REPORT 60638

GEOTECHNICAL/HYDRAULIC DESIGN
VANGORDA CREEK DIVERSION FACILITY
VANGORDA PLATEAU DEVELOPMENT

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REPORT 60638

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1.0 INTRODUCTION

In accordance with your recent authorization, Steffen Robertson and Kirsten (B.C.) Inc. (SRK) completed a preliminary geotechnical and hydraulic design for the proposed Vangorda Creek diversion near Faro, Yukon, for Curragh Resources Inc. A vicinity map of the project site is shown on Figure 1. The purpose of the study is to complete a preliminary design for the embankment and culvert comprising the proposed diversion works for the Vangorda Creek. This report presents our recommendations for the design and construction of the proposed facility based on a recently completed geotechnical investigation.

Based on the information provided by Kilborn Engineering, SRK understands that Vangorda creek will be diverted approximately 700 metres upstream from the Vangorda Pit at the location of an existing stream crossing. The existing crossing consists of a 1500 mm diameter corrugated steel pipe (CSP) installed beneath a 5-m high road embankment. The stream would pass through the existing culvert and would then be diverted into a half round CSP pipe. The half-round pipe would be installed within an open channel ditch. The half-round pipe would convey flow along the northern hillside above the pit. The half-round CSP pipe traverses around the pit, eventually discharging back into the original stream bed below the pit. It is proposed to utilize the existing 1500 mm diameter CSP culvert and incorporate the existing road embankment into the final dam structure. The culvert through the dam would be required to pass the 100-year flood event with a peak flow of 10 cubic metres per second. This event would cause flood waters to impound upstream of the embankment. Consequently, the crest of the existing road embankment would have to be raised to accommodate the surcharge flood storage. A layout of the facilities is presented in Figure 2.

In addition to the extension of the road embankment and the construction of the half round diversion pipe, based on the design presented in this report it will also be necessary to build a low embankment at the south end of the open pit to prevent backflow during a flood event from entering the pit.

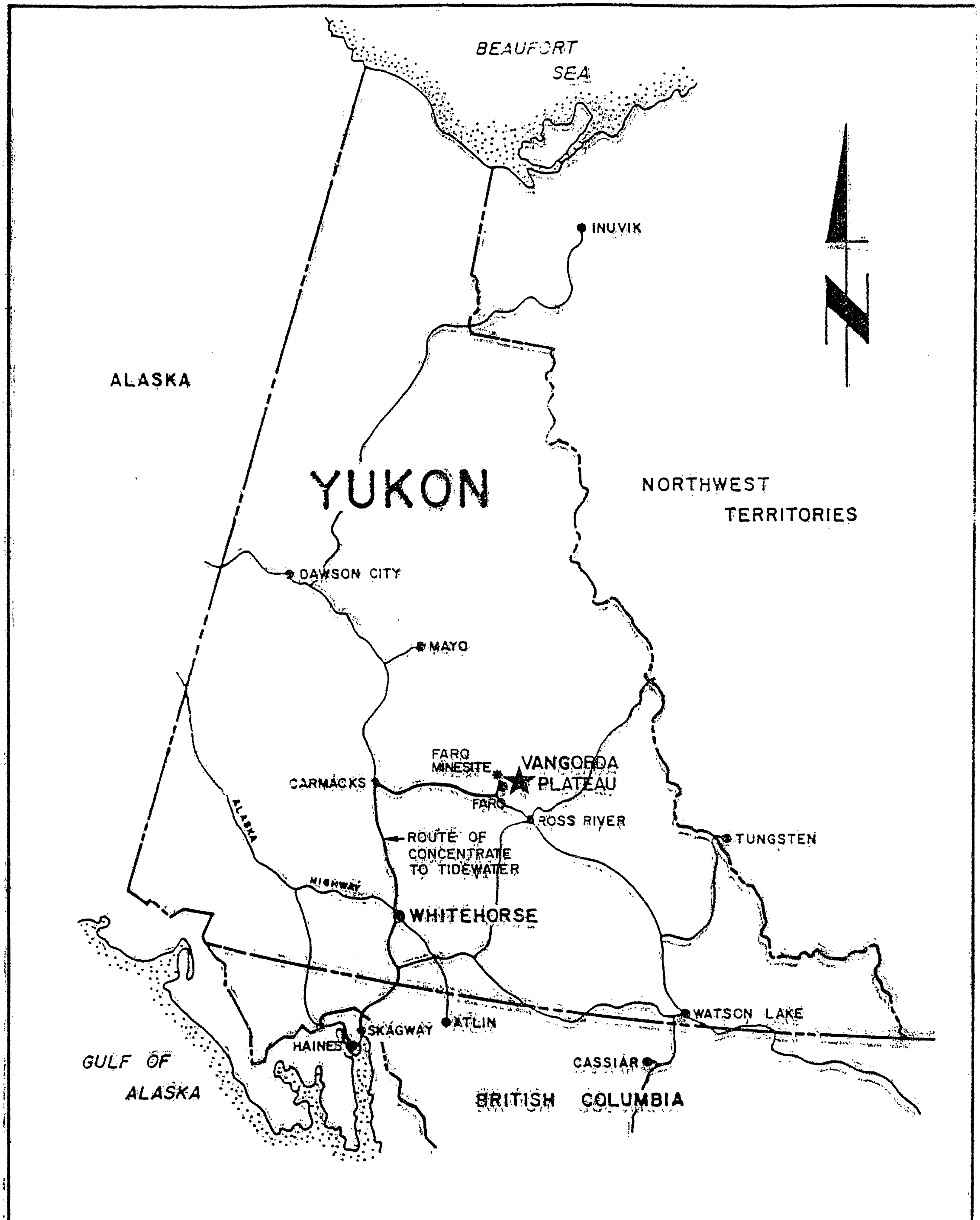


FIGURE 1

CURRAGH RESOURCES INC.

2.0 FIELD INVESTIGATION

To determine the conditions of the existing road embankment and the subsurface soil conditions for the foundation of the proposed embankment, a geotechnical investigation was completed recently by SRK.

The field work involved excavation of 8 test pits at the locations shown on Figure 3. The test pits were excavated using a track-mounted CAT 235 excavator provided by Curragh Resources Inc. The depths of the test pits varied from about 1.2 to 6.1 m below the existing ground surface. The field work was completed under the supervision of our field inspector. Representative soil samples encountered from each stratum were obtained and returned to our laboratory for visual examination and moisture content determinations. The logs of the test pits, together with the results of the moisture content determinations, are presented in the Appendix A, Figures A1 to A8.

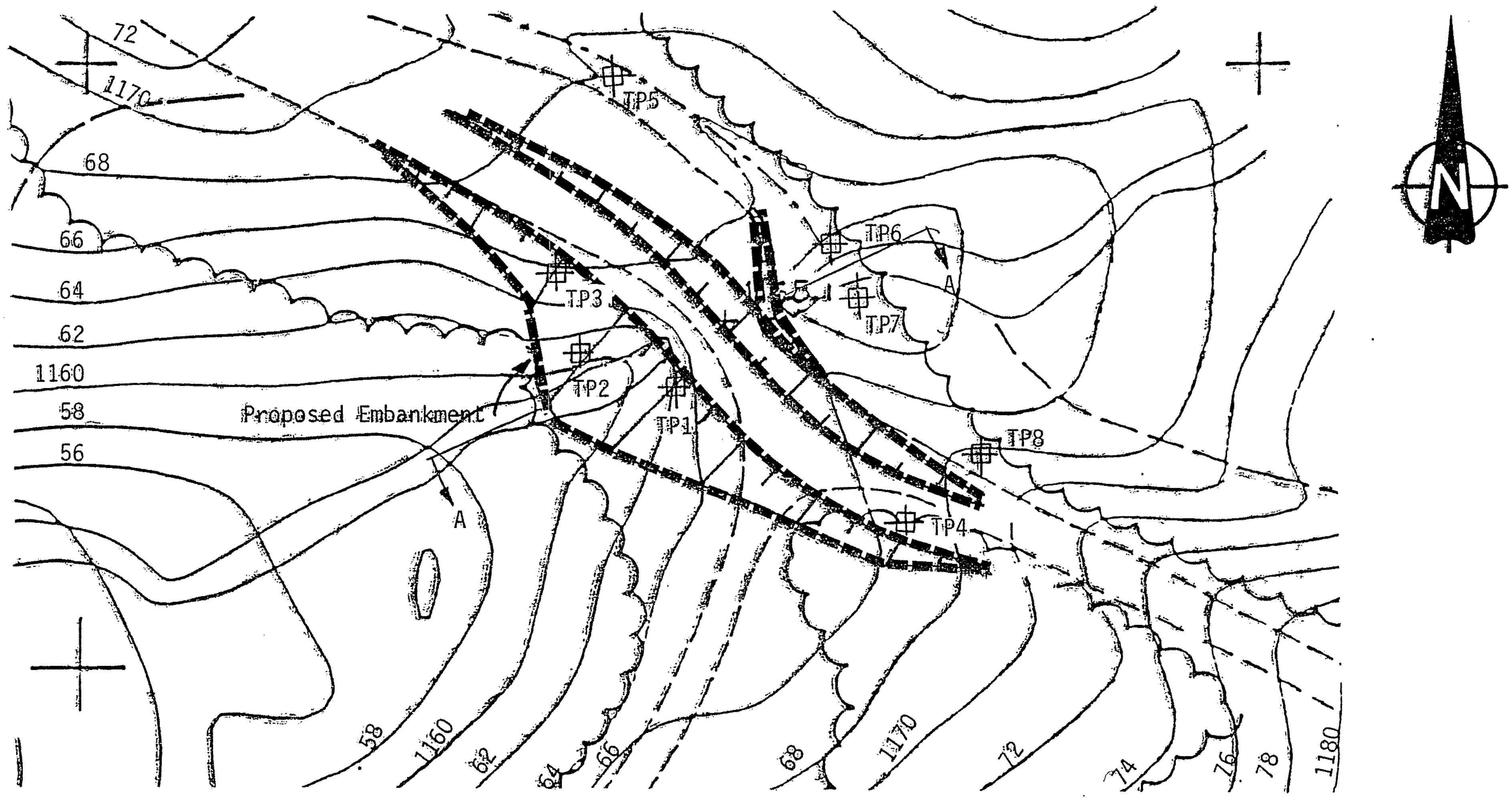
3.0 SITE DESCRIPTION

3.1 Surface Conditions

Vangorda Creek flows from north to south in a U-shaped gully about 4 to 5 metres wide at the base. The slope of the natural channel averages about 5 to 6 percent. At the bottom and sides of the creek, alluvial sand, gravel and cobbles in the order of about 1.2 m diameter were observed. The average slope of the banks above the creek is about 2.5 horizontal(H):1 vertical(V). At the time of the investigation, the depth of flow in the creek was about 0.5 metre.

At the top end of the creek where the proposed diversion begins, flow from the creek passes into the 1500 mm diameter CSP culvert installed at the base of the existing road embankment. At the location of the culvert, the crest of the road embankment is at El. 1165.1 m. Beyond the culvert, the profile of the road embankment rises towards the two abutments at about 7 percent. The crest width of the road embankment varies from about 10 to 25 metres, increasing from east to west. Near the east abutment, but in the downstream slope of the existing embankment, is a 10-metre access road which apparently dead-ends at a distance of about 250 metres south of the road embankment. The side slopes of the road embankment are close to the angle of repose of the granular fill materials at 1.3H:1V. At the culvert location, the crest of the road embankment extends about 4 and 5 metres above the invert of the culvert at the inlet and outlet of the culvert, respectively. At the outlet of the culvert, a 1.5 m high wing wall, constructed with sawn timber was observed. The wing walls were about 4.5 m long.

The proposed alignment of the 2400 mm diameter half-round pipe would traverse along the hillside around the north end of the Vangorda Pit. According to the local topography, the hillside slopes down from north to south at about 25 percent.



Test Pit Location

10 30 50
0 20 40

Scale - 1:1000

Site Plan referenced to a Topographic Map for Faro Plateau, Scale 1:2000, compiled by The Orthoshop, Calgary, dated November 1990.

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Consulting Engineers

FIGURE 3

SITE PLAN
PROPOSED EMBANKMENT

The outlet at the low end of the diversion would include an existing 1800 mm diameter culvert under the Vangorda Pit haul road from the Faro Mine. Under the haul road, there are also two 1800 mm diameter existing culverts. The lower culverts are in the existing creek bed and have partially collapsed under the haul road. However, the lower culverts still permit the existing creek to flow through. The upper overflow culvert, which would be utilized in the proposed diversion facility, is located about 2.5 metres above the lower culverts.

Scrub, deciduous trees and brush line the present creek banks.

3.2 Subsurface Conditions

The subsurface soil conditions at the proposed embankment site consist of a glacial till overlying bedrock. The thickness of the glacial till varies from about 1.5 to 4 metres. The upper 1.5 metres of the till consists of a silty/clayey fine sand with some gravel. Below the weathered zone, the till grades more cobbly with depth. Occasional boulders in the order of 1200 mm diameter were encountered in the test pits. The consistency of the glacial till is medium dense to dense.

Below the glacial till is a thinly foliated phyllite bedrock. The bedrock is slightly weathered, but hard. Moderate oxidization and staining were observed on the bedrock samples.

The road embankment fill mainly consists of materials similar in the gradation to the glacial till soil described previously. The consistency of this fill is medium dense.

4.0 HYDROLOGY

4.1 General

The Vangorda Creek diversion channel has been designed to convey the peak flow for the 100 year return period event. Estimation of this flood event was based on a regional relationship between peak instantaneous discharge and catchment area. The methodology used to derive this relationship is described in the Vangorda Plateau Water Licence Application, Volume I. A discussion on the flood hydrology of the Vangorda Creek catchment is included in Volume I and has been reproduced in Appendix B. The design discharge is 10 m^3/s.

Due to environmental considerations, the diversion channel was also designed to be reasonably watertight. Leakage from the diversion system will enter the Vangorda open pit and thus increase the quantity of contaminated water that must be treated. Reduction of leakage will be realized by lining the diversion channel with a half-round corrugated steel pipe (CSP).

4.2 Blind Creek Road Culvert Crossing

The road embankment at the Blind Creek culvert crossing will be raised in order to provide sufficient headwater depth for the peak flow to pass through the existing 1500 mm culvert. Under design conditions the culvert will require 6.5 m of headwater depth to pass the design flow of 10 m^3/s under inlet control. The outlet of this culvert will be made to overlap the 2400 mm CSP liner of the diversion channel. A bulkhead will be used to seal the upstream end of the CSP liner to the culvert.

5.0 EMBANKMENT DESIGN

5.1 General

The design criterion for establishing the crest level of the embankment extension is based on the peak discharge for the 100-year flood event. As an additional requirement, the stability of the existing road embankment should be improved by construction of the proposed embankment.

Without detailed information for the invert elevation of the culvert, we have assumed that the invert elevation at the intake of the 1500 mm diameter culvert is at El. 1161.5 m. For the existing 1500 mm diameter culvert, the difference between full discharge capacity of the pipe and the peak storm runoff would cause water to pond above the pipe inlet to a height of about 6.5 m measured from the invert of the pipe. To allow for a minimum freeboard of 1 m, the crest of the extension would therefore be established at El. 1169 m. At this elevation, the height of the proposed extension at the culvert location would extend about 10 metres above the downstream toe of the dam.

Construction of the dam extension could be completed either using the Upstream or the Downstream Method of Construction. The main disadvantage associated with the upstream extension technique involves diverting the flow of the creek during construction. Construction of a cofferdam would be required upstream of the road embankment to maintain a dry working environment during placement of the dam fill and construction of the culvert extension. Water retained behind the cofferdam would have to be pumped over the existing road embankment and discharged downstream. The advantages of the upstream extension would include less fill for the construction of the dam, and probably less seepage loss through the structure.

The alternative method of construction would involve building the extension on the downstream side of the existing road embankment. Although considerably more fill would be required for construction of the extension, problems associated with construction dewatering would be minimized. To reduce seepage loss through the foundation beneath the embankment, construction of a cutoff trench would be required below the downstream portion of the embankment. The stability of the existing road embankment would be improved by the construction of the buttress downstream. Therefore, SRK recommends that the proposed extension be constructed using the downstream construction technique.

5.2 Layout

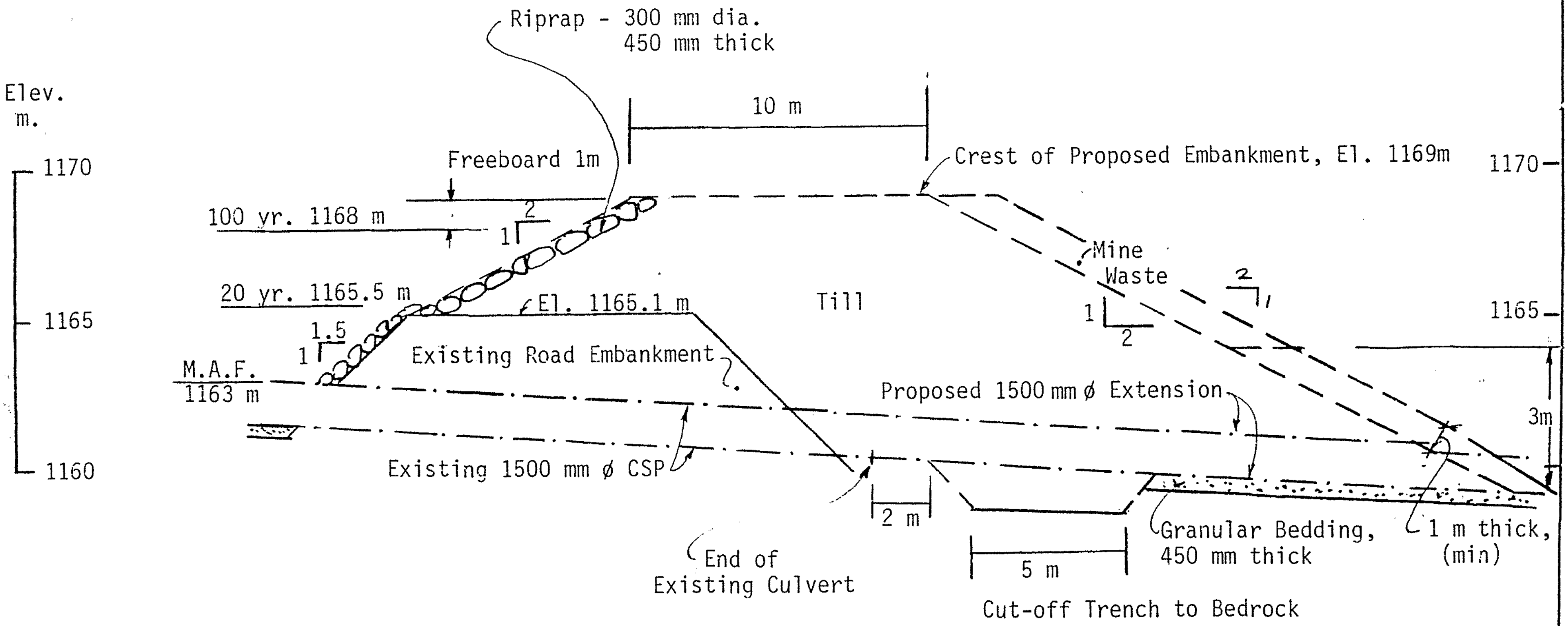
The layout of the proposed extension is shown on Figure 3.

The proposed extension would be constructed to El. 1169 m with upstream and downstream slopes of 2 horizontal:1 vertical. The crest width would be 10 m. The proposed extension would add a maximum height of 4 m to the existing embankment, and would be about 115 m in length. The main section of the embankment would be constructed of glacial till. For seepage control, a cut-off trench would be excavated below the foundation of the proposed extension immediately below the existing road embankment. The cut-off trench would extend to the shallow bedrock, generally at a depth of about 1.5 m below the ground surface. The base of the cut-off trench would be at least 5 m wide, and the cut-off would extend across the full width of the creek. The cut-off trench would be backfilled with compacted glacial till. The upstream slope of the proposed extension would be protected with riprap. In addition, riprap would be required for the upstream slope of the existing road embankment. The final slope of the riprap for the road embankment would be constructed no steeper than 1.5:1. A minimum thickness of 450 mm of riprap should be placed on the upstream slopes of the proposed extension and the existing road embankment. A cross-section of the proposed extension at the culvert location is shown on Figure 4.

SRK recommends that a cover of coarse granular fill be placed on the downstream slope. The purpose of the cover is to reduce the risk of surface sloughing of the glacial till due to seasonal freezing and thawing cycle. The granular cover should consist of mine waste and should be at least 1 m thick. To improve the stability of the cover, the lower 3 m of the cover should be compacted. To achieve compaction of the mine waste, it will be necessary to bring up the lower 3 m in horizontal lifts at the same time the glacial till is being placed for the main extension. The remaining section of the mine waste may be end-dumped from the crest of the proposed extension.

A granular bedding material should be placed beneath the culvert extension. The bedding material should be at least 450 mm thick, and extend at least 300 mm beyond the maximum cross section of the culvert. To reduce seepage loss through the bedding material, the bedding should be deleted upstream of the cut-off trench.

For the recommended embankment configuration, SRK estimates that about 8200 m^3 of fill would be required. Of this volume, about 300 m^3 would be granular bedding, and 500 m^3 for riprap. The remaining 7,400 m^3 would be glacial till. The key trench would involve about 40 m^3 for excavation and backfill. The granular cover on the downstream slope of the proposed embankment would require about 500 m^3 of material.



CROSS SECTION AT THE CULVERT LOCATION

Scale - 1:200

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FIGURE 4

CROSS-SECTION AT THE CULVERT, 'A-A'

5.3 Borrow Materials

Materials required for construction of the proposed embankments would include glacial till, riprap, granular bedding and mine waste.

The glacial till would be obtained from stripping of the overburden at the Vangorda Pit or from other sources approved by the Engineer.

The typical glacial till encountered in the pit consists of an olive brown clayey gravelly sand with occasional cobbles up to about 500 mm in diameter. The insitu moisture content of the till ranges from about 9 to 12 percent. The glacial till is well-graded with about 37 to 43 percent finer than No. 200 sieve size. A gradation envelope for the glacial till is shown on Figure 5. The maximum dry density of the soil would range from 2180 kg/m^3 (135.5 pcf) to 2204 kg/m^3 (137.0 pcf). The corresponding optimum moisture content varies from 7.5 to 8.0 percent. A compaction curve is presented in Appendix A on Figure A-9.

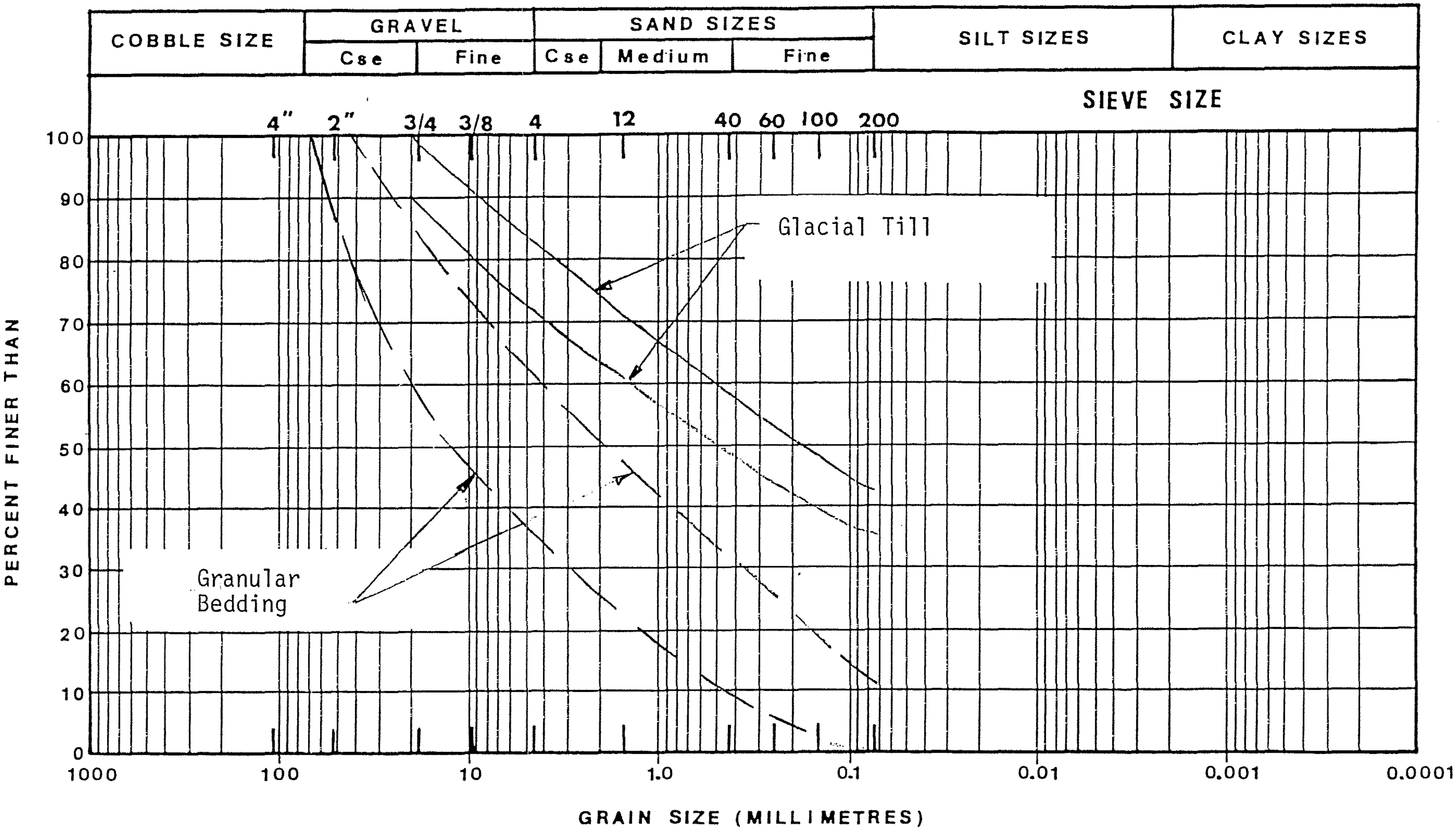
Riprap for protection of the upstream slope of the proposed extension and the existing road embankment as well as for the creek banks upstream of the existing culvert, would be obtained from processing local materials including deposits of granular materials, glacial till and mine waste. The riprap would be well graded between 100 and 400 mm, with a D_{50} of 300 mm.

Bedding for the proposed 1500 mm diameter culvert extension would consist of a well-graded coarse sand or fine gravel with less than 10 percent finer than No. 200 size sieve. A gradation envelope for the granular bedding is shown on Figure 5.

The downstream slope of the proposed extension should be protected by mine waste. The mine waste shall consist of non-acid generating material.

5.4 Stability

The stability of the proposed extension was evaluated using a computer program STABL 2B, which models the Modified Bishop Method of analysis. The program completes a search of different sliding configurations to establish the five most critical sliding surfaces. The factor of safety of each sliding configuration is obtained by comparing the soil strength necessary to maintain equilibrium with the estimated soil parameters. Static and pseudo-static loading conditions were investigated. A horizontal seismic loading of 0.07 g was used. Analyses were performed for the upstream and downstream slopes at the maximum section of the proposed extension.



PROJECT No. 60638	PREPARED BY: STEFFEN ROBERTSON & KIRSTEN Consulting Engineers	FIGURE 5
DATE Nov.15/90		GRADATION ENVELOPE FOR GLACIAL TILL AND GRANULAR BEDDING
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For static conditions, effective strength parameters were used in the analyses. For earthquake loading, total strength parameters were used for potential failure surfaces below the water table.

The following parameters were used in our stability analyses:

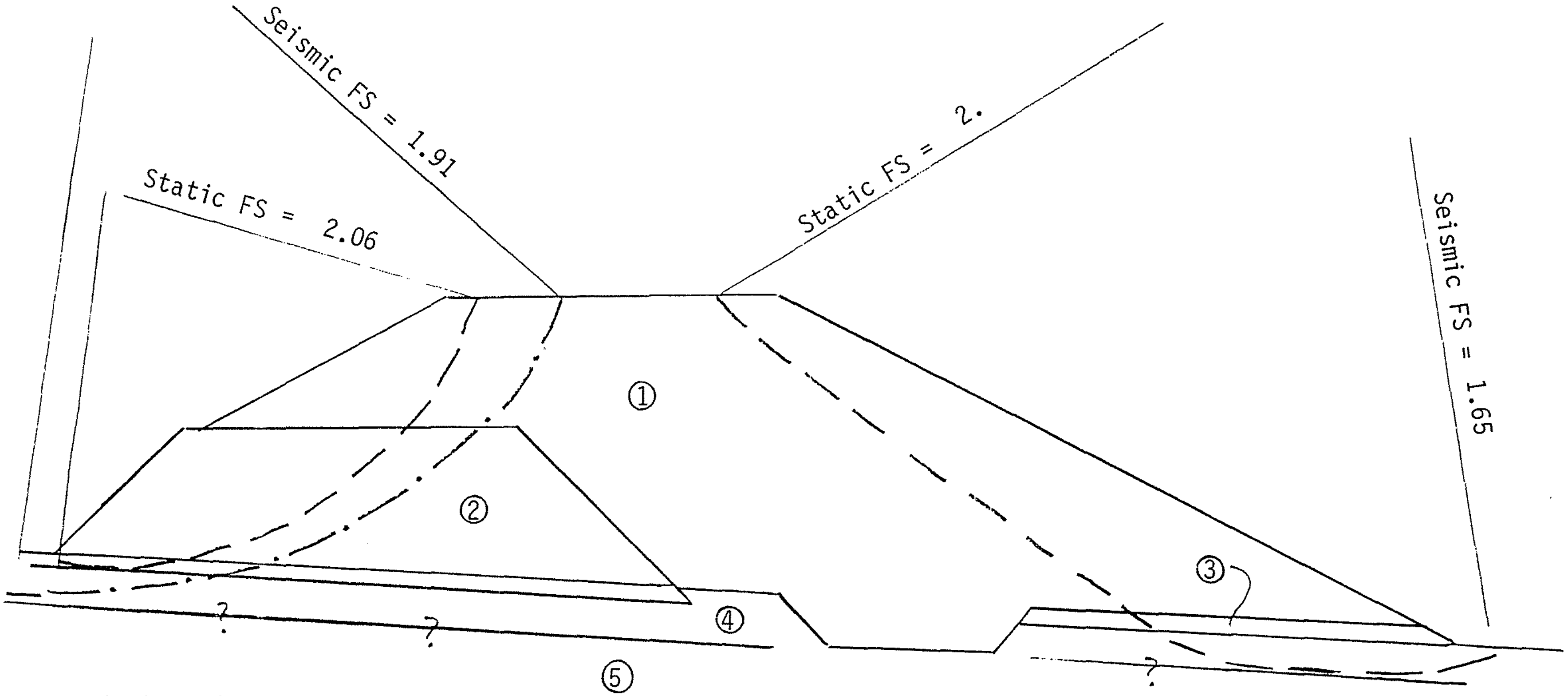
			Strength Parameters			
		Unit	Effective Stress		Total Stress	
Material		Weight (kg/m$^{3)}$	Friction (deg.)	Cohesion (kPa)	Friction (deg.)	Cohesion
Fill	- glacial till	2163	42	0	38	0
	- sandy gravel	2091	35	0	35	0
Foundation	- till	2252	45	1200	42	0

The results of the analyses are summarized below, and also presented in Figure 6:

	Computed Factor of Safety			
Condition	Downstream Slope		Upstream Slope	
	Static	Seismic	Static	Seismic
Water level at 100-year flood	2.06	1.91	2.0	1.65

5.5 Seepage

For the design section, seepage losses would be related to flow through the foundation of the embankment which would be roughly equal to the natural baseflow. Because of the short duration that water would be ponded against the embankment, it is unlikely that the ponding of water would increase foundation seepage or initiate seepage through the embankment.



SOIL PARAMETERS

		γTotal (kg/m^3)	Friction Angle	Cohesion (kPa)
①	Compacted Till	2163	38 - 42	0
②	Sandy Gravel	2091	35	0
③	Granular Bedding	2083	33	0
④	Foundation Till	2252	45	1200
⑤	Bedrock			

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FIGURE 6

STABILITY ANALYSES FOR
THE MAXIMUM SECTION OF
THE PROPOSED EMBANKMENT

6.0 DIVERSION CHANNEL DESIGN

The main diversion channel, which is 800 m in length, is designed as a 2400 mm half round CSP with 0.6 m wide berm set within a riprap lined trapezoidal cross section (see Figure 7). The longitudinal slope of the channel is 0.5%, which ensures subcritical flow at all discharges. A slope of 2%, for example, would lead to supercritical flows for all but the lowest flows. Subcritical flow is considered to be preferable for this application for the following reasons:

- subcritical flow is associated with lower velocities than supercritical flow, and therefore is less likely to cause problems with abrasion of the culvert by suspended load and bed load, which may enter the creek under flood conditions;

- the velocities that would occur during supercritical flow would require much heavier riprap protection to be used on the channel side slopes;

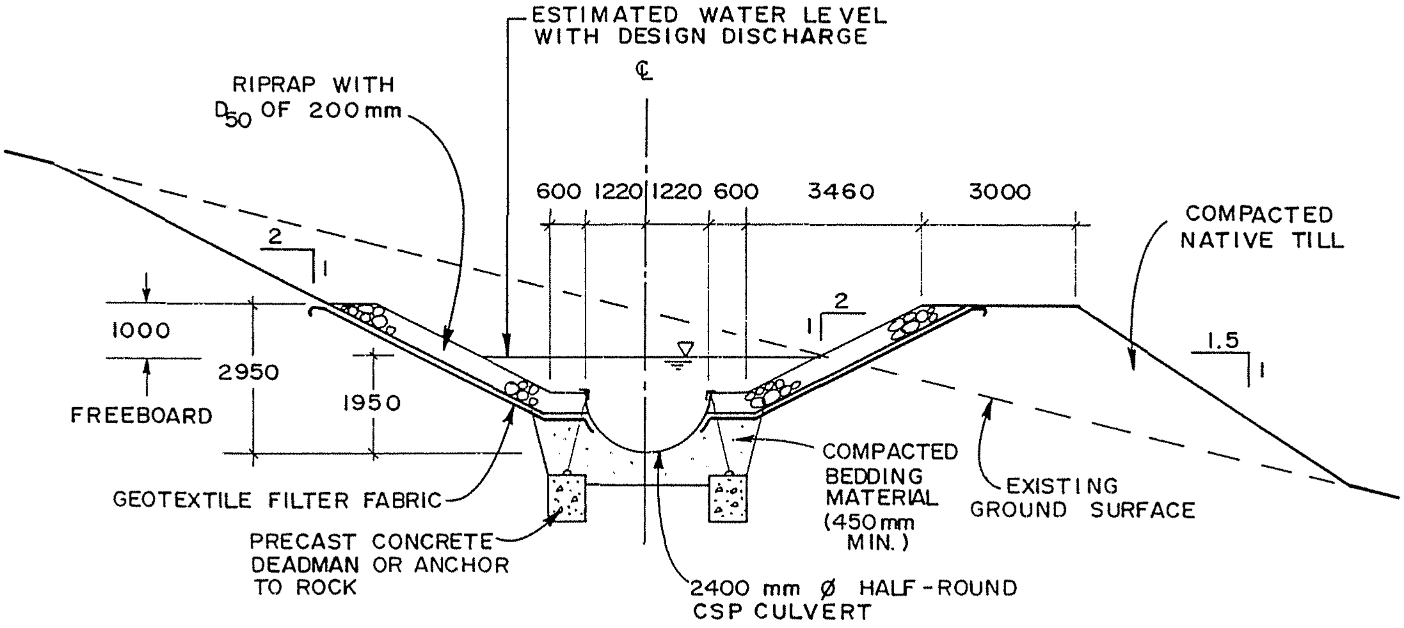
- supercritical flow near the critical state tends to be unstable and small changes in specific energy (such as a change in cross section) cause large changes of depth;

- channel bends cause superelevation of the water surface which is difficult to predict or control, and could cause spillage from the channel;

- air bulking can increase the cross sectional area of the flow by up to 25%, thus negating some of the advantage of the smaller cross section which is required theoretically;

- the mild slope proposed for this section of the diversion will almost certainly ensure that an ice cover forms. If sufficient depth is available in the channel the ice layer will provide insulation and allow free flow beneath it. In flow with a mean velocity of above about 1 to 1.2 m/s, a surface layer does not form until the temperature drops well below zero. In the absence of surface ice, anchor ice and frazil ice can form and reduce the hydraulic capacity of the channel.

The hydraulics of the main diversion under a range of discharges are summarized in Table 1. For most of the diversion life, the flows of Vangorda Creek will be entirely contained within the CSP liner. Exceedances of the half-round culvert will only occur for short periods during flood events. During the mean annual flood the crest of the CSP liner is predicted to be overtopped by about 0.2 m. During the design event, the depth of flow will be about 1.95 m, or 0.73 m above the crest of the CSP liner. For low flow conditions (i.e. estimated average March flows), the mean velocity in the diversion channel will be about 0.3 m/s.



SCALE- 1: 150

DIMENSIONS SHOWN IN MILLIMETRES

DESIGN FACTORS

CHANNEL SLOPE = 0.005

MANNING'S COEFFICIENT FOR HALF-ROUND CULVERT = 0.027

MANNING'S COEFFICIENT FOR TRAPEZOIDAL SECTION = 0.04

DESIGN DISCHARGE (100-YEAR RETURN PERIOD) = 10 m^3/s

CURRAGH RESOURCES INC.	VANGORDA CREEK DIVERSION	DATE NOV. 1990
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Table 1: Hydraulic Summary for Main Diversion Channel

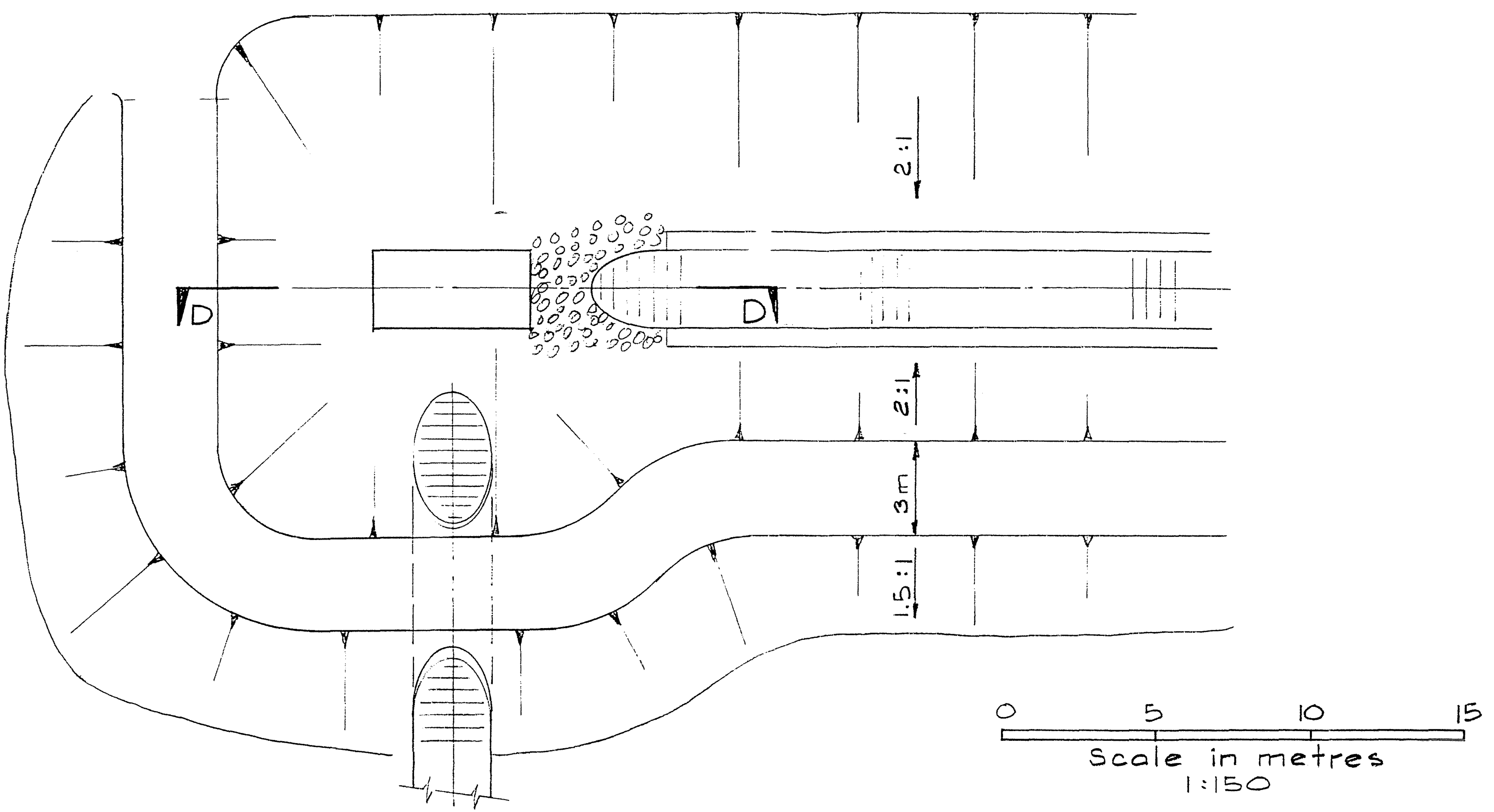
Channel Slope %	Discharge Type	Discharge (m^3/sec)	Normal Depth (m)	Mean Velocity (m/s)	Froude Number[2]
Adopted Slope					
0.5	100-yr flood	10	1.95	1.66[1]	0.55
	20-yr flood	7.6	1.75	1.57[1]	0.55
	MAF	4.5	1.42	1.44[1]	0.55
	low flow[3]	0.016	0.077	0.36	0.50
Steep Slopes					
2	100-yr flood	10	1.48	2.91[1]	1.09
	20-yr flood	7.6	1.12	3.63	1.25
	MAF	4.5	0.84	3.15	1.28
	low flow[3]	0.016	0.056	0.58	0.96
5	100-yr flood	10	1.01	5.47	2.00
	20-yr flood	7.6	0.87	5.07	2.02
	MAF	4.5	0.66	4.38	2.03
	low flow[3]	0.016	0.05	0.80	1.46

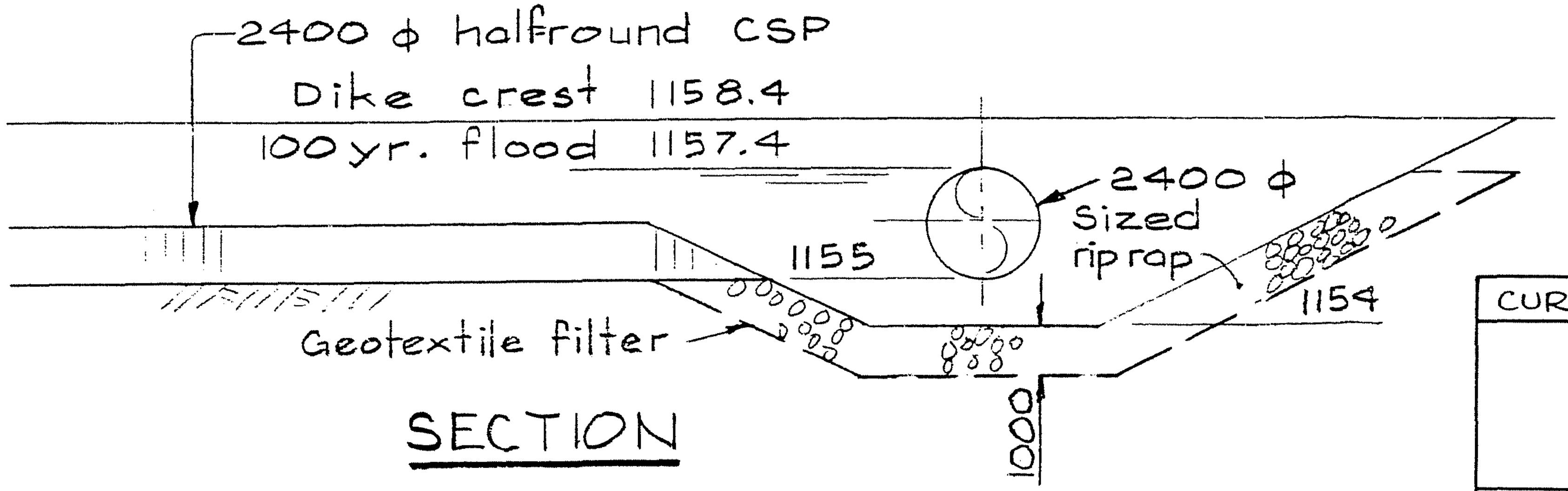
[1] Flow above culvert

[2] Froude No >1, Supercritical flow

[3] Estimated average flow during March, the month which typically experiences lowest annual runoff.For comparison purposes, the hydraulics of channels with steeper slopes (2% and 5%) are also presented in Table 1.

The subcritical section of the diversion channel will end in a transition structure excavated in the hillside (see Figure 8). This will serve as a sediment trap in addition to directing the flow into a chute.





SECTION

CURRAGH RESOURCES INC. VANGORDA CR. DIVERSION	DATE NOV. 1990
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CHUTE TRANSITION STRUCTURE	APPROVED
	FIG. NO. 8
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6.1 Half Round Culvert Chute

A 2400 mm half round culvert will convey the flow from the transition structure at elevation 1155 m to the valley bottom at elevation 1118 m. The flow will be supercritical, consequently the chute will have a uniform grade and straight alignment. The gradient will be 17%. The high velocities anticipated in this chute require that it have a bituminous paved invert, unless the owner is willing to accept the possibility of failure of this structure during the life of the project.

The kinetic energy at the outlet of the chute will be dissipated in an excavated rock lined stilling basin (see Figure 9).

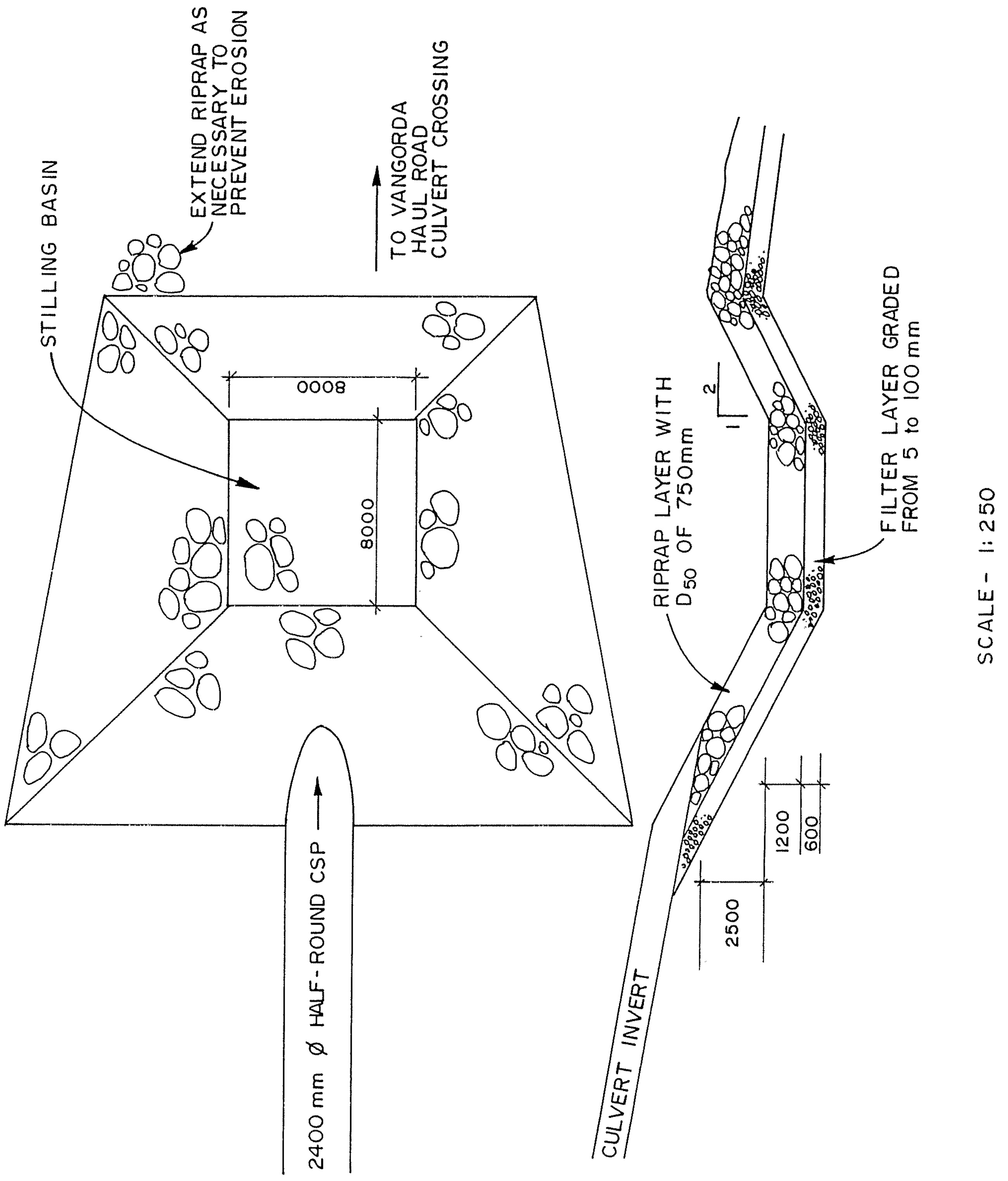
6.2 Vangorda Haul Road Culvert Crossing

Three culverts have been installed in the haul road to pass the flows of Vangorda Creek. Two of these are located in the stream bed and have collapsed somewhere beneath the haul road fill. The hydraulic capacities of these two culverts are unknown. The third culvert is located some 3 m above the stream bed and has the purpose of carrying flood discharges. This upper culvert has a diameter of 1800 mm and an entrance invert of 1122.2 m.

Under design conditions the water level in the valley bottom will pond to elevation 1126 m to enable the flow to pass under the haul road. This calculated headwater depth is based on the conservative assumption that the two lower culverts have minimal capacity and, therefore, the design discharge must be accommodated entirely by the upper culvert. As more information on the geometry of the lower culverts becomes available, the estimated headwater depth may be revised to a lower elevation which would also impact on the 6 m high earthfill embankment which would be constructed near the pit perimeter to prevent back flow into the pit (see Figure 10).

6.3 Back Flow Embankment

At the end of the proposed 1800 mm half round culvert chute located below the pit haul road, a back flow embankment would be required to keep the water from flowing into the Vangorda Pit. The back flow embankment would be constructed with compacted glacial till to Elevation 1127 m. According to the contours and the proposed crest elevation,the back flow embankment would be 5 m high and about 50 m long. The crest of the embankment should have a minimum width of 6 m. The side slopes should be 2H:1V. The fill quantity for the construction of this embankment would be about 4,000 m^3.



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STILLING BASIN DETAIL

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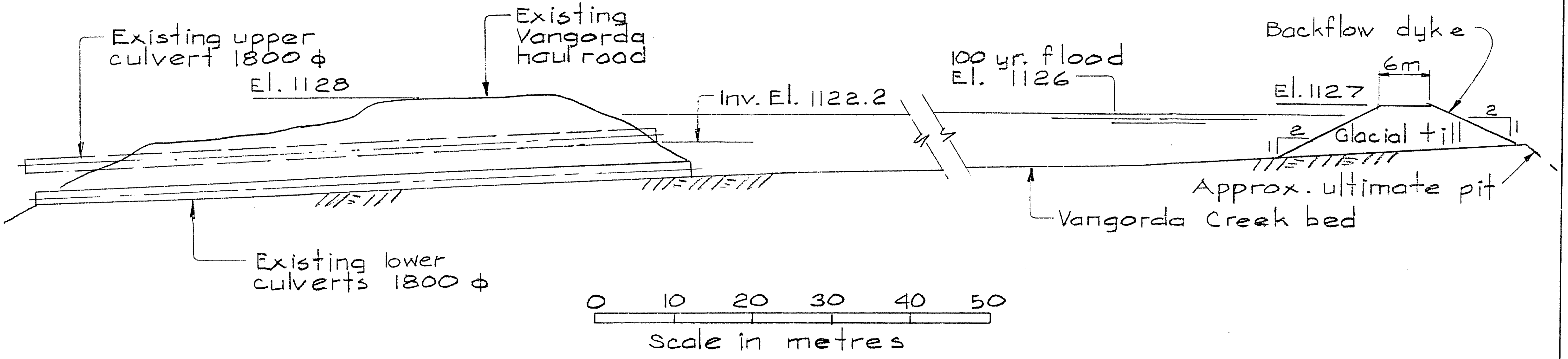

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SECTION C-C

FIG. NO. 10

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7.0 CONSTRUCTION REQUIREMENTS

7.1 Site Preparation

The initial site preparation would include removal of the existing timber wing walls at the outlet of the 1500 mm diameter culvert.

All snow, frozen soil, deleterious material, and vegetation should be stripped from the foundation area and for a distance of at least 3 m beyond the footprint of the proposed structure. Where fill is to be placed on the existing road embankment, the surfaces of the road embankment should also be cleared of snow and vegetation as well as any loose or deleterious materials. At the foundation of the proposed extension, soft sediments accumulated in the creek should also be stripped to acceptable foundation. After site grading, the foundation of the proposed extension should be proof-rolled with 6 passes or to 90 percent Modified Proctor maximum dry density using a 5-ton vibratory roller.

During initial construction of the proposed extension and installation of the proposed 1500 mm diameter culvert extension, some dewatering of the creek would be required to allow fill placement under dry working condition.

7.2 Excavation

Excavation for the cutoff trench should be completed with slopes of 1 : 1. Elsewhere where excavation would be required for the proposed 2400 mm diameter half culvert, for temporary excavation in the clayey till, the unsupported slopes should not exceed 1 (H) : 2 (V). Where the height of excavation exceeds 2.4 m, a safety bench about 2 m wide should be incorporated in the excavated slope. For temporary excavation in sandy gravelly material, the unsupported slope should not exceed 1.3H:1V.

For temporary excavation in good sound rock, the unsupported slope should be cut no steeper than 1H:3V. For poor quality rock, the unsupported slope should be reduced to 1:1. All loose rocks shall be scaled to avoid toppling.

All permanent excavated slopes should be trimmed to a maximum inclination of 1.5H:1V.

7.3 Fill Placement

SRK understands that construction of the proposed embankments would begin in the winter. For construction in extreme cold temperatures, potential for frozen lumps of soil to be incorporated in the embankment fill would be high. In addition, the risk of inadequate binding being developed between successive lifts of fill is high. The two conditions may lead to internal erosion (piping) of the dam fill. Consequently, it would affect the overall performance of the structure. SRK strongly recommends that no fill placement activity be allowed when the ambient temperature drops below -10 degrees Celsius.

Construction would commence with excavating the key trench below the downstream crest of the proposed extension. The key trench should be excavated to bedrock, about 1.5 m below the creek bed. The key trench should be backfilled with glacial till. The till should be placed in horizontal lifts not exceeding 300 mm thick and compacted to at least 98 percent Modified Proctor maximum dry density.

Beneath the 1500 mm diameter culvert extension, and extending at least 300 mm beyond the maximum cross section of the culvert, the granular bedding material should be compacted to 90 percent Modified Proctor maximum dry density.

Placement of glacial till around the proposed culvert extension should be completed in 150 mm thick layers alternating on both sides of the culvert. A small mechanical tamper should be used for compaction in the immediate vicinity of the proposed culvert. In the restricted area around the proposed culvert, hand tamping by a 2 x 4 timber would be required. The heavy vibratory compactor may be used after a minimum soil cover of 1.5 m is provided around the culvert.

Beyond the limits of the soil cover discussed above, the glacial till should be placed in horizontal lifts not exceeding 450 mm, loose measure. The fill should be compacted to a minimum 95 percent Modified Proctor maximum dry density.

To increase the stability for the downstream slope of the proposed extension, the bottom 3 metres of the mine waste should be compacted. The mine waste should be placed in horizontal lifts of 1.5 m thick loose measure and compacted to 90 percent Modified Proctor maximum dry density. The remaining portion of the mine waste should be placed by end-dumping from the crest of the proposed embankment.

For placement of glacial till in extreme cold weather conditions, SRK has prepared the following guidelines:

a) The active mining area of the borrow pit and the proposed embankment sites should be free of snow, ice and ponding water.

b) All fill materials should be free of frozen particles, and maintained unfrozen during initial placement and compaction. To reduce the potential for freezing, this may require constant "working" of the newly placed fill, until it is ready for compaction.

c) Prior to placement of subsequent lifts of fill, the compacted surface should be scarified. Where frozen soils are encountered in the compacted material, the soils should be removed and replaced with unfrozen soil.

d) To reduce time of exposure of fill (in-place or at the borrow pit) to the ambient temperature, it would be prudent for the Contractor to consider operating on a round-the-clock basis.

e) Close inspection for the conditions of fill in the borrow pit and in the embankment, is critical during fill placement.

o O o

If you have any questions regarding this report, please do not hesitate to contact us.

Respectfully submitted,

STEFFEN ROBERTSON AND KIRSTEN (B.C.) INC.

P. Healey

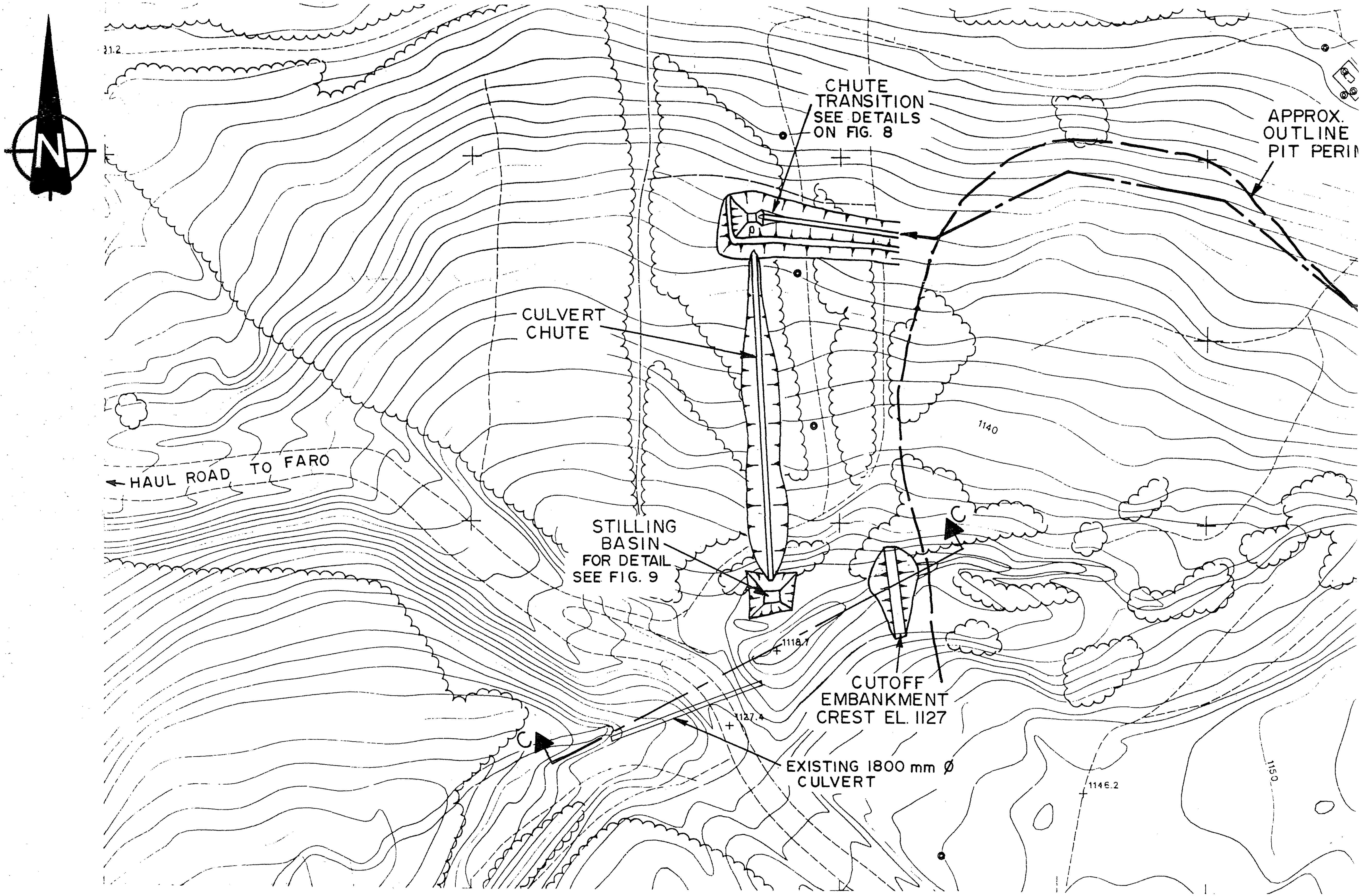
P. M. Healey, P. Eng.
Project Manager

A. G. Chantler

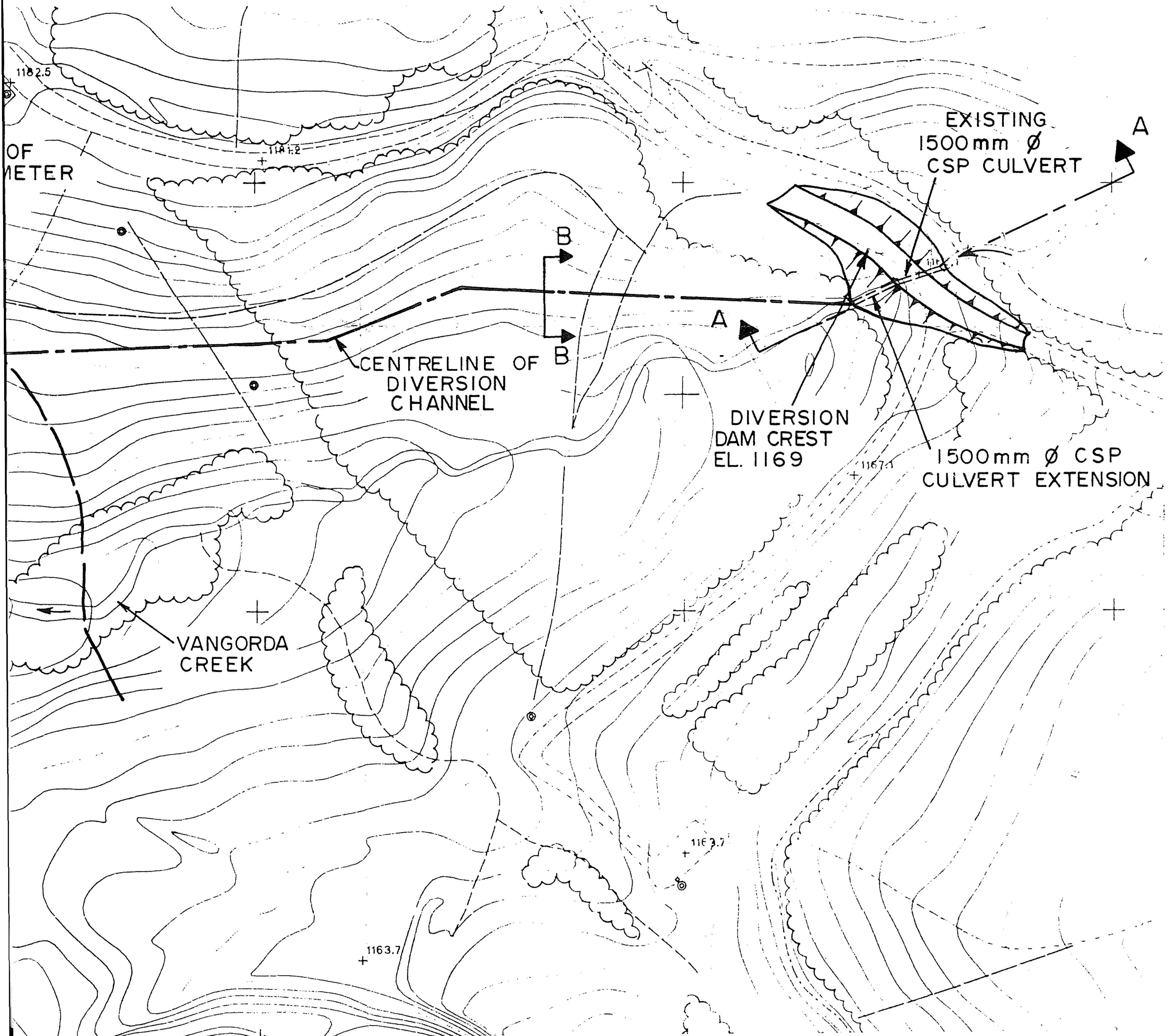
A. G. Chantler, P. Eng.
Review Engineer

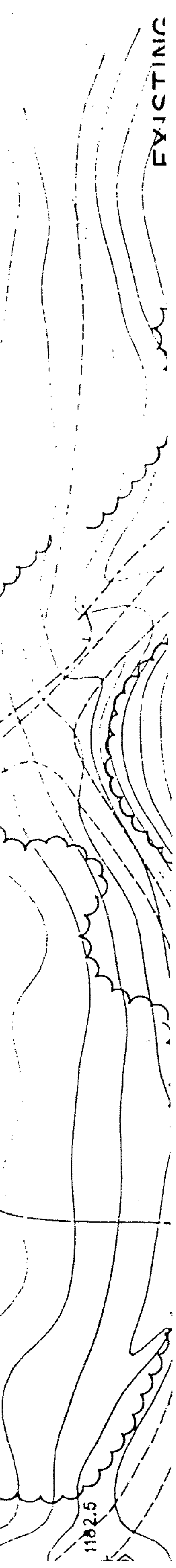
APPENDIX A

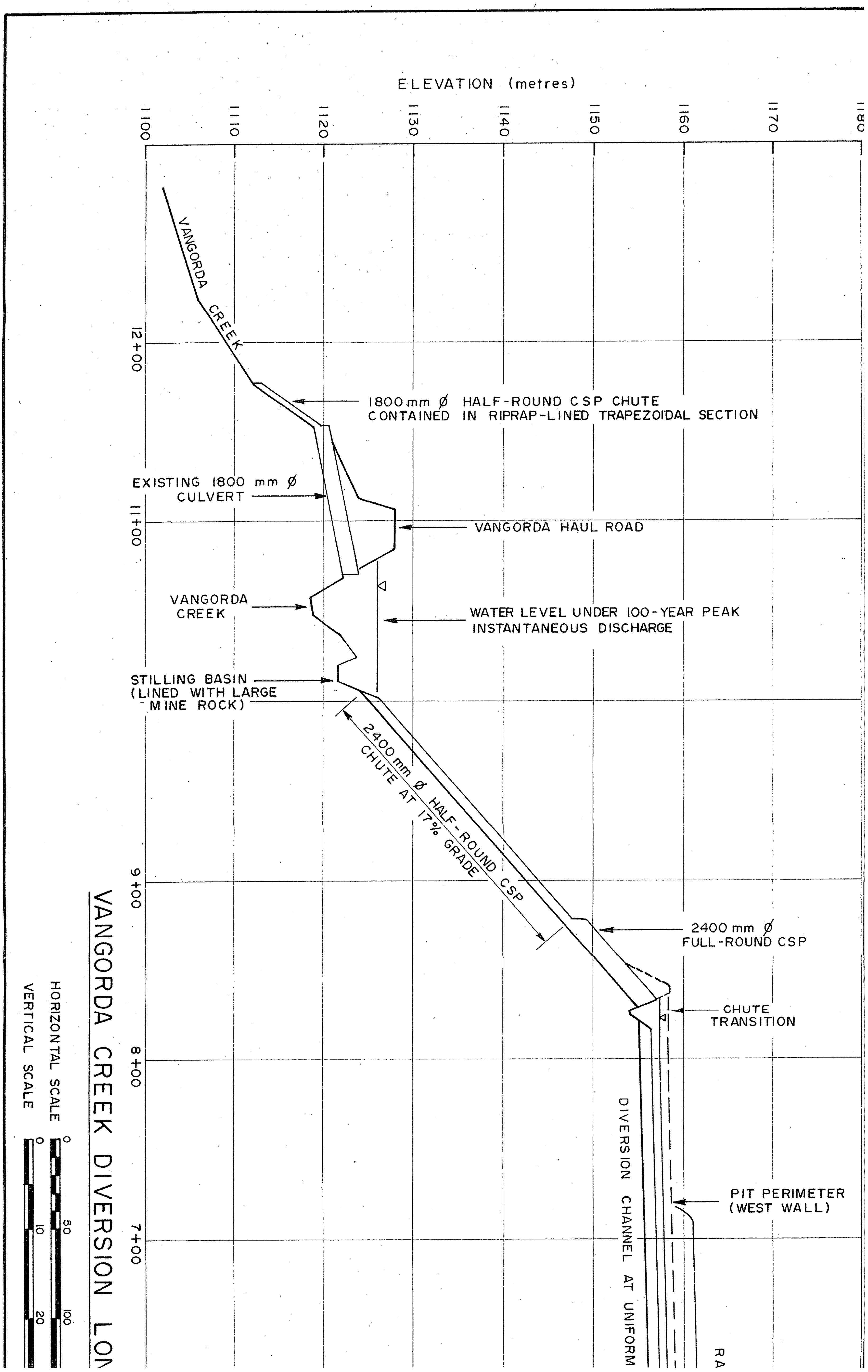
Borehole Logs



PLAN OF VANGORDA CREEK DI


1182.5
OF
METER
1181.2
EXISTING
1500mm Ø
CSP CULVERT
A
B
B
A
CENTRELINE OF
DIVERSION
CHANNEL
DIVERSION
DAM CREST
EL. 1169
1500mm Ø CSP
CULVERT EXTENSION
1167.1
VANGORDA
CREEK
1163.7
1163.7

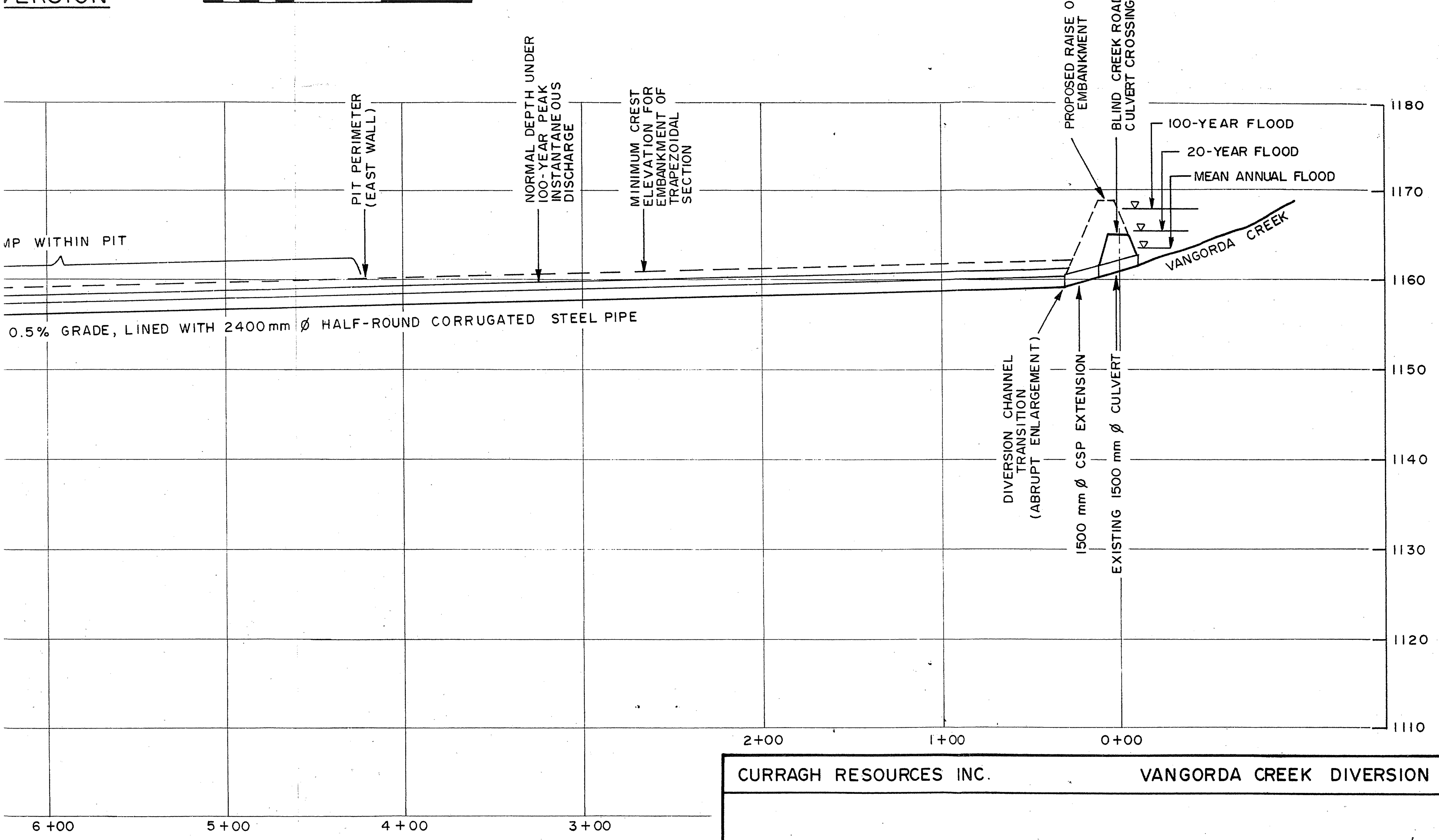




VANGORDA CREEK DIVERSION LON

HORIZONTAL SCALE 0 50 100

VERTICAL SCALE 0 10 20



GITUDINAL PROFILE

150 metres

30 metres

CURRAGH RESOURCES INC.	VANGORDA CREEK DIVERSION	DATE NOV. 1990
		PROJ. NO. 60638
LOCATION PLAN & PROFILE		APPROVED
		NO. 2
STEFFEN ROBERTSON & KIRSTEN, Consulting Engineers		

STEFFEN ROBERTSON & KIRSTEN
Consulting Engineers
LOG OF TEST PIT

TEST PIT NO. 1

LOCATION D/S of culvert, east bank of creek

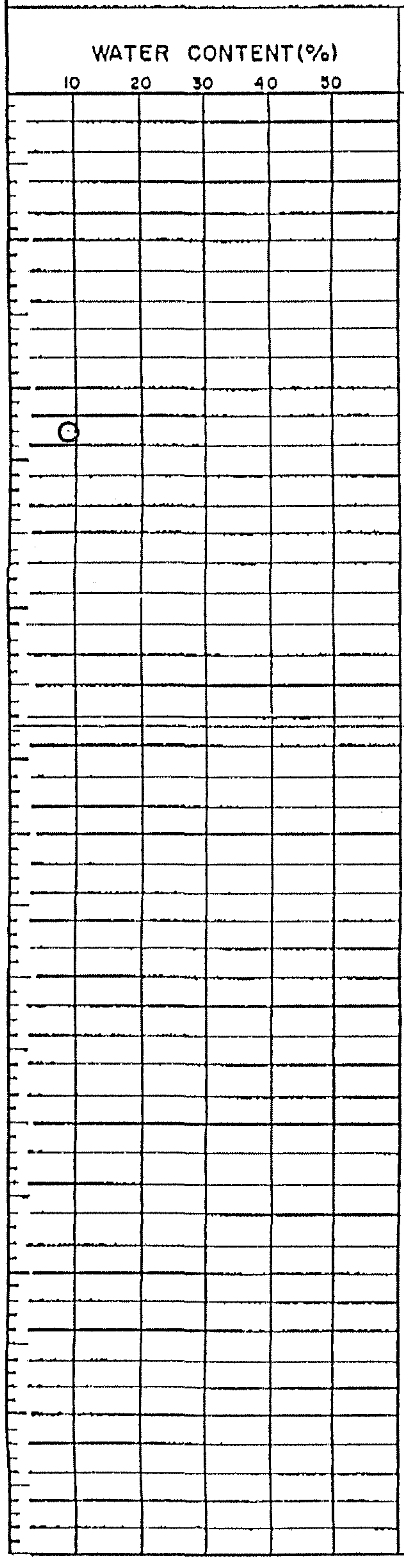
PROJECT Vangorda Creek Diversion Facility, Curragh Resources

DATE November 7, 1990

METHOD Excavator: CAT 235

INSPECTOR C. Gay

TOP OF PIT ELEV. 1164 m (approx)



REMARKS	UNIFIED SOIL CLASSIFICATION SYSTEM		TEST PIT LOG	DEPTH (METRES)
	GP	☐	Brown sandy gravel; with cobbles and boulders up to 1200 mm dia. (dense)	1
		☐	Bedrock; Phyllite thinly foliated, slightly weathered, moderately oxidized, hard	2
			Bottom of pit at 2.1 m.	3

LEGEND
- Gs GRAINSIZE TESTING
- C COMPACTION TEST
- ☐ DISTURBED SAMPLE
- ■ UNDISTURBED SAMPLE
- W_L LIQUID LIMIT
- W_P PLASTIC LIMIT

STEFFEN ROBERTSON & KIRSTEN

Consulting Engineers

LOG OF TEST PIT

TEST PIT NO. 2

LOCATION D/S of Culvert; west bank of creek

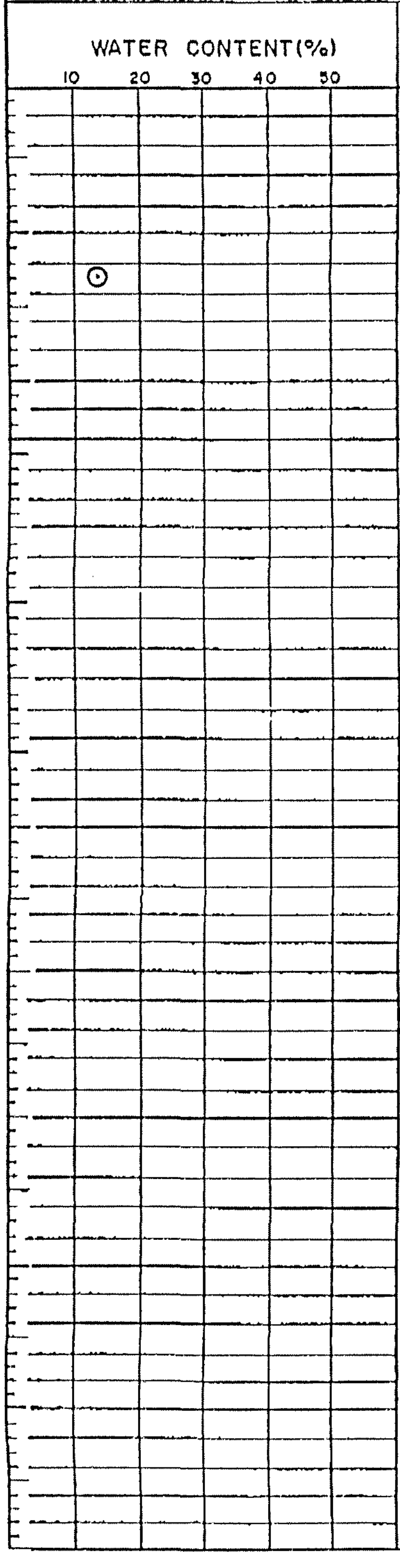
PROJECT Vangorda Creek Diversion Facility, Curragh Resources

DATE November 7, 1990

METHOD Excavator: CAT 235

INSPECTOR C. Gay

TOP OF PIT ELEV. 1161 m (approx.)



ELEV. (FEET)	REMARKS	UNIFIED SOIL CLASSIFICATION SYSTEM		TEST PIT LOG	DEPTH (METRES)
		SM/GM	☐	Brown silty sand and cobbles, some boulders (dense)	
			☐	Bedrock; Phyllite thinly foliated, slightly weathered, moderately oxidized, hard.	1
				Bottom of Pit at 1.2 m	1.5

LEGEND
- Gs GRAINSIZE TESTING
- C COMPACTION TEST
- ☐ DISTURBED SAMPLE
- ■ UNDISTURBED SAMPLE
- W_L LIQUID LIMIT
- W_P PLASTIC LIMIT

STEFFEN ROBERTSON & KIRSTEN

Consulting Engineers

LOG OF TEST PIT

TEST PIT NO. 3

LOCATION D/S Slope of Existing Road Embankment, 25 m west of the 1500 mm dia. culvert

PROJECT Vangorda Creek Diversion Facility, Curragh Res.

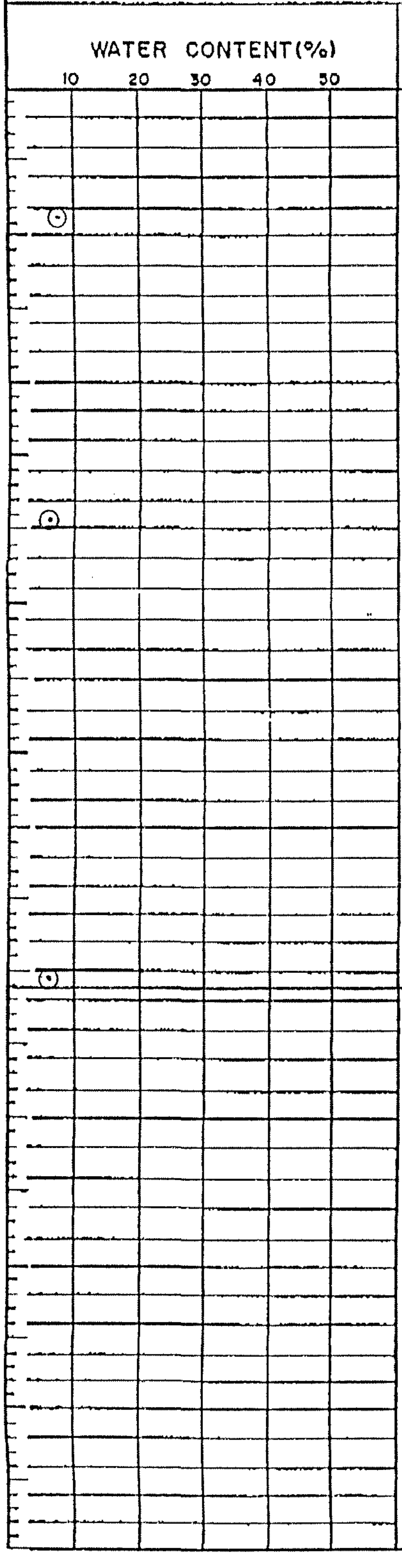
DATE November 7, 1990

METHOD Excavator: CAT 235

INSPECTOR C. Gay

REMARKS

TOP OF PIT ELEV. 1166 m (approx.)



ELEV. (FEET)

UNIFIED SOIL CLASSIFICATION SYSTEM		TEST PIT LOG	DEPTH (METRES)
SP GP	☐	Fill - brown silty sand with cobbles and boulders (m.dense)	1
		Original Topsoil	2
SP GP	☐	Brown sandy gravel, with cobbles and boulders	3 4 5
	☐	residual soil grading to weathered bedrock	6
		Bottom of pit at 6.1m	

LEGEND

- Gs GRAINSIZE TESTING
- C COMPACTION TEST
- ☐ DISTURBED SAMPLE
- ■ UNDISTURBED SAMPLE
- W_L LIQUID LIMIT
- W_P PLASTIC LIMIT

STEFFEN ROBERTSON & KIRSTEN
Consulting Engineers
LOG OF TEST PIT

TEST PIT NO. 4

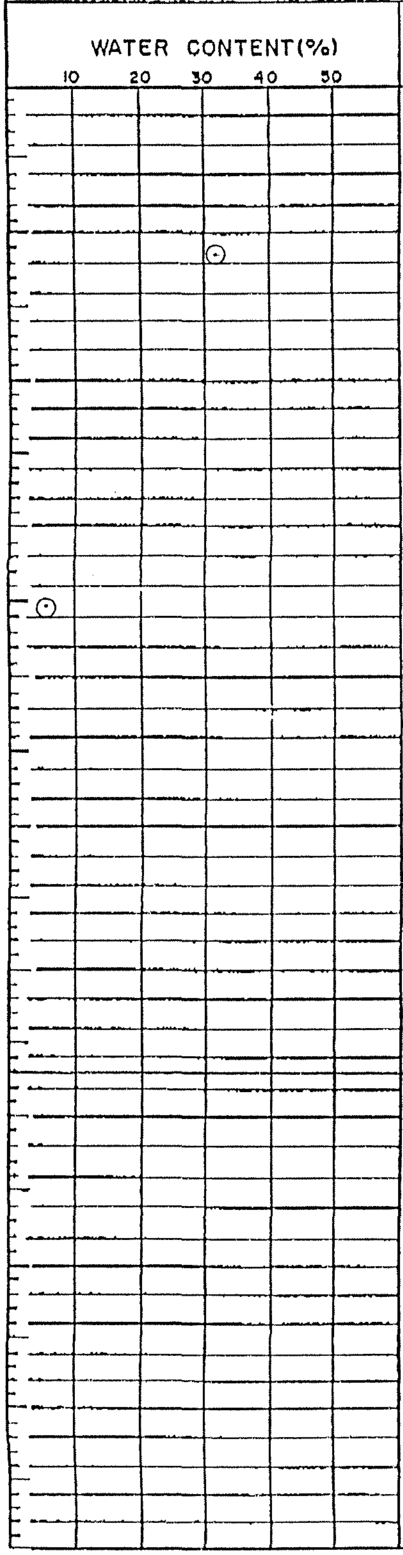
LOCATION D/S Slope of Existing Road Embankment, 30 m east of the 1500 mm dia. culvert

PROJECT Vangorda Creek Diversion Facility, Curragh Resources

DATE November 7, 1990

METHOD Excavator: CAT 235

INSPECTOR C. Gay



ELEV. (FEET)

REMARKS

TOP OF PIT ELEV. 1167 (approx)

UNIFIED SOIL CLASSIFICATION SYSTEM

TEST PIT LOG

DEPTH (METRES)

GM

Brown silty sand and gravel, with cobbles and boulders (m. dense)

☐

1

☐

2

☐

3

Bedrock; Phyllite thinly foliated, slightly weathered, moderately oxidized, hard

Bottom of Pit at 3.4 m.

LEGEND

G	GRAINSIZE TESTING
C	COMPACTION TEST
☐	DISTURBED SAMPLE
■	UNDISTURBED SAMPLE
W_L	LIQUID LIMIT
W_P	PLASTIC LIMIT

STEFFEN ROBERTSON & KIRSTEN
Consulting Engineers

LOG OF TEST PIT

TEST PIT NO. 5

LOCATION U/S Slope of Existing Road Embankment, 55 m west of the 1500 mm dia. culvert

PROJECT Vangorda Creek Diversion Facility, Curragh Resources

DATE November 7, 1990

METHOD Excavator: CAT 235

INSPECTOR C. Gay

WATER CONTENT(%) 10 20 30 40 50

ELEV. (FEET)

REMARKS

TOP OF PIT ELEV. 1168 m (approx)

TEST PIT LOG

DEPTH (METRES)

UNIFIED SOIL CLASSIFICATION SYSTEM

SM GM	☐	Till - brown silty sand with cobbles and boulders (dense)
		1
		2
		3
		Bedrock; Phyllite thinly foliated, slightly weathered, moderately oxidized, hard.
	☐	4
		Bottom of Pit at 4.6 m.
		5

LEGEND
Gs GRAINSIZE TESTING
C COMPACTION TEST
☐ DISTURBED SAMPLE
■ UNDISTURBED SAMPLE
W_L LIQUID LIMIT
W_P PLASTIC LIMIT

STEFFEN ROBERTSON & KIRSTEN

Consulting Engineers

LOG OF TEST PIT

TEST PIT NO. 6

PROJECT Vangorda Creek Diversion Facility, Curragh Resources

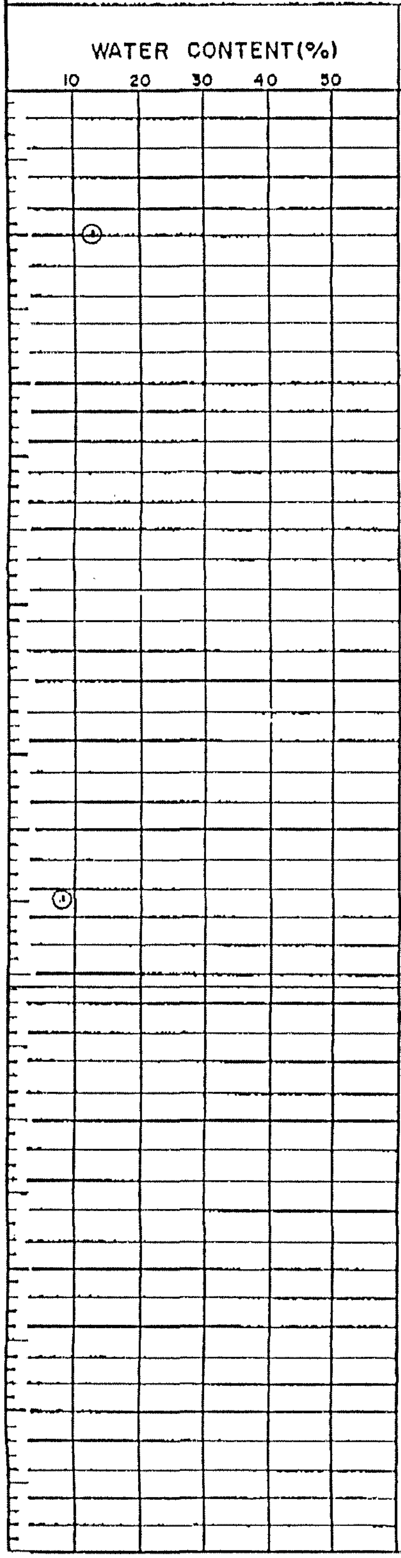
LOCATION U/S of Culvert; west bank of creek.

DATE November 7. 1990

METHOD Excavator: CAT 235

INSPECTOR C. Gay

TOP OF PIT ELEV. 1164 m. (approx)



ELEV. (FEET)

REMARKS

UNIFIED SOIL CLASSIFICATION SYSTEM

UNIFIED SOIL CLASSIFICATION SYSTEM	Sample	TEST PIT LOG	DEPTH (METRES)
GM/GP		Fill - brown sandy gravel, with cobbles and boulders (m.dense)	
	☐		1
			2
		Original Topsoil	
GP		Brown sandy gravel with cobbles and boulders (dense)	3
			4
			5
	☐	Bedrock; Phyllite thinly foliated, slightly weathered, moderately oxidized, hard.	6
		Bottom of Pit at 6.1 m.	
			7

LEGEND

Gs GRAINSIZE TESTING

C COMPACTION TEST

☐ DISTURBED SAMPLE

■ UNDISTURBED SAMPLE

W_L LIQUID LIMIT

W_P PLASTIC LIMIT

STEFFEN ROBERTSON & KIRSTEN
Consulting Engineers

LOG OF TEST PIT

TEST PIT NO. 7

LOCATION U/S of Culvert; East Bank of Creek.

PROJECT Vangorda Creek Diversion Facility, Curragh Resources

DATE November 7, 1990

METHOD Excavator: CAT 235

INSPECTOR C. Gay

WATER CONTENT(%) 10 20 30 40 50

ELEV. (FEET)

REMARKS

TOP OF PIT ELEV. 1164 m (approx)

UNIFIED SOIL CLASSIFICATION SYSTEM

TEST PIT LOG

DEPTH (METRES)

USCS		Test Pit Log	Depth (metres)
GP		Brown sand and gravel, cobbly with lots of boulders	0.5
	☐	Bedrock; Phyllite thinly foliated, slightly weathered, moderately oxidized, hard	1
		Bottom of Pit at 1.2 m.	

LEGEND
- Gs GRAINSIZE TESTING
- C COMPACTION TEST
- ☐ DISTURBED SAMPLE
- ■ UNDISTURBED SAMPLE
- W_L LIQUID LIMIT
- W_P PLASTIC LIMIT

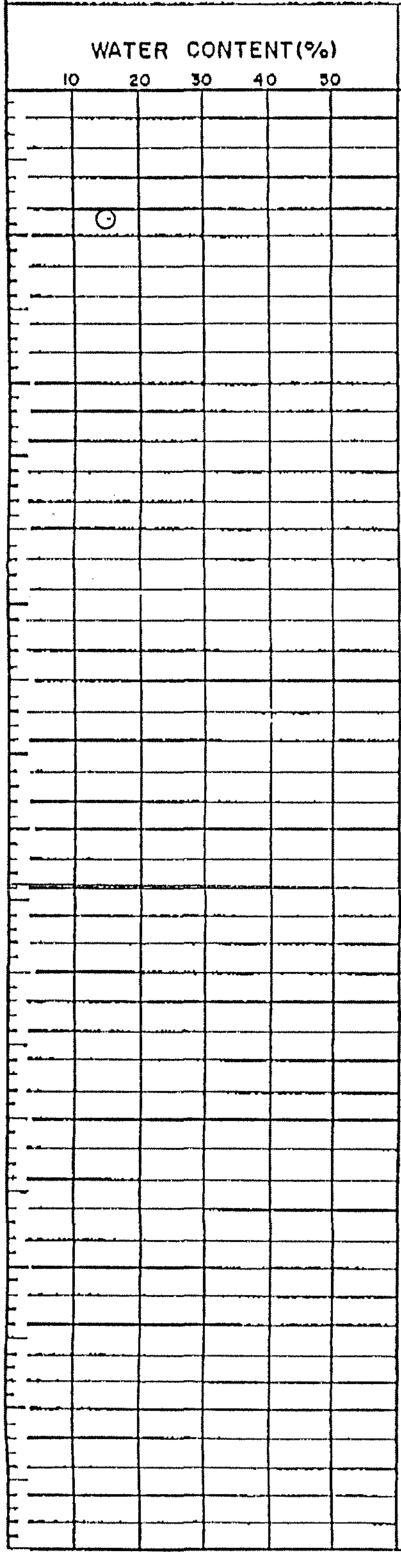
STEFFEN ROBERTSON & KIRSTEN
Consulting Engineers

LOG OF TEST PIT

TEST PIT NO. 8

LOCATION U/S Slope of Existing Road Embankment; 45 m east of the 1500 mm dia. culvert

PROJECT Vangorda Creek Diversion Facility, Curragh Resources
DATE Nov. 7, 1990
METHOD Excavator, CAT 235
INSPECTOR C. Gay



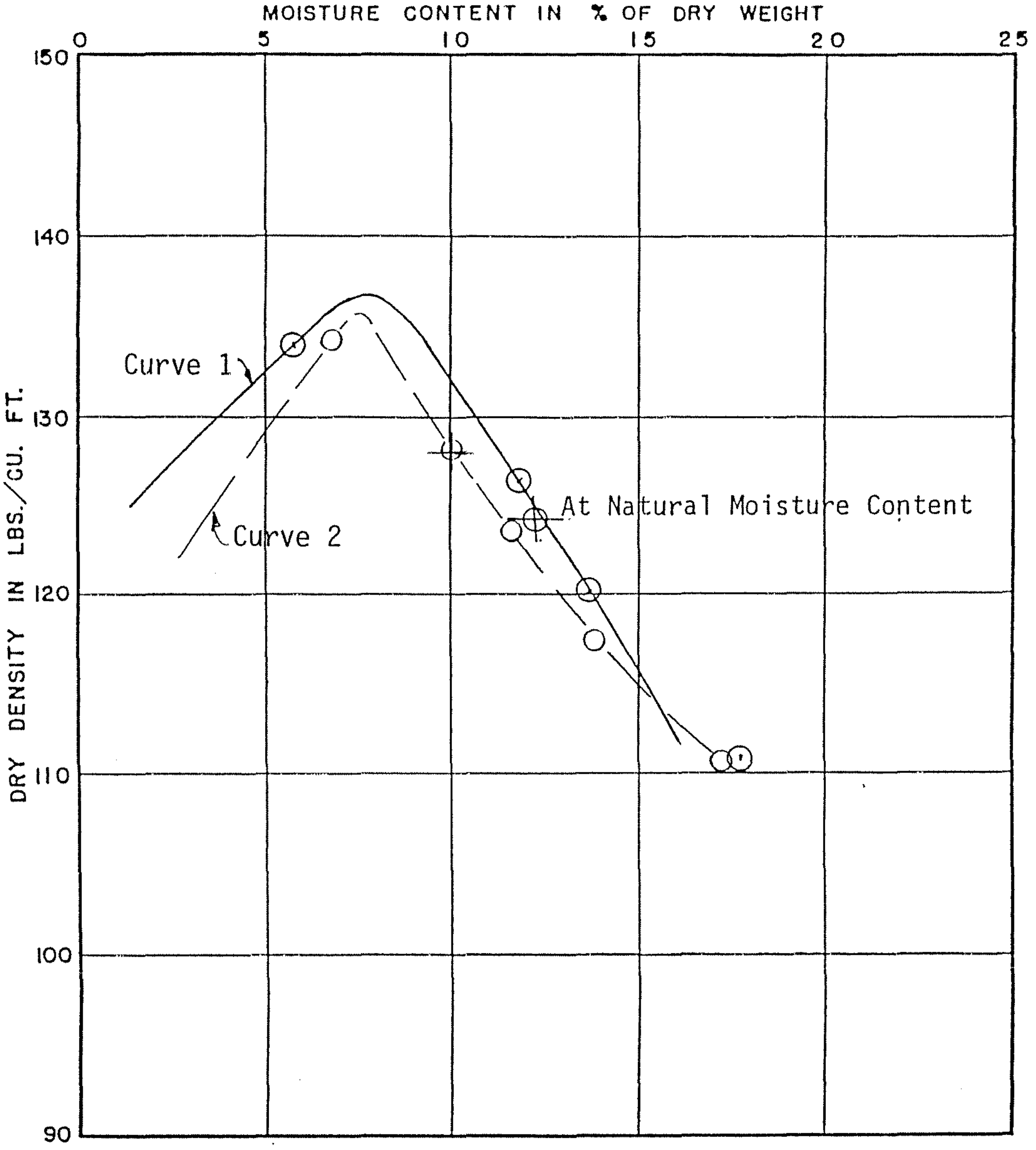
ELEV. (FEET)

REMARKS

TOP OF PIT ELEV. 1168 m (approx)

UNIFIED SOIL CLASSIFICATION SYSTEM		TEST PIT LOG	DEPTH (METRES)
SM GM	☐	Till - grey silty sand with some gravel (m. dense)	1
GP		Brown sandy gravel; abundance of cobbles and boulders up to 1200 mm dia. (dense)	2 3 4
	☐	Bedrock; Phyllite thinly foliated, slightly weathered, moderately oxidized, hard.	5
		Bottom of Pit at 5.5 m.	6

LEGEND
- Gs GRAINSIZE TESTING
- C COMPACTION TEST
- ☐ DISTURBED SAMPLE
- ■ UNDISTURBED SAMPLE
- W_L LIQUID LIMIT
- W_P PLASTIC LIMIT



Soil Type: Olive brown clayey sand/gravel

	Curve 1	Curve 2
Optimum Moisture Content (%)	8	7.5
Maximum Dry Density (pcf)	137.0	135.5

* Tests completed in accordance with ASTM D1557-78, Modified Proctor Maximum Dry Density.

PROJECT No.
60625

DATE
May 24/90

REVISION

PREPARED BY:

STEFFEN ROBERTSON & KIRSTEN
Consulting Engineers

FIGURE A-9

COMPACTION CURVES FOR COMPOSITE TILL SAMPLES OBTAINED FROM VANGORDA PIT

APPENDIX B

Hydrology

3.2.1.3 Site Hydrology

The watershed for the Vangorda Plateau, which covers an area of 18.8 square kilometres, drops about 760 metres in elevation from the highest point, over a distance of about 8 kilometres. In the upper reaches of the basins, there are extensive granitic rock outcrops with alpine tundra. In the lower areas of the basin the rock outcrops are less extensive and much of the area is underlain by relatively impervious tills. In general, as the watershed has very little topsoil or organic litter and virtually no tree cover, the potential for runoff is high. An outline of the drainage boundary for the Vangorda minesite watershed is shown on Figure 1.2.

3.2.1.4 Design Peak Flows

Estimates of peak flows in the Vangorda Creek are required to design and evaluate dams, culverts and diversion structures both during the mining operation and in preparing for abandonment. Several hydrological studies on Vangorda Creek have been completed by a number of consultants and Government agencies over the last few years. These studies include:

- Vangorda Creek Flood Estimates for Kerr-AEX Grum Joint Venture (Montreal Engineering 1978)
- Hydrologic and Hydraulic Design of Down Valley Tailings Disposal Project (Hydrocon Engineering 1980)
- Method of Flood Estimates for the Rose Creek Reservoir Study (Acres 1985)
- Yukon River Basin Flood Risk Study (Underwood McLellan Ltd., 1983)
- Yukon Regional Analysis - DIAND (Janowicz 1986)

Curragh Resources was asked by the Regional Environmental Review Committee (RERC) to re-evaluate the flood estimates for Vangorda Creek as part of the IEE. The results of this review were presented in a report titled "Review of Peak Flow Estimates for Rose and Vangorda Creeks, June 1988".

The Yukon Regional Analysis Method developed by DIAND (Janowicz, 1986), which adopts a regional approach, was developed for estimating peak flows for the design of hydraulic structures in the Yukon Territory. Single station flow-frequency analyses were carried out on 94 hydrometric stations (DIAND and WSC stations) with at least 12 years of record using a two parameter log-normal theoretical probability distribution (no skew factor) which was considered by the author to be the most appropriate for sparse data regions. Simple linear regression relationships were developed between maximum annual instantaneous discharge at selected return periods and drainage area for two hydrologic regions. A second analysis was conducted including a storage index factor to account for lake storage within a basin.

The peak flow estimates for the Vangorda watershed using the DIAND (1986) method were calculated using the following equation:

$Qn = a(D.A.)^b$
where Q_n = Peak flow with a return period of n years in m^3/s
D.A. = Drainage Area in square kilometres.
a = regression constant
b = regression coefficients.

A summary of the peak flow estimates at various return periods for the Vangorda Watershed above the proposed diversion using the DIAND (1986) method is presented in Table 3.8

TABLE 3.8

PEAK FLOW ESTIMATES FOR
VANGORDA CREEK ABOVE DIVERSION
(Drainage Area 18.8 sq.km)

Return Period	a	b	Peak Discharge (cu.m./s)
MAF	.23	.87	2.95
10	.39	.85	4.72
50	.629	.811	6.78

The primary motivation for developing the DIAND regional analysis was to provide a simple methodology for the design of temporary structures such as the haul road culverts. For the permanent hydraulic structures and the in-pit culvert, DIAND method was considered inadequate. A fair degree of uncertainty exists in applying this method to the mine site drainages because of three limitations in the data set used to develop the DIAND regional analysis. Firstly, the majority of the regional streamflow gauging stations command significantly larger catchments than the mine site drainages and, therefore, may experience a different flood-producing mechanism. The greatest floods on large catchments are due solely to snowmelt, whereas those on small catchments are probably caused by extreme rainfall events. Secondly, the streamflow records span relatively short periods, especially in the case of small catchments, making the projection of extreme flood events difficult. Thirdly, the DIAND regional analysis makes use of streamflow gauging stations that cover a large region and that encompass catchments with a wide range of physiographic characteristics (such as elevation and aspect). The flood-prediction relationships derived by this regionalization are therefore characteristically general in nature (i.e. the regional flood data exhibit a fairly wide scatter around the relationships developed for the DIAND method). This scatter is of little consequence when applying the DIAND method to the design of temporary structures but becomes important when dealing with the permanent mine site structures. Due to the uncertainty associated with application of the DIAND regional analysis, an independent study of the Vangorda Creek flood hydrology was conducted.

Two methods were employed for predicting flood discharges on the mine site drainages. The first method was a "focused" regional analysis which was derived in a similar fashion as the DIAND regional analysis except that it only made use of streamflow gaugings in relatively close proximity to the proposed mine development. The rationale behind adopting this approach was that, by using only local streamflow data, a set of specific flood-prediction relationships could be developed for the project area that exhibit narrower confidence intervals than shown by the more general DIAND relationships. The second method used involved the application of a rainfall/runoff simulation model. This was in recognition that rain storms are probably the primary flood-generation mechanism on small catchments.

A total of 9 regional streamflow gaugings, 5 operated by the Water Survey of Canada (WSC) and 4 operated by DIAND, were selected for developing the "focused" regional analysis. The WSC streamflow records span relatively long periods (12 to 35 years), represent runoff from large catchments (997 to 49,000 km^2) and are operated throughout the year. In contrast, the DIAND records are short (9 to 12 years), represent runoff from small catchments (83.1 to 183 km^2) and are operated seasonally. Table 3.9 provides details of the selected streamflow gauging stations.

The DIAND have recently published an updated version of their flood-estimation technique which is contained in the report entitled "Design Flood Estimating Guidelines for the Yukon Territory" (Janowicz, 1989). The flood frequency analyses used in preparation of the above mentioned publication were obtained from DIAND and adopted for the "focused" regional analysis. From these frequency analyses, the 2-, 10-, 50- and 100-year projected flood estimates were abstracted for each of the 9 regional streamflow gauging stations. The 2-year return period floods for the regional stations were then plotted on a logarithmic graph of unit discharge versus catchment area (see Figure 3.1). Inspection of this graph reveals that all but one of the regional stations generally follow the same trend between flood hydrology and catchment area. The exception is the gauging station located near the mouth of Vangorda Creek, the station that would be expected to best characterize the flood hydrology at the mine site. However, the Vangorda Creek streamflow record has a low level of reliability due to extensive periods of missing data, especially during freshet conditions. For this reason, it was decided to reject the Vangorda Creek streamflow record in deriving the regional flood hydrology relationships. Figure 3.1 shows the linear regression relationship fitted to the flood hydrology data.

TABLE 3.9
DETAILS OF REGIONAL STREAMFLOW GAUGING STATIONS

Gauging Station ID No.	Name	Catchment Area (Km^2)	Latitude		Longitude		Record Length (years)
			deg	min	deg	min	
WATER SURVEY OF CANADA							
09BC001	Pelly River at Pelly Crossing	49000	62	50	136	35	35
09BC004	Pelly River Below Vangorda Creek	22100	62	13	133	23	15
09BC002	Pelly River at Ross River	18400	61	59	132	27	21
09BA001	Ross River at Ross River	7250	62	00	132	23	24
09BB001	South MacMillan River at Kilometre 407 Canol Road	997	62	55	130	32	12
DEPARTMENT OF INDIAN AFFAIRS AND NORTHERN DEVELOPMENT							
29BC003	Vangorda Creek at Faro * Townsite Road	91.2	62	14	133	23	9
29BA002	180 Mile Creek at Kilometre 295.8 North Canol Highway	83.1	62	18	131	41	12
29BB001	Boulder Creek at Kilometre 387.0 North Canol Highway	84.1	62	52	130	51	9
29BB002	South MacMillan River #2 at Kilometre 438.6 North Canol Highway	183	63	06	130	12	11

* Incomplete data where peak freshet not always recorded

A similar procedure as detailed above for the 2-year event was undertaken for the 10-, 50- and 100-year return period floods. The results of the analyses are graphically presented in Figures 3.1 and 3.2. For comparison, the appropriate relationship derived by DIAND is also drawn on each of the figures. As can be observed, the "focused" regionalization predicts greater flood discharges than does the DIAND regionalization for small catchments. However, the differences in the two methods decrease as the return period increases. The predicted peak instantaneous flood magnitudes are presented in Figure 3.3 in the form of a frequency diagram.

The analysis for the second flood-prediction method, the SCS Unit Hydrograph, was performed using HEC-1 (U.S. Army Corps of Engineers, 1987) which is a standard computer model for simulating the precipitation/surface runoff process. Interception and infiltration were determined using the SCS curve number index which groups catchments according to soil type, land use and antecedent soil moisture conditions. A curve number of 80 was selected for the mine site drainage and represents relatively low interception and infiltration losses for a rural catchment with clayey soils and average antecedent soil moisture condition (McCuen, 1982).

The SCS dimensionless unit hydrograph was adapted to the Vangorda Creek drainage by providing an estimate of the lag time between the centroid of rainfall excess and the peak flow. The lag time was assessed at 2.2 hours, as calculated using an empirical equation developed by the SCS that depends on catchment characteristics (McCuen, 1982).

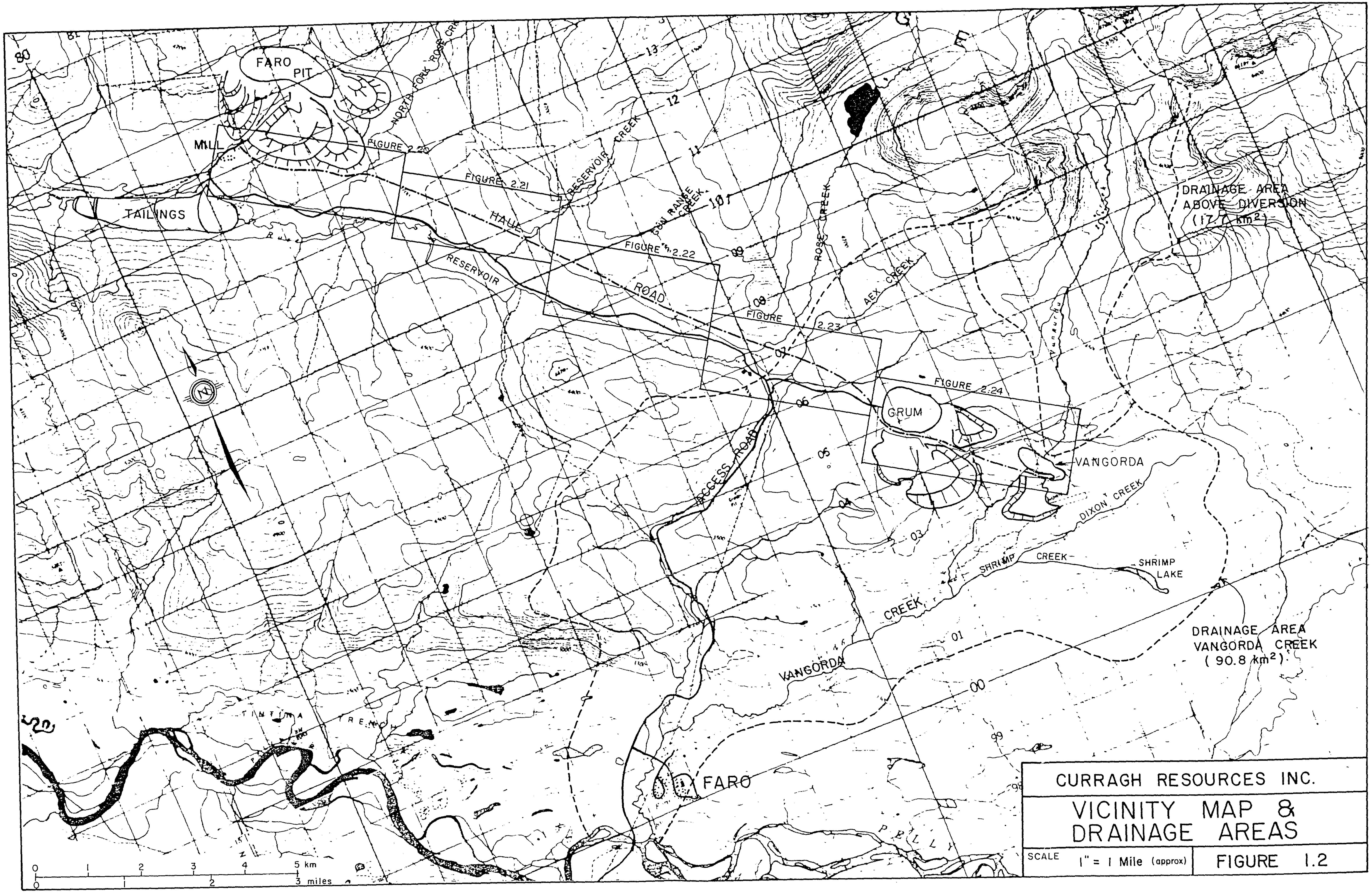
In order to run the HEC-1 model, it was necessary to provide rainfall storm data in the form of an intensity - duration - frequency (IDF) curve. Representative IDF curves for the mine site drainage were derived from the "Rainfall Frequency Atlas for Canada" (Environment Canada, 1985) which is a compilation of maps giving rainstorm amounts for various durations and return periods. The constructed intensity - duration curves for various return intervals are presented in Figure 3.4.

The results derived from executing the HEC-1 model for the 50-,100-,200- and 500- year rain storm events are plotted on Figure 3.3

Inspection of Figure 3.3 shows that the "focused" regionalization predicts higher flood discharges than does the SCS Unit Hydrograph for return periods less than about 100 years. The situation is reversed for return periods greater than 100 years. The character of the frequency diagram shown in Figure 3.3 reflects the assumption that two mechanisms, (i.e., snowmelt and intense rainfall, generate the major flooding on the Vangorda Creek catchment. The results of the flood analysis suggest that small (i.e. more frequent) floods are due to snowmelt, as projected by the focused regionalization, while greater floods are caused by intense summer storms, as simulated by the rainfall/surface runoff modelling.

For design of permanent hydraulic structures at the minesite, including the in-pit culvert it is recommended that the higher value predicted by the two methods be adopted. The peak instanteous flood discharges at the diversion site on Vangorda Creek for various return periods would be as follows:

Return Period (years)	Peak Discharge (cu.m/s)
2	4.1
MAF	4.2
10	6.6
20	7.6
50	9.0
100	10.0
500	18.0



CURRAGH RESOURCES INC.

VICINITY MAP &
DRAINAGE AREAS

SCALE 1" = 1 Mile (approx)

FIGURE 1.2

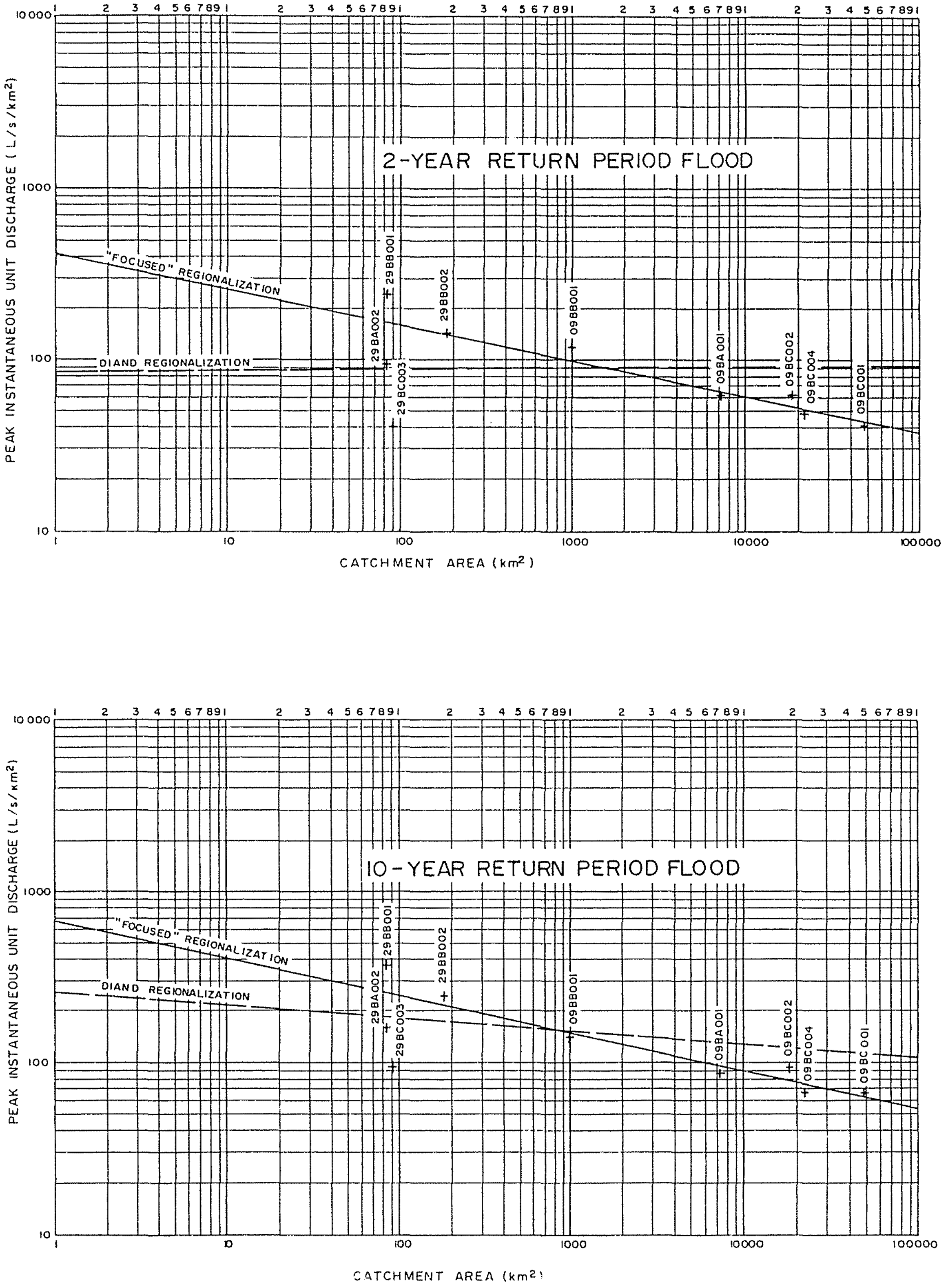


FIG. 3.1 REGIONAL RELATIONSHIPS BETWEEN FLOOD HYDROLOGY AND CATCHMENT AREA

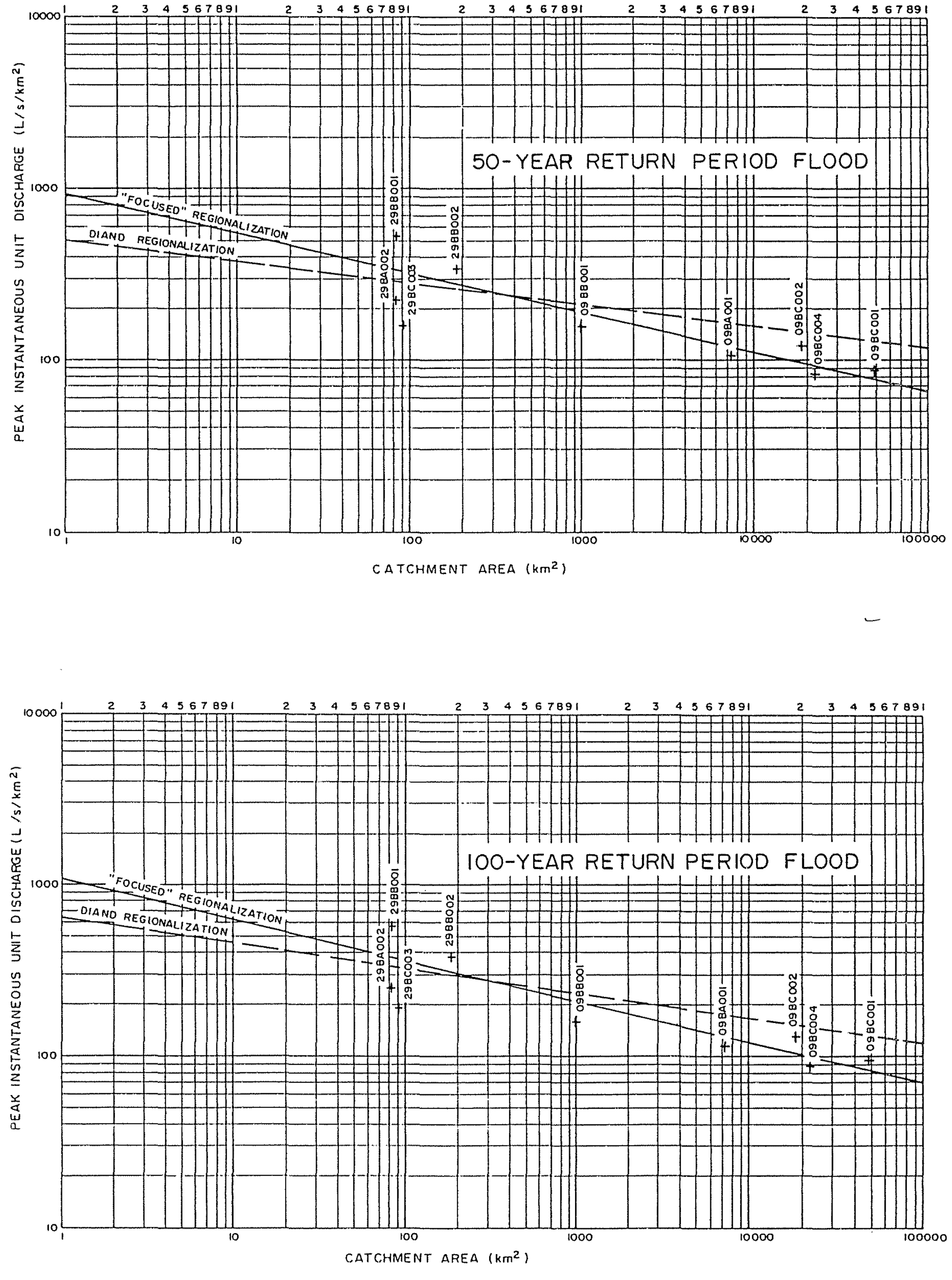


FIG. 3.2 REGIONAL RELATIONSHIPS BETWEEN FLOOD HYDROLOGY AND CATCHMENT AREA

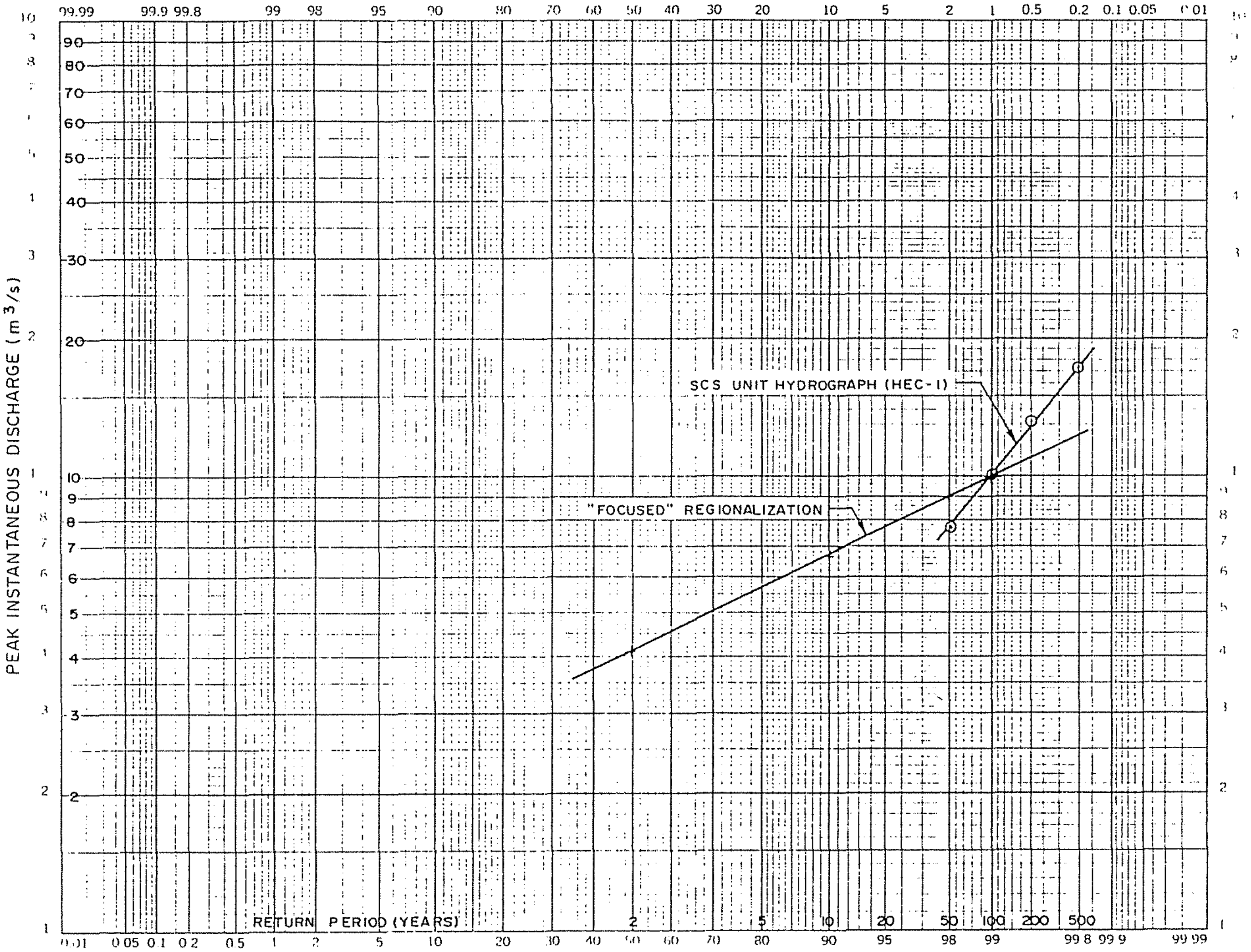


FIG. 3.3 ESTIMATED PEAK INSTANTANEOUS FLOOD DISCHARGES FOR VANGORDA CREEK AT DIVERSION SITE

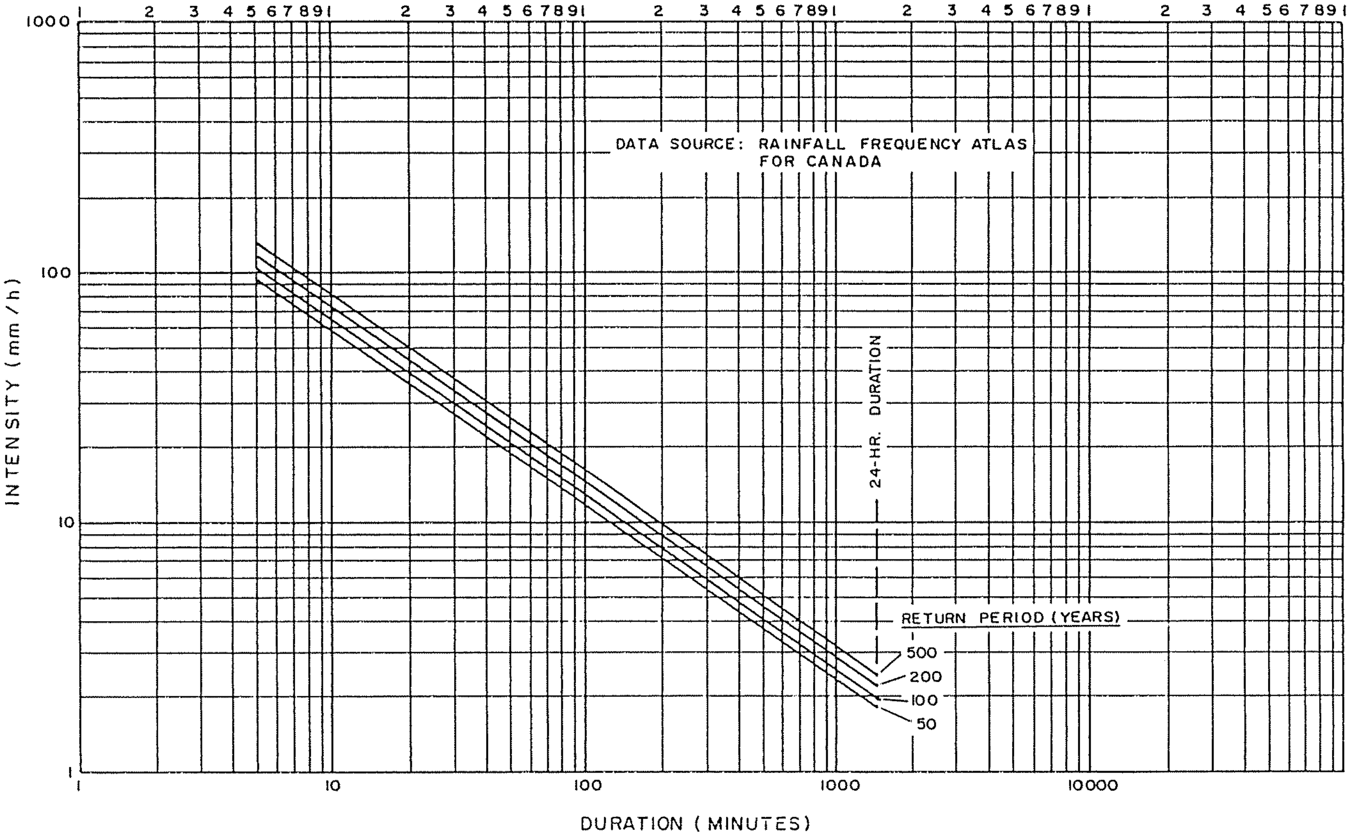


FIG. 3.4 ESTIMATED RAINFALL INTENSITY-DURATION-FREQUENCY CURVES FOR PROJECT AREA