MOUNT POLLEY MINING CORPORATION
MOUNT POLLEY PROJECT
TAILINGS STORAGE FACILITY

REPORT ON
ON-GOING CONSTRUCTION REQUIREMENTS
(REF. NO. 10162/9-3)

DECEMBER 2, 1997

Knight Piésold
CONSULTING ENGINEERS

Access
95-2018
Box 18

MOUNT POLLEY MINING CORPORATION
MOUNT POLLEY PROJECT
TAILINGS STORAGE FACILITY

REPORT ON
ON-GOING CONSTRUCTION REQUIREMENTS
(REF. NO. 10162/9-3)

DECEMBER 2, 1997

MP00008

Suite 1400
750 West Pender Street
Vancouver, British Columbia
Canada V6C 2T8
Telephone (604) 685-0543
Telefax (604) 685-0147
CIS: 72360,477

Knight Piésold Ltd.
CONSULTING ENGINEERS

GRTT # 2806

Knight Piésold Ltd.
CONSULTING ENGINEERS

MOUNT POLLEY MINING CORPORATION
MOUNT POLLEY PROJECT
TAILINGS STORAGE FACILITY

REPORT ON
ON-GOING CONSTRUCTION REQUIREMENTS
(REF. NO. 10162/9-3)

MOUNT POLLEY MINING CORPORATION
MOUNT POLLEY PROJECT
TAILINGS STORAGE FACILITY

REPORT ON
ON-GOING CONSTRUCTION REQUIREMENTS
(REF. NO. 10162/9-3)

TABLE OF CONTENTS

TABLES

FIGURES

DRAWINGS

APPENDICES

MOUNT POLLEY MINING CORPORATION
MOUNT POLLEY PROJECT

TAILINGS STORAGE FACILITY
REPORT ON
ON-GOING CONSTRUCTION REQUIREMENTS
(REF. NO. 10162/9-3)

SECTION 1.0 - INTRODUCTION

1.1 PROJECT DESCRIPTION

The Mount Polley Project is located in central British Columbia, approximately 56 kilometres north-east of Williams Lake, as shown on Figure 1.1. The nearest settlement is the community of Likely, on the northern tip of Quesnel Lake.

The project derives its name from Mount Polley, a low mountain with a peak elevation of 1260 metres, approximately 300 metres above the surrounding terrain. Mount Polley is situated on a topographic ridge with Polley Lake to the east and Bootjack Lake to the southwest. The site is accessible by paved road from Williams Lake to Morehead Lake and then by gravel forestry road for the final 12 kilometres.

The Mount Polley open pit mine contains an estimated 82.3 million tonnes of copper and gold ore in three ore bodies. After loading in the pit, the ore will be hauled to the crusher where it will be crushed. The ore is then transported to the nearby concentrator where it will be processed by selective flotation to produce a copper-gold concentrate at a rate of approximately 17,808 tonnes per day (6.5 million tonnes per year). Approximately 92.6 million tonnes of waste rock will be stored immediately east of the Millsite.

The mill tailings will be discharged as a slurry into the Tailings Storage Facility which has been designed to provide environmentally secure storage of the solid waste. As the solids settle out of the slurry, process fluids are collected and recycled back to the mill for re-use in the milling process. No surface discharge of any process solution from the tailings facility is required or anticipated.

1.2 SCOPE OF REPORT

This report presents the details and concepts for the on-going construction and operation of the Mount Polley Mine Tailings Storage Facility. It will be used as support to obtain a permit to operate the facility to its final elevation. Although the facility will be permitted to the final elevation, a detailed design report will be prepared for each embankment raise. Based on the results of the design report, technical specifications and construction drawings will also be prepared for each raise.

SECTION 2.0 - SITE CHARACTERISTICS

2.1 HYDROMETEOROLOGY

2.1.1 General

The area is subject to a relatively temperate climate with warm summers and cool winters. Precipitation is well distributed throughout the year. Climate records are available for Likely (6 years of record) and Horsefly (11 years), which are located in similar terrain within 40 km of the site.

The mean annual temperature at Likely, the nearest station, is 4.0° C with an extreme maximum of 33.9° C and an extreme minimum of -37° C. Quesnel, with approximately 70 years of record, has extremes of 40.6° C and -46.7° C. Frost free days in the area range from 199 at Horsefly Lake (elevation 788 m) to 244 at Barkerville (elevation 1244 m).

2.1.2 Precipitation and Evaporation

mt. Polley is hy. rainfall area (orogenic?)

Precipitation data at the site is limited and thus precipitation records for climatologically similar stations in the area were used to estimate mean annual site precipitation values. The mean annual precipitation at Likely is 699.7 mm and at Barkerville (over 70 years of record) is 1043.9 mm. Site precipitation is expected to fall within this range. Data for Likely, Barkerville and the site are presented in Table 2.1. A coefficient of variation of 0.16 was determined from regional values. This translates to a standard deviation of 121 mm. These conditions were applied to the Tailings Storage Facility.

B'ville better

A mean annual precipitation of 755 mm was determined for the Tailings Storage Facility. The waste dumps, pit areas and Millsite, (all at higher elevations) were modelled with a mean precipitation of 810 mm, a coefficient of variation of 0.16 and a standard deviation of 130 mm. The increased precipitation value was determined by applying an orographic factor of 1.07285 to the values for the Tailings Storage Facility. The

sig figs

orographic factor is consistent with elevation correlations developed in previous studies. This data is summarized on Table 2.2.

too high ?? decreas in high precip yr.

An annual evaporation rate of 423 mm at the site has been assumed to be constant for all years of operation and precipitation conditions.

Still not a big deal!

2.1.3 Runoff Coefficients

Site water balances include runoff coefficients based on average precipitation conditions only. The runoff coefficients are summarized below:

Component Description	Runoff Coefficient (%)		
	Dry	Average	Wet
Unprepared Tailings Basin	20	24	29
Tailings Beach	90	90	90
Open Pit	45	50	55
Millsite (Disturbed)	65	70	75
Waste Rock Dumps	58	60	62
Undisturbed Catchments	20	24	29

Seems low must discuss c KP

2.1.4 Storm Events

Intensity-duration-frequency curves were developed for the site based on data from the Rainfall Frequency Atlas for Canada (RFAC), as shown on Figure 2.1. Probable maximum precipitation values for the site were also estimated, as shown on Table 2.3. As outlined in the RFAC, the 1 and 6 hour values are not influenced by orographic factors, while the 24 hour and 10 day values are significantly affected. A conservative orographic factor of 1.5 was used to evaluate the storm events at the higher elevations.

The 10 day PMP storm event of 406 mm was estimated by assuming a ratio of 10 day to 1 day PMP of 2.0. This value was used in the evaluation of embankment storage requirements.

Time period June - Oct.
* Event precip.

2.2 REGIONAL GEOLOGY

Mount Polley is located in an alkalic intrusive complex in the Quesnel Trough, a 35 km wide north-west trending volcanic sedimentary belt of regional extent. The rock units are segmented into blocks by several faults, including an inferred north westerly trending normal fault that extends along Polley Lake. The predominant structure of the region is northwest trending and dipping steeply to the northeast.

The topography is generally subdued and the area has been glaciated. Surficial deposits of well graded dense glacial till are common throughout the region and are typically present in greater thickness in topographic lows. Bedrock exposures are common at higher elevations.

2.3 SEISMICITY

2.3.1 Regional Seismicity

Mount Polley is situated in an area of historically low seismicity. The site is located within the Northern B.C. (NBC) source zone, close to the boundary with the Southeastern B.C. (SBC) source zone, as defined by Basham et al (1982). Basham assigns a maximum earthquake magnitude of 5.0 for the NBC zone. However, in March, 1986 a magnitude 5.4 did occur close to Prince George, approximately 200 km north-east of the project site. A maximum magnitude of 6.5 has been set for the SBC zone, based on historic earthquake data.

There has been much debate in recent years concerning the possibility of a large interplate earthquake of magnitude 8 or 9 along the Cascadia subduction zone. Such an event would be located at over 400 km west of the project site, and therefore ground motions amplitudes would be relatively low due to attenuation over such a large distance. However, the duration of shaking experienced at the site may be very long for such an event.

Southwest of the site lies the Northern Cascades region where a maximum earthquake magnitude of 7.5 has been estimated, based on historic seismic records and geologic data, (LaVassar, 1991). This potential source zone lies at a minimum distance of about 200 km and therefore is unlikely to have a significant impact at the site.

2.3.2 Seismic Design Parameters

A seismic hazard assessment for the site has been completed using both probabilistic and deterministic methods. Seismic ground motion parameters for both the Design Basis Earthquake (DBE) and Maximum Design Earthquake (MDE) have been determined.

The probabilistic analysis was carried out by the Pacific Geoscience Centre based on the method presented by Cornell (1968). The results are:

Return Period (Years)	100	200	475	1000
Maximum Ground Acceleration (g)	0.021	0.028	0.037	0.046
Maximum Ground Velocity (m/sec)	0.043	0.056	0.077	0.094

Four potential source zones were considered for estimation of the maximum ground acceleration at the site for the deterministic analysis. These source zones are the Northern B.C., Southeastern B.C., Northern Cascades and Cascadia Subduction Zones. The results are tabulated below together with the maximum magnitude and estimated minimum epicentral distance for each zone:

Source Zone	Maximum Magnitude	Epicentral Distance,(km)	Maximum Acceleration, (g)
Northern B.C.	5.0	0	0.13
Southeastern B.C.	6.5	40	0.13
Northern Cascades	7.5	200	0.04
Cascadia Subduction Zone	9.0	450	0.08

The Northern B.C. magnitude 5.0 earthquake corresponds to a worst case event occurring directly beneath the site with a focal depth of 20 km. Maximum accelerations were calculated using the ground motion attenuation relationship given by Idriss (1993), using the Mean +1 standard error relationship. Based on this, a Maximum Credible Earthquake (MCE) of magnitude 6.5, causing a bedrock acceleration of 0.13 g has been assigned to the site. However, for seismic stability analyses a magnitude 9 event with a bedrock acceleration of 0.08 g has also been considered as an alternative MCE. This earthquake has the potential to be more damaging due to the long duration of ground shaking associated with such an event.

The selection of appropriate design earthquakes is based on criteria given by the Canadian Dam Safety Association's "Dam Safety Guidelines for Existing Dams". These criteria are given on Table 2.4. A "LOW" consequence category has been assessed for the Tailings Storage Facility. For post-closure conditions a conservative "HIGH" consequence category has been adopted for design.

The seismic ground motions adopted and implications for design are summarized below:

- The Design Basis Earthquake (DBE) for operations will be taken as the 1 in 475 year return period event. This corresponds to a maximum firm ground acceleration of 0.037 g and maximum ground velocity of 0.077 m/sec. A design earthquake magnitude of 6.0 has been selected. These parameters will be used for the design of all earthwork structures. These values are also recommended for the design of all site buildings and structures, consistent with the National Building Code of Canada. The above ground motion parameters place the site in seismic zone 0 for acceleration and zone 1 for velocity, ($Z_a < Z_v$).

- The Maximum Design Earthquake (MDE) for closure of the Tailings Storage Facility shall conservatively be taken as 50% of the MCE.

This MDE corresponds to approximately the 1 in 2500 year return period event, based on extrapolation of data from the probabilistic analysis. This event gives a maximum firm ground acceleration of 0.065g and design magnitude of 6.5. A design earthquake of magnitude 9.0 with a maximum firm ground acceleration of 0.04g has also been selected for seismic design. These events have been adopted for the design of the embankment for post-closure conditions.

Due to the dense nature of the overconsolidated foundation soils at the site, the amplification of seismic waves as they propagate from bedrock to the ground surface will not be significant. Case studies have shown that ground motion amplification is negligible through dense soil deposits overlying bedrock. Therefore, maximum bedrock ground motion parameters have been used for embankment design.

wet sands?
[illegible]
as a layer

SECTION 3.0 - TAILINGS CHARACTERISTICS

3.1 PROCESS DESCRIPTION

Mount Polley tailings will be produced from conventional milling of copper and gold ore. The anticipated tailings stream will be as follows:

- Solids throughput: 17,808 tpd (6.5 million tonnes per year)
- Percent solids: 35 percent
- Solids specific gravity: 2.78

Tailings slurry will be deposited from spigots located on the inside crest of the embankments. Tailings will initially be deposited into make-up water and a submerged beach with a slope of 15 to 20 percent is expected to develop from the coarser tailings fraction. Steep Finer tailings particles will be transported further before settling. The overall slope of the tailings solids is expected to be about 1.5 percent. (Tailings beach slopes were estimated from experience at other mines and are based on the results from the publication "Tailings Beach Slopes" by B. H. Conlin).

Huh??

A sandy beach will develop as the coarser tailings fraction settles more rapidly adjacent to the embankment. The average beach slope above water will be about 1.5 percent. The finer tailings particles will be transported further out into the supernatant pond before settling at a minimum anticipated slope of about 0.25 percent. Overall, seems flat the tailings solids are assumed to have an average slope of about 0.5 percent.

3.2 PHYSICAL CHARACTERISTICS

3.2.1 Initial Testwork

Preliminary metallurgical testwork was conducted on tailings from drill core samples in 1989 and 1990. The bulk tailings were comprised of 64 percent silt, 30 percent fine sand and 6 percent clay sized particles. The gradation limits for the 1990 bulk tailings are shown on Figure 3.1. The tailings were non-plastic, yellow grey in colour, with a solids specific gravity of 2.78.

Settling tests were conducted at slurry solids contents ranging from 25 to 45 percent. The particles settled rapidly, with a pronounced segregation of coarse to fine material. The colloidal clay fraction remained suspended in the supernatant water for several days. The tailings initially settled to relatively low dry densities of 0.9 to 1.1 tonnes/m^3. Consolidation from evaporative drying resulted in final dry densities of 1.3 tonnes/m^3.

The initial volume of water recovered from the tailings depends on the initial solids content of the slurry. At 35 percent solids, the initial water recovery was about 64 percent of the total water in the slurry. The vertical permeability of the settled tailings varied from 1.0 to 2.0 x 10^{-5} cm/s. The permeability will be reduced due to on-going consolidation. Detailed results were presented in Knight Piésold report "Design Report, Ref. No. 1625/1".

3.2.2 1996 Testwork

Additional tailings testwork was conducted in 1996 by MET Engineers Ltd. Tailings were separated into two streams called the finer Slime Tails and the coarser Sand Tails. The Slime tails comprised about 57 percent of the tailings stream. The Sand Tails made up the remaining 43 percent.

The Slime Tails were comprised of 85 to 90 percent well graded silt. The remaining 10 to 15 percent was clay sized particles. The Sand Tails comprised about 26 percent fine sand and about 70 to 74 percent coarse silt.

Bulk Tailings were estimated by combining results from the Slime and Sand Tails. Bulk Tailings comprised about 13 percent fine sand, 77 to 82 percent silt and 5 to 10 percent clay sized particles. The gradation limits of the Slime, Sand and Bulk tailings are also shown on Figure 3.1.

3.2.3 1997 Testwork

In October 1997, tailings samples were obtained from the Tailings Storage Facility and Mill. A testing program was conducted to determine the index properties and settling and consolidation characteristics of the tailings. Index tests included specific gravity, Atterberg limits and particle size distribution (sieve and hydrometer). Slurry tests included undrained and drained settling tests, falling-head permeability tests and slurry consolidation tests.

Three tailings samples were tested. A composite sample (BK2) was collected from the mill on-stream analyzer, which retrieves a sample hourly. Samples collected over a three week period were filtered, dried and combined to form the composite sample. Two additional samples were collected at the Tailings Storage Facility. One sample of bulk tailings slurry (BK1) was taken at one of the discharge spigots. Another sample (BH1)was taken directly from the exposed tailings beach adjacent to the Main Embankment.

where along line & was bias by spigot position on line

Index Tests

The specific gravity of the BK1 tailings solids was determined in two tests to be 2.75 and 2.73. An average of 2.74 was used for all calculations. The bulk tailings (BK1) were determined to be non-plastic, with a liquid limit of 19%.

Pg. 9 2.78

See page 10.

The bulk tailings (BK1) were fine-grained sandy silt (21% fine sand, 68% silt) with a trace of clay (11%). The composite sample (BK2) consists of 31% fine sand, 61% silt and 8% clay. The coarser beach tailings (BH1) consist of 66% fine sand, 31% silt and only 3% clay. The grain size distributions for each of the 1997 samples are also shown on Figure 3.1.

The results on the bulk (BK1) and composite (BK2) samples were compared to the 1990 bulk tailings sample. These tailings have a very similar gradation to the composite tailings (BK2). The 1997 bulk tailings (BK1) sampled from a discharge point are similar but have a higher percentage of fine silt particles.

Tailings grain size distribution is routinely carried out by the mine on samples from the on-stream analyzer. From mid-October to mid-November, 1997, the fines content of the tailings, (percent passing a No. 200 sieve) has varied from 51% to 69% and has averaged about 61%. Both the bulk tailings sampled from a discharge point within the storage facility (BK1), and the composite tailings sample (BK2), have a fines content greater than this average.

Slurry Tests

For the slurry tests, water was added to the tailings samples to create slurries with an initial solids content of approximately 35 percent. The samples were mixed to produce a consistent slurry prior to testing. In addition to the bulk (BK1), beach (BH1) and composite (BK2) tailings samples, a fine tailings slurry (SS1) was also tested. The fine tailings slurry was prepared in the laboratory by allowing some of the bulk tailings sample to settle and segregate in a large pail. The finer material on the top was then carefully collected and remixed to form a slurry. This slurry is likely to be representative of the fine-grained material located within the supernatant pond. ??

Undrained settling, drained settling, and falling head permeability tests were conducted on each of the four samples. Slurry consolidation tests were performed on the bulk tailings (BK1) and fine tailings (SS1) samples.

For the undrained settling tests, each sample was placed in a one litre graduated cylinder. The settling rate was recorded and the dry density of the settled solids was calculated with time. These tests estimate the density to which the tailings settle in an undrained, sub-aqueous environment. Undrained settled dry densities of 0.81 tonne/m^3 and 1.10 tonne/m^3 were achieved for the bulk (BK1) and composite (BK2) tailings, respectively. An undrained settled dry density of 0.89 tonne/m^3 was recorded from the 1990 bulk tailings sample.

Settled dry densities of 1.19 and 0.49 tonne/m^3 were achieved for the beach (BH1) and fine tailings (SS1), respectively. These dry densities represent the range of initial densities achieved by the tailings upon settling within the Tailings Storage Facility. The settled dry densities for each of the tailings slurries are summarized below. same low !!

Tailings Sample	Settled Dry Density (t/m^3)		Falling-Head Permeability (cm/sec)	Slurry Consolidation, C_v (m^2/year)
	Undrained	Drained		
Beach (BH1)	1.19	1.20	5.5×10^{-5}	-
Bulk (BK1)	0.81	0.92	4.7×10^{-5}	10
Fine (SS1)	0.49	0.57	5.4×10^{-6}	1
Composite (BK2)	1.10	1.10	2.2×10^{-5}	-

1. Initial solids content of tailings slurries approximately 35% prior to settling.
2. Cv = Coefficient of Consolidation

Drained settling tests were performed using one litre graduated cylinders with bottom drainage and recovery of downward seepage. These tests provide an indication of the dry density that will be achieved from settling with drainage at the base of the sample. Drained settled dry densities of 0.92 and 1.10 tonne/m^3 were achieved for the bulk (BK1) and composite (BK2) tailings, respectively. A drained settled dry density of 1.10 tonne/m^3 was recorded from the 1990 testwork. Dry densities of 1.20 tonne/m^3 and 0.57 tonne/m^3 were achieved for drained settling of the beach (BH1) and fine (SS1) tailings respectively. The resulting drained settled densities are summarized above.

Falling head permeability tests were conducted after completion of the drained settling tests. Water was applied to the surface of the tailings, imposing a vertical gradient across the sample. The drainage rate and drop in water level were recorded with time. The following results were obtained:

Tailings Sample	Average Void Ratio (After Drained Settling) (e)	Coefficient of Vertical Permeability (cm/sec)
Beach (BH1)	1.45	5.5×10^{-5}
Bulk (BK1)	1.94	4.7×10^{-5}
Fine (SS1)	3.75	5.4×10^{-6}
Composite (BK2)	1.39 ?	2.2×10^{-5}

These results provide an indication of the coefficient of permeability of the tailings at low effective stresses and corresponding low densities (high void ratios). The tailings permeability will decrease as consolidation increases the density and reduces the void ratio.

Slurry consolidation tests were performed on the bulk (BK1) and fine (SS1) samples to determine their coefficient of consolidation at low effective stresses (high void ratios). The tests were carried out by introducing a pre-measured quantity of tailings slurry into a one litre burette with the bottom stopcock closed. After settling of the slurry the bottom stopcock was opened to permit drainage and dissipation of pore pressures, causing an increase in the effective stress across the sample. The decrease in volume with time was recorded.

The calculated coefficients of consolidation and the corresponding average void ratios for the tailings slurry for both tests are as follows:

Tailings Sample	Average Void Ratio (e)		Coefficient of Consolidation (m^2/year)
	Settled	Consolidated	
Bulk (BK1)	2.32	1.62	10
Fine (SS1)	4.54	2.57	1

A coefficient of consolidation of 10 m^2/year for the bulk tailings is in good agreement with the tailings parameters used for the consolidation analyses in the design of the Tailings Storage Facility, (Knight Piésold Design Report, Ref. No. 1625/1). The coefficient of consolidation of 1 m^2/year is a typical lower bound value for fine tailings material.

3.3 GEOCHEMICAL CHARACTERISTICS

Geochemical testwork on a locked cycle tailings sample was conducted in 1989 by Coastech Research Inc. The testwork included the following:

- Determination of net acid generating potential
- Special Waste Test using acetic acid
- ASTM waste extraction test using carbonic acid

The acid base accounting procedures used were based on recommendations by the U.S. Environmental Protection Agency. The method includes an evaluation of the balance between acid producing components (primarily pyrite) and acid consuming components (carbonates and other rock types with neutralizing capabilities). The results are:

Sulphur (percent)	Paste pH	Acid Potential (kg $CaCO_3$/t)	Neutralization Potential (kg $CaCO_3$/t)	Net Neutralization Potential (kg $CaCO_3$/t)
0.02	8.22	0.6	24.6	24.0

These results indicate that the tailings are not acid producing and have a significant net neutralization potential.

major leap of faith based on one sample

A special waste classification test was conducted in accordance with the procedure published by the B.C. Ministry of the Environment, entitled "B.C. Special Waste List". The test indicates that the tailings from the locked cycle tests do not exceed the B.C. Waste Management Branch regulations for special wastes.

In addition to the special waste test, an ASTM waste extraction test using carbonic acid at pH 5.5 was carried out. The test uses carbonic acid for leaching of the tailings and is a more realistic indication of actual long term water leachable constituents under slightly acidic rainfall. The test showed very low levels of water leachable constituents in the extract, all at concentrations below the lower range concentration for the pollution control objectives for final effluent discharge.

Detailed results of the geochemical characteristics of the tailings were presented in the Knight Piésold document "Tailings Storage Facility Design Report, Ref. No. 1625/1".

SECTION 4.0 - GEOLOGICAL AND GEOTECHNICAL CONDITIONS

The tailings area geological and geotechnical conditions have been confirmed based on results of the following site investigation programs:

- The initial site investigations in 1989.
- The final design investigations in 1995
- Investigations during Stage 1a/1b construction in 1996/1997.

The results of these investigations are presented in detail in the Knight Piésold document "Tailings Storage Facility, Updated Design Report, Ref. No. 1627/2".

Additional borrow area investigations for Stage 2 construction were conducted in October, 1997. The results are presented in the Knight Piésold document "Stage 2A Tailings Facility Construction, Selected Excerpts from Reference Information, Ref. No. 10162/9-2". This document also contains a summary of all geotechnical investigations conducted prior to the Stage 2 borrow area investigations.

A geologic summary was prepared based on the above listed investigations and accompanying laboratory test data to define the surficial overburden conditions, including the continuity of the surficial glacial till and the location and extent of the underlying sedimentary units. In summary, the geology of the tailings basin is characterized by four units:

Surficial Till

A surficial layer of melt-out or Ablation glacial till underlies all areas of the tailings basin investigated to date. This glacial till is typically comprised of 50 to 65 percent sandy silt (passing No. 200). It is slightly weathered, firm to stiff, and wet for the top 0.5 to 1 metres in the lower areas of the tailings basin. It is very stiff and is moist to very moist below 1 to 2 metres depth and at higher elevations. No appreciable fissuring was observed in the surficial till unit in the lower areas of the tailings basin. This is likely due to the shallow groundwater table, which is typically less than 0.3 metres below the ground surface.

The surficial till thickness varies, but generally thins from north (4 to 6 m) to south (2 to 3 m) along the valley and may pinch out completely downstream of the Main Embankment Seepage Collection Pond. Field and laboratory permeability testing on the surficial till typically yielded results in the order of 10^{-8} to 10^{-9} cm/s.

The surficial till thickness exceeds 2 metres over most of the tailings basin. Figure 4.1 shows the extent and thickness of the surficial till near the Main Embankment. The surficial till is less than 2 metres thick over two areas, including the right abutment (approx. Ch. 16+00 to 16+75) and at the bottom of the basin (approx. Ch. 19+50 to 21+50). A glacial till basin liner was constructed over these areas. The locations of the as-built basin liners are also shown on Figure 4.1.

Glaciolacustrine/Glaciofluvial Sediments

Glaciolacustrine/glaciofluvial sediments underlie the surficial glacial till. This unit is primarily comprised of glaciolacustrine layers (silt, some clay), with lesser fine grained glaciofluvial layers (sand). The glaciolacustrine/glaciofluvial sequence thickens from west to east and from north to south, and terminates at approximately El. 928 m. It is not present along the right abutment where the surficial till directly overlies bedrock. The glaciolacustrine/glaciofluvial sequence transforms from a continuous sequence near the Main Embankment into thin (0.5 to 3.0 m) layers within the glacial till unit to the northwest. The glaciolacustrine/glaciofluvial sequence is generally 6 to 8 metres thick at the west and increases to as much as 25 metres towards the eastern edge of the tailings basin.

The glaciolacustrine/glaciofluvial sequence consists predominantly of interbedded layers of silt with either clay or fine sand. The glaciolacustrine (silt, clay) sediments are often highly over-consolidated and very stiff to hard, with a low permeability. Within the glaciolacustrine sediments, occasional seams of fine sand with only a trace of silt are present. These seams vary in thickness from 0.1 metres to greater than 3 metres.

One continuous sandy unit is present below the surficial till over a 450 metre stretch (approx. Ch. 16+50 to 21+00) directly beneath or upstream of the Stage 1b Main

Embankment footprint, as shown on Figure 4.2. The unit consists of fine-grained sand with 20 to 40 percent coarse silt. The permeability is estimated to be 10^{-5} to 10^{-6} cm/s, based on the results of gradation analyses. From Ch. 16+50 to Ch. 18+75 the unit grades into a fine to medium grained sand and contains localized areas of coarser gravelly sand. The permeability of this coarser unit is estimated to be 10^{-4} to 10^{-5} cm/s. Groundwater seeped into excavations and some of the pit walls were unstable when exposed, indicating that the unit is likely a confined aquifer. Two foundation drains, one pressure relief trench and one pressure relief well were extended into the sandy unit to contain groundwater flows in this area. A groundwater monitoring well (GW96-9) was installed in this unit just downstream of the ultimate toe of the Main Embankment.

The glaciofluvial sand unit extends into the Tailings Storage Facility for approximately 200 metres before it grades into a lower permeability silt or is overlain by more than 2 metres of surficial till. Laboratory testwork on samples of the glaciolacustrine/glaciofluvial materials from further upstream of the Main Embankment (such as the sediment layers exposed in some areas of the Reclaim Barge Channel) have shown that the material is primarily silt with variable clay content and occasional narrow seams of coarse silt with trace to some fine sand.

Near the Perimeter Embankment Seepage Collection Pond (GW96-1), the glaciolacustrine sequence consists of a 3 metre layer of firm to stiff, low permeability silts with variable clay content and thin silty sand laminations. No higher permeability layers were identified. East of the Perimeter Embankment (GW96-2), the glaciolacustrine unit is 7.5 metres thick and consists primarily of silt with rare thin (5 to 30 mm) fine to medium grained sand laminations. A 10 to 13 metre thick sequence of high permeability glaciofluvial sands and gravels was encountered at depths of 27 and 32 metres in GW96-1 and GW96-2. The higher permeability sandy gravel unit is not connected to the tailings impoundment. It is therefore not considered to be a significant seepage pathway due to the thick layer of low permeability surficial till and glaciolacustrine sediments above this zone.

Basal Till

The glaciolacustrine/glaciofluvial sedimentary sequence is underlain by a very dense, well graded silt and sand basal glacial till. The basal till dips and thickens slightly from west to east and north to south, likely following bedrock topography. It is typically 10 to 20 metres thick, massive, highly consolidated and contains some gravel and trace to some clay. The basal till has a low permeability, estimated to be less than 10^{-6} cm/s.

Bedrock

At the Main Embankment, the bedrock surface dips from west to east and more gently from north to south. Bedrock drops off quickly and is less than 1 metre below surface on the ridge at the right abutment. The bedrock surface is greater than 30 metres deep at the left abutment. Because of the thick cover of low permeability overburden soils, the bedrock permeabilities will not greatly influence seepage from the Tailings Storage Facility

except... West perimeter emb. W. end main emb.

Bedrock is predominantly a red-brown sedimentary conglomerate composed of hematitically altered volcanic tuffs and fragmentals. It is moderately to highly weathered near the surface. Weathering decreases with depth. Rock quality is typically poor to very poor for the top 15 m and improves with depth. The unit appeared to be free of large fault fractures and measured permeabilities were typically 10^{-6} cm/s or lower.

A coarse-grained syenite intrusive unit underlies much of the hill up-slope (west) of the Bootjack-Morehead Connector Road (GW96-5). The unit is massive and is generally free of large fractures. Rock quality ranged from fair to good. Smaller isolated units of mudstone, sandstone and basalt were also identified.

SECTION 5.0 - TAILINGS STORAGE FACILITY DESIGN

5.1 GENERAL

The main components of the Tailings Storage Facility are the Main, Perimeter and South Embankments. All three embankments are modified centreline zoned earthfill structures with low permeability glacial till core zones, chimney drains, upstream toe drains and downstream random fill zones constructed from mine waste rock. The tailings embankments are shown in section on Drawing No. 10162-9-201. A plan view of the final arrangement of the Tailings Storage Facility is shown on Drawing No. 10162-9-200.

The tailings embankments have been designed for staged expansion during operations in order to minimize initial capital expenditures and to maintain an inherent flexibility to allow for variations in operation and production throughout the life of the mine.

5.2 DESIGN BASIS AND CRITERIA

5.2.1 General

The principal objectives of the Tailings Storage Facility are to ensure that regional groundwater and surface water flows are not adversely affected during mining operations and in the long term, and also to permit effective reclamation at mine closure. The principal requirements of the design are to:

- Provide permanent, secure, and total confinement of all solid waste materials within an engineered storage facility.
- Control, collect and remove free draining liquids from the tailings for recycling as process water to the maximum practical extent.
- Include monitoring features for all aspects of the facility to ensure performance goals are achieved.
- Develop the facility in stages to distribute capital expenditure over the life of the project.

The design basis and criteria are based on appropriate and conservative design parameters from hazard classification, seismic data, hydrological studies, on results of site investigations and on review comments by the Ministry of Employment and Investment (MEI). The design basis and criteria for all aspects of the design, construction and operations are presented on Table 5.1.

5.2.2 Consequence Classification

A hazard classification based on the Canadian Dam Safety Association's (CDSA) "Dam Safety Guidelines for Existing Dams" has been assessed to establish design flood and seismic criteria. Details of each consequence category and the corresponding potential consequences of failure are presented on Table 5.2.

A "LOW" hazard classification or consequence category has been assessed for the Tailings Storage Facility, as discussed in Section 2.3. In accordance with the "LOW" hazard classification, a Design Basis Earthquake (DBE) corresponding to the 1 in 475 year return period event has been adopted for design of the facility during operations. For post-closure conditions, a conservative "HIGH" consequence category has been assigned.

The embankment has been designed to accommodate a maximum design earthquake (MDE) corresponding to 50% of the maximum credible earthquake (MCE) and the probable maximum flood (PMF) flood event.

5.2.3 Tailings Storage Capacity

The depth-area-capacity-filling rate relationships for the Tailings Storage Facility are shown on Figure 5.1. The projected filling rate and rate of rise are based on a production rate of 17,808 tpd (6.5 million tpy). The tailings facility has been designed to contain 84.5 million tonnes of tailings solids at an average dry density of 1.28 tonnes/m^3 (1.1 tonnes/m^3 for Year 1, 1.2 tonnes/m^3 for Year 2, and 1.3 tonnes/m^3 for Years 3 through 13). Provisions for the following are been incorporated into the design:

- Up to 2.5 million cubic metres of process (reclaim) water on top of the tailings surface.
- An emergency storage volume of at least 0.68 million cubic metres for runoff from the design storm event, the 24-hour probable maximum precipitation (PMP).
- An additional one metre of freeboard for wave run-up and emergency flood storage.

5.3 STAGE 1 CONSTRUCTION

The Stage 1 Tailings Storage Facility was constructed from May, 1996 to March, 1997. The Stage 1a Main Embankment was completed to El. 927 metres in December, 1996 to enable the impoundment of runoff water from the 1997 freshet. The Stage 1b Main and Perimeter Embankments were completed to El. 934 metres in March, 1997. Stage Ib provides sufficient storage capacity to contain the above mentioned runoff, plus additional make-up water from Polley Lake and tailings from approximately one year of production.

The main components of the Stage 1 construction included the following:

- Tailings basin clearing, grubbing and topsoil stripping and stockpiling.
- Soils investigations to determine the extent of the basin liners, including laboratory and in-situ field testing.
- Construction of the Lower and Upper Basin Liners, and additional Basin Liners in the Original Borrow Area (No. 1), where required.
- Preparation of the embankment foundations to ensure a tie-in with dense, natural ground.
- Placement and compaction of the embankment fill materials in the respective zones in accordance with the Technical Specifications. Fill materials were placed during freezing and non-freezing conditions.
- Installation and monitoring of the Main Embankment Foundation and Chimney Drain systems.
- Installation and monitoring of vibrating wire piezometers.
- A construction quality assurance (CQA) program to evaluate the construction techniques and embankment fill materials through detailed testing on the fill

and in the site soils laboratory. During cold weather construction periods, additional Knight Piésold personnel were provided to ensure that the design objectives were achieved in spite of the freezing conditions.

- Excavation of the seepage collection ponds and installation of the drain monitoring sumps, seepage recycle sumps and pipework.
- The Bootjack-Morehead Connector Relocation to replace the section of the Gavin Lake Forest Service Road that was inside the Tailings Storage Facility.
- Tailings and Reclaim pipeline access roads, complete with pipe containment channels, separate runoff diversion ditches and a crossing of Bootjack Creek.
- Installation of the HDPE tailings pipeline, including construction of the T2 tailings dropbox, construction of the spigot offtakes (M1 dump valves and movable discharge section), pipeline anchoring and pipeline testing.
- Installation of the HDPE reclaim pipeline, including construction of the reclaim booster pumpstation, pipeline anchoring and pipeline testing.
- Installation of the floating barge pumpstation, steel ball joint and steel pipe.
- Construction of the make-up water supply system components, including the Millsite Sump, the Southeast Sediment Pond and the Polley Lake Pumpstation.

Fill placement during freezing conditions was permitted only if the materials were placed and compacted to the specified densities which would normally be achieved if freezing conditions did not prevail. The criteria for placing fill materials during freezing conditions are summarized in the Knight Piésold document "Tender Documents for Stage 2A Tailings Facility Construction, Ref. No. 10162/9-1". X

Stage 1 construction was generally completed in compliance with the design intent and according to the technical specifications and construction drawings. Details are provided in the Knight Piésold document "Report on Stage 1a/1b Construction, Ref. No. 10162/7-5".

5.4 WATER MANAGEMENT AND RATE OF FILLING

5.4.1 General

The components of the water management plan include disturbed and undisturbed areas at the open pits, waste dump, Millsite, Tailings Storage

Facility, the undisturbed catchment area immediately upstream of the Tailings Storage Facility and the diverted areas downstream of the tailings embankments. A water management plan schematic is shown on Figure 5.2.

The objective of the water management plan in the early years of operation is to route all project water flows from disturbed areas into mine activities. In later years, the objective will be to monitor and release selected surface water inflows in order to manage the final volume of ponded water in the tailings impoundment at closure. These objectives will be met by :

- Maximizing the capture of surface and groundwater flows from within the project area.
- Maximizing the use of the poorest quality water recovered from within the project area in the milling process.
- Minimizing the volume of fresh water extracted from Polley Lake.
- Monitoring the quality of surface runoff from disturbed areas and groundwater flows within the project site.
- Releasing only the highest quality water from within the project boundaries in accordance with permitted requirements.
- Managing the tailings supernatant pond to optimize the volume of water stored on the tailings surface during operations and at closure.

Maintaining adequate storage during summer/fall. - discharge if needed.

The key to implementing the water management plan is developing and maintaining a detailed data base so that water balances can be as accurate as possible. This will enable the water balances to become useful tools for predicting annual make-up water requirements and for scheduling releases of clean surface runoff water.

5.4.2 Water Balance Results

The overall project water balance was originally presented in the Knight Piésold document "Report on Project Water Management, Ref. No. 1624/1". The current water balance has been modified as other catchment areas will no longer be utilized as a source of surface runoff. Instead, water will be extracted annually from Polley Lake during the spring freshet high flow period.

The water balance requires a sufficient volume of water to be available to the milling circuit during the cold winter months when precipitation accumulates as snow and surface runoff is at a minimum or if drier than average conditions occur. This requirement can be met by:

- Providing up to 2.5 million cubic metres of water in the tailings impoundment prior to start-up.
- Ensuring that 1.9 to 2.5 million cubic metres of water is available in the tailings impoundment after freshet during on-going operations.
- Allowing for contingency water extraction from Polley Lake during peak flow months. The amount of water that can be extracted from Polley Lake is limited by the minimum fish flow requirements for Hazeltine Creek.

The staged development plans for the various components of the project are included in the water balances. For this report, average annual water balances for years 1 and 13 have been prepared.

The water balance components for years 1 and 13 are shown on Figures 5.3 and 5.4. The results of the water balance for years 1 and 13, shown on Tables 5.3 and 5.4, indicate that water stored in the Tailings Storage Facility will be at a minimum in March of every year, just prior to the freshet. The subsequent snowmelt significantly increases the water storage in the tailings impoundment. However, it is unlikely that the freshet alone will provide enough water for operations for the following winter and additional make-up water will be required from Polley Lake to make up the difference. Current plans include the annual withdrawal of up to 1 million cubic metres of water from Polley Lake. A schematic illustration of the water balances for years 1 and 13 is presented on Figure 5.5.

The annual withdrawal of up to 1 million cubic metres of water from Polley Lake will likely only be required prior to start-up and for the first three to four years of operations. Withdrawal requirements for subsequent years will ?? decrease progressively with time due to the increased consolidation of tailings in the pond (greater release of pore water) and the progressive development of the open pit and waste dump.

Site water balances have been developed and are being updated based on actual production and monitoring data. The site water balances are in general agreement with the original water balance.

5.4.3 Rate of Filling

The Filling Schedule and anticipated Staged Construction sequence is shown on Figure 5.6. The filling schedule includes tailings deposition and reclaim water storage. Provisions for the 24 hour PMP and required 1 metre of freeboard are also included. These variables, combined with preferred construction seasons, are used to define the stage construction sequences. Variations in the reclaim water volume are based on the amount of water available in the pond from the water balances.

Recorded pond levels to date are also shown on Figure 5.6. The recorded pond levels indicate that the rate of filling of the impoundment is slightly behind anticipated levels. However, no modifications are currently planned for the embankment staging as additional make-up water may be supplied from the Polley Lake Pumping System.

The rate of filling for the tailings accounts for consolidation, which occurs continuously within the tailings deposit during deposition and will continue after completion of operations until all excess pore pressures have dissipated. Expulsion of pore fluids during consolidation produces settlement of the tailings surface and a corresponding increase in the average density of the deposit.

Analyses conducted to predict tailings surface settlements and average densities during operations and at closure are discussed in detail in the Knight Piésold document "Updated Design Report, Ref. No. 1627/2". In summary, an average dry density of 1.1 tonnes/m3 was predicted after the first year of operation. The average dry density will likely increase to 1.2 to 1.3 tonnes/m3 and will be maintained until closure.

5.5 ON-GOING EMBANKMENT CONSTRUCTION

5.5.1 General

On-going embankment construction requirements for the staged expansions are shown on Figure 5.6 and on Drawing No. 10162-9-201. The staged expansions will incorporate a combination of centreline and modified centreline construction methods. The on-going raises will each provide incremental storage capacity for one or two years of production. The proposed raises will be re-evaluated during operations to ensure that adequate storage capacity and embankment freeboard are maintained throughout the mine life.

The embankment design will be reviewed on an on-going basis. Drainage systems will be evaluated during operations and will be extended during on-going embankment expansions as required. Any modifications to the drainage systems will be based on operating experience, monitoring records and availability of various embankment construction materials. All pipework will include suitable levels of redundancy to compensate for minor embankment settlements or earthquake induced deformation.

See design detail

Staged embankment fill quantities for on-going construction are shown on Table 5.5.

The Tailings Storage Facility can be expanded if the ore reserves are increased above the projected total of 84.5 million tonnes. Embankment raises above the proposed final crest elevation of El. 965 metres would be constructed as required by incorporating a downstream extension of the embankment toe. This would also ensure that embankment stability is maintained.

5.5.2 Embankment Settlement

Settlement of the embankment fill materials occurs progressively as the embankment raises extend over the tailings beaches. Analyses carried out to predict the magnitude of these settlements using a one-dimensional finite element computer model are discussed in detail in the Knight Piésold document "Updated Design Report, Ref. No. 1627/2". Thea analyses are summarized below.

Two tailings columns were evaluated at increasing distance from the Stage 1b embankment crest.

- Column A - 6 metres of tailings overlain by Stages 3 to 7.
- Column B - 30 metres of tailings overlain by Stages 6 and 7.

Void ratio vs. effective stress and coefficient of consolidation vs. effective stress relationships for the tailings beach materials were based on data for similar coarse tailings from existing tailings facilities. Parameters used for the tailings consolidation analyses were adopted to represent these tailings.

Estimates of embankment settlements were made for staged expansions up to the final Stage 7 crest at El. 965 metres. These represent the maximum settlements at the deepest section of the embankment.

The "bulk" tailings adjacent to the Stage 1b embankment crest will be approximately 50% consolidated prior to construction of the Stage 2 raise. A ?? settlement of about 0.3 metres is expected during construction of Stage 2 onto these tailings. This settlement will occur during initial placement of the coarse bearing layer on the tailings and during placement of the remaining fill. Consolidation will occur rapidly during fill placement and the underlying tailings are predicted to be over 90% consolidated immediately after construction. These "bulk" tailings do not have a significant effect on predicted embankment settlements because the compressibility of this material is likely to be higher than the coarser beach tailings only at low effective stresses. Due to confinement from additional tailings, effective stresses will increase in this underlying material. Therefore, the compressibility of these tailings will be similar to the overlying beach tailings by the time on-going embankment raises are constructed.

Prove it - circular failure

For the staged expansions, the majority of the settlement for both columns occurs during placement of embankment fill, as described above. The coarse bearing layer and fill placement during construction routinely compensates for these initial tailings settlements. Excess pore pressures generated in the tailings during fill placement dissipate rapidly and the degree of consolidation is typically 70 to 90% by the end of construction of each raise.

Embankment settlements after construction of each raise in Column A will be negligible (less than 0.1 metres) and the underlying tailings will be consolidated shortly after each raise. Settlements for Column B are also expected to be minor, approximately 0.6 metres and 0.2 metres after construction of Stages 6 and 7, respectively.

On-going settlements due to additional embankment raises generally reduce as the tailings become less compressible at the high confining pressures from the overlying fill. Settlements will also vary laterally along the embankment crest due to the variable thickness of the underlying tailings. The minor settlements given above correspond to maximum values in the deepest section of the facility and therefore differential settlements will not be significant.

On-going fill placement during staged expansion of the embankment routinely compensates for settlement of the embankment crest. Sloping internal embankment zones and the chimney drain will deform slightly but will result in only a very slight flattening of the embankment drainage systems. This will not reduce the efficiency or integrity of the systems.

5.6 SEEPAGE ANALYSES

5.6.1 General

Seepage analyses were performed using the finite element computer program SEEP/W to establish the pore water pressures within the embankments for stability analyses and to estimate the amount of seepage discharge from the Tailings Storage Facility.

To reflect variability in embankment design and foundation conditions, the Final Embankment seepage analysis was conducted by dividing the Tailings Storage Facility into four sections as shown on Figure 5.7. The sections examined were:

- The Perimeter Embankment.
- The Main Embankment with a varved silt unit within the foundation (Section A).
- The Main Embankment with a sand unit within the foundation (Section B).
- The South Embankment.

Finite element models were generated for each section to estimate the seepage rates into the upstream toe drains, the chimney drain, the foundation drains (Main Embankment sections only), and the groundwater system. Seepage rates for the entire facility were determined by adding the seepage rates for each section.

Two cases were considered in the analyses. These were:

- Case 1: estimated seepage flows with the upstream toe drains functioning.
- Case 2: estimated seepage flows with the upstream toe drains not functioning.

Finite element models were also generated to determine the pore pressure conditions following completion of the Stage 2 Main Embankment and of the Post Closure Main Embankment for use during stability analysis. The conditions considered for the Stage 2 and the Final Embankment seepage analyses are summarized on Table 5.6.

During the initial year of operations, tailings will be discharged into stored make up water, resulting in limited beach development. As a conservative approximation, fine tailings, have been assumed to extend to the upstream face of the embankment up to the maximum stored make up water elevation of 925 m.

5.6.2 Summary of Parameters

Saturated and unsaturated hydraulic conductivities were determined for each material in the embankment and foundation. Typical conductivity functions for similar soil types were used in assigning hydraulic conductivity values. These functions were adjusted to correspond with the actual saturated conductivities of the material. Hydraulic conductivity values for the tailings mass, embankment and foundation were determined as follows:

- The tailings mass was sub-divided into three zones with decreasing hydraulic conductivity to account for the less permeable consolidated tailings at depth. Hydraulic conductivity values were assigned based on falling head permeability test results for tailings samples collected in October 1997.
- Hydraulic conductivity values for Zone S, Zone B, and the Basin Liner were estimated from Stage 1 Record and Control Test results.
- Hydraulic conductivity values for Zone F, Zone T, the Free Draining Fill, and Zone C were based on Stage 1 Record and Control Test results and from empirical estimations relating to particle size analysis (Hazen formula, Crum, Blein, and Munk formula, and the USBR formula).

- Hydraulic conductivity values for the foundation materials were based on field permeability test results, estimations recorded during geological investigations, and typical values for similar materials.

The material parameters used during the analyses are provided on Figures 5.8, 5.9 and 5.10.

Conductivity ratio values (ratio of the vertical hydraulic conductivity divided by the horizontal hydraulic conductivity) for the embankment materials, foundation, and the tailings were estimated based upon typical values for similar materials. The vertical and horizontal conductivities for Zone S and Zone B are expected to be equal (conductivity ratio of one), however a sensitivity analysis was completed to determine the effects a higher horizontal conductivity.

5.6.3 Boundary Conditions and Flux Sections

Boundary conditions were imposed on the modelled sections to more accurately represent hydrogeologic conditions in the field. These conditions are summarized as follows:

- A no-flow boundary condition was assigned along the left side of the model (upstream of the embankment).
- A total head boundary was imposed at the tailings surface to model a supernatant pond.
- The upstream embankment toe drains and the foundation drains were modelled by applying elevation head nodes at those locations (pore water pressure equal to zero).
- The longitudinal drain and the outlet pipe were modelled by applying an elevation head node at the base of the chimney drain.
- A hydrostatic pore pressure profile with the water table 2 metres below the ground surface was assigned to the right boundary of the model (downstream of the embankment). what happens if 1 m above?

Flux sections were included in the model to estimate seepage flow across the various geological units, as well as the engineered components. The following locations, in particular, were examined closely:

- Seepage collected by the upstream toe drain.
- Seepage collected by foundation drains.
- Seepage collected by the chimney drain.
- Seepage flow which bypasses the seepage collection systems.

Flows captured in the seepage collection systems (i.e. the upstream toe drains, the chimney drain, and foundation drains) will drain to the Seepage Collection Ponds and will be recycled to the tailings impoundment. Seepage flows which bypass the seepage collection systems are the only component lost to groundwater.

5.6.4 Results

The results of the seepage analysis are provided in Table 5.7. All seepage flow estimates are projected increases over baseline flow rates. In particular, the embankment foundation drains include a baseline groundwater flow component which is not factored into the following flow projections.

In Case one, with the upstream toe drains functioning as designed, a total seepage rate of 39.9 l/s was calculated from the seepage analysis. The solution flow contribution made by each of the components is as follows:

- The upstream toe drain collected 36.4 l/s.
- The chimney drain system collected 0.6 l/s.
- The embankment foundation drain system collected 1.1 l/s.
- Seepage loss through the foundation was 1.8 l/s.

The results from Case 2, with the upstream toe drains not functioning, indicate a total seepage of 4.2 l/s The flow contribution made by each of the components is as follows:

- The chimney drain system collected 1.2 l/s.
- The embankment foundation drain system collected 1.1 l/s.
- Seepage loss through the foundation was 1.9 l/s.

Construction of the upstream toe drains will reduce the expected losses into the groundwater system. Seepage into the upstream drains will also contribute to the consolidation of the tailings and decrease the hydraulic conductivity of the tailings mass.

The seepage rates presented above are expected maximum incremental values which occur late in the project. However, during the early years of operation, seepage rates are expected to be lower, particularly at the Perimeter and South Embankments where the natural groundwater table provides complete hydraulic confinement during the first year. As the tailings surface rises, the seepage rate is expected to gradually increase to the maximum values presented above. Following closure, seepage rates will decrease as the supernatant pond becomes remote to the embankment and the tailings continue to consolidate.

The results of the sensitivity analysis to determine the effects of a higher horizontal conductivity within the Zone S and the Zone B indicate that a conductivity ratio of 0.1 within these zones would increase the total seepage through the embankment by 40 percent. However, the seepage losses into the groundwater system would actually be reduced and the additional flow would be collected by the chimney drain system and the foundation drain system.

5.7 STABILITY ANALYSES

5.7.1 General

Embankment stability analyses were conducted using the limit equilibrium computer program SLOPE/W. This program performs a systematic search to obtain the minimum factor of safety from a number of potential slip surfaces. Factors of safety were computed using Bishop's Simplified Method of Slices. The conditions considered during stability analyses are summarized in Table 5.8 and outlined below.

Downstream Stability - Analyses were performed to investigate the downstream stability of the Stage 2 Main Embankment, the Final Main Embankment during operations, and the Post Closure Main Embankment for the following conditions:

- Static conditions during operations and post-closure. Minimum acceptable factors of safety of 1.3 (during operations) and 1.5 (post-closure) have been adopted for these cases.
- Earthquake loading during operations and post-closure. The stability of the embankment under earthquake loading was analyzed using the pseudostatic method, by applying a horizontal seismic coefficient (acceleration) to the potential sliding mass. Factors of safety greater than 1.0 imply that there will be no deformations of the embankment initiated by earthquake loading. For conditions during operations, a seismic coefficient of 0.04 was used to represent the Design Basis Earthquake (as determined by the hazard classification for the Tailings Storage Facility). A conservative seismic coefficient of 0.065 was used to represent the Maximum Design Earthquake, for post-closure (long-term) conditions.
- Residual (post-liquefaction) tailings strength conditions. A preliminary tailings liquefaction analysis indicates the tailings will not liquefy during operations if subjected to the Design Basis Earthquake with a magnitude of 6.0. However, for post-closure conditions a

Maximum Design Earthquake with a magnitude of 9.0 has been considered. This earthquake has the potential to initiate liquefaction within the tailings due to the long duration of ground shaking associated with such an event. Therefore, for post-closure conditions, liquefaction of the entire depth of tailings was assumed. Good.

The tailings were assumed to be partially consolidated during operations (based on the results of the consolidation analysis) and an appropriate undrained shear strength was assigned to the tailings. Tailings effective strength parameters were used for the long-term post-closure condition when complete consolidation has been achieved.

The location of the phreatic surface was based on seepage analysis for the cases with the upstream toe drains functioning and with the upstream toe drains not functioning. Both cases were considered in the analyses.

Upstream Stability - The upstream stability of the Stage 2 Main Embankment, the Final Main Embankment, and the Post Closure Main Embankment has also been evaluated.

The influence of construction pore pressures on embankment stability has been previously considered. These results are presented in Section 5.7.3.

5.7.2 Material Parameters and Assumptions

The following parameters and assumptions were used in the stability analyses:

- Bulk unit weights for the embankment and foundation materials are based on Stage 1 Record and Control Test results, testwork conducted on representative samples as part of the 1995 geotechnical investigations, and typical values for similar materials. An average bulk unit weight for the tailings deposit adjacent to the embankment was estimated from the results of consolidation analysis.

- Partially consolidated tailings during operations were assigned typical undrained shear strengths ranging from 10 kPa to 55 kPa at depth. For fully consolidated tailings an average effective friction angle of 30° was adopted. These are based on lower bound strengths from in-situ Shear Vane and Cone Penetration Testing obtained at other mine sites for similar tailings materials.
- An undrained shear strength of 10 kPa was conservatively adopted to represent the residual (post-liquefaction) strength of the tailings. This is based on lower bound values obtained for similar tailings and is also consistent with lower bound data presented by Seed (1990) for the residual undrained shear strength of sand.
- An average effective friction angle of 30° was adopted to represent the coarse beach tailings beach underlying on-going embankment raises. These coarser, more free draining tailings will consolidate rapidly. Modelling has shown that these tailings achieve complete consolidation shortly after placement of the embankment raise.
- Effective strength parameters for the embankment fill and foundation materials were obtained from consolidated-undrained triaxial testwork performed on representative samples obtained during the 1995 geotechnical investigations.
- An undrained shear strength of 85 kPa was adopted to represent the strength of the top two metres of the Stage 1 and 2 foundation soils during Stage 2 analysis. This value is based on the lower third bound strength obtained from 1996 cone penetration tests.
- An effective friction angle of 26° was used to represent the strength parameters of the top two metres of the Final Main Embankment foundation soils. These strength parameters account for long-term consolidation conditions of the foundation soils. This value was based on the consolidated undrained triaxial testwork performed on glacial till samples obtained during the 1995 geotechnical investigations. a bit high?
- An effective friction angle of 40° was adopted for the free draining fill based on typical values for similar materials.
- A hydrostatic pore pressure of 1.5 metres above ground was applied to the foundation soils on the downstream side of the embankment. This

piezometric condition has been added to simulate baseline artesian pore water pressures within the foundation materials. This value is based on the initial readings from the foundation piezometers, the pore pressure dissipations from the cone penetration tests and the observation of nearby groundwater monitoring wells.

The geometry, material parameters and location of the phreatic surface for the Stage 2 Main Embankment and the Final and Post Closure Main Embankment stability analyses are illustrated on Figures 5.11 and 5.12 respectively.

5.7.3 Results of Analyses

The results of the stability analysis are summarized in Table 5.9 and outlined below.

Stage 2 Main Embankment Stability Analysis- The factors of safety of the downstream slope of the Stage 2 Main Embankment are 1.67 for the static condition and 1.44 under seismic loading. Similarly, results of analysis of the upstream slope indicates the factors of safety are also within acceptable levels. The factors of safety for the upstream slope are 1.64 under static conditions and 1.49 under seismic loading conditions. The results of Stage 2 stability analysis is illustrated on Figure 5.13.

Final Main Embankment Downstream Stability Analysis - For the static case during operations a minimum factor of safety of 1.58 was calculated when the toe drains are functioning and 1.55 when the toe drains are not functioning. Under seismic loading conditions, these values are reduced to 1.42 when the toe drains are functioning and to 1.35 when the toe drains are not functioning. The Final Main Embankment downstream stability analysis results are summarized on Figures 5.14 and Figure 5.15.

Final Main Embankment Upstream Stability Analysis - Under static conditions, the factor of safety for the upstream slope of the Final Main Embankment is 1.93 when the upstream toe drains are functioning and 1.82

when the upstream toe drains are not functioning. Under seismic loading, the factor of safety with the upstream toe drains functioning is 1.67 and 1.41 with the upstream toe drains not functioning. The Final Main Embankment upstream stability analysis results are illustrated on Figures 5.16 and 5.17.

Post Closure Main Embankment Stability Analysis - For post closure stability analysis, an increase in tailings strength and a lowering of the phreatic surface resulted in factors of safety for the downstream slope of 1.77 under static conditions and 1.49 under seismic loading. Calculated values for the minimum factor of safety of the upstream slope were 2.09 for static conditions and 1.72 for seismic loading conditions. The results of post closure stability analysis are provided on Figure 5.18.

Residual (post-liquefaction) Tailings Strength Analysis - Under the worst case conditions with the upstream toe drains not functioning, the calculated factors of safety for the upstream and downstream slopes were 1.71 and 1.39 respectively. This indicates that the embankment is not dependent on tailings strength to maintain overall stability. The results of the post liquefaction analysis are provided on Figure 5.19.

A sensitivity analysis was previously conducted to evaluate the downstream static stability of the Final Main Embankment for various hydrostatic pore pressures in the foundations soils. The results, shown on Figure 5.20, indicate that a minimum Factor of Safety of 1.1 is approached as the foundation pore pressures reach a height about 8.5 metres above ground for the final embankment. The pore pressures will be monitored during embankment construction and appropriate actions will be taken to assure embankment stability.

5.8 PRECEDENTS FOR DESIGN CONCEPTS

The Mount Polley Mine tailings embankments are modified centreline zoned earthfill structures with low permeability glacial till core zones, chimney drains, upstream toe drains and downstream random fill zones constructed from mine waste rock.

The tailings embankments have been designed for staged expansion during operations in order to minimize initial capital expenditures and to maintain an inherent flexibility to allow for variations in operation and production throughout the life of the mine. Key design concepts which may be subject to review by regulatory agencies include:

- Modified Centreline Construction
- Drainage systems provided upstream of the embankment core zones.
- Drainage system pipeworks which penetrate the embankments.

Modified centreline construction is similar to conventional centreline construction in that the contact between the compacted fill and tailings slopes slightly upstream. However, it differs because no construction on the downstream face of the embankment is required. It is different from upstream construction in that the stability of the embankment is independent of tailings strength.

Modified centreline construction has been successfully used at other mines, including the Montana Tunnels Mine in Montana, the Nickel Plate and Premier Mines in British Columbia and the Alumbrera Mine in Argentina. In addition, modified centreline construction has been permitted for the Kensington Mine in Alaska. A collection of papers which discuss modified centreline construction is included in Appendix A.

The drainage system located upstream of the embankment core zone has been included to facilitate drainage of the tailings mass and to control the phreatic surface within the embankments. Upstream toe drains will be included along the full length of the embankments at selected elevations during future staged expansions. The locations and elevations of the drains will be reviewed after an observation period during operations when parameters such as the tailings characteristics, available borrow materials and the performance of the facility have been established.

Upstream drainage systems have also been successfully implemented at the above mentioned mines. It is presently anticipated that the Mount Polley upstream drainage system will initially include a Longitudinal Drain and will be similar to the Chimney Drain System.

The upstream drainage systems require conveyance pipework which penetrates the embankment core zones. The Mount Polley conveyance pipework will comprise solid HDPE pipe with seepage collars bedded in concrete. The pipework will be installed in the abutments in dense natural ground. The details and elevation of future core zone pipe penetrations will be finalized during the detailed design for the staged expansions. The Toe Drain conveyance pipe downstream of the Main Embankment ultimate toe was installed during Stage 1a/1b construction.

SECTION 6.0 - STAGE 2 CONSTRUCTION

6.1 GENERAL

Stage 2 is the first of the staged expansions for the Tailings Storage Facility, as shown on the Staged Construction and Filling Schedule on Figure 5.6. The construction stages are based on a throughput rate of 17,808 tpd, a tailings dry density of 1.1 tonnes/m^3 and a full production start-up date of August 1, 1997. The total Stage 2 expansion includes raising the embankments 6 metres to El. 940 metres and will be completed in three separate stages, as follows:

- Stage 2A includes the first modified centreline raise of 2 metres, to El. 936 metres. The Stage 2B haul road at the toe of the existing Stage 1b embankment is also included in Stage 2A construction.
- Stage 2B includes an additional raise of 2 metres to El. 938 metres and the remainder of the Stage 2 downstream work.
- Stage 2C includes a 2 metre modified centreline raise, to El. 940 metres.

The overall site plan showing the Stage 2 tailings embankments is shown on Drawing No. 10162-9-100. The Stage 2 General Arrangement is shown on Drawing No. 10162-9-101. Cross-sections of the Main and Perimeter Embankments are shown on Drawing Nos. 10162-9-102 and 103, respectively. Details of each of the Stage 2 construction programs are provided below.

6.2 STAGE 2A

Stage 2A includes raising the Main and Perimeter Embankments by 2 metres to crest El. 936 metres. The raise on the existing Stage 1b embankments will be a modified centreline raise, where fill materials will be placed on the tailings beaches adjacent to the embankments. The scope of work for Stage 2A embankment construction includes the items listed below.

- Relocate tailings pipeline as required. Pipeline moves must be scheduled to allow tailings discharge from movable discharge section to be continued for as long as possible. The M1 dump valves are to be used as a last resort.
- Survey Stage 1b embankments to evaluate settlement and/or deformations.
- Clear, strip and grub and remove topsoil or unsuitable material from the embankment extensions at the abutments (from El. 934 to El. 936 m) and on ground down to the tailings surface.
- Prepare the foundation areas for the embankment fill.
- Install vibrating wire piezometers in tailings beaches.
- Place Type 1 Geotextile Filter Fabric on the tailings beach at the Main Embankment.
- Place the coarse bearing layer on the Geotextile Filter Fabric at the Main Embankment.
- Prepare the Stage 1b embankment crests for fill placement.
- Supply, place and compact glacial till fill in Zones B and S to El. 936 m.

The Stage 2B haul road at the toe of the existing Stage 1b embankment is also included in Stage 2A construction. The scope of work for the Stage 2B Haul Road includes the items listed below.

- Clear, strip and grub and remove topsoil or unsuitable material from the foundation area for the haul road.
- Prepare the foundation area for the haul road.
- Install outlet drains OD-4, 5, and 6 at the Perimeter Embankment.
- Install foundation drain FD-5 at the Main Embankment, complete with the required pressure relief wells and trenches. Pressure relief details are shown on Drawing Nos. 10162-9-105 and 155.
- Install vibrating wire piezometers in the foundations below Zone T and in Zone T, as required.
- Place Type 2 geotextile filter fabric on the prepared and approved foundations as required.
- Supply, place and compact Zone T material for the haul road.
- Install inclinometers in the Main Embankment foundations as required.

The haul road is within the footprint of future embankment raises and therefore must be constructed to the same standards for Stage 2A embankment construction. Stage 2A construction is planned for early 1998. Winter construction methods established during Stage 1 construction will be followed. Details for Stage 2A construction are shown on Drawing Nos. 10162-9-104, 105, 110, 111, 120 and 121.

6.3 STAGE 2B

Stage 2B includes raising the Main and Perimeter Embankments by 2 metres to El. 938 metres. It also includes the remainder of the downstream work for Stage 2. The scope of work for Stage 2B construction includes the items listed below.

- Relocate tailings pipeline as for Stage 2A construction.
- Clear, strip and grub and remove topsoil or unsuitable material from the embankment extensions at the abutments (from El. 936 to El. 938 m.).
- Prepare the foundation areas for the embankment fill.
- Install foundation drains FD-6 and 7 at the Main Embankment, complete with the required pressure relief wells and trenches.
- Place Type 2 geotextile filter fabric on the prepared and approved foundations as required.
- Supply, place and compact Zone T and C materials.
- Extend outlet drains OD-1, 2 and 3 at the Main Embankment.
- Install Longitudinal at the Perimeter Embankment and extend Longitudinal Drain at the Main Embankment.
- Extend Chimney Drains to El. 936 m.
- Prepare the Stage 2A embankment crests for fill placement.
- Supply, place and compact glacial till fill in Zones B and S and extend the Longitudinal and Chimney Drains to El. 938 m.
- Install vibrating wire piezometers in fill zones as required. Extend the piezometer leads to the instrumentation monitoring huts.

Stage 2B construction is planned for mid 1998.

6.4 STAGE 2C

Stage 2C includes raising the Main and Perimeter Embankments by 2 metres to crest El. 940 metres. The raise on the Stage 2B embankments will be a modified centreline raise, where fill materials will be placed on the tailings beaches adjacent to the embankments. The scope of work for Stage 2C embankment construction includes the following items:

- Relocate tailings pipeline as required for Stages 2A and 2B.
- Survey Stage 2B embankments to evaluate settlement and/or deformations.
- Clear, strip and grub and remove topsoil or unsuitable material from the embankment extensions at the abutments (from El. 938 to El. 940 m) and on ground down to the tailings surface.
- Prepare the foundation areas for the embankment fill.
- Place Type 1 Geotextile Filter Fabric on the tailings beaches..
- Place the coarse bearing layer on the Geotextile Filter Fabric.
- Prepare the Stage 2B embankment crests for fill placement.
- Supply, place and compact glacial till fill in Zones B and S to El. 940 m.
- Extend Chimney Drains and Zone T and C materials to El. 940 m.

Stage 2C construction is planned for late 1998 or mid 1999. Winter construction methods established during Stage 1 construction will be followed if construction takes place in freezing conditions. Construction details for Stage 2B and 2C are shown on Drawing Nos. 10162-9-130, 131, 140 and 141.

SECTION 7.0 - PIPEWORK

7.1 GENERAL

A brief description of the pipework and pump systems required to operate the tailings and reclaim pipelines and the seepage recovery systems is included in this section. Detailed descriptions of the pipework are presented in the Knight Piésold documents "Operation, Maintenance and Surveillance Manual for Stage 1b Embankment (El. 934m), Ref. No. 10162/7-3" and "Tailings Storage Facility, Updated Design Report, Ref. No. 1627/2".

7.2 TAILINGS PIPEWORK

The tailings pipeline extends approximately 7,000 metres from the Millsite to the right abutment of the Main Embankment. The system is designed for gravity flow for the full mine life, to the final tailings embankment crest El. 965 metres. The pipeline has a continuous downhill grade to ensure it is free draining and to prevent potential sanding and freezing problems. The pipe diameter was selected for gravity flow over a range of operating conditions. All pipework is butt fusion welded High Density Polyethylene (HDPE) pipe of varying diameter. Pipe wall thickness (pressure rating) was selected to accommodate the anticipated operating pressures and vacuum conditions and includes an allowance for internal abrasive wear.

A dropbox (T2) is provided for surge protection and to allow the addition of waste dump runoff from the Southeast Sediment Pond to the tailings stream. The dropbox also functions as an overflow for the reclaim booster sump.

Spill containment is provided for the full length of all pipelines. The pipelines are buried through the Millsite area and are laid in a pipe containment channel cut in or lined with glacial till from the Millsite to the Tailings Storage Facility. The pipelines are sleeved at the Bootjack Creek crossing for additional spill containment. How effective

The tailings pipeline has two sections, with different pressure ratings and diameters. The first section extends from the Millsite to the T2 Dropbox and is comprised of 22 inch (556 mm) DR 17 HDPE pipe. The second section extends from the T2 Dropbox to the Tailings Storage Facility and comprises 24 inch (610 mm) DR 15.5 HDPE pipe. Two sections of 30 inch (762 mm) DR 15.5 HDPE pipe are also included at the start of the two pipeline sections (at the Millsite and at the T2 Dropbox) to ensure that flows are not restricted at the inlets.

The pipeline runs along the inside crest of the embankment at the Tailings Storage Facility. It is provided with a movable discharge section with six 150 mm offtakes that will allow controlled deposition of tailings over the length of the embankment. The pipeline has a number of flanged connections where the movable discharge section can be installed. The tailings pipeline is secured on the embankment crest by straps and concrete blocks or guide posts to restrict thermally induced movements.

For the first year of operations, discharge will be concentrated from the Main Embankment at the deepest part of the impoundment to establish the tailings beach, and from the right abutment of the Main Embankment to cover the Upper Basin Liner. Additional discharge will be provided at the M1 dump valves, as required. After the tailings beach is established at the Main Embankment, discharge will be rotated so that tailings beaches are established over the full length of the Perimeter and Main Embankments. Following construction of the South Embankment during Stage 3, a bifurcation will be added to tailings pipeline and a new pipeline section will be installed along the South Embankment. Tailings deposition will be concentrated from the South Embankment at this time so as to blanket the near surface bedrock with a layer of low permeability tailings. Good

7.3 RECLAIM PIPEWORK

The reclaim system was designed to provide adequate pipeline and pumping capacity to recycle process water from the Tailings Storage Facility to the Millsite so as to meet process requirements. Reclaim pipework includes the reclaim pipeline, a reclaim booster pump station a pump barge in the Tailings Storage Facility. All pipework except a 300 metre stretch of steel pipe at the reclaim barge is butt fusion welded High Density Polyethylene (HDPE) pipe of varying diameter. Pipe wall thickness (pressure rating) was selected to accommodate the anticipated operating pressures.

The reclaim pipeline was constructed in two sections. The first section extends from the pump barge to the booster pump station and includes approximately 300 metres of steel pipe at the pump barge. The remainder is HDPE pipe which decreases in thickness (pressure rating) as the booster pump station is approached and the pressure head is decreased. The second pipe section is similar to the first, but does not have any steel pipe sections. Nominal 24 inch (610 mm) HDPE pipe with varying pressure ratings was selected to provide the required water transfer capacity.

The reclaim booster pump station was built at the midpoint of elevation to reduce pressure rating requirements.. An inter-linked control system co-ordinates pump operations with process water demand at the millsite. The control system and pipework design will include the necessary provisions for spill prevention.

The reclaim barge is a prefabricated floating pump station complete with perimeter trash screens, internal wet well(s), pump(s), valving, piping, electrical power, instrumentation and control circuitry. A hinged walkway/pipe bridge is provided for access to the barge from the side of the reclaim barge channel. The reclaim barge was designed by Others. Identical pumps will be used at the barge and booster station to reduce spare part requirements and to simplify maintenance.

7.4 SEEPAGE RECOVERY SYSTEMS

Seepage recovery systems return seepage water collected from the foundation drains, chimney drains and upstream toe drains to the tailings impoundment. Seepage recovery systems have been installed at the Main and Perimeter Embankments. An additional system will be installed at the South Embankment in the future.

The seepage recovery systems include seepage recycle sumps and seepage recycle pumps and pipelines. Seepage recycle sumps have been installed at the Main and Perimeter Embankment Seepage Collection Ponds. They house the seepage recycle pumps, which are connected to six inch diameter HDPE pipes that extend from the pumps to the crest of the tailings embankment. Seepage water discharges directly onto the tailings beach.

SECTION 8.0 - INSTRUMENTATION AND MONITORING

8.1 GENERAL

Instrumentation and monitoring are essential to evaluate the performance of the embankments and associated structures and to detect abnormal conditions relevant to dam safety. A detailed description on the instrumentation and monitoring requirements is presented in the Knight Piésold document "Operation, Maintenance and Surveillance Manual for Stage 1b Embankment (El. 934m), Ref. No. 10162/7-3". Maintenance and inspection requirements are also described in this document.

8.2 MONITORING PROGRAM

The monitoring program described in the Operation, Maintenance and Surveillance Manual for Stage 1b Embankment includes the following:

a) Measurement of the rate of filling with water and/or tailings. + elevation
b) Measurement of the Foundation Drain flow quantities and sampling for water quality analyses.
c) Monitoring of the Chimney Drain outlets. ?
d) Monitoring of the vibrating wire piezometers.
e) Monitoring of Survey Monuments and Control Points.
f) Monitoring of water levels in groundwater monitoring wells.
g) Sampling of groundwater monitoring wells for water quality analyses.
h) Sampling of surface water streams for water quality analyses.
i) Sampling of process water in the tailings pond and seepage recycle ponds for water quality analyses.
j) Flow monitoring in diversion ditches, runoff collection ditches, and Polley Lake Pumping System.
k) Meteorological (rain, snow, evaporation) and air quality data collection.

Monitoring and reporting frequencies and contingency procedures for all components of the Tailings Storage Facility are also provided in the Operation, Maintenance and Surveillance Manual.

8.3 RESULTS TO DATE

A plot of the up to date Main Embankment foundation drain flows is presented on Figure 8.1. The plot shows that the flows have continued to remain low, with total flows typically below 0.5 litres/second, even though the pond level is rising. This indicates that the impounded water has not significantly influenced the underlying soils and that the glacial till liner (natural and constructed basin liner) is working to seal off the tailings basin.

Summary plots for the four piezometer planes are shown on Figures 8.2, 8.3, 8.4 and 8.5. The results from each instrumentation plane are discussed below.

Plane A (Main Embankment Ch. 20+00)

- Piezometers in the drain zones include A1-PE1-01, A1-PE1-02 (foundation drains) and A1-PE1-03 (chimney drain). All pore pressures are below zero, indicating that the drains are unimpeded and functioning well. Minor fluctuations have occurred since installation.
- Piezometers in the foundation soils include A2-PE2-01 and A2-PE2-02. A2-PE2-01 is deeper (9 m) and is installed in the fine grained glaciolacustrine sediments. It has approx. 11.5 m excess pore pressure, which is about 2.5 m above ground (artesian), an increase of approx. 2 m since installation. A2-PE2-02 is shallower (2.9 m) and is also in the fine grained glaciolacustrine sediments. There is approx. 2 m excess pore pressure, (not artesian). It is relatively unchanged since installation , with only minor fluctuations.
- Piezometers in the embankment fill zones include A2-PE2-03, A2-PE2-04 and A2-PE2-05. All piezometers showed significant pore pressure increases during fill placement. A2-PE2-03 is slowly dissipating and currently has approx. 8 m excess pore pressure. A2-PE2-04 increased dramatically after installation and fill placement and stopped working shortly after installation. A2-PE2-05 is fully dissipated and is showing very little excess pore pressure (0.15 m).

Plane B (Main Embankment Ch. 22+40)

- Piezometers in the drain zones include B1-PE1-01, B1-PE1-02 (foundation drains) and B1-PE1-03 (chimney drain). All pore pressures are below zero, indicating that the drains are unimpeded and functioning well. Minor fluctuations have occurred since installation.
- Piezometers in the foundation soils include B2-PE2-01 and B2-PE2-02. B2-PE2-01 is deeper (15 m) and is installed in a sandy glaciofluvial layer. There is approx. 14.1 m excess pore pressure (not artesian), an increase of approx. 2.7 m since installation. B2-PE2-02 is shallower (7.9 m) and is also a sandy layer in the glaciofluvial sediments. There is approx. 8.1 m excess pore pressure, which is about 0.2 m above ground (artesian), an increase of about 3.2 m since installation.
- Piezometers in the embankment fill zones include B2-PE2-03, B2-PE2-04 and B2-PE2-05. All piezometers showed significant pore pressure increases during fill placement. B2-PE2-03 increased dramatically after installation and fill placement and has approx. 16.8 m excess pore pressure. B2-PE2-04 increased dramatically after fill placement and is still dissipating. It currently has approx. 5.7 m excess pore pressure. B2-PE2-05 is fully dissipated and is showing zero excess pore pressure.

Plane C (Main Embankment Ch. 18+50)

- Piezometers in the drain zones include C1-PE1-01 and C1-PE1-02 (foundation drains). All pore pressures are below zero, indicating that the drains are unimpeded and functioning well. Minor fluctuations have occurred since installation.
- Piezometers in the foundation soils include C2-PE2-01 and C2-PE2-02. C2-PE2-01 is deeper (8.2 m) and is installed in a sandy layer in the glaciofluvial/ glaciolacustrine sediments. There is approx. 12.4 m excess pore pressure, which is about 4.1 m above ground (artesian), an increase of approx. 1.8 m since installation. C2-PE2-02 is shallower (5.2m) and is also a sandy layer in the glaciofluvial/ glaciolacustrine sediments. This piezometer has stopped functioning. There was approx. 6.3m excess pore pressure, which is about 1.1

m above ground (artesian) when it stopped functioning. This was an increase of approx. 0.7 m.

- Piezometers in the embankment fill zones include C2-PE2-03 and C2-PE2-05. Both piezometers showed significant pore pressure increases during fill placement. C2-PE2-03 increased dramatically after installation and is fully dissipated, with a pore pressure below zero C2-PE2-05 increased after fill placement. It currently has approx. 0.9 m excess pore pressure.

Plane D (Perimeter Embankment Ch. 39+86)

- One piezometers has been installed at the Perimeter Embankment. Piezometer D2-PE2-01 was installed in the embankment fill. It is showing approx. 0.3 m excess pore pressure and is essentially unchanged since installation.

In summary, monitoring to date has shown that:

- The drain piezometers are all showing pore pressures are below zero, indicating that the drains are functioning well.
- Pore pressures in the foundation soils have typically increased by 2 to 3 m, due to the loading from the embankment and impounded tailings and water. Only C2-PE2-02 is exhibiting significant artesian pore pressures (4.1 m). The frequency of piezometer readings will be increased if the pressure rises closer to the trigger level (6 m artesian).
- Embankment fill piezometers responded quickly to the placement of fill materials and were monitored accordingly. Some high pressures were observed because of the piezometer installation method, where the saturated tips were immersed in a loose slurry in a small hole and were then quickly loaded. These pore pressures are not considered to be indicative of general pore pressure conditions in the embankment fill, but only provide an indication of the confined slurry pressure at the piezometer tip. The high pressures are slowly dissipating and illustrate the low permeability nature of the surrounding fill.

Trigger values have been established for all piezometers, as discussed in the Knight Piésold documents "Operation, Maintenance and Surveillance Manual for Stage 1b Embankment (El. 934m), Ref. No. 10162/7-3" and "Tailings Storage Facility, Updated Design Report, Ref. No. 1627/2". These values, if exceeded, will require that investigations and contingency or remedial actions be taken.

8.4 ON-GOING MONITORING REQUIREMENTS

The established monitoring program must be followed for the life of the facility. In the future, if very good results are continually obtained, some of the monitoring frequencies may be reduced. However, the Design Engineer must approve any modifications to the monitoring frequencies.

For Stage 2 construction, an another instrumentation plane will be added at the Main Embankment, near the right abutment (Ch. 17+60) where artesian pore pressures from the glaciofluvial/glaciolacustrine sediments have been identified. A total of 28 new vibrating wire installations are planned, including 2 in the tailings beach at Planes A, B, C and D. Also, additional foundation piezometers will be installed in boreholes at Planes A and C. The piezometers will be closely monitored during embankment fill placement. Placement rates will be modified as required to ensure that any excess pore pressures which may be generated during fill placement have dissipated before additional fill materials are placed. Instrumentation details for Stage 2 are shown on Drawing Nos. 10162-9-150, 151, 152, 153 and 154.

Inclinometers are to be installed just past the final toe of the Main Embankment at Planes A, B and C during Stage 2. The inclinometers will also require weekly monitoring.

Embankment crest surveys will be completed before and after construction of each of Stage 2A, 2B and 2C to evaluate deformation and settlement.

SECTION 9.0 - CLOSURE AND RECLAMATION

9.1 GENERAL

In accordance with requirements under the B.C. Mines Act and Health, Safety and Reclamation Code for Mines in British Columbia, the primary objective of the proposed Reclamation Plan will be to "return all mine-disturbed areas to an equivalent level of capability to that which existed prior to mining on an average property basis, unless the owner, agent or manager can provide evidence which demonstrates to the satisfaction of the chief inspector the impracticality of doing so". The following goals are implicit in achieving this primary objective for the Tailings Storage Facility:

- Long-term preservation of water quality within and downstream of decommissioned operations.
- Long-term stability of the tailings impoundment.
- Removal of all access roads, ponds, ditches, pipelines, structures and equipment not required after the mine closes.
- Long-term stabilization of all exposed materials that are susceptible to erosion.
- Natural integration of disturbed lands into surrounding landscape, and restoration of the natural appearance of the area after mining ceases, to the greatest possible extent.
- Establishment of a self-sustaining vegetative cover consistent with existing forestry, grazing, wildlife and outdoor recreation needs.

As an overall approach to achieving these objectives, the Reclamation Plan is sufficiently flexible to allow for future changes in the mine plan and to incorporate information obtained from ongoing reclamation research programs such as trial tailings re-vegetation plots.

The detailed Reclamation Plan for the Mount Polley Mine is presented in the Hallam Knight Piésold document "The Mount Polley Mine Project Reclamation Plan".

9.2 DECOMMISSIONING AND CLOSURE

Testwork on the tailings has indicated that the tailings solids will not be acid-generating. Therefore, no special remediation measures will be required. The general concept is that the surface of the tailings impoundment will be decommissioned as a mixed forested/wetlands complex with a gradual transition towards a ponded area with an overflow spillway. The downstream face of the tailings embankments will be covered with topsoil from stockpiles and revegetated progressively during operations to the greatest extent possible, once the final toe position and slope have been established.

At mine closure, surface facilities will be removed in stages, salvaged and sold. The tailings delivery system will be dismantled and removed immediately following cessation of operations. The reclaim barge, pumps and pipeline will be utilized for supplementary flooding of the open pit and will then be dismantled and removed. The seepage collection ponds and recycle pumps will be retained for a period after closure until monitoring results indicate that tailings area seepage is of suitable quality for direct release to the environment. At that time, the seepage collection pond and recycle pumps will be removed. The groundwater monitoring wells and piezometers in the tailings embankment will be retained for long term monitoring.

Before flooding the wetlands complex to the required pond elevation, the area along the final water level will be sculptured using conventional earthmoving equipment to create a series of small bays and channels which will become a margin environment conducive to the creation of waterfowl breeding and staging habitat. The tailings embankments and the upland portions of the exposed tailings beach will be covered with a layer of topsoil from stockpiles and revegetated with indigenous species of conifer and deciduous trees and willow and marsh land grasses. The moist transition zone between the topsoiled beach and final pond will be revegetated as an early seral stage meadow, leading to aquatic tolerant, emergent and submerged species of plant. Native vegetation species that are accustomed to swampy areas will be utilized for these transition zones. Where necessary, the final tailings surface will be treated with amendments suitable for sustaining permanent growth. The shoreline will then be planted with native emergent plant species for cover. The expected species will be transplanted from nearby wetlands of a similar aspect and elevation or propagated from root cuttings, turf squares or offsets.

A spillway will be constructed to accommodate the Probable Maximum Flood (PMF) flood flows within the tailings basin. The spillway will be constructed in competent ground along the northwest side of the Tailings Storage Facility and will discharge to the Edney Creek north tributary drainage. The elevation of this spillway and outflow channel will be designed to establish a set water elevation over the tailings surface (approximately 15% coverage).

Final seeding of the embankment slopes with grasses and legumes will provide a stable vegetation mat that resists erosion. Once open pit flooding is complete, the surface water diversion system will be dismantled to allow for natural runoff to be routed through the tailings area.

The advice of organizations such as the B.C. Fish and Wildlife Branch, Ducks Unlimited and local trappers/guided outfitters will be sought during final design and implementation of the Reclamation Plan.

SECTION 10.0 - REFERENCES

The following select Knight Piésold documents provide background information to support this report:

1. Mount Polley Mining Corporation, Mount Polley Project, Tailings Storage Facility, Operation, Maintenance and Surveillance Manual for Stage Ib Embankment (El. 934 m), Ref. No. 10162/7-3, November 24, 1997.

2. Mount Polley Mining Corporation, Mount Polley Mine, Tender Documents for Stage 2A Tailings Facility Construction, Ref. No. 10162/9-1, November 11, 1997.

3. Mount Polley Mining Corporation, Mount Polley Mine, Stage 2A Tailings Facility Construction, Selected Excerpts from Reference Information, Ref. No. 10162/9-2, November 6, 1997.

4. Mount Polley Mining Corporation, Mount Polley Project, Tailings Storage Facility, Report on Stage Ia/Ib Construction, Ref. No. 10162/7-5, August 14, 1997.

5. Mount Polley Mining Corporation, Mount Polley Project, Tailings Storage Facility, Updated Design Report, Ref. No. 1627/2, June 6, 1997.

6. Imperial Metals Corporation, Mount Polley Project, Tailings Storage Facility, Design Report, Ref. No. 1625/1, May 26, 1995.

Other references include the following:

1. Hallam Knight Piésold Ltd., "Imperial Metals Corporation, The Mount Polley Mine Project Reclamation Plan, April, 1996 ".

2. LaVassar, J.M. and Ordonez, G.A., (1991), "Leader Lake Seismic Risk Assessment," OFTR 91-6, Water Resources Program, Dam Safety Section, Washington State Department of Ecology.

Bruce S. Brown, P.Eng.
Director

Ken D. Embree, P.Eng.
Senior Engineer

TABLE 2.1

MOUNT POLLEY MINING CORPORATION
MOUNT POLLEY PROJECT

MEAN MONTHLY AND ANNUAL PRECIPITATION

Location:	Likely, B.C.		Mine Site		Barkerville	
Elevation:	724 m		1000 m		1265	
Location:	52° 36'N 121° 32'W		52° 30'N 121° 35'W		53° 4'N 121° 31'W	
	Mean (mm)	Std. Dev. (mm)	Mean (mm)	Std. Dev. (mm)	Mean (mm)	Std. Dev. (mm)
Jan	74.2	27.0	75.5	27.0	103.0	44.4
Feb	60.2	27.7	58.1	27.7	85.6	42.5
Mar	37.8	13.5	44.5	13.5	85.3	29.1
Apr	42.2	20.9	43.1	20.9	61.8	24.5
May	36.6	15.4	50.6	15.4	65.9	28.9
June	66.3	29.7	81.5	29.7	89.2	28.8
July	47.0	27.4	65.7	27.4	81.7	31.0
Aug	82.0	35.7	83.1	35.7	102.3	53.0
Sept	50.4	27.1	60.4	27.1	85.4	39.9
Oct	61.6	42.3	60.4	42.3	88.4	37.4
Nov	58.4	18.8	57.3	18.8	86.6	28.2
Dec	83.0	36.9	74.8	36.9	108.7	42.5
Annual	699.7	116.4	755	116.4	1043.9	112.7

data source?

Source :

Canadian Climate Normals, 1951-1980, Temperature and Precipitation Atmospheric Environment Service, Environment Canada.

TABLE 2.2

Knight Piésold Ltd.
CONSULTING ENGINEERS

MOUNT POLLEY MINING CORPORATION
MOUNT POLLEY PROJECT

PRECIPITATION DETAILS USED IN ANALYSIS

J:\JOB\REPORT\10162-9\3-TBL2-2.XLS 11/27/97 15:29

DESCRIPTION	VALUE			
Lower Elevations (ie. TSF)				
Mean annual precipitation (mm)	755			
"Dry" annual precipitation (mm)	601			
"Wet" annual precipitation (mm)	909			
"Max." annual precipitation (mm)	1050			
"Min." annual precipitation (mm)	450			
Mean annual rainfall (mm)	451			
Mean annual snowfall (mm)	304			
Coefficient of variation	0.16			
Standard deviation (mm)	121			
Higher Elevations (ie. mill site, waste dumps, etc.)				
"Elevation" factor	1.07285			
Mean annual precipitation (mm)	810			
"Dry" annual precipitation (mm)	645			
"Wet" annual precipitation (mm)	975			
Coefficient of variation	0.16			
Standard deviation (mm)	130			
Proportions of Total Precipitation:				
Rainfall	0.60			
Snowfall	0.40			
Monthly Proportions of Precipitation:				
	Rainfall (mm)	Proportion as Rainfall	Snowfall (mm)	Proportion as Snowfall
Oct	48.3	0.11	12.1	0.04
Nov	17.3	0.04	40.0	0.13
Dec	7.6	0.02	67.2	0.22
Jan	6.8	0.02	68.7	0.23
Feb	6.0	0.01	52.1	0.17
Mar	6.0	0.01	38.5	0.13
Apr	24.2	0.05	18.9	0.06
May	45.3	0.10	5.3	0.02
Jun	81.5	0.18	0.0	0.00
Jul	65.7	0.15	0.0	0.00
Aug	83.1	0.18	0.0	0.00
Sep	58.9	0.13	1.5	0.00
Total (mm)	450.7		304.3	

TABLE 2.3

MOUNT POLLEY MINING CORPORATION
MOUNT POLLEY PROJECT

PROBABLE MAXIMUM PRECIPITATION

1 hour PMP	= 78 mm	= 78 mm/hour
6 hour PMP	= 88 mm	= 14.6 mm/hour
24 hour PMP	= 203 mm	= 8.5 mm/hour
10 day PMP	= 406 mm	= 1.7 mm/hr

Source :

Rainfall Frequency Atlas for Canada, W.D. Hogg, D.A. Carr, Supply and Services Canada 1985.

Note:

1. 24 hr. PMP value conservatively assumes an orographic factor of 1.5.
2. 10 day PMP value assumes a 10 day to 24 hour PMP ratio of 2.0.

TABLE 2.4

MOUNT POLLEY MINING CORPORATION
MOUNT POLLEY PROJECT

USUAL MINIMUM CRITERIA FOR DESIGN EARTHQUAKES

Consequence Category	Maximum Design Earthquake (MDE)	
	Deterministically Derived	Probabilistically Derived (Annual exceedence probability)
Very High	MCE[a][b][c]	1/10,000[b][c]
High	50% to 100% MCE[d][e]	1/1000 to 1/10,000[e]
Low	[f]	1/100 to 1/1000[f]

[a] For a recognized fault or geographically defined tectonic province, the Maximum Credible Earthquake (MCE) is the largest reasonably conceivable earthquake that appears possible. For a dam site, MCE ground motions are the most severe ground motions capable of being produced at the site under the presently known or interpreted tectonic framework.

[b] In Hydro-Quebec's practice, the MDE for Very High Consequence structures involves a combination of deterministic and probabilistic approaches that reflect current knowledge of seismo-tectonic conditions in Eastern Canada. Hydro-Quebec's deterministically derived MDE magnitude is the maximum historically recorded earthquake, increased by one-half magnitude, while their probabilistically derived earthquake has an estimated probability of exceedence of 1/2000.

[c] An appropriate level of conservatism shall be applied to the factor of safety calculated from these loads, to reduce the risks of dam failure to tolerable values. Thus, the probability of dam failure could be much lower than the probability of extreme event loading.

[d] MDE firm ground accelerations and velocities can be taken as 50% to 100% of MCE values. For design purposes the magnitude should remain the same as the MCE.

[e] In the High Consequence category, the MDE is based on the consequences of failure. For example, if one incremental fatality would result from failure, an AEP of 1/1000 could be acceptable, but for consequences approaching those of a Very High Consequence dam, design earthquakes approaching the MCE would be required.

[f] If a Low Consequence structure cannot withstand the minimum criteria, the level of upgrading may be determined by economic risk analysis, with consideration of environmental and social impacts.

TABLE 5.1

Knight Piésold Ltd.
CONSULTING ENGINEERS

MOUNT POLLEY MINING CORPORATION

MOUNT POLLEY PROJECT

DESIGN BASIS AND OPERATING CRITERIA

j:\job\report\10162-9\3-tbl5-1.doc

ITEM	DESIGN CRITERIA
1.0 GENERAL DESIGN CRITERIA	
Regulations	MEI, MELP (Water Management Branch)
Codes and Standards	ASTM, ACI, ANSI, CSA, CDSA, HSRC (Health, Safety and Reclamation Code for Mines in BC), NBC and related codes
Design Operating Life	14 Years
Tailings Production Information	17,808 tonnes/day, 35% solids, 2.78 SG, 81.3 million tonnes total production, 1.28 tonnes/m^3 final average tailings dry density
Hazard Rating: During Operations After Closure	 LOW by CDSA Hazard Classification HIGH by CDSA Hazard Classification
Site Elevation	910 to 1150 metres
Climate	Average Annual Rainfall = 755 mm, Annual Evaporation = 423 mm, Mean Annual Temp = 4.0 C (Likely), Design 24 hour PMP storm = 203 mm.
Design Floods and Freeboard: During Operations: After Closure:	 Sufficient freeboard to store 1 in 10 year 24 hour PMP on top of maximum pond volume. Additional 1 m freeboard provided. No spillway. Final spillway in place, freeboard to pass the Probable Maximum Flood (PMF) in the tailings basin.
Design Earthquakes: During "Operations: Design Basis Earthquake (DBE) Maximum Design Earthquake (MDE) After Closure: Maximum Credible Earthquake (MCE):	 1 in 475 Year Event (M = 6.5, A max. = 0.037 g). 50% of the 1 in 2500 Year Event or MCE (M = 6.5, A max. = 0.065 g). 1 in 2500 Year Event (MCE).
Seepage Control	Glacial Till Liners (natural and constructed) in basin, with Foundation Drain System below Main Embankment. Seepage reports to Seepage Collection Ponds.
Tailings Pipework	Butt fusion welded HDPE pipe, gravity flow, discharge predominantly from embankment, spill containment by gravity flow to tailings basin.
2.0 TAILINGS BASIN	
Site Selection	See Section 4.0 of 1627/2 and based on: • Capacity and filling characteristics. • Hydrology and downstream water usage. • Hydrogeology and groundwater regime. • Aesthetics and visual impact. • Foundation conditions. • Construction requirement. • Closure and reclamation requirements. • Capital and operating costs.
Geological and Geotechnical Conditions	See Section 4.0 of 10162/9-3.
Basin Liner	• Natural fine grained till, or

DESIGN BASIS AND OPERATING CRITERIA

Basin Liner (cont'd)		• Compacted glacial till with frost protection layer required in areas with <2 m in-situ glacial till. • Liner placed in 3 - 150 mm lifts. • Liner compacted to 95% Std. Proctor max. dry density (ASTM D698) at optimum moisture content minus 1% to plus 2%.
Embankment Foundation Drains		• Installed in Main Embankment Foundation. • Geotextile wrapped 1000 mm x 800 mm gravel/drain with 100 mm perforated CPT drain pipe. • Drain conveyance pipes are solid HDPE. • Discharge to Main Embankment Seepage Collection Pond via Drain Monitoring Sump.
Stripping		• Required at areas directly affected by construction (embankments, basin liners, seepage collection ponds, reclaim barge channel, stockpiles, roads etc.). • Remove organic soil to topsoil stockpiles.
3.0 TAILINGS EMBANKMENT		
Function		• Storage of tailings and process water for design life. • Provide storage for 24 hour PMP storm. • Provision for routing PMF at closure.
Embankment Crest Width		8 m starter dam and 12 m final dam.
Embankment Height (Max.):	Starter	15 m (Crest El. 927 m)
	Final	53 m (Crest El. 965 m)
Embankment Crest Length:	Starter	1000 m
	Final	4500 m
Design Tonnage		6,500,000 tpy (17,808) tpd
Solids Content of Tailings Stream		35% (before Millsite and waste dump runoff added to tailings stream)
Freeboard:	Operations	24 hour PMP event (679,000 m^3) plus 1.0m wave run-up on 2.5 million m^3 operational storage pond.
	Closure	Sufficient to provide routing of PMF plus wave run-up.
Storage Capacity		84.5 million tonnes.
Tailings Density:	Year 1	1.1 t/m^3
	Year 2	1.2 t/m^3
	Year 3-13	1.3 t/m^3
Tailings Specific Gravity		2.78
Borrow Material Properties		See Section 3.0 of 10162/7-5.
Construction Diversion		Not required.
Emergency Spillway Flows:	Operations	Not required.
	Closure	Design flow for routing PMF event.
Filling Rate		See Figures 5.1 and 5.6 from 10162/9-3.
Fill Material Properties		See Drawing No. 10162-9-104.
Compaction Requirements		See Drawing No. 10162-9-104.
Geotechnical Data		See Section 3.0 of 10162/7-5 and Section 2 of 10162/9-2.
Seepage Analysis		Section 5.6 of 10162/9-3..
Stability Analysis		Section 5.7 of 10162/9-3..

TABLE 5.1

MOUNT POLLEY MINING CORPORATION

MOUNT POLLEY PROJECT

DESIGN BASIS AND OPERATING CRITERIA

Sediment Control	Primary control from Main Embankment. Main Embankment Seepage Collection Pond provides secondary sediment control.
Seepage Control	Seepage collection ponds and pumpback well systems.
Seismic Parameters	See Section 2.3 of 10162/9-3.
Spillway Discharge Capacity	Not required during operations.
Settlement	See Section 5.5 of 10162/9-3.
Surface Erosion Protection	Re-vegetation with grasses on final embankment slope.
4.0 PIPEWORKS	
4.1 Tailings Delivery and Discharge Pipework	See Section 7.0 of 10162/9-3.
Function	Transport tailings slurry and mill site and waste dump runoff to Tailings Storage Facility (TSF).
Tailings Pipeline	• Free draining, gravity flow pipeline. • Butt fusion welded HDPE with 30" DR15.5, 22" DR17 and 24" DR15.5.
Spigots	• Movable discharge section placed on tailings embankment crest.
Flow Rate	• Design throughput 900 tonnes/hr dry solids. • Slurry solids content 35%. • Design flow 19.6 cfs ($0.55m^3/s$). Increases to 23.8 cfs ($0.67m^3/s$) at 30% solids content with addition of 4.2 cfs storm water runoff.. • Waste dump and Millsite runoff will be added to tailings stream, increasing flow and decreasing solids content.
Spill Containment: - Mill site to Bootjack Creek - Bootjack Creek Crossing - Bootjack Creek to TSF	 • Pipeline laid in pipe containment channel. There is an overflow pond for the T2 Dropbox. • Pipeline sleeved in pipe containment channel. • Pipeline laid in pipe containment channel.
4.2 Reclaim Water System	
Function	Primary source of water for milling process. (Pump and Barge System Designed by Others.)
Reclaim Barge	• Prefabricated pump station on barge in excavated channel in TSF. • Local and remote control from Millsite.
Reclaim Pipeline	• 24" pipeline with a steel section at the reclaim barge and HDPE with varying pressure ratings along length.
Reclaim Booster Pump Station	• Prefabricated pump station located between TSF and Millsite. • Identical pumps, sensors and controls as reclaim barge for ease of maintenance.
Spill Containment	• See Item 4.1 above, all same for pipelines. • Booster pump station has closed sump.
4.3 Seepage Recycle System	
Function	Return seepage and foundation drain flows to TSF.
Drain Monitoring Sumps	Flow quantity and water quality measurements on individual drains.
Seepage Collection Ponds	• Sized to hold 10 times max. weekly seepage flow quantity. • Excavated in low permeability natural soil liner, operated as groundwater sink.

Knight Piésold Ltd.
CONSULTING ENGINEERS

MOUNT POLLEY MINING CORPORATION
MOUNT POLLEY PROJECT

DESIGN BASIS AND OPERATING CRITERIA

Seepage Recycle Pumps	• Set in vertical pump sumps. • Submersible pumps, system by Others. • Pumps discharge back to TSF via 150 mm HDPE pipes.
5.0 MAKE-UP WATER SUPPLY	
5.1 General	
Function	To direct runoff from the Millsite and Southeast Sediment pond to the TSF, providing additional water for recycle to the mill. Also, to implement the Polley Lake Pump Station when and as required to meet the project Water Management Plan objectives.
5.2 Millsite Sump	
Catchment Area	Approx. 20 ha direct catchment, plus pit dewatering.
Design Storm	1.5 x 1 in 10 yr. 24 hour event runoff (6,000 m^3)
Sump Cross-Section	3:1 inside slope, 2:1 outside slope, 4m crest width.
Normal Operating Level	1102.7 m
Maximum Operating Level	1106.2 m
Flow Control Structures	See Drawing No. 1625.232 for layout details.
Discharge Pipe	300 mm HDPE DR 21 to plant or tailings line.
Flow Monitoring	None.
5.3 Southeast Sediment Pond	
Catchment Area	Approx. 150 ha direct catchment.
Design Storm	1 in 10 yr. 24 hour event runoff (25,000 m^3)
Sump Cross-Section	3:1 inside slope, 2:1 outside slope, 4m crest width.
Normal Operating Level	1054.5 m
Maximum Operating Level	1057.4 m
Flow Control Structures	See Drawing No. 1625.232 for layout details.
Discharge Pipe	250 mm HDPE DR 21 to Reclaim sump or T2 Dropbox
Flow Monitoring	None.
5.4 Polley Lake Pump Station	See Report 1628/5.
Max. Volume to be extracted	1,000,000 m^3 annually
Period for water extraction	Freshet
Max. Intake Velocity	0.11 m/s
Intake Screen Opening	0.1 inch (No. 8 Mesh wire cloth)
Spill Containment at Pump	Collection into a Holding Basin
Discharge Pipe	22 ½ inch ID, 350 ft of 19 ½ inch ID and 5200 ft of 17 ½ inch ID pipe.
Max. Flow	5,500 US GPM
Flow Monitoring	Flows in Hazeltine Creek, water level on Polley Lake, pumping hours times measured flow rate.
Security and Access	Signs for buried or submerged components, buoys attached to intake in Polley Lake.
6.0 INSTRUMENTATION AND MONITORING	
6.1 General	
Function	To quantify environmental conditions and performance characteristics of the TSF to ensure compliance with design objectives.

MOUNT POLLEY MINING CORPORATION

MOUNT POLLEY PROJECT

DESIGN BASIS AND OPERATING CRITERIA

6.2 Geotechnical Instrumentation and Monitoring	
Piezometers	• Measure pore pressures in drains, foundations, fill materials and tailings. • Vibrating wire piezometers. • Installed by qualified technical personnel. • Three instrumentation planes for Main Embankment and one for Perimeter Embankment.
Survey Monuments	• Deformation and settlement monitoring of embankments.
6.3 Flow Monitoring	• To provide data for on-going water balance calculations. • Drain flows regularly monitored. • Reclaim and seepage pump systems flow meters. • Tailings output monitored at millsite. • Stream flow monitoring.
6.4 Water Quality Monitoring	• To ensure environmental compliance. • Water quality samples taken at regular intervals from sediment ponds, drains (at drain monitor sump), groundwater monitoring wells, seepage ponds and tailings pond. • Upstream and downstream samples for impact analysis.
6.5 Hydrometeorology	• Operator weather station for input to water balance calculations. • Precipitation (rain and snow). • Evaporation. • Air quality monitoring (dust, etc.).
6.6 Operational Monitoring	• Quantify operation of tailings storage facility. • Rate of tailings accumulation in terms of mass and volume. • Tailings characteristics and water recovery. • Supernatant pond (depth, area and volume).
7.0 CLOSURE REQUIREMENTS	
7.1 General	Return impoundment to equivalent pre-mining use and productivity by establishing a wetland area adjacent to a final spillway and re-vegetating remainder of tailings surface with indigenous species of trees, shrubs and grasses adjacent to embankment grading to aquatic species along and adjacent to final pond.
7.2 Spillway	Two stage spillway with lower channel outlet designed to pass 1 in 200 yr. 24 hour flood event and upper wider outlet section designed to pass Probable Maximum Flood without overtopping embankments.

Notes:

1. The closure plan will remain flexible during operations to allow for future changes in the mine plan and to incorporate information from on-going reclamation programs.

TABLE 5.2

MOUNT POLLEY MINING CORPORATION
MOUNT POLLEY PROJECT

CONSEQUENCE CLASSIFICATION OF DAMS

J:\JOB\REPORT\10162-9\3-TBL5-2.XLS 11/28/97 11:49

Consequence Category	Potential Incremental Consequences of Failure [a]	
	Loss of Life	**Economic, Social, Environmental**
Very High	Large increase expected [b]	Excessive increase in social, economic and/or environmental losses.
High	Some increase expected [b]	Substantial increase in social, economic and/or environmental losses.
Low	No increase expected	Low social, economic and/or environmental losses.
Very Low	No increase	Small dams with minimal social, economic and/or environmental losses. Losses generally limited to the owner's property; damages to other property are acceptable to society.

[a] Incremental to the impacts which would occur under the same natural conditions (flood, earthquake event) but without failure of the dam. The type of consequences (e.g. loss of life, or economic losses) with the highest rating determines which category is assigned to the structure.

[b] The loss-of-life criteria which separates the High and Very High categories may be based on risks which are acceptable to society, taken to be 0.001 lives per year for each dam. Consistent with this tolerable societal risk the minimum criteria for a Very High Consequence dam (PMF and MCE) should result in an annual probability of failure of less than 1/100,000.

Knight Piésold Ltd.
CONSULTING ENGINEERS

TABLE 5.3
MOUNT POLLEY MINING CORPORATION
MOUNT POLLEY PROJECT
TAILINGS STORAGE FACILITY
MONTHLY WATER BALANCE - AVERAGE PRECIPITATION CONDITIONS
YEAR 1

Assumptions:
Daily Ore and Tailings Throughput (tpd) = 17,808
Solids Content = 35%
Tailings S.G. = 2.78
Water Content of Ore = 4%
Tailings Initial Dry Density (t/m^3) = 0.9
Tailings Final Dry Density (t/m^3) = 1.1
Minimum Fresh Water Makeup = 2.4%
Open Pit Groundwater Discharge (m^3/mo) = 39,818 (200 Igpm)

Catchment Areas:
Total Tailings Facility Area (ha) = 233
Pond Area (ha) = 48.6
Beach Area (ha) = 50.3
Unprepared Area (ha) = 134.1
Upstream Undiverted Area (ha) = 61
Total Pit Area (ha) = 17.6
Downstream Area (ha) = 63.1
Millsite Area-disturbed (ha) = 20
Millsite Area-undisturbed (ha) = 38.9
Waste Dump-disturbed (ha) = 27.2
Waste Dump-undisturbed (ha) = 106.8
Area North of Millsite (ha) = 22.6

Runoff Coefficients:
Tailings Pond = 1.0
Unprepared Basin = 0.24
Tailings Beach = 0.9
Open Pit Area = 0.5
Undisturbed Catchment Areas = 0.24
Millsite Area-disturbed = 0.70
East Waste Dump-disturbed = 0.60
Beach Evaporation Factor = 0.80
Downstream Area Factor = 0.70

11/28/97 12:01
J:\JOB\DATA\10162-8\WATERBAL\WBAL-3.XLS

	DESCRIPTION	JUN	JUL	AUG	SEP	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	ANNUAL	
A	Rainfall (mm/month)	81.5	65.7	83.1	58.9	48.3	17.3	7.6	6.8	6.0	6.0	24.2	45.3	450.7	A
B	Snowfall (mm/month)	0.0	0.0	0.0	1.5	12.1	40.0	67.2	68.7	52.1	38.5	18.9	5.3	304.3	B
C	Evaporation (mm/month)	112.0	107.0	92.0	50.0	15.0	0.0	0.0	0.0	0.0	0.0	0.0	47.0	423.0	C
	<WATER INTO TAILINGS IMPOUNDMENT> (m^3)														
1	With Slurry	1,005,940	1,005,940	1,005,940	1,005,940	1,005,940	1,005,940	1,005,940	1,005,940	1,005,940	1,005,940	1,005,940	1,005,940	12,071,280	1
2	Tailings Pond Precipitation	39,647	31,961	40,425	29,382	23,496	8,416	3,697	3,308	2,919	2,919	88,731	92,379	367,278	2
3	Tailings Beach Runoff	36,861	29,715	37,585	27,318	21,845	7,825	3,437	3,076	2,714	2,714	82,497	85,889	341,475	3
4	Undiverted Runoff From Within Tailings Facility	26,230	21,145	26,745	19,439	15,545	5,568	2,446	2,189	1,931	1,931	58,704	61,117	242,989	4
5	Runoff from Upstream Undiverted Area	11,932	9,618	12,166	8,843	7,071	2,533	1,113	996	878	878	26,703	27,801	110,532	5
5a	Runoff from Downstream Area	35,999	29,020	36,705	26,679	21,334	7,641	3,357	3,004	2,650	2,650	80,566	129,740	379,344	5a
6	Waste Dump Runoff (Disturbed and Undisturbed)	36,682	29,570	37,402	26,510	21,739	7,786	3,421	3,061	2,700	2,700	82,095	85,471	339,137	6
7	Water Available From Polley Lake	0	0	0	0	0	0	0	0	0	0	500,000	500,000	1,000,000	7
8	>>> Total	1,193,290	1,156,969	1,196,968	1,144,110	1,116,971	1,045,709	1,023,411	1,021,572	1,019,733	1,019,733	1,925,235	1,988,337	14,852,036	8
	<WATER OUT OF TAILINGS IMPOUNDMENT> (m^3)														
	Supernatant Recovery														
9	(+) Recovery from Tailings	598,937	598,937	598,937	598,937	598,937	598,937	598,937	598,937	598,937	598,937	598,937	598,937	7,187,247	9
10	(+) Total Net Precipitation and Runoff =(2)+(3)+(4)+(5)+(5a)+(6)-(18)-(19)	87,838	55,960	109,286	93,746	97,703	39,769	17,471	15,632	13,793	13,793	419,295	440,638	1,404,923	10
11	(+) Consolidation to Final Density	109,426	109,426	109,426	109,426	109,426	109,426	109,426	109,426	109,426	109,426	109,426	109,426	1,313,115	11
12	(+) Water Available From Polley Lake	0	0	0	0	0	0	0	0	0	0	500,000	500,000	1,000,000	12
13	(-) *Seepage*	(63,940)	(63,940)	(63,940)	(63,940)	(63,940)	(63,940)	(63,940)	(63,940)	(63,940)	(63,940)	(63,940)	(63,940)	(767,280)	13
14	Sub-Total (recovered water in supernatant pond)	732,262	700,384	753,710	738,169	742,127	684,192	661,894	660,055	658,216	658,216	1,563,719	1,585,061	10,138,006	14
	Underdrainage Recovery														
15	(+) Underdrainage	58,100	58,100	58,100	58,100	58,100	58,100	58,100	58,100	58,100	58,100	58,100	58,100	697,200	15
16	Sub-Total (total recovered water) =(14)+(15)	790,362	758,484	811,810	796,269	800,227	742,292	719,994	718,155	716,316	716,316	1,621,819	1,643,161	10,835,206	16
	Unrecoverable Water														
17	(-) *Water Retained in Tailings*	297,576	297,576	297,576	297,576	297,576	297,576	297,576	297,576	297,576	297,576	297,576	297,576	3,570,917	17
18	(-) *Evaporation from Supernatant Pond*	54,484	52,051	44,754	24,323	7,297	0	0	0	0	0	0	22,864	205,773	18
19	(-) *Evaporation from Beach*	45,027	43,017	36,987	20,102	6,030	0	0	0	0	0	0	18,895	170,059	19
20	(-) *Seepage Losses*	5,840	5,840	5,840	5,840	5,840	5,840	5,840	5,840	5,840	5,840	5,840	5,840	70,080	20
21	Sub-total (unrecoverable water)	402,928	398,485	385,158	347,841	316,744	303,416	303,416	303,416	303,416	303,416	303,416	345,176	4,016,830	21
22	>>> Total	1,193,290	1,156,969	1,196,968	1,144,110	1,116,971	1,045,709	1,023,411	1,021,572	1,019,733	1,019,733	1,925,235	1,988,337	14,852,036	22
	Water Required at Millsite														
23	Water for slurry	1,005,940	1,005,940	1,005,940	1,005,940	1,005,940	1,005,940	1,005,940	1,005,940	1,005,940	1,005,940	1,005,940	1,005,940	12,071,280	23
24	Water for Dust Control on Roads	25,000	25,000	25,000	25,000	25,000	0	0	0	0	0	0	25,000	150,000	24
25	Mill Water Required	1,030,940	1,030,940	1,030,940	1,030,940	1,030,940	1,005,940	1,005,940	1,005,940	1,005,940	1,005,940	1,005,940	1,030,940	12,221,280	25
26	(-) Minimum Fresh Water Input to Mill (from open pit groundwater) = 2.4%*(1)	24,143	24,143	24,143	24,143	24,143	24,143	24,143	24,143	24,143	24,143	24,143	24,143	289,711	26
27	(-) Water in Ore	21,982	21,982	21,982	21,982	21,982	21,982	21,982	21,982	21,982	21,982	21,982	21,982	263,784	27
28	Water Required from Additional Sources =(25)-(26)-(27)	984,815	984,815	984,815	984,815	984,815	959,815	959,815	959,815	959,815	959,815	959,815	984,815	11,667,785	28
	<WATER DISTRIBUTION IN SYSTEM> (m^3)														
29	Open Pit Surface Runoff	7,694	6,203	7,846	5,702	4,560	1,633	718	642	566	566	17,221	17,929	71,280	29
30	Open Pit Groundwater (39,818 - fresh water input to mill)	15,675	15,675	15,675	15,675	15,675	15,675	15,675	15,675	15,675	15,675	15,675	15,675	188,105	30
31	Mill Site Runoff (Disturbed and Undisturbed)	20,404	16,449	20,805	14,746	12,092	4,331	1,903	1,702	1,502	1,502	45,666	47,543	188,646	31
32	Catchment Area North of Millsite	4,743	3,823	4,836	3,427	2,811	1,007	442	396	349	349	10,614	11,051	43,847	32
33	Recovered Water from Tailings Facility (excluding storage)	790,362	758,484	811,810	796,269	800,227	742,292	719,994	718,155	716,316	716,316	1,621,819	1,643,161	10,835,206	33
34	Total Water Available in the System	838,879	800,634	860,972	835,821	835,365	764,939	738,732	736,571	734,409	734,409	1,710,995	1,735,359	11,327,085	34
35	Water Surplus/(Deficit) =(34)-(28)	(145,937)	(184,181)	(123,844)	(148,994)	(149,450)	(194,876)	(221,083)	(223,245)	(225,406)	(225,406)	751,179	750,544	-340,700	35
36	Cummulative Water Surplus/(Deficit)	(145,937)	(330,118)	(453,962)	(602,956)	(752,406)	(947,283)	(1,168,366)	(1,391,611)	(1,617,017)	(1,842,423)	(1,091,244)	(340,700)	-340,700	36

Notes:
1. Snowfall is provided in equivalent depth of rainfall and is assumed to accumulate on catchment areas until April and May when it melts equally over the two months.
2. Fresh water imput to mill to be supplied from Open Pit dewatering wells.

Knight Piésold Ltd.
CONSULTING ENGINEERS

TABLE 5.4
MOUNT POLLEY MINING CORPORATION
MOUNT POLLEY PROJECT
TAILINGS STORAGE FACILITY
MONTHLY WATER BALANCE - AVERAGE PRECIPITATION CONDITIONS
YEAR 13

Assumptions:
Daily Ore and Tailings Throughput (tpd) =17,808
Solids Content = 35%
Tailings S.G. = 2.78
Water Content of Ore = 4%
Tailings Initial Dry Density (t/m^3) = 0.9
Tailings Final Dry Density (t/m^3) = 1.3
Minimum Fresh Water Makeup = 2.4%
Open Pit Groundwater Discharge (m^3/mo) =39,818
(200 Igpm)

Catchment Areas:
Total Tailings Facility Area (ha) = 233
Pond Area (ha) = 101
Beach Area (ha) = 122
Unprepared Area (ha) = 10
Upstream Undiverted Area (ha) = 61
Total Pit Area (ha) = 64.7
Downstream Area (ha) = 63.1
Millsite Area-disturbed (ha) = 20
Millsite Area-undisturbed (ha) = 39
Waste Dump-disturbed (ha) = 134
Waste Dump-undisturbed (ha) =
Area North of Millsite (ha) = 24.3

Runoff Coefficients:
Tailings Pond = 1.0
Unprepared Basin = 0.24
Tailings Beach = 0.9
Open Pit Area = 0.5
Undisturbed Catchment Areas = 0.24
Millsite Area-disturbed = 0.70
East Waste Dump= 0.60
Beach Evaporation Factor = 0.80
Downstream Area Factor = 0.70

11/28/97 12:00

J:\JOB\DATA\10162-8\WATERBAL\WBAL-3.XLS

	DESCRIPTION	JUN	JUL	AUG	SEP	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	ANNUAL	
A	Rainfall (mm/month)	81.5	65.7	83.1	58.9	48.3	17.3	7.6	6.8	6.0	6.0	24.2	45.3	450.7	A
B	Snowfall (mm/month)	0.0	0.0	0.0	1.5	12.1	40.0	67.2	68.7	52.1	38.5	18.9	5.3	304.3	B
C	Evaporation (mm/month)	112.0	107.0	92.0	50.0	15.0	0.0	0.0	0.0	0.0	0.0	0.0	47.0	423.0	C
	<WATER INTO TAILINGS IMPOUNDMENT> (m^3)														
1	With Slurry	1,005,940	1,005,940	1,005,940	1,005,940	1,005,940	1,005,940	1,005,940	1,005,940	1,005,940	1,005,940	1,005,940	1,005,940	12,071,280	1
2	Tailings Pond Precipitation	82,152	66,226	83,765	60,883	48,686	17,438	7,661	6,854	6,048	6,048	183,859	191,419	761,040	2
3	Tailings Beach Runoff	89,707	72,316	91,468	66,482	53,164	19,042	8,365	7,485	6,604	6,604	200,768	209,023	831,029	3
4	Undiverted Runoff From Within Tailings Facility	1,936	1,561	1,974	1,435	1,148	411	181	162	143	143	4,334	4,512	17,939	4
5	Runoff from Upstream Undiverted Area	11,932	9,618	12,166	8,843	7,071	2,533	1,113	996	878	878	26,703	27,801	110,532	5
5a	Runoff from Downstream Area	35,999	29,020	36,705	26,679	21,334	7,641	3,357	3,004	2,650	2,650	80,566	129,740	379,344	5a
6	Waste Dump Runoff (Disturbed and Undisturbed)	70,300	56,671	71,680	50,805	41,662	14,922	6,556	5,865	5,175	5,175	157,333	163,802	649,948	6
7	Water Available From Polley Lake	0	0	0	0	0	0	0	0	0	0	0	0	0	7
8	>>> Total	1,297,965	1,241,352	1,303,698	1,221,067	1,179,005	1,067,928	1,033,172	1,030,305	1,027,439	1,027,439	1,659,503	1,732,238	14,821,111	8
	<WATER OUT OF TAILINGS IMPOUNDMENT> (m^3)														
	Supernatant Recovery														
9	(+) Recovery from Tailings	598,937	598,937	598,937	598,937	598,937	598,937	598,937	598,937	598,937	598,937	598,937	598,937	7,187,247	9
10	(+) Total Net Precipitation and Runoff =(2)+(3)+(4)+(5)+(5a)+(6)-(18)-(1	69,548	22,867	115,009	115,807	143,269	61,988	27,232	24,365	21,499	21,499	653,563	632,937	1,909,584	10
11	(+) Consolidation to Final Density	185,183	185,183	185,183	185,183	185,183	185,183	185,183	185,183	185,183	185,183	185,183	185,183	2,222,195	11
12	(+) Water Available From Polley Lake	0	0	0	0	0	0	0	0	0	0	0	0	0	12
13	(-) *Seepage*	(63,940)	(63,940)	(63,940)	(63,940)	(63,940)	(63,940)	(63,940)	(63,940)	(63,940)	(63,940)	(63,940)	(63,940)	(767,280)	13
14	Sub-Total (recovered water in supernatant pond)	789,729	743,047	835,190	835,987	863,449	782,168	747,412	744,545	741,679	741,679	1,373,743	1,353,117	10,551,746	14
	Underdrainage Recovery														
15	(+) Underdrainage	58,100	58,100	58,100	58,100	58,100	58,100	58,100	58,100	58,100	58,100	58,100	58,100	697,200	15
16	Sub-Total (total recovered water) =(14)+(15)	847,829	801,147	893,290	894,087	921,549	840,268	805,512	802,645	799,779	799,779	1,431,843	1,411,217	11,248,946	16
	Unrecoverable Water														
17	(-) *Water Retained in Tailings*	221,820	221,820	221,820	221,820	221,820	221,820	221,820	221,820	221,820	221,820	221,820	221,820	2,661,838	17
18	(-) *Evaporation from Supernatant Pond*	112,896	107,856	92,736	50,400	15,120	0	0	0	0	0	0	47,376	426,384	18
19	(-) *Evaporation from Beach*	109,581	104,689	90,013	48,920	14,676	0	0	0	0	0	0	45,985	413,863	19
20	(-) *Seepage Losses*	5,840	5,840	5,840	5,840	5,840	5,840	5,840	5,840	5,840	5,840	5,840	5,840	70,080	20
21	Sub-total (unrecoverable water)	450,137	440,205	410,409	326,980	257,456	227,660	227,660	227,660	227,660	227,660	227,660	321,021	3,572,165	21
22	>>> Total	1,297,965	1,241,352	1,303,698	1,221,067	1,179,005	1,067,928	1,033,172	1,030,305	1,027,439	1,027,439	1,659,503	1,732,238	14,821,111	22
	Water Required at Millsite														
23	Water for slurry	1,005,940	1,005,940	1,005,940	1,005,940	1,005,940	1,005,940	1,005,940	1,005,940	1,005,940	1,005,940	1,005,940	1,005,940	12,071,280	23
24	Water for Dust Control on Roads	25,000	25,000	25,000	25,000	25,000	0	0	0	0	0	0	25,000	150,000	24
25	Mill Water Required	1,030,940	1,030,940	1,030,940	1,030,940	1,030,940	1,005,940	1,005,940	1,005,940	1,005,940	1,005,940	1,005,940	1,030,940	12,221,280	25
26	(-) Minimum Fresh Water Input to Mill (from open pit groundwater) = 2.4%*(1)	24,143	24,143	24,143	24,143	24,143	24,143	24,143	24,143	24,143	24,143	24,143	24,143	289,711	26
27	(-) Water in Ore	21,982	21,982	21,982	21,982	21,982	21,982	21,982	21,982	21,982	21,982	21,982	21,982	263,784	27
28	Water Required from Additional Sources =(25)-(26)-(27)	984,815	984,815	984,815	984,815	984,815	959,815	959,815	959,815	959,815	959,815	959,815	984,815	11,667,785	28
	<WATER DISTRIBUTION IN SYSTEM> (m^3)														
29	Open Pit Surface Runoff	28,286	22,802	28,841	20,963	16,763	6,004	2,638	2,360	2,082	2,082	63,305	65,908	262,036	29
30	Open Pit Groundwater (39,818 - fresh water input to mill)	15,675	15,675	15,675	15,675	15,675	15,675	15,675	15,675	15,675	15,675	15,675	15,675	188,105	30
31	Mill Site Runoff (Disturbed and Undisturbed)	20,404	16,449	20,805	14,746	12,092	4,331	1,903	1,702	1,502	1,502	45,666	47,543	188,646	31
32	Catchment Area North of Millsite	5,099	4,111	5,199	3,685	3,022	1,082	476	425	375	375	11,413	11,882	47,145	32
33	Recovered Water from Tailings Facility (excluding storage)	847,829	801,147	893,290	894,087	921,549	840,268	805,512	802,645	799,779	799,779	1,431,843	1,411,217	11,248,946	33
34	Total Water Available in the System	917,294	860,184	963,811	949,157	969,103	867,362	826,203	822,809	819,414	819,414	1,567,902	1,552,226	11,934,879	34
35	Water Surplus/(Deficit) =(34)-(28)	(67,522)	(124,631)	(21,005)	(35,658)	(15,713)	(92,454)	(133,612)	(137,007)	(140,401)	(140,401)	608,087	567,410	267,094	35
36	Cummulative Water Surplus/(Deficit)	(67,522)	(192,153)	(213,158)	(248,816)	(264,528)	(356,982)	(490,594)	(627,601)	(768,002)	(908,403)	(300,316)	267,094		36

Notes:
1. Snowfall is provided in equivalent depth of rainfall and is assumed to accumulate on catchment areas until April and May when it melts equally over the two months.
2. Fresh water input to mill to be supplied from Open Pit dewatering wells.

Knight Piésold Ltd.
CONSULTING ENGINEERS

TABLE 5.5

MOUNT POLLEY MINING CORPORATION
MOUNT POLLEY PROJECT
TAILINGS STORAGE FACILITY

STAGED EMBANKMENT FILL QUANTITIES

J:\JOB\DATA\10162-9\VOLUMES\TOTALVOL.XLS]Overall 3-Dec-97 14:05

		FILL QUANTITY BY STAGE (m^3)[1]									
	Stage	1b	2A	2B	2C	3	4	5	6	7	
	El. (m)	934	936	938	940	946	951	956	961	965	TOTAL
Item	**ZONE AND MATERIAL** Year	1996/97	1998	1999	1999	2000	2002	2004	2006	2008	(m^3)
1.0	COARSE BEARING LAYER (CBL)[2]	0	9,000	0	15,500	0	0	0	0	0	24,500
2.0	ZONE B	220,000	84,000	29,400	66,500	0	0	0	0	0	399,900
3.0	FREE DRAINING RANDOM FILL (FDF)[3]	0	0	0	0	256,455	221,375	229,625	235,510	175,400	1,118,365
4.0	ZONE S	352,000	21,000	45,500	46,900	163,500	162,500	167,500	172,500	140,500	1,271,900
5.0	FILTER SAND										
5.1	Chimney Drain	22,000	0	16,800	5,900	20,280	20,150	20,850	21,425	0	127,405
5.2	Longitudinal Drain	2,450	0	5,400	0	2,400	1,100	500	350	0	12,200
5.3	Outlet Drain	50	800	700	0	700	0	0	0	0	2,250
6.0	DRAIN GRAVEL										
6.1	Longitudinal Drain	450	0	900	0	400	200	100	50	0	2,100
6.2	Outlet Drain	50	200	100	0	100	0	0	0	0	450
6.3	Foundation Drain	1,500	200	800	0	0	0	0	0	0	2,500
7.0	ZONE T	0	52,000	137,500	23,500	93,420	80,600	83,400	85,700	0	556,120
8.0	ZONE C	0	0	506,600	29,300	812,500	130,800	6,800	1,368,900	61,600	2,916,500
	TOTALS	598,500	167,200	743,700	187,600	1,349,755	616,725	508,775	1,884,435	377,500	6,434,190

Notes:
[1] All quantities listed above are neat line. No allowance has been added for cut to fill shrinkage.
[2] Coarse Bearing Layer only included for Stage 2. It may be required for additional expansions, to be determined prior to construction.
[3] Free Draining Random Fill material type to be determined prior to construction.

Knight Piésold Ltd.
CONSULTING ENGINEERS

TABLE 5.6

MOUNT POLLEY MINING CORPORATION
MOUNT POLLEY PROJECT
TAILINGS STORAGE FACILITY

SUMMARY OF REQUIRED SEEPAGE ANALYSES

J:\JOB\REPORT\10162-9\3TBLTMP.XLS 1-Dec-97

CASE	EMBANKMENT SECTION					
	Main Embankment		Perimeter Embankment		South Emankment	
	With U/S Toe Drain	Without U/S Toe Drain	With U/S Toe Drain	Without U/S Toe Drain	With U/S Toe Drain	Without U/S Toe Drain
Stage 2	---	yes[1]	---	---	---	---
Final Embankment - Operations	yes[1], [2]	yes[1], [2]	yes[2]	yes[2]	yes[2]	yes[2]
Final Embankment - Post Closure	yes[1]	---	---	---	---	---

Notes:

[1] Required to determine phreatic surface for stability analyses.

[2] Required to determine seepage flows for each component (foundation drains, chimney drain, U/S toe drains and seepage losses).

Knight Piésold Ltd.
CONSULTING ENGINEERS

TABLE 5.7

MOUNT POLLEY MINING CORPORATION
MOUNT POLLEY PROJECT
TAILINGS STORAGE FACILITY

RESULTS OF SEEPAGE ANALYSES

J:\JOB\DATA\10162-9\SEEPW\[SEPSUM.XLS]Table 5.6b 3-Dec-97

Embankment Section	Section Length (m)	Estimated Flow from Seepage Analysis (L / s)				
		Into Toe Drains	Into Chimney Drain	Into Foundation Drains	Into Groundwater System	Total Flux Through Embankment
Case 1 - Upstream Toe Drains Functioning						
Perimeter Embankment	2130	19.31	0.32	N/A	0.81	20.45
Main Embankment - Section A (varved silt unit within foundation)	950	8.36	0.17	0.25	0.90	9.73
Main Embankment - Section B (sand unit within foundation)	480	3.50	0.06	0.85	0.04	4.40
South Embankment	885	5.23	0.06	N/A	0.02	5.31
Case 1 Totals	4445	36.40	0.61	1.10	1.77	39.89
Case 2 - Upstream Toe Drains Not Functioning						
Perimeter Embankment	2130	N/A	0.70	N/A	0.87	1.57
Main Embankment - Section A (varved silt unit within foundation)	950	N/A	0.26	0.23	0.94	1.43
Main Embankment - Section B (sand unit within foundation)	480	N/A	0.08	0.91	0.04	1.03
South Embankment	885	N/A	0.19	N/A	0.02	0.21
Case 2 Totals	4445	N/A	1.23	1.14	1.87	4.24

OK

Knight Piésold Ltd.
CONSULTING ENGINEERS

TABLE 5.8

MOUNT POLLEY MINING CORPORATION
MOUNT POLLEY PROJECT
TAILINGS STORAGE FACILITY

SUMMARY OF REQUIRED STABILITY ANALYSES

J:\JOB\REPORT\10162-9\3TBLTMP.XLS 1-Dec-97

CASE	UPSTREAM ANALYSES[12]					DOWNSTREAM ANALYSES				
	With U/S Toe Drain		Without U/S Toe Drain			With U/S Toe Drain		Without U/S Toe Drain		
	Static	Seismic	Static	Seismic	Residual	Static	Seismic	Static	Seismic	Residual
Stage 2 - End of Construction	---[1]	---[1]	yes[2]	yes[3]	---[13]	---[1]	---[1]	yes[2]	yes[3]	---[13]
Final Embankment - Operations	yes[4]	yes[3]	yes[4]	yes[3]	---[13]	yes[4]	yes[3]	yes[4]	yes[3]	---[13]
Final Embankment - Post Closure	---[5]	---[5]	yes[6]	yes[3]	yes[13]	---[5]	---[5]	yes[6]	yes[3]	yes[13]

Notes:

[1] No U/S toe drains installed for Stage 2, no analyses required.
[2] Minimum Factor of Safety for End of Construction Static Case is 1.3.
[3] Minimum Factors of Safety for all seismic cases are:
- Pseudostatic case 1.0 (or limited displacement).
- Displacement analyses to ensure acceptable movements during the earthquake may be required.
- Post-Liquefaction (flow slide) case 1.1.

[4] Minimum Factor of Safety for Final Emabankment Static Case During Operations is 1.3.
[5] Upstream analyses for full height Post Closure required for worst case only (without U/S toe drain).
[6] Minimum Factor of Safety for Final Embankment Static Post Closure case is 1.5.
[7] Peak shear strength parameters are to be used for static cases (End of Construction or Post Closure).
[8] Residual shear strength parameters are to be used for seismic cases.
[9] In materials predicted to liquify, post-liquefaction shear strengths are to be used.
[10] Steady state seepage conditions are to be used for Post Closure cases.
[11] Steady state pore pressure conditions for relevant pond level are to be used for End of Construction and During Operations cases.
[12] All upstream analyses for loss of freeboard critical slip surfaces.
[13] Residual Tailings Strength Case only required for Post Closure worst case (without U/S toe drain). Minimum Factor of Safety is 1.1.

Knight Piésold Ltd.
CONSULTING ENGINEERS

TABLE 5.9

MOUNT POLLEY MINING CORPORATION
MOUNT POLLEY PROJECT
TAILINGS STORAGE FACILITY

RESULTS OF STABILITY ANALYSES

J:\JOB\REPORT\10162-9\3TBLTMP.XLS 1-Dec-97

CASE	UPSTREAM ANALYSES[12]					DOWNSTREAM ANALYSES				
	With U/S Toe Drain		Without U/S Toe Drain			With U/S Toe Drain		Without U/S Toe Drain		
	Static	Seismic	Static	Seismic	Residual	Static	Seismic	Static	Seismic	Residual
Stage 2 - End of Construction	---[1]	---[1]	1.64[2]	1.49[3]	---[13]	---[1]	---[1]	1.67[2]	1.44[3]	---[13]
Final Embankment - Operations	1.93[4]	1.67[3]	1.82[4]	1.41[3]	---[13]	1.58[4]	1.42[3]	1.55[4]	1.35[3]	---[13]
Final Embankment - Post Closure	---[5]	---[5]	2.09[6]	1.72[3]	1.71[13]	---[5]	---[5]	1.77[6]	1.49[3]	1.39[13]

Notes:

[1] No U/S toe drains installed for Stage 2, no analyses required.

[2] Minimum Factor of Safety for End of Construction Static Case is 1.3. Good

[3] Minimum Factors of Safety for all seismic cases are:
- Pseudostatic case 1.0 (or limited displacement).
- Displacement analyses to ensure acceptable movements during the earthquake may be required.
- Post-Liquefaction (flow slide) case 1.1.

[4] Minimum Factor of Safety for Final Emabankment Static Case During Operations is 1.3.

[5] Phreatic surface below upstream toe drains for Final Embankment Post Closure Analysis. Analysis not required.

[6] Minimum Factor of Safety for Final Embankment Static Post Closure case is 1.5.

[7] Peak shear strength parameters are to be used for static cases (End of Construction or Post Closure).

[8] Residual shear strength parameters are to be used for seismic cases.

[9] In materials predicted to liquify, post-liquefaction shear strengths are to be used.

[10] Steady state seepage conditions are to be used for Post Closure cases.

[11] Steady state pore pressure conditions for relevant pond level are to be used for End of Construction and During Operations cases.

[12] All upstream analyses for loss of freeboard critical slip surfaces.

[13] Residual Tailings Strength Case for worst case only (without U/S toe drain).

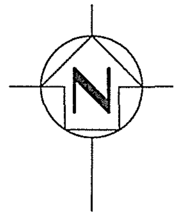
A5

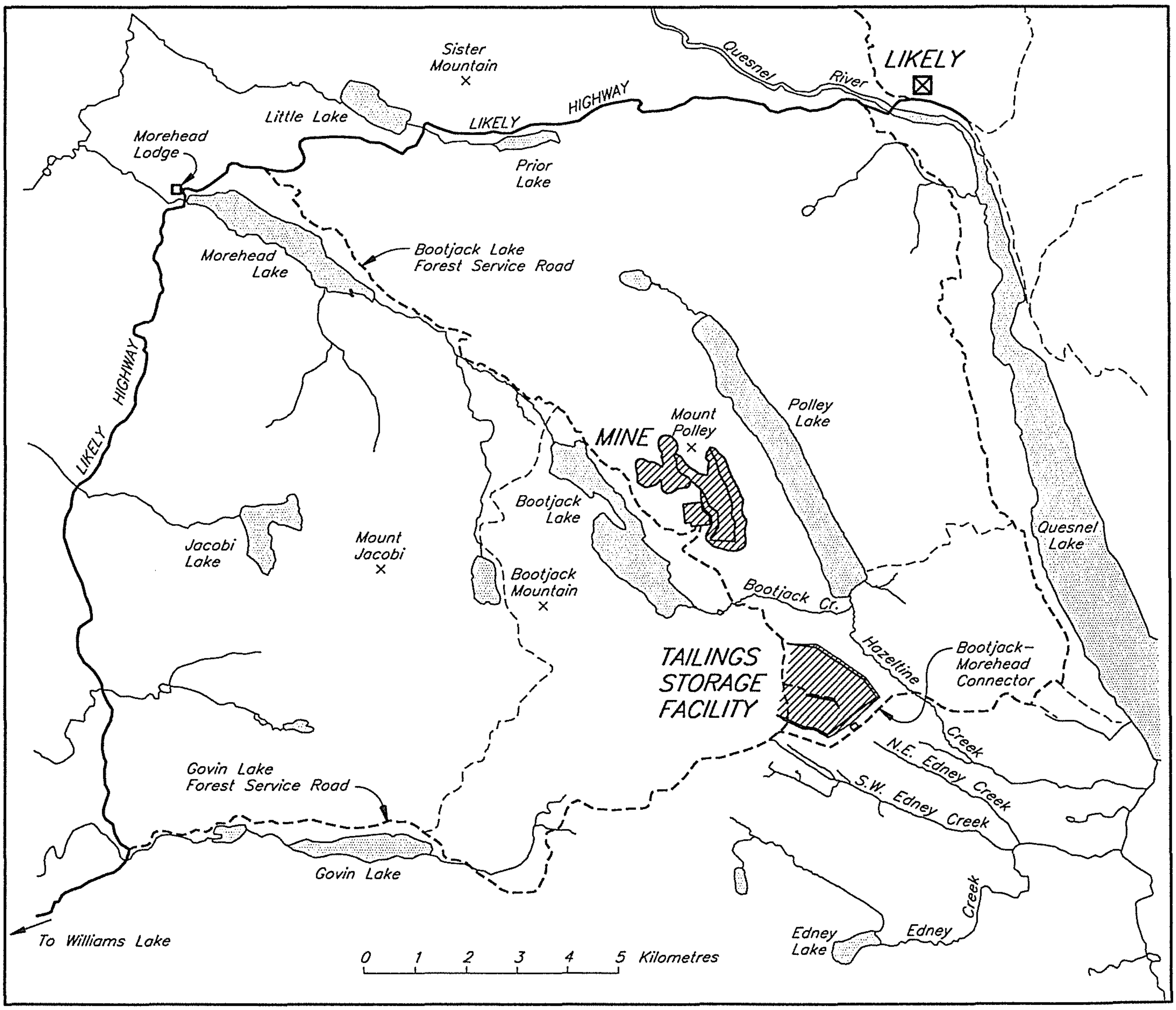
MOUNT POLLEY MINING CORPORATION

MOUNT POLLEY PROJECT

PROJECT LOCATION AND ACCESS PLAN

KEY MAP





Nov. 28, 1997

LOCATION MAP

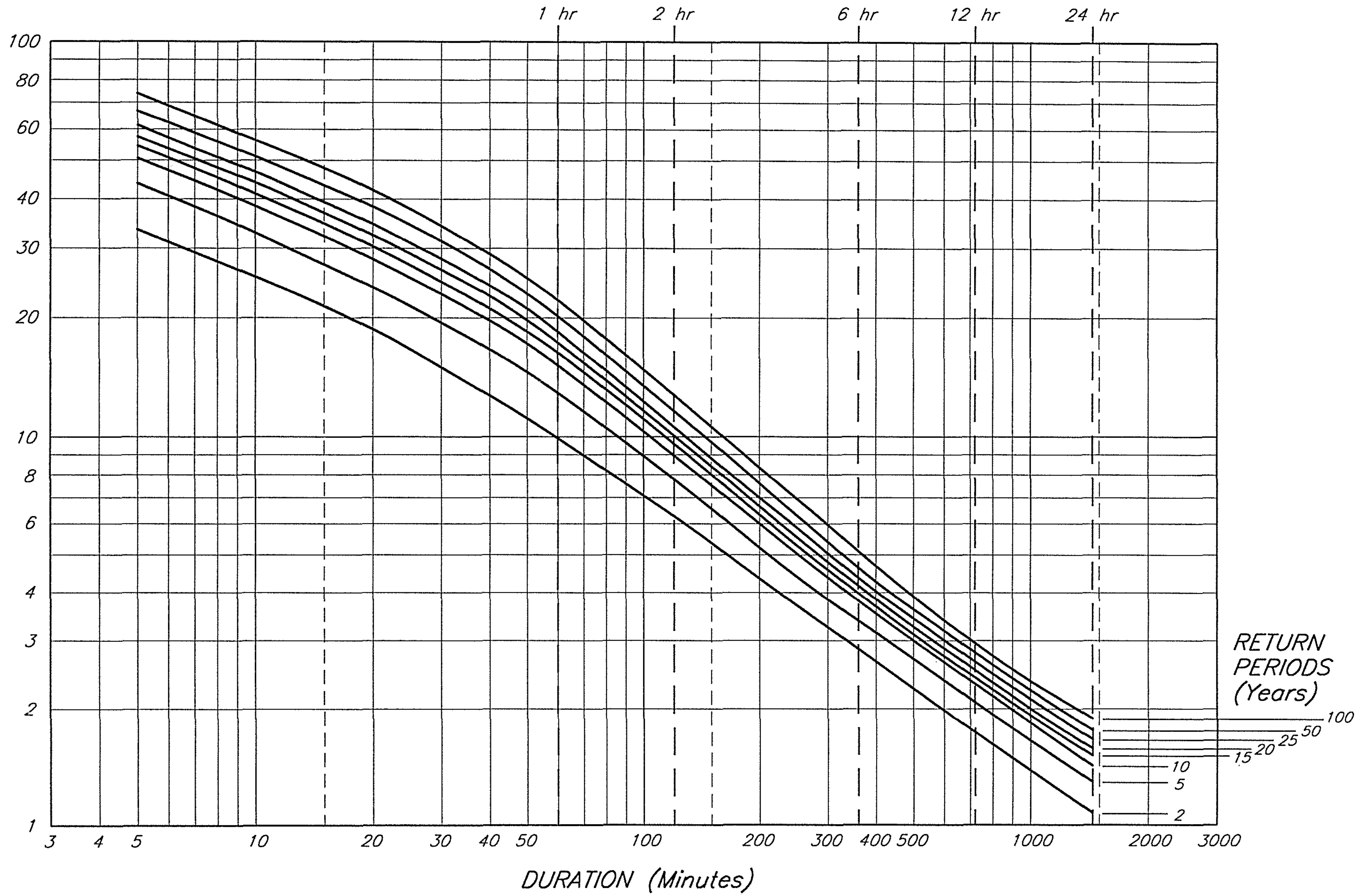
KNIGHT PIESOLD LTD.
CONSULTING ENGINEERS

FIGURE 1.1

CAD FILE: 10162\9\FIG\A5 1:125,000 Plot 1=125

CAD FILE: \10162\9\FIG\A6 Plot 1=1

MOUNT POLLEY MINING CORPORATION
MOUNT POLLEY PROJECT
SHORT DURATION RAINFALL INTENSITY – DURATION VS FREQUENCY DATA FOR MINE SITE



Nov. 28, 1997

KNIGHT PIESOLD LTD.
CONSULTING ENGINEERS

FIGURE 2.1

A6

J:\JOB\DATA\10162-9\TAILSLAB\MP-PSD.XLS Gradation 11/28/97 12:10 PM

KNIGHT PIESOLD LTD.
CONSULTING ENGINEERS

MOUNT POLLEY MINING CORPORATION.
MOUNT POLLEY PROJECT
GRAIN SIZE DISTRIBUTION FOR TAILINGS SAMPLES

Project No.: 10162/9
Date : November 27, 1997

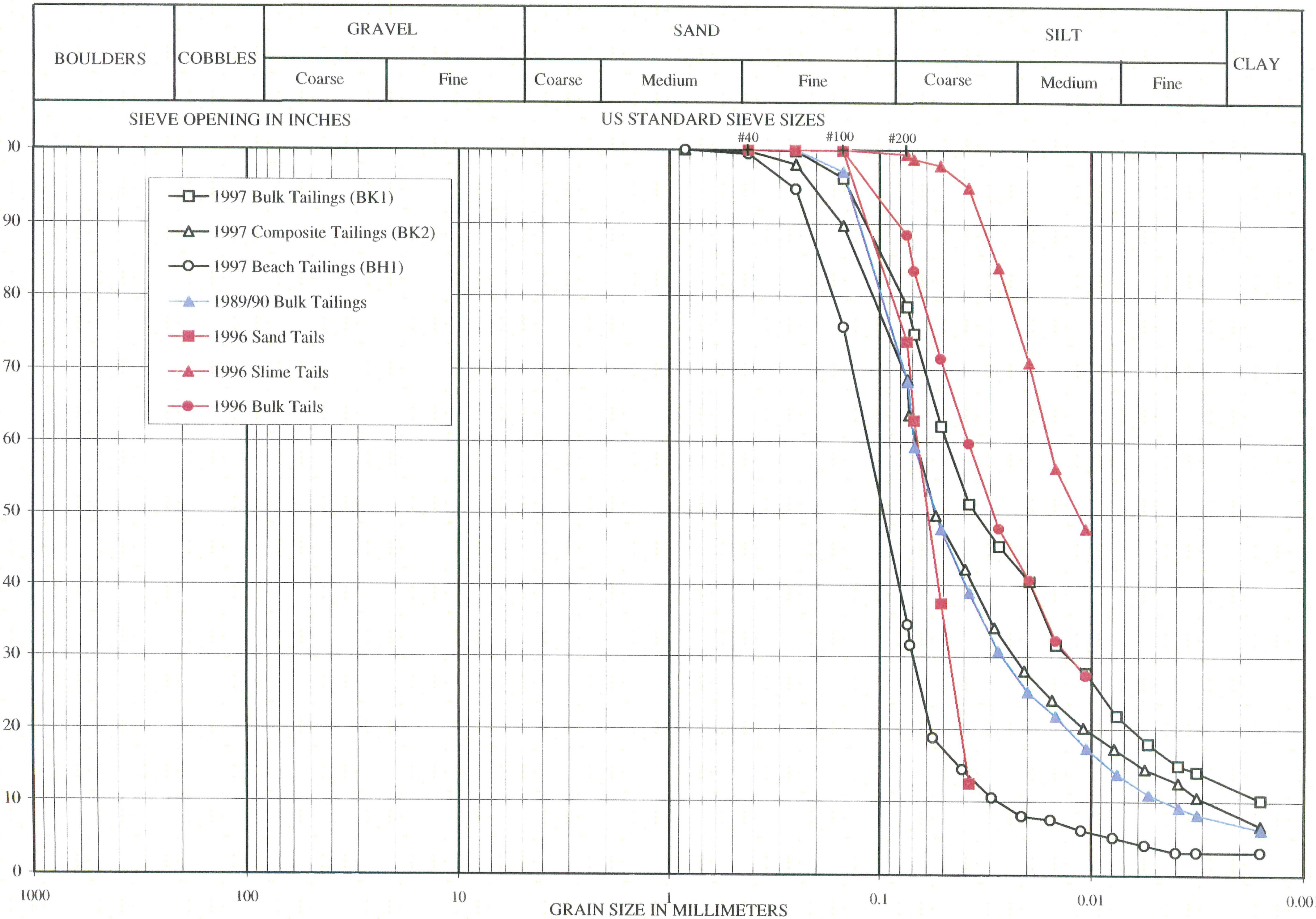
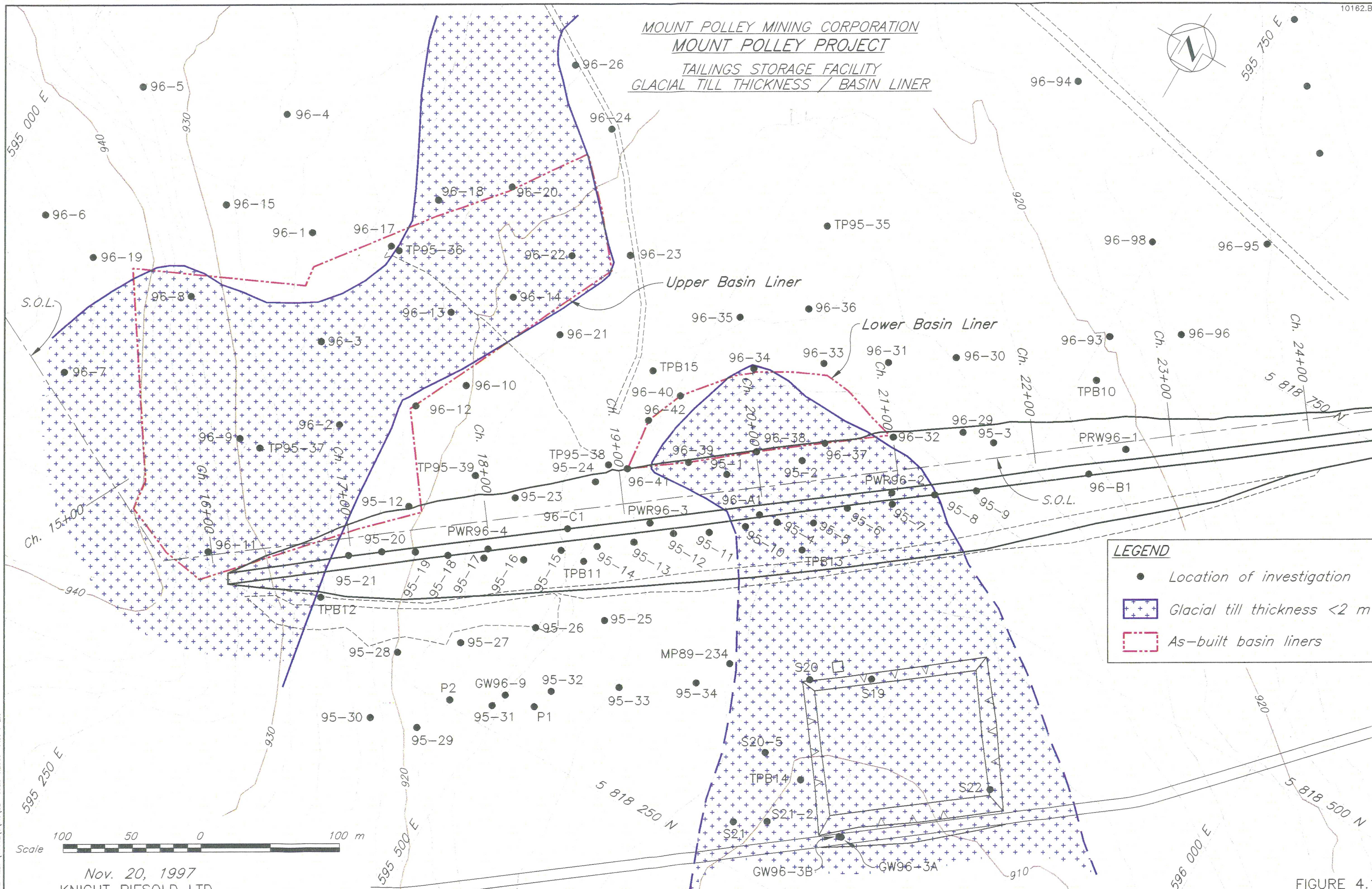


FIGURE 3.1

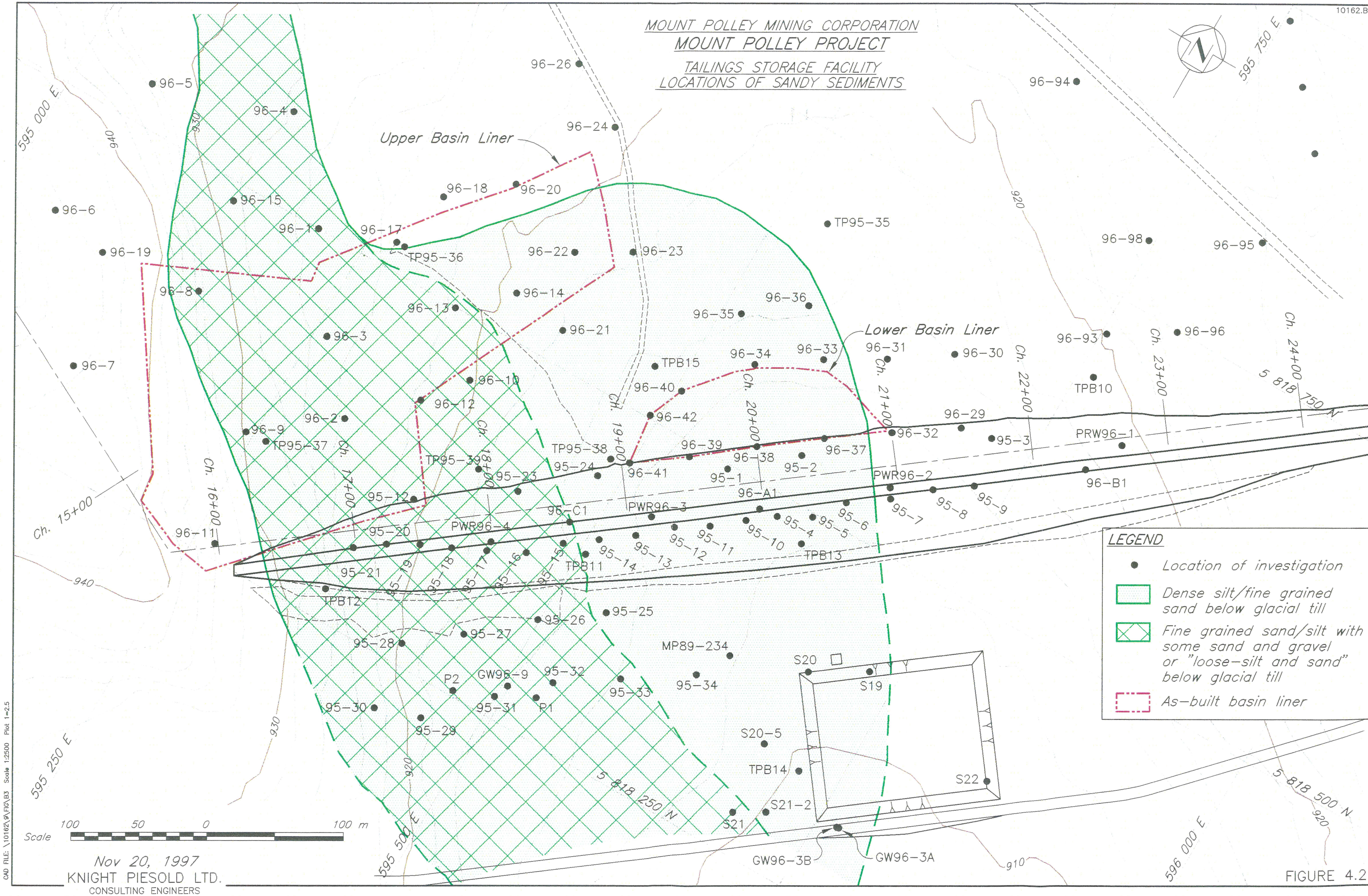


Nov. 20, 1997

KNIGHT PIESOLD LTD.
CONSULTING ENGINEERS

CAD FILE: \10162\9\FIG\B2 Scale 1:2500 Plot 1=2.5

FIGURE 4.1

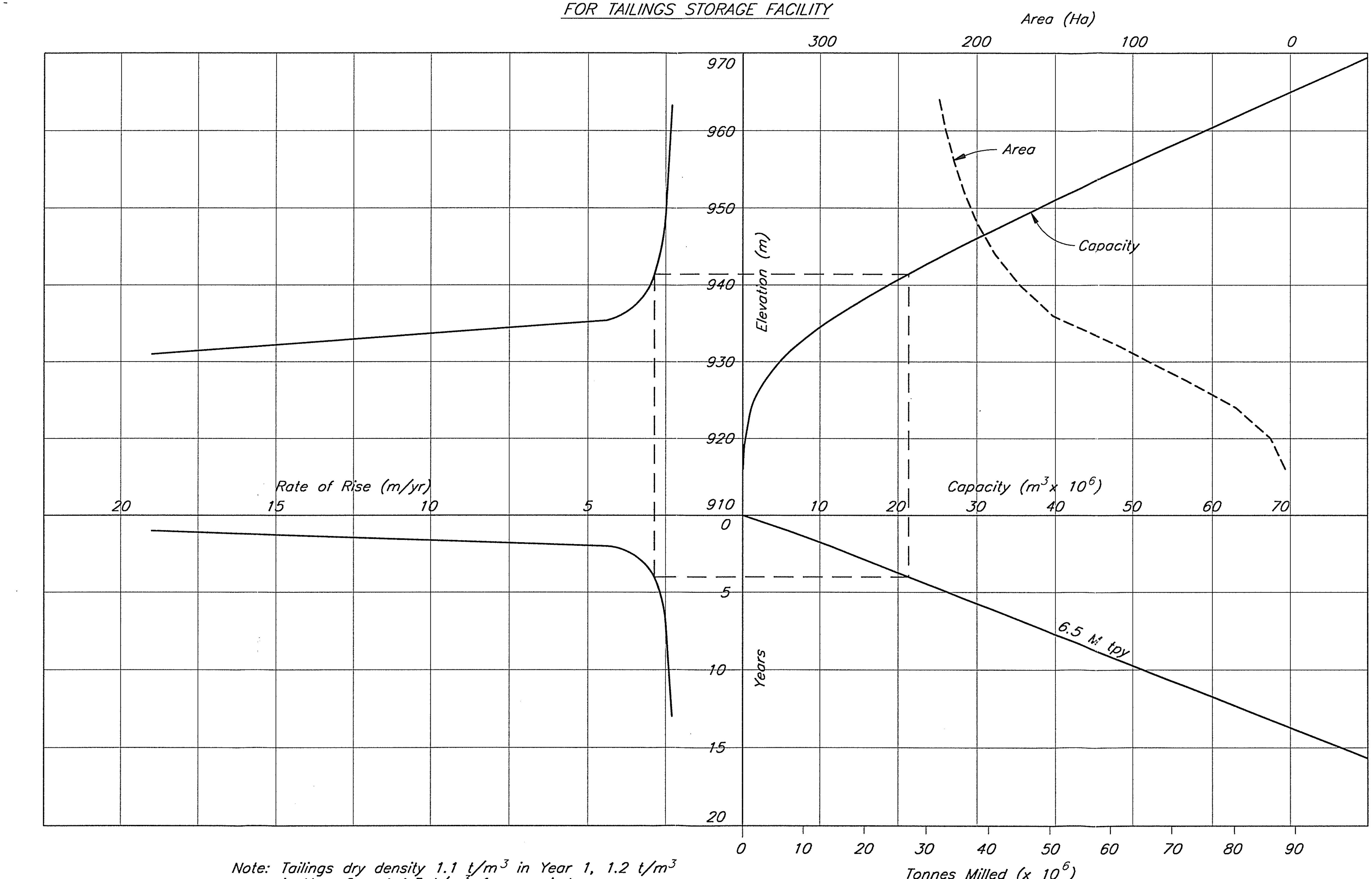


10162.B3
MOUNT POLLEY MINING CORPORATION
MOUNT POLLEY PROJECT
TAILINGS STORAGE FACILITY
LOCATIONS OF SANDY SEDIMENTS
Upper Basin Liner
Lower Basin Liner
595 000 E
595 250 E
595 500 E
595 750 E
596 000 E
5 818 750 N
5 818 500 N
5 818 250 N
Ch. 15+00
Ch. 16+00
Ch. 17+00
Ch. 18+00
Ch. 19+00
Ch. 20+00
Ch. 21+00
Ch. 22+00
Ch. 23+00
Ch. 24+00
LEGEND
Location of investigation
Dense silt/fine grained sand below glacial till
Fine grained sand/silt with some sand and gravel or "loose-silt and sand" below glacial till
As-built basin liner
Scale
100 50 0 100 m
Nov 20, 1997
KNIGHT PIESOLD LTD.
CONSULTING ENGINEERS
CAD FILE: \10162\9\FIG\B3 Scale 1:2500 Plot 1=2.5
FIGURE 4.2

MOUNT POLLEY MINING CORPORATION

MOUNT POLLEY PROJECT

DEPTH/AREA/CAPACITY/FILLING RATE FOR TAILINGS STORAGE FACILITY



Note: Tailings dry density 1.1 t/m^3 in Year 1, 1.2 t/m^3 in Year 2 and 1.3 t/m^3 for remainder.

Nov. 28, 1997

KNIGHT PIESOLD LTD.
CONSULTING ENGINEERS

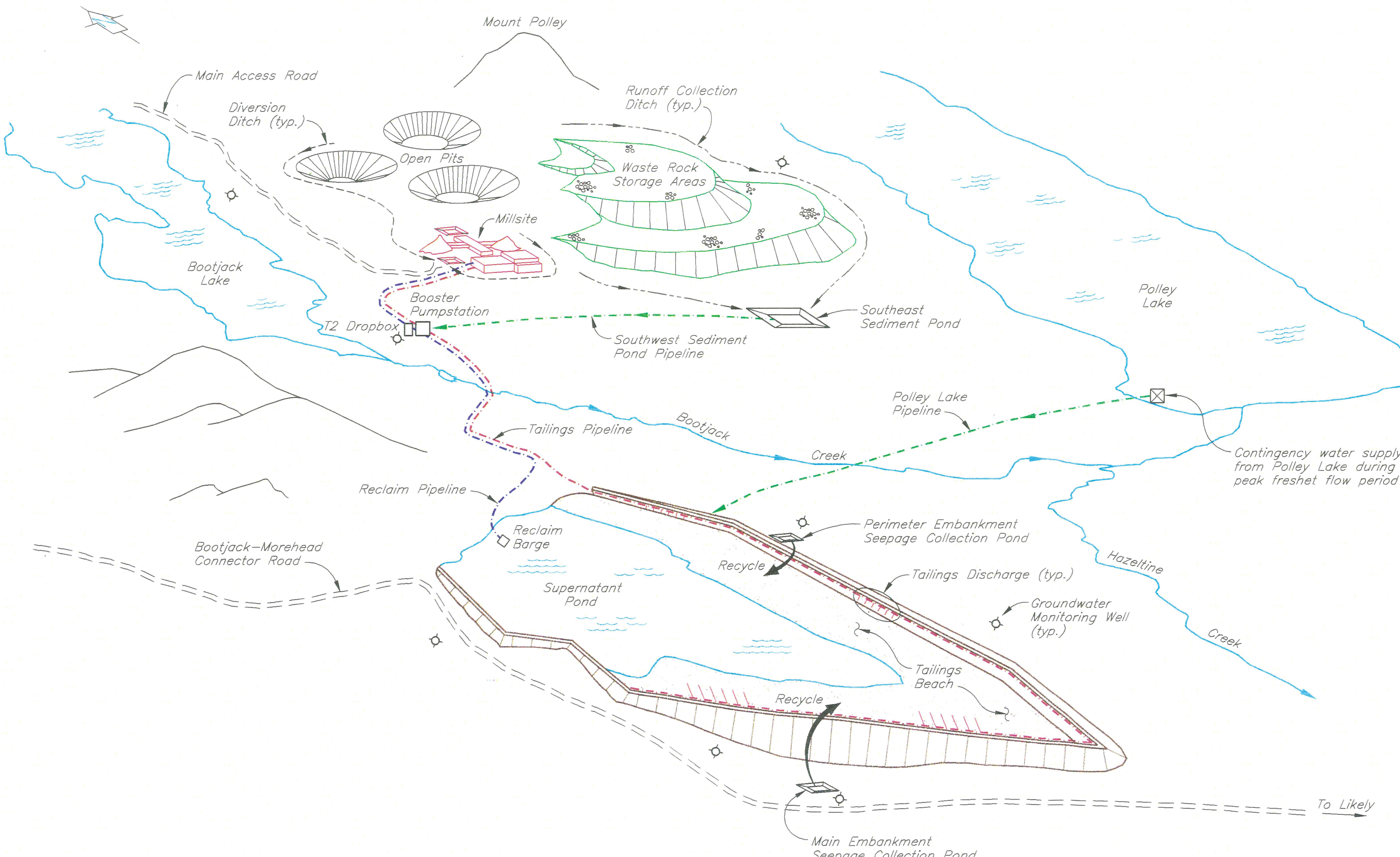
FIGURE 5.1

CAD FILE: \10162\9\FIG\B9 Plot 1=1

MOUNT POLLEY MINING CORPORATION

MOUNT POLLEY PROJECT

SITE WATER MANAGEMENT SCHEMATIC



Nov. 28, 1997

KNIGHT PIESOLD LTD.
CONSULTING ENGINEERS

FIGURE 5.2

B10

CAD FILE: \10162\9\FIG\B10 Plot scale 1=2 STD. 1

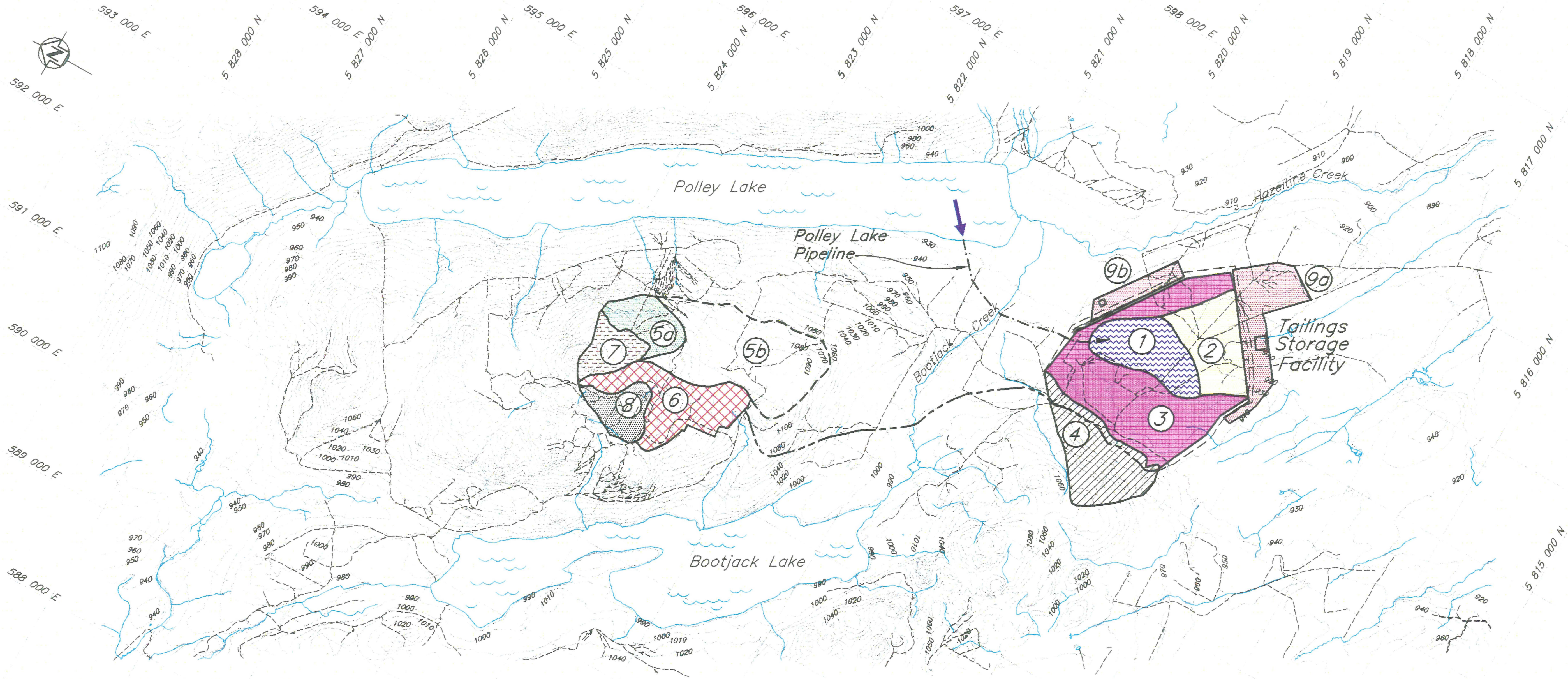
B11

MOUNT POLLEY MINING CORPORATION

MOUNT POLLEY PROJECT

SCHEMATIC OVERVIEW OF WATER BALANCE COMPONENTS

YEAR 1



CATCHMENT AREAS IN WATER BALANCE FOR YEAR 1		
Catchment Area	Area No.	Area (ha)
Tailings Facility :		
Tailings Pond	1	48.6
Beach	2	50.3
Unprepared area	3	134.1
Upstream undiverted area	4	60.9
Downstream area	9a	47.9
Downstream area	9b	15.2
Waste Dump :		
Disturbed area	5a	27.2
Undisturbed area	5b	106.8
Mill Site	6	58.9
Area north of Mill Site	7	22.6
Open Pit	8	17.6

CAD FILE: \10162\9\FIG\B11 Plot scale 1=40

Nov. 28, 1997

KNIGHT PIESOLD LTD.

CONSULTING ENGINEERS

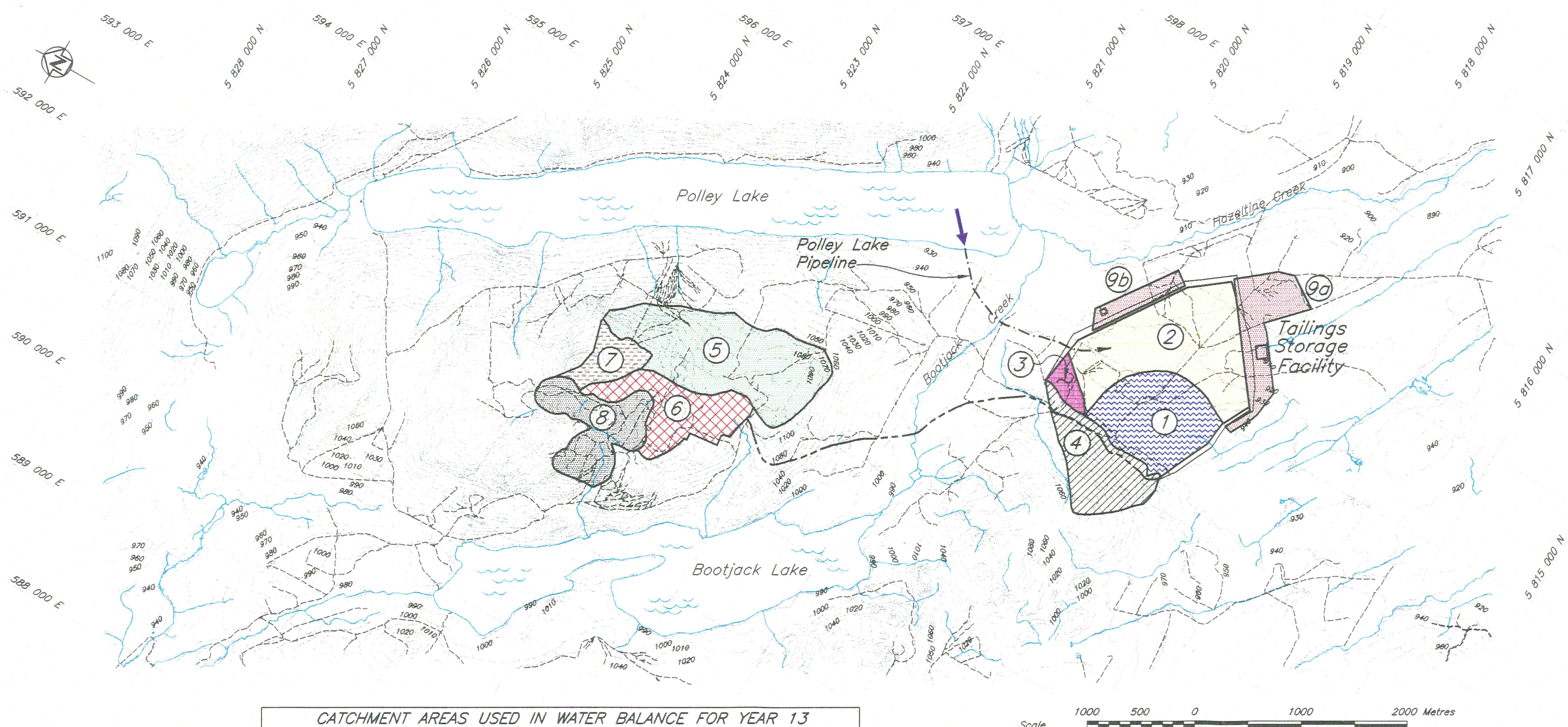
FIGURE 5.3

MOUNT POLLEY MINING CORPORATION

MOUNT POLLEY PROJECT

SCHEMATIC OVERVIEW OF WATER BALANCE COMPONENTS

YEAR 13



CATCHMENT AREAS USED IN WATER BALANCE FOR YEAR 13		
Catchment Area	Area No.	Area (ha)
Tailings Facility :		
Tailings Pond	1	101.0
Beach	2	122.0
Unprepared area	3	10.0
Upstream undiverted area	4	60.9
Downstream area	9a	47.9
Downstream area	9b	15.2
Waste Dump	5	134.0
Mill Site	6	58.9
Area north of Mill Site	7	24.3
Open Pit	8	64.7

CAD FILE: \10162\9\FIG\B12 Plot scale 1=40

Nov. 28, 1997

KNIGHT PIESOLD LTD.
CONSULTING ENGINEERS

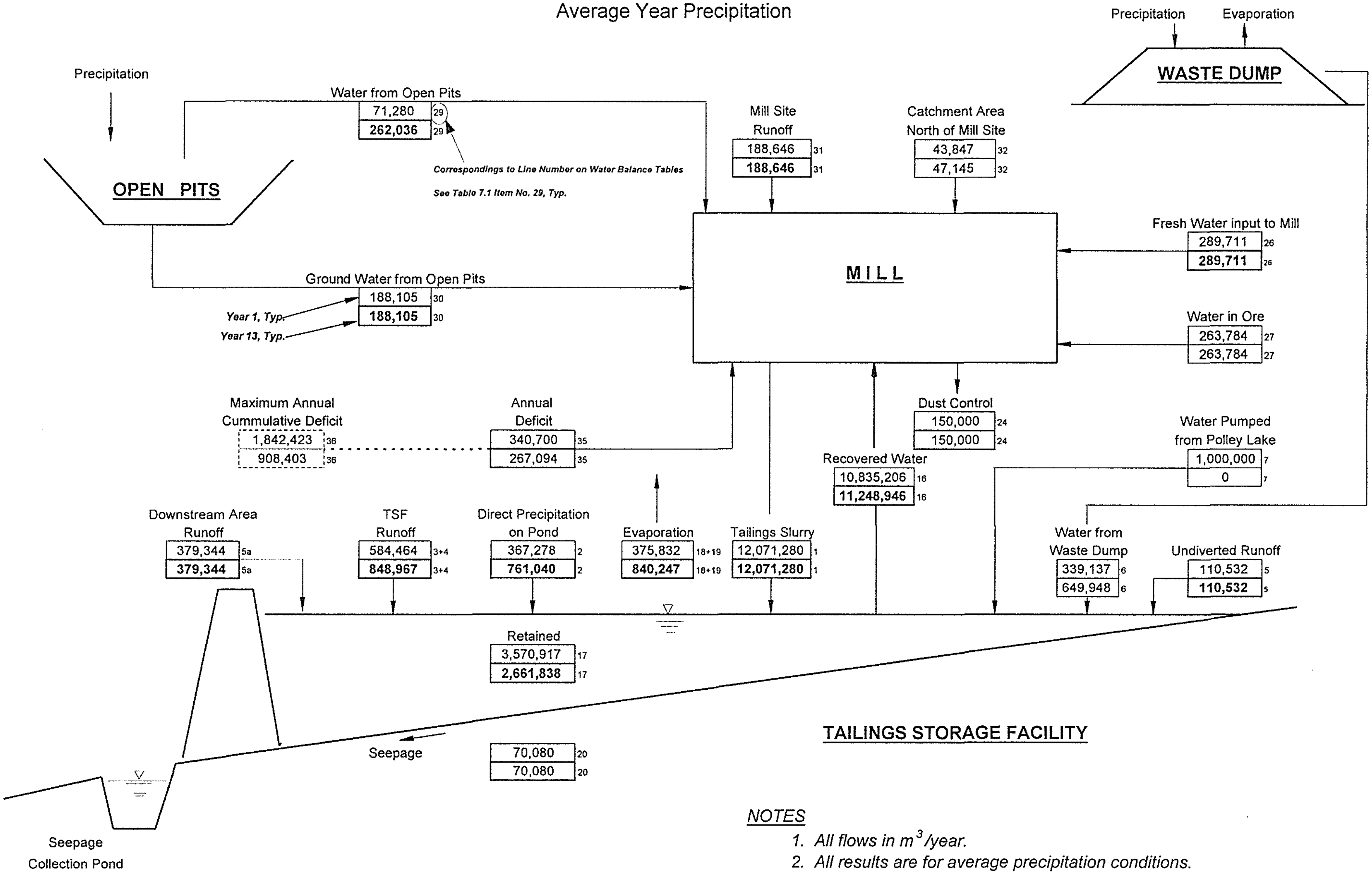
FIGURE 5.4

J:\JOB\DATA\1628\WATERBAL\WBAL-3.XLS fig 11/28/97 10:21 AM

MOUNT POLLEY MINING CORPORATION
MOUNT POLLEY PROJECT

TAILINGS STORAGE FACILITY
SCHEMATIC ILLUSTRATION OF WATER BALANCE - YEAR 1 & 13

Average Year Precipitation



NOTES

1. *All flows in m^3/year.*
2. *All results are for average precipitation conditions.*

KNIGHT PIESOLD LTD.
CONSULTING ENGINEERS

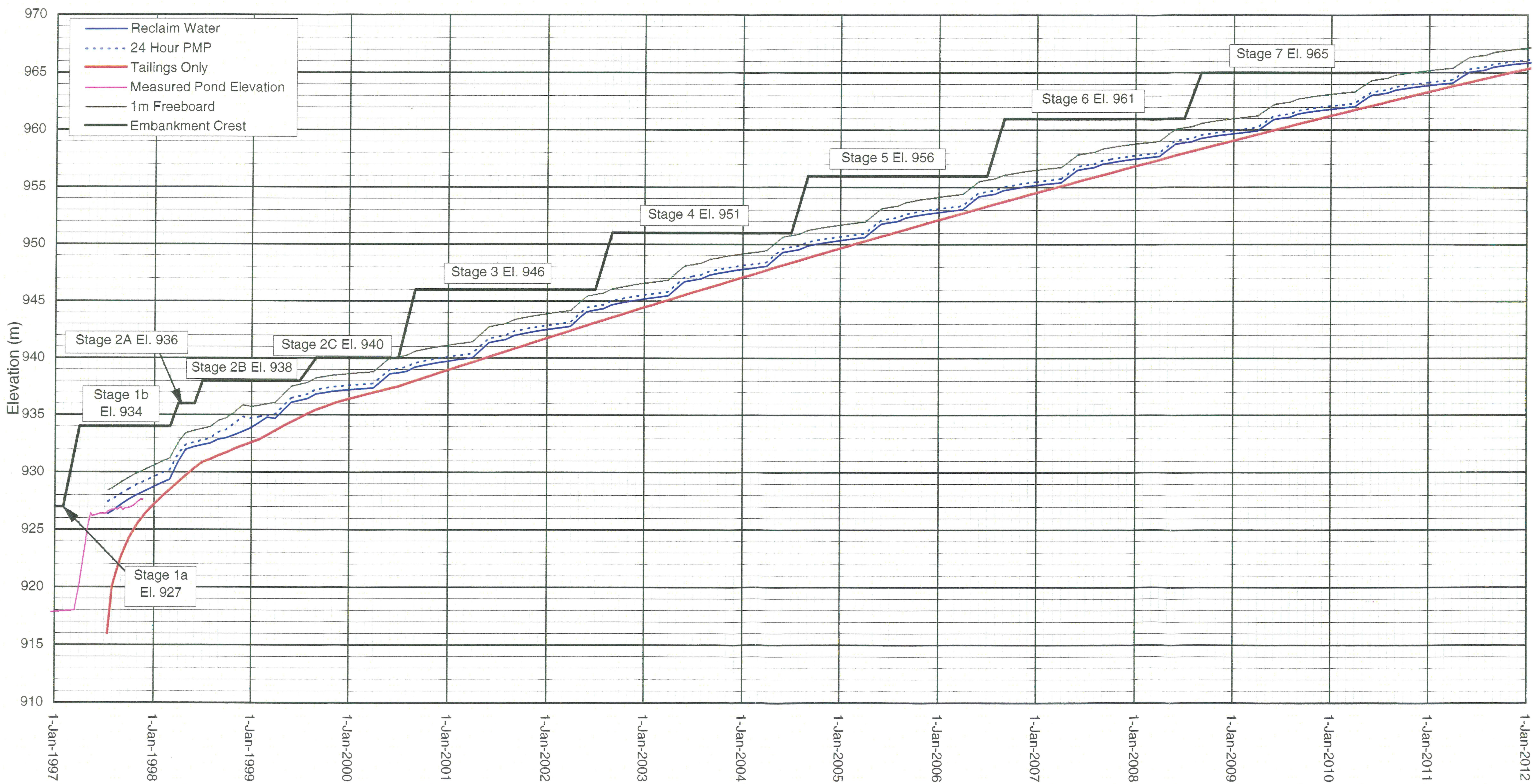
FIGURE 5.5

J:\JOB\DATA\10162\9\misc\DAC.XLS(fillsch) 11/28/97

MOUNT POLLEY MINING CORPORATION
MOUNT POLLEY PROJECT

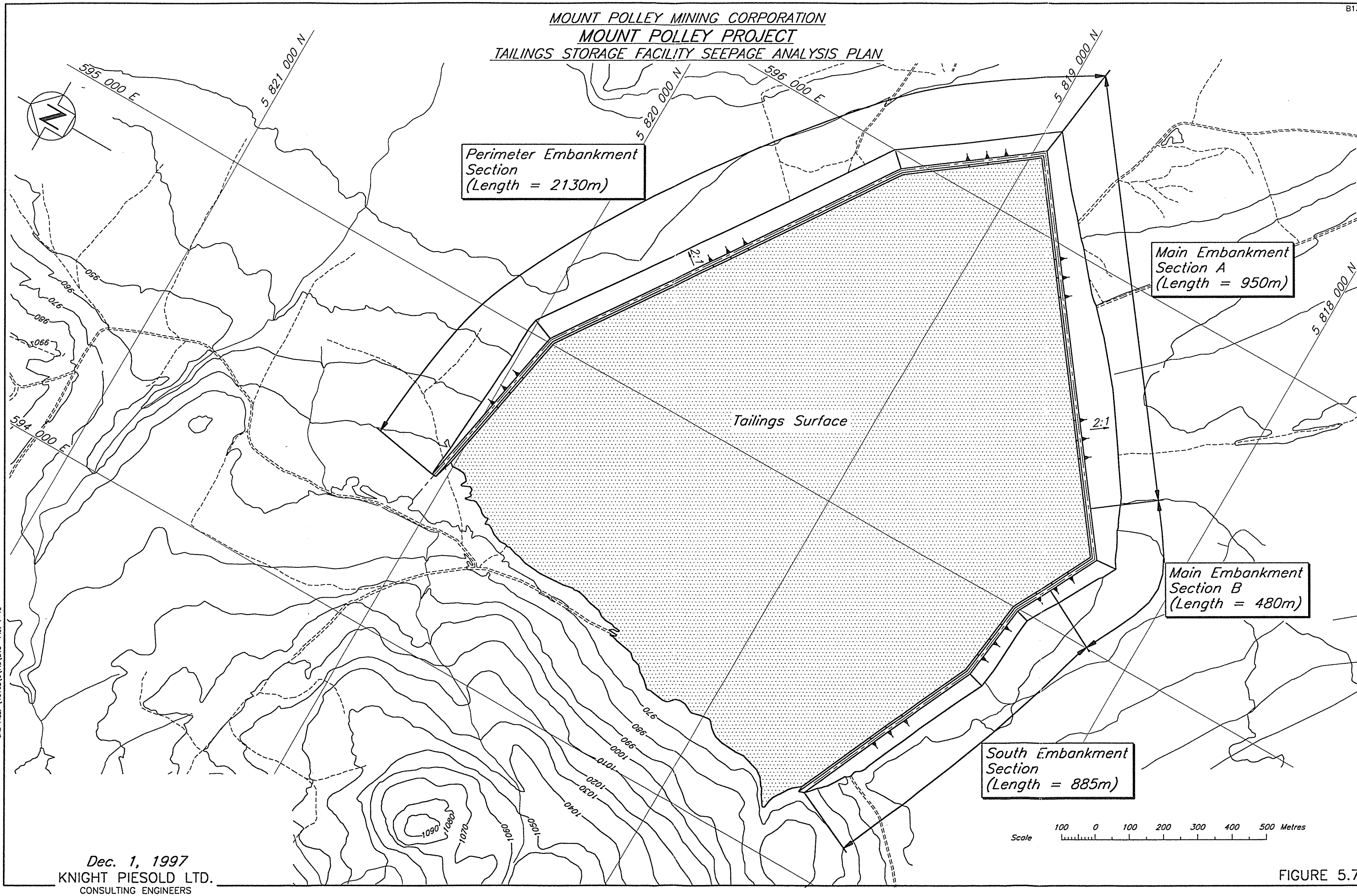
TAILINGS STORAGE FACILITY
FILLING SCHEDULE AND STAGED CONSTRUCTION

November 28, 1997



KNIGHT PIESOLD LTD.
CONSULTING ENGINEERS

FIGURE 5.6


B13
MOUNT POLLEY MINING CORPORATION
MOUNT POLLEY PROJECT
TAILINGS STORAGE FACILITY SEEPAGE ANALYSIS PLAN
595 000 E
596 000 E
594 000 E
5 821 000 N
5 820 000 N
5 819 000 N
5 818 000 N
Perimeter Embankment
Section
(Length = 2130m)
Main Embankment
Section A
(Length = 950m)
Main Embankment
Section B
(Length = 480m)
South Embankment
Section
(Length = 885m)
Tailings Surface
2:1
2:1
Scale
100 0 100 200 300 400 500 Metres
CAD FILE: \10162\9\FIG\B13 Plot 1=10
Dec. 1, 1997
KNIGHT PIESOLD LTD.
CONSULTING ENGINEERS
FIGURE 5.7

MOUNT POLLEY MINING CORPORATION
MOUNT POLLEY PROJECT
PERIMETER EMBANKMENT SEEPAGE ANALYSES
SUMMARY OF MATERIAL PARAMETERS

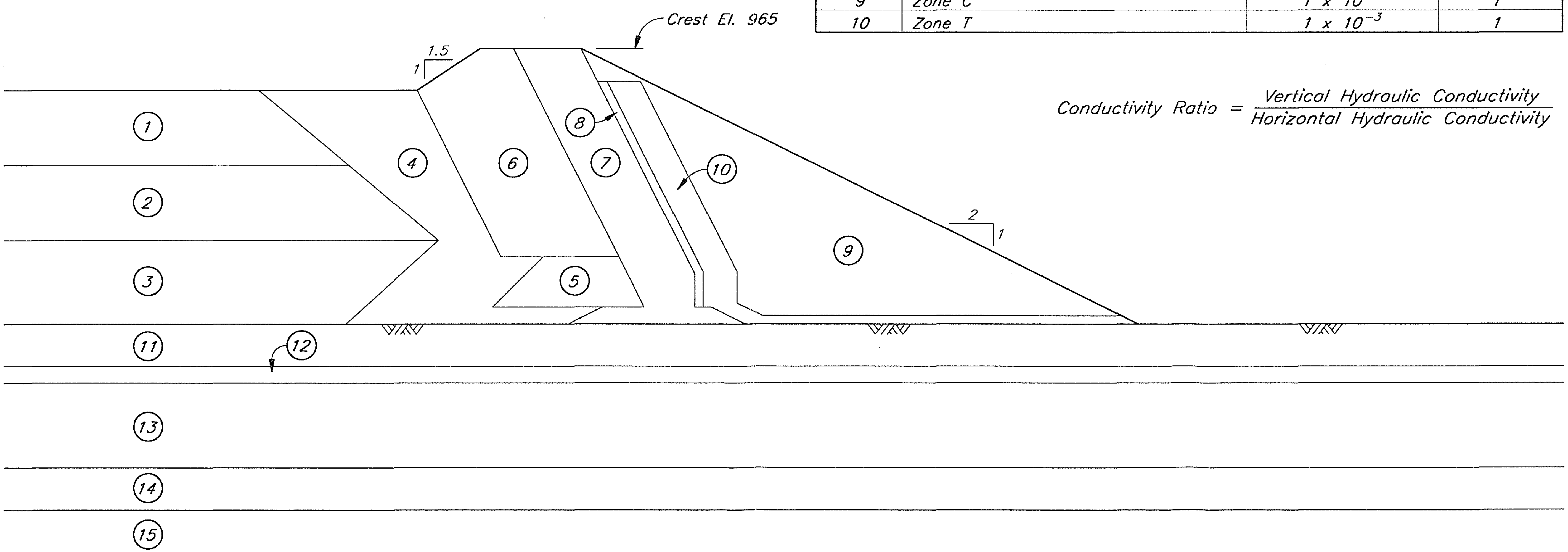
FOUNDATION

ZONE NUMBER	ZONE	DEPTH (m)	VERTICAL HYDRAULIC CONDUCTIVITY (cm/s)	CONDUCTIVITY RATIO
11	Dense to Very Dense Till	0 – 5	1×10^{-7}	1
12	Varved Silt	5 – 7	1×10^{-5}	0.1
13	Basal Till	7 – 17	1×10^{-6}	1
14	Varved Silt	17 – 21	1×10^{-5}	0.1
15	Glaciofluvial Sand and Gravel	>21	1×10^{-4}	1

EMBANKMENT FILL

ZONE NUMBER	ZONE	VERTICAL HYDRAULIC CONDUCTIVITY (cm/s)	CONDUCTIVITY RATIO
1	Tailings El. >946 m	5×10^{-5}	1
2	Tailings El. 934–946 m	1×10^{-5}	1
3	Tailings El. <934 m	1×10^{-6}	1
4	Coarse Tailings	5×10^{-5}	0.1
5	Zone B	1×10^{-6}	0.1–1.0
6	Free Draining Random Fill	1×10^{-4}	1
7	Zone S	5×10^{-7}	0.1–1.0
8	Chimney Drain	1×10^{-2}	1
9	Zone C	1×10^{-3}	1
10	Zone T	1×10^{-3}	1

$$\text{Conductivity Ratio} = \frac{\text{Vertical Hydraulic Conductivity}}{\text{Horizontal Hydraulic Conductivity}}$$



Dec. 1, 1997

KNIGHT PIESOLD LTD.
CONSULTING ENGINEERS

FIGURE 5.8

CAD FILE: \10162\9\FIG\B5 Scale 1:500 Plot 1=0.5

MOUNT POLLEY MINING CORPORATION
MOUNT POLLEY PROJECT
MAIN EMBANKMENT SEEPAGE ANALYSES – SECTIONS A AND B
SUMMARY OF MATERIAL PARAMETERS

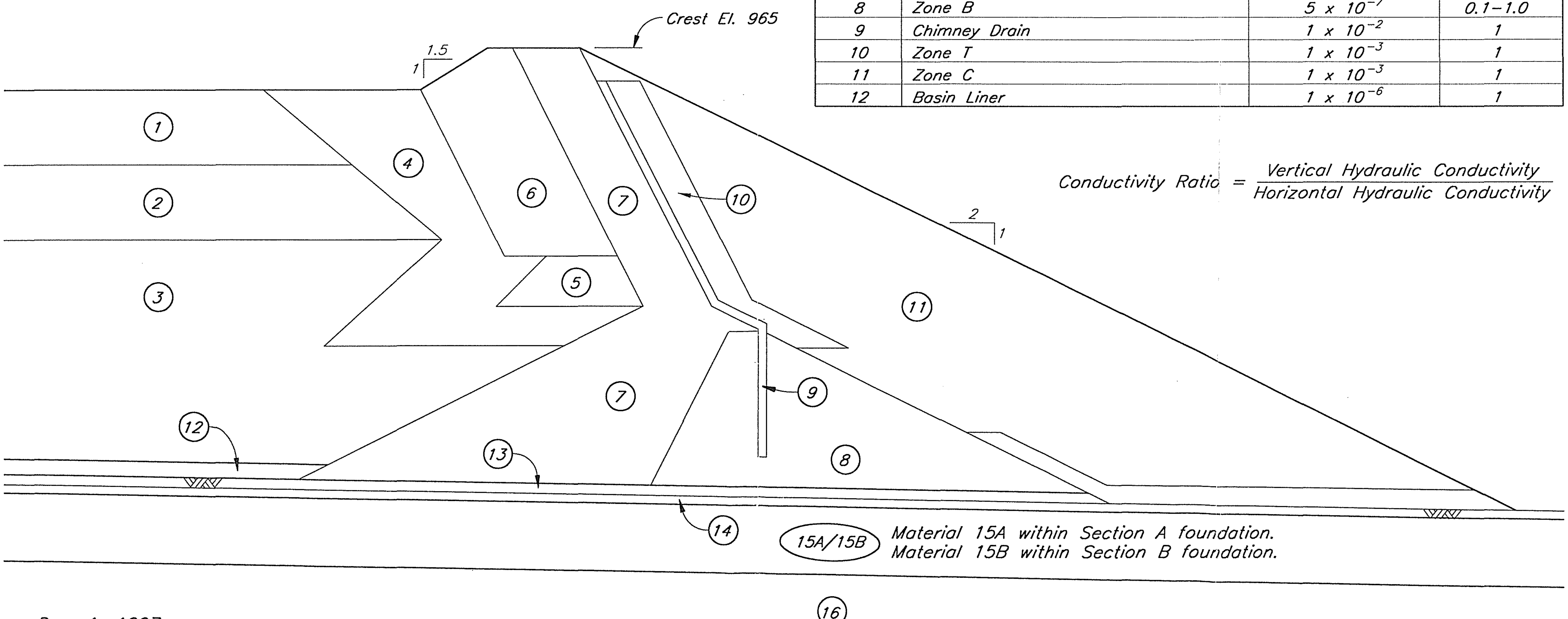
FOUNDATION

ZONE NUMBER	ZONE	DEPTH (m)	VERTICAL HYDRAULIC CONDUCTIVITY (cm/s)	CONDUCTIVITY RATIO
13	Loose to Medium Dense Till	0 – 1.2	1×10^{-7}	1
14	Dense to Very Dense Till	1.2 – 2.2	1×10^{-7}	1
15A	Varved Silt	2.2 – 12.7	1×10^{-5}	0.1
15B	Sand	2.2 – 12.7	1×10^{-4}	1
16	Basal Till	>12.7	1×10^{-6}	1

EMBANKMENT FILL

ZONE NUMBER	ZONE	VERTICAL HYDRAULIC CONDUCTIVITY (cm/s)	CONDUCTIVITY RATIO
1	Tailings El. >946 m	5×10^{-5}	1
2	Tailings El. 934–946 m	1×10^{-5}	1
3	Tailings El. <934 m	1×10^{-6}	1
4	Coarse Tailings	5×10^{-5}	0.1
5	Zone B	1×10^{-6}	0.1–1.0
6	Free Draining Random Fill	1×10^{-4}	1
7	Zone S	5×10^{-7}	0.1–1.0
8	Zone B	5×10^{-7}	0.1–1.0
9	Chimney Drain	1×10^{-2}	1
10	Zone T	1×10^{-3}	1
11	Zone C	1×10^{-3}	1
12	Basin Liner	1×10^{-6}	1

$$\text{Conductivity Ratio} = \frac{\text{Vertical Hydraulic Conductivity}}{\text{Horizontal Hydraulic Conductivity}}$$



Material 15A within Section A foundation.
Material 15B within Section B foundation.

Dec. 1, 1997

KNIGHT PIESOLD LTD.
CONSULTING ENGINEERS

FIGURE 5.9

CAD FILE: \10162\9\FIG\B1 Scale 1:500 Plot 1=0.5

MOUNT POLLEY MINING CORPORATION
MOUNT POLLEY PROJECT
SOUTH EMBANKMENT SEEPAGE ANALYSES
SUMMARY OF MATERIAL PARAMETERS

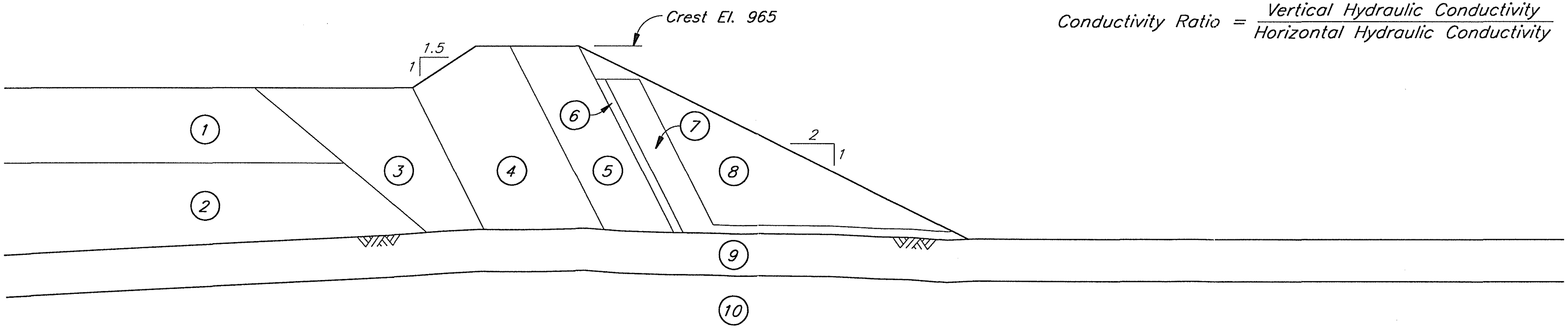
FOUNDATION

ZONE NUMBER	ZONE	DEPTH (m)	VERTICAL HYDRAULIC CONDUCTIVITY (cm/s)	CONDUCTIVITY RATIO
9	Loose to Medium Dense Till	0 – 5	1×10^{-7}	1
10	Volcanic Conglomerate Bedrock	>5	1×10^{-6}	1

EMBANKMENT FILL

ZONE NUMBER	ZONE	VERTICAL HYDRAULIC CONDUCTIVITY (cm/s)	CONDUCTIVITY RATIO
1	Tailings El. >946 m	5×10^{-5}	1
2	Tailings El. <946 m	1×10^{-5}	1
3	Coarse Tailings	5×10^{-5}	0.1
4	Free Draining Random Fill	1×10^{-4}	1
5	Zone S	5×10^{-7}	0.1–1.0
6	Chimney Drain	1×10^{-2}	1
7	Zone T	1×10^{-3}	1
8	Zone C	1×10^{-3}	1

$$\text{Conductivity Ratio} = \frac{\text{Vertical Hydraulic Conductivity}}{\text{Horizontal Hydraulic Conductivity}}$$



Nov. 28, 1997

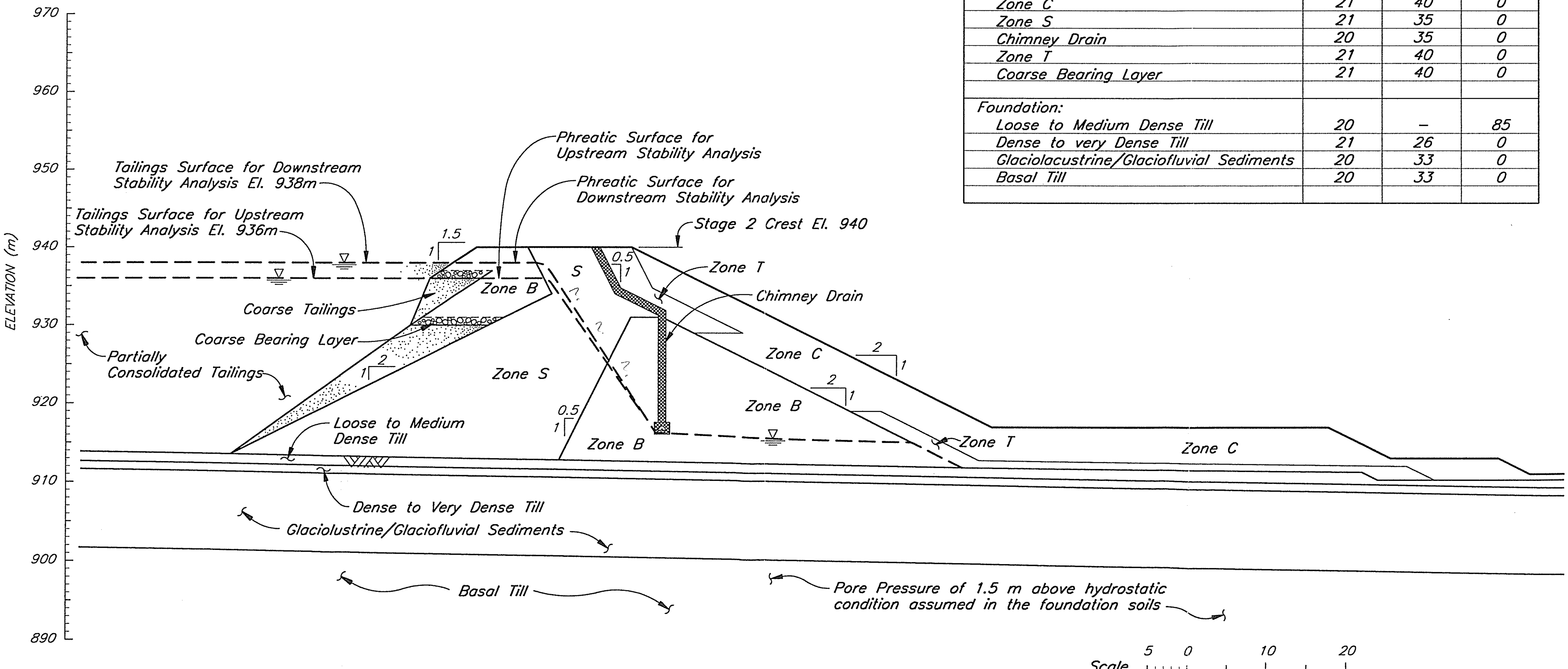
KNIGHT PIESOLD LTD.
CONSULTING ENGINEERS

FIGURE 5.10

CAD FILE: \10162\9\FIG\B6 Scale 1:500 Plot 1=0.5

MOUNT POLLEY MINING CORPORATION
MOUNT POLLEY PROJECT
STAGE 2 MAIN EMBANKMENT STABILITY ANALYSES
GEOMETRY AND MATERIAL PARAMETERS

MATERIAL TYPE	Unit wt. (kN/m^3)	ø' (degrees)	c_u, c' (kPa)
Tailings:			
Partially Consolidated	18	–	10–35
Coarse Tailings	19	30	0
Embankment Fill:			
Zone B	22	35	0
Zone C	21	40	0
Zone S	21	35	0
Chimney Drain	20	35	0
Zone T	21	40	0
Coarse Bearing Layer	21	40	0
Foundation:			
Loose to Medium Dense Till	20	–	85
Dense to very Dense Till	21	26	0
Glaciolacustrine/Glaciofluvial Sediments	20	33	0
Basal Till	20	33	0



Dec. 1, 1997

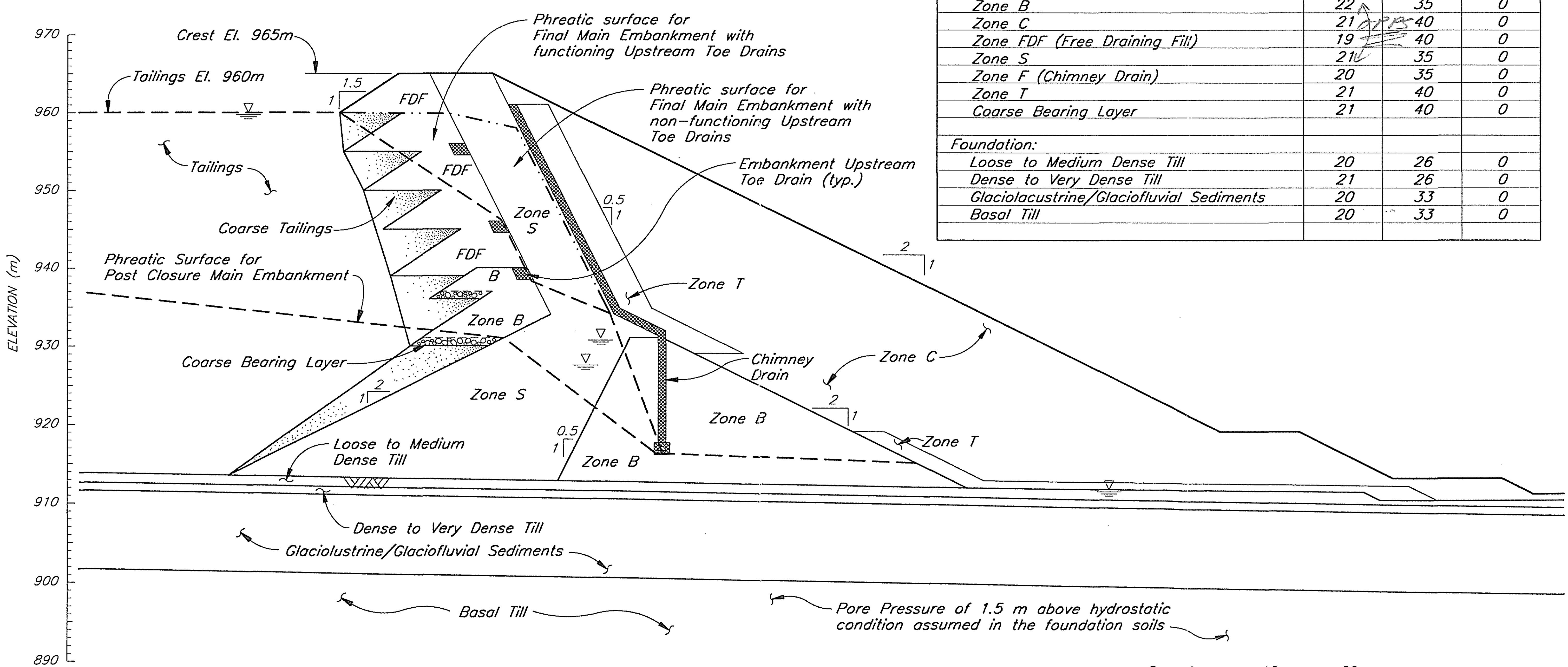
KNIGHT PIESOLD LTD.
CONSULTING ENGINEERS

FIGURE 5.11

CAD FILE:/10162/9/FIG/B7 Plot 1=1

MOUNT POLLEY MINING CORPORATION
MOUNT POLLEY PROJECT
FINAL MAIN EMBANKMENT STABILITY ANALYSES
GEOMETRY AND MATERIAL PARAMETERS

MATERIAL TYPE	Unit wt. (kN/m^3)	ø' (degrees)	c_u, c' (kPa)
Tailings:			
Partially Consolidated	18	–	10–55
Fully Consolidated (Post Closure Analysis)	19	30	0
Residual (Post Liquefaction) Strength	18	–	10
Coarse Tailings	19	30	0
Embankment Fill:			
Zone B	22	35	0
Zone C	21	40	0
Zone FDF (Free Draining Fill)	19	40	0
Zone S	21	35	0
Zone F (Chimney Drain)	20	35	0
Zone T	21	40	0
Coarse Bearing Layer	21	40	0
Foundation:			
Loose to Medium Dense Till	20	26	0
Dense to Very Dense Till	21	26	0
Glaciolacustrine/Glaciofluvial Sediments	20	33	0
Basal Till	20	33	0



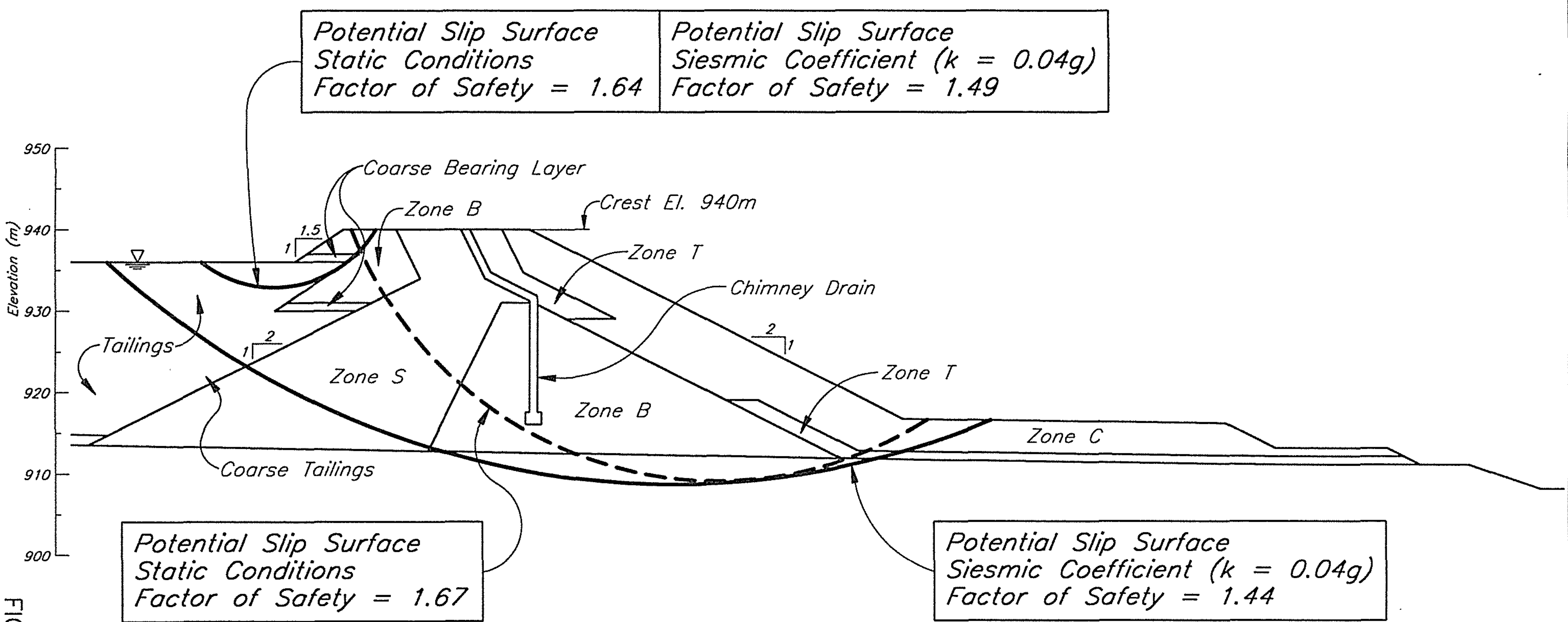
Dec. 1, 1997
KNIGHT PIESOLD LTD.
CONSULTING ENGINEERS

FIGURE 5.12

CAD FILE:/10162/9/FIG/B4 Plot 1=1

CAD FILE: \10162\9\FIG\A7 Plot 1=0.75

MOUNT POLLEY MINING CORPORATION
MOUNT POLLEY PROJECT
RESULTS OF STAGE 2 MAIN EMBANKMENT STABILITY ANALYSES



KNIGHT PIESOLD LTD.
CONSULTING ENGINEERS

Dec. 1, 1997

FIGURE 5.13

A7

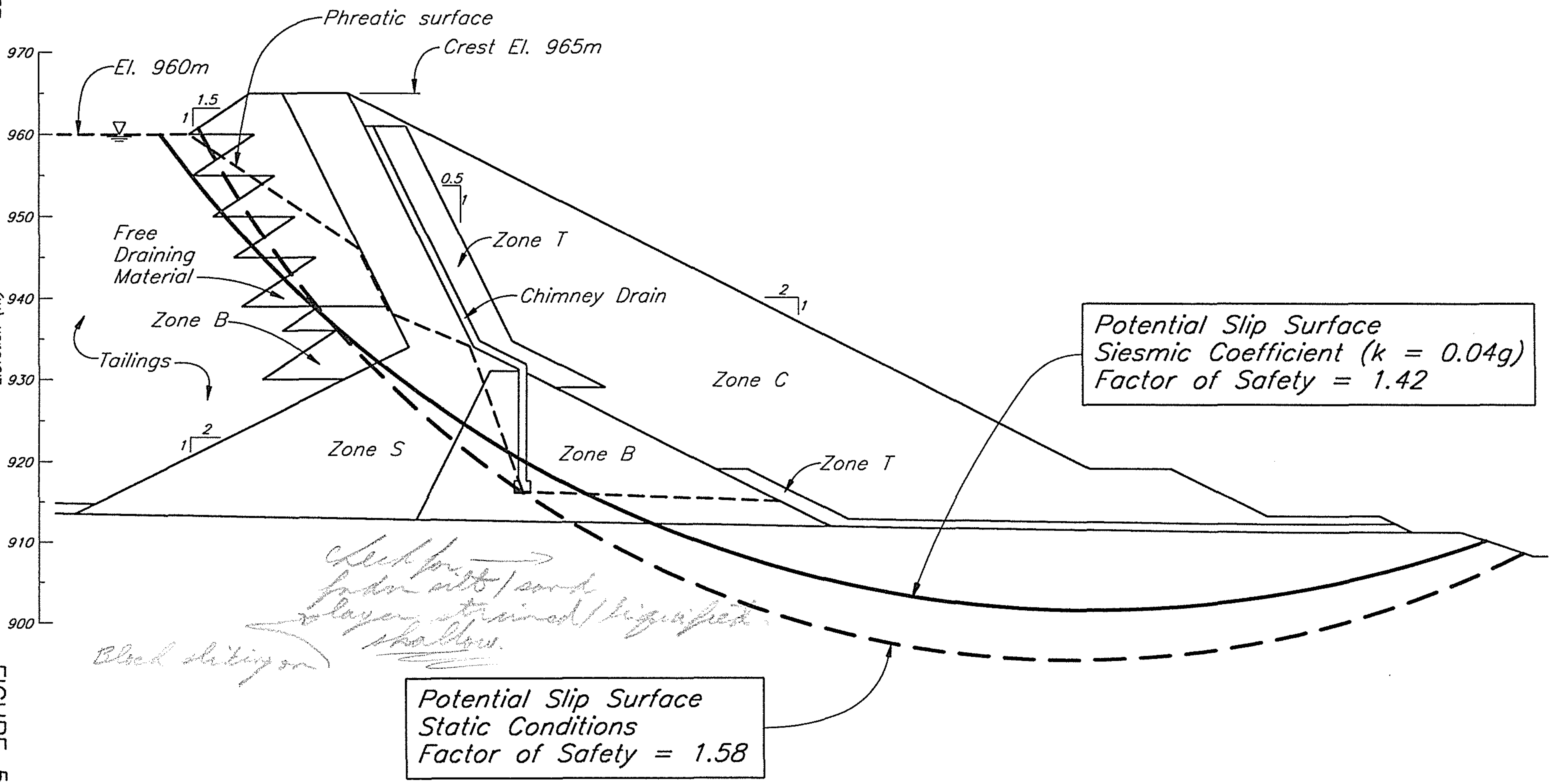
CAD FILE: \10162\9\FIG\A4 Plot 1=0.75

MOUNT POLLEY MINING CORPORATION

MOUNT POLLEY PROJECT

RESULTS OF FINAL MAIN EMBANKMENT DOWNSTREAM STABILITY ANALYSES

UPSTREAM TOE DRAINS FUNCTIONING



Dec. 1, 1997

KNIGHT PIESOLD LTD.

CONSULTING ENGINEERS

FIGURE 5.14

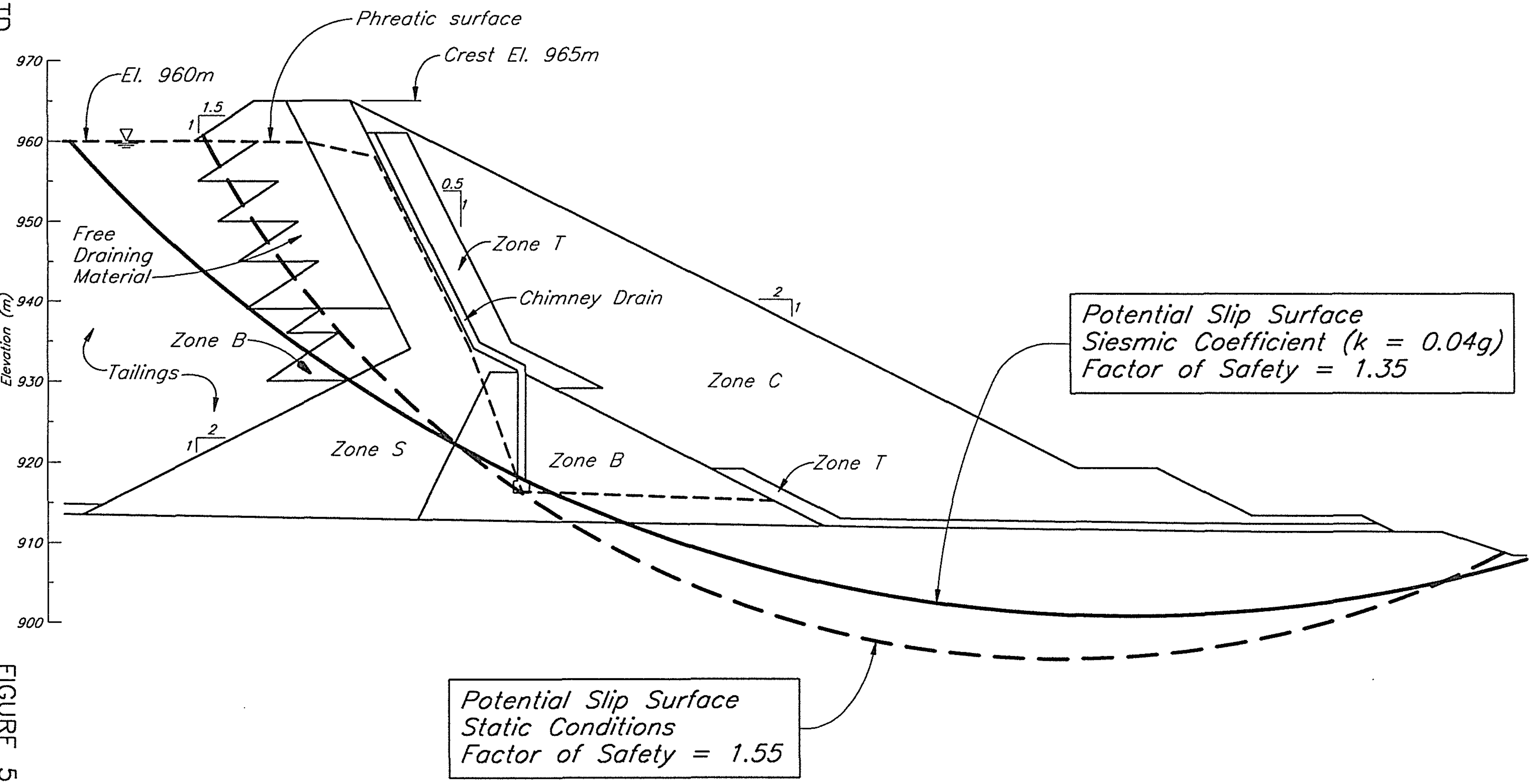
A4

CAD FILE: \10162\9\FIG\A8 Plot 1=0.75

MOUNT POLLEY MINING CORPORATION

MOUNT POLLEY PROJECT

RESULTS OF FINAL MAIN EMBANKMENT DOWNSTREAM STABILITY ANALYSES
UPSTREAM TOE DRAINS NOT FUNCTIONING



A8

Dec. 1, 1997

KNIGHT PIESOLD LTD.
CONSULTING ENGINEERS

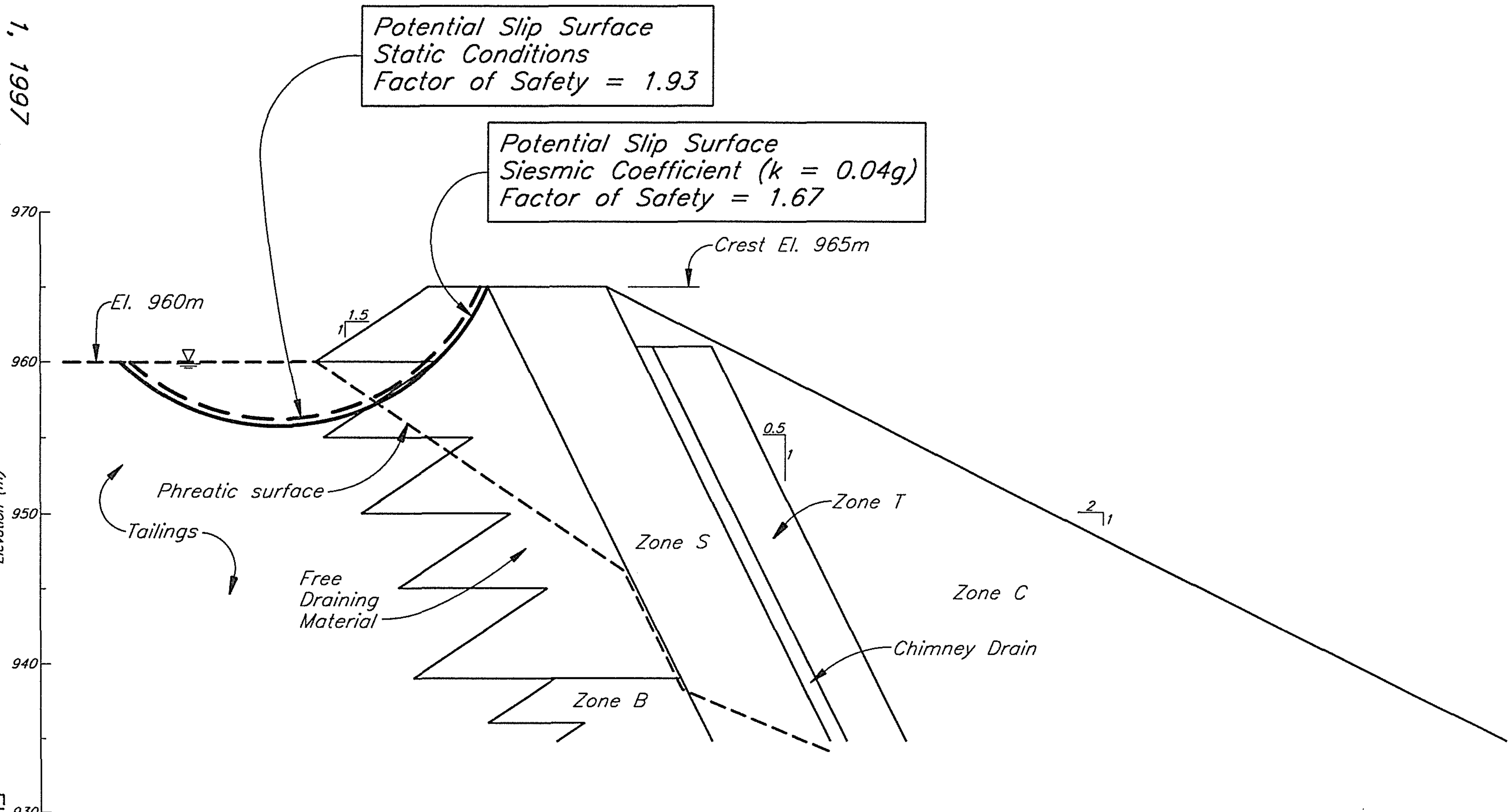
FIGURE 5.15

CAD FILE: \10162\9\FIG\A9 Plot 1=0.40

MOUNT POLLEY MINING CORPORATION

MOUNT POLLEY PROJECT

RESULTS OF FINAL MAIN EMBANKMENT UPSTREAM STABILITY ANALYSES
UPSTREAM TOE DRAINS FUNCTIONING



Dec. 1, 1997

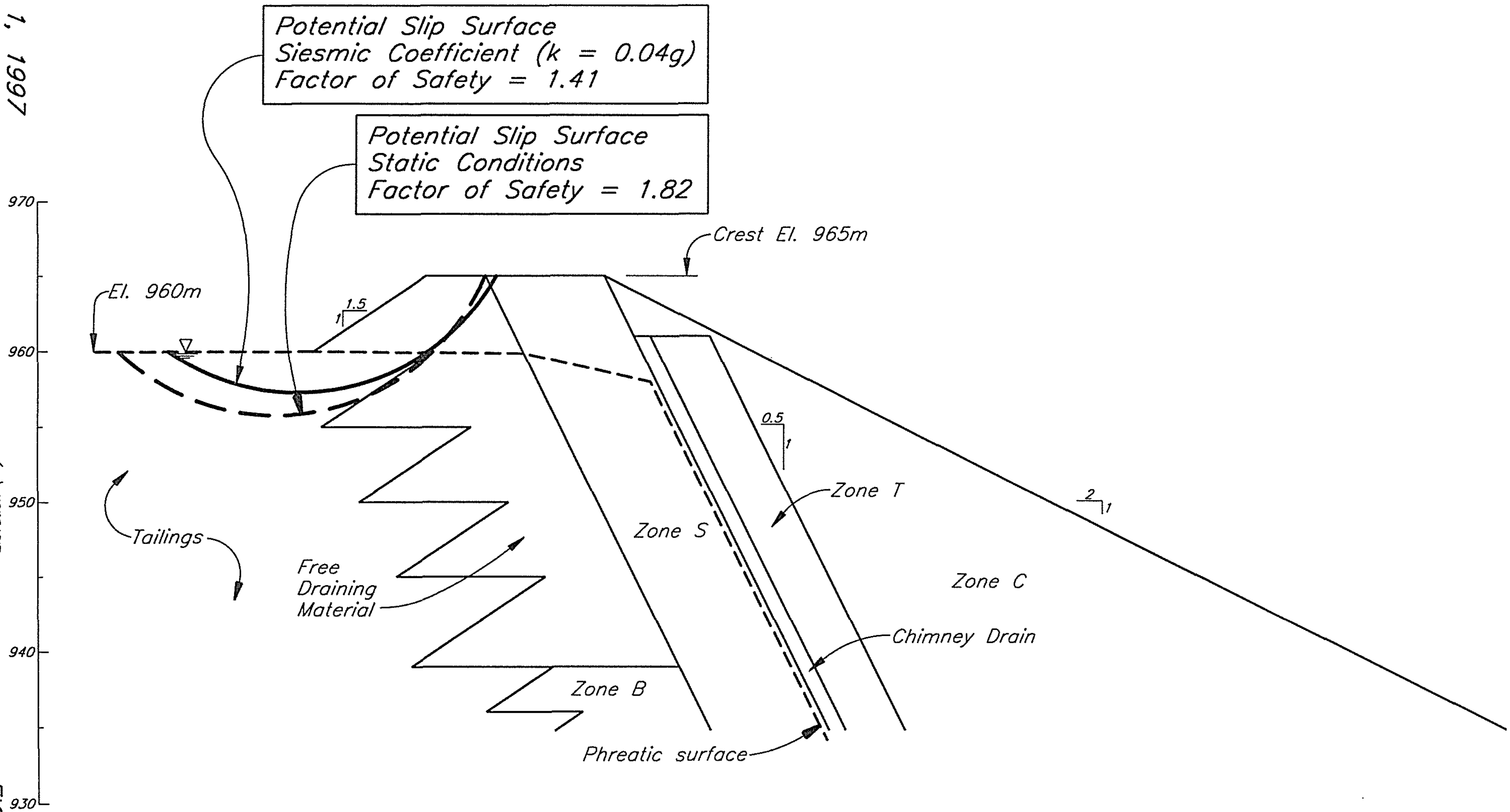
KNIGHT PIESOLD LTD.
CONSULTING ENGINEERS

FIGURE 5.16

A9

CAD FILE: \10162\9\FIG\A10 Plot 1=0.40

MOUNT POLLEY MINING CORPORATION
MOUNT POLLEY PROJECT
RESULTS OF FINAL MAIN EMBANKMENT UPSTREAM STABILITY ANALYSES
UPSTREAM TOE DRAINS NOT FUNCTIONING



Dec. 1, 1997
KNIGHT PIESOLD LTD.
CONSULTING ENGINEERS

FIGURE 5.17

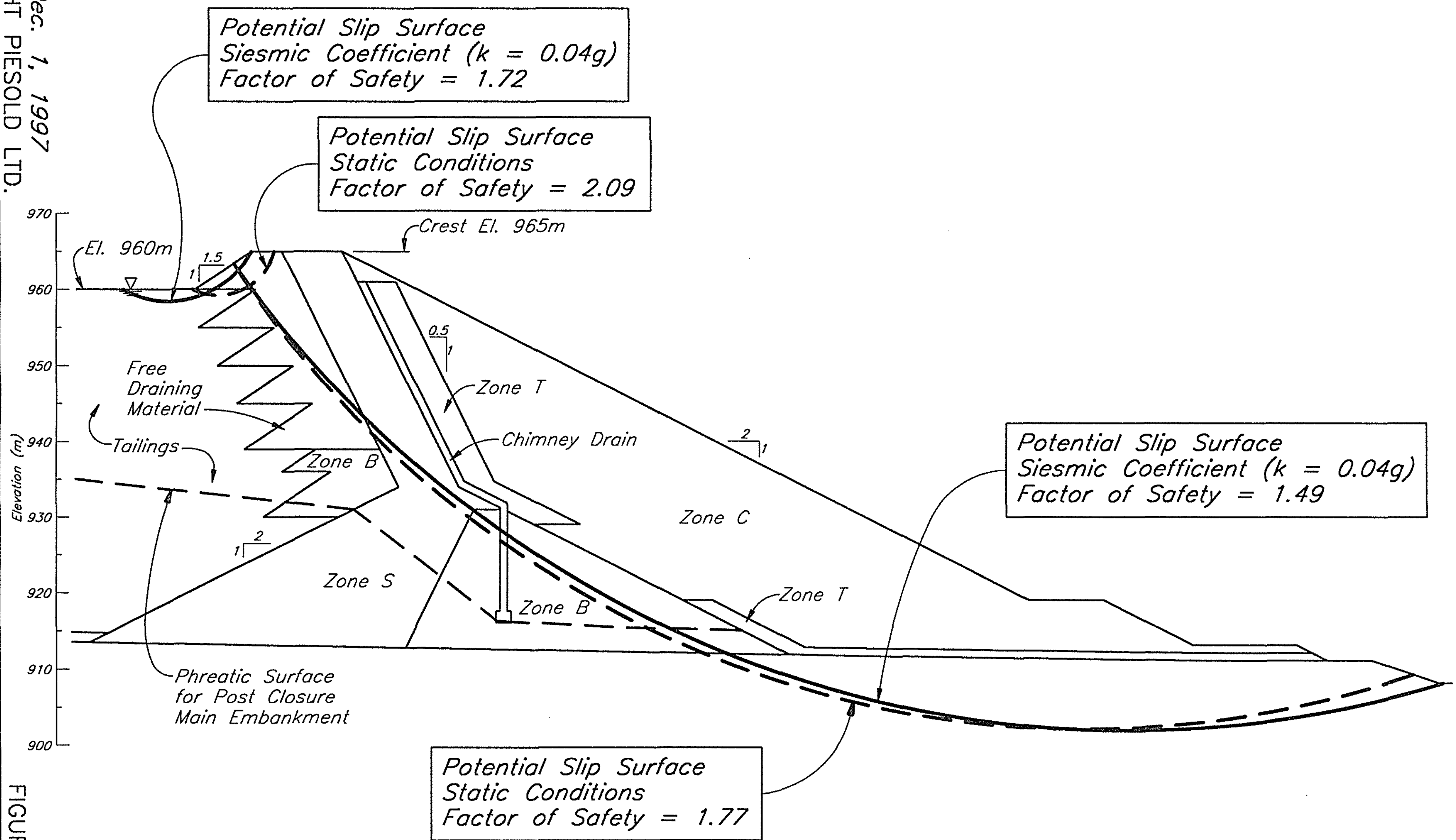
A10

CAD FILE: \10162\9\FIG\A12 Plot 1=0.75

MOUNT POLLEY MINING CORPORATION

MOUNT POLLEY PROJECT

POST CLOSURE MAIN EMBANKMENT STABILITY ANALYSES



Dec. 1, 1997

KNIGHT PIESOLD LTD.
CONSULTING ENGINEERS

FIGURE 5.18

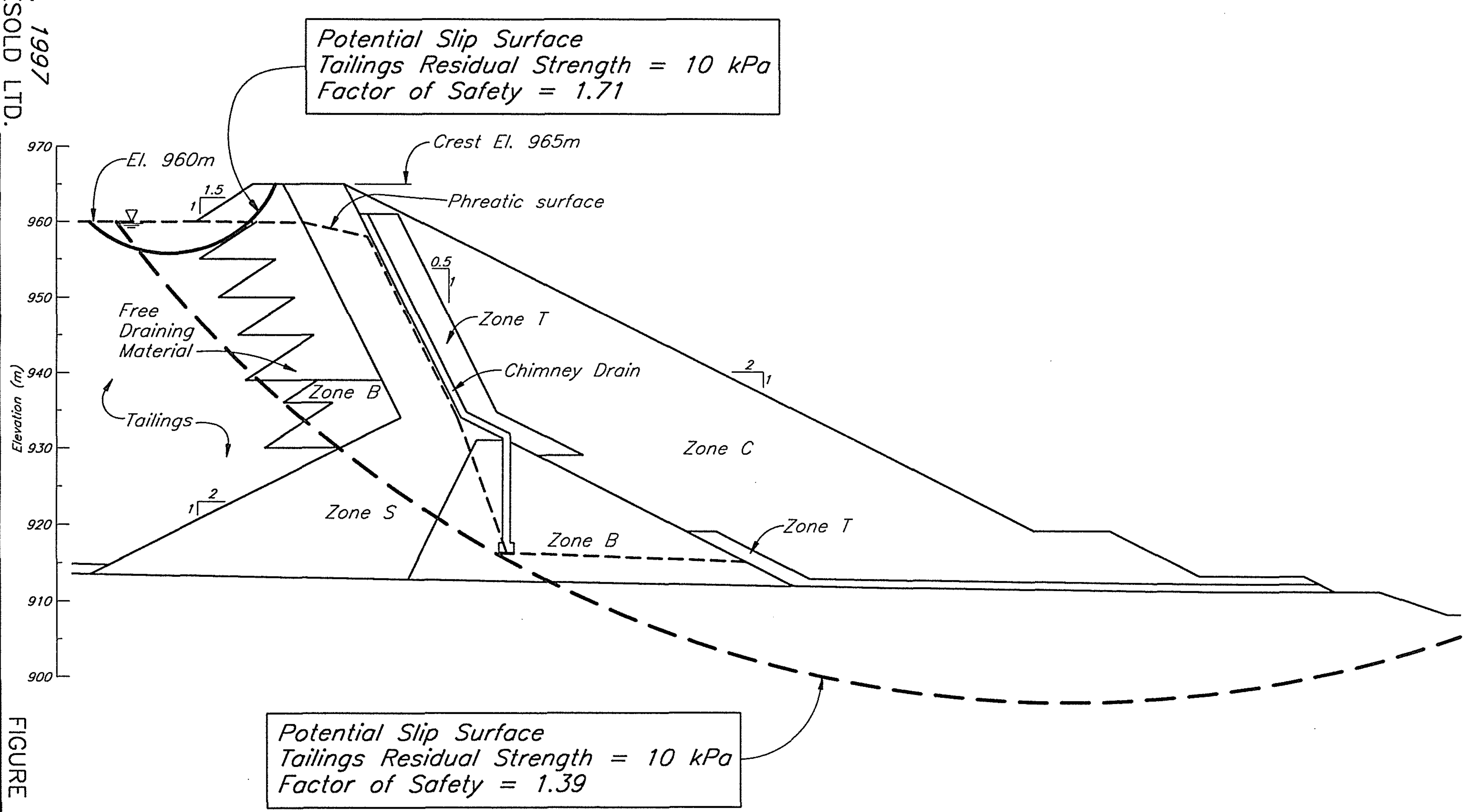
A12

CAD FILE: \10162\9\FIG\A11 Plot 1=0.75

MOUNT POLLEY MINING CORPORATION

MOUNT POLLEY PROJECT

RESULTS OF FINAL MAIN EMBANKMENT STABILITY ANALYSES
POST LIQUIFACTION

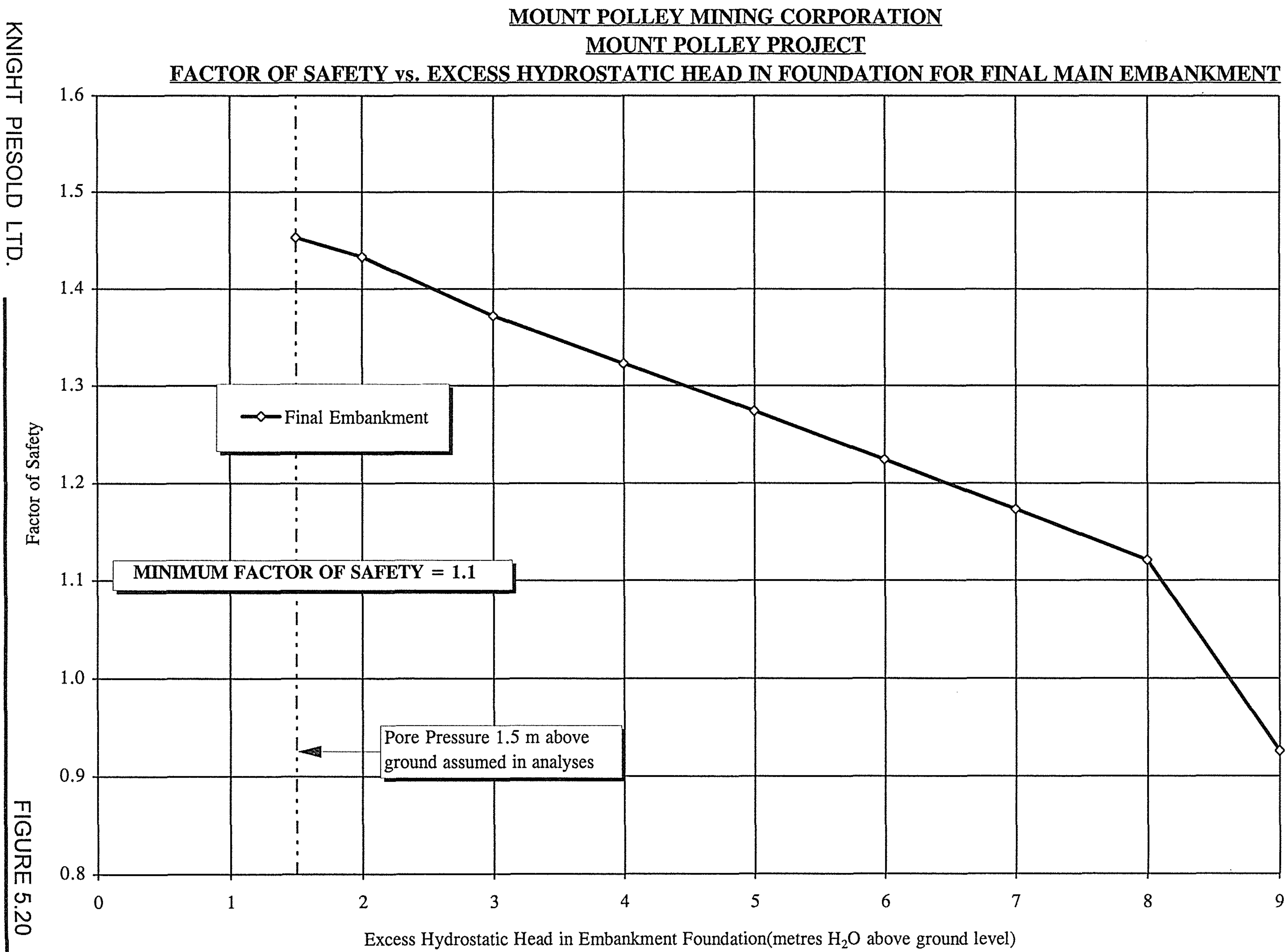


Dec. 1, 1997

KNIGHT PIESOLD LTD.
CONSULTING ENGINEERS

FIGURE 5.19

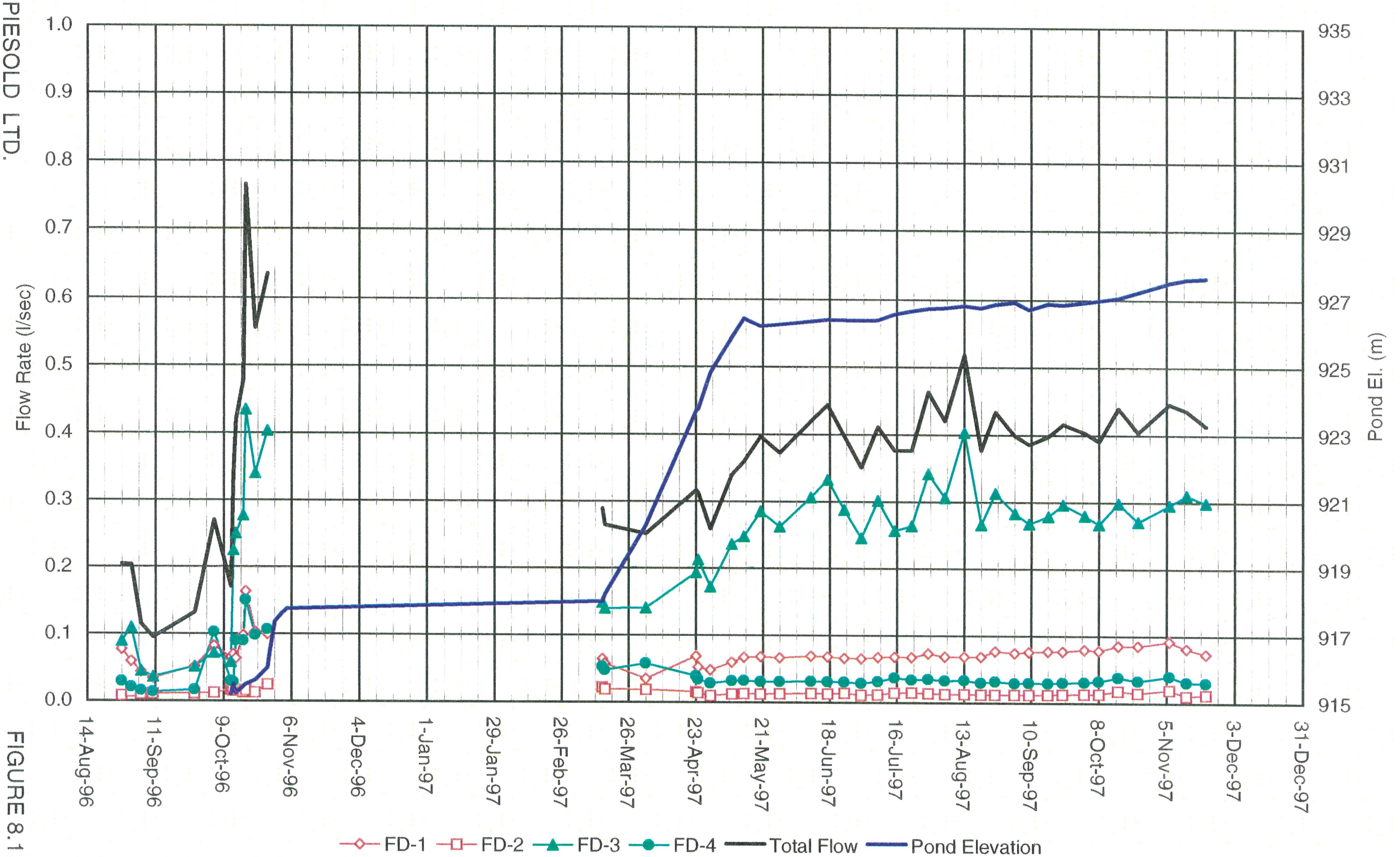
A11



KNIGHT PIESOLD LTD.
CONSULTING ENGINEERS

FIGURE 5.20

MOUNT POLLEY MINING CORPORATION
MOUNT POLLEY PROJECT
MAIN EMBANKMENT FOUNDATION DRAIN FLOWS



KNIGHT PIESOLD LTD.
CONSULTING ENGINEERS

FIGURE 8.1

J:\JOB\DATA\1(\MISC\PIEZOS.XLS * Plane A plots Chart 1 11/28/97 10:53 AM

MOUNT POLLEY MINING CORPORATION
MOUNT POLLEY PROJECT
TAILINGS STORAGE FACILITY
SUMMARY PLOT PIEZOMETER PLANE A

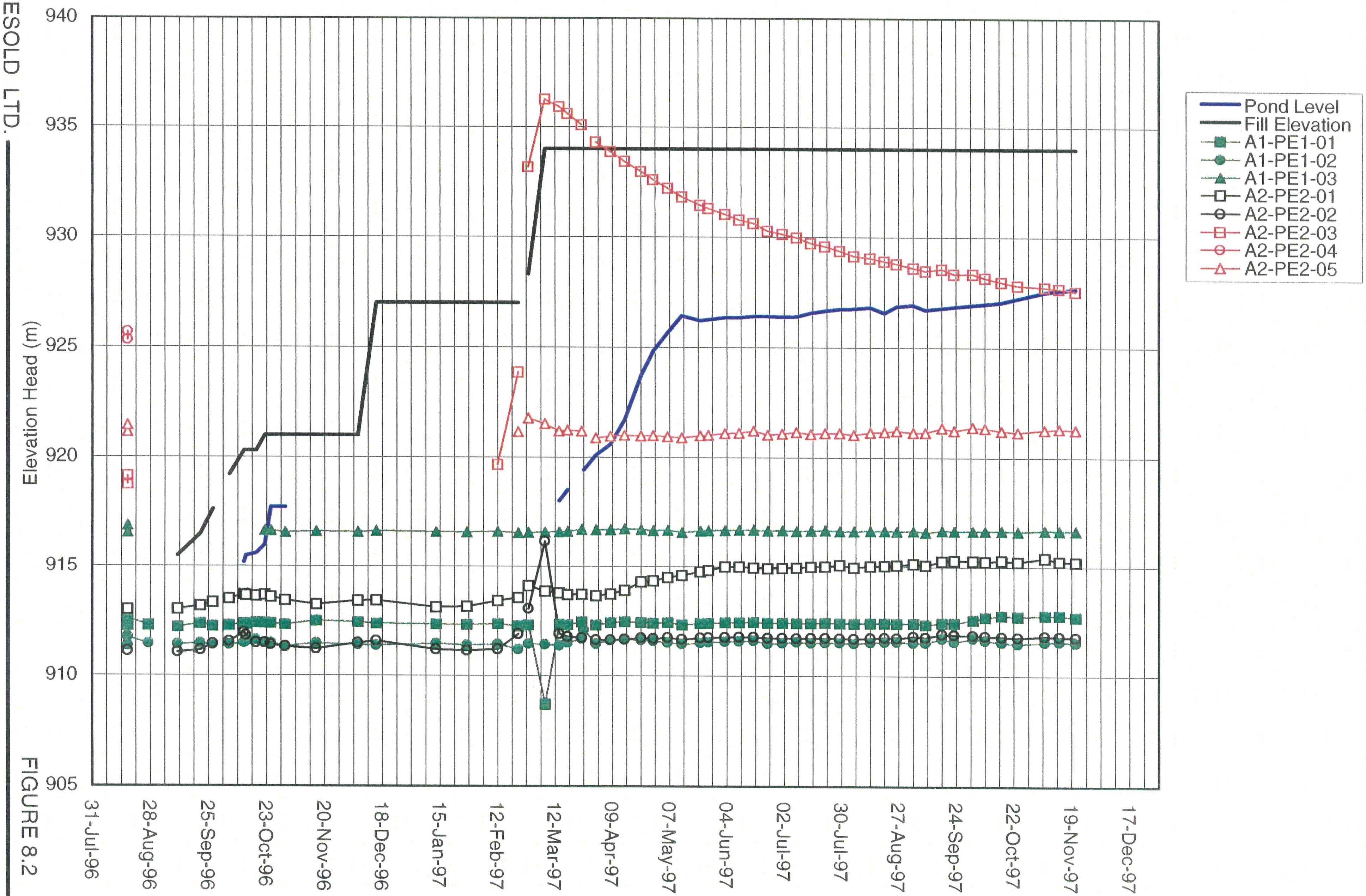
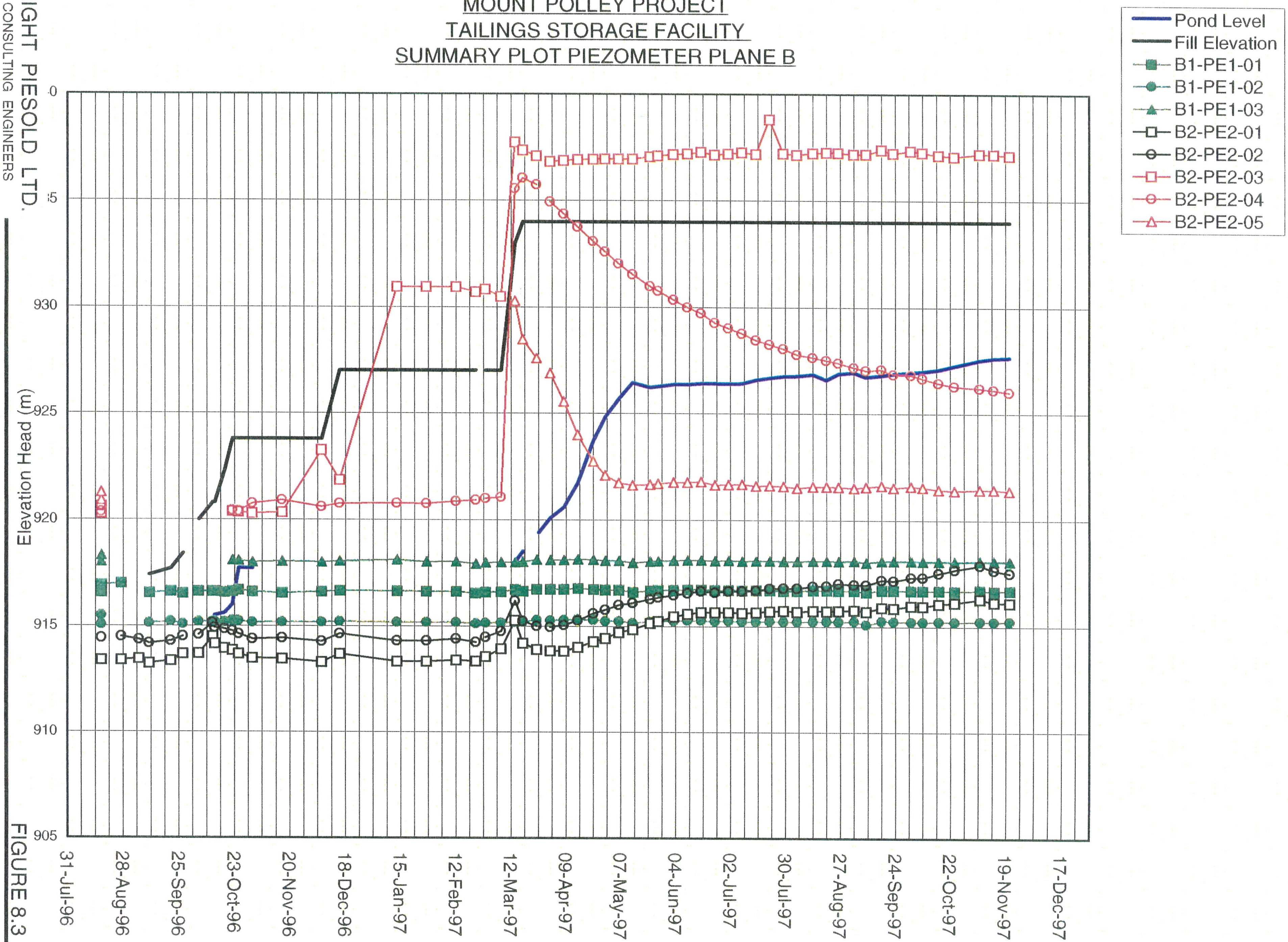


FIGURE 8.2

KNIGHT PIESOLD LTD.
CONSULTING ENGINEERS

J:\JOB\DATA\10... 9\MISC\piezos.xlx * Plane B plots Chart 1 11/28/97 10:53 AM

MOUNT POLLEY MINING CORP0RATION
MOUNT POLLEY PROJECT
TAILINGS STORAGE FACILITY
SUMMARY PLOT PIEZOMETER PLANE B



KNIGHT PIESOLD LTD.
CONSULTING ENGINEERS

FIGURE 8.3

J:\JOB\DATA\1 \MISC\piezos.xls * Plane C plots Chart 1 11/28/97 10:49 AM

MOUNT POLLEY MINING CORP0RATION
MOUNT POLLEY PROJECT
TAILINGS STORAGE FACILITY
SUMMARY PLOT PIEZOMETER PLANE C

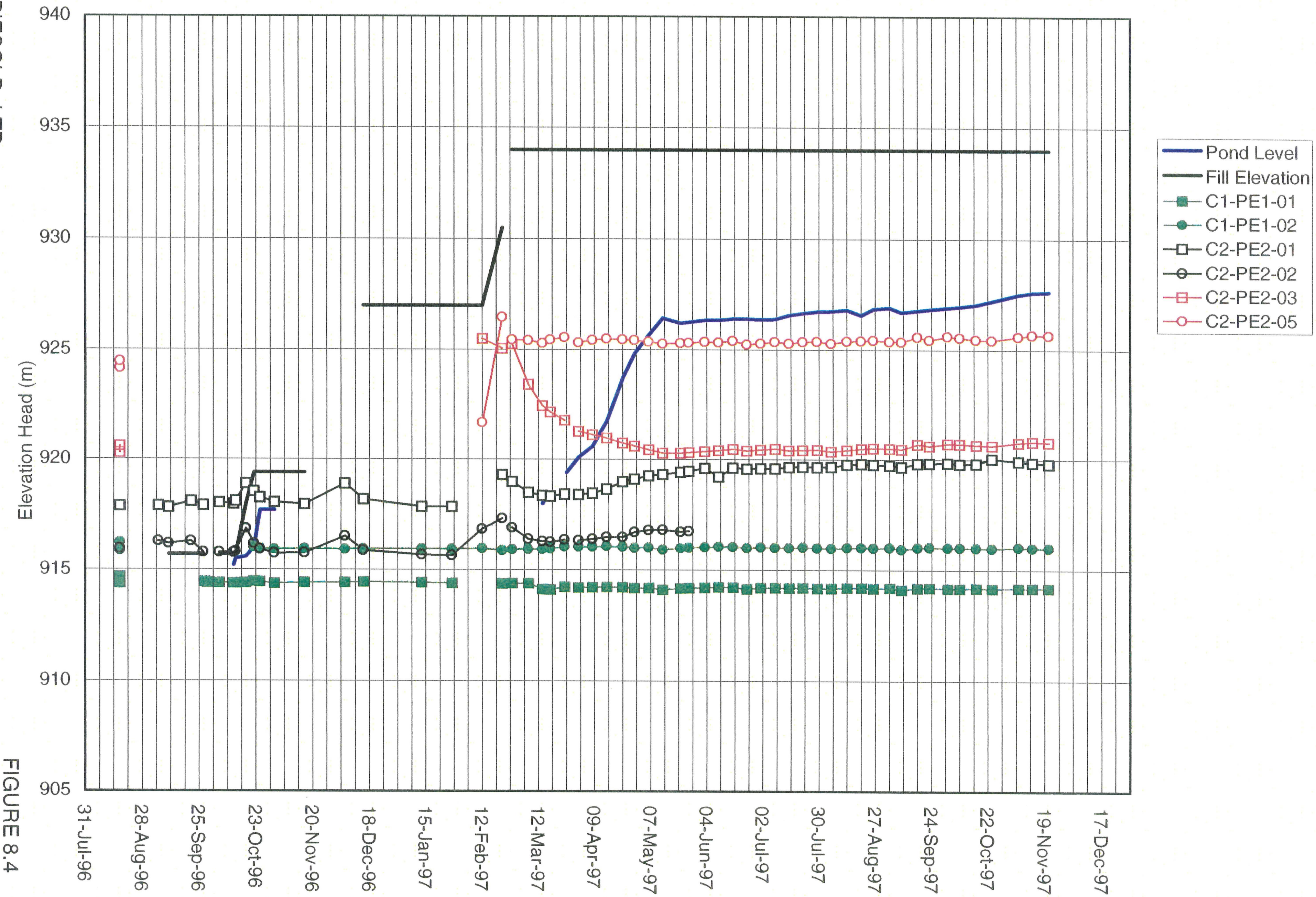
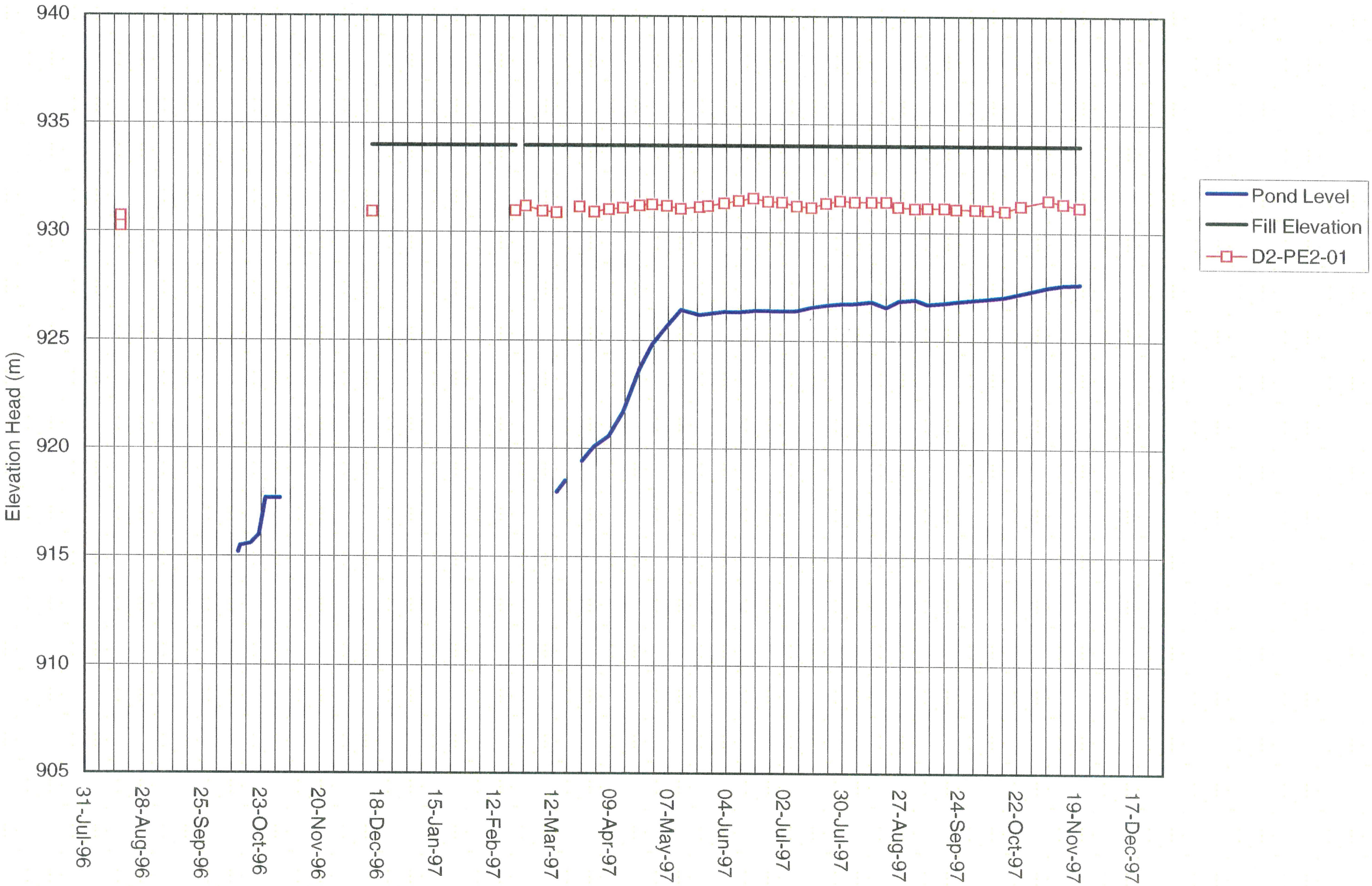


FIGURE 8.4

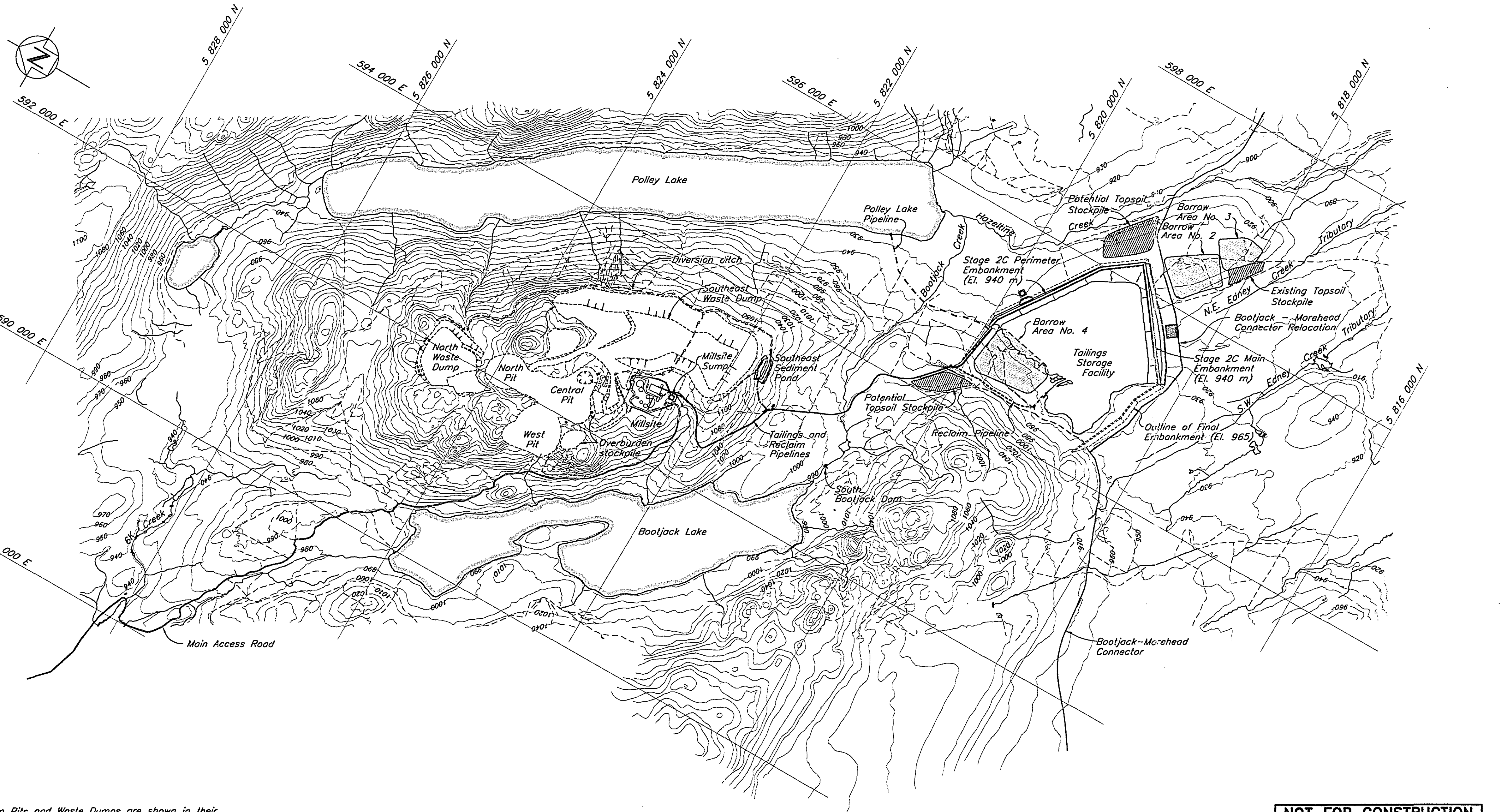
KNIGHT PIESOLD LTD.
CONSULTING ENGINEERS

MOUNT POLLEY MINING CORP0RATION
MOUNT POLLEY PROJECT
TAILINGS STORAGE FACILITY
SUMMARY PLOT PIEZOMETER PLANE D



KNIGHT PIESOLD LTD.
CONSULTING ENGINEERS

FIGURE 8.5



NOT FOR CONSTRUCTION

Scale 1000 500 0 1000 2000 m

NOTES

1. Open Pits and Waste Dumps are shown in their final configurations.
2. Pond level in Tailings Storage Facility projected at El. 936 (July 1999).
3. Topography has not been updated from 1997 Flyover.

DRG. NO.	DESCRIPTION
REFERENCE DRAWINGS	

REV.	DATE	DESCRIPTION	APPROVED
REVISIONS			

REV.	DATE	DESCRIPTION	APPROVED
0	NOV 10/97	ISSUED FOR TENDER	
REVISIONS			

KNIGHT PIESOLD LIMITED
CONSULTING ENGINEERS – VANCOUVER, B.C.

PROFESSIONAL PROVINCE OF BRITISH COLUMBIA K. D. K. EMBREE ENGINEER

DESIGNED	KDE/KJB
DRAWN	NSD/NAR
CHECKED	SG
APPROVED	

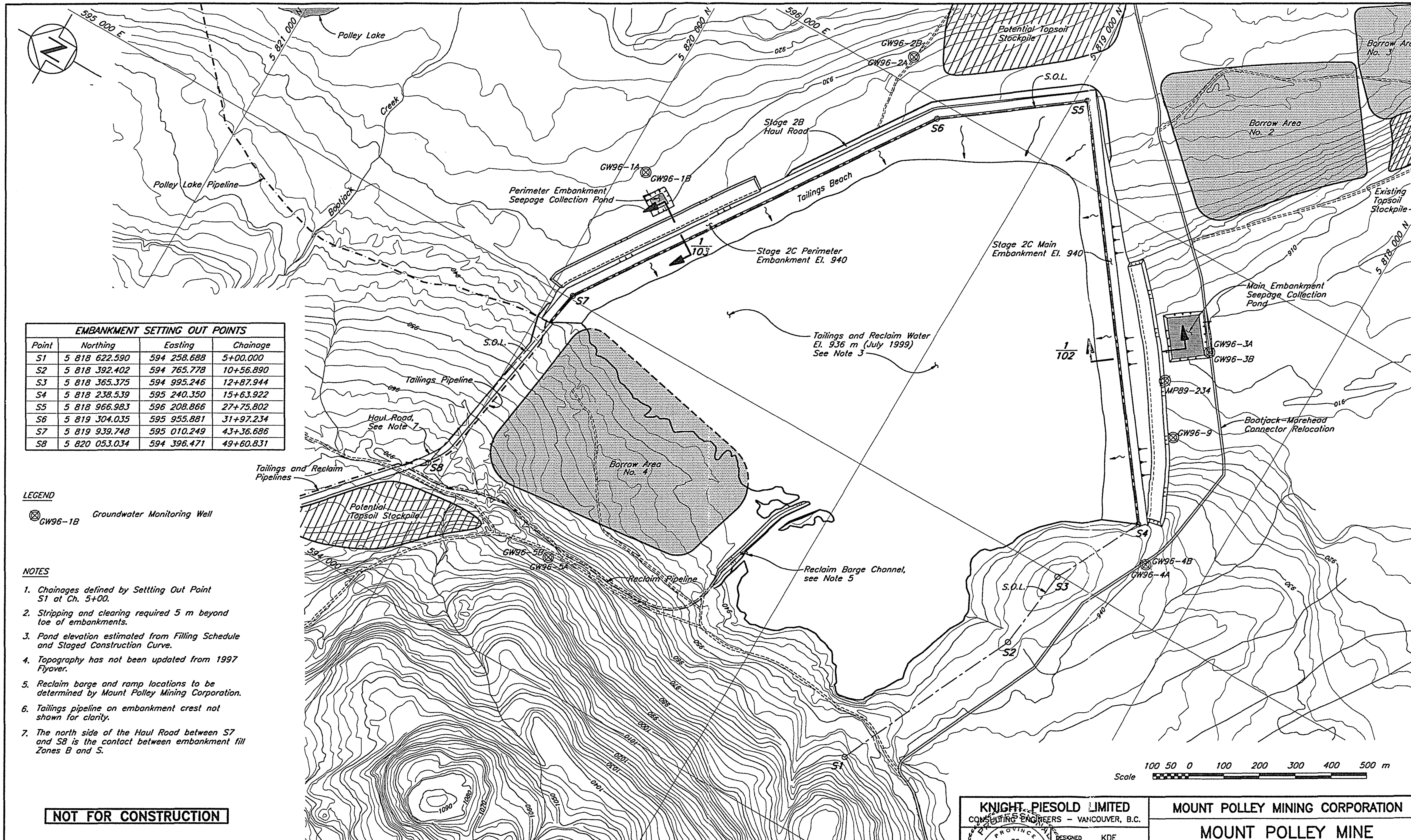
MOUNT POLLEY MINING CORPORATION

MOUNT POLLEY MINE

OVERALL SITE PLAN
SHOWING STAGE 2C TAILINGS EMBANKMENT

DATE	NOV. 10, 1997	SCALE AS SHOWN	DRG. NO.	10162-9-100	REV.	0

CAD FILE:\10162\9\04\04 1=20000 Plot 1=20 NOV 10, 1997



EMBANKMENT SETTING OUT POINTS			
Point	Northing	Easting	Chainage
S1	5 818 622.590	594 258.688	5+00.000
S2	5 818 392.402	594 765.778	10+56.890
S3	5 818 365.375	594 995.246	12+87.944
S4	5 818 238.539	595 240.350	15+63.922
S5	5 818 966.983	596 208.866	27+75.802
S6	5 819 304.035	595 955.881	31+97.234
S7	5 819 939.748	595 010.249	43+36.686
S8	5 820 053.034	594 396.471	49+60.831

LEGEND

GW96-1B Groundwater Monitoring Well

NOTES

1. Chainages defined by Settting Out Point S1 at Ch. 5+00.
2. Stripping and clearing required 5 m beyond toe of embankments.
3. Pond elevation estimated from Filling Schedule and Staged Construction Curve.
4. Topography has not been updated from 1997 Flyover.
5. Reclaim barge and ramp locations to be determined by Mount Polley Mining Corporation.
6. Tailings pipeline on embankment crest not shown for clarity.
7. The north side of the Haul Road between S7 and S8 is the contact between embankment fill Zones B and S.

NOT FOR CONSTRUCTION

CAD FILE: \10162\9\D1\01 1:5000 Plot 1=5 NOV 26, 1997

DRG. NO.	DESCRIPTION
103	TSF - STAGE 2 PERIMETER EMBANKMENT - SECTION
102	TSF - STAGE 2 MAIN EMBANKMENT - SECTIONS

REFERENCE DRAWINGS

REV.	DATE	DESCRIPTION	APPROVED
0	DEC 1/97	ISSUED FOR DESIGN REPORT	

REVISIONS

KNIGHT PIESOLD LIMITED
CONSULTING ENGINEERS - VANCOUVER, B.C.

PROFESSIONAL ENGINEER PROVINCE OF BRITISH COLUMBIA K. D. K. EMBREE

DESIGNED KDE
DRAWN RDT/DHS/NAR
CHECKED
APPROVED

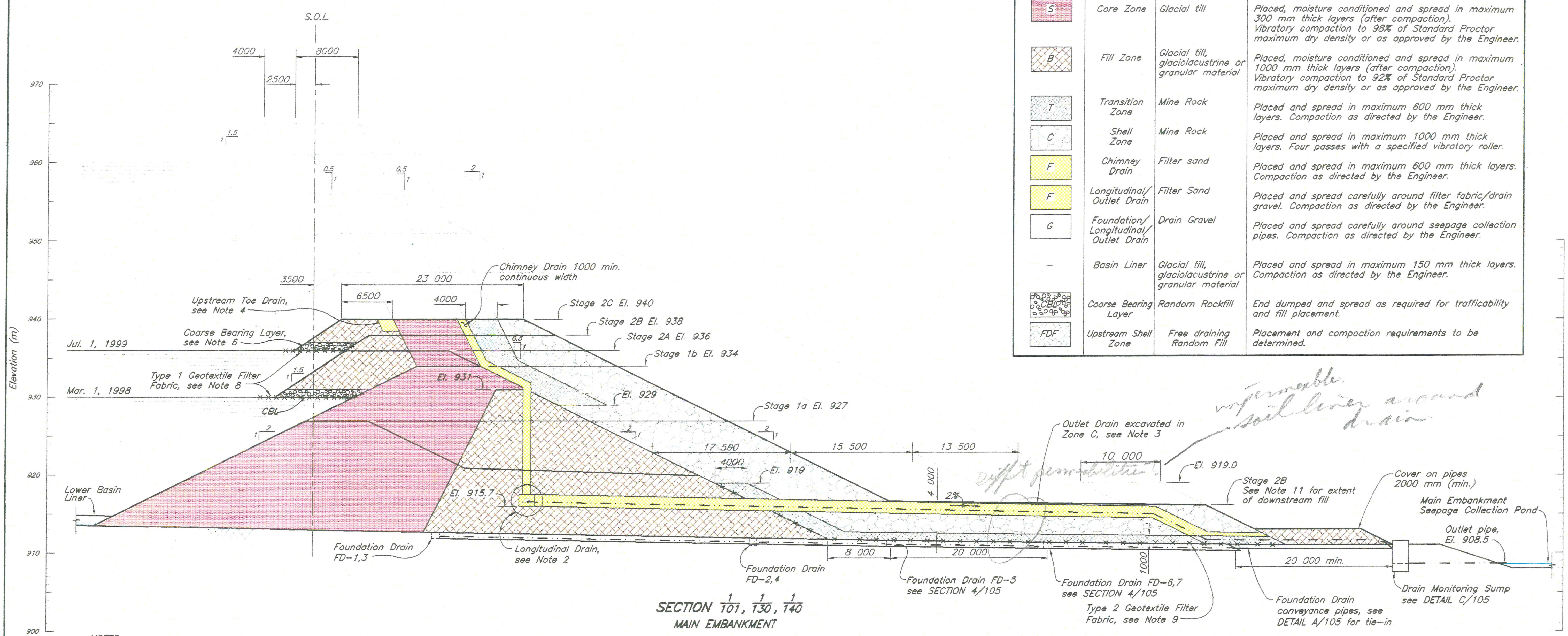
MOUNT POLLEY MINING CORPORATION

MOUNT POLLEY MINE

TAILINGS STORAGE FACILITY
STAGE 2 EXPANSION
GENERAL ARRANGEMENT

DATE DEC. 1, 1997 | SCALE AS SHOWN | DRG. NO. 10162-9-101 | REV. 0

ZONE	LOCATION	MATERIAL TYPE	PLACEMENT AND COMPACTION REQUIREMENTS
S	Core Zone	Glacial till	Placed, moisture conditioned and spread in maximum 300 mm thick layers (after compaction). Vibratory compaction to 98% of Standard Proctor maximum dry density or as approved by the Engineer.
B	Fill Zone	Glacial till, glaciolacustrine or granular material	Placed, moisture conditioned and spread in maximum 1000 mm thick layers (after compaction). Vibratory compaction to 92% of Standard Proctor maximum dry density or as approved by the Engineer.
T	Transition Zone	Mine Rock	Placed and spread in maximum 600 mm thick layers. Compaction as directed by the Engineer.
C	Shell Zone	Mine Rock	Placed and spread in maximum 1000 mm thick layers. Four passes with a specified vibratory roller.
F	Chimney Drain	Filter sand	Placed and spread in maximum 600 mm thick layers. Compaction as directed by the Engineer.
F	Longitudinal/ Outlet Drain	Filter Sand	Placed and spread carefully around filter fabric/drain gravel. Compaction as directed by the Engineer.
G	Foundation/ Longitudinal/ Outlet Drain	Drain Gravel	Placed and spread carefully around seepage collection pipes. Compaction as directed by the Engineer.
–	Basin Liner	Glacial till, glaciolacustrine or granular material	Placed and spread in maximum 150 mm thick layers. Compaction as directed by the Engineer.
CBL	Coarse Bearing Layer	Random Rockfill	End dumped and spread as required for trafficability and fill placement.
FDF	Upstream Shell Zone	Free draining Random Fill	Placement and compaction requirements to be determined.



SECTION 1/101, 1/130, 1/140
MAIN EMBANKMENT

NOTES

1. Pond elevations estimated from Filling Schedule and Staged Construction Curve.
2. Longitudinal Drain to be extended from invert El. 929 (Stage 1b) to El. 938.5 in original ground during Stage 2B.
3. Outlet Drains to be extended to Drain Monitoring Sump during Stage 2B. Drains require min. 2% slope.
4. Stage 2C Upstream Toe Drain to be designed and installed during Stage 3.
5. Fill placement rates to be monitored by the Engineer. Placement rates to be modified if excess pore pressures observed in fill or foundation piezometers.
6. Coarse Bearing Layer required on tailings. To be added on ground as required to provide a firm bearing layer for fill placement.
7. All dimensions in millimetres with elevations in metres, unless noted otherwise.
8. Type 1 Geotextile Filter Fabric required on tailings. Specification provided in Tender Documents (12 oz/sq. yd).
9. Type 2 Geotextile Filter Fabric required on the right abutment (approx. Ch. 15+75) to El. 920 on the left abutment (approx. Ch. 23+00). Specification provided in Tender Documents (8 oz/sq. yd).
10. Foundation drains are not intersected on this cross-section.
11. Downstream fill (1000 thick for Zone T and 4000 thick for Zone C) to be placed up to final Stage 2B Crest El. 938 at right abutment and to ground El. 920 (approx. Ch. 23+00) at left abutment.

NOT FOR CONSTRUCTION

Scale 10 5 0 10 20 Metres

DRG. NO.	DESCRIPTION
140	TSF – STAGE 2C MAIN EMBANKMENT – PLAN
130	STAGE 2 EXPANSION – STAGE 2B MAIN EMBANKMENT
105	TSF – STAGE 2 EXPANSION –FOUNDATION PREPARATION–SECTIONS & DETAILS
101	TSF – STAGE 2C EXPANSION – GENERAL ARRANGEMENT

REFERENCE DRAWINGS

REV.	DATE	DESCRIPTION	APPROVED

REVISIONS

REV.	DATE	DESCRIPTION	APPROVED
0	DEC 1/97	ISSUED FOR DESIGN REPORT	

REVISIONS

KNIGHT PIESOLD LIMITED
CONSULTING ENGINEERS – VANCOUVER, B.C.

PROVINCE OF BRITISH COLUMBIA
K. D. K. EMBREE
ENGINEER

DESIGNED KDE
DRAWN WAL/DHS
CHECKED GBW
APPROVED

MOUNT POLLEY MINING CORPORATION
MOUNT POLLEY MINE
TAILINGS STORAGE FACILITY
STAGE 2 MAIN EMBANKMENT
SECTIONS

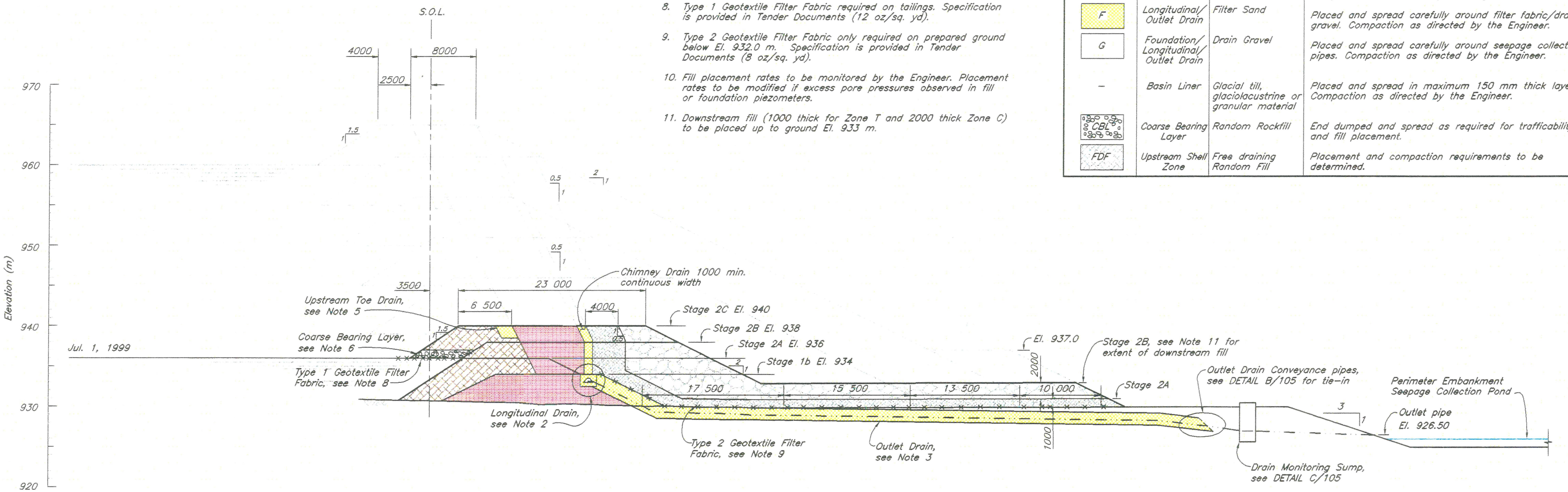
DATE DEC. 1, 1997
SCALE AS SHOWN
DRG. NO. 10162-9-102
REV. 0

CAD FILE: \10162\9\102.DS2 1:250 Plot 1=0.25 NOV 28, 1997

NOTES

1. Pond elevations estimated from Filling Schedule and Staged Construction Curve.
2. Longitudinal Drain to be installed during Stage 2B.
3. Outlet Drains to be installed to Stage 1b crest during Stage 2A.
4. Chimney Drain to be installed during Stage 2B and 2C.
5. Stage 2C Upstream Toe Drain to be designed and installed during Stage 3.
6. Coarse Bearing Layer required on tailings. To be added on ground as required to provide a firm bearing layer for fill placement.
7. All dimensions in millimetres with elevations in metres, unless noted otherwise.
8. Type 1 Geotextile Filter Fabric required on tailings. Specification is provided in Tender Documents (12 oz/sq. yd).
9. Type 2 Geotextile Filter Fabric only required on prepared ground below El. 932.0 m. Specification is provided in Tender Documents (8 oz/sq. yd).
10. Fill placement rates to be monitored by the Engineer. Placement rates to be modified if excess pore pressures observed in fill or foundation piezometers.
11. Downstream fill (1000 thick for Zone T and 2000 thick Zone C) to be placed up to ground El. 933 m.

ZONE	LOCATION	MATERIAL TYPE	PLACEMENT AND COMPACTION REQUIREMENTS
S	Core Zone	Glacial till	Placed, moisture conditioned and spread in maximum 300 mm thick layers (after compaction). Vibratory compaction to 98% of Standard Proctor maximum dry density or as approved by the Engineer.
B	Fill Zone	Glacial till, glaciolacustrine or granular material	Placed, moisture conditioned and spread in maximum 1000 mm thick layers (after compaction). Vibratory compaction to 92% of Standard Proctor maximum dry density or as approved by the Engineer.
T	Transition Zone	Mine Rock	Placed and spread in maximum 600 mm thick layers. Compaction as directed by the Engineer.
C	Shell Zone	Mine Rock	Placed and spread in maximum 1000 mm thick layers. Four passes with a specified vibratory roller.
F	Chimney Drain	Filter sand	Placed and spread in maximum 600 mm thick layers. Compaction as directed by the Engineer.
F	Longitudinal/ Outlet Drain	Filter Sand	Placed and spread carefully around filter fabric/drain gravel. Compaction as directed by the Engineer.
G	Foundation/ Longitudinal/ Outlet Drain	Drain Gravel	Placed and spread carefully around seepage collection pipes. Compaction as directed by the Engineer.
–	Basin Liner	Glacial till, glaciolacustrine or granular material	Placed and spread in maximum 150 mm thick layers. Compaction as directed by the Engineer.
CBL	Coarse Bearing Layer	Random Rockfill	End dumped and spread as required for trafficability and fill placement.
FDF	Upstream Shell Zone	Free draining Random Fill	Placement and compaction requirements to be determined.



SECTION 1/101, 1/131, 1/141
PERIMETER EMBANKMENT

NOT FOR CONSTRUCTION

Scale 10 5 0 10 20 Metres

DRG. NO.	DESCRIPTION
141	TSF – STAGE 2C PERIMETER EMBANKMENT – PLAN
131	TSF – STAGE 2B PERIMETER EMBANKMENT – PLAN
105	TSF – STAGE 2 EXPANSION –FOUNDATION PREPARATION–SECTIONS & DETAILS
101	TSF – STAGE 2C EXPANSION – GENARAL ARRANGEMENT

REFERENCE DRAWINGS

REV.	DATE	DESCRIPTION	APPROVED
0	DEC 1/97	ISSUED FOR DESIGN REPORT	

REVISIONS

KNIGHT PIESOLD LIMITED
CONSULTING ENGINEERS – VANCOUVER, B.C.

PROFESSIONAL ENGINEER, PROVINCE OF BRITISH COLUMBIA, K. D. K. EMBREE

DESIGNED KDE
DRAWN DSR/DHS
CHECKED
APPROVED

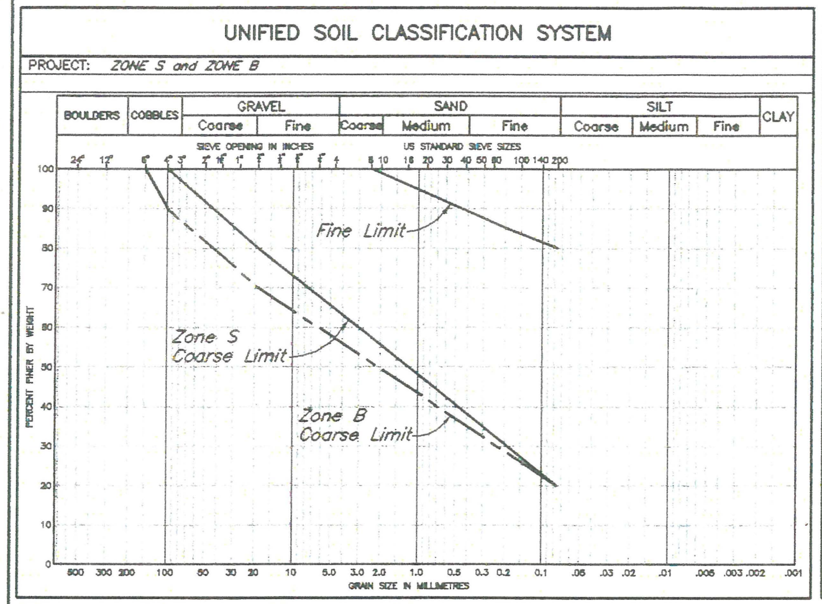
MOUNT POLLEY MINING CORPORATION

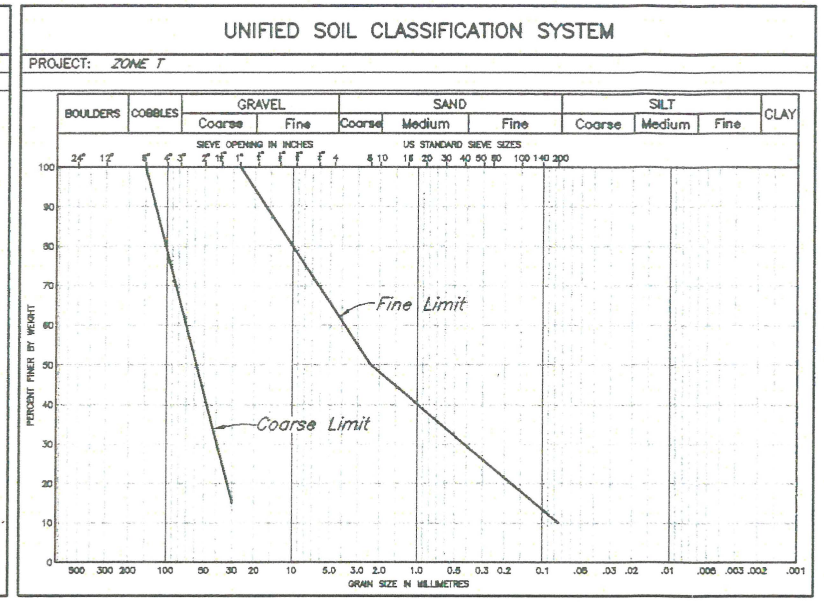
MOUNT POLLEY MINE

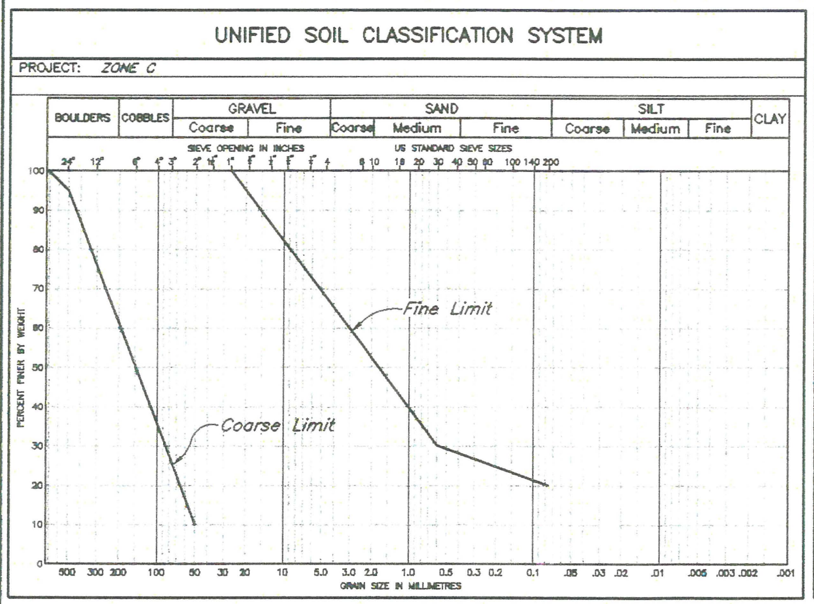
TAILINGS STORAGE FACILITY
STAGE 2 PERIMETER EMBANKMENT
SECTION

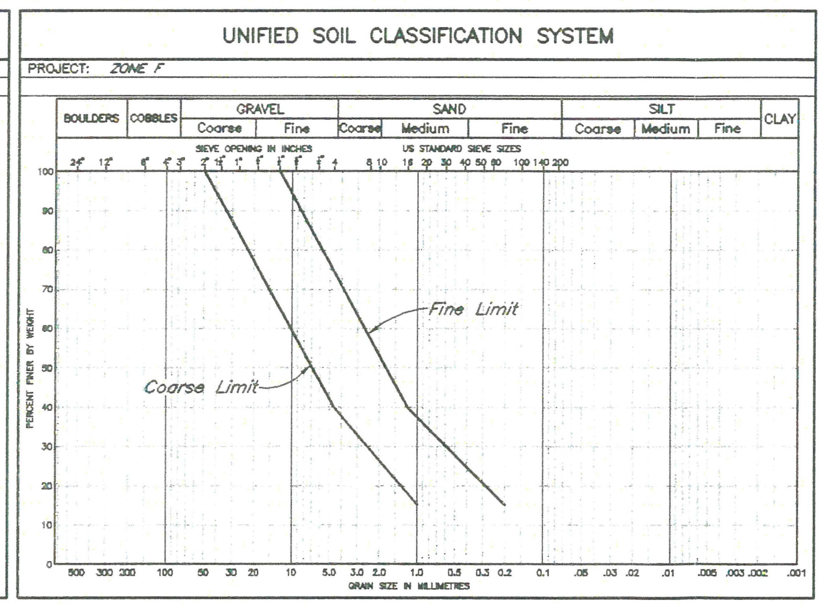
DATE DEC. 1, 1997 | SCALE AS SHOWN | DRG. NO. 10162-9-103 | REV. 0

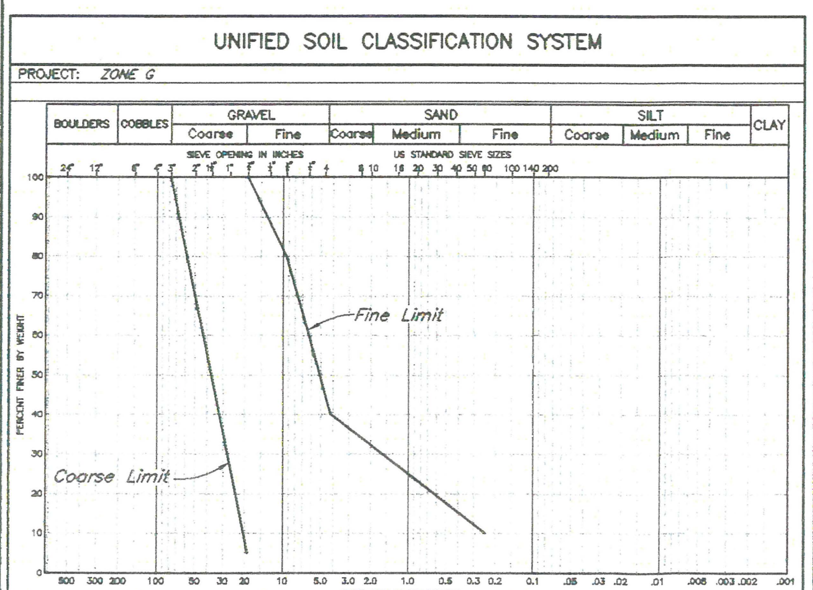
CAD FILE:\10162\9\010\010 1:250 Plot 1=0.25 NOV 28, 1997

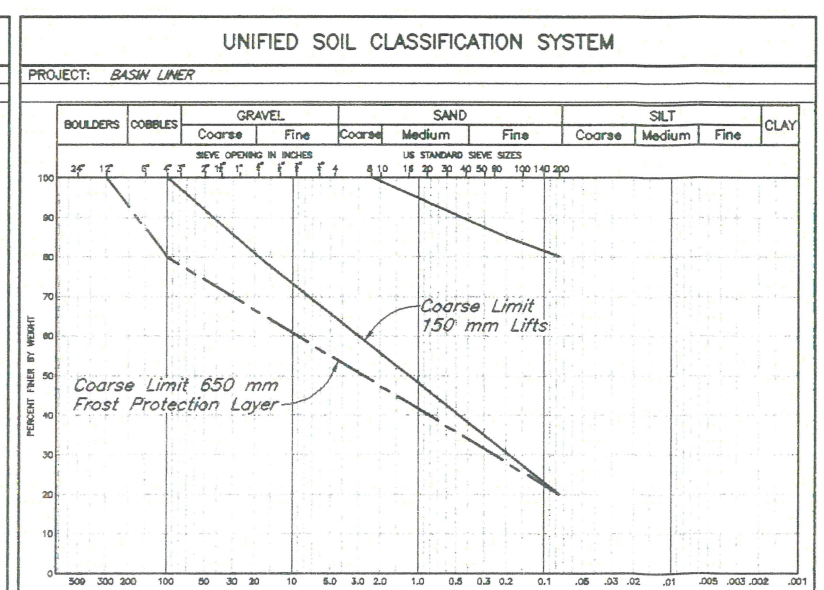












ZONE	LOCATION	MATERIAL TYPE	PLACEMENT AND COMPACTION REQUIREMENTS
S	Core Zone	Glacial till	Placed, moisture conditioned and spread in maximum 300 mm thick layers (after compaction). Vibratory compaction to 98% of Standard Proctor maximum dry density or as approved by the Engineer.
B	Fill Zone	Glacial till, glaciolacustrine or granular material	Placed, moisture conditioned and spread in maximum 1000 mm thick layers (after compaction). Vibratory compaction to 92% of Standard Proctor maximum dry density or as approved by the Engineer.
T	Transition Zone	Mine Rock	Placed and spread in maximum 600 mm thick layers. Compaction as directed by the Engineer.
C	Shell Zone	Mine Rock	Placed and spread in maximum 1000 mm thick lifts. Four passes with a specified vibratory roller.
F	Chimney Drain	Filter sand	Placed and spread in maximum 600 mm thick lifts. Compaction as directed by the Engineer.
F	Longitudinal/ Outlet Drain	Filter Sand	Placed and spread carefully around filter fabric/drain gravel. Compaction as directed by the Engineer.
G	Foundation/ Longitudinal/ Outlet Drain	Drain Gravel	Placed and spread carefully around seepage collection pipes. Compaction as directed by the Engineer.
–	Basin Liner	Glacial till, glaciolacustrine or granular material	Placed and spread in maximum 150 mm thick lifts. Compaction as directed by the Engineer.
CBL	Coarse Bearing Layer	Random Rockfill	End dumped and spread as required for trafficability and fill placement.

CAD FILE:\10162\9\103\03-COL 1:1250 Plot 1=1.25 OCT. 24, 1997

NOT FOR CONSTRUCTION

DRG. NO.	DESCRIPTION

REFERENCE DRAWINGS

REV.	DATE	DESCRIPTION	APPROVED

REVISIONS

REV.	DATE	DESCRIPTION	APPROVED
0	NOV 10/97	ISSUED FOR TENDER	

REVISIONS

KNIGHT PIESOLD LIMITED
CONSULTING ENGINEERS – VANCOUVER, B.C.

PROFESSIONAL ENGINEER
PROVINCE OF BRITISH COLUMBIA
K. D. K. EMBREE

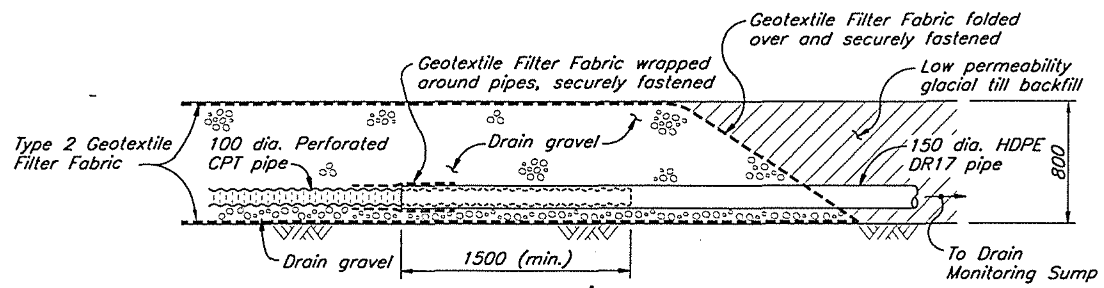
DESIGNED KDE/PJP
DRAWN NSD/DHS
CHECKED
APPROVED

MOUNT POLLEY MINING CORPORATION

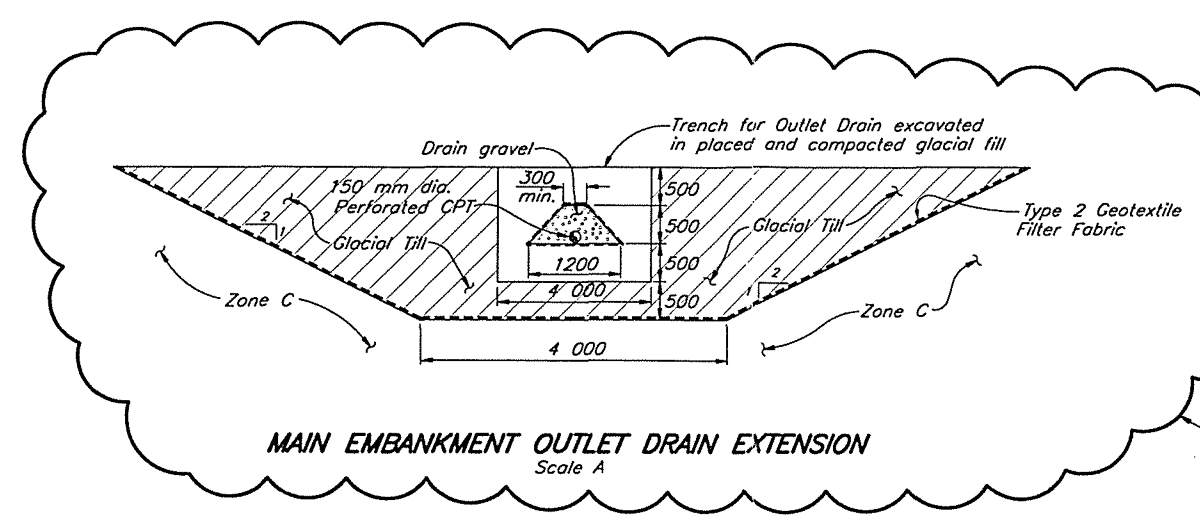
MOUNT POLLEY MINE

TAILINGS STORAGE FACILITY
STAGE 2 EXPANSION
MATERIAL SPECIFICATIONS

DATE NOV. 10, 1997 | SCALE AS SHOWN | DRG. NO. 10162-9-104 | REV. 0

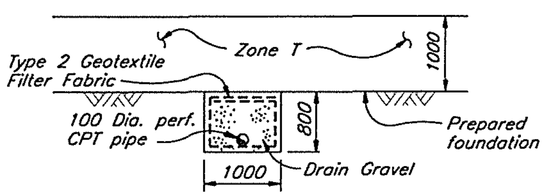


DETAIL A/111
EMBANKMENT FOUNDATION DRAIN TIE-IN (IN SECTION)
Scale B

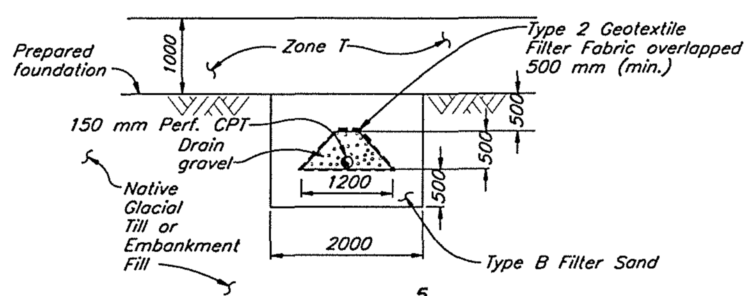


MAIN EMBANKMENT OUTLET DRAIN EXTENSION
Scale A

HOLD
For Stage 2B Construction

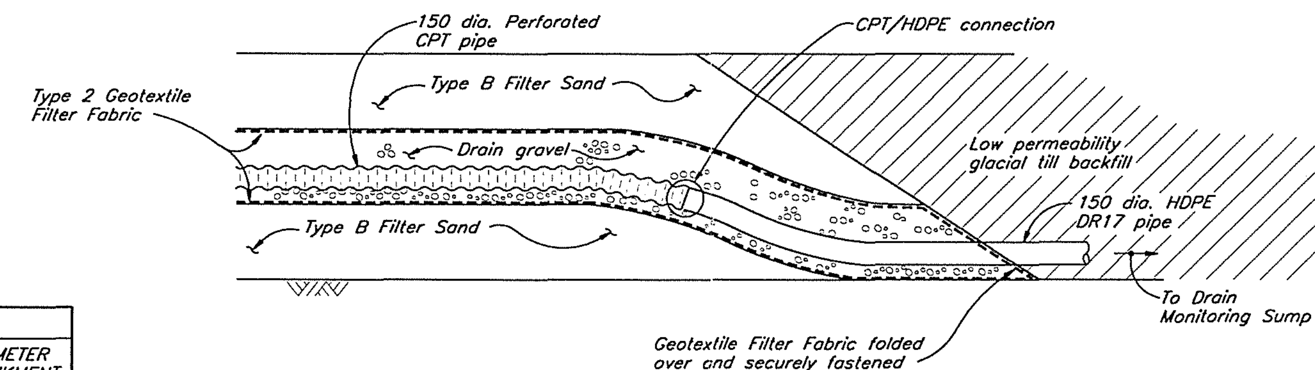


SECTION 4/110
FOUNDATION DRAIN DETAIL
Scale A

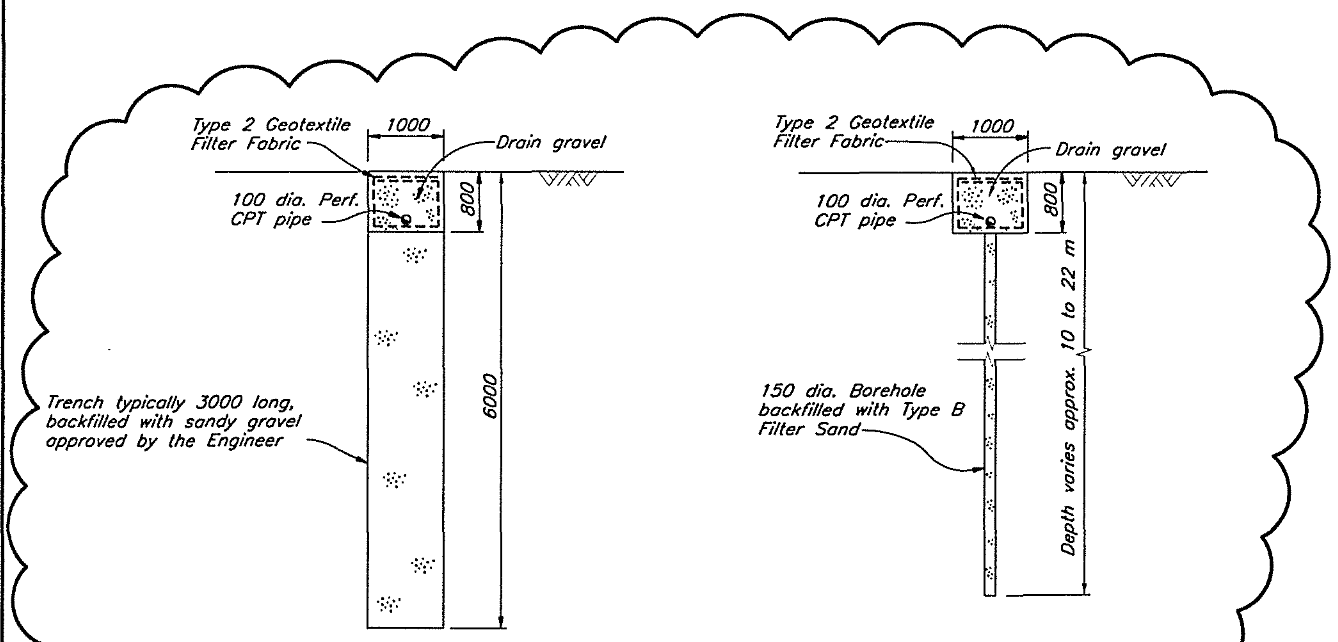


SECTION 5/120
OUTLET DRAIN DETAIL
Scale A

Table 1		
ELEV.	MAIN EMBANKMENT	PERIMETER EMBANKMENT
A	907.5	925.5
B	908.5	926.5
C	909.3	927.0
D	911.5	930.5



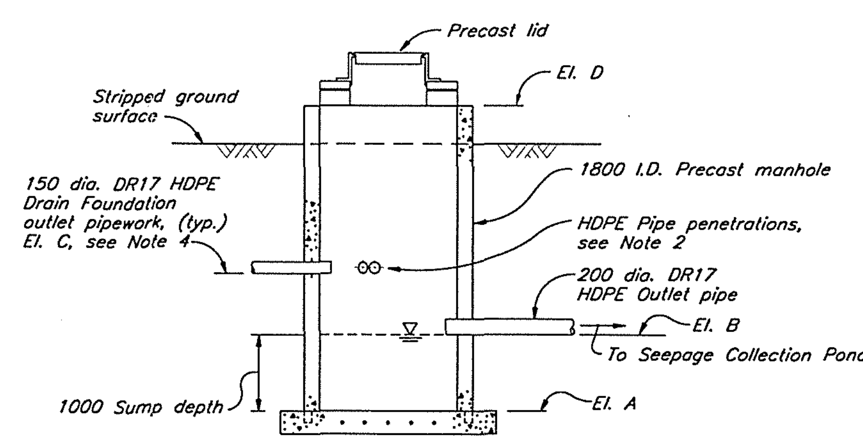
DETAIL B/121
OUTLET DRAIN TIE-IN (IN SECTION)
Scale B



EMBANKMENT FOUNDATION DRAIN PRESSURE RELIEF TRENCH TYPICAL DETAIL
Scale A

EMBANKMENT FOUNDATION DRAIN PRESSURE RELIEF WELL TYPICAL DETAIL
Scale A

HOLD
For Stage 2B Construction or as required by the Engineer

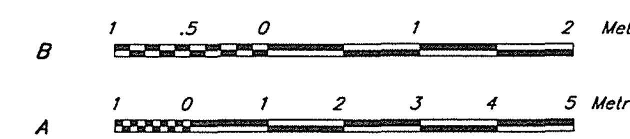


DETAIL C/111, C/121
EXISTING DRAIN MONITORING SUMP (Typ.)
Scale A

NOTE: See Table 1 for key elevations

NOTES

1. All pipework to have a minimum of 1 m of cover for frost protection.
2. All HDPE pipe penetrations to be water tight.
3. Buried pipework to be covered by a berm and clearly marked by a line of stakes.
4. Elevation of new HDPE pipe penetrations at Drain Monitoring Sump to be determined in the field, by the Engineer.



NOT FOR CONSTRUCTION

DRG. NO.	DESCRIPTION
121	TSF - STAGE 2A PERIMETER EMBANKMENT - SECTIONS
120	TSF - STAGE 2A PERIMETER EMBANKMENT -PLAN
111	TSF - STAGE 2A MAIN EMBANKMENT -SECTIONS
110	TSF - STAGE 2A MAIN EMBANKMENT -PLAN

REFERENCE DRAWINGS

REV.	DATE	DESCRIPTION	APPROVED

REVISIONS

REV.	DATE	DESCRIPTION	APPROVED
0	NOV 10/97	ISSUED FOR TENDER	

REVISIONS

KNIGHT PIESOLD LIMITED
CONSULTING ENGINEERS - VANCOUVER, B.C.

PROFESSIONAL ENGINEER PROVINCE OF BRITISH COLUMBIA K. D. K. EMBREE

DESIGNED KDE
DRAWN DSR/NSD/DHS
CHECKED LSG
APPROVED

MOUNT POLLEY MINING CORPORATION

MOUNT POLLEY MINE

TAILINGS STORAGE FACILITY
STAGE 2 EXPANSION
FOUNDATION PREPARATION
SECTIONS AND DETAILS

DATE NOV. 10, 1997 | SCALE AS SHOWN | DRG. NO. 10162-9-105 | REV. 0

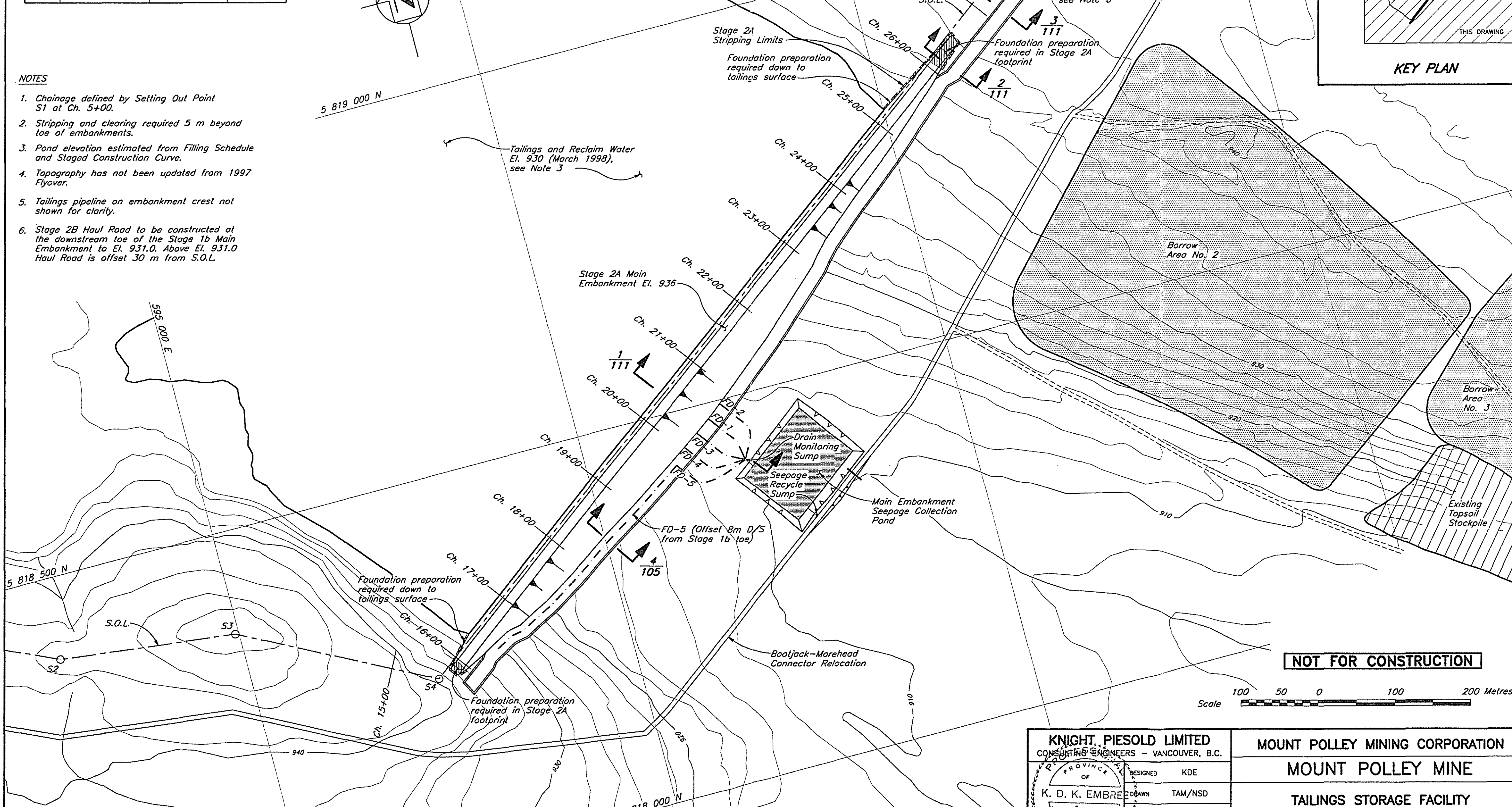
CAD FILE: \10162\9\D15\015 1:50 Plot 1=0.05 NOV. 10, 1997

EMBANKMENT SETTING OUT POINTS

Point	Northing	Easting	Chainage
S1	5 818 622.590	594 258.688	5+00.00
S2	5 818 392.402	594 765.778	10+56.89
S3	5 818 365.375	594 995.246	12+87.94
S4	5 818 238.539	595 240.350	15+63.92
S5	5 818 966.983	596 208.866	27+75.80
S6	5 819 304.035	595 955.881	31+97.23
S7	5 819 939.748	595 010.249	43+36.69
S8	5 820 053.034	594 396.471	49+60.83

NOTES

1. Chainage defined by Setting Out Point S1 at Ch. 5+00.
2. Stripping and clearing required 5 m beyond toe of embankments.
3. Pond elevation estimated from Filling Schedule and Staged Construction Curve.
4. Topography has not been updated from 1997 Flyover.
5. Tailings pipeline on embankment crest not shown for clarity.
6. Stage 2B Haul Road to be constructed at the downstream toe of the Stage 1b Main Embankment to El. 931.0. Above El. 931.0 Haul Road is offset 30 m from S.O.L.



NOT FOR CONSTRUCTION

Scale 100 50 0 100 200 Metres

DRG. NO.	DESCRIPTION
105	TSF - STAGE 2 EXPANSION - FOUNDATION PREPARATION - SECTIONS & DETAILS
111	TSF - STAGE 2A MAIN EMBANKMENT - SECTIONS

REFERENCE DRAWINGS

REV.	DATE	DESCRIPTION	APPROVED
0	NOV 10/97	ISSUED FOR TENDER	

REVISIONS

KNIGHT PIESOLD LIMITED
CONSULTING ENGINEERS - VANCOUVER, B.C.

PROVINCE OF BRITISH COLUMBIA — K. D. K. EMBREE — ENGINEER

DESIGNED	KDE
DRAWN	TAM/NSD
CHECKED	LSG
APPROVED	

MOUNT POLLEY MINING CORPORATION

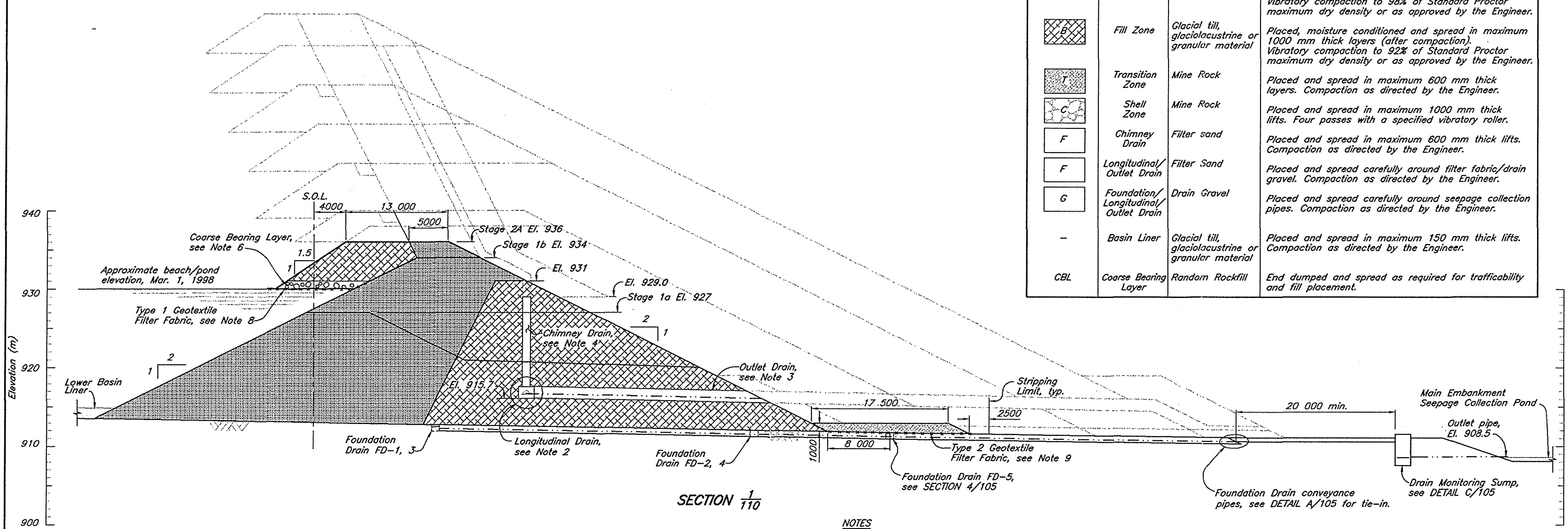
MOUNT POLLEY MINE

TAILINGS STORAGE FACILITY
STAGE 2A MAIN EMBANKMENT
PLAN

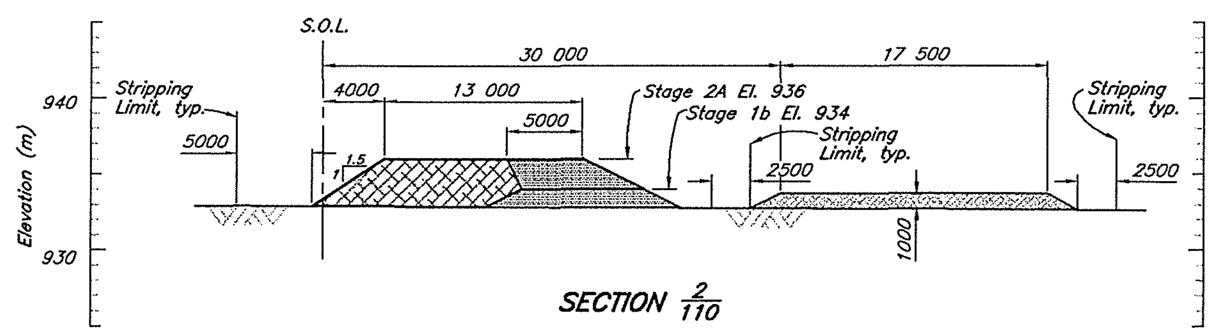
DATE NOV. 10, 1997 | SCALE AS SHOWN | DRG. NO. 10162-9-110 | REV. 0

CAD FILE: \10162\9\09\09 1:2500 Plot 1=2.5 Nov. 10, 1997

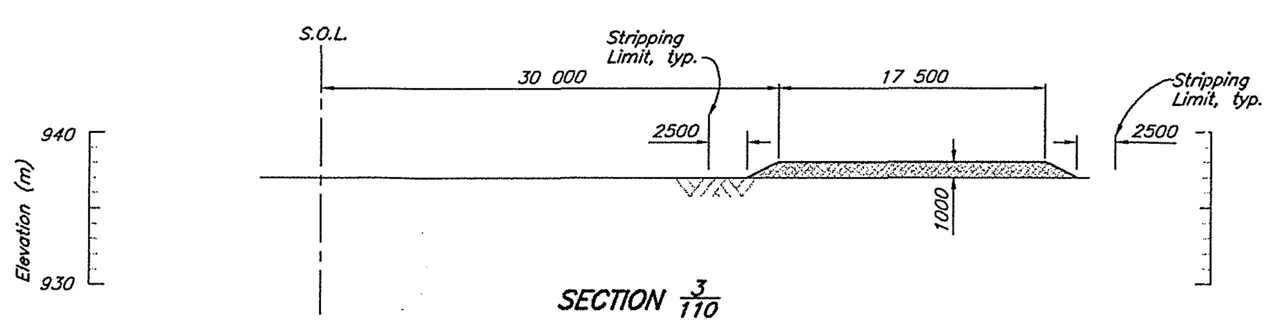
ZONE	LOCATION	MATERIAL TYPE	PLACEMENT AND COMPACTION REQUIREMENTS
S	Core Zone	Glacial till	Placed, moisture conditioned and spread in maximum 300 mm thick layers (after compaction). Vibratory compaction to 98% of Standard Proctor maximum dry density or as approved by the Engineer.
B	Fill Zone	Glacial till, glaciolacustrine or granular material	Placed, moisture conditioned and spread in maximum 1000 mm thick layers (after compaction). Vibratory compaction to 92% of Standard Proctor maximum dry density or as approved by the Engineer.
T	Transition Zone	Mine Rock	Placed and spread in maximum 600 mm thick layers. Compaction as directed by the Engineer.
C	Shell Zone	Mine Rock	Placed and spread in maximum 1000 mm thick lifts. Four passes with a specified vibratory roller.
F	Chimney Drain	Filter sand	Placed and spread in maximum 600 mm thick lifts. Compaction as directed by the Engineer.
F	Longitudinal/ Outlet Drain	Filter Sand	Placed and spread carefully around filter fabric/drain gravel. Compaction as directed by the Engineer.
G	Foundation/ Longitudinal/ Outlet Drain	Drain Gravel	Placed and spread carefully around seepage collection pipes. Compaction as directed by the Engineer.
–	Basin Liner	Glacial till, glaciolacustrine or granular material	Placed and spread in maximum 150 mm thick lifts. Compaction as directed by the Engineer.
CBL	Coarse Bearing Layer	Random Rockfill	End dumped and spread as required for trafficability and fill placement.



SECTION $\frac{1}{110}$



SECTION $\frac{2}{110}$



SECTION $\frac{3}{110}$

NOTES

1. Pond elevations estimated from Filling Schedule and Staged Construction Curve.
2. Longitudinal Drain to be extended from invert El. 929 (Stage 1b) to El. 938.5 in original ground during Stage 2B .
3. Outlet Drains to be extended to Drain Monitoring Sump during Stage 2B.
4. Chimney Drain to be extended during Stages 2B and 2C.
5. Fill placement rates to be monitored by the Engineer. Placement rates to be modified if excess pore pressures observed in fill or foundation piezometers.
6. Coarse Bearing Layer required on tailings. To be added on ground as required to provide a firm bearing layer for fill placement.
7. All dimensions in millimetres and elevations in metres, unless noted otherwise.
8. Type 1 Geotextile Filter Fabric specification provided in Tender Documents (12 oz/sq. yd).
9. Type 2 Geotextile Filter Fabric is required from the right abutment (approx. Ch. 15+25) to El. 920.0m on the left abutment (approx. Ch. 23+00). Specification is provided in Tender Documents (8 oz/sq. yd).
10. All Foundation Drains are not intersected on this cross-section.

NOT FOR CONSTRUCTION

Scale 10 5 0 10 20 Metres

CAD FILE: 10162\9\011\011-sol 1:250 Plot 1=0.25 NOV. 10, 1997

DRG. NO.	DESCRIPTION
110	TSF – STAGE 2A MAIN EMBANKMENT – PLAN
105	TSF – STAGE 2 EXPANSION – FOUNDATION PREPARATION – SECTIONS & DETAILS

REFERENCE DRAWINGS

REV.	DATE	DESCRIPTION	APPROVED
0	NOV 10/97	ISSUED FOR TENDER	

REVISIONS

KNIGHT PIESOLD LIMITED
CONSULTING ENGINEERS – VANCOUVER, B.C.

PROVINCE OF BRITISH COLUMBIA — K. D. K. EMBREE — ENGINEER

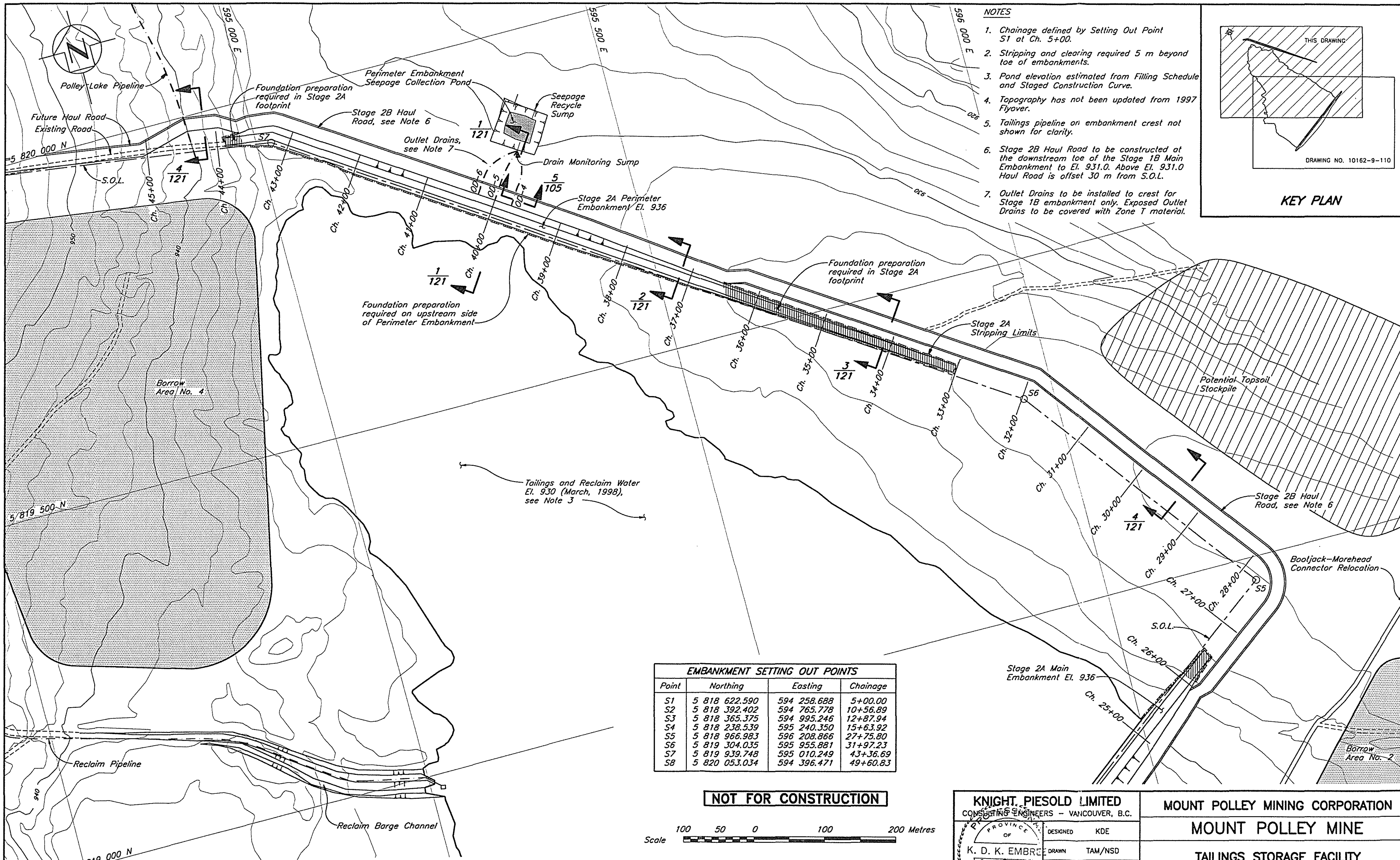
DESIGNED KDE
DRAWN WAL/DHS
CHECKED LSG
APPROVED

MOUNT POLLEY MINING CORPORATION

MOUNT POLLEY MINE

TAILINGS STORAGE FACILITY
STAGE 2A MAIN EMBANKMENT
SECTIONS

DATE NOV. 10, 1997 | SCALE AS SHOWN | DRG. NO. 10162-9-111 | REV. 0



NOTES

1. Chainage defined by Setting Out Point S1 at Ch. 5+00.
2. Stripping and clearing required 5 m beyond toe of embankments.
3. Pond elevation estimated from Filling Schedule and Staged Construction Curve.
4. Topography has not been updated from 1997 Flyover.
5. Tailings pipeline on embankment crest not shown for clarity.
6. Stage 2B Haul Road to be constructed at the downstream toe of the Stage 1B Main Embankment to El. 931.0. Above El. 931.0 Haul Road is offset 30 m from S.O.L.
7. Outlet Drains to be installed to crest for Stage 1B embankment only. Exposed Outlet Drains to be covered with Zone T material.

EMBANKMENT SETTING OUT POINTS

Point	Northing	Easting	Chainage
S1	5 818 622.590	594 258.688	5+00.00
S2	5 818 392.402	594 765.778	10+56.89
S3	5 818 365.375	594 995.246	12+87.94
S4	5 818 238.539	595 240.350	15+63.92
S5	5 818 966.983	596 208.866	27+75.80
S6	5 819 304.035	595 955.881	31+97.23
S7	5 819 939.748	595 010.249	43+36.69
S8	5 820 053.034	594 396.471	49+60.83

NOT FOR CONSTRUCTION

Scale 100 50 0 100 200 Metres

DRG. NO.	DESCRIPTION
105	TSF – STAGE 2 EXPANSION – FOUNDATION PREPARATION – SECTIONS & DETAILS
121	TSF – STAGE 2A PERIMETER EMBANKMENT – SECTIONS

REFERENCE DRAWINGS

REV.	DATE	DESCRIPTION	APPROVED

REVISIONS

REV.	DATE	DESCRIPTION	APPROVED
0	NOV 10/97	ISSUED FOR TENDER	

REVISIONS

KNIGHT PIESOLD LIMITED
CONSULTING ENGINEERS – VANCOUVER, B.C.

PROFESSIONAL ENGINEER – PROVINCE OF B.C. – K. D. K. EMBREE

DESIGNED	KDE
DRAWN	TAM/NSD
CHECKED	LJG
APPROVED	

MOUNT POLLEY MINING CORPORATION

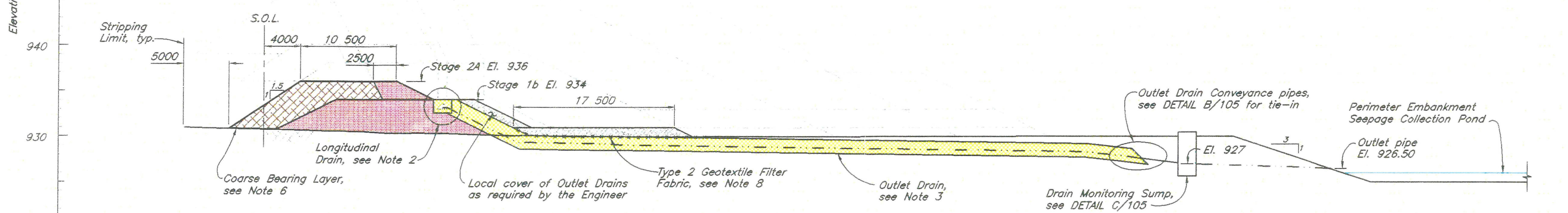
MOUNT POLLEY MINE

TAILINGS STORAGE FACILITY
STAGE 2A PERIMETER EMBANKMENT
PLAN

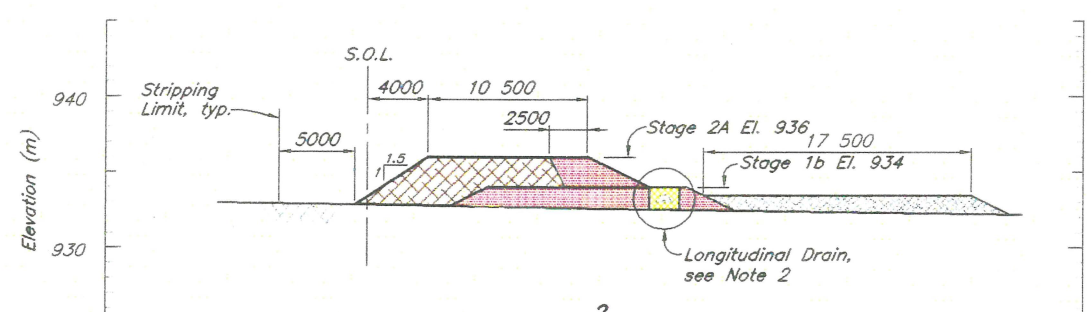
DATE NOV. 10, 1997 | SCALE AS SHOWN | DRG. NO. 10162-9-120 | REV. 0

NOTES

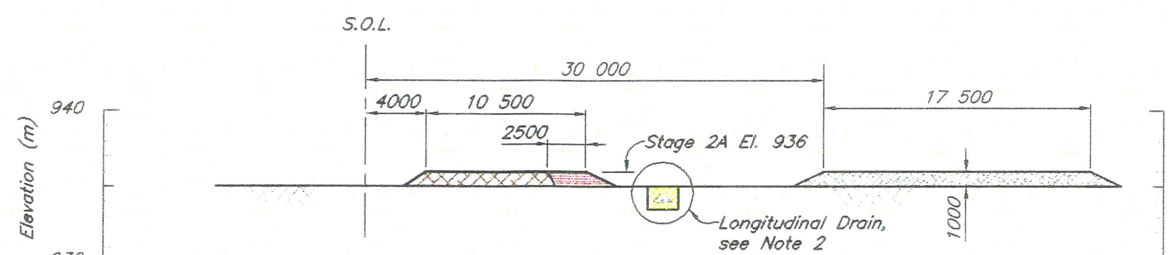
1. Pond elevations estimated from Filling Schedule and Staged Construction Curve.
2. Longitudinal Drain to be installed during Stage 2B.
3. Outlet Drains to be installed to Stage 1b crest during Stage 2A. Penetration at Drain Monitoring Sump already made. Pipe stubs are capped and backfilled.
4. Chimney Drain to be installed during Stage 2B and 2C.
5. Fill placement rates to be monitored by the Engineer. Placement rates to be modified if excess pore pressures observed in fill or foundation piezometers.
6. Coarse Bearing Layer to be added on ground as required to provide a firm bearing layer for fill placement.
7. All dimensions in millimetres with elevations in metres, unless noted otherwise.
8. Type 2 Geotextile Filter Fabric only required on prepared ground below El. 932.0m. The specification is provided in Tender Documents (8 oz/sq. yd).



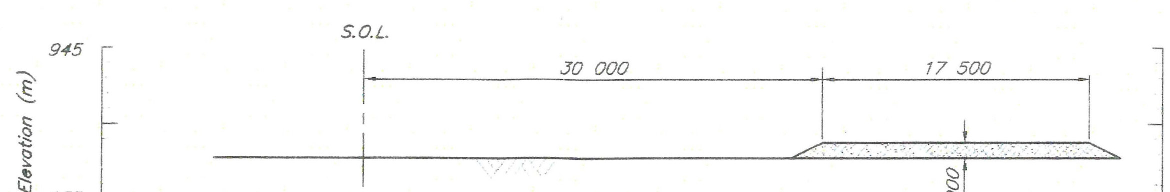
SECTION 1/120



SECTION 2/120



SECTION 3/120



SECTION 4/120

ZONE	LOCATION	MATERIAL TYPE	PLACEMENT AND COMPACTION REQUIREMENTS
S	Core Zone	Glacial till	Placed, moisture conditioned and spread in maximum 300 mm thick layers (after compaction). Vibratory compaction to 98% of Standard Proctor maximum dry density or as approved by the Engineer.
B	Fill Zone	Glacial till, glaciolacustrine or granular material	Placed, moisture conditioned and spread in maximum 1000 mm thick layers (after compaction). Vibratory compaction to 92% of Standard Proctor maximum dry density or as approved by the Engineer.
T	Transition Zone	Mine Rock	Placed and spread in maximum 600 mm thick layers. Compaction as directed by the Engineer.
C	Shell Zone	Mine Rock	Placed and spread in maximum 1000 mm thick lifts. Four passes with a specified vibratory roller.
F	Chimney Drain	Filter sand	Placed and spread in maximum 600 mm thick lifts. Compaction as directed by the Engineer.
F	Longitudinal/ Outlet Drain	Filter Sand	Placed and spread carefully around filter fabric/drain gravel. Compaction as directed by the Engineer.
G	Foundation/ Longitudinal/ Outlet Drain	Drain Gravel	Placed and spread carefully around seepage collection pipes. Compaction as directed by the Engineer.
–	Basin Liner	Glacial till, glaciolacustrine or granular material	Placed and spread in maximum 150 mm thick lifts. Compaction as directed by the Engineer.
CBL	Coarse Bearing Layer	Random Rockfill	End dumped and spread as required for trafficability and fill placement.

Scale 10 5 0 10 20 Metres

NOT FOR CONSTRUCTION

DRG. NO.	DESCRIPTION
121	TSF – STAGE 2 EXPANSION–FOUNDATION PREPARATION–SECTIONS & DETAILS
120	TSF – STAGE 2A PERIMETER EMBANKMENT – PLAN

REFERENCE DRAWINGS

REV.	DATE	DESCRIPTION	APPROVED
0	NOV 10/97	ISSUED FOR TENDER	JMA

REVISIONS

KNIGHT PIESOLD LIMITED
CONSULTING ENGINEERS – VANCOUVER, B.C.

PROFESSIONAL ENGINEER, PROVINCE OF BRITISH COLUMBIA, K. D. K. EMBREE

DESIGNED KDE
DRAWN DHS
CHECKED LSG
APPROVED JMA

MOUNT POLLEY MINING CORPORATION

MOUNT POLLEY MINE

TAILINGS STORAGE FACILITY
STAGE 2A PERIMETER EMBANKMENT
SECTIONS

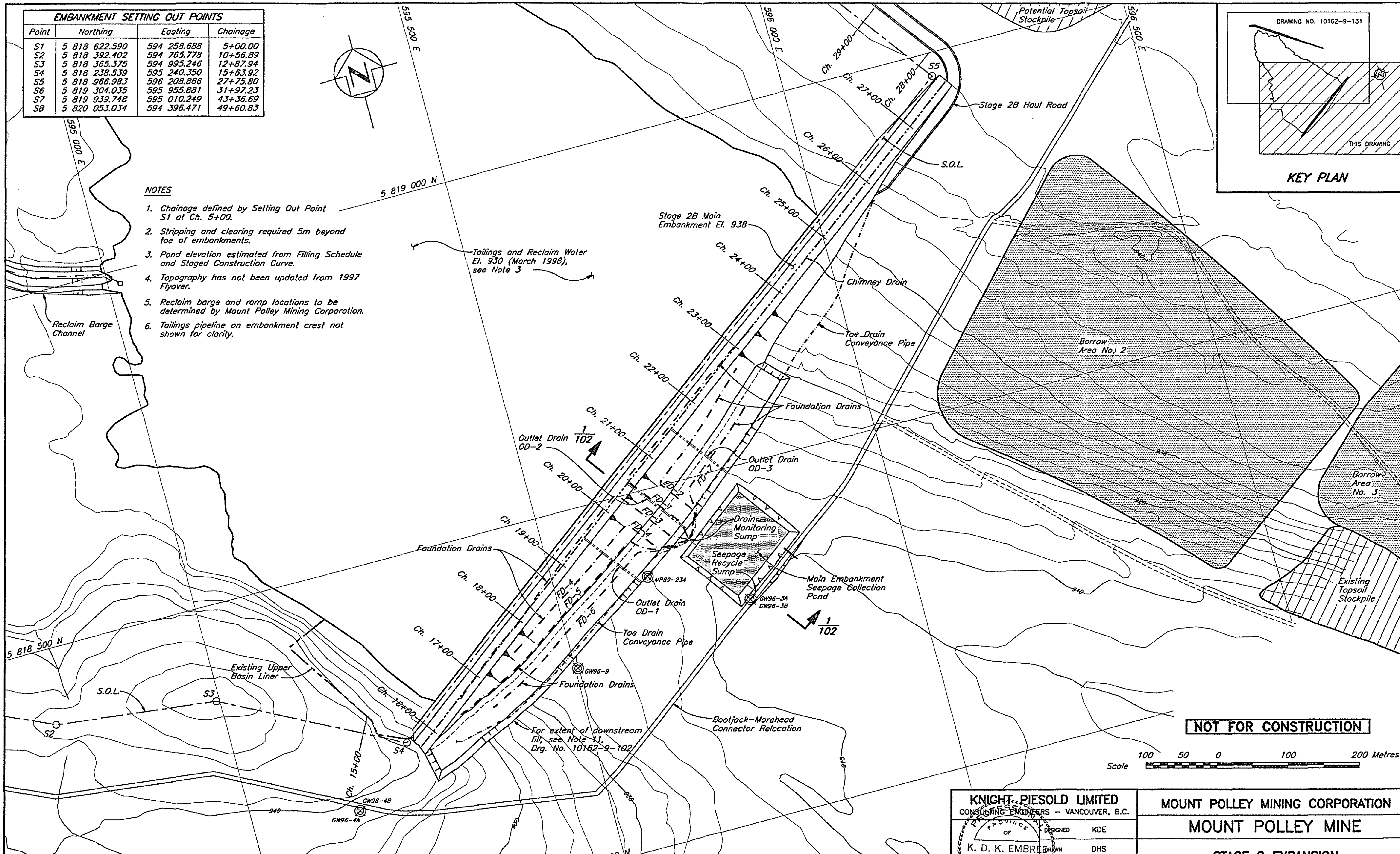
DATE NOV. 10, 1997 | SCALE AS SHOWN | DRG. NO. 10162-9-121 | REV. 0

CAD FILE: \10162\9\012\012-col 1:250 Plot 1=0.25 NOV. 10, 1997

EMBANKMENT SETTING OUT POINTS			
Point	Northing	Easting	Chainage
S1	5 818 622.590	594 258.688	5+00.00
S2	5 818 392.402	594 765.778	10+56.89
S3	5 818 365.375	594 995.246	12+87.94
S4	5 818 238.539	595 240.350	15+63.92
S5	5 818 966.983	596 208.866	27+75.80
S6	5 819 304.035	595 955.881	31+97.23
S7	5 819 939.748	595 010.249	43+36.69
S8	5 820 053.034	594 396.471	49+60.83

NOTES

1. Chainage defined by Setting Out Point S1 at Ch. 5+00.
2. Stripping and clearing required 5m beyond toe of embankments.
3. Pond elevation estimated from Filling Schedule and Staged Construction Curve.
4. Topography has not been updated from 1997 Flyover.
5. Reclaim barge and ramp locations to be determined by Mount Polley Mining Corporation.
6. Tailings pipeline on embankment crest not shown for clarity.



KEY PLAN

DRAWING NO. 10162-9-131

THIS DRAWING

DRG. NO.	DESCRIPTION
102	TSF - STAGE 2 MAIN EMBANKMENT - SECTIONS
131	TSF - STAGE 2B PERIMETER EMBANKMENT - PLAN

REFERENCE DRAWINGS

REV.	DATE	DESCRIPTION	APPROVED
0	DEC 1/97	ISSUED FOR DESIGN REPORT	

REVISIONS

KNIGHT PIESOLD LIMITED
CONSULTING ENGINEERS - VANCOUVER, B.C.

PROFESSIONAL ENGINEER PROVINCE OF BRITISH COLUMBIA K. D. K. EMBREE

DESIGNED KDE
DRAWN DHS
CHECKED ABW
APPROVED

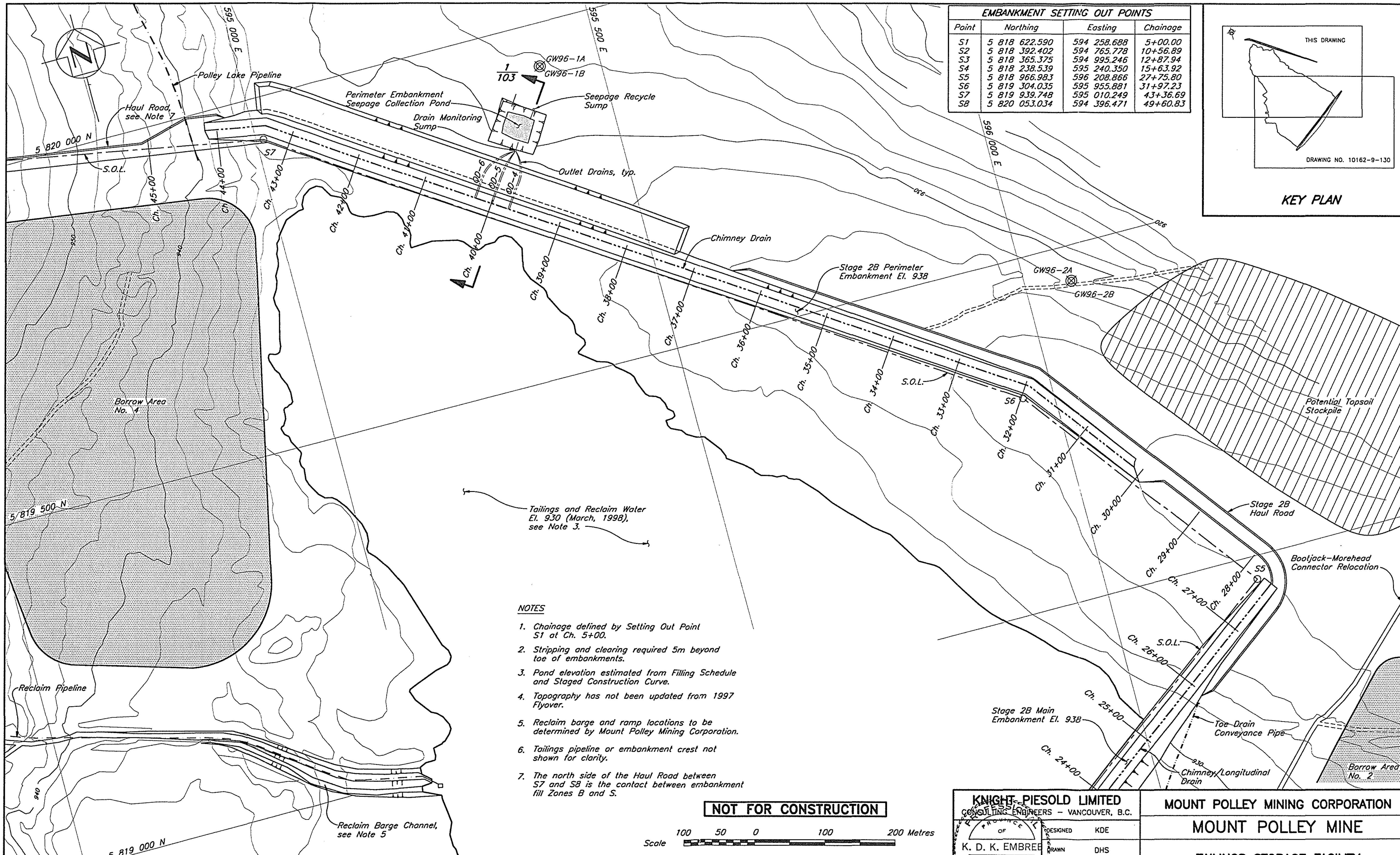
MOUNT POLLEY MINING CORPORATION

MOUNT POLLEY MINE

STAGE 2 EXPANSION
STAGE 2B MAIN EMBANKMENT
PLAN

DATE DEC. 1, 1997 | SCALE AS SHOWN | DRG. NO. 10162-9-130 | REV. 0

CAD FILE: 10162\9\022\022 1:2500 Pol 1=2.5 NOV 27, 1997



EMBANKMENT SETTING OUT POINTS

Point	Northing	Easting	Chainage
S1	5 818 622.590	594 258.688	5+00.00
S2	5 818 392.402	594 765.778	10+56.89
S3	5 818 365.375	594 995.246	12+87.94
S4	5 818 238.539	595 240.350	15+63.92
S5	5 818 966.983	596 208.866	27+75.80
S6	5 819 304.035	595 955.881	31+97.23
S7	5 819 939.748	595 010.249	43+36.69
S8	5 820 053.034	594 396.471	49+60.83

NOTES

1. Chainage defined by Setting Out Point S1 at Ch. 5+00.
2. Stripping and clearing required 5m beyond toe of embankments.
3. Pond elevation estimated from Filling Schedule and Staged Construction Curve.
4. Topography has not been updated from 1997 Flyover.
5. Reclaim barge and ramp locations to be determined by Mount Polley Mining Corporation.
6. Tailings pipeline or embankment crest not shown for clarity.
7. The north side of the Haul Road between S7 and S8 is the contact between embankment fill Zones B and S.

NOT FOR CONSTRUCTION

Scale 100 50 0 100 200 Metres

DRG. NO.	DESCRIPTION
103	TSF - STAGE 2 PARIMETER EMBANKMENT - SECTIONS
130	TSF - STAGE 2B MAIN EMBANKMENT - PLAN

REFERENCE DRAWINGS

REV.	DATE	DESCRIPTION	APPROVED

REVISIONS

REV.	DATE	DESCRIPTION	APPROVED
0	DEC 1/97	ISSUED FOR DESIGN REPORT	

REVISIONS

KNIGHT PIESOLD LIMITED
CONSULTING ENGINEERS - VANCOUVER, B.C.

PROFESSIONAL ENGINEER PROVINCE OF BRITISH COLUMBIA K. D. K. EMBREE

DESIGNED KDE
DRAWN DHS
CHECKED
APPROVED

MOUNT POLLEY MINING CORPORATION

MOUNT POLLEY MINE

TAILINGS STORAGE FACILITY
STAGE 2B PERIMETER EMBANKMENT
PLAN

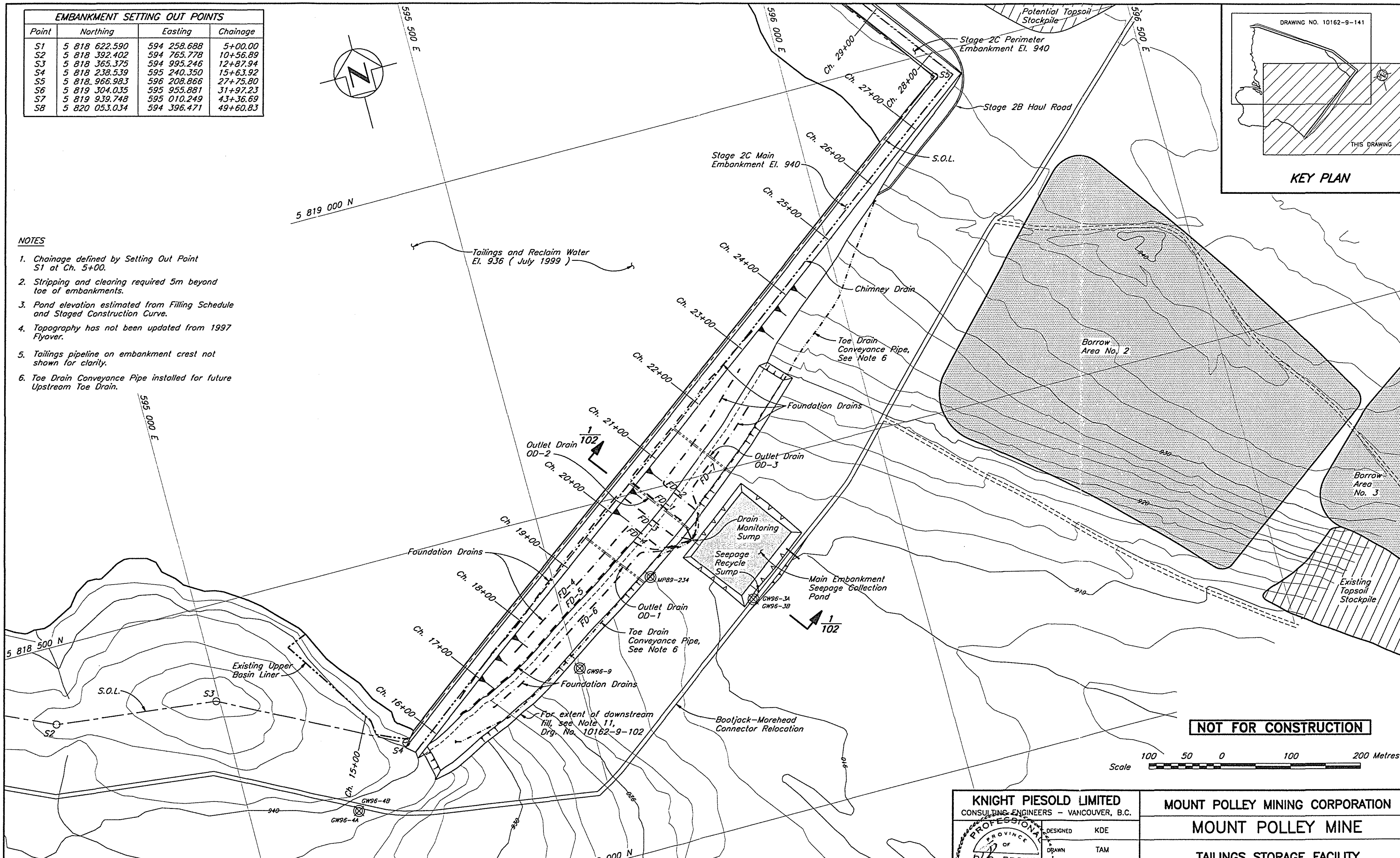
DATE DEC. 1, 1997 | SCALE AS SHOWN | DRG. NO. 10162-9-131 | REV. 0

EMBANKMENT SETTING OUT POINTS

Point	Northing	Easting	Chainage
S1	5 818 622.590	594 258.688	5+00.00
S2	5 818 392.402	594 765.778	10+56.89
S3	5 818 365.375	594 995.246	12+87.94
S4	5 818 238.539	595 240.350	15+63.92
S5	5 818 966.983	596 208.866	27+75.80
S6	5 819 304.035	595 955.881	31+97.23
S7	5 819 939.748	595 010.249	43+36.69
S8	5 820 053.034	594 396.471	49+60.83

NOTES

1. Chainage defined by Setting Out Point S1 at Ch. 5+00.
2. Stripping and clearing required 5m beyond toe of embankments.
3. Pond elevation estimated from Filling Schedule and Staged Construction Curve.
4. Topography has not been updated from 1997 Flyover.
5. Tailings pipeline on embankment crest not shown for clarity.
6. Toe Drain Conveyance Pipe installed for future Upstream Toe Drain.



KEY PLAN

DRAWING NO. 10162-9-141

THIS DRAWING

NOT FOR CONSTRUCTION

Scale 100 50 0 100 200 Metres

DRG. NO.	DESCRIPTION
141	TSF – STAGE 2C PERIMETER EMBANKMENT – PLAN
102	TSF – STAGE 2 MAIN EMBANKMENT – SECTIONS

REFERENCE DRAWINGS

REV.	DATE	DESCRIPTION	APPROVED
0	DEC 1/97	ISSUED FOR DESIGN REPORT	

REVISIONS

KNIGHT PIESOLD LIMITED
CONSULTING ENGINEERS – VANCOUVER, B.C.

PROFESSIONAL ENGINEER – PROVINCE OF BRITISH COLUMBIA – B.S. BROWN

DESIGNED	KDE
DRAWN	TAM
CHECKED	
APPROVED	

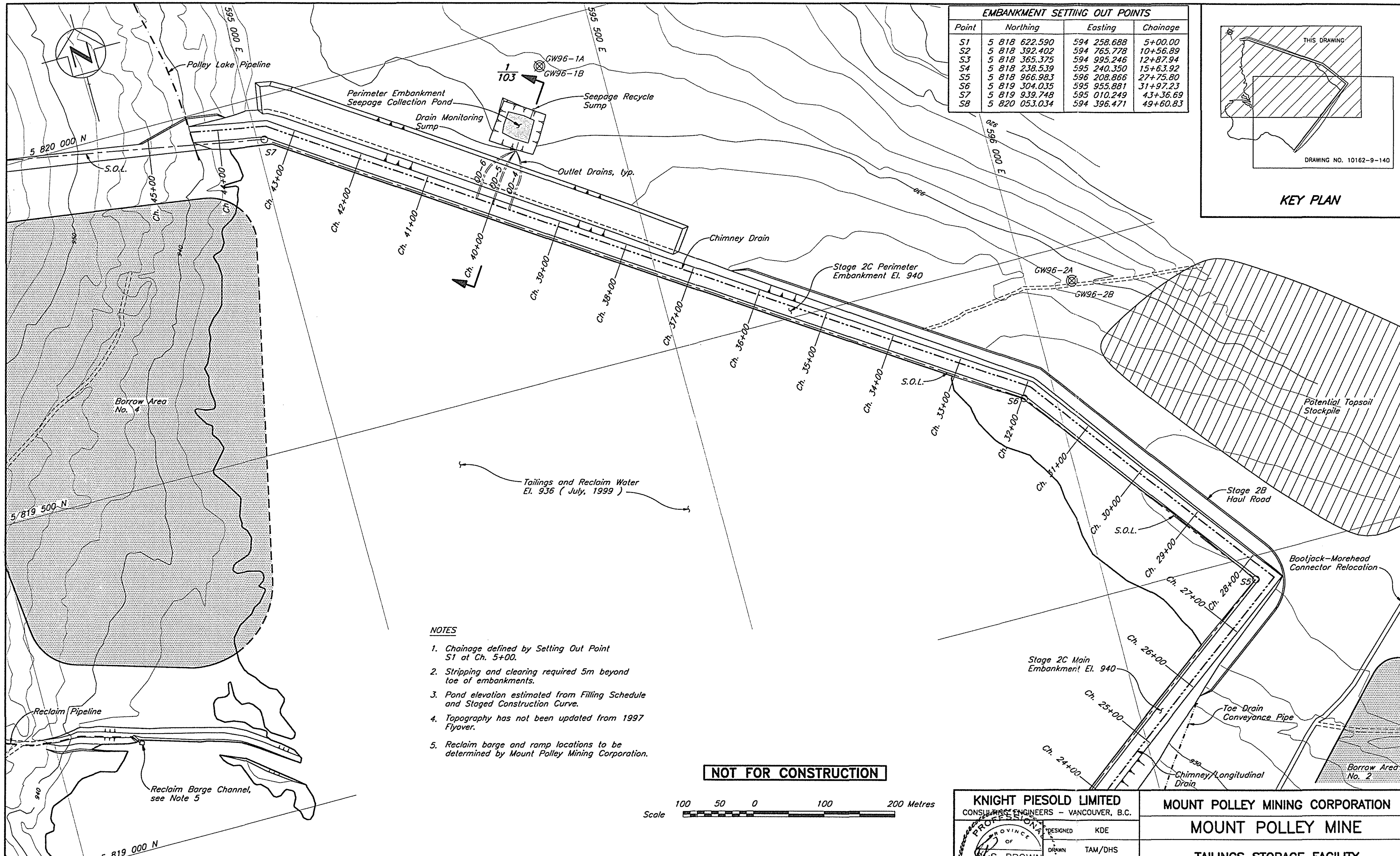
MOUNT POLLEY MINING CORPORATION

MOUNT POLLEY MINE

TAILINGS STORAGE FACILITY
STAGE 2C MAIN EMBANKMENT
PLAN

DATE DEC. 1, 1997 | SCALE AS SHOWN | DRG. NO. 10162-9-140 | REV. 0

CAD FILE: 10162\9_05\08 1:2500 Plot 1=2.5 NOV 28, 1997



EMBANKMENT SETTING OUT POINTS			
Point	Northing	Easting	Chainage
S1	5 818 622.590	594 258.688	5+00.00
S2	5 818 392.402	594 765.778	10+56.89
S3	5 818 365.375	594 995.246	12+87.94
S4	5 818 238.539	595 240.350	15+63.92
S5	5 818 966.983	596 208.866	27+75.80
S6	5 819 304.035	595 955.881	31+97.23
S7	5 819 939.748	595 010.249	43+36.69
S8	5 820 053.034	594 396.471	49+60.83

KEY PLAN

NOTES

1. Chainage defined by Setting Out Point S1 at Ch. 5+00.
2. Stripping and clearing required 5m beyond toe of embankments.
3. Pond elevation estimated from Filling Schedule and Staged Construction Curve.
4. Topography has not been updated from 1997 Flyover.
5. Reclaim barge and ramp locations to be determined by Mount Polley Mining Corporation.

NOT FOR CONSTRUCTION

Scale 100 50 0 100 200 Metres

DRG. NO.	DESCRIPTION
140	TSF - STAGE 2C MAIN EMBANKMENT - PLAN
103	TSF - STAGE 2 PARIMETER EMBANKMENT - SECTIONS

REFERENCE DRAWINGS

REV.	DATE	DESCRIPTION	APPROVED
0	DEC 1/97	ISSUED FOR DESIGN REPORT	

REVISIONS

KNIGHT PIESOLD LIMITED
CONSULTING ENGINEERS - VANCOUVER, B.C.

PROFESSIONAL ENGINEER PROVINCE OF BRITISH COLUMBIA S. BROWN

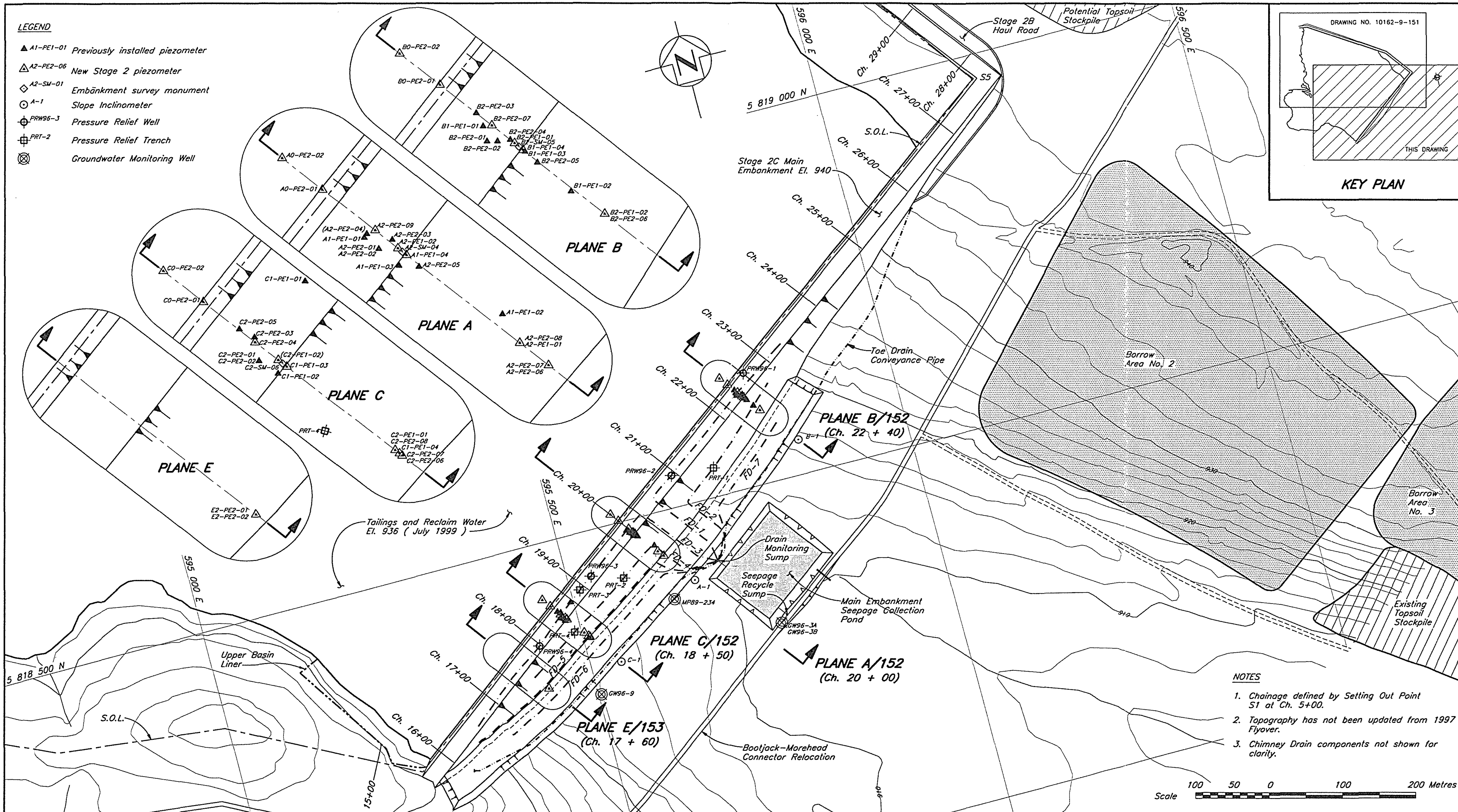
DESIGNED KDE
DRAWN TAM/DHS
CHECKED ABW
APPROVED

MOUNT POLLEY MINING CORPORATION

MOUNT POLLEY MINE

TAILINGS STORAGE FACILITY
STAGE 2C PERIMETER EMBANKMENT
PLAN

DATE DEC. 1, 1997 | SCALE AS SHOWN | DRG. NO. 10162-9-141 | REV. 0



NOTES

1. Chainage defined by Setting Out Point S1 at Ch. 5+00.
2. Topography has not been updated from 1997 Flyover.
3. Chimney Drain components not shown for clarity.

NOT FOR CONSTRUCTION

DRG. NO.	DESCRIPTION
153	TSF – STAGE 2 EXPANSION – INSTRUMENTATION SECTIONS – SHEET 2 OF 2
152	TSF – STAGE 2 EXPANSION – INSTRUMENTATION SECTIONS – SHEET 1 OF 2
151	TSF – STAGE 2 EXPANSION – PERIMETER EMBANKMENT INSTRUMENTATION – PLAN

REFERENCE DRAWINGS

REV.	DATE	DESCRIPTION	APPROVED
0	DEC 1/97	ISSUED FOR DESIGN REPORT	

REVISIONS

KNIGHT PIESOLD LIMITED
CONSULTING ENGINEERS – VANCOUVER, B.C.

PROFESSIONAL ENGINEER – PROVINCE OF BRITISH COLUMBIA – S. BROWN

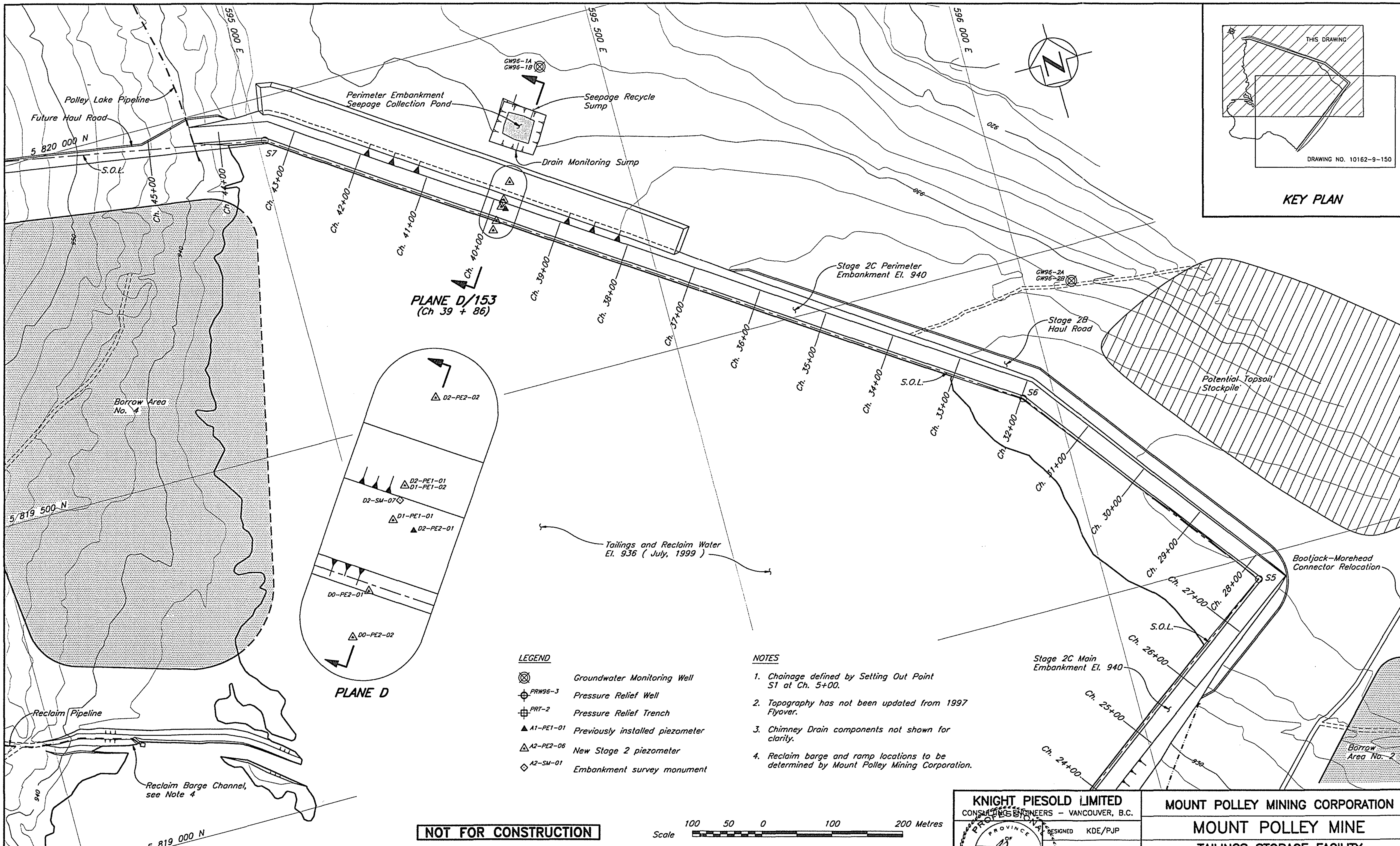
DESIGNED KDE/PJP
DRAWN TAM/DHS/RDT
CHECKED
APPROVED

MOUNT POLLEY MINING CORPORATION

MOUNT POLLEY MINE

TAILINGS STORAGE FACILITY
STAGE 2 EXPANSION
MAIN EMBANKMENT
INSTRUMENTATION PLAN

DATE DEC. 1, 1997 | SCALE AS SHOWN | DRG. NO. 10162-9-150 | REV. 0



LEGEND

(symbol)	Groundwater Monitoring Well
PRW96-3	Pressure Relief Well
PRT-2	Pressure Relief Trench
A1-PE1-01	Previously installed piezometer
A2-PE2-06	New Stage 2 piezometer
A2-SM-01	Embankment survey monument

NOTES

1. Chainage defined by Setting Out Point S1 at Ch. 5+00.
2. Topography has not been updated from 1997 Flyover.
3. Chimney Drain components not shown for clarity.
4. Reclaim barge and ramp locations to be determined by Mount Polley Mining Corporation.

NOT FOR CONSTRUCTION

DRG. NO.	DESCRIPTION
153	TSF - STAGE 2 EXPANSION - INSTRUMENTATION - SECTIONS - SHEET 2 OF 2
152	TSF - STAGE 2 EXPANSION - INSTRUMENTATION - SECTIONS - SHEET 1 OF 2
150	TSF - STAGE 2 EXPANSION - MAIN EMBANKMENT INSTRUMENTATION - PLAN

REFERENCE DRAWINGS

REV.	DATE	DESCRIPTION	APPROVED

REVISIONS

REV.	DATE	DESCRIPTION	APPROVED
0	DEC 1/97	ISSUED FOR DESIGN REPORT	

REVISIONS

KNIGHT PIESOLD LIMITED
CONSULTING ENGINEERS - VANCOUVER, B.C.

PROFESSIONAL ENGINEER — PROVINCE OF BRITISH COLUMBIA — B. BROWN

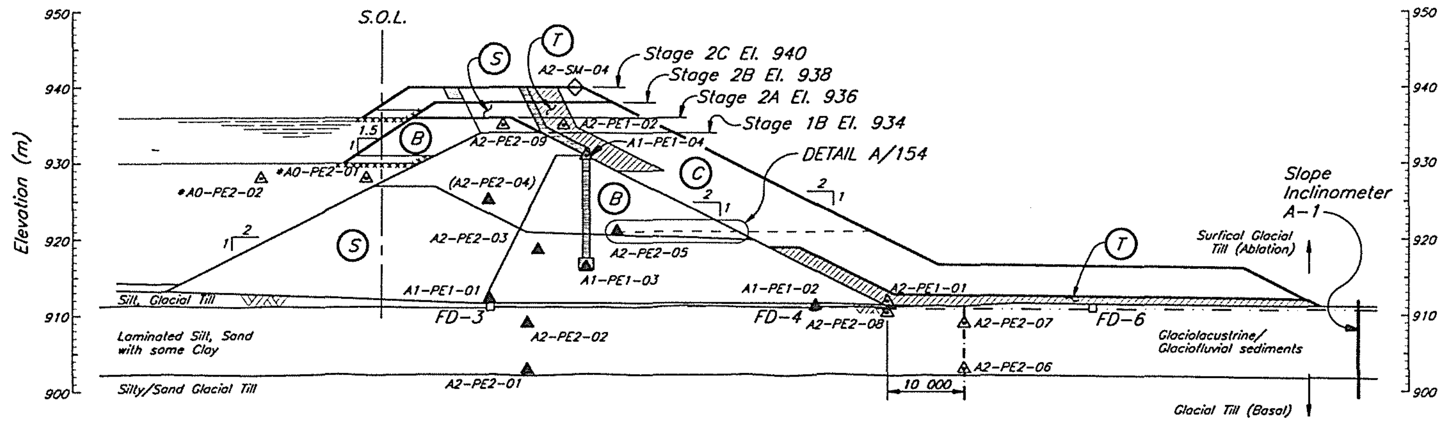
DESIGNED	KDE/PJP
DRAWN	TAM/DHS/RDT
CHECKED	CBW
APPROVED	

MOUNT POLLEY MINING CORPORATION

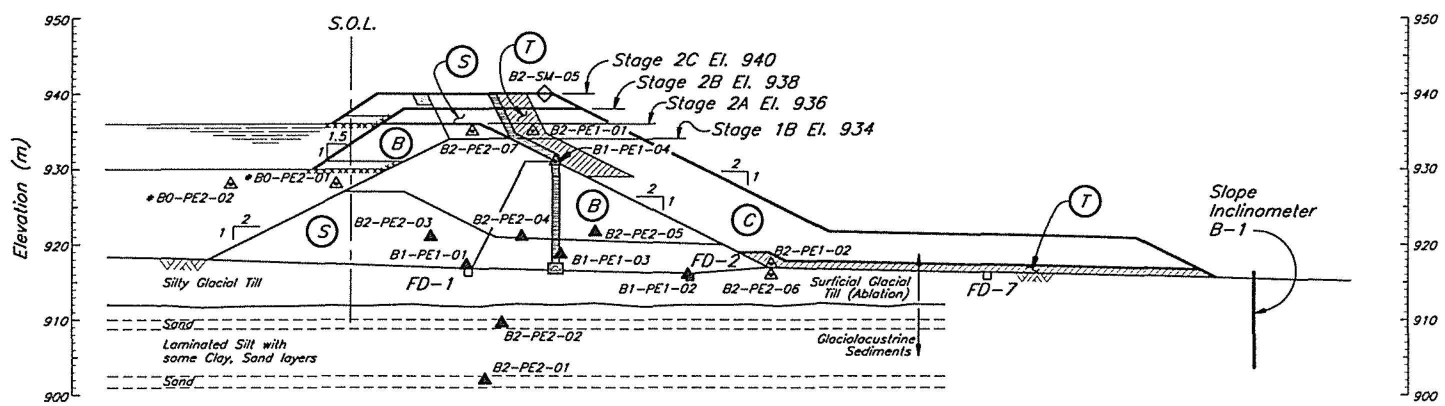
MOUNT POLLEY MINE

TAILINGS STORAGE FACILITY
STAGE 2 EXPANSION
PERIMETER EMBANKMENT
INSTRUMENTATION PLAN

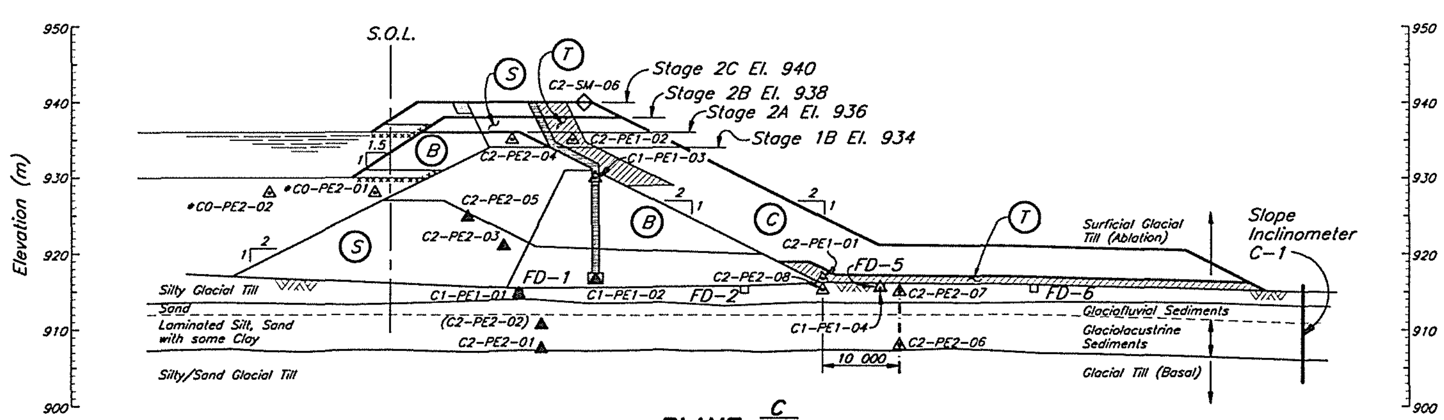
DATE DEC. 1, 1997 | SCALE AS SHOWN | DRG. NO. 10162-9-151 | REV. 0



PLANE A/150
CH. 20+00



PLANE B/150
CH. 22+40



PLANE C/150
CH. 18+50

SUMMARY OF PIEZOMETER INSTALLATIONS

PIEZOMETER ID	LEAD LENGTH (m)	NORTHING	EASTING	ELEV.	DATE INSTALLED
* A0-PE2-01				928	
* A0-PE2-02				928	
A1-PE1-01	175	5 818 486.650	595 595.060	912.99	27/08/96
A1-PE1-02	150	5 818 456.420	595 626.250	912.14	27/08/96
A1-PE1-03	200	5 818 476.822	595 602.380	917.17	22/10/96
** A1-PE1-04				931	
** A2-PE1-01				912	
** A2-PE1-02				935	
A2-PE2-01	200	5 818 482.710	595 598.140	903.7	25/07/96
A2-PE2-02	200	5 818 482.710	595 598.140	909.8	25/07/96
A2-PE2-03	175	5 818 484.196	595 598.140	919.43	12/02/97
(A2-PE2-04)	200	5 818 487.510	595 595.995	926.07	22/02/97
A2-PE2-05	175	5 818 475.061	595 607.560	921.87	22/02/97
** A2-PE2-06				903	
** A2-PE2-07				909	
** A2-PE2-08				910	
** A2-PE2-09				935	
* B0-PE2-01				928	
* B0-PE2-02				928	
B1-PE1-01	300	5 818 632.550	595 787.910	917.27	10/09/96
B1-PE1-02	275	5 818 609.040	595 806.770	915.95	10/09/96
B1-PE1-03	305	5 818 622.780	595 797.260	918.69	22/10/96
** B1-PE1-04				931	
** B2-PE1-01				935	
** B2-PE1-02				917.5	
B2-PE2-01	325	5 818 628.270	595 787.880	902.00	25/07/96
B2-PE2-02	325	5 818 627.470	595 790.660	909.50	25/07/96
B2-PE2-03	325	5 818 636.530	595 786.970	921.00	22/10/96
B2-PE2-04	330	5 818 626.940	595 794.190	921.00	22/10/96
B2-PE2-05	325	5 818 619.014	595 799.804	921.70	14/03/97
** B2-PE2-06				916	
** B2-PE2-07				935	
* C0-PE2-01				928	
* C0-PE2-02				928	
C1-PE1-01	325	5 818 410.500	595 496.070	914.70	28/09/96
C1-PE1-02	330	5 818 410.500	595 496.070	916.60	22/10/96
** C1-PE1-03				930	
** C1-PE1-04				915	
** C2-PE1-01				917	
** C2-PE1-02				935	
C2-PE2-01	350	5 818 392.410	595 478.240	907.50	25/07/96
(C2-PE2-02)	350	5 818 392.410	595 478.240	910.50	25/07/96
C2-PE2-03	325	5 818 399.106	595 478.824	920.97	12/02/97
** C2-PE2-04				908	
C2-PE2-05	325	5 818 402.343	595 475.326	924.84	12/02/97
** C2-PE2-06				913	
** C2-PE2-07				915	
** C2-PE2-08				935	
*** D0-PE2-01				935	
*** D0-PE2-02				935	
** D1-PE1-01	90			933	
** D1-PE1-02				929	
D2-PE2-01	85	5 819 756.360	595 316.210	931.00	15/12/96
** D2-PE2-02				922	
** E2-PE2-01				908	
** E2-PE2-02				913	

* To be installed during Stage 2A construction.
** To be installed during Stage 2B construction.
*** To be installed during Stage 2C construction.
( ) Piezometer no longer functioning.

NOTES

1. Piezometers are vibrating wire type, RST model VW-2100 with a pressure rating of 100 psi or equivalent, connected to a readout panel via standard non-vented model VW-232 direct burial cable.
2. Piezometer leads are to be extended to Instrumentation Monitoring Hut after foundation preparation for final embankment during Stage 2 construction.
3. Future survey monuments not shown. A minimum of 2 monuments will be installed for each embankment raise.
4. Installation details for borehole piezometers as shown on Drg. No. 10162-9-154.

LEGEND

A0-PE1-01
- A — Plane I.D. (A, B etc.)
- 0 — Area (0-Tailings, 1-Drain, 2-Embankment)
- 01 — Number I.D.
- 1 — Pressure Rating (1-Low, 2-High)
- PE — Type of Instrumentation (PE-Piezometer electric, SM-Survey Monument)

A1-PE1-01 ▲ Previously installed piezometer
A2-PE2-06 △ New Stage 2 piezometer
A2-SM-01 ◇ Embankment survey monument

NOT FOR CONSTRUCTION

Scale 10 0 10 20 30 40 50 m

CAD FILE: \10162\9\014\014 1:500 Plot 1=0.5 NOV 28, 1997

KNIGHT PIESOLD LIMITED
CONSULTING ENGINEERS - VANCOUVER, B.C.

PROFESSIONAL ENGINEER — PROVINCE OF BRITISH COLUMBIA — S. BROWN

DESIGNED	KDE/PJP
DRAWN	NSD/DHS/RDT
CHECKED	GBW
APPROVED	

MOUNT POLLEY MINING CORPORATION

MOUNT POLLEY MINE

TAILINGS STORAGE FACILITY
STAGE 2 EXPANSION
INSTRUMENTATION SECTIONS
SHEET 1 OF 2

DATE DEC. 1, 1997 | SCALE AS SHOWN | DRG. NO. 10162-9-152 | REV. 0

DRG. NO.	DESCRIPTION
154	TSF - STAGE 2 EXPANSION - INSTRUMENTATION - DETAILS
153	TSF - STAGE 2 EXPANSION - INSTRUMENTATION - SECTIONS - SHEET 2 OF 2
150	TSF - STAGE 2 EXPANSION - MAIN EMBANKMENT INSTRUMENTATION - PLAN

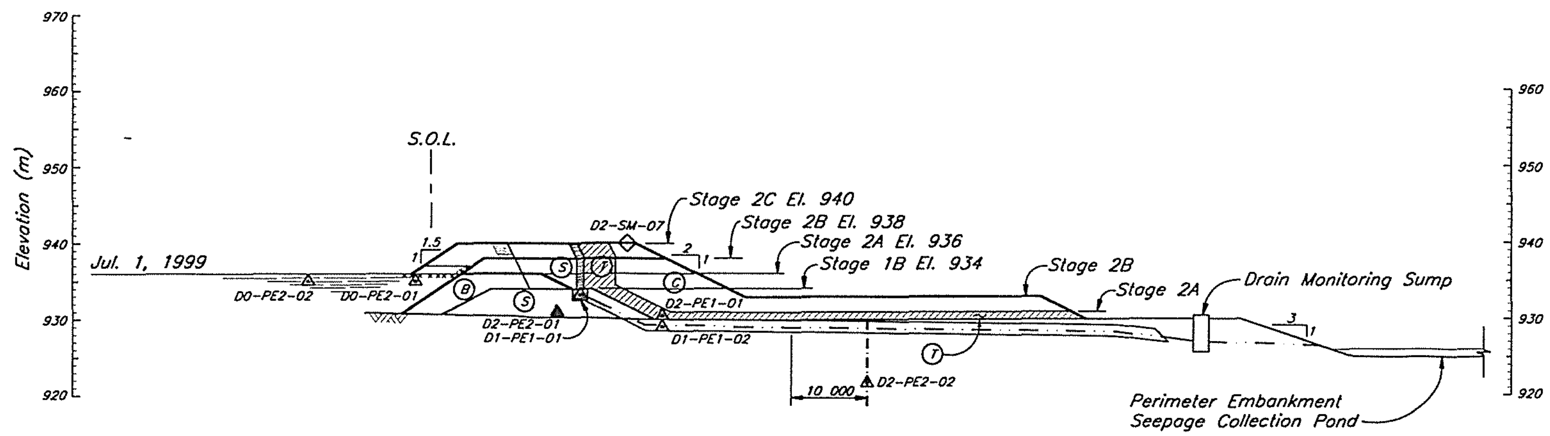
REFERENCE DRAWINGS

REV.	DATE	DESCRIPTION	APPROVED

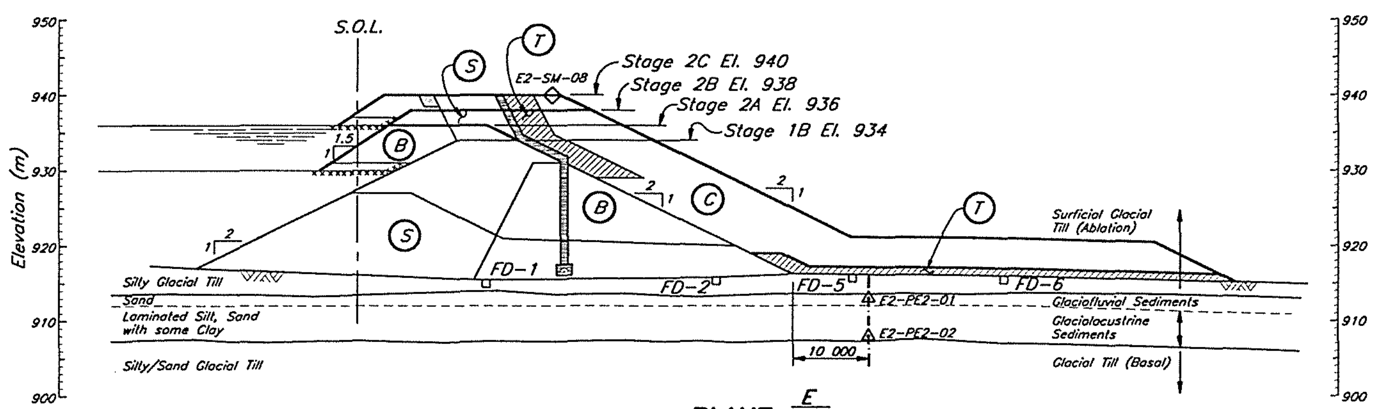
REVISIONS

REV.	DATE	DESCRIPTION	APPROVED
0	DEC 1/97	ISSUED FOR DESIGN REPORT	

REVISIONS



PLANE $\frac{D}{151}$
CH. 39+86



PLANE $\frac{E}{150}$
CH. 17+60

NOTES

1. Piezometers are vibrating wire type, RST model VW-2100 with a pressure rating of 100 psi or equivalent, connected to a readout panel via standard non-vented model VW-232 direct burial cable.
2. Piezometer leads are to be extended to Instrumentation Monitoring Hut after foundation preparation for final embankment during Stage 2 construction.
3. Future survey monuments not shown. A minimum of 2 monuments will be installed for each embankment raise.
4. Installation details for borehole piezometers as shown on Drg. No. 10162-9-154.
5. See Drg. No. 10162-9-152 for Summary of Piezometer Installations.

LEGEND

D0-PE1-01
- Plane I.D. (A, B etc.)
- Area (0-Tailings, 1-Drain, 2-Embankment)
- Number I.D.
- Pressure Rating (1-Low, 2-High)
- Type of Instrumentation (PE-Piezometer electric, SM-Survey Monument)

D1-PE1-01 ▲ Previously installed piezometer

D2-PE2-02 △ New Stage 2 piezometer

D2-SM-07 ◇ Embankment survey monument

NOT FOR CONSTRUCTION

Scale 10 0 10 20 30 40 50 m

CAD FILE: \10162\9\016\016 1:500 Plot 1=0.5 NOV 28, 1997

DRG. NO.	DESCRIPTION
154	TSF - STAGE 2 EXPANSION - INSTRUMENTATION - DETAILS
152	TSF - STAGE 2 EXPANSION - INSTRUMENTATION - SECTIONS - SHEET 1 OF 2
151	TSF - STAGE 2 EXPANSION - PERIMETER EMBANKMENT INSTRUMENTATION - PLAN
150	TSF - STAGE 2 EXPANSION - MAIN EMBANKMENT INSTRUMENTATION - PLAN

REFERENCE DRAWINGS

REV.	DATE	DESCRIPTION	APPROVED

REVISIONS

REV.	DATE	DESCRIPTION	APPROVED
0	DEC 1/97	ISSUED FOR DESIGN REPORT	

REVISIONS

KNIGHT PIESOLD LIMITED
CONSULTING ENGINEERS - VANCOUVER, B.C.

PROFESSIONAL ENGINEER, PROVINCE OF BRITISH COLUMBIA, B. BROWN

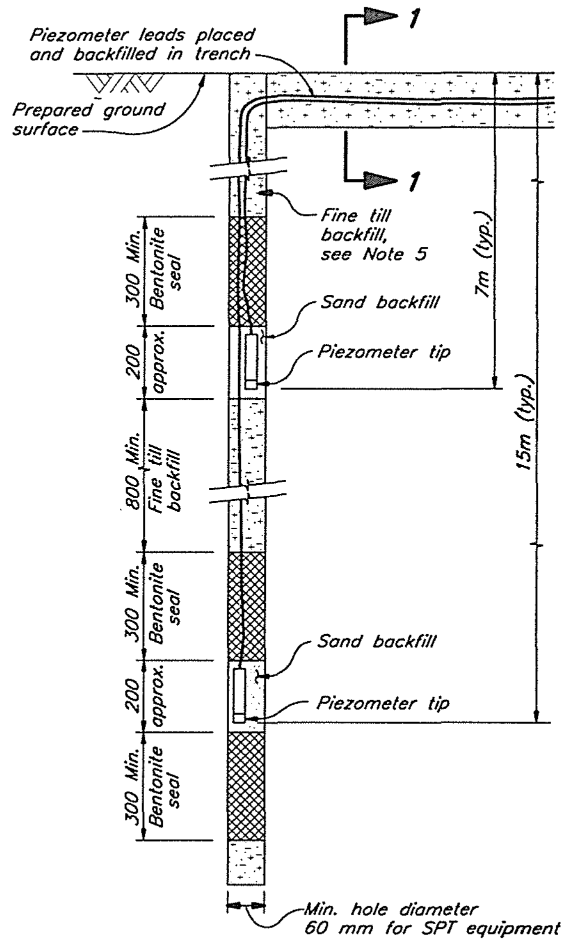
DESIGNED KDE/PJP
DRAWN NSD/DHS/RDT
CHECKED ABW
APPROVED

MOUNT POLLEY MINING CORPORATION

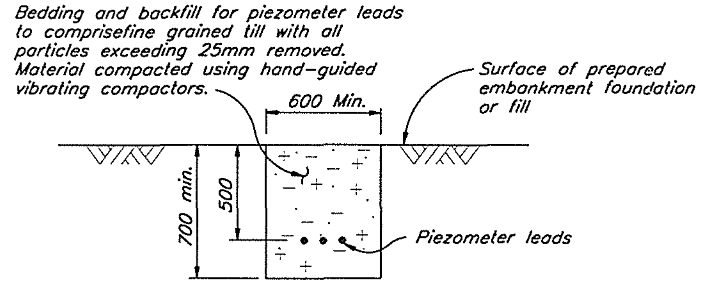
MOUNT POLLEY MINE

TAILINGS STORAGE FACILITY
STAGE 2 EXPANSION
INSTRUMENTATION SECTIONS
SHEET 2 OF 2

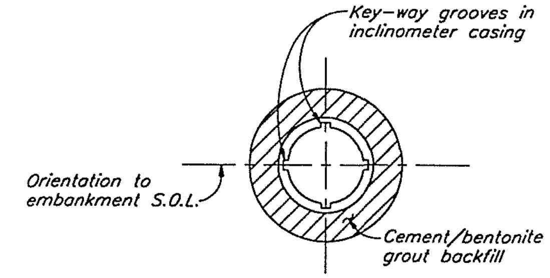
DATE DEC. 1, 1997 | SCALE AS SHOWN | DRG. NO. 10162-9-153 | REV. 0



TYPICAL INSTALLATION OF PIEZOMETERS IN BOREHOLES
NTS

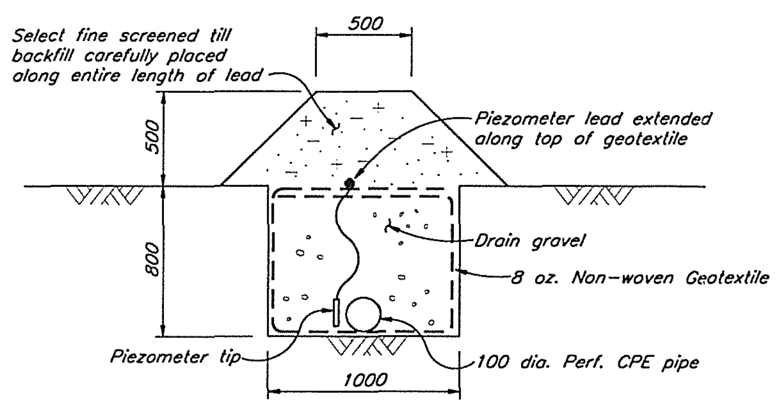


SECTION 1
TYPICAL SECTION THROUGH PIEZOMETER LEAD TRENCH IN PREPARED EMBANKMENT FOUNDATION OR FILL
Scale A

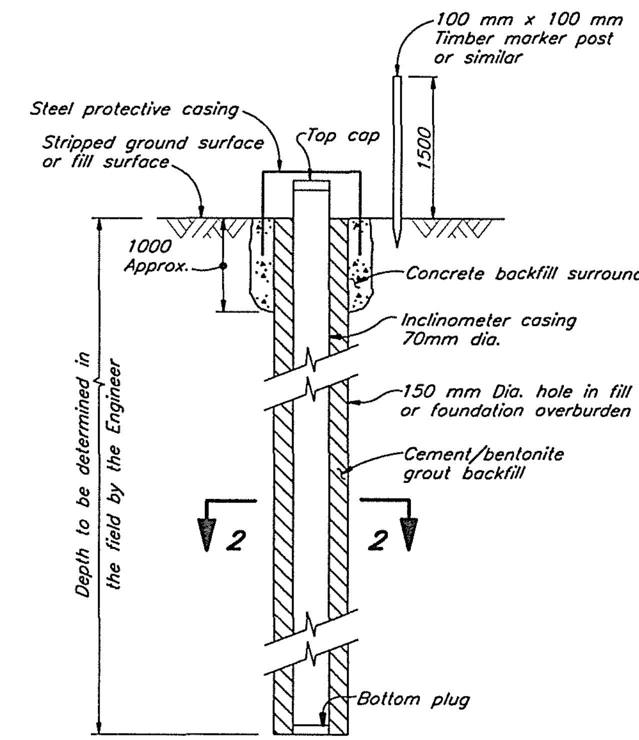


SECTION 2
N.T.S.

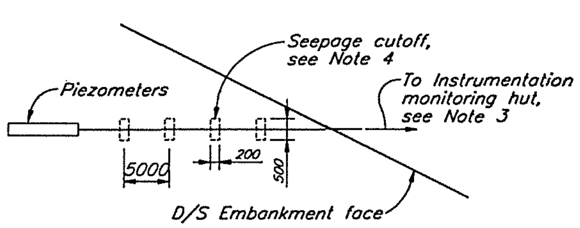
NOTES
1. Dimensions are in millimeters unless otherwise noted.
2. Tailings piezometers to be installed during future investigation programs.
3. Piezometer leads are to be extended to Instrumentation Monitoring Hut after foundation preparation during Stage 2 construction.
4. Seepage cutoffs placed at 5 m intervals with 10% bentonite added to fine grained till backfill.



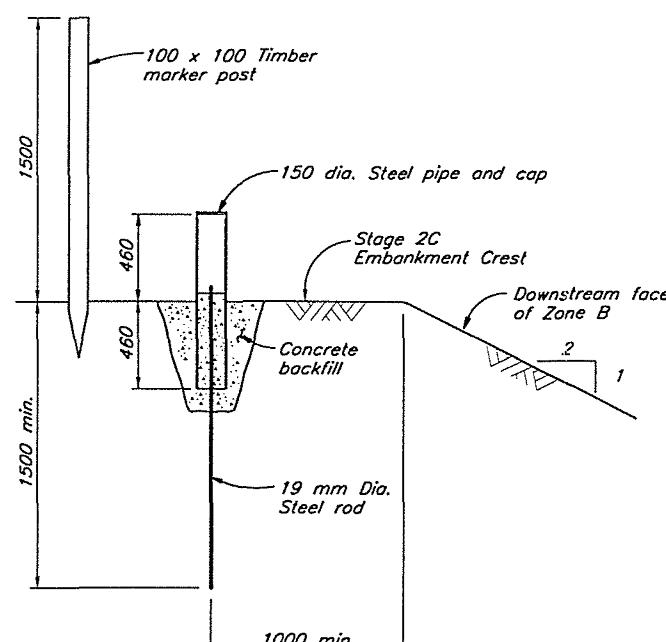
TYPICAL PIEZOMETER INSTALLATION IN EMBANKMENT FOUNDATION DRAIN OR TOE DRAIN
Scale A



DETAIL C
TYPICAL INCLINOMETER INSTALLATION IN BOREHOLE
N.T.S.



DETAIL A/140
TYPICAL DETAIL OF SEEPAGE CUTOFF FOR PIEZOMETER LEADS IN GLACIAL TILL
NTS



DETAIL OF SURFACE MOVEMENT MONUMENT
Scale A

NOT FOR CONSTRUCTION

Scale 1000 500 0 1000 2000 mm

DRG. NO.	DESCRIPTION
153	TSF - STAGE 2 EXPANSION - INSTRUMENTATION - SECTIONS - SHEET 2 OF 2
152	TSF - STAGE 2 EXPANSION - INSTRUMENTATION - SECTIONS - SHEET 1 OF 2
151	TSF - STAGE 2 EXPANSION - PERIMETER EMBANKMENT INSTRUMENTATION - PLAN
150	TSF - STAGE 2 EXPANSION - MAIN EMBANKMENT INSTRUMENTATION - PLAN

REFERENCE DRAWINGS

REV.	DATE	DESCRIPTION	APPROVED

REVISIONS

REV.	DATE	DESCRIPTION	APPROVED
0	DEC 1/97	ISSUED FOR DESIGN REPORT	

REVISIONS

KNIGHT PIESOLD LIMITED
CONSULTING ENGINEERS - VANCOUVER, B.C.

PROFESSIONAL ENGINEER — PROVINCE OF BRITISH COLUMBIA — B. S. BROWN

DESIGNED KDE
DRAWN NSD/DHS/RDT
CHECKED
APPROVED

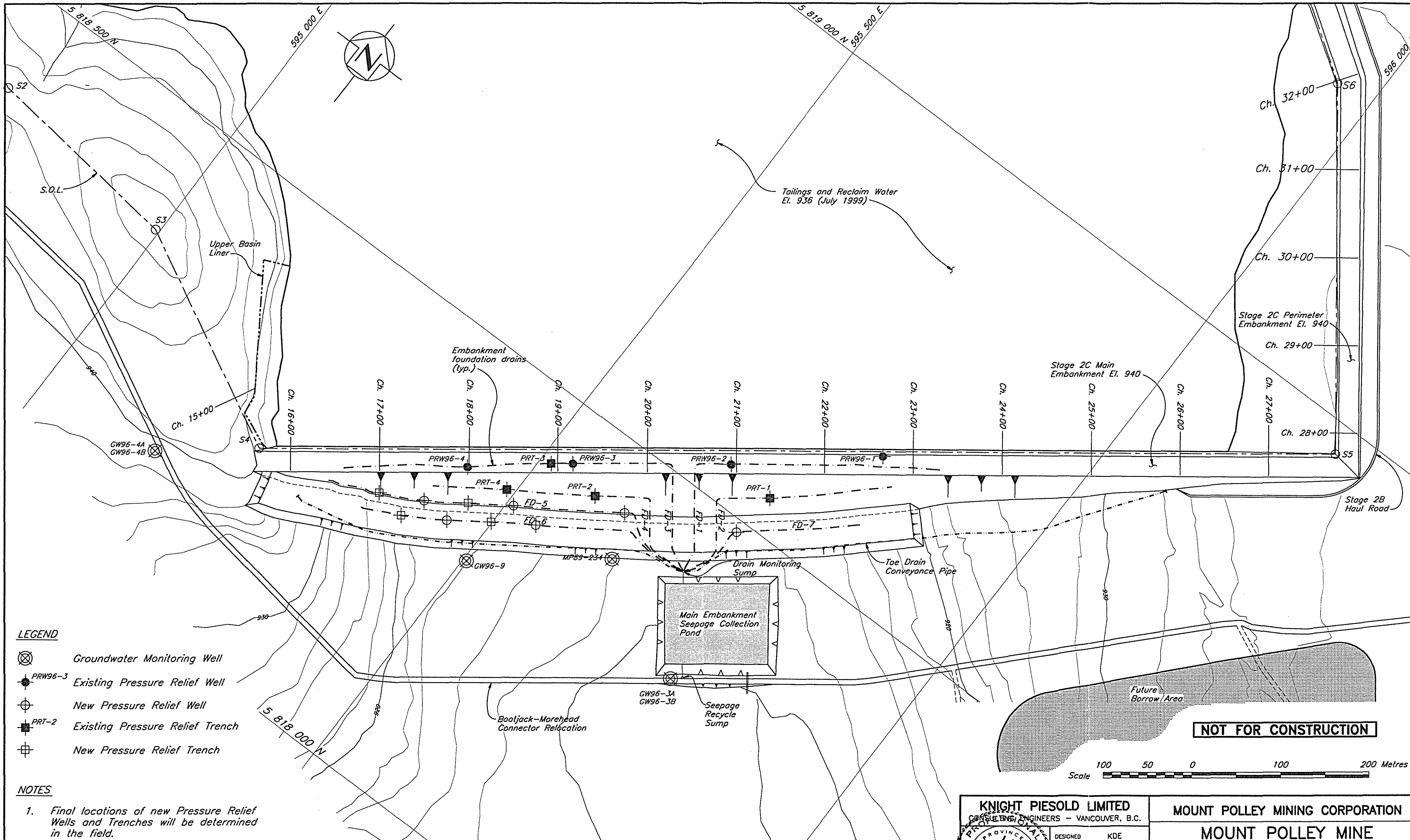
MOUNT POLLEY MINING CORPORATION

MOUNT POLLEY MINE

TAILINGS STORAGE FACILITY
STAGE 2 EXPANSION INSTRUMENTATION
DETAILS

DATE DEC. 1, 1997 | SCALE AS SHOWN | DRG. NO. 10162-9-154 | REV. 0

CAD FILE:\10162\9\018\018 1:20 Pxl 1=0.02 DEC 1, 1997



KNIGHT PIESOLD LIMITED
CONSULTING ENGINEERS – VANCOUVER, B.C.

PROVINCE OF BRITISH COLUMBIA
B. S. BROWN
PROFESSIONAL ENGINEER

DESIGNED KDE
DRAWN DHS/RDT
CHECKED ABW
APPROVED

MOUNT POLLEY MINING CORPORATION

MOUNT POLLEY MINE

TAILINGS STORAGE FACILITY
STAGE 2 EXPANSION
MAIN EMBANKMENT
PRESSURE RELIEF DETAILS

DATE DEC. 1, 1997 | SCALE AS SHOWN | DRG. NO. 10162-9-155 | REV. 0

REV.	DATE	DESCRIPTION	APPROVED
0	DEC 1/97	ISSUED FOR DESIGN REPORT	

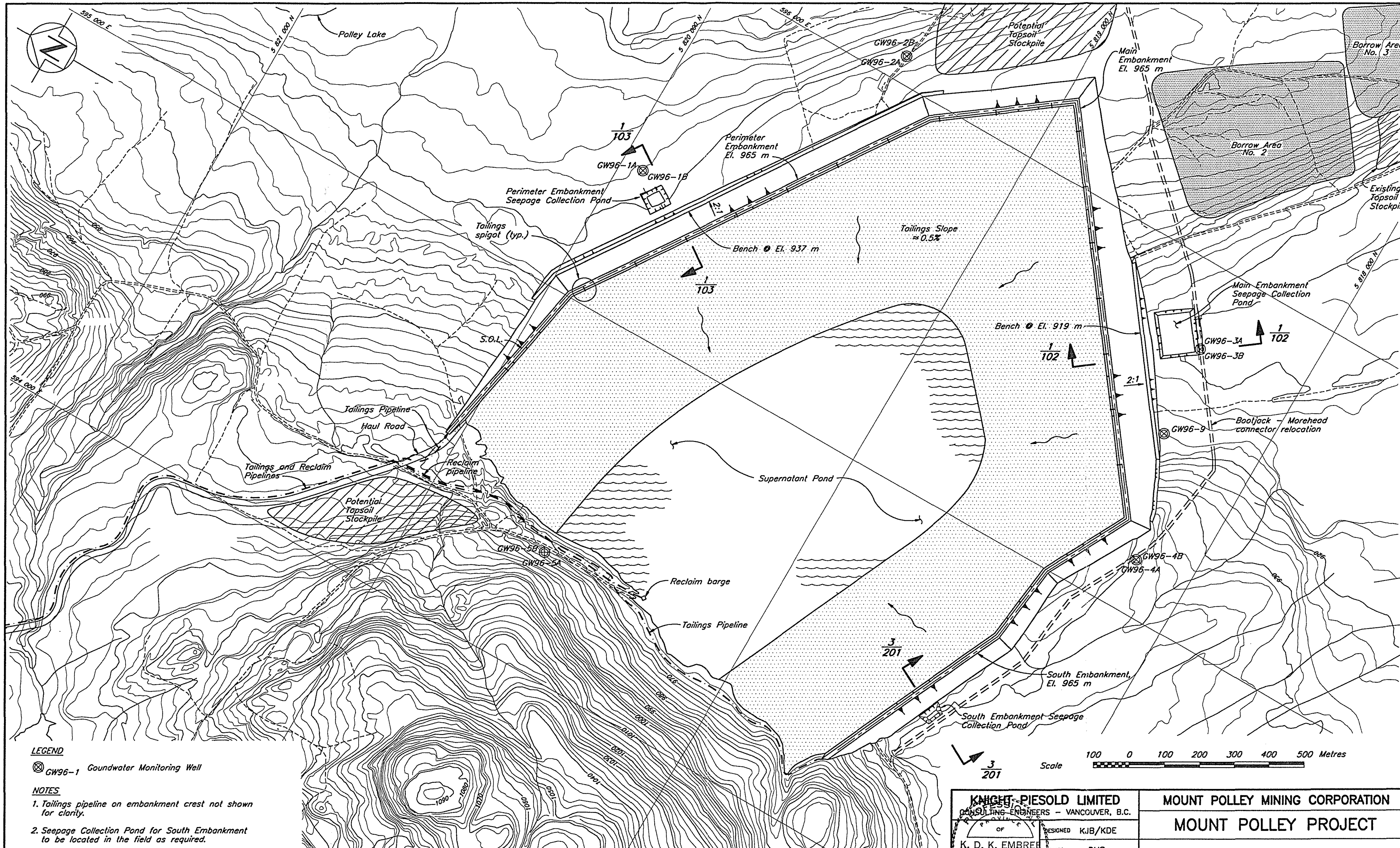
DRG. NO.	DESCRIPTION

REFERENCE DRAWINGS

REV.	DATE	DESCRIPTION	APPROVED

REVISIONS

REVISIONS



LEGEND

GW96-1 Goundwater Monitoring Well

NOTES

1. Tailings pipeline on embankment crest not shown for clarity.
2. Seepage Collection Pond for South Embankment to be located in the field as required.

DRG. NO.	DESCRIPTION
201	TSF - FINAL TAILINGS EMBANKMENT - SECTIONS

REFERENCE DRAWINGS

REV.	DATE	DESCRIPTION	APPROVED
0	DEC 1/97	ISSUED FOR DESIGN REPORT	

REVISIONS

KNIGHT PIESOLD LIMITED
CONSULTING ENGINEERS - VANCOUVER, B.C.

K. D. K. EMBREE

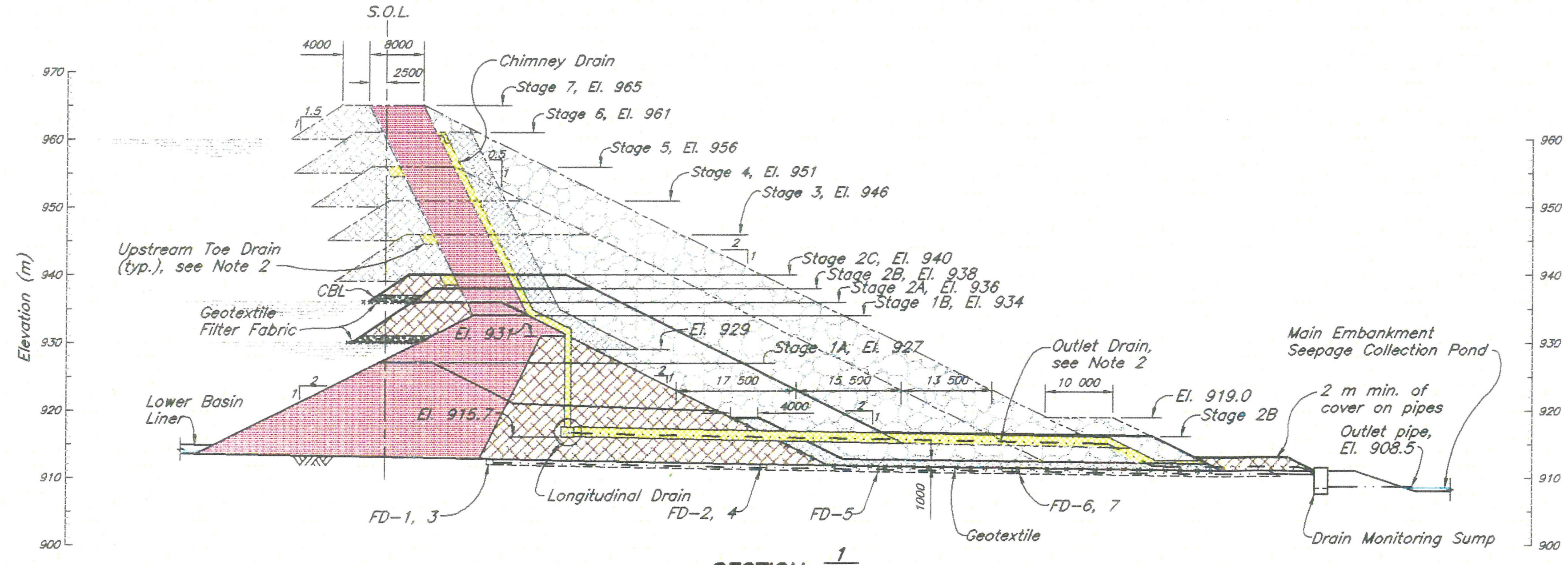
DESIGNED KJB/KDE
DRAWN DHS
CHECKED
APPROVED

MOUNT POLLEY MINING CORPORATION

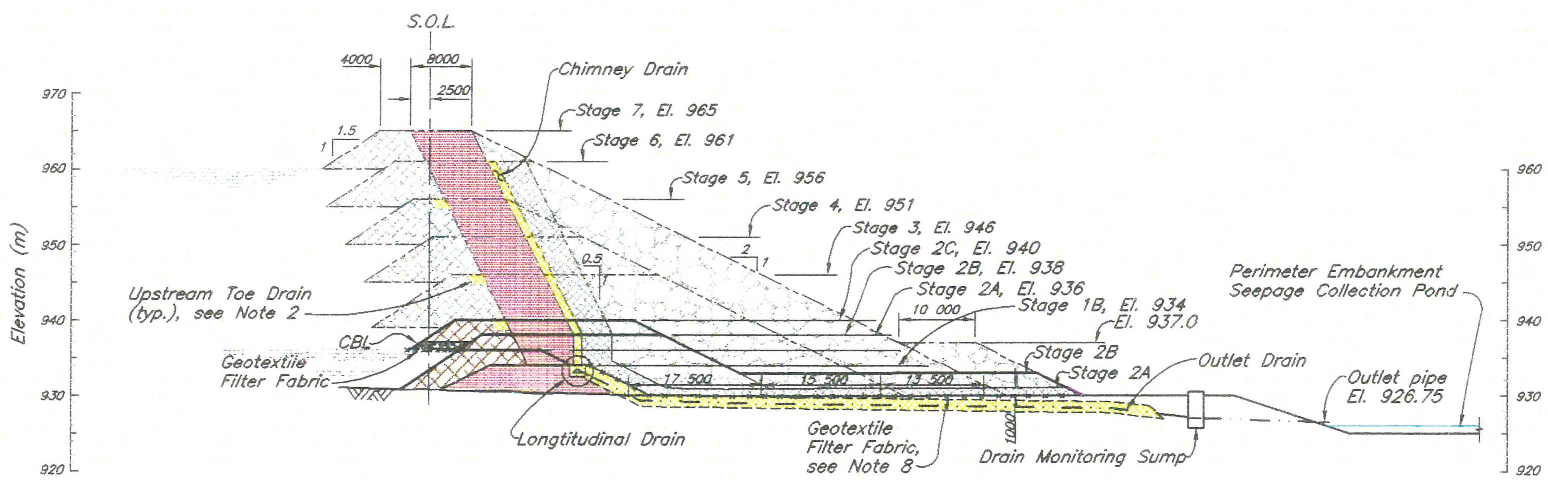
MOUNT POLLEY PROJECT

TAILINGS STORAGE FACILITY
FINAL ARRANGEMENT

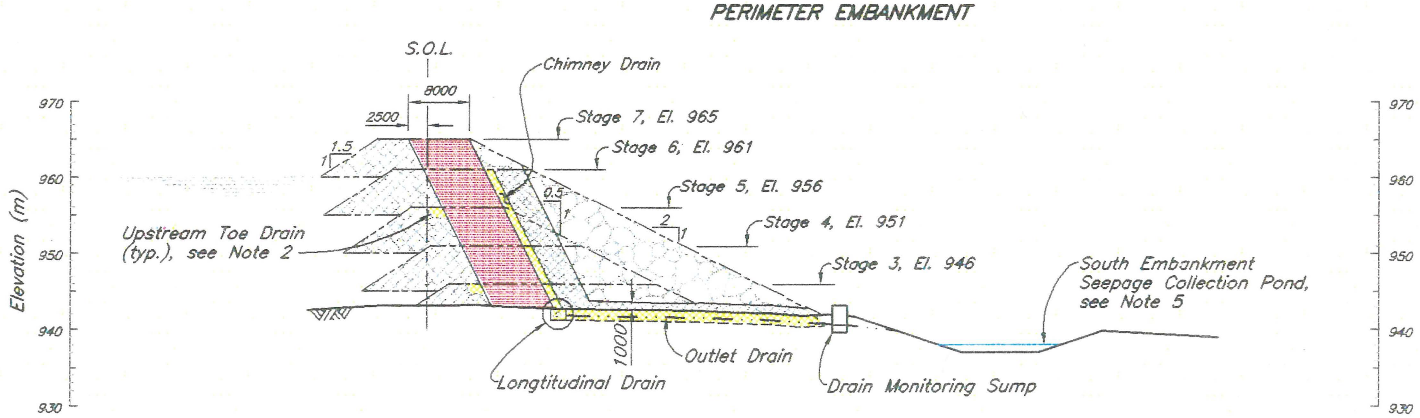
DATE DEC. 1, 1997 | SCALE AS SHOWN | DRG. NO. 10162-9-200 | REV. 0



SECTION $\frac{1}{101}$
MAIN EMBANKMENT



SECTION $\frac{2}{200}$
PERIMETER EMBANKMENT



SECTION $\frac{3}{200}$
SOUTH EMBANKMENT

ZONE	LOCATION	MATERIAL TYPE	PLACEMENT AND COMPACTION REQUIREMENTS
S	Core Zone	Glacial till	Placed, moisture conditioned and spread in maximum 300 mm thick layers (after compaction). Vibratory compaction to 98% of Standard Proctor maximum dry density or as approved by the Engineer.
B	Fill Zone	Glacial till, glaciolacustrine or granular material	Placed, moisture conditioned and spread in maximum 1000 mm thick layers (after compaction). Vibratory compaction to 92% of Standard Proctor maximum dry density or as approved by the Engineer.
T	Transition Zone	Mine Rock	Placed and spread in maximum 600 mm thick layers. Compaction as directed by the Engineer.
C	Shell Zone	Mine Rock	Placed and spread in maximum 1000 mm thick lifts. Four passes with a specified vibratory roller.
F	Chimney Drain	Filter sand	Placed and spread in maximum 600 mm thick lifts. Compaction as directed by the Engineer.
F	Longitudinal/ Outlet Drain	Filter Sand	Placed and spread carefully around filter fabric/drain gravel. Compaction as directed by the Engineer.
G	Foundation/ Longitudinal/ Outlet Drain	Drain Gravel	Placed and spread carefully around seepage collection pipes. Compaction as directed by the Engineer.
–	Basin Liner	Glacial till, glaciolacustrine or granular material	Placed and spread in maximum 150 mm thick lifts. Compaction as directed by the Engineer.
CBL	Coarse Bearing Layer	Random Rockfill	End dumped and spread as required for trafficability and fill placement.
FDF	Upstream Shell Zone	Free draining Random Fill	Placement and compaction requirements to be determined.

NOTES

1. Pond elevations estimated from Filling Schedule and Staged Construction Curve and include provision for 2.5 million cubic metres of reclaim water.
2. Stage 2 Upstream Toe Drains to be designed and installed during Stage 3. Future Upstream Toe Drains to be added as required.
3. Dashed lines imply preliminary design. Ongoing design and crest elevations to be modified as required based on filling records and monitoring data.
4. Chimney Drain extension requirements to be reviewed for each raise. Chimney Drain to have a minimum continuous width of 1000 mm.
5. South Embankment Seepage Collection Pond and Drain Monitoring Sump to be constructed during Stage 3.
6. Coarse Bearing Layer required on tailings. To be added on ground as required to provide a firm bearing layer for fill placement.
7. All dimensions in millimetres with elevations in metres, unless noted otherwise.
8. Extent of Geotextile Filter Fabric on foundation to be determined in the field.

NOT FOR CONSTRUCTION

Scale 10 5 0 10 20 30 40 50 m

CAD FILE:10162\9\D20\D20 1:500 Plot 1=0.5 DEC 1, 1997

KNIGHT PIESOLD LIMITED
CONSULTING ENGINEERS – VANCOUVER, B.C.

PROFESSIONAL ENGINEER PROVINCE OF BRITISH COLUMBIA K. D. K. EMBREE

DESIGNED	KJB/KDE
DRAWN	DHS
CHECKED	ABW
APPROVED	

MOUNT POLLEY MINING CORPORATION

MOUNT POLLEY PROJECT

TAILINGS STORAGE FACILITY
FINAL TAILINGS EMBANKMENT
SECTIONS

DATE	SCALE	DRG. NO.	REV.
DEC. 1, 1997	AS SHOWN	10162-9-201	0

DRG. NO.	DESCRIPTION
200	TSF – FINAL ARRANGEMENT

REFERENCE DRAWINGS

REV.	DATE	DESCRIPTION	APPROVED

REVISIONS

REV.	DATE	DESCRIPTION	APPROVED
0	DEC 1/97	ISSUED FOR DESIGN REPORT	

REVISIONS

APPENDIX A

SELECTED PAPERS
ON MODIFIED CENTRELINE
TAILINGS EMBANKMENT CONSTRUCTION

Modified Centreline Construction of Tailings Embankments

J.P. Haile & K.J. Brouwer

Knight Piésold Ltd.
Consulting Engineers
Suite 1400 - 750 West Pender Street
Vancouver, B.C., Canada
V6C 2T8

Abstract: A new approach to compacted fill embankments for tailings storage facilities has been developed which is seismically stable and minimizes the fill requirements, and hence costs, for embankment construction. Modified centreline construction is similar to conventional centreline construction but with the contact between the compacted fill and the tailings sloping slightly upstream. It is, however, different from upstream construction as the stability of the embankment relies on the relatively wide thickness of compacted fill at any elevation, is independent of the tailings strength and is inherently stable even with complete liquefaction of the tailings mass. The design approach significantly reduces the quantity of fill required for on-going raises compared to conventional centreline and downstream construction as on-going construction on the downstream face is not required. This also allows for reclamation of the downstream embankment face during operations. It has been successfully implemented at the Montana Tunnels Mine in Montana, where a final embankment height of over 100 metres is planned, and forms the basis for the tailings embankment design for new projects in Alaska and British Columbia, Canada. This paper describes the principal features of this construction technique, analytical procedures and case histories.

Key Words: mine tailings storage, embankment construction, waste reclamation, seismic stability

1. Introduction

The design of tailings facility embankments in seismically active areas, or for fine-grained, low strength tailings, has historically utilized conventional earth or rockfill embankments constructed as a full embankment section similar to a water retaining dam. No reliance is placed on the strength of the tailings and the embankment section is stable under all conditions of static and seismic loading. In some instances centreline construction using either the coarse fraction of the tailings or compacted fill is used to achieve the same design objectives.

Both of these approaches require a relatively large volume of fill material for the embankment section. With staged construction the volume of fill required for each incremental raise of the embankment crest gets larger as the height of the embankment increases, and requires construction on the downstream face of the embankment over the full height. This has the added disadvantage of not allowing reclamation of the downstream face to be carried out during mining operations. Staged construction of downstream and centreline embankments is shown schematically in Figure 1.

In most instances where these embankment cross-sections are required, upstream construction on the tailings mass itself would not be an appropriate alternative, either because of poor consolidation and/or drainage conditions within the tailings, potential liquefaction and low strength of the tailings. Upstream tailings embankments can only be constructed with fine grained tailings and in seismically active areas if proper measures are taken to ensure full consolidation and drainage of the tailings [1].

The modified centreline embankment, however, offers a cost effective alternative to downstream or centreline construction in areas of high seismic risk and for tailings with little or no strength. This paper describes the principal features of this construction technique, along with analytical procedures and case histories.

3rd International Conference on Environmental Issues and Waste Management in Energy and Mineral Production, August, 1994. Perth, Australia

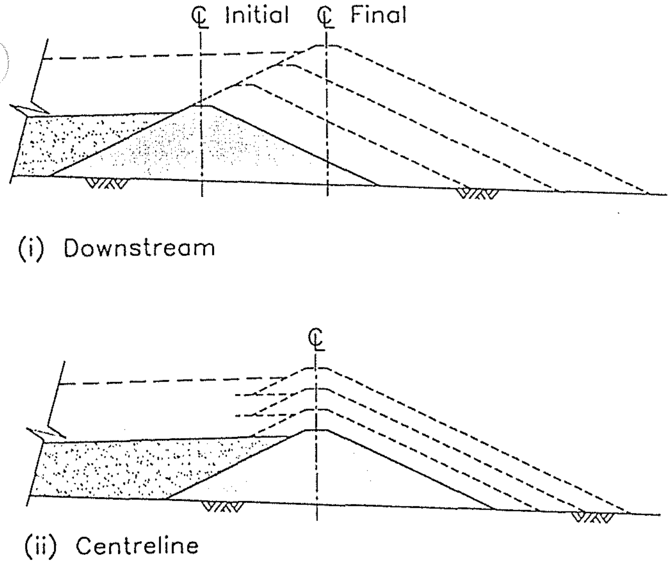


Figure 1 Downstream and centreline embankments

2. Design Concept

The modified centreline cross-section is similar to a centreline cross-section but with the contact between the embankment fill and the tailings sloping slightly upstream. It results in the minimum volume of embankment fill for an embankment that is stable under all conditions of static and seismic loading. Furthermore, on-going construction on the downstream face is not required and reclamation can be carried out during operations. A schematic cross-section through a modified centreline embankment is shown on Figure 2.

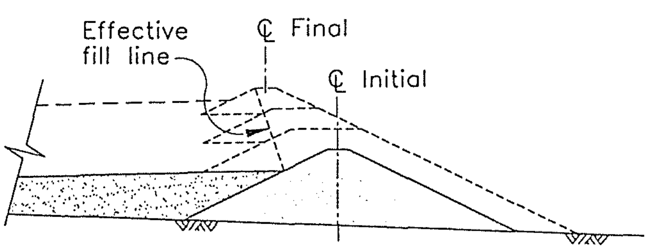


Figure 2 Modified centreline embankment

The modified centreline embankment achieves its stability from the relatively wide thickness of compacted fill at any elevation, and is independent of the strength of the tailings. The embankment is designed to be stable even if the tailings are fully liquefied and imposing both full fluid pressure and hydrodynamic loading on the upstream contact. The upstream contact remains stable even if the tailings are fully liquefied, when they would act as a dense fluid. The analogy is that of a slurry wall, where a dense fluid such as bentonite mud can be used to support very deep excavations.

The construction technique does require some placing of fill on the tailings beach, and hence deposition of at least a portion of the tailings stream from the embankment face is required. Ideally, the beach should be at least strong enough to support the first lift of fill. This can be achieved on very soft tailings with the assistance of a geotextile separation layer. If the beach cannot support the first lift, then the tailings can be displaced using dumped rockfill.

Modified centreline tailings embankments can be designed as either water retaining structures or fully drained embankments. When designed to be water retaining, which is obviously a more severe loading condition than if fully drained, the water retaining zone, or core, should be located as far upstream as possible, in order to provide the necessary width of drained granular material downstream of the core for stability.

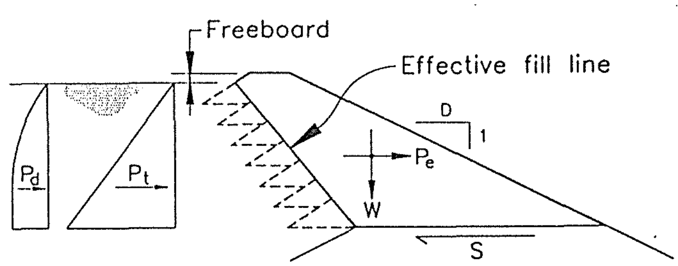
3. Stability and Deformation Analyses

Stability analyses of a modified centreline embankment can be considered under three separate headings:

(i) Downstream stability,
(ii) Upstream stability,
(iii) Deformation Analyses.

Downstream Stability

Downstream stability can be analyzed initially as pseudo-static loading on the modified centreline portion of embankment only, i.e. that portion of the embankment above the full section. The forces acting on this section of the embankment are shown schematically on Figure 3.



SUMMARY OF LOADING CONDITIONS	
SYMBOL	DESCRIPTION
P_t	LIQUEFIED TAILINGS
P_d	HYDRODYNAMIC THRUST
P_e	EARTHQUAKE LOADING ON EMBANKMENT
W	WEIGHT OF EMBANKMENT
S	SHEAR RESISTANCE

Figure 3 Downstream pseudo-static loading for stability analyses

In designing a modified centreline embankment the main variables to be considered in the geometry of the section are the height of the modified centreline portion, the downstream slope and the upstream contact slope between the fill and tailings.

The downstream slope will generally be dictated by the construction materials available, but the height of the modified centreline portion and the upstream contact slope will be a function of the seismicity of the site. The height of the modified centreline portion can be considered in terms of Critical Height (H_c), which is defined as that height at which the pseudo-static factor of safety is equal to 1.0 under a given acceleration. The relationships between H_c, acceleration and the upstream contact slope are shown on Figure 4, for a given set of assumptions and the loading conditions shown on Figure 3.

The concepts presented in Figure 4 can be used for an initial determination of H_c. However, it is important to realize that this critical height is not a limiting height and only defines the height at which the critical acceleration for the embankment section k_c, is equal to the design acceleration for the site, a_{max}. Higher embankments, with a value of k_c less than a_{max}, can be safely designed but will be subject to some deformation during the earthquake shaking.

The modified centreline embankment must also incorporate suitable provisions for seepage control and for piping prevention. Since the embankment fill extends slightly over more compressible tailings materials, consolidation settlement may result in cracking of the embankment core zone. Therefore, the embankment design must incorporate suitable filter criteria and drainage provisions. In general, the tailings mass forms an ideal crack stopping filter medium so that piping failure is not a major consideration. Embankment stability can also be enhanced by incorporating drainage features such as chimney drains to reduce pore pressures within the structural zone of the embankment.

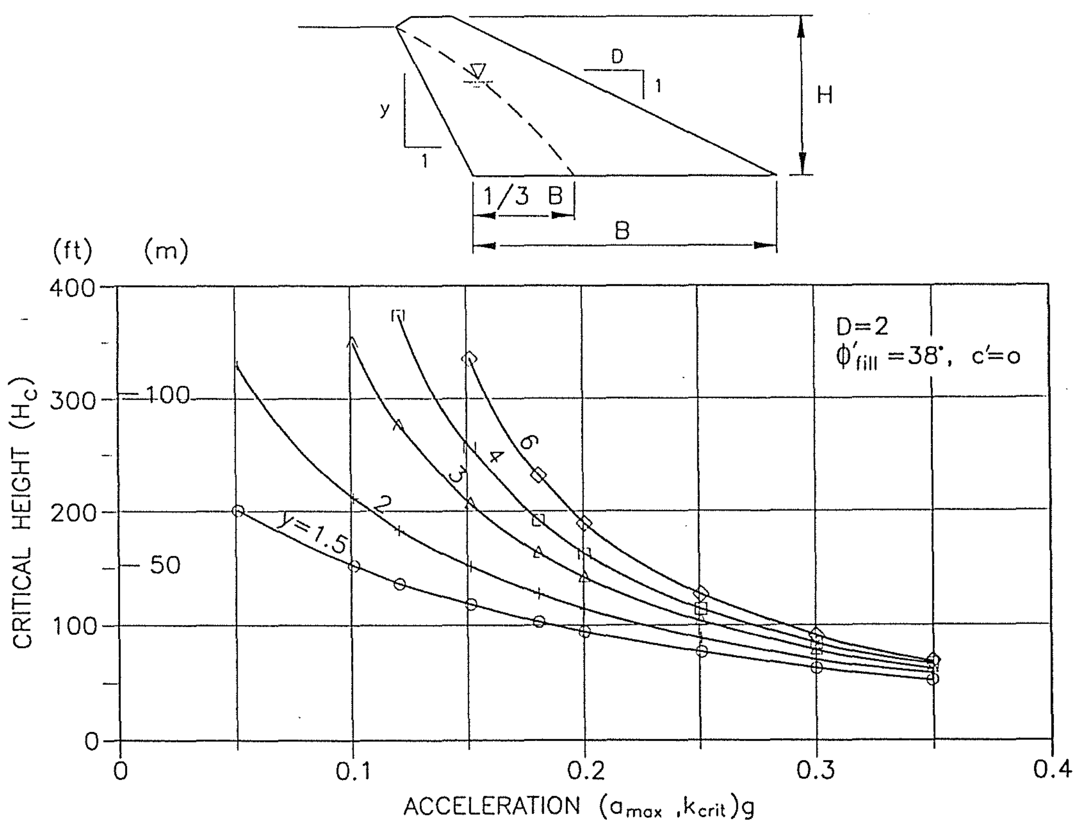
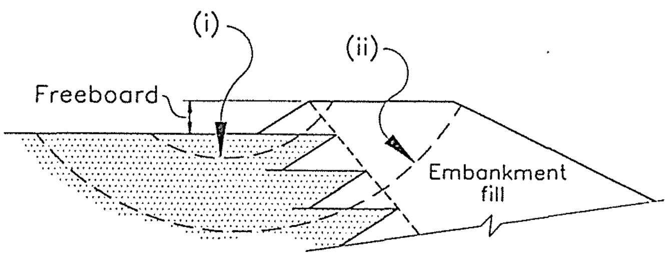


Figure 4 Relationship between critical height and acceleration

Upstream Stability

Upstream stability needs to consider two critical loading conditions: short-term loading on the tailings beach during embankment crest raising; and post-seismic upstream stability when the tailings would have only post liquefaction residual strength. In the first case, the principal concern is safety, whereas for the second case the principal concern is for failures causing loss of freeboard. Both cases need to be analyzed to determine the maximum allowable freeboard, which can then be related to flood storage requirements (Figure 5). In both analyses the appropriate strength characteristics of the tailings need to be known, in addition to those of the embankment fill materials.



(i) Short term construction.
Tailings strength, $c_u/p' \simeq 0.2 - 0.3$

(ii) Post earthquake loss of freeboard.
Tailings residual strength, $c_u/p' \simeq 0.1 - 0.2$

Figure 5 Upstream stability loading cases to determine maximum freeboard

Deformation Analyses

Deformation analyses can be carried out using the simplified procedures of Newmark [2] and Makdisi and Seed[3]. The analyses compare the critical acceleration k_c, with the site design acceleration, a_{max}, and compute displacements using empirical relationships and case history data from conventional water retaining dams. Modification of the amplitude of the ground acceleration as it propagates up through the embankment can be determined using the SHAKE [4] program. Similarly, the value of k_c at any elevation in the embankment can be determined from standard stability analysis programs. In order to compensate for the geometry of the modified centreline embankment and uncertainties in the mode of deformation, the largest value of acceleration determined from SHAKE can be used together with the smallest value of k_c to compute potential deformations.

A pseudo-dynamic finite element displacement analysis has been developed by Byrne *et al* [5,6]. This analysis can be used to determine deformations under both upstream and downstream earthquake loading, and to define the location and magnitude of the largest deformations. In general it predicts deformations somewhat larger than those from the simplified Newmark analyses using the extreme values.

The stability analyses discussed above have only considered the more extreme loading conditions. In all embankment designs, all loading cases must be analyzed using relevant material parameters to ensure that acceptable factors of safety exist for each loading case.

4. Case Histories

Montana Tunnels Mine, Montana, USA.

The Montana Tunnels Mine is an open pit operation which involves processing gold, lead, zinc and silver ore at a rate of approximately 13,700 tonnes per day. The mine has been operating since 1987. Total mineable reserves from inception of mining have recently been expanded from 38 to 62 million tonnes.

The original tailings embankment was designed using a downstream method of construction for the annual staged expansions[7]. The compacted rockfill embankment layout was modified in 1990, when on-going expansions were constructed using the modified centreline method in order to minimize fill quantities and preserve a downstream process water pond[8]. The modified centreline section was changed again in 1993 to enable expansion of the tailings impoundment to provide storage for the increased ore reserves. The embankment is presently designed to reach a maximum ultimate height of 105 metres. A schematic cross-section through the embankment is shown on Figure 6.

The redesign of the modified centreline embankment in 1993 included an extensive site investigation program which incorporated drilling, sampling, standard penetration testing, seismic piezocone testwork and installation of vibrating wire piezometers. A line of wick drains was installed along the tailings beach to enhance drainage into the free-draining embankment. A second wick drain program[9] was also completed within the tailings impoundment to dissipate excess pore pressures, accelerate consolidation and enhance seismic stability.

The stability assessment for the embankment included conventional limit equilibrium analyses for static, pseudo-static and post-earthquake conditions. Additional pseudo-dynamic finite element analyses, using the procedure described by Byrne *et al*[5], were also used to evaluate potential embankment deformations for a maximum credible earthquake with a peak horizontal ground acceleration of 0.22 g. The analysis includes both the inertia forces from the earthquake as well as the softening effect of the soil during cyclic loading. The fifth modified centreline embankment raise will be completed at the Montana Tunnels Mine during 1994, with annual expansions planned through 2001.

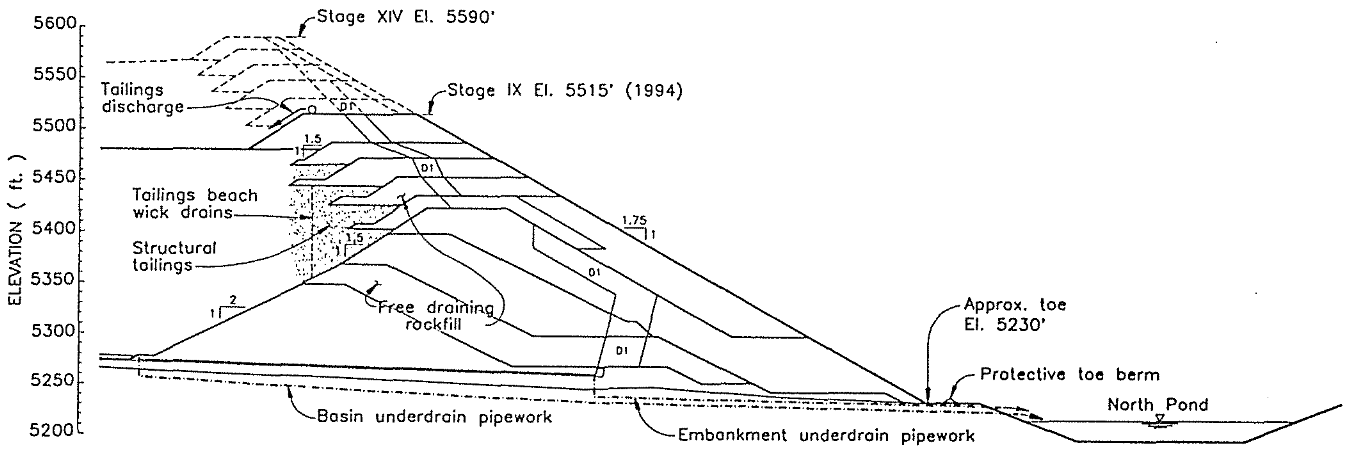


Figure 6 Typical section through Montana Tunnels embankment

Kensington Venture, Alaska, USA

The Kensington Project is a proposed underground gold mine located 40 miles north of Juneau, Alaska, on the east side of the Lynn Canal. The mine will require construction of a 89 metre high dam to contain the tailings from the mining operations. The dam is to be constructed in stages using compacted earthfill and rockfill and a modified centreline arrangement. The project is located in an area of high potential seismicity and earthquake-induced liquefaction of the tailings is possible. The stability of the top portion of the dam and the potential displacements resulting from earthquake loading are therefore of extreme importance. A cross-section through the proposed final embankment is shown on Figure 7.

Conventional limit equilibrium and Newmark analyses, including hydrodynamic loading from the liquefied tailings, indicate that the embankment is stable and deformations would be very small. Deformation analyses were also carried out using the pseudo-dynamic finite element procedure developed by Byrne *et al* [5]. The analysis allows both the inertia forces from the earthquake as well as the softening effect of the liquefied soil to be considered.

Peak horizontal ground accelerations ranging from 0.2 g to 0.6 g were considered with corresponding peak ground velocities of 0.2 and 0.6 metre/second. The predicted peak displacements of the crest of the dam are 0.48 metre horizontal and 0.09 metre vertical. The maximum movement of the dam predicted from the Newmark analysis using the same soil strengths was 0.14 metres.

The Kensington Venture is currently in the final stages of permitting.

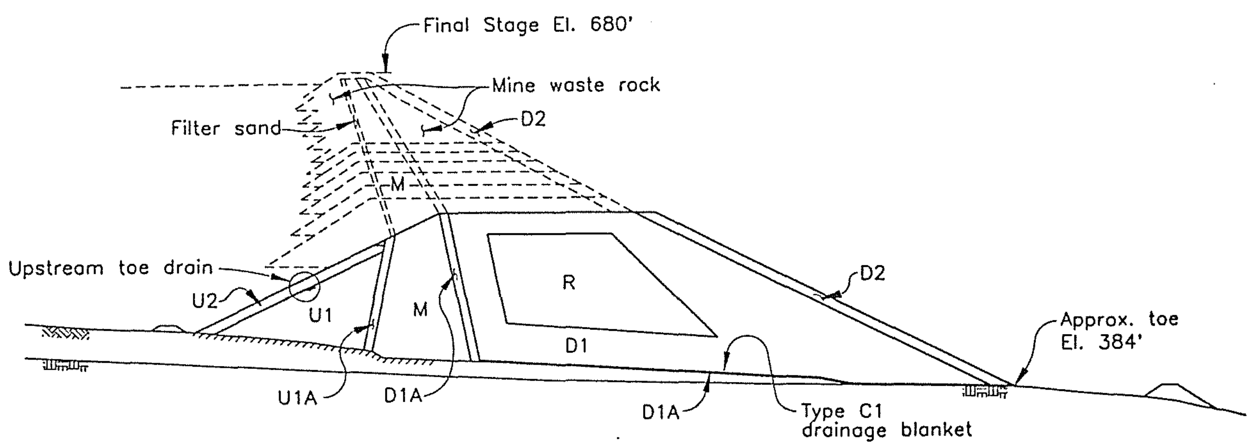


Figure 7 Typical section through Kensington embankment

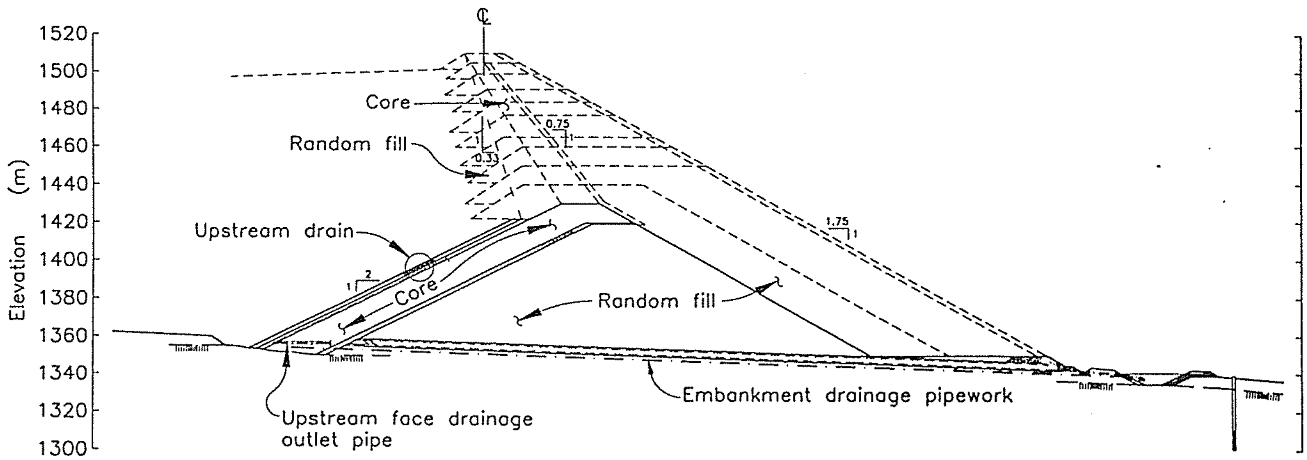


Figure 8 Typical section through Kemess South embankment

Kemess South Project, B.C., Canada

The Kemess South Project, situated in north central British Columbia, is presently in the final stages of permitting and is scheduled for development in 1995. A total reserve of 220 million tonnes of gold and copper ore will be processed at a rate of 40,000 tonnes per day. The project will include the staged construction of a compacted earthfill tailings embankment using the modified centreline technique to an ultimate height of 150 metres. A schematic embankment section is shown on Figure 8.

The project site is situated in an area of low seismicity and conventional pseudo-static limit equilibrium analyses indicate an adequate factor of safety against embankment deformation. The modified centreline embankment section was selected in order to minimize the quantity of fill required for staged expansions, and thus reduce on-going capital expenditures. Also, the downstream face of the embankment will be incrementally revegetated to minimize environmental impacts during operations and to reduce post-closure reclamation requirements.

5. Conclusions

The modified centreline embankment provides the least cost compacted fill embankment for tailings storage facilities in areas of high seismicity and for low strength tailings. These embankments are intrinsically stable under earthquake loading even with the tailings fully liquified. They can be constructed in stages using standard mining equipment and overburden materials from on-going mining operations. After the initial one or two stages no further construction is required on the downstream face, which allows for on-going reclamation during operations.

The modified centreline design has been successfully implemented at the Montana Tunnels Mine in Montana, where a final embankment height of over 100 metres is planned. A detailed design has been developed for the Kensington Venture in Alaska and is in the final stages of the review process. Designs for new projects in B.C. and elsewhere in North America are currently at the development stage.

References

1. Haile, J.P. and Kerr, T.F., 1989. "Design and Operation of the Myra Falls Tailings Disposal Facility." Vancouver Geotechnical Society's 4th Annual Symposium on Geotechnical Aspects of Tailings Disposal and Acid Mine Drainage. May 26, 1989, Vancouver, B.C.

2. Newmark, N.M., 1965. "Effects of Earthquakes on Dams and Embankments." Geotechnique 15: 139-160.

3. Makdisi, F.I. and Seed, H.B., 1977. "A Simplified Procedure for Estimating Earthquake Induced Deformations in Dams and Embankments." Report No. UCB-EERC-77/19, College of Engineering, University of California, Berkeley.

4. Schnabel, P.B., Lysmer, J. and Seed, H.B., 1972. "SHAKE - A Computer Program for Earthquake Response Analysis of Horizontally Layered Sites." Report No. EERC 72-12, University of California, Berkeley, California.

5. Byrne, P.M., Jitno, H, Haile, J, 1992. "A Procedure for Predicting the Seismic Response of Tailings Impoundments." First Canadian Symposium on Geotechnique and Natural Hazards. May 6-9, 1992, Vancouver, B.C.

6. Byrne, P.M, Jitno, H., Anderson, D.L, Haile, J., 1994. "A Procedure for Predicting Seismic Displacements of Earth Dams." Report No. XIII CSMFE, 1994, New Delhi, India/XIII CIMSTF, 1994, New Delhi.

7. Haile, J.P. and Brouwer, K.J., 1987. Design and Construction of the Montana Tunnels Tailings Disposal Facility: 89th Annual General Meeting of the Canadian Institute of Mining. May 3-7/87, Toronto, Ontario.

8. Brouwer, K.J., Weitz, T.J. and Haile, J.P., 1992. "Design and Operation of the Montana Tunnels Tailings Disposal Facility." Second International Conference on Environmental Issues & Management of Waste in Energy & Mineral Production. September 1 - 4, 1992, Calgary, Alberta.

9. Brouwer, K.J., Brown, B.S. and Stewart, D, 1994. "Tailings Consolidation Using Wick Drains." 3rd International Conference on Environmental Issues and Waste Management in Energy and Mineral Production, 1994. Perth, Australia.

DESIGN AND SEISMIC DISPLACEMENT ANALYSES FOR THE KENSINGTON TAILINGS DAM, ALASKA

Jeremy P. Haile, Ken J. Brouwer, Ian B. Manning
Knight Piésold Ltd.
Vancouver, B.C.
Canada

and

Dr. Peter M. Byrne
University of B.C.
Vancouver, B.C.
Canada

ABSTRACT

The Kensington Project is a proposed new underground gold mine located near Juneau, Alaska. A 90 m high dam has been designed to contain the tailings from the mill process. The project is located in an area of high potential seismicity with a design bedrock acceleration of 0.6 g, and earthquake induced liquefaction of the tailings is probable. The design for the tailings dam uses compacted earthfill and rockfill and a modified centreline arrangement. The design of the dam is presented together with an analysis of earthquake induced displacements using the computer code FLAC. The results of these analyses are compared with previous displacement analyses using the pseudo-dynamic finite element procedure developed by Byrne et al (1992, 1994).

INTRODUCTION

The Kensington Project is a proposed underground gold mine located 40 miles north of Juneau, Alaska, on the east side of the Lynn Canal. The mine will process the ore at a rate of 4000 tons per day and requires on-land storage for up to 30 million tons of tailings. One alternative for on-land storage is behind a 90 m high dam designed to contain the tailings and for water management at the site. This dam has been designed with a modified centreline earth rockfill embankment which will be constructed in stages. The embankment cross-section differs from conventional centreline construction in that the upstream contact between the compacted fill and the tailings is located upstream of the starter dam centreline for each additional construction stage. Examples of other applications of the modified centreline construction method are presented.

The Kensington project is located in an area of high potential seismicity and earthquake induced liquefaction of the tailings is probable. Several cases of earthquake induced failure of tailings dams built using the tailings sands and upstream construction have been reported in the literature, e.g. two Chilean tailings dams (Dobry and Alvarez (1967)), and the Mochikoshi tailings dam in Japan (Ishihara (1984)). Therefore, key considerations in the design of the Kensington dam are the seismic stability and the potential displacements resulting from earthquake loading.

The stability of the Kensington tailings dam has been evaluated in several ways. Conventional limit equilibrium and Newmark analyses including hydrodynamic loading from the liquefied tailings were originally carried out and indicated that the embankment is stable and deformations would be very small for a design bedrock acceleration of 0.6 g. Subsequent more detailed deformation analyses using a pseudo-dynamic finite element procedure were carried out and are reported by Byrne et al (1992, 1994). This procedure allows inertia forces from the earthquake as well as the softening effect of liquefied tailings to be taken into account.

Further deformation analyses have been carried out using the finite difference computer program FLAC and a simple total stress approach. The procedures involved in these analyses, and the results of the analyses are presented in this paper.

MODIFIED CENTRELINE CONSTRUCTION

Modified centreline construction is a new approach to compacted fill embankments for tailings storage facilities which is seismically stable and minimizes the fill requirements, and hence costs, for embankment construction. Modified centreline construction is similar to conventional centreline construction but with the contact between the compacted fill and the tailings sloping slightly upstream. It is, however, different from upstream construction as the stability of the embankment relies on the relatively wide thickness of compacted fill at any elevation, is independent of the tailings strength and is inherently stable even with complete liquefaction of the tailings mass. The principal features of this construction technique, analytical procedures and case histories have been presented by Haile and Brouwer (1994). This design concept has been used for the Montana Tunnels Mine tailings embankment in Montana, USA, and is currently being implemented for the Alumbrera Project in Argentina and the Kemess South Project in B.C., Canada. Overview case histories for these projects are presented below.

Montana Tunnel Mine, Montana, USA.

The Montana Tunnels Mine is an open pit operation which involves processing gold, lead, zinc and silver ore at a rate of approximately 14,500 tonnes per day. The mine has been operating since 1987. Total mineable reserves from inception of mining have recently been expanded from 38 to 62 million tonnes.

The original tailings embankment was designed using a downstream method of construction for the annual staged expansions as described by Haile and Brouwer (1987). The compacted rockfill embankment layout was modified in 1990, when on-going expansions were constructed using the modified centreline method in order to minimize fill quantities and preserve a downstream process water pond. These modifications are described by Brouwer et al (1992). The modified centreline section was changed again in 1993 to enable expansion of the tailings impoundment to provide storage for the increased ore reserves. The embankment is presently designed to reach a maximum ultimate height of 105 metres. A schematic cross-section through the embankment is shown in Figure 1.

The redesign of the modified centreline embankment in 1993 included an extensive site investigation program which incorporated drilling, sampling, standard penetration testing, seismic piezocone testwork and installation of vibrating wire piezometers. A line of wick drains was installed along the tailings beach to enhance drainage into the free-draining embankment. A second wick drain program was also completed within the tailings impoundment to dissipate excess pore pressures, accelerate consolidation and enhance seismic stability. The wick drain programs are described by Brouwer et al (1994).

The stability assessment for the embankment included conventional limit equilibrium analyses for static, pseudo-static and post-earthquake conditions. Additional pseudo-dynamic finite element analyses, using the procedure described by Byrne (1991), were also used to evaluate potential embankment deformations for a maximum credible earthquake with a peak horizontal ground acceleration of 0.22 g. The analysis includes both the inertia forces from the earthquake as well as the softening effect of the soil during cyclic loading.

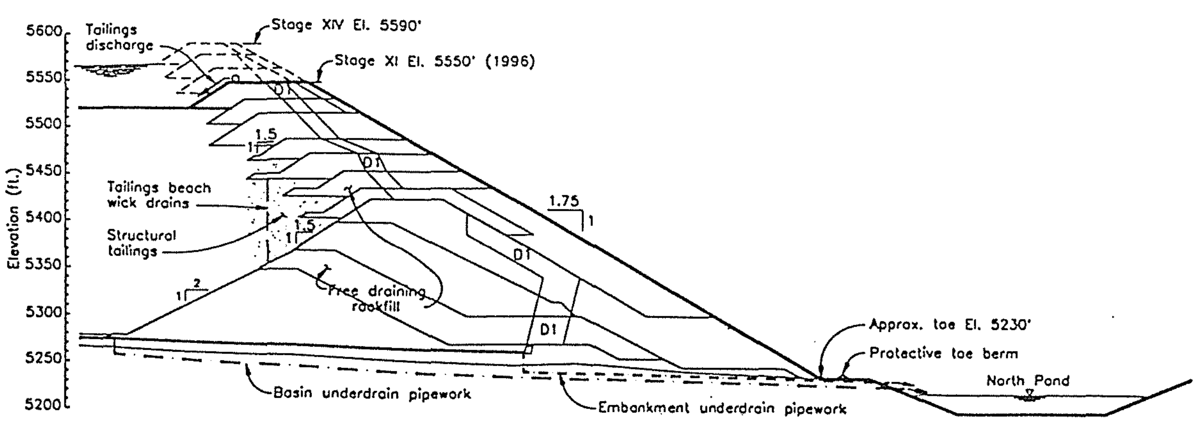


Figure 1: Typical section through Montana Tunnels embankment

Alumbrera Project, Argentina.

The Alumbrera project is a large open pit copper-gold mine currently being developed in Catamarca Province, Argentina. The mine will process ore at an initial rate of 80,000 tonnes per day, increasing to 120,000 tonnes per day after five years. The tailings facility has been designed for a total storage capacity of 1 billion tonnes. The project is located in an area of moderate historical seismicity. The presence of a significant linear feature running through the footprint of the embankment, however, has been used as the basis for a maximum design acceleration at the site of 0.58 g.

The design of the tailings facility incorporates a free draining, modified centreline embankment with an ultimate height of 165 m. The 45 m high starter embankment, currently under construction, uses local alluvial materials and incorporates an upstream drainage system. On-going raises will be constructed using waste rock from the open pit for the structural shell zones and alluvial materials for the transition zones between the waste rock and tailings. A typical section through the embankment is shown on Figure 2.

The embankment has been designed assuming full liquefaction of the tailings mass under the design earthquake. However, due to the coarse grind of the tailings, the drainage system incorporated in the embankment and the significant depth of the tailings, it is considered unlikely that liquefaction of the tailings would in fact occur.

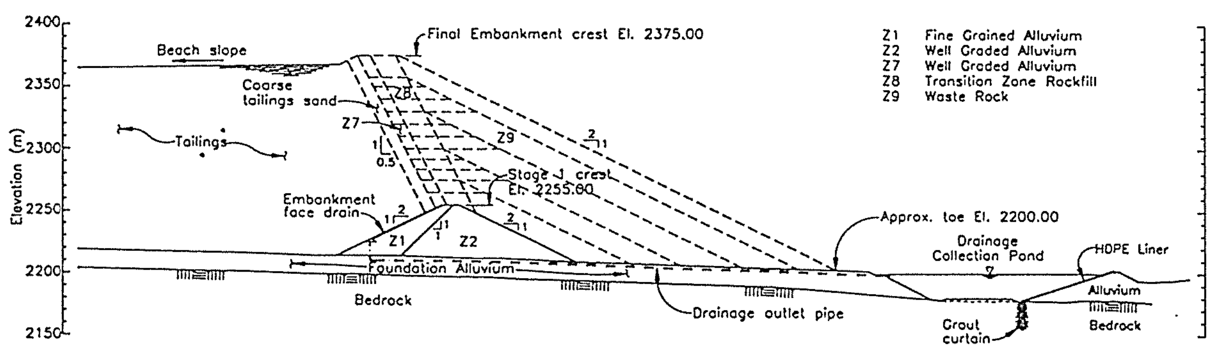


Figure 2: Typical section through Alumbrera embankment

Kemess South Project, B.C., Canada.

The Kemess South Project, situated in north central British Columbia is currently under construction. A total reserve of 200 million tonnes of gold and copper ore will be processed at a rate of 45,000 tonnes per day. The project includes the staged construction of a compacted earthfill tailings embankment using the modified centreline technique to an ultimate height of 150 metres. A schematic embankment section is shown on Figure 3.

The project site is situated in an area of low seismicity and conventional pseudo-static limit equilibrium analyses indicate an adequate factor of safety against embankment deformation for a design acceleration of 0.19 g. The modified centreline embankment section was selected in order to minimize the quantity of fill required for staged expansions, and thus reduce on-going capital expenditures. Also, the downstream face of the embankment will be incrementally revegetated to minimize environmental impacts during operations and to reduce post-closure reclamation requirements.

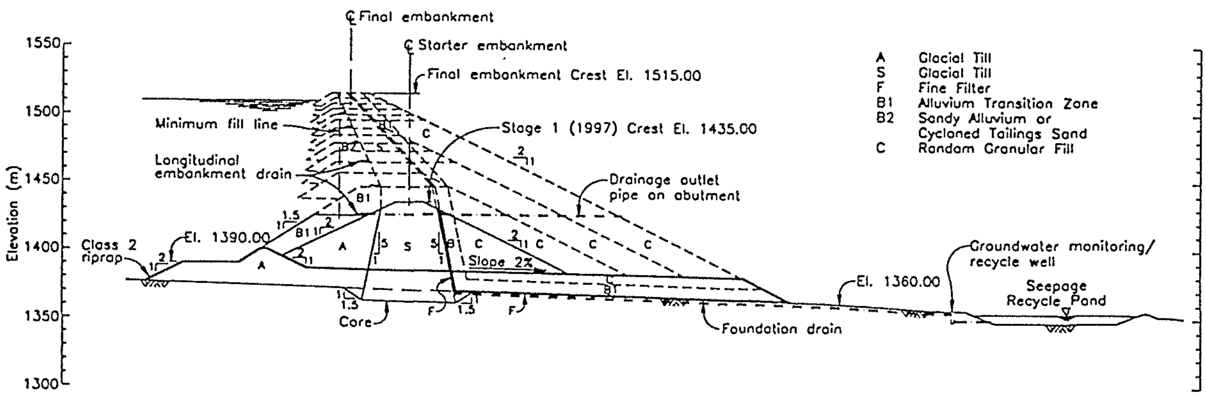


Figure 3: Typical section through Kemess South embankment

KENSINGTON TAILINGS DAM

The proposed Kensington Tailings Dam is designed as a modified centreline embankment using compacted earthfill and mine waste rock, and constructed in stages to an ultimate height of 90 m. A typical section through the embankment is shown on Figure 4.

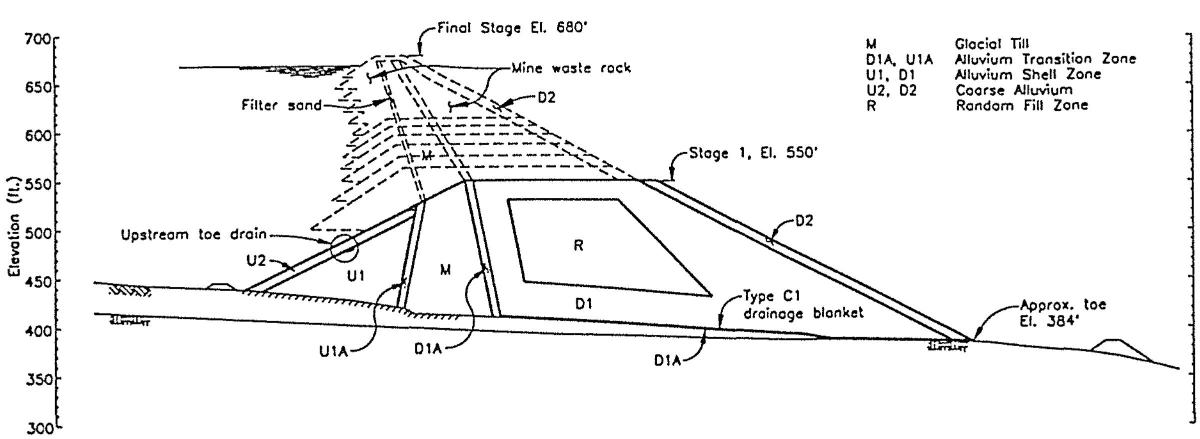


Figure 4: Typical Section through Kensington embankment

The initial stage of the embankment will be constructed using local borrow materials from within the tailings basin. These consist of a well graded glacial till that will comprise the core zone (M), and glaciofluvial alluvial deposits that will be used for the upstream and downstream shell zones (U1, U1A, U2, D1, D1A, D2). On-going construction raises will utilize underground mine development waste for the main structural zones, as well as a processed filter sand, glacial till (M) and alluvium (D1A, D2).

Foundation conditions at the site consists of dense glacial till deposits with a thickness varying from 0 m on the valley bottom to over 50 m on the right abutment. The underlying bedrock is a fresh, fractured phylite.

The site is located in an area of moderate historical seismic activity. However, the Lynn Canal forms a linear extrapolation of the Chatham Strait fault to the south and the Denali fault to the north. Both of these faults have been ascribed an MCE of Magnitude 7.0, and hence the Maximum Design Earthquake for the site has been based on a Magnitude 7.0 event centred in the Lynn Canal at a horizontal distance of 3 miles from the site. This results in a maximum design acceleration of 0.6 g and maximum velocity of 0.6 m/s for the site, which have been used as the basis for the liquefaction assessment of the tailings and for the embankment displacement analyses.

LIQUEFACTION

Cyclic shear loading of granular soils causes slip at grain contacts that results in compaction under drained conditions. If drainage is prevented from occurring, then grain slip still occurs but results in a rise in porewater pressure and a loss in strength and stiffness in place of compaction. If porewater pressure rises to equal the total stress, the effective stress drops to zero, and a complete loss in stiffness occurs. The soil temporarily act as a liquid, and large strains and deformations will occur in the presence of a driving stress. However, as the liquefied soil strains, porewater pressures will drop and the soil will strain harden and recover some strength and stiffness, the amount depending on its density or penetration resistance. Typical pre- and post-liquefaction curves are shown in Figure 5.

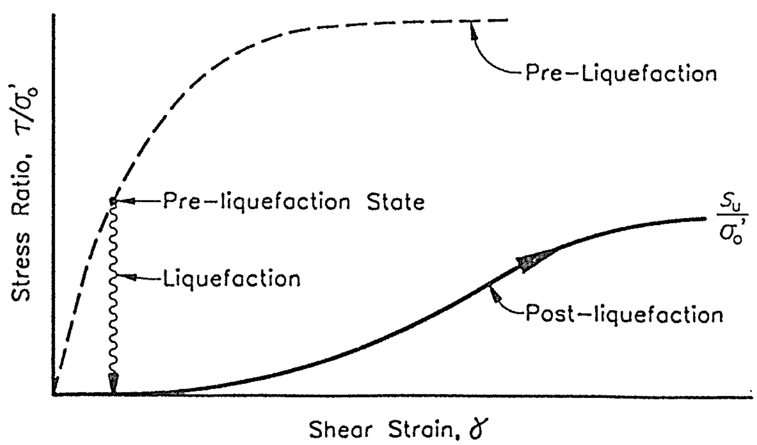


Figure 5: Stress - Strain Response of Soil

Test data indicate that post-liquefaction strength, termed the residual strength, s_u, depends on the effective confining stress σ'_o prior to liquefaction and the density, i.e. $s_u = \alpha\sigma'_o$. Typical α values as a function of normalized penetration resistance (Byrne, 1996), are shown in column 2 of Table 1.

The shear strain required to mobilize the residual strength, γ_f, may be very large and depend on relative density and the degree of liquefaction, which can be expressed in terms of the factor of safety against triggering, F_{TRIG}. Estimates of γ_f from Byrne (1996), are also shown in Table 1. The data shown in Table 1 can be used to prescribe a simple bilinear post-liquefaction stress-strain curve as shown in Figure 6.

$(N_1)_{60}$	s_u/σ'_o (α)	γ_f (%)	
		$F_{TRIG} \approx 1.0$	$F_{TRIG} \approx 0.5$
0 - 4	0.05 - 0.10	25 - 50	> 100
4 - 10	0.10 - 0.20	10 - 25	30 - >100
10 - 15	0.15 - 0.40	8 - 15	20 - 35
15 - 20	0.30 - 0.50	5 - 10	15 - 25
>20	>0.50	<5	< 15

Table 1. Post-Liquefaction Stress-Strain and Strength Parameters

Ideally, the post-liquefaction shear stress-strain curves should be obtained directly from testing of representative undisturbed samples. However, estimates can be obtained indirectly from penetration resistance tests and comparison with laboratory tests on similar materials and/or field experience during past earthquakes (Byrne, 1996). It should be noted that the post-liquefaction curves may be 50 to 500 times softer than the pre-liquefaction curves, so that ranges of values rather than precise values should be used.

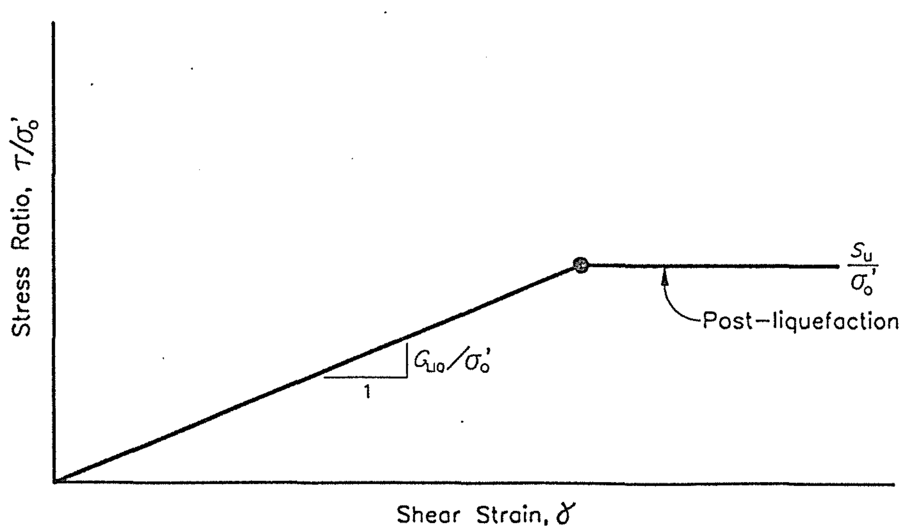


Figure 6: Idealized Post-Liquefaction Stress-Strain Response

ANALYSIS PROCEDURE

In dealing with earthquake induced liquefaction, there are three basic concerns:

1) Will the cyclic loading induced by the earthquake trigger liquefaction?, and if so
2) Is the residual strength adequate to prevent a flow slide? and if so,
3) Are the deformations tolerable?

These questions can be addressed by either an effective stress or a total stress dynamic analysis procedure as described by Byrne et al. (1994).

The current state-of-practice is to carry out a total stress analysis procedure. Briefly, this involves:

1) A triggering analysis, wherein the factor of safety against triggering liquefaction is assessed by comparing the cyclic resistance ratio (CRR) with the cyclic stress ratio (CSR) caused by the design earthquake. The CSR is usually obtained from an equivalent viscoelastic dynamic analysis using SHAKE or FLUSH.

2) A flow slide analysis. Here the residual strength is assigned to those zones deemed to have triggered, and limit equilibrium analyses are carried out to assess stability. If the computed factor of safety is less than unity, a flow slide is predicted and remedial measures are generally required.

3) Deformation analysis. If a flow slide is not predicted, the deformations associated with liquefaction could still be quite large, and must be assessed. The simplest and most common method used for this is the Newmark (1965) approach. Here, a potential slide block is modelled as a single-degree-of-freedom mass on an inclined plane (rigid plastic condition) and subjected to inertia forces corresponding to the design earthquake. The resulting downslope movements are computed and used as an estimate of soil movements. The assumption of a rigid plastic soil response is not appropriate for liquefied soil conditions, and Newmark never intended that it be used for this condition (Byrne, 1991).

The following simple total stress approach is proposed here for seismic assessment of tailing impoundments:

1) Determine the pre-earthquake static stresses using a finite element or finite difference code such as FLAC (Fast Lagrangian Analysis of Continua, ITASCA, 1995), using appropriate drained stress-strain and strength parameters for each soil type. The computer code FLAC solves the equations of motion in explicit form using very small time steps. The solution technique is valid for both static and dynamic conditions and has an advantage when solving static problems that while the problem may not be statically stable, the solution technique is stable and the failure pattern is predicted.

2) Determine the zones that will liquefy during the design earthquake. This may require a triggering analysis as discussed earlier, or it may be simply assumed that all saturated tailings will liquefy.

3) Assign post-liquefaction stress-strain and strength parameters to the liquefied zones and assess the flow slide potential. Conventional limit equilibrium analyses are commonly used for this. If the computed factor of safety is <1, a flow slide is predicted and deformations will be very large.

4) If a flow slide is not predicted, displacements can be computed by considering the effects of liquefaction as well as the inertia forces induced by the base motions. The liquefaction effect involves setting the initial stress redistribution to $\sigma_x = \sigma_y$ and $\tau_{xy} = 0$ in the zones of liquefaction. Upon shearing, the liquefied soil will strain harden and gain strength and stiffness as depicted in Figure 5. Loading of the model may be simulated in the following ways:

a) *Gravity Only*: Here the displacements are considered to be caused by gravity loading only, without the inertia forces caused by seismic loading of the base. Liquefaction of the tailings causes a redistribution in stress state and a greatly reduced stiffness, which results in disequilibrium under gravity loading, and causes movements that will be arrested when the soil strains and develops sufficient strength. This becomes a dynamic problem as soil elements initially accelerate under the out of balance forces, and finally decelerate and come to rest. The computer code FLAC can be used to carry out such an analysis.

b) *Gravity plus Base Acceleration*: The approach described above neglects the inertia forces due to movement of the base. In the time history approach, a representative time history of base motion is chosen, and a dynamic analysis is carried out. Liquefaction of the tailings is assumed to occur after a prescribed time, after which the post-liquefaction stress-strain curves are assigned to the liquefied zone together with a stress state $\sigma_x=\sigma_y$ and $\tau_{xy}=0$. The computer code FLAC can be used to carry out this analysis.

c) *Gravity plus Velocity Pulse*: Newmark found that the effect of a base time history could be roughly accounted for by considering that the soil block is subjected to a number of velocity pulses with magnitude equal to the maximum ground velocity, V_{max}. Byrne (1991) argued that once soil liquefaction occurred, only one pulse equal to the maximum value need be considered. In this approach, the whole tailings impoundment is considered to have a velocity V_{max} at the time liquefaction occurs. The liquefied zones are then assigned post-liquefaction parameters and a dynamic analysis carried out to assess the displacement resulting from the velocity pulse. FLAC can be used for this analysis.

APPLICATION TO THE KENSINGTON PROJECT

As previously mentioned, original analyses for the Kensington tailings dam included conventional limit equilibrium and Newmark displacement analyses as well as pseudo-dynamic displacement analyses, as reported by Byrne et al (1992, 1994). Results from these previous analyses indicated that the embankment is stable even with complete liquefaction of the tailings. A flow slide would not occur and deformations would be very small.

The previous analyses assumed that the cyclic loading induced by the design earthquake would trigger liquefaction, and showed that a flow slide would not occur. The new analyses presented in this paper do not present these results but provide a new assessment of the deformations associated with earthquake loading on the embankment. As described in the previous section, three approaches were considered: Gravity Only, Gravity plus Base Acceleration and Gravity plus Velocity Pulse. All analyses were conducted using the finite-difference computer code FLAC. The model grid was defined by four distinct zones: tailings mass, embankment sand/gravel and till core zones, and the underlying foundation, as shown on Figure 7. Static as well as post-liquefaction material input parameters required for the elastic-plastic Mohr-Coulomb model are shown in Table 2.

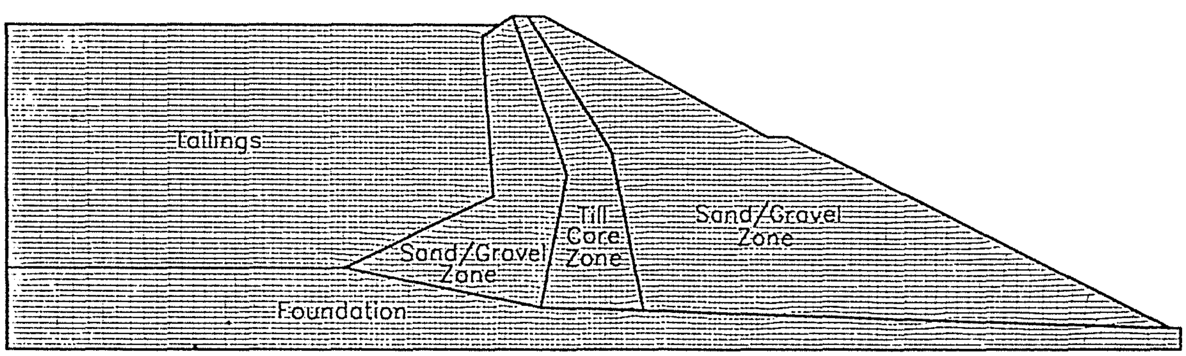


Figure 7: Kensington Tailings Embankment - FLAC Grid with Various Material Zones

	Elastic Parameters				Plastic Parameters		Density
MATERIAL	K_g	n	K_b	m	Ø(deg.)	c'(kPa)	ρ_b(t/m³)
Tailings							
–preliquefaction	375	0.5	1490	0.25	36.6°	0	1.9
–post liquefaction	3.75	0.0	1490	0.25	8.5°	0	1.9
Sand/Gravel Zones	455	0.465	1085	0.25	39°	0	2.2
Till Core Zone	725	0.46	1700	0.25	42°	0	2.2
Foundation	2600	0.0	3400	0.0	40°	0	2.2

NOTES

1. Post–liquefied Shear Modulus for Tailings modelled as being independent of stress, ie. G=375 kPa
2. Shear and Bulk Modulii for Foundation materials modelled as average values over depth of foundation, ie. $G = 2.6\times10^5$ kPa , $B = 3.4\times10^5$ kPa
3. For all other materials, elastic Shear and Bulk Modulii are considered to depend on the level of confining stress.
   $G = K_g P_a (\frac{\sigma'_m}{P_a})^n$ (kPa) $B = K_b P_a (\frac{\sigma'_m}{P_a})^m$ (kPa) P_a = Atmospheric Pressure

Table 2: Kensington Dam Displacement Analyses - Material Input Parameters for FLAC Model

For all analyses, the entire tailings mass was considered saturated and liquefiable. The static stresses prior to liquefaction were obtained by initially setting the cohesion for all zones to a very high value, and allowing the model to reach equilibrium under gravity loading. Cohesion was then dropped to zero, and the mean effective stress within each element was determined as the model was allowed to reach a new equilibrium. At this point, bulk and shear moduli values for each element were calculated, through their relationships with mean effective stress. After reaching this static condition within the model, each of the three displacement analyses were carried out, as described previously.

RESULTS

Horizontal and vertical displacements of the dam crest have been predicted for three deformation analyses approaches: Gravity Only, Gravity plus Base Acceleration and Gravity plus Velocity Pulse. Displacements resulting from the Gravity plus Velocity Pulse approach may be compared with the previous pseudo-dynamic SOILSTRESS analyses, as both cases utilize a similar analytical approach. As shown on Table 3, displacements resulting from the FLAC Gravity plus Velocity Pulse analysis are greater than from the SOILSTRESS pseudo-dynamic analysis. Slightly revised material input parameters and model geometry may be factors contributing to this discrepancy, specifically an unrealistically high value for s_u for the liquefied tailings used in the SOILSTRESS analysis.

ANALYTICAL PROCEDURE	Crest Displacement (m)	
	Horizontal Displacement	Vertical Displacement
FLAC – Gravity plus Velocity Pulse	1.2	−0.5
SOILSTRESS – Pseudo-dynamic approach (Byrne, 1992)	0.5	−0.1

NOTE
1. Positive horizontal displacements – downstream: positive vertical displacements – up.

Table 3: Results of Displacement Analyses for Kensington Embankment

The results from the three different approaches used in the FLAC analyses are compared in Table 4. Of the three deformation analyses conducted, the Gravity Only case resulted in the lowest horizontal and vertical deformations. For this case, the post-liquefaction parameters are assigned to the tailings, and the embankment is allowed to deform under gravity, with no input base motion considered.

FLAC ANALYTICAL APPROACH	Crest Displacement (m)	
	Horizontal Displacement	Vertical Displacement
Gravity Only	0.1	0.0
Gravity plus Base Acceleration		
(i) San Fernando (Griffith Park) Time History	3.7	−2.8
(ii) Imperial Valley (El Centro) Time History	0.1	−2.9
Gravity plus Velocity Pulse	1.2	−0.5

NOTES
1. San Fernando and Imperial Valley time histories scaled to peak ground acceleration of 0.6g
2. Positive horizontal displacements – downstream: positive vertical displacements – up.

Table 4: Results of FLAC Displacement Analyses for Kensington Embankment

For the Gravity plus Base Acceleration approach, two separate earthquake time histories were applied to the model. Although each time history was scaled to a peak ground acceleration of 0.6 g, the records were significantly different. As shown on Figures 8 and 9, the San Fernando time history is a "balanced" record, with accelerations distributed evenly in both directions. On the other hand, the Imperial Valley record contains peak input accelerations of greater magnitude in the negative (upstream) direction. These differences in the earthquake time histories may explain the resulting displacements shown on Table 4.

Reversing the time history records for these two earthquakes resulted in lower displacements for the San Fernando time history and a much larger horizontal displacement, in the order of 7 m downstream, for the Imperial Valley record. Due to the nature of this time history, an inverted Imperial Valley record applies large base accelerations in the downstream direction. This may explain the resulting displacements for the inverted Imperial Valley record.

To further investigate the results of the Gravity plus Base Acceleration approach, other cases were analyzed in which a decreased thickness of the tailings mass was considered to liquefy during the input base time history. Resulting displacements of the embankment crest from these analyses were only slightly smaller than for the case in which the entire tailings mass was considered to liquefy. This indicates that for these cases, the embankment geometry may have more of an influence on the patterns of deformation from a Gravity plus Base Acceleration analysis than the corresponding strength of the adjacent tailings deposit.

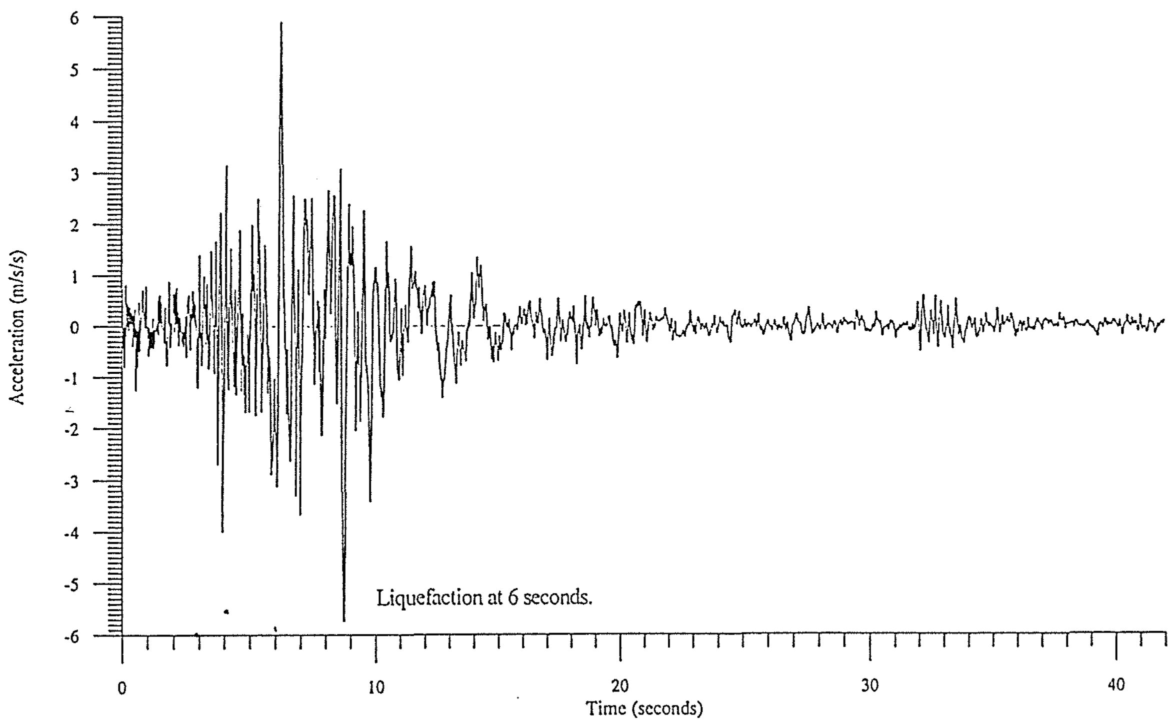


Figure 8: San Fernando Earthquake, Griffith Park Time History

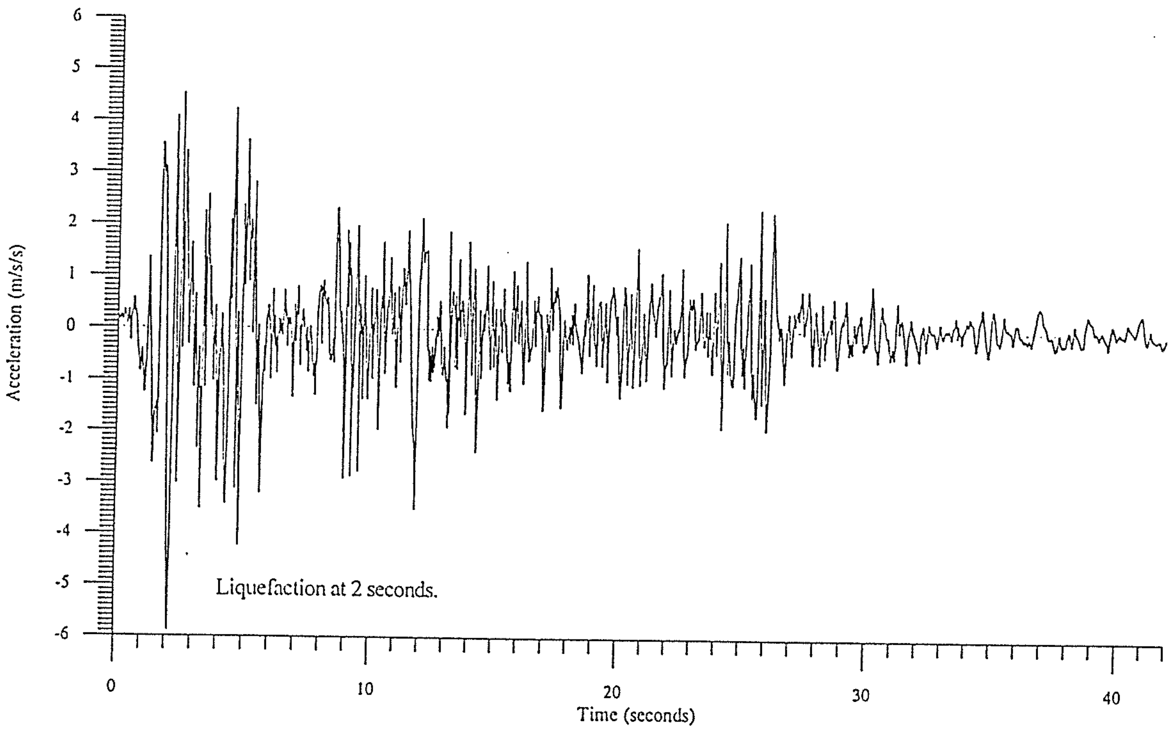


Figure 9: Imperial Valley Earthquake - El Centro Time History

As shown on Table 4, the Gravity plus Velocity Pulse approach resulted in displacements somewhere between the other two methods. For all approaches the predicted pattern of displacements was an outward (downstream) bulging of the crest with vertical slumping of the embankment. The "worst case" displacements from the inverted Imperial Valley time history are shown on Figure 10.

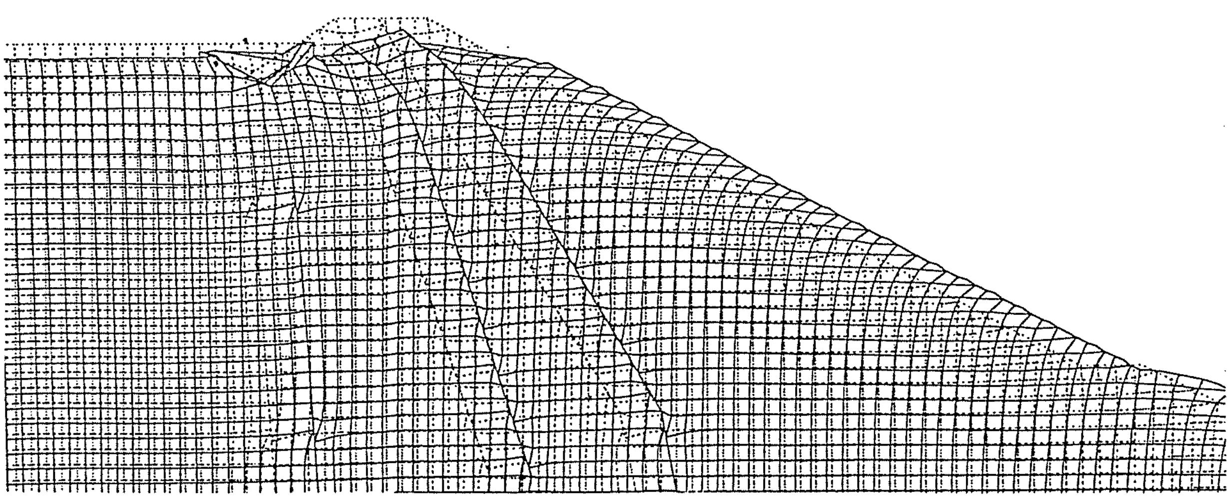


Figure 10: Kensington Tailings Embankment - Pattern of Displacements for the Inverted Imperial Valley Time History

SUMMARY

The Modified centreline method is a new approach to tailings embankment construction, and has been shown to be seismically stable, while minimizing fill requirements. This construction method has been used in the design of the proposed tailings dam for the Kensington Project located in an area of high potential seismicity near Juneau, Alaska. Previous displacement analyses were carried out using the pseudo-dynamic finite element procedure developed by Byrne (1991), and resulted in small and acceptable displacements of the dam crest.

A new total stress approach for seismic assessment of tailings impoundments is presented, using a finite element or finite difference code such as FLAC. Three displacement analysis approaches are proposed: Gravity Only, Gravity plus Base Acceleration and Gravity plus Velocity Pulse. As shown in Table 4, the Gravity Only case resulted in very small deformations of the dam crest, indicating that the methods which include an input base motion are more conservative. The Gravity plus Velocity Pulse approach resulted in larger crest displacements than the Gravity Only case, but smaller displacements than the Gravity plus Base Acceleration method. This result seems reasonable, due to the more conservative nature of the Gravity plus Base Acceleration approach, in which case the embankment continues to be subjected to a base input motion even after liquefaction of the tailings mass has occurred. A more conservative approach to the Gravity plus Velocity Pulse method would be to subject the embankment to several velocity pulses, rather than just a single pulse.

Of the three approaches presented, the Gravity plus Base Acceleration displacement analysis has been shown to be the most conservative, resulting in the largest displacements of the embankment crest. It has also been shown that although different earthquake records can be scaled to contain identical peak base accelerations, they may provide significantly different results when incorporated into displacement analyses. When conducting a Gravity plus Base Acceleration displacement analysis, several representative earthquake time histories should be applied to the model, in order to ensure a rigorous evaluation.

Predicted displacements resulting from this new approach using FLAC for conducting deformation analyses were generally found to be higher than from the previously conducted pseudo-dynamic finite element analyses using SOILSTRESS. In particular, the Gravity plus Base Acceleration approach was found to result in the largest patterns of deformation, providing a conservative approach for the seismic assessment of tailings impoundments.

ACKNOWLEDGEMENTS

The authors are grateful to Mr. John Wilkinson for some of the FLAC runs, Mr. Wayne Lahoda for the figures and Ms. Sue Priest for her typing and presentation of the paper.

REFERENCES

1. Brouwer, K.J., Weitz, T.J. and Haile, J.P., 1992. "Design and Operation of the Montana Tunnels Tailings Disposal Facility." Second International Conference on Environmental Issues & Management of Waste in Energy & Mineral Production. September 1-4, 1992, Calgary, Alberta.

2. Brouwer, K.J., Brown, B.S. and Stewart, D., 1994. "Tailings Consolidation Using Wick Drains." 3rd International Conference on Environmental Issues and Waste Management in Energy and Mineral Production, 1994. Perth, Australia.

3. Byrne, P.M., 1991. "A Model for Predicting Liquefaction Induced Displacements Due to Seismic Loading." The 2nd Int. Conf. on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, St. Louis, Missouri, Paper No. 7.14, March.

4. Byrne, P.M., 1996a. "Liquefaction Case Histories: Can We Predict Response." Recent Developments in Seismic Liquefaction Assessment, ConeTec Seminar, 1996, Vancouver, B.C.

5. Byrne, P.M., 1996b. "Modelling Liquefaction", ITASCA FLAC Seminar, March 1996, Vancouver, B.C.

6. Byrne, P.M., Imrie, A.S. and Morgenstern, N.R., 1994. "Results and Implications of Seismic Performance Studies for Duncan Dam." Canadian Geotechnical Journal, Vol. 31, No. 6, pp. 979-988.

7. Byrne, P.M., Jitno, H., Haile, J., 1992. "A Procedure for Predicting the Seismic Response of Tailings Impoundments." First Canadian Symposium on Geotechnique and Natural Hazards. May 6-9, 1992, Vancouver, B.C.

8. Byrne, P.M., Jitno, H., Anderson, D.L., Haile, J., 1994. "A Procedure for Predicting Seismic Displacement of Earth Dams." Report No. XIII CSMFE, 1994, New Delhi, India/XIII CIMSTF, 1994, New Delhi.

9. Dobry, R. and Alvarez, L., Nov. 1967. "Seismic Failures of Chilean Tailings Dams", Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 93, SM6, pp. 237-260.

10. Haile, J.P. and Brouwer, K.J., 1987. "Design and Construction of the Montana Tunnels Disposal Facility." 89th Annual General Meeting of the Canadian Institute of Mining. May 3-7/87, Toronto, Ontario.

11. Haile, J.P. and Brouwer, K.J., 1994. "Modified Centreline Construction of Tailings Embankments." 3rd International Conference on Environmental Issues and Waste Management in Energy and Mineral Production, 1994. Perth, Australia.

12. Ishihara, K., 1984. "Post-Earthquake Failure of a Tailings Dam Due to Liquefaction of the Pond Deposit", Proc. International Conf. on Case Histories in Geotechnical Eng., St. Louis, Vol. 3, pp. 1129-1143.

13. Newmark, N.M., 1965. "Effects of Earthquakes on Dams and Embankments." Geotechnique, Vol. 15, No. 2, pp. 139-160.

Design and operation of the Montana Tunnels tailings disposal facility

K.J. Brouwer
Knight and Piesold Ltd., Vancouver, B.C.

T.J. Weitz
Montana Tunnels Mining Inc., Jefferson City, Montana

J.P. Haile
Knight and Piesold Ltd., Vancouver, B.C.

2nd International Conference on Environmental Issues & Management of Waste in Energy & Mineral Production
September 1-4, 1992
Calgary, Alberta

ABSTRACT: The Montana Tunnels Project is an open pit mining operation located 37 km south of Helena, Montana. It involves mining of a weathered diatreme orebody at a rate of 11,500 tpd to produce lead and zinc concentrates with significant precious metal values. Total mineable reserves are currently estimated at 46 million tons. The tailings from the mineral extraction process are discharged into a drained tailings impoundment with supernatant water decanted into a process water pond prior to recycling to the mill. Significant features of the initial design of the tailings disposal facility included a partial soil-bentonite liner and drainage system within the tailings basin and a free-draining embankment with a design flow-through capacity of 4000 USgpm and an ultimate height of 260 ft. During initial start-up, some difficulties were encountered with respect to erosion of the soil-bentonite liner and solids passing through the embankment drainage system. The latter problem led to redesign of the filter system on the embankment face and the incorporation of a decant system for recovery of supernatant water. The construction technique for the main embankment has also been changed from downstream construction to a modified centreline method, which differs from conventional centreline construction in that the contact between compacted fill and tailings slopes slightly upstream. The embankment does not, however, rely on the strength of the tailings and is stable even if the tailings are fully liquefied.

This paper provides an overview of the original design basis for the tailings disposal facility, operating performance to date, the reasons for design modifications and a stability assessment of the modified centreline construction technique.

1 INTRODUCTION

The Montana Tunnels mine is operated by Montana Tunnels Mining Inc., a subsidiary of Pegasus Gold Corporation. Montana Tunnels began operation in the spring of 1987 following 25 months of environmental review under the Montana hard rock mine permit process and 13 months of construction. The mine is located in north Jefferson County, Montana, approximately 25 miles south of Helena, Montana. The mine permit area encompasses 1,497 acres of which 860 acres will be disturbed by the mining operations.

The Montana Tunnels orebody occurs in the throat of an ancient volcano which was active approximately 50 million years ago. Mineralization is hosted in a tuffaceous breccia called a "diatreme". The mineralization consists of small veins and disseminated mineral grains of pyrite, galena, and sphalerite, along with electrum (a mixture of gold and silver). The ore grade averages 0.019 ounces per ton gold, 0.34 ounce per ton silver, 0.24 percent lead, and 0.62 percent zinc.

Mining is by conventional open pit methods of drilling, blasting, loading and haulage. The mine moves approximately 4 million tons of ore and 6 million tons of waste annually.

Ore processing in the 11,500 tons per day concentrator involves primary crushing to -8" size or smaller, followed by autogenous and ball mill grinding to approximately 75% minus 100 mesh. A small gravity circuit has been added to the grinding section to collect coarse particles of free electrum which are refined to produce a dorè bullion. Following grinding, the slurry enters the flotation circuit where lead, followed by zinc minerals, are selectively floated to produce lead and zinc concentrates. In 1991, Montana Tunnels produced 62,600 ounces of gold, 1.17 million ounces of silver, 7,000 tons of lead and 18,000 tons of zinc.

Tailings from the flotation circuit flow by gravity to the tailings disposal facility which is located downslope from the concentrator and is the primary focus of this paper. A general arrangement of the overall mine site is shown on Figure 1.

2 PERMITTING AND RECLAMATION

Mining operations at Montana Tunnels are regulated by over 20 state and federal laws through eight different state and federal agencies.

Before Montana Tunnels was constructed, Pegasus Gold Corporation prepared a comprehensive plan for environmental management. It included programs for air and water quality monitoring, a system to recycle process water, and a reclamation plan to assure that lands disturbed by mining would be restored to other productive uses when mining is completed. Montana Tunnels operates four air quality stations around the perimeter of the mine. Samples are collected weekly and analyzed for concentrations of particulate dust and the presence of heavy minerals. Water quality monitoring is conducted at 25 groundwater wells and three surface water stations. Samples are collected at weekly, monthly, and quarterly intervals, depending upon the location of the monitoring point. Water is checked for a wide variety of metals as well as pH, sulphates, and nitrates. Since the start of construction in 1986, the mine has been in compliance with Montana's strict water quality standards. Montana Tunnels is a zero discharge facility. No water is discharged into the waters of Montana. All water used to process ore as well as storm runoff is collected in the tailings disposal facility where it is filtered through the tailings dam and collected in a reclaim water pond for reuse at the mine. Approximately 80% of the mine's water requirement is met through recycling.

The reclamation objectives at Montana Tunnels are to restore the land to its prior uses which were mainly livestock grazing, wildlife habitat, and recreation. Through 1991, Montana Tunnels has salvaged over 1.4 million cubic yards of topsoil for future reclamation. Wherever feasible, reclamation is conducted concurrently with mining operations. Montana Tunnels has also established a number of reclamation research projects at the mine. Test plots have been established to examine the feasibility of reclamation using different topsoil depths, seed and mulch mixtures, and fertilizer application rates. The research program will continue throughout Montana Tunnels' operating life to ensure implementation of the best possible reclamation program for the mine.

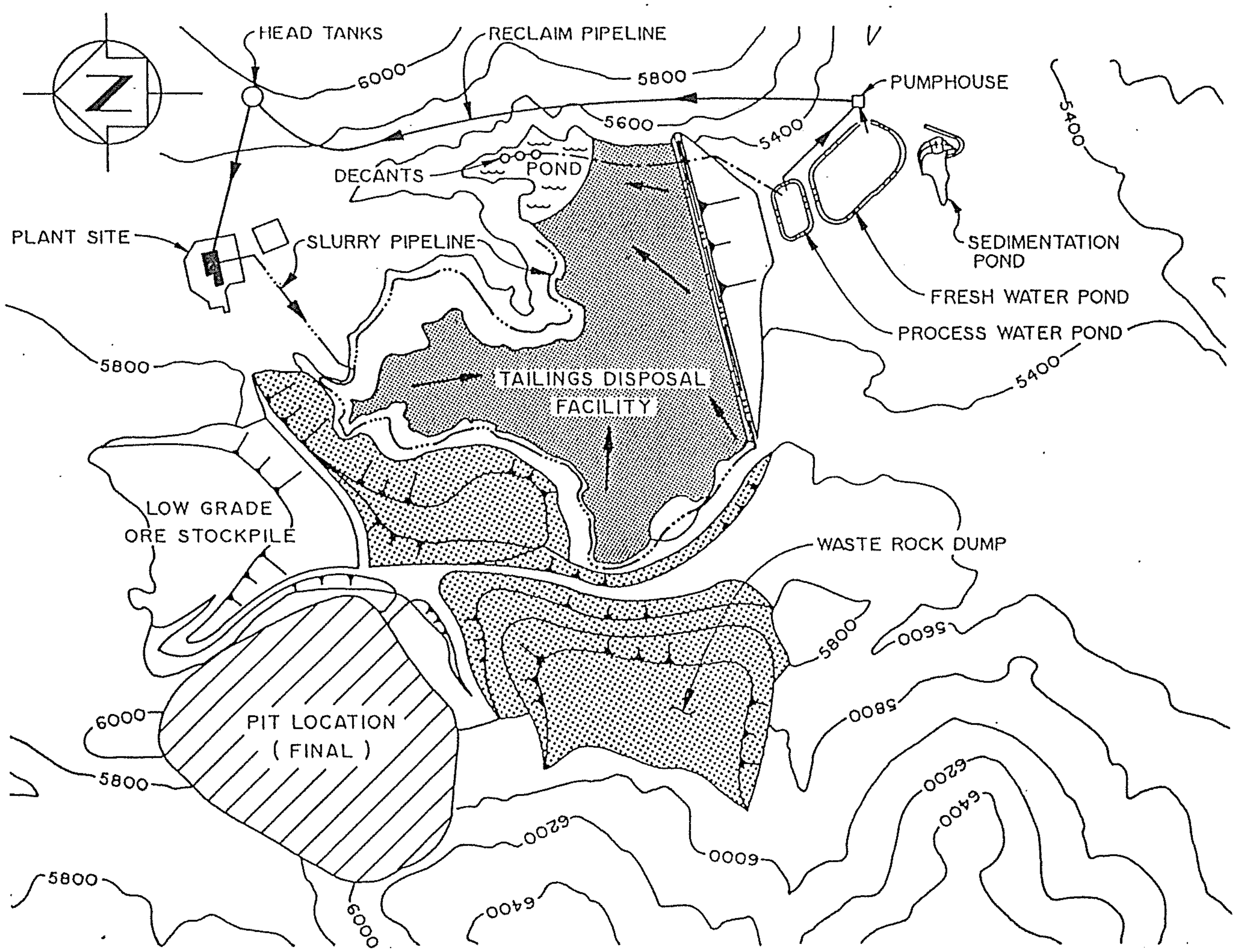


Fig. 1 Overall site layout of the Montana Tunnels mine

3 TAILINGS FACILITY DESIGN

The tailings disposal facility for the Montana Tunnels mine comprises a drained tailings impoundment for storage of the tailings solids and a process water pond for storage of all decanted water prior to recycling to the mill. The overall facility is designed to achieve the basic objectives of minimizing seepage to the environment in the short and long-term and achieving a fully drained stable tailings mass suitable for surface reclamation on completion of mining. The design and initial construction are described by Haile and Brouwer (1987).

Specific features of the initial design include a partial soil-bentonite liner and drainage system within the tailings basin and a free-draining embankment with a design flow-through capacity of 4000 USgpm and an ultimate height of 260 ft. (80 m), constructed in stages using waste rock from open pit mining. A schematic cross-section through the facility is shown on Figure 2.

4 INITIAL OPERATIONS

Milling operations at the mine started in the spring of 1987. During initial operations some difficulties were encountered with various

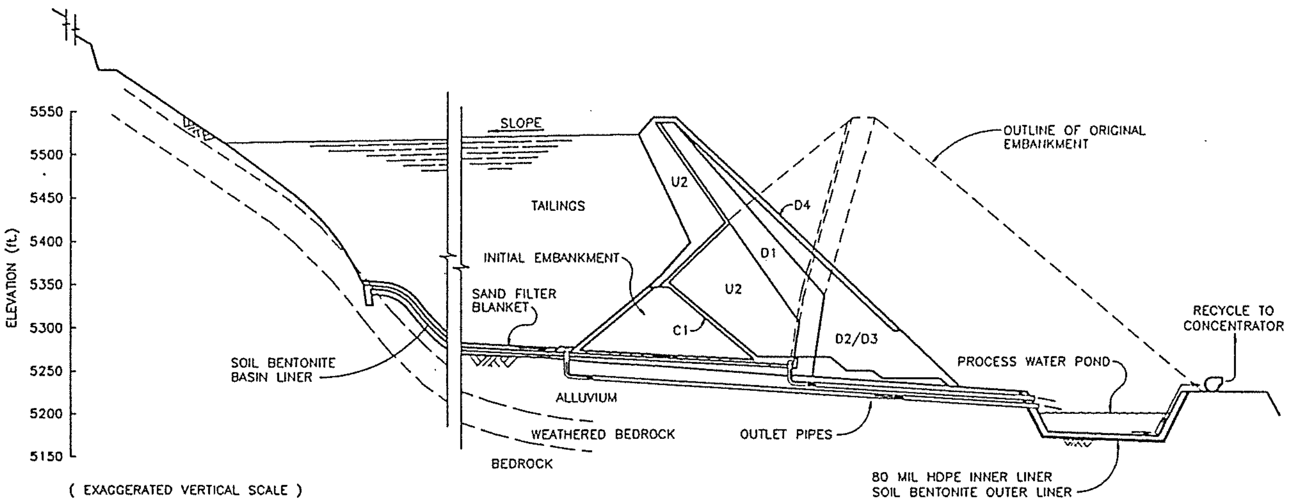


Fig. 2 Schematic cross-section of tailings disposal facility

aspects of the tailings facility which have led to some on-going design modifications.

The tailings distribution system consists of two header pipes around the uphill perimeter of the tailings basin, with multiple valved spigot offtakes along each header to distribute the tailings slurry. Deposition of the tailings slurry was initially carried out on a controlled rotational basis towards the main embankment, the intent being to create low energy laminar flow over the tailings beach to promote liquid-solid separation. Supernatant water and rainfall runoff within the tailings basin flows towards the embankment and, in the original design, is decanted through the embankment drainage system to the process water pond.

Despite the efforts made to distribute the tailings slurry and the provision of erosion control berms, some concentration of flows inevitably occurred on the drainage blanket overlying the soil bentonite liner in the lower, flatter portion of the basin. This led to scour of the gravelly sand making up the drainage blanket on slopes of between 1 and 3 percent, and some localized erosion of the soil-bentonite liner. These difficulties were overcome by repairs to the liner and by placing a rockfill blanket over the drainage blanket using clean waste rock from the open pit mining operations.

The offtake pipes for tailings discharge were also locally extended to ensure deposition of the slurry took place on previously deposited tailings.

A second difficulty arose with respect to the ability of the embankment filter system to prevent the fine tailings particles from entering the embankment. The embankment drainage system was designed with a flow-through capacity of 4000 USgpm, with a series of processed gravel drainage zones and perforated pipework upstream of a low permeability central core. Water entering the drainage system flows by gravity through concrete encased outlet pipes to the process water pond beyond the downstream toe. A needle punched polyester geotextile embedded in the upstream face was designed to prevent the ingress of the tailings fines. The design was based on the particle size distribution of the tailings from pilot testwork and filter criteria published by Koerner (1986), giving an equivalent opening size (EOS) $<2 \times d_{85}$. This resulted in the use of a 12 oz/yd^2 geotextile with an EOS of 80 microns, similar to what was successfully employed at the Jamestown Mine in California. (Skolasinski et al (1990)).

Once tailings deposition started, it was immediately apparent that the filtration system was not working and, while adequate water recovery was achieved, a significant volume of tailings fines were passing through the geotextile. Sampling of the tailings showed that significant segregation was occurring and that the material adjacent to the embankment face was a very fine colloidal rock flour with a particle size distribution as shown on Figure 3.

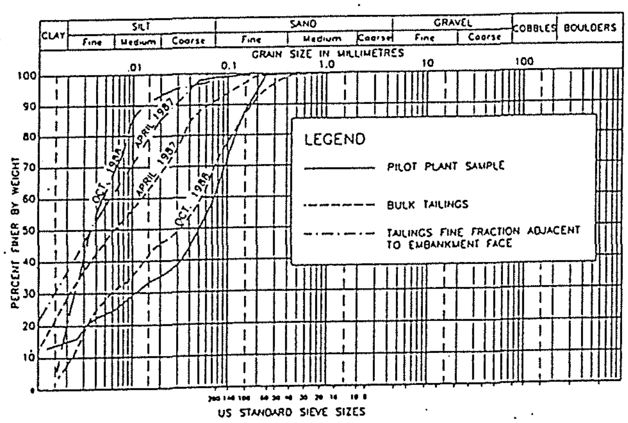


Fig. 3 Particle size distribution of tailings samples

The immediate remedy involved the placing of a layer of sand on the upstream face of the embankment which provided some temporary relief.

5 SUPERNATANT WATER RECOVERY

A testwork program was initiated to identify combinations of geotextile and sand that would effectively prevent ingress of the tailings fines, but maintain the required flow-through capacity for on-going operations. This was not entirely successful as the use of sand appeared to be necessary with a resulting reduction in overall permeability. In practice, a combined sand/geotextile filter would work but a significant ponded depth above the level of the tailings fines would be required to achieve the required flow-through capacity.

An alternative was adopted which involved the construction of decant towers on the embankment face connected into the embankment drainage system. However, persistent cloudy water in the process water pond led to a complete re-design of the embankment and water recovery system.

This involved extension of a tailings header pipe and deposition of tailings from the embankment, relocating the surface pond into the northeast gulley of the tailings basin, and construction of a new decant system in the northeast gulley for recovery of the supernatant water.

The decant system comprises a 20 inch diameter steel and HDPE conveyance pipe located along the east side of the basin and leading to the process water pond. At the upstream end, offtakes to the conveyance pipe are located at 5 foot vertical intervals and are connected to decant towers as shown on Figure 4. The decant towers allow for very fine control of the pond water level while removing the cleanest surface water as the level of the tailings solids rises. As each decant becomes fully submerged it is plugged with concrete. The decant system continues to work extremely well.

6 MODIFIED CENTRELINE CONSTRUCTION

Deposition of tailings from the embankment and the resulting segregation of the sand fraction has allowed a complete re-design of the main embankment from the original downstream construction to modified centreline construction.

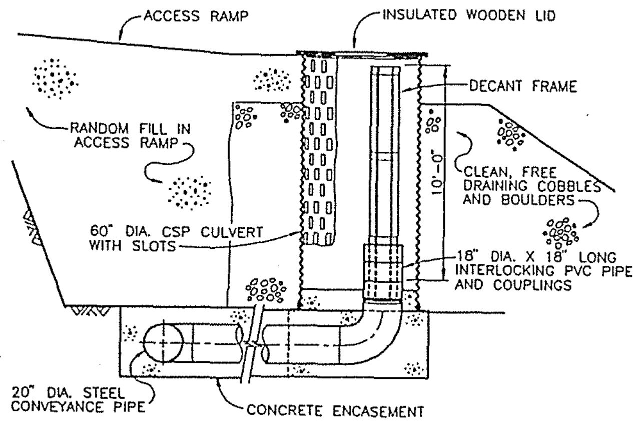


Fig. 4 Schematic arrangement of decant

Modified centreline construction is similar to conventional centreline construction in that the contact between compacted fill and the tailings slopes slightly upstream. It is, however, different from upstream construction as the stability of the embankment is independent of the tailings strength. The revised cross-section for the embankment is shown on Figure 5.

The basic features of modified centreline embankment construction are:

- The construction differs from conventional centreline construction in that no construction on the downstream face is required for on-going raises.

- The stability of the embankment relies on the relatively wide thickness of compacted fill at any elevation and is independent of the strength of the tailings.

- The embankment is stable even if the tailings are fully liquefied and is intrinsically stable under earthquake loading. The analogy for the upstream face is that of a slurry wall, where a dense fluid, i.e. bentonite mud, can be used to support very deep excavations.

This construction technique requires some placing of fill on the tailings beach. If the beach is strong enough to support the first lift, then the strength only increases as the tailings

MATERIAL PLACING AND COMPACTION REQUIREMENTS

MATERIAL TYPE	LOCATION	PLACING AND COMPACTION REQUIREMENTS
DIATREME	D1 ZONE	PLACED, MOISTURE CONDITIONED AND SPREAD IN MAX. 12" LAYERS AFTER COMPACTION. VIBRATORY COMPACTION AS DIRECTED BY THE ENGINEER.
RANDOM ROCKFILL	D3 ZONE	PLACED AND SPREAD IN MAX. 36" LAYERS AFTER COMPACTION. VIBRATORY COMPACTION AS DIRECTED BY THE ENGINEER.
NON ACID GENERATING ROCKFILL	D4 ZONE	PLACED AND SPREAD IN MAX. 36" LAYERS AFTER COMPACTION. VIBRATORY COMPACTION TO 90% OF MAXIMUM DENSITY.
FREE DRAINING ROCKFILL	U1,U2a,U2b,U3 ZONES	PLACED AND SPREAD IN VARYING LIFT THICKNESSES COMPACTION AS DIRECTED BY THE ENGINEER.
UNPROCESSED COLLUVIUM	CF ZONE	PLACED AND SPREAD IN MAX. 36" LAYERS. VIBRATORY COMPACTION AS DIRECTED BY THE ENGINEER.
COARSE FILTER MATERIAL	C1 DRAINAGE ZONE	PLACED AND SPREAD IN MAX. 36" LAYERS AFTER COMPACTION. VIBRATORY COMPACTION AS DIRECTED BY THE ENGINEER.

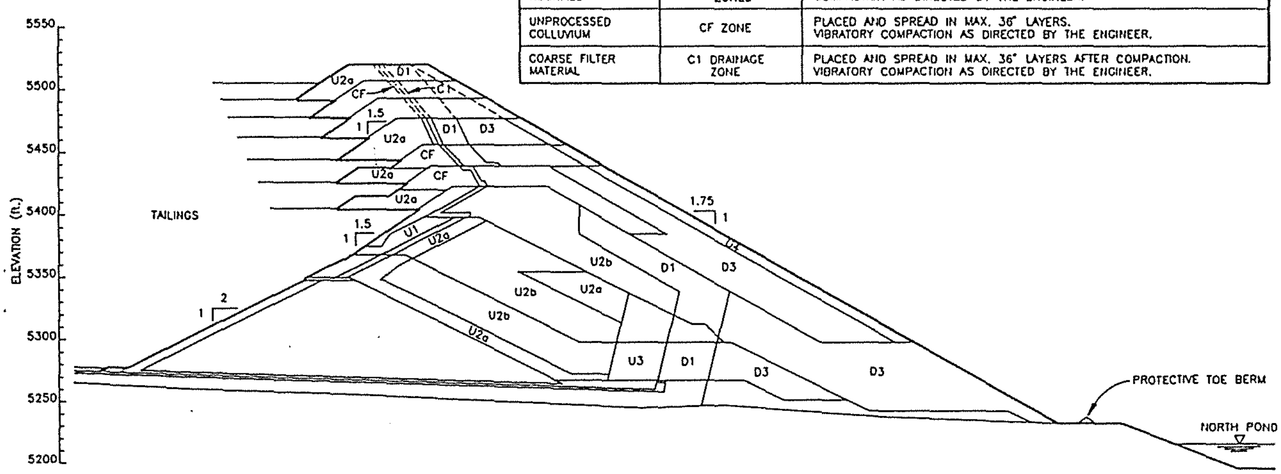
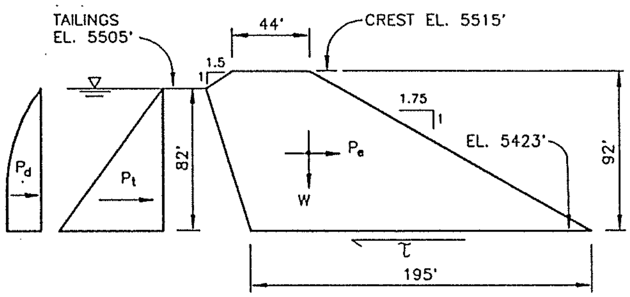


Fig. 5 Modified centreline embankment cross-section

consolidate. If the beach cannot support the first lift, then it can be displaced using rockfill.

Stability analyses carried out on the embankment for worst case seismic loading assume full liquefaction of the tailings and hydrodynamic forces imposed by this liquefied tailings mass. This is synonymous with the loading imposed by a liquefied silt on a water retaining gravity dam. Shear resistance is provided by the relatively wide thickness of compacted fill at any elevation. A schematic diagram of the forces acting on the modified centreline portion of the embankment is shown on Figure 6.



SUMMARY OF LOADING CONDITIONS	
SYMBOL	DESCRIPTION
P_t	LIQUEFIED TAILINGS
P_d	HYDRODYNAMIC FORCE
P_e	EARTHQUAKE
W	EMBANKMENT WEIGHT
τ	SHEAR RESISTANCE

Fig. 6 Stability analysis of modified centreline raise

Static factors-of-safety for upstream and downstream failure are 1.81 and 1.50, respectively.

The design MCE acceleration for the Montana Tunnels site is 0.22 g. For the permitted final crest at El. 5515 feet, an overall embankment height of 265 feet, the critical acceleration assuming fully liquefied tailings is 0.24 g.

Displacement analyses using the procedures of Newmark (1965) and Makadisi and Seed (1977) for an MCE event result in negligible potential displacement within the embankment. A more rigorous displacement analysis has been developed by Byrne et al (1992) and has been used to analyze a similar modified centreline embankment. This will be used to investigate raising the embankment by a further 40 feet to accommodate increased ore reserves.

Modified centreline construction involves placing fill on the tailings beach for each embankment raise. The first upstream fill placement took place in 1990 on a beach only 55 feet wide after only six months of tailings deposition from the embankment. Construction involved placement of a geotextile directly on the tailings to form a separation layer and an initial 3 foot lift of coarse rockfill placed with Caterpillar 777 trucks and a D8N bulldozer travelling along the lift. Monitoring of porewater pressures within the tailings was carried out using electric transducers. A typical response is shown on Figure 7. It shows an initial rise in pore pressure during lift placement, followed by dissipation within 24 hours. This was accompanied by numerous sand boils beyond the lift and evidence of horizontal dissipation of excess pore pressures. After dissipation of the excess pressures, placement of the second lift was authorized without further concerns for stability.

A second upstream lift was successfully placed in the summer of 1991. In this case, the geotextile separation layer was augmented with a geogrid to provide additional horizontal strength. While the initial pore pressure rise in the tailings under the first lift in 1990 generally equalled the vertical stress, in 1991 significantly lower pressure rises were observed due to the improved tailings beach development and some air-drying of the tailings.

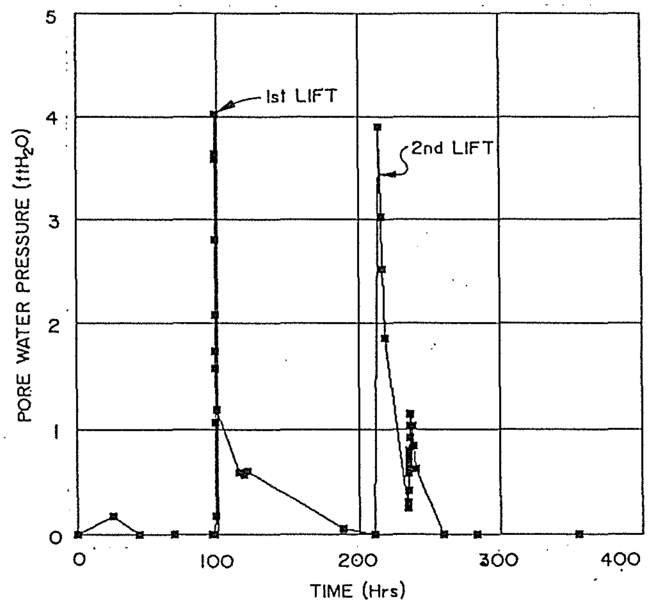


Fig. 7 Pore-pressure response in tailings

The change in the embankment construction method has resulted in a large reduction in the compacted fill requirements with a resulting significant cost saving. It has also allowed on-going reclamation of the downstream face.

7 TAILINGS CONSOLIDATION

The drainage system overlying the soil-bentonite liner at the base of the tailings mass, together with the drained embankment, were incorporated into the original design to decrease drainage path lengths and enhance consolidation of the tailings. The current reclamation concept is to obtain access to the final tailings surface as soon as possible after mine closure for placement of a cap rock layer and topsoil to return the tailings area to its original grazing potential.

On-going filling of the tailings basin, together with pore pressure measurements within the tailings mass, has indicated that the tailings are not achieving the degree of consolidation anticipated in the original design. This is attributed to the actual very fine gradation of the material. Consolidation is also limited by the geometry of the tailings basin and relatively rapid rate of rise.

Consolidation within the tailings basin has been analyzed using a one-dimensional large strain finite element analysis and measured tailings consolidation parameters. The resulting tailings density profile is as shown on Figure 8, with an anticipated average dry density at closure in the order of 82 pounds per cubic foot. Different options are currently being investigated to enhance beach development and evaporation from the tailings surface, in order to improve consolidation and reduce potential long-term settlements.

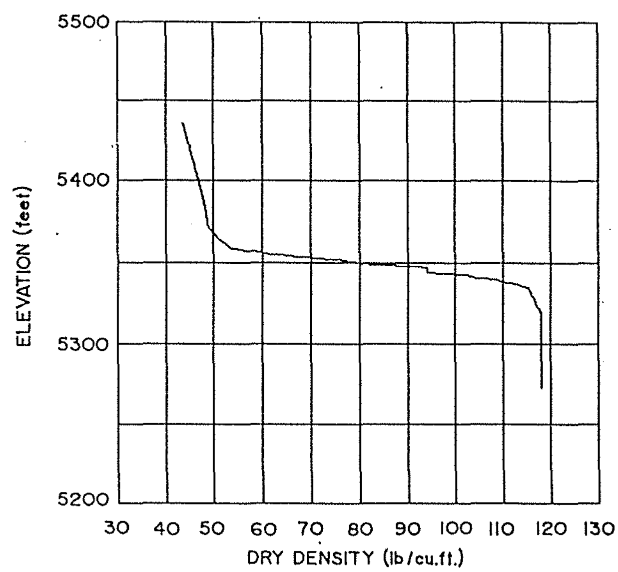


Fig. 8 Tailings density profile

8 SUMMARY

The Montana Tunnels tailings disposal facility was designed in 1985 as a drained tailings facility for storage of the tailings solids, together with a lined process water pond. The basic objectives of the design are the protection of waters of the State and ease of long-term reclamation.

The mine started operations in 1987. During initial start-up, some difficulties were encountered with respect to erosion of the soil-bentonite liner and solids passing through the embankment drainage system. The latter problem led to immediate re-design of the filter system on the embankment face and the incorporation of a decant system for recovery of supernatant water. In order to reduce on-going construction costs for the embankment, which is built from mine waste material, the construction technique has also been changed from a full downstream section to a modified centreline method.

Modified centreline embankment construction differs from conventional centreline construction in the contact between compacted fill and tailings slopes slightly upstream. The embankment does not, however, rely on the strength of the tailings and is stable even if the tailings are fully liquefied. It provides a cost-effective method of embankment raising and allows for on-going reclamation of the downstream slope.

REFERENCES

Newmark, N.V. 1965. Effects of Earthquakes on Dams and Embankments: *Geotechnique 15: 139-160*

Makadisi, F.I. and Seed, H.B. 1977. A Simplified Procedure for Estimating Earthquake Induced Deformations in Dams and Embankments: *Report No. UCB/EERC-77/19, College of Engineering, University of California, Berkeley*

Koerner, R.M. 1986. Designing with Geosynthetics: *Prentice-Hall, Englewood Cliffs, N.J.*

Haile, J.P. and Brouwer, K.J. 1987. Design and construction of the Montana Tunnels Tailings Disposal Facility: *89th Annual General Meeting of the Canadian Institute of Mining, May 3-7/87, Toronto, Ontario*

Skolasinski, D.Z., Haile, J.P., and Smith, A. 1990. Design Objectives and Performance of the Tailings Management System for the Jamestown Mine, California: *Western Regional Symposium on Mining and Mineral Processing Wastes, Society for Mining, Metallurgy Processing Wastes, May 30-June 1/90, Berkeley, California*

Byrne, P., Jitno, H., and Haile, J.P. 1992. A Procedure for Predicting the Seismic Response of Tailings Impoundments: *First Canadian Symposium on Geotechnique and Natural Hazards, May 6-9/92, Vancouver, B.C.*