

0+890 to 0+920

1+080 to 1+220

1+380 to 1+400

1+665 to 1+700

Test Pit No. 79-21 [1+900 to 1+930]

Chainage Test Pit 2+155 to 2+180

2+500 to 2+530

2+890 to 2+930

3+150 to 3+175

GENERAL DESCRIPTION OF LOGGING FORMAT

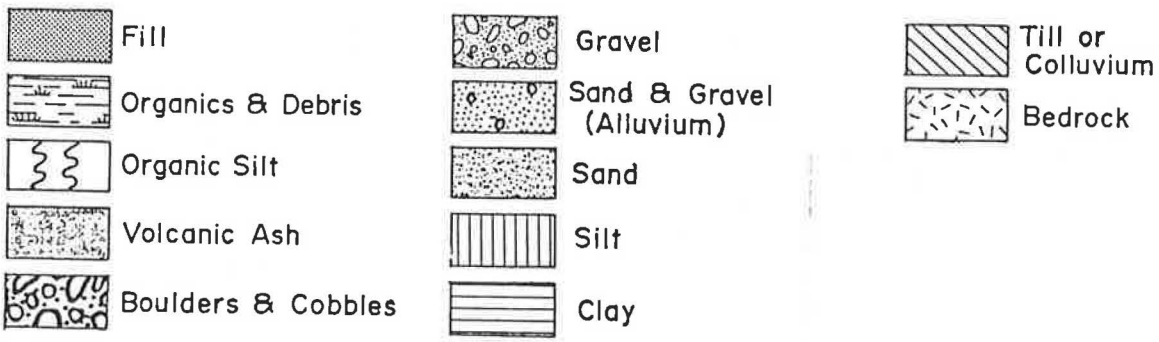
The record of test pit and borehole sheets are presented in this Appendix as a documentation of field and laboratory observations and testing. The pertinent information on the logs should be considered in light of List of Abbreviations and the List of Symbols and with the following in mind:

(i) ELEV/DEPTH - The depths observed during drilling of a borehole or excavation of a test pit are recorded. Borehole depths are estimated by penetration of the drilling rods. Test pit depths are estimated by measuring down a slope face and correcting for slope inclination where possible. For test pits in sloping ground the uphill cut face has usually been logged and depth in such instances does not reflect depth at the center of the test pit. The coordinates of the boreholes and test pits shown in the location heading of the logs were determined by stadia and/or EDM survey methods. Test pit coordinates are the average of the pit end coordinates.

(ii) DESCRIPTION - The field description of the material is stated as completely as possible with respect to geological origin, material composition and gradation. It is noted that, as the project developed, familiarity with the stratigraphy and laboratory test results permitted more standardized classification of the materials in investigations subsequent to the initial July-August investigation.

(iii) STRATIGRAPHIC PLOT - An office classification of the materials is made, based on both field and laboratory data. The legend for this stratigraphic plot is shown below.

LEGEND



(iv) SAMPLE TYPE -

CS - chunk sample
DO - driven open
TO - shelby tube

(v) SAMPLE NUMBER - Consecutive numbering of all samples whether recovered or not.

(vi) BLOWS/FT - The number of blows of a 63.5 kg hammer dropped 76.2 cm required to drive a 75 mm or 50 mm dia. split steel tube a distance of 30 cm (Standard Penetration Test).

(vii) WATER CONTENT PERCENT - The weight of water in the soil expressed as a percentage of dry soil weight.

(viii) PIEZOMETER OR STANDPIPE INSTALLATION - All details pertinent to groundwater are recorded.

(ix) LABORATORY DATA - The laboratory data is presented adjacent to the samples for which it was obtained. The data includes:

- gradation, symbols as follows:

The percentage of each soil component is indicated

Cbl - cobble
G - gravel
S - sand
M - silt
C - clay

- organic content - weight of organics expressed as a percentage of air dry weight.

- unit weight expressed in kg/m^3 obtained by standard procedures or, for frozen samples, obtained by measured weight and volume measured by displacement of antifreeze.

- permeability expressed in cm/sec and determined
  a) in the field by falling or rising head permeability tests
  b) in the laboratory by compacting minus 20 mm material at field moisture content with Standard Proctor effort.

- strength testing is noted but presented in Appendix VI.

- other testing results or indication of testing for specific gravity, Standard Proctor Density, consolidation, and thaw consolidation parameters are indicated where appropriate.

- other data including details of water sample locations analyzed by Cyprus Anvil and estimated flow rates from air rotary advanced boreholes are noted on the logs.

RECORD OF BOREHOLE SHEETS

Borehole Nos. 1 to 41 incl.
46 to 52 incl.

Project No. 792-2025

RECORD OF BOREHOLE 79-1

2320·85 N ; 122·32 E
LOCATION (See Figure 3)

BORING DATE July 27, 1979

BOREHOLE TYPE Rotary

BOREHOLE DIAMETER 120 mm

SAMPLER HAMMER WEIGHT 63.5 kg. DROP 76.2 cm DATUM Ground Surface

ELEV. / DEPTH (m)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS / FOOT	ELEVATION SCALE	WATER CONTENT PERCENT W_P W W_L	PIEZOMETER OR STANDPIPE INSTALLATION / ADDITIONAL LAB. TESTING
1049.38 / 0.0	Fill Surface							Fill surface
	Gravel (Fill)							
1048.38 / 1.0	Alluvium, brown medium to coarse sand, some fine to medium gravel, occ. bedrock cobbles and boulders - compact to dense.		1	51mm D.O	31			W.L. 27 Sept 79
			2	"	54			Cement bentonite grout
	Shattered bedrock		3	"	>50			
	Mostly medium to coarse sand		4	"	29			19mm P.V.C. pipe
			5	"	24			
	Shattered bedrock		6	"	35			
1038·10 / 11.28	Brown to grey fine sandy SILT, occ. pebbles, low plasticity (Till-like composition		7	"	60			Peltonite seal
	Smooth drilling, very few pebbles		8	"	28			Slotted 25mm P.V.C. pipe
			9	"	72			coarse sandpack
1032.92 / 16.46	Shattered bedrock							Cave-in material
1031.54	Bedrock (black schist)							
17.84	End of Borehole							

VERTICAL SCALE 1 : 125

Golder Associates

B.H. 79-1

DRAWN RD
CHECKED DG

Project No. 792-2025

RECORD OF BOREHOLE 79-2

2377.04 N ; 740.06 E
LOCATION (See Figure 3)

BORING DATE July 28, 1979

BOREHOLE TYPE Rotary with air

BOREHOLE DIAMETER 120 mm

SAMPLER HAMMER WEIGHT 63.5 kg. DROP 76.2 cm

DATUM Approx. 1m below natural ground surface

SOIL PROFILE										PIEZOMETER OR STANDPIPE INSTALLATION / ADDITIONAL LAB. TESTING
ELEV. / DEPTH (m)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS / FOOT	ELEVATION SCALE	WATER CONTENT PERCENT W_P W W_L			
1128.63	Cleared Surface									
0.00	← W > Wp ←									
	Brown to brown grey fine sandy SILT TILL, very scattered boulders W < Wp									
1124.97										
3.66 / 1124.67	Schisty BEDROCK									
3.96	End of Borehole									

VERTICAL SCALE 1 : 50

Golder Associates

B.H. 79-2

DRAWN R.W.
CHECKED DG

Project No. 792-2025

RECORD OF BOREHOLE 79-3

2361·95N ; 814·80 E
LOCATION (See Figure 3)

BORING DATE July 28, 1979

BOREHOLE TYPE Rotary with air

BOREHOLE DIAMETER 120 mm

SAMPLER HAMMER WEIGHT 63.5kg. DROP 76.2cm

DATUM Approx. 0.6 m below natural ground surface

SOIL PROFILE									
ELEV. / DEPTH (m)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS / FOOT	ELEVATION SCALE	WATER CONTENT PERCENT W_P W W_L	PIEZOMETER OR STANDPIPE INSTALLATION / ADDITIONAL LAB. TESTING
1132.74 / 0.0	Cleared Surface							
	Brown grey fine sandy silt TILL							
1127.56	Hard drilling (bedrock)							
5.18	End of Borehole							

VERTICAL SCALE 1 : 50

Golder Associates BH. 79-3

DRAWN R.D.
CHECKED DG

Project No. 792-2025

RECORD OF BOREHOLE 79-4

2418·36 N; 833·67 E
LOCATION (See Figure 3)

BORING DATE July 28, 1979

BOREHOLE TYPE Rotary with air

BOREHOLE DIAMETER 120 mm

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 IN.

DATUM Approx. 0.3 m below natural ground surface

SOIL PROFILE									
ELEV. / DEPTH (m)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS / FOOT	ELEVATION SCALE	WATER CONTENT PERCENT W_P W W_L	PIEZOMETER OR STANDPIPE INSTALLATION / ADDITIONAL LAB. TESTING
1140.35	Cleared Surface							
0.0	Brown grey sandy silt TILL with significant proportions and inclusions of broken schist bedrock							
1134.56								
5.79	Schist bedrock. Drilling is easier and cuttings are plastic at about depth 7.9m							
1131.21								
9.14	End of Borehole							

VERTICAL SCALE 1:50

Golder Associates

B.H. 79-4

DRAWN R.D.
CHECKED DG

Project No. 792-2025

RECORD OF BOREHOLE 79-5

2439.05 N ; 147.66 E
LOCATION (See Figure 3)

BORING DATE July. 28, 1979

BOREHOLE TYPE Rotary with mud

BOREHOLE DIAMETER 114 mm

SAMPLER HAMMER WEIGHT 63.5 kg. DROP 76.2 cm

DATUM Ground Surface

SOIL PROFILE									
ELEV. / DEPTH (m)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS / FOOT	ELEVATION SCALE	WATER CONTENT PERCENT W_P W W_L	PIEZOMETER OR STANDPIPE INSTALLATION / ADDITIONAL LAB. TESTING
1077.37 0.0	Ground Surface							
	Sandy silt TILL and broken rock		1	51mm D.O.	21			
1074.32 3.05			2	"	>100			
	Frozen sandy silt TILL and broken rock, some coarse sand and gravel pockets. Easier drilling in vicinity of 5.2 m depth.		3	"	>100		G=36 S=51 M+C=13	No installations
1065.17 12.2	End of Borehole NOTE Borehole had to be abandoned due to continuous caving of borehole wall							

VERTICAL SCALE 1:125

Golder Associates

B.H. 79-5

DRAWN R.Q.
CHECKED DG

G.B. 102

PROJECT No. 792-2025

RECORD OF BOREHOLE 79-6

1924.93 N; - 26.74 E
LOCATION (See Figure 3)

BORING DATE July 29, 1979

DATUM Ground Surface

BOREHOLE TYPE Rotary with mud

BOREHOLE DIAMETER 120 mm

SAMPLER HAMMER WEIGHT 63.5 kg. DROP 76.2 cm.

PEN. TEST HAMMER WEIGHT - LB. DROP - INCHES

ELEVN. / DEPTH (m)	DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/FT.	ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS/FT. / SHEAR STRENGTH C_u, LB./SQ.FT.	COEFFICIENT OF PERMEABILITY k, CM./SEC. / WATER CONTENT, PERCENT W_P W W_L	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
1050.0	Ground Surface									Ground surface
0.0 1048.35	Gravel, cobbles and boulders some sand (alluvium)		1	51mm D.O.	23					W.L. 27 Sept 79
1.68	Variable alluvium Oxidized sandy and silty gravel, compact.		2	"	10				G-55 S-30 M+C-15	
			3	"	push					
	Oxidized silty fine to medium sand and gravel, compact		4	"	30					
			5	"	27					
			6	"	36					
	Boulder		7	"	22					Cement-bentonite grout
1029.91 20.92	Boulder		8	WS						19 mm P.V.C. pipe
	Grey silty medium to coarse sand, very dense (alluvium)		9	51mm D.O.	>100				G-5 S-74 M+C 21	Sand
1017.23										Peltonite seal
32.8	Dark grey clayey silt to sandy silt TILL, laminated, low plasticity, hard. - occasional cobbles									Sandpack Slotted 25mm P.V.C. pipe Caved-in material
1010.09										
39.94	End of Borehole									Permeability: 4.5×10^{-7} to 2.3×10^{-6} cm/s

15 / 10 - 5 Percent axial strain at failure

VERTICAL SCALE 1 : 125

Golder Associates

B.H. 79-6

DRAWN R.O.

CHECKED DG

G.B. 102

PROJECT No. 792-2025

RECORD OF BOREHOLE 79 - 7

2081·44 N ; - 43·55 E
LOCATION (See Figure 3)

BORING DATE July 31, 1979

DATUM Ground Surface

BOREHOLE TYPE Rotary with Mud

BOREHOLE DIAMETER 120 mm

SAMPLER HAMMER WEIGHT 63.5 kg. DROP 76.2 cm.

PEN. TEST HAMMER WEIGHT - LB. DROP INCHES

ELEVN. / DEPTH (m)	DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/FT.	ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS/FT. / SHEAR STRENGTH C_u, LB./SQ.FT.	COEFFICIENT OF PERMEABILITY k, CM./SEC. / WATER CONTENT, PERCENT (W_P W W_L)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
1048·34	Cleared surface									Ground surface
0.0	Alluvium: Oxidized fine to coarse SAND & GRAVEL, very dense (frozen?)] boulder [		1	51mm D.O.	62					W.L. 27 Sept 79
			2	"	60					
1042·55										
5.79	Alluvium, compact Brown gravelly coarse sand		3	"	>30					
			4	"	14				G·31 S·47 M+C -23	
			5	"	push					
	Oxidized coarse sand and fine to medium gravel.		6	"	23					Cement-bentonite grout
			7	"	13					
1036·14	] losing some circulation [									
12.2	Brown oxidized gravelly fine to coarse SAND, little silt, compact		8	"	24					
			9	"	46					
			10	"	18					19 mm P.V.C. pipe
			11	"	30				Cbl·1 G·40 S·45 M+C -14	
	] cobble [several cobbles & boulders									
1027·61			12	"	66					Peltonite seal
20.73	Grey sandy silt TILL, very dense, non plastic to low plasticity									Coarse sandpack
			13	"	>100				Cbl·1 G·54 S·28 M+C -17	Slotted 25mm P.V.C. pipe
1022·43										
25.91	End of Borehole									Permeability 1.69×10^{-5} to 3.17×10^{-5} cm/s

15 / 10 / 5 Percent axial strain at failure

VERTICAL SCALE 1 : 125

Golder Associates

B.H. 79 - 7

DRAWN R.D

CHECKED DG

Project No. 192-2025

RECORD OF BOREHOLE 79-8

1806.18N ; -92.71E
LOCATION (See Figure 3)

BORING DATE Aug. 2, 1979

BOREHOLE TYPE Rotory with air

BOREHOLE DIAMETER 120 mm

SAMPLER HAMMER WEIGHT 63.5 kg. DROP 76.2 cm DATUM Ground Surface

ELEV. / DEPTH (m)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS / FOOT	ELEVATION SCALE	WATER CONTENT PERCENT W_P W W_L (10 20 30 40)		PIEZOMETER OR STANDPIPE INSTALLATION / ADDITIONAL LAB. TESTING
1078.80	Cleared Surface								Cleared surface
0.0	Some boulders								
	Frozen greenish grey sandy silt TILL, no visible ice		1	51mm O.D.	48			G-20 S-34 M-36 C-10	
			2	"	62			G-20 S-30 M-34 C-16	Sand backfill
			3	"	86			G-12 S-33 M-35 C-20	25mm P.V.C. pipe
	] boulder [		4	"	100			G-37 S-34 M-24 C-5	
	] ice [Drills more easily - may not be frozen		5	"	100			G-15 S-40 M+C-45	
1068.43			6	"	100				
10.37	Brown to red-brown medium to coarse sand (alluvium) probably frozen								Caved-in material
1066.0									
12.8	] cobble / boulder [								
	] cobble [								
	shist bedrock								
1062.65									
16.15	End of Borehole								

VERTICAL SCALE 1:125

Golder Associates B.H. 79-8

DRAWN R.D.
CHECKED JG

Project No. 192-2025

RECORD OF BOREHOLE 79-9

2305 N 851 E
LOCATION (See Figure 3)

BORING DATE Aug. 2, 1979

BOREHOLE TYPE Rotary with air

BOREHOLE DIAMETER 114 mm

SAMPLER HAMMER WEIGHT 63.5 kg. DROP 76.2 cm DATUM Cleared 0.3 m below natural ground surface

SOIL PROFILE									PIEZOMETER OR STANDPIPE INSTALLATION / ADDITIONAL LAB. TESTING
ELEV. / DEPTH (m)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS / FOOT	ELEVATION SCALE	WATER CONTENT PERCENT W_P W W_L		
1141.82	Cleared Surface								
0.0	Olive brown to grey sandy to clayey silt TILL								No Installation
1137.55									
4.27	Schist BEDROCK								
1136.64									
5.18	End of Borehole								

VERTICAL SCALE 1:50

Golder Associates 79-9

DRAWN R.D
CHECKED JPG

Project No. 792-2025

RECORD OF BOREHOLE 79-9A

2394.65 N; 913.28 E
LOCATION (See Figure 3)

BORING DATE Aug. 2, 1979

BOREHOLE TYPE Rotary with air

BOREHOLE DIAMETER

SAMPLER HAMMER WEIGHT 63.5 kg. DROP 76.2 cm DATUM Cleared ground surface (removed organics)

SOIL PROFILE ELEV. / DEPTH (m)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS / FOOT	ELEVATION SCALE	WATER CONTENT PERCENT Wp W WL (10 20 30 40)	PIEZOMETER OR STANDPIPE INSTALLATION / ADDITIONAL LAB. TESTING
1141.82	Cleared Surface							
0.0	Brown grey clayey/ sandy silt TILL containing several pockets of shattered bedrock.		1	51mm D.O.	29		○	G-51 S-32 M+C-17 No Installations
			2	"	21		○	Cbl-10 G-55 S-20 M+C-15
1138.16 3.66	Schist Bedrock							
1137.25 4.57	End of Borehole							

VERTICAL SCALE 1:50

Golder Associates

B.H. 79-9A

DRAWN R.D.
CHECKED DG

Project No. 792-2025

RECORD OF BOREHOLE 79-10

2378·12 N; 979·62 E
LOCATION (See Figure 3)

BORING DATE Aug. 2, 1979

BOREHOLE TYPE Rotary with air

BOREHOLE DIAMETER 114 mm

SAMPLER HAMMER WEIGHT 63.5 kg. DROP 76.2cm DATUM Cleared ground surface

SOIL PROFILE ELEV. / DEPTH (m)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS / FOOT	ELEVATION SCALE	WATER CONTENT PERCENT W_P W W_L (10 20 30 40)	PIEZOMETER OR STANDPIPE INSTALLATION / ADDITIONAL LAB. TESTING
1142·82	Cleared Surface							
0.0 / 1142·22	Organic silt and debris							
0.6 / 1140·69	] cobble/boulder [Brown sandy silt TILL with high proportion of disturbed schist		1	51mm D.O			○ G - 62 S - 29 M+C - 9	No Installations
2.13 / 1138·25	Schist Bedrock			=2= refusal				
4.57	End of Borehole							

VERTICAL SCALE 1:50

Golder Associates B.H. 79-10

DRAWN R.D
CHECKED JG

RECORD OF BOREHOLE 79-11

Project No. 792-2025

2386.95N ; 1039.04E
LOCATION (See Figure 3)

BORING DATE Aug. 2, 1979

BOREHOLE TYPE Rotary with air

BOREHOLE DIAMETER 114 mm

SAMPLER HAMMER WEIGHT 63.5 kg. DROP 76.2 cm DATUM Cleared surface 0.6 m below natural ground surface

SOIL PROFILE									
ELEV. / DEPTH (m)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS / FOOT	ELEVATION SCALE	WATER CONTENT PERCENT W_P — W — W_L	PIEZOMETER OR STANDPIPE INSTALLATION / ADDITIONAL LAB. TESTING
1148.19	Cleared Surface							
0.0	Brown sandy silt TILL with high proportion of bedrock schist							No Installations
1145.75								
2.44	Weathered schist BEDROCK (dry)							
1143.62								
4.57	End of Borehole							

VERTICAL SCALE 1 : 50

Golder Associates B.H. 79-11

DRAWN R.D.
CHECKED DG

Project No. 192-2025

RECORD OF BOREHOLE 79-12

2290·54 N; 1267·85 E
LOCATION (See Figure 3)

BORING DATE Aug. 2, 1979

BOREHOLE TYPE Rotary with air

BOREHOLE DIAMETER 114 mm

SAMPLER HAMMER WEIGHT DROP

DATUM Cleared surface 0.3 m below natural ground surface

SOIL PROFILE									PIEZOMETER OR STANDPIPE INSTALLATION / ADDITIONAL LAB. TESTING
ELEV. / DEPTH (m)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS / FOOT	ELEVATION SCALE	WATER CONTENT PERCENT W_P W W_L (10 20 30 40)		
1151·91	Cleared Surface								
0.0	Brown sandy silt TILL, high proportion of bedrock schist								
1150·39								G-42 S-33 M-20 C-5	
1.52	Grey sandy/clayey silt TILL		1	51mm D.0	36		O		No Installations
1149·17									
2.74	Schist BEDROCK								
1147·95									
3.96	End of Borehole								

VERTICAL SCALE 1:50

Golder Associates B.H. 79-12

DRAWN R.D
CHECKED DG

Project No. 192-2025

RECORD OF BOREHOLE 79-13

2284.47 N; 1372.13 E
LOCATION (See Figure 3)

BORING DATE Aug. 2, 1979

BOREHOLE TYPE Rotary with air

BOREHOLE DIAMETER 114 mm

SAMPLER HAMMER WEIGHT 63.5 kg. DROP 76.2 cm DATUM Cleared surface 0.46 m below natural ground surface

SOIL PROFILE ELEV. / DEPTH (m)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS / FOOT	ELEVATION SCALE	WATER CONTENT PERCENT W_P W W_L (10 20 30 40)	PIEZOMETER OR STANDPIPE INSTALLATION / ADDITIONAL LAB. TESTING
1157.36	Cleared Surface							
0.0	Brown to dark grey sandy / clayey silt TILL		1	51mm O.D.	38		○	G-39 S-36 M-19 C-6 — No Installations
	] cobbly drilling [							
1154.01			2	"	55 refusal		○ 6.8%	G=20 S=35 M+C= 45
3.35	Grey becoming brown schist BEDROCK							
1152.79								
4.57	End of Borehole							

VERTICAL SCALE 1 : 50

Golder Associates B.H. 79-13

DRAWN R.D.
CHECKED DG

Project No. 792-2025

RECORD OF BOREHOLE 79 - 14

2237.31 N; 1249.75 E
LOCATION (See Figure 3)

BORING DATE Aug. 2, 1979

BOREHOLE TYPE Rotary with air

BOREHOLE DIAMETER 114 mm

SAMPLER HAMMER WEIGHT 63.5 kg. DROP 76.2cm

DATUM Cleared surface 1.2 m below natural ground surface

ELEV. / DEPTH (m)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS / FOOT	ELEVATION SCALE	WATER CONTENT PERCENT W_P W W_L	PIEZOMETER OR STANDPIPE INSTALLATION / ADDITIONAL LAB. TESTING
1144.76	Cleared Surface							
0.0	Brown grey sandy / clayey silt TILL							No Installations
1142.63								
2.13	Grey schist BEDROCK (dry)							
1140.19								
4.57	End of Borehole							

VERTICAL SCALE 1 : 50

Golder Associates B.H. 79-14

DRAWN R.D
CHECKED DG

G.B. 102

PROJECT No. 792-2025

RECORD OF BOREHOLE 79-15

2374·12 N; 57·74 E
LOCATION (See Figure 3)

BORING DATE Aug. 7, 1979

DATUM Ground Surface

BOREHOLE TYPE Rotary with mud

BOREHOLE DIAMETER 114 mm

SAMPLER HAMMER WEIGHT 63.5 kg. DROP 76.2 cm.

PEN. TEST HAMMER WEIGHT – LB. DROP INCHES

ELEVN. / DEPTH (m)	DESCRIPTION	NUMBER	TYPE	BLOWS/FT.	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
1056·10	Ground Surface					Ground surface
0.0	Oxidized sandy gravel and cobbles	1	51mm D.O.	–		
1053·66						
2.44	Brown coarse sand with some fine to medium, little fine to medium gravel, compact.	2	"	12		
		3	"	16	G-36 S-51 M+C -13	
		4	"	26		
1048·94						
7.16	← cobble →	5	"	39		W.L. ▽ 27 Sept 79
	Grey sandy to silty till or colluvium, high proportion of bedrock schist, red oxidation mottling in schist, compact to dense.	6	"	27		
		7	"	26	G-41 S-41 M+C -18	
		8	"	64		
	]black organic silt[	9	"	61		
	harder drilling	10	"	–		
	]cobble boulder[	11	"	>100		Cement/ bentonite grout
						19 mm P.V.C. pipe
	Occasional clayey silt pockets	12	"	46	G-30 S-51 M+C -19	Peltonite seal
1032·32						
23.78	Grey silty fine sand with some pebbles, loose to compact. Extensive loss of circulation below 24.4m and continuing to 32m	13	"	11	G-58 S-29 M+C -13	
		14	"	20	Organic Content 1.2%	Slotted 25 mm P.V.C. pipe
1027·44	Organic fine sandy silt with pebbles					
28.66	Grey silty sand and gravel (alluvium) very dense	15	"	>100		Coarse sandpack
1025·00						
31.10	Greenish grey becoming grey schist BEDROCK	16	"	>100		
		17		–		
		18	"	–	G_S = 2.77	Caved-in material
1017·99						
3811	End of Borehole					Permeability: 7.49 X10⁻⁶ to 1.61 X10⁻⁵ cm / s

DYNAMIC PENETRATION RESISTANCE BLOWS/FT.

SHEAR STRENGTH C_u, LB./SQ.FT.

COEFFICIENT OF PERMEABILITY k, CM./SEC.

WATER CONTENT, PERCENT — W_P W W_L — 10 20 30 40

15 / 10 5 Percent axial strain at failure

VERTICAL SCALE 1:125

Golder Associates

B.H. 79-15

DRAWN R.D.

CHECKED D.G.

G.B. 102

PROJECT No. 792-2025

RECORD OF BOREHOLE 79-15

2374·12 N; 57·74 E
LOCATION (See Figure 3)

BORING DATE Aug. 7, 1979 DATUM Ground Surface

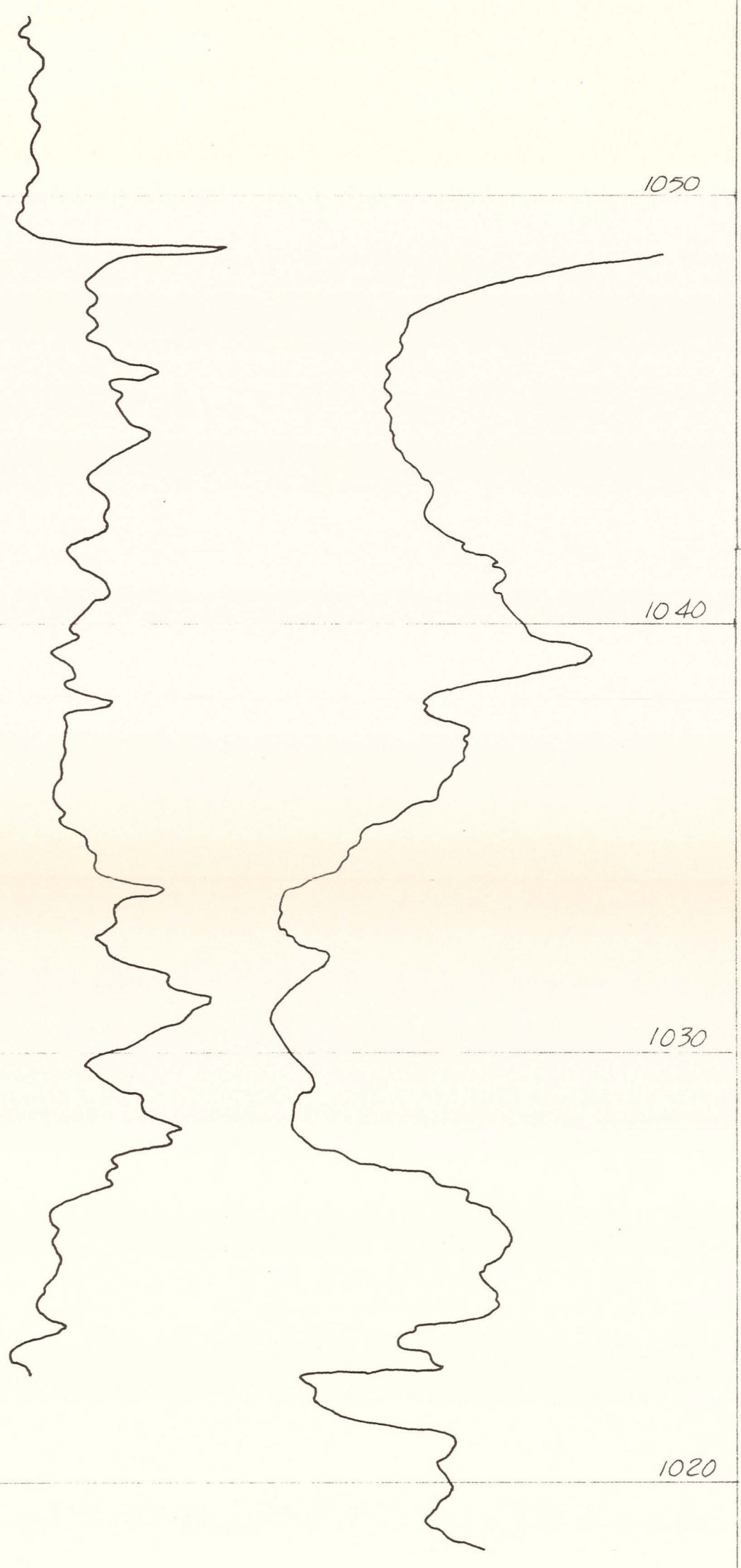
BOREHOLE TYPE Rotary with mud BOREHOLE DIAMETER 114 mm

SAMPLER HAMMER WEIGHT 63.5 kg. DROP 76.2 cm. PEN. TEST HAMMER WEIGHT – LB. DROP INCHES

ELEVN. / DEPTH (m)	DESCRIPTION	STRAT PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS/FT
1056·10	Ground Surface				
0.0	Oxidized sandy gravel and cobbles		1	51mm D.O.	–
1053·66					
2.44	Brown coarse sand with some fine to medium little fine to medium gravel, compact.		2	"	12
			3	"	16
			4	"	26
1048·94					
7.16	← cobble →		5	"	39
	Grey sandy to silty till or colluvium, high proportion of bedrock schist, red oxidation mottling in schist, compact to dense.		6	"	27
			7	"	26
			8	"	64
	] black organic [silt		9	"	61
	harder drilling		10	"	–
	] cobble boulder [		11	"	>100
	Occasional clayey silt pockets		12	"	46
1032·32					
23.78	Grey silty fine sand with some pebbles, loose to compact Extensive loss of circulation below 24.4m and continuing to 32m		13	"	11
			14	"	20
1027·44	Organic fine sandy silt with pebbles				
28.66	Grey silty sand and gravel (alluvium)				
1025·00	very dense		15	"	>100
31.10	Greenish grey becoming grey schist BEDROCK		16	"	>100
			17	"	–
			18	"	–
1017·99					
38·11	End of Borehole				

DOWN HOLE GEOPHYSICAL LOGGING

GAMMA/GAMMA RESISTIVITY



PIEZOMETER OR STANDPIPE INSTALLATION

Ground surface
W.L. 27 Sept 79
Cement/bentonite grout
19 mm P.V.C. pipe
Peltonite seal
Slotted 25 mm P.V.C. pipe
Coarse sandpack
Caved-in material

Permeability: 7·49 X10^{-6} to 1·61 X10^{-5}

VERTICAL SCALE 1:125

Golder Associates

B.H. 79-15

DRAWN R.D.
CHECKED DG

G.B. 102

PROJECT No. 792-2025

RECORD OF BOREHOLE 79-16

2256·12 N; 5·12 E
LOCATION (See Figure 3)

BORING DATE Aug. 9, 1979

DATUM Ground Surface

BOREHOLE TYPE Rotary with mud

BOREHOLE DIAMETER 114 mm

SAMPLER HAMMER WEIGHT 63.5 kg DROP 76.2 cm

PEN. TEST HAMMER WEIGHT – LB. DROP – INCHES

SOIL PROFILE			SAMPLES				DOWN HOLE GEOPHYSICAL LOGGING	PIEZOMETER OR STANDPIPE INSTALLATION
ELEVN. / DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	BLOWS/FT.		GAMMA/GAMMA RESISTIVITY	
1043.25	Fill Surface							Fill surface
0.0 1042.34	Gravel Fill							
0.91 1040.51	Muskeg and organic debris Frozen to 1.83m.		1	76mm D.O.	13			27 Sept 79 W.L. ▽ #1
2.74 1036.85	Medium to coarse SAND and GRAVEL, compact		2	"	13		El. 1040	Thermistor #2
			3	"	14			
			4	"	18			#3
6.40 1034.71	Compact grey medium to coarse sand some organic		5	"	20			
8.54	Compact interlayered sand and gravel (alluvium)		6	"	30			#4
			7	"	59			
			8	"	38			19 mm PVC pipe
	Coarse sand and fine gravel		9	"	30		1030	#5
	] loose to compact [		10	"	40			
			11	"	10			
1024.04	Losing some circulation		12	"	27			
19.21	Grey medium to coarse sandy silt, high proportion of broken bedrock dense		13	"	36			Peltonite seal
1019.47			14	"	38		1020	
23.78 1017.64	Dense grey silty fine to coarse SAND, little gravel		15	51 mm D.O.	>100			Sandpack
25.61 1015.20	Very hard schist BEDROCK							Slotted 25mm PVC pipe
28.05 1012.76	Relatively soft schist BEDROCK							Cave-in material
30.49	End of Borehole							Permeability: 5.89 X 10^{-6}

VERTICAL SCALE
1 : 125

Golder Associates

B.H. 79-16

DRAWN R.D.
CHECKED D.G.

G.B. 102

PROJECT No. 792-2025

RECORD OF BOREHOLE 79-16

2256·12 N; 5·12 E
LOCATION (See Figure 3)

BORING DATE Aug 9, 1979

DATUM Ground Surface

BOREHOLE TYPE Rotary with mud

BOREHOLE DIAMETER 114 mm

SAMPLER HAMMER WEIGHT 63.5 kg DROP 76.2 cm

PEN. TEST HAMMER WEIGHT – LB. DROP – INCHES

ELEVN. / DEPTH (m)	DESCRIPTION	NUMBER	TYPE	BLOWS/FT.
1046.25	Fill Surface			
0.0 / 1047.34	Gravel Fill			
0.91 / 1045.51	Muskeg and organic debris. Frozen to 1.83m	1	76mm D.O.	13
2.74 / 1043.68	Medium to coarse SAND and GRAVEL, compact	2	"	13
		3	"	14
		4	"	18
6.40 / 1039.71	Compact grey medium to coarse sand some organic	5	"	20
8.54	Compact Interlayered sand and gravel (alluvium)	6	"	30
		7	"	59
		8	"	38
	Coarse sand and fine gravel] loose to compact [	9	"	30
		10	"	40
		11	"	10
10.29·04	Losing some circulation	12	"	27
19.21	Grey medium to coarse sandy silt, high proportion of broken bedrock dense	13	"	36
1024.47		14	"	38
23.78 / 1022.64	Dense grey silty fine to coarse SAND, gravel	15	51mm D.O.	>100
25.61 / 1020.80	Very hard schist BEDROCK			
28.05 / 1017.76	Relatively soft schist BEDROCK			
30.49	End of Borehole			

DYNAMIC PENETRATION RESISTANCE BLOWS/FT.

SHEAR STRENGTH C_u, LB./SQ.FT

TEMPERATURE °C △ (-0.5, 0, +0.5, +1.0)

WATER CONTENT, PERCENT W_P W W_L (10, 20, 30, 40)

Organic Content 16.3% → >110%

Sept. 27, 1979 ②

ADDITIONAL LAB. TESTING

G-61 S-32 M+C -7

Cbl-1 G-31 S-59 M+C -9

Cbl-8 G-64 S-23 M+C 5

G-65 S-25 M+C -18

G-70 S-21 M-7 C-2

G-54 S-36 M+C -10

PIEZOMETER OR STANDPIPE INSTALLATION

Fill surface
27 Sept 79 W.L
#1
Thermistor #2
#3
19 mm PVC pipe
#4
#5
Peltonite seal
Sandpack
Slotted 25mm PVC pipe
Cave-in material

15-○-5 Percent axial strain at failure (10)

Permeability: 5.89×10^{-6} cm/s

VERTICAL SCALE 1 : 125

Golder Associates

B.H. 79-16

DRAWN R.D.

CHECKED D.G.

Project No. 792-2025

RECORD OF BOREHOLE 79-17

2448·63 N ; 109·94 E
LOCATION (See Figure 3)

BORING DATE 11 Aug 1979

BOREHOLE TYPE Rotary with mud

BOREHOLE DIAMETER 114 mm

SAMPLER HAMMER WEIGHT 63·5 kg DROP 76·2 cm

DATUM Ground surface

ELEV. / DEPTH (m)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS / 300 mm	ELEVATION SCALE	WATER CONTENT PERCENT W_P — W — W_L	PIEZOMETER OR STANDPIPE INSTALLATION / ADDITIONAL LAB. TESTING
1076·83	Ground surface							
0·00	↑ Unfrozen ↑ Sandy SILT TILL (frozen)							
	Boulder							
1072·56								
4·27	Olive brown silty fine SAND and fine sandy SILT							
1071·34								
5·49	↖ cobbly drilling ↗ Fine GRAVEL and coarse SAND, mostly frozen, some broken rock zones. Caving of frozen gravel tends to mask cuttings. Refer to BH.79-20 for more accurate stratigraphy		1	76mm D.O.	50/140mm			
			2	51mm D.O.	>100			
1065·86								
10·97	Grey brown fine sandy SILT TILL							
1064·64								
12·19	End of Borehole							

VERTICAL SCALE 1 : 125

Golder Associates BH.79-17

DRAWN RK
CHECKED DG

G.B. 102

PROJECT No. 792-2025

RECORD OF BOREHOLE 79-18

2138·41 N; 44·98 E
LOCATION (See Figure 3)

BORING DATE 18 Aug 1979 DATUM Fill surface

BOREHOLE TYPE Rotary with mud BOREHOLE DIAMETER 114 mm

SAMPLER HAMMER WEIGHT 63·5kg DROP 76·2 cm PEN. TEST HAMMER WEIGHT DROP

ELEVN. / DEPTH (m)	DESCRIPTION	NUMBER	TYPE	Blows/300mm
1049·23	Fill surface			
0·00	Gravel FILL			
1047·71				
1·52	Frozen black organic SILT, pieces of wood & ice crystals to 12 mm dia.	1	76mm DIA.	>100
1043·13				
6·10	Frozen, brown to grey layered fine to medium SANDS, some fine gravelly layers	2	"	>100
		3 A/B	"	>100
		4	"	>100
1038·87				
10·36	Layered grey & brown fine SAND & coarse SAND to fine GRAVEL, trace organics marginally compact	5 A/B	"	11
		6	"	12
1036·12				
13·11	Layered grey fine to medium SAND & non-plastic SILT, some organic, compact	7	"	13
		8	"	20
		9	"	13
		10	"	15
		11	"	20
	Scattered pebbles	12	"	22
1024·85				
24·38	Grey sandy GRAVEL, dense	13	"	51
		14	"	38
	] Boulder			
1017·23				
32·00	Grey fine to coarse SAND, little medium gravel, very dense	15	"	79
1015·09				
34·14	Possibly fractured BEDROCK			
1014·18				
35·05	Dark grey schist BEDROCK			
1008·08				
41·15	End of Borehole			

Consolidation Test
Gs 2.66

TEMPERATURE, °C: −0·5 0 +0·5 +1·0

WATER CONTENT, PERCENT: W_P W W_L — 10 20 30 40

organic content 2.9%

Sept. 27, 1979

ADDITIONAL LAB. TESTING	
G-43 S-30 M-24 C-3	
G-56 S-39 M+C -5	
3A G-6 S-86 M+C -8	3B G-41 S-55 M+C -4
G-4 S-84 M+C -12	
5A G-29 S-65 M+C -6	5B G-42 S-57 M+C -1
G-20 S-73 M+C 7	
S-2 M-87 C-11	
G-14 S-6 M-62 C-18	
Cbl-5 G-64 S-22 M+C -9	
Cbl-2 G-27 S-62 M+C 9	

PIEZOMETER OR STANDPIPE INSTALLATION

Fill surface
W.L. 27 Sept 79
Thermistor #1
#2
#3
#4
#5
19 mm PVC pipe
Peltonite seal
Petur pneumatic piezometer
Sandpack
Cave-in material

15-○-5 Percent axial strain at failure
10

VERTICAL SCALE 1:125

Golder Associates BH. 79-18

DRAWN RK
CHECKED DG

G.B. 102

PROJECT No. 792-2025

RECORD OF BOREHOLE 79-18

2138·41 N; 44·98 E
LOCATION (See Figure 3)

BORING DATE 13 Aug 1979 DATUM Fill surface

BOREHOLE TYPE Rotary with mud BOREHOLE DIAMETER 114 mm

SAMPLER HAMMER WEIGHT 63·5 kg DROP 76·2 cm PEN. TEST HAMMER WEIGHT DROP

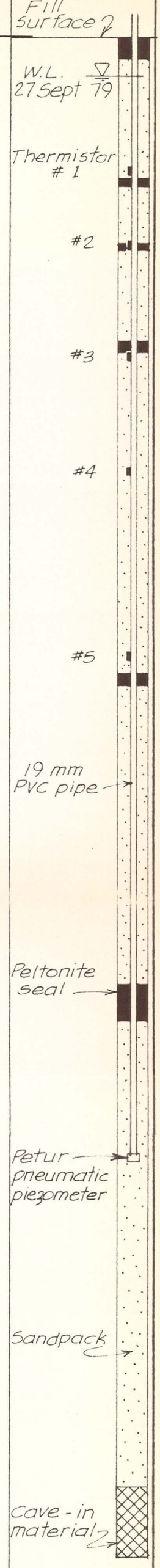
SOIL PROFILE ELEVN/DEPTH (m)	DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	Blows/300mm
1049·23	Fill surface				
0·00 / 1047·71	Gravel FILL				
1·52 / 1043·13	Frozen black organic SILT, pieces of wood & ice crystals to 12 mm dia.		1	76 mm Dia.	>100
6·10 / 1038·87	Frozen, brown to grey layered fine to medium SANDS, some fine gravelly layers		2	"	>100
			3	"	>100
			4	"	>100
10·38 / 1036·12	Layered grey & brown fine SAND & coarse SAND to fine GRAVEL, trace organics marginally compact		5	"	11
			6	"	12
13·11	Layered grey fine to medium SAND & non-plastic SILT, some organic, compact		7	"	13
			8	"	20
			9	"	13
			10	"	15
	Scattered pebbles		11	"	20
1024·85			12	"	22
24·38	Grey sandy GRAVEL, dense		13	"	51
	] Boulder				
1017·23					
32·00 / 1015·09	Grey fine to coarse SAND, little medium gravel, very dense		15	"	79
34·14 / 1014·18	Possibly fractured BEDROCK				
35·05	Dark grey schist BEDROCK				
1008·08					
41·15	End of Borehole				

DOWN HOLE GEOPHYSICAL LOGGING

GAMMA / GAMMA RESISTIVITY

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION



VERTICAL SCALE
1: 125

Golder Associates BH. 79-18

DRAWN RK
CHECKED DG

G.B./102

PROJECT No. 792-2025

RECORD OF BOREHOLE 79-19

2216·56 N; 73·73 E
LOCATION (See Figure 3)

BORING DATE 15 Aug 1979

DATUM Fill surface

BOREHOLE TYPE Rotary with mud

BOREHOLE DIAMETER 114 mm

SAMPLER HAMMER WEIGHT 63·5 kg DROP 76·2 cm

PEN. TEST HAMMER WEIGHT DROP

ELEVN / DEPTH (m)	DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/300mm	ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS/FT. / SHEAR STRENGTH C_u, LB./SQ.FT	WATER CONTENT, PERCENT W_P W W_L (10 20 30 40)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
1048·26	Fill surface									Fill surface
0·00	Gravel FILL									W.L. 27 Sept 79
1046·74										Cuttings backfill
1·52	Organic SILT & debris, loose to compact		1	76mm DO	14					
1042·16									γ_d - 1820 Kg/m³	G- 55 S- 8 m- 33 C- 4
6·10	Silty TILL with high proportion of schist BEDROCK									Void
										19mm dia PVC pipe
	possibly alluvium									Peltonite seal
										Sandpack
										Slotted 25 mm PVC pipe
1028·75										
19·51	Gravelly SAND, some loss of circulation									Cave-in material
1026·31										
21·95	Schist BEDROCK, slightly softer drilling below 22·86 m									
1023·88										
24·38	End of Borehole									

NOTE: Stratigraphy based on interpretation of cuttings and drilling action and therefore may vary from that shown.

15 / 10 / 5 Percent axial strain at failure

Permeability: $8·11 \times 10^{-5}$ cm/s

VERTICAL SCALE 1:125

Golder Associates BH. 79-19

DRAWN RK

CHECKED DG

G.B. 102

PROJECT No. 792-2025

RECORD OF BOREHOLE 79-19

2216.56 N; 73.73 E
LOCATION (See Figure 3)

BORING DATE 15 Aug 1979

DATUM Fill surface

BOREHOLE TYPE Rotary with mud

BOREHOLE DIAMETER 114 mm

SAMPLER HAMMER WEIGHT 63.5 kg DROP 76.2 cm

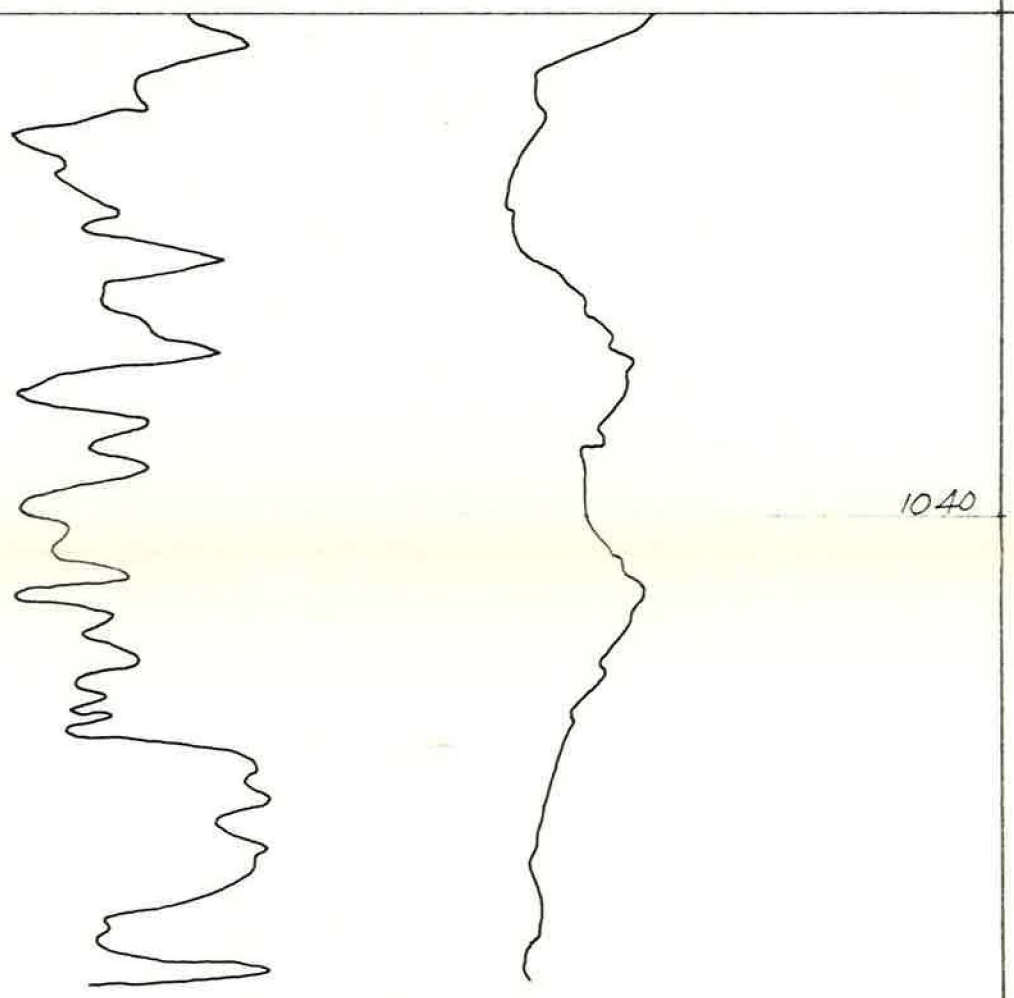
PEN. TEST HAMMER WEIGHT

DROP

SOIL PROFILE ELEVN / DEPTH (m)	DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/300mm
1048.26	Fill surface				
0.00 / 1046.74	Gravel FILL				
1.52 / 1042.16	Organic SILT & debris, loose to compact		1	76mm O.D.	14
6.10 / 1028.75	Silty TILL with high proportion of schist BEDROCK - possibly alluvium				
19.51 / 1026.31	Gravelly SAND, some loss of circulation				
21.95 / 1023.88	Schist BEDROCK, slightly softer drilling below 22.86 m				
24.38	End of Borehole				

DOWN HOLE GEOPHYSICAL LOGGING

GAMMA / GAMMA RESISTIVITY



ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

Permeability: 8.11 x 10^{-5}

VERTICAL SCALE 1:125

Golder Associates BH. 79-19

DRAWN RK

CHECKED DG

RECORD OF BOREHOLE 79-20

2447·49 N; 114·36 E
LOCATION (See Figure 3)

BORING DATE 16 Aug 1979

DATUM Ground surface

BOREHOLE TYPE Rotary with air

BOREHOLE DIAMETER 149mm

SAMPLER HAMMER WEIGHT 63·5 kg DROP 76·2 cm

PEN. TEST HAMMER WEIGHT

DROP

ELEVN / DEPTH (m)	DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	SAMPLES TYPE	BLOWS/300mm
1076·80	Ground surface				
0·00	Brown fine sandy SILT TILL		1	W.S.	
1073·75					
3·05	Brown silty GRAVEL, coarse SAND		2	"	
1072·84					
3·96	Brown fine sandy SILT		3	"	
			4	"	
1070·09					
6·71	Grey fine sandy SILT TILL, some fine gravel		5	"	
1067·96					
8·84	Coarse SAND & fine GRAVEL		6	"	
1067·05					
9·75	Frozen grey fine sandy SILT TILL, low plasticity, occ. ice cuttings to 12 mm dia.		7	"	
			8	"	
1063·69					
13·11	Grey fine sandy SILT TILL, some containing high proportion of rock		9	"	
			10	"	
	TILL becoming much harder & drier		11	"	
			12	"	
1057·60					
19·20	Brown grey - becoming clayey, low plasticity		13	"	
	Grey SILT, non-plastic to low plasticity (rock flour)		14	"	
			15	"	
			16	"	
			17	"	
1047·54	Gravelly coarse SAND		18	"	
29·26					
			19	"	
	Grey schisty BEDROCK		20	"	
			21	"	
			22	"	
1039·61					
37·19	End of Borehole				

ELEVATION SCALE

DYNAMIC PENETRATION RESISTANCE BLOWS/FT.

SHEAR STRENGTH C_u, LB./SQ.FT.

TEMPERATURE, °C
-0·5 0 +0·5 +1·0

WATER CONTENT, PERCENT
W_P W W_L
10 20 30 40

Sept. 27, 1979

March 9, 1980 thermister began to function.

G_s = 2.77

NOTE: Pneumatic piezometer & Thermistor #9 not functional. Presumably these were destroyed during installation.

15 10 5 Percent axial strain at failure

ADDITIONAL LAB. TESTING

+2·9
G·58 S·32 M+C ·10
G·1 S·50 M·41 C·8
G·19 S·40 M·28 C·13
G·52 S·40 M+C ·8
G·28 S·43 M·27 C·8
G·5 S·47 M·34 C·14
G·16 S·51 M+C ·33
G·28 S·32 M+C ·40
S·4 M·68 C·28
G·1 S·3 M·72 C·24
G·66 S·33 C·1

PIEZOMETER OR STANDPIPE INSTALLATION

Ground surface
Thermistor #2
#3
#1
#4
#5
Peltonite seal
#6
#7
19 mm PVC pipe
Pneumatic W.L. 27 Sept 79
#8
Petur pneumatic piezometer
Sandpack
Slotted 25mm PVC pipe
#9

Permeability: 3.64×10^{-6} to 4.18×10^{-5} cm/s

VERTICAL SCALE 1:125

Golder Associates BH. 79-20

DRAWN RK

CHECKED DG

RECORD OF BOREHOLE 79-20

2447·49 N; 114·36 E
LOCATION (See Figure 3)

BORING DATE 16 Aug 1979

DATUM Ground surface

BOREHOLE TYPE Rotary with air

BOREHOLE DIAMETER 149 mm

SAMPLER HAMMER WEIGHT 63·5 kg DROP 76·2 cm

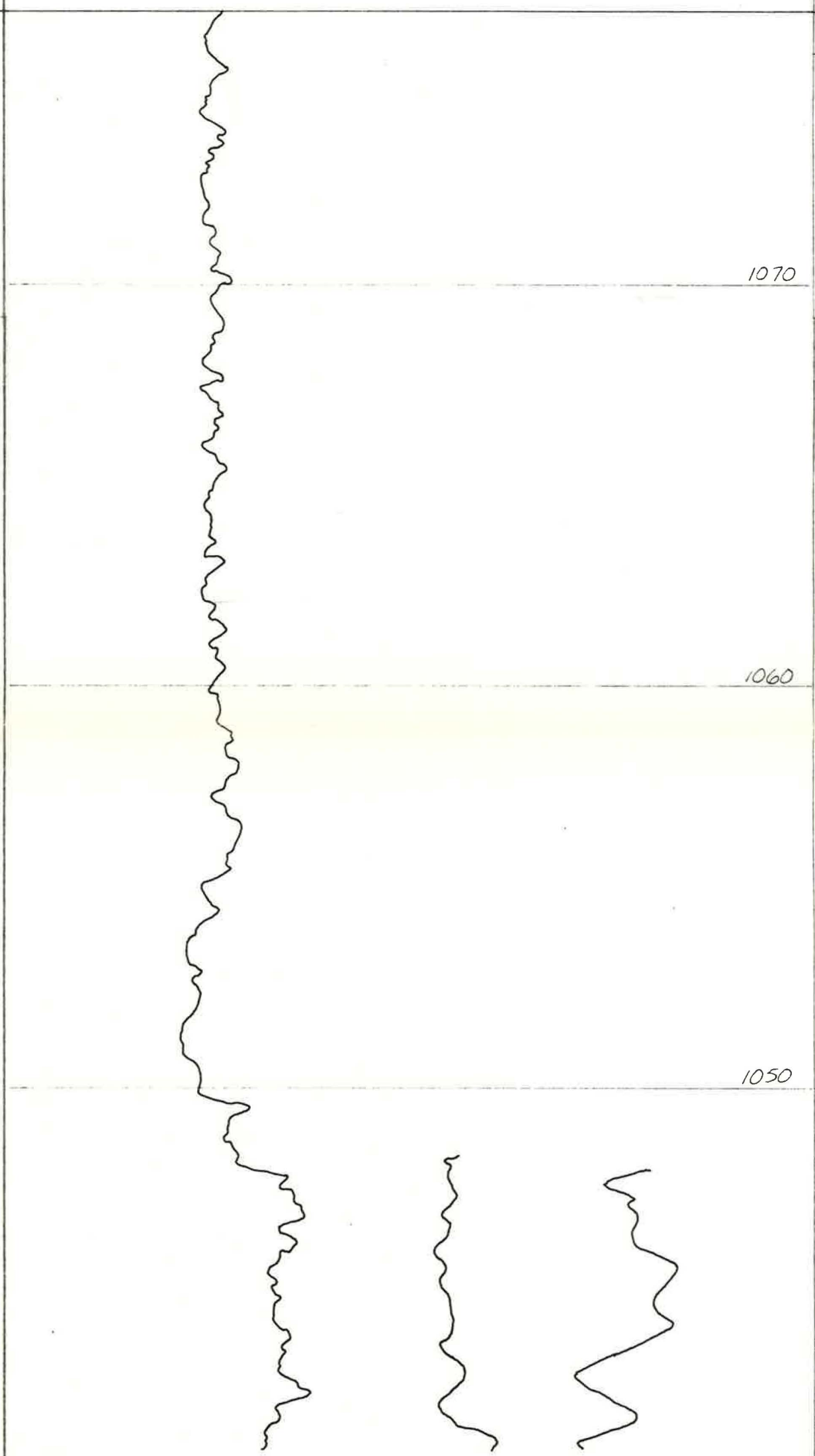
PEN. TEST HAMMER WEIGHT

DROP

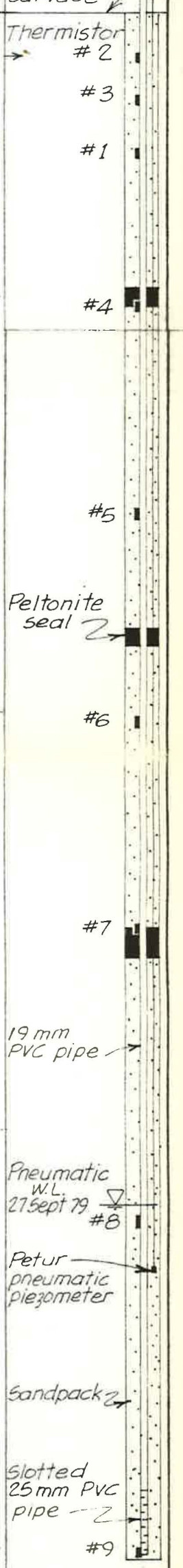
SOIL PROFILE ELEVN. / DEPTH (m)	DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	BLOWS/300mm
107·80	Ground surface				
0·00	Brown fine sandy SILT TILL		1	W.S.	
1073·75					
3·05	Brown silty GRAVEL & coarse SAND		2	"	
1072·84					
3·96	Brown fine sandy SILT		3	"	
1070·09			4	"	
6·71	Grey fine sandy SILT TILL, some fine gravel		5	"	
1067·96					
8·84	Coarse SAND & fine GRAVEL		6	"	
1067·05					
9·75	Frozen grey fine sandy SILT TILL, low plasticity, occ. ice cuttings to 12 mm dia.		7	"	
			8	"	
1063·69					
13·11	Grey fine sandy SILT TILL, some containing high proportion of rock		9	"	
			10	"	
	TILL becoming much harder & drier		11	"	
			12	"	
1057·60					
19·20	←Brown grey→		13	"	
			14	"	
	Grey SILT, non-plastic to low plasticity (rock flour)		15	"	
			16	"	
	-becoming clayey, low plasticity		17	"	
1041·54	Gravelly coarse SAND		18	"	
29·26					
	Grey schisty BEDROCK		19	"	
			20	"	
			21	"	
			22	"	
1039·61					
37·19	End of Borehole				

DOWN HOLE GEOPHYSICAL LOGGING

NATURAL GAMMA GAMMA/GAMMA RESISTIVITY



PIEZOMETER OR STANDPIPE INSTALLATION



Permeability: 3.64×10^{-6} to 4.18×10^{-5}

VERTICAL SCALE
1:125

Golder Associates BH. 79-20

DRAWN RK
CHECKED DG

RECORD OF BOREHOLE 79-21

1974.58 N ; -119.73 E
LOCATION (See Figure 3)

BORING DATE 16 Aug 1979 DATUM Cleared ground surface

BOREHOLE TYPE Rotary with mud BOREHOLE DIAMETER 114 mm

SAMPLER HAMMER WEIGHT 63.5 kg DROP 76.2 cm PEN TEST HAMMER WEIGHT DROP

ELEVN / DEPTH (m)	DESCRIPTION	NUMBER	TYPE	BLOWS/300mm
1050.07	Cleared surface			
0.00	COBBLES & BOULDERS			
1047.94				
2.13	Grey fine sandy SILT TILL or COLLUVIUM, low plasticity, loose	1	76mm OD	7
1046.11				
3.96	Brown, oxidized ALLUVIUM, including coarse SAND, fine gravel & silty gravel, compact to dense	2	"	27
		3	"	>100
1043.06				
7.01	Coarse SAND & fine GRAVEL	4	"	38
		5	CS.	-
		6	76mm OD	53
	Layered grey & brown silty fine SAND & fine sandy SILT, dense	7	"	45
1034.53		8	1 1/8"	>100
15.54	Grey layered SAND & GRAVEL ALLUVIUM, very dense	9	"	>100
		10	"	>100
1025.69				
24.38	Layered med. SAND & TILL	11	"	>100
	Grey sandy SILT TILL, very dense	12	"	>100
1015.93				
34.14	Grey schist BEDROCK			
1013.49				
36.58	End of Borehole			

DYNAMIC PENETRATION RESISTANCE BLOWS/FT. ----------

SHEAR STRENGTH C_u, LB./SQ.FT

COEFFICIENT OF PERMEABILITY k, CM./SEC.

WATER CONTENT, PERCENT
W_P W W_L
10 20 30 40

ADDITIONAL LAB. TESTING

G - 25
S - 19
M+C - 56

G - 55
S - 36
M+C - 9

G - 37
S - 15
M - 30
C - 18

S - 12
M - 83
C - 5

G - 11
S - 39
M - 17
C - 3

PIEZOMETER OR STANDPIPE INSTALLATION

Cleared surface

W.L. 27 Sept 79

19 mm dia. PVC pipe

Peltonite seal

Sandpack

Slotted 25mm PVC pipe

Permeability: 6.33×10^{-6} cm/s

15-○-5 Percent axial strain at failure
10

VERTICAL SCALE 1:125

Golder Associates BH. 79-21

DRAWN RK
CHECKED DG

RECORD OF BOREHOLE 79-21

1974·58 N ; -119·73 E
LOCATION (See Figure 3)

BORING DATE 16 Aug 1979

DATUM Cleared ground surface

BOREHOLE TYPE Rotary with mud

BOREHOLE DIAMETER 114 mm

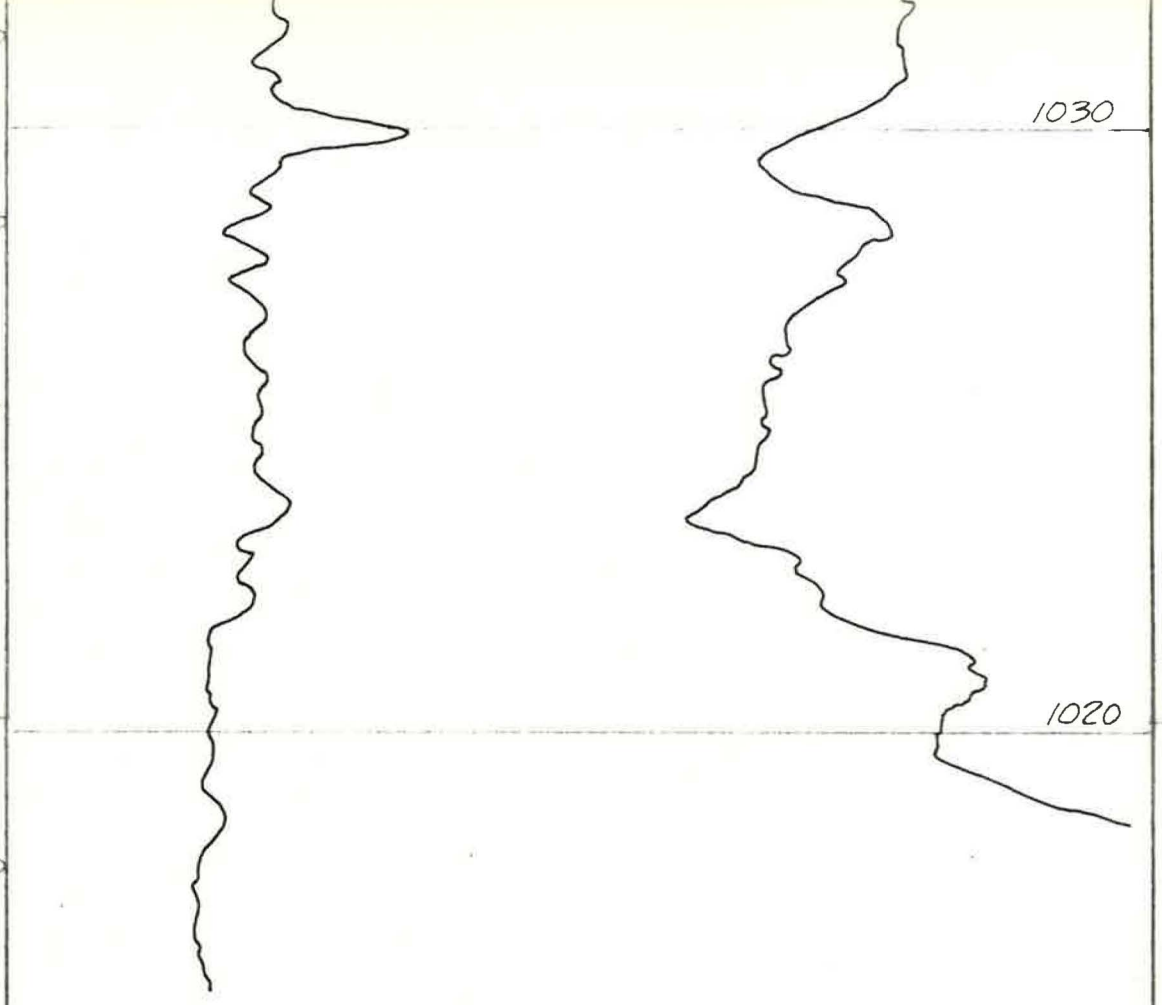
SAMPLER HAMMER WEIGHT 63·5 kg DROP 76·2 cm

PEN. TEST HAMMER WEIGHT DROP

ELEVN / DEPTH (m)	DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/300mm
1050·07	Cleared surface				
0·00	COBBLES & BOULDERS				
1047·94					
2·13	Grey fine sandy SILT TILL or COLLUVIUM, low plasticity, loose		1	76mm DO	7
1046·11					
3·96	Brown, oxidized ALLUVIUM, including coarse SAND, fine gravel & silty gravel, compact to dense		2	"	27
			3	"	>100
1043·06					
7·01	Coarse SAND & fine GRAVEL		4	"	38
			5	CS.	-
			6	76mm DO	53
	Layered grey & brown, silty fine SAND & fine sandy SILT, dense		7	"	45
1034·53			8	"	>100
15·54	Grey layered SAND & GRAVEL ALLUVIUM, very dense		9	"	>100
			10	"	>100
1025·69					
24·38	Layered med. SAND & TILL		11	"	>100
	Grey sandy SILT TILL, very dense		12	"	>100
1015·93					
34·14	Grey schist BEDROCK				
1013·49					
36·58	End of Borehole				

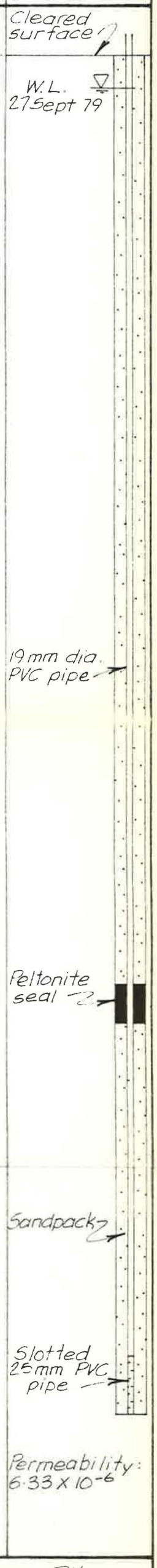
DOWN HOLE GEOPHYSICAL LOGGING

GAMMA / GAMMA RESISTIVITY



ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION



Permeability: 6.33×10^{-6}

VERTICAL SCALE 1:125

Golder Associates BH. 79-21

DRAWN RK

CHECKED DG

Project No. 792-2025

RECORD OF BOREHOLE 79-22

1869.83 N ; -198.86 E
LOCATION (See Figure 3)

BORING DATE 19 Aug 1979

BOREHOLE TYPE Rotary with mud

BOREHOLE DIAMETER 114 mm

SAMPLER HAMMER WEIGHT 63.5 kg DROP 76.2cm DATUM Cleared ground surface

SOIL PROFILE									PIEZOMETER OR STANDPIPE INSTALLATION / ADDITIONAL LAB. TESTING
ELEV. / DEPTH (m)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS / 300 mm	ELEVATION SCALE	WATER CONTENT PERCENT W_P W W_L		
1079.20	Cleared ground surface								
0.00	Frozen olive brown to grey fine sandy SILT TILL, occ. coarse sand and gravel pockets		1	76mm DO.	>100			G-5.9 S-26 M+C -15	No installations
			2	"	>100				
1070.06									
9.14	End of Borehole								

NOTE: Borehole abandoned due to loss of pit seal. Four boreholes were attempted with mud in this area, but all had to be abandoned. Borehole 79-22A was drilled with air

VERTICAL SCALE 1:125

Golder Associates BH. 79-22

DRAWN RK
CHECKED DG

Project No. 792 2025

RECORD OF BOREHOLE 79-22A

1840.84 N; -185.90 E
LOCATION (See Figure 3)

BORING DATE 21 Aug 1979

BOREHOLE TYPE Rotary with air

BOREHOLE DIAMETER 114 mm

SAMPLER HAMMER WEIGHT 63.5 kg DROP 76.2 cm DATUM Cleared ground surface

ELEV. / DEPTH (m)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS / 300 mm	ELEVATION SCALE	TEMPERATURE, °C (-0.5, 0, +0.5, +1.0) / WATER CONTENT PERCENT (Wp, W, WL)	PIEZOMETER OR STANDPIPE INSTALLATION / ADDITIONAL LAB. TESTING
1081.90	Cleared ground surface							Cleared surface
0.00	Frozen alternating brown & grey fine sandy SILT TILL, non-plastic to low plasticity, occ. organic layers							19mm dia. PVC pipe
1079.46								
2.44	Frozen grey to greenish grey sandy SILT TILL							Thermistor #1, #2, #3, #4, #5
	easier drilling							Sept. 27/79
1066.66								Sandpack
15.24	organic		1	W.S				Cave-in material
1063.61	Dark grey SILT, non-plastic, occ. cobble or pebble							
18.29	End of Borehole							

VERTICAL SCALE 1:125

Golder Associates BH. 79-22A

DRAWN RK
CHECKED DG

Project No. 792-2025

RECORD OF BOREHOLE 79-23

1910·41 N ; -387·72 E
LOCATION (See Figure 3)

BORING DATE 22 Aug 1979

BOREHOLE TYPE Rotary with mud to 2·2m; air to 18·29 m

BOREHOLE DIAMETER 114 mm

SAMPLER HAMMER WEIGHT 63·5 kg DROP 76·2 cm DATUM Cut ground surface

ELEV. / DEPTH (m)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS / 300 mm	ELEVATION SCALE	WATER CONTENT PERCENT (Wp, W, WL; 10 20 30 40)	PIEZOMETER OR STANDPIPE INSTALLATION / ADDITIONAL LAB. TESTING
1083·17	Cut ground surface							Ground surface
0·00	] Ice seams to 12 mm thickness		1	76mm D.O.	>100		G-35 S-31 M+C-34	19 mm dia. PVC pipe
	Frozen brown to grey fine sandy SILT TILL, some visible ice near surface		2	"	>100		Cbl-1 G-29 S-46 M+C-24	Peltonite seal
	] organic							Sandpack
1074·03							Cbl-G G-45 S-36 M+C-13	W.L. 27 Sept 79
9·14	]Organic black fine to coarse SAND & GRAVEL (appears to be unfrozen)		3	"	90			Petur pneumatic piezometer
	] cobble							
1070·06	SAND & GRAVEL							
13·11	Grey fine sandy SILT TILL							
	] Boulder							
1064·88								
18·29	End of Borehole							

NOTE: Borehole abandoned due to rotting of permafrost & caving. Hole size at surface ≈ 0·7m dia.

NOTE: Piezometer installed in auxilliary borehole drilled with air, about 4m East of BH. 79-23

VERTICAL SCALE 1:125

Golder Associates BH. 79-23

DRAWN RK
CHECKED DG

Project No. 792-2025

RECORD OF BOREHOLE 79-25

1050.57N; 1432.59E
LOCATION (See Figure 3)

BORING DATE 27 Aug 1979

BOREHOLE TYPE Rotary with air

BOREHOLE DIAMETER 114 mm

SAMPLER HAMMER WEIGHT 63.5kg DROP 76.2cm DATUM Cleared ground surface

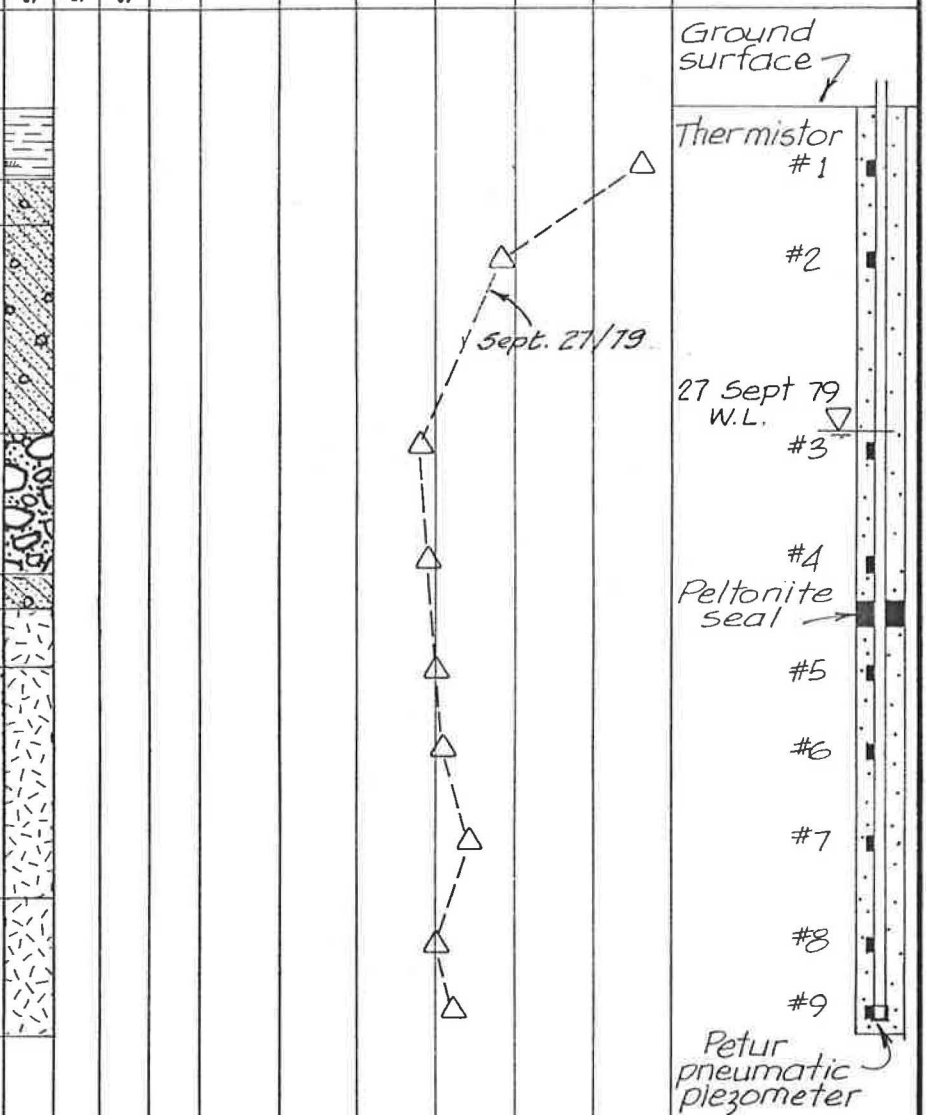
ELEV. / DEPTH (m)	DESCRIPTION
1087.00	Cleared ground surface
0.00 / 1086.09	Organic SILT & debris
0.91 / 1085.48	Sandy SILT TILL
1.52 / 1082.73	Frozen SILT TILL and schisty TILL
4.27 / 1080.90	SCHIST BOULDER
6.10	Sandy SILT TILL
6.55 / 1079.68	SCHIST BEDROCK
7.32 / 1076.64	Relatively hard SCHIST BEDROCK
10.36 / 1074.81	Relatively soft SCHIST BEDROCK
12.19	End of Borehole

SOIL PROFILE | STRATIGRAPHY PLOT | SAMPLE NUMBER | SAMPLE TYPE | BLOWS / 300 mm | ELEVATION SCALE

TEMPERATURE, °C
-0.5 0 +0.5 +1.0

WATER CONTENT PERCENT
W_P W W_L

PIEZOMETER OR STANDPIPE INSTALLATION
ADDITIONAL LAB. TESTING



VERTICAL SCALE 1: 100

Golder Associates BH. 79-25

DRAWN RK
CHECKED DG

Project No. 792-2025

RECORD OF BOREHOLE 79-26

2379.13 N ; 126.90E
LOCATION (See Figure 3)

BORING DATE 27 Aug 1979

BOREHOLE TYPE Rotary with air

BOREHOLE DIAMETER 114 mm

SAMPLER HAMMER WEIGHT 63.5 kg DROP 76.2cm DATUM Ground surface

SOIL PROFILE									PIEZOMETER OR STANDPIPE INSTALLATION / ADDITIONAL LAB. TESTING
ELEV. / DEPTH (m)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS / 300mm	ELEVATION SCALE	WATER CONTENT PERCENT W_P W W_L 10 20 30 40		
1061.99	Ground surface								
0.00	Organic SILT & debris								
0.61 / 1059.86	Brown fine sandy SILT TILL								
2.13	← organics → ⊐ boulder Red brown fine to coarse SAND & GRAVEL ALLUVIUM								No installations Borehole backfilled with sand, Borehole dry
1054.98									
7.01	End of Borehole								

VERTICAL SCALE 1:125

Golder Associates BH. 79-26

DRAWN RK CHECKED DG

G.B. 102

PROJECT No. 792-2025

RECORD OF BOREHOLE 79-24

1161·40 N ; 1222·52 E
LOCATION (See Figure 3)

BORING DATE 22 Aug 1979

DATUM Cleared ground surface

BOREHOLE TYPE Rotary with mud

BOREHOLE DIAMETER 114 mm

SAMPLER HAMMER WEIGHT 63·5 kg DROP 76·2 cm

PEN. TEST HAMMER WEIGHT DROP

ELEVN / DEPTH (m)	DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/300mm	ELEVATION SCALE
108464	Ground surface					
0·00	Olive brown fine sandy SILT TILL, compact to dense		1	76mm O.D.	29	
108235						
2·29	Grey fine sandy SILT TILL, low plasticity, fissured, compact to dense (cold but not frozen)		2	"	18	
			3	"	25	
			4	"	31	
	] sample seems frozen		5	"	>100	
107580						
8·84	Grey clayey SILT to sandy SILT TILL, hard					
1070·01						
1463	Blue sandy to clayey SILT TILL, hard					
106788						
1676	Light grey clayey SILT TILL, low to medium plasticity, hard					
1061·78						
22·86	End of Borehole					

DYNAMIC PENETRATION RESISTANCE BLOWS/FT.

SHEAR STRENGTH C_u, LB./SQ.FT.

TEMPERATURE, °C
-0·5 0 +0·5 +1·0

WATER CONTENT, PERCENT
W_P W W_L
10 20 30 40

G_s = 2.74

Composite sample formed from samples 2, 3, and 5 for strength testing. $\phi' = 33°$

Sept. 27/79

ADDITIONAL LAB. TESTING

Cbl 6
G-52
S-30
M+C 12

G-9
S-24
M-40
C-17

G-15
S-32
M-32
C-21

S-31
M-30
C-18

PIEZOMETER OR STANDPIPE INSTALLATION

Ground surface
W.L. 27 Sept 79
Thermistor #1
#2
#3
19 mm dia. PVC pipe
#4
Peltonite seal
#5
Sandpack
Petur pneumatic piezometer

15 / 10 Percent axial strain at failure

VERTICAL SCALE
1 : 100

Golder Associates BH. 79-24

DRAWN RK
CHECKED DG

RECORD OF BOREHOLE 79-27

1946.49 N. 427.73 E.
LOCATION (See Figure 3)

BORING DATE Nov. 30, 1979

DATUM Project

BOREHOLE TYPE Cased Rotary with Air

BOREHOLE DIAMETER 149 m.m.

SAMPLER HAMMER WEIGHT 63.5 Kg. DROP 76.2 cm.

PEN. TEST HAMMER WEIGHT DROP

ELEVN. / DEPTH	DESCRIPTION	STRAT. PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: BLOWS/FT.
1066.71	Ground surface				
0.0	Brown peaty ORGANICS				
0.46	Brown SAND and GRAVEL trace silt occ. sandy layers occ. cobbles		1	W.S.	
1063.05					
3.66	Brown fine to coarse SAND some gravel		2	"	
1060.31			3	"	
6.40	Brown silty SAND & GRAVEL		4	"	
			5	"	
			6	"	
			7	"	
			8	"	
			9	"	
			10	"	
1048.42					
18.29	Brown SAND and GRAVEL turning brownish red at elevation 1042.93 m.		11	"	
			12	"	
			13	"	
			14	"	
			15	"	
			16	"	
1038.66					
28.05	Grey medium to coarse SAND		17	"	
1037.73					
28.98	Grey sandy SILT				
1036.83			18	"	
29.88	Brown sandy SILT				
1035.61					
31.10	Brown silty fine SAND occ. gravel occ. sand and gravel layers		19	"	
			20	"	
			21	"	
			22	"	
1028.60					
38.11	Brown SAND & GRAVEL		23	"	
38.72	Brown silty SAND occ. gravel		24	"	
1025.86					
40.85	Brown silty fine SAND turning grey at elevation 1024.91 m.		25	"	
			26	"	
1023.72					
42.99	Weathered schist BEDROCK in a till-like matrix		27	"	
			28	"	
1019.51					
47.20	Greenish grey schist BEDROCK		29	"	
1014.88			30	"	
51.83	End of Borehole				

ELEVATION SCALE

DYNAMIC PENETRATION RESISTANCE BLOWS/FT. ----------

SHEAR STRENGTH C_u, LB./SQ.FT.

COEFFICIENT OF PERMEABILITY k, CM./SEC.

WATER CONTENT, PERCENT W_P W W_L

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

Pumping Approx. 20 g.p.m. (1.3 l./sec.)

Pumping Approx 100 g.p.m. 6 l./sec.

Pumping Approx. 3 g.p.m (0.2 l./sec.)

Pumping Approx. 50+ g.p.m. (3+ l./sec.)

Pumping Approx. 5 g.p.m. or less (0.3 l./sec.)

no water return

cuttings backfill

19 m.m. diameter PVC pipe

water level Dec. 11, 1979 (Elev. 1052.21 meters)

Peltonite seal

caved-in material

Sand pack filter

Slotted 25 m.m P.C.V. pipe

caved-in material

15 / 10 / 5 Percent axial strain at failure

VERTICAL SCALE 1:125

Golder Associates B.H. 79-27

DRAWN R.W.

CHECKED DG

G.B. 102

PROJECT No. 792 2025

RECORD OF BOREHOLE 79-28

2056.32 N. 490.48 E.
LOCATION (See Figure 3)

BORING DATE Dec. 1, 1979

DATUM Project

BOREHOLE TYPE Cased Rotary with Air

BOREHOLE DIAMETER 149 m.m.

SAMPLER HAMMER WEIGHT 63.5 kg DROP 76.2 cm

PEN. TEST HAMMER WEIGHT

DROP

ELEVN. / DEPTH	DESCRIPTION	STRAT. PLOT	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/FT.
1065.17	Ground Surface				
0.0					
0.30	Brown SAND & GRAVEL with some organics				
1063.65	Brown sandy GRAVEL		1	WS	
1.52			2	"	
	Brown silty SAND & GRAVEL Layer of grey sandy silt at elevation 1056.0 meters		3	"	
			4	"	
			5	"	
			6	"	
			7	"	
			8	"	
			9	"	
			10	"	
			11	"	
			12	"	
1045.35			13	"	
19.82	Brown SAND & GRAVEL occ. silty sand layers turning brownish red at elevation 1036.8 meters 0.45 m. diameter boulder encountered at elevation 1036.77 meters		14	"	
			15	"	
			16	"	
			17	"	
			18		
			19	"	
			20	"	
			21	"	
1032.55					
32.62	Grey weathered schist BEDROCK in clayey till-like matrix		22	"	
1030.11					
35.06	Greenish grey schist BEDROCK		23	"	
			24	"	
1025.54			25	"	
39.63	End of Borehole				

ELEVATION SCALE

DYNAMIC PENETRATION RESISTANCE BLOWS/FT. ----------

SHEAR STRENGTH C_u, LB./SQ.FT.

COEFFICIENT OF PERMEABILITY k, CM./SEC.

WATER CONTENT, PERCENT W_P W W_L

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

cuttings backfill

19 m.m. diameter P.V.C. pipe

W.L. Dec. 10, 1979 in standpipe Elev. 1052.67 meters

W.L. Nov. 31, 1979 during drilling Elev. 1049.57 meters

Pumping Approx 100-150 g.p.m. (6 l./sec.)

Pumping Approx. 100 g.p.m (6 l./sec.

Pumping Approx. 5 g.p.m. (0.3 l./sec)

Sandpack Filter 2

Slotted 25mm PVC pipe

15 10 5 Percent axial strain at failure

VERTICAL SCALE 1:125

Golder Associates B.H. 79-28

DRAWN R.W.

CHECKED DG

G.B. 102

PROJECT No. 792-2025

RECORD OF BOREHOLE 79-29

2250.56 N. 559.17 E.
LOCATION (See Figure 3)

BORING DATE Dec. 2, 1979

DATUM Project

BOREHOLE TYPE Cased Rotary with Air

BOREHOLE DIAMETER 149 m.m.

SAMPLER HAMMER WEIGHT 63.5 Kg DROP 76.2 cm

PEN. TEST HAMMER WEIGHT

DROP

ELEVN / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: BLOWS/FT
1084.95	Gravel pad surface				
0.00	SAND & GRAVEL FILL		1	CS	
1081.90			2	WS	
3.05	Brown medium to coarse SAND trace gravel		3	"	
1081.30					
3.65	Brown SAND & GRAVEL		4	"	
			5	"	
7.32	Grey silty SAND and GRAVEL TILL		6	"	
			7	"	
1073.36			8	"	
11.59	Greenish grey schist BEDROCK		9	"	
			10	"	
			11	"	
1067.88			12	"	
17.07	End of Borehole				

ELEVATION SCALE

DYNAMIC PENETRATION RESISTANCE BLOWS/FT. ----------

SHEAR STRENGTH C_u, LB./SQ.FT

TEMPERATURE °C
-1.0 -0.5 0 0.5

WATER CONTENT, PERCENT
W_P W W_L

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

Dry during Drilling

March 9, 1980

Cuttings Backfill
Thermistor #1
#2
19 m.m. diameter P.V.C. pipe
#3
Peltonite seal
Sandpack Filter
#4
W.L. Dec 10, 1979 (Elev. 1073.35 m.)
Peltonite seal
Sandpack Filter
#5
Slotted 25 m.m PVC Pipe

Borehole dry during drilling Dec. 2, 1979

15 / 10 S Percent axial strain at failure

VERTICAL SCALE
1:125

Golder Associates B.H. 79-29

DRAWN R.W.
CHECKED DG

G.B. 102

PROJECT No. 792 2025

RECORD OF BOREHOLE 79-30

2138.07 N. 521.03 E
LOCATION (See Figure 3)

BORING DATE Dec. 2 & 3, 1979

DATUM Project

BOREHOLE TYPE Cased Rotary with Air

BOREHOLE DIAMETER 149 mm

SAMPLER HAMMER WEIGHT 63.5 Kg DROP 76.2 cm

PEN. TEST HAMMER WEIGHT DROP

ELEVN / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: BLOWS/FT.	ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS/FT. / SHEAR STRENGTH C_u, LB./SQ.FT.	COEFFICIENT OF PERMEABILITY k, CM./SEC. / WATER CONTENT, PERCENT W_P W W_L	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
1060.73	Cleared ground Surface									
0.00	Brown silty SAND some gravel		1	WS						19 m.m. diameter P.V.C. pipe
1056.16			2	"						
4.57	Brown SAND & GRAVEL occ. silty sand layers 0.3 m boulder encountered at elevation 1052.8		3	"						W.L. Dec. 10, 1979 in Standpipe Elev. 1054.80 meters
			4	"						
			5	"						W.L. Dec. 2, 1979 During drilling Elev. 1051.58 m.
			6	"						Peltonite seal
			7	"						
			8	"						Sandpack Filter
1047.01										
13.72	Grey SAND & GRAVEL TILL occ. silt layers		9	"						
			10	"			Pumping Approx. 150 g.p.m. (9 l./sec.)			
			11	"						Peltonite Seal
			12	"						
1041.22			13	"			Pumping Approx. 5 g.p.m. (0.3 l./sec)			Sandpack Filter
19.51	Greenish grey schist BEDROCK		14	"						
1036.34			15	"						Slotted 25 m m PVC pipe
24.39	End of Borehole									

15-○-5 (10) Percent axial strain at failure

VERTICAL SCALE 1:125

Golder Associates BH 79-30

DRAWN R.W.

CHECKED DG

G.B. 102

PROJECT No. 792-2025

RECORD OF BOREHOLE 79-31

1796.72 N. 334.19 E.
LOCATION (See Figure 3)

BORING DATE Dec. 3 & 4, 1979

DATUM Project

BOREHOLE TYPE Cased Rotary with Air

BOREHOLE DIAMETER 149 mm

SAMPLER HAMMER WEIGHT 63.5 Kg DROP 76.2 cm

PEN. TEST HAMMER WEIGHT DROP

ELEVN. / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	TYPE	BLOWS/FT.	ELEVATION SCALE
1053.62 / 0.00	Cleared Ground Surface					
1052.71 / 0.91	Brown silty SAND trace clay					
1050.88 / 2.74	Brown SAND & GRAVEL		1	WS		
1050.27 / 3.35	Black silty SAND & GRAVEL trace organics		2	"		
1046.91 / 6.71	Greyish black organic SILT		3	"		
1045.08 / 8.54	Grey clayey SILT trace organics occ. gravel		4	"		
1042.64 / 10.98	Brown SAND & GRAVEL		5	"		
1038.07 / 15.55	Brownish-red silty SAND occ. gravel		6	"		
1027.10 / 26.52	Brownish-red SAND and GRAVEL		7	"		
1015.51 / 38.11	Grey SAND & GRAVEL some silt occ. silt & sand layers		8	"		
1009.12 / 43.90	Weathered schist BEDROCK in a till like matrix		9	"		
1004.84 / 48.78	Greenish grey schist BEDROCK		10	"		
	End of Borehole		11	"		
			12	"		
			13	"		
			14	"		
			15	"		
			16	"		
			17	"		
			18	"		
			19	"		
			20	"		
			21	"		
			22	"		
			23	"		
			24	"		
			25	"		
			26	"		
			27	"		
			28	"		
			29	"		
			30	"		
			31	"		

DYNAMIC PENETRATION RESISTANCE BLOWS/FT. _____

SHEAR STRENGTH C_u, LB./SQ.FT.

COEFFICIENT OF PERMEABILITY k, CM./SEC.

WATER CONTENT, PERCENT
W_P W W_L
10 20 30 40

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

Non Plastic G- 5 S- 12 M- 69 C- 14
Organic Content 0.8%

Non Plastic G- 10 S- 24 M- 56 C- 10

Pumping Approx. 20-50 g.p.m (1-3 l./sec).

Pumping Approx. 50 g.p.m. (3 l./sec.)

Pumping Approx. 50+ g.p.m (3+ l./sec.)

- water sample taken @ 17.9 m.

Pumping Approx. 50 g.p.m. (30 l./sec)

- water sample taken at 24.4 m.

Pumping Approx. 100 g.p.m. (6 l./sec.)

- water sample taken at 30.4m

- water sample taken at 36.6 m

Pumping approx. 150 - 200 g.p.m. (9-13 l./sec.)

15-5 Percent axial strain at failure
10

During drilling Dec. 3, 1979 W.L.
W.L. In Standpipe Dec. 10, 1979 (elevation 1051.02 m)

cuttings backfill

Peltonite seal

Sand pack Filter

Slotted 25 mm PVC pipe

VERTICAL SCALE 1:125

Golder Associates B.H. 79-31

DRAWN RW.

CHECKED DG

G.B. 102

PROJECT No. 792-2025

RECORD OF BOREHOLE 79-32

1578.76 N 235.42 E
LOCATION (See Figure 3)

BORING DATE Dec. 4, 1979

DATUM Project

BOREHOLE TYPE Cased Rotary with Air

BOREHOLE DIAMETER 149 mm

SAMPLER HAMMER WEIGHT 63.5 Kg DROP 76.2 cm.

PEN. TEST HAMMER WEIGHT DROP

SOIL PROFILE			SAMPLES			ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS/FT. / SHEAR STRENGTH C_u, LB./SQ.FT.	COEFFICIENT OF PERMEABILITY k, CM./SEC. / WATER CONTENT, PERCENT W_P W W_L	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
ELEVN / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	BLOWS/FT.					
1085.37	Cleared Ground Surface									
0.00	Brown SAND & Gravel TILL		1	CS						
1082.63										
2.74	Greenish grey schist BEDROCK occ. silty layer		2	WS						Dry
			3	"						
1077.44			4	"						
7.93	End of borehole									

15-○-5 Percent axial strain at failure
10

VERTICAL SCALE
1:125

Golder Associates B.H. 79-32

DRAWN R.W.
CHECKED DG

G.B. 102

PROJECT No. 792 2025

RECORD OF BOREHOLE 79-33

1747.61 N. 301.65 E
LOCATION (See Figure 3)

BORING DATE Dec. 5, 1979

DATUM Project

BOREHOLE TYPE Cased Rotary with Air

BOREHOLE DIAMETER 149 m.m.

SAMPLER HAMMER WEIGHT 63.5 Kg. DROP 76.2 cm.

PEN. TEST HAMMER WEIGHT

DROP

ELEVN. / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/300 m.m.	ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS/FT. / SHEAR STRENGTH C_u, LB./SQ.FT.	COEFFICIENT OF PERMEABILITY k, CM./SEC. / WATER CONTENT, PERCENT (W_P W W_L)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
1054.07	Cleared Ground Surface									
0.00	Frozen sandy SILT trace clay									
0.80	Brown SAND and GRAVEL		1	WS					G. 12 S. 8 M. 42 C. 8	
1051.33			2	"						
2.74	Grey fine sandy SILT trace clay occ. gravel trace organics in upper portion		3	"				Organic Content 1.2 %		
			4	51mm D.O.	PH					
			5	WS				Organic Content 1.1 % Non Plastic	S. 18 M. 71 C. 11	
			6	"						
			7	"				Organic Content 1.0 % Non-Plastic	S. 13 M 77 C 10	
			8	51mm D.O.	14					
1041.27								Water sample taken		
12.80	Grey silty SAND and GRAVEL		9	WS.						
			10	"			Pumping Approx. 50-100 g.p.m. (3-6 l./sec)			
1036.39			11	"						
17.68	Brown SAND and GRAVEL		12	"			Pumping Approx. 50-100 g.p.m (3-6 l./sec)			
1033.95			13	"						
20.12	Greenish grey schist BEDROCK		14	"			Pumping Rate Increasing From 20 to 50 g.p.m. (1-3 l./sec.)			
			15	"						
1029.07			16	"				Water sample taken @ 25.0 m.		
25.00	End of Borehole									

NOTE: Standpipe piezometer, Petur piezometer and thermister set were destroyed during installation Dec. 5, 1979 Water level in borehole encountered at about elevation 1051.02 meters during drilling Dec 5, 1979

15 / 10 / 5 Percent axial strain at failure

VERTICAL SCALE 1:125

Golder Associates B.H. 79-33

DRAWN R.W.

CHECKED DG

G.B. 102

PROJECT No. 792 2025

RECORD OF BOREHOLE 79-34

2177.44 N. -2300 E
LOCATION (See Figure 3)

BORING DATE Dec. 6 1979

DATUM Project

BOREHOLE TYPE Cased Rotary with Air

BOREHOLE DIAMETER 149 mm

SAMPLER HAMMER WEIGHT DROP

PEN. TEST HAMMER WEIGHT DROP

ELEVN. / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	TYPE	BLOWS/FT.	ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS/FT. / SHEAR STRENGTH C_u, LB./SQ.FT.	COEFFICIENT OF PERMEABILITY k, CM./SEC. / WATER CONTENT, PERCENT W_P W W_L	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
1049.0	Ground Surface									
0.00	Brown silty SAND some clay, occ gravel									
1048.09										
0.91	Reddish brown SAND & GRAVEL		1	WS						
			2	"						
			3	"						
			4	"						
1041.37										
7.62	Brown silty SAND occ. gravel		5	"						
1040.7										
8.23	Grey silty fine to medium SAND trace organics		6	"			Pumping Approx. 50 g.p.m. (3 l./sec)			Casing Remains in Borehole
1038.3										
10.67	Brown silty fine to coarse SAND		7	"						
1036.5			8	"						
12.50	Grey clayey SILT some sand		9	"						
1031.9										
17.07	Brown silty SAND & GRAVEL		10	"			Pumping Approx. 20 g.p.m. (1 l./sec) 50 g.p.m. (3 l./sec)			
1028.5			11	"						
20.43	Brown SAND & GRAVEL						Water Sample Taken at 21.3m (75-100 g.p.m.) (5-6 l./sec)			
			12	"			Pumping Approx. 100-150 g.p.m. (6-9 l./sec.)			
			13	"						
1022.17										
26.83										

15 / 10 / 5 Percent axial strain at failure

VERTICAL SCALE 1:125

Golder Associates BH. 79-34

DRAWN RW.

CHECKED DG

Project No. 792-2025

RECORD OF BOREHOLE B.H. 80-35

1709.7 N 258.5 E
LOCATION (See Figure 3)

BORING DATE Feb. 20, 1980

BOREHOLE TYPE Rotary with air

BOREHOLE DIAMETER 100mm

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 IN.

DATUM Cleared ground surface

ELEV. / DEPTH (metres)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS / FOOT	ELEVATION SCALE	WATER CONTENT PERCENT W_P W W_L (10 20 30 40)	PIEZOMETER OR STANDPIPE INSTALLATION / ADDITIONAL LAB. TESTING
1056.08	Cleared ground surface							
0.0	Brown, organic SILT, some boulders							Bentonite seal
0.5	Light bwn. SILT, occ. organic layers, appears frozen							
1053.38	cobble							Sand and gravel backfill
2.7	Greenish brown, SAND and GRAVEL TILL, some silt, nonplastic, occasional sand pockets.		1			Cbl = 1 G = 57 S = 25 M = 14 C = 3	○	W.L. March 8/80 Cave in material
1049.08						G = 45 S = 26 M = 23 C = 6		Slotted 20mm PVC pipe
7.0 1047.98	Grey, fine, gravelly SILT TILL		2				○	
8.1	End of Borehole Note: Cuttings became wet at depth of about 1.5m. Hole had to be terminated due to water inflow which began at about 7.6 m depth.							Water level in open hole after drilling to 7.6m at depth ~ 1.5m.

VERTICAL SCALE 1:125

Golder Associates B.H. 80-35

DRAWN R.W.
CHECKED DG

Project No. 792-2025

RECORD OF BOREHOLE BH. 80-36

1776.7 N 250.9 E
LOCATION (See Figure 3)

BORING DATE Feb. 20, 1980

BOREHOLE TYPE Rotary with air to 3 m. Rotary with mud.

BOREHOLE DIAMETER 100mm.

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 IN.

DATUM Cleared ground surface

SOIL PROFILE									PIEZOMETER OR STANDPIPE INSTALLATION
ELEV. / DEPTH (metres)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS / FOOT	ELEVATION SCALE	WATER CONTENT PERCENT W_P W W_L (10 20 30 40)	ADDITIONAL LAB. TESTING	
1052.73	Cleared ground surface								
0.0 / 1052.13	Brown, fine SAND								
0.6 / 1050.03	Brown SAND and GRAVEL (alluvium) dry								
2.7 / 1040.53	Brown SAND and GRAVEL (alluvium) saturated. Pockets of brown silty fine SAND.						No testing		No installation
12.2.	End of Borehole								Standpipe in caved hole shows water level 1.3m b.g.s. March 8/80

VERTICAL SCALE 1:125

Golder Associates B.H. 80-36

DRAWN R.W.
CHECKED DG

Project No. 792-2025

RECORD OF BOREHOLE B.H. 80-37

1711.7 N 300.9 E
LOCATION (See Figure 3)

BORING DATE Feb. 20, 1980

BOREHOLE TYPE Rotary with air

BOREHOLE DIAMETER 100 mm.

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 IN.

DATUM Cleared ground surface

ELEV. / DEPTH (metres)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS / FOOT	ELEVATION SCALE	WATER CONTENT PERCENT W_P W W_L (10 20 30 40)	PIEZOMETER OR STANDPIPE INSTALLATION / ADDITIONAL LAB. TESTING
1053.41	Cleared ground surface							
0.0	Brown, organic SILT, frozen							Bentonite Seal
0.3	Brown SAND and GRAVEL TILL							
1050.71								
2.7	Grey, silty SAND TILL interlayered with grey to black organic silt and fine sand.		1	75mm D.D.	10	G=1. S=14 M=70 C=15	Organic content = 1.1%	Sand and Gravel Backfill
1045.31			2	50mm D.D.	19			Slotted 25mm. PVC pipe
8.1	End of Borehole Note: Encountered first water at depth 2.7 m.							Frozen at 1.2m March 8/80

VERTICAL SCALE 1:125

Golder Associates B.H. 80-37

DRAWN R.W.
CHECKED DG

Project No. 122-2025

RECORD OF BOREHOLE B.H. 80-38

2215.32 N. 41.17 E.
LOCATION (See Figure 3)

BORING DATE Feb. 21, 1980

BOREHOLE TYPE Rotary with air

BOREHOLE DIAMETER 120mm

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 IN.

DATUM Pond ice surface

SOIL PROFILE									PIEZOMETER OR STANDPIPE INSTALLATION
ELEV. / DEPTH (metres)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS / FOOT	ELEVATION SCALE	WATER CONTENT PERCENT W_P W W_L		ADDITIONAL LAB. TESTING
1048.85	Pond ice surface								
0.0 1047.95	ICE (seasonal)								
0.9 1046.15	Organic debris and organic silt (frozen)								No installations
2.7 1044.25	SAND and GRAVEL alluvium, unfrozen								
4.6	End of Borehole Note: Water encountered in substantial amounts (~100 i.g.p.m.) at a depth of 1.5 m and flowed under artesian pressure probably due to confinement by surface ice On March 21, 1980 borehole was still open through ice indicating continuing flow up the borehole.								

VERTICAL SCALE 1:125

Golder Associates B.H. 80-38

DRAWN R.W.
CHECKED DG

Project No. 792 2025

RECORD OF BOREHOLE B.H. 80-38A

2198.99 N. 41.94 E.
LOCATION (See Figure 3)

BORING DATE Feb. 21, 1980

BOREHOLE TYPE Rotary with air

BOREHOLE DIAMETER 120 mm

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 IN.

DATUM Pond Ice Surface

SOIL PROFILE ELEV. / DEPTH (metres)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS / FOOT	ELEVATION SCALE	WATER CONTENT PERCENT W_P W W_L	PIEZOMETER OR STANDPIPE INSTALLATION / ADDITIONAL LAB. TESTING
1048.9	Pond Ice Surface							
0.0	ICE, (seasonal)							
0.5 / 1047.7	Frozen organic silt & debris, sm. volcanic ash.							
1.2	Probably silt and fine sand, unfrozen							No installations
1044.2 / 4.1	Black SAND and GRAVEL alluvium, trace of organics		1	75mm D.O.	23		G=72 S=23 M+C=5 Organic content 0.41%	
1043.9 / 5.0	End of Borehole							

Note:
Water encountered in substantial amounts (~100 i.g.p.m.) at a depth of 1.5m and flowed under artesian pressure probably due to confinement by surface ice.

On March 21, 1980 ice surface was closed over borehole

VERTICAL SCALE 1 : 125

Golder Associates B.H. 80-38A

DRAWN R.W.
CHECKED DG

Project No. 792-2025

RECORD OF BOREHOLE B.H. 80-39

2153.38 N. 21.98 E.
LOCATION (See Figure 3)

BORING DATE Feb. 21, 1980

BOREHOLE TYPE Rotary with air

BOREHOLE DIAMETER 120 m.m.

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 IN.

DATUM Cleared ground surface

ELEV. / DEPTH (metres)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS / FOOT	ELEVATION SCALE	WATER CONTENT PERCENT W_P W W_L	PIEZOMETER OR STANDPIPE INSTALLATION / ADDITIONAL LAB. TESTING
1048.84	Cleared ground surface							
0.0	Frozen organic SILT, pieces of wood and ice.							20mm PVC pipe →
1046.34								Backfill with sand & gravel →
2.5	Gravel and fine to med. sand, frozen, dry							
	Frozen SAND and GRAVEL TILL							
1042.34	Becoming unfrozen							
6.5	End of Borehole							open hole water level at depth 1.5 m.
	Note: Water was encountered at depth about 6m.							Frozen at depth 1.2 m. March 9, 1980

VERTICAL SCALE 1:125

Golder Associates B.H. 80-39

DRAWN R.W.
CHECKED DG

Project No. 7922025

RECORD OF BOREHOLE BH. 80-40

2113.5N 7.5E
LOCATION (See Figure 3)

BORING DATE Feb. 21, 1980

BOREHOLE TYPE Rotary with air

BOREHOLE DIAMETER 120 mm to 2.4 m
100 mm to 3.2 m

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 IN.

DATUM Cleared ground surface

SOIL PROFILE									PIEZOMETER OR STANDPIPE INSTALLATION / ADDITIONAL LAB. TESTING
ELEV. / DEPTH (metres)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS / FOOT	ELEVATION SCALE	WATER CONTENT PERCENT W_P W W_L (10 20 30 40)		
1049.11	Cleared ground surface								
0.0	Frozen organic SILT, pieces of wood, ice crystals.						No testing		sand & gravel backfill → 20mm PVC pipe →
1046.71									
2.4 1045.91	Brown SAND & GRAVEL								
3.2	End of Borehole Note: water was encountered at depth of about 3m.								Frozen at 1.5m March 9/80

VERTICAL SCALE 1:125

Golder Associates BH. 80-40

DRAWN R.W.
CHECKED DG

Project No. 772 2025

RECORD OF BOREHOLE B.H. 80-41

2069.5 N -8.6 E
LOCATION (See Figure 3)

BORING DATE Feb. 21, 198

BOREHOLE TYPE Rotary with air

BOREHOLE DIAMETER 120 mm

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 IN.

DATUM Cleared ground surface

SOIL PROFILE ELEV. DEPTH (metres)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS / FOOT	ELEVATION SCALE	WATER CONTENT PERCENT W_P W W_L (10 20 30 40)	PIEZOMETER OR STANDPIPE INSTALLATION ADDITIONAL LAB. TESTING
1049.15	Cleared ground surface							2mm PVC pipe
0.0	Frozen SAND and GRAVEL						No testing	sand & gravel backfill
1047.75			1A	75 mm D.O.	34			
1.4	Organic silt and debris becoming silt & fine sand		1B	75 mm D.O.	34			
1046.85								
2.3	End of Borehole							Frozen at 1.32 m March 9/80

Note: water encountered at about 2 m. depth.

VERTICAL SCALE 1:125

Golder Associates B.H. 80-41

DRAWN R.W.
CHECKED DG

Project No. 792-2025

RECORD OF BOREHOLE B.H. 80-46

1807.6 N 303.9 E
LOCATION (See Figure 3)

BORING DATE Feb. 22, 1980

BOREHOLE TYPE Rotary with mud

BOREHOLE DIAMETER 100mm

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 IN.

DATUM Ground Surface

ELEV. / DEPTH (metres)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS / FOOT	ELEVATION SCALE	WATER CONTENT PERCENT W_P W W_L (10 20 30 40)	PIEZOMETER OR STANDPIPE INSTALLATION / ADDITIONAL LAB. TESTING
1054.11	Ground Surface							
0.0 / 1053.21	Peat & organic debris							
0.9 / 1052.61	Brown silty SAND							
1.5 / 1049.81	Brown SAND & GRAVEL		1	75mm D.O.	88			W.L. March 10/80 ▽
4.3 / 1039.91	Brown silty fine SAND interlayered with fine to medium sand		2	75mm D.O.	67			Sand & gravel backfill
			3	75mm D.O.	21			Peltonite seal
			4	75mm D.O.	58	cbl=10 G=36 S=47 M+C=3		Slotted 20mm PVC pipe
			5	75mm D.O.	49			Cave-in material
14.2	End of Borehole Note: Log based in part on drillers observations							

VERTICAL SCALE 1:125

Golder Associates B.H. 80-46

DRAWN R.W.
CHECKED DG

Project No. 792-2025

RECORD OF BOREHOLE 80-47

Canal Sta. 2+370
1600.2 N 164.8 E
LOCATION (See Figure 3)

BORING DATE Feb. 23, 1980

BOREHOLE TYPE Rotary with air

BOREHOLE DIAMETER 100 mm

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 IN.

DATUM Cleared ground surface

ELEV. / DEPTH (m.)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS / FOOT	ELEVATION SCALE	WATER CONTENT PERCENT W_P W W_L (10 20 30 40)	PIEZOMETER OR STANDPIPE INSTALLATION / ADDITIONAL LAB. TESTING
1090.66	Cleared ground surface							
0.0 / 1090.06	Brown, organic SILT, frozen							
0.6	— Frozen —] organic layer							
	Greenish-brown to brown SAND and GRAVEL TILL (frozen)						No testing	Backfilled with cuttings
	] Brown fine SAND becoming dry							
1084.56								
6.1 / 1083.36	Brown to very light grey disturbed BEDROCK, dry							Bentonite seal
7.3	Schisty BEDROCK with brown silt infilling in joints or layers.							Sand & gravel pack; Slotted 20mm PVC pipe
1079.96								
10.7	End of Borehole							Dry at 10.0 m March 8/80

VERTICAL SCALE 1:100

Golder Associates B.H. 80-47

DRAWN RW
CHECKED DG

Project No. 792 2025

RECORD OF BOREHOLE B.H. 80-48

Canal Sta. 2+300
156B.5N 227.7 E
LOCATION (See Figure 3)

BORING DATE Feb. 23, 1980

BOREHOLE TYPE Rotary with air

BOREHOLE DIAMETER 100 mm

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 IN.

DATUM Cleared ground surface

ELEV. / DEPTH (metres)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS / FOOT	ELEVATION SCALE	WATER CONTENT PERCENT W_P W W_L (10 20 30 40)	PIEZOMETER OR STANDPIPE INSTALLATION / ADDITIONAL LAB. TESTING
1089.89	Cleared ground surface							
0.00 1089.29	Frozen organic SILT							
0.6	Frozen, greenish-brown SAND & GRAVEL TILL, some ice in cuttings							Cuttings backfill
1083.79								Bentonite seal
6.1 1082.59	Disturbed bedrock							sand & gravel pack
7.3 1080.79	Grey, schisty, BEDROCK		1	W.S.				Slotted 20mm PVC pipe
9.1	End of Borehole							Dry at 8.2 m March 8/80

VERTICAL SCALE 1:100

Golder Associates B.H. 80-48

DRAWN R.W.
CHECKED DG

Project No. 792 2025

RECORD OF BOREHOLE B.H. 80-49

Canal Stn. 2+220
1528.8 N 299.2 E
LOCATION (See Figure 3)

BORING DATE Feb. 23, 1980

BOREHOLE TYPE Rotary with air

BOREHOLE DIAMETER 100 mm

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 IN.

DATUM Cleared ground surface

ELEV. / DEPTH (metres)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS / FOOT	ELEVATION SCALE	WATER CONTENT PERCENT Wp W WL (10 20 30 40)	PIEZOMETER OR STANDPIPE INSTALLATION / ADDITIONAL LAB. TESTING
1088.83	Cleared ground surface							
0.0 / 1088.23	Frozen organic SILT							
0.6	←boulder↗ [boulder Greenish-brown to brown SAND and GRAVEL TILL frozen to about 4m. depth Very dry (possibly disturbed bedrock)						No testing	Cuttings backfill Bentonite seal Sand & gravel backfill Slotted 20mm PVC pipe
1079.73								
9.1 / 1078.13	Grey schisty BEDROCK							
10.7	End of Borehole							Dry at 10.0 m March 8/80

VERTICAL SCALE 1:100

Golder Associates B.H. 80-49

DRAWN RW. CHECKED DG

Project No. 792 2025

RECORD OF BOREHOLE B.H. 80-50

Canal Sta. 2+100
1475.8 N 404.4 E
LOCATION (See Figure 3)

BORING DATE February 23, 1980

BOREHOLE TYPE Rotary with air

BOREHOLE DIAMETER 100 mm.

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 IN.

DATUM Cleared ground surface

ELEV. / DEPTH (metres)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS / FOOT	ELEVATION SCALE	WATER CONTENT PERCENT W_P W W_L (10 20 30 40)	PIEZOMETER OR STANDPIPE INSTALLATION / ADDITIONAL LAB. TESTING
1085.66	Cleared ground surface							
0.0 1084.46	Frozen brown organic SILT							
1.2 1083.26	Greenish brown SAND and GRAVEL TILL (unfrozen)							Cuttings backfill
2.4 1080.76	Brown SAND and GRAVEL, unfrozen						No testing	Bentonite seal Sand & gravel backfill Slotted 20mm PVC pipe
4.9 1077.76	Grey silt and sand cuttings, probably disturbed BEDROCK, very dry, probably rippable							
7.9 1076.56	Schisty BEDROCK							
9.1	End of Borehole							Dry at 8.0 m March 8/80.

VERTICAL SCALE 1:100

Golder Associates B.H. 80-50

DRAWN R.W.
CHECKED DG

Project No. 792-2025

RECORD OF BOREHOLE B.H. 80-51

Canal Sta. 2+000
1438.0 N 482.5 E
LOCATION (See Figure 3)

BORING DATE Feb. 23, 1980

BOREHOLE TYPE Rotary with air

BOREHOLE DIAMETER 100 mm

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 IN.

DATUM Cleared ground surface

ELEV. DEPTH (metres)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS / FOOT	ELEVATION SCALE	Temperature °C (-2, -1, 0, +1) / WATER CONTENT PERCENT W_P W W_L (10, 20, 30, 40)	PIEZOMETER OR STANDPIPE INSTALLATION / ADDITIONAL LAB. TESTING
1089.54	Cleared ground surface							
0.0 1088.34	Frozen, brown, organic SILT							
1.2	Greenish-grey, frozen SAND and GRAVEL TILL, bits of ice in cuttings						March 8/80; March 26/80; April 30/80	Cuttings backfill; Thermistor #1; Bentonite seal; #2
	Till as above but with several boulders					No testing		#3; Sand & gravel backfill (probably fill is discontinuous); #4
1078.84	possibly fractured or disturbed BEDROCK							
10.7	Grey schisty BEDROCK							Slotted 20mm PVC pipe; #5
1075.84								
13.7	End of Borehole							Dry at 13.4 m March 8/80

VERTICAL SCALE 1:100

Golder Associates BH 80-51

DRAWN RW
CHECKED DG

Project No. 792 2025

RECORD OF BOREHOLE B.H. 80-52

Canal Sta. 1+890
1369.5 N 618.5 E
LOCATION (See Figure 3)

BORING DATE Feb. 23, 1980

BOREHOLE TYPE Rotary with air

BOREHOLE DIAMETER 100mm

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 IN.

DATUM Cleared Ground Surface

ELEV. / DEPTH (metres)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS / FOOT	ELEVATION SCALE	WATER CONTENT PERCENT W_P W W_L (10 20 30 40)	PIEZOMETER OR STANDPIPE INSTALLATION / ADDITIONAL LAB. TESTING
1089.41	Cleared ground surface							
1088.81	Frozen, bwn., organic SILT							
0.6	Frozen, greenish-grey, SAND and GRAVEL TILL					No testing		Sand & gravel pack Bentonite seal Sand & gravel pack Slotted 20 mm PVC pipe
1083.31								
6.1	SAND and GRAVEL TILL with sandy silt pockets (unfrozen)							
1080.31								
9.1	End of Borehole Note: Log based on driller's observations							Dry at 7.8m March 8/80

VERTICAL SCALE 1:100

Golder Associates B.H. 80-52

DRAWN R.W.
CHECKED DG

RECORD OF TEST PIT SHEETS

Other Than Along Diversion Canal

Test Pit No. 1 to 20 incl.
23 to 25 incl.

Project No. 792-2025

RECORD OF TEST PIT T.P. 79-1

2415.7N 725.6E LOCATION (See Figure) DATE 27 July 1979 DATUM Ground Surface

METHOD OF EXCAVATION D8 Cat & ripper PROJECT DOWN VALLEY

SOIL PROFILE: ELEV. / DEPTH (m)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	ELEVATION SCALE	WATER CONTENT, PERCENT (W_P, W, W_L) 10 20 30 40 50	ADDITIONAL LAB. TESTING	GROUNDWATER CONDITIONS
1132.66	Ground surface							
0.00 / 1132.28	Organic SILT & debris, .1 m volcanic ash							
0.38 / 1131.90	Brown grey fine sandy SILT TILL, occ. brown fine to med. sand lense							
0.76	Grey to brown grey sandy SILT TILL, some zones of high schist content		1	C.S.		○	Cbl-1 G-48 S-31 M-15 C-5	non-plastic
1128.70			2	C.S.		○	Cbl-5 G-24 S-34 M-28 C-9	
3.96	Bottom of Test Pit							

Sample 2 combined with sample 2 from TP-79-2 for strength testing and compaction.

VERTICAL SCALE 1:50

Golder Associates T.P. 79-1

DRAWN RK CHECKED DG

Project No. 792-2025

RECORD OF TEST PIT T.P. 79-2

2389.2N 689.6E LOCATION (See Figure) DATE 28 July 1979 DATUM Ground surface

METHOD OF EXCAVATION DB Cat & ripper PROJECT DOWN VALLEY

ELEV N. / DEPTH (m)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	ELEVATION SCALE	WATER CONTENT, PERCENT W_P W W_L (10 20 30 40 50)	ADDITIONAL LAB. TESTING	GROUNDWATER CONDITIONS
1121.68 / 0.00	Ground surface							
0.00	Organic debris & moss							
0.25	White volcanic ash							
0.43	Brown grey fine sandy SILT TILL, w < Wp, very stiff to hard		1	C.S.		O	Cbl-6 G-38 S-31 M-24 C-1	non-plastic
1117.87 / 3.81	Bottom of Test Pit		2	C.S.		O	Cbl-0 G-30 S-31 M-29 C-10	

Sample 2 combined with sample 2 from TP-79-1 for strength testing and compaction.

VERTICAL SCALE 1:50

Golder Associates T.P. 79-2

DRAWN RK CHECKED DG

Project No. 792-2025

RECORD OF TEST PIT T.P. 79-3

3143.0N -824.7E
LOCATION (See Figure)

DATE 5 Aug 1979

DATUM Ground surface

METHOD OF EXCAVATION D8 Cat & ripper PROJECT

DOWN VALLEY

ELEV N. / DEPTH (m)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	ELEVATION SCALE	WATER CONTENT, PERCENT W_P W W_L (10 20 30 40 50)	ADDITIONAL LAB. TESTING	GROUNDWATER CONDITIONS
1068.93 / 0.00	Ground surface							
1068.63 / 0.30	organic debris & volcanic ash							
1068.32 / 0.61	Brown fine sandy SILT, occ. pebble, moist							
	Brown grey coarse sandy GRAVEL, moist below 3m, occ. grey silt inclusions, some frozen		1	C.S.		O	Cbl-10 G-63 S-25 M+C-2	
			2	C.S.		O	Cbl-5 G-64 S-28 M+C-3	
1063.60 / 5.33	Bottom of Test Pit							

VERTICAL SCALE 1:50

Golder Associates T.P. 79-3

DRAWN RK
CHECKED DG

Project No. 792-2025

RECORD OF TEST PIT T.P. 79-4

2501.6N 1021.6E
LOCATION (See Figure)

DATE 5 Aug 1979

DATUM Ground surface

METHOD OF EXCAVATION D8 Cat & ripper

PROJECT DOWN VALLEY

ELEV N. / DEPTH (m)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	ELEVATION SCALE	WATER CONTENT, PERCENT W_P W W_L (10 20 30 40 50)	ADDITIONAL LAB. TESTING	GROUNDWATER CONDITIONS
1159.78 / 0.00	Ground surface							
1159.37 / 0.41	Organic debris & volcanic ash							
	Grey brown clayey SILT TILL, non-plastic, very dense, $w < w_p$		1	C.S.			G-46 S-28 M-21 C-5	
			2	C.S.				
1154.60 / 5.18	Bottom of Test Pit							

VERTICAL SCALE 1:50

Golder Associates T.P. 79-4

DRAWN RK

CHECKED JG

Project No. 792-2025

RECORD OF TEST PIT T.P. 79-5

2503.4 N -589.3 E LOCATION (See Figure) DATE 6 Aug 1979 DATUM Ground surface

METHOD OF EXCAVATION D8 Cat & ripper PROJECT DOWN VALLEY

ELEV N. / DEPTH (m)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	ELEVATION SCALE	WATER CONTENT, PERCENT W_P W W_L (10 20 30 40)	ADDITIONAL LAB. TESTING	GROUNDWATER CONDITIONS
1055.83	Ground surface							
0.00	Organic SILT & debris							
0.25	White volcanic ash							
0.46	Brown to brown grey dense coarse sandy GRAVEL		1	C.S.		O		
1053.33								
2.50	Frozen, oxidized med. to coarse SAND, some gravel (to .1m. dia.), dense, thaws quickly		2	C.S.		O	G-49 S-49 M+C-2	
1051.83								
4.0	Olive brown fine to coarse SAND, little med. gravel		3	C.S.		O	G-48 S-50 M+C-2	
1049.83								
6.00	Bottom of Test Pit							

VERTICAL SCALE 1:50

Golder Associates T.P. 79-5

DRAWN RK CHECKED DG

G.B. ...

Project No. 792-2025

RECORD OF TEST PIT T.P. 79-6

2300.2N 467.9E
LOCATION (See Figure)

DATE 7 Aug 1979

DATUM Ground surface

METHOD OF EXCAVATION D8 Cat & ripper

PROJECT DOWN VALLEY

ELEV N. / DEPTH (m)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	ELEVATION SCALE	WATER CONTENT, PERCENT W_P W W_L (10 20 30 40)	ADDITIONAL LAB. TESTING	GROUNDWATER CONDITIONS
1055.69	Ground surface							
0.00 / 1055.39	Organic SILT & debris, moss							
0.30	White volcanic ash							
0.51 / 1052.49	Brown coarse sandy GRAVEL, (max. size 0.5m)							
3.20 / 1050.09	Brown grey med. to coarse SAND, some frozen, some gravel to .3m dia., becoming sandier with depth		1	C.S.		O	Cbl-10 G-54 S-35 M+C = 1%	
			2	C.S.		O		
5.60	Bottom of Test Pit							

VERTICAL SCALE 1:50

Golder Associates T.P. 79-6

DRAWN RK
CHECKED DG

Project No. 792-2025

RECORD OF TEST PIT T.P. 79-7

2041.7N 380.7E
LOCATION (See Figure)

DATE 8 Aug 1978

DATUM Ground surface

METHOD OF EXCAVATION D8 Cat & ripper

PROJECT DOWN VALLEY

SOIL PROFILE ELEV N. / DEPTH (m)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	ELEVATION SCALE	WATER CONTENT, PERCENT W_P W W_L	ADDITIONAL LAB. TESTING	GROUNDWATER CONDITIONS
1057.58	Ground surface							
0.00 / 1057.28	Moss & organic debris							
0.30 / 1056.36	Black frozen organic SILT, $w >> W_P$, material flows when thawed							
1.22 / 1054.23	Brown gravelly SAND becoming finer with depth		1	C.S.			G-78 SM, C-22	
3.35	Bottom of Test Pit Depth varies along length of pit							

VERTICAL SCALE 1:50

Golder Associates T.P. 79-7

DRAWN RK
CHECKED DG

G.B. 101

Project No. 792-2025

RECORD OF TEST PIT T.P. 79-B

2346.0N 1017.9E
LOCATION (See Figure)

DATE 9 Aug 1979

DATUM Ground surface

METHOD OF EXCAVATION D8 Cat & ripper PROJECT DOWN VALLEY

SOIL PROFILE									
ELEV N. / DEPTH (m)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	ELEVATION SCALE	WATER CONTENT, PERCENT W_P W W_L	ADDITIONAL LAB. TESTING	GROUNDWATER CONDITIONS
1143.73	Ground surface							
0.00	Broken SCHIST in brown sandy silt matrix (TILL)							
1141.83								
1.90	Brown grey fine sandy SILT TILL, non plastic							
1140.13								
3.60	Broken grey SCHIST BEDROCK							
1139.83								
3.90	Bottom of Test Pit							
NOTE	Area has been previously stripped							

VERTICAL SCALE 1:50

Golder Associates T.P. 79-8

DRAWN RK
CHECKED DG

Project No. 792-2025

RECORD OF TEST PIT T.P. 79-9

2277.7N 1307.7E
LOCATION (See Figure)

DATE 9 Aug 1979

DATUM Ground surface

METHOD OF EXCAVATION D8 Cat & ripper

PROJECT DOWN VALLEY

ELEV N. / DEPTH (m)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	ELEVATION SCALE	WATER CONTENT, PERCENT W_P W W_L	ADDITIONAL LAB. TESTING	GROUNDWATER CONDITIONS
1152.68	Ground surface							
0.00 / 1152.40	Organic debris & moss							
0.28	White volcanic ash							
0.41	Brown sandy SILT TILL, some frozen							
1150.75								
1.93 / 1150.32	Disturbed SCHIST BEDROCK							
2.36	SCHIST BEDROCK dipping to southwest at approx. 30°							
1148.41								
4.27	Bottom of Test Pit							

VERTICAL SCALE 1:50

Golder Associates T.P. 79-9

DRAWN RK
CHECKED DG

Project No. 792-2025

RECORD OF TEST PIT T.P. 79-10

2502.6N 51.6E
LOCATION (See Figure)

DATE 24 Aug 1979

DATUM Ground surface

METHOD OF EXCAVATION D8 Cat & ripper

PROJECT Down valley

SOIL PROFILE ELEV N. / DEPTH (m)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	ELEVATION SCALE	WATER CONTENT, PERCENT W_P W W_L (10 20 30 40)	ADDITIONAL LAB. TESTING	GROUNDWATER CONDITIONS
1075.16	Ground surface							
0.00 / 1074.86	Organic debris & volcanic ash							
0.30	Frozen silty fine SAND TILL with alluvium pockets		1	C.S.		○	G-25 S-55 M-19 C-1	
1072.97								
2.19 / 1072.33	Frozen fine to coarse SAND, some gravel, trace silt							
2.83	Bottom of Test Pit							

VERTICAL SCALE 1:50

Golder Associates T.P. 79-10

DRAWN RK
CHECKED DG

Project No. 792-2025

RECORD OF TEST PIT T.P. 79-11

2131.4N -441.5E LOCATION (See Figure) DATE 25 Aug 1979 DATUM Ground surface

METHOD OF EXCAVATION D8 Cat & ripper PROJECT DOWN VALLEY

ELEV N. / DEPTH (m)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	ELEVATION SCALE	WATER CONTENT, PERCENT W_P W W_L	ADDITIONAL LAB. TESTING	GROUNDWATER CONDITIONS
1060.82	Ground surface							
0.00	Frozen organic SILTS & debris with moss at top of stratum, some fine & medium to coarse sand pockets							
1059.30								
1.52	Frozen gravelly SAND (no cobbles or coarse gravel)							
1057.16								
3.66	Bottom of Test Pit							

VERTICAL SCALE 1:50

Golder Associates T.P. 79-11

DRAWN RK CHECKED DG

Project No. 792-2025

RECORD OF TEST PIT T.P. 79-12

2060.2 N - 287.2 E
LOCATION (See Figure 3)

DATE 25 Aug 1979

DATUM Ground surface

METHOD OF EXCAVATION D8 Cat & ripper

PROJECT Down Valley

SOIL PROFILE			SAMPLE NUMBER	SAMPLE TYPE	ELEVATION SCALE	WATER CONTENT, PERCENT W_P W W_L 10 20 30 40	ADDITIONAL LAB. TESTING	GROUNDWATER CONDITIONS
ELEVN. / DEPTH (m)	DESCRIPTION	STRATIGRAPHY PLOT						
1057.36	Ground surface							
0.00 / 1057.06	Surficial organics							
0.30	Sandy fine GRAVEL with occ. silty pockets & organic inclusions, material is highly variable							
1055.99								
1.37	Bottom of Test Pit							Pit abandoned due to seepage

VERTICAL SCALE 1:50

Golder Associates T.P. 79-12

DRAWN RK

CHECKED DG

Project No. 792-2025

RECORD OF TEST PIT T.P 79-13

1941.8N 847.5E
LOCATION (See Figure 3)
DATE Dec. 1, 1979
DATUM Ground Surface
METHOD OF EXCAVATION D-8 Cat & Ripper
PROJECT Down Valley Tailings

SOIL PROFILE: ELEV N. / DEPTH (m)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	ELEVATION SCALE	WATER CONTENT, PERCENT W_P W W_L	ADDITIONAL LAB. TESTING	GROUNDWATER CONDITIONS
1069.4	Ground Surface							
0.0	Organic debris							
1068.1	Frozen black organic SILT					No testing		
1.3	Brown SAND and GRAVEL mostly frozen		1	C.S.				seepage forcing termination of pit.
1066.4								
3.0	End of Test Pit							

VERTICAL SCALE 1:50

Golder Associates T.P 79-13

DRAWN RW.
CHECKED DG

Project No. 722-2025

RECORD OF TEST PIT T.P. 79-14

1726.0N 1008.6E LOCATION (See Figure 3) DATE Dec. 1, 1979 DATUM Ground Surface

METHOD OF EXCAVATION D-8 Cat & Ripper PROJECT Down Valley Tailings

SOIL PROFILE ELEV. / DEPTH (m.)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	ELEVATION SCALE	WATER CONTENT, PERCENT W_P W W_L (10 20 30 40)	ADDITIONAL LAB. TESTING	GROUNDWATER CONDITIONS
1069.55	Ground Surface							
0.0 1069.25	Organics & volcanic ash							
0.3 1068.95	SAND and GRAVEL							
0.6 1068.45	Coarse SAND (lense)							
1.1 1064.75	Gravelly SAND some frozen N.B. permafrost dips from surface at NW and tapers off in bottom of pit		1	CS	cbl=20 G=50 S=27 M+C=3	O		Dry
4.8 1063.05	Coarse SAND some gravel		2	C.S.	cbl=2 G=57 S=39 M+C=2	O		
6.5	End of Test Pit Surveyed bottom Elevation 1064.57							

VERTICAL SCALE 1:50

Golder Associates T.P. 79-14

DRAWN R.W. CHECKED DG

G.B 101

Project No. 792-2025

RECORD OF TEST PIT T.P. 79-15

1959.0N 614.8E
LOCATION (See Figure 3)
DATE Dec. 1, 1979
DATUM Ground Surface
METHOD OF EXCAVATION D-8 Cat & Ripper
PROJECT Down Valley Tailings

ELEV N. / DEPTH (m.)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	ELEVATION SCALE	WATER CONTENT, PERCENT (W_P — W — W_L) 10 20 30 40	ADDITIONAL LAB. TESTING	GROUNDWATER CONDITIONS
1066.22	Ground Surface							
0.0 1065.72	Organic SILT & volcanic ash							
0.5 1064.12	Red brown GRAVEL some coarse sand little cobble		1	C.S.	cbl = 27 G = 41 S = 28 M+C = 2	O		
2.1 1063.42	Red brown coarse SAND and GRAVEL							
2.8 1061.92	Frozen red brown coarse SAND, some layers of gravel & sand dipping steeply to north							Dry
4.3 1060.42	Red brown coarse SAND, little fine to coarse gravel		2	C.S.	G = 70 S = 27 M+C = 1	O		
5.8	End of test pit							

VERTICAL SCALE 1:50

Golder Associates T.P. 79-15

DRAWN R.W.
CHECKED DG

Project No. 122-2025

RECORD OF TEST PIT T.P. 79-16

2096.8 N 414.9 E
LOCATION (See Figure 3)

DATE Dec. 8, 1979

DATUM Ground Surface

METHOD OF EXCAVATION D-8 Cat & Ripper

PROJECT Down Valley Tailings

ELEV. / DEPTH (m.)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	ELEVATION SCALE	WATER CONTENT, PERCENT (W_P — W — W_L) 10 20 30 40	ADDITIONAL LAB. TESTING	GROUNDWATER CONDITIONS
1060.61	Ground Surface							
0.0 / 1059.41	Organic debris Olive laminated fine sandy SILT scattered pebbles							
1.2 / 1058.01	Brown fine to medium GRAVEL little sand and cobbles, layered					No testing		
2.6 / 1057.61	Grey brown coarse SAND & GRAVEL, some silt							
3.0 / 1056.91	Layered grey fine SAND & brown med. to coarse SAND							
3.7	GRAVEL & coarse SAND, few cobbles and boulders							Dry
1053.91	Large lense of silty fine sand					NOTE: Test Pit is cut in wall of old borrow pit.		
6.7 / 1052.41	GRAVEL, some cobble, little coarse sand near vertical imbrication							
8.2	End of Test Pit							

VERTICAL SCALE 1:50

Golder Associates T.P. 79-16

DRAWN R.W.
CHECKED D.G.

Project No. 792-2025

RECORD OF TEST PIT T.P. 79-17

2041.7N 380.7E
LOCATION (See Figure 3)

DATE Dec. 9, 1979

DATUM Ground Surface

METHOD OF EXCAVATION D-8 Cat & Ripper

PROJECT Down Valley Tailings

SOIL PROFILE									
ELEV N. / DEPTH (m.)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	ELEVATION SCALE	WATER CONTENT, PERCENT W_P W W_L (10 20 30 40)	ADDITIONAL LAB. TESTING	GROUNDWATER CONDITIONS
1064.34	Ground Surface							
0.0 / 1063.74	Organic debris, Volcanic ash & silty SAND							
0.6 / 1062.54	Layered fine SAND, sandy SILT, fine GRAVEL and med. to coarse SAND							
1.8 / 1061.14	Fine to coarse GRAVEL, few cobbles, little coarse sand							Dry
3.2 / 1057.94	Layered fine to coarse SAND, some gravelly layers		1	CS	cbl=6 G=38 S=55 M+C=0.5	O		
6.4	End of test pit							

VERTICAL SCALE 1:50

Golder Associates T.P. 79-17

DRAWN R.W.
CHECKED DG

Project No. 792-2025

RECORD OF TEST PIT TP 79-18

2139.2 N 539.4 E
LOCATION (See Figure 3)

DATE Dec. 9, 1979

DATUM Ground Surface

METHOD OF EXCAVATION D-8 Cat & Ripper

PROJECT Down Valley Tailings

ELEV. / DEPTH (m)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	ELEVATION SCALE	WATER CONTENT, PERCENT (W_P — W — W_L) 10 20 30 40	ADDITIONAL LAB. TESTING	GROUNDWATER CONDITIONS
1062.26	Ground Surface							
0.0 / 1061.66	Frozen organic debris, organic silt and volcanic ash							
0.6	Frozen light brown							Dry
	frozen olive layered silty fine SAND, occasional fine to medium sand layer		1	CS	Cbl=20 G=59 S=15 M+C=6	O		
1058.16	SAND and GRAVEL TILL							
4.1	End of Test Pit							

VERTICAL SCALE 1:50

Golder Associates TP 79-18

DRAWN R.W.
CHECKED DG

Project No. 792-2025

RECORD OF TEST PIT T.P. 79-19

2124.4 N 781.8 E
LOCATION (See Figure 3)

DATE Dec. 10, 1979

DATUM Ground surface

METHOD OF EXCAVATION D-8 Cat & Ripper

PROJECT Down Valley Tailings

SOIL PROFILE										
ELEV N. / DEPTH (m.)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	ELEVATION SCALE	WATER CONTENT, PERCENT W_P W W_L (10, 20, 30, 40)	ADDITIONAL LAB. TESTING	GROUNDWATER CONDITIONS
1081.89	Ground surface							
0.0 / 1081.55	Organic silt & debris							
0.3	Gravelly, silty fine SAND till		1	CS	cbl. 2 G=37 S=33 M=20 C=8	○		Dry
1076.69	Sandy pockets		2	CS	cbl.<1 G=30 S=39	○		
5.2	End of Test Pit				M=22 C=8			

VERTICAL SCALE 1:50

Golder Associates T.P. 79-19

DRAWN R.W.

CHECKED DG

U.B. 101

Project No. 792-2025

Dam 2127.6N 27.7E

RECORD OF TEST PIT T.P. 79-20

LOCATION (See Figure 3) DATE Dec. 11, 1979 DATUM Ground Surface

METHOD OF EXCAVATION D-8 Cat & Ripper PROJECT Down Valley Tailings

ELEV N. / DEPTH (m.)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	ELEVATION SCALE	WATER CONTENT, PERCENT (W_P — W — W_L) 30 40 50 60	ADDITIONAL LAB. TESTING	GROUNDWATER CONDITIONS
1049.35	Ground Surface							
0.0 / 1049.05	Organic silt & debris							
0.3 / 1048.75	Brown fine to medium SAND							
0.6	Frozen, layered brown grey silty fine SAND and fine sandy SILT some organic inclusions (wood) and occasional ice crystals. Becoming more difficult to rip and to obtain traction with depth.		1	CS		thaw consolidation ○	1A γ_w = 1485 kg/m³ 1B γ_w = 1181 kg/m³	Frozen Dry
1045.05			2	CS	dol=27 G=50	○	γ_w = 1345 kg/m³ D=7.5%	
4.3 / 1044.75	Frozen SAND & GRAVEL TILL		3	CS	S=17 M=5 C=1	○	γ_w = 2241 kg/m³	
4.6	End of Test Pit Surveyed bottom elevation 1046.16							

VERTICAL SCALE 1:50

Golder Associates T.P. 79-20

DRAWN R.W.

CHECKED DG

G.B. 101

Project No. 782-2025

RECORD OF TEST PIT T.P. 80-23

2184.05 N. 851.46 E
LOCATION (See Figure 3)

DATE Feb. 24, 1980

DATUM Ground Surface

METHOD OF EXCAVATION D-8 Cat & Ripper

PROJECT Down Valley Tailings

SOIL PROFILE									
ELEVN. / DEPTH (metres)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	ELEVATION SCALE	WATER CONTENT, PERCENT W_P W W_L (10 20 30 40)	ADDITIONAL LAB. TESTING	GROUNDWATER CONDITIONS
1108.5	Ground Surface							
0.0 1108.0	Organic silt, moss and debris							
0.5	Greenish brown SAND and GRAVEL TILL, ice lenses to 5cm. thickness] Sandy Some ice stratification in pit walls estimated to be 10 percent by volume T = 31° F.		1	CS		obl = 11 G - 64% S - 16% M - 7% C - 2% ○		No Seepage
1104.5								
4.0	End of Test Pit Surveyed pit bottom Elev. 1104.13							

VERTICAL SCALE 1:50

Golder Associates T.P. 80-23

DRAWN R.W.
CHECKED DG

Project No. 722 2025

RECORD OF TEST PIT T.P. 80-24

2155.6N 1171.9E
LOCATION (See Figure 3) DATE Feb. 24, 1980 DATUM Ground Surface

METHOD OF EXCAVATION D8 Cat & Ripper PROJECT Down Valley Tailings

SOIL PROFILE									
ELEVN. DEPTH (metres)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	ELEVATION SCALE	WATER CONTENT, PERCENT W_P W W_L 10 20 30 40	ADDITIONAL LAB. TESTING	GROUNDWATER CONDITIONS
1129.6	Ground Surface							
0.0 1129.1	Organic silt, moss and debris							
0.5 1127.2	Unfrozen greenish-brown SAND & GRAVEL TILL, moist		-1-	CS	cbl. 1 G=26 S=32 M=31 C=10	○		No Seepage
2.4 1125.6	Frozen, greenish brown SAND & GRAVEL TILL some ice stratification to 2.5mm thickness ~ 10% by volume T=31°F.		-2-	CS	cbl=1 G=37 S=30 M=23 C=9	○		
4.0	End of Test Pit. Surveyed pit bottom Elev. 1125.6							

VERTICAL SCALE 1:50

Golder Associates T.P. 80-24

DRAWN R.W.
CHECKED DG

G.B. 101

Project No. 782-2025

RECORD OF TEST PIT 80-25

2103.9N 1234.0E
LOCATION (See Figure)

DATE Feb. 24, 1980

DATUM Ground Surface

METHOD OF EXCAVATION D-8 Cat & Ripper

PROJECT Down Valley Tailings

SOIL PROFILE ELEV N. / DEPTH (metres)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	ELEVATION SCALE	WATER CONTENT, PERCENT W_P W W_L (10 20 30 40)	ADDITIONAL LAB. TESTING	GROUNDWATER CONDITIONS
1122.1	Ground Surface							
0.0 / 1121.8	Moss & Organic debris							
0.3 / 1119.4	Frozen dark brown to brown silty sand, few pebbles, mixed with organics and pieces of wood, gravel content increasing with depth							No Seepage
2.7 / 1118.8	Frozen & unfrozen greenish-bwn. SAND & GRAVEL TILL, occ. gravelly pockets		1	CS	Cbl = 2 G = 24 S = 32 M = 29 C = 13	O		
4.0	End of Test Pit Surveyed pit bottom Elev. 1118.04 m							

VERTICAL SCALE 1:50

Golder Associates T.P. 80-25

DRAWN R.W.
CHECKED DG

RECORD OF TEST PIT SHEETS

Along Diversion Canal

Test Pit No. 79-22 [0-060 to 0-80]

Chainage Test Pit	0+030 to 0+060
	0+180 to 1+210
	0+360 to 0+390
	0+545 to 0+575
	0+750 to 0+770
	0+890 to 0+920
	1+080 to 1+220
	1+380 to 1+400
	1+665 to 1+700

Test Pit No. 79-21 [1+900 to 1+930]

Chainage Test Pit	2+155 to 2+180
	2+500 to 2+530
	2+890 to 2+930
	3+150 to 3+175

Project No. 792 2025

RECORD OF TEST PIT T.P. 79-22

638.2 N 2398.3 E
LOCATION (See Figure 3)

DATE Dec. 22, 1979

DATUM Ground Surface

METHOD OF EXCAVATION D-8 Cat & Ripper

PROJECT Down Valley Tailings

SOIL PROFILE: ELEV N. / DEPTH (m.)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	ELEVATION SCALE	WATER CONTENT, PERCENT (W_P — W — W_L) 10 20 30 40	ADDITIONAL LAB. TESTING	GROUNDWATER CONDITIONS
91.09	Ground Surface							
0.0 / 90.84	Organic debris & volcanic ash	PEAT						
0.25								
90.09	Red-brown gravelly fine SAND							
1.0	Grey SAND and GRAVEL TILL, significant proportion of fines							Dry
88.69			1	C.S.	dbl. 1 G = 50 S = 31 M = 11 C = 6	○		
2.4 / 88.39	Weathered BEDROCK							
2.7 / 88.09	BEDROCK, not rippable		2	C.S.			G_s = 2.80	
3.0	End of Test Pit							

VERTICAL SCALE 1:50

Golder Associates T.P. 79-22

DRAWN R.W.

CHECKED DG

Project No. 792-2025

RECORD OF TEST PIT T.P. 0+045

LOCATION (See Figure) DATE 26 Aug 1979 DATUM Ground surface

METHOD OF EXCAVATION D8 Cat & ripper PROJECT Down valley

SOIL PROFILE: ELEVN. / DEPTH (m)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	ELEVATION SCALE	WATER CONTENT, PERCENT (W_P W W_L)	ADDITIONAL LAB. TESTING	GROUNDWATER CONDITIONS
1091.3	Ground surface							
0.00	Moss & organic debris	FILL						
0.30	Frozen organic SILT, volcanic ash, some boulders							
1090.2								
1.07	Frozen brown sandy GRAVEL (ALLUVIUM), some silty pockets		1	C.S.		O	Cbl-5 G-47 S-39 M+C-9	
1087.6								
3.66	Bottom of Test Pit Surveyed pit bottom elevation 1088.0							

WATER CONTENT, PERCENT: W_P W W_L; 10 20 30 40

VERTICAL SCALE 1:50

Golder Associates T.P.0+045

DRAWN RK

CHECKED DG

G.P. 1

Project No. 792-2025

RECORD OF TEST PIT T.P. 0+195

LOCATION (See Figure) DATE 26 August 1979 DATUM Ground surface

METHOD OF EXCAVATION D8 Cat & ripper PROJECT Down valley

SOIL PROFILE									
ELEVN. / DEPTH (m)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	ELEVATION SCALE	WATER CONTENT, PERCENT W_P W W_L 10 20 30 40	ADDITIONAL LAB. TESTING	GROUNDWATER CONDITIONS
1092.3	Ground surface							
0.00	Organic debris & moss							
0.28	Brown silty GRAVEL TILL, high proportion of schist rock boulder lag at approx. 1.83 m		1	C.S.		O	G - 73 S - 18 M - 9	non-plastic
1089.9								
2.44	Bottom of Test Pit Surveyed pit bottom elevation 1090.2							

VERTICAL SCALE 1:50

Golder Associates T.P. 0+195

DRAWN RK

CHECKED DG

G.B

Project No. 792-2025

RECORD OF TEST PIT T.P. 0+375

LOCATION (See Figure) DATE 26 Aug 1979 DATUM Ground surface

METHOD OF EXCAVATION D8 Cat & ripper PROJECT Down Valley

ELEV N. / DEPTH (m)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	ELEVATION SCALE	WATER CONTENT, PERCENT (W_P W W_L) 10 20 30 40	ADDITIONAL LAB. TESTING	GROUNDWATER CONDITIONS
1089.3	Ground surface							
0.00 / 1088.89	Organic debris & moss							
0.41 / 1088.54	Frozen organic SILT							
0.76 / 1086.1	Frozen brown fine sandy SILT TILL, large schist pebble content							
3.20 / 1084.73	Frozen, mostly grey fine sandy/clayey SILT TILL, large platy pebble content		1	C.S.		O	G-55 S-30 M-14 C-1	non-plastic
4.57	Bottom of Test Pit							
	Surveyed pit bottom elevation 1084.6							

VERTICAL SCALE 1:50

Golder Associates T.P. 0+375

DRAWN RK CHECKED DG

G.B 101

Project No. 792-2025

RECORD OF TEST PIT T.P. 0+560

LOCATION (See Figure) DATE 26 Aug 1979 DATUM Ground surface

METHOD OF EXCAVATION D8 Cat & ripper PROJECT Down Valley

SOIL PROFILE								
ELEV N. / DEPTH (m)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	ELEVATION SCALE	WATER CONTENT, PERCENT W_P W W_L	ADDITIONAL LAB. TESTING	GROUNDWATER CONDITIONS
1090.8	Ground surface							
0.00	Moss & organic debris							
0.15	White volcanic ash							
1090.19	Interlayered organic SILT & brown sandy SILT TILL							
0.61	Greenish grey brown silty fine SAND to fine sandy SILT TILL, high tabular cobble/ boulder content of greenish grey metavolcanic rock							
1088.51								
2.29	Grey essentially massive BEDROCK, mainly at east end of pit; sub-vertical joints occur in the rock							
1086.53								
4.27	Bottom of Test Pit							
	Surveyed pit bottom elevation 1087.8							

VERTICAL SCALE 1:50

Golder Associates T.P.0+560

DRAWN RK

CHECKED DG

RECORD OF TEST PIT T.P. 0+760

LOCATION (See Figure) DATE 26 Aug 1979 DATUM Ground surface

METHOD OF EXCAVATION D8 Cat & ripper PROJECT Down Valley

Project No. 792-2025

SOIL PROFILE									
ELEVN. / DEPTH (m)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	ELEVATION SCALE	WATER CONTENT, PERCENT W_P W W_L 10 20 30 40	ADDITIONAL LAB. TESTING	GROUNDWATER CONDITIONS
1091.9	Ground surface							
	Organic debris & moss							
0.15	White volcanic ash							
	Greenish brown silty fine SAND TILL		1	C.S.		O	G-51 S-28 M-20 C-1	non-plastic
1090.22								
1.68	Greenish grey massive metavolcanic rock dipping into slope							
1087.94								
3.96	Bottom of Test Pit							
	Surveyed pit bottom elevation 1088.8							

VERTICAL SCALE 1:50

Golder Associates T.P. 0+760

DRAWN RK CHECKED DG

G.B. 101

Project No. 792-2025

RECORD OF TEST PIT T.P. 0+905

LOCATION (See Figure) DATE 26 Aug 1979 DATUM Ground surface

METHOD OF EXCAVATION D8 Cat & ripper PROJECT Down Valley

ELEV N. / DEPTH (m)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	ELEVATION SCALE	WATER CONTENT, PERCENT W_P W W_L (10 20 30 40)	ADDITIONAL LAB. TESTING	GROUNDWATER CONDITIONS
1088.9	Ground surface							
0.00 / 1088.60	Moss & organic debris							
0.30 / 1088.23	Organic fine SAND with volcanic ash layer at base							
0.67 / 1087.07	Brown & red brown fine sandy SILT to silty fine SAND, some organic layers, some frozen							
1.83 / 1085.85	Brown sandy SILT TILL, probably frozen, occ. silty fine sand layers							
3.05 / 1084.63	Gravelly fine to coarse SAND ALLUVIUM (probably frozen)							
4.27 / 1084.02	Brown sandy SILT TILL (probably frozen)		1	C.S.		O	G-49 S-30 M-20 C-1	non-plastic
4.88	Bottom of Test Pit							

Surveyed pit bottom elevation 1084.30

VERTICAL SCALE 1:50

Golder Associates T.P. 0+905

DRAWN RK CHECKED DG

G.B.

Project No. 792-2025

RECORD OF TEST PIT T.P. 1+095

LOCATION (See Figure) DATE 26 Aug 1979 DATUM Ground surface

METHOD OF EXCAVATION D8 Cat & ripper PROJECT Down Valley

ELEV N. / DEPTH (m)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	ELEVATION SCALE	WATER CONTENT, PERCENT W_P W W_L (10 20 30 40)	ADDITIONAL LAB. TESTING	GROUNDWATER CONDITIONS
1089.5	Ground surface							
0.00 / 1089.2	Moss & organic debris							
0.30	Olive brown fine sandy SILT TILL (frozen below 1.22 m)							
	Frozen black organic SILT							
1.92 / 1087.1	Frozen SAND & GRAVEL							
2.44	Frozen greenish brown fine sandy SILT TILL		1	C.S.		O	G-51 S-35 M-13 C-1	
1084.6								
4.88	Bottom of Test Pit							
	Surveyed pit bottom elevation 1085.4							

VERTICAL SCALE 1:50

Golder Associates T.P. 1+095

DRAWN RK

CHECKED DG

Project No. 292-2025

RECORD OF TEST PIT T.P. 1+390

LOCATION (See Figure) DATE 26 Aug 1979 DATUM Ground surface

METHOD OF EXCAVATION DB Cat & ripper PROJECT Down Valley

SOIL PROFILE: ELEV N. / DEPTH (m)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	ELEVATION SCALE	WATER CONTENT, PERCENT W_P W W_L	ADDITIONAL LAB. TESTING	GROUNDWATER CONDITIONS
1087.3	Ground surface							
0.00 / 1087.0	Organic debris & moss							
0.30 / 1086.2	Organic SILT, organic debris, lenses of volcanic ash (most of ash above 0.9 m)							
1.07 / 1085.9	BOULDER & COBBLE lag							
1.37 / 1084.9	Brown fine sandy SILT TILL, nonplastic, dense, some fine to medium sand pockets at top of stratum							Water in bottom of Test Pit
2.44	Bottom of Test Pit							
	surveyed pit bottom elevation 1084.7							

VERTICAL SCALE 1:50

Golder Associates T.P. 1+390

DRAWN RK CHECKED DG

Project No. 792-2025

RECORD OF TEST PIT T.P. 1+680

LOCATION (See Figure) DATE 26 Aug 1979 DATUM Ground surface

METHOD OF EXCAVATION D8 Cat & ripper PROJECT Down valley

ELEV. / DEPTH (m)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	ELEVATION SCALE	WATER CONTENT, PERCENT (W_P, W, W_L) 10 20 30 40	ADDITIONAL LAB. TESTING	GROUNDWATER CONDITIONS
1086.5	Ground surface							
0.00 / 1086.1	Organic debris & moss							
0.43 / 1085.3	Frozen organic SILT & brown sandy SILT COLLUVIUM							
1.22 / 1084.7	Brown sandy SILT to silty fine SAND COLLUVIUM with several organic seams		1	C.S.			G-39 S-32 M-28 C-1	
1.83 / 1082.2	Greenish brown fine sandy SILT TILL		2	C.S.			G-53 S-30 M-15 C-2	
4.27	Bottom of Test Pit							
	Surveyed pit bottom elevation 1082.6							

VERTICAL SCALE 1:50

Golder Associates T.P. 1+680

DRAWN RK CHECKED DG

G.B. 101

Project No. 792_2025

Canal Sta. 1+915
1408 N 578 E
LOCATION (See Figure 3)

RECORD OF TEST PIT T.P. 79-21

DATE Dec. 12. 1979 DATUM Ground Surface

METHOD OF EXCAVATION D-8 Cat & Ripper PROJECT Down Valley Tailings

ELEV N. / DEPTH (m)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	ELEVATION SCALE	WATER CONTENT, PERCENT W_P W W_L (10 20 30 40)	ADDITIONAL LAB. TESTING	GROUNDWATER CONDITIONS
1087.4	Ground Surface							
0.0 / 1086.8	Frozen organic silt & debris							
0.6 / 1083.4	Frozen greenish grey SAND & GRAVEL TILL occasional ice lenses. most of material is highly friable		1	CS		O	δ_w = 1999 kg/m³ δ_w = 1917 kg/m³ δ_w = 1781 kg/m³ δ_w = 2082 kg/m³ δ_w = 1866 kg/m³	Frozen Dry
			Several samples taken					
4.0	End of Test Pit NOTE: This pit subsequently incorporated into the test excavation.							

VERTICAL SCALE 1:50

Golder Associates T.P. 79-21

DRAWN R.W.
CHECKED DG

Project No. 792-2025

RECORD OF TEST PIT T.P. 2+155

LOCATION (See Figure) DATE 25 Aug 1979 DATUM Ground surface

METHOD OF EXCAVATION D8 Cat & ripper PROJECT Down Valley

ELEVN. / DEPTH (m)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	ELEVATION SCALE	WATER CONTENT, PERCENT W_P W W_L (10 20 30 40)	ADDITIONAL LAB. TESTING	GROUNDWATER CONDITIONS
1084.0	Ground surface							
0.00 / 1083.7	Moss & organic debris							
0.30	Frozen, mixed brown sandy SILT (TILL) COLLUVIUM & organic SILT, some visible ice		1	C.S.		O	G-25 S-34 M-32 C-9	
1080.65								
3.05	Brown silty SAND						G-35 S-38 M-24 C-3	
1076.69			2	C.S.		O		
3.96 / 1072.42	Blue green massive BEDROCK							
4.27	Bottom of Test Pit							

NOTE: Test pit elevation estimated from diversion canal baseline profile.

VERTICAL SCALE 1:50

Golder Associates T.P. 2+155

DRAWN RK CHECKED DG

G.E

Project No. 792-2025

RECORD OF TEST PIT T.P. 2+515

LOCATION (See Figure) DATE 25 Aug 1979 DATUM Ground surface

METHOD OF EXCAVATION D8 Cat & ripper PROJECT Down Valley

ELEV N. / DEPTH (m)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	ELEVATION SCALE	WATER CONTENT, PERCENT W_P W W_L (10 20 30 40)	ADDITIONAL LAB. TESTING	GROUNDWATER CONDITIONS
1083.5 / 0.00	Ground surface							
1083.1 / 0.41	Moss & organic SILT							
	Brown fine sandy SILT TILL, non-plastic to low plasticity, friable boulder layer at 5.5 m		1	C.S.		O	G-45 S-25 M-25 C-5	
			2	C.S.		O	G-45 S-34 M-20 C-1	
			3	C.S.				
1076.7 / 6.83	Bottom of Test Pit		4	C.S.				

VERTICAL SCALE 1:50

Golder Associates T.P. 2+515

DRAWN RK CHECKED DG

G.P. 101

Project No. 792-2025

RECORD OF TEST PIT T.P. 2+910

LOCATION (See Figure) DATE 25 Aug 1979 DATUM Ground surface

METHOD OF EXCAVATION D8 Cat & ripper PROJECT Down Valley

SOIL PROFILE ELEVN. / DEPTH (m)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	ELEVATION SCALE	WATER CONTENT, PERCENT W_P W W_L (10 20 30 40)	ADDITIONAL LAB. TESTING	GROUNDWATER CONDITIONS
1086.0	Ground surface							
0.00 / 1085.7	Organic debris & moss							
0.30 / 1085.09	Frozen olive brown fine sandy SILT COLLUVIUM							
0.61 / 1083.57	Frozen brown sandy SILT COLLUVIUM, contains visible ice crystals, organic layer & cobbles at 1.2 m							
1.52 / 1079.91	Frozen brown sandy SILT TILL, ice rich, non plastic to low plasticity							
3.66 / 1074.73	Frozen grey & brown fine sandy SILT TILL, organic seam at 4.6 m		1	C.S.		O	G-19 S-36 M-39 C-6	NOTE: Surficial mud flows occurring thawing progresses
5.18	Bottom of Test Pit							

NOTE: Test pit elevation estimated from diversion canal base-line profile.

VERTICAL SCALE 1:50

Golder Associates T.P. 2+910

DRAWN RK

CHECKED DG

G.B 101

Project No. 792-2025

RECORD OF TEST PIT T.P. 3+160

LOCATION (See Figure) DATE 25 Aug 1979 DATUM Ground surface

METHOD OF EXCAVATION D8 Cat & ripper PROJECT Down Valley

SOIL PROFILE ELEVN. / DEPTH (m)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	ELEVATION SCALE	WATER CONTENT, PERCENT W_P W W_L	ADDITIONAL LAB. TESTING	GROUNDWATER CONDITIONS
1072.3	Ground surface							
0.00	Frozen organic SILT & debris & thin volcanic ash seams, cobble lag at base of stratum							
1071.69 / 0.61	Frozen brown grey fine sandy SILT with organic seams at approx. .15 m intervals, some coarse sand seams adjacent to organics							
1069.25 / 2.44	Unfrozen GRAVEL & SAND with boulder lag at top of stratum, some silty pockets							
1065.59 / 3.66	Bottom of Test Pit							

NOTE:
Test pit elevation estimated from diversion canal baseline profile.

VERTICAL SCALE 1:50

Golder Associates T.P. 3+160

DRAWN RK
CHECKED DG

Selected Site Photographs

Mine Waste Dumps

Water supply storage reservoir

Existing Tailings Pond

Proposed Intermediate Dam

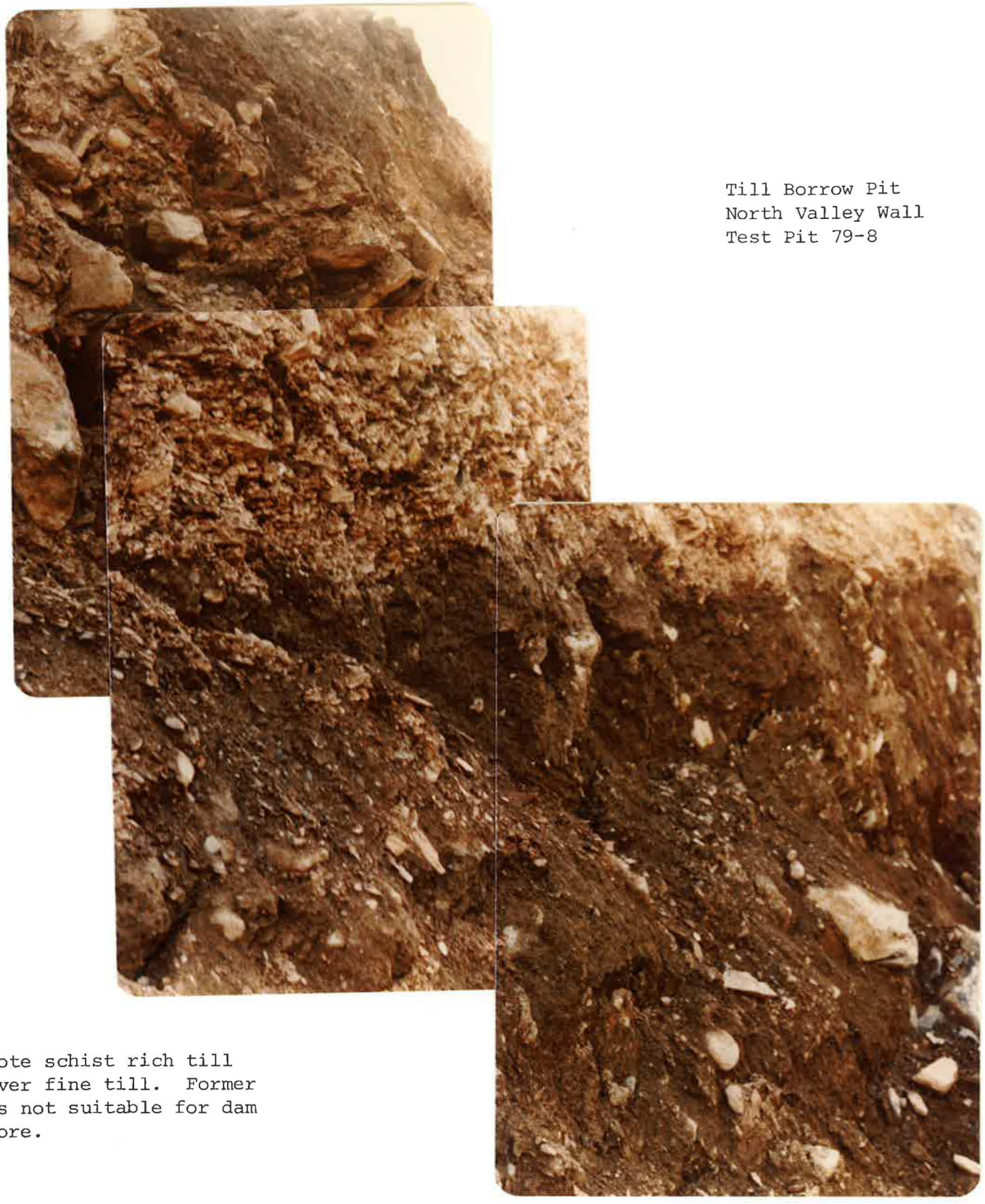
Proposed Cross Valley Dam

An aerial view of the Rose Creek Valley - June 1, 1979
(looking southeast and upstream)

Photos of Creek near Sta. 0+600 June 1, 1979. Several boulders and bedrock outcrops occur in the vicinity of the creek near the proposed alignment of the diversion canal.

Photos of Creek in outfall area of proposed diversion canal (approx. Sta. 3+000)

June 1, 1979



Till Borrow Pit
North Valley Wall
Test Pit 79-8

Note schist rich till over fine till. Former is not suitable for dam core.

Till Excavation at Test Pit 0+750

Till Excavation at Test Pit 2+515

Examples of excavated frozen till material from Test Pit 79-21. (Sta. 1+900 of diversion canal)

Temperature -40° C.
December, 1979

Material had to be ripped but usually crumbled on excavation unless ice lensed.

Sample of frozen till recovered from Test Pit 79-21 near Sta. 1+900 of diversion canal.

Sample contains several prominant ice lenses.

It is estimated that 25% of the excavation would be this type of material.

Unit weight:

119.7 lb/cu.ft.
1920 kg/m^3

Sample of frozen till recovered from Test Pit 79-21 near Sta. 1+900 of diversion canal.

This sample is quite dry.

Unit weight:

111.2 lbs/cu.ft.
1783.6 kg/m^3

Access roads on south valley wall at west end of proposed diversion canal.

August, 1979

Note coarser gravel in upper 1.5 m

Borrow Pit near West Tailings Dam

Gravel shell of West Tailings Dam

Drain Material manufactured from Borrow Pit gravels

Example Photographs of Riprap Quarry Developed in 1974.

Example Photographs of Riprap Quarry Developed in 1974.

Looking west towards working face. Note rock size centre right and poor rock quality at right and left edges of pit. Area below man has been blasted. Note it is not excessively loosened. Note traces of drill holes on left hand face.

Example of Developed Gravel Terrace Borrow Pits.

Use cat tracks (about 0.5 m wide) for scale.

March 21, 1980

Test Excavation
in progress.

Final touches on excavation March 22, 1980 a.m. (looking N.W.)

Note completed dyke and pilot channel notch.

Final survey being conducted.

Starting to place gravel on ramp.

Placement of Gravel Slope Protection.

Note width of gravel

As the buggy rounds the bend (which will not be the case in the final excavation) there is a tendency for width to increase because back wheels move out on slope.

Completed test excavation.

March 27, 1980

Note:

1. Borrow area left of centre.

2. Drainage trench. Subsequent to photo gravel plug removed and gravel dozed over bottom of trench ramp (trench has two loads gravel in bottom).

3. Excess gravel in S.W. corner dozed up onto area where stake is.

APPENDIX V

TEST EXCAVATION

NOTE: A full report regarding construction experience, compaction densities and monitoring of the test section is in progress. The final design of the canal will be further modified if the test section performance indicates need.

Golder Associates
CONSULTING GEOTECHNICAL ENGINEERS

A REPORT
to
CYPRUS ANVIL MINING
Providing

FINAL DESIGN RECOMMENDATIONS
For The
DOWN VALLEY TAILINGS DISPOSAL PROJECT
Faro Yukon Territory

VOLUME THREE
OF
THREE VOLUMES

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792-2025 June, 1980

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Cross Valley Dam Design

Golder Associates

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1.0 INTRODUCTION

The purpose of this section of the report is to provide specific information regarding the design of the Cross Valley earth dam. Information regarding the overall project has been previously discussed and detailed geotechnical data related to the foundation stratigraphy and the proposed borrow pit materials may be found in Appendices III and VII.

The function of the Cross Valley Dam is to impound the water after it has been separated from the tailings effluent of the mill for a sufficiently long period of time to permit chemical adjustment and to allow deleterious particles to settle out of suspension. It is required that this water be kept out of contact with the tailings which will be contained upstream by an Intermediate Dam. Because of this requirement, it is not possible to capitalize on the seepage reduction benefits which a tailings dam gets from the upstream blanket of fine tailings sludge which typically covers the ground surface.

It is necessary, therefore, to treat the Cross Valley Dam as a normal water retaining embankment dam rather than a tailings retention dam. Consequently, the dam design discussed in this report is based on procedures appropriate to an earth dam situated in an area of moderate seismicity. The only advantage which may be taken of the mining context of this construction project is that we may presume construction equipment and manpower will be immediately available should emergency remedial work become necessary.

The presence of permafrost in the foundation, the possibility of construction work in severe winter conditions, and the scarcity of precedents for similar structures in this environment are major reasons for concern and demand that a reasonably conservative approach be applied in all significant aspects of the design.

2.0 GEOLOGY OF SITE

The detailed geology of the site has been discussed in Section 6 of this report. The results of the site investigation work are presented in Appendix III.

From the viewpoint of dam stability and seepage control, it is important to note the following major geological features of this site which influence the design.

i) The valley is rock controlled and infilled by alluvium or glacio-fluvial deposits to a depth of 25 m to 50 m.

ii) The deep alluvial deposits vary in composition from pervious sands and gravels to relatively impervious clayey silts or fine sandy silts. The distribution of these different materials is discontinuous and appears to be random both in depth and lateral extent.

iii) The soil-bedrock contact zone appears to be highly pervious at some locations, consisting either of highly fractured schist or coarse grained alluvium.

iv) The near surface strata contain significant amounts of organic matter varying from muskeg to organic silts.

v) There is evidence of discontinuous permafrost in the valley bottom and the abutments. There is evidence that permafrost has invaded most of the soil types and extends to about 9 m depth in the central valley bottom and beyond 16 m in the south abutment. Moisture contents as high as 56 per cent have been recorded for frozen organic silts.

vi) In the north abutment at depth there is a deposit of uniformly graded silt which may be rock flour.

vii) The valley bottom and abutment groundwater levels are generally consistent with the present Rose Creek level.

The significance of the above aspects of the site geology are discussed separately in the appropriate sections of this report.

3.0 EARTH DAM DESIGN CONSIDERATIONS

There are several special considerations which are specifically applicable to the design of a water retention embankment at the Cyprus Anvil site.

3.1 Pollution Considerations

Water seepage through the embankment should be insignificant under normal conditions, particularly in comparison to seepage through the foundation.

Underseepage beneath the dam will be directly related to the permeability of the dominant foundation strata. Since the impounded water may be chemically contaminated, it is necessary in the dam design to provide measures to ensure the quantity of seepage is restricted to reasonable amounts and that water which unavoidably bypasses the control provisions is largely recaptured downstream of the dam.

3.2 Distortion of Structure

To prevent a large discharge of impounded water in an uncontrolled manner, it is necessary to ensure that the embankment does not suffer distortions or settlements in excess of design allowances. Cracking of either the embankment core or the upstream blanket, or overtopping of the spillway or embankment itself must, therefore, be addressed and appropriate mitigative design developed.

The presence of discontinuous permafrost in the foundations, to depths which are inaccessible in practical terms, requires that the

embankment design allow for abrupt differential settlements which are in excess of those normally encountered in such structures.

3.3 Increased Dam Height

The possibility exists that the future storage needs of the mine may necessitate that the volume of the reservoir be enlarged. This would be accomplished by substantially increasing the crest level of the embankment. To allow for this contingency, the critical zones of the dam (core and drain) must be designed to accommodate a significant increase in the reservoir level.

3.4 On Site Equipment

As stated earlier, the proximity of this project to an operating mine is a distinct advantage in that construction equipment and manpower would be accessible at short notice. This would allow maintenance and/or remedial work to be performed in areas of the dam which might require repair as and when it became necessary. Since the foundation settlements in areas underlain by deep permafrost cannot be accurately predicted, either in time or magnitude, it is practical and prudent to rely to some degree upon the "observational method" of design and to plan for some contingency repair work.

3.5 Design Modifications During the Construction Phase

It is normal engineering practice in earth dam construction to modify the design during the course of construction, especially at the stage of foundation preparation. Due to the complicated nature of the alluvium at this site, it is reasonable to anticipate that some design modifications will be required here also.

4.0 RECOMMENDED ZONING OF DAM

Figure VI-1 shows the dam zoning for a typical embankment section.

The main elements of the structure are discussed below.

4.1 Crest Width and Elevation

i) The crest width should be sufficient to allow safe passage of all construction, maintenance or mining equipment which may be allowed access to the crest. We recommend that the crest should be at least 6 m wide.

ii) The crest should be surfaced with crushed granular fill of sufficient thickness to provide an all-weather road without permitting damage to the core.

iii) The design crest elevation should provide a freeboard of about 1.5 m. Therefore, for a maximum normal pond level of 1063.5 m, the design crest level should be 1065 m.

iv) The constructed crest elevation should allow for 0.25 m embankment fill settlement in addition to a conservative allowance for foundation settlement.

4.2 Side Slopes

Provided the fill used in the construction of the embankment shells is compacted to 100 percent of standard Proctor maximum dry density and that it can provide shear strengths equivalent to at least $\emptyset' = 33$ degrees, both the upstream and downstream side slopes may be constructed at 2 horizontal to 1 vertical. This recommendation is made on the understanding that the pond will fill relatively slowly and that there is no provision for rapid reservoir drawdown.

4.3 Impervious Zones

The fill material used in the central core, the partial cut-off, and the upstream blanket beneath the shell should consist of a well graded silt till with between 20 percent and 50 percent passing the No. 200 sieve. The till should be compacted to 100 percent of standard Proctor dry density. The placement moisture content for the primary blanket and

the lower core should be at, or slightly below, optimum compaction moisture content.

i) A central core has been chosen so that the primary upstream blanket section beneath the shell (i.e. that part of the blanket resting on reliable foundation material) could be maximized. The width of the core at tail water level should be at least 7 m; should the dam be subsequently raised to 1082 m this would provide a ratio of core width to differential head of 22 per cent. The core top width should be 4 m minimum.

ii) The partial cut-off trench should be a minimum of 3 m deep and have a 4 m wide base. Where foundation treatment requires excavation in excess of 3 m, the impervious fill should be carried down to the bottom of the excavation. The downstream face of the trench should be inspected to ensure that the impervious fill cannot enter the natural ground and if necessary, a filter layer of well graded sand and gravel should be interposed.

iii) The contact between the core, the trench, and the blanket should be made in such a manner as to ensure that preferential seepage along the contacts will not occur. The trench should preferably be excavated in the dry to allow inspection of the side walls and the base; backfilling and compaction in the dry is also desirable.

However, if it proves impractical to dewater the excavation, it would be acceptable to excavate and backfill through water, provided:-

(a) sufficient on site inspection is available to ensure that the trench has been adequately cleaned,

(b) backfilling is performed according to a specified placement technique,

(c) the slurry accumulation at the forward edge of the fill be removed, and

(d) a maximum amount of stratigraphic detail is acquired from the spoil.

iv) The impervious blanket should be extended 60 m upstream of the upstream toe using a 2 m depth of relatively impervious fill. It is anticipated that the clayey till which is scheduled to be recovered from the channel excavation in the winter of 1980 will be a suitable material. Some effort should be made to compact this soil after it has thawed. If there is a surplus of clayey till available, this material would be more usefully usefully employed by increasing the blanket thickness closer to the dam rather than extending the blanket further upstream.

Both the primary and secondary blankets should extend up the valley walls to elevation 1065 m, or to make contact with a impervious strata which is continuous to crest level.

4.4 Upstream Filter/Transition Zone

Should the impervious zones crack for one reason or another, it is desirable to have a soil type upstream of the crack which would be washed into the hole to help seal off the leak. This provision avoids depending on the core material itself to expand into the space and the consequent reduction of the density of the neighbouring core material.

Ideally a fine, non-cohesive fill such as a silty sand should be used upstream of the core. The coarse fraction of the silty till or the fine tail of the sand and gravel would be suitable materials. This filter transition fill should be placed against the upstream face of the core and also over the primary section of the upstream blanket.

4.5 Downstream Seepage Control Zones

The most critical shell fill sections are the chimney drain and drainage blanket which control seepage downstream of the core (see Figure VI-1). Both these zones should be constructed of clean, well graded sand and gravel fill. Every effort must be made to avoid contamination of these fill zones by silty or clayey materials.

The chimney drain should be 4 m thick and be continuous with the horizontal blanket drain which should be 2 m deep. The drain should terminate in a shallow seepage collector ditch at the downstream toe to facilitate seepage monitoring and/or collection and treatment of water.

The foundation material in contact with the base of the horizontal blanket drain should be inspected during foundation treatment to determine if it is sufficiently coarse to prevent contamination of the drain and if necessary, a filter layer should be interposed. Also, the gradation of the drainage zones should be checked during construction for compliance with the filter criteria with respect to the core. Finally, the downstream shell fill should be checked to ensure that it cannot contaminate the horizontal drain by being washed into the selected gravel fill during rainfall percolation.

4.6 Upstream and Downstream Shell Fills

Select fill materials are required in the core and primary blanket and also in the downstream chimney and blanket drains. The shell materials are less important to the integrity of the dam but, nevertheless, must not contain organic, frozen or other deleterious matter.

In the upstream shell, the finer fill materials should be placed near the base and near the core, with the coarser materials being placed towards the outside slope. In the downstream shell, the coarser materials should be placed on the side closer to the drainage zones. In addition, the outside face should contain gravel sizes for erosion protection.

4.7 Frozen Fill Materials

Special care must be taken throughout construction to ensure that frozen fills are not placed in the dam. Measures must also be taken to avoid freezing of exposed compacted fill horizontal surfaces, especially where the fill materials are frost susceptible.

5.0 SITE PREPARATION

Permafrost has been encountered at the dam site during foundation investigation work.

Based upon existing theoretical computational techniques, the data in Figure VI-2 is presented to underline the significance of frozen ground beneath a structure. It may be seen from this figure that, for materials similar to the frozen organic silts present at this site, thawing would progress at the rate of 5 m to 7 m in one year, depending upon the moisture content of the soil. The bottom set of curves indicates the pore pressures which would be generated in soils of various consolidation characteristics. From this, it may be seen that unacceptably high pore pressures would develop beneath the dam during thawing of soils exhibits a coefficient of consolidation, c_v, below 10^{-6} m^2/s.

Because of relatively warm water being impounded above the blanket and upstream core, and because of the inevitability of seepage water entering the downstream foundation area, it is necessary to recognize that thawing of permafrost beneath the dam is unavoidable. Consequently, we recommend that all frozen soils which could result in either high pore pressure development or large foundation settlements be removed by excavations prior to placing embankment fill.

5.1 Choice of Alignment

Since the main problem related to this site is associated with the presence of discontinuous permafrost in the foundations and abutments it is highly desirable to locate the dam alignment in an area which minimizes

the adverse effects of frozen ground which cannot be readily removed. The present alignment, close to the active creek channel, was chosen for this reason. The available investigation records indicate that the permafrost beneath the dam on this alignment is minimal and this is consistent with the fact that the nearby creek is a large source of heat which would provide the energy required to thaw the frozen ground locally.

5.2 Foundation Preparation

i) The entire foundation area of the embankment should be cleared, grubbed and stripped.

ii) Organic matter and the frozen organic silts (or other frozen materials with a c_v less than about 10^{-5} m^2/s) should be completely removed.

iii) Frozen uniform deposits, whether silt, sand or gravel, of significant lateral extent should be assessed to determine their probable density after they become thawed. Should the predicted density fall below 60 per cent relative density, it would be necessary to remove such deposits to avoid the possibility of liquefaction during seismic loading.

iv) Where natural materials are removed, the excavation should be backfilled with compacted granular materials or non-plastic till, depending on the location.

v) Because of the differential settlements which are likely to occur in the upstream secondary blanket due to permafrost degradation, it is not considered reasonable to expend a lot of effort on foundation preparation in this area. However, the area should be cleared of any small trees, bushes or logs. Where coarse gravels or cobbles appear at the surface, these should be covered by well graded sand and gravel.

5.3 Further Foundation Investigation

During the foundation clearing stage, and also during the core trench excavation, it is recommended that the existance and extent of frozen ground be further defined by means of drilling and seismic methods.

6.0 SEEPAGE CONTROL

Earth dams are designed to both minimize the amount of water which seeps through and beneath a dam and also to remove this pressurized water in a controlled manner so as to avoid loss of material from the foundation strata and to prevent destabilization of the downstream face of the embankment.

6.1 Seepage Reduction

i) The central core will be relatively impervious and the amount of water which will pass through the embankment itself will be insignificant provided the core remains intact. Should a crack open in the core, it is most likely that the period of high seepage would be limited since the core and transition fills would consist of non-cohesive soil which would have the capability to plug the crack through seepage induced migration.

ii) The upstream blanket is intended to present a relatively impervious barrier to reservoir water entering the foundation within the neighbourhood of the dam. The longer the blanket, up to a point, the smaller the volume of seepage which could enter the foundation.

Figure VI-3 shows the theoretical relationship between seepage water flowing beneath the dam and the length of blanket constructed. As can be seen from this plot, for the parameters listed above the curve, a 30 m blanket is distinctly beneficial, whereas a blanket length greater than 90 m has not apparent incremental advantage. Consequently, the total blanket length recommended for the Cross Valley Dam is 90 m. The initial 30 m,

the most efficient length, is termed the primary blanket and should be constructed over foundations from which permafrost has been removed. The remaining 60 m, the secondary blanket, is less efficient and because of this, it is considered unwarranted to perform expensive foundation preparation to avoid some cracking of this section of the blanket.

The underseepage which is expected to pass through the foundations cannot be reliably estimated since the quantity is directly related to the hydraulic conductivity of the most highly permeable strata beneath the blanket. If the permeabilities and foundation parameters shown in Figure VI-3 are correct, the seepage through the Cross Valley Dam foundation would amount to about 0.03 m^3/s (400 igpm) for an intact blanket, or to about 0.04 m^3/s (550 igpm) if only the primary blanket were intact.

It must be emphasized, however, that it is not possible to determine the average permeability of the foundation within one order of magnitude although an understanding of the site geology helps to some degree in assessing the probable effective conductivity of the aquifer. At Cyprus Anvil, it is likely, judging from the site investigation records and the geomorphology of the creek channel, that the vertical permeability is probably low due to the alternation of the fine grained and coarse grained deposits. Also, it appears unlikely that there are extensive continuous horizontal strata of high permeability extending from points upstream of the blanket to areas downstream of the core.

iii) While a partial cut-off is quite ineffective in reducing underseepage in theory, these trenches prove to be quite successful in practice. A partial cut-off interrupts the horizontal continuity of the upper strata and these are typically more loose and disturbed or weathered than those at

greater depth. The trench would also cut across animal holes and remove soils which might have contained shrinkage cracks or decomposing roots.

6.2 Reduction of Seepage Pressures

i) The chimney drain directly behind the core is designed to relieve water pressure due to seepage through the core. The chimney is also purposely graded to prevent core material being washed away in the event of a crack developing in the core fill.

ii) The horizontal blanket drain provides a discharge conduit for the core seepage and allows the foundation underseepage to vent harmlessly at low pressure. Consequently, the possibility of high, destabilizing pressures beneath the downstream shell are greatly reduced.

iii) The discharge from the blanket drain, in turn, would be collected in the seepage ditch where it could be observed and monitored. If desired, this water might then be collected and treated or pumped back into the reservoir.

iv) If, through observation of instrumentation, seepage pressures in the downstream foundation after construction are found to be undesirably high, or boils appear in the ground downstream of the dam, relief wells may be installed at spacings which would be adequate to bring the pressures under control again.

7.0 STABILITY OF DAM

The stability of the proposed earth dam and its shallow foundations have been computed to determine the factor of safety (F) against shear failure for the most critical loading conditions anticipated during the embankment's construction and operating life.

7.1 Method of Analysis

The Janbu method of stability analysis has been used to determine F for each of the loading and strength conditions shown on Figures VI-4 and VI-5. Relatively steep side slopes of 2 horizontal to 1 vertical have been used in the embankment design so to minimize the area beneath the dam which would require foundation treatment.

In order to approximate the loading conditions pertaining during seismic vibrations resulting for an earthquake the Janbu analysis was modified to incorporate a pseudostatic horizontal load in the equation.

The earthquake accelerations used were those recommended for this site by the Dominion Observatory in Victoria, B.C. For the 100 year return period earthquake an acceleration of 0.054 g was used and for the 200 year shock 0.097 g was used.

7.2 End of Construction Condition

The first critical stability condition which develops in an earth dam is the "end of construction" case, when the embankment reaches its full height and before pore pressure in the impervious fills have had time to dissipate. It is our opinion that for a dam of this height, utilizing non-plastic impervious fill materials, and designed with a relatively slender central core which is compacted at or below optimum moisture content, pore pressures immediately following construction will not be sufficient to threaten the stability of the section.

7.3 Upstream Slope Stability

Figure VI-4 summarizes the results of analyses performed on the failure wedge shown in the upper diagram. The F values computed for the slope for a range of impervious fill or foundation soil strengths is shown in the lower diagrams. Two conditions were examined.

i) Considering a full pool, i.e. reservoir water impounded to maximum normal water level, the F value for static (i.e. no earthquake) as well as for horizontal accelerations equivalent to the 200 year return period earthquake are shown. It may be seen for the plots that the F value remains considerably above unity (limit equilibrium or the point of failure) for both conditions even when unreasonably weak soil conditions are assumed.

ii) Normally the critical stability condition for the upstream face of an embankment dam is during rapid reservoir drawdown. Although there is no provision for drawdown of the reservoir at this site, there is still the possibility that the water level could be surcharged to crest level due to failure of the new Rose Creek canal followed by drawdown below normal pool level because of spillway erosion.

A series of analyses were, therefore, carried out for the condition of 2.5 m instantaneous drawdown, i.e. a sudden drop in pool level without any attendant benefits in reduced blanket or core pore pressure reductions. The results plotted on Figure VI-4 show F value substantially above unity even for the remote possibility of a coincidental 100 year earthquake.

7.4 Downstream Slope Stability

Figure VI-5 summarizes the results of stability analyses performed on the downstream slope for the wedge failure surface indicated in the sketch at the top of the figure. The soil strengths used in the analyses are also noted.

Three conditions were examined for both the static loading case and for the loading associated with the 200 year return earthquake.

i) Assuming the entire blanket remains intact, the hydraulic gradient beneath the downstream slope would be low and consequently the F value of the slope would be high. The results for this, the most optimistic condition, are labelled A.

ii) If it is assumed that the secondary blanket is ruptured to the extent that it becomes totally ineffective, the resulting higher seepage pressures beneath the downstream shell would reduce the F values to those shown on line B.

iii) If it is assumed that the primary blanket is seriously ruptured or becomes parted from the core, the F values would drop to those shown on line C.

It may be seen that in all cases the computed values of F remain above unity for foundation strengths as low as $\emptyset'$ = 30 degrees.

7.5 Foundation Stability

The foundation soils are generally competent and would normally not be considered as a possible cause for concern with regard to shear failure. However, the foundation strata at Cyprus Anvil contain permafrost, and consequently caution must be exercised in assessing the foundation stability for two reasons:-

i) High pore pressure may be generated when soils having a low coefficient of consolidation thaw, and

ii) When soils of high moisture content thaw the resulting soil may be of very low density.

During foundation preparation, it is intended to remove the frozen organic silts entirely and, therefore, the influence of permafrost will to a large degree be removed. However, it will still be necessary to

establish that no significant frozen deposits of uniformly graded silts, sands, or gravels remain which, when thawed, might produce a loose deposit which could liquify during an earthquake.

In the event that it becomes necessary at some later date to increase the crest height of the dam, we recommend that the rock flour deposit on the north abutment be further investigated to determine its density and extent.

8.0 EROSION PROTECTION

i) The upstream slope should be protected from erosion by wave action. At Cyprus Anvil, it should prove adequate to push the coarser granular fills out to the face of the slope. Since the pond level is likely to remain relatively stationary at about 1063.5 m, it would be prudent to place about 0.5 m of coarse gravel to cobble sizes in the elevation range 1062 m to 1065 m.

ii) The downstream slope should be protected against rutting due to rain-water run-off. Again, pushing the coarse materials to the outside face should provide adequate protection.

iii) To avoid overtopping of the embankment, it is necessary to provide a spillway which can safely discharge the design run-off. In this case, it would be prudent to assume that the spillway should be capable of carrying the flow from Rose Creek. This would allow for the contingency of canal failure or a temporary diversion for canal maintenance.

9.0 INSTRUMENTATION

9.1 Visual Monitoring

It is imperative that, both during construction and during the working life of the dam, routine observations be made of the embankment and the area downstream of the dam by an informed conscientious inspector. Cracks or settlements of the embankment, or the appearance of springs or

boils in the dam or the natural ground downstream, should be immediately reported to the designers. A particularly careful examination should be made following earthquakes or other instances of unusual loading or pond operation.

The visual observations should be supplemented by readings taken from the installed instrumentation discussed below.

9.2 Instrumentation

Four simple types of monitoring devices are recommended for installation in the embankment.

i) Piezometers should be placed behind the core at the three positions shown on Figure VI-6. The purpose of these devices is to measure water pressure in the drainage layers and the foundations. These measurements would provide useful data regarding the functioning of the seepage control zones of the dam. Sets of piezometers should be installed at the three quarter points of the embankment alignment as a minimum. During construction other areas of concern may be identified which require installation of piezometers.

ii) Settlement gauges should be installed at the two locations shown on Figure VI-6, i.e. beneath the core-to-blanket contact and at the base of the partial cut-off trench. These devices will provide data on the differential settlements in this important contact zone.

Sets of settlement gauges should be installed on the same sections used for piezometers.

iii) Surface movement monuments, consisting of metal pins driven or concreted into the crest should be placed along the upstream side of the crest at intervals of 50 m. Routine surveys of

these monuments will help to identify any unusual embankment movements. Each of these pins should be carefully protected from accidental damage due to traffic.

(vi) One or more V-notch weirs should be constructed in the seepage ditch to measure the rate of water flow. This simple device, when read at regular intervals, is perhaps the most useful indicator of dam performance.

It is most important that readings be taken on a routine basis and that the data be reduced and plotted on the same day it is measured.

10.0 LABORATORY TESTING

10.1 Site Control Laboratory

During construction it will be necessary to have a materials laboratory on site. This laboratory should be capable of performing routine soil mechanics classification and compaction tests.

10.2 Laboratory Shear Strength Test Results

Laboratory triaxial compression tests have been performed in Golder Associates' Vancouver laboratory on three samples of soil which are being considered for embankment construction. These soils represents the principal types of materials which are proposed for use for embankment fills. The results are summarized in Figures VI-7, VI-8 and VI-9.

Figure VI-7 shows the results for a silty till which is proposed for use in the core, the primary blanket and the partial cut-off trench.

Figure VI-8 shows the results for a clayey till which is proposed for use in the secondary blanket and may be used in non-critical areas of the shell fills.

Figure VI-9 shows the results for a sand and gravel which is proposed for use in the drainage zones of the dam and in the sensitive areas of the shell fills.

All three materials indicate satisfactory shear strengths at the stress levels which will pertain in the Cross Valley dam. The absence of a cohesion intercept in the silty till sample indicates it is more suitable for core fill material than the clayey till.

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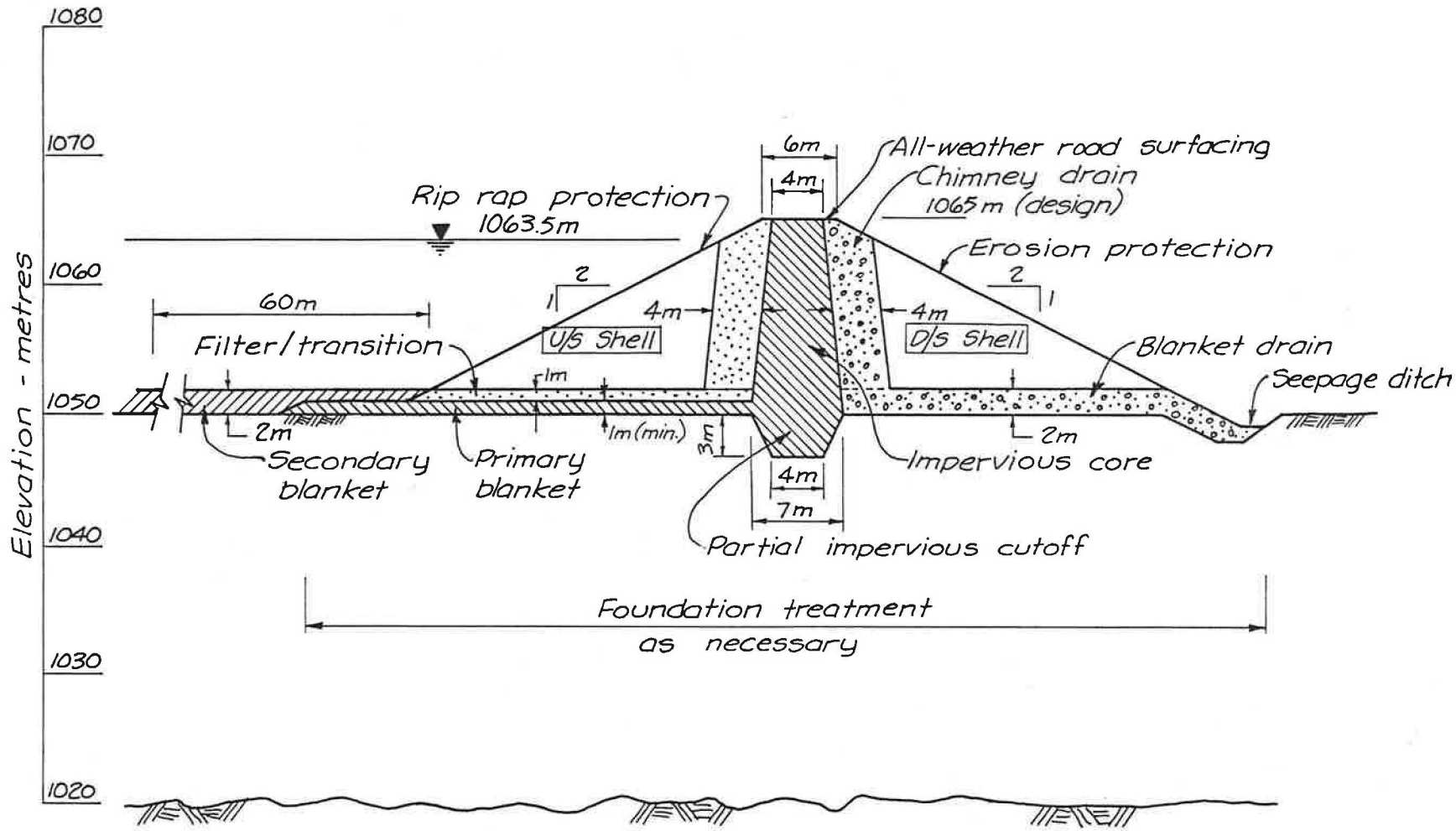

CROSS VALLEY DAM, TYPICAL SECTION Figure VI-1

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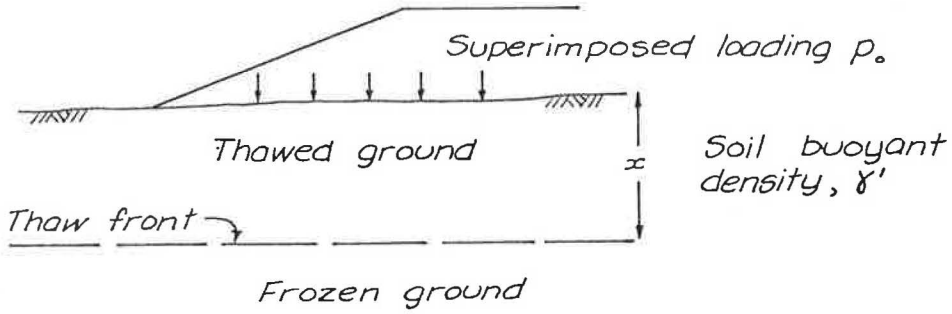
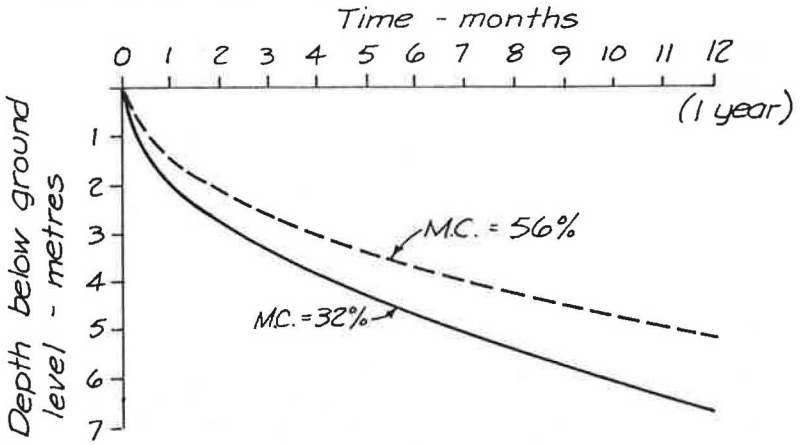
DEGRADATION OF PERMAFROST

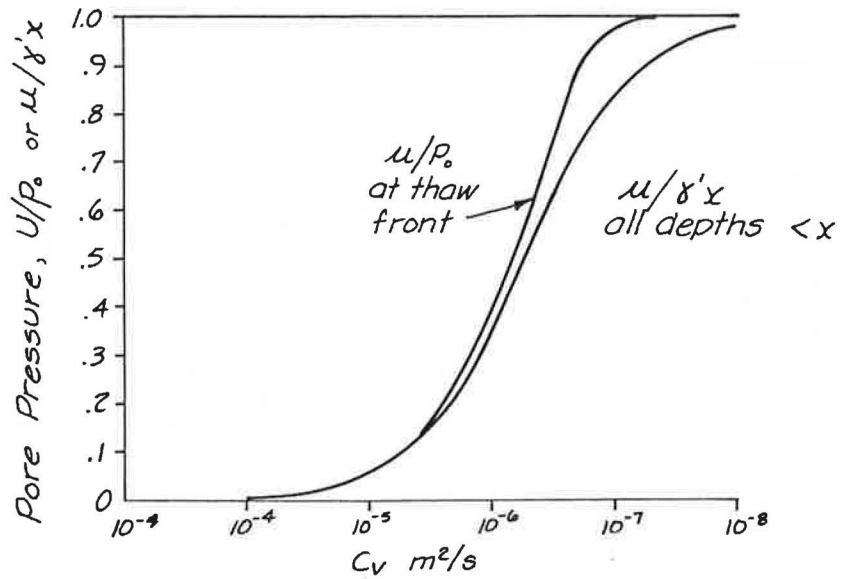
Figure VI-2



RATE OF PENETRATION OF THAW FRONT IN ORGANIC SILTS (APPROXIMATELY)

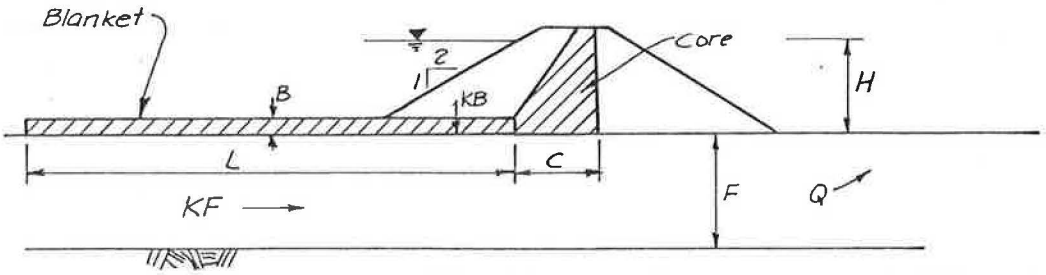


PORE PRESSURES GENERATED BY DEGRADING PERMAFROST



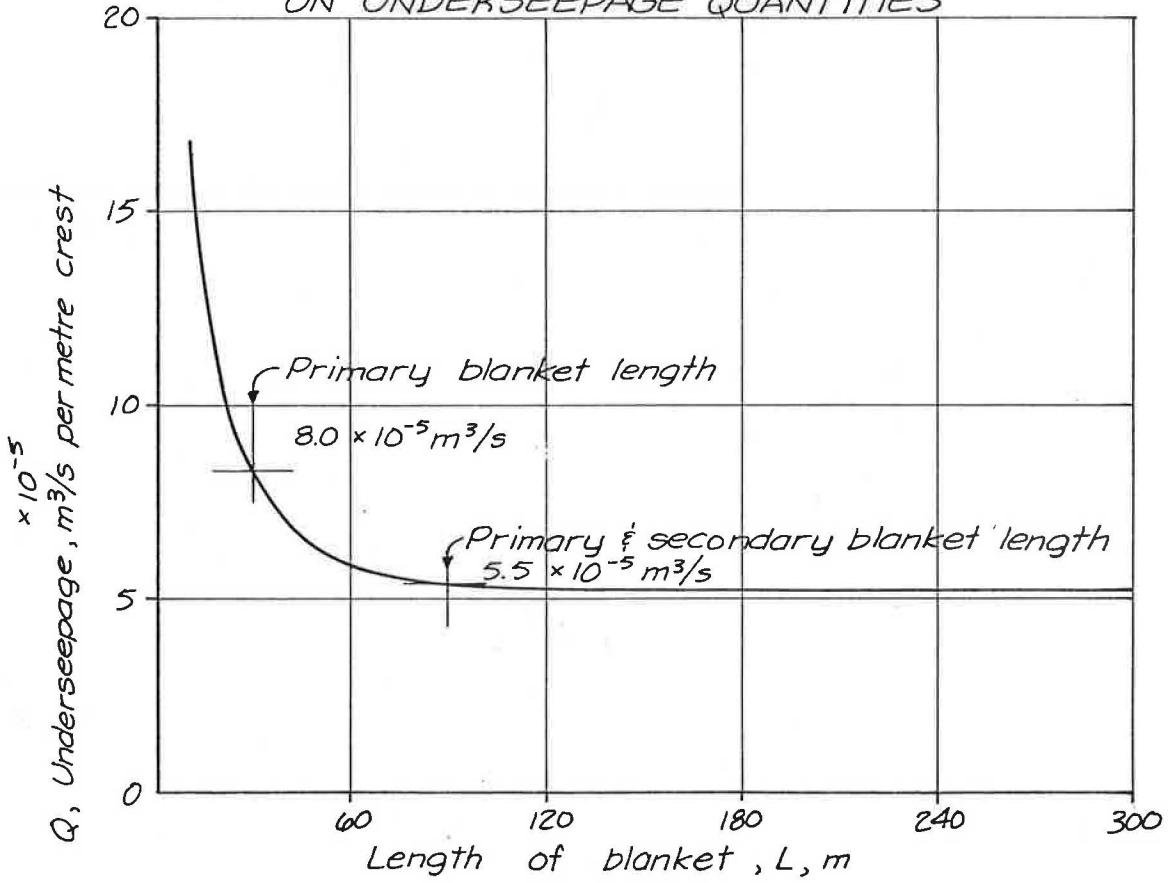
Project No. 792-2025 Drawn R.Q. Reviewed ____ Date 5-80

EFFICIENCY OF UPSTREAM IMPERVIOUS BLANKET Figure VI- 3



KF = Foundation permeability 10^{-5} m/s
KB = Blanket permeability 10^{-7} m/s
C = Core base width 7m
F = Depth of pervious foundation 20m
B = Blanket thickness 1m
H = Differential head 13.5m
Q = Underseepage, m^3/s

INFLUENCE OF INCREASING BLANKET LENGTH ON UNDERSEEPAGE QUANTITIES

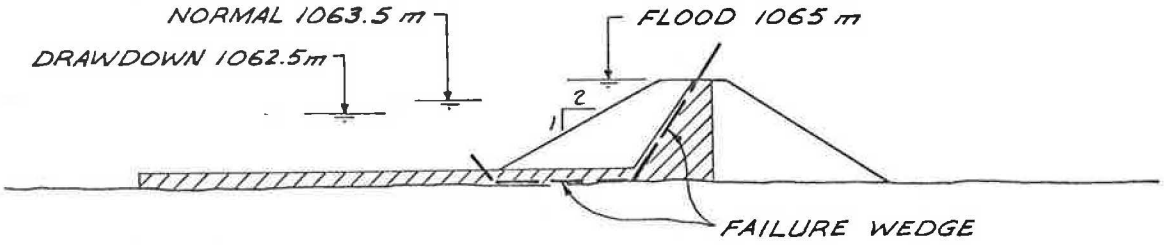


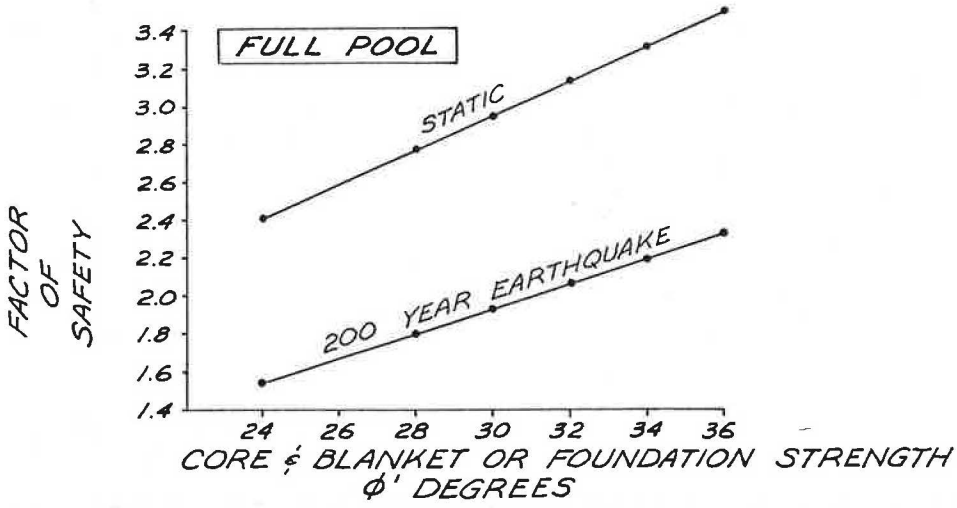
Computations based on P.T. Barnett's mathematical solution.

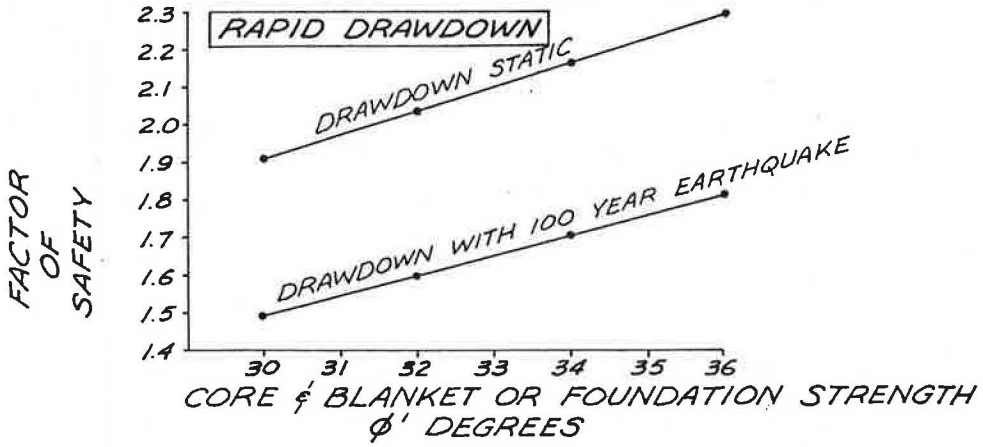
Project No. 792-2025 Drawn R.D. Reviewed ..... Date 5-80

STABILITY ANALYSES - UPSTREAM SLOPE

Figure VI-4







Notes:
(1) Janbu analyses used to assess stability.
(2) Earthquake loading simulated by simple pseudostatic method.
(3) Maximum drawdown of 2.5 m assuming no pore pressure dissipation from core or blanket.

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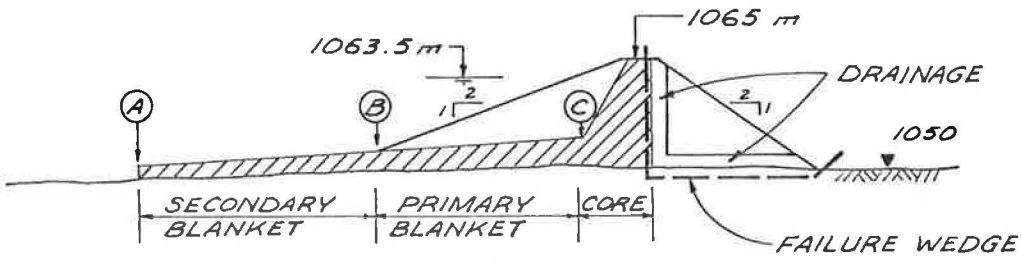
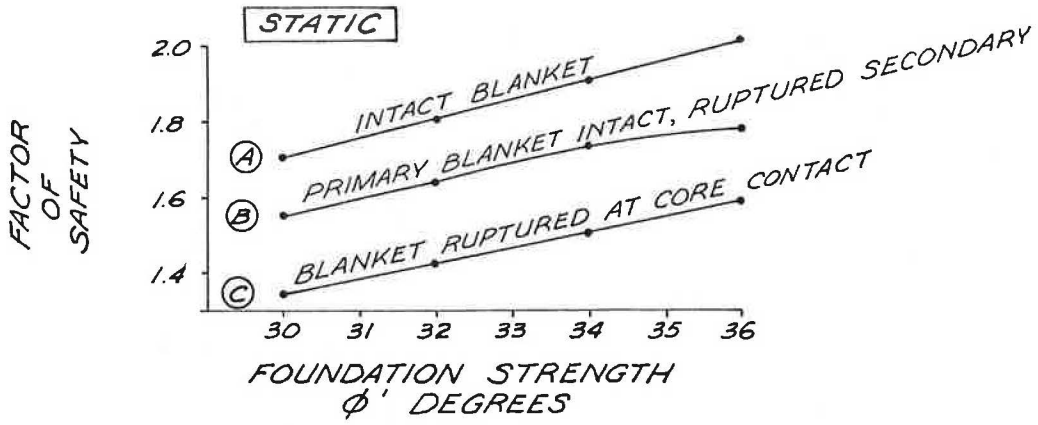
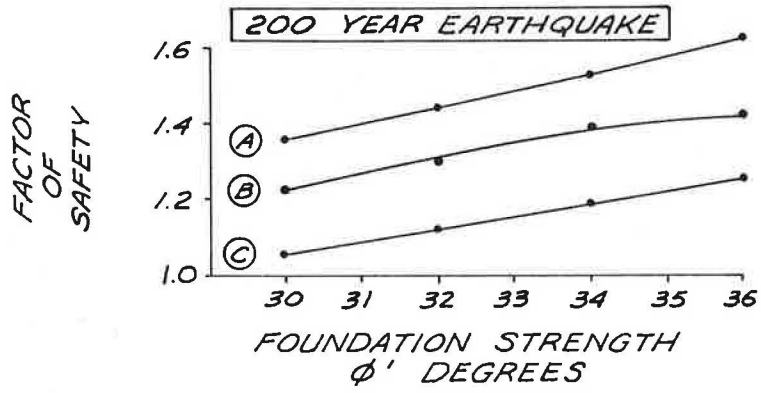
STABILITY ANALYSES - DOWNSTREAM SLOPE Figure VI - 5







Notes:
(1) Janbu analyses used to assess stability.
(2) Earthquake loading simulated by simple pseudodynamic method.
(3) Core strength chosen as C' = 0 Ø = 35°
(4) Core considered to be leaking.
(5) Impervious blanket considered effective downstream of points A, B, or C, eg. B assumes secondary blanket ruptured.

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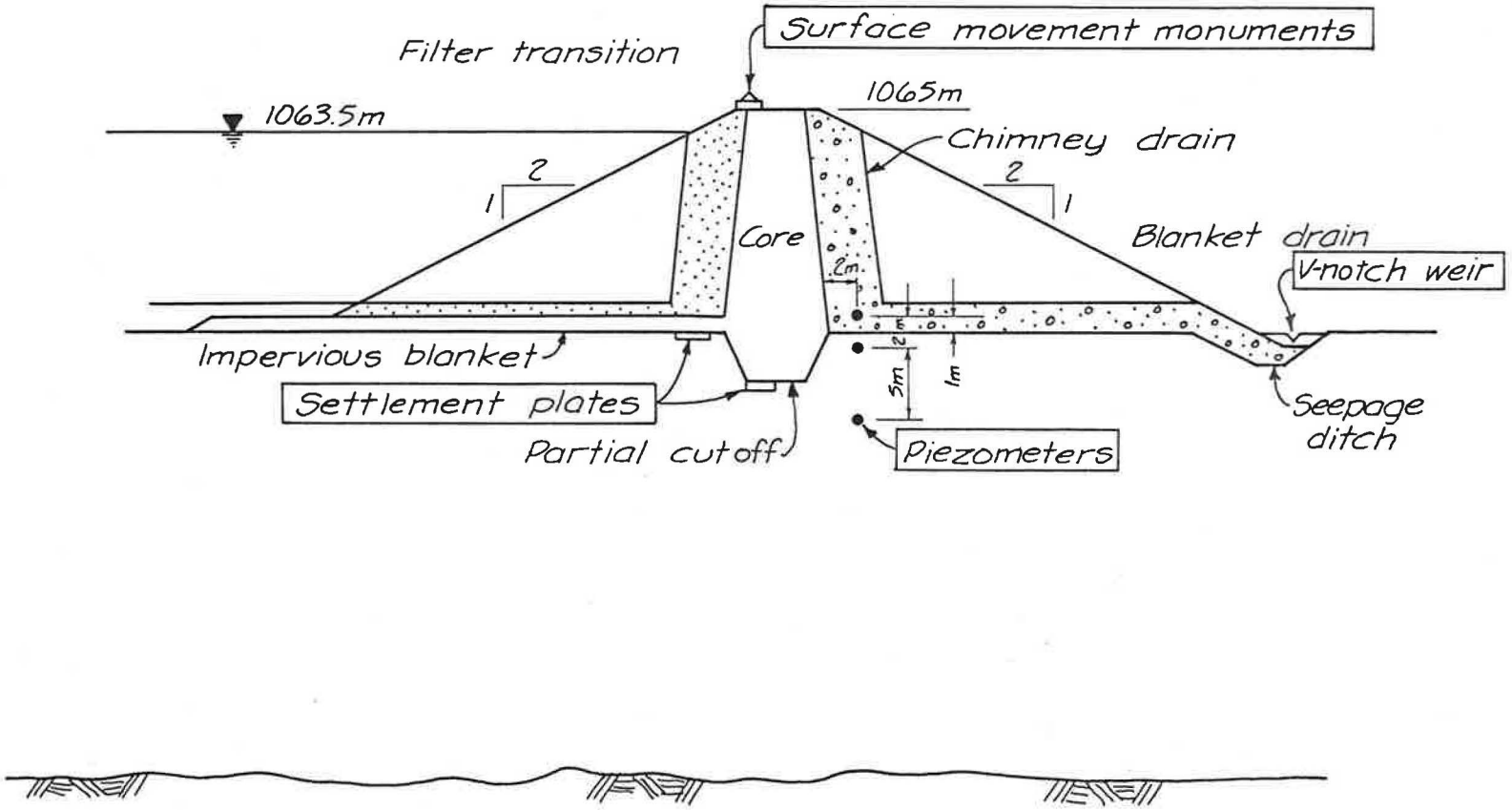


INSTRUMENTATION OF EMBANKMENT

Figure VI-6

Golder Associates

LABORATORY TRIAXIAL COMPRESSION RESULTS Figure VI-7

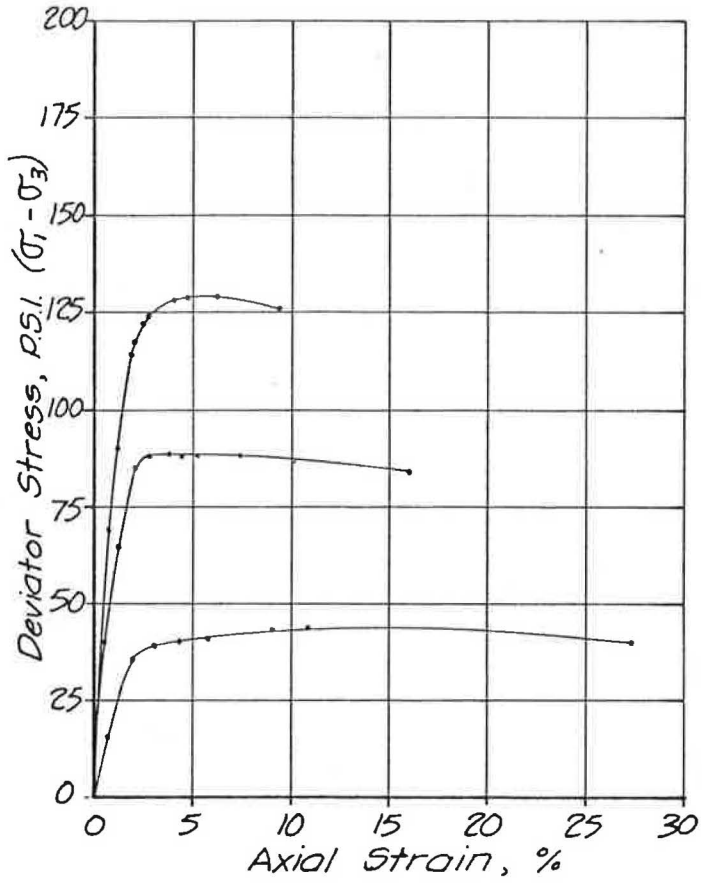
TEST PROCEDURE :
Consolidated Drained on 3 separate specimens

SAMPLE IDENTIFICATION:
Test Pits 1-Sa2 and 2-Sa2 combined

SOIL DESCRIPTION :

Glacial till with silty matrix

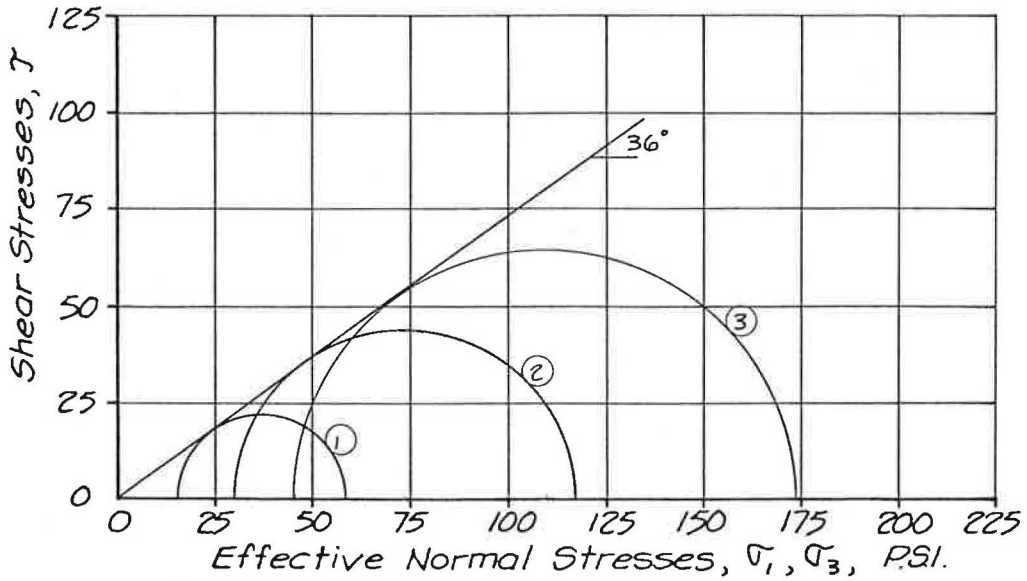
STRESS-STRAIN RECORDS



SPECIMEN DATA

Specimen #	1	2	3
Diameter, in.	2.01	2.01	2.01
Height, in.	4.05	4.20	4.15
Dry Unit Weight, P.C.F.	126.1	129.7	129.5
Water Content at Compaction, %	10.0	9.9	9.7
Cell Pressure, P.S.I.	154.9	149.9	155.1
Back Pressure, P.S.I.	139.4	120.2	110.1
Effective Consolidation Pressure, P.S.I.	15.5	29.8	45.0
Pore Pressure Parameter, B	0.95	0.96	0.96
Average Rate of Strain, %/Hr.	1.35	1.29	1.28
Max. Deviator Stress, P.S.I.	43.7	88.1	128.3
Axial Strain at Max. Deviator Stress, %	10.7	5.2	6.3
Max. Effective Principal Stress Ratio	3.82	3.96	3.85
Axial Strain at Max. Effective Principal Stress Ratio	10.7	5.2	6.3

MOHR'S CIRCLES OF STRESS AT SPECIMEN FAILURE



Project No. 792-2025 Drawn ..... Reviewed ..... Date 5-80

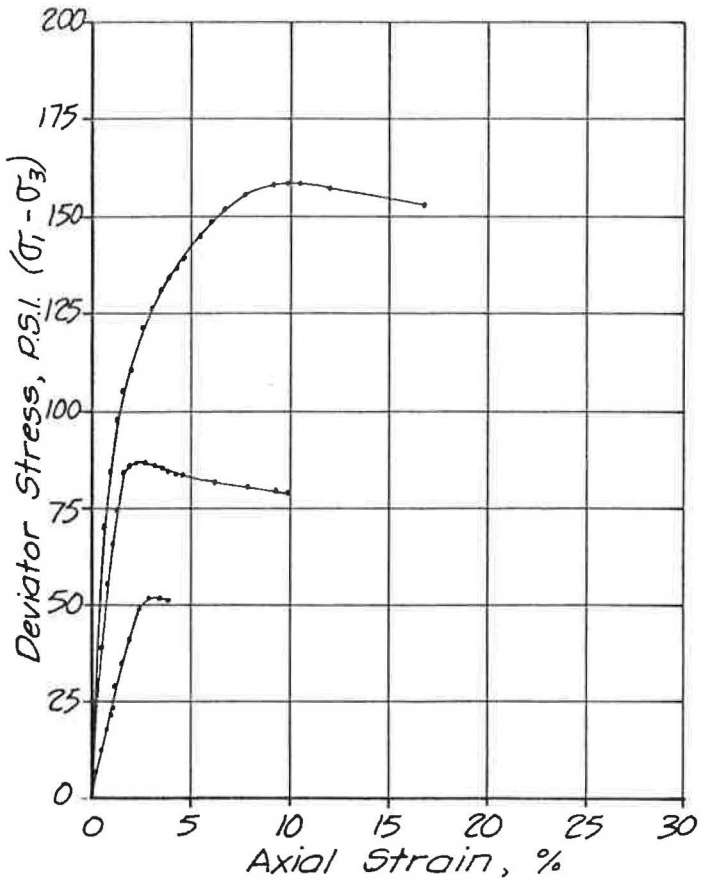
LABORATORY TRIAXIAL COMPRESSION RESULTS Figure VI-8

TEST PROCEDURE:
Consolidated Drained, two separate specimens, ① & ② multistage

SAMPLE IDENTIFICATION:
Borehole 24 Sa.3 and 5 combined

SOIL DESCRIPTION:
Glacial till with clayey matrix

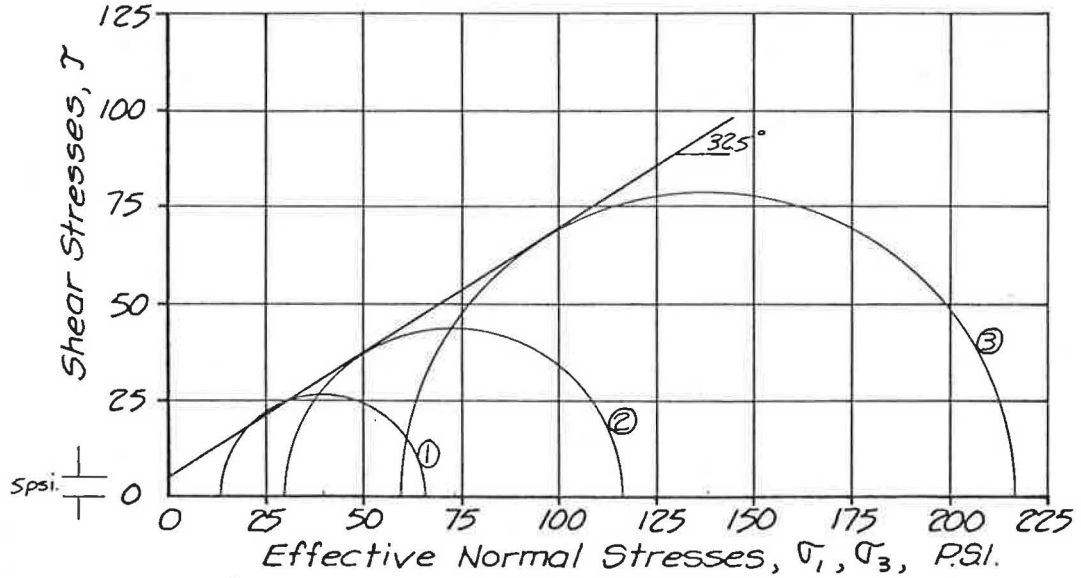
STRESS-STRAIN RECORDS



SPECIMEN DATA

Specimen	#1	#2	#3
Diameter, in.	2.01	2.04	2.00
Height, in.	4.20	4.00	4.07
Dry Unit Weight, P.C.F.	128.0	130.4	131.5
Water Content at Compaction, %	12.3	11.9	9.5
Cell Pressure, P.S.I.	114.1	129.9	180.4
Back Pressure, P.S.I.	100.6	100.6	121.2
Effective Consolidation Pressure, P.S.I.	13.5	29.4	59.2
Pore Pressure Parameter, B	0.95	-	0.96
Average Rate of Strain, %/Hr.	1.29	1.34	1.32
Max. Deviator Stress, P.S.I.	52.1	87.6	158.5
Axial Strain at Max. Deviator Stress, %	3.0	3.3	10.5
Max. Effective Principal Stress Ratio	4.86	3.98	3.68
Axial Strain at Max. Effective Principal Stress Ratio	3.0	3.3	10.5
	Multistage		

MOHR'S CIRCLES OF STRESS AT SPECIMEN FAILURE



Project No. 792-2025 Drawn _____ Reviewed _____ Date 5-80

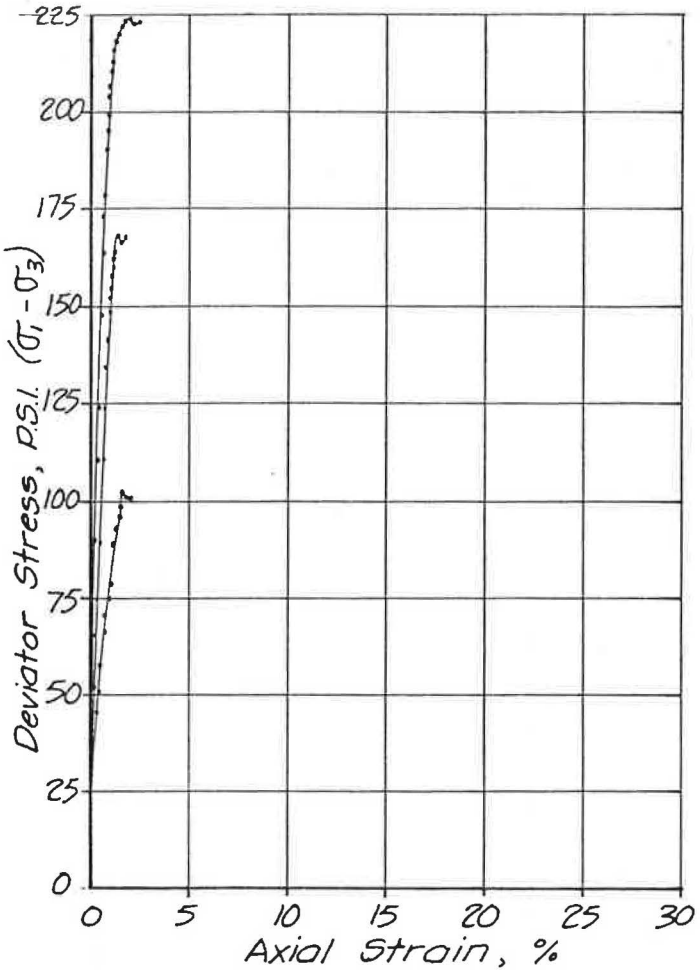
LABORATORY TRIAXIAL COMPRESSION RESULTS Figure VI-9

TEST PROCEDURE :
Consolidated drained, multistage on single specimen.

SAMPLE IDENTIFICATION:
Proposed shell borrow area.

SOIL DESCRIPTION :
Sand and gravel with a trace of silt (minus 1 inch fraction)

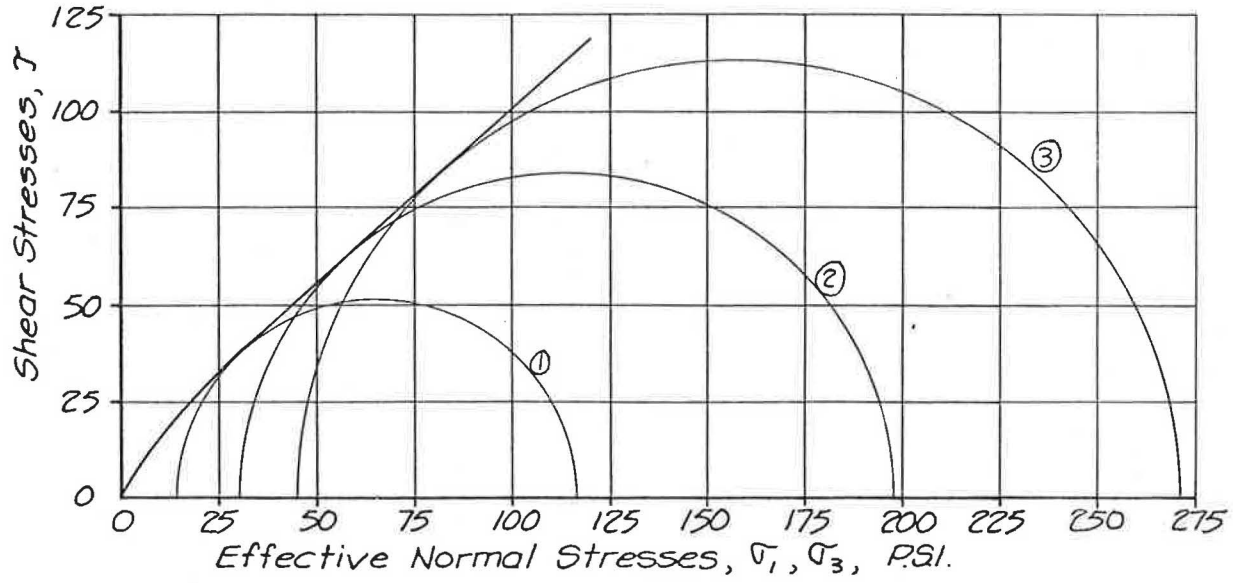
STRESS-STRAIN RECORDS



SPECIMEN DATA

Specimen #	1	2	3
Diameter, in.	6.00	6.07	6.11
Height, in.	12.4	12.2	12.0
Dry Unit Weight, P.C.F.	131.0	130.4	130.9
Water Content at Compaction, %	7.3		
Cell Pressure, P.S.I.	54.8	69.9	85.2
Back Pressure, P.S.I.	40.5	39.8	37.9
Effective Consolidation Pressure, P.S.I.	14.3	30.1	45.3
Pore Pressure Parameter, B			
Average Rate of Strain, %/Hr.	8.6	9.9	10.6
Max. Deviator Stress, P.S.I.	102.2	168.5	226.1
Axial Strain at Max. Deviator Stress, %	1.6	1.7	2.0
Max. Effective Principal Stress Ratio	8.14	6.6	5.99
Axial Strain at Max. Effective Principal Stress Ratio	1.6	1.7	2.0

MOHR'S CIRCLES OF STRESS AT SPECIMEN FAILURE



Project No. 792-2025 Drawn _____ Reviewed _____ Date 5-80

Borrow Pit Investigations

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APPENDIX VII

DISCUSSION OF BORROW PITS

For location of Borrow Pits see Figure VII-1.

Borrow Pit No. 1

This borrow possibility is located in the gravel terrace adjacent to Rose Creek near the West Dam of existing tailings facility. No boreholes or test pits have been done in this area but nearby excavation experience suggests that a good yield of gravel can be obtained. Estimated total volume of gravel available is 6×10^4 m^3 down to Elev. 1068 m. However, this volume may not be totally available depending on usage and water elevation in the nearby filtration pond.

Borrow Pit No. 2

This borrow possibility is located in the gravel terrace to the north of Borrow Pit No. 1. No test pits or boreholes have been done in this area. Experience with the nearby borrow pit for the existing West Dam indicates that good quality gravels and sands may be expected. Some volcanic ash has been observed at the surface and depths of stripping up to 1 m should be expected. The upper 1.5 m of gravel is usually coarser.

Estimated total volume of gravel available is 33×10^4 m^3 down to Elev. 1067 m.

Borrow Pit No. 3

This borrow pit is located in the gravel terrace to the west of Pit No. 2. One test pit has been excavated near the centre of this area and one in the northwest corner. Most of the area has no tree cover and was found to be unfrozen. However, as the northern boundary is approached some patches of permanently frozen gravel may be found. Gravel is also

present north of this borrow area but much of it is frozen (at least near surface). This area is tree covered and is overlain by frozen organic silts about 1.3 m in thickness.

The gravel is relatively coarse near the top of the profile and appears to become sandier with depth. Permafrost gravel in this area does not appear to contain any visible ice and should thaw quickly under summer conditions. Occasional perched water table conditions should be anticipated at the northwest corner of the pit.

About 0.4 m of waste material will need to be stripped from the area. Estimated total volume of gravel available is 49 x 10^4 m^3 down to Elev. 1059 m.

Borrow Pit No. 4

This area is just east of the Intermediate Dam and several boreholes and a test pit have been done in this area. The remarks above for Pit No. 3 apply to this pit as well.

Estimated total volume of gravel available is 32 x 10^4 m^3 down to Elev. 1056 m.

Borrow Pit No. 5

This borrow pit consists of a gravel ridge situated between the proposed Cross Valley and Intermediate Dams and falling adjacent to Rose Creek. Much of the area was stripped during the summer of 1979 revealing some pockets of frozen silty sand (difficult to rip) and organic silt. It is estimated that 70 percent of the materials in this area could be used as gravel fill for the dam. Also it is possible that, when thawed, some of the silty sand would be desirable as an upstream sand zone for the cross valley dam core.

The available amount of all materials in this pit is estimated at 30 x 10^4 m^3 to Elev. 1053 m assuming full utilization.

Borrow Pit No. 6

This pit is located on the north side of the valley between the Cross Valley and Intermediate Dams. Several old borrow pits have been opened in this area. The extremely complex stratigraphy consists of interlayered and mixed gravel alluvium, schist rich till and silty sand. About 0.6 m of stripping will be required in this area although locally greater depths of stripping may be required. It is considered that this area should not be categorized as a general fill source but rather as a source of specific materials such as filter sands.

Total available volume of material in the area is about 22×10^4 m^3 to Elev. 1053 m.

Borrow Pit No. 7

This pit area is located just downstream of the north abutment of the Cross Valley Dam. It is intended that this area not be used in the Stage I construction because it will be one of the few sources of gravel which will be readily accessible after the terrace area within the new tailings impoundment area is flooded. Furthermore, the Cross Valley Dam Spillway will traverse this area.

Stratigraphically this area is considered similar to that at Borrow Pit No. 6 although a greater gravel potential exists here as indicated by an abandoned borrow pit. Nevertheless a schist rich till wedge traverses the pit.

Estimated total available volume is 8×10^4 m^3 to Elev. 1050 m and about 65 percent of this should be good gravel fill.

Borrow Pit No. 8

This pit area is located on the south valley wall gravel terrace adjacent to the alluvial fan in the outfall section of the diversion canal. Again it is intended that this area not be depleted during Stage I construction because it will be readily accessible for post Stage I construction.

This generally treeless area is underlain by discontinuously frozen gravels which thaw readily under summer conditions. Again the gravels appear to become sandier with depth as indicated by test pits within the area. On approaching the southwest boundary of the deposit, tree growth becomes thicker and frozen organic silts up to 2 m thick overlie the gravels which sometimes contain ice and are difficult to excavate.

About 0.5 m of stripping would be required to develop this pit which would yield about 10×10^4 m^3.

Borrow Pits No. 9

These borrow pits are not shown on the mapping of the area. These borrow pits are up to 1200 m downstream of the Cross Valley Dam and may never be used if the Cross Valley Dam is not raised above Elev. 1063 m. Borrow pit No. 9B is about 900 m downstream from the Cross Valley Dam but it is not readily accessible in summer due to soft ground. Borrow pit 9A is about 400 m further downstream, but on the north side of Rose Creek, and Borrow Pit 9 is situated adjacent to a north valley wall tributary which enters the Rose Creek Valley about 1200 m downstream of the Cross Valley Dam.

A test pit was excavated in the Pit 9B area revealing discontinuously frozen gravels overlain by about 0.4 m of organic silt and volcanic ash which would need to be stripped. The available volumes are estimated as follows:

Borrow Pit 9: 57×10^4 m^3
Borrow Pit 9A: 5.5×10^4 m^3
Borrow Pit 9B: 13.6×10^4 m^3

Additional gravel deposits appear to be available further downstream.

Borrow Pit No. 10

This borrow area is located on the north wall in the valley in the designated core borrow area and it has been investigated using test pits and boreholes. The depth to bedrock is about 4 m. The north and eastern

pit often contain schist rich till inclusions (see photo sheet 4 of Appendix IV) which should not be used in the dam core but should be acceptable in certain portions of the dam section (i.e. Shell Type I). It is estimated that about 26 x 10^4 m^3 of material are available in this area of which 60 percent would be usable as core. Some oversize (+150mm) rejects occur in the till. Between 0.4 and 0.5 m of stripping should be expected.

No evidence of permanently frozen ground was encountered in this area. The till is generally compact to dense and ripping was used for test pits. Except for a moist till layer near the top of the deposit, the till is far below its optimum water content and will need to be wetted to obtain desired core material performance.

Borrow Pit No. 11

This area just to the east of Borrow Pit No. 10 has been investigated using test pits and boreholes. The southern half of the area is ridden with schist rich till deposits and shallow bedrock (as shallow as 2.5 m). The northern half is much more promising and it is expected that a good till borrow pit could be opened there.

The total available volume in this area is about 9 x 10^4 m^3 of which about 30 percent is expected to be useable as core borrow.

Borrow Pit No. 12

This area is located to the east of Borrow Pit No. 11 with only the southern half (south of the creek) having been investigated using boreholes and test pits. Bedrock is relatively shallow in this area (about 3 m) and a stripping depth of 0.4 m should be expected. Isolated occurrences of permafrost were noted in this area.

The total available volume in this area is about 9 x 10^4, of which about 60 percent is expected to be usable as core material.

Borrow Pit No. 13

Test Pit Nos. 80-24 and 80-25 have been excavated in this area and indicate good core potential although some till appears to be partially frozen and stratigraphic variations occur. The potential yield of till material in this pit is up to 55 x 10^4 m^3.

Borrow Pit No. 14

This borrow pit is located close to the intermediate dam and it appears to hold promise of yielding substantial core material of good quality. Test Pit Nos. 79-19 and 80-23 indicated good quality till but some discontinuous permafrost is present. It is expected that exposure of this till would introduce rapid thawing because it is not ice rich. About 0.4 m of stripping would be required.

The potential volume available is about 30 x 10^4 m^3 and a yield of 75 percent is suggested.

Early spring operation of the above borrow pits means that the near surface materials will be frozen to the depth of seasonal frost. Frozen till materials will generally be unsuitable for use in the dams unless allowed to thaw. Such a consideration must be applied to the estimated yields above.

A borrow pit summary is shown on Table VII-1.

TABLE VII - 1
Borrow Pit Summary

Borrow Pit No.	Material	Est'd Volume m^3
1	Gravel	6.2×10^4
2	Gravel	33×10^4
3	Gravel	49×10^4
4	Gravel	32×10^4
5	Gravel	30×10^4
6	Gravel & Broken Rock	22×10^4
7	Gravel & Broken Rock	8×10^4
8	Gravel	9 to 15×10^4
9	Gravel	76×10^4
10	Till	26×10^4
11	Till	12.5×10^4
12	Till	9.3×10^4
13	Till	55×10^4
14	Till	30×10^4

LOCATION OF BORROW PITS AND PROJECT ELEMENTS

FIGURE VII-1

Note: This drawing to be read in conjunction with Appendix VII

Drawn R.W.
Rev'd
Date May, 1980

Diversion Canal Geothermal, Seepage and Stability Considerations

Golder Associates

APPENDIX VIII
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Figure VIII-4 Channel Backslope First Thaw Season Factor of Safety vs. Slope Gradient (depth of thaw 0.9 m)

Figure VIII-5 Channel Backslope First Thaw Season Factor of Safety vs. Slope Gradient (depth of thaw 1.2 m)

Figure VIII-6 Channel Backslope First Thaw Season Factor of Safety vs. Thaw Depth Beneath Base of Gravel

Figure VIII-7 Factor of Safety vs. Slope Gradient for Subaquaceous Slope at Toe of Excavation Spoil

Figure VIII-8 Integration of Accelerograms to Determine Downslope Displacements

Figure VIII-9 Computation of Embankment Displacments Due To Earthquake Indunced Ground Motions

1.0 INTRODUCTION

This report deals with the geothermal and geotechnical aspects of the proposed Rose Creek diversion canal to be constructed on the south wall of the Rose Creek Valley adjacent to the proposed new Cyprus Anvil tailings and water retention facilities. The location and approximate alignment of the proposed canal are discussed in the main body of the report (see Figure 7-1). Specifically, this report deals with that portion of the canal between the downstream end of the existing upstream diversion and the beginning of the outfall section at the downstream end of the proposed canal.

2.0 GEOTHERMAL AND THAW CONSOLIDATION CONSIDERATIONS

2.1 General Comments

Excess pore water pressures induced by thaw consolidation processes which take place when permafrost thaws have obvious implications for slope stability considerations. In order to predict the consequences of permafrost degradation in terms of the excess pore water pressures, the rate of degradation must be defined.

In terms of the proposed Rose Creek diversion canal, the most important concern involving slope stability and permafrost degradation are the thaw slope stability in the cut back slope of the canal, the failure of the waste piles being considered a secondary concern in terms of the performance of the canal. These are particularly critical within the region where the canal is adjacent to the proposed water retention portion of the development because rapid thawing of permafrost at the toe of the construction could take place due to free water storage.

The geothermal response of permafrost soils is sensitive to many environmental factors. As well as ground and ambient air temperatures and soil thermal properties and conditions, these factors include snow cover, ice (water) content, peat or other insulating cover, disturbance of the natural ground cover, terrain aspect, and surface drainage, to name a few of the most important factors.

Experience has shown that in permafrost areas, where clearing is done in a manner that avoids disturbance of the insulating organic cover and ground surface disturbance is minimal, little if any degradation of permafrost will take place. Such clearing is most easily achieved during the winter season when the insulating cover is least fragile. In cases where there was no attempt to preserve the insulating moss cover during clearing, high rates of degradation inevitably occurred during the ensuing thaw season and this produced slope failures, even on very shallow slopes. Consequently, clearing and/or excavation for channel construction must be undertaken during the winter season, to permit placement of adequate amounts of insulating material on the exposed permafrost before the following thaw season. If it is necessary to clear areas beyond the perimeter of the channel construction the least rate of permafrost degradation will occur if such areas are cleared by hand, or alternatively, if these areas are also covered with an adequate insulating layer of gravel or other suitable material.

2.2 Geothermal Considerations

A one-dimensional geothermal analysis has been carried out for the diversion canal section using the mean daily air temperature data for the Cyprus Anvil site (Table 1) together with pertinent soil thermal properties selected from Jumikis (1977). This analysis was applied to the soils observed to exist at the site as well as to borrow materials anticipated for use in canal construction. On the basis of the soil parameters and canal geometry selected, thaw in the vicinity of the proposed channel section can be expected to reach a depth of approximately 2.9 m (9.5 ft.) below the gravel surface of the constructed channel during the first thaw season. Total freezeback during the following winter season can be expected where water is not in contact with the ground surface. Due to reductions in water contents caused by consolidation during seasonal thawing, subsequent annual thaw depths will be slightly greater, however, induced excess pore water pressures will reach their worst condition, with respect to stability, during the first thaw season.

Where water is in contact with the ground surface on a continuous, or near continuous basis, more rapid thaw will occur. This will be the case in the water storage area between the cross valley and intermediate dams, as well as for a portion of the bottom of the diversion canal.

2.3 Theory of Stability for Thawing Slopes

Because of the likelihood of substantial thaw, it has been deemed necessary to undertake stability assessments involving thaw consolidation considerations. Details of these assessments are discussed in Section 3 of this report. These assessments are based on the argument that the factor of safety against sliding for a two layered infinite slope in thawing permafrost has been shown by Nixon (1973) to be:

$$F_s = \frac{P_o + \gamma' d}{P_o + \gamma d} \left[\frac{1 + k_2 H / k_1 d}{1 + 2R^2 + k_2 H / k_1 d}\right] \frac{\tan \phi'}{\tan \theta}$$

where:- P_o = the overburden pressure applied by soils above the ground water table

γ & γ' = the total and effective unit weights, respectively, of thawing soil

k_1 & k_2 = the thermal conductivities of the upper and lower layers, respectively, of the two layer system

H = thickness of the upper layer measured normal to the face of the slope

d = depth of thaw in the second layer, measured normal to the face of the slope (i.e. total depth of thaw normal to the face of the slope is H + d)

ϕ' = the effective angle of internal friction of the lower layer of thawing soil

θ = the overall slope angle

R = the thaw consolidation ratio.

All of these variables can be adequately defined in terms of classical slope stability usage except for R, which describes the relative influence on excess pore water pressure of the rate at which water is produced by thawing, and the rate at which it can be expelled from the thawed soil overlying the thaw front.

The value of R has been shown by Morgenstern and Nixon (1971), in a one-dimensional analysis, to be:-

$$R = \frac{d}{2\sqrt{c_v t}}$$

where:- d = the depth of thaw normal to the slope surface

t = time

c_v = coefficient of consolidation of the thawing soil.
(The component $d/\sqrt{t}$ in this equation is normally a constant for a given soil/thaw condition)

The coefficient of consolidation and the density related soil stress also define the excess pore water pressure at the thaw front and in a two layered system the relationship was shown by Morgenstern and Nixon (1971) to be:-

$$u = \frac{P_o + \gamma' d}{1 + \frac{1}{2R^2}\left(1 + \frac{k_2' H}{k_1 d}\right)}$$

where:- $P_o + \gamma' d$ = the effective stress at the thaw front at depth H+d after complete dissipation of excess pore pressures.

As stated previously, the evaluation of excess pore water pressures is central to the ability to assess the stability of thawing permafrost slopes. This requires the evaluation of the thaw consolidation ratio R. In these considerations, the variables d and t reflect the geothermal analysis of the climatic and soil conditions of the site (see Section 2.2).

Although there are a number of laboratory, field and analytical techniques available for use in the evaluation of c_v, it is notoriously difficult to evaluate to accuracies within even one order of magnitude. The techniques available have been discussed by McRoberts, Fletcher and Nixon (1978). Based on this work, as well as the results of a number of case studies carried out in the MacKenzie Valley, a range of values for c_v of between 2.5×10^{-3} and 2.5×10^{-2} cm^2/sec has been chosen for use in the evaluation of the thaw consolidation ratio, R, for the materials through which the proposed diversion canal will be constructed.

2.4 Summary

When the above geothermal/geotechnical considerations are applied in the analysis of a thawing permafrost slope to assess its stability over a period of years it can be shown (McRoberts et al (1978)) that the maximum normalized pore pressure, and hence the lowest factor of safety, will occur during the first thaw season and that stability will tend to improve with time. Keeping in mind that conditions may also improve with depth of degradation, due to the presence of denser soils and the influence of two-dimenstional effects in enhancing stability, the most critical conditions can be seen to occur during the initial stages of thaw. The worst condition is believed closely approximated using the method proposed by Morgenstern and Nixon (1971), provided an average value for the ratio of $d/\sqrt{t}$ can be predicted for the first thaw season.

3.0 STABILITY ANALYSES

3.1 Backslope of Diversion Canal

Analyses using the criteria outlined in Section 2 have been used to assess the stability of the backslope portion of the diversion canal. These analyses were performed for materials with in situ shear strengths represented by effective angles of internal friction of 30, 35 and 40 degrees. This range of strengths was chosen on the basis of the laboratory triaxial tests carried out on materials from the cross valley

dam abutment area and that are discussed in that section of this report, and from considerations of the in situ moisture (ice) contents and grain size distributions of the materials which have been observed along the proposed channel alignment.

For these analyses it has been assumed that the gravel placed on the channel backslope is free draining and that the phreatic surface will be at the boundary between the gravel and the underlying native materials.

Figures VIII-1 to VIII-5, inclusive, show the dependence of factor of safety on backslope gradient for in situ materials with varying shear strengths, on coefficients of consolidation for a range of gravel insulation thicknesses and on depths of thaw beneath the gravel insulation. The width of gravel corresponding to the thickness required in each case is also shown on these figures.

Figure VIII-1 is representative of both the case when thaw does not penetrate beneath the base of the gravel at any time and the case in which the thaw consolidation process has reached equilibrium and all excess pore water pressures have been completely dissipated. This figure, in other words, is representative of the long term stability of the channel backslope irrespective of the amount of gravel insulation (assuming that the slope remains intact until the equilibrium condition is achieved). Figures VIII-2 to VIII-5 introduce the additional variable of thaw depth, coefficient of consolidation and slope angle.

Figure VIII-6 is a composite of the 2 horizontal to 1 vertical portions of all of the above analyses. It permits determination of the conclusion that, where fine grained, weaker soils are encountered (e.g. $\emptyset' = 30$ degrees and $c_v = 2.5 \times 10^{-3}$ cm^2/sec) the width, B, of gravel insulation required to achieve a factor of safety of approximately 1.05 would be approximately 5.2 m (17.1 ft.). Where stronger and more free

draining materials are encountered along the backslope excavation (e.g. $\emptyset'$ = 37 degrees, and c_v = 2.5 x 10^{-2} cm^2/sec) then the minimum practicable width, B, of gravel of 3.66 m (12.0 ft.) would result in a factor of safety of approximately 1.20 against slope stability failure due to thawing. (The d = 0 axis on this figure is also representative of the long term factor of safety for thawed consolidated slopes with gradients of 2 horizontal to 1 vertical under the conditions of seepage forces related to the high water table shown.

The curves on Figure VIII-6 for materials with c_v = 2.5 x 10^{-4} cm^2/sec are shown to demonstrate the sensitivity of the analysis to c_v when its value becomes smaller than approximately 2.5 x 10^{-3} cm^2/sec. None of the materials observed at the site are believed to have values of c_v of any less than the chosen lower bound of 2.5 x $10^{-3}cm^2$/sec.

For the conservative range of strength and coefficient of consolidation parameters which have been chosen for the analyses, it is considered that minimum factors of safety of 1.05 are reasonable considering the transient stability conditions being addressed and because the long term factor of safety against instability as shown on the d = 0 axis of Figure VIII-6 is above an acceptable minimum as represented in Figure VIII-1 and in Figure VIII-6 along the d = 0 axis.

3.2 Toe Region of the Overall Channel Section

The toe region of the overall channel section between the cross valley and intermediate dams will ultimately be below the surface level of the water stored in this area. Consequently, it is prudent to assume that rapid thaw will occur with resulting high pore water pressures at the thaw front in the native soils.

A one-dimensional analysis has been carried out, assuming a continuous temperature of +4° C at the ground surface in this area and indicates that the $d/\sqrt{t}$ ratio is approximately 0.06 $cm/sec^{1/2}$.

Figure VIII-7 is a graphical representation of the factor of safety versus slope gradient which highlights thaw consolidation considerations for the region downslope of the toe of the channel excavation spoil pile. It is noted that the ground surface at the toe of the soil in the region between the two proposed dams varies between approximately 10 and 12 degrees with some localized slopes up to 16 degrees in steepness.

The results of the analysis, shown on Figure VIII-7, indicate that there is little risk of local underwater slope failures along the downslope toe of the canal construction due to rapid thawing of permafrost.

If the region downslope of the toe of the overall channel construction within the proposed tailing retention area is not disturbed by construction activities, or alternatively, is properly treated after any such disturbance (Section 2.1), thaw slope stability failures in this area should not occur.

3.3 Overall Channel Stability

As addressed above, the critical areas with respect to channel stability are considered to be the back slope section (Section 3.1) and the submerged toe region of the valley wall downslope from the channel. Overall stability of the channel section will be greater than for either of the above cases with one possible exception. The exception may occur if channel construction spoil materials are placed over actively thawing ground or ground in which a talic, or unfrozen sandwiched zone related to the previous thaw season remains below the ground surface. If this situation occurs, and the slope gradient is sufficiently steep, the normal loads applied to the thawing surface could induce excess soil porewater pressures sufficient to initiate slope movement.

Once initiated, these movements would continue until static equilibrium is achieved under the thawing condition of the first summer erosion, and the consequence would be distortion or destruction of some portions of the channel spoil pile. Choice of winter construction is therefore adviseable because post-sliding mitigative measures are not attractive. They would include either the reduction of loads applied to the thawed or thawing soil, or alternatively, the application of sufficient loads at the toe of the fill slope to stop the movements. The first of these may infringe on the design requirements of the proposed channel section and the second, particularly where original ground surface gradients are steep, could involve buttressing fill founded on the floor of the valley.

If spoil pile foundation failures occur because of high pore water pressures in the talic, (unfrozen sandwich zone) they will take place during construction and will not extend to depths greater than that of the talic, nor to distances further upslope than the limit of the fills which induce these failures.

The most desirable solution is to avoid the problem by placing these fills when the ground is frozen. This requirement includes planning contractor activities to minimize the likelihood that residual talic from the previous thaw season remains beneath the ground surface at the onset of local spoil pile building.

3.4 Spoil Section Failure

In addition to the slight possibility of foundation-related stability failures at the toe region of the excavation spoil section (Section 3.2) and of overall creep movements occurring in the soils beneath it (Section 3.3), the nature of the spoil materials and the manner in which they are to be placed is expected to result in some subsidence and perceptible downslope movements within the spoil mass itself, as well as shallow slope failures on its surface. Aside from shallow surficial movements,

these events, if they occur at all, can be expected to take place in late summer and early fall. One or any combination of these phenomena may cause some minor distress to the engineered fill portion of the channel construction. Although these are not conditions which can be economically or practicably prevented, they are conditions which, if they occur at all, will decrease in severity with time and, over a period of several years, will stop altogether. It is suggested, therefore, that they be dealt with as a maintenance item and that careful record be kept of the nature, location and frequency of remedial activity.

3.5 Solifluction of Materials Upslope of Channel Construction

It is believed that the surficial layer of moss and earth materials on the slope in which the proposed channel is to be constructed are experiencing slow downslope movements, particularly in the steeper slope sections and where the permafrost has relatively higher ice content. The indications are, however, that if such movements are occurring, they are occurring at rates perhaps not exceeding a few centimeters per year. One area of solifluction movement sufficient to cause tilting of the tree cover is located at Chainage 2+000 (see Figure 7-1). These areas therefore present a need for careful post-construction scrutiny to determine if there is a need to install cross slope sand drains to reduce the near-surface ice content through year-by-year drainage of the active layer.

3.6 Deep Seated Creep

There is no evidence of deep seated creep having occurred on the south valley wall of the Rose Creek Valley in the past. The permafrost is, according to the water content determinations, not ice rich, nor is there conclusive evidence of continuous ice lensing in the slope. Consequently, the excavations proposed for the channel construction are not expected to initiate creep movements in the materials upslope from the canal.

3.7 Seismic Considerations

3.7.1 Introduction

A seismic risk analysis for the Cyprus Anvil site was provided by the Pacific Geoscience Center. A copy of this document is included in Appendix X of the main report. The risk analysis suggests that a k = 5.4 and 9.7 per cent gravity be used respectively for the 100 and 200 year return periods.

3.7.2 Pseudo-Static Stability Analyses

The application of pseudo-static techniques to the analyses discussed in the previous sections can be summarized as follows:-

$$[F_S \text{ (pseudo-static)}] = \frac{\tan \theta}{\tan [\tan^{-1}(k)+\theta]} [F_S \text{ (static)}]$$

For example, in the case of a slope with a gradient of 2 horizontal to 1 vertical, the factors of safety calculated using the pseudo-static approach are:-

For 100-Year return period (k = 0.054 g)
(F_S pseudo-static) = 0.88 x (F_S static)

For 200-Year return period (k = 0.097 g)
(F_S pseudo-static) = 0.80 x (F_S static)

The assumptions are made in the pseudo-static analysis that the applied earthquake force is continuous and not cyclic and that it is acting in the plane of the section being analyzed. These are severe assumptions when assumed simultaneously additive, and the factors of safety so calculated are considered to be unrealistically low.

The pseudo-static analysis does not, however, recognize or differentiate between the two potential effects caused by the imposition of earthquake forces on natural or man-made slopes. These are displacements

as a result of ground accelerations and liquefaction of the slope or its foundation soils as a consequence of ground shaking. These phenomena are dealt with in the following sections.

3.7.3 Displacements as a Result of Ground Accelerations

In a slope of relatively uniform density subjected to a reasonably uniform acceleration throughout its height, experimental evidence (Goodman and Seed (1966) and Makdisi and Seed (1978)) suggests that failure occurs by mass sliding of a thin surface zone. During acceleration of the earthquake, inertia forces may induce momentary instability of this mass causing it to slide downhill. However, before any appreciable movements occur, the direction of ground acceleration is reversed and the motion of the surface layer is arrested, only to be started again in the following acceleration cycle. The overall effect is a progressive displacement causing flattening of the slope at its toe and settlement at its crest. If the initial inclination of the slope is less than the friction angle, movement can only occur when the inertia forces induce temporary instability, and no further displacement will occur when the ground motion has ceased. Techniques to calculate earthquake induced displacements have been developed by Goodman and Seed (1966) and Makdisi and Seed (1978). As explained below, neither of these techniques relies on the pseudo-static method of anlaysis to arrive at estimates of final displacements.

a) Goodman and Seed (1966) assumes that the friction angle and shear strength intercept decrease with shear deformation and that each successive acceleration pulse produces a corresponding displacement of the slope. Figure VIII-8 illustrates this phenomenon. Numerical integration can be used to calculate displacement from an expression for the velocity or, alternatively, there is a simplified expression for displacement.

b) Makdisi and Seed (1978) utilize three steps in the calculation of displacement:

i) First, static analyses are used to determine the horizontal or "yield" acceleration $k_y(g)$ required to because failure of the slope.

ii) Second, the earthquake induced accelerations, k_{max}, for the slope can be computed by using a simplified procedure described by Makdisi and Seed (1978), or by resorting to statistical data such as that provided by the Pacific Geoscience Center, or use of appropriate computer programs.

iii) Thirdly, the quantities determined from Steps one and two are used to determine displacement through application of design charts which have been drawn up using numerical integration.

To apply these techniques to all of the static analyses that have been done would involve extensive analysis. However, application of the latter techniques to the worst probable case of a high channel backslope, low materials shear strength ($\emptyset'$ = 30 degrees) and a low coefficient of consolidation (c_v = 2.5 x 10^{-3} cm^2/sec) for a 1 in 200 year earthquake event occurring at the time of maximum thaw depth during the first thaw season after construction, predicts about 0.2 m (0.62 ft.) of downslope movements of the gravel insulation and the underlying thawed soil. After the completion of thaw consolidation and the dissipation of excess pore water pressures, a similar earthquake event is predicted to produce a downslope movement in the gravel and weaker soils of less than a centimeter. The generation of these solutions is shown on Figure VIII-9 for illustration.

The subaqueous soils downslope from the toe of the channel construction in the water storage area may experience skin flow failures during an earthquake event, in isolated areas where shear strengths are low and where pore pressures due to thaw consolidation are high. Again, once geothermal equilibrium is achieved and thaw induced pore water

pressures have been dissipated, earthquake induced stability failures should not occur.

Briefly then, if a severe summer time earthquake (i.e.k = 0.10^{+}g) occurs during the first several years of canal operation, some isolated sections of the construction (i.e. those sections with low $\emptyset'$ and c_v) may experience damage. As thawing pore pressures dissipate with time, stability of the construction will increase and the probability of serious damage to the canal due to earthquake induced motions will decrease to the point where only slight damage would occur in areas with relatively weak foundation soils, even during a severe earthquake.

3.7.4 Liquefaction

In cases where the soil is cohesionless, the second effect of the imposition of earthquake forces mass is the possible liquefaction of the material. For the case being considered, the insulating gravel is essentially free draining and well graded and, therefore, not susceptible to liquefaction because it will be fully drained. The only effect on the canal construction, exclusive of its foundation soils, of seismic loading will be lateral displacement as discussed in the previous section. However, the soils underlying some portions of the canal construction may be saturated and loose when they are thawed, and consequently, minor susceptibility to liquefaction may be present. The glacial till at the site is considered to be resistant to liquefaction during ground shaking. In any areas where loose saturated silts or sands are present, liquefaction may occur during an earthquake event. Fortunately, field investigation has shown that these materials are not present to any significant extent beneath the channel except, possibly, under isolated sections of the spoil excavation portion of the construction. Because of the low probability of liquefaction failures involving these piles it is recommended that such failures should be dealt with on a maintenance basis if they occur.

4.0 SEEPAGE CONSIDERATIONS

Seepage losses from the proposed diversion channel have been calculated for three flow conditions. These include:

(1) the fall/winter low flow situation (10 cfs or 0.28 m^3/s),
(2) spring runoff flow (100 cfs or 2.8 m^3/s),
(3) and the 5 year flood (1320 cfs or 37 m^3/s).

Seepage analyses have been performed with consideration of the pertinent flow widths and depths corresponding to these flow stages. The channel has been considered to be perched above the existing groundwater table in the valley wall and that the seepage flow consists of vertical underdrainage to this existing groundwater regime. Seepage losses in the outfall portion of the diversion channel were not considered.

Cyprus Anvil has indicated that seepage losses from the diversion channel should not exceed 20 percent of the flow to a maximum of 0.15 m^3/sec. (2000 gpm). In order to satisfy this criterion, the maximum permeabilities of the soil over which an unlined channel may be constructed have been calculated to be 4 x 10^{-4}cm/s for wintertime low flow condition, 7.7 x 10^{-4}cm/s for the spring runoff flow, and 2.65 x 10^{-4}cm/s for the 5 year flood. Since the soils found along the diversion channel alignment are quite variable and because there is a potential deviation between laboratory testing and field performance, it is considered prudent to recommend that the canal be lined in areas where the soil exhibits permeability of greater than 10^{-4}cm/s.

It is presently anticipated that it will be necessary to line up to 1000 m of the channel length. This includes areas between Sta. 0+000 and 0+700 and where excavation in bedrock is required (i.e. near Sta. 1+500

and 2+300). Additional areas where a till blanket is applied as a seepage control measure will be dilineated at the discretion of the supervising engineer as excavation progresses.

The lining for the canal could be either a compacted blanket of fine grained material or a plastic liner. While the latter may be preferable for reasons of low permeability and resistance to freeze-thaw degredation, economics and the practicability of its application over short sections at different time periods may preclude its use. Furthermore, a 0.3 m thick layer of bedding sand would be necessary beneath the plastic liner. The environmental acceptability of a plastic liner is also in question.

If a blanket of relatively fine grained material is used, the material should have a minimum of 20 percent of the soil particles passing the No. 200 sieve. This material will, in all probablility, be the silty till found near the site. The blanket should be 1 m thick and placed in 0.3 m thick well-compated lifts.

The lining material should have a permeability of less than 10^{-6} cm/s in a laboratory test. While a permeability value of only 10^{-5} is necessary to substantially cut off the seepage, test results have indicated that the permeability of fine grained compacted soils typical to the Anvil site may increase by more than an order of magnitude when subjected to repeated freeze-thaw cycles.

5.0 SUMMARY OF CONCLUSIONS

a) One-dimensional geothermal considerations indicate that first season thaw can reach a computed depth of approximately 2.9 m (9.5 ft.) beneath the surface of a practicable width of gravel lining/insulation to be placed over the constructed channel section.

b) Using effective angles of internal friction of 30, 35, and 40 degrees together with coefficients of consolidation of 2.5 x 10^{-3} and 2.5 x 10^{-2} cm^2/sec, thaw slope stability analyses incidate that 2 horizontal to 1 vertical side slopes will provide adequate short term factors of safety for the back cut slope of the channel. The analyses also indicate that the factors of safety increase markedly over the long term as thaw consolidation induced pore water pressures dissipate with time.

c) One-dimensional geothermal analyses for the slopes in the vicinity of the downslope portion of the overall construction in the area of the proposed water retention area between the cross-valley and intermediate dams indicate that continuous thaw in the subaqueous soils have a $d/\sqrt{t}$ ratio of approximately 0.06 $cm/sec^{1/2}$.

d) Using effective angles of internal friction of 30, 35, and 40 degrees, together with coefficients of consolidation of 2.5 x 10^{-3} and 2.5 x 10^{-2} cm^2/sec, thaw slope stability analyses indicate that slope failures near the toe of the channel spoil pile are unlikely.

e) Overall channel stability could be threatened if construction materials or excavation spoil are placed on thawing permafrost or over areas in which an unfrozen sandwich layer (talic) from the previous thaw season exists beneath the ground surface. If this occurs over significant areas during construction, continuous downslope movements of those portions of the construction placed on the original ground surface could be initiated.

f) Some subsidence, surficial sloughing and downslope movements within the unengineered portion of the excavation spoil can be expected due to the nature and methods of placement of these

materials. These movements are not expected to compromise the integrity of the channel construction proper and are expected to cease after the first two or three thaw seasons.

g) There is no evidence of deep seated creep movements taking place in the south wall of the Rose Creek Valley and excavations for the proposed diversion canal are not expected to initiate such movements in the materials upslope from the canal.

h) Seismic risk analyses indicate that 1 in 100 year and 1 in 200 year earthquake events would involve firm ground accelerations of 5.4 and 9.7 per cent gravity, respectively. Pseudo-static analyses using these criteria are believed to provide unrealistically low factors of safety for the proposed channel construction.

i) Assessment of earthquake induced slope displacements involving low strength soil and a low coefficient of consolidation ($\emptyset'$ = 30 degrees and $c_v = 2.5 \times 10^{-3}$ cm^2/sec) indicates that downslope movements less than 0.2 m (0.66 ft.) would be induced by a 1 in 200 year earthquake event if the earthquake occurred at the time of maximum thaw depth during the first thaw season after construction. After permafrost thaw has reached equilibrium a similar earthquake is expected to produce negligible downslope movement.

j) In subaqueous areas where shear strengths are low and pore water pressures are high during the early stages of thaw, some slopes may experience shallow skin flow failures during an earthquake. These could become retrogressive.

Once geothermal equilibrium is achieved in these areas and thaw induced pore water pressures have been dissipated, earthquake induced stability failures on these slopes are not expected to occur.

k) The channel construction and foundation materials are not considered to be generally susceptible to liquefaction during earthquakes.

l) The canal should be lined with a 1 m thick till blanket in areas where the permeability of the natural materials is estimated to be greater than 10^{-4}cm/sec. The till material should have 20 percent by weight passing the No. 200 sieve and should be placed in 0.3 m thick well compacted lifts.

TABLE VIII-1

Climatic Data - Cyprus Anvil Mine Site

Month	Mean Daily Air Temperature (°C)
January	-20.8
February	-14.3
March	-11.3
April	- 3.9
May	3.6
June	10.6
July	11.1
August	9.6
September	3.9
October	- 3.1
November	-12.3
December	-17.8

Mean Annual Air Temperature - 3.7°C.

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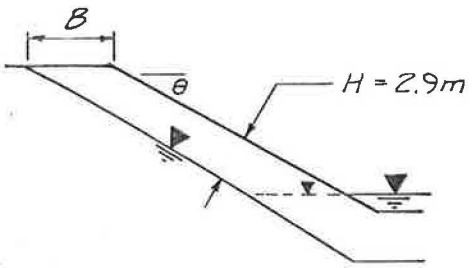
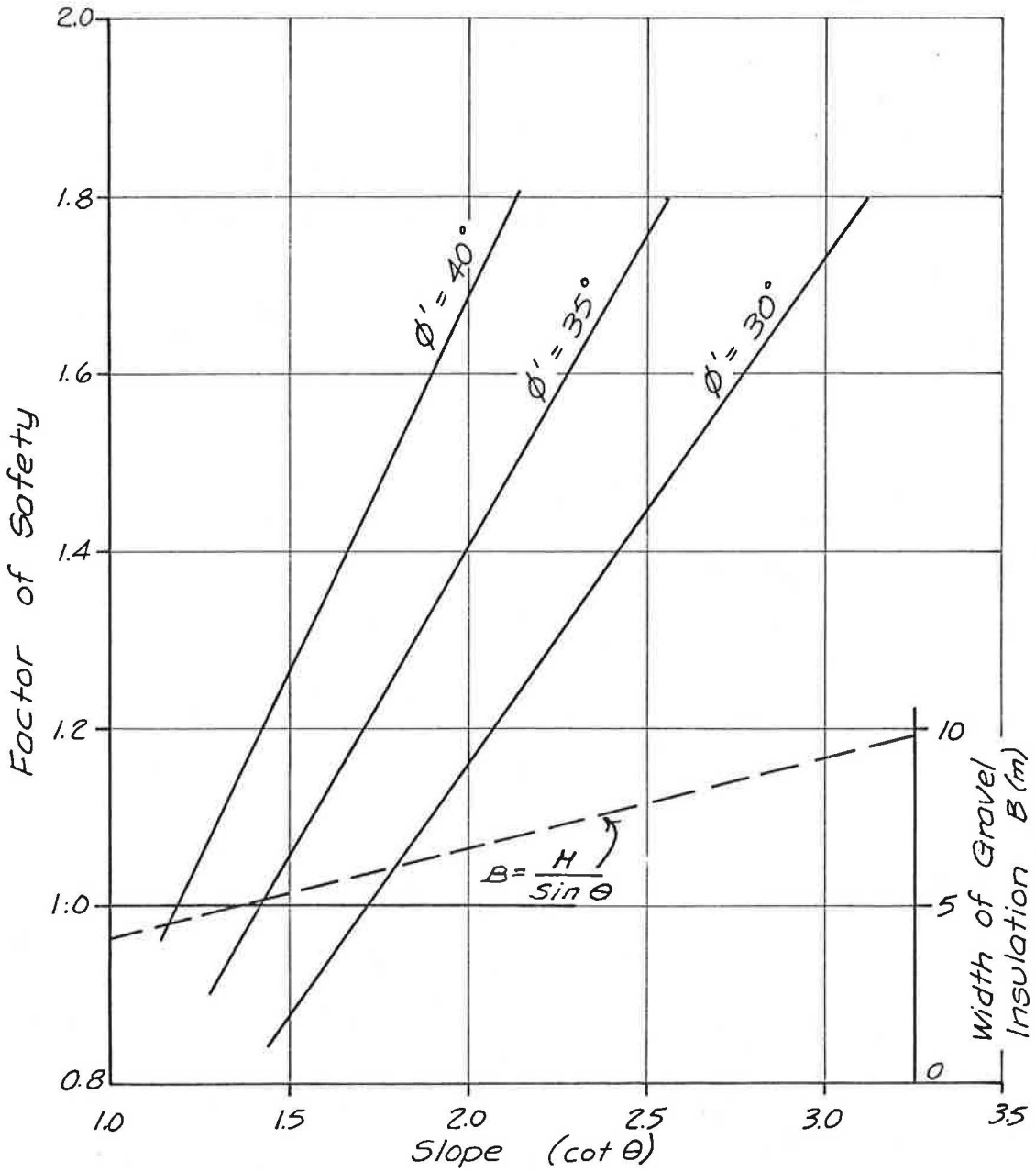
CHANNEL BACKSLOPE - LONG TERM FACTOR OF SAFETY vs SLOPE GRADIENT

Figure VIII- 1



$$F_s = \frac{\tan \phi}{\tan \theta}$$

$$B = \frac{H}{\sin \theta}$$



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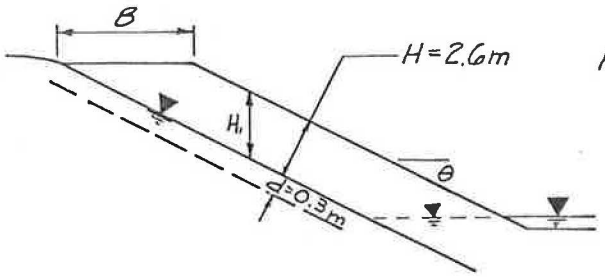
CHANNEL BACKSLOPE – FIRST THAW SEASON
FACTOR OF SAFETY vs. SLOPE GRADIENT
(DEPTH OF THAW BENEATH GRAVEL IS 0.3m)

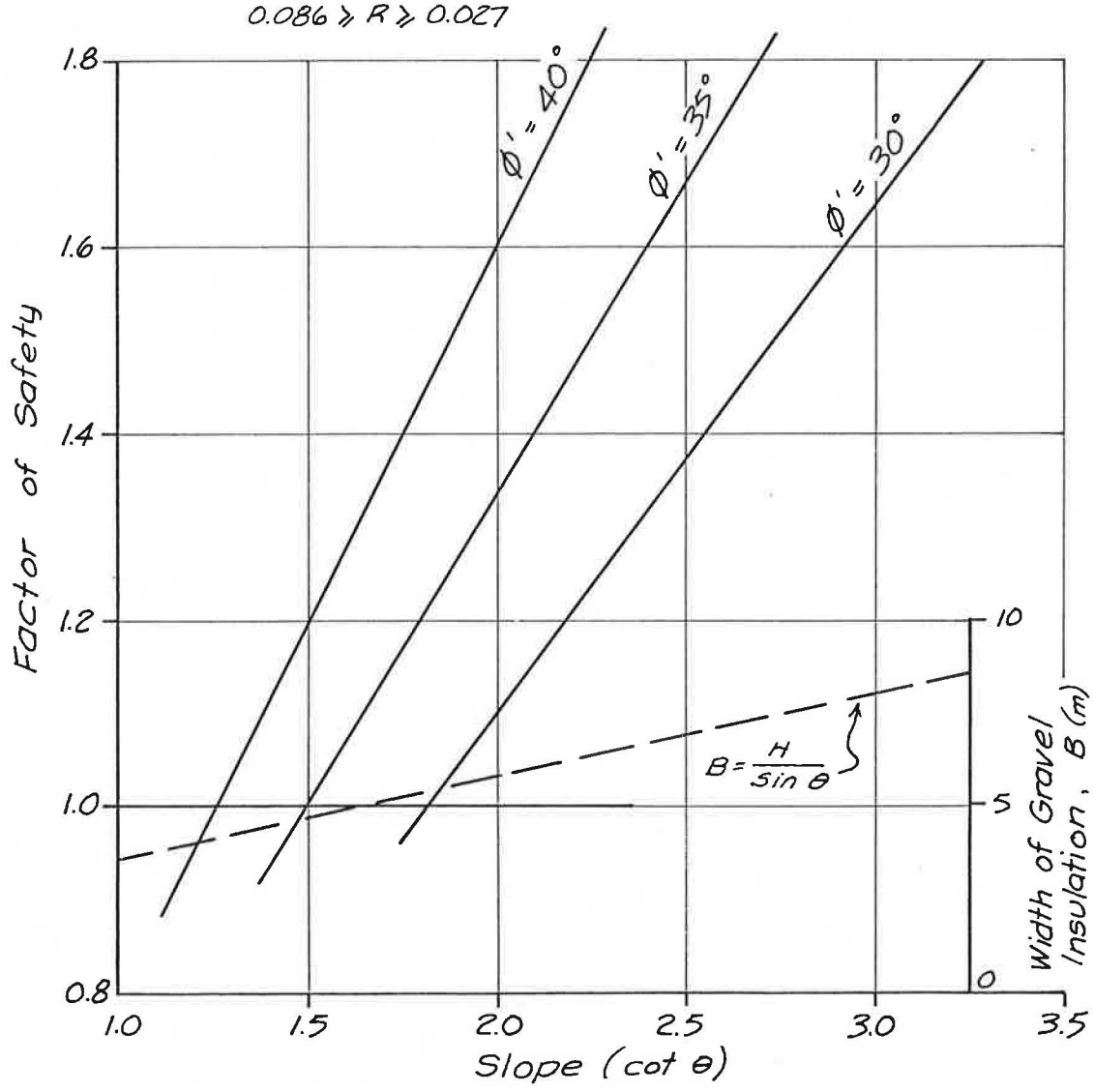
Figure VIII: 2



$$F_S = \frac{P_o + \gamma' d}{P_o + \gamma d}\left[\frac{1 + k_2 H/k_1 d}{2R^2 + 1 + k_2 H/k_1 d}\right]\frac{\tan\phi'}{\tan\theta}$$

$$B = \frac{H}{\sin\theta}$$

For all curves
$2.5 \times 10^{-3}\ cm^2/sec. \leqslant cv \leqslant 2.5 \times 10^{-2}\ cm^2/sec.$
$0.086 \geqslant R \geqslant 0.027$



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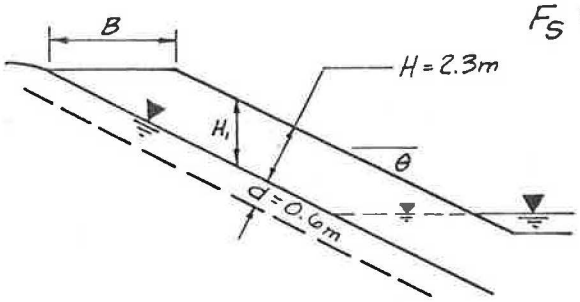
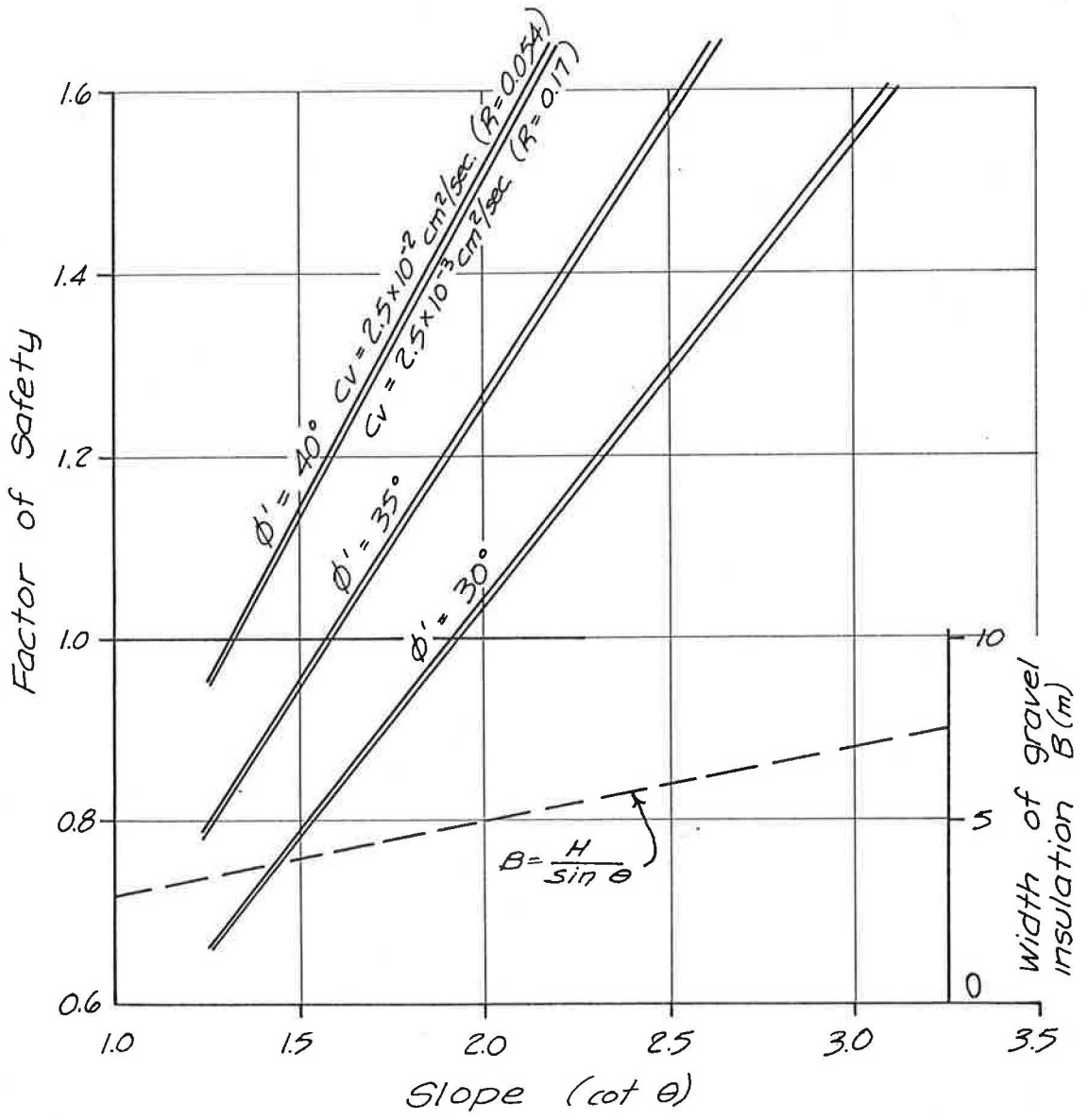
CHANNEL BACKSLOPE - FIRST THAW SEASON
FACTOR OF SAFETY vs. SLOPE GRADIENT
(DEPTH OF THAW BENEATH GRAVEL IS 0.6m)

Figure VIII-3



$$F_S = \frac{P_o + \gamma' d}{P_o + \gamma d}\left[\frac{1 + k_2 H/k_1 d}{2R^2 + 1 + k_2 H/k_1 d}\right]\frac{\tan \phi'}{\tan \theta}$$

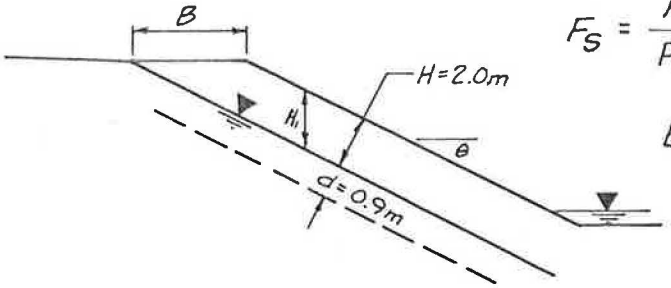
$$B = \frac{H}{\sin \theta}$$



Project No. 192-2025 Drawn R.P. Reviewed _____ Date Jan. '80

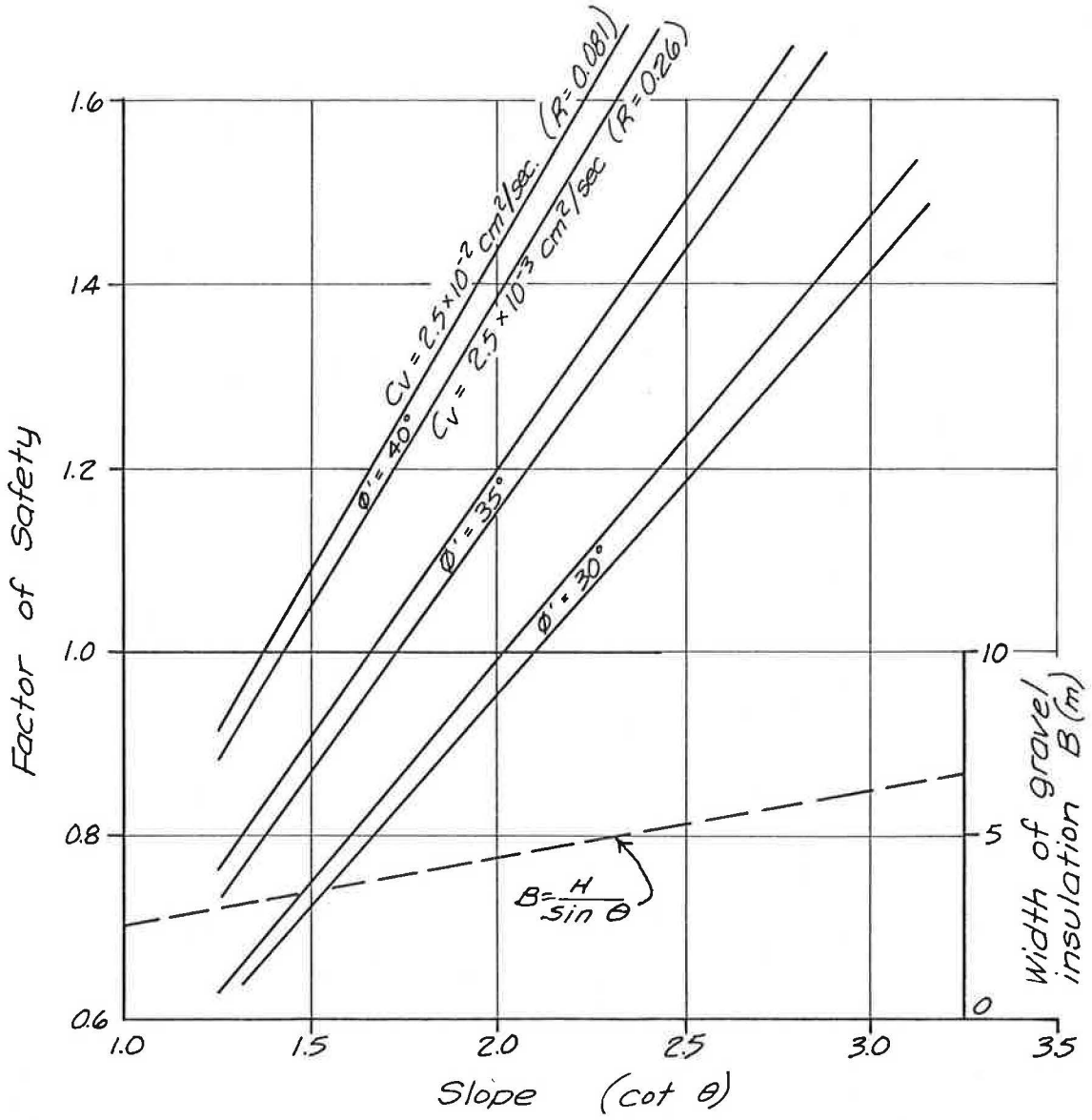
CHANNEL BACKSLOPE - FIRST THAW SEASON
FACTOR OF SAFETY vs. SLOPE GRADIENT
(DEPTH OF THAW BENEATH GRAVEL IS 0.9m)

Figure VIII 4



$$F_S = \frac{P_o + \gamma' d}{P_o + \gamma d}\left[\frac{1 + k_2 H/k_1 d}{2R^2 + 1 + k_2 H/k_1 d}\right]\frac{\tan\phi'}{\tan\theta}$$

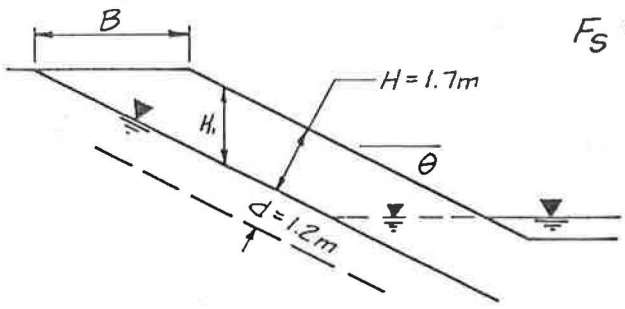
$$B = \frac{H}{\sin\theta}$$



Project No. 192-2025 Drawn R.C. Reviewed _____ Date Jan. '80

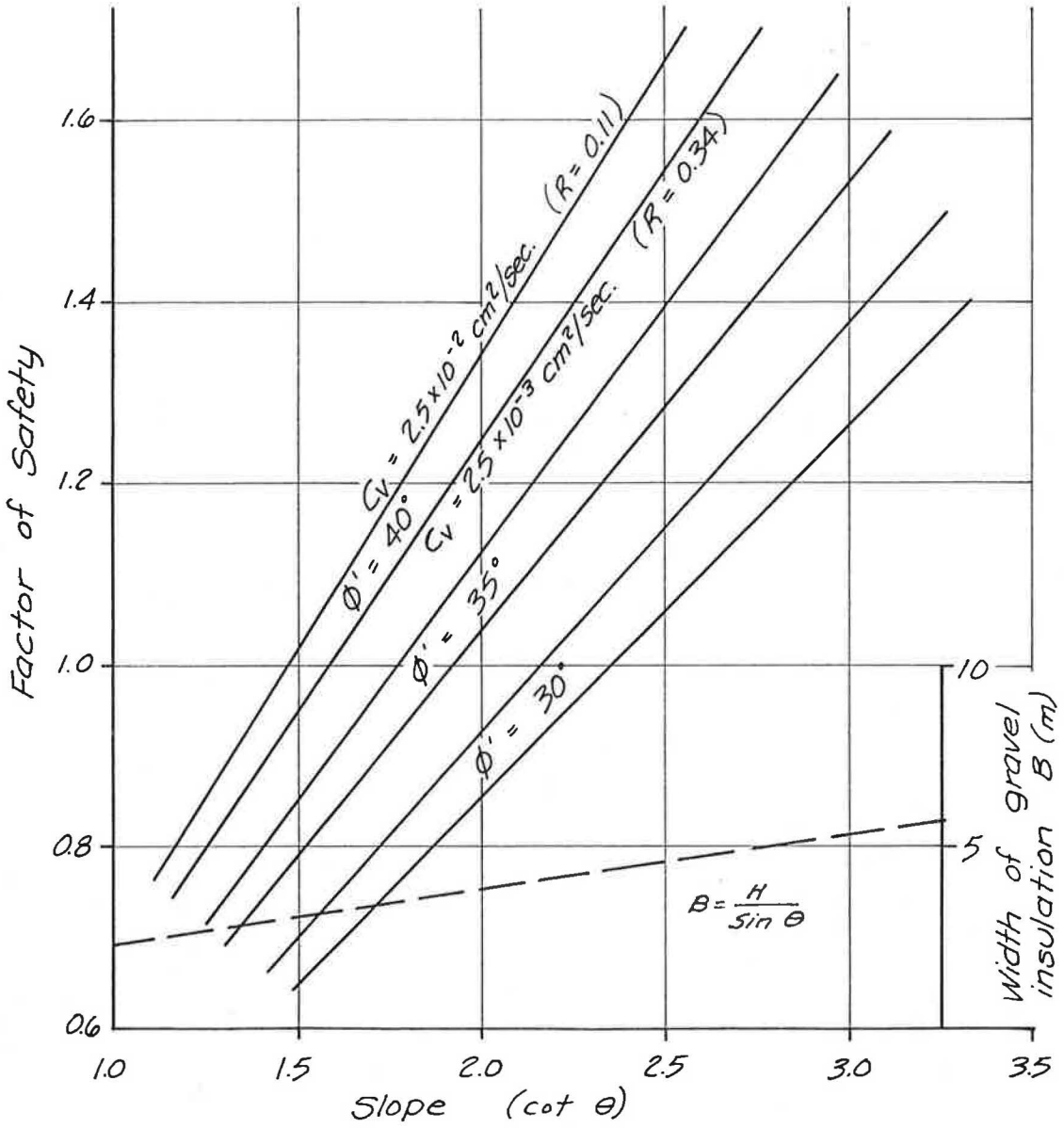
CHANNEL BACKSLOPE - FIRST THAW SEASON FACTOR OF SAFETY vs. SLOPE GRADIENT (DEPTH OF THAW BENEATH GRAVEL IS 1.2 m)

Figure VIII 5



$$F_S = \frac{P_o + \gamma' d}{P_o + \gamma d}\left[\frac{1 + k_2 H/k_1 d}{2R^2 + 1 + k_2 H/k_1 d}\right]\frac{\tan \phi'}{\tan \theta}$$

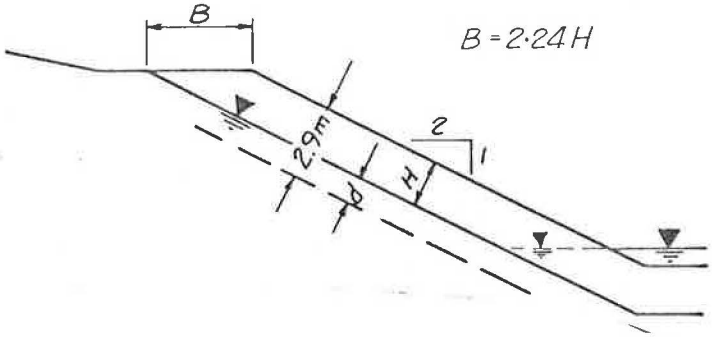
$$B = \frac{H}{\sin \theta}$$

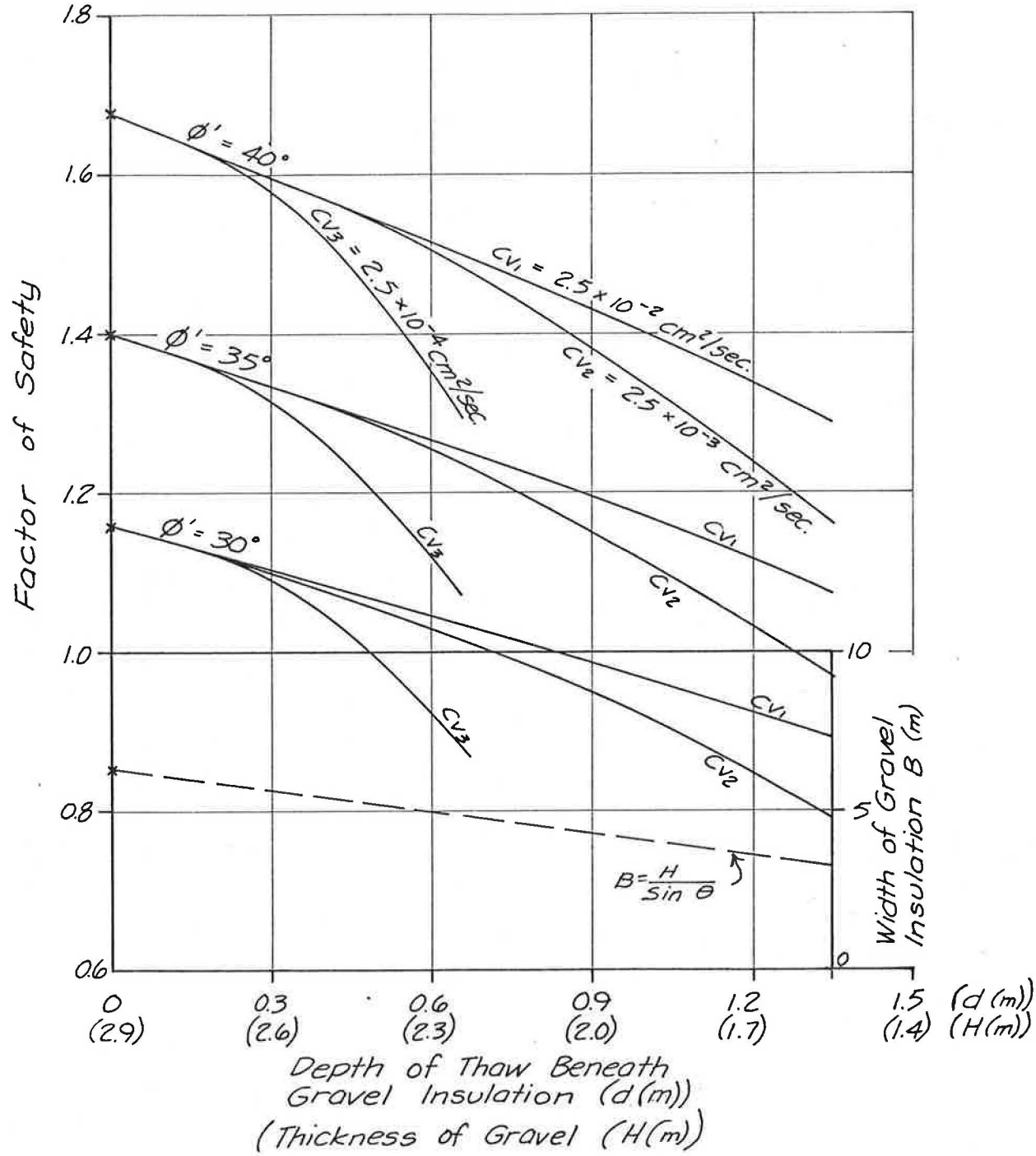


Project No. 792-2025 Drawn R.R. Reviewed ..... Date Jan. '80

CHANNEL BACKSLOPE - FIRST THAW SEASON FACTOR OF SAFETY vs. THAW DEPTH BENEATH BASE OF GRAVEL

Figure VIII-6

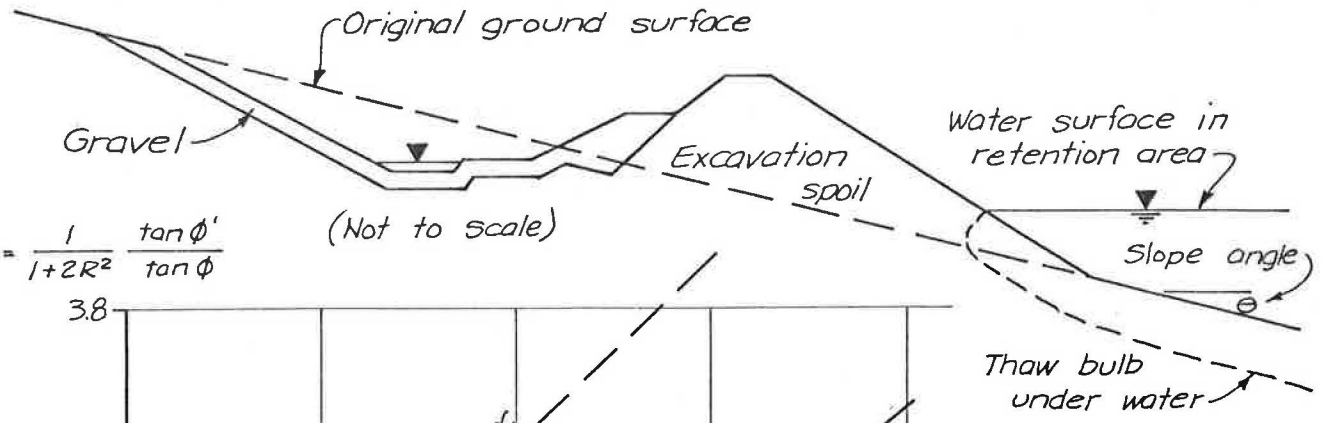




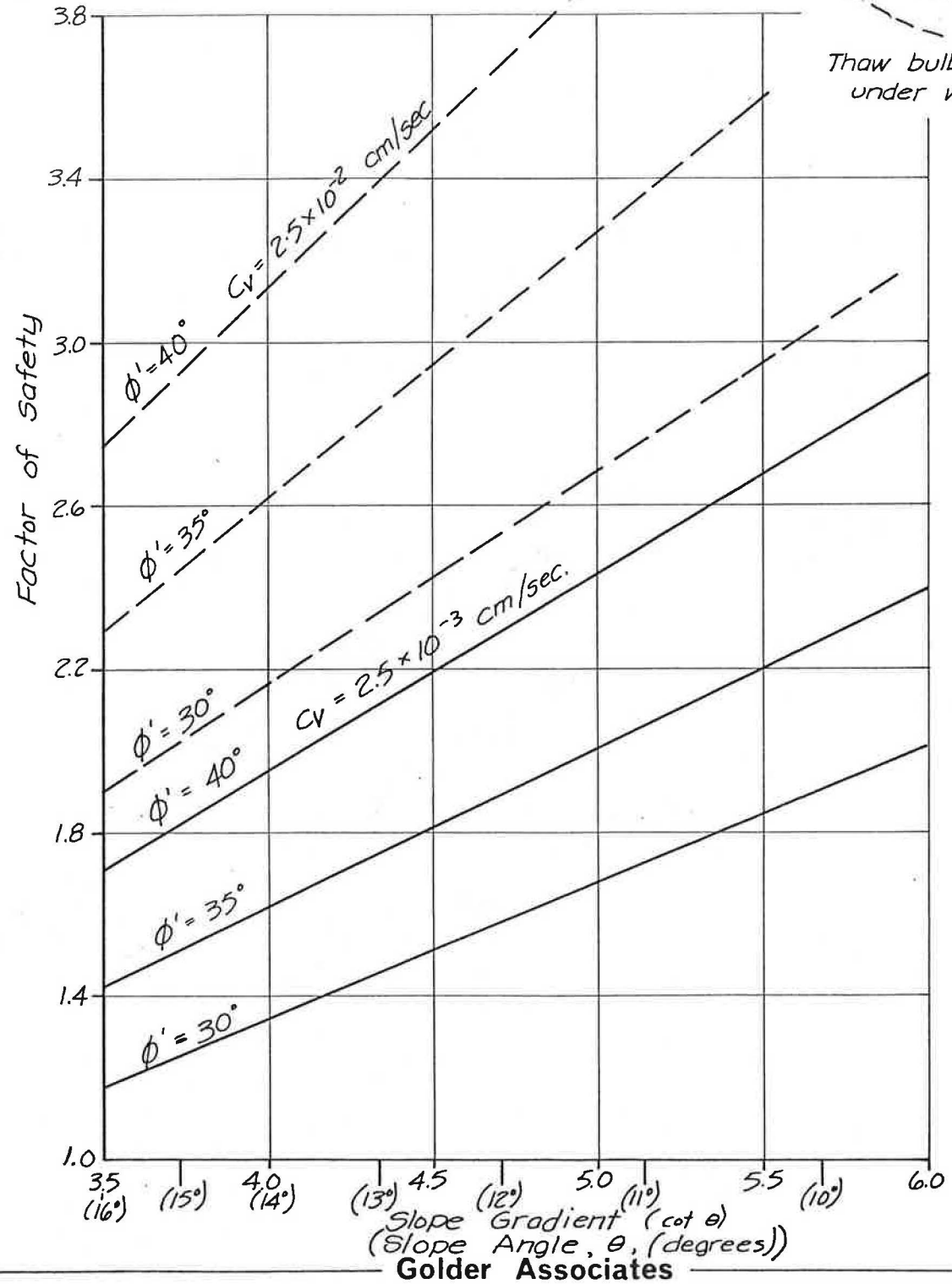
Project No. 192-2025 Drawn R.P. Reviewed ..... Date Jan. 20

FACTOR OF SAFETY vs. SLOPE GRADIENT FOR SUBAQUEOUS SLOPE AT TOE OF EXCAVATION SPOIL

Figure VIII-7



$$F_S = \frac{1}{1+2R^2}\,\frac{\tan\phi'}{\tan\phi}$$



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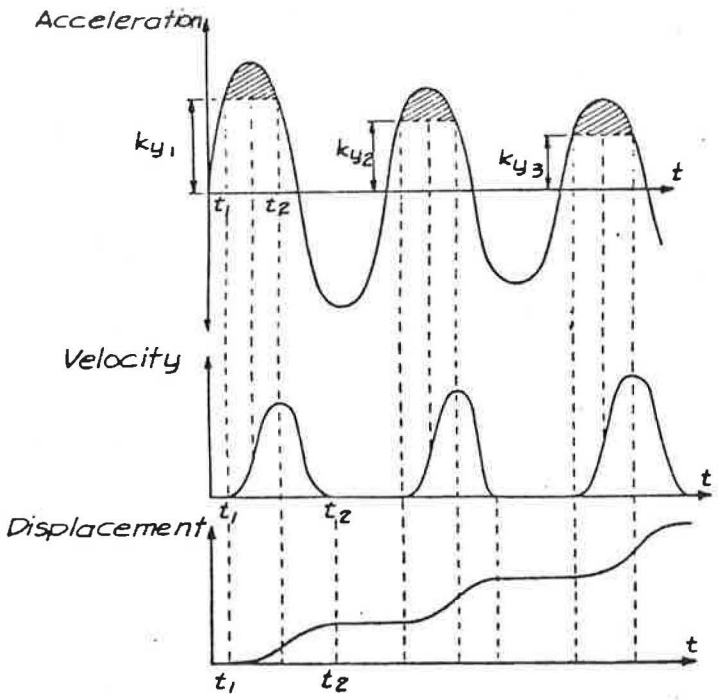
Golder Associates

INTEGRATION OF ACCELEROGRAMS TO DETERMINE DOWNSLOPE DISPLACEMENTS

Figure VIII-8

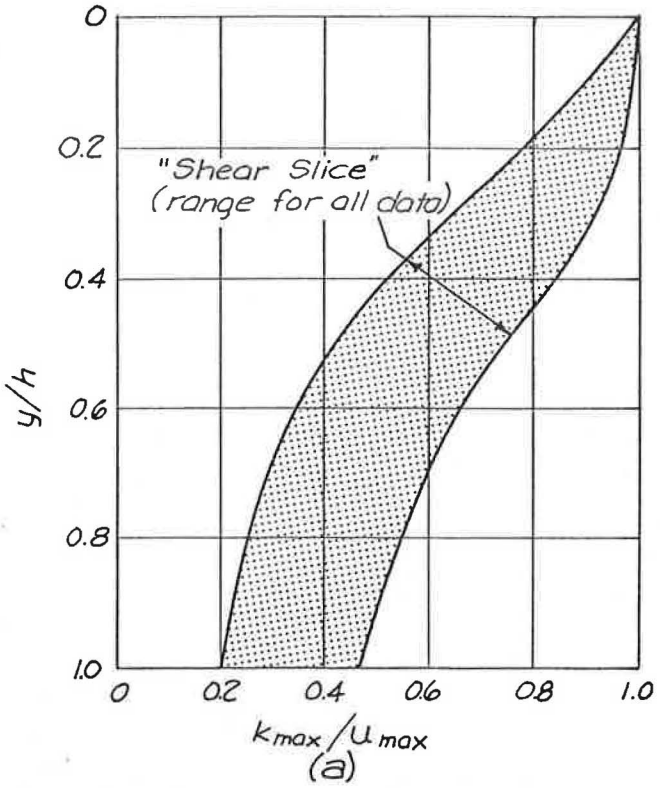


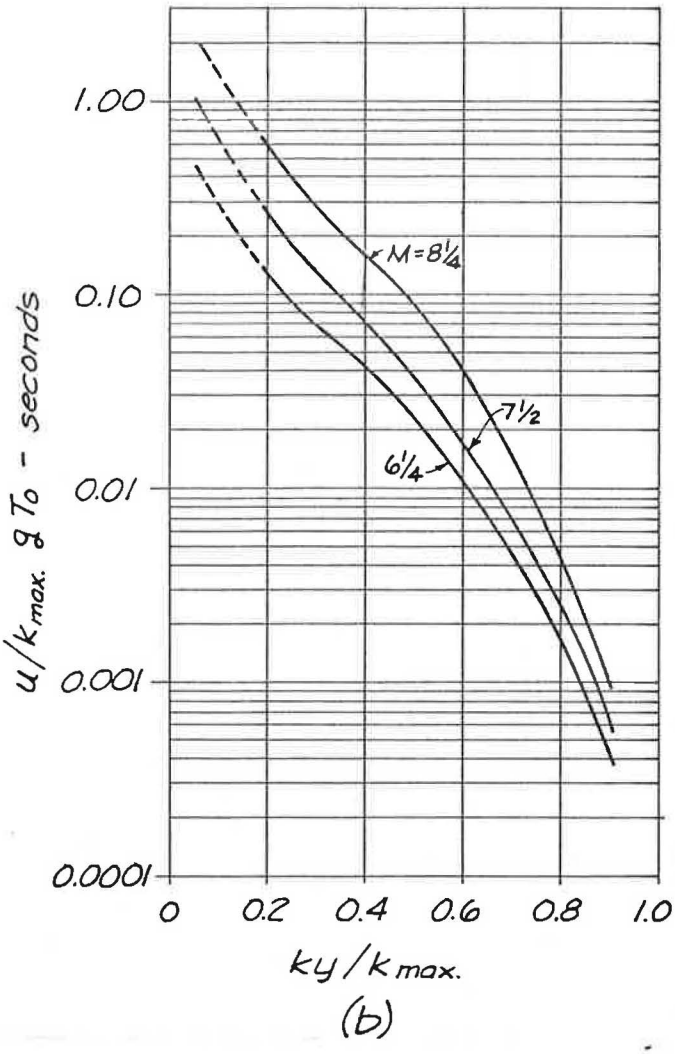
(REF: Goodman & Seed (1966))

Project No. 192-2025 Drawn R.D. Reviewed _____ Date Jan. '80

COMPUTATION OF EMBANKMENT DISPLACEMENTS DUE TO EARTHQUAKE INDUCED GROUND MOTIONS

Figure VIII-9





(Ref:- Makdisi and Seed (1978)

Example:- for embankment with 2:1 side slopes, $\phi' = 30°$ and $c_v = 2.5 \times 10^{-3}$ and $c_v = 0$ cm^2/sec

1. The response of the embankment crest to ground accelerations has been assumed to be the same as the firm ground accelerations since the thickness of the first season thaw layer is of the order of only 3 m (10 ft.).

2. The first natural period, T_o, of the embankment has been calculated to be of the order of 0.1 to 0.2 sec. For conservatism a value for T_o of 1.0 is used.

3. For a 2:1 slope with $\phi' = 30°$ and $c_v = 2.5 \times 10^{-3}$ cm^2/sec with a depth of thaw of 0.6 m (2 ft.) beneath the gravel, and $c_v = 0$, representing conditions when thaw induced pore water pressures are completely dissipated, yield accelerations of $k_y = 0.012$ and 0.062 g, respectively, have been calculated.

4. From Figure 10a), for y/h = 0, k_{max} has been taken as 0.097 g for a 1:200 year earthquake of Richter magnitude = 5.75.

5. Applying these criteria with Figure 10b), displacements are estimated to be:-

   a) worst conditions - first thaw season:-
   $U/k_{max}\ g\ T_o = 0.20$ $\quad U = 0.097\ (32.2)(1.0)(0.20) = 0.62$ ft. = 0.2 m
   b) long term conditions - after dissipation of excess pore water pressures:-
   $U = 0.097\ (32.2)(1.0)(0.008) = 0.025$ ft. = 0.008 m

Note:- 1. The influence of T_o could cause these estimates to be as much as an order of magnitude higher than actual.
2. This analysis applies to compacted clayey embankments and to dry or dense cohesionless soils that do not lose strength during cyclic loading.

Project No. 792-2025 Drawn RQ Reviewed _____ Date Jan. '80

HYDROLOGIC AND HYDRAULIC DESIGN
OF
DOWN VALLEY TAILINGS DISPOSAL PROJECT
CYPRUS ANVIL MINING CORPORATION
FARO, Y.T.

Prepared for:
GOLDER ASSOCIATES

Prepared by:
HYDROCON ENGINEERING (Continental) LTD.

MAY 8, 1980

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HYDROCON ENGINEERING (Continental) LTD.
ENGINEERING CONSULTANTS IN WATER RESOURCES

May 8, 1980 File: 8-7.1

Mr. H. Glen Gilchrist, P. Eng.
Calgary Manager
Golder Associates
5915 - 3rd Street S.E.
Calgary, Alberta
T2H 1K3

SUBJECT: HYDROLOGIC AND HYDRAULIC DESIGN OF
DOWN VALLEY TAILINGS DISPOSAL - CYPRUS ANVIL MINING CORPORATION

Dear Mr. Gilchrist:

We are hereby pleased to submit 5 copies of our study and recommendations for the above noted project. The preliminary November 1, 1979 report has been revised in accordance with your comments and discussions with Mr. Dave Goodall, P. Eng. following completion of the past winter's test section. Added in the present scope of work are our recommendations for the emergency overflow spillways and the north valley interceptor ditch.

We trust that our design recommendations are adequate to enable you to prepare the detailed design drawings and specifications. Some of the design details, particularly for the tributaries to Rose Creek Diversion, the emergency spillways and interceptor ditch can probably best be finalized by the field engineer during construction.

If we can be of any assistance in further finalizing or clarifying our recommendations, please call us at your convenience.

Yours truly,
HYDROCON ENGINEERING (Continental) LTD.

per D. L. Cooper
Wim M. Veldman, P. Eng.

WMV:dc

Enclosures

INTRODUCTION

To provide adequate storage for tailings disposal from the base metal mine operated by Cyprus Anvil Mining Corporation at Faro, Yukon, the Rose Creek valley near the mine is required. Two tailings dam are proposed to contain waste material. The storage capacity of the existing dam are inadequate for the mine's needs over the next 20-30 years.

Golder Associates, the general consultants to Cyprus Anvil Mining Corporation Ltd. for the expansion of the tailings facilities, requested Hydrocon Engineering (Continental) Ltd. to assist in the hydrologic and hydraulic design of:

1) The diversion of Rose Creek around the new tailings dam,
2) Emergency overflow spillways for the proposed intermediate and cross valley dams,
3) North valley interceptor ditch.

A preliminary report on the diversion was submitted on November 1, 1979. This report is a finalization of the design recommendations contained therein plus the recommended design of the emergency overflows and north valley interceptor ditch.

The report and drawings are sufficiently detailed to enable the preparation of tender and construction drawings by Golder Associates. Items such as design details of tributaries into Rose Creek Diversion, orientation and armoring of the decant channel, orientation of the inlet and outlet channels for the emergency spillways and orientation and armor details for the north valley interceptor ditch will need further refinement and/or field engineering during layout and construction.

HYDROLOGY

PREVIOUS STUDIES FOR ROSE CREEK

The existing Rose Creek diversion channel designed by Golder Associates in 1974[1] was sized for a flow of 48m^3/sec. (1700 cfs.)[2]. The average return period of the flow is not documented, however Sigma Resource Consultants in their report on the 1975 runoff[3] state "...peak flow of 1700 cfs. (48m^3/sec.) which it is understood, has an average return period of 25 years." Montreal Engineering Company (MECO) in a general report on waste management procedure for Yukon base metal mines[4] computed a 20 year flow of 60.3m^3/sec. (2130 cfs.) Extrapolation of a frequency curve for Rose Creek prepared by Water Survey of Canada (WSC)[5] results in a 20 year instantaneous flood magnitude of 113.2m^3/sec. (4000 cfs.).

It is not clear how Golder Associates and Cyprus Anvil arrived at their design flow of 48m^3/sec. MECO it appears, used the Rational Method to compute their estimate. The WSC frequency relationship was based on data points for the years 1967 - 1969 inclusive only.

---

1. *"A Preliminary Report to Anvil Mining Corporation Limited on Proposed New Tailings Facilities for Anvil Mine, Faro, Yukon."* Golder Brawner Associates, June 1974.

2. Personal communications with H.G. Gilchrist of Golder Associates and N.G. Cornish, Environmental Control Manager for Cyprus Anvil Mining Corporation.

3. *"Rose Creek Diversion Works, Observations and Measurements Made During Spring Runoff - 1975."* Sigma Resource Consultants Ltd. June 1975.

4. *"Waste Management-Yukon Base Metal Mines."* MECO report to Department of Indian and Northern Affairs, Whitehorse, April 1974.

5. *"Magnitude of Floods, Yukon Territory."* Planning and Studies Section, Water Survey of Canada, Vancouver, May 1978.

AVAILABLE FLOW DATA FOR ROSE CREEK

WSC gauged Rose Creek below Faro Creek in September 1966 and from January 1967 to July 1969. Daily flow data, monthly averages and annual extremes are given in Appendix 1. Sigma Resource Consultants gauged the diversion channel during the 1975 spring runoff from May 10 to June 4 results of which are shown in the form of a hydrograph in Appendix 1.

Yearly recorded or estimated extremes are given in Table 1 below. Minimum winter flows are governed by releases from the South Fork reservoir and flow over the pumphouse pond.

TABLE 1

ANNUAL FLOW EXTREMES-ROSE CREEK

Year	Maximum Daily Discharge		Minimum Daily Discharge	
	cfs.	m^3/sec.	cfs.	m^3/sec.
1967	1150	32.6	6.1	0.2
1968	548	15.5	7.0	0.2
1969	354	10.0	7.3	0.2
1975	950	26.9	Not Available	

It is believed that the 1975 flow was the largest experienced by the existing diversion channel since its construction in 1974.

FREQUENCY ANALYSIS - ROSE CREEK

To compute the design flow for Rose Creek, the following methods were assessed:

1) Preparation of a discharge - frequency relationship from recorded flow,

2) Rational Method,

3) Methods used by other researchers and,

4) Correlation with nearby streams of similar physiographic characteristics.

Extrapolation of the frequency curve prepared by WSC, based on only three data points, is considered to be highly inaccurate. The Rational Method was developed for small urban watersheds and is generally not used for large, steep mountainous areas.

Other studies[6,7,8] on runoff characteristics of small watersheds in permafrost regions are generally site specific and often contradictory in their general assessment of factors affecting runoff. The Kinematic Wave Model for flood frequency analysis of ungauged streams is used by the Department of Indian and Northern Affairs for flow forecasting in the Yukon.[9] At this time it is not fully calibrated or defined, however it shows good promise for the future. Thus correlation

---

6. *"Estimation of Small Basin Flood Flows in the Discontinuous Permafrost Zone."* D.G. Vincent and S.O. Russell, in Canadian Hydrology Symposium: 79 Proceeding, National Research Council Canada.

7. *"Permafrost Runoff: The Consulting Engineer's Experience."* G.C. Seagel and R.P. Parish, Bolten, Parish, Trimble, Edmonton in Permafrost Hydrology Proceedings of Workshop Seminar, Canadian National Committee, The International Hydrological Decade, 1974.

8. *"River Hydrology in Permafrost Areas."* R.W. Newbury, Same as No. 7 above.

9. *"The Kinematic Wave Model for Flood Frequency Analysis in Ungauged Areas: An Engineering Application in the Yukon Territory."* M. Orecklin, Hydrologist, DIANA Whitehorse. Canadian Hydrology Symposium, 1979 Proceedings, National Research Council Canada.

with nearby streams having adequate flow data was utilized.

In the hydrological study of the Dempster Lateral Pipeline river crossings,[10] flood coefficients, computed as discharge divided by a fractional power of the drainage area, were determined for the gauged streams for the mean annual (MAF) and 100 year floods. For the Pelly River at Pelly Crossing (Sta. 9BC1) the MAF and 100 year coefficients were computed to be 0.22 and 2.47 respectively.

From equations:

$$Q_{MAF} = C\ D.A.^{0.85}$$

and $$Q_{100} = C\ D.A.^{0.7}$$

where C is the flood coefficient and D.A. the drainage area in $km.^2$ The MAF and 100 year floods for Rose Creek were computed to be $22m^3/sec.$ (775 cfs.) and $109m^3/sec.$ (3855 cfs.) respectively. The drainage area of 224 $km.^2$ (86 $mi.^2$) was obtained from 1:50,000 topographic plans and is as indicated on Figure 1. The resultant frequency curve is as plotted on Figure 2. MECO's 20 year flood estimate is quite similar.

To assess the similarity between runoff patterns in the Pelly River and Rose Creek in more detail, the return period of recorded maximum instantaneous flows on Rose Creek, as determined from the computed frequency curve, were compared to the return period of maximum instantaneous flows for Pelly River for the same years from the frequency curve computed for Pelly River for the period 1952-1977. The results are as given in Table 2 below.

---

10. *"River Engineering and Geotechnical Considerations at Selected River Crossings Along the Proposed Dempster Lateral Gas Pipeline."* Prepared for Foothills Pipe Lines (Yukon) Ltd. by W.M. Veldman previously with Northwest Hydraulic Consultants Ltd. and F.B. Claridge (Komex Consultants Ltd.).

TABLE 2

FLOW CORRELATION BETWEEN ROSE CREEK AND PELLY RIVER AT PELLY CROSSING

Year	Maximum Daily Flow in m^3/sec.		Return Period in Years	
	Pelly River	Rose Creek	Pelly River	Rose Creek
1967	2971	32.6	4.5	7
1968	1964	15.5	1.5	2
1969	1372	10.0	-	-
1975	2583	26.9	3.5	4.5

Flow patterns on Pelly River and Rose Creek are thus quite similar for the years of record on the latter. This is to be expected for snowmelt generated floods. However, a localized summer storm could produce high flows on Rose Creek and have an insignificant effect on the Pelly River at Pelly Crossing. Generally though it can be concluded that the Rose Creek frequency curves derived from flow characteristics of the Pelly River, is representative and reasonable.

SELECTION OF DESIGN FLOOD

The design flood selected for a particular project depends, in part, on:

1) duration of the project,

2) consequences of a flood larger than design and,

3) availability of equipment that can be readily mobilized for emergency repairs.

The economic life of the mine is in the order of 20-30 years. During this time Cyprus Anvil will be undertaking maintenance and upgrading measures on the proposed diversion similar to periodic maintenance and upgrading work they have performed on the existing diversion channel. This period, particularly if a large flood occurs, will serve as the "learning curve" in determining whether any further upgrading measures are required for the long term. If the capacity of the channel was exceeded, and flow spilled into the tailings pond, retention time in the pond would become minimal. The dam will be designed for this probability thus its integrity is not of concern.

Based on the above discussion, a design flow with an average return period of 50 years is recommended plus the channel should be capable of withstanding without being overtopped and serious damage, a "contingency flood" with an average return period of 500 years. Table 3 summarizes flow magnitudes for various return periods as obtained from Figure 2.

TABLE 3

ROSE CREEK DIVERSION FLOW MAGNITUDES FOR VARIOUS RETURN PERIODS

Return Period in Years	Flow in m^3/sec.	(cfs.)
10	51	1802
50	88	3110
100	109	3851
500	160	5650

NORTH VALLEY INTERCEPTOR DITCH

Drainage area for the North Valley Interceptor Ditch is approximately 33.6 km^2 (13 mi^2) as measured from Figure 1. The precise area may be somewhat different because of interruption of natural drainage patterns in the area by the mining activity. Design flows for the ditch were computed, based on a ratio of drainage areas plus 25%,[11] from Rose Creek flows shown in Table 3. Flows were calculated for 3 locations corresponding to the location of the tributaries entering the interceptor ditch (see Figure 9 for a partial plan of the ditch and tributaries). Data is as summarized in Table 4.

TABLE 4

DESIGN FLOWS IN M^3/SEC. FOR NORTH VALLEY INTERCEPTOR DITCH

Average Return Period In Years	Location - See Fig. 9		
	Sta. 0+000 To 0+650	Sta. 0+650 To 1+255	Sta. 1+255 To 2+000
10	5.1	7.2	10.2
50	8.9	12.4	17.7
100	11.0	15.4	21.9
500	16.1	22.5	32.2

11. Increase in unit runoff due to smaller drainage area of the North Valley Interceptor Ditch.

HYDRAULIC DESIGN

ROSE CREEK DIVERSION

Alignment - The selected alignment of the diversion channel was dependent on:

1) required tailings dam height,
2) design cross section and depth. The objective was to try and ensure that the 50 year flood level was fully contained in a cut rather than a fill configuration,
3) required channel slope,
4) local topographic features such as the high rock outcropping near the inlet area.

The dam height and location selected by Golder Associates was based on maximizing tailings storage. The cross section, design depth and slope of the channel were computed as more fully described in subsequent sections. Containment of the design section fully within a cut section was not always possible. Field data such as profiles and cross sections were obtained by Underhill and Underhill under the direction of Golder Associates.

Hydraulic - Assuming 2:1 channel side slopes, the excavation volumes are proportional to the base width of the channel. A 12m. (40') channel width was selected to maintain design velocities at a reasonable level, and more importantly, to approximate the natural regime width of the channel. The "regime theory" postulates there is a definite relationship between dominant discharge (usually the mean annual flood) and channel dimensions

and slope. Design depth for this width and a slope of 0.19%, and Manning's 'n' value of 0.030 is 2.4 metres (7.8 feet). Design depth for the 500 year flood would be approximately 1 metre greater. Design velocities for the 50 and 500 year floods would be 2.2 and 2.6m metres/sec. respectively.

A low-flow channel on the uphill side is recommended to:

1) increase winter velocities and thus reduce the tendency for ground fast ice to develop in the channel,

2) concentrate the zone of highest velocity along the uphill side of the channel and thus as far as possible removed from the more vulnerable downhill side. A bottom width of 3.65 metres was selected for ease of construction. The capacity of the low flow channel, 2 m^3/sec. (70 cfs.) is adequate to convey winter flows.

Minimum curve radius of 35m (115 feet) was established to prevent excessive superelevation of flow around the curves. This requirement along with other hydraulic design characteristics are illustrated on Figure 3.

Channel Armoring - Test pits by Golder Associates and the test section constructed in February 1980 indicated insufficient quantities of cobbles and boulders in the native material to ensure an armor layer can develop naturally from an unlined channel. To reduce thermal degradation of the native material, a 1.7m (5.6 feet)[12] thick layer of granular material will be

---

12. As specified by Golder Associates.

used to line the channel walls except where thaw stable material is present. In pervious native material, it will be necessary to place a till lining to minimize seepage losses into the tailings ponds. It is also anticipated that rock will be encountered in several locations.

These alternative methods of natural or placed channel lining require various armoring techniques to ensure the channel's stability under the design velocities mentioned in the previous section. Without armoring, general erosion that would occur during design conditions could lead to undermining of the uphill slope, downcutting of the channel bed and erosion of the downhill cut or fill slope.

The various armoring techniques are illustrated on Figure 4 and are as more fully described below.

a) Typical Section Armoring - Armoring by windrowing cobbles from gravel borrow pits was initially proposed. However, following test excavations by Golder Associates, this technique was considered unsuitable and, to reduce maintenance, armoring as shown in Figure 4 is recommended. The 500mm. thick armor layer is to consist of rock with a D_{50} of from 180 to 210mm. The need for a filter layer should be field determined according to the criteria specified in Figure 4. The Engineer should ensure that after placement of the rock and filter layer, the net channel cross section meets the hydraulic characteristics as depicted on Figure 3. Armoring is required to the 500 year water level.

b) Till lining on channel slopes and bottom - The same armoring requirements apply as above except a thinner layer of filter is overlying the easily erodible till lining. The lining will be placed only to the 50 year water level as infrequent seepage out of the channel above that level is tolerable. Armoring between the 50 year and 500 year levels is as per option (a). A filter layer will be required between the armor and the till lining to prevent leaching of the till through the armor at design velocities.

c) Channel in rock - Armoring is not required in rock cuts except where the rock is rather pervious in which case b) above applies. Armoring above the rock surface to the 500 year contingency level will be as per option a) or b).

Drop Weirs - Drop weirs will be required in the 5% channel slope area between the crest of the cross valley dam and where the diversion re-enters Rose Creek to avoid excessive velocities in the channel. The drops as illustrated on Figure 5 will be quite similar to those on the existing diversion channel since they have performed reasonably well and meet the requirements of the fisheries - an important criteria. Placement of cobbles on the channel bed between the structures is recommended to avoid excessive erosion downstream of the drops - a problem experienced with some of the existing structures. The cobbles can be the reject

material from the quary or stockpile used for the large Class 'B' rock.

To perform as designed, the rock for the drop weirs must be adequately sized and well placed and keyed-in particularly at the crest. If necessary the crest could be grouted at a later date (when settlement has occurred). Some maintenance initially should be anticipated.

Gradient Control Sills - Gradient control sills as shown on Figure 5 are recommended for the 0.19% slope area of the diversion channel. The sills are intended to stabilize the channel's grade at periodic intervals. The sills can be eliminated in rock cut areas (see Figure 4) or where special armoring is placed in the channel for tributary inflow (Figures 6 and 7).

Inlet Diversion Structure - A relatively impervious low height dam on the existing diversion channel will be required downstream of the diversion channel inlet. The dam will probably be constructed, in the wet, to force the flow into the new channel. After diversion is complete, the necessary materials would be added to the dam to result in an impervious section.

To minimize the possibility of serious damage to the channel during a "larger-than-contingency-flood" or during severe icing conditions the diversion structure or a channel section near the inlet could be designed to be overtopped. This would permit a controlled flow magnitude into the tailings pond rather than an uncontrolled amount if a particular section of the channel was breached during an extreme flood event. The detailed design of the dam and its location will be addressed by Golder Associates in the final design.

Local Tributaries - The proposed diversion intercepts several creeks and springs. The creeks at Stations 0 + 614, 0 + 622 and 3 + 100 probably flow continuously. Others at Stations 0 + 586, 1 + 900 and 2 + 175 are intermittent and/or springs.

The design of the tributaries into the main channel are site specific and shall be generally field determined. The design conditions which must be fullfilled are:

1) High flow in the tributary and low to moderate flows in the main channel,
2) High flow in both the tributary and main channel,
3) Surface winter flow and,
4) Subsurface winter flow.

Figure 6 shows a typical design approach for major tributaries. If the grade of the tributary channel is much higher than that of the main channel, drop weirs similar in design approach as for the main channel will be necessary. Beyond the main channel area the tributary creek bed and banks should be well restored and/or armored to their natural conditions within the limits of the disturbed area.

Intermittent creeks and springs could pose a different problem particularly during the winter period. To try and avoid surface icings to develop as the creek or spring enters the main channel, very coarse riprap shall be placed in the creek bed or in the general area where the spring enters the main channel, to enable the low flows to percolate into the bottom of the low flow main channel. It will be very important to restore the disturbed area to prevent surfacing of the spring or creek prior to reaching the coarse rock section. Field engineering will be of prime importance.

A typical design is shown on Figure 7. If an approach similar to that shown is not successful and severe icings are created in the diversion channel other options such as inducing an icing to form uphill from the diversion channel will have to be investigated.

NORTH VALLEY INTERCEPTOR DITCH

Design Criteria - Capacity to handle the 1:50 year design flood plus 0.5 metre of freeboard was selected as the minimum cross section for the north valley interceptor ditch. As armoring is not considered to be necessary at this time, some downcutting of the channel grade as a natural armor layer is developed, is anticipated. This will provide additional capacity. Based on test hole data and the boulder-strewn beds of the existing drains, natural armoring is anticipated to be adequate.

Hydraulic Design - As stated above the channel will not be armored. In two locations as shown on Figure 8, the natural creeks will be used to convey the diverted flows. The creeks are presently stable and well armored. Some degrading might occur due to the increased flows but this is not expected to be serious. The steep drops from the diversion ditch to the natural creek bed will probably have to be armored. Field engineering of the drops and armoring is recommended. Since fish passage is not a constraint, there is no limit on the height of the drops.

Diversion structures, field engineered to suit site specific conditions, will be required to divert the flow into the excavated sections. Typical designs are shown on Figure 8.

The interceptor ditch can be joined up with the decant channel located in the north abutment of the cross-valley dam as shown on Figure 10. Again field engineering is recommended to determine final details such as the drop into the decant channel, armoring in the drop area, size and armoring downstream from the confluence and the necessary height of the dike between the decant channel and emergency overflow spillway.

EMERGENCY OVERFLOW SPILLWAYS

Purpose - Rose Creek flows could exceed the design capacity of the diversion channel in the event of:

a) The occurrence of flows greater than the design and contingency floods,

b) The occurrence of a slope failure which partially or wholly blocks the channel,

c) Severe channel icings combined with high flows

and flow into the reservoirs created by the intermediate and cross valley dams. The decant channels would not be sufficient to handle the flow and the dam crest would be overtopped. Depending on the magnitude and duration of the flow, damage to the dam could be significant. To minimize and localize the damage and overflow area respectively, designed emergency overflow sections for the dam are recommended.

Design Criteria - Sizing of the emergency overflow sections is complicated by the unknowns inherent in the events, as stated in the above section, which could lead to the utilization of the overflow. For example a specific return period cannot be assigned to a severe icing condition

combined with spring flow. The diversion channel would remain to have some capacity[13] and significant flows would, due to their velocity and temperature, quickly downcut through the ice. Slope failures, if they occur, will not necessarily coincide with severe flood conditions and if they occur, the entire channel would not necessarily be blocked. It is also assumed that the sloughed material would be cleaned out as quickly as possible. Since some flow over the dam's crest will not create intolerable environmental conditions and damage resulting from it can be reasonably repaired, the emergency overflow need not be designed for the worst possible set of conditions. A 1:50 year design capacity without causing overtopping of the remainder of dam's crest is recommended.

Hydraulic Design - The crest of the overflow section is designed to be non-erodible during the 1:50 year flood. Sections and plans are as illustrated on Figures 9 and 10. Erosion would occur in the unarmored outlet channel downstream from the riprapped section but the length of riprapped apron downstream from the crest is sufficiently long to prevent undermining of the critical section - the crest. Only minor erosion would occur at the intermediate dam as the head difference between the reservoirs of the intermediate and cross valley dams is only approximately 3 metres.

A short stub dike should be constructed downstream from the spillway crest on the dam side to prevent overflows from eroding the downstream dam face. The dike is shown schematically for the intermediate and cross valley dams on Figures 9 and 10 respectively. The precise length of the dike should be field determined.

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13. If a severe icing condition occurred, it is assumed that steps would be taken to remove the ice prior to breakup.

To prevent the seepage of decant through the pervious rock section, upstream layers of till and gravel will be placed to the same height as the rock crest. Some washout of this meterial could occur at high flows. Impact of this widened crest on hydraulic capacity is not significant.

APPENDIX 1

ROSE CREEK - FLOW DATA

ROSE CREEK BELOW FARO CREEK - STATION No. 9BC-3

Location: Lat. 62° 20' 00", long. 133° 24' 00", Yukon Territory, approximately one thousand feet below Faro Creek, and forty-five miles west-northwest of Ross River P.O.
Gauge: Manual.
Period of Record: August and September 1966.
Extremes Recorded: Maximum daily discharge, 45.2 cfs on September 6, 1966. Minimum daily discharge, 34.0 cfs at various times in 1966.
Remarks: Records fair.

Daily Discharge in Cubic Feet per Second for the 1966 Water Year

Day	Oct.	Nov.	Dec.	Jan.	Feb.	March	April	May	June	July	Aug.	Sept.
1.....	-	-	-	-	-	-	-	-	-	-	-	34.0
2.....	-	-	-	-	-	-	-	-	-	-	-	36.0
3.....	-	-	-	-	-	-	-	-	-	-	-	38.4
4.....	-	-	-	-	-	-	-	-	-	-	-	40.2
5.....	-	-	-	-	-	-	-	-	-	-	-	41.4
6.....	-	-	-	-	-	-	-	-	-	-	-	45.2
7.....	-	-	-	-	-	-	-	-	-	-	-	43.6
8.....	-	-	-	-	-	-	-	-	-	-	-	40.8
9.....	-	-	-	-	-	-	-	-	-	-	-	38.4
10.....	-	-	-	-	-	-	-	-	-	-	-	38.4
11.....	-	-	-	-	-	-	-	-	-	-	-	36.6
12.....	-	-	-	-	-	-	-	-	-	-	44.4	35.6
13.....	-	-	-	-	-	-	-	-	-	-	42.0	35.2
14.....	-	-	-	-	-	-	-	-	-	-	41.4	36.0
15.....	-	-	-	-	-	-	-	-	-	-	38.4	37.2
16.....	-	-	-	-	-	-	-	-	-	-	39.6	37.2
17.....	-	-	-	-	-	-	-	-	-	-	41.4	37.2
18.....	-	-	-	-	-	-	-	-	-	-	42.0	36.0
19.....	-	-	-	-	-	-	-	-	-	-	39.0	36.0
20.....	-	-	-	-	-	-	-	-	-	-	40.2	35.6
21.....	-	-	-	-	-	-	-	-	-	-	42.0	35.2
22.....	-	-	-	-	-	-	-	-	-	-	38.4	35.2
23.....	-	-	-	-	-	-	-	-	-	-	38.4	37.8
24.....	-	-	-	-	-	-	-	-	-	-	38.4	37.8
25.....	-	-	-	-	-	-	-	-	-	-	38.4	36.0
26.....	-	-	-	-	-	-	-	-	-	-	35.6	35.6
27.....	-	-	-	-	-	-	-	-	-	-	35.2	35.2
28.....	-	-	-	-	-	-	-	-	-	-	35.2	34.0
29.....	-	-	-	-	-	-	-	-	-	-	34.4	34.0
30.....	-	-	-	-	-	-	-	-	-	-	34.0	34.8
31.....	-	-	-	-	-	-	-	-	-	-	34.0	-
Total	-	-	-	-	-	-	-	-	-	-	-	1,114.6
Mean	-	-	-	-	-	-	-	-	-	-	-	37.2
Acre-feet	-	-	-	-	-	-	-	-	-	-	-	2,210

For the Period..... Maximum daily discharge, 45.2 cfs on September 6
Minimum daily discharge, 34.0 cfs at various times

SOURCE: Water Survey of Canada

ROSE CREEK BELOW FARO CREEK - STATION No. 9BC-3

Location: Lat. 62° 20' 30", long. 133° 24' 30", (revised), Yukon Territory, approximately one thousand feet below Faro Creek and forty-five miles west-northwest of Ross River P.O.
Gauge: Manual.
Period of Record: Mainly continuous, August and September 1966 and December 1966 to September 1967.
Extremes Recorded: Maximum daily discharge, 1,150 cfs on May 31, 1967. Minimum daily discharge, 6.1 cfs on March 10, 1967.
Remarks: Records fair.

Daily Discharge in Cubic Feet per Second for the 1967 Water Year

Day	Oct.	Nov.	Dec.	Jan.	Feb.	March	April	May	June	July	Aug.	Sept.
1.....	-	-	-	-	-	-	-	-	1,070	105	108	41.0
2.....	-	-	-	-	-	-	-	-	810	99.0	93.0	38.2
3.....	-	-	-	-	-	-	-	-	616	155	87.8	79.4
4.....	-	-	-	-	-	-	6.3 b	-	732	110	82.5	60.5
5.....	-	-	-	-	-	-	-	-	801	129	77.2	59.5
6.....	-	-	12.3 b	-	-	-	-	-	783	121	77.2	62.6
7.....	-	-	-	-	-	-	-	-	512	100	78.3	68.8
8.....	-	-	12.4 b	-	-	-	-	-	323	90.9	82.5	78.3
9.....	-	-	-	-	-	-	-	-	334	90.9	77.2	93.0
10.....	-	-	-	-	-	6.1 b	-	-	591	105	77.2	79.4
11.....	-	-	-	-	-	-	-	-	543	95.4	74.1	71.0
12.....	-	-	-	-	-	-	-	-	292	115	77.2	79.4
13.....	-	-	-	-	-	-	-	-	246	93.0	71.0	71.0
14.....	-	-	-	-	-	-	-	-	292	94.2	68.8	68.8
15.....	-	-	-	-	-	-	-	-	431	135	64.6	72.0
16.....	-	-	-	-	-	-	-	-	467	115	61.5	73.0
17.....	-	-	-	-	-	-	-	-	414	110	59.5	71.0
18.....	-	-	-	-	-	-	-	-	467	105	59.5	77.2
19.....	-	-	-	-	-	-	-	-	337	105	61.5	80.4
20.....	-	-	-	-	-	-	-	-	273	82.5	53.5	72.0
21.....	-	-	-	-	-	-	-	-	278	101	46.8	69.9
22.....	-	-	-	-	-	-	-	-	204	72.0	51.5	66.8
23.....	-	-	-	-	-	-	-	-	200	80.4	53.5	67.8
24.....	-	-	-	-	-	-	-	-	180	86.7	45.8	66.8
25.....	-	-	-	-	-	-	-	-	169	82.5	42.0	64.6
26.....	-	-	-	-	-	-	-	-	196	86.7	42.0	61.5
27.....	-	-	-	-	-	-	-	-	202	82.5	36.3	51.5
28.....	-	-	-	-	-	-	-	792	200	83.6	32.5	61.5
29.....	-	-	-	-	-	-	-	595	163	105	34.4	59.5
30.....	-	-	-	-	-	-	-	1,110	121	173	36.3	57.5
31.....	-	-	-	-	-	-	-	1,150	-	163	34.4	-
Total	-	-	-	-	-	-	-	-	12,247	3,272.3	1,947.6	2,023.9
Mean	-	-	-	-	-	-	-	-	408	106	62.8	67.5
Acre-feet	-	-	-	-	-	-	-	-	24,290	6,490	3,860	4,010

For the Period..... Maximum daily discharge, 1,150 cfs on May 31
Minimum daily discharge, 6.1 cfs on March 10

b - Ice conditions.

SOURCE: Water Survey of Canada

ROSE CREEK BELOW FARO CREEK - STATION No. 09BC003

Location: Lat. 62° 20' 30", long. 133° 24' 30", Yukon Territory, approximately one thousand feet below Faro Creek and forty-five miles west-northwest of Ross River P.O.
Gauge: Manual.
Period of Record: Mainly continuous, August and September 1966 and December 1966 to December 1968.
Extremes Recorded: Maximum daily discharge, 1,150 cfs on May 31, 1967.
Minimum daily discharge, 6.1 cfs on March 10, 1967.
Remarks: Records fair. Since October 4, 1968, flows have been affected by dam on south fork of Rose Creek.

Daily Discharges in Cubic Feet per Second for the Period October to December 1967

Day	Oct.	Nov.	Dec.	Day	Oct.	Nov.	Dec.	Day	Oct.	Nov.	Dec.
1	56.5	23.9	18.2	11	49.6	22.3	16.2	21	30.7	20.3	14.0
2	55.5	23.7	18.1	12	49.6	22.1	16.0	22	27.1	20.1	13.8
3	42.0	23.5	17.9	13	46.8	21.9	15.8	23	26.8	19.9	13.6
4	42.0	23.3	17.7	14	40.1	21.7	15.6	24	26.5	19.7	13.4
5	40.1	23.2	17.4	15	40.1	21.5	15.2	25	26.0	19.5	13.1
6	41.0	23.1	17.2	16	41.0	21.3	15.0	26	25.7	19.3	13.0
7	40.1	23.0	17.0	17	40.1	21.1	14.8	27	25.4	19.1	12.9
8	61.5	22.9	16.8	18	32.5	20.9	14.6	28	25.1	18.9	12.8
9	40.1	22.7	16.6	19	32.5	20.7	14.4	29	24.8	18.9	12.6
10	42.0	22.5	16.4	20	32.5	20.5	14.2	30	24.5	18.9	12.3
								31	24.0	-	12.0
								Total	1,152.2	640.4	468.6
								Mean	37.2	21.3	15.1
								Acre-feet	2,290	1,270	929

Ice conditions October 22 to December 31.

Daily Discharges in Cubic Feet per Second for the Year 1968

Day	Jan.	Feb.	March	April	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
1	11.8	8.0	7.1	7.1	12.4	212 E	152	261	218	115	36.5	28.0
2	11.6	7.9	7.1	7.1	13.5	230 E	139	316	280	111	36.1	27.7
3	11.4	7.8	7.1	7.1	14.5	205 E	133	316	229	90.0	35.7	27.4
4	11.2	7.7	7.1	7.2	16.0	185 E	114	351	253	70.0	35.3	27.0
5	11.0	7.7	7.1	7.2	17.5	181	198	236	241	57.0	35.0	26.5
6	10.9	7.6	7.1	7.2	19.5	185 E	548	170 E	218	54.0	34.7	26.2
7	10.8	7.5	7.1	7.2	22.0	190 E	372	133	174	53.0	34.4	25.9
8	10.7	7.4	7.1	7.3	27.0	204 E	211	115	196	47.1	34.1	25.6
9	10.6	7.4	7.1	7.3	32.0 E	220 E	178	106	210 E	46.0	33.8	25.3
10	10.4	7.4	7.1	7.3	35.8	250 E	168	97.0	220 E	45.0	33.8	25.0
11	10.3	7.4	7.1	7.3	44.0 E	236 E	159	88.0 E	234	44.0	33.6	24.6
12	10.2	7.3	7.1	7.4	52.0 E	228 E	140	84.0 E	227	43.0	33.4	24.2
13	10.0	7.3	7.1	7.4	60.0 E	213	170	83.0	211	42.0	33.0	23.8
14	9.8	7.2	7.1	7.5	64.0 E	201	150	80.6	198	42.0	32.8	23.4
15	9.7	7.2	7.1	7.6	68.0 E	190 E	160	78.2	181	42.0	32.5	23.0
16	9.6	7.2	7.1	7.7	70.0 E	210 E	150	84.0 E	202	42.0	32.3	22.2
17	9.5	7.2	7.1	7.8	74.6	220 E	150	90.0 E	200	42.0	32.1	22.4
18	9.4	7.1	7.1	7.9	52.0	240 E	176	96.0 E	200	41.0	31.9	22.1
19	9.2	7.1	7.1	8.0	220 E	260 E	166	100 E	198	41.0	31.7	21.8
20	9.0	7.1	7.1	8.2	270 E	295 E	160	108	183	40.0	31.5	21.5
21	8.9	7.1	7.1	8.5	330 E	302	140	106	168	39.5	31.2	21.1
22	8.8	7.1	7.1	8.8	390 E	274	129	105	168	39.0	30.9	20.7
23	8.7	7.1	7.1	9.0	440	236	131	97.0	160	38.5	30.6	20.3
24	8.6	7.1	7.1	9.2	348	220 E	114	148	156	38.0	30.3	19.9
25	8.5	7.1	7.1	9.5	300 E	206 E	114	148	148	38.0	30.0	19.5
26	8.5	7.1	7.1	10.0	280 E	200 E	106	158	146	38.0	29.3	19.0
27	8.4	7.0	7.1	10.5	260 E	194 E	98.5	168	139	38.0	29.2	18.5
28	8.3	7.0	7.1	11.0	230 E	185 E	98.5	168	142	37.5	29.1	18.0
29	8.2	7.0	7.1	11.5	215 E	178	84.4	168	131	37.0	28.8	17.5
30	8.1	-	7.1	11.9	210 E	160 E	91.4	180 E	122	37.0	28.5	17.0
31	8.1	-	7.1	-	205 E	-	114	198 E	-	37.0	-	16.5
Total	300.2	212.1	220.1	248.7	4,392.8	6,510	5,014.8	4,636.8	5,753	1,524.6	972.1	701.6
Mean	9.7	7.3	7.1	8.3	142	217	162	150	192	49.2	32.4	22.6
Acre-feet	595	421	437	493	8,710	12,900	9,950	9,200	11,400	3,020	1,930	1,390

For the Year 1968..... Maximum daily discharge, 548 cfs on July 6
Minimum daily discharge, 7.0 cfs, February 27 to 29
Mean discharge, 83.3 cfs
Total discharge, 60,400 ac-ft

Ice conditions January 1 to May 8 and October 5 to December 31. E - Estimated.

SOURCE: Water Survey of Canada

ROSE CREEK BELOW FARO CREEK - STATION NO. 09BC003

DAILY DISCHARGE IN CUBIC FEET PER SECOND FOR 1969

DAY	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	DAY
1	16.5 B	7.6 B	7.6 B	7.4 B	14.3 B	117	42.1	---	---	---	---	---	1
2	15.8 B	7.6 B	7.5 B	7.4 B	15.0 B	84.4	45.7	---	---	---	---	---	2
3	15.1 B	7.6 B	7.5 B	7.4 B	15.7 B	75.8	49.3	---	---	---	---	---	3
4	14.4 B	7.6 B	7.5 B	7.4 B	16.5 B	127	52.0	---	---	---	---	19.4	4
5	13.7 B	7.6 B	7.5 B	7.4 B	17.3 B	160	63.0	---	---	---	---	---	5
6	13.3 B	7.6 B	7.5 B	7.4 B	19.4 B	179	103	---	---	---	---	---	6
7	12.9 B	7.6 B	7.5 B	7.4 B	19.3 B	216	122	---	---	---	---	---	7
8	12.5 B	7.6 B	7.5 B	7.5 B	21.0 B	200	122	---	---	---	---	---	8
9	12.0 B	7.6 B	7.5 B	7.6 B	23.0 B	239	90.0	---	---	---	---	---	9
10	11.5 B	7.6 B	7.5 B	7.6 B	27.6	354	72.2	---	---	---	---	---	10
11	11.2 B	7.6 B	7.4 B	7.6 B	38.5	277	72.2	---	---	---	---	---	11
12	10.9 B	7.6 B	7.4 B	7.7 B	46.6	213	179	---	---	---	---	---	12
13	10.6 B	7.6 B	7.4 B	7.8 B	49.3	202	198	---	---	---	---	---	13
14	10.2 B	7.6 B	7.4 B	7.9 B	58.3	193	139	---	---	---	---	---	14
15	9.8 B	7.6 B	7.4 B	8.0 B	65.0	174	102	---	---	---	---	---	15
16	9.6 B	7.6 B	7.4 B	8.2 B	95.6	164	94.2	---	---	---	---	---	16
17	9.4 B	7.6 B	7.4 B	8.4 B	105	140	84.4	---	---	---	---	---	17
18	9.2 B	7.6 B	7.4 B	8.6 B	109	127	98.5	---	---	---	---	---	18
19	9.0 B	7.6 B	7.4 B	8.7 B	98.5	139	81.8	---	---	---	---	---	19
20	8.7 B	7.6 B	7.4 B	8.8 B	105	111	73.4	---	---	---	---	---	20
21	8.7 B	7.6 B	7.3 B	9.1 B	144	90.0	68.0	---	---	---	---	---	21
22	8.6 B	7.6 B	7.3 B	9.3 B	196	77.0	66.0	---	---	---	---	---	22
23	8.4 B	7.6 B	7.3 B	9.6 B	277	39.4	71.0	---	---	---	---	---	23
24	8.2 B	7.6 B	7.3 B	10.0 B	305	53.8	69.0	---	---	---	---	---	24
25	8.0 B	7.6 B	7.3 B	10.5 B	328	50.2	90.0	---	---	---	---	---	25
26	7.9 B	7.6 B	7.3 B	11.0 B	158	47.5	124	---	---	---	---	---	26
27	7.8 B	7.6 B	7.3 B	11.7 B	81.8	45.7	103	---	---	---	---	---	27
28	7.7 B	7.6 B	7.3 B	12.4 B	56.5	43.9	90.0	---	---	---	---	---	28
29	7.6 B		7.3 B	13.1 B	61.0	41.2	81.8	---	---	---	---	---	29
30	7.6 B		7.3 B	13.8 B	102	42.1	76.0 E	---	---	---	---	---	30
31	7.6 B		7.4 B		139		74.0 E	---		---		---	31
TOTAL	324.4	212.8	229.5	266.7	2808.2	4023.0	2796.6	---	---	---	---	---	TOTAL
MEAN	10.5	7.6	7.4	8.9	90.6	134	90.2	---	---	---	---	---	MEAN
AC-FT	643	422	455	529	5570	7980	5550	---	---	---	---	---	AC-FT
MAX	16.5	7.6	7.6	13.8	328	354	198	---	---	---	---	---	MAX
MIN	7.6	7.6	7.3	7.4	14.3	39.4	42.1	---	---	---	---	---	MIN

SUMMARY FOR THE MONTHS JAN TO JUL

MEAN DISCHARGE, 50.3 CFS
TOTAL DISCHARGE, 21100 AC-FT
MAXIMUM DAILY DISCHARGE, 354 CFS ON JUN 10
MINIMUM DAILY DISCHARGE, 7.3 CFS ON MAR 21

TYPE OF GAUGE - MANUAL
LOCATION - LAT 62 20 30 N
LONG 133 24 30 W

B-ICE CONDITIONS
E-ESTIMATED

REGULATED

ROSS RIVER AT ROSS RIVER - STATION NO. 09BA001

DAILY DISCHARGE IN CUBIC FEET PER SECOND FOR 1969

DAY	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	DAY
1	312 B	184 B	181 B	182 B	630 B	5810	1910 E	3660	3890	2650	320 B	600 B	1
2	304 B	184 B	182 B	184 B	680 B	5570	2100 E	4010	3740	2550	350 B	600 B	2
3	296 B	184 B	182 B	186 B	740 B	5540	2300 E	4180	3710	2450	400 B	594 B	3
4	288 B	184 B	181 B	189 B	820 B	5740	2500 E	4090	4070	2400	430 B	570 B	4
5	278 B	184 B	181 B	191 B	920 B	5910	2750 E	3820	4500	2340	440 B	550 B	5
6	270 B	184 B	180 B	195 B	1010 B	6320	2930 E	3560	4740	2270	435 B	520 B	6
7	262 B	184 B	180 B	198 B	1110 B	7390	3050 E	3380	4740	2230	426 B	490 B	7
8	254 B	184 B	180 B	200 B	1270 B	7930	3340 E	3250	4650	2230	415 B	470 B	8
9	246 B	184 B	180 B	205 B	1450 B	8360	3700 E	2700	4500	2250	407 B	460 B	9
10	240 B	184 B	179 B	212 B	1650 B	8900	4150 E	2880	4440	2300	400 B	450 B	10
11	234 B	184 B	179 B	216 B	1810	8980 A	5000 E	3000	4620	2320	410 B	450 B	11
12	228 B	184 B	179 B	222 B	2060	8480 E	6350 E	3130	4800	2270	430 B	445 B	12
13	222 B	184 B	178 B	228 B	2500	7700 E	8300 E	3300	5370	2230	430 B	440 B	13
14	216 B	184 B	178 B	235 B	3280	6950 E	10400 E	3300	6490	2190	420 B	430 B	14
15	212 B	184 B	178 B	243 B	3870	6380 E	11600 A	3980	6860	2090	380 B	425 B	15
16	210 B	183 B	178 B	251 B	4150	5900 E	9540	4210	6590	1920	365 B	420 B	16
17	206 B	183 B	178 B	261 B	4270	5200 E	7570	4470	6080	1770	360 B	395 B	17
18	202 B	183 B	178 B	274 B	4740	4650 E	6350	4650	5540	1830	375 B	380 B	18
19	198 B	183 B	178 B	289 B	5040	4200 E	5370	4620	5100	1830	415 B	370 B	19
20	196 B	183 B	178 B	305 B	5190	3900 E	4680	4500	4710	1710	460 B	350 B	20
21	194 B	183 B	178 B	320 B	5250	3570 E	4120	4560	4380	1690	500 B	335 B	21
22	192 B	183 B	177 B	337 B	5160	3340 E	3710	4950	4090	1350	520 B	320 B	22
23	190 B	183 B	177 B	355 B	5100	3100 E	3380	5470	3840	908	560 B	310 B	23
24	189 B	183 B	178 B	375 B	5470	2850 E	3180	5640	3660	654	550 B	305 B	24
25	188 B	182 B	178 B	391 B	6350	2650 E	3030	5430	3480	524	560 B	300 B	25
26	188 B	181 B	178 B	420 B	7170	2450 E	3000	5070	3350	372	580 B	300 B	26
27	187 B	181 B	179 B	450 B	7390	2250 E	3150	4740	3230	290 A	580 B	298 B	27
28	186 B	180 B	179 B	488 B	7070	2150 E	3400	4590	3030	288 B	580 B	298 B	28
29	185 B		179 B	530 B	6790	2010 E	3690	4530	2850	290 B	600 B	298 B	29
30	185 B		180 B	588 B	6280	1940 E	3710	4360	2750	295 B	590 B	298 B	30
31	184 B		180 B		6010		3630	4120		300 B		300 B	31
TOTAL	6942	5131	5551	8720	115230	156120	141890	128150	133800	50791	13688	12773	TOTAL
MEAN	224	183	179	291	3720	5200	4580	4130	4460	1640	456	412	MEAN
AC-FT	13800	10200	11000	17300	229000	310000	281000	254000	265000	101000	27200	25300	AC-FT
MAX	312	184	182	588	7390	8980	11600	5640	6860	2650	600	600	MAX
MIN	184	180	177	182	630	1940	1910	2700	2750	288	320	298	MIN

SUMMARY FOR THE YEAR 1969

MEAN DISCHARGE, 2130 CFS
TOTAL DISCHARGE, 1540000 AC-FT
MAXIMUM DAILY DISCHARGE, 11600 CFS ON JUL 15
MINIMUM DAILY DISCHARGE, 177 CFS ON MAR 22

TYPE OF GAUGE - RECORDING
LOCATION - LAT 61 59 40 N
LONG 132 22 40 W
DRAINAGE AREA 2800 SQ MILES

A-MANUAL GAUGE
B-ICE CONDITIONS
E-ESTIMATED

NATURAL FLOW

SOURCE: Water Survey of Canada

ROSE CREEK BELOW FARO CREEK - STATION NO. 09BC003

MONTHLY AND ANNUAL MEAN DISCHARGES IN CUBIC FEET PER SECOND FOR THE PERIOD OF RECORD

YEAR	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	MEAN	YEAR
1966	---	---	---	---	---	---	---	---	37.2	---	---	---	---	1966
1967	---	---	---	---	---	408	106	62.8	67.5	37.2	21.3	15.1	---	1967
1968	9.7	7.3	7.1	8.3	142	217	162	150	192	49.2	32.4	22.6	83.3	1968
1969	10.5	7.6	7.4	8.9	90.6	134	90.2	---	---	---	---	---	---	1969
MEAN	10.1	7.5	7.3	8.6	116	253	119	106	98.9	43.2	26.9	18.9	83.3	MEAN

LOCATION - LAT 62 20 30 N
LONG 133 24 30 W

REGULATED

ROSE CREEK BELOW FARO CREEK - STATION NO. 09BC003

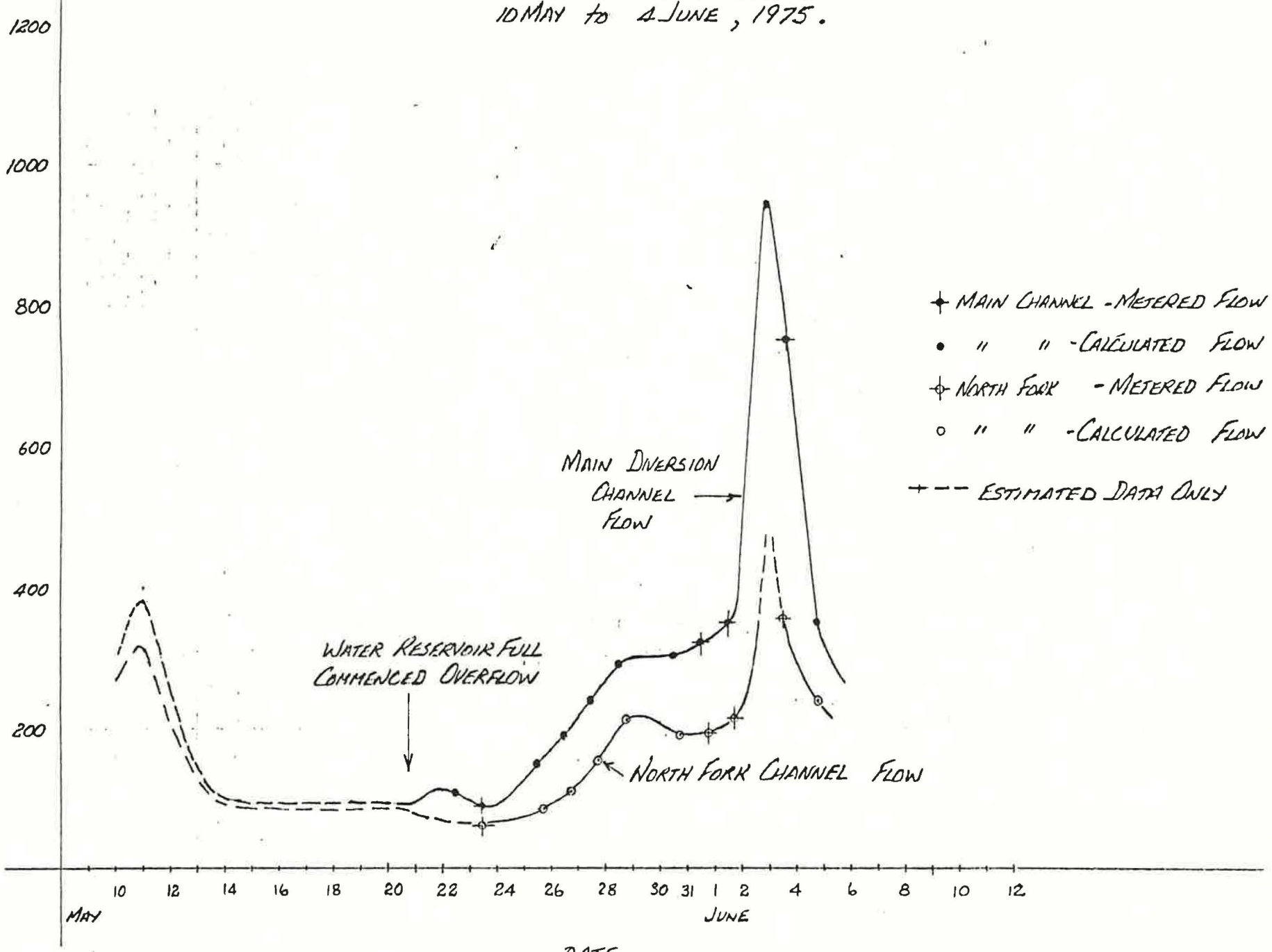
ANNUAL EXTREMES OF DISCHARGE IN CFS AND ANNUAL TOTAL DISCHARGE IN AC-FT FOR THE PERIOD OF RECORD

YEAR	MAXIMUM INSTANTANEOUS DISCHARGE	MAXIMUM DAILY DISCHARGE	MINIMUM DAILY DISCHARGE	TOTAL DISCHARGE	YEAR
1966	---	---	---	---	1966
1967	---	1150 CFS ON MAY 31 *	6.1 CFS ON MAR 10 *	---	1967
1968	---	548 CFS ON JUL 6	7.0 CFS ON FEB 27	60400 AC-FT	1968
1969	---	354 CFS ON JUN 10	7.3 CFS ON MAR 21	---	1969
		* - EXTREME RECORDED FOR THE PERIOD OF RECORD		60400 AC-FT	MEAN

SOURCE: Water Survey of Canada

FLOW HYDROGRAPH

10 MAY to 4 JUNE, 1975.



SOURCE: Sigma Resource Consultants Ltd.

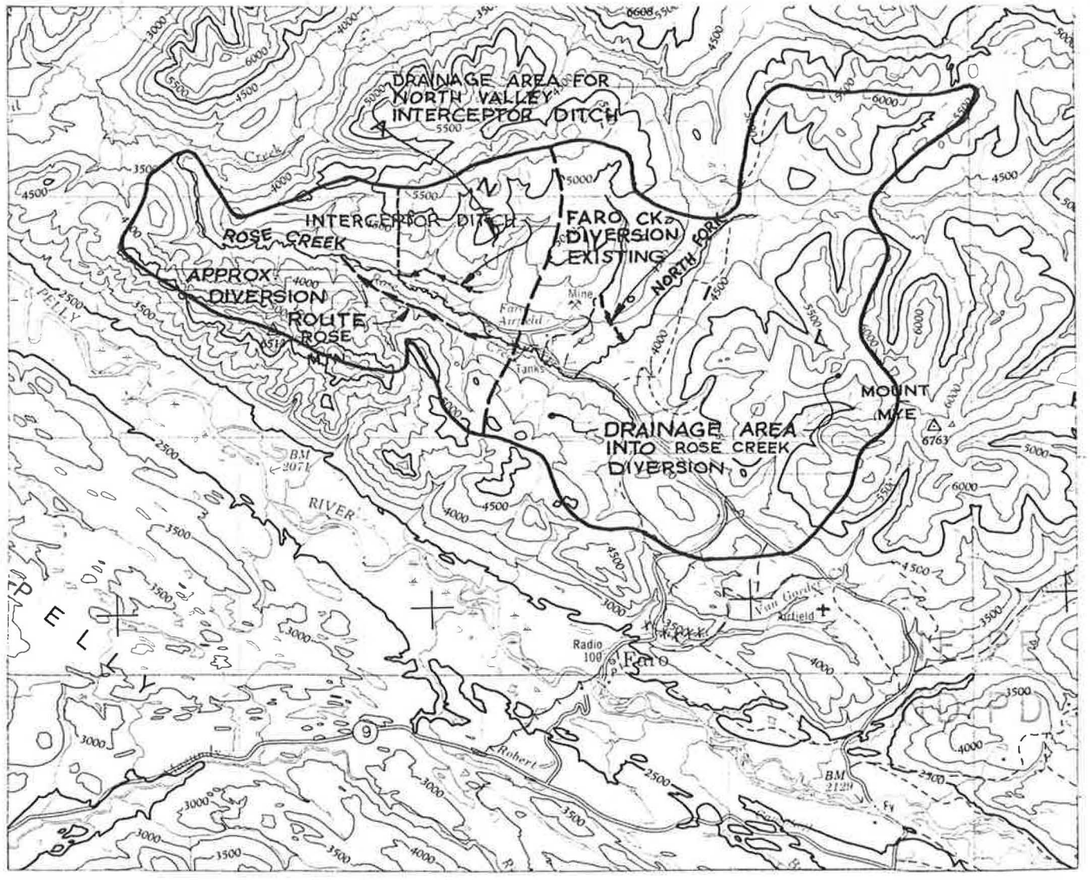
FIGURE 2

DWG. DATE 10 JUNE 75 SUBJECT

BY DATE SHEET NO. OF

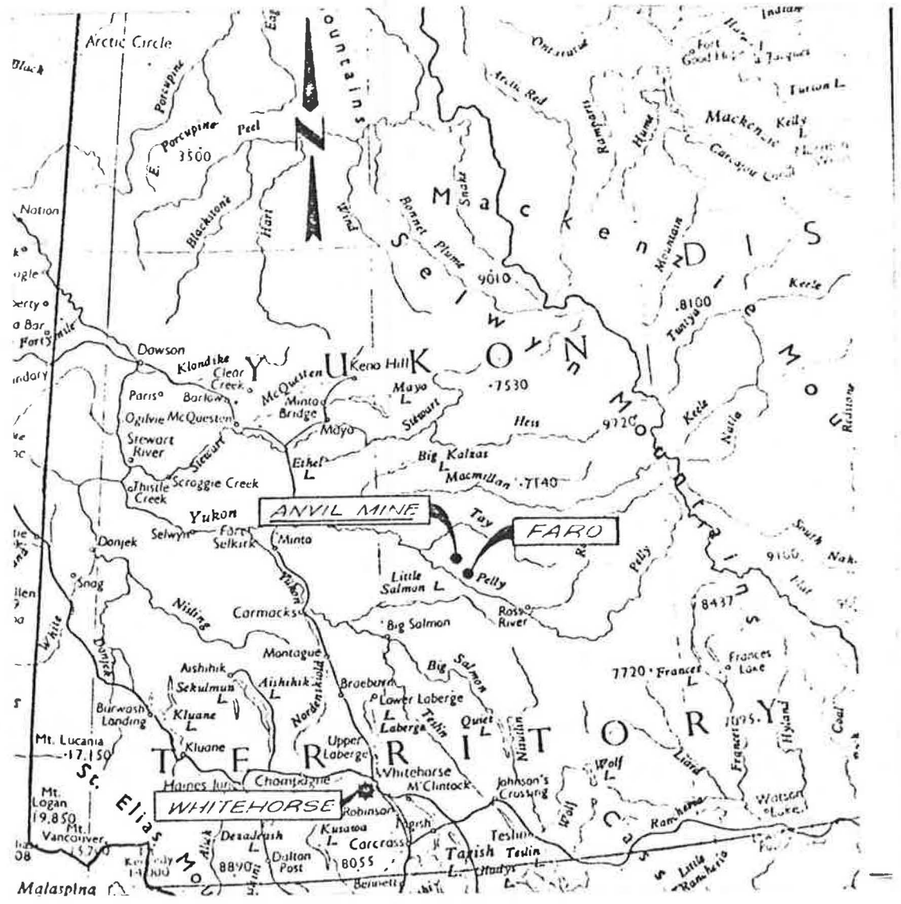
JOB NO.

FIGURES



DRAINAGE AREA

SCALE = 1: 250 000



KEY PLAN

SCALE = 1 cm = 56 km

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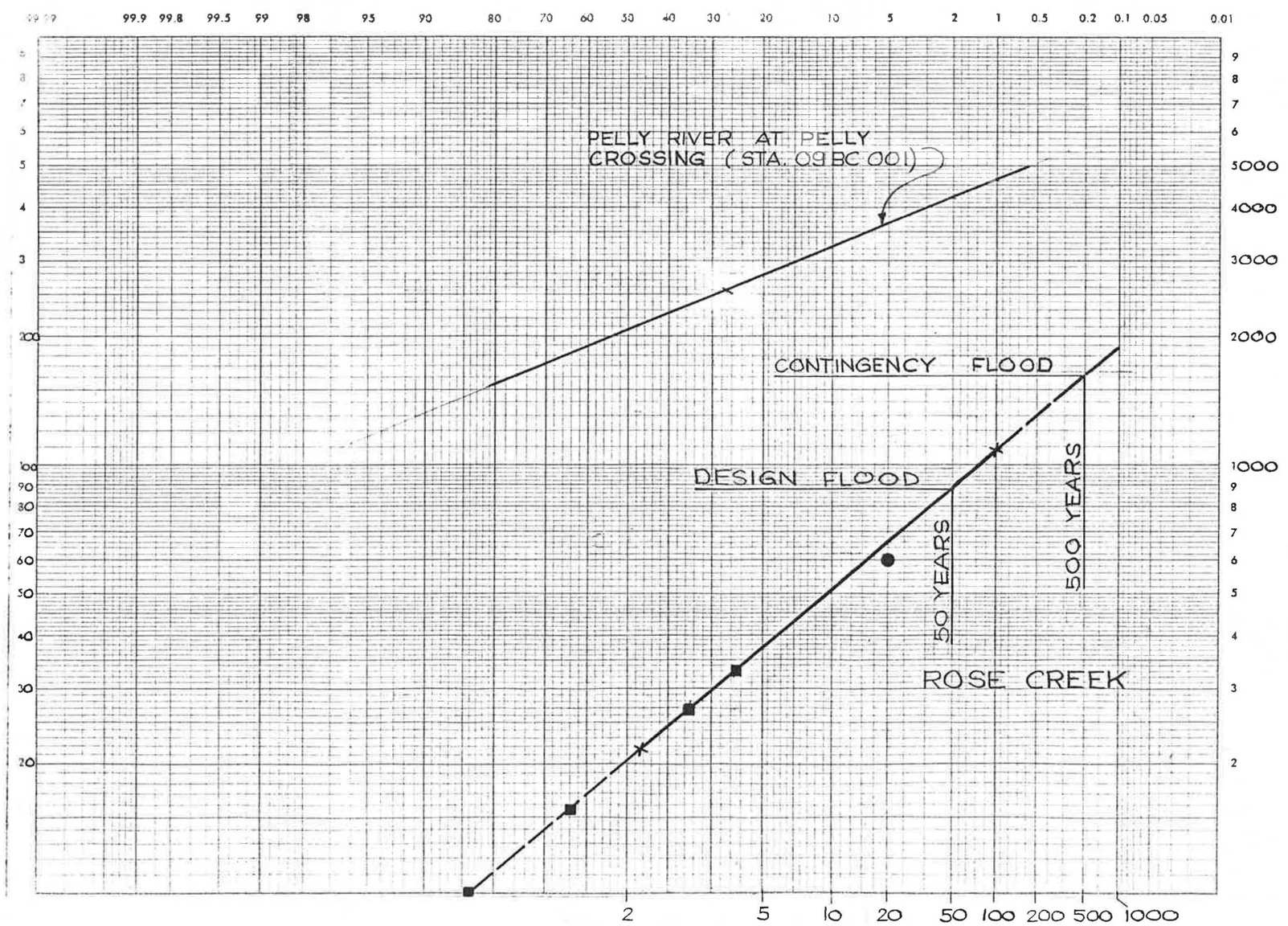
W.M. VELDMAN
22992

FIGURE 1

GOLDER ASSOCIATES		
DOWN VALLEY TAILINGS DISPOSAL		
KEY PLAN AND DRAINAGE AREAS		
HYDROCON ENGINEERING (Continental) LTD.		
Drawn: U.G.	Date: APR. 80	Job. No. 8-7.1

PROBABILITY OF EXCEEDANCE



ROSE CREEK MAXIMUM INSTANTANEOUS DISCHARGE IN $m^3/sec.$

PELLY RIVER AT PELLY CROSSING MAXIMUM INSTANTANEOUS FLOW IN $m^3/SEC.$

LEGEND

X FREQUENCY CURVE POINTS AS DETERMINED FROM REGIONAL CORRELATIONS.

● 20 YEAR FLOOD COMPUTED BY MECO

■ RECORDED MAXIMUM INSTANTANEOUS FLOWS

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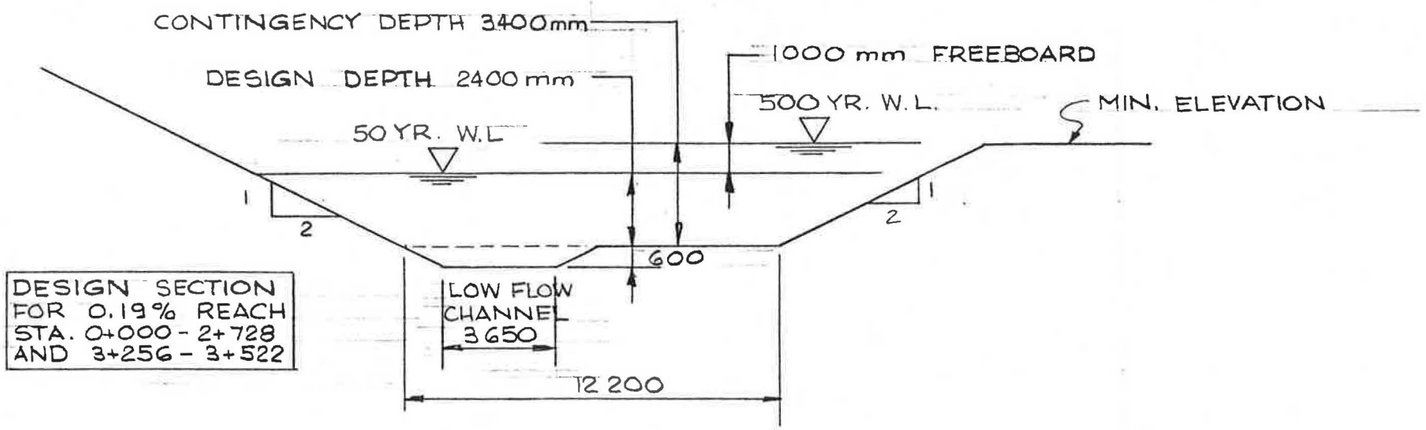
FIGURE 2

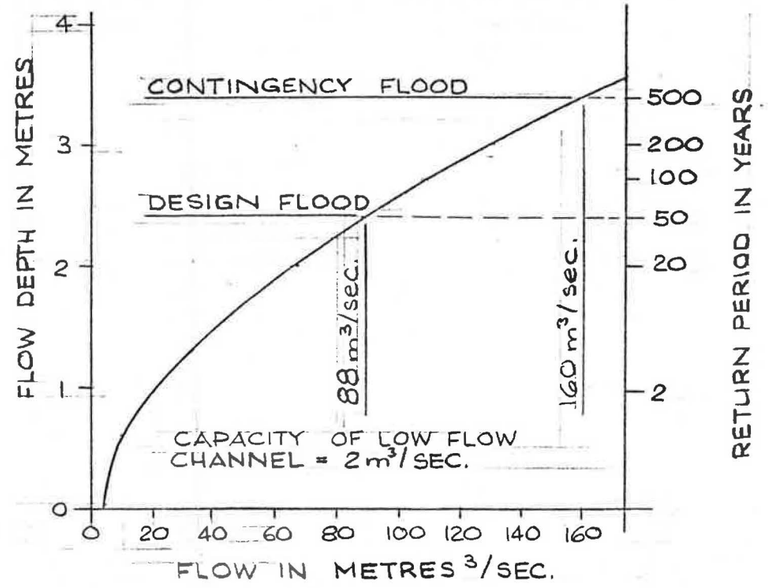
GOLDER ASSOCIATES		
DOWN VALLEY TAILINGS DISPOSAL		
ROSE CREEK DIVERSION FLOW - FREQUNCY RELATIONSHIP		
HYDROCON ENGINEERING (Continental) LTD.		
Drawn: U.G.	Date: APR. 80	Job. No. 8-7.1



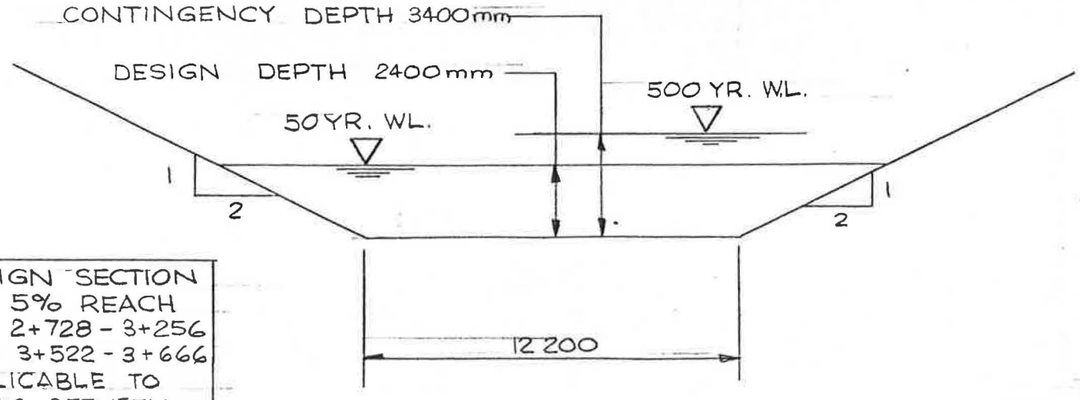
DESIGN SECTION FOR 0.19% REACH STA. 0+000 - 2+728 AND 3+256 - 3+522

CROSS - SECTION

SCALE = 1: 200



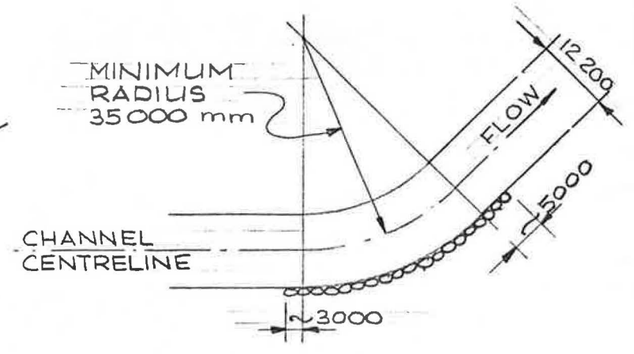
FLOW VERSUS DEPTH AND RETURN PERIOD



DESIGN SECTION FOR 5% REACH STA. 2+728 - 3+256 AND 3+522 - 3+666 (APPLICABLE TO AREAS BETWEEN DROP WEIRS. SEE FIG. 5)

CROSS - SECTION

SCALE : 1: 200



DESIGN OF CHANNEL CURVE

SCALE = 1: 1000

NOTES RE CURVES:

1. OUTSIDE OF CURVES TO BE ARMORED TO THE LIMITS SHOWN USING ROCK $D_{50} \geq 200$ PLACED TO A THICKNESS OF 400 mm.
2. INSIDE OF BEND TO BE ARMORED AS PER FIG. 4
3. FOR CURVES LESS THAN 5°, ARMORING AS PER NORMAL REQUIREMENTS OF FIG. 4.

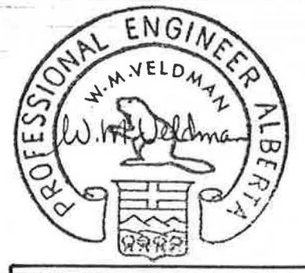
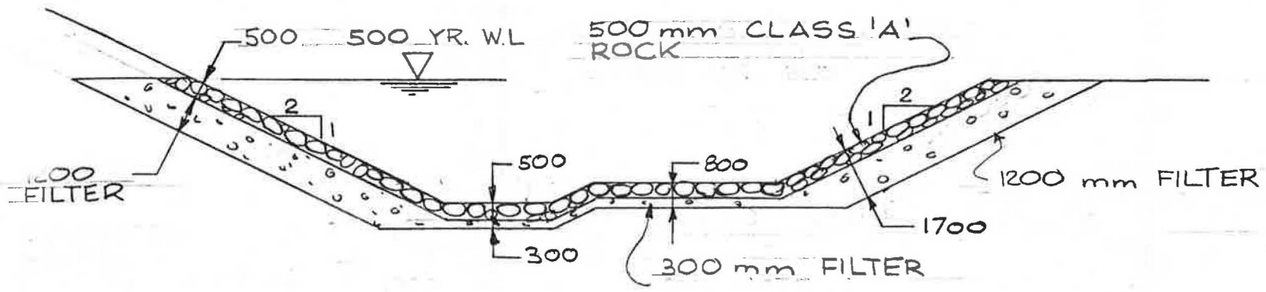


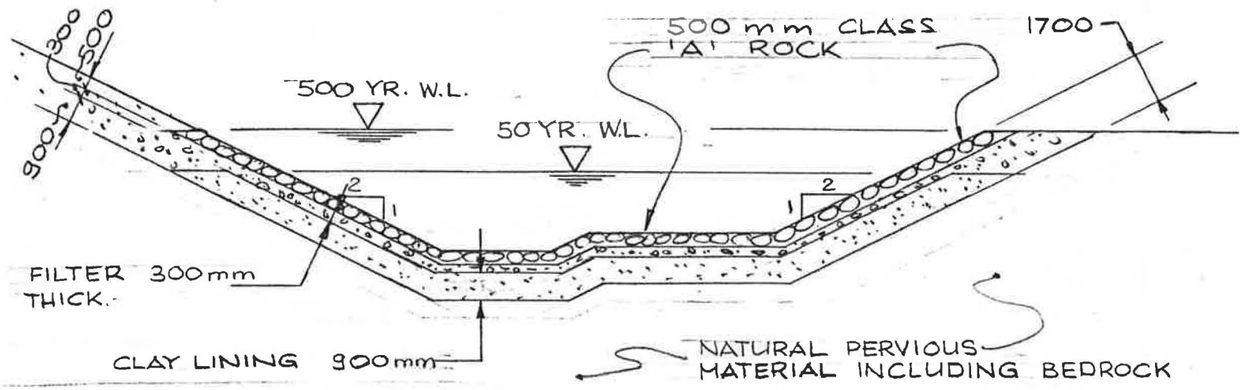
FIGURE 3

GOLDER ASSOCIATES		
DOWN VALLEY TAILINGS DISPOSAL		
ROSE CREEK DIVERSION HYDRAULIC DESIGN CHARACTERISTICS		
HYDROCON ENGINEERING (Continental) LTD.		
DRAWN: LIG	DATE: APR 80	JOB: 8 - 7.1



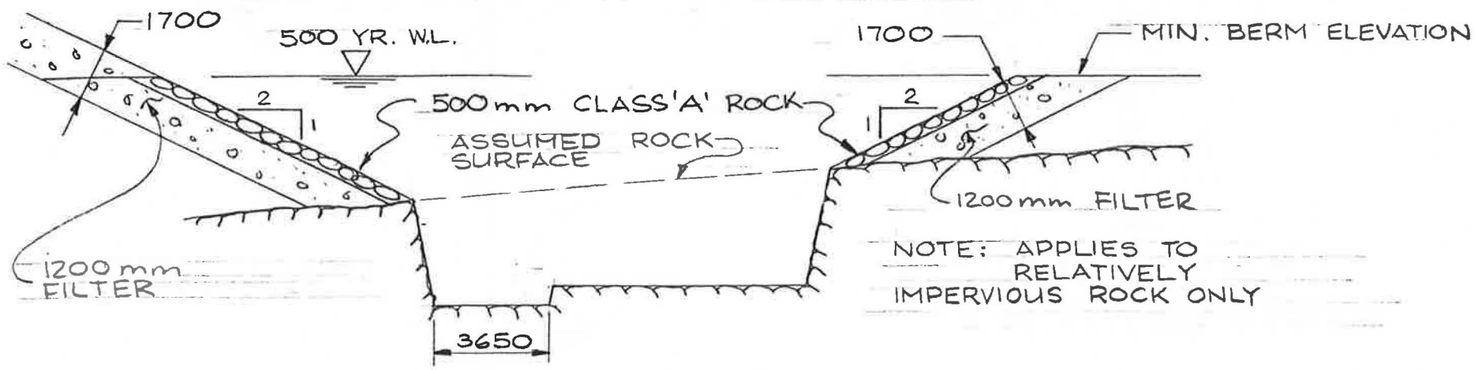
ALTERNATIVE 1 TYPICAL SECTION

SCALE = 1: 200



ALTERNATIVE 2 CHANNEL IN PERVIOUS MATERIAL

SCALE = 1: 200



ALTERNATIVE 3 CHANNEL EXCAVATED IN ROCK

SCALE = 1: 200

NOTES:

1.) SEE NOTE 3 OF FIG. 5 FOR FILTER SPECIFICATIONS.
2.) RIPRAP SPECIFICATIONS AS PER NOTE 4 OF FIG. 5.
3.) SEE FIG. 3 FOR ARMORING DETAILS ON OUTSITE OF BEND.
4.) PLASTIC LINING MAY BE USED AS SUBSTITUTE TO TILL (ALTERNATIVE 2)

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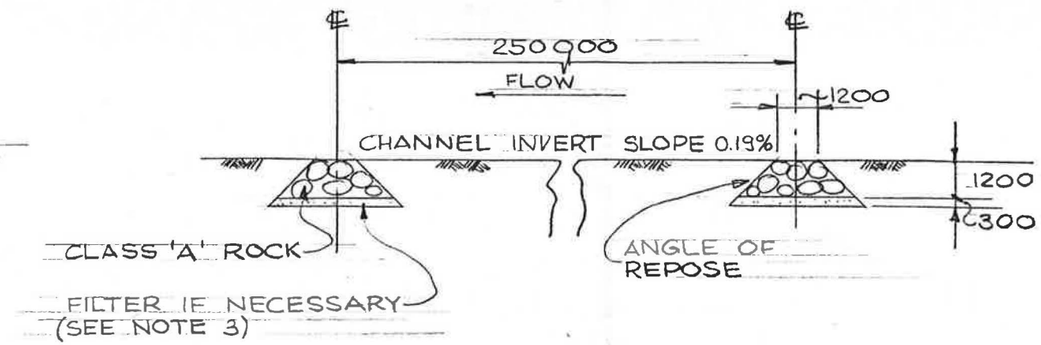
REV. 1
FIGLIRE 4

GOLDER ASSOCIATES
DOWN VALLEY TAILINGS DISPOSAL
ROSE CREEK DIVERSION ALTERNATIVE ARMORING METHODS
HYDROCON ENGINEERING (Continental) LTD.
DRAWN: UG | DATE: APR. 80 | JOB. 8-7.1

REV. 1 RIPRAP CHANGED JUNE 80 UG. D.C.

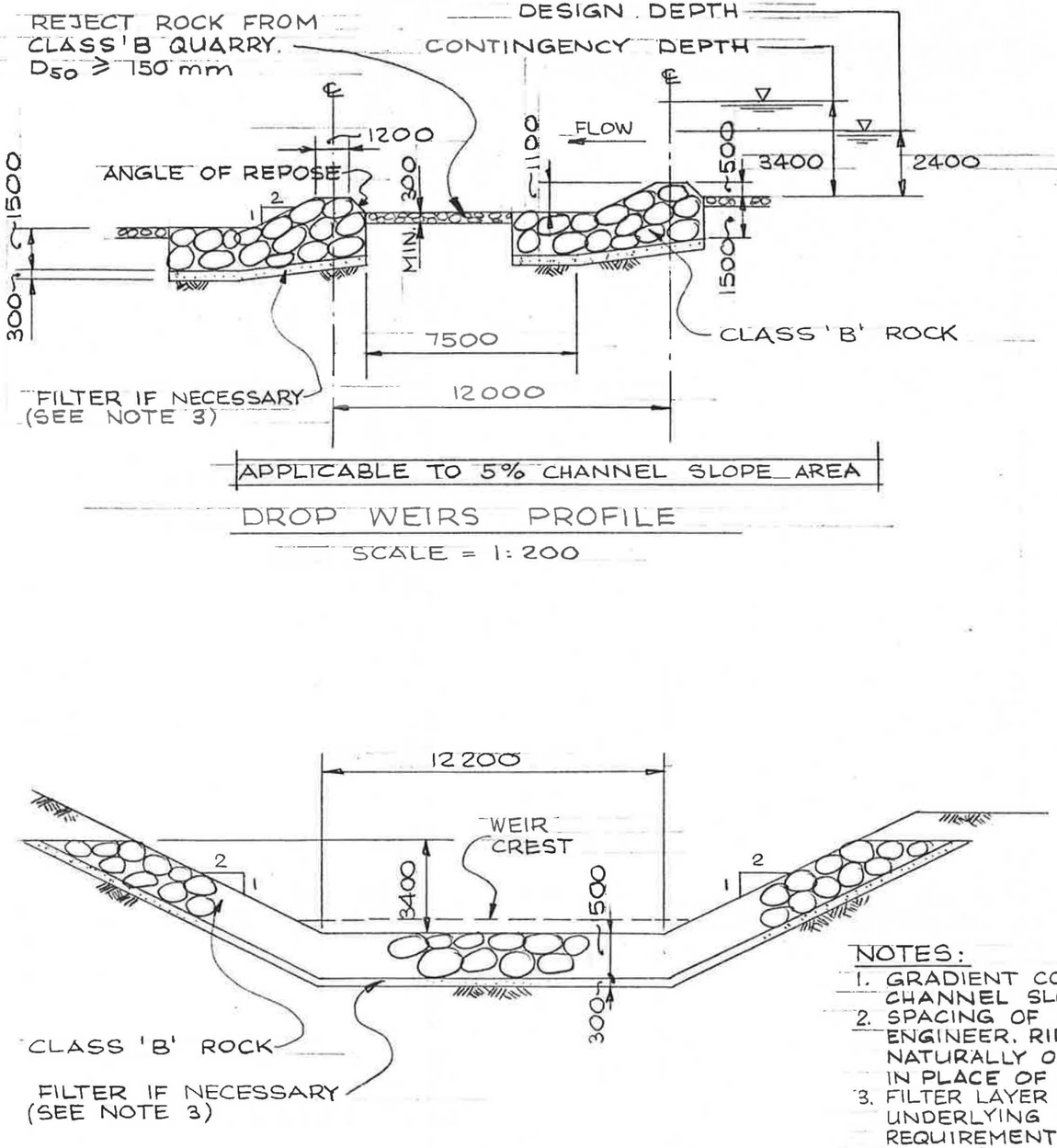
DROP WEIRS PROFILE

SCALE = 1:200



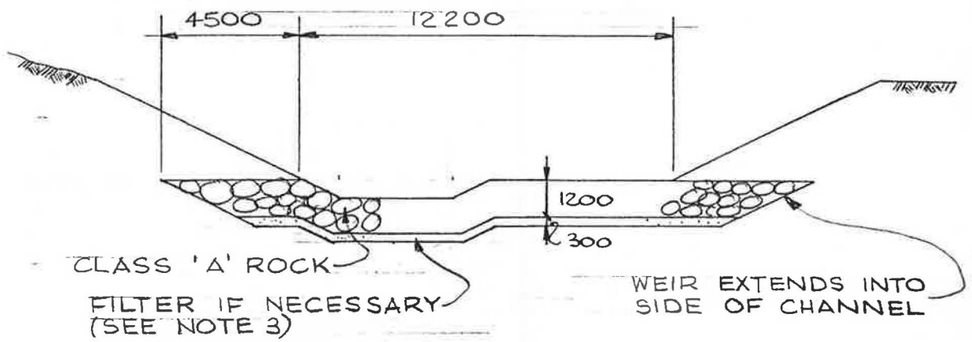
GRADIENT CONTROL SILLS PROFILE

SCALE = 1:200



DROP WEIRS CROSS-SECTION

SCALE = 1:200



GRADIENT CONTROL SILLS CROSS-SECTION

SCALE = 1:200

NOTES:

1. GRADIENT CONTROL SILLS TO BE PLACED ON 0.19% CHANNEL SLOPE AREA.
2. SPACING OF SILLS CAN BE VARIED AT DISCRETION OF FIELD ENGINEER. RIPRAP PLACED FOR TRIBUTARIES (FIG. 6) OR NATURALLY OCCURRING ROCK AREAS (FIG. 4) CAN BE USED IN PLACE OF THE SILLS SHOWN HEREIN.
3. FILTER LAYER REQUIRED IF RATIOS OF ROCK (COARSE) TO UNDERLYING MATERIAL (FINE) DO NOT MEET THE REQUIREMENTS BELOW. RATIOS OF ROCK/FILTER AND FILTER/UNDERLYING MATERIAL SHALL BE CHECKED IN SAME MANNER TO ENSURE THE PROPER DESIGN OF THE FILTER. THE NEED FOR AND DESIGN OF FILTERS SHALL BE FIELD DETERMINED.

$$\frac{(D_{15})\ \text{COARSE}}{(D_{15})\ \text{FINE}} < 40\ , \quad \frac{(D_{15})\ \text{COARSE}}{(D_{85})\ \text{FINE}} < 5\ , \quad \frac{(D_{50})\ \text{COARSE}}{(D_{50})\ \text{FINE}} < 40$$

4. RIPRAP SIZES IN mm TO BE AS FOLLOWS:

	CLASS A	CLASS B
D_{15}	120 - 180 mm	520 - 730 mm
D_{50}	180 - 210 mm	730 - 880 mm
D_{max}	210 - 300 mm	880 - 1250 mm

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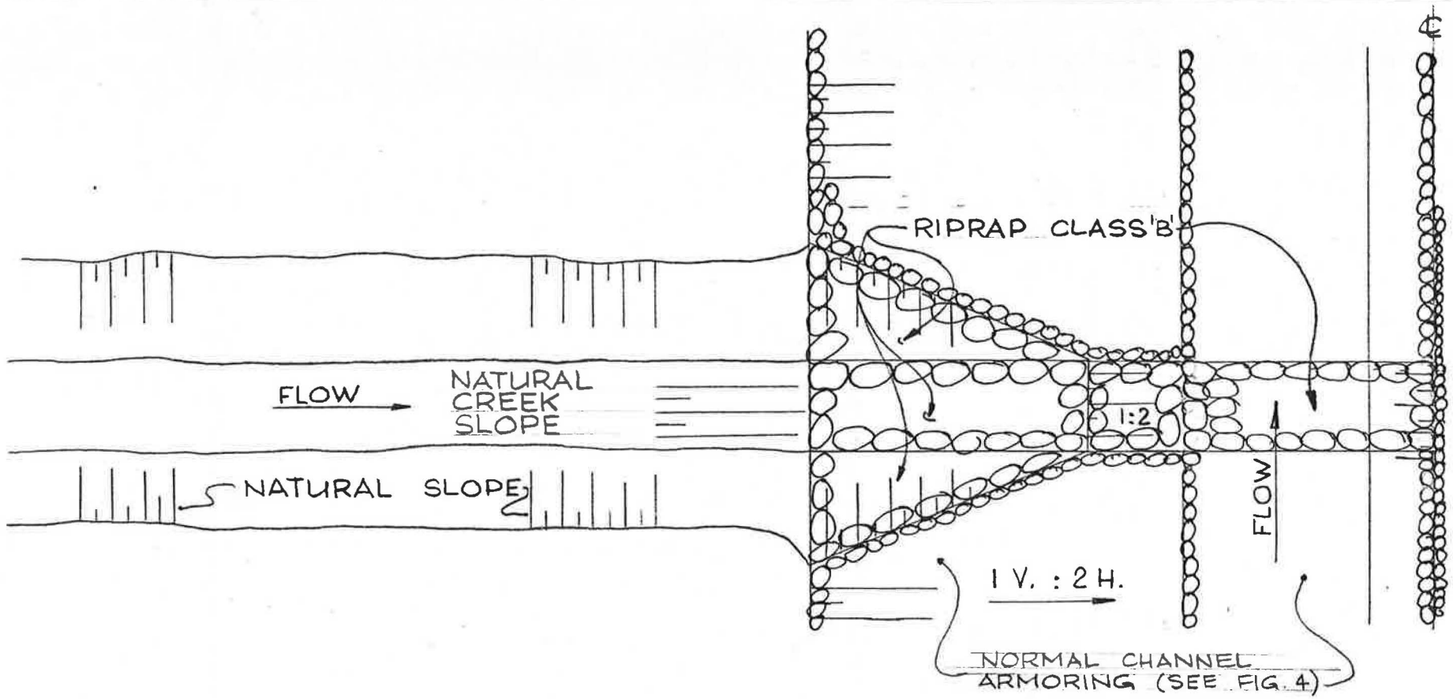
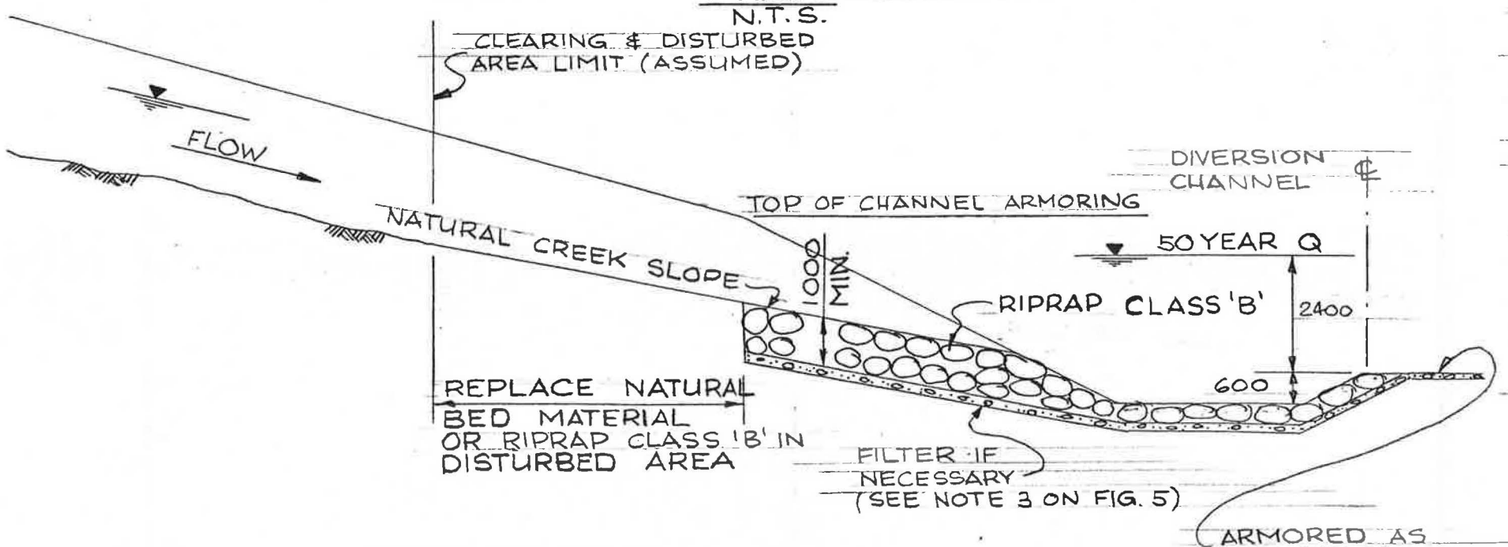
FIGURE 5

GOLDER ASSOCIATES		
DOWN VALLEY TAILINGS DISPOSAL		
ROSE CREEK DIVERSION DETAILS OF DROP WEIRS AND GRADIENT CONTROL SILLS		
HYDROCON ENGINEERING (Continental) LTD.		
DRAWN: UG	DATE: APR 30	JOB. 8-71



PLAN
N.T.S.



CROSS - SECTION
N.T.S.

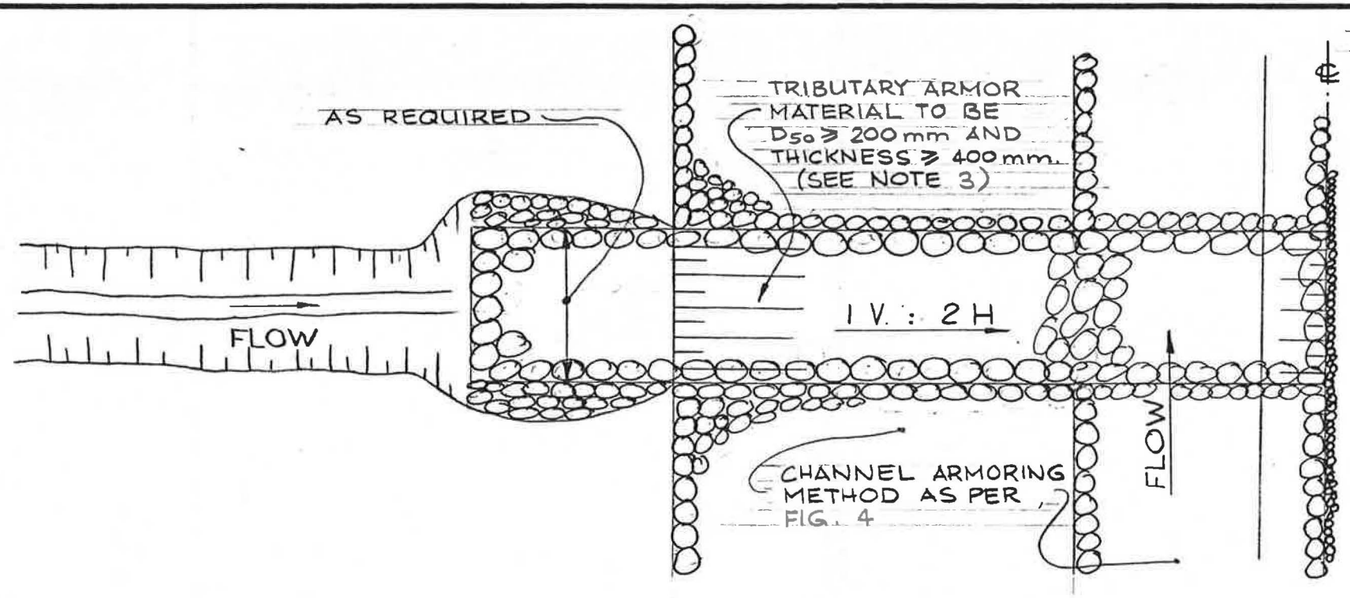
PROFESSIONAL ENGINEER ALBERTA
W.M. VELDMAN

NOTES:

1. IF TRIBUTARY GRADE AT INTERSECTION WITH MAIN CHANNEL IS MORE THAN 1000 mm (1 metre) ABOVE THE GRADE OF LOW FLOW CHANNEL, INSTALL DROP WEIRS IN THE TRIBUTARY AS PER FIGURE 5
2. TRIBUTARY INLETS TO BE FIELD ENGINEERED TO SUIT LOCAL CONDITIONS AND CHANNEL ARMORING METHOD (FIG. 4)
3. POSSIBLE APPLICABLE LOCATIONS
   STA. 0+614
   0+622
   3+100
   FOR THE FIRST TWO LOCATIONS:
   A) ELIMINATE RIPRAP IF ROCK IS PRESENT
   B) JOIN THE TWO UPHILL FROM DIVERSION CHANNEL INTO A SINGLE TRIBUTARY IF DEEMED TO BE DESIRABLE BY THE FIELD ENGINEER.
4. CLASS 'B' SPECIFICATION SEE NOTE 4 ON FIG. 5

FIGURE 6

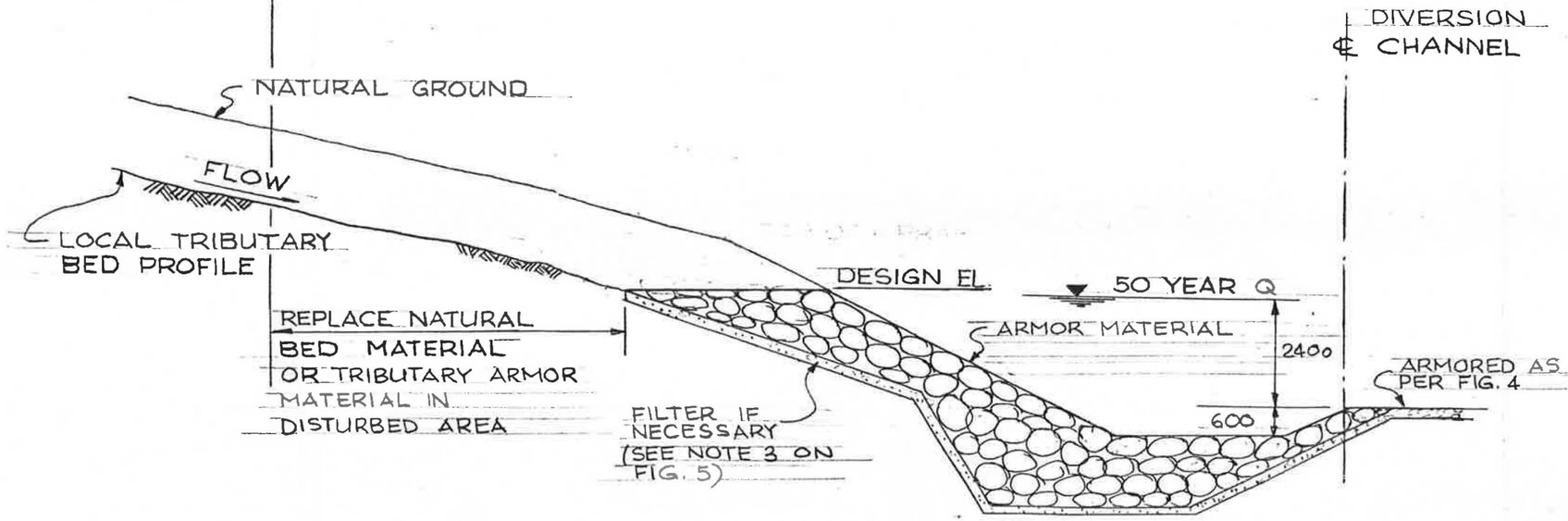
GOLDER ASSOCIATES
DOWN VALLEY TAILINGS DISPOSAL
ROSE CREEK DIVERSION TYPICAL MAJOR TRIBUTARY INFLOW DETAILS
HYDROCON ENGINEERING (Continental) LTD.
Drawn: U.G | Date: APR 80 | Job No. 8-71



PLAN
N.T.S.

NOTES:

1. TYPICAL DETAIL IS SHOWN TO ILLUSTRATE DESIGN PHILOSOPHY. SITE SPECIFIC DESIGN TO BE FIELD ENGINEERED TO SUIT TOPOGRAPHIC CONDITIONS AND CHANNEL ARMORING METHOD (FIG. 4.)
2. POSSIBLE APPLICABLE LOCATIONS:
   STA. 0+586
   1+900
   2+175
3. SPECIAL RIPRAP CAN BE ELIMINATED IF:
   A.) ROCK IS PRESENT,
   B.) CHANNEL ARMOR METHOD USED (SEE FIG. 4) IS DEEMED ADEQUATE BY THE FIELD ENGINEER.

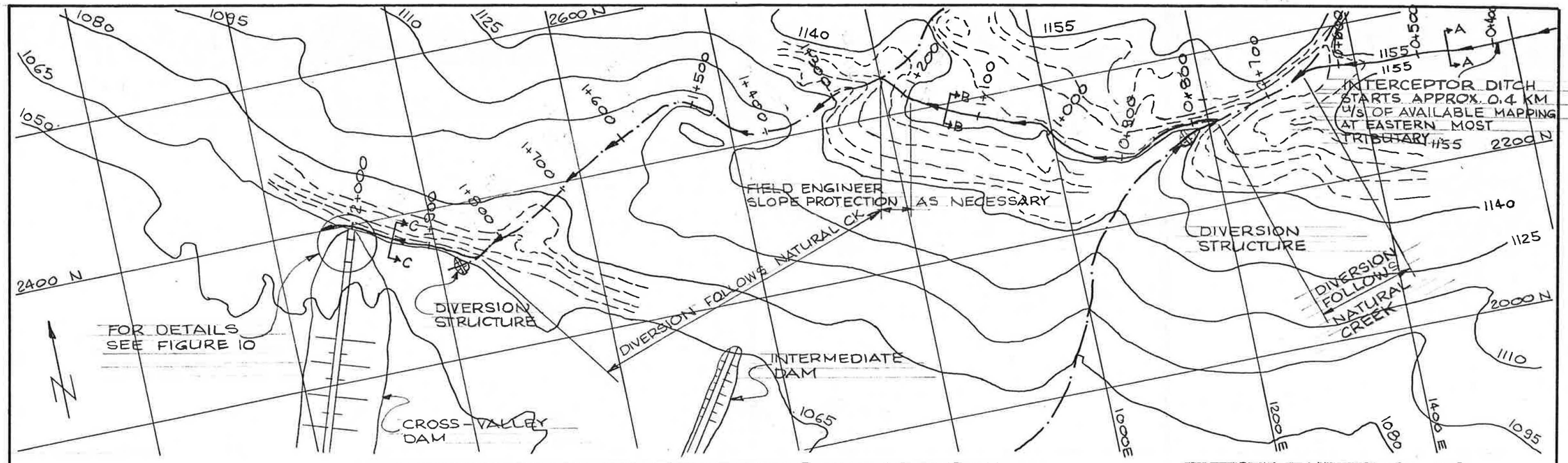


CROSS - SECTION
N.T.S.

PROFESSIONAL ENGINEER ALBERTA
W.M. VELDMAN

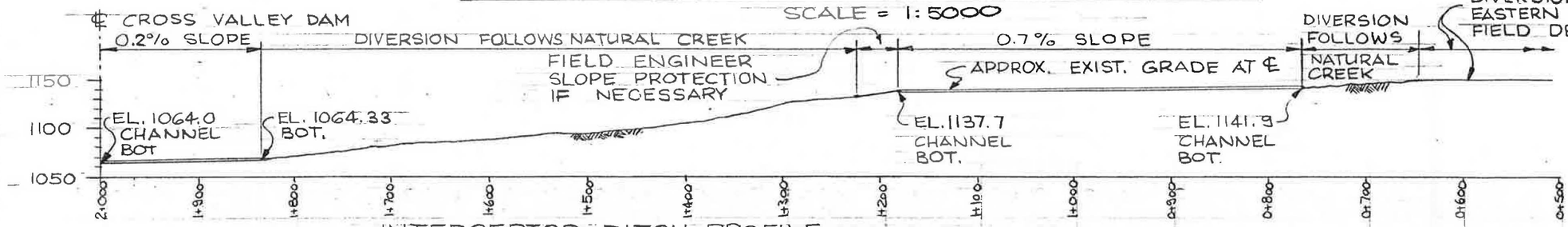
FIGURE 7

GOLDER ASSOCIATES
DOWN VALLEY TAILINGS DISPOSAL
ROSE CREEK DIVERSION TYPICAL INTERMITTENT CREEK OR SPRING INFLOW DETAILS
HYDROCON ENGINEERING (Continental) LTD.
Drawn. UG. | Date. APR. 80 | Job. No. 8-71



PLAN OF NORTH VALLEY INTERCEPTOR DITCH

SCALE = 1:5000



INTERCEPTOR DITCH PROFILE

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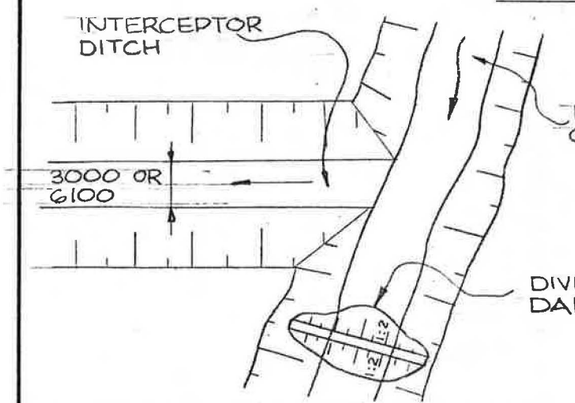
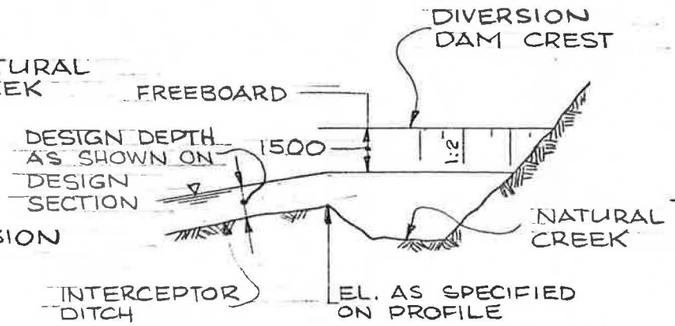


NATURAL CREEK OUTLET

N.T.S.



OUTLET - TYPICAL CROSS-SECTION

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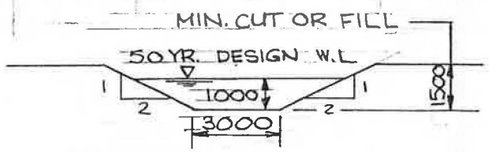
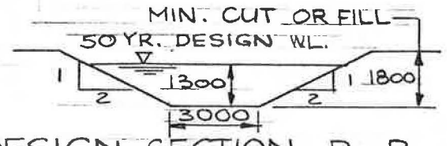


DESIGN SECTION A-A

BEGINNING TO STA. 0+650

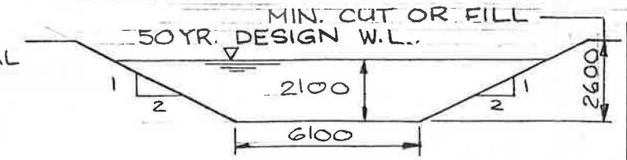
SCALE = 1:200



DESIGN SECTION B-B

STA. 0+770 TO 1+185

SCALE = 1:200



DESIGN SECTION C-C

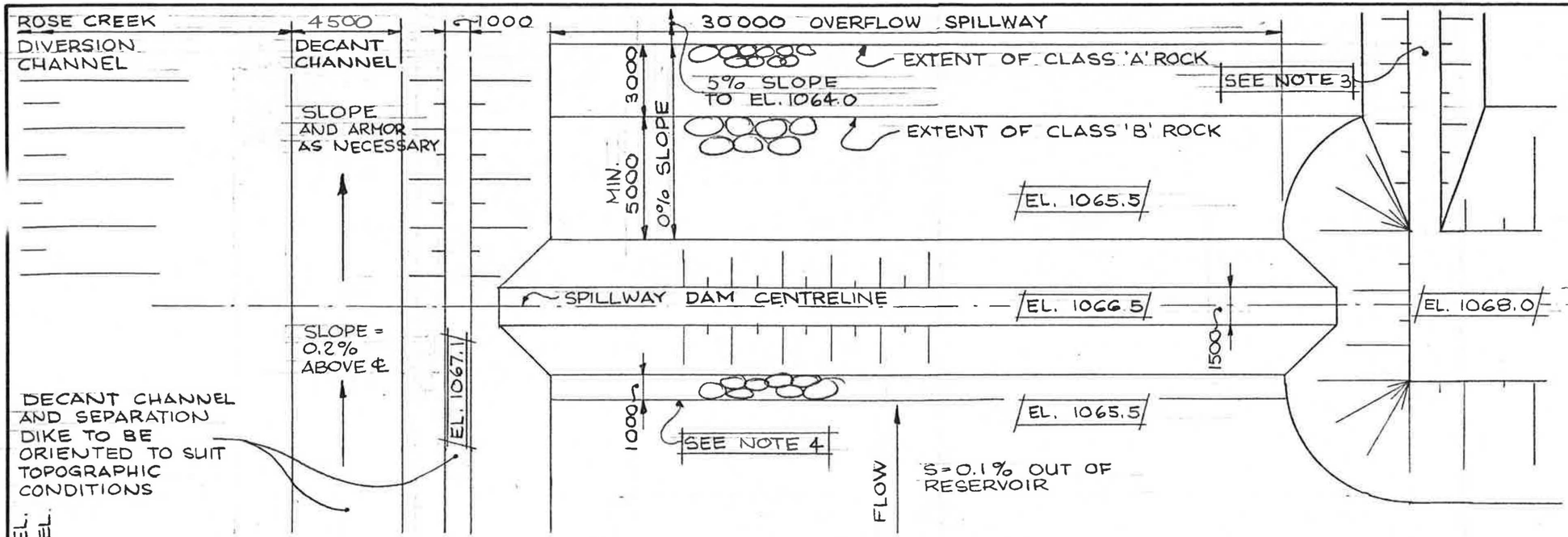
STA. 1+835 TO 2+015

SCALE = 1:200

FIGURE 8

GOLDER ASSOCIATES		
DOWN VALLEY TAILINGS DISPOSAL		
NORTH VALLEY INTERCEPTOR DITCH PLAN AND DETAILS		
HYDROCON ENGINEERING (Continental) LTD.		
DRAWN: UG	DATE: APR 80	JOB. 8-7.1

PROFESSIONAL ENGINEER ALBERTA W.M. VELDMAN

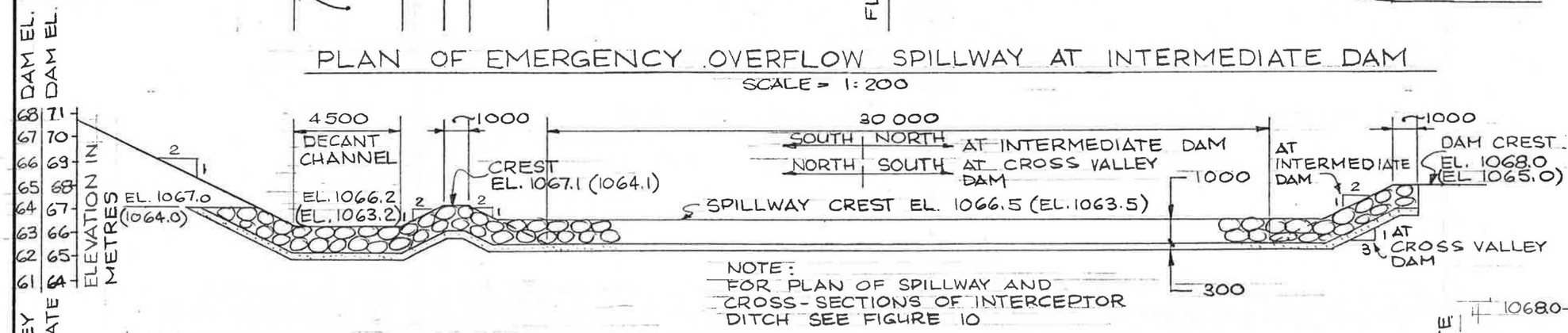


PLAN OF EMERGENCY OVERFLOW SPILLWAY AT INTERMEDIATE DAM

SCALE = 1:200

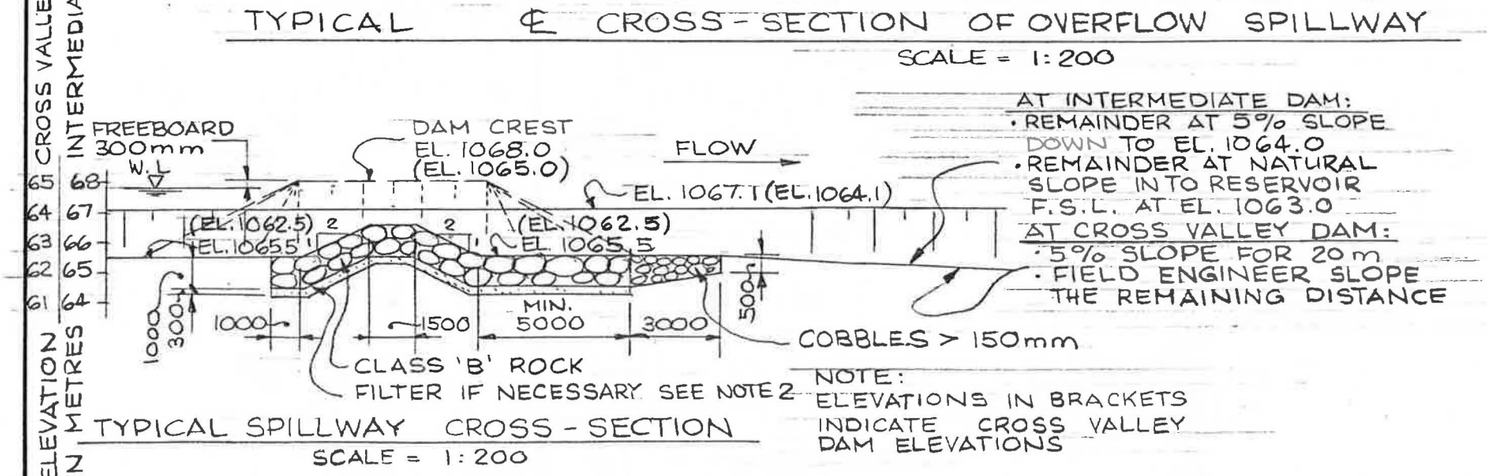
NOTES:

1. RIPRAP SPECIFICATIONS FOR CLASS 'A' AND 'B' AS PER GOLDER ASSOCIATES.
2. SEE NOTE 3 OF FIG. 5 FOR FILTER SPECIFICATIONS.
3. FIELD ENGINEER DIKE DOWNSTREAM OF SPILLWAY CREST SUFFICIENTLY LONG AND HIGH TO PREVENT IMPINGMENT OF SPILLWAY FLOW ON DOWNSTREAM FACE OF DAM.
4. TILL AND GRAVEL TO BE PLACED UPSTREAM OF ROCK SPILLWAY CREST TO PREVENT SEEPAGE OF DECANT THROUGH THE EMERGENCY SPILLWAY. DETAILS TO BE DETERMINED BY GOLDER ASSOCIATES.
5. RIPRAP SPECIFICATIONS AS PER NOTE 4. OF FIG. 5.



TYPICAL ℄ CROSS-SECTION OF OVERFLOW SPILLWAY

SCALE = 1:200



TYPICAL SPILLWAY CROSS-SECTION

SCALE = 1:200

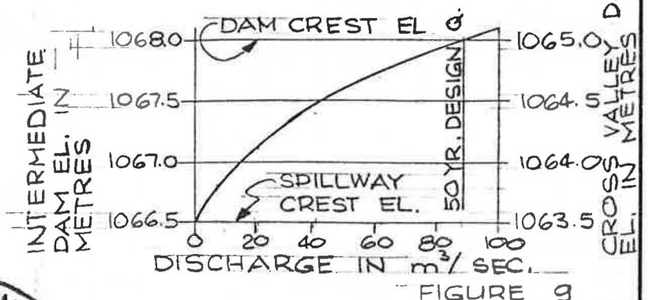
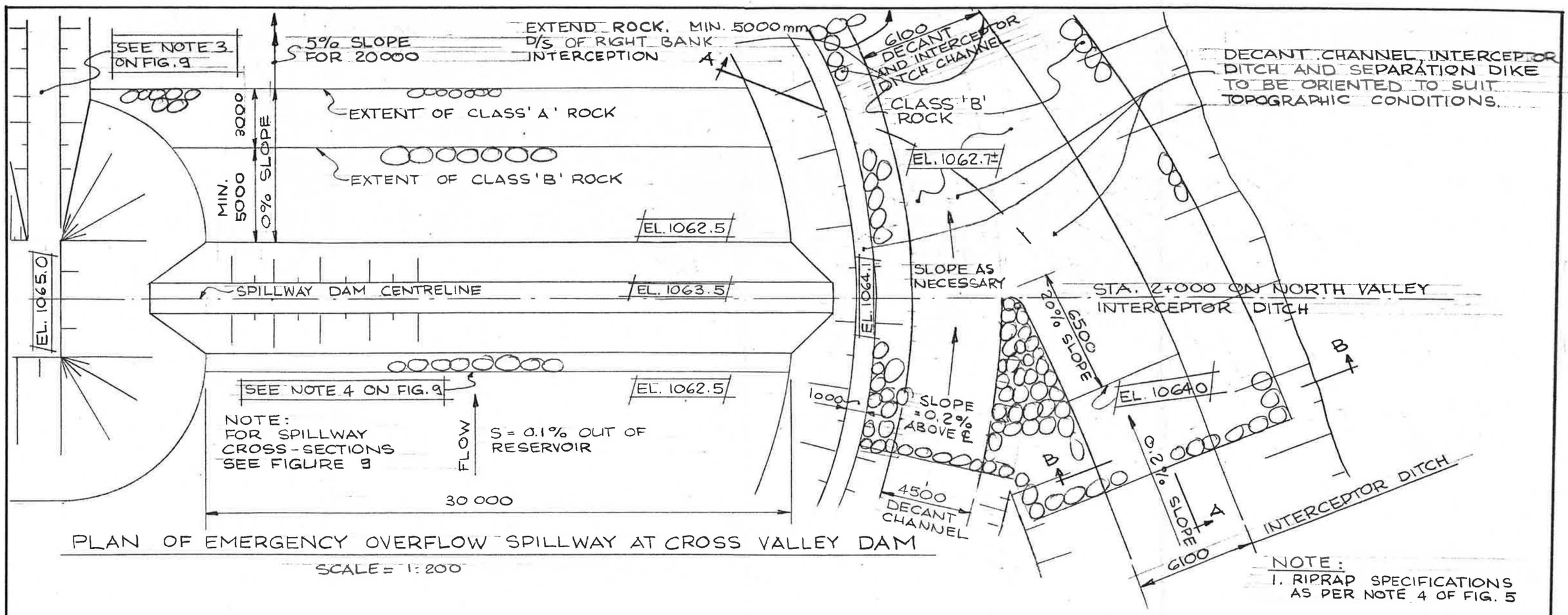


FIGURE 9

PROFESSIONAL ENGINEER ALBERTA W.M. VELDMAN

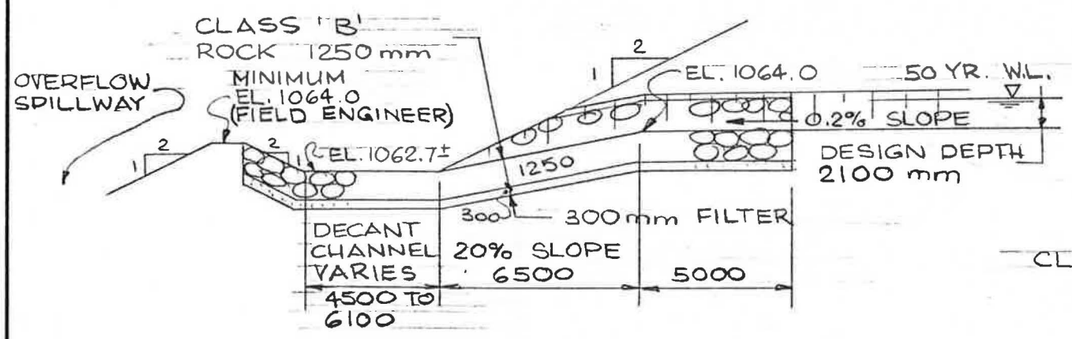
GOLDER ASSOCIATES		
DOWN VALLEY TAILINGS DISPOSAL		
INTERMEDIATE DAM EMERGENCY OVERFLOW SPILLWAY PLAN AND TYPICAL CROSS-SECTIONS		
HYDROCON ENGINEERING (Continental) LTD.		
DRAWN: UG.	DATE: APR 80	JOB. 8-7.1



PLAN OF EMERGENCY OVERFLOW SPILLWAY AT CROSS VALLEY DAM

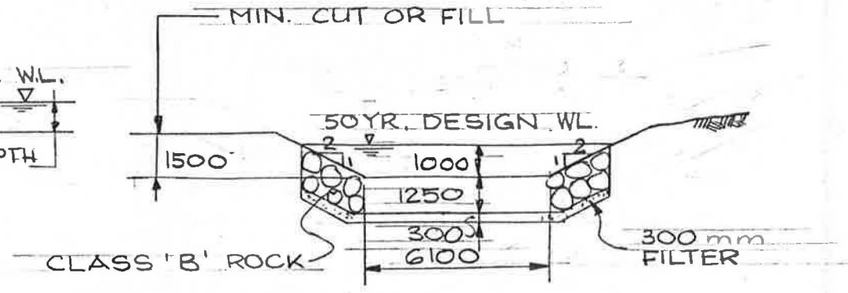
SCALE = 1:200

NOTE:
1. RIPRAP SPECIFICATIONS AS PER NOTE 4 OF FIG. 5



NORTH VALLEY WALL INTERCEPTOR DITCH
CROSS-SECTION AT CENTRELINE A-A

N.T.S.



INTERCEPTOR DITCH
CROSS-SECTION B-B
IN 20% SLOPE AREA
STA. 1+993.500 TO 2+010

SCALE = 1:200

PROFESSIONAL ENGINEER ALBERTA
W.M. VELDMAN

FIGURE 10

GOLDER ASSOCIATES
DOWN VALLEY TAILINGS DISPOSAL
CROSS VALLEY DAM EMERGENCY OVERFLOW SPILLWAY - PLAN AND INTERCEPTOR DITCH CROSS-SECTIONS
HYDROCON ENGINEERING (Continental) LTD.
DRAWN LIG | DATE: APR. 80 | JOB. 8-7.1

Earthquake Data

4669 Osler Street
Vancouver, B.C.
V6H 2Y3

January 4, 1980

Mr. D.C. Goodall, P. Eng.
Golder Geotechnical Consultants Ltd.
5915 3rd Street SE
Calgary, Alberta T2H 1K3

RE: Earthquake Stability of the Proposed Tailings Impoundment at the Cyprus Anvil Mine, Faro, Yukon Territory

Dear Dave:

Further to our meeting in Calgary on November 19, 1979, I am making recommendations regarding the earthquake stability of the proposed extension to the tailings impoundment at the Cyprus Anvil Mine, Faro, Yukon Territory. My recommendations are based on the following sources of information:

1. Your letter to me dated October 22, 1979.
2. Discussions with Mr. Bill Hodge, November 15 and 17, 1979.
3. Discussions with Mr. Glen Gilchrist and yourself, November 19, 1979.
4. Your report "Geotechnical Recommendations Concerning a Proposed Sloping Tailings Disposal Method for Cyprus Anvil Mining Corporation, Faro Mine", 1977.

SITE CONDITIONS

A plan of the proposed impoundment extension is shown in Figure 1. It is my understanding that two retention dams will be constructed. The upper dam will retain the slimes while the lower dam will retain contaminated water as shown schematically in Figure 2. It is intended that the surface of the slimes will slope either up valley or cross valley as shown in Figure 2. The present tailings comprise of fine sand and silt but a finer grind which will produce finer tailings is being considered.

THE EFFECT OF EARTHQUAKES ON TAILINGS IMPOUNDMENTS

An earthquake has basically two effects on an earth structure: (1) It causes oscillating stresses on the structure for a period of about 10-40 seconds and (2) the stress oscillations may cause the soil to lose some or all of its strength. If the soil loses all of its strength it is said to have liquefied and most of the earthquake damage to earth structures has been caused by such liquefaction. Loose saturated sand and slime deposits are very prone to failure by liquefaction. Casagrande (1950) pointed out the dangers inherent in tailings impoundments as follows: "Chemical and mining wastes consisting of flour sized material are usually deposited hydraulically in a very loose condition. Although they may appear stable

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behind the flimsiest dikes and in some cases supported only by a thin wall of the same material which has dried along the slopes, they are extremely sensitive to disturbance and constitute treacherous conditions which in some instances have caused disasterous flow slides with much loss of life and property". The flow slides or liquefaction may be initiated by local shocks caused by small slips or by dynamic stresses caused by earthquakes.

Many failures of tailings dams attributed to liquefaction have been recorded but the most prominent examples in recent times are provided by the failures of Chilean tailings dams in the La Liqua earthquake of 1965 (Dobry and Alvarez, 1967). Ten out of fourteen tailings dams inspected in the area failed. At the El Soldado mine there were 3 tailings dams which had been constructed by the upstream method. Two of these dams failed emptying 2 million tons of tailings which flowed 12 kilometers, flooding and destroying part of the El Cobre village and causing more than 200 casualties. It is important, therefore, that tailings impoundments be designed to resist failure by liquefaction during the design earthquake.

METHOD OF ANALYSIS

A rational assessment of the liquefaction potential at a site requires the following steps:

1. Determination of the design earthquake (time history of accelerations)
2. Dynamic analysis to determine the dynamic stresses caused by the earthquake.
3. Determination of the dynamic resistance of the soil.

A factor of safety against the occurrence of liquefaction is then computed by comparing the dynamic resistance of the soil with the dynamic stresses caused by the earthquake. A factor of safety in the range 1.2 to 1.5 is generally considered acceptable.

SEISMIC ACTIVITY AND DESIGN ACCELERATION LEVEL

Dr. Milne's report on the seismic activity of the site is appended to this letter. The maximum acceleration levels to be expected for 100 and 200 year return periods are 5 and 10 percent gravity. The earthquake design of buildings in Canada is governed by the National Building Code of Canada (NBCC) and is based on the 100 year return period acceleration level. It might appear therefore, that the tailings impoundment should be designed for the same acceleration level. However, the code requirements are such that buildings are expected to suffer only minor damage for the 100 year earthquake. Much higher acceleration levels corresponding to 200 to 500 year return periods would be required to cause collapse of such buildings. On the other hand, tailings impoundments unlike most civil engineering structures evolve over a long period of time. The most critical condition from the earthquake point of view may only last for a relatively short period of time and the probability of an earthquake occurring in that short period would be very low. The consequences of failure at any stage of construction must also be considered. A rational method of analysis should consider both the probability and the consequences of failure at the various stages of construction. This is beyond the scope of this report. It is suggested that the impoundment be designed to resist at least an acceleration level of 5 percent gravity during the active life of the mine and 10 percent gravity thereafter. These acceleration levels correspond to 100 and 200 year return periods.

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DYNAMIC ANALYSIS

A detailed dynamic analysis requires an evaluation of the dynamic stress-strain properties of the soil and a solution of the equations of motion to obtain the time history of stresses. An approximate method of obtaining an average dynamic stress, τ_{av}, that would be equivalent to those of the design earthquake was presented by Seed (1976) and is as follows:

$$\frac{\tau_{av}}{\sigma'_o} \simeq 0.65 \frac{a_{max}}{g} \frac{\sigma_o}{\sigma'_o} r_d \qquad (1)$$

where a_{max} = maximum acceleration at ground surface

σ_o = total overburden pressure

σ'_o = effective overburden pressure

r_d = a stress reduction factor varying from one at the surface to a value of about 0.9 at a depth of 30 ft.

Now the design accelerations of 5 and 10 percent of g are for rock or firm ground. When the motion propogates up through the tailings impoundment it may be amplified and the NBCC suggests a foundation factor of 1.5 for a depth of soft soil in excess of 50 ft. The a_{max} value for use in equation 1 are therefore 7.5 and 15 percent g.

DYNAMIC RESISTANCE OF TAILING IMPOUNDMENT MATERIAL

The proposed tailings impoundment is comprised of two retention dams. The upper dam will retain tailings while the lower dam will retain contaminated water. These dams will over-lie natural granular deposits up to 100 ft. in depth and it is imperative that neither the dams nor their foundations should fail by liquefaction during the design earthquake. The stored tailings behind the dam could liquefy but provided the slope of the tailing surface is sufficiently flat, a flow of the tailings leading to overtopping of the dams would not occur. Thus, there are two questions to be addressed here: 1) what densities of material are required in the dams and their foundations to prevent liquefaction and 2) what is a safe slope for the tailings should they liquefy.

1) Dynamic resistance of dams and their foundations

The materials of the dams could be subjected to cyclic loading tests in the laboratory and their dynamic resistance obtained. However, from experience it has been found that compacted cohesive material is not susceptable to strength loss and liquefaction and consequently if the dams are constructed of such materials, liquefaction is not a concern. Saturated granular material is susceptable to strength loss and liquefaction depending on the magnitude of the earthquake stresses and the density of the material. Based on field experience Seed (1976) has presented a design chart for assessing the liquefaction potential of granular material (Figure 3). The density of the soil is expressed in terms of its standard penetrative resistance corrected to a confining stress of $1T/Ft^2$. The correction for confining pressure C_N is shown in Figure 4. The stresses caused by the earthquake, τ/σ'_o, can be computed from Eq. 1.

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The required N_1 for the foundation soil during the active life of the mine can be estimated as follows:

$$\text{From Eq. 1} \quad \frac{\tau_{av}}{\sigma'_o} = 0.65\ (1.5)(.05)\ \frac{\sigma_o}{\sigma'_o}\ r_d = .05\ \frac{\sigma_o}{\sigma'_o}\ r_d \qquad (2)$$

If the water table is close to the surface $\frac{\sigma_o}{\sigma'_o} \simeq 2$ and taking r_d at the foundation level = 0.8, $\frac{\tau_{av}}{\sigma'_o} = 0.08$ and from Figure 3, the required $N_1 \geq 7$.

If we wish to have a factor of safety of say 1.3 against the occurrence of liquefaction, then the required $N_1 \geq 9$ which corresponds to a relative density of about 50 to 55 percent. The granular material of the dams should have a similar density. After mining has ceased the higher acceleration of 10 percent g should be considered. However, at this stage the lower water storage dam will not be required and the high porewater pressures it causes should dissipate so that the ratio σ_o/σ'_o should now be close to unity and $\tau_{av}/\sigma'_o \simeq 0.08$ as before. The required blow count and density are therefore, 9 and 55 percent as before.

2) Safe slope of liquefied tailings

It is proposed to discharge the tailings from the North side of the valley so that they slope across the valley and towards the dam. This method of disposal is similar to that described by Robinsky (1978). He argues that since the tailings are deposited and come to rest in a liquefied state, they would not further flatten on reliquefaction caused by an earthquake. In fact, since drainage would occur after deposition, he argues that a large reserve of stability exists. On the other hand, Williams (1978) describes an interesting failure of a tailings slope in Australia which I am appending to this report. Briefly, tailings were being deposited into an excavation as shown in Figure 5. The tailings had a surface slope of 3^o to 4^o and although Vane shear tests indicated a factor of safety of about 2, a failure occurred. Surface boils suggest that high porewater pressures, probably due to statically induced liquefaction, were the cause of failure. The ground water conditions in the proposed tailings impoundment could be quite similar to those described above and hence, slopes in the range of 3^o or 5 percent could fail by static or earthquake induced liquefaction.

I am not aware of any laboratory studies to determine whether material deposited in a liquefied state would further flatten during simulated earthquake shaking. I intend to initiate research into this topic. Lacking further evidence and based on the failure described by Williams, I would suggest that you limit the tailings slope to 2 percent.

SUMMARY

The Cyprus Anvil mine is in an area of moderate seismic activity. It is important that the retention dams should not fail by liquefaction during an earthquake, and to prevent such a failure, I recommend that the granular material of the dams and their foundations have a relative density in excess of 55 percent. The tailings will likely liquefy during the design earthquake and to prevent the tailings from flowing and overtopping the dams, I suggest their slope be limited to 2 percent.

Yours truly, . . . 5

P.M. Byrne

PMB/sl

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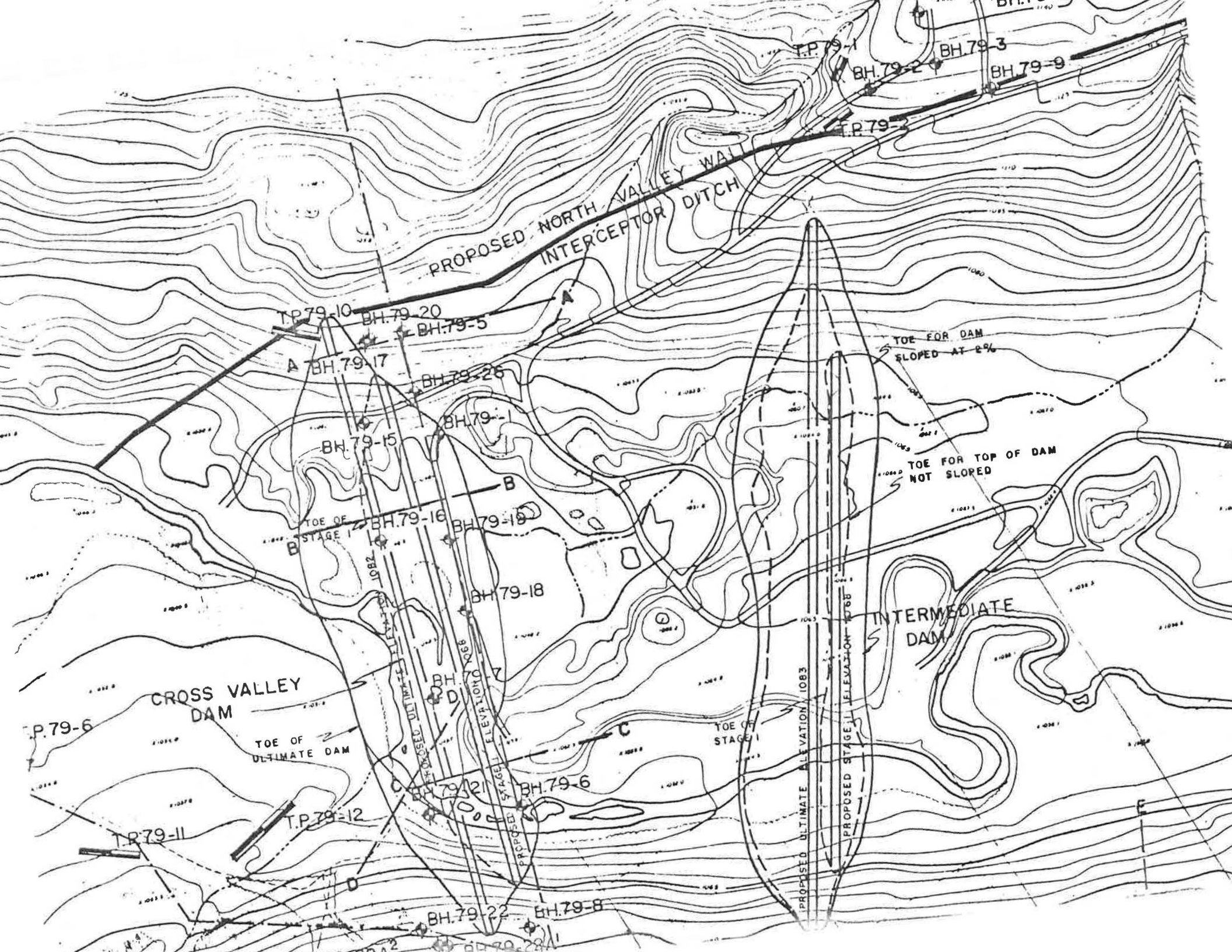
Casagrande, A. (1950). "Notes on Design of Earth Dams", Journal, Boston Society of Civil Engineers, Oct., pp. 231-255.

Dobry, R. and Alvarez, L. (1967). "Seismic Failures in Chilean Tailings Dams", Journal, Soil Mechanics and Foundation Eng. ASCB, Vol. 93, No. SM66, pp. 237-260.

Robinsky, E.L. "Tailings Disposal by the Thickened Discharge Method for Improved Economy and Environmental Control", Tailings Disposal Today, Vol. 2, Proc. of the Second International Tailings Symposium, Denver, Colorado, 1978, pp. 75-95.

Seed, H.B. "Evaluation of Soil Liquefaction Effects on Level Ground During Earthquakes" ASCE, National Convention, Philadelphia, 1976.

Williams, M.P.A. "Tailing Dam Failure Case History", Tailings Disposal Today, Vol. 2, pp. 428-436.


T.P.79-1
BH.79-3
BH.79-2
BH.79-9
T.P.79-3
PROPOSED NORTH VALLEY WALL
INTERCEPTOR DITCH
T.P.79-10
BH.79-20
BH.79-5
A
BH.79-17
BH.79-26
BH.79-1
BH.79-15
B
TOE FOR DAM
SLOPED AT 2%
TOE FOR TOP OF DAM
NOT SLOPED
TOE OF
STAGE I
BH.79-16
BH.79-19
BH.79-18
PROPOSED ULTIMATE ELEVATION 1082
PROPOSED STAGE I ELEVATION 1068
INTERMEDIATE
DAM
CROSS VALLEY
DAM
T.P.79-6
TOE OF
ULTIMATE DAM
BH.79-7
D
C
TOE OF
STAGE I
BH.79-12
BH.79-6
T.P.79-12
T.P.79-11
D
BH.79-22
BH.79-8
PROPOSED ULTIMATE ELEVATION 1083
PROPOSED STAGE II ELEVATION 1068
E

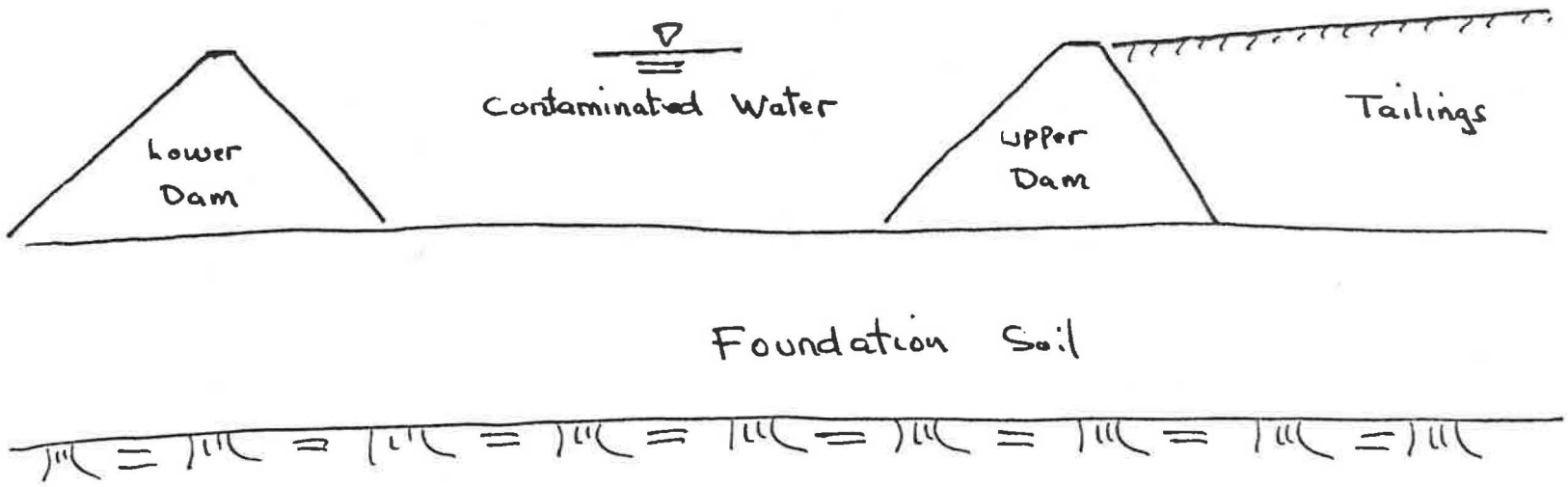


Fig. 2 Schematic Section of Tailings Impoundment.

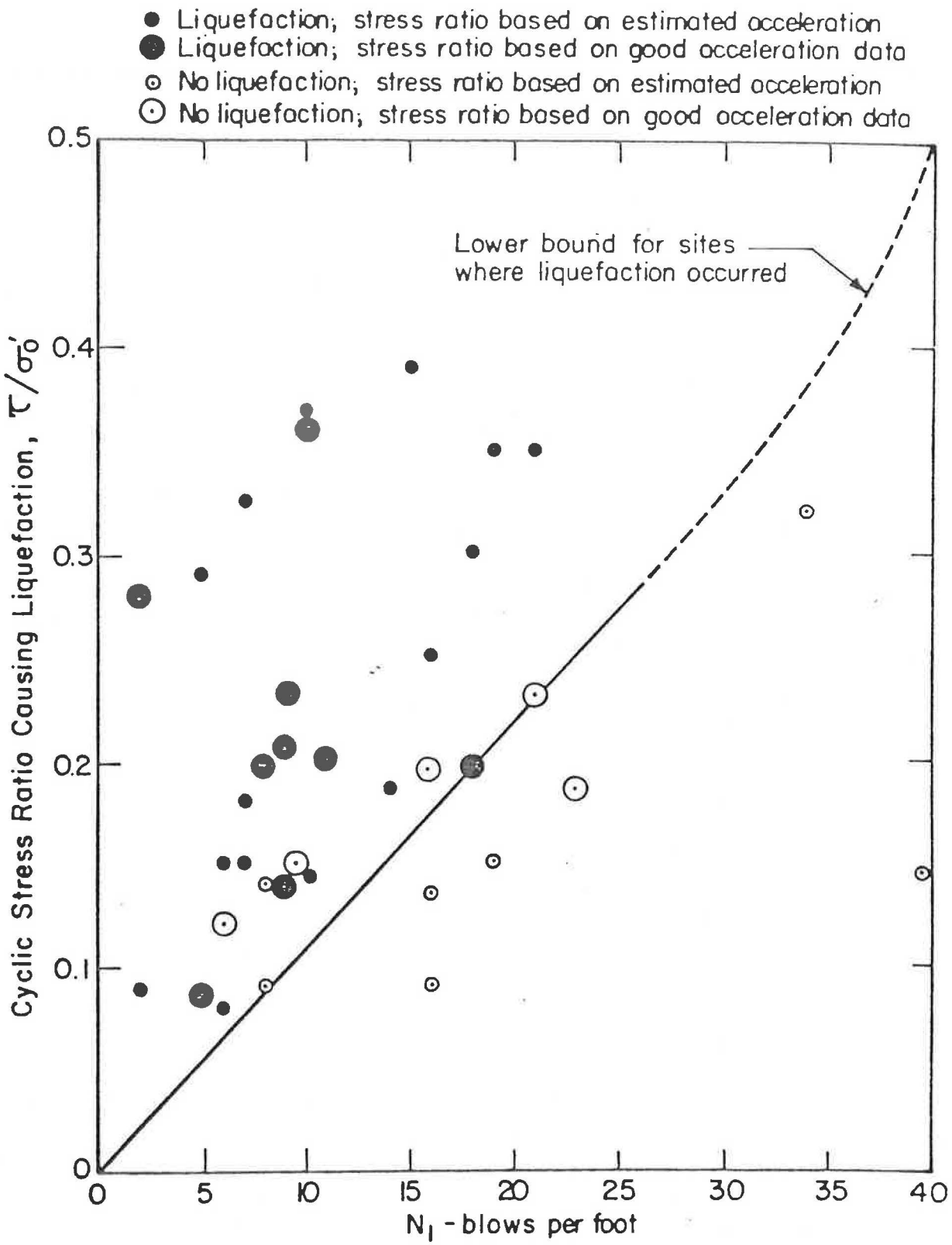


FIG. 3 CORRELATION BETWEEN STRESS RATIO CAUSING LIQUEFACTION IN THE FIELD AND PENETRATION RESISTANCE OF SAND

(After Seed et. al. 1975)

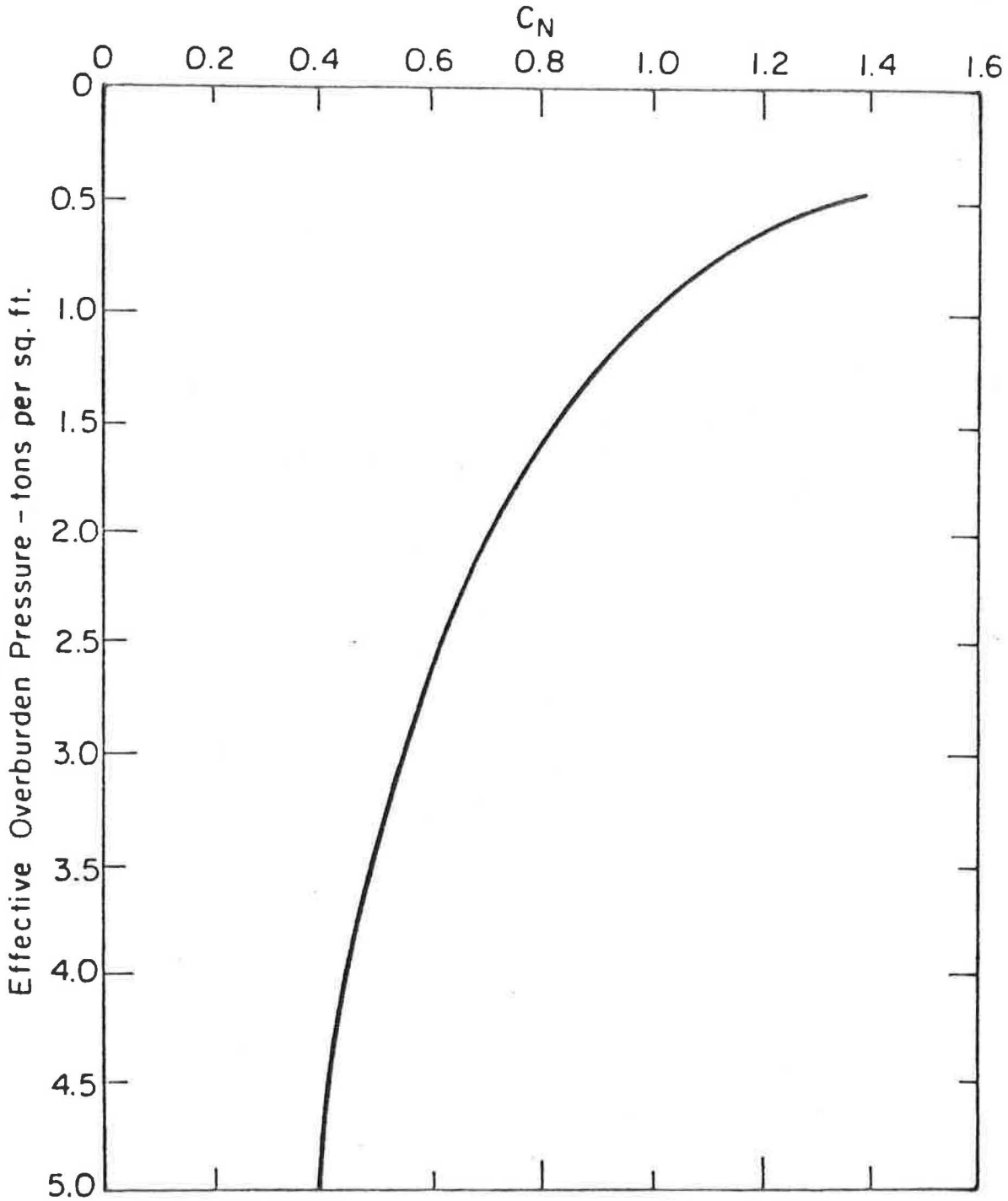


FIG. 4 RELATIONSHIP BETWEEN C_N AND EFFECTIVE OVERBURDEN PRESSURE

(After Seed et. al. 1977)

20
Tailing Dam Failure Case History

By M. P. A. Williams, Associate Director
Golder Associates Pty. Limited
Melbourne, Australia

This case history is of a beach sand mining operation in Western Australia. The sands are wind-blown deposits and they are mined for ilmenite, rutile, and zircon.

The ore body is mixed from shallow pits about 30 to 40 feet deep covering large areas - in this case of the order of 3,000 by 1,500 feet - and with a cross fall of about 3 to 4°, The pit floors consist of dense clayey sands with some cleaner sand lenses and some cementing. The mined sand is not ground, but treated directly in the mill as mined. The objective of the tailing disposal operation is simply to fill the pits back up to initial ground level.

The procedure initially adopted was to discharge the tailing from a single pipe at the lower edge of the pit, see Figure No. 1. A beach and a pond quickly formed. Once the beach level had risen to ground level, the pipe was moved upstream and filling of the pit advanced in this way. To minimize the frequency of moving the pipe, it became the practice to allow the beach to rise above the original ground level so that a localized crest was formed up to 15 to 20 feet in height. The crest was gradually moved up slope by bulldozers. There

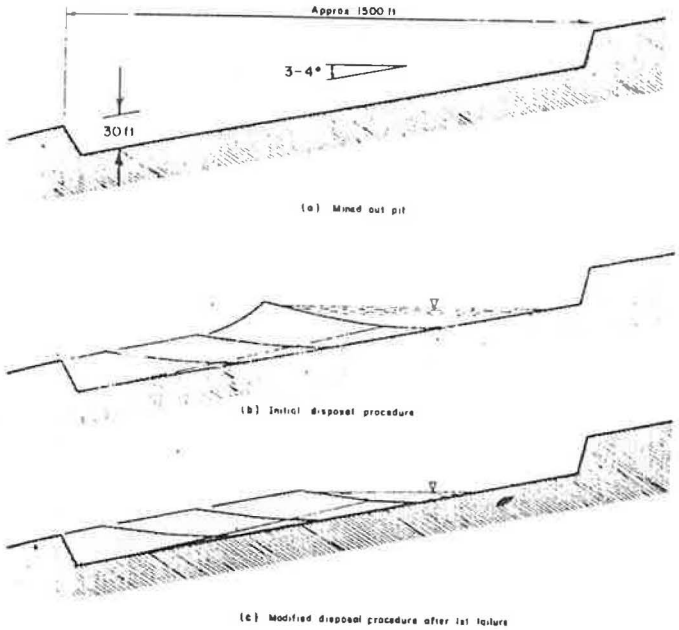


Fig 1 PIT CROSS SECTIONS

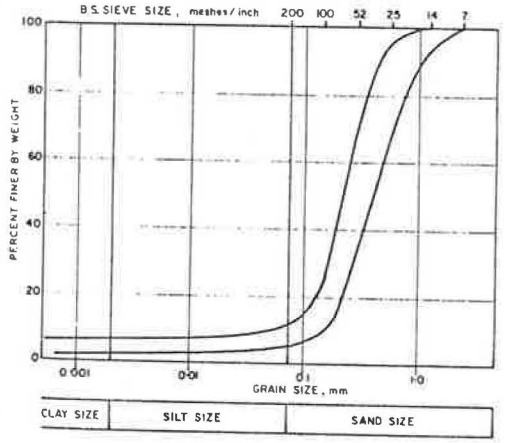


Fig 2 SAND GRADINGS

was some segregation as the tailing slurry flowed across the beach, but most of the slime settled out in the pond. The result tended to be that deposition occurred over an ever thicker slimes layer as disposal progressed up the slope.

In August 1976, a failure of the localized embankment occurred in Pit. No. 2. The mine is a remote, semi-arid area and there was no loss of life or damage to property associated with the event.

Investigation drilling and testing in the vicinity of the failure yielded the grading envelope shown on Figure No. 2 for the sand deposited under beach conditions. It can be seen to be a fine sand with the 4 to 12 percent passing the 200 sieve mostly consisting of clay size particles. The drilling encountered clay layers (slimes) up to 4 to 5 feet within the deposited sand. The results of it in situ shear vane tests in the slimes are shown on Figure No. 3. Shear vane tests were also undertaken in the clayey sands and gave valves indistinguishable from those obtained in the slimes. From the plot it can be seen that despite the considerable scatter, the tendency is for the strength to increase with depth to about 20 feet and to remain more or less constant thereafter. It was concluded that both the slimes and the clayey sands were behaving as normally consolidated materials, and they were treated as such in subsequent stability and analyses. The Standard Penetration Test blow counts recorded are also plotted on Figure No. 3. They were within the range o to 5 for the full depth tested.

Conventional slip circle and wedge stability analyses were undertaken using the design strengths shown in Figure No. 3. The downstream slope of the localized embankment was of the order of 5 or 6:1. The freeboard maintained during the operation was often only a few feet and since fairly heavy rain had occurred in the period leading up to the failure the phreatic surface was assumed to have been at ground level. In these circumstances, a factor of safety of less that unity was obtained in the analyses and it was concluded that a slip failure had occured, as was supported by a visual inspection of the failed area.

Several alternative disposal strategies were put forward, mostly based on separating the slimes from the

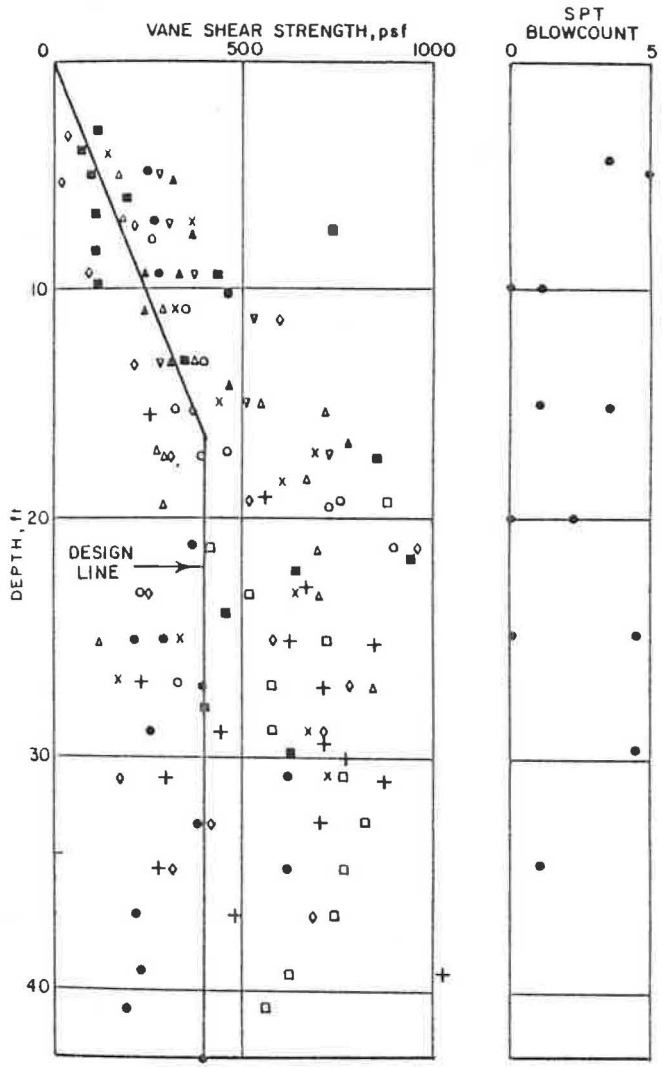


Fig. 3 TEST RESULTS

sand fraction, and using the latter for embankment construction behind which the slimes would be stored. The separation could have been achieved either by using cyclones at the disposal site or alternatively by taking advantage of the fact that the coarse and fine materials were found to be unacceptable however, on the grounds that the increased costs would have rendered the mining operation uneconomic. It was finally agreed that the original disposal method would be continued, but that the local embankment aspect would be eliminated. Thus, the downstream slope would effectively be the cross fall slope of 3 to 4° (between 14 and 19:1), see Figure No. 1(c). A minimum freeboard of 6 feet was specified. Conventional analyses gave a minimum factor of safety of two for this configuration.

The installation of piezometers to monitor the phreatic surface was discussed, but large numbers would have been required to cover the crest length of some 3,000 feet and the eventual filled pit width of 1,500 feet. The cost of this program was a significant factor once again, and since conservative assumptions concerning the phreatic surface had already been made, it was decided to proceed without them.

Almost exactly 12 months later, another failure occurred. The crest had advanced some 650 feet up the slope during this period. From its appearance afterwards, the nature and extent of the failure were clearly different from the first one. The failed area was much larger. There was evidence of substantial upthrusting at the toe of the pit and of the mass movement of blocks downslope. Boils occurred in places on the surface of the sand after the failure.

Further investigation drilling and testing were carried out and quick response piezometers were pushed into deep slimes deposits in an adjacent pit, but the work only served to confirm the shear strengths obtained earlier and the assumed pore water pressure profile.

At this stage it is assumed that failure occurred as a reult of liquefaction of the tailing. However, no obvious trigger for the liquefaction is apparent. There were no seismic shocks recorded in the area. A small bulldozer was employed virtually full time spreading the sand in the vicinity of the crest, but it is hard to see how it could have provided the energy necessary to mobilize the failure.

One possibility is that more permeable lenses in the tailing sand or in the sands in the pit floor permitted the full hydrostatic head from the pond to be transmitted to beneath the toe of the slope. The upheaval associated with the relief of this pressure could then have triggered liquefaction.

Current proposals to solve the problem include the installation of piezometers - probably a limited number towards the toe of the pit aimed if possible at providing forewarning of the build-up of artesion hydrostatic pressures both in and beneath the tailing - and modifications to the disposal method to try to minimize pond development and increase the density of the deposited tailing.

DAVE GOODALL

THIS IS AN ESTIMATE OF THE EARTHQUAKE PROBABILITY OF A SITE NEAR

CYPRUS ANVIL MINE

THE GEOGRAPHICAL CO-ORDINATES OF THE SITE ARE
62 DEGREES 24 MIN NORTH LATITUDE
133 DEGREES 18 MIN WEST LONGITUDE

EARTHQUAKES BETWEEN 1899 AND 1977 INCLUSIVE ARE INCLUDED IN THE ANALYSIS.

DURING THIS PERIOD THERE ARE 1410 EARTHQUAKES IN THE EARTHQUAKE CATALOGUE FOR THIS REGION, THERE WERE 67 EARTHQUAKES WHICH COULD HAVE BEEN FELT AT THIS LOCATION DURING THIS 79 YEAR PERIOD.

THIS SITE HAS EXPERIENCED A MAXIMUM ACCELERATION OF 4. PERCENT GRAVITY
OR A MAXIMUM INTENSITY OF VI

TABLE 1 IS A LIST OF THE EARTHQUAKES WHICH REACHED AN INTENSITY OF II OR GREATER AT THIS SITE. ALL OF THE EVENTS IN THIS LIST MAY HAVE BEEN FELT.

TABLE 2 IS A LIST OF THE PREDICTIONS FOR THIS SITE. PLEASE READ THE FOLLOWING REMARKS BEFORE USING THE DATA.

THE VALUES FOUND IN TABLE 2 ARE BASED UPON A STATISTICAL ANALYSIS OF THE EARTHQUAKES WHICH HAVE OCCURRED OVER THE ABOVE INTERVAL OF TIME. THIS IS TOO SHORT A PERIOD OF OBSERVATION FOR ACCURATE PREDICTIONS ABOUT TECTONIC ACTIVITY. HOWEVER, THE DATA HAVE BEEN PROCESSED TO PROVIDE AN ESTIMATE OF THE POSSIBLE LEVEL OF EARTHQUAKE ACTIVITY IN THE NEAR FUTURE ASSUMING A CONTINUATION OF THE STATISTICAL PATTERN OF THE PAST YEARS. ALTHOUGH PREDICTIONS OF ACCELERATION LEVELS MAY BE MADE FOR MANY PROBABILITIES OF EXCEEDANCE, THE RESULTS FOR PROBALITIES LESS THAN ONE IN ONE HUNDRED SHOULD BE USED WITH CONSIDERABLE CAUTION.

THE EXTREME VALUE METHODS OF STATISTICS ARE USED FOR THESE CALCULATIONS. THE SLOPE OF THE CURVE IS FOUND BY A LEAST SQUARES SOLUTION OF THE DATA FOR THE YEARS WHEN THERE WAS ACTIVITY ABOVE A FIXED THRESHOLD LEVEL. THE CURVE IS REPRESENTED BY THE EQUATION

LOG(ACC) = MODE + K(-LOG(-LOG(P))) $\leftarrow$ LN

P = 1-N where N is probability of being exceeded

WHERE NATURAL LOGARITHMS ARE USED. P IS THE PROBABILITY THAT ACC WILL NOT BE EXCEEDED IN 1 YEAR, R IS THE RETURN PERIOD (IN YEARS) OF ACCELERATION AMPLITUDE LOG(ACC) AND EQUALS 1/1-P .

FROM THIS THE ACCELERATION AMPLITUDE IS

ACC = EXP(LOG(ACC))

AND THE INTENSITY

I = 3 LOG10(ACC) + 4.5

I IS THE INTENSITY BY THE MODIFIED MERCALLI INTENSITY SCALE OF 1931. IN THE TABLES THE VALUE IS ROUNDED TO THE NEAREST INTEGER AND PRINTED IN ROMAN NUMERALS. ACC IS THE ACCELERATION AS A PERCENTAGE OF GRAVITY.

THE VALUES OF ACCELERATION LISTED IN TABLE 2 ARE FOR FIRM SOIL. OTHER

TYPES OF FOUNDATION MATERIAL MAY ALTER THE VALUES BY AT LEAST ONE UNIT OF INTENSITY.

THE SEISMIC ZONING MAP IN THE NATIONAL BUILDING CODE OF CANADA (1977) USES THE ACCELERATION AMPLITUDES WITH AN ANNUAL PROBALITY OF EXCEEDANCE OF .01 AS THE CRITERIA FOR ZONE BOUNDARIES. THIS IS CALLED ACC100. THE FOLLOWING ARE THE ZONE BOUNDARIES IN THE 1977 CODE..

ZONE 0		ACC100 < 1.	A = 0
ZONE 1	1. <=	ACC100 < 3.	A = 2
ZONE 2	3. <=	ACC100 < 6.	A = 4
ZONE 3	6. <=	ACC100 <	A = 8

CYPRUS ANVIL MINE

62 24 133 18

TABLE 1 DATA

DAY	MONTH	YEAR	MAG	LAT	LONG	DISTANCE MILES	DISTANCE KMS	ACCELERATION PERCENT G	INTENSITY	
4	9	1899	8.20	60.00	-142.00	333	536	2	VI	2.20
10	9	1899	7.80	60.00	-140.00	277	446	2	V	1.90
10	9	1899	8.60	60.00	-140.00	277	446	4	VI	4.23
9	10	1900	8.20	60.00	-142.00	333	536	2	VI	2.19
24	9	1907	5.00	59.50	-135.25	210	339	0	II	.17
15	5	1908	7.00	59.00	-141.00	350	563	1	IV	.62
18	6	1912	7.00	59.00	-143.00	402	648	1	IV	.51
18	5	1919	6.00	56.00	-136.00	452	727	0	II	.16
7	7	1920	6.00	60.20	-138.00	217	350	0	III	.44
10	4	1921	6.50	53.20	-133.70	635	1023	0	II	.16
25	4	1923	5.70	59.00	-138.00	283	456	0	III	.23
24	10	1927	7.10	57.50	-137.00	361	582	1	IV	.65
31	8	1933	5.20	59.25	-137.50	259	417	0	II	.16
19	9	1933	5.50	60.00	-138.00	227	366	0	III	.25
6	7	1935	6.00	59.00	-139.00	303	488	0	III	.28
18	9	1939	6.00	58.00	-136.00	317	511	0	III	.26
28	1	1940	5.20	61.75	-137.50	143	230	0	III	.36
29	5	1940	6.20	66.60	-132.70	290	467	0	III	.36
5	6	1940	6.50	67.00	-135.50	324	521	0	III	.42
10	8	1941	5.20	59.25	-137.50	259	417	0	II	.16
12	6	1942	5.70	61.00	-138.00	181	292	0	III	.42
3	2	1944	6.50	60.50	-137.50	190	307	1	IV	.87
16	10	1945	5.60	58.00	-137.00	329	529	0	II	.17
16	11	1945	5.50	58.00	-137.40	309	498	0	II	.16
30	4	1947	6.30	59.00	-139.00	303	488	0	III	.37
8	5	1947	5.00	61.00	-139.00	210	338	0	II	.17
28	2	1948	6.50	53.90	-132.10	588	947	0	II	.18
22	8	1949	8.00	53.75	-133.25	597	961	1	IV	.79
31	10	1949	6.70	56.00	-136.00	452	727	0	III	.32
9	3	1952	6.00	59.50	-136.00	219	353	0	III	.44
15	6	1952	5.50	65.70	-134.50	230	371	0	III	.25
11	1	1953	6.50	65.30	-133.20	200	322	1	IV	.82
12	8	1953	4.50	62.48	-134.60	41	67	1	IV	.93
1	3	1955	6.60	65.30	-132.90	200	322	1	IV	.90
7	1	1956	6.50	65.50	-133.50	214	344	1	IV	.74
1	8	1956	5.80	66.00	-133.50	248	400	0	III	.30
3	11	1956	5.70	61.00	-139.00	210	338	0	III	.34
4	11	1956	5.40	61.00	-139.00	210	338	0	III	.25
17	11	1956	6.50	59.50	-134.00	546	879	0	II	.20
30	1	1957	5.80	65.00	-134.00	180	291	0	IV	.47
23	6	1957	5.60	58.50	-137.00	297	478	0	II	.19
9	12	1957	5.70	65.50	-133.00	214	344	0	III	.33
10	12	1957	5.70	65.50	-133.00	214	344	0	III	.33
10	7	1958	7.90	58.60	-137.10	292	470	2	V	1.95
13	7	1958	5.60	58.30	-136.90	308	496	0	II	.18
31	8	1958	6.20	63.00	-144.50	356	574	0	III	.27
24	9	1958	6.20	59.50	-143.50	395	636	0	III	.23

11	10	1958	4,90	65,50	-132,50	215	346	0	II	,15
17	10	1959	5,60	60,10	-138,50	239	385	0	III	,26
15	8	1961	5,30	59,10	-136,90	258	415	0	II	,17
17	6	1963	5,40	60,50	-140,60	279	450	0	II	,17
15	7	1963	4,20	61,70	-134,90	70	114	0	III	,34
15	7	1963	3,50	62,00	-134,40	44	72	0	III	,42
22	2	1964	4,10	63,47	-130,70	110	177	0	II	,17
12	2	1965	3,80	63,40	-134,20	74	120	0	II	,21
4	10	1965	5,60	65,30	-134,00	201	324	0	II	,18
5	10	1965	5,20	65,30	-134,00	201	324	0	III	,22
23	12	1965	5,80	60,60	-140,70	273	440	0	III	,26
26	3	1966	5,00	65,00	-133,50	179	289	0	II	,21
13	3	1969	5,00	63,50	-120,50	174	280	0	III	,22
20	3	1971	5,80	61,33	-140,94	290	467	0	III	,24
10	6	1972	5,20	61,56	-140,25	232	373	0	II	,18
30	7	1972	6,20	56,77	-135,91	399	642	0	III	,23
1	7	1973	6,10	57,64	-137,33	344	553	0	III	,26
3	7	1973	6,00	57,98	-136,02	345	555	0	III	,23
5	3	1975	4,10	63,35	-130,09	120	193	0	II	,15
16	2	1977	3,00	62,32	-133,38	6	9	1	V	1,17

TABLE 2

62 24 133 1F

MODE FOR THIS SITE = -2.23
K FOR THIS SITE = .85

ACC100 FOR THIS SITE = 5.4 PERCENT GRAVITY

PROBABILITY OF ACC BEING EXCEEDED IN ONE YEAR.	ACC IN PERCENTAGE GRAVITY	INTENSITY (MODIFIED MERCALLI)	EQUIVALENT RETURN PERIOD
.333	0	III	3.
.100	1	IV	10.
.033	2	V	30.
.020	3	VI	50.
.010	5	VII	100.
.005	10	VII	200.

0FIN

P R

$R = -1/(P-1)$ where $N+P=1$

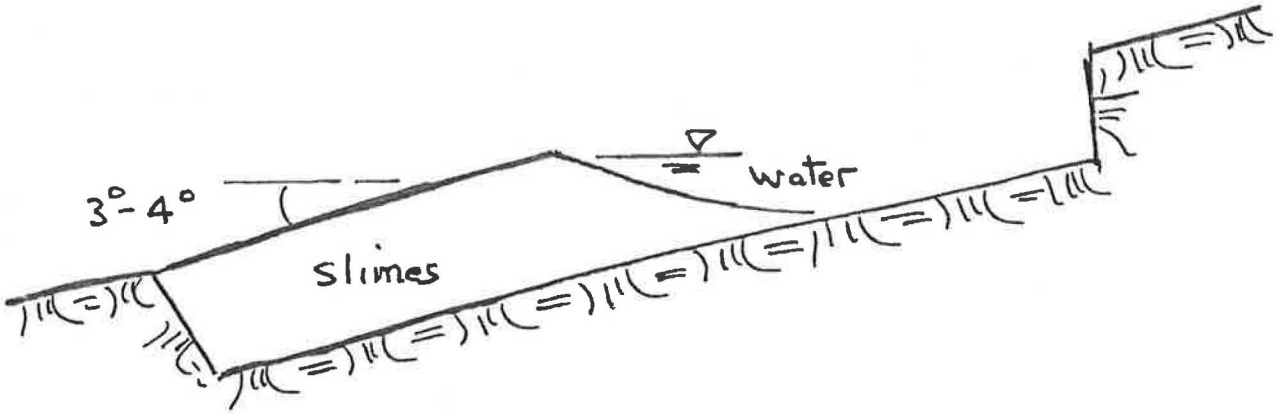


Fig. 5 Diagramatic Section of Tailings Impoundment Prior to Failure.