Appendix M

Memo (4 December 2001) – RTP Dam Design, Seepage Estimates & Precedents

Memo (17 December 2001) – Review Comments on RTP Dam Seepage Analysis

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Pogo Project Memorandum

To	**Rick Zimmer** **Karl Hanneman**	File No.	**VM00172 V-2**
From	**Mike Davies**	cc	
Date	**4 December 2001**		
Subject	**RTP Dam** **Design Elements, Seepage Estimates and Membrane-Lined Rockfill Dam Precedents**		

1.0 INTRODUCTION

The purpose of this memorandum is to summarize the design elements for the Pogo RTP Dam, summarize seepage estimates completed for the facility and to comment on the precedence for membrane-lined rockfill dams. Being a summary memorandum, more detailed information is available as required on the topics presented.

The proposed dam has been designed as a rockfill structure, incorporating a number of seepage control works, such as:

- Upstream liner system composed of a reinforced geocomposite clay liner underlying a 60 mil textured HDPE liner;
- Positive cutoff at the upstream toe of the dam. The positive cutoff works include the excavation and removal of the coarse granular overburden soils down to the level of the existing weathered bedrock. The excavated overburden is to be replaced with a compacted rockfill zone with an upstream impermeable liner system equivalent to that proposed for the dam face; and
- Grouted Cutoff Wall at the upstream toe of the positive cutoff. The grout is to be injected through the weathered bedrock zone to the maximum practical depth.
- Grouted zones of bedrock fracturing – any specific zones noted in evaluative drilling or during dam construction will be specifically grouted off.
- A seepage collection system down-gradient of the dam that is comprised of several wells that will penetrate the weathered bedrock and be placed to intercept specific conveyance features, if there are any indicated.

These works are summarized in this memorandum.

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2.0 DESIGN OVERVIEW

The proposed RTP will be formed by a combination of an excavation and development of a dam contiguous to the excavation. The dam will be developed using rockfill and, with the basin excavation, has been sized for feasibility purposes to have a 40 million gallon (Mgal) capacity although the design is flexible for both larger and smaller RTP capacities. The RTP will have several functions including:

- Provision of water to the processing facilities
- To provide collection, storage, and treatment of surface run-off and near surface groundwater that may come into contact with mining operations, particularly with the tailings stack

The geographic setting at the Pogo site, consisting of steep narrow V shaped valleys, does not provide storage to impoundment structure volume ratios typically available in broader valleys with lesser valley gradients. A relatively large amount of dam material is therefore required for a relatively small reservoir capacity. Assessments of earth, concrete and rockfill dams for use at Pogo were made based on safety, cost and construction issues.

For the Pogo site, earth and concrete dams as well as an earth core rockfill dam would have significant logistical problems. All would constrain construction to the summer months and would require construction materials not locally available. No fine-grained materials have been found near the Pogo site suitable for an impervious earth core. This is not surprising given the lack of glacial activity in the project site area. In the case of a potential concrete dam, significant quantities of cement would have to be transported by truck 130 miles from Fairbanks.

By comparison, rockfill can be placed year round under typical conditions encountered at the Pogo site and there are significant sources of competent rockfill available, both non-mineralized development rock or locally quarried rock. Furthermore, rockfill dams with an upstream impervious lining (such as concrete or geosynthetics) have historically proven to be stable, to incur little deformation in earthquakes, to withstand high seepage flows without suffering internal erosion should an insufficient liner be installed and have been overtopped without catastrophic failure of the dam (e.g. Bureau of Reclamation (1987), Sherard and Cooke (1987), ICOLD (1993), Cooke (1993)). Further examples are provided later in this memorandum that demonstrate that seepage concerns can be addressed by a well-designed and well-constructed rockfill lined facility.

Based on the factors presented above, a rockfill structure was recommended for the Pogo RTP dam. Complete with seepage control, collection and monitoring systems, this type of dam would provide a cost effective, technically sound and environmentally compliant option for the impoundment of RTP water.

3.0 RTP DAM – SUMMARY OF SELECTED DETAILS

3.1 Foundation Conditions

The proposed RTP dam would be located at approximately ¾ miles downstream of the headwaters of Liese Creek. The valley walls at that location are steep, particularly on the south side, varying from 1.5H:1V to 3H:1V. Site investigations have been completed over the past four years to characterize the geotechnical conditions. The vegetation on the slopes was mainly moss and small black spruce alder or birch trees. Portions of the south slopes were bare talus slopes overlain by a thin layer of organics. The overburden, consisting mainly of sand and gravel colluvium, and depth of bedrock weathering tended to thicken in the creek bottom and on the lower portions of the south-facing slopes.

Twenty-three boreholes, four seismic refraction lines and a number of shallow test pits have been completed in the vicinity of the RTP dam over the course of the four site investigations and have been used to develop the geologic model near the RTP dam. The locations of the boreholes and survey lines are shown on Figure B00172-03-001. Seven of the boreholes and a seismic refraction survey line 960 ft in length were located near the proposed centerline of the RTP dam. Two of these boreholes were inclined holes that intersected beneath Liese Creek. A seismic refraction survey 1595 feet long was carried out near the proposed alignment of the upstream toe of the dam with six boreholes also being placed along this alignment. Two of the boreholes were drilled to investigate conditions in the proposed RTP pond and one was drilled in the center of a potential diorite quarry. The area downstream of the proposed seepage collection and return system of wells was also investigated through one borehole and a seismic refraction line 420 feet in length. The boreholes within the RTP and RTP dam footprints carried out in the 1998, 1999, 2000 and 2001 programs were advanced between a minimum of 20 feet and a maximum of 290 feet into bedrock. In 1998 (EBA, 1998) and 1999, the drilling was carried out using a mud rotary drilling method whereas in 2000 and 2001 the drilling was carried out using an Odex system, which utilizes an air driven percussion hammer. The mud rotary drilling returned a continuous core sample once in bedrock but only standard penetration samples were obtained during the overburden drilling. The Odex system, returns a continuous sample although it is highly altered from the drilling process as the drill cutting are forced up inside the casing and out a hose. The mud rotary drilling focused largely on characterizing the bedrock while the Odex drilling focused more on characterizing the overburden materials and confirming overburden thickness at the locations investigated.

In-situ testing of the overburden in the vicinity of the proposed RTP site consisted of 26 standard penetration tests (SPT) of the granular material. For the bedrock, measurements of structural discontinuities and rock quality designation (RQD), and 15 borehole packer tests were completed. The SPT's carried out during the 1998 and 1999 mud rotary drilling were carried out under standard testing conditions and the recorded SPT values should be appropriate for applying to standard correlations. The SPT's carried out using the Odex system was carried out in more disturbed soil conditions. Air was introduced to the soil formation from the percussion hammer during drilling and the pounding of the hammer would also alter the soil conditions. The SPTs were generally carried out under an appropriate head of water and field measurements generally did not indicate heaving of the soils, but due to the drill set up, the water could not be introduced prior to lifting the hammer resulting in an initial upward flow gradient and therefore a change in stress conditions at the bottom of the borehole. In spite of the differences in conditions when carrying out the SPTs in 1998, 1999 and 2000, the SPT values recorded during the three site investigations correlate relatively well. The SPT values were used in standard correlations between blowcounts and soil properties. The SPT blowcounts indicated generally dense conditions except in the material immediately near the surface (less than 10 foot depth). As noted below, carrying out SPT sampling was difficult due to the very coarse nature of the materials encountered.

Laboratory testing was carried out in conjunction with the first three site investigations. The laboratory testing on samples procured near the proposed RTP consisted of 13 gradation tests and 7 Atterberg Limits carried out on selected SPT samples. The tested material ranged from silty sands to sandy gravels although a significant portion of the overburden material was too coarse to be collected in an SPT sampler, particularly at depths greater than 15 to 20 feet. None of the soils tested exhibited any signs of plasticity. Complete details of the laboratory results can be found in AGRA (2000) and AMEC (2001).

3.2 Dam Geometry

The proposed RTP rockfill dam would have a downstream slope of 2H:1V and an upstream slope of 3H:1V based on stability, constructability and post-construction deformation considerations. This basic geometry would be constant for any size of facility. A rockfill key will be constructed at the upstream toe of the dam to reduce the influence of potential deformations on the liner system. The exact size of this key will be determined upon excavation and proof rolling with the current design being a robust (conservative) section. Geosynthetics and a grout curtain will form the upstream impervious membrane. To obtain the required 40Mgal capacity and freeboard, the face of the RTP dam will be lined to an average crest elevation of 2090 feet. For dam crest width, a minimum 20-foot running surface would be required for maintenance vehicle access. Lock blocks will be used for vehicle safety berms. As a result, the upstream side of the RTP dam crest would have an elevation 3.5 feet higher than the stated average crest elevation because of the cover layers overlying the impervious liner, the lock blocks and the riprap placed against the lock blocks. Allowing for the width of the outer cover and riprap layers and for lock blocks on both side of the dam crest would an overall crest width of 35 feet would be required. The general configuration of the Option 1 RTP dam is shown on Figures B00172-03-002 and B00172-03-003. Figure B00172-03-004 presents a summary of the rockfill placement locations whereas Figure B00172-03-005 presents dam crest, filter and liner details. More discussion on the filter and liner aspects of the dam is presented later in the memorandum.

3.3 Dam Stability

Rockfill dams are inherently stable entities when designed and constructed to acceptable practice guidelines. The proposed RTP dam has been conceptualized and designed with full attention to applicable guidelines. Furthermore, AMEC Earth & Environmental Limited is very familiar with earth and rockfill design and construction practices and that experience was also utilized in the design process.

Both upstream and downstream stability (static, seismic and sudden drawdown) stability evaluations were completed. Deformation analyses were also carried out with these analyses focused on finding potential areas of strain concentration that may be a concern to liner performance. Industry standard analytical tools were used with hand checks of all calculations. Figure B00172-03-006 presents a summary of some of the stability evaluation work completed.

3.4 Seepage Control

3.4.1 General

Control measures for seepage through both the rockfill dam itself and the foundation would be required. The control for the rockfill is obvious whereas the foundation controls are not as obvious. However, a conservative approach to the foundation conditions has been taken as described in this section. Upstream seepage control is preferred over an impervious central earth core option for limiting seepage through the dam at Pogo. In addition to the simpler construction noted previously, an upstream seepage barrier is preferred over a central barrier for the following reasons:

- An upstream barrier allows the majority of the rockfill to remain unsaturated resulting in a more stable dam when compared to a dam of similar geometry with a central barrier
- An upstream membrane is more readily available for inspection and repair if ever required
- An upstream membrane can be constructed after completion of the rockfill section resulting in fewer settlement problems
- An upstream barrier would facilitate raising the dam if required
- An upstream barrier is far simpler to remove/modify for mine closure purposes

The recommended upstream impervious liner for the proposed rockfill dam at Pogo is a composite system of geosynthetics consisting of a textured high-density polyethylene liner (HDPE) and a geosynthetic clay liner (GCL) adhered to a thin HDPE membrane. The geocomposite system (dual liner) was selected based on the following advantages:

- lower hydraulic conductivity than concrete
- two layers of protection against seepage
- not affected by freeze thaw cycles
- allow construction to be accelerated and simplified
- GCLs tend to be somewhat self healing due to their swelling properties
- Excellent case history precedence for single layered HDPE systems for rockfill dams so the proposed dual liner system provides a very secure system

Seepage from the RTP through the foundation will be naturally limited by the nature of the weathered and fresh bedrock. In addition, any seepage potential will be controlled through the construction of an upstream seepage cut-off. The cutoff would involve the construction of a grout curtain at the upstream toe of the rockfill dam. A permanent excavation through the overburden would extend the upstream face of the RTP dam to the weathered bedrock surface which can be inspected and, if required, overexcavated to expose suitable material for the base of the face liner system. The geosynthetic liners on the upstream face of the rockfill dam would then be extended over the overburden excavation to the weathered bedrock surface and tied into a grout curtain installed through the weathered bedrock and into competent bedrock. A continuous, relatively impervious membrane would thus be created, effectively containing surface runoff and flow or seepage in both the weathered bedrock and the overburden. The grout curtain would also tend to lower the water table downstream of the cut-off leading to a more stable foundation for the dam though this additional stability is not required for the design. The lower water table would induce an upward gradient in the fresh and weathered bedrock downstream of the cutoff, promoting any limited seepage under or through the cut-off to the collection system downstream of the dam where the seepage could be returned to the RTP.

3.4.2 Dam Cutoff System

The proposed seepage cutoff for the RTP dam would consist of an upstream geosynthetic system that would tie in to a grout curtain constructed at the upstream toe of the dam or at the base of the excavated trench upstream of the dam. There would be an excavated trench in all areas where the toe of the dam is completed in the excavated weathered bedrock. On the left abutment, particularly at higher elevations, there may be limited weathered bedrock and the geosynthetic system may be more appropriately tied into the fresh bedrock without need for a trench. If the rock quality is high, a trench would not be required or even desirable, as the creation of the trench could introduce discontinuities not initially present in the rock. The grout curtain would be place in all rock, regardless of location along the upstream toe.

The proposed grout curtain would consist of boreholes drilled through weathered bedrock and, where necessary, several feet into competent bedrock; the exact spacing and length of boreholes would be determined during construction once the cutoff alignment was fully excavated. The current plan is to utilize a line of primary grouted holes at 20-foot centres with secondary holes between these holes (e.g. 10-foot centres), tertiary and quaternary holes similarly between preceding holes. For project estimating purposes, it is assumed that there would be 100% alignment coverage with the primary holes, 75% secondary, 25% tertiary and 5% quaternary. In addition, an allowance for grouting any specific structures in either the weathered bedrock or, if having reservoir daylighting potential, the fresh bedrock. The grout holes would be inclined to maximize the amount of grout and to intercept the largely sub-vertical structure present in the foundation area. An appropriate grout mixture would be injected into the boreholes under sufficient pressure but not so high as to cause any hydro-fracturing of the weathered bedrock. Packer testing during the grouting program would be carried out to optimize grout mixtures, pressures and to locate any areas requiring more than primary or secondary treatment.

A 60mil textured high-density polyethylene (HDPE) geomembrane, underlain by and in intimate contact with a reinforced geocomposite clay liner (GCL) adhered to a minimum 20mil texture HDPE liner is proposed for the upstream impervious liner system. HDPE was chosen for its superior strength, ability to be textured, its ultraviolet resistance and its demonstrated longevity in extreme conditions (climatic and chemical) when compared to other synthetic liner materials. The HDPE liner should be installed to manufacturers specification (e.g. no horizontal seams, minimum overlap of 6 inches, standard quality control testing) so that typical membrane properties are applicable. Similarly, the GCL seams should not be horizontal, should be overlapped a minimum of 6 inches and should be covered with extra bentonite to create a low permeability seam. Bedding and cover layers will be required to protect the materials and to ensure proper installation. A bedding layer should be placed on the upstream face of the RTP dam, consisting of the finest material available on site in order to provide a smooth installation surface for the liner system. The specification for this bedding layer is 3 mm minus material, compacted to a thickness of one foot perpendicular to the dam face (horizontal layer thickness nearly 3 feet). A riprap layer should be placed on the upstream face of the RTP dam to proved protection against wind and ice action. A cover layer would be required between the riprap and the liner system to protect the liner against damage from the riprap. Figure B00172-03-005 illustrates the proposed liner system including the material specifications.

The geosynthetic liner system would be terminated with an tie-in system that is connected to a grout trench. This seepage control system will effectively contain flow and seepage in both the weathered bedrock and the overburden. It will also tend to lower the water table downstream of the cut-off leading to a more stable foundation. The lower water table will induce an upward gradient in the fresh and weathered bedrock promoting any seepage under or through the cut-off towards the collection system downstream of the dam where it can be returned to the RTP. The proposed collection system is summarized later in the memorandum.

The toe area of the liner can expect the highest potential gradients. As such, the tie-in system was provided with the redundancy of the grout trench. Moreover, the threaded bolts would have two neoprene gaskets for added seepage security. The secondary GCL will also be extended to the grout trench to provide seepage security at the tie-in. The estimated potential flow gradient across the liner under a full pool reservoir was not of pressure concern to either geosynthetic product, the HDPE liner or the GCL with membrane backing, based upon typical manufacturer information. The maximum head differential on any portion of the liner and tie-in system will be equal to the maximum depth of pool in the RPT. This maximum value is approximately 100 feet or just under 45 psi. The proposed liner and tie-in system would be capable of withstanding pressures at least five times that high. For example, as shown in Section 4, many membrane systems have been installed with far greater reservoir heads, and hence toe gradients, with no adverse effects on the liner system.

3.4.3 Filter Zones

Although the dam has been provided with a dual liner system, internal erosion concerns were still addressed for design completeness. To mitigate against piping or migration of processed bedding fines into the rockfill dam, filter compatibility should exist between adjacent soil and rock materials. There are three distinct zones that would require filter compatibility:

- Between the liner bedding layer and the rockfill
- Between the rockfill and the overburden foundation
- Between the cover material and the riprap

The filter beneath the RTP dam should be 3 feet thick and the filters on the face of the RTP dam should be 3 feet wide to enable proper placement and compaction. The filters would be end dumped in lifts of not more than 24 inches loose measure and compacted to a minimum of 95% of the material's standard Proctor density in accordance with ASTM D-698. The placement would take place either as the rockfill dam is constructed or following the complete rockfill placement. Specified gradations of the filters, riprap and assumed rockfill are provided on Figure B00172-03-007.

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3.5 Seepage Estimates

3.5.1 Background

Seepage estimates for the RTP pond have been developed as part of the feasibility design of the project water management facilities. The seepage estimates were carried out using the SEEP/W modeling program and checked with manual calculations. The details of the algorithms used by SEEP/W are presented in GEO-SLOPE International Inc. (1998).

In addition to base conditions, seepage estimates have also been carried out assuming the presence of a permeable fault along the alignment of Liese Creek that is in excess of that currently envisioned by the subsurface investigations. It has been noted that the existence or non-existence of this fault has not yet been conclusively determined although the site investigations to date indicate if such a feature(s) exists, it has relatively limited cross-valley extent and would therefore be limited in size.

Likewise, the evaluation assessed the effectiveness of the grouted cutoff wall in the weathered bedrock. It is possibly argued that potential construction, climate and ground condition variation issues may make it difficult to effectively grout off the weathered bedrock zone. Seepage estimates considered the possibility that it may only be possible to construct the grout cutoff to a maximum depth of 20 ft. Thus leaving a zone of ungrouted weathered bedrock below above the intact bedrock zone. AMEC Earth & Environmental have been involved in a number of similar projects in similar climates and not had this lack of success for creation of an effective grout curtain. The conditions at Pogo are favourable for an effective grout curtain.

This section of the memorandum summarizes the seepage estimates using the subsurface information available, e.g. the "base case" and for the "what if" scenarios of a large, pervious fault and ineffectual grouting. For all seepage estimates presented, it is assumed that a full pond for the 40 Mgal dam will be present. This is a highly conservative condition as project water balance modeling for the RTP has indicated that this condition would be present for no more than a few weeks for any given year. Consequently, average seepage rates (from normal operating pond levels) would be estimated as being considerably lower than the values in the following sections.

3.5.2 Seepage Estimates Based on Existing Information

Figure 3.1 presents the cross section used to model the seepage regime of the RTP demonstrating the dam geometry and materials as well as the original ground stratigraphy with the overburden, weathered bedrock and intact bedrock layers. Table 3.1 outlines the material properties used in the seepage model. Table 3.2 presents a summary of the seepage losses estimated by the model.

FIGURE 3.1

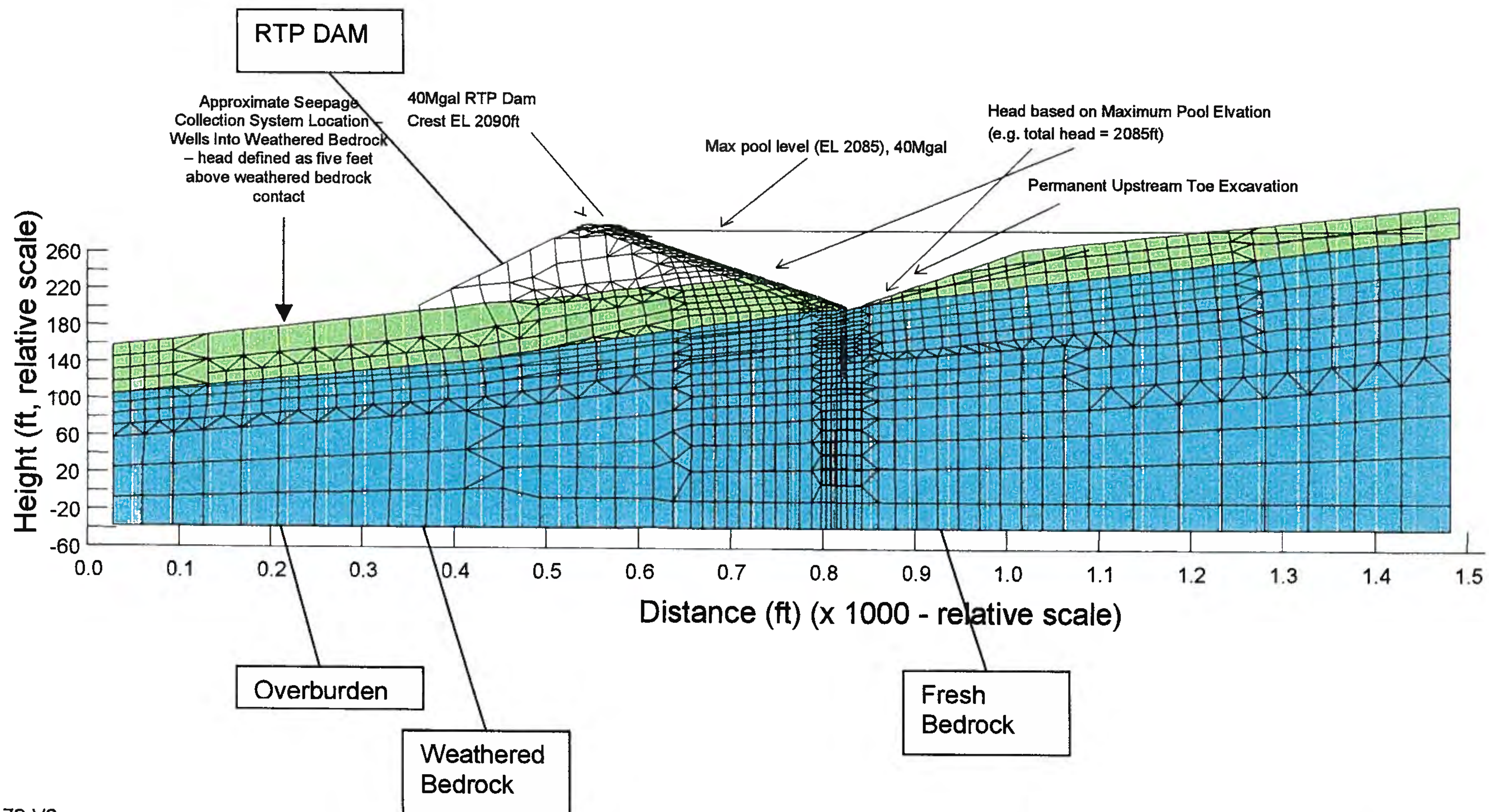
RTP DAM
Approximate Seepage Collection System Location – Wells Into Weathered Bedrock – head defined as five feet above weathered bedrock contact
40Mgal RTP Dam Crest EL 2090ft
Max pool level (EL 2085), 40Mgal
Head based on Maximum Pool Elvation (e.g. total head = 2085ft)
Permanent Upstream Toe Excavation
Height (ft, relative scale)
260
220
180
140
100
60
20
-20
-60
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
1.1
1.2
1.3
1.4
1.5
Distance (ft) (x 1000 - relative scale)
Overburden
Weathered Bedrock
Fresh Bedrock

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Table 3.1 Seepage Model Material Properties

Material	K_s (m/s)
Overburden[1]	10^{-4}
Finest upstream filter material[1]	10^{-7}
Rockfill[2]	10^{-1}
Weathered Bedrock[3]	10^{-6}
Intact Bedrock[3]	10^{-9}
Cutoff Wall[4]	10^{-7} - 10^{-8}
Geomembrane[5]	10^{-14}

Notes:
1. Based on laboratory gradation results
2. Values based on engineering judgment.
3. Conductivity values based on packer test data gathered during the 1999 site investigation.
4. Cutoff wall conductivity based on engineering judgment.
5. Saturated hydraulic conductivity based on published values.

The estimates in Table 3.2 include an assumed effective grout curtain (cutoff wall), liner performance per typical installations and no specific structural defect in the bedrock daylighting into the reservoir. "What-if" seepage analyses are summarized in the next section of the memorandum.

Table 3.2 Estimated Seepage – Base Case Parameters and As-Investigated Foundation Conditions

Flow Component	Estimated Flow from RTP (gpm)
From Reservoir	3.2
Through Cut-off Wall	4.5
Into Seepage Collection System	5.2

The results in Table 3.2 considered that a grouted cutoff wall, at the upstream toe of the dam, would serve to "homogenize" the weathered bedrock zone and effectively reduce potential seepage through any preferential flowpaths. As such, the majority of seepage from the reservoir, along with natural groundwater inflows, is forced through the cutoff wall. In the analyses, the flow through the cut-off well and into the collection system are higher than the "from reservoir" value as there is introduction of natural groundwater flow to these components.

A sensitivity analyses was carried out assuming that the fresh bedrock hydraulic conductivity was lesser than the base case value. Table 3.3 presents the results of this sensitivity analyses.

Table 3.3 Estimated Seepage – Sensitivity Analyses for Fresh Bedrock Hydraulic Conductivity

Bedrock Hydraulic Conductivity	Estimated Flow from RTP Reservoir (gpm)
Base Case – $K = 10^{-9}$ m/s	3.2
Case Two – $K = 10^{-8}$ m/s	4.0
Case Three – $K = 10^{-7}$ m/s	4.8

Raising the hydraulic conductivity assumed for the fresh bedrock by two orders of magnitude increased the estimated leakage from the RTP reservoir by roughly 50%. However, without an assumed pervious structure, the absolute change in estimated seepage is only a few gallons per minute.

3.5.3 "What if" Seepage Estimates

3.5.3.1 General

Table 3.4 presents a summary of the material hydraulic conductivities used in a revised seepage model to evaluate the presence of a pervious structure(s) that has yet to be discovered by the site investigation work. Several "what if" scenarios are evaluated as described below.

Table 3.4 Material Hydraulic Conductivities for Revised "What-if" Seepage Estimates

Material	Hydraulic Conductivity K_s (m/s)
Overburden[1]	10^{-4}
Finest upstream filter material[1]	10^{-7}
Rockfill[2]	10^{-1}
Weathered Bedrock[3]	10^{-5}
Grouted Weathered Bedrock	10^{-7} to 10^{-8}
Intact Bedrock[3]	10^{-9}
Cutoff Wall[4]	10^{-8}
Geomembrane[5]	10^{-14}

Notes:
1. Based on laboratory gradation result
2. Values based on engineering judgment.
3. Conductivity values reduced to model potential fault.
4. Cutoff wall conductivity based on engineering judgment.
5. Saturated hydraulic conductivity based on published values.
6. Hydraulic conductivity consistent with work by Adrian Brown (170 ft/year).

Note that all material hydraulic conductivities have remained unchanged except for the weathered bedrock zone. The hydraulic conductivity for the weathered bedrock zone, in a 100 ft wide section, has been increased from 10^{-6} m/s to 10^{-5} m/s; e.g. values consistent with "dirty" (silty) sand. This increase is intended to model the potential existence of the geologic fault outlined previously. As mentioned previously the cutoff wall depth was also reduced to a maximum of 20 ft.

3.5.3.2 Ungrouted Fault Condition

Table 3.5 presents a summary of the estimated seepage assuming the "what if" hydraulic conductivity conditions of Table 3.4 and assuming that the faulted bedrock is left ungrouted.

Table 3.5 Estimated Seepage for "What if" Condition with Ungrouted Fault

Flow Component	Total Foundation Flow from RTP (gpm)
From Reservoir	30
Through Cutoff Wall	0
Under Cutoff Wall	32
Into Weathered Bedrock	5
Into Collection System	27

Note that the predicted seepage from the reservoir now passes underneath the 20-foot deep cutoff wall instead of through it. The total estimated downstream flow reporting to the collection system has increased from about 5 gpm to nearly 30 gpm.

A further "what-if" evaluation was carried out assuming the fault condition described above and assuming no grout-curtain cutoff wall. The seepage from the RTP reservoir in that extreme case was estimated at approximately 65 gpm.

3.5.3.3 Grouted Fault Condition

If the fault above were grouted, the seepage estimate would trend back to the base case scenario depending upon the chosen hydraulic conductivity for the grouted mass.

3.5.4 Seepage Estimate Summary

Table 3.6 presents a summary of the groundwater flows estimated to report to the collection system downstream of the RTP. Also included is the assessment of a 100 ft wide fault with no grouting whatsoever. This latter scenario would provide an estimated 65 gpm. The flows are presented for each of the seepage conditions outlined in the preceding sections.

The presence of a conceptual large, and permeable, fault that daylights in the reservoir leads to an increase in the predicted groundwater flows reporting to the collection system, from 5 gpm to nearly 30 gpm. The 20 ft depth limitation of the cutoff wall results in most of the predicted seepage shortcutting the cutoff wall by flowing through the higher conductivity weathered bedrock zone. The effective grouting off of the faulted material leads to reductions in seepage that would be consistent with whatever the assumed effectiveness would be. With an assumption that the effectiveness would be to bring the fault to at least (as impervious as) the low end of the hydraulic conductivity of the grouted weathered bedrock (10^{-7} m/s), then the seepage rate would trend towards the 5 gpm reporting to the seepage collection well system.

Table 3.6 Summary of Seepage Estimates

Condition	Geologic Fault Hydraulic Conductivity (m/s)	Estimated Peak Flow to Collection System (gpm)
Base Case Model	N/A	5
Ungrouted Fault 20 ft Deep Grout Curtain Cutoff Wall	10^{-5}	27
Ungrouted Fault No Grout Curtain Cutoff	10^{-5}	65

Past and the current site investigation programs have not been able to provide evidence of the day-lighting fault as modeled. However the analyses presented provide a summary of the "what if" scenarios.

3.6 Seepage Collection System

The proposed seepage collection system consists of not fewer than four bedrock penetrating pumping wells. These wells, inclined as approximate to intersect any conveyance structures, would be concentrated in the valley bottom within approximately 150 feet of the downstream toe of the dam. The wells would nominally be 12 inches in overall diameter with 8-inch screen diameters. Granular material will be placed between the screens and the outer slotted casing to have an effective conveyance layer.

The effectiveness of the wells when modeled is typically 100% as a head is simply provided to the node in the finite element mesh to simulate pumping drawdown. In reality, recovery efficiency is a function of well size, completion details and geological conditions. To assist with modeling reality, a five-foot head of well inefficiency was assumed. With this assumed inefficiency, more than 90% capture efficiency was indicated. Based upon the modeling, it is estimated that any seepage bypassing the cut-off grout curtain should be largely collected by the well system and, based upon engineering judgment, at least 80% capture efficiency should be expected for a well-designed and installed well system.

Figure B00172-03-008 presents a schematic representation of how the RTP seepage collection system would work under mine operating conditions. The collection wells shown in two-dimensions would draw flow laterally between adjacent wells as well as any flows coming from upgradient areas.

4.0 MEMBRANE LINED ROCKFILL DAMS

4.1 General

The use of a membrane lined rockfill dam for the RTP dam is in keeping with a growing worldwide trend. A few of these installations have neglected the growing and extensive design guidance available through literature and experience giving these facilities a poor reputation in some jurisdictions. For the design of the Pogo RTP dam, current governance literature and experience on a number of similar projects has been used to provide a sound basis for this project.

As noted in Scuero (1997):

> *"The use of geomembranes as impervious facings on embankment dams dates back to 1959. More than 100 large dams all over the world have since been lined with geomembranes".*

Scuero (1997) further notes:

> *"Installations accomplished so far have proved that this technology, if properly designed and installed, performs well. Installations with exposed membranes, in severe climates such as the Italian Alps at high elevation (over 6600 feet), have successfully performed since the middle of the 1970's. Life expectancy, as it can be extrapolated form existing case histories and from improvements in technologies, exceeds 50 years, with future possibilities of over 100 years".*

As noted earlier in this memorandum, not only would the RTP dam have a geomembrane (HDPE) liner, a GCL composite would also be utilized giving a system redundancy not common in the case records but consistent with a commitment to an effective water retention structure.

4.2 Existing Database

4.2.1 Overview

There are numerous examples of geomembrane applications as waterproof lining on dams. A partial list of case histories is shown in Table 4.1. As to the theory and testing, Koerner (1986, 1990, 1994 and 1998) and Rollin & Rigo (1991) present detailed information.

Table 4.1 Sample Case Histories of Geomembrane Facing on Dam

Dam	Country	Height (feet)	Notes
Bovilla Dam (rockfill)	Albania	300	Completed in 1996, in preference to original CFRD design, using CAPRI system (PVC based geocomposite).
Alzeau Dam (rockfill)	France	131	Commissioned in 2000, 22,000 m^2 of geomembrane used.
Borfloc'h Dam (rockfill)	France	62	Commissioned in 1993. See Figure 4.1.
Ospedale Dam (rockfill)	France	82	Completed in 1978, believed to be the first one, bituminous membrane reinforced with polyester felt used.
Codole Dam (rockfill)	France	92	Completed in 1983, non-reinforced PVC used.
Pactola Dam (earth core rockfill)	USA	220	Used for 4.5 m raising, 1 mm HDPE geomembrane used, connected to the original earth core, completed in 1987.
Golden Giant Mine (rockfill)	Canada	164	The dam has an upstream 80 mil HDPE liner in conjunction with a low permeability till zone (0.5 m thick) (more in Section 5.3)
Dobsina Dam (rockfill)	Czechoslovakia	33	Post construction dam water proofing.
Mission (Terzaghi) Dam (rockfill)	Canada	61	In 1960, K. Terzaghi designed and implemented a PVC liner placed on upstream side of clay core to assist with clay core function.
Alpe Gera (concrete)	Italy	571	More than 120,000 ft^2 PVC geocomposite used in 1993 for seepage repair on dam face
Gold Camp (rockfill)	USA	98	1.5 mm CSPE placed on upstream face to minimize seepage
Crueize Dam (rockfill)	France	98	Unprotected 2 mm PVC geocomposite for seepage barrier on upstream face
Sa' Forada (rockfill)	Italy	101	Original upstream bituminous facing (1960) was upgraded to PVC geocomposite (1992) and seepage essentially eliminated
Muna Reservoir	Saudi Arabia	>80	The whole reservoir lined with 2.5 mm Carbofol geomembrane.

Figure 4.1 shows a typical installation of a geomembrane liner on a rockfill dam during construction.

4.2.2 Example Case Histories of Geomembrane facings

There are a number of case histories of geomembrane applications on upstream faces of dams. For example, Raymond & Giroud (1993) present several cases. Following are some examples to expand the example list in Table 4.1. Metric measures are used if they were cited in the case history.

**Figure 4.1 Borfloc'h Dam (rockfill), France,
Picture above shows Geomembrane Liner under Construction**

4.2.2.1 Ospedale and Codole dams, France

The Ospedale and Codole dams, France, were completed in 1978 and 1983, respectively. The details of the dam and geomembrane are listed in Table 4.2.

The geomembrane used at Ospedale was a bituminous membrane reinforced with polyester felt, while at Codole, non-reinforced PVC was used. The geomembrane system used in each case involved three main components: the support layer, the waterproof lining and the protective layer.

Table 4.2 Details of Ospedale and Codole Dams

Details of dam			Details of geomembrane		
Item	**Ospedale**	**Codole**	**Item**	**Ospedale**	**Codole**
Reservoir level (m)	950	115	Geomembrane	Coletanche	TERSOM
Reservoir capacity (m^3)	3,000,000	6,500,000	Thickness (mm)	4.8	2.0
Dam Height (m)	25	28	Tensile strength (KN/m)	14 - 21	17
Crest length (m)	135	460	Strain (%)	59	170
Base width (m)	90	95	Bursting pressure (kPa)	800	>1200
Upstream slope	1.7H:1V	1.7H:1V			

4.2.2.2 Pactola Dam, South Dakota, USA

The original Pactola Dam was a 67 m earth core rockfill dam with a crest length of 381 m. The dam was raised 4.5 m to safely route the Probable Maximum Flood (PMF). A 1 mm HDPE geomembrane was used for the raising on a geotextile with a mass unit area of 400 g/m^2. The geomembrane was connected to the original earth core. The raising was constructed in 1985 – 1987. This use of a geomembrane as the impervious barrier for a storage reservoir is believed to be the first such application by the US Bureau of Reclamation (Hammer & Lippert, 1993). The use of such a lining for seepage reduction was approved by the Bureau and this dam continues to function extremely well in that regard.

From the experience gained in this project, the following general recommendations were made regarding the use of geomembranes in large dams:

- Geomembrane thickness less than 1.5 mm should be avoided in a rocky environment, even when geotextile cushioning is used.
- The number of seams on steep or curved slopes should be minimized.
- On steep, rough slopes, the geomembrane should be anchored loosely prior to placement of the backfill against the geomembrane.
- To the extent possible, construction activities on slopes above the geomembrane should be kept to a minimum.
- A contractor with experience in construction involving geosynthetic materials should be selected.

4.2.2.3 Mt. Elbert Forebay Reservior, Colorado, USA

In 1980, the Mt. Elbert Forebay Reservoir was lined with 1.15 mm thick reinforced chlorinated polyethylene (CPER) geomembrane. The 117 ha installation was the largest single cell lining installation in the world (Frobel, 1993).

The reservoir serves as the forebay for the Mt. Elbert Pumped Storage Powerplant. The reservoir has a capacity of 14,250,000 m^3. The original reservoir was completed in 1977 by construction of a 27 m high-zoned earth/rockfill dam. Because the original compacted lining failed to prevent leakage from the reservoir, it was decided to install a continuous geomembrane over the entire forebay bottom and side slopes. The piezometers and observation wells installed in the hillside south of the reservoir indicated no appreciable rise in groundwater levels since the relining of the forebay confirming the effective seepage barrier provided by the geomembrane system.

4.2.2.4 Drained Geomembrane System

Scuero & Vaschetti (1998) present a drained geomembrane system developed by CARPI, Italy. The CARPI system is essentially a PVC geomembrane bonded with a layer of geotextile. This system has been used on Alpe Gera Dam (concrete, 174 m, Italy), Bovilla Dam (embankment, 91 m, Albania) and Balambano Dam (RCC, 93 m, Indonesia).

When the CARPI system is attached to the existing concrete surface, the drainage layer relieves potential uplift pressure between the waterproofing liner and the concrete surface. At the same time, the bonded geotextile can provide higher dimensional stability and protect from a puncturing substrate. A geomembrane coupled during manufacturing to a geotextile provides a geocomposite whose performance is superior to the performance that can be obtained if geomembrane and geotextile are kept separate.

Scuero & Vaschetti (1998) conclude that the service performance of these membrane lined facilities has is excellent, with dramatic reduction of seepage in existing dams. They suggest that there will be virtually no leaks in new dams if implemented properly.

4.3 Example AMEC Earth & Environmental Project

In addition to the examples available from literature, AMEC Earth & Environmental Limited has considerable experience with the use of membrane liner systems. This experience includes heap leach installations (with rockfill), dams, reservoirs, canals, landfills etc. A particularly salient project to the proposed Pogo installation is the Battle Mountain Golden Giant mine in Ontario. This area experiences extremely cold and prolonged winters that are equally cold to the Pogo site.

The Main tailings dam, a rockfill structure, is over 100 feet high and has an upstream membrane liner. The total length of dam crest lined in this fashion, between the Main Dam, Dam 43 and Dam 44, is 3,600 feet. Seepage through this liner system, which is anchored in a trench that is, in turn, contiguous with a grout curtain, has been extremely limited over the past ten years of monitoring. The seepage rates are within the range predicted for the Pogo Project (e.g. highest recorded rates are less than 10 gpm). The driving head and dam size (height and width) are both larger than for the proposed RTP dam at Pogo.

5.0 SUMMARY

The proposed RTP rockfill dam for the Pogo Project includes a robust design with sufficient redundancies for usual concerns of stability, seepage and deformation. The dam, which will not be a permanent structure, includes a very stout section with quite flat upstream and downstream slopes for a rockfill dam. The use of flattish slopes is partially related to the use of an upstream membrane liner system to facilitate good installation practices.

The construction of the dam will be straightforward and optimizations are likely during detailed design and construction. However, the basic concept for the dam will be the same and the facility should join the extensive database of well-performing rockfill dams during its intended service life for the Pogo project.

Respectfully submitted,

AMEC Earth & Environmental Limited

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Project Engineer

Michael P. Davies, Ph.D., P.Eng., P.Geo.
Project Manager

Reviewed by:

Peter C. Lighthall, P.Eng.
Vice President, Vancouver

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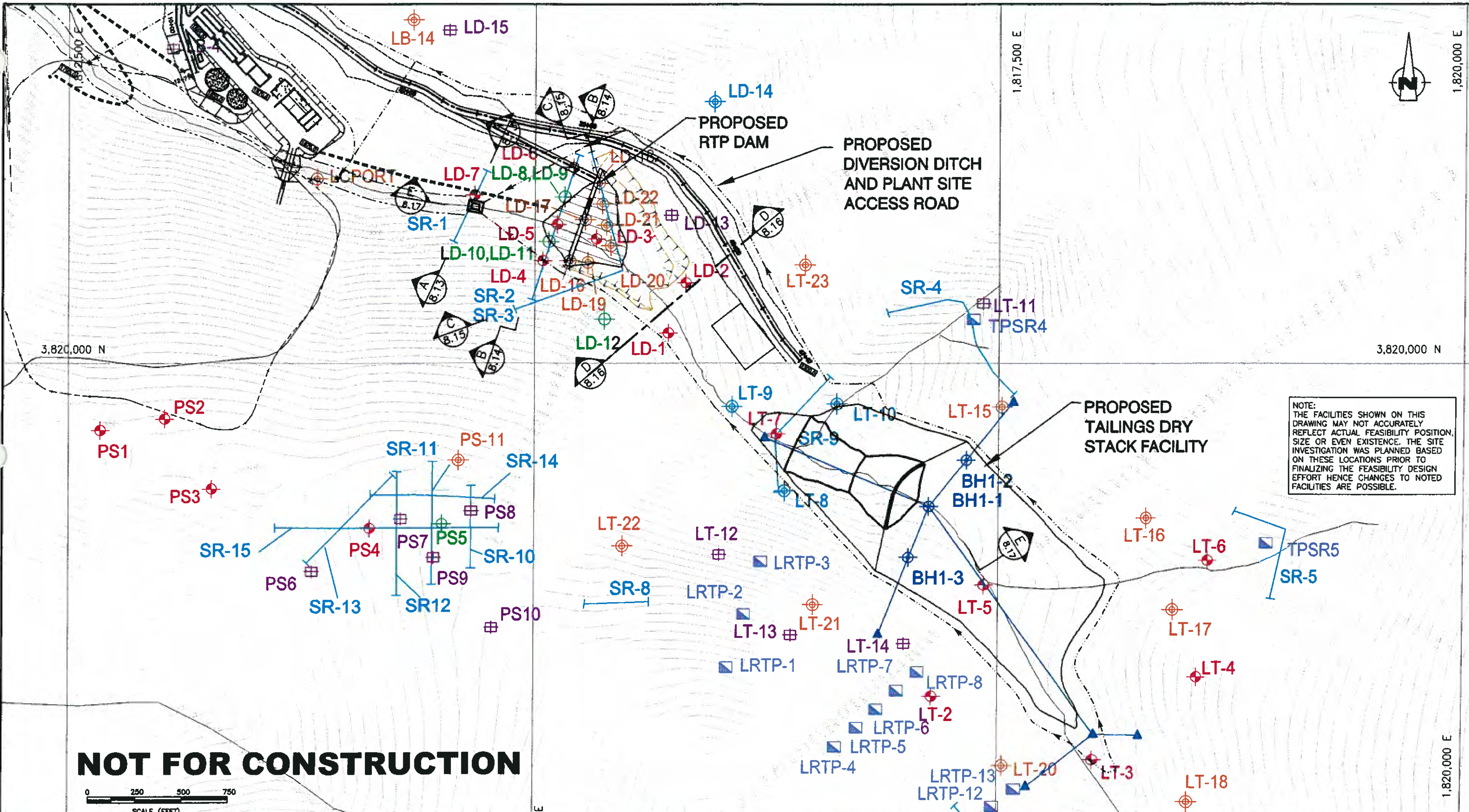

LEGEND

2000 ABANDONED PORTABLE ROTARY RIG BOREHOLE	2000 HAND TEST PITS
1999 EXISTING BOREHOLE (AGRA)	2000 GEOPHYSICS SURVEY LINE
2000 EXPLORATION RIG BOREHOLE	1998 GEOPHYSICS SURVEY LINE (EBA)
2000 PORTABLE ROTARY RIG BOREHOLE	1998 EXISTING BOREHOLE (GOLDER)
2000 PORTABLE ROTARY RIG BOREHOLE, WELL INSTALLED	1998 EXISTING BOREHOLE (EBA)
2001 INVESTIGATION	EXPLORATION BOREHOLE

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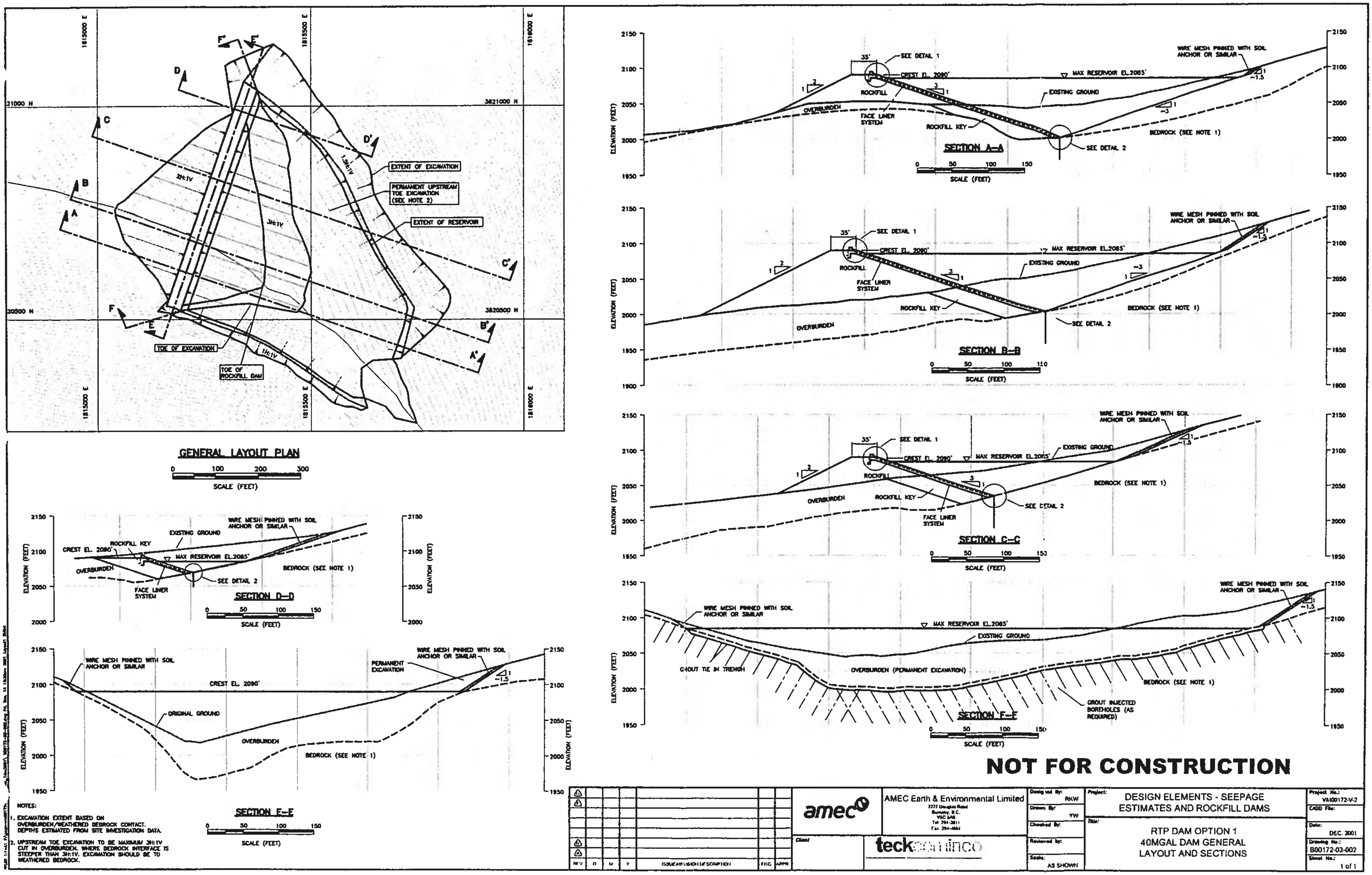
CHK'D BY: LS
APP.
SCALE: AS SHOWN

PROJECT: DESIGN ELEMENTS - SEEPAGE ESTIMATES AND ROCKFILL DAMS

TITLE: PROPOSED RTP EXPLORATION LAYOUT

DATE: DEC. 2001
PROJECT NO: VM00172-V-2
REV. NO.: -
FIGURE No. B00172-03-001


GENERAL LAYOUT PLAN
SCALE (FEET)
EXTENT OF EXCAVATION
PERMANENT UPSTREAM TOE EXCAVATION (SEE NOTE 2)
EXTENT OF RESERVOIR
TOE OF EXCAVATION
TOE OF ROCKFILL DAM
21000 N
3821000 N
20500 N
3820500 N
1815000 E
1815500 E
1816000 E
SECTION A–A
SECTION B–B
SECTION C–C
SECTION D–D
SECTION E–E
SECTION F–F
SEE DETAIL 1
SEE DETAIL 2
CREST EL. 2090'
MAX RESERVOIR EL.2085'
WIRE MESH PINNED WITH SOIL ANCHOR OR SIMILAR
EXISTING GROUND
ROCKFILL
ROCKFILL KEY
FACE LINER SYSTEM
OVERBURDEN
BEDROCK (SEE NOTE 1)
ORIGINAL GROUND
PERMANENT EXCAVATION
OVERBURDEN (PERMANENT EXCAVATION)
GROUT TIE IN TRENCH
GROUT INJECTED BOREHOLES (AS REQUIRED)
ELEVATION (FEET)
SCALE (FEET)
NOT FOR CONSTRUCTION
NOTES:
1. EXCAVATION EXTENT BASED ON OVERBURDEN/WEATHERED BEDROCK CONTACT. DEPTHS ESTIMATED FROM SITE INVESTIGATION DATA.
2. UPSTREAM TOE EXCAVATION TO BE MAXIMUM 3H:1V CUT IN OVERBURDEN. WHERE BEDROCK INTERFACE IS STEEPER THAN 3H:1V, EXCAVATION SHOULD BE TO WEATHERED BEDROCK.
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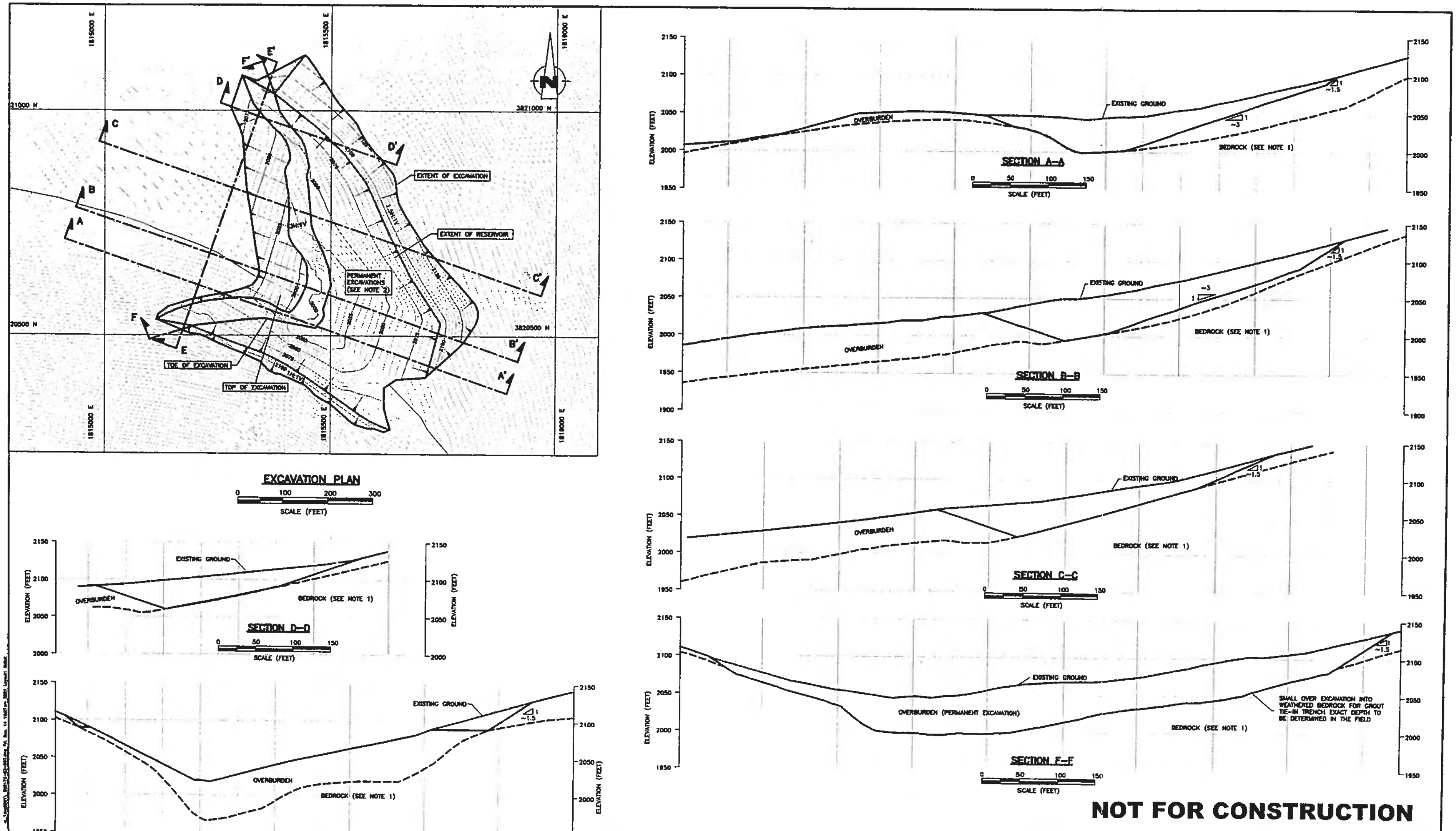
teckcominco
Designed By: RKW
Drawn By: YW
Checked By:
Reviewed by:
Scale: AS SHOWN
Project: DESIGN ELEMENTS - SEEPAGE ESTIMATES AND ROCKFILL DAMS
Title: RTP DAM OPTION 1 40MGAL DAM GENERAL LAYOUT AND SECTIONS
Project No.: VA100172-V-2
Date: DEC. 2001
Drawing No.: B00172-03-002
Sheet No.: 1 of 1



NOTES:

1. EXCAVATION EXTENT BASED ON OVERBURDEN/WEATHERED BEDROCK CONTACT. DEPTHS ESTIMATED FROM SITE INVESTIGATION DATA.
2. UPSTREAM TOE EXCAVATION TO BE MAXIMUM 3H:1V CUT IN OVERBURDEN. WHERE BEDROCK INTERFACE IS STEEPER THAN 3H:1V. EXCAVATION SHOULD BE TO WEATHERED BEDROCK. EXCAVATION CONTOURS SHOWN ARE ESTIMATED BASED ON SITE INVESTIGATION DATA.
3. ESTIMATED EXCAVATION VOLUME 188,300 YD^3

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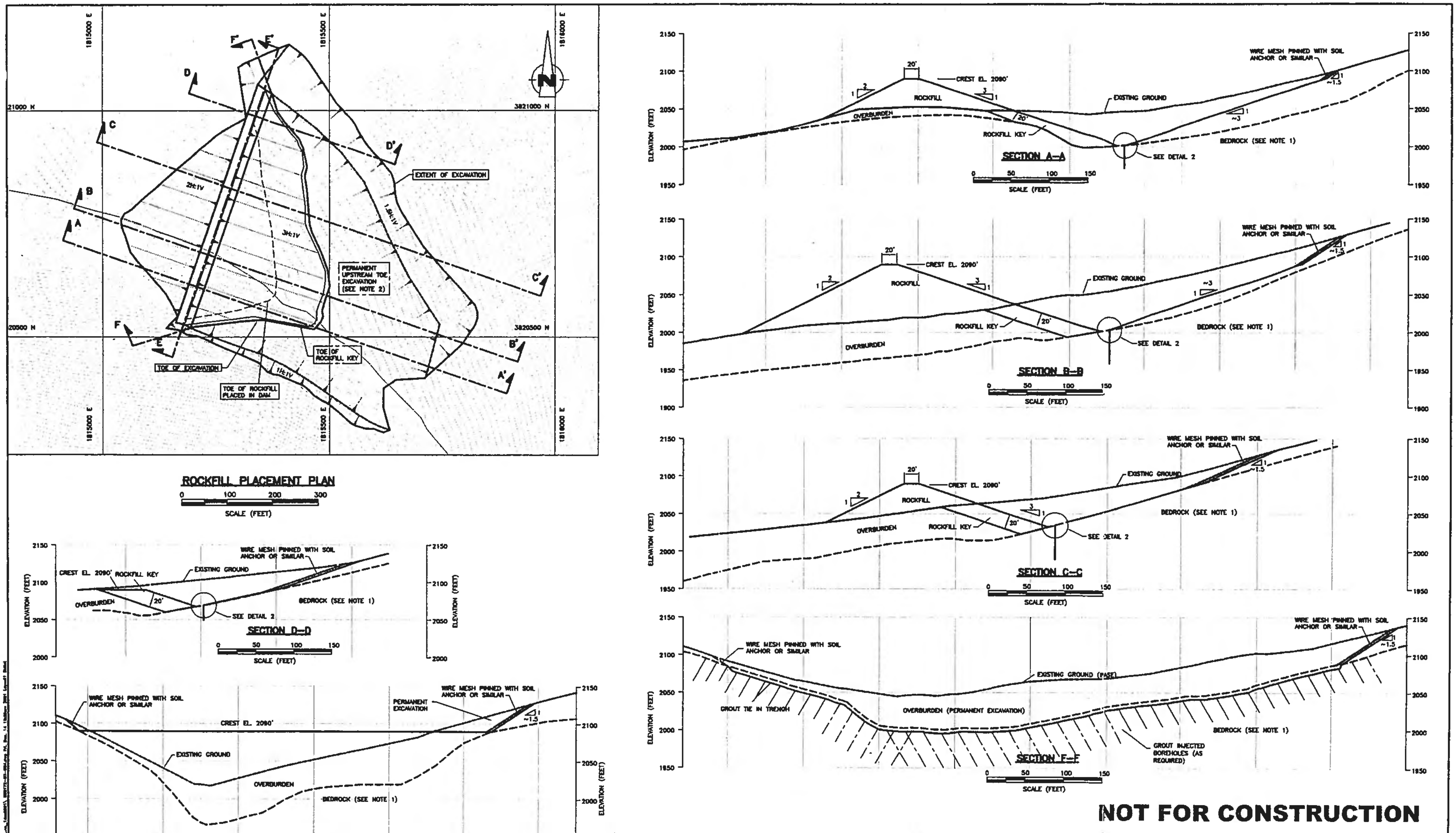
Drawn By:	YW
Checked By:	
Reviewed by:	
Scale:	AS SHOWN
Project:	DESIGN ELEMENTS - SEEPAGE ESTIMATES AND ROCKFILL DAMS
Title:	RTP DAM OPTION 1 40MGAL DAM EXCAVATION LAYOUT
Project No.:	VM00172-V-2
CADD File:	
Date:	AUG. 2001
Figure No.:	B00172-03-003
Sheet No.:	1 of 1



NOTES:

1. EXCAVATION EXTENT BASED ON OVERBURDEN/WEATHERED BEDROCK CONTACT. DEPTHS ESTIMATED FROM SITE INVESTIGATION DATA.
2. UPSTREAM TOE EXCAVATION TO BE MAXIMUM 3H:1V CUT IN OVERBURDEN. WHERE BEDROCK INTERFACE IS STEEPER THAN 3H:1V, EXCAVATION SHOULD BE TO WEATHERED BEDROCK.
3. ESTIMATED ROCKFILL VOLUME 144,400 YD^3

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REV	D	M	Y	ISSUE/REVISION DESCRIPTION	ENG	APPR

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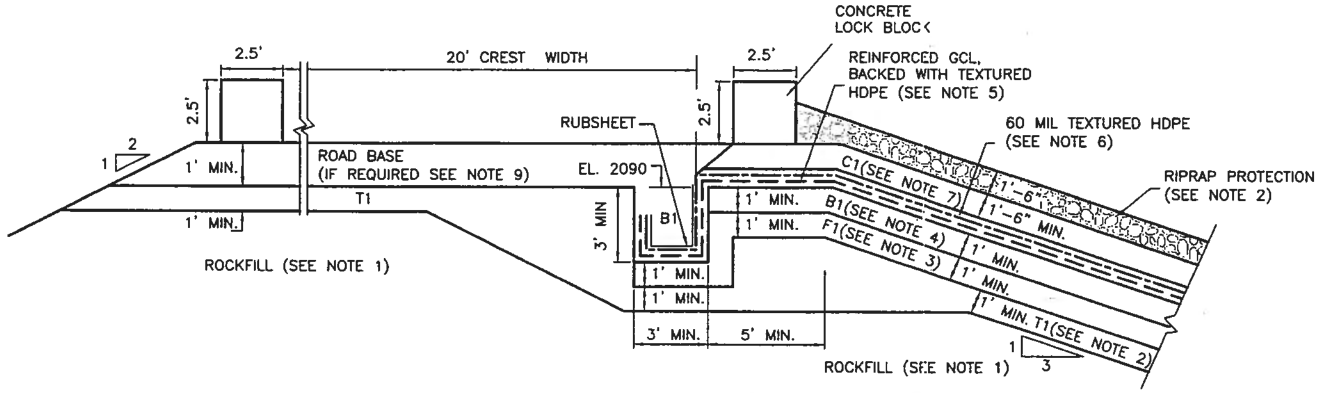
Project: DESIGN ELEMENTS - SEEPAGE ESTIMATES AND ROCKFILL DAMS

Title: RTP DAM OPTION 1 40MGAL DAM ROCKFILL PLACEMENT LOCATION

Project No.:	VM00172-V-2
CADD File:	
Date:	DEC. 2001
Figure No.:	B00172-03-004
Sheet No.:	1 of 1



DETAIL 1 RTP LINER & MATERIALS

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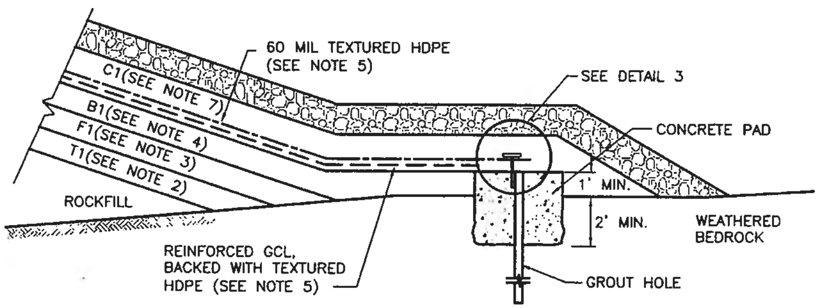
SCALE (FEET)



DETAIL 2 GROUT CURTAIN/HDPE LINER TIE-IN

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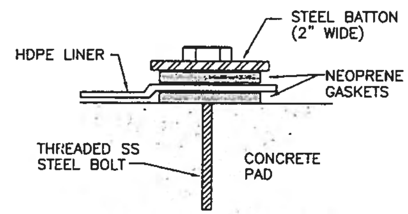
SCALE (FEET)



DETAIL 3 HDPE TIE-IN

SCALE (NTS)

NOTES:

1. ROCKFILL
CLEAN, 2ft MINUS ROCKFILL PLACED IN COMPACTED 3ft LIFTS

2. T1: TRANSITION LAYER
CLEAN 8in MINUS MATERIAL, D_{10} : 3.5 – 10mm
MINIMUM COMPACTED THICKNESS OF 1ft
(EQUIVALENT TO 3 FEET WIDTH HORIZONTALLY)

3. F1: FILTER LAYER
CLEAN (LESS THAN 5% PASING THE #200 SIEVE), 1in MINUS MATERIAL, D_{10} : 0.6 TO 1.5mm
MINIMUM COMPACTED THICKNESS OF 1ft
(EQUIVALENT TO 3 FEET HORIZONTALLY WIDTH)

4. B1: BEDDING LAYER
3mm MINUS MATERIAL
MINIMUM COMPACTED THICKNESS OF 1ft
(EQUIVALENT TO 3 FEET HORIZONTALLY WIDTH)

5. REINFORCED GCL, BACKED WITH A TEXTURED HDPE GEOMEMBRANE
NOMINAL 0.5mm HDPE (MINIMUM) ADHERED TO BACK OF GCL
TYPICAL BENTONITE CONTENT 0.75–1.00lb/ft^2
PLACED WITH HDPE AGAINST BEDDING LAYER
MAXIMUM SATURATED HYDRAULIC CONDUCTIVITY OF $5X10^{-11}$ m/s ($1.6x10^{-11}$fps)
MINIMUM INTERNAL SHEAR STRENGTH OF 500 psf
MINIMUM INTERFACE FRICTIONAL ANGLES OF 26°

6. 60mil TEXTURED HDPE
NOMINAL 1.5mm (0.06in) THICKNESS
MINIMUM INTERFACE FRICTION ANGLES OF 26°

7. C1: COVER MATERIAL
2in MINUS, GOODPASTER ALLUVIUM TYPICAL GRADATION
MINIMUM COMPACTED THICKNESS OF 1'–6"

8. RIP RAP
8in MINUS, D_{50} = 6in
MINIMUM COMPACTED THICKNESS OF 1'–6"

9. ROAD BASE (TO COME FROM AMEC E/C AS REQUIRED)

10. GROUT INJECTED BOREHOLES
BOREHOLES SHOULD BE INJECTED WITH HIGH SLUMP GROUT UNDER HIGH PRESSURE
LENGTH AND SPACING OF BOREHOLES TO BE DETERMINED IN THE FIELD

11. SLUSH GROUT TIE IN TRENCH
SHOULD BE POURED IN TWO BATCHES SO THAT LINER SYSTEM CAN BE SECURED TO THE INITIAL LOWER SEGMENT AND ENCAPSULATED BY THE UPPER SEGMENT.
SHOULD COMPLETELY COVER ALL GROUT CURTAIN BOREHOLES TO ENSURE A CONTINUOUS IMPERVIOUS LINER.

12. DETAILS ARE NOT INTENDED FOR CONSTRUCTION PURPOSES.

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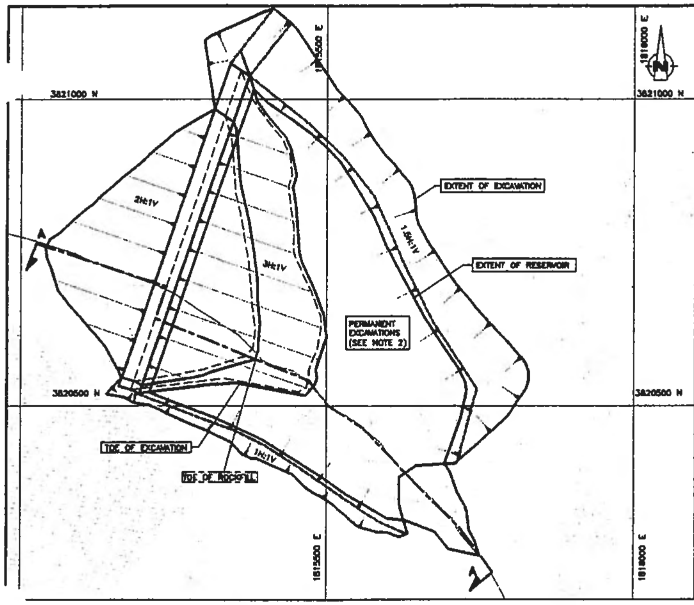
amec	AMEC Earth & Environmental Limited 2227 Douglas Road Burnaby, B.C V5C 5A9 Tel. 294-3811 ~~Fax. 294-4664~~	DWN BY. YW CHK'D BY: PM APP. SCALE AS SHOWN	DESIGN ELEMENTS - SEEPAGE ESTIMATES AND ROCKFILL DAMS RTP DAM CREST, FILTER & LINER DETAILS
Client	teckcominco		DATE: DEC. 2001 PROJECT NO: VM00172-V-2 REV. NO. - FIGURE No. B00172-03-005



GENERAL PLAN

SCALE (FEET) 0 100 200 300

DOWNSTREAM FAILURE SURFACES

STEADY STATE SEEPAGE (STATIC & EARTHQUAKE)
END OF CONSTRUCTION (STATIC & EARTHQUAKE)
SUDDEN DRAWDOWN

1 – MINIMUM FAILURE THROUGH ROCKFILL

2 – MINIMUM FAILURE THROUGH OVERBURDEN

3 – MINIMUM FAILURE THROUGH CREST

4 – MINIMUM FAILURE THROUGH WEATHERED BEDROCK

UPSTREAM FAILURE SURFACES

END OF CONSTRUCTION (STATIC & EARTHQUAKE)
SUDDEN DRAWDOWN

A – MINIMUM FAILURE THROUGH ROCKFILL

B – MINIMUM FAILURE THROUGH OVERBURDEN

C – MINIMUM FAILURE THROUGH CREST

Potential Slip Surface	Maximum Pool Factor of Safety		End of Construction		Sudden Drawdown	
	Static	Pseudo Static (MDE)	Static	Pseudo Static (MDE)	Static	Pseudo Static (MDE)
1	2.3	1.9	n/a	n/a	2.3	1.9
2	2.1	1.7	n/a	n/a	2.1	1.7
3	2.4	1.8	n/a	n/a	2.4	1.8
4	2.7	2.2	n/a	n/a	2.7	2.2
A	n/a	n/a	3.5	2.6	3.5	2.6
B	n/a	n/a	2.8	2.1	2.8	2.1
C	n/a	n/a	3.2	2.3	3.2	2.3

NOTE: FACTORS OF SAFETY ESTIMATED BASED ON LIMIT EQUILIBRIUM THEORY

MATERIAL PROPERTIES:

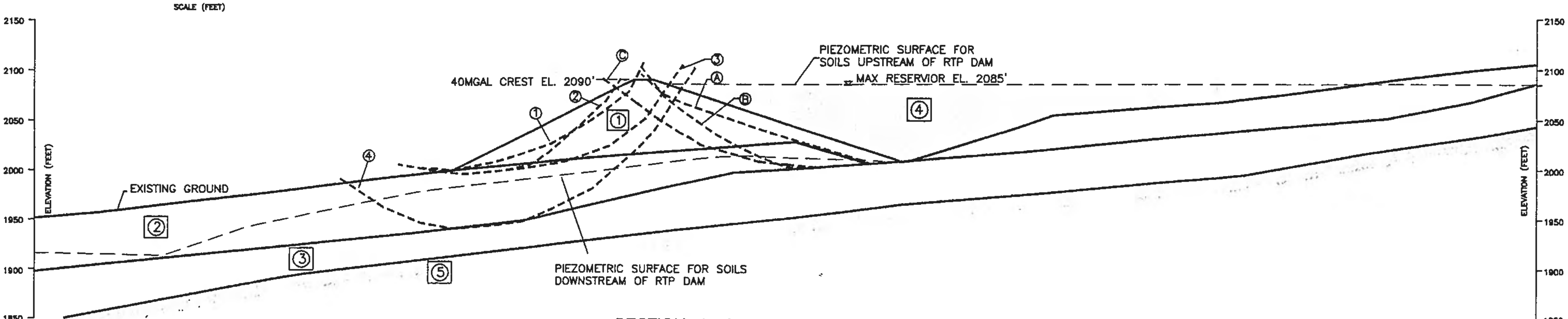
1 ROCKFILL:
SHEAR/NORMAL STRENGTH FUNCTION (LEPS, 1970)
γ = 130pcf

2 OVERBURDEN:
ϕ = 36°
c = 0
γ =127pcf

3 WEATHERED BEDROCK:
ϕ = 50°
c = 0
γ = 140pcf

4 WATER:
NO STRENGTH MATERIAL
γ = 62.4pcf

5 BEDROCK:
NOT CONSIDERED IN ANALYSES



SECTION A–A

SCALE (FEET) 0 50 100 150

NOTES:

1. EXCAVATION EXTENT BASED ON OVERBURDEN/WEATHERED BEDROCK CONTACT. DEPTHS ESTIMATED FROM SITE INVESTIGATION DATA.
2. UPSTREAM TOE EXCAVATION TO BE MAXIMUM 3H:1V CUT IN OVERBURDEN. WHERE BEDROCK INTERFACE IS STEEPER THAN 3H:1V. EXCAVATION SHOULD BE TO WEATHERED BEDROCK.

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REV	D	M	Y	ISSUE/REVISION DESCRIPTION	ENG	APPR

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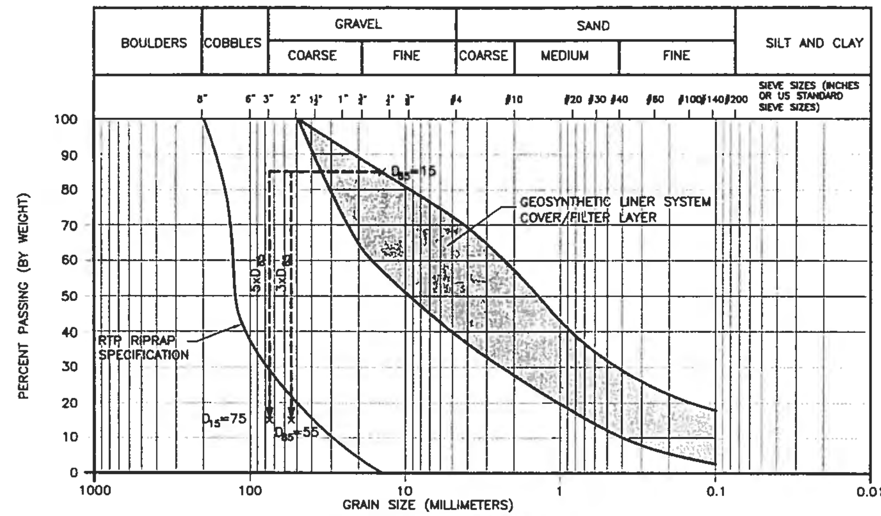
Client: teckcominco

Designed By: RW
Drawn By: YW
Checked By:
Reviewed by:
Scale: AS SHOWN

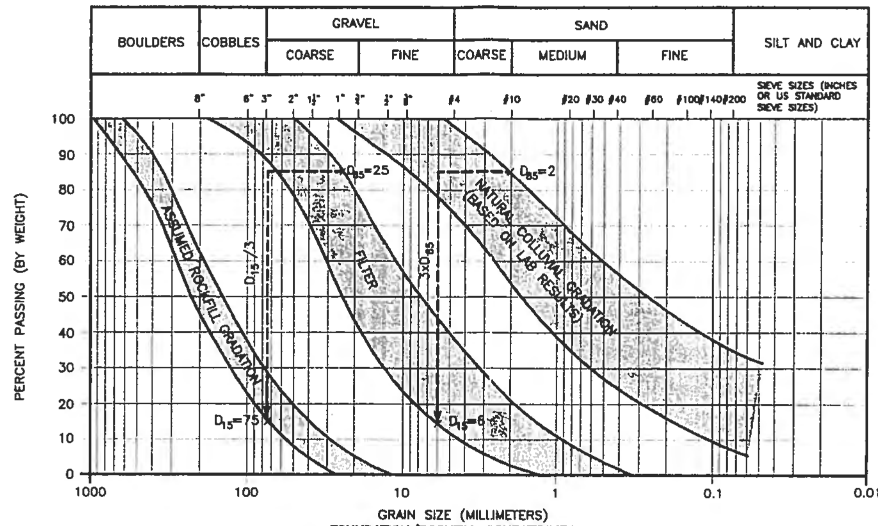
Project: DESIGN ELEMENTS - SEEPAGE ESTIMATES AND ROCKFILL DAMS

Title: RTP DAM OPTION 1 GEOMETRY & POTENTIAL SLIP SURFACES FOR STABILITY ANALYSES

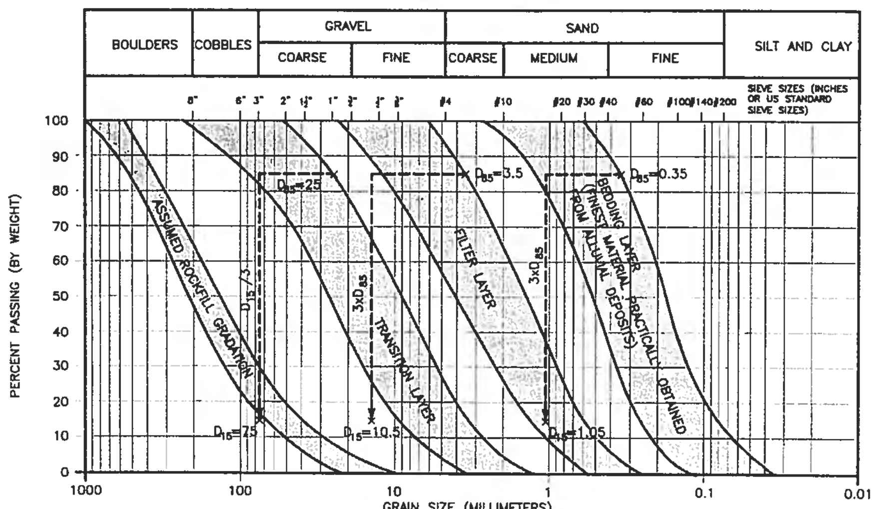
Project No.: VM00172-V-2
CADD File:
Date: DEC. 2001
Figure No.: B00172-03-006
Sheet No.: 1 of 1



COVER LAYER/RIPRAP COMPATIBILITY



FOUNDATION/ROCKFILL COMPATIBILITY



BEDDING LAYER/ROCKFILL COMPATIBILITY

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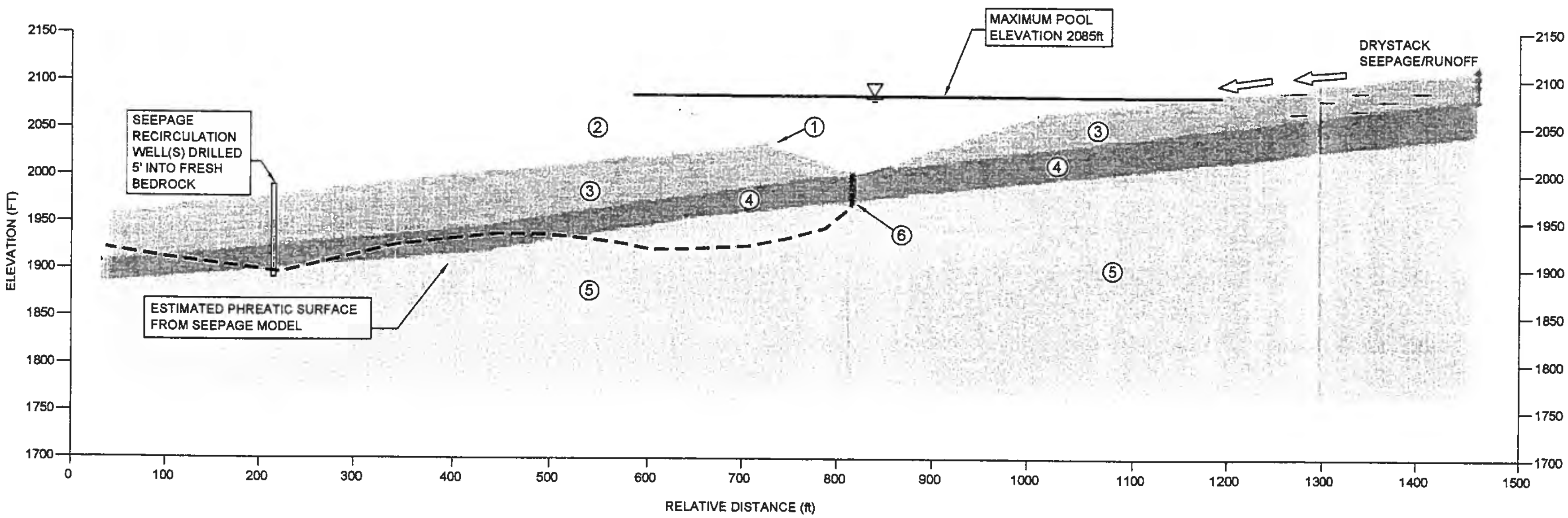
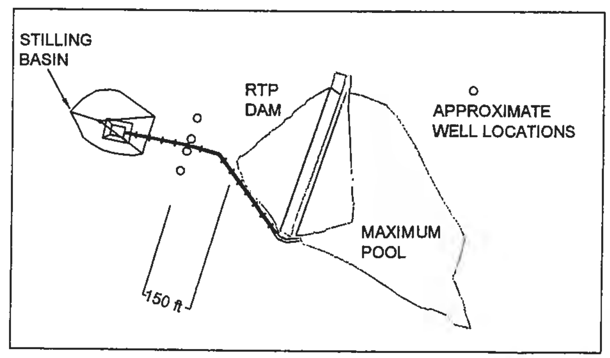
NOTE: SOIL CLASSIFICATION BASED ON UNIFIED SOIL CLASSIFICATION SYSTEM

amec	AMEC Earth & Environmental Limited 2227 Douglas Road Burnaby, B.C V5C 5A9 Tel 294-3811 Fax. 294-4664	DWN BY: RW/YW CHK'D BY: RW	DESIGN ELEMENTS - SEEPAGE ESTIMATES AND ROCKFILL DAMS	DATE DEC. 2001 PROJECT NO. VM00172-V-2
Client	teckcominco	APP. SCALE	FILTER COMPATIBILITY OF RTP DAM MATERIALS	REV NO - FIGURE No B00172-03-007



LEGEND

1. LINER
2. ROCKFILL
3. OVERBURDEN
4. WEATHERED BEDROCK
5. FRESH BEDROCK
6. SEEPAGE CUT-OFF GROUT CURTAIN



NOTE:
WELL LOCATIONS, AND NUMBER OF WELLS, TO BE OPTIMIZED BASED ON HYDRAULIC TESTING AND SCREENED INTERVALS

NOT FOR CONSTRUCTION

amec	AMEC Earth & Environmental Limited 2227 Douglas Road Burnaby, B.C. V5C 5A9 Tel. 294-3811 Fax. 294-4664	DWN BY: SM CHK'D BY: MPD APP: - SCALE: NTS	PROJECT: DESIGN ELEMENTS - SEEPAGE ESTIMATES AND ROCKFILL DAMS TITLE: RTP SEEPAGE COLLECTION SYSTEM CONCEPT
Client: teckcominco			DATE: DEC. 2001 PROJECT NO: VM00172-V-2 REV. NO.: - FIGURE No. B00172-03-008

Robertson GeoConsultants Inc.

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Memorandum

DATE: December 17, 2001

TO: Rick Zimmer, Teck Corporation
CC: Mike Davies, AMEC

FROM: Dr. Andy Robertson
Dr. Christoph Wels, Robertson GeoConsultants Inc

RE: Review Comments - RTP Dam Seepage Analysis, Pogo Project (RGC Project No. 064 002)

Rick:

As requested, we have completed a review of the site investigation data for the Liese Creek Basin, which were collected as part of the feasibility study for the Pogo gold mine project, Alaska. Our review focused on the data pertaining to the design of the Recycle Tailings Pond (RTP).

The proposed RTP Dam would store local surface and subsurface runoff and seepage from the dry stack tailings facility for reuse in the milling and processing facilities (and storage prior to treat/release). The proposed retention pond is to be built by constructing a rock fill dam in upper Liese Creek, some 1300 ft downstream of the proposed tailings dry stack. The maximum design capacity of the reservoir used in this analysis is 40 Mgal. The following measures have been proposed to minimize seepage from this reservoir (AMEC, 2001a):

- An impervious dual liner (a GCL sandwiched between two HDPE liners) at the upstream side of the rock fill dam;
- A positive cut-off at the upstream toe of the dam by excavation and removal of the coarse, granular overburden soils down to the existing weathered bedrock surface; and
- A grouted cut-off wall at the upstream toe of the dam installed through the weathered bedrock zone to the depth required for seepage control.

The overall objective of our review was to assess the potential for seepage from this reservoir and to assess the implications for the downstream environment. The following specific questions were to be addressed:

1. Is the work carried out to date on foundation characteristics reasonable, prudent and commensurate with a facility of this size?
2. What is the potential for the presence of structural features (faults and/or fractures) beneath the proposed RTP Dam?
3. What is the potential for seepage from the RTP reservoir (with or without the presence of potential structures)?
4. What are the potential impacts of any seepage from the RTP reservoir on downstream water quality? and
5. What can be done to minimize any impact on downstream water quality?

The following information was reviewed to address these questions:

- A structural map of the Pogo project site provided by Teck – Pogo Inc. and entitled "Significant Faults within the Liese Zone and Proposed Facility Sites" (date unknown);
- Appendix A of Water Management Plan for Pogo Project (July 2001) written by Adrian Brown and entitled "Inflow to the Pogo Mine, Alaska"
- Borehole logs and photo logs of 19 boreholes drilled during the 1999, 2000 and 2001 site investigations;
- Various plan views and cross-sections summarizing the drill information;
- Results of packer tests and slug tests (summary data only, a detailed analysis of the raw data was beyond the scope of this work);
- Seepage analyses carried out for the RTP Dam by AMEC Earth & Environmental Ltd. ("AMEC").

Review of Structural Information

Information on the structural geology in the Pogo project area was provided in a structural map developed by Teck's geologists. The hydrogeological characteristics of existing structures are discussed in Adrian Brown's report on mine inflow to the development drift for the proposed Pogo underground mine (Adrian Brown, 2001).

In the vicinity of the orebody, three sets of high angle faults have been identified:

- Northwest trending faults which include the Liese Creek Fault and Mid-Ridge fault; and

- North-east trending faults; and
- East-West Faults (including South Basalt Fault).

To date, only the Liese Fault and the Mid-Ridge fault have produced any significant water when intercepted by drill holes from the underground drift. Liese Creek fault runs subparallel to Liese Creek and exhibits right-lateral strike-slip offset of a few hundred feet (Adrian Brown, 2001). Hole 00U98C intersected a six foot wide shear zone which initially produced about 35 gpm. During a subsequent shut-in pressure test the hole produced 150 gpm with the majority of this flow believed to be originating from the Liese Creek fault (Adrian Brown, 2001). This flow has decreased over time and is presently about 115 gpm (R. Zimmer, pers. Comm.). A second hole (00U98D) intersected a 90ft zone containing faulted material, gouge and breccia but did not produce any measurable additional flow (Adrian Brown, 2001).

The Mid-Ridge fault runs approximately parallel to the Liese Creek fault and also has a right-lateral strike slip of several hundred feet. This fault appears to be water-bearing but high pressures encountered near the fault indicated that it may represent a barrier to flow (Adrian Brown, 2001).

Interpretation of the inflow and shut-in pressures suggested a high effective fracture transmissivity (K*width) of T=16,800 ft^2/yr (5E-5m^2/s) for the Liese Creek Fault. Assuming a fracture width of 10ft this represents an effective hydraulic conductivity of 1,680ft/yr (or 1.6E-5m/s). No direct estimates of fracture transmissivity were available for the Mid-Ridge fault. However, calibration of the flow model to total inflow into the drift and the drawdown observed in response to drifting suggested a transmissivity of T=17,000ft^2/yr (5E-5m^2/s) for this fault.

Note that the southeastern projection of the two water-bearing structures that have been identified (Liese Creek Fault and Mid-Ridge Fault) do not directly intersect the RTP Dam area (the Liese Creek fault runs about 800' south of the RTP Dam). The only high angle fault identified during the u/g drilling program that may intersect the RTP Dam area is the South Basalt Fault. This fault is a vertical fault that exhibits evidence of left-lateral strike slip motion of approximately 50ft. However, no water was encountered in this fault during u/g drilling (Adrian Brown, 2001).

The review of the structural geology in the project area suggests that the preferred orientation of permeable, water-bearing faults is in a northwest-southeast direction. This observation is consistent with the shape of the cone of depression created by the underground drifting. The data also suggest, that the transmissivity of any given fault may vary very significantly within relatively short distances. Hence caution should be exercised when trying to apply estimates of fracture transmissivity (or equivalent hydraulic conductivity) obtained at depth in the underground development area to any potential faults in the vicinity of the RTP Dam site.

Review of Site Investigation

Nineteen boreholes and four seismic refraction lines have been completed in the immediate vicinity of the RTP Dam and several more in the general area. All photo logs of the bedrock cores were reviewed and compared against the borehole logs completed by AMEC to allow an independent assessment of the weathering and fracturing in bedrock. In general, borehole logs were found to be in very good agreement with the photo logs. The foundation of the RTP Dam consists of sandy to very coarse overburden and a zone of highly to moderately weathered bedrock overlying competent, fresh bedrock (predominantly diorite). Fresh bedrock was encountered close to surface (0-4ft bgs) on the southern abutment of the proposed dam and about 80-90ft bgs near the north abutment. The thickest zone of weathered bedrock (~60ft) was encountered in the lower portions of the north abutment (in LD-9b). The bore logs suggested that the highly weathered diorite was typically relatively weak (i.e. crumbled on handling) with an RQD of 0-20%. Average RQD values in excess of 60% were reported for the moderately weathered and fresh diorite.

The photo logs and the borehole logs suggested that the "fresh" bedrock was fractured with a wide range of fracture orientations (ranging from 5°-85° TCA). The majority of these fractures were reported to be healed with mineral infilling and/or clay-gouge filled (AMEC, 2000b). Some fractures showed signs of oxidation suggesting that groundwater movement and/or unsaturated conditions have occurred within these fractures at some point in the past.

The photo logs of the inclined borehole (LD-9b drilled from the southfacing slope towards the center of the valley at an angle of 50° to the horizontal) showed three zones of weathered bedrock and/or broken rock, at greater depth within the "fresh" bedrock that may be associated with smaller faults. These fracture zones were encountered at 178'-196', 239'-257' and 280'-292' (all distances along the orientation of the borehole) as indicated on Figure A-XIV in AMEC (2001b). The intersected thickness of these "disturbed" zones ranged from 12-18ft (equivalent to a horizontal width of less than 10ft on average, depending on the angle of intersection).

These potential fault zones could not be traced, however, in the other inclined hole (LD-10) drilled at 70° from the north-facing slope towards the center of the valley (see Figure A-XIV in AMEC 2001b). This suggests that these fracture zones are either discontinuous and/or subparallel to the inclination of borehole LD10 (70°).

Thirteen packer tests were carried out in five boreholes drilled in the immediate vicinity of the proposed RTP dam location. The packer tests were aimed at testing the higher K material (M. Davies, personal communication). The packer tests yielded a wide range of hydraulic conductivity estimates and showed no clear correlation with degree of weathering, degree of fracturing and/or RQD of the bedrock. Very low hydraulic conductivities (<1.0E-9m/s) were determined for slightly-highly weathered, fractured bedrock (in LD6) as well as fresh, unfractured bedrock (in LD4). Moderate hydraulic conductivity values (~1E-5 to 5E-6m/s) were obtained for test intervals in weathered bedrock (LD2) as well as in fresh, fractured bedrock (in LD3 and LD5). No effective packer testing was completed in the potential fault zones encountered in LD9b.

It is the author's experience that packer tests often overestimate the actual permeability of fractured bedrock. This is because fractures may allow short-circuiting between the packer tested interval and other portions of the open borehole (allowing pressure to dissipate during the test). This may explain the seemingly high hydraulic conductivity estimates for fresh, fractured bedrock. In addition, the packer tests generally only test the permeability of a fractured bedrock in immediate vicinity of the borehole. A high hydraulic conductivity in a packer test does not necessarily imply a similarly high hydraulic conductivity at a larger scale. Caution should therefore be exercised when extrapolating such data to the field scale (over hundreds of feet).

Four of the boreholes in the RTP Dam area were completed as monitoring wells (LD3, LD5, and LD7A and LD7B) and slug tests were carried out in two of those wells ((LD3 and LD5). Both of those wells were screened in the overburden and hydraulic conductivity estimates ranged from 1.3E-7 to 1.3 E-6m/s. A slug test carried out in monitoring well LT7A screened in fresh, but moderately fractured bedrock some 1500ft upstream (at the proposed dry stack facility) suggested a hydraulic conductivity of 5E-7m/s. A packer test over a similar interval suggested a hydraulic conductivity of 1E-6m/s.

Review of Seepage Analyses

AMEC estimated seepage from the RTP pond to a downstream seepage collection system using the software SEEP/W. A series of sensitivity analyses were carried out to assess the potential range of seepage from the RTP Dam reservoir for a wide range of potential hydraulic conditions in the foundation soils and underlying bedrock. Table 1 below summarizes the hydraulic conductivity values assumed for representative seepage analyses carried out by AMEC.

Table 1. Summary of hydraulic conductivity values assumed for seepage sensitivity analyses (Scenarios 1 & 2).

Material	K_s (m/s)
Overburden[1]	10^{-4}
Finest upstream filter material[1]	10^{-7}
Rockfill[2]	10^{-1}
Weathered Bedrock[3]	10^{-6}
Bedrock[3]	
Sound ("Tight")	10^{-7}
Highly Fractured	10^{-6}
Cutoff Wall[4]	10^{-8}
Geomembrane[5]	10^{-14}

Notes:
1. Based on laboratory gradation results
2. Values based on engineering judgment.

3. Conductivity values based on packer test data gathered during the 1999 site investigation. Sound "tight" value at upper bound (most pervious) of available data and published values
4. Cutoff wall conductivity based on engineering judgment.
5. Saturated hydraulic conductivity based on published values.

Table 2(a) summarizes the seepage estimates from the RTP Dam reservoir for three selected scenarios, which, in our opinion, bracket the potential range of hydraulic conditions that may be encountered at the RTP Dam location. In Scenario 1, all bedrock underlying the weathered bedrock consists of sound, "tight" bedrock of low permeability. A seepage rate of about 5 gpm was estimated for this scenario. This analysis made the conservative assumption that the reservoir is at full capacity all the time (in reality hydraulic modeling indicates that it would only be full for 2-3 months per year).

Table 2. Summary of Seepage Estimates.

	Seepage Loss from Reservoir[1]	
Scenario	Maximum Pond level (El.=2085.0ft)	Average Pond Level (El.=2063.2ft)
(a) Results of Seepage Sensitivity Analyses carried out by AMEC		
Scenario 1. All sound bedrock "tight" (w/o fracture zones)	5 gpm[2]	N/A
Scenario 2. 25% of bedrock "highly fractured"; remaining 75% sound ("tight") bedrock	30 gpm[2]	21 gpm[3]
Scenario 3. Highly permeable Fault Zone (T=16,800ft2/yr)	23 gpm[4]	19 gpm[4]
(b) RGC Estimates of Seepage		
(i) Seepage Estimate for Poor Conditions: Seepage through 3 zones of highly fractured bedrock (10ft wide each) with K=100ft/yr plus flow through grouted weathered bedrock	N/A	10 gpm[3]
(ii) Seepage Estimate for "Reasonable" Worst-Case: Seepage through highly permeable fault zone (T=16,800ft2/yr) plus flow through grouted weathered bedrock	N/A	24 gpm[3,4]

Notes:
1) assuming effective grouting only to 40ft depth
2) assuming a cross-sectional width of 506ft (=length of dam in contact with pond)
3) assuming a cross-sectional width of 435ft (=length of dam in contact with pond)
4) assuming a high K fault zone with a nominal width of 10ft

Because of the low hydraulic conductivity of the geomembrane there is essentially no seepage through the dam and seepage through the foundation is mainly controlled by the hydraulic conductivity values assumed for the Cut-off Wall and for the sound bedrock beneath the weathered bedrock zone (Note that the grout curtain was assumed to fully penetrate the zone of weathered bedrock). The assumed hydraulic conductivity for the grout curtain is a design parameter that, we believe, is achievable with the proposed grouting program (one or more rows of inclined drill holes at approximately 10ft centers). The hydraulic conductivity value selected for "sound" bedrock (10^{-7} m/s, see Table 1) is significantly higher than K values typically obtained in packer tests conducted in sound (unfractured) bedrock ($\leq 10^{-9}$ m/s, see above). Hence, the estimated seepage rate of 5 gpm has to be considered a conservative (high) estimate for scenario 1.

In the second scenario, 25% of the bedrock beneath the weathered zone was assumed to be highly fractured with an effective hydraulic conductivity of $K=10^{-6}$ m/s (100 ft/yr). The relative proportion of 25% of fractured bedrock versus 75% tight bedrock is based on the relative proportion of highly disturbed (fractured) zones in fresh bedrock with low RQD (typically <30) encountered in the inclined borehole LD9b (approximately 46ft of fractured rock over 190ft of fresh bedrock drilled). The hydraulic conductivity assumed for fractured rock ($K=10^{-6}$ m/s) is consistent with packer test results obtained in fractured rock (beneath the zone of weathering). The estimated seepage losses from the RTP Dam reservoir for this second scenario range from 21 to 30 gpm depending on the assumed elevation of the reservoir (Table 2(a)).

The third scenario examined the potential for seepage losses from the RTP Dam reservoir through a discrete, highly permeable structure such as the Liese Creek fault zone. It was assumed that this hypothetical fault zone is aligned along the center of the Liese Creek valley thus maximizing the potential for seepage from the reservoir. A transmissivity of $T=16{,}800ft^2/yr$ (or $5*10^{-5}$ m2/s) was assumed for this hypothetical structure, i.e. a value equivalent to that estimated for the Liese Creek fault based on observed flows into the underground workings (see above). The estimated seepage from such a discrete structure would be in the order of 20 gpm (Table 2(a)). Note that the model assumes that this structure is laterally extensive over several hundred feet in distance and many tens of feet in depth below the weathered bedrock.

In our opinion, the seepage analyses carried out by AMEC provide a good indication of the potential seepage rates that might be expected under different hypothetical hydraulic conditions for the bedrock underlying the weathered zone. RGC used the results of these sensitivity analyses to estimate seepage losses from the RTP Dam reservoir for a conservative scenario of "poor conditions" and a "reasonable" worst-case scenario (see Table 2(b) discussed below).

Conclusions

Based on the review summarized above we conclude the following:

Question 1: Is the work carried out to date on foundation characteristics reasonable, prudent and commensurate with a facility of this size?

In our opinion, the degree of field investigation goes beyond what is typically required and carried out for the feasibility design of a structure the size of the proposed RTP Dam. However, as there is limited data on packer testing near the center of the Liese Creek valley, we recommend that three additional boreholes (inclined at ~45°) be drilled into fresh bedrock underlying the center of the valley to assess the possibility of significant fracturing/faulting along the axis of the valley along the proposed seepage cutoff. The bulk hydraulic properties of fractured bedrock beneath the zone of weathering should be evaluated by carrying out single packer tests during advance of the borehole. In all tests the packer should be set as high as possible in fresh bedrock in a section with high RQD. The effectiveness of the packer seal and any potential short-circuiting through the rock mass at that location should first be evaluated by testing a very small zone of competent bedrock. The final packer test (over the entire length of the borehole in fresh bedrock) would provide an estimate of the bulk hydraulic conductivity of the fractured rock mass. A comparison of relative changes in water take between subsequent packer tests would be indicative of the relative permeability of any individual fracture zones encountered during drilling.

Such drilling and testing could be carried out during construction of the dam, provided contingency measures are in place to deal with potentially permeable fracture zones (see below).

It is the authors' opinion, that if no extensive permeable structures (with an effective hydraulic conductivity $>10^{-6}$ m/s) can be identified in these additional inclined boreholes than the lower bound of seepage estimates (i.e. Scenario 1 in Table 2(a)) would be the most appropriate estimate for final design. However, the AMEC designed grouting program would still be recommended.

Question 2: What is the potential for the presence of structural features (faults and/or fractures) beneath the proposed RTP Dam?

The drilling in the vicinity of the RTP Dam suggests the presence of individual fractures and fractured zones of broken rock within the competent bedrock (in particular in the center of Liese Creek valley). Based on the information provided in the drill logs and the structural interpretation of underground drill holes it is our expectation that the fracture zone(s) run along Liese Creek valley and are vertical or sub-vertical. Under a conservative assumption, there is potential for a hydraulic connection of such structures with the weathered bedrock and the overburden (including the area upstream of the RTP Dam).

Based on the drilling to date, the width of individual fracture zones appeared to be in the order of 10 ft or less. However, a larger fault zone consisting of several distinct zones of broken rock cannot be ruled out at this point. As noted above additional drilling would be required to clarify the nature and extent of structural features in the deeper bedrock prior to the construction grouting program.

The bedrock drilling program was not designed to address the potential for a major water-bearing structure running perpendicular to the Liese Creek valley in the RTP Dam area (i.e. in a NE-SW direction). We concur and believe that the potential for such a fault is rather small (note that none of the north-south trending faults encountered during underground drilling were water-bearing). The generally good rock conditions encountered on the north-facing slope of Liese Creek support this (preliminary) conclusion. Further, a fault with this orientation would be subject to high piezometric heads on either valley flanks such that a hydraulic gradient away from the RTP would not occur, preventing seepage losses along such faults. Consequently, we do not recommend any additional drilling to study the potential presence of a cross-valley fault.

Question 3: What is the potential for seepage from the RTP pond (with or without the presence of potential structures)?

The potential for seepage from the RTP Dam reservoir is a function of the hydraulic characteristics of the more permeable foundation units that are present under the dam and could connect to the reservoir and that remain ungrouted. In our opinion, the seepage analyses carried out by AMEC provide a good framework for estimating the likely range of seepage rates from the reservoir.

Table 2(b) summarizes our estimates of seepage losses from the RTP Dam reservoir for conservative scenarios of "poor conditions" and a "reasonable" worst-case. Those estimates are based on the cross-sectional seepage analyses carried out by AMEC (Scenarios 1-3 in Table 2(a)).

In our opinion, a conservative (high) estimate of seepage from the RTP Dam reservoir (based on the existing information) is likely one with preferred seepage along some discrete fractures and/or fracture zones below the grout curtain. Assuming 3 vertical fracture zones (with an average width of 10ft each) extending several hundred feet upstream and downstream of the grout curtain and with an average effective hydraulic conductivity of $K \cong 100$ ft/yr ($T=1{,}000 ft^2/yr$) each, would produce a combined seepage of about 5 gpm from the reservoir. Adding this fracture flow to the seepage through the grout curtain in the remainder of the cross-section (5 gpm, Table 2(a)) our best estimate of seepage for poor conditions (i.e. presence of fractured rock) would be about 10 gpm (Table 2(b)).

In our opinion, a "reasonable" worst-case scenario for the RTP Dam location would constitute the presence of a structural feature equivalent to the Liese fault zone ($T=16{,}800 ft^2/yr$). Such a fault zone would produce a seepage loss of about 19 gpm from the reservoir under average pond operating conditions (Table 2(a)). Hence the total seepage loss from the reservoir (including seepage losses through the grouted weathered bedrock) would be 19+5=24 gpm (Table 2(b)). Note that this combined seepage flow is equivalent to the seepage losses that would occur from the RTP Dam Reservoir if ~25% of all the fresh foundation bedrock (beneath the foot print area of the proposed reservoir) was "highly fractured" with a bulk hydraulic conductivity of K=100ft/yr (1E-6m/s). These calculations likely represent an upper (conservative) bound but they emphasize the need to refine current estimates of hydraulic properties for the fractured bedrock and/or a potential fault zone that may or may not be present beneath the proposed RTP Dam.

Question 4: What are the potential impacts of any seepage from the RTP reservoir on downstream water quality?

It is our understanding that the water quality of the water stored in the RTP reservoir is marginally above standards (i.e. less than 2 times above standards). This implies that a dilution factor of 2 would be adequate to reduce the concentrations to acceptable levels. Assuming a recharge factor of 1 inch/year to groundwater (Adrian Brown, 2001) a catchment area of about 78 ha would be required to reduce our current estimate of seepage (~10gpm) to acceptable levels. However, in the current feasibility design provisions are made for the interception of any seepage from the RTP reservoir about 100-200 ft downstream of the dam. In our opinion, the efficiency of such an interception system (if properly designed) could be as high as 80-90% of total seepage for the hydrogeological conditions considered here (see Table 2). Assuming an efficiency of 85% for the interception system, the requirement for dilution from groundwater with background water quality would reduce to a catchment area of 7.8 ha. Note that the Liese Creek (at the mouth) commands a catchment area of about 610 ha. Based on these calculations it is our opinion, that there is no threat for seepage from the RTP Dam to result in unacceptable levels of contaminants in groundwater in the Goodpaster River valley.

Note that runoff (i.e. surface water and groundwater flow combined) in the Pogo area is significantly higher than groundwater recharge (estimated to be ~7.5 inches per year). Hence, the required catchment area for diluting streamflow to acceptable levels would be significantly smaller than those derived above for groundwater.

It should be pointed out that there is some, albeit small, potential that groundwater levels in the RTP dam area itself or in the Liese Creek area further downstream will be drawn down due to dewatering of the underground workings. Given the relatively low permeability of the intact bedrock such a propagation of drawdown will likely only occur if permeable high angle faults connect from the ore zone to upper Liese Creek and the RTP Dam area. In this event, any seepage from the RTP Dam in the bedrock aquifer (likely confined to permeable fractures and/or fracture zones) would be captured in the underground workings.

Question 5: What can be done to minimize any impact on downstream water quality?

As mentioned in our response to Question 1, there is some uncertainty as to the degree of fracturing and the hydraulic properties of the bedrock underlying the proposed RTP dam location. Although it is never possible to remove all uncertainty, the level of uncertainty can be significantly reduced with a minimum of three additional inclined holes be drilled (at an angle of 45° to the horizontal) which penetrate some 100 ft into fresh bedrock near the center of Liese Creek valley along the proposed seepage cutoff alignment. Single packer tests should then be performed on progressively larger sections of the borehole in fresh bedrock. Further hydraulic testing in these holes (using slug tests and/or pump tests) should be considered if the packer tests suggests hydraulic conductivities in the order of 10-6m/s or higher.

If the suggested drilling and hydraulic testing should indicate that a permeable structure (or several structures) is present and that resulting seepage from the reservoir would be unacceptable from an environmental point-of-view then one of three options (or a combination thereof) could be pursued:

- Grout the permeable structure(s) to considerable depth and perform hydraulic tests to verify that a specified (low) bulk hydraulic conductivity has been achieved (as provided for in the design); and/or:
- Grout the permeable structure(s) along the entire length of the reservoir (upstream of the dam); and/or
- Adjust the seepage interception system below the RTP dam (as provided for in the design) to intercept seepage flowing in such a permeable structure (e.g. active pumping from inclined boreholes completed in the permeable structure).

The selection of which option(s) to pursue would have to be made after completion of the additional site investigation. We also recommend that the seepage interception system be designed with extra capacity to handle flows up to 100 gpm to provide an appropriate measure of contingency.

Provided appropriate contingency measures are in place (and are approved by the Alaskan regulatory authorities) the proposed additional drilling and hydraulic testing could be carried out during actual construction of the RTP dam.

We trust that these review comments meet your requirements at this time. Please contact the undersigned if you have any questions regarding this letter.

Best Regards,

ROBERTSON GEOCONSULTANTS INC.

Andy Robertson, Ph.D., P. Eng..
President

Christoph Wels, Ph.D, M.Sc.
Principal Hydrogeologist

REFERENCES

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