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WRIGHT ENGINEERS LIMITED

PROVINCE OF BRITISH COLUMBIA

MINISTRY OF THE ATTORNEY GENERAL

PRELIMINARY GEOTECHNICAL ASSESSMENTS AND RELATED CONCERNS FOR THE PROPOSED SHERWOOD PROJECT STRATHCONA PARK, BRITISH COLUMBIA

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SUMMARY AND CONCLUSIONS

This report provides a preliminary geotechnical assessment and describes the principal geotechnical concerns relating to the proposed Sherwood Project in the Drinkwater Valley on Vancouver Island, British Columbia. The mine components addressed in this assessment include the access road, the underground mine, the sedimentation facility, the plant site, tailings pond, campsite, and water supply. The study is based on a review of available geological information, an examination of topographic maps and aerial photographs and a one-day site reconnaissance using helicopter and ground traverses. Hence, all conclusions and cost estimates are preliminary and would have to be verified by detailed investigations and analyses at the detailed design stage of the project.

The main geotechnical concerns and preliminary order of magnitude cost estimates for each of the mine components are summarized on Table IV and described in detail in Sections 3 and 4. A breakdown of the estimated costs for the access road, sedimentation facility and tailings disposal facility are included in Tables I, II and III, respectively. These costs are preliminary and all designs must be confirmed or modified during the detailed design phase of the project.

Based on the brief site reconnaissance, examination of airphotos and review of the information provided for the preliminary study, it is my opinion that the project is feasible from a geotechnical point of view, provided careful attention is given to the geotechnical design criteria determined and operating constraints identified and described in Sections 3 and 4 above. The project does have a number of significant geotechnical constraints (risks) and related environmental constraints which are related to the steep terrain and adverse climatic conditions at the project site. These constraints will add significantly to the cost of construction and maintenance of the facilities, particularly the access road and infrastructure. It is noteworthy that development of infrastructure (roads and supply system) has been a major cost at a number of mines recently developed in severe terrain in British Columbia.

The environmental impact of the various mine components as they relate to the geotechnical design is also extremely important, as addressed by Mr. J. Villamere, P.Eng. of Hatfield Consultants Ltd.

Practical engineering solutions can be designed to mitigate the problems and/or optimize the risks arising from most geotechnical constraints. The cost of these engineering solutions must be accounted in the overall feasibility assessment and used as a basic input into the overall economic feasibility of the project.

Respectfully submitted,

PITEAU ASSOCIATES ENGINEERING LTD.

Dennis C. Martin, M.Sc. DIC, P.Eng.

1. INTRODUCTION

1.1 TERMS OF REFERENCE

Piteau Associates Engineering Ltd. were retained by Wright Engineers Limited to review the geotechnical aspects of the proposed Sherwood Project, as part of the overall feasibility assessment commissioned by the Ministry of the Attorney General of the Province of British Columbia. This report summarizes the results of the geotechnical studies, and provides background for the geotechnical concerns relating to the access road and infrastructure, the underground workings, waste rock disposal, the treatment of mine water, the plant site, the tailings disposal scheme and the camp facilities.

This study has been undertaken without the benefit of specific subsurface investigations, such as digging of test pits or drilling test holes, because of time constraints and the need for special permits to conduct such work within the park area. Although these studies must be considered preliminary, it is our opinion that the conceptual layouts prepared for the various facilities are appropriate for the type and size of project defined by Wright Engineers Limited. The main geotechnical concerns raised would have applied to this mine in the mid-1980's, had the project proceeded at that time.

1.2 STUDY PERSONNEL

The geotechnical studies have been conducted by Mr. Dennis C. Martin, M.Sc. DIC, P.Eng, President of Piteau Associates Engineering Limited, with the assistance of geotechnical engineers and hydrogeologists from Piteau Associates Engineering Ltd. Mr. Martin is a registered professional engineer in the provinces of British Columbia and Alberta; he has over 19 years experience in geotechnical studies for mining projects worldwide. A copy of Mr. Martin's resume is included in Appendix A.

The environmental aspects of the project are being handled by Mr. John Villamere, P.Eng. of Hatfield Consultants Limited. Mining geology aspects are addressed by Mr. Dave Barr, P.Eng. of Barrda Minerals Inc. All other engineering and construction aspects are being addressed by Messrs. Bill Norquist, P.Eng. and Dave Wortman, P.Eng. of Wright Engineers Limited.

1.3 DESCRIPTION OF THE INVESTIGATION

The geotechnical studies have consisted of a review of available geological and mining reports, an examination of topographic maps and aerial photographs, and a one-day site reconnaissance of the project using helicopter and ground traverses. Mr. Martin conferred with members of the project team during the site reconnaissance and during the conceptual development of the project, to ensure that a feasible approach was being developed and that the geotechnical aspects were addressed in a rational manner, based on sound engineering principles.

2. SITE DESCRIPTION

2.1 LOCATION AND ACCESS

The Sherwood Project is located in steep mountainous terrain within the boundaries of Strathcona Provincial Park on Vancouver Island, British Columbia as shown in Fig. 1. The site is located on the north valley wall of Drinkwater Creek, approximately 12 km from the head of Great Central Lake. The existing underground workings are located within an avalanche chute and slide area, which extends from Drinkwater Creek at an elevation of about 640m (2100 ft) to about 1400m (4600 ft) elevation at a point near the ridge crest. The valley walls slope at an angle of about 45° in the vicinity of the proposed mine (see Fig. 2). The existing underground workings are located about 500m above the creek, and extend from approximately 1128m to 1375m elevation (3698 to 4512 ft elevation).

Current access to the site is by barge or boat to the head of Great Central Lake, and by a Ministry of Parks hiking trail from the east end of Great Central Lake to Della Falls and Love Lake. The trail follows the route of a logging railroad and truck road built along Drinkwater Creek, and abandoned in 1946. Bridges used for the abandoned road have rotted out, washed out, or have been removed and replaced with footbridges at various locations.

2.2 PHYSIOGRAPHY

Drinkwater Creek occurs in a glacially carved valley with steep side slopes and high relief typical of many valleys in central Vancouver Island. Relief typically exceeds 2000m (6,500 ft), and valley side slopes vary from less than 10° to 75°. The upper section of Drinkwater Creek above about 300m elevation consists of a narrow valley bottom with steep side walls, and an average creek gradient of 5°. Two waterfalls occur in this area, as noted on Fig. 2. Below about 300m elevation, the valley bottom widens and a more pronounced U-shaped valley, indicative of more extensive glacial action, occurs to the mouth of the creek. Creek gradients in the lower section of the creek are generally less than 2°.

2.3 CLIMATE

Climate data has not been collected by the mine proponent, as would normally be desirable for the engineering assessments and design of the various facilities. Climatic characteristics in the Drinkwater Valley may be similar to the existing mine site at Myra Falls, for which climatic data is available. Information provided by Wright Engineers Limited for that site indicates the following pertinent climatic information:

Mean annual precipitation	2921mm
Maximum monthly precipitation	1218mm
Maximum daily precipitation	159mm
Mean winter snowfall	3920mm
Maximum winter snowfall	7900mm

It must be appreciated that climatic conditions in the Drinkwater Valley could be considerably different than at Myra Falls due to the relative location and orientation of Great Central Lake, the different orientation of the valley with respect to the prevailing winds, and specific relief within the very narrow valley. It is possible that considerable variations in climate could occur between Great Central Lake and the upper sections of Drinkwater Valley, and also from the valley bottom to the top of the ridge (Love Lake).

In terms of normal engineering practice, the use of climate data from Myra Falls for final design of facilities at the Sherwood Project would be questionable. Myra Falls data is only suitable for a preliminary assessment of the likely range of climatic effects which may impact the proposed development. In order to collect sufficient climate data for detailed design and permitting, it would most likely be necessary to establish climate monitoring stations at both the mine site and at the plant site, and to collect climatic records for a minimum period of one to two years. Ongoing data collection would be required at both stations during operation, to develop a sound climate data base for preparation of the reclamation plans at closure.

3. ENGINEERING GEOLOGY

A detailed account of the regional geology and local geology has been prepared by Barr (1990) based on previous reports by Sargent (1940, 1941), Muller (1980), Heard et al (1989), and others. The following geological descriptions are based on a review of those documents and the site reconnaissance. The salient engineering geology features as they relate to the specific geotechnical concerns for the various facilities required for the project are described in Section 4.

3.1 BEDROCK GEOLOGY

3.1.1 Sicker Group Volcanic and Sedimentary Rocks

Bedrock in the upper Drinkwater Valley is expected to consist primarily of hard, fractured volcanic and sedimentary rocks of the Sicker Group. Dr. Muller, of the Geological Survey of Canada, has mapped the Sicker Group over large sections of Vancouver Island and has indicated this formation to have a thickness of 1000 to 3000m. Reports and maps by Dr. Muller (1977, 1980) indicate that within the Sherwood Project area, the Sicker Group consists mainly of volcanic rocks described as agglomeratic lava flows, fine grained banded tuffs and volcanic breccias and denoted as CP_{SV} in Fig. 2. Outcrops of limestone and chert of the Buttle Lake Formation (CP_{BL}) are also indicated to occur within the Sicker Group in the area (see Fig. 2). These rocks are expected to be generally durable, as evidenced by the formation of steep cliffs on the valley sides and waterfalls within the creek.

The Sicker Group rocks are considered to be approximately 300 million years old (i.e. Paleozoic age), and were subsequently metamorphosed as a result of a probable low grade regional metamorphism approximately 250 million years ago.

The mineralized zone at the Sherwood Project occurs in a shear zone identified within the Sicker Group Rocks. This zone, which is designated the Sherwood Shear on Fig. 2, is likely to consist of fractured and sheared rock, which may have contributed to the formation of the avalanche gully and rockfall hazard area where the shear zone outcrops on the valley walls. Rock strengths are expected to be lower within the shear zone, and rock may be less durable.

3.1.2 Island Intrusions

Bedrock in the lower portion of the Drinkwater Valley downstream of the waterfalls at about 300m elevation is indicated to consist of intrusive rocks of the Island Intrusions (Fig. 2). Dr. Muller (1977) indicates that these rocks consist of bodies of granite to quartz diorite which were intruded into the surrounding bedrock between 140 and 180 million years ago. These rocks are generally expected to be hard and fractured.

3.3 STRUCTURAL GEOLOGY

Examination of available geological maps and airphotos indicates several well developed sets of structural lineations, which are likely related to development of geological faults or related throughgoing discontinuities. Lineations are strongly developed in the Sicker Group, and generally fall into three well developed trends, as follows:

i) well developed lineations which strike west-northwest subparallel to an alignment of major creeks and gullies

ii) well developed lineations which strike approximately N10E subparallel to an alignment of major creeks

iii) less well developed shears which strike northeast and may be subparallel to the Sherwood Shear shown on Fig. 2

It is likely that major faults are formed along all of these trends, particularly within the valley bottoms of Drinkwater Creek or other linear valley segments in the area. Lineations are less well developed within the Island Intrusions in the lower section of Drinkwater Valley. However, it is anticipated that faults and shears may occur within steep gullies and valley bottoms in these areas.

Formation of the valleys and gullies along faults is generally accepted, because the fault zones are susceptible to erosion. The presence of faults in valley bottoms and on the valley sides could be expected to result in less competent rock masses in some areas, and could also result in increased rockfall activity and landslides, as noted in some areas along the Drinkwater Valley. The effect of faults on groundwater flows, and possible losses of mine effluent or contaminants into more permeable fault zones, would have to be addressed for detailed design.

3.4 SURFICIAL GEOLOGY AND LANDFORMS

Bedrock outcrops over much of the valley walls along the entire length of Drinkwater Creek (Fig. 2). Materials encountered in the valley bottom, and on the lower valley sidewalls, range from bedrock and colluvium in the upper sections of the creek above about 300m elevation, to a range of colluvial, alluvial and morainal deposits in the lower portions of the valley. A preliminary interpretation of the distribution of the surficial soil deposits based on an examination of available airphotos and the brief site reconnaissance is given in Fig. 2. The specific occurrence of the various surficial deposits is related to the history of formation of the valley and the geomorphological processes which have been active during and after glaciation, as discussed in the following.

3.4.1 Morainal Deposits

Morainal deposits are derived from the action of glaciers which grind and pulverize bedrock and pre-existing soils. Glacial action results in the formation of a range of soil types below, on the edges and in front of advancing and retreating glaciers. These deposits can range from dense clay silt to silty sand and gravel, typical of "till-like" deposits throughout British Columbia, to looser sand and gravel materials formed from end moraines, eskers and other glacial features. The diagnostic U-shape of the Drinkwater Valley downstream of the waterfalls at 300m elevation indicates that extensive glaciation has occurred in that area.

Although no morainal deposits were identified during the site reconnaissance, there is some indication from the airphotos that such deposits may exist on the flanks of the north valley side between Great Central Lake and the end of the abandoned railroad. The occurrence and nature of suitable deposits of fine grained morainal soils (tills) in an accessible location outside of Strathcona Park would be advantageous for use as a source of borrow for pond liners and other uses.

3.4.2 Colluvial Deposits

Colluvium is a general term used to describe soils which have been transported downslope to their present location by gravity, with limited transportation by water. Several different types of colluvium were noted within the Drinkwater Valley, as follows:

i) Talus (T)

Talus typically forms as an apron along the base of slopes as a result of rockfalls and related eresion of bedrock. Talus typically consists of very large blocks in excess of 1m dimension in a matrix of sand and gravel. Talus slopes are subject to downslope creep and sliding resulting in considerable maintenance requirements for roads, bridges or structures located on such deposits. Presence of large blocks in these deposits makes excavation difficult and blasting is normally required to break large blocks into manageable sized pieces.

ii) Avalanche/Debris Fans (F)

Avalanches from steep gullies and chutes on valley walls will transport considerable organic debris, rock and soil to the base of the slope, resulting in the formation of large fans of mixed debris. Floods and washouts during heavy rains or snowmelt will also carry organic debris, rock and soil to these fans. Avalanche chutes and gullies were identified over large portions of the walls of Drinkwater Valley, particularly on the south side of the creek upstream of Margaret Creek. A total of ten active fans were identified on the south side of Drinkwater Creek upstream of Margaret Creek. A total of four active fans were noted on the north side of the creek over the same interval. There is evidence that two avalanches have blocked Drinkwater Creek and may even have led to floods and washouts in the creek.

It is important to note that avalanches could occur almost anywhere along the steep valley walls of Drinkwater Creek. In addition, washouts, debris flows and debris torrents could be expected on many creeks and gullies at any time of the year. Some creeks may washout more than once in a season. Such events could destroy bridges/culverts or even block creeks, leading to flooding and washouts.

iii) Undifferentiated Colluvium (U)

Deposits of undifferentiated colluvium are indicated at various locations along the Drinkwater Valley, as shown in Fig. 2. There was insufficient time to assess the nature of these deposits during the brief site reconnaissance. These deposits are expected to consist of mixed deposits of silty sand and gravel which are transported and deposited under less dynamic conditions than the talus and fan deposits. Detailed site reconnaissance would be required to define the distribution and nature of these colluvial deposits.

iv) Landslides (L) and Rockfalls (R)

One landslide, one large rockfall and several smaller rockfalls were identified in the Drinkwater Valley during the site reconnaissance (Fig. 2). These deposits indicate that active mass wasting processes are continuing to occur in the valley. We did not have an opportunity to examine all the valley walls in detail to assess the potential for additional landslides or rockfalls. Such events could result in closure of the access road for lengthy periods.

3.4.3 Alluvial/Organic Deposits

Alluvial deposits consisting of sand and gravel were noted at several locations within the floodplain of Drinkwater Creek (Fig. 2). These deposits are likely to consist of sand and gravel which could be used for embankment construction and general fill. Use of these materials as concrete aggregate or other specialized uses might not be feasible, due to the large percentage of cobbles and boulders. Use of this material could not be considered without further sampling and testing of possible aggregate sources within the valley. Special permits would likely be required for development of borrow pits within any areas which may impact the fisheries resource. If a source of suitable aggregate could not be identified and permitted, the required materials would have to be transported to the site by barge and/or truck.

Several areas within the floodplain or on terraces above Drinkwater Creek consist of marshes and swamps which may contain appreciable quantities of peat, organic silt and related soft sediments. These deposits should generally be avoided for any type of construction or borrow due to the high water content and organic content.

3.5 HYDROGEOLOGY

A detailed hydrogeology assessment has not been conducted for this preliminary review. It is generally expected that groundwater flow in bedrock in the study area will be confined primarily to fault and fracture zones. It is unlikely that significant aquifers will be encountered within the volcanic rocks in Drinkwater Valley.

Groundwater aquifers are likely to occur in the surficial deposits, particularly within the alluvial deposits on the floodplain of Drinkwater Creek, and colluvial deposits on the valley sides and upper Drinkwater Valley. The distribution of aquifers and groundwater flow systems within the surficial deposits could only be determined by detailed site investigations which would be required for any structures, water supply, sewage disposal, etc. A detailed assessment of the various aquifers in the valley and the environmental impact of the various structures on groundwater quality would be required for permitting.

4. GEOTECHNICAL CONSIDERATIONS AND CONCERNS

4.1 ACCESS ROAD

A preliminary feasibility and geotechnical assessment of the access road for the proposed Sherwood Project has included a review of geotechnical hazards and construction feasibility for two possible routes. Conceptual designs and preliminary order of magnitude cost estimates have been prepared for two possible alignments on the preferred route.

4.1.1 Geotechnical Concerns

The main geotechnical concerns for the proposed access road are to establish the road on stable terrain, with a minimum of geotechnical hazards. Review of the airphotos and preliminary terrain analyses as described above indicates that it will be impossible to avoid all geotechnical hazards, particularly those developed from avalanches, debris flows, debris torrents, floods and rockfalls or landslides from the valley walls. It would be prudent to lay out the road on such an alignment as to minimize the hazard from these features. It is important to note that mitigative measures for geotechnical hazards will add considerably to the construction costs, as well as the maintenance and repair costs for the road and creek crossings.

4.1.2 Route Selection

Two alternate routes were initially reviewed for the access road:

i) Ash River/Gretchen Creek Access

An "all-road" access involving extension of the existing logging roads along the Ash River and Gretchen Creek into the Drinkwater Valley was considered. This road would require construction of 11 km of new road from the headwaters of Gretchen Creek, through the valley of Margaret Creek and into the Drinkwater Valley, north of Margaret Creek. Construction costs for this route are estimated to range from $1.7 million to $2.8 million. This option would also require winter maintenance of about 30 km of road to service the plant site and camp, and these costs could be anticipated to be significantly greater than the Great Central Lake/Drinkwater Creek access described in the following.

ii) Great Central Lake/Drinkwater Creek Access

Site reconnaissance and review of available information indicates that a practical access would be to transport equipment and supplies by barge to a dock or landing at the head of Great Central Lake, and by truck along Drinkwater Valley to the project site. This approach would reduce the costs of new road construction, because the access road would generally follow the previously established railroad and truck road which was developed to the old sawmill site near Della Creek. This route would substantially reduce the winter maintenance costs, particularly if the road were to be operated only during snow free periods. Construction methodology and preliminary order of magnitude costs for two possible alignments of a road constructed along Drinkwater Creek are discussed in the following.

4.1.3 Docking Facility on Great Central Lake

The site reconnaissance has shown that a number of docking facilities have been established at the terminus of the railway at the head of Great Central Lake in the past. These facilities appears to have consisted of a pier and a log dumping trestle built on piles. The only remaining remnants of these facilities are the rotted pilings from the trestle and pier.

We have not conducted any detailed assessments of the methodology and costs of establishing the required docking facility. It is likely that a suitable facility could be constructed using a pier supported on piles, or a dock constructed on fill. Alternatively, one or two barges moored at the terminus of the road may provide the least expensive temporary docking facility. Detailed engineering and environmental studies would be required to determine the optimum design of the docking facility. The studies required and procedures involved to obtain the required agency approvals must also be considered.

4.1.4 Assessment of Alternate Access Road Alignments

Two alternate alignments were considered for the access road up the Drinkwater Valley. Both alignments are shown on Fig. 2, and the various components and estimated construction costs are summarized in Table I. The methodology and construction approach for each alignment were the same. Differences in costs arise due to the relative amounts of the different type of road construction, and the cost of constructing adequate crossings of Drinkwater Creek.

i) Alignment 1

Alignment 1 would consist of rehabilitating the entire length of the abandoned railway and previously constructed truck road from the end of the railway to the old sawmill near Della Creek. The section from Della Creek to the proposed portal would require construction of approximately 1040m of new road in bedrock and talus. Road rehabilitation would require clearing, grubbing, regrading and resurfacing of the abandoned road sections, and reestablishment of all river and creek crossings using bridges or culverts, depending on the estimated flows and hydraulic constraints.

While this alignment would at first appear the most practical and also the most economic, a number of geotechnical and environmental concerns were noted for this alignment which result in a higher cost for developing this option. These constraints are summarized as follows:

a. Two major crossings of Drinkwater Creek will be required on the old road alignment. These crossings appear to have been washed out in previous floods, and hence it is likely that relatively costly bridges would have to be constructed to cross the creek. As the crossings would be removed subsequent to mining, it is likely that temporary bridges such as Bailey bridges would be feasible at these sites. Approximately 25m long bridges would be required, at an estimated cost of about $125,000 per bridge.

b. Permission to place the highly visible bridges across Drinkwater Creek may have been difficult to obtain even in 1987/88.

c. By crossing to the south side of the creek, the road would be subject to a number of severe avalanche/debris flow chutes which have virtually eliminated the old road, and would pose a very serious and ongoing hazard to the rehabilitated road.

d. A portion of the old road on the south side of the valley was originally constructed on the floodplain of the creek or in the creek channel, and has subsequently washed out. Although the road grade appears to have been constructed in the creek in 1946, it is unlikely that permission would be obtained to reconstruct the road in the creek in 1988, even if it was not located in the park. Reestablishing the grade in this area would require very costly remedial measures to preserve the water quality in the creek, or alternately would require relocating the road outside the creek channel.

ii) Alignment 2

Alignment 2 would consist of rehabilitating the abandoned railroad and previously constructed truck road to a location near the first major crossing of Drinkwater Creek. Instead of crossing the creek, approximately 1300m of new road would be constructed in talus/colluvium and bedrock on the north side of the creek, and would rejoin the old road near the second crossing. New road construction would cost an estimated $127,000 to $196,000, which is considerably less than the estimated cost of $365,000 to construct two large span bridges across Drinkwater Creek and rehabilitate the old road in environmentally sensitive and geotechnically hazardous areas.

4.1.5 Costs

A preliminary order of magnitude cost comparison of the two alignments up the Drinkwater Valley is included in Table I. The unit costs in this table are based on our general experience with logging road construction in similar terrain, and a brief review of the costs of road and bridge construction with equipment suppliers and road construction personnel. Wherever possible, temporary construction has been assumed using easily removed or reclaimed structures. In this regard, use of surplus railway flatcars for the crossing of major creeks would be relatively inexpensive, and less environmentally sensitive, than use of galvanized steel culverts. Road grading and surfacing would be accomplished using available materials excavated along the road, or from borrow pits outside the park. It should be recognized that relatively high unit costs have been used for new road construction, because it is likely that side casting of excavated material, which has been a common practice in logging road construction in British Columbia, would not be allowed for environmental and visual impact reasons. The Ministry of Parks have indicated that all excavated material would have to be endhauled to reduce the environmental and visual impact of the road.

The preliminary order of magnitude cost estimates included in Table I indicate that construction of an access road up the Drinkwater Valley would cost approximately $600,000 to $800,000 for the most feasible option, with alternate options costing up to $1,000,000.

4.1.6 Maintenance

Maintenance of the access road is likely to be extremely costly, particularly if the road is operated during the winter season. It is likely that a grader, a loader and/or other suitable snow removal equipment will be required on a full time basis for winter maintenance. A snow control program will also likely be required, which would consist of snow surveys, helicopter reconnaissance and removal of avalanches using explosive charges. It may have been difficult to obtain permits for use of explosive charges on surface outside the immediate mine area in 1988, even for snow control.

It would have to be anticipated that regular maintenance of the road and bridges would be required, even during summer-only operation. There is evidence of severe washouts, floods and debris flows/debris torrents on Drinkwater Creek and many of its tributaries in the past. Experienoe has shown that such events can occur at almost any time of the year in the coastal regions of British Columbia. Design of remedial measures to totally eliminate interruption of the road as a result of these events would be impractical. Allowance should be made for the rebuilding of road grade and the replacement of bridges, which may be lost as a result of such events throughout the year. The proponent would also have to consider the cost of recovering debris from bridges which were destroyed in such events, as well as the cost of repairing any environmental damage as a result of washouts in the creeks. While it is recognized that washouts and floods are natural events, it would be necessary to assess what environmental liability the operator would assume, in the event that such events occurred while the project is in operation.

Establishing a cost for maintenance of the access road is difficult, until experience is gained with the number and frequency of events at the various hazard sites identified along the road. It would be prudent for the operator to have sufficient equipment available on a standby basis to remove debris, snow, etc., and to reestablish the road grade and rebuild bridges. It is likely that a grader, a loader, a haul truck, a dozer (D7 or larger), and possibly a track mounted backhoe, would be required on a full time standby basis. A number of spare bridges and a trailer to transport them to specific bridge sites would also be recommended.

It is likely that the road maintenance equipment could be operated by the mine crew as required. However, if the mine crew were required to operate the road maintenance equipment, production would be lost. It is likely that production losses could be minimized by having at least one, and preferably two, full time equipment operators to conduct routine and emergency road maintenance.

4.1.7 Reclamation

Reclamation of the road as proposed could be accomplished by removal of bridges and landscaping/contouring of the road surface to control erosion. Alternatively, the Ministry of Parks may require the road only to be reclaimed to trail status, leaving the bridges in place for recreational use. In this case, the costs of reclamation are expected to be minimal.

4.2 UNDERGROUND MINE

As advised by Mr. Bill Norquist of Wright Engineers Limited, we have assumed that the proposed underground mine could consist of two stopes approximately 1.5 to 2m wide, 60m long and 60m high, which would be contained within a fault/shear zone which strikes approximately N40E, and dips approximately 70° to the northwest. Heard et al (1989) have indicated that ore extraction would be conducted using a shrinkage stoping method. The reserve area outlined by Barr (1990) would break through to surface if mined, and would form a permanent visible opening within the avalanche/rockfall chute, and on the top of the ridge between about 1277m and 1375m (4190 and 4512 ft) elevation unless it is filled after excavation (see Fig. 3). The implications of this opening will have to be addressed in terms of the operational stability of stopes which break through to surface, the visual impact from Della Falls, possible drainage of surface water into the underground workings during operation and after abandonment, as well as the long term slope stability and safety of this slope after abandonment.

4.2.1 Portal Selection and Development

The steep valley walls on the north side of Drinkwater Creek and the documented geotechnical hazards preclude the construction of an access road to the existing workings. Hence, access to the workings would be established from an adit from the lower valley walls. The portal of the adit would be in a protected location about 60m above the valley bottom, south of the active avalanche/rockfall chute developed along the Sherwood Shear zone. No specific problems are anticipated to establish the portal, provided that a protected site in sound bedrock is selected. A pad and staging area will be required to establish access to the portal, and to provide space for power generators, ventilation equipment and service buildings. This pad could be constructed using development waste rock or excess rock endhauled from the access road construction. The pad must be constructed on stable ground in such a manner that the stability of the natural slopes is not affected.

A study will be required to determine the optimum location of the portal and pad from a geotechnical and environmental/visual impact perspective.

4.2.2 Rock Mechanics Considerations for Development Openings

We understand that an approximately 600m long adit would be driven from the portal to the main extraction area. We further understand that a series of raises would be developed in ore and waste to enable removal of ore, to provide access to the underground workings, and to provide ventilation. We understand that existing development openings are reported to have extensive timber support in some areas, most probably within fault/shear zones.

In general, development openings should be laid out to avoid zones of weaker rock and faults. Openings in sound rock would generally be expected to require minimal support or limited support using rock bolts and wire mesh, as is standard practice. If unstable fault zones are encountered, additional support using shotcrete (sprayed concrete) and timber sets or steel arches could be utilized. During the development phase, it would be important to determine the location and assess the significance of any extensive zones of weaker rock, and lay out the proposed development openings to avoid these areas, if possible.

4.2.3 Rock Mechanics Considerations for Stopes

The size of the proposed stopes and the proposed extraction method will result in relatively narrow openings, which will be supported by the broken rock during the operation. It is anticipated that any local instability encountered during mining could be controlled by the installation of rock bolts and mesh, or incorporating small "post" pillars in the stopes where required. Both these measures are standard practice for ground control and safety in stopes of the size and type proposed by Wright Engineers Ltd. If extensive shear zones and weak ground are encountered in stopes, heavier support and/or a modification of the mining method may be required for ground control.

Where a stope breaks through to surface, it would be necessary to conduct additional studies to insure adequate stability to the backs and stope side walls, and also to enable rock to be drawn from the stope. In this regard, it may be necessary to leave a pillar of undisturbed rock between the stope and the valley side wall to confine the broken rock in the stope, to maintain a working platform for drilling and blasting within the stope, to maintain stability of the natural rock slope within the avalanche/rockfall chute, and to prevent buildups of snow and debris within the stope. The Ministry of Parks may require that a pillar be left to reduce the visual impact from Della Falls, and The Ministry of the Environment may require the pillar to be left to preserve the natural drainage course and prevent changes to the existing runoff from the ridge. The Ministry of Mines may require the pillar to be maintained for operational safety and for safety of future recreational users.

While a mining method might be devised to remove the pillar, the cost of removal may be significant, and would likely require reasonably high grades of ore to justify the additional cost of recovery. Any mining scheme would have to address the fact that a portion of a proposed mining block may have to be left as a permanent pillar.

At the end of the operation, the rock will be drawn from the stopes leaving an open and unsupported stope. Therefore, while the stopes are likely to remain stable during the operation, some collapse of the abandoned workings could occur after mining, particularly if the stopes are mined through to surface, as indicated by mining the reserves indicated by Heard et al (1989) and Barr (1990).

If subsequent engineering studies were to indicate that stopes could collapse in the long term after the stopes had been emptied, it could be necessary to backfill the stopes with broken rock or coarse sand for long term stability. If uncontrolled drainage of water into the stopes were considered to be unacceptable by the various permitting agencies, it would be necessary to prevent water inflow by sealing the top levels of the stope with a sand/cement mixture, or other low permeability barrier system. The cost of such a system, if required, would have to be addressed during the detailed design phase of the project.

4.2.4 Waste Rock Disposal

We understand that approximately 42,000 tonnes of waste would be generated during the development and production phases of the mine. Assuming a material density of about 2.8 tonnes/m^3, approximately 15,000m^3 of waste rock would be developed, of which about 4000m^3 would be generated during the development phase. Any waste rock which is non-toxic and non-acid generating could be used in construction of the pads at the portal, or placed on the access road for road grading and construction of pullouts. In this regard, all of the non-toxic waste rock could likely be disposed of without significant impact.

Toxic or acid generating waste rock would require specialized handling and treatment. Various methods for handling this material could consist of the following:

i) Stockpile the toxic waste near the portal and deposit it in an abandoned underground stope after closure.

ii) Transport the toxic waste to an approved dump site;

iii) Place the toxic waste in an engineered containment.

iv) Incorporate the waste into the tailings disposal scheme, which will also be required to control generation of acid or contaminants to the groundwater.

An engineering study will be required to determine the amount of toxic and/or acid generating waste rock, and to determine the optimum method of handling and treatment of the waste rock. Provision must be made to conduct the required engineering and environmental assessments relating to the identification and handling of toxic or acid generating waste rock. This will involve an extensive program of testing representative waste rock samples from the project.

4.2.5 Water Supply

Water supply requirements for the underground mine are estimated to be about 1.1 to 1.4 L/s (17 to 22 gpm). Review of available climate data indicates that the average flow from Love Lake is expected to be approximately 70 L/s. Hence, the required mine water supply of 1.4 L/s could easily be obtained from Love Lake or from Drinkwater Creek with minimal reduction in the flows. Removal of water from Love Lake, even during a period of prolonged drought (three months), would only be expected to draw the lake level down by several centimetres.

In view of the probable difficulty in maintaining a water supply system from Love Lake during severe winter conditions, it may be more practical to establish a shallow infiltration well adjacent to Drinkwater Creek upstream of Love Creek, and to pump the required flows along the access road to the portal. This supply would be used to supplement any shortfall in the mine water requirements which could not be recovered from the sedimentation facility, as described in Section 4.3.

The costs for the mine water supply established in this manner are not expected to exceed $10,000.

4.3 SEDIMENTATION FACILITY

Water flow from the underground workings will consist of waste water from the operation, plus the natural groundwater flow to the stopes and development openings. If stopes are allowed to break through to surface, considerable additional flow could occur to the underground workings. Regulations set by the Ministry of Energy Mines and Petroleum Resources, in consultation with the Waste Management Branch of the Ministry of the Environment, require that the concentration of suspended sediment and other objectionable materials in an effluent stream be reduced to an acceptable level, prior to release to the environment. Effluent discharge permits for mining projects normally restrict the maximum concentration of total suspended solids (TSS) to within the range of 25 to 75 mg/litre. Removal of metals, organic contaminants and neutralization of any acid mine waters will also be required.

The accepted practice for treatment of mine waters is to construct a sedimentation facility to restrict the TSS, and to provide a location for any required treatment to reduce the concentration of other contaminants, and neutralize any acid. We understand that the mine waters are not expected to contain unacceptable concentration of contaminants, or to be acidic. Hence, the main purpose of the sedimentation facility will be to restrict the concentration of total suspended solids. It should be noted that the proponent will have to conduct sufficient studies to confirm these assumptions.

The sizing and design of the sedimentation facility will depend on the flows through the facility, the TSS accepted by the permit and the settling velocity of the suspended material. A range of possible flows and settling parameters is assessed in the following as a guide to assessing the design concept, and preparing a preliminary cost estimate for the facility.

4.3.1 Estimation of Flows

Possible flows from the workings are estimated as follows:

Mine operating water	1.4 L/s
Groundwater flow to the mine	3.5 L/s
Total groundwater flow	4.9 L/s

In addition, if the pillars are mined and the stopes are allowed to break through to surface, an additional 32 L/s of direct inflow to the workings could be anticipated. The required size of the sedimentation facility for both these alternatives is considered in the following.

4.3.2 Estimated Size of the Sedimentation Facility

The Ministry of the Environment publication entitled "Guidelines for the Design and Operation of Settling Ponds used for Sediment Control in Mining Operations" suggests that, where the particle size distribution of the inflow sediments is not known, the required area of the settling pond may be calculated using the following formula:

$$A = 1.2\ Q/Vsc$$

where A = settling pond area, m^2

Q = peak outflow rate, m^3/s

Vsc = critical settling velocity

$= 2.0 \times 10^{-5}$ to 5.0×10^{-5} m/s

For the design outflow rate for the sedimentation facility of 4.9 L/s, the surface area of the pond calculated from this equation is in the range 117 to 294 m^2. A pond with approximate dimensions of 8m x 40m would be recommended for this flow. The required size of the pond would increase to 900 to 2200m^2 if the inflow to the pond were to include direct runoff into the open stopes as a result of the removal of the pillars. In this case, the recommended pond dimensions would be up to 20m x 110m.

4.3.3 Preliminary Design Considerations

The conventional approach for a sedimentation pond would be to construct a 3m deep impoundment to accommodate 1m of sediment storage, 1m of settling and 1m of freeboard. Construction would normally be undertaken using cut and fill techniques on relatively flat ground to balance the amount of excavation and dyke construction. A site which has minimal geotechnical hazards would be required to avoid damage to the structure during avalanches, floods, debris flows, etc.

In terms of the Sherwood Project, the closest location to the portal where there is sufficient flat ground to construct a sedimentation pond of the required size is at the abandoned sawmill site near Della Creek. This site is currently occupied by a campsite used by hikers in the park. The geotechnical feasibility of this site must be confirmed by appropriate engineering studies. A more important consideration may be to verify the overall acceptability of the site from discussions with the various permitting agencies. It would also be necessary to determine what measures would be required to relocate the existing campsite and avoid disruption of the recreational use of this area.

In terms of geotechnics, this site may be underlain by relatively coarse grained soils and large boulders from the nearby talus deposits. Hence, there is a strong possibility that a cut and fill type excavation may not be feasible or practical. In such a case, it would be necessary to construct the dyke to the full height, using granular fill which was transported to the site from a borrow pit outside the park boundary.

The conceptual design of the sedimentation pond as shown in Fig. 4 includes a width to length ratio of approximately 1:5. The design calls for a compacted fill embankment with an emergency spillway and overflow. It will also be necessary to occasionally remove accumulated sediment from the pond, and transport it to an approved disposal location.

4.3.4 Preliminary Cost Estimate

A preliminary estimate of quantities and costs has been prepared for the case assuming that the inflow to the pond is based on only groundwater flow and operational flows (4.9 L/s), and for the case that the inflow is based on groundwater flow, operational flows and surface runoff into the stopes which are broken through to surface (37.0 L/s). We have further assumed that the pond may be constructed using cut and fill techniques as shown in Fig. 4, or that the embankments are constructed entirely above the existing ground surface using borrow material imported from outside the park.

The preliminary cost estimate included in Table II indicates that, if the cut and fill excavation were feasible, the costs will be substantially reduced. It is to be noted that the costs for the sedimentation pond do not include the cost of environmental studies, relocation of the existing park facilities, or reclamation at the end of operation.

4.3.5 Alternate Sedimentation Control Concept

After completion of the necessary engineering and environmental assessments and permitting enquiries, it may not be possible to establish the sedimentation facility at the proposed location, for environmental or aesthetic reasons. If this were the case, it could be possible to place the sedimentation facility in an underground cavern excavated in sound bedrock within the underground workings. The cost of the facility would be determined by the volume of rock to be excavated to develop the pond. Excavation costs could exceed $100,000 for the lower water inflow (4.9

L/s). If the higher flows arising from the flow of surface water into the stopes were to be handled, excavation of an appropriate size facility may be impractical. Alternatively, a flocculating station may have to be incorporated into the design to reduce the required excavation size. While use of a flocculant system would be feasible, the design of the system and costs would have to be included in the cost of the facility.

As the cost of an underground sedimentation facility exceeds the cost of a surface pond, this design would only be adopted if the surface location could not be permitted.

4.4 PLANT SITE AND CAMP

4.4.1 Site Selection

The plant site and camp will require an area of approximately 20,000m^2 (i.e. an area with approximate plan dimensions of 100m x 200m). Ideally, the tailings pond and other related facilities should be located nearby. Site development would be most economic on relatively flat ground underlain by compact to dense soils or bedrock. Possible plant sites in the upper Drinkwater Valley were considered but have been eliminated due to limited space, steep slopes and presence of severe geotechnical hazards. Two suitable areas of relatively gently sloping or flat ground well above the maximum flood level of Drinkwater Creek were identified along the abandoned railroad outside the boundaries of Strathcona Park, as shown in Fig. 2. These sites are considered feasible and practical because natural hazards can be controlled, site development costs would be optimized, and access could be more easily maintained for year round operation of the plant. It appears that deposits of suitable borrow material could likely be obtained close to these sites, and a groundwater supply for the mill could also be developed for the plant and camp, using shallow wells in the floodplain of Drinkwater Creek.

4.4.2 Foundation Considerations

A plant site located on dense native soils, such as the alluvial/colluvial deposits identified at either of the preferred locations shown in Fig. 2, would be geotechnically feasible. Specific subsurface investigations would be required to assess the distribution and nature of soils below the site, and to determine the geotechnical parameters for design of footings and drainage. Such a study would require a one to two day site investigation for each plant site investigated using a large track mounted backhoe or a soils drilling machine.

Foundation preparation costs are likely to be similar to any conventional structure on granular soils. Light structures and buildings would likely be placed on spread footings below the depth of frost penetration. Any heavy or vibrating structures may require a deeper foundation system, depending on subsurface conditions. Costs of site clearing, grubbing, stripping of topsoil and grading are expected to be of the order of $4/m^2, i.e. $60,000 to $80,000 for the entire plant site. Installation of deep foundation systems, such as piles under heavy equipment, could be expected to cost $50 to $80/m^2, depending on the subsurface conditions.

4.4.3 Geotechnical Hazards and Remedial Measures

The main geotechnical concerns for the preferred plant site would be the potential for flooding and or debris flows/torrents, which have been documented on some creeks in the Drinkwater Valley. A study of geotechnical hazards and the remedial measures required to mitigate the hazards would be required during the detailed design phase for the plant site area. Remedial measures for these hazards will likely consist of placing the plant site in a protected area, well above the probable maximum flood, and installing training berms and ditches to divert any runoff water or debris flows away from the plant site area. The cost of the required remedial measures is expected to be less than $100,000, including engineering services.

4.4.4 Water Supply

Water supply requirements are expected to be approximately .26 L/s (3.5 gpm) for the camp, and 20 L/s (260 gpm) for the mill make up water. The most reliable and practical source for this flow would be from a well installed in a strategic location in the gravels on the floodplain of Drinkwater Creek. A groundwater supply would have no impact on the water quality or fisheries resource in the creek, and would not require any treatment. Normal practice would be to install two wells to depths of about 30m each. One well would be used for the required supply, while the other well would be used as a backup in the event of pump failure, during servicing, or for fire protection. The cost of drilling and installing two wells as specified is estimated to be about $30,000, not including the cost of mobilizing the drill rig and materials to the site, or the installation of the pumps, power supply and pipe lines.

4.4.5 Sewage Disposal

The most practical method of sewage disposal for the camp would be to install a septic tank and drainfield for ground disposal of domestic waste water. The granular soils noted in the floodplain and colluvial sediments at the proposed plant site are considered suitable for a drainfield, provided that appropriate environmental concerns are satisfied and waste disposal permits are obtained. Costs of installation of a septic tank and drainfield are expected to be of the order of $30,000 to $35,000 in 1988.

4.5 TAILINGS DISPOSAL

We understand that approximately 50,000 tonnes of tailings would be generated from the milling process. Based on a dry density of 1.5 tonnes/m^3, a permanent impoundment facility sized to contain approximately 33,000m^3 of tailings would be required. We have assumed that the tailings may be acid generating, and that the impoundment facility must be designed to protect the fisheries resource by inhibiting acid generation, and restricting seepage losses following abandonment.

We have further assumed that clarified supernatant is of adequate quality that it may be routinely discharged from the impoundment to the environment, or recycled to the mill, and that retention of excess supernatant and any precipitation that occurs directly on the impoundment is not required. If the supernatant is not of adequate quality for direct discharge, a supernatant treatment facility may be required. Alternatively, the tailings facility may have to be operated as a closed system, which could require a substantially larger impoundment. Depending on climatic conditions, effluent discharge restrictions, available dilution and other factors, some combination of supernatant retention and treatment may be required. Facility requirements and costs for constructing or operating the tailings disposal facility as a closed or restricted discharge system, or for treatment of effluent, could be substantial and have not been addressed in this assessment.

Based on the above assumptions, a conventional tailings impoundment, consisting of a basin formed partly by excavation and partly by construction of an earthfill retention dyke, and incorporating a liner, as required to control seepage, is considered the most appropriate approach to tailings disposal for the project. Following completion of the project, the impoundment would be permanently flooded to inhibit oxidation of the tailings and generation of acidic leachate. Other methods of tailings disposal, such as direct disposal into Great Central Lake, dyke construction using cycloned tailings, or subaerial deposition were also considered; however, these were rejected based on environmental or permitting concerns, or higher capital or operating costs than a conventional impoundment.

A conceptual design of a conventional tailings impoundment scheme is presented in Fig. 5, and a preliminary estimate of quantities and costs is given in Table III.

4.5.1 Geotechnical Concerns

The main geotechnical concerns for the proposed conceptual tailings disposal facility are :

i) The tailings site must be suitably located in an area or areas with minimal geotechnical hazards and potential environmental impacts.

ii) Access must be such that the facility may be constructed and operated at reasonable cost.

iii) A site with limited direct catchment and which does not require diversion of any perennial streams would be preferred. However, collection and/or diversion of surface flows to maintain flooded conditions upon abandonment may be required, and a nearby perennial stream would be an asset in this regard.

iv) A site on gently sloping ground underlain by impervious soils, such as glacial till, would be preferred. In the alternative, a site underlain by granular soils above the water table would be suitable, provided the impoundment is lined to restrict seepage losses.

v) A site where soils are of sufficient thickness and quality that excavation of bedrock is not required, and where soils available from the excavation are suitable for use as general fill for dyke construction (e.g. till, sand and gravel, some types of colluvium) is preferred.

vi) A locally available source of borrow materials for construction of liners (e.g. low permeability morainal or clay deposits, sand for construction of a bentonite admix liner), filters, etc. would be an asset.

vii) A site located close to the mill facility with enough space to accommodate expansion, if required.

viii) The site should be removed from sensitive environments, such as active floodplains, marsh lands, or fish bearing streams, to minimize overall environmental and aesthetic impacts.

4.5.2 Site Selection

Suitable tailings sites on granular soils removed from the active floodplain of Drinkwater Creek have been identified adjacent to the two plant sites discussed above, and shown in Fig. 2. Depending on the exact location and layout of the facilities, some remedial works may be required for protection of the facility from avalanches and debris flows and flooding in tributary creeks. It must be appreciated that the suitability of these sites requires confirmation by additional geotechnical investigations, such as detailed reconnaissance and mapping, drilling and/or excavation of test pits and detailed assessments of all of the geotechnical and environmental aspects discussed above.

4.5.3 Design Concept

As indicated above and illustrated on Fig. 5, the selected design concept involves construction of a conventional impoundment formed partly by excavation and partly by construction of a fill dyke. The impoundment would be constructed on gently sloping terrain (i.e. 3 to 5°). The dyke portion would be constructed from soils obtained from the excavation, and cut and fill quantities would balanced to minimize excess spoil or excess fill requirements.

The shape and depth of the impoundment (i.e. 110 x 110 x 6m) was selected to achieve a reasonable balance between dyke height, required impoundment area, liner requirements, and cut and fill balance, and incorporates 1m of freeboard and 1m for flooding on abandonment. Alternative shapes and depths may also be feasible, and the overall concept lends itself well to accommodating some variation in site topography, as well as possible expansion, either by increasing dyke height, or by constructing adjacent cells.

Excavated slopes would be no steeper than 2:1 (horizontal:vertical). Dykes are designed as full water retaining structures with relatively flat (i.e. 2.5 horizontal to 1 vertical) slopes and 4.5m wide crests. Dykes would be constructed using compacted, free-draining sand and gravel from the excavation. Given the flat slopes, limited height and granular construction materials, both static and dynamic (i.e. seismic) stability are considered adequate. However, a more thorough evaluation of seismic hazards would be required for final design.

As both of the sites are located on pervious sand and gravel, the design concept includes a full liner for the impoundment. If suitable low permeability natural materials are available at or close to the site, these would be used to construct the liner. Alternatively, a bentonite/sand admix liner could be constructed with imported materials. As this is a permanent facility, use of geotextiles, synthetics or HDPE as liners was not considered, as these would likely not be accepted by regulatory authorities.

4.5.4 Borrow and Materials Requirements

It will be necessary to confirm that the materials excavated for the impoundment will be suitable for embankment construction. In addition, it will be necessary to identify sources of clean granular soils for rip rap armouring, filters or sand for the bentonite admix liner. Sources of low permeability soils such as glacial till or other fine grained soil would also be required for construction of a natural liner.

4.5.5 Construction Aspects

It is envisaged that the bulk of the facility could be constructed using basic earth moving equipment (e.g. bulldozers, backhoes, self-propelled compactors). If material must be hauled to the site from remote borrow sources, or barged to the site, dump trucks and loaders would also be required. Based on the volumes of excavation and filling required, and nature of the materials, it should be possible to construct the basic facility in a relatively short period (i.e. about one month).

5. REFERENCES

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FIGURES

JOB NUMBER 91-419

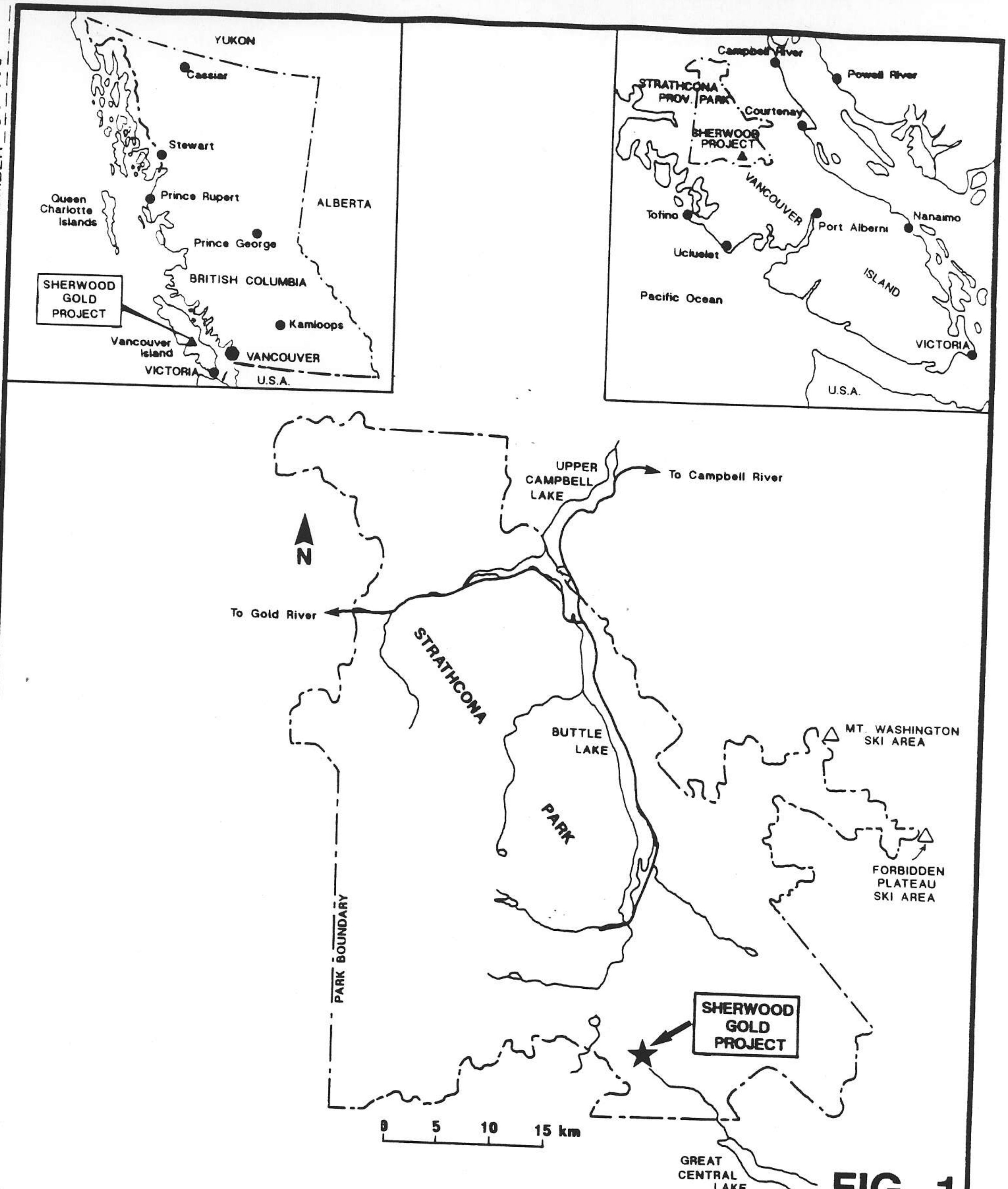


FIG. 1

WRIGHT ENGINEERS LIMITED
MINISTRY OF THE ATTORNEY GENERAL
SHERWOOD PROJECT

PITEAU & ASSOCIATES
GEOTECHNICAL CONSULTANTS
VANCOUVER CALGARY

SITE LOCATON

BY:	DATE:
/BL	AUG.91
APPROVED:	DWG:

JOB NUMBER 91-419

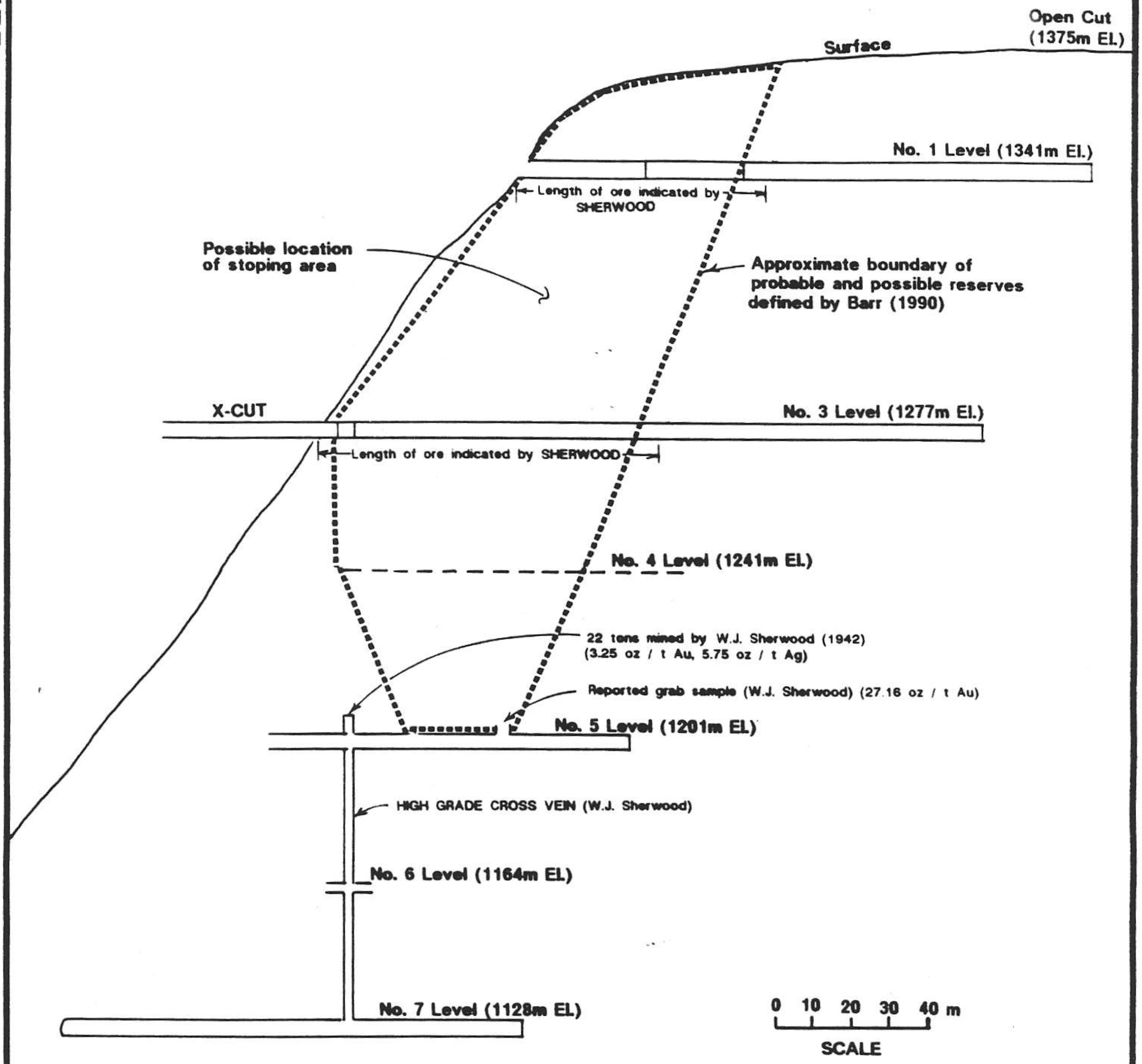


FIG. 3

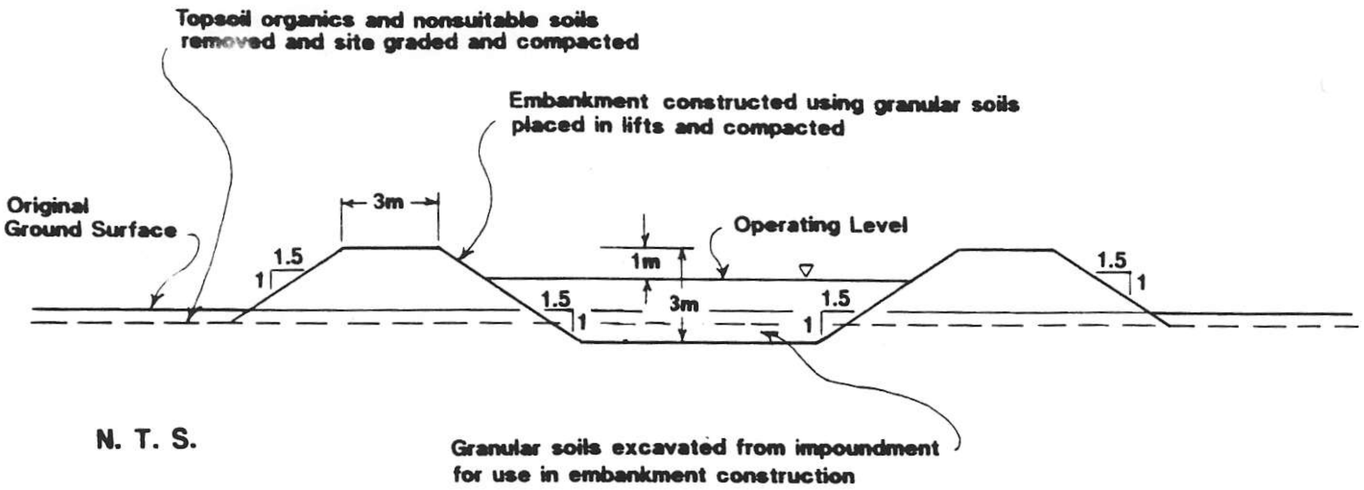
WRIGHT ENGINEERS LIMITED
MINISTRY OF THE ATTORNEY GENERAL
SHERWOOD PROJECT

PITEAU ASSOCIATES
GEOTECHNICAL CONSULTANTS
VANCOUVER CALGARY

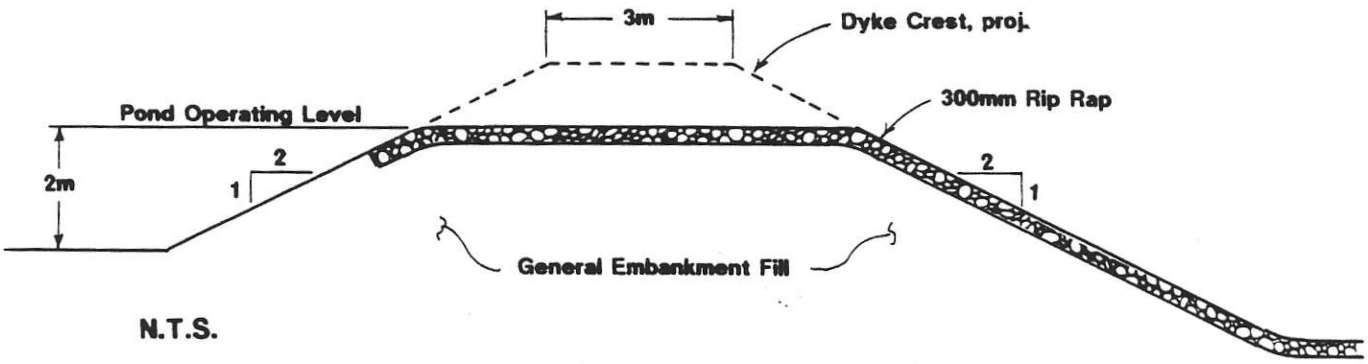
LONGITUDINAL SECTION SHOWING POSSIBLE LOCATION OF STOPING BLOCKS

BY:	DATE:
DCM/BL	AUG.91
APPROVED:	DWG:

JOB NUMBER 91-419



EXCAVATION AND IMPOUNDMENT CONSTRUCTION DETAILS



POND OVERFLOW AND EMERGENCY SPILLWAY DETAILS

FIG. 4

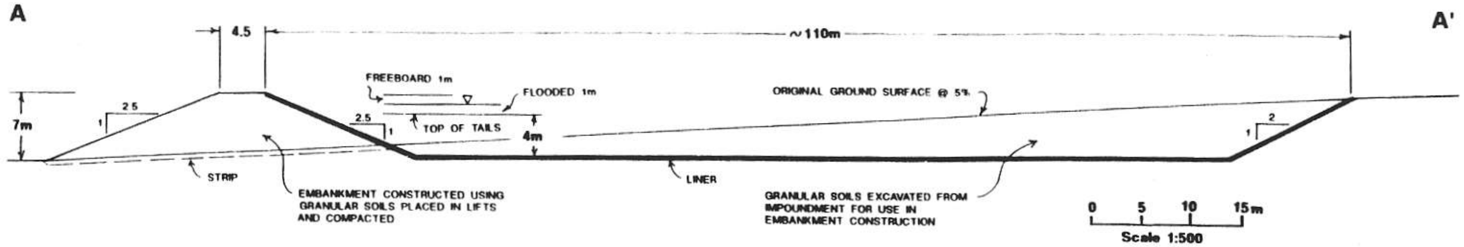
WRIGHT ENGINEERS LIMITED
MINISTRY OF THE ATTORNEY GENERAL
SHERWOOD PROJECT

PITEAU & ASSOCIATES
GEOTECHNICAL CONSULTANTS
VANCOUVER CALGARY

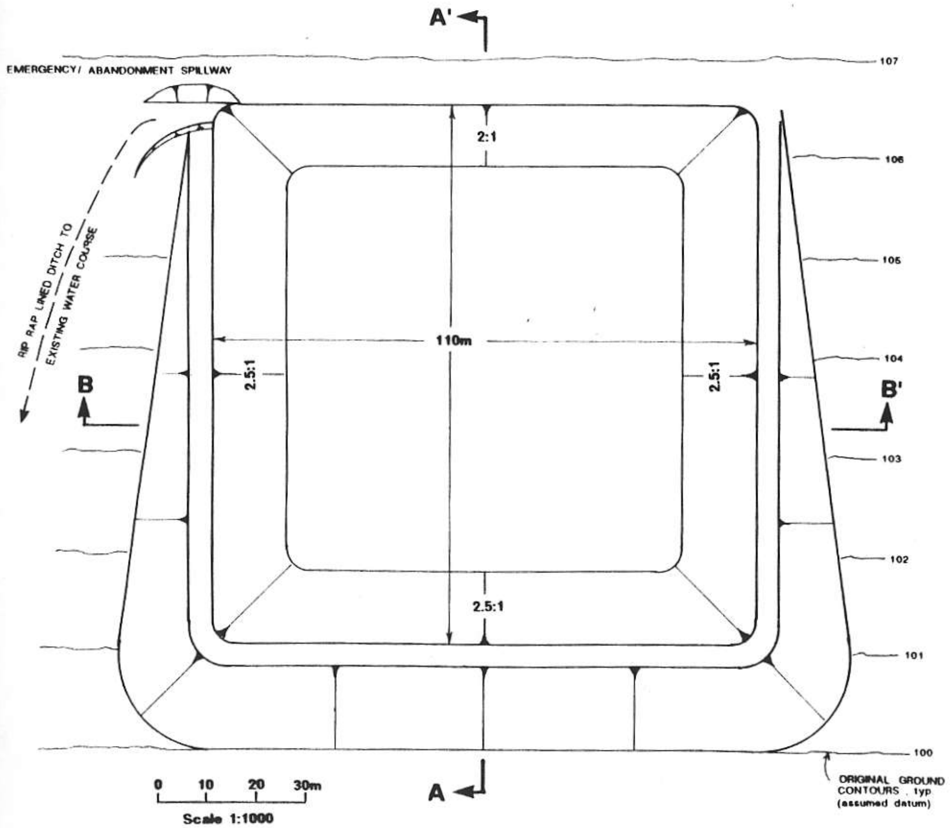
CONCEPTUAL LAYOUT AND DESIGN OF SEDIMENTATION FACILITY

BY:	DATE
DCM/BL	AUG.91
APPROVED:	DWG:

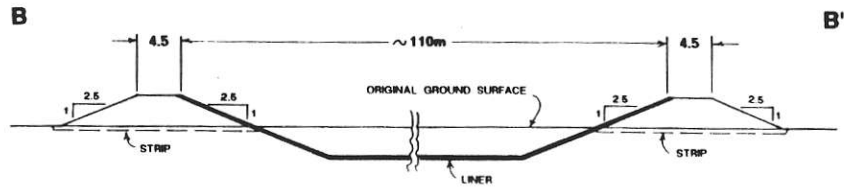
JOB NUMBER 91-419



SECTION A-A'



PLAN VIEW



SECTION B-B'

0 5 10 15m
Scale 1:500

FIG. 5

WRIGHT ENGINEERS LIMITED MINISTRY OF THE ATTORNEY GENERAL SHERWOOD PROJECT	PITEAU ASSOCIATES GEOTECHNICAL CONSULTANTS VANCOUVER CALGARY	
CONCEPTUAL LAYOUT AND DESIGN OF TAILINGS DISPOSAL FACILITY	BY: PMH/BL APPROVED: DCM.	DATE: AUG.91 DWG:

TABLES

TABLE I

PRELIMINARY ESTIMATES OF QUANTITIES AND COSTS FOR ALTERNATE ACCESS ROAD ALIGNMENTS ALONG DRINKWATER CREEK FROM GREAT CENTRAL LAKE TO THE PORTAL

ITEM	ESTIMATED UNIT COST	ALIGNMENT 1 QUANTITY	ALIGNMENT 1 COST	ALIGNMENT 2 QUANTITY	ALIGNMENT 2 COST
Rehabilitation of Abandoned Railroad	15,000/km	8.82 km	132,300	8.82 km	132,300
Rehabilitation of Abandoned Truck Road - North Side of Creek	40,000/km	2.74 km	109,600	2.74 km	109,600
Rehabilitation of Abandoned Truck Road - South Side of Creek	80,000/km	1.30 km	104,000	-	-
Construction of New Road in Talus/Colluvium	80,000 - 120,000/km	0.72 km	57,600 - 86,400	1.52 km	121,600 - 182,400
Construction of New Road in Bedrock	120,000 - 200,000/km	0.32 km	38,400 - 64,000	0.82 km	98,400 164,000
Bridge Crossing of Drinkwater Creek	125,000	2	250,000	-	-
Bridge/Culvert Crossing of Tributary Creeks	5000 - 10,000	11	55,000 - 110,000	10	50,000 - 100,000
SUBTOTAL		746,900 - 856,300		511,900 - 688,300	
Contingency (15%)		112,000 - 128,400		76,800 - 103,200	
ESTIMATED TOTAL COST		858,900 - 984,700		588,700 - 791,500	

Notes:

1. Alignment 1 follows the abandoned railroad and truck road and involves two major crossings of Drinkwater Creek.
2. Alignment 2 follows the abandoned railroad and truck road with new road construction to avoid major crossings of Drinkwater Creek and geotechnical hazards on the south side of the creek.
3. Costs for maintenance and repair of roads and bridges during operation and reclamation after closure are not included.

TABLE II
PRELIMINARY ESTIMATES OF QUANTITIES AND COSTS FOR CONCEPTUAL SEDIMENTATION FACILITY

ITEM	UNITS	UNIT COST	8m x 40m POND QUANTITY	8m x 40m POND COST	10m x 22m POND QUANTITY	10m x 22m POND COST
Site investigation and detailed engineering design	LS	-	-	5000	-	5000
Mobilization/demobilization	LS	-	-	5000	-	5000
Clearing and grubbing	m^2	1	2500	2500	6000	6000
Stripping dyke foundations	m^3	2	1250	2500	3200	6400
Grading and foundation prep	m^2	1	1250	1250	3200	3200
Excavation and placement of fill for balanced cut and fill construction	m^3	2	600	1200	2500	5000
Excavation and transportation of fill from borrow pit outside park and placement and compaction for dyke construction	m^3	10	2100	21,000	6000	60,000
Emergency spillway and rip rapped outlet channel	LS	-	-	2500	-	2500
Survey layout and control	LS	-	-	2500	-	3000
Engineering supervision	LS	-	-	2500	-	3000
ESTIMATED TOTAL COST (balanced cut and fill)			24,950		39,100	
ESTIMATED TOTAL COST (embankment material imported)			44,750		94,250	

Notes:

1. Assumes the bulk of the construction would be carried out by a D8 dozer, Cat 235 Backhoe and self-propelled compactor. A loader and haul truck would be required for imported material.
2. Assumes construction would require approximately 1 month to complete.
3. Does not include costs associated with tendering or construction management.
4. No contingencies have been included.
5. Costs of relocation of existing recreational facilities and reclamation of pond site after closure are not included.

TABLE III
PRELIMINARY ESTIMATES OF QUANTITIES AND COSTS FOR CONCEPTUAL TAILINGS DISPOSAL FACILITY

ITEM	UNITS	QUANTITY	UNIT COST	ITEM COST
Site investigation and detailed engineering design	LS	-	-	25,000 - 35,000
Mobilization/demobilization	LS	-	-	10,000 - 15,000
Clearing and grubbing	m^2	17500	1	17,500
Stripping dyke foundations	m^3	4300	2	8,600
Grading and foundation prep	m^2	8600	1	8,600
Mass excavation, liner excavation, placement and compaction of general embankment fill, disposal of excess	m^3	35000	1-2	35,000 - 70,000
Provision and placement of bentonite/sand admix liner	m^2	13000	7-10	91,000 - 130,000
Provision and placement of natural liner (materials available locally at no cost)	m^2	13000	2-4	26,000 - 52,000
Emergency/abandonment spillway and rip rapped outlet channel	LS	-	-	5,000
Survey layout and control	LS	-	-	5,000
Engineering supervision	LS	-	-	10,000 - 15,000
ESTIMATED TOTAL COST (Bentonite/Sand liner)				215,700 - 309,700
ESTIMATED TOTAL COST (Natural liner)				150,700 - 231,700

Notes:

1. Assumes tailings is not a closed system and that supernatant is discharged to the receiving environment. Costs for a closed system or a treatment plant, if required, are not included in this estimate.
2. Assumes the bulk of the construction would be carried out by a D8 dozer, Cat 235 Backhoe and self-propelled compactor.
3. Assumes construction would require approximately 1 month to complete.
4. Does not include costs associated with tendering or construction management.
5. No contingencies have been included.

TABLE IV

SUMMARY OF GEOTECHNICAL CONCERNS AND PRELIMINARY ORDER OF MAGNITUDE COST ESTIMATES FOR GEOTECHNICAL COMPONENTS

COMPONENT	ASPECT	GEOTECHNICAL CONCERN	COMMENTS/CONCLUSIONS	OF MAGNITUDE COSTS
Access road	Location/ construction	-excavation in steep bedrock and talus slopes -natural hazards such as debris flows, avalanches, floods, rockfalls, landslides -crossing of Drinkwater Creek and tributaries	-construction costs higher than normal logging road -construction in creek not permitted -cannot avoid all hazards -must anticipate periodic interruptions of access	590,000 to 985,000
	Operation	-natural hazards such as debris flows, avalanches, floods, rockfalls, landslides	-high maintenance costs -maintenance crew and equipment required -winter operation may not be practical	
Underground mine	Portal location	-protected location and road access required	-impractical to construct road to level of existing workings -suitable site has been identified near valley bottom	Mining costs prepared by Wright Engineers Limited
	Development openings	-ground control	-openings should be located to avoid bad ground -standard ground control measures anticipated including rock bolts, mesh and shotcrete	
	Stopes	-ground control during operation	-sheared rock may form backs and walls -standard ground control measures anticipated includin rockbolts, mesh and shotcrete -may require timber or post pillars in bad ground	
		-subsidence and breakthrough of stopes at surface -increased groundwater inflow with breakthrough	-may require crown pillar to provide long term stability and prevent water inflow -removal of crown pillar may not be economic or may have adverse visual effects	
	Waste rock	-disposal site in stable ground -acid mine drainage	-approved disposal may require rehandling or transport of rock to approved location -remedial measures may be required to control acid mine drainage	
	Water supply	-access to site and system maintenance -continuous supply	-Love Lake source may be impractical to maintain during winter operation -shallow infiltration well/sump in valley bottom may be most practical	10,000
Sedimentation facility	Location	-flat ground without natural hazards	-relocation of recreational campsite required	45,000 to 95,000
	Size	-must provide adequate retention time	-pond size will increase substantially if stopes break through to surface	
	Construction	-method, sources of borrow	-cut and fill may not be feasible depending on soils at site -may have to import borrow	
	Operation/ reclamation	-possible treatment requirements for effluent and sediment	-may require flocculation or treatment to remove fine sediment or metals or adjust pH -may have to remove embankment and return recreational campsite	
Plant site and camp foundation	Location and hazard protection	-flat ground without natural hazards -compact to dense soils preferred	-two suitable areas identified in lower Drinkwater Valley -creek training and debris flow protection may be required	50,000 to 100,000
	Foundation preparation	-compact to dense soils or rock required -soft soils may require deep foundation system	-alluvial/colluvial soils identified at preferred locations are geotechnically feasible -specific subsurface investigations are required	60,000 to 130,000
	Water supply	-appropriate aquifer required -access to site and system maintenance -continuous supply	-two wells installed in gravels of Drinkwater Creek floodplain most reliable and practical source	30,000
	Sewage disposal	-drainfield location -soil properties -groundwater contamination	-septic tank and drainfield most practical -environmental impact and permitting must be addressed	30,000
Tailings disposal	Location	-flat to gently sloping ground without natural hazards	-ideal location is next to plant site and camp -creek training and debris flow protection may be required	150,000 to 310,000
	System requirements	-must provide adequate storag space -must protect environment by inhibiting acid generation -supernatant quality	-33,000m³ of tailings expected -natural liner will be required -may require supernatant treatment facility or operation of tailings as a closed system -pond size will increase if closed system is required	
	Design and construction	-construction method -sources of borrow for embankment liner -embankment stability	-conventional impoundment formed by cut and fill most practical -seismic stability must be assessed	
	Operation/ Reclamation	-minimal potential environmental impact -water balance	-facility will be flooded to inhibit acid generation -revegetation required on dykes and cut slopes -detailed assessments required	

Notes:

1. All costs have been rounded. A breakdown of various components and items included in the cost estimate are given i Tables I,II and III.

APPENDIX A

RESUME OF

DENNIS C. MARTIN, M.Sc. DIC, P.Eng.

DENNIS C. MARTIN, P.ENG.

Principal
Geotechnical Engineer/Engineering Geologist

BIOGRAPHY

Mr. Dennis Martin received a B.A.Sc. in Geological Engineering from the University of British Columbia in 1973, and an M.Sc./DIC (with distinction) from the Royal School of Mines in 1978. Mr. Martin has over nineteen years' experience in engineering geology, rock mechanics and geotechnical design for mines throughout Canada and overseas.

Mr. Martin is a member of The Association of Professional Engineers and Geoscientists of British Columbia, The Association of Professional Engineers, Geologists and Geophysicists of Alberta, the Canadian Institute of Mining and Metallurgy, the Association of Engineering Geologists, and the Canadian Geotechnical Society. In 1978 he was co-recipient of the Leonard Medal awarded by the Engineering Institute of Canada for the best paper in rock mechanics.

Mr. Martin is currently President of Piteau Associates Engineering Ltd., a firm of geotechnical and hydrogeological consultants with offices in Vancouver and Calgary. He is also an adjunct professor at the University of British Columbia, where he presents a course in Applied Engineering Geology.

DENNIS C. MARTIN, P.ENG.

Principal
Geotechnical Engineer/Engineering Geologist

Education:

1973. B.A.Sc. in Geological Engineering, University of British Columbia.

1978. M.Sc./D.I.C. (with distinction) in Engineering Geology, Imperial College of Science and Technology, London, England.

Currently conducting Ph.D. research (part time) on time dependent behaviour of high rock slopes.

Qualifications and Experience:

Mr. Martin is President of Piteau Associates Engineering Ltd. Mr. Martin has over 19 years' experience in engineering geology, rock mechanics and soil mechanics as applied to design and behaviour of slopes, underground openings, tunnels, shafts and foundations.

Mr. Martin has carried out general geological and detailed structural mapping programs, related structural and stability analyses and design for numerous mines in Quebec, Ontario, Alberta, British Columbia and the Yukon; also in Arizona, California, Idaho and Nevada in the United States; and in South Africa, Chile, Peru, China and Spain.

Mr. Martin has been involved on several major highway and tunnelling projects in Canada and the United States, dealing with stability assessments and design of remedial measures for rockfalls, debris flows and debris torrents, and construction supervision.

Mr. Martin has carried out field work and co-authored reports for various phases of the Pit Slope Project sponsored by Energy, Mines and Resources Canada. He has also presented a "Workshop on Rock Slope Engineering" for transportation, mining and civil engineering applications to various governmental agencies and private organizations. He is also an Adjunct Professor in Engineering Geology at the University of British Columbia.

Memberships:

Association of Professional Engineers and Geoscientists of British Columbia
Association of Professional Engineers, Geologists and Geophysicists of Alberta
Association of Engineering Geologists
Canadian Institute of Mining and Metallurgy
Canadian Geotechnical Society
Society of Mining Engineers of AIME
Tunnelling Association of Canada

DENNIS C. MARTIN, P.ENG.

Principal
Geotechnical Engineer/Engineering Geologist

The following presents a detailed summary of Mr. Martin's experience:

1984/Present **PITEAU ASSOCIATES ENGINEERING LTD.**
President and Senior Geotechnical Engineer:

- Detailed geotechnical assessments and numerical modelling of deep seated toppling failures at several mines in western North America.
- Detailed geotechnical assessments and review of numerical modelling studies for slope steepening at palabora Mine, South Africa.
- Design and supervision of anchor tie down systems for earthquake loading on shipping centre in Surrey, B.C.
- Rock mechanics design and blasting aspects for construction excavation along the coast of British Columbia.
- Review of ground support and related problems for the McDame Underground Deposit, Cassiar, B.C. and the ore transport decline at Palabora Mine, South Africa.
- Engineering geology and geotechnical investigations for grading contracts and bridges on the Coquihalla Highway Project, B.C.
- Rock mechanics investigations and geotechnical assessments of rock rippability and design of slopes for the Gold Bar Project and Rawhide Project, Nevada.
- Rock mechanics investigation and design of slopes and artificial support systems. Coordinated geotechnical aspects of monitoring and related research for Alberta Energy and Natural Resources research project on footwall anchoring, Smoky River Coal, Alberta.
- Rock mechanics investigations and preliminary designs for underground mines including Lawyers Mine, B.C.; Raul Mine and Milpo Mine, Peru; and Los Santos Mine, Spain.
- Rock mechanics investigation and design of Big Missouri/Silbak Premier Mines and Golden Bear Mine, B.C.

DENNIS C. MARTIN, P.ENG.

Principal
Geotechnical Engineer/Engineering Geologist

- Engineering geology and tunnel assessments for proposed 50 MW hydroelectric scheme on Rio Tarma, Peru.

- Rock mechanics review and remedial measures for slopes at Palabora Mine, South Africa; Cyprus Bagdad Copper, Arizona; Barrick Goldstrike Mine, Nevada; Cassiar Mine, B.C.; and Brenda Mine, B.C.

- Assessment of large postglacial landslides near Telegraph Creek and Williams Lake, B.C.

- Rock mechanics assessments and design of rock reinforcement systems for Waterfront Centre, Vancouver, B.C.

- Assessment of major landslide/debris flow in central highlands of Peru.

- Engineering geology and geotechnical assessments for waste dumps at Premier Mine, B.C. and Cyprus Thompson Creek Mine, Idaho.

- Specialist advice regarding debris torrent hazards at Lions Bay, B.C. and Revelstoke, B.C

1978/1984 **D.R. PITEAU & ASSOCIATES LIMITED**
Associate and Senior Geotechnical Engineer:

- Rock mechanics and slope design investigations for Cassiar Mine, B.C. and Reocin Mine, Spain.

- Rock mechanics investigation and design for Y Gold Mine, Shandong Province, Peoples Republic of China.

- Assistance with artificial support and remedial measures for Dixie Knot Tie Road, Oregon and retaining wall collapse on Interstate 70, Charleston, West Virginia.

- Geotechnical data collection, analysis, design and cost estimates and preparation of contract documents for the Km 80 and Km 86 Tunnels on the Tumbler Ridge Branchline of British Columbia Railway.

- Data collection, analysis and preliminary design of open pit slopes for supergene enriched copper deposit in northern Chile for Cerro Colorado.

DENNIS C. MARTIN, P.ENG.

Principal
Geotechnical Engineer/Engineering Geologist

- Data collection, analysis and design of open pit slopes for tungsten deposit at Mt. Carbine Mine in Australia. Work also involved design of artificial support for slopes in deeply weathered residual soil adjacent to the mill.
- Data collection, analysis, design, cost estimates and preparation of contract documents and construction supervision of remedial measures for a large debris flow at Agassiz Mountain, B.C.
- Geotechnical data collection, analysis and design of proposed 200m deep open pit at Robb, Alberta for Denison Mines. Also acted as project coordinator for preparation of geotechnical report for submission to licensing authority.
- Data collection, analysis and design of remedial measures for footwall slope at Line Creek Mine and geotechnical investigation and design of slopes for the proposed Line Creek Expansion.
- Geotechnical data collection, analyses and preliminary design of slopes for the proposed open pit at the Los Bronces project, Chile as well as analysis and preliminary design of the proposed underground crushing station and 36 km long ore transport tunnel.
- Data collection, analysis and design of open pit slopes for 700m deep pit at Palabora Mine, South Africa.
- Data collection, back analysis and design of open pit slopes in volcanic rocks subject to numerous failures and adverse groundwater conditions at the Savage River Mine, Tasmania.
- Engineering geology and rock slope analyses and design for Afton Mine in B.C. and highway projects in North Carolina.

1977/1978 **M.Sc. COURSE, IMPERIAL COLLEGE OF SCIENCE AND TECHNOLOGY, LONDON.**

Dissertation entitled "The Influence of Fabric Geometry and Fabric History on the Stability of Rock Slopes".

DENNIS C. MARTIN, P.ENG.

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Geotechnical Engineer/Engineering Geologist

1976/1977 — **D.R. PITEAU & ASSOCIATES LIMITED**
Intermediate Engineer:

Field data collection and slope stability analysis for rock slopes at Granisle and Rexpar Mines in B.C., Griffith Mine in Ontario and for rock cuts on the British Columbia Railway near Dease Lake, B.C.

1973/1976 — **PITEAU GADSBY MACLEOD LTD.**
Junior Engineer:

- Field data collection, geological mapping, core logging, rock testing, slope stability analyses and design for slopes at Hilton Mine, Quebec; Clinton Mine and Anvil Mine, Yukon; Cassiar Mine, B.C.; and Woodrat Mine in Idaho, U.S.A.

- Completed field work and co-authored reports for some phases of the Pit Slope Project of Energy, Mines and Resources - Canada.

Prior to 1973 — Structural mapping in Fording Coal Mine and Granisle Mine, B.C. Graduation thesis discussed the geological structural and slope stability analyses of the Granisle pit.

DENNIS C. MARTIN, P.ENG.

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PUBLICATIONS

Field Mapping for Slope Stability Analysis. 1976 - Proc. 11th Canadian Rock Mechanics Symposium, Structural Geology Session, Vancouver, B.C. October.

Application of Waviness of Structural Discontinuities to Rock Slope Design. 1976 - With D.R. Piteau. Proc. 29th Canadian Geotechnical Conference, Vancouver, B.C. October 13-25.

Slope Stability Analysis and Design Based on Probability Techniques at Cassiar Mine, British Columbia. 1977 - With D.R. Piteau. CIM Bulletin, Vol. 70, No. 779. March. pp. 139-150.

Select Berm Width to Contain Local Failures. 1977 - With D.R. Piteau. E&MJ Operating Handbook of Surface Mining, Vol. 2, pp. 103-106.

The Influence of Fabric Geometry and Fabric History on the Stability of Rock Slopes. 1978 - M.Sc. Dissertation. University of London (Imperial College). 99p.

Handbook - Workshop on Rock Slope Engineering. Reference Manual on Rock Slope Engineering. 1979 - With D.R. Piteau and A.F. Stewart. Prepared for Federal Highways Administration, U.S. Department of Transportation, Washington, D.C.

Mechanics of Rock Slope Failure. 1982 - With D.R. Piteau. Proc. Third International Conference on Stability of Surface Mining, AIME, New York, Chapter 6. pp. 113-169.

Design Examples of Open Pit Slopes Susceptible to Toppling. 1982 - With D.R. Piteau and A.F. Stewart. Proc. Third International Conference on Stability in Surface Mining, AIME, New York, Chapter 29. pp. 679-712.

Remedial Measures for Debris Flows at Agassiz Mountain Institution, B.C. 1984 - With D.R. Piteau, P.M. Hawley and R.A. Pearce. Canadian Geotechnical Journal, Vol. 21, No. 3. August.

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Assessing Stability of Coal Footwall Slopes. 1985 - With T. Zehir. Engineering and Mining Journal, McGraw Hill. December. pp. 140-141.

Assessment and Design of Artificial Support for Optimum Excavation of a Footwall Slope at Smoky River Coal, Alberta. 1985 - With P. Sheehan and D. Fawcett. CIM District 5 Meeting, Hinton, Alberta.

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Failure Mechanics and Design Considerations for Footwall Slopes. 1986 - With P.M. Hawley and C.P. Acott. CIM Bulletin Volume 79, No. 896. December. pp. 47-53.

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Application of Rock Reinforcement and Artificial Support in Surface Mines. 1987. Proceedings 6th Congress of International Society for Rock Mechanics. Montreal. August 31 to September 3. pp. 437-443.

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Assessment of Instrumentation Systems and Behaviour of A Steep Unbenched Footwall Slope. 1988 - With P. Sheehan, CIM Annual Meeting. Edmonton. May 14-15.

Remedial Measures for Debris Flows in British Columbia. 1988 - Bulletin of the Association of Engineering Geologists. Vol. XXV, No. 2. May. pp. 265-267.

Footwall Anchoring at Smoky River Coal Limited, Alberta. 1988 - With D. Fawcett and P. Sheehan. CIM Bulletin, Vol. 81, No. 915. July. pp. 78-85.

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Geotechnical Assessment of the Southeast Wall Slope Failure at Highmont Mine, B.C. 1988 - With H.W. Newcomen. CIM Bulletin, Vol. 81, No. 917. September. pp. 71-76.

Assessment of a Major Landslide/Debris Flow at San Vicente Mine, Chanchamayo, Peru. 1989 - With A.L. Galpin and R. Tejada. Proceeding International Erosion Control Association Conference, Vancouver. February 15 to 18. 12p.

Use of Percussion Drilling Information for Pit Slope Design. 1990 - With Z. Zavodni. AIME Annual Meeting, Salt Lake City, Feb. 28 to March 1. 8p.

Engineering Geology and Geotechnical Assessment of the Muddy Lake Landslides, Northwestern British Columbia. 1990 - With J.D. Tod. Symposium on Landslide Hazards in the Canadian Cordillera. Geological Association of Canada Annual Meeting, May. 14 p.

Deformation of Open Pit Slopes by Deep Seated Toppling. 1991. International Journal of Surface Mining and Reclamation. In press.

Numerical Modelling Studies for Design of High Rock Slopes at Palabora Copper Mine. 1991 - With L. duPlessis. 7th Congress of the International Society for Rock Mechanics. Aachen, West Germany. September 16-19, 1991. 6p.